"ENGINEERING ANALYSIS OF A TIMBER CRIBWORK RETAINING WALL"

PREPARED FOR: THE DEPARTMENT OF FISHERIES AND OCEANS, AND THE DIVISION OF CO-ORDINATION, MEMORIAL UNIVERSITY OF NEWFOUNDLAND

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December 4, 1987.

Mr. Paul Batstone, P.Eng.,
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Dear Paul:

Enclosed is my work report entitled "Engineering Analysis of a Timber Crib Retaining Wall". I am currently employed for Work Term VI by the Harbour & Infrastructure Branch of the Department of Fisheries & Oceans.

This report analyses a timber crib retaining wall constructed by Fisheries & Oceans at Long Harbour, Nfld. This report besides analysing this particular structure presents some helpful guidelines that should be followed in designing any typical timber crib retaining wall.

In closing, I would like to thank Dan Blundon for the opportunity to work for the Department of Fisheries and Oceans and Jeff Greene for his valuable technical assistance in the compiling of this report.

Yours truly,

Richard Tiller

Richard Tiller

Department of Fisheries and Oceans
Harbours and Infrastructure

St. John's, Newfoundland

"Engineering Analysis of a Timber Crib Retaining Wall"

Prepared for: The Division of Co-Ordination
Memorial University of Newfoundland
St. John's, Newfoundland

By: Richard Tiller
Workterm VI

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TABLE OF CONTENTS

	Page
Summary	iv
Introduction	1
Section I. Timber Crib Retaining Walls in General	4
Design Considerations	4
- Earth Pressure	4
- Geotechnical Considerations	5
- Drainage	7
- Stability Against Sliding and Overturning	9
Timber Crib Construction	10
Section II. Long Harbour Retaining Wall	12
Design Assumptions and Site Conditions	12
Design Results	13
Structural Analysis of Timber Crib Members	14
Conclusions	18
Recommendations	19
References	20
Appendix A: "Active Soil Pressure Determination and External Analysis"	21
Appendix B: "Structural Analysis Calculations of Timber Crib"	30
Appendix C: "Drainage"	46
Appendix D: "Unified Classification System for Soils"	53
Appendix E: "Photographs"	61

SUMMARY

This report details the analysis of a timber crib retaining wall constructed at the Department of Fisheries & Oceans Bait Depot in Long Harbour in 1986.

Lateral active earth pressure is the major design load on a retaining wall, and can be calculated by the Rankine or Coulomb Theories. Furthermore to accurately calculate this pressure, the backfill soil properties must be obtained using geotechnical investigations and by subsequently applying accepted design values.

Since no geotechnical investigation was performed at Long Harbour before construction, a worst case soil condition was assumed and the wall analysed under these conditions. As built, the wall was found to have a factor of safety against sliding of 1.52 and a factor of safety against overturning of 3.31. The bearing capacity of the underlying soil was also determined to be satisfactory.

The longitudinals and the ballast floor beam members were structurally analysed against the internal factors of bending, shear and deflection and found to be structurally secure.

It was concluded that timber crib retaining walls in general can be successfully used in retaining wall applications. This is clearly illustrated by the Long Harbour retaining wall which is safe against the external and internal factors which may cause failure. These walls have several advantages over other designs including ease of construction and inherent drainage properties. To properly design a safe, economical timber crib retaining wall, the analysis should be performed in accordance to the guidelines established in this report.

INTRODUCTION

Retaining walls are structures used to provide stability for soil or other materials where conditions disallow the mass to assume its natural slope.

Retaining walls are classified based on their method of achieving stability. Gravity walls depend upon their self weight to provide stability. Cantilever walls are reinforced concrete walls which use cantilever action to retain a mass. Counterfort walls are modified cantilever walls used generally for heights greater than six meters.

Timber crib construction is more commonly used in wharf design, but, can also be used in retaining wall applications. Timber crib walls have members connected to form rectangular cells or cribs. These cribs are filled with rock ballast to provide stability for the structure.

The Department of Fisheries and Oceans (DFO) used a timber crib retaining wall to retain a steep excavation in the construction of a storage yard at the DFO Bait Depot in Long Harbour, Nfld. in 1986. This 68.1 meter wall was constructed of 1.8 meter wide cribs 2.8 meters long and reaching a height of 3.2 meters at the highest section.

Detailed engineering analysis were not performed for this structure. The wall was designed using basic handbook calculations and the past experience of the project team involved. This report provides a post-construction engineering analysis of this timber crib retaining wall to ensure the structure is safe against the external factors of sliding, overturning and bearing capacity as well as internally structurally secure. The report also proposes some helpful guidelines to ensure future timber crib retaining walls are analysed and designed correctly.

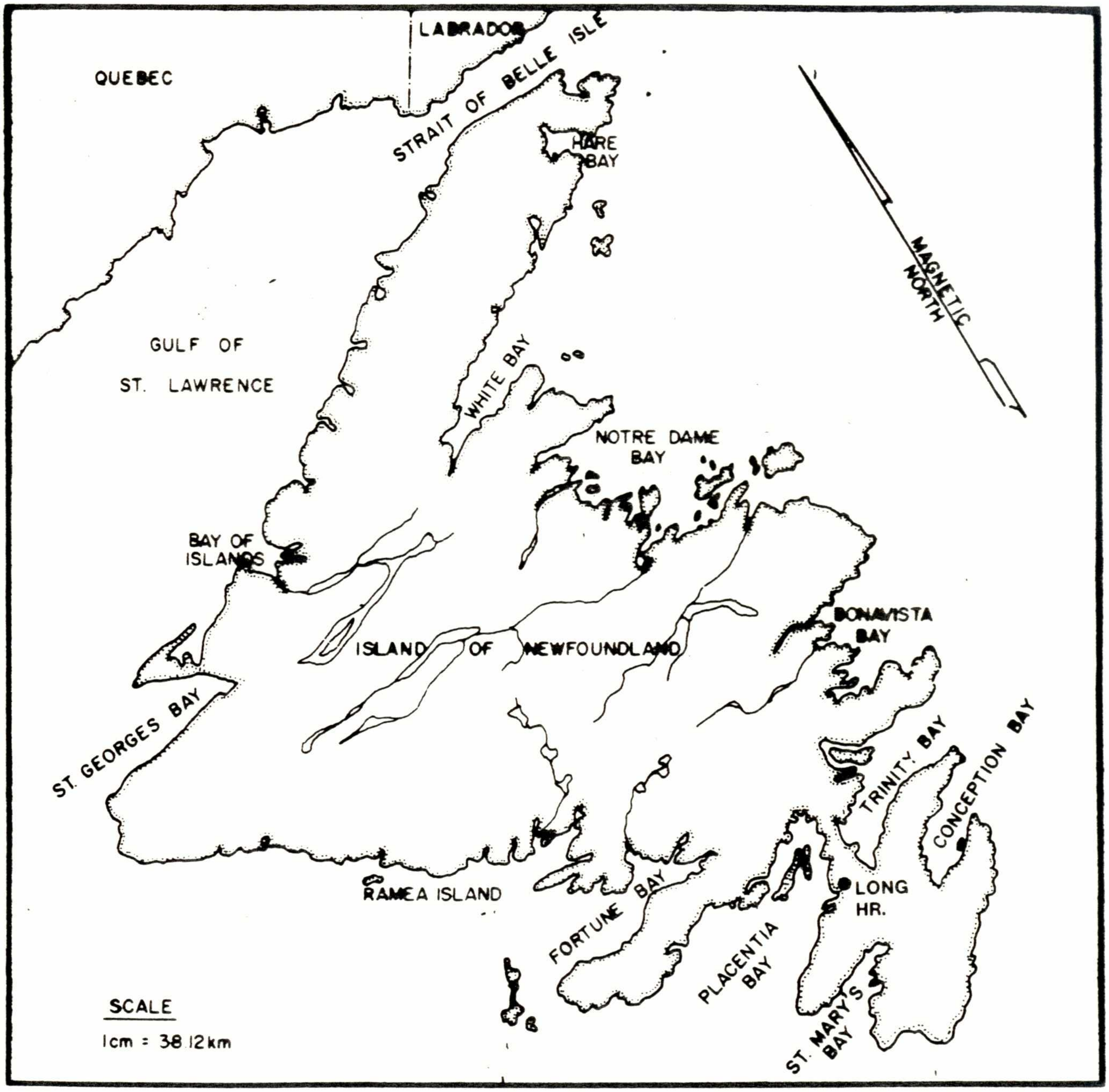

Figure 1. Location of Long Harbour.

SECTION I - TIMBER CRIB RETAINING WALLS IN GENERAL

DESIGN CONSIDERATIONS

EARTH PRESSURE

Retaining walls are subject to lateral pressures from the earth they retain. This lateral pressure is usually the major design parameter in retaining wall analysis.

Under the effect of soil pressure, retaining walls may deflect or move a small amount causing an active soil pressure condition to develop. If the wall moves towards the soil mass, a passive soil condition develops. Both active and passive pressures are dependent on the backfill soil type with granular soils such as sand behaving entirely different than cohesive soils such as clay. The pressures are assumed to vary linearly with the height of the wall and are calculated commonly by either the Rankine or Coulomb theories[1].

Retaining walls are usually designed for the active pressure soil state. This recognizes that if the lateral force is large enough that the system starts to translate or rotate about the toe, the lateral displacement allows the backfill pressure to reduce to the active value[2].

This report will use the Rankine theory of earth pressure. This theory assumes that the retaining wall yields a sufficient amount to develop a state of plastic equilibrium in the soil mass at the wall surface. The rest of the soil remains in the state of elastic equilibrium.

Active soil pressure develops when the retaining wall deflects from the backfill under the lateral soil pressure. The active soil pressure at a depth, h, on a retaining wall is as follows:

$$Pa = Ca \cdot \gamma' \cdot h \quad \& \quad Ha = \tfrac{1}{2} \cdot (Pa) \cdot h$$

where

Ca = Active Earth Pressure Coefficient

$$= \cos\delta \left(\frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \right)$$

ϕ = angle of internal friction of the soil

γ' = unit weight of soil (effective value)

h = height of wall

Ha = resultant active force[3]

GEOTECHNICAL CONSIDERATIONS

The purpose of a geotechnical investigation is to disclose the subsurface soil conditions. The physical properties of soils that most commonly enter into the design of retaining structures are: the angle of internal friction, unit weight of the soil, and location of the water table.

The backfill soil should be sampled and classified according to the Unified Classification System (U.C.S.) presented in Appendix D. A variety of situ and insitu tests exist that can be performed to determine the

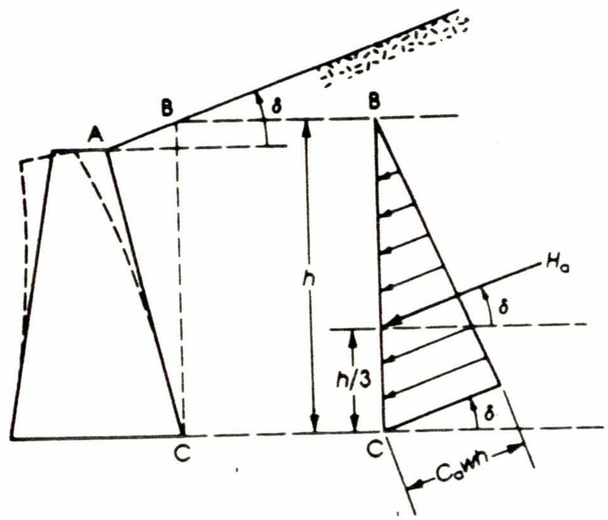

Figure 2. Active Pressure Distribution Behind A Retaining Wall.

Active-earth-pressure coefficients K_a for the Rankine equation

δ, β	φ = 26	28	30	32	34	36	38	40	42
0	0.3905	0.3610	0.3333	0.3073	0.2827	0.2596	0.2379	0.2174	0.1982
5	0.3959	0.3656	0.3372	0.3105	0.2855	0.2620	0.2399	0.2192	0.1997
10	0.4134	0.3802	0.3495	0.3210	0.2944	0.2696	0.2464	0.2247	0.2044
15	0.4480	0.4086	0.3730	0.3405	0.3108	0.2834	0.2581	0.2346	0.2129
20	0.5152	0.4605	0.4142	0.3739	0.3381	0.3060	0.2769	0.2504	0.2262
25	0.6999	0.5727	0.4936	0.4336	0.3847	0.3431	0.3070	0.2750	0.2465
30	0.0000	0.0000	0.8660	0.5741	0.4776	0.4105	0.3582	0.3151	0.2784
35	0.0000	0.0000	0.0000	0.0000	0.0000	0.5971	0.4677	0.3906	0.3340
40	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.7660	0.4668

Table 1. From: Bowles, J.E. "Foundation Analysis and Design", McGraw Hill, Montreal, 1982, p. 389.

required soil properties. Several excellent literature sources have sections devoted exclusively to the subject of situ and insitu testing, including the "Canadian Foundation Engineering Manual" and "Foundation Analysis and Design" by Joseph E. Bowles (see references).

Attempting to save time or money by eliminating or reducing the geotechnical investigation is generally a false economy, as unanticipated conditions frequently result in costly design changes during construction. Undisclosed subsurface conditions may result in an unsafe design or an over-designed structure which results in wasted materials and labor. However, for walls less than six meters in height and where the total cost is not relatively great, a detailed geotechnical investigation need not be undertaken[4]. The backfill should be appropriately sampled and classified according to the U.C.S. and appropriate design values for the required soil properties applied as given in Table 2.

DRAINAGE

If water accumulates behind the retaining wall, the hydrostatic pressure must be included in the design. It is much more desirable to provide soil drainage than to design a retaining wall for the larger lateral pressure which will be induced if the backfill does not readily drain[5].

Drainage is inherent in the design of timber crib retaining walls. The spaces between the crib members along with the voids in the rock ballast provide permanent channels of escape for water that accumulates behind the retaining wall. The possibility of clogging of these channels due to

Table 2. Values of w and ϕ

Type of backfill	Unit weight w, γ pcf	Unit weight w, γ kg/m^3	Angle of internal friction, ϕ
Soft clay	90-120	1440-1920	0°-15°
Medium clay	100-120	1600-1920	15°-30°
Dry loose silt	100-120	1600-1920	27°-30°
Dry dense silt	110-120	1760-1920	30°-35°
Loose sand and gravel	100-130	1600-2100	30°-40°
Dense sand and gravel	120-130	1920-2100	25°-35°
Dry loose sand, well graded	115-130	1840-2100	33°-35°
Dry dense sand, well graded	120-130	1920-2100	42°-46°

From: "Design of Reinforced Concrete Structures", PWS Publishersd, 1985, p. 426.

leaching out of the finer backfill particles may occur. To prevent this drainage systems using gradated filter designs as found in the Canadian Foundation Engineering Manual should be incorporated. (See Appendix C).

STABILITY AGAINST SLIDING AND OVERTURNING

The horizontal component of all forces acting on a retaining wall tend to push it in a horizontal direction. The total frictional resistance force of the base of the wall resisting the sliding effect is

$$F = \mu R$$

where

μ = the coefficient of friction

R = the vertical force acting on the base.

The factor of safety against sliding is

$$F.S = \frac{F}{Hah} \geq 1.5$$

where

Hah = the horizontal component of the active pressure, Ha.

