design

Of

Rein-Forced Concrete

Bridge

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- To our parents who teach us the love of work faithfully.

- To our brothers and sisters who encouraged us.

- To all who wished our success in this project

- To all our friends.

Contents

Introduction:

Reinforced concrete is particularly well suited for use in bridges of all kinds because of its durability, rigidity, and economy as well as the comparative ease with which a pleasing appearance can be achieved.

For very short spans, from about 10 to 25 ft., one way slab bridges are economical. Some longer spans. Up to about 50 ft., can be achieved with bridges of this same general type by forming longitudinal void with fiber board tubes there by reducing dead load.

Cast in place concrete girder may be used for spans up to about 100 ft., although their use is less common now than before because of the advantages of pre-casting for this range of spans. For cast-in-place bridges monolithic concrete deck spans as a slab transversely and also provided a abroad compression flange for the main, the main girders. Such bridges maybe single span and simple.

Types of R.C Bridge:

There are numerous types of bridges, which can be constructed from R.C.C list of important types of R.C.C bridges is given below:

1- R.C.C slab bridge,

2- simple girder⨍slab bridge.

3- T. beam bridge.

4- Rigid frame R.C.C bridges.

5- R.C.C arch bridges.

6- Bow sting girder bridges.

7- Cantilever bridges etc.

Cast is generally the governing factor for the section of type of bridges. There are so many factors which are to be considered before any section of type of bridge is made foundation conditions, loading conditions, availability of materials of construction are some of the factors which influenced the choice 1.

Bridge water ways:

The determination of adequate water way opening for ⨍ stream crossings is essential to the design of safe f economical bridges, hydraulic studies of bridges site are necessary part of the primary design of bridge and reports of such studies should include applicable parts of the following on time.

A- Site date:

1- Maps, stream cross sections, aerial photography.

2- Complete data on existing bridges including dates of construction and performance during past floods.

3- Available high water marks with dates of occurrence.

4- Information on ice, debus and channel stability.

5- Factors affecting water stages such as high water from other streams, reservoirs, flood control projects and tides.

B- Hydrologic Analysis:

1- Compute flood data application to estimating floods at side including both historical floods and maximum floods of record.

2- Plot flood frequency curve for site.

3- Determine distribution of flow and velocities at site for flood discharges to be considered in design of structure.

4- Plot stage- discharge curve for site.

C- Hydraulic Analysis.

1- Compute back water and mean velocities at bridge opening for various trail bridge lengths and selected discharge.

2- Estimate scour depth at piers and abutment of proposed structures.

Usually bridge water ways are sized to pass a design flood of magnitude and frequency consistent with the type or class of high ways. In the section of the water way opening. Consideration should be given to the mount of up streams ponding, the passage of ice and debus and possible scour of the bridge foundation where flood exceeding the design flood have occurred or where super floods would cause extensive damage to adjoin property or the loss of a costly structure a larger water way opening maybe warranted, due consideration should be given to any Federal state and local requirements.

Relief opening super-clicks, derbies deflectors and channel training works should be used where needed to minimize the effect of adverse flood flow condition where scour is likely to occur protection against of bridge piers and should be provided for in the design of bridge pairs and abutments. Embankment slopes adjacent to structures subject to erosion should be adequately protected by rip-rap flexible mattresses retards spur dikes or other appropriate construction cleaning of brush and the tress along

Embankment in the vacating of bridge openings should be avoided to prevent high flow velocities and possible scour, borrow pits should not be located in areas which would increase velocities and the possibility of scour at bridges. 2

Economic span of Bridge:

Total cost of the bridge construction is the sum of costs of construction of sub- structure ⨍ super-structure for small changes in spans. The cost of pier will not be affected cost of decking it means floor way. Generally remains unaffected due to variation in the span, but cost of main girders and cross-girders is directly proportional to the span of the bridge. The span of the bridge is considered to be most economical when cost of main girders ⨍ cross girders of one span. Equals the cost of piers of one span.1

Loading on the bridge:

Before any structural member is designed, values of max. B.M.F.S.F are to be determined in it. Following loads are considered for computing maximum values B.M, and S.F in the cause of bridges.

1- Dead load

2- Live load.

3- Impact effect.

4- Wind loads.

5- Seismic forces.

6- Forces like centrifugal force, longitudinal force and lateral forces on parapets etc.

While designing a bridge. Beside all the forces mentioned above, temporary stresses, secondary stresses and erection stresses should also be suitably considered.1

Dead load:

This load consists of weight of the bridge structure and any other load permanently attached to the bridge, before proceeding with the design work, dead load of the structure has to be assumed first, but after actually designing. The bridge it is actual weight should be check with assumed dead load. If actual load exceeds by more than 2 ½ % the assumed value, design should be revised adopting actual dead loads. 1

Live Load:

For foot bridge:

a- for foot bridges. Subjected to normal pedestrian traffic live load should be taken as (400)kg1m².

b- where foot bridges are likely to be crowded by pedestrians, at times the live load should be taken as (500)kg/m².

Krebs having width of (60)cm or more, are designed for above a said live loads. In case width of Krebs is less than (60)cm, no live load should be assumed to be acting on the kerb.

Main, girders, trusses, arches etc. supporting the foot paths should be designed for following live loads:

a- For foot bridges up to (7.5)cm effective span live load should be taken (400) kg/m² for ordinary conditions and (500)kg/m² where crowing in expected.

b- For foot bridges having span between 7.5m and 3.0 m live load should be calculated using the following

Formula:

$$P=P\left(\frac{40L-300}{9}\right)$$

c- For spans more than (30)m live load on foot bridge should be determined using following formula:

$$P=\left(p^1-260-\frac{4800}{L}\right)\left(\frac{165-\omega}{15}\right)$$

Where: p^1= 400kg/m² or 500kg/cm² as the case may be.

P= live load to be adopted in kg/m².

L= Effective span of the main girder, truss or arch in (m)

W= width of the foot way in meters.

Where vehicles are likely to mount the footway, each part of the foot way should be capable of carrying a wheel load of (4)t which shall be deemed to include impact, distributed over a contact area of (30)cm in diameter, the working stresses should be increased by 25%. 1

Wind loads:

Bridges are also subjects to lateral wind loads the wind force is considered acting horizontally and at such a location on the structure that maximum streets are developed by wind. Intensity of wind force vanes with the height.1

Longitudinal forces:

The bridges are also subjected to longitudinal forces due to tractive and braking effects, the braking effect on a simply supported span or a continuous unit of spans or any other type of bridge unit shall be assumed to have the following values:

(¿) in case of single lane or two lane bridges:

20% of the first train load plus 5% of the load of the succeeding trains or part there of the train loads in one lane only being considered for this purpose.

(¿¿) in the case of bridges having more than two lanes:

As in (¿) above for the first two lanes plus 5% of the loads on the lanes in excess of two. The force due to braking effect should be assumed to act along a line parallel to the road way and (1.2) m above it. While transferring the force to the bearing the change in the vertical reaction at the bearing should be accounted for; the longitudinal force at any free bearing should be limited to the sum of the dead and live load reactions at the bearing multiplied by the appropriate coefficient of friction. The coefficient of affrication at the bearings should be assumed to have the following values:

1- For roller bearing =0.13
2- For sliding bearings of hard copper alloy=0.15
3- For sliding bearing of steel or cast iron or steel =0.25
4- For sliding bearing of steel on Ferro asbestos=0.2. (1)

Piers spacing orientation and type:

Piers shall be located to meet navigational clearance requirements and to give a minimum interference to flood flow. In general, piers should be placed parallel with the direction of the steam current at flood stage adequate provision should be made for daft and ice by increasing span lengths and by vertical clearance by selecting proper pier. Types and by using debris deflectors. Special precautions against scour are requires when large cofferdam are placed in unstable steam beds.

Abutments:

Abutments shall be designed to with stand earth pressure the weight of Abutment and super structure live load over any portion of the super structure live load over any portion of the super structure or approach fill, wind forces longitudinal force when the bearings are fixed and longitudinal forces due to friction or shear resistance of bearing the design shall be investigated for any combination of these forces which any produce the most severe condition of loading. Abutments shall be designed to be safe against turning about the toe of footing base and against crushing of foundation material or over loading of piles at the point maximum pressure.

In computing stresses in abutment the weight of filling material directly over an inclined or stepped rear face or over a reinforced concrete spread footing extending back from the force wall. May be considered as part of the effective weight of the abutment, in the case of a spread footing, the rear projection shall be designed as a cantilever supported at the abutment stem and load with the full weight of the super imposed material, unless a more exact method is used.

The cross section of stone masonry or plain conceited abutments shall be proportioned to avoid the introduction of tensile stress in the material. (2)

Reinforcement of temperature:

Except in gravity abutments not less than1/8 square inch of horizontal reinforcement per fat of height shall be provided near exposed surfaces not otherwise reinforced to resist the formation of temperature and shrinkage cracks.

Wing wall:

Wings walls shall be sufficient length to retain the road way embankment to the required extent and to furnish protection against erosion, the wing lengths shall be computed on the bases of the required road way slopes. Where deflection joint are not used. Reinforcement rods or other suitable rolled sections preferably shall be spaced a cross the junction between all wing walls and abutments to thoroughly tie them together, such bars shall extend in the masonry on each side of the joint for enough to develop the strength of the bars as specified for bar reinforcement, and shall vary in length so as to avoid places of weakness in the concrete at their ends if bars are not used on expansion joint shall be provided at this point in which the wings shall be mortised in the body of the abutment.

Drainage:

The filling material behind abutment shall be effectively drained by weep holes with French drains placed at suitable at suitable intervals.(2)

Piers:

Piers shall be designed to with stand the dead and live loads super imposed thereon. Wind pressure. The forces due to stream current, flooding ice and drift, and longitudinal forces at the fixed ends of spans. Where necessary piers shall be protected against abrasion by facing them with granite Vitrified brick timber or other suitable material with in the limits of damage by floating ice or debris.

Piers Nose:

In stream carrying ice or drift, the pier nose shall be designed as an ice breaker, when a steel angle or other metal nosing is used it shall be, effectively secured to the masonry by means of suitable anchors.

Elastomeric bearing

General:

Elastomeric bearings shall be subject to the requirements of this section and to the sections applicable to the particular types of construction with which they are used.

The elastomers to be used shall conform to requirements. (2)

Design:

Bearings may be plain (consisting of elastomer only) or laminated (consisting of layers of elastomer restrained at their interfaced by bonded laminates). Elastomer compounds of nominal 70 udometer hardness shall not be used in laminated bearings plain bearings. Generally will be restricted by the requirements of this specification to conditions where little movement is anticipated.

L= length of a rectangular bearing parallel to the direction of translation.

W= width of rectangular bearing perpendicular to the direction of translation.

R= Radius of a circular bearing.

t= average thickness of a plain bearing or the thickness of any individual layer of elastomer in a laminated bearing (including the top and bottom layer).

T= total effective elastomer thickness (summation of it's)

S=shape factor (the area of the loads face divided by the side area free to bulge).

$S = \frac{LW}{2t(L+W)}$ for rectangular bearings.

$S = \frac{R}{2t}$ for circular bearings.

The size of the elastomeric pad shall be such that both surfaces are in complete contact with the bearing areas.

The compressive strain of a plain bearing or of any individual layer of a laminated bearing is a function of the average unit compressive stress, the hardness of the elastomer and the shape factor. The compressive deflection of each layer is the product of the strain and the thickness of the layer. The total deflection of the bearing is the sum of the layer deflections. The shear strain of a bearing is a function of the temperature, hardness of the elastomer and average shearing unit stress. The shear deflection of a bearing is the product of the shear strain and the total effective elastomer thickness. These relationships maybe taken from existing test reports but for large bearing or group of standard designs. They shall preferably by verified by test of the particular designs involved.

Bearings shall have built in toper when non parallel load surfaces would otherwise produce a compressive deflection of 0.6T under dead load. Such taper shall be limited to5/8 per foot.

