# DRAINAGE AND DRAINAGE STRUCTURES OF BEIRUT-DAMASCUS TURNPIKE.

Edward Balabanian

1953 Copy 1

SCHOOL OF ENGINEERING

PROJECT REPORT



AMERICAN UNIVERSITY OF BEIRUT

# DRAINAGE AND DRAINAGE STRUCTURES
# OF
# BEIRUT-DAMASCUS TURNPIKE

By

Edward Balabanian
Engineering IV

April 27th, 1953

AMERICAN UNIVERSITY OF BEIRUT
SCHOOL OF ENGINEERING

*****
***
*

# PART I

# D R A I N A G E .

## Introduction

Drainage of highway, strictly speaking, refers to the removal of surplus water within the highway itself and satisfactory disposal of it. The term is commonly used, however, to include also the prevention of water from reaching the highway and the controlled movement of water along and under the highway.

The water involved may be that precipitation directly upon the highway itself; it may be the surface run-off from adjacent areas; it may be underground water moving through subterranean channels or strata, or it may be moisture rising by capillarity from the water table underneath the roadbed.

The nature and extent of drainage that must be provided will vary:

1. with the type of soil on which the road is built.
2. the precipitation in the area.
3. the topography.

## Types of Drainage Problems

1. Surface drainage. It involves the disposal of storm or snow water in so expeditious a manner that the

percolation into the soil immediately under the road surface will be reduced to a minimum. Its standard solution is one of providing open channels to carry the storm water from the proximity of the travelled part of the road.

2. the second type of drainage problem is the prevention of the flow or percolation of water, other than that which is being carried in the ditches, into the soil in the upper 50 or 75 cm of the subgrade under the road surface. This water, of course, originates in precipitation but sinks into the soil and collects in underground pockets of porous materials or flow in underground seams and porous layers, sometimes for great distances. It is usually referred to as "free water" to differentiate it from water held in the soil by surface tensions or capilarity. Free water is a prolific cause of landslips along the side slopes of cuts or embankments that have been made in grading the highway. It may also serve as a reservoir to supply water that seeps into the soil immediately under the roadway surface or is drawn in by capillary action, rendering the supporting subgrade unstable.

## Surface drainage

Surface drainage is considered to be the removal, control and disposal of water which has been precipitated directly upon the surface of the roadway and immediately adjacent areas. It also includes the water produced by melting snow and ice on these same areas.

Pavement surface. The pavement should always be as watertight as practicable to make it in order to prevent the water on the surface from entering. It should be sufficiently smooth and have sufficient crown so that the water will flow to the edges rather than stand in puddles on the surface or flow longitudinally. If steps are not taken to prevent such action the water flowing off the pavement will seep downward at the edge of the pavement to the subgrade below, where it may result in frost heaving, mud pumping or subgrade unstability.

Shoulders. The water on the shoulders, whether falling there directly or running off the pavement may be prevented from causing damages by waterproofing the shoulders with asphalt or other suitable material. As an alternate the shoulders may be constructed of materials through which the water may rapidly pass downward and outward without causing damage to the shoulder or seeping into the subgrade. Densely graded granular material is best for this purpose as it does not become unstable when wet.

Side ditches. The surface drainage of highway is usually accomplished by side ditches which flank the portion of the roadway that carries the traffic. In most instances a simple side ditch is all that is required, but sometimes special provisions are needed for the effective removal of a very large amount of storm water that may be expected during certain seasons of the year.

The bottom of the ditch should be at least 25 cm lower than the subgrade elevation in order to drain water from the base course under the pavement and to provide snow storage space. To prevent water standing in the ditches the flowline should be smooth and have a fall of preferably not less than 0.2 % for paved ditches and 0.5 % for unpaved ditches. If the ditch is in easily eroded material and on a steep grade, it should be lined with sod, crushed rock, gravel or other locally available erosion resisting materials.

Outlets should be provided at intervals frequent enough to avoid erosion or exceeding the capacity of the ditch. In general the outlets should be not more than 200 m apart for unlined ditches. A suitable inlet or headwall should be constructed to permit the water to pass readily from the road ditch to the storm sewer or cross-drain pipe below. At the end of a cut the water from the ditch should not be turned loose to flow down the toe of fill slope, but to reduce erosion, a definite channel, sodded or paved if necessary, should be provided to take the water to a natural drainage channel or culvert.

Generally there are two shapes of side ditches used:

1. V-shaped which is used when the capacity of water is not great and when there is no traffic into the farms during the dry season of the year.

2. Trapezoidal, which is used when the capacity of water is great. This type has advantage over the V-shaped since it can secure great capacity without the necessity of using a depth which will introduce side-slope problems. The various designs used for side ditches are shown in figure II

Run-off. The amount of water to be carried by the side ditches along the highway is the run-off from the area contributing thereto. Primarily this water comes from the portion of the road between the ditches, upon which there is more or less impervious wearing surface, and from the earth shoulders flanking the roadway surface, which are usually fairly hard. It may be assumed that for storms of short duration, about 75 % of the water that falls on the area between side ditches will run to the ditches and that for storms of exceeding 15 minutes in duration all of the water falling on this area will run to the ditches.

Some water will reach the road ditches from the area between the ditches and the right of way lines and from adjacent lands which slopes toward the ditches. The rate of run-off from this area will depend on:

1. slope
2. the character of soil
3. nature of the growth

For general purposes it may be assumed that for storms of more than 40 minutes' duration and for short downpours at very high rates all the water falling on the drainage area will reach the side ditch. Knowing the total area, the rainfall characteristic of the region, and the slope that may be obtained for the ditch, the area of cross-section may be computed by determining the velocity by means of Chezy formula, using Kutter's formula for determining C.

Although under many conditions it is not necessary to go to the requirement of computing the ditch capacity it is imperative to provide sufficient longitudinal slope to insure that the ditch empties quickly.

Supplementary ditches. It is often desirable to intercept, by means of a supplementary ditch, roughly parallel to the road, water flowing from the adjacent lands towards the road. Such water is frequently the cause of landslips in highway cuts. A supplementary ditch is especially advantageous where the amount of drainage area to be provided against is large. The slopes of cuts can be protected in this way by means of supplementary ditches few meters back of the upper edge of the side slope. The use of supplementary ditches is illustrated in figure II

Tile to supplement side ditches. If side ditches on flat grades must carry storm water for long distances and in considerable quantities, it is to be expected that during rainy weather the ditches will impound water for a considerable period of time. This creates a condition favorable to the percolation of water into the subgrade in quantity sufficient to lower appreciably the supporting strength of the subgrade soil. To expedite the removal of the ditch water and thus minimize this hazard, the drains are laid below side ditches to supplement the ditch capacity. To be effective, such a tile msut be of ample size and have a free outlet so that full advantage may be taken of the capacity.

The principle advantage of supplementary tile is realized when soil or accumulation of trash partially block the ditches, thus preventing the free flow of the storm water along the ditch. It will escape through the inlets, rather slowly to be sure, and thus eliminate impounding in the ditch. In the area of considerable snowfall, there will usually be a period during spring that when the ditches are only partially effective, and considerable water will be held in the ditch if tile is not used or if there are not adequate inlets to the tile.

Drainage of embankments. The surface drainage of road surface located on an embankment is not much of a problem when the fill is not more than about 2 m high and composed of carefully compacted soil. The low fills can be built with a reasonable flat side slope which, in humid areas, will soon be covered with grass which will protect the slope from erosion. When the side slope is made of sand and when the embankment is more than 2 m, the maintenance of the side slope becomes a troublesome problem. Storm water that flows down the side slope of the fill will carry away great quantities of soil when preventive measures are taken. This type of erosion may be minimized by constructing a ridge or low curb along the edge of the roadway to confine the water that falls on the top of the embankment and carry it to paved flumes arranged to carry the water down the side slopes at appropriate intervals.

## Underground Drainage

Storm water that percolates into the soil flows downward and laterally, its course depends upon a great many conditions of soil and topography. The underground water flowing freely through the soil may be the cause of unstability in the soil under a road surface and very often is the underlying reason for the failure of the wearing surface to with-stand the pounding of traffic. The level of free underground water may be held to such a depth that it will not jeopardize the stability of soil supporting the road surface it properly designed tile drains are employed.

It will be convenient to consider 3 distinct conditions that are encountered in designing tile underdrains for highways.

The first condition is that the tile is employed to supplement the surface drainage in addition to lowering the ground water level

The second condition is that which is encountered in relatively flat country where the water must be carried for long distances to reach an outlet, and where the soil is of such a type that it reaches a state of saturation rather quickly after a storm.

The third condition is one that encountered in hilly country more frequently than elsewhere but may exist under any topographical condition. This particular condition is manifested by seepages of water in the most unexpected places and by partial saturation of road surface in locations remote from accumulation of surface water.

The drainage requirement of a road where such conditions exist will be exceedingly variable and can be determined only by a careful survey prior to begining construction work and during the process of construction, using test-hole experiment whenever ground water is a probability. The drainage method applicable to all these locations is to provide tile lines to intercept the flow of water to an outlet remote from the road surface. The size of the tile to be used in such a case is largely a matter of judgement as there is no reliable basis for computing it. Obviously, it would be unnecessary to lay tiles when the road surface is built on the porous soils such as sand and on the very sandy loans.

There are three objectives to be attained in an underground drainage:

1. removal of stationary free water in the soil by lowering the water table and providing outlets for basins surrounded by impermeable material.

2. collecting and disposing of water arising in springs under the road or seeping down from the surface of the highway.

3. intercepting seepage water from outside sources before it reaches the highway area.

Types of underdrains: An underdrain usually is a trench in which a pipe line of clay tile, concrete tile or perforated corrugated metal is laid, the trench then being backfilled with porous material. Sometimes the pipe line is omitted in which case the backfilled trench is called a blind drain or French drain. The type of drain to be used will depend upon the source and the volume of water to be handled, the availability and cost of the pipe, and the chemical properties of soil and water.

Location of underdrains. If the water is coming toward the road from one side only, a single drain on that side to intercept the water before it reaches the road will be sufficient. If the highway is in a valley where seepage comes from both sides or if the intent is to lower the water table, drains may be required on both sides of the highway. If the water is moving along on impervious layer the trench should extend 2-3 cm into the layer. If it is held in a basin by impervious material, the bottom of the trench should be slightly below the low point of the basin. If the removal of free water is desired, the drain should be deep enough to prevent the water rising by capillarity above the desired elevation. Generally the drains should be at least 1 m underground and in areas subject to freezing temperatures they should be low enough to hold the water below the depth to which frost will penetrate.

Size of underdrains: The ordinary range of size of pipes for underdrains is 4-12 inches in diameter, with the 6-inch and 8-inch sizes predominating in usage. Smaller pipes than 4-inch size are generally undesirable because of likelihood of their becoming clogged.

Size and location of perforations: When perforated metal is used best results will be attained by double or triple lines of holes about 0.75 cm in diameter and spaced approximately 3 cm centers on each side of the pipe and in the area just above a line 30 degrees down from horizontal axis. There should be no perforation in the bottom third of the pipe so that there will be no outward seepage from the pipe where it passes through dry material. Holes in the top half or third are unnecessary and should be omitted to lessen the opportunity for backfill material to enter.

Filling trench. Trenches for perforated pipe should be filled to the level of the perforation with well-tamped impervious material and the remainder of way to the top with porous filler material;for nonperforated pipe the porous filler material should extend from the level of the bottom of the pipe to the top of the trench.

The filter material in the main body of the trench should be well tamped and should generally consist of sand all of which pass a 3/8 inch sieve and is retained on a No. 40 sieve. It is desirable that fine rather than coarse material be used because the coarse materials will be clogged with clay or silt working in from the sides or top of the trench until the drain is useless .

If the backfill is too fine the moisture film surrounding the individual particles may merge and completely fill the pore spaces so taht no water will pass through the filter. Also if a densely graded material is used the small particles will fill the interstices between the larger particles and clog the filter. In no case should a pipe be placed on a bed of porous material because the water bebeath the pipe cannot enter it and so will either flow along under the pipe as in a blind drain or be trapped under the pipe to cause a permanently wet condition.

Vertical sand drains. Occasionally a layer of water-bearing clay, silt or mud supplied by underground streams may be situated under the area to be occupied by an embankment and may extend to such a depth below the surface that it cannot be drained by horizontal underdrains because of lack of outlet. Nevertheless, it must be drained or it will be a constant threat to the stability of the embankment. In such cases vertical sand drains with horizontal outlets at the natural ground surface may be a satisfactory remedy. The vertical drain is constructed by drilling vertical holes from the ground surface to the bottom of the water-bearing layer and filling the holes with sand. A still casing should be driven to follow the drill to the bottom. Then as sand is poured into the hole to completely fill it, the casing should be gradually withdrawn. The size and spacing of the holes will depend upon particular condition at the site. At the ground surface the vertical drains should connect with a system of horizontal perforated pipe, tile or blind underdrains which will carry the water outside the roadway area.

Subgrade drainage : The purpose of subgrade drainage is to minimize the effect of water upon the load-supporting capacity of the material upon which the roadway surface is constructed. It is important in this connection to differentiate between free water and water that is held by capillarity. Free water being in motion, the quantity in the pores of the soil at any time depends upon the texture of the soil, the supply of free water available for replenishing the evaporation loss and the extent of evaporation.