The horizontal component of the active pressure tends to overturn the retaining wall about the toe of the wall. The overturning moment is equal to $M_0 = Ha\ (h/3)$

where h = the height of the wall.

The weight of the wall tends to develop a balancing moment to resist the overturning moment. The balancing moment, Mb, is equal to the product of the weight of the wall and the distance of its center of gravity from the toe of the wall.

The factor of safety against overturning is

$F.s. = Mb/M_o \geq 2.0$ [6].

TIMBER CRIB CONSTRUCTION

The major advantage of timber crib construction is its ease of construction. Timber crib can be assembled using relatively unskilled labour and a minimum of specialized equipment. Figure 3 shows a typical crib retaining wall and its components.

It is highly recommended that timber crib be perservative treated and the connections should be galvanized to prevent decay and to increase the structures service life.

Timber crib retaining walls are designed as gravity walls with the weight of the ballast material and the timber weight taken as the structural self weight.

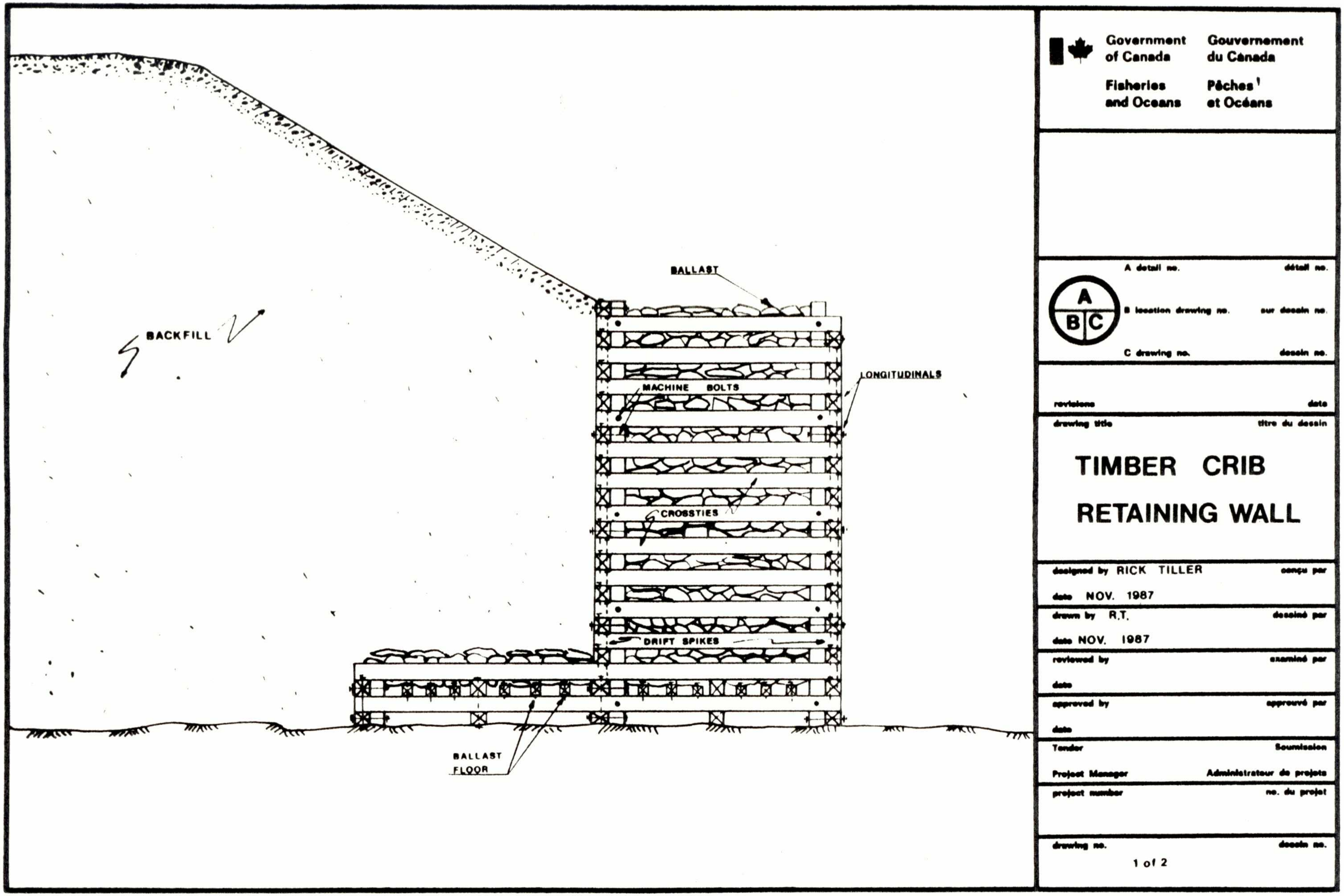

Figure 3. Typical Timber Retaining Wall.

SECTION II

LONG HARBOUR RETAINING WALL

DESIGN ASSUMPTIONS AND SITE CONDITIONS

There was no geotechnical investigation performed at Long Harbour. The backfill was simply identified as a "cohesionless - granular material". In future retaining wall construction it is strongly recommended, if not imperative, that at least a soil identification be performed! As a result of this dilemma, it was decided that a backfill soil representing the worst case scenario would be assumed and the wall analysed under these conditions. If the wall was deemed safe under these conservative conditions, it would perform more than adequately under the existing conditions.

The wall at Long Harbour was constructed of 140 x 140 mm preservative treated timbers connected with 15 mm galvanized drift spikes and 15 mm machine bolts in the configuration shown in Figure 3. Boulders approximately 200-400 mm made up the ballast.

The wall was constructed in three different sections of 24.6, 19.4 and 24.1 meter lengths with heights of 1.7, 3.2, and 2.6 meters respectively. The backfill sloped away from the wall at an approximate angle of 30° to the horizontal.

Longitudinals were extended back from the main crib section to form a tie-back system to add further structural stability to the wall.

The water table was observed to be near or at the top of the excavation, consequently the water table was assumed to be at the surface level and the backfill was analysed under fully saturated conditions.

The design section was taken at the 3.2 m wall section without considering the presence of tie-back longitudinals, this again represented the worst case scenario.

DESIGN RESULTS

A lateral active earth pressure of 40.03 kn/m length was computed to be acting at the base of the retaining wall (see Appendix A). The wall was checked for the following external factors

a) sliding

b) overturning

c) bearing capacity.

a) Using a unit weight of 23.38 kn/m^3 the retaining wall was found to impose a 135.92 kn reaction on the underlying soil. The frictional resistance force of the wall against the soil was 53.00 kn. Therefore the factor of safety against sliding, which is computed by dividing the frictional resistance force by the horizontal active pressure coefficient, was 1.52 which is greater than the recommended value of 1.50.

b) The overturning moment, which is the product of the horizontal component of the lateral active earth pressure and the vertical distance from the toe of the wall, was 36.95 kn/m. The balancing moment for the wall was computed by taking the product of the weight of the wall and the distance from its center of gravity to the toe of the wall. The balancing moment was 122.33 kn/m. Therefore the factor of safety against overturning, which is the balancing moment divided by the overturning moment, was computed as 3.31 which is greater than the recommended value of 2.0 (see Appendix A).

c) The pressure exerted on the underlying soil was found to be a uniform value of 75.51 kn/m^2. Applying a recommended factor of safety of 3.0 against bearing capacity failure[7], any soil other than a "very-soft clay" will yield safe bearing capacity for this wall (see Appendix A). The likelihood of the underlying soil being a "very-soft clay" is very remote.

Consequently, the wall is safe against sliding, overturning, and bearing capacity. This conclusion was confirmed during a recent site visit in which the wall showed no signs of any external failure mode.

STRUCTURAL ANALYSIS OF TIMBER CRIB

The following components of the timber crib wall were analysed.

1) Structural integrity of longitudinals due to active lateral earth pressure.

2) Structural integrity of ballast floor beams with the superimposed load consisting of rock ballast.

3) General analysis of connections.

A) The worst case scenario for loading of the longitudinal occurs at the base of the wall where the active soil pressure of 40.03 kn/m acts. Every fourth longitudinal is bolted to the verticals using 15 mm diameter machine bolts and spiked to the crossties while the intermediate longitudinals are simply spiked to the crossties. In both cases, the members can be modelled by a simply supported beam with a uniformly distributed load of 40.03 kn/m superimposed upon it.

Besides being supported by the connections the longitudinal and crossties are also supported by the rock ballast. This leads to a highly indeterminate system which can be modelled by the beam-system shown in Figure 4.

The longitudinals were analysed for bending, shear, and deflections using the guidelines used in the Timber Design Manual[8]. The members satisfied all design criteria easily except for shear. During the analysis of shear the greatest span between the ballast was calculated only 0.21 meters. However, this considered a member at the base of the crib, a longitudinal half way up the crib can have a safe span of 0.85 meters between the individual ballast. The spacing between the ballast was not likely to be less than 0.21 meters unless undermining was experienced, therefore, the members were concluded to be satisfactory.

B) The ballast floor consists of 140 x 140 mm members spanning the width of the crib-work wall, i.e. 1.8 m. The floor beams are at 300 mm spacing and support ballast of 2.7 m at the highest section.

The ballast was assumed to have a conservative unit weight of 2700 kg/m^3. This led to a uniform distributed load of 21.17 kn/m on each ballast floor beam. Assuming a clear span of 1.8 meters for each beam, the 140 x 140 mm members adequately supported the loading condition (see Appendix B).

C) The connection detail consisted of every fourth longitudinal bolted to the verticals using 15 mm machine bolts. The intermediate longitudinals where spiked to the crossties using 15 mm drift spikes. This connection detail is shown in Figure 3.

No calculations were performed on these connections due to the fact this connection arrangement has been used for years with proven success in situations under for higher loading conditions.

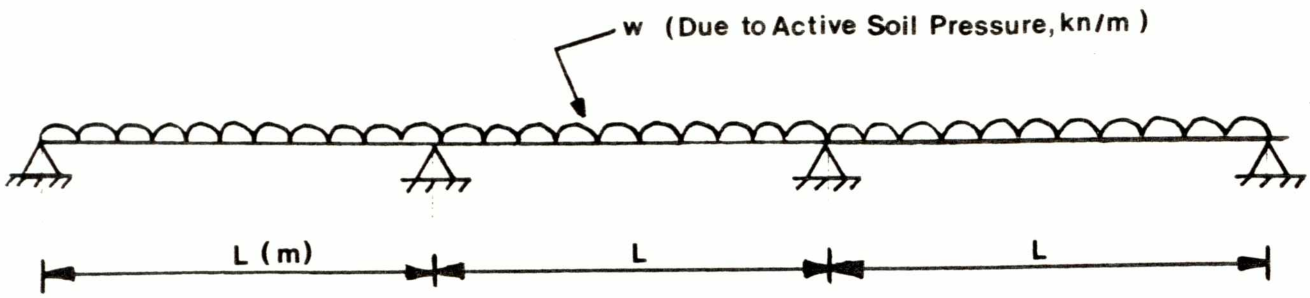

Figure 4. Modelled Beam System for Longitudinal Members.

CONCLUSIONS

Lateral active earth pressure is the major parameter of retaining wall design, and is dependent on the soil properties of the backfill. A properly conducted geotechnical investigation leads to an accurate assessment of these properties. From these a prediction of the lateral active earth pressure may be calculated resulting in a safe, more economical design.

Timber crib construction can be used quite effectively in retaining wall applications. Its advantages over other retaining wall designs are: ease of construction and its drainage properties.

The Long Harbour timber crib retaining wall is safe against the external factors of sliding, overturning and bearing capacity. The wall is also adequate internally, that is, its members adequately resist the bending moments and shear forces imposed upon them by the lateral active earth pressure.

RECOMMENDATIONS

1) In future retaining wall designs a geotechnical investigation consisting of at least a backfill soil identification should be performed.

2) Future retaining wall designs should be performed in a similar manner to the one outlined in this report.

3) Periodic monitoring of the timber crib retaining wall at Long Harbour should be performed. Any visual signs of failure should be noted in order that corrective measures can be implemented to rectify the problem.

REFERENCES

1) Hassoun, M. Nadim, "Design of Reinforced Concrete Structures", PWS Publishers, Boston, 1985, p. 421.

2) Bowles, J.E., "Foundation Analysis and Design", McGraw Hill, Montreal, 1982, p. 436.

3) IBID. 1), pp. 422, 423

4) Canadian Geotechnical Society, "Canadian Foundation Engineering Manual, 2nd Edition", 1985, p. 430.

5) IBID, 2), p. 458.

6) IBID, 1) pp. 427, 428

7) IBID, 4) p. 156

8) Laminated Timber Institute of Canada, "Timber Design Manual", 1980.

APPENDIX A

Active Soil Pressure Determination

SUBJECT:	PAGE NO.	DATE	
LATERAL EARTH PRESSURE CALC.	A-1	NOV 30, 1987	
DESIGNED BY: RICK TILLER (RT)	CHECKED BY:		REFERENCE

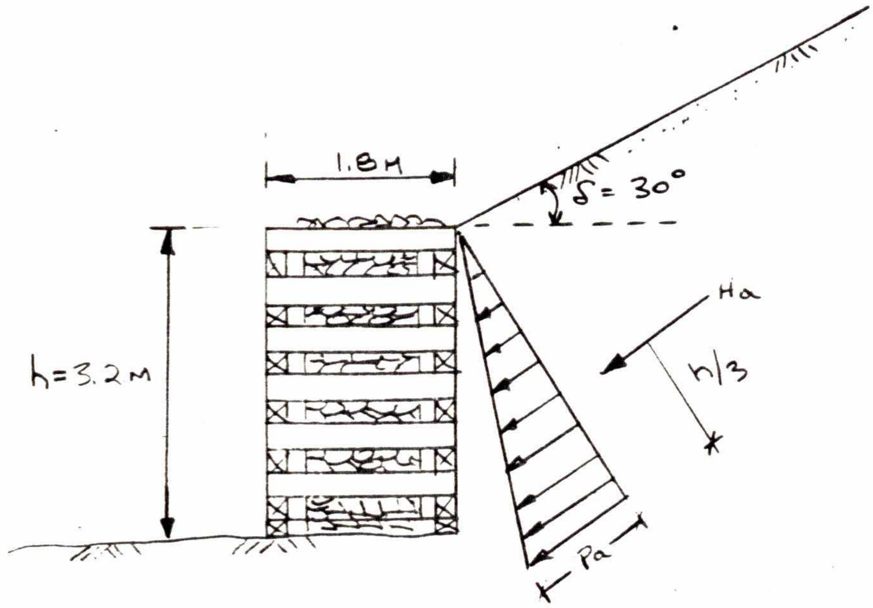

FIG. 1 TIMBER CRIB WALL AT LONG HARBOUR

RANKINE'S THEORY OF LATERAL EARTH PRESSURE WAS USED IN THE CALCULATIONS.

THE BACKFILL WILL BE INITIALLY ASSUMED TO BE A CLAY BACKFILL.

ASSUMED SOIL PROPERTIES:

γ = UNIT WEIGHT OF SOIL = 1920 kg/m^3 = 18.84 kN/m^3 — TABLE 2.

ϕ = ANGLE OF INTERNAL FRICTION = 30° — "

C_a ≡ ACTIVE SOIL PRESSURE COEFFICIENT = 0.866 — TABLE 1.