To insure stability the following limits shall be observed:

Plain bearings –minimum L or R=5T

Minimum W=5T

Laminated bearings-Minimum L or R=3T

Minimum W=2T

The total of the positive and negative movements caused by anticipated temperature change shall not exceed 5t.

The average unit pressure on elastomeric bearing shall not exceed 800psi under a combination of dead load plus live load, not including impact. The average unit pressure due to dead load plus live load up lift reduce the average pressure to less than 200psi the bearing shall be secured against horizontal crawling preferably by positive attachment to the top surface or to the top and bottom surfaces the bearing maybe subject to momentary light tension.
The initial compressive deflection in a plain bearing or in any layer of a laminated bearing, under dead load plus live load, not including impact shall not exceed 0.7t. the deflection can be determined from a plot showing the relationship of shape factor, load and the udometer hardness of the elastomer under consideration. These curves are generally available from manufactures for their product.

Footings:

Depth: the depth of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Except where solid rock is encountered or in other special cases, the footings of all structures, other than culverts which are exposed to the erosive action of stream currents preferably, shall be founded at a depth of not less than 4 feet below the permanent bed of the steam piers and arch abutments, preferably, shall be founded at a depth of not less than 6 feet below stream bed. The above preferred minimum depths shall be increased as conditions may require.
Footing not exposed to the action of stream currents shall be founded on a firm foundation and below frost.

Footing for culverts shall be carried to an elevation sufficient to secure a firm foundation, or heavy reinforced floor shall be used to distribute the pressure over the entire horizontal area of the structure in any location liable to erosion, a proms or cut-off walls shall be used at both ends of the culvert and where necessary, the entire floor area between the wing walls shall be paused. Baffle wall or struts a cross the unpaved bottom of a culvert barrel shall not be used where the stream bed is subjected to erosion. When conditions require, culvert footings shall be reinforced longitudinally.

Anchorage:

Footing on inclined smooth solid rock surfaces which are not restrained by an over burden of resistant material, shall be effectively anchored by means of anchor bolt, dowels, keys or other suitable means. (2)
All footings shall be designed to keep the maximum soil pressures with in safe bearing values in order to prevent unequal settlement footing shall be designed to keep the pressure as nearly uniform as practicable. In footings having unequal pressures and requiring piling, the spacing of the piles shall be such as to secure as nearly equal loads on each pile as may be practicable. (2)

Spread footings:

Spread footings which act as contrivers maybe decreased in thickness from the junction of the footing slab which column or wall toward the edge of the footing. Provided sufficient section is maintained at all points to provide the necessary resistance to diagonal tension and bending stresses. This decrease in section may be accomplished by sloping the upper surface of the footing or by means of vertical steps. Stepped footings shall be cast monolithically. (2)

Internal stresses in spread footings:

Spread footings shall be considered as under the action of down ward forces, due to the super imposed loads, resisted by an upward pressure exerted by foundation materials and distributed over the area of the footings as determined by the eccentricity of the result of the down ward forces. Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being assumed to carry its computed proportion of the total footing load. (2)

Reinforcement:

Footing slabs shall be reinforced by bending stresses and where necessary for diagonal tension. The computed stress in the bar shall be developed in band.

The reinforcement for square footings hall consists of two or more bands of bars. The reinforcement necessary to resist the bending moment in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth of the fact shall be the depth from the top to the plane of the reinforcement. The required reinforcement shall be spaced uniformly a cross the footing, unless the footing width is greater than the side of the column or pedestal plus twice the effective depth of the footing plus one half the remaining width of the footing. In order than no considerable area of the footing shall remain un reinforced additional bars shall be placed outside of the width specified, but such bars shall not be considered as effective in resisting the calculated bending moment . for the extra bars a spacing double that used for the reinforcement with in the effective belt may be used.

Transfer of stress from vertical reinforcement

The stresses in the vertical reinforcement of columns or walls shall be transferred to the fact by extending the reinforcement in to them a sufficient distance to develop the strength of the bars in bond or by means of dowels anchored in the footings and over lapping or fastened to the vertical bars in such manner as to develop their strength. If the dimensions of the footings are not sufficient to permit the use of straight bars may be hooked or otherwise mechanically anchored in the footings. (2)

Concrete structures General:

The work covered by this section of the specification consists in furnishing all plant equipment. Material and labor in performing all operations in connection with all concrete structure (except concrete pipes) including box culverts, retaining wall concrete and steel, or concrete and timber. Such structures shall be as indicated on the drawings and in conformity with the lines grades dimensions and details there shown. They shall also be in accordance with the prevision of the specification pertaining to the various material and concrete items which enter into and form a part of the complete structure and with the directions of the engineered representation. (3)

Construction requirements:

1-all construction details shall be in a correct with the requirements.

2- water proofing for structures.

3- bridge bearing and movement joints.

4-bridge railing :bridge railing including material parapets hand railings and safety fencing, guard or collision, rails.

5- Excavation and backfill, backfilling around and over concrete structures and behind retaining walls shall not be done unit the concrete has set at least (21days)
6- Drainage of structure: the filling material to back of abutments retaining wall and wing walls shall be thoroughly and effectively drained by means of titling French or rock drains, and weep holes or other adequate

construction all material to the back of the abutments or wall shall be graded filter material.

7- Opening to traffic: bridges having decks made with Portland cement concrete shall remain closed to all traffic subject to the results of tests made of the concrete for not less than (21days) after placing of concrete, if rapid hardening cement concrete is specified. The opening time shall be depend on the result of tests made of the concrete but in no case shall the time before opening to traffic be less than seven days after the concrete is placed.

The above time of opening to traffic is applicable when the temperature is above 10C when the temperature is below 10 C , the time of opening to traffic shall be increased at discretion of the Engineer's Representative. Bridges with concrete decks shall not be opened to traffic without the approval of the Engineer's representative payment. Payment for the various items involved in the contraction.

"concrete structures" will be made in according with the specification at the price tendered in the bill of quantities for the several items involved. Any item not included in the bill of quantities which is shown on the drawings or called for by the specification shall be understood, to be include in the rates quoted for other items.

Payment shall include the furnishing of all materials, labor equipment and all items required to complete the work; the construction of weep holes and drains and the back filling with coarse gravel or rock as specified above. Shall be included in the price tendered in the bill of qualities per cubic meter for concrete in place. (3)

Piling for structures:

The work covered by this section of specification consists in furnishing all materials plant, equipment and labor and in performing all operations in connection with the execution of piles, including test piles complete subject to the conditions of contract and in strict accordance with the section of the specification, the applicable drawings and the special specification for particular application.

General

1- Pilling shall conform in all respects with principles.

2- In addition to submitting the records required the contractor shall report immediately to the engineer's representative any circumstance which indicates that the ground conditions differ from those expected by the contractor from his metrically the bearing capacity of the pile.

3- Where piles have not been positioned within the limits described in the contract, no method of forcible correction will be permitted.

4- Eder list: all piles which are intended to remain in place in the completed structure shall be furnished by the contractor in accordance with an itemized list which will be supplied by the engineer. This list will show the number and length of piles required and will be based on information secured from the driving of test piles or other data available to the employer the length shown on this list shall be length required below cut off and the contractor shall, at this own expense, increase the lengths the necessary amount to provide the stripping of reinforcing steel, fresh heading and to reach from cut-off level up to the position of this driving equipment.

5- Piling ordered and not driven: piling furnished in according will the engineer's itemized list, but not incorporated in the finished works, shall be immediately delivered to the employer and shall become his property. The furnishing of additional piles or piles of a greater length than those shown on the engineer's. list shall be at the contractor's risk.
6- Piles destroyed in healing or driving any pile which is damaged or destroyed before or at the time it is being driven shall be replaced by the contractor at his own expense.
7- Preparation for driving: foundation pits , including the construction of cofferdams or cribs where required shall be excavated in accordance before the driving of foundation piles is begun unless otherwise approved by the engineer's be made , the amount of the allowance depending up on the character of the material through which the piles are to be driven. Any material forced up between the piles to above the level shown for the button of the foundation pit shall be removed to the correct level before the foundation concrete is placed if too great an allowance is made for upheaval due to the driving of piles. Back filling with gravel will normally be permitted to raise the pit bottom to the correct level.
8- Elevation of cut off: the tops of all piles shall be sawn cut or stripped to a true plane.
9- Piles driven below cut off: piles driven below the cut off level without the engineer's authentic shall be withdrawn and replaced by new and if necessary longer piles or shall be extended at the expose of the contractor.
10- Equipment for driving:
1- Hammers
2- Leads

3- Water jets

11- Test piles: the contractor shall drive test pile to determine the length of pilling required to obtain the necessary load carrying capacity or penetration.

12- Loading test: the size and number of the piles shall be determined by actual loading tests. These tests shall consist of the application of test loads together with suitable apparatus for accurately determining the super imposed weight and the settlement of the pile under each increment of load.

Scour protection and channel training works:

Protection of foundations against scour:

General principles of design:

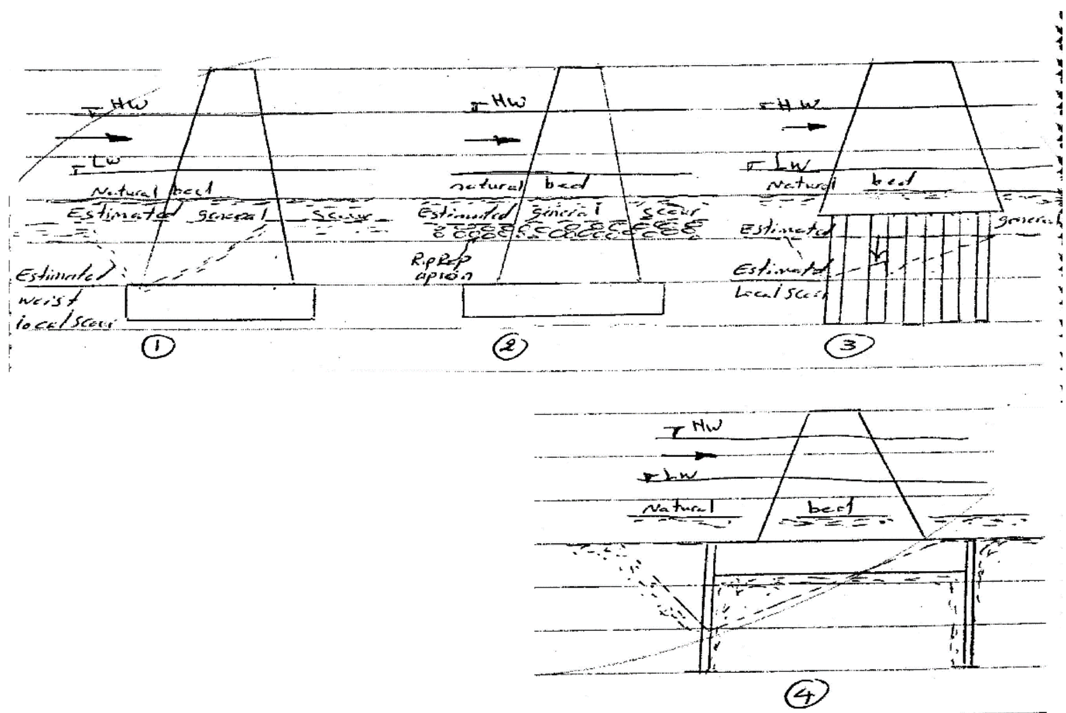
The need for scour protection can be minimized by locating bridges on stable tangential reaches of channels and by placing foundations on in erodible materials such a solution is not always practicable, economical, or desirable from a road alignment stand point.

Having estimated the probable lowest scour levels several choices are open to the designer in selecting the type and elevation of the foundation. (3)

1- Place the bottom of the pier footing below estimated lowest scour levels, making allowance for local scour caused by the pier shaft and footing and including an appropriate margin safety.

2- Place the bottom of the pier shaft below estimated lowest general scour and provide protection against local scour effect.