Suitable underdrain system will minimize the deleterious effects to eliminate capillary water and in humid regions the soil under the road crust is usually moist, even in dry weather. If the soil is a type that swells when wet, this promiscuous wetting in small areas will cause unevenness in the road surface. Even concrete slabs will be lifted by the swelling of the soil. Types of soil that do not swell will be softened sufficiently to effect the supporting strength of the area reached by this free water. Good maintenance of cracks and joints will minimize the infiltration of water.

Relation of drainage to frost action. The surface manifestation of the disturbance of a road surface from the expansion of freezing water in the supporting soil is frequently an irregular dome-shaped bulge which becomes unstable as soon as the supporting ice melts. From its characteristic appearance this has been dubbed as "frost boil". A road surface of any flexible type on top of or adjacent to a frost boil is rendered unstable and often impassable when the ice melts, and even heavy wearing surface are seriously damaged in some cases.

So far as is now known, two methods are available for minimizing the disturbance of road surface from the expansion of freezing water in the supporting soil. The first is to provide drainage that will prevent free water under a slight hydrostatic head flowing under the road in seams and then rising in subgrade. The second method is one that must be resorted to when the subgrade is composed of types of soils from which water cannot be removed by underdrainage. It involves removing the soil and replacing it with more stable material.

## Cross-road Structures

Drainage conduits are concrete or metal pipe, burned clay or cement concrete tile, and monolithic concrete culvert barrels.

Culverts. The tem culvert is used by engineers to designate the structure employed to carry water through highway embankments or under a roadway surface. In this treatise the discussion of culverts will be limited to those drainage channels which are entirely burried in the embankment.

The function of culvert is to carry water from one side of travelled way to the other and the controlling factors in the design of culverts are :

1. provision of sufficient area of waterway to accomodate the flow of water through the culvert.

2. structural strength sufficient to carry the weight of the fill and the traffic loads that pass over the culvert.

Ordinarily the culverts are circular or square in cross-sectional area, but where headroom is limited a rectangular cross-section may be employed in which the height of the opening is much less than the width. Under similar circumstances of restricted headroom a battery of circular culverts may be employed instead of a single circular larrel.

Size of openings; The structure opening should be amply large to handle the volume of water without seriously impeding the flow and to permit the passage through it of boulders, logs and debris carried by the stream. The required size may be determined by calculating the drainage area. Even though a small amount of water is involved, it is essential, in order to provide for cleaning of the culvert, that, except for underdrains no culvert having a diameter less than 24 inches be installed under the highway.

In the determination of the size of culvert opening, consideration should be given to the fact that the normal flow of most streams is often considerably exceeded at infrequent intervals when cloudbursts or flood occur. It must be decided whether to pay the heavy initial cost to provide a structure adequate for the extreme conditions or to provide a smaller, less costly culvert and suffer the consequences when the occasional flood waters exceed its capacity. The decision will depend upon the circumstances in each individual case.

<u>Culvert inlet.</u> The headwall, wings and other appurtenances at the culvert entrance have the following important functions to perform:

1. to retain in its proper position the natural ground and ambankment slopes near the entrance.

2. to prevent the water from undercutting or bypassing the structure and from seeping into the roadway.

3. to cause the transition of the cross-section of the stream from the shape of the approach channel to the shape of the culvert.

4. to provide for entrance of the water into the culvert with maximum efficiency.

5. to prevent erosion of the approach channel.

Ideal inlet structure: As a rule, the above functions will be best performed if the entrance structure includes the following features:

a. headwall and wings extending high ehough above the ground slope to prevent materials from sloughing into the inlet basin.

b. wings extending well upstream and flared and warped to form the best transition from the approach channel cross-section to the culvert cross-section.

c. an apron extending the full length of the wings with a cut off wall deep enough to prevent undercutting.

d. steeper grade on apron than on the channel bed beyond to increase the velocity of the stream before the more restricted area of the culvert is reached.

e. rounded-lip, beveled-edge or flared entrance to the culvert itself.

f. intersection of the wings and the headwall face flush with the edge of the culvert.

g. face of the headwall perpendicular to the axis of the stream.

Although such construction will be ideal and the most desirable type under critical conditions, it may be impractical or the cost may be exorbitant in many cases and frequently some other type will serve satisfactorily.

For small velocity streams with either direct or undefined approach, a straight headwall without wings or apron will be satisfactory and because of low cost and ease of construction will be most desirable.

Culvert outlets: The objectives to be accomplished at the outlet of a culvert are different from those at the inlet and, except perhaps for appearance and simplicity of design and constructions there is no reason for the outlet to be the same as the entrance. The natural ground and embankment slopes must be prevented from encroaching on the channel and together with the side and bottom of the channel must be protected from erosion. Pipe culverts carrying small or low velocity streams, or having submerged outlets, or discharging onto rock, paved spillway or other stable materials are generally satisfactory without any endwall whatever. If no wall is used the pipe should extend beyond the embankment slope far enough to avoid undercutting or eroding the slope. If the end of the culvert is on filled ground and endwall structure may be desirable because settlement of the fill might leave the endwall without support.

Usually some type of endwall is needed, especially if an abrupt change in direction of flow is required or if the channel or natural ground has little resistance to erosion. Ordinarily the discharge velocity of the culvert is considerably higher than normal velocity of the stream. Also the release of the water from the restricted cross-section of the culvert to the wider and different shape channel beyond causes turbulence and eddies which together with the high velocity may result in serious erosion if not controlled. A straight headwall without wings and apron is useless for this purpose. Wings parallel to the direction of the flow are merely a continuation of the culvert sidewall and so accomplish nothing towards restoring the stream to its natural flow. For best results the wings should be flared and warped but if the flare angle is too great the eddying action may be increased rather than decreased. Considerable reduction in velocity may be obtained by means of vertical drops from the culvert floor to the apron especially if a low curb is then constructed across the end of the apron so that a basin is formed. The velocity may also be reduced by making the grade of the apron less steep than that of the culvert.

Types of culverts. Culverts are constructed of a number of kinds of materials and in several forms. Any one of these may be used according to convenience and cost of construction in a given locality. It should be certain that the structural strength of the particular type selected is adequate for the load it will be called upon to sustain in that location.

Reinforced concrete culverts: Reinforced concrete is a desirable and widely used material for culverts because the structure is monolithic and consequently particularly satisfactory for culverts, where the rectangular section is desired and for culverts that pass through side hill fills on considerable slope or for culverts that are built with drop inlets.

Materials for this type of culverts are obtainable in almost every locality and can be transported to the site at low cost because they are carried in small units. The strength of the concrete culvert can be adapted readily to the location and load to which it will be subjected.

Inverted siphons: Occasionally both the inlet and the oulet of a culvert must be at a higher elevation than is possible for the flowline elevation under the highway itself. Resort is then taken to what are commonly known as vertical siphons although this is not a correct designation because no siphonic action is involved. Vertical wells are constructed at each end of the culvert to pass the water down from the inlet channel to the culvert and back up to the outlet channel which must be lower than the inlet channel by the amount of head lost in the structure. Both the wells should extend below the culvert flowline to provide basins for sediment and provision should be made to permit cleaning the wells. Because of the loss of head, the danger of obstructing the waterway, the difficulty in cleaning and the stagnant water when flow is intermitted, inverted siphones are undesirable and should only be used when no better solution is practicable.

Debris control devices: Streams frequently carry with them, either floating or rolling along the bottom considerable quantities of debris ranging from leaves, weeds and small twigs to large logs and boulders. Some provision should be made at or near the culvert entrance to reduce the interference of such debris with the satisfactory operation of the structure.

For small structures the best barrier will usually be a grating or grillwork of steel or timber constructed directly across the culvert entrance. The spacing of the barrier members should be such that nothing will pass larger than the culvert can handle. The barrier should preferably be high enough to prevent the water and floating debris going over the top even when the water level is raised because of the obstruction of the lower portion of the barrier by debris, but in many cases such construction will not be feasible.

## Stream Channels

Importance of erosion prevention: Because of the danger of damage to the highway itself and also to adjoining lands, it is essential that provision be made to prevent erosion of adjacent streams channels. This is especially necessary when new channels have been created or when old ones have been changed or disturbed. A channel which has existed for a number of years has usually become so well established that it is little affected by the erosive force of the stream. If the alignment, grade or cross-section is changed, or if the channel is partly filled so that the stream is crowded into a narrower channel, the new currents and eddies developed, the changed velocity and the new surfaces exposed to the stream may be the source of considerable trouble if precautionary measures are not taken.

When a new channel or revised channel is necessary, abrupt changes in the alignment and grade should be avoided, the bed and the banks should be reasonably smooth, and the cross-section should be as uniform as the conditions permit. The grade line will usually be controlled by ground conditions but preferably should be the minimum required for satisfactory drainage. Occasionally adequate protection can be provided only through the use of such special construction features as lining, riprapping, cribbing, retaining walls, check dams and vegetation.

Lining; A non-erosible lining of some type may be essential in special cases if the grade is steep, if the velocity is high, or if the material of which the channel is composed is unstable or pervious. The lining may be of portland cement concrete, stone masonry, grouted rubble or bituminous material.

Riprapping: Riprapping is particularly adapted for use when only intermittent short sections on one or both sides need protection such as curves, at structure approaches or outlets, where the highway encroaches on the original channel or where channel changes have been introduced.

Check dams: When steep grades for channels cannot be avoided, erosion can be prevented by construction of check dam. They consist of series of dams built across the channels so that the flow-line grade between dams does not exceed the maximum which can be used without causing erosion. The dams may be constructed of concrete, masonry, metal, wood or any other suitable material.

Vegetation: Trees, brush, sod and other vegetation along the stream banks will assist considerably in the control of the erosive forces and at the same time improve the appearance. Their growth should therefore, be encourages provided they do not detrimentally impede the flow nor cause debris to become lodged. If there is insufficient natural growth, planting may be desirable.

## Foundation & Excavation

Foundation for structures. It is essential that the foundation under a structure provide support as firm and as nearly uniform as possible under the entire bearing surface. Whenever conditions permit, the bottom of the excavation should be on solid ground for its full length and width. If it can be provided culverts should not be placed partly on filled ground and partly on undisturbed natural ground because of the probability of unequal settlement which might distort or break the structure. This applies transversely as well as longitudinally and when a side-hill location is used the culvert should be benched into the hillside far enough to be entirely on solid ground. If part of the culvert must be on filled ground, the filled material should be placed in thin layers thoroughly compacted so that it will provide a foundation as nearly comparable to that afforded by the natural ground as possible.

The installation of drainage structures or systems in embankments should be avoided whenever practicable to do so because of the possibility of not providing a firm foundation and of settlement which will cause breakage of the structure or low spots which would not drain. When such an installation must be made the embankment should be constructed and thoroughly compacted to a height at least three feet above the elevation of the bottom of the structure. The excavation should then be made in the compacted fill.

Unstable foundation material should be removed and replaced with satisfactory material to the extent practicable. If this cannot be done reasonably, a layer of sand, gravel or other suitable material should be placed on the foundation and worked into the unsatisfactory material until a stable foundation is formed. If a pipe culvert is placed in rock excavation the rock should be removed slightly below the elevation of the bottom of the pipe and a well-compacted cushion of gravel, sand or other suitable material should then be placed as a bed for the pipe.

<u>Excavation for channels, ditches and structure inlets and outlets.</u> Economy of construction will usually require that the excavation for stream channels, drainage ditches and structure inlets take care of the water to be handled. In some cases, however, it will be more economical to construct a larger excavation in order to make efficient use of mechanized methods instead of doing

the work by manual labor.

Excavation for structure and underdrains: The excavation for a structure should be adequate to accomodate the structure to be installed and should provide sufficient working space and room for forms and bracing if required. Economy will generally dictate that the excavation be the minimum necessary but this is usually desirable for stability also because the undisturbed natural ground under and at the sides of the structure will ordinarily furnish better support for the structure itself and the embankment to be constructed over it, than will backfill which may be placed instead.

The trench for an underdrain should be at least 15 cm wider than the diameter of the pipe, the width ordinarily being the minimum which it is practicable to dig. If dug by hand labor the trench must have a minimum width of 40 cm as a man cannot work efficiently in a narrower width.

## Installation of Drainage Structures

Installation of pipe culverts and underdrains: Pipes may be lowered into position by means of ropes only. When this method is used the pipe is placed alongside the trench directly above its final position. Long ropes are passed under the pipes, one near each end with addition rope at the intermediate points if needed. One end of each rope is anchored and the other end is manipulated and paid out with the pipe rolling in the bight until it reaches the bottom of the trench. Cranes mounted on the tractors or trucks may be needed for the larger pipes. The tractor mounted cranes are especially useful in rough terrain where the pipes cannot be delivered by truck to the installation site. The tractor crane can then be used to carry the pipe to the site as well as to lower it into position.

Pipes for culverts and underdrains should be carefully placed so that they will be accurately aligned both vertically and horizontally and will rest on a sufficently firm and uniform bed to prevent settlement or displacement. Bell and spigot pipe should be placed with the bell ends upgrade and with the spigot ends fully entered into the adjacent bells. The outside laps of circumferential joints of corrugated metal pipe should point up grade and the longitudinal laps should be on the sides.

Backfilling: Backfill material should be the best available in order that uniform bearing may be provided. Granular material is preferable and in any event the material should be free from large stones, lumps and rubbish. To obtain uniform pressure against the pipe or structure the backfill material should be placed in layers about 15 cm thick and thoroughly compacted, water being added if necessary to bring the material to the optimum moisture content for maximum consolidation. To avoid displacing or unduly stressing the structure backfilling should be done simultaneously on both sides.