SUBJECT:	PAGE NO.	DATE
LATERAL EARTH PRESSURE CALC.	A-2	NOV 30/87

DESIGNED BY:	CHECKED BY:		REFERENCE
RC			

$P_a \equiv$ ACTIVE SOIL PRESSURE

$= C_a(\gamma')h$

WHERE:

$\gamma' \equiv$ EFFECTIVE UNIT WEIGHT OF SOIL

$= \gamma_{SAT} - \gamma_{WATER}$

$h \equiv$ HEIGHT OF WALL.

$P_a = 0.866(18.34 - 9.81\ ^{kN}/_{m^3})(3.2m)$

$= 25.02\ kN/m^2$

$H_a \equiv$ ACTIVE SOIL FORCE

$= \frac{1}{2}(P_a)(h) = \frac{1}{2}(25.02\ kN/m^2)(3.2m)$

$= \underline{40.03\ kN/m}$ LENGTH.

ASSUMING A DENSE SAND AND GRAVEL.

$\phi = 35°$

$\gamma = 2100\ ^{kg}/_{m^3} \equiv 20.60\ ^{kN}/_{m^3}$

$C_a = 0.444$

$P_a = 0.444(20.60 - 9.81\ ^{kN}/_{m^3})\ 3.2\ m$

$= 15.33\ kN/m^2$

$H_a = \frac{1}{2}(15.33\ ^{kN}/_{m^2})(3.2\ m) = \underline{24.53\ kN/m}$ LENGTH

SUBJECT: SLIDING CALCULATIONS	PAGE NO. A-3	DATE NOV 30/87
DESIGNED BY: RT	CHECKED BY:	REFERENCE

* THE CLAY BACKFILL EXERTS A HIGHER PASSIVE FORCE THAN THE DENSE SAND CONSEQUENTLY THIS VALUE WILL BE TAKEN AS THE DESIGN VALUE.

SLIDING CALCULATIONS

INORDER TO CALCULATE THE FRICTIONAL FORCE, F, THE REACTION, R, OF THE WALL IS REQUIRED.

VOLUME OF RETAINING
WALL PER METER LENGTH $= 1m \times 3.2\,m \times 1.8\,m$
$= 5.76\ m^3$

UNIT WEIGHT OF RETAINING
WALL PER METER LENGTH

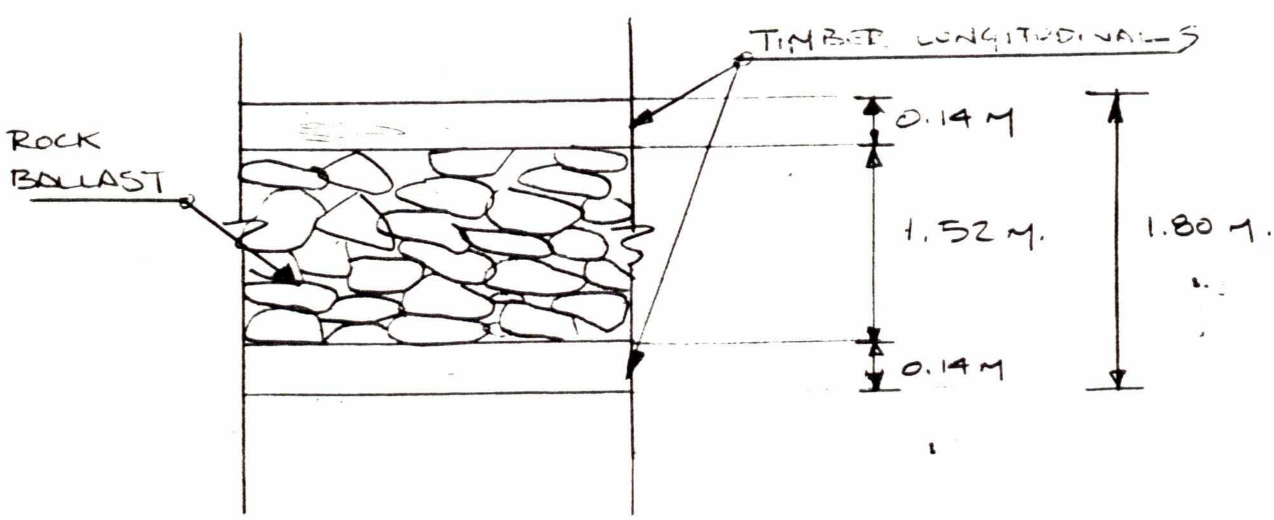

FIG. 2. PLAN VIEW OF CRIB SECTION.

SUBJECT: SLIDING CALCULATIONS	PAGE NO. A-4	DATE NOV 30/97
DESIGNED BY: RT	CHECKED BY:	REFERENCE

THE TIMBER CONSTITUTES $(0.14+0.14)/1.8$ m

$= 15.5\%$

OF THE TOTAL STRUCTURAL VOLUME. THE ROCK BALLAST (AND IT'S VOID SPACES) MAKE UP THE REMAINING 84.5%.

THE COMPOSITE UNIT WEIGHT OF THE TIMBER CRIB IS:

0.155(UNIT WEIGHT OF PRESERVATIVE TIMBER) + 0.845(UNIT WEIGHT OF ROCK BALLAST)

$= 0.155(800\ kg/m^3) + 0.845(2700\ kg/m^3)$ TABLE 4-11 p.167 FOUNDAT ANALYSI AND DESIGN BY BOW

$= 2405.50\ kg/m^3$

$= 23.60\ kN/m^3$

REACTION, R, OF WALL ON SOIL

$= 23.60\ kN/m^3\ (5.76\ m^3)$

$= \underline{135.92\ kN}$

COEFFICIENT OF FRICTION BETWEEN THE WALL AND THE UNDERLYING SOIL $= 0.67 \tan \phi$ p44 BOWLE

$= 0.67 \tan 30°$

$= 0.39$

SUBJECT: SLIDING AND OVERTURNING CALC.		PAGE NO. A-5	DATE NOV 30/97
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FRICTIONAL RESISTANCE FORCE, $F = \mu R$

$F = 0.39(135.92)$ kN

$F = \underline{\underline{53.00 \text{ kN}}}$

$Ha = 40.03$ kN/m

$Ha \cos\delta = (40.03 \text{ kN/m}) \cos 30° = \underline{\underline{34.67 \text{ kN}}}$

FACTOR OF SAFETY AGAINST SLIDING, $F.S. = F/(Ha \cos\delta)$

$= 53.00/34.67$

$= \underline{1.52}$ WHICH IS GREATER THAN THE RECOMMENDED VALUE OF 1.5 ∴ THE WALL IS SAFE AGAINST SLIDING!

OVERTURNING CALCULATIONS

OVERTURNING MOMENT (ABOUT TOE OF WALL) $= (Ha \cos\delta) \times h/3$

$= 34.64 \text{ kN} \times 3.2^{m}/3$

$= 36.95$ kN·m

BALANCING MOMENT = (WEIGHT OF WALL) x (DIST. CENTER GRAVITY TO TOE OF WALL)

$= W(\bar{x})$

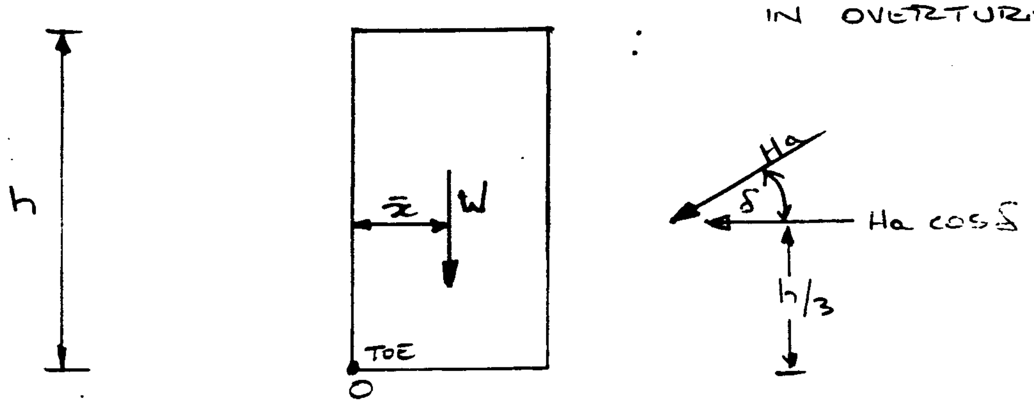
$= 135.92 \text{ kN} (0.9 \text{ m})$

$= 122.33$ kN·m

SUBJECT:	PAGE NO.	DATE
OVERTURNING AND BEARING CAPACITY ANALYSIS.	A-6	NOV 30/97

DESIGNED BY:	CHECKED BY:		REFERENCE
RT			

FIG. 3 FORCES INVOLVED IN OVERTURNING



FACTOR OF SAFETY AGAINST SLIDING

$$= \frac{\text{BALANCING MOMENT}}{\text{OVERTURNING MOMENT}}$$

$$= \frac{122.33 \text{ kN m}}{36.95 \text{ kN m}} = \underline{\underline{3.31}}$$

WHICH IS GREATER THAN THE RECOMMENDED VALUE OF 2.0 ∴ THE WALL IS SAFE AGAINST OVERTURNING!

ALLOWABLE BEARING CAPACITY OF UNDERLYING SOIL

THE PRESSURE EXERTED ON THE UNDERLYING SOIL, q, IS UNIFORM.

FOR A METER LENGTH, AREA OF BASE OF RETAINING WALL = $1 m \times 1.8 m = 1.8 m^2$

$$q = \frac{W}{A} = 135.92 \text{ kN} / 1.8 m^2 = \underline{75.51 \text{ kN/m}^2}$$

SUBJECT:	PAGE NO.	DATE
BEARING CAPACITY ANALYSIS	A-7	NOV 30/87

DESIGNED BY:	CHECKED BY:		REFERENCE
RD			

USING TABLE 8.1 OF THE "CANADIAN FOUNDATION ENGINEERING MANUAL" ANY SOIL OTHER THAN SOFT CLAY AND SILT WILL YIELD A SAFE BEARING CAPACITY VALUE FOR THIS STRUCTURE

p 139 "CANADIAN FOUNDATION ENGINEERING MANUAL", 1985

TABLE 8.1 Presumed allowable bearing pressure

presumed values of the allowable bearing pressure are estimates and may need to be adjusted upwards or downwards in ...ific case. No consideration has been made for the depth of embedment of the foundation. Reference should be made ...r parts of the Manual when using this table.

	Types and conditions of rocks and soils	Strength of Rock Material	Presumed Allowable Bearing Pressure (kPa)	Remarks
	Massive igneous and metamorphic rocks (granite, diorite, basalt, gneiss) in sound condition (2)	High to very high	10 000	These values are based on the assumption that the foundations are carrried down to unweathered rock.
	Foliated metamorphic rocks (slate, schist) in sound condition (1) (2)	Medium to high	3000	
	Sedimentary rocks: cemented shale, siltstone, sandstone, limestone without cavities, thoroughly cemented conglomerates, all in sound condition (1) (2)	Medium to high	1000 - 4000	
	Compaction shale and other argillaceous rocks in sound condition (2) (4)	Low to medium	500	
	Broken rocks of any kind with moderately close spacing of discontinuities (0.3 m or greater), except argillaceous rocks (shale)		1000	
	Limestone, sandstone, shale with closely spaced bedding		(See note 3)	
	Heavily shattered or weathered rocks		(See note 3)	
Coarse-grained soil	Dense gravel or dense sand and gravel		>600	Width of foundation (B) not smaller than 1 m. Groundwater level is assumed to be at a depth equal to B or more than B below the base of the foundation.
	Compact gravel or compact sand and gravel		200 - 600	
	Loose gravel or loose sand and gravel		<200	
	Dense sand		>300	
	Compact sand		100 - 300	
	Loose sand		<100	
Fine-grained soil	Very stiff to hard clays or heterogeneous mixtures such as till		300 - 600	Fine-grained soils are susceptible to long-term consolidation settlement due to imposed loads and are often susceptible to severe swelling or shrinking due to changed moisture conditions. If the Plasticity Index (Ip) exceeds 30 and the clay content exceeds 25%, the long-term performance of the foundation may be significantly affected by swelling or shrinking of the subsoils, and a complete assessment of these possibilities is necessary as discussed in Chapter 17.
	Stiff clays		150 - 300	
	Firm clays		75 - 150	
	Soft clays and silts		<75	
	Very soft clays and silts		not applicable	
Organic soils	Peat and organic soils		not applicable	
Fill	Fill		not applicable	

NOTES

(1) The above values for sedimentary or foliated rocks apply where the strata or the foliation are level or nearly so, and, then, only if the area has ample lateral support. Tilted strata and their relation to nearby slopes or excavations should be assessed by a person knowledgeable in this field of work.

(2) Sound rock conditions allow minor cracks at spacing not closer than 1 m.

(3) To be assessed by examination in-situ, including test loading if necessary.

(4) These rocks are apt to swell on release of stress, and on exposure to water they are apt to soften and swell.