3- Support the pier shaft or footing on piles or column sunk well below lowest scour levels and designed to be secure when their upper parts are exposed by scour.
4- Construct the pier in the form of arrow of piles or columns with out a footing or solid sinking these well below estimated scour levels and designing them to be secure when their upper parts are exposed by scour.
5- As in (3) above but protect the upper parts of piles or columns against exposure by local scour.
6- Protect the spread footing or piles against undermining by means of a sheet pile skirting tied to the foundation the skirting must itself be designed against scour and loss of support. (3)



Various ways of catering scour in the design of pier foundations.

1- Bottom of footing placed well below the estimated lowest scour level.
2- Footing placed below the general scour level and protected by a rip- rap apron against local scour.

3- Pier supported on piles design to with stand the exposure of their upper parts.

4- Footing protected by a sheet pile skirting.

The choice among these and other possible alternative depends on a great many factors including load bearing requirements, subsoil conditions actually encountered, economics feasible construction methods and schedules inspection procedure…etc the following points maybe relived in making a choice.

1- Since the worst local scour at a pier or other obstruction it is important that pier shafts and footings liable to the exposed to strong currents should be as narrow as is consistent will structural requirements noses should be stream lined and long axes should be aligned parallel to the principal local current direction. In shifting channels where current directions are difficult to predict it may be advisable to control them by means of training works or to employ a pier consisting of arrow of separate circular columns (the last solution may not be attractive where the stream carries heaving ice or floating debris.

2- Footings or caissons should if possible be kept well below estimated general scour levels.

3- Protecting shallow foundations by helping stone around them(a practice often adopted with existing bridges). Is often an unsatisfactory solution because the stone tends to require continual replacement and the heaps reduce the available water area there by increasing average velocity and scour between the pier.

4- Placing spread directly on non-cohesive material not far below the estimated scour level necessarily entails a certain amount of risk because of the present uncertainty of scour estimates and of possible accidents and unforeseen regime changes but might produce worse scour. The advisability of using piles consideration because the expense of sinking a caisson or footing in non-cohesive material to level easily reached by piles is usually prohibitive.

5- Where there is a history of log or ice jams the possibility that serve scour may occur as a result of massive blockage of water way opening should be considered.

6- Use of a sheet-pile cofferdam to protect footings and piles maybe detrimental if the cofferdam is much wider than the pier itself- since its effect may be to greatly increase local depth of scour.(3)

Protective apron around piers and footing:

Where it's economical to prevent the development of local scour holes immediately around piers. Aprons consisting of stone rip-rop, flexible matters, or other suitable material may be provided. The apron should preferable laid so that it's surface is below the expected general scour level. Where this is not practicable it should be designed as a lunching apron and the reduction in water way area caused by its presence after launching should be taken into account the apron should projected around the nose of the pier by distance equal to(1.5) times the pier width, and should be equal in thickness to twice the DSO size of the stone. Talking the local velocity as approximately (1.5) times the mean velocity through the water way opening where the cost of aprons is appreciable. It may be advisable to conduct model tests as an aid to estimating the required stone size and apron extend.

Maintenance and construction procedures:

Debris caught on piers should be removed as soon as possible serving should be made after floods and jams using divers where warranted. Foundation excavation in granular beds may usually be back filed with the natural river- bed material. During winter construction it may be necessary to back fill using dry imported granular material. For this purpose pit-run sounds and gravels with grain size reasonably representative of the bed material may be used. Day should not be used for back filling pier excavations in sand beds.

Since the clay plug will become an obstruction to flow increasing local scour beyond expectations and losing the advantage of an otherwise tender and hydraulically efficient pier shaft. Excavations for spread footings in cohesive material should be poured neat where possible the back fill material may either be clay. A suitable coarse pit –rum gravel or other selected elevation.

Temporary sheet-pile cofferdams used construction purpose should preferably be removed unless this is liable to caves deterioration of the foundation material. If left in place, they should be cut off at or below bed level, and any projection above expected general scour levels should be taken into account when estimating local scour.

Berms used for cofferdams and for construction cause ways should be removed and the river-bed restored to it is original nature condition immediately upon completion of the work or pier to the flood season. The use of earth berms in lieu of sheet pile cofferdams and work bridges should be viewed with caution and consideration must be given to the adverse effects on pier scour and changing regime that may result from temporarily constricting the river. Work cause ways should be designed to wash out in the event of an unusual flood emergency.

Natural vegetation which serves as protection against erosion should as far as possible be preserved during construction and due weight should be given to the effects of alternative construction procedures on the natural regime of the stream and on the biological habitat. (3)

Application of bank protection and training works:

Reasons for use:

Many bridges require the use of some types of bank protection or training works to protect the bridge and it is approaches from damage by flood water. Modern standard of road alignment are leading to increasing use of training works to make naturally unfavorable sites usable such use should always be considered as an alternative to routing the road way to a more favorable bridges sited a good appreciation of river behavior is required for optimum use of training works.

Specific functions of bank protection and training works in relation to bridges and their apprehenders includes:

- To stabilize eroding river-bank and channel location in the case of shifting streams.
- To economize on bridges lengths by constricting the natural water way.
- To direct flow parallel to piers and thereby to minimize local scour.
- To improve the hydraulic efficiency of water way opening there by reducing afflux and secure and facilitating passage of ice and drift.
- To protect road approaches from stream attach and to prevent meanders from folding on to the approaches.

- To permit construction of a square bridges crossing by diverting the channel from skewed alignment.
- To reduce the overall cost of a road project by diversity the channel away from the base of a valley stop, thereby allowing a reduction in bridge length and height.
- To secure existing works or to repair damage and improve initial designs. (3)

Types:

The principle types of banks protection and training work recommended for use at bridges are as follows.

-bank and slope revetment: rip- rap paving or other covering places on a stream bank. Embankment or head slope to prevent erosion.

-guide banks (sometimes referred to as spur dikes) embankments constructed more or less parallel to the stream to direct the flow smoothly through the water way opening.

-spurs (sometimes referred to as grains) embankments or walls constructed more or less square to a stream bank or shore.

-dikes embankments or walls constructed to prevent flooding of lands adjacent to roads and bridges, channel, diversion, artificial cuts made in order to improve flow alignment through the bridge opening or to facilitate lament and construction of adequate training works of other types.(3)

General principles of designed construction:

The following principle should be followed in designing and constructing bank protection and training works:

1- The cost should not exceed the benefits to be derived permanent works should be used for important bridges on main roads and where the results of failure would be intolerable. Expendable works may be used where traffic volumes are light alternative routes are a reliable and the risk of failure is acceptable.
2- Designs should be based on studies of channel trends and processes and on experience with comparable situations. The ultimate effects of the work on the natural channel both downstream and upstream should be considered.
3- Side reconnaissance by the designer is highly desirable. If circumstances prevent on-site inspection aerial reconnaissance or air photo study are possible substitutes.
4- The possibility of using model studies as a design aid should receive consideration at an early stage.
5- The works should be inspected periodically after construction with the aid of surveys. To check results and modify the design if necessary the first design may require modification continuity in treatment as opposed to sporadic attention is advisable.
6- In lieu of maintaining and existing road or bridge consideration should be given to reacting it away from the river hazard. (3)

Bank and slope revetment:

Selection of revetment type:

The type of revetment to be used will depend upon the cast of materials and upon considerations of durability, safety, and appearance commonly used types of flexible revetment include stone rip-rap, stone- filled wire baskets (gabions), wire much over a layer of stones bagged concrete, and articulated concrete slabs rigid types include poured concrete slabs, soil-cement, asphaltic concrete and log cribs.

Attention is concentrate here on stone rip-rap because of its considerable advantages over other types in many circumstances.

-it is flexible and is not impaired by slight movement of the embankment resulting from settlement.

-local damage is easily repaired.

-no special equipment or construction practices are necessary.

-vegetation will often grow through the rocks.

-additional thickness can be provided at the toe to offset possible scare.

-ware run-up is less than with smooth types.

-an additional advantage is that, far pedestrians it presents less danger of slipping than concrete slab or other smooth types of revetment. (3)

Determination of rip-rap stone size against stream flow:

Methods of selecting the stone size maybe divided into three categories- local experience, empirical rules and hydrometric charts or formulas.

1- Local experience may re lied upon where sufficient installations have been tested in services under floods approximately to design conditions. On the basis of such experience some organization have development empirical data on required stone sizes for the various types of situation they encounter such an uproar was used extensively in the past and still finds application.

2- Empirical rules that may be mentioned include that of blench which reads as follows, although guide is that a large sand bed river will normally need stone about 150 Ib if it does not have a very large bed-load a small bed load charge should use stone at least twice the diameter of the largest material that rolls on the bed if moderate attack is expected for very violent attack, as at a major spur nose three times size is safer.

3- The required size of stone for stability depend theoretically on the local flow velocity adjacent to the slope, stone density, depth of flow, degree of turbines or eddying, curvature of flow and slope angle practical formulas and charts published by different agencies ray considerably in their predictions, however a general difficulty in using hydraulic criteria is that the size obtained is quite sensitive to the assumed flow velocity which may be difficult to estimate in practice. A systematic method of allowing for curvature and larger-scale turbulence is also lacking.(3)

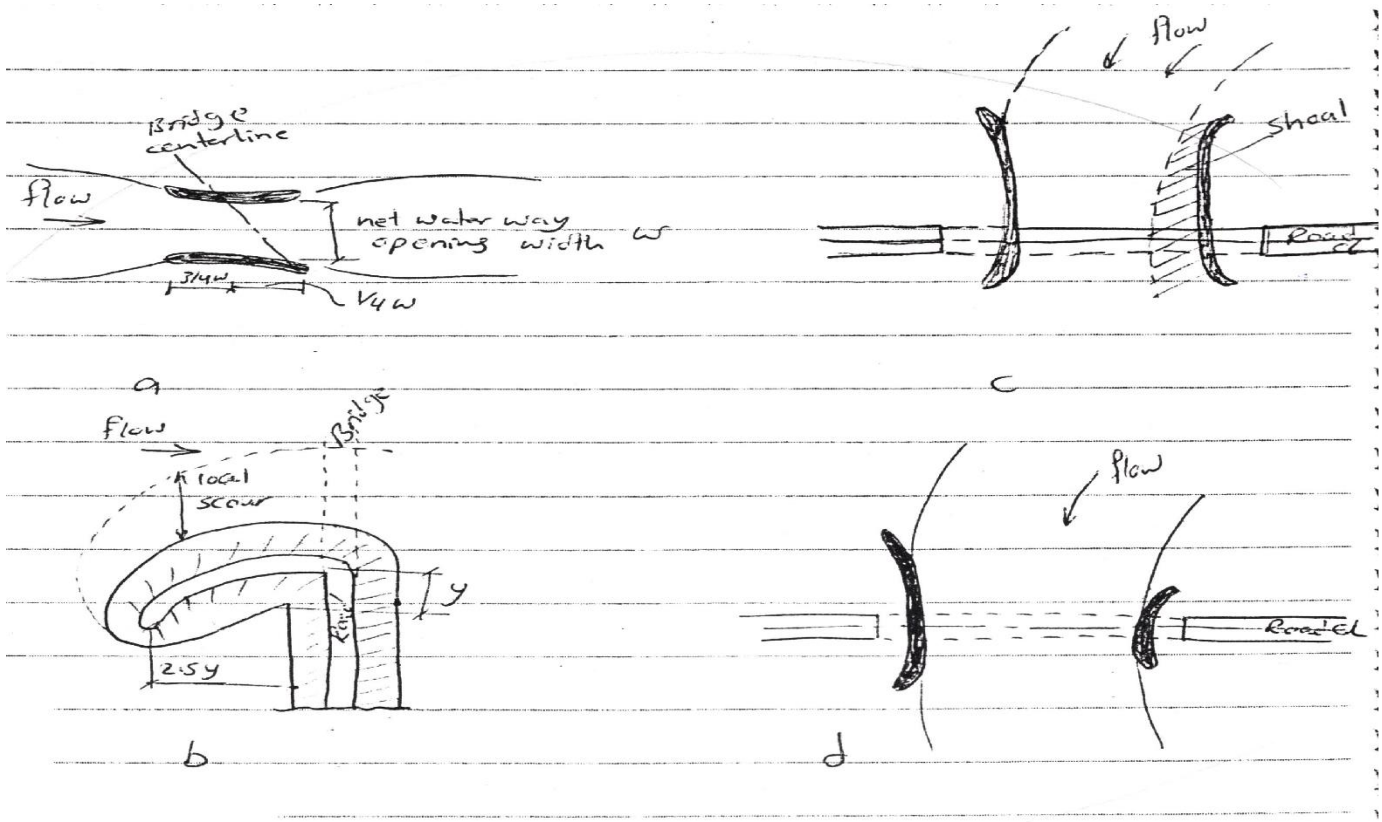
Guide banks:

Uses:

Guide banks may be used to confine that flow to a single channel, to improve the distribution of discharge across the water way opening, to control the angle of attract on piers to break up meander patterns, and to prevent erosion of approach roads. Two guide banks are generally required when the water way opening is located in the middle of a wide flood plain or braided stream where the direction of the main flow can shift from side to side. A single guide bank may be sufficient when the stream is confined to one side a valley or where advantage can be taken of a natural in erodible bank on one side. (3)

Width between guide banks:

The minimum width between guide banks should be selected to provide the required water way opening area through the bridge in additional to limitations imposed by allowable scare, back way etc. the degree of constriction may also be limited by construction procedures it is difficult and expensive to place an earth embankment in flowing water and preferable to construct it on dry land or in still water. When choosing the location of guide banks, and before proceeding with a final design it is essential to check that the sub oil is capable of carrying their weight.



