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OVERSEAS ROAD NOTE 9



A design manual for small bridges



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A design manual for small bridges

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OVERSEAS ROAD NOTES

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1 Introduction

This manual offers highway engineers a comprehensive set of guidelines to assist and simplify the process of designing small bridges and culverts. These structures are an essential part of every road network. They are far more common than large bridges and are simpler to design and construct. For the purposes of the manual, 'small bridges' are defined as single or multispan structures with individual spans no more than 12m long, ie taking one span to bridge a two-lane highway with shoulders or two spans to bridge a dual carriageway.

The guidelines cover the entire design process, from the planning stage through site investigations and materials analysis, hydraulic design and structural design, to the final preparation of drawings and detailed specifications. There are many textbooks and other technical publications that provide excellent treatments of all these aspects of bridge design: some are listed in the manual as useful reference material for readers wishing to pursue subjects in more detail. These sources, however, are all intended for bridge engineers or students of bridge engineering. The present manual is meant to be of use in a bridge design office, but it is aimed also at the general civil engineer who is not a bridge specialist but who may nonetheless be required occasionally to construct a road that crosses a river or other obstruction. He/she may be a provincial roads engineer, extending a regional network of feeder roads with permanent bridges, an army engineer or an engineer involved in famine relief distribution, needing rapid but temporary solutions to bridging problems

Because these non-specialist bridge builders have other professional responsibilities, they rarely have the time or expertise to work out all the necessary bridge design calculations from first principles. For this reason, the manual gives as much guidance as possible in the form of drawings and tables, covering two standards of traffic loading, single or multiple spans a range of bridge materials - concrete, steel, timber and masonry - and a range of in situ soils.

Though the **structural** design of small bridges can be simplified by the use of stock solutions, the process of **hydraulic** design cannot be shortened in the same way. The chapters that deal with river hydraulics, hydraulic design and river works (Chapters 4 to 6) contain all the background information and procedures that the bridge designer will need in order to apply the detailed structural tables set out in subsequent chapters, but they assume the knowledge and experience of a qualified engineer as well as the availability of basic facilities for field investigations and soils analysis.

Where there are several possible methods of calculating a variable - for example, allowable

bearing pressure and scour depth - the manual presents only the simplest of these methods but includes references to others. When it is thought likely to be helpful, typical calculations are worked out in the appendices to chapters.

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2 Planning

In this initial stage of design the highway engineer identifies a preferred location for the bridge and decides on the type, size and capacity of the structure.

These decisions are made on the basis of field surveys and information about:

- the local terrain and site conditions;
- the required design life of the bridge;
- the likely traffic volumes;
- the resources he/she has available.

The local terrain and site conditions dictate the height, length and number of spans, and the design of the substructure foundations. The required design life and the resources available to construct the bridge

will influence the choice of materials and building methods. The traffic predictions enable the engineer to determine the necessary width of the bridge and the load-bearing capacity of the superstructure and substructures. Figure 2.1 shows how the information and surveys in this planning stage lead to the preparation of a general specification for the bridge, followed by further field investigations and the development of detailed designs and specifications.

The data required specifically for the geotechnical analyses, site investigations and hydraulic design are discussed in Chapters 3 and 4.

2.1 Site selection

There are three initial considerations to bear in mind:

- a bridge site must offer appropriate vertical and horizontal alignments;

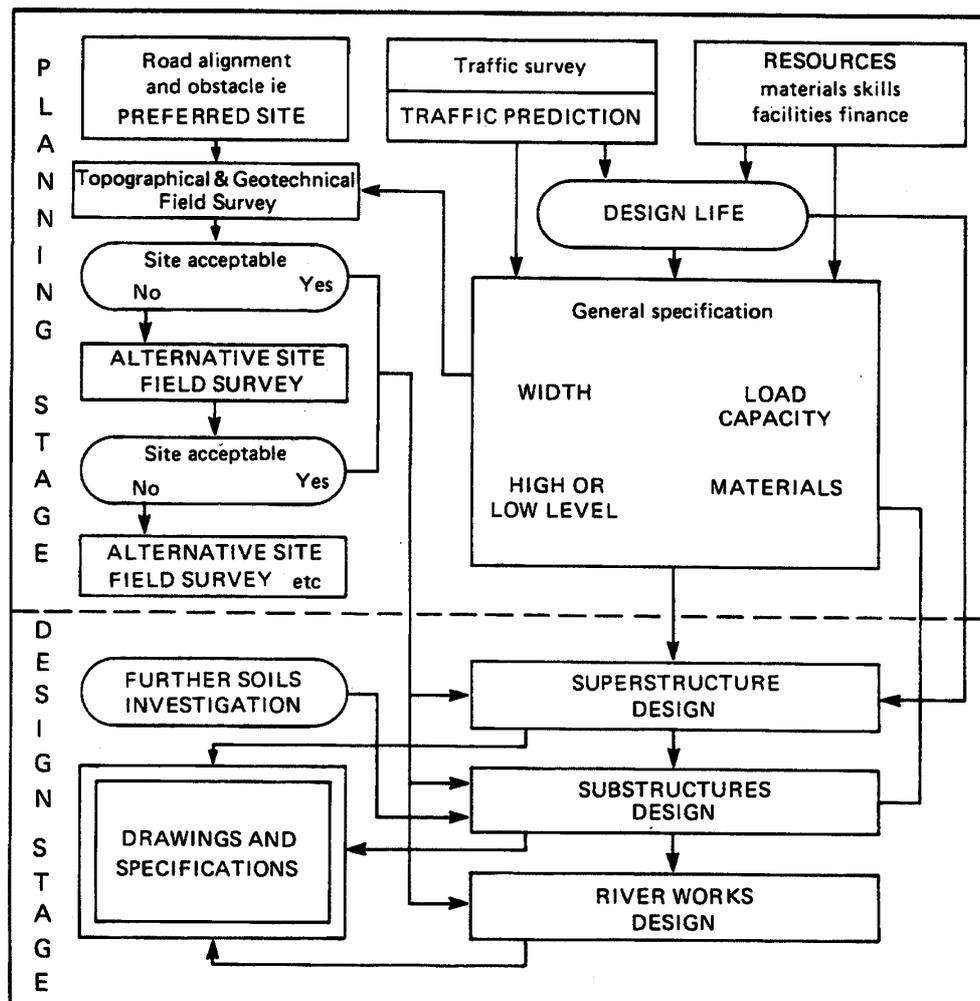


Figure 2.1 Flow diagram of the design process

- its soils must be strong enough to ensure the stability of the structure;
- the bridge and its associated works should not have an adverse impact on adjoining land or buildings, or themselves be susceptible to damage from the local environment.

For the highway engineer, rivers are the most common obstructions needing to be bridged. Occasionally he/she may be called upon to design a rail or road crossing but these are relatively simple compared to river crossings because they involve considerations only of height and span, whereas the design of a river crossing has also to take hydraulic requirements into account.

2.1.1 River morphology

Rivers are classed as either alluvial or incised.

Alluvial rivers erode their banks and scour their beds; they have flood plains on either side of the channel and the flow regularly overtops the channel banks to spread across the flood plain. They are continually active, scouring and depositing material on the banks and transporting quantities of sediment. Their channels are winding, and the meanders move downstream as scour and deposition of sediment takes place (Figure 2.2a). When an alluvial river is fed with larger quantities of sediment than it can transport, it deposits sediment as shoals over short lengths of the channel. These shoals deflect the flow, causing a number of minor channels to develop between islands. The river is then said to be braided

(Figure 2.2b). The main channels and any of the minor channels can change position and islands can disappear in the course of a single major flood.

Incised rivers have relatively stable banks and are generally narrower and deeper than alluvial rivers.

Some overtop their banks during flood, but the flow returns to the existing channel when the flood subsides.

Steeply graded tributary streams flowing into a major river commonly exhibit abrupt changes in channel width and bed gradient where they enter the main flood plain. These changes result in the deposition of large quantities of sediment in the form of alluvial fans (Figure 2.3). The fans consist typically of gravel to clay size debris, are usually conical in shape and have a maximum slope of about 10 per cent. The main channel across the fan can shift its position considerably in a single flood.

2.1.2 Bridge location

In selecting the location for a small bridge, the engineer often has to reach a compromise between the easiest river crossing and the shortest road alignment. The choice of location then becomes an economic decision. The cheapest bridge site and the one that has potentially the longest service life is a location that:

- is on a straight reach of the river;
- is beyond the disturbing influence of larger tributaries;

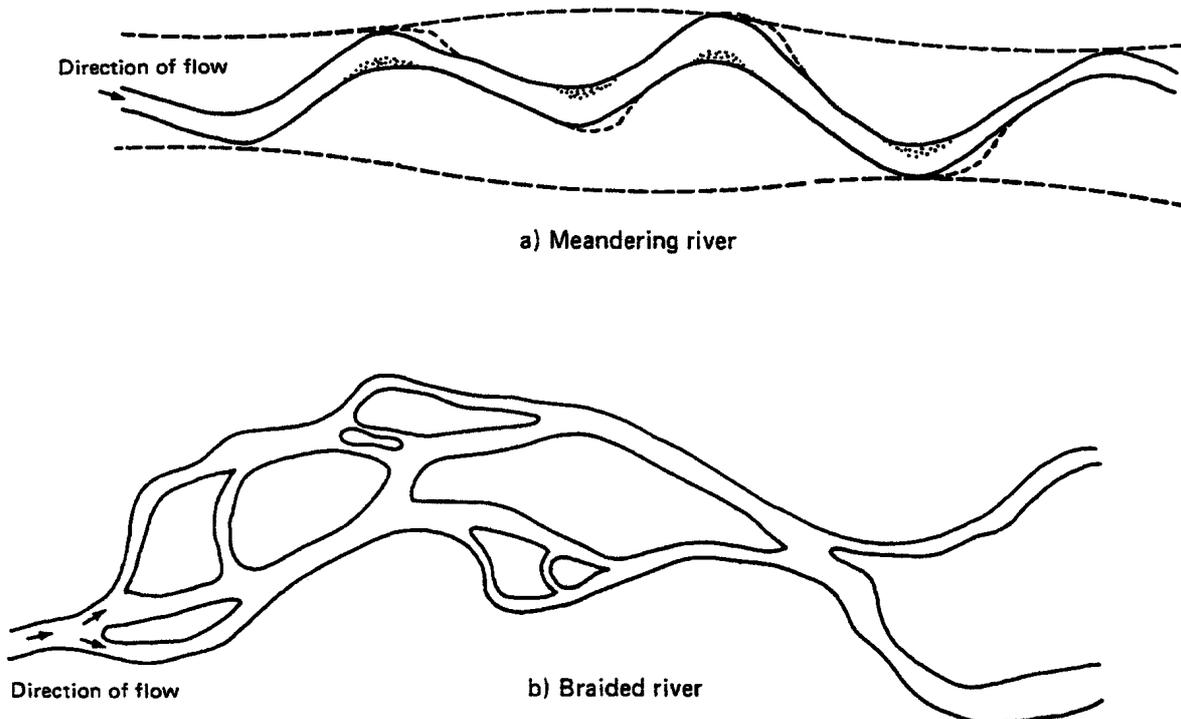


Figure 2.2 Alluvial rivers

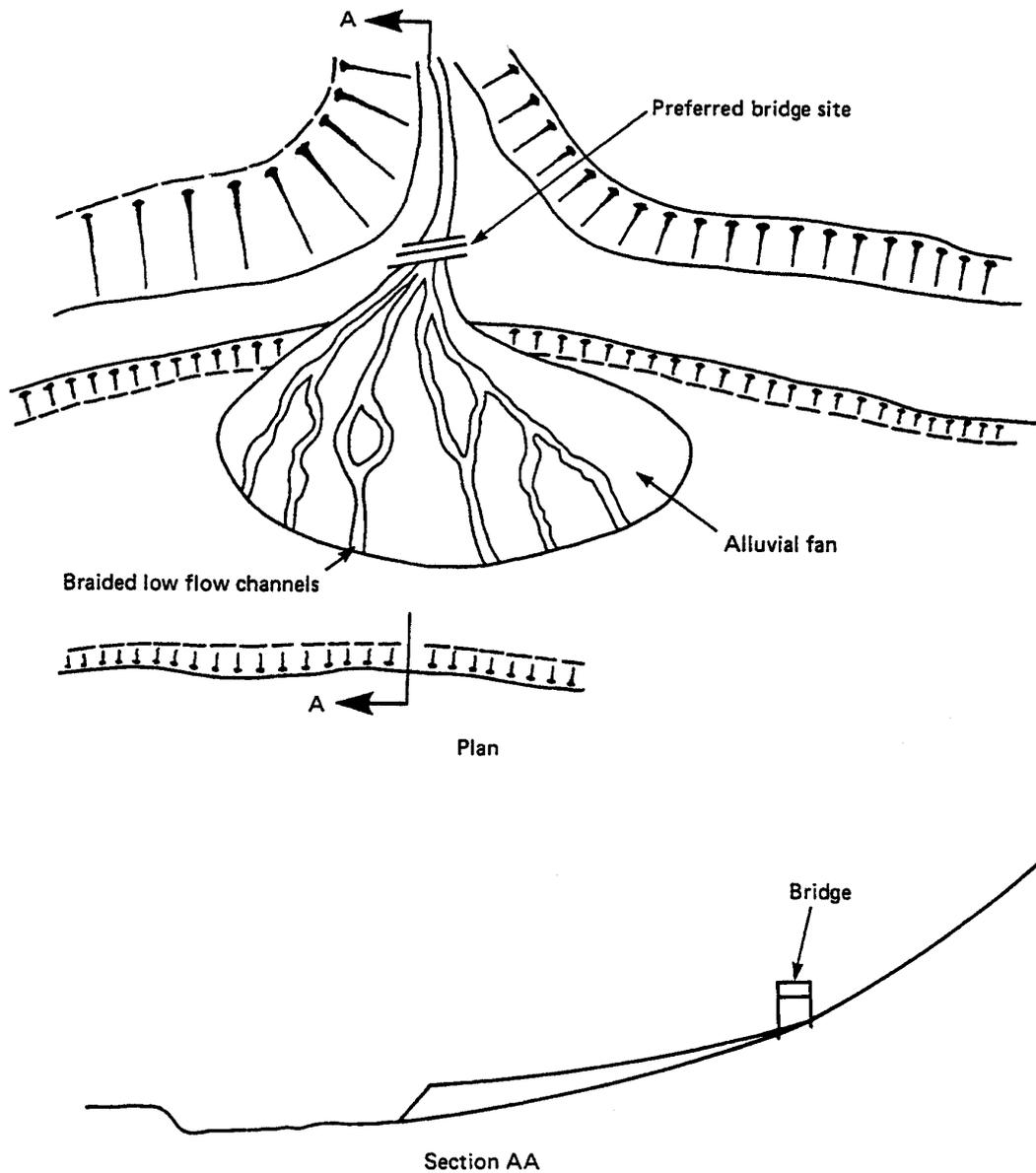


Figure 2.3 Crossing an alluvial fan

- has well defined banks;
- has reasonably straight approach roads;
- permits as square a crossing as possible;
- has good foundation conditions.

The site should allow the maximum gradient of the approach roads to be appropriate to the types of vehicle likely to travel on the road as well as offering vertical curves and sight distances suitable for the maximum speed of vehicles using the bridge.

A bridge aligned at right angles to the river results in the shortest superstructure. A skew bridge requires more material and is more complicated to design and construct. If a skew is unavoidable the angle should

not exceed 200 and the abutments and piers should be set parallel to the direction of flow during maximum flood (Section 2.2.2), which may not be the same as during normal flow.

When crossing an alluvial fan the bridge should be located as near to the throat of the fan as possible where the tributary stream has sufficient gradient to be self-scouring (Figure 2.3). If this is not practicable and the crossing has to be located across the body of the fan, there will be a requirement for extensive training works to confine the flood flow and for the regular removal of sediment from under the bridge. In this situation a causeway will often be a more practical solution. Bridge crossings over alluvial rivers nearly always require training works to stabilise the channel flow within the bridge waterway opening.

2.2 Site conditions

Once the engineer has identified a likely site for the bridge, he/she needs to obtain field information on the local terrain and river conditions in addition to the soil information and hydraulic data that are outlined in Chapters 3 and 4. The key points of field information relate to:

- the catchment area of the river;
- water levels;
- navigational and other clearance requirements.

2.2.1 Catchment area

The extent of the river catchment area determines the area to be included in plans and sections, and can be used to estimate flow volumes. If maps to an appropriate scale or aerial photographs are available, the limits of the catchment area can be marked on them and its total size calculated. Transparent squared graph paper is useful for this purpose. In the absence of suitable cartography, the size of the catchment area and its average gradient should be estimated by means of a traverse.

2.2.2 Water levels

Information is needed on the highest known flood level, the ordinary flood level and the low water level at the proposed site.

The **highest known flood level (HFL)** should be determined by local observation wherever possible, supplemented by inquiries in the locality. The silt marks that high floods generally leave on tree trunks and buildings remain visible for several years. If there are old trees in the vicinity of the site, they should be examined for the presence of small twigs left adhering to the bark at high water levels. It is usually helpful to ask people who have been living in the area for a long time about their recollections of particularly high floods, but this source of information is variable in its reliability. It is better to make such inquiries by talking to people individually rather than in groups.

The **ordinary flood level (OFL)** is the level to which the river normally rises during the wettest part of the year.

The **low water level (LWL)** is the level prevailing in the river during dry weather. If there is little or no flow in dry weather, the period during which the river bed remains dry should be noted.

2.2.3 Navigational and other clearance requirements

The height of the bridge superstructure has to allow for the passage of any regular or occasional river craft as well as the clearance of floating debris at times of

flood. Even where a river is not used by regular traffic, drainage channels and other alluvial waterways periodically require dredging and river-borne equipment may need to pass the bridge. Though it is unlikely that river maintenance equipment will travel when the river is in full spate, this is the time when trees and other floating debris may be carried by floodwater. Experience from other structures on the same river, together with inquiries locally, will help to determine the required clearance between the design flood level (Chapter 5, Introduction) and the underside of the superstructure (Section 5.2).

2.3 Plan and sections

The engineer should produce:

- a plan and longitudinal section of the river to scales of 1/1000 horizontal and 1/100 vertical;
- at least three cross-sections plotted to a natural scale of 1/100, one at the proposed site and one each at the upstream and downstream limits of the plan.

Table 2.1 indicates the distances that should be covered by the plan and longitudinal section in relation to the size of the river catchment area. These distances may be reduced if large-scale aerial photographs are available and show a simple river channel shape.

Table 2.1 Distances to be covered by site plans

Catchment area, km ²	Upstream	Downstream
up to 2.5	150m	150m
2.5 – 10	200–400m	150m
over 10	400–1500m	250m

Information on the terrain and other features in the vicinity of the proposed site should be marked on the plan, as well as the direction of river flow and the location of cross-sections. The longitudinal section should indicate the highest and ordinary flood levels and the low water level (Section 2.2.2). Figure 2.4 shows a typical plan and longitudinal section. Section 16.1 lists in detail the information to be included on the site plan and longitudinal section prepared as part of the final bridge drawings and specifications.

Figure 2.5 shows a simple cross-section, with the relevant water levels and the channel shape indicated. The cross-section at the bridge site should show contours at close intervals and indicate any rock outcrops, scour holes and other river bed features. If a road or track already crosses the river at the proposed bridge site, the cross-section should be taken a short distance upstream, in order to obtain an undisturbed section.

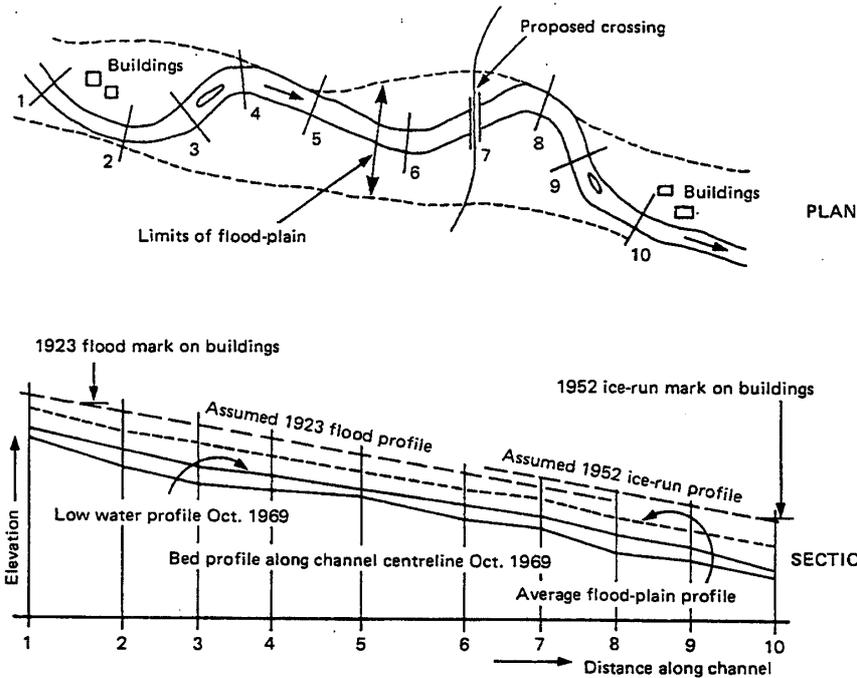


Figure 2.4 Plan and section of the river at a proposed crossing

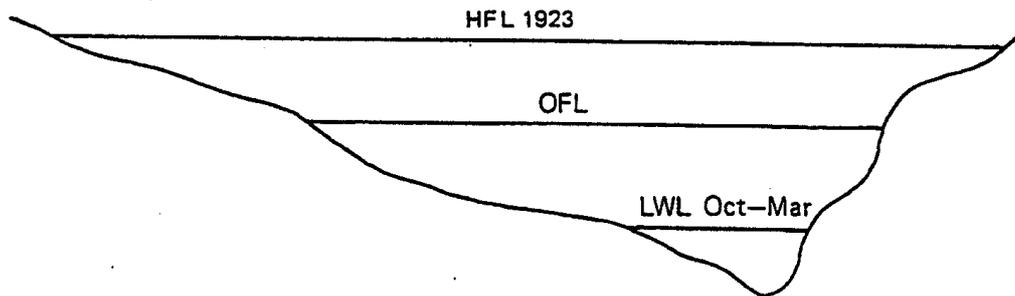


Figure 2.5 Cross section showing relevant water levels

2.4 Design life

Unlike roads, bridges are not designed to sustain a total number of load cycles. The choice to be made is usually between a 'permanent' structure to carry specified loads with a service life of more than 40 years, and a temporary structure. The engineer's decision will be influenced by traffic predictions and by the resources available at present and in the foreseeable future.

Where it is expected that future development will increase the desired capacity, the choice is between building a low-cost bridge to serve until the development occurs or building a structure that is wider, longer or stronger than initially required but which will cope with future needs. An alternative solution is to build permanent abutments and a light deck that can be upgraded or replaced when the development occurs.

Service requirements can seldom be predicted with any confidence beyond a ten-year horizon. Local experience

and available funds are likely to be the strongest factors influencing the design life of the bridge.

2.5 Traffic

The engineer should estimate the composition and volume of the vehicular traffic likely to use the road throughout the design life of the bridge. The volume of current traffic can be determined from a simple traffic count. The growth rate over the design life of the bridge is difficult to estimate, but the engineer should attempt to do so, taking into account the local factors which influence traffic growth, such as agricultural or industrial development, and national factors such as development planning and the general increase in gross domestic product. Traffic counting and estimating future flows are discussed in TRRL (1988).

Vehicle weights can vary according to the season. Unless good quality data on vehicle weights are

available it is advisable to carry out an axle weighing exercise at the time of year when the heaviest loads are transported, as described in TRRL (1978).

2.6 Bridge width

Apart from bridges for special applications, there are three alternative widths to be considered:

- single lane;
- one and a half lanes;
- two lanes.

2.6.1 Single lane bridges

Single lane bridges are suitable for predicted traffic flows lower than about 200 vehicles per day. They involve only minimal disturbance to traffic flow and there is normally no safety problem, given adequate sight distance and waiting areas on the bridge approaches and clear advance signing of the width restriction. The width clearance for vehicles is usually 3.65m. Additional provision can be made for pedestrians and two-wheeled vehicles on one side of the roadway, or on both sides when the bridge is located close to a village. Footways should be a minimum of 1.5m wide.

2.6.2 One and a half lane bridges

In some districts there may be a preponderance of light traffic, with only the occasional bus or heavy commercial vehicle. In this situation, the most cost effective design may be a bridge allowing two lanes of light traffic, but not wide enough for two large vehicles to pass. This solution offers economies over a full, two-lane bridge in terms of both width and load carrying capacity.

A carriageway width of 4.6m is sufficient for two lanes of light vehicles but restricts the loading to one lane of heavy vehicles, which are normally 2.5m wide. Adequate sight distances, waiting areas and warning signs are required at both ends of the bridge, and there is likely to be a need to make additional provision for pedestrians.

Some authorities consider this width of bridge dangerous and may give preference to a wider two lane bridge.

2.6.3 Two lane bridges

These should be designed to conform to the appropriate national standards in terms of load capacity, width and safety provisions.

2.6.4 Culverts

Culverts occur more frequently than bridges and are not so noticeable to drivers on fast stretches of road. It is recommended that carriageway width remains constant over culverts.

2.6.5 Low water crossings

Low water crossings are considered separately in Chapter 7.

2.7 Paths for pedestrians and cyclists

Safety authorities recommend that segregated footways are provided for pedestrians to cross bridges, TRRL (1991). They are particularly necessary on long bridges built to minimum widths where the traffic is fast.

It is possible to add a pathway for pedestrians and cyclists by means of supports cantilevered from the main deck, but the engineer has to bear in mind the effect of asymmetric loading should a large number of people congregate on the pathway. An alternative and generally more satisfactory approach is to widen the main deck by about 2m and provide a suitable barrier and parapet, as discussed in Sections 9.2.10 and 11.4.

2.8 Design loading

Most countries have some form of design loading standards for bridges, but some have not yet determined an appropriate standard for short span rural bridges which carry low traffic volumes and weights. These short span rural bridges often do not need to be designed for the heavy goods vehicles that are common in industrial areas as an appropriate loading specification for a bridge is one which caters for the heaviest predicted loads expected during the life of the structure.

This manual offers standard designs that conform with two of the most commonly adopted loading standards. These are the British Standard loading for 40 tonne gross weight vehicles (BS.HA.LOAD) and the American AASHTO loading for 20 tonne gross weight vehicles (HS 20-44). These loading levels have been used in the standard designs presented in the following chapters and should be sufficient to cover the loading requirements of the majority of rural bridges.

2.8.1 BS.HA.LOAD (40 tonne maximum gross vehicle weight)

The BS.HA.LOAD loading was adopted from British Standard BS 5400 (BSI 1978). The loading includes 38 tonne heavy goods vehicles as well as the new European 40 tonne, five- and six-axle trailer combinations. The revised loading specifications, now a requirement for bridges in Britain, are presented in BD 37 (Highways Agency 1988). The loading is presented in the form of a uniformly distributed load imposed on the full lane and a knife edge load placed across the lane in the most critical location and represents a fully loaded lane with the worst combination of vehicles expected in the life of the bridge. The loading includes the effect of impact

(80% on the heaviest axle), overloading (up to 40%) and a 10% contingency for future increases in traffic load and is therefore a conservative estimate of the worst expected loading. The range of vehicle configurations covered by this loading, as well as an indication of which vehicles are critical for each span, are given in BD 21 (Highways Agency 1997).

2.8.2 HS 20-44(20 tonne maximum gross vehicle weight)

Loading HS 20-44 has been taken from the AASHTO standard (AASHTO 1983) to cover vehicles with a maximum gross vehicle weight of 20 tonnes. Most two-axle medium weight commercial vehicles are loaded within the 15 tonne AASHTO limits, but when overloaded these limits may be exceeded. HS 20-44 has therefore been adopted here as a conservative standard. The vehicle configurations (axle weights, spacing, etc) covered by this loading are described in the AASHTO standard.

Bridge loading specifications can vary considerably between countries. One reason for this is that standards are frequently derived from the range of legal vehicle loads in that country. When choosing a standard to use, it is important to take into account the actual vehicle loads that may use the bridge. In some cases, these can be higher than the maximum legal vehicle loads. The 40 tonne loading (BS.HA.LOAD) should be used where traffic cannot be restricted to 20 tonne vehicles. In addition, it should be used where overloaded three axle lorries, forestry or quarry vehicles and construction plant are likely to be in use.

In both of the above design standards, an allowance for even heavier vehicles can be made by additional loading requirements. These are not included in the standard designs given in this manual. Where these heavier vehicles are expected, the standard designs can be checked, and modified appropriately, by a suitably qualified engineer.

2.9 Resources

There are three categories of resources to be considered:

- design ability;
- skills for the various bridge construction processes;
- available materials.

2.9.1 Design

It is the purpose of this manual to provide all the necessary procedural guidance, tables, dimensions and material specifications to enable a civil or mechanical engineer with some field experience to prepare appropriate designs for small bridges. The

use of these procedures and tables will lead to serviceable, conservatively designed structures. Engineers with experience of bridge design may wish to carry out more of their own calculations or to interpolate between recommended dimensions, and to produce designs tailored more specifically to individual requirements.

2.9.2 Trade skills

The engineer should be conversant with the standards of workmanship and supervision required for structures using plain concrete, reinforced concrete, steel and timber. He/she should also be aware of the facilities he/she has available for other functions such as site investigations, and so will be the best person to judge when external skills should be engaged for processes outside the experience of his own workforce. Working alongside crews specialising in fields such as Site investigation, boring, piling or scaffolding can provide a useful opportunity to broaden the skills of technicians and labourers in the engineer's own organisation.

2.9.3 Materials

All construction materials must either be found at or close to the bridge site, or must be transported there. Bringing in heavy materials in long sections such as steel beams can be difficult, especially when the bridge is being constructed in advance of the road.

Steel panel bridges and steel beam bridges are often made from imported parts and may be subject to delivery delays affecting key items. They may also be the most expensive option, unless a substantial amount of the required materials can be found in the district in good second-hand condition. Although panel bridges are designed to be dismantled and used again, they are rarely used more than once.

The site investigations can be extended to include a search for suitable aggregates for concrete and timber for use in the deck or as temporary support during construction. The planning stage includes making provision for processing the resources to be developed locally and storing all the materials in a form that will prevent their deterioration.

2.10 References

AASHTO (1983). *Standard specification for highway bridges*. Washington: American Association of State Highway and Transportation Officials.

Highways Agency (1997). *BD 21/97: The assessment of highway bridges and structures*. Design Manual for Roads and Bridges, Volume 3, Section 4, Part 3, Highways Agency, London.

Highways Agency (1988). *BD 37/88: Loads for highway bridges.* Design Manual for Roads and Bridges, Volume 1, Section 3, Highways Agency, London.

TRRL (1978). *A guide to the measurement of axle loads in developing countries using a portable weighbridge.* Road Note 40. Stationery Office, London.

TRL (1988). *A guide to road project appraisal.* Overseas Road Note ORN5. Transport Research Laboratory, Crowthorne.

TRL (1991). *Towards safer roads in developing countries - a guide for planners and engineers.* Transport Research Laboratory, Crowthorne.

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3 Site investigations

The weight of the traffic, superstructure, abutments and piers will all be carried by the soils supporting the abutment and pier foundations. In order to design appropriate foundations (Chapter 8) the engineer has to determine the nature and location of the different soil types occurring at the site of the bridge and its approaches, to depths containing strata sufficiently strong to support the bridge and embankments without significant deformation.

This information is obtained by analysing samples taken from a grid of bore-holes or test pits covering the whole of the proposed site, and by testing the samples for density, shear strength, plasticity and penetration, in order to provide quantitative data for foundation design.

The level and nature of the ground water also affect the foundation design and the engineer must take into account the impact of bridge construction on the ground water and hence the stability of nearby works and slopes.

Since the overall behaviour of the ground is often dictated by planes or zones of weakness, it is possible to obtain a large sample of material that may not be indicative of the behaviour of the mass. For this reason, and because of the frequent need to modify the sampling technique to suit the ground conditions, strict supervision of sampling is essential.

Nearby cut slopes can reveal soil and rock types and their stability characteristics, as can old excavations and quarries. There may be embankments or buildings and other structures in the vicinity of the bridge site that have a settlement history due to the presence of compressible or unstable soils.

This chapter contains several extracts from Tomlinson (1986), who states, 'An engineer undertaking a site investigation may engage local labour for trial pit excavation or hand auger boring, or he/she may employ a contractor for boring and soil sampling. If laboratory testing is required, the boring contractor can send the samples to an independent testing laboratory. The engineer then undertakes the soil mechanics analysis for foundation design or he/she may ask the testing laboratory to do this analysis. Alternatively, a specialist organization offering comprehensive facilities for boring, sampling, field and laboratory testing, and soil mechanics analysis may undertake the whole investigation. This is much to be preferred to the system whereby one organization does the borings, another the testing, and yet another the analysis. A single organization has an advantage of providing the essential continuity and close relationship between field, laboratory and office work. It also permits the boring and testing programme to be readily modified in the light of information made available as the work proceeds.

Additional samples can be obtained as necessary from soil layers shown by laboratory testing to be particularly significant. In situ testing can be substituted for laboratory testing if desired. In any case, the engineer responsible for the day-to-day direction of the field and laboratory work should keep the objective of the investigation closely in mind and should make a continuous appraisal of the data in the same way as is done at the stage of preparing the report. In this way vital information is not overlooked and the significance of such features as weak soil layers, deep weathering of rock formations and sub-artesian water pressure can be studied in such greater detail as may be required, while the field work is still in progress.'

3.1 Methods of site investigation

This section outlines the following methods:

- test pits;
- hand auger boring;
- cable percussion boring;
- rotary drilling;
- geophysical surveying.

The descriptions are brief and intended only to remind the engineer about the uses and limitations of the tests, which should be carried out under the supervision of an experienced technician according to BSI (1981) or other accepted standard. Tomlinson (1986) and Geotechnical Control Office (1987) contain more detailed descriptions.

3.1.1 Test pits

A test pit, which should be at least 1m square at the bottom, is a cheap and simple method of subsurface exploration. The pit is normally dug by hand, but a mechanical excavator may be used to remove the bulk of the material before the sides and bottom are squared and cleaned for examination. This method of investigation supplies excellent data on subsurface conditions within the depth to which the pit is excavated and enables a clear picture to be obtained of the stratification of the soils, the presence of any lenses or pockets of weaker material and the level of the water table. The maximum practical depth to which a pit can be excavated is about 3m; below a depth of about 1.5m the sides of the pit will require support or will need to be excavated at a safe angle.

Pits excavated through cohesive soils below ground water level are unlikely to need dewatering by pumps. They should be left open for some time so that seepage lines on the pit sides can indicate the ground water level.

In medium and fine sands it may be possible to dewater the pit by pumping. This will rarely be possible in coarse sands and gravel, where boring may therefore be necessary.

Figure 3.1 reproduced from Geotechnical Control Office (1987) gives an example of how to record the data obtained from a test pit.

3.1.2 Hand auger boring

The hand auger boring method uses light hand-operated equipment. The auger and drill rods are normally lifted out of the borehole without the aid of a tripod, and no borehole casing is used. Boreholes up to 200mm diameter may be made in suitable ground conditions to a depth of about 5m. The method can be used in self-supporting ground without hard obstructions or gravel-sized to boulder-sized particles. Hand auger boreholes can be used for

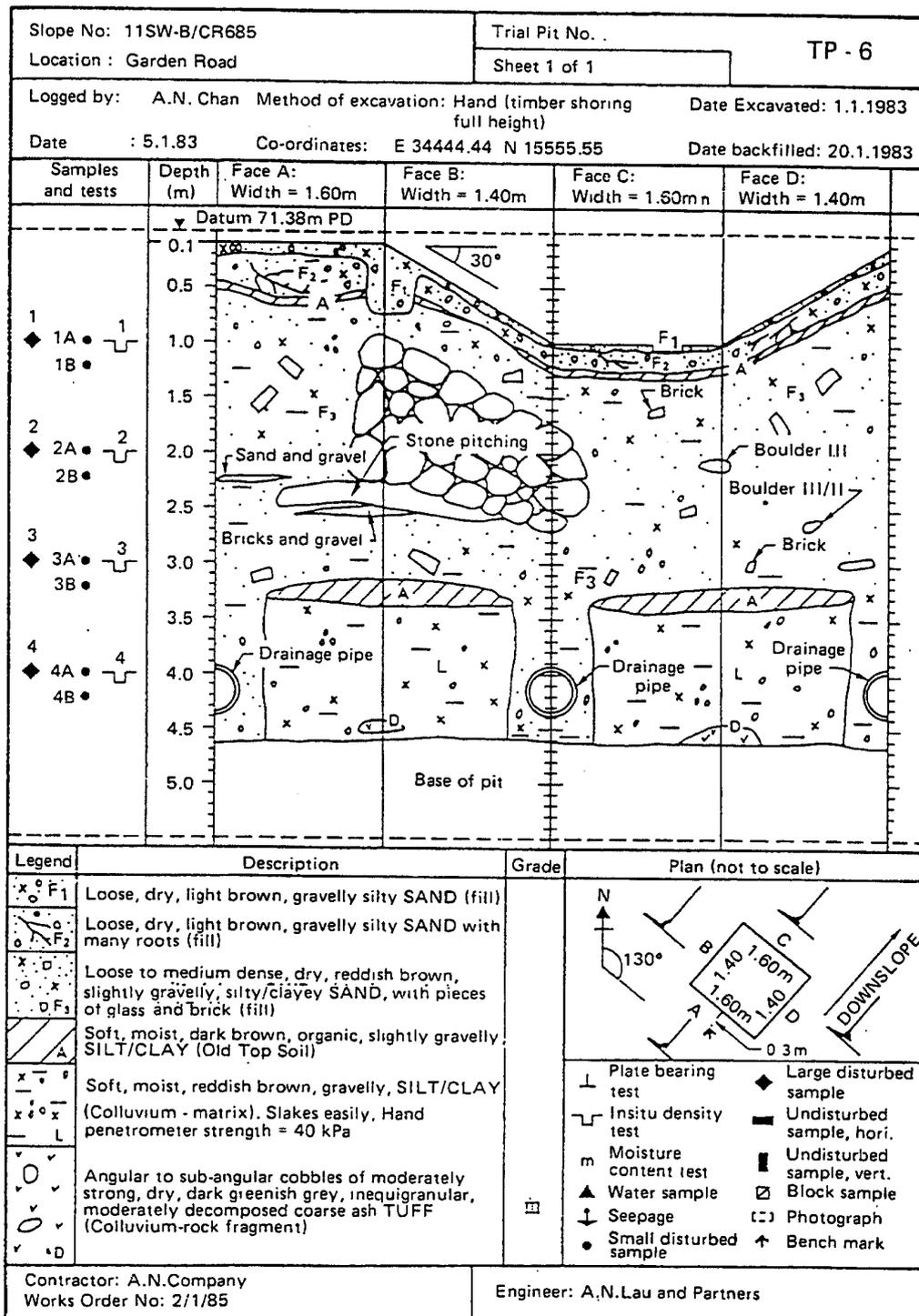


Figure 3.1 Trial pit log

ground water observations and to obtain disturbed samples and small open-tube samples.

3.1.3 Cable percussion boring

This is an adaptation of standard well-boring methods, suitable for soil and weak rock. The sizes of borehole casings and tools are generally 150mm, 200mm, 250mm, and 300mm, giving a maximum borehole depth of about 60m in suitable strata. The drill tools, worked on a wire rope using the clutch of the winch for the percussive action, are a clay cutter for dry cohesive soils, a shell or baler for cohesionless soils and a chisel for breaking up rock and other hard layers. The clay cutter and shell bring up disturbed material for laboratory testing and identification of the strata.

3.1.4 Rotary drilling

Rotary drilling rigs are available in a wide range of weights and power ratings. They require a certain expertise in operation, not least because water supplied to lubricate the drilling head can adversely affect the stability of the surrounding ground and the samples obtained from the bore. **Open hole drilling**, in which the drill bit cuts all the material within the diameter of the borehole, is used for more rapid progress in hard material. Better quality samples of soil and rock are obtained using **core drilling**, in which an annular bit fixed to the outer rotating tube of a core barrel cuts a core that is returned within the inner stationary tube of the barrel.

3.1.5 Geophysical surveying

Geophysical tests may be helpful in supplementing the data obtained from test pits and bores, eg by tracing the boundary between two soil types, but they are rarely necessary for the planning and design of small bridges. The tests, whether sonic, magnetic or seismic, require expert handling and interpretation, and should therefore be entrusted to an organisation specialising in this work.

3.1.6 Backfilling

Poorly compacted backfill will cause settlement at the ground surface and can act as a path for ground water. For boreholes in dry ground it is possible to use compacted soil as a fill material, though cement based grout is usually more successful. The back-fill of excavations can be compacted by using an excavator bucket, but hand tamping will be required at corners.

Only temporary backfilling will be required where abutment or pier foundations are to be constructed. All other pits and boreholes should be properly reinstated.

3.2 Extent of investigations

The site investigations should reveal a clear pattern of soils, rock strata and ground water over the whole site. Strength tests are required in the soils that are to be loaded by the structure (Figure 3.2 and Section 8.3.3): these tests should be taken down below foundation level, or below all deposits that are unsuitable for foundations, to a depth at least 1.5 times the expected width of the foundation, unless bed-rock is reached and confirmed by drilling with a rotary coring rig. Usually a penetration of 3m into the rock will be sufficient; in residual profiles it may be necessary to drill farther to ensure that the rock is not just a boulder or core stone.

At least three boreholes should be drilled for each bridge abutment. Fewer bores may be permitted for piers if a clear picture emerges of the strata and soil properties. Each borehole and pit should be numbered and the numbers entered on a plan of the site.

3.3 Sampling

The choice of sampling technique depends on the purpose for which the sample is required and the character of the ground.

There are four main techniques for obtaining samples:

- taking disturbed samples from drill tools or from excavating equipment in the course of boring or excavation
- drive sampling in which a tube or split tube sampler with a sharp cutting edge at its lower end is forced into the ground, either by static thrust or by dynamic impact
- rotary sampling, in which a tube with a cutter at its lower end is rotated into the ground, so producing a core sample
- taking block samples cut by hand from a trial pit, shaft or heading.

Samples obtained by the last three techniques will be sufficiently intact to enable the ground structure within the sample to be examined. However, the quality of these samples can vary considerably, depending on the sampling technique and ground conditions, and most samples will exhibit some degree of disturbance.

Table 3.1 indicates the mass of sample required for identification purposes, Atterburg tests, moisture content, sieve analysis and sulphate tests.

Care should be taken to ensure that samples are as pure and undisturbed as possible. Before a sample is taken, the bottom of the borehole or surface of the pit must be cleared of loose or disturbed material. When

Table 3.1 Soil sample mass required for identification

Soil type	Mass required kg
Clay, silt, sand	2
Fine & medium gravel	5
Coarse gravel	30

taking undisturbed samples by drive sampling, it is necessary to maintain the water level in the borehole above the surrounding ground water level, so as to prevent the sample being disturbed by a flow of water into the borehole due to the differential head.

Samples should be packed and labelled in appropriate containers, according to the laboratory testing programme. Block samples should be marked for orientation and protected from evaporation so far as possible until they are properly wrapped and boxed.

Each sample should be labelled with a reference number for location, date, brief description and depth below ground level of the top and bottom of the sample. The sample reference numbers should be related to the borehole and pit numbers (Section 3.2)

3.4 Soil Testing

Table 3.2 sets out the basis on which soils are classified for engineering purposes, and outlines simple field tests that help identification. Laboratory tests of soil samples and in situ field tests should be carried out according to recognised standards such as BSI (1981 and 1990). These tests include sieve analysis of non-cohesive soils, liquid limit and plastic limit tests of cohesive soils, density tests, strength tests and acidity tests.

Some presumed bearing values are listed in Chapter 8, but it is recommended that wherever laboratory facilities and field equipment are available, the most appropriate of the tests outlined in Section 3.4.1 are carried out for each site.

3.4.1 Field tests

These may include:

- density measurements;
- shear vane tests;
- penetration strength tests;
- dynamic cone penetrometer soundings;
- plate bearing tests.

It is assumed that if the facilities for these tests are available, the procedures are known and need not be

described in detail here. Relevant standards are BSI (1981) and ASTM(1985).

3.4.1.1 Density measurements

Bulk density of soils and rock is measured by sand or water replacement methods or by nuclear methods.

3.4.1.2 Shear vane tests

Shear vane tests are usually confined to uniform, cohesive, fully saturated soils. The presence of even small amounts of coarse particles, rootlets or thin laminations of sand may lead to unreliable results.

3.4.1.3 Penetration strength tests

The strength of coarse-grained, non-cohesive soils can be assessed by the Standard Penetration test using a percussion boring rig and a split-barrel sampler. The N value obtained is used directly in the design calculations outlined in Chapter 8. The value is the number of blows required to drive the sampler 300mm into the layer under study, and it may be affected by large stones and rock. For this reason, any very high values obtained by this method should be treated with suspicion. Table 3.3 indicates an approximate correlation between N values and the relative density of granular materials.

3.4.1.4 Dynamic cone penetrometer soundings

The Dynamic Cone Penetrometer (DCP) is cheap and quick to use, and it causes minimal disturbance to the ground. It can be applied between boreholes or test pits to obtain a continuous profile of soil layers, or to find the boundaries of boulders. DCP tests should be made close to each borehole or test pit, to provide a correlation between soil types and penetration specific to the locality of the site, and then at small intervals between boreholes and test pits. Table 3.4 shows a typical correlation between DCP and SPT values.

3.4.1.5 Plate bearing tests

There are a number of procedures for measuring the bearing capacity of soils and weak rocks by the use of a steel plate to which either a continuous load or a constant rate of penetration is applied, BSI (1990). If possible the plate should fill the borehole and be bedded on undisturbed material: where the diameter of the plate is significantly less than the diameter of the borehole, the results of the test are hard to interpret. Ground water should be at its natural level during the test, which may make seating of the plate in the bottom of the borehole difficult. Since the resulting bearing capacity applies only to the soil or rock immediately below the plate, a number of tests will be required to cover the surface area and depth of material to be stressed by an abutment or pier (Figure 3.2)

Table 3.2 General basis for field identification and classification of soils

	1. Principal soil types		2. Size & nature of particles		3. Composite Types		4. Strength		5. Structure	
							Term	Field Test	Term	Field Identification
Coarse grained non-cohesive	Boulders Cobbles		Larger than 200mm in diameter. Mostly between 200mm and 60mm.	Boulder gravels	Loose	Can be excavated with spade. 50mm wooden peg can be easily driven.	Homogenous	Deposit consisting essentially of one type.		
		Gravels	Mostly between 60mm and 2mm.	Hoggin Sandy gravels	Dense	Requires pick for excavation, 50mm wooden peg hard to drive more than a few inches.	Stratified	Alternating layers of varying types.		
	Uniform		Composed of particles mostly 2.0 – 0.06mm and visible to the naked eye. Very little or no cohesion when dry.	Silty sands Micaceous sands	Slightly cemented	Visual examination. Pick removes soil in lumps which can be abraded with thumb.				
		Graded	Sands Sands may be classified as uniform or well graded according to the distribution of particle size. Uniform sands may be divided into: coarse sands: 2.0 – 0.6mm, medium sands: 0.6 – 0.2mm, fine sands: 0.2 – 0.06mm.	Lateritic sands Clayey sands						
Fine grained cohesive	Low Plasticity		Particles mostly invisible or barely visible to the naked eye. Some plasticity and exhibits marked dilatancy. Dries moderately quickly and can be dusted off the fingers. Dry lumps possess cohesion but can be powdered easily in the fingers.	Loams Clayey silts Organic silts Micaceous silts	Soft Firm	Easily moulded in the fingers. Can be moulded by strong pressure in the fingers.	Homogenous Stratified	Deposit consisting essentially of one type Alternating layers of varying types.		
		Medium Plasticity								
	High Plasticity	Clays	Dry lumps can be broken but not powdered. They also disintegrate under water. Smooth touch and plastic, no dilatancy. Sticks to the fingers and dries slowly. Shrinks appreciably on drying, usually showing cracks. Lean and fat clays show those properties to a moderate and high degree respectively.	Boulder clays Sandy Clays Silty clays Marls Organic clays Lateritic clays	Very soft Soft Firm Stiff Hard	Exudes between fingers when squeezed in fist. Easily moulded in fingers. Can be moulded by strong pressure in the fingers. Cannot be moulded in the fingers. Brittle or very tough.	Fissured Intact Homogenous Stratified Weathered	Breaks into polyhedral fragments along fissure planes. No fissures. Deposits consisting essentially of one type. Alternating layers of varying types. If layers are thin, the soil may be described as laminated. Usually exhibits crumb or columnar structure.		
Organic	Peats	Fibrous organic material, usually brown or black in colour	Sandy, silty or clayey peats	Firm Spongy	Fibres compressed together Very compressible and open structure					

Table 3.3 Correlation between SPT value, N, and density of granular soils

N value	Relative density
Less than 10	Loose
1-30	Medium dense
3-50	Dense
More than 50	Very dense

Table 3.4 Typical correlation between DCP and SPT values

DCP value mm/blow	SPT value blows/300mm
5	50
6	44
7	38
8	33
9	28
10	24
12	22
14	18
16	16
18	15
20	14

3.4.2 Laboratory tests

The following laboratory tests are specified in BSI (1990):

- liquid limit;
- plastic limit;
- density.

The liquid and plastic limits of cohesive soils may be determined using disturbed or undisturbed samples taken from bores or test pits, and tested in their natural state.

If quantities of material of suitable size can be obtained, the bulk density of soil samples of natural material may be determined by the immersion in water or water displacement methods.

Sieve analyses carried out on coarse-grained, non-cohesive soils also assist in their identification and classification.

3.5 Aggressive chemicals

The ground or ground water may contain chemicals capable of causing damage to concrete or steel. These chemicals may emanate from nearby industrial processing or may occur naturally. Measures to protect concrete or reinforcement against chemical attack are described in Section 14.1 of the manual.

The principal constituents that cause concrete to deteriorate are sulphates, which are most common in clay soils and acidic waters. Total sulphate contents of more than 0.2 per cent by weight in soil and 300 parts per million in ground water are potentially aggressive (BRE, 1981).

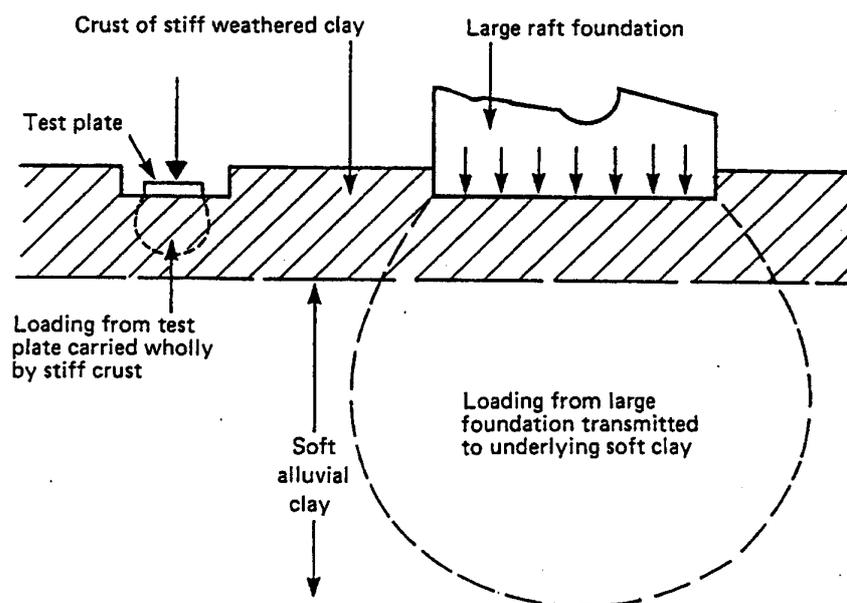


Figure 3.2 Loading effects of small and large plates

Corrosion of metal is caused by electrolytic or other chemical or biological action. In industrial areas, corrosive action may result from individual waste products dumped on the site. In river and marine works, the possible corrosive action of water, sea water and other saline waters, and industrial effluents may also require investigation. In a marine environment, the most severe corrosion is found in the 'splash zone' that is only wetted occasionally. The saline concentration in ground water near the sea may approach that of sea water, particularly where land has been reclaimed. Near estuaries, there may be an adverse condition caused by alternation of water of different salinities.

Laboratory tests to assess the aggressiveness of the ground and ground water against Portland cement concrete include determination of pH value and sulphate content (B SI, 1990). Since the pH value may be altered if there is a delay between sampling and testing, determinations should be made in the field whenever possible.

Water sampled from boreholes may be altered by the flushing water used in drilling, or by other flushing media employed: this means that sulphate and acidity tests carried out on samples from boreholes may not be representative unless special precautions are taken.

3.6 Design review

There is often difficulty in specifying ground conditions before the excavations for construction are complete. For this reason the engineer should be prepared to review his plans, both during the site investigation and during construction, if evidence is found of unexpected soil conditions.

3.7 References

ASTM (1985). *Annual book of ASTM standards, vol 0408*. Philadelphia: American Society for Testing and Materials.

BRE (1981). *Concrete in sulphate bearing soils and ground waters*. Digest No 250. Watford: Building Research Establishment.

BSI (1981). *Code of practice for site investigations*. BS 5930. London: British Standards Institution.

BSI (1990). *Methods of test for soils for civil engineering purposes*. BS 1377. London: British Standards Institution.

Geotechnical Control Office (1987). Guide to site investigation, Geoguide 2. Hong Kong: Civil Engineering Services Department.

Tomlinson M J (1986). *Foundation design and construction, fifth edition*. Singapore: Longman Singapore Publishers Pte Ltd.

4. RIVER HYDRAULICS

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4 River hydraulics

This chapter deals with the acquisition of the hydraulic data necessary for the efficient design of a river crossing in relation to the flow characteristics of the river (Chapter 5). The engineer has to ensure that the flow of water can pass the structure without causing damage to the bridge, the road embankment or the surrounding land. Damage can occur in a number of ways:

- the river may react against obstructions such as piers and abutments, and scour beneath them causing failure;
- the approach embankments may act as a dam during high floods, sustaining damage or causing more extensive flooding upstream;
- a river flowing on a shifting path may bypass a bridge and cut a new channel across the highway;
- a river may over-top a bridge if sufficient clearance is not provided.

In order to design a structure that avoids these problems and costs no more than is necessary, the hydraulic characteristics of the river must be understood and quantified. The most economical structure is usually one which is just wide and high enough to accommodate the design flood, minimising the total cost of abutments, piers, superstructure, approach embankments, relief culverts and river training works.

The hydraulic data required for the design process detailed in Chapters 5 and 6 relate to:

- design flood level (defined in Chapter 5), flow volume and velocity;
- maximum flood level, flow volume and velocity;
- bed characteristics - particle size, vegetation;
- channel shape and flood plain width;
- sedimentation and meander characteristics;
- navigational requirements and clearance of floating debris.

Flow velocity measurement and estimation are treated in Section 4.1, flow volume calculation in Section 4.2. Characteristics of the river bed and navigational requirements were discussed in Chapter 2. Flood levels and channel shape are drawn on the longitudinal and cross sections described in Section 2.3.

Using the hydraulic data, calculations may be made to determine:

- the geometry of waterway required at the bridge site;
- the backwater caused by the restriction of flow due to piers and abutments;
- the scour caused by the restriction;
- the river training works required.

4.1 Flow velocity

4.1.1 Direct measurement

Though it may be difficult to measure flow velocities directly during a flood, the engineer should attempt to do so wherever possible, because this is the critical value and alternative methods of estimating a maximum value are less accurate.

After a suitable method of depositing and retrieving a float on the river has been contrived, its travel should be timed over a distance of at least four times the channel width on a straight reach of preferably uniform section. If the shape of the channel is complex, velocities should be measured at several stations across the width.

Where the channel is deep, a double float may be used to measure velocities at several depths. This device consists of a small, buoyant float attached to a large weighted object by a cord (Figure 4.1). The velocity of the small float is assumed to be that of the water at depth 'd', the length of the cord, which is varied to measure the velocity at different depths. On simple sections the mean velocity is approximately equal to the surface velocity at the centre multiplied by 0.85. Figure 4.2 illustrates typical flood velocities at different stations and depths.

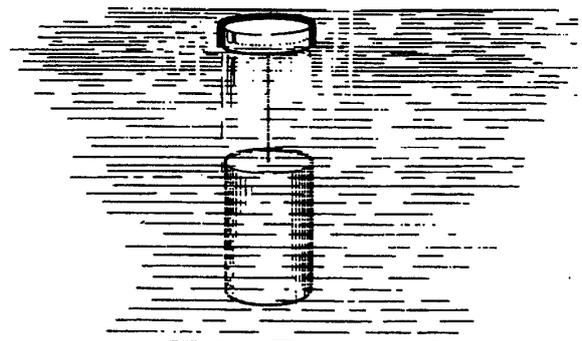


Figure 4.1 Double float for measuring flow velocity

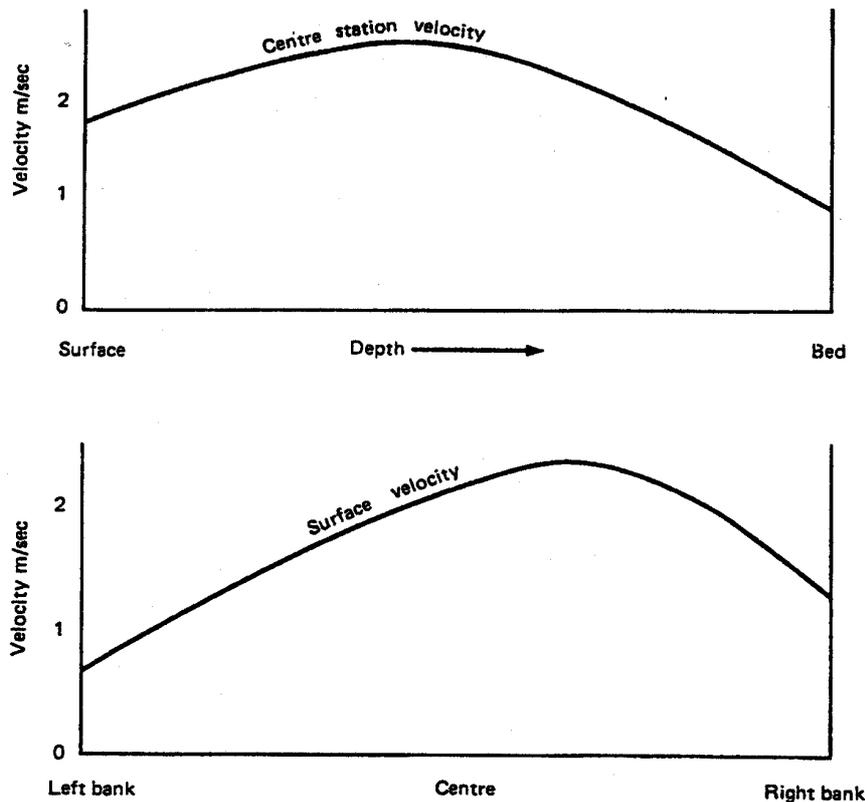


Figure 4.2 Typical flow velocity patterns

4.1.2 Calculation using bed characteristics.

The alternative to direct measurement is to use Manning's formula to estimate the mean velocity:

$$V = \frac{1}{n} \left(\frac{A}{P} \right)^{2/3} S^{1/2} \quad (4.1)$$

Where $V =$ velocity (m/sec)

$A =$ area of cross-section of the flooded channel (m^2)

$P =$ length of the wetted bed across the channel (m)

$s =$ gradient of the surface or bed slope

and $n =$ value of rugosity coefficient taken from Table 4.1.

Note: When using Table 4.1 to find the rugosity coefficient (n), choose Minimum if the sides are relatively smooth and Maximum if relatively rough. For example, tree stumps on cleared land, (C)3 would have $n = 0.050$, while a dense mass of trees would have $n = 0.200$.

4.2 Flow volume (discharge)

When a value for mean velocity of flow (V) has been obtained, the flow volume (Q) at the design flood

level may be calculated from the cross-sectional area and velocity.

4.2.1 Area-velocity method

Where there is free flow, the volume of flow may be calculated using the formula:

$$Q = A.V \quad (4.2)$$

Where $Q =$ volume of flow (m^3/sec)

$A =$ cross sectional area (m^2)

$V =$ mean velocity of the water (m/sec)

If the cross-section is not a simple shape, it may be divided into several parts as shown in Figures 4.3 and 4.4, and the total volume flow may be obtained by adding the flows from each calculation. This method is necessary when a river tops its banks during flood. Each part cross-section is chosen to be a simple shape and the value of V is measured for that part section, or calculated for it using Manning's formula. Appendix 4.1 sets out an example of flood discharge calculation by this method.