* FROM THE "CANADIAN FOUNDATION ENGINEERING MANUAL, 2ND EDITION", 1995

APPENDIX B

Structural Analysis Calculations of Timber Crib

SUBJECT: TIMBER STRUCTURAL ANALYSIS	PAGE NO. B-1	DATE DEC 1, 1987
DESIGNED BY: RICK TILLER (RT)	CHECKED BY:	REFERENCE

SECTION I: LONGITUDINALS

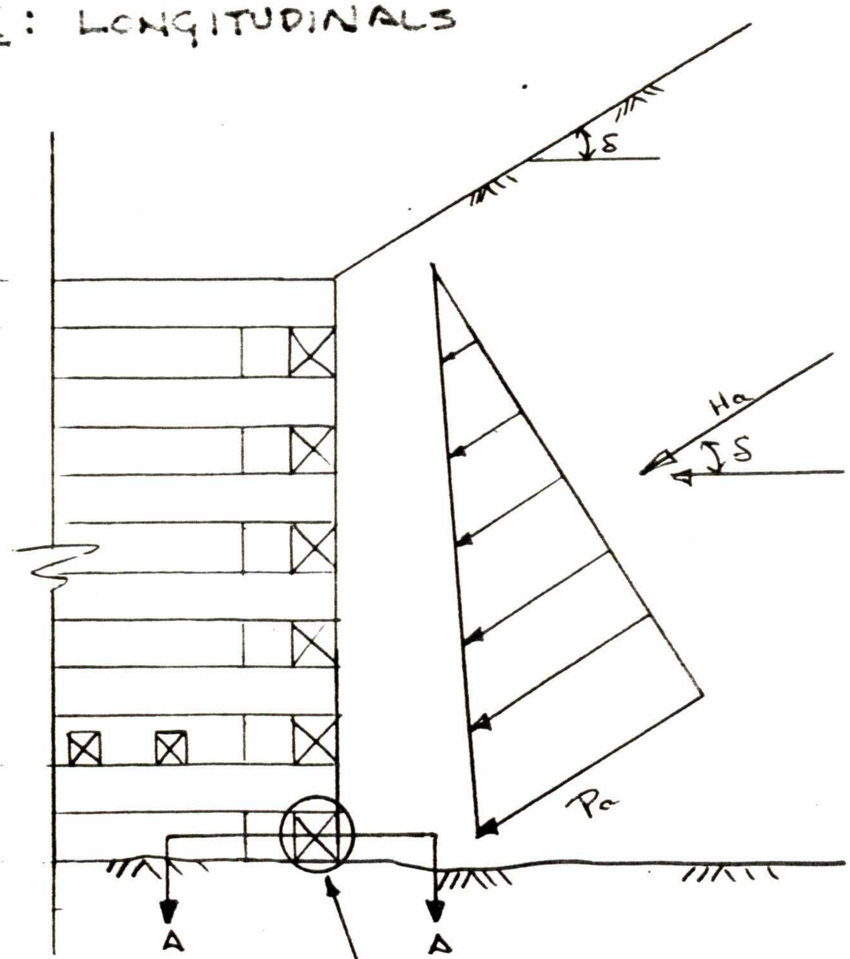
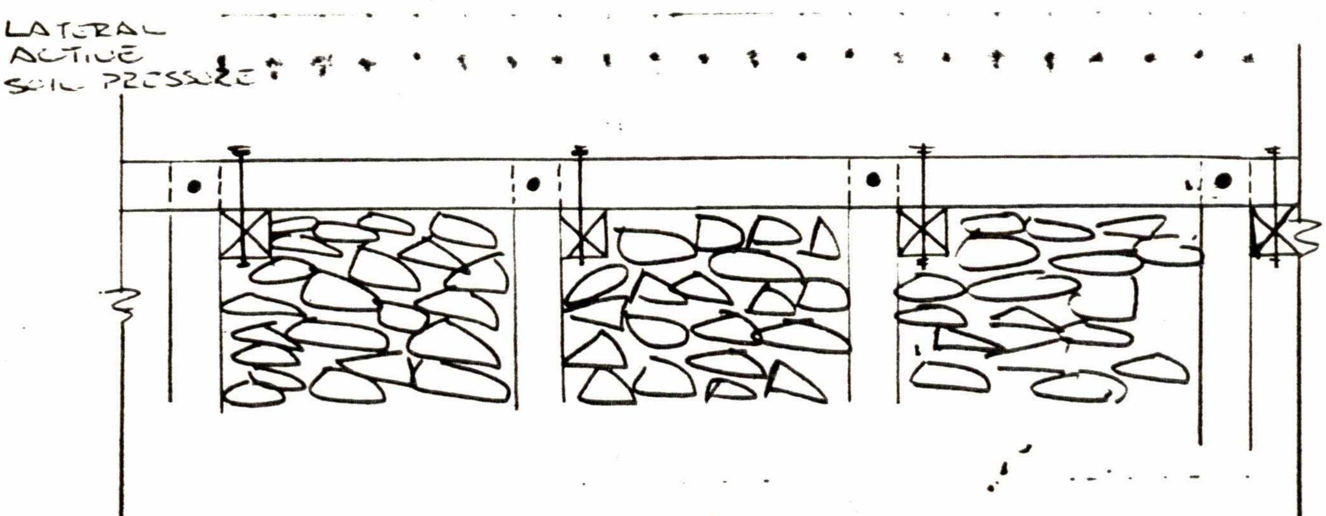


FIG. 1

SECTION A-A

DESIGN MEMBER



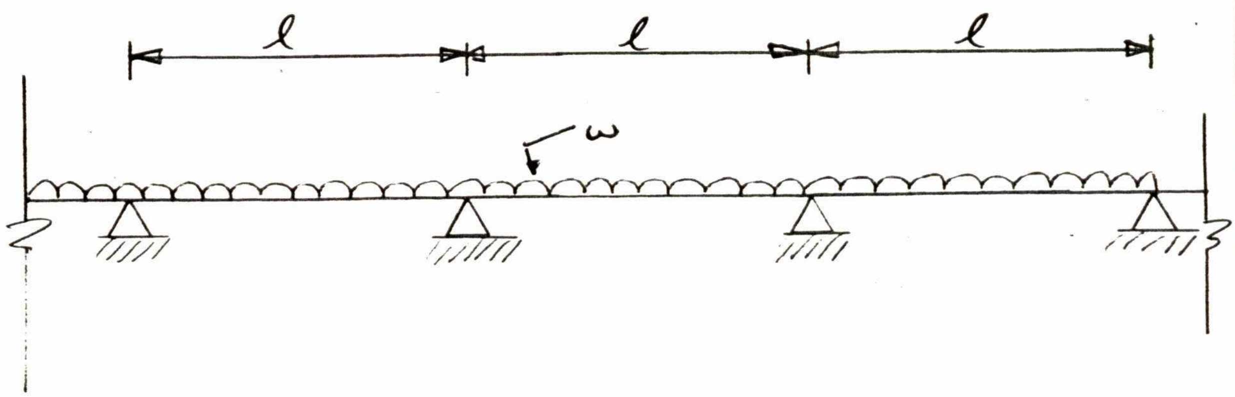
PLAN VIEW

SUBJECT: TIMBER STRUCT. ANALYSIS	PAGE NO. B-2	DATE DEC 1/87
DESIGNED BY: RT	CHECKED BY:	REFERENCE

ANALOGOUS BEAM SYSTEM FOR SECTION A-A

ASSUMPTIONS :

- MACHINE BOLTS, AND DRIFT SPIKES ARE SIMPLY-SUPPORTED CONNECTIONS.
- BALLAST PROVIDES SUPPORT TO LONGITUDINALS AND CAN ALSO BE CONSIDERED AS SIMPLY SUPPORTED CONNECTIONS.
- THE SPACING OF THE BALLAST, ℓ, IS EQUAL.



$w \equiv$ ACTIVE SOIL PRESSURE

$= 40.03$ kN/m (CALCULATED IN APPENDIX A.)

SUBJECT: TIMBER: STRUCT. ANALYSIS	PAGE NO. B-3	DATE DEC 1/87	
DESIGNED BY: RICK TILLER	CHECKED BY:		REFERENCE

THE LONGITUDINAL MEMBERS ARE:

- 140 x 140 MM., No. 1 GRADE, PRESERVATE (CRESOTE) TREATED.

- SPECIES GROUP, D ⇒ S.P.F. (SPRUCE, PINE, FIR). — p. 12 "TIMBER DESIGN MANUAL" (T.D.M. 1980.

THE DESIGN OF TIMBER BENDING MEMBERS, SUCH AS THE LONGITUDINALS OF CRIBS, IS GOVERNED BY:

1) BENDING
2) SHEAR
3) DEFLECTION.

1) BENDING

THE GENERAL EQ. USED FOR BENDING IS:

$$\boxed{S = \frac{bd^2}{6} = \frac{M \cdot K_M}{F'b} \times 10^6}$$

p 56 T.D.M

WHERE:

$S =$ SECTION MODULUS, mm^3
$b =$ WIDTH, mm
$d =$ DEPTH, mm
$K_M =$ MOMENT FACTOR
$= 1.00$
$F'b =$ ALLOWABLE WORKING STRESS IN BENDING.

SUBJECT:	PAGE NO.	DATE
TIMBER: STRUCT. ANALYSIS	B-4	DEC 1/97

DESIGNED BY:	CHECKED BY:		REFERENCE
RT			

FOR SAWN TIMBER:

$$F'b = Fb \cdot K_{sb} \cdot K_F \cdot K_D$$ p 56 T.D.M

WHERE

Fb = ALLOWABLE UNIT STRESS IN BENDING AT EXTREME FIBER AT NORMAL DURATION OF LOAD FOR THE APPROPRIATE STRESS GRADE AND SPECIES GROUP. = 5.7 MPa — TABLE p.18 T.D.M

K_{sb} = SERVICE CONDITION FOR BENDING = 1.00 — p.22 T.D.M

K_F = TREATMENT FACTOR
= 1.00 FOR PERSERVATIVELY TREATED TIMBER — p 56 T.D.M.

K_D = LOAD DURATION FACTOR.
= 0.90 FOR RETAINING WALLS — TABLE 9, p2 T.D.M

$$\therefore \; F'_b = (5.7 \text{ MPa})(1.00)(1.00)(0.90)$$

$$= 5.13 \text{ MPa}.$$

SUBJECT:	PAGE NO.	DATE
TIMBER: STRUCTURAL ANALYSIS	B-5	DEC 1/87

DESIGNED BY:	CHECKED BY:		REFERENCE
RT			

$$S = \frac{(140)(140)^2}{6} = \frac{M\,(1.0)}{5.13} \times 10^6$$

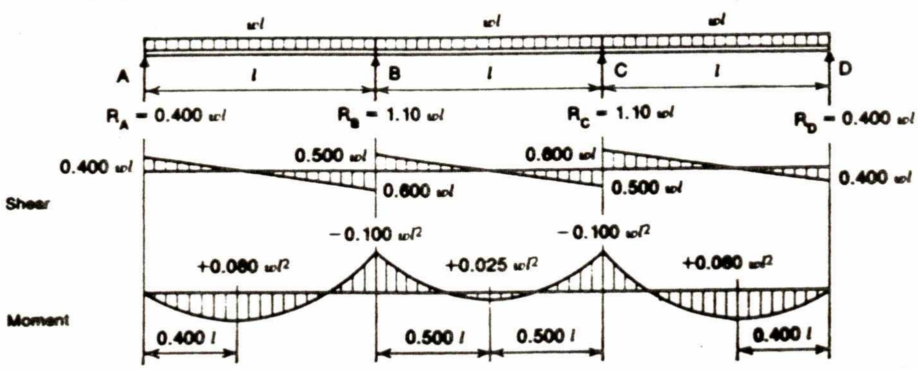
M ≡ MAX. MOMENT WHICH CAN BE APPLIED

$= \underline{\underline{2.36 \text{ kN·m}}}$.

FOR A CONTINUOUS BEAM – THREE EQUAL SPANS – ALL SPANS UNIFORMLY LOADED. THE FOLLOWING SHEAR AND MOMENT DIAGRAMS RESULT :

40. CONTINUOUS BEAM—THREE EQUAL SPANS— ALL SPANS LOADED

p 156
T.D.M



Δ Max. (0.446 l from A or D) = 0.0069 wl⁴/EI

THE MAXIMUM MOMENT IS $-0.100\, w\, \ell^2$ OCCURING OVER THE SUPPORTS.

THE MAXIMUM SPAN (ie DIST. BETWEEN BALLAST) , ℓ, IS :

$$\text{Moment}_{MAX} = -0.100\, w\, \ell^2$$

$$2.36 \text{ kN}\cdot\text{M} = -0.100\,(40.03 \text{ kN/m})\, \ell^2$$

$$4.00\, \ell^2 = 2.36$$
$$\ell^2 = 0.59$$

$$\ell = \underline{\underline{0.77 \text{ M.}}}$$

ANY SPACING LESS THAN 0.77M WILL SATISFY BENDING REQUIREMENTS. THE SPACING BETWEEN BALLAST IS UNLIKEY TO BE GREATER THAN 0.77M
∴ THE LONGITUDINALS SATISFY BENDING REQUIREMENTS.

2) SHEAR

THE GENERAL EQUATION IS :

$$\boxed{A = bd = \frac{1.5 V_e \cdot K_N}{F'_v} \times 10^3}$$

p.57 TDM

SUBJECT: TIMBER STRUCT. ANALYSIS

PAGE NO. B-7

DATE DEC 1/97

DESIGNED BY: RT

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REFERENCE

WHERE:

$A =$ AREA, mm^2

$V_e =$ EFFECTIVE SHEAR FORCE, KN

$K_N =$ NOTCH FACTOR = 1.0 FOR NO NOTCH.

$F'_V =$ ALLOWABLE WORKING STRESS IN LONGITUDINAL SHEAR, MPa

$$F'_V = F_V \cdot K_{SV} \cdot K_F \cdot K_D$$

$F_V = 0.43$ MPa — TABLE p 18 TDM

$K_{SV} = 1.00$ — TABLE 1 p 22 TDM

$K_F = 1.00$

$K_D = 0.90$

$$\therefore \quad F'_V = 0.43(1.0)(1.0)(0.9)$$
$$= 0.39 \text{ MPa}.$$

$$A = bd = (140)^2 = \frac{1.5(V_e)\,1.0}{0.39} \times 10^3$$

$$V_e = 5.09 \text{ kN}$$

$\therefore$ MAX SHEAR PERMITTED IS 5.09 kN.

SUBJECT: TIMBER STRUCT. ANALYSIS		PAGE NO. B-8	DATE DEC 1/07
DESIGNED BY: RU	CHECKED BY:		REFERENCE

FROM THE SHEAR DIAGRAM FOR THE BEAM SYSTEM, MAX. SHEAR = $0.60\, w \cdot (\ell)$

$$\therefore\ 5.09\ kN = 0.60\ w\,(\ell)$$
$$= 0.60\,(40.03)\,\ell$$

$$5.09 = 24.02\,\ell$$

$$\ell = \underline{0.21\ M}.$$

THIS VALUE FOR SPAN IS RELATIVELY LOW. HOWEVER, ONE MUST CONSIDER THIS IS FOR A LONGITUDINAL AT THE BASE OF THE WALL.

A LONGITUDINAL HALF-WAY UP THE HEIGHT OF THE WALL WILL NOW BE EXAMINED

$$P_a = C_a(\gamma')\,h$$

$$h = 1.6\ M \quad \text{NOW}$$

$$P_a = (0.866)(18.84 - 9.81)(1.6)$$
$$= 12.51\ kN/M^2$$

$$H_a = \tfrac{1}{2}(P_a)\,h$$
$$= \tfrac{1}{2}(12.51)(1.6) = 10.01\ kN/M$$

$$5.09\ kN \cdot M = 0.60\,(10.01)\,\ell$$

$$\ell = \underline{\underline{0.85\ M}}$$

SUBJECT:	PAGE NO.	DATE
TIMBER : STRUCT ANALY.	3-9	DEC 1/87

DESIGNED BY:	CHECKED BY:		REFERENCE
RC			

IF THE BALLAST SPACING IS GREATER THAN 0.21 M. FOR A BASE LONGITUDINAL SHEAR MAY PRESENT A PROBLEM TO THE STRUCTURAL INTEGRITY OF THE WALL. HOWEVER, THE DESIGN SHOULD BE SAFE FOR THE FOLLOWING REASONS ...

* 1) THE ACTIVE SOIL PRESSURE IS OVERESTIMATED.

2) A LONGITUDINAL HALF-WAY UP THE WALL CAN HAVE A SAFE SPAN OF 0.85 M, A REASONABLE SPACING.

3) EVEN IF THESE MEMBERS EXPERIENCED SHEAR FAILURE IT WOULD NOT AFFECT THE WALL'S STABILITY TO ANY GREAT EXTENT.

3) DEFLECTION

$\Delta \equiv$ DEFLECTION

$$\Delta MAX = \left(0.0069\ w\ l^4\right) / (E'I) \times 10^{12}$$ p 64 TDM

ASSUME $l = 0.85$ M

$w = 40.03$ kN·M

$E' = E(K_{SE})\,K_F$ p 19, 22 TDM

$= 7500\,(1.00)(1.00) = 7500$ MPa.

$$I = \frac{b(h)^3}{12} = \frac{(140)^4}{12} = 32.01 \times 10^6 \text{ mm}^4$$

$$\Delta MAX = \frac{\left[0.0069 \cdot (40.03) \cdot (0.85)^4\right]}{7500 \cdot 32.01 \times 10^6} \times 10^{12}$$

$= 0.60$ MM

SUBJECT:	PAGE NO.	DATE
TIMBER : STRUCT ANAY	B-10	DEC 1/87

DESIGNED BY:	CHECKED BY:		REFERENCE
RJ			

THE MAXIMUM ALLOWABLE DEFLECTION

$= L/180$ p.65
$= 0.85m/180$ T.D.M

$= 4.72$ mm.