Length and plan shape of guide banks.

a- Suggested length of guide banks in shifting alluvial river.

b- Plan shape of short up stream guide bank,

c- Straight parallel guide banks tending to case formation of a shoal on one side an elliptical shape is preferable on the inner bank here.

d- Combination of straight and curved banks on a channel bend.(3)

Spurs:

Uses and construction:

Spurs may be used singly or in groups to permit the erosion of road embanks, dikes or natural river banks or in lieu of guide banks to direct the flow into a constricted bridge water way opening, they can consist of embankments structures such as a double row of piles filled with cut trees, suitably selected embankments are recommended as the first choice.

Orientation: when constructed in the form of earth embankments, spur should generally be pointed upstream so as to create a dead water pond which provides a cushion to prevent erosion of the upstream face it is then necessary to place protection on the spur nose only if pointed downstream to act as flow deflectors the upstream faces many require protection against erosion along their full length. (3)

University Bridge / Cairo Egypt:

There are eight across the Nil in, Cairo. The university bridge is the longest one among them. The central shipping area is (100) meters wide and secures a clear height of (12.3) meters above the water level it was complete in (1958)
Length of bridge,484 meters(7spans)
Width of bridge, 30 meters
Weight of steel, 2870 tons. (4)

Old timber bridge/ Tokyo

Old Japanese bridge were generally Timber bridge made of wooden piles and beans this wood cut picture, drawn by Hiroshige in about 1850. Show a representative old Japanese wooden bridge and also a peaceful land scape of the Sumida river in afternoon shower. (4)

Tay road bridge/ Dundee Scotland:

The Tay road bridge which spans the river Tay on the east coast of Scotland and links the busy manufacturing city of Dundee in Anus with Newport in fife completed in (1966) the (U2) span bridge is (2250) meters long and (9) meters wide. Along with the firth road bridge it route from London to Alberdeen via Edinburgh.

Arch bridge/ Hortobagy Hungary:

A concrete arch bridge over the Tisza river which flows through the Hortobagy the north eastern province of Hungary the land escape around here is famous. (4)

Double Deck bridge/ Moscow V.S.S.R:

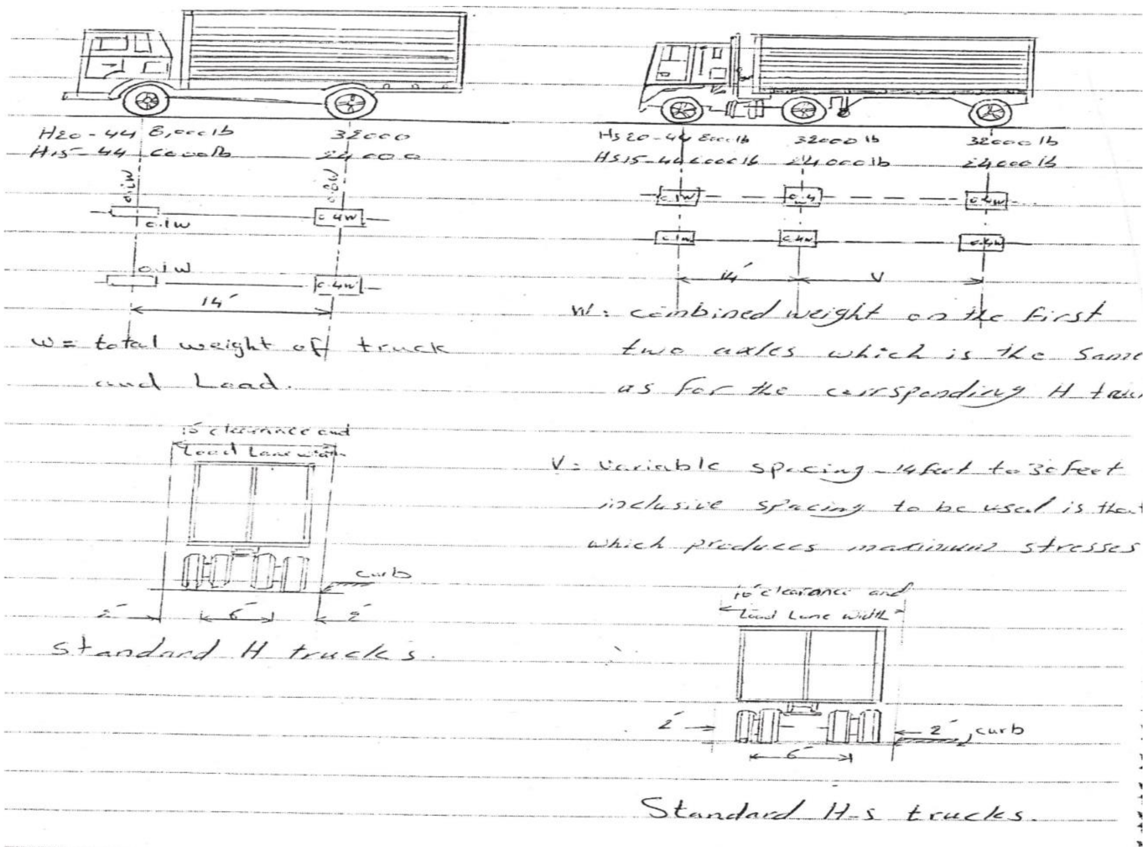
This double deck bridge over the Moscow river concretes the central street with the south eastern district of the city. The upper deck is used for traffic and pedestrian and the lower is for subway. (4)
Suspension Bridge:

Maracaibo bridge over the lake Mara Caibo/ Vene Zuela:

This bridge is difficultly called the general Rafael urbanite bridge. It's greatest of the pre stressed concrete bridges ever built with the cost of (90) million dollar and a term of (40)months, it was completed in (1962). (4)
Length of bridge 8678 meters.
Length of span from (22) to (160) meters.
Width of bridge 17.4 meters.

Standard H. truck loading

standard Hs truck loading



W= total weight of truck and load

w: combined weight on the first two axles which

is the same as for the

corresponding h truck.

V: variable spacing 14 feet to 30

feet inclusive

spacing to be used is that which

produces

maximum stresses

Standard H trucks.

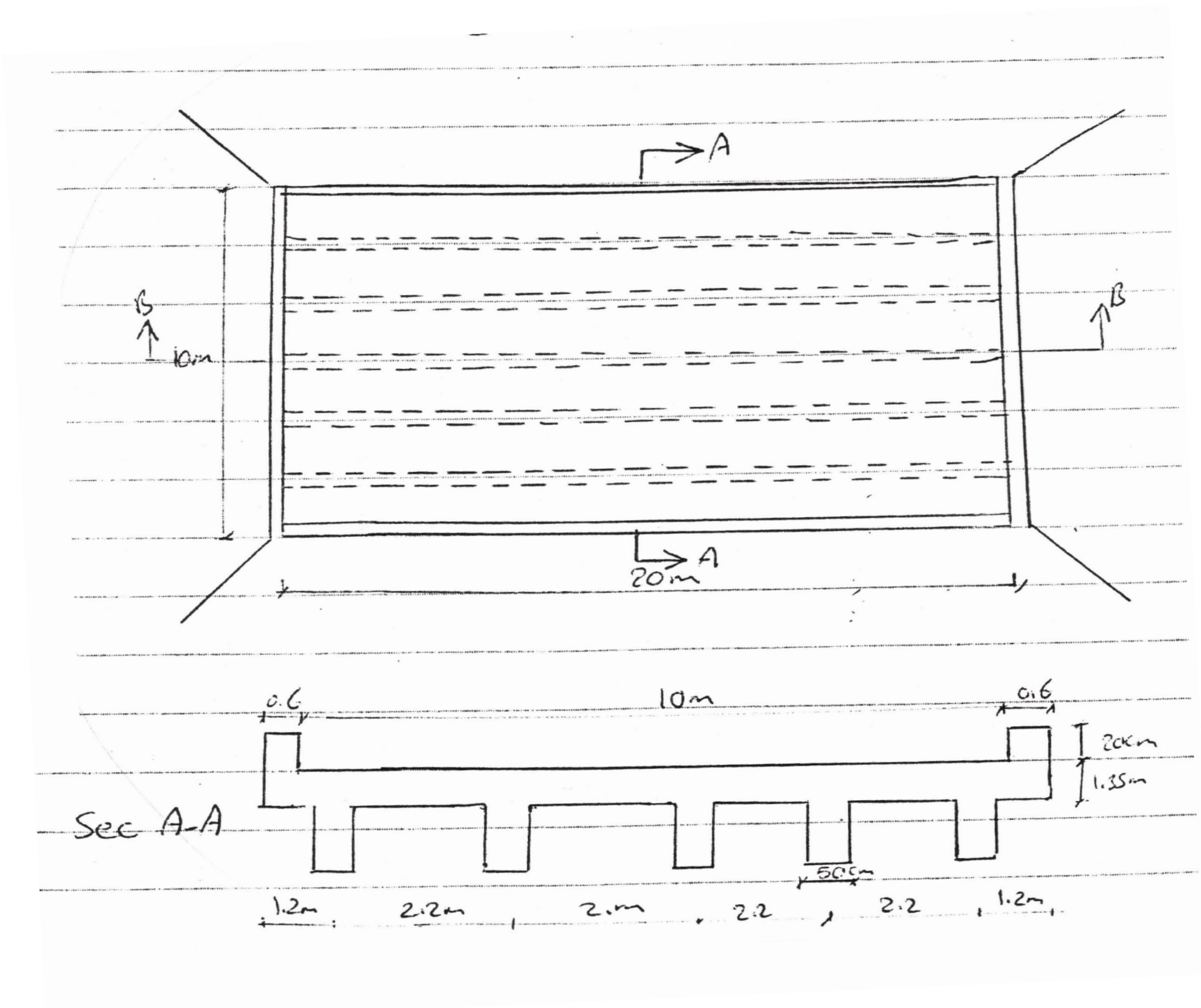
Standard H-s trucks.