In the case of pipe culverts there should be a berm of compacted material on each side of the pipe at least equal to the diameter of the pipe, except in so far as undisturbed material obtrudes upon this area, and the compacted backfill should extend at least 30 cm and preferably a distance of two diameters, above the top of the pipe. For structure such as box culverts, abutments and retaining walls the berm of compacted material should extend behind the structures a distance at least equal to the height of the wall being backfilled against, except insofar as undisturbed material obtrudes upon this area. Especial care should be given to tamping material under the haunches of pipes.

The backfilling for trenches and other small areas should be deposited by hand shovels and compacted in thin layers by hand tampers or mechanical tampers. Whenever space may permit the backfilling may be done by means of tractors and bulldozers, special backfilling attachments for tractors and power shovels, or other suitable equipment, and the compacting may be accomplished by means of rollers. Pipe culverts and other structures should be adequately protected from damage before any heavy equipment is operated near or over them. Water can sometimes be used to facilitate the settlement of granular backfills but it should never be used where conditions are such that liquid or semiliquid pressure may be developed within the berm area.

Loads on culverts under fill: The load that a culvert must support consists of the portion of the weight of the embankment carried by the culvert and a part of the weight of traffic on the road surface over the culvert.

Experiments have shown that the portion of the weight of the embankment that is carried by the culvert depends upon the method of placing the fill, the method of installing the culvert, the nature of the soil that supports it, and the unit weight of the material. The load due to the fill material may vary from the weight of a prism of fill directly over the conduit to a load to three times that amount.

Effects of traffic load: A part of the weight of the traffic over the surface is transmitted through embankment to the culvert. The extent to which traffic loads are transmitted to the culvert depends upon the nature of the surface of the road. For unsurfaced roads or light gravel or oil-mixed surfaces it is best to assume that the road surface does not effect any distribution of the load. If a wearing surface is employed, like a concrete or heavy bitumenous macadam, the portion of the traffic load that reaches the smaller culverts is much less than for the case of the unsurfaced road. Although no dependable data are available to indicate the extent of the reduction of loads on culverts through the load-spreading action of stiff wearing surface, it is safe to estimate a 50% reduction.

## PART II.

## DESIGN OF STRUCTURES

### Drainage Areas Measured by Planimeter

| | | |
|---|---|---|
| $a_1$ | = 0.3 $km^2$ | = 74.5 acres * |
| $a_2$ | = 0.44 " | = 108.0 " |
| $a_3$ | = 0.80 " | = 198.5 " |
| $a_4$ | = 0.34 " | = 84.5 " |
| $a_5$ | = 0.28 " | = 69.5 " |
| $a_6$ | = 0.5 " | = 124.0 " |
| $a_7$ | = 0.20 " | = 49.6 " |
| $a_8$ | = 0.24 " | = 59.6 " |
| $a_9$ | = 0.28 " | = 69.5 " |
| $a_{10}$ | = 0.5 " | = 124.0 " |
| $a_{11}$ | = 0.5 " | = 124.0 " |
| $a_{12}$ | = 1.1 " | = 273.0 " |
| $a_{13}$ | = 2.0 " | = 496.0 " |

* 1 $km^2$ = 248 acres.

Dividing the areas into general groups:

Type I - $a_{13}$ = 496 acres = 0.775 sq. miles

Type II- $a_{12}$ and $a_3$ , area=273 acres = 0.426 sq.mile

TypeIII- $a_{10}$, $a_{11}$, $a_6$ and $a_2$ = 124 acres=0.194 sq.mile

Type IV- $a_1$, $a_4$, $a_5$, a7, $a_8$ and $a_9$ area = 84.5 acres
= 0.132 sq.mile

Type V - other smaller culverts.

Size of openings by Talbot's formula

Type I

$A = c\sqrt[4]{a^3}$

A = cross-section area

c = 1 in mountenous areas.

a = drainage area in acres

$\therefore A = 1 \times 496^{3/4} = 104.5$ sq. ft.

The area is for stream of normal velocity of maximum 10 fps. To reduce the section we increase the velocity of the water through the culvert by increasing the slope of the culvert.

$\therefore$ We have 104.5 x 10 = 1045 cfs of water to be discharged.

If we allow the slope of the culvert to be 2%. The discharge of 2 parallel culverts of 5 ft x 4 ft box culvert will be 523 x 2 = 1046 cfc with a velocity of 26.2 fps (from table I) Page 40

Type II.

$A = c\sqrt[4]{a^3}$

c = 1 in mountenous areas

$\therefore a = 1 \times 273^{3/4} = 67$ sq. ft.

This area is for streams with a normal velocity of 10 fps. ∴ the quantity to be discharged is 67 x 10 = 670 ft.sec.

From table I we find that for a slope of 2 % the discharge of a 5'x5' box culvert is 700 cfs and its velocity is 28 fps.

---

Type III

$A = 1 \times 124^{3/4} = 37.2$ sq.ft.

Discharge = 37.2 x 10 = 371 cfs

from table I for a slope of 2 % the discharge of 4' x 4' box culvert is 378 at a velocity of 23.4 fps.

---

Type IV

$A = 1 \times 84{,}5^{3/4} = 27.9$ sq.ft.

D = 27.9 x 10 = 279 cfs

from table I for 2 % slope the discharge of 4' x 3' box culvert is 264 cfs.

Type V

For smaller discharge use box culverts of 2'x2' which discharge 59 cfs with a 2 % slope.

Size of opening by the use of Table II.

Type I

For an area of 0.8 sq.mile the waterway area required is 88 sq.ft. (from Table II) Page 41

The discharge will be 88 x 10 = 880 cfs (since the velocity of the water is 10 fps)

For 2 % slope of culvert the velocity will be 26.2 fps; the discharge of 2 culverts 5'x4' in parallel is 2 x 523 = 1046 cfs (from Table I)

---

Type II

For an area of 0.5 sq.mile the waterway area required is 66 sq.ft. (Table II)

D = 66 x 10 = 660 cfs.

for 2 % slope of culvert the velocity will be 28 fps of a 5'x5' box culvert. D = 700 cfs. (Table I)

Type III

For an area of 0.2 sq.mile the waterway area required is 32 sq.ft. (Table II)

D = 32 x 10 = 320

For 2% slope of culvert the velocity will be 23.4 fps of a 4'x4' box culvert. D = 378 cfs (Table I)

---

Type IV

For an area of 0.15 sq.mile the waterway area required is 25 sq.ft. (Table II)

D = 25 x 10 = 250 cfs

For 2 % slope of culvert the velocity of water will be 22 fps of 4'x3' box-culvert.

D = 264 (Table I)

---

Use 24 in round reinforced concrete pipe as felief culvert. Although this size is very big but it is not allowed to use culverts smaller than 24 in in diameter for road of 4 or more lanes .

Table I. Discharge capacity of box-culverts.

| Slope | 2'x2' Vel. | Dis. | 4'x3' Vel. | Dis. | 4'x4' Vel. | Dis. | 5'x4' Vel. | Dis. | 5'x5' Vel. | Dis. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.5 | 7.7 | 31 | 10.8 | 130 | 11.6 | 186 | 13.0 | 260 | 13.8 | 340 |
| 1.0 | 10.4 | 42 | 15.0 | 180 | 16.7 | 267 | 18.5 | 370 | 19.0 | 475 |
| 2.0 | 14.8 | 59 | 22.0 | 264 | 23.6 | 378 | 26.2 | 523 | 28.0 | 700 |
| 3.0 | 18.3 | 73 | 26.5 | 318 | 29.0 | 464 | | | | |
| 4.0 | 21.0 | 84 | 30.5 | 366 | | | | | | |
| 5.0 | 23.5 | 94 | | | | | | | | |
| 6.0 | 26.0 | 104 | | | | | | | | |

Note: - Table I is figured from church's diagrams of kutter's formula using n = 0.011

Kutter's Formula expressed in metric units is :

$$V = \frac{23 + \frac{1}{n} + \frac{0.00155}{s}}{1 + \left(23 + \frac{0.00155}{s}\right)\frac{n}{\sqrt{r}}}\sqrt{rs}$$

Expressed in English units, the formula becomes

$$V = \frac{41.6 + \frac{1.811}{n} + \frac{0.0028}{s}}{1 + \left(41.6 + \frac{0.00281}{s}\right)\frac{n}{\sqrt{r}}}\sqrt{rs}$$

Table II. Santa Fé Size of openings

| Area drained in sq.miles | Area of waterway sq.ft. | Area drained in sq.miles | Area of waterway in sq.ft. |
|---|---|---|---|
| 0.1 | 16.0 | 0.45 | 62.0 |
| 0.15 | 25.0 | 0.5 | 66.0 |
| 0.20 | 32.0 | 0.55 | 70.0 |
| 0.25 | 38.0 | 0.6 | 74.0 |
| 0.30 | 44.0 | 0.65 | 78.0 |
| 0.35 | 51.0 | 0.7 | 81.0 |
| 0.40 | 56.0 | 0.80 | 88.0 |

Table II is prepared from observation of streams in Southwest Missouri, Eastern Kansas, Western Arkansas and Southeastern of Indian Territory. In all of this region steep rocky slopes prevail like here in Lebanon, and the soil absorbs but a small percentage of the rainfalls. The velocity through openings is 10 fps.

Size of Openings

| Type | No. | By Talbot's Formula Height ft. | m | width ft. | m. | By Table II height ft. | m. | width ft. | m |
|---|---|---|---|---|---|---|---|---|---|
| I | 2 | 4 | 1.2 | 5 | 1.5 | 4 | 1.2 | 5 | 1.5 |
| II | 1 | 5 | 1.5 | 5 | 1.5 | 5 | 1.5 | 5 | 1.5 |
| III | 1 | 4 | 1.2 | 4 | 1.2 | 4 | 1.2 | 4 | 1.2 |
| IV | 1 | 3 | 0.9 | 4 | 1.2 | 3 | 0.9 | 4 | 1.2 |

## Structural Design of Culverts

Assumptions:

1. 2/3 of the weight of the fill and 1/2 of the traffic load will be transferred to the culvert.
2. The fill will be sand and gravel rammed.
3. Live load 20 tons.
4. The live load as well as dead load will be considered as a uniform load.
5. The unit stress of steel = 16,000 psi and of concrete = 650 psi.
6. The pressure on the sides of the culverts are uniform and it is equal to the average of the top and bottom pressure.

Justifications for above assumptions.

1. due to the arch action the weight of the fill above the culvert is distributed and it is carried to the sides of culvert which in this case acts as a column.
2. sand and gravel will be the best material for roads, mixed with a small percentage of clay or silt. It is also the heaviest. In case other materials are used it would be lighter than sand and gravel and the design will be on the safer side.

3. The live load will very rarely exceed 20 tons; so it is not economical to design for heavier loads.

4. due to the arch action and rigidness of the road, the live load will be distributed uniformly on the culvert.

5. to be on the safe side it is better to assume low values of unit stresses because you cannot tell what kind of steel you will get and how well the concrete will be mixed and poured.

6. The difference of moment found by correct and long method will be so small from the approximate method, due to the short span, that it is not necessary to go into that trouble as it will be shown later.

## Design of Type I under a Fill of 9 ft.

Dead load one foot length and one foot width of road

is: $120 \times 10 = 1200$ #/ft.

Live load $20 \times 2000 \times 1/2 \times 1/30 \times 1/5 = 133$ #/ft.

$M = Kbd^2$

$K = 108$ for 16,000 psi unit stress of steel and 650 psi unit stress of concrete.

$M = 1/8 \times 1330 \times 5^2 = 4160$'#

$b = 12$ in

$$\therefore d^2 = \frac{4160 \times 12}{12 \times 108} = 38.5 \text{ in}^2$$

$d = 6.2$" make total thickness 7 in

$$A_s = \frac{M}{f_s jd} = \frac{4160 \times 12}{16000 \times 0.874 \times 6} = 0.595$$

area of 5/8" $\phi$ = 0.3068

number of bars required per foot $= \frac{0.595}{0.3068} = 1.94$

spacing of bars $= \frac{12}{1.94} = 6.2$ in ; make it 6 in C to C

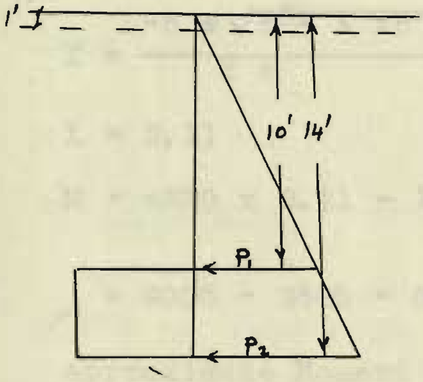
This design will apply both to the top and the bottom of the culvert.