∴ THE MEMBERS ARE MORE THEN ADEQUATE CONSIDERING DEFLECTON.

SUBJECT:	PAGE NO.	DATE
TIMBER: STRUCT ANALYSIS	B-11	DEC 1/97

DESIGNED BY:	CHECKED BY:		REFERENCE
(RT)			

SECTION II: BALLAST FLOOR BEAMS

1) BENDING

THE MAXIMUM MOMENT THAT A 140 x 140 MM PRESERVATIVELY TREATED, No. 1 GRADE, S.P.F. MEMBER SUCH AS THE BALLAST FLOOR BEAMS CAN WITHSTAND WAS PREVIOUSLY CALCULATED IN SECTION I OF THIS APPENDIX.

$M_{MAX} = \underline{2.88\ kN \cdot M}$

THE BALLAST FLOOR BEAMS HAVE A CLEAR SPAN OF 1.52 M. AND ARE PLACED 300 MM ON CENTER.

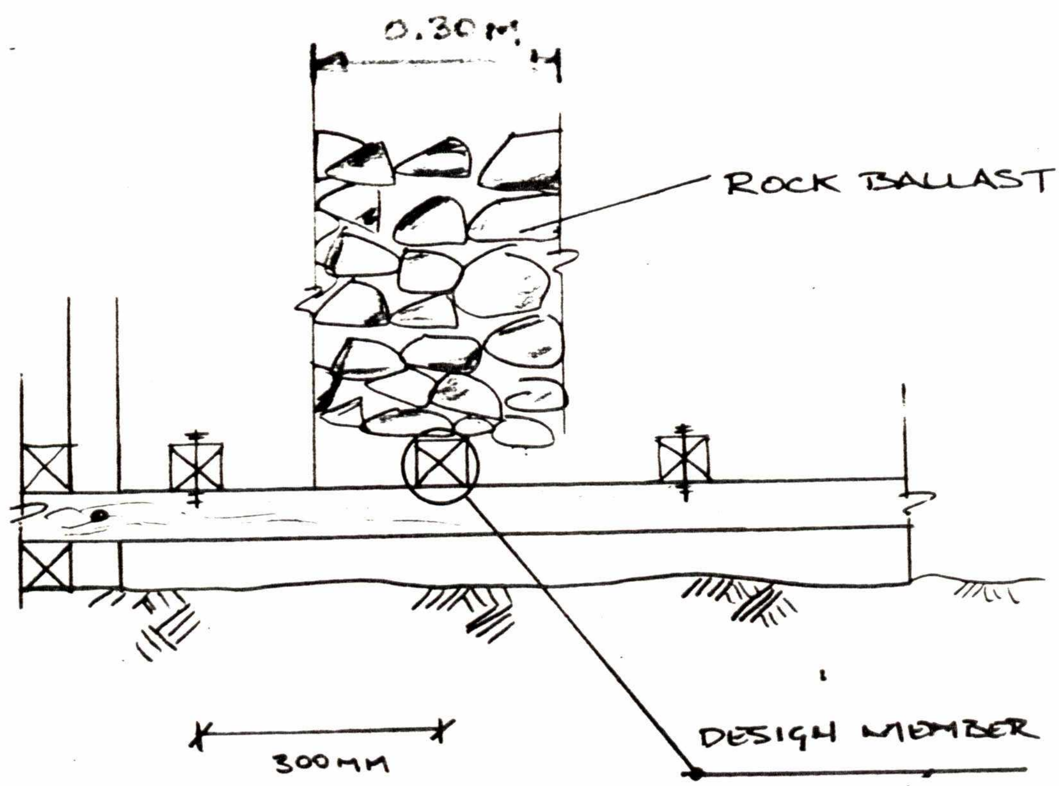

FIG. 2. LOADING ON DESIGN MEMBER

SUBJECT: TIMBER : STRUCT ANALYSIS

PAGE NO. B-12

DATE DEC 1 /87

DESIGNED BY: RT

CHECKED BY:

REFERENCE

EACH DESIGN MEMBER SUPPORTS A LOAD OVER 0.30 M BY 1.52 M LONG. THIS YIELDS A TRIBUTARY AREA = 0.45 M^2

∴ BALLAST LOADING
<u>PER DESIGN MEMBER</u> = TRIBUT. AREA × HEIGHT OF BALLAST × UNIT WEIGHT OF BALLAST

$= (0.45\ m^2) \times (2.7\ m) \times 2700\ kg/m^3$

$= 3280.50\ kg$
$= 32.18\ kN$

UNIFORM DISTRIBUTED
<u>LOAD PER DESIGN MEMBER</u> = 32.61 kN / 1.52 M SPAN

$= 21.17\ kN/M \equiv w$

ASSUMING A SIMPLY-SUPPORTED SPAN OF 1.8 M, THE MAXIMUM MOMENT = $\frac{wL^2}{8}$

$= \frac{21.17\,(1.8)^2}{8}$

$= 8.57\ kN \cdot M$ WHICH IS > 2.88 kN·M
∴ <u>NOT</u> OK!

SUBJECT: TIMBER: STRUCT. ANALYSIS	PAGE NO. B-13	DATE DEC 1/87
DESIGNED BY: R-D	CHECKED BY:	REFERENCE

THE FLOOR BEAM OBVIOUSLY FAILS. HOWEVER AS WITH LONGITUDINAL MEMBERS, THE FLOOR BEAMS ARE INTERMEDIATELY SUPPORTED BY THE UNDERLYING SOIL LEADING TO A HIGHLY INDETERMINATE SYSTEM.

THE MAXIMUM MOMENT FOR A CONTINUOUS BEAM WITH A UNIFORM DIST. LOAD

$= -0.100\, w\, l^2$ (OVER THE INTERMEDIATE SUPPORTS) — p 157 T.D.M.

THE MAXIMUM ALLOWABLE APPLIED MOMENT $= 2.88$ kN·m

THE MAXIMUM SPAN, l =

$$2.88 \text{ kN·m} = -0.100\, w\, l^2$$
$$= 0.100\,(21.46)\, l^2$$
$$= 2.146\, l^2$$

$$l = 1.16 \text{ m.}$$

A SPAN OF GREATER THAN 1.16 M WILL RESULT IN BEAM FAILURE, HOWEVER IT IS UNLIKELY THAT THE MAJORITY, IF ANY OF THE FLOOR BEAMS, ACTUALLY SPAN THIS LENGTH ∴ THE BALLAST FLOOR BEAMS MET THE MOMENT REQUIREMENTS.

2) SHEAR

THE BALLAST FLOOR BEAMS ARE IDENTICAL TO LONGITUDINALS. THE MAXIMUM ALLOWABLE SHEAR PERMITTED FOR THESE MEMBERS IS 5.09 kN (CALCULATED IN SECTION I APPENDIX B)

$$5.09 = 0.60(w)\,l$$
$$= 0.60(21.17)\,l$$
$$= 12.88\,l$$

$$\underline{l = 0.40\ m}$$

SHEAR MAY PRESENT A PROBLEM

THIS SPAN LENGTH IS UNDER-ESTIMATED FOR ANY OF THE FOLLOWING REASONS:

1) UNIT WEIGHT OF BALLAST IS OVER-ESTIMATED A MORE CONSERVATIVE VALUE IS 2200 kg/m^3 WHEN VOID SPACE IS CONSIDERED.

2) IN MOST WALL LOCATIONS THE FLOOR BEAMS ARE RESTING ENTIRELY ON THE SOIL.

3) EVEN IF A BEAM DOES FAIL BY SHEAR, THIS WILL NOT RESULT IN CATASTROPHIC STRUCTURAL FAILURE.

SUBJECT:	PAGE NO.	DATE
TIMBER STRUCT. ANALYSIS	B-15	DEC 1/97

DESIGNED BY:	CHECKED BY:		REFERENCE
(RJ)			

AS A RESULT OF THESE STATEMENTS THE FLOOR BEAM MEMBERS ARE SATISFACTORY FOR SHEAR AND ARE THEREFORE OVERALL SATISFACTORY UNDER THE LOADING CONDITIONS.

APPENDIX C

Drainage

From: "Canadian Foundation Engineering Manual, 2^{nd} Edition", 1985.

CHAPTER 14

DRAINAGE

14.1 INTRODUCTION

Below-ground facilities, such as building basements, inevitably collect water unless protected by adequate subsurface interceptor drains. These drains must provide permanent channels of escape for water that would otherwise impair the use of the structure. The detrimental effects of water on below-ground facilities and structures are manifested in three general ways: (1) probably the most significant effect is the ingress and presence of water in spaces that should be and were intended to be dry; (2) a secondary effect is from water containing salt, which is corrosive to Portland cement concrete and is particularly detrimental in parking garages; and (3) a third effect is the potential reduction of shear strength in soils when subjected to an increase of pore-water pressure.

Subsurface drain pipes are surrounded by a filter and are intended to provide the necessary channels of escape. These must possess hydraulic, structural, and durability characteristics such that they will adequately carry away the water and safely support the loads to which they will be subjected during and after construction. The satisfactory functioning of many projects is related directly to the adequate control of subsurface water.

14.2 FILTER DESIGN

Filter materials, such as grades of natural sands and gravels and geotextiles, are used to retain erodable soils. Filter materials must possess grain-size and permeability characteristics compatible with the grain-size distribution of the soil being drained and the size, location, and distribution of perforations in the drain pipe. The term 'compatible' used in this context means that:

- the pore spaces in the filters must be small enough to prevent particles from adjacent erodible soils from being washed through them, 'filtration criteria';

- the pore spaces must be large enough to permit water to escape freely and thus provide control over seepage forces and hydrostatic pressures, 'permeability criteria';

- the filter particles must be coarse enough to prevent any significant amount of the filter material being washed through the perforations in the drain pipes;

- the filter material must be chemically stable and inert to the water and soil with which it will be in contact; and

- the filter must be physically strong and sufficiently durable to support the loads that will be placed on them during and after construction.

14.2.1 SOIL FILTERS

The theory and rational approach to filter design has been presented by Bertram (1940), Karpoff (1955), and Sherard et al., (1984a,b). Filter design is based on the phenomenon that if perfect spheres have diameters greater than six and one-half times the diameter of a smaller sphere, the smaller sphere can move between the larger spheres (Taylor, 1948).

Because soils vary in particle size, shape, and grading, criteria have been developed from both experimental and theoretical considerations. Various criteria are available; a common one is that of the U.S. Bureau of Reclamation (1974).

For the filtration criteria:

(a) for uniform soils,

when $D_{60}(\text{soil})/D_{10}(\text{soil}) \leq 1.5$

then $D_{15}(\text{filter}) \leq 6 \times D_{85}(\text{soil})$

(b) for well graded soils,

when $D_{60}(\text{soil})/D_{10}(\text{soil}) \geq 4.0$

then $D_{15}(\text{filter}) \leq 40 \times D_{15}(\text{soil})$

For the permeability criteria:

$$D\ (\text{filter}) > 4 \times D\ (\text{soil})$$

Throughout this section, D_{15} (filter) is used to designate the 15% size of the filter material, that is, the size of the sieve that allows 15% by weight of the filter material to pass through it. Similarly, D_{85} (soil) designates the size of sieve that allows 85% by weight of the base soil to pass through it.

Particle sizes smaller than the No. 200 sieve (0.075 mm) refer to results by hydrometer analysis.

An alternative criterion is that of the U.S. Bureau of Reclamation (1974).

For the filtration criteria:

$$D_{15}(\text{soil}) \leq 5\, D_{85}(\text{soil})$$

$$D_{50}(\text{filter}) \leq 25\, D_{50}(\text{soil})$$

For the permeability criteria:

$$D_{15}(\text{filter}) \geq 5\, D_{15}(\text{soil})$$

The design of the filter is dependent upon the gradation curve of the soil to be protected; therefore, gradation curves should be included in site investigation reports, particularly when it is suspected that filter criteria will be a design consideration.

14.2.2 CONCRETE SAND

Experience shows that filters consisting of well graded sand provide adequate protection for a wide range of soils.

14.2.3 DRAINAGE PIPE OR TILE

To avoid clogging of perforated drainage pipes, the maximum circular pipe-hole sizes should be equal to D_{85}(filter) of the immediate backfill. For slotted pipes, the width of the slot should be equal to D_{70}(filter).

In the event that the size of opening supplied is incompatible with the filtration, permeability, or pipe-opening criteria, provision should be made for the regular cleaning and rodding of foundation drains to remove any sediment and fine-grained soils that may accumulate in them. Alternatively, the pipes can be wrapped with geotextiles.

14.2.4 GEOTEXTILES

Geotextiles, or filter fabrics, may be used as suitable filtering media to prevent the ingress of native soil into the drainage system, provided that the following performance criteria are used to select the fabric; i.e., it should:

- permit the free passage of water;
- retain the fine particles of the soil to be protected;

- minimize ingress of soil particles into drains;

- be chemically and physically stable and inert to the groundwater; and

- be sufficiently strong and durable to perform over the anticipated life of the structure.

The location of the geotextile within the filtering/drainage system may vary. Consequently, in some cases, the fabric may be in direct contact with the native soil to separate it from the filtering soil, whereas in other cases, the fabric may be installed between the filtering soil and the drainage pipe. The specifications for the fabric will differ for each case.

14.3 PIPES AND TRAPS

For drain pipes to function adequately, it is imperative that they be installed with sufficient slope (or grade) to induce the velocity required to carry any fine particles (i.e., clays, silts, and fine sands) that may have passed through the filter and perforations in the pipe. In the absence of specific design calculations, it is suggested that a minimum slope of 1% should be used for the installation of the weeping-tile system. Moreover, when the trench for the filter and pipe is dug, the invert of the trench should be shaped and sloped parallel to that of the pipe in order that water in the filter below the pipe will drain away. Such a sloped system will not result in a continuous man-made perched water table in the vicinity of the foundation structure.

Particular attention should be given to the installation of weeping-tile drains at the exterior corners of buildings. Frequently the drainage tile is looped over the foundation pad, resulting in an inefficient drainage path, which only operates when the drain system is full. Because it is inevitable that some fine-grained soil will fall into the perforated drain pipes, soil will eventually collect in traps. Therefore, it is imperative that the traps and backwater valving arrangements be designed and installed in such a manner that they can be maintained by periodic inspections and cleaning.

14.4 CONSTRUCTION OF SUBSURFACE DRAINS

Few other single features of civil engineering works are more vital to long trouble-free performance than drainage. The need for high quality of workmanship in the construction of drains cannot be over-emphasized. Adequately prepared plans and specifications of an enforceable nature are a prerequisite for good quality of construction.

Figure 14.1 shows a suggested arrangement of a subsurface perimeter drainage system around shallow foundations. The figure illustrates three typical locations of drains and filter soils relative to the foundation footings and walls. Traditionally, clay or concrete drain pipes have been used, but in recent years, slotted plastic pipes are more common. To avoid silting up inside the drains, it is good practice to wrap the drains in geotextile (Subsection 14.2.4). The slope of the drains should always be greater than 1% and, preferably, greater than 2%. An additional drop of about 10 mm should be provided near each bend (90°) of the drain pipe. The drain pipe must not be placed underneath the footings.

