GABION STRUCTURES

ANALYSIS/DESIGN AND CONSTRUCTION PRACTICES

PREPARED & PRESENTED BY [illegible]

Construction Control Services Corporation
ACLU - Project #306-0205
University Town, Peshawar, Pakistan

Gabion Structures
Seminar

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Particulars

Dates of Seminar
7 December 1991
14 December 1991

Location of Seminar

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GABION STRUCTURES SEMINAR

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Construction Control Services Corporation
ACLU Project
University Town, Peshawar, Pakistan

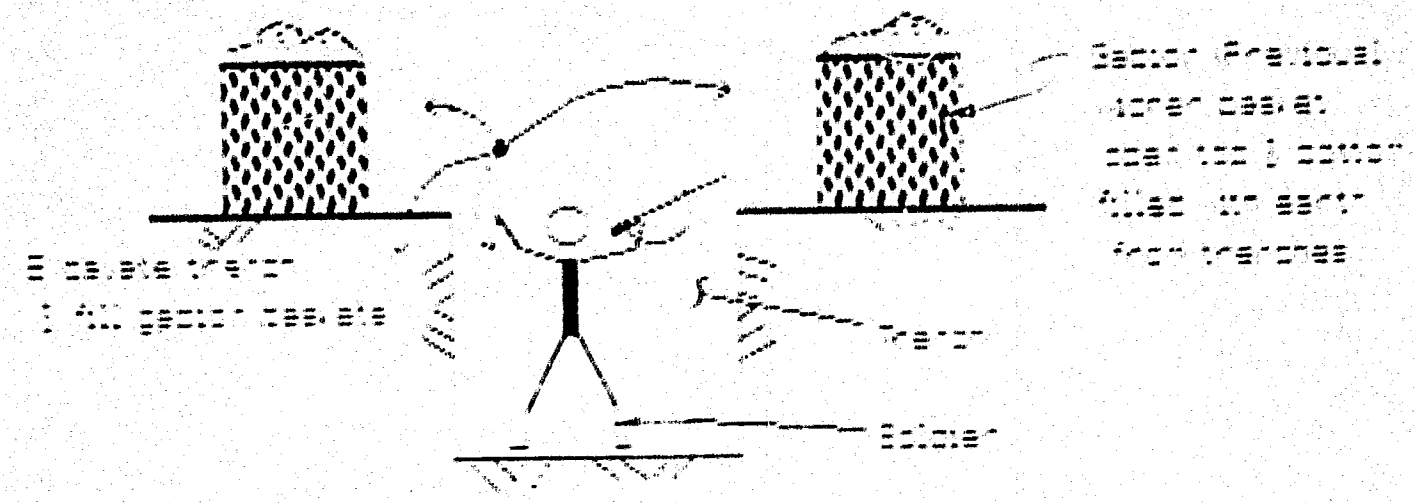
PART I

GABION CONSTRUCTION TECHNIQUES AND PRACTICES

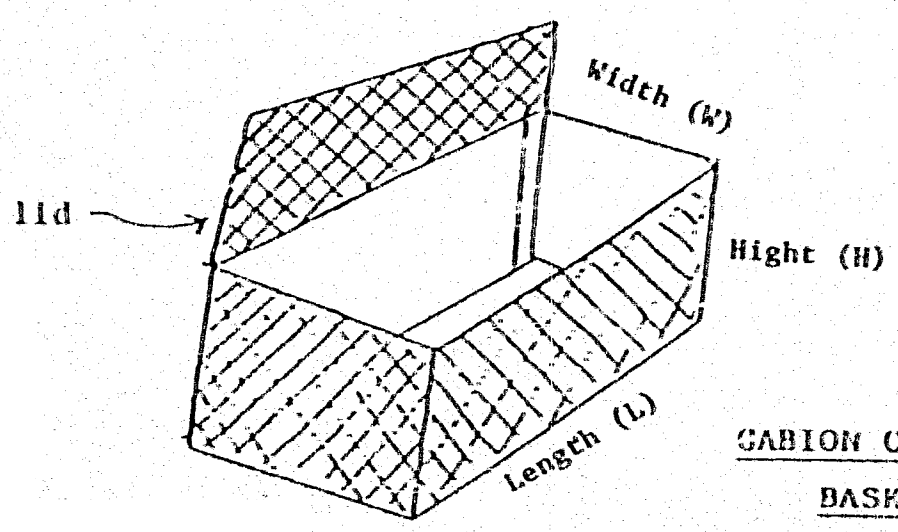
Gabion Construction Techniques And Practices

Gabions

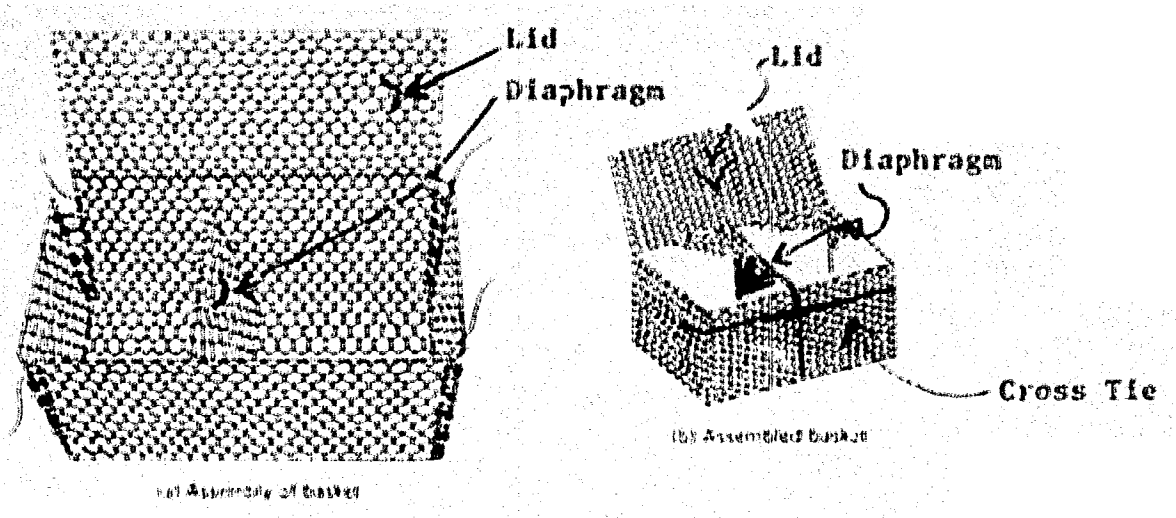
Originally gabions were bottomless wicker baskets, about .6M in diameter, placed on the edge of a trench to protect soldiers from enemy rifle fire. They stood adjacent to each other in a row and were filled with earth dug from the trenches.



Gabions have evolved over the years, today they come in flat sheets of galvanized hexagonal wire mesh, are folded & wired together on site to form compartmentalized, rectangular cages or baskets that are subsequently filled with stones. Gabions have many different uses but are primarily used to control bank erosion.

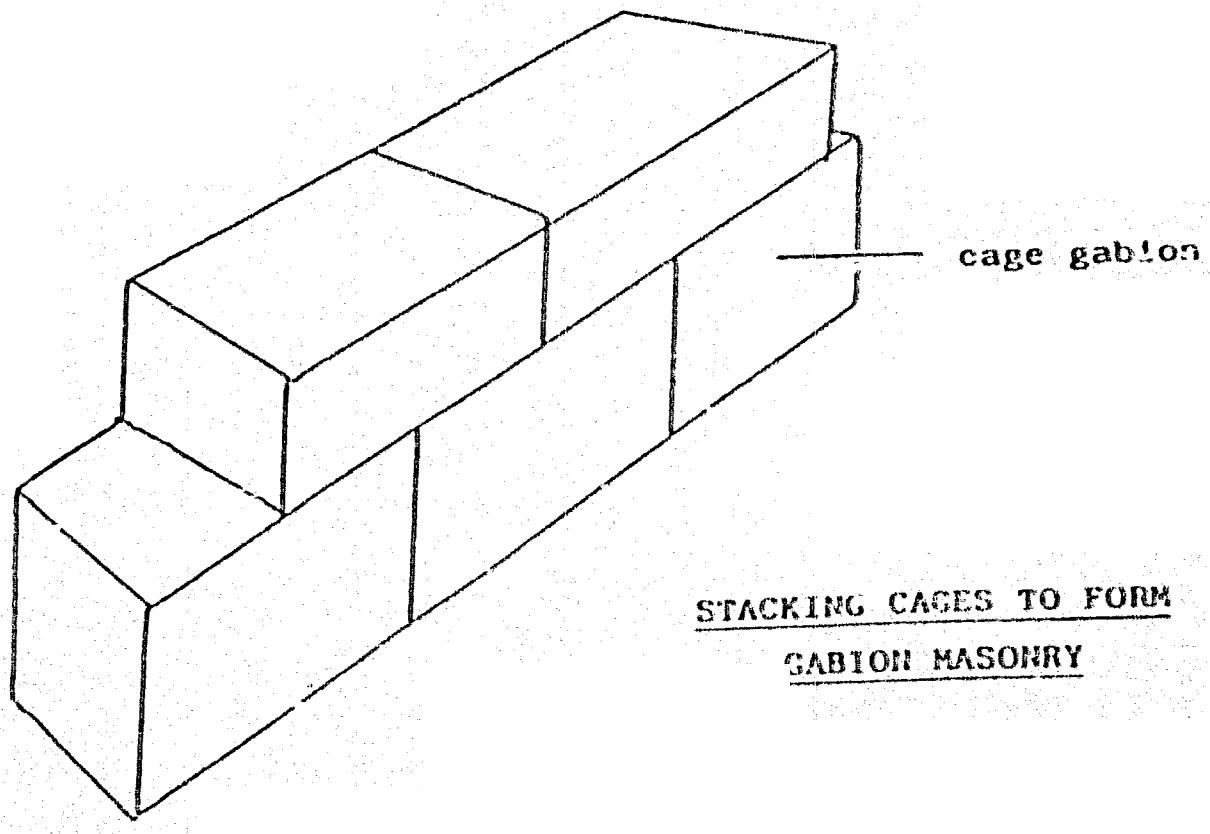


GABION CAGE OR BASKET



Once the gabion is filled, its lid is lowered and bound to the adjacent side of the box. In this way, the gabion becomes a single unit with definite and relatively stable dimensions. Once boxes are filled with even small stone, the mesh will prevent a water flow from removing the stones. The boxes are heavy enough to resist movement even by large water flows and high stream velocities, unlike concrete they do not crock. Fabrication is done completely on site.

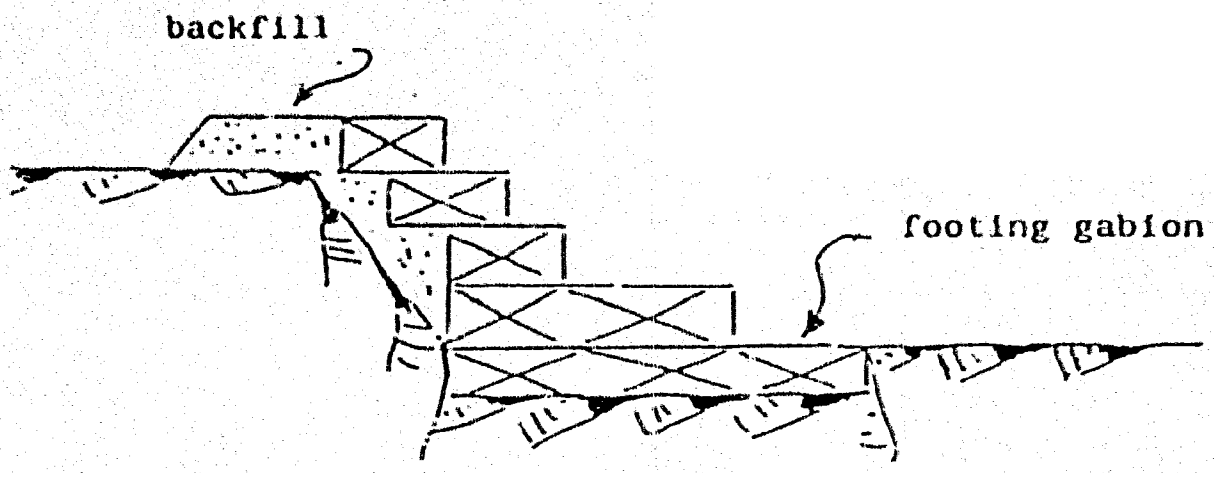
Gabion masonry can be defined as a collection of blocks arranged, following certain rules, in stacks and attached together by wire.

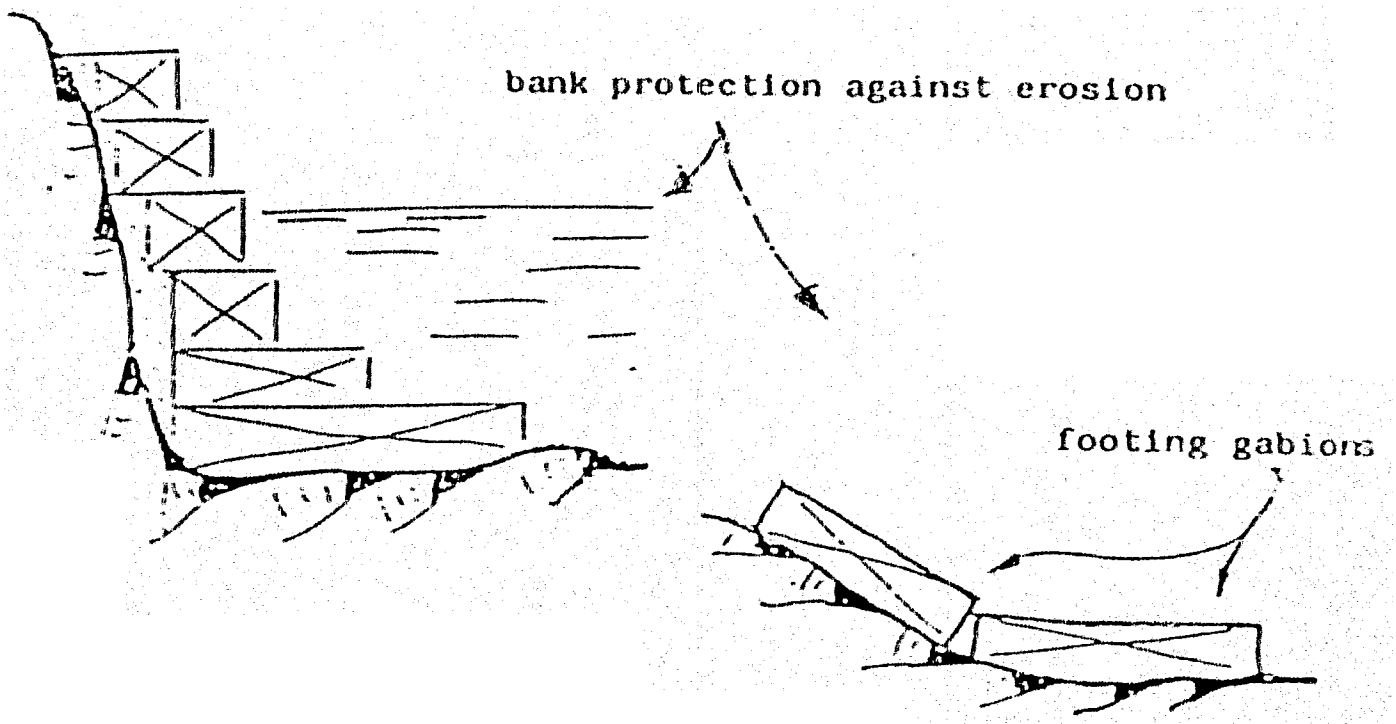


STACKING CAGES TO FORM GABION MASONRY

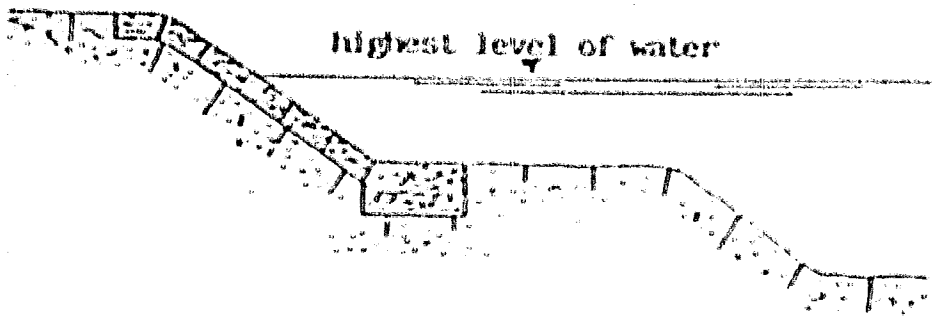
In a sense gabion masonry walls can be compared to bonded dry stone masonry walls. Gabions can be used in a number of different ways, to name a few:

<u>Bank Protection</u> against flooding

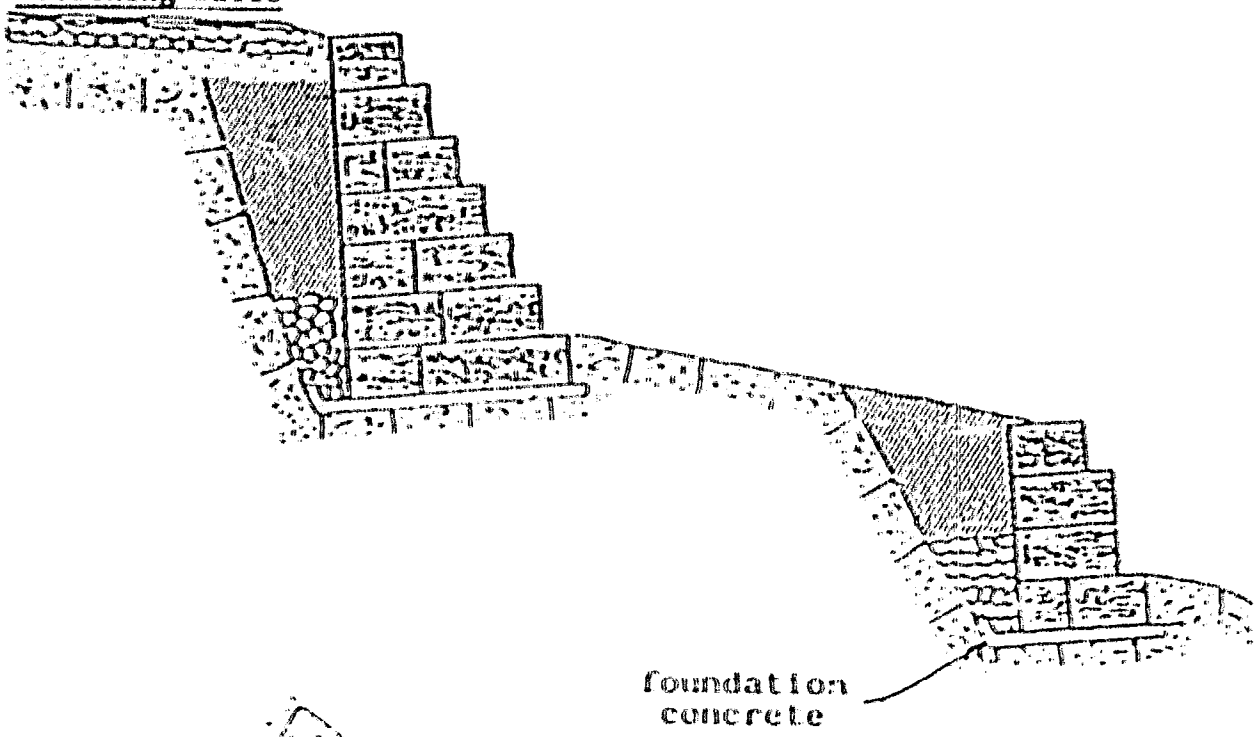


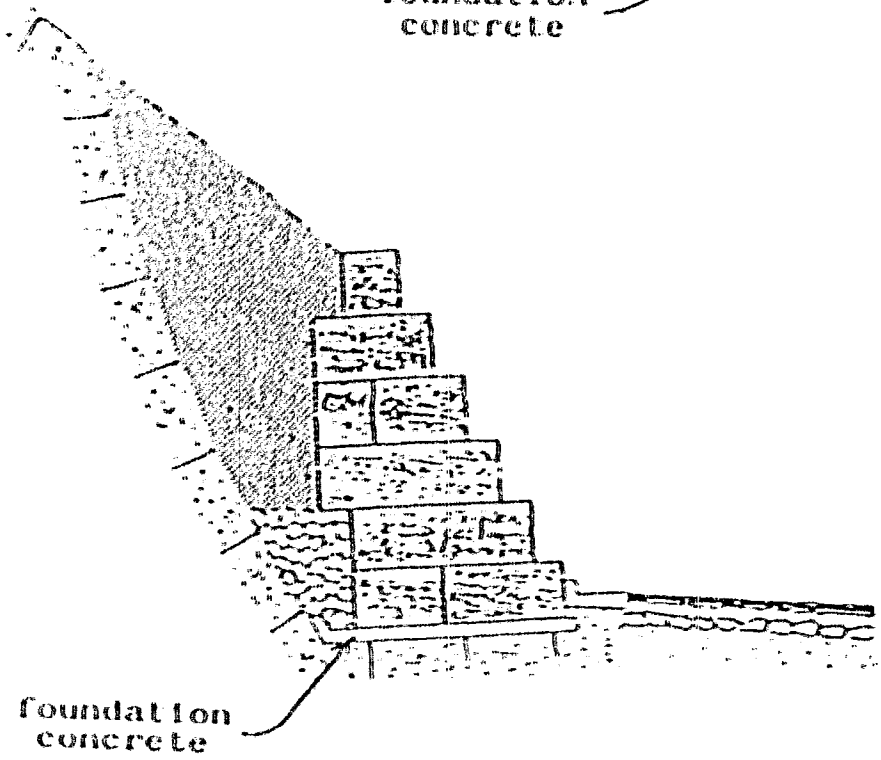


Bank Protection

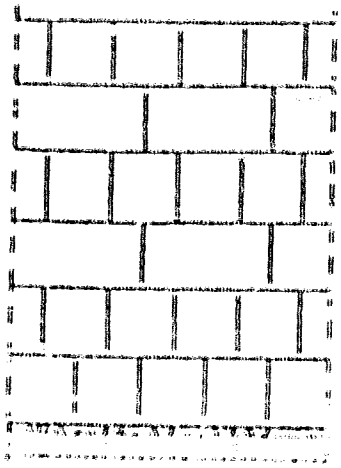


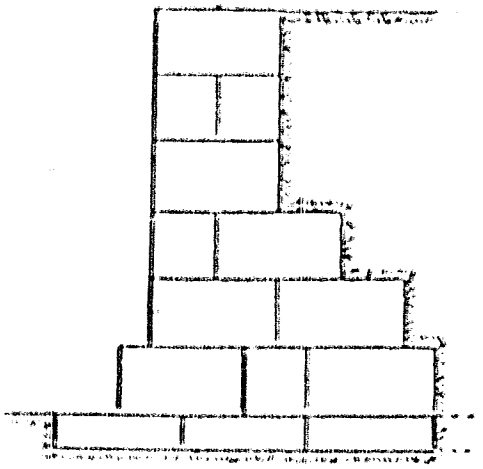
Retaining Walls

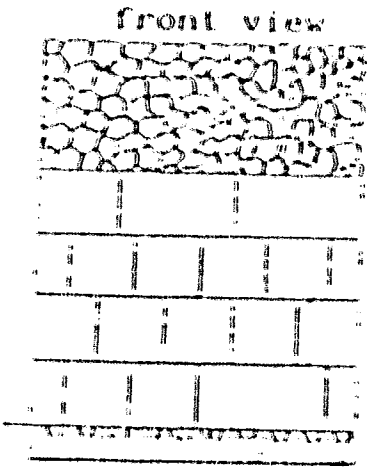


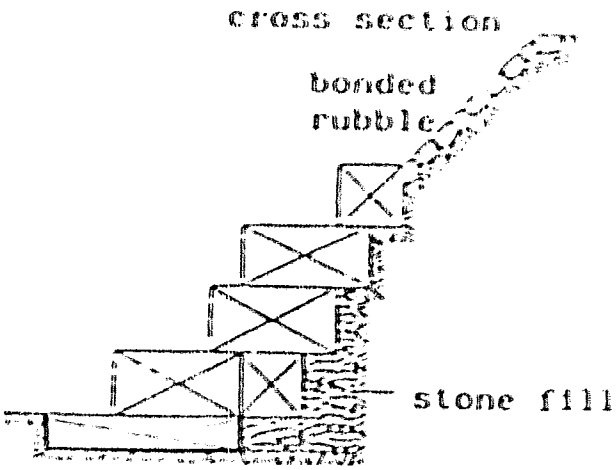


Retaining Walls





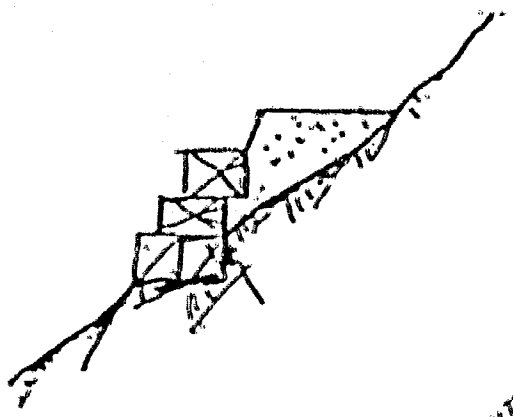




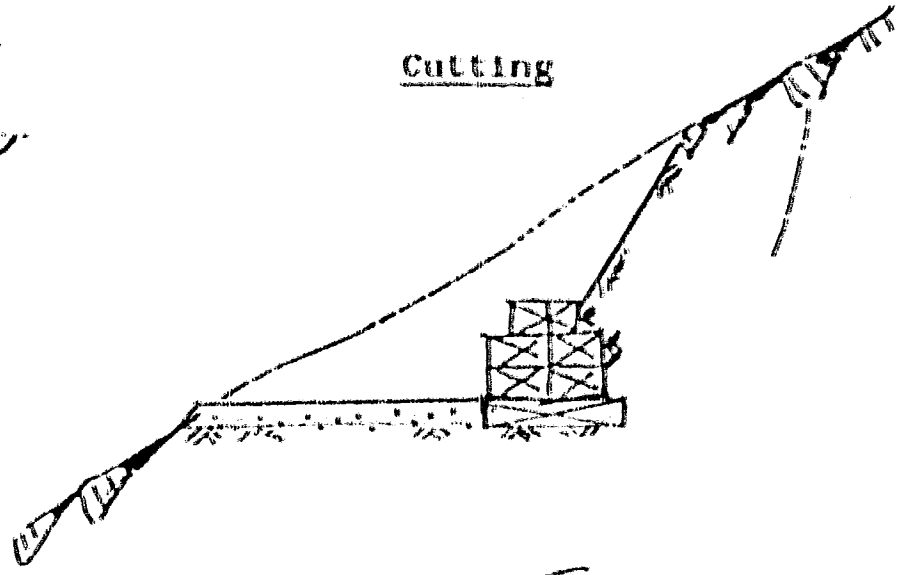
wall at foot of slope

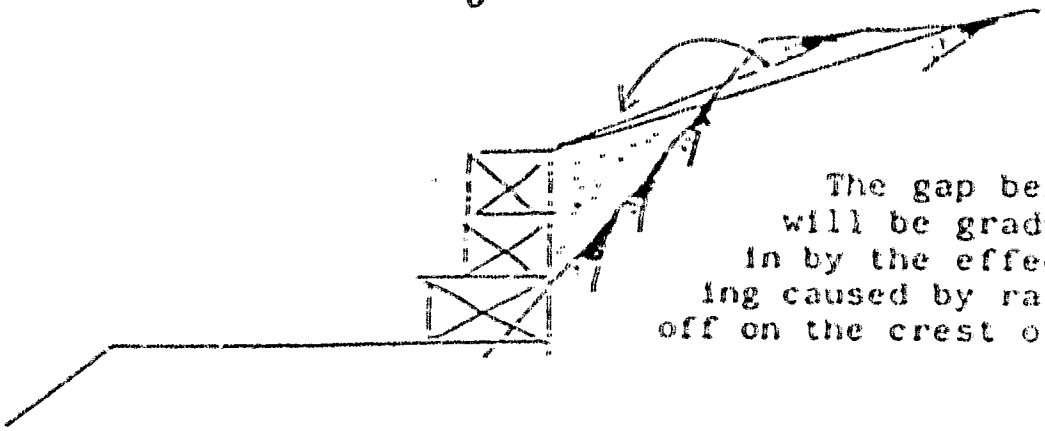
Roadway retaining walls

Embankment



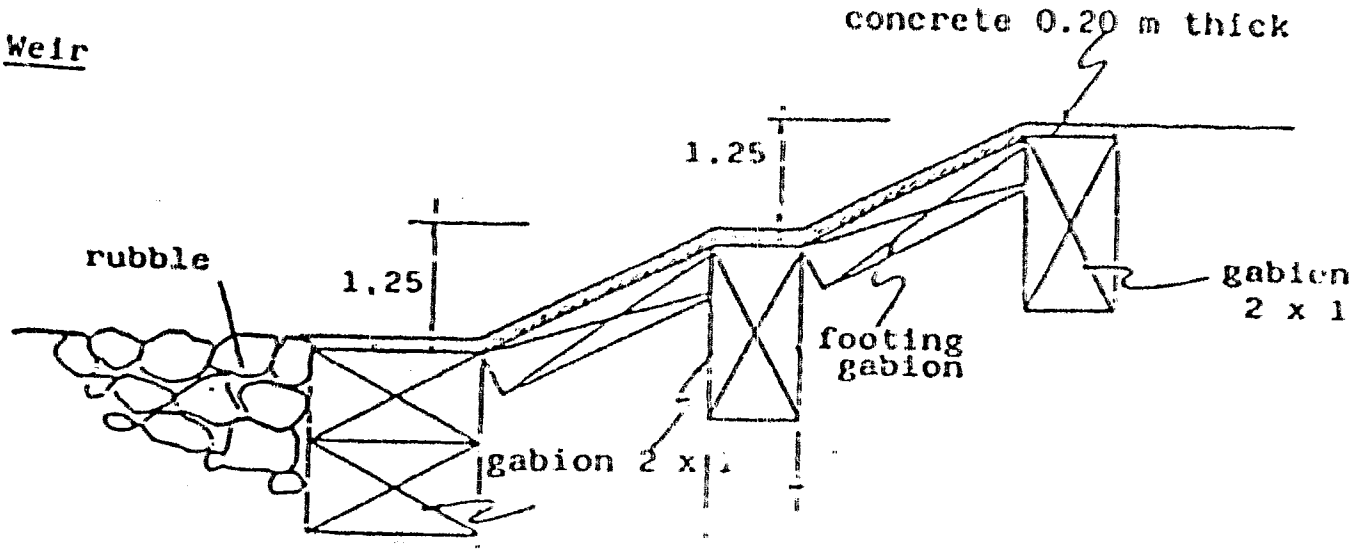
Cutting



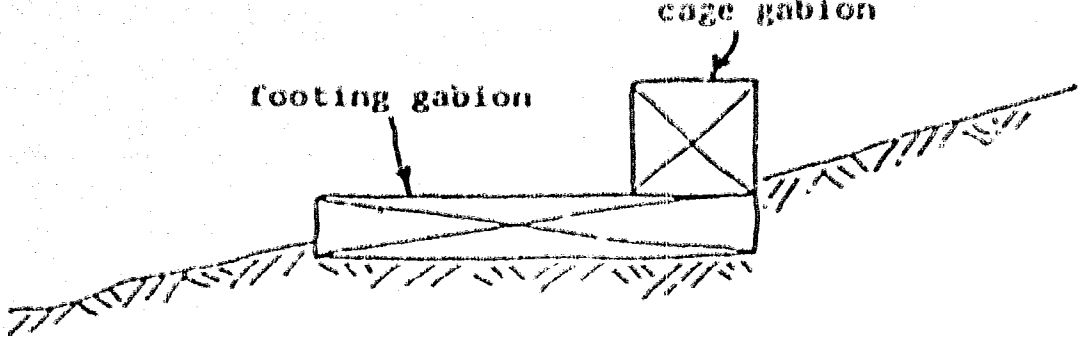


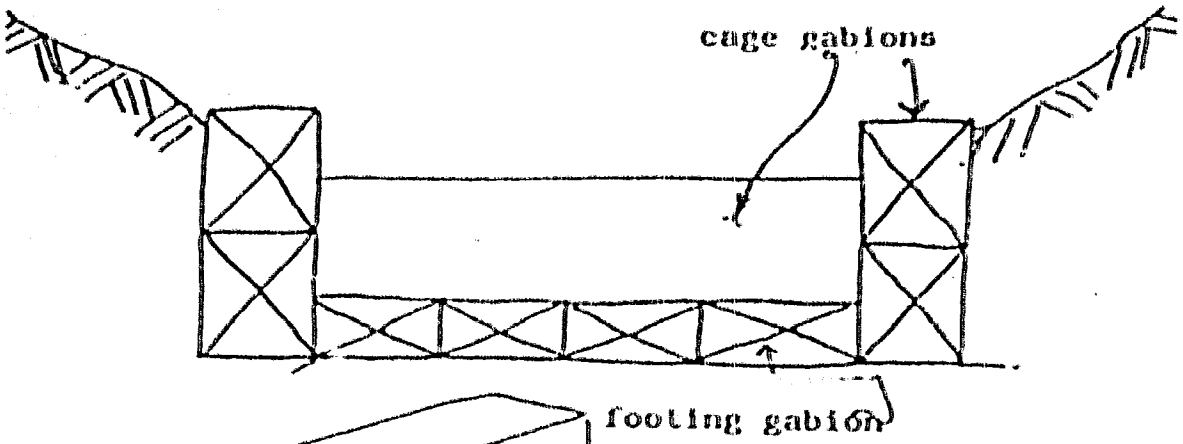
The gap behind the wall will be gradually filled in by the effects of gullying caused by rainfall run-off on the crest of the slope.

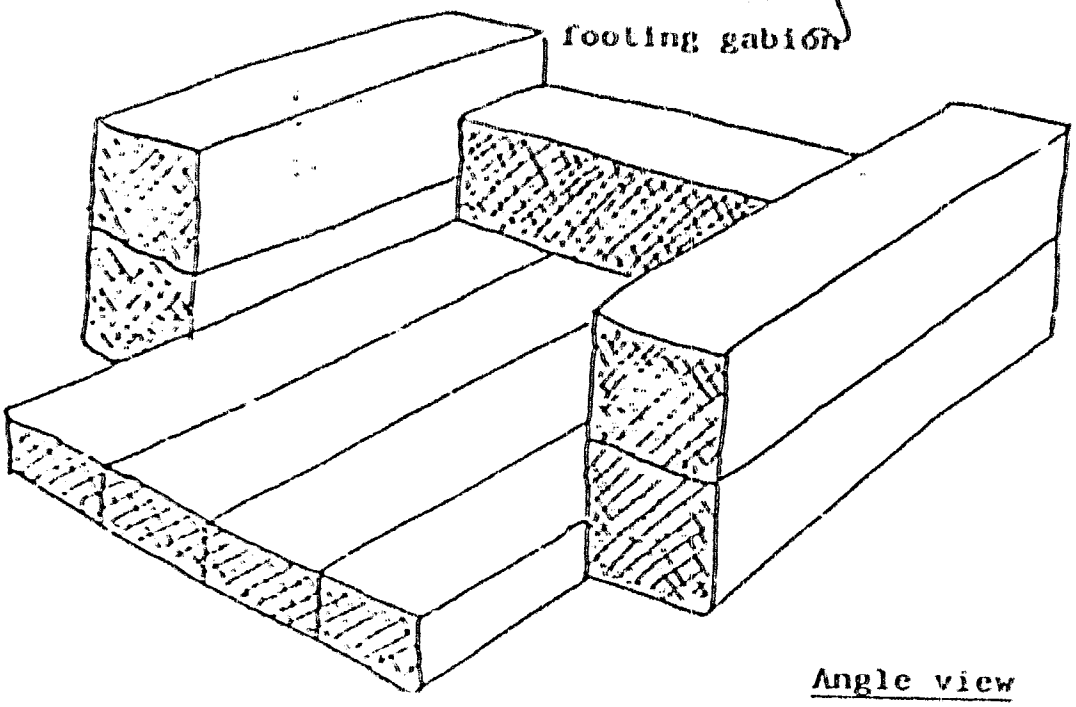
Weir



Weirs

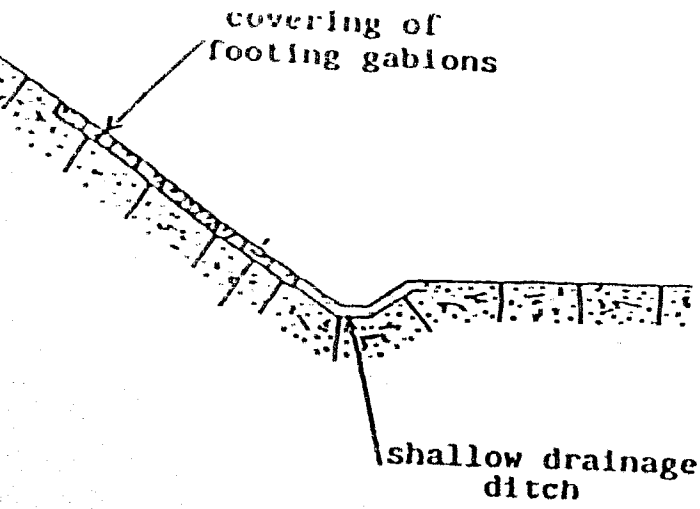


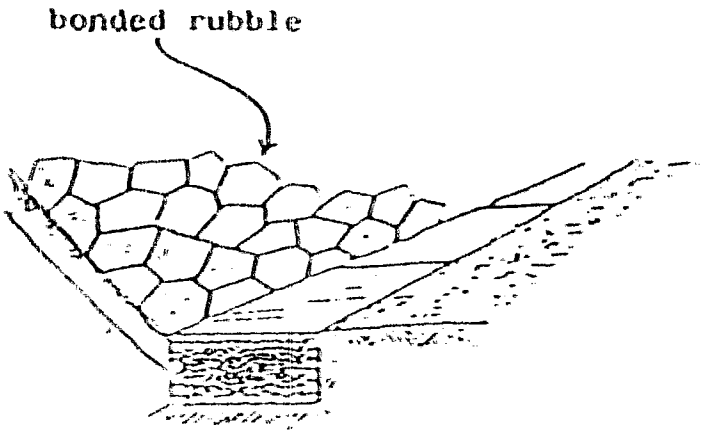




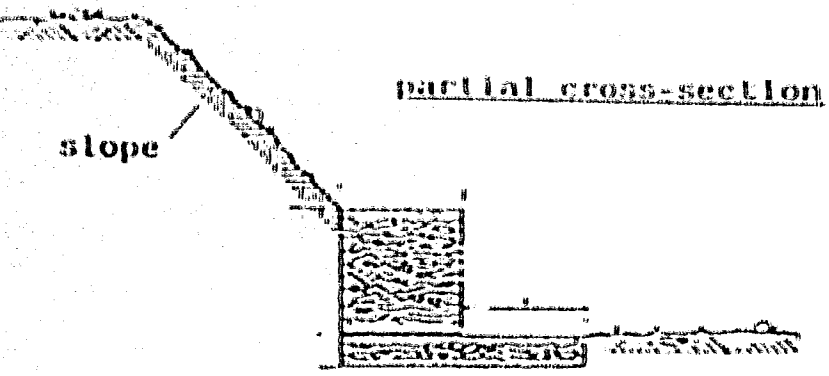
Angle view

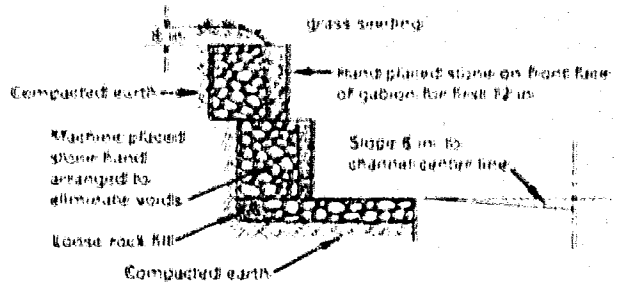
Bank and Slope Protection

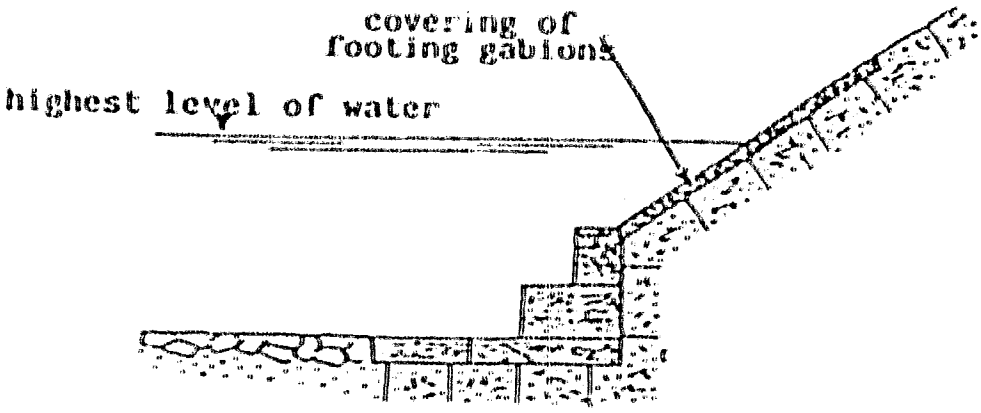


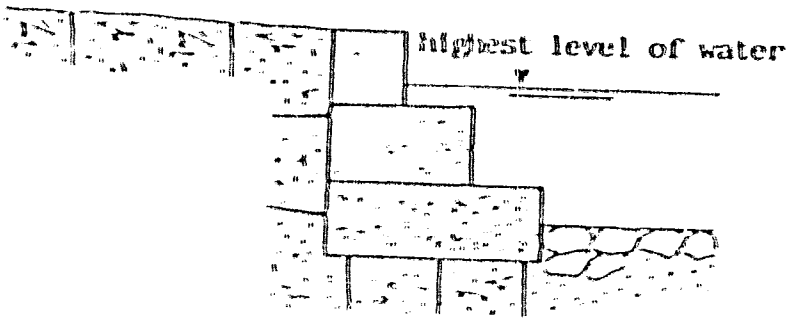


Bank Slope Protection



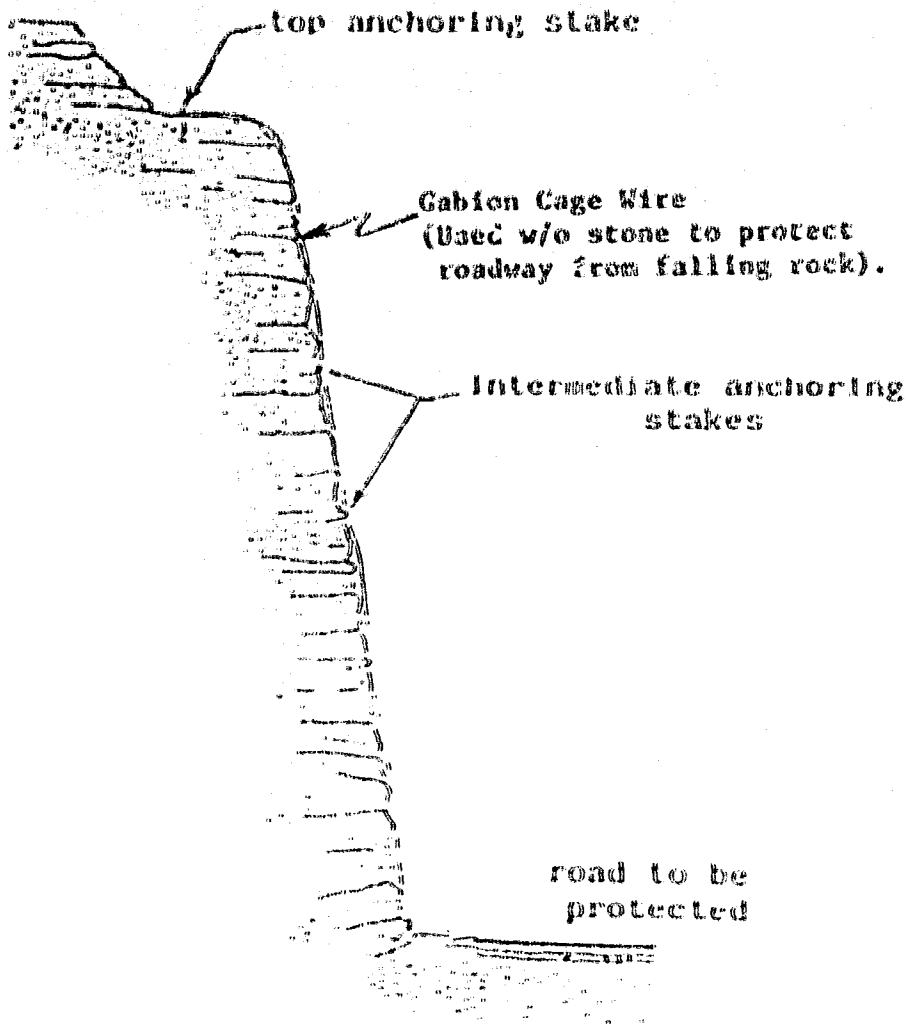




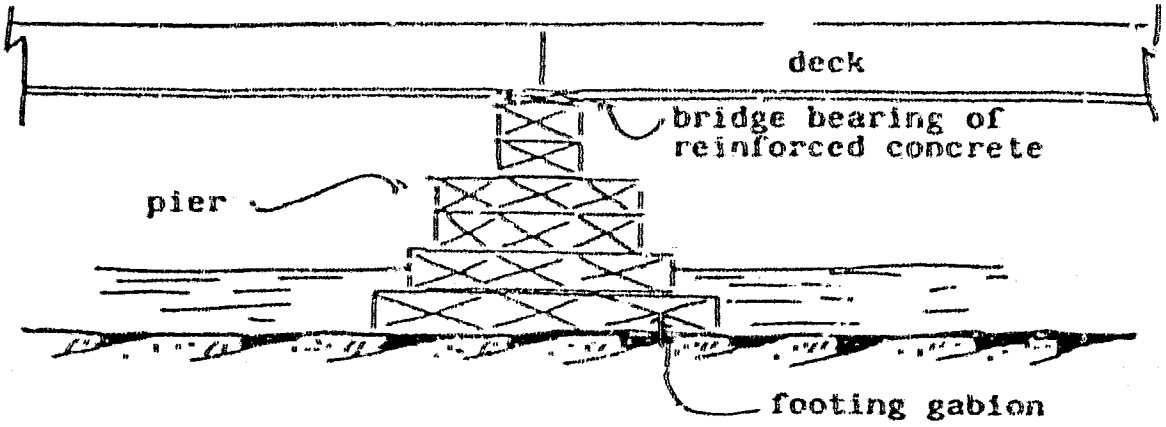


CANAL BANK PROTECTION

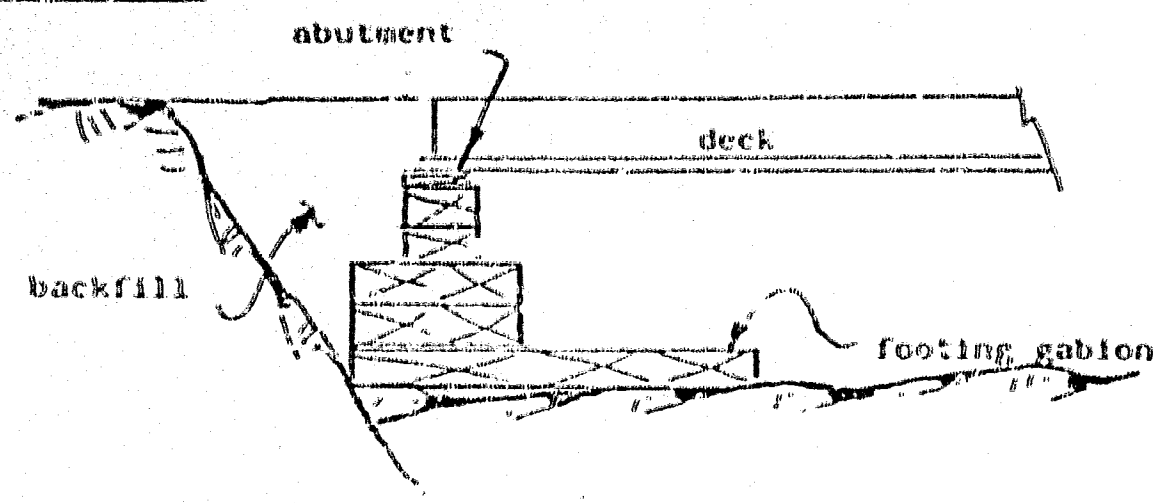
Slope Protection



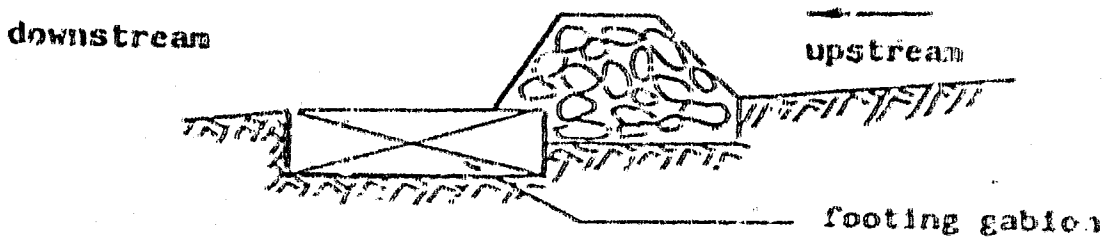
Bridge Foundations



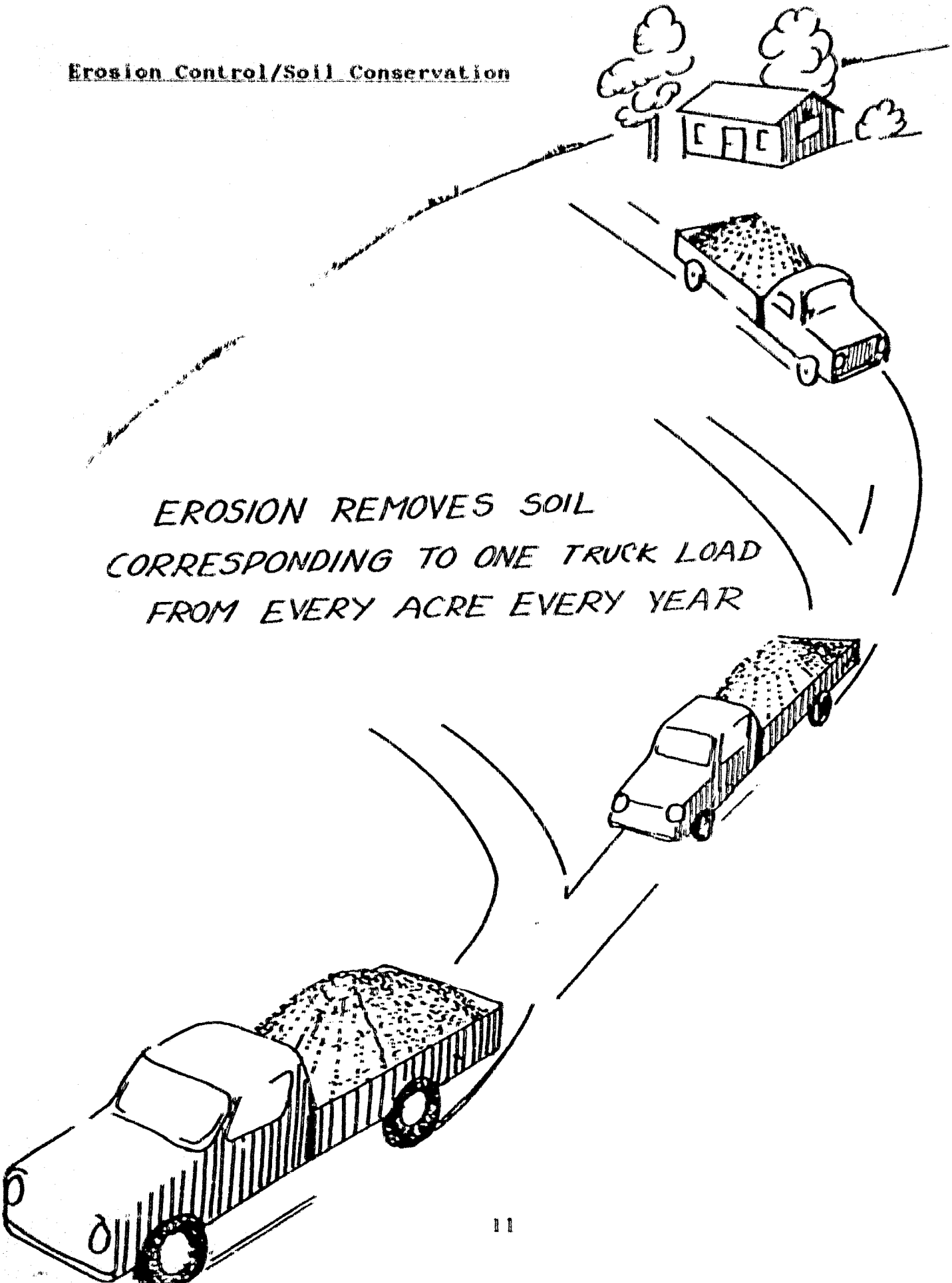
Bridge Foundations



Gully Correction



Erosion Control/Soil Conservation



EROSION

Rain and water flows remove the top soil from arable land.

The annual loss of top soil is much higher than you might think, usually much more than a big truck load of soil from every acre every year. If not stopped the top soil will get thinner every year and yields will decrease. Rills can develop into gullies dissecting the land, making it unsuitable for agriculture or others uses.

Erosion is more severe in loamy-silty-sandy soils than in clay soils, and it is also more severe on long and steep slopes. Basically erosion is dependent on the pattern of cultivation, especially in regard to the vegetative cover and the structure of the soil.

Erosion (soil loss) consequently depends on several factors:

amount and energy of rainfall	+	erodibility of the soil	+	length and steepness of slope	+	crop and land management

you cannot change the rainfall or the soil, but you can change the length and also the steepness of a slope through "terracing".

Terraces can be constructed as ditches across the slope or as benches or ridges across the slope - these terraces help to reduce the slope of a hillside & thereby reduce the velocity of the flow of water. If the velocity of water flow is not reduced by terracing the production of rills & gullies will be increased.

Rills are channels in the ground made by a small stream of water, whereas gullies are larger trenches in the ground made by running water.

GULLY CONTROL

When rills have increased in size so that they cannot be levelled out through ploughing or other simple means they are called gullies. Such channels can rapidly increase in size, especially in silty- fine sandy soils, even if the water flows are small.

Erosion Processes in Gullies

At the head of a gully a "waterfall" type of erosion causes a rapid cutting back into the slope (fig. a).

fig. a

If the earth layer is thick enough, the flow over the floor of the gully will deepen the gully. This deepening does not stop until the gully has reached solid rock (fig. b, showing 3 stages of erosion in the longitudinal direction of the gully).

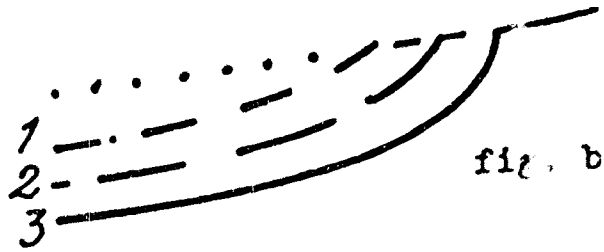


fig. b

The deep erosion makes the sides of the gully unstable and causes movement of the soil from the sides of the gully to the bottom. In this way the gully is not only deepened but also widened (fig. c, showing a cross-section of the gully with 3 stages of erosion).

fig. c

Measures

There are two main measures to control gully erosion:

1) Diverting the water which enters the gully by means of a cutoff drain, or a ridge of soil.

2) Preventing the water in the gully from causing erosion by protecting the head and the floor with erosion resistant materials. The measures will differ according to the shape of the gully:

   a) A wide, shallow and not very steep gully, compared to
   b) a narrow and steep gully (with concentrated large water flows).

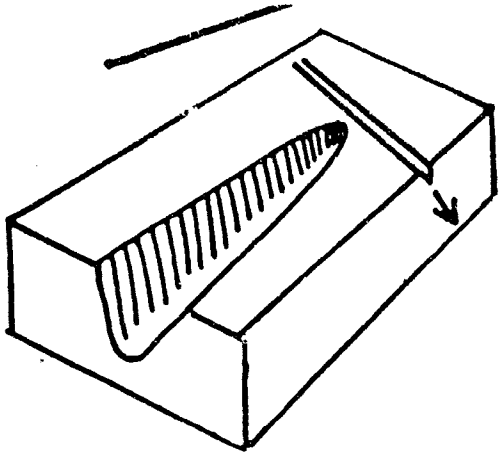
When choosing the requisite measure it is necessary to consider the availability and cost of construction materials.

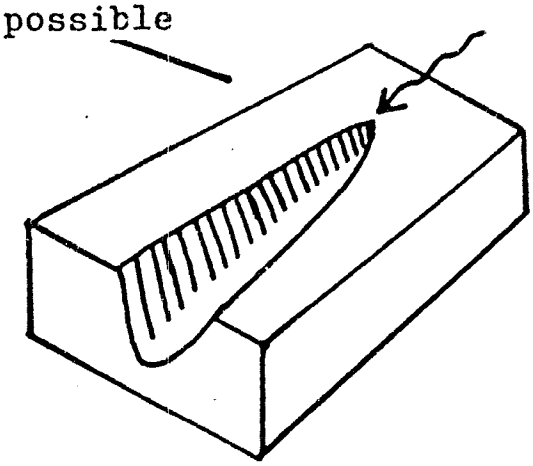
GULLIES

Diversion of water

possible

not possible





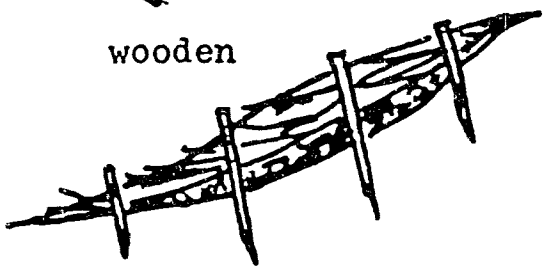
Head of gully

Side walls
(usually no measure taken)

Floor of gully

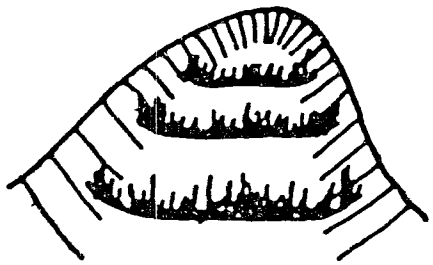
stones

wooden



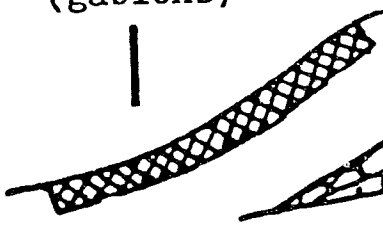
wide
and shallow

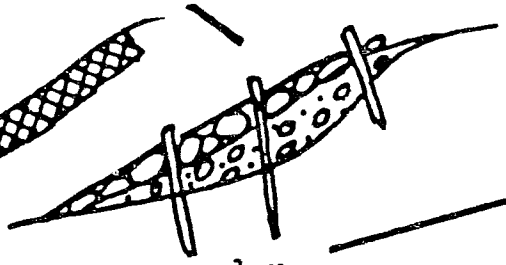
narrow
and steep



small stones
(gabions)

large stones

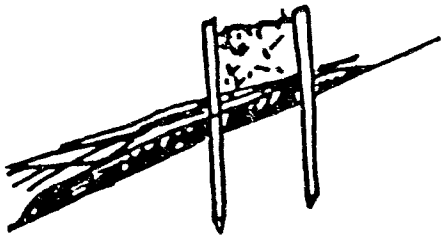


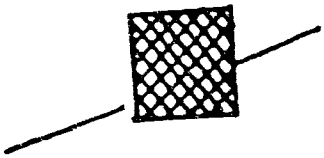


wooden

large stones

small stones
(gabion)





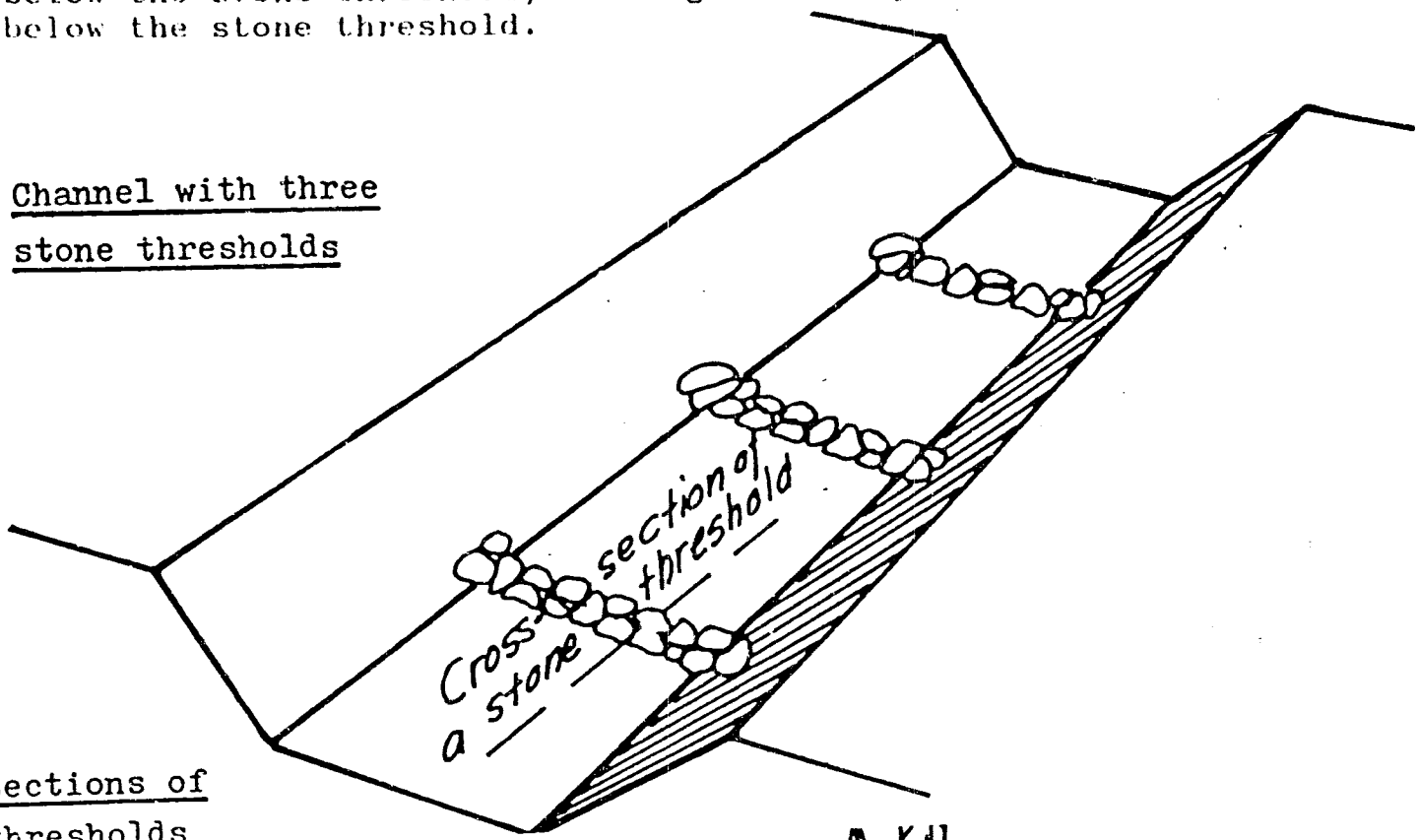
Floor of Gully, Wide and Shallow

Erosion on the floor of a gully can often be stopped by strips of grass, placed at every 2nd, 4th or 6th meter.

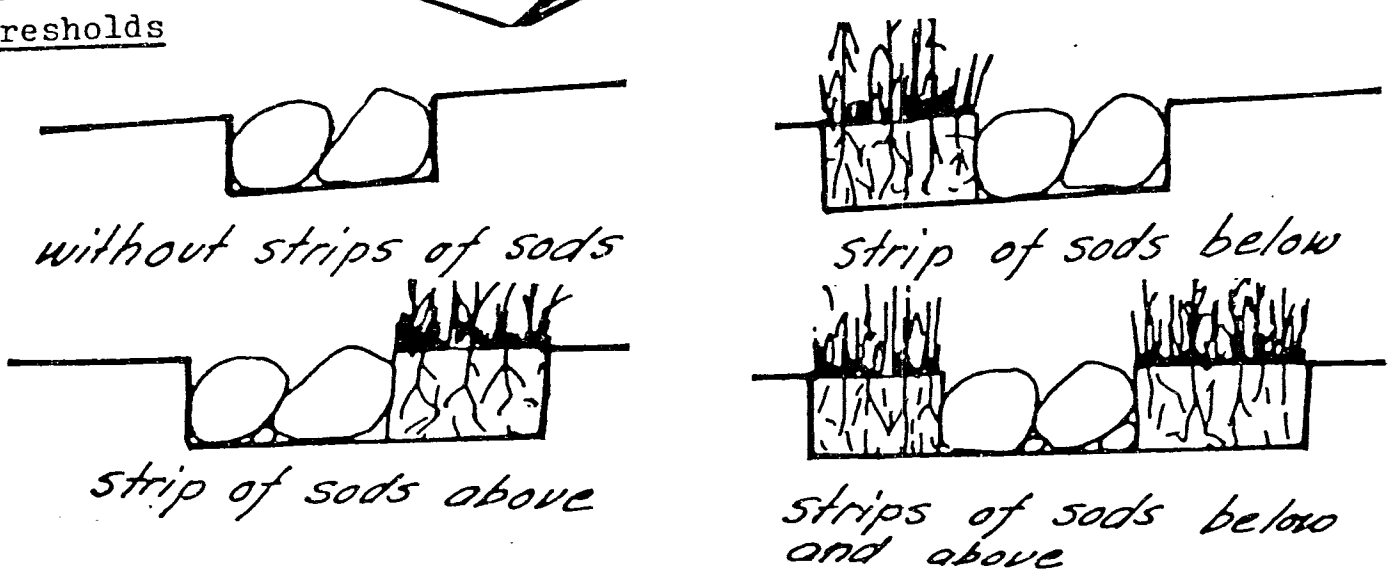
Alternatively stone walls called check dams can be arranged across the floor instead of vegetation. The stones should be put into an excavation (approximately 0.5 feet deep), so the upper parts of the stones are level with the floor of the gully. It is also possible to combine the two types of materials:

1) a grass strip above the stone threshold, 2) a grass strip below the stone threshold, or 3) grass strips both above and below the stone threshold.

Channel with three stone thresholds



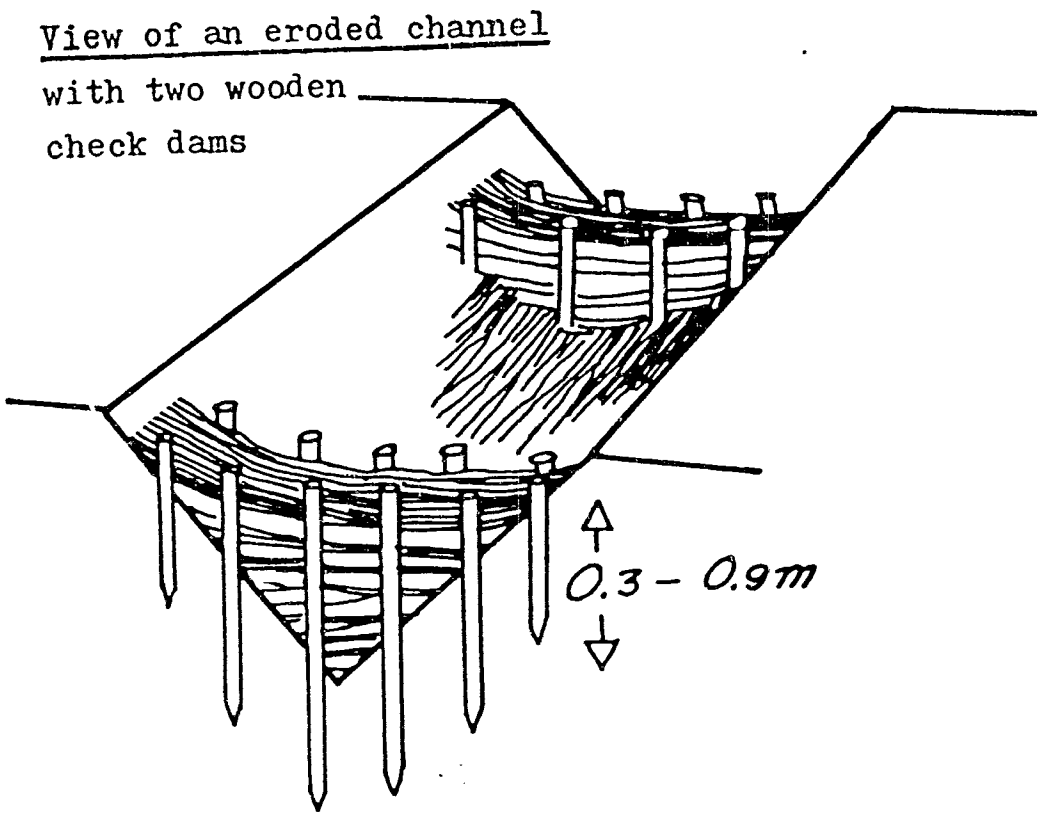
Cross-sections of stone thresholds



Floor of a Gully, Narrow and Steep

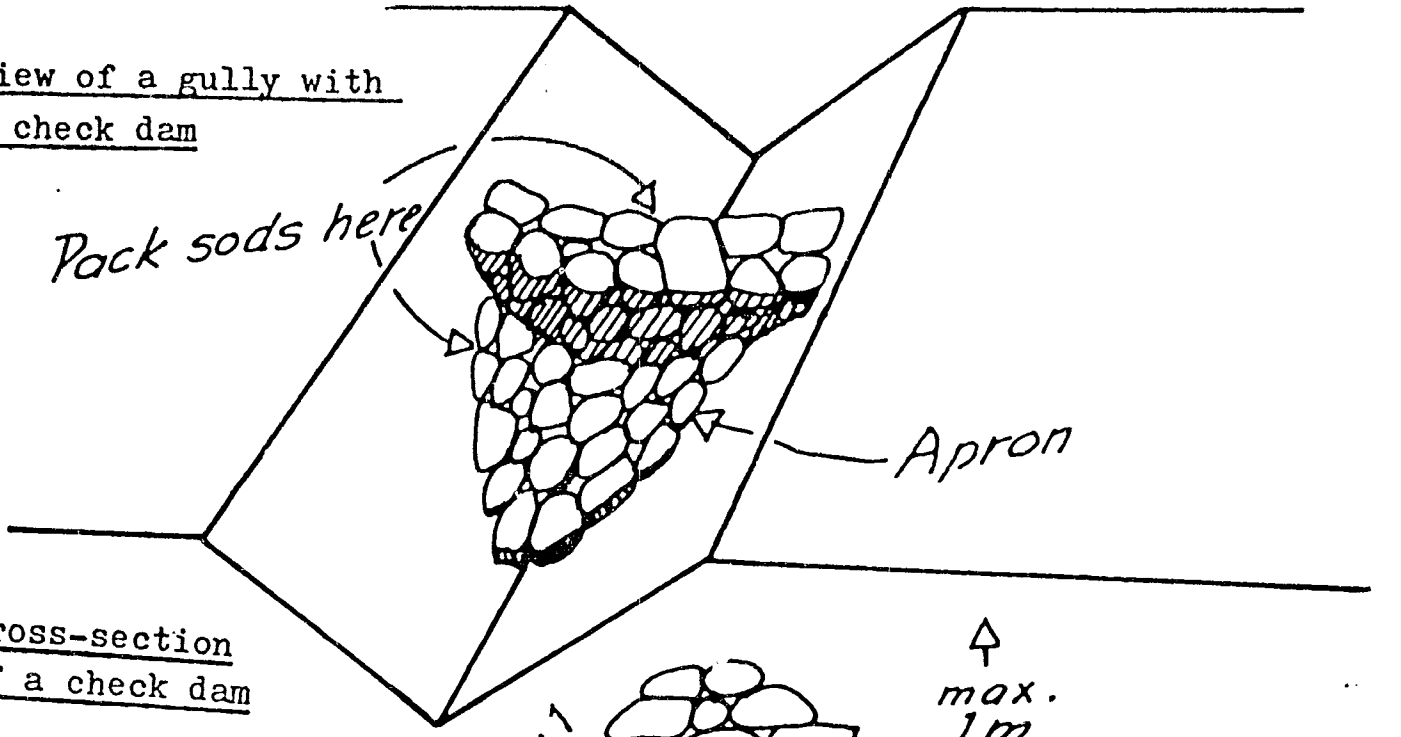
In steep and narrow gullies the check dams have to be made more solid to reduce the the slope and to resist the velocity of the water. The ends of the check dam should be somewhat higher to prevent water from cutting round them.

Wooden Material (No Stones Available)

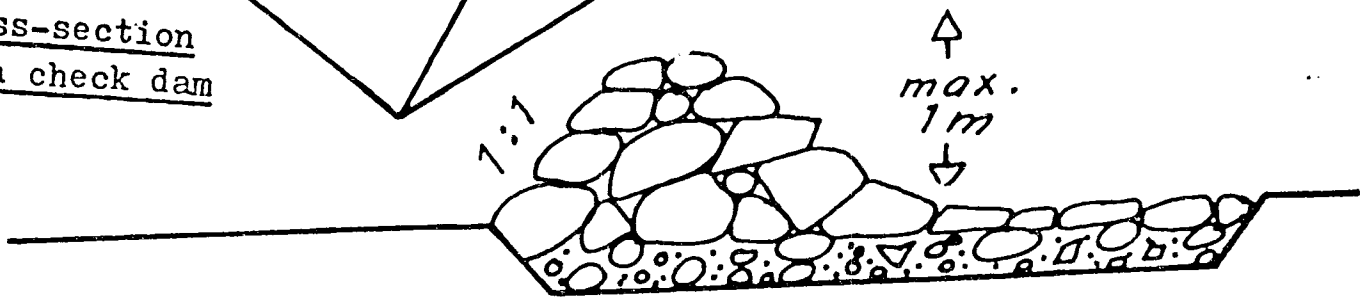


CHECK DAM USING STONE MATERIAL
(Stone Material, Large Stones Available)

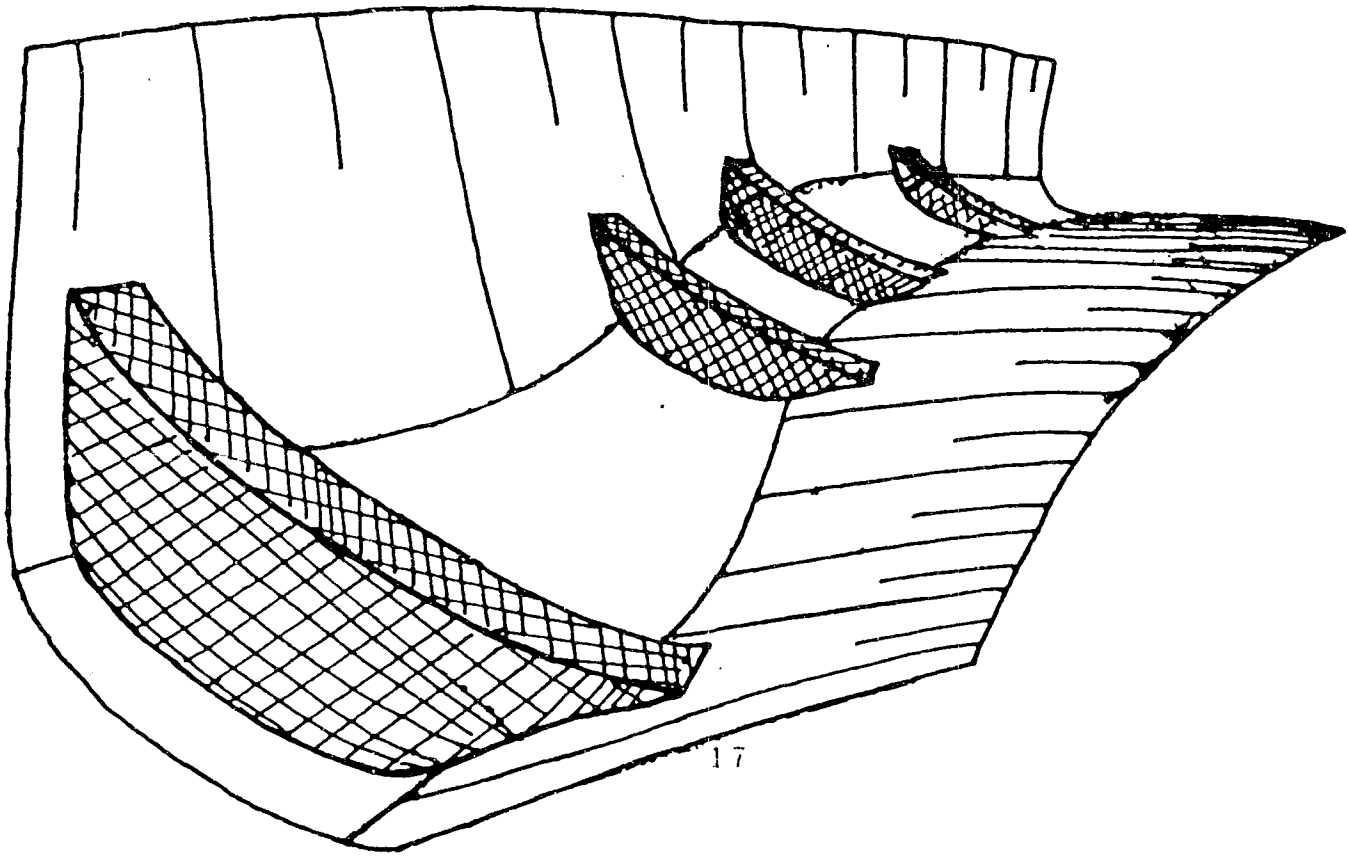
View of a gully with a check dam



Cross-section of a check dam



Stone Material, Large Stones Not Available
In such a case gabions can be used.



Advantages of Using Gabions

The use of gabions offers several advantages over that of other construction materials, especially where work on soft or unstable ground is to be undertaken.

Ease and Flexibility of Use

Gabion work can be interrupted at any stage without any risk to the solidity of the finished structure. This means that the progress of work can be modified according to the labor actually available in the project area.

Work can be interrupted, for example during the rainy season. This is not always possible and always difficult on sites where masonry or concrete works are being undertaken.

Uniformity of Gabion Structures

Gabion structures are made up of large blocks, joined one to another and are, therefore, extremely stable. Gabion work resists, remarkably, well, erosion and undercutting by water.

Elasticity

The elastic quality of gabion work is one of its main advantages. The use of hexagonal wire mesh allows a gabion to modify its form to compensate for irregularities in the natural ground on which it lies without weakening its structure.

It's suppleness makes its use particularly appropriate on soft ground which is prone to erosion and undercutting.

A gabion moulds itself perfectly to the profile of the ground.

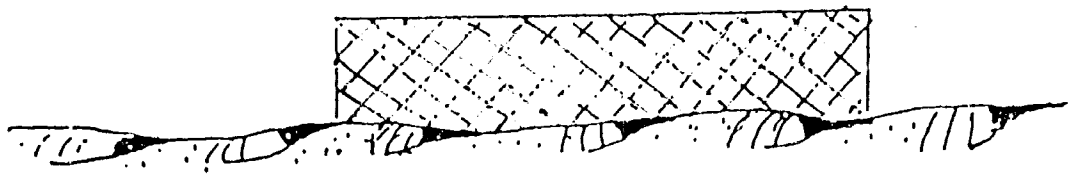
Ease of Modifying, Raising and Repairing Gabion Structures

The simple concept of gabion structures means that, in certain instances, work can be undertaken in stages, for example:

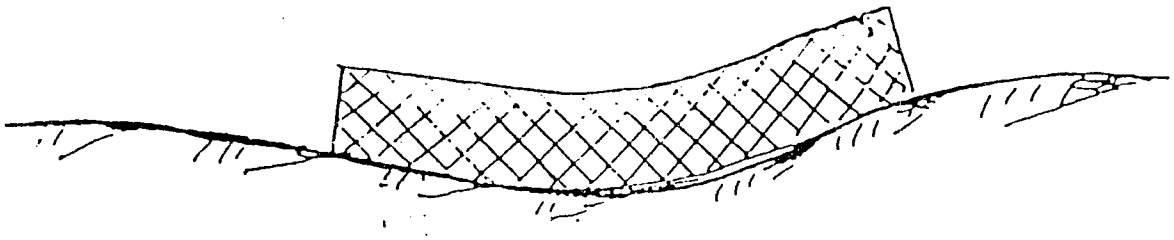
- raising an overflow lip (spillway);
- raising a bank protection wall;
- extending a bank protection groin;

This means that the pace of work can be adapted to the availability of local labor which will vary throughout the year.

Any subsidence of the ground under the gabion will be absorbed by the gabion's mass, with no significant risk to the stability of the structure as a whole.



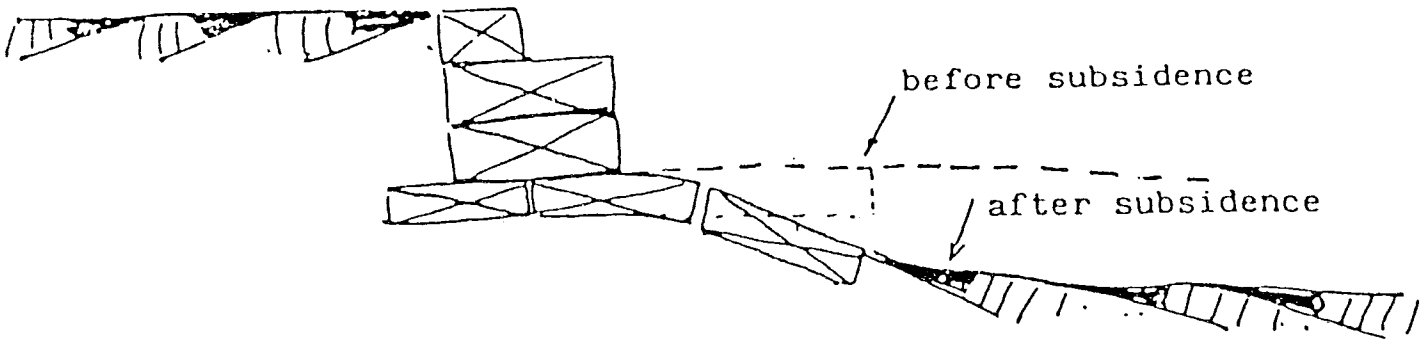
BEFORE UNDERMINING



AFTER UNDERMINING

AFTER UNDERMINING

By virtue of its ability to modify its shape, moulding itself to the form of the ground, the footing gabion is very useful for foundation work.



The technique of gabion work is of great value in development projects.

The material used is inexpensive and the simple assembly techniques involved require no special tools or skills.

Nevertheless, a certain amount of care needs to be taken as far as the conception and execution of projects using gabions are concerned.

Indeed, even minor mistakes made in the planning and implementation stages of gabion work can bring about the rapid destruction of the work. This occurrence, which gives rise to a certain distrust and dislike of the technique, is unfortunate since, when properly designed and executed, gabion work provides an economical solution to the construction of numerous types of structures.

Moreover, the decision to adopt this technique benefits unemployed or underemployed communities since it necessitates the mobilization of a large work force of unskilled labor.

Simplicity of Gabion Work

The conception of gabion structures is very simple. Stability can be increased by using oversized gabions, without pushing up the cost of manufacture unduly.

Increased Utilization of Local Resources

Apart from wire mesh which, in the majority of cases, is imported, and entails foreign currency spending, the rest of the resources needed for the manufacture of gabions is available locally.

These resources are:

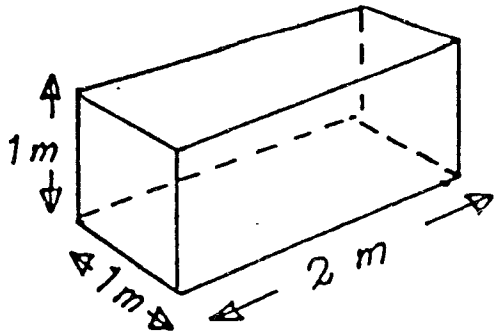
- Unskilled labor;
- Skilled labor;
- Gangleaders;
- Materials (Stones and Pebbles);
- Tools for stone extraction and gabion-handling.

CHARACTERISTICS

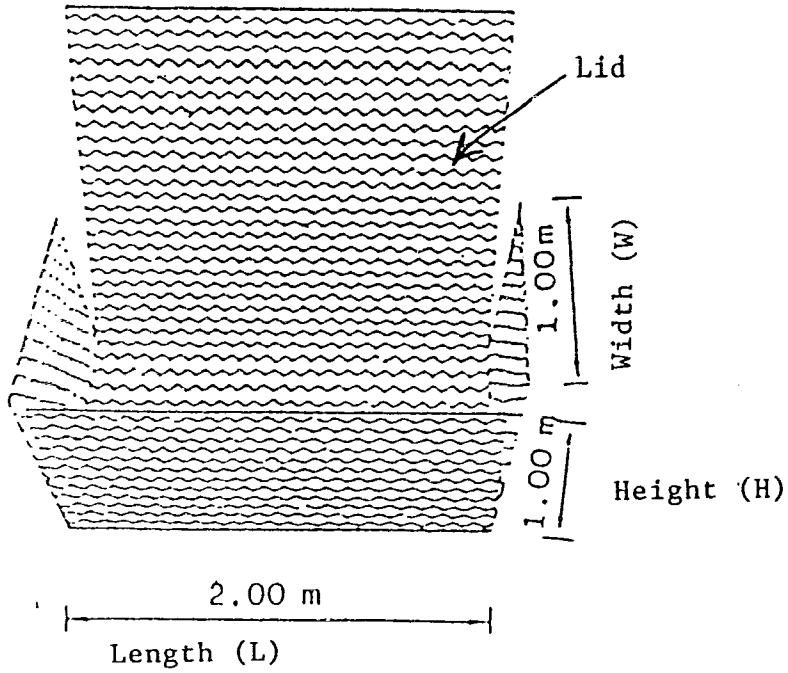
TYPES OF GABION

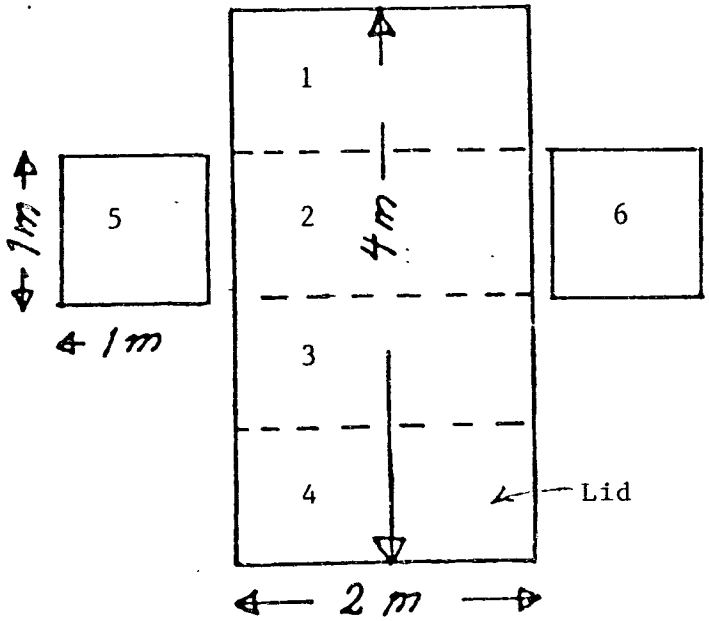
The most common and versatile gabion is a rectangular paralelepiped (box shape) in form, 1 meter high and wide and between 2 to 5 meters long, according to specific requirements.

Additional partition walls, known as diaphragms can be fitted to strengthen the gabion, preventing it from becoming too misshapen on ground which is liable to subsidence.



STANDARD CAGE GABION





The above standard size needs a total of 10 sqmtr of mesh i.e.

a) for 4 sides 4x2 m = 8 sq.m of wire mesh.

b) for 2 ends 2 pieces of 1 x 1 m = 2 sq.m.

OTHER TYPES OF GABIONS

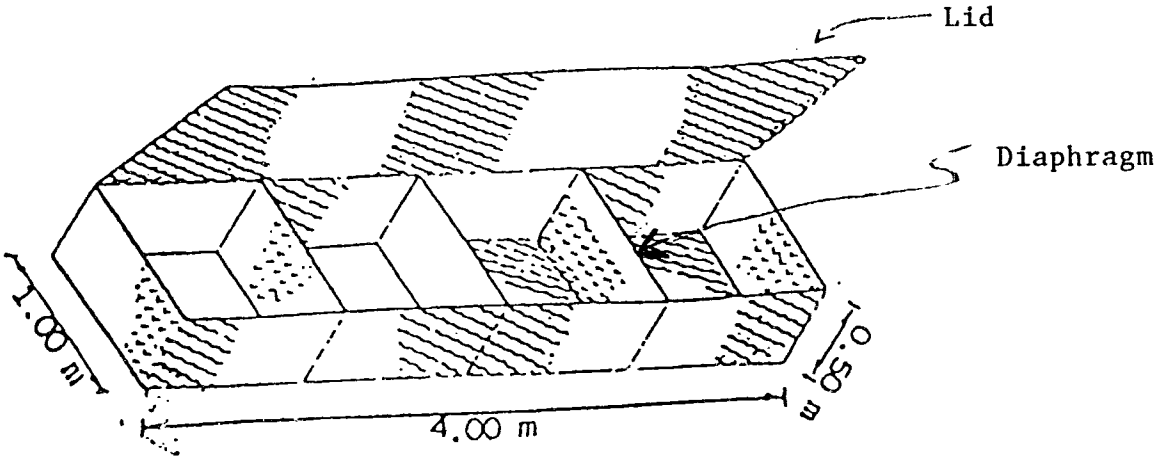
Other gabion types than the standard cage gabion exist, although less frequently employed they can be adapted to meet the specific requirements of certain types of structures.

Footing Gabions

The only distinction between the footing gabion and standard gabion is its height which is generally 50 centimeters.

Footing gabions also exist in heights of 15, 23, 25 and 30 centimeters. Such gabions are especially suitable for bank protection work and for reinforcing canal banks.

Like ordinary gabions, footing gabions can be fitted with diaphragm walls inside, for use on unstable ground.



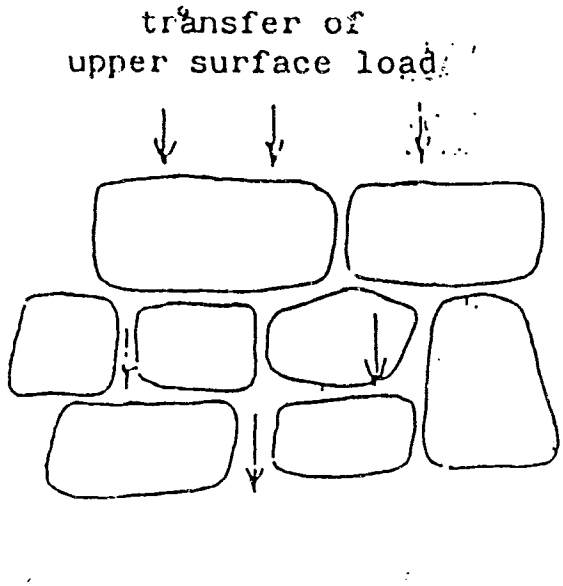
FOOTING GABION FITTED WITH DIAPHRAGMS

Stones

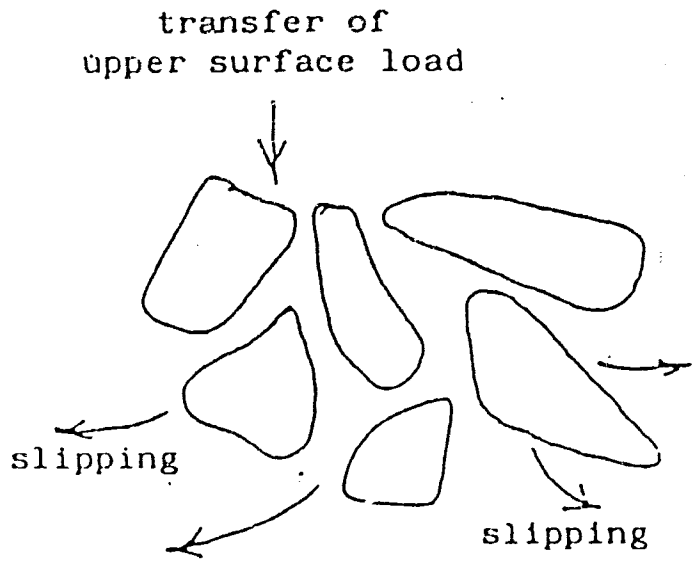
The purpose of the stones contained within gabion cages is to add weight and to absorb compression which is the only force gabion structures have to withstand.

Movement caused by stones sliding against each other is absorbed by the wire mesh of the cage. The main consideration is to prevent too much movement within the cage by carefully arranging the stones in such a way that there is the maximum contact between their surfaces.

If care is not taken when stacking the stones, they will slide about, deforming the mesh cage and even breaking it.



CORRECTLY PACKED STONES

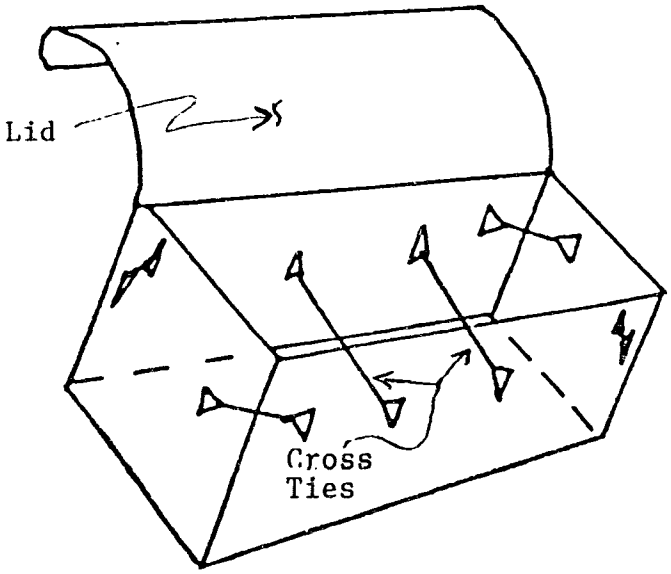


BADLY PACKED STONES

Nevertheless, even when every precaution is taken in filling gabions they have a tendency to deform when the stonework suffers forces other than of compression.

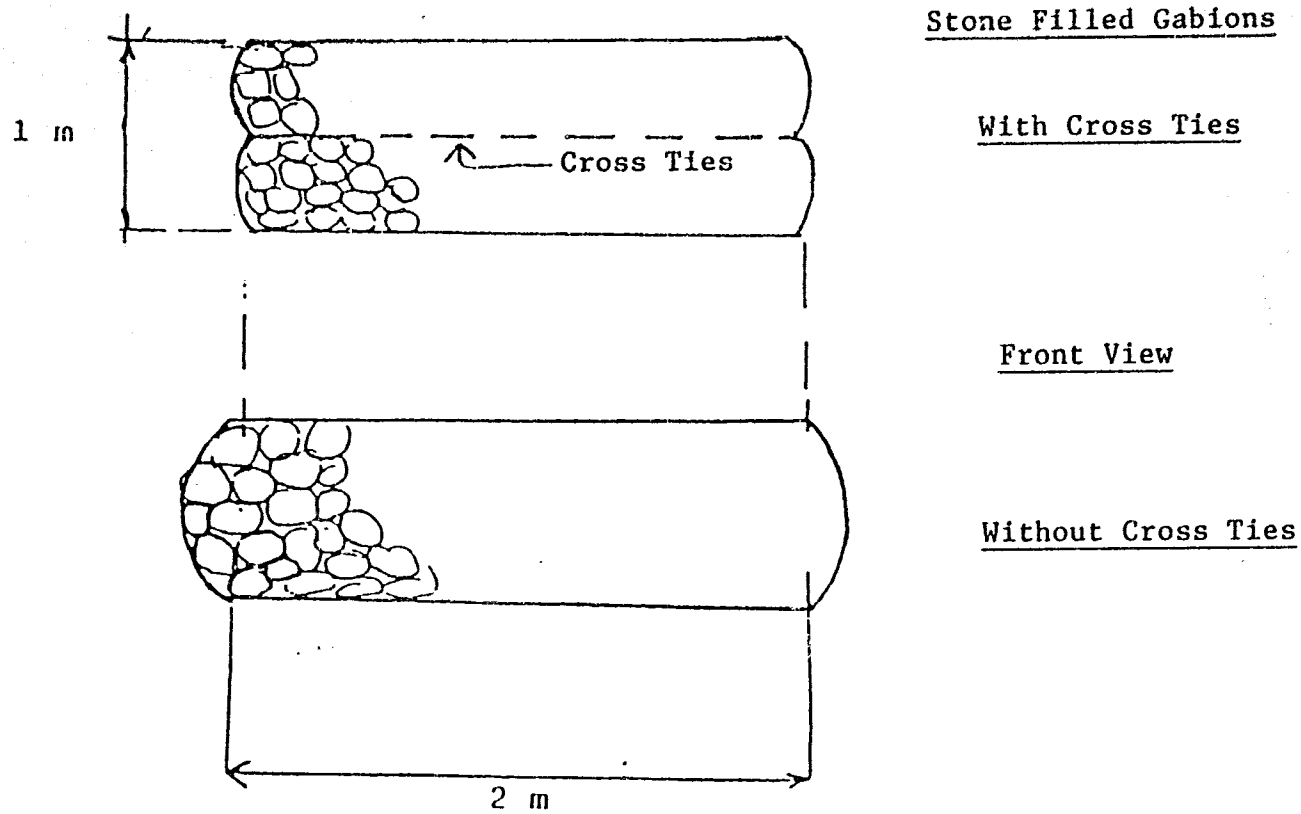
Cross Ties

To combat this eventuality, the gabion can be cross tied using galvanized wire to link opposite and adjacent walls together, which will help reduce the tendency of the gabion to become misshapen.



During the filling of the gabion box with stones, cross-ties help to stabilize the box; ties can be placed across the box in either direction and/or diagonally in the corners.(as shown in sketch)

Diaphragms and cross ties strengthen the wire mesh cage and help the gabion keep its shape during and after filling. Cross ties also contribute to the even distribution of stones after stacking. The addition of cross ties should always be used for gabions over two meters in length.



SIZES AND SPECIFICATIONS

The most common specifications for gabions are the following (hexagonal wire mesh):

Standard Gabions

Length m	Width m	Height m	Volume m3	Weight of basket (Kg)
2	1	1	2	14
3	1	1	3	20
4	1	1	4	25
5	1	1	5	31
6	1	1	6	37

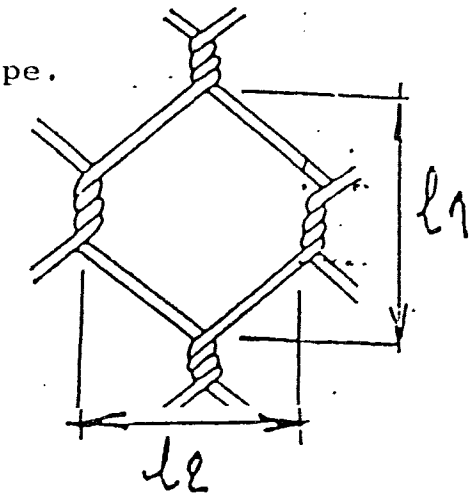
Footing Gabions

Length m	Width m	Height m	Volume m3	Weight of basket (Kg)
2	1	0.5	1.0	10
2	1	0.3	0.6	7
3	1	0.5	1.5	14
3	1	0.3	0.9	11
4	1	0.5	2.0	18
4	1	0.3	1.2	13
5	1	0.5	2.5	23
6	1	0.5	3.0	27

Smaller-sized mesh exists i.e. 80 x 110 mm, 50 x 70 mm, but is reserved for instances where only small stones or pebbles are available or where a structure needs to be exceptionally resistant.
It is always more economical to use large-sized mesh gabions (120 x 100 mm) filled with large stones.

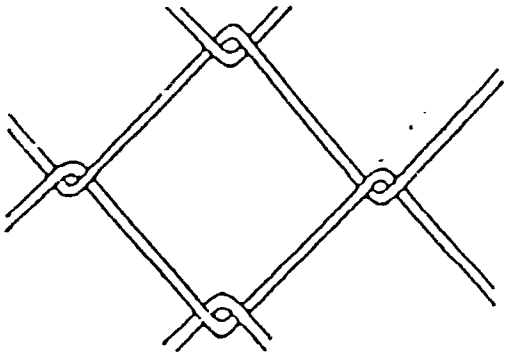
Mesh should be of the hexagonal Double-twist type.

DOUBLE-TWIST
HEXAGONAL MESH



Gabions may also be made using single-twist square mesh which can be manufactured locally from imported rolls of iron wire.

Although less expensive, this type or gabion is not recommended for large structures or for those which will have to stand up to great stress. The single twist square mesh can be used, however, in constructing low-stress structures, e.g. small supporting walls, 1-2 m high.

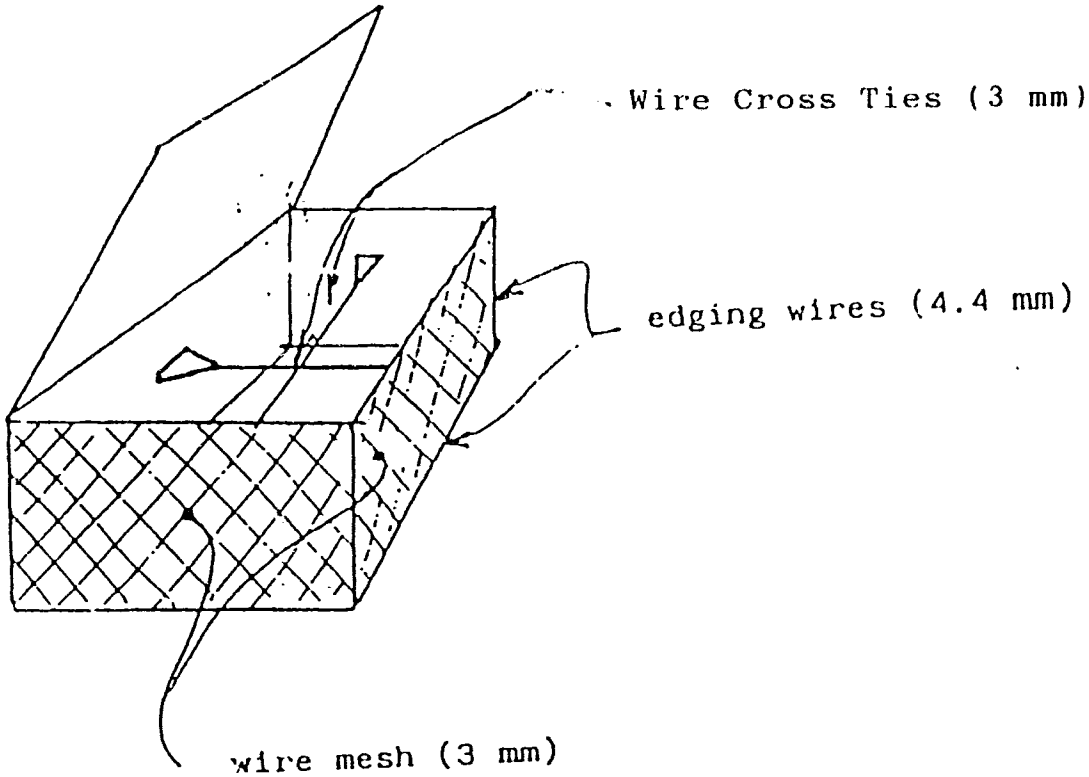


SINGLE-TWIST
SQUARE MESH

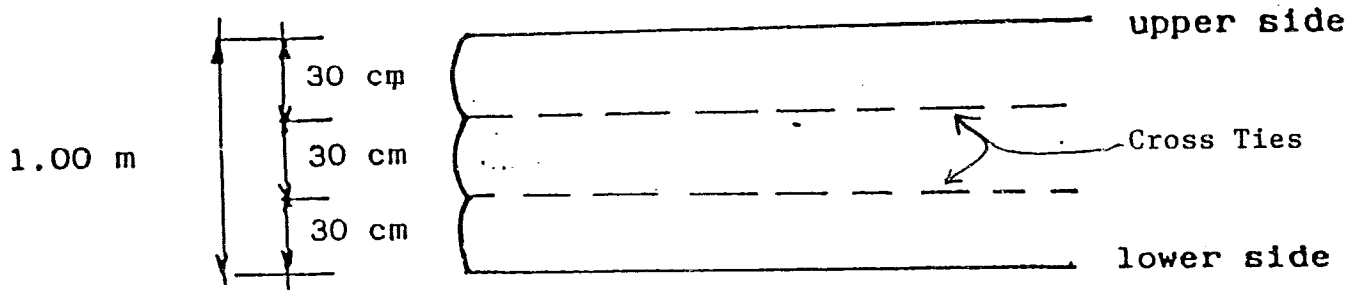
It is imperative that gabion cages be made from galvanized steel wire. Plastic - (PVC) covered galvanized steel mesh is used to advantage in corrosive environments (e.g. for marine structures).

Wire Specifications

Binding wire, which is used to secure the lid of the cage, should be about 3.5 to 4.0 mm in diameter. 3mm diameter wire may be used if there is difficulty manipulating and tying the heavier wire.

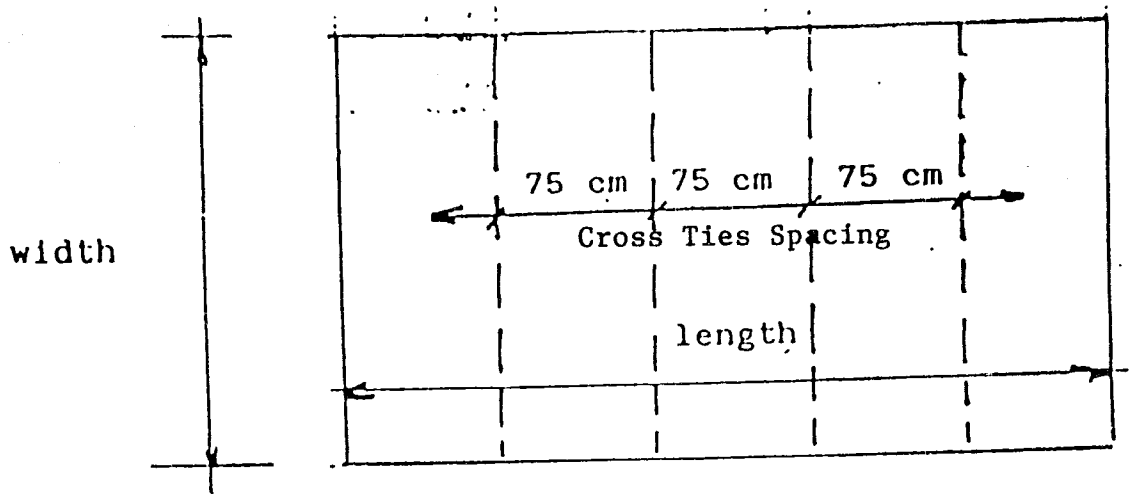


Horizontal cross ties are placed at 30 cm intervals, e.g. a 1 meter high gabion will be fitted with two cross ties.