4.2.2 Orifice formula

The volume of flow may also be calculated from measurements taken on an existing bridge over the

Table 4.1 Value of rugosity co-efficient, 'n' in Manning's formula for streams and rivers up to 30m

Surface	Minimum	Normal	Maximum
(A) Natural streams on PLAINS			
1. Clean, straight bank, full stage, no rifts or deep pools	0.0275	0.030	0.033
2. Same as 1, but some weeds and stones	0.033	0.035	0.040
3. Winding, some pools and shoals, clean	0.035	0.040	0.045
4. Same as 3, some weeds and stones	0.035	0.045	0.050
5. Same as 3, lower stages, more ineffective slope and sections	0.045	0.050	0.055
6. Same as 5, stony sections	0.050	0.055	0.060
7. Sluggish river, rather weedy or with very deep pools	0.060	0.070	0.080
8. Very weedy	0.100	0.125	0.150
(B) Natural streams in MOUNTAINS			
Banks usually steep, trees and bushes along banks which are submerged when stream in flood			
1. Bottom: gravel, cobbles and not many large boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
(C) Natural Streams on FLOOD-PLAINS			
1. Grass or crops with no bushes or trees	0.025	0.032	0.050
2. Bushes on sides which are in the water when in flood	0.035	0.060	0.160
3. Trees	0.050		0.200
(D) Unnatural Channels			
1. Sides lined with Reno mattresses or gabions		0.025	
2. Rough rip rap		0.040	
3. Machine dug channels not maintained		0.030	
4. Concrete lined		0.016	

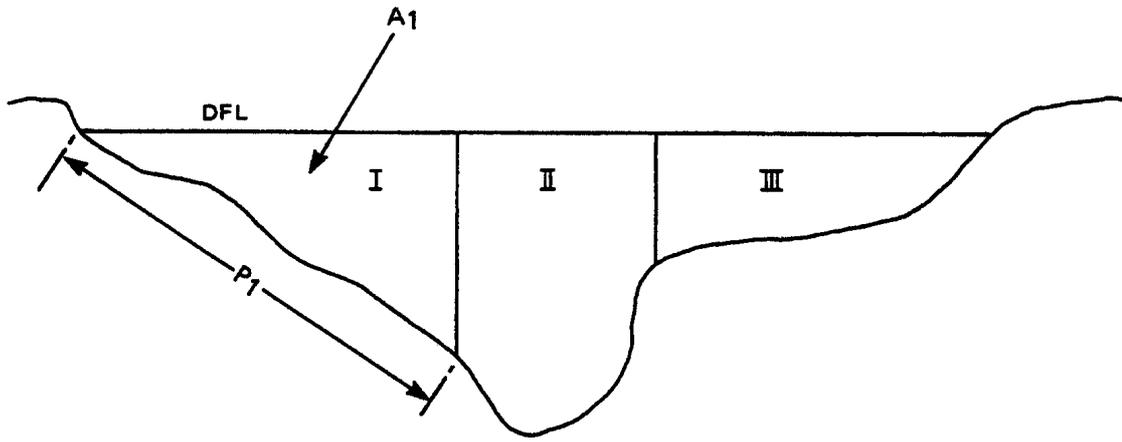


Figure 4.3 River cross section divided into simple shapes

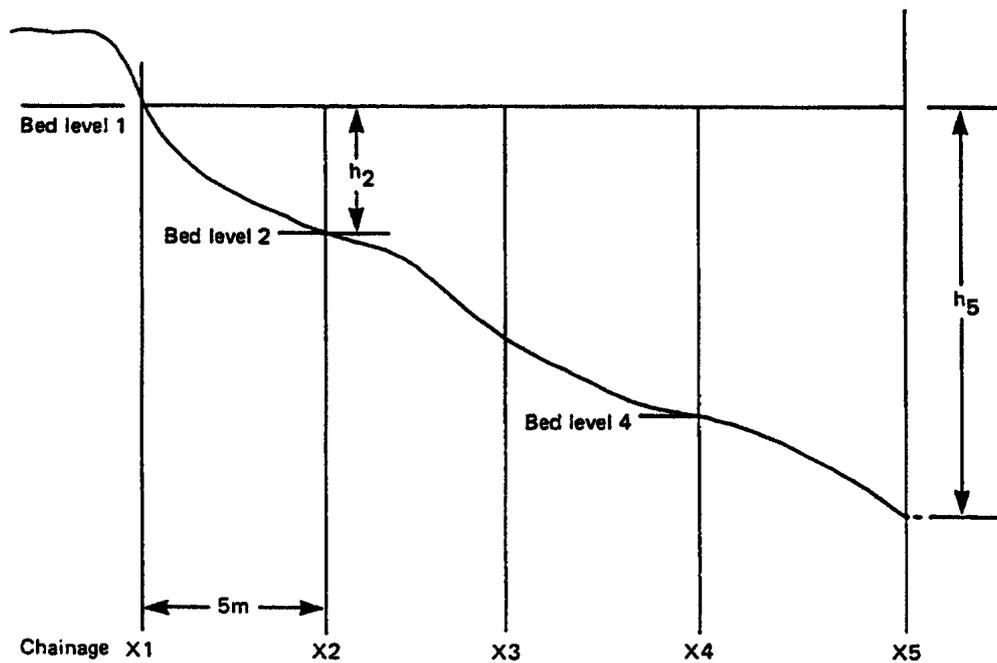


Figure 4.4 Part section I labelled for calculations

same river, by using the orifice formula:

$$Q = C_o(2g)^{1/2} \cdot L \cdot Dd \left[(Du - Dd) + (1 + e) \frac{V^2}{2g} \right]^{1/2} \quad (4.3)$$

where: Q = volume of flow (m^3/sec)

g = acceleration due to gravity ($9.8m/sec^2$)

L = linear waterway, ie distance between abutments minus width of piers, measured perpendicular to the flow (m)

Du = depth of water immediately upstream of the bridge measured from marks left by the river in flood (see the definitions in the introduction to Chapter 5) (m)

Dd = depth of water immediately downstream of the bridge measured from marks on the piers, abutments or wing walls (m)

V = mean velocity of approach (m/sec).

C_o and e are coefficients to account for the effect of the structure on flow, as listed in Table 4.2

These dimensions are shown in Figures 4.5 and 4.6.

The formula is claimed to give nearly correct volumes for most waterway shapes, but Q should be increased by 5 per cent when $Du - Dd$ is greater than

$$\frac{Dd}{4}$$

Table 4.2 Values of C_0 and e in the orifice formula

L/W	C_0	e
0.5	0.892	1.05
0.55	0.88	1.03
0.6	0.87	1.00
0.65	0.867	0.975
0.7	0.865	0.925
0.75	0.868	0.86
0.8	0.875	0.72
0.85	0.897	0.51
0.9	0.923	0.285
0.95	0.96	0.125

(Note: intermediate values may be obtained by interpolation)

L = width of waterway as defined above

W = unobstructed width of the stream

Appendix 4.2 gives an example of a calculation using this formula.

Whenever possible, flow volumes should be calculated by both the area-velocity and orifice formula methods. The higher of the two volumes

should be adopted as the design discharge, provided the results are not too dissimilar. If they differ considerably, the engineer has to form a judgement based on the reliability of the data on which each calculation was based.

4.2.3 Other methods

The rational method of estimating flow volumes from catchment area, run-off coefficients, rainfall intensity and a time factor is a further means of checking estimates. Details of this and other methods may be found in Farraday and Charlton (1983) and Fiddes (1976).

For small catchment areas up to about 15 sq kms, the peak discharge is often between 1 and 2m³/sec per 25 hectares.

4.3 References

Farraday and Charlton (1983). *Hydraulic factors in bridge design.* Wallingford: Hydraulics Research Station Ltd

Fiddes D (1976). *The TRRL East African flood model.* Laboratory Report LR706. Transport Research Laboratory, Crowthorne.

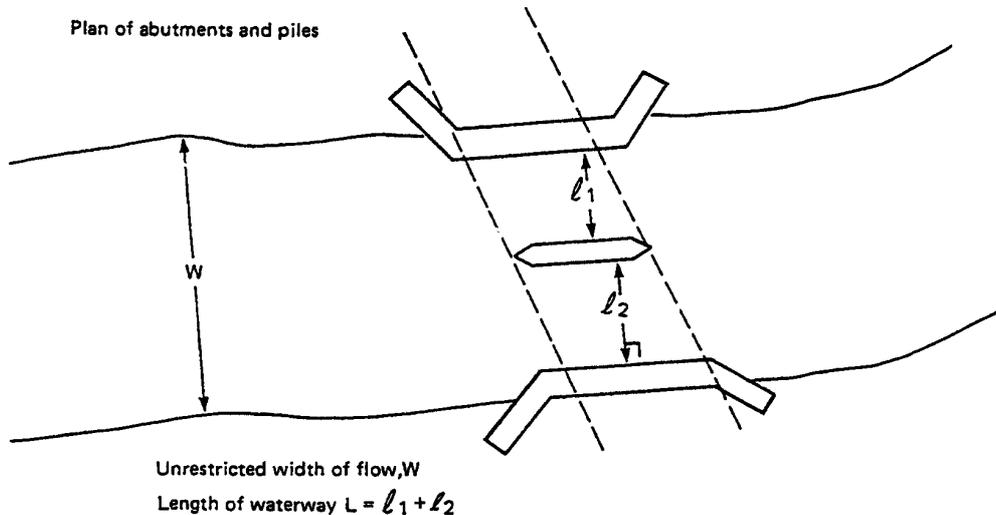


Figure 4.5 Waterway at a bridge

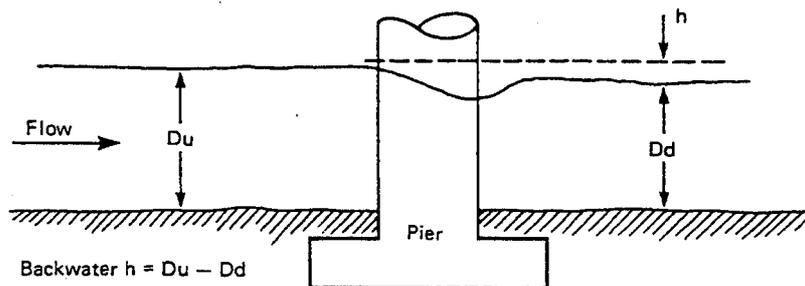


Figure 4.6 Backwater

Appendix 4

Appendix 4.1

Flood discharge calculation by the area-velocity method

Calculate the discharge of a stream with clean straight banks, no rifts or deep pools and bed slope of 0.2% having a cross section, as given in Figure 4.3.

- Since the cross section is irregular, divide the channel into three sub-sections (I, II, and III) as shown in Figure 4.3.
- Calculate the hydraulic conditions of sub-section I as follows:

If necessary, divide the sub-section into small strips as in Figure 4.4. The area and the length of the wetted bed surface of each strip is calculated using the following expressions:

$$\text{Bed surface } P = \left[(h_2 - h_1)^2 + (x_2 - x_1)^2 \right]^{\frac{1}{2}}$$

$$\text{Area } A = \frac{(h_1 + h_2)}{2} \cdot (x_2 - x_1)$$

where h_1 and h_2 are the depths of water at chainages x_1 and x_2 respectively.

Example calculations are shown in Table A4.1

Using Manning's formula:

$$V = \frac{1}{n} \left(\frac{A}{P} \right)^{\frac{2}{3}} S^{1/2} \quad (4.1)$$

$$\text{Now hydraulic mean depth } \frac{A}{P} = \frac{13.87}{20.05} = 0.692\text{m}$$

From Table 4.1, section (A) 1, the coefficient of rugosity (n) is 0.030.

Bed slope is 0.2%, i.e. $s = 0.002$

$$\text{Hence } V = \frac{1}{0.03} \times 0.692^{\frac{2}{3}} \times 0.002^{1/2} = 1.22\text{m/sec}$$

The flow volume for sub-section I (Q_1):

$$Q_1 = A \cdot V = 13.87 \times 1.22 = 16.92\text{m}^3/\text{sec}$$

- Similarly, the hydraulic conditions for the other sub-sections are calculated as follows:

		<i>Sub-Section</i>	
		II	III
Wetted bed surface, P(m)		15.22	10.05
Area, A (m ²)		26.81	4.40
Velocity, V (m/sec)		2.17	0.86
Discharge, Q (m ³ /sec)		58.18	3.78
Total Discharge	= $Q_1 + Q_2 + Q_3$		
	= $16.92 + 58.18 + 3.78$		
	= $78.88\text{m}^3/\text{sec}$		
	= Approx $80\text{m}^3/\text{sec}$		

Appendix 4.2

Flood discharge calculation using the orifice formula

Calculate the discharge passing through a bridge with a waterway width of 18m across a stream 30m wide. In flood the average depth of flow upstream is 2.2m.

Discharge at a section just upstream of the bridge, assuming a rectangular cross section:

$$Q = A \cdot V \quad (4.2)$$

$$Q = 2.2 \times 30 \times V = 66 \cdot V\text{m}^3/\text{sec} \quad (4.4)$$

Table A4.1 Flood discharge calculation by the area-velocity method

Chainage (m)	Bed Level (m)	Depth of water (m)	$x_2 - x_1$ etc (m)	Bed Surface P (m)	Area A (m ²)
$x_1 = 0$	100.00	$h_1 = 0.00$			
$x_2 = 5$	99.60	$h_2 = 0.40$	5.0	$(0.40^2 + 5^2)^{\frac{1}{2}} = 5.02$	$\frac{0.4}{2} \times 5.0 = 1.0$
$x_3 = 10$	99.30	$h_3 = 0.70$	5.0	$(0.30^2 + 5^2)^{\frac{1}{2}} = 5.01$	$\frac{1.10}{2} \times 5.0 = 2.75$
$x_4 = 15$	99.00	$h_4 = 1.00$	5.0	$(0.30^2 + 5^2)^{\frac{1}{2}} = 5.01$	$\frac{1.70}{2} \times 5.0 = 4.25$
$x_5 = 20$	98.65	$h_5 = 1.35$	5.0	$(0.35^2 + 5^2)^{\frac{1}{2}} = 5.01$	$\frac{2.35}{2} \times 5.0 = 5.87$
			TOTAL	20.05	13.87

Discharge at a section just downstream of the bridge will be the same and will be given by the orifice formula:

$$Q = C_o(2g)^{1/2}L.Dd \left[(Du - Dd) + (1+e)\frac{V^2}{2g} \right]^{1/2} \quad (4.3)$$

$$L = 18, W = 30 \text{ ie } \frac{L}{W} = 0.6$$

and from Table 4.2, $C_o = 0.87$ and $e = 1.00$

$$Q = 0.87(2 \times 9.8)^{1/2} \times 18 \times 2 \left[0.2 + \frac{(1+1)V^2}{2 \times 9.8} \right]^{1/2}$$

$$Q = 138.6 \left[0.2 + 0.102V^2 \right]^{1/2}$$

$$66V = 138.6 \left[0.2 + 0.102V^2 \right]^{1/2} \text{ Substituting (4.4)}$$

$$V^2 4.41 \left[0.2 + 0.102V^2 \right] \text{ Squaring both sides}$$

$$V^2 = 0.881 + 0.4498V^2$$

$$\text{Hence } V^2(1 - 0.4498) = 0.881$$

$$V^2 = \frac{0.881}{0.5502}$$

$$V = 1.265$$

$$\text{Now } Q = 66V = 83.5 \text{m}^3 / \text{sec}$$

5. HYDRAULIC DESIGN

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5 Hydraulic design

When a bridge structure and its associated embankments encroach upon the flow of a river in flood, there is a risk to the structure, the embankments and the surrounding land. It is not economical, however, to build a bridge to clear a wide floodplain because bridge works cost more than earth embankments. Often bridges are designed to accommodate annual high flows without excessively restricting the flow of the river or incurring damage either to the structure or the surrounding land. In these cases, it is accepted that unusually high floods may over-top the superstructure and cause temporary disruption.

The annual high flow is termed the **design flood** (DF) and the highest known flood, the **high flood** (HF). Precautions may be necessary to ensure that restricting the flow of the design flood does not cause either excessive backwater, resulting in flooding upstream, or scour that may damage the structure. The bridge parts should also be designed to withstand the high flood, though some damage may result to adjoining land and it may subsequently be necessary to repair scour holes, etc, in the river channel.

On important roads the bridges may be designed for a design flood occurring no more than once every ten years and a high flood once every hundred years. A minor road may be served by bridges or low water crossings designed to be overtopped for a few days every year.

The bridge designer is required to select a design flood level (DFL), a design discharge ie flow volume, and a design velocity, on which to base calculations of waterway geometry, foundation depth, scour protection and vertical clearance.

In summary:

The **design flood** is the maximum flow that can pass through the bridge without:

- causing unacceptable disruption to traffic;
- endangering the pier and abutment foundations with scour;
- damaging approach embankments;
- causing flood damage on the upstream side of embankments.

The **high flood** is the rarely occurring flow that it is uneconomical to include in the design flood, but which may be considered when designing the superstructure and piers of the bridge.

The simplest situation is shown in Figure 5.1a, where the incised river is contained within its banks at the design flood, and is narrow enough to be bridged

with a single span. The abutments are built clear of the level of the design flood and hence there is no restriction of its flow. River training works are unlikely to be required, and no backwater or additional scour is expected as a result of the presence of the bridge.

Figure 5.1b illustrates a crossing of a wider flood channel. The superstructure is longer and will be very expensive if piers are not used. Both pier and abutment foundations are below DFL and will require protection from scour. The abutment walls and piers will impede the design flood. This restriction to flow will cause some backwater and additional scour of the bed, which must be taken into account in foundation design.

Where a small river has a wide flood plain (Figure 5.1c), the most economical solution may be a relatively small bridge, approached by earth embankments. The structure will restrict the flow during the design flood but not during the normal flow. In order to prevent flood water rising too high on the upstream side, relief culverts are installed in the embankments while retaining walls and river training works channel the main flow through the bridge. The abutment foundations are designed to withstand the scour caused by the restriction to flow.

5.1 The hydraulic design process

The sequence of data collection and hydraulic design is illustrated in the flow diagram, Figure 5.2, taken from Farraday and Charlton (1983). Their book, 'Hydraulic factors in bridge design', describes the process in more detail than can be shown here and is recommended for its explanations and worked examples.

The general order of design is:

- 1 Establish the height of the superstructure, ie clearance above the DFL.
- 2 Establish trial positions for the abutments according to the bed shape at the proposed crossing.
- 3 Make a provisional decision regarding the number of piers that will result in the lowest overall cost of superstructure, piers and abutments.
- 4 Calculate the general scour and the local scour due to abutments and piers, and draw the worst case profiles on the cross sections.
- 5 Check that the backwater caused by the restriction to flow does not cause damage to surrounding land upstream of the bridge or affect the height set for the superstructure.
- 6 Prepare preliminary designs of abutments and piers.

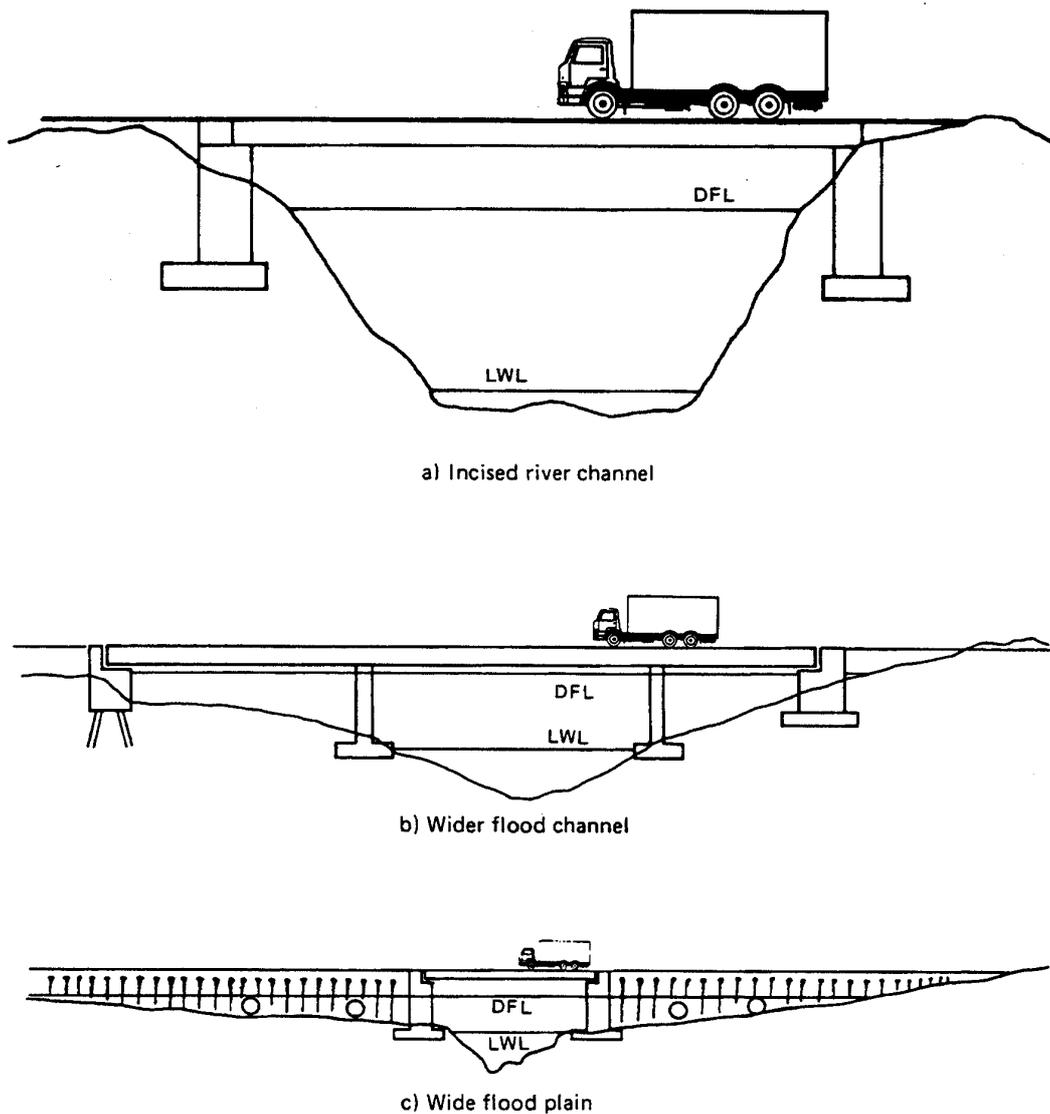


Figure 5.1 Bridges for different flood widths

- 7 Check scour and backwater effects and make adjustments as required, recalculating the effects of any changes to the waterway.
- 8 Calculate the cost of the superstructure once its length is decided, and the costs of the substructures, embankments, river training works and relief culverts.
- 9 In order to obtain the most economical design, or to compare the costs of structures catering for different design floods, it may be necessary to repeat the above procedure on the basis of alternative waterway conditions.

5.2 Bridge height

The waterway below the superstructure must be designed to pass the design flood and the floating debris carried on it. Table 5.1 gives recommended

Table 5.1 Vertical clearance at DFL minimum measurements for vertical clearance between the lowest part of the superstructure and the DFL, taking into account backwater effects described in Section 5.5. These clearance measurements should be increased on rivers with a history of unusually large floating items or a navigational requirement.

Table 5.1 Vertical clearance at DFL

Discharge (m ³ /sec)	Vertical clearance (mm)
<0.3	150
0.3 to 3.0	450
3.0 to 30.0	600
30 to 300	900
>300	1200

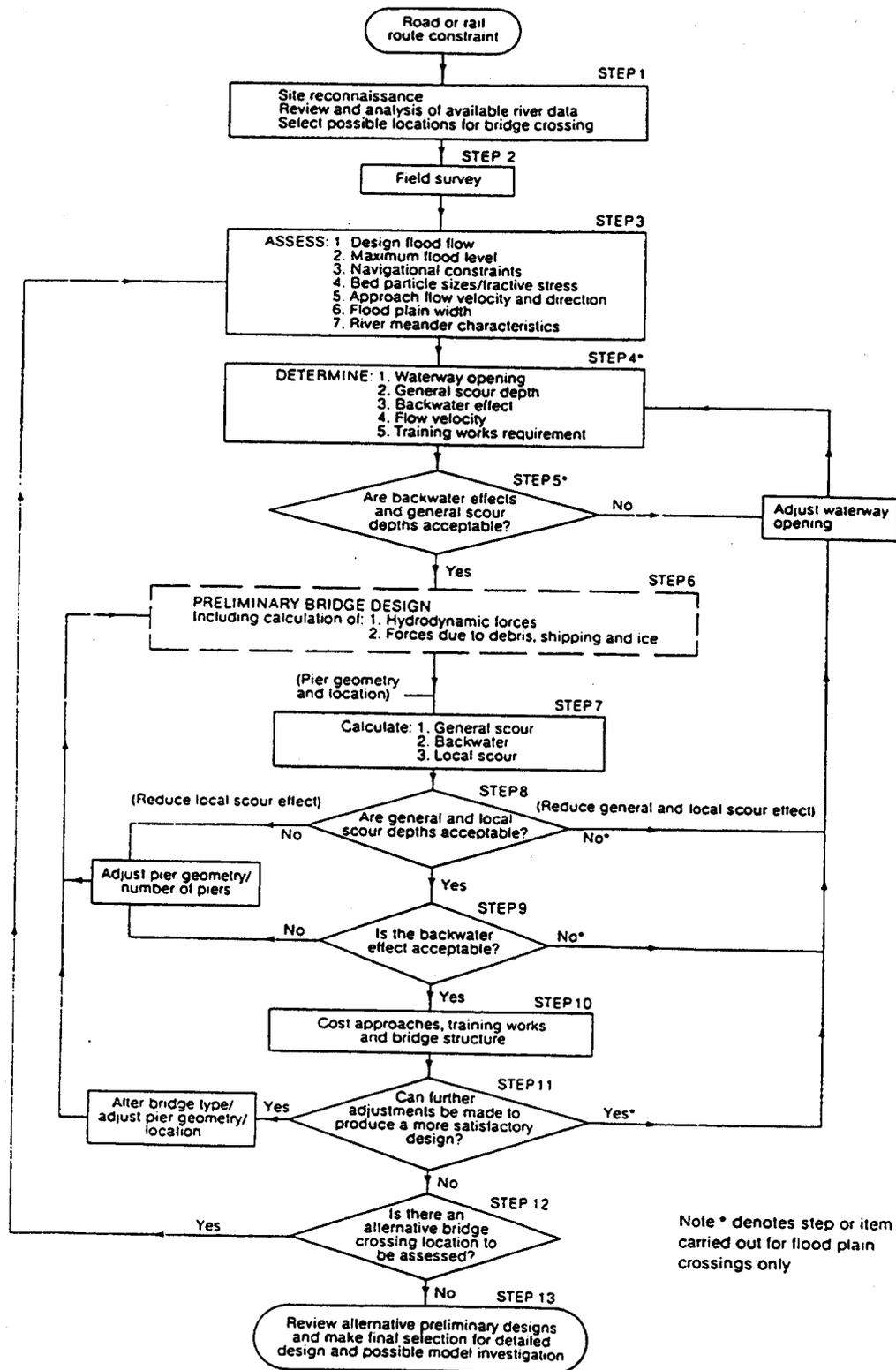


Figure 5.2 Flow diagram of the hydraulic design of a bridge

5.3 Positioning the abutments and piers

The engineer should choose trial positions for the abutments, bearing in mind the guidelines in the introduction to this chapter and Figure 51. The requirement for piers will depend on several factors: the availability and cost of material for the structural members of the superstructure; traffic loading;

distance between the abutments; the length of timber or steel beams available; HA or HS20 loading and suspended length. A configuration of abutments and piers should be selected and the length of waterway measured, ie the distance between the abutments, measured perpendicular to the flow, minus the sum of the pier widths (Figure 4.5).

Using these measurements and the estimates or calculations of flow volume and velocity, outlined in Sections 4.1 and 4.2, the engineer can calculate the likely scour and backwater effects. If these prove unacceptable, a different configuration of abutments and piers should be tried.

5.4 Scour

Scour is the erosive effect of water flow on the river bed or banks. Bridge works may alter the existing scour pattern by restricting the free flow of the stream and/or causing local changes to the current. Approximately half of all river bridge failures are due to scour alone.

There are four types of scour:

- natural scour and channel shifting on alluvial rivers (Section 2.1.1);
- scour caused by changes to the river channel upstream or downstream of the bridge site;
- 'general' scour caused by reduction in the channel width at bridge works;
- 'local' scour at the base of piers, abutments and river training works where these divert the general flow.

Examination of the river at the bridge site and at any existing structures, preferably during and soon after a time of flood, will indicate if and where river training works are required to combat channel shifting and to guide the flow through the waterway at the bridge. Any changes to the river channel should be considered as likely to affect the proposed structure and other nearby structures on the river by changing scour patterns.

At the bridge site estimates are required for general scour and local scour.

5.4.1 General scour

There are many relationships that can be used to predict general scour. All of them assume a fairly simple river channel shape. At complex locations, particularly near the junction of two streams, these methods are known to be inaccurate.

5.4.1.1 The area-velocity method for predicting general scour

Despite the rise in water level on the upstream side of a bridge during flood, the flow through the structure is assumed to be equal to the unrestricted flow (Section 4.2), as calculated on the basis of velocity measurement (Section 4.1.1) or estimation (Section 4.1.2).

To predict the likely general scour of a river bed between confined banks or abutments, a probable scour line is drawn on the cross-section at the bridge: the area (A) and wetted perimeter (P) below the design flood level are measured as explained in Section 4.2. Water velocity is given by Manning's formula,

$$V = \frac{1}{n} \left(\frac{A}{P} \right)^{2/3} S^{1/2} \quad (4.1)$$

where n is Manning's rugosity coefficient found in Table 4.1 and

s is the gradient of the water or average bed gradient.

This process is repeated, ie probable scour lines are drawn and the corresponding water velocities calculated until the product of calculated V and area A is approximately the same as the unrestricted discharge. The shape of the additional scour area should be adjusted as shown in Figure 5.3.

5.4.1.2 The competent velocity method for predicting general scour

The competent velocity of flow is the greatest velocity that will not cause scour to the river bed. In this method, the waterway at the bridge, ie the cross-sectional area of the flow, is increased until the resultant mean velocity of flow is reduced to the competent velocity, V_c .

In order to select an appropriate value for V_c , a probable scour depth at the bridge is estimated and the corresponding competent velocity is read from Table 5.2 for cohesive bed material, or from Figure 5.4 for non-cohesive bed material.

$$\text{Using } A = \frac{Q}{V_c} \quad (5.1)$$

Where Q = unrestricted flow volume (m³/sec)

V_c = competent velocity (m/sec)

Table 5.2 Table of competent velocities for erodible materials.

Depth of flow (m)	Competent mean velocity, V_c		
	Low values (easily erodible material) (m/s)	Average values (m/s)	High values (resistant material) (m/s)
1.5	0.6	1.0	1.8
3	0.65	1.2	2.0
6	0.7	1.3	2.3
15	0.8	1.5	2.6

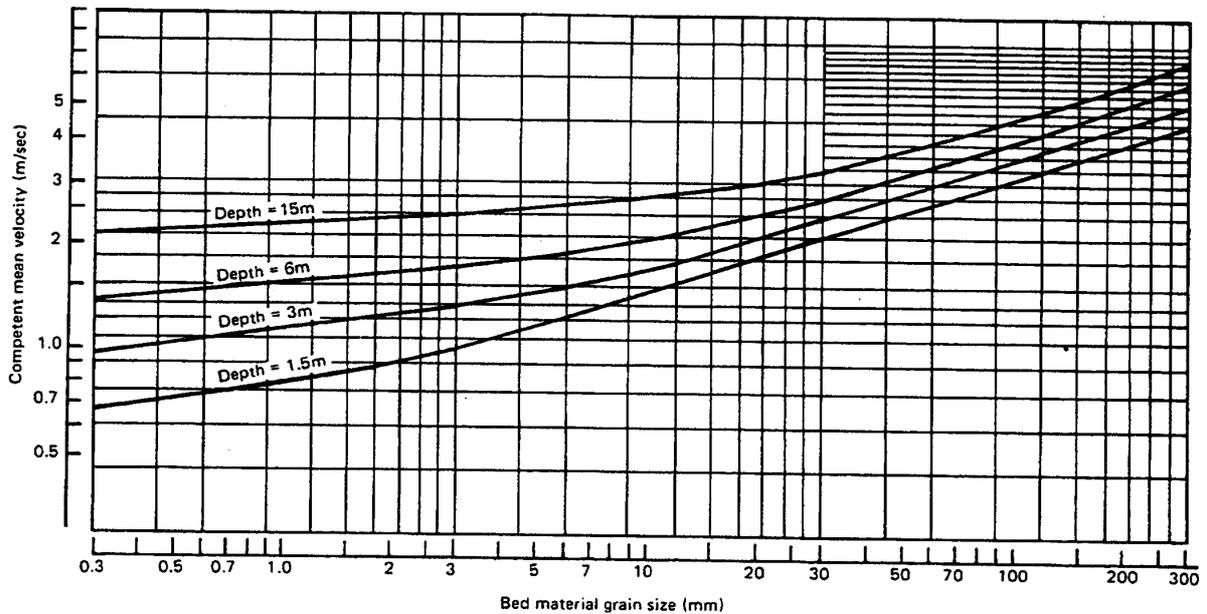


Figure 5.4 Competent velocity chart for river beds of non-cohesive soils.

The area of the waterway, A , is calculated and adjusted as shown in Figure 5.3. The mean depth used for the calculation must correspond with the area derived.

5.4.2 Local scour

In addition to the general scour discussed in 5.4.1, there will be local scour of the river bed at the bridge site caused by turbulent flow around obstacles such

As piers, unless the bed is armoured. The localised turbulence also removes bed material from critical areas wherever the flow of water is diverted downwards or sideways around the ends of wing walls, abutments or river training works. Since local scour is related to bed material and water velocity, a study of scour patterns at existing structures on the same river will give an indication of conditions likely to occur at the new structure.

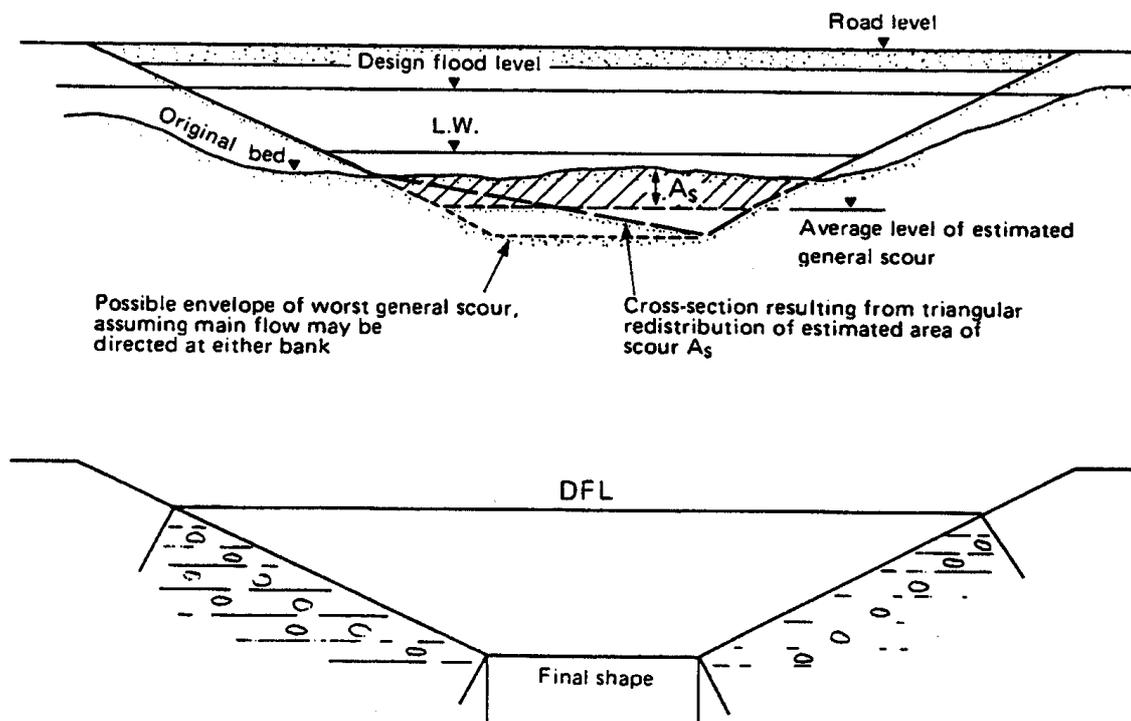


Figure 5.3 Adjustment of waterway to accommodate general scour

5.4.2.1 Local scour at piers

Figure 5.5 gives a basis for estimating the local scour likely to occur at the upstream end of piers. If the piers are skewed in relation to the direction of flow, the depth of scour should be multiplied by a factor from Table 5.3.

Table 5.3 Multiplying factors for local scour at skewed piers

Angle of attack	Length-to-width ratio of pier		
	4	8	12
0°	1.00	1.00	1.00
5°	1.17	1.40	1.65
10°	1.33	1.70	2.12
15°	1.50	2.00	2.50
20°	1.67	2.25	2.83
25°	1.83	2.50	3.17
30°	2.00	2.75	3.50

5.4.2.2 Local scour at abutments

One means of estimating local scour at abutments and river training works is to use the general scour prediction as illustrated in Figure 5.3. Blench (1969) suggests that the general scour depth should be used with a multiplier from Table 5.4. Where abutments protrude into the flow, it is suggested that the greater of the scour estimates for the piers and the level

calculated by multiplying the general scour depth by 2.25 should be used

Table 5.4 Multipliers for estimating scour depth at abutments and training works

Nature of location	Multiplier
Nose of groynes or guide banks	2.0 to 2.75
Flow impinging at right angles on bank	2.25
Flow parallel to bank	1.5 to 2.0

5.5 Backwater effect (afflux)

The restriction to free flow due to piers and abutments causes the head of water to rise on the upstream side of the structure. The water level downstream may also be lowered, but is usually assumed to be unaffected by the bridge. The engineer should estimate the rise on the approach side to ensure that the river bank will not be breached where floods would cause damage, and to check that the desired clearance for floating debris or river traffic is maintained below the superstructure.

The magnitude of backwater can be affected by the shape of the river bed, its eccentricity, scour holes and related factors. It is also influenced by the shape

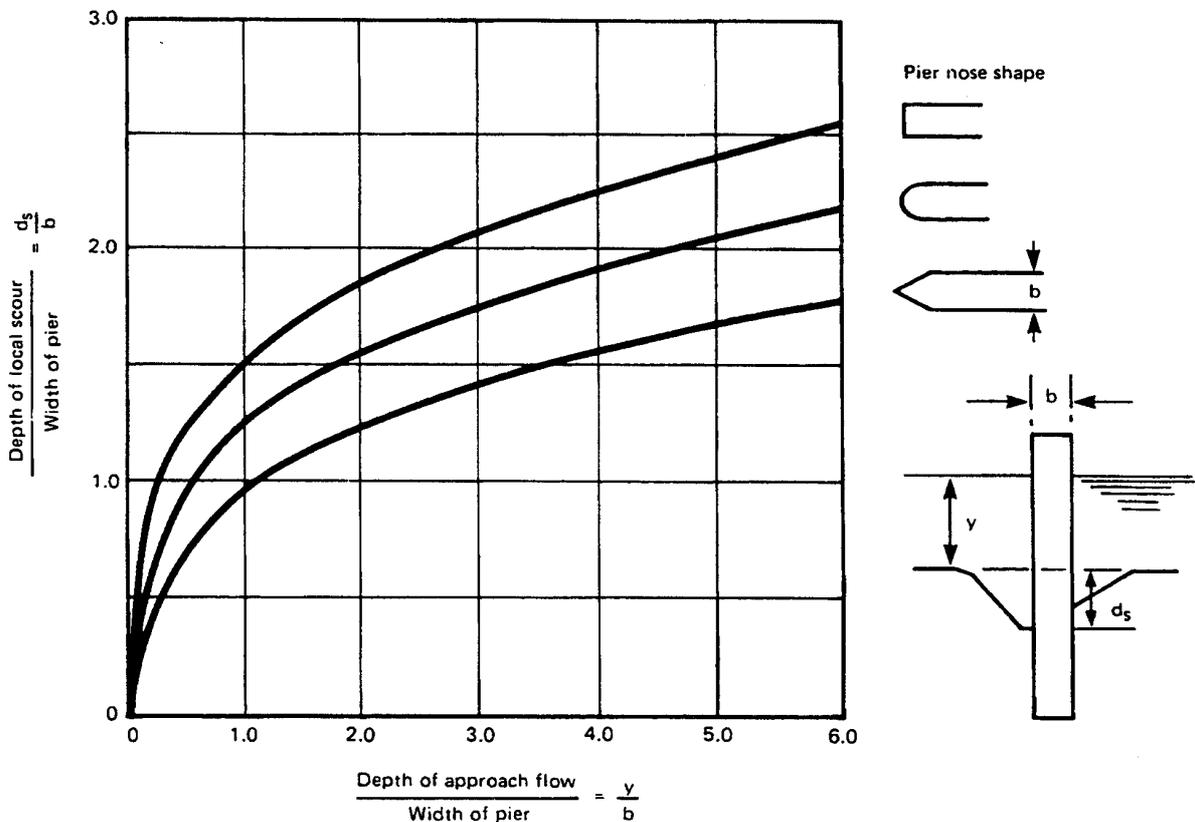


Figure 5.5 Curves for estimating depth of local scour at bridge piers

of the bridge works, ie by any skew, river training, embankments and relief culverts. For simple configurations, a good estimate may be made using the formula:

$$h = \frac{V^2}{2g} \left(\frac{W^2}{C^2 L^2} - 1 \right) \quad (5.2)$$

where h = afflux (m)
 V = average velocity of flow (m/sec)
 g = acceleration due to gravity
 (9.8m/sec²)
 W = unobstructed width of the stream (m)
 L = linear waterway as defined in Section 4.2.2 (m)
 C = coefficient of discharge through the bridge, taken as 0.7 for sharp entry and 0.9 for bell mouthed entry.

The formula (5.2) is taken from Victor (1980). More detailed methods of calculation, including use of the orifice formula, are found in Bradley (1972) and Henderson (1966).

It bears repetition that the calculations set out here and elsewhere in this chapter can produce only broad estimates of scour depth, afflux and other variables, since each river bed and bridge location has a distinct and complex set of characteristics. The bridge engineer is required to produce a conservative design, taking into account the required service life and a realistic estimate of the quality of the maintenance available to detect and repair at an early stage any damage to the river banks and bed that may affect the structure.

5.6 Culverts

Where a required waterway opening is less than about 15m² and in particular, where a road crosses a stream on a relatively high embankment, it is usually cheaper to provide a culvert than a bridge.

Before the hydraulic requirements of a culvert can be calculated, an assumption has to be made about the type of construction to be used. The most common forms of culvert construction are:

- precast concrete joined pipes;
- prefabricated corrugated steel pipes;
- precast or built in situ concrete boxes.

Concrete pipe culverts are commonly used for small openings up to about 2m² and multiple pipes, with or without common headwalls, are used for larger areas. For areas greater than 2m², reinforced concrete box

culverts or sometimes corrugated steel pipe culverts are used, singly or in multiples.

Culvert design is the subject of Chapter 12.

5.6.1 Culvert hydraulics

Like bridges, culverts are designed to be large enough to pass the design flood without damage to the embankment or surrounding land. In practice this usually means limiting the height of the flood on the upstream side. The required size of the culvert is found by calculating the area required to permit a flow that will maintain the upstream head of water below the critical level. The head downstream is taken to be either the design flood level before the embankment is built or the top of the culvert, whichever is the higher.

The operating head H is defined as

where h_u is the upstream head and
 h_d is the downstream head.

The head, H , can then be said to equal the sum of the losses at the culvert, ie

$$H = h_e + h_f + h_o \quad (5.3)$$

where h_e is the loss on entry
 h_f is the friction loss and
 h_o is the loss at the outlet.

These losses are estimated using the velocity head:

$$h_e = k_e \cdot \frac{V^2}{2g}$$

$$h_o = k_o \cdot \frac{V^2}{2g}$$

$$h_f = f \cdot \frac{L}{D} \cdot \frac{V^2}{2g}$$

where V = flow velocity (m/sec)
 g = acceleration due to gravity (m/sec²)
 L = culvert length (m)
 D = culvert internal diameter (m)

The coefficients K_E and f depend on the entry size and shape. For the concrete box culvert designs detailed in Chapter 12,

k_e can be taken as 0.15 and
 f can be taken as 0.016
 k_o is assumed to be 1.0 for all pipes.

$$\text{therefore } H = \frac{V^2}{2g} \left[f \cdot \frac{L}{D} + k_e + k_o \right] \quad (5.4)$$

Hence for concrete culverts

$$H = \frac{V^2}{2g} \left[0.016x \frac{L}{D} + 1.15 \right] \quad (5.5)$$

A typical calculation is set out in Appendix 5.1.

For corrugated metal pipes projecting from the fill, K_e can be as high as 0.9. Use of head-walls can reduce this to 0.5. The f value for corrugated pipes should be 0.075.

5.7 Use of existing abutments and piers

When replacing a bridge deck, the sub-structure foundations should be examined to verify that they are sufficiently deep to be unaffected by the maximum calculated scour. If they are not, scour protection as outlined in Chapter 6 will be required.

5.8 References

Blench T (1969), *Mobile bed fluviology*. Edmonton: University of Alberta Press.

Bradley J N (1972), *Hydraulics of bridge waterways*. Washington: US Bureau of Public Roads.

Farraday and Charlton (1983), *Open channel flow*, Collier-Macmillan, London.

Victor D J (1980), *Essentials of bridge engineering*. New Delhi: Oxford and IBH Publishing Co.

Appendix 5

Appendix 5.1

Calculation of culvert operating head.

Determine the head required to produce a discharge of $2.0 \text{ m}^3/\text{sec}$ through a concrete culvert having a length of 20.0m and a diameter of 1.0m.

$$\text{Now } A = \frac{\pi D^2}{4} = \frac{\pi \times 1^2}{4}$$

$$\text{so } V = \frac{Q}{A} = \frac{2 \times 4}{\pi \times 1^2} = 2.55 \text{ m/sec}$$

$$H = \frac{V^2}{2g} \left[0.016x \frac{L}{D} + 1.15 \right] \quad (5.5)$$

$$H = \frac{2.55^2}{2 \times 9.8} \left[0.016 \times \frac{20}{1} + 1.15 \right]$$

$$= \frac{2.55^2}{2 \times 9.8} \times 1.47$$

$$= 0.49$$

The head required is 0.49m.

6 RIVER AND SCOUR PROTECTION

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6 River and scour protection

Scour protection is usually required where a bridge is built across a meandering stream or when some restriction to the flow of the design flood occurs at a bridge. Protection measures can take the form of:

- rip rap on slopes or river bed;
- gabion or Reno mattress aprons or revetments;
- groynes;
- piled walls;
- vegetation.

With the exception of groynes and some forms of vegetation, the firm rule is that these protective measures must not protrude in any way into the design waterway at the crossing, since a protrusion would restrict the flow even further, causing additional scour.

Experience is by far the best guide as to where river training works are required. Whether the engineer is considering measures to protect a new structure or to prevent further damage to an existing one, he will find it useful to examine protective works on other structures in the area, for their effectiveness and their durability. Observation of the currents at the site in question, preferably during a flood, and examination of plans showing how the river shape progresses, may also indicate where bank strengthening and guide walls are needed. Farraday and Charlton (1983) provide details and further references.

6.1 Protection materials

6.1.1 Rip rap

Rip rap river bed protection consists of a carpet of loose stones, which prevents the water current from eroding the soft bed material. The stone elements must be heavy enough to resist being washed away by maximum water velocities during a flood, and they should not be installed in a manner which reduces the area of waterway designed to accommodate general scour.

The main advantages of rip rap are that:

- it is relatively cheap;
- it is a flexible protection and often finds its own effective level;
- if appropriate lifting gear is available it is easy to install and repair.

Since stone of uniform size is unlikely to be available and does not bed down well, a judicious amount of

grading is recommended. The stone should have a size distribution such that the ratios between the maximum and median size and between the median and minimum size are both about 2:1. Table 6.1 gives two examples of suitable stone gradings by weight.

Table 6.1 Stone rip rap gradings for stream bank revetment

Class I	
Nominal 0.3m diameter or 35kg weight. Allowable local velocity up to 3m/sec.	
Grading:	kg
100% smaller than	140
at least 20% larger than	70
at least 50% larger than	35
at least 80% larger than	10
Class II	
Nominal 0.5m diameter or 180kg weight. Allowable local velocity up to 4m/sec.	
Grading:	kg
100% smaller than	700
at least 20% larger than	320
at least 50% larger than	180
at least 80% larger than	30

Note The percentages are by weight:
the sizes are equivalent spherical
diameters = $1.24 \times \sqrt[3]{\text{volume}}$.

Figure 6.1 shows recommended stone sizes for different **local** flow velocities. Figure 6.2 is a conversion chart to change nominal stone diameter to weight. The thickness of the stone blanket should be at least the length of the largest stones and about twice the nominal median diameter. Durable stone of a near cubic shape is best. Flaky material should be discarded.

6.1.2 Gabions and Reno mattresses

The standard gabion is a rectangular basket made of steel wire mesh (Figure 6.3). It is strengthened at the corners by heavier wire and by mesh diaphragms which divide it into typically 1m compartments. The gabion is supplied as a flat pack for assembly on site and is normally filled in situ with quarried stone or rounded shingle of sufficient size to prevent the stones passing through the mesh. The gabion structure is more stable and durable if the stones are packed carefully by hand, almost as if building a stone wall. Internal tie wires at about 0.5m intervals help to retain the shape. Sections of gabions are securely wired together in position to form a continuous revetment or wall (Figures 6.7 and 6.8).

Standard sizes are 2m, 3m and 4m long by 1m wide by 0.5m or 1m high.

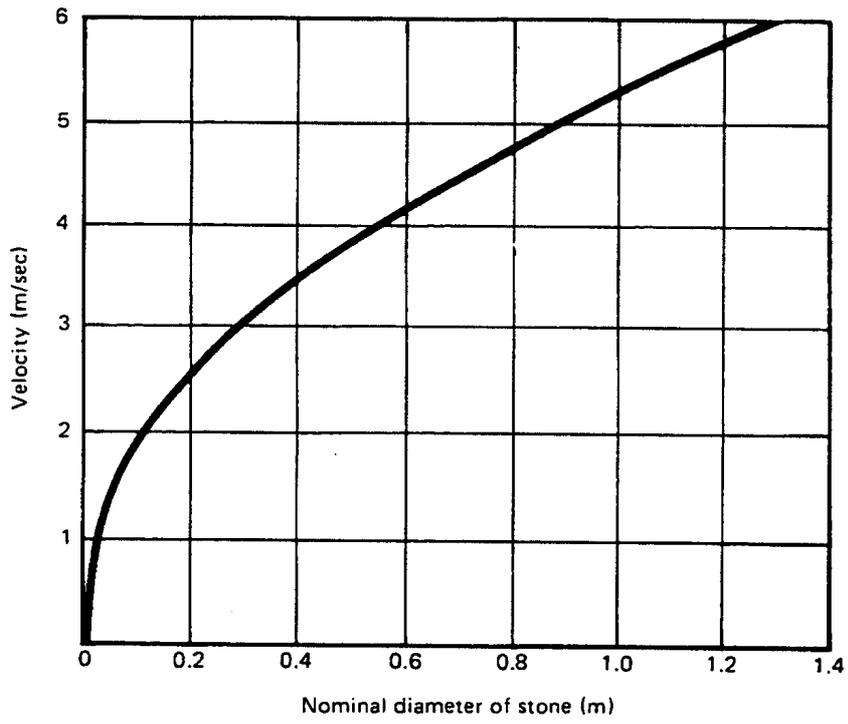


Figure 6.1 Rip rap stone size

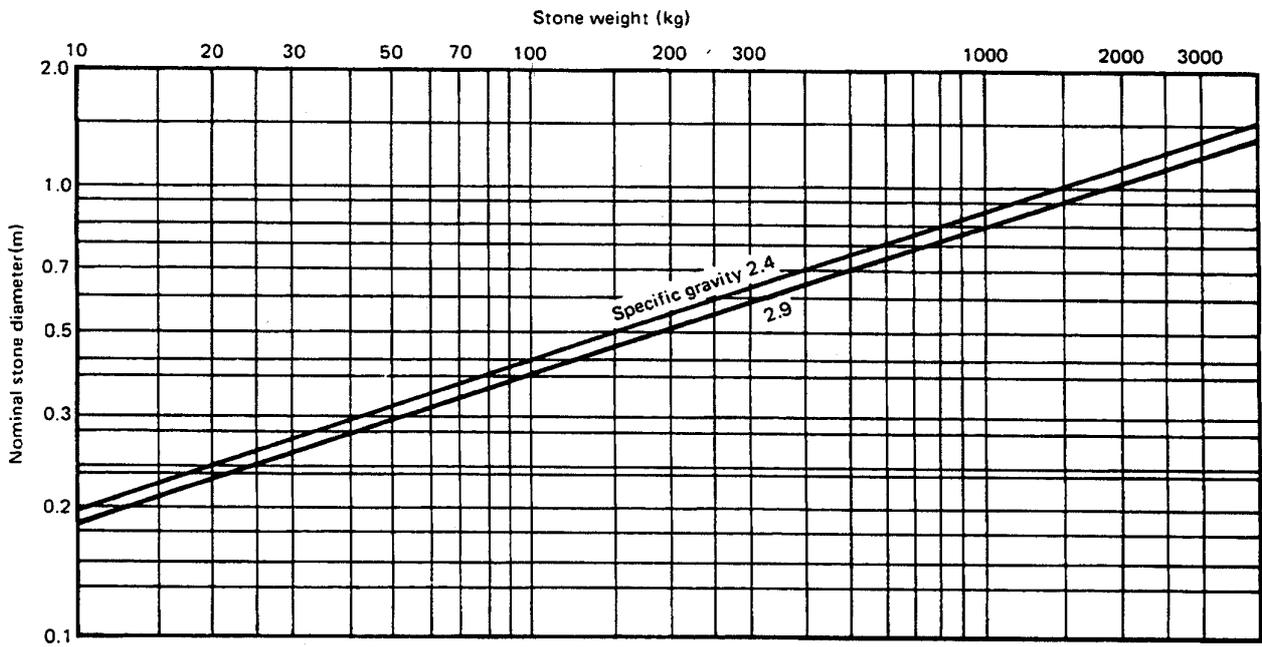


Figure 6.2 Stone diameter/weight conversion chart

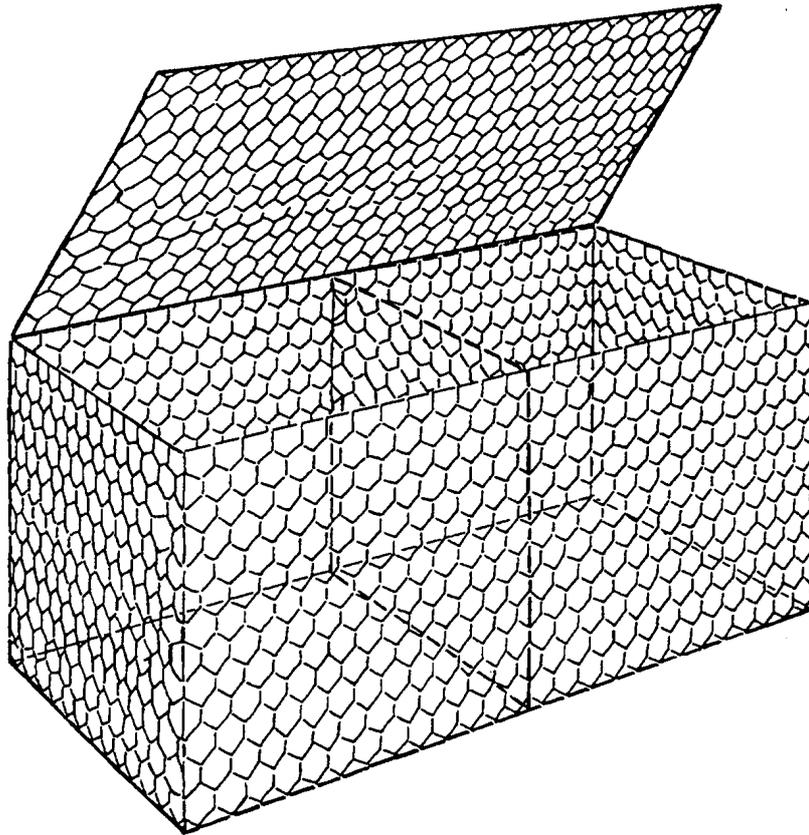


Figure 6.3 Gabion basket

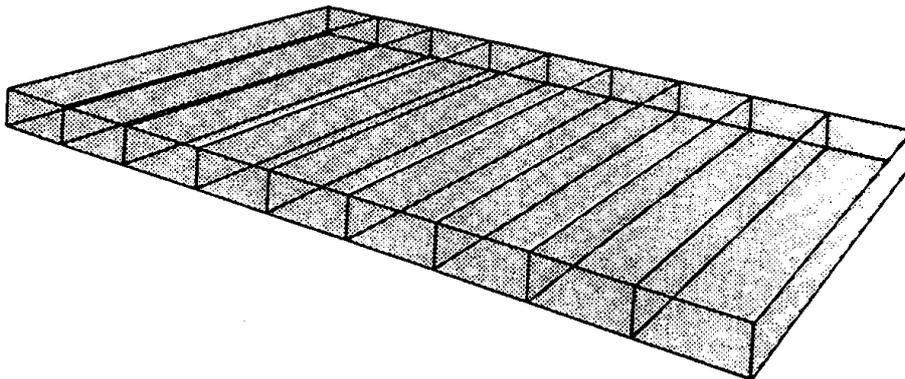


Figure 6.4 Reno mattress

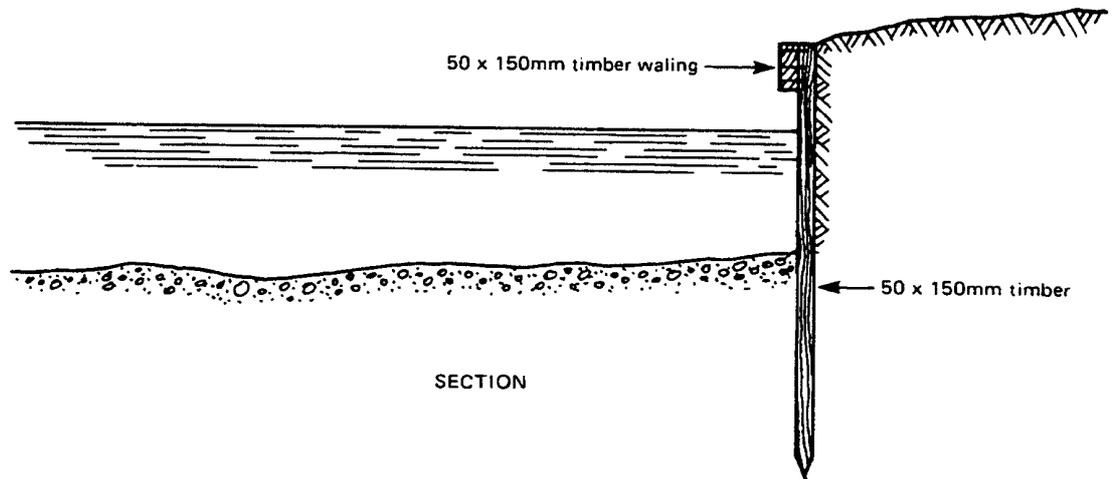
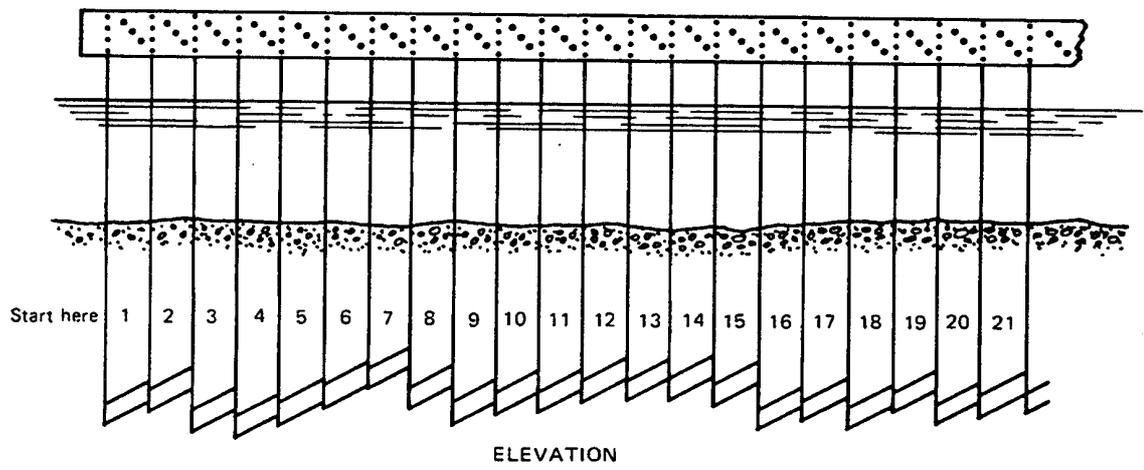