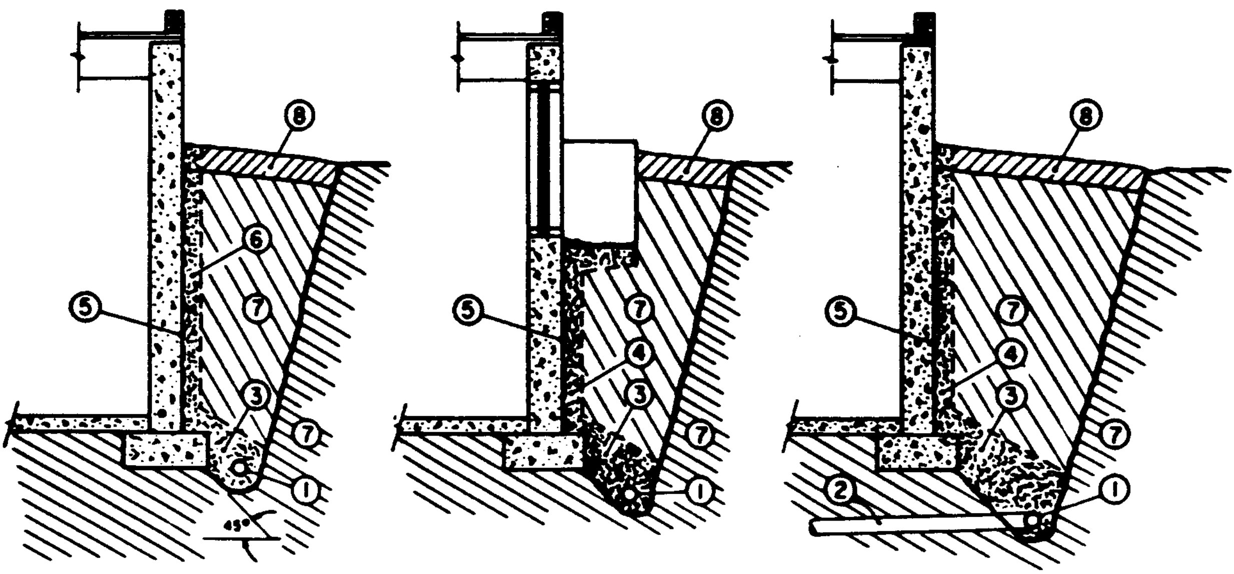

Fig. 14.1. Typical sections showing arrangement of subsurface perimeter drains around shallow foundations.

(1) Perforated or slotted pipe placed about 300 mm below the upper level of the basement floor slab; (2) unperforated drain pipe connected to appropriate trap and backwater valve before connecting to a sewer. The trap shall have provisions for inspection and cleaning; (3) filter material that is compatible with the grain size characteristics of the fine-grained foundation and backfill soils, as well as with the perforations of the pipe; (4) filter material continously or intermittently placed next to the foundation wall to intercept water from window wells and from low areas near the building (see also 6); (5) damp-proofing on wall — optional depending on the quality of the concrete wall; (6) optional use

of sheet drain, or synthetic filter blanket, next to the foundation wall to replace the soil filter according to (4); (7) foundation and backfill soils, which may contain fine-grained and erodable materials; and (8) 'topping-off' material sloping outward to lead off the surface water. It is usually desirable to use low permeability soil to reduce the risk of overloading the pipe.

APPENDIX D

Unified Classification System for Soils

TECHNICAL NOTE

Amster K. Howard[1]

VI (I)

The Revised ASTM Standard on the Unified Classification System

REFERENCE: Howard, A. K., "**The Revised ASTM Standard on the Unified Soil Classification System,**" *Geotechnical Testing Journal*, GTJODJ, Vol. 7, No. 4, Dec. 1984, pp. 216–222.

ABSTRACT: ASTM Test Method for Classification of Soils for Engineering Purposes (D 2487) was significantly revised in 1983. The revisions require that soil is to be classified by using both a symbol and a name, and the group names were standardized. Organic silts and clays were redefined to recognize that organic soils occur that plot above the "A" line on the plasticity chart. More precise guidelines were established, particularly with regard to plasticity, so that only one particular classification will result. If boarderline classifications are used, the classification symbols are separated with a slash with the classification symbol indicated using the standard appearing first. Appendixes give example written descriptions, preparation of soil for testing, and guidelines for using the system for materials such as shale, mudstone, crushed rock, and slag

KEYWORDS: soil classifications, soils, sands, clays, silts

Introduction

Classification is the mirror in which the present condition of science is reflected; a series of classifications reflect the phases of its development. Aristocristane, 98 A.D.

ASTM Test Method for Classification of Soils for Engineering Purposes (D 2487) was significantly revised in 1983. The modifications were the result of several years of discussion by ASTM Subcommittee D18.07 on Identification and Classification of Soils and a special meeting of Federal agencies using the Unified Soil Classification System (USCS) held in Denver, CO, in 1980.

The USCS has become the most popular and widely used soil classification system for engineering purposes. The Federal Aviation Administration (FAA) recently adopted the use of the USCS in place of the system they had developed earlier. Personnel using the American Association of State Highway and Transportation Officials (AASHTO) soil classification system for highway construction are seriously looking at using the USCS. The USCS began as the Airfield Classification System developed by Arthur Casagrande during World War II. With the adoption of the system by the U.S. Bureau of Reclamation and the Corps of Engineers in 1952, with standardized terms and procedures, it became known as the "Unified" System.

[1]Supervisory civil engineer, U.S. Bureau of Reclamation, Code 1542, P.O. Box 25007, Denver, CO 80225. Member of ASTM.

In the ensuing years, it became apparent that certain approaches in the system needed to be better defined and standardized. Where insufficiencies or gaps existed, various organizations and agencies found it necessary to develop their own standards or practices. In an attempt to bring uniformity to this important means of communicating engineering information, ASTM Subcommittee D18.07 sought to refine and standardize the ASTM version of the system.

The significant changes and revisions adopted include the following:

1. Soil classification consists of both a name and a symbol.
2. The names were standardized.
3. Organic silts and clays were redefined.
4. More precise classification was established.

In addition, information presented in appendixes gives example written descriptions to encourage uniformity, detail methods of preparation and testing, and shows how the system can be used to assist in describing materials such as shale, siltstone, crushed rock, and so forth.

ASTM Recommended Practice for Description of Soils (Visual-Manual Procedure) (D 2488-69) is currently undergoing similar revisions.

Classification—Name and Symbol

The classification of a soil should consist of both a name and a symbol. Often only a symbol is used, and this can be misleading. For example, the symbol CL is used for the following three soils:

(1) 100% fines,
(2) 55% fines, 45% fine-to-medium sand, and
(3) 55% fines, 25% fine and coarse gravel, 20% fine to coarse sand.

These are three different materials based on their gradation and on their engineering properties. The new ASTM D 2487-83 would classify the soils as follows:

(1) CL—lean clay,
(2) CL—sandy lean clay, and
(3) CL—gravelly lean clay with sand.

It is obvious that the name and symbol together give a better indication of what the soil is like.

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MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve*	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than 50% of coarse fraction passes No. 4 sieve	CLEAN SANDS	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS 50% or more passes No. 200 sieve*	SILTS AND CLAYS Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

* Based on the material passing the 3-in. (75-mm) sieve

FIG. 1—*Soil classification chart (ASTM D 2487-69).*

Standardization of Group Names

On Figure 1 of ASTM D 2487-69 (see Fig. 1, this paper), one column of the soil classification chart shows "group symbols" and the adjacent column "typical names." The typical names were more like descriptions of the soil, but some of the descriptions evolved in time to become a name associated with the symbol. The committee decided to formalize these names with a single unique name for each symbol (except for organic silts and clays). The names and corresponding symbols are:

GW well-graded gravel
GP poorly graded gravel
GM silty gravel
GC clayey gravel
SW well-graded sand
SP poorly graded sand
SM silty sand
SC clayey sand
CL lean clay
ML silt
OL organic silt or organic clay
CH fat clay
MH elastic silt
OH organic silt or organic clay
PT peat

Although some of the names were often unpalatable (for example, fat clay and elastic silt), it was decided to go with the venacular that had evolved, recognizing that it would be impossible to change.

In addition, modifiers to the basic group name were standardized. Most engineering organizations recognized the need to change the soil name or modify it to better reflect the characteristics of the soil. However, the names varied widely between users. For example, soil with 20% sand, 15% gravel, and 65% fines has been variously described as:

lean clay,
sandy clay,
sandy gravelly clay,
sandy lean clay,
sandy gravelly lean clay,
lean sandy clay,
lean clay with sand and gravel, or
clay with well-graded sand and gravel.

Since only the symbol, CL, does not convey enough information, a group name should be associated with the symbol and that group name should be standardized. According to the revised standard, every user would describe this soil as

sandy lean clay with gravel, CL

Thus, the name and symbol alone relate the facts that the fines are clayey with a liquid limit less than 50; there is between 30 and 49% coarse-grained particles, predominantly sand, with at least 15% gravel.

The standard group name is listed in Table 1 of the new standard (see Fig. 2, this paper) for each group symbol and information given as to what to add to the group by a "with" statement. The flow charts, Figs. 1 and 2, also illustrate the use of the group name and "withs" (see Figs. 3 through 5, this paper).

Organic Silts and Clays Redefined

In ASTM D 2487-69, organic silts (OL) and organic clays (OH) could only occur below the "A" line. A liquid limit of 50 was the dividing line between OL and OH (see Fig. 1 ASTM D 2487-69).

The standard was changed so that OL and OH soils can be both below and above the "A" line. A liquid limit of 50 remains as the division between the symbols OL and OH (see Fig. 6, this paper). However, the group name will depend on whether the soil plots above or below the "A" line. The group names "organic clay" will apply to soils on or above the "A" line and "organic silt" will apply to soils below the "A" line. The possible classifications then are

organic clay, OL,
organic silt, OL,
organic clay, OH, or
organic silt, OH.

The criterion for determining whether or not a soil is organic remains as the comparison of the liquid limit values of an oven-dried specimen and a nondried specimen. The change was made for the following reasons:

1. Organic soils occur that plot above the "A" line. The following comments are by A. Casagrande [1].

> Originally the A-line was defined by the writer as an empirical boundary between typical inorganic clays and plastic organic soils. He was then not aware of the existence of fairly tough organic clays which fall above the A-line. (They have more the characteristics of inorganic clays except for the substantial loss in plasticity due to drying.) It was suggested to move the A-line so as to assure that all organic soils would fall below it. However, this would also bring most inorganic soils below the A-line. The writer believes that the A-line has proven its value as an important reference line and that it should be kept essentially in its original position, but that in the expanded system a new group should be provided for the organic soils located above the A-line.

The following are comments by R. A. Barron [2]:

> After a year's use, comments were sent in from the various field offices to the Office of the Chief of Engineers. There were a few comments on the system which indicated some minor revisions may be necessary. One, for instance, is the fact that some organic soils plot above the "A" line of the plasticity charts.

In addition, Richard S. Ladd, of Woodward-Clyde Consultants of Clifton, NJ, reported in subcommittee meetings of D18.07 that his laboratory has encountered organic soils that plot above the "A" line.

2. For inorganic soils, the "A" line is the division between clays and silts. This division is now logically extended to organic soils.

3. The name "organic clay" according to ASTM D 2487-69 could have been applied to a soil with a liquid limit (LL) > 50 and a plasticity index (PI) < 10. For a soil with such low plasticity, the name organic "clay" is inappropriate.

More Precise Classification

ASTM D 2487-69 recommended giving a soil a borderline classification if the LL and PI values plotted "on or practically on" the "A" line or the LL = 50 line.

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [a]				Soil Classification: Group Symbol	Soil Classification: Group Name [b]
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS Less than 5% fines [c]	$C_u \geq 4$ and $1 \leq C_c \leq 3$ [e]	GW	Well-graded gravel [f]
			$C_u < 4$ and/or $1 > C_c > 3$ [e]	GP	Poorly graded gravel [f]
		GRAVELS WITH FINES More than 12% fines [c]	Fines classify as ML or MH	GM	Silty gravel [f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel [f,g,h]
	SANDS 50% or more of coarse fraction passes No. 4 sieve	CLEAN SANDS Less than 5% fines [d]	$C_u \geq 6$ and $1 \leq C_c \leq 3$ [e]	SW	Well-graded sand [i]
			$C_u < 6$ and/or $1 > C_c > 3$ [e]	SP	Poorly graded sand [i]
		SANDS WITH FINES More than 12% fines [d]	Fines classify as ML or MH	SM	Silty sand [g,h,i]
			Fines classify as CL or CH	SC	Clayey sand [g,h,i]
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50%	inorganic	PI > 7 and plots on or above "A" line [j]	CL	Lean clay [k,l,m]
			PI < 4 or plots below "A" line [j]	ML	Silt [k,l,m]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay [k,l,m,n] Organic silt [k,l,m,o]
	SILTS AND CLAYS Liquid limit 50% or more	inorganic	PI plots on or above "A" line	CH	Fat clay [k,l,m]
			PI plots below "A" line	MH	Elastic silt [k,l,m]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay [k,l,m,p] Organic silt [k,l,m,q]
Highly organic soils		Primarily organic matter, dark in color, and organic odor		PT	Peat

GRAVEL = No. 4 sieve.

a. Based on the material passing the 3-in (75-mm) sieve.
b. If field sample contained cobbles and/or boulders, add "with cobbles and/or boulders" to group name.
c. Gravels with 5 to 12% fines require dual symbols
   GW-GM well graded gravel with silt
   GW-GC well graded gravel with clay
   GP-GM poorly graded gravel with silt
   GP-GC poorly graded gravel with clay
d. Sands with 5 to 12% fines require dual symbols
   SW-SM well graded sand with silt
   SW-SC well graded sand with clay
   SP-SM poorly graded sand with silt
   SP-SC poorly graded sand with clay
e. $C_u = D_{60}/D_{10}$ $\quad C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
f. If soil contains ≥ 15% sand, add "with sand" to group name.
g. If fines classify as CL-ML, use dual symbol GC-GM, SC-SM.
h. If fines are organic, add "with organic fines" to group name.
i. If soil contains ≥ 15% gravel, add "with gravel" to group name.
j. If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
k. If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel" whichever is predominant.
l. If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.
m. If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
n. PI ≥ 4 and plots on or above "A" line.
o. PI < 4 or plots below "A" line.
p. PI plots on or above "A" line.
q. PI plots below "A" line.