Front view

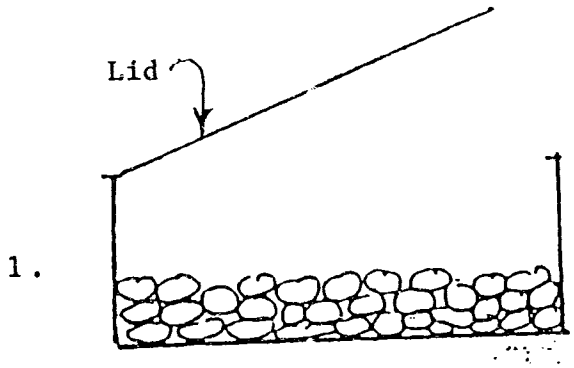
These same cross ties are spaced 75 cm apart along the length of the gabion.



Plan view

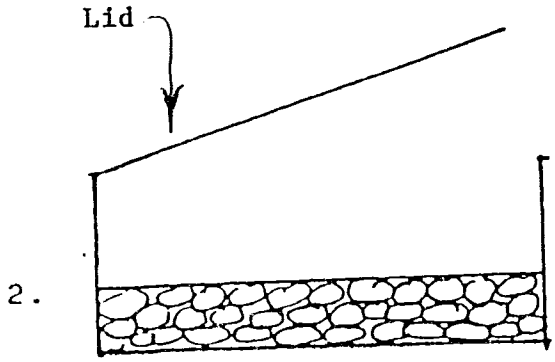
Horizontal cross ties are positioned one at a time while the gabion is being filled with stones.

The number of horizontal and vertical cross ties required is determined according to the size and shape of the filling stones. For example: More ties should be fitted with round pebbles, (which tend to slide, causing a gabion to become misshapen and to bulge) than in a gabion filled with angular stones which "hold" better.

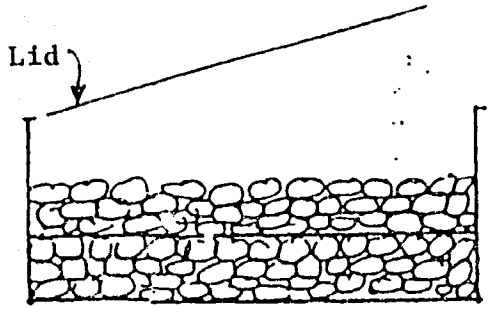


Sequence for fitting trusses

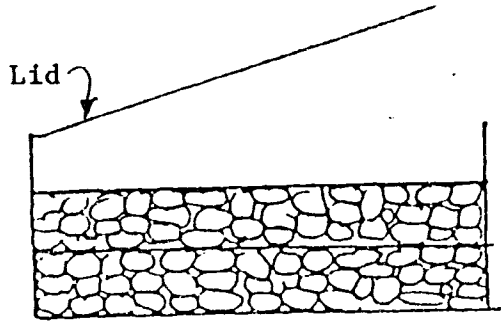
Fill up to level of first set of cross ties.



Fit first set of cross ties.



3. Fill with layer of stones up to level of second set of cross ties.



4. Fit second set of cross ties.

PLACING THE GABIONS

Required tools:

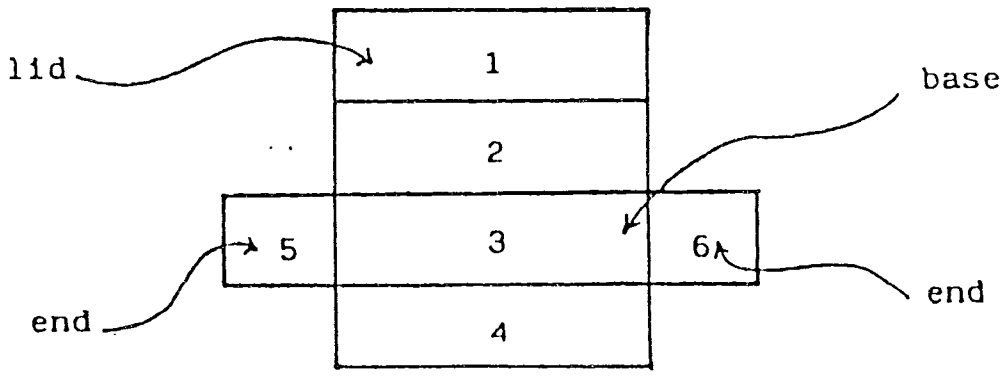
- Pliers, wire cutters, iron jumper bar, 8 kg hand rammer, shovel, pickaxe for levelling the ground, if necessary.

The first operation to be undertaken is the unpacking of the wire cages, which are usually delivered in flat parcels of five to ten gabions, weighing up to 200-300 kg.

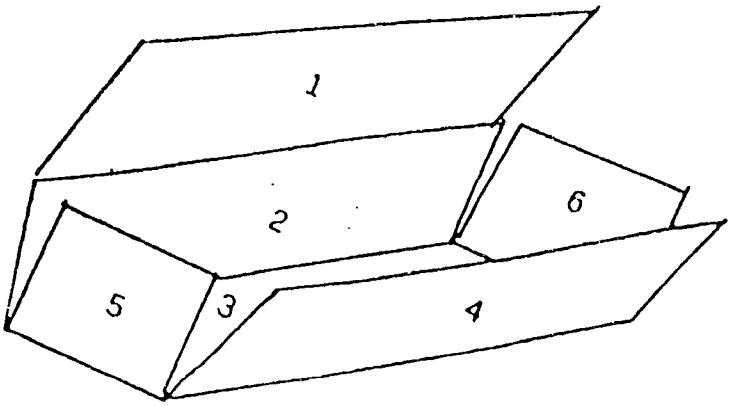
After checking that the goods conform to the order, the parcels should be unpacked before being unloaded. The parcels of gabion cages should be unloaded directly in the place where they will be used so as to save time and effort.

Assembling the Cages

STEP 1 Unfold the cage and place it a few meters away from the place where it will be used.

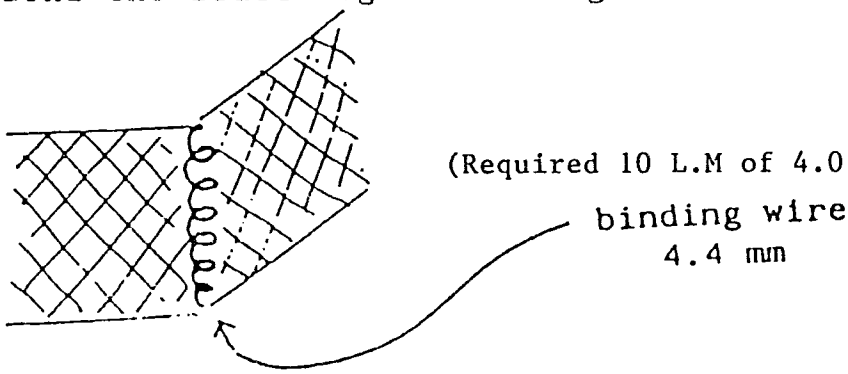


STEP 2 Bring sides 2, 5, 4 and 6 together to form a box, leaving the lid open.



BRINGING THE SIDES OF THE GABION TOGETHER

STEP 3 Bind the sides together using wire.

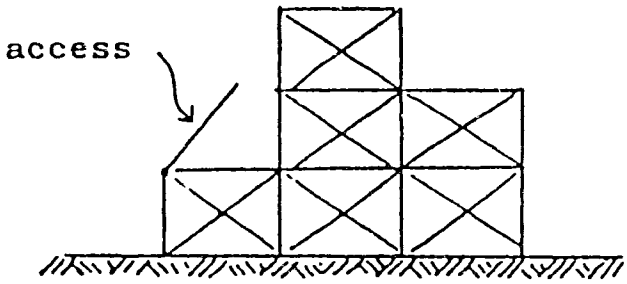


(Required 10 L.M of 4.0 to4.5 MM deam. wire)

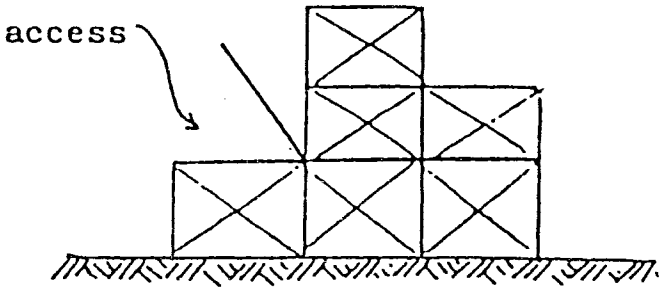
Wire cutters should not be used for this. Pliers alone must be used as these will not nick the wire. To ease binding, 4 to 4.5 mm binding wire can be replaced by a double binding of 3 mm wire.

STEP 4 Put the empty cage in place in the construction.

Care should be taken that the open lid does not hamper the filling of the cages.

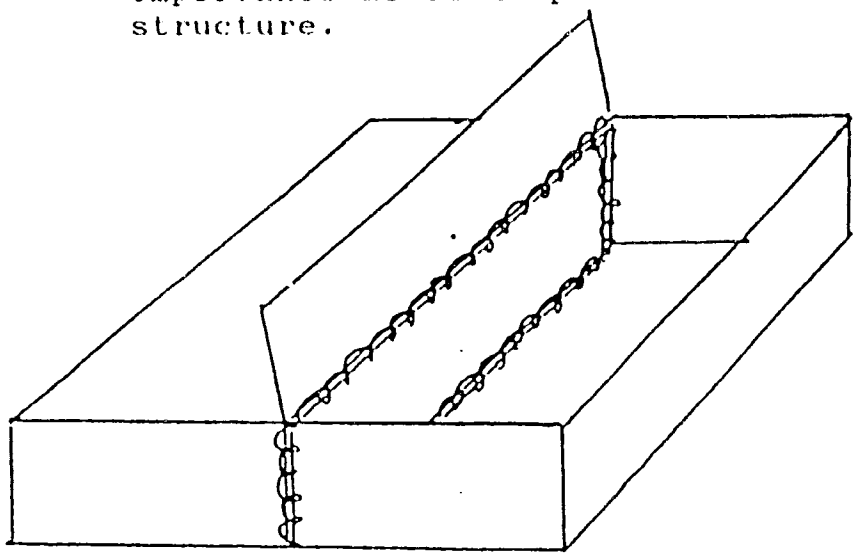


NO



YES

STEP 5 Bind the gabion by its edges to those of the gabions which are already in place. This step is of particular importance as it helps ensure the solidity of the structure.



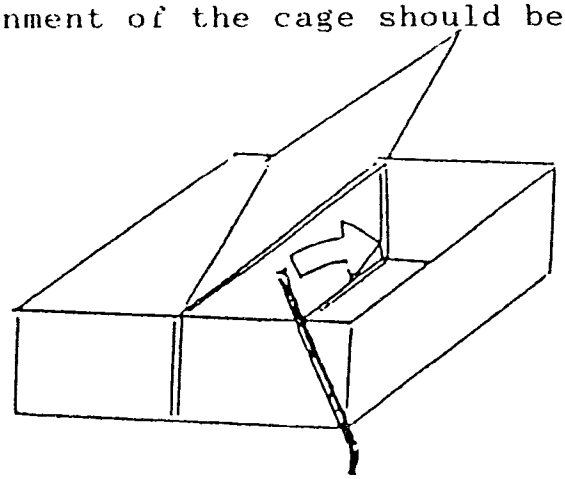
CAGE BEFORE FILLING

The cage is secured by binding adjacent edges together using 4 mm wire or a double binding of 3 mm wire.

STEP 6 Using a jumper bar (a heavy iron bar of 18 or 22 mm diameter), the base of the cage is stretched into its final position.

The bar is used as a lever. It is placed through a hole in the mesh near a corner and pressure is exerted on it to force the free sides of the cage into their final positions.

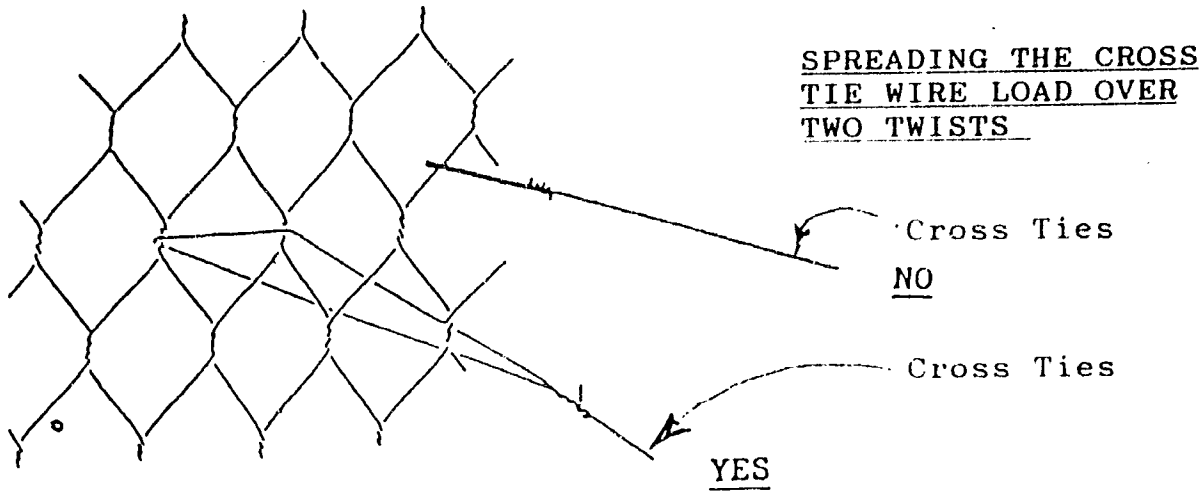
The alignment of the cage should be checked.



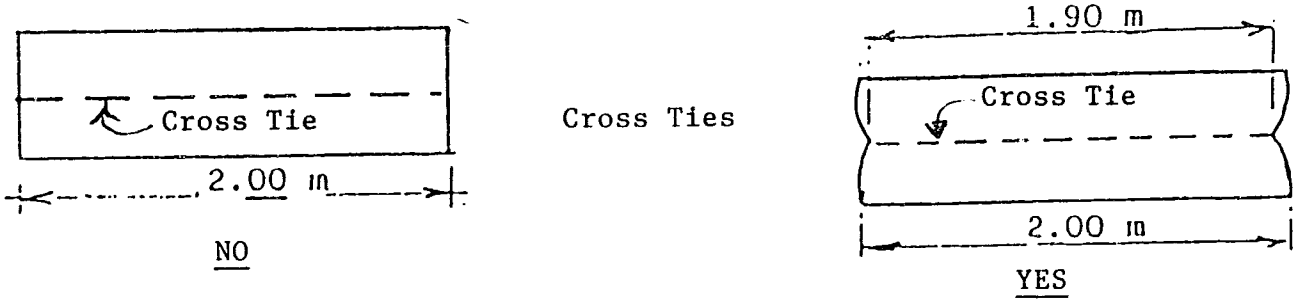
USING A JUMPER BAR TO STRETCH THE BASE INTO POSITION

STEP 7 Fitting Cross Tie Wires

Precaution should be taken during this operation. The tie wires should be attached to two or three wires of the mesh to prevent them stretching or breaking, by spreading the tension borne by them over an area of the mesh.



Longitudinal cross ties should be slightly shorter than the distance between the sides they join. A 5 percent reduction in the length of the tie is normally provided for.



STEP 7 Angle cross ties are then fitted, taking the same precaution.

Vertical cross ties are attached to the bottom of the wire mesh cage, their other end being left free until the gabion has been filled with stones. They are then attached to the closed lid.

It should be borne in mind that the ultimate strength and rigidity of the finished structure depends, to a great extent, on the care taken in assembling and preparing the gabion cages.

Workers' Productivity

These are only suggested norms. In fact, productivity norms vary for site to site, depending on various factors, notably:

- the type of setting
- organization of works
- workers' training
- kind and size of stones
- quality of tools
- climate, etc., etc.

The productivity of workers engaged in unpacking and assembling gabion cages can be estimated using the following mean values (time taken unloading gabion parcels and positioning assembled gabions within the construction is not included in these indications).

Productivity in m^3/man days

Volume of the cage	Dimension	Surface area of wire mesh	Productivity
1.00 m^3	2 x 1 x 0.5	7 m^2	8 m^3/md
1.50 m^3	3 x 1 x 0.5	10 m^2	6 m^3/md
2.00 m^3	2 x 1 x 1	10 m^2	4 m^3/md
4.00 m^3	4 x 1 x 1	18 m^2	3 m^3/md

CHOOSING THE STONES AND FILLING THE GABION

To be chosen for gabion-filling, stones need to have the following qualities.

- they should be non-porous;
- they should be sufficiently hard.

These conditions fulfilled, most stones are suitable for gabion-filling, provided they are of suitable dimensions.

The stones selected should weigh between 5 and 10 kg apiece.

Stone Collection

Stone may be obtained from quarries, river beds or gathered and extracted from the ground.

The workers unearth and collect stones, making piles of different sized stones beside a road, track or right-of-way.

The stones will then be loaded and transported to the place where they will be used to fill gabions.

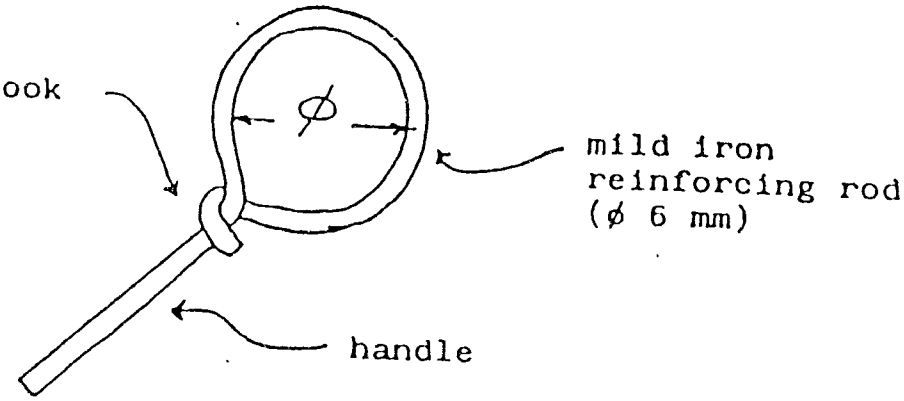
Payment of workers (if they are compensated by quantity of stone gathered) is easily calculated, provided the quantity, that is to say, the volume of each pile is known.

Dimensions of the Stones

The stones which will be placed next to the walls of the gabion must be at least one-and-a-half times larger than the largest size of the wire mesh.

Examples:	mesh	stones
	80/110	165 mm
	100/120	180 mm

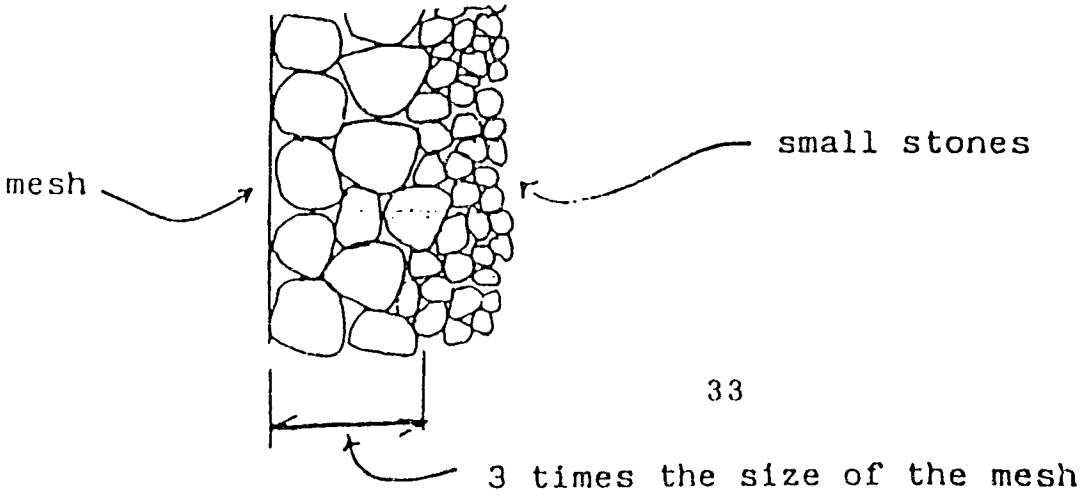
The size of the stones can be checked using an iron ring (a gauge made from a mild iron reinforcing rod), the diameter of which is 1.5 times larger than the size of the wire mesh.



Checking is usually carried out by simply judging the size of stones visually. But, to avoid any dispute from workers when it comes to payment at the piecework rate for the piles of stones, it is advisable to have an iron ring gauge on had check of sizes chosen.

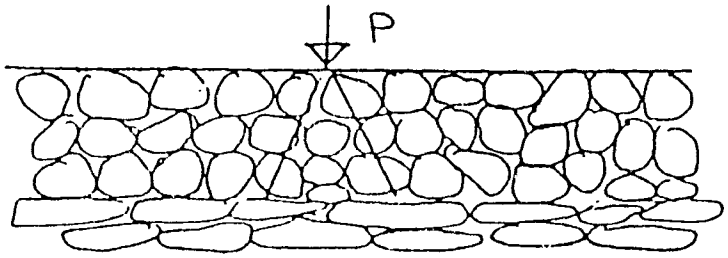
Smaller-sized stones may also be used, provided they are not placed next to the mesh but are put at a distance at least three times the size of the wire mesh.

However, on no account must small stones of less than 8 cm diameter (using an iron ring gauge to check) be used to fill the gabion, not even if they are placed well inside.



Flat Stones

Where structures have to be withstand considerable stress, care should be taken to avoid, as far as possible, placing flat stones outermost in the gabion, since flat stones are less resistant. For this reason, they should be placed beneath a layer of rounded stones which will spread the load.

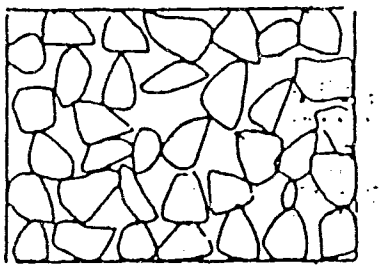


Pebbles

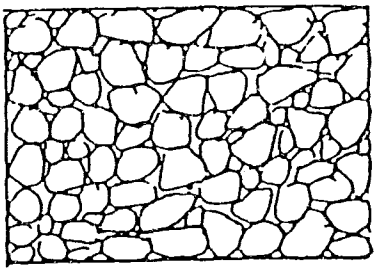
Pebbles are particularly suitable for filling footing gabions, since they increase the flexibility of the gabion and thus its ability to mould itself to the irregularities of the ground. It must be kept in mind, however, that the use of rounded pebbles increases the number of horizontal and vertical cross ties required.

Packing the gabion

Stones must be packed inside the gabion in such a way as to leave the least possible space. The worker should strive to pack the stones so that they touch as much as possible.



WRONGLY FILLED GABION



CORRECTLY FILLED GABION

From time to time during filling, it may be helpful to use a hand rammer to pack the stones down, taking care not to break the stones or deform the gabion walls.

PART II

TYPICAL GABION RETAINING WALL/CHECK DAM ANALYSIS AND DESIGN

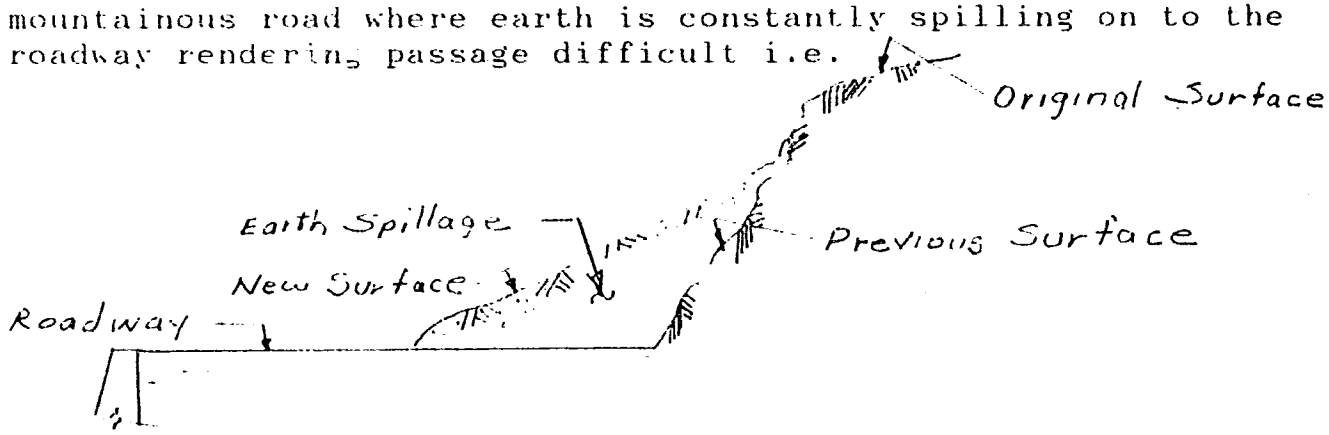
Typical Gabion Retaining Wall/Check Dam Analysis & Design

Gabion retaining walls and/or check dams, although they are relatively flexible structures, are generally analyzed &/or designed in the same way as any mass gravity, rigid structure of reinforced concrete or masonry. The fundamental characteristics of the gabion i.e. its flexibility and its ability to re-distribute forces and pressures, are generally not taken into account. In this sense, therefore, the gabion wall if designed properly (because of its in inherent flexibility and adoptability) can be considered relatively more stable than an equivalent gravity structure design accomplished utilizing masonry or concrete.

With the above in mind we will proceed to analyze & design a typical gabion check dam. The analysis and/or design of a gabion retaining wall would be accomplished in more or less exactly the same manner.

Check Dam Design

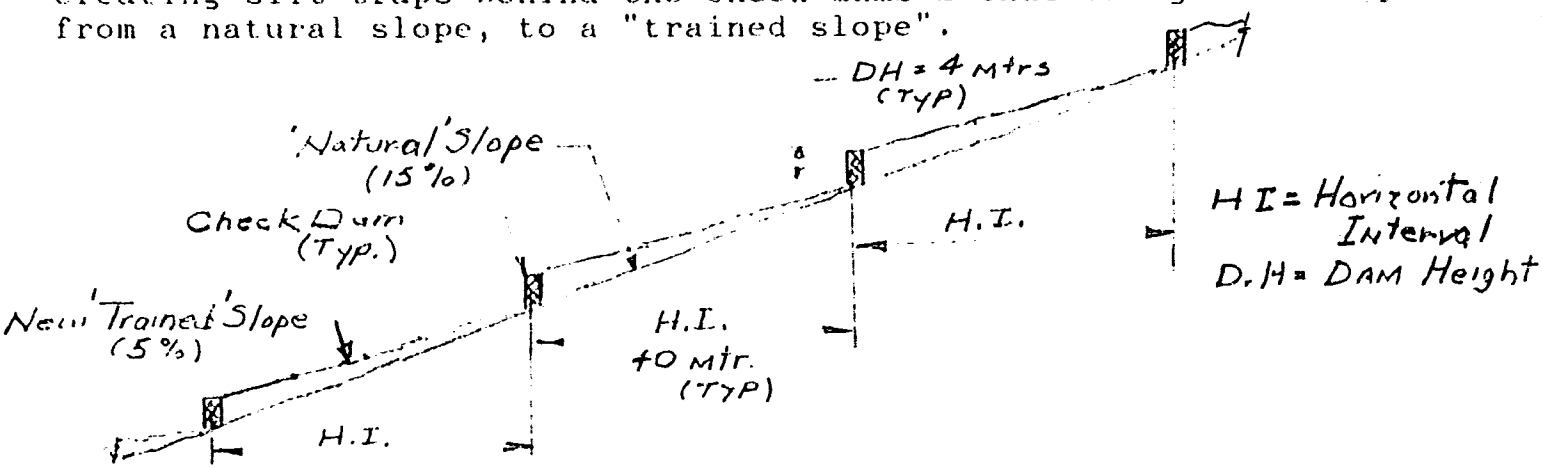
Normally when a design is begun the first step is to establish the design criteria because in the practical world, as opposed to the 'academic', a design is begun without any 'givens'. For example, it is determined that a retaining wall is required at a mountainous road where earth is constantly spilling on to the roadway rendering passage difficult i.e.



Where do we begin with the design? How high should the wall be? What type of material do we use for the wall (i.e. concrete, masonry, gabions)? What design conditions/criteria do we need to know? How do we determine the criteria? What is the ground water level? Is there flooding from the mountain? And if so how often? Are we in an earthquake zone? What are the soil characteristics? Where is the frost level? What strength of concrete do we use? What reinforcing steel do we use? Where do we purchase the steel and how do we insure uniform quality? And so on.

For a check dam (which is essentially a dam placed in a gully or stream to control the flow of water) the decisions are a bit easier. Generally a check dam is constructed of masonry or gabions, very seldom, except in the case of large structures is reinforced concrete used.

To illustrate the design of a typical gabion structure we'll take a practical problem; an actual problem encountered in the field - the design of check dams to control and check the flow of flood waters in a gully and to reduce the steep slope of the gully by creating silt traps behind the check dams & thus change the slope from a natural slope, to a "trained slope".



Assume it is necessary to "train" a stream or gully bed slope from 15% to say, 5% - using rule of thumb estimates (see Appendix). The dam height is dependent upon the dam intervals - say we keep the dam intervals to 40 meters then, using rule of thumb formula:

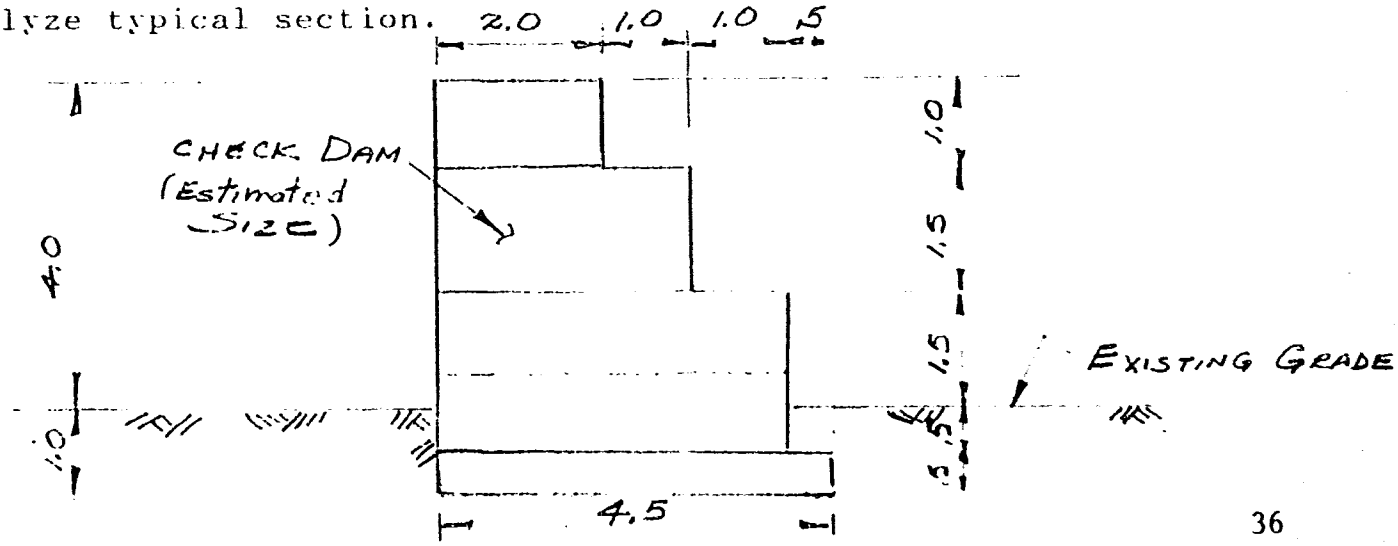
$$D.H. = (s - S_t)\ H.I$$

Where D.H = Dam Height

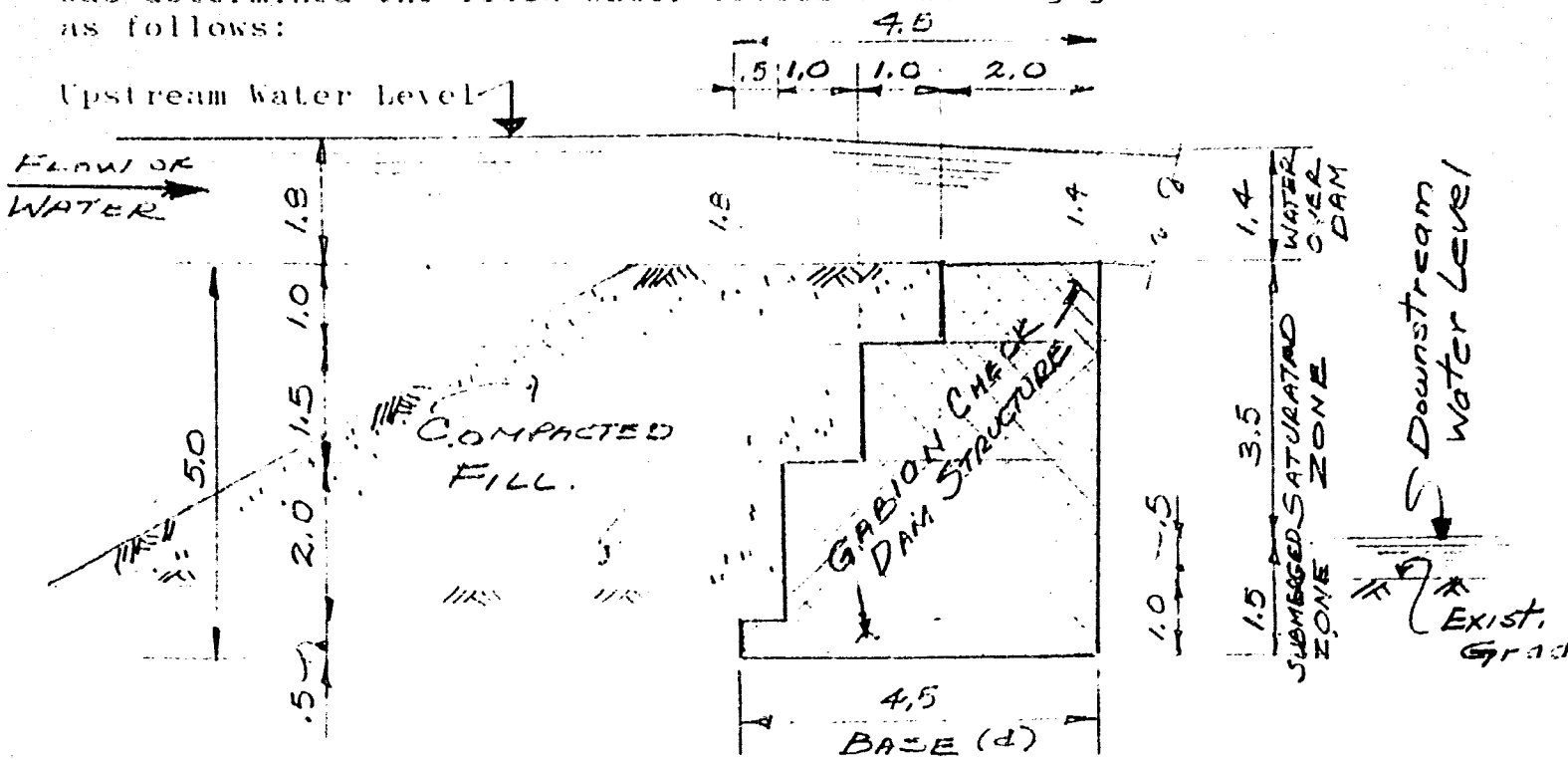
S = "Natural" Slope
St= "Trained" Slope

$$DH = (15 - .05)\ 40M = 4 \text{ meters high}$$

Now design a 4 meter high gabion check dam across stream/gully. Start with 'rule of thumb' sizes (modified) as shown in Appendix & analyze typical section.



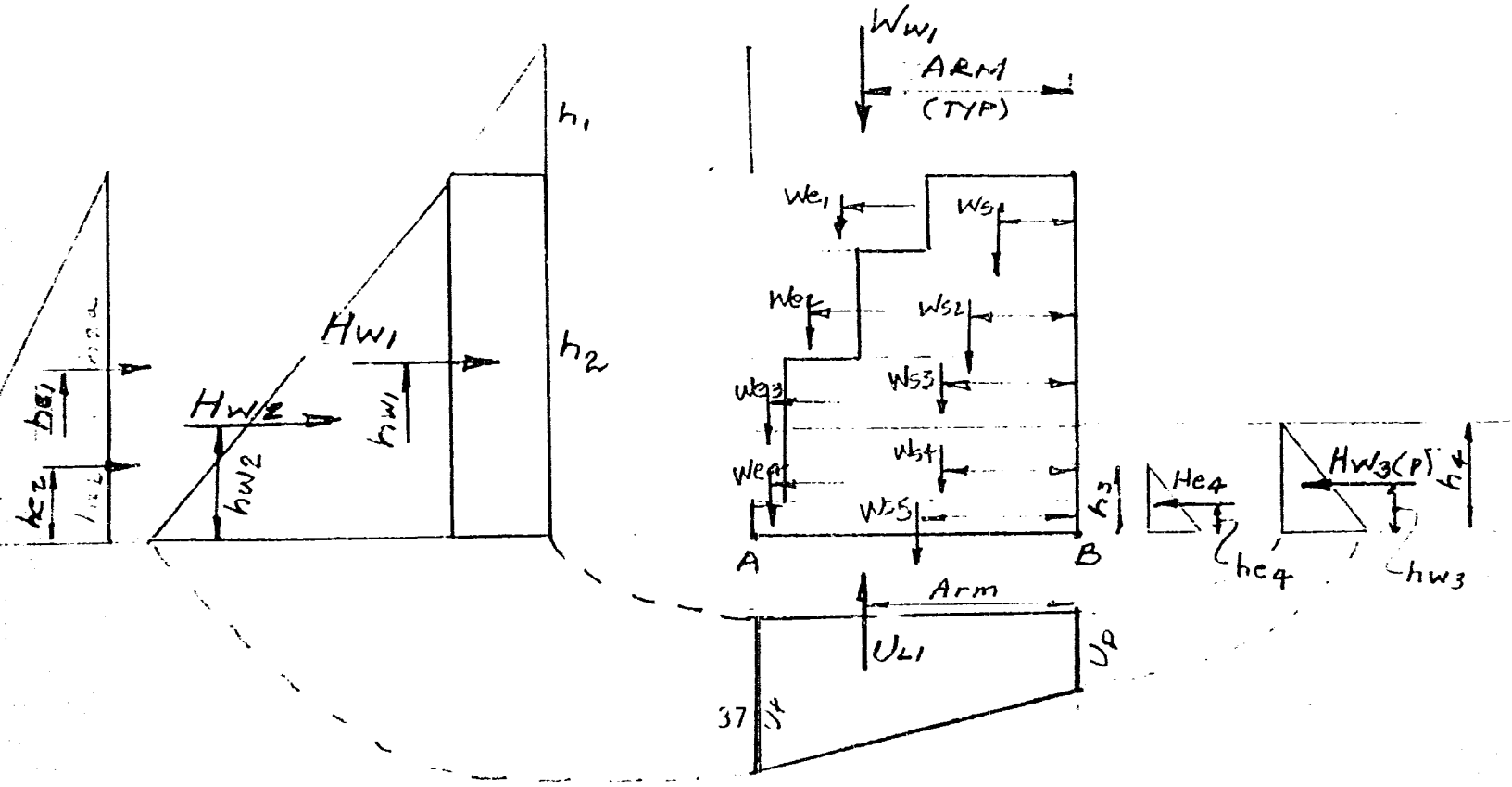
Based upon flood & drainage calculations and a field survey it was determined the flood water levels & existing grade levels are as follows:



Compacted fill is placed in front of the check dam as a precaution to prevent heavy flood waters from undermining the dam.

Force Diagram

Force diagram for the above configuration is as follows:



Design Criteria & Weight of Material

Weight of Various Materials

Masonry Wall (W/Mortar):

$\delta st. = 2.6\ T/M^3$ Dry
$\delta st. = 2.9\ T/M^3$ Saturated
$\delta gt. = 1.6\ T/M^3$ Submerged

Gabion Wall (W/O Mortar):

$\delta g = 1.82\ T/M^3$ Dry
$\delta g = 2.1\ T/M^3$ Saturated
$\delta g = 1.1\ T/M^3$ Submerged

Dry $\delta g = \delta st\ (1 - n)$ where $n = .3$
or $\delta g = \delta st \times .7$
Sat. $\delta g = [\delta s(1-n) + nu\ \delta w$
$.7\delta s + .3 \times 1.0$
$1.82 + .3 = 2.12$
Sub. $\delta g = (\delta s - \delta w)\ (1 - n)$
$= (2.6 - 1.0)\ (.7) = 1.1$

Where n=porosity
(Assume 30% voids)
u=degree of saturation(say 100%)
δw = Unit Wgt. of water

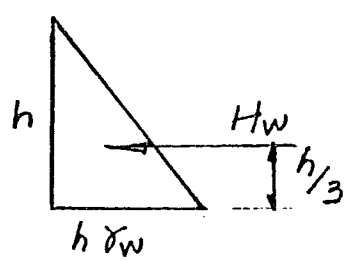
Earth (Assume Ungraded)
$\delta e = 1.68\ T/M^3$ Dry
$\delta e = 2.02\ T/M^3$ Sat.
$\delta e = .91\ T/M^3$ Sub.

δw varies betwn. 1.0 & $1.2 T/M^3$
(Use Avg. $1.0\ T/M^3$
Assume not turbid)

Horizonal Forces

Water Pressure

Use General Formula. $Hw = \frac{1}{2}\ \delta w h^2$



Uplift Pressure on Gabions

Assume hydrostatic pressure on upstream & downstream faces act at base as shown in Force Diagram, resulting in a trapezoidal distribution.

Earth Pressure

Use either of two classic theories, the trial wedge method first developed by the French Engineer, C.A Coulomb in 1776 or that developed in 1857 by the Englishman, W.I. Rankine, for horizontal earth pressure, in cohesionless soil, based upon the general expression;

$$P = \tfrac{1}{2}\, wh^2\, K_A$$

Values of K_A:

Per Coulomb

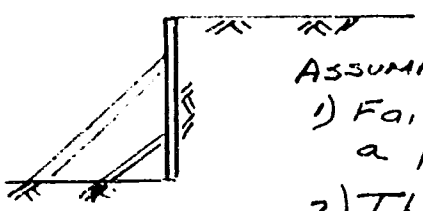
$$K_A = \frac{\cos \Phi}{[1 + (2 \sin^2 \Phi - 2 \sin\Phi \,.\, \cos\Phi \,.\, \tan \delta)^{\frac{1}{2}}]^2}$$

Per Rankine

$$K_A = \frac{(1 - \sin \Phi)}{(1 + \sin \Phi)}$$

Where Φ = Angle of repose of soil (assume equal to internal friction angle).

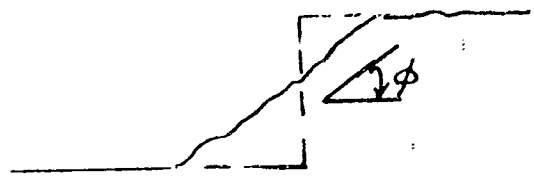
Under Coulomb Theory (which we'll use) it is assumed:

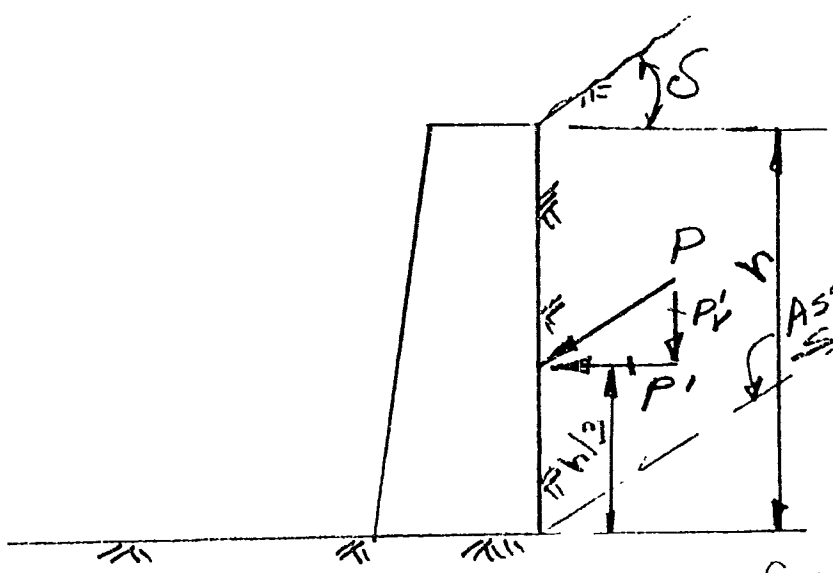


ASSUMPTIONS:

1) Failure surface is a plane.

2) Thrust acts in a known direction





Assume $P' = P$ (Conservative)

If $\delta = 0$

Fill is Horizontal

Then $\tan \delta = 0$

If $\tan \delta = 0$, K_A becomes: (per Coulomb)

$$K_A = \frac{\cos \phi}{[1 + (2 \sin^2 \phi)^{1/2}]^2}$$

or

$$\boxed{K_A = \frac{\cos \phi}{[1 + \sqrt{2} \sin \phi]^2}}$$

Use This Equation

where h = height of Wall

W = Unit weight of earth

P earth for passive pressure (in cohesionless soil)

Per Coulomb

$$K_p = \frac{\cos \Phi}{[1 - (2\sin^2 \Phi + 2\sin^2 \Phi . \cos \Phi . \tan \delta)^{\frac{1}{2}}]^2}$$

or where $\delta = \Phi$

Per Rankine

$$K_p = \frac{1 + \sin \Phi}{1 - \sin \Phi}$$

Use this equation

$$K_p = \frac{\cos \Phi}{[1 - \sqrt{2} \sin \Phi]^2}$$

Design Factors Assumed

Angle of repose (internal friction) = 30^0

Friction factor (f)

Gabion on coarse sand/gravel
Soil f = .60

Gabion on Rock f = .70

Allowable Bearing Pressure

On Hard Sound Rock Use 300 Tons/M^2

On Boulders/Gravel Use 70 ton/m
(Hardpan)

On Earth
Compact/Gravel/Sand
25 to 30 ton/M^2 } use 28 tons/M^2
Med. to hard coarse Sand

On Compacted Fill Use 15 tons/M^2
Med. to stiff inorganic clay
or sandy clay stiff.

Factors of Safety

(Assume No Earthquake)

Overturning

For small structures (hgt. to 2.5 M above grade)
use F.S = 1.2
(with all forces acting including uplift & passive pressures)

For larger structures (hgt. above 3.0 M above grade)

use F.S = 1.5

Sliding

For small structures (hgt. to 2.5 mts)
Use F.S. = 1.4

For larger structures (hgt. over 3.0m)
Use F.S = 1.5
(Absolute Min. - 1.4 use if necessary)

Vertical Forces - Net Stabilizing Moments
(See Force Diagram, Pg. 37)

Sour--ce	Force Symbol Ltr.Ref.	Dimensions Meters	Area M2	Unit δ T/M³	Total δ TONS	Arm Meter	Moment(abt Pt.B) Stabil. + Over Turning − Ton Meters	Remarks
Water	W_{w1}	$\frac{1.8+1.1}{2} \times 4.5$	7.2	1.0	7.2	2.34	+ 16.85	
Uplt. Water	U_{x1} U_x	$wh_1 = 1.0 \times 1.8 +$ wh_2 1.0×5.0						*Total Force = 16.69T (Assume abt. 22% of total force acts)
	U_p	$wh_4 = 1.0 \times 1.5$ $u_p = 1.5^T$ $\frac{(6.8 + 1.5)}{2}$	-	1.0	*4.15	2.73	- 11.33	
Earth	W_{e1}	2.5 x 1.0	2.5	2.02	5.1	3.25	+ 16.58	
	W_{e2}	1.5 x 1.5	2.25	2.02	4.55	3.75	+ 17.06	
	W_{e3}	.5 x 1.0	.5	2.02	1.01	4.25	+ 4.29	
	W_{e4}	.5 x 1.0	.5	.91	.46	4.25	+ 1.93	
Structure (Gbn)	W_{s1}	2.0 x 1.0	2.0	2.1	4.2	1.0	+ 4.2	
	W_{s2}	3.0 x 1.5	4.5	2.1	9.45	1.5	+14.18	
	W_{s3}	4.0 x 1.0	4.0	2.1	8.4	2.0	+16.80	
	W_{s4}	4.0 x 1.0	4.0	1.1	4.4	2.0	+ 8.80	
	W_{s5}	4.5 x .5	2.25	1.1	2.48	2.25	+ 5.58	
Total Wgt. (ΣW_{RF}) (Resultant Force)					+43.10T			
Net Stabilizing Mo (ΣMstab)							+ 94.94 T-M	

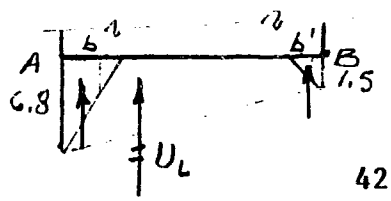
Resultant Force Location (From Pt.B)

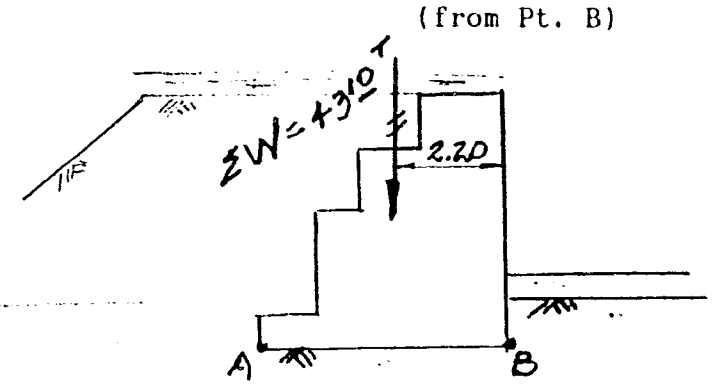
$$\frac{\Sigma M_{stabilizing}}{\Sigma W_{resultant}} = \frac{94.94}{43.10} = 2.20 \text{ Mtrs}$$

(from Pt. B)

* i.e. $\frac{6.8 + 1.5}{2} \times 4.5 = 18.68^T$ (too hi)

Assume 78% loss thru gabions uplift pressure dissipates rapidly thru gabions - similar to sketch below:





Horizontal Forces - Net Overturning Moments
(See Force Diagram, Pg. 37)

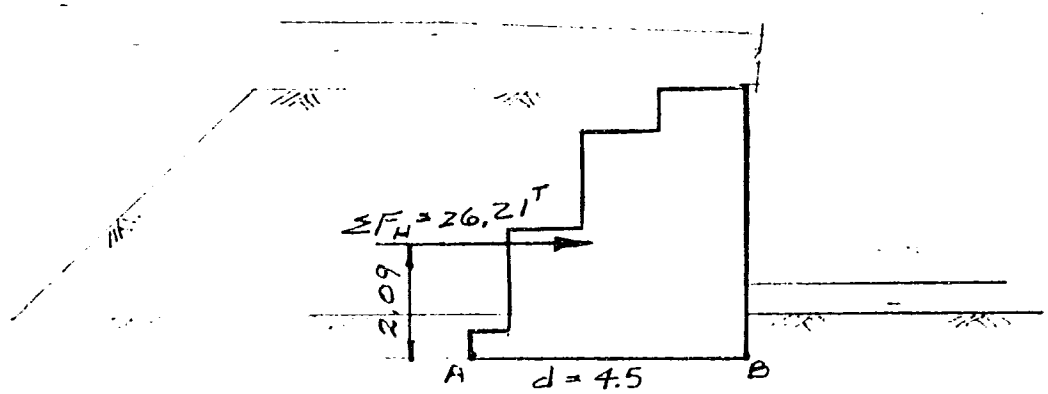
Sour--ce	Force Desig-nation Symbol	Formula Horiz. Force	Active or Passive Cost	Heights h	Unit Wgt δ	Horiz. Force --> + <-- -	Arm	Over turning Moments +	Over turning Moments -	Remarks
			KA/KP	METERS	TONS/M³	TONS	MTRS	TON - MTRS		
Water	H_{w1}	$\gamma_w h_1 \times h_2$	--	$h_1=1.8$ $h_2=5.0$	1.0	+9.00	2.5	+22.50		
	H_{w2}	$.5\ \gamma_w h^2$	--	$h_2=5.0$	1.0	+12.50	1.67	+20.88		
	H_{w3}	$.5\ \gamma_w h^2$	--	$h_4=1.5$	1.0	-1.125	.50	- .56		
Earth	H_{e1}	$.5\gamma_e h_{2a}^2 \cdot K_A$	K_A .30	$h_{2a}=3.5$	2.02	+ 3.71	2.67	+ 9.91		
	H_{e2}	$\gamma_e h_{2a} h_{2b} K_A$	K_A .30	$h_{2b}=1.5$	2.02	+ 3.18	.75	+ 2.39		
	H_{e3}	$.5\ \gamma_e h_{2b}^2 K_A$	K_A .30	$h_{2b}=1.5$	.91	+ .31	.50	+ .16		
	H_{e4}	$.5\ \gamma_e h_4^2 K_P$	K_P 3.0	$h_3=1.0$	.91	-1.37	.33	- .45		
	Net Total Horiz. Force					+26.21				
	Net Total Overturning Moment							+ 54.83 T-M		

Summary

ΣF_H [Net Horiz. Force] (-->) = 26.21T (Driving Force)
ΣM_{OT} [Net Overturn. Mo] ( ) = 54.83 T-M

Resultant Force Location (From Line A-B)

$$\text{Arm} = \frac{\Sigma M_{OT}}{\Sigma F_H} = \frac{54.83}{26.21} = 2.09 \text{ Mtrs.}$$



Factors of Safety

* **Overturning**

$$F.S. = \frac{\text{Stabilizing Moment}}{\text{Overturning Moment}} = \frac{\Sigma Mstab.}{\Sigma M o.t.}$$

$$F.S. = \frac{94.94}{54.83} = 1.73 > 1.5$$

More than allowable. Therefore OK for overturning (will not overturn)

* **Sliding**

$$F.S. = \frac{\text{Resisting Force}}{\text{Driving Force}} =$$

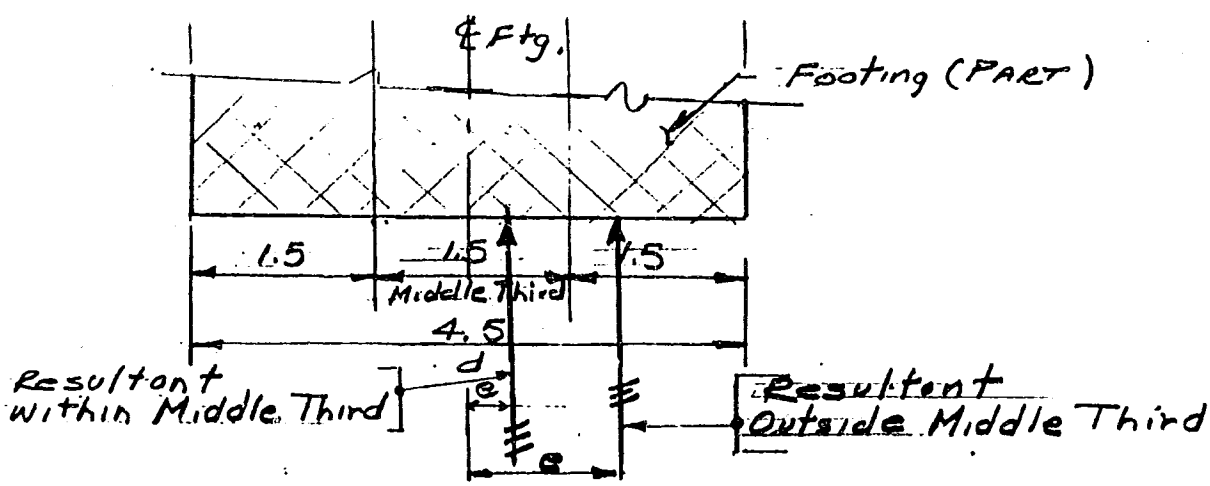
Resisting force = $\Sigma W \cdot f$
ΣW=Net total Wgt. f= coef. of friction/use .60
therefore resistant F= 43.10 x .60 = 25.86 T
Driving Force = 24.93T

$$F.S. = \frac{43.10 \times .6}{26.21} = .99 < 1.5 \text{ Allowable}$$

No good, less than allowable - must increase size and weight of wall or place wall lower into ground to develop passive resistance to sliding. Can reduce **F.S. to 1.4 absolute minimum**.

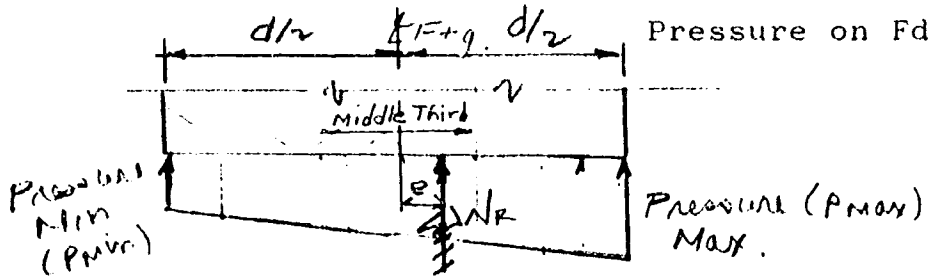
* Bearing Values.

First determine if resultant force is within middle third of footing i.e.



If resultant within middle third then pressures is trapezoidal & can be obtained as follows.

Use Standard Column/Beam Bending+Vert. Load Equation:



Pressure on Fdn. $= \frac{P}{A} \pm \frac{Mc}{I}$

Where P=W
A= Area Base used for 1 meter deep ftg. i.e. width = 1
C= d/2 ($\frac{1}{2}$ depth of footing)

M=We

$$\text{Press} = \frac{W_R}{d} \pm \frac{We.\ d/2}{1/12\ bd^3}$$

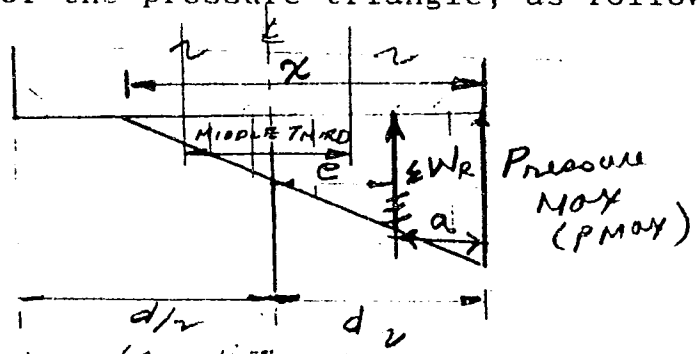
$$I = \frac{bd^3}{12} \quad \text{(Moment of Inertia)} \qquad \text{Press} = \frac{W_R}{d} \pm \frac{6\ We}{d^2} \text{ or}$$

<u>Final Equation</u>

$$\text{Max Pres.} = \frac{W_R}{d}\left(1 + \frac{6e}{d}\right)$$

$$\text{Min Pres.} = \frac{W_R}{d}\left(1 - \frac{6e}{d}\right)$$

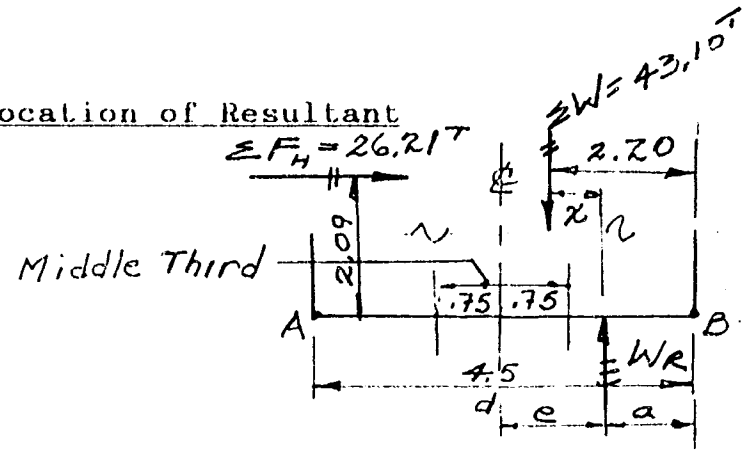
If resultant outside of middle third then pressure is triangular where the triangle area is equal to the resultant load W, located at the centroid of the pressure triangle, as follows:



W=.5 (x) pm (Area of Triangle)

<u>Then</u> $$Pm = \frac{W}{.5(x)} = \frac{2W}{3a}$$ Where x = 3a

Check Location of Resultant



Take Moments About Wr (i.e. clockwise + counter clockwise = 0)

$$(+\ 26.21 \times 2.09\ ) + (-43.10\ x) = 0$$

$$54.78 = 43.10\ x$$

$$x = \frac{54.78}{43.10} = 1.27\ M$$

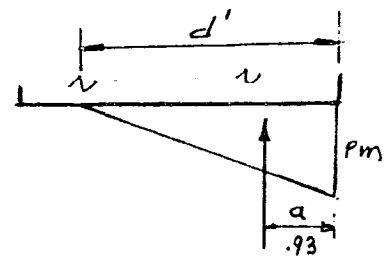
Therefore e (dist of Wr from center line)

$$e = \frac{4.5}{2} - a \quad \text{but } a = 2.20 - x$$

$$a = 2.20 - 1.27 = .93$$

$$e = \frac{4.5}{2} - .93 = 1.32M$$

Therefore e outside middle third. Use triangular distribution to obtain maximum pressure.



$$Pm = \frac{2\ W}{3x\ .93} = \frac{2x43.10}{2.79}$$

$Pm = 30.90\ T/M^2 > 28\ T/M^2$ No good

$d^1 = 3a$
$= .93x3 =$
$d^1 = 2.79$

Conclusion

Design inadequate - bearing pressure overstresses supporting soil plus factor of safety against sliding too low; only factor of safety against overturning adequate - must redesign wall by increasing height of wall & bearing area.

Assignment/Home Work

Complete the previous inadequate design by making another design that is adequate. Use the same criteria as shown in the previous calculations and redesign the structure so that it fits the following criteria. The final design should not be overly conservative, but it must meet these requirements:

* Overturning Factor of Safety
  No less than 1.5

* Sliding Factor of Safety
  No less than 1.4 (where f = .60)

* Bearing Values
  Maximum no more than 28 tons/M^2

Show all calculations as well as a final sketch of the completed structure - all other assumptions are the same as the first design presented (and included here) in lecture. Gabions should be in .5 meter segments i.e. no dimensions such as of 1.23 should be used.

Completed design papers are to be turned in to CCSC Engineering Department no later than 12 noon, 11 December 1991.

Further Home Work

1) Change the upstream & downstream water levels to upstream - 1.9 M downstream - 1.6 M

   For the sake of simplicity use exact same "uplift" pressures as in previous design; present this work on 11 Dec. 1991 (All other forces change according to new dimensions)

2) Also check your design, assuming there is earth pressure but no water pressure on upstream or downstream forces, check for bearing pressure, sliding and over turning stability for Part 1) design.

   If wall is over designed without water pressure show the size the wall should be, to meet all requirements i.e. overturning sliding & bearing, when the structure allows water to totally drain thru or proper drains are placed so that there is no hydrostatic pressure on dam. (i.e. redesign wall w/o water pressure).

   This redesign can be presented on Saturday, 14, December 1991.

APPENDICES

A) Rule of Thumb Procedures

B) Sketches - Dock Side Gravity Retaining Wall

C) Gabion Literature

D) Group Design Calculations

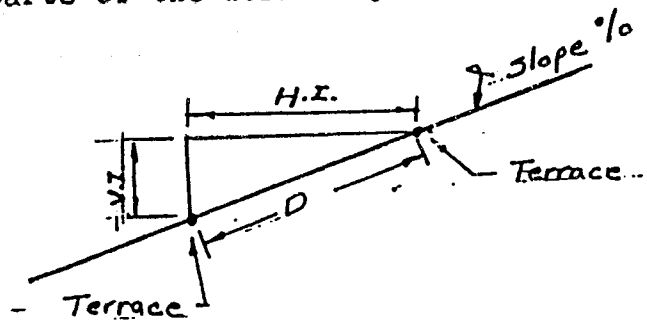
Appendix A

Rule of Thumb Procedures

RULE OF THUMB PROCEDURES

Spacing Between Terraces

Any variation of a number of formulas used in various parts of the world may be used.



Where:

H.I. = Horizontal Interval
V.I. = Vertical Interval
D = Distance Along Ground
S = Slope (%)

Formulas:

				V.I. In
Method	#1	V.I. = .6 + S%/13		Meter
US Soil Con.	#2	V.I. = aS + b	a=.3 & b = 1.0	Feet
Zimbabwe	#3	V.I. = + f/2	f = 5 to 6 Say 3	Feet
South Africa	#4	V.I. = S/a + b	a = 2.5 & b = 1.0	Feet
Algeria	#5	V.I. = S/10 + 2		Meters
Israel	#6	V.I. = XS + y	X = .25 & y = 1.5	Meters
Australia	#7	V.I. = .18(S + X	X = 1.9	Meters
Kenya	#8	V.I. = %S/8 + 1	From 12% to 35%	Feet
		V.I. = %S/4 + 2	From 0% to 12%	Feet

Relation of H.I. to V.I. to D & S:

$S/100 = V.I./H.I.$ ..... $H.I. = V.I. \times 100/S$

$$D = (V.I.^2 + H.I.^2)^{1/2}$$

Example: Distances for Method #1 (Formula Shown Above)

Slope (%)	V.I. (Meters)	H.I. (Meters)
5	.98	19.6
10	1.37	13.7
15	1.75	11.7
20	2.14	10.7
25	2.52	10.1
30	2.91	9.7
35	3.29	9.4
40	3.68	9.2
45	4.06	9.0
50	4.45	8.9
55	4.83	8.8
60	5.22	8.7

RULE OF THUMB PROCEDURES
For Runoff Calculations

Use Variation of Rational Formula

$Q = ACI/360$

Where:

Q=Runoff in Cubic Meters/Second
A=Area of Watershed in Hectares
C=Coefficient of Runoff i.e. ratio of rate of run-off to rate of ra fall (runoff/rainfall)
I=Intensity of rainfall in mm/hour (based on time of concentration

Method

1) Obtain A from Topography or measurement

2) Obtain C-From Table B (attached)

3) Obtain I-For Cape Verdi
   Say 20 Year Rain = 1.5 inches per hour or 38.1 mm/hr
   3a) Obtain T.C. From Tables 7.8 or 7.9 Attached
   3b) Obtain I from Fig. J Attached (in "/hour

4) Calculate $Q = ACI/360$

For Weir Sizes

Use Formula $b = Q/2.0\ h_d^{3/2}$

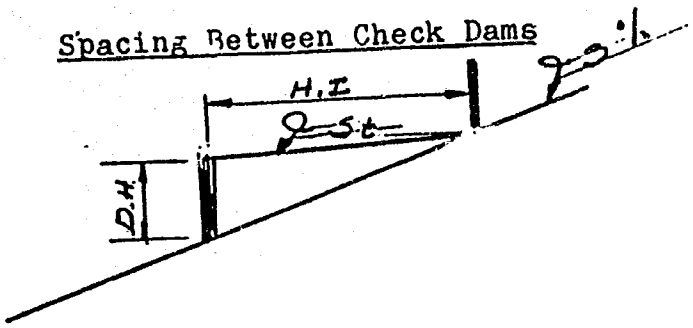
Where:
Q = Runoff in cu.m./sec
b = Width of Weir in meters
h_d= Height of Water over Weir

Assume H_d , Calculate b

or Assume b, Calculate h_d

RULE OF THUMB PROCEDURES

Spacing Between Check Dams



Where:

H.I. = Horizontal Interval
S = Natural Slope %
S_t = Trained Slope %
D.H. = Dam Height

$D.H. = (S - S_t)\ H.I.$

Normal D.H. Varies from 1.5 to 6 Meters

- For Existing Slope to S = 10% Use

  D.H. = .07 H.I.

- For Existing Slope to S= 20% Use

  D.H. = .1 H.I.

- For Existing Slope to S = 30% Use

  D.H. = .15 H.I.

- For Existing Slope to S = 50% Use

  D.H. = .3 H.I.

Number of Dams (N)

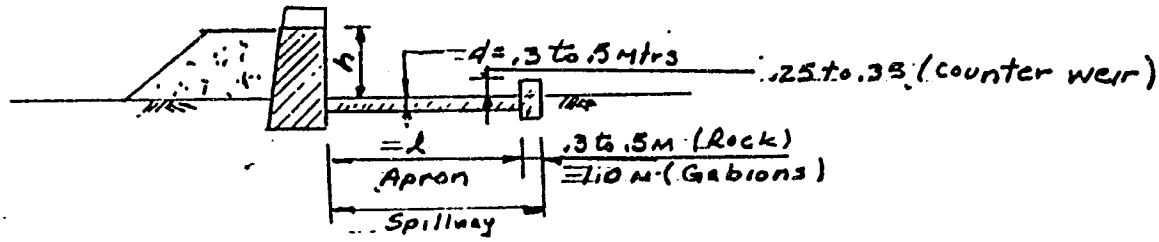
$N = L/H.I.$

Where

L = River Length Meters

RULE OF THUMB PROCEDURES

For Spillway & Apron Sizes



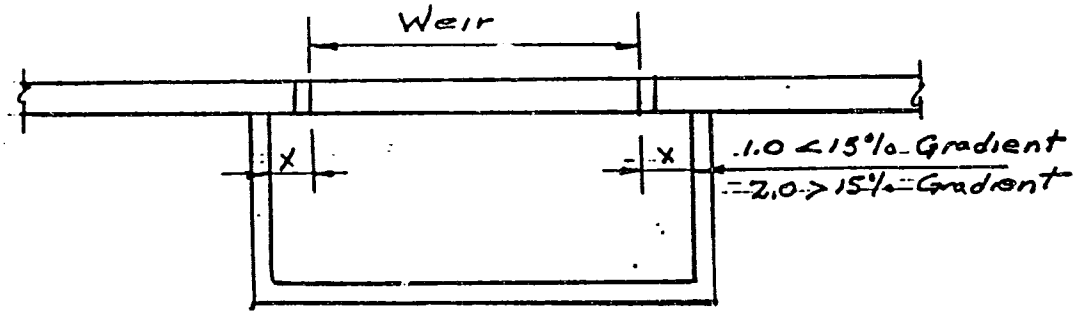
Existing Gradient	0 to 10%	l= 1.5 h
	10 to 15%	L= 1.7 h
	15 to 20%	L= 2.3 h
	20% +	L= 3.0 h

Use Apron at all locations where downstream spillway is on earth.

Omit Apron where downst eam spillway is on rock.

Extend aprons at least 1 meter beyond weir on either side - for light flows i.e. to 15% gradient

For heavier flows (i.e. higher gradients) extend apron two meters, i.e. above 15%.

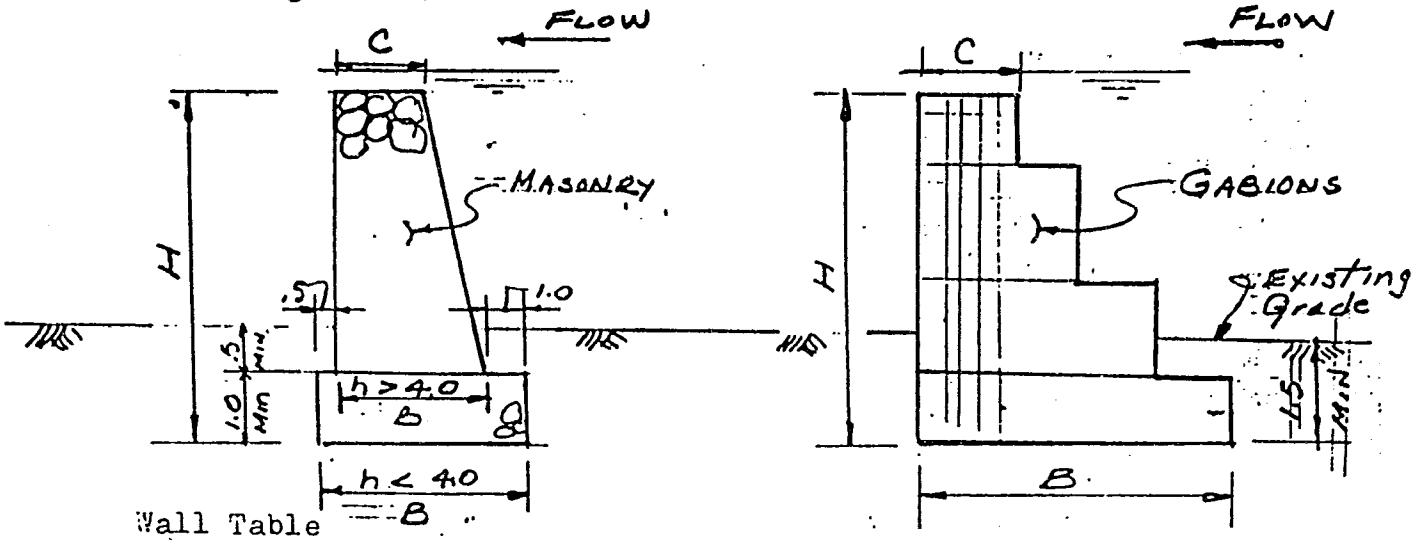


RULE OF THUMB PROCEDURES

For Dam Height & Dimensions

(Assumes file hydrostatic plus portial uplift forces acting on dam)

Wall Diagram



Wall Table

Height Meters (H)	Masonry Dam B	Masonry Dam C	Gabion Dam B	Gabion Dam C
1	1.0	.6	3.0	1.0
2	2.0	1.2	3.5	1.5
3	3.0	1.8	3.5	1.5
4	3.5	2.0	4.0	2.0
5	4.0	2.5	4.5	2.5
6	4.5	3.0	5.5	2.5

Where H is Over 6 meters - Dam dimensions should be calculated.

DRAINAGE — RUNOFF-2

Q=Aci RATIONAL FORMULA (Logical approach).

Q = RUNOFF = Peak discharge of watershed in cubic feet per second (c.f.s.) due to maximum storm assumed. See Fig. A to F, Pg. 5-00 (Usually 10 – 25 years).
A = Area of watershed in acres.
C = Coefficient of runoff, Table B below (Measure of losses due to infiltration, etc.).
i = Intensity of rainfall in inches per hour based on concentration time, See Pg. 5-00.
Concentration time = time required for rain falling at most remote point to reach discharge point. Concentration time may include Overland flow time, Fig. H, Pg. 5-00, and Channel flow time, Pg. 5-04, 5-05, 5-27 and 5-29.

TABLE A-COMPUTATION FORM FOR RATIONAL FORMULA.

LOCATION			A			TIME OF FLOW - MIN.					DESIGN					PROFILE				
STREET	FROM	TO	INCREMENT	TOTAL	C	TO INLET	IN CHANNEL	TIME OF CONC	L *	Q c.f.s	CHANNEL OR PIPE SIZE	SLOPE ft. per ft.	n	CAPACITY FULL c.f.s.	V ft. per sec.	LENGTH ft.	FALL ft.	OTHER LOSSES ft. †	INV. ELEV. UPPER END	INV. ELEV. LOWER END
FIRST ST.	A	B	1.8	1.8	.44	16.5	0.3	16.5	3.8	3.0	15"	.008	.015	4.6	3.9	60	0.48	0	82.00	81.52
MAIN RD.	B	C	1.9	3.7	.50		2.5	16.8	3.7	6.8	D-2	.011	.030	12.0	2.8	420	4.62	0	81.52	76.90
" "	C	D	2.0	5.7	.50		1.8	19.3	3.5	10.0	21"	.007	.015	11.1	4.5	480	3.36	2.20	74.70	70.34

* Note that the sequence of design as in example, Fig. J, Pg. 5-00, involves trial assumptions in determining i.
† Fall in manhole.

TABLE B – VALUES OF $C = \frac{RUNOFF}{RAINFALL}$

SURFACES			VALUE PROPOSED MIN.	VALUE PROPOSED MAX.	VALUE BY OTHER AUTHORITY MIN.	VALUE BY OTHER AUTHORITY MAX.	
ROOFS, slag to metal.			0.90	1.00	0.70	0.95	①
PAVEMENTS	Concrete or Asphalt.		0.90	1.00	0.95	1.00	③
	Bituminous Macadam, open and closed type.		0.70	0.90	0.70	0.90	③
	Gravel, from clean and loose to clayey and compact.		0.25	0.70	0.15	0.30	①
R.R. YARDS			0.10	0.30	0.10	0.30	①
EARTH SURFACES	SAND, from uniform grain size, no fines, to well graded, some clay or silt.	Bare	0.15	0.50	0.01	0.55	④
		Light Vegetation	0.10	0.40	0.01	0.55	④
		Dense Vegetation	0.05	0.30	0.01	0.55	④
	LOAM, from sandy or gravelly to clayey.	Bare	0.20	0.60			
		Light Vegetation	0.10	0.45			
		Dense Vegetation	0.05	0.35			
	GRAVEL, from clean gravel and gravel sand mixtures, no silt or clay to high clay or silt content.	Bare	0.25	0.65			
		Light Vegetation	0.15	0.50			
		Dense Vegetation	0.10	0.40			
	CLAY, from coarse sandy or silty to pure colloidal clays.	Bare	0.30	0.75	0.10	0.70	④
		Light Vegetation	0.20	0.60	0.10	0.70	④
		Dense Vegetation	0.15	0.60	0.10	0.70	④
COMPOSITE AREAS	City, business areas.		0.60	0.75	0.60	0.95	⑤
	City, dense residential areas, varying as to soil and vegetation.		0.50	0.65	0.30	0.60	⑤
	Suburban residential areas, " " "		0.35	0.55	0.25	0.40	⑤
	Rural Districts, " " "		0.10	0.25	0.10	0.25	②
	Parks, Golf Courses, etc., " " "		0.10	0.35	0.05	0.25	①

NOTE: Values of "C" for earth surfaces are further varied by degree of saturation, compaction, surface irregularity and slope, by character of subsoil, and by presence of frost or glazed snow or ice.

① Bryant & Kuichling, Report, Back Bay Sewerage District, Boston, 1909.
② Metcalf and Eddy, American Sewerage Practice, 1928. Mc Graw-Hill.
③ Used by City of Boston, reported by Metcalf & Eddy.
④ Used by City of Detroit, reported by Metcalf & Eddy.
⑤ L.C. Urquhart, Civil Engineering Handbook, 1940. Mc Graw-Hill.

Appendix B

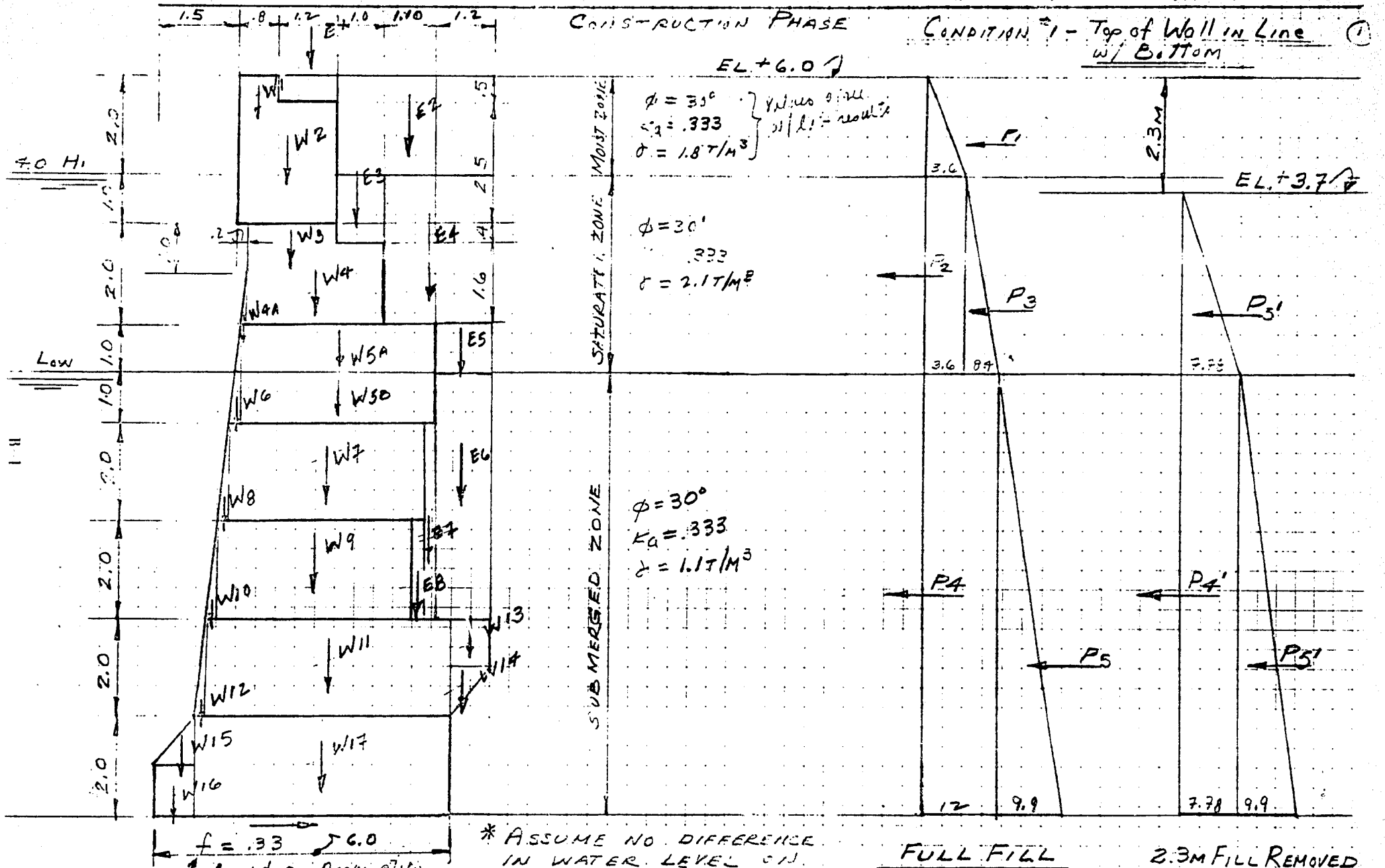
Sketches - Dock Side

Gravity Retaining Wall

STABILITY OF WALL (AS BUILT CONDITION) FULL FILL & WITH 2.3M REMOVED

CONSTRUCTION PHASE

CONDITION #1 - Top of Wall in Line w/ Bottom



* Based on Assumption silt w/ ϕ of 18° filtered thru a reduce of value DM7 Table 10-1

* ASSUME NO DIFFERENCE IN WATER LEVEL ON EITHER SIDE OF WALL - THIS IS CONSISTENT WITH BORING INFORMATION, SHOWS WATER LEVEL ON LAND SIDE VARIES WITH TIDE.

FULL FILL

2.3M FILL REMOVED

PRESSURE DIAGRAMS

Final Design

6/80

As Built Condition Wgts & Stab. Mos

CONCRETE WALL (Full & Part Fill Same)

CONDITION #1
TOP OF WALL IN LINE W/BOTTOM

(2)

FORCE	DIMENSIONS	AREA	UNIT WGT.	WGT.	ARM	STAB. Mo
W_1	.8 x .5 M.	.40 M^2	2.32 T/M^3	.93 T/LM	1.9 M	1.76 M.T.
W_2	2.5 x 2.0	5.0		11.6	2.5	29.0
W_3	1.8 x .4	.72		1.67	2.6	4.34
W_4	2.8 x 1.6	4.48		10.39	3.1	32.21
W_{4A}	.1 x 1.0 x .5	.05		.12	1.67	.20
W_{5A}	4.0 x 1.0	4.0	2.32	9.28	3.6	33.41
W_{5B}	4.0 x 1.0	4.0	1.32	5.28	3.6	19.01
$W_{6A/6D}$	.2 x .2 x .5	.2	1.82	.36	1.53	.56
W_7	4 x 2.0	8.0	1.32	10.56	3.4	35.90
W_8	.2 x .2 x .5	.2		.26	1.33	.35
W_9	4.0 x 2.0	8.0		10.56	3.2	33.79
W_{10}	.2 x .2 x .5	.2		.26	1.13	.29
W_{11}	5.0 x 2.0	10.0		13.20	3.5	46.20
W_{12}	.2 x .2 x .5	.2		.26	.93	.24
W_{13}	.8 x 1.0	.8		1.06	6.4	6.78
W_{14}	.8 x 1.0 x .5	.4		.53	6.27	3.32
W_{15}	.8 x 1.0 x .5	.4		.53	.53	.28
W_{16}	.8 x 1.0	.8		1.06	.40	.42
W_{17}	5.2 x 2.0	10.4	1.32	13.73	3.4	46.68
				$\Sigma W = 91.64$ T/LM		$\Sigma SM_o = 294.74$ M.T.

$$ARM = \frac{\Sigma \text{ STAB. } M_o}{\Sigma \text{ WGT}} = \frac{294.74}{91.64} = 3.22 \text{ Meters}$$

6/83
Final Design

(N)

As Built Condition - Wgts / Stat. Mts

Full Earth Fill

CONDITION #1

FORCE	DIMENSIONS	AREA	UNIT WGT	WGT.	ARM	STAB. Mo
E1	1.2 x .5 M	.6 M²	1.8 T/M³	1.08 T/LM	2.9 M	3.13 M.T.
E2	3.3 x 2.0	6.6	1.8	11.88	5.15	61.18
E3	1.0 x 1.4	1.4	2.1	2.94	4.0	11.76
E4	2.3 x 3.0	6.9	2.1	14.49	5.65	81.87
E5	1.2 x 1.0	1.2	2.1	2.52	6.2	15.62
E6	1.2 x 5.0	6.0	1.1	6.6	6.2	40.92
E7	.2 x 4.0	.8	1.1	.88	5.5	4.84
E8	.2 x 2.0	.4	1.1	.44	5.3	2.33

$\Sigma W_{(E)} = 40.83$ T/LM $\quad \Sigma S.M_o = 221.65$ M.T/LM

$$ARM = \frac{\Sigma \text{ STAB. } M_o}{\Sigma WGT_{(E)}} = \frac{221.65}{40.83} = 5.43 \text{ Metres}$$

WITH 2.3 MTRS. EARTH FILL REMOVED

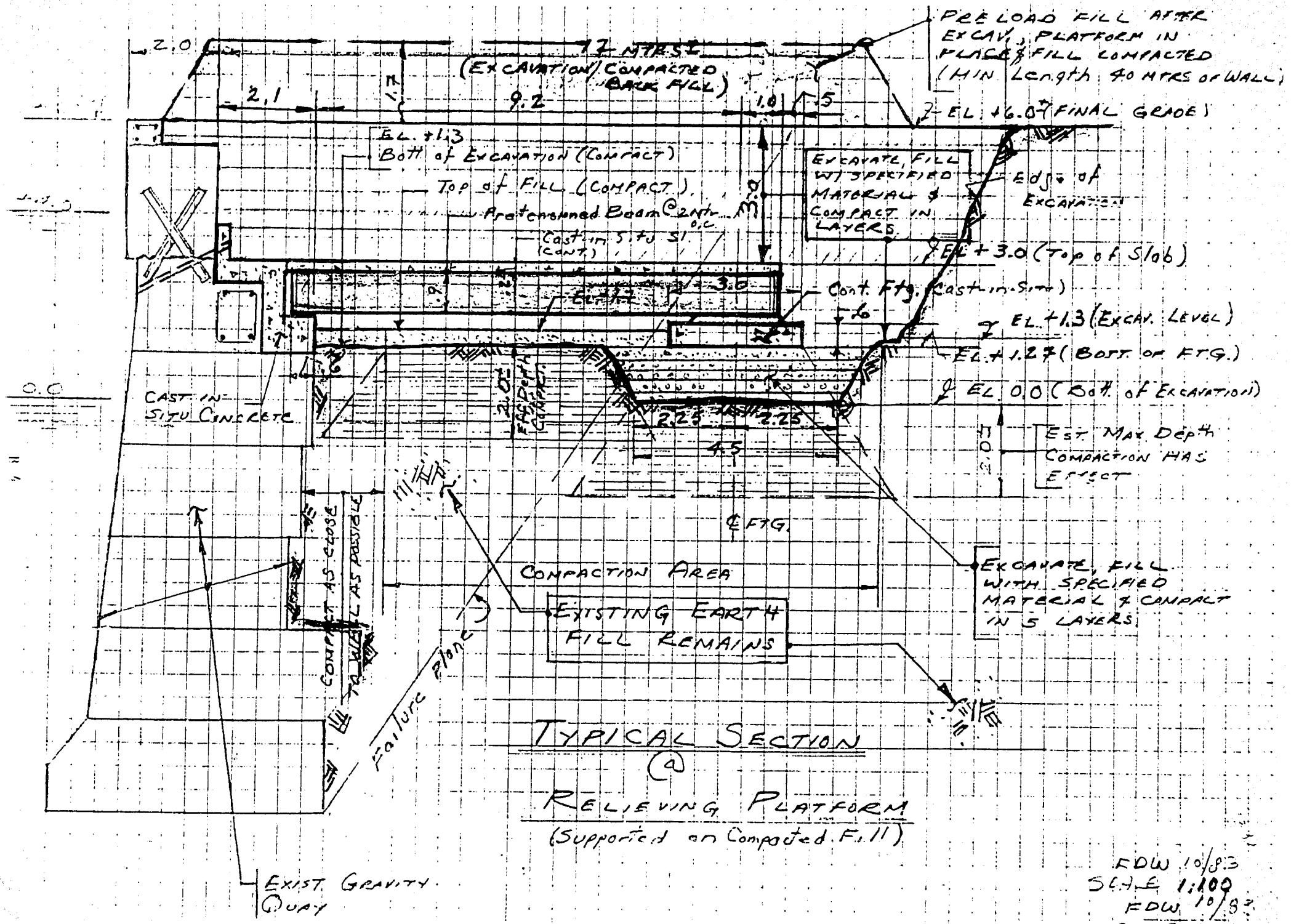
CONDITION #1

E1	No Force/Wgt			—	—	—
E2	No Force/Wgt			—	—	—
E3	1.0 x 1.1	1.1	2.1	2.31	4.0	9.24
E4	2.3 x 2.7	6.21	2.1	13.04	5.65	73.68
E5 Thru E8	same as above			10.44 (TOT. per above)		63.71 (TOTAL per above)

$\Sigma W_{(E)} = 25.79$ T/LM $\quad \Sigma S.M_o = 146.63$ MT/LM

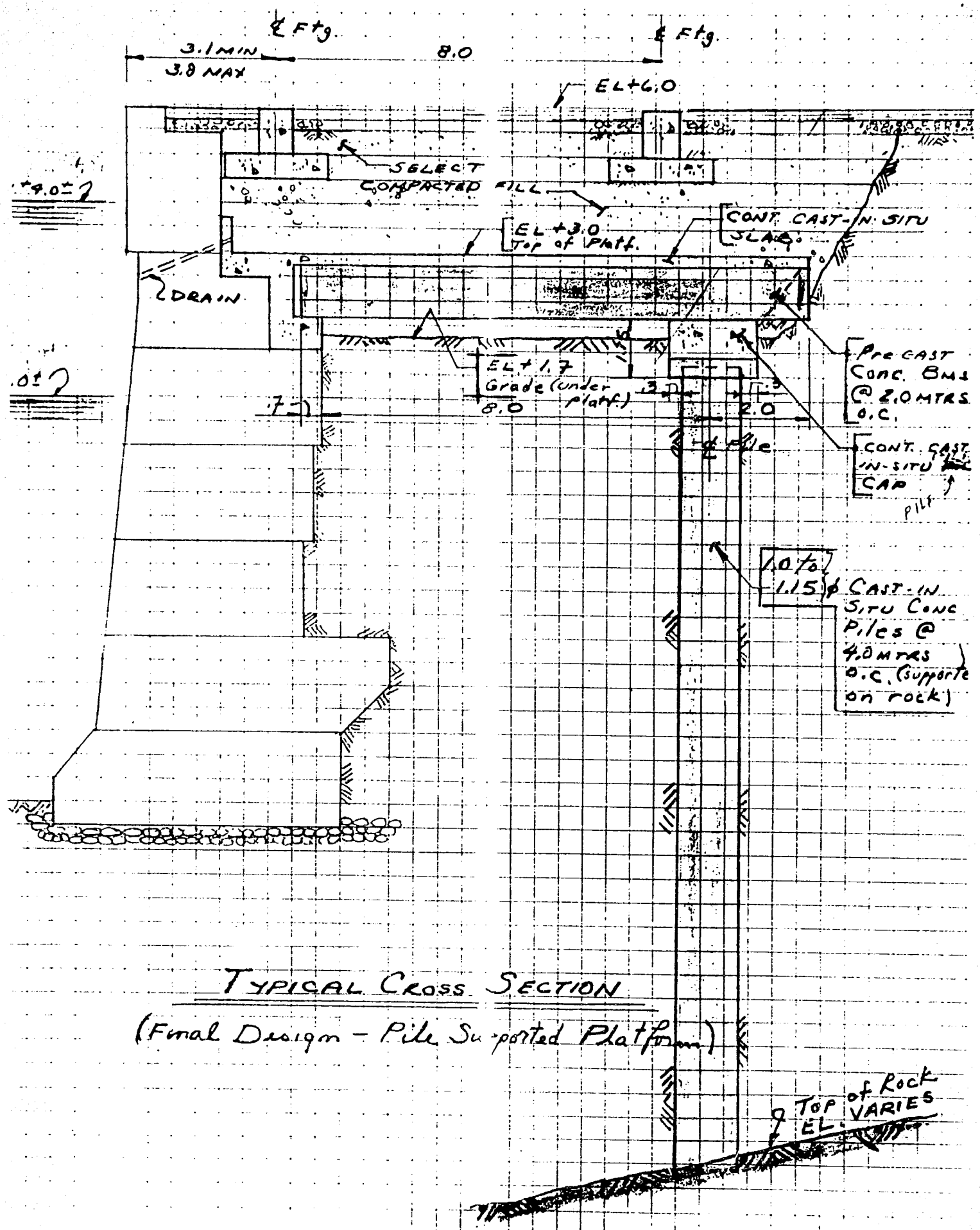
$$ARM = \frac{146.63}{25.79} = 5.69 \text{ Metres}$$

6/83 (W)
Final Design



TYPICAL SECTION @ RELIEVING PLATFORM (Supported on Compacted Fill)

FDW 10/83
SCALE 1:100
FDW 10/83



TYPICAL CROSS SECTION

(Final Design - Pile Supported Platform)

Appendix C

Gabion Literature Prepared
by Officine Maccaferri
Bologna, Italy

Flexible gabion structures in earth retaining works

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Preface

Flexible structures have been successfully used for some time for soil retention and in river works for erosion control, for land reclamation and the stabilization of steep slopes. In the following pages after a brief review of the various types of structures and materials generally used for soil retention, only the Maccaferri gabion structures, their methods of construction and their functional characteristics will be discussed in detail. Other publications in this series focus on the numerous uses of Reno mattress and gabions in water courses and road construction: specifically river training works, linings for channels and canalized water courses, bank protection for navigable waterways, protection of coastal and harbour structures and lining of dams. With the publication of this technical series the Officine Maccaferri S.p.A. propose to illustrate the salient characteristics of the flexible structures built of gabions that constitute the base of its production, and also to advance the knowledge of how these structures perform in their various applications and suggest rational criteria for their selection and design. Officine Maccaferri S.p.A. trust that the manual will assist the engineers and designers interested in the study and design of these structures. It is hoped that it may serve to increase the use of flexible structures which in numerous applications offer advantages, both technical and economical, over other types.

For the help and expert advice they gave in the editing of the manual and for their review of the experimental programme carried out in Bologna, my grateful thanks are due to Prof. Eng. Claudio Ceccoli and Dr. Eng. Pier Paolo Diotallevi of the Institute of Construction Technology, Prof. Eng. Giorgio Folloni and Arch. Giuseppe Lombardini of the Institute of Topography, Geodesy and Mineral Geophysics of the Engineering Department of the University of Bologna.

Bologna, January 1987

dr. eng. Andrea Papetti
GENERAL MANAGER

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Foreword

This manual presents some guidelines, general criteria and new developments useful in the process of selecting, designing and building soil retaining structures using flexible gabions.

To this end the results of recent experiments and research programmes investigating the behaviour and strength characteristics of these structures will be examined.

Officine Maccaferri S.p.A. intend to promote a new and useful contribution to the field of design and construction of retaining walls by bringing the results of the research to the attention of engineers and offering advice on methods of the design and building of gabion works.

For more detailed study of the topics discussed here, the reader should consult authoritative references cited in the bibliography.

The manual also presents detailed examples of calculations to illustrate the principles discussed, as well as an extensive review of earth retaining projects using gabions.

Officine Maccaferri S.p.A., drawing upon the back-ground and experience their engineers have acquired throughout the world, will be happy to offer assistance in the resolution of any particular problems.

CHAPTER I

Retaining structures

1.1 Introduction

In the technical literature a retaining structure is defined as any structure capable of resisting soil pressure.

Planning of these structures must take careful account of many structural and functional considerations:

type of construction and structural characteristics;
- overall shape and dimensions;
- analysis of the static and dynamic forces involved;
- loads transmitted to the foundation.

The function and the selection of structure type is also influenced by environmental factors such as:

- morphology of the soil surface;
- surface water runoff;
- level of the local water table.

In the long run climatic factors have decided effects on the stability of structures and therefore they need to be considered.

The project must, therefore, be examined within its climatic and hydro-geologic setting, keeping in mind the changes that can result to the natural environment.

1.1.1 Types of structures

Structural and functional differences allow retaining structures to be classified into various groups:

- retaining walls
- sheet piling
- wells and caissons
- reinforced soil
- walls tied and anchored to rock
- shoring of excavations.

This classification is related to the characteristics of the structures, their possible movement during and/or after construction and their method of achieving stability.

Gabion retaining walls are the subject of this manual and hence they are treated in detail in the following chapters. For the other types of earth retaining structures the reader is referred to the bibliography [1, 2, 3, 4, 5].

1.2 Retaining walls

1.2.1 Generalities

Retaining walls are permanent structures usually built at the toe of a slope or to contain backfill.

The analysis of the location where the wall is to be placed is important with regard to both permeability and uniform or differential settlement.

It should be remembered that the backfill behind the wall is often imported. This permits a selection of fill material with different characteristics from those of the native soil.

When retaining walls are used to contain earth fill, the embankment must be protected as much as possible from the effect of seasonal weather variations, especially from surface and deep penetration freezing.

Therefore clay backfill and foundations should be avoided because they absorb water, swell up and give rise to unforseen increases in pressure. It is advisable to use non-cohesive materials, preferably coarse grain sand and gravel that permit the flow of water to the lower strata and to existing drains.

If the structure is not self-draining, it is necessary to avoid the accumulation of water behind the wall by providing a well thought out and carefully executed network of drains to reduce hydrostatic pressure and the consequences of frost.

1.2.2 Types and materials

To approach the problem correctly, it is necessary to determine the strength characteristic of the materials so that they may be best utilized in accordance with the installation procedure and requirements dictated by design calculations and assumptions.

To find the most technically and economically appropriate solution and taking the climatic and environmental factors into account, the designer must consider:

- the nature of the construction materials locally available at the most advantageous conditions, (sand, gravel, cement, stone);
- time required for construction (urgency);
- available space at the construction site;
- availability of adequate equipment;
- special circumstances such as existing structures on the site.

Design criteria are dependent on the calculation assumptions and on the technological requirements of the type of construction and materials adopted as well as on the desired safety factors and the allowable stresses of the materials.

The design process may be more or less sophisticated with the possibility of using computer-generated preliminary solutions to simplify and optimize the design.

Since the material and the type of construction can not be separated when classifying retaining walls, it is a good idea to categorize them using a single criterion based on both factors.

With this in mind, the following types can be identified:

1) gravity walls,
2) reinforced concrete walls,
3) walls made of pre-fabricated elements,
4) gabion walls.

For the first three types the reader should consult the authoritative references [1, 2, 3, 4, 5, 6].

1.2.3 Gabion walls

These are "cellular" structures formed of rectangular cages made of zinc coated steel wire mesh, filled with stone of the proper size and mechanical characteristics.

The individual units are firmly tied to each other with zinc coated wire in such a way as to form a monolithic structure (Fig. 1).

The choice of the materials to be used, both for the filling material and for the wire mesh, is fundamental if a truly effective structure is to be obtained.

In particular, the mesh, must satisfy the following requirements:

- high mechanical resistance;
- high resistance to corrosion;
- good deformability;
- will not unravel easily.

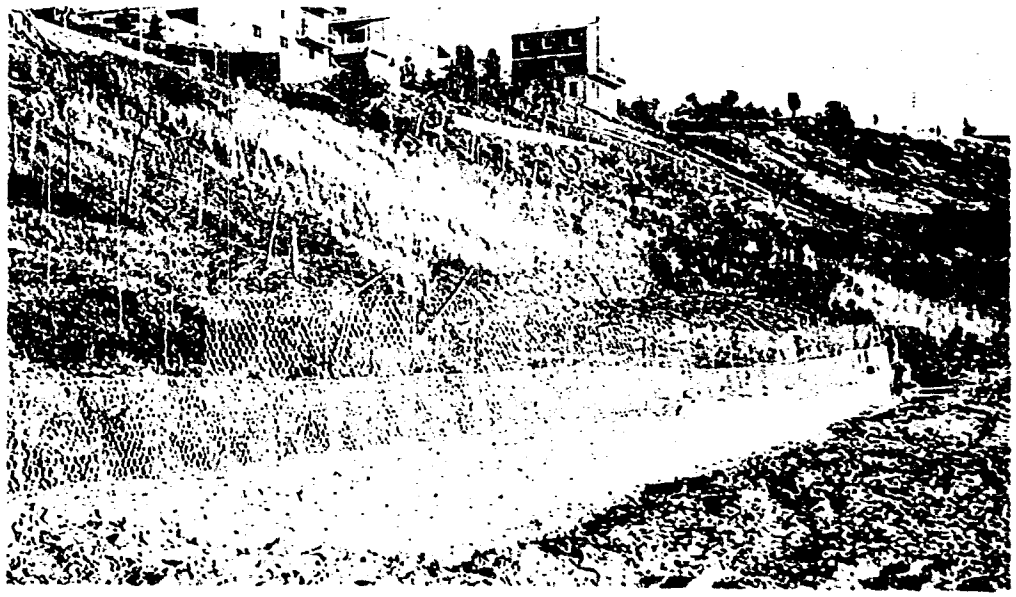
Hexagonal double-twisted wire mesh, either zinc coated to international standards or coated with zinc plus PVC, meets all the requirements better than any other type.

In designing gabion walls it has previously been assumed that they will function as gravity walls without considering the contribution of the mesh which through its resistance to tension provides a further safety factor.

Recently a series of experiments were carried out on prototypes of gabion walls, (see Chap. III). The results have permitted us to determine the static behaviour of these structures and to define their principal characteristics, assuming that the mesh serves to contain the stone filling as an active part of the structure.

These characteristics place gabion walls in a category of their own.

The types of gabion walls, their characteristics and applications are discussed in Chapter II.



1.3 Uses

Retaining walls are designed to provide restraint against the lateral earth thrust. They are used for the containment of fill and in the stabilization of both natural and artificial slopes such as road embankments, river and canal banks and excavated sections.

Both for natural and artificial slopes, knowledge of the environmental conditions such as the distribution of interstitial pressures, the hydrologic cycle, the parameters of earth resistance and its deformation rate permit verification of the stability conditions and the selection of action in case of settlement.

In cases in which the instability of the slope is caused by superficial or deep landslides, possible courses of action fall into two large categories.

The first includes:

a) Measures that preserve the geometry of the slope and therefore protect it from all occurrences, such as erosion, degradation, etc. that tend to modify it. Included among these measures are check dams, longitudinal and transverse bank protection works and various types of slope protection. In the final analysis, these are river training works and slope consolidation projects carried out with the use of various structures [7].

b) Works for stabilizing slopes include gravity, sheet piles, and diaphragm walls. Since rigid walls would require prohibitive dimensions, either flexible structures able to withstand significant deformation or tension rods with anchor plates are generally used [8].

The second category of measures includes:

a) Works to collect surface water from various sources and rapidly dispell it, particularly in an area subject to landslides.

b) Works to collect and dispose of the water present in the soil. These consist in either installing relatively superficial drains such as trenches filled with granular materials and drainage pipes of various types, tunnels, drainage wells and columns; or deep draining walls in which the function of earth support is subordinate to that of intercepting the subterranean water that is too deep to be reached by trenching.

Unstable slopes may also be found in excavations for foundations or road alignment construction. In these cases it is necessary to determine the safe slope based on the characteristics of the embankment and foundation soil.

Cuttings are created by excavating through the existing natural soil whereas in embankments, the soil is imported and stabilized by mechanical compaction. Thus the same soil can present different conditions depending on whether it is in embankments or in cuttings.

To limit the amount of earth works, retaining walls are used to shore the faces of the sides of the excavation. On the other hand a wall supporting a road embankment is a wall supporting fill. The height of this type of wall usually does not reach the level of the road surface but the ground is sloped down at the natural slope from the road to the top of the wall.

Bridge abutments and their wing walls are often retaining walls, built to support the road embankment and to protect its toe from possible erosion that could cause slips and subsequent undermining of the road.

In some cases retaining structures are intended mainly to protect the earth from the erosive effects of the weather and are often called revetments. Here the support function is minimal because the revetment is resting against compacted stable soil and the protective function predominates.

Particular attention must be paid to the drainage of the backfill being reveted to avoid possible hydrostatic uplift pressure caused by ground water.

CHAPTER II

Gabion retaining walls
Technical and construction characteristics

2.1 Introduction

In the planning of all manner of projects, whether they are for the improvement and or protection of water courses, canals, roads or railways in mountainous, forest or urban areas, among all the alternatives of structure types and materials which the designer can choose, the box gabion, in comparison to rigid forms of construction, often offers greater reliability and wider application [9, 10].

In the cases of unconsolidated soils with modest mechanical strength and those subject to settlement, the gabion structure having characteristics of flexibility, permeability and reinforcement is particularly applicable.

A retaining wall built of box gabions is an homogeneous monolithic structure, and is able to function under tension and to absorb unforseen stresses.