Design of sides:

The pressure of the earth from the sides $= \frac{Kh^2w}{2}$

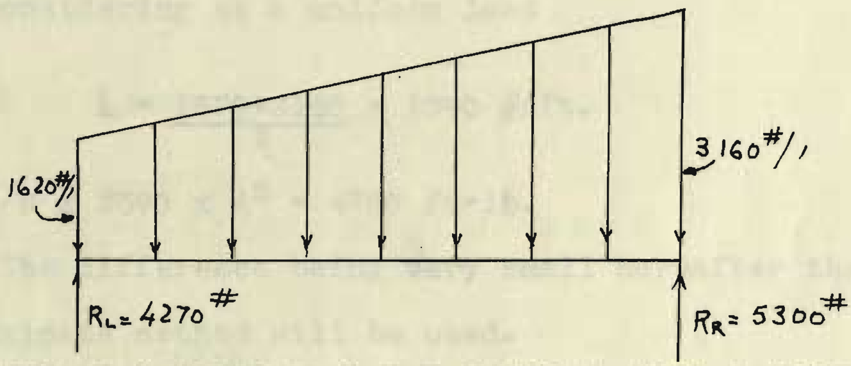
$K = 0.27$ for angle of friction of $35^\circ$ and $0^\circ$ surcharge.

The live load will be substituted by one ft. of fill material.




$$p_1 = 0.27 \times 10^2 \times \frac{120}{2}$$
$$= 1620\ \#/\text{ft.}$$

$$P_2 = 0.27 \times 14^2 \times \frac{120}{2}$$
$$= 3160\ \#/\text{ft.}$$




$$R_L = 1620 \times \frac{4}{2} + 1540 \times \frac{4}{2} \times \frac{1}{3} = 3240 + 1030 = 4270$$

$$R_R = 1620 \times \frac{4}{2} + 1540 \times \frac{4}{2} \times \frac{2}{3} = 3240 + 2060 = 5300$$

The moment will be maximum where the shear is zero.

Let X be the distance from the left reaction to the place where the shear is 0.

$$\text{Then } (1620 + \frac{1540}{4 \times 2} X)X = 4270$$

$$1620X + 193X^2 = 4270$$

$$193X^2 + 1620X - 4270 = 0$$

We find the value of X by quadratic formula

$$X = \frac{-b \pm \sqrt{b^2 - 4\,ac}}{2\,a} = \frac{-1620 \pm \sqrt{1620^2 + 4 \times 193 \times 4270}}{2 \times 193}$$

$X = 2.11$

$$M = 4270 \times 2.11 - 1620 \times 2.11 \times \frac{2.11}{2} - \frac{1540}{4} \times 2.11 \times \frac{2.11}{2} \times \frac{2.11}{3}$$

$= 9000 - 3600 - 600 = \underline{\underline{4800 \text{ ft-lb.}}}$

approximate Moment found by averaging the end pressure and considering as a uniform load

$$L = \frac{1620+3160}{2} = 2390 \text{ \#/ft.}$$

$M = 1/8 \times 2390 \times 4^2 = 4780$ ft-lb.

The difference being very small hereafter the approximate method will be used.

$M = Kbd^2$

$$d^2 = \frac{4780 \times 12}{12 \times 108} = 43.8 \text{ in}^2$$

$d = 6.6$ in make total thickness 7.5 in

$$As = \frac{4780 \times 12}{16{,}000 \times 0.874 \times 6.5} = 0.632$$

using 5/8" we need $\frac{0.632}{0.3068} = 2.06$ bars per foot.

The spacing $= \frac{12}{2.06} = 5.83$ in ; make 5.5 in C to C

## Design of Type I under a fill of 5 ft.

Dead load $6 \times 120 = 720$ #/ft.

Live load the same = 133 #/ft.

$M = 1/8 \times 853 \times 5^2 = 2670$ ft-lb.

$d^2 = \frac{2670 \times 12}{12 \times 108} = 24.5 \text{ in}^2$

d = 5 in make total thickness 6 in

$As = \frac{2670 \times 12}{16000 \times 0.874 \times 5} = 0.46$

Using ½ in Ø we need $\frac{0.46}{0.1963} = 2.34$ bars per foot

Spacing $\frac{12}{2.34} = 5.13$ in make 5 in C.to C.

Design of the sides

$P_1 = 0.27 \times 6^2 \times \frac{120}{2} = 583$ #/ft.

$P_2 = 0.27 \times 10^2 \times \frac{120}{2} = 1620$ #/ft.

Average uniform pressure = $\frac{1620 + 580}{2} = 1100$ #/ft.

$M = 1/8 \times 1100 \times 4^2 = 2200$ ft-lb.

$d^2 = \frac{2200 \times 12}{12 \times 108} = 20.4$

d = 4.51 in make total thickness 5.5 in.

$$As = \frac{2200 \times 12}{16000 \times 0.874 \times 4.5} = 0.42 \text{ in}^2$$

Using 1/2 in ∅ we need $\frac{0.42}{0.1963}$ = 2.13 bars per foot

Spacing $\frac{12}{2.13}$ = 5.64 in ; make 5.5 in C. to C.

## Design of Type II under a Fill of 9 ft.

The top and bottom slabs will be the same as type one since it has the same span.

Design of sides

$$P_1 = 0.27 \times 10^2 \times \frac{120}{2} = 1620 \text{ \#/ft.}$$

$$P_2 = 0.27 \times 15^2 \times \frac{120}{2} = 3640 \text{ \#/ft.}$$

Average uniform pressure = $\frac{3640 + 1620}{2}$ = 2630 #/ft.

$$M = 1/8 \times 2630 \times 5^2 = 8230 \text{ ft-lb.}$$

$$d^2 = \frac{8230 \times 12}{12 \times 108} = 76.1 \text{ sq.in.}$$

d = 8.7 in ; make total thickness 10 in.

$$As = \frac{8230 \times 12}{16000 \times 01874 \times 9} = 0.786$$

Using 5/8" ∅ bars we need $\frac{0.786}{0.3068}$ = 2.56 bars per foot

Spacing = $\frac{12}{2.56}$ = 4.7 in ; make 4.5 in C to C

Design of Type II under a Fill of 5 ft.

The top and bottom slabs are the same as that of Type I .

Design of sides

$P_1 = 0.27 \times 6^2 \times \frac{120}{2} = 580$ #/ft.

$P_2 = 0.27 \times 11^2 \times \frac{120}{2} = 1960$ #/ft.

Average uniform pressure $= \frac{580 + 1960}{2} = 1270$ lb/ft.

$M = 1/8 \times 1270 \times 5^2 = 3970$ ft-lb.

$d^2 = \frac{3970 \times 12}{12 \times 108} = 36.7$ sq.in.

d = 6.05 in. ; make total thickness 7 in.

$As = \frac{3970 \times 12}{16000 \times 0.874 \times 6} = 0.566$ sq.in.

Using 5/8 in ∅ we need $\frac{6.566}{0.3068} = 1.94$ bars per ft.

Spacing $= \frac{12}{1.94} = 6.2$ in ; make 6 in Center to Center.

## Design of Type III under a Fill of 9 ft.

Design of top and bottom slabs.

Uniform load is the same = 1330 #/ft.

$M = 1/8 \times 1330 \times 4^2 = 2660$ ft-lb.

$d^2 = \dfrac{2660 \times 12}{12 \times 108} = 24.6$ in sq.

d = 5 in make total thickness 6 in.

$As = \dfrac{2660 \times 12}{16000 \times 0.874 \times 5} = 0.456$ sq.in.

Using 1/2 in Ø we need $\dfrac{0.456}{0.1963} = 2.32$ bars per foot

Spacing $= \dfrac{12}{2.32} = 5.17$ in.; make 5 in C to C

The side slabs are the same as the sidesslabs of Type I

## Design of Type III under a Fill of 5 ft.

Design of top and bottom slabs

Uniform load is 855 #/ft.

$M = 1/8 \times 855 \times 4^2 = 1710$ ft-lb.

$d^2 = \dfrac{1710 \times 12}{12 \times 108} = 15.85$ sq.in.

d = 4 in ; make total thickness 5 in.

$As = \dfrac{1710 \times 12}{16000 \times 0.874 \times 4} = 0.367$ sq.in.

Using 1/2 in Ø we need $\dfrac{0.367}{0.1963} = 1.87$ bars per ft.

Spacing $= \dfrac{12}{1.87} = 6.4$ in ; make 6 in C to C.

The side slabs are the same as Type I.

## Design of Type IV under a Fill of 9 ft.

The top and bottom slabs are the same as Type III

Design of side slabs

$P_1 = 0.27 \times 10^2 \times \frac{120}{2} = 1620$ #/ft.

$P_2 = 0.27 \times 13^2 \times \frac{120}{2} = 2740$ lb/ft.

Average uniform pressure $= \frac{1620 + 2740}{2} = 2180$ #/ft.

$M = 1/8 \times 2180 \times 3^2 = 2460$ ft-lb.

$d^2 = \frac{2460 \times 12}{12 \times 108} = 22.7$ sq.in.

d = 4.77 in ; make total thickness 6 in.

$As = \frac{2460 \times 12}{16000 \times 0.875 \times 5} = 0.423$ sq.in.

Using 1/2 in ∅ we need $\frac{0.423}{0.1963} = 2.15$ bars per foot

Spacing $\frac{12}{2.15} = 5.58$ in ; make 5.5 in Center to Center

## Design of Type IV under a Fill of 5 ft.

Top and bottom slabs are the same as type III.

Design of side slabs

$P_1 = 0.27 \times 6^2 \times \frac{120}{2} = 580$ #/ft.

$P_2 = 0.27 \times 9^2 \times \frac{120}{2} = 1320$ #/ft.

Average uniform pressure $= \frac{580 + 1320}{2} = 1450$ #/ft.

$M = 1/8 \times 1450 \times 3^2 = 1630$ ft-lb.

$d^2 = \frac{1630 \times 12}{12 \times 108} = 15.1$ in sq.

d = 3.9 in ; make total thickness 5 in.

$As = \frac{1630 \times 12}{16000 \times 0.874 \times 4} = 0.349$ sq.in.

Using 1/2 in ⌀ we need $\frac{0.349}{0.1963} = 1.78$ bars per foot

Spacing $= \frac{12}{1.78} = 6.8$ ; make 6.5 Center to Center.

Design of Type V under a fill of 9 ft.

Design of top and bottom slabs

Uniform load is equal to 1330 #/ft.

$M = 1/8 \times 1330 \times 2^2 = 665$ ft-lb.

$d^2 = \frac{665 \times 12}{12 \times 108} = 6.15$ sq. in.

d = 2.5 ; make total thickness 3.5 in.

$As = \frac{665 \times 12}{16000 \times 0.974 \times 2.5} = 0.228$ sq. in.

Using 3/8 in ⌀ we need $\frac{0.228}{0.1104} = 2$ bars per foot

Spacing $\frac{12}{2}$ 6 in C to C.

Design of side slab

$$P_1 = 0.27 \times 10^2 \times \frac{120}{2} = 1620$$

$$P_2 = 0.27 \times 12^2 \times \frac{120}{2} = 2340$$

$$\text{Average uniform pressure} = \frac{1620 + 2340}{2} = 1980 \text{ \#/ft.}$$

$$M = 1/8 \times 1980 \times 2^2 = 990 \text{ ft-lb.}$$

$$d^2 = \frac{990 \times 12}{12 \times 108} = 9.16 \text{ sq.in.}$$

d = 3.1 in make total thickness 4 in.

$$As = \frac{990 \times 12}{16000 \times 0.874 \times 3} = 0.284 \text{ sq.in.}$$

Using 3/8 in ø we need $\frac{0.284}{0.1104}$ = 2.5 bars per foot

Spacing = $\frac{12}{2.5}$ = 4.8 ; make 4.5 in C to C.

Design of Type V under a fill of 5 ft.

Uniform load is equal to 855 #/ft.

$M = 1/8 \times 855 \times 2^2 = 430$ ft-lb.

$d^2 = \frac{430 \times 12}{12 \times 108} = 3.98$ sq.in.

d = 2 in ; make total thickness 3.5 in.

$As = \frac{430 \times 12}{16000 \times 0.874 \times 2.5} = 0.147$ sq.in.

Using 3/8 in ∅ we need $\frac{0.147}{0.1104} = 1.34$ bars per foot

Spacing $= \frac{12}{1.34} = 8.9$ ; make 8 in C to C.

Design of side slabs

$P_1 = 0.27 \times 6^2 \times \frac{120}{2} = 583$ #/ft.

$P_2 = 0.27 \times 8^2 \times \frac{120}{2} = 1360$ #/ft.

Average uniform pressure $= \frac{1360 + 580}{2} = 970$ #/ft.

$M = 1/8 \times 970 \times 2^2 = 485$ ft-lb.

$d^2 = \frac{485 \times 12}{12 \times 108} = 4.5$

d = 2.12 in ; make total thickness 3.5

$As = \frac{485 \times 12}{16000 \times 0.875 \times 2.5} = 0.166$

Using 3/8 in ∅ we need $\frac{0.166}{0.1104} = 1.51$ bars per foot

Spacing $= \frac{12}{1.51} = 7.95$ ; make 7.5 Center to Center.

## Design of Side Ditches

It is very convenient to divide the ditches into 3 different types. Type I having slope of 8.5-7 %; type II 5-2.5 % and type III level surface.

General assumptions

1. The shape of the side ditches will be triangular except where the ground is level.

2. The ditches will be lined wherever the slope is more than 2.5 %

3. n will be taken as 0.03

4. Maning's Formula will be used to find the velocity of water in the ditch.

5. The maximum intensity of rainfall will be considered as 6 in/hr.

6. All the rain falling on the road and the side slopes will flow into the ditches.

7. Half the right of way will contribute to each side ditch.

Reasons for above assumptions:

1. Triangular shape of side ditches will be used because the capacity of the water to be carried is small and it is to be carried a short distance.