FIG. 2—*Soil classification chart (ASTM D 2487-83).*

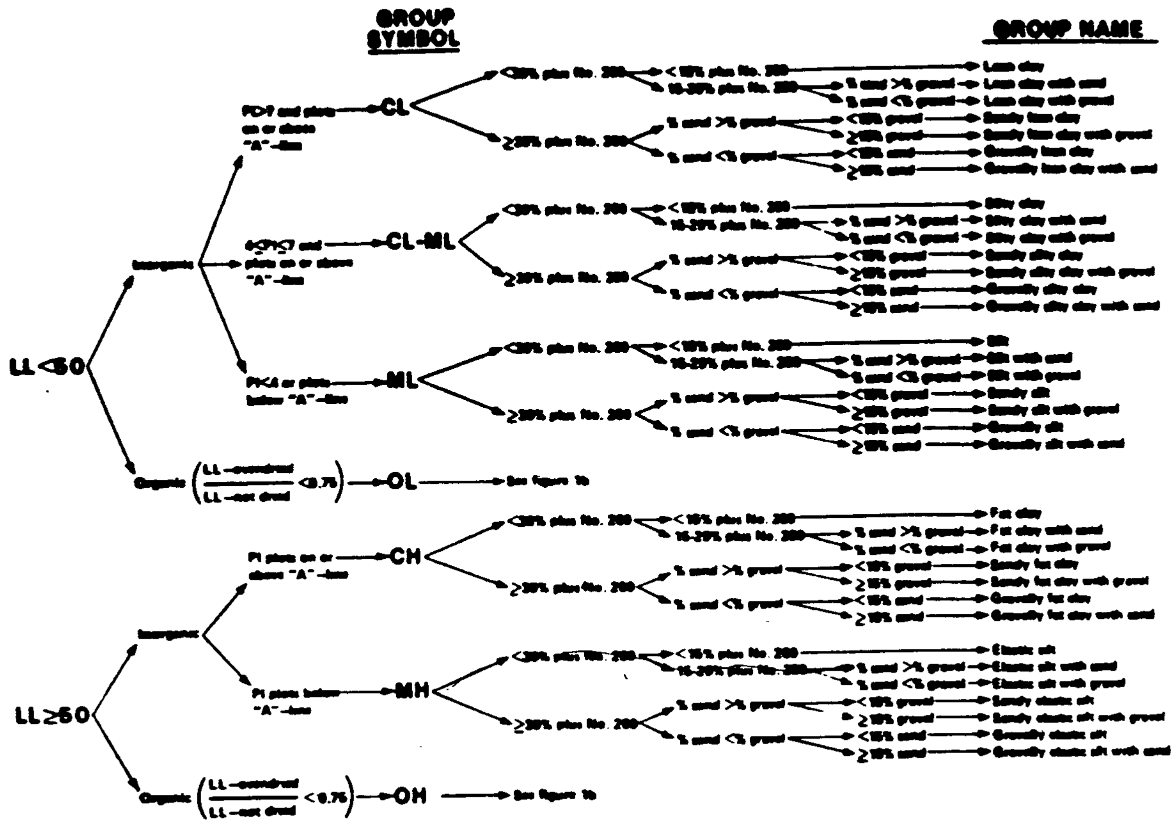


FIG. 3—*Flow chart for classifying fine-grained soil.*

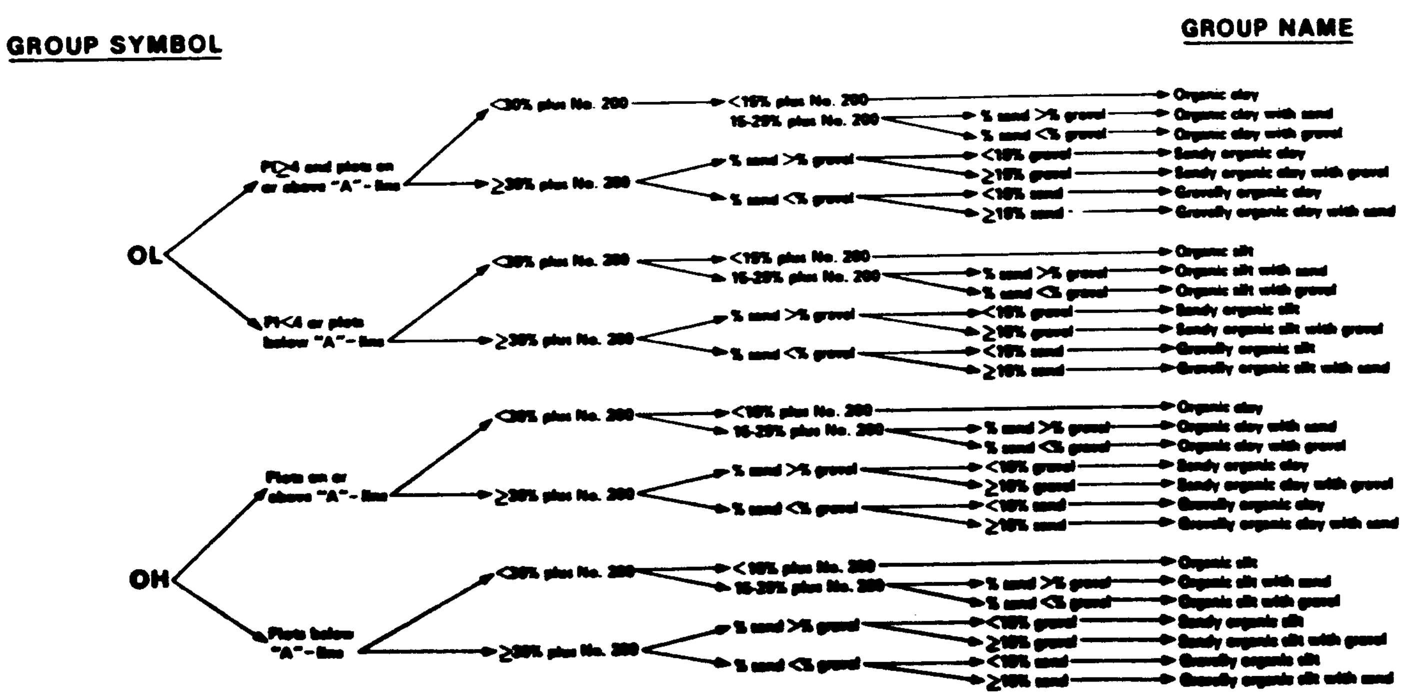


FIG. 4—*Flow chart for classifying organic soil.*

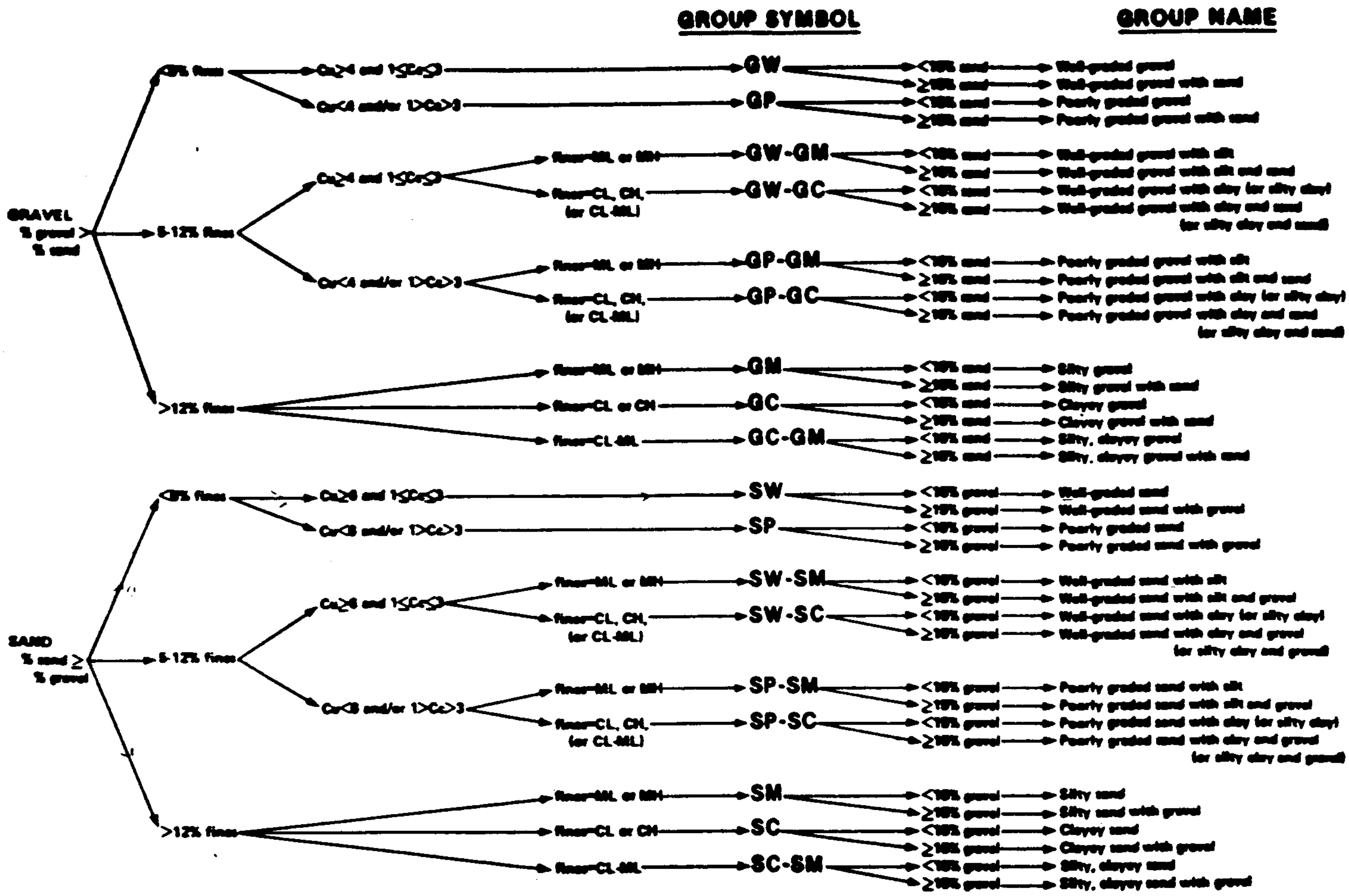


FIG. 5—*Flow chart for classifying coarse-grained soil.*

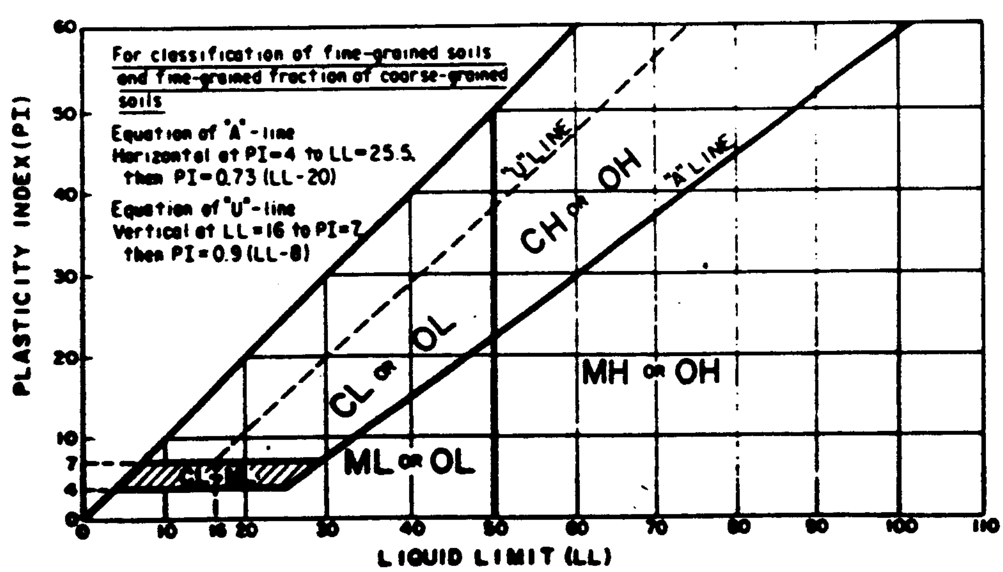


FIG. 6—*Plasticity chart.*

The standard was changed so that these borderline classifications are eliminated. Fines to be described as clay have an LL and PI value that plot on or above the "A" line while fines to be described as silt would plot below the "A" line. The symbols CH, MH, and OL refer to soils with a liquid limit of 50 or greater; and CL, ML, and OL refer to soils with a liquid limit less than 50.

The change was made for the following reasons:

(1) to eliminate the confusion and profusion of using borderline classifications,

(2) so people using the same laboratory test results would classify the soil exactly the same, and

(3) so inexperienced personnel and computer programs would have a set of prescribed rules to follow.

Dual Versus Borderline Symbols

The USCS requires some soils to have dual symbols. Soils with 5 to 12% fines must have a dual symbol composed of a clean, coarse-grained symbol followed by a coarse-grained soil with fines symbol (for example, SP-SM and GW-GC). Soils with LL and PI values that plot in the cross-hatched area of the plasticity chart must have a dual symbol of CL-ML, SC-SM, or GC-GM. These classifications are a required part of the system as presented in ASTM D 2487-83.

However, it is often desired to indicate that a soil is close to the boundary or borderline between two different soil classifications. When the laboratory tests indicate that a soil is close to a borderline (either plasticity or gradation values), it can be given a borderline symbol of two symbols separated by a slash. The first symbol is the one based on ASTM D 2487 (for example, CL/CH, CL/ML, ML/CL, and GP/SP).

Emphasis Placed on More Plastic Classification

The new standard emphasizes or favors the more plastic classification or the finer-grained classification.

1. ASTM D 2487-69 defined fine-grained soils as "50% or more passes the No. 200 sieve" and coarse-grained soils as "more than 50% retained on No. 200 sieve" while sands were soils with "more than 50% of coarse fraction passes No. 4 sieve" and gravels as "50% or more of coarse fraction retained on No. 4 sieve." In the former case, the fine-grained material was favored while in the latter case the coarse-grained material was favored. The new standard changes the latter case to describe sands as "50% or more of coarse fraction passes the No. 4 sieve" and gravels as "more than 50% of coarse fraction retained on No. 4 sieve."

2. ASTM D2487-69 favored the less plastic classification in one note (Note 5), while another note (Note 6) stated the more plastic classification was to be favored.

The new standard favors the more plastic classification in the following ways:

1. New Note 7 (old Note 5) was changed to favor the more plastic classification.

2. When the LL and PI for a soil fall on the "A" line, the soil is classified as a clay, not a silt.

3. When the LL = 50, the soil is to be classified as a CH, not CL, and MH, not ML, emphasizing the more compressible material.

4. A soil with LL and PI plotting in the hatched area of the plasticity chart is to be classified as a CL-ML, silty clay.

Use of the System as a Secondary Classification System

The USCS is often used for classifying and describing materials such as shale, siltstone, claystone, mudstone, sandstone, crushed rock, slag, cinders, shells, and so forth.

Lithified or partially lithified material (shale, claystone, and so forth), is sometimes classified as a soil after the material has been processed (grinding, slaking, and so forth). The material should be "classified" according to its original state. A secondary classification according to USCS can be reported. However, as presented in Appendix X2 in ASTM D 2487-83, it is suggested that the group name and symbol be in quotation marks to distinguish them from the classification of true soils.

Material, such as shells and slag, should not be considered as soil, but the USCS can be used to describe the material. Again, the primary classification should be shells or slag with a secondary USCS classification in quotation marks.

Crushed rock is not a naturally occurring soil and any classification should also be in quotation marks.

Examples of written descriptions were included in Appendix X2, some of which are shown below:

1. *Shale Chunks*—retrieved as 50- to 101-mm (2- to 4-in.) pieces of shale from power auger, dry, brown, no reaction with HCl. After laboratory processing by slaking in water for 24 h material classified as "Sandy Lean Clay (CL)," 61% clayey fines, LL = 37, PI = 16; 33% fine to medium sand; 6% gravel-size pieces of shale.