With the passing of time, the concept of gabion retaining structures has been accepted and established throughout the world. A testimonial to their ability to meet customers' requirements of durability and resistance to corrosion.

2.2 Description of gabions and characteristics of materials

2.2.1 Box gabions

Gabions are rectangular cages made of hexagonal double-twisted wire mesh filled with appropriately sized cobbles or quarry stone.

The wire used is soft steel, annealed and zinc coated to international standards.

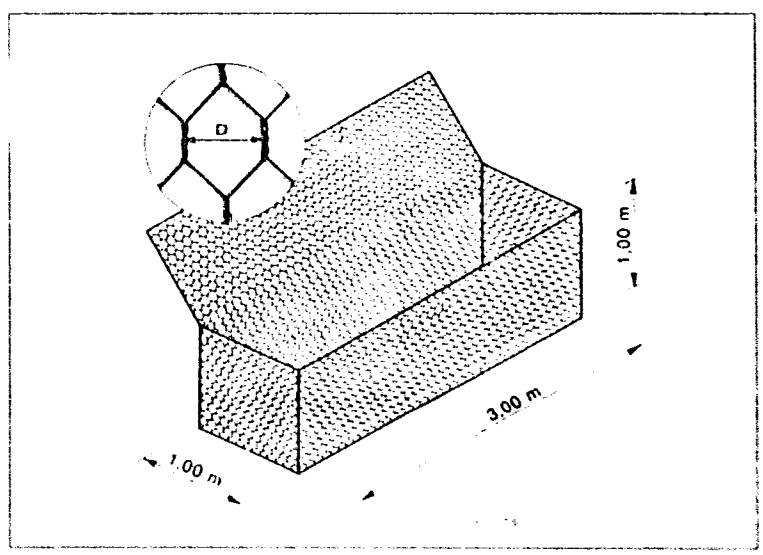
The double twisted hexagonal mesh and high quality zinc coating are determining factors for the good quality of a box gabion. Double twisting ties together the wires that form the mesh and guarantees that the mesh will not unravel should one or more wires break (Fig. 2). Zinc coating provides long-term protection for steel wire against oxidation.

The mechanical and qualitative characteristics of the wire, i.e. breaking strength, elongation and quality of zinc coating, meet the most rigid international standards.

The gabions can be subdivided into cells by inserting diaphragms which are mesh panels with the same characteristics as the external sides, spaced 1 m from each other to strengthen the structure and to facilitate its assembly. The gabions can be 1 to 2 m. wide. In the first case the lid and the base are formed out of a single sheet of mesh, see standard gabion Fig. 3a; in the second case the lid is made from a separate sheet of mesh, see multiple cell gabion, Fig. 3b.

The zinc coated wire, before being woven, can be coated with a special PVC (polyvinylchloride) 0.4 to 0.6 mm thick. This additional coating gives full protection from corrosion in marine or heavily polluted environments.

The dimensions of standard gabions are shown in Table I.



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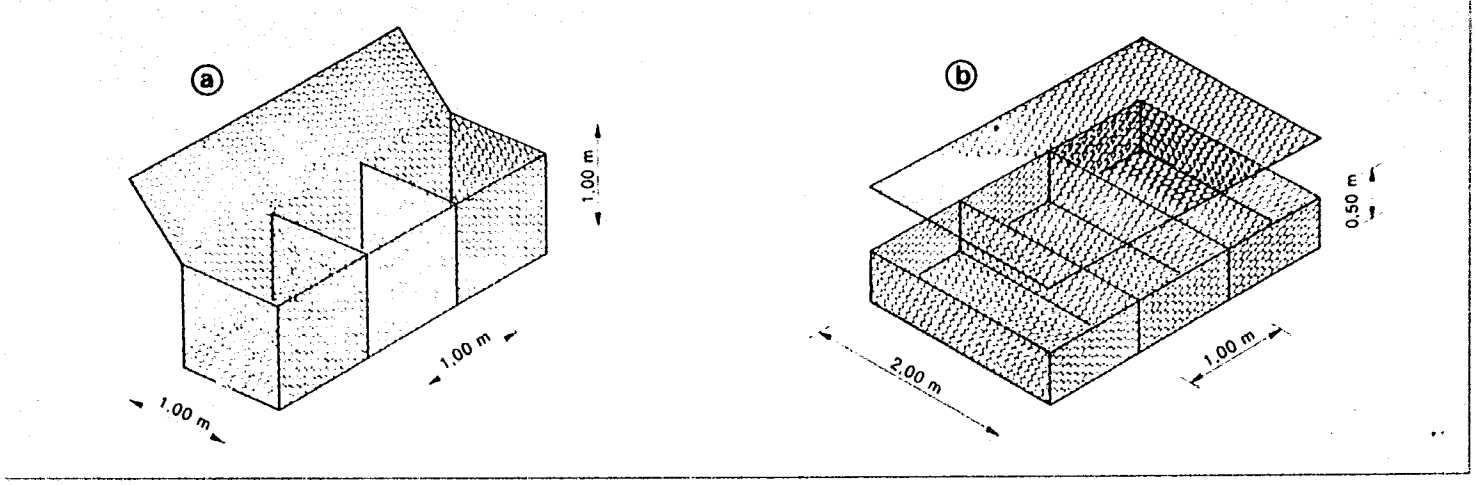


Table 1 - Gabion dimensions

Gabions - Zinc coated			Gabions - Zinc coated with PVC sleeve			
Mesh type	Wire ø mm	Thickness m	Mesh type	Wire ø inner mm	Wire ø outer mm	Thickness m
10 × 12	2.40 2.70 3.00	0.50	10 × 12	2.40 2.70	3.40 3.70	0.50
8 × 10	2.40 2.70 3.00		8 × 10	2.40 2.70	3.40 3.70	1.00
6 × 8	2.20 2.70	1.00				
5 × 7	2.00 2.40					

Width. 1.00 - 2.00 m - Length: 2.00 - 3.00 - 4.00 m for gabions 1.0 m wide; 3.00 - 4.00 - 5.00 m for gabions 2.0 m wide.

Reno mattress and gabions are manufactured with double-twisted hexagonal mesh made with annealed mild steel wire, zinc coated according to Circ. Cons. Sup. LL. No. 2078 dd. 27.8.1962, to BS 443-1982 and to U.S. Federal Specification QQ-W-461 H finish 5-class 3.
PVC-coated mattress are made with wire coated by extrusion with a speci- , highly corrosion resistant PVC. The types indicated in the table are standard: for more detailed information, see general catalogue.

2.2.2 Characteristics of filling materials

Any stone or other material may be used to fill the gabion as long as its density and other characteristics meet the structural, functional and durability requirements of the project. The most commonly used materials are round or quarried stones. Materials of higher specific gravity (Table 2) are preferable particularly if the gravity function of the structure is predominant or if the structure is submerged or exposed to running water. To ensure the durability of the structure the stone must be weather resistant, non friable, insoluble and sufficiently hard.

Table 2 - Density of different types of stone.

Type of rock	Density γ_s (kg/m^3)
Basalt	2900
Granite	2600
Hard limestone	2600
Trachytes	2500
Sandstone	2300
Soft limestone	2200
Tuff	1700

Figure 4 shows a diagram for the determination of the apparent density of the filled gabion given the density of the filling material γ_s and the porosity of the gabion "n", which generally varies from 0.30 to 0.40 depending on the hardness and angularity of the stone, the granulometric curve and whether it is round or quarried [11].

The most appropriate size for stone varies from 1 and 1.5 to 2 times the dimension D of the mesh (Fig. 2) that is, the stone should be large enough to prevent its escape through the mesh.

The use of smaller sized stone, 1 to 1.5 D, permits an improved and more economical filling of the cage, it also allows a better distribution of the imposed loads (Figs. 5 and 6) and adaptability of the structure to deformation.

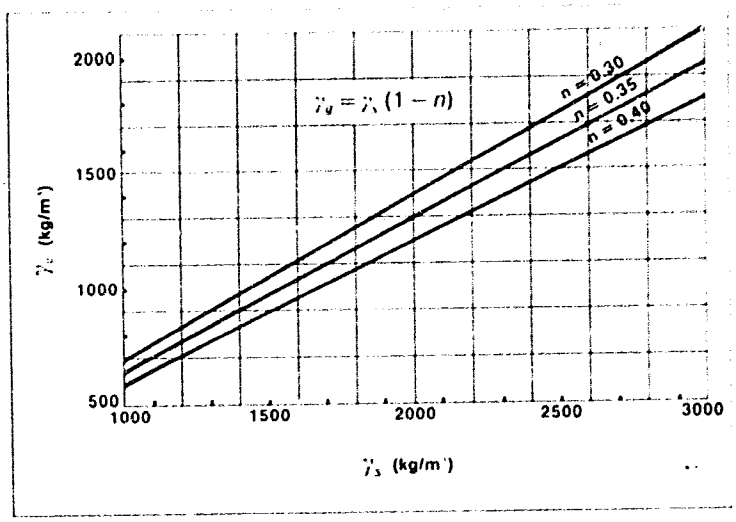


Fig. 4 - Diagram showing the determination of the apparent density γ_g of the filled gabion, given the density γ_s of the fill material and the porosity "n".

2.3 Assembly and installation

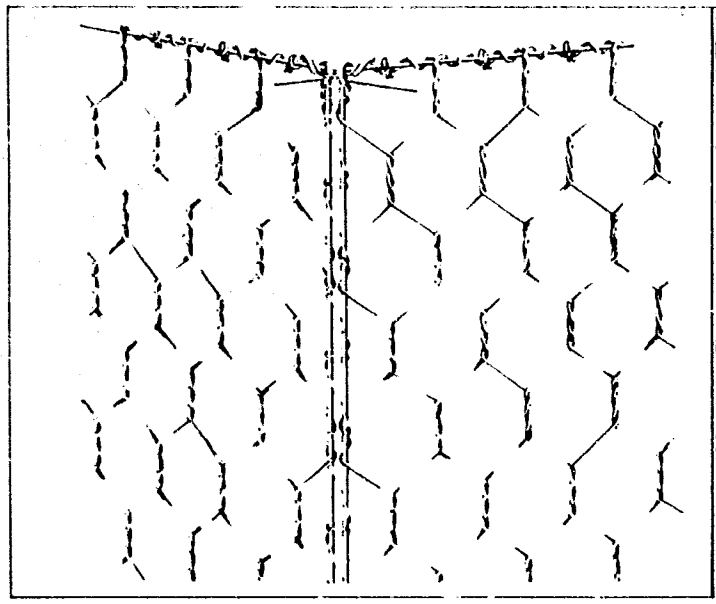
The gabions are dispatched from the factory folded and packed to reduce their volume and make transport to site easier and more economical. At the construction site the single gabions are opened and assembled; the edges are tied and the diaphragms, where fitted, are bound to the side wall panels. Then several empty gabions are joined together, set in place and firmly laced to the adjacent gabions along the edges in contact, both horizontally and vertically.

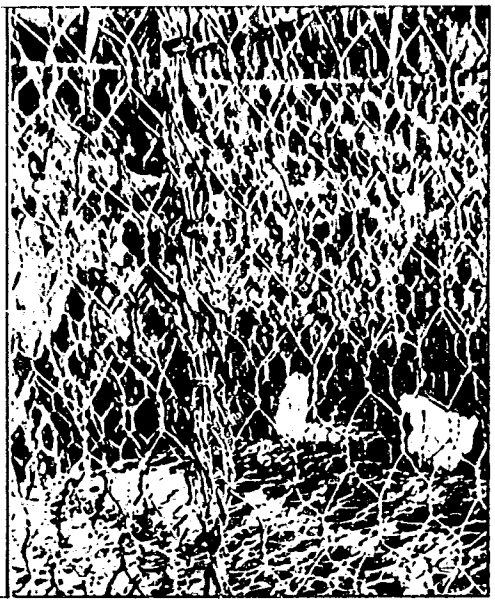
The most efficient type of lacing is illustrated in Fig. 7.

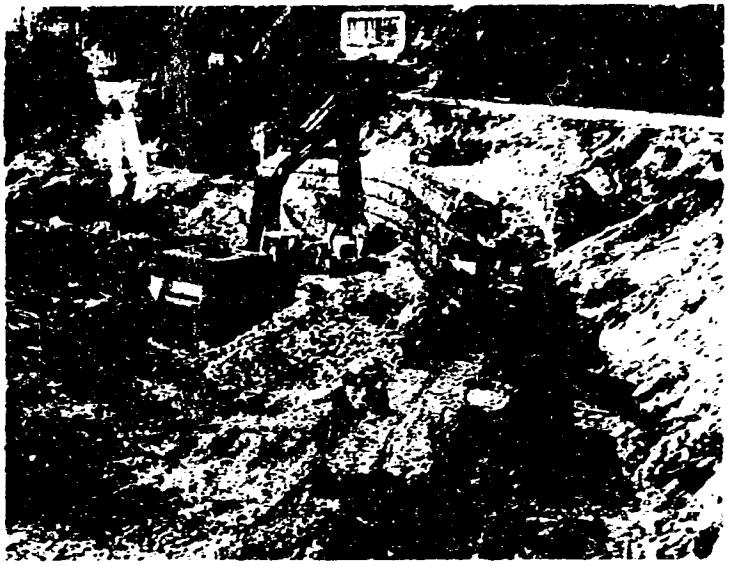
The placing of the gabions depends on the types used and on the structural characteristics of the project.

Filling is done by mechanical means (Figs. 8, 9, 10). The material is arranged inside the gabion to minimize the voids.

Once the filling is completed, the gabion is closed by lacing the lid along all edges and internal cell diaphragms.







2.4 Characteristics of gabion retaining structures

2.4.1 Technical characteristics

As we have seen, the assembly and installation of the gabion consists of a series of very simple operations. However the resulting structure possesses some outstanding technical advantages:

a) it is a *reinforced* structure, capable of resisting any type of stress and in particular tension and shear. The wire mesh acts not only to contain the stone, but it also provides a comprehensive reinforcement throughout the structure. To give the structure the greatest possible strength, particular attention should be paid to details such as the selection of the proper type of gabion, careful filling and correct lacing methods.

b) it is a *deformable* structure. Deformability, contrary to popular opinion, does not diminish the strength, but increases it by drawing into action all the resisting elements. The structure takes on the characteristics of a complex reinforced one capable of adapting in a wide sense to the redistribution of loads due to the movement of the stone fill (Fig. 11).

It is pointless therefore, and often damaging, to stiffen gabion structures, or even only parts of them, by placing stone or concrete on the face of the wall.

It is worth stressing the importance that the degree of deformability of the structure contributes to the final aims of the design. In fact the evolution, the distribution and the final value of the soil stresses depend on the way these structures deform and on the extent of the deformation.

c) it is a *permeable* structure, capable of collecting and carrying away ground water, so eliminating or attenuating one of the principal causes of soil instability.

The drainage function is augmented by evaporation generated by the natural circulation of air through the voids between the filling stones.

Auxiliary means should be installed for the disposal of the water collected via the structure.

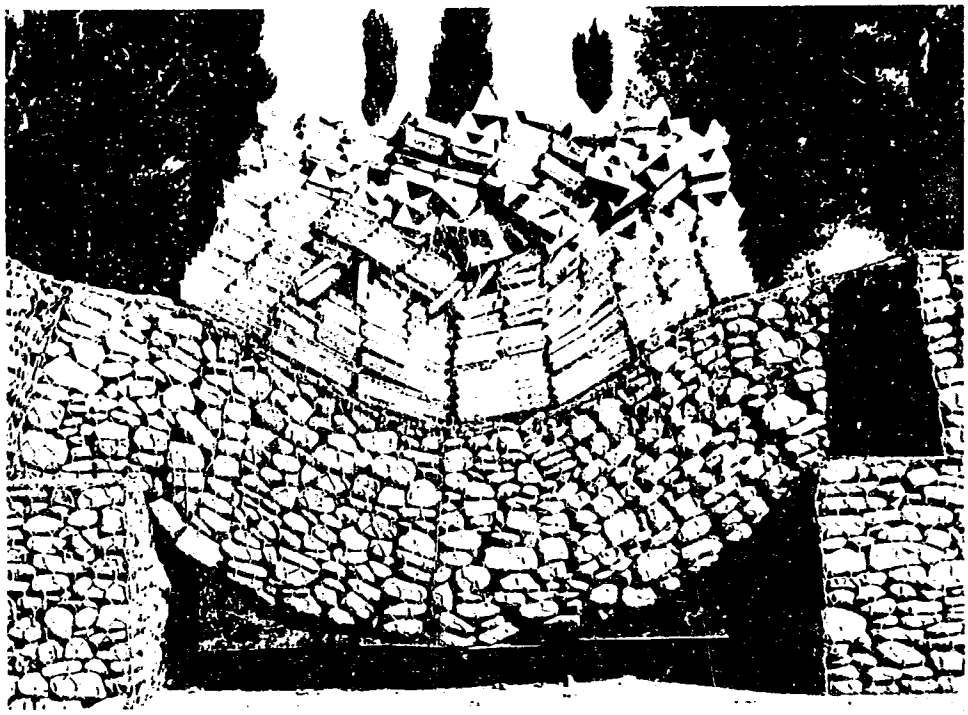


Fig. 11 - Results of tests made on the bending and deformation properties of a gabion of 8 x 10 mesh type, 3 mm diameter wire, fitted with diaphragms. The filled gabion was loaded with zinc bars on a 3 m clear span. The weight of the filled gabion was 5,100 kg. The wire mesh failed at a load of 15,000 kg.

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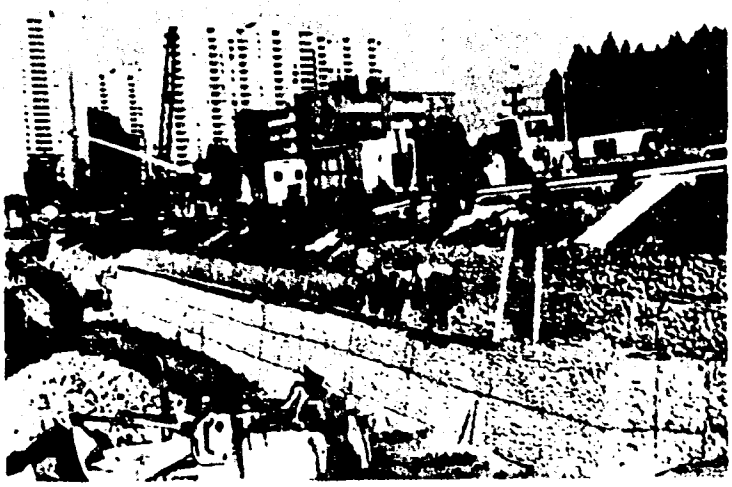
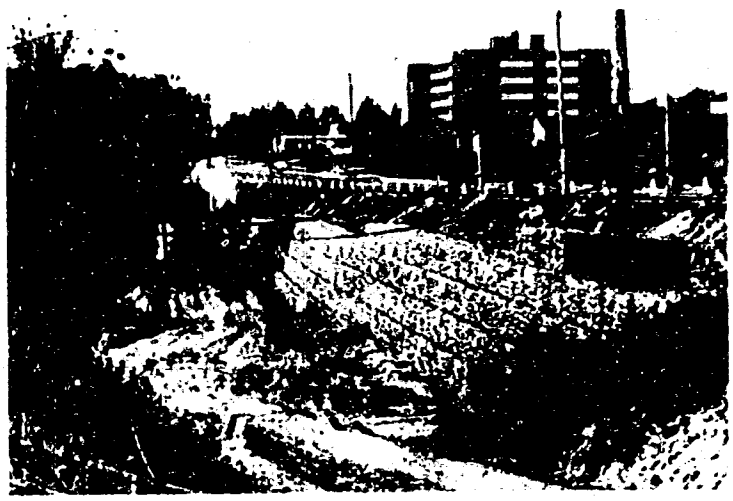
2.4.2 Functional characteristics

Over many years, gabion retaining structures have gained evergrowing success in the fields of soil protection and conservation and in the construction of river and canal structures, road and railroad embankments and cuttings. This is due, at least in part, to certain functional qualities:

long term durability, which means the structure may be considered permanent: deterioration of the mesh from oxidation is a very slow process with effects no more serious than the aging of any other structure. In addition to high quality zinc protection, the PVC coating greatly increases the durability. Furthermore the characteristics of gabion structures, in time, favor the establishment of a natural state of





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equilibrium. This means that the structure has to hold up under less severe conditions in comparison to the original situation when its presence was absolutely indispensable.
– *ease of installation*, in whatever environment, without the aid of specialized personnel or special equipment. This aspect takes on notable importance in marine and river reclamation projects, where rapid intervention is often necessary to retain the soil particularly in areas of difficult access and where all sorts of weather conditions prevail.
– *possible modification of the structure* at a later date: new units can be installed along side those already in place and others can be installed above them to increase the height.
– *capacity for effective functioning* even in the absence of certain complementary structures such as collecting and cut off drains. This is very important where the soil is saturated and where ground water levels fluctuate.
– *ease of maintenance*, low cost, and often not even necessary.

These characteristics which appertain to gabion structures, permit their use in any type of terrain, not only for structures to combat soil erosion, but also in works protecting roads, railways and buildings where higher safety factors and stricter time limits on completion dates are dictated.

2.5 Types of gabion walls for earth retaining structures

The design of retaining walls is a process of verification in which the designer must define the geometry of the structure, while guaranteeing its safety from overturning, sliding, and settlement, plus its overall stability in addition to the economic, functional and aesthetic requirements.

In the definition of the wall's geometry one must take into account the modularity of a gabion structure, whether it should be stepped on the front face or rear face, the angle of inclination of the foundation, and the type of foundation (simple, extended with a footing, or deepened).

Gabion retaining structures can be classified into four different types (Fig. 14).

1) gravity retaining structures
2) semi-gravity retaining structures
3) structures to stabilize embankments
4) thin walls anchored back into the soil by panels of hexagonal mesh.

The several types are distinguished primarily with regard to the calculation criteria that are most appropriate to each.

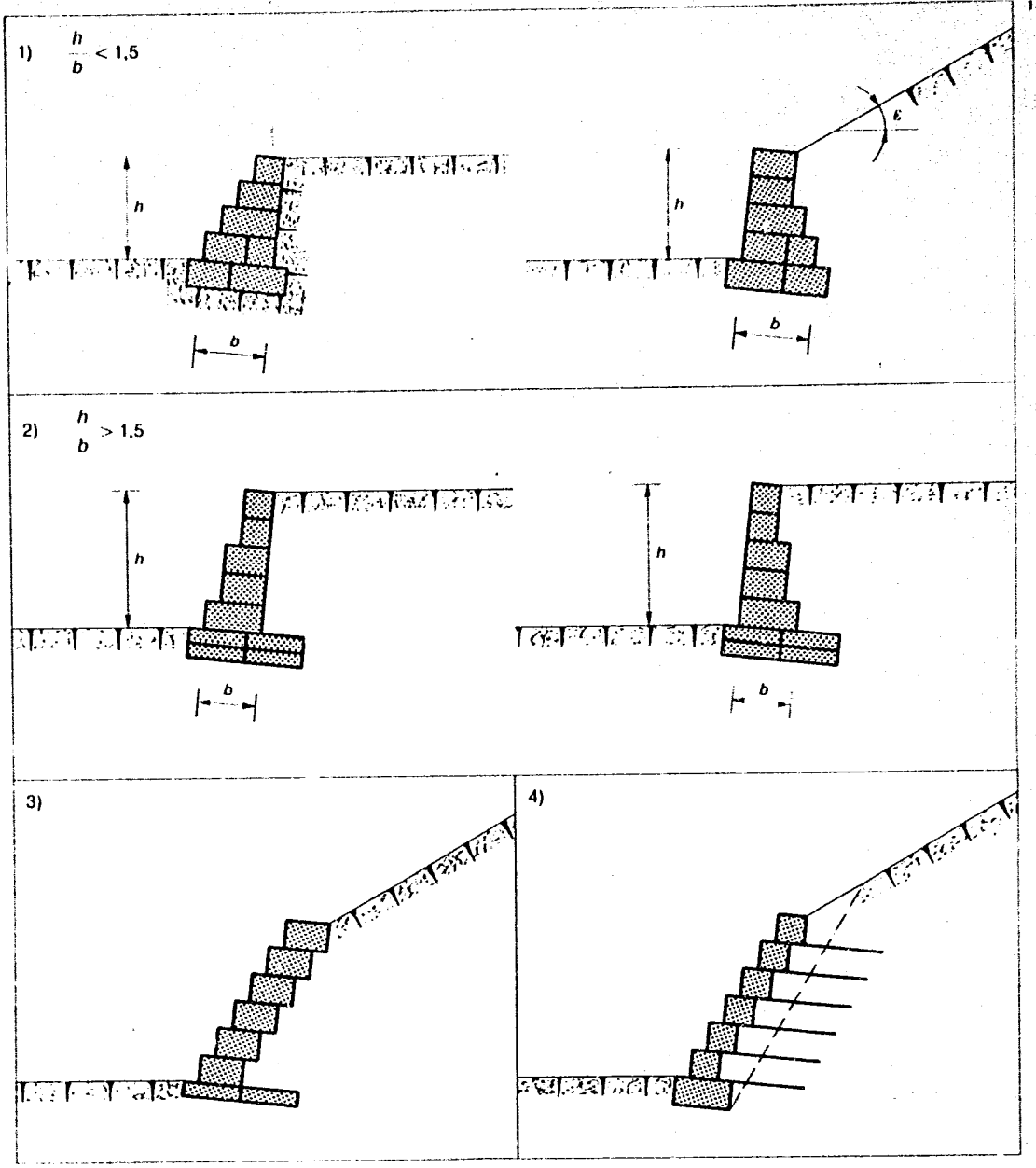
Gravity retaining structures are dimensioned using traditional methods of analysis.

Semi-gravity structures rely for their resistance on the reinforcement provided by the mesh to an extent that they can be considered as a special type of reinforced structure. In Section 4.3 the calculation criteria for gravity and semi-gravity gabion structures are set out. Structures for the stabilization of embankments may be treated as gravity structures.

Section 4.6.2.2. shows procedures which are based on the results of the original full scale experiments for the design of walls anchored with one or more layers of hexagonal mesh panels which provide static support and limit deformation.

Gabion retaining structures will, due to their nature, deform to quite a degree, thus for them to function properly it is unwise to have vertical faces for heights greater than 3.0 m. For heights greater than 3.0 m the face should be battered at not less than 6 degrees, or should be stepped (Fig. 15).

Fig. 14 - Types of gabion walls: 1) gravity, 2) semi-gravity, 3) wall supporting sloping surcharge, 4) thin walls with tie-back mesh panels.



CHAPTER III

Design criteria for gabion structures Experimental work and observations

3.1 Introduction

It follows from the study of gabion structures in Chapter II that the calculations for those designed for retaining earth must take into account the particular nature of the materials composing the filled gabion and their physical and mechanical characteristics.

A series of tests in three phases was devised to obtain data for the examination of these characteristics and of the behaviour of gabion structures under load. In the first phase, compression and shear tests on full size gabions and on single mesh panels were carried out in the laboratory for the purpose of determining strength and deformation parameters. In the second phase, load tests were performed on full size 4.0 metre high retaining walls for the purpose of verifying the previous results, identifying the behaviour of the structure as a whole and to establish the most appropriate calculation parameters and their limits of application. The purpose of the third set of tests was to evaluate the use of hexagonal woven mesh panels in soil reinforcement and as a form of anchorage for retaining structures.

The observations made during the tests and the results are detailed in the following paragraphs. The purpose is to identify the specific problems relating to gabion structures to assist the designer, who, in view of the numerous parameters involved, must make the final decision.

3.2 Tests on full size gabions

The mechanical stress tests were performed in the Laboratorio di Scienza delle Costruzioni, (Structural Science Laboratory), of the University of Bologna (1979) using specimens made of one or more gabions wired together. Their nominal size was 0.50 × 0.50 × 0.50 m, and they were fabricated from 6 × 8 woven hexagonal mesh of wire diameter 2.7 mm. The gabions were assembled, laced with 2.4 mm diameter wire and filled by hand with quarried stone (Fig. 16).

Three series of tests were made: simple compression, compression with lateral expansion restrained on two opposite sides, and shear [12, 13].

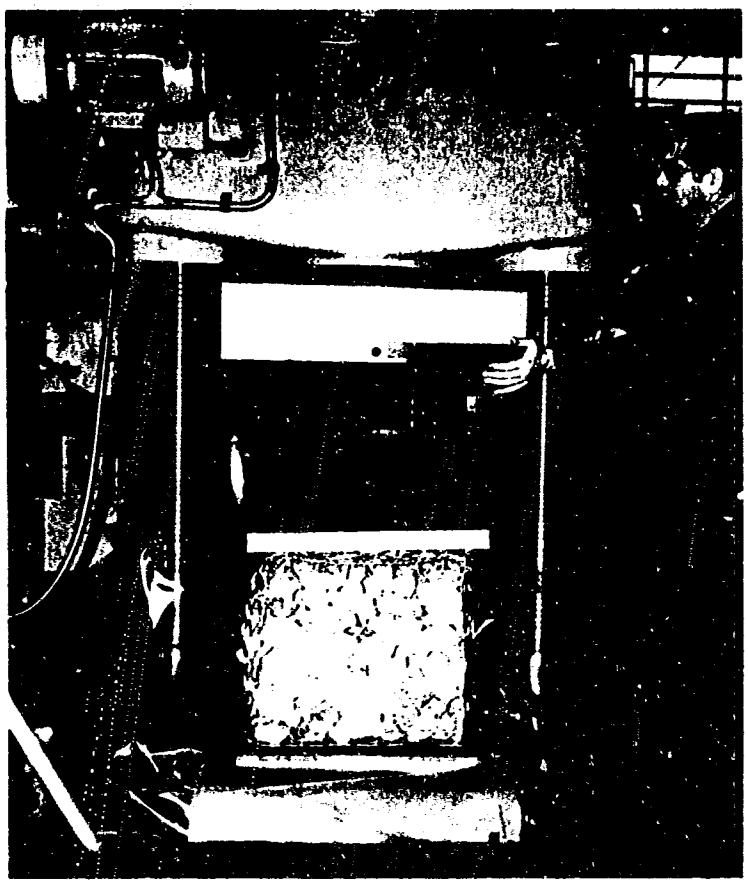
Fig. 16 - Full size gabion test samples.

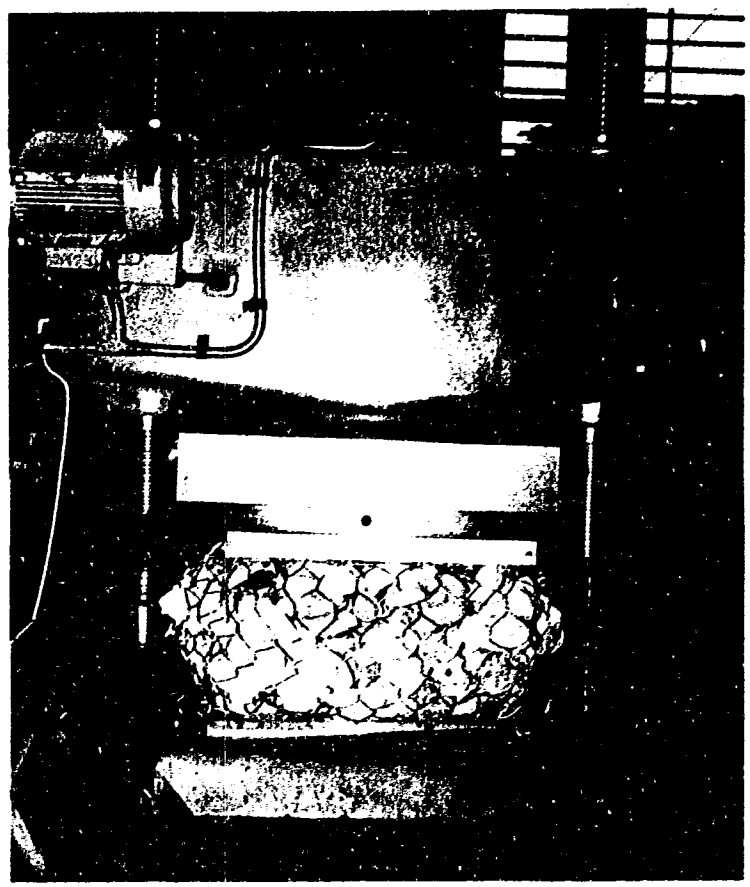
3.2.1 Simple compression tests

The first series, to examine compression with free deformation of the vertical sides, was intended to provide general indications of the resistance of gabions to compression. The points requiring investigation were:

(i). The sequence of events that accompany the progressive deformation induced by increased loading.
(ii). To determine the point of final failure of the structure involving the fracture of the stone fill by compaction, deformation and rupture of the mesh resulting in the spillage of the stone fill.
(iii). Influence of the orientation of the weave of the mesh in relation to the direction of the applied load.

Figs. 17, 18 - Simple compression tests.





(iv). The effect of diaphragms incorporated in the gabions (Figs. 17, 18).

To verify the possible influence of the method of applying the load on the mesh containing the stone, some of the tests made on single gabion units were repeated on two units, one on top of the other.

For each type of gabion three or four tests were made. Table 3 shows, for each type of gabion tested, a diagram indicating the direction of the hexagonal weave on the free vertical sides and the position of internal diaphragms, if any, the number of the test specimen, and the initial dimensions of the test specimen before the vertical load was applied.

The direction of the weave and the position of the internal diaphragm in the types of gabion tested generally correspond to those occurring in actual construction. The opportunity afforded by the observation of the behaviour of gabions of different configurations and the need to generate a wider spread of data has prompted additional investigation. The terms "horizontal mesh" and "vertical mesh" refer to the direction of the major axis of the weave of the mesh.

Figure 19 shows curves for the experimental values of $\sigma = P/A$ and of $\varepsilon = \Delta h/h$ derived from type A tests, where h is the initial height of the specimen subjected to compression, Δh is the reduction height from its initial value due to the application of the vertical load P, and A is the area of the horizontal sides in contact with the loading plates.

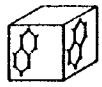
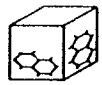
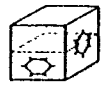
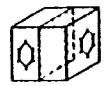
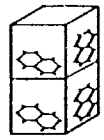
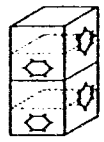
The load on the specimen was applied at a rate of about one kilogramme per square centimetre per minute [(1 kg/cm^2)/min] and was halted when the mesh had become so disintegrated that no further increase could be tolerated.

A comparative analysis of the test results for single gabions suggests the following comments:

- The crushing of the gabion takes place in successive stages. It begins with the progressive readjustment of the stone fill, the individual stones remaining intact, followed by an extensive readjustment under loads which are small compared to the ultimate load, but greater than those expected in actual practice. This phase is characterised by values of $\Delta\sigma/\Delta\varepsilon$ usually below 20 kg/cm^2 except in the case of two specimens of the "B" type.

A second phase begins with the fracture of some stones inside the specimen coinciding with a deformation $\Delta h/h$ of about 10%. Intensification of the load and hence the compression, leads to greater stone fragmentation which spreads through the entire central core. At the same time the increasing tension in the steel mesh and in the peripheral stone on the exterior, exercise a containing action on the central nucleus, which by the end of the test is reduced to minute fragments (Fig. 20). Coincidental with the increasing load and the crushing and bulging of the specimen, rupture of

Table 3 - Simple compression tests on gabions [13]

Type of gabion	Scheme	Test No.	Initial dimensions (m)		P_{max}	σ_{max}	Final dimensions (m)	
			Base	Height	(t)	(kg/cm²)	Base	Height
Rectangular box with vertical mesh on the side panels		A/1	0.50 x 0.52	0.490	90.5	34.8	not recorded	0.215
		A/2	0.53 x 0.55	0.470	120.0	41.2	0.81 x 0.85	0.235
		A/3	0.54 x 0.57	0.460	75.0	24.4	0.82 x 0.80	0.245
		A/4	0.53 x 0.56	0.500	93.0	31.3	0.82 x 0.85	0.260
Rectangular box with horizontal mesh on the side panels		A_o/1	0.47 x 0.57	0.530	25.0	9.3	0.67 x 0.74	0.390
		A_o/2	0.49 x 0.53	0.520	31.0	11.9	not recorded	0.405
		A_o/3	0.48 x 0.58	0.530	31.0	11.1	0.74 x 0.72	0.360
Rectangular box with horizontal diaphragm, vertical mesh on two opposite sides and horizontal mesh on the other two sides		B/1	0.48 x 0.50	0.520	105.0	43.8	not recorded	0.260
		B/2	0.48 x 0.50	0.530	85.0	35.4	0.79 x 0.76	0.280
		B/3	0.47 x 0.51	0.550	112.5	46.9	not recorded	0.225
		B/4	0.47 x 0.51	0.550	100.0	41.7	0.78 x 0.78	0.270
Rectangular box with vertical diaphragm: vertical mesh on side panels		C/1	0.50 x 0.53	0.470	67.7	25.6	0.76 x 0.86	0.230
		C/2	0.50 x 0.59	0.480	120.0	40.7	0.80 x 0.88	0.210
		C/3	0.51 x 0.60	0.500	136.0	40.4	not recorded	0.230
Specimen made of two box elements, superimposed and tied, with horizontal mesh on the side panels		(A_o + A_o)/1	0.50 x 0.58	1.000	30.0	10.3	not recorded	0.685
		(A_o + A_o)/2	0.50 x 0.56	1.050	28.5	10.2	0.75 x 0.72	0.775
		(A_o + A_o)/3	0.52 x 0.61	1.020	27.0	8.5	not recorded	0.790
Specimen made of two box elements, superimposed and tied, with horizontal diaphragms and vertical mesh on two opposite sides and horizontal mesh on the other two		(B + B)/1	0.50 x 0.54	1.100	45.0	16.7	not recorded	0.570
		(B + B)/2	0.46 x 0.51	1.090	34.5	15.7	0.70 x 0.74	0.610
		(B + B)/3	0.48 x 0.50	1.080	40.0	16.7	0.73 x 0.80	0.580

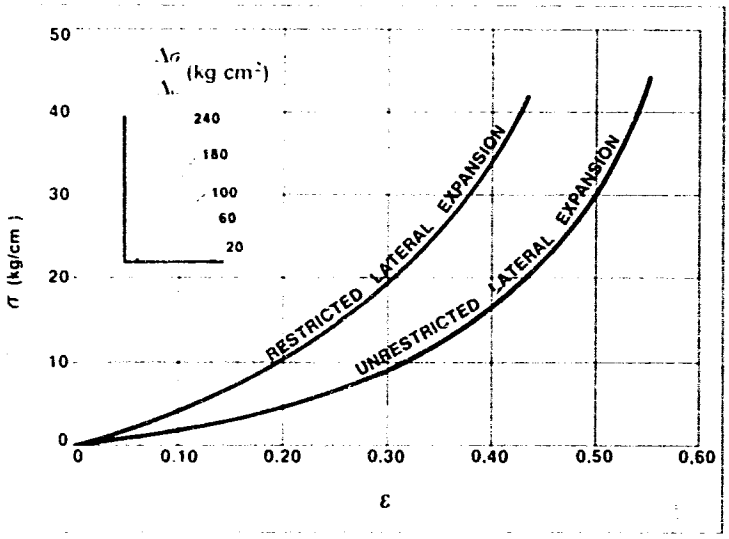


Fig. 19 - Results of compression tests on gabions with restricted and unrestricted lateral expansion.

Fig. 20 - Stone filling after testing.

the steel wire mesh occurs in the areas of concentrated pressure which are either at the contact points between the gabion and the loading plates or where very sharp edges of stone are in contact with the mesh. The fracture of a single wire does not affect the containing function of the netting since the woven hexagonal mesh will not unravel.

The causes of fracture of the wire are unpredictable and they present themselves in the various tests under a variety of loading and compression values. They are visibly influenced by the disposition of the mesh weave in relation to the direction of the applied load. The last phase is marked by extensive rupture of the wire mesh rendering it incapable of containing the stone fill and the specimen, unable to sustain any further increase in load, fails.

In Fig. 19 all the tests show an almost linear variation between $\sigma = P/A$ as a function of $\varepsilon = \Delta h/h$ and values of $\Delta\sigma/\Delta\varepsilon$ between 20 and 40 kg cm^{-2}, provided $\Delta h/h$ remains smaller than about 30%. Above this value, as the core of the stone fill is progressively fractured and the external mesh increasingly exercises its containing function, the curve rises sharply so that when values of $\Delta h/h$ approach 50%, $\Delta\sigma/\Delta\varepsilon$ reaches values over 200 kg cm^{-2}.

Comparison of the results of tests A and A_0 with vertical and horizontal mesh respectively, demonstrates that those with horizontal mesh are slightly more rigid. This indicates that horizontal mesh on the sides of the units deforms less rapidly than the vertical even under small loads and therefore contains the stone fill more efficiently. The most outstanding difference in the behaviour of horizontal and vertical mesh units is demonstrated by the values of the deformation $\Delta h/h$ and average pressure P/A at the point of failure. In the case of horizontal mesh units, (see tests A_0), extensive rupturing of the wire occurs at loads of less than 10 kg cm^{-2} with corresponding deformation of less than 30%. Vertical mesh units, (see tests A), fail at loads greater than 30 kg cm^{-2} with a corresponding deformation of about 50%. This can also be attributed to a lesser tendency of the horizontal mesh to deform.

Comparison of the results of test C, (with diaphragm), and of test A (without diaphragm), shows that the inclusion of a vertical diaphragm has no appreciable effect on the behaviour of the gabion.

Test conducted on pairs of gabions, one on top of the other and indicated by the symbols $A_0 + A_0$ and $B + B$ give the following results:

- Test $(A_0 + A_0)$ yielded results substantially the same as those of test A_0.
- Test $(B + B)$ yielded results similar to those of test B for deformation in the 30-35% range. For deformation greater than 35% it was impossible to keep the specimens in place under the applied load.
- In general, the tests conducted on double gabions did not indicate any appreciable influence on the scale factor.

3.2.2 Compression tests with lateral expansion restrained on two opposite sides

These tests have been done on specimens similar to those used in tests A (gabions with vertical mesh on the side panels), by placing two vertical stiffened steel plates against the two opposite side panels in order to restrict their deformation.

The plates were lubricated to reduce the amount of friction between them and the gabion. These tests, conducted in the same manner as described earlier for the simple compression test, were terminated when the steel plates became too distorted or when the unrestrained gabion faces made contact with the guide bars of the press. The results do not differ substantially from those of type A tests, showing, also in this case, an initial adjustment of the stone, the crushing of the internal core, and the rupture of the mesh wire (which in this case happened only in two of the three tests and under very high loads). It was not possible even with the maximum pressure of $P/A = 30$ kg cm^{-2} to bring the gabion to the failure stage.

When the two curves in Fig. 19, representing the average behaviour of the specimens of type A and those specimens with restricted lateral deformation, are compared, a greater rigidity especially for small loads can be observed in the last series of tests.

3.2.3 "Simple shear" tests

This terminology refers to a type of test where shear stress prevails.

Table 4 illustrates the types of structure tested with a dimensioned section of the arrangement, the maximum load P attained, the average maximum shear stress τ, and the maximum deflection H. The test results are shown in Fig. 21 and they show a noticeable shear resistance in the gabions accompanied by considerable deformation. The shear resistance is provided by the mesh and is therefore increased by increasing the strength of the mesh itself or by introducing appropriate diaphragms (Fig. 22).

In the shear tests a small initial readjustment of the stone fill, with a relative large deformation was noticed, followed by a stiffening phase in which the structure became more rigid as the resistance of the mesh was brought into action.

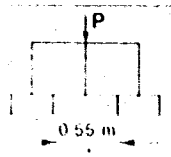
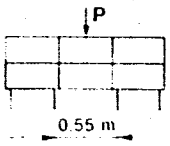
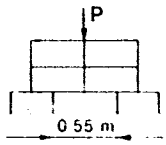
Table 4 shows the values of the shear module $G = \tau/(2H/l)$, where l is the free span of 0.55 m betwen the supports, calculated for the maximum load and for a load $P = 2500$ kg which corresponds on average to the beginning of the stiffening phase (see diagram H-τ in Fig. 21).

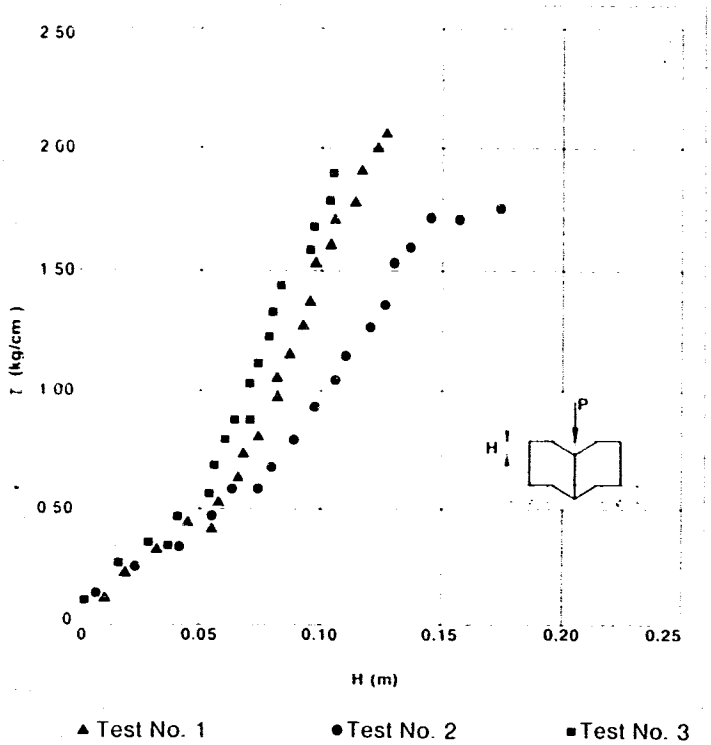
3.2.4 Interpretation of the test results

The results of the experiments which were summarised and discussed in the preceeding paragraphs may serve to clarify some aspects of the behaviour of the materials which constitute the gabion structure.

a) First of all the simple shear and compression tests have demonstrated that during the initial application of the load, it is impossible to interpret a process of deformation that is in fact quite reversible, particularly as the irreversible portion of the distorted mass grows considerably as the deformation

Table 4 - "Simple shear" tests on gabions.

	Arrangement of gabions	Cross section dimensions (m)	Maximum load P_{max} (t)	Maximum shear stress τ_{max} (kg/cm²)	Max. deflection H (m)	Shear modulus for $P = P_{max}$ τ_{max} / 2H l (kg/cm²)	Shear modulus for P = 2500 kg τ / 2H l (kg/cm²)
1		0.54 x 0.53	12.2	2.13	0.16	3.65	2.12
2		0.48 x 0.55	9.6	1.82	0.21	2.40	1.75
3		0.53 x 0.53	11.1	1.98	0.13	4.20	2.55



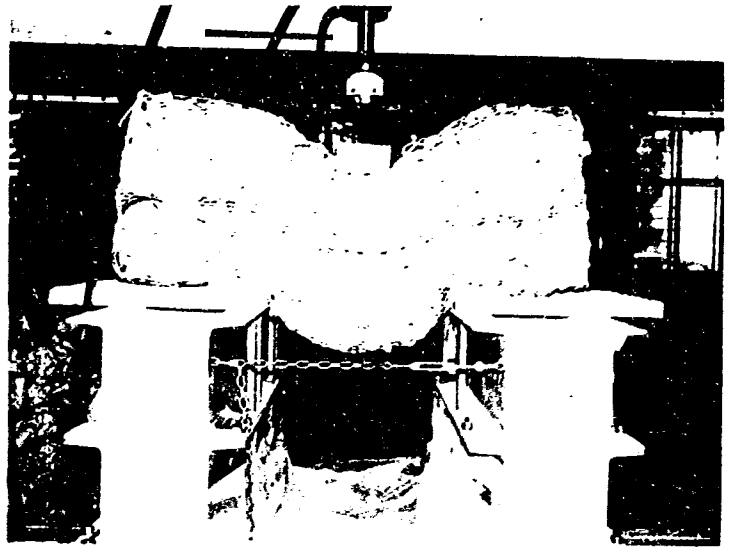


Fig. 21 - Results of "simple shear" tests on gabions.

Fig. 22 - "Simple shear" test in progress.

process increases. In other words, the behaviour of the material may only be considered to be elastic when the values of stress are low and it is obvious that conditions are improved if the gabions are loaded in such a way as to prevent deformation on two opposite sides, that is by restricting the occurrence of deformation to one plane only.

It should be noted that this condition occurs frequently in practice, so that in some respects the compression tests on units with restricted lateral deformation happen to be more significant than those of simple compression.

b) Once the phase beyond the "elastic" condition is reached, fracture of the stone in the core begins and is accompanied by internal movement and an increase in density. The associated deformation has an irreversible nature and in this sense we can speak of "plastic" behaviour of the material. The load — deformation diagrams clearly reveal that the rigidity of the material increases directly in proportion to the load, and in the case of extensive deformation, the test samples did not reach complete failure condition. These two cases prompt us to describe the behaviour of the material as hardened-plastic. It is accompanied by a mode of deformation that for all practical purposes must be considered as indefinable.

c) The slope of the load — deformation curve, taking the simple compression test as an example, has a difficult [illegible]

tion with the orientation of the mesh provided all the other conditions, and particularly the density and grading of the stone in the gabions, can be considered equal. The mesh, when set in such a way as to effectively oppose stretching in a transverse direction, will reduce the ductility of the specimens. The horizontal diaphragms also seem to have a certain effect in this sense. These results confirm prior impressions, which albeit were intuitive, that the resistance of a filled gabion is for the most part provided by the containing action performed by the mesh on the stone fill.

If the behaviour of the gabion material is interpreted in terms of the criteria commonly used in respect of soils, e.g. Mohr-Coulomb, it can be concluded that the containing action of the mesh on the stone fill corresponds to the active thrust. On the other hand, the conditions in actual structures are similiar to those in the experiments except that no expansion can take place because the unit is contained by adjacent gabions which add immeasurably to the retaining action of the mesh. This additional resistance is seen in Fig. 19 where the supported load with restrained expansion is about double that with free expansion, the deformation being equal in each case.

This is equivalent to an increase of about 5% in the angle of internal friction of the retained material. However, it is not consistent to compare a filled gabion with cohesionless soil considering the shear strength obtained in the tests. This strength in shear can be regarded as being similar to that provided by shear reinforcement in a short concrete beam, or in terms of soil, assuming that the filled gabion possesses cohesion (see § 4.4.2) and therefore has a high angle of internal friction and also a high value of cohesion.

3.3 Laboratory tests on panels of mesh

The steel mesh from which gabions are made must possess specific properties to guarantee an adequate performance with regard to both structural strength and durability. Tests to check the mechanical properties of the mesh were carried out at these centres: the "Strength of Materials" Laboratory of the Engineering Department of Bologna University, the Colorado Test Center Inc., Denver, U.S.A., 1983 [14], and in the Officine Maccaferri S.p.A. laboratory in their works in Bologna (Fig. 23).

The breaking load in these tests was assumed to be that which caused the first wire to fracture. Table 5 gives average values of tensile loads in kg/m length for mesh tensioned in the direction of the weave.

Similar tests were made on panels loaded at right angles to the weave of the mesh. The breaking strengths obtained in these tests are approximately 1200 kg/m for the 6 x 8 mesh in wire diameter 2.2 mm, and 2200 kg/m for the 8 x 10 mesh and wire diameter 3.0 mm.

In some tests the elongation of the mesh at breaking point was recorded. For the loads applied longitudinally the elongation was from 6 to 7%, for those applied at right angles to the weave it was 20 to 22%.

Table 5 - Ultimate failure loads on hexagonal woven steel wire mesh (kg/m).

Mesh type	Ultimate loads (kg/m)				
	Wire diameter (mm)				
	2.0	2.2	2.4	2.7	3.0
5 x 7	3500	4000	4500	–	–
6 x 8	3000	3500	4200	4700	–
8 x 10	–	–	3400	4300	5300
10 x 12	–	–	–	3500	4300

The last test was designed to check the resistance of the mesh against punching. This was performed by applying a vertical load via a metal disc 0.35 m in diameter to the centre of a panel of mesh fixed on all four sides but spanning 0.8 m in each direction. The first wires failed at the rounded edges of the plate at an average recorded load of 3250 kg.

Fig. 23 - Tensile test on wire mesh panels.

3.4 Load tests on full size structures

3.4.1 Test arrangements

The tests on full size 4 m high walls were made at the Zola Predosa factory of Officine Maccaferri S.p.A., near Bologna between December 1981 and February 1982 in collaboration with the Istituto di Tecnica delle Costruzioni of the University of Bologna.

They had proved to be necessary because the scaling factor used in the model tests could not be reliably established in quantitive terms. A number of practical problems had to be overcome to perform the full scale tests, and the procedure was complex, however, in the final analysis some very useful and significant results were produced.

The simulation of the earth thrust against the face of a retaining wall presents several problems with regard to the measurement of both the load intensity and the most probable load distribution. To eliminate these problems it was decided to use hydrostatic pressure of which there are no unknown factors. Fig. 24 shows the testing arrangement consisting of two gabion walls set up opposite each other, one metre apart at the base.

The height of the wall was 4 m with bottom width of 2 m, a vertical front face, and a stepped back face that reduced the wall thickness to 1.5 m from the mid height to the top of the wall. The 0.50 m thick base platform extended 0.50 m on both sides and the total length of each wall was 5 m (See Fig. 25).

The ends of the space contained between the internal faces of the walls were closed off with steel plates interconnected by wire rope ties (See Figs. 26 and 27).

Inside the enclosure were placed flexible water tanks arranged to form two separate chambers as shown in the drawing in Fig. 24. To ensure that the system would continue to function at the maximum loading and greatest deformation, the tanks had to be completely impermeable and have a high tensile strength. They were significantly oversized to accommodate the large movements expected of the structure. Between the tanks and the interior faces of the walls was

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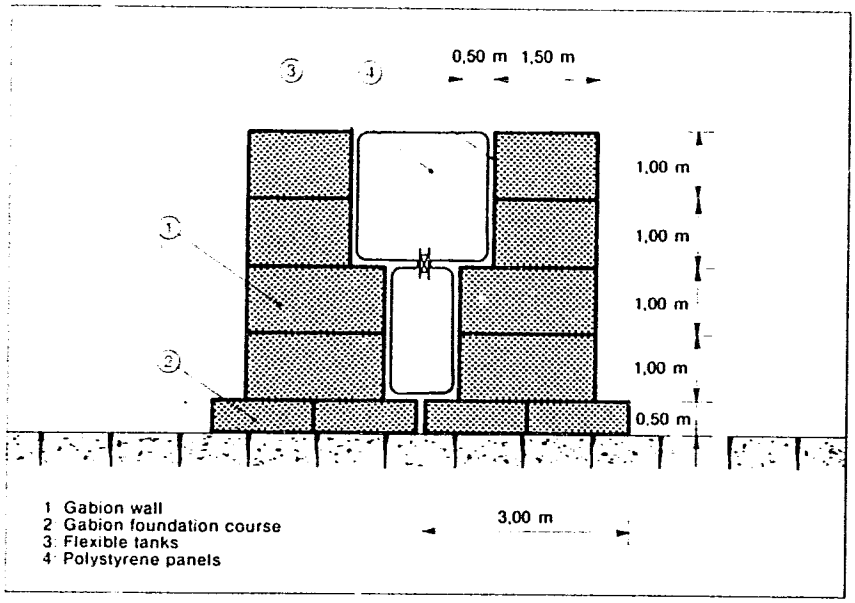

Fig. 24 - Sketch showing the arrangement for tests on full size gabion structures.

Fig. 25 - The gabion walls before testing.

Fig. 26 - Installation of the steel plates at the ends of the water tanks.

Fig. 27 - The upper flexible tank almost completely filled with water.

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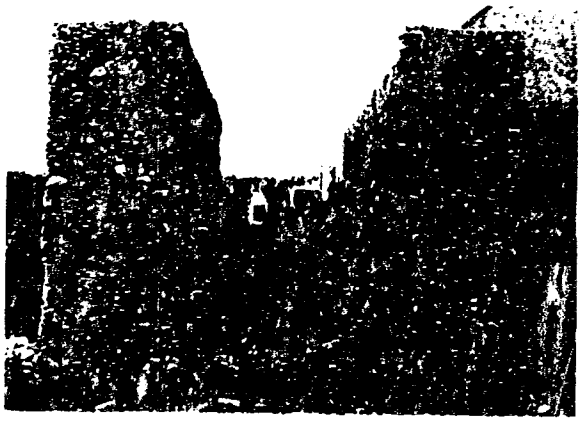

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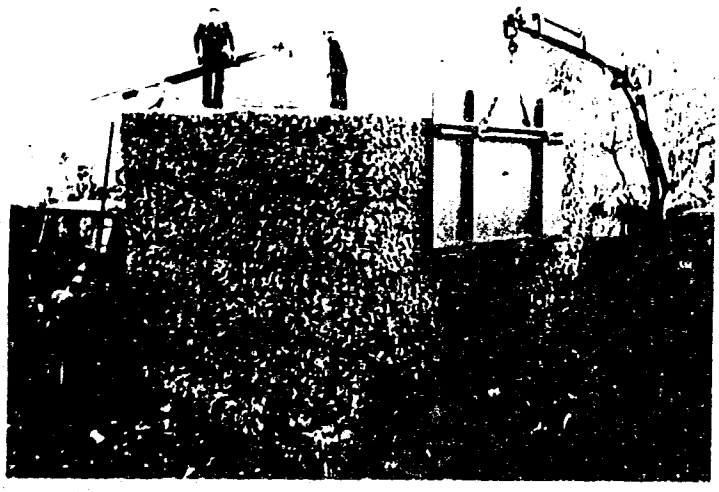

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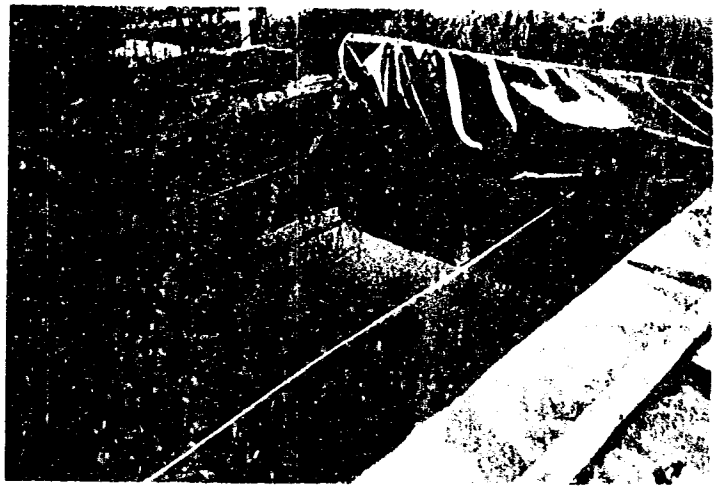


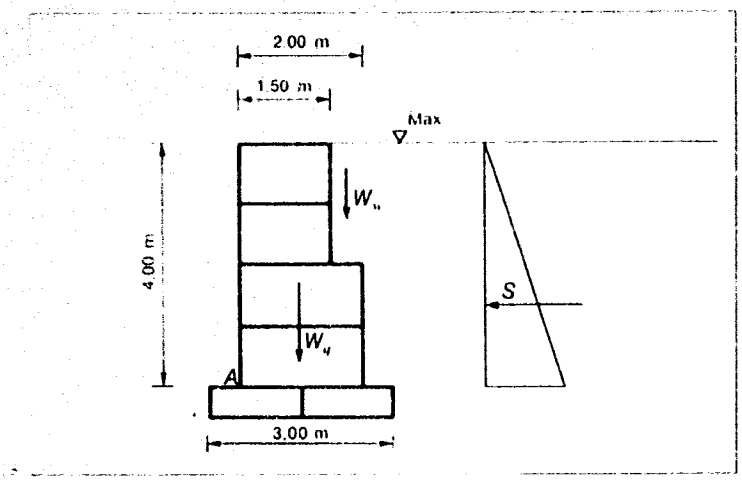


Fig. 28 - Forces acting on the hydrostatically loaded gabion wall.

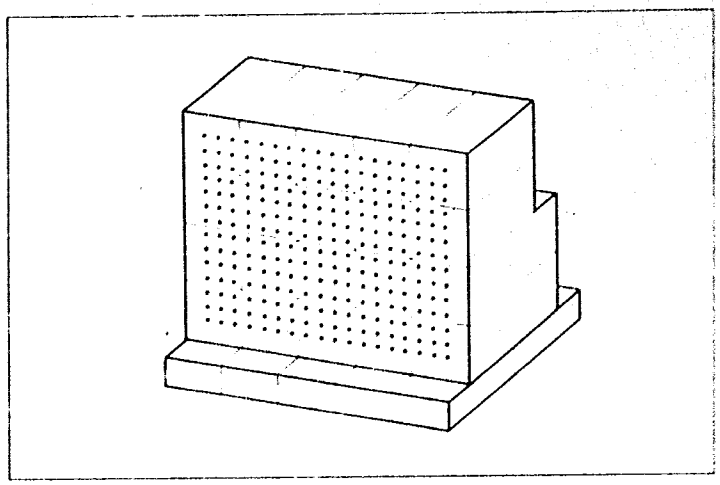


Fig. 29 - Diagram showing the matrix of points on the wall at which deformation was measured.

placed a 5 cm thick sheet of polystyrene to separate the tanks from the steel mesh and the ties which might have torn the tanks or interfered with their movement.

The positions of the end plates were carefully adjusted so that the ties interconnecting them would not damage the tanks.

To ensure a linear distribution of the water thrust, the upper and lower tanks were hydraulically connected, see Fig. 24. The depth of the water during the successive phases of the load test was measured from the outside by means of a piezometer.

Since the loading system on both walls provided a situation which was not affected by the end condition, it was possible to assume a plane of deformation for the central portion of the walls.

The walls were formed of gabions of nominal dimensions of 1.50 × 1.00 × 1.00 m, 2.00 × 1.00 × 1.00 m and 1.50 × 1.00 × 0.50 m without interior diaphragms, in 6 × 8 double twisted mesh of wire diameter 2.00 mm and laced together with 2.00 mm diameter wire. The filling material was rounded river stone of average size 90 to 120 mm.

The gabions were supplied from the current production line, but were lighter and more deformable than those normally used for retaining structures, i.e., mesh 8 × 10, wire diameter 2.70 mm. The usual construction procedure was followed [9].

The cross section of the wall was determined on the basis of conventional methods of design for gravity walls, and using reduced safety factors.

A maximum hydrostatic head of 4 m and a gabion density of 1800 kg/m^3 were assumed and the section had a safety factor of 1.22 against overturning about point *A*, see Fig. 28, and 1.06 against sliding on a horizontal plane through point *A* using an angle of friction of 32° for the stone fill. The maximum bearing pressure on the base, assuming zero tension and compression on only part of it, was 5.3 kg/cm^2, while the average shear stress was in the order of 0.4 kg/cm^2, taking into consideration the interlocking action of the stone.

3.4.2 Method of measurement

A photogrammetric system was used to measure the extent of deformation on the exterior face. It was chosen in preference to direct linear measurement for two reasons. Firstly it was expected that the deformation would be very large, judging by observations made on numerous completed structures and by the results of test on individual gabions as outlined in Section 3.2. Secondly, obtaining an average value for the deformation at a specific height, would entail taking horizontal measurements at a large number of points, in view of the unusual character of the gabion material.

In the photogrammetric method a grid reference system was introduced to cover the 5 m × 4 m wall surface. The matrix of 266 points was based on 14 horizontal and 19 vertical lines and had an average dimension between points of 0.25 to 0.30 m which was dictated by the irregular distribution of the stone in the gabions (See Fig. 29).

This large number of measurements would have been practically impossible using conventional equipment especially as movements varying from zero to over a metre beyond the original line of the wall were expected. The photogrammetric method was capable of providing this service and at a precision greater than that which was required.

The measuring apparatus consisted of two cameras mounted on fixed tripods to take simultaneous pictures of the surface of the wall. From the results of the photographs taken at various stages of loading and using an arbitrary but identical reference plane for all measurements, it was possible to plot the position of the wall surface to within one millimetre. For any given load, the difference between the coordinates on the adopted cartesian system [15] of the positions before and after the load was applied denote the movement of the wall.

The measurements were performed at night to avoid any possible effect due to temperature changes during the day and to facilitate the photogrammetric operations.

3.4.3 Loading system

The load was applied by pumping water into the tanks between the walls in successive increments. This allowed spot checks to be made on the behaviour of the structures as loading progressed.

The diagram in Fig. 30 shows the depth of water at various stages of the test. The evolution of the deformation of the wall can be studied in Figs. 31, 32, 33.

After a water depth of 3.4 m from the base of the wall had been reached, the structure was left under load for 20 days to obtain data on the effect of uniform loading over a period of time. Subsequently on 4/1/1982 the structure was partially unloaded by lowering the water level to 2.00 metres and then increasing it again on 6/1/1982 to a level of 3.50 m (6/1/1982, Figs. 30 and 34). The load was maintained at this level of 3.50 m, while the structure was kept under observation until 15/2/1982 before increasing it to the final point when a deflection of 0.60 m at the top of the wall was attained. During this period the increase in deformation was only minor, (See Fig. 53). The water depth was then reduced to 2.3 m before proceeding to the final phase.

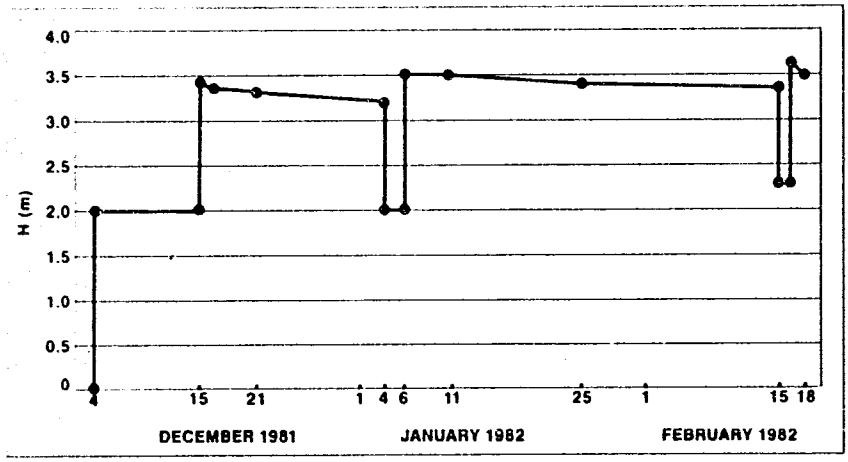
After the 3.5 m mark was passed, sizeable increments in deformation took place without significant increases in load. A maximum value of 3.6 m was finally reached and the structure was unloaded for the last time on 18/2/1982 after the deflection had attained a limit value of 1.10 m (Figs. 36, 37).

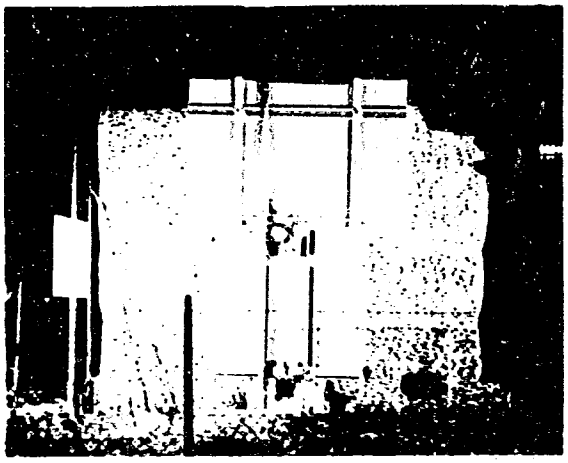
3.4.4 Comments on the observed measurements

The coordinates of the 266 points on the grid reference matrix recorded at all stages of loading were transferred on to

Fig. 30 - Levels af loading in metres of water during the duration of the test.

Fig. 31, 32, 33, 34 - Sequence of photographs showing the deformation at various phases of the test.





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a magnetic tape. This tape was later used in a computer programme to analyse the deformation at every point at each test stage with respect to its initial vertical position.

Fig. 38 illustrates graphically the average values of deformation at various wall heights at some of the loading stages. On plotting these results it was found that the scatter was relatively minor, and therefore after discarding the high and low values the averages obtained for the deflection were considered to be very dependable. The evolution of the average deflection of the top of the wall as a function of the hydrostatic load is illustrated in Fig. 35.

After the diagrams and photographs were examined, it was concluded that the ability of a gabion structure to deform derives mainly from the internal sliding or "shearing" which takes place within the filling material when it is subjected to thrust. In addition to this fundamental observation, it is possible to deduce the following:

a) The deformation induced by the load is essentially irreversible. As already indicated by the results of the test on individual gabions, these structures do not behave elastically. (See Fig. 35).

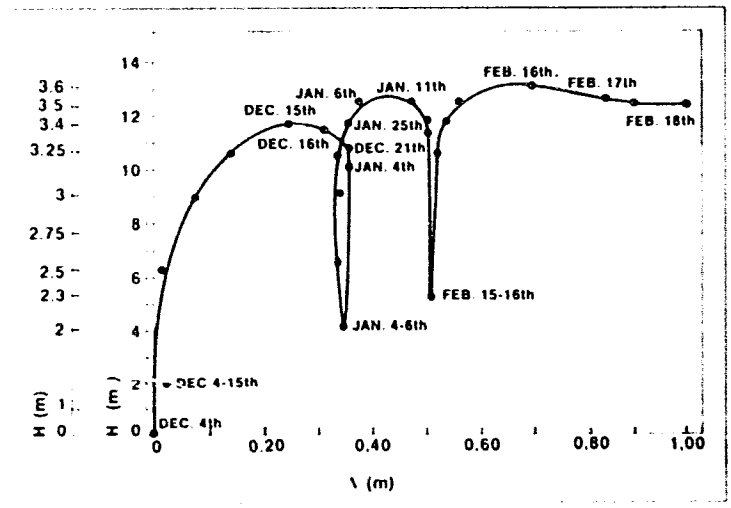
b) During the loading and unloading cycles, unloading does not substantially decrease the deformation nor does loading essentially increase it. In the course of the operation the structure was never completely unloaded (See Fig. 35). It will be recognized that these cycles undoubtedly represent the successive changes in the thrust of the soil that occur at back of a retaining wall due to the changes in the water content of the backfill and other natural conditions.

Fig. 35 - Deflection of the top of the wall at corresponding load levels during the period of test.

Fig. 38 - Average values of the deflection at various levels of the wall at selected stages of loading

Figs. 36, 37 - Gabion wall as it appeared at the end of the load test.

35



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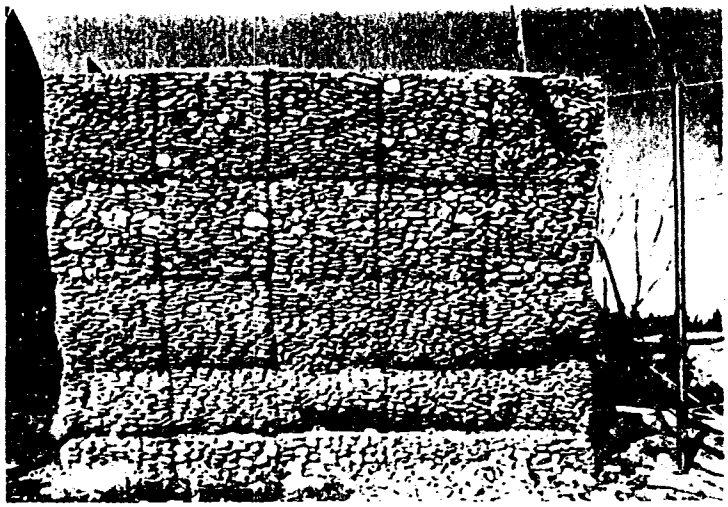

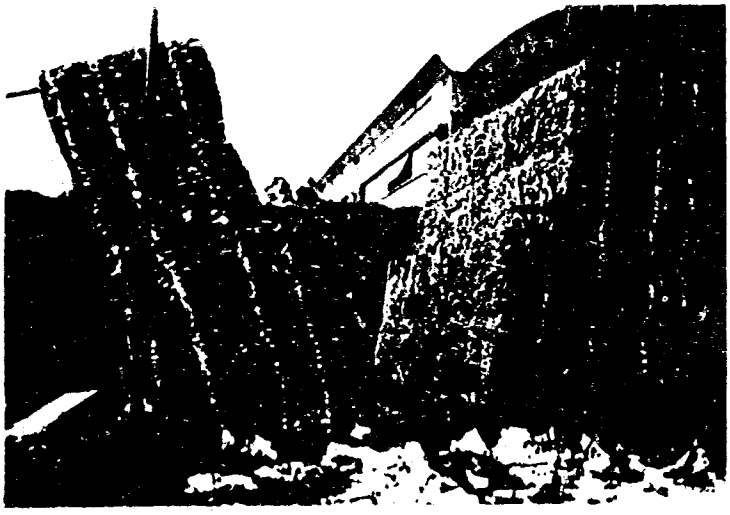

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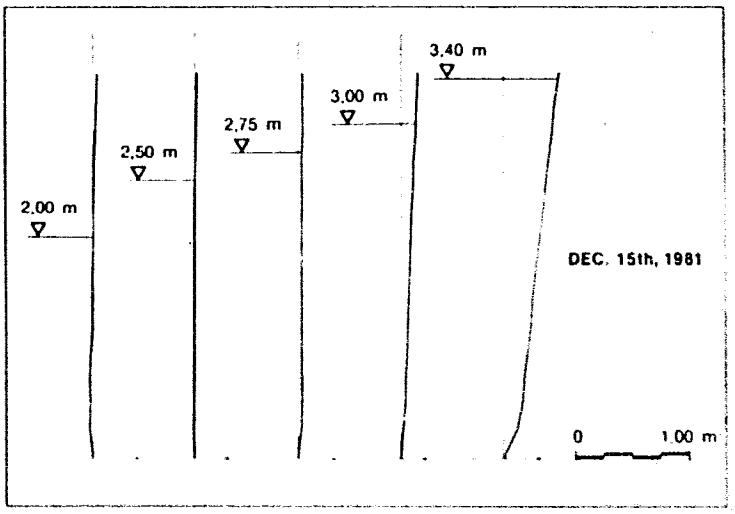

c) When the structure is subjected to a constant load over a long period, the deformation increases progressively. This is clearly shown in Fig. 35 in the periods between 15 th and 16 th December 1981 and between 6 th and 11 th February 1985 when the structure was continuously loaded.

This condition can be attributed to an unstable equilibrium which relies mainly on the internal friction between the stones of the filling material, but is also reinforced by the containing action of the steel wire mesh.

d) When the test structure had been loaded to its maximum capacity and was incapable of withstanding any additional thrust, the process of failure was very gradual and although extensive deformation occurred, there was no sudden or abrupt collapse. This confirms the supposition that gabion structures are ductile in that they can tolerate massive deformation without losing their ability to resist thrust.

The description "ductile" is somewhat imprecise in view of the complex and composite characteristics of the material, and its behaviour under load. It is however possible to speak, in a wide sense, of a "ductility coefficient". This coefficient can be considered as the ratio between the degree of deformation occurring at the point at which the structure ceases to support a load increment, and that at which the first mesh failure occurs. Using this criterion it is possible to obtain ratios of the order of 20 and higher, which would allow gabion structures to be classified as very ductile.

e) The significance of the influence of the tensile strength of the mesh on the stability of the structure was clearly demonstrated in the tests. It is illustrated graphically in Fig. 39 which shows a detail of the mesh in the tension zone at the junction between the wall and the foundation course.

Fig. 39 - Close-up of the area subjected to tensile stress at the joint between the wall and the foundation course.

Fig. 40 - Curves illustrating the deformation and corresponding shear values.

39

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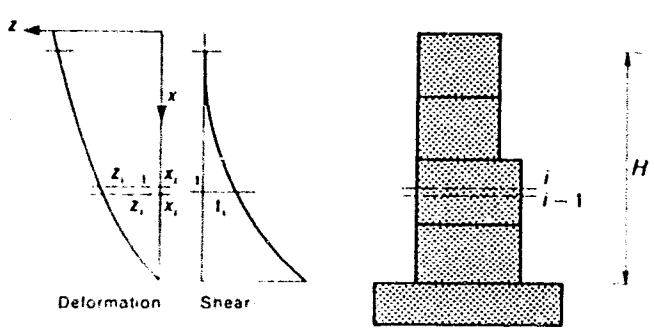


3.4.5 Observations on the test results

3.4.5.1 The non linear behaviour of gabion structures and criteria for assessing the degree of deformation. The examination of the test results disclosed a remarkable "non linear" behaviour of the material which, if ignored, could lead to a misunderstanding of the principles that determine it. On the basis of these observations and bearing in mind that the deformation mainly results from shear stress, a study was made of the relation between the shear stress and the shear modulus of elasticity, taking the test structure as an example.

The photogrammetric records give the deflection z at height x at each point on the grid reference system for each stage of loading. Consequently one can evaluate the specific shear t at any point and obtain the average shear stress τ by dividing t by the wall thickness b so that $\tau_i = t_i/b_i$.

The deflection at the point i in Fig. 40 can be approximated from the relation:

$$\gamma_i = \frac{z_i - z_{i-1}}{x_i - x_{i-1}}$$

and therefore the shear modulus at point i is:

$$G_i = \frac{\tau_i}{\gamma_i}$$

Using the numerical results extracted from the photogrammetric records, the value of τ, and the average deflection in the vicinity of the point, can be established at every grid point, and hence the value of the shear modulus G, known as the secant modulus. It has therefore been possible to obtain a relation between the shear stress τ and the shear modulus of elasticity G. It is linear as illustrated in Fig. 41 and can be expressed in the forms(*):

$$G = 5.733\ \tau + 0.13 \qquad \text{when } b = 1.5 \text{ m}$$

$$G = 5.333\ \tau + 0.44 \qquad \text{when } b = 2.0 \text{ m}$$

Hence the results show that G varies linearly with τ and that the slope of the curve expressing this relation is practically independant of the value of b which, however, establishes the initial value of G for $\tau = 0$. The curve of τ plotted against G varies with the thickness of the wall and also with the weight of the gabion mesh. Since the duration of the load can modify the slope of the curve, there are two versions of the modulus G according to the time scale, i.e.: immediately after the load is applied and over a period of time (See Fig. 35).

Fig. 42 shows a comparison between the actual deflection of the wall during the test when it was loaded to a water depth of $h = 3.40$ m on 16/12/81 and the value computed from the above expression. The computed values are remarkably close to the actual ones but the figures for deformation given by the expressions can only be regarded as indicative.

From the examination of the most significant load test results, the following conclusions may be drawn:

a) As far as gabion structures are concerned, the above expression for the relation between τ and G should only be used for obtaining rough guideline values. It should be remembered that the gabions in the test wall were manufactured from light weight mesh which deforms easily, and were not fitted with diaphragms (See § 3.4.1).

b) The simplified but perfectly tenable assumption can be made that, for gabions under shear stress of a typical value of τ, the modulus G has constant average values of between 1.5 and 3.5 kg cm^2. The selection of one or the other value depends on a variety of circumstances which the designer must consider in every individual case.

c) In view of the wide scatter of the plotted results, it was decided to extract from them criteria in general terms only. This simplified criteria has been employed to formulate some quite tenable hypotheses without resorting to elaborate mathematics.

3.4.5.2 Analysis of stress in gabion materials. The analysis of a horizontal section through a conventional retaining wall would normally cover the tension, compression and shear stresses, and the bending moment. However, in the case of a gabion structure, the analysis would only usefully serve to check that the compression stress was within the specified tolerance for the stone fill, since the steel wire mesh in every known case is always under stressed.

As an example, consider a gabion wall loaded with a head of water of 3.5 m. The density of the filled gabion is 1.8 t/m^3, the vertical component N at the base is 13.4 t, the bending moment M is 7.15 tm and the horizontal component T is 6.13 t.

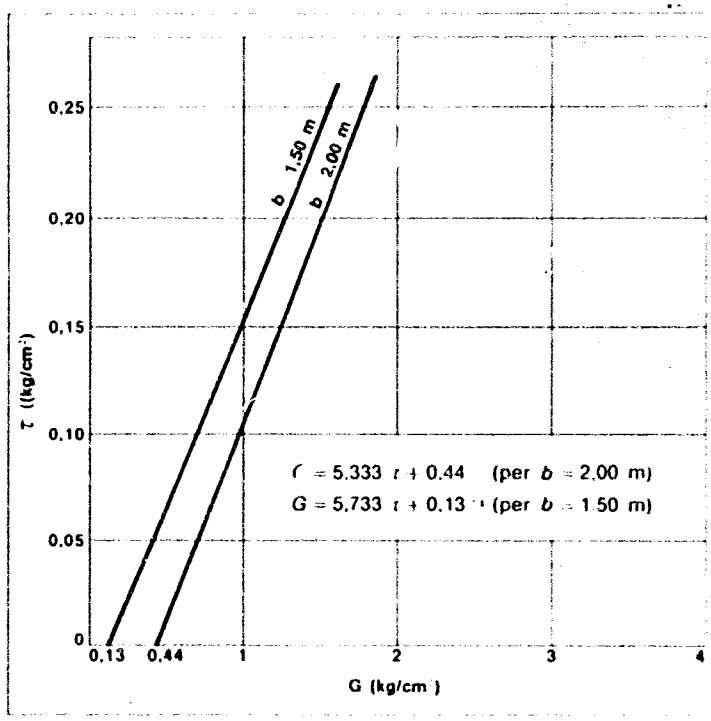
Assuming that only part of the base is carrying stress, as indicated in Fig. 43, the results would show:
$e = M/N = 0.536$ m and therefore(**)

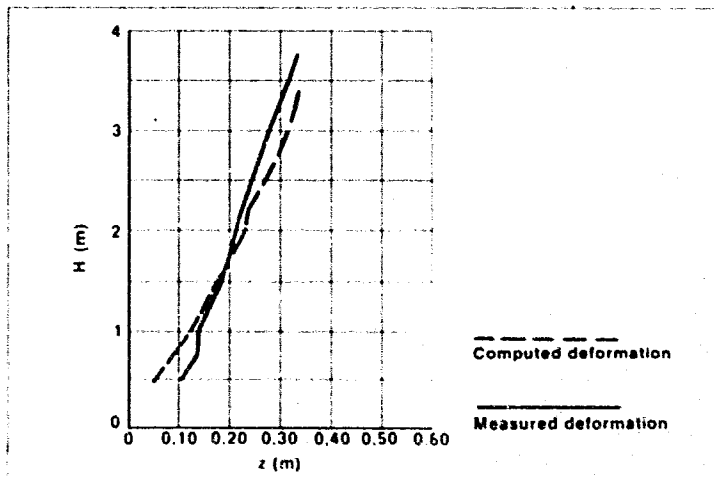
$$\sigma_{max} = 1.92 \text{ kg/cm}^2$$

$$\tau_{max} = 0.67 \text{ kg/cm}^2$$

Fig. 41 - The relationship between shear and the shear modulus of elasticity in gabion structures.

Fig. 42 - Comparison between the measured and the computed values of deformation for the wall under test.





(*) The value of G in these expressions is in kg cm^2.

(**) Compression stresses are considered as being positive in this analysis.

If it is assumed that the whole of the base is either in compression or tension as in Fig. 44, the results change to:

$$\sigma_{max} = 1.74 \text{ kg/cm}^2$$

$$\sigma_{min} = -0.405 \text{ kg/cm}^2$$

$$\tau_{max} = 0.46 \text{ kg/cm}^2$$

In this case the bearing pressure is low but in any case it must not exceed that allowable for an average compacted gravel which may be taken as equivalent to gabion filling material (See Section 4.4.1).

The tension on the mesh is usually low. The stress which it has to resist can be simply computed as being equivalent to the resultant of the tensile stress acting on the whole of the section. In this case, the part in tension covers a width of 0.38 m and the resultant tensile stress is: $0.5 \times 4.05 \times 0.38 = 0.76$ tonnes per metre length, which is well below the strength of the mesh (See § 3.3).

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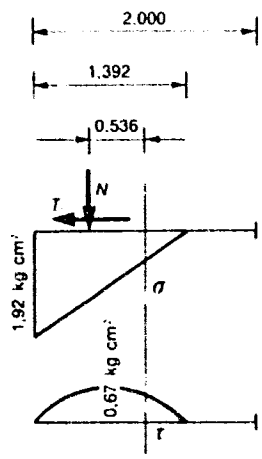

Fig. 43 - Characteristics of the external loads and internal stresses acting on the base section of the wall loaded with a 3.5 m depth of water, assuming that only part of the base is in compression.

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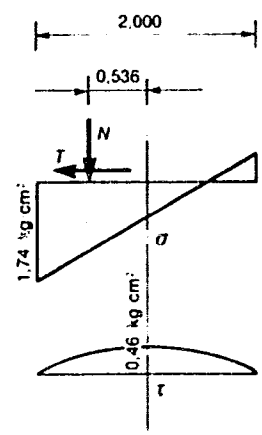


Fig. 44 - Characteristics of the external loads and internal stresses on the base section of the wall loaded with water to a level of 3.5 m assuming that part is in compression and part in tension.

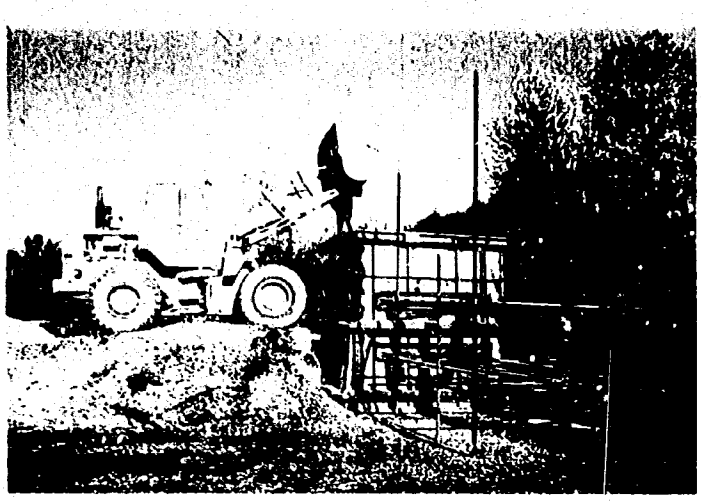
3.5 Experiments on the use of wire mesh panels in soil reinforcement and as earth anchors

The final tests on gabion construction materials were made to investigate the use of panels of mesh as anchorage elements. A conventional gabion wall, in certain situations, can be replaced by a simple thin outer face tied into the backfill at one or more vertical intervals by panels of mesh, or thin gabion mattress units, anchored in the soil. To establish the necessary anchorage length, an experiment was set up to measure the minimum required to cause tensile failure of the mesh rather than that to extract the panel intact from the soil. A test was also made to establish the relation between the applied traction and the yield point of the anchors.

Only the basic essentials of the collected data is presented below. Nonetheless, it should be sufficient to provide useful guidelines for designers.

The test equipment consisted of a steel box shaped as a bottomless truncated pyramid $3.00 \times 3.00 \times 3.50$ metres in size and having horizontal slots cut in two opposite faces at two levels (See Fig. 45). The procedure for testing was as follows: soil was placed in the boxes to the level of the bottom of the slots, and panels of mesh laid over it so that their ends protuded through the slots (See Figs. 46 47). Further soil was then added to a predetermined height. To find the anchorage resistance, traction was applied at the end of the panel and measured by a dynamometer.

The load was applied to the panel via two steel clamps which gripped the mesh over its full width to ensure a uniform distribution of stress. Traction was measured by a dynamometer with digital controls which had been designed for the purpose. It had a maximum capacity of 12,000 kg and



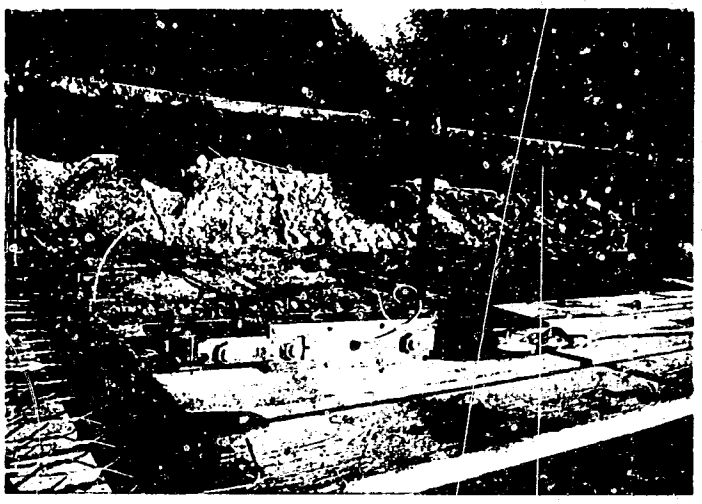


Fig. 45 - Loading soil into the steel box used for extraction tests.

Fig. 46 - Mesh panels in the steel testing box prior to placing the surcharge soil

Fig. 47 - A mesh panel protruding from a slot and gripped by the steel clamp which ensured a uniform distribution of load.

Fig. 48 - The dynamometer used to measure the traction applied to the mesh panel via the steel clamp.

a measurement accuracy of 4 kg. (See Fig. 48). The lengths of the test samples were taken before and after testing to measure the elongation.

Parameters which were taken into consideration for the purposes of the tests were the mesh type, the direction of the weave of the mesh with reference to the direction of pull, the length of the panel buried in the soil, and the type and depth of soil over the sample. The types of mesh that were tested were the 8 × 10 in 3.00 mm diameter wire, and the 6 × 8 in 2.2 mm diameter wire.

The mesh was tested by traction parallel to, and perpendicular to the direction of the weave. Obviously it is important to know in which direction the mesh should be laid to obtain the optimum anchorage value. Further information on this subject can be found in Section 3.3 and 4.6.2.3

Tests were also made with panels of the same specification of mesh but PVC coated. No substantial differences were recorded. Panels of electrically welded mesh were also tested. It was found that they had low ductility and tended to fail suddenly and instantaneously over the full width of the panel. When compared to hexagonal woven mesh which fails gradually in stages, it would seem that it is less suitable for this particular purpose.

The width of the panels used in the tests was 2.00 m and the encased lengths were 1.00, 2.00 and 3.00 m. The soil surcharge depths over the panels were 1.00 and 2.00 m.

The soil types which were tested were naturally occurring gravel, crushed stone 30 to 50 mm in size, and a medium grade of sand.

The soils in the test were not compacted and therefore it can be assumed that if the backfill is compacted either mechanically or by a superimposed surcharge, the anchorage value will be greater than that achieved in the tests.

Although it is stated above that the object in the tests was to achieve anchorage failure by rupture of the mesh rather than by its extraction from soil, there were some failures of the latter kind (See Figs. 49, 50).

Figs. 51, 52 and 53 show the results of the tests on the 8 × 10 mesh. S is the tension in the mesh in kilogrammes per metre width, occurring at the commencement of movement in the case of extraction, ($S < 5000$ kg/m), otherwise at the point of rupture, ($S > 5500$ kg/m). The graphs illustrate the relation between S and L_a, the anchorage length, for two values of the soil depth H above the sample.

The commencement of movement of the panel through the soil was noted by keeping the mesh protuding through the rear of the box under observation, and measuring the traction at the point when it started to move. Fig. 54 shows values of S for the 8 × 10 mesh plotted against V the volume of soil superimposed on the panel in cubic metres per metre width.

As the graphs show, the values of anchorage obtained in

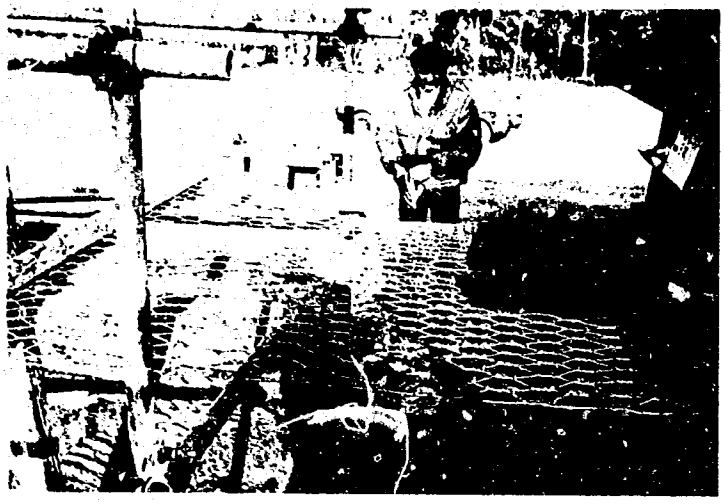


Fig. 49 - Panel extracted intact. The anchorage length was insufficient in this case.

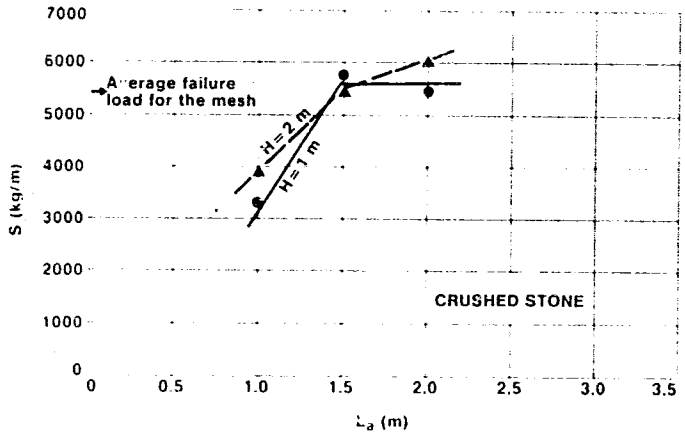
Fig. 50 - Panel failure caused by a load exceeding its tensile strength.

gravel and crushed stone are similiar, and it can be concluded that a 2.00 m anchorage length is sufficient to develop a resistance against extraction which is at least equal to the strength of the mesh. An exception is sand, particularly where the depth of surcharge is small, when a 3.00 m anchorage length applies. The general rule is that the width of the panel must never be less than 2.00 m.

It is interesting to note that the failure values were nearly equal to the breaking strength of the mesh in the cases where the anchorage was adequate, but where the mesh failed (See Fig. 33). An exception, mentioned above, was the test in sand at a surcharge depth of 1.0 m in which failure occurred at a much lower load. This may be due possibly to the lack of compaction provided by only one metre of sand, this being insufficient to prevent the mesh from contracting laterally. This condition does not occur in soil of a larger particle size and it would certainly improve as the surcharge of sand is increased.

Further observations and guidelines for the design of anchorages using panels of mesh can be found in section 4.6.2.3.

Fig. 51, 52, 53 - Diagrams showing anchorage strength S as a function of anchorage length l



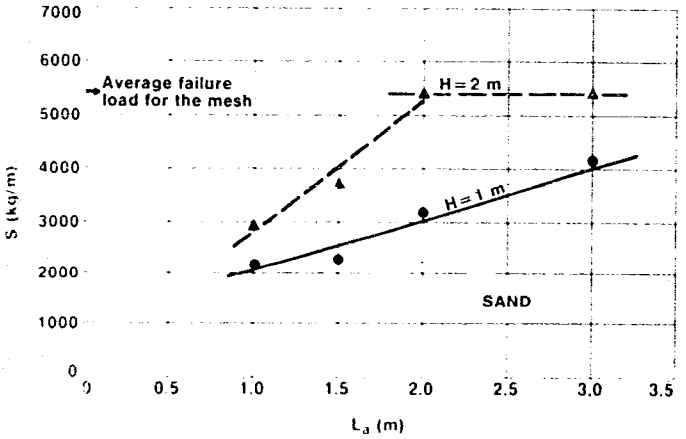
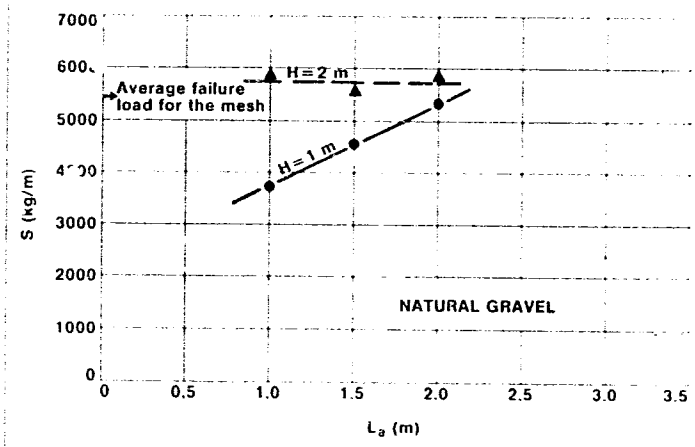
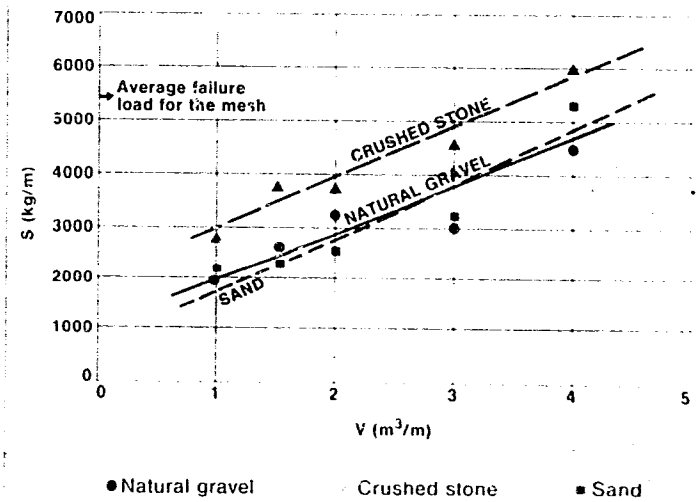


Fig. 54 - Diagram showing the anchorage strength S as a function of the volume V of soil surcharge over the panel.





CHAPTER IV

Computation criteria

4.1 Introduction

In the present chapter the criteria, useful for designing gabion retaining structures, will be reviewed. If the behaviour of gabion structures were similar to that of concrete structures there would be no need for such a review, for in that case it would be sufficient to refer the reader to the existing technical literature.

As it is, the peculiar characteristics of gabion structures require various refinements with regard to both the most appropriate criteria for computing the loads transmitted by the soil to the retaining wall and the criteria for verification of the adequacy of the structure and its safety factors.

In writing this chapter therefore the most recent technical literature on retaining structures and on experiments carried out on gabion elements and on full size structures was taken into account.

Theoretical and experimental references useful to a better understanding of the text or for a more thorough study of the subject may be found in the bibliography [1, 2, 5, 8, 16, 17, 18].

Gabion type retaining structures, like any other retaining structures, require:

a) An analysis of the loads transmitted by the soil to the structure;

b) A stability analysis of the combination of the wall and surrounding soil as a whole.

c) A structural analysis and design of the wall and soil foundation;

d) An evaluation of the potential movements and deformations.

First of all in order to carry out these computations the mechanical properties and engineering characteristics of both the foundation soil and the soil being retained must be known. It is the responsibility of the project designer to make an adequate geological investigation which will conform with the requirements of the competent authorities and existing codes of practice. The degree of sophistication of the investigation will depend on the merits of the project and the required refinement of computation.

For a better understanding of what follows it is necessary to posses an adequate knowledge of Soil Mechanics, and the reader should refer to the technical literature and to the mechanical characteristics and strength of gabion structures (See Chaps. II e III).

4.2 Loads imposed by the backfill

4.2.1 General

The loads imposed by the soil on a retaining wall depend not only on the mechanical characteristics of the soil, defined by its internal angle of friction φ and/ or cohesion c, but also on the extent to which the structure can deform.

It is known that the active pressure in an earth retaining structure can only develop when the retaining structure has deformed sufficiently. Table 6 lists the lower limits of movement necessary to bring into action the active pressure.

It follows that in the case of very rigid retaining walls, pressures are developed in excess of the active pressure, whereas with flexible walls the soil is mobilized at its lower limit of equilibrium and it exerts the active pressure over the

Table 6 - Minimum displacement and rotation required to develop the active pressure in retaining structures.

Type of soil	Deflection δ	Rotation θ
Non cohesive	0.001 H	0.001
Cohesive	0.004 H	0.004

entire height H.

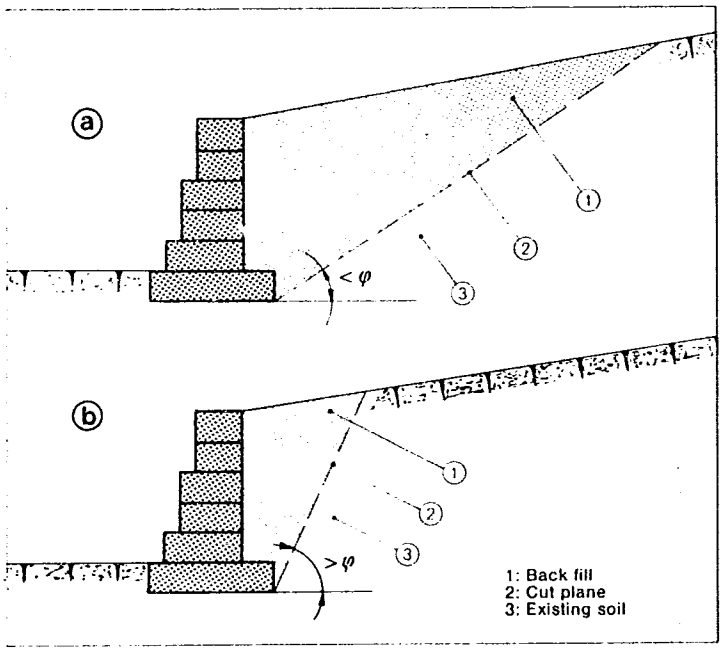
It is also worth noting that where impervious walls have no weep holes, it is, generally speaking, inadvisable to rely on the cohesion factor (5) as it can diminish rapidly with the rise of the water content. For instance this can occur after heavy rainfall where drainage facilities are inadequate. Gabion walls on the other hand provide through drainage and it is permissible to take cohesion into account..

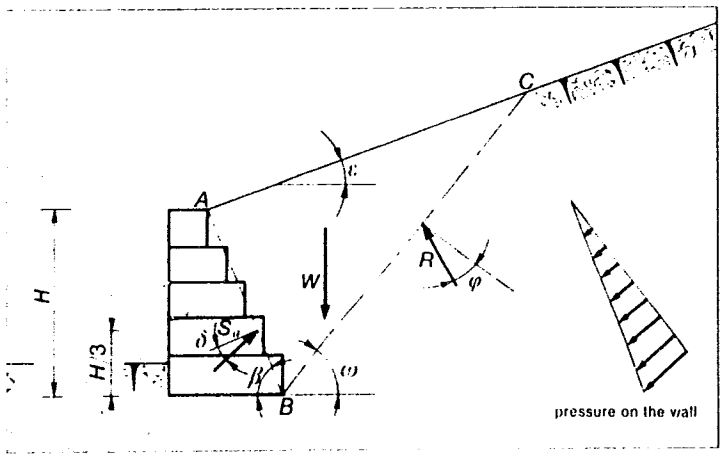
Active pressure is usually computed with reference to the well known Coulomb Wedge theory, and a typical example of calculation for a general type of wall loaded with a given soil as seen in Fig. 55 is considered below.

If the excavation behind the wall is made in such a way as to assure the stability of the cut (Fig. 55a), the pressure on the structure is due essentially to the fill only and therefore it can be computed on the basis of the φ and c of the fill.

Fig. 55 - Schemes for the backfilling of gabion retaining structures.

Fig. 56 - Forces exerted on the gabion retaining structure according to Coulomb's theory.





On the contrary, if the excavation is made with too steep a slope (Fig. 55b) and the stability of the excavated face during construction is dependent on friction and/or cohesion, it is then appropriate to compute the loads using φ and c characteristics of the weaker of the soils, the naturally occurring or the imported.

The Coulomb theory (1776) is still substantially valid and it is based on the following principal assumptions:

a) The surface of rupture of the soil behind the wall is a plane surface;
b) The friction forces are uniformly distributed over the rupture surface;
c) The soil included between the wall and the surface rupture is considered a non deformable mass;
d) The load imposed by the wall on the soil acts at an angle δ with the perpendicular to the soil-wall interface and in such a direction as to oppose the motion;
e) The problem is two dimensional in the sense that a unit length of retaining wall is analyzed and the wall is assumed to be of unlimited length.

4.2.1.1 Cohesionless soils. Assuming the conditions just described and with the additional assumption that the wall is sufficiently flexible, a condition that is always the case with gabion structures, it can be shown that the total thrust S_a exerted by a cohesionless soil of unit weight γ_t on a wall of height H is

$$S_a = \frac{H^2 \gamma_t K_a}{2}$$

where the symbols are those illustrated in Fig. 56 and K_a is:

$$K_a = \frac{\sin^2(\beta + \varphi)}{\sin^2\beta \sin(\beta - \delta)\left[1 + \sqrt{\dfrac{\sin(\varphi + \delta)\sin(\varphi - \varepsilon)}{\sin(\beta - \delta)\sin(\beta + \varepsilon)}}\right]^2}$$

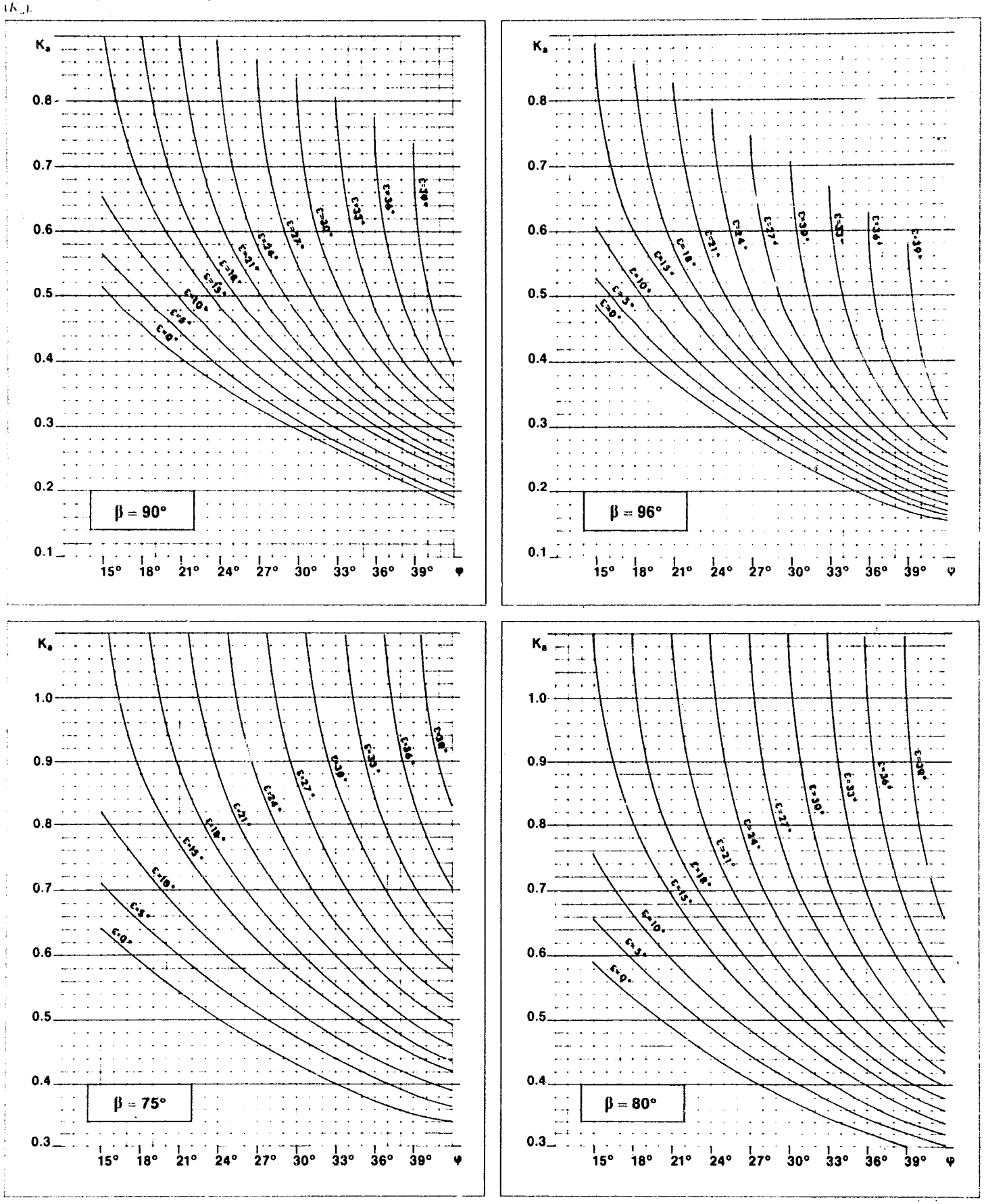
The coefficient K is called the coefficient of active pressure. The resultant of the active pressure is applied at $H/3$ from the base and its direction makes an angle δ with the perpendicular to the wall. In gabion walls δ can be assumed equal to φ, ($\delta = \varphi$) due to the considerable roughness of the gabion surface, which can therefore be assumed to be a soil to soil friction surface.