Figure 6.5 Timber piled bank protection

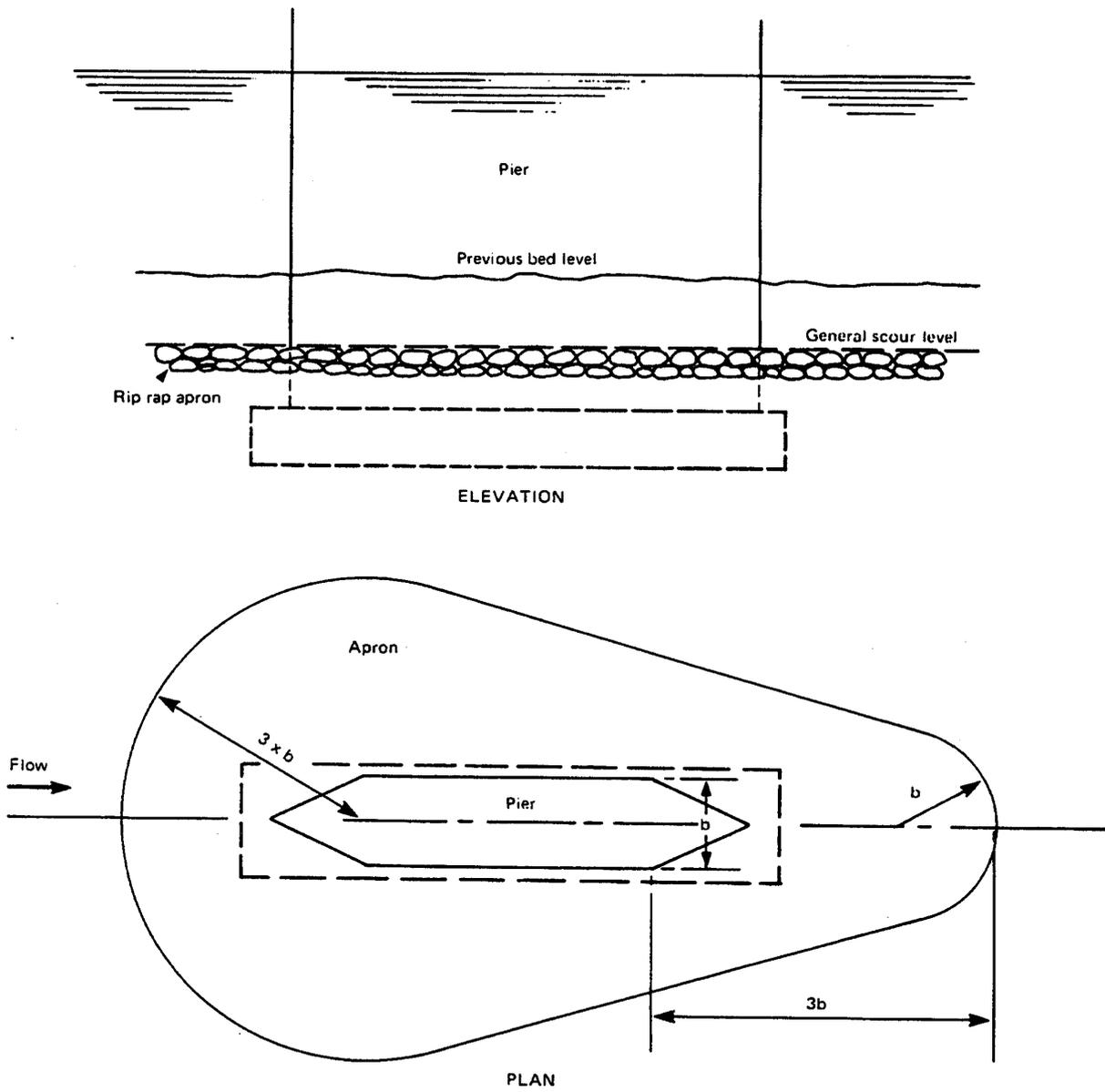


Figure 6.6 Area requiring scour protection around a pier base

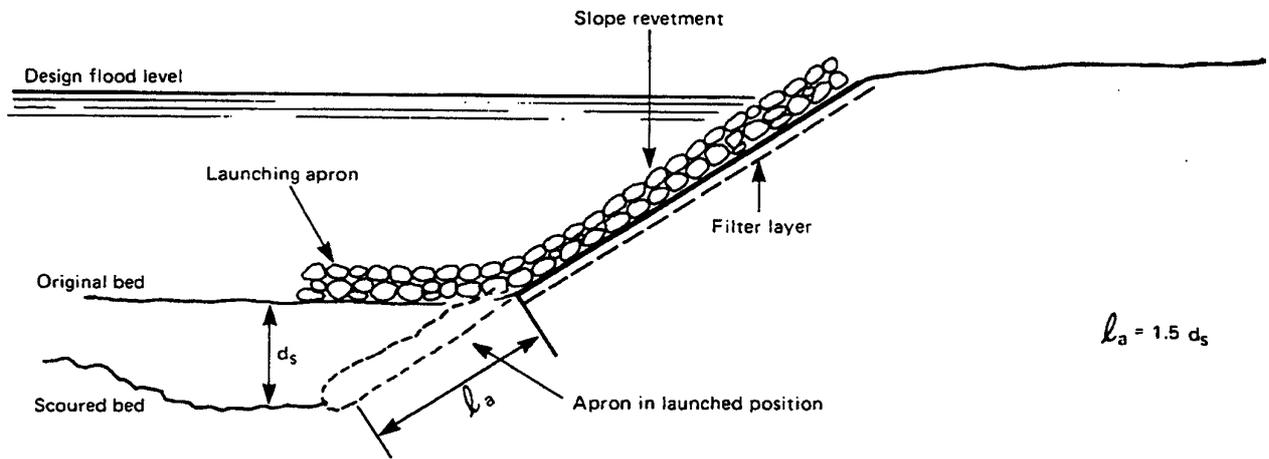


Figure 6.7 Slope revetment with a launching apron

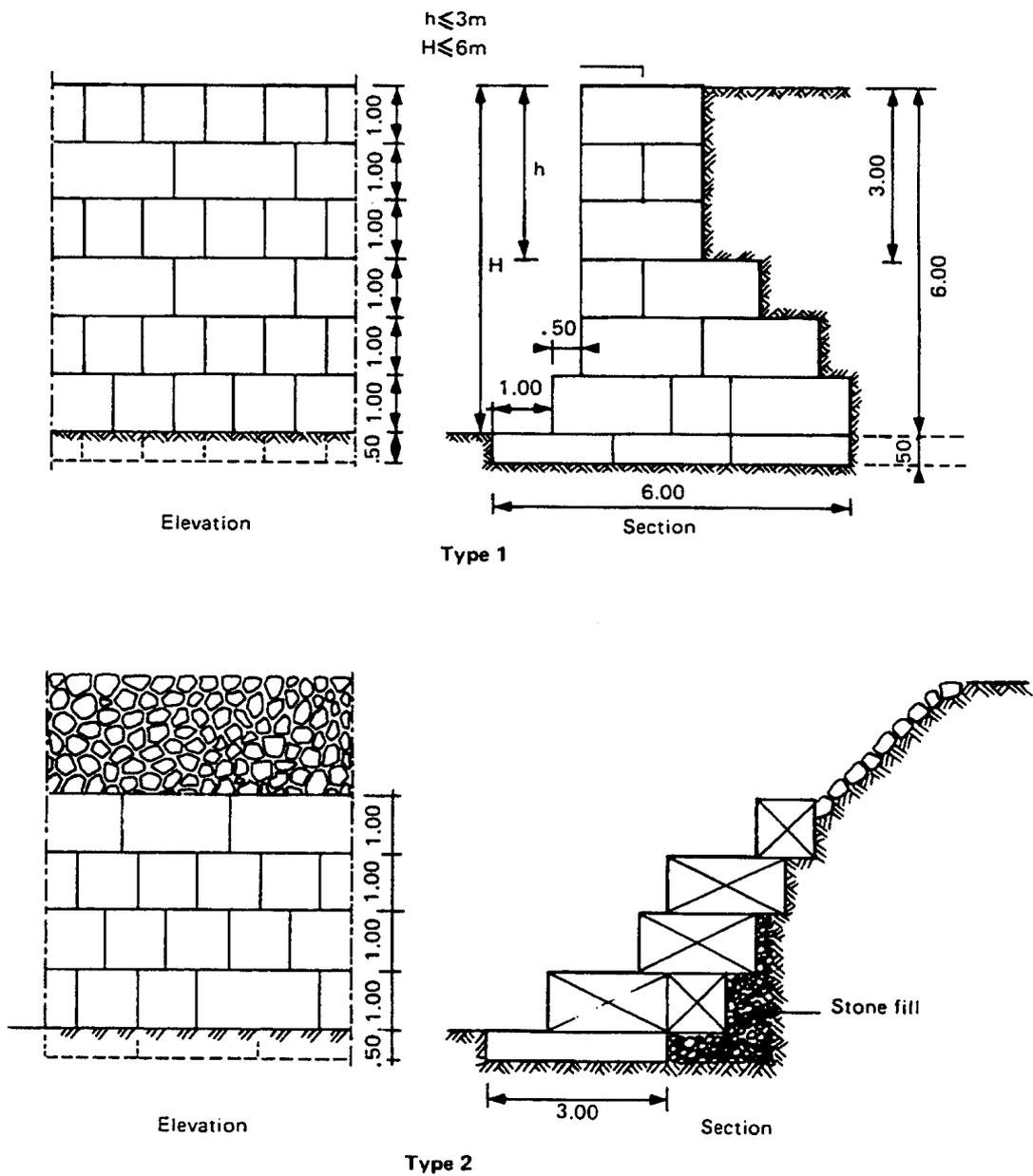


Figure 6.8 Gabion walls

The Reno mattress is a mattress-shaped version of the gabion, for use where the latter's height and bulk are not needed, as in lining some river banks and beds. The standard 6m length of mattress is divided into six 1m compartments or into as many as ten compartments on special order as illustrated in Figure 6.4. After assembly on site, the mattresses are wired together and filled to form a continuous blanket or lining.

Details of the construction and filling of gabions and Reno mattresses are given in ILO (1986) which also contains a list of sources and material specifications.

Table 6.2 indicates minimum mattress thicknesses for a range of bed soil types and water flow speeds.

Table 6.2 Minimum Reno mattress thicknesses

Bed soil type	Maximum water velocity (m/sec)	Minimum mattress thickness (mm)
(a) Clays and heavy cohesive soils	2	170
	3	230
	4.5	300
(b) Silts and fine sands	2	230
	3	300
(c) Shingle with gravel	3.5	170
	5	230
	6	600

6.1.3 Filter blankets

If the river bank is a fine, non-cohesive material, it may be washed away through the voids in rip rap or gabion lining. This can be prevented by using a textile filter, or retarded by means of a gravel filter layer half as thick as the rip rap layer with a grading conforming to the following calculation:

$$\frac{D_{50} \text{ filter}}{D_{50} \text{ base}} < 40$$

$$5 < \frac{D_{15} \text{ filter}}{D_{15} \text{ base}} < 40$$

$$\frac{D_{15} \text{ filter}}{D_{85} \text{ base}} < 5$$

where D_{15} , D_{50} and D_{85} are stone sizes such that 15 per cent, 50 per cent and 85 per cent respectively of the stones are smaller by weight. 'Base' refers to the river bed material.

6.1.4 Piled walls

Timber, steel or concrete piles may be driven to form a continuous wall, with the aim of either providing protection against erosion, or channelling the main

flow through the bridge (Figure 6.5). Piled walls are less flexible than gabions or rip rap and may fail under small amounts of movement. The engineer has to make sure that the piles are driven deep enough to withstand the general and local scour, and still retain sufficient stability to withstand earth pressure on one side when the water level is low on the other.

If timber is to be used, local knowledge of durability under wet and dry conditions will help selection. The shaping of the timber piles shown in Figure 6.5 contributes to achieving a tight wall.

Purpose-made steel piles interlock and so make a stronger wall than lengths of W section guardrail, used railway sleepers or other improvised sections.

6.1.5 Vegetation

Many plant types can be used to protect river banks against erosion. The most successful ones are almost always those found growing naturally along the river being bridged. Protection from annual floods in the form of gabion groynes (6.2.4), tethering to strong stakes or rip rap, may help tender plants to become established.

6.1.6 Slab concrete and stone pitching

Where there is an erosive stream, such as a drainage ditch joining the river at a bridge, slab concrete or stones set in cement mortar may be required to armour the river bank at the point of entry.

6.2 Protection methods

6.2.1 Pier and abutment foundations

Mass concrete foundations set on erodible soils need protection against local scour, though they should be unaffected by general scour if the size of the waterway is adequate. Figure 6.6 indicates the recommended position for rip rap. The top of the protective layer must not protrude above the lowest general scour level, as determined by the calculations outlined in Chapter 5. The engineer may select a suitable stone size from Figure 6.1, remembering that local velocities can be as high as 1.5 times the mean calculated velocity through the waterway opening.

If large enough stone for rip rap is too expensive, gabions or Reno mattresses may be used. The appropriate mattress thickness can be read from Table 6.2.

6.2.2 Bank protection

The same guidelines apply to rip rap stone size for bank protection as for substructure foundations (Figure 6.1). The local velocity against the slope of a bank aligned more or less parallel to the flow may be

taken as 0.7 times mean flood velocity where the channel is straight. The local velocity at the outer bank on a severe bend may be as high as 1.5 times mean velocity.

Reno mattresses, laid with their longest dimension up the bank, provide effective protection against bank erosion and scour in a similar manner to rip rap. Both methods may be used with a launching apron, as shown in Figure 6.7. Apron length l_a should be 1.5 times the total scour depth DS . On most river banks, grasses and shrubs grow up between the stones of rip rap and mattresses, their roots protected by the revetment which they in turn help to stabilise.

Steep banks are better protected by driven pile walls or gabion walls with the front face vertical or raked back at a slope of about 1 in 10 (Figure 6.8 type 1).

6.2.3 Guide banks

On meandering and braided rivers, whose widths are greater than necessary to take the flood discharge, guide banks can be used:

- to confine the flow to a single channel
- to control the angle of attack on piers and prevent meanders encroaching on and eroding the approach roads.

A single guide bank may be sufficient when the stream can be confined to one side of a valley possessing a natural inerodible bank (Figure 6.9). Two guide banks are necessary when the waterway opening is in the middle of a flood plain.

The minimum width between guide banks is the width necessary to provide the required waterway opening through the bridge. A greater width may be preferable because it is more difficult and expensive to place an earth embankment in flowing water than to construct it on dry land or in still water. The guide banks should extend upstream by about three quarters of the waterway opening and downstream by about one quarter, unless a longer length is necessary to prevent an upstream meander working behind the guide bank and breaching the approach embankment (Figure 6.10).

The plan shape of the guide bank often takes the form of a quarter section of an ellipse with a ratio of major to minor axis of 2.5 to 1, the major axis being parallel to the principal diversion of flood flow through the opening (Figure 6.11).

Guide banks should be protected by revetments and by launching aprons, as shown in Figure 6.7. The top of the bank should be wide enough for maintenance vehicles and the ends of the banks should be rounded (Figure 6.11). They should extend above design HEL with a suitable freeboard allowance.

6.2.4 Groynes

Groynes can take the form of permeable or impermeable walls and may be constructed using piles, gabions, rip rap or mass concrete. The purpose of a groyne, or series of groynes, is to divert the river flow, which is also influenced by the other features of the river bed. It follows that the length, height and location of the groynes depends not only on the required path of the river in flood but also on the river banks, bed material and any other obstructions in the vicinity, including the substructures of the bridge.

Groynes should never impinge on the area of general scour. Nonetheless, they may affect the pattern of local scour at the bridge by diverting the current and may generate local scour holes, depending on the angle between the groyne and the water current. Figure 6.12 shows some typical scour effects of river groynes, but local features and experience drawn from other structures on the same river will provide the best guidance.

6.3 References

IL O (1986). *Training element and technical guide for SPWP workers. Booklet No 3.* International Labour Office, Geneva.

Farraday and Charlton (1983). *Hydraulic factors in bridge design.* Hydraulic Research Station Ltd., Wallingford.

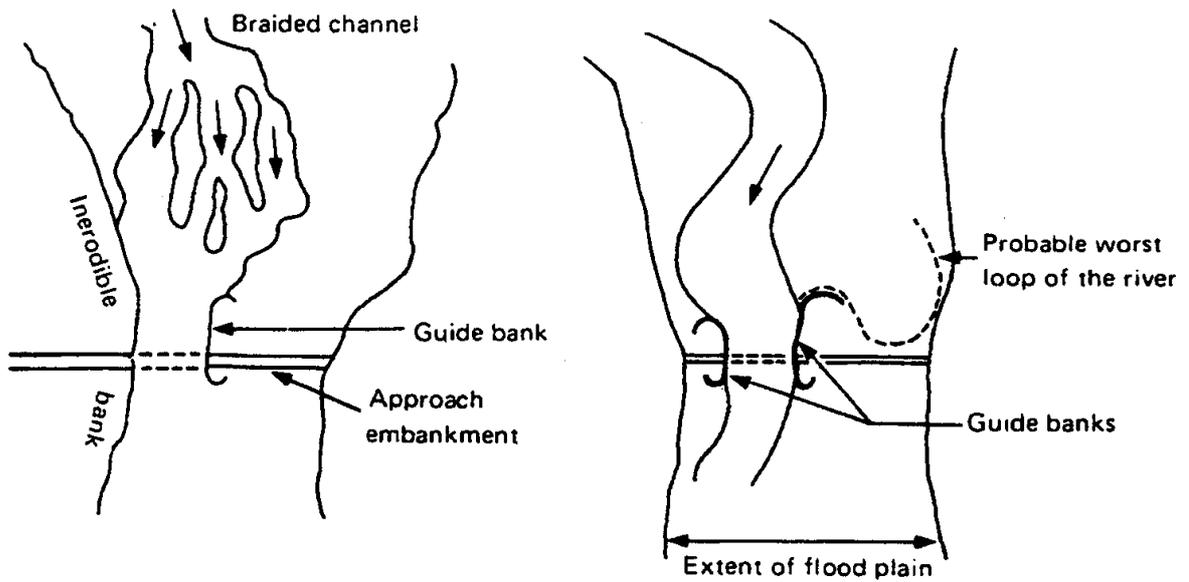
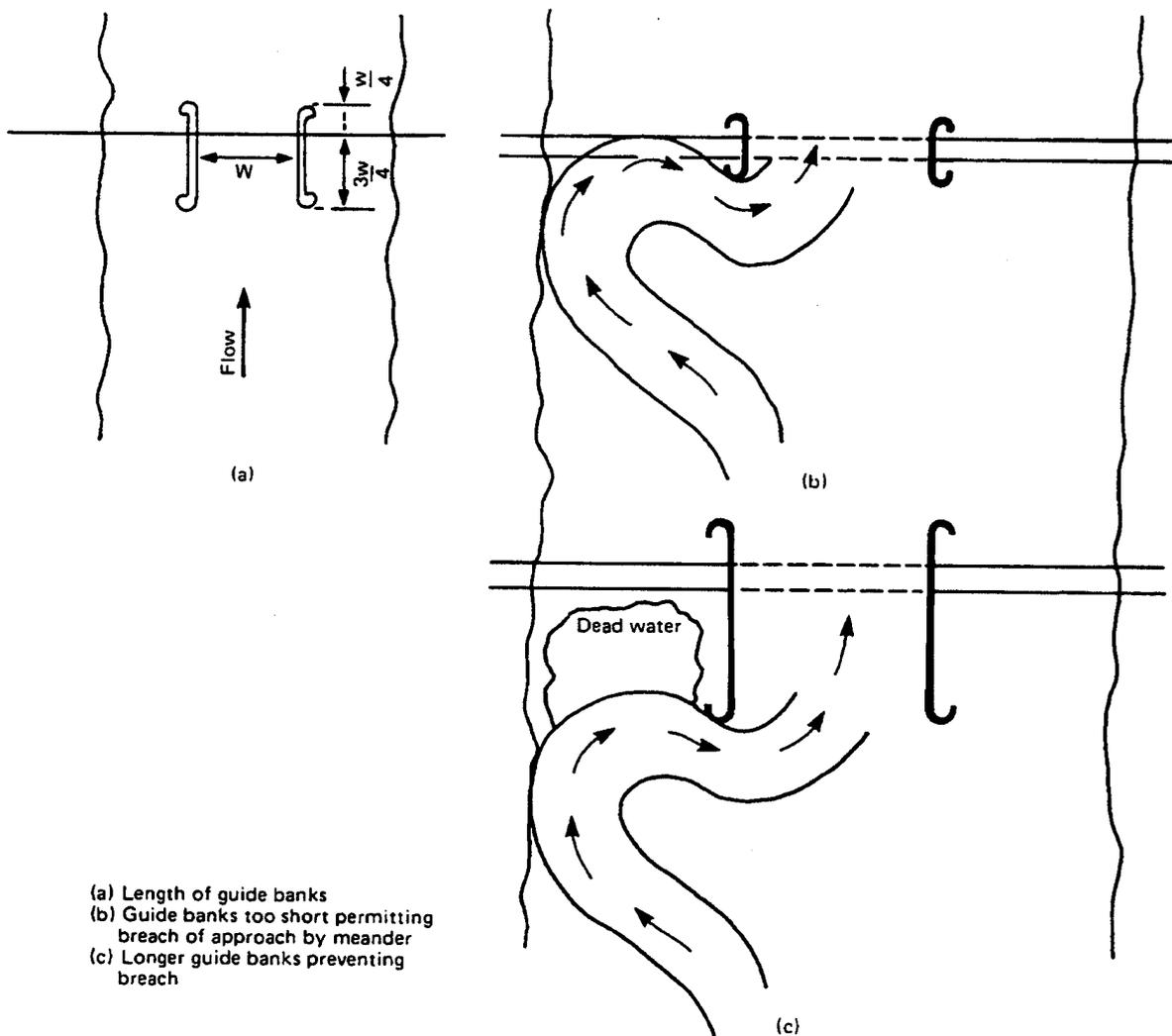


Figure 6.9 Single and twin guide banks



- (a) Length of guide banks
- (b) Guide banks too short permitting breach of approach by meander
- (c) Longer guide banks preventing breach

Figure 6.10 Guide banks – general principles

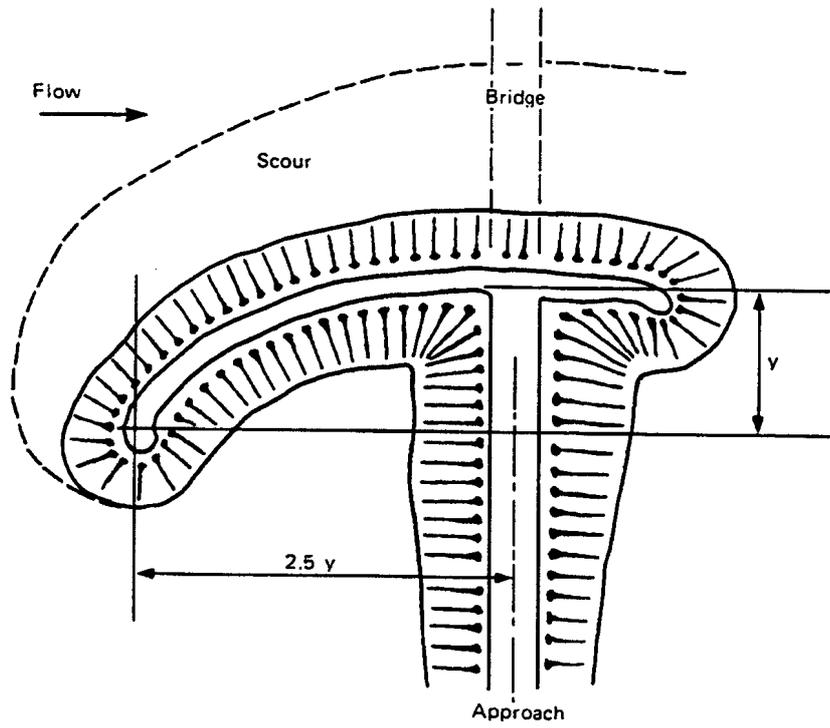


Figure 6.11 Guide bank profile

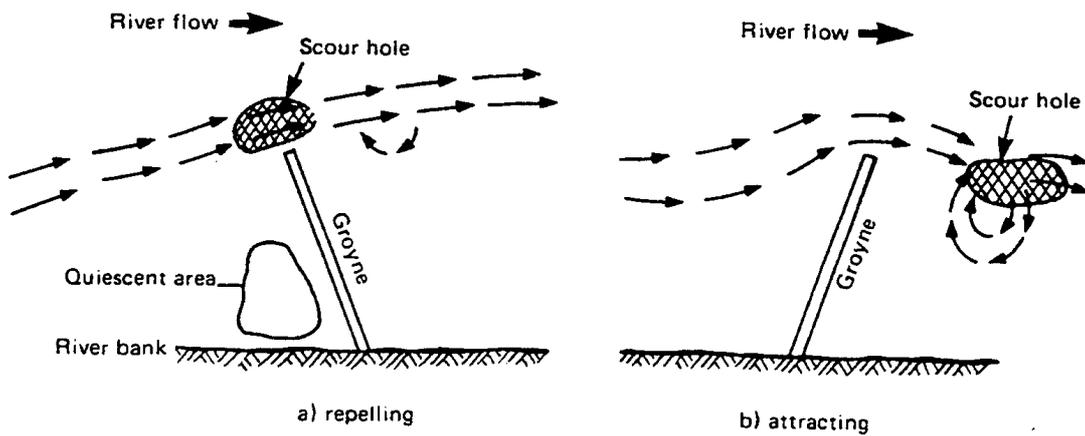


Figure 6.12 Typical effects of groynes on river flow

7 LOW LEVEL WATER CROSSING

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7 Low level water crossing

In favourable conditions, low level water crossings can provide economical and relatively simple alternatives to conventional bridges. There are two basic types of low level crossing:

- fords and bed-level causeways;
- vented causeways and submersible bridges.

Both types are appropriate for sites where traffic volumes are low or where a reasonably short detour provides access to an all-weather bridge. For most of the year the maximum depth of water over the carriageway should be less than 150mm.

The success or service life of the structure will depend considerably on its hydraulic design as outlined in Chapter 5. Fords and bed-level causeways, like conventional bridges, may be constructed so that they cause little interference with the design flood. Vented causeways and submersible bridges inevitably disrupt river flow, and so are liable to sustain damage or indirectly cause scour to the river bed or banks, which in turn may affect the road approaches to the crossing.

Since fords and bed-level causeways are overtopped by any water flowing in the river channel there is no reason to raise the road surface above the stream bed. Vented causeways and submersible bridges usually present a dry carriageway for ordinary flows and are overtopped during the design flood.

7.1 Road approaches

The best location for a low level crossing is similar to that recommended for a conventional bridge (Section 2.1.2), with the exception that a broad, straight stretch of river will provide slower and shallower water as well as easier road approaches.

The maximum gradient for motor traffic should be 10 per cent, and for animals 5 per cent. The change of gradient should be gradual to prevent the underside of vehicles touching the road and to preserve long sight distances. A small change in horizontal alignment of the road at the crossing helps to draw the attention of drivers to a dip that may conceal an obstacle.

The equal cut and fill construction of the approach roads shown in Figure 7.1a requires less work than the cut and remove spoil of Figure 7.1b, but the placing of the spoil in the river channel may cause scour problems during a flood. On roads where speeds are naturally slow and where the horizontal curve on the approach side is clearly visible, steep approaches may be relieved by a diagonal descent of the river bank, as shown in Figure 7.2.

Even where the road is a single track, it is advisable to make the crossing and its approaches two lanes in width, so as to allow traffic to pass any vehicle which breaks down or falls to mount the gradient.

7.2 Fords

Fords are the simplest form of river crossing at places where the stream is wide, shallow and slow, the approaches gentle, and the surface firm. Improvements to the approaches are usually confined to reducing the gradient. The running surface in the

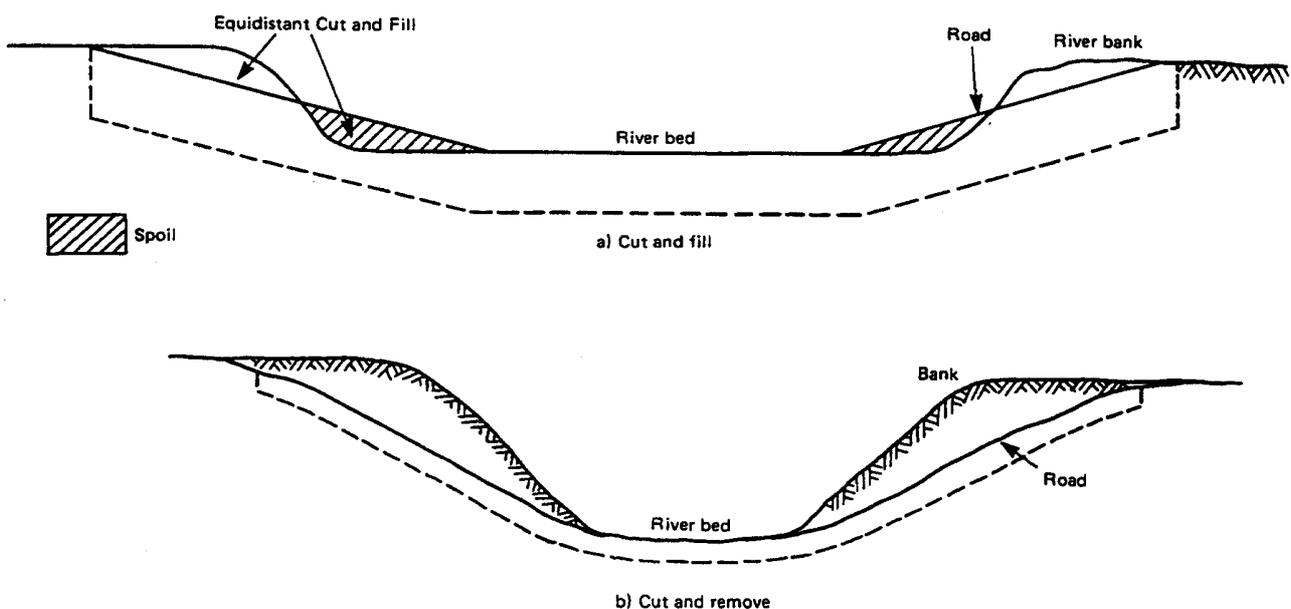


Figure 7.1 Sections at a ford showing alternative road approaches

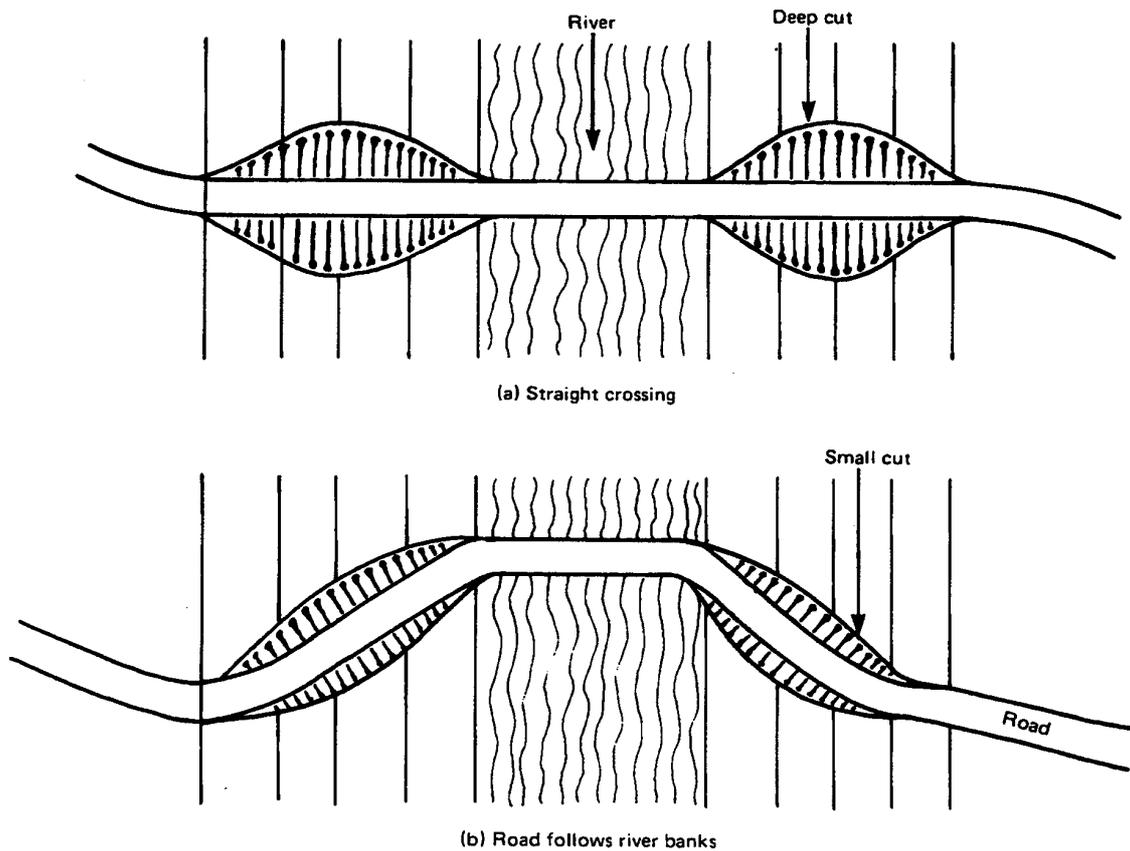


Figure 7.2 Plan views showing steep and shallower approaches to a ford

stream can be strengthened and made more even using stones brought in and buried just below the surface. Alternatively, if stones are carried in the flow, these may be trapped by barriers made of boulders, gabions or piles; see Bingham (1979), Hindson (1983), Metschies (1978), Leydecker (1973).

7.2.1 Boulders

Large stones placed across the river bed at the downstream side of the crossing are claimed to filter the flow of water and retain gravel and sand, which eventually form a more level and even surface for vehicles. However, if the stones are too large or form too high a wall (>300mm), scour may result; if they are not heavy enough, they will be washed away at the first flood. Figure 7.3a shows a typical cross section of a ford with downstream boulders.

7.2.2 Gabions

A more durable improvement may be made to the running surface by replacing the boulders with gabions as shown in Figure 7.3b. It is important that the gabions protrude no more than 150-300mm above the natural bed level of the river, otherwise they will act as a weir and cause heavy scour downstream of the crossing.

The gabion baskets are wired together and dropped into a prepared trench. The central gabion is filled first and, using that as an anchor, the line of gabions is pulled taut and straightened by a chain attached to a truck or winch. This tension is maintained while the remaining baskets are filled. When filling is complete, final adjustments are made to the top course of rock and the baskets are closed. Section 6.1.2 provides a more detailed account of the use of gabions.

As an alternative to conventional gabions, a tube basket can be made from a roll of fencing mesh laid across the bed of the stream and filled with rock. During filling, the edges are raised and bent over at the top to form a tube and then tied. Finally a wire rope is attached as shown in Figure 7.3c and securely anchored at each end. Tube baskets need to be installed in a previously excavated trench approximately half the depth of the basket, ie 0.2 to 0.3m. After installation, sand and gravel transported by the stream is trapped behind the basket, building up a fairly level surface suitable for the passage of vehicles.

7.2.3 Piles

Where gabions are unobtainable, timber piles driven into the river bed with or without stone protection make an acceptable cut-off wall (Figure 7.3d).

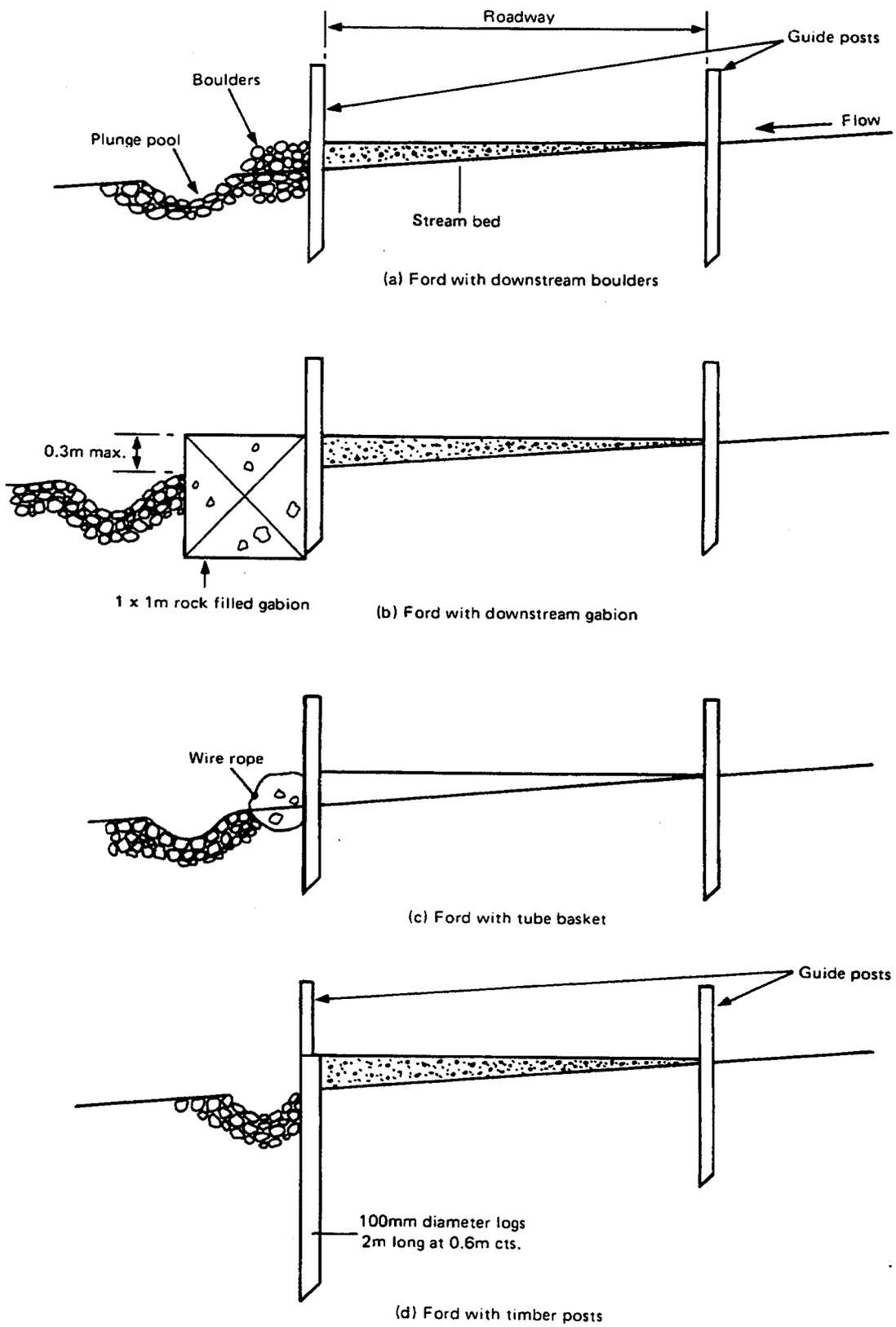


Figure 7.3 Ways of improving the running surface at a ford

To be fully effective, timber piles should have a length of about 2m, and should be placed at no more than 0.6m centres. If the river is fast flowing a continuous line of piles may be necessary.

A curtain wall may be required on the upstream as well as the downstream side if the road bed is erodible.

7.3 Bed level causeways

Where the type of traffic or the lack of a nearby all-weather crossing justifies the expense, a pavement may be laid on the river bed. A paved ford is also called a bed-level causeway, drift, paved dip or Irish bridge (Roberts 1986). Figure 7.4 illustrates three common designs. Masonry can also be used instead of concrete: see Chapter 15 for more details.

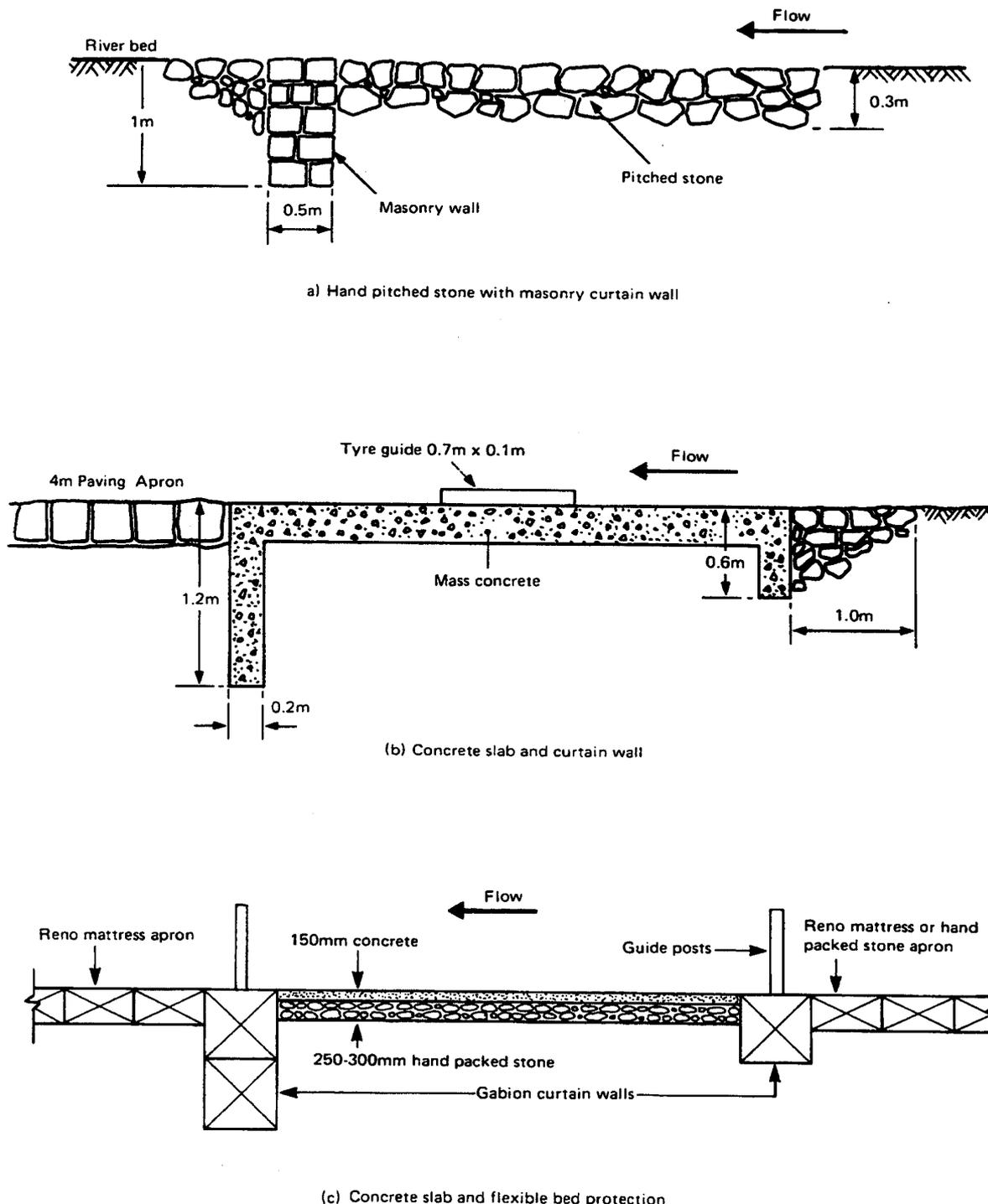


Figure 7.4 Bed level causeways

To protect the pavement from scour damage, curtain walls are usually required on both sides of the roadway and these must continue up the approaches to the height of the design flood. It is recommended that curtain walls should be 1m deep on the upstream side and 2m deep on the downstream side, unless rock is reached before that depth.

If the bed is not susceptible to erosion, the causeway need not have curtain walls but the bed on both the upstream and downstream sides of the crossing should be trimmed flat to reduce turbulence.

Figure 7.4a shows a Section through a basic bed level causeway suitable for light traffic and maximum water flows below 2m/sec. The crossing shown in Figure 7.4b requires good concrete technology and may sustain damage to the apron that is difficult to repair. Figure 7.4c shows a design employing a good combination of concrete pavement with flexible protection.

Generally, a 1:2:4 concrete, by volume, is used and slabs are jointed using crack inducers every 5m.

7.4 Vented causeways

Vented causeways are built where the river flow is too great for too many days in the year to allow the traffic to cross a ford or bed level causeway. Structures like the one depicted in Figure 7.5 are, for hydraulic purposes, multiple pipe culverts and should be designed following the guidelines in Chapter 5 and Chapter 12. However, the design flood used to calculate the vent sizes will be less than the annual flood, provided it is acceptable that the carriageway

may be overtopped for a few days each year during the annual high flood.

Since these structures present a considerable obstacle to the free flow of both an ordinary flood and the design flood, they must be built sufficiently massive to withstand water pressure and debris impact. They must also have scour protection (Chapter 6), where the bed is erodible, and marker posts.

The vents are usually concrete or corrugated steel pipes from 0.6m to 1.0m diameter, set in a block of concrete or masonry. Where pre-cast concrete pipes are unavailable, vaulted masonry tunnels have proved successful. Concrete or masonry retaining walls and aprons are needed to channel the flow and prevent scour at both entrance and exit.

To prevent blockage of the stream by debris or silting, the pipes must be set level with the stream bed and at the same slope. No part of the vents should be narrower than the entrances. Wedge-shaped deflector walls may be required on the upstream side to guide large floating debris above the vents.

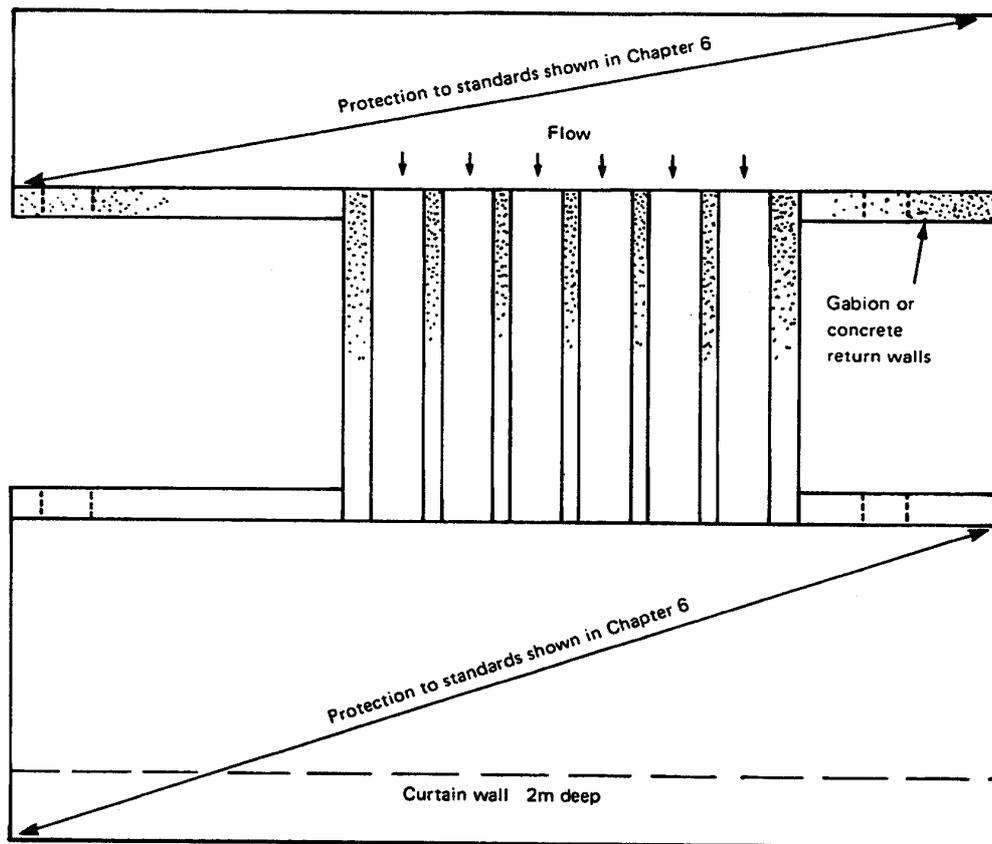
Figure 7.6 shows a typical section and elevation of a vented causeway. Figure 7.7 gives dimensions for the concrete cover and reinforcement details.

7.5 Submersible bridges

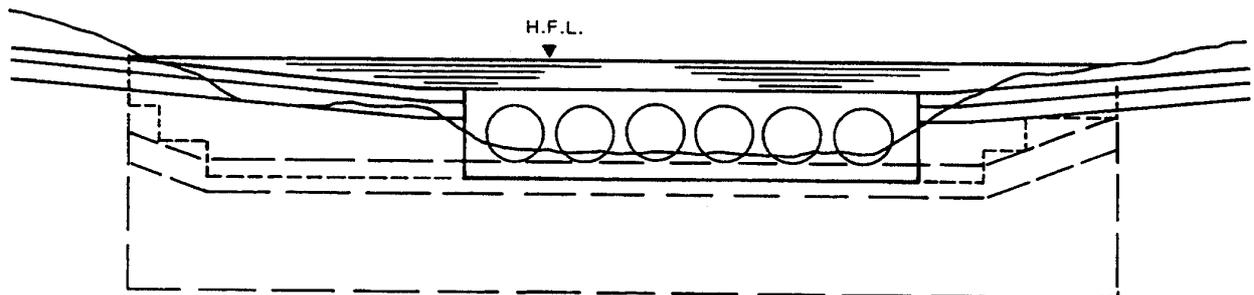
Where the traffic density justifies a dry crossing of a substantial ordinary flood and the annual high flood is much greater, a submersible bridge designed to pass the ordinary flood but to be overtopped by the high



Figure 7.5 Vented causeway

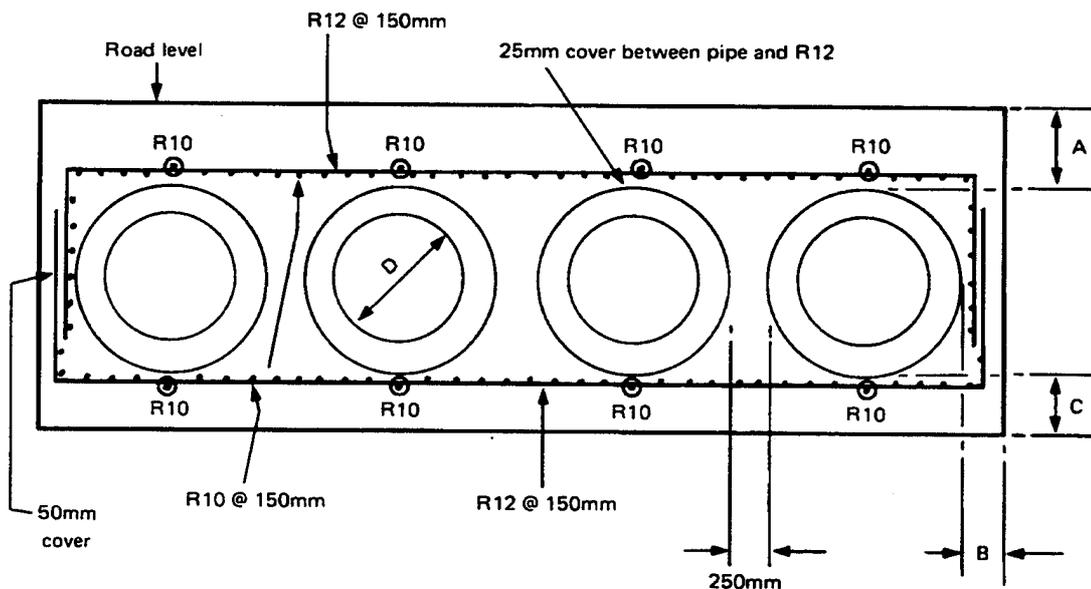


(a) Horizontal section



(b) Elevation

Figure 7.6 Vented causeway – horizontal section and elevation



Concrete strength 30 N/mm ² (28 day cube)			
Dimensions (mm)			
D	A	B	C
600	250	175	150
750	250	175	150
900	275	200	175
1000	300	225	200

Figure 7.7 Vented caueway - vertical section

flood will be considerably cheaper than a high level, all weather bridge. Submersible bridges have the advantage of being able to pass a larger flow than the vents of a causeway of the same height, but are more susceptible to damage by the river. The overturning moment at the pier foundations becomes very large unless the piers are kept short, and the horizontal and vertical forces on the decks require solid restraint.

Because of these difficulties submersible bridges are not recommended for any foundation other than rock, and even then a vented causeway or conventional bridge is likely to be a more durable alternative. However, submersible bridges have served well in some areas, and should the designer wish to adapt for this purpose the shorter standard concrete deck drawings included in Chapter 9, the only major modification required is to replace the standard fixed and free bearings by monolithic concrete with continuous reinforcement. Spans greater than 6m are unlikely to be economical.

7.6 Signs and markers

Like all other water crossings, low level crossings should be well signed in advance. Signs should give

warning of the change in vertical and horizontal alignment, and indicate the possibility of encountering water on the carriageway.

In addition to advance warnings, there should be a depth gauge at each end of the crossing and guide posts on both sides for the whole length.

7.6.1 Depth gauges

Depth gauges should indicate the depth of water at the lowest point of the crossing. Simple black and white markings are best with an indication of the units used. Posts should be about 300mm diameter or square, placed within easy vision of the approach but well away from possible impact damage by vehicles.

7.6.2 Guideposts

Guide posts should be set each side of the carriageway between 2m and 4m apart, depending on the likelihood of catching floating debris. There are two opinions about their height:

- They should be visible only when the water is shallow enough for vehicles to cross.

- b) They should be high enough to be visible during the highest expected floods. This is the preferred method where depth gauges are well maintained, as it permits drivers to exploit the capabilities of different types of vehicle.

The posts should be of durable timber, metal or concrete, depending on the materials used for the carriageway surface and the curtain walls, eg using concrete posts on concrete bases and timber posts set into gabions.

An additional guide for vehicles may be provided by building a ridge down the centre of concrete causeways, as show Figure 7.4b. This ridge also offers restraint against sideways drifting of vehicles in strong currents.

7.7 Designing for maintenance

Submersible crossings of all types require more maintenance than most conventional bridges. The structural design should allow for easy repair of routine damage, which is usually due to scour. On highly erodible beds it is often more successful to build gabion curtain walls and use Reno mattress aprons than to build a rigid concrete structure, recognising that some rebuilding will be required

each year. Figure 7.8 from BCEOM (1975) illustrates the problem.

7.8 References

BCEOM (1975). *Manuel d'exécution de petits ouvrages routiers en Afrique.* Ministère de la Cooperation, Pans.

Bingliam J (1979). *Low water crossings.* Compendium 4. Transportation Research Board, Washington.

Hindson J (1983). *Earth Roads - A practical guide to earth road construction and maintenance.* Intermediate Technology Publications, London.

Leydecker A (1973). *Use of gabions for low water crossings on primitive or secondary roads.* Engineering Technical Information Service, Field Notes 5, No.6. Washington: US Forest Service.

Metschies G (1978). *Manual on road construction. Economic and Social Commission for Asia and the Pacific, Bangladesh.*

Roberts P (1986). *The Irish Bridge - a low cost river crossing.* University of Southampton, Department of Civil Engineering, Southampton.

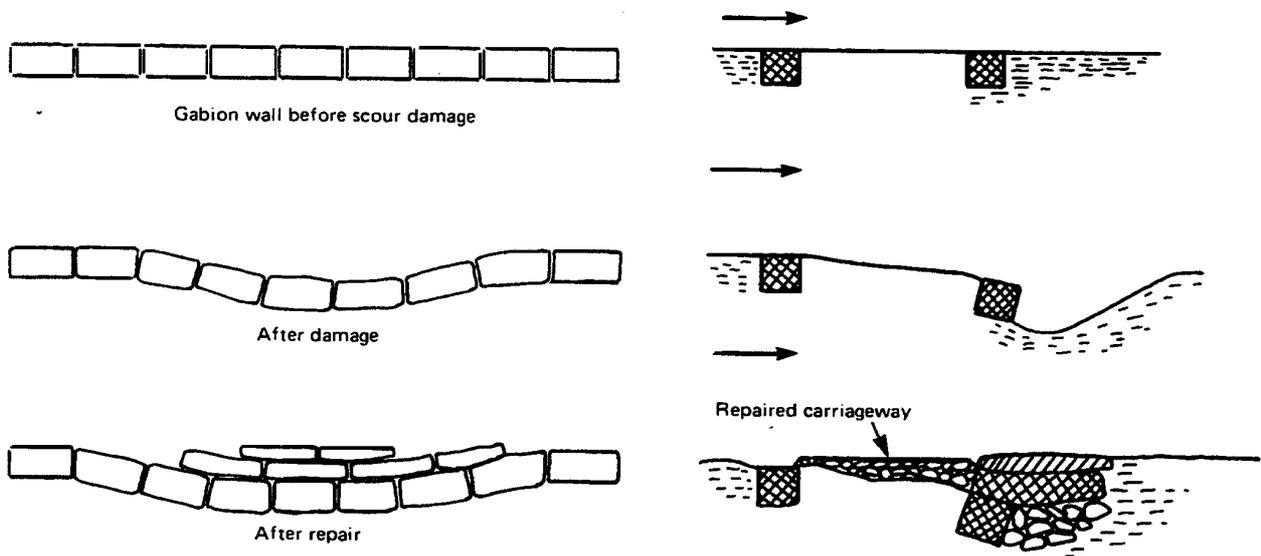


Figure 7.8 Repair to flexible curtain walls at a ford

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8 Substructures and foundations

This chapter deals with the elements of a bridge that support the superstructure, ie abutments, piers and their foundations. Figure 8.1 shows some typical shapes of substructures in concrete. To assist the highway engineer by removing the necessity for complex individual calculations, the chapter includes a set of standard PC concrete abutment and pier drawings, with tables specifying dimensions, reinforcement and other details, for spans up to 12m and a range of soil conditions. The designs are conservative and take account of all the external forces from live and dead loads that substructures are normally expected to sustain. Should the engineer prefer to work from first principles or modify any of the drawings presented here, reference to Victor (1980), Pennells (1978) and Tomlinson (1986) will provide procedures and formulae.

If experience is available in structural masonry and this material is an economical alternative, it may be used as a substitute for the mass concrete detailed here. However, the engineer must be satisfied about the strength of the materials used, particularly when they are submerged in flowing water. The mass concrete abutments and retaining walls detailed in Figures 8.11 and 8.13 are conservatively designed but any changes, such as replacing the concrete stem with masonry, should be carefully evaluated, taking into account all the forces and reactions from the superstructure, the embankments and the soil under the foundations. A reinforced concrete bearing shelf for use with masonry abutments is illustrated in Figures 8.10 and 8.11, and detailed in Figures 8.20 and 8.21.