2. *Crushed Rock*—processed gravel and cobbles from Pit 7; "Poorly Graded Gravel (GP)," 89% fine, hard, angular gravel-size particles; 11% coarse, hard, angular sand-size particles; dry, tan; no reaction with HCl; Cc = 2.4, Cu = 0.9.

"U" Line

The upper limit or "U" line was added to the plasticity chart (Fig. 6) to aid in the evaluation of test data. This line was recommended by Casagrand as an empirical boundary for natural soils. It provides a check against erroneous data, and any test results that plot above or to the left of it should be verified.

There is no formal documentation as to the origin of the "U" line. Students in classes given by Casagrande reported that it was presented as part of his lectures, and they have the sketch in their class notes. The Corps of Engineers does include the "U" line, described as the upper limit line, in their manual *Laboratory Soils Testing*. Casagrande served as a consultant for this manual and did review it. The Corps' manual states that the "U" line begins at an LL of 8 and PI of 0 and rises on a slope of 0.9 (PI − 0.9 [LL = 8]). However, the line is not shown on their plasticity chart below a PI of 7 (the top of the cross-hatched area).

The 1983 revision of ASTM D 2487 also shows the "U" line on the plasticity chart, but below a PI of 7 the line is vertical at LL = 16. LL's below 16 are felt to be unreasonable values as the soil is probably sliding on the surface of the cup rather than a flowing or shearing of the material. A computer search revealed that of over a thousand soil specimens tested and reported by the USBR geotechnical laboratory, four had LL = 17, one had LL = 16, and none had LL below 16.

Expanded Liquid Limit Scale

Stopping the LL scale at 100 on the plasticity chart tends to reinforce the erroneous assumption that the LL of a soil cannot be greater than 100. Expanding the scale to 110 to help correct this misunderstanding was incorporated in the 1983 revision of ASTM D 2487 (Fig. 6).

Symbol for Coefficient of Curvature

The most controversial change in the revised standard was the symbol for the coefficient of curvature. In the USCS, as adopted by the Corps of Engineers and the Bureau of Reclamation, the symbol used was C_c. Unfortunately, this is also the soil mechanics symbol for the compression index; the slope of the linear portion of the pressure-void ratio curve on a semilog plot. In ASTM 2487-69, the symbol C_z was used for the coefficient of curvature in order to avoid the confusion of using the same symbol for two different terms. During the balloting process preceding the 1983 version, it became apparent that a strong and vociferous faction wanted to return to the traditional C_c as the symbol. After a ballot incorporating the C_c symbol went out, it became obvious that the advocates of not using the C_c symbol were also indeed numerous and vocal. Following hours of deliberating, cogitating, and arbitrating, the symbol Cc, with the lower case c on the same line (not a subscript) was selected as the symbol that least offended all the parties involved.

Cobbles and Boulders

Although the soil that is classified is the 75-mm (3-in.) minus material, the new standard requires that if plus 75-mm (3-in.) particles (cobbles or boulders) were present in the field sample, then the name of the soil should reflect their presence (for example, silty gravel with cobbles, GM). Suggested criteria for what is a cobble or a boulder were given.

Summary

ASTM D 2487 was significantly revised in 1983. The revisions include:

1. Requiring soil to be classified by stating both a symbol and a name.
2. Standardizing the names associated with the symbols and what modifiers or additional terms must be included in the name.
3. Redefining organic silts and clays to recognize that organic soils occur that plot above the "A" line on the plasticity chart.
4. More precise guidelines were established, particularly with regard to plasticity, to eliminate borderline classifications. Using the standard, only one particular classification will result. In the case of soils with 5 to 12% fines or plotting in the hatched area of the plasticity chart, dual symbols are used (for example, SP-SM, and CL-ML). However, if it is desired to indicate that the soil properties are close to another classification group, the two groups can be indicated using a slash, for example, CL/CH, with the classification indicated from the standard appearing first.
5. Provision was made to apply the classification system to materials such as shale, mudstone, crushed rock, slag, and so forth.

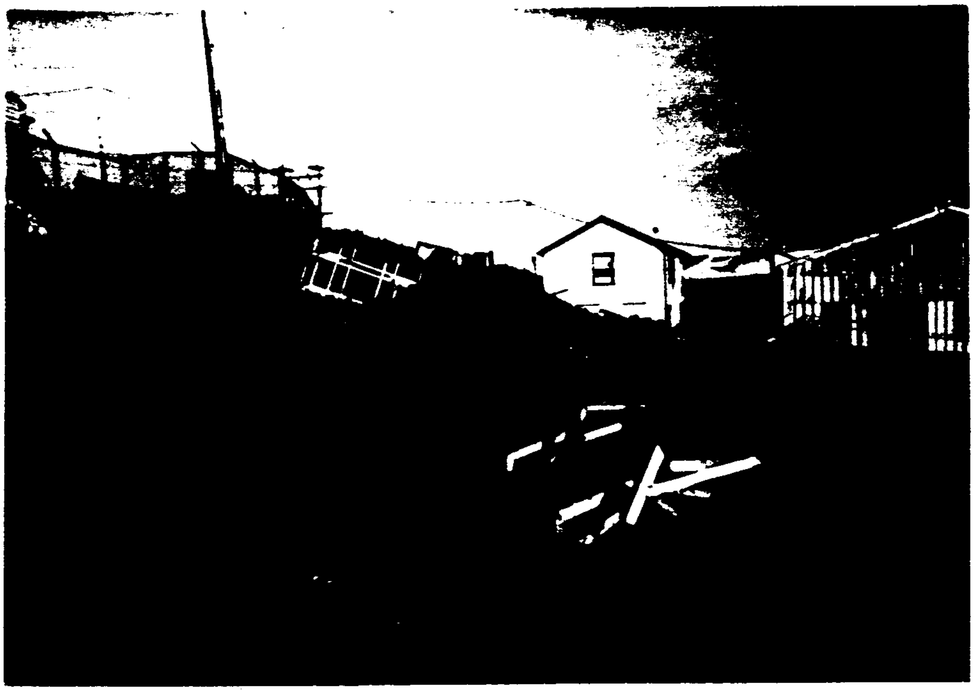
References

[1] Casagrande, A., "Proposal for a Unified Soil Classification," U.S. Bureau of Reclamation, no date.

[2] Barron, R. A., "Discussion of 'A Standard Classification of Soils as Proposed by the Bureau of Reclamation' by E. A. Abdun-Nur," *Symposium on the Identification and Classification of Soils, STP 113*, American Society for Testing and Materials, Philadelphia, 1950, p. 6.

APPENDIX E

Photographs

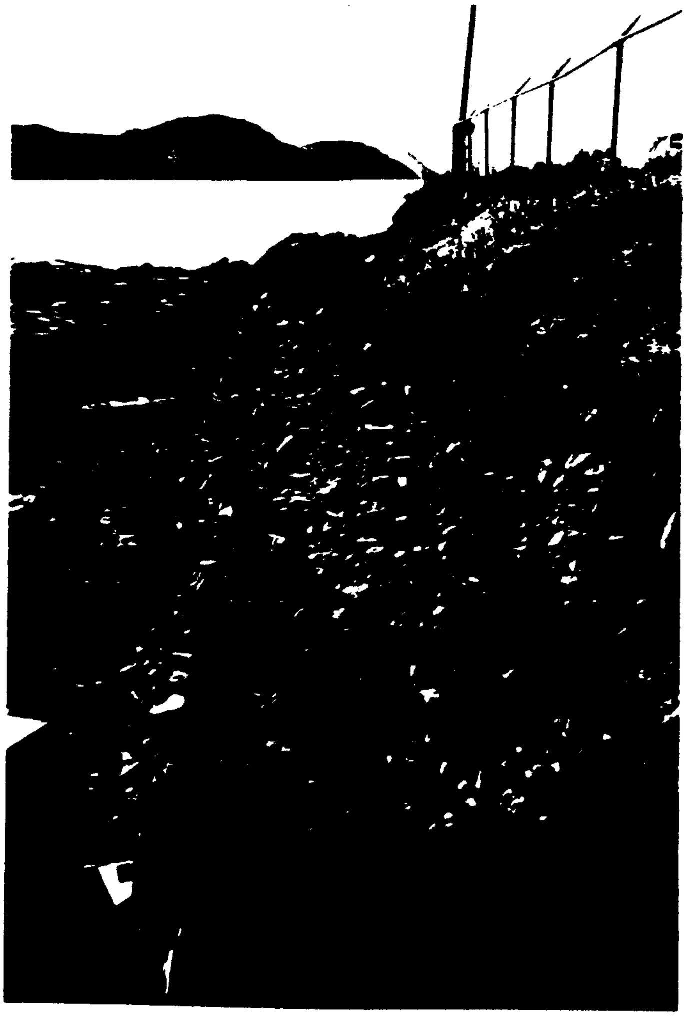


Area behind Long Harbour Bait Depot prior to excavation.

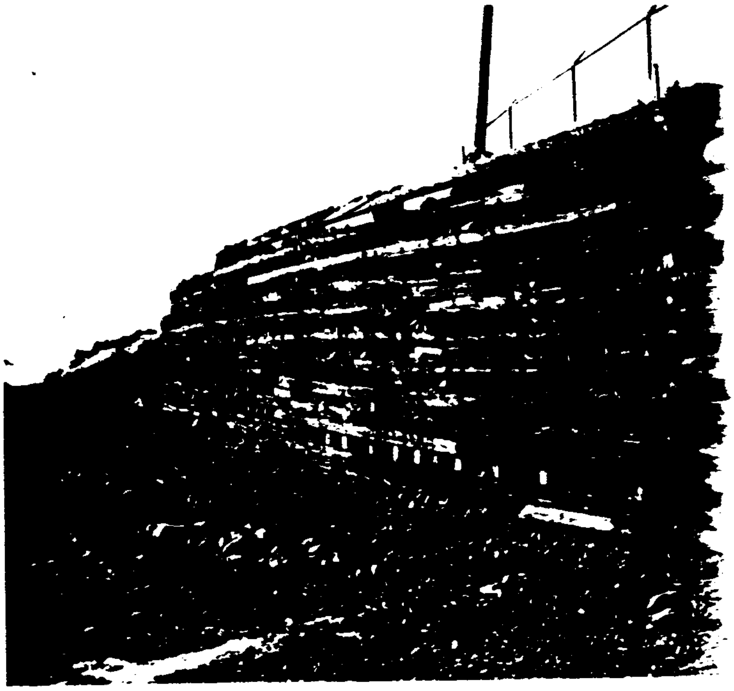
Area during excavation.

Overall view of Timber Crib Retaining Wall.

Profile view of wall showing entire length.

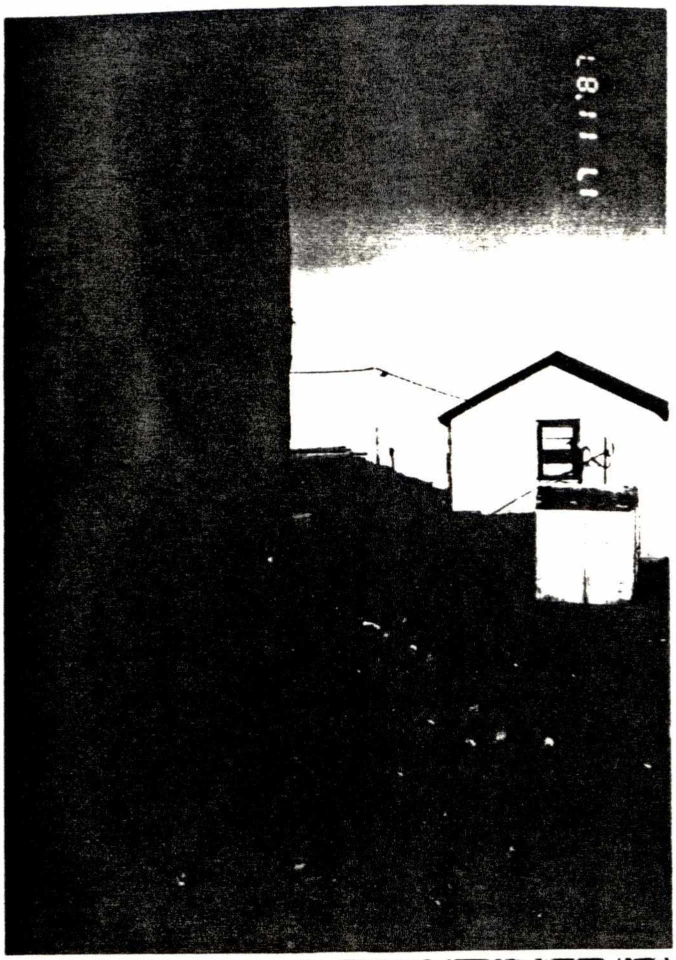


View of ballast placed in cribs.

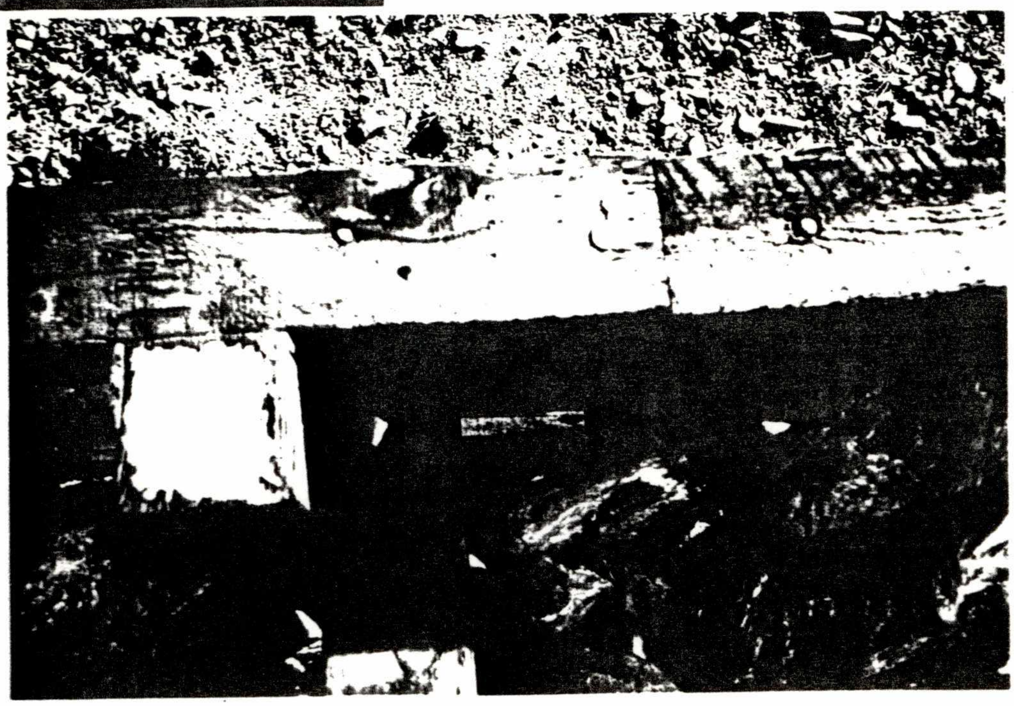


Highest section of wall.

View showing components of timber crib construction.



Elevation view of wall. Note the absence of translation or wall deformation.



Connections arrangement. Note drift spikes and machine bolted through vertical.