The diagrams in Fig. 57 show the value of K_a for various values of the friction angle φ and for various inclinations of the thrust surface β and the soil surface ε, thus permitting the immediate computation of the thrust.

For stepped rear face walls the surface where the soil pressure is assumed to be applied is the plane connecting the points A and B in Fig. 56.

Rankine's theory for the computation of earth pressure (1857) deals with the condition of the plastic equilibrium of

Fig. 57 - Diagrams for the computation of the coefficient of active pressure (K_a).



the soil, without consideration of the friction between wall and soil, thus obtaining:

$$K_a = \cos\varepsilon \frac{\cos\varepsilon - \sqrt{\cos^2\varepsilon - \cos^2\varphi}}{\cos\varepsilon + \sqrt{\cos^2\varepsilon - \cos^2\varphi}}$$

The thrust is assumed to be inclined at an angle ε from the horizontal.

Coulomb's theory in the case where $\beta = 90°$ and $\delta = \varepsilon$ produces the same expression as Rankine's theory (Fig. 58). In addition where $\delta = \varepsilon = 0°$, the value of K_a becomes:

$$K_a = \tan^2(45° - \varphi/2)$$

The angle ω (Figs. 56, 58) defines the inclination of the rupture surface that corresponds to the active pressure S_a. Its general expression is found for instance in reference [18]. In practice it is possible to assume:

$$\omega = 45° + \varphi/2$$

4.2.1.2 Cohesive soil. With silty soils and/or clay soils a limited amount of cohesion can be assumed.

In such a case the active pressure can be computed as:

$$S_a = \frac{\gamma_t H^2 K_a}{2} - 2cH\sqrt{K_a}$$

4.2.1.3 Observations. Some clarification will be helpful when applying the above theories, since, as will be seen later, the diversity of practical situations requires further approximations, which experience shows to be generally acceptable.

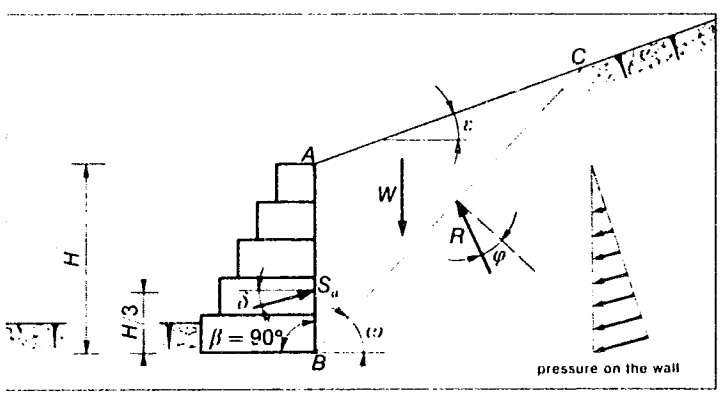
In the theories just reviewed the distribution of the earth pressure exerted by the soil on the structure is assumed to be linear, whereby in the absence of cohesion and surcharge, the resultant pressure acts at a height $H/3$ from the base.

Fig. 59 schematically illustrates three different types of walls frequently used in practice. In case a) the wall surface against which the soil pressure is exerted is precisely the wall surface against which the soil rests.

In case b) the earth thrust can be computed along the imaginary section b-b as shown by the broken line and the mass of the soil included between the inside face of the wall and section b-b is the mass that acts on the heel of the wall, and is computed in the overturning and sliding analysis of the wall.

Case c) concerns a wall whose footing is exceptionally long and consequently rendered excessively flexible. In such a case it is not possible to rely on the bending strength of the footing and the computation of the earth thrust is made with reference to the ideal section b'-b' at a distance back from point B equal to or less than the footing.

The smaller portion of the footing set off by section b'-b' behind the wall contributes to the translational equilibrium and can be interpreted as an anchorage element.



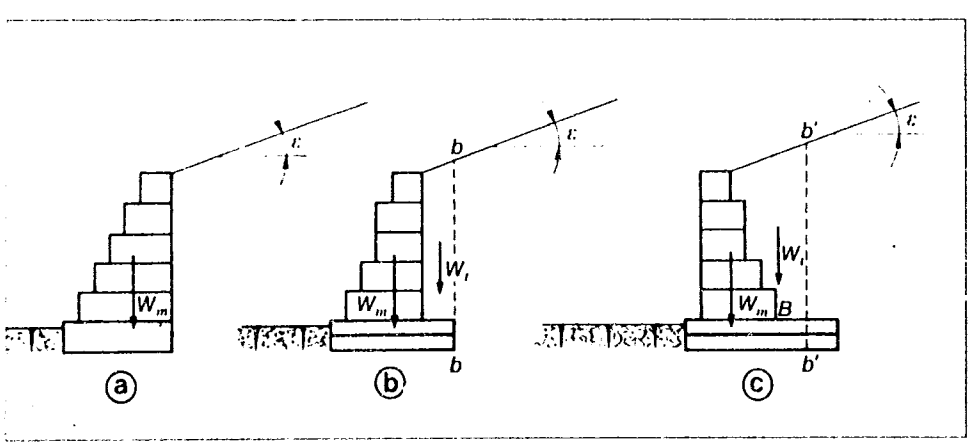


Fig. 58 - Forces exerted on gabion retaining structures according to Rankine's Theory.

Fig. 59 - a) Gabion gravity wall; b) semi-gravity gabion wall; c) semi-gravity gabion wall with anchorage heel.

4.2.2 Earth pressure due to surcharge

The thrust from a distributed surcharge load on the horinzontal backfill (Fig. 60) is easily computed. In practice:

$$S_a = p_o H K_a$$

Where the backfill soil is homogeneous the contact pressure between soil and wall due to the surcharge is uniformly distributed, and the resultant is applied at $H/2$.

In cases where retaining walls support elevated roads it is common practice to consider the uniform surcharge load p_o above the wall as a layer of soil of weight equivalent to the given pressure and of thickness $H_s = p_o/\gamma_t$. The effective thrust against the face of the wall due to the surcharge and the weight of the cohesionless soil can be expressed as:

$$S_a = \frac{\gamma_t H^2 K_a}{2}\left(1 + \frac{2H_s}{H}\right)$$

and the line of action S_a intercepts the back surface of the wall at a height d from the base, where:

$$d = \frac{H}{3}\left(\frac{H + 3H_s}{H + 2H_s}\right)$$

The evaluation of the thrust due to nearby concentrated loads or loads along a strip of ground parallel to or at an angle to the line of the wall is more complex as are the loads transmitted by vehicles and some buildings etc.

Such cases can be handled in a simplified manner (see § 4.2.3), or the soil can be assumed to behave as an elastic homogenous and isotropic medium (Spengler and Gerber, Terzaghi). The expression that can be obtained can be found in references [1, 2].

4.2.3 Evaluation of earth pressure in complex loading conditions

Often in practice the st face of the ground above the wall is irregular, as shown for instance in Fig. 61.

In such cases it may be convenient to refer to the limiting conditions and making conservative assumptions. A more precise analysis can be made using the Cullman method which is based on substantially the same assumptions as Coulomb's. This method seeks the maximum value of the reaction which the wall must provide in order to guarantee the equilibrium of the wedge of moving earth.

For a detailed discussion of the Cullman method the reader is referred to the specific technical literature [1, 2]. In chapter V the computation of the thrust acting on a wall supporting a road, a case in which this method is frequently used, is presented as an example.

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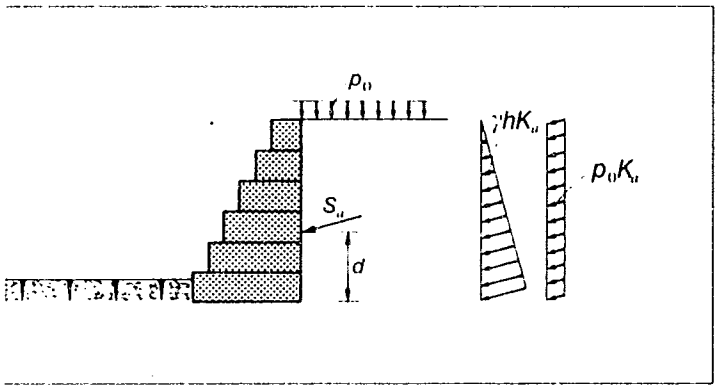

Fig. 60 - Lateral soil pressure distribution due to a uniform surcharge.

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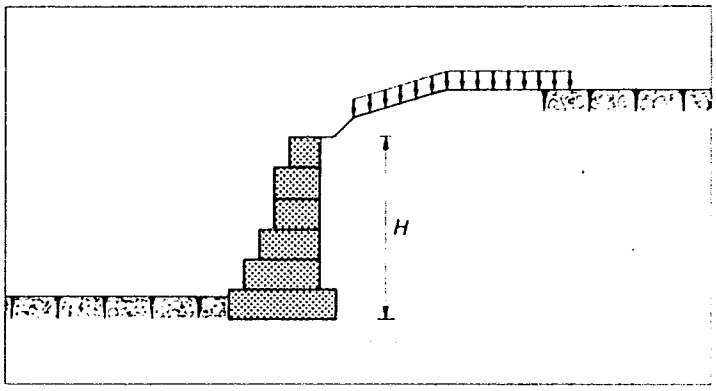

Fig. 61 - Example of a wall with a surcharge non-uniformly distributed.

4.3 Criteria for the stability analysis of gabion walls

Whatever the type of the retaining structure, whether gravity, semigravity, or reinforced soil (see Chap. II and § 4.2.1 c) it is usually necessary to check the stability of the work with respect to:

a) sliding
b) overturning
c) overall stability of the wall and surrounding soil.

It is also necessary to check the stress developed at critical points of the wall (§ 4.4.1) and the soil bearing pressures (§ 4.4.2), and it may be useful to check potential wall move-

4.3.1 Check for sliding

This check is made with reference to an horizontal plane (Fig. 62). Specifically, for gabion walls, the stabilizing forces (F_s) resisting sliding are friction (fN) and cohesion (cB) at the sliding surface, passive pressure (S_p) at the toe of the wall and anchorage forces (S_r) at the heel of the wall:

$$F_s = fN + cB + S_p \cos\delta + S_r$$

Some of these components may not be present depending

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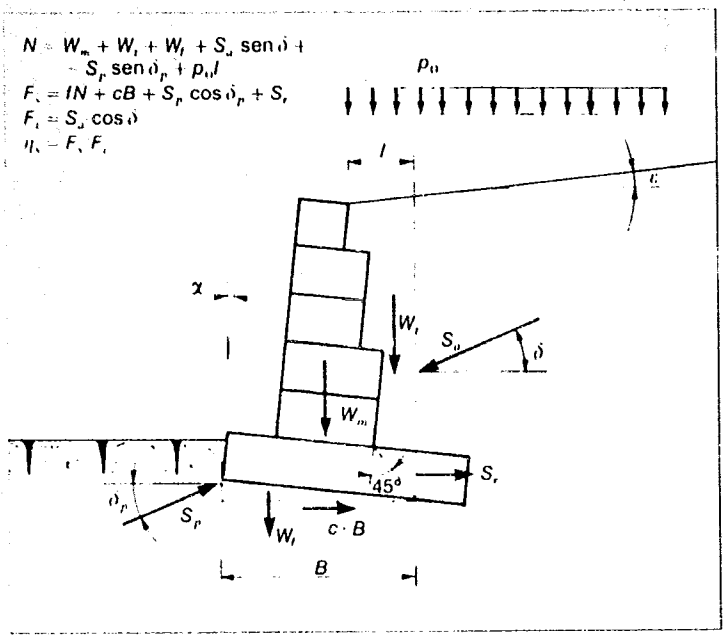

Fig. 62 - Forces to be considered in analysing the resistance to sliding.

Fig. 63, 64 - Tests to determine the coefficient of friction between gabions and soil.

Fig. 65 - a) Increased foundation depth; b) widening of the foundation

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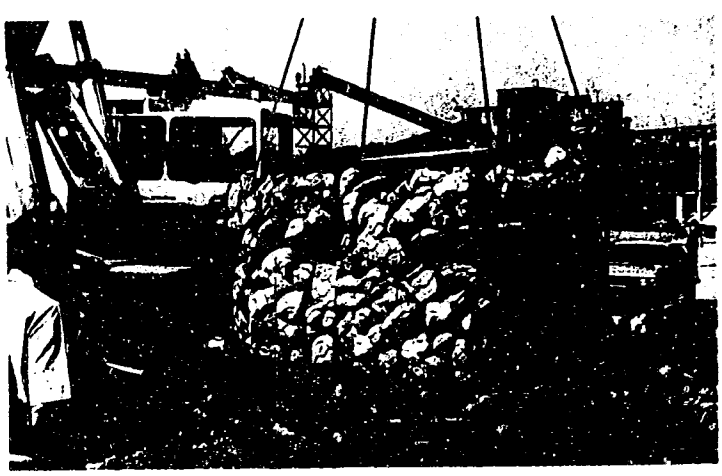

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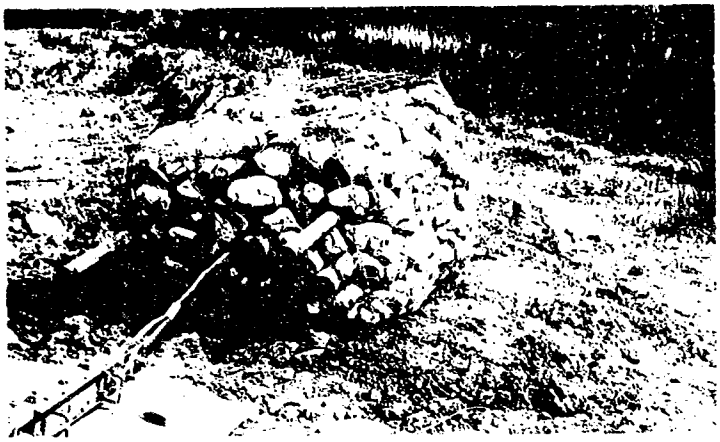

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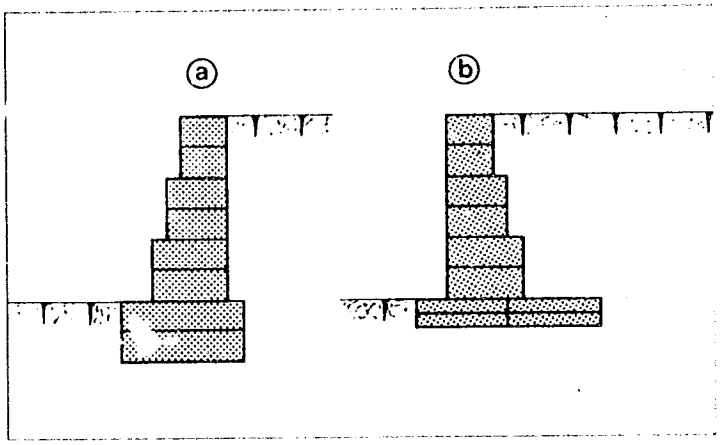


on the type of wall and existing conditions. The normal force N is the sum of the vertical forces perpendicular to the sliding surface i.e. soil weight, wall weight, vertical component of the soil thrust, and surcharge and f is the coefficient of friction between soil and footing or between soil and soil; c is the cohesion factor, and B is the length of the sliding surface (Fig. 62). For a computation of these contributing factors refer to Chapter V, "Examples of Computations".

With regard to the coefficient of friction, specific experiments were carried out to determine what values should be used for gabion walls, sic: (Rio Reventado, Instituto Constaricense de Electricidad 1965 [19]; Bologna Officine Maccaferri S.p.A. 1985 Figs. 63, 64). For cohesive soils the ratio that was obtained between the horizontal thrust and the filled gabion was rather high, 0.7 to 0.75, but since the contribution of cohesion needs to be taken into account in these cases, the resulting coefficient of friction is lower. In cohesive soils values are slightly higher than the traditional tan φ, where φ is the angle of internal friction of the soil. In the analysis of the resistance to sliding at the base of a gabion wall, it is reasonable to use:

$$f = \tan\varphi$$

since it can be assumed that sliding occurs on a soil to soil face and not at the gabion to soil interface.

Conservative values of cohesion on the sliding plane may be used in addition, in the case of cohesive soils. In the experiments referred to above, the kinematic coefficient of friction between gabions and artificial surfaces was determined. For concrete surfaces the coefficient of friction was determined to be 0.64.

The horizontal thrust tending to produce sliding is:

$$F_i = S_a \cos\delta$$

The safety factor against sliding is η_s

$$\eta_s = F_s / F_i$$

Most authorities recommend that this safety factor should not be less than 1.5, while others that it should be 1.3.

In any case the selection of the factor of safety is usually a matter of judgement where the merit of the work, the degree of refinement sought in the computations, and the accuracy with which the soil characteristics are known all need to be taken into consideration. It should be noted that in this respect clay soils are the most treacherous on various counts, mainly because their characteristics vary significantly with change in water content.

To design walls with adequate safety factors against sliding is often difficult especially where the foundation soils have low friction factors.

In these cases it is sensible to consider a deeper foundation as shown in Fig. 65a or a longer one as in Fig. 65b.

4.3.2 Check for overturning

For a retaining wall that is resisting active earth pressure and where its own mass is a resisting force the overturning moment (Fig. 66) is:

$$M_i = S_a l_a$$

and the restoring moment is:

$$M_s = W_{tot} l_w + S_p l_p$$

where W_{tot} is the resultant of the wall's weight and the weight of the "boxed" soil above the heel and l_a, l_w, l_p, are the lever arms for the forces S_a, W_{tot}, and S_p respectively.

The safety factor against overturning is given by:

$$\eta_r = M_s / M_i$$

and its minimum value is usually taken to be 1.5.

In this analysis it is assumed that the foundation slab has adequate rigidity and for gabion walls this usually means that the minimum thickness of the heel and toe is equal to or greater than their respective widths.

In cases where the foundation section is wider than usual it is necessary to assume a hinge point as shown in Fig. 67.

In addition since the strength of the heel in bending is negligible, only the "boxed" soil lying on a length of the heel equal to its thickness can be assumed when computing the restoring moment. The remainder of the heel can be considered effective as an anchorage element in the computation of the resistance to sliding.

When computing the anchorage effect at Section *b-b* only the horizontal top and bottom panels of gabions are considered to be effective. The vertical panels carry the anchorage stress developed between Sections *a-a* and *b-b* and in conjunction with the interlocking action of the stone fill they also transfer the shear and bending stresses at Section *a-a* to the main structure.

4.3.3 Check on overall stability

A retaining wall may fail on a semi-circular slip surface located within the soil below and behind the wall.

The analysis for this condition consists in searching for the slip circle, i.e. that circle which has the smallest safety factor against failure.

The true shape of the slip surface is complex but in practice a number of simplifications can be used to reduce its complexity to a simple geometry of linear, circular and logarithmic spiral curves.

For a discussion on the general methods of solution the reader should refer to the bibliography. In the special case where the fill behind the wall consists of cohesionless soil the failure surface can be assumed to be a plane inclined at an angle ω with the horizontal. This surface is assumed to pass through point A, the lowest extreme point on the heel of the foundation as shown in Fig. 68. To find the failure plane i.e. the one with the lowest factor of safety against sliding, the angle ω is made to vary.

The condition for stability is $S < S_p$, where S is the thrust along plane OT and S_p is the passive resistance corresponding to the limiting equilibrium condition of the prism TOC'.

For every ω there needs to be found a W, the combined weight of the soil and wall in the element CTE. This weight is a force of known magnitude and direction. Also for every ω the direction of the resultant R of the combination of soil and wall on side CT can be found by noting that in its limiting condition R makes an angle φ with the perpendicular to CT.

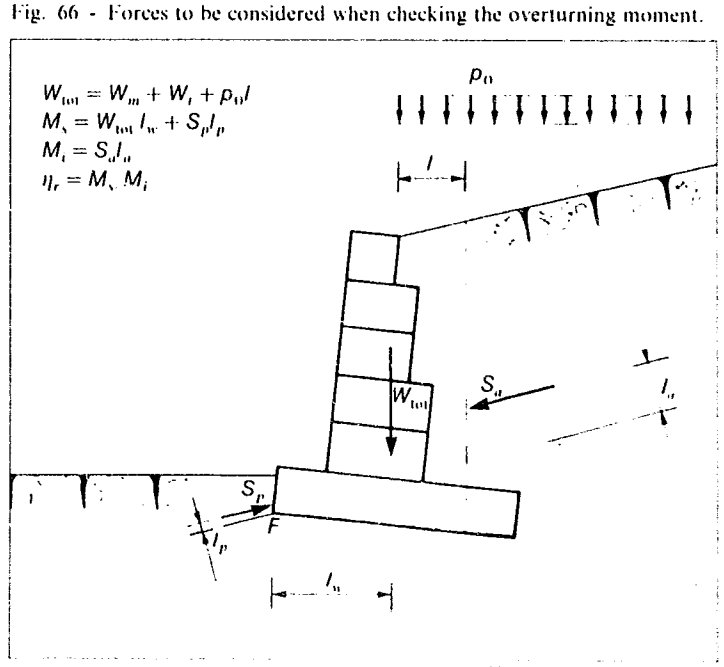
Knowing that the direction of the force S along side OT of the mass under consideration makes an angle δ with the horizontal, it is possible to determine the magnitude of S by using a simple force polygon. Finally it is possible to determine the intensity of the passive pressure S_p, for which the reader should refer to [1, 2] and Chapter V.

The critical surface for the whole mass is the one with the smallest safety factor:

$$\eta_i = S_p / S = \min$$

To ensure stability the minimum value of η_i to be used is 1.2 to 1.3.

Fig. 66 - Forces to be considered when checking the overturning moment.



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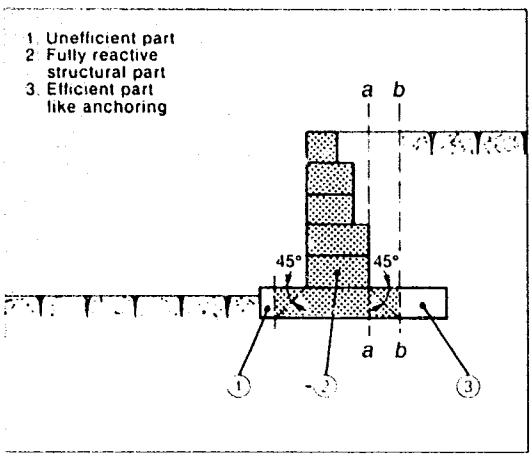

Fig. 67 - Diagram illustrating the structurally active portions of a gabion retaining structure.

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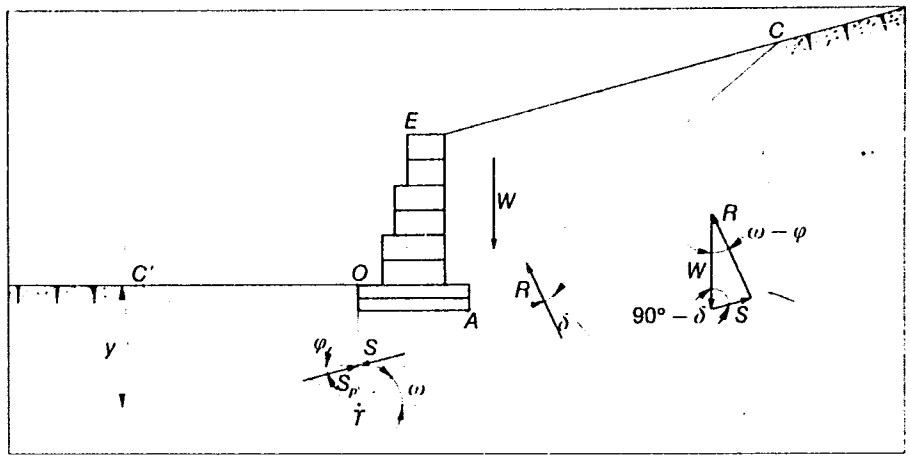


Fig. 68 - Diagram for the analysis of overall stability.

4.4 *Analysis and design of the wall and its foundation*

4.4.1 Wall design

It is generally necessary to check the design of the wall and its foundation at every change in the cross section.

The bending moment M, the shear stress T, and the normal stress N are the values to be checked at a typical cross section.

The bending moment and the shear stress are functions of the horizontal thrust of the soil and of the eccentricity of the perpendicular resultant. The perpendicular resultant is a function of the weight of the wall, the weight of the soil resting on the foundation heel and toe, and of the vertical component of the active soil pressure (Fig. 62).

Knowing all the external forces acting on a wall section it is possible to compute the internal stress induced in the wall.

The eccentricity of the perpendicular forces on a wall section of height B and of unit width is computed as:

$$e = \frac{M}{N}$$

Due to the tensile strength of the gabion structure and its adaptability to plastic readjustment, it is possible to assume that the maximum tension developed on a given section of wall for any value of eccentricity is:

$$\sigma_{max} = \frac{N}{B - 2e}$$

This expression can be obtained by making the following assumptions: the section reacts partially when $e > B/10$, the length of the effective resistant part of the section is $x = (B/2 - e)\,0.4$ and the maximum stress is $\sigma_{max} = N/(0.8\ x)$. The values of the maximum stress thus obtained are smaller than those obtained using the expression $\sigma_{max} = (N/B)(1 + 6e/B)$.

The maximum computed normal stress (σ_{max}) must not exceed the allowable stress (σ_{am}) which is a function of the density γ_g of the gabions, i.e. the type of rock fill and its compaction, (See Table 7).

The limits suggested are for the purpose of controlling the deformation of the wall rather than for safety reasons. In fact with higher normal stresses the deformation may be excessive. When the stress increases to twice the suggested value, instantaneous settlements of the order of 5% can be expected and long term settlements can reach up to three times this value.

In an approximate fashion the developed shear stress can be expressed as:

$$\tau = \frac{T}{B}$$

The allowable shear stress is:

$$\tau \leqslant \sigma \tan \varphi^* + c_g$$

Table 7 - Allowable compression stresses on gabion structural elements.

γ_g (t/m³)	σ_{am} (Kg/cm²)
1.4 1.6 1.8	4 5 6
Linear interpolation can be used for intermediate value: $\sigma_{am} = 5\gamma_g - 3$	

where σ is the average normal stress at the height of the section under analysis, φ^* is the internal "fictitious" angle of friction of the aggregate, deduced from the angle of friction of the soil in order to take into account the effect of the compaction of the gabion fill; φ^* is related to the density γ_g (t/m^3) of the filled gabions by the empirical expression:

$$\varphi^* = 25\ \gamma_g - 10^\circ$$

The term c_g represents the overall cohesive effect of the wire mesh on the gabion structure. The value of c_g depends on the ratio of weight of mesh to the volume of the gabion structure, and it increases as the gabion depth decreases, with gabions fitted with diaphragms, and with gabions constructed of heavier mesh. Conversely the value of c_g decreases with the increase in gabion thickness, with gabions without diaphragms, with large mattress type gabions (See reference [9]), and with gabions made of lighter mesh.

For example, for reference purposes, for gabions $4 \times 1 \times 1$ m in size, without diaphragms and with 8×10 mesh of wire diameter 2.7 mm, c_g is 0.15 kg/cm^2; and for gabions $2 \times 1 \times 0.5$ m in size, with diaphragms and with 8×10 mesh of wire diameter 3.0 mm, c_g is 0.40 kg/cm^2. It is possible to compute the representative c_g value using the empirical espression:

$$c_g = 0.03\ P_u - 0.05$$

where P_u is the weight of the metallic mesh per cubic metre of wall [9] expressed in kg/cm^3 and c_g is expressed in kg/cm^2.

The limits of the shear stress are also fixed in order to control the degree of deformation rather than for reasons of safety.

4.4.2 Check on foundation bearing pressure

Soil stresses transmitted locally at the base of the wall need to be checked, especially in the case of relatively high retaining walls and where the angle of internal friction of the soil is low.

The ultimate pressure can be computed using Hansen's expression, see Fig. 69 and references [1, 2]:

$$p_{\lim} = cN_c d_c + qN_q d_q i_q + \frac{1}{2}\gamma_t B N_\gamma d_\gamma i_\gamma$$

where:

$$q = \gamma_t y$$

$$i_q = 1 - 0.5\ T/N$$

$$i_\gamma = i_q^2$$

$$d_c = d_q = 1 + 0.35\ y/B$$

$$d_\gamma = 1$$

and the coefficients N_c, N_q and N_γ are given by the following expressions (*):

$$N_q = e^{\pi \tan\varphi} \tan^2(45^\circ + \varphi/2)$$

$$N_c = (N_q - 1)\cot\varphi$$

$$N_\gamma = 1.8\ (N_q - 1)\tan\varphi$$

For convenience of computation these coefficients, as functions of the internal angle of friction φ are given in Table 8. The maximum soil pressure developed at the wall foundation is usually found by the expression:

$$\sigma_{\max} = \frac{N}{B}\left(1 + \frac{6e}{B}\right)$$

The maximum pressure thus obtained must be lower than the allowable soil bearing stress:

$$\sigma_{am} = \frac{p_{\lim}}{3}$$

Some authors suggest that for cohesionless soil:

$$\sigma_{am} = \frac{p_{\lim}}{2}$$

In the case where only a portion of the base section is in compression (eccentricity $e > B/6$) the soil stress can be evaluated with the usual espression:

$$u = \frac{B}{2} - e$$

$$\sigma_{\max} = \frac{2N}{3u}$$

where u indicates the distance from the normal force to the element under stress.

Table 8 - Coefficients for the computation of maximum bearing pressure on soil.

φ	N_c	N_q	N_γ
0	–	1.00	0.00
5	6.48	1.57	0.09
10	8.34	2.47	0.47
15	10.97	3.94	1.42
20	14.83	6.40	3.54
25	20.72	10.66	8.11
30	30.14	18.40	18.08
35	46.13	33.29	40.69
40	75.32	64.18	95.41
45	133.89	134.85	240.85
50	266.89	318.96	681.84

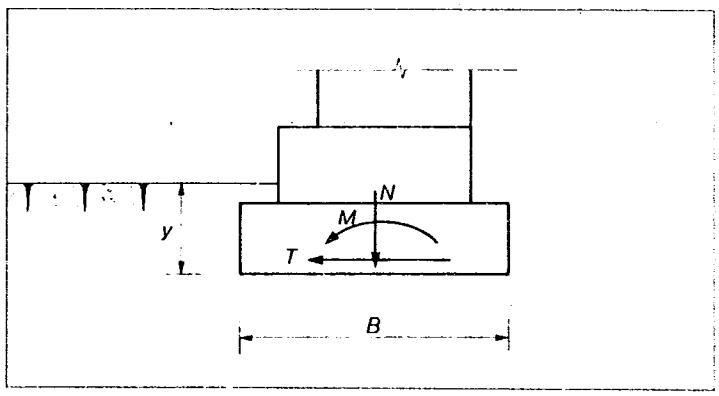


Fig. 69 - Characteristics of the stresses acting on the foundation of a

4.5 Criteria for the evaluation of deformation

As determined in the experiments described in Chapter III, gabions exhibit greater strength in shear than in bending. This was demonstrated in tests on single elements and on full size structures, and is confirmed by the appearance of deformed walls.

So in order to evaluate possible movements in wall structures it is necessary to define an appropriate value of the tangential modulus of elasticity G. On the basis of the experiments mentioned above it has been possible to observe (§ 3.4.5) a correlation, for a given wall section, between the value of G (tangential) and the value of τ.

This relation can be used to determine deflection. In the examples above it can be shown that the results are related to the specific procedure followed in loading. In practice the value of tangential stress in the walls is generally less thus resulting in a linear expression of τ against γ and therefore a constant value of G.

The value of G can be expected to range between 2.5 and 3.5 kg/cm^2 and in any case these can serve as guideline values.

4.6 Analysis of the behaviour of particular retaining structures

4.6.1 General

The component elements of gabions or panels of mesh can be used to advantage in the construction of reinforced soil structures. They function in a fashion similar to that of reinforcing strips or geogrids in increasing the strength of the soil in shear and tension when placed in the backfill. The purpose of the examples of calculation that follow is to demonstrate the principles rather than their practical application while in Chapter V more detailed examples will show the entire calculations for the same procedures.

At this juncture it is opportune to remind the reader that projects of this nature not only require careful planning in the office but also a meticulous soil investigation of the site to provide accurate data for the preparation of the design.

4.6.2 Retaining structures using gabions or panels of mesh as soil reinforcement

This type of structure is shown in Fig. 70 and consists of a thin gabion wall anchored into the soil with one or more layers of ties.

Obviously the computation must include checks on the following:

a) The strength of the ties and their anchorage length.
b) Stress concentration at critical points on the thin wall with particular attention paid to the effect of vertical loads and connections to the ties.
c) Soil resistance at the wall foundation.
d) Overall stability of the structure and surrounding soil.
e) Any necessary control of vertical and horizontal deflection.

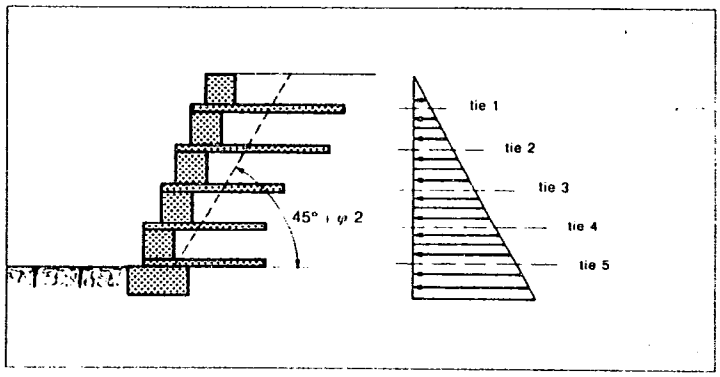
4.6.2.1 Dimensions of the soil reinforcement elements. The combination as a whole of all the reinforcement elements in the structure must be capable of resisting the thrust of the active pressure with an adequate factor of safety, generally taken as equal to 2.

The position of reinforcement elements is planned by distributing them along the height of the wall using areas of influence and assuming triangular distribution of the earth pressure on the wall, see Fig. 70.

Consequently ties or elements that are uniformly stressed will be closer together at the bottom of the wall.

Table V in Chapter 3 serves as a guide to the design of reinforcement elements and it also gives the breaking

Fig. 70 - Retaining structures stiffened with reinforcement ties made of gabions and Reno mattress.



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strength S_r for the various types of mesh. The allowable strength in the mesh can be obtained by reducing the given breaking strength value by a factor of safety, which in this case can be assumed equal to 1.2 to 1.3.

The anchorage length of the tension element can be determined using Fig. 71 and considering the ultimate tangential strength τ_{lim} related to the normal pressure of the soil σ by the usual expression:

$$\tau_{lim} = \sigma \tan \varphi$$

The anchorage length of the reinforcement element must extend into the backfill beyond the plane of expected failure (Figs. 70, 71) this length is:

$$l = \frac{S_{lim}}{\tau_{lim}} = \frac{nS_r}{\sigma \tan \varphi}$$

where n is the number of mesh panels lined up along the direction of anchorage stress per unit width of the tension element.

The resulting anchorage length includes a safety factor in as much as the mesh is stressed to a lower limit than its capacity. The allowable stress for the mesh is equal to the breaking strength divided by 1.2 or 1.3.

Although these may appear to be high, it should be remembered that in this case particularly, they affect the overall stability of the structure.

4.6.2.2 Design of the thin gabion wall. The data for the design of thin walls anchored with mesh panels was obtained from the tests described in Paragraph 3.5.

The test results show that 2 to 3 metres is the minimum anchorage length required to provide a "pull-out" resistance equal to or greater than the breaking strength of the mesh itself when loaded with soil to a depth of one metre only. The length of anchorage is computed by starting from the theoretical failure surface of a wedge of earth resting on the base. However, recent studies indicate that the rupture plane critical for masses of reinforced soil is transposed towards the toe. This results in an increased effective strength of the anchorages as seen in References [20, 21 and 22].

The project designer needs to consider such data to determine whether the anchorage length should be extended beyond the 2 to 3 metre dimension notwithstanding the statement above that longer anchorage lengths are not generally required.

It is advisable to analyse the overall stability of the wall and surrounding soil as a whole, taking a series of slip surfaces passing through the ends of the anchorage panels. This entails checking the stability of a wedge of earth resting against the wall and exerting a thrust less than the active thrust, but not having the stabilizing effect of the anchors which lie within the wedge.

The evaluation of the overall stability at connection points of the anchorage panels to the wall face presents no difficulty. This is clearly illustrated in § 3.2.3; the shear strength of the gabions is in the order of 1.5 to 2 kg/cm^2 and therefore 15 to 20 tonnes on one metre length of wall having a thickness of one metre. It can be seen that the shear stress at the connection points is approximately half the tension stress in the anchorage panels, therefore even where the panels are carrying their maximum load it does not exceed 3000 kg. The panels are placed with the dimension of maximum strength, (the direction in which the mesh is woven), lying parallel to the direction of soil stress. Therefore there will be panels at least 2 metres, and preferably 3 to 4 metres wide and of a length which will vary according to the wall height and with the predetermined anchorage length.

The anchorage panels should be extended through the gabion wall for a least one metre and be securely tied to ensure a positive connection between the wall and the anchorage.

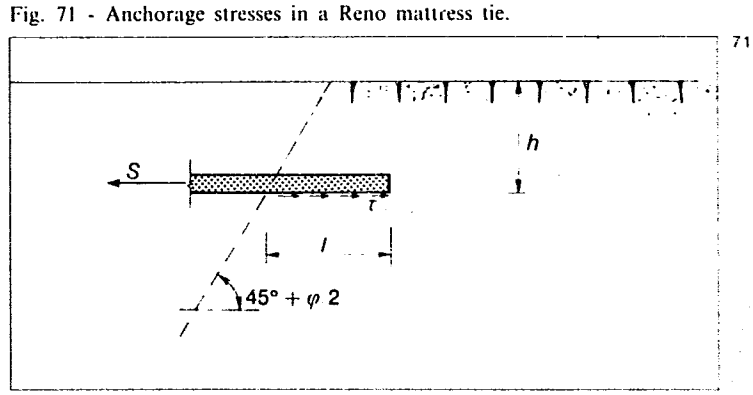
For economy purposes it is useful to note that the anchorage panel can be used as the lid for the gabion to which the panel is anchored.

4.6.2.3 Localized resistance of the gabion wall. The vertical stresses are produced mainly by the wall's own weight and the remainder by the bending stress imposed locally by the soil thrust. They are computed using the procedures described in the preceding paragraphs.

To conclude, the virtually concentrated loads carried by the reinforcement panels impose tangential tensions on the wall. These have to be contained within the limits already mentioned. See § 3.2.3 and 4.6.2.2.

4.6.2.4 Foundation soil stresses. The bearing pressure on the foundation soil must be checked under both the backfill and the wall. If it is desirable to avoid settlement due to soil movement and hence failure of any magnitude it would be preferable to check the local stresses, i.e. verify that at specific critical points the allowable soil bearing capacity is not exceeded (See § 4.4.2). The safety factor can be checked using the same method as that for local stability (See § 4.3.3).

Fig. 71 - Anchorage stresses in a Reno mattress tie.



4.7 Computer programme for the design of gabion walls

The calculations required for the design of gabion walls can be programmed for computer solution. This is confirmed in the above paragraphs which show that the sequence of operations and design criteria remain unchanged, whatever the wall section, soil or gabion characteristics. There are however some exceptions for special cases. A programme has been prepared for the FORTRAN system and called GAWAC for «Gabion Wall Code». Gabion walls of any shape and size can be analysed with this programme. It is immaterial whether the section is vertical, or built to a batter, or whether it is stepped on the front on rear face.

In this programme the geometry of the project is described by means of a matrix, each element of which corresponds to a structural characteristic. As illustrated in the example of Fig. 72 the gabion characteristics are assigned the number 1, the characteristics of the soil are assigned number 2 and the unoccupied space is assigned number 0.

The GAWAC programme, which is also designed to deal with sloping or level backfill loaded with a uniformly distributed surcharge p_0, computes the active earth thrust S_a acting on the wall. In cases of simple gravity walls as shown in Figs. 56 and 59a the programme assumes the pressure surface to be the back of the wall. But in the case of walls with foundation heels and balancing «boxed» soil over the heel, the load surface is assumed to be the vertical plane represented by line *b-b* in Figs. 59b, 59c and 67.

The thrust is assumed to make an angle $\delta = \varphi$ with the perpendicular to the thrust surface.

The GAWAC programme using the known density of the gabion and soil determines the overturning moment *MI*, the restoring moment *MS*, and the overturning safety factor SR. The check for sliding is performed according to paragraph 4.3 by computing *FI*, *FS* and the safety factor *SS*. The eventual anchorage force exerted on the extreme portion of the heel beyond section *b-b*, in Fig. 67, is assumed equal to the smaller of the two values, namely the allowable tension resistance of the mesh itself and the anchorage value developed by the soil. When sp ied, the contribution of the passive soil pressure at the toe is also evaluated.

The programme computes the soil pressure both in the case where the soil pressure acts over the whole foundation width and where it acts on part of the width only. To do this it uses the usual reactions for sections affected by stress but not subject to tension. The output gives the maximum and minimum soil pressures with the understanding that if the minimum pressure is zero the stress exists on only part of the foundation.

The programme excludes the possibility of having zero pressure at the toe. This eventuality even though rare can occur if the soil thrust is low and the front face of the wall is extensively stepped.

The stress at the bottom of the stem wall is also checked and here too the case where the stress exists over part only of the section is singled out.

The check for overall stability of the wall and surrounding soil is carried out according to the criteria quoted in

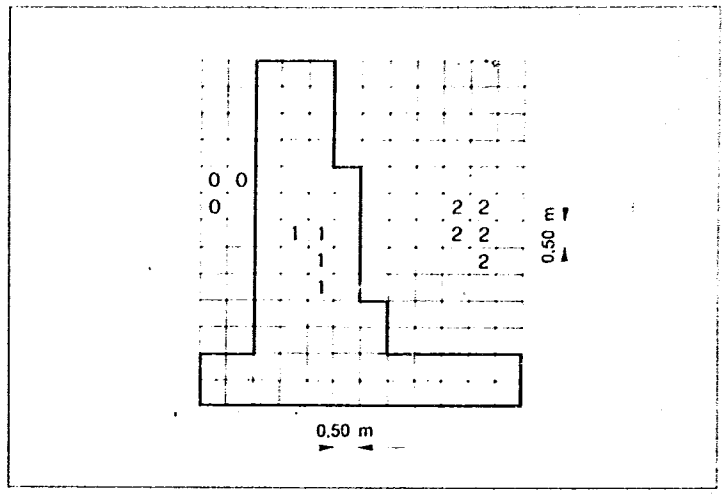


Fig. 72 - Diagram of the geometry of a gabion structure illustrating the input for GAWAC computer program.

paragraph 4.3, yielding, after a trial and error computation, the minimum value of the safety factor (*SI*) for the stability of the system as a whole.

Finally the maximum deflection of the top of the wall is computed. For every 0.5 m increment of wall height, using the average values for the combination of wall and soil for that increment, an average tangential stress is computed. From this stress the value of *G* and the average deflection for the given wall increment and wall height are obtained using relations of the type indicated in paragraph 3.4.5.

The maximum deflection at the top is obtained by adding the deflection contributions of all the increments of wall height.

For the sake of comparison the programme also evaluates the horizontal deflection at the top of the wall using a constant tangential modulus of elasticity i.e. *G* of value 2.5 kg/cm^2 which was derived from experiments on both gabion elements and full scale projects.

It is to be understood that this total deflection, even though computed, only serves as a guideline value and in any case is a maximum range.

The GAWAC programme is arranged so that for a given wall cross section it will automatically cycle through for different values of the density of soil, different angles of friction, for increasing values of the density of the gabions and various loading conditions, including sloping and level backfill and uniform surcharge.

Several tables in the appendix were developed with the use of the GAWAC programme. These tables make it easier for the project designer to establish preliminary basic sizes for gravity and semi-gravity walls and for different loading and soil conditions.

4.8 Comments concerning the design criteria for gabion structures

4.8.1 Layout and sizes of gabions in the structure

The best results for this type of construction are obtained by exercising the utmost care in the filling of the gabions and in arranging their layout in courses in such a way that the distribution of the mesh in the structure is balanced with respect to its characteristic functions. This means that the fill material should be as small as possible for the given size of the mesh to obtain a uniform density which will ensure the best load distribution and also the maximum and most efficient utilization of the wire mesh.

In this respect gabions in 6 × 8 mesh are preferable to those in 8 × 10 or 10 × 12 mesh, and 0.5 m deep to 1.00 m deep units, since the increased ratio of mesh to stone fill will noticeably improve the static behaviour of the wall.

It follows that where walls are higher than 4 to 5 metres, it is advantageous to use 0.5 m deep in preference to 1.0 m deep gabions in the lower courses and foundation where the compression and shear stresses are highest.

In addition, by increasing the number of panels aligned perpendicular to the face of the wall and therefore parallel to the soil thrust, the deformation caused by shear will be effectively reduced. This is simply achieved by placing the gabions with their longest dimension perpendicular to the wall.

4.8.2 Filling

Gabions may be filled by hand or by mechanical means. Every effort shall be made to keep voids and bulges in the gabions to a minimum in order to ensure proper alignment and a neat, compact, square appearance, without using stone of such size and shape as to obtain a rigid construction or face.

4.8.3 Walls with stepped front or rear faces

Gabion walls can be built with a stepped front-face, or with a stepped rear-face, see Chapter II. Walls with stepped rear-face are often chosen for functional and aesthetic reasons. However from the technical point of view the walls with stepped front-faces are preferable, and certainly advisable for walls 5 to 6 metres high. For other walls the stepped rear-face is acceptable providing that walls are built to a batter of not less than 6 degrees.

4.8.4 Allowable deformation

Gravity walls are subject to less deformation than semi-gravity walls and are therefore more suitable in situations where movement of the top of the structure is to be avoided. But even in these cases it is possible to use semi-gravity walls and control the movement by means of one or more layers of ties as described in paragraph 4.6.2, depending on the height of the wall and the soil characteristics.

4.8.5 Drainage

When compared to other types of retaining structures, gabions walls have an additional safety factor in that they are permeable and self draining. They are therefore not subject to pressures exceeding those contemplated, nor to the deterioration of the soil characteristics in the foundation and the backfill areas.

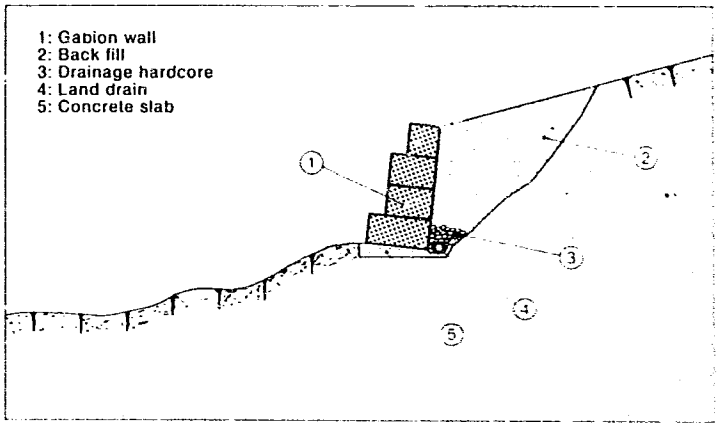
Notwithstanding the above, it is advisable to take proper measures to improve the drainage in the back of the wall using larger size material at the lower level of the fill (Fig. 73), and possibly providing a concrete apron shaped to drain off collected water in a convenient direction, and installing land drains.

At regular intervals along the walls drainage should be provided to carry water away from the foundation.

4.8.6 Use of filter fabrics

In fine silty or sandy soils, it is most important to select a filling material which can serve as a filter to prevent migration of the soil through the structure. It is inadvisable to use filter fabrics against the rear face of the wall because of the possibility, often inevitable, of the fabric becoming clogged, with the resulting dangerous increase in hydrostatic

Fig. 73 - Drainage system for a gabion retaining structure.



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pressure.

The use of fabric can be recommended in river and marine works where a continuous and frequent water movement will flush the wall and the soil behind it. Obviously in these cases provision must be made in the design to cater for the relief of hydrostatic pressure.

Fabric is sometimes useful for providing a filter between the excavated surface and the backfill, and for preventing the washout of fines from the foundation (Fig. 74).

4.8.7 Drainage counterforts

The drainage in the area behind the wall can be effectively improved by the installation of gabion counterforts (See Fig. 75). These project from the rear face of the wall to the limit of the area to be drained which is usually scallop shaped. They therefore tend. to be more widely spaced at the outer limit than at the wall (See § 4.3). Their thickness is generally 1 to 2 m and they are spaced at distances which depend on the dimensions of the wall, that is, empirically, twice the height of the wall. Their function is essentially to provide drainage, they therefore serve a different purpose to the stiffeners and ribs used in concrete walls. This does not mean that they do not contribute some structural reinforcement to the wall. They can be considered as efficient anchorage elements similar in nature to the foundation heel and mesh anchorage panels, in that they perform as counterweights acting on the rear face and any forward movement in the structure is resisted by friction on their sides. They also stiffen the top of the wall by preventing its deformation.

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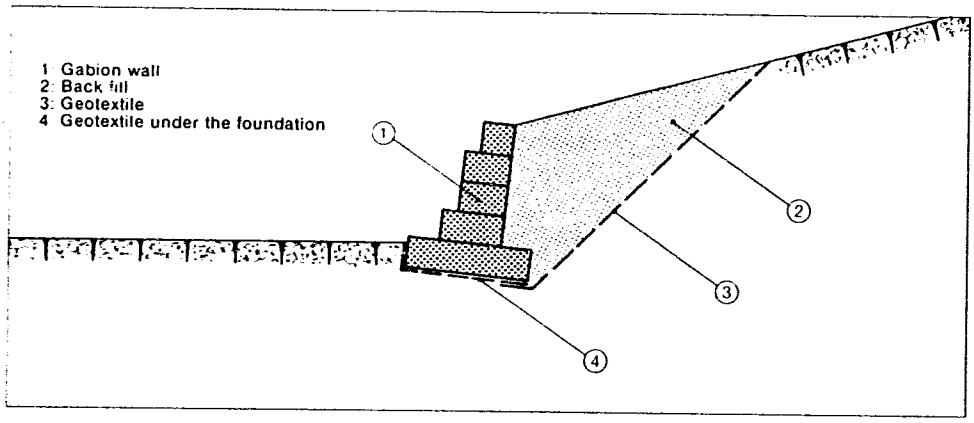

Fig. 74 - Use of filter fabrics in a gabion retaining structure.

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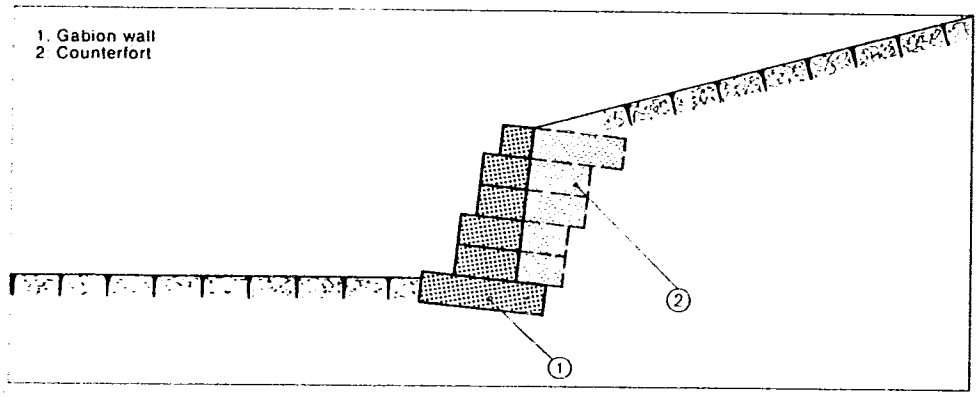


Fig. 75 - Diagram illustrating gabion drainage counterforts.

CHAPTER V

Examples of calculations

In this chapter the criteria developed in the preceding chapters are clarified by the practical application of the solutions to some specific examples of gabion retaining structures. The procedure followed is probably too demanding to be carried out manually, and this applies especially to the one in Example 1a. In a preliminary evaluation the designer should therefore proceed with simplified calculations, concentrating mainly on the behaviour of the structure as described in Example 1b. The more precise computations can be made when the final dimensions of the wall have been extracted or when preparing a computer program solution. "Officine Maccaferri S.p.A." will be pleased to offer assistance to interested designers in the examination of specific problems arising during the planning of gabion retaining structures and in the formulation of project proposals using the GAWAC programme outlined in § 4.7.

Example 1a

Calculations are required for the gabion retaining wall shown in Fig. 76.

The density of the gabion is $\gamma_g = 1.65$ t/m³; the density of the fill behind the wall and that of the foundation soil is $\gamma_t = 1.8$ t m³, their internal angle of friction is $\varphi = 30°$; for resistance against sliding the cohesion factor $c = 1$ t/m² is assumed for the foundation. The gabions are of hexagonal mesh 8 × 10 with wire diameter $\varnothing = 3.00$ mm.

The total wall height measured parallel to the face of the wall is $H = 8.00$ m of which 1.00 m is buried; the structure is battered at an angle $\alpha = 6°$. In addition to the soil thrust the wall is supporting a surcharge load $p_0 = 1.8$ t/m², equivalent to a soil layer of depth 1.00 m., and applied on top of the horizontal backfill ($\varepsilon = 0$).

a) The design calculations are carried out on a unit length of wall.

The lines extended at 45° from points F' and A' intersect the base at hinge points F and A. The effective length of the base element is therefore:

$$B = FA = 4.50 \text{ m}$$

The base is therefore full effective.

The vertical plane against which the pressure is assumed to act passes through point D.

b) Let A_m, X_m, Y_m and A_t, X_t, Y_t be the areas and the coordinates of the centres of gravity G_m and G_t of the wall section and of the soil confined between the back of the wall and QND respectively, in the coordinate system OXY with origin at 0 and has axes parallel to the faces of the gabions:

$A_m = 16$ m² $\quad X_m = 2.00$ m $\quad Y_m = 1.94$ m

$A_t = 13$ m² $\quad X_t = 3.50$ m $\quad Y_t = 4.04$ m

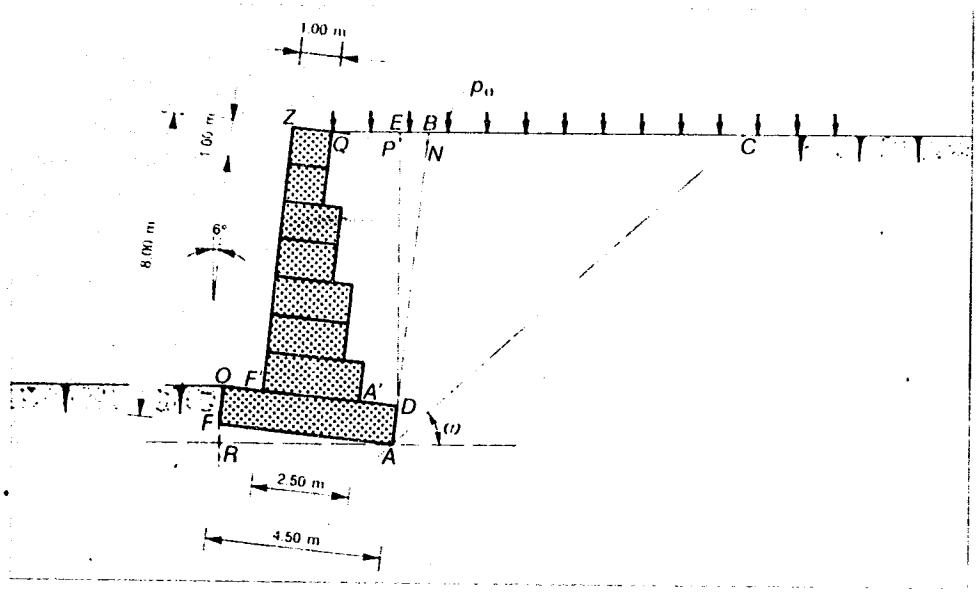
The coordinates of the points G_m, G_t, Q, N, D, F, A in the coordinate system Oxy which has its origin at 0 and vertical and horizontal axes, and is therefore rotated 6° with reference to the OXY coordinate system, are computed using the following relationships:

$$x = X \cos\alpha + Y \sin\alpha$$

$$y = -X \sin\alpha + Y \cos\alpha$$

The results are:

$x_m = 2.19$ m	$y_m = 1.72$ m
$x_t = 3.90$ m	$y_t = 3.65$ m
$x_Q = 2.72$ m	$y_Q = 6.75$ m
$x_N = 5.21$ m	$y_N = 6.49$ m
$x_E = x_D = 4.48$ m	$y_D = -0.47$ m
$x_F = -0.10$ m	$y_F = -0.99$ m
$x_A = 4.37$ m	$y_A = -1.46$ m



c) Let A_{QEP}, M_{QEP}, A_{PND}, M_{PND} be the areas and the static moments about hinge point F of the triangles QEP and PND:

$$A_{QEP} = (x_E - x_Q)^2 (\tan\alpha + \tan\varepsilon)/2 = 0.162 \text{ m}^2$$

$$M_{QEP} = A_{QEP}(x_E - x_F - (x_E - x_Q)/3) = 0.646 \text{ m}^3$$

$$A_{PND} = (DN)^2 \tan\alpha/2 = 2.58 \text{ m}^2$$

$$M_{PND} = A_{PND}(x_E - x_F + (x_N - x_E)/3) = 12.42 \text{ m}^2$$

Let W_m, M_m, W_t, M_t be the weights and the moments with respect to hinge point F of the wall section and the "boxed" soil i.e. the soil enclosed between the wall and the plane on which the soil pressure is assumed to act.

$$W_m = A_m \gamma_a = 26.4 \text{ t/m}$$

$$M_m = W_m(x_m - x_F) = 60.62 \text{ tm/m}$$

$$W_t = (A_t + A_{QEP} - A_{PND})\gamma_t = 19.06 \text{ t/m}$$

$$M_t = (A_t(x_t - x_F) + M_{QEP} - M_{PND})\gamma_t = 72.58 \text{ tm/m}$$

Let W_c, M_c be the resultant and the moment of the resultant about hinge point F of that portion of the surcharge located above the boxed-in soil.

$$W_c = p_0(x_E - x_Q) = 3.16 \text{ t/m}$$

$$M_c = W_c(x_Q - x_F + (x_E - x_Q)/2) = 11.69 \text{ tm/m}$$

Let W_f be the weight of the soil included between the base of the wall FA and the horizontal sliding plane RA:

$$W_f = (x_A - x_F)^2 \gamma_t \tan\alpha/2 = 1.89 \text{ t/m}$$

d) Computation of the active pressure. Since the earth thrust is exerted on a soil to soil plane it can be assumed, (see § 4.7) that:

$$\delta = \omega = 30°$$

δ being the angle included between the line of action of the earth thrust and the perpendicular to the plane on which the pressure acts. Let β be the angle between the plane of earth thrust and the horizontal, and ω the angle between the line of the earth thrust and the horizontal:

$$\beta = 90°$$

$$\omega = \delta + (90° - \beta) = 30°$$

Let H_a be the height where the active thrust acts, and K_a the coefficient of active pressure:

$$H_a = y_Q + (x_E - x_Q)\tan\varepsilon - y_A = 8.22 \text{ m}$$

$$K_a = \frac{\sin^2(\beta + \varphi)}{\sin^2\beta \sin(\beta - \delta)\left(1 + \sqrt{\dfrac{\sin(\varphi + \delta)\sin(\varphi - \varepsilon)}{\sin(\beta - \delta)\sin(\beta + \varepsilon)}}\right)^2} =$$

$$= 0.297$$

Let S_a be the active earth pressure, S_h and S_v its vertical and horizontal components and M_a its moment about the hinge point:

$$S_a = \gamma_t H_a^2 K_a/2 + p_0 H_a K_a = 22.46 \text{ t/m}$$

$$S_h = S_a \cos\omega = 19.45 \text{ t/m}$$

$$S_v = S_a \sin\omega = 11.23 \text{ t/m}$$

$$M_a = S_h\left(\frac{H_a}{3} \cdot \frac{H_a + 3p_0/\gamma_t}{H_a + 2p_0/\gamma_t} - (y_F - y_A)\right) - S_v(x_D - x_F) =$$

$$= -2.09 \text{ tm/m}$$

The moment of the earth thrust has a negative sign because the line of the thrust intersects the base of the wall.

e) In the computation of the passive pressure it is assumed that

$$\beta = 90^\circ \qquad \delta_p = \varphi = 30^\circ \qquad \varepsilon = 0^\circ$$

Let K_p be the coefficient of passive earth pressure:

$$K_p = \frac{\sin^2(\beta - \varphi)}{\sin^2\beta \sin(\beta + \delta_p)\left(1 - \sqrt{\frac{\sin(\varphi + \delta_p)\sin(\varphi + \varepsilon)}{\sin(\beta + \delta_p)\sin(\beta + \varepsilon)}}\right)^2} =$$

$$= 10.10$$

The passive pressure is divided into S'_p which is the portion acting up to the height of the hinge point and S''_p which is the portion acting up to the level of the plane of sliding (y_A). In addition let S'_{ph} and S'_{pv} and M'_p be the components and the moment about hinge point F of S'_p and let S''_{ph} and S''_{pv} be the components of S''_p:

$$S'_p = \gamma_t y_F^2 K_p/2 = 8.99 \text{ t/m}$$

$$S'_{ph} = S'_p \cos\delta_p = 7.78 \text{ t/m}$$

$$S'_{pv} = S'_p \sin\delta_p = 4.49 \text{ t/m}$$

$$M'_p = S'_{ph} y_F/3 = 2.58 \text{ tm/m}$$

$$S''_p = \gamma_t y_A^2 K_p/2 = 19.50 \text{ t/m}$$

$$S''_{ph} = S''_p \cos\delta_p = 16.89 \text{ t/m}$$

$$S''_{pv} = S''_p \sin\delta_p = 9.75 \text{ t/m}$$

f) Check the resistance to sliding along the horizontal plane RA passing through the lowest point of the fondation. Let F_i be the sum of the forces causing sliding and F_s the sum of the forces resisting sliding and η_S the safety factor, then:

$$F_i = S_h = 19.45 \text{ t/m}$$

$$F_s = S''_{ph} + cB\cos\alpha + (W_m + W_t + W_f + W_c + S_v - S''_{pv})\tan\varphi =$$

$$= 51.39 \text{ t/m}$$

$$\eta_s = F_s/F_i = 2.64$$

Let M_i be the sum of the overturning moment and M_s the sum of the restoring moment. The safety factor against overturning $\eta_r = M_s/M_i$ is not computed because as already noted M_i is negative:

$$M_i = M_a = -2.09 \text{ tm/m}$$

$$M_s = M_m + M_t + M_c + M'_p = 147.47 \text{ tm/m}$$

g) Let N be the normal force acting on the base section FA, e the eccentricity of the resultant, and σ_1 and σ_2 the soil pressures at points F and A:

$$N = (W'_m + W'_t + W'_c)\cos\alpha + S_a \sin(\omega + \alpha) - S'_p \sin(\delta_p + \alpha) =$$

$$= 56.27 \text{ t/m}$$

$$e = B/2 - (M_s - M_i)/N = -0.408 \text{ m}$$

$$\sigma_1 = (1 + 6e/B)N/B = 5.7 \text{ t/m}^2$$

$$\sigma_2 = (1 - 6e/B)N/B = 19.3 \text{ t/m}^2$$

Since the computed values of the soil pressure are significantly higher at the heel than at the toe, it can be assumed that these results are on the safe side and that in practice the soil pressures will be more uniform.

Computation of the allowable soil bearing pressure.

Let T be the shear force acting at section FA, let $p_{\lim}$ be the limiting pressure and σ_{am} the allowable soil bearing pressure, N_c, N_q, N_γ, i_q, i_γ, d_c, d_q, d_γ the coefficients to be used in Hansen's formula (see § 4.4.2):

$$T = -(W_m + W_t + W_c)\sin\alpha + S_a\cos(\omega + \alpha) -$$

$$- S'_p \cos(\delta_p + \alpha) = 5.82 \text{ t/m}$$

$$N_q = e^{\pi\tan\varphi}\tan^2(45^\circ + \varphi/2) = 18.40$$

$$N_c = (N_q - 1)\cot\varphi = 30.14$$

$$N_\gamma = 1.8(N_q - 1)\tan\varphi = 18.08$$

$$i_q = 1 - 0.5T/N = 0.948$$

$$i_\gamma = i_q^2 = 0.899$$

$$d_q = d_c = 1 + 0.35 y_F/B = 1.077$$

$$d_\gamma = 1$$

$$p_{\lim} = cN_c d_c + y_F \gamma_t N_q d_q i_q + \frac{1}{2}\gamma_t B N_\gamma d_\gamma i_\gamma = 131.8 \text{ t/m}^2$$

$$\sigma_{am} = p_{\lim}/3 = 43.9 \text{ t/m}^2 > \sigma_{\max}$$

h) Computation of the safety factor for overall stability, assuming the failure surfaces to be planes (see § 4.3.3) and $\delta = \varphi$. Let x_B, y_B be the coordinates of the point of intersection B of the top of the fill with the line NA (Fig. 77):

$$\overline{QN} = [(x_N - x_Q)^2 + (y_N - y_Q)^2]^{1/2} = 2.50 \text{ m}$$

$$x_B = x_N + \overline{QN}\sin(\alpha + \varepsilon)\sin\alpha = 5.24 \text{ m}$$

$$y_B = y_N + \overline{QN}\sin(\alpha + \varepsilon)\cos\alpha = 6.75 \text{ m}$$

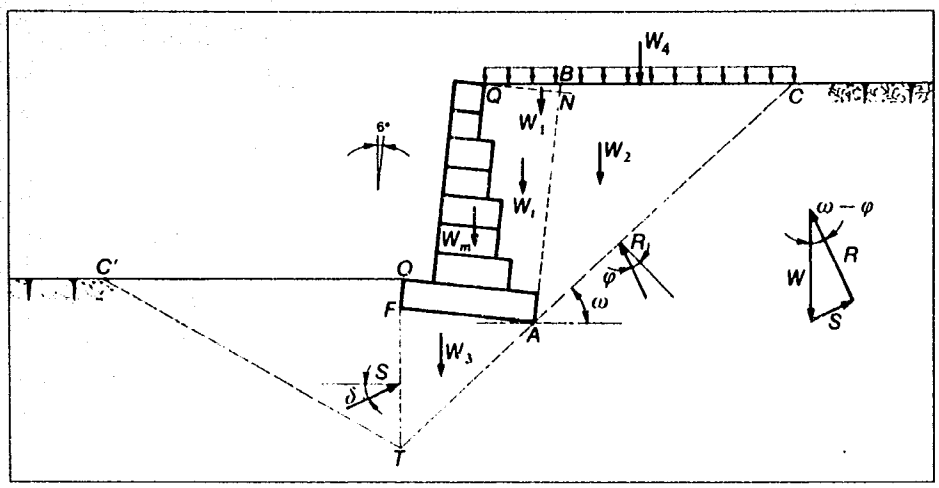
$$\overline{AB} = [(x_A - x_B)^2 + (y_A - y_B)^2]^{1/2} = 8.26 \text{ m}$$

Let the weight of the soil of section QNB be:

$$W_1 = \overline{QN}^2 \tan(\alpha + \varepsilon)\gamma_t/2 = 0.591 \text{ t/m}$$

Varying ω the angle at which the failure plane passing through point A makes with the horizontal, it is found that the safety factor is least for $\omega = 43.58^\circ$.

With this value of ω, let C be the point of intersection of the failure plane with the soil surface, let T be the intersection point of the failure plane with the vertical through F, W_2 the



weight of the soil section ABC, W_3 the weight of the soil section AFT, W_4 the resultant of the surcharge acting on QC:

$$W_2 = \frac{1}{2}\overline{AB}^2 \frac{\cos(\alpha+\varepsilon)\cos(\alpha+\omega)}{\sin(\omega-\varepsilon)}\gamma_t = 57.48 \text{ t/m}$$

$$W_3 = (x_A - x_F)^2(\tan\alpha + \tan\omega)\gamma_t/2 = 19.05 \text{ t/m}$$

$$\overline{BC} = \overline{AB}\cos(\alpha+\omega)/\sin(\omega-\varepsilon) = 7.77 \text{ m}$$

$$W_4 = [(x_B - x_Q) + \overline{BC}\cos\varepsilon]\,p_0 = 18.51 \text{ t/m}$$

Let W be the sum of the destabilizing weights and S the thrust on the plane OFT, which closes the force triangle with side S, W and the reaction along TC:

$$W = W_1 + W_2 + W_3 + W_4 + A_t\gamma_t + W_m = 145.43 \text{ t/m}$$

$$S = W\sin(\omega-\varphi)/\cos(2\varphi-\omega) = 35.60 \text{ t/m}$$

Let R_p be the passive thrust acting on the plane OFT:

$$y_T = y_F - (x_A - x_F)(\tan\alpha + \tan\omega) = -5.72 \text{ m}$$

$$R_p = y_T^2 K_p \gamma_t/2 = 297.7 \text{ t/m}$$

Let η_i be the safety factor for overall stability:

$$\eta_i = R_p/S = 8.361$$

The search for the minimum value of η_i was carried out by using the following values:

$\omega = 45°$ $\quad \eta_i = 8.399$ (first attempt)

$\omega = 43.4°$ $\quad \eta_i = 8.362$

$\omega = 43.58°$ $\quad \eta_i = 8.361$ (minimum)

i) Computation of tension at section $F'A'$ at the bottom of the stem, assuming the load is acting along the line QA'.

The weight, the moment arm about hinge point F' and the moment exerted on the wall section are:

$$W_s = 18.98 \text{ t/m} \quad b = 1.20 \text{ m} \quad M_s = 22.76 \text{ tm/m}$$

Let β be the included angle between the plane on which the active earth pressure acts and the horizontal, let δ, assumed equal to φ, be the angle between the plane on which the passive earth pressure acts and the horizontal, ω the angle between the line of the active earth pressure and the horizontal, H_a the height at which the active earth pressure acts which is the vertical distance between the points Q and A', K_a the coefficient of active pressure, S_a the active pressure, M_a the moment of S_a about hinge point F'. The computations are carried out in a manner similar to those in Para d) above, thus:

$$\beta = \alpha + \tan^{-1}\overline{ZF'}/(\overline{F'A'} - \overline{ZQ}) = 83.91°$$

$$\delta = 30°$$

$$\omega = \delta + (90° - \beta) = 36.09°$$

$$H_a = 7.12 \text{ m}$$

$$K_a = 0.348$$

$$S_a = 20.32 \text{ t/m}$$

$$M_a = 12.55 \text{ tm/m}$$

Let M be the moment about hinge point F', N the normal force and T the tangential force acting at section $F'A'$, e the eccentricity, σ_{max} the maximum tension on the section, and σ_{am} the allowable tension (see § 4.4.1):

$$M = M_s - M_a = 10.21 \text{ tm/m}$$

$$N = W_s\cos\alpha + S_a\sin(\omega+\alpha) = 32.49 \text{ t/m}$$

$$T = -W_s\sin\alpha + S_a\cos(\omega+\alpha) = 13.10 \text{ t/m}$$

$$B = \overline{F'A'} = 2.5 \text{ m}$$

$$e = B/2 - M/N = 0.936 \text{ m}$$

$$x = (B/2 - e)/0.4 = 0.786 \text{ m}$$

$$\sigma_{max} = N/(0.8\ x) = 51.69 \text{ t/m}^2$$

$$\sigma_{am} = 40 + 50(\gamma_g - 1.4) = 52.50 \text{ t/m}^2 > \sigma_{max}$$

Assuming the contribution of the weight of the wire mesh on the lower side of the foundation to be $P_u = 10.3$ kg/m³ (see [9]), then the cohesion of gabion surface is (see § 4.4.1).

$$c_g = 0.03\ P_u + 0.05 = 0.26\ \text{kg cm}^2 = 2.6\ \text{t/m}^2$$

Let φ^* be the fictitious angle of friction of the stone filling in the gabions (see § 4.4.1); τ the average shear stress on the section, τ_{am} the allowable shear stress.

$$\varphi^* = 25\ \gamma_g - 10 = 31.25°$$

$$\tau = T/B = 5.24\ \text{t/m}^2$$

$$\tau_{am} = N \tan\varphi^* / B + c_g = 10.49\ \text{t/m}^2 > \tau$$

l) Computation of the deflection Δ of point Z at the top of the wall, assuming F' to be fixed. The structure is divided in elements of thickness $\Delta H = 0.50$ m and a table is drafted. The surface on which the earth pressure acts is assumed to be QA', as in Para i), above, with $H_a = 7.12$ m. Let h be the distance from Z measured parallel to the face of the gabions to the centre of gravity of the element, let S be the thrust, W the weight of the structure above, and T the shear at the height of the centre of gravity of the element, τ the average shear stress and b the width of the element, γ the angle of motion and $\Delta\xi$ the contribution of the element to the movement in the direction ZQ:

$$W = A\gamma_g = 1.65\ A$$

$$S = \frac{1}{2} K_a \gamma_t H_a^2 (h/\overline{ZF'})^2 + K_a p_0 H_a (h/\overline{ZF'}) =$$

$$= 0.3238\ h^2 + 0.637\ h$$

$$T = S\cos(\omega + \alpha) - W \sin\alpha = 0.742\ S - 0.1045\ W$$

$$\tau = T/b$$

$$\gamma = \tau/G = \tau/25$$

$$\Delta\xi = \gamma \Delta H = 0.5\ \gamma$$

Let also σ be the average normal tension, η_τ the ratio between the allowable tangential tension and the average tangential tension on the element.

$$\sigma = (S \sin(\omega + \alpha) + W \cos\alpha)/b = (0.6704\ S + 0.9945\ W)/b$$

$$\eta_\tau = (\sigma \tan\varphi^* + c_g)/\tau = (0.6068\ \sigma + 1.5)/\tau$$

The total movement of point Z in the direction ZQ is obtained from Table E1, and finally Δ, the horizontal deflection of Z is computed:

$$\Sigma(\Delta\xi) = 0.685\ \text{m}$$

$$\Delta = \Sigma(\Delta\xi)\cos\alpha = 0.680\ \text{m}$$

The computed Δ is the sum of the movements due to the earth pressure and that due to the surcharge load; it is worthwhile to separate the two. In fact the structures eventually superimposed on the backfill will be affected by the Δ' due to the surcharge, while the «permanent» deformation Δ'' of the structure is due to the soil pressure.

Assuming a constant value for the tangential modulus of elasticity the results are:

$$\Delta' = \Sigma \frac{1}{bG} K_a p_0 H_a (h/\overline{ZF'}) \cos(\omega + \alpha)\, \Delta H \cos\alpha =$$

$$= 0.0094\ \Sigma \frac{h}{b} = 0.256\ \text{m}$$

$$\Delta'' = \Delta - \Delta' = 0.424\ \text{m}$$

The effect of the surcharge is to increase the horizontal deflection by 0.26 m remembering that the wall is inclined at 6°, a point on top of the wall is 0.31 m behind a vertical line passing through the corresponding point at the bottom of the wall, when it is only loaded by earth pressure. When the surcharge is in position this distance becomes 0.05 m.

Table E1

h (m)	b (m)	A (m²)	S (t/m)	T (t/m)	τ (t/m²)	γ (–)	$\Delta\xi$ (m)	σ (t/m²)	η_τ (–)
0.25	1.0	0.250	0.180	0.090	0.090	0.0036	0.0018	0.53	20.22
0.75	1.0	0.750	0.660	0.360	0.360	0.0144	0.0072	1.67	6.98
1.25	1.0	1.250	1.302	0.751	0.751	0.0300	0.0150	2.92	4.36
1.75	1.0	1.750	2.106	1.261	1.261	0.0504	0.0252	4.28	3.25
2.25	1.5	2.375	3.073	1.870	1.247	0.0499	0.0249	3.97	3.14
2.75	1.5	3.125	4.200	2.578	1.719	0.0687	0.0344	5.30	2.74
3.25	1.5	3.875	5.190	3.406	2.270	0.0908	0.0454	6.69	2.45
3.75	1.5	4.625	6.942	4.354	2.902	0.1161	0.0580	8.16	2.22
4.25	2.0	5.500	8.556	5.400	2.700	0.1080	0.0540	7.38	2.21
4.75	2.0	6.500	10.331	6.545	3.273	0.1309	0.0655	8.80	2.09
5.25	2.0	7.500	12.269	7.810	3.905	0.1562	0.0781	10.26	1.98
5.75	2.0	8.500	14.368	9.196	4.598	0.1839	0.0920	11.79	1.88
6.25	2.5	9.625	16.630	10.680	4.272	0.1709	0.0854	10.78	1.88
6.75	2.5	10.875	19.053	12.262	4.905	0.1962	0.0981	12.24	1.82
							$\Sigma = 0.685$ m		

Example 1b

Simplified design calculations for the wall in example 1a.

The stability of the mass comprising the wall and surrounding soil as designated by OZNAF in Fig. 76 is to be investigated. The average density is 1.7 t/m³. The surface on which the pressure acts is the plane whose projection is line AB, so that $\beta = 96°$ and $\omega = 24°$.

In the computation of the stabilizing weight, a surcharge of 1 m of soil distributed over QE is combined with 0.5 m of soil distributed over the full width of ZN.

The combined weight of the mass comprising the wall, surrounding soil and surcharge and their moment about hinge point F, neglecting the step at the toe are:

$$W = 8.5 \times 3.5 \times 1.7 = 50.6 \text{ t/m}$$

$$M_w = W[(1.75 + 0.5)\cos 6° + 4.25 \sin 6°] = 135.6 \text{ tm/m}$$

$K_a = 0.251$; is obtained from Fig. 57 using $\beta = 96°$. Then the height of the resultant thrust becomes:

$$H_a = AN \cos 6° + QN \sin 6° = 8.22 \text{ m}$$

then:

$$S_a = \frac{1}{2} K_a \gamma_t H^2 + K_a p_0 H = 18.97 \text{ t/m}$$

$$S_h = S_a \cos \omega = 17.32 \text{ t/m}$$

$$S_v = S_a \sin \omega = 7.71 \text{ t/m}$$

$$M_a = S_h \frac{H_a}{3} \left(\frac{H_a + 3p_0/\gamma_t}{H_a + 2p_0/\gamma_t} \right) - S_v B = 17.4 \text{ tm/m}$$

The weight W_f of the soil FAR and the horizontal component S_{ph} of the passive pressure is computed:

$$W_f = \frac{1}{2} \gamma_t B \cos 6° B \sin 6° = 1.89 \text{ t/m}$$

$$S_{ph} \geqq \frac{1}{2} \gamma_t OR^2 \tan^2(45 + \varphi/2) = 5.8 \text{ t/m}$$

The resistance to sliding on plane RA, and to overturning about F, and the bearing pressure on the base FA are then calculated using the expressions below:

$$\eta_s = [(W + W_f + S_v)\tan\varphi + S_{ph} + cB]/S_h = 2.60$$

$$\eta_r = M_w/M_a = 7.80$$

$$N = (W + S_v)\cos 6° + S_h \sin 6° = 58.8 \text{ t/m}$$

$$e = B/2 - (M_w - M_a)/N = 0.24 \text{ m}$$

$$\sigma_{max} = \frac{N}{B}\left(1 + \frac{6e}{B}\right) = 17.2 \text{ t/m}^2 = 1.7 \text{ kg/cm}^2$$

Example 1c

In this example the heel of the foundation in Example 1a is extended back 1 m as shown in Fig. 78 and the resulting safety factor against sliding is compared to that in Example 1a.

a) The effective width of the foundation remains the same as that in Fig. 1a; the increased length is assumed to serve only for anchorage purposes.

b) The anchorage force can not exceed the strength of the mesh panels forming the anchors and only the top and bottom panels of the gabions are assumed to serve as anchors since the vertical panels carry the stress, which is mainly shear, developed at the section through A', see Fig. 76. The effective panels for anchorage are therefore two, at the top and bottom. Their load capacity is the ultimate load capacity of the mesh 5.3 t/m, (see paragraph 3.3), reduced by an appropriate safety factor to 4.5 t/m for each panel. Therefore the capacity of the mesh is:

$$S_{mesh} = 4.5 \times 2 = 9 \text{ t/m}$$

where S_{mesh} indicates the anchorage load carried by the gabion mesh.

c) Analysis of the anchorage force provided by friction in the portion of the foundation DE, see paragraph 4.6.2.2.

The soil pressure at the foundation level is:

$$\sigma = \gamma_t(y_Q - y_D) + p_0 + \gamma_g AD = 16.45 \text{ t/m}^2$$

and therefore the friction force S_{att} is:

$$S_{att} = \sigma \cdot \tan\varphi \cdot l = 9.50 \text{ t/m}$$

Since the friction force is greater than the mesh capacity, $S_{att} > S_{mesh}$, the anchorage force provided by the section of foundation beyond DE is assumed to be $S_{mesh} = 9$ t/m.

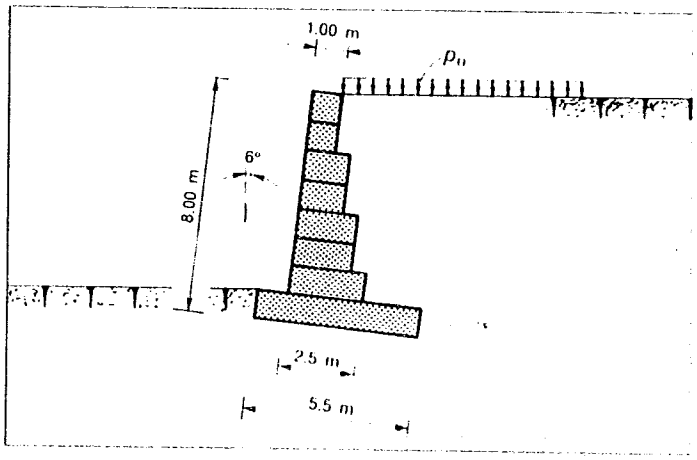
d) Adding the mesh components contributing to the other forces resisting sliding as computed at para f) in example 1a, the results are:

$$F_i = 19.45 \text{ t/m}$$

$$F_s = 51.39 + 9 = 60.39 \text{ t/m}$$

$$\eta_s = F_s/F_i = 3.10$$

which compares with $\eta_s = 2.64$ as computed in Example 1a without the foundation extension.



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Example 1d

Analysis of a wall similar to that in Example 1a but with stepped front-face, and a smooth rear-face as shown in Fig. 79.

In this case the stabilizing weight of the boxed soil is absent. If the pressure surface is assumed to be the rear face of the wall, that is a wall to soil surface, the following assumptions can be made for the computation of the active pressure:

$\beta = 96°$

$\delta = \varphi = 30°$

Proceeding in a way analogous to that in Example la, we obtain:

$\eta_s = 2.0$

$\eta_r = 4.9$

$\sigma_1 = 5.30 \text{ t/m}^2$

$\sigma_2 = 10.91 \text{ t/m}^2$

$\eta_i = 7.80$

$\sigma_{F'A'} = 15.56 \text{ t/m}^2$

$(\tau/\tau_{am})_{F'A'} = 1.9$

$\Delta = 0.54 \text{ m}$

Example 1e

Analysis of a structure similar to that in Example 1a but with a gabion at the top anchored by a panel of mesh as shown in Fig. 80.

a) The anchorage panel has a total length of 12 m and consists of two Reno mattress units 6.00 m long and 0.30 m thick, with 6 × 8 mesh and wire diameter 2.2 mm., tied end to end over every two metre length of wall.

To be effective great care must be taken in lacing the two mattress units together and the front unit to the wall. The top of the mattress consists of a mesh panel 13 m long and extends into the body of the wall.

b) The 6 × 8 hexagonal mesh is made of 2.2 mm steel wire with an ultimate strength (see paragraph 3.3), of 3.5 t/m. The total strength of the anchorage is therefore 3.5(2+0.30) = 8.05 t/m which is the sum of the loads that can be taken by the top, bottom and side mesh panels assuming that their edges are securely laced together.

The tie is anchored back in the stable zone of the soil, see § 4.2.1, for a length of approx. 6 m and therefore the anchorage resistance induced by friction amounts to:

$(1 \times 1.8 + 0.30 \times 1.65) \tan 30° \times 6 = 7.95 \text{ t/m}.$

Assuming 7.95 t/m to be the limiting strength of the tie and reducing it to 6 t/m after applying an appropriate safety factor, it is apparent that the strength is equivalent to the lateral thrust acting on the first 3.75 m of structure, including the surcharge. For that reason it can be assumed that the anchor achieves its object of reducing the deflection at the top of the wall.

c) The presence of the anchorage increases the safety factors computed in Example la, but above all it reduces the deformation which will be much less than that computed in Example 1a.

Furthermore, the point at which the anchor is tied to the structure is a "fixed" point and even without a more detailed analysis of the deformation of the soil-anchor-wall entity, the deformation at the top of the wall can be shown to be virtually nil:

$\Delta = 0$

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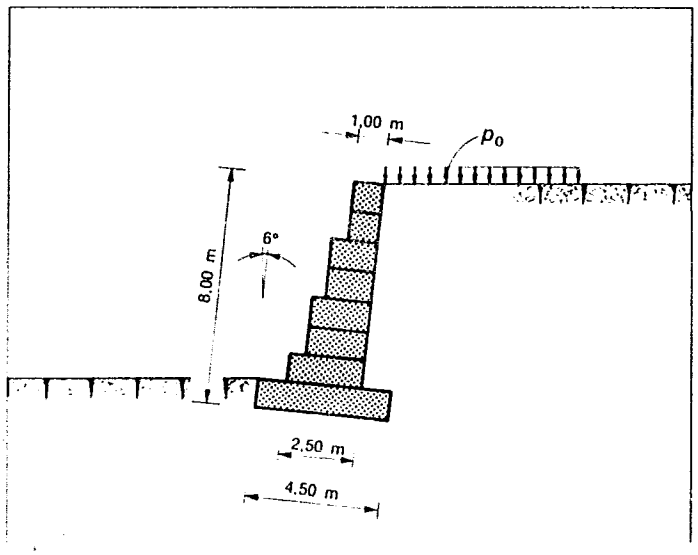

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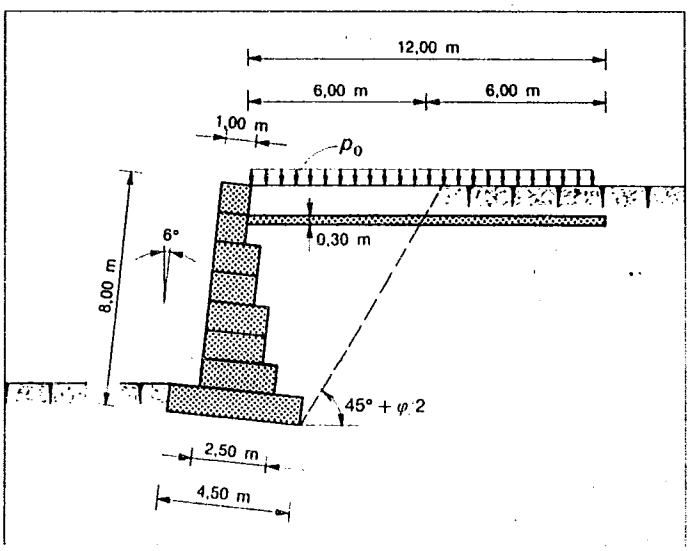

Example 1f

Analysis of a thin gabion wall anchored with mesh panels as shown in Fig. 81 and withstanding the lateral thrust of a soil similar to that of Example 1a.

The thin wall is formed of gabions without lids, fabricated of hexagonal 8 × 10 mesh and wire diameter ⌀ = 3.00 mm. The anchorage panels which also serve as gabion lids are of the same wire mesh.

In the following calculation the symbols are the same as those in Example 1a. The computations are carried out in a simplified manner.

a) Active thrust.

With $\beta = 96°$, $\varphi = \delta = 30°$, $\varepsilon = 0$, K_a becomes 0.2535, and the thrust acts on the internal wall face.

$$S_a = \left(\frac{1}{2}\gamma_t H^2 + p_0 H\right) K_a = 18.25 \text{ t/m}$$

$$S_h = S_a \cos(\delta - \alpha) = 16.67 \text{ t/m}$$

$$S_v = S_a \sin(\delta - \alpha) = 7.42 \text{ t/m}$$

$$M_a = S_h \cdot \frac{H}{3} \frac{H + 3p_0/\gamma_t}{H + 2p_0/\gamma_t} - S_v \cdot b_v =$$

$$= 2.93\ S_h - 1.81\ S_v = 35.46 \text{ tm/m}$$

b) Passive pressure.

With $\beta = 90°$, $\varphi = \delta = 30°$, $\varepsilon = 0$, K_p becomes 10.10

$$S_p = \frac{1}{2}\gamma_t H_p^2 K_p = 9.09 \text{ t/m}$$

$$S_{ph} = S_p \cos\delta = 7.87 \text{ t/m}$$

c) Weight of the wall.

$$W_m = A_m \gamma_g = 14.85 \text{ t/m}$$

$$M_m = W_m b_m = 1.38\ W_m = 20.49 \text{ tm/m}$$

d) Tension in the anchorage ties.

The tension in the ties is computed assuming that the stress is distributed equally to all the ties and uniformly to the soil ($e = 0$).

If the tension in each of the ties is S, the moment due to all the ties about hinge point F becomes 28 S and therefore:

$$e = \frac{B}{2} - (M_m - M_a + 28\ S)/N = 0$$

where $N = W_m + S_v = 22.3$ t/m and it follows that:

$$S = 1.33 \text{ t/m}$$

e) Final results:

$$\eta_s = \frac{S_{ph} + N \tan\varphi + 7\ S}{S_h} = 1.80$$

$$\eta_r = \frac{M_m + 28\ S}{M_a} = 1.63$$

$$\sigma = N/B = 11.1 \text{ t/m}^2$$

f) Observations

The assumption that all the ties are carrying the same load S is a simplification which eliminates the need to determine exactly to what degree the thin wall has deformed.

Strictly speaking this would be required to evaluate the specific contribution of each anchor. On the other hand the distribution of the soil pressure is influenced by the anchorage elements.

The mechanics of the processes that take place in the structure and backfill can be said to be compatible with limited sliding on the one hand, and with limited rotation, (the stresses and movement are greater at the top), coupled with the distribution of soil pressure which is greater at the base, on the other.

The load is assumed to be acting at the centre of the foundation ($e = 0$) to avoid the tendency of the foundation to rotate upwards, a situation which in the case of a rigid structure would result in the anchors being unloaded.

The panels appear to carry only small loads, i.e. 1.33 t/m, compared to their limit capacity of 5.3 t/m, and this means that the structure has spare capacity, probably in excess of that indicated by the safety factors.

g) Check on essential points.

The stress on anchor No. 6 from the top, is evaluated assuming that it carries the thrust from a vertical strip of soil whose depth is equal to the spacing of 1 m, between the ties. This anchor can be considered to be carrying the greatest stress since the load on No. 7 is decreased by the effect of passive pressure:

$$S^{(6)} \simeq (\gamma_t h + p_0) K_a \times 1 =$$

$$= (1.8 \times 6 + 1.8)(0.2535 \times 1) = 3.19 \text{ t/m}$$

This load is far less than the tensile strength of the panel and can be safely carried by it.

Finally the shear stress is checked at the most critical section $F'A'$:

$$T \cong \left(\frac{1}{2}\gamma_t h^2 + p_0 h\right) K_a \cos\delta - 6\ S = 4.47 \text{ t/m}$$

$$N \cong \left(\frac{1}{2}\gamma_t h^2 + p_0 h\right) K_a \sin\delta + 7\ \gamma_g = 18.73 \text{ t/m}$$

$$\tau_{am} = N \tan\varphi^*/B + c_g = 18.73 \tan 31.25°/1 + 2.1 = 13.47 \text{ t/m}^2$$

$$\tau = T/B = 4.47 \text{ t/m}^2 < \tau_{am}$$

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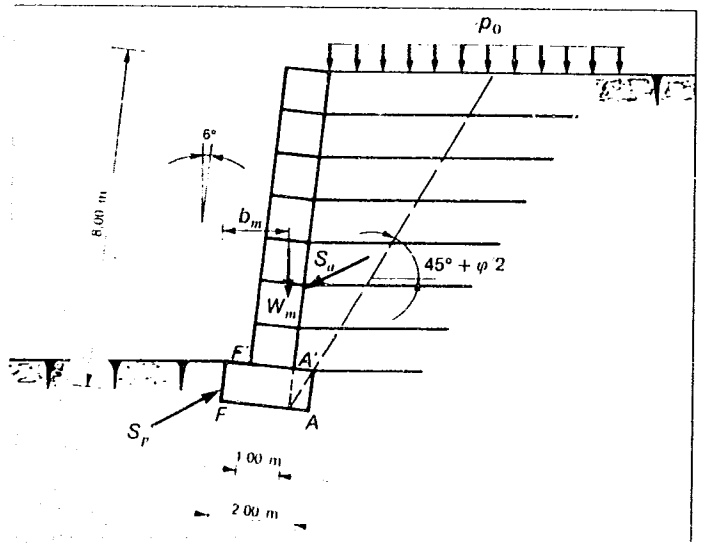

Example 2

Calculations for the design of the retaining wall shown in Fig. 82.

The wall is built with gabions of hexagonal mesh 8 × 10 with wire diameter 3.00 mm. Using the same symbols as in Example 1a, then:

$\gamma_g = 1.60$ t/m³	$\gamma_t = 1.80$ t/m³
$\varphi = 25°$	$G = 25$ t/m²
$c = 1$ t/m²	$c_g = 2.6$ t/m²
$\alpha = 6°$	$H = 5.00$ m
$\varepsilon = 24°$	$p_0 = 0$

The computations are carried out in an identical manner to those in Example 1. Note however that the slope is at an angle $\varepsilon = 24°$, and there is no surcharge. The more important results show:

$W_m = 16$ t/m	$M_m = 29.92$ tm/m
$W_t = 5.85$ t/m	$M_t = 18.66$ tm/m
$W_c = 0$	$M_c = 0$
$W_f = 1.15$ t/m	
$K_a = 0.713$	$K_p = 5.60$
$S_a = 18.27$ t/m	$M_a = -4.30$ tm/m (stabilizing)
$S'_p = M_p = 0$	$S''_p = 0.67$ t/m
$F_i = 16.56$	$F_s = 24.27$ t/m

The value of F_s includes 5.7 t/m which is the stabilising effect provided by the foundation.

$M_i = 0$	$M_s = 55.75$ tm/m
$\eta_s = 1.47$	$\eta_r = \infty$
$N = 31.14$ t/m	
$\sigma_1 = 8.3$ t/m²	$\sigma_2 = 9.5$ t/m²
$\omega = 28.5°$	$\eta_i = 1.37$
$\sigma_{(F,A)} = 9.5$ t/m²	
$\Delta = 0.26$ m	$\tau_{am}/\tau_{(F,A)} = 1.93$

Example 3

The wall shown in Fig. 83 supporting a road is loaded with a uniformly distributed surcharge $p_0 = 2$ t/m² and the soil unit weight is $\gamma_t = 1.8$ t/m³, and $\varphi = 30°$.

It is required to determine the stresses imposed on the wall.

The problem is solved by Culmann's method. See § 4.2.3 and compare with § 4.3.3.

Varying the angle ω, the thrust S which balances the weight of A, B, C, D, is computed:

Per unit length of wall the total vertical load W (soil + surcharge) for $\omega < 59°$ is:

$$W = \left(\frac{12.5}{\tan\omega} - 3\right)\gamma_t + \left(\frac{5}{\tan\omega} - 3\right)p_0$$

and the thrust is:

$$S = W\frac{\sin(\omega - \varphi)}{\cos(\delta + \varphi - \omega)}$$

Using $\gamma_t = 1.8$ t/m³, $p_0 = 2$ t/m², $\delta = \varphi = 30°$, and varying the value of ω with the help of a programmable pocket calculator the maximum value obtained is:

$$S = 5.68 \text{ t/m} \qquad \text{for} \qquad \omega = 46.35°$$

This thrust, assuming it is the resultant of linear distributed pressures, is the value to be used in the stability analysis of the wall. As mentioned in paragraph 4.2.3, this method makes use of all the simplifying assumptions basic to Coulomb's method. In fact the actual stress distribution is more complex than an elementary linear distribution.

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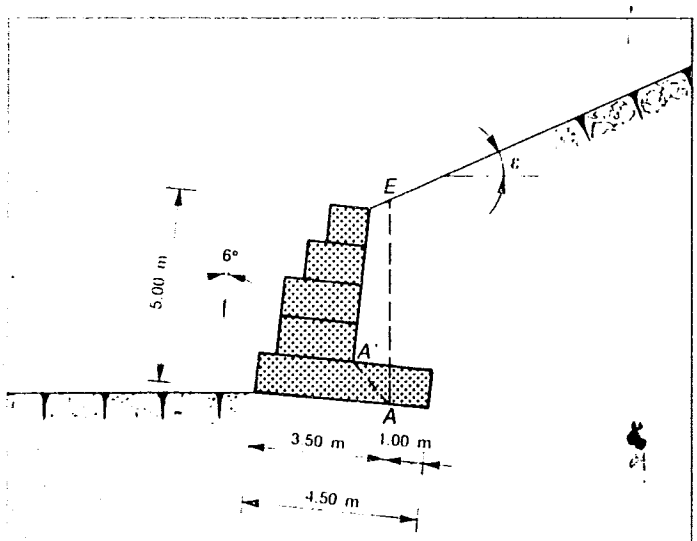

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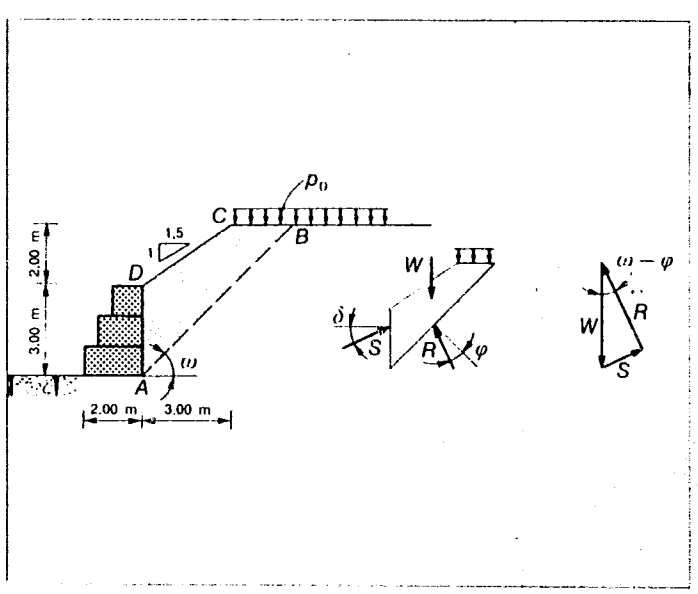


CHAPTER VI

Review of completed projects

6.1 Introduction

In the final chapter of this manual a number of examples of completed projects are illustrated. In the interest of clarity they have been divided into five principal categories:

a) Road and railway work.
   Structures to support slopes and roads
   Bridge abutments and piers
   Culverts
b) Slope and cliff face stabilization
   Works in urban areas
   Works on unstable slopes
   Rockfall and avalanche protection
c) Revetments on embankments and other slopes.
d) River and marine works.
e) Work appertaining to buildings.

6.2 Road and railway work

6.2.1 Structures to retain slopes and roads

84

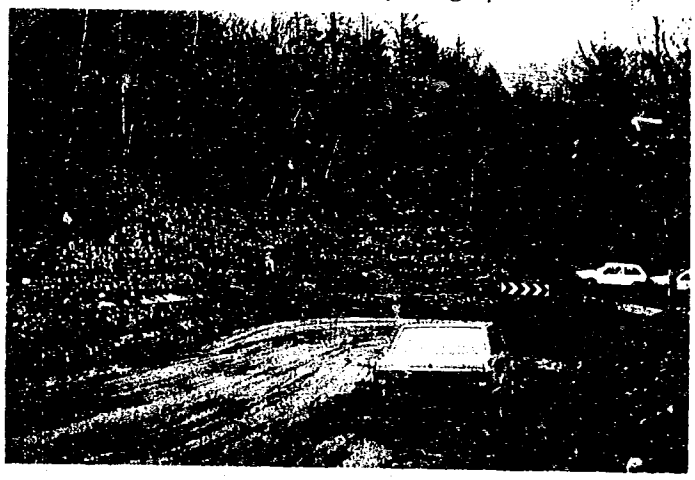
ITALY - Emilia-Romagna - 1913
Structure built along the Setta River to protect the railway embankment on the Bologna-Florence line (Fig. 84).

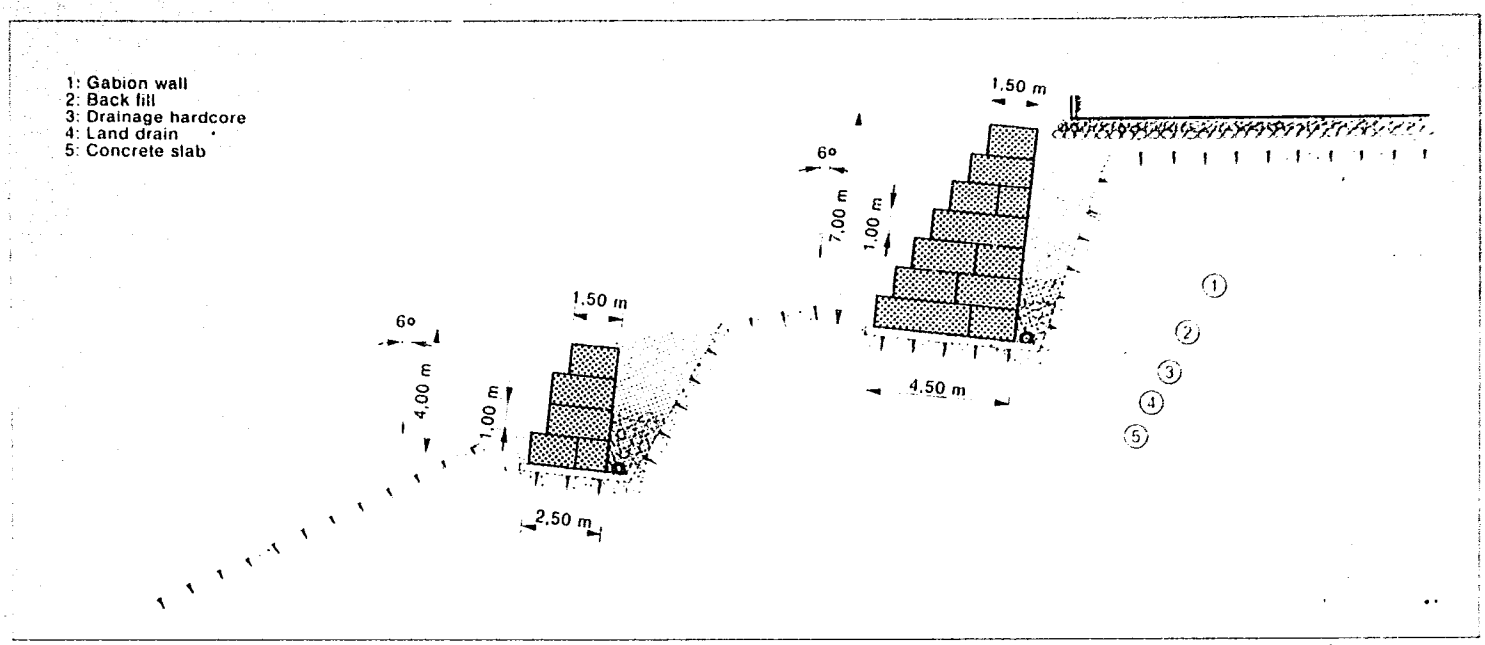
ITALY - Piedmont - 1927
Bulwark protection of the railway embankment on the side of the Rea Stream in Monchiero, Cuneo (Fig. 85).

ITALY - Piedmont - 1960
Wall to support an unstable slope on the Susa-Oulx road, (Fig. 86 - photographed in 1984).

85

86





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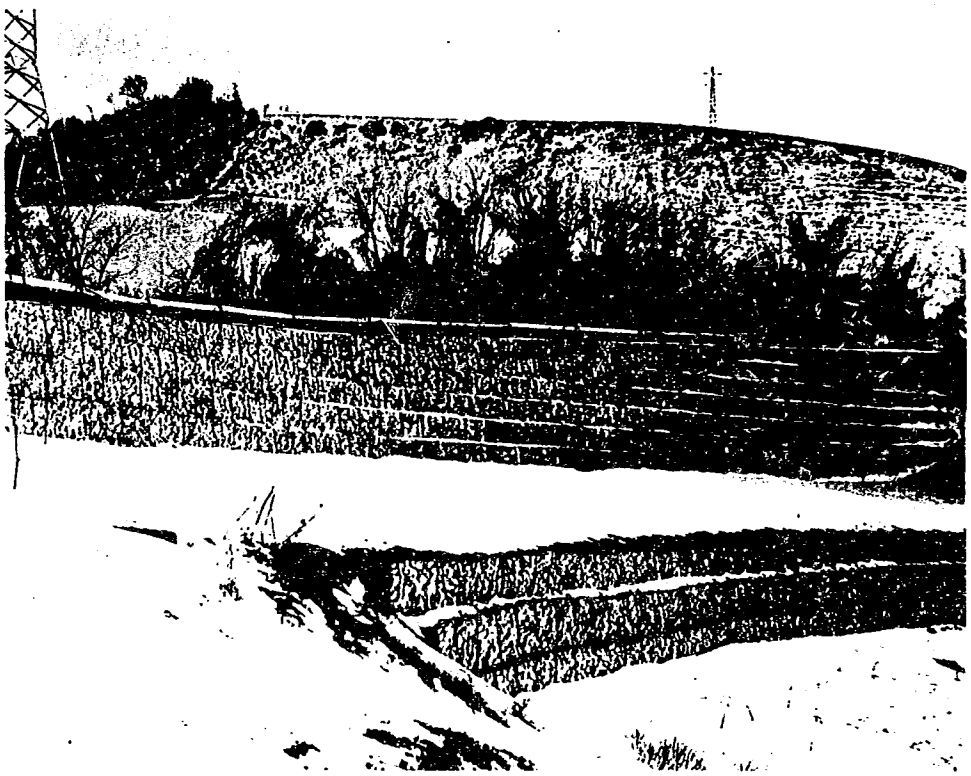

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ITALY - Emilia-Romagna - 1964

Retaining wall for a road embankment located on an unstable slope near Bologna. The schistose clay of this calanque zone is in unstable equilibrium. For this reason it was decided to build two structures, providing upper and lower sections and a double drainage effect.