Timber abutments or gabion abutments may be used

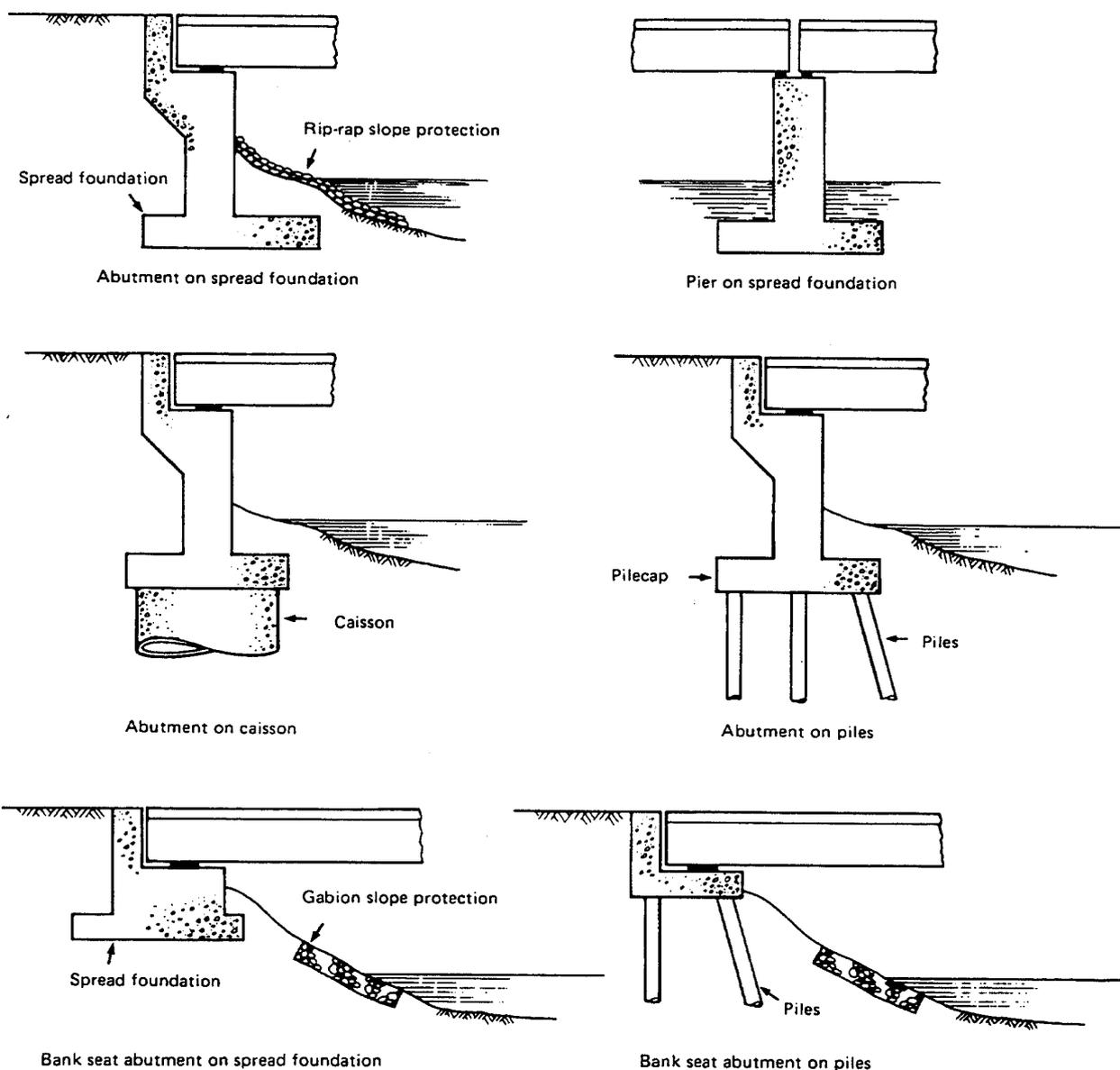


Figure 8.1 Typical substructures

for temporary structures but are not recommended for permanent bridges, even those with timber superstructures. Reference to Section 13.3 will provide details of timber and gabion abutments.

Concrete abutments and piers as described here may be built to support a timber superstructure that will be replaced at a later date with a more permanent material. In this case the final dead weight and width, etc., must be used in the abutment design. All concrete decks must have rigid substructures like those detailed in this chapter, because uneven settlement of either abutment or pier can result in unacceptably high stresses in the materials of the decks.

The positioning of the abutment and pier foundations is critically important: the alignment of the bridge and its interaction with the river are discussed extensively in Chapters 2, 4, 5 and 6. Since the most likely cause of substructure failure is scour, the engineer should pay careful attention to the estimation of general and local scour described in Chapter 5. Pier foundation depths are specified according to foundation type and protection method (Figure 8.2). There are two key points to bear in mind:

- the general scour area must not be obstructed or the flow will be impeded and further scour damage will result;
- local scour is caused by turbulence and may be contained by armouring the bed.

Abutments also fail when the soil under the foundation is not strong enough to counter the combined forces from the structure and the embankment. It is recommended that spread foundations be used wherever possible, but if adequate support as defined in Section 8.3 is unavailable, a piled foundation is required. Piling techniques and materials vary from region to region. Tomlinson (1977) is a good general guide on this subject. Piling for permanent bridges is a specialist technique that should be entrusted only to experienced contractors.

Where a satisfactorily strong foundation soil is found not too far below preferred foundation level, caisson support may be considered. The technique is simple if the caissons are short, but the engineer must take care that:

- the maximum soil reaction at the sides does not exceed the maximum passive pressure at any depth
- the soil pressure at the base remains compressive throughout and the maximum pressure does not exceed the allowable pressure as defined in Chapter 3.

Victor (1980) contains guidance on the use of both piles and caissons. The drawings of spread foundations included in this chapter need to be amended if piles or caissons are employed.

8.1 Abutments

In addition to supporting the dead load of the superstructure, the abutments of a bridge must:

- resist the vertical and horizontal live loads placed on them by vehicles and the elements;
- retain the approach embankments and the live loads applied to them;
- provide a smooth transition from the road surface to the deck running surface.

Figure 8.3 indicates the main elements of abutments. Their essential features are:

- a foundation slab, which transmits the weight of the abutment and a proportion of the superstructure directly to the supporting soil, or which forms a capping slab to a system of load-bearing piles;
- a front wall with bearing shelf that supports the superstructure and usually retains the soil of the embankment;
- wing walls or retaining walls may be separate from the abutments or, if they are short, may be built integrally with them. These walls retain the road embankment or river bank adjacent to the abutment and are usually built so as to bisect the angle between the road and the river bank, though they can be set at any angle to the abutments and may be built parallel to the road or perpendicular to it.

8.2 Piers

Piers also perform a support function. They convey vertical and horizontal loads from the superstructure via the bearing shelf, stem and foundation slab to the supporting soil. In many instances, piers stand on saturated soils for most or all of the year: they do not retain soil embankments but are designed to withstand hydraulic pressures and impact loads.

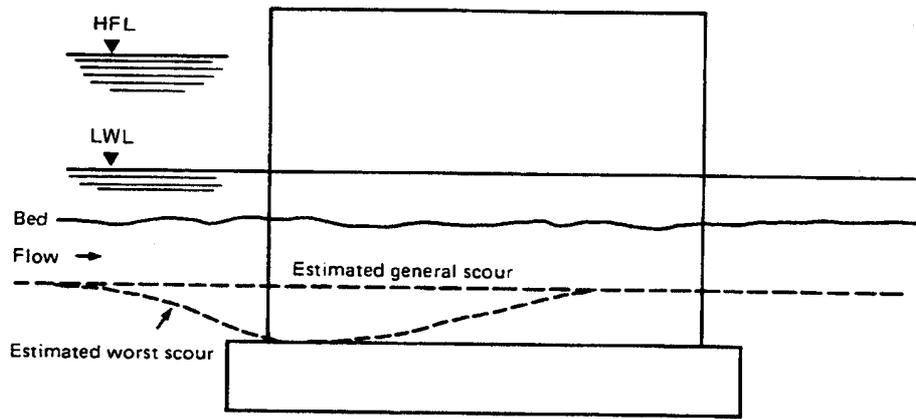
Piers are often more susceptible to scour damage than abutments and need to be orientated carefully with respect to flow direction. Their foundations should be located well below maximum scour depth.

8.3 Determination of allowable bearing Pressure

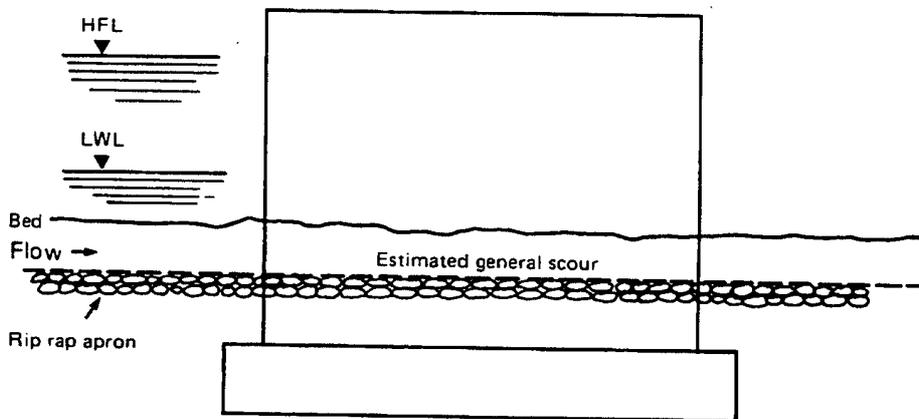
8.3.1 Non-cohesive soils

The allowable bearing pressure under foundations in non-cohesive soils is governed by the permissible settlement of the structure due to consolidation of the soils under the applied loading.

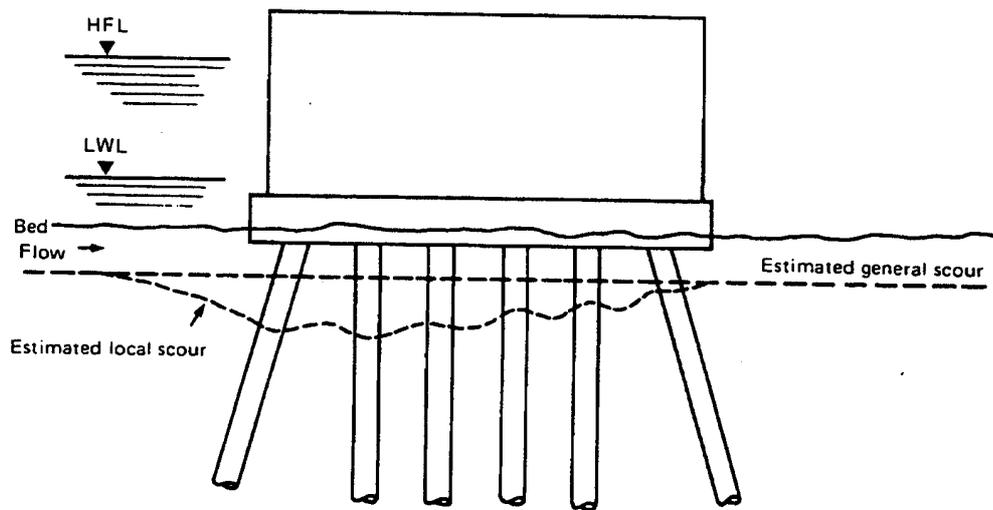
If standard penetration tests have been performed in boreholes (Section 3A.1.3) the values of N can be



(a) Bottom of foundation below estimated lowest scour level



(b) Foundation below general scour level and protected against local scour by rip rap apron



(c) Pier supported on piles designed to be exposed

Figure 8.2 Pier foundations in relation to scour

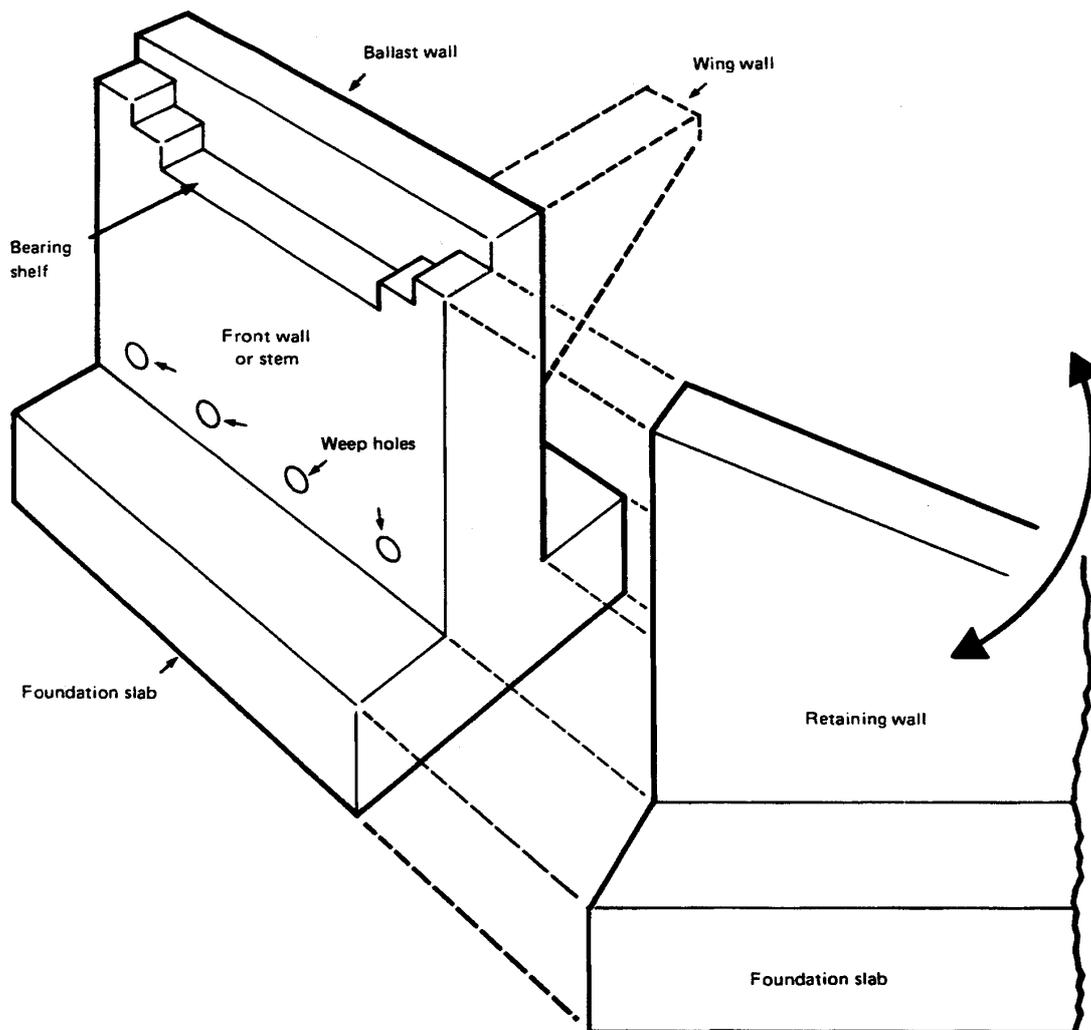


Figure 8.3 Main elements of an abutment

used to obtain allowable bearing pressures for various foundation dimensions (Figure 8.4).

The allowable bearing pressure in this context is that which causes 25mm of settlement under the given breadth of foundation front to back, B_r , ie measured perpendicular to the direction of water flow, on the assumption that the water table always remains at a depth of at least B_r below foundation level. If the water table can be higher than this, the allowable pressures should be halved.

Before applying the relationships shown in Figure 8.4, the engineer may need to correct the standard penetration values measured in the boreholes, since it has been found that the test seriously underestimates the relative densities of cohesionless soils at shallow depths. To allow for this, a correction factor, obtained from Figure 8.5, should be applied to the measured values. Appendix B.1 provides an example of the calculation procedure.

Where the N-value of a fine or silty sand below the water table is greater than 15, the density of the soil should be assumed to be equal to that of a sand having the N-value of $15 + \frac{1}{2}(N-15)$.

Very loose uniformly graded sands with N equal to 5 or less and subject to rapid changes of water level are liable to suffer large settlements under load. In these circumstances, either the sand should be dug out and thoroughly recompacted or the foundation should be supported on piles.

In areas that experience seismic activity, foundations must not bear on single size material.

8.3.2 Cohesive soils

Most cohesive soils at foundation level *are* saturated and have an angle of shearing resistance equal to zero, provided that no water is expelled from the soil as the load is applied. This is accepted as the basis for

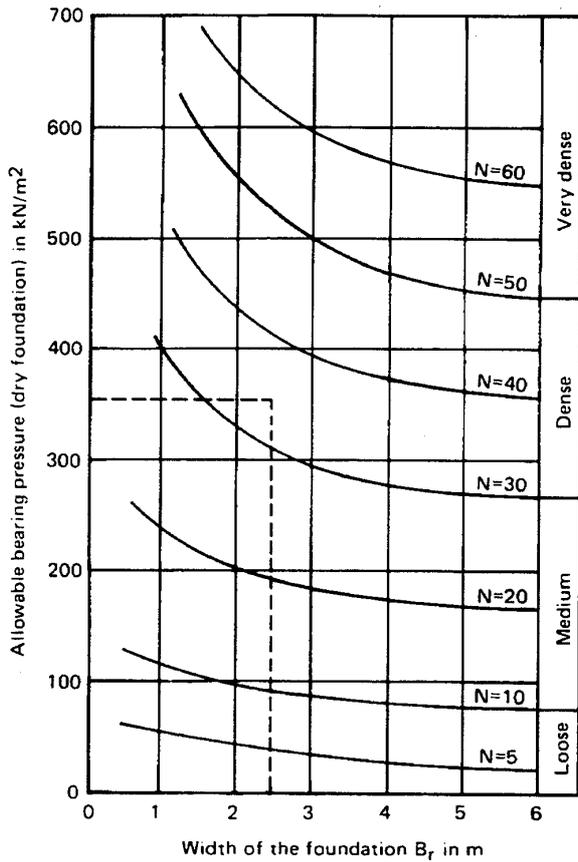


Figure 8.4 Allowable bearing pressure/penetration value relationships

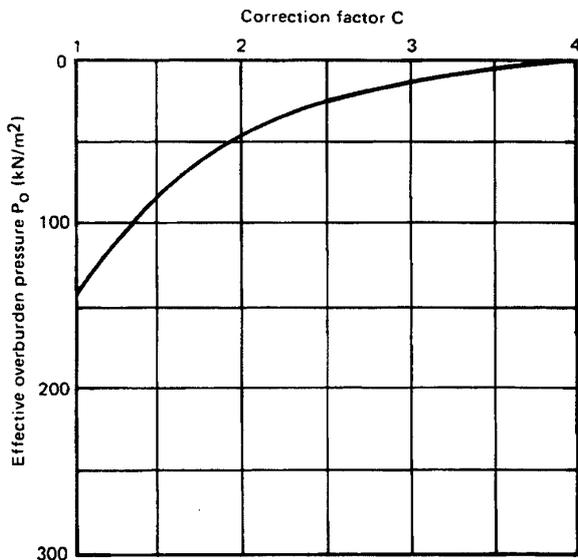


Figure 8.5 Correction factors for cohesionless soils at shallow depths

calculating the ultimate bearing capacity of foundations where the load is applied relatively quickly.

The ultimate bearing capacity of cohesive soils can be calculated from the following formula:

$$\text{Ultimate bearing capacity } q_f = c_u N_c + p \quad (8.1)$$

Where

C_u = undrained shear strength (kN/m^2) (section 3.4)

N_c = bearing capacity factor

P = total overburden pressure at Foundation level (kN/m^2)

$P = \gamma \cdot D$

where

g = density of soil above foundation level (kN/m^3) (submerged density where below water level)

D = depth of foundation level below ground surface (m).

Values of the bearing capacity factor N_c for square or circular foundations can be read from the graph in Figure 8.6.

For rectangular foundations

$$N_{c \text{ rectangle}} = \left(0.84 + 0.16 \frac{B_f}{L} \right) \times N_{c \text{ square}} \quad (8.2)$$

Where: B_f = breadth of foundation front to back

L = length of foundation

The undrained shear strength, c_u of soft clays can be measured by means of field vane tests, but these results need to be corrected because the soil is sheared in a horizontal direction. The value of c_u , to shear strength multiplied by the correction factor read from the graph in Figure 8.7. This factor is dependent on the plasticity index of the soil.

The **allowable** bearing pressure is one-third of the calculated **ultimate** bearing capacity.

8.3.3 Plate bearing tests

Ultimate bearing capacity values from plate bearing tests may be used to check the results calculated on the basis of Section 8.3.1 or 8.3.2. However, these tests are carried out with relatively small plates and stress only the soil immediately below the plate. Since pier and abutment foundations cover a larger area and their influence penetrates to greater depths (Figure 3.2), the whole volume of soil affected by the structure requires examination.

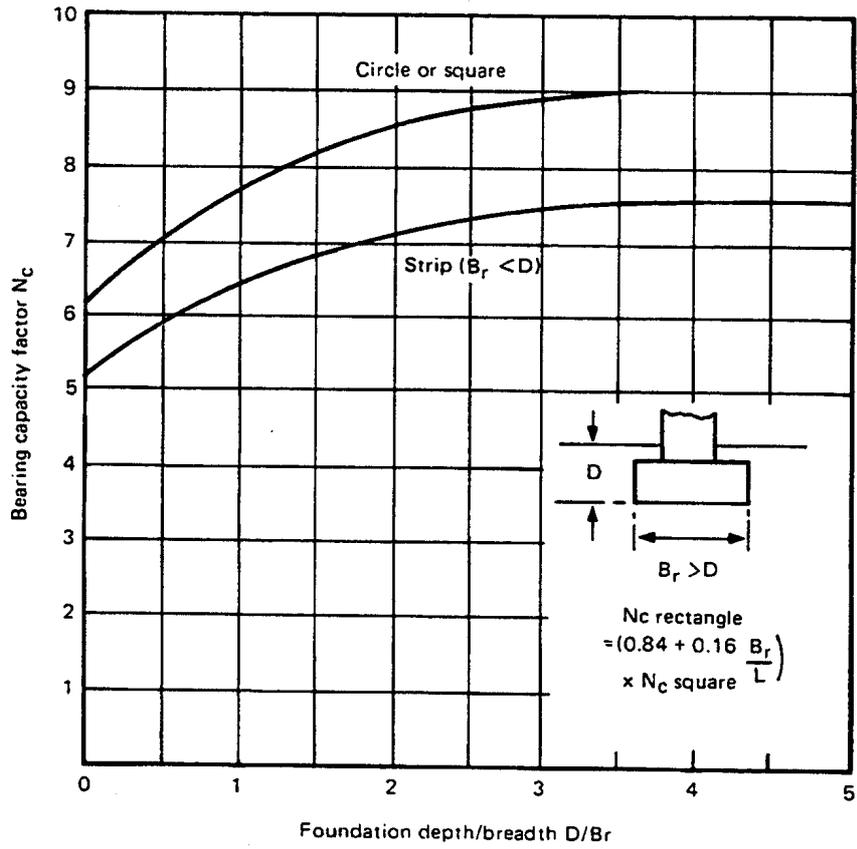


Figure 8.6 Bearing capacity factors

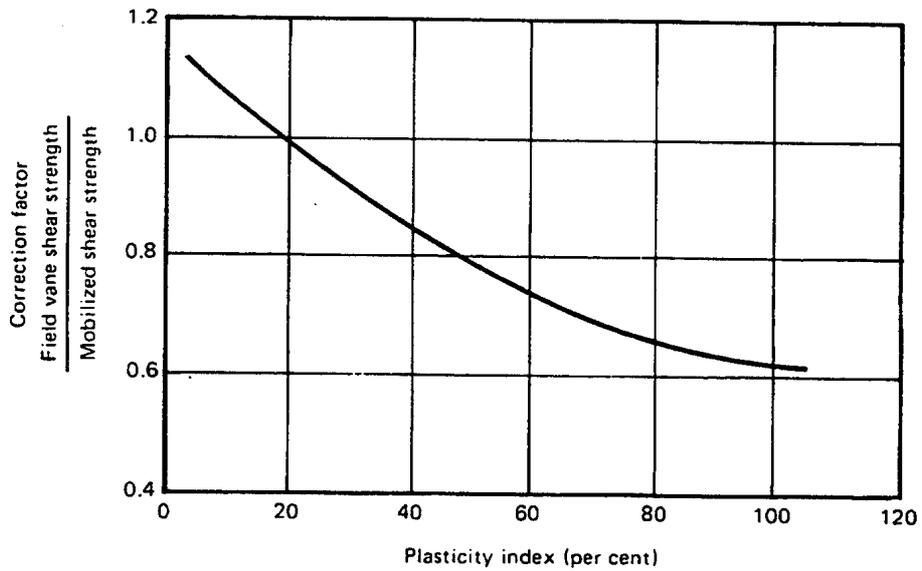


Figure 8.7 Vane shear strength correction factor

8.3.4 Presumed values

At the preliminary stage of design there may be no measured values of soil density or field strengths available. For purposes of estimation, Table 8.1 lists approximate values of allowable bearing pressures for different soil types.

8.4 Abutment design

The standard designs Figures 8.8 to 8.30, set out in Section 8.10, show typical shapes and dimensions for abutments and retaining walls, built with mass concrete and reinforced concrete for heights up to 6.5m and for a range of allowable bearing pressures. Though correct in themselves, these drawings require careful adaptation, taking into account all the individual characteristics of the Site and the superstructure, eg foundation conditions, deck thickness, expansion joints etc. Specifications for the concrete and steel are given in Chapter 14, while Chapter 16 discusses the presentation of drawings and specifications.

8.4.1 Moss concrete abutments (Figures 8.9 to 8.11)

The width of the top of the abutment is fixed at 1200mm to provide an adequate bearing seat for the bridge deck. Since the front face of the wall stem slopes at 1 in 10 and the rear face at 1 in 2, the width of the bottom of the wall stem is a function of the wall height H. If H is 5m, the width of the bottom of the wall stem is:

$$1.2 + \frac{5}{10} + \frac{5}{2} = 4.2\text{m}$$

The width of the abutment foundation, which is of reinforced concrete, depends on the allowable bearing pressure of the ground, the span and the height of the abutment. The width of the toe A and the heel B and the foundation thickness D are given in Table 8.2 for a range of waterway openings from 4m to 12m, wall heights from 2m to 6.5m and presumed bearing pressures from 400kN/m² to 100kN/m².

Table 8.3 gives details of the high yield steel reinforcement required in the foundation slab and Table 8.4 indicates the alternative mild steel reinforcement.

8.4.2 Moss concrete retaining walls (Figures 8.12 and 8.13)

These are shown in Figure 8.10 in the common configuration bisecting the angle between river and road. The width of the bottom of the wall stem varies with the height as in the case of the abutment, but it should be noted that the slope of the front face of the stem is shown in Figures 8.12 and 8.13 as 1 in 14:

Table 8.1 Presumed bearing values

(a) Foundations bearing on ROCK
width not exceeding 3m and length not more than 10 times width.

Description of rock	Presumed bearing value kN/m ²
Massive strong igneous and metamorphic rocks and limestones	10,000
Unweathered medium to fine grained sandstones	4,000
Schists and slates	3,000
Hard shales, mudstones and soft sandstones	1,500
Soft limestones	600

(b) Foundations in non-cohesive soils
at a minimum depth of 1m below ground level.

Description of soil	Presumed bearing value, kN/m ² Foundation width:		
	1m	2m	4m
Very dense sands and gravels	600	500	400
Dense sands and gravels	500	400	300
Medium-dense sands and gravels	250	200	150
Loose sands and gravels	100	75	75

(c) Foundations in cohesive soils
at a minimum depth of 1m below ground level.

Description of soil	Presumed bearing value, kN/m ² Foundation width:		
	1m	2m	4m
Hard boulder clays, hard fissured clays, weathered shales and weathered mudstones	800	600	400
Very stiff boulder clays, very stiff marls	600	400	200
Stiff boulder clays, stiff fissured clays and stiff marls	300	200	100
Firm clays	150	100	75
Soft alluvial clays	75	35	0

this is geometrically necessary for alignment with the front face of the abutment only when the wing wall is set at 45°. Widths of the toe and heel, the thickness of the wall foundation and details of the required high yield or mild steel reinforcement are set out in Tables 8.5 and 8.6.

8.4.3 Reinforced concrete abutments (Figures 8.14 and 8.15)

Like the mass concrete abutments, these are shown with a standard width of 1200mm at the top but the front and rear faces are vertical. This structure is considerably lighter than its mass concrete counterpart and so requires different foundation details. The bearing shelf now becomes an integral part of the stem.

The widths of the toe and heel, and the thickness of the foundation for various heights, span and bearing pressures are shown in Table 8.7. High yield and mild steel reinforcement for the foundations, detailed in Figure 8.17, are specified in Tables 8.8, and 8.9 respectively. Reinforcement for the wall stem is detailed in Table 8.10.

8.4.4 Reinforced concrete retaining walls (Figures 8.16 and 8.17)

These match the abutments with reinforced concrete stems. Foundation dimensions and reinforcement are detailed in Tables 8.12, 8.13 and the stem reinforcement in Table 8.11. If the retaining wall is not long, the weep holes can usually be omitted. These details apply to the typical case of walls set at 45° to the abutments supporting road embankments with slopes of 1 in 2.

8.4.5 Raft foundations (Figure 8.18)

Where waterway openings are less than 4m wide and where the allowable bearing pressure values are too low to support standard abutment foundations, single or multiple reinforced box culverts should be used. Alternatively a continuous raft, forming a foundation to both abutments and an invert to the waterway, may be built with an articulated deck. Table 8.14 indicates the thickness of the raft and details of high yield and mild steel reinforcement. Scour protection is usually required both upstream and downstream of the raft, as in the case of vented causeways (Section 7.4).

8.4.6 Bearing shelves (Figures 8.19 to 8.21)

These designs are generally suitable for concrete, composite or timber decks, though the bearing details will be specific to the deck type. Figures 9.2 and 10.12 show the dowel and bearing pad details required for concrete and composite decks.

Good drainage and the facility for removal of debris are important requirements on all bearing shelves.

The road approaches should be built to prevent water draining onto the bridge, but some water falling on the deck will penetrate expansion joint seals and leak through to the bearing shelves. This is particularly likely to occur when no seal at all is provided (Figure 14.2). A number of drainage configurations are available, Hambly (1979), but the two main principles to be observed are:

- slope horizontal surfaces to direct water away from the bearing pads;
- provide good access for the removal of stones, vegetation, bird nests and other debris.

8.5 Reinforced concrete piers (Figures 8.22 to 8.25)

Though piers may be built using masonry or mass concrete, reinforced concrete has several advantages, notably a more slender stem presenting less interference to flow and hence causing less induced scour. The superstructure spans detailed in Chapters 9, 10 and 11 are designed to be simply supported at the abutments and at the piers. Each span should have one fixed and one free end. It is usual practice, though not essential, to provide one fixed bearing and one free bearing on the bearing shelf of each pier (Figure 9.2).

Pier foundations are even more susceptible to damage by erosion than abutment foundations. They must be constructed on soils of well established allowable bearing pressure, on the basis of the guidelines in Chapter 3 and Section 8.3. Foundation sizes are detailed in Table 8.16 according to height, spans and allowable bearing pressures. Tables 8.17 and 8.18 detail the reinforcement sizes for the foundations and Table 8.15 for the stem.

8.6 Foundations on rock

The foundation designs presented in the preceding sections are for soils readily excavated by hand or mechanical digger. Modifications may be required to suit individual site conditions, particularly when bedrock is encountered. Where foundations are set on rock at ground level or on the river bed, substantial keying will be necessary in the form of steel dowels and notching.

8.7 Earthquake restraint (Figures 8.26 and 8.27)

The bridge bearings detailed in drawings Figures 9.2, 9.3, 10.11 and 10.12 are designed to support and restrain the superstructures under normal loading from traffic, wind and temperature change, but would not be robust enough to resist the severe forces that can result from seismic activity. Bridge authorities in earthquake zones can give guidance on the precautions normally required in the area. These precautions most often take the form of stronger lateral restraint to prevent

sideways movement of the bridge superstructure on the abutments and/or wider bearing shelves to prevent the spans falling off their supports. Extra restraint can be provided quite easily for composite decks; a typical solution is shown in Figure 8.27. This solution is not appropriate for concrete slab decks, which are usually restrained by upstands on the bearing shelf, as shown in Figure 8.26.

In areas prone to earthquakes these precautions are required at both abutments and piers.

8.8 Run-on slabs

Almost all earth embankments are subject to settlement. The amount they settle will depend on the height, the degree of compaction of the material and the strength of the subgrade. Relatively uniform settlement can be expected from most embankments until a cause of uneven compaction is met, such as a bridge or a box culvert with little fill above it (Figure 8.28a). It is difficult to compact fully the embankment material close to the bridge abutments or the culvert walls, and the result of poor compaction is more pronounced settlement. The resulting longitudinal profile is uncomfortable for road users and causes impact loads on the structure, owing to vehicle bounce.

These local depressions in the carriageway close to drainage structures may be bridged using run-on slabs (Figures 8.28b, 8.29 and 8.30). They are more easily constructed at the same time as the structure, rather than afterwards as a remedial measure, and they span the fill material susceptible to settlement. One end of the slab rests on a small shelf cast onto the culvert wall or on the abutment ballast wall, while the other rests on well compacted material several metres away.

Run-on slabs are usually between 3 and 6 metres long. The concrete and reinforcement details may be abstracted from the culvert detailing in Chapter 12, assuming that the slab is resting on good support for one third of its length, ie a 6m slab will have similar details to a 4m wide culvert lid. Construction details of the hinged end are shown in Figures 8.29 and 8.30. Run-on slabs are usually made wide enough to support the kerbs on the approach roads.

Run-on slabs should not be required where efficient maintenance facilities are readily available (Hambley 1979).

8.9 References

Hambley E C (1979). *Bridge foundations and substructures*. Building Research Establishment, Stationery Office, London.

Pennells E (1978). *Concrete bridge designer's manual*. Cement and Concrete Association, London.

Tomlinson M J (1977). *Pile design and Construction practice*. Cement and Concrete Association, London.

Tomlinson M J (1986). *Foundation design and construction*. Longman Singapore Publishers Pte. Ltd., Singapore.

Victor D J (1980). *Essentials of bridge engineering*. Oxford and IBH Publishing Co., New Delhi.

8.10 Standard drawings and tables for abutments, retaining walls and piers

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Embankment settlement profiles at a culvert or bridge	8.28
Culvert run-on slab	8.29
Bridge run-on slab	8.30

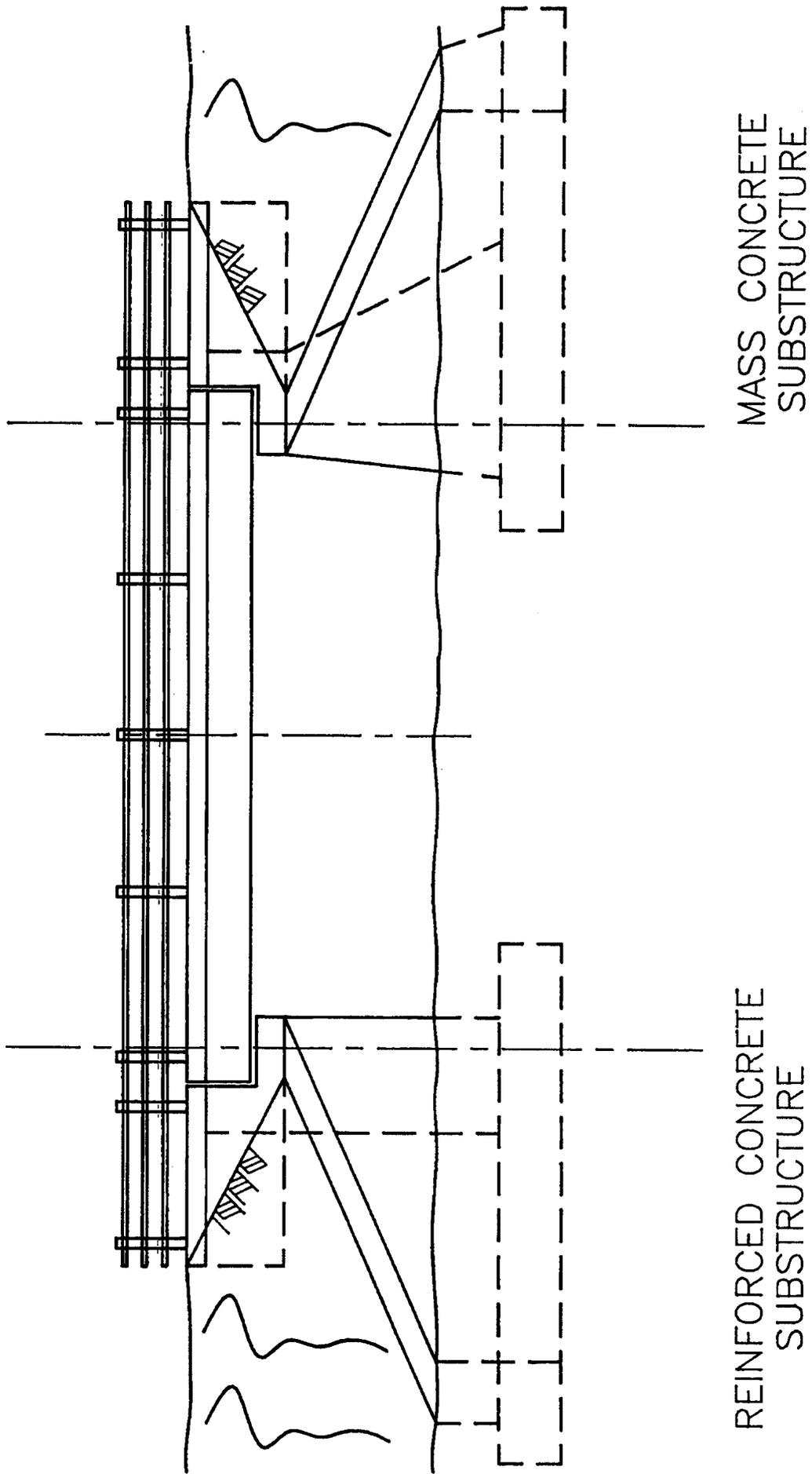


Figure 8.8 Typical elevation of a single span bridge

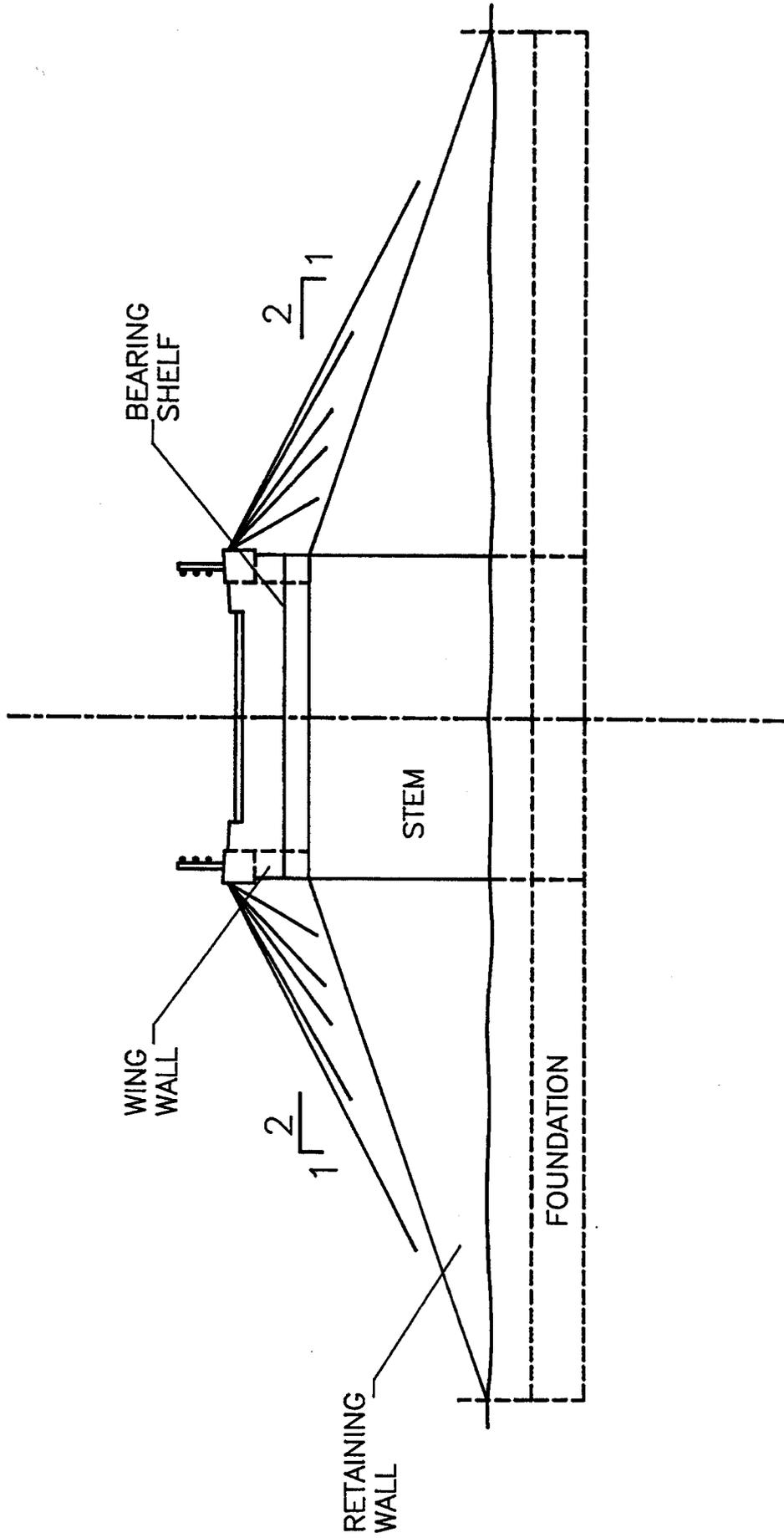


Figure 8.9 Abutment – front elevation

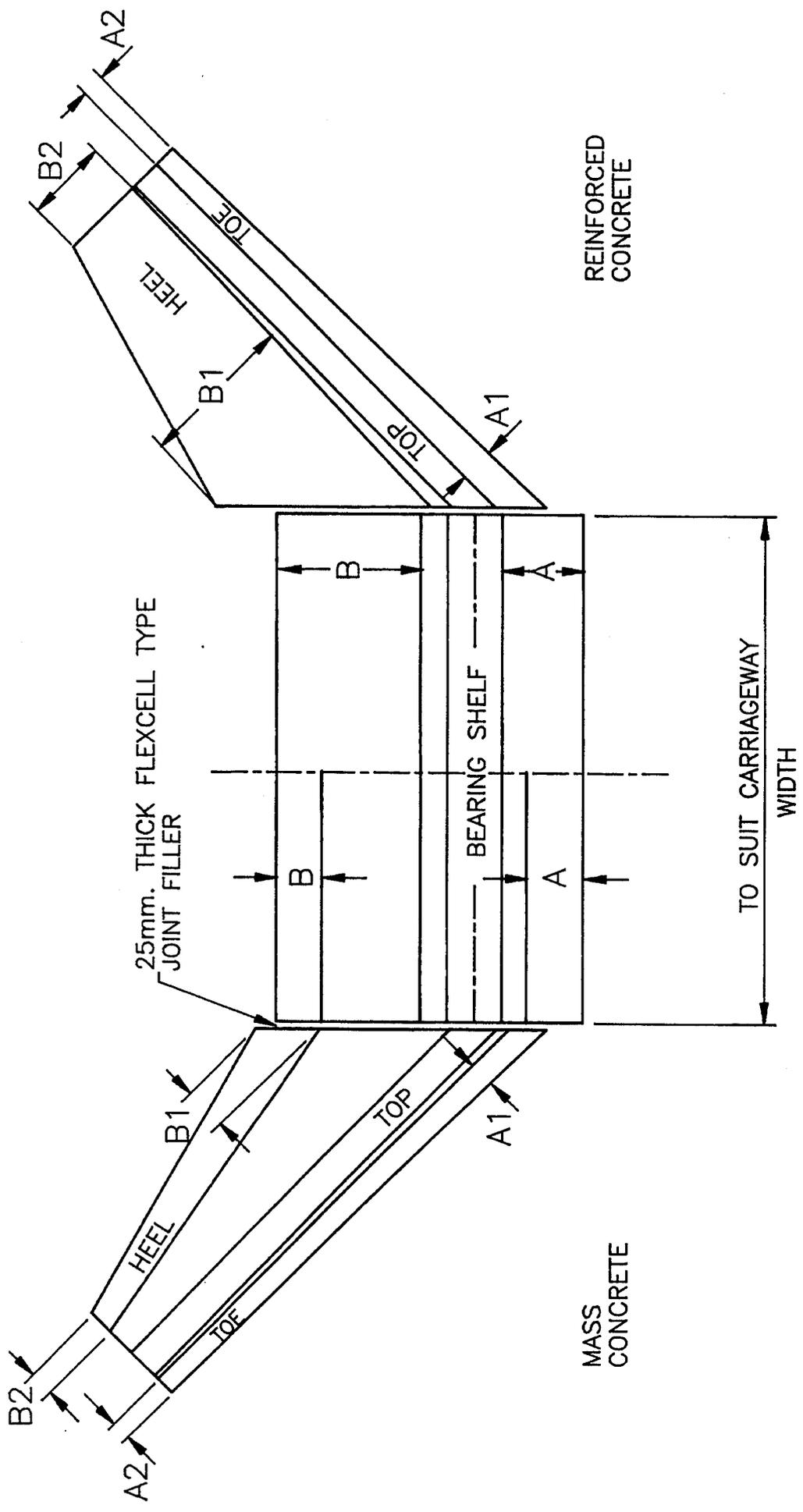
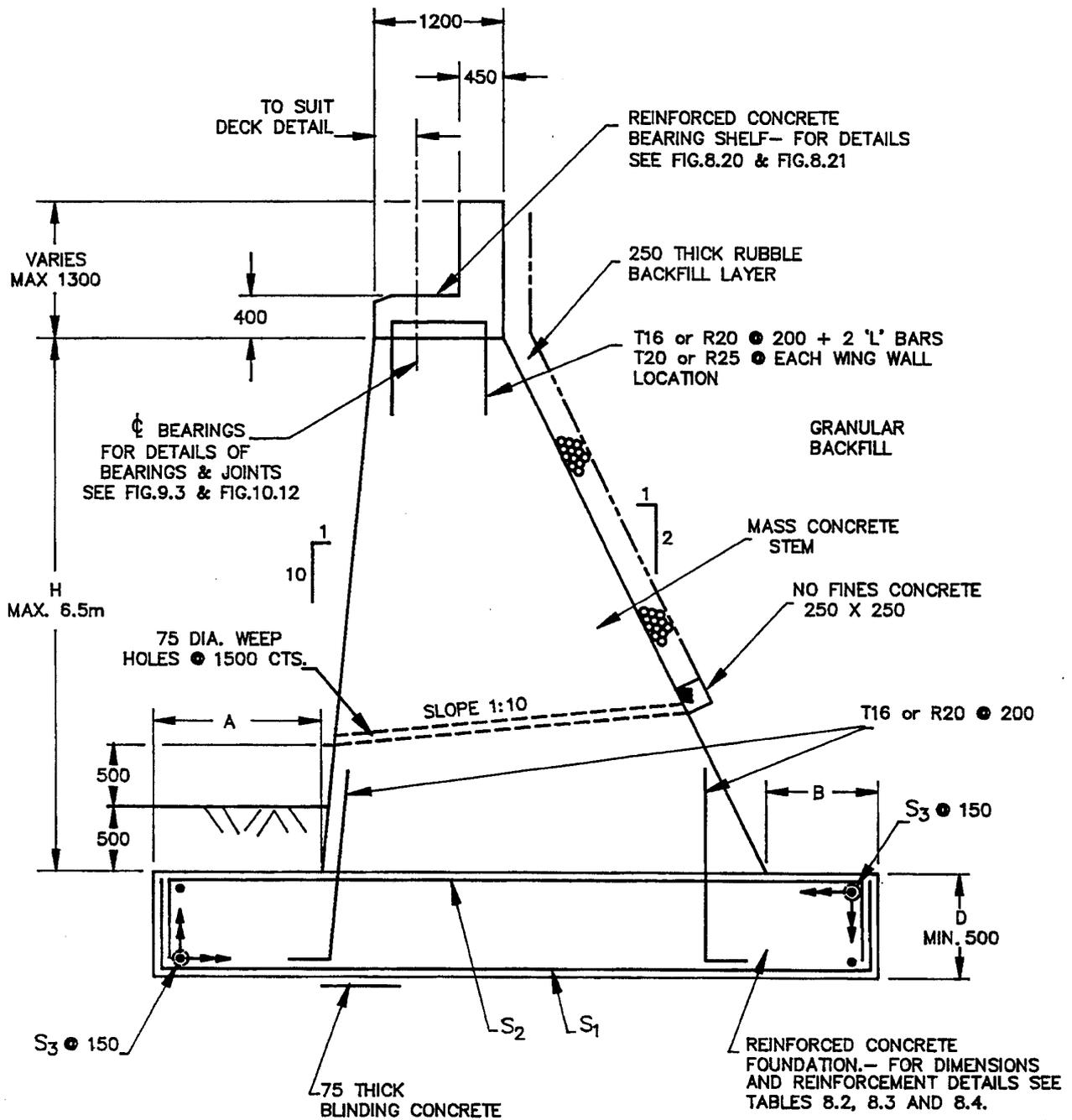


Figure 8.10 Abutment - plan



ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.11 Abutment, mass concrete – vertical section

FOUNDATION REINFORCEMENT IS DETAILED IN TABLE 8.6
 FOUNDATION DIMENSIONS ARE GIVEN IN TABLE 8.5

NOTE THAT DIMENSIONS:

A1, B1 AND D CORRESPOND TO H MAX.

A2 AND B2 CORRESPOND TO H MIN.

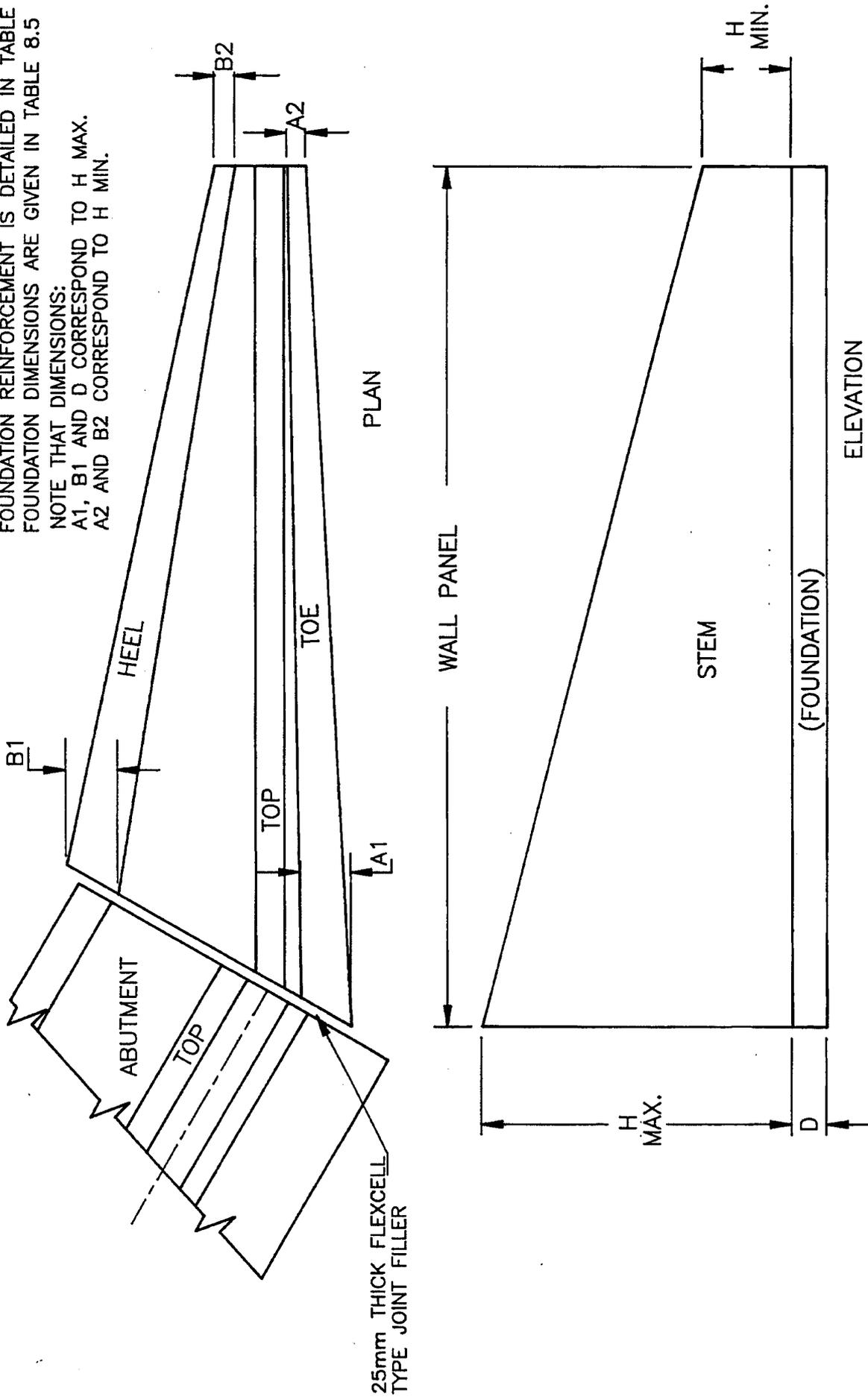
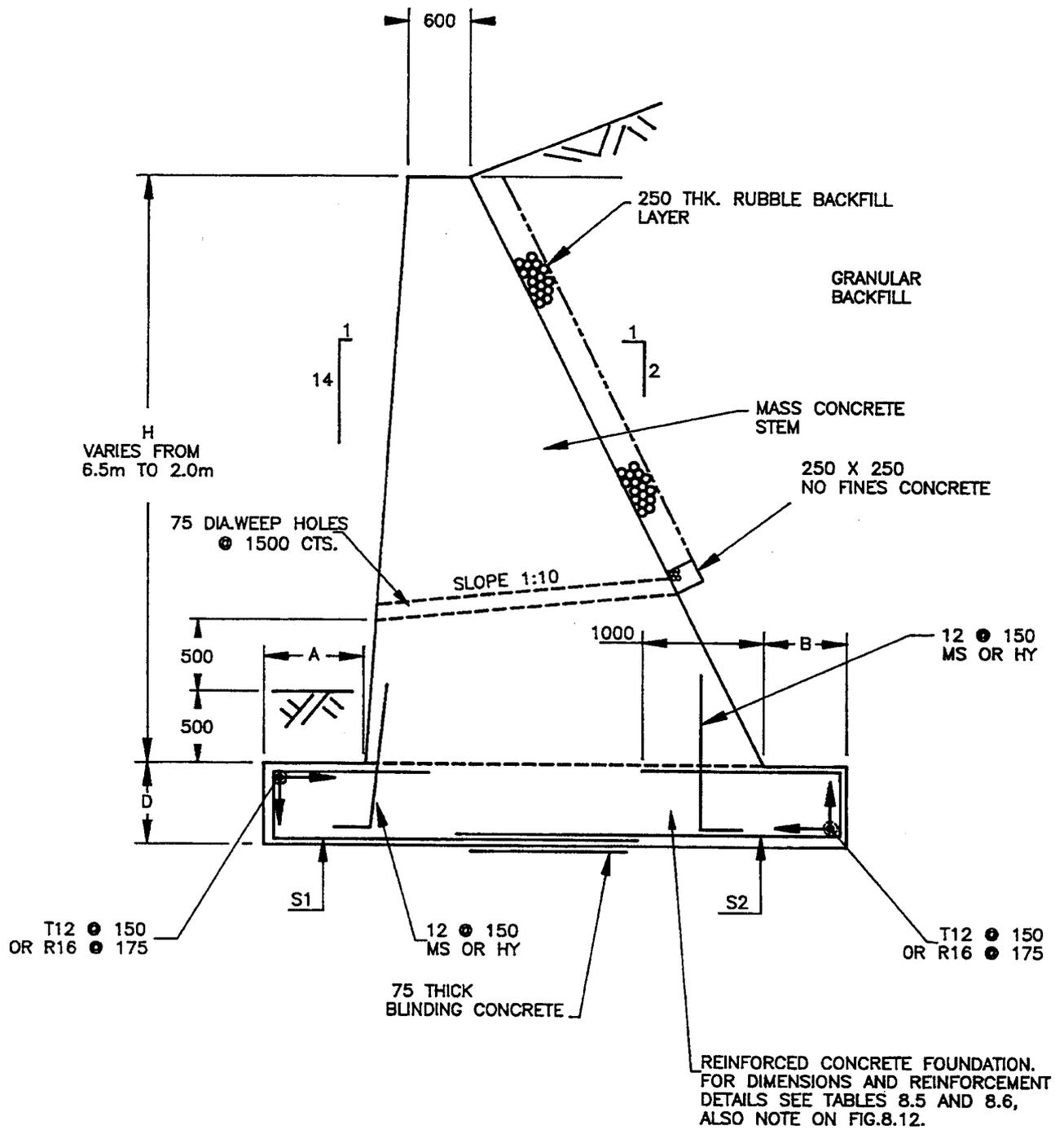
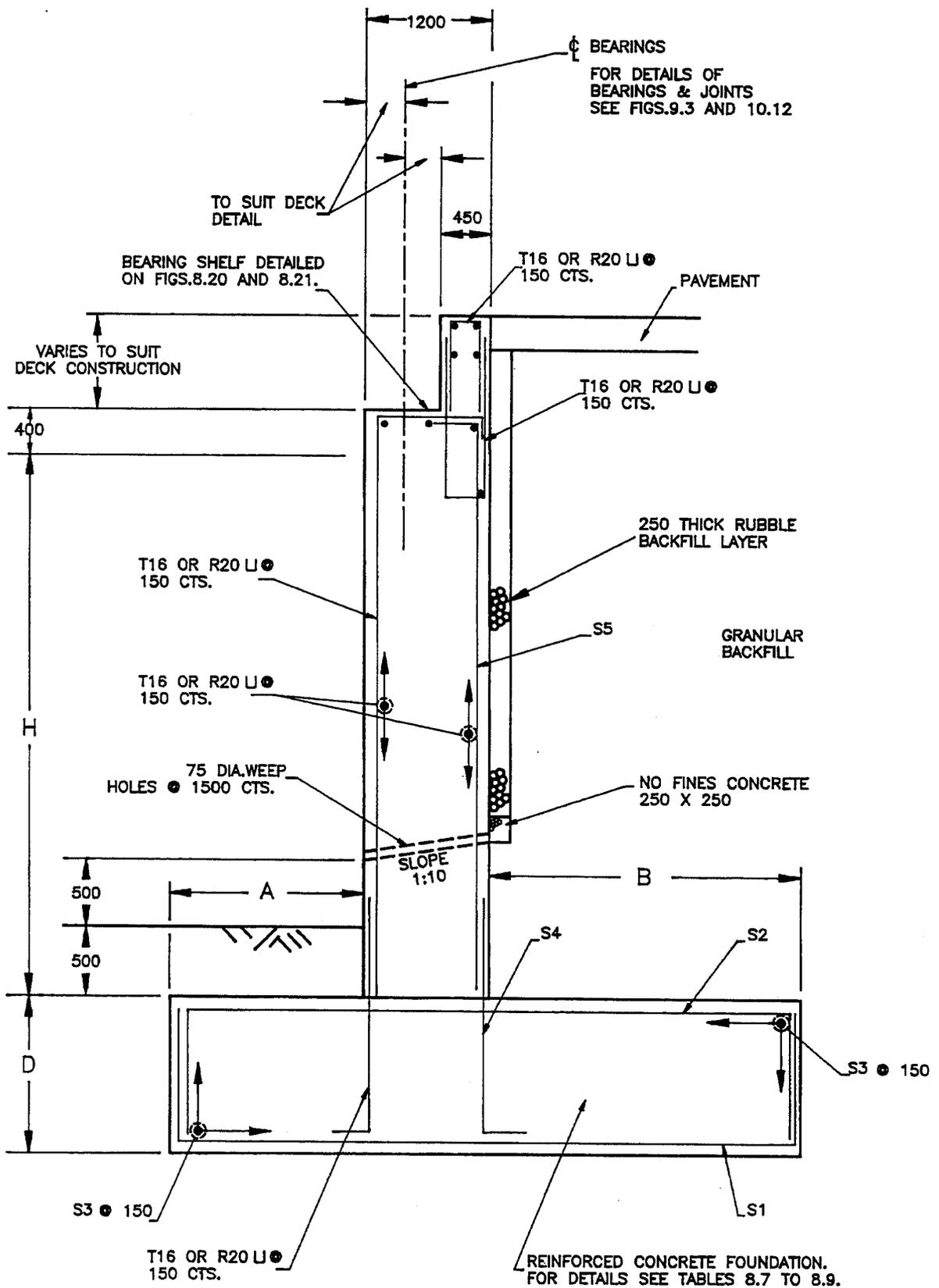


Figure 8.12 Retaining wall, mass concrete – plan and elevation



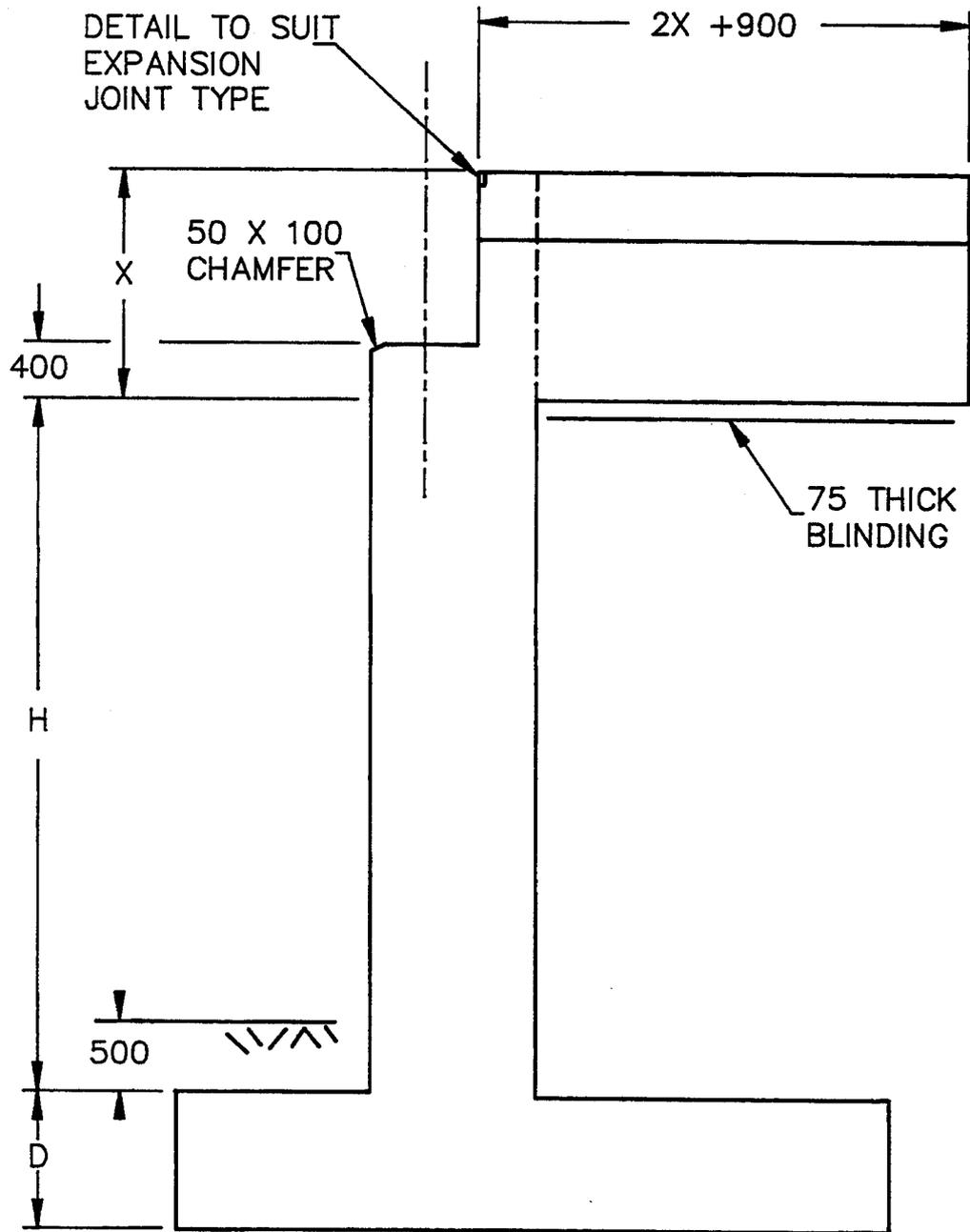
ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.13 Retaining wall mass concrete – vertical section



ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.14 Abutment, reinforced concrete – vertical section

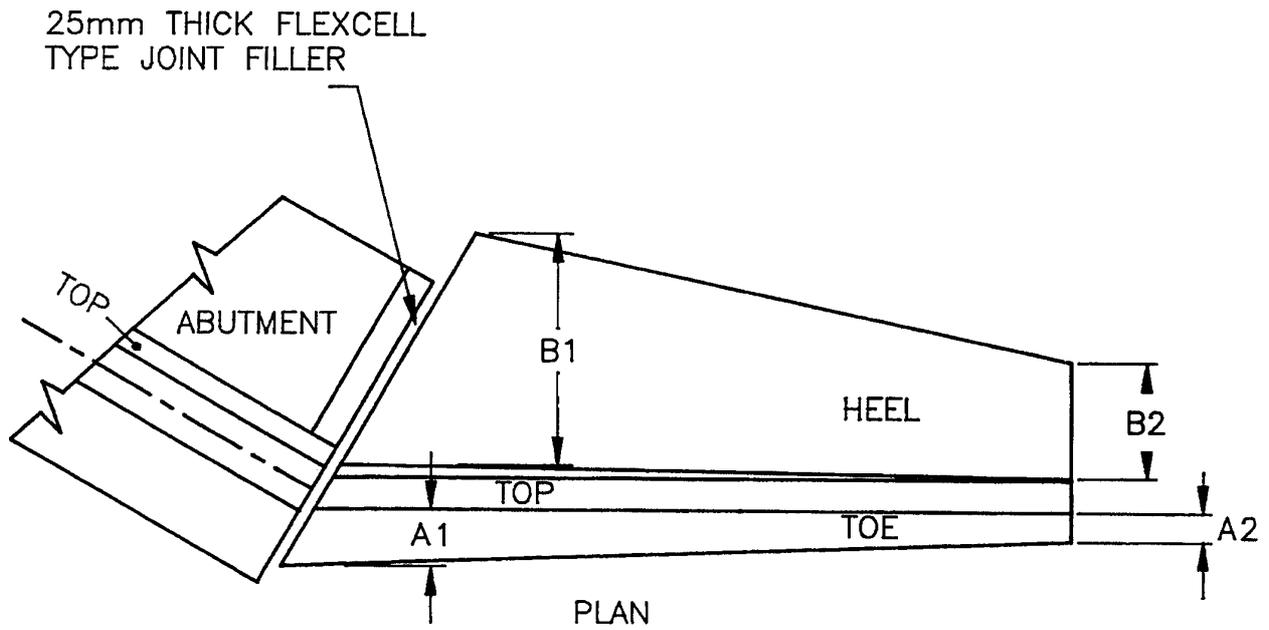


FOR WING WALL REINFORCEMENT
DETAILS SEE FIG.8.21.