2. If the ditches are not lined at slopes of more

than 2.5 % the high velocity of water will erode the bottom and sides of the ditch and harm the road proper.

3. n minimum for smooth canal is 0.025 and n maximum is 0.035. n = 0.03 is the average of these 2 values.

4. Maning Formula and Kutter's formula give the same results for values of r (hydraulic radius) 3.28 ft. when r is less than 3.28 ft Manning formula will give slightly larger values than the Kutter's formula. Therefore the advantages of the Manning formula is that it is on the safe side in this design because r is less than 3.28 ft and scondly it is much more easier to deal with than with Kutter's formula.

5. Since there are no data available for the maximum intensity of rainfall, it is wise to take the maximum intensity of California which has quite a similar condition as Lebanon. In California the maximum rainfall has occured on Aug.12, 1891 which is 8.63 in/hr. It is not economical to design culverts which will be good for storms happening once in a 50 years. So it is quite reasonable to consider the maximum intensity of rainfall as 6 in per hr.

6. The surface of the road as well as side slope will be impervious. So the water will not be able to percolate into the soil but instead it will flow into the ditches.

7. It is not wise to consider only half the width of the road because the rain that falls on the side slopes and adjacent lands which slope toward the ditch will convey the water to the canal. On the other hand it is not economical to assume more than half the width of right of way because the water which comes from adjacent land sloping toward the road will be carried away by intercepting or supplementary ditches.

Design of Type I (7 %)

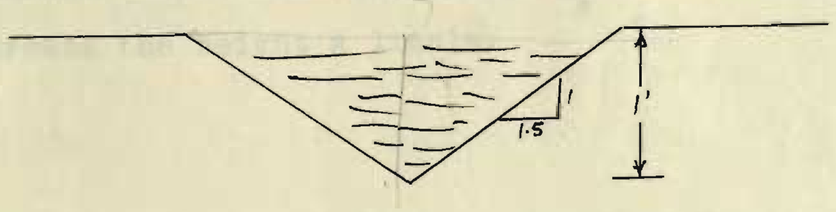
The maximum spacing of culverts in steep places (7-8 %) is 300 ft.

The width of the right of way is 50 m = 164 ft.

The maximum run-off will be $\frac{164}{2} \times 300 \times \frac{6}{12} \times \frac{1}{60 \times 60}$

$= 3.44$ cfs

Assuming a triangular section of height 1 ft and side slope 1: 1½



$$A = \frac{3 \times 1}{2} = 1.5 \text{ sq.ft.}$$

$$p = 2\sqrt{2.25+1} = 2\sqrt{3.25} = 3.6 \text{ ft.}$$

$$r = \frac{1.5}{3.6} = 0.416$$

$$V = \frac{1.49}{0.03} \times 0.416^{2/3} \times 0.07^{\frac{1}{2}} = 49.7 \times 0.568 \times 0.264 = 7.45 \text{ fps.}$$

$$D = 1.5 \times 7.45 = 11.2 \text{ cfs}$$

This section is although a little larger than it is required but a smaller section will be too small. Therefore this section is satisfactory.

Design of Type II (2.5 % slope)

The maximum spacing of culverts is 500 ft; the maximum run-off will be $500 \times \frac{164}{2} \times \frac{6}{12 \times 60 \times 60} = 5.73$ cfs

Using the same section

$$V = \frac{1.49}{0.03} \times 0.416^{2/3} \times 0.025^{\frac{1}{2}} = 49.7 \times 0.568 \times 0.158 = 4.45 \text{ fps.}$$

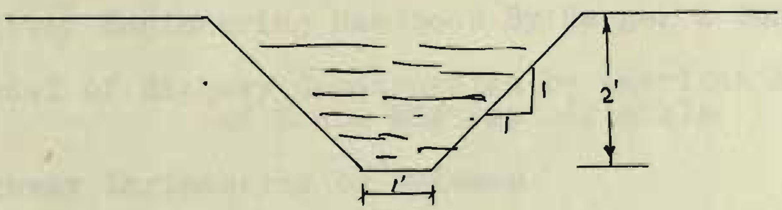
$$D = 4.45 \times 1.5 = 6.68 \text{ cfs.}$$

Although this section is enough it is better to make increase the height a little.

## Type III (Level Ground 0.1% slope)

Since the ground is level it is better to use trapezoidal shape culvert.

Assume a trapezoidal culvert of 1 ft base, width 2 ft height and side slope 1:1




$A = \frac{5+1}{2} \times 2 = 6$ sq.ft.

$P = 1 + 2\sqrt{8} = 1 + 5.65 = 6.5$

$r = \frac{6}{6.5} = 0.923$

$V = \frac{1.49}{0.03} \times 0.923^{2/3} \times 0.001^{\frac{1}{2}} = 49.7 \times 0.95 \times 0.0316 = 1.49$

$D = 6 \times 1.49 = 8.95$ cfs

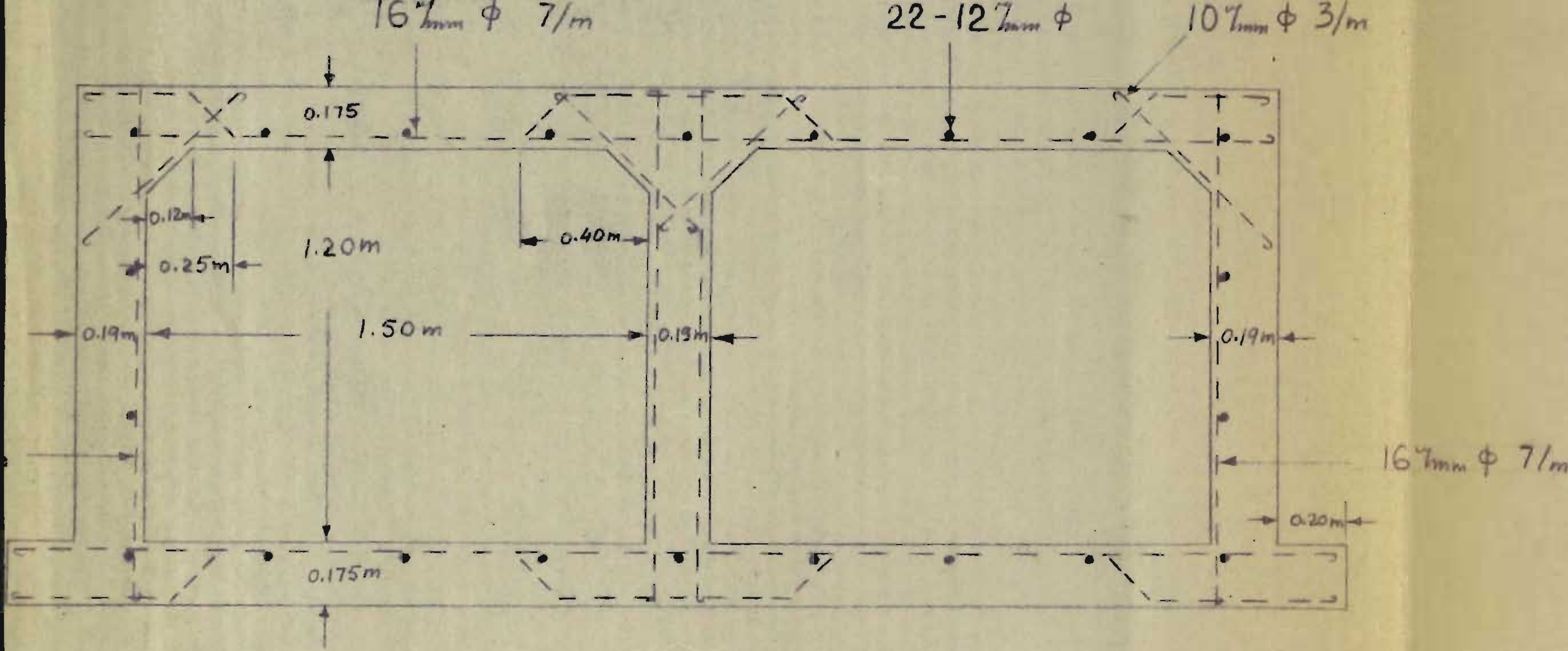
This section is satisfactory becuase the run off will be the same as type III that is 5.73 cfs.

## R E F E R E N C E S

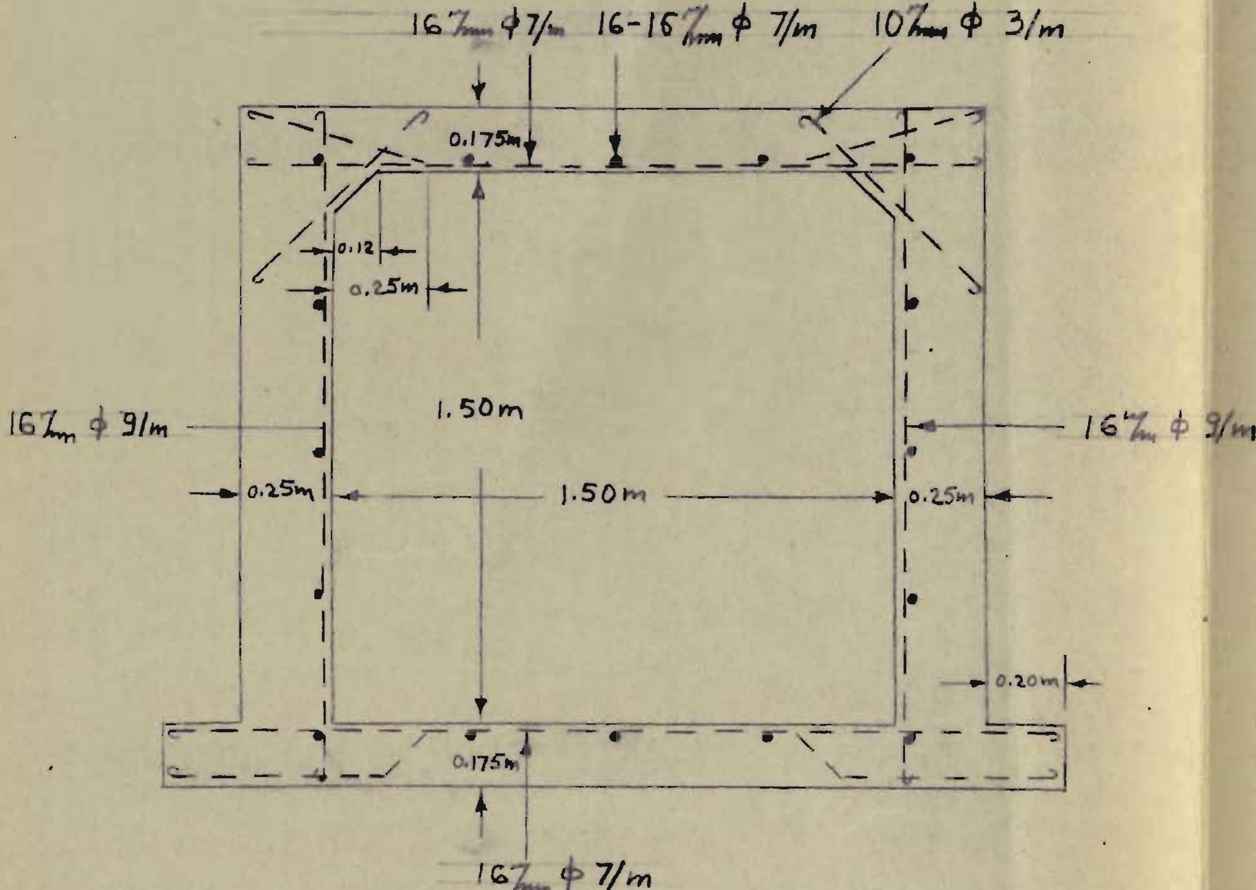
In studying the drainage problems and designing the drainage structures the following books were used as references:


1. Handbook of Applied Hydraulics
2. Drainage and Flood Control Engineering by Pickel
3. Highway Enginnering Handbook By Harger & Bonnet
4. Manual of Highway Construction by American Association of State Highway officials
5. Highway Engineering by Bateman
6. Railroad Construction by Webb
7. Construction of roads and Pavement by Agg
8. Design of Concrete Structures by Urquhart