Figure 87 shows the layout of the scheme: Figure 88 is a photograph taken during the winter shortly after the construction of the walls. It should be pointed out that the gabion structure does not suffer from damage due to frost action. Figure 89 shows the scheme twenty years after its construction. The great success of the project is evident from the growth of vegetation on the embankment and near the wall.

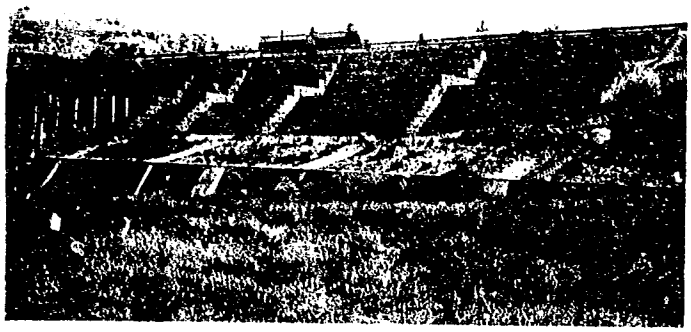
89

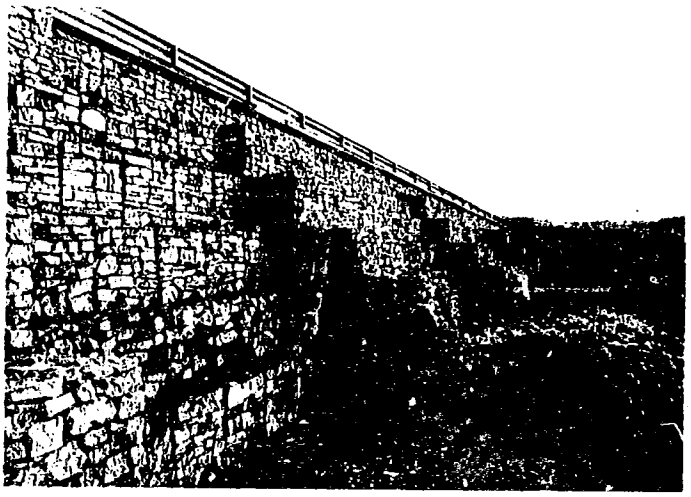
ITALY - Umbria - 1966

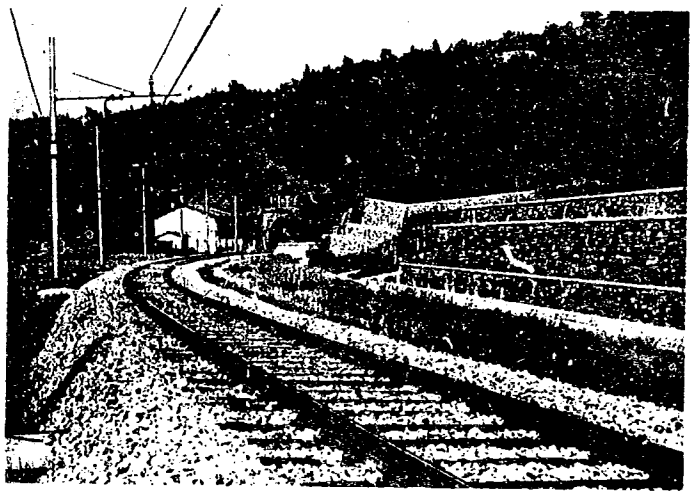
A wall of noteworthy dimensions with external buttresses supporting the embankment of the Autostrada del Sole, in Fabro, province of Terni, between Florence and Rome. (Fig. 90). Figure 91 shows how well gabions can support vegetation (photographed in 1984).

ITALY - Emilia-Romagna - 1969

Wall with stepped rear-face retaining a very unstable schistose clay slope, for which it also provides drainage. Constructed near Vergato in the Province of Bologna, it is perfectly aligned with the road on a double bend, demonstrating the ease with which gabion walls can be adapted to awkward situations (Fig. 92).

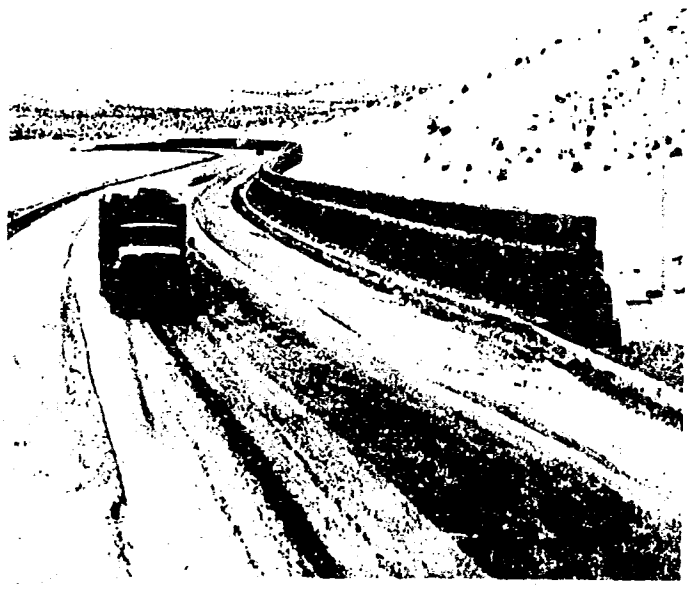






ITALY - Emilia-Romagna - 1970

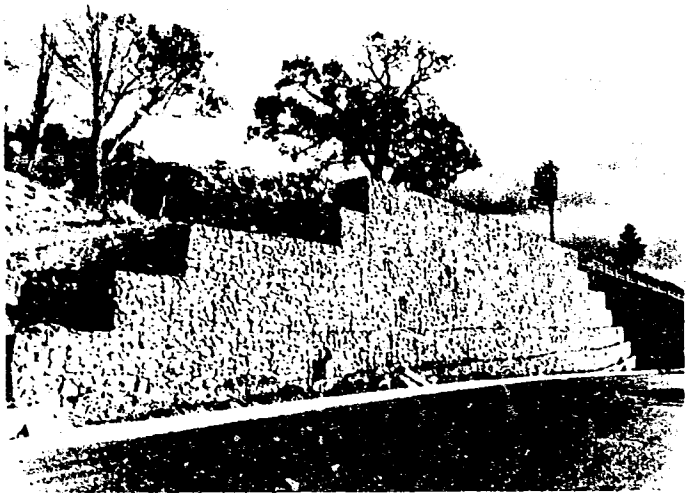
A drainage wall with a stepped front-face safeguarding the Bologna-Pistoia railway in the locality of Silla near Porretta Terme, Province of Bologna. One of the purposes of the structure was to provide drainage to a considerable depth, consequently the foundation is five metres below the level of the track (Fig. 93).

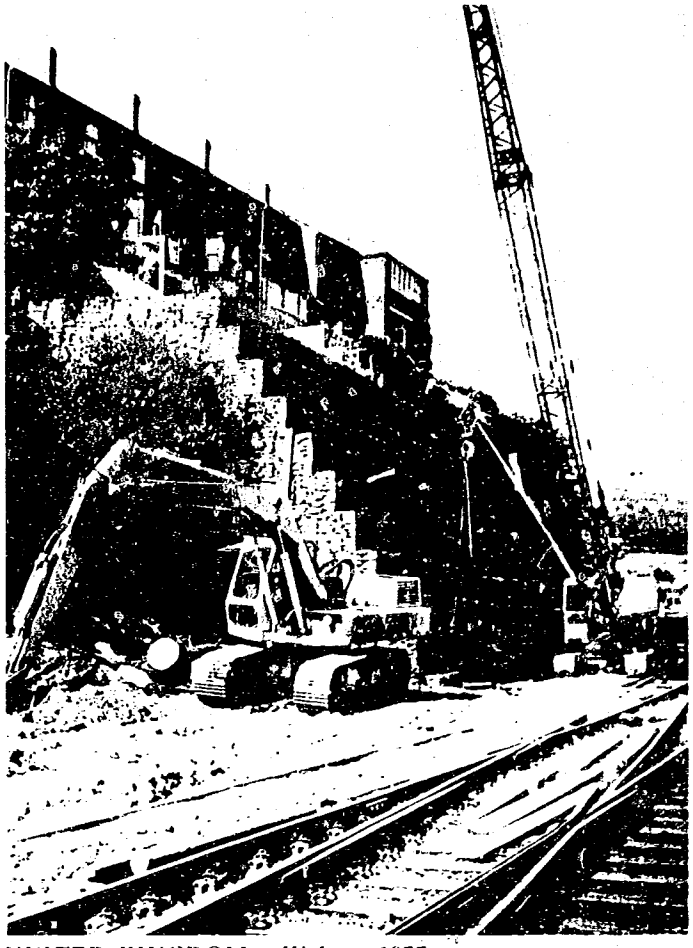


SAUDI ARABIA - 1973

Gabion walls on the Alzulfi-Buraidah road. It is interesting to note that the wall below the road is supporting the embankment, while the one above was constructed to prevent the sand dunes from encroaching on the road (Fig. 94).

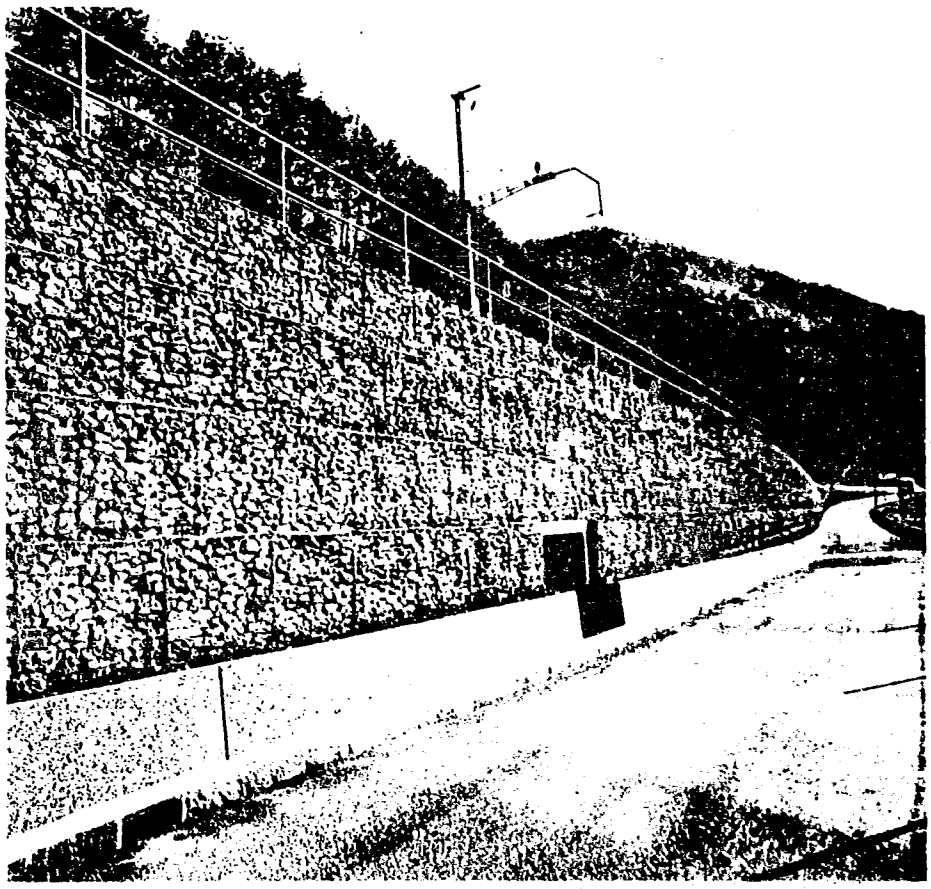
UNITED KINGDOM - England - 1974
Toe wall on the A30 road, near Exeter, Devon (Fig. 95).





UNITED KINGDOM - Wales - 1977
Wall with stepped front-face to protect the railway and to support dwellings in Pontypridd, Mid Glamorgan (Fig. 96).

SWITZERLAND - 1975
An interesting wall near Alpnach supporting the railway that runs parallel to the road located on a lower level. The structure was constructed with PVC coated gabions (Fig. 97).



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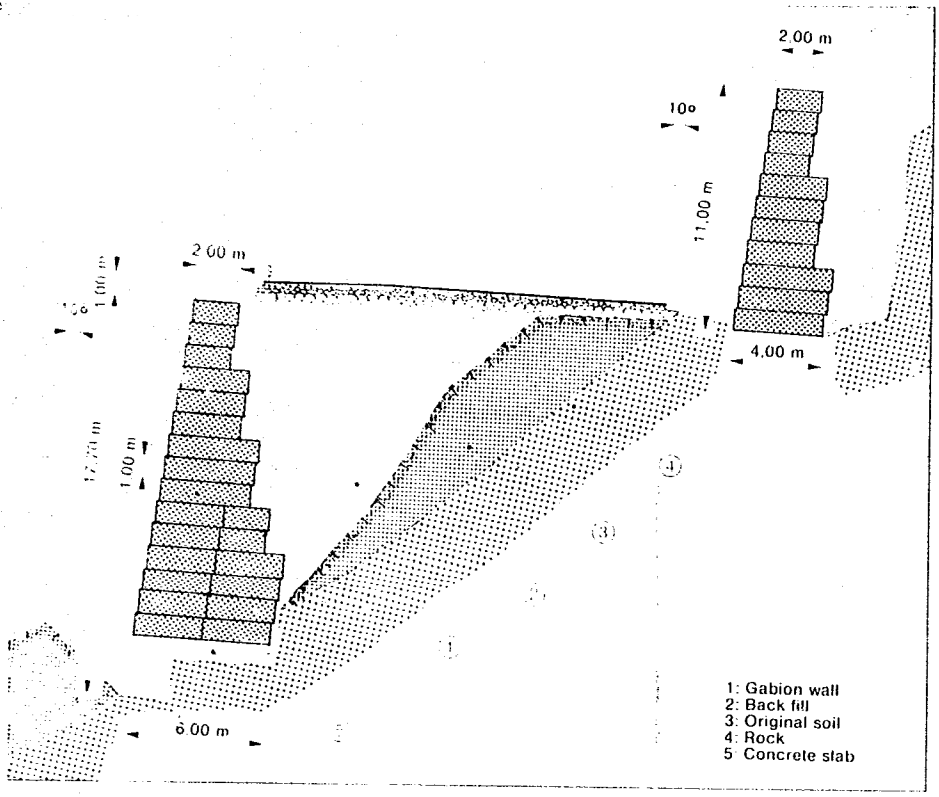

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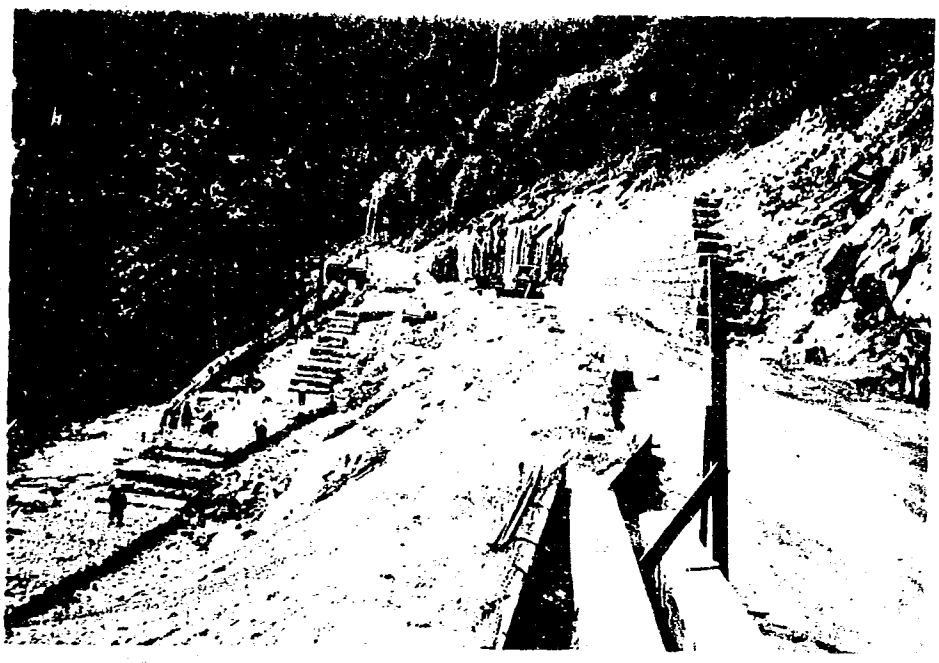


U.S.A - Washington - 1976-1977

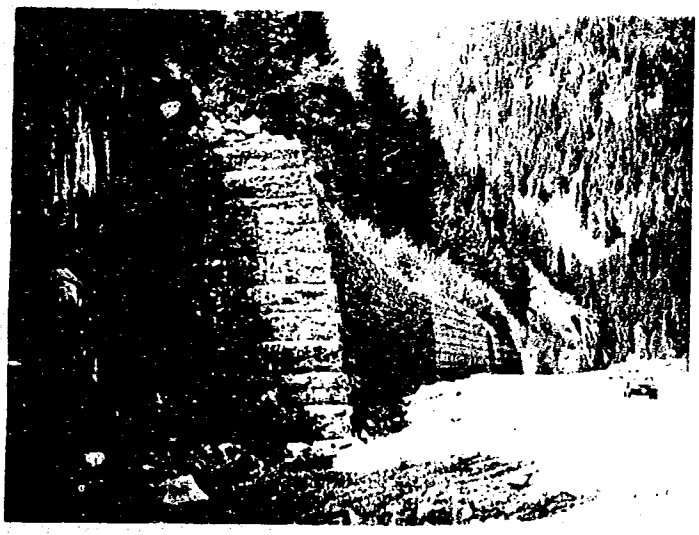
Gabion walls on Interstate Highway 90, near Snoqualmie Pass. The road is the principal east-west corridor in the state, linking urban centres and important tourist areas.

The main concern of the Washington State Highway Department in planning this six-lane highway, was to cause the least possible damage to the surrounding countryside. For this reason, and to minimize the excavation and construction costs, a gabion structure was chosen for the retaining walls.

Along one stretch of road a 15 m high wall supporting the road and an 11 m high wall on the upper side of the road to retain the slope were planned. The walls were founded either directly on bed rock or on a concrete slab on top of the rock where it proved to be too uneven. At other points smaller walls were built on soil consisting mostly of gravel.

The gabions were filled with selected quarry stone and backfilled with stone of about 0.60 m in diameter, dumped to a slope of 1:1.5. The fill was then packed with soil and seeded afterwards. Shrubs were planted to stabilize the surface of the embankment. Figure 98 and 99 illustrate typical cross sections of walls and their construction. Figure 100 shows a cross section of a wall supporting the slope, while Figure 101 shows the same wall after seven years.

100



101

SWEDEN - 1976

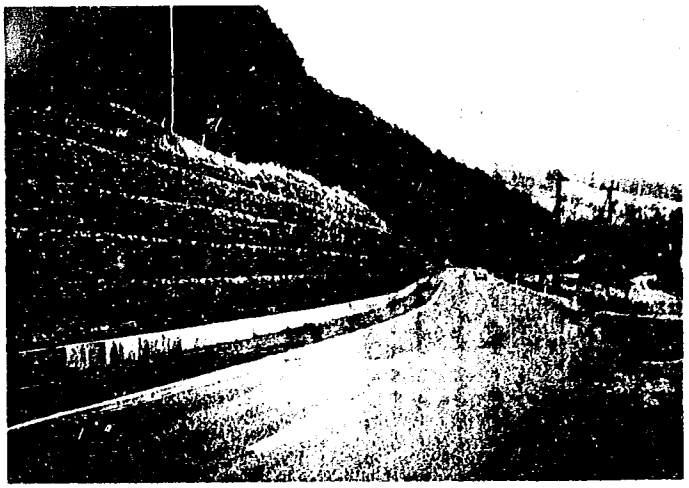
Stepped front-face wall in Sundswall in Västernorrland. (Fig. 102).

102

SWITZERLAND - 1977

Imposing stepped front-face wall in Tosstal Bauma (Fig. 103).

103



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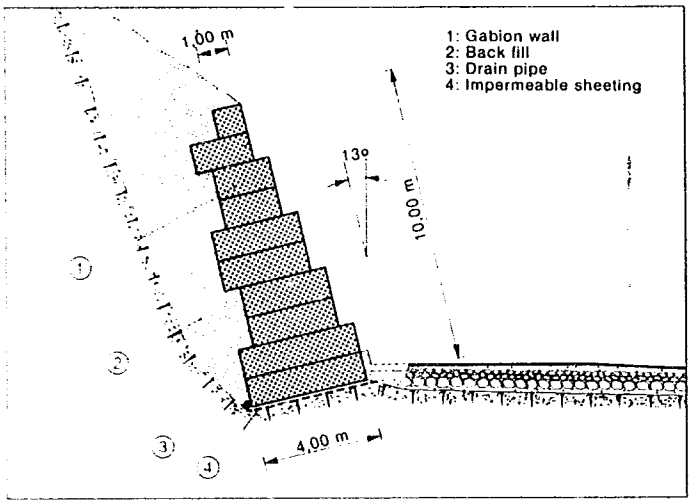

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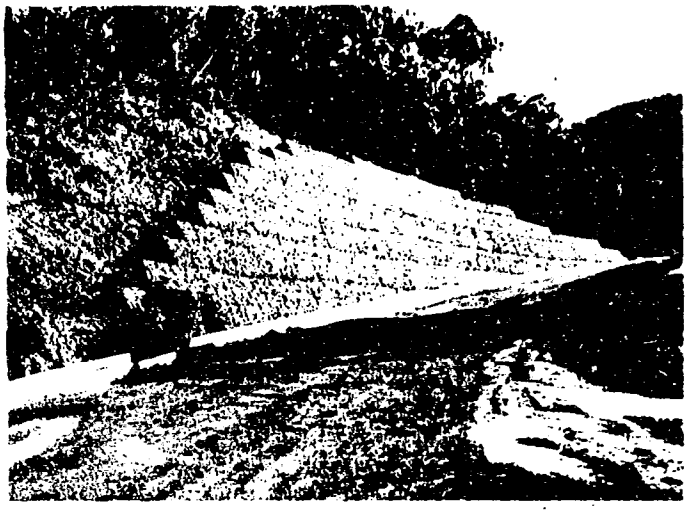
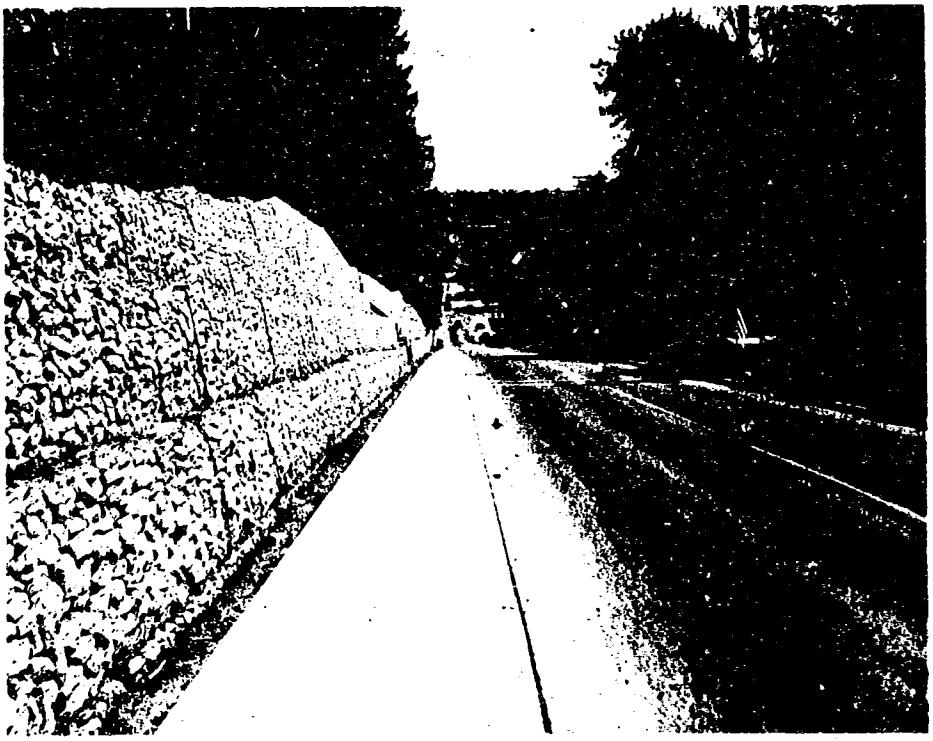


AUSTRALIA - 1977

Wall installed during the widening of State Highway 95 at Mount Ousley, not far from the city of Wollongong in New South Wales.
Figure 104 illustrates the cross section of the wall;
Figure 105 shows it just after completion.

106



CANADA - Ontario - 1978

Walls to slopes on both sides of the road in Owen Sound (Fig. 106).

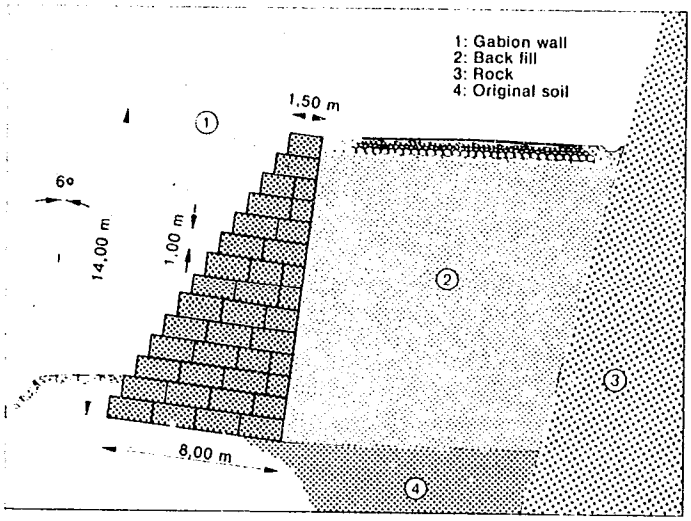
JORDAN - 1978

Stepped front-face wall, 14 m high, on the Amman-Aqaba road near Maan Junction. Figure 107 shows a cross section of the project. The road, cut into the rock on the side of the mountain, crosses a deep gully having a silty sand bed. The wall was built to bridge across the gully and the road laid on material imported for the purpose placed between the wall and rock face.

Because of the bearing capacity of the foundation soil (about 1 kg/cm^2) a gabion structure was chosen for its ability to distribute pressure uniformly and to adapt to any future settlement (Fig. 108).

Figures 109 and 110 show a 6 m wall supporting the slope along another stretch of the road, during construction and on completion, respectively.

10



108

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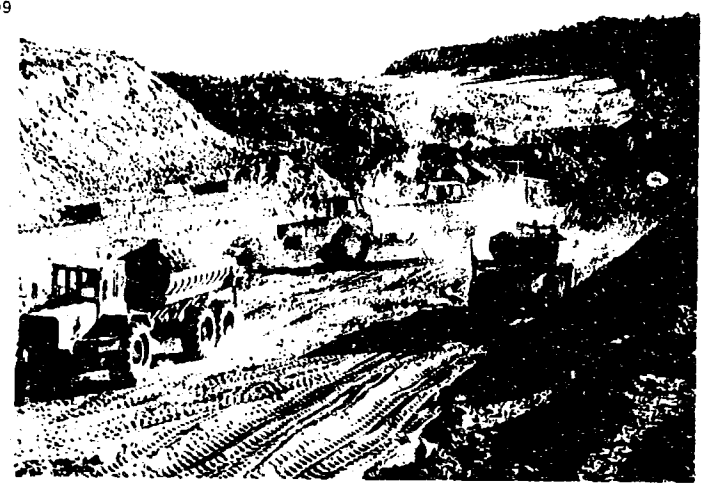

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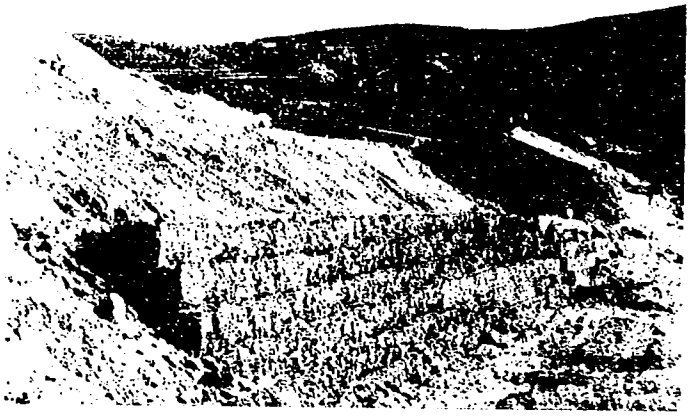

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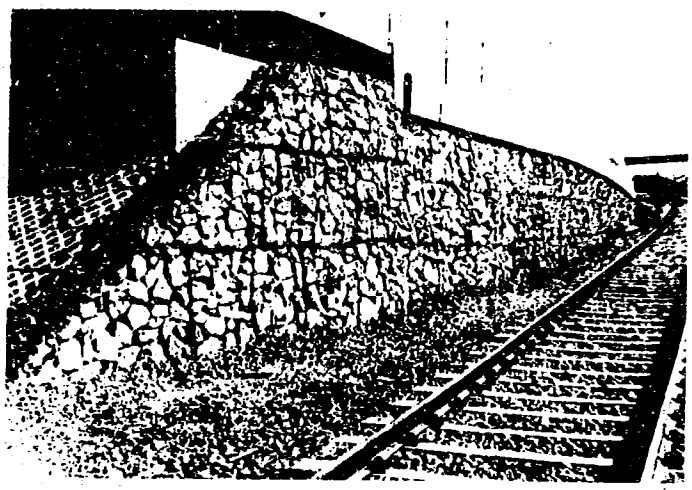


FINLAND - 1978

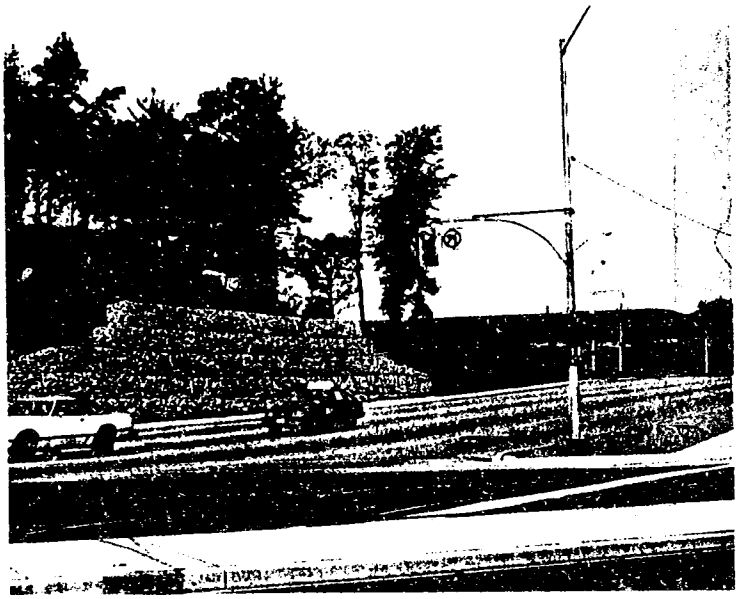
Retaining wall on an urban railway supporting the end of a flyover in Helsinki (Fig. 111).

112

UNITED KINGDOM - England - 1979

Retaining wall supporting a slope along a steep road near Milton Coombe in Devon (Fig. 112).

113



CANADA - Ontario - 1979
Toe wall to a slope in Toronto (Fig. 113).

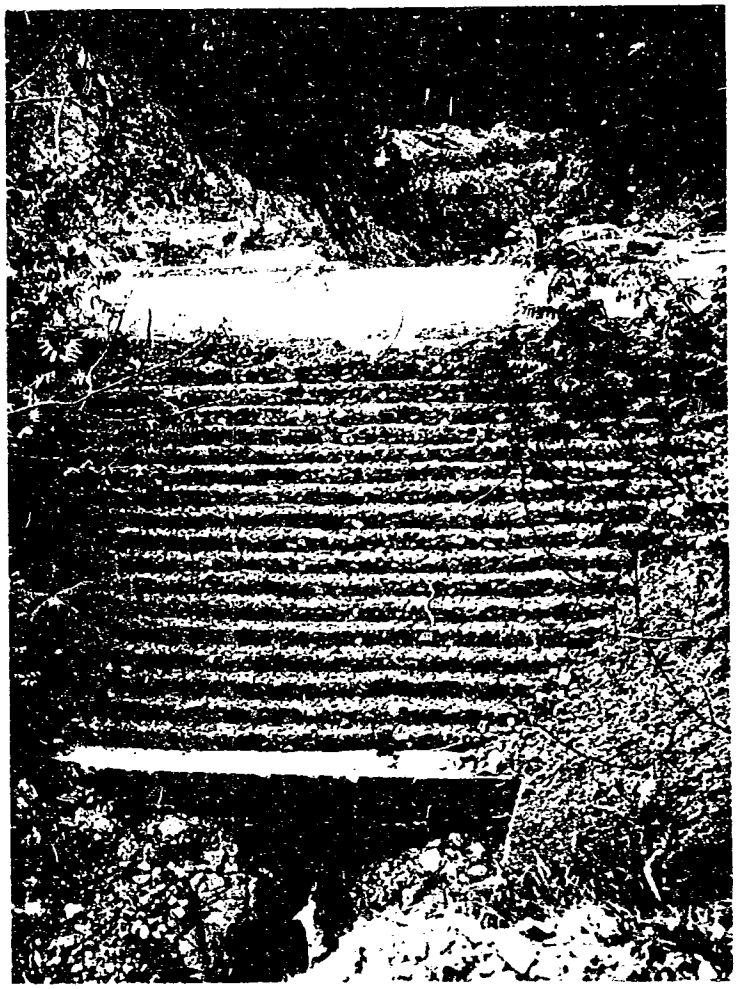
114

BOLIVIA - 1980
Wall supporting a steep slope above a road in La Paz (Fig. 114).

115

AUSTRALIA - 1981
Wall with rear stepped face at Guildford Station in the western suburbs of Sydney, N.S.W. (Fig. 115).

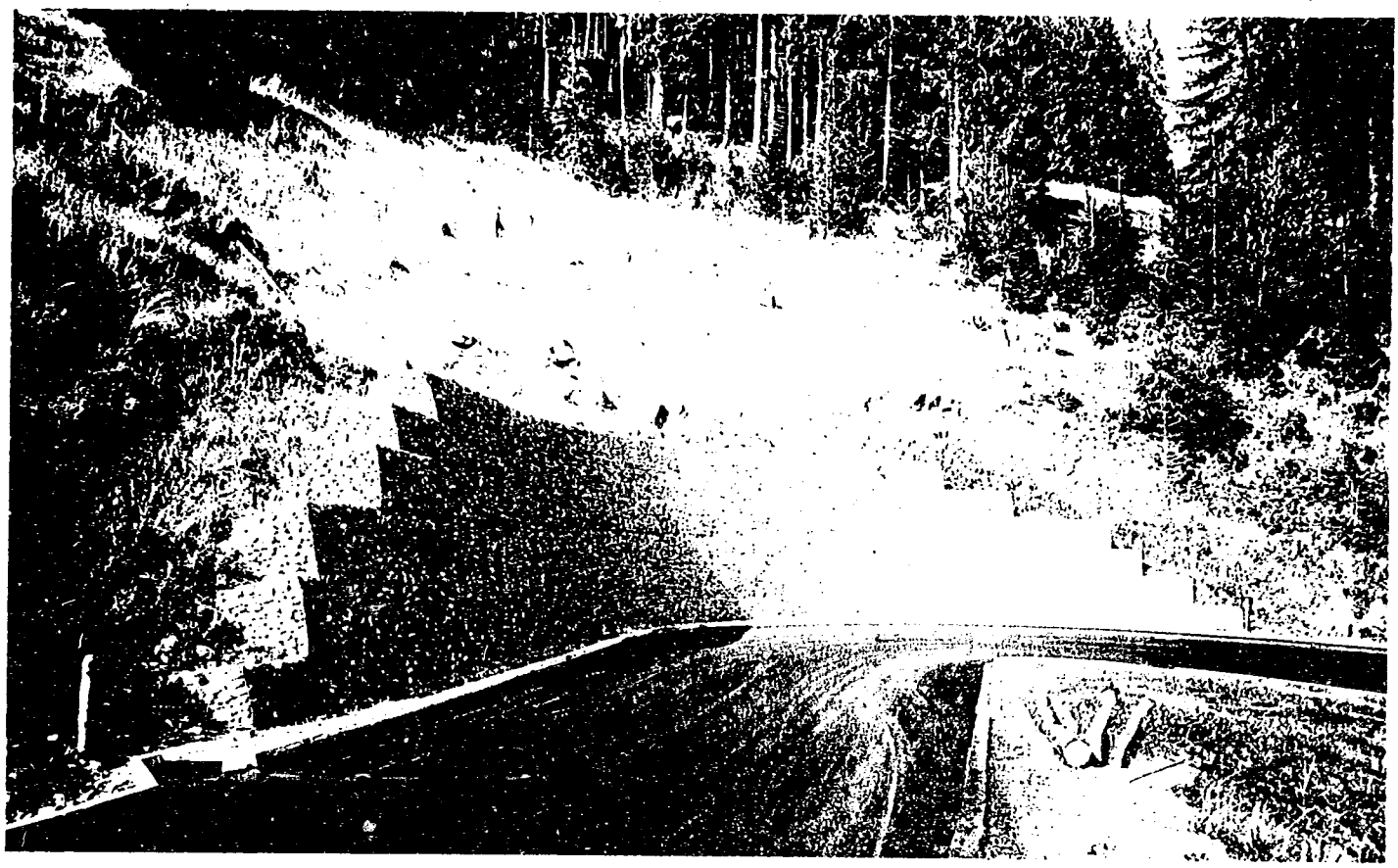
116



DOMINICAN REPUBLIC - 1981
Wall to a road embankment crossing a deep gully (Fig. 116).

SAUDI ARABIA - 1982

Walls both above and below the road along a stretch of the "Shaar descents" (Fig. 117).

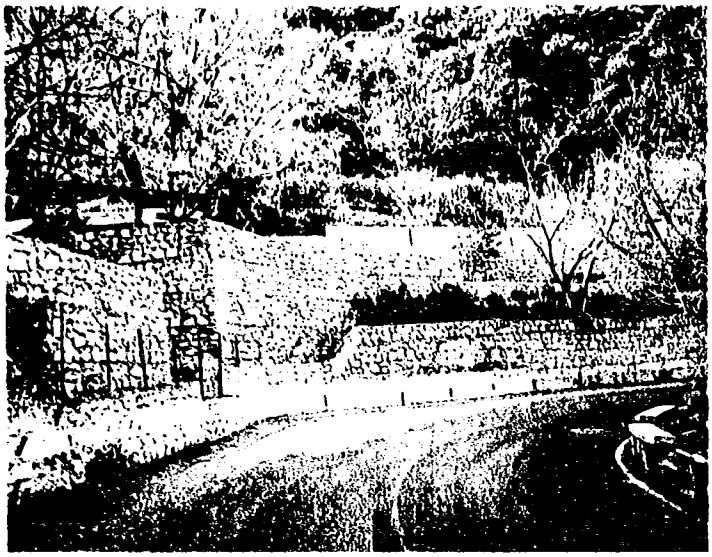


SWITZERLAND - 1982

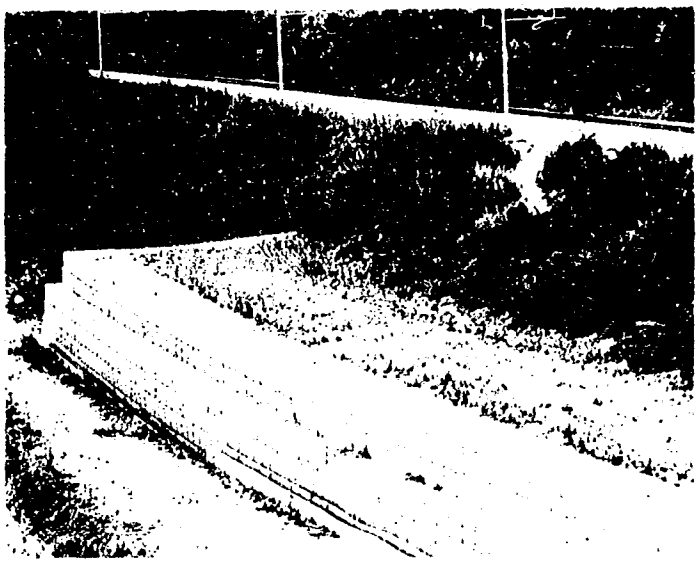
Wall on a road in Sarnen Entlebuch. Worth noting is the excellent workmanship on the structure along the curve and its perfect integration into the natural environment (Fig. 118).

KENYA - 1983

Stabilization of the slopes by means of gabion stepped front-face walls along the Thuchi-Nkubu road under construction (Fig. 119).



ITALY - Campania - 1982
Series of walls to stabilise the slopes along State road 163 Amalfitana near Naples (Fig. 120).



ITALY - Campania - 1982
Wall supporting the road at the side of the Caserta-Benevento railway (Fig. 121).

SWITZERLAND - 1982
Wall retaining the slope above a road in Birmensdorf. Earth containing seed was placed in the gabions with the stone fill during construction. The resulting vegetation ensures a pleasing appearance and the integration of the structure into the local environment (Fig. 122).

U.S.A. - Kentucky - 1982

Slope retention beside the railway near Corbin (Fig. 123).

SAUDI ARABIA - 1983

Longitudinal wall with stepped front-face to protect a section of the Zayma-Al Sayl-Al Kabir road that runs parallel to a «wadi» (Fig. 124).

123

124

125

126

127

DOMINICAN REPUBLIC - 1983

Wall below a road embankment.
The structure also functions as a weir (note the lateral crest) which carries water from the culvert under the road and discharges it into a deep ditch (Fig. 125).

SWITZERLAND - 1983

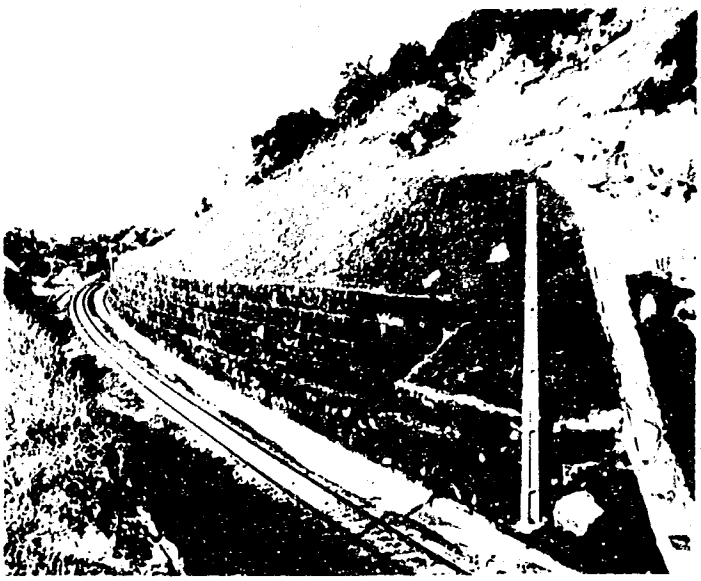
Wall supporting the road near Lake Sarnen (Fig. 126).

BRAZIL - Rio Grande do Sul - 1983

Wall supporting the slope in front of the piers of a federal railway viaduct near Santa Maria (Fig. 127).

BRAZIL - Minas Gerais - 1983

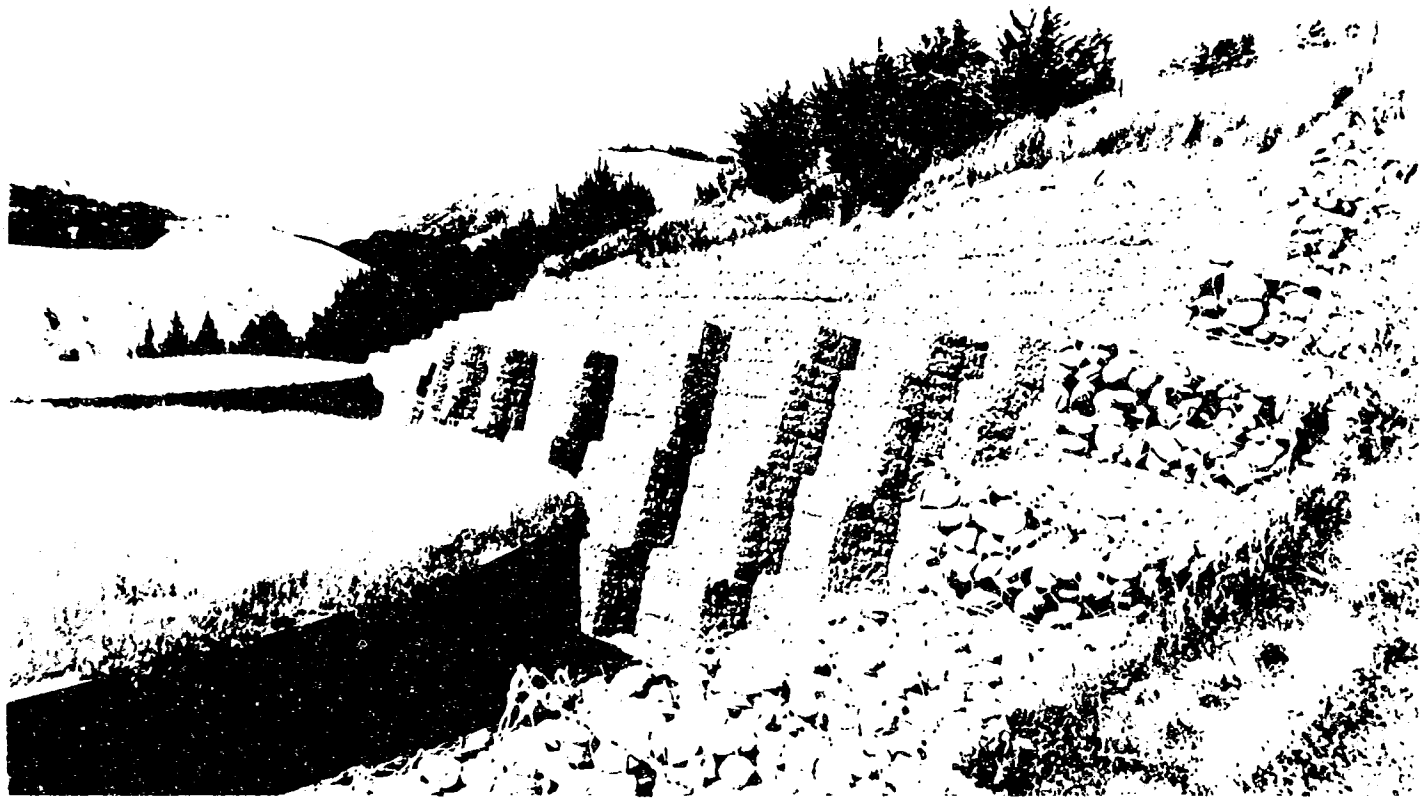
Wall at km. 15 of the ES 050 Araguari-Goiandira railway, built by the Second Railway Battalion of the Brazilian Army (Fig. 128).

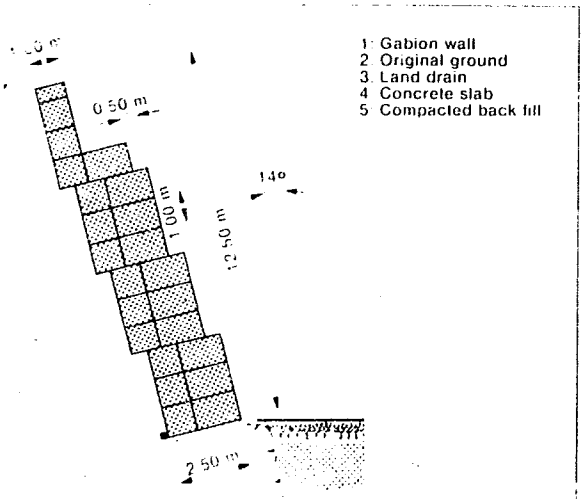


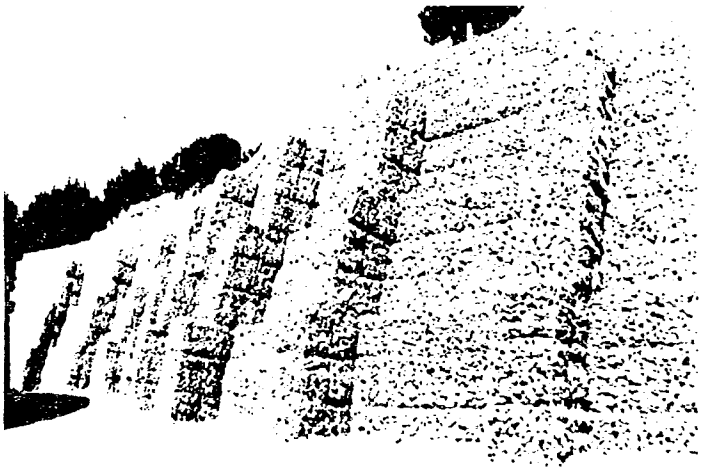
NEW ZEALAND - 1983

Imposing counterfort wall at Worsley Spur Reservoir, Christchurch.

Figures 129 and 130 show a view from above and a cross section, and Figure 131 a close up of part of the work.







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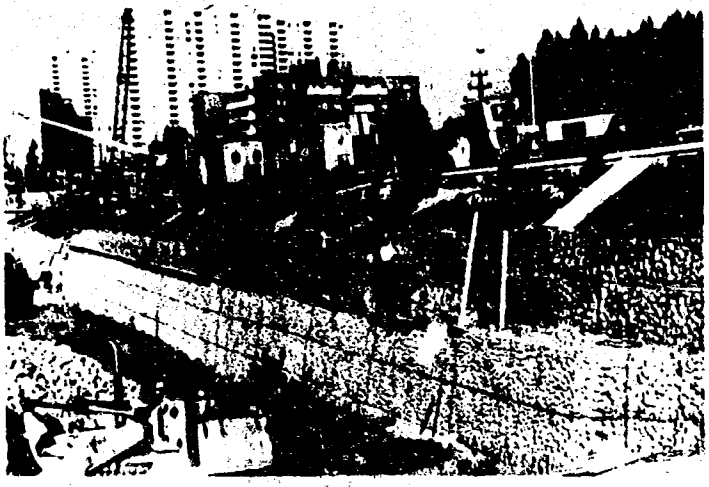

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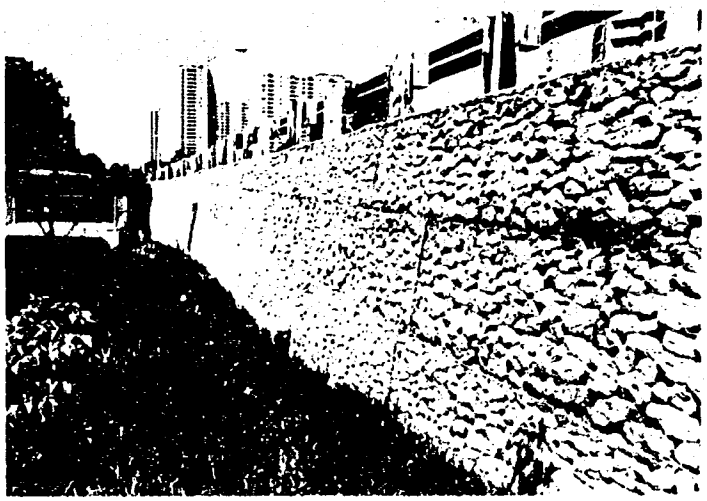


CANADA - Ontario - 1977-1983

Along side a wall built in Toronto in 1977 to support the road (Fig. 132); a second wall of gabions was built so the road could be widened (Fig. 133).

SENEGAL - 1984

Wall on the East Tangential road to Dakar (Fig. 134).

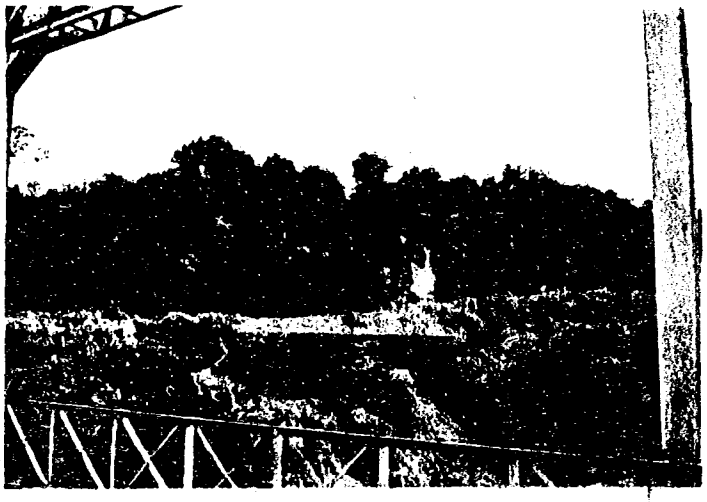
ITALY - Basilicata - 1984

Stepped front-face wall, under construction, on the Potenza-Ferrandina railway (Fig. 135).

134

135

136



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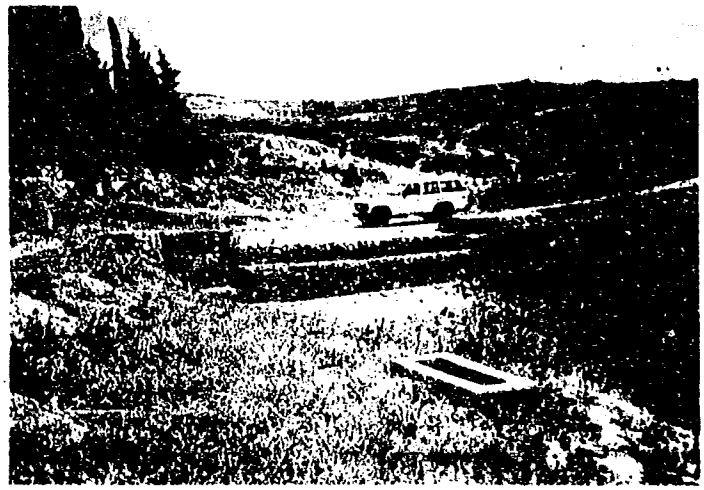


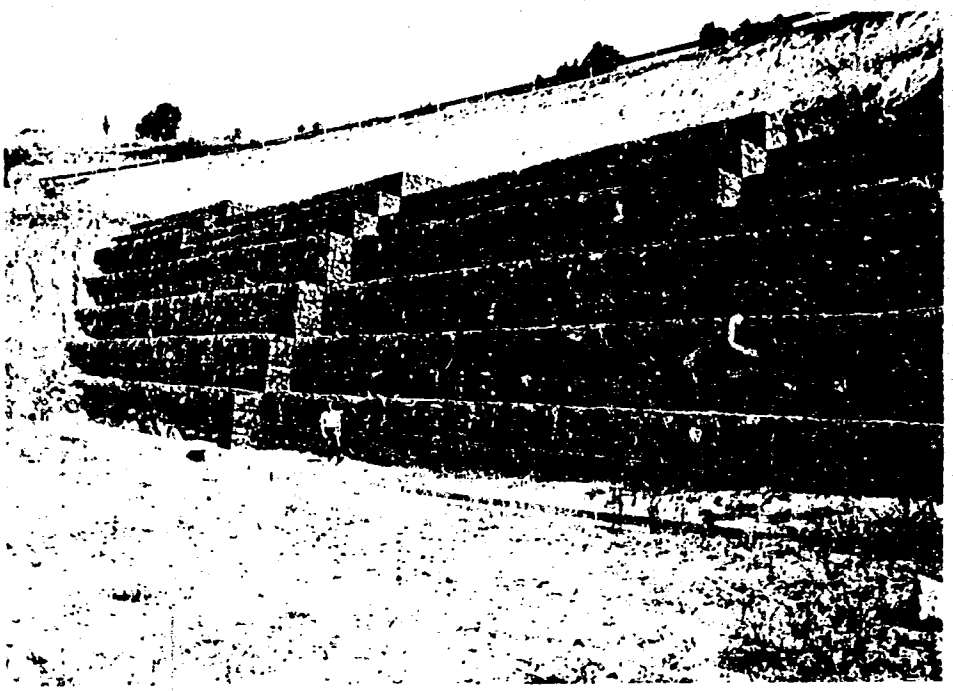
COSTA RICA - 1984

Wall supporting a road on the Barranca River in Puntarenas (Fig. 136).

JORDAN - 1984

Completed works on a road embankment where a stream crosses the alignment (Fig. 137).

138



CAMEROUN - 1984

Embankment walls on the Bafoussam to Bamenda Road (Figs. 138, 139).

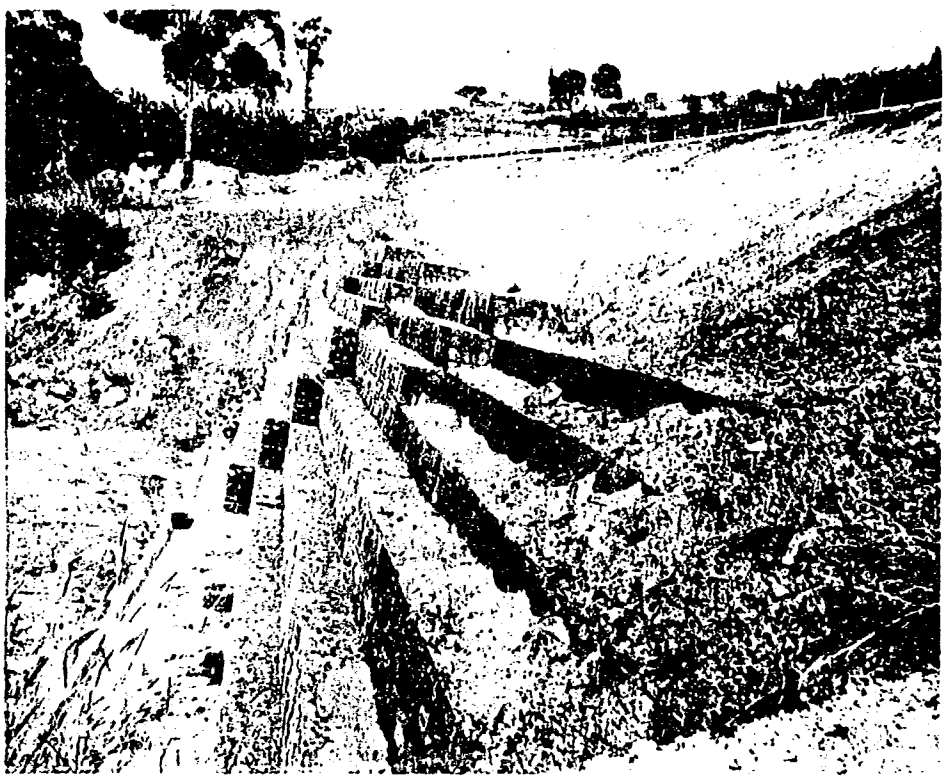
CAMEROUN - 1984

Wall at the toe of a slope on the Eseka to Maloume section of the Trans-Cameroun Road (Fig. 140).

FEDERAL REPUBLIC OF GERMANY - 1984

Wall to protect the slopes overlooking the Ulm-Wuerzburg highway in Reichenbach-Aalen seen at the beginning of the project (Fig. 141).

139



140

141

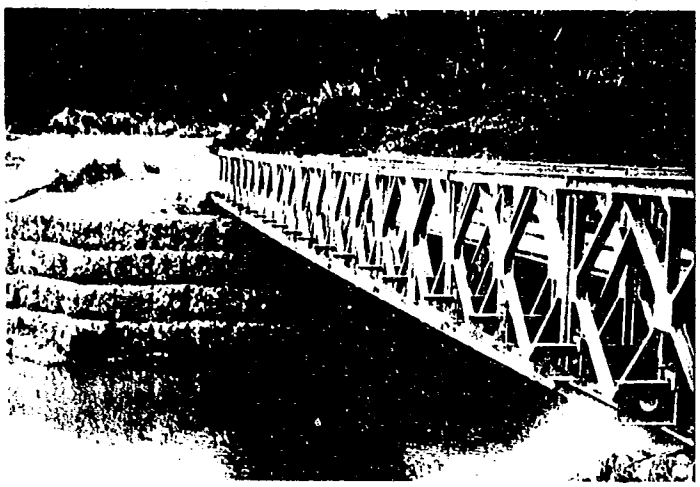
PAPUA - NEW GUINEA - 1975

Gabion abutments for a Bailey bridge on the Kwatit River (Fig. 148).

CANADA - Ontario - 1978

To protect the abutments of a bridge, this interesting structure is joined together, both upstream and downstream, by a simple revetment (Fig. 149). The project was constructed on Mullet Creek in the city of Mississagua.

148



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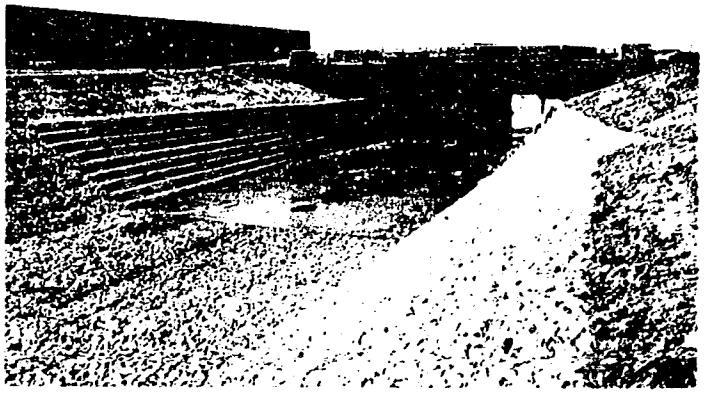

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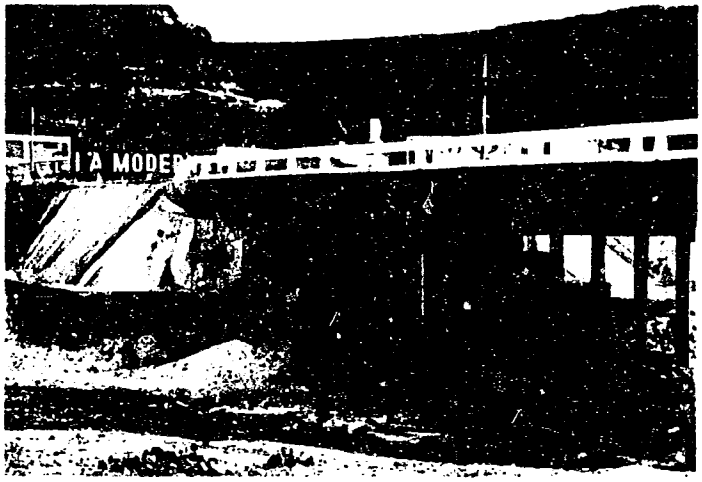


DOMINICAN REPUBLIC - 1979

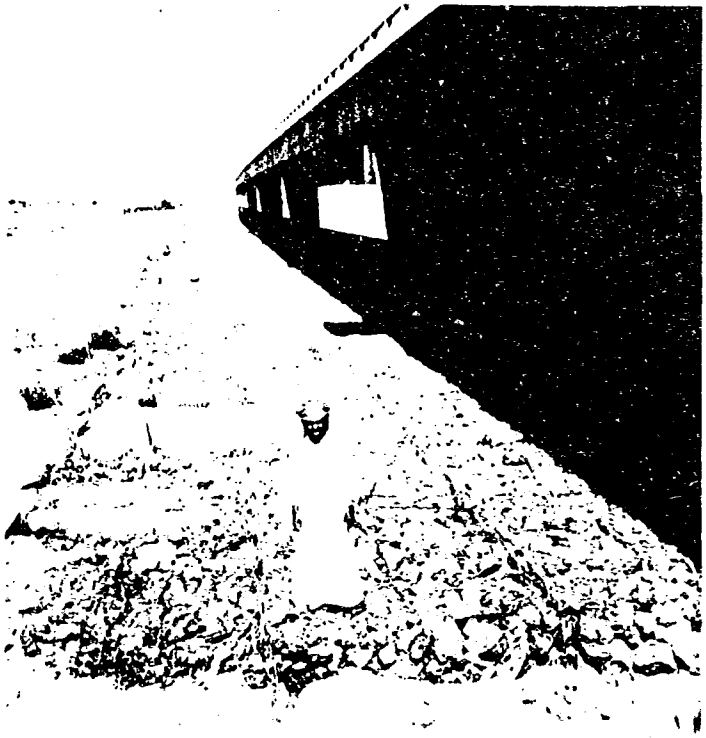
Gabion abutments for temporary bridges built after Hurricane David (Fig. 150).

GUATEMALA - 1980

Wall to protect the abutments of a bridge (Fig. 151).

152

153



SAUDI ARABIA - 1980

Long wall protecting the bridge abutments and the embankment on the Jumum to Zaymah Feeder Road (Fig. 152).

SAUDI ARABIA - 1980

Protection of bridge piers on the Jeddah-Meccah

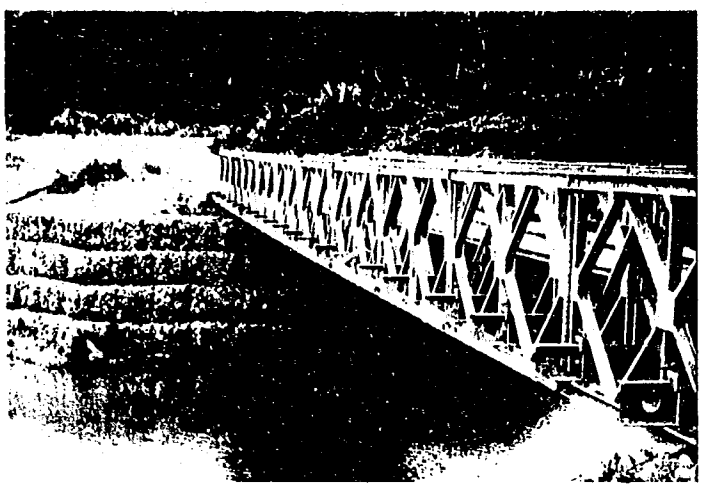
PAPUA - NEW GUINEA - 1975

Gabion abutments for a Bailey bridge on the Kwatit River (Fig. 148).

CANADA - Ontario - 1978

To protect the abutments of a bridge, this interesting structure is joined together, both upstream and downstream, by a simple revetment (Fig. 149). The project was constructed on Mullet Creek in the city of Missisagua.

148



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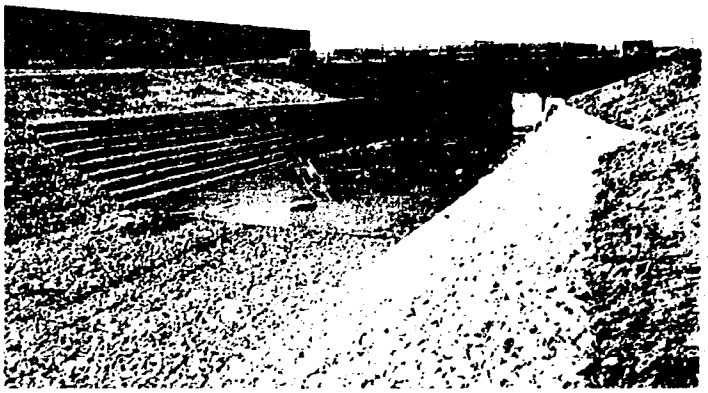

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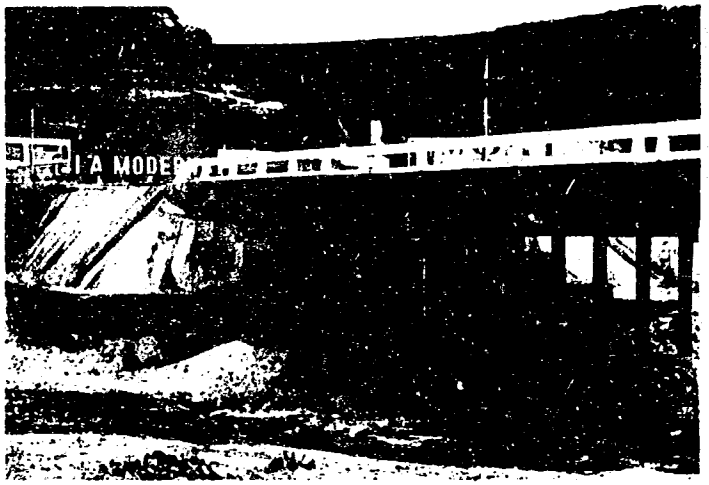


DOMINICAN REPUBLIC - 1979

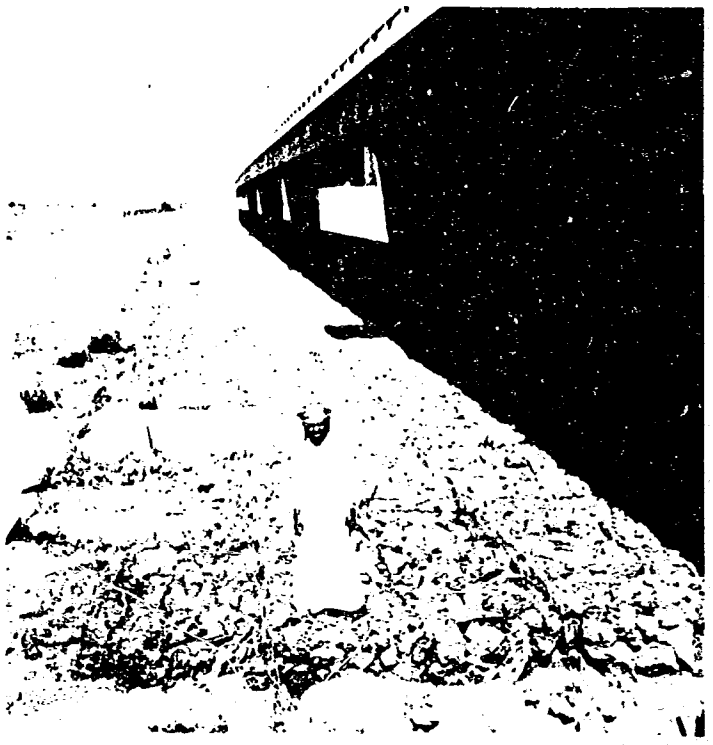
Gabion abutments for temporary bridges built after Hurricane David (Fig. 150).

GUATEMALA - 1980

Wall to protect the abutments of a bridge (Fig. 151).

152

153



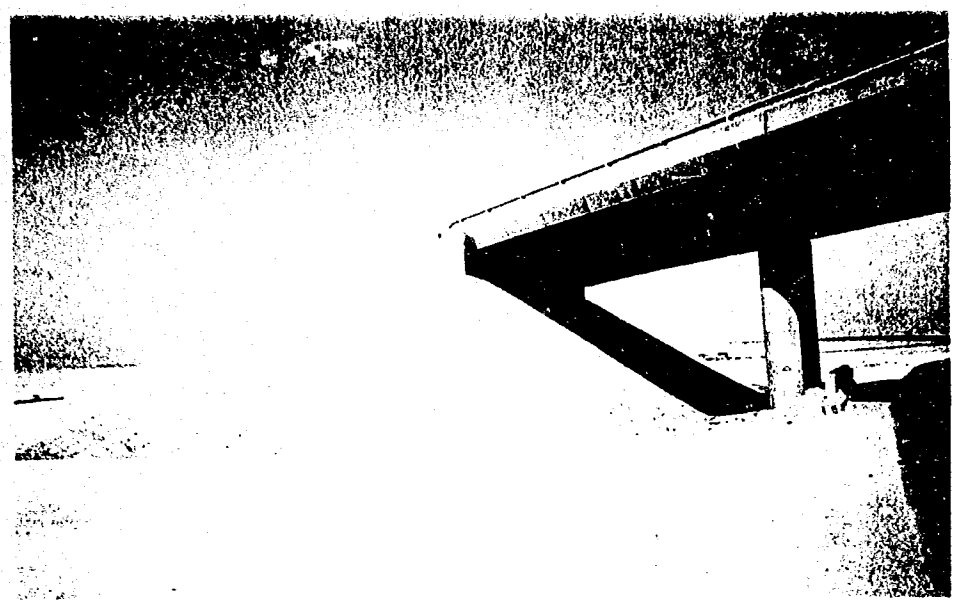
SAUDI ARABIA - 1980

Long wall protecting the bridge abutments and the embankment on the Jumum to Zaymah Feeder Road (Fig. 152).

SAUDI ARABIA - 1980

Protection of bridge piers on the Jeddah-Meccah Expressway.

154



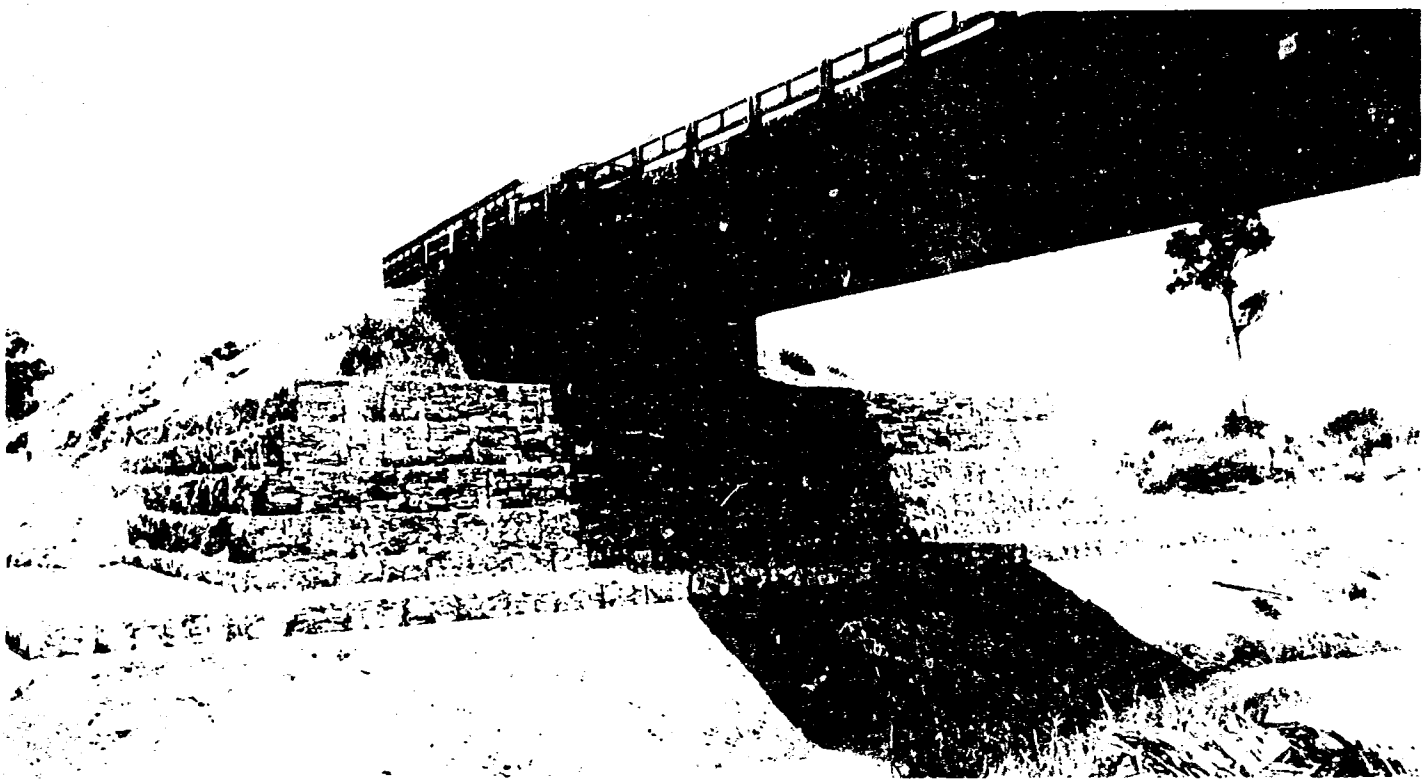
SAUDI ARABIA - 1982

Protection of bridge abutments on the Riyadh-Dammam Road (Fig. 154).

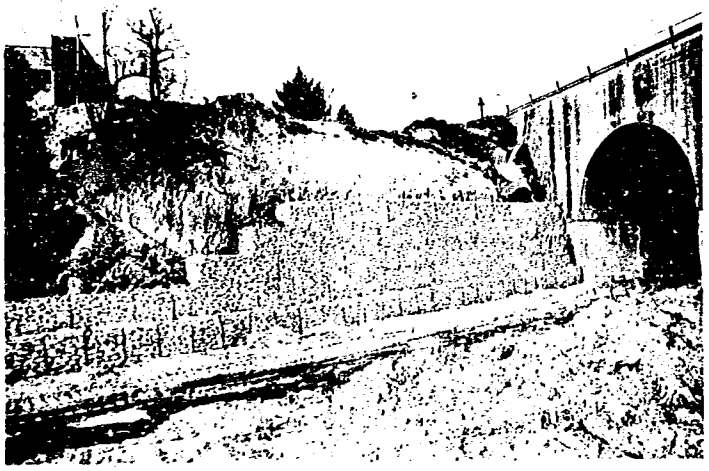
BRASIL - Goias - 1982

Stepped front-face wall to protect a bridge abutment in Mozarlandia. The wall is founded on a gabion apron 0.50 m thick which will check erosion at the toe as it flexes downwards (Fig. 155).

155



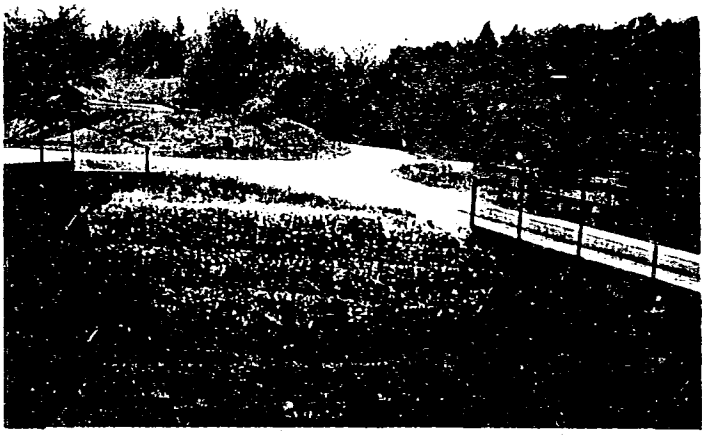
156



ITALY - Abruzzi - 1984

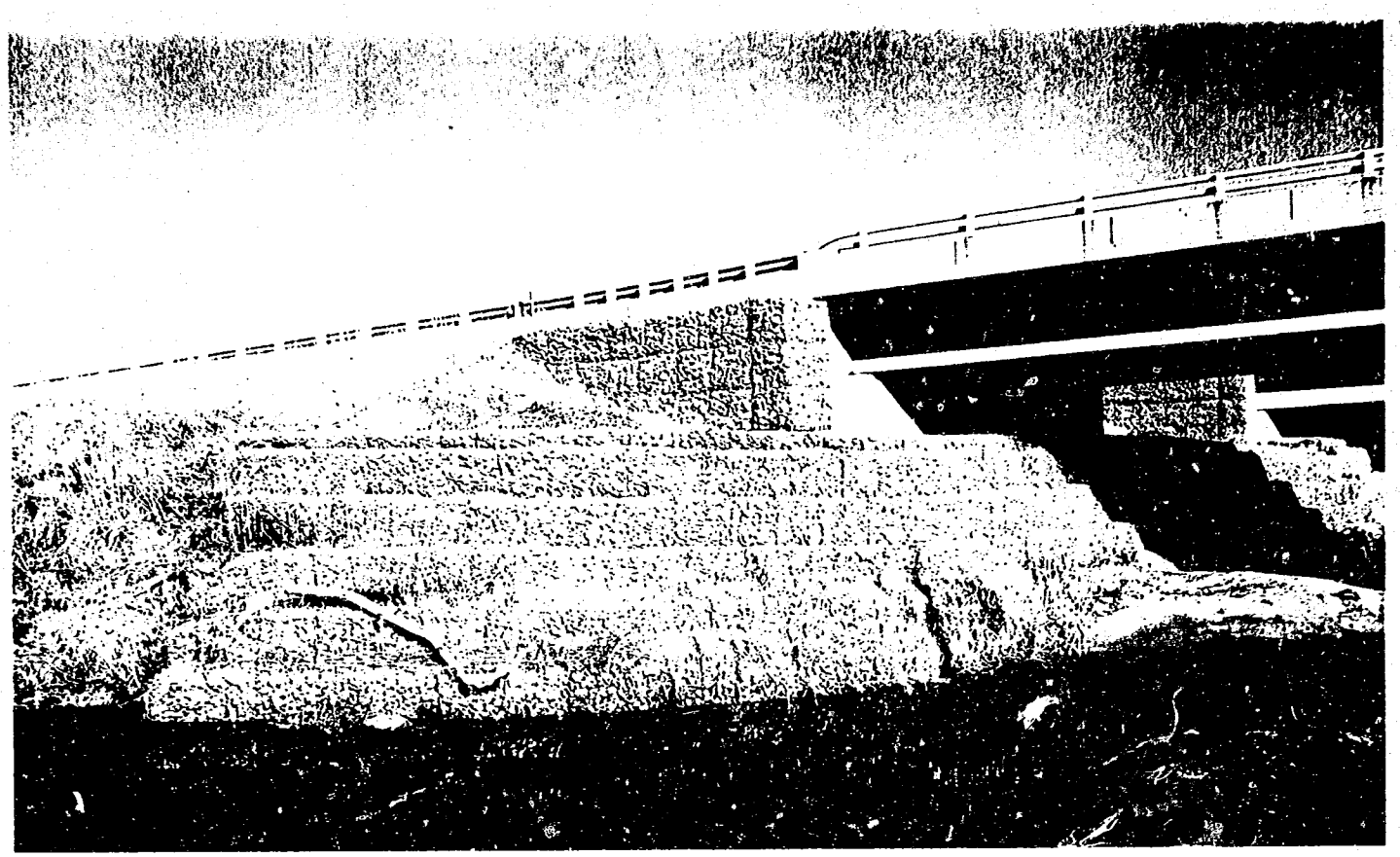
Abutment protection at a bridge on Paradiso stream near Chieti (Fig. 156).

157



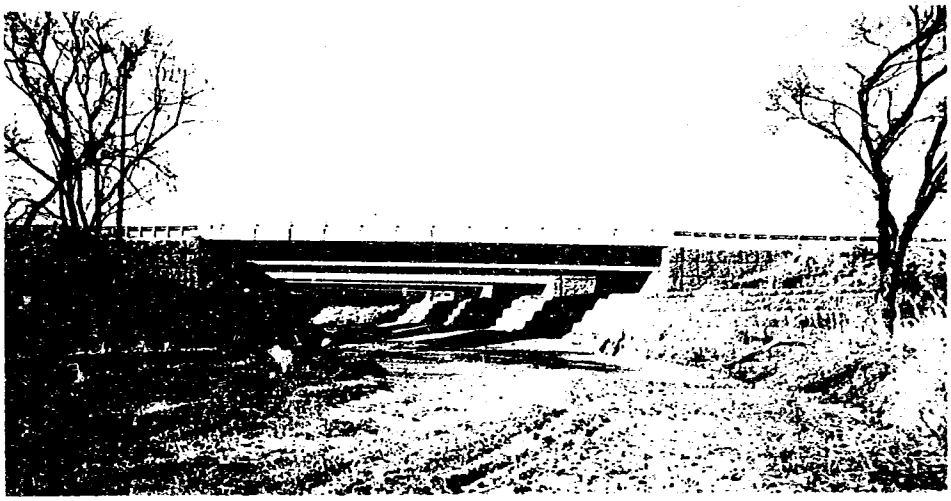
CANADA - Ontario - 1984

Gabion wall protecting bridge abutments near Toronto (Fig. 157).



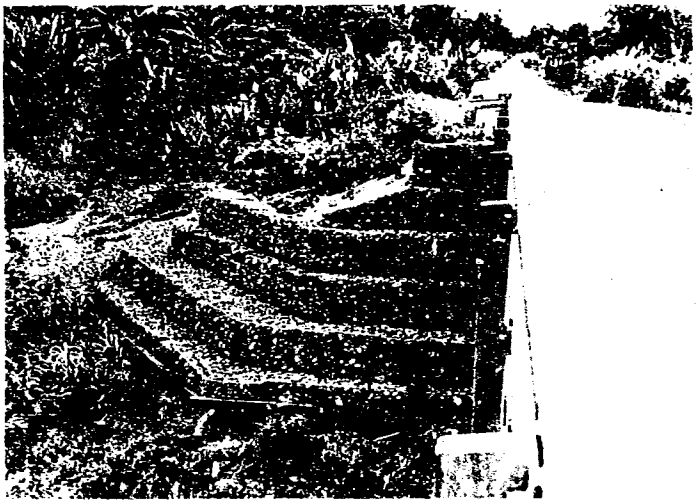
U.S.A. - Oklahoma - 1984

Imposing wall built to protect the abutments of four adjacent bridges on Interstate Highway 35 near Oklahoma City.
Figure 158 shows a detail of the project. Figure 159: a panoramic view.



PAPUA - NEW GUINEA - 1984

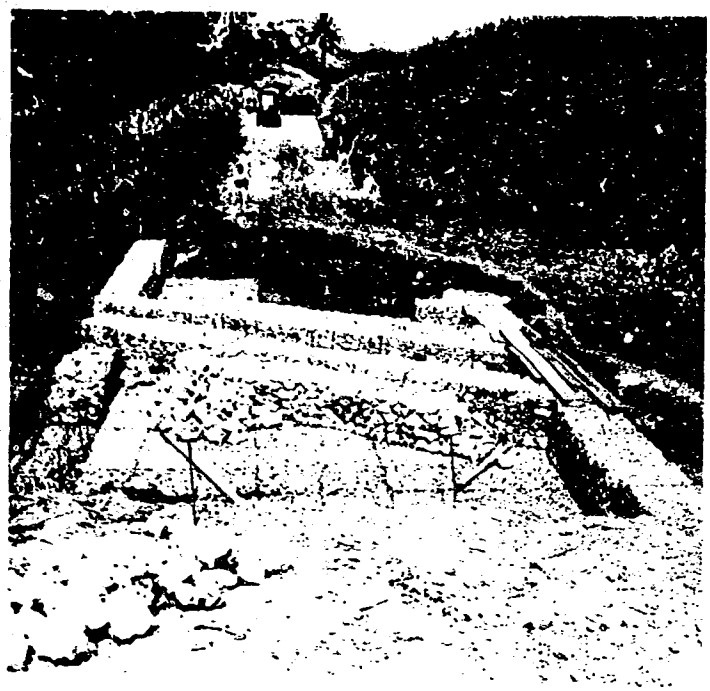
Revetment protecting the end of a bridge on the Mount Hagen to Wabag Road (Fig. 160)

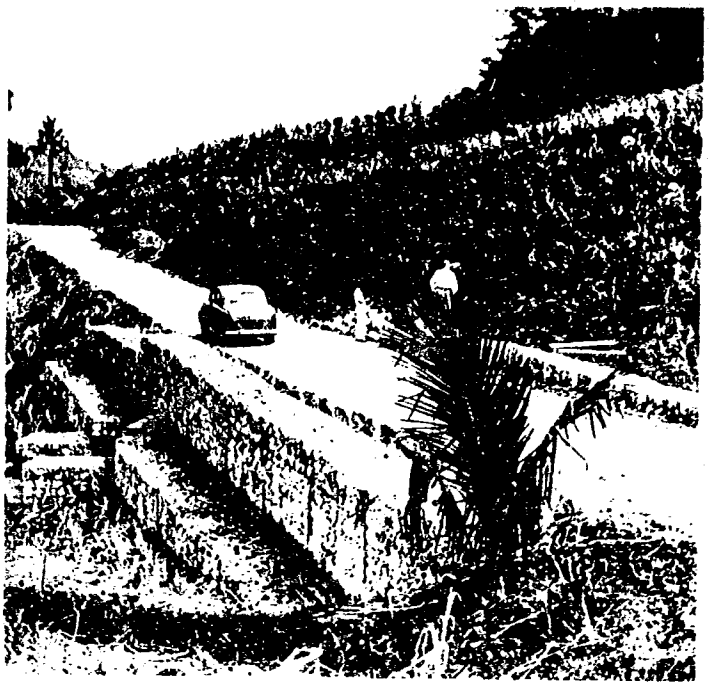


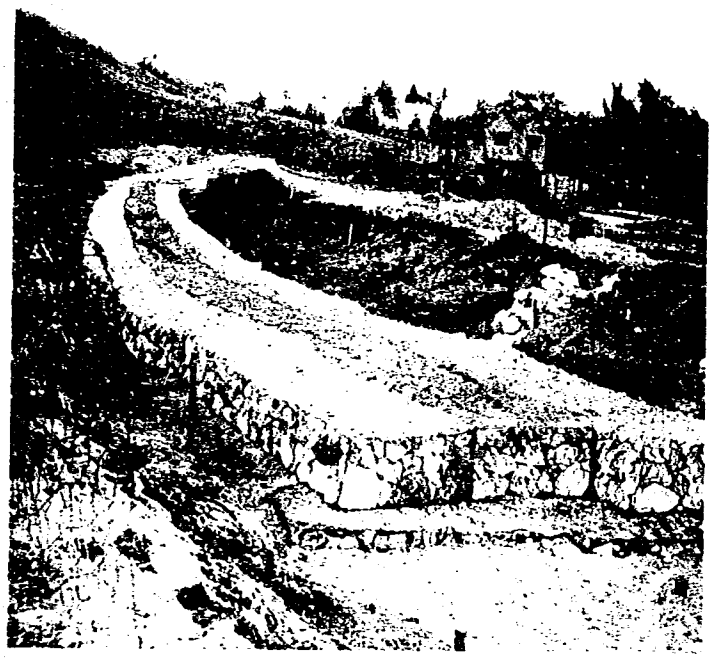
PAPUA - NEW GUINEA - 1984

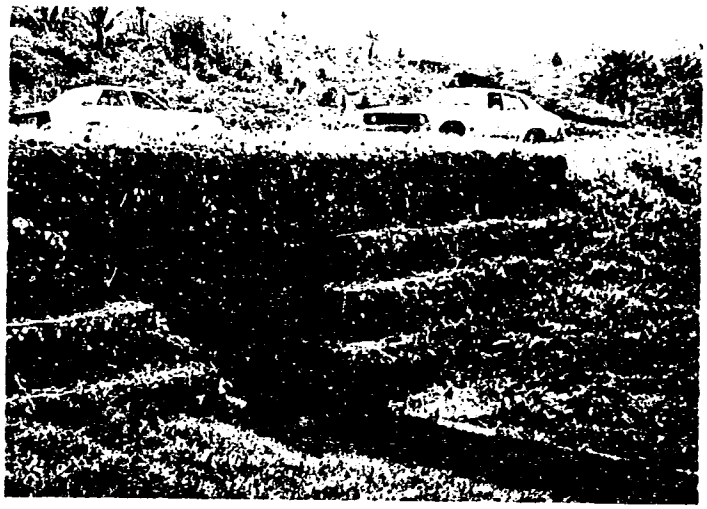
False abutments at a bridge on the Golgol River near Lae (Fig. 161)

6.2.3 Culvert protection









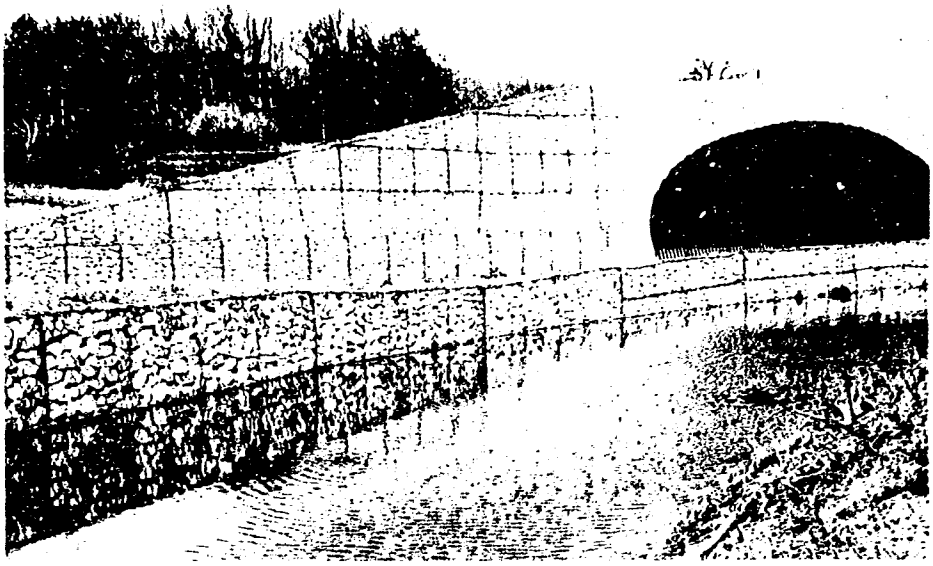
BARBADOS - 1966

Consolidation of a marshy area traversed by a road. Figures 162 and 163 show the construction of a culvert to drain away the water, and the completed project. Figure 164 illustrates the process of restoring the road sub-grade by using 0.50 m thick gabions.
Figure 165 shows the project about twenty years after it was built.

UNITED KINGDOM - England - 1975

Wall to contain the sides of an embankment over a culvert on the A 303 road at Yarty on the border between Devon and Somerset (Figs. 166, 167).

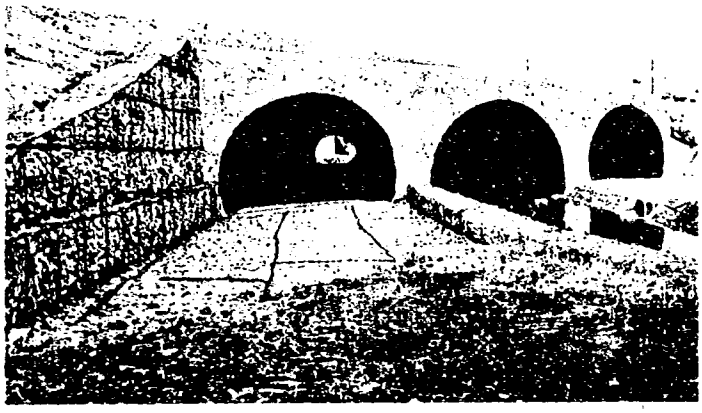
166



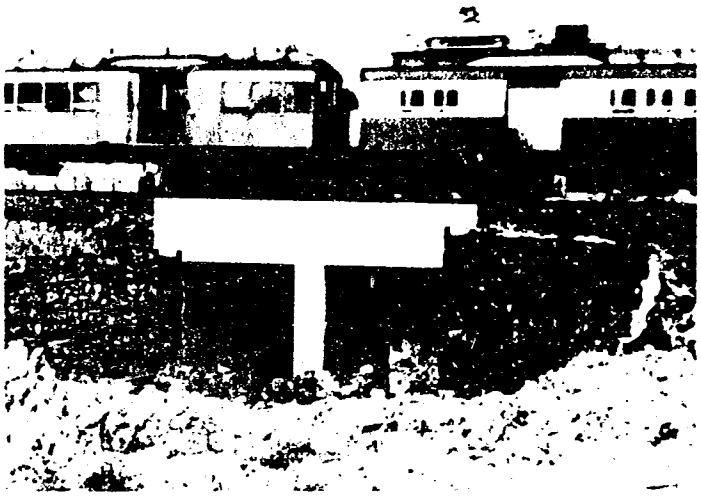
AUSTRALIA - 1976

Gabion wing walls to a box culvert under a railway at Marayong Victoria (Fig. 168).

167



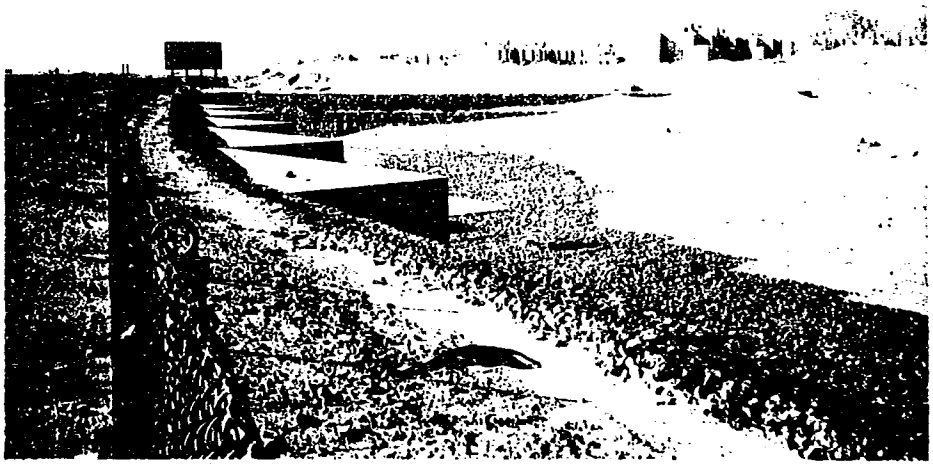
168



169

SAUDI ARABIA - 1979

Retaining wall in PVC-coated gabions on the Jeddah-Meccah Expressway. The wall is continued to protect a series of large box culverts. Also note the extensive PVC-coated Reno mattress apron built downstream of the culverts to control erosion (Fig. 169).

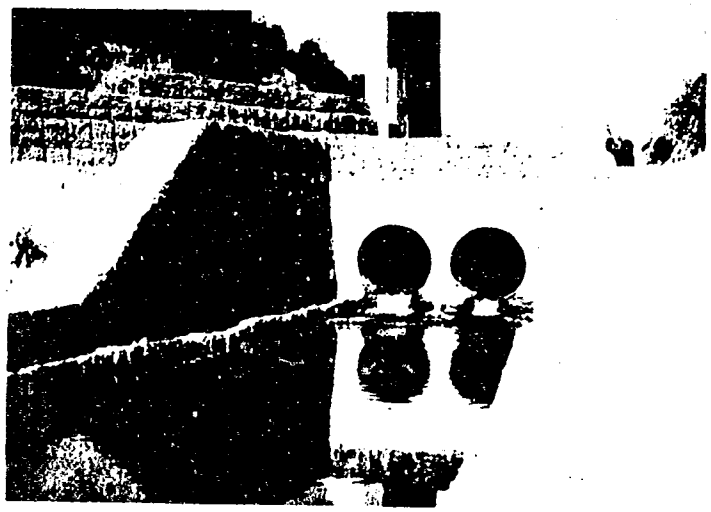


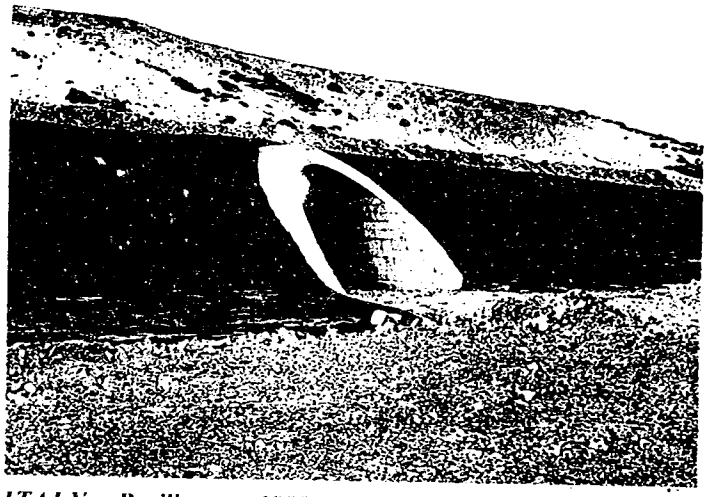
BARBADOS - 1979

High gabion wall under the outfall of a culvert (Fig. 170).

BRAZIL - Sao Paolo - 1979

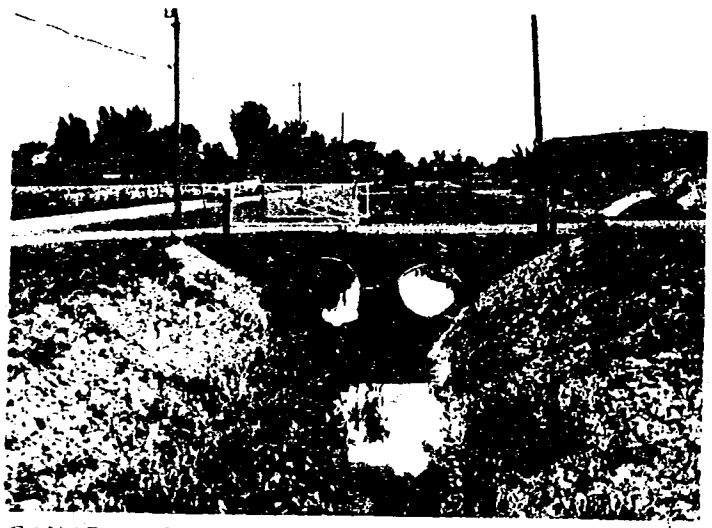
Composite structure in San José Dos Campos consisting, in the background, of a bridge abutment and, in the foreground, a culvert protection (Fig. 171).





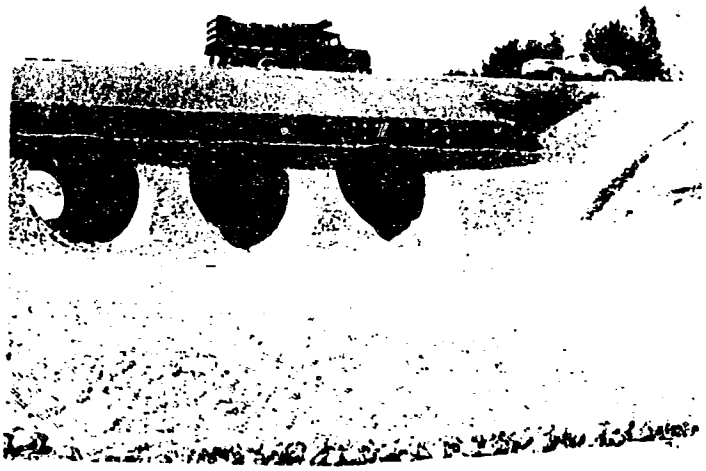
ITALY - Basilicata - 1980

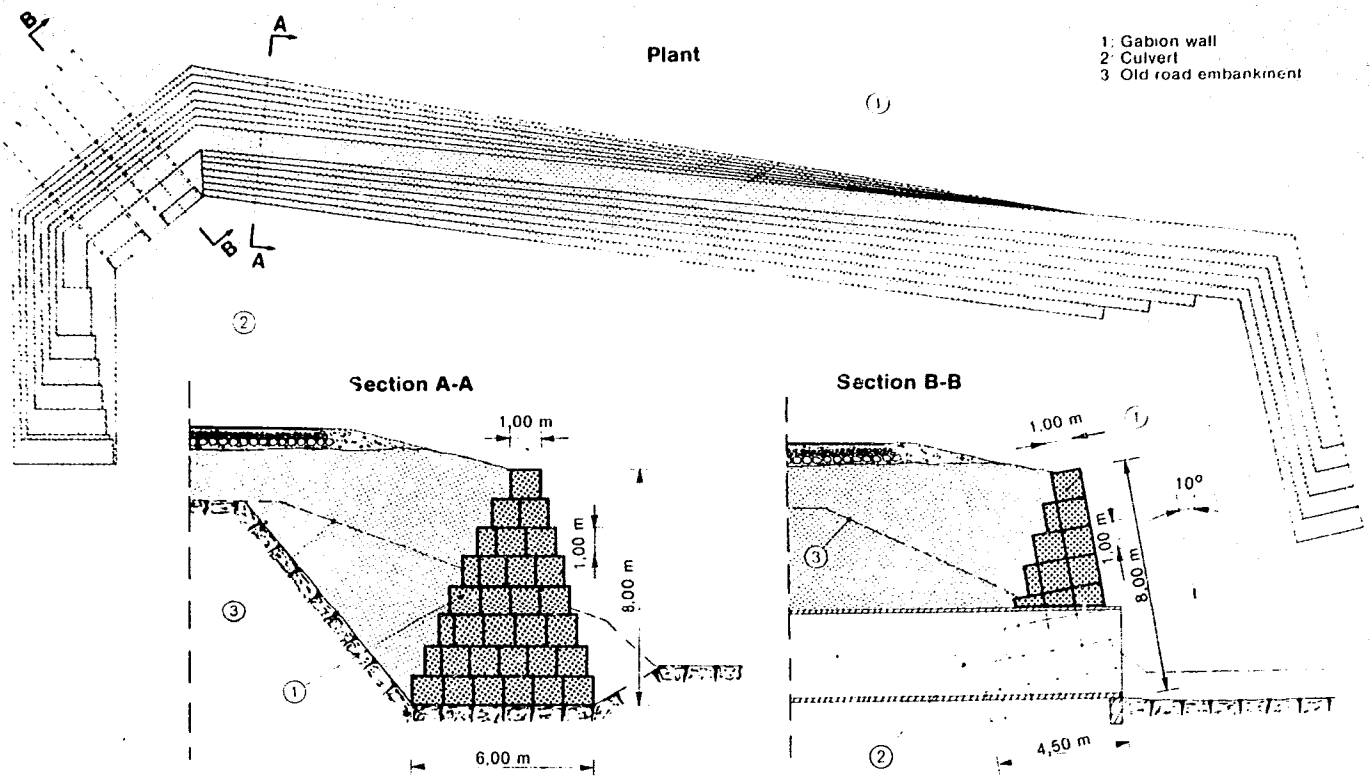
Protection of a road culvert using Reno mattresses consolidated with hydraulic sand asphalt mastic in the Sinni River Valley in Metaponto, Province of Matera (Fig. 172).