75 THICK
BLINDING

ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.15 Abutment, reinforced concrete – side elevation



FOUNDATION REINFORCEMENT IS DETAILED IN TABLE 8.13
 FOUNDATION DIMENSIONS ARE GIVEN IN TABLE 8.12
 NOTE THAT DIMENSIONS:
 A1, B1 AND D CORRESPOND TO H MAX.
 A2 AND B2 CORRESPOND TO H MIN.

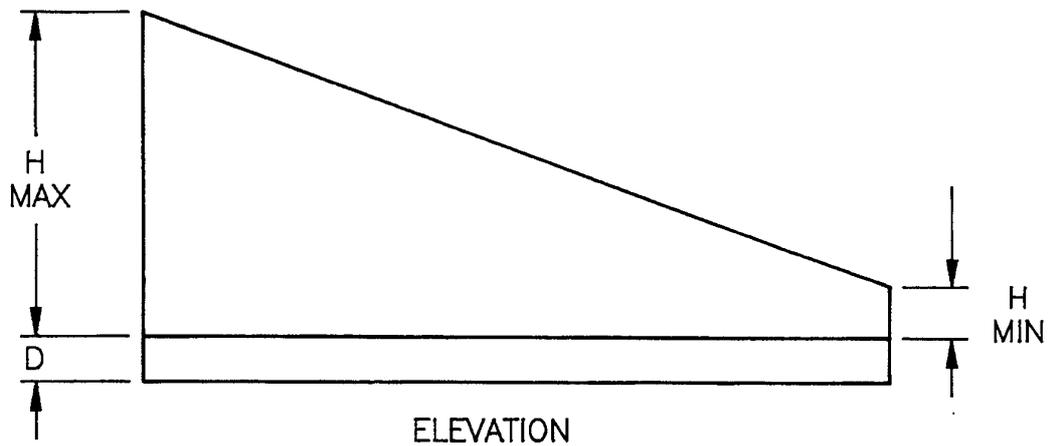
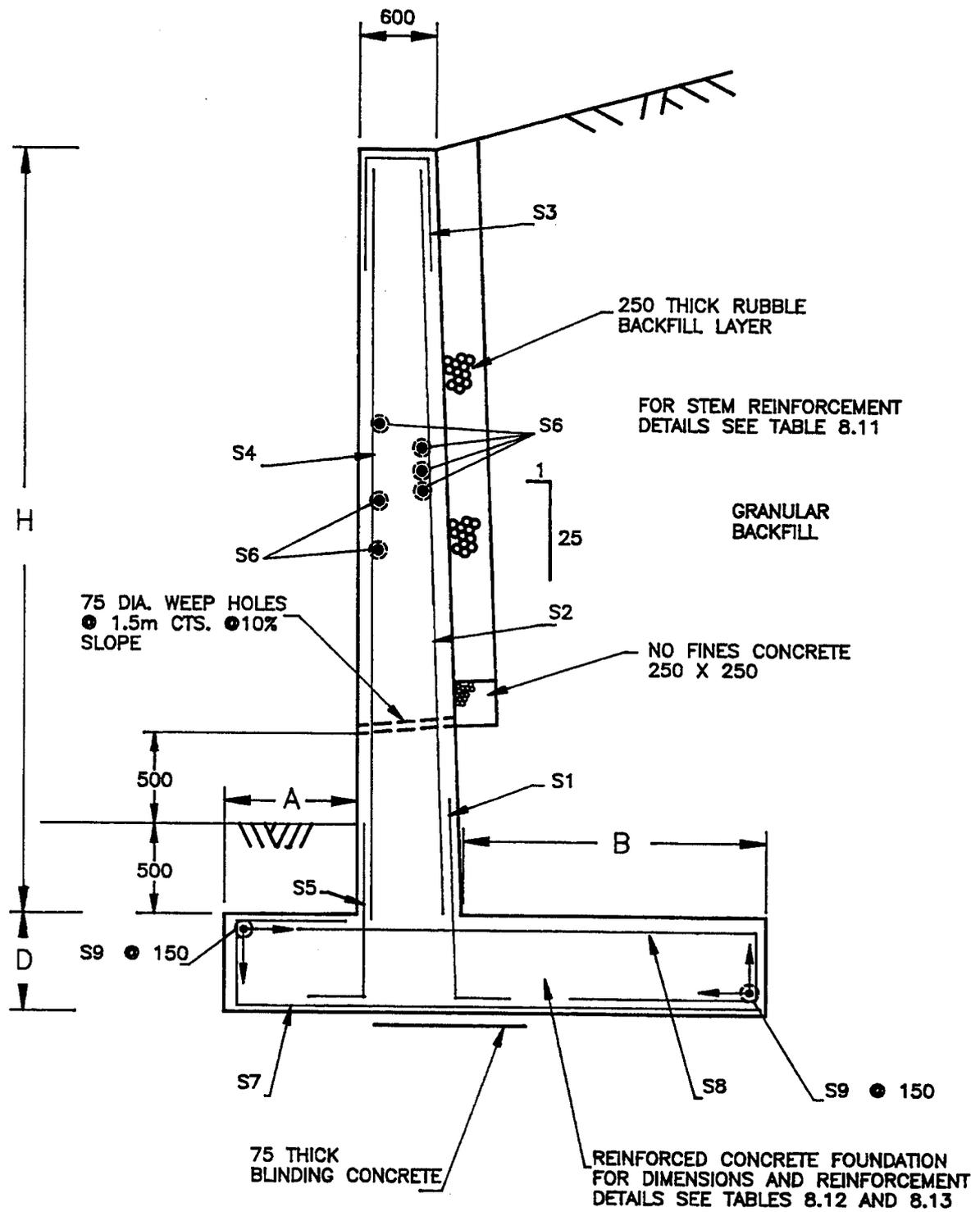


Figure 8.16 Retaining wall, Reinforced concrete – plan and elevation



ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.17 Retaining wall, reinforced concrete – vertical section

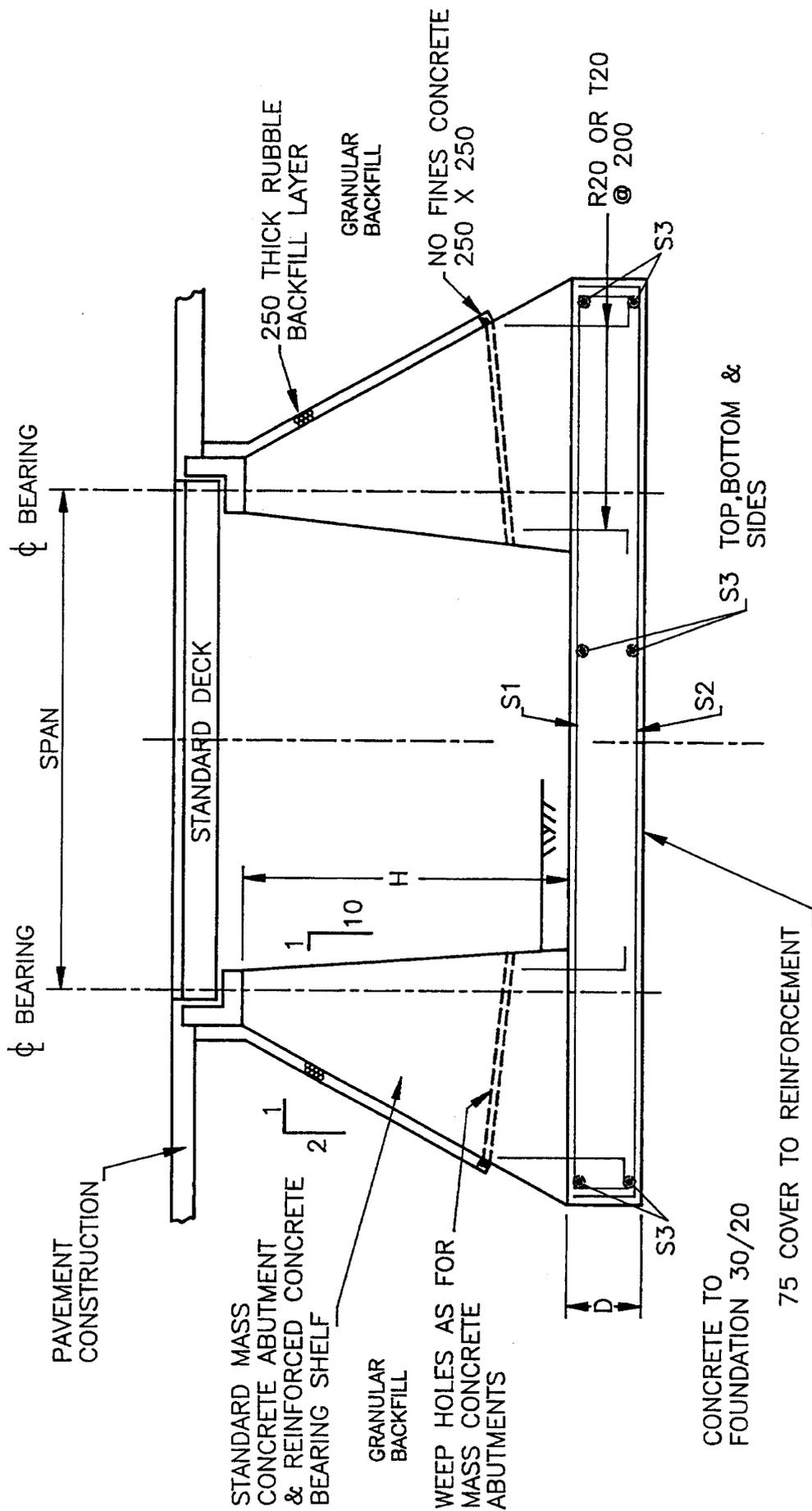
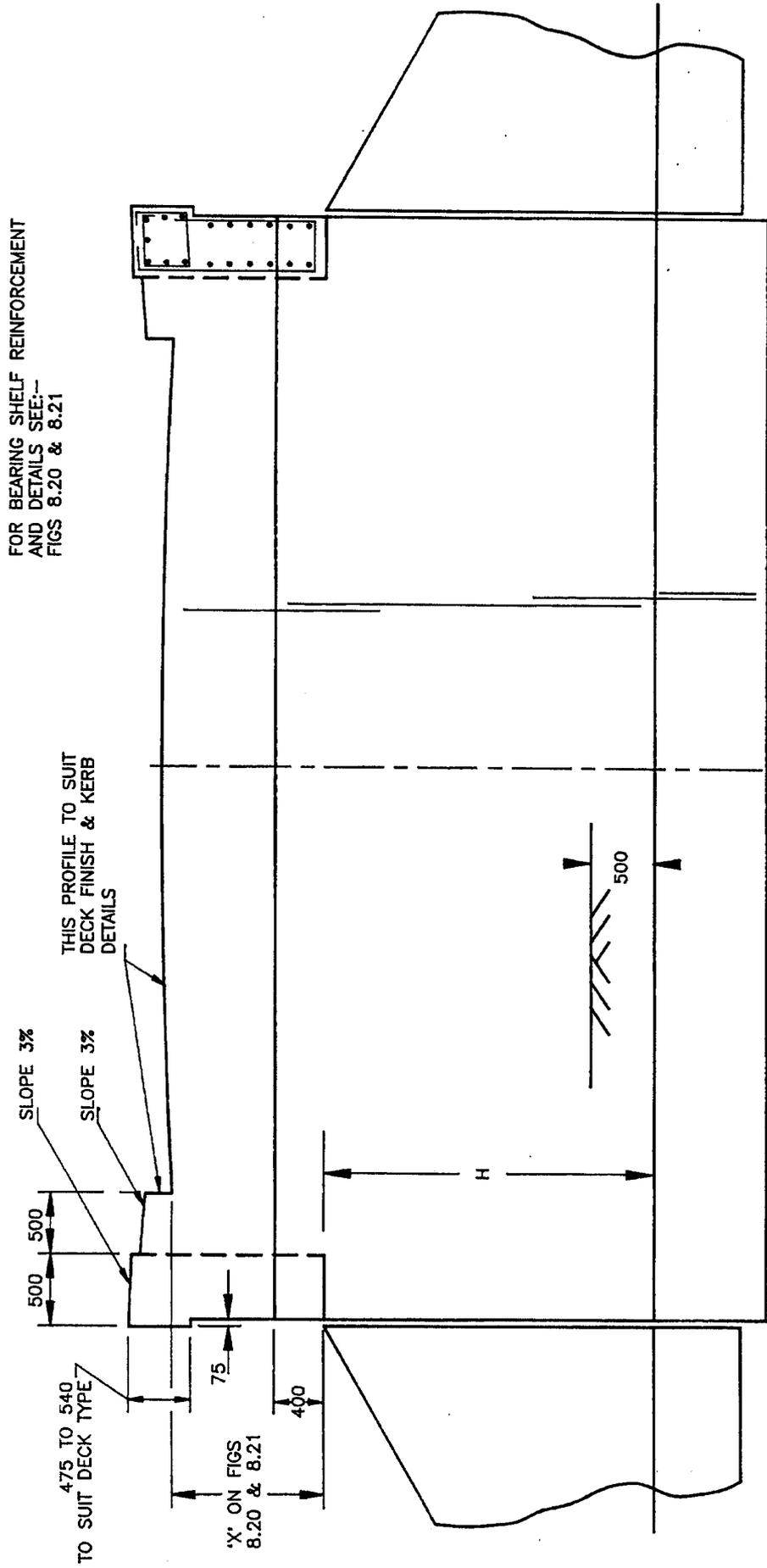


Figure 8.18 Raft foundation

ALL DIMENSIONS IN MILLIMETRES (mm).

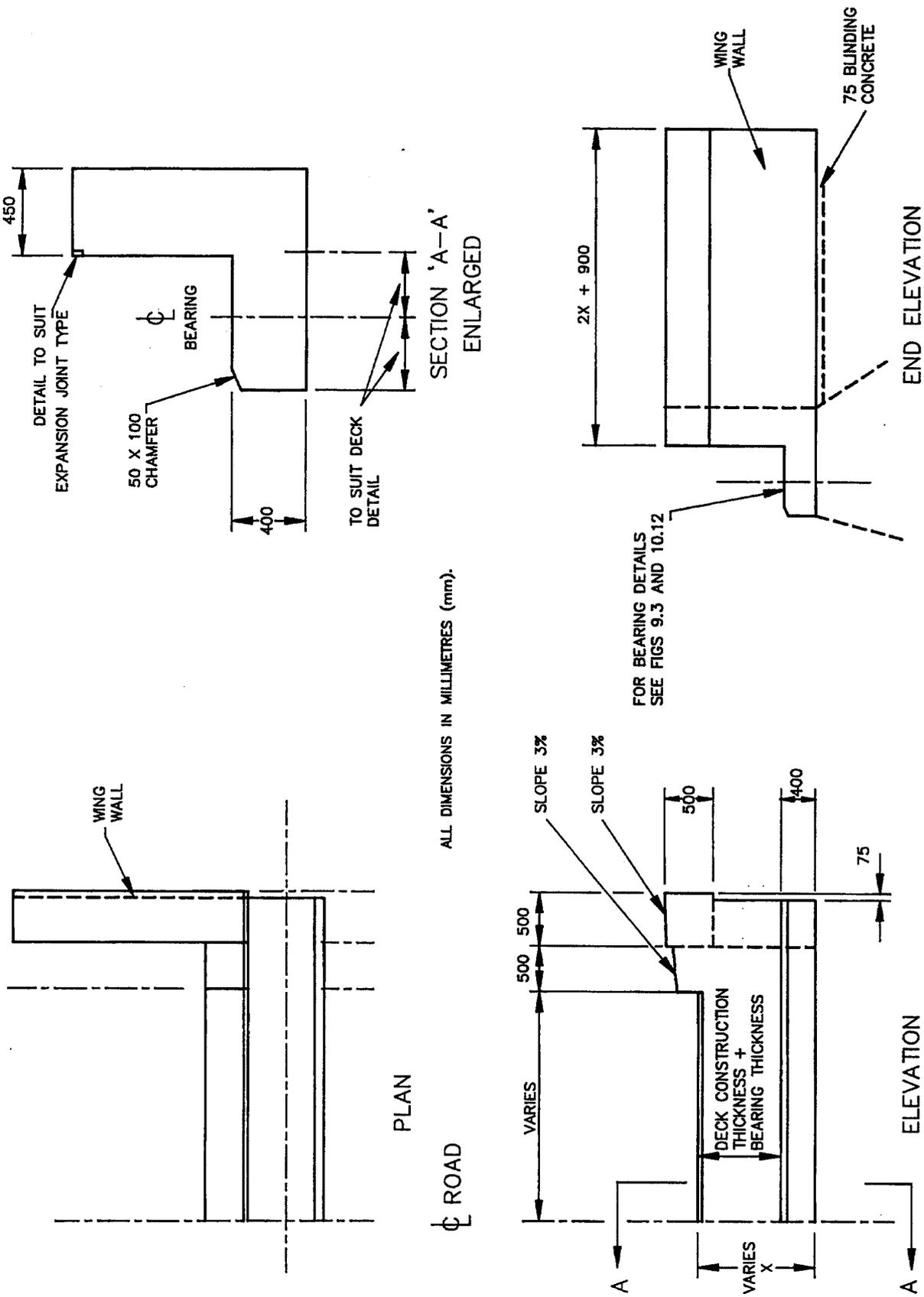
FOR FOUNDATION DEPTH AND REINFORCEMENT DETAILS, SEE TABLE 8.14

Figure 8.18 Raft foundation



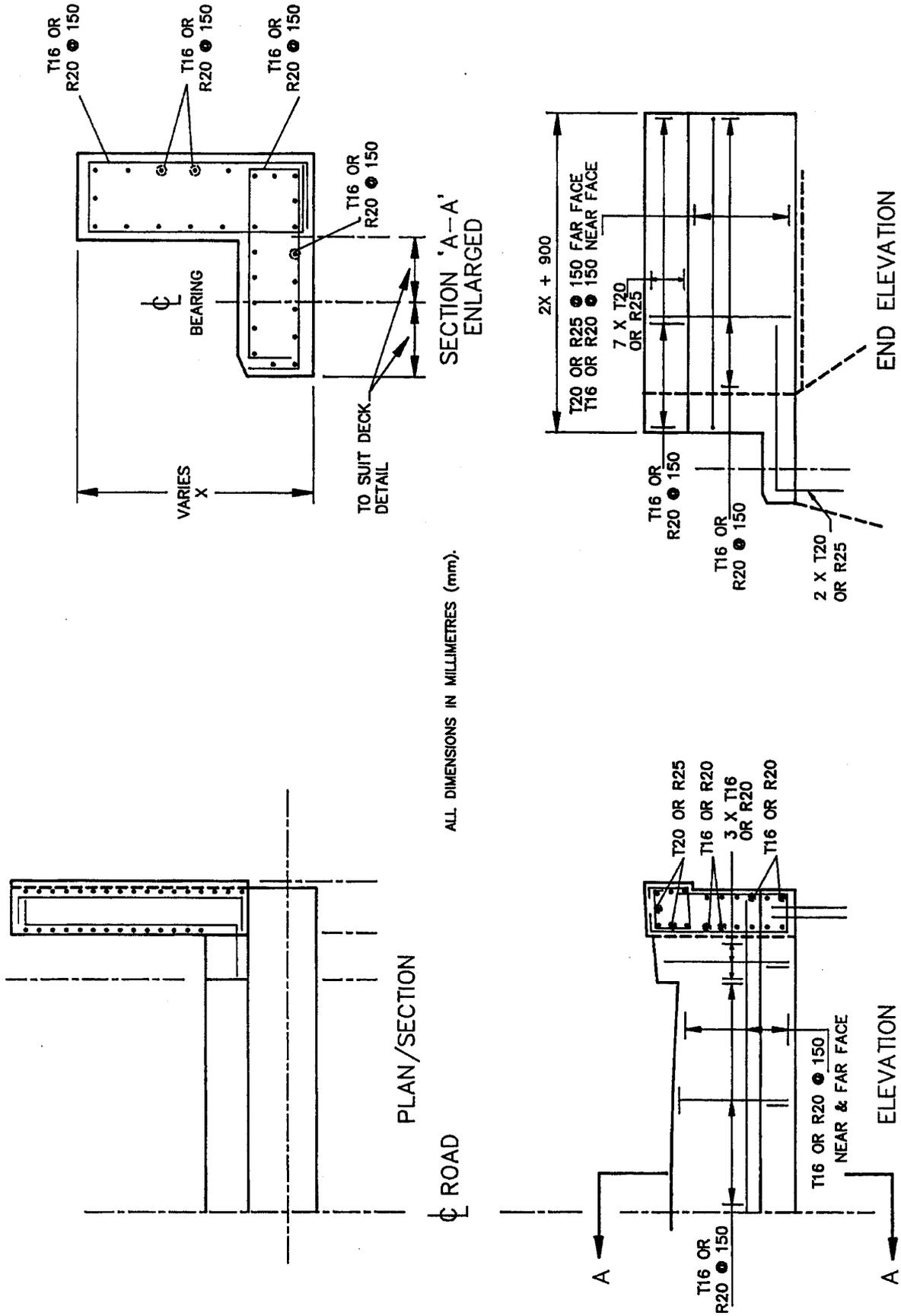
ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.19 Bearing shelf on reinforced concrete abutment – front elevation



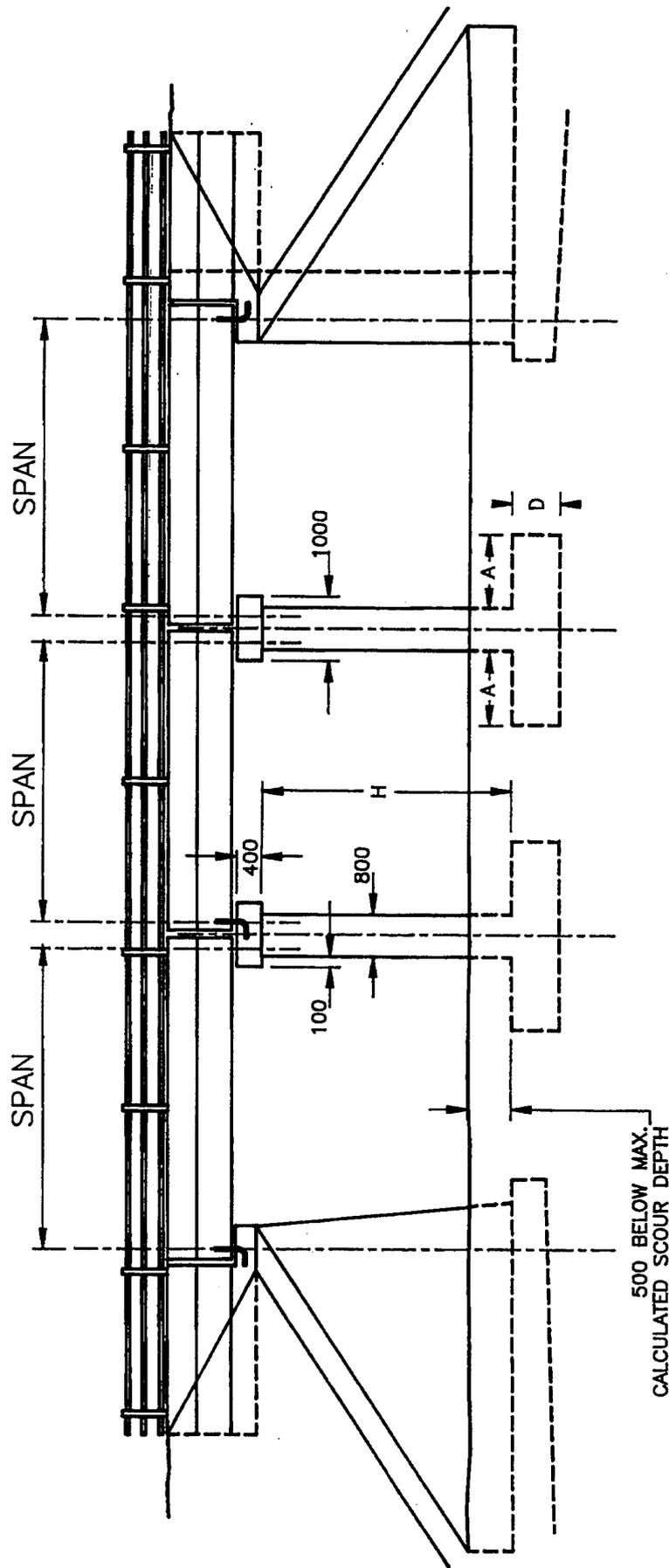
ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.20 Bearing shelf details



ALL DIMENSIONS IN MILLIMETRES (mm).

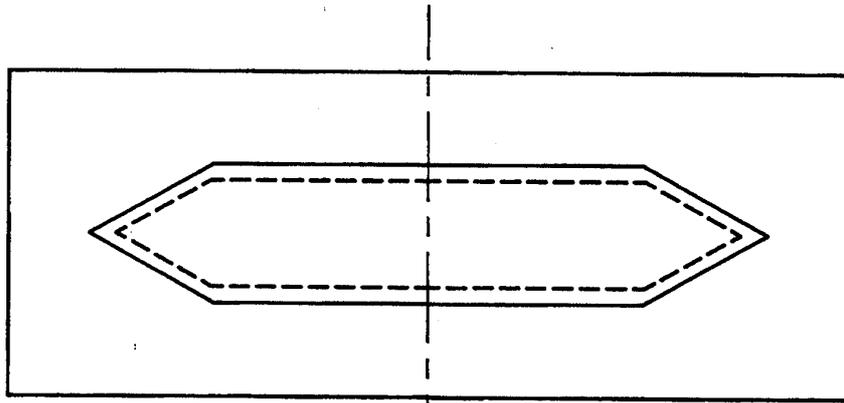
Figure 8.21 Bearing shelf - reinforcement



PIERS WITH
 REINFORCED CONCRETE STEM
 REINFORCED CONCRETE FOUNDATION

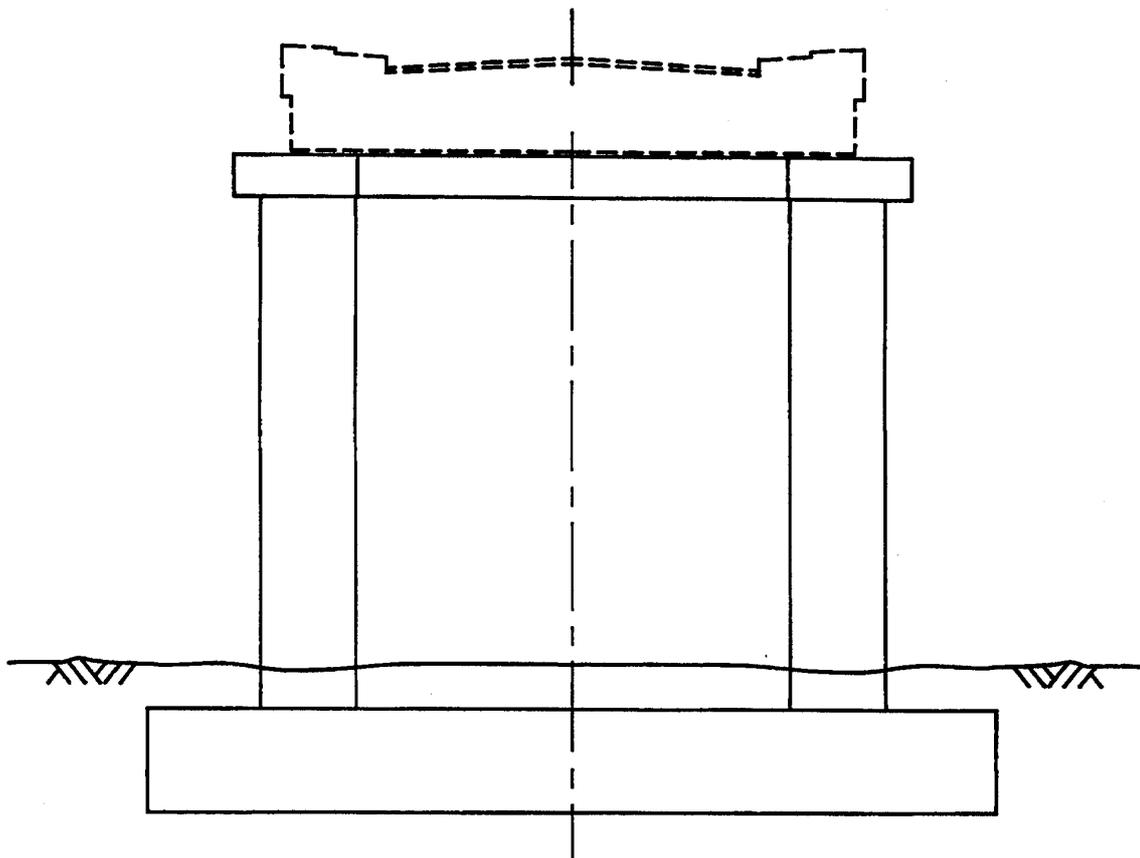
ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.22 Piers – general arrangement



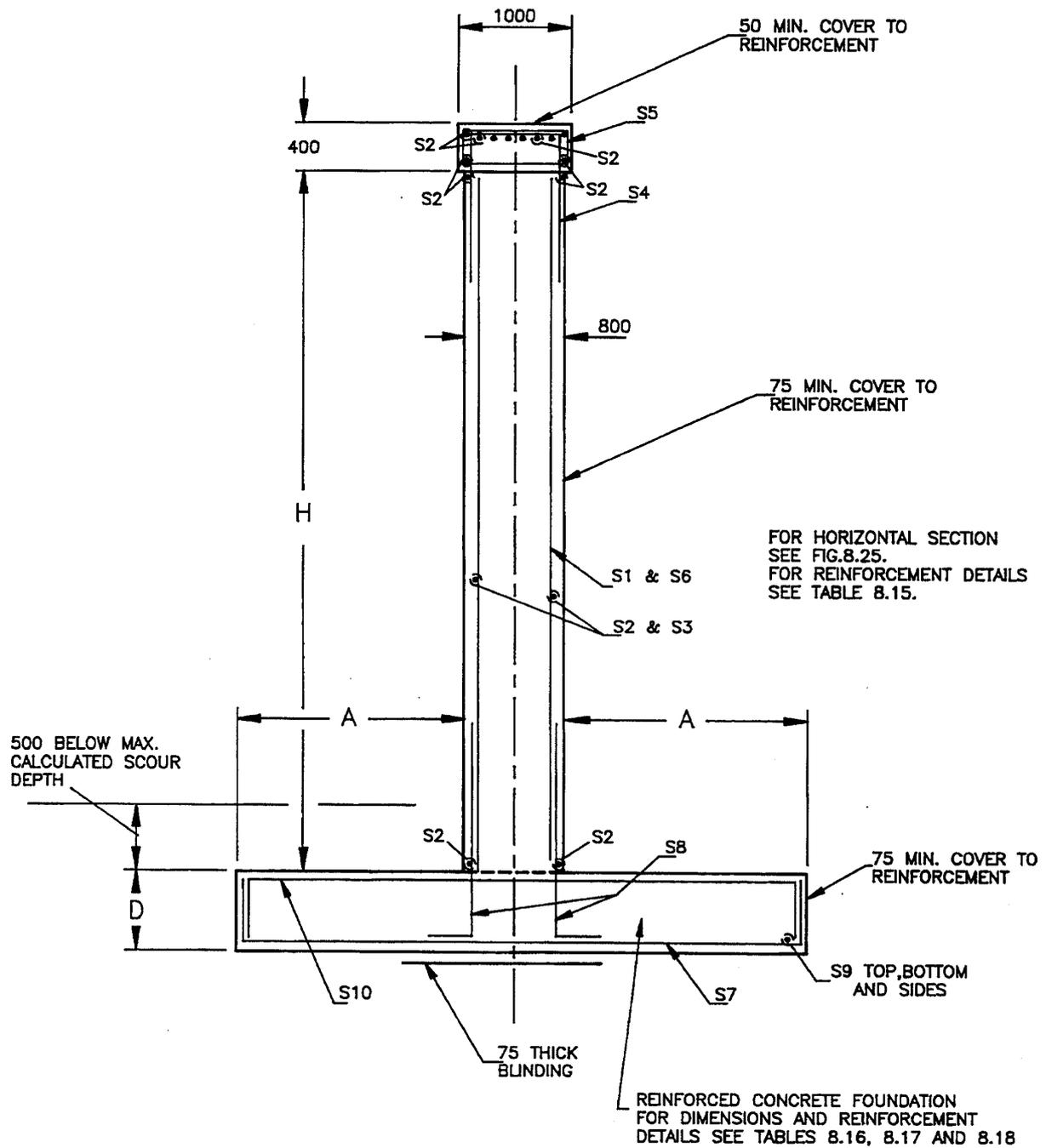
PLAN

FOR BEARINGS AND DOWEL DETAILS SEE:
FIGS 9.2 OR 10.11.



ELEVATION

Figure 8.23 Pier – plan and elevation



ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 8.24 Pier, reinforced concrete – vertical section

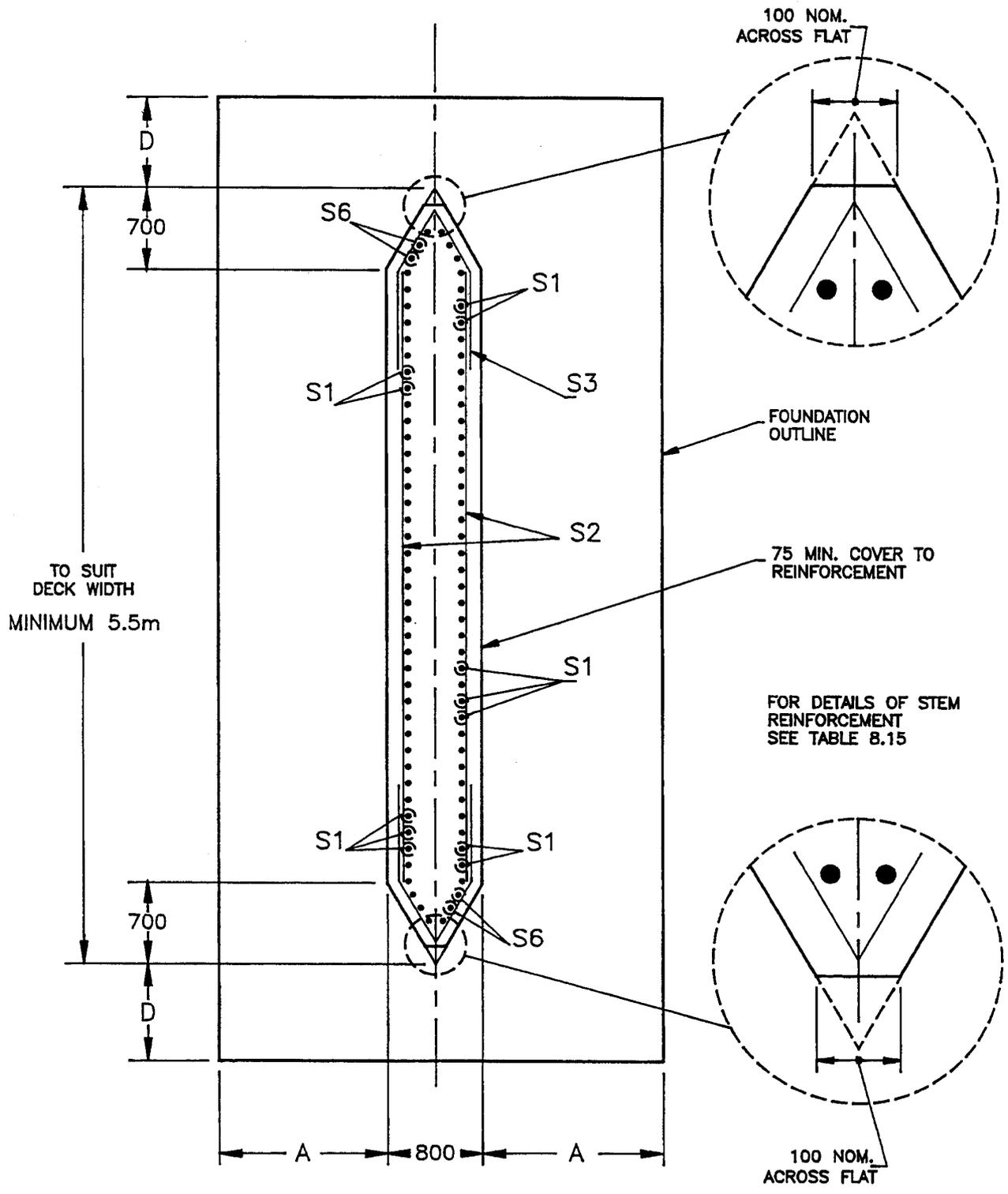
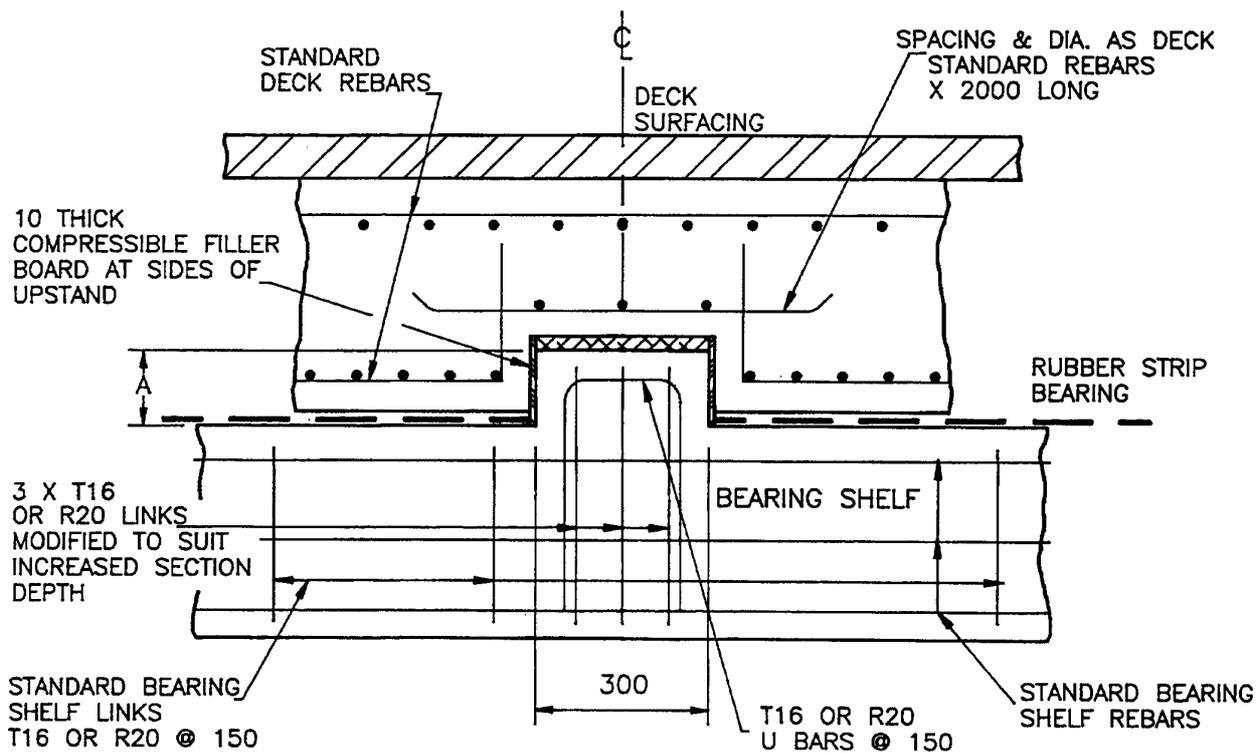
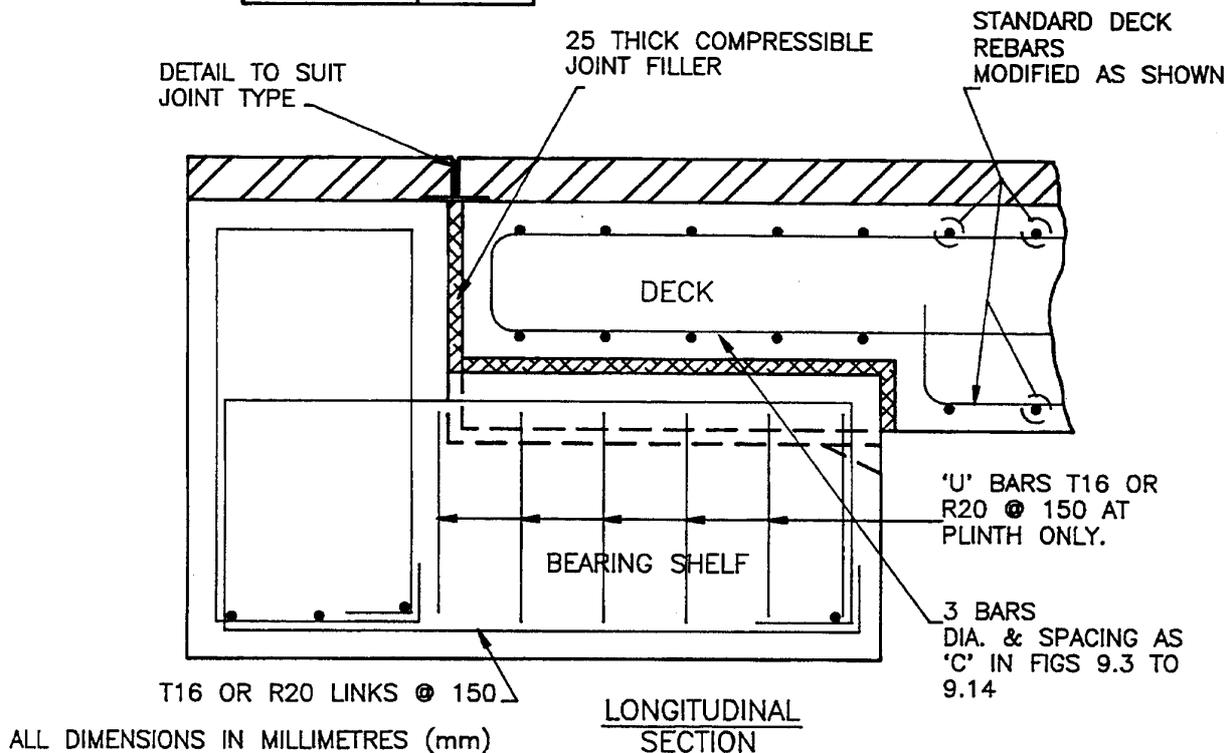


Figure 8.25 Pier, reinforced concrete – horizontal section



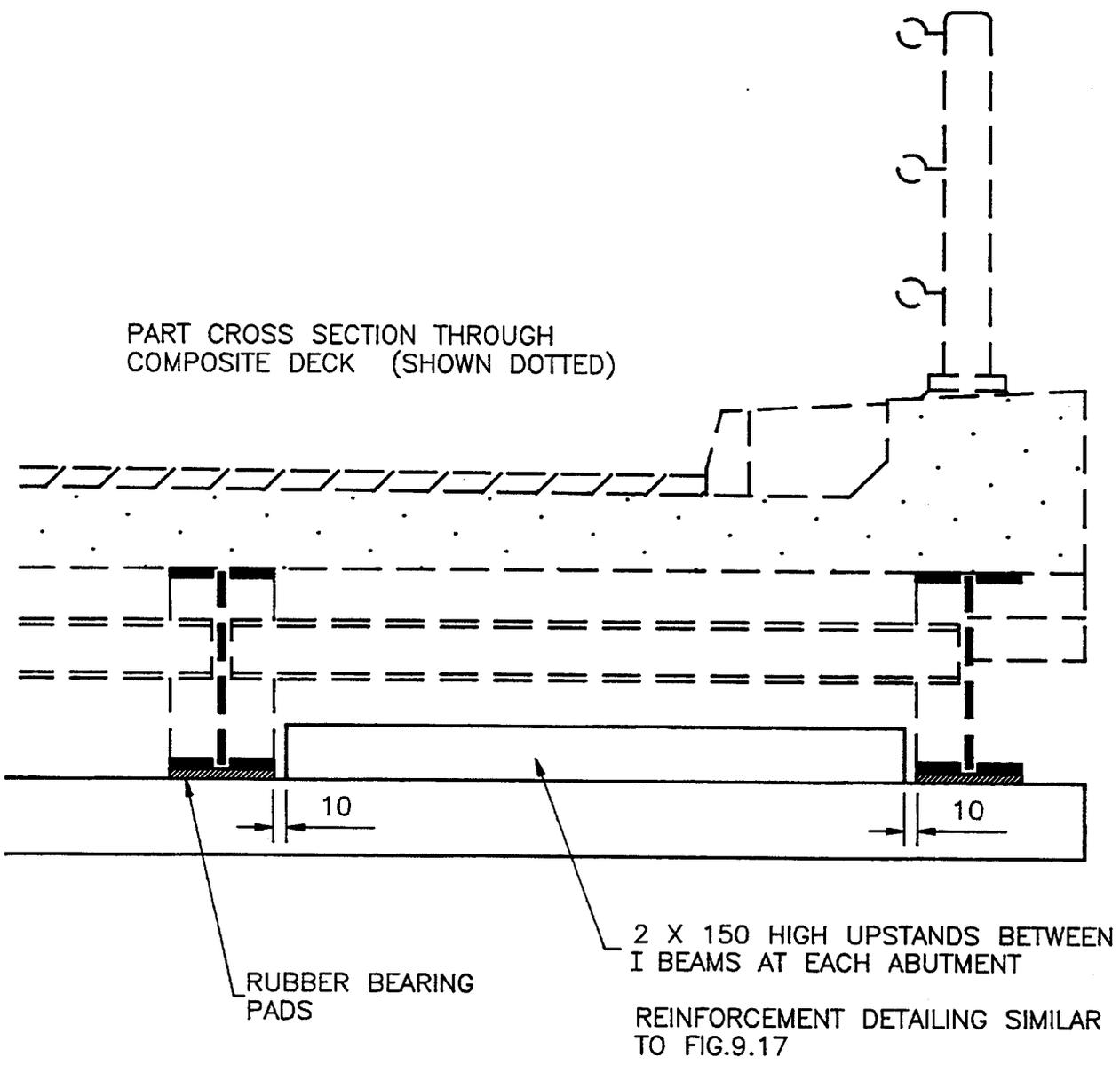
DECK THICKNESS	A
300,400	100
500-750	150

CROSS SECTION



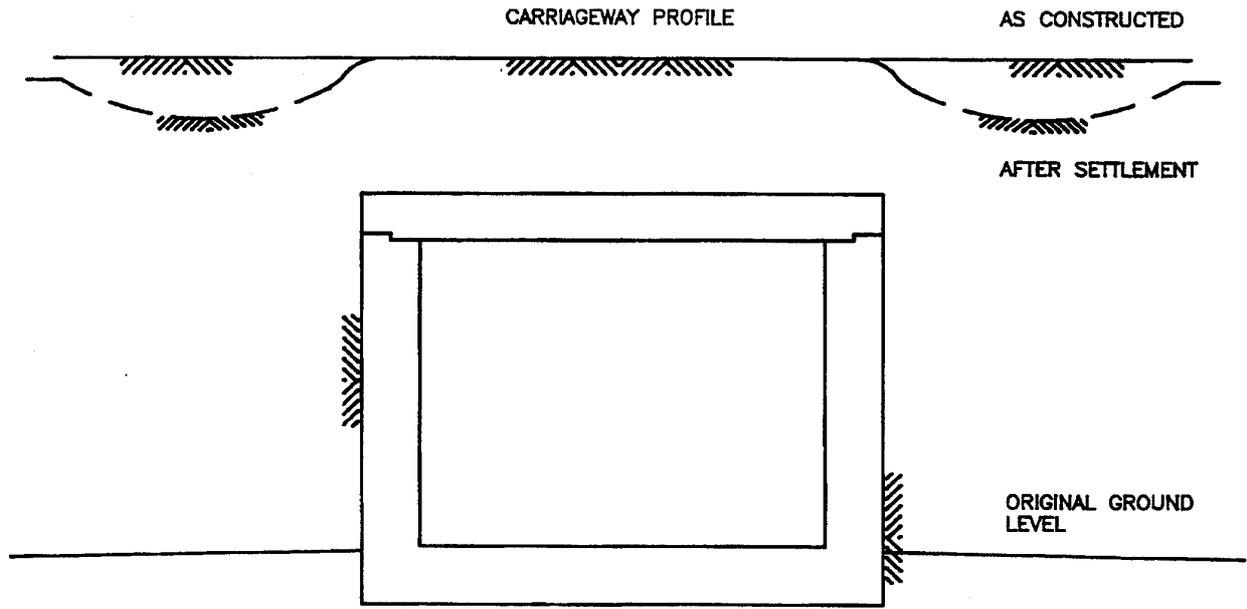
LONGITUDINAL SECTION

Figure 8.26 Earthquake resstraint concrete superstructures

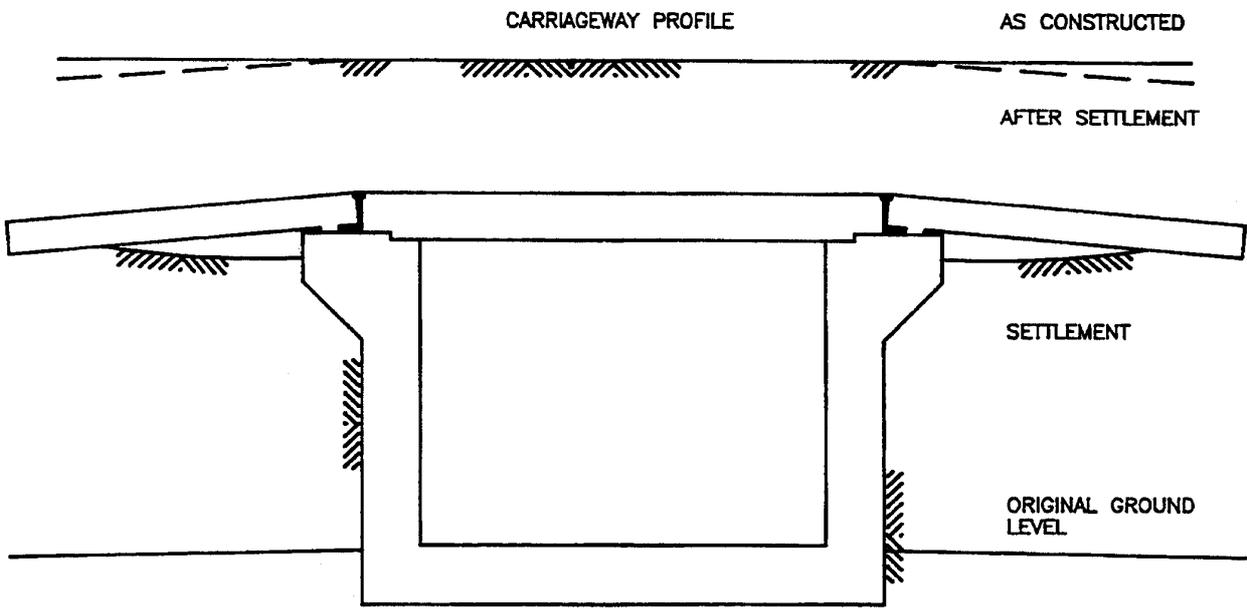


ALL DIMENSIONS IN MILLIMETRES (mm)