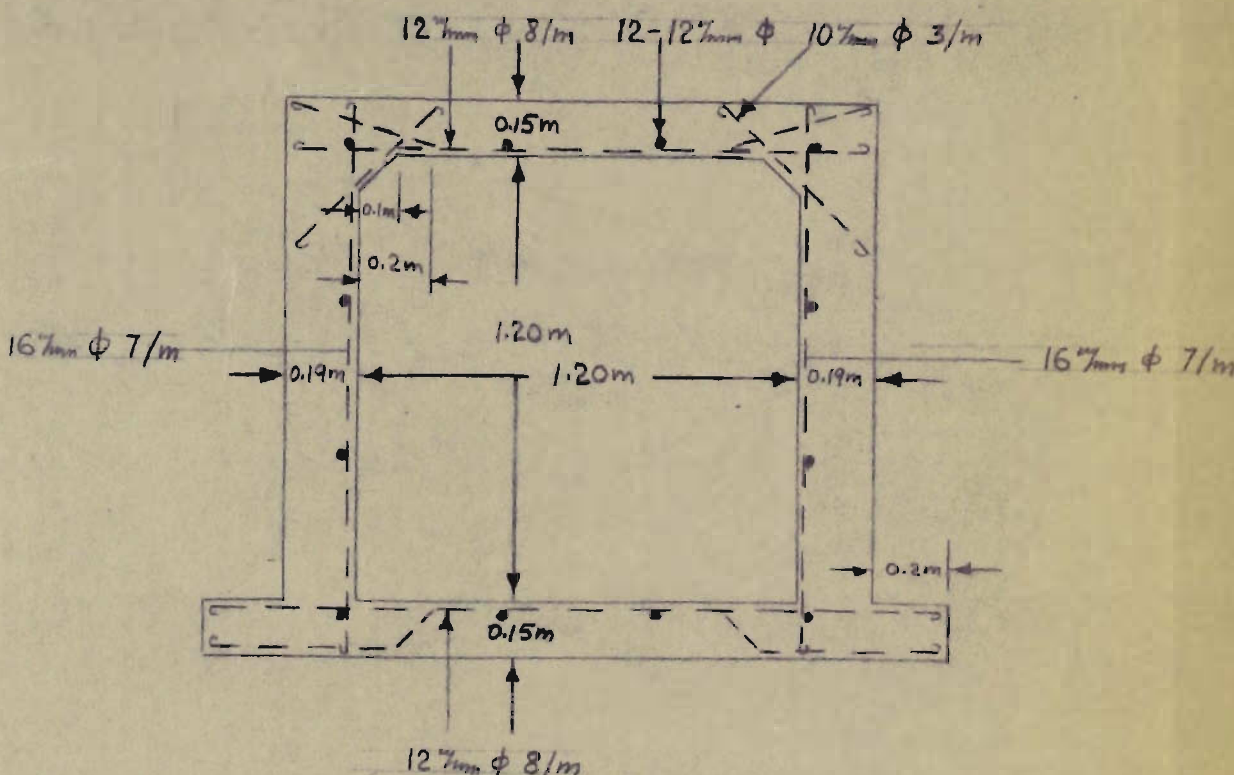

2 additional charts in copy 2



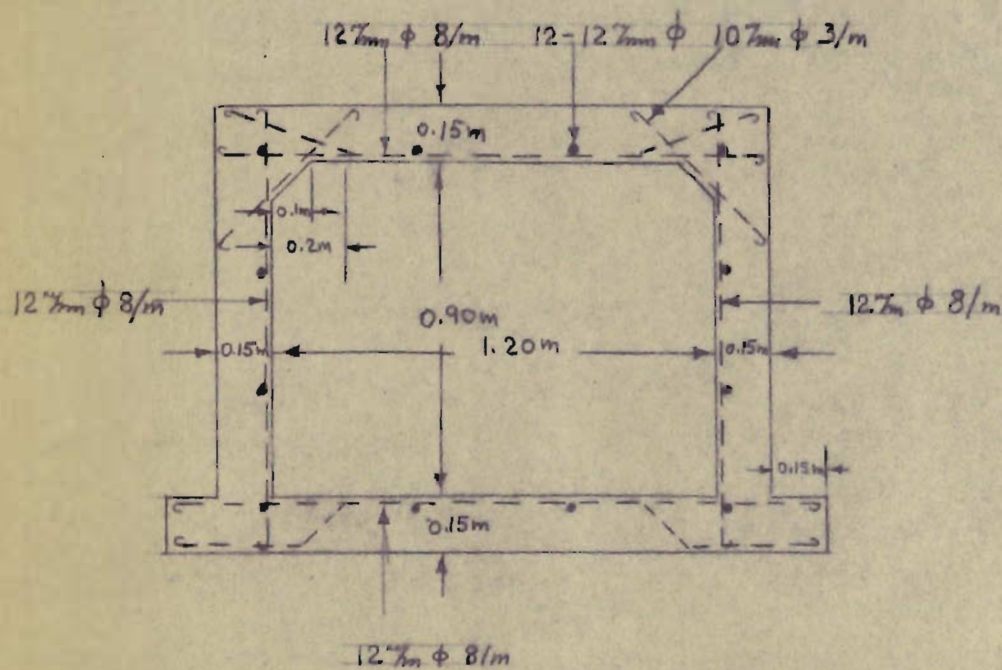

TYPE I UNDER FILL OF 9 Ft. (3m)



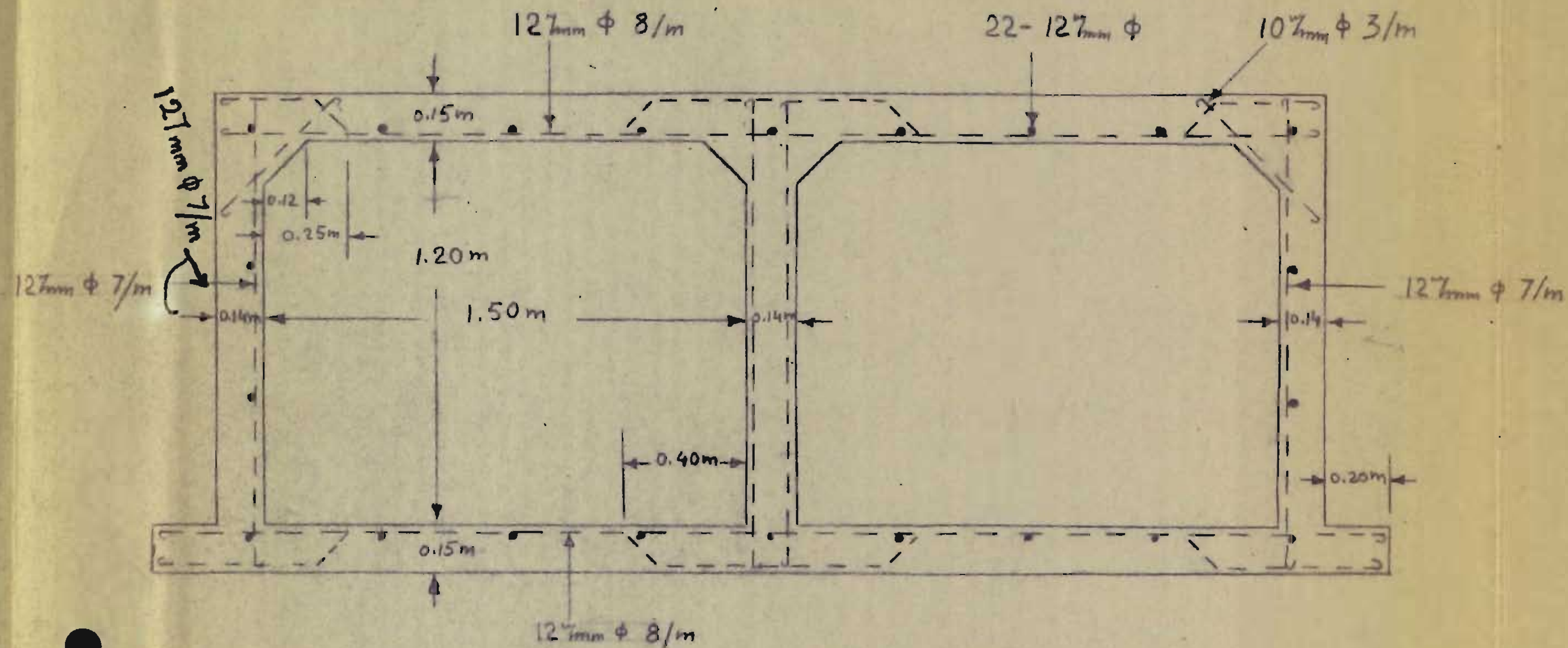

TYPE II UNDER FILL OF 9 Ft.



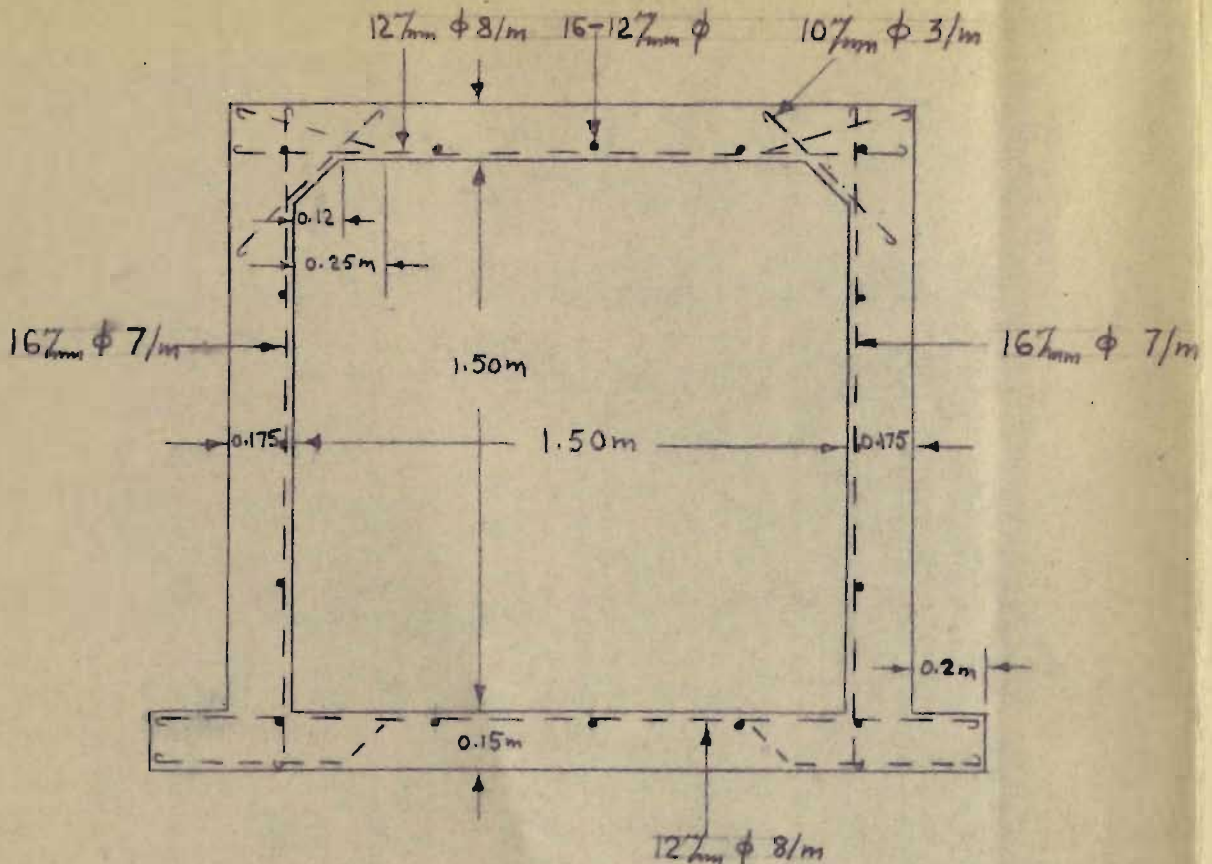

TYPE III UNDER FILL OF 9 Ft.



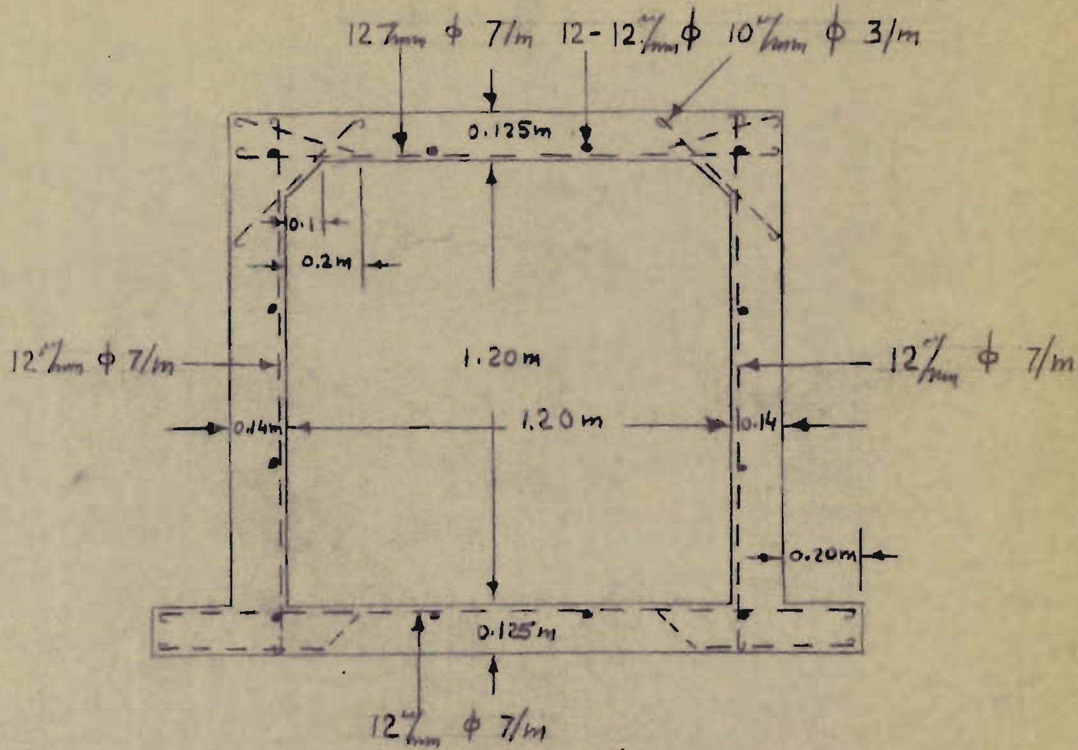

TYPE IV UNDER FILL OF 9 Ft.