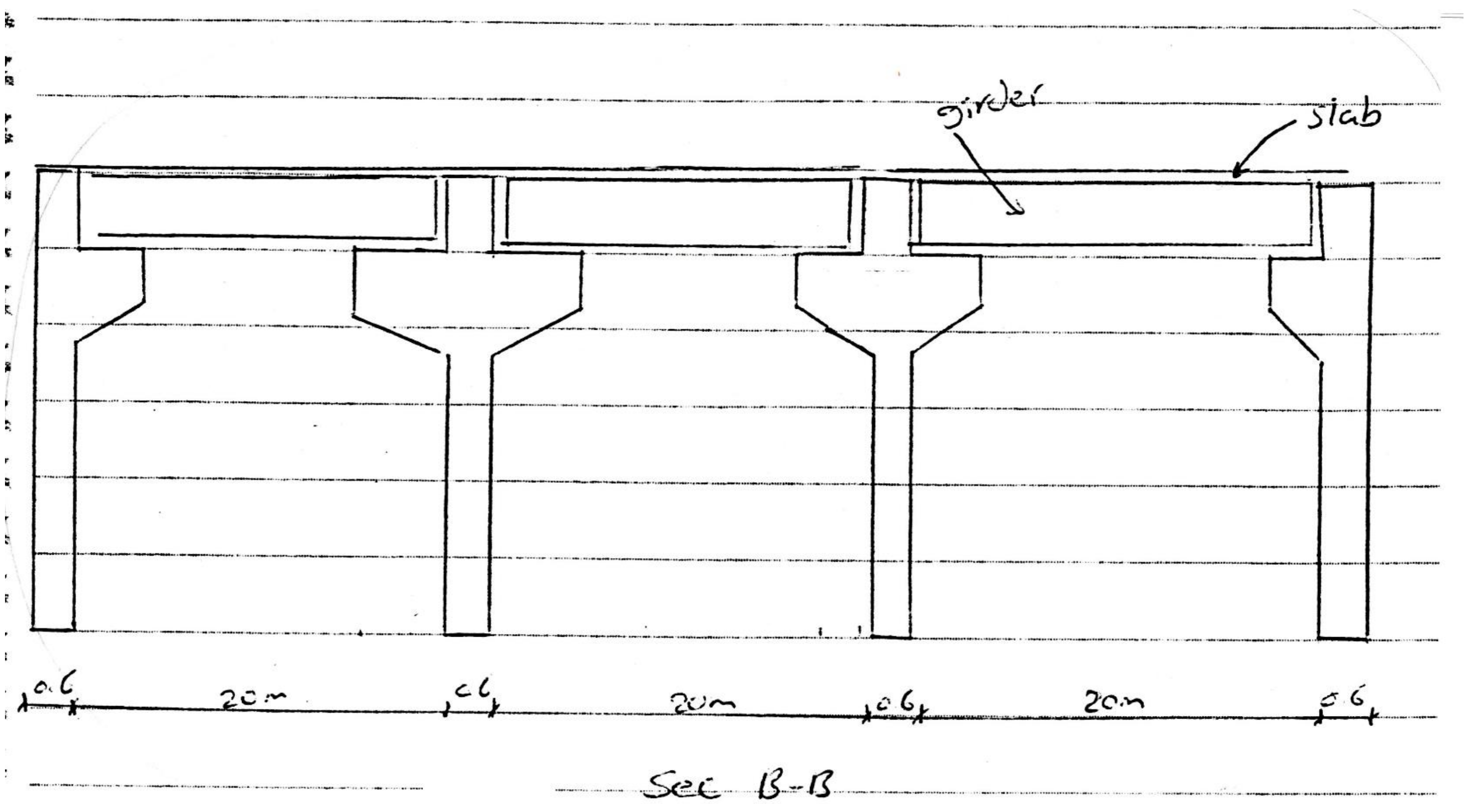
Design a reinforced concrete bridge its cross-section and its dimension shown below for the following given data and according to AASHTO specification, bridge width =10m, clearspans20m. wearing surface=1kn1m³

Fc=21Mpa fc=0.4fc

Fy=276 Mpa fs=0.sfy

H5 15-44 loading





Design of R-C slab:

The slab is continuous over five supports for the continuous span

S=2.2-0.5=1.7m

Assume thickness of the slab= 16cm

Wd.L=0.16x 24+1= 4.84 knlm²

(Md.L)max= $\frac{Wd.Ls^2}{10}$

=$\frac{4.84+(1.7)^2}{10}$ = 1.4 KN.m 1m for +M

and –M

ML.L= $(\frac{S+0.61}{9.74}$x p13.5 =$(\frac{1.7+0.61}{9.74})$ * 54=12.8kn.mlm

For continuous span reduction factor = 20%

MLL=0.8*12.8= 10.25 KN.mlm

I= $\frac{15.24}{5+38}$ = $\frac{15.24}{1.7+38}$ =0.38 >0.3 *use I=0.3

Mtotal= Md.L+MLL+MI= 14.72 KN.mlm

Check thickness d= $\sqrt{\frac{2Mt}{\text{fcbkJ}}}$

Mt=14.72x106 Nmm lm

Fc=0.4 fc^1=8.4Mpa

Fs=0.5fy=138Mpa

B=1m=1000mm

K=0.378

J=0.874

d=$\sqrt{\frac{2*14.72*10 ₍}{8.4*10*0.374*0.874}}$

=103mm

Assume using Ø16mm ⨍20mm clear cover

ts 103+20+$\frac{16}{2}$ =131 <160 0.k

*use ts 16cm

dprovide=160-20-$\frac{16}{2}$ =132 mm

as main (+ve) =$\frac{Mt}{2fs\ Jd}$

-ve

As= $\frac{14.72x10_6}{2138x0.274*132}$ =924.5mm²lm

Use Ø 16.mm @21cm clc as=9s7>924.5 0.k

As distribution=$\frac{121}{\sqrt{s}} \div \frac{as\ main}{100}$

=$\frac{121}{\sqrt{107}}$ x $\frac{92405}{100}$ =857 mm²

Use Ø16mm @23cm clc as=874>857 0.k

Design of reinforced concrete girders:

Effective span of the flange width = (bf)

1- Span 14=20.614
2- Clc of girders =2.2m
3- 16hf+ bw=3.06m

Bf=2.2m

Wd.L=4.84x2.2 +0.5 x1.19x24=25knlm

Md.L= $\frac{wd.LxL2}{8}$ =$\frac{25x20.6^2}{8}$ =1326 kn.m

Live load moment (truck load)

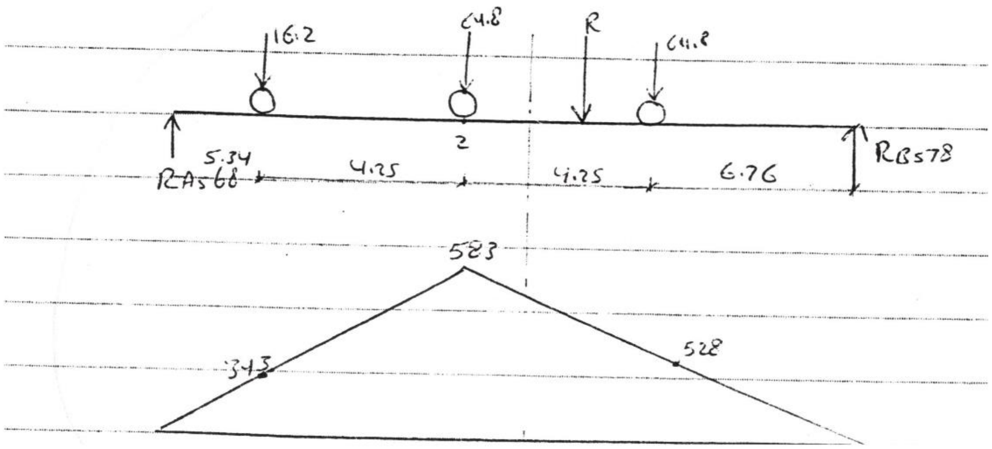
Fraction of the wheel load=511.83=2.2/1.83=1.2m

(No. of lanes=3)

Load from near and middle wheel= 1.2x13.5=16.2kn

Load from near and middle wheel=102x13.5=16.2kn

Critical case of live load



R=16..2 +64.8=146kn

R x= 64.8x 4.25-16.2x4.25

X=1.42

$\sum MB = 0$ Rax20.6-146x9.s9

=Ra=68

$\sum Fy = 0$ Rb=78kn

ML.L max =583 kn.m

Reduction factor for three lanes =10%

MLL= 0.9x583=523=525kn.m

I=$\frac{15.24}{(20.6+38)}$ =0.26 <30%

ML=0.26xML.L=136kn

Mt=1987 kn.m

K=0.378

J=1350mm

Assuming Ø32mm as main reinforcement and Ø 12mm as stirrups and

40mm clear cover.

D=1350-40-32/2 =1282mm

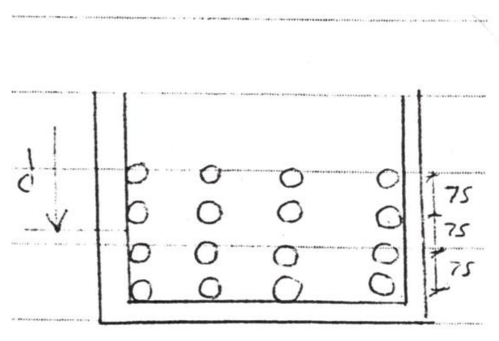
B=500mm

As $\frac{1987x10_6}{fsJd}$ =$\frac{1987x10_6}{138x0.274x1282}$ =12850mm²

As1=214mm²

No.of bars = $\frac{12850}{819}$ = 16bar

Use 16 Ø32mm in four layer



Dact=1282-112.5=1169.5

As $\frac{1987x106}{138x0.874.1169.5}$ =14086.8mm²

No of bars =$\frac{14086.8}{214}$ =17.2

Use 17Ø32mm in four layers

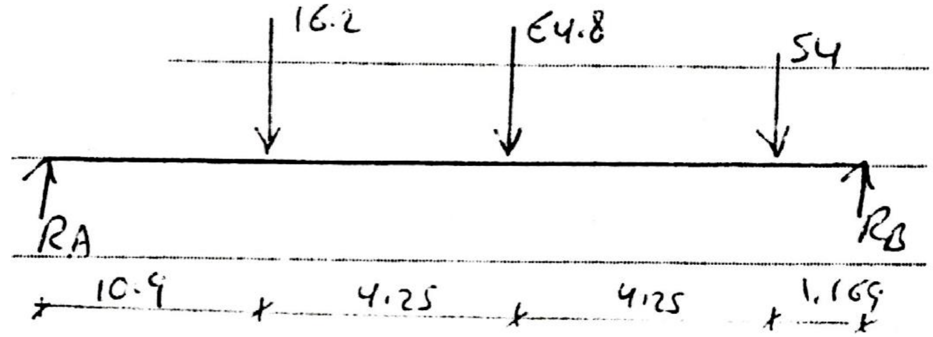
Check spacing, actual spacing = $\frac{(500-2x40-2x12-4x32)}{3}$

=89>db o.k.

Check for shear:

Vd.L @ sup=25x$\frac{20.6}{2}$ =258 kn

V @ d= 258-(25x1.169)=228kn



$\sum MA = 0$

-RBx20.6+16.2x10.9+64.8x15.15+54x19.4

RB=107 kn

VL.L = 0.9x107 =96.4 kn

VI=0.26x 107=27.8 kn

Vu=27.8+9604+228=352 kn

Check cross sectional size and calculating web reinforcement.