CANADA - Ontario - 1981

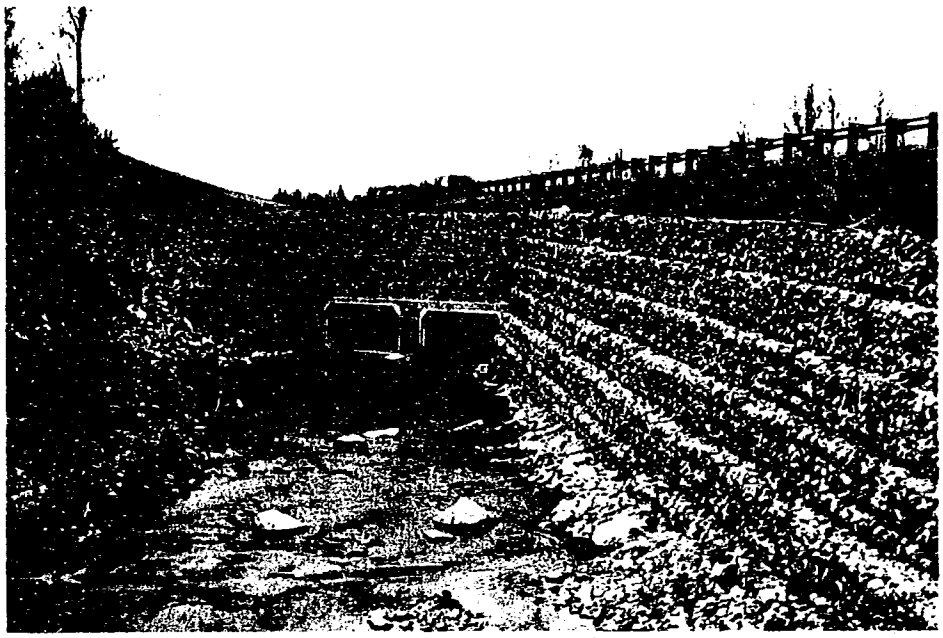
Series of culvert protection walls in Cornwall (Fig. 173). Figure 174 illustrates a project of larger dimensions consisting of a retaining wall to protect a culvert, and of a gabion apron that extends several metres downstream to give greater protection against soil erosion. Two lateral channels lined with gabions carry the water collected from the sides of the road to the river.

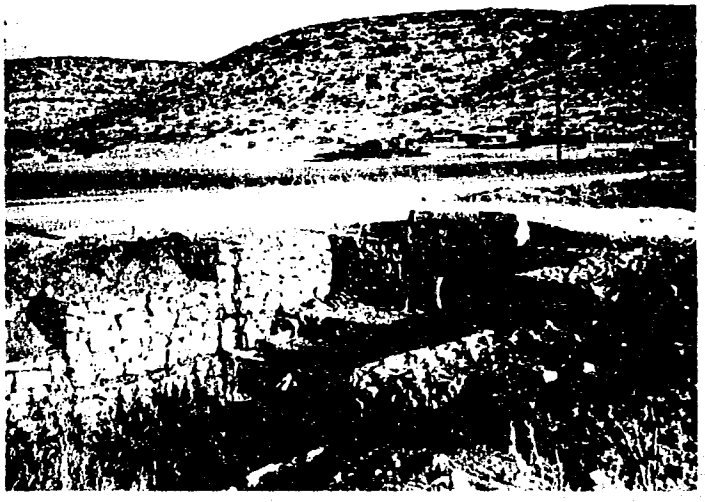




U.S.A. - Minnesota - 1983

Structure supporting the embankment of Highway 53 and the culvert that runs through it, allowing the Ash River to flow into the lake of the same name near Virginia, Mn. Figure 175 shows the plan and some cross sections of the project which was carried out by the Department of Transportation.
Figure 176 illustrates the completed project.





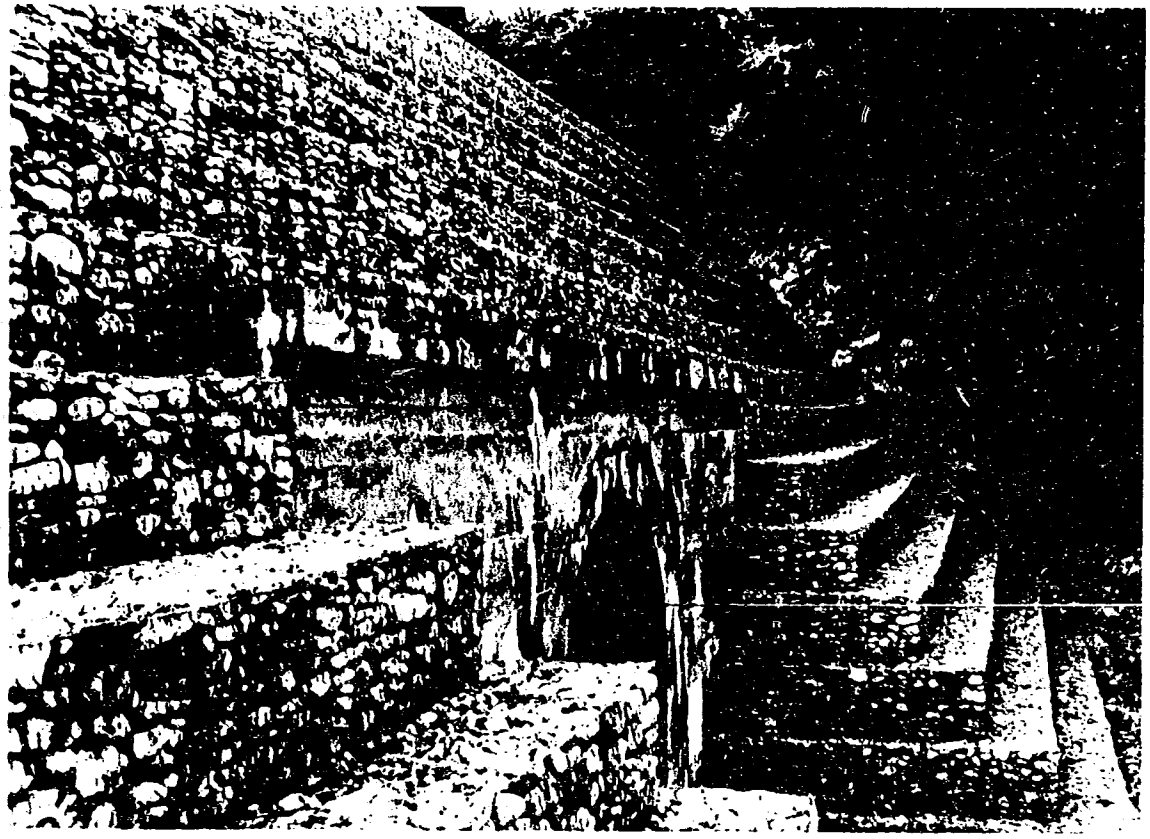
LIBYA - 1983

Protection of a culvert near the village of El Mebni at the foot of the Jebel Akhdar Mountains. The gabion structure continues to form a long canal that carries the water to the sea (Fig. 177)

6.3 Slope and cliff face stabilization

6.3.1 Works in urban areas

8



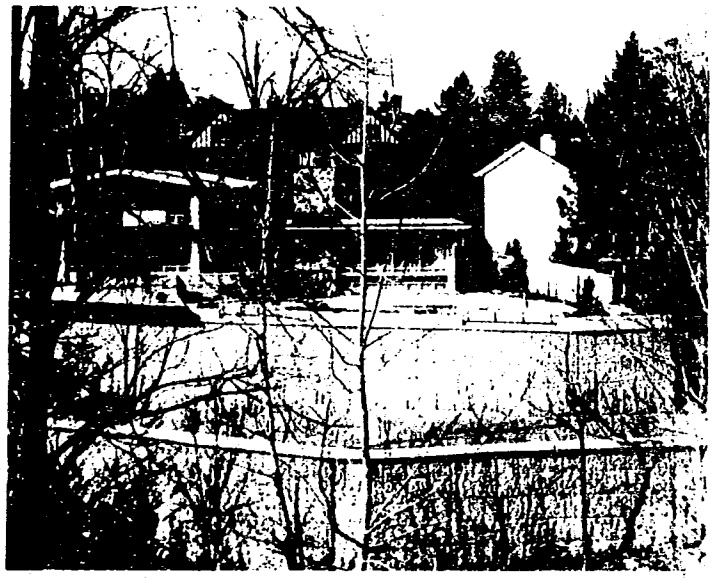
ITALY - Friuli Venezia Giulia - 1964

Check dam across the valley near St. Osvald pass. Its purpose is to protect the town of Cimolais from being inundated should landslides cause Lake Vajont to overflow (Fig. 178).

CANADA - Ontario - 1969

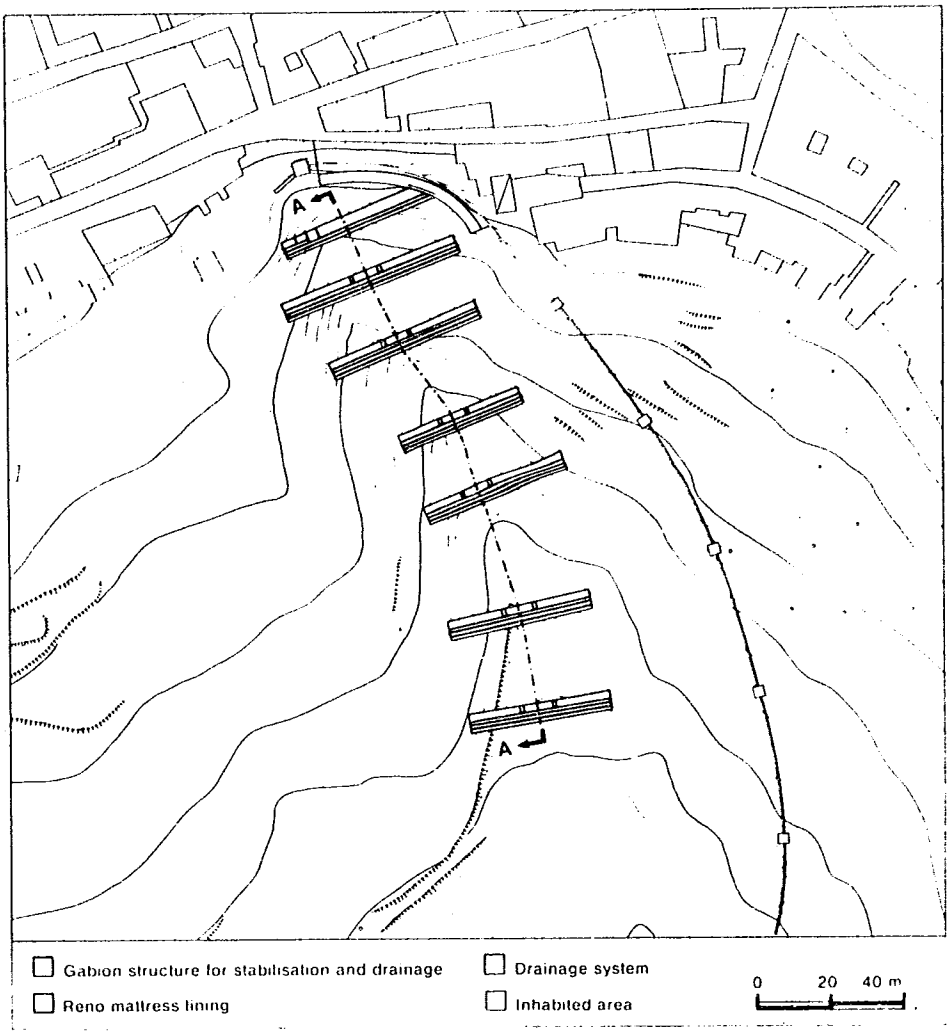
Series of retaining walls to protect dwellings in the Bayview area (Toronto).
Figure 179 shows a view of the project just after its construction, while figure 180 shows the uppermost wall fifteen years later.

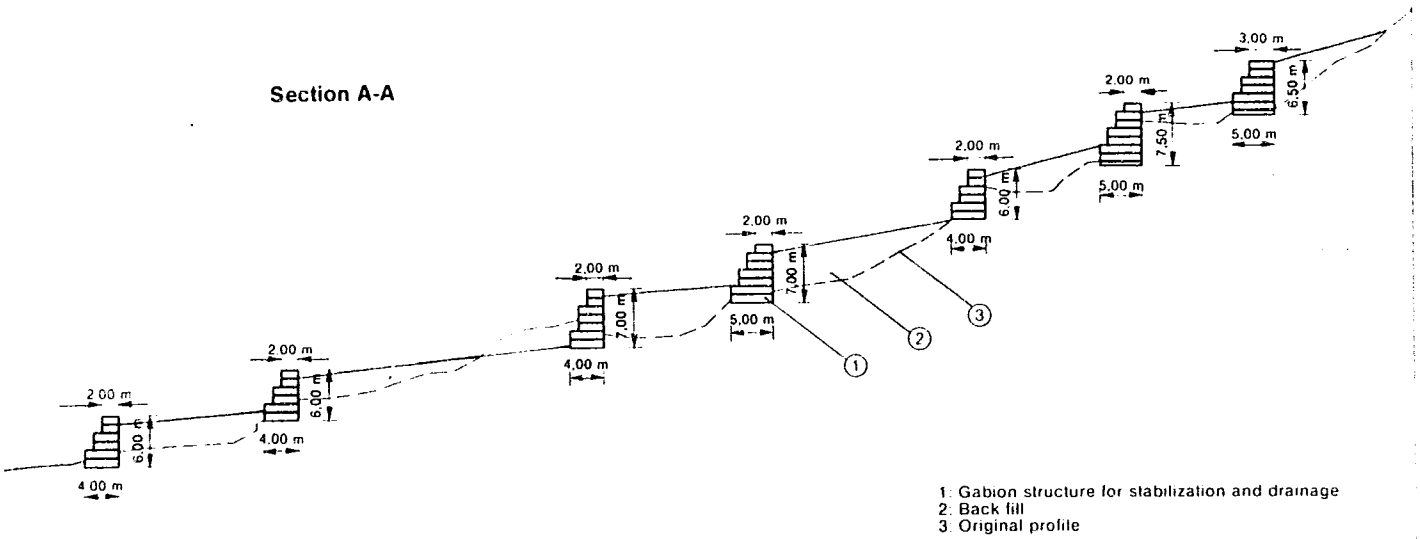
9

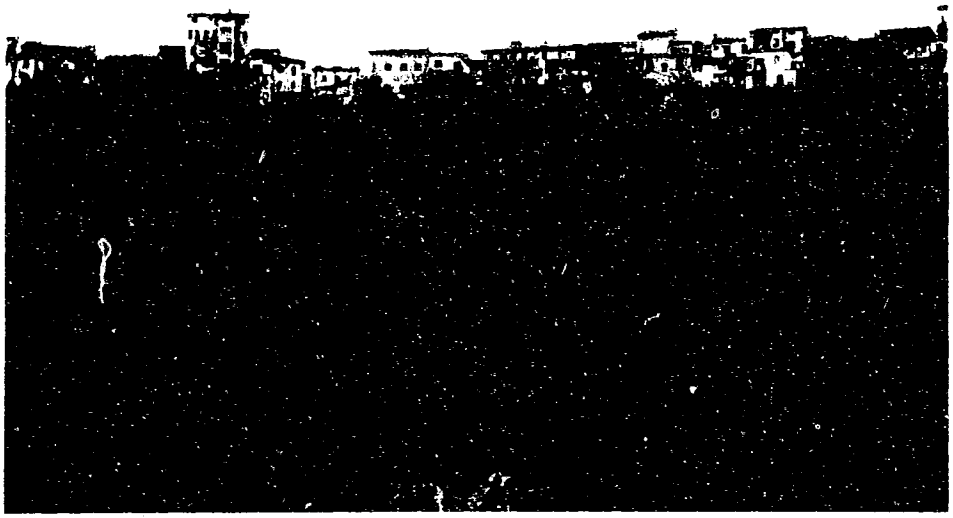


ITALY - Tuscany - 1969

Stabilization of an unstable slope in San Miniato, Province of Pisa. The slide movement was of major dimensions and extended to the inhabited area, as can be seen from the site plan (Fig. 181).
The project involved the construction of a series of combined walls and weirs in gabions for stabilisation and drainage, as shown in the cross section in Fig. 182. The upper part of the slope was lined with Reno Mattress.
Figure 183 shows the project just after completion.
Fifteen years after their construction, the structures are still in excellent condition and perfectly integrated into the natural environment (Fig. 184).



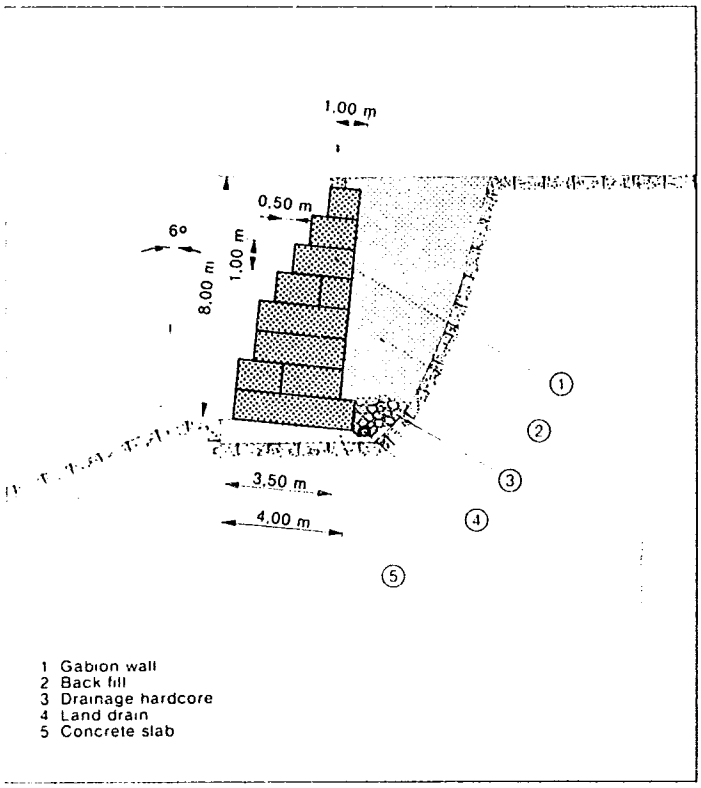




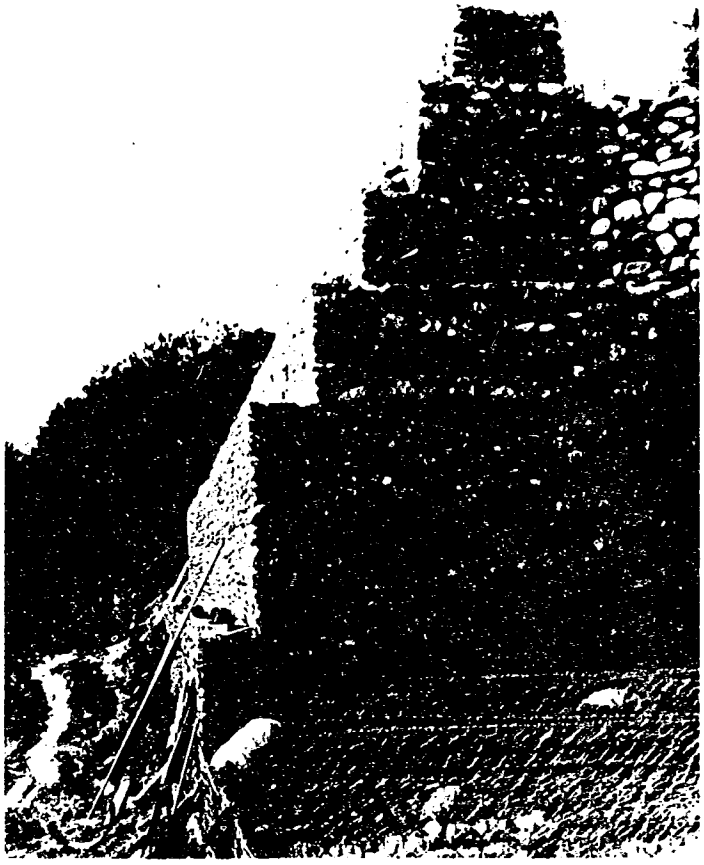
185

186

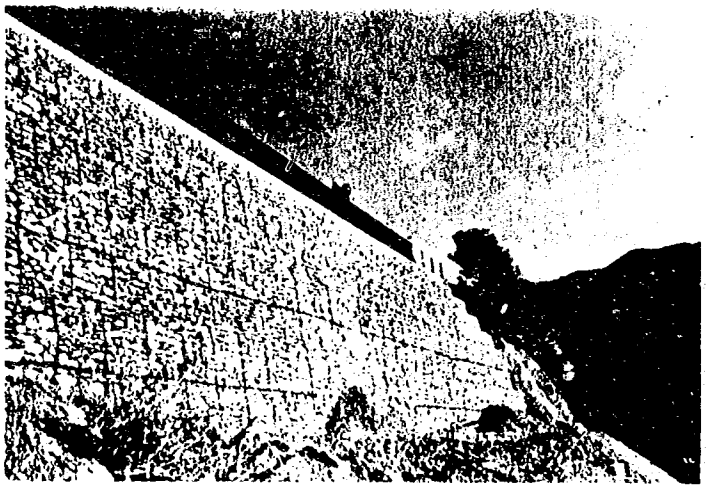
187



188



189

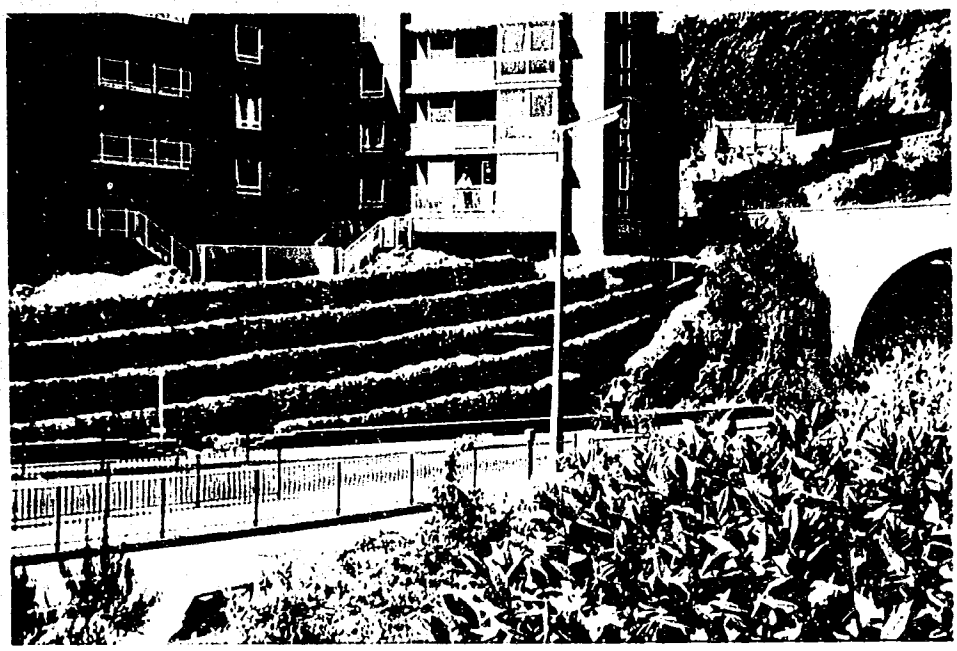


ITALY - Emilia-Romagna - 1971

The archeological remains of the Etruscan "City of Misa" are located on the embankment of the Reno River in Marzabotto, Province of Bologna. In 1935, to protect them from the erosive action of the river, a series of groynes and a gabion wall (Fig. 185) were constructed. Figure 186 shows these structures after more than forty years. On the left note the fence that surrounds the archeological zone. Due to considerable water seepage and surface erosion, the embankment became unstable and, at the beginning of 1970, the situation became critical.

During the rainy seasons the water washed away the surface soil, wetted the slip plane surface and caused landslides. Action was taken in 1971, in the form of a new wall 8 m high, shown in Figures 187, 188, and 189. After fourteen years, the present condition of the wall confirms that the structure has achieved a stable natural equilibrium.

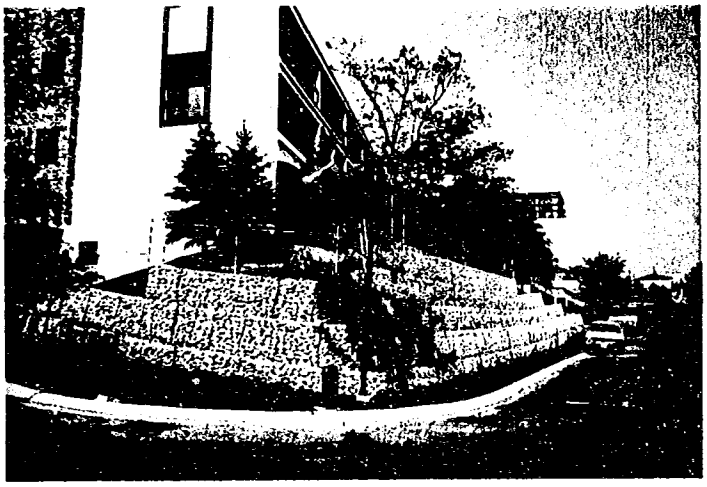
190



UNITED KINGDOM - Channel Islands - 1973

Retaining wall supporting the ground in front of a group of houses at Gothic Cedars, Jersey, shown ten years after its construction (Fig. 190).

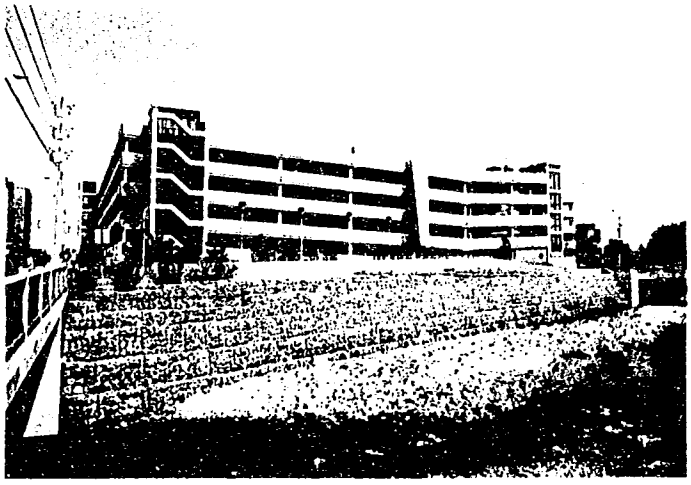
191



CANADA - Ontario - 1974

Retaining wall for an embankment in a residential part of Toronto (Fig. 191).

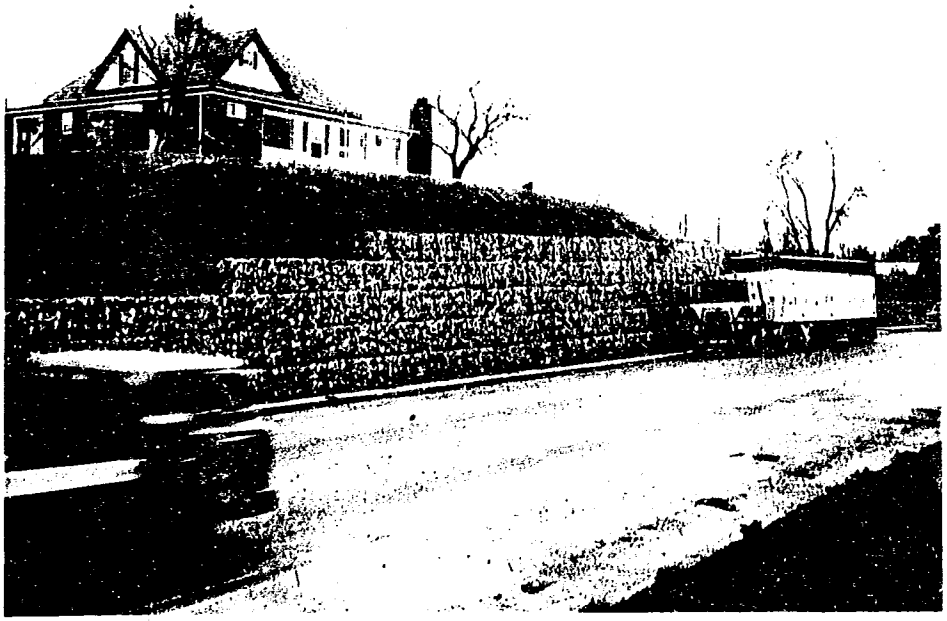
192



CANADA - Ontario - 1974

Interesting longitudinal structure protecting the abutments of two bridges and the foundation of an imposing building. Figure 192 shows the wall ten years after its construction.

193



U.S.A. - Oklahoma - 1977

Wall to protect a house in Binger (Fig. 193).

PORTUGAL - 1980

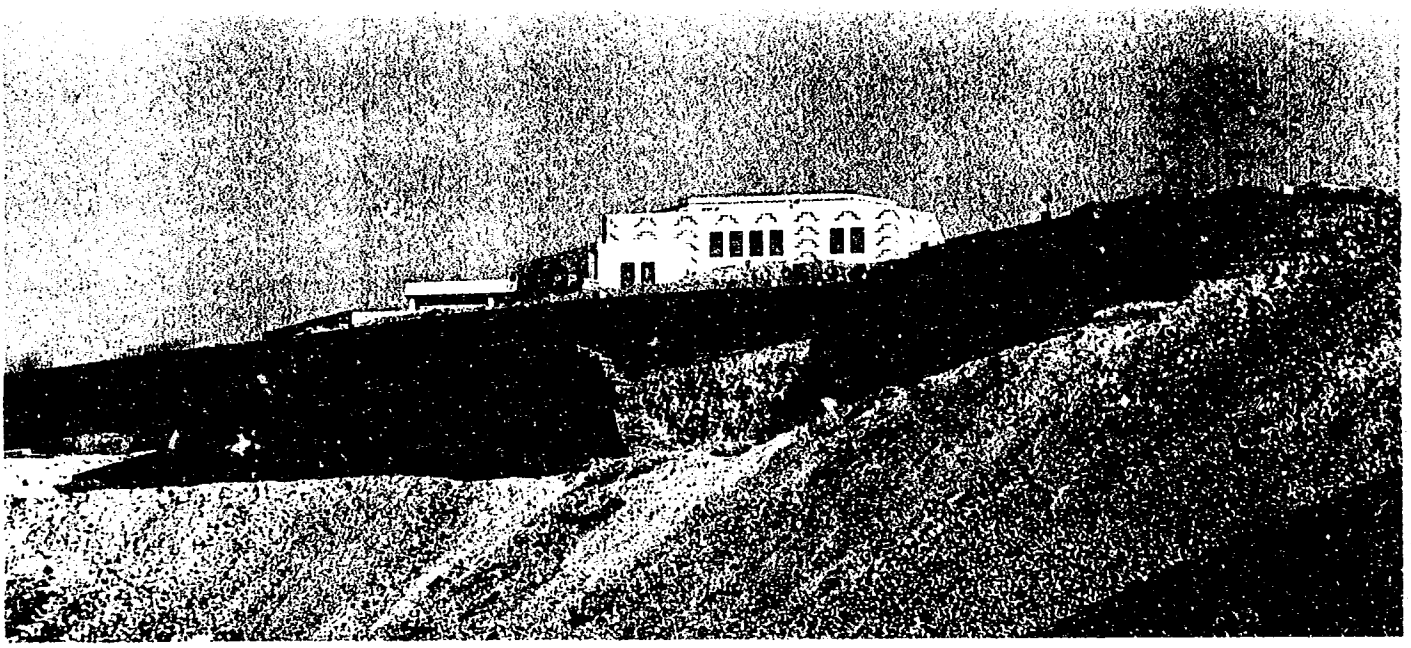
Gabion wall protecting some buildings in the residential complex of Queluz (Fig. 194).

194

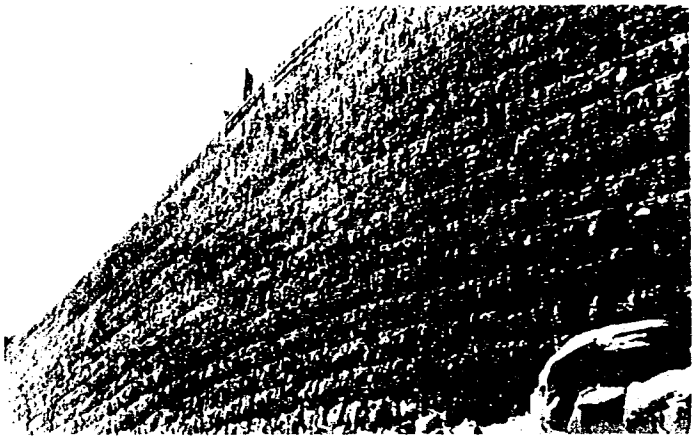
PHILIPPINES - Mindanao - 1978

Structure to protect an inhabited area in Salay on the Cagayan de Oro-Butuan City Road (Fig. 195).

195



196

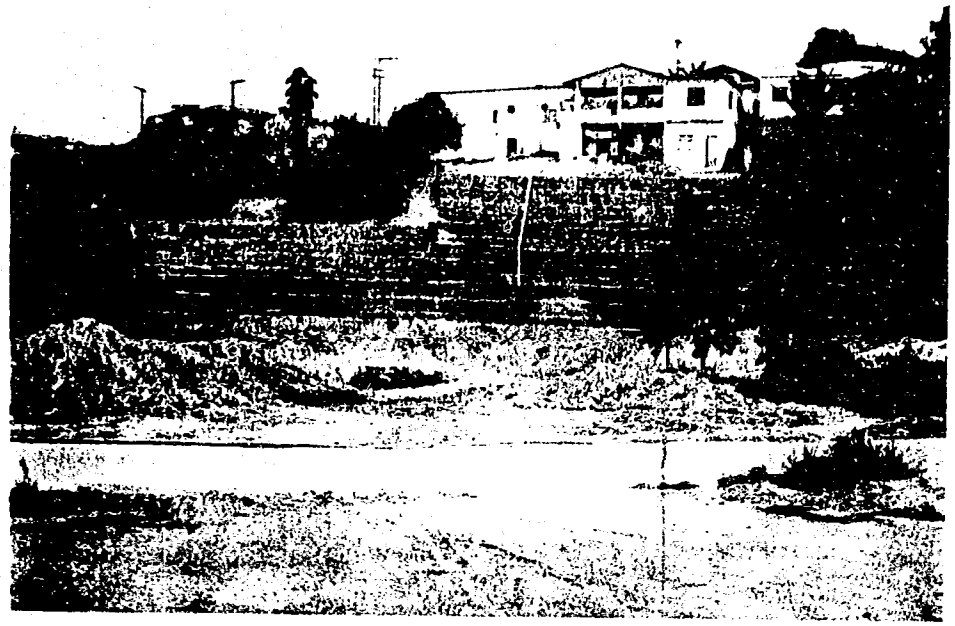


197

SAUDI ARABIA - 1981

Wall supporting the embankment on which the royal residence in Medina stands (Fig. 196). This photograph is from the Saudi-Oger 1982 brochure, published by Saudi-Oger Ltd., Riyadh, Saudi Arabia. Figure 197 shows a detail of the structure.

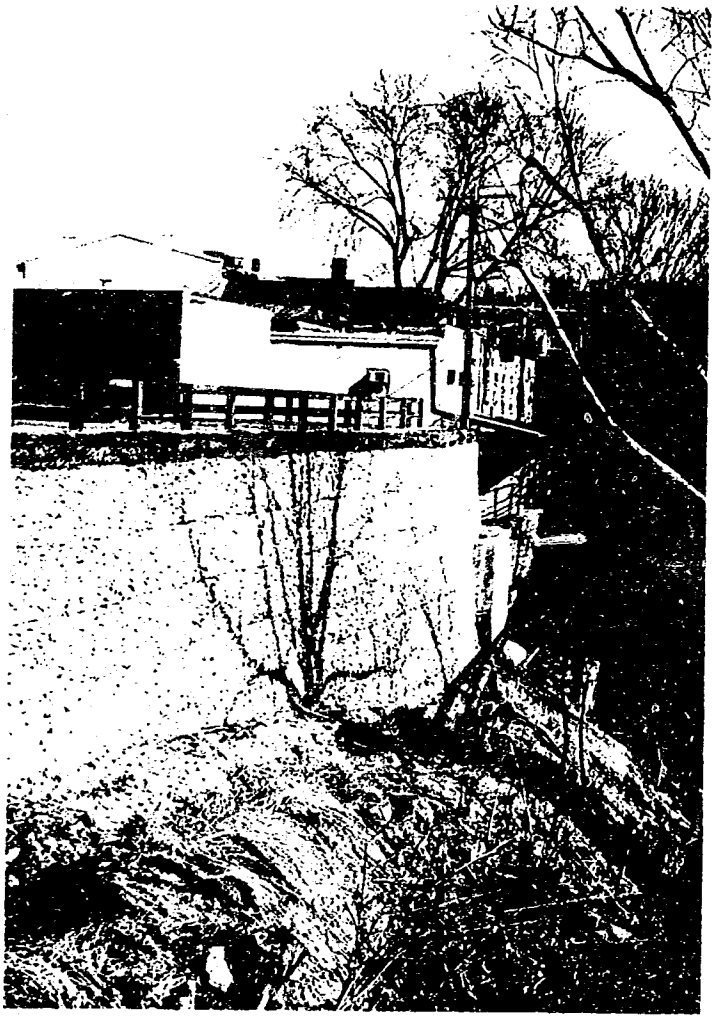
198

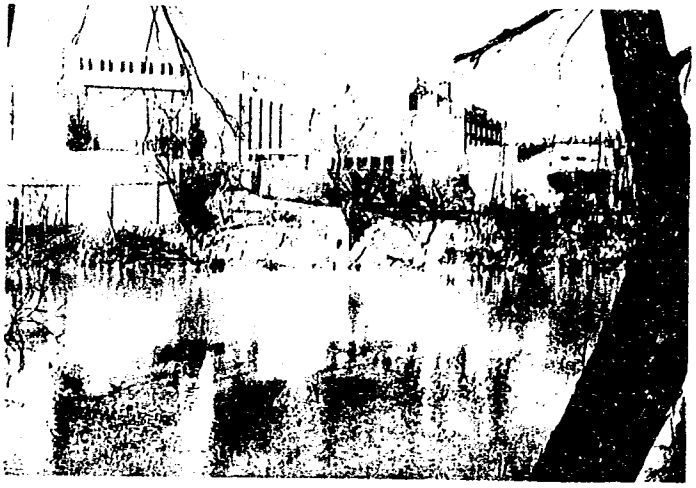


BRASIL - Minas Gerais - 1981

Retaining wall to protect a group of houses in Timoteo built on soil that is very easily eroded (Fig. 198).

199

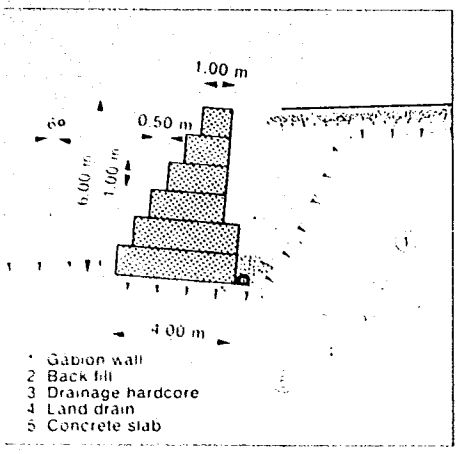




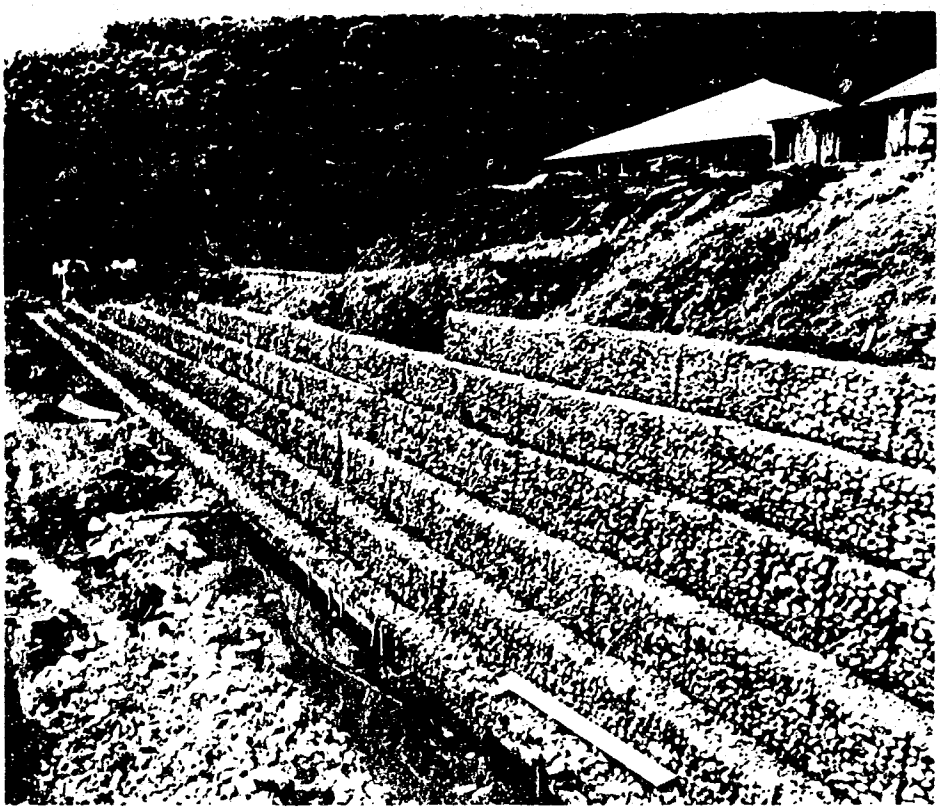
U.S.A. - Kentucky - 1982

Series of wall in the historic section of Frankfort to protect several houses that face the Kentucky River. Figure 199 shows a detail of the project. Figure 200, a general view.

201



202



BRAZIL - Santa Catarina - 1982

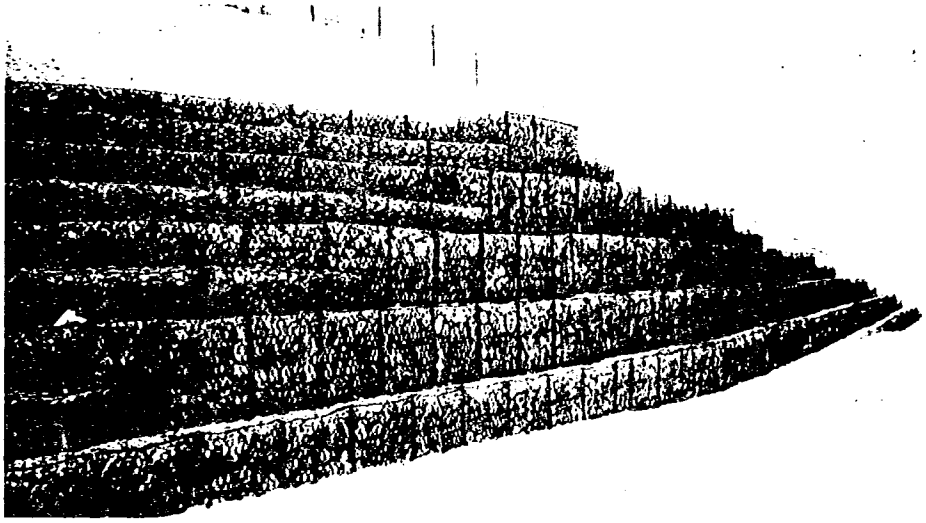
Wall adjoining the athletic grounds and facilities of the Athletic Association Club of the Banco do Brasil in Joinville (Fig. 202).

Figure 201 shows a cross section of the wall which features a concrete sub-foundation that collects water that seeps through from the highly permeable sandy-silt backfill.

SAUDI ARABIA - 1985

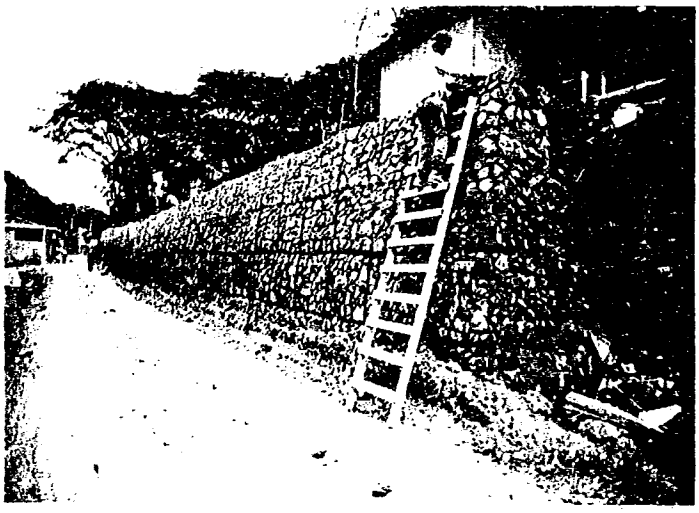
Wall in PVC-coated gabions at the toe of an embankment, constructed for an urban settlement on Ar Rabiah Hill in North Dharan (Fig. 203).

203



204

205



REPUBLIC OF SAN MARINO - 1983

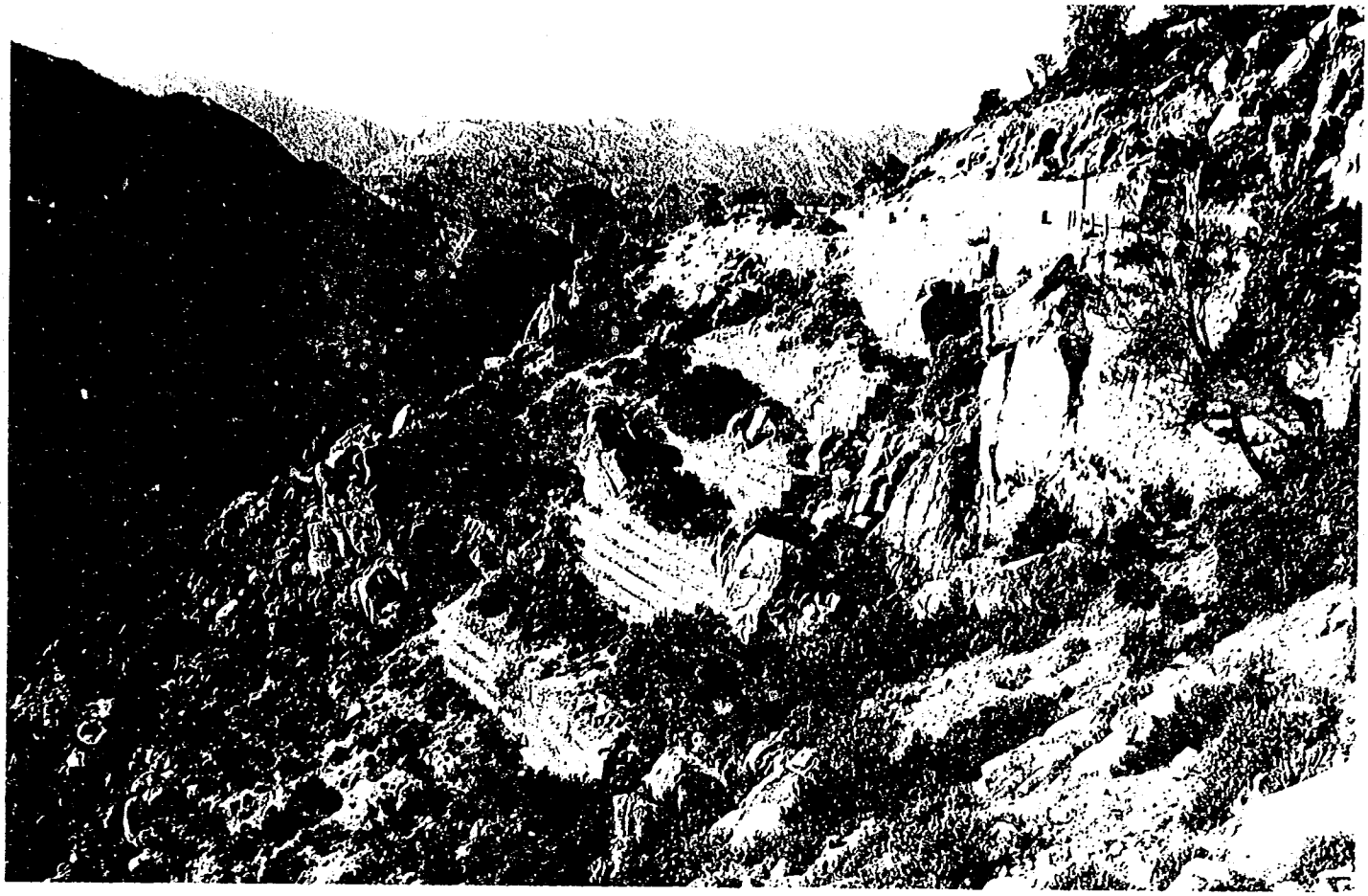
Retaining wall below industrial buildings in Domagnano (Fig. 204).

PUERTO RICO - 1984

Wall under construction to support an embankment near the new psychiatric hospital in Bayanon (Fig. 205).

6.3.2 Works on unstable slopes

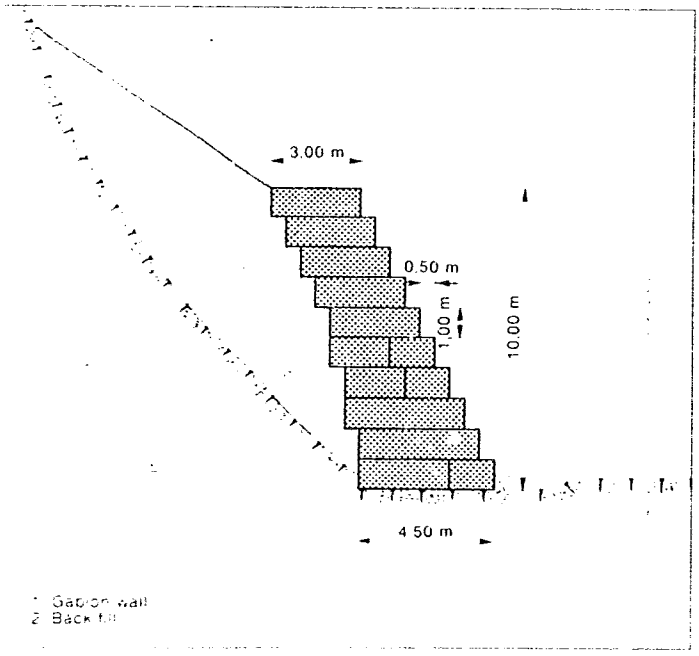
206



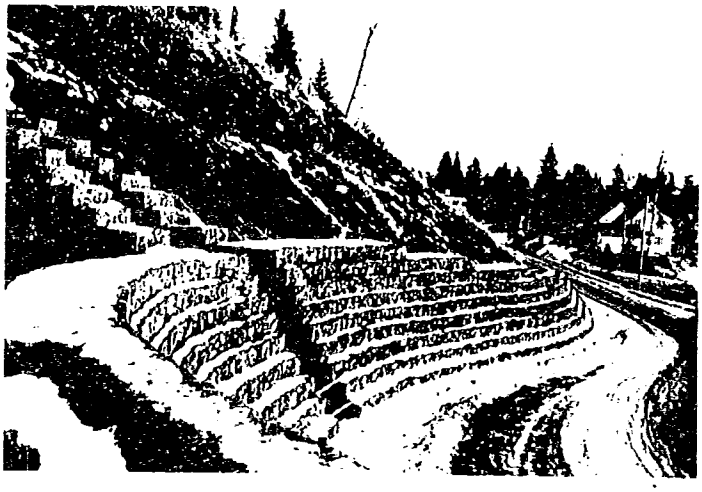
ITALY - Sardinia - 1963

Series of retaining walls to stabilize a treacherous slope in Dorgali, Province of Nuoro. A road has been built at the top of the slope (Fig. 206), and twenty years later the structures are still in perfect condition.

207



208

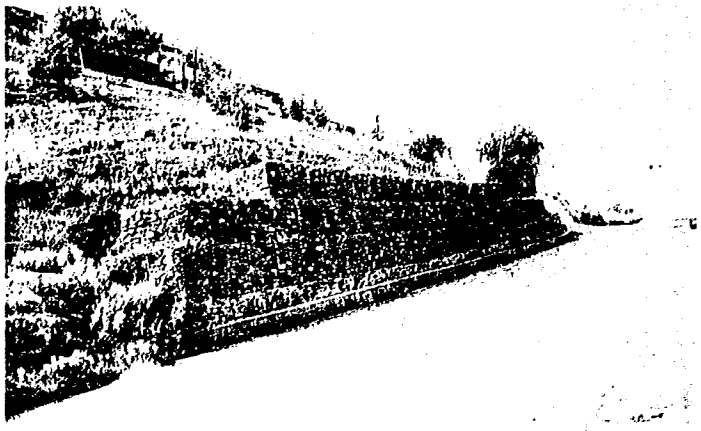


NORWAY - 1973

Wall to consolidate a slope in Tiovik. Figure 207 illustrates a cross section through the structure. As we can see in Figure 208, a gabion structure is as suited to arctic condition, as to the tropical countries featured in other illustrations.

ITALY - Emilia-Romagna - 1977

Wall at the foot of an unstable slope near the inhabited area of Porretta Terme, Province of Bologna (Fig. 209).

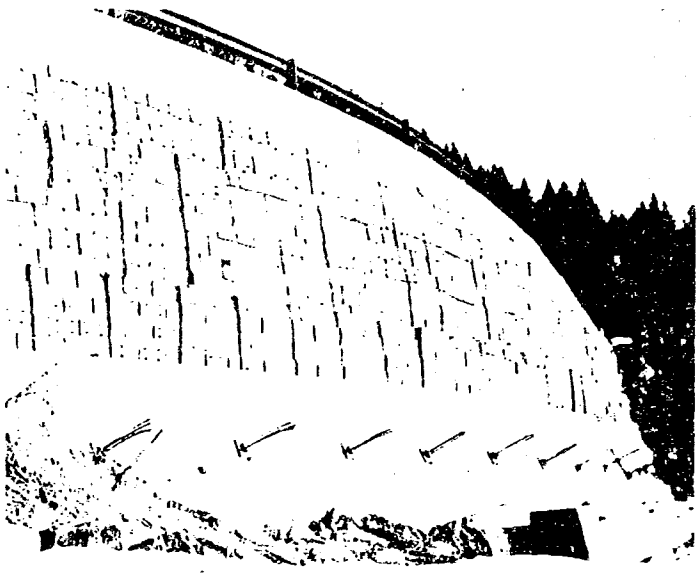


ITALY - Trentino Alto Adige - 1978

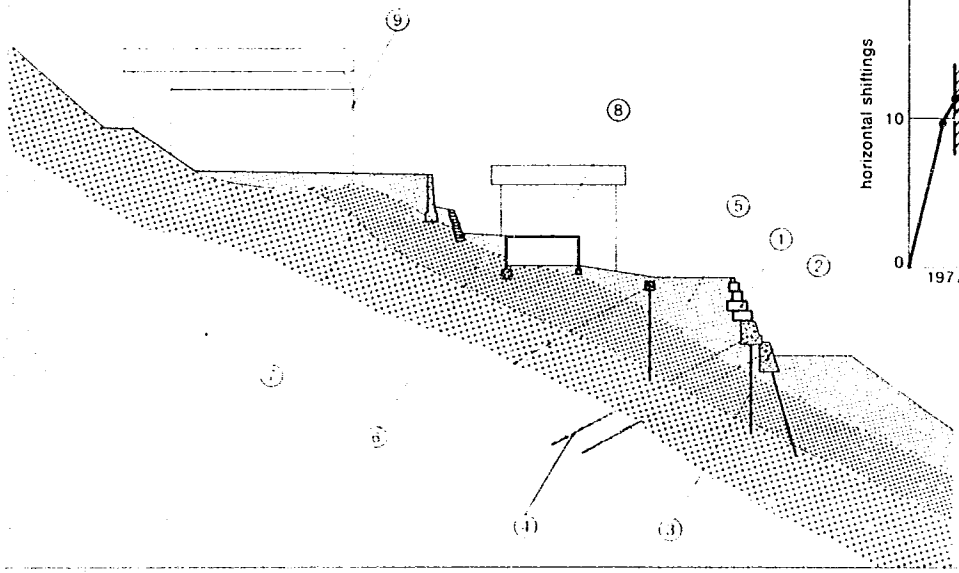
Gabion wall built on piles anchored by tension rods and interconnected with reinforced concrete beams, to consolidate an unstable slope in Vetriolo, Province of Trento.

The slip which was first observed in 1965, occurred in quartziferous filladis soil at a level near the surface, and involved a volume of about 55,000 m^3.

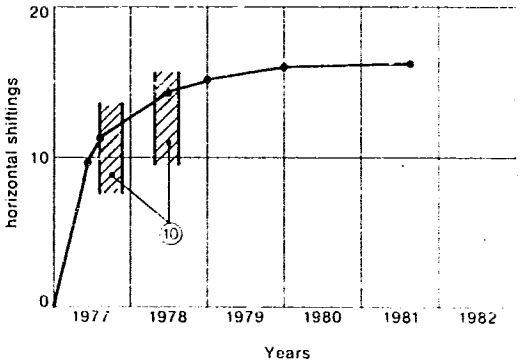
The movement caused serious damage to the local road and to a large number of dwellings. Boreholes were taken to find the depth of the slip surface (from 3 to 8 metres), and to obtain soil samples from which the shear parameters were determined. The low shear strength of the soil and the necessity of preserving the existing drainage pattern demanded a structure of optimum flexibility. The design included a combination of tension rods and piles interconnected by reinforced concrete beams, to precompress the soil. Gabions were chosen for the vertical structure for their flexibility and drainage capability (Fig. 210). The general layout of the scheme is shown in Figure 211, and the graph in the insert illustrates how the slip has been almost completely halted by these remedial measures.



STABILIZATION WORKS SCHEME



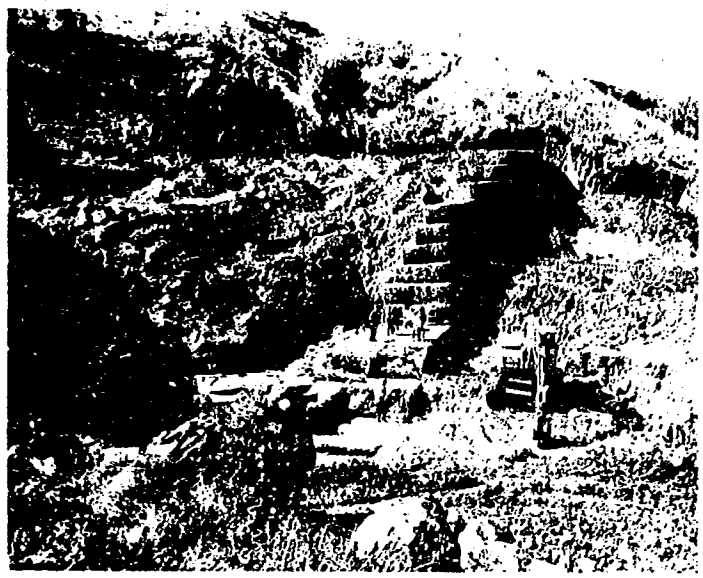
HORIZONTAL SHIFTINGS OBSERVED



1: Gabion wall
2: Reinforced concrete beam
3: Piles
4: Anchorage
5: Slide volume
6: Degraded filladis
7: Quartziferous filladis
8: Villa
9: Hotel
10: Stabilization work

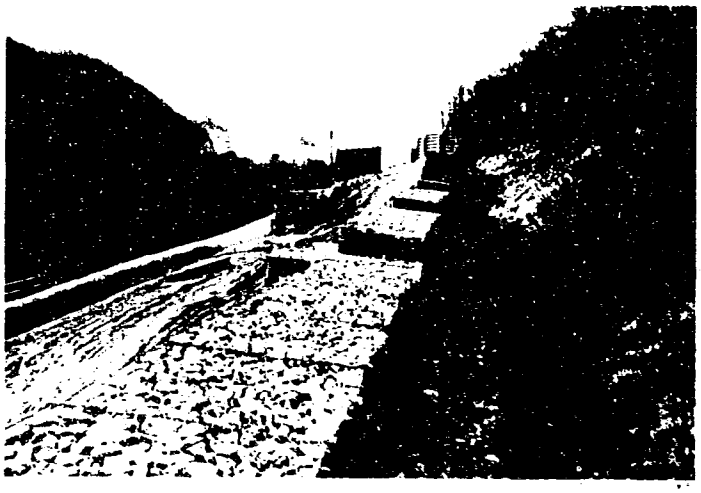
REPUBLIC OF SAN MARINO - 1978

Stabilisation works on a slope at Baldasserona. The wall is keyed deeply into the ground which consists primarily of weak clay (Fig. 212). The wall serves as a weir. Surface water resulting from rainfall on the catchment area above is channeled over the central crest (Fig. 213). Figure 214 shows the scheme six years after its completion.



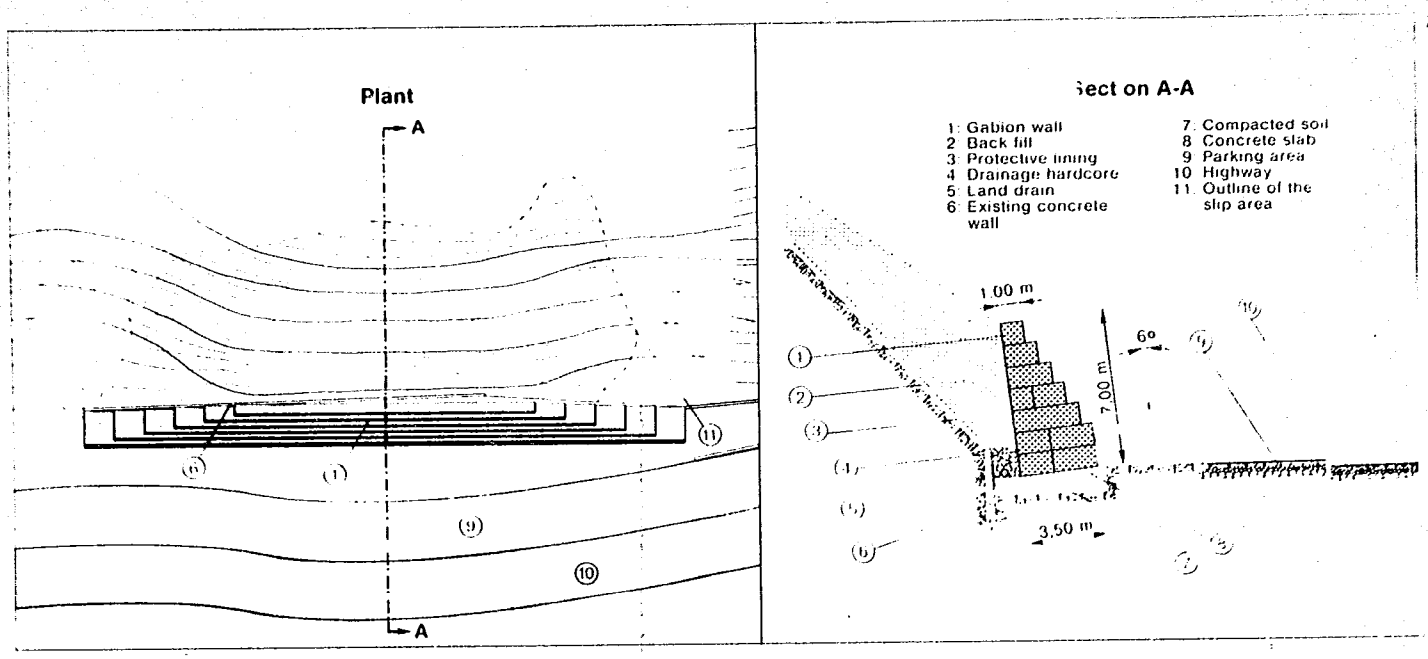
ITALY - Emilia-Romagna - 1979

Series of walls for earth retention and drainage on an unstable slope in Ca Domenicali, Province of Bologna (Fig. 215). The ground, composed of schistose clay, has a low bearing capacity for which a flexible gabion structure is the most suitable solution.



ITALY - Emilia-Romagna - 1983

Wall retaining an unstable hillside above the railway between Bologna and Florence near Grizzana, Province of Bologna (Fig. 216).



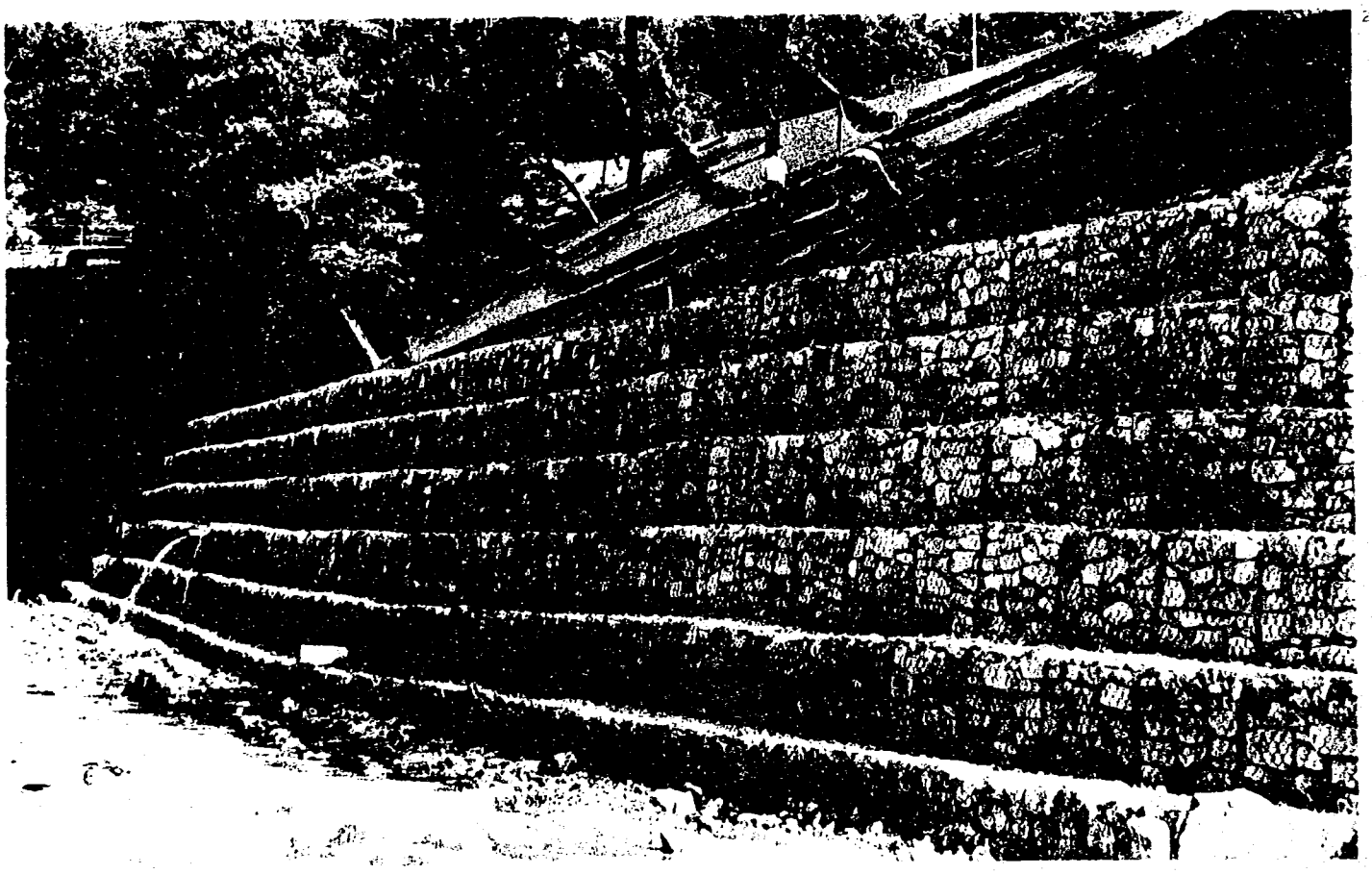
U.S.A. - California - 1983

An unstable slope on Yerba Buena Island, San Francisco is retained by a gabion wall chosen in preference to two other solutions, namely a crib wall and reinforced concrete. Fig. 217 shows the plan, the extent of the area of slip and a cross section. Figure 218 shows the completed structure.

ITALY - Campania - 1983

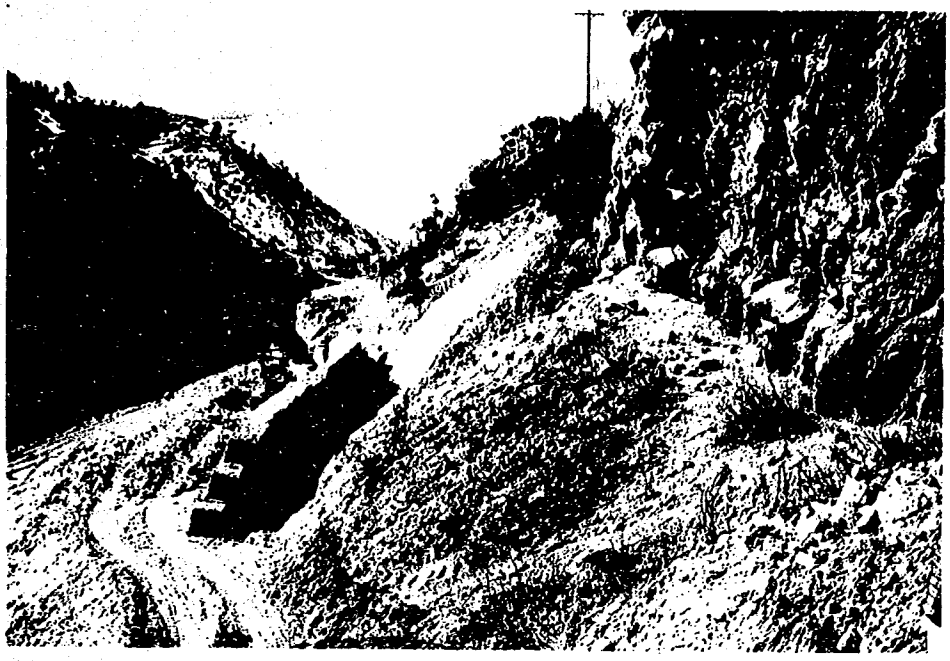
Imposing retaining wall supporting a slope subject to slips and protecting a SNAM station at Capo Sele, Province of Avellino (Fig. 219).

The photograph shows seeding and planting of shrubs in progress to consolidate the surface of the slope with vegetation.



6.3.3 Rockfall and avalanche protection

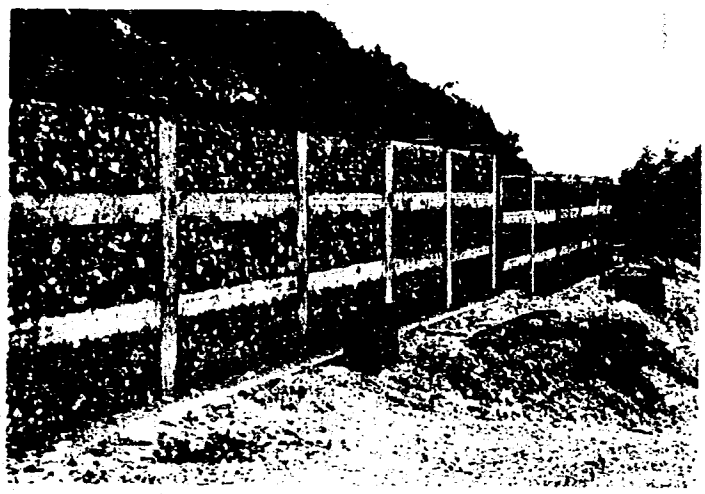
220



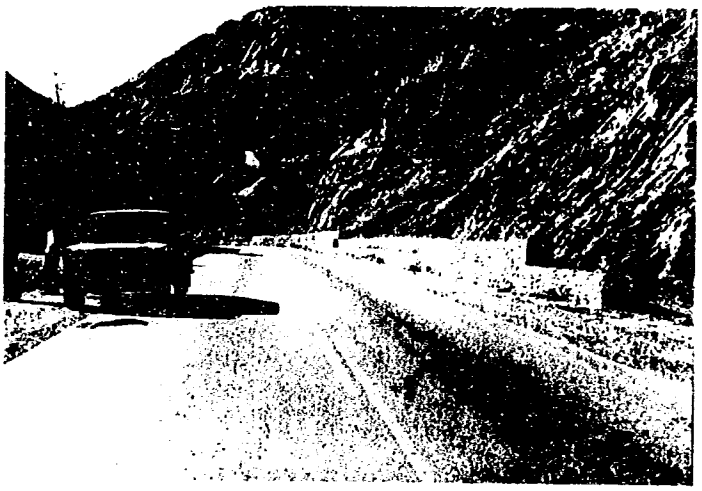
U.S.A. - Colorado - 1969

Rockfall barrier wall near Glenwood Canyon, Denver (Fig. 220).

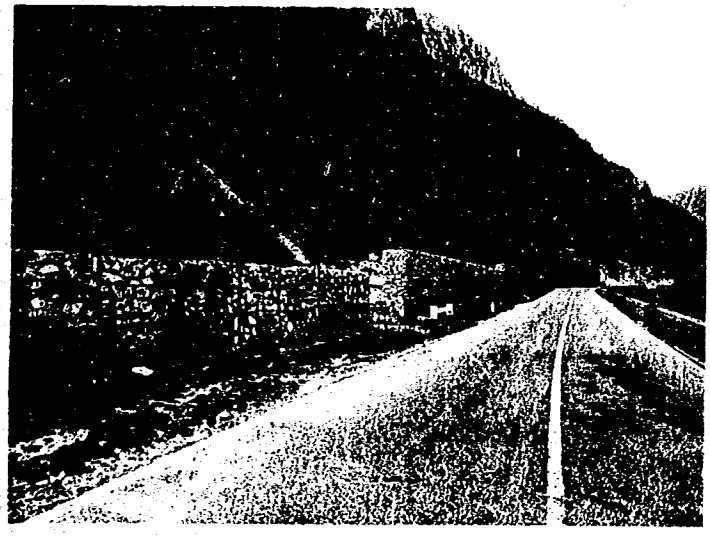
221



222



223



ITALY - Friuli Venezia Giulia - 1981

Rockfall and avalanche protection project near Udine. Gabion works are combined with a network of reinforced concrete beams (Fig. 221).

BOLIVIA - 1982

Rockfall barrier wall along the road that links Santa Cruz with Cochabamba (Fig. 222).

ITALY - Latium - 1982

Gabion wall to prevent boulders rolling on to the Salaria State Highway 4 in the vicinity of Posta, Province of Rieti (Fig. 223).
A smaller wall can be seen half way up the slope.

BRAZIL - Rio de Janeiro - 1983

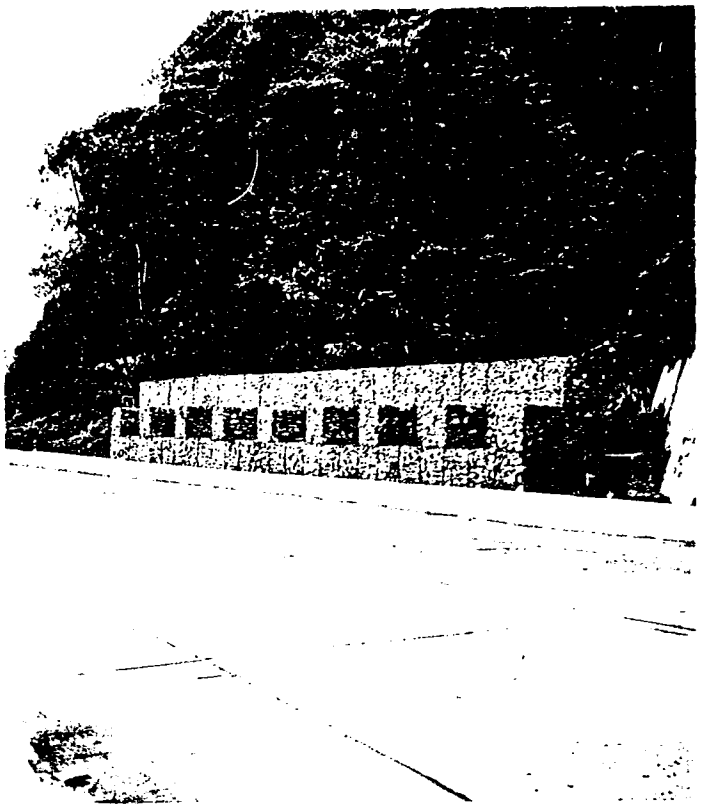
Steep slope lined with rockfall protection netting. At the foot of the slope a wall serves to contain any rocks falling down the slope (Fig. 224). The site is near Teresopolis.

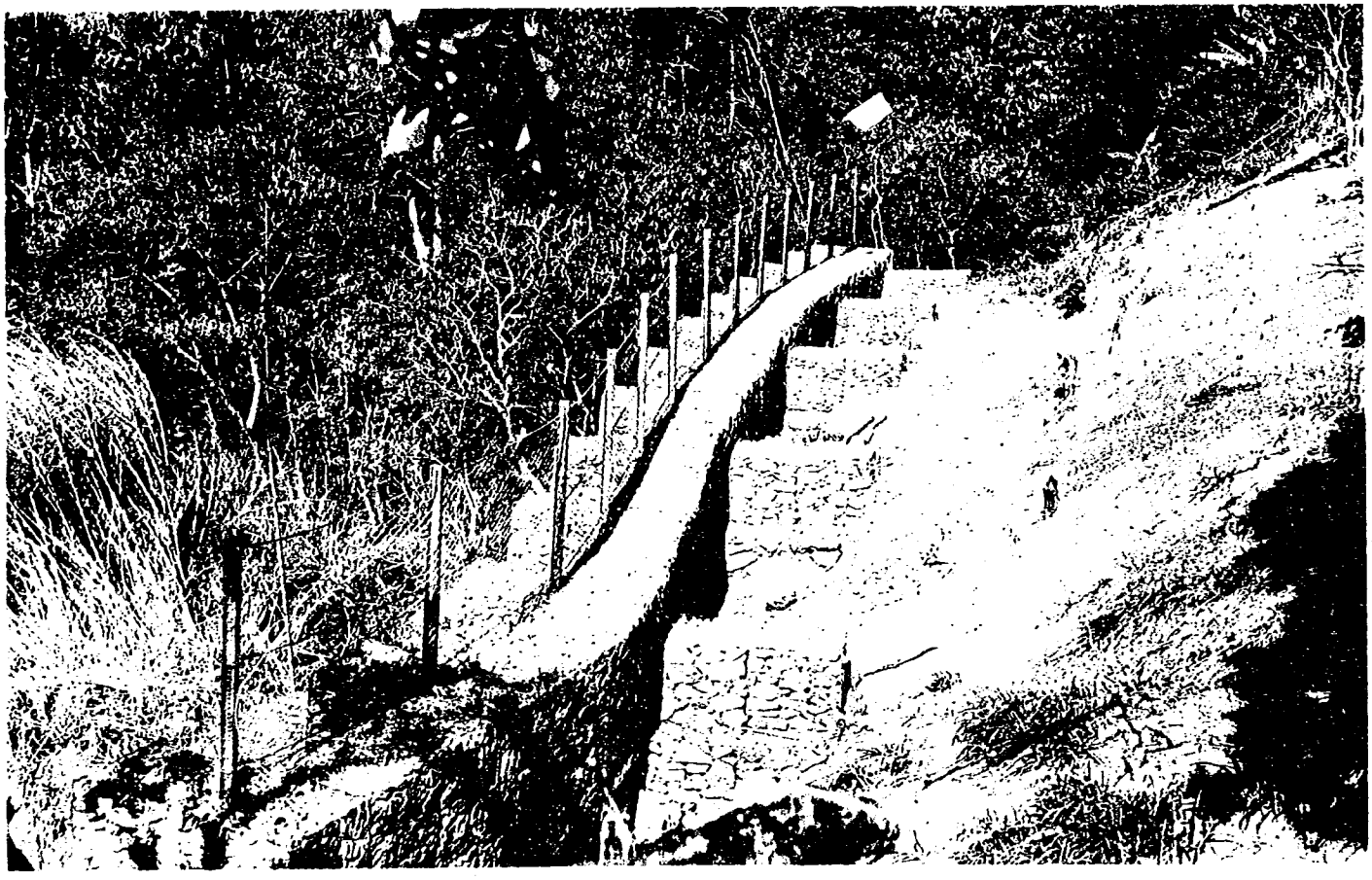
BRAZIL - Rio de Janeiro - 1984

Rockfall barrier wall with external counterforts on the Rio de Janeiro - Teresopolis road (Fig. 225).

BRAZIL - Rio de Janeiro - 1984

A villa in Petropolis is protected by a gabion barrier having internal drainage counterforts. Note the double fence of hexagonal woven mesh at the top of the wall (Fig. 226). As part of the same scheme, a second wall with identical features is now under construction.





6.3.4 Terracing

227

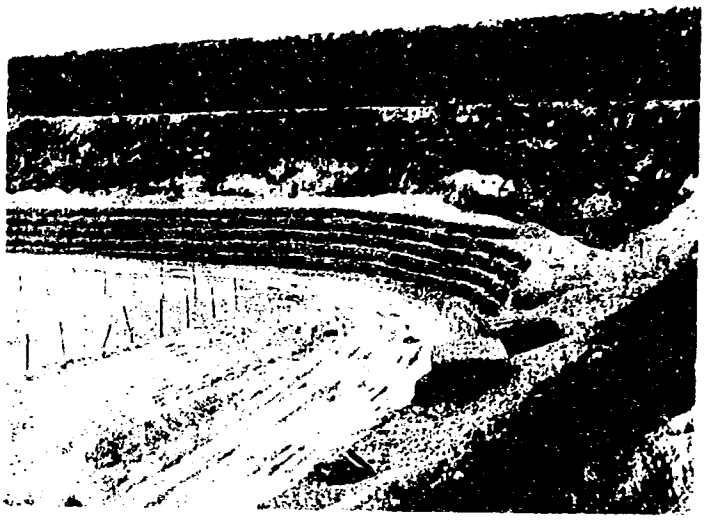
ITALY - Emilia-Romagna - 1959

Stabilization of a slope by terracing with gabion walls on the Bologna - Castiglione de' Pepoli Road, in the Lagaro area (Fig. 227).

FEDERAL REPUBLIC OF GERMANY - 1971

Terracing with gabion walls in Kaisersthul, Freiburg, for new vineyards in the Rhine Valley (Fig. 228).

228



229

ITALY - Campania - 1976

Gabion wall terracing in an olive grove in Vietri sul Mare, Province of Salerno.

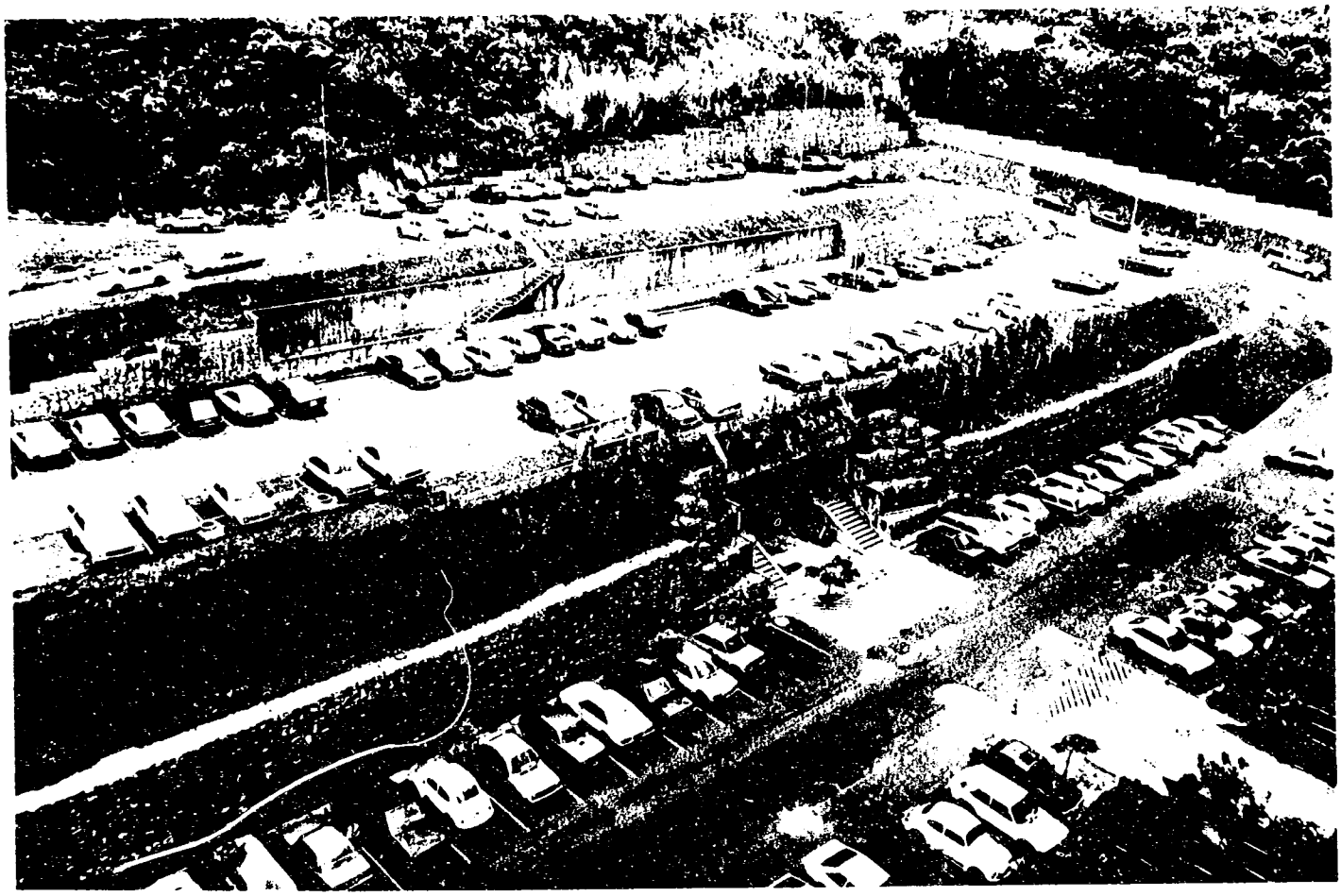
The gabions, ideally suited to the purpose, blend perfectly into the south Italian countryside (Fig. 229).

BRAZIL - Minas Gerais - 1979

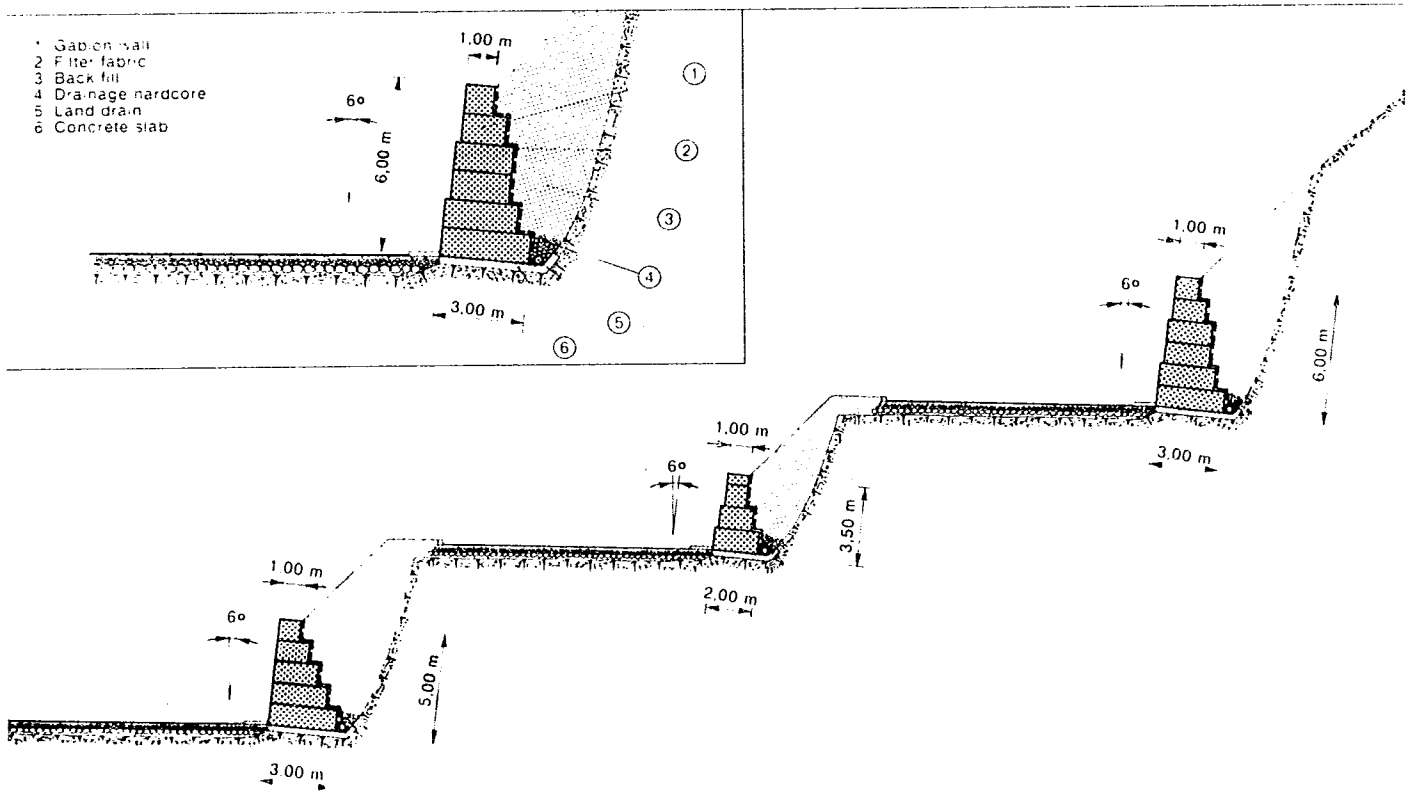
Interesting and imposing terracing to provide parking areas for the offices of Usiminas in Belo Horizonte (Fig. 230).

Figure 231 shows a cross section of the scheme.

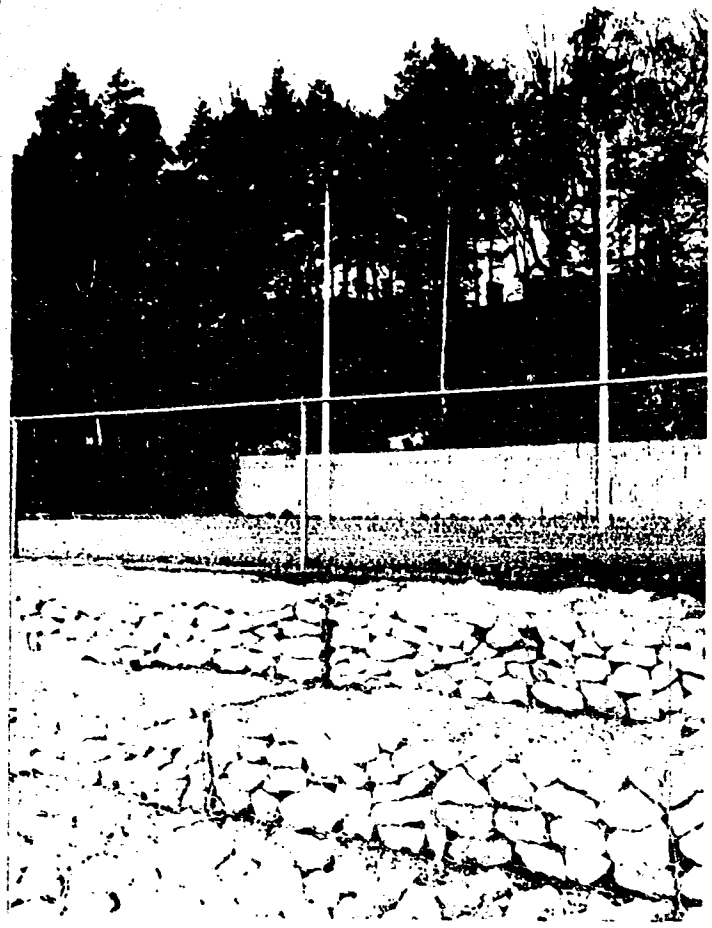
230



231



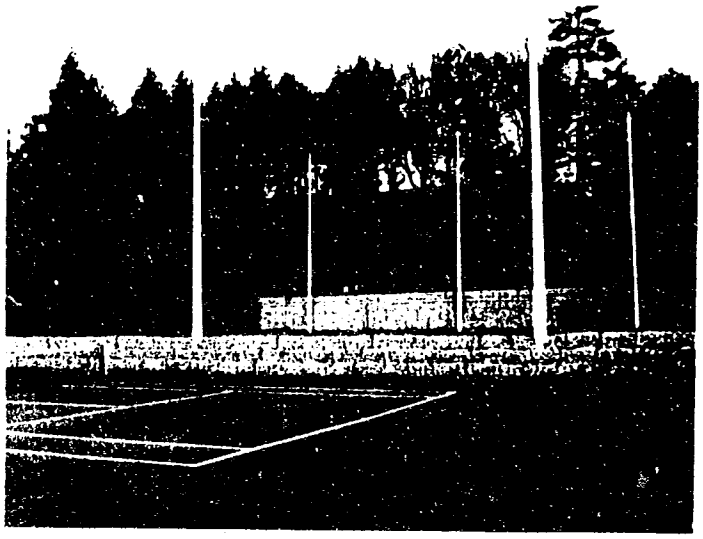
232



CANADA - Ontario - 1974

Gabion terracing for tennis courts in Toronto (Figs. 232, 233).

233



234

235

REPUBLIC OF CAPE VERDE - 1981

Terracing for cultivation on the island of San Tiago. The area suffers from severe erosion caused by intense rainfall (Fig. 235).

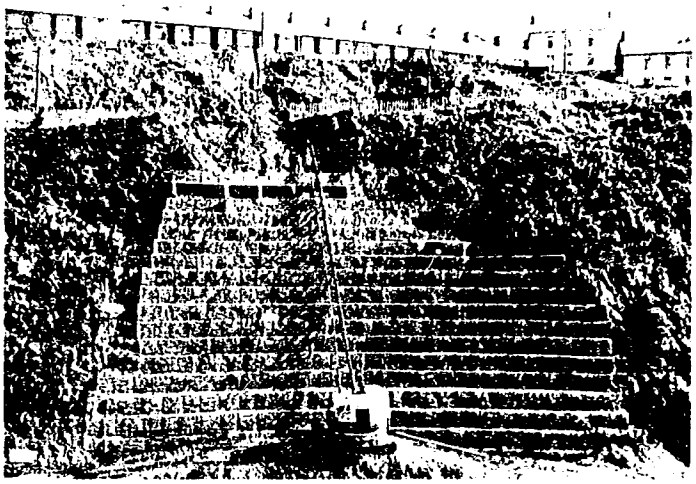
ITALY - Valle d'Aosta - 1980

Terracing with retaining walls to consolidate a steep unstable slope in Lis (Fig. 234).

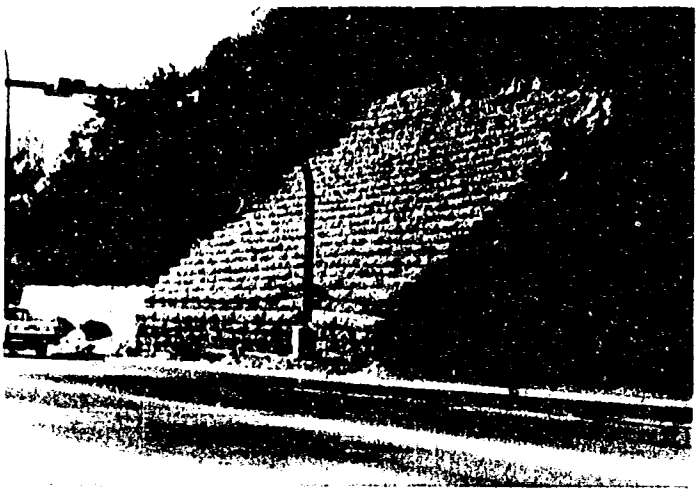
UNITED KINGDOM - Wales - 1968

A 17 metre high stepped revetment at Tylorstown in Mid Glamorgan. Fig. 236 shows the slip area as work was started. The structure safeguards a road and dwellings above it (Fig. 237).

236



235



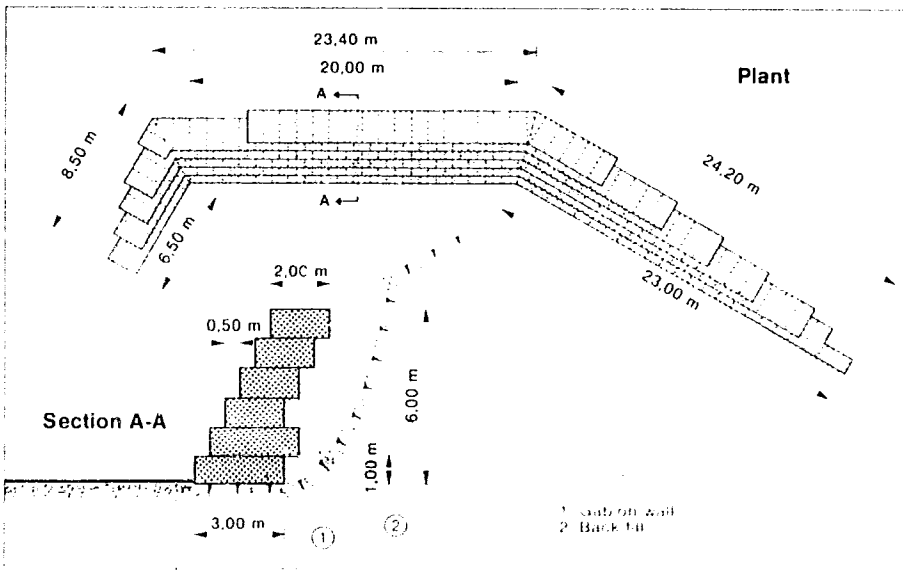
U.S.A. - California - 1973

Revetment stabilizing a vulnerable slope (Fig. 238).

UNITED KINGDOM - England - 1973

Wall in the British Leyland works complex at Longbridge, Birmingham (Fig. 239).

Figure 240 shows the plan and a cross section through the structure in which PVC coated gabions were used in view of the acid nature of the backfill

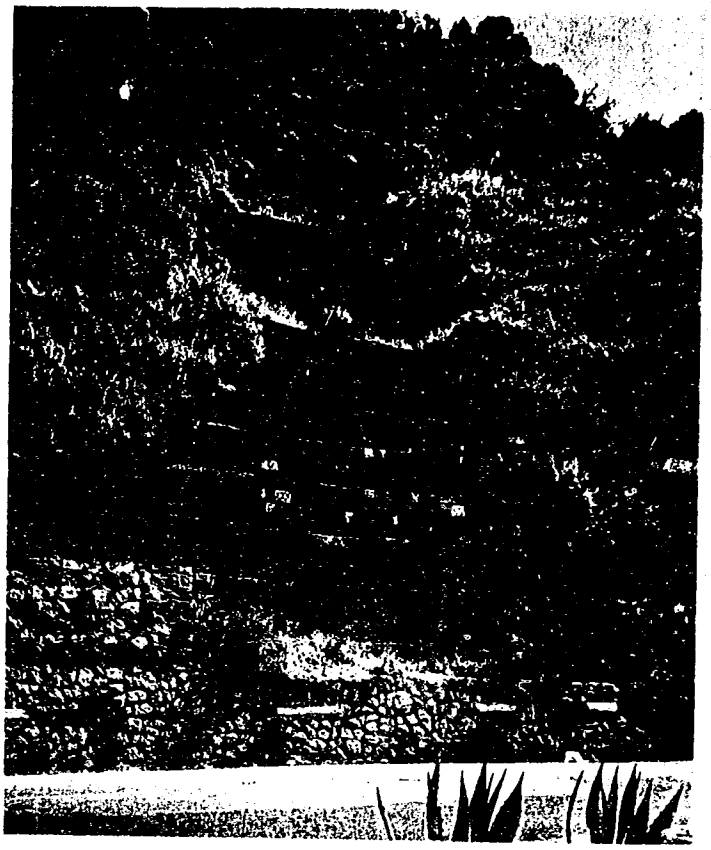


241

NEPAL - 1976

High wall retaining a steep slope and highway (Fig. 241).

242



ITALY - Sicily - 1979

Revetments built at various levels on a slope to protect a road in Taormina, province of Messina (Fig. 242).

243

244

ITALY - Lazio - 1981

State Highway 509 near Frosinone-a slope revetment (Fig. 243).

ITALY - Marche - 1982

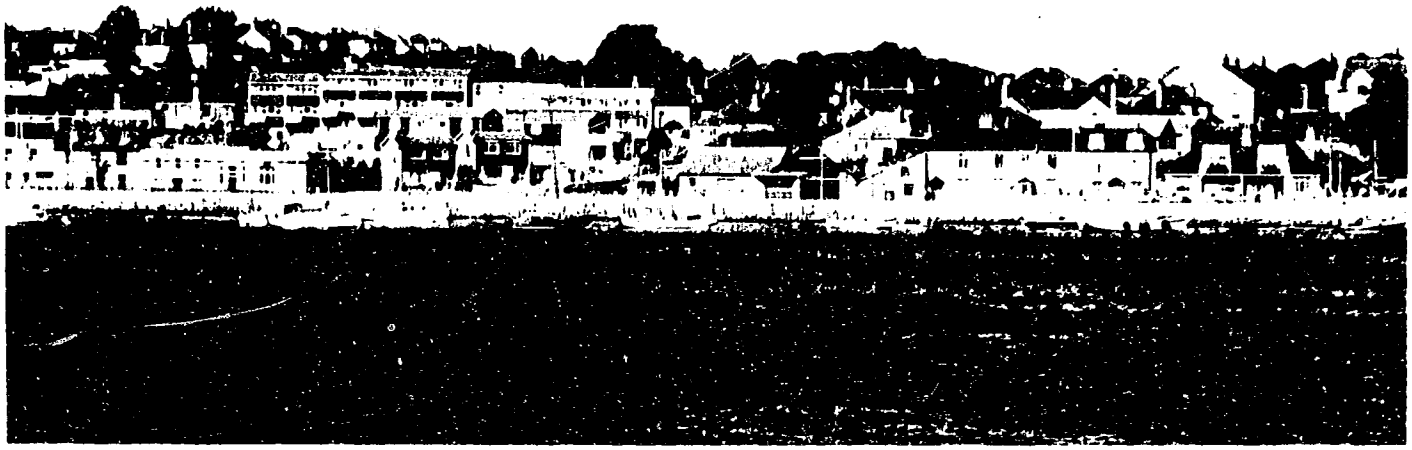
Revetment to stabilise a slope on the Pietramaura-San Leo Road, Province of Pesaro (Fig. 244).

245

BRUNEI - 1983

An unstable zone on a slope near the capital has been re-established with a gabion revetment (Fig. 245).

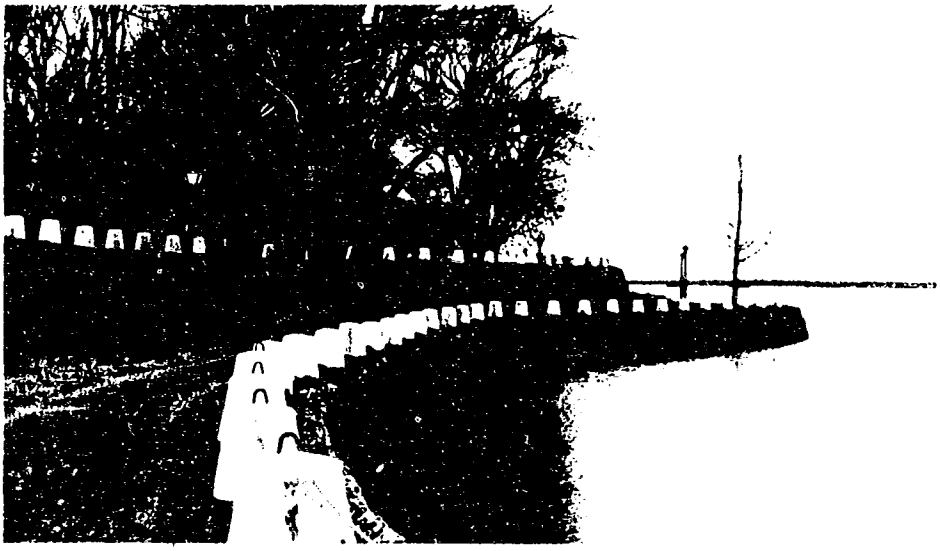
246



UNITED KINGDOM - England - 1963

PVC gabion river wall at Oreston, Plymouth as part of a reclamation and beautification scheme. Constructed in 1964 (Fig. 246).

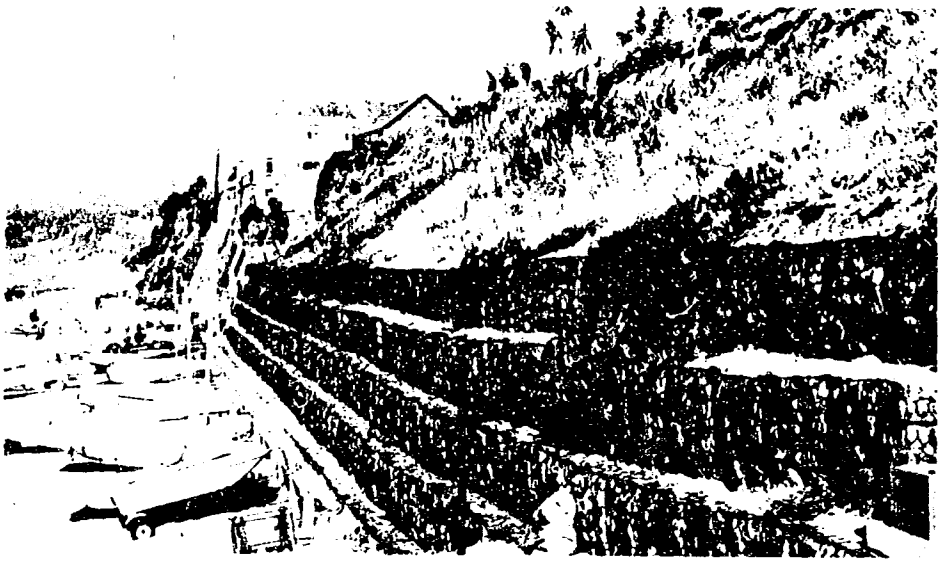
247



CANADA - Quebec - 1971

Walls to protect a city park on the banks of the St. Lawrence River in Candiac (Fig. 247).