Figure 8.27 Earthquake restraint composite superstructures



a) PROFILE WITHOUT RUN-ON SLABS



b) PROFILE WITH RUN-ON SLABS

Figure 8.28 Embankment settlement profiles at a culvert or bridge

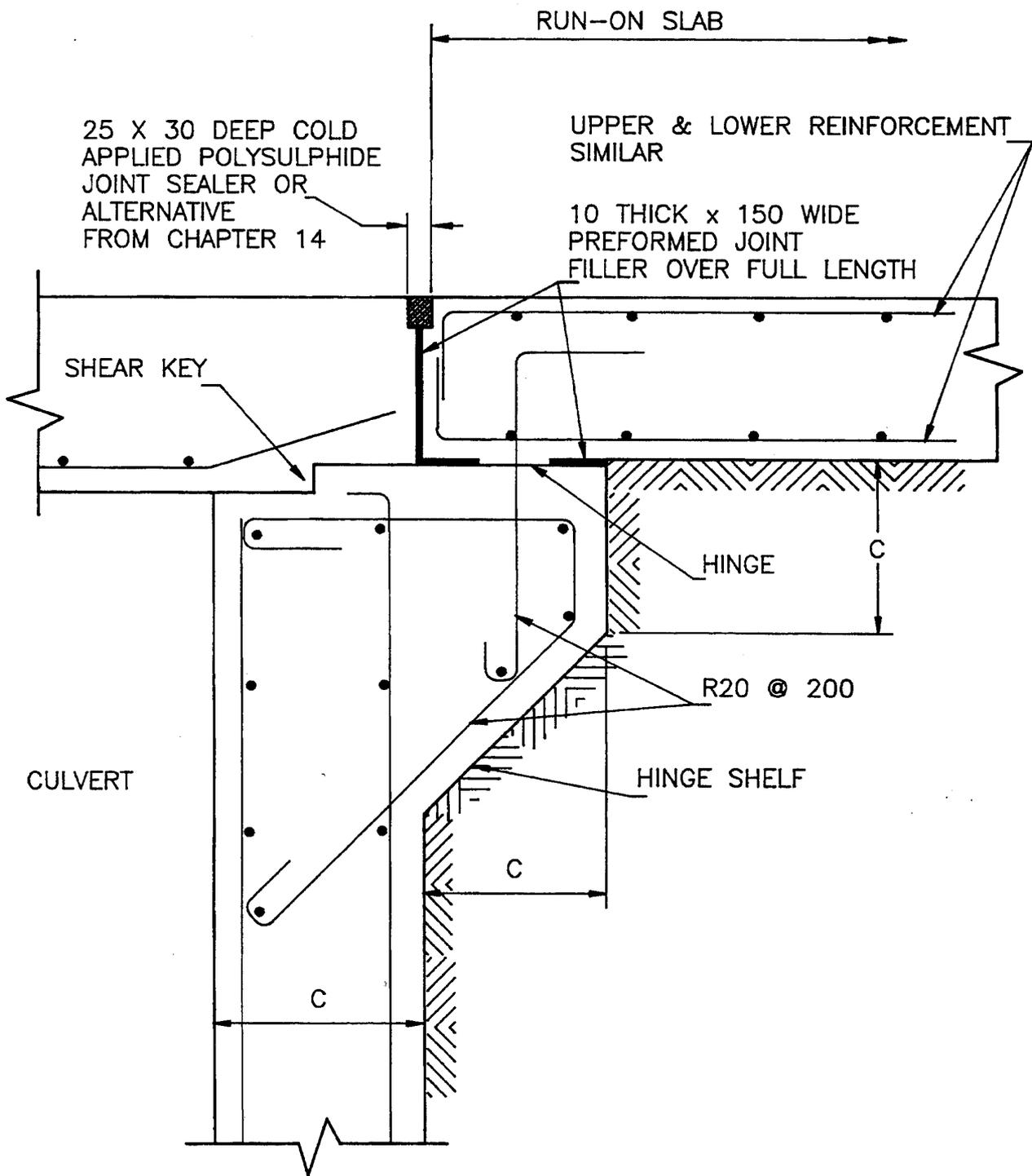


Figure 8.29 Culvert run-on slab

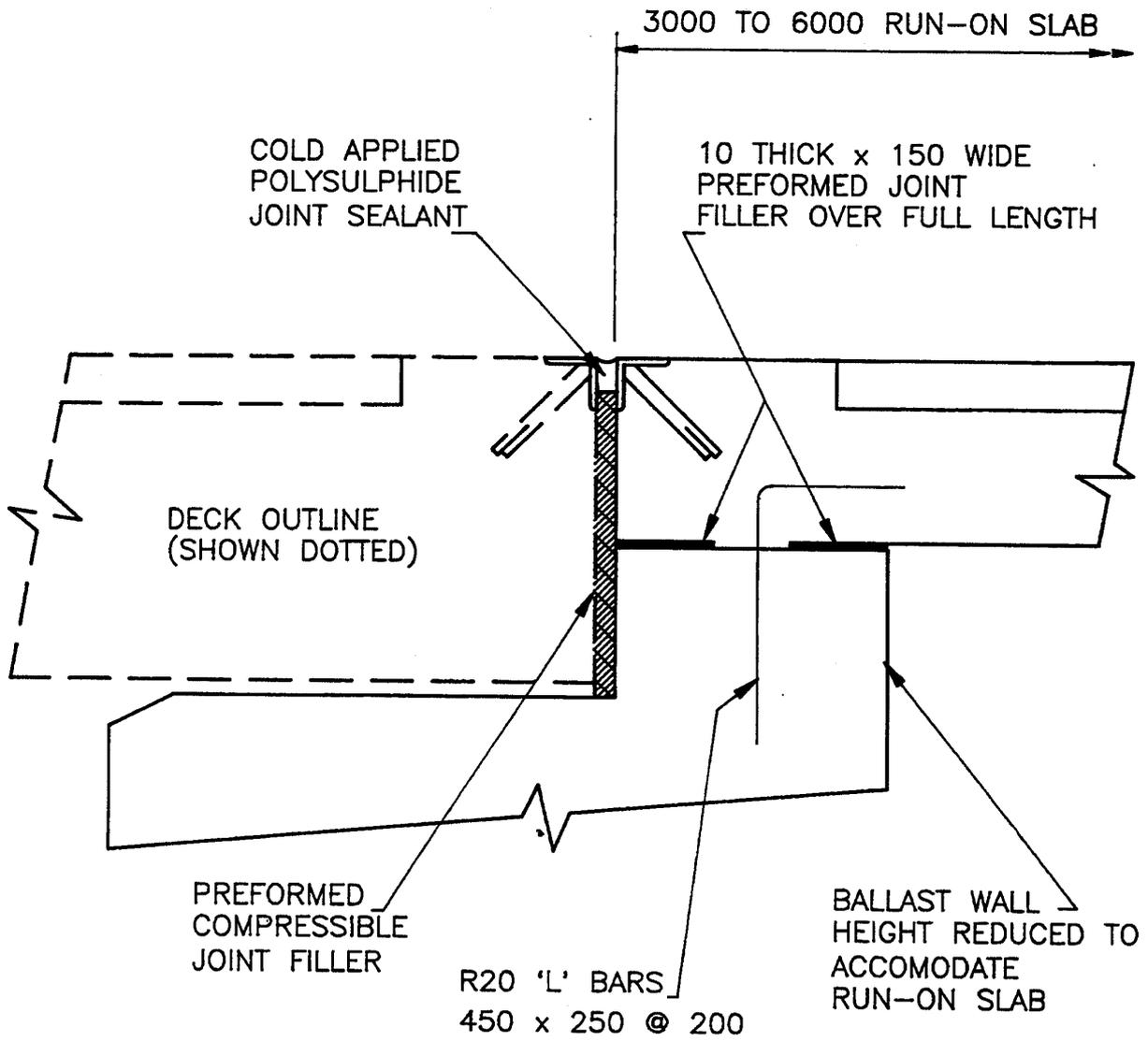


Figure 8.30 Bridge run-on slab

8.10.2 Index of tables

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Abutment mass concrete – mild steel reinforcement in the foundation	8.4
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Abutment, reinforced concrete – high yield steel reinforcement in the foundation	8.8
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Note: The steel reinforcement used in these tables is either:

MS – Plain round mils steel bar, or

HYS – round deformed high tensile steel bar according to the specifications detailed in Section 14.2

Table 8.2 Abutment, mass concrete – foundation sizes (m)

SPAN (m)	HEIGHT H (m)	ALLOWABLE BEARING PRESSURE VALUES (kN/m ²)																							
		400				300				200				150				100							
		A	B	D		A	B	D		A	B	D		A	B	D		A	B	D					
4	2	0.6	0.8	0.5	0.6	0.8	0.5	0.6	0.8	0.5	0.6	0.8	0.5	0.8	0.8	0.5	0.8	0.5	1.2	0.8	0.5				
	3	0.6	0.6	0.5	0.6	0.6	0.5	0.6	0.6	0.5	0.6	0.6	0.5	1.0	0.6	0.5	1.0	0.6	1.5	0.6	0.7				
	4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	1.2	0.5	0.5	1.2	0.5	0.6						
	5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	1.0	0.5	0.5	0.5	0.5												
	6	0.5	0.4	0.5	0.5	0.4	0.5	0.5	1.3	0.4	0.7														
	6.5	0.5	0.3	0.5	0.8	0.3	0.5																		
	6	2	0.6	0.8	0.5	0.6	0.8	0.5	0.6	0.8	0.5	0.6	0.8	0.5	0.9	0.8	0.5	0.9	0.8	1.4	0.8	0.6			
	3	0.6	0.6	0.5	0.6	0.6	0.5	0.6	0.6	0.5	0.6	0.6	0.5	1.1	0.6	0.5	1.1	0.6	1.7	0.6	0.7				
	4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	1.3	0.5	0.5	1.3	0.5	0.7						
	5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	1.2	0.6	0.6	1.7	0.5	0.6	1.7	0.5	0.9						
	6	0.5	0.4	0.5	0.6	0.4	0.5	1.6	0.4	0.8	2.0	0.4	1.0												
	6.5	0.5	0.3	0.5	0.8	0.3	0.5	1.8	0.4	0.9	2.2	0.3	1.0												
8	2	0.5	1.2	0.5	0.5	1.2	0.5	0.6	1.2	0.5	0.6	1.2	0.5	1.0	1.2	0.5	1.0	1.2	1.7	1.2	0.7				
	3	0.5	1.0	0.5	0.5	1.0	0.5	0.8	1.0	0.5	0.8	1.0	0.5	1.3	1.0	0.7	1.3	1.0	1.9	1.0	0.8				
	4	0.5	0.9	0.5	0.5	0.9	0.5	1.1	0.9	0.6	1.6	0.9	0.8	1.6	0.9	0.8	1.6	0.9							
	5	0.5	0.8	0.5	0.6	0.8	0.5	1.3	0.8	0.7	2.0	0.9	1.0	2.0	0.9	1.0	2.0	0.9							
	6	0.5	0.7	0.5	0.8	0.7	0.5	1.6	0.8	0.8	2.4	0.9	1.1	2.4	0.9	1.1	2.4	0.9							
	6.5	0.5	0.7	0.5	1.0	0.7	0.6	1.8	0.8	0.9	2.6	0.9	1.2	2.6	0.9	1.2	2.6	0.9							
	10	2	0.5	1.2	0.5	0.5	1.2	0.5	0.8	1.2	0.5	0.8	1.2	0.5	1.2	1.2	0.5	1.2	1.2	1.8	1.2	0.7			
	3	0.5	1.0	0.5	0.5	1.0	0.5	1.0	1.0	0.5	1.0	1.0	0.5	1.4	1.0	0.7	1.4	1.0							
	4	0.5	0.9	0.5	0.5	0.9	0.5	1.2	0.9	0.6	1.8	1.0	0.9	1.8	1.0	0.9	1.8	1.0							
	5	0.5	0.8	0.5	0.7	0.8	0.5	1.6	0.9	0.8	2.0	1.0	1.0	2.0	1.0	1.0	2.0	1.0							
	6	0.5	0.7	0.5	0.9	0.7	0.6	1.8	0.8	0.9	2.4	0.9	1.1	2.4	0.9	1.1	2.4	0.9							
	6.5	0.5	0.7	0.5	1.1	0.7	0.7	2.0	0.8	1.0	2.6	0.9	1.2	2.6	0.9	1.2	2.6	0.9							
12	2	0.5	1.2	0.5	0.5	1.2	0.5	1.0	1.2	0.5	1.0	1.2	0.5	1.4	1.2	0.7	1.4	1.2	2.0	1.2	0.8				
	3	0.5	1.0	0.5	0.5	1.0	0.5	1.2	1.0	0.5	1.2	1.0	0.6	1.6	1.0	0.8	1.6	1.0							
	4	0.5	0.9	0.5	0.6	0.9	0.5	1.4	0.9	0.7	1.8	1.0	0.9	1.8	1.0	0.9	1.8	1.0							
	5	0.5	0.8	0.5	0.8	0.8	0.5	1.6	0.9	0.8	2.2	0.9	1.0	2.2	0.9	1.0	2.2	0.9							
	6	0.5	0.7	0.5	1.0	0.8	0.6	2.0	0.9	1.0	2.6	0.9	1.2	2.6	0.9	1.2	2.6	0.9							
	6.5	0.5	0.7	0.5	1.2	0.8	0.7	2.2	0.8	0.7	2.8	0.9	1.3	2.8	0.9	1.3	2.8	0.9							

Table 8.8 Abutment, reinforced concrete – high yield steel reinforcement in the foundation (mm)

SPAN (m)	HEIGHT H (m)	ALLOWABLE BEARING PRESSURE VALUES (kN/m ²)																	
		400			300			200			150			100					
		S1	S2	S3	S1	S2	S3	S1	S2	S3	S1	S2	S3	S1	S2	S3			
4	2	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12
	3	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 200	25 @ 200	12	16 @ 200	25 @ 200	12	16 @ 200	25 @ 200	12	16 @ 150	20 @ 150	12
	4	16 @ 200	32 @ 200	16	16 @ 200	32 @ 200	16	16 @ 200	32 @ 200	16	16 @ 200	32 @ 200	16	16 @ 200	32 @ 200	16			
	5	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	16 @ 150	32 @ 150	16									
	6	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20						
	6.5	20 @ 150	40 @ 150	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20						
6	2	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 200	20 @ 200	12	16 @ 150	20 @ 150	16
	3	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	20 @ 150	16
	4	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16			
	5	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16			
	6	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20			
	6.5	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20						
8	2	16 @ 200	25 @ 200	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 175	20 @ 175	12	20 @ 200	20 @ 200	16
	3	16 @ 200	32 @ 200	12	20 @ 200	32 @ 200	12	20 @ 200	32 @ 200	12	20 @ 200	32 @ 200	12	20 @ 175	25 @ 175	12	25 @ 175	25 @ 175	16
	4	16 @ 150	32 @ 150	16	20 @ 200	32 @ 150	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16			
	5	16 @ 150	40 @ 150	16	16 @ 125	40 @ 125	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	20 @ 150	32 @ 150	16			
	6	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20			
	6.5	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20			
10	2	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	20 @ 200	20 @ 200	12	20 @ 200	20 @ 200	12	20 @ 200	20 @ 200	12	16 @ 150	20 @ 150	16
	3	20 @ 200	32 @ 200	12	20 @ 200	32 @ 200	12	20 @ 200	32 @ 200	12	20 @ 200	32 @ 200	12	20 @ 200	25 @ 200	16			
	4	20 @ 150	32 @ 150	16	16 @ 150	32 @ 150	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	25 @ 200	32 @ 200	16			
	5	20 @ 150	40 @ 150	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	25 @ 150	32 @ 150	20	25 @ 150	32 @ 150	20			
	6	16 @ 125	40 @ 125	20	16 @ 125	40 @ 125	20	20 @ 150	40 @ 150	20	25 @ 175	40 @ 175	20	25 @ 175	40 @ 175	20			
	6.5	20 @ 150	40 @ 125	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20			
12	2	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	16	25 @ 200	25 @ 200	16
	3	20 @ 200	32 @ 200	12	20 @ 200	32 @ 200	12	20 @ 175	25 @ 175	12	20 @ 175	25 @ 175	12	25 @ 150	20 @ 150	16			
	4	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	25 @ 200	32 @ 200	16	32 @ 200	32 @ 200	16	32 @ 200	32 @ 200	16			
	5	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	25 @ 175	40 @ 175	16	32 @ 175	32 @ 175	20	32 @ 175	32 @ 175	20			
	6	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	32 @ 150	40 @ 150	20	32 @ 175	40 @ 175	20	32 @ 175	40 @ 175	20			
	6.5	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	32 @ 150	40 @ 150	20	32 @ 150	40 @ 150	20	32 @ 150	40 @ 150	20			

Table 8.9 Abutment, reinforced concrete – mild steel reinforcement in the foundation (mm)

SPAN (m)	HEIGHT H (m)	ALLOWABLE BEARING PRESSURE VALUES (kN/m ²)																	
		400			300			200			150			100					
		S1	S2	S3	S1	S2	S3	S1	S2	S3	S1	S2	S3	S1	S2	S3			
4	2	20 @ 200	25 @ 200	12	20 @ 200	25 @ 200	12	16 @ 200	25 @ 200	12	16 @ 200	25 @ 200	12	16 @ 200	25 @ 200	12	20 @ 150	20 @ 150	16
	3	20 @ 175	32 @ 175	12	16 @ 175	32 @ 175	12	20 @ 175	32 @ 175	12	25 @ 200	32 @ 200	16	20 @ 150	25 @ 150	16			
	4	16 @ 150	32 @ 150	16	16 @ 150	32 @ 150	16	16 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16			
	5	20 @ 175	40 @ 175	20	20 @ 175	40 @ 175	20	25 @ 200	40 @ 200	20	25 @ 200	40 @ 200	20						
	6	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20						
	6.5	20 @ 125	40 @ 125	25	20 @ 125	40 @ 125	20	25 @ 125	40 @ 125	25	20 @ 200	25 @ 200	12	20 @ 200	25 @ 200	12	20 @ 200	20 @ 200	16
6	2	20 @ 200	25 @ 200	12	20 @ 200	25 @ 200	12	20 @ 200	25 @ 200	12	20 @ 175	32 @ 175	12	20 @ 175	32 @ 175	12	25 @ 175	32 @ 175	16
	3	20 @ 175	32 @ 175	12	20 @ 175	32 @ 175	12	20 @ 175	32 @ 175	12	20 @ 175	32 @ 175	12	20 @ 175	32 @ 175	12	20 @ 200	25 @ 200	16
	4	16 @ 150	32 @ 150	16	16 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 125	25 @ 125	16			
	5	20 @ 175	40 @ 175	20	20 @ 175	40 @ 175	20	25 @ 200	40 @ 200	20	25 @ 200	40 @ 200	20	32 @ 200	40 @ 200	20			
	6	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20			
	6.5	20 @ 125	40 @ 125	25	20 @ 125	40 @ 125	25	25 @ 125	40 @ 125	25	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	20 @ 200	25 @ 200	16
8	2	20 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	20 @ 200	25 @ 200	16
	3	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	25 @ 200	32 @ 200	16
	4	16 @ 125	32 @ 125	16	16 @ 125	32 @ 125	16	25 @ 200	32 @ 200	16	25 @ 200	32 @ 200	16	25 @ 150	32 @ 150	20			
	5	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20			
	6	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20	32 & 20 @ 250	40 @ 125	20			
	6.5	20 @ 125	40 @ 125	25	20 @ 125	40 @ 125	25	25 @ 125	40 @ 125	25	25 @ 125	40 @ 125	25	32 & 20 @ 250	40 @ 125	25			
10	2	20 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	20 @ 150	25 @ 150	12	25 @ 200	25 @ 200	16	25 @ 200	25 @ 200	16
	3	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	25 @ 150	25 @ 150	16			
	4	16 @ 125	32 @ 125	16	16 @ 125	32 @ 125	16	25 @ 200	32 @ 200	16	25 @ 200	32 @ 200	16	25 @ 150	32 @ 150	20			
	5	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20	32 @ 200	40 @ 200	20			
	6	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20	32 @ 150	40 @ 150	25			
	6.5	20 @ 125	40 @ 125	25	20 @ 125	40 @ 125	25	25 @ 125	40 @ 125	25	20 @ 150	25 @ 150	12	25 @ 200	25 @ 200	16	25 @ 200	25 @ 200	16
12	2	20 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	16 @ 150	25 @ 150	12	20 @ 150	25 @ 150	12	25 @ 200	25 @ 200	16	25 @ 200	25 @ 200	16
	3	20 @ 150	32 @ 150	16	20 @ 150	32 @ 150	16	25 @ 200	32 @ 200	16	25 @ 200	32 @ 200	16	25 @ 150	25 @ 150	16			
	4	25 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	32 & 25 @ 200	40 @ 200	16	32 & 25 @ 200	40 @ 200	16	32 & 20 @ 150	32 @ 150	20			
	5	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20	25 @ 150	40 @ 150	20	32 & 25 @ 200	40 @ 200	20			
	6	20 @ 125	40 @ 125	20	25 @ 125	40 @ 125	20	32 & 25 @ 125	40 @ 125	20	32 @ 150	40 @ 125	20	32 @ 150	40 @ 150	25			
	6.5	20 @ 125	40 @ 125	25	20 @ 125	40 @ 125	25	32 & 25 @ 125	40 @ 125	25	25 @ 150	25 @ 150	12	32 & 25 @ 125	40 @ 125	25	25 @ 200	25 @ 200	16

Table 8.10 Abutment, reinforced concrete – stem reinforcement (mm)

HEIGHT H (m)	HIGH YIELD STEEL S4 & S5	MILD STEEL S4 & S5
2	16 @ 150	20 @ 175
3	16 @ 150	20 @ 175
4	16 @ 150	20 @ 150
5	20 @ 150	25 @ 150
6	25 @ 150	32 @ 150
6.5	32 @ 150	40 @ 175

Table 8.11 Retaining wall, reinforced concrete – stem reinforcement (mm)

HEIGHT H (m)	HIGH YIELD STEEL REINFORCEMENT						MILD STEEL REINFORCEMENT					
	S1	S2	S3	S4	S5	S6	S1	S2	S3	S4	S5	S6
2	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150
3	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150
4	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	20 @ 150	20 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150
5	20 @ 150	20 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	25 @ 150	25 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150
6	25 @ 175	25 @ 175	16 @ 175	16 @ 175	16 @ 175	16 @ 150	32 @ 175	32 @ 175	16 @ 175	16 @ 175	16 @ 175	16 @ 150
6.5	25 @ 150	25 @ 150	16 @ 150	16 @ 150	16 @ 150	16 @ 150	32 @ 125	32 @ 125	16 @ 125	16 @ 125	16 @ 125	16 @ 150

Table 8.12 Retaining wall, reinforced concrete – foundation sizes (m)

HEIGHT H (m)	ALLOWABLE BEARING PRESSURE VALUES (kN/m ²)																											
	400				300				200				150				100				75							
	A	B	D		A	B	D		A	B	D		A	B	D		A	B	D		A	B	D					
1	0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5	
2	0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5		0.5	2.2	0.5	
3	0.5	2.6	0.6		0.5	2.6	0.6		0.5	2.6	0.6		0.5	2.6	0.6		0.8	2.6	0.6		0.8	2.6	0.6		1.2	2.6	0.6	
4	0.6	3.2	0.8		0.6	3.2	0.8		0.6	3.2	0.8		0.8	3.2	0.8		1.2	3.2	0.8		1.2	3.2	0.8		1.8	2.9	0.8	
5	0.5	3.7	1.0		0.5	3.7	1.0		0.6	3.7	1.0		1.1	3.7	1.0													
6	0.5	4.2	1.1		0.5	4.2	1.1		1.1	3.7	1.1		1.7	3.7	1.1													
6.5	0.5	4.6	1.2		0.5	4.6	1.2		1.4	4.6	1.2		2.0	4.6	1.2													

Table 8.13 Retaining wall, reinforced concrete – foundation reinforcement (mm)

HEIGHT H (m)	ALLOWABLE BEARING PRESSURE VALUES (kN/m ²)																								
	400				300				200				150				100				75				
	S7	S8	S9		S7	S8	S9		S7	S8	S9		S7	S8	S9		S7	S8	S9		S7	S8	S9		
HIGH YIELD STEEL	2	16 @ 150	16 @ 150	12	16 @ 150	16 @ 150	12	16 @ 150	16 @ 150	12	16 @ 150	16 @ 150	12	16 @ 150	16 @ 150	12	16 @ 150	16 @ 150	12	16 @ 150	16 @ 150	12	16 @ 150	16 @ 150	12
	3	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12	16 @ 150	20 @ 150	12
	4	16 @ 150	25 @ 150	16	16 @ 150	25 @ 150	16	16 @ 150	25 @ 150	16	16 @ 150	25 @ 150	16	16 @ 150	25 @ 150	16	16 @ 150	25 @ 150	16	16 @ 150	25 @ 150	16	16 @ 150	25 @ 150	16
	5	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16	20 @ 200	32 @ 200	16
	6	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16	20 @ 200	40 @ 200	16
	6.5	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16	20 @ 175	40 @ 175	16
MILD STEEL	2	20 @ 150	20 @ 150	12	20 @ 150	20 @ 150	12	20 @ 150	20 @ 150	12	20 @ 150	20 @ 150	12	20 @ 150	20 @ 150	12	20 @ 150	20 @ 150	12	20 @ 150	20 @ 150	12	20 @ 150	20 @ 150	12
	3	20 @ 150	25 @ 150	16	20 @ 150	25 @ 150	16	20 @ 150	25 @ 150	16	20 @ 150	25 @ 150	16	20 @ 150	25 @ 150	16	20 @ 150	25 @ 150	16	20 @ 150	25 @ 150	16	20 @ 150	25 @ 150	16
	4	16 @ 175	32 @ 175	16	16 @ 175	32 @ 175	16	16 @ 175	32 @ 175	16	16 @ 175	32 @ 175	16	16 @ 175	32 @ 175	16	16 @ 175	32 @ 175	16	16 @ 175	32 @ 175	16	20 @ 175	32 @ 175	16
	5	20 @ 200	40 @ 200	20	20 @ 200	40 @ 200	20	20 @ 200	40 @ 200	20	20 @ 200	40 @ 200	20	20 @ 200	40 @ 200	20	20 @ 200	40 @ 200	20	20 @ 200	40 @ 200	20	20 @ 200	40 @ 200	20
	6	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20	20 @ 150	40 @ 150	20
	6.5	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20	20 @ 125	40 @ 125	20

Table 8.14 Raft foundation – depth and reinforcement

SPAN (m)	HEIGHT H (m)	ALLOWABLE BEARING PRESSURE (kN/m ²)	DEPTH D (m)	HIGH YIELD STEEL REINFORCEMENT (mm)			MILD STEEL REINFORCEMENT (mm)		
				S1	S2	S3	S1	S2	S3
4	2	90	0.5	16 @ 150	12 @ 150	12 @ 150	20 @ 200	16 @ 200	12 @ 150
	3	105	0.6	16 @ 150	12 @ 150	12 @ 150	20 @ 150	12 @ 150	16 @ 175
	4	120	0.7	20 @ 150	12 @ 150	16 @ 200	25 @ 175	16 @ 175	16 @ 175
	5	140	0.8	20 @ 150	16 @ 150	16 @ 200	25 @ 150	16 @ 150	16 @ 150
	6	160	0.8	25 @ 175	16 @ 175	16 @ 200	32 @ 175	16 @ 175	16 @ 150
	6.5	170	0.9	25 @ 175	16 @ 175	16 @ 175	32 @ 175	16 & 20 @ 175	16 @ 150
6	2	85	0.7	20 @ 150	12 @ 150	16 @ 200	25 @ 175	16 @ 175	16 @ 175
	3	95	0.7	25 @ 200	16 @ 200	16 @ 200	32 @ 200	20 @ 200	16 @ 175
	4	110	0.8	25 @ 175	16 @ 175	16 @ 175	32 @ 175	16 @ 175	16 @ 150
	5	130	1.0	25 @ 175	16 @ 175	16 @ 150	32 @ 175	16 & 20 @ 175	20 @ 200
	6	150	1.1	25 @ 150	16 @ 150	16 @ 150	32 @ 150	20 @ 150	20 @ 175
	6.5	160	1.1	32 @ 200	20 @ 200	16 @ 150	40 @ 200	20 @ 200	20 @ 175
8	2	80	0.8	25 @ 200	16 @ 200	16 @ 200	32 @ 175	16 @ 175	16 @ 150
	3	95	0.9	25 @ 150	16 @ 150	16 @ 150	32 @ 175	20 @ 175	16 @ 150
	4	110	1.1	25 @ 150	16 @ 150	16 @ 150	32 @ 150	20 @ 150	20 @ 175
	5	125	1.2	25 & 32 @ 175	16 @ 150	20 @ 200	32 & 40 @ 175	20 @ 175	20 @ 175
	6	145	1.3	32 @ 200	20 @ 200	20 @ 200	40 @ 175	20 @ 175	20 @ 150
	6.5	155	1.4	32 @ 175	20 @ 175	20 @ 175	40 @ 150	20 @ 150	20 @ 150
10	2	85	1.0	25 @ 150	16 @ 150	16 @ 150	32 @ 150	16 @ 150	20 @ 200
	3	95	1.1	25 & 32 @ 175	20 @ 175	16 @ 150	32 & 40 @ 150	20 @ 175	20 @ 175
	4	110	1.2	32 @ 200	20 @ 200	20 @ 200	32 & 40 @ 150	20 @ 150	20 @ 175
	5	125	1.4	32 @ 175	20 @ 175	20 @ 175	40 @ 175	20 & 25 @ 175	20 @ 150
	6	140	1.5	32 @ 150	20 @ 150	20 @ 175	40 @ 150	20 @ 150	25 @ 200
	6.5	150	1.6	32 @ 150	20 @ 150	20 @ 150	40 @ 150	20 & 25 @ 150	25 @ 200
12	2	85	1.2	32 @ 200	20 @ 200	20 @ 200	32 & 40 @ 175	20 @ 175	20 @ 175
	3	95	1.3	32 @ 175	20 @ 175	20 @ 200	40 @ 175	20 @ 175	20 @ 150
	4	110	1.4	32 @ 150	20 @ 150	20 @ 175	40 @ 150	20 @ 150	20 @ 150
	5	125	1.6	32 @ 150	20 @ 150	20 @ 150	40 @ 150	25 @ 200	25 @ 200
	6	140	1.7	32 & 40 @ 175	20 @ 175	20 @ 150	40 @ 125	20 @ 125	25 @ 175
	6.5	150	1.8	40 @ 200	20 @ 200	20 @ 150	40 @ 125	20 @ 125	25 @ 175

Table 8.15 Pier stem reinforcement (mm)

HEIGHT H (m)	HIGH YIELD STEEL REINFORCEMENT						MILD STEEL REINFORCEMENT					
	S1	S2	S3	S4	S5	S6	S1	S2	S3	S4	S5	S6
2	20 @ 150	16 @ 175	16 @ 175	16 @ 150	16 @ 150	20	20 @ 150	16 @ 150	16 @ 150	20 @ 150	20 @ 150	20
3	20 @ 150	16 @ 175	16 @ 175	16 @ 150	16 @ 150	20	25 @ 150	16 @ 150	16 @ 150	20 @ 150	20 @ 150	25
4	20 @ 150	16 @ 175	16 @ 175	16 @ 150	16 @ 150	20	25 @ 150	16 @ 150	16 @ 150	20 @ 150	20 @ 150	25
5	25 @ 150	16 @ 175	16 @ 175	16 @ 150	16 @ 150	25	32 @ 200	16 @ 150	16 @ 150	20 @ 200	20 @ 200	32
6	25 @ 150	16 @ 175	16 @ 175	16 @ 150	16 @ 150	25	32 @ 150	16 @ 150	16 @ 150	20 @ 150	20 @ 150	32
6.5	25 @ 150	16 @ 175	16 @ 175	16 @ 150	16 @ 150	25	32 @ 150	16 @ 150	16 @ 150	20 @ 150	20 @ 150	32

Table 8.16 Pier foundation sizes (m)

SPAN (m)	HEIGHT H (m)	ALLOWABLE BEARING PRESSURE VALUES (kN/m ²)														
		400			300			200			150			100		
		A	D	A	D	A	D	A	D	A	D	A	D			
8	2	1.6	1.0	1.6	1.0	1.6	1.0	1.6	1.0	2.0	1.1	3.0	1.4			
	3	1.8	1.0	1.8	1.0	1.8	1.0	1.8	1.0	2.2	1.2	3.2	1.5			
	4	1.8	1.0	1.8	1.0	1.8	1.0	1.8	1.0	2.4	1.2	3.5	1.5			
	5	2.0	1.1	2.0	1.1	2.0	1.1	2.0	1.1	2.6	1.3					
	6	2.0	1.2	2.0	1.2	2.2	1.2	2.2	1.2	2.8	1.3					
	6.5	2.2	1.2	2.2	1.2	2.2	1.2	2.2	1.2	2.8	1.3					
10	2	1.4	1.0	1.4	1.0	1.6	1.0	1.6	1.0	2.2	1.2	3.2	1.5			
	3	1.4	1.0	1.4	1.0	2.0	1.2	2.4	1.2	2.4	1.2	3.5	1.6			
	4	1.6	1.0	1.6	1.0	2.0	1.2	2.6	1.2	2.6	1.3	3.8	1.7			
	5	1.6	1.0	1.6	1.0	2.2	1.2	2.8	1.2	2.8	1.4					
	6	1.8	1.2	1.8	1.2	2.3	1.2	3.0	1.2	3.0	1.4					
	6.5	1.8	1.2	1.8	1.2	2.4	1.2	3.0	1.2	3.0	1.4					
12	2	1.0	0.9	1.2	1.0	1.8	1.0	1.8	1.2	2.4	1.2	3.6	1.8			
	3	1.2	1.0	1.5	1.2	2.0	1.3	2.6	1.3	2.6	1.4	4.0	2.0			
	4	1.4	1.1	1.5	1.2	2.2	1.3	2.8	1.3	2.8	1.4	4.5	2.2			
	5	1.4	1.2	1.6	1.2	2.4	1.4	3.0	1.4	3.0	1.5	4.5	2.2			
	6	1.5	1.2	1.8	1.2	2.5	1.4	3.2	1.4	3.2	1.6					
	6.5	1.6	1.2	1.8	1.2	2.5	1.4	3.4	1.4	3.4	1.6					

Appendix A8

Appendix A8.1

Example of bearing pressure calculation

Determine the allowable bearing pressure of a 2.5m wide foundation, bearing on dry sand at a depth of 1.2m below ground level, Figure A8.1. The average N value measured in boreholes over a depth of 2.5m (ie foundation width) below foundation level is 17.

Now the average overburden pressure (over a depth equal to the foundation width below the foundation) =
Overburden density x Average depth of penetration test x Gravitational force.

Assuming an Overburden density = 1.8 Mg/m^3 ,

Average depth = foundation depth + (0.5 x foundation width)

$$\begin{aligned} &= 1.2\text{m} + (0.5 \times 2.5\text{m}) \\ &= 2.45\text{m} \end{aligned}$$

Hence the average overburden pressure = $1.8 \times 2.45 \times 9.8 = 43 \text{ kN/m}^2$

From Figure 8.5 the correction factor is 2.0, so the N value to be used in Figure 8.4 is 17×2.0 , ie 34.

Reading up from the Br value, 2.5, in Figure 8.4 to an N value of 34, the allowable bearing pressure is 350 kN/m^2 .

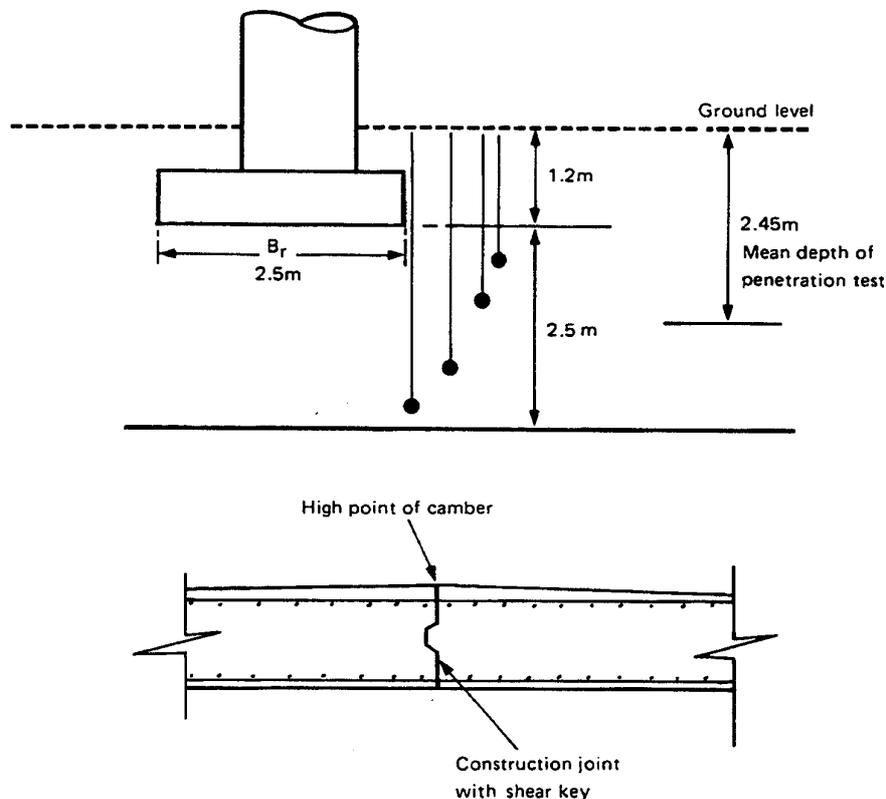


Figure A8.1 Bearing pressure calculation

9 CONCRETE SUPERSTRUCTURES

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9 Concrete superstructures

Concrete will often be the natural choice of material for the bridges on roads where a long service life and minimum maintenance are required. Not all concrete structures are trouble-free, however, and it is important for engineers and contractors to understand the factors which influence concrete durability. It is assumed here that the engineer is familiar with good concrete practice and that he and guidelines in Section 14.1.

A solid concrete slab provides an economical and practical solution for spans up to 12m. It has a number of advantages over the concrete beam-and-slab form, due mainly to its simpler construction. This makes it easier to obtain both good compaction around the reinforcement and a dense surfacing which prevents the ingress of pollutants, both advantages leading to greater durability. Solid slab decks also show better load distribution qualities than beam and slab forms.

This chapter gives standard designs for concrete slab decks from 4m to 12m spans for one, one and a half, and two lanes of traffic, for both BS-HA and HS20-44 loadings, using mild steel (MS) and high yield steel (HYS) reinforcement. Span lengths are measured between bearing centres as shown Figure 8.18 8.22. Traffic loadings and bridge width are discussed in Sections 2.5 and 2.6.

9.1 Standard designs

The standard designs at the end of this chapter conform to either BSI (1990a) or AASHTO (1983) (Section 2.8). These designs are conservative in strength and may be modified in detail, but structural changes should be made only by a qualified civil or structural engineer.

9.2 Design details

9.2.1 Vertical profile over the bridge

The vertical profile of the road over the bridge can be determined once the high flood level, floating debris

or navigation clearance and slab depth are known. If the deck level is required to be higher than the adjacent road, the superstructure will describe a hogging vertical curve over the bridge. This curve should be designed to the applicable highway standards, Odier et al (1974). The slab should be cast to follow this profile: thus a longitudinal section through the bridge would show a constant depth of slab and surfacing, (Figure 9.1). This detail must be made clear on the drawings, otherwise there is a risk that the contractor will cast the soffit horizontally and make up the vertical profile by varying the thickness of the surfacing, which would add excessive deadload to the deck.

9.2.2 Bearings

The simplest form of bearing is made by casting the concrete slab onto the abutment bearing shelf, with only a layer of bitumen felt separating the two concrete surfaces. This may be satisfactory for very short spans, say up to 6m, but for longer spans there is a risk that thermal movement will damage both the supporting structure and the slab. For this reason, simple rubber strip bearings are shown supporting the slab in Figures 9.2 and 9.3

These bearings consist of discrete strips of black natural rubber (Section 14.6), extending over the full width of the slab soffit at the support point, with a maximum width of approximately 300mm and a maximum thickness of approximately 25mm. At the free end of the span, the bridge deck locates by friction between the rubber strip and the concrete deck, with no positive mechanical means to develop resistance against transverse loadings. At the fixed end, dowel bars passing through the pad at frequent intervals provide the necessary restraint, both longitudinally and transversely. To allow rotations to occur and the deck to expand laterally, the dowel bars are usually fitted with rubber caps where they pass into the concrete deck slab.

9.2.3 Expansion joints

An expansion joint is required at each end of each span. Though the fixed end moves less than the free end, similar joints are usually installed at both.

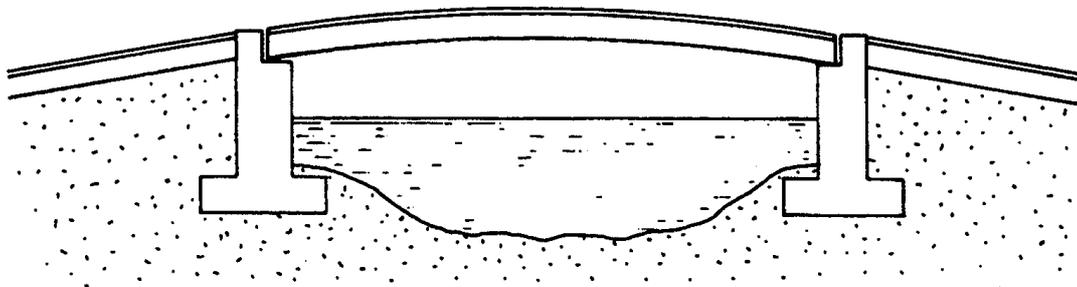


Figure 9.1 Bridge with hogging vertical profile

Figure 9.2 shows a suitable joint between adjacent slabs of a multi-span bridge. This same type of joint can be used between the abutments and deck slab (Figure 9.3), one end of the deck being doweled and the other free. Flexible joint seals at the road surface are not recommended for bridges on gravel roads; Section 14.7 describes a more suitable alternative.

9.2.4 Construction joints

Construction joints often provide paths for the seepage of water, leading to leaching of lime from the concrete and corrosion of the reinforcement. For this reason, it is preferable for the slab to be cast in one continuous pour. If the engineer knows that this will not be possible because the batch volume required is too much for one day's working, it is better for him to mark on the drawings a permissible location for construction joints. If a joint is unavoidable, it should be parallel to the centre line and preferably located at the high point of the transverse camber (Figure 9.16).

9.2.5 Slab reinforcement

The maximum length of reinforcing bar generally available is 12m. When a longer bar is needed, eg for a 12m span, two bars must be lapped. As a general rule the lap length should be at least 40 times the diameter of the bar. The main bars should not be lapped near the centre of the span, and laps should be staggered so that they do not cause a line of weakness across the slab. The lapping of bars should be avoided whenever possible so as to ensure good continuity, minimise congestion of reinforcement and so produce better compaction of concrete around the steel.

The anchorage and positioning and reinforcement are particularly important where the slab sits on its bearings. For the standard designs all bars have been continued to the supports. Main bars are L-bars, marked C in Figures 9.4 to 9.15, and are positioned with the hooks at alternate ends. It is important to position the 90 degree hook so that the bend begins behind the bearing, as shown on the drawings.

Accurate positioning of the reinforcement is essential in order to maintain the minimum cover of 50mm of well compacted concrete, and so avoid moisture and pollutants penetrating the slab and corroding the steel reinforcement.

9.2.6 Drainage

If the bridge is single span with a hogging vertical curve and the joints between deck and abutments are sealed, surface water can be allowed to run off the bridge into the road drainage channels and no drains are required on the bridge superstructure.

If the bridge is multi-span, or has a flat or sagging

profile with a low point on the bridge, drains must be installed on the deck. The simplest way is to provide a 100mm diameter PVC pipe at the kerb line, dishing the concrete around it, and setting a metal gully grating in the surfacing (Figure 9.16). The engineer must take care to see that the discharge does not cause damage to services or sub-structures, and ensure that close attention is given to the compaction of the deck concrete around the pipe.

9.2.7 Parapets

Figure 9.17 shows details of a suitable steel parapet system. These follow the general principles of the Highways Agency (1993) but circular sections have been selected for the rails since they are more readily available in most countries. Badly damaged posts or rails can be unbolted and replaced. If proprietary square section beams are available, they should be used because they cause less damage to vehicles on impact. 'W section' guardrails may also be used to good effect.

Provision should be made for differential expansion and contraction between steel and concrete elements, particularly at the superstructure expansion joints.

To reduce the danger of a rail penetrating a vehicle on impact, parapet rails should present a continuous face to traffic, even at expansion joints.

Care should be taken with details of the holding-down fixing into the concrete plinth. As well as installing the metre-long bars through the U-bars, two extra links (reinforcement bar marked D in Figures 9.3 to 9.14) should be used at each post position, one each side close to the U-bars. Bridge parapets usually extend several metres beyond the superstructure at each end, and may be supported by the wing walls, in order to provide a barrier at the approaches.

9.2.8 Surfacing

The standard slabs are designed to be used with 75mm of asphalt surfacing. This provides some degree of waterproofing and protection of the surface of the structural slab. In some countries it is common practice to use the upper surface of the slab as a running surface. This is not recommended, but should the engineer wish to avoid using asphalt, the minimum cover to reinforcement in the top of the slab should be increased to 75mm and kerb joint and drainage details have to be modified accordingly. Bridges serving gravel roads may need more than 75mm cover to counteract the abrasive action of stones embedded in vehicle tyres.

9.2.9 Services carried on the bridge

It will sometimes be necessary to make provision for services and utilities such as electricity or telephone

cables to be carried on the bridge. These services can be readily accommodated in PVC ducts cast into the verges behind the kerbs, as shown in Figure 9.16. The ducts cost very little and could be included even if there were no immediate need. If steel pipes are attached to the deck, they must have provision for expansion at the fixings.

9.2.10 Provision for pedestrians and cyclists

Wherever there is likely to be significant movement of pedestrians and cyclists across the bridge, a segregated pathway is recommended (Section 2.7). This pathway can be provided by an addition cantilevered out from the bridge superstructure; but as noted in Section 2.7, such additions tend to be less robust than the main structure and there is a possibility of critical asymmetric overloading of the deck, should a large number of people congregate on the pathway.

A more durable, safer and often simpler solution is shown in Figure 9.18: widening the deck by 2m beyond the width required for motorised traffic, protecting the pedestrians and cyclists with the standard parapet, and installing a lighter parapet at the edge of the walkway.

Comparison of the cross sections through the single, one and a half, and two lane decks shows that the

construction is identical in terms of concrete and steel. Only the width is changed. Additions can be made for pedestrians and cyclists on one or both sides by extending the same section and maintaining the spacing of the steel reinforcement.

A wider superstructure will require wider abutments and piers. Should these additional features prove to be too expensive, a separate structure might be considered as an acceptable means of providing a safe crossing for non-motorised travellers.

9.3 References

AASHTO (1983). *Standard Specification for Highway Bridges. 13th edition.* American Association of State Highway and Transportation Officials, Washington.

BSI (1990a). *Steel, concrete and composite bridges Part 4. Code of practice for design of concrete bridges. BS 5400 Pt 4: 1990.* British Standards Institution, London.

Highways Agency (1993). *BD 52/93: The design of highway bridge parapets.* Department of Transport, London.

Odier et al (1974). *Low Cost Roads – design, construction and maintenance.* UNESCO publication by Butterworth, London,

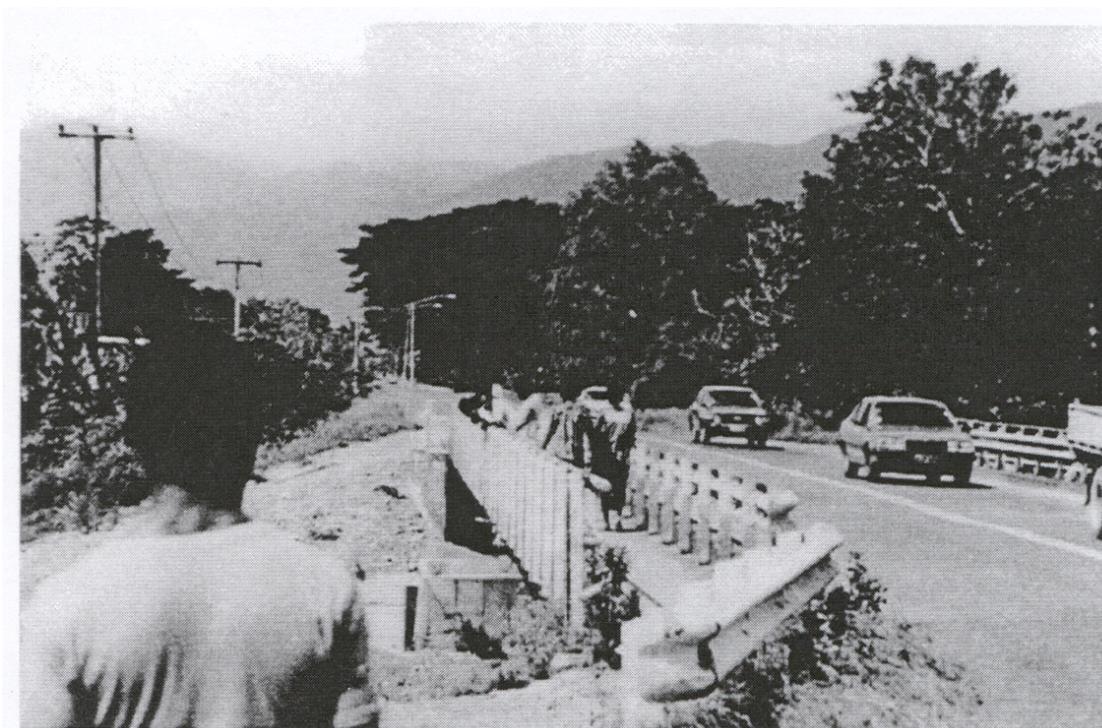


Figure 9.18 Segregated walkway

9.4 Standard drawings and tables for concrete superstructures

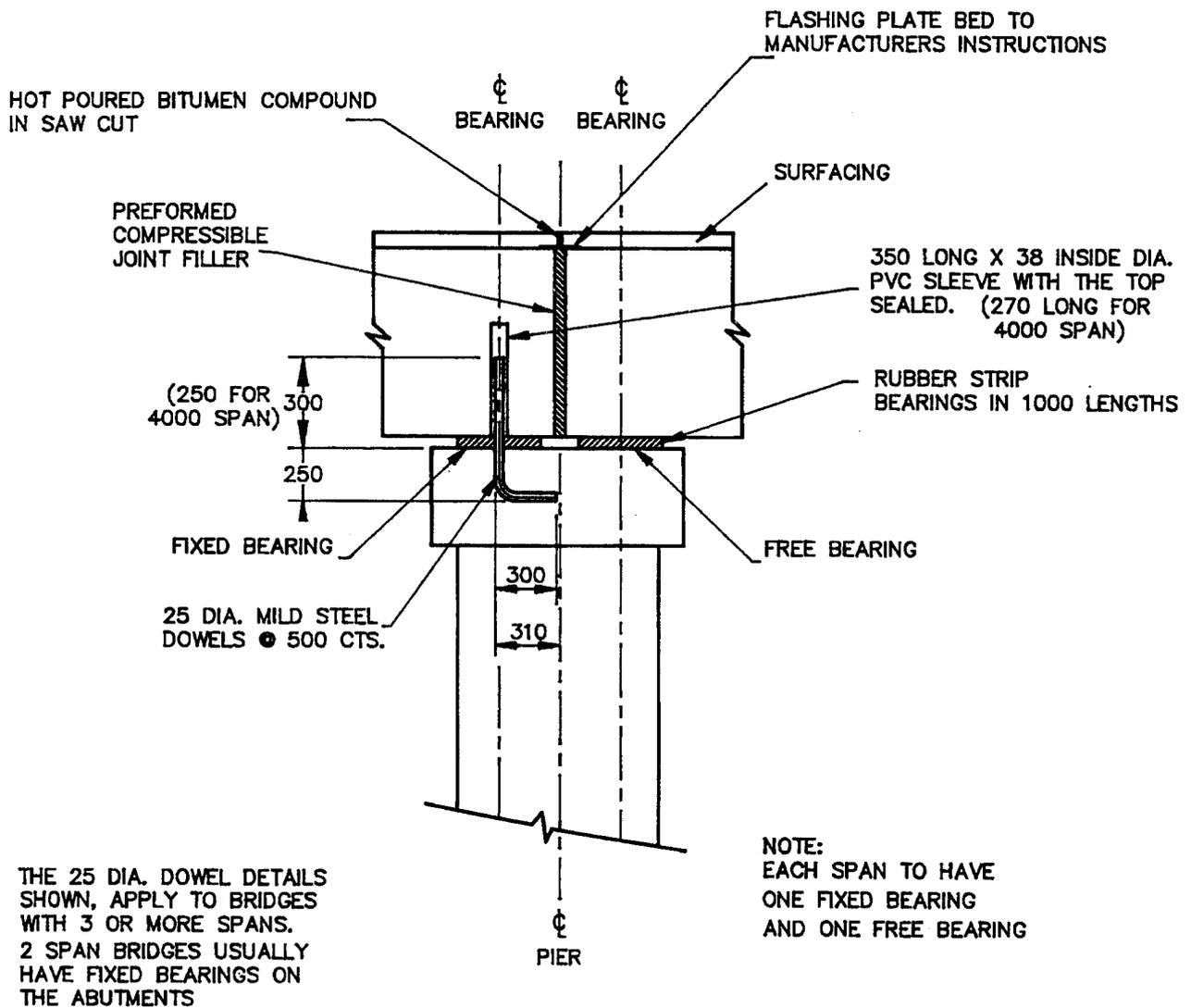
<i>Title</i>			<i>Figure number</i>
Pier expansion joint and bearing details -----			9.2
Abutment expansion joint and bearing details -----			9.3
<i>Bridge design loading</i>	<i>Bridge width</i>	<i>Reinforcement</i>	
BS-HA	1 lane	MS -----	9.4
		HYS -----	9.5
	1½ lanes	MS -----	9.6
		HYS -----	9.7
	2 lanes	MS -----	9.8
		HYS -----	9.9
HS20-44	1 lane	MS -----	9.10
		HYS -----	9.11
	1½ lanes	MS -----	9.12
		HYS -----	9.13
	2 lanes	MS -----	9.14
		HYS -----	9.15
Ducts, drainage and construction joints -----			9.16
Parapets -----			9.17

In the following drawings, the steel reinforcement is either

MS - plain round mild steel bar or

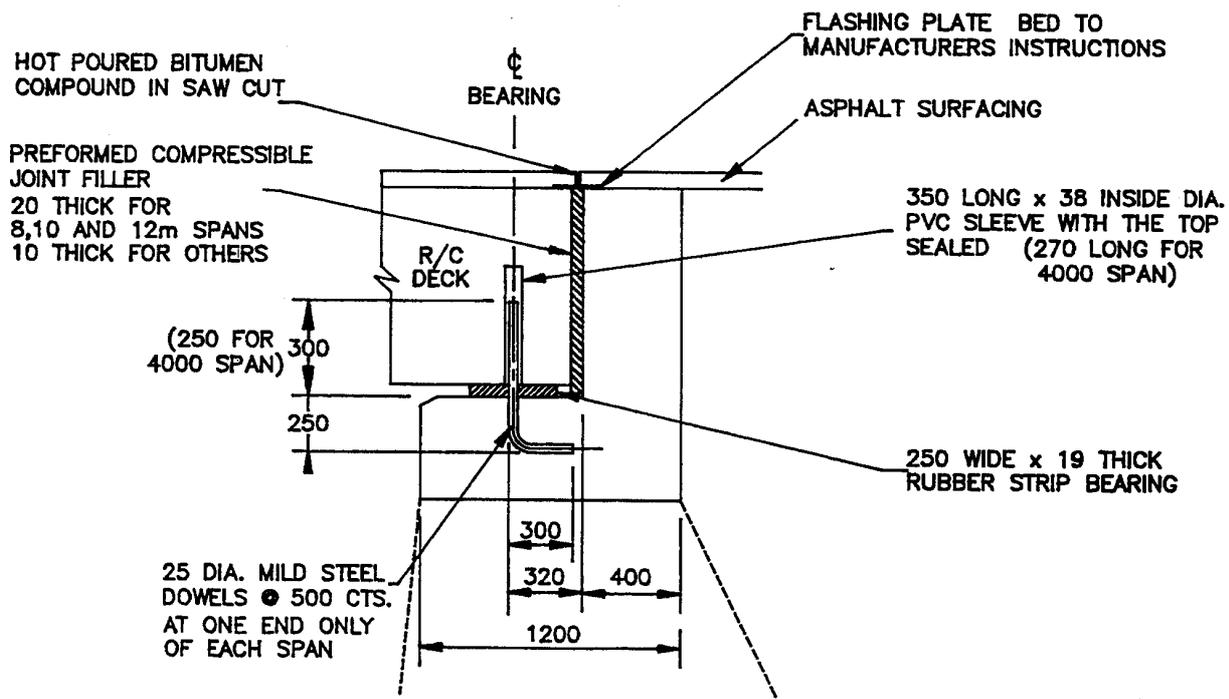
HYS - round deformed high tensile steel bar, according to the specifications detailed in Section 14.2

A wearing course of asphaltic concrete 75mm thick is specified. Any change to this may require corresponding changes to the abutments.

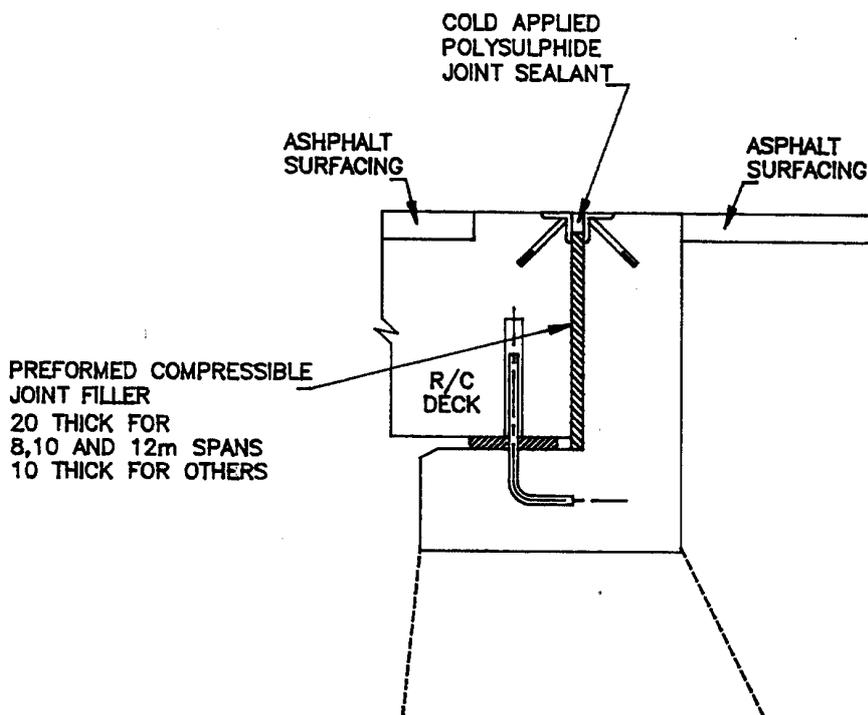


ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 9.2 Concrete superstructure, pier expansion joint and bearing details

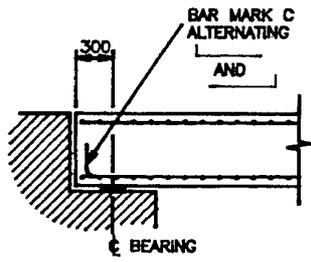


ALL DIMENSIONS IN MILLIMETRES (mm).



FOR COMPOSITE SUPERSTRUCTURE BEARING DETAILS SEE FIG.10.12.

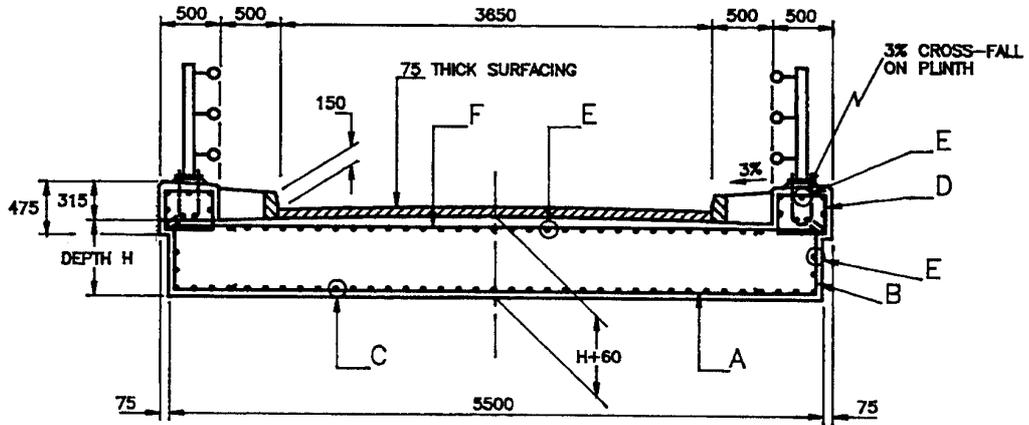
Figure 9.3 Concrete superstructure, abutment expansion joint and bearing details



DETAIL AT SUPPORT

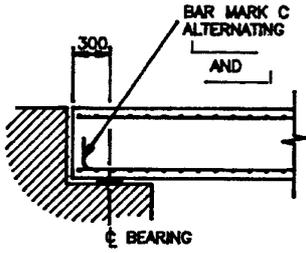
BS.HA.LOAD	✓
HS20-44	
SINGLE LANE	✓
1½ LANES	
2 LANES	
MILD STEEL REINFORCEMENT	✓
HIGH YIELD STEEL REINFORCEMENT	

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



SPAN(m)	BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)				E	F	DEPTH H (mm)
		A	B	C	D			
4	16 @ 125	12 @ 200	32 @ 175	16 @ 200	12 @ 200	12 @ 200	300	
6	16 @ 125	12 @ 200	32 @ 150	16 @ 200	12 @ 200	12 @ 200	400	
8	16 @ 125	16 @ 200	32 @ 100	16 @ 200	16 @ 200	16 @ 200	500	
10	16 @ 150	16 @ 200	32 @ 100	16 @ 200	16 @ 200	16 @ 200	650	
12	16 @ 150	16 @ 200	32 @ 90	16 @ 200	16 @ 200	16 @ 200	750	

Figure 9.4 Reinforced concrete deck slab



DETAIL AT SUPPORT

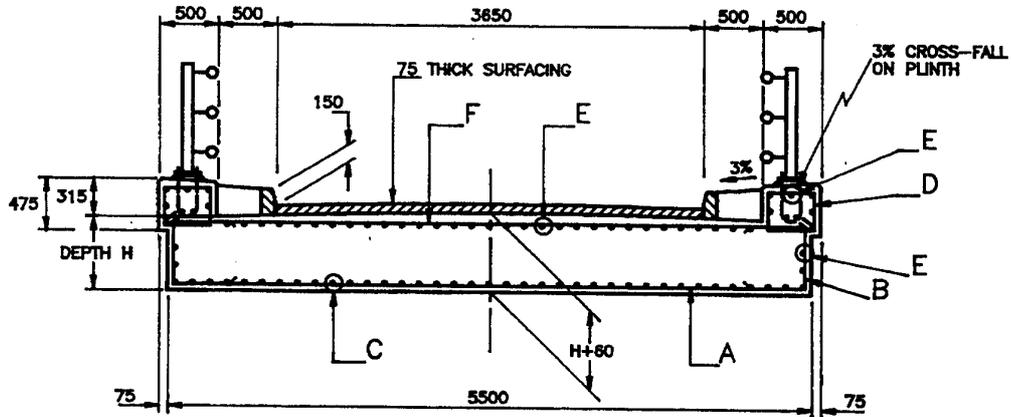
BS.HA.LOAD	✓
HS20-44	
SINGLE LANE	✓
1½ LANES	
2 LANES	
MILD STEEL REINFORCEMENT	
HIGH YIELD STEEL REINFORCEMENT	✓

NOTES :

ALL DIMENSIONS IN MILLIMETRES (mm).