V:normal shear strength at service loading Nlmm²

V= $\frac{v}{bwd} \leq 0.41\sqrt{fC}$

=$\frac{352x1000}{500x11695}$ =0.6 mpa < 0.41$\sqrt{21}$ =1.88 0.k

Vc=0.079 $\sqrt{21}$ =0.362 MPa

v>vc shear reinforcement required

vs=v-v=0.238 MPa < 0.125$\sqrt{21}$ =0.57 0.k

s= $\frac{226x138}{0.238x500}$ =262

<d12

<600mm 0.k

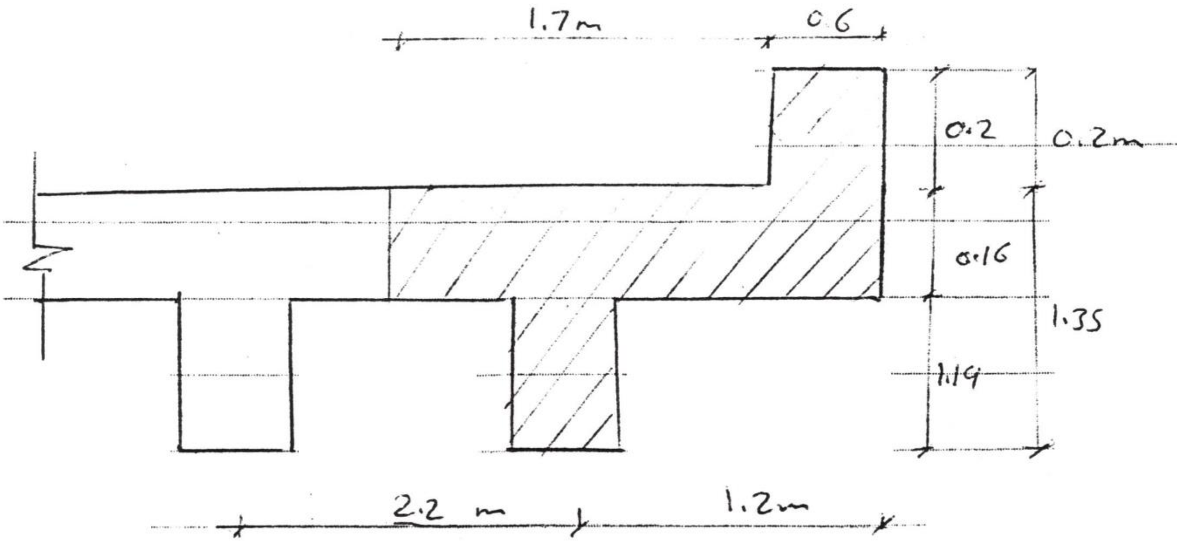
Use Ø 12mm @ 25cm clc

From support of distance which v<vc where v<vc use min

S= $\frac{avfy}{o.345\ bw}$ =362 <d12 , <600mm o.k

Ø12mm @35cm clc

b. Design of extensor girder:



Wd.L=4.84x1.7x +(0.5 x1.19x24)+(0.6x0.36x24)

=228 knlm

Md.l=$\frac{wd\ell 2}{8}$ = $\frac{28x(20.6)^2}{8}$ =1485.3 kn.m

Live load moment

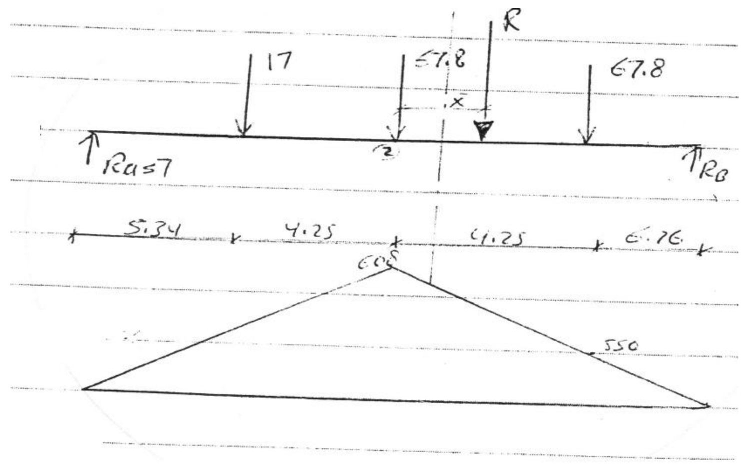
Fraction of wheel load = $\frac{2.3}{1.83}$= 1485.31.257

Load from rear ∱ middle when = 1.257 x 54=67.8 kn

Load from front wheel = 10257 x 54=67.8 kn

Load from front wheel= 10257x 13.5 = 17 kn

Resultant= 67.8+67.8+17= 152.6 kn



$\sum 4250$

67.2x4.25-17-4.25=152.6 x

X=1.42

$$\sum MB = 0$$

Ra x20.6-152.6x9.29=0

Ra=≥ kn

$\sum Fy = 0$ Ra=81.55kn

$(MU)max$ @point 2=608 knm Reduction =10%

MLL =0.9x608=547knm

MI= 0.26x547=142knm

Mt=2174.3 knm

K=0.378

J=0.874

H=1350

J=1169.3

As=$\frac{2174.3x10_6}{138x0.874x1169.3}$ =15417.1 mm²

As=819mm²

No. of bars= $\frac{15417.1}{819}$ =19 bars

Use 19 Ø 32mm as=15561>15417.1mm² o.k

Design for shear:

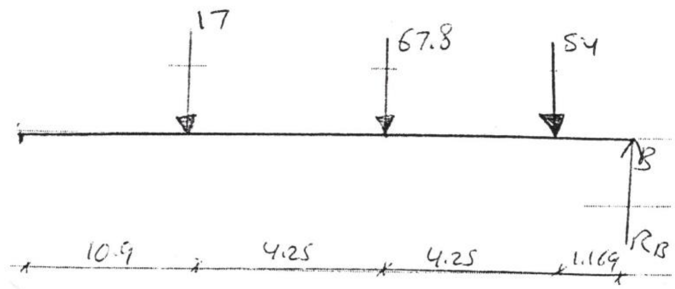
V0.L=28x$\frac{20.6}{2}$ =20.6x1.169 =264kn

$\sum MA = 0$: −RB=20.6 + 17x10.9 + 67.8x15.15 + 54−19.4=0

$RB = 110$ kn

$Reduction\ 10\%$

VLL=110x0.9=99kn



VI= 26 kn

VU=389kn

V=$\frac{VU}{bwd}$= $\frac{389x1000}{500x1169.5}$ = 0.66 <1.88 o.k

Vc=0.079$\sqrt{21}$ =0.362 MPa

Vs=0.66−0.362 = 0.298 MPa

S= $\frac{226x138}{0.298x500}$ =209 mm <d/2 ok

Use Ø 12mm @1.8cm clc

Design of the column:

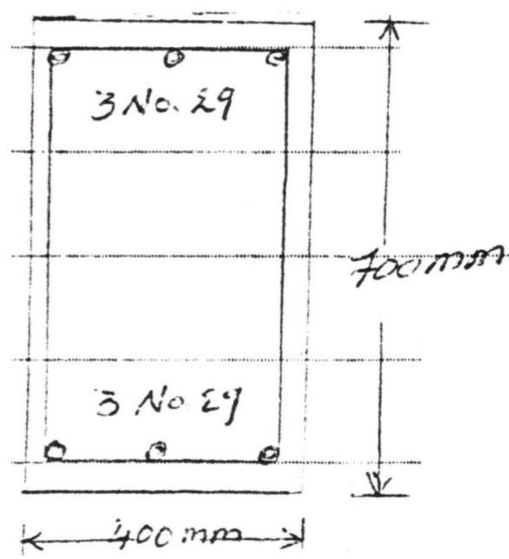
Assume: b=400 mm

H=700 mm

Fc=12 Mpa

Fy=276Mpa

As=As =3*645 =1935mm²



Ast= As+As =2*1935

=3870 mm²

Pn(max)=0.8 (0.85 fc (Ag-Ast)+Asfy)

=0.8(0.85*21(700*400-380)+3870x276)

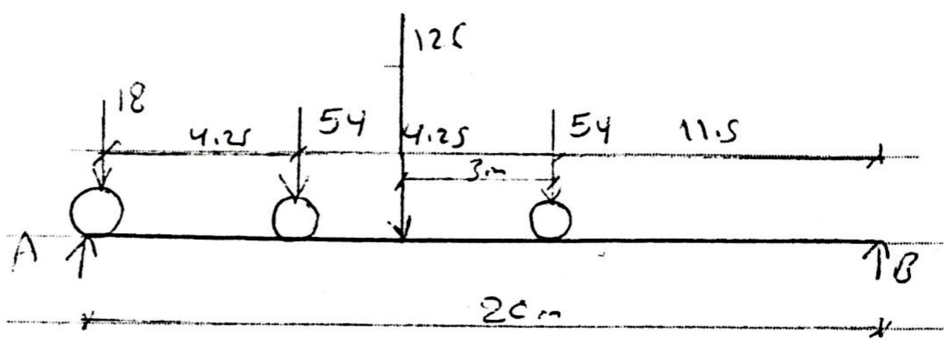
Pn(max)=4797.6 kn

Pu(max) =0.7*4797.6 = 3358kn

Pu(external) =Wd.L + WL.L

Wd.L =25*3*20 + 28x2*20

=260kn

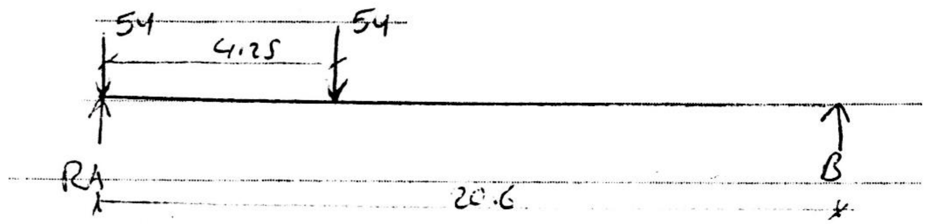


$R= \frac{p.r}{L}$

$= \frac{126x(3+11.5)}{20}$ =91.33 kn

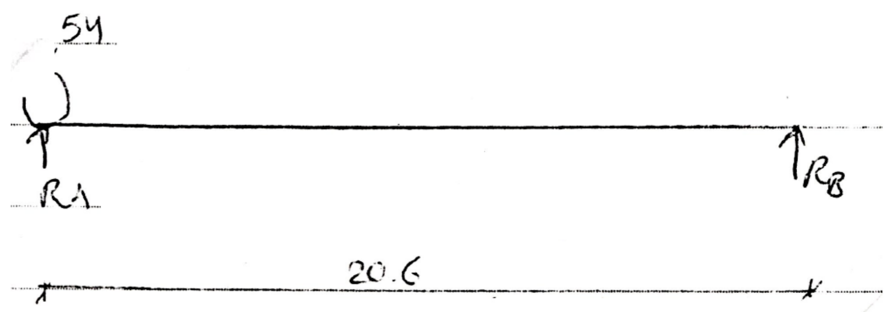
$R=\frac{108(3+11.5+4.25)}{20}$

=101.25kn



$R=\frac{54(3+11.5+8.5)}{20}$

=62.1 kn



Rmax=101.25kn

Pu(external)= Wd.L +WL.L

=2620+ 202.5 =2822 kn

Pu external= 2822 <3358 kn 0.k

Design of pedestals:

Wd.L =25x3 +28x2 =131 knlm

wL.L= 4x101 =404 knlm

wa=Wd.L +WL.L

=131+404 =535 knlm²

Mu=$\frac{WuL^2}{2}$ =$\frac{535(5)^2}{2}$ =6687 kn.m

Mu=Ø Mn=ØAsfy (d−$\frac{a}{2}$)

A=$\frac{AsFy}{0.85fcb}$ =$\frac{Asx276}{0.85x12x1000}$ =0.0154As

Assume d=400mm

6687= 0.9As x276 x 10^3 (0.4 −$\frac{0.0154A}{2}$)

6687= 99360 As−1912.68 As^2

As^2−52As +3.49 =0

Solve As= 0.067m^2

Vu =we−dw

Vu=535 x5 −0.4 x 535 =2461kn

Vc= $\frac{Ø}{6}\sqrt{fc}$ =$\frac{0.85}{6}$ $\sqrt{21}$ =649kn

Vu=Ø vc+ Øvs

Vs=$\frac{vu-Ø}{Ø}$ = $\frac{2461-649}{0.85}$ =2131 kn

Av=226mm^2 , fs=0.5fy =138Mpa

S=$\frac{Av.fs}{vs.bw}$ =$\frac{226x138*1000}{2131\ x500}$ =29.3mm

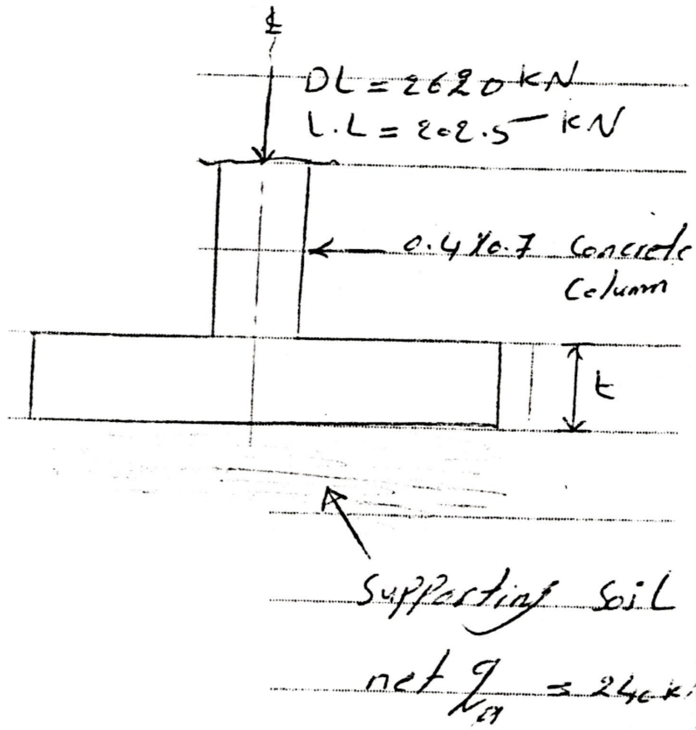
Design a square R.C single column footing

DL=2620kn

Assume design bearing capacity of or soil pressure =240kn

Rein cement = concrete column (400x700)mm with Fc for concrete of belt the footing and the column =21 Mpa.