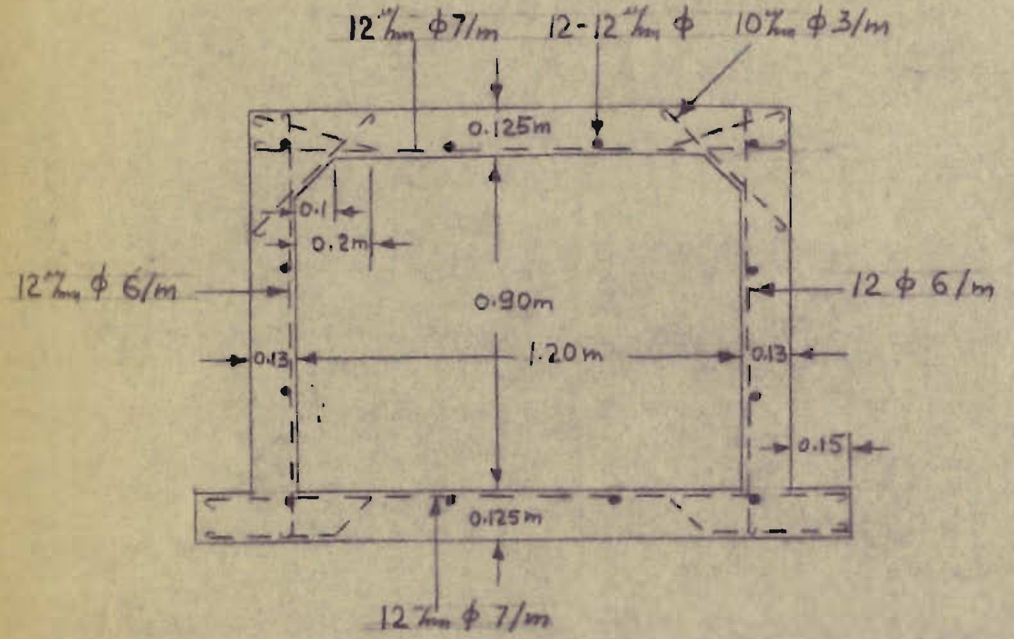

TYPE I UNDER FILL OF 5 Ft OR LESS (1·5m)