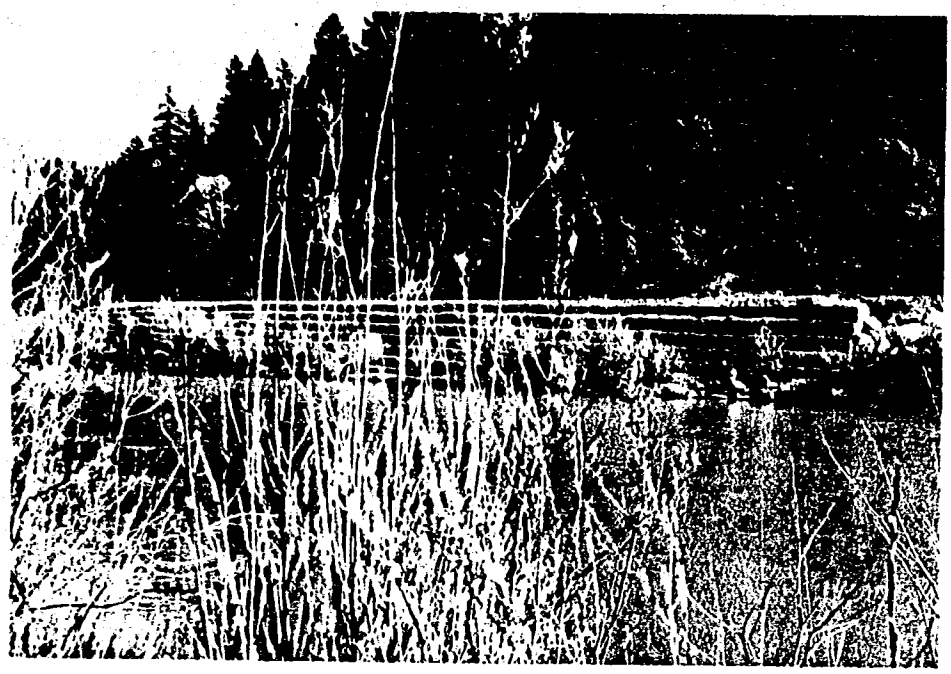
248



UNITED KINGDOM - England - 1971

A substantial gabion wall at Downderry, in Cornwall, designed to retain the cliff face and to protect it from wave erosion (Fig. 248). The marine environment dictated the use of PVC coated gabions.

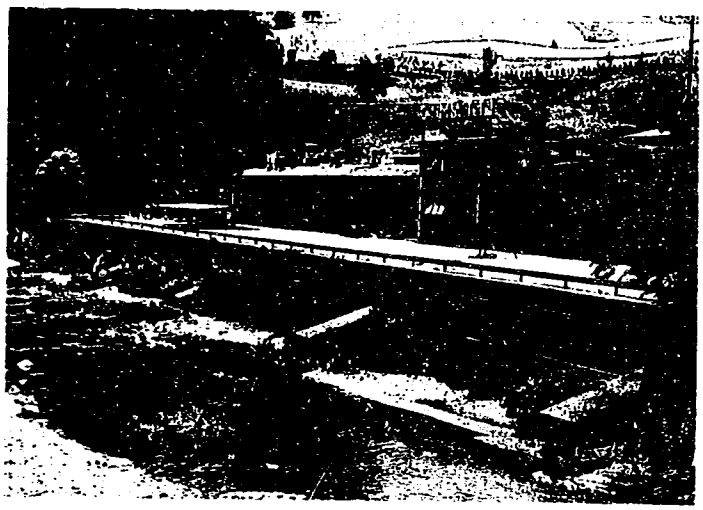
249



U.S.A. - California - 1974

Retaining wall to support a road adjacent to the Eel River (Fig. 249).

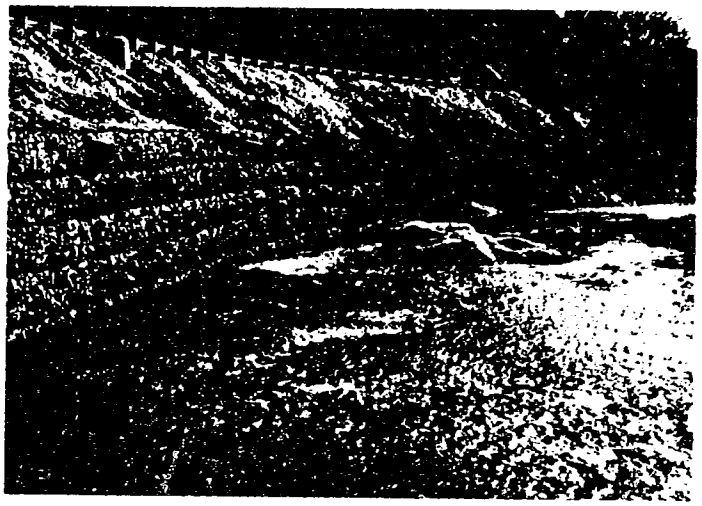
250



ITALY - Emilia-Romagna - 1974

Retaining wall to support a large parking area in Pioppe di Salvaro, Province of Bologna, on the River Reno. A series of small groynes were combined with the wall to direct the flow away from the foundation and prevent undermining of the structure (Fig. 250).

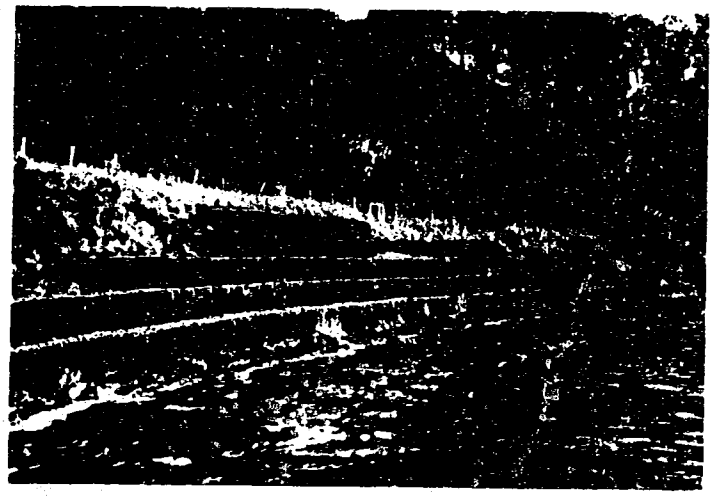
251



U.S.A. - Maryland - 1975

Retaining wall of PVC-coated gabions to protect Falls Road from possible erosion by the Jones Falls River near Baltimore (Fig. 251).

252



UNITED KINGDOM - Scotland - 1970

PVC gabion wall providing support and protection for a road at Dunkeld in Tayside Region (Fig. 252).

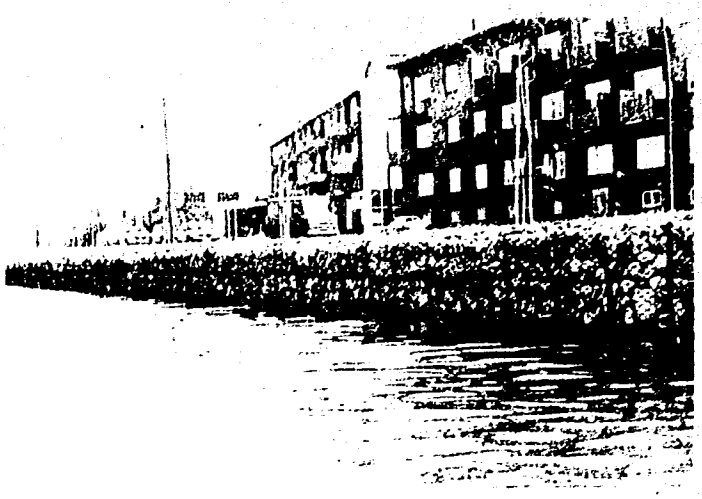
253

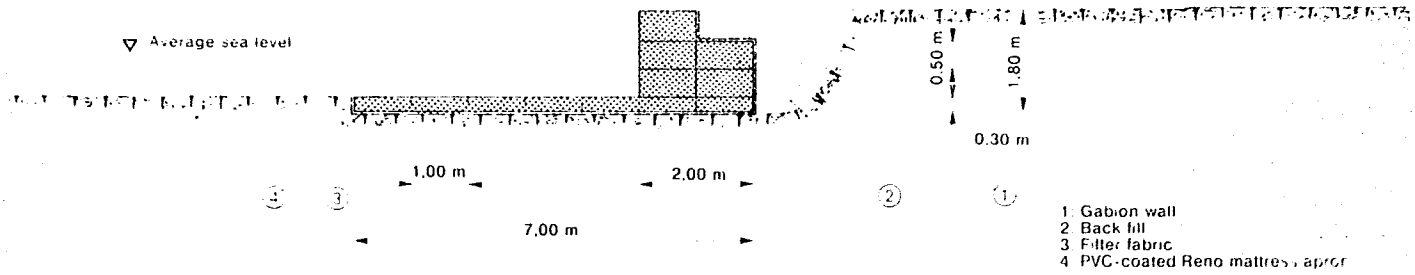
CANADA - Ontario - 1978

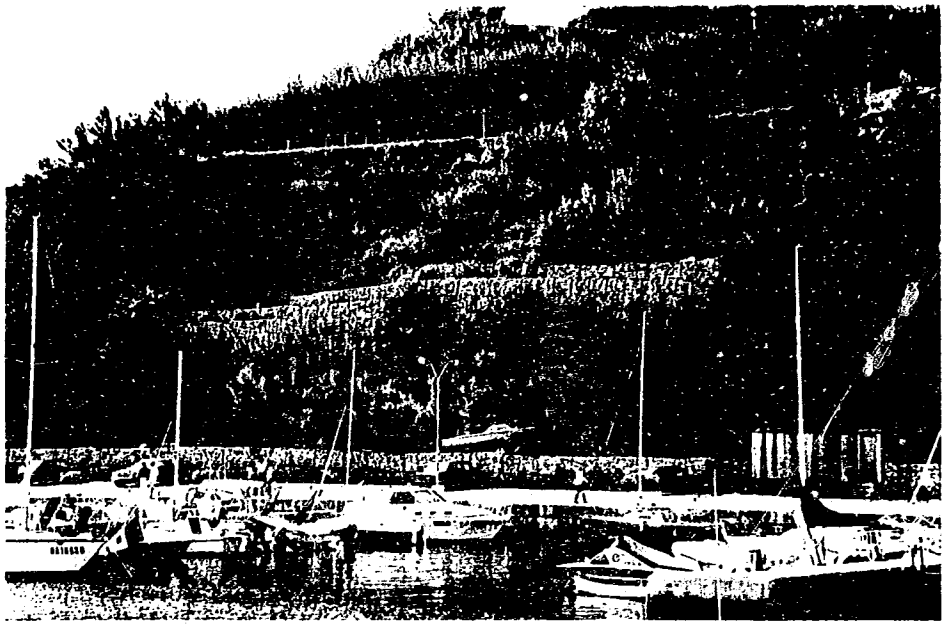
Gabion wall to protect a mansion on Lake Ontario in Oakville, near Toronto (Fig. 253).

SWEDEN - 1978

Wall in Karlskrona harbour (Fig. 254). A crane was used to install the Reno mattress apron and gabion section. To prevent migration of fines, a filter fabric was placed between the gabions and the backfill (Fig. 255).

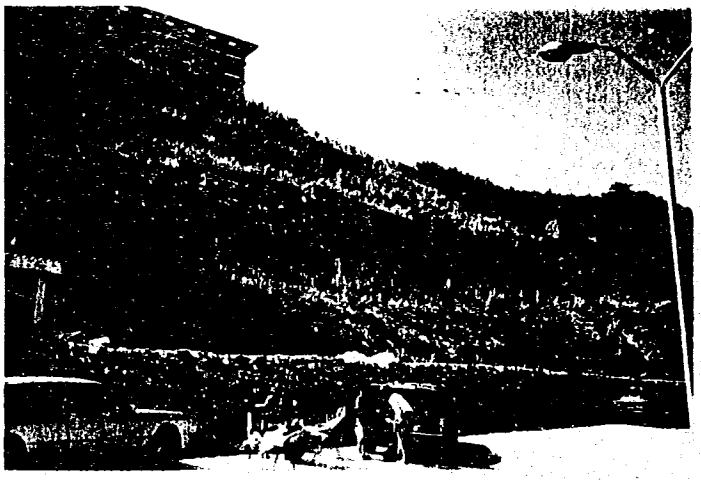






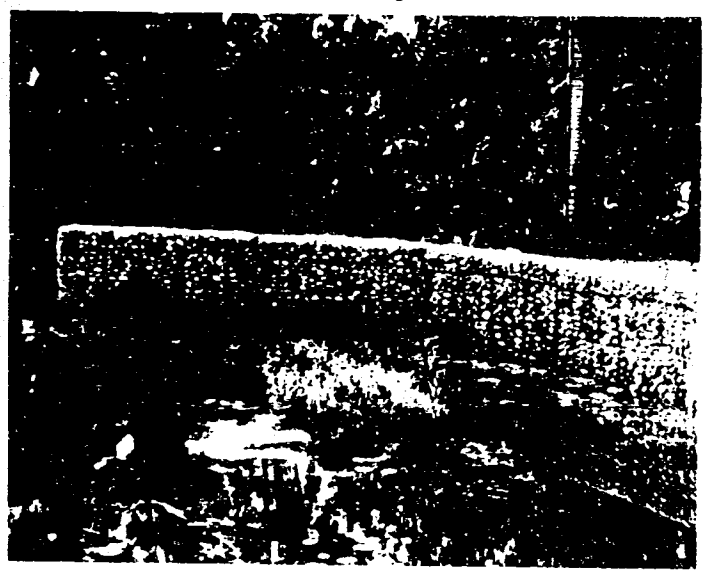
ITALY - The Marche - 1978

Wall supporting a slope overlooking the little port of Baia Vallugola near Gabicce Monte, Province of Pesaro (Figs. 256 and 257).



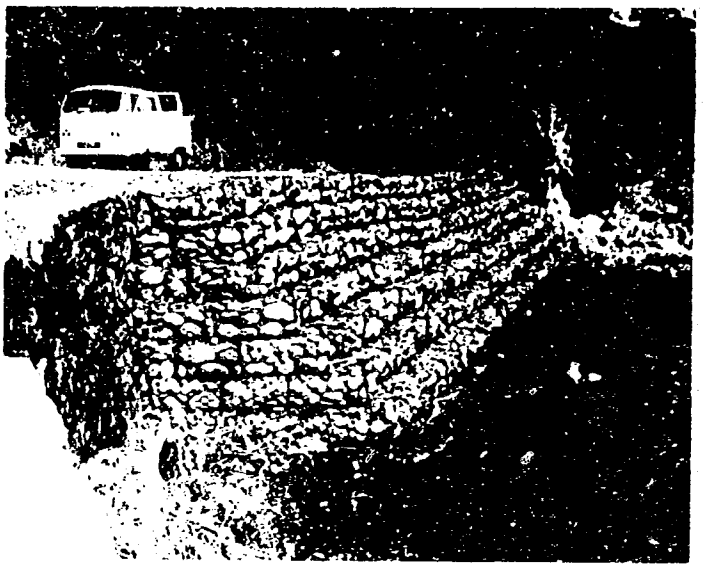
HAITI - 1979

Small wall at the side of the Port au Prince - Les Cayes road to counter river erosion (Fig. 258).



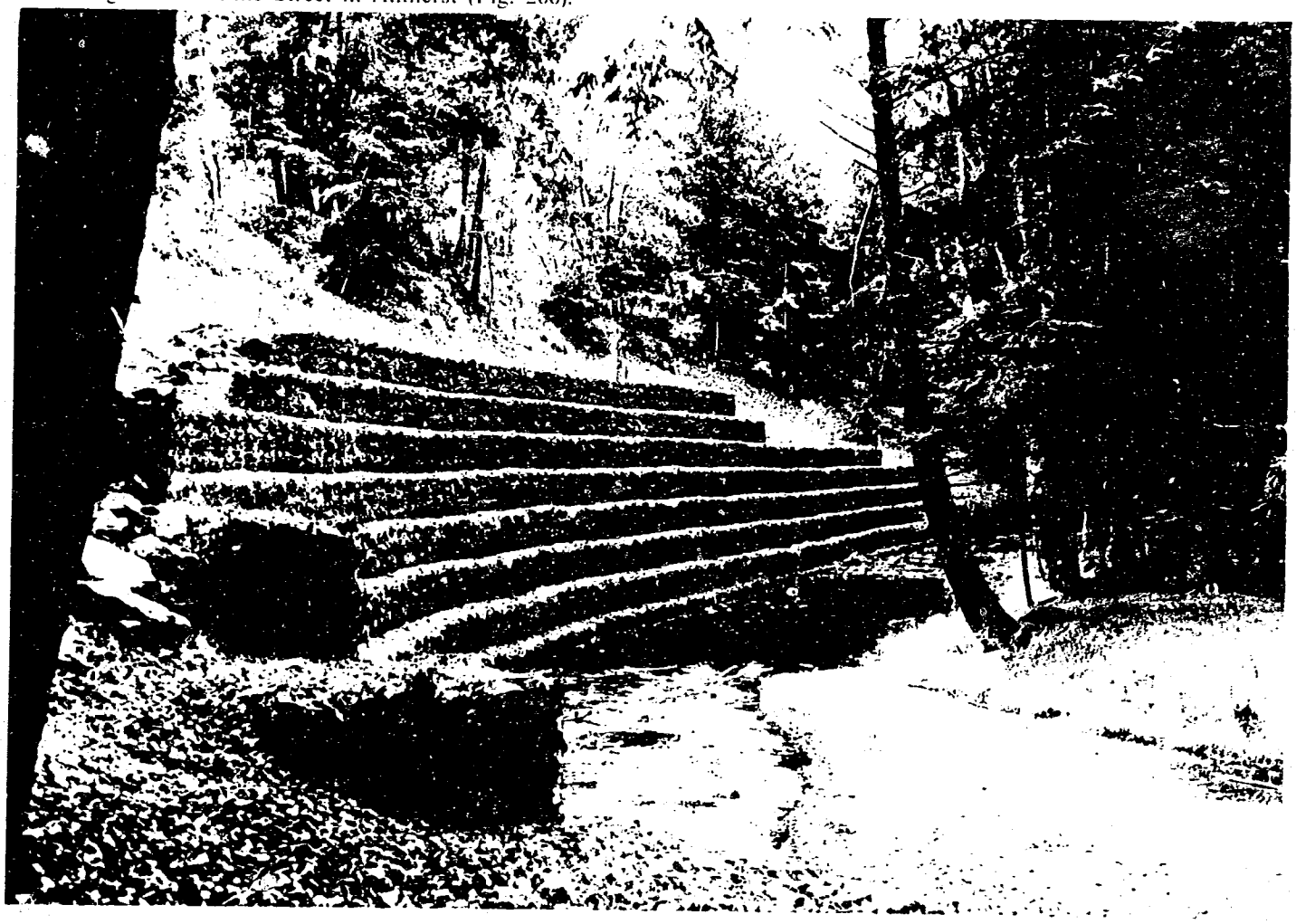
DOMINICAN REPUBLIC - 1980

Foreshore protection to the coastal road bordering Mangoe Bay near Margoz airport (Fig. 259).



U.S.A. - Massachussetts - 1980

Retaining wall on Pine Street in Amherst (Fig. 260).



BRAZIL - Sao Paolo - 1981

Retaining wall on Anchieta Road in Sao Paolo (Fig. 261).

TRINIDAD AND TOBAGO - 1982

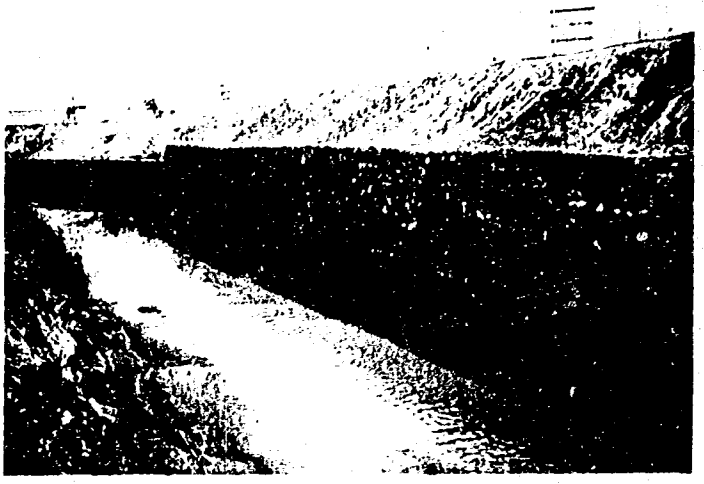
Stepped wall protecting a road along a reach of the Malik River near Port of Spain on the island of Trinidad (Fig. 262).

ARGENTINA - 1983

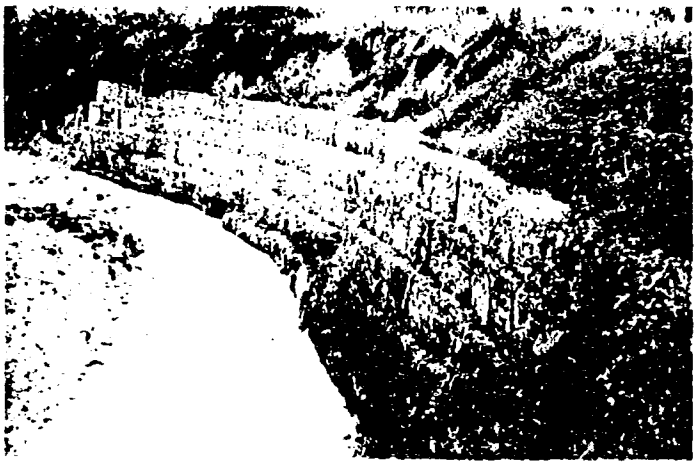
Wall on the access road to the Nihuil I hydro-electric plant on the Atuel River in Mendoza (Fig. 263).

BOLIVIA - 1984

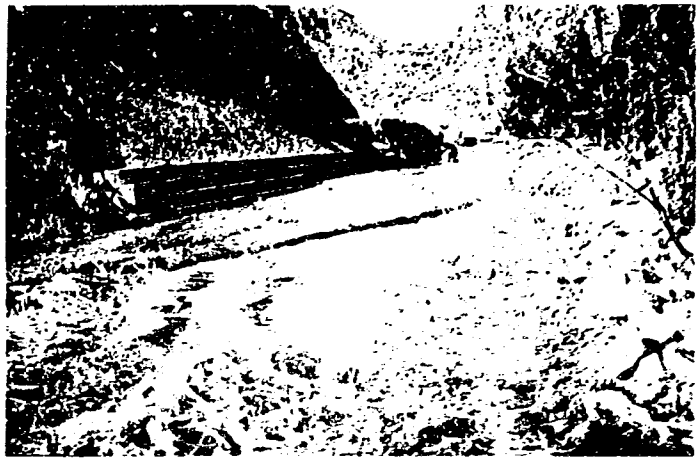
Protection for the transformer compound of the Santa Isabel power station near Cochabamba (Fig. 264).



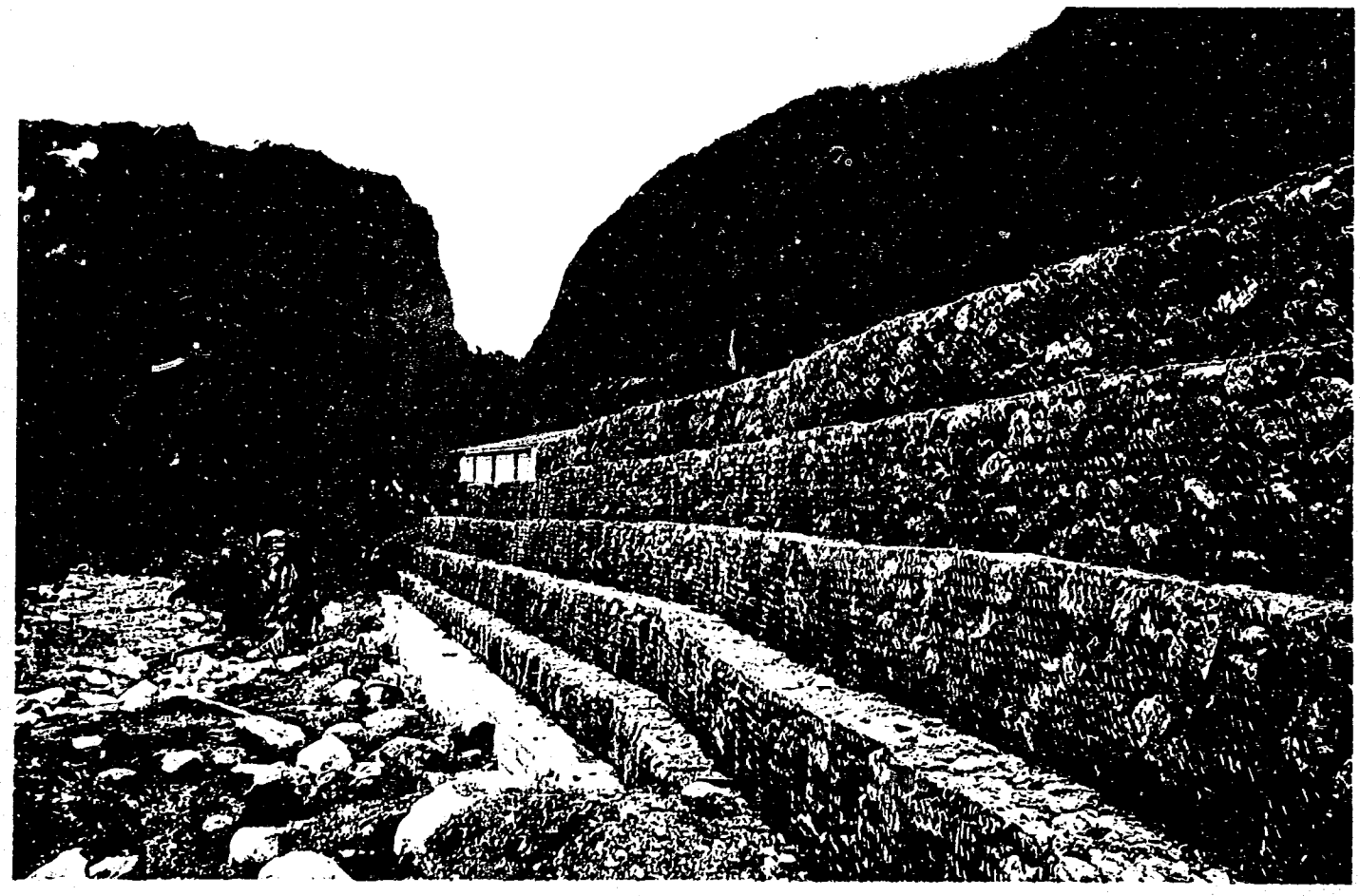
262



263

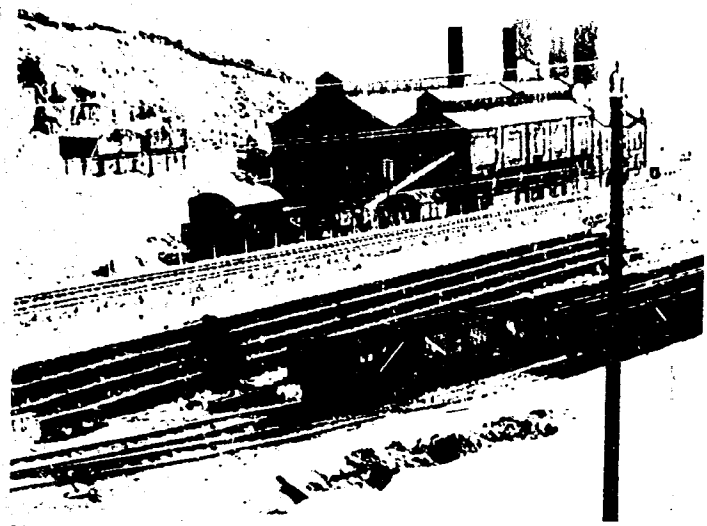


264



6.6 Works appertaining to buildings

265



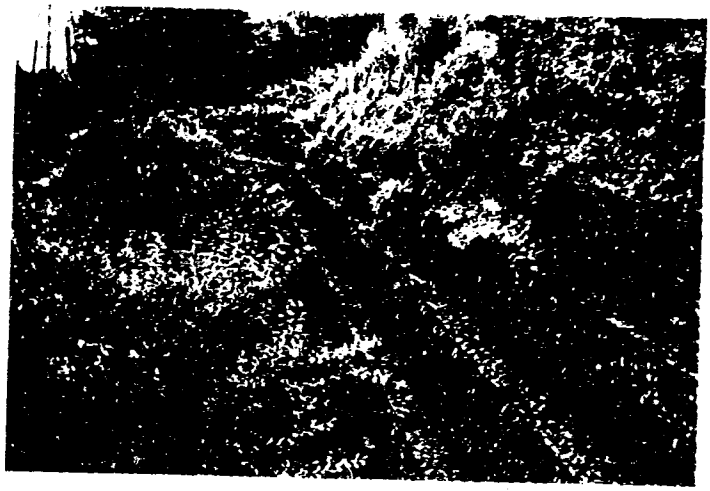
UNITED KINGDOM - Wales - 1954

Wall at British Railways depot in the Rhondda, Mid Glamorgan (Fig. 265).

U.S.A. - Maryland - 1977

Lining to prevent erosion by rainwater on the sides of an earth bank surrounding the Hess Oil Co. tank farm in Baltimore (Fig. 266).

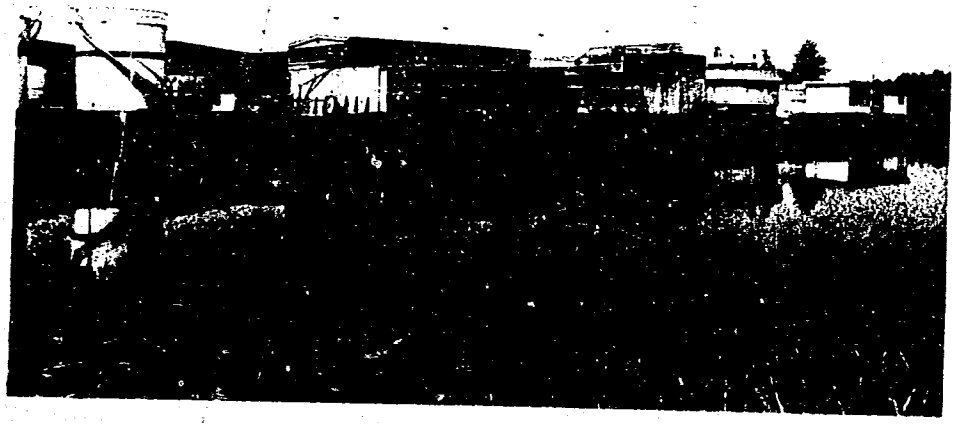
Such embankments serve to contain any leaks of crude oil.



268

UNITED KINGDOM - South Wales - 1973

Gabion wall retaining the hillside around a transimission tower of the Central Electricity Generating Board near Pontypridd (Fig. 267). In Figure 268 the same wall is shown after more than ten years.

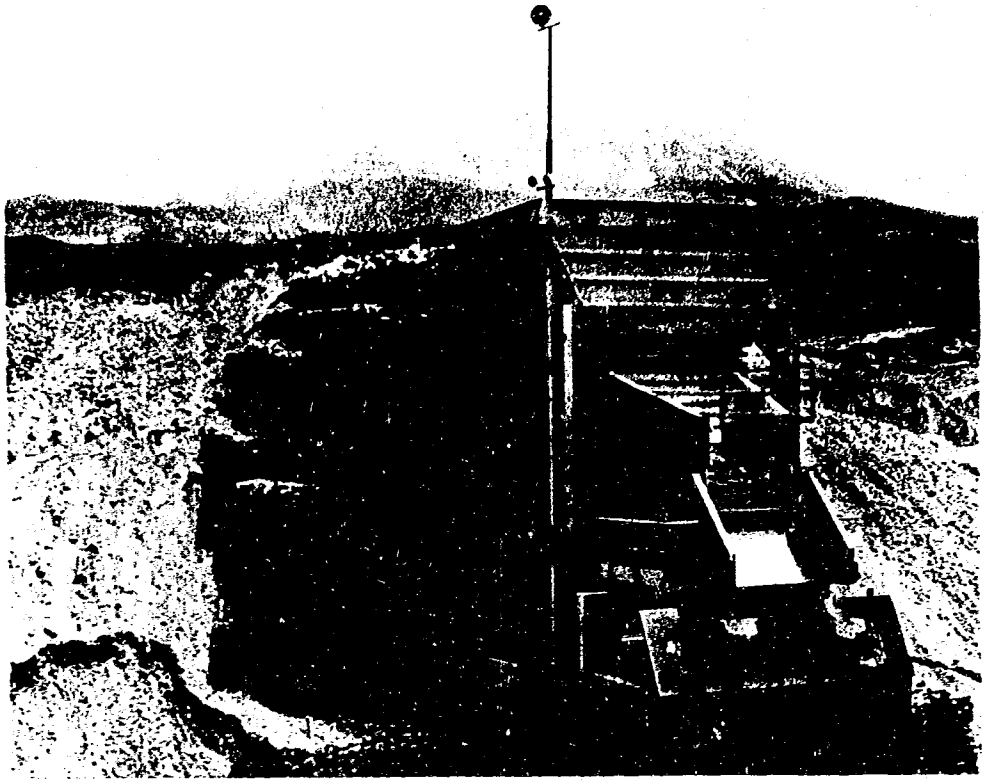


ITALY - Lombardy - 1977

Gabion protection at the thermo-electric power station on the side of the La Muzza canal in Tavazzano and Montenaso (Fig. 269).

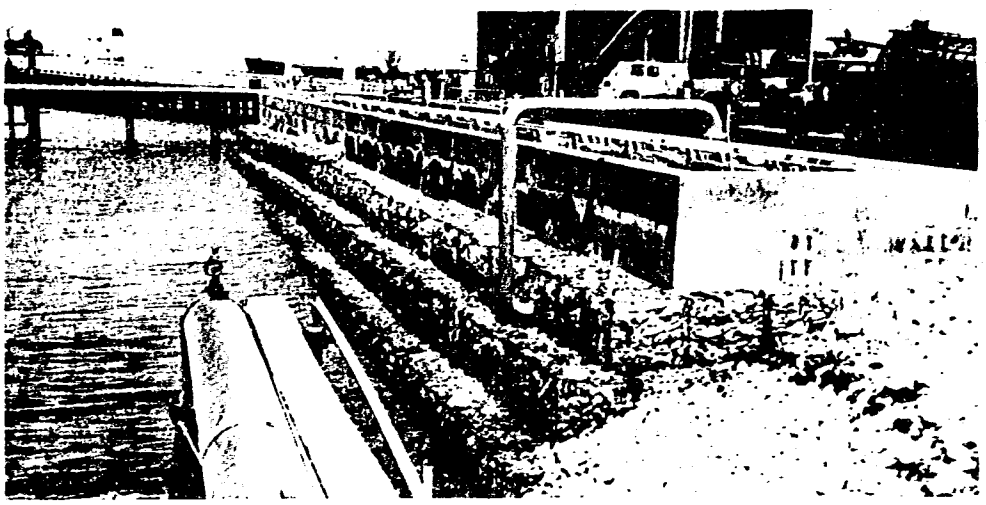
IRAN - 1978

Temporary gabion structures in the sand and gravel production area in the contractor's compound during the construction of the Lar Dam (Fig. 270).



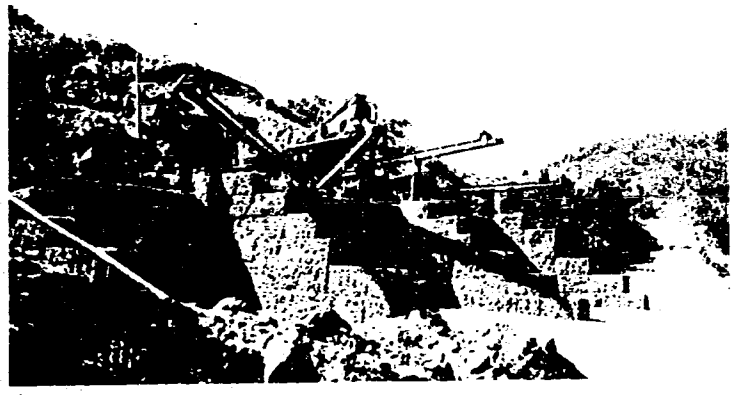
BAHRAIN - 1979

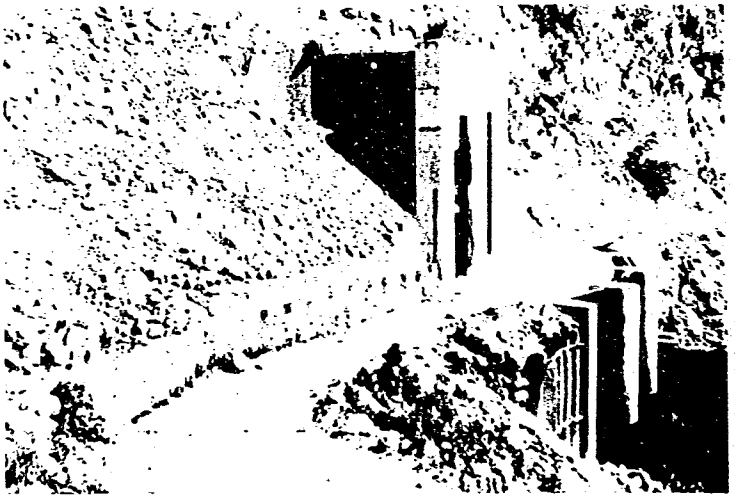
Structure in PVC coated gabions carrying pipelines at the side of the B.A.P.C. causeway (Fig. 271).

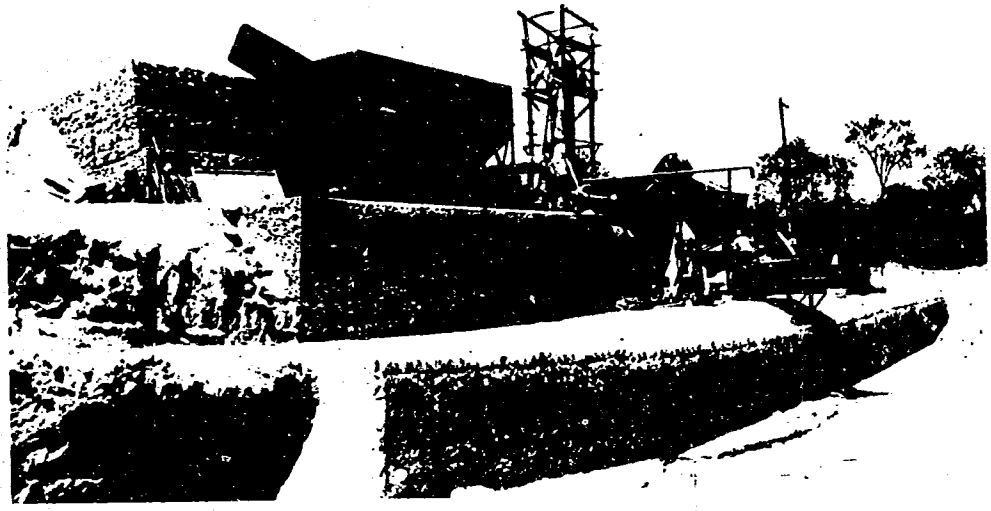


HONDURAS - 1980

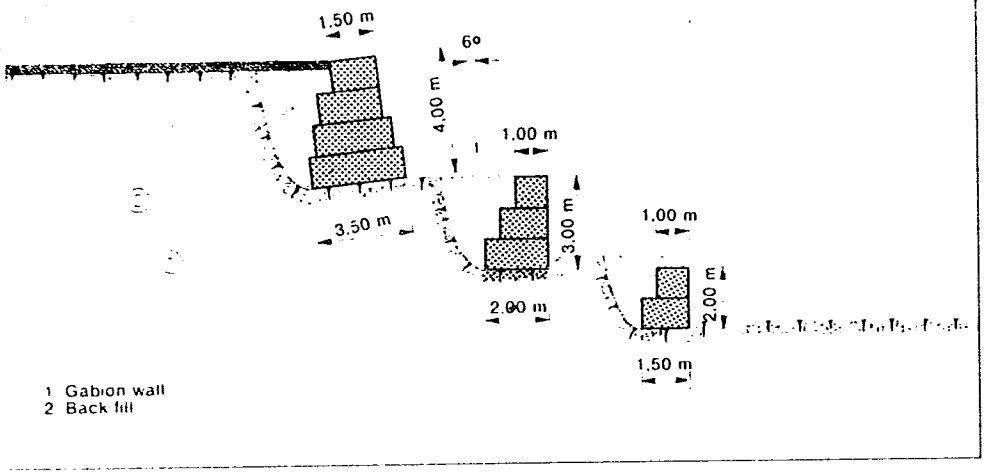
Wall supporting the crushing plant in the contractor's compound during the construction of the El Cajon dam (Fig. 272).
Figure 273 shows a wall built in 1982. It is anchored with tie rods and supports a slope along the outfall below the dam.

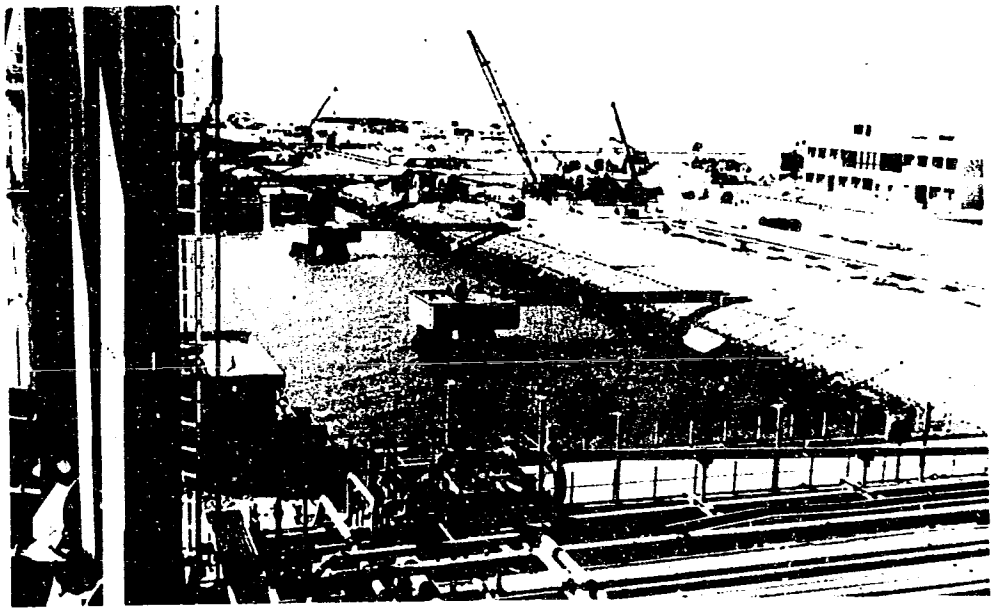




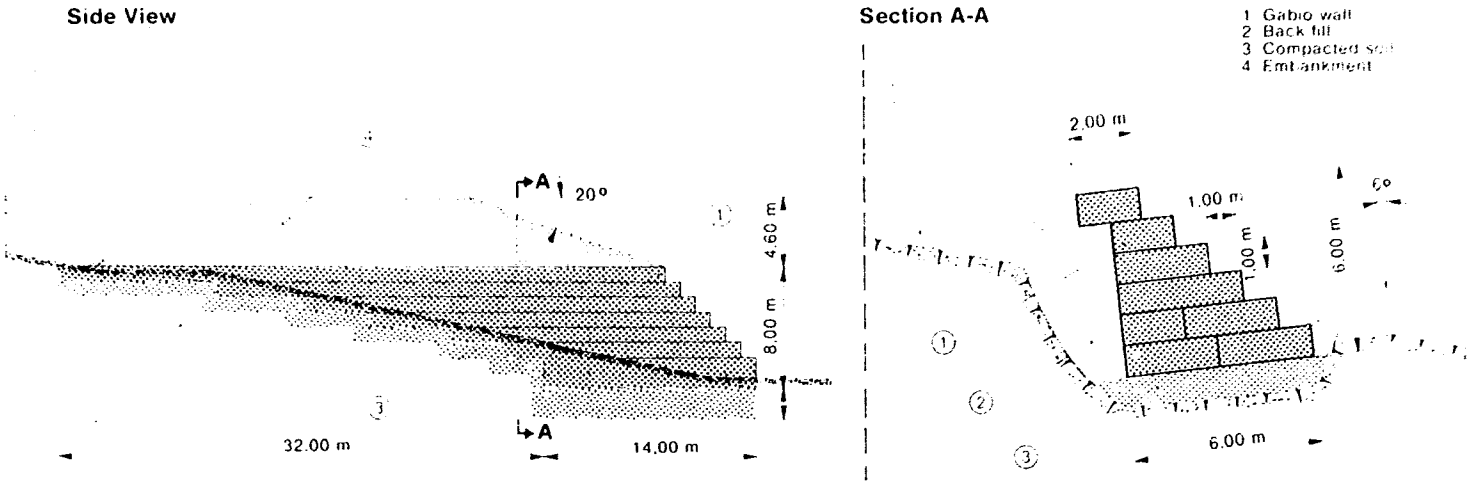


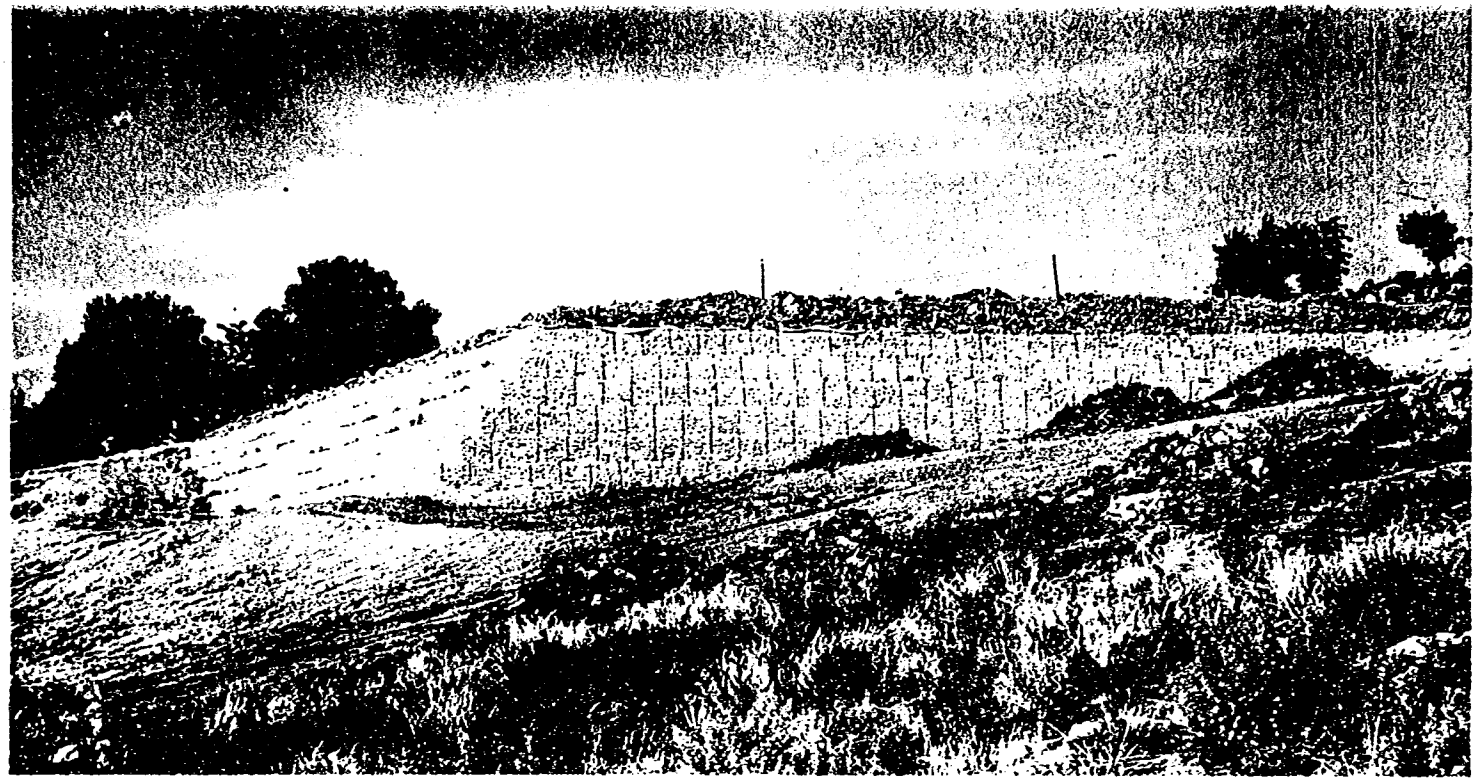
BRAZIL - Sao Paolo - 1982
Structure, built on three levels, to support a crushing plant in Jundiai, near Sao Paolo (Fig. 274).
Figure 275 shows a cross section of the scheme.

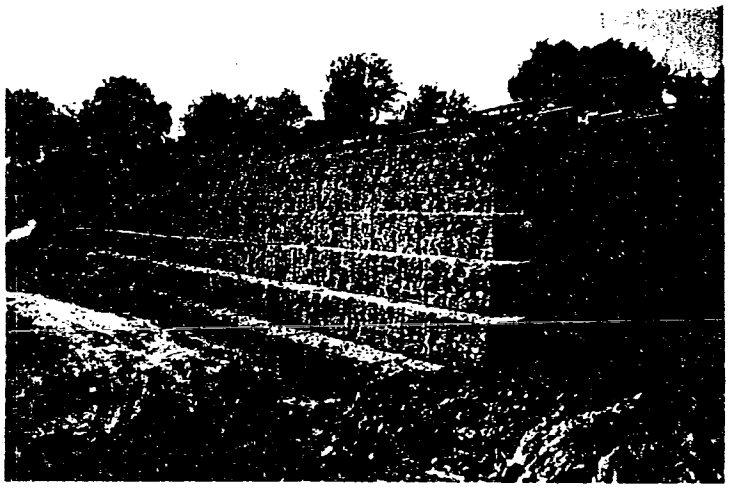




UNITED ARAB EMIRATES - Abu Dhabi - 1982
Plastic-coated Reno mattress lining a bank at the oil pipeline terminal at Umm Al Nar (Fig. 276).

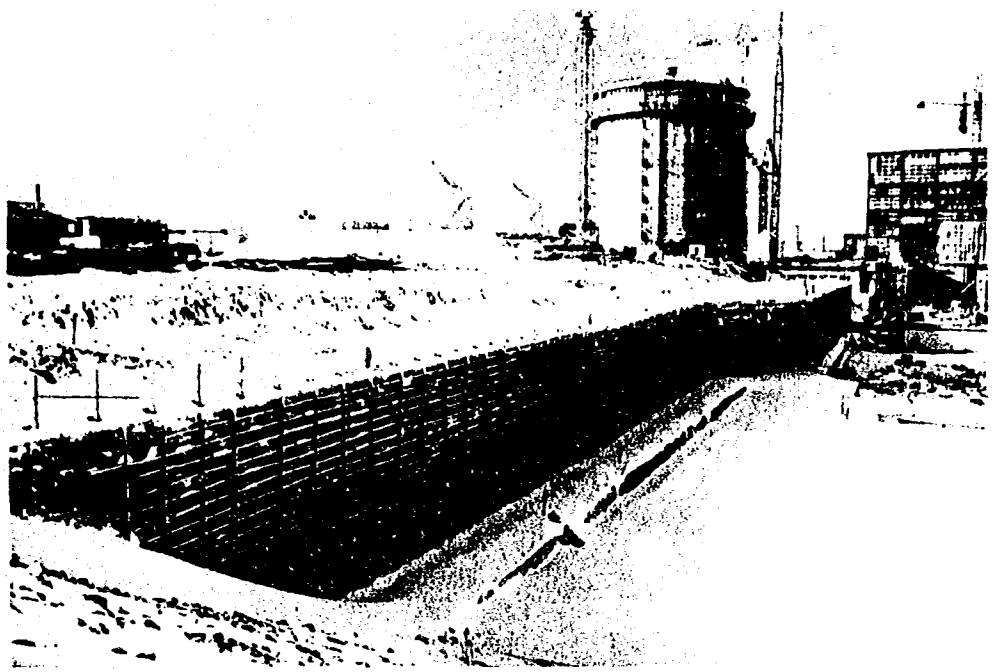






ITALY - Emilia-Romagna - 1984

Wall under construction to retain backfill at San Patrignano near Rimini. An agricultural establishment will be set up later on the backfill. Figure 277, 278, and 279 show a cross section of the project and the wall under construction.

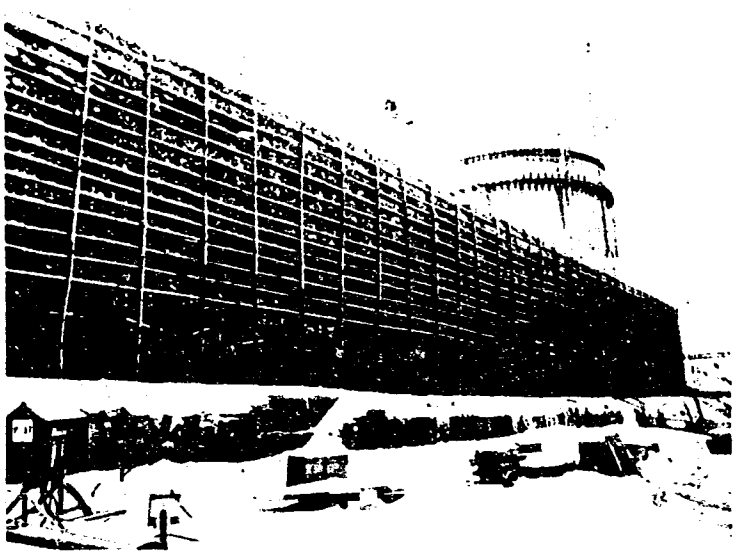


CANADA - Ontario - 1984

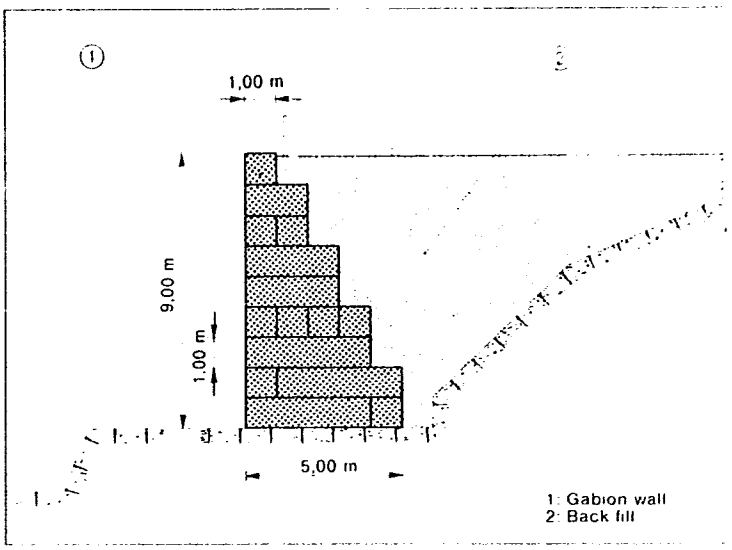
Nine metre high wall to support the turning area for the crane used to lift the generators for the nuclear power station at Darlington near Newcastle (Figs. 280, 281, 282 and 283).

The crane, among the largest in the world, is 56 m. high and has a reach of 49 m.

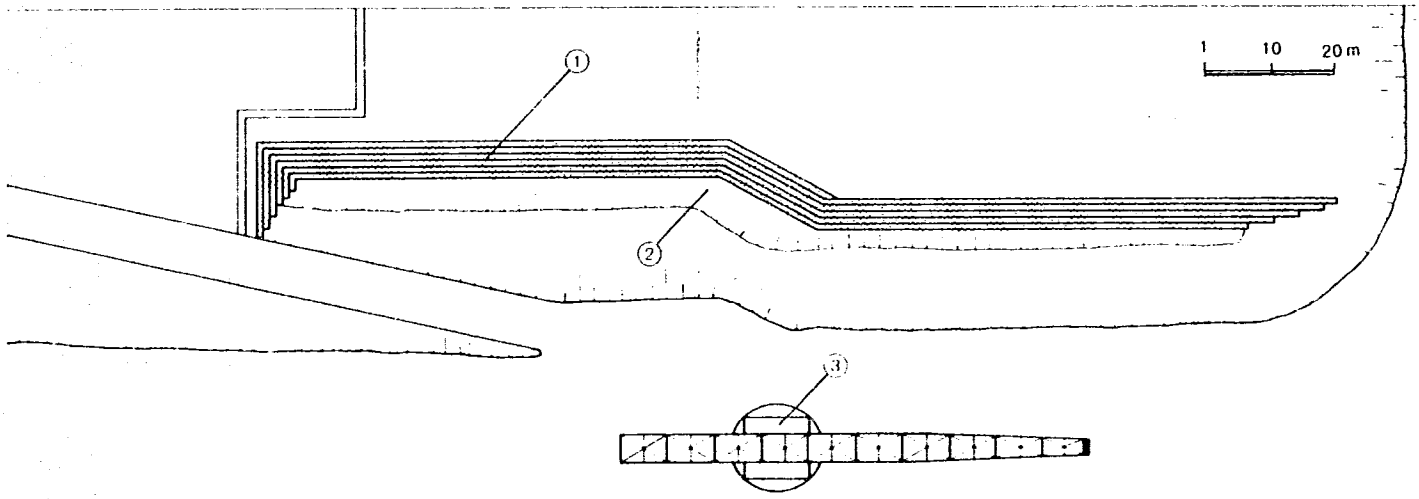
281



282



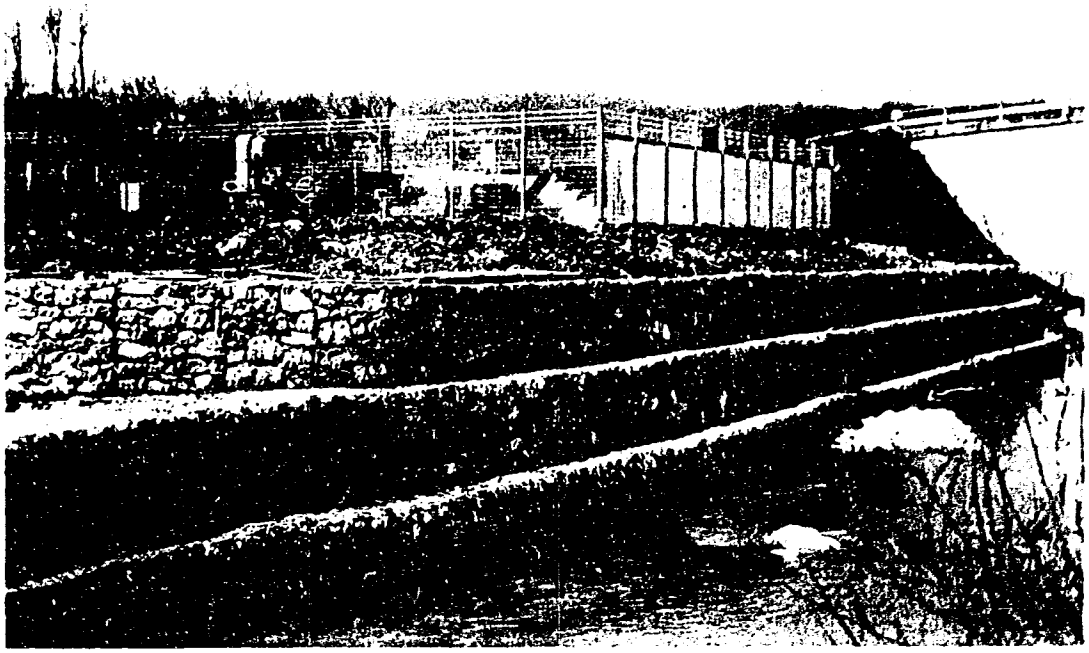
283

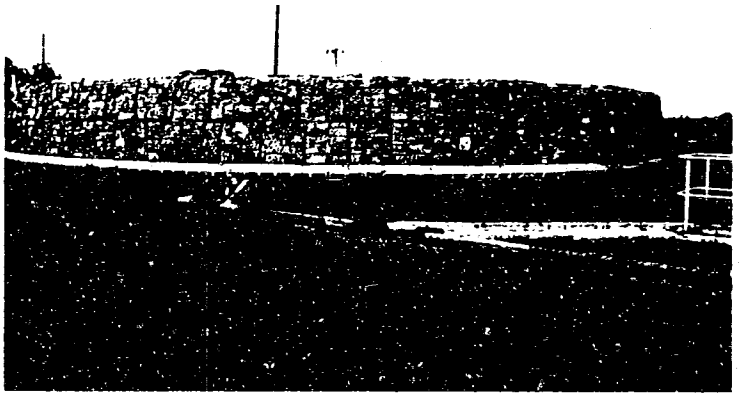


1. Gabion wall
2. Back fill
3. Crane

ITALY - Campania - 1983

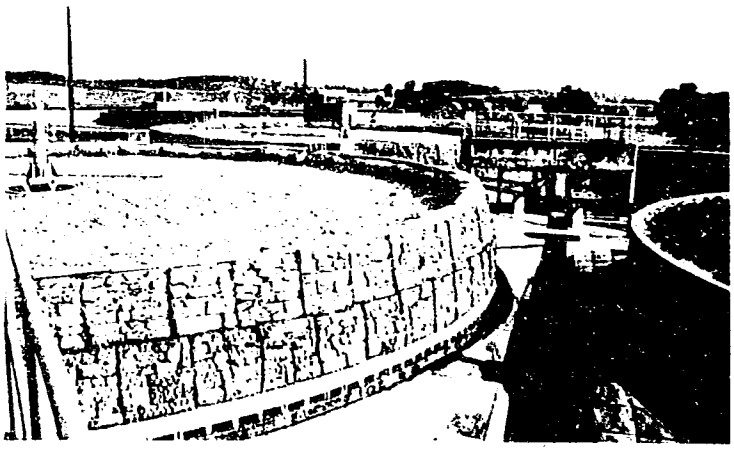
Gabion wall at an industrial establishment at Atripalta in the Province of Avellino (Fig. 284).





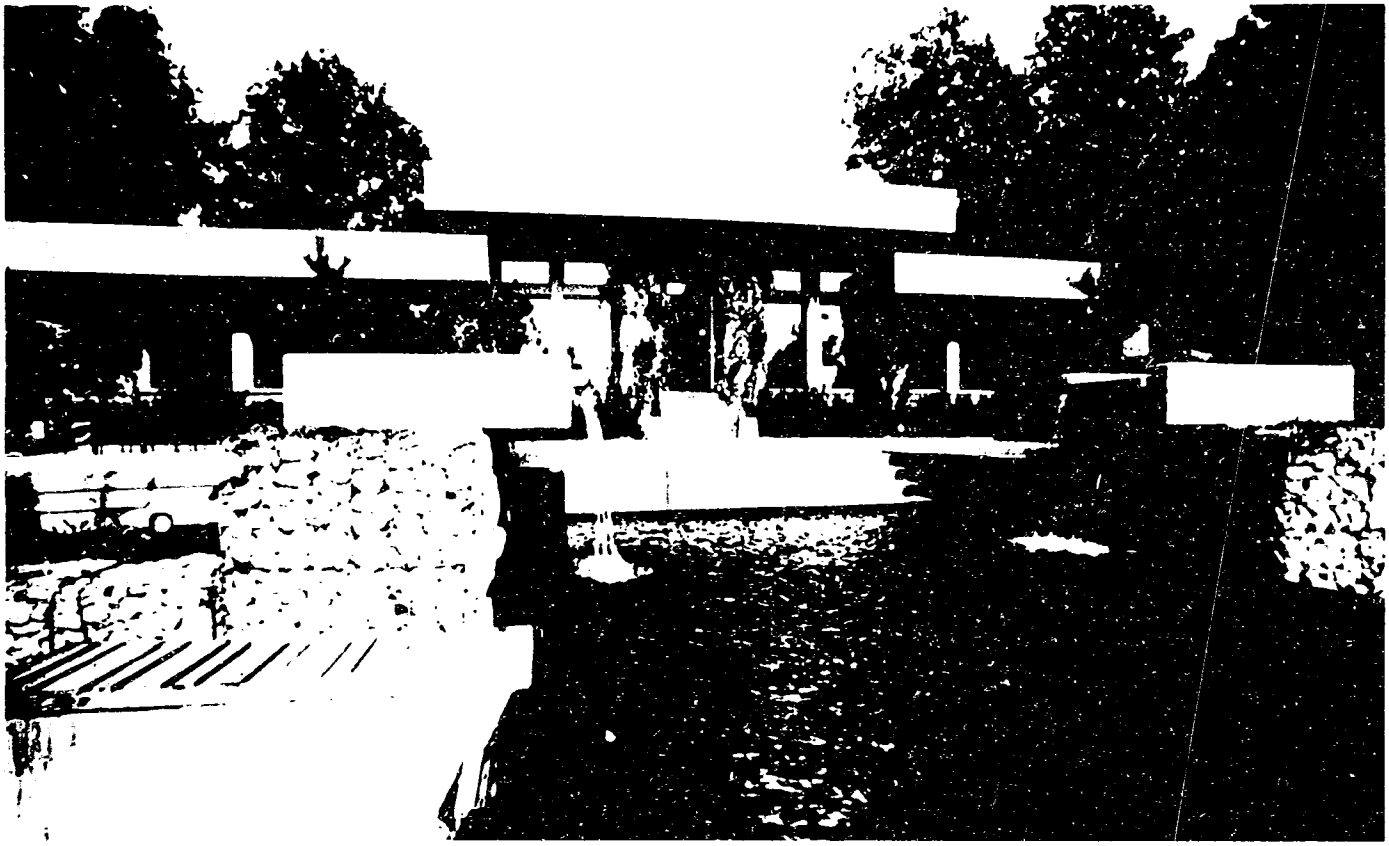
UNITED KINGDOM - England - 1984

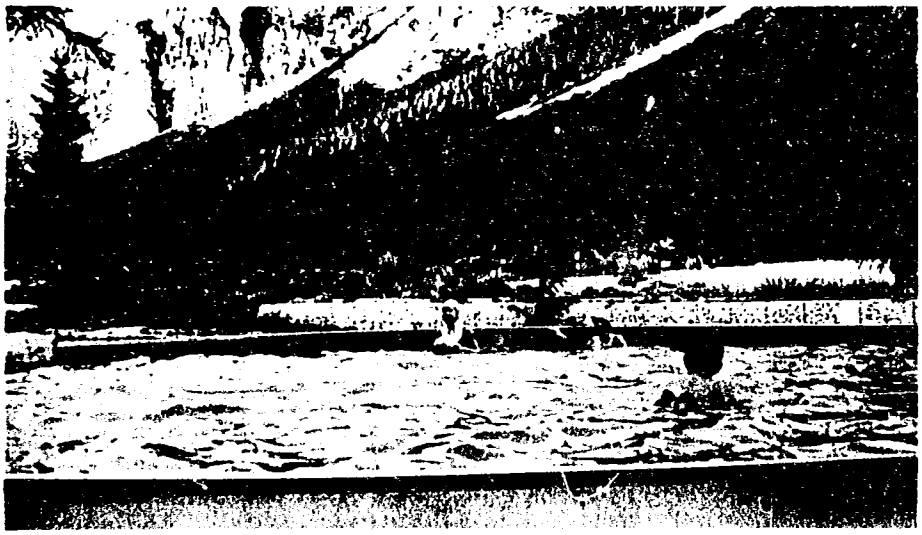
Biological filters in the sewage treatment plant at Barnoldswick in Lancashire (Fig. 285). The circular walls containing the filter material are built of PVC-coated gabions (Figs. 286 and 287).



GABIONS AS ARCHITECTURAL FEATURES!!

U.S.A. - Missouri - 1980
Gabion architectural feature in a house in the Meramec River Valley near St. Louis (Figs. 288, 289)





ITALY - Trentino Alto Adige - 1980
Gabion wall supporting a bank close to a swimming pool in Cortina d'Ampezzo (Fig. 290)

Appendix - Table of conversion system metric - imperial.

Quantity	Metric units	Imperial units	Conversion factor
FORCE	Newton Newton per metre	Pound force Pound force per foot	1N* = 0.22482 lbf 1N/m = 0.06854 lbf/ft
PRESSURE - STRESS	Newton per square metre Meganewton per square metre	Pound force per square foot Pound force per square inch	$1N/m^2 = 0.02089\ lbf/ft^2$ $1MN/m^2 = 145.032\ lbf/in^2$
LENGTH	Metre Millimetre	Foot Inch	1 m = 3.2808 ft 1 mm = 0.03937 in
AREA	Square metre	Square foot	$1\ m^2 = 10.7643\ ft^2$
VOLUME	Cubic metre Cubic metre	Cubic fott Cubic yards	$1\ m^3 = 35.3107\ ft^3$ $1\ m^3 = 1.3079\ yd^3$
MASS	Tonne Kilogramme	Ton Pound	1 ton = 0.98425 t 1 kg = 2.20459 lb
DENSITY	Kilogramme per cubic metre	Pound per cubic foot	$1\ kg/m^3 = 0.06242\ lb/ft^3$

(*) To obtain metric international system values from M.K.S. system used in this book, this additional conversion factor should be considered: 1 kg = 9.80665 N

APPENDIX

Approximate dimensions of gravity and semigravity gabion retaining structures

The tables in this appendix were prepared to simplify the selection of approximate dimensions in the initial stages of designing gravity (1, 2, 3, 8, 9), and semi-gravity gabion retaining structures. The semi-gravity type being that which relies on the weight of the soil over the heel to maintain stability. See Chap. 4, Tables 4, 5, 6, 10, 11, 12.

The tables numbered from 1 to 6 are for smooth rear-face and stepped front-face walls, and the tables numbered from 7 to 12 are for walls with a smooth front-face and a stepped rear-face.

For gravity walls 2 to 10 m high a choice of three separate slenderness ratios (H/B) is offered. For semi-gravity walls, 3 to 10 m high, two H/B ratios are offered for the stem plus an additional solution using an extended foundation as an anchorage.

The foundation courses of walls 5 to 6 m high, especially in the case of semi-gravity walls, should only be 0.5 metre deep.

In Fig. 291 the basic results extracted from the stability analysis produced by the GAWAC programme are shown against the principal dimensions of the structure, H, B, and b. See Chap. IV.

- Safety factor against sliding, η_s;
- Allowable soil pressure, σ_t;
- Allowable compression stress, σ_m, in the gabion structure, for semi-gravity walls only;
- Deformation, Δ, at the top of the structure.

The safety factor against overturning is not given because in this type of structure the requirements for resisting overturning are always exceeded.

The units of measurement of the above data are metres for length and kg/cm^2 for stress.

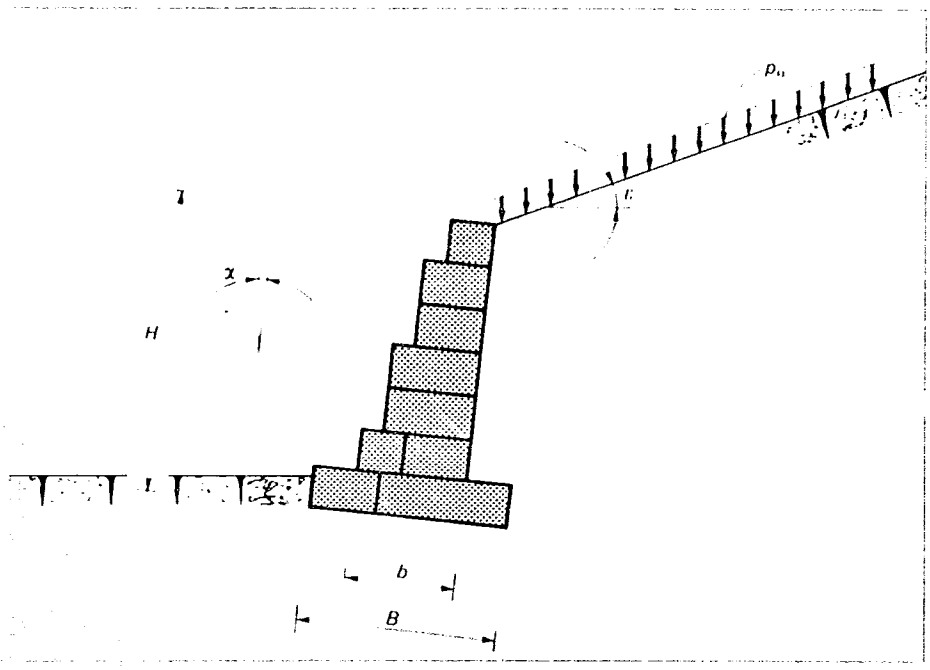
In the computation the unit weight of the filled gabions was assumed to be $\gamma_1 = 1.7$ t/m^3, the density of the soil $\gamma_t = 1.6$ t/m^3, the shear modulus of elasticity of the gabions $G = 40$ t/m^2, the angle of batter $\alpha = 6°$, the depth of the foundation below ground level to be equal to one tenth of the «total» height, H.

The results are given for four different values of the internal angle of friction of the soil, φ, beginning with the smallest value required for stability.

Four different loading conditions are considered:

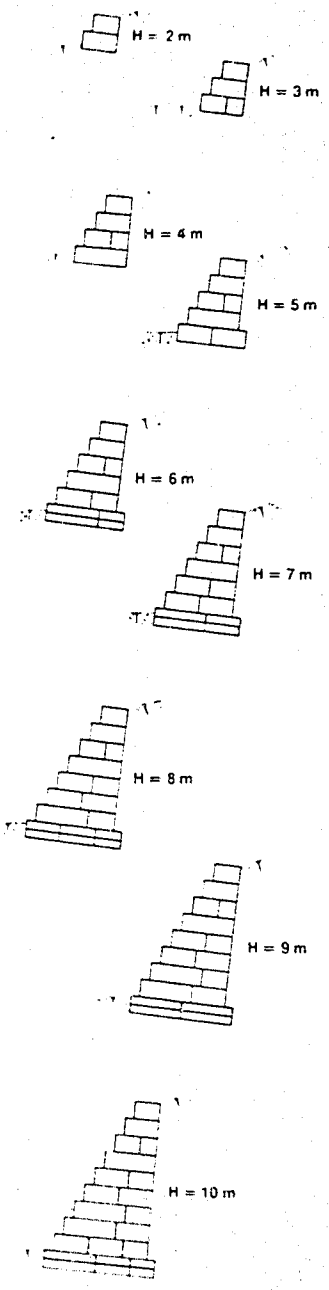
- level backfill without surcharge ($\varepsilon = 0$, $p_0 = 0$);
- level backfill and surcharge equivalent to one metre of soil ($\varepsilon = 0$, $p_0 = \gamma_t \times 1$ m);
- backfill sloped at an angle approximately equal to φ and with no surcharge ($\varepsilon \simeq \varphi$, $p_0 = 0$);
- backfill sloping at 1:2 and with no surcharge ($\varepsilon \simeq 26.6°$ and $p_0 = 0$).

291



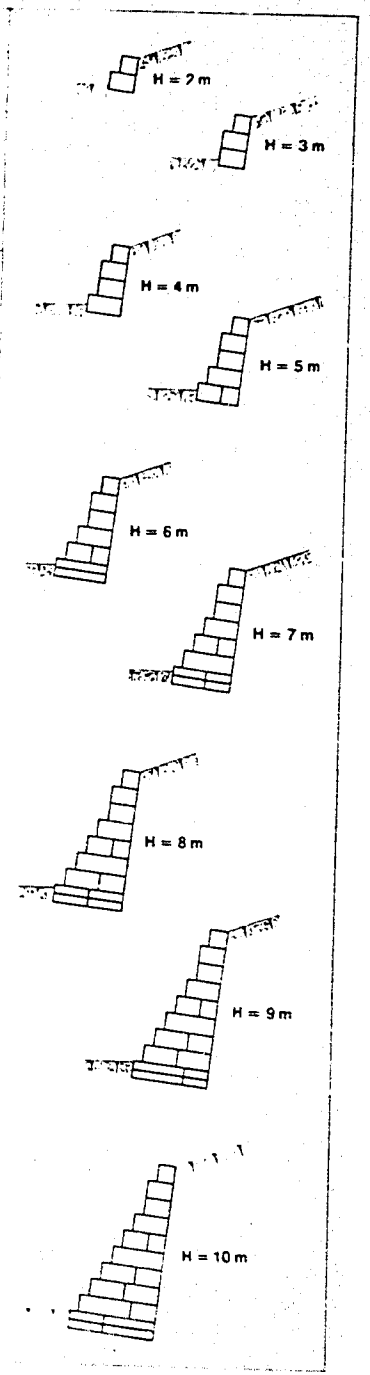
Gravity Walls with a Stepped Front-Face H/B < 1.5

H	B	$\varepsilon = 0$, $p_0 = 0$				$\varepsilon = 0$, $p_0 = \gamma_t$				$\varepsilon > \varphi$, $p_0 = 0$				$\varepsilon = 26.6°$, $p_0 = 0$			
		φ	η_k	σ_t	λ	φ	η_k	σ_t	λ	φ	η_k	σ_t	λ	φ	η_k	σ_t	λ
2	2	15°	1.36	0.4	0.00	21	1.35	0.4	0.01	20	1.37	0.4	0.01	27°	2.20	0.5	0.01
		17°	1.70	0.4	0.00	23	1.68	0.4	0.01	22	1.61	0.4	0.01	29°	3.12	0.5	0.00
		19	2.11	0.4	0.00	25	2.01	0.4	0.01	24	1.88	0.5	0.01	31°	4.10	0.5	0.00
		21	2.60	0.5	0.00	27°	2.44	0.5	0.01	26	2.20	0.5	0.01	33°	5.29	0.5	0.00
3	2.5	17°	1.32	0.6	0.01	22°	1.39	0.6	0.03	23	1.37	0.6	0.02	27°	1.74	0.7	0.02
		19	1.64	0.6	0.01	24°	1.69	0.6	0.02	25	1.60	0.7	0.02	29°	2.46	0.8	0.01
		21°	2.02	0.7	0.01	26°	2.06	0.6	0.02	27°	1.87	0.7	0.02	31°	3.23	0.8	0.01
		23°	2.48	0.7	0.00	28°	2.50	0.7	0.02	29	2.19	0.8	0.02	33°	4.17	0.8	0.00
4	3	19°	1.41	0.8	0.02	22°	1.32	0.7	0.05	25	1.39	0.8	0.04	27°	1.52	0.9	0.05
		21°	1.74	0.9	0.02	24°	1.61	0.8	0.05	27	1.63	0.9	0.04	29°	2.14	1.0	0.03
		23°	2.13	0.9	0.02	26°	1.96	0.8	0.04	29	1.91	1.0	0.04	31°	2.81	1.0	0.02
		25°	2.60	0.9	0.01	28°	2.38	0.9	0.03	31	2.24	1.0	0.03	33°	3.62	1.1	0.01
5	4	20°	1.43	0.8	0.05	23°	1.42	0.7	0.09	26	1.36	0.9	0.09	27°	1.37	0.9	0.10
		22°	1.74	0.8	0.04	25°	1.72	0.8	0.08	28	1.58	1.0	0.08	29°	1.90	1.0	0.07
		24°	2.10	0.9	0.03	27°	2.07	0.9	0.07	30	1.84	1.0	0.08	31°	2.45	1.0	0.05
		26°	2.54	0.9	0.03	29°	2.50	0.9	0.06	32	2.15	1.1	0.07	33°	3.13	1.1	0.04
6	4.5	20°	1.32	0.9	0.08	23°	1.38	0.9	0.13	27°	1.37	1.0	0.14	28°	1.54	1.1	0.13
		22°	1.61	1.0	0.07	25°	1.67	0.9	0.11	29°	1.60	1.1	0.13	30°	2.03	1.2	0.09
		24°	1.96	1.0	0.06	27°	2.02	1.0	0.10	31°	1.87	1.2	0.12	32°	2.61	1.3	0.07
		26°	2.37	1.1	0.05	29°	2.43	1.1	0.08	33°	2.20	1.3	0.11	34°	3.32	1.4	0.06
7	5	21°	1.38	1.1	0.11	23°	1.35	1.0	0.18	27°	1.31	1.2	0.21	28°	1.47	1.2	0.19
		23°	1.68	1.1	0.09	25°	1.63	1.1	0.15	29°	1.53	1.3	0.19	30°	1.94	1.3	0.14
		25°	2.04	1.2	0.08	27°	1.98	1.2	0.13	31°	1.79	1.4	0.18	32°	2.49	1.5	0.11
		27°	2.47	1.3	0.07	29°	2.39	1.2	0.12	33°	2.10	1.5	0.16	34°	3.18	1.6	0.09
8	5.5	21°	1.32	1.2	0.15	23°	1.32	1.1	0.23	28°	1.36	1.3	0.28	28°	1.41	1.3	0.26
		23°	1.61	1.3	0.13	25°	1.60	1.2	0.20	30°	1.59	1.5	0.26	30°	1.87	1.5	0.20
		25°	1.96	1.4	0.11	27°	1.94	1.3	0.18	32°	1.87	1.6	0.24	32°	2.40	1.6	0.15
		27°	2.37	1.4	0.10	29°	2.35	1.4	0.15	34	2.21	1.8	0.22	34°	3.07	1.8	0.12
9	6	22°	1.41	1.3	0.19	23°	1.30	1.2	0.30	28°	1.32	1.4	0.37	28°	1.37	1.5	0.35
		24°	1.72	1.5	0.17	25°	1.58	1.3	0.26	30°	1.54	1.6	0.34	30°	1.81	1.7	0.26
		26°	2.08	1.6	0.14	27°	1.92	1.5	0.23	32°	1.82	1.8	0.31	32°	2.33	1.8	0.21
		28°	2.52	1.7	0.12	29°	2.32	1.6	0.20	34°	2.15	2.0	0.29	34°	2.98	2.0	0.16
10	6.5	22°	1.37	1.5	0.25	24°	1.42	1.4	0.35	29°	1.39	1.6	0.46	28	1.33	1.6	0.45
		24°	1.67	1.6	0.22	26°	1.72	1.5	0.30	31°	1.63	1.9	0.42	30	1.77	1.8	0.34
		26	2.02	1.7	0.19	28°	2.09	1.7	0.26	33°	1.93	2.1	0.39	32°	2.28	2.0	0.27
		28°	2.45	1.8	0.16	30	2.53	1.8	0.23	35	2.30	2.3	0.35	34°	2.91	2.2	0.21



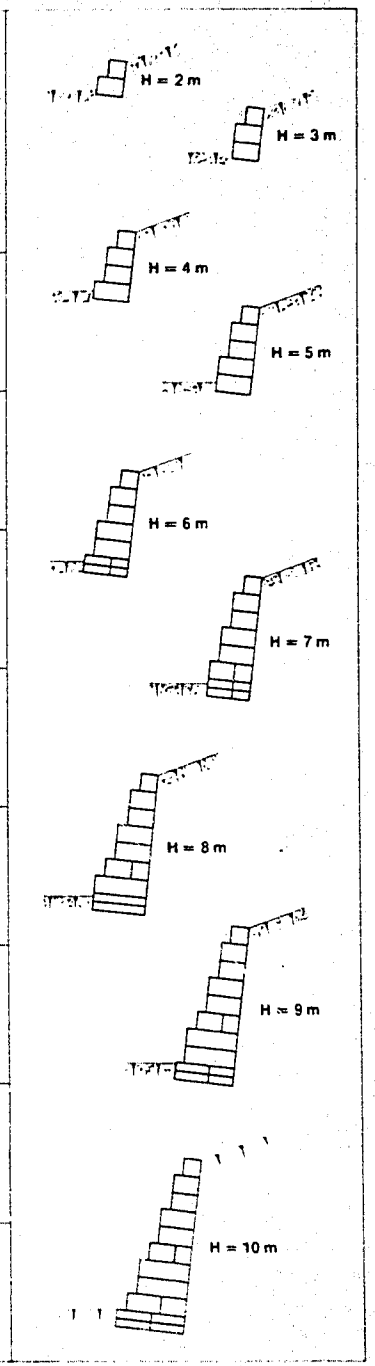
Gravity Walls with a Stepped Front-Face 1.5 < H/B < 2

H	B	$\varepsilon = 0$, $p_0 = 0$				$\varepsilon = 0$, $p_0 = \gamma_t$				$\varepsilon = \varphi$, $p_0 = 0$				$\varepsilon = 26.6°$, $p_0 = 0$			
(m)	(m)	φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (mm)	φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (mm)	φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (mm)	φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (mm)
2	1.5	18°	1.38	0.4	0.00	24°	1.34	0.3	0.02	24°	1.38	0.4	0.01	27°	1.63	0.4	0.01
		20°	1.70	0.4	0.00	26°	1.63	0.4	0.01	26°	1.62	0.4	0.01	29°	2.30	0.5	0.01
		22°	2.09	0.4	0.00	28°	1.98	0.4	0.01	28°	1.90	0.5	0.01	31°	3.02	0.5	0.00
		24°	2.55	0.5	0.00	30°	2.40	0.4	0.01	30°	2.22	0.5	0.01	33°	3.88	0.5	0.00
3	1.5	21°	1.36	0.6	0.02	26°	1.41	0.8	0.04	28°	1.40	0.7	0.03	28°	1.45	0.4	0.03
		23°	1.66	0.5	0.01	28°	1.71	0.7	0.03	30°	1.64	0.6	0.03	30°	1.94	0.6	0.02
		25°	2.02	0.6	0.01	30°	2.07	0.6	0.03	32°	1.93	0.6	0.03	32°	2.51	0.7	0.02
		27°	2.46	0.6	0.01	32°	2.52	0.6	0.02	34°	2.29	0.7	0.02	34°	3.21	0.8	0.01
4	2	22°	1.33	0.6	0.04	26°	1.37	0.8	0.07	29°	1.36	0.7	0.07	28°	1.30	0.7	0.07
		24°	1.62	0.7	0.03	28°	1.67	0.6	0.06	31°	1.60	0.7	0.06	30°	1.73	0.8	0.05
		26°	1.97	0.8	0.03	30°	2.03	0.7	0.05	33°	1.89	0.9	0.06	32°	2.24	0.9	0.04
		28°	2.40	0.9	0.02	32°	2.47	0.8	0.05	35°	2.26	1.0	0.05	34°	2.88	1.1	0.03
5	2.5	23°	1.37	0.8	0.06	26°	1.37	0.8	0.11	30°	1.39	0.8	0.12	29°	1.42	0.8	0.10
		25°	1.67	0.9	0.05	28°	1.66	0.8	0.10	32°	1.64	1.0	0.11	31°	1.86	1.0	0.08
		27°	2.03	1.0	0.05	30°	2.02	0.9	0.08	34°	1.95	1.1	0.10	33°	2.39	1.2	0.06
		29°	2.47	1.1	0.04	32°	2.47	1.0	0.07	–	–	–	–	35°	3.07	1.2	0.05
6	3	23°	1.31	0.9	0.10	26°	1.37	0.8	0.16	30	1.34	0.9	0.18	29°	1.37	0.9	0.16
		25°	1.60	1.0	0.09	28°	1.66	0.9	0.14	32	1.58	1.1	0.17	31°	1.79	1.1	0.12
		27°	1.95	1.1	0.07	30°	2.03	1.0	0.12	34	1.88	1.3	0.15	33°	2.30	1.4	0.10
		29°	2.37	1.3	0.06	32°	2.48	1.2	0.11	–	–	–	–	35°	2.96	1.4	0.08
7	3.5	24°	1.41	1.1	0.14	26°	1.37	0.9	0.21	30°	1.30	1.0	0.26	29°	1.33	1.0	0.23
		26°	1.72	1.2	0.12	28°	1.67	1.0	0.19	32°	1.54	1.2	0.24	31°	1.74	1.3	0.18
		28°	2.09	1.3	0.10	30°	2.04	1.2	0.16	34°	1.84	1.5	0.22	33°	2.25	1.5	0.14
		30°	2.55	1.5	0.09	32°	2.49	1.4	0.14	–	–	–	–	35°	2.89	1.5	0.11
8	4	24°	1.38	1.2	0.19	26°	1.38	1.0	0.28	31°	1.39	1.2	0.34	29°	1.31	1.1	0.31
		26°	1.68	1.3	0.16	28°	1.68	1.2	0.24	33°	1.66	1.5	0.31	31°	1.71	1.4	0.24
		28°	2.05	1.5	0.14	30°	2.05	1.4	0.21	35°	1.99	1.8	0.28	33°	2.21	1.7	0.20
		30°	2.50	1.6	0.12	32°	2.51	1.6	0.18	–	–	–	–	35°	2.84	1.7	0.16
9	4.5	24°	1.36	1.3	0.25	26°	1.39	1.1	0.35	31°	1.38	1.3	0.44	30°	1.48	1.4	0.36
		26°	1.66	1.5	0.21	28°	1.69	1.3	0.30	33°	1.63	1.6	0.40	32°	1.92	1.7	0.29
		28°	2.02	1.6	0.18	30°	2.06	1.5	0.27	35°	1.96	2.0	0.37	34°	2.47	2.0	0.23
		30°	2.46	1.8	0.16	32°	2.52	1.8	0.23	–	–	–	–	–	–	–	–
10	5	24°	1.34	1.4	0.31	26°	1.39	1.2	0.42	31°	1.36	1.4	0.55	30°	1.47	1.5	0.45
		26°	1.64	1.6	0.27	28°	1.70	1.5	0.37	33°	1.62	1.7	0.51	32°	1.90	1.9	0.36
		28°	2.00	1.8	0.24	30°	2.07	1.7	0.33	35	1.95	2.1	0.46	34°	2.44	2.1	0.29
		30°	2.44	2.0	0.20	32°	2.53	2.0	0.28								



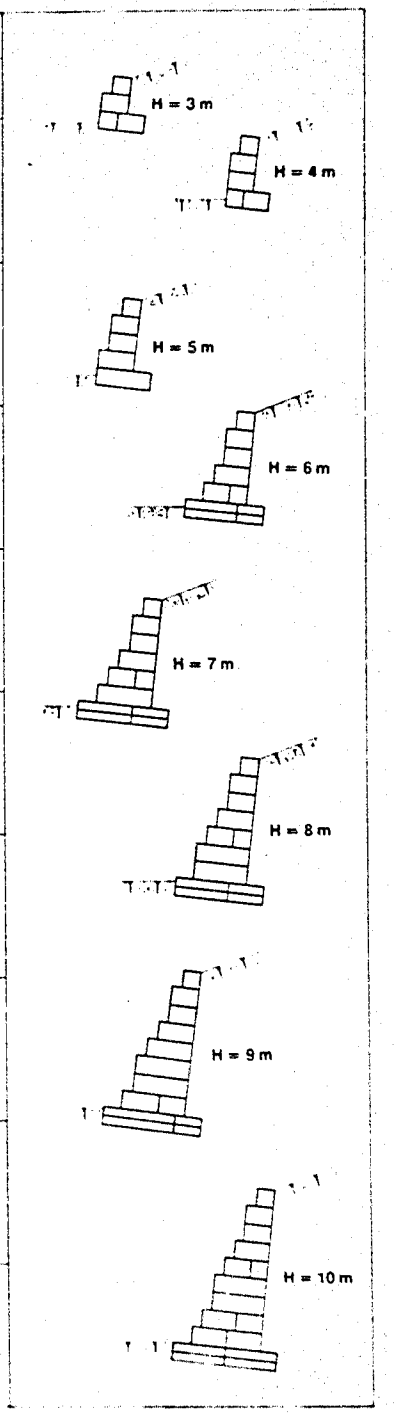
Gravity Walls with a Stepped Front-Face H/B > 2

H (m)	B (m)	$\varepsilon = 0$, $p_0 = 0$				$\varepsilon = 0$, $p_0 = \gamma_t$				$\varepsilon \geq \varphi$, $p_0 = 0$				$\varepsilon = 26.6°$, $p_0 = 0$			
		φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)
2	1.5	18°	1.38	0.4	0.00	24°	1.34	0.3	0.02	24°	1.38	0.4	0.01	27°	1.63	0.4	0.01
		20°	1.70	0.4	0.00	26°	1.63	0.4	0.01	26°	1.62	0.4	0.01	29°	2.30	0.5	0.01
		22°	2.09	0.4	0.00	28°	1.98	0.4	0.01	28°	1.90	0.5	0.01	31°	3.02	0.5	0.00
		24°	2.55	0.5	0.00	30°	2.40	0.4	0.01	30°	2.22	0.5	0.01	33°	3.88	0.5	0.00
3	1.5	21°	1.36	0.6	0.02	26°	1.41	0.8	0.04	28°	1.40	0.7	0.03	28°	1.45	0.7	0.03
		23°	1.66	0.5	0.01	28°	1.71	0.7	0.03	30°	1.64	0.6	0.03	30°	1.94	0.6	0.02
		25°	2.02	0.6	0.01	30°	2.07	0.6	0.03	32°	1.93	0.6	0.03	32°	2.51	0.7	0.02
		27°	2.46	0.6	0.01	32°	2.52	0.6	0.02	34°	2.29	0.7	0.02	34°	3.21	0.8	0.01
4	2	22°	1.33	0.6	0.04	26°	1.37	0.8	0.07	29°	1.36	0.7	0.07	28°	1.30	0.7	0.07
		24°	1.62	0.7	0.03	28°	1.67	0.6	0.06	31°	1.60	0.7	0.06	30°	1.73	0.8	0.05
		26°	1.97	0.8	0.03	30°	2.03	0.7	0.05	33°	1.89	0.9	0.06	32°	2.24	0.9	0.04
		28°	2.40	0.9	0.02	32°	2.47	0.8	0.05	35°	2.26	1.0	0.05	34°	2.88	1.1	0.03
5	2	24°	1.40	1.0	0.06	27°	1.40	1.4	0.11	31°	1.40	1.3	0.12	29°	1.32	1.3	0.11
		26°	1.71	0.8	0.05	29°	1.70	1.1	0.10	33°	1.66	1.0	0.11	31°	1.72	0.9	0.08
		28°	2.07	0.9	0.04	31°	2.06	0.9	0.08	35°	1.98	1.0	0.10	33°	2.21	1.0	0.07
		30°	2.52	1.0	0.04	33°	2.52	0.9	0.07	–	–	–	–	35°	2.84	1.3	0.05
6	2.5	24°	1.32	0.9	0.11	27°	1.38	1.2	0.17	31°	1.34	1.3	0.19	30°	1.44	1.1	0.16
		26°	1.61	0.9	0.09	29°	1.68	1.0	0.15	33°	1.58	0.9	0.18	32°	1.86	1.1	0.13
		28°	1.96	1.1	0.08	31°	2.04	1.0	0.13	35°	1.90	1.2	0.16	34°	2.39	1.4	0.10
		30°	2.39	1.3	0.07	33°	2.50	1.2	0.11	–	–	–	–	–	–	–	–
7	2.5	25°	1.34	1.5	0.15	27°	1.31	2.1	0.23	32°	1.35	2.1	0.28	30°	1.33	2.0	0.24
		27°	1.63	1.2	0.13	29°	1.59	1.7	0.20	34°	1.60	1.6	0.26	32°	1.72	1.3	0.19
		29°	1.98	1.1	0.11	31°	1.94	1.3	0.18	–	–	–	–	34°	2.20	1.3	0.15
		31°	2.42	1.3	0.09	33°	2.37	1.1	0.15	–	–	–	–	–	–	–	–
8	3	26°	1.43	1.2	0.21	27°	1.30	1.9	0.32	32°	1.31	2.0	0.40	31°	1.47	1.5	0.31
		28°	1.74	1.2	0.18	29°	1.58	1.5	0.28	34°	1.57	1.4	0.37	33°	1.89	1.3	0.25
		30°	2.12	1.4	0.15	31°	1.93	1.1	0.24	–	–	–	–	35°	2.43	1.8	0.20
		32°	2.60	1.7	0.13	33°	2.37	1.4	0.21	–	–	–	–	–	–	–	–
9	3.5	26°	1.40	1.2	0.28	27°	1.30	1.8	0.41	33°	1.41	1.7	0.51	31°	1.45	1.5	0.41
		28°	1.71	1.3	0.24	29°	1.59	1.3	0.36	35°	1.70	1.4	0.47	33°	1.86	1.5	0.33
		30°	2.09	1.6	0.21	31°	1.94	1.3	0.31	–	–	–	–	35°	2.40	2.0	0.27
		32°	2.55	1.9	0.18	33°	2.38	1.7	0.27	–	–	–	–	–	–	–	–
10	4	26°	1.39	1.2	0.36	27°	1.31	1.7	0.50	33°	1.40	1.7	0.65	31°	1.44	1.5	0.52
		28°	1.69	1.4	0.31	29°	1.60	1.3	0.44	35°	1.69	1.6	0.60	33°	1.85	1.7	0.42
		30°	2.07	1.7	0.27	31°	1.95	1.5	0.39	–	–	–	–	35°	2.38	2.2	0.35
		32°	2.53	2.0	0.23	33°	1.39	1.9	0.34	–	–	–	–	–	–	–	–



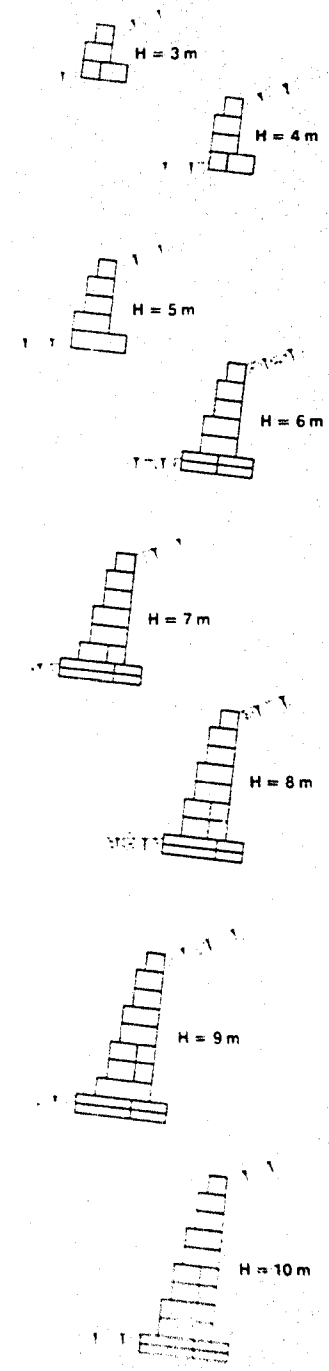
Semi-gravity Walls with a Stepped Front-Face H/b < 2.5

H	B	b	$\varepsilon = 0$, $p_0 = 0$					$\varepsilon = 0$, $p_0 = \gamma_t$					$\varepsilon \simeq \varphi$, $p_0 = 0$					$\varepsilon = 26.6°$, $p_0 = 0$				
(m)	(m)	(m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)
—	—	—	—					—					—					—				
3	2.5	1.5	17°	1.31	0.5	0.3	0.02	21°	1.35	0.6	0.4	0.04	25°	1.31	0.6	0.4	0.03	27°	1.39	0.7	0.4	0.03
			19°	1.62	0.6	0.3	0.01	23°	1.64	0.7	0.4	0.04	27°	1.49	0.7	0.4	0.02	29°	1.92	0.7	0.4	0.02
			21°	1.97	0.6	0.4	0.01	25°	1.98	0.7	0.3	0.03	29°	1.70	0.7	0.4	0.02	31°	2.47	0.8	0.4	0.01
			23°	2.39	0.6	0.4	0.01	27°	2.39	0.8	0.3	0.03	31°	1.95	0.8	0.4	0.02	33°	3.13	0.8	0.4	0.01
4	2.5	1.5	20°	1.38	0.8	0.5	0.04	23°	1.37	1.0	0.8	0.08	28°	1.35	1.0	0.7	0.07	28°	1.41	1.0	0.6	0.06
			22°	1.69	0.7	0.5	0.03	25°	1.65	0.9	0.7	0.07	30°	1.55	1.0	0.6	0.06	30°	1.84	0.8	0.6	0.05
			24°	2.04	0.7	0.5	0.03	27°	1.99	0.8	0.7	0.06	32°	1.79	0.9	0.6	0.05	32°	2.34	0.9	0.6	0.03
			26°	2.46	0.8	0.5	0.02	29°	2.39	0.7	0.6	0.05	34°	2.07	0.9	0.6	0.05	34°	2.95	1.0	0.6	0.03
5	3	2	21°	1.38	0.8	0.6	0.07	24°	1.42	1.0	0.8	0.12	29°	1.38	1.1	0.7	0.12	28°	1.33	1.1	0.7	0.11
			23°	1.67	0.8	0.6	0.06	26°	1.71	0.9	0.7	0.11	31°	1.59	1.0	0.7	0.11	30°	1.74	1.0	0.7	0.09
			25°	2.02	0.9	0.6	0.05	28°	2.05	0.9	0.7	0.09	33°	1.84	1.1	0.7	0.10	32°	2.22	1.1	0.7	0.07
			27°	2.43	1.0	0.7	0.04	30°	2.47	1.0	0.6	0.08	35°	2.15	1.2	0.7	0.09	34°	2.80	1.2	0.7	0.05
6	4.5	2.5	21°	1.40	1.1	0.7	0.11	23°	1.35	1.1	0.9	0.18	28°	1.34	1.2	0.9	0.18	28°	1.39	1.2	0.8	0.17
			23°	1.70	1.2	0.7	0.10	25°	1.63	1.2	0.8	0.16	30°	1.55	1.3	0.8	0.17	30°	1.83	1.3	0.8	0.13
			25°	2.06	1.2	0.7	0.08	27°	1.97	1.2	0.8	0.14	32°	1.81	1.4	0.8	0.1[illegible]	32°	2.34	1.4	0.8	0.10
			27°	2.48	1.3	0.8	0.07	29°	2.37	1.3	0.7	0.12	34°	2.11	1.6	0.8	0.14	34°	2.96	1.5	0.8	0.08
7	5	3	21°	1.30	1.2	0.8	0.16	24°	1.42	1.2	0.9	0.24	29°	1.38	1.4	1.0	0.26	28°	1.33	1.4	1.0	0.26
			23°	1.58	1.3	0.8	0.14	26°	1.71	1.3	0.9	0.21	31°	1.61	1.5	0.9	0.24	30°	1.75	1.5	0.9	0.20
			25°	1.92	1.3	0.8	0.12	28°	2.07	1.4	0.8	0.18	33°	1.88	1.7	0.9	0.22	32°	2.23	1.6	0.9	0.16
			27°	2.31	1.4	0.9	0.11	30°	2.49	1.5	0.8	0.16	35°	2.22	1.8	1.0	0.20	34°	2.84	1.6	1.0	0.12
8	5	3	22°	1.30	1.2	1.2	0.22	24°	1.31	1.2	1.4	0.32	30°	1.38	1.5	1.5	0.35	[illegible]	[illegible]	1.4	1.4	0.32
			24°	1.57	1.3	1.1	0.20	26°	1.58	1.3	1.3	0.29	32°	1.62	1.6	1.4	0.33	[illegible]	[illegible]	1.6	1.2	0.25
			26°	1.90	1.4	1.0	0.17	28°	1.90	1.4	1.2	0.25	34°	1.90	1.9	1.4	0.31	[illegible]	[illegible]	1.8	1.1	0.20
			28°	2.29	1.5	1.0	0.15	30°	2.30	1.5	1.1	0.22	—	—	—	—	—	[illegible]	[illegible]	2.0	1.1	0.16
9	5.5	3.5	23°	1.36	1.4	1.2	0.28	25°	1.39	1.3	1.4	0.39	30°	1.34	1.6	1.6	0.49	29°	1.37	1.5	1.5	0.43
			25°	1.65	1.5	1.1	0.25	27°	1.68	1.4	1.3	0.35	32°	1.57	1.8	1.5	0.45	31°	1.77	1.8	1.3	0.34
			27°	1.99	1.6	1.1	0.21	29°	2.03	1.6	1.2	0.30	34°	1.85	2.0	1.4	0.41	33°	2.25	2.0	1.2	0.28
			29°	2.40	1.7	1.1	0.18	31°	2.45	1.7	1.1	0.26	—	—	—	—	—	35°	2.86	2.0	1.2	0.22
10	6	4	23°	1.31	1.4	1.2	0.37	25°	1.36	1.3	1.4	0.49	30°	1.31	1.7	1.7	0.63	29°	1.34	1.6	1.6	0.56
			25°	1.58	1.6	1.2	0.32	27°	1.64	1.5	1.3	0.43	32°	1.54	1.9	1.6	0.58	31°	1.73	1.9	1.4	0.44
			27°	1.91	1.7	1.2	0.28	29°	1.88	1.7	1.2	0.38	34°	1.82	2.2	1.5	0.53	33°	2.19	2.1	1.3	0.36
			29°	2.31	1.8	1.2	0.24	31°	2.39	1.9	1.2	0.33	—	—	—	—	—	35°	2.79	2.1	1.3	0.29



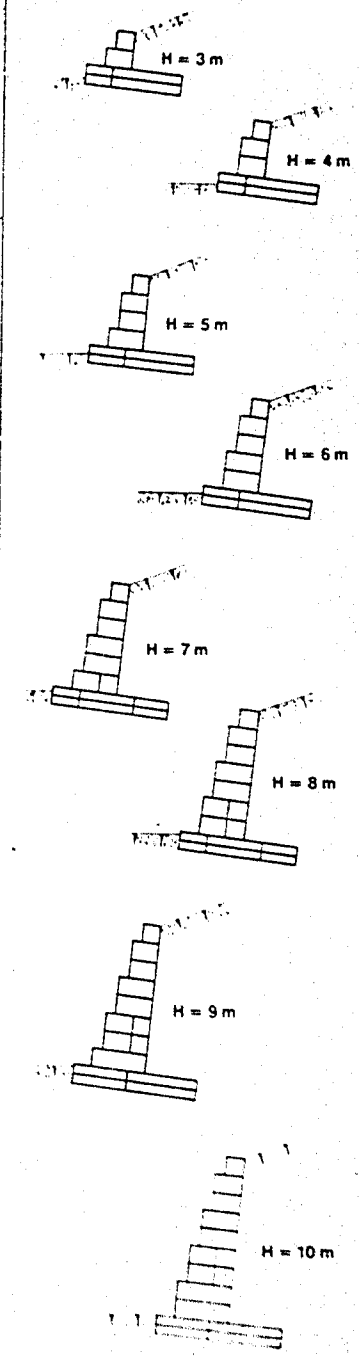
Semi-gravity Walls with a Stepped Front-Face H/b > 2.5