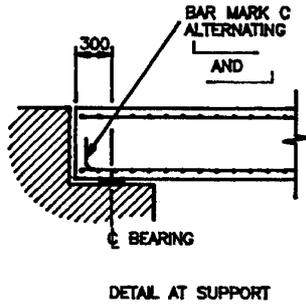
MIN. COVER TO REINFORCEMENT - 50

CHARACTERISTIC CONCRETE CUBE STRENGTH AT 28 DAYS - 30 N/mm²



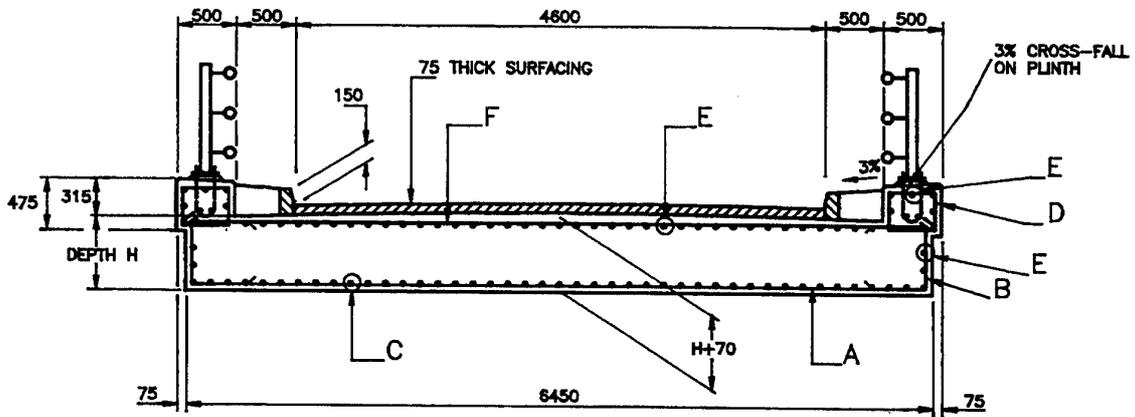
SPAN(m)	BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)				DEPTH H (mm)
		A	B	C	D	
4	12 @ 125	12 @ 200	20 @ 125	16 @ 200	12 @ 200	300
6	12 @ 125	12 @ 200	20 @ 100	16 @ 200	12 @ 200	400
8	12 @ 125	12 @ 200	25 @ 100	16 @ 200	12 @ 200	500
10	12 @ 125	16 @ 200	25 @ 100	16 @ 200	16 @ 200	650
12	12 @ 125	16 @ 200	32 @ 125	16 @ 200	16 @ 200	750

Figure 9.5 Reinforced concrete deck slab



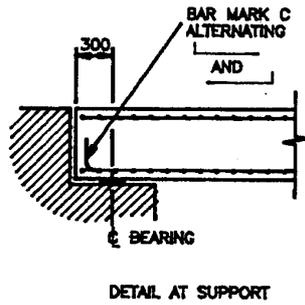
BS.HA.LOAD	✓
HS20-44	
SINGLE LANE	
1½ LANES	✓
2 LANES	
MILD STEEL REINFORCEMENT	✓
HIGH YIELD STEEL REINFORCEMENT	

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



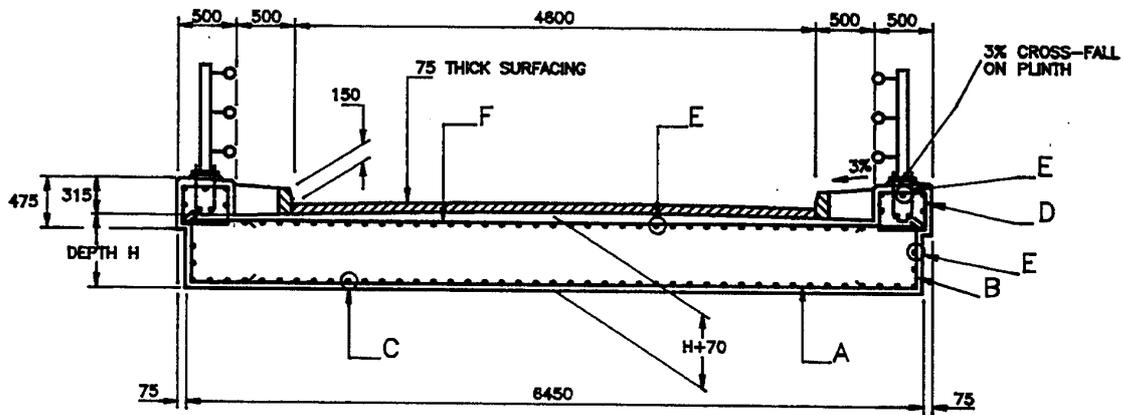
SPAN(m)	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)		BAR				DEPTH H (mm)
	STRAIGHT				STRAIGHT SPAN+500	STRAIGHT	
	A	B	C	D	E	F	
4	16 @ 125	12 @ 200	32 @ 175	16 @ 200	12 @ 200	12 @ 200	300
6	16 @ 125	12 @ 200	32 @ 150	16 @ 200	12 @ 200	12 @ 200	400
8	16 @ 125	16 @ 200	32 @ 100	16 @ 200	16 @ 200	16 @ 200	500
10	16 @ 125	16 @ 200	32 @ 100	16 @ 200	16 @ 200	16 @ 200	650
12	16 @ 125	16 @ 200	32 @ 90	16 @ 200	16 @ 200	16 @ 200	750

Figure 9.6 Reinforced concrete deck slab



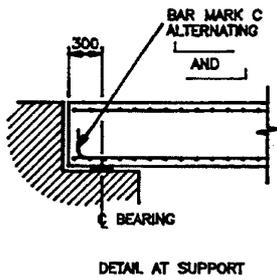
BS.HA.LOAD	✓
HS20-44	
SINGLE LANE	
1½ LANES	✓
2 LANES	
MILD STEEL REINFORCEMENT	
HIGH YIELD STEEL REINFORCEMENT	✓

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH AT 28 DAYS - 30 N/mm²



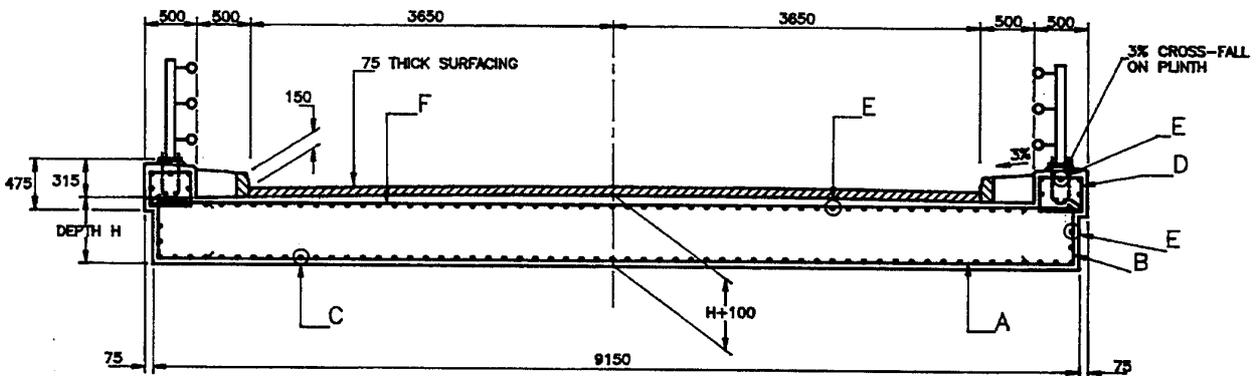
SPAN(m) \ BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)	STRAIGHT				STRAIGHT SPAN+500	STRAIGHT	DEPTH H (mm)
	A	B	C	D	E	F		
4	12 @ 125	12 @ 200	20 @ 125	16 @ 200	12 @ 200	12 @ 200	300	
6	12 @ 100	12 @ 200	20 @ 100	16 @ 200	12 @ 200	12 @ 200	400	
8	12 @ 100	12 @ 200	25 @ 100	16 @ 200	12 @ 200	12 @ 200	500	
10	12 @ 100	16 @ 200	25 @ 100	16 @ 200	16 @ 200	16 @ 200	650	
12	12 @ 100	16 @ 200	32 @ 125	16 @ 200	16 @ 200	16 @ 200	750	

Figure 9.7 Reinforced concrete deck slab



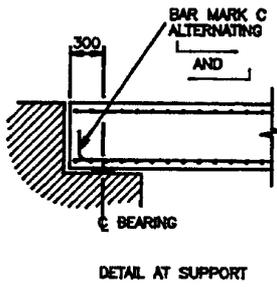
NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²

BS.HA.LOAD	✓
HS20-44	
SINGLE LANE	
1½ LANES	
2 LANES	✓
MILD STEEL REINFORCEMENT	✓
HIGH YIELD STEEL REINFORCEMENT	



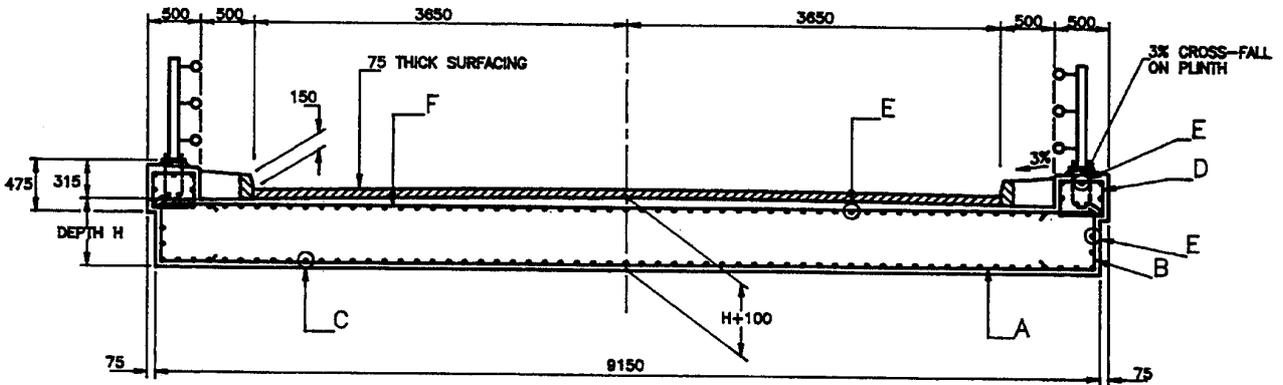
SPAN(m)	BAR	STRAIGHT				STRAIGHT	STRAIGHT	DEPTH H (mm)
		A	B	C	D	E	F	
4	16 @ 125	12 @ 200	32 @ 175	16 @ 200	12 @ 200	12 @ 200	300	
6	16 @ 100	12 @ 200	32 @ 150	16 @ 200	12 @ 200	12 @ 200	400	
8	16 @ 100	12 @ 200	32 @ 100	16 @ 200	16 @ 200	16 @ 200	500	
10	16 @ 100	16 @ 200	32 @ 100	16 @ 200	16 @ 200	16 @ 200	650	
12	16 @ 100	16 @ 200	32 @ 90	16 @ 200	16 @ 200	16 @ 200	750	

Figure 9.8 Reinforced concrete deck slab



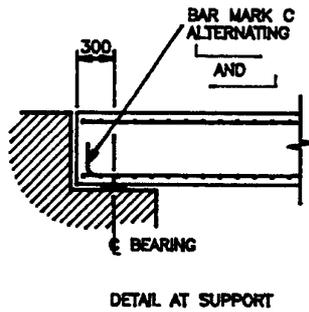
BS.HA.LOAD	✓
HS20-44	
SINGLE LANE	
1½ LANES	
2 LANES	✓
MILD STEEL REINFORCEMENT	
HIGH YIELD STEEL REINFORCEMENT	✓

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



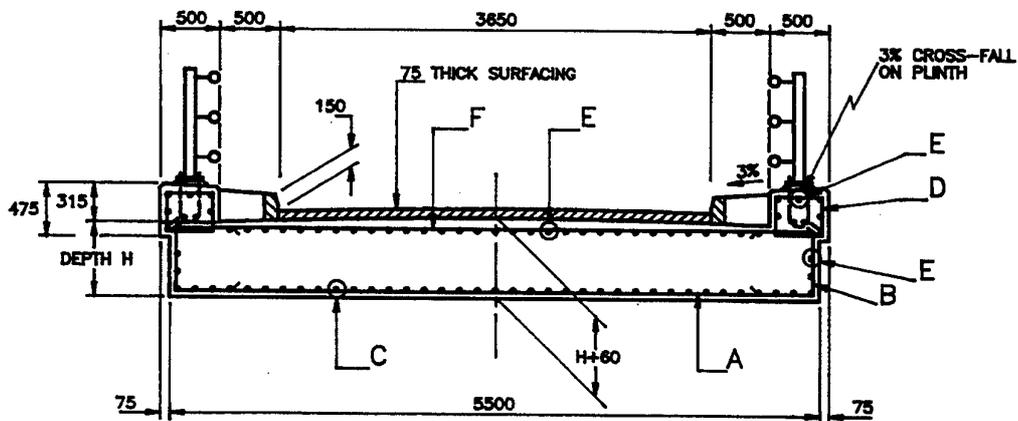
SPAN(m)	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)		BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)				DEPTH H (mm)
	STRAIGHT				STRAIGHT SPAN+500	STRAIGHT	
	A	B	C	D	E	F	
4	12 @ 100	12 @ 200	20 @ 125	16 @ 200	12 @ 200	12 @ 200	300
6	12 @ 100	12 @ 200	20 @ 100	16 @ 200	12 @ 200	12 @ 200	400
8	12 @ 100	12 @ 200	25 @ 100	16 @ 200	12 @ 200	12 @ 200	500
10	12 @ 100	16 @ 200	25 @ 100	16 @ 200	16 @ 200	16 @ 200	650
12	12 @ 100	16 @ 200	32 @ 125	16 @ 200	16 @ 200	16 @ 200	750

Figure 9.9 Reinforced concrete deck slab



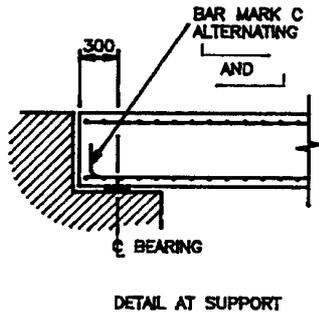
BS.HA.LOAD	
HS20-44	✓
SINGLE LANE	✓
1½ LANES	
2 LANES	
MILD STEEL REINFORCEMENT	✓
HIGH YIELD STEEL REINFORCEMENT	

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



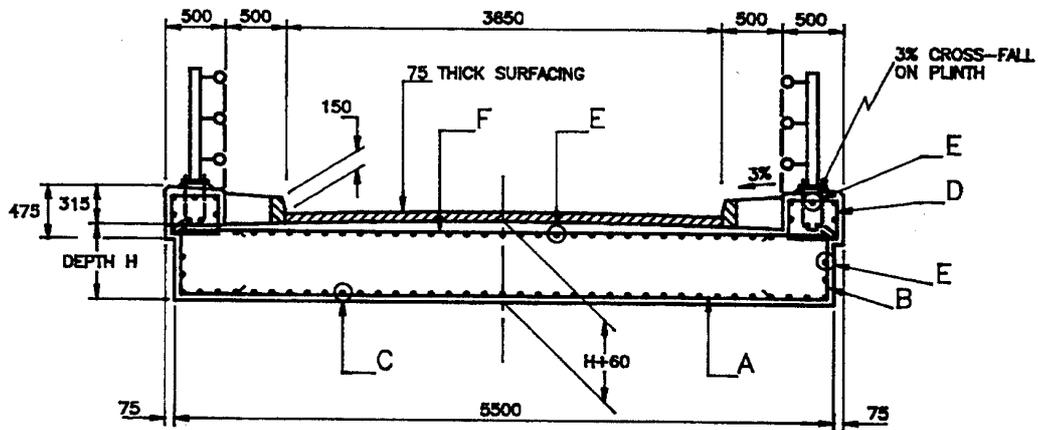
SPAN(m) \ BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)	STRAIGHT				STRAIGHT SPAN+500	STRAIGHT	DEPTH H (mm)
	A	B	C	D	E	F		
4	16 @ 125	16 @ 200	25 @ 125	16 @ 200	16 @ 200	16 @ 200	16 @ 200	300
6	16 @ 125	16 @ 200	25 @ 100	16 @ 200	16 @ 200	16 @ 200	16 @ 200	400
8	16 @ 125	20 @ 200	32 @ 150	16 @ 200	20 @ 200	20 @ 200	20 @ 200	500
10	16 @ 100	20 @ 200	32 @ 125	16 @ 200	20 @ 200	20 @ 200	20 @ 200	650
12	20 @ 125	20 @ 200	32 @ 100	16 @ 200	20 @ 200	20 @ 200	20 @ 200	750

Figure 9.10 Reinforced concrete deck slab



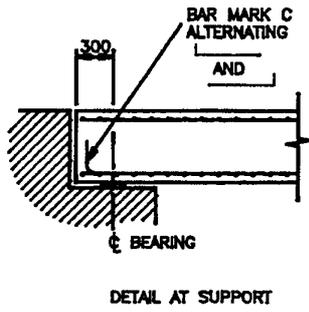
BS.HA.LOAD	
HS20-44	✓
SINGLE LANE	✓
1½ LANES	
2 LANES	
MILD STEEL REINFORCEMENT	
HIGH YIELD STEEL REINFORCEMENT	✓

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



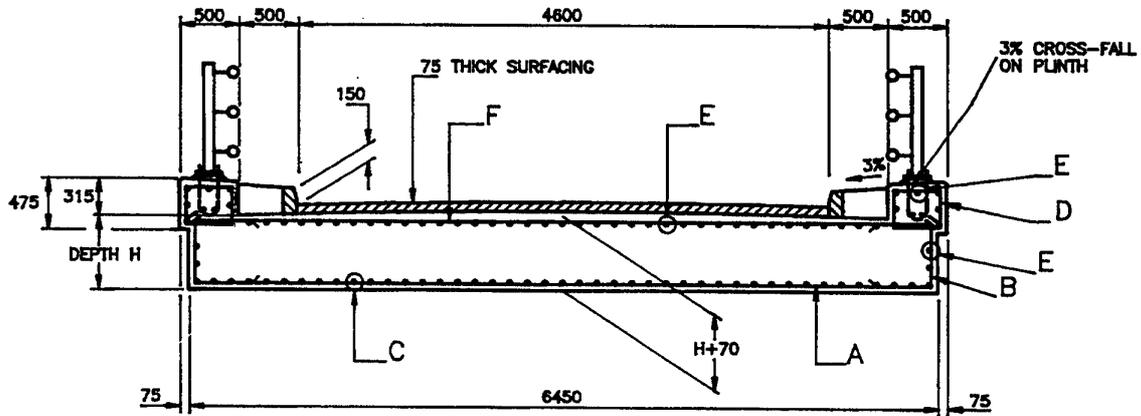
SPAN(m)	BAR	STRAIGHT				STRAIGHT	STRAIGHT	DEPTH H (mm)
		A	B	C	D	E	F	
4		12 @ 125	12 @ 200	20 @ 125	16 @ 200	12 @ 200	12 @ 200	300
6		12 @ 125	16 @ 200	25 @ 175	16 @ 200	16 @ 200	16 @ 200	400
8		12 @ 100	16 @ 200	25 @ 150	16 @ 200	16 @ 200	16 @ 200	500
10		16 @ 150	20 @ 200	25 @ 125	16 @ 200	16 @ 200	20 @ 200	650
12		16 @ 125	20 @ 200	25 @ 100	16 @ 200	20 @ 200	20 @ 200	750

Figure 9.11 Reinforced concrete deck slab



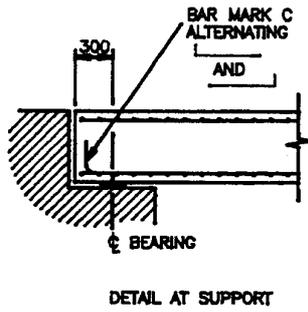
BS.HA.LOAD	
HS20-44	✓
SINGLE LANE	
1½ LANES	✓
2 LANES	
MILD STEEL REINFORCEMENT	✓
HIGH YIELD STEEL REINFORCEMENT	

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



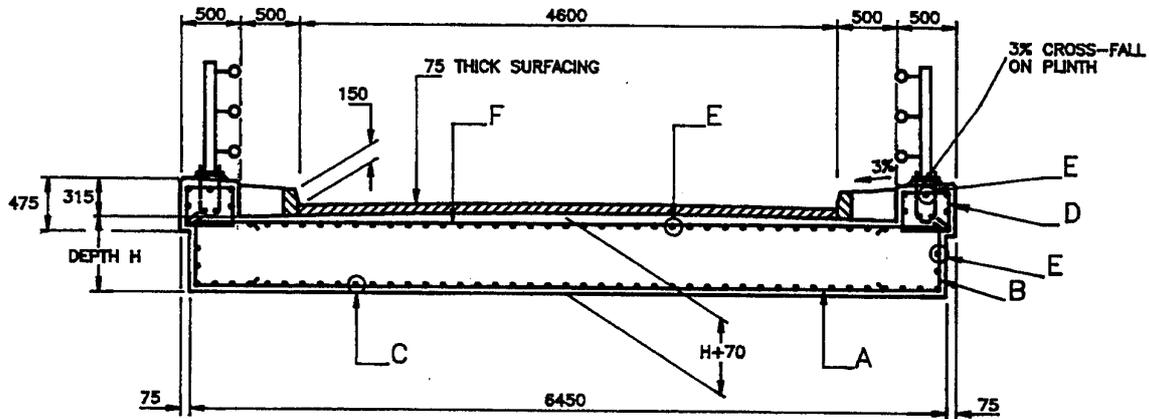
SPAN(m)	BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)	STRAIGHT				STRAIGHT SPAN+500	STRAIGHT	DEPTH H (mm)
		A	B	C	D	E	F		
4		16 @ 125	16 @ 200	25 @ 125	16 @ 200	16 @ 200	16 @ 200	16 @ 200	300
6		16 @ 125	16 @ 200	25 @ 100	16 @ 200	16 @ 200	16 @ 200	16 @ 200	400
8		16 @ 125	20 @ 200	32 @ 150	16 @ 200	20 @ 200	20 @ 200	20 @ 200	500
10		16 @ 100	20 @ 200	32 @ 125	16 @ 200	20 @ 200	20 @ 200	20 @ 200	650
12		20 @ 125	20 @ 200	32 @ 100	16 @ 200	20 @ 200	20 @ 200	20 @ 200	750

Figure 9.12 Reinforced concrete deck slab



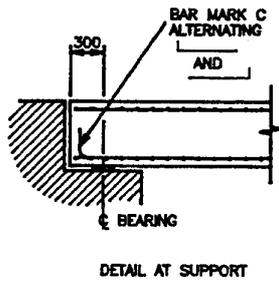
BS.HA.LOAD	
HS20-44	✓
SINGLE LANE	
1½ LANES	✓
2 LANES	
MILD STEEL REINFORCEMENT	
HIGH YIELD STEEL REINFORCEMENT	✓

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



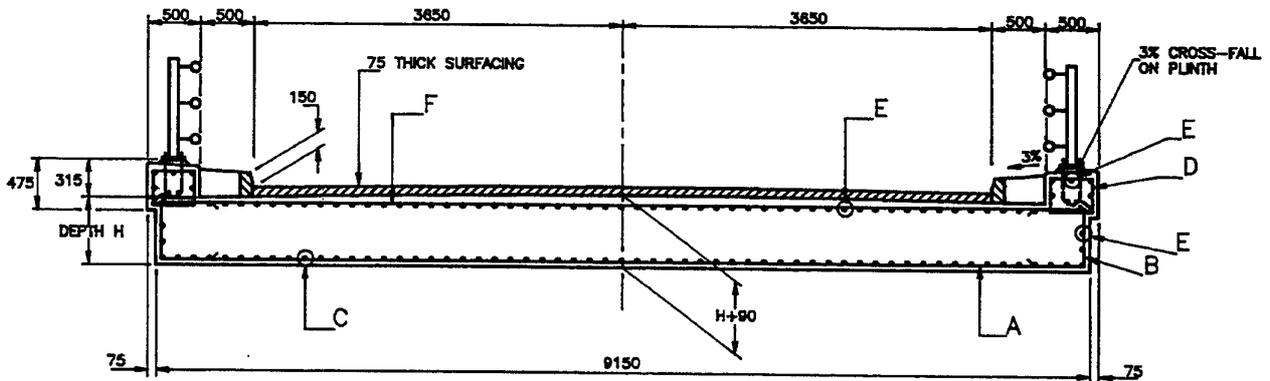
SPAN(m)	BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)	STRAIGHT				STRAIGHT	STRAIGHT	DEPTH H (mm)
		STRAIGHT	STRAIGHT	STRAIGHT	STRAIGHT	STRAIGHT			
		A	B	C	D	E	F		
4		12 @ 100	12 @ 200	20 @ 125	16 @ 200	12 @ 200	12 @ 200	300	
6		12 @ 100	16 @ 200	25 @ 175	16 @ 200	16 @ 200	16 @ 200	400	
8		12 @ 100	16 @ 200	25 @ 150	16 @ 200	16 @ 200	16 @ 200	500	
10		16 @ 150	20 @ 200	25 @ 125	16 @ 200	20 @ 200	20 @ 200	650	
12		16 @ 125	20 @ 200	25 @ 100	16 @ 200	20 @ 200	20 @ 200	750	

Figure 9.13 Reinforced concrete deck slab



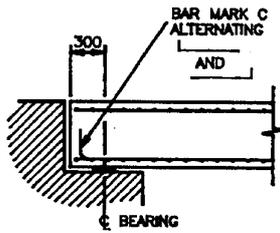
BS.HA.LOAD	
HS20-44	✓
SINGLE LANE	
1½ LANES	
2 LANES	✓
MILD STEEL REINFORCEMENT	✓
HIGH YIELD STEEL REINFORCEMENT	

NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²



SPAN(m)	BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)				STRAIGHT SPAN+500	STRAIGHT	DEPTH H (mm)
		STRAIGHT						
		A	B	C	D	E	F	
4		16 @ 125	16 @ 200	25 @ 125	16 @ 200	16 @ 200	16 @ 200	300
6		16 @ 125	16 @ 200	25 @ 100	16 @ 200	16 @ 200	16 @ 200	400
8		16 @ 125	20 @ 200	32 @ 150	16 @ 200	20 @ 200	20 @ 200	500
10		16 @ 100	20 @ 200	32 @ 125	16 @ 200	20 @ 200	20 @ 200	650
12		20 @ 125	20 @ 200	32 @ 100	16 @ 200	20 @ 200	20 @ 200	750

Figure 9.14 Reinforced concrete deck slab



DETAIL AT SUPPORT

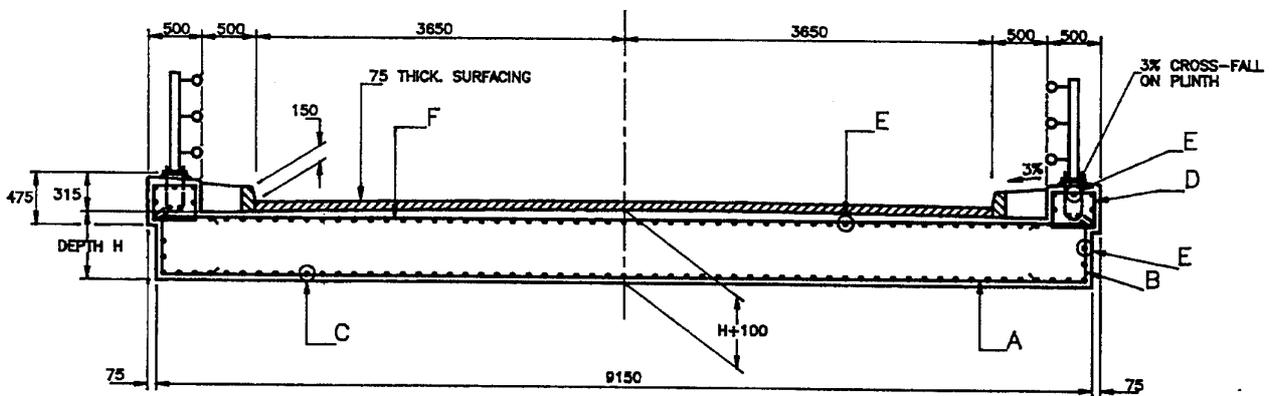
BS.HA.LOAD	
HS20-44	✓
SINGLE LANE	
1½ LANES	
2 LANES	✓
MILD STEEL REINFORCEMENT	
HIGH YIELD STEEL REINFORCEMENT	✓

NOTES :

ALL DIMENSIONS IN MILLIMETRES (mm).

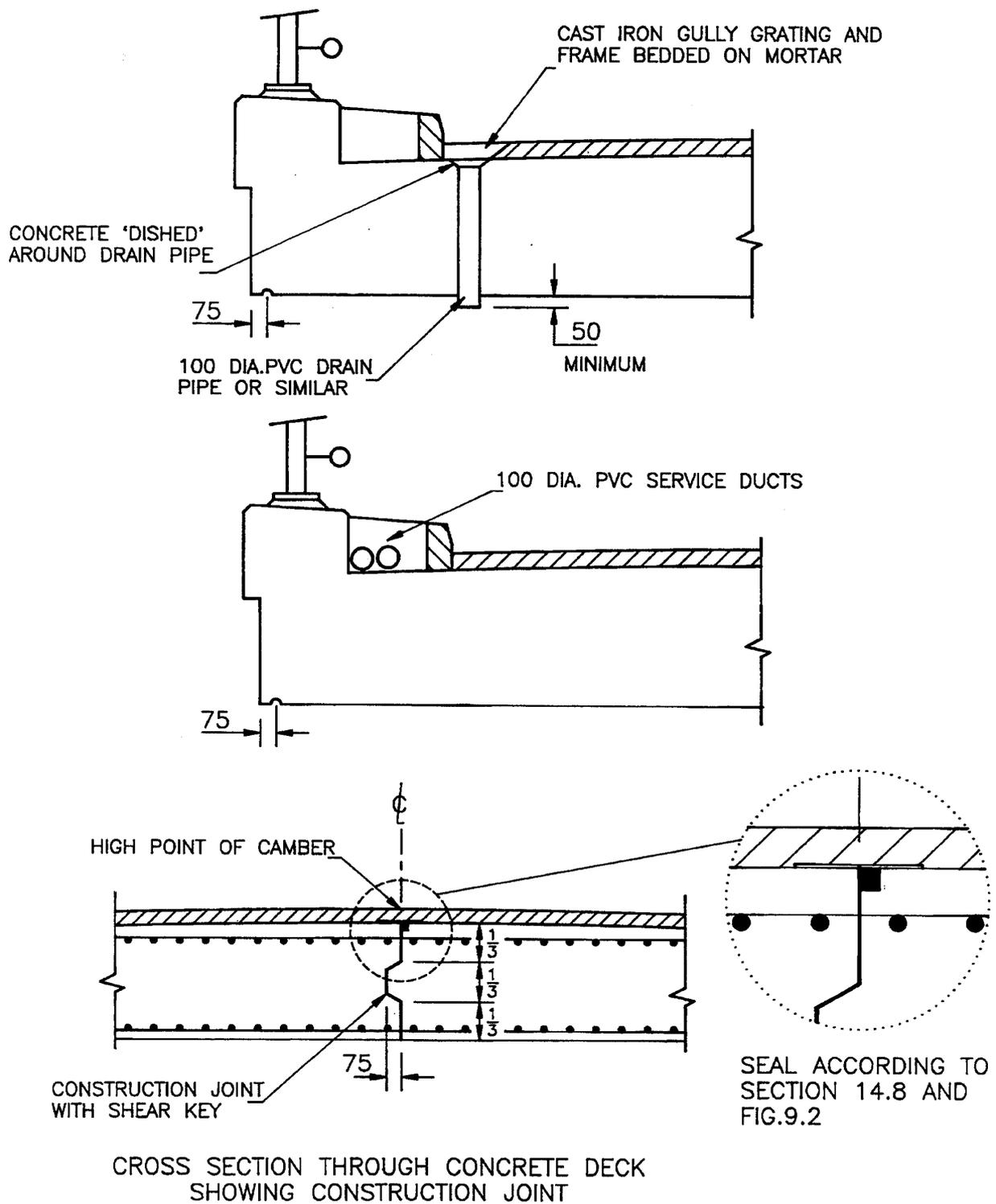
MIN. COVER TO REINFORCEMENT - 50

CHARACTERISTIC CONCRETE CUBE STRENGTH AT 28 DAYS - 30 N/mm²



SPAN(m)	BAR	BAR SHAPE & MAX. OVERALL DIMENSIONS(mm)				STRAIGHT SPAN+500	STRAIGHT	DEPTH H (mm)
		A	B	C	D			
4	12 @ 100	12 @ 200	20 @ 125	16 @ 200	12 @ 200	12 @ 200	300	
6	12 @ 100	16 @ 200	25 @ 175	16 @ 200	16 @ 200	16 @ 200	400	
8	12 @ 100	16 @ 200	25 @ 150	16 @ 200	16 @ 200	16 @ 200	500	
10	16 @ 150	20 @ 200	25 @ 125	16 @ 200	20 @ 200	20 @ 200	650	
12	16 @ 125	20 @ 200	25 @ 100	16 @ 200	20 @ 200	20 @ 200	750	

Figure 9.15 Reinforced concrete deck slab



ALL DIMENSIONS IN MILLIMETRES (mm)

Figure 9.16 Ducts, drainage and construction joints

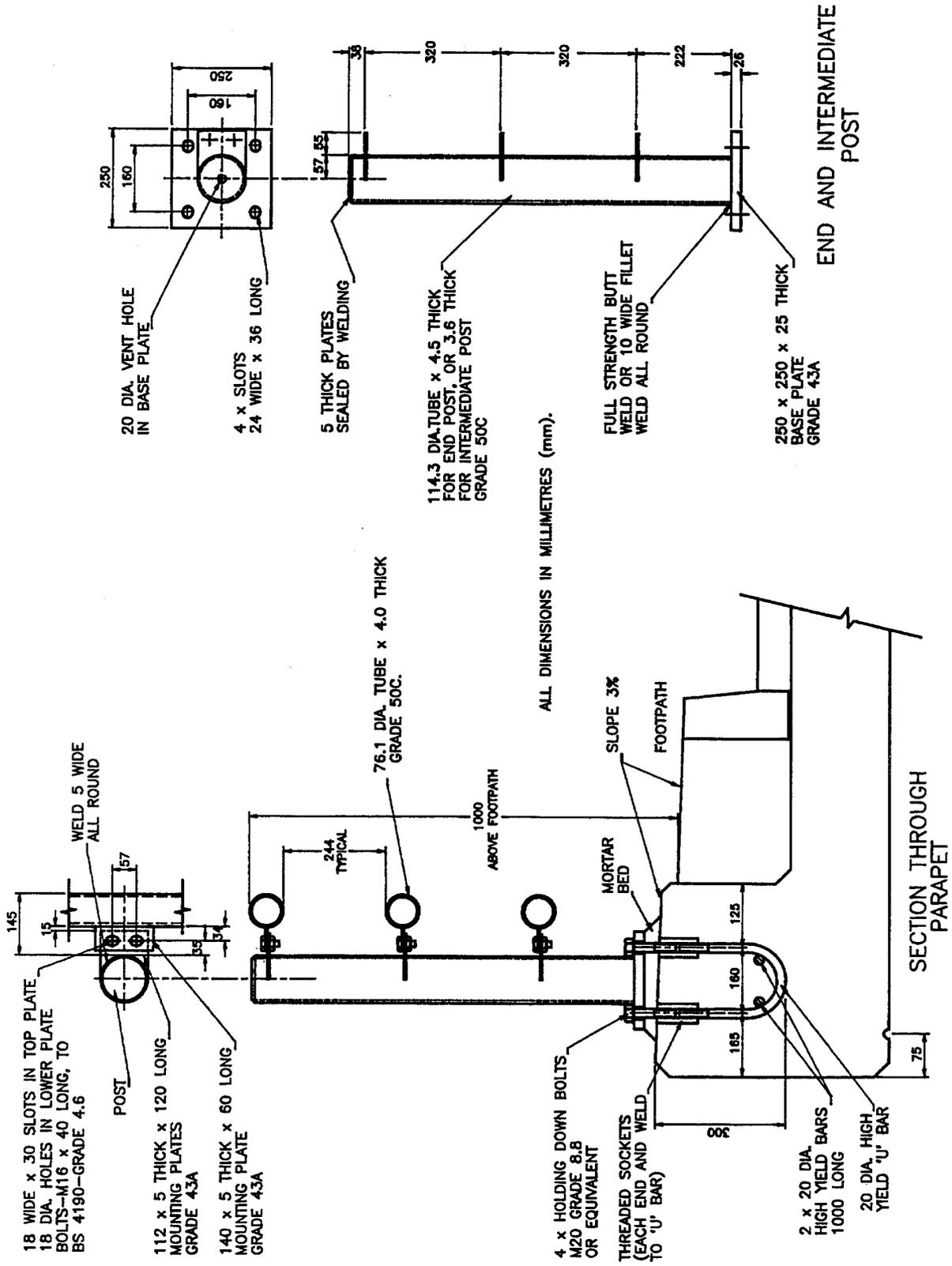


Figure 9.17 Parapets

10 STEEL/CONCRETE COMPOSITE SUPERSTRUCTURES

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10 Steel/concrete composite superstructures

In countries without an indigenous steel industry, the need to use foreign exchange for structural steelwork often inhibits designers from considering steel bridges. However, there are situations in which the use of steelwork will be both technically appropriate and cost-effective.

Because they are easy both to erect and remove, and can be used again elsewhere, steel beams with a timber decking are often a good solution for structures intended to have a short service life (ie no more than 10 years). Their durability, however, is limited by the inability of the decking to prevent surface water and dirt from penetrating through to the steel main beams. Though regular maintenance painting can prevent deterioration of the beam webs and bottom flanges, serious corrosion can occur at the inaccessible beam top flanges, where the timber and steel meet.

Where structures are intended to be permanent, the durability of the steel over a service life of 50 years or more can be achieved more readily by the use of a cast in situ concrete deck slab. Composite action of the slab and beams is secured by the use of shear connectors welded to the top flanges of the beams and cast into the concrete.

As an alternative to the solid concrete slab decks described in Chapter 9, this chapter details a series of standard bridge decks constructed from steel universal beams (UB) with a composite concrete deck slab. The main beams and cross members are of standard rolled carbon steel sections (yield stress 274N/mm^2 , with deck slab reinforcement in either mild steel (MS) or high yield steel (HYS). The steel and concrete specifications are given in Chapter 14.

Steel/concrete composite deck structures have the following advantages:

- The deck self weight can be less than that of an equivalent all-concrete structure.
- The off-site prefabrication of the main load-carrying elements of the bridge substantially reduces the work necessary on site, resulting in more rapid construction.
- No temporary supports are required during of the deck slab, since the soffit shutters can be supported directly from the steel beams. This can be a particular advantage at locations with poor ground conditions, steeply sloping terrain, or with a fast stream.
- Steel is a reliable material which is supplied with guaranteed strength properties, enabling structures of high and consistent reliability to be produced.

10.1 Design standards

The bridge decks detailed in this chapter range from 6m to 12m in length with roadway widths of one, one and a half, and two lanes. Span lengths are measured between bearing centres as shown in Figures 8.18 and 8.22: the designs are in accordance with BSI (1979) for HA loading and AASHTO (1985) for HS20-44 loading.

10.2 Design details

10.2.1 Vertical profile over the bridge

The bridge should preferably be constructed either to a level profile or to a constant longitudinal grade, if this is required by the road alignment. The steel beams can be cambered to give some degree of hogging vertical curvature should the equipment be available, but the cost is unlikely to be justifiable.

10.2.2 Construction

The heaviest beams required for the standard bridges detailed in Figures 10.1 to 10.11, ie those for a 12m span length, weigh less than two tonnes. They may therefore be erected using a small mobile crane or improvised lifting equipment. Alternatively, the beams may be pushed or pulled out over the gap being bridged using some form of intermediate temporary support and/or a light launching nose attached to the beams.

Propping of the steel beams during deck concreting is unnecessary, since the steel section has been designed to be adequate to support the wet concrete, soffit shutters and construction loading.

The engineer must appreciate that the wet concrete loading constitutes a critical design case for the steel beams. Wind loading at this stage adds significantly to the stresses in the beams. For this reason, it is necessary to ensure that the concrete slab is not poured if wind speeds of 1.8m/sec (40mph) or more are occurring or are expected within the following 48 hours.

10.2.3 Shear connectors

These connectors are the only links between the concrete slab, acting in compression and bending when under load, and the steel beams, acting in tension and bending. The largest shear forces act at each end of the deck, where the shear connectors are spaced closer together. Figure 10.7 gives the dimensions and spacing for each span and load rating.

Welded channel shear connectors are specified, since they can be produced from readily available material and fixed locally. The joints, however, should be of good quality welding and be protected from corrosion in storage.

10.2.4 Protective treatment to steelwork

The degree of protection which the steelwork will require depends on the local environment. Particular care will be needed for structures in coastal locations or where there is significant atmospheric pollution.

Types of paint used and surface preparation methods will depend on the local availability of materials and equipment. The engineer should try to achieve the following standard in order to ensure a reasonable life to first maintenance:

- 1) grit blasting to remove millscale, loose rust, welding scale etc., and produce a clean surface for painting;
- 2) application of a multi-coat paint system to a total dry film thickness of 0.25mm.

At least one paint coat should be applied at site after completion of construction, so that damage to paintwork incurred during transport, steel erection and concreting can be repaired.

10.2.5 Bolts and nuts

Ordinary bolts, grade 8.8 to ISO (1982) together with grade 8 nuts, are specified for fixing cross members. Alternatives should match the tensile strength of 80 kgf/mm² with a minimum elongation at fracture of 12%.

10.2.6 Bearings

Elastomeric bearings, as detailed in Sections 9.2.2 and 14.6, are specified because they are durable, inexpensive and simple to install.

10.2.7 Expansion joints

At these relatively short deck lengths, joint movements due to temperature and live loading are small and are readily accommodated by a simple gap joint, as indicated in Figures 10.11 and 10.12. The joint is sealed by a polysulphide sealant to prevent water penetration. An alternative unsealed joint, more suitable for bridges on gravel roads, is described in Section 14.7.

10.2.8 Construction joints

As discussed in Section 9.2.4, it is best if the slab can be cast in one continuous pour. If this will not be practicable, permissible locations for construction joints should be marked on the drawings. If a joint is unavoidable, it should be perpendicular to the centre line at a location least likely to promote corrosion in underlying steelwork.

10.2.9 Slab reinforcement

As noted in Section 9.2.5, the maximum length of reinforcing bar generally available is 12m. Where a longer bar is required, eg for a 12m span bridge, two bars must be lapped. The lap length should be at least 40 times the diameter of the lapped bars and laps should be staggered both to avoid a line of weakness and to minimise congestion of reinforcement. Main bars are L-bars, marked C in Figures 10.4 to 10.6, and are positioned with the hooks at alternate ends.

Accurate positioning of the reinforcement is essential in order to maintain the minimum cover of 50mm of well compacted concrete, so that moisture and pollutants cannot penetrate the slab as far as the reinforcing bar and corrode it.

10.2.10 Drainage

Drainage should be provided if required, generally as detailed for concrete slab bridges (Section 9.2.6. and Figure 9.16). Down pipes must be of sufficient length to ensure that run-off water is discharged at least 150mm clear below the beam lower flanges.

10.2.11 Parapets, surfacing and services

Parapet and surfacing details are the same as for concrete slab bridges (Sections 9.2.7 and 9.2.8). If ducts for services are required, they can be provided as set out in Section 9.2.9.

10.2.12 Provision for pedestrians and cyclists

The recommendations set out in Section 9.2.10 on pathways for pedestrians and cyclists on concrete decks apply equally to composite decks. The addition of a 2m wide walkway to a composite deck of the type detailed here would also require one more I beam to match those for the road bridge.

10.3 References

AASHTO (1985). *Standard specifications for highway bridges, 13th edition.* American Association of State Highway and Transportation Officials, Washington.

BSI (1979). *Steel, concrete and composite bridges Part 5. Code of practice for the design of composite bridges. BS 5400 Pt 5: 1979.* British Standards Institution, London.

ISO (1982). *Fasteners - hexagon products-widths across flats.* ISO 272:1982

10.4 Standards drawings and tables for composite superstructures

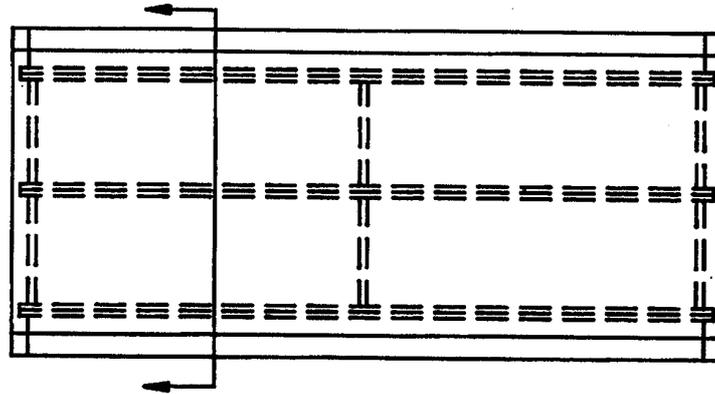
<i>Title</i>	<i>Figure number</i>
Single lane dimensions	10.1
One and a half lane dimensions	10.2
Two lane dimensions	10.3
Single lane reinforcement	10.4
One and a half lane reinforcement	10.5
Two lane reinforcement	10.6
Shear connector details	10.7
Fixing details - mid span cross members	10.8
Fixing details - end trimmers 6m and 8m spans	10.9
Fixing details - end trimmers 10m and 12m spans	10.10
Pier expansion joint and bearing details	10.11
Abutment expansion joint and bearing details	10.12

In the following drawings, the steel reinforcement is either:

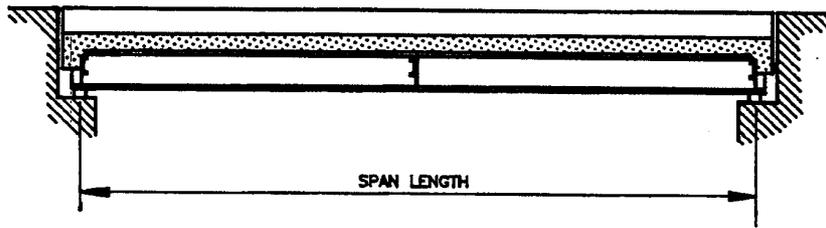
MS - plain round mild steel bar or

HYS - round deformed high tensile steel bar, according to the specifications detailed in Section 14.2

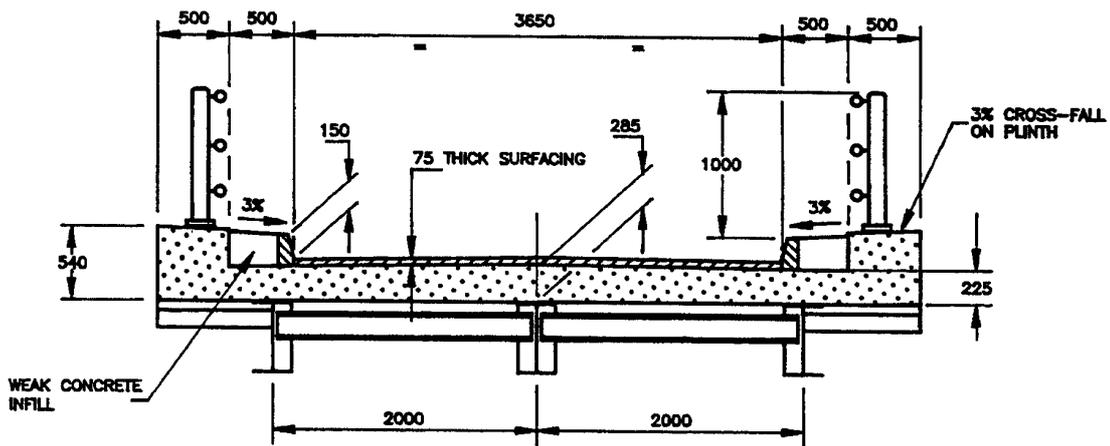
A wearing course of asphaltic concrete 75mm thick is specified. Any change to this may require corresponding changes to the abutments.



PLAN



ELEVATION



CROSS SECTION

B.S. HA LOADING	
SPAN(m)	BEAM SIZE (mm)
6	457 X 191 X 67 UB
8	533 X 210 X 82 UB
10	610 X 229 X 113 UB
12	610 X 229 X 140 UB

AASHTO HS20-44 LOADING	
SPAN(m)	BEAM SIZE (mm)
6	406 X 178 X 54 UB
8	457 X 191 X 67 UB
10	533 X 210 X 82 UB
12	610 X 229 X 113 UB

NOTES :

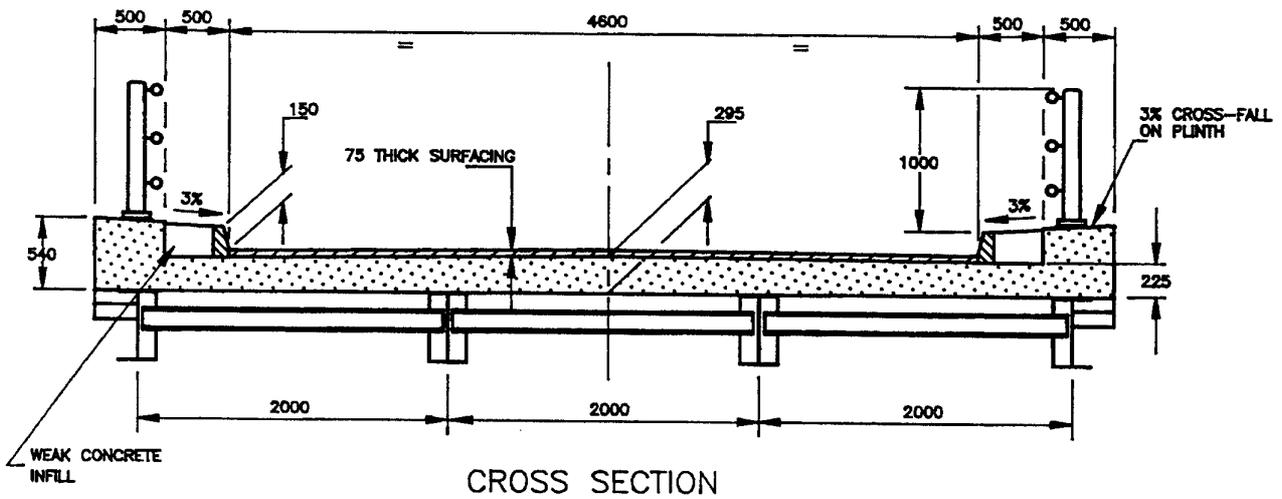
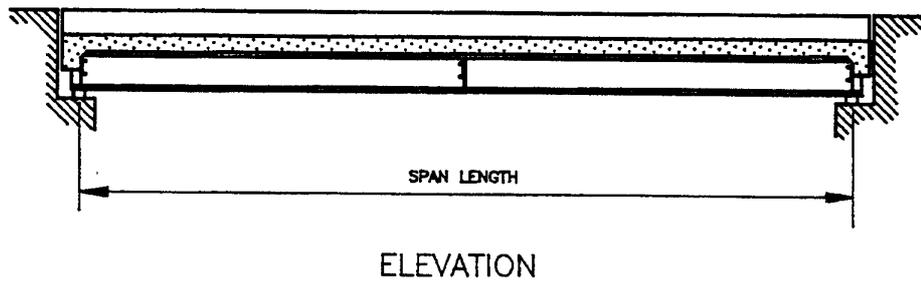
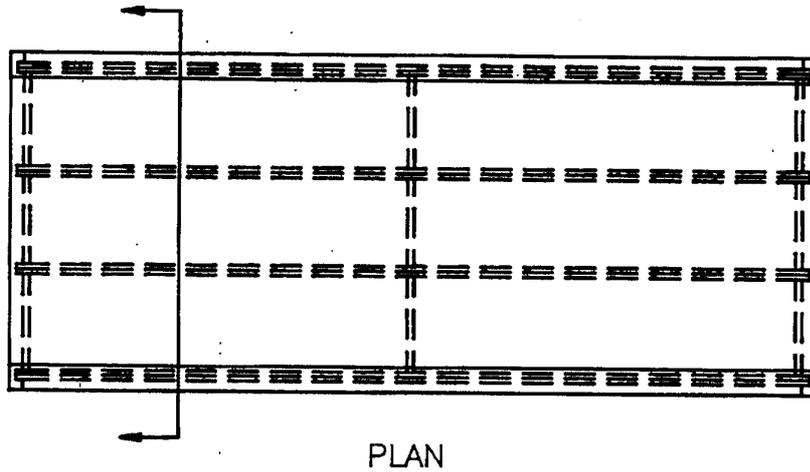
ALL DIMENSIONS IN MILLIMETRES (mm).

MIN. COVER TO REINFORCEMENT - 50

CHARACTERISTIC CONCRETE CUBE STRENGTH
AT 28 DAYS - 30 N/mm²

STEEL BEAMS ARE GRADE 43c TO BE EN 10025
OR EQUIVALENT

Figure 10.1 Composite deck - 1 lane width



B.S. HA LOADING	
SPAN(m)	BEAM SIZE(mm)
6	457 X 191 X 67 UB
8	533 X 210 X 82 UB
10	610 X 229 X 113 UB
12	610 X 229 X 140 UB

AASHTO HS20-44 LOADING	
SPAN(m)	BEAM SIZE(mm)
6	406 X 178 X 60 UB
8	457 X 191 X 74 UB
10	533 X 210 X 92 UB
12	610 X 229 X 125 UB

NOTES :

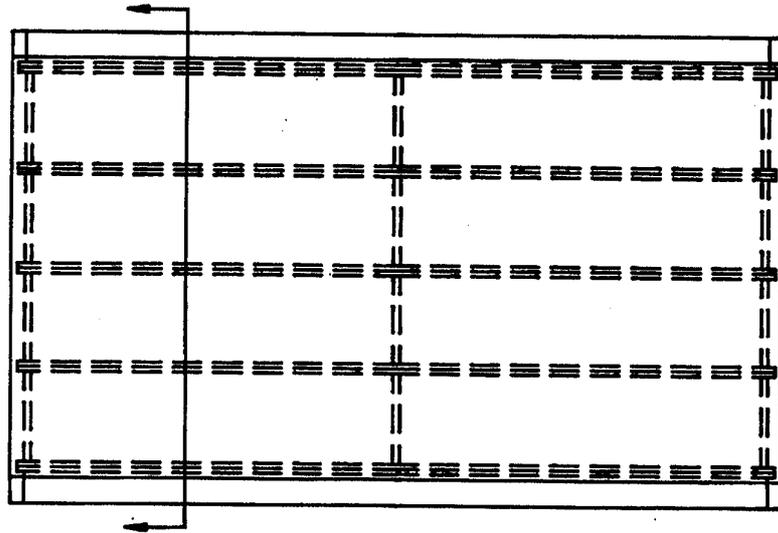
ALL DIMENSIONS IN MILLIMETRES (mm).

MIN. COVER TO REINFORCEMENT - 50

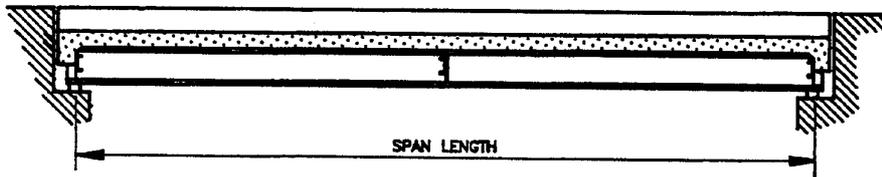
CHARACTERISTIC CONCRETE CUBE STRENGTH AT 28 DAYS - 30 N/mm²

STEEL BEAMS ARE GRADE 43c TO BE EN 10025 OR EQUIVALENT

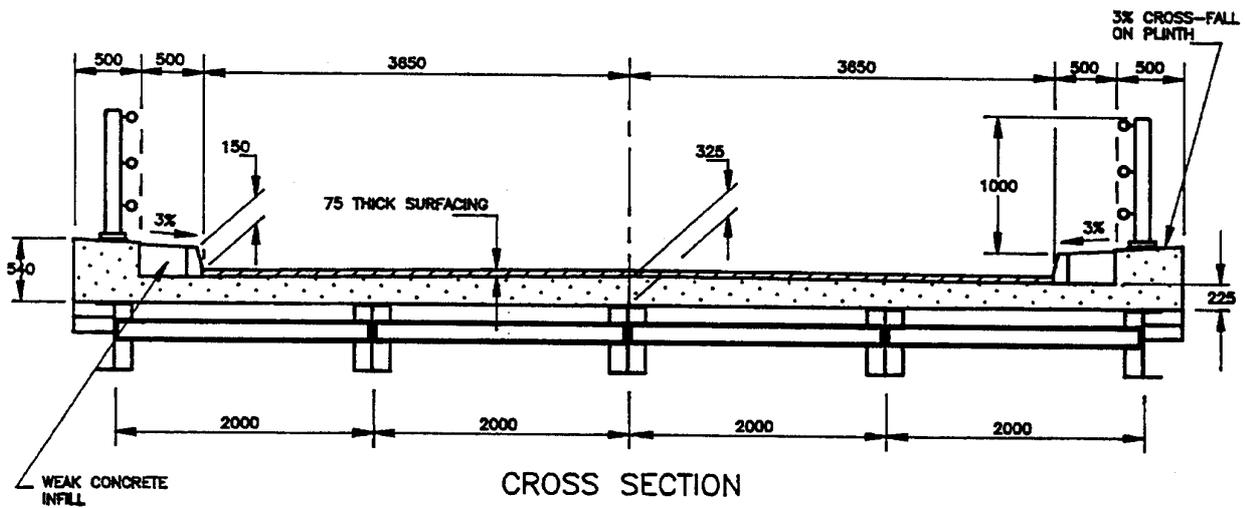
Figure 10.2 Composite deck - 1 ½ lane width



PLAN



ELEVATION



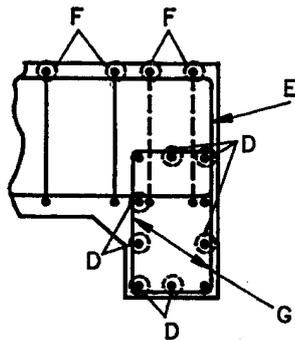
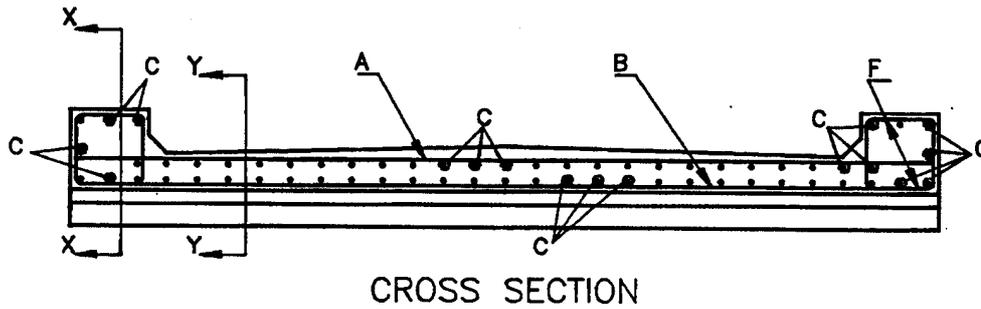
CROSS SECTION

B.S HA LOADING	
SPAN(m)	BEAM SIZE(mm)
6	457 X 191 X 67 UB
8	533 X 210 X 82 UB
10	610 X 229 X 113 UB
12	610 X 229 X 140 UB

AASHTO HS20-44 LOADING	
SPAN(m)	BEAM SIZE(mm)
6	457 X 191 X 67 UB
8	533 X 210 X 82 UB
10	616 X 229 X 101 UB
12	610 X 229 X 140 UB

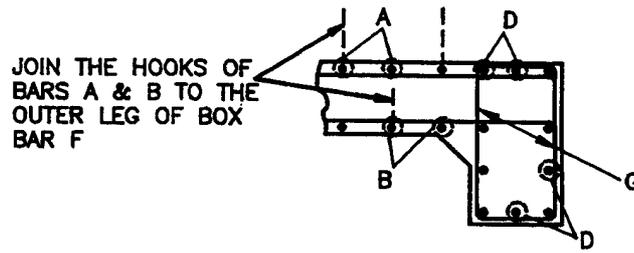
NOTES :
 ALL DIMENSIONS IN MILLIMETRES (mm).
 MIN. COVER TO REINFORCEMENT - 50
 CHARACTERISTIC CONCRETE CUBE STRENGTH
 AT 28 DAYS - 30 N/mm²
 STEEL BEAMS ARE GRADE 43c TO BE EN 10025
 OR EQUIVALENT

Figure 10.3 Composite deck – 2 lane width



LONGITUDINAL SECTION X-X
THROUGH END TRIMMER
AT DECK EDGE BEAM

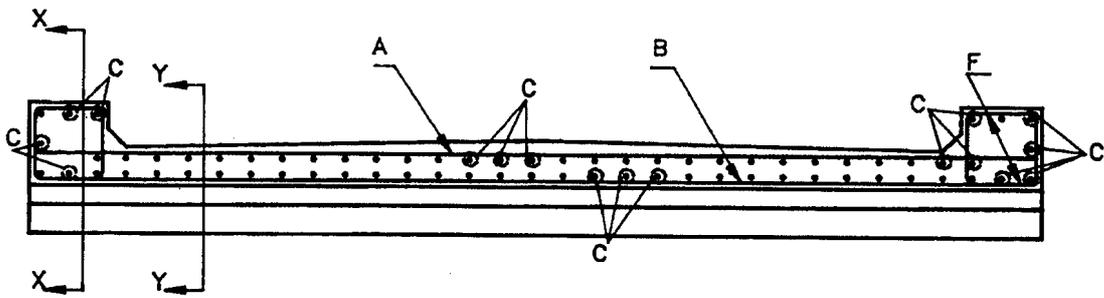
MIN. COVER TO REINFORCEMENT – 50 mm
ALTERNATE THE HOOK-ENDS OF
BARS A,B,C AND D.