TYPE II UNDER FILL OF 5Ft. OR LESS




TYPE III UNDER FILL OF 5Ft. OR LESS



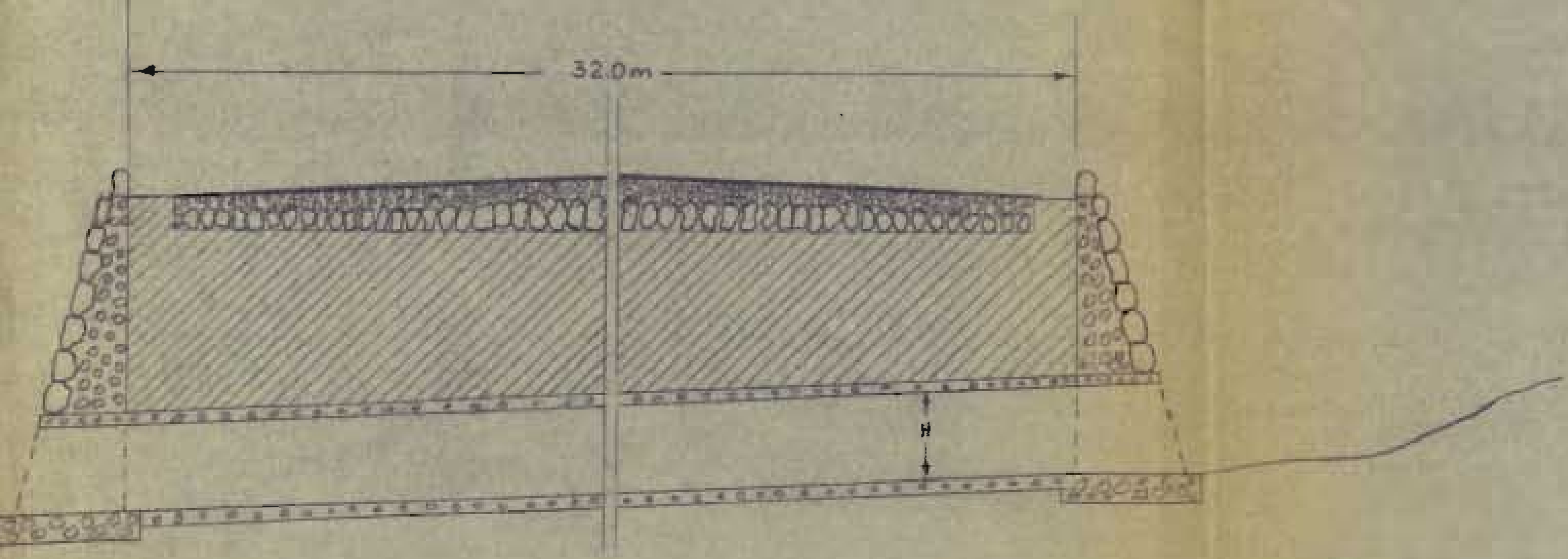

TYPE IV UNDER FILL OF 5FT OR LESS

SCHOOL OF ENGINEERING A. U. B.
PROJET: DRAINAGE STRUCTURES

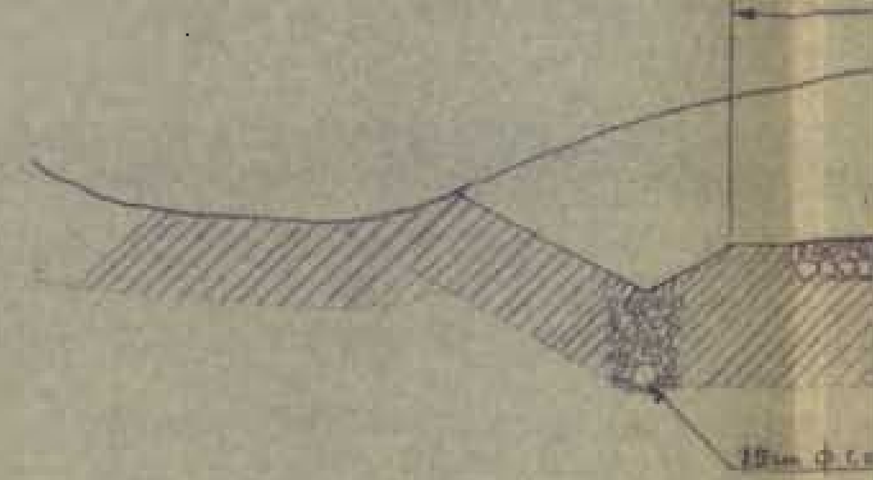
SECTIONS

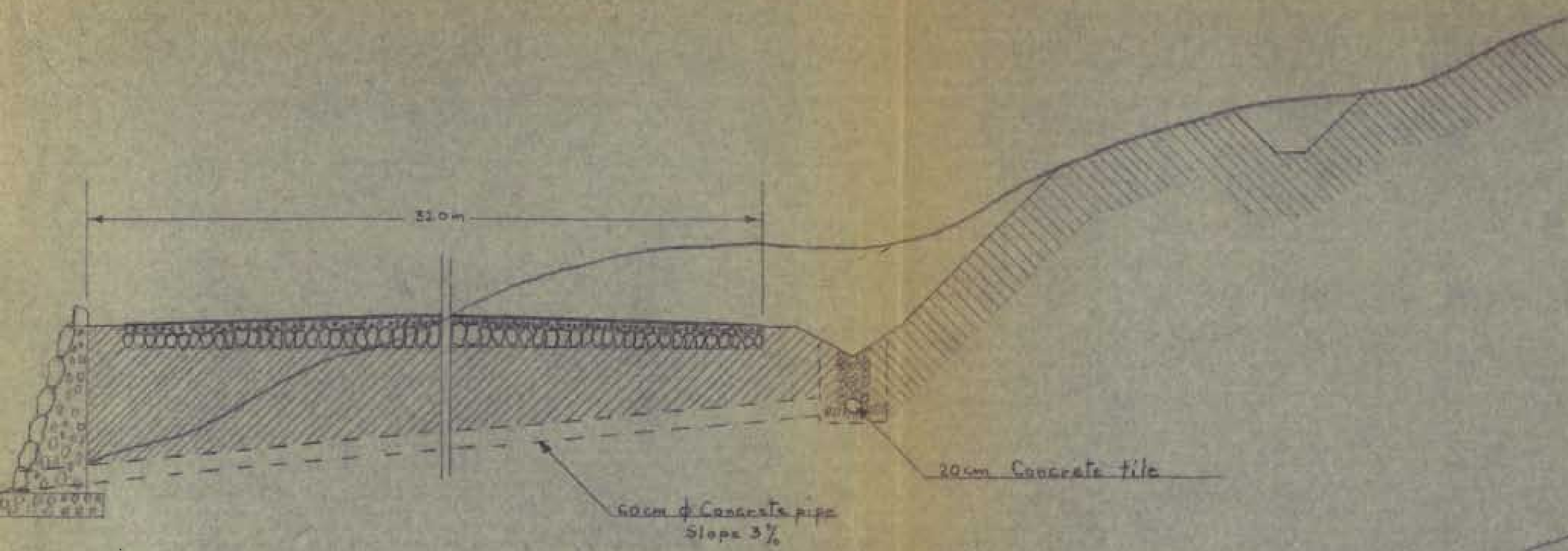
DESIGNED BY: EDWARD BALABANIAN

DATE APRIL 27, 1953 SCALE 1:20 SHEET 1 OF 3

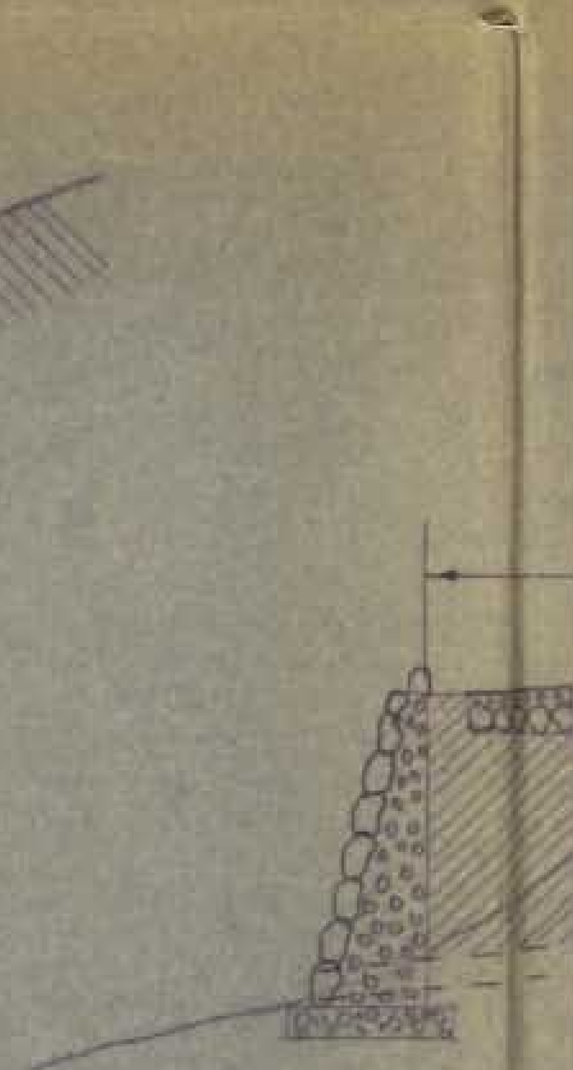


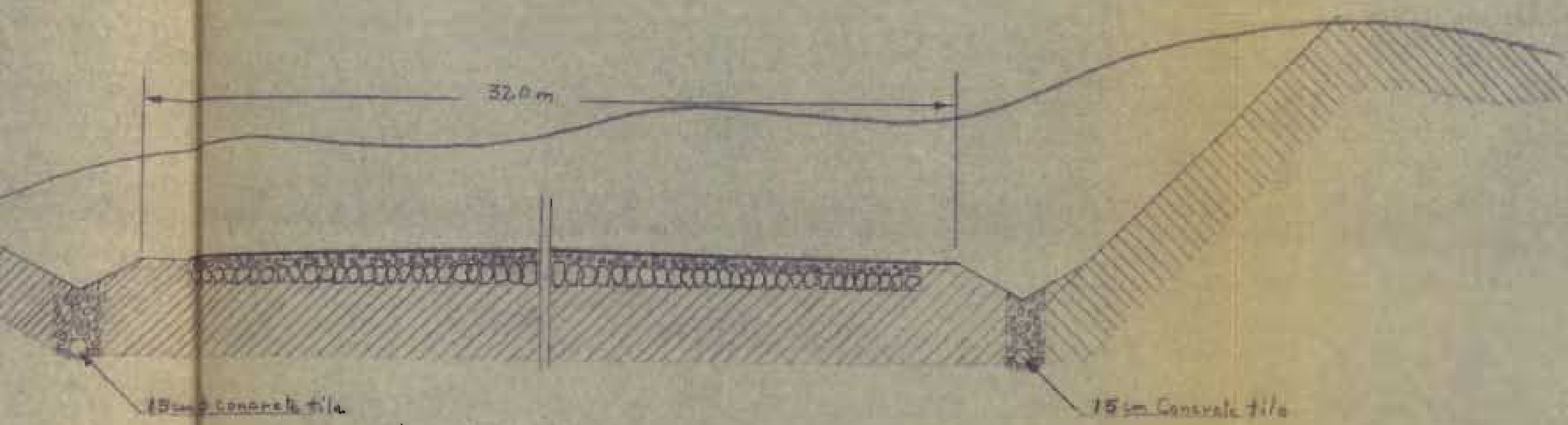
32.0m
H

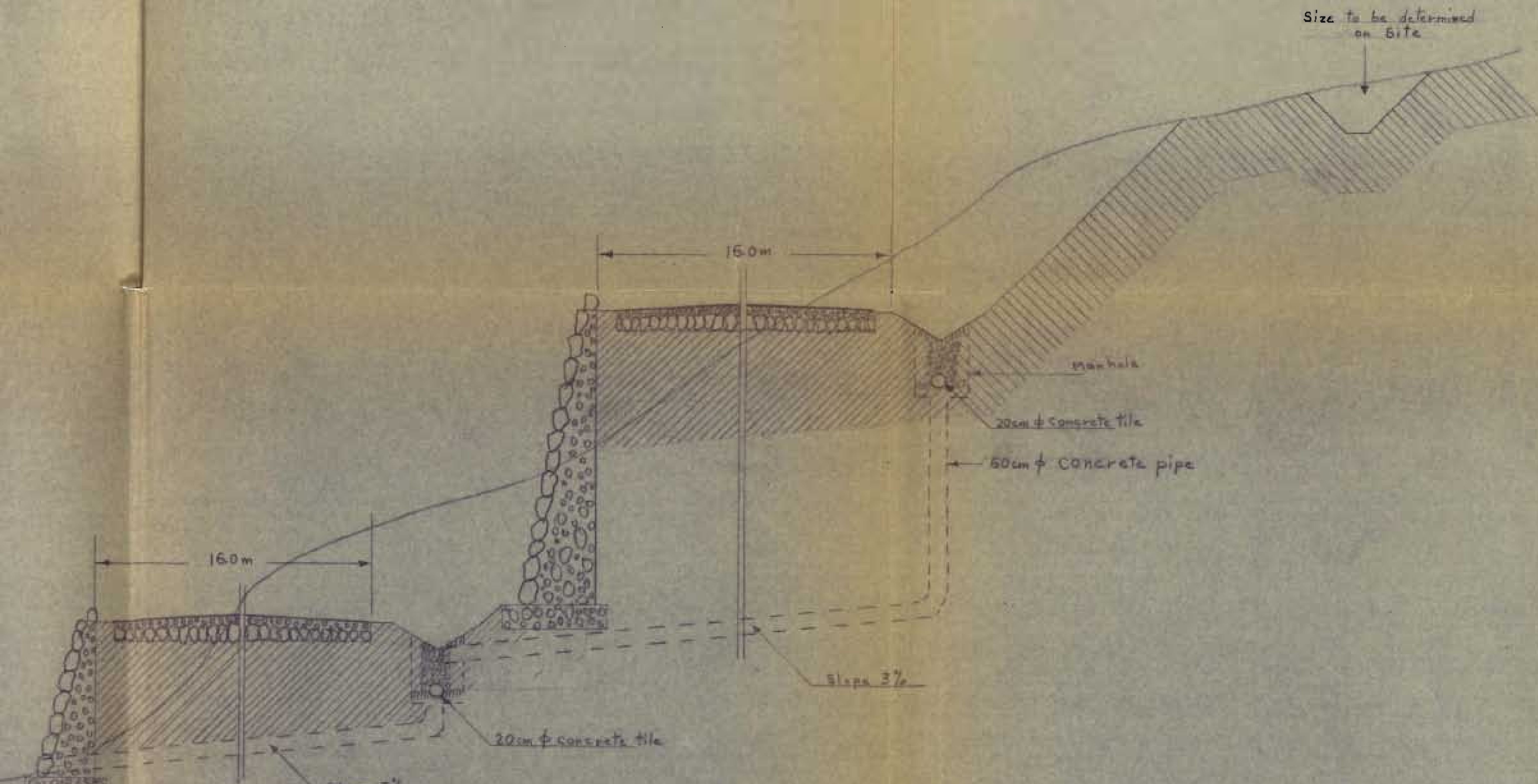





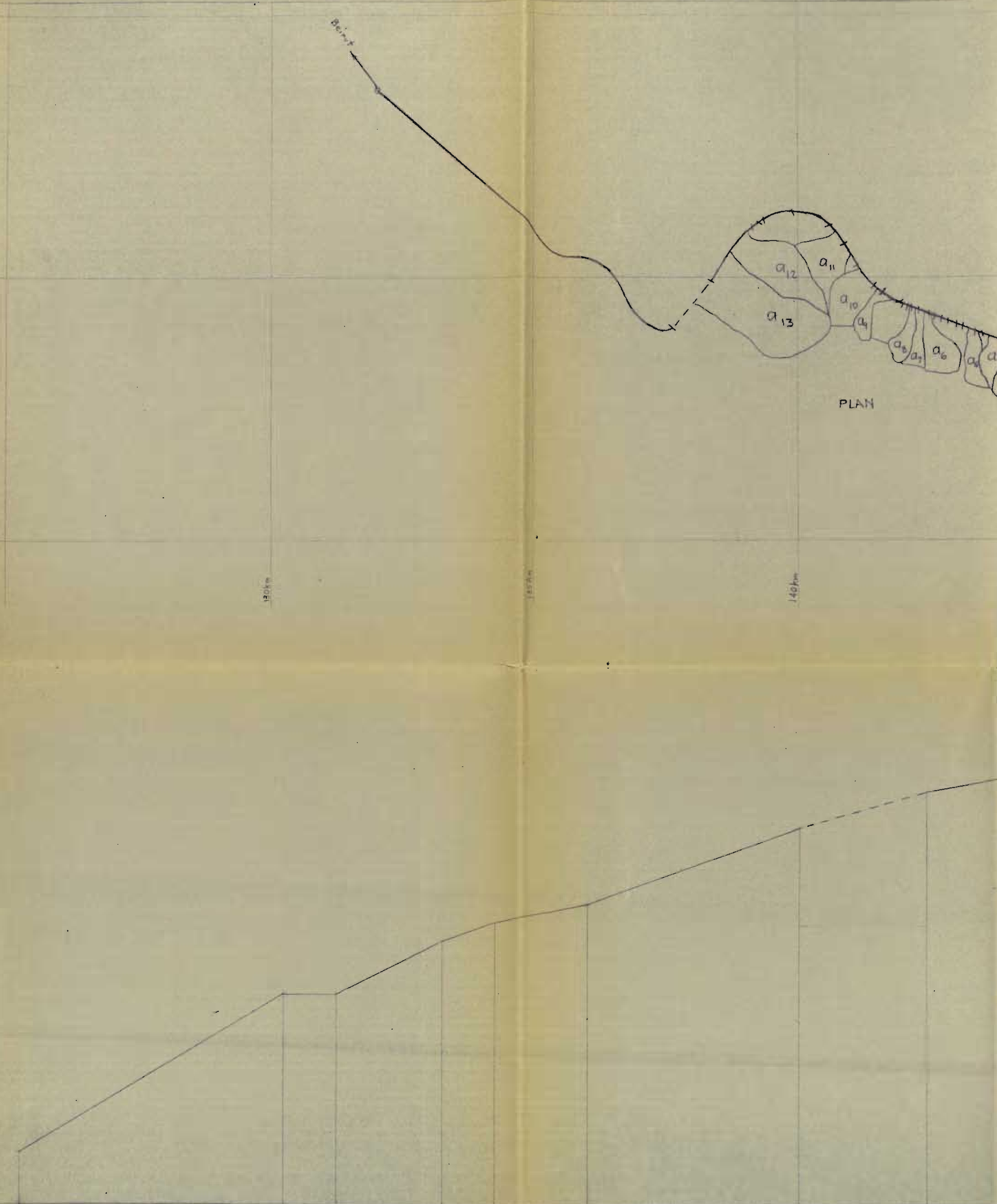
32.0m
60cm ϕ Concrete pipe
Slope 3%
20cm Concrete tile



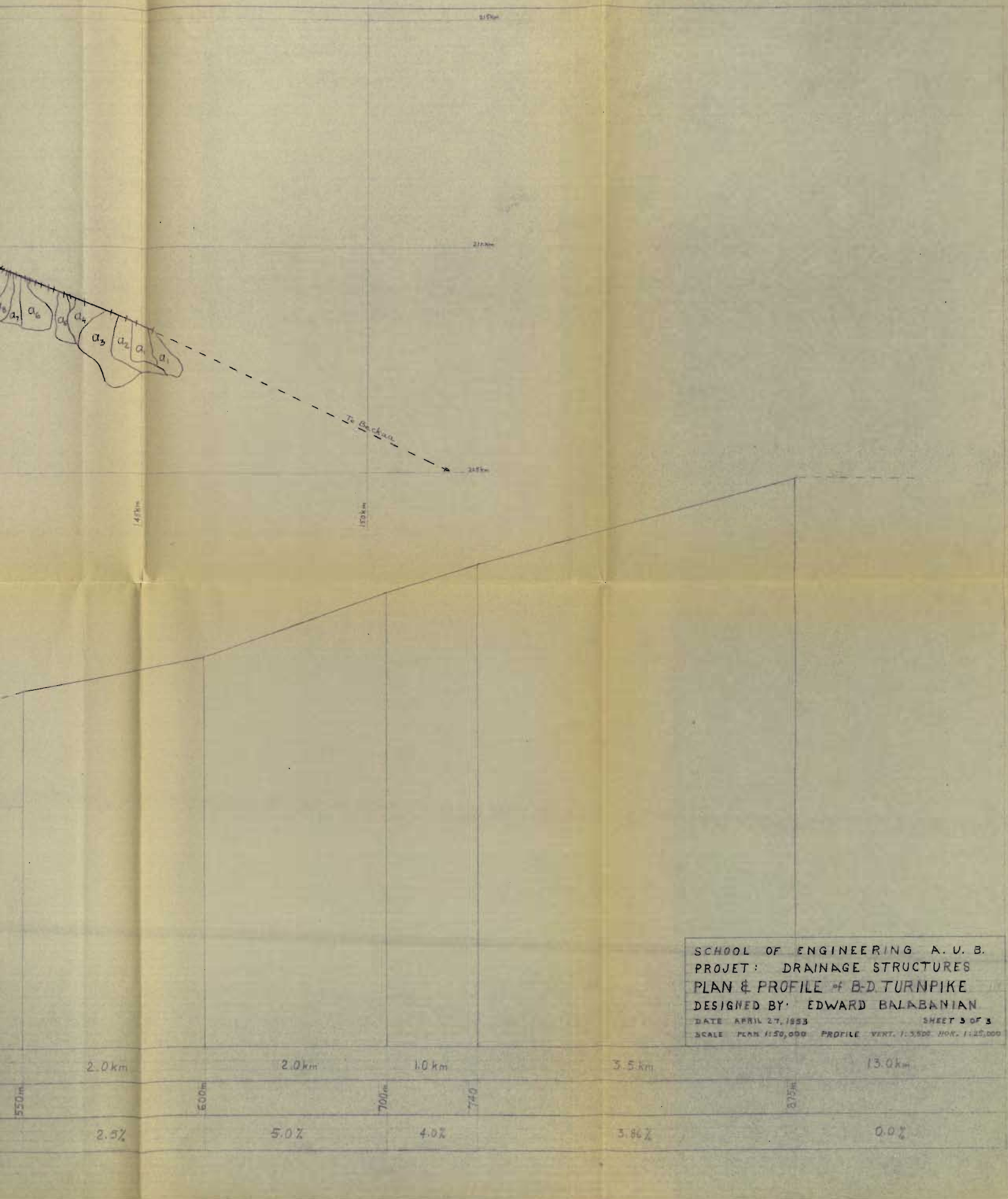




SCHOOL OF ENGINEERING A. U. B.
PROJECT: DRAINAGE STRUCTURES
ROAD CROSS-SECTION
DESIGNED BY: EDWARD BALABANIAN
DATE APRIL 27, 1953 SCALE 1:100 SHEET 2 OF 3




| DISTANCE (DIST-ANCE) | 2.5 km | 0.5 km | 1.0 km | 0.5 km | 0.9 km | 2.0 km | 1.2 km |
|---|---|---|---|---|---|---|---|
| SLOPE (SLOPE) | 8.5% | 0% | 7.0% | 5.0% | 2.5% | 5.0% | 4.15% |

| ELEVATION (ELE-VATION) | | | | | | | |
|---|---|---|---|---|---|---|---|
| 70 m | 280 m | 280 m | 350 m | 375 m | 400 m | 500 m | 550 m |




SCHOOL OF ENGINEERING A. U. B.
PROJET: DRAINAGE STRUCTURES
PLAN & PROFILE of B-D TURNPIKE
DESIGNED BY: EDWARD BALABANIAN
DATE APRIL 27, 1953 SHEET 3 OF 3
SCALE PLAN 1:50,000 PROFILE VERT. 1:3,500 HOR. 1:25,000