H	B	b	$\varepsilon = 0$, $p_0 = 0$					$\varepsilon = 0$, $p_0 = \gamma_t$					$\varepsilon = \varphi$, $p_0 = 0$					$\varepsilon = 26.6°$, $p_0 = 0$				
(m)	(m)	(m)	φ (°)	η_s (-)	σ_t (kp/cm²)	σ_m (kp/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kp/cm²)	σ_m (kp/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kp/cm²)	σ_m (kp/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kp/cm²)	σ_m (kp/cm²)	Δ (m)
3	2.5	1.5	17°	1.31	0.5	0.3	0.02	21°	1.36	0.6	0.4	0.04	25°	1.31	0.6	0.4	0.03	27°	1.39	0.7	0.4	0.03
			19°	1.62	0.6	0.3	0.01	23°	1.64	0.7	0.4	0.04	27°	1.49	0.7	0.4	0.02	29°	1.92	0.7	0.4	0.02
			21°	1.97	0.6	0.4	0.01	25°	1.98	0.7	0.3	0.03	29°	1.70	0.7	0.4	0.02	31°	2.47	0.8	0.4	0.01
			23°	2.39	0.6	0.4	0.01	27°	2.39	0.8	0.3	0.03	31°	1.95	0.8	0.4	0.02	33°	3.13	0.8	0.4	0.01
4	2.5	1.5	20°	1.38	0.8	0.6	0.04	23°	1.37	1.0	0.8	0.08	28°	1.35	1.0	0.7	0.07	28°	1.41	1.0	0.6	0.06
			22°	1.69	0.7	0.5	0.03	25°	1.65	0.9	0.7	0.07	30°	1.55	1.0	0.6	0.06	30°	1.84	0.8	0.6	0.05
			24°	2.04	0.7	0.5	0.03	27°	1.99	0.8	0.7	0.06	32°	1.79	0.8	0.6	0.05	32°	2.34	0.9	0.6	0.03
			26°	2.46	0.8	0.5	0.02	29°	2.39	0.8	0.6	0.05	34°	2.07	0.9	0.6	0.05	34°	2.95	1.0	0.6	0.03
5	3	2	21°	1.38	0.8	0.6	0.07	24°	1.42	1.1	0.8	0.12	29°	1.38	1.1	0.7	0.12	28°	1.33	1.1	0.7	0.11
			23°	1.67	0.8	0.6	0.06	26°	1.71	0.9	0.7	0.11	31°	1.59	0.9	0.7	0.11	30°	1.74	1.0	0.7	0.09
			25°	2.02	0.9	0.6	0.05	28°	2.05	0.9	0.7	0.09	33°	1.84	1.1	0.7	0.10	32°	2.22	1.1	0.7	0.07
			27°	2.43	1.0	0.7	0.04	30°	2.47	1.0	0.6	0.08	35°	2.15	1.2	0.7	0.09	34°	2.80	1.2	0.7	0.05
6	4	2	21°	1.31	1.0	1.0	0.12	24°	1.39	1.0	1.4	0.19	29°	1.36	1.2	1.3	0.20	28°	1.31	1.1	1.4	0.19
			23°	1.59	1.1	0.9	0.11	26°	1.68	1.1	1.2	0.17	31°	1.57	1.3	1.2	0.18	30°	1.72	1.3	1.1	0.15
			25°	1.92	1.2	0.9	0.09	28°	2.02	1.2	1.1	0.15	33°	1.83	1.4	1.2	0.17	32°	2.19	1.4	1.0	0.12
			27°	2.32	1.2	0.8	0.08	30°	2.44	1.3	1.0	0.13	35°	2.16	1.6	1.1	0.15	34°	2.77	1.5	0.9	0.09
7	4.5	2.5	22°	1.33	1.2	0.7	0.18	24°	1.32	1.1	0.8	0.27	29°	1.30	1.3	0.9	0.30	29°	1.44	1.3	0.8	0.25
			24°	1.62	1.2	0.8	0.15	26°	1.59	1.2	0.8	0.24	31°	1.51	1.4	0.9	0.28	31°	1.85	1.5	0.9	0.20
			26°	1.96	1.3	0.8	0.13	28°	1.92	1.3	0.8	0.21	33°	1.76	1.6	0.9	0.25	33°	2.35	1.6	0.9	0.16
			28°	2.36	1.4	0.9	0.11	30°	2.32	1.4	0.8	0.18	35°	2.08	1.8	0.9	0.23	35°	2.98	1.6	1.0	0.13
8	4.5	2.5	23°	1.32	1.2	1.5	0.25	25°	1.33	1.1	2.1	0.35	31°	1.40	1.4	2.2	0.41	29°	1.32	1.2	2.1	0.38
			25°	1.60	1.3	1.3	0.22	27°	1.61	1.2	1.8	0.31	33°	1.63	1.6	1.9	0.38	31°	1.70	1.5	1.6	0.30
			27°	1.93	1.4	1.2	0.19	29°	1.94	1.4	1.5	0.27	35	1.93	1.9	1.8	0.34	33°	2.16	1.7	1.4	0.24
			29°	2.33	1.5	1.1	0.16	31°	2.34	1.5	1.4	0.24	–	–	–	–	–	35°	2.73	1.9	1.3	0.19
9	5	3	24°	1.38	1.3	1.4	0.31	26°	1.42	1.2	1.7	0.43	31°	1.36	1.5	2.1	0.55	30°	1.46	1.5	1.8	0.45
			26°	1.67	1.4	1.3	0.27	28°	1.71	1.4	1.5	0.38	33°	1.59	1.7	1.9	0.51	32°	1.86	1.7	1.5	0.36
			28°	2.02	1.6	1.2	0.24	30°	2.06	1.6	1.4	0.33	35°	1.89	2.0	1.7	0.46	34°	2.35	2.0	1.3	0.30
			30°	2.44	1.7	1.1	0.21	32°	2.50	1.7	1.3	0.29	–	–	–	–	–	–	–	–	–	–
10	5	3	25°	1.40	1.3	1.9	0.40	26°	1.33	1.5	2.7	0.56	32°	1.39	1.5	3.2	0.71	30°	1.37	1.5	2.8	0.61
			27°	1.70	1.5	1.7	0.35	28°	1.60	1.3	2.2	0.49	34°	1.64	1.8	2.8	0.65	32°	1.75	1.7	2.2	0.49
			29°	2.05	1.7	1.6	0.30	30°	1.94	1.5	1.9	0.43	–	–	–	–	–	34°	2.21	2.0	1.8	0.40
			31°	2.48	1.9	1.4	0.26	32°	2.34	1.7	1.7	0.37	–	–	–	–	–	–	–	–	–	–



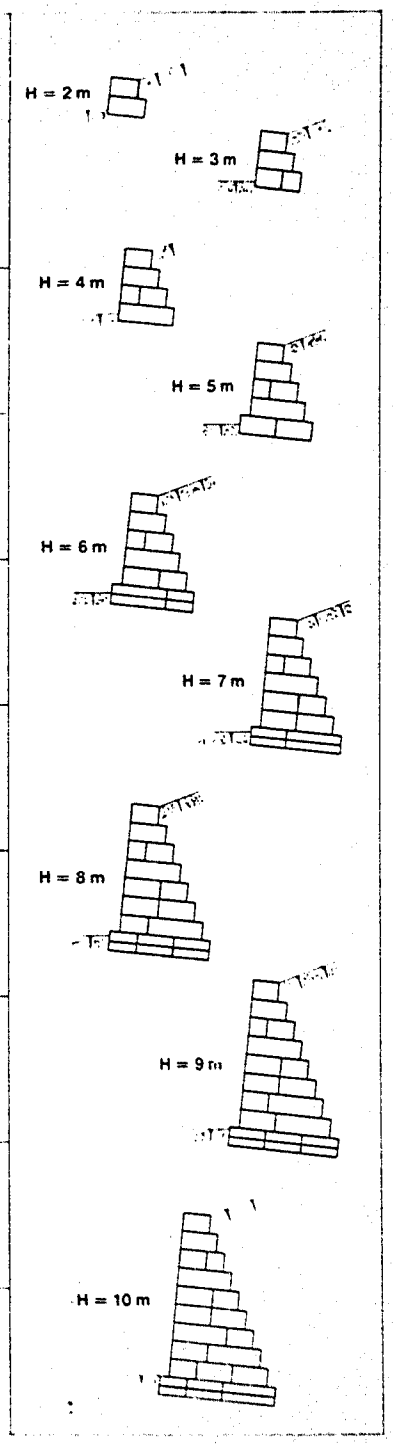
Semi-gravity Walls with a Stepped Front-Face and Extended Foundation

H	B	b	$\varepsilon = 0$, $p_0 = 0$					$\varepsilon = 0$, $p_0 = \gamma_t$					$\varepsilon \geq \varphi$, $p_0 = 0$					$\varepsilon = 26.6°$, $p_0 = 0$				
(m)	(m)	(m)	φ (°)	η_s (-)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)
3	5.5	1.5	14°	1.88	0.7	0.3	0.01	14°	1.33	0.7	0.5	0.05	16°	1.40	0.7	0.3	0.03	27°	2.77	1.0	0.4	0.02
			16°	2.35	0.7	0.3	0.01	16°	1.67	0.8	0.4	0.04	18°	1.62	0.7	0.3	0.02	29°	3.87	0.9	0.4	0.01
			18°	2.89	0.7	0.3	0.01	18°	2.06	0.8	0.4	0.04	20°	1.87	0.8	0.3	0.02	31°	5.01	0.9	0.4	0.01
			20°	3.54	0.8	0.3	0.01	20°	2.52	0.9	0.4	0.03	22°	2.15	0.8	0.3	0.02	33°	6.34	0.9	0.4	0.01
4	5.5	1.5	14°	1.30	0.8	0.7	0.05	16°	1.32	0.8	1.4	0.10	19°	1.35	0.8	0.8	0.07	27°	2.20	1.1	0.7	0.06
			16°	1.73	0.9	0.6	0.04	18°	1.63	0.9	1.1	0.09	21°	1.56	0.9	0.8	0.07	29°	3.05	1.2	0.6	0.04
			18°	2.14	0.9	0.6	0.04	20°	2.00	1.0	1.0	0.08	23°	1.79	1.0	0.7	0.06	31°	3.94	1.2	0.6	0.03
			20°	2.61	1.0	0.6	0.03	22°	2.43	1.0	0.8	0.07	25°	2.06	1.1	0.7	0.06	33°	4.99	1.1	0.6	0.02
5	6	2	15°	1.30	0.9	0.7	0.09	18°	1.42	0.9	1.1	0.16	21°	1.35	1.0	0.9	0.14	27°	1.92	1.3	0.8	0.12
			17°	1.62	1.0	0.7	0.08	20°	1.75	1.0	0.9	0.14	23°	1.56	1.1	0.8	0.13	29°	2.67	1.4	0.7	0.09
			19°	1.99	1.0	0.6	0.07	22°	2.12	1.1	0.8	0.13	25°	1.79	1.2	0.8	0.12	31°	3.44	1.4	0.7	0.07
			21°	2.42	1.1	0.6	0.06	24°	2.56	1.2	0.8	0.11	27°	2.06	1.3	0.8	0.11	33°	4.27	1.3	0.7	0.05
6	6	2	17°	1.35	1.0	1.3	0.15	19°	1.37	0.9	2.5	0.25	23°	1.34	1.0	1.9	0.24	27°	1.67	1.3	1.6	0.23
			19°	1.67	1.1	1.1	0.13	21°	1.67	1.0	1.9	0.22	25°	1.55	1.1	1.6	0.22	29°	2.27	1.4	1.2	0.16
			21°	2.03	1.2	1.0	0.12	23°	2.02	1.2	1.5	0.20	27°	1.79	1.3	1.5	0.21	31°	2.86	1.6	1.0	0.13
			23°	2.47	1.3	0.9	0.10	25°	2.44	1.3	1.3	0.17	29°	2.03	1.4	1.3	0.19	33°	3.53	1.6	0.9	0.10
7	6.5	2.5	18°	1.35	1.1	1.2	0.22	20°	1.39	1.0	1.8	0.33	24°	1.32	1.1	1.7	0.35	27°	1.49	1.3	1.7	0.35
			20°	1.66	1.2	1.1	0.20	22°	1.69	1.1	1.5	0.30	26°	1.51	1.2	1.6	0.33	29°	2.01	1.5	1.2	0.25
			22°	2.01	1.3	1.0	0.17	24°	2.04	1.3	1.3	0.26	28°	1.70	1.4	1.4	0.30	31°	2.54	1.7	1.0	0.20
			24°	2.44	1.4	0.9	0.15	26°	2.39	1.4	1.2	0.23	30°	1.92	1.5	1.3	0.28	33°	3.16	1.8	0.9	0.16
8	6.5	2.5	19°	1.33	1.0	2.0	0.31	21°	1.39	0.9	3.5	0.44	26°	1.32	1.4	3.2	0.48	27°	1.32	1.5	3.4	0.51
			21°	1.62	1.2	1.7	0.28	23°	1.65	1.1	2.5	0.39	28°	1.50	1.2	2.7	0.45	29°	1.78	1.3	2.1	0.37
			23°	1.97	1.3	1.5	0.24	25°	1.93	1.2	2.1	0.35	30°	1.70	1.4	2.3	0.42	31°	2.24	1.5	1.6	0.29
			25°	2.34	1.5	1.3	0.21	27°	2.26	1.4	1.8	0.31	32°	1.94	1.6	2.0	0.39	33°	2.79	1.8	1.4	0.24
9	7	3	20°	1.36	1.1	1.8	0.40	22°	1.39	1.3	2.5	0.55	27°	1.30	1.5	2.7	0.64	28°	1.44	1.3	2.3	0.58
			22°	1.65	1.3	1.5	0.36	24°	1.64	1.2	2.0	0.48	29°	1.48	1.4	2.3	0.59	30°	1.86	1.6	1.8	0.45
			24°	1.95	1.4	1.4	0.31	26°	1.92	1.3	1.7	0.43	31°	1.69	1.6	2.1	0.55	32°	2.33	1.9	1.5	0.36
			26°	2.29	1.6	1.3	0.27	28°	2.27	1.5	1.5	0.38	33°	1.95	1.9	1.9	0.50	34°	2.90	2.1	1.3	0.29
10	7	3	21°	1.35	1.5	2.7	0.51	23°	1.36	1.8	4.0	0.67	29°	1.36	1.9	4.2	0.79	28°	1.32	2.0	4.3	0.78
			23°	1.60	1.2	2.2	0.45	25°	1.61	1.5	3.0	0.59	31°	1.56	1.6	3.5	0.74	30°	1.70	1.5	2.8	0.61
			25°	1.89	1.4	1.9	0.40	27°	1.90	1.3	2.4	0.52	33°	1.80	1.7	3.0	0.68	32°	2.13	1.8	2.2	0.49
			27°	2.22	1.6	1.7	0.35	29°	2.24	1.5	2.1	0.46	35°	2.10	2.2	2.6	0.62	34°	2.66	2.2	1.8	0.40



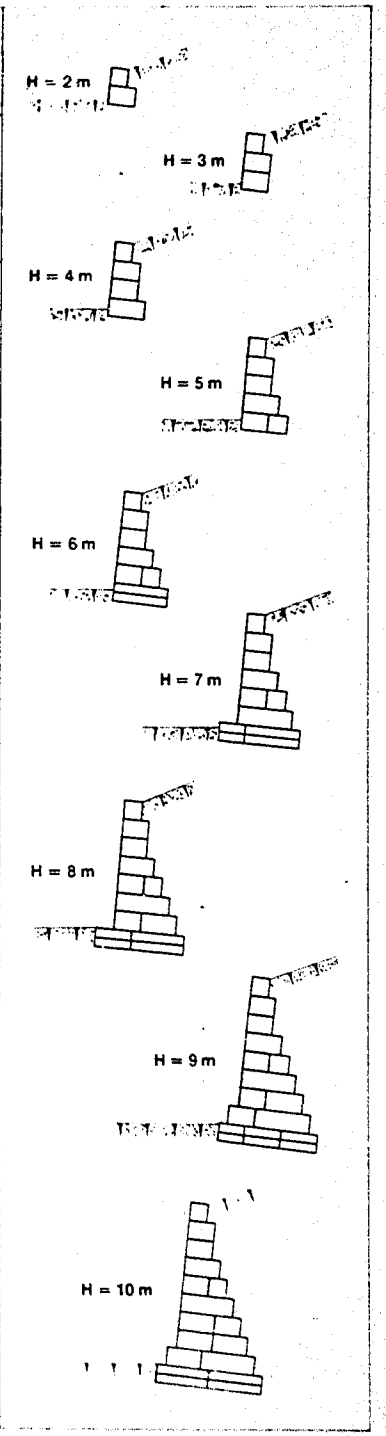
Gravity Walls with a Stepped Rear-Face H/B < 1.5

H (m)	B (m)	$\varepsilon = 0$, $p_0 = 0$				$\varepsilon = 0$, $p_0 = \gamma_t$				$\varepsilon > \varphi$, $p_0 = 0$				$\varepsilon = 26.6°$, $p_0 = 0$			
		φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)
2	2	16°	1.37	0.4	0.00	22°	1.36	0.5	0.02	21°	1.30	0.4	0.01	27°	1.84	0.4	0.01
		18°	1.68	0.4	0.00	24°	1.63	0.4	0.01	23°	1.50	0.4	0.01	29°	2.52	0.4	0.01
		20°	2.04	0.4	0.00	26°	1.95	0.4	0.01	25°	1.72	0.4	0.01	31°	3.22	0.4	0.00
		22°	2.47	0.3	0.00	28°	2.33	0.4	0.01	27°	1.97	0.4	0.01	33°	4.04	0.4	0.00
3	2.5	18°	1.31	0.7	0.01	23°	1.38	0.8	0.04	25°	1.36	0.7	0.03	27°	1.47	0.7	0.03
		20°	1.58	0.6	0.01	25°	1.65	0.8	0.03	27°	1.56	0.7	0.03	29°	1.57	0.6	0.02
		22°	1.91	0.6	0.01	27°	1.96	0.7	0.03	29°	1.79	0.6	0.03	31°	2.49	0.6	0.02
		24°	2.28	0.6	0.01	29°	2.33	0.7	0.02	31°	2.05	0.6	0.03	33°	3.10	0.6	0.01
4	3	20°	1.38	1.0	0.03	23°	1.33	1.2	0.07	26°	1.30	1.1	0.07	27°	1.31	1.1	0.08
		22°	1.65	0.9	0.03	25°	1.58	1.1	0.06	28°	1.48	1.1	0.07	29°	1.74	0.9	0.05
		24°	1.98	0.9	0.02	27°	1.88	1.0	0.05	30°	1.70	1.0	0.06	31°	2.19	0.8	0.04
		26°	2.35	0.8	0.02	29°	2.23	1.0	0.05	32°	1.95	0.9	0.06	33°	2.71	0.8	0.03
5	4	20°	1.32	0.9	0.06	23°	1.35	1.0	0.11	27°	1.35	1.0	0.12	28°	1.46	1.0	0.11
		22°	1.59	0.8	0.05	25°	1.61	1.0	0.10	29°	1.54	1.1	0.12	30°	1.89	1.0	0.09
		24°	1.90	0.8	0.05	27°	1.91	0.9	0.09	31°	1.77	1.2	0.11	32°	2.36	1.1	0.07
		26°	2.26	0.7	0.04	29°	2.27	0.8	0.07	33°	2.04	1.3	0.11	34°	3.92	1.1	0.06
6	4.5	21°	1.35	1.1	0.09	23°	1.31	1.4	0.16	28°	1.36	1.3	0.20	28°	1.40	1.3	0.19
		23°	1.62	1.1	0.08	25°	1.56	1.3	0.14	30°	1.56	1.2	0.19	30°	1.78	1.1	0.15
		25°	1.93	1.0	0.07	27°	1.85	1.2	0.13	32°	1.79	1.4	0.18	32°	2.21	1.2	0.12
		27°	2.29	0.9	0.06	29°	2.20	1.1	0.11	34°	2.07	1.5	0.17	34°	2.73	1.3	0.10
7	5	22°	1.40	1.4	0.14	24°	1.40	1.6	0.21	28°	1.31	1.7	0.30	28°	1.35	1.7	0.29
		24°	1.67	1.3	0.12	26°	1.66	1.5	0.19	30°	1.50	1.6	0.29	30°	1.71	1.4	0.22
		26°	1.99	1.2	0.11	28°	1.97	1.4	0.17	32°	1.73	1.5	0.28	32°	2.11	1.3	0.18
		28°	2.37	1.1	0.09	30°	2.34	1.3	0.15	34°	1.99	1.7	0.26	34°	2.61	1.4	0.15
8	5.5	22°	1.35	1.7	0.20	24°	1.37	2.0	0.29	29°	1.36	2.1	0.42	28°	1.31	2.1	0.40
		24°	1.61	1.6	0.17	26°	1.63	1.8	0.26	31°	1.56	1.9	0.40	30°	1.65	1.8	0.32
		26°	1.91	1.5	0.15	28°	1.94	1.7	0.23	33°	1.80	1.7	0.38	32°	2.04	1.5	0.26
		28°	2.27	1.4	0.13	30°	2.30	1.5	0.20	35°	2.09	2.0	0.36	34°	2.52	1.5	0.22
9	6	22°	1.30	2.0	0.27	24°	1.35	2.3	0.37	29°	1.33	2.6	0.56	29°	1.44	2.4	0.48
		24°	1.56	1.9	0.24	26°	1.61	2.1	0.33	31°	1.53	2.4	0.54	31°	1.79	2.0	0.39
		26°	1.85	1.8	0.21	28°	1.91	2.0	0.30	33°	1.76	2.1	0.52	33°	2.21	1.6	0.32
		28°	2.20	1.6	0.18	30°	2.27	1.8	0.27	35°	2.04	2.2	0.49	35°	2.72	1.8	0.27
10	6.5	23°	1.39	2.3	0.33	24°	1.34	2.6	0.47	29°	1.31	3.1	0.73	29°	1.41	2.8	0.62
		25°	1.65	2.1	0.29	26°	1.59	2.4	0.42	31°	1.50	2.8	0.70	31°	1.76	2.4	0.51
		27°	1.97	2.0	0.26	28°	1.89	2.3	0.37	33°	1.73	2.5	0.67	33°	2.16	1.9	0.42
		29°	2.33	1.8	0.23	30°	2.24	2.0	0.33	35°	2.01	2.1	0.64	35°	2.66	1.9	0.35



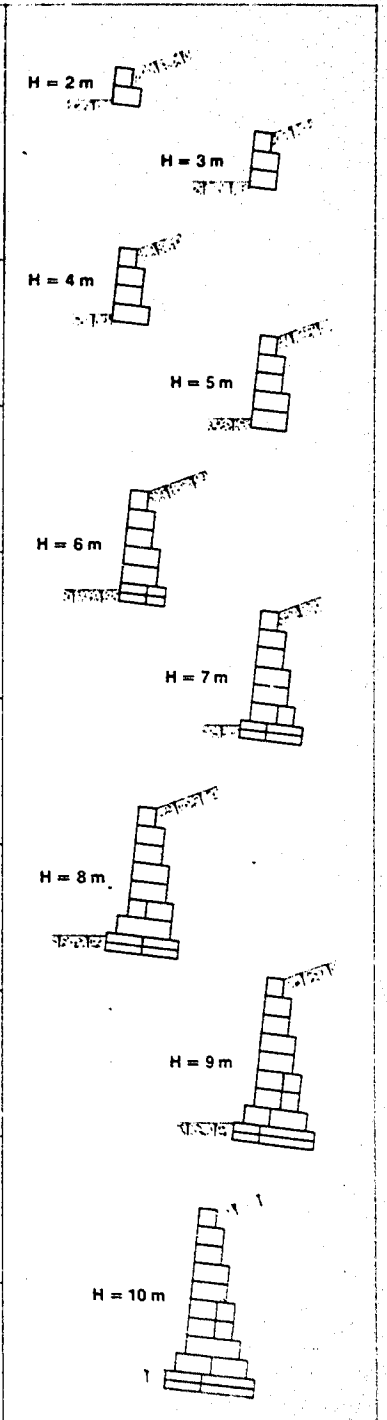
Gravity Walls with a Stepped Rear-Face 1.5 < H/B < 2

H (m)	B (m)	$\varepsilon = 0$, $p_0 = 0$				$\varepsilon = 0$, $p_0 = \gamma_t$				$\varepsilon \simeq \varphi$, $p_0 = 0$				$\varepsilon = 26.6°$, $p_0 = 0$			
		φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	Δ (m)
2	1.5	19°	1.38	0.4	0.01	25°	1.36	0.6	0.02	25°	1.32	0.5	0.01	27°	1.42	0.5	0.01
		21°	1.67	0.4	0.01	27°	1.62	0.6	0.02	27°	1.51	0.5	0.01	29°	1.92	0.4	0.01
		23°	2.00	0.4	0.00	29°	1.93	0.5	0.02	29°	1.76	0.4	0.01	31°	2.44	0.4	0.01
		25°	2.40	0.4	0.00	31°	2.30	0.5	0.01	31°	2.00	0.4	0.01	33°	3.06	0.4	0.01
3	1.5	22°	1.39	0.8	0.02	26°	1.32	1.2	0.05	29°	1.38	1.0	0.04	28°	1.33	1.0	0.04
		24°	1.67	0.8	0.02	28°	1.58	1.1	0.04	31°	1.60	0.9	0.04	30°	1.72	0.8	0.03
		26°	2.01	0.7	0.01	30°	1.89	1.0	0.04	33°	1.85	0.8	0.03	32°	2.17	0.7	0.02
		28°	2.41	0.6	0.01	32°	2.26	0.8	0.03	35°	2.17	0.6	0.03	34°	2.72	0.6	0.02
4	2	23°	1.33	1.3	0.05	27°	1.40	1.6	0.09	30°	1.32	1.6	0.10	29°	1.35	1.5	0.08
		25°	1.59	1.2	0.04	29°	1.66	1.5	0.08	32°	1.53	1.5	0.09	31°	1.70	1.3	0.07
		27°	1.90	1.1	0.04	31°	1.98	1.3	0.07	34°	1.78	1.3	0.08	33°	2.11	1.0	0.05
		29°	2.27	1.0	0.03	33°	2.37	1.1	0.06	–	–	–	–	35°	2.63	0.8	0.04
5	2.5	24°	1.36	1.6	0.08	27°	1.39	2.0	0.14	31°	1.35	2.1	0.17	30°	1.43	1.9	0.14
		26°	1.63	1.5	0.07	29°	1.65	1.8	0.13	33°	1.57	1.9	0.16	32°	1.78	1.6	0.11
		28°	1.94	1.4	0.06	31°	1.96	1.6	0.11	35°	1.82	1.6	0.15	34°	2.20	1.3	0.09
		30°	2.31	1.2	0.06	33°	2.35	1.4	0.10	–	–	–	–	–	–	–	–
6	3	24°	1.31	2.0	0.13	27°	1.39	2.4	0.20	31°	1.32	2.7	0.28	30°	1.39	2.4	0.22
		26°	1.56	1.8	0.12	29°	1.65	2.2	0.18	33°	1.53	2.4	0.26	32°	1.72	2.0	0.18
		28°	1.86	1.7	0.10	31°	1.97	1.9	0.16	35°	1.78	2.1	0.25	34°	2.13	1.6	0.15
		30°	2.22	1.5	0.09	33°	2.35	1.7	0.15	–	–	–	–	–	–	–	–
7	4.5	24°	1.39	1.1	0.19	26°	1.39	1.4	0.29	30°	1.30	1.5	0.40	29°	1.31	1.4	0.35
		26°	1.66	1.0	0.17	28°	1.65	1.2	0.26	32°	1.50	1.3	0.39	31°	1.63	1.2	0.29
		28°	1.97	0.9	0.15	30°	1.96	1.1	0.23	34°	1.74	1.1	0.37	33°	2.02	1.2	0.24
		30°	2.35	0.8	0.13	32°	2.34	0.9	0.21	–	–	–	–	35°	2.51	1.4	0.20
8	5	24°	1.35	1.4	0.26	26°	1.38	1.7	0.38	31°	1.37	1.9	0.55	30°	1.43	1.7	0.44
		26°	1.61	1.3	0.23	28°	1.64	1.5	0.34	33°	1.58	1.6	0.52	32°	1.78	1.3	0.37
		28°	1.92	1.1	0.21	30°	1.95	1.3	0.31	35°	1.84	1.3	0.50	34°	2.19	1.4	0.31
		30°	2.28	1.0	0.19	32°	2.33	1.1	0.27	–	–	–	–	–	–	–	–
9	5.5	24°	1.32	1.7	0.35	26°	1.37	2.0	0.48	31°	1.35	2.3	0.73	30°	1.41	2.0	0.59
		26°	1.58	1.6	0.31	28°	1.63	1.8	0.43	33°	1.56	2.0	0.69	32°	1.75	1.6	0.49
		28°	1.87	1.4	0.28	30°	1.94	1.6	0.39	35°	1.82	1.6	0.66	34°	2.15	1.5	0.41
		30°	2.23	1.2	0.25	32°	2.32	1.3	0.35	–	–	–	–	–	–	–	–
10	6	24°	1.30	2.0	0.44	26°	1.37	2.3	0.59	31°	1.34	2.0	0.93	30°	1.39	2.4	0.75
		26°	1.55	1.8	0.40	28°	1.63	2.1	0.53	33°	1.55	2.4	0.89	32°	1.72	2.0	0.62
		28°	1.84	1.6	0.36	30°	1.94	1.8	0.48	35°	1.80	2.0	0.84	34°	2.12	1.6	0.53
		30°	2.20	1.4	0.32	32°	2.31	1.6	0.43	–	–	–	–				–



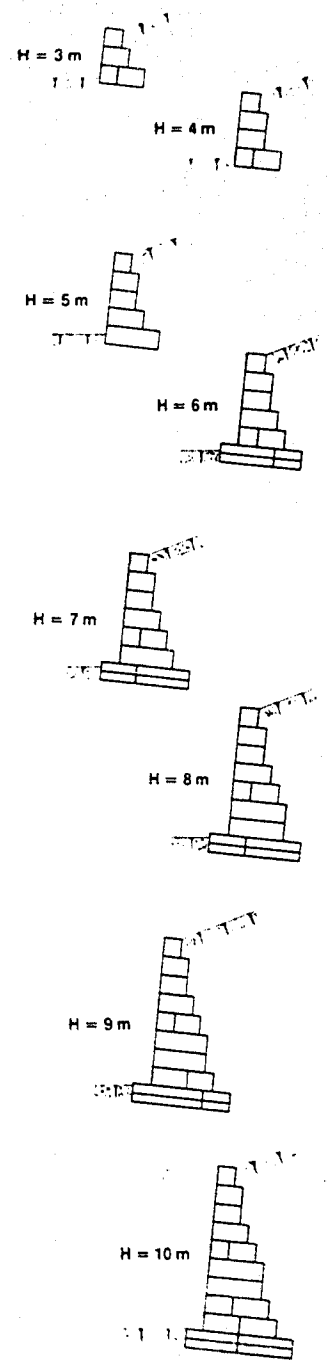
Gravity Walls with a Stepped Rear-Face H/B > 2

H (m)	B (m)	$\varepsilon = 0$, $p_0 = 0$				$\varepsilon = 0$, $p_0 = \gamma_t$				$\varepsilon \simeq \varphi$, $p_0 = 0$				$\varepsilon = 26.6°$, $p_0 = 0$			
		φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (m)	φ (°)	η_s (-)	σ_t (kg/cm²)	Δ (m)
2	1.5	19°	1.38	0.4	0.01	25°	1.36	0.6	0.02	25°	1.32	0.5	0.01	27°	1.42	0.5	0.01
		21°	1.67	0.4	0.01	27°	1.62	0.6	0.02	27°	1.51	0.5	0.01	29°	1.92	0.4	0.01
		23°	2.00	0.4	0.00	29°	1.93	0.5	0.02	29°	1.74	0.4	0.01	31°	2.44	0.4	0.01
		25°	2.40	0.4	0.00	31°	2.30	0.5	0.01	31°	2.00	0.4	0.01	33°	3.06	0.4	0.01
3	1.5	22°	1.39	0.8	0.02	26°	1.32	1.2	0.05	29°	1.38	1.0	0.04	28°	1.33	1.0	0.04
		24°	1.67	0.8	0.02	28°	1.58	1.1	0.04	31°	1.60	0.9	0.04	30°	1.72	0.8	0.03
		26°	2.01	0.7	0.01	30°	1.89	1.0	0.04	33°	1.85	0.8	0.03	32°	2.17	0.7	0.02
		28°	2.41	0.6	0.01	32°	2.26	0.8	0.03	35°	2.17	0.6	0.03	34°	2.72	0.6	0.02
4	2	23°	1.33	1.3	0.05	27°	1.40	1.6	0.09	30°	1.32	1.6	0.10	29°	1.35	1.5	0.08
		25°	1.59	1.2	0.04	29°	1.66	1.5	0.08	32°	1.53	1.5	0.09	31°	1.70	1.3	0.07
		27°	1.90	1.1	0.04	31°	1.98	1.3	0.07	34°	1.78	1.3	0.08	33°	2.11	1.0	0.05
		29°	2.27	1.0	0.03	33°	2.37	1.1	0.06	–	–	–	–	35°	2.63	0.8	0.04
5	2	25°	1.41	1.8	0.07	27°	1.31	2.7	0.14	32°	1.39	2.6	0.16	30°	1.37	2.3	0.38
		27°	1.69	1.6	0.07	29°	1.56	2.3	0.12	34°	1.62	2.1	0.15	32°	1.72	1.9	0.35
		29°	2.03	1.4	0.06	31°	1.87	1.9	0.11	–	–	–	–	34°	2.14	1.4	0.32
		31°	2.43	1.2	0.05	33°	2.24	1.6	0.10	–	–	–	–	–	–	–	–
6	2.5	25°	1.33	2.3	0.13	28°	1.41	3.0	0.20	32°	1.33	3.2	0.27	30°	1.30	3.0	0.23
		27°	1.59	2.1	0.12	30°	1.68	2.5	0.18	34°	1.55	2.8	0.26	32°	1.62	2.4	0.19
		29°	1.90	1.8	0.10	32°	2.00	2.2	0.16	–	–	–	–	34°	2.02	1.9	0.16
		31°	2.27	1.6	0.09	34°	2.41	1.8	0.14	–	–	–	–	–	–	–	–
7	3.5	25°	1.36	1.2	0.19	27°	1.35	1.5	0.28	32°	1.36	1.7	0.38	30°	1.33	1.5	0.31
		27°	1.63	1.0	0.17	29°	1.61	1.3	0.25	34°	1.59	1.3	0.35	32°	1.67	1.1	0.26
		29°	1.95	0.9	0.15	31°	1.93	1.1	0.23	–	–	–	–	34°	2.09	1.2	0.22
		31°	2.35	1.1	0.13	33°	2.32	1.0	0.20	–	–	–	–	–	–	–	–
8	4	25°	1.30	1.6	0.28	27°	1.32	1.9	0.40	32°	1.31	2.2	0.57	31°	1.43	1.8	0.43
		27°	1.55	1.4	0.25	29°	1.58	1.7	0.36	34°	1.53	1.8	0.54	33°	1.79	1.3	0.36
		29°	1.86	1.2	0.22	31°	1.89	1.4	0.32	–	–	–	–	35°	2.23	1.3	0.30
		31°	2.23	1.0	0.20	33°	2.27	1.1	0.29	–	–	–	–	–	–	–	–
9	4.5	26°	1.38	1.8	0.37	27°	1.31	2.3	0.53	33°	1.39	2.5	0.77	31°	1.40	2.2	0.60
		28°	1.65	1.6	0.33	29°	1.56	2.0	0.48	35°	1.63	2.0	0.73	33°	1.74	1.7	0.50
		30°	1.97	1.4	0.29	31°	1.87	1.7	0.43	–	–	–	–	35°	2.16	1.3	0.43
		32°	2.37	1.1	0.26	33°	2.24	1.4	0.38	–	–	–	–	–	–	–	–
10	5	26°	1.36	2.2	0.48	27°	1.31	2.6	0.67	33°	1.38	3.0	1.01	31°	1.38	2.7	0.79
		28°	1.62	1.9	0.43	29°	1.56	2.3	0.60	35°	1.61	2.4	0.95	33°	1.71	2.1	0.66
		30°	1.94	1.7	0.39	31°	1.86	2.0	0.54	–	–	–	–	35°	2.13	1.4	0.56
		32°	2.32	1.3	0.34	33°	2.23	1.6	0.48	–	–	–	–	–	–	–	–



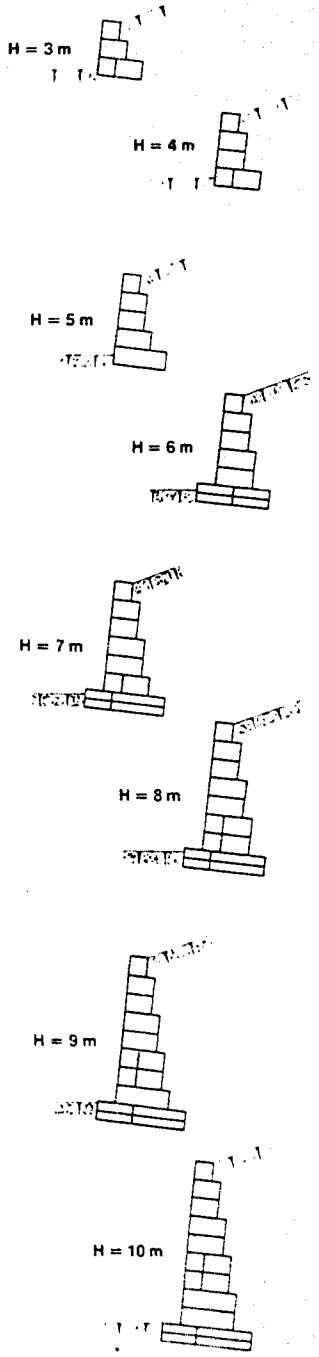
Semi-gravity Walls with a Stepped Rear-Face H/b < 2.5

H	B	b	$\varepsilon = 0$, $p_0 = 0$					$\varepsilon = 0$, $p_0 = \gamma_t$					$\varepsilon \simeq \varphi$, $p_0 = 0$					$\varepsilon = 26.6°$, $p_0 = 0$				
			φ	η_s	σ_t	σ_m	Δ	φ	η_s	σ_t	σ_m	Δ	φ	η_s	σ_t	σ_m	Δ	φ	η_s	σ_t	σ_m	Δ
(m)	(m)	(m)	(°)	(-)	(kg/cm²)	(kg/cm²)	(m)	(°)	(-)	(kg/cm²)	(kg/cm²)	(m)	(°)	(-)	(kg/cm²)	(kg/cm²)	(m)	(°)	(-)	(kg/cm²)	(kg/cm²)	(m)
3	2.5	1.5	17°	1.35	0.6	0.4	0.02	20°	1.32	0.7	0.6	0.05	26°	1.32	0.8	0.4	0.04	27°	1.30	0.8	0.5	0.04
			19°	1.66	0.6	0.4	0.02	22°	1.60	0.7	0.5	0.05	28°	1.49	0.7	0.4	0.03	29°	1.79	0.7	0.4	0.03
			21°	2.03	0.6	0.4	0.02	24°	1.94	0.7	0.5	0.04	30°	1.69	0.7	0.4	0.03	31°	2.31	0.7	0.4	0.02
			23°	2.46	0.6	0.4	0.01	26°	2.33	0.8	0.5	0.04	32°	1.92	0.8	0.4	0.03	33°	2.91	0.8	0.4	0.02
4	2.5	1.5	20°	1.41	0.9	0.7	0.05	22°	1.31	1.1	1.2	0.10	29°	1.37	1.3	0.9	0.08	28°	1.33	1.3	0.8	0.08
			22°	1.72	0.8	0.7	0.04	24°	1.59	1.0	1.0	0.09	31°	1.56	1.2	0.9	0.08	30°	1.74	1.0	0.8	0.06
			24°	2.08	0.7	0.7	0.04	26°	1.91	0.9	1.0	0.08	33°	1.79	1.1	0.8	0.07	32°	2.21	0.8	0.7	0.05
			26°	2.51	0.8	0.6	0.03	28°	2.30	0.9	0.9	0.07	35°	2.07	0.9	0.8	0.07	34°	2.78	0.9	0.7	0.04
5	3	2	20°	1.34	1.2	1.1	0.10	22°	1.32	1.5	1.7	0.17	29°	1.30	1.8	1.4	0.16	29°	1.46	1.6	1.3	0.14
			22°	1.64	1.1	1.0	0.08	24°	1.60	1.3	1.5	0.15	31°	1.49	1.7	1.3	0.16	31°	1.87	1.3	1.1	0.11
			24°	1.98	1.0	1.0	0.08	26°	1.92	1.2	1.4	0.14	33°	1.71	1.5	1.3	0.15	33°	2.35	1.0	1.0	0.09
			26°	2.39	0.9	0.9	0.07	28°	2.31	1.1	1.3	0.12	35°	1.97	1.3	1.2	0.14	35°	2.95	1.2	1.0	0.08
6	4.5	2.5	20°	1.41	1.1	1.5	0.15	22°	1.42	1.2	2.2	0.25	29°	1.35	1.3	1.9	0.26	28°	1.31	1.2	1.8	0.26
			22°	1.71	1.2	1.4	0.14	24°	1.72	1.3	1.9	0.22	31°	1.54	1.4	1.8	0.25	30°	1.72	1.4	1.6	0.20
			24°	2.08	1.3	1.3	0.12	26°	2.07	1.4	1.7	0.20	33°	1.77	1.5	1.7	0.24	32°	2.18	1.5	1.4	0.17
			26°	2.50	1.3	1.2	0.11	28°	2.50	1.5	1.6	0.18	35°	2.05	1.7	1.6	0.23	34°	2.75	1.6	1.3	0.14
7	5	3	20°	1.35	1.3	1.8	0.23	22°	1.40	1.4	2.6	0.34	29°	1.30	1.4	2.3	0.40	29°	1.45	1.4	2.1	0.34
			22°	1.65	1.4	1.7	0.21	24°	1.70	1.5	2.3	0.31	31°	1.49	1.5	2.2	0.38	31°	1.87	1.6	1.9	0.28
			24°	2.00	1.4	1.6	0.18	26°	2.05	1.6	2.0	0.28	33°	1.71	1.7	2.1	0.36	33°	2.36	1.8	1.7	0.23
			26°	2.41	1.5	1.5	0.16	28°	2.47	1.7	1.9	0.25	35°	1.97	1.9	2.0	0.34	35°	2.97	2.0	1.6	0.20
8	5	3	21°	1.32	1.3	2.6	0.31	23°	1.39	1.4	3.7	0.44	31°	1.39	1.8	3.3	0.52	29°	1.35	1.7	3.2	0.47
			23°	1.60	1.4	2.3	0.27	25°	1.68	1.5	3.2	0.39	33°	1.60	1.6	3.1	0.49	31°	1.73	1.6	2.6	0.38
			25°	1.94	1.5	2.1	0.25	27°	2.02	1.7	2.8	0.36	35°	1.85	1.9	2.9	0.47	33°	2.18	1.8	2.3	0.32
			27°	2.34	1.6	1.9	0.22	29°	2.43	1.8	2.5	0.32	–	–	–	–	–	35°	2.74	2.1	2.1	0.27
9	5.5	3.5	21°	1.30	1.4	3.0	0.42	23°	1.40	1.5	4.2	0.57	31°	1.36	2.2	3.9	0.72	29°	1.32	2.1	3.7	0.65
			23°	1.58	1.5	2.7	0.38	25°	1.69	1.6	3.6	0.52	33°	1.57	2.0	3.6	0.68	31°	1.69	1.7	3.1	0.53
			25°	1.91	1.7	2.5	0.34	27°	2.03	1.8	3.1	0.47	35°	1.82	2.0	3.4	0.65	33°	2.14	2.0	2.7	0.45
			27°	2.30	1.8	2.3	0.30	29°	2.44	2.0	2.8	0.42	–	–	–	–	–	35°	2.69	2.3	2.4	0.38
10	6	4	22°	1.42	1.6	3.3	0.52	23°	1.40	1.8	4.6	0.72	31°	1.34	2.7	4.4	0.94	29°	1.31	2.5	4.2	0.85
			24°	1.72	1.7	2.9	0.47	25°	1.69	1.8	3.9	0.65	33°	1.55	2.4	4.1	0.90	31°	1.67	1.9	3.5	0.70
			26°	2.08	1.9	2.7	0.42	27°	2.04	1.9	3.5	0.59	35°	1.79	2.1	3.9	0.85	33°	2.11	2.1	3.1	0.59
			28°	2.50	2.0	2.5	0.37	29°	2.45	2.1	3.1	0.53	–	–	–	–	–	35°	2.65	2.5	2.8	0.50



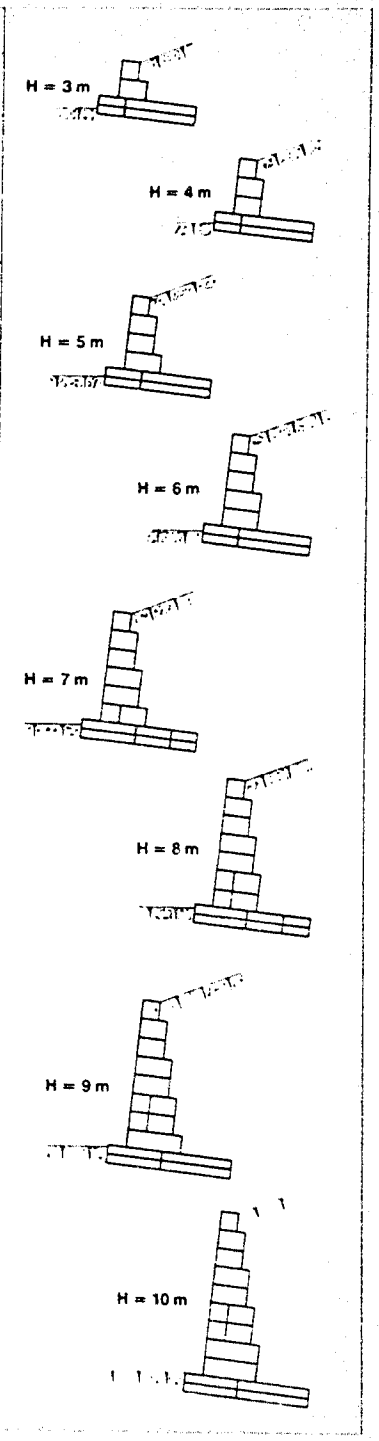
Semi-gravity Walls with a Stepped Rear-Face H/b > 2.5

H	B	b	$\varepsilon = 0$, $p_0 = 0$					$\varepsilon = 0$, $p_0 = \gamma_t$					$\varepsilon = \varphi$, $p_0 = 0$					$\varepsilon = 26.6°$, $p_0 = 0$				
(m)	(m)	(m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Λ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Λ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Λ (m)	φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Λ (m)
—	—	—																				
3	2.5	1.5	17°	1.35	0.6	0.4	0.02	20°	1.32	0.7	0.6	0.05	26°	1.32	0.8	0.4	0.04	27°	1.30	0.8	0.5	0.04
			19°	1.66	0.6	0.4	0.02	22°	1.60	0.7	0.5	0.05	28°	1.49	0.7	0.4	0.03	29°	1.79	0.7	0.4	0.03
			21°	2.03	0.6	0.4	0.02	24°	1.94	0.8	0.5	0.04	30°	1.69	0.7	0.4	0.03	31°	2.31	0.7	0.4	0.02
			23°	2.46	0.6	0.4	0.01	26°	2.33	0.8	0.5	0.04	32°	1.92	0.8	0.4	0.03	33°	2.91	0.8	0.4	0.02
4	2.5	1.5	20°	1.41	0.9	0.7	0.05	22°	1.31	1.1	1.2	0.10	29°	1.37	1.3	0.9	0.08	28°	1.33	1.3	0.9	0.08
			22°	1.72	0.8	0.7	0.04	24°	1.59	1.0	1.0	0.09	31°	1.56	1.2	0.9	0.08	30°	1.74	1.0	0.8	0.06
			24°	2.08	0.7	0.7	0.04	26°	1.91	0.9	1.0	0.08	33°	1.79	1.1	0.8	0.07	32°	2.21	0.8	0.7	0.05
			26°	2.51	0.8	0.6	0.03	28°	2.30	0.9	0.9	0.07	35°	2.07	0.9	0.8	0.07	34°	2.78	0.9	0.7	0.04
5	3	2	20°	1.34	1.2	1.1	0.10	22°	1.32	1.5	1.7	0.17	29°	1.30	1.8	1.4	0.16	29°	1.46	1.6	1.3	0.14
			22°	1.64	1.1	1.0	0.08	24°	1.60	1.3	1.5	0.15	31°	1.49	1.7	1.3	0.16	31°	1.87	1.3	1.1	0.11
			24°	1.98	1.0	1.0	0.08	26°	1.92	1.2	1.4	0.14	33°	1.71	1.5	1.3	0.15	33°	2.35	1.0	1.0	0.09
			26°	2.39	0.9	0.9	0.07	28°	2.31	1.1	1.3	0.12	35°	1.97	1.3	1.2	0.14	35°	2.95	1.2	1.0	0.08
6	4	2	21°	1.38	1.1	1.8	0.15	23°	1.38	1.1	3.2	0.24	30°	1.37	1.2	2.4	0.25	29°	1.42	1.2	2.2	0.22
			23°	1.67	1.2	1.6	0.13	25°	1.67	1.2	2.6	0.22	32°	1.57	1.3	2.2	0.24	31°	1.83	1.3	1.8	0.18
			25°	2.02	1.2	1.5	0.12	27°	2.01	1.3	2.2	0.19	34°	1.82	1.5	2.1	0.23	33°	2.31	1.5	1.6	0.15
			27°	2.44	1.3	1.4	0.10	29°	2.41	1.4	1.9	0.17	—	—	—	—	—	35°	2.91	1.6	1.4	0.12
7	4.5	2.5	21°	1.34	1.2	2.3	0.24	23°	1.38	1.2	3.9	0.35	30°	1.32	1.4	3.1	0.41	29°	1.38	1.3	2.8	0.36
			23°	1.63	1.3	2.1	0.21	25°	1.67	1.4	3.2	0.32	32°	1.52	1.4	2.8	0.39	31°	1.77	1.5	2.3	0.29
			25°	1.97	1.4	1.9	0.19	27°	2.01	1.5	2.7	0.29	34°	1.76	1.6	2.6	0.36	33°	2.23	1.7	2.0	0.24
			27°	2.38	1.5	1.7	0.17	29°	2.42	1.6	2.3	0.26	—	—	—	—	—	35°	2.81	1.9	1.8	0.20
8	4.5	2.5	22°	1.31	1.2	3.6	0.32	25°	1.51	1.3	6.0	0.43	31°	1.32	2.0	5.4	0.56	30°	1.44	1.6	4.3	0.45
			24°	1.59	1.3	3.0	0.29	27°	1.81	1.5	4.4	0.39	33°	1.53	1.7	4.6	0.53	32°	1.83	1.5	3.3	0.38
			26°	1.92	1.5	2.6	0.26	29°	2.18	1.6	3.6	0.35	35°	1.78	1.7	4.0	0.49	34°	2.31	1.8	2.8	0.31
			28°	2.31	1.6	2.3	0.23	31°	2.63	1.8	3.0	0.31	—	—	—	—	—	—	—	—	—	—
9	5	3	22°	1.31	1.4	4.4	0.45	25°	1.53	1.5	6.5	0.58	31°	1.30	2.4	6.1	0.78	30°	1.42	2.0	5.0	0.64
			24°	1.58	1.4	3.6	0.40	27°	1.84	1.6	4.9	0.52	33°	1.50	2.1	5.2	0.74	32°	1.81	1.6	3.9	0.53
			26°	1.91	1.6	3.1	0.36	29°	2.22	1.8	4.0	0.47	35°	1.75	1.7	4.6	0.70	34°	2.28	2.0	3.2	0.45
			28°	2.30	1.7	2.7	0.32	31°	2.67	2.0	3.4	0.42	—	—	—	—	—	—	—	—	—	—
10	5	3	24°	1.44	1.7	5.7	0.53	29°	2.04	1.7	6.1	0.59	35°	1.66	2.3	7.4	0.90	32°	1.70	2.0	6.0	0.69
			26°	1.74	1.5	4.6	0.47	31°	2.46	2.0	4.8	0.53	—	—	—	—	—	34°	2.14	1.9	4.6	0.58
			28°	2.09	1.7	3.8	0.42	33°	2.97	2.2	4.0	0.47	—	—	—	—	—	—	—	—	—	—
			30°	2.52	1.9	3.3	0.38	35°	3.62	2.6	3.5	0.42	—	—	—	—	—	—	—	—	—	—



Semi-gravity Walls with a Stepped Rear-Face and Extended Foundation

H (m)	B (m)	b (m)	$\varepsilon = 0$, $p_0 = 0$: φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	$\varepsilon = 0$, $p_0 = \gamma_t$: φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	$\varepsilon \simeq \varphi$, $p_0 = 0$: φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)	$\varepsilon = 26.6°$, $p_0 = 0$: φ (°)	η_s (°)	σ_t (kg/cm²)	σ_m (kg/cm²)	Δ (m)
—	—	—	—					—					—					—				
3	5.5	1.5	14°	1.88	0.7	0.4	0.02	14°	1.37	0.8	0.7	0.06	16°	1.33	0.7	0.5	0.03	27°	2.54	1.0	0.5	0.03
			16°	2.35	0.7	0.4	0.02	16°	1.71	0.9	0.6	0.05	18°	1.53	0.8	0.5	0.03	29°	3.54	1.0	0.4	0.02
			18°	2.90	0.8	0.4	0.01	18°	2.12	0.9	0.6	0.05	20°	1.76	0.8	0.5	0.03	31°	4.58	1.0	0.4	0.01
			20°	3.55	0.8	0.4	0.01	20°	2.59	1.0	0.6	0.04	22°	2.00	0.9	0.5	0.03	33°	5.79	1.0	0.4	0.01
4	5.5	1.5	14°	1.39	0.8	0.9	0.06	16°	1.35	0.9	2.0	0.12	20°	1.38	0.9	1.1	0.08	27°	2.08	1.2	1.0	0.08
			16°	1.73	0.9	0.8	0.05	18°	1.67	1.0	1.6	0.11	22°	1.59	1.0	1.0	0.08	29°	2.85	1.2	0.8	0.06
			18°	2.14	0.9	0.8	0.04	20°	2.04	1.1	1.3	0.09	24°	1.81	1.0	1.0	0.08	31°	3.68	1.3	0.7	0.04
			20°	2.62	1.0	0.7	0.04	22°	2.48	1.1	1.2	0.08	26°	2.07	1.1	0.9	0.07	33°	4.65	1.3	0.7	0.04
5	6	2	15°	1.34	0.9	1.3	0.11	17°	1.36	1.0	2.7	0.20	22°	1.36	1.0	1.7	0.17	27°	1.77	1.2	1.5	0.17
			17°	1.67	1.0	1.2	0.10	19°	1.67	1.1	2.2	0.18	24°	1.56	1.1	1.6	0.17	29°	2.45	1.4	1.3	0.13
			19°	2.05	1.1	1.1	0.09	21°	2.04	1.2	1.8	0.16	26°	1.78	1.2	1.5	0.16	31°	3.07	1.5	1.1	0.10
			21°	2.51	1.2	1.1	0.08	23°	2.48	1.3	1.6	0.15	28°	2.03	1.3	1.4	0.15	33°	3.79	1.6	1.0	0.08
6	6	2	17°	1.39	1.0	2.4	0.18	20°	1.59	1.1	5.3	0.27	24°	1.36	1.2	3.5	0.29	27°	1.53	1.1	3.2	0.29
			19°	1.71	1.1	2.0	0.16	22°	1.93	1.3	3.7	0.25	26°	1.56	1.1	3.0	0.27	29°	2.06	1.4	2.2	0.22
			21°	2.09	1.2	1.8	0.14	24°	2.34	1.4	2.9	0.22	28°	1.73	1.3	2.7	0.26	31°	2.59	1.6	1.8	0.17
			23°	2.54	1.3	1.6	0.13	26°	2.81	1.5	2.4	0.20	30°	1.93	1.4	2.4	0.25	33°	3.20	1.7	1.6	0.14
7	6.5	2.5	17°	1.30	1.1	3.3	0.29	20°	1.52	1.2	6.5	0.40	25°	1.31	1.6	4.3	0.45	27°	1.36	1.6	4.1	0.47
			19°	1.60	1.2	2.7	0.26	22°	1.84	1.3	4.5	0.36	27°	1.46	1.5	3.7	0.43	29°	1.83	1.4	2.8	0.35
			21°	1.95	1.3	2.3	0.23	24°	2.22	1.5	3.5	0.33	29°	1.63	1.3	3.3	0.41	31°	2.31	1.6	2.3	0.29
			23°	2.36	1.4	2.1	0.21	26°	2.60	1.6	2.9	0.30	31°	1.83	1.5	3.0	0.39	33°	2.87	1.8	2.0	0.24
8	6.5	2.5	19°	1.40	1.3	5.4	0.37	25°	2.08	1.5	6.0	0.43	29°	1.46	2.0	6.6	0.58	28°	1.44	2.0	6.8	0.56
			21°	1.71	1.3	4.0	0.34	27°	2.45	1.6	4.4	0.38	31°	1.65	1.8	5.4	0.55	30°	1.85	1.4	4.3	0.45
			23°	2.08	1.4	3.3	0.30	29°	2.88	1.8	3.6	0.34	33°	1.87	1.5	4.6	0.52	32°	2.31	1.7	3.3	0.37
			25°	2.46	1.6	2.8	0.27	31°	3.40	2.0	3.0	0.31	35°	2.14	1.8	4.0	0.49	34°	2.86	2.0	2.8	0.31
9	7	3	20°	1.49	1.5	5.6	0.50	26°	2.17	1.7	5.6	0.54	30°	1.46	2.4	6.7	0.80	29°	1.54	2.2	6.0	0.71
			22°	1.80	1.4	4.4	0.45	28°	2.56	1.8	4.4	0.49	32°	1.66	2.2	5.6	0.76	31°	1.95	1.6	4.3	0.58
			24°	2.12	1.6	3.6	0.40	30°	3.02	2.0	3.7	0.44	34°	1.89	1.8	4.9	0.72	33°	2.47	1.9	3.5	0.48
			26°	2.50	1.7	3.1	0.36	32°	3.58	2.2	3.2	0.39	—	—	—	—	—	35°	3.01	2.3	3.0	0.41
10	7	3	24°	1.88	1.5	5.7	0.53	29°	2.51	1.9	6.1	0.59	35°	1.90	2.1	7.4	0.90	32°	2.01	1.8	6.0	0.69
			26°	2.22	1.7	4.6	0.47	31°	2.97	2.1	4.8	0.53	—	—	—	—	—	34°	2.50	2.1	4.6	0.58
			28°	2.63	1.9	3.8	0.42	33°	3.54	2.4	4.0	0.47	—	—	—	—	—	—	—	—	—	—
			30°	3.11	2.1	3.3	0.38	35°	4.25	2.7	3.5	0.42	—	—	—	—	—	—	—	—	—	—



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Publication edited by
OFFICINE MACCAFERRI S.p.A.
Study: Research and Development Dept.
Layout: L. Monti
Design: P. Angelini - L. Monti - R. Rimondi
Photographic Archive
of Officine Maccaferri S.p.A.
and its Overseas Subsidiary Offices
Printed by: Labanti & Nanni S.r.l.
Via Segantini, 32 - Bologna
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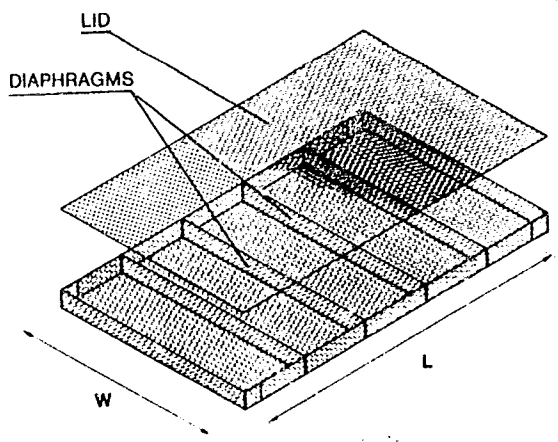
TECHNICAL SPECIFICATIONS

RENO MATTRESS - ZINC COATED

Reno mattress

The Reno mattress is a large and thin structure, made of hexagonal double twisted wire mesh (Fig. 1); the mattress is divided into cells by panels positioned at 1 m centres. The wire used for manufacturing the mattress is heavily zinc coated mild steel. The standard combinations mesh/wire are shown in Tab. 1. In order to reinforce the structure, all edges are selvedged with a wire having greater diameter (Tab. 3).
Dimensions and tolerances on sizes are shown at Tab. 2.

Fig 1

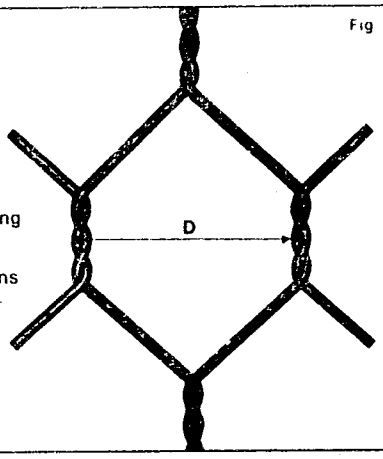


Wire

1) Tensile strength: both the wire used for the manufacture of mattress and the lacing wire, shall have a tensile strength of 38-50 Kg/mm². according to BS 1052/80 "Mild Steel Wire". Above values are referred to wire before manufacturing mesh. Tolerances of wire, shown at Tab. 4 meet the requirements of BS 1052/80.
2) Elongation: the test must be carried out before manufacturing mesh on a sample at least 30 cm long. Elongation shall not be less than 12%.
3) Zinc coating and tolerances: minimum quantities, shown at Tab. 4, meet the requirements of BS 443/82 and exceed those of ASTM A641-71 A, DIN 1548, QQ-W-461 H.
4) Adhesion of zinc: the adhesion of the zinc coating to the wire should be such that, when the wire is wrapped six turns round on a mandrel having four times the diameter of the wire, it does not flake or crack when rubbing it with the bare fingers.

Fig 2

MESH TOLERANCE
The tolerance on the opening of mesh "D", being the distance between the axes of the twists, is according to specifications UNI 8018.



Tab 1

STANDARD COMBINATIONS MESH-WIRE	
D (cm)	o Wire mm
5 x 7*	2.00
6 x 8*	2.00-2.20

* Code number, without references to dimensions

Tab 2

STANDARD SIZES	TOLERANCES
L = 3.00 m - 4.00 m - 5.00 m - 6.00 m	± 3%
W = 2.00 m - 3.00 m	± 3%
H = Mesh 5 x 7: 0.15 - 0.20 - 0.25 m Mesh 6 x 8: 0.17 - 0.23 - 0.30 m	± 2.5 cm.

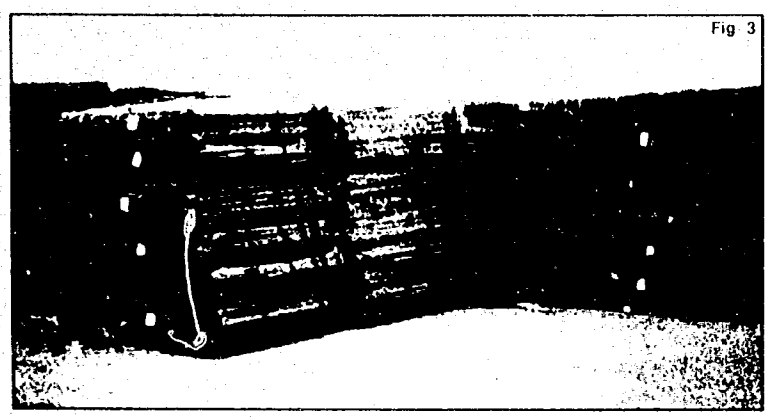


Fig 3

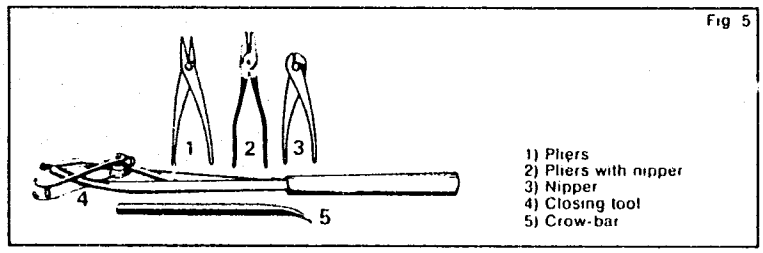


Fig 5

Fig. 4

Fig 3 - Reno Mattress bundles at stock.
Fig 5 - Tools used in assembly and erection.
Fig 4 - Shipment of Reno Mattress.

Tab 3

Mesh wire	ø mm	2.00	2.20	2.40
Selvedge wire	ø mm	2.40	2.70	3.00
Lacing wire*	ø mm	2.00	2.00	2.00

* Lacing wire is supplied at the rate of 5% on mattress weight

Tab. 4

Mesh wire	ø mm	2.00	2.20	2.40	2.70	3.00
Wire tolerance	ø mm ±	0.06	0.06	0.06	0.08	0.08
Quantity of zinc	gr/m²	240	240	260	260	275

Assembly and erection

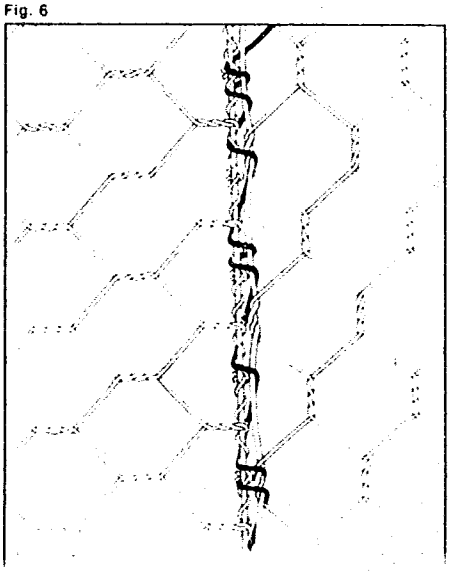
1) Unfold the units, erect corners and diaphragms and bind them to the side panels. Lacing wire is supplied together with the Reno mattress. For a correct lacing operation, the wire should be passed through each mesh, making a double twist every other mesh (Fig. 6).
2) Fill the Reno mattress with stones, whose minimum size is not less than dimension "D" of mesh, and maximum size is about 2.5 times "D". Bigger stones are accepted, provided that their total volume does not exceed 5% of the cell volume. Stones must be durable and, in case of cold climate, non-porous.
3) Check filling at the corners. Compaction is not necessary.
4) Bind the lid down with the usual lacing operation.

NOTE: all Reno mattress must be connected to each other along all corners with the same lacing operation.

Request of offer: When requesting an offer, please specify: quantities per each size; size of units (length x width x height, see Fig. 1); type of mesh; wire diameter.

Example: No. 1500 Reno mattress 6 x 2 x 0.15 m - Mesh Type 5 x 7 - Wire

Fig. 6



MACCAFERRI GABIONS

OFFICINE MACCAFERRI S.p.A.
40123 Bologna (Italy)
Via Agresti, 6 - P.O. Box 396
Tel.: (051) 234303-279701
Cable: Gabbionimac
Telex: 510649-510371 Gabion I

TECHNICAL SPECIFICATIONS

BOX GABION - ZINC COATED

Box gabion

The box gabion is a structure made of hexagonal double twisted wire mesh (Fig. 1 and 2).
The wire used in the manufacture of the gabion is heavily zinc coated mild steel.
The standard combinations mesh/wire are shown in Tab. 1.
In order to reinforce the structure, all edges are selvedged with a wire having greater diameter (Tab. 3).
The gabion can be divided into cells by means of diaphragms positioned at 1 m centres (Fig. 1).
Dimensions and tolerances on sizes are shown at Tab. 2.

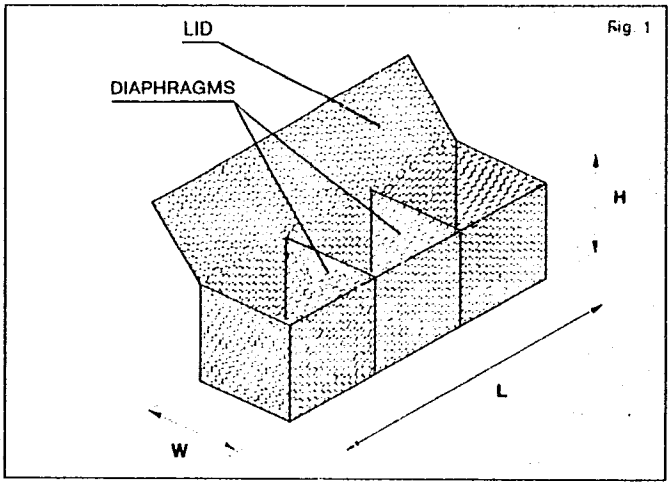


Fig. 1

Wire

1) Tensile strength: both the wire used for the manufacture of gabions and the lacing wire, shall have a tensile strength of 38-50 Kg/mm^2. according to BS 1052/80 "Mild Steel Wire". Above values are referred to wire before manufacturing mesh. Tolerances of wire, shown at Tab. 4 meet the requirements of BS 1052/80.
2) Elongation: the test must be carried out before manufacturing mesh on a sample at least 30 cm long. Elongation shall not be less than 12%.
3) Zinc coating and tolerances: minimum quantities of zinc, shown at Tab. 4, meet the requirements of BS 443/82 and exceed those of ASTM A641-71 A, DIN 1548, QQ-W-461 H.
4) Adhesion of zinc: the adhesion of the zinc coating to the wire should be such that, when the wire is wrapped six turns round on a mandrel having four times the diameter of the wire, it does not flake or crack when rubbing it with the bare fingers.

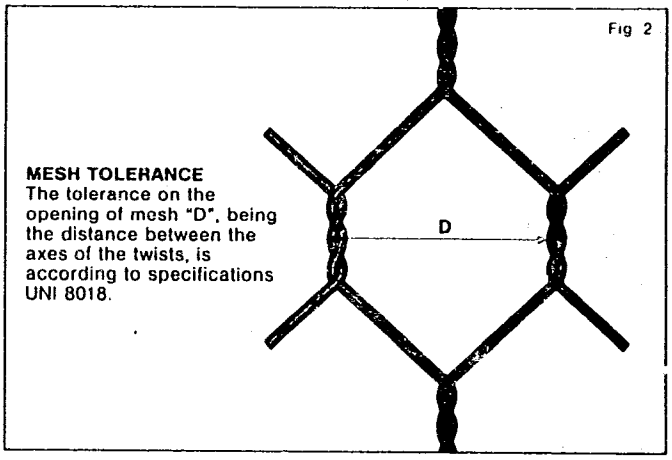


Fig. 2

MESH TOLERANCE
The tolerance on the opening of mesh "D", being the distance between the axes of the twists, is according to specifications UNI 8018.

Tab. 1

STANDARD COMBINATIONS MESH-WIRE	
D (cm)	ø Wire mm
5 x 7*	2.00 - 2.40
6 x 8*	2.20 - 2.70
8 x 10*	2.40 - 2.70 - 3.00
10 x 12*	2.70 - 3.00

* Code number, without reference to dimensions

Tab. 2

STANDARD SIZES	TOLERANCES
L = 1.50 m - 2.00 m - 3.00 m - 4.00 m	± 3%
W = 1.00 m	± 5%
H = 0.50 m - 1.00 m	± 5%

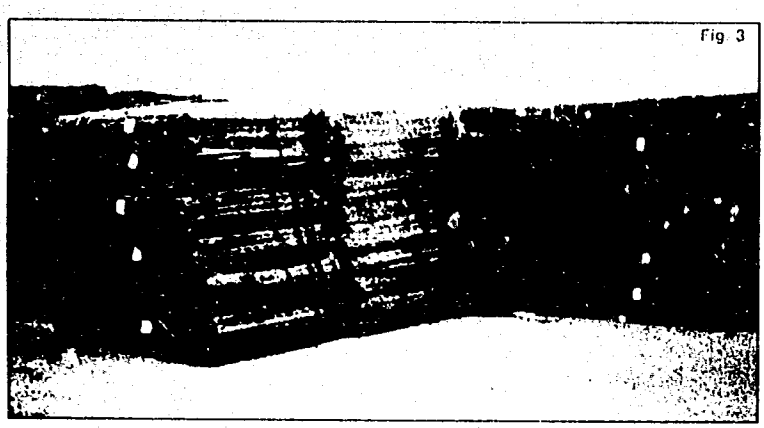


Fig. 3

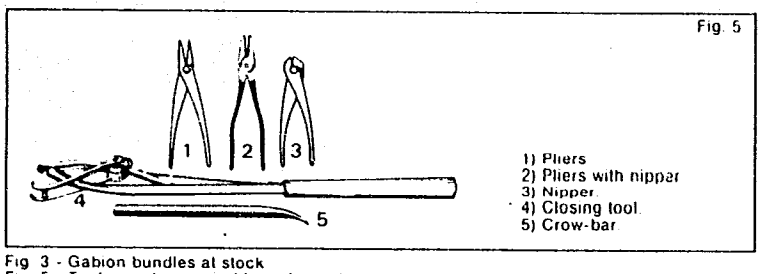


Fig. 5

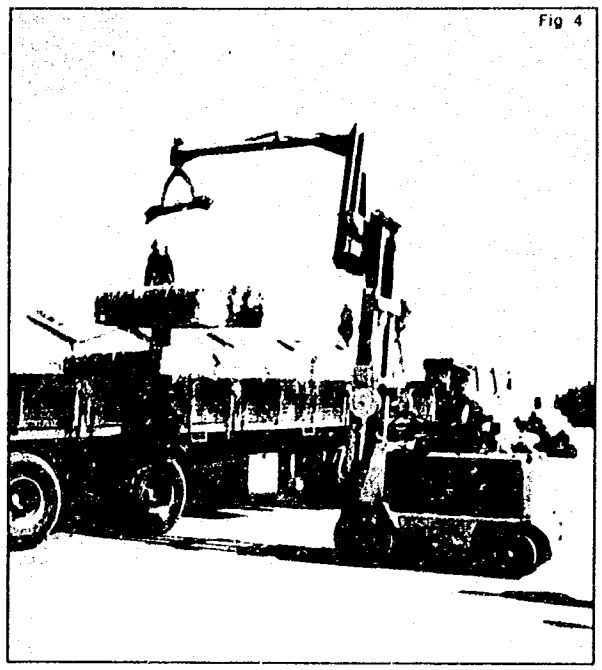


Fig. 4

Fig. 3 - Gabion bundles at stock
Fig. 5 - Tools used in assembly and erection
Fig. 4 - Shipment of gabions

Tab. 3

Mesh wire	ø mm	2.00	2.20	2.40	2.70	3.00
Selvedge wire	ø mm	2.40	2.70	3.00	3.40	3.90
Lacing wire*	ø mm	2.00	2.00	2.00	2.20	2.40

* Lacing wire is supplied at the rate of 5% on gabion weight

Tab. 4

Mesh wire	ø mm	2.00	2.20	2.40	2.70	3.00	3.40	3.90
Wire tolerance	ø mm ±	0.06	0.06	0.06	0.08	0.08	0.10	0.10
Quantity of zinc	gr/m²	240	240	260	260	275	275	290

Assembly and erection

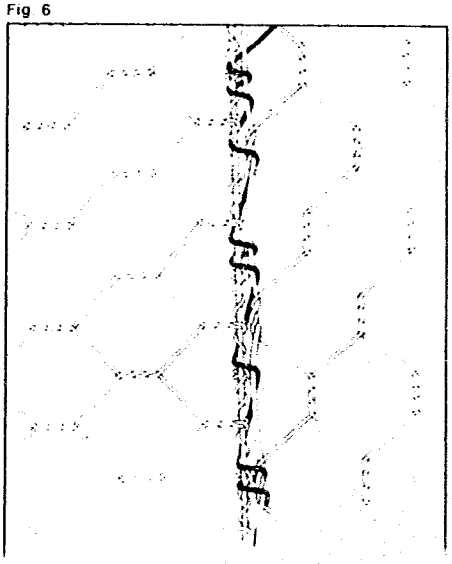
1) Unfold the units, erect corners and diaphragms and bind them to the side panels. Lacing wire is supplied together with the gabions. For a correct lacing operation, the wire should be passed through each mesh, making a double twist every other mesh (Fig. 6).
2) Fill the gabion with stones, whose minimum size is not less than dimension "D" of mesh, and maximum size is about 2.5 times "D". Bigger stones are accepted, provided that their total volume does not exceed 5% of the cell volume. Stones must be durable and, in case of cold climate, non-porous.
3) Check filling at the corners. Compaction is not necessary.
4) Bind the lid down with the usual lacing operation.

NOTE: all gabions must be connected to each other along all corners with the same lacing operation.

Request of offer: When requesting an offer, please specify: quantities per each size; size of units (length x width x height, see Fig. 1); type of mesh; wire diameter; if with or without diaphragms.

Example: No. 100 gabions 2 x 1 x 1 m - Mesh Type 8 x 10 - Wire dia. 2.7 mm.

Fig. 6



Appendix D

Group Design Calculations

Group 1 - Calculations

Participants:

Engr. Roohullah, Leader
Engr. M. Azim
Engr. Abdul Hamid
Engr. Bashirullah

Afghan Construction & Logistics Unit
(ACLU)

DESIGN OF GABION CHECK DAM

Prepared by: Group # 1

1. Engr. Roohullah (Leader)
2. Engr. Bashirullah
3. Engr Azim
4. Engr. A. Hamid

Submitted To:

Mr. Fred Wilson

Date: December 11, 1991

12/91

Design concept correct, design incorrect. Only minor adjustments required

Question:

Design a (4M) meter high wall as check dam across stream/gully.
start with rule of thumb, use picture?

Weight of material is in following:

1. Gabion Wall without Mortar:

$\gamma_g = 1.82\ T/M^3$ Dry.

$\gamma_g = 2.1\ T/M^3$ saturated

$\gamma_g = 1.1\ T/M^3$ submerged.

2. Fill

Assume $\rightarrow \gamma_e = 2.02\ T/M^3$ Saturated

$\gamma_e = 1.68\ T/M^3$ Dry

Ungraded $\gamma_e = 0.91\ T/M^3$ submerged.

3. Water:

$\gamma_w = 1.0$ to $1.2\ T/M^3$

Average (not turbid) $\gamma_w = 1.0\ T/M^3$

Engr. Koohullah

Assignment/Home Work

Complete the previous inadequate design by making another design that is adequate. Use the same criteria as shown in the previous calculations and redesign the structure so that it fits the following criteria. The final design should not be overly conservative, but it must meet these requirements:

* Overturning Factor of Safety
  No less than 1.5

* Sliding Factor of Safety
  No less than 1.4 (where f = .60)

* Bearing Values
  Maximum no more than 28 tons/M^2

Show all calculations as well as a final sketch of the completed structure - all other assumptions are the same as the first design presented (and included here) in lecture. Gabions should be in .5 meter segments i.e. no dimensions such as of 1.23 should be used.

Completed design papers are to be turned in to CCSC Engineering Department no later than 12 noon, 11 December 1991.

Further Home Work

1) Change the upstream & downstream water levels to upstream - 1.9 M downstream - 1.6 M

   For the sake of simplicity use exact same "uplift" pressures as in previous design; present this work on 11 Dec. 1991 (All other forces change according to new dimensions)

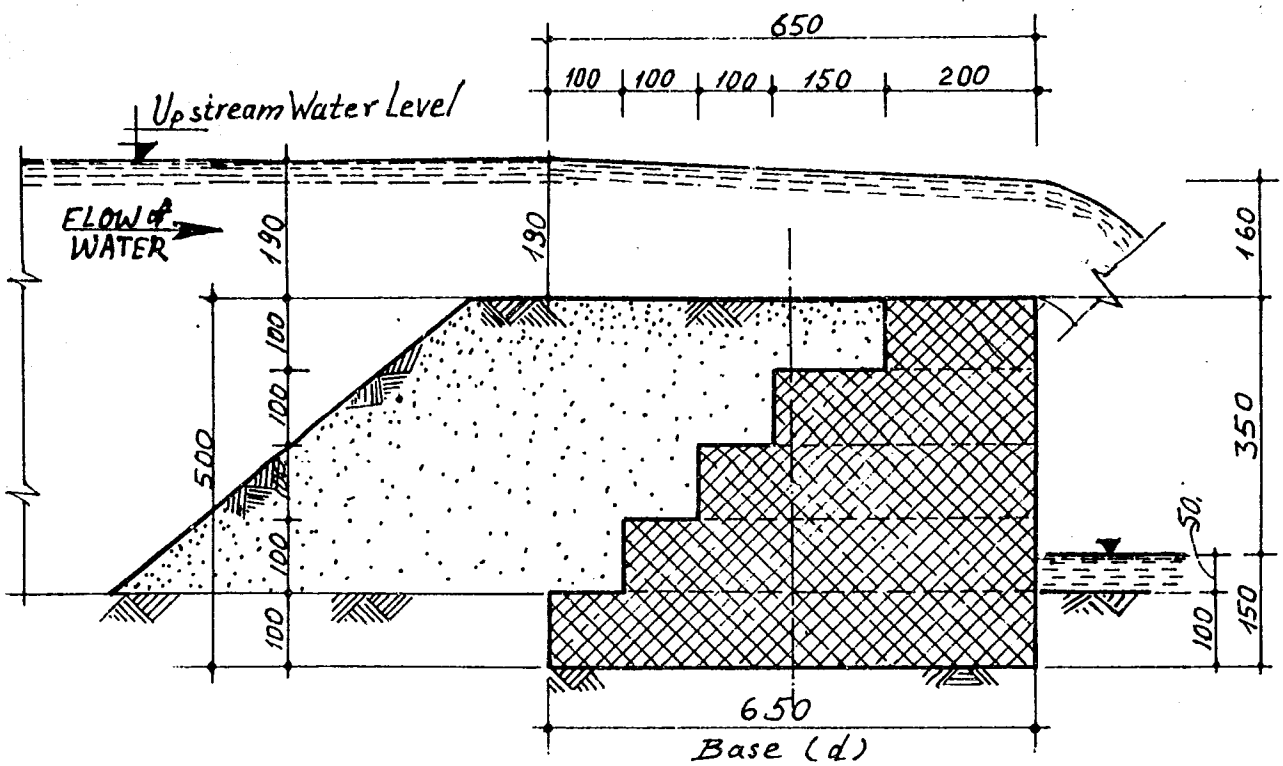
2) Also check your design, assuming there is earth pressure but no water pressure on upstream or downstream forces, check for bearing pressure, sliding and over turning stability for Part 1) design.

   If wall is over designed without water pressure show the size the wall should be, to meet all requirements i.e. overturning, sliding & bearing, when the structure allows water to totally drain thru or proper drains are placed so that there is no hydrostatic pressure on dam. (i.e. redesign wall w/o water pressure).

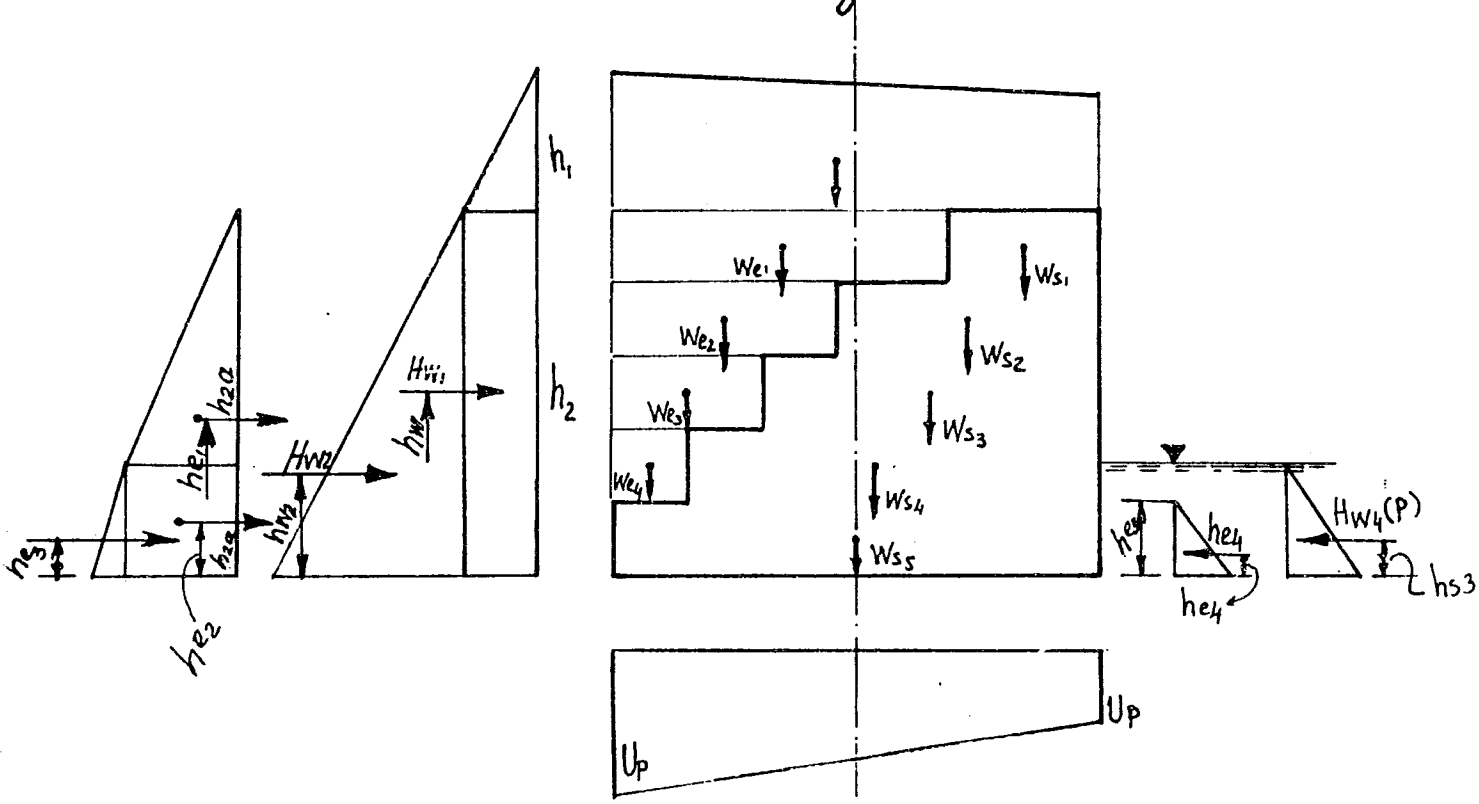
   This redesign can be presented on Saturday, 14, December 1991.

DSENG-2, GABION2, FW/ah

FIGURE (1) : GABION CHECK DAM



Force Diagram



Calculation table

Source	Force Symbol Ltr. Ref.	Dimensions [M]	Area [M2]	Unit γ [T/M3]	Total W [Tons]	Arm [M]	Moment (abt. Pt.B) Stabil. + Overturning − Ton Meters	Remarks
					-4.2	3.95	-16.59	↷
Water	W_{w_1}	$\frac{1.9+1.6}{2} \cdot 6.5$	11.38	1	11.38	3.34	38.01	↶
Earth	We_1	4.5 x1	4.50	2.02	9.09	4.25	38.63	↶
	We_2	3x1	3,00	2.02	6.06	5.00	30.30	↶
	We_3	1x 2	2	2.02	4.04	5,50	22,22	↶
	We_4	1x0,5	0,50	2,02	1.01	6,00	6.06	↶
	We_5	1x0.5	0,50	0,91	0.45	6,00	2,70	↶
	Ws_1	1x2	2,00	2,10	4.20	1.0	4,20	↶
	Ws_2	3.50x1	3.50	2.10	7.35	1,75	12,86	↶
	Ws_3	4.50x1	4.50	2.10	9.45	2,25	21,26	↶
	Ws_4	0.50x5.50	2,75	2,10	5,78	2,75	15,88	↶
	Ws_5	0,5x5.50	2,75	1.10	3,03	2,75	8,33	↶
	Ws_6	1x 6,50	6.50	1,10	7.15	3,25	23,25	↶
	Total weight (Resultant force)				64.79			
	Net Stabilizing moment						207.11	↷

C: Gabion

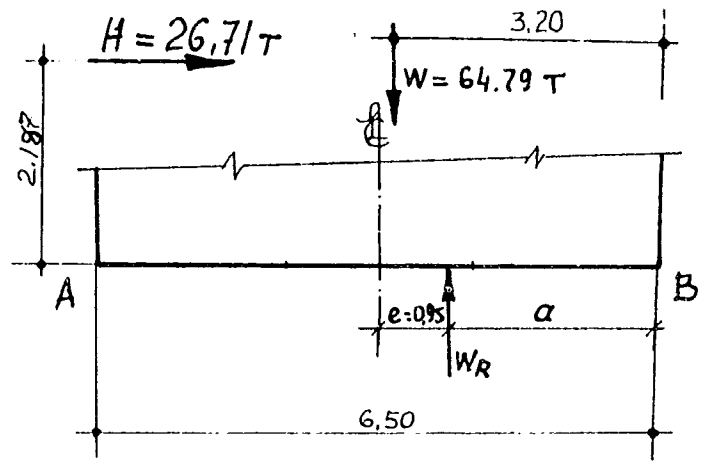
$$a_1 = \frac{207.11}{64.79} = 3.196$$

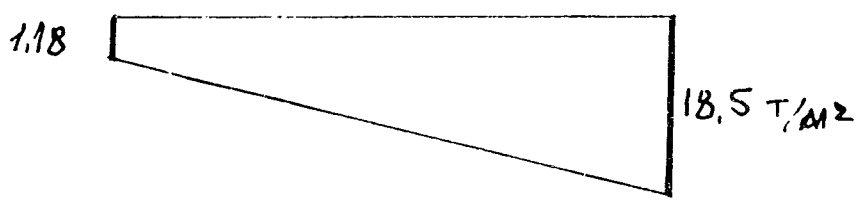
Calculation table

Source	Force Designation Symbol	Formula Horiz. forces	Active or Passive cost [Ka/Kp]	Heights h [M]	Unit Weight γ [T/M3]	Horiz. Force ---> + <--- [T]	Arm [M]	Overturning Moments − + Ton meter	Remarks
	HW1	$\gamma_w h_1 h_2$	Hydrau-static	$h_1=1.9$ $h_2=5$	1	-9.5	2.75	-26.125	
	HW2	$0.5\gamma_w h_2^2$	—"—	$h_2=5$	1	-12.5	1.67	-20.875	
	HW3	$0.5\gamma_w h_3^2$	—"—	$h_3=1.5$	1	1.125	0.5	0.563	
	He1	$0.5\gamma_e h_{2a}^2 \cdot Ka$	Ka=0.3	$h_{2a}=3.5$	2.02	-3.71	2.67	-9.906	
	He2	$\gamma_e h_{2a} ka\, h_{2b}$	Ka=0.3	$h_{2a}=3.5$ $h_{2b}=1.5$	2.02	-3.18	0.75	-2.385	
	He3	$0.5\gamma_e h_{2b}^2 Ka$	Ka=0.3	$h_{2b}=1.5$	0.91	-0.31	0.5	-0.155	
	Hey	$0.5\gamma_e h_4^2 Kp$	Kp=3	$h_4=1$	0.91	+1.365	0.333	0.455	
Net total horizontal force						-26.71			
Net Overturning moment								-58.43	

C: Gabionh

$$h_{equivalent} = \frac{58.43}{26.71} = 2.187\ m.$$





1. Checking against overturning :

$$F.S = \frac{M_R}{M_o} = \frac{207.11}{58.43} = 3,61 > 1.50 \quad OK.$$

2. Checking against sliding :

$$F.S = \frac{\Sigma W_i \times 0,6}{\Sigma H_i} = \frac{64.79 \times 0,6}{26.71} = 1.455 > 1.4 \; OK$$

3. Checking of bearing capacity :

Distance $a = \frac{\Sigma M_i}{W} = \frac{207.11 - 58.43}{64.79} = \frac{148.48}{64.79} =$

$a = 2,295 M.$

Eccentricity

~~Excentricity~~ $e = \frac{d}{2} - a = 3,25 - 2,295 =$

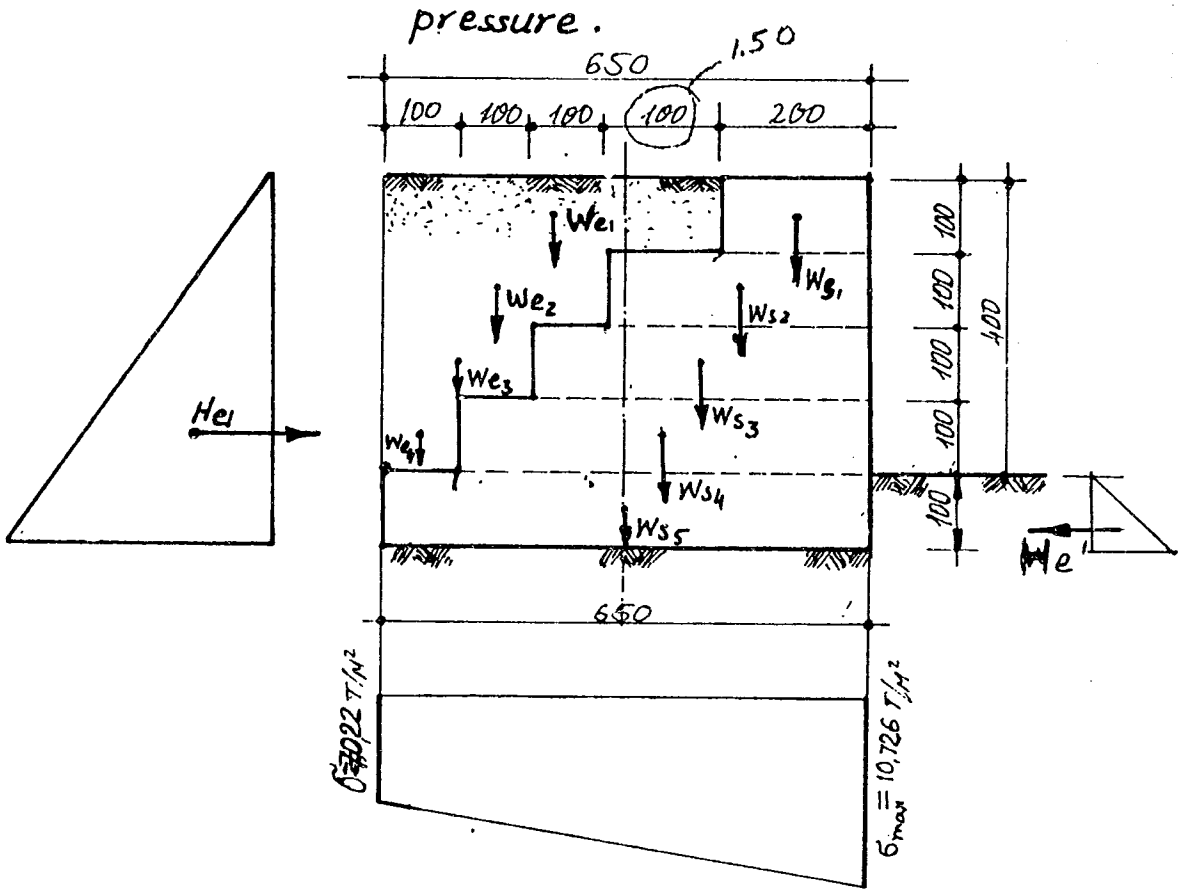
$e = 0,955 M.$

$$\sigma_{\substack{max\\min}} = \frac{W}{d}\left(1 \pm \frac{6 \cdot e}{d}\right) = \frac{64.79}{6.50}\left(1 \pm \frac{6 \times 0.955}{6.50}\right)$$

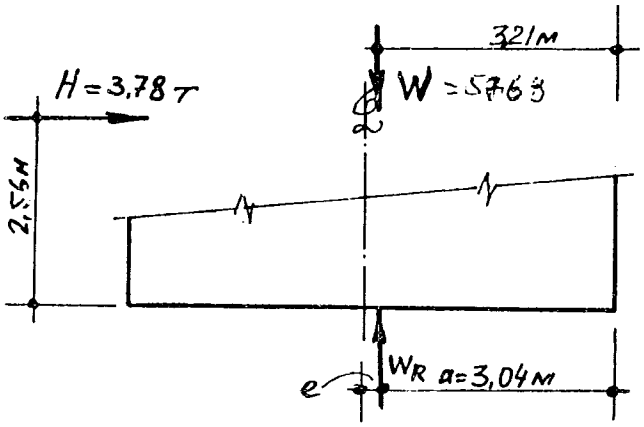
$\sigma_{max} = 18.75 < 28$ OK

$\sigma_{min} = 1.18 << 28$

Figure 2. : Checking the Dam ~~just in~~ against dry earth pressure.



Bearing diagram (Force Diagram)



Calculation table

Source	Force Symbol Ltr. Ref.	Dimensions [M]	Area [M2]	Unit γ [T/M3]	Total W [Tons]	Arm [M]	Moment (abt. Pt.B) Stabil. + Overturning − Ton Meters	Remarks
Earth	W_{e1}	1x4.5	4.5	1.68	7.56	4.25	32.12	+
	W_{e2}	1x3	3	1.68	5.04	5.0	25.20	
	W_{e3}	1x2	2	1.68	3.64	5.5	18.48	
	W_{e4}	1x1	1	1.68	1.68	6.0	10.08	
	W_{s1}	1x2	2	1.82	3.64	1.0	3.64	
	W_{s2}	1x3.5	3.5	1.82	6.37	1.75	11.15	
	W_{s3}	1x4.5	4.5	1.82	8.19	2.25	18.43	
	W_{s4}	1x5.5	5.5	1.82	10.01	2.75	27.53	
	W_{s5}	1x6.5	6.5	1.82	11.83	3.25	38.45	
Total weight (Resultant force)					57.68 T		185.08	
Net Stabilizing moment							185.09	

C: Gabion

$$a = \frac{185.09}{51.68} = 3.21$$

Calculation table

Source	Force Designation Symbol	Formula Horiz. forces	Active or Passive cost [Ka/Kp]	Heights h [M]	Unit Weight γ [T/M3]	Horiz. Force ---> + <--- - [T]	Arm [M]	Overturning Moments or Stabil mom. Ton meter	Remarks
Earth	He1	$0.5\gamma_e h_2^2 k_a$	$k_a=0.3$	$h_2=5$	1.68	-6.3	5/3	-10.5	↻ -
	He2	$0.5\gamma_e h_4^2 k_p$	$k_p=3$	$h_4=1$	1.68	+2.52	1/3	+0.84	↺ +
	Net total horizontal force					-3.78			
	Net Overturning moment							-9.66	↻

C: Gabionh

$$h_e = \frac{9.66}{\gamma} = 2.56$$

1- Overturning stability:

$$F.S = \frac{M_R}{M_o} = \frac{185.09}{9.66} = \underline{\underline{19.16 >> 1.5}}$$

2- sliding:

$$\sum W_i = 57.68\ T$$

$$\sum H_i = 3.78\ T$$

$$F.S = \frac{57.68 \times 0.6}{3.78} = \underline{\underline{9.15 >> 1.4}}$$

3- Bearing capacity:

$$\sum M_i = 185.09 - 9.66 = 175.43\ T.M$$

$$a = \frac{\sum M_i}{\sum W_i} = \frac{175.43}{57.68} = 3.041\ m$$

$$e = \frac{d}{2} - a = 3.25 - 3.04 = 0.21\ m$$

$$M = \sum W_i \times e = 57.68 \times 0.21 = 12.11\ T.M$$

$$\sigma_{\substack{max \\ min}} = \frac{57.68}{6.5} \pm \frac{12.11 \times 3.5}{22.885} =$$

$$\sigma_{max} = 8.874 \pm 1.852 = 10.726\ T/m^2 \ll 28$$

$$\sigma_{min} = 8.874 - 1.852 = 7.022\ T/m^2$$

The structure is over designed

Redesign of structure against soil pressure

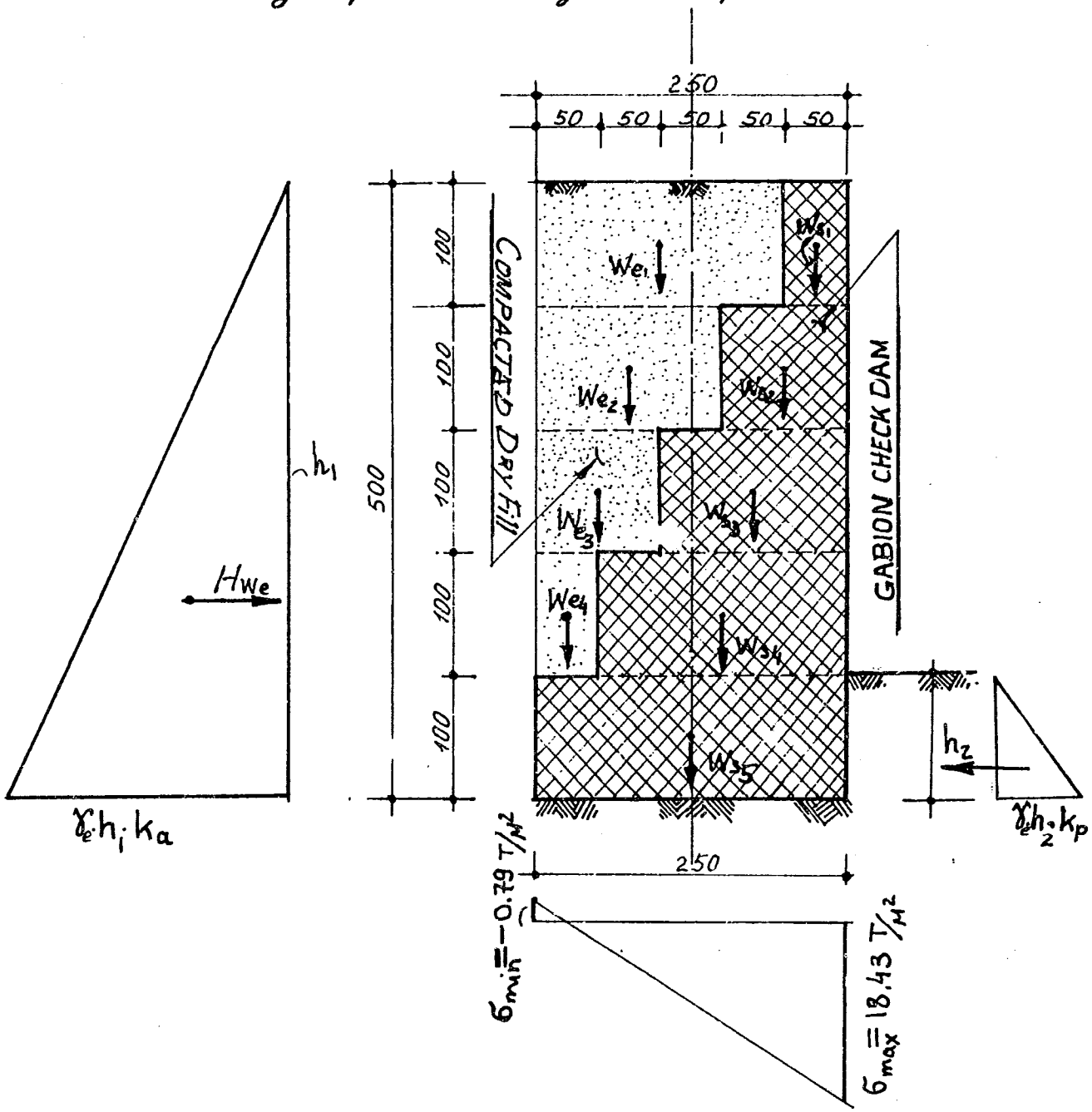


Figure (2): GABION Ret. wall. ~~STRAUCTUR~~ Structure AND DIAGRAMS

Calculation table

Source	Force Symbol Ltr. Ref.	Dimensions [M]	Area [M2]	Unit γ [T/M3]	Total W [Tons]	Arm [M]	Moment (abt. Pt.B) Stabil. ↶+ Overturning ↷- Ton Meters	Remarks
	We1	2 x 1	2	1.68	3.36	1.5	5.04	↶
	We2	1.5 x 1	1.5	1.68	2.52	1.75	4.41	↶
	We3	1 x 1	1	1.68	1.68	2.0	3.36	↶
	We4	0.5 x 1	0.5	1.68	0.84	2.25	1.89	↶
	Ws1	0.5 x 1	0.5	1.82	0.91	0.25	0.228	↶
	Ws2	1 x 1	1	1.82	1.82	0.5	0.91	↶
	Ws3	1.5 x 1	1.5	1.82	2.73	0.75	2.048	↶
	Ws4	2 x 1	2	1.82	3.64	1	3.64	↶
	Ws5	2.5 x 1	2.5	1.82	4.55	1.25	5.68	↶
	Total weight (Resultant force)				22.05			
	Net Stabilizing moment						27.206	↶

C: Gabion

$$a_1 = \frac{27.206}{22.05} = 1.23 \text{ m}$$

Calculation table

Source	Force Designation Symbol	Formula Horiz. forces	Active or Passive cost [Ka/Kp]	Heights h [M]	Unit Weight [T/M3]	Horiz. Force ---> − <--- + [T]	Arm [M]	Overturning Moments or stabil. Mom. Ton meter	Remarks
	H_{e1}	$0.5\gamma h_2^2 Ka$	$ka = 0.3$	$h_2 = 5$	1.68	−6.3	1.67	−10.5	↷
	H_{e2}	$0.5\gamma_e h_4^2 K_p$	$Kp = 3.0$	$h = 1$	1.68	2.52	0.333	0.84	↶
Net total horizontal force						−3.78			
Net Overturning moment								−9.66	↷

C. Gabionh

[illegible] / 3.78 = [illegible]

1. Overturning Stability:

$$F.S = \frac{M_R}{M_O} = \frac{27}{9.66} = 2.8 > \underline{1.5 \quad C.K}$$

2. Sliding control:

$$F.S = \frac{22.05 \times 0.6}{3.78} = 3.5 > 1.4 \quad OK$$

3. Bearing capacity:

$$a = \frac{\sum M}{\sum W_i} = \frac{27.206 - 9.66}{22.05} = \frac{17.546}{22.05} = 0.796\,m$$

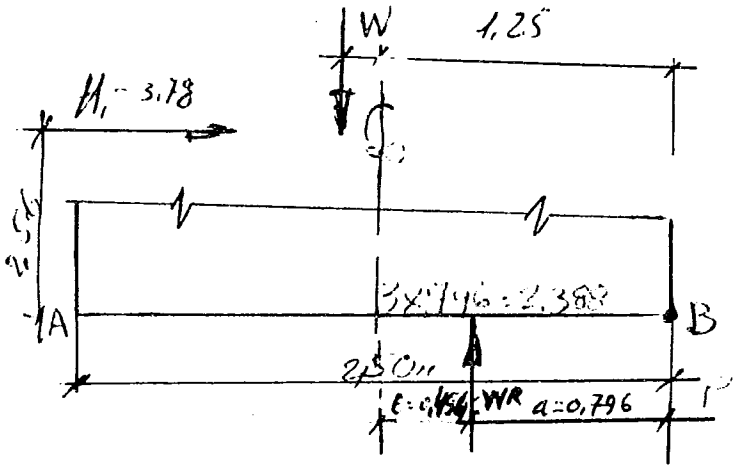
$$e = 1.25 - a = 1.25 - 0.796 = 0.454\,m$$

$$\sigma_{\substack{Max\\Min}} = \frac{22.05}{2.5}\left(1 \pm \frac{6 \times 0.454}{2.5}\right) =$$

$$\sigma_{max} = +18.43\ T/m^2 < 28.7\ T/m^2$$

$$\sigma_{min} = -0.79\ T/m^2$$

Approximate. Not exactly cannot because outside middle third - more accurate triangular distribution used



$22.05 = .5 \times 2.388\,P$

$P = \frac{22.05}{.5 \times 2.388}$

$P = 18.43\ T/m^2$

Note: We did not go further to decrease the size of retaining wall; because the resultant of soil pressure is out of the middle third of the foundation base, ~~So~~ We didn't want to increase the excentricity and move the resultant force ~~more~~ further away from the middle third.

Good