Fy of reinforcing bars =276Mpa



The base are $\frac{DL+L.L}{net\ \vartheta a}$

A=$\frac{2620+202.5}{240}$ =11.76 m²

The footing is square B=C

A=B² or L²=11.76m

B or L =$\sqrt{11.76}$ =3.5m

Use square footing 35m x3.5m

Compute the factored soil pressure $\vartheta\ fact$

$\vartheta\ factored$ =$\frac{factored\ \ load}{Area,A}$ =$\frac{1.4\ DL+107L.L}{A}$ = $\frac{104(2620)+107(202.5)}{3.5x3.5}$

$= 32.8\ knlm^2$

Vc $= \frac{0.85}{6} = \sqrt{21}$ =0.65 *1000 =650knlm^2

The applied shear force = B^2 $\vartheta fact$ =(3.5)^{2}x 328

=4018 kn

$\sum fv$=0

The applied shear force= resisting shear force

4018 =(2 (0.4+d)+(0.7+d) *d*650 +(0.4+d)

(0.7+d) *328

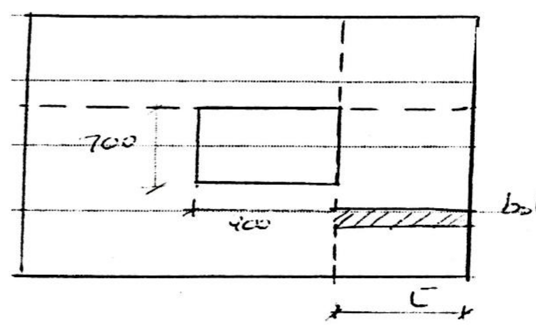
4018=2928d^2 +1790.8d +91.84

d^2+0.61d-1.34=0

solve: d=0.89m

the applied moment /lm of the critical section =Øfact * $\frac{C^2}{2}$

=328 *$(\frac{33}{2})^2$ =1786 kn mlm



Resisting BM=Ø Asfy (d-$\frac{a}{2}$)

A= $\frac{asfy}{0.85*fcb}$ = $\frac{AS*276}{0.85*21*1000}$ = 0.0154 As

Resisting B.M=0.9As *276*1000 (0.89- $\frac{0.0154As}{2}$)

=221076As -1912.68As^2

Applied B.M = resisting B.M

1786=221076 As-1912.68

As^2-115.6 As+0.933 =0

As= 0.0087=0

As=0.0087 m^2lm of section

Fmin for grade 50 → 0.002

Asmin=0.002(1*t)

Ts thickness of footing =0.29 +0.025 +0,07 =0.985m

Asmin =0.002 *1 *0.485m =0.00197m^2

As provided >Asmin

Asprovi =0.0087 x3.5 =0.03045m^2

Area of 25 mmØ = 491mm^2

No. of bar required in each direction= $\frac{30450}{491}$ =62 bars

Assure using 63 bars in each direction check max spacing of bars (smax)

Smax=3d or 0.46m

=3x0.89 =2.67m

No. of spacing = number of bar -1

=63-1=62 spacing

The minimum con c cover for concrete footing = 70mm

3500 = 62 sprov +2*70 +25

Sprov= 0.053 m

0.46 controls > sprovided =0.053 0.k

P provided =$\frac{30450}{3500*890}$ =0.00977

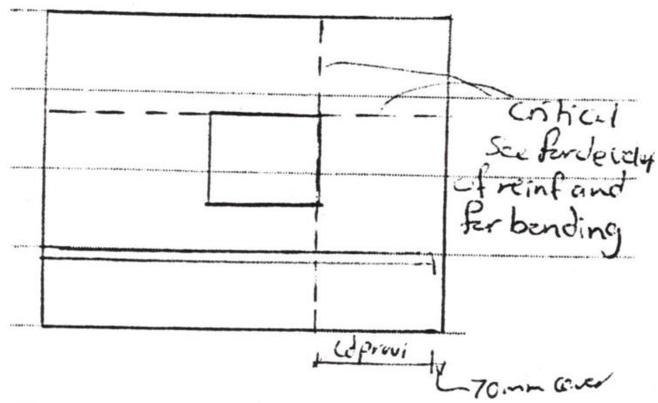
Pmax=0.75 (0.85 x 0.85 *$\frac{21}{276}\frac{600}{600+276}$) =0.0282

Provided < pmax o.k

Ld provided =3300-70=3230mm

Ldmin=305

3230> Ldmin O.K



Check column bearing on footing

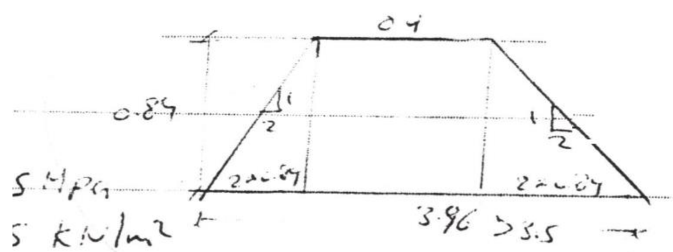
(the applied compression stress at the interface) =$\frac{1.40L+107LL}{104dead\ area}$

=$\frac{104(2620)+107(2c2.5)}{0.4\ z0.7}$ = 14329 knlm²

Øa bearing =0.85 Øfc

=0.85-0.7*21=21 =12.4aSMpa

=124asknlm²



12.495 MPa<21 MPa O.K

Øa bears 12.44s<14.329 Øabears=14.329

Asmin:0.005 A1= 0.005x0.7=0.0014m²

This amount of steel may be provided by extending the 4 Ø25mmcl steel bars (As= 4x4a1=1963> 1400mm²)

Down to footing Assume using 4Ø23mm dowel bars

As provided = 4*415 = 1662 > 1400 mm^2 o.k

Length of dowels embedded in the footing ? for this purpose consider these bars as compression reinf

Ldmin=406.4 mm

D=840mm

d>ldmin o.k

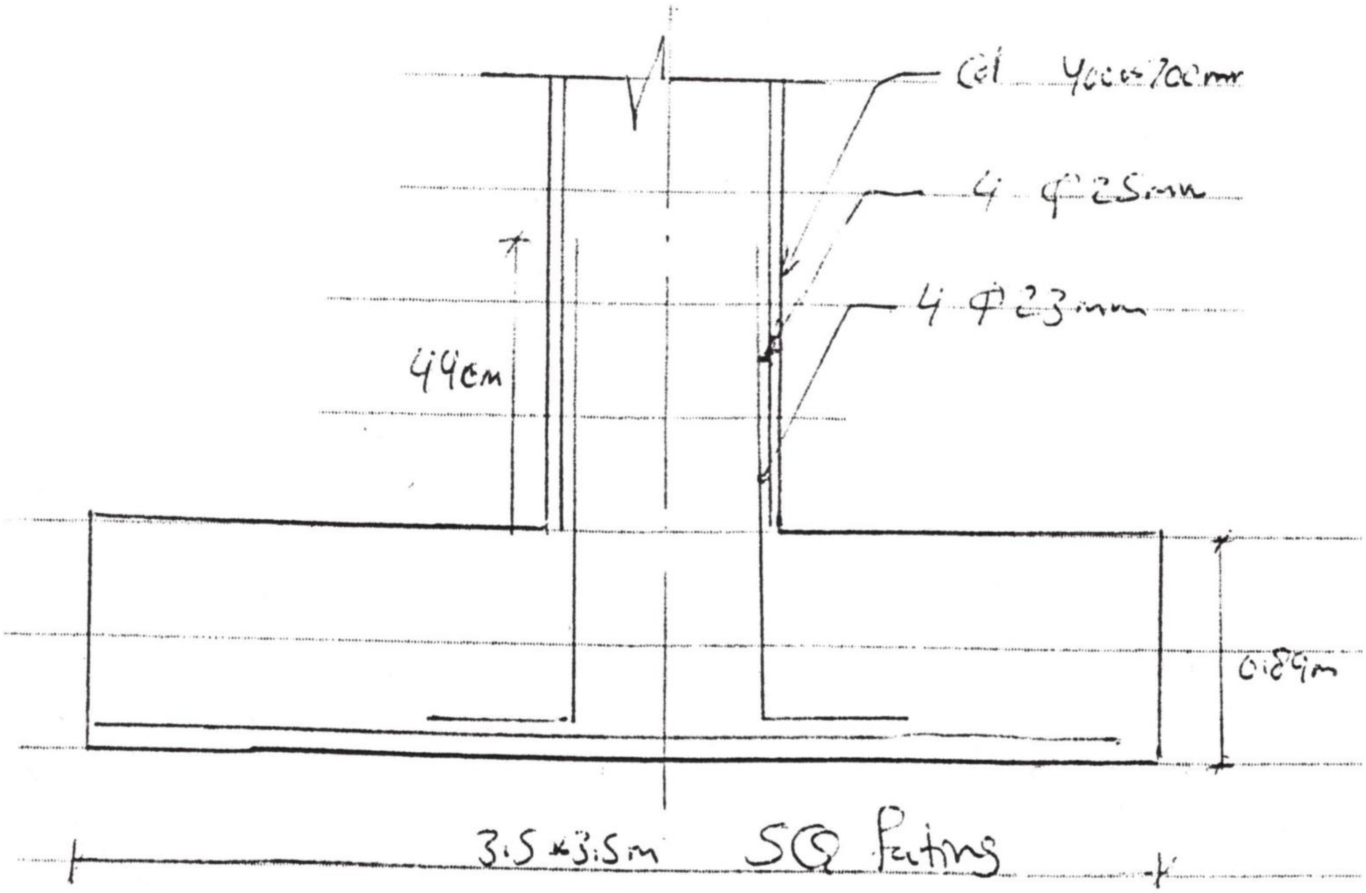
length od dowels above the footing in the column?

Consider the dowels as compression reinf, using length of lap splice

Ldmin= 0.07 * 25 *276* =483mm =48.3 cm

Ld min= 30 cm

Use Ldmins = 48.3 cm say 49cm



Design of Abutment:

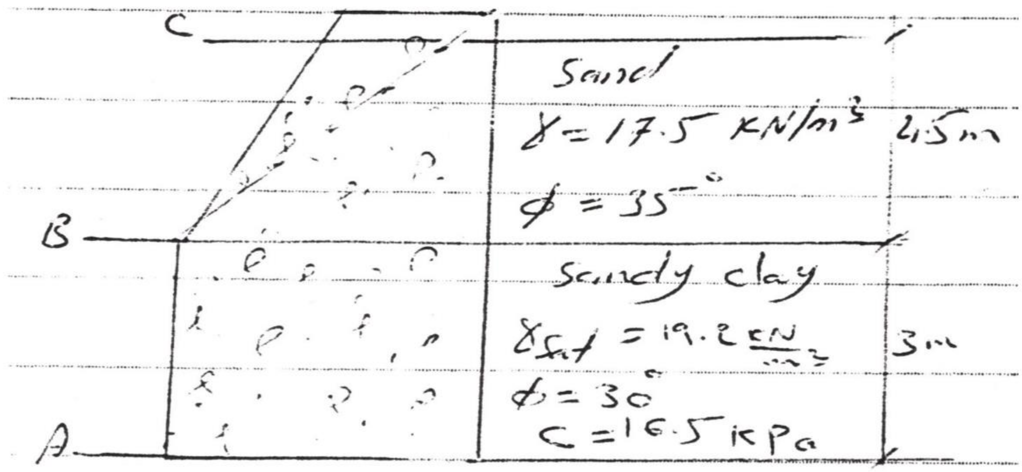
Assume a retaining wall (7.5m)high.

Soil supported consisted (4.5m) of sand $\delta = 17.5knlm^2$

Ø=35 overlying saturated sand clay ($\delta sat = 19.2knlm^2$)

Ø=30, c=165 knlm²

The graind water level is at the surface of the sandy clay.



At level©

$\delta = 0$ pa=0 c=0

At level B

Ka =tan²(45– $\frac{35}{2}$)=0.27

$\delta a = H1\, \delta \,.\, ka1 = 17.5$ (4.5)(0.27)=21.26 kpa)for soil I

Pa1=$\frac{1}{2}$ H1 $\delta a1 = \frac{1}{2}$ (4.5) (21.26) = 47.84 kn

At level (A)

Ka2= tan² (45–$\frac{30}{2}$)= 0.333

$\delta a3 = \ \delta Hi, Ka2\ = 45(17.5)0.333 = 26.22\ kpa$

Pa2= $\frac{1}{2}$ H2 $\delta a2 = \frac{1}{2}$ (3) 26.22 =78.67 kn

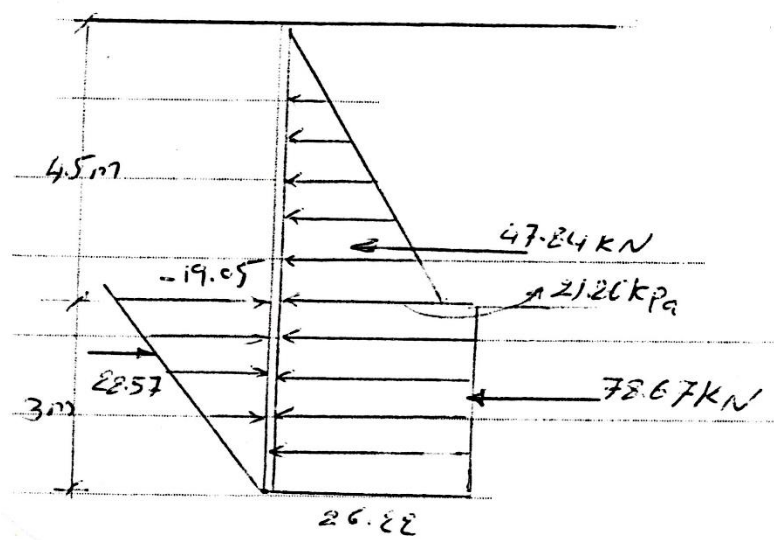
At level (B) soil (II)

$\delta a3 = -2\sqrt{ka2}$ =-2 (16.5) $\sqrt{0.333}$ =-19.05 kpa

Pa3=$\frac{1}{2}$ ht,$\delta a3$

Ht=$\frac{2c}{\delta\ sub.\sqrt{Ka2}}$ =$\frac{2(1605)}{(19.2\ \sqrt{Ka2}}$ =2.97m≅ sm

Pa3 = $\frac{1}{2}$ (3) 19.05 =-28.57kn.

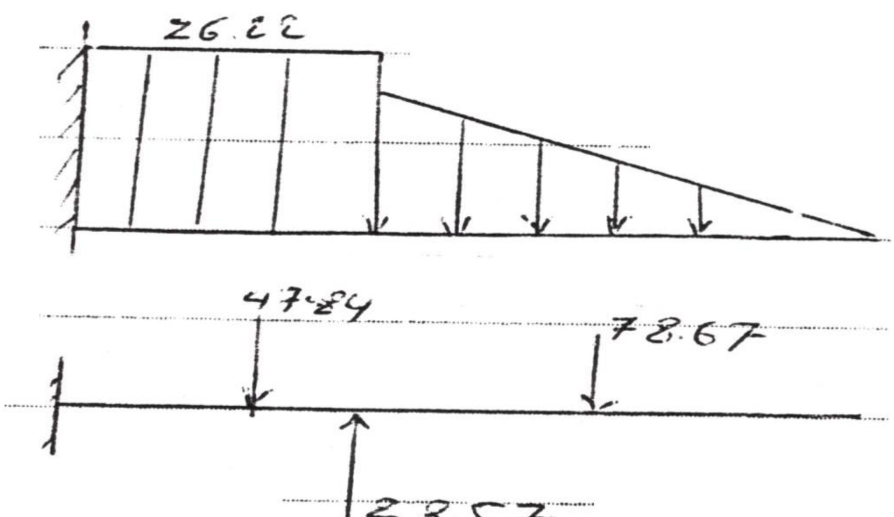


$$\sum Pa = 47.84 + 78.67 - 28.57 = 97.94kn$$

97.94y=47.84($\frac{4.5}{3} + 3$) + 78.67($\frac{3}{2}$) +28.57 ($\frac{2}{3}$ *3)

Y=4.88m1

from (0.0) level.



M.at fixed end

Mu=47.84(4.5)+7867*1.5-2857*2

=276.15 kn.m/1m

Mn = $\frac{Mu}{\emptyset}$ =307

Mn=Asfy(d-$\frac{a}{2}$)

Assume d=0.9m

A=d-$\sqrt{d^2 - \frac{Mu*10_6}{0.425*21*1000!}}$

=22mm

307=As*$\sum 76$ (0-9-22/2)

As=1252 mm²/ 1m

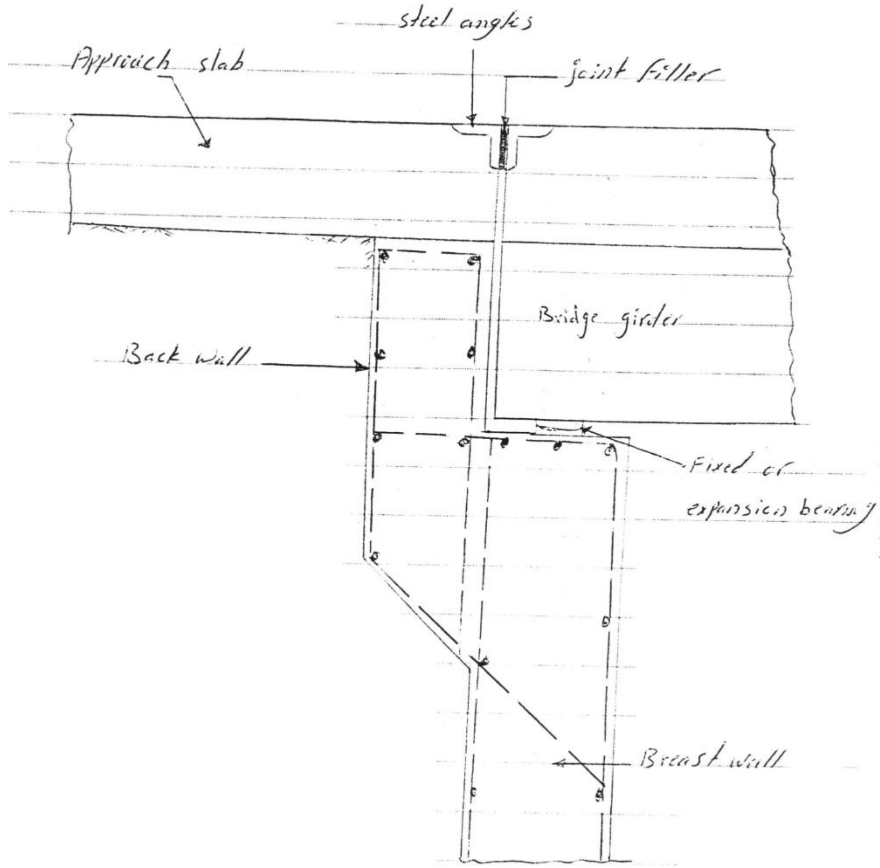
Assume use Ø =25mm

Area one bar =490mm²

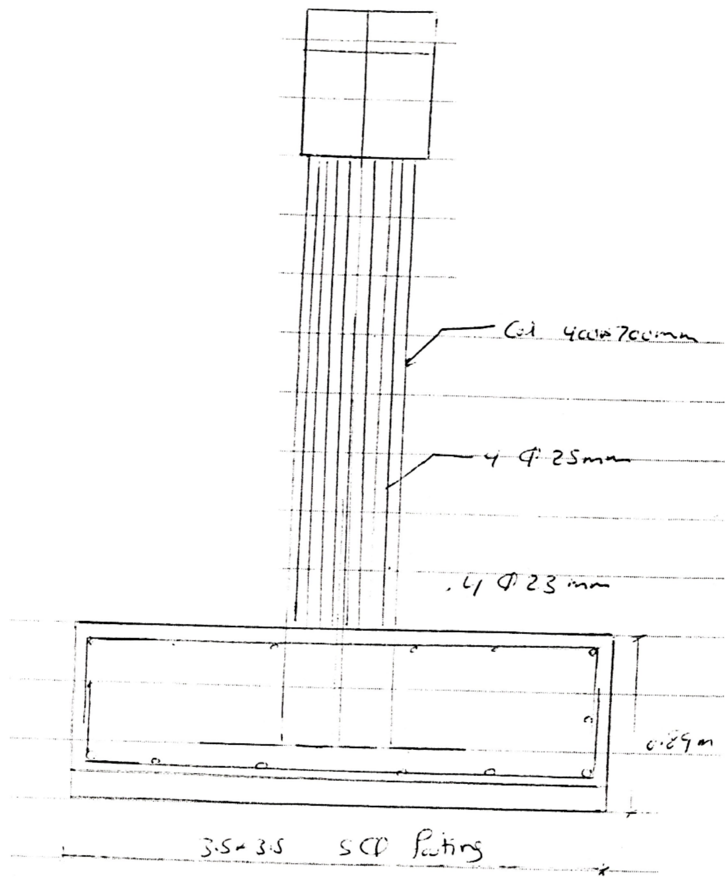
No. of bar =3bar

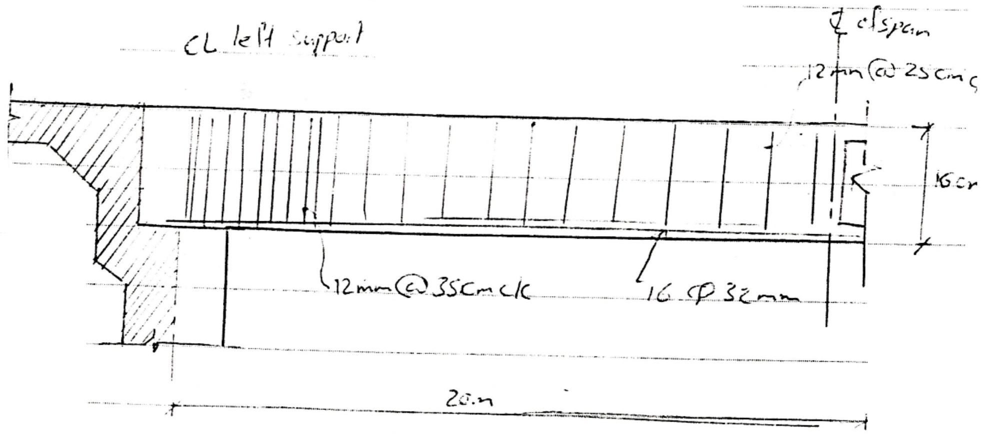
Spacing= $\frac{1000}{3}$ =333mm

Use S=25 cm



" Detail at bridge seat





Detail of interior girder

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