LONGITUDINAL SECTION Y-Y
THROUGH SLAB END
TRIMMER

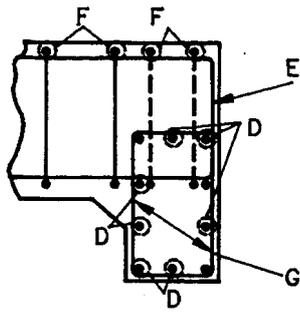
BAR MARK	SHAPE	B.S. HA LOADING		AASHTO HS20-44 LOADING	
		M.S.	H.Y.	M.S.	H.Y.
A		16 mm DIA. 100 mm SPACING	16 mm DIA. 100 mm SPACING	25 mm DIA. 200 mm SPACING	12 mm DIA. 100 mm SPACING
B		16 mm DIA. 100 mm SPACING	16 mm DIA. 100 mm SPACING	25 mm DIA. 200 mm SPACING	12 mm DIA. 100 mm SPACING
C		12 mm DIA. 150 mm SPACING	12 mm DIA. 150 mm SPACING	20 mm DIA. 200 mm SPACING	12 mm DIA. 150 mm SPACING
D		25 mm DIA. 10 No.EACH TRIMMER	20 mm DIA. 10 No.EACH TRIMMER	25 mm DIA. 10 No.EACH TRIMMER	20 mm DIA. 10 No.EACH TRIMMER
E		12 mm DIA. 6 No.EACH TRIMMER	12 mm DIA. 6 No.EACH TRIMMER	16 mm DIA. 6 No.EACH TRIMMER	16 mm DIA. 6 No.EACH TRIMMER
F		12 mm DIA. 200 mm SPACING	12 mm DIA. 200 mm SPACING	16 mm DIA. 200 mm SPACING	16 mm DIA. 200 mm SPACING
G		16 mm DIA. 200 mm SPACING			

Figure 10.4 Composite deck – single lane reinforcement



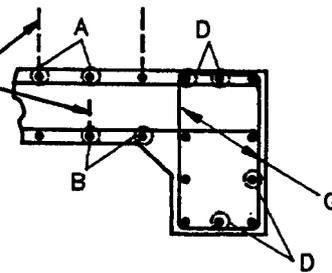
CROSS SECTION

MIN. COVER TO REINFORCEMENT - 50 mm
ALTERNATE THE HOOK-ENDS OF BARS A,B,C AND D.



LONGITUDINAL SECTION X-X
THROUGH END TRIMMER
AT DECK EDGE BEAM

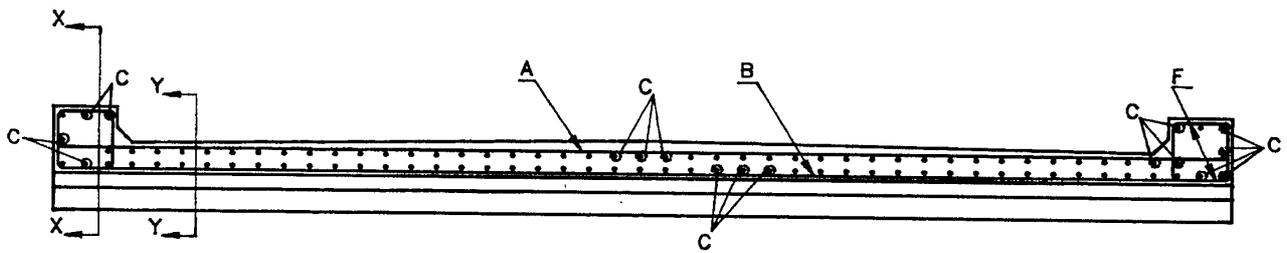
JOIN THE HOOKS OF BARS A & B TO THE OUTER LEG OF BOX BAR F



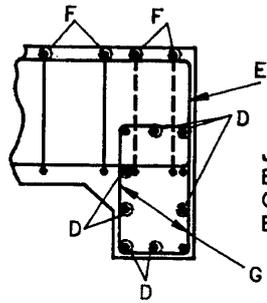
LONGITUDINAL SECTION Y-Y
THROUGH SLAB END TRIMMER

BAR MARK	SHAPE	B.S. HA LOADING		AASHTO HS20-44 LOADING	
		M.S.	H.Y.	M.S.	H.Y.
A		16 mm DIA. 100 mm SPACING	16 mm DIA. 100 mm SPACING	25 mm DIA. 200 mm SPACING	12 mm DIA. 100 mm SPACING
B		16 mm DIA. 100 mm SPACING	16 mm DIA. 100 mm SPACING	25 mm DIA. 200 mm SPACING	12 mm DIA. 100 mm SPACING
C		12 mm DIA. 150 mm SPACING	12 mm DIA. 150 mm SPACING	20 mm DIA. 200 mm SPACING	12 mm DIA. 150 mm SPACING
D		25 mm DIA. 10 No.EACH TRIMMER	20 mm DIA. 10 No.EACH TRIMMER	25 mm DIA. 10 No.EACH TRIMMER	20 mm DIA. 10 No.EACH TRIMMER
E		12 mm DIA. 6 No.EACH TRIMMER	12 mm DIA. 6 No.EACH TRIMMER	16 mm DIA. 6 No.EACH TRIMMER	16 mm DIA. 6 No.EACH TRIMMER
F		12 mm DIA. 200 mm SPACING	12 mm DIA. 200 mm SPACING	16 mm DIA. 200 mm SPACING	16 mm DIA. 200 mm SPACING
G		16 mm DIA. 200 mm SPACING			

Figure 10.5 Composite deck - 1 1/2 lane reinforcement

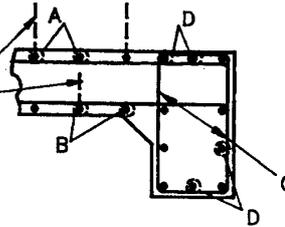


CROSS SECTION



LONGITUDINAL SECTION X-X
THROUGH END TRIMMER
AT DECK EDGE BEAM

MIN. COVER TO REINFORCEMENT - 50 mm
ALTERNATE THE HOOK-ENDS OF
BARS A,B,C AND D.

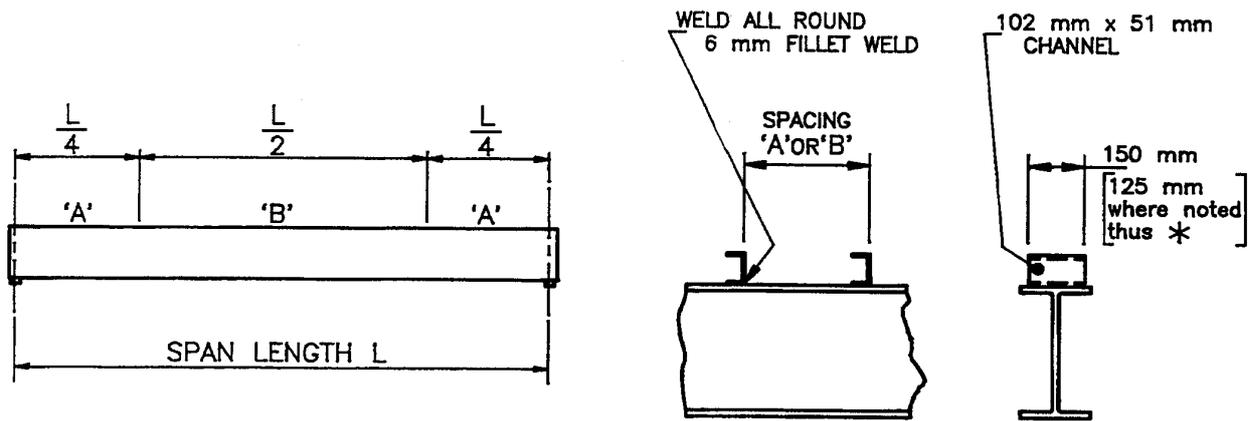


LONGITUDINAL SECTION Y-Y
THROUGH SLAB END
TRIMMER

JOIN THE HOOKS OF
BARS A & B TO THE
OUTER LEG OF BOX
BAR F

BAR MARK	SHAPE	B.S. HA LOADING		AASHTO HS20-44 LOADING	
		M.S.	H.Y.	M.S.	H.Y.
A		16 mm DIA. 100 mm SPACING	16 mm DIA. 100 mm SPACING	25 mm DIA. 200 mm SPACING	12 mm DIA. 100 mm SPACING
B		16 mm DIA. 100 mm SPACING	16 mm DIA. 100 mm SPACING	25 mm DIA. 200 mm SPACING	12 mm DIA. 100 mm SPACING
C		12 mm DIA. 150 mm SPACING	12 mm DIA. 150 mm SPACING	20 mm DIA. 200 mm SPACING	12 mm DIA. 150 mm SPACING
D		25 mm DIA. 10 No.EACH TRIMMER	20 mm DIA. 10 No.EACH TRIMMER	25 mm DIA. 10 No.EACH TRIMMER	20 mm DIA. 10 No.EACH TRIMMER
E		12 mm DIA. 6 No.EACH TRIMMER	12 mm DIA. 6 No.EACH TRIMMER	16 mm DIA. 6 No.EACH TRIMMER	16 mm DIA. 6 No.EACH TRIMMER
F		12 mm DIA. 200 mm SPACING	12 mm DIA. 200 mm SPACING	16 mm DIA. 200 mm SPACING	16 mm DIA. 200 mm SPACING
G		16 mm DIA. 200 mm SPACING			

Figure 10.6 Composite deck – 2 lane reinforcement

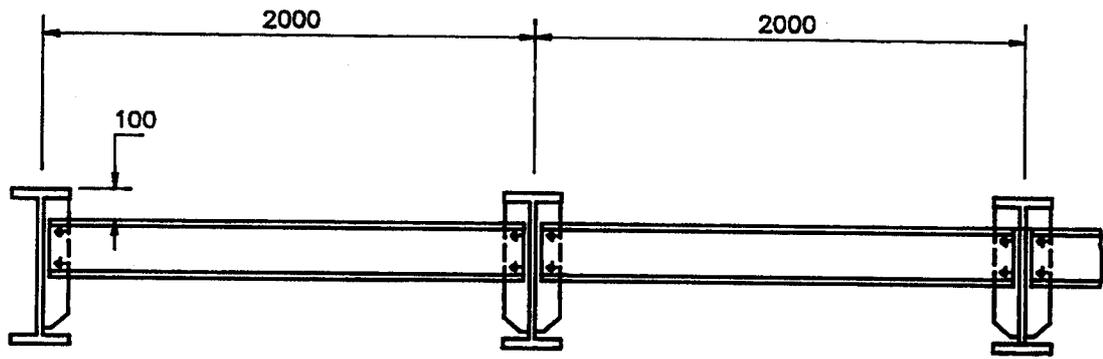


'A' AND 'B' INDICATE SHEAR CONNECTOR SPACING IN MILLIMETRES FOR THE LOCATIONS SHOWN ABOVE

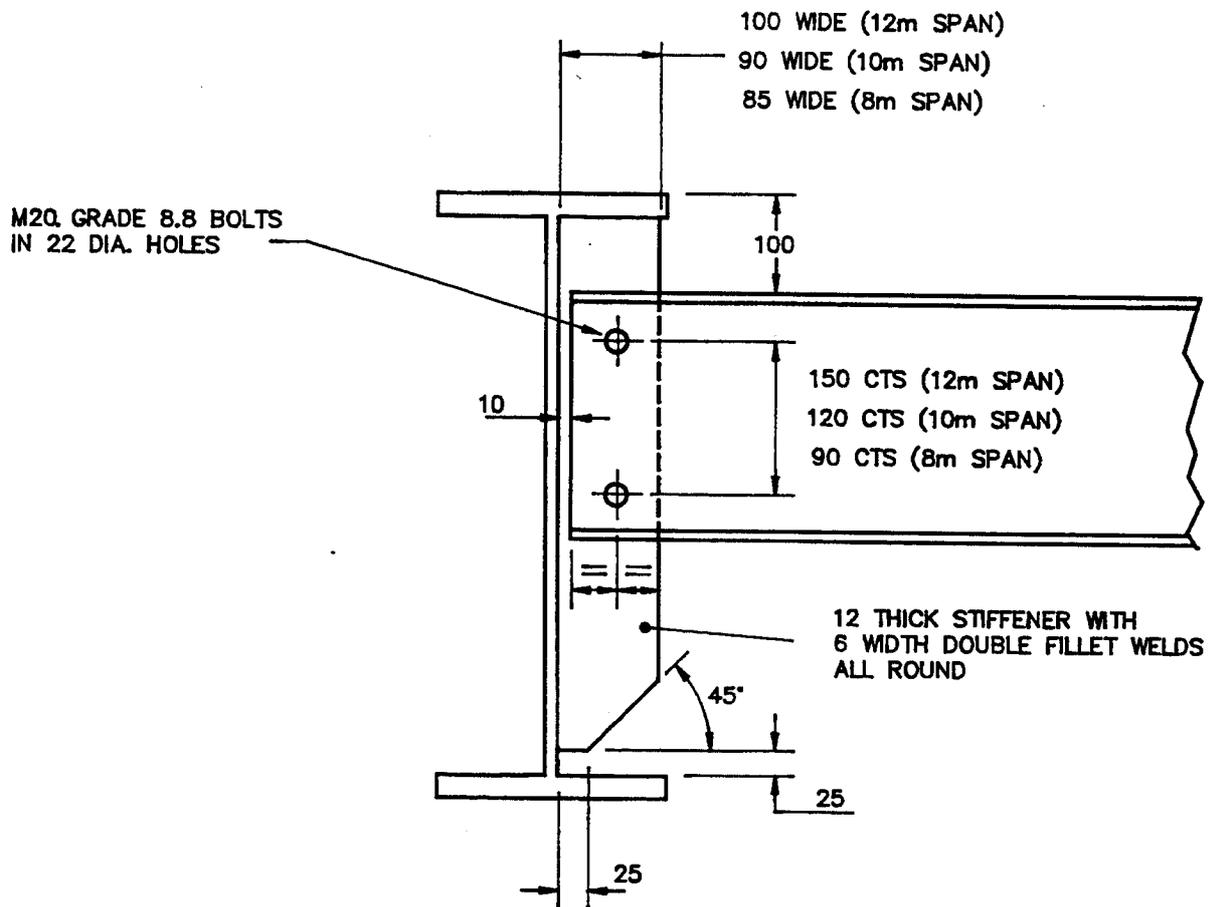
DETAIL OF SHEAR CONNECTORS

ROADWAY WIDTH	SPAN LENGTH (m)	B.S. HA LOADING		AASHTO HS20-44 LOADING	
		A	B	A	B
1 - LANE	6	300 *	400 *	300 *	400 *
	8	300	400	300 *	400 *
	10	250	400	300	400
	12	250	400	300	400
1 1/2 - LANES	6	300 *	400 *	300 *	400 *
	8	300	400	300 *	400 *
	10	250	400	300	400
	12	250	400	300	400
2 - LANES	6	300 *	400 *	300 *	400 *
	8	300	400	300	400
	10	250	400	300	400
	12	250	400	300	400

Figure 10.7 Composite decks – shear connector details



ELEVATION ON MID-SPAN BRACING FOR 8m,10m AND 12m SPACINGS

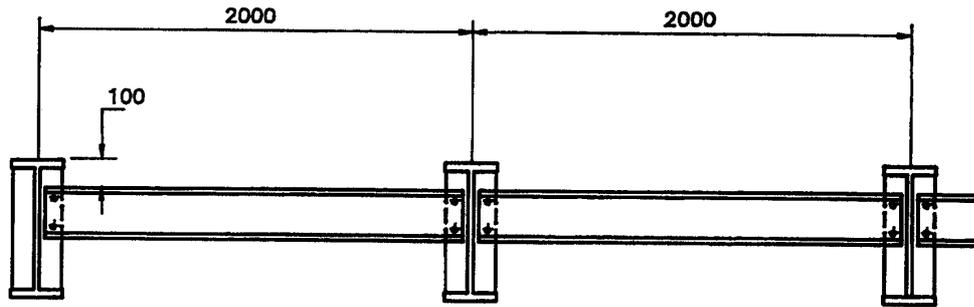


MID-SPAN BRACING DETAIL

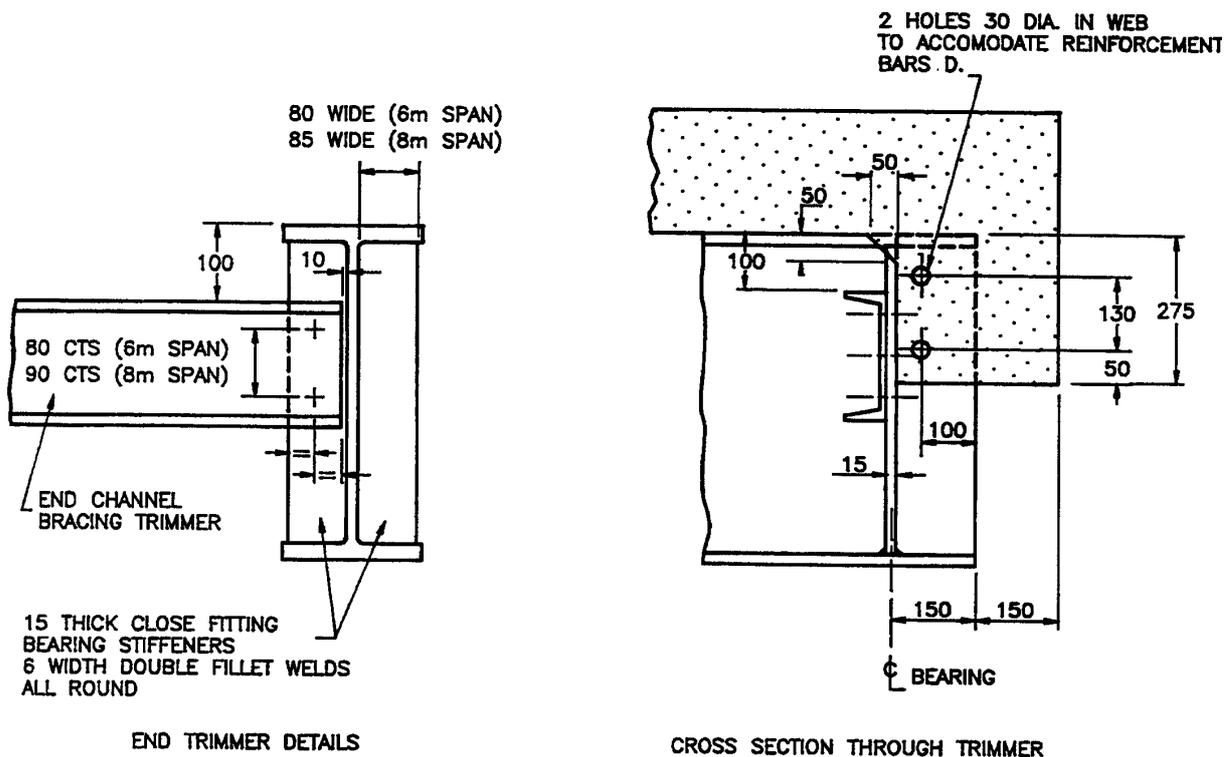
NOTES:

ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 10.8 Composite decks – fixing details of mid-span cross members



ELEVATION ON END TRIMMER



END TRIMMER DETAILS

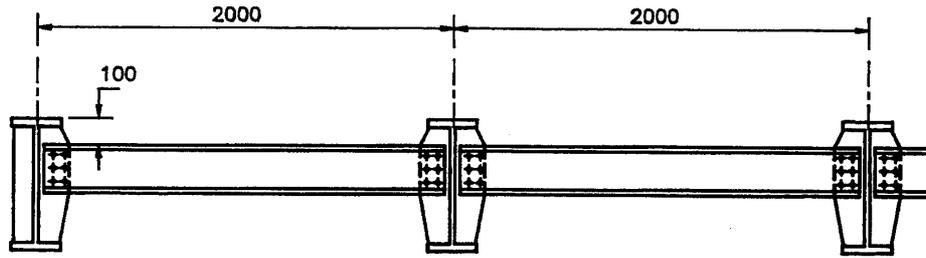
CROSS SECTION THROUGH TRIMMER

NOTE: ALL BOLTS TO BE GRADE 8.8
M20. IN 22 DIA. HOLE
ALL DIMENSIONS IN MILLIMETRES (mm)

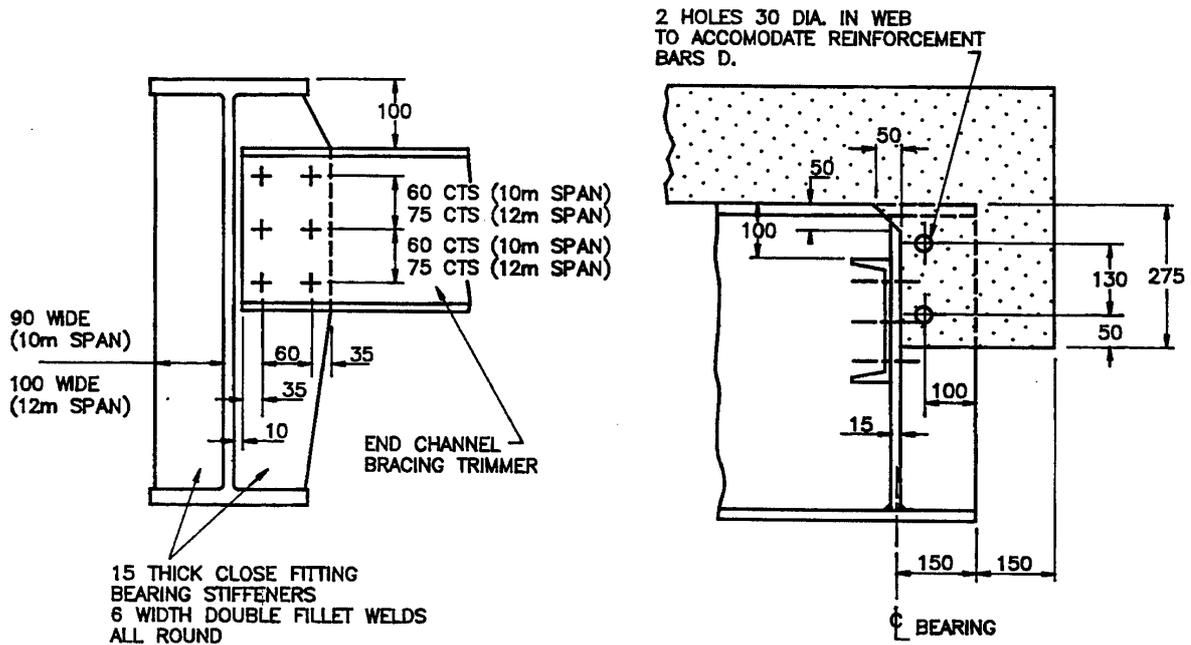
SPAN(m)	MEMBER SIZE(mm)	
	B.S. HA LOADING	AASHTO HS20-44 LOADING
6	203 X 76 CHANNEL	152 X 76 CHANNEL
8	203 X 76 CHANNEL	178 X 76 CHANNEL

TABLE OF BRACING MEMBER SIZES

Figure 10.9 Composite decks – fixing details of end trimmers 6m and 8m spans



ELEVATION ON END TRIMMER



END TRIMMER DETAILS

CROSS SECTION THROUGH TRIMMER

NOTE: ALL BOLTS TO BE GRADE 8.8
M20. IN 22 DIA. HOLE
ALL DIMENSIONS IN MILLIMETRES (mm)

SPAN(m)	MEMBER SIZE(mm)	
	B.S. HA LOADING	AASHTO HS20-44 LOADING
10	229 X 76 CHANNEL	203 X 76 CHANNEL
12	229 X 76 CHANNEL	254 X 76 CHANNEL

TABLE OF BRACING MEMBER SIZES

Figure 10.10 Composite decks – fixing details of end trimmers 10m and 12m spans

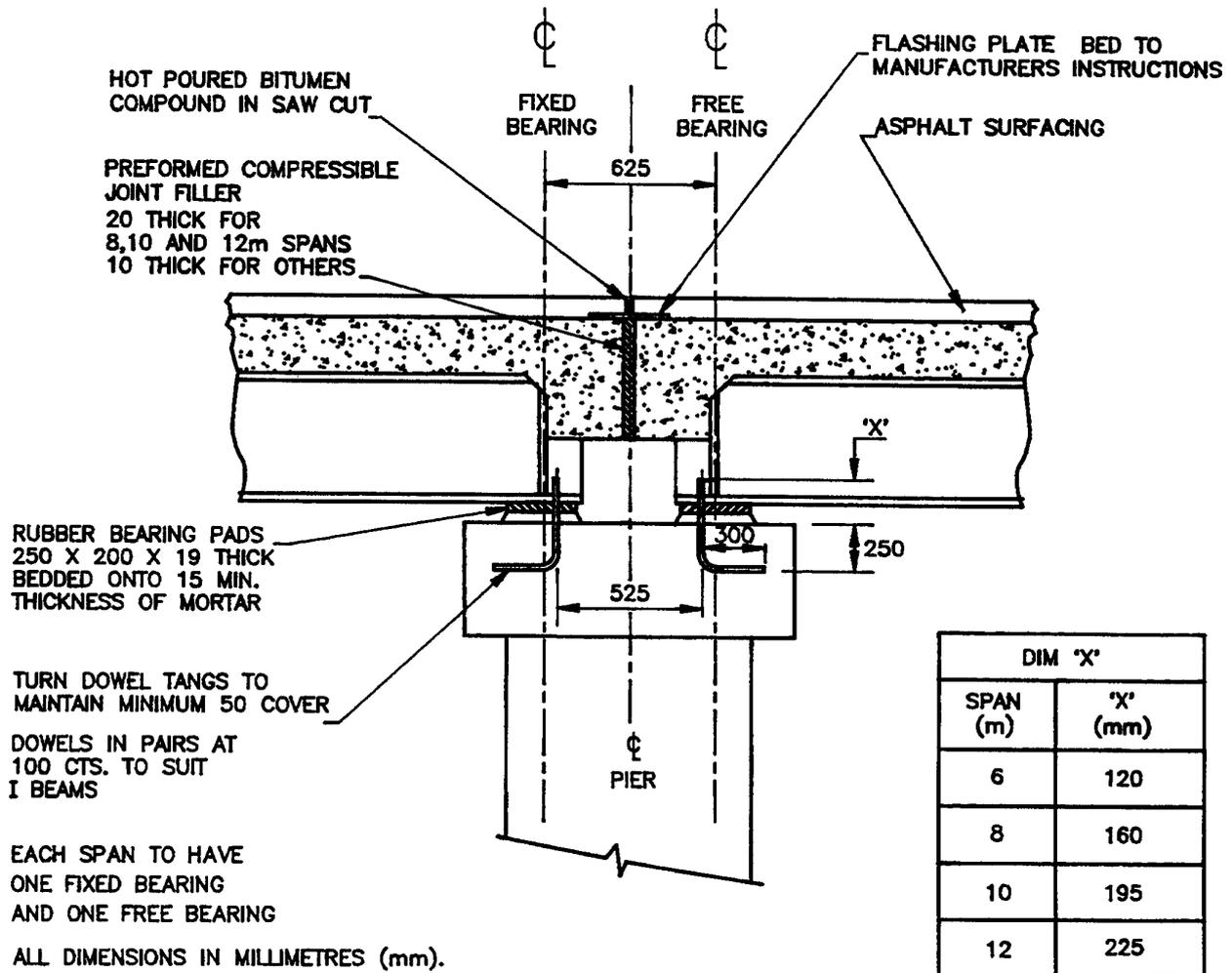
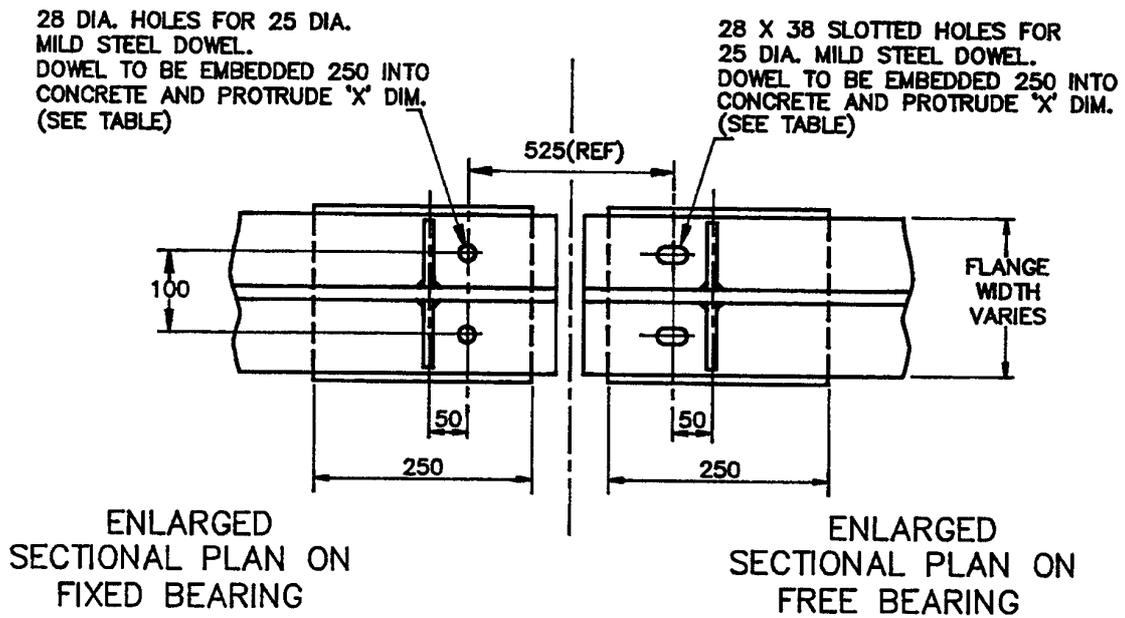


Figure 10.11 Composite decks – pier expansion joint and bearing details

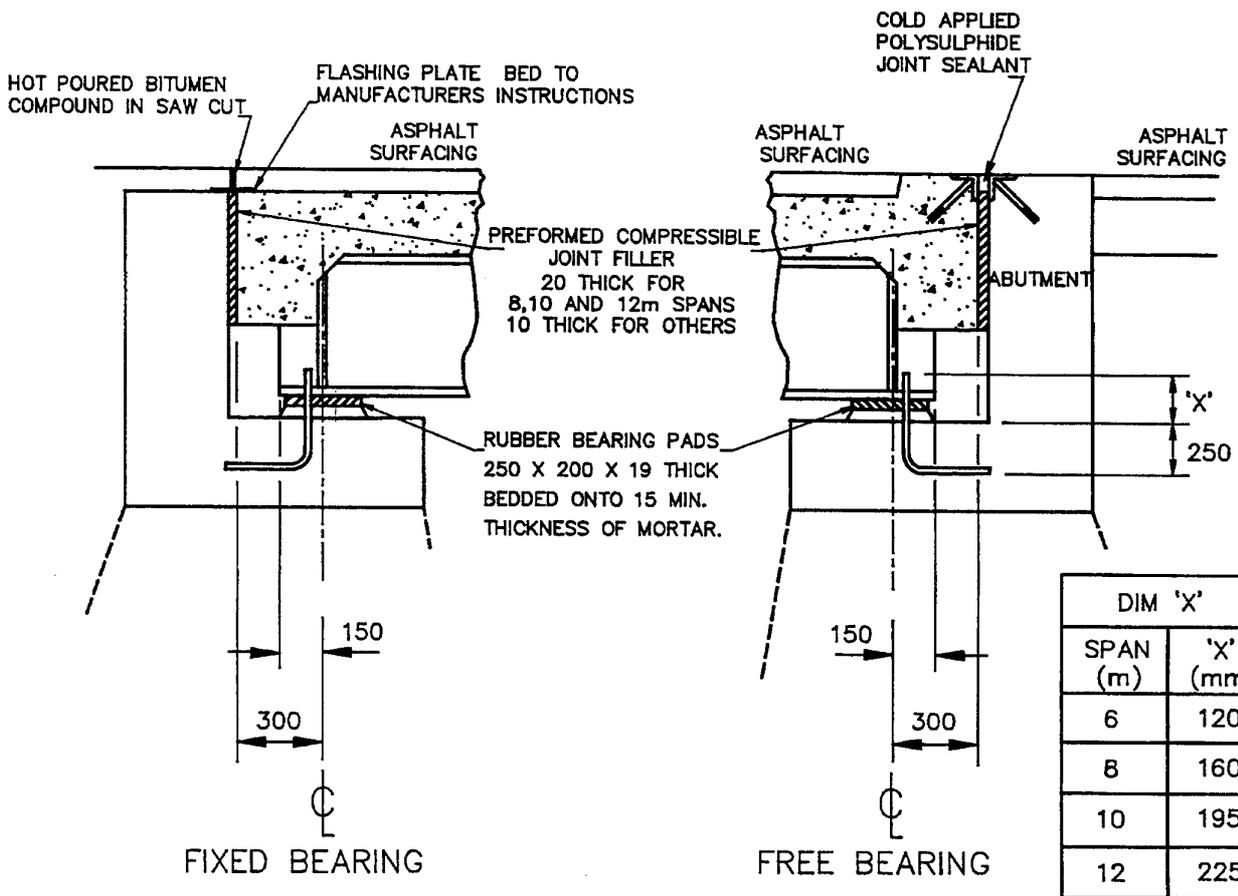
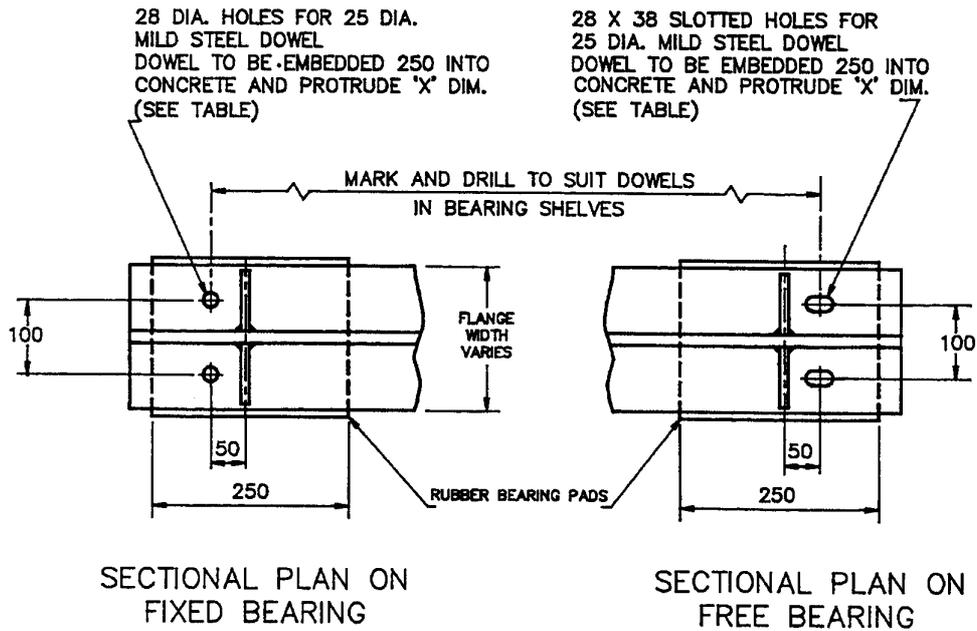


Figure 10.12 Composite decks – abutment expansion joint and bearing details

11 TIMBER SUPERSTRUCTURES

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11 Timber superstructures

Timber is a viable load-bearing material for the smaller bridge decks and can offer the following potential advantages:

- low material cost for short spans;
- relatively modest requirements for skilled labour and equipment;
- no need to have a dry river bed for building shuttering during construction;
- ability to compensate for a certain amount of settlement of substructures without strain.

The limitations of timber as a bridge deck material include:

- susceptibility to rotting and insect attack
- need for frequent maintenance
- heavy duty or long span timber decks require a lot of material, and so may be more expensive than steel or concrete alternatives.

This chapter contains designs for single-lane timber beam decks, suitable for spans up to 12m. They are simple to construct and are particularly useful for the fast replacement of superstructures that have suffered damage (Chapter 13).

There are many designs of timber decks, ranging from the famous covered truss bridges in the USA, some of

which have been in service for more than a hundred years, to knee-braced beams and glue-laminated arches. Though these designs are not considered appropriate to the present manual, which focuses on small bridges and simple procedures, they may be viable where specialised expertise is available. Further designs of timber beam decks, braced and trussed, may be found in Masani (1952), Federal Highway Administration (1969), Canadian Institute of Timber Construction (1970) and Ritter (1990).

Where logs or sawn timber beams are unavailable or very expensive, truss-type decks may be viable. One such design, which uses multiples of a standard prefabricated frame, is described and detailed in TRADA (1985) and Parry (1981).

Two other useful reference books for engineers wishing to design timber decks from first principles are Pearson, Kloot and Boyd (1962) and American Society of Civil Engineers (1975). The classification of structural timber for use in bridges is discussed in Section 14.4.2.

11.1 Log bridges

Log bridges, such as the one with no parapet shown in Figure 11.1, are common on rural access roads in many countries. Figures 11.2 and 11.3 give details of a standard design for a single lane log bridge to carry AASHTO H520-44 loading. Whenever possible seasoned logs should be used. They should be closely matched and positioned the same way, ie with the larger diameter ends all at the same end of the deck, so that the decking planks can be fixed in contact



Figure 11.1 Timber bridge on a minor road

with all the logs. It is better to notch the underside of those logs which are high at the abutment than to remove a greater quantity of material along the upper surface of the logs. The maximum span is usually limited by the size of tree found locally, or by the maximum length which can be transported.

11.2 Sawn beam bridges

Rectangular timber beams are also used as the main spanning members for bridges. Construction is easier with such a regular shape, since each member rests on a flat surface and fixing of the decking to the beams is more positive. Cutting allows close examination, so timber can be carefully selected to avoid serious defects. Considerable time, effort and cost, however, may be incurred in transporting logs to a saw mill, and the sawn section is not as strong as the log from which it was cut.

Figure 11.4 shows standard designs for sawn beam bridges for spans up to 12m to carry AASHTO HS 20-44 loading.

11.3 Timber decking

The deck is constructed by using timber boards, usually 100mm thick, set across the top of the logs. To allow water to drain easily and to avoid dirt and moisture being trapped between deck planks, an air gap of 20mm is left between the planks. In practice the logs are rarely uniform enough to fix all decking planks directly to each log. Rather than force a plank down onto a log, hardwood packing pieces should be used to make up the gap, as shown in Figure 11.3.

To protect the deck timbers and encourage drivers to position their vehicles centrally on the bridge, two strips of longitudinal running boards are fixed to the decking at centres that will accommodate the wheel spacing of conventional motorised vehicles. The strips should also be sufficiently wide and spaced to cater for all common local vehicles, such as bullock carts and rickshaws as well as lorries. It is also beneficial to fix transverse threshold boards across the width of the bridge at each end of the running strips. They lessen the impact on the ends of the running strips and reduce the tendency for them to work loose.

Planks for the deck and running strips should be placed with the surface that was nearest to the centre of the tree face-down, so that their edges do not curl upwards on further drying.

Nailing is the least successful method of fixing the decking and running boards, since movements of the deck under traffic loads tend to pull the nails loose. Galvanised coach screws or coach bolts and nuts are far more satisfactory. The running strips in particular should be bolted to the deck planks.

An edge beam serves as a kerb and ties the ends of the deck planks together. A pedestrian handrail is provided by extending deck planks at the post positions only, to carry an inclined post brace.

11.4 Footways

Even though timber bridges tend to be confined to rural areas, some carry significant numbers of pedestrians. When there is motorised traffic using the bridge at the same time as high pedestrian flows, eg at times when children are going to and from school, a segregated footway should be provided on the main structure or separate from it.

The footway should be at least 1.5m wide: if attached to the main structure, it may be supported on one or two additional beams as shown in Figure 11.5. A standard parapet is provided at the outer edge and another between the pedestrians and motorised traffic. The posts for this parapet are best braced below the deck in order to avoid obstructions to the footway.

If all deck planks are continued across the footway, there is no need for longitudinal running strips. Alternatively, the engineer can extend only those deck planks that are required to support the outer parapet and attach continuous running strips across the full width of the footway.

11.5 Abutments

The abutments can be made of masonry or concrete, as shown in Chapter 8. It is important that the abutment bearing shelf is well drained and that air is able to circulate freely around the ends of the logs. These should sit on a raised bearer, clear of moisture, soil and debris which may collect on the bearing shelf, and they may be secured in position with timber wedges fixed to the bearer. Abutments for temporary timber bridges are discussed in Section 13.3.

11.6 References

American Society of Civil Engineers (1975). *Wood Structures - a design guide and commentary*. American Society of Civil Engineers, New York.

Canadian Institute of Timber Construction (1970). *Modern Timber Bridges - some standard details*. Canadian Institute of Timber Construction, Ottawa.

Federal Highway Administration (1969). *Standard plans for highway bridges, Volume 3 Timber Bridges*. Federal Highway Administration, Bureau of Public Roads, Washington.

Masani N J (1952). *Simple calculations in the design of forest bridges of stock spans of 152030 and 40 feet*. Survey of India, Delhi.

Parry J D (1981). *The Kenyan low cost modular timber bridge.* Laboratory Report LR970. Transport Research Laboratory, Crowthorne.

Pearson R G, Kloot N H and Boyd J D (1962). *Timber engineering design handbook.* Jacaranda Press Pty Ltd, second edition, Melbourne.

Ritter M A (1990). *Timber Bridges - design, construction, inspection and maintenance.* U S Department of Agriculture, Forest Service, Washington.

TRADA (1985). *Prefabricated modular wooden bridges.* Timber Research and Development Association, High Wycombe.

11.7 Standard drawings and tables for timber superstructures

<i>Title</i>	<i>Figure number</i>
Log or timber beam bridge - longitudinal section	11.2
Log bridge - cross section	11.3
Timber beam bridge - cross section	11.4
Timber decks with segregated footway	11.5

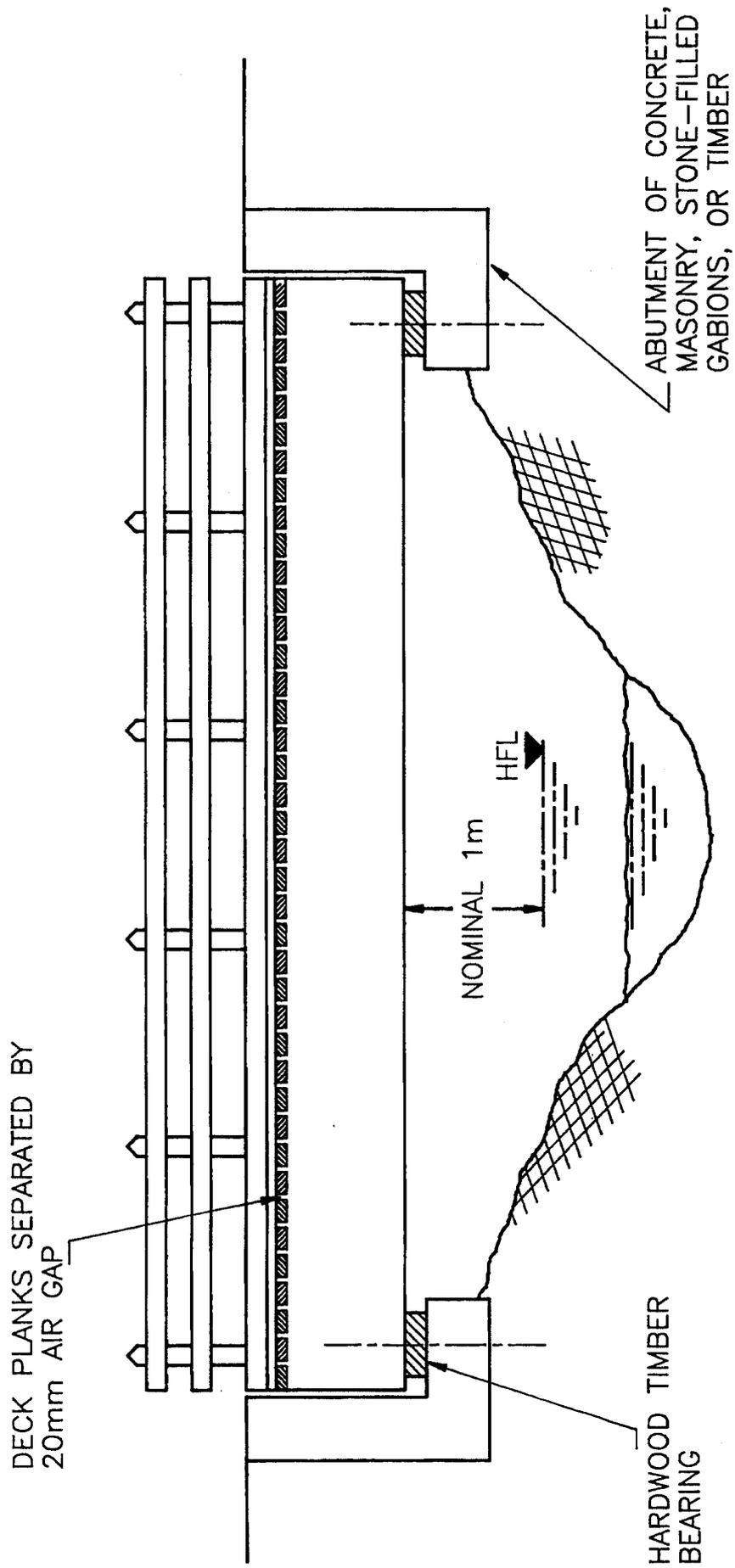
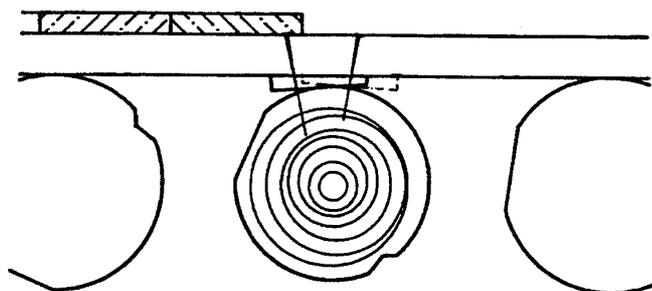
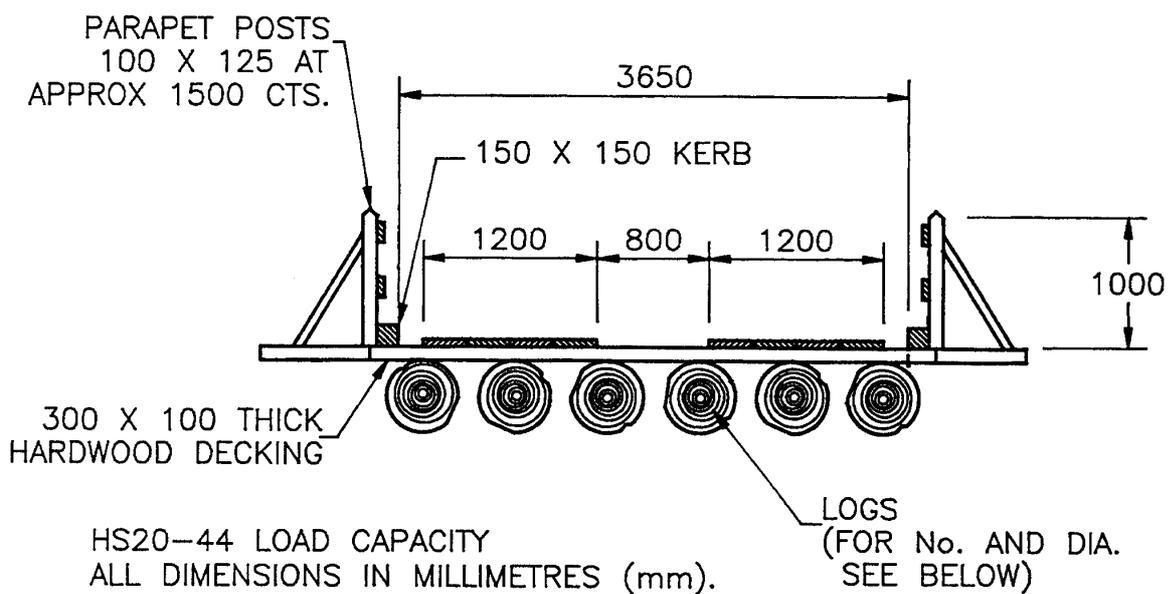


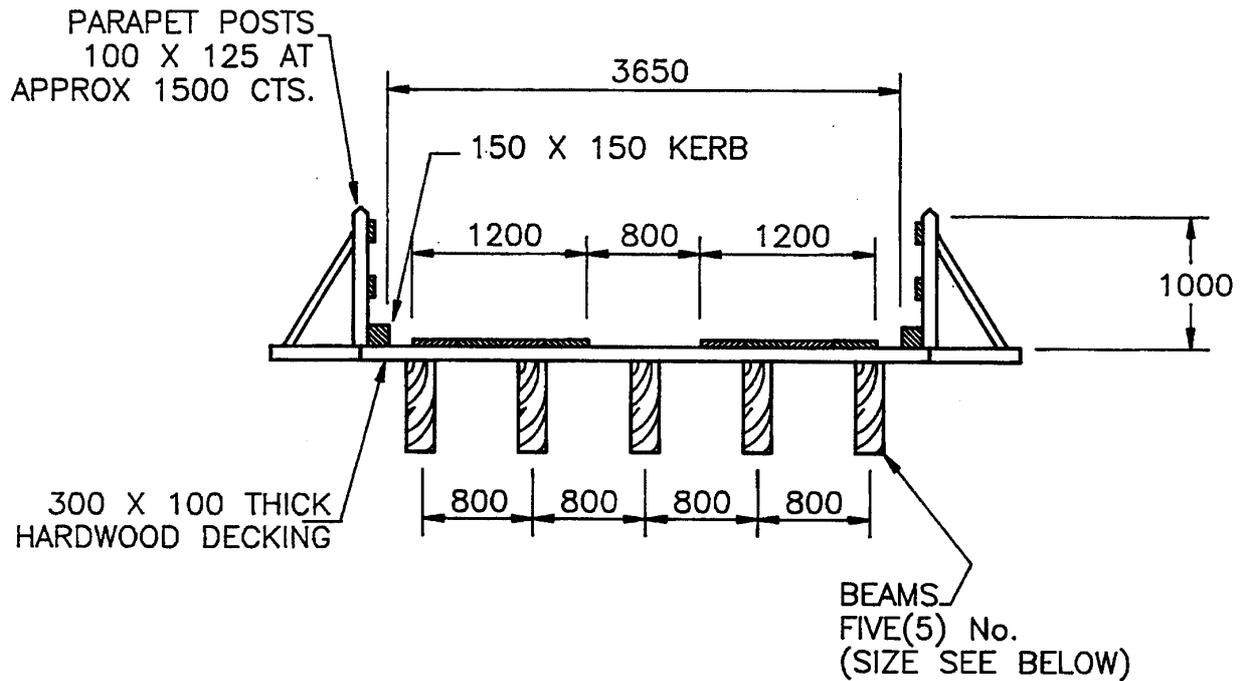
Figure 11.2 Log or timber beam bridge longitudinal section



SPAN (m)	No.OF LOGS	DIAMETER OF LOGS FOR TIMBER GROUP		
		GROUP A	GROUP B	GROUP C
4	7	325	400	475
6	6	375	475	550
8	6	450	525	625
10	5	500	600	725
12	5	550	675	800

NOTE: DIAMETER GIVEN IS MINIMUM ALONG CENTRAL THIRD OF SPAN
AND REFERS TO LOGS STRIPPED OF BARK

Figure 11.3 Log bridge cross section



HS20-44 LOAD CAPACITY
ALL DIMENSIONS IN MILLIMETRES (mm).

SPAN (m)	BEAM SIZE FOR TIMBER GROUP		
	GROUP A	GROUP B	GROUP C
4	150 X 375	150 X 500	200 X 550
6	150 X 475	200 X 550	200 X 700
8	200 X 500	200 X 650	250 X 750
10	200 X 600	250 X 725	300 X 850
12	200 X 700	250 X 850	300 X 1000

Figure 11.4 Timber beam bridge cross section

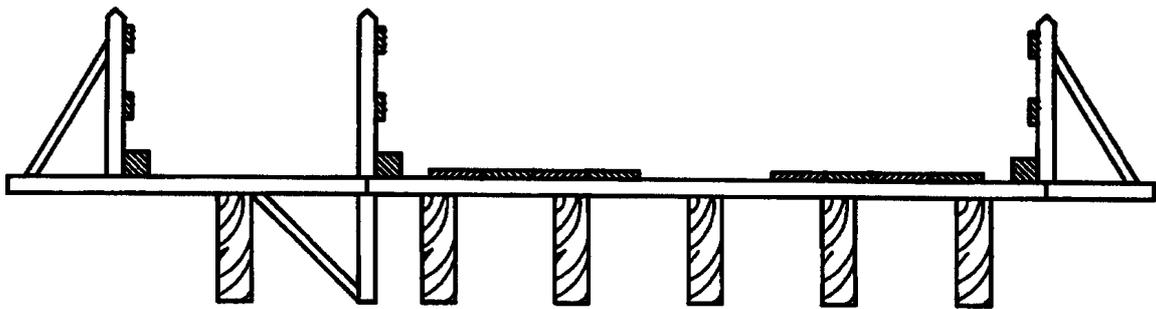
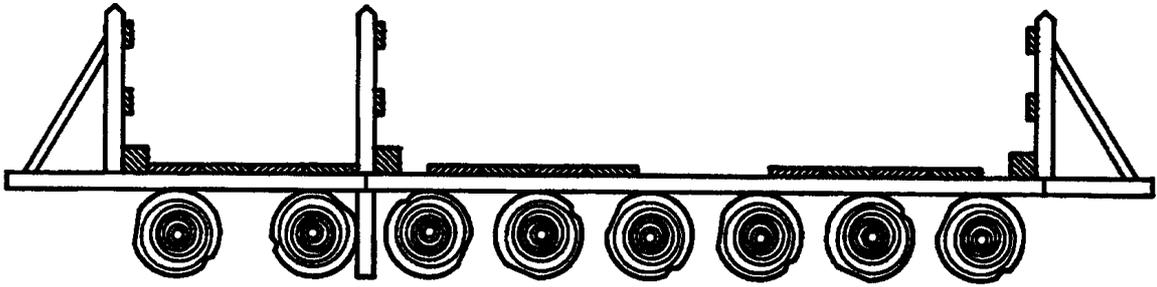


Figure 11.5 Timber decks with segregated footway

21 Culverts

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12 Culverts

There are many similarities between bridges and culverts, and they perform similar tasks. Bridges, however, usually accommodate longer spans: they consist of free-standing abutments and a separate articulated superstructure which carries the traffic. Culverts are often made of pre-fabricated pipes or boxes, or are cast in one or two pieces: they are usually set low in an embankment and less often bear the direct weight of traffic.

Where the waterway opening is less than about 15m², and particularly where the road crosses the waterway on a relatively high embankment, a culvert will usually be cheaper than a bridge.

The hydraulic design of culverts is discussed in Section 5.6.1.

Culverts may be constructed using a variety of materials from vitreous clay to glass reinforced plastics, but the most common and cheapest forms of construction are:

- precast concrete jointed pipes;
- corrugated steel pipes made of prefabricated panels;
- single or multi-cell reinforced concrete boxes, prefabricated or built in situ.

Single precast concrete pipe culverts are commonly used for small openings up to 2m² while multiple concrete pipes with common headwalls or corrugated steel pipes cater for larger areas. Alternatively, reinforced concrete box culverts are used with internal box sizes up to 4m x 3.5m. Twin or multiple boxes may be required for larger waterway openings.

In difficult ground conditions a flexible steel pipe has an advantage over a rigid concrete culvert through its ability to accommodate a certain amount of differential settlement over the length of the culvert without overstressing the material. A culvert made of rigid concrete sections will not be tolerant to differential settlement unless it is specifically designed for such conditions either by increasing its structural strength or by segmenting the culvert along its length to allow it to flex. In theory such a solution may appear to be satisfactory, but the provision of watertight joints to permit flexing to occur can be costly and the satisfactory performance of the joints cannot always be guaranteed. Leakage from culverts, particularly those carrying embankments, can have serious consequences, since weakening of the embankment material may lead to an embankment failure.

If properly constructed, a reinforced concrete culvert is likely to have a service life in excess of 60 years and will almost certainly be more durable and require less maintenance than a steel pipe. By comparison, a

corrugated steel pipe culvert, well protected against corrosion by plating and bitumen coating, can be expected to have a working life in the order of 30 to 40 years in a non-aggressive environment. It is usual to design culverts to last the life of the highway. Despite the best efforts of the engineer, it must be expected that some culverts will become silted or obstructed by debris. For this reason, pipes of internal diameter less than 0.6m are not recommended since they are difficult to clean.

12.1 Erosion control

The construction of a road and associated drainage structures may have the effect of taking general water run-off from a slope and channelling it. This concentration of flow is almost certain to cause erosion damage to many types of soil.

Before or at the entrance to culverts it is important to control the flow velocity, particularly since an abrupt change in direction of flow may occur at these points making erosion damage more likely. Control may be achieved by reducing the gradient, installing check-drains or by building a drop inlet.

It is also important to control the velocity of flow at the outlet, because erosion damage to the road embankment or surrounding farmland occurs more frequently at the discharge end. For this reason, where gradients are steep, the arrangements shown in Figures 12.1 b and 12.1 c are preferred to that shown in Figure 12.1 a.

A curtain wall is often necessary at the outlet of a culvert carrying more than a minimal flow, in order to prevent scour undermining the structure.

A concrete or rip rap apron may be required to dissipate the energy of the discharge, or it may be necessary to contain the flow in a paved channel until the natural gradient becomes small enough not to induce fast flows with consequential deepening of the channel. Figure 12.2 shows the most common failure of culverts installed in soft soils, due to erosion and undermining which progress, if not checked, to the point of embankment collapse.

12.2 Location and alignment

The proper location of each culvert is important because it affects the drainage pattern and hence the strength of the surrounding soils, the area of waterway, ditch and culvert maintenance, and erosion damage.

There are four rules which, though obvious to experienced engineers, are worth listing here:

- Where a road crosses a valley, the lowest point requires a vent, whether there is an established stream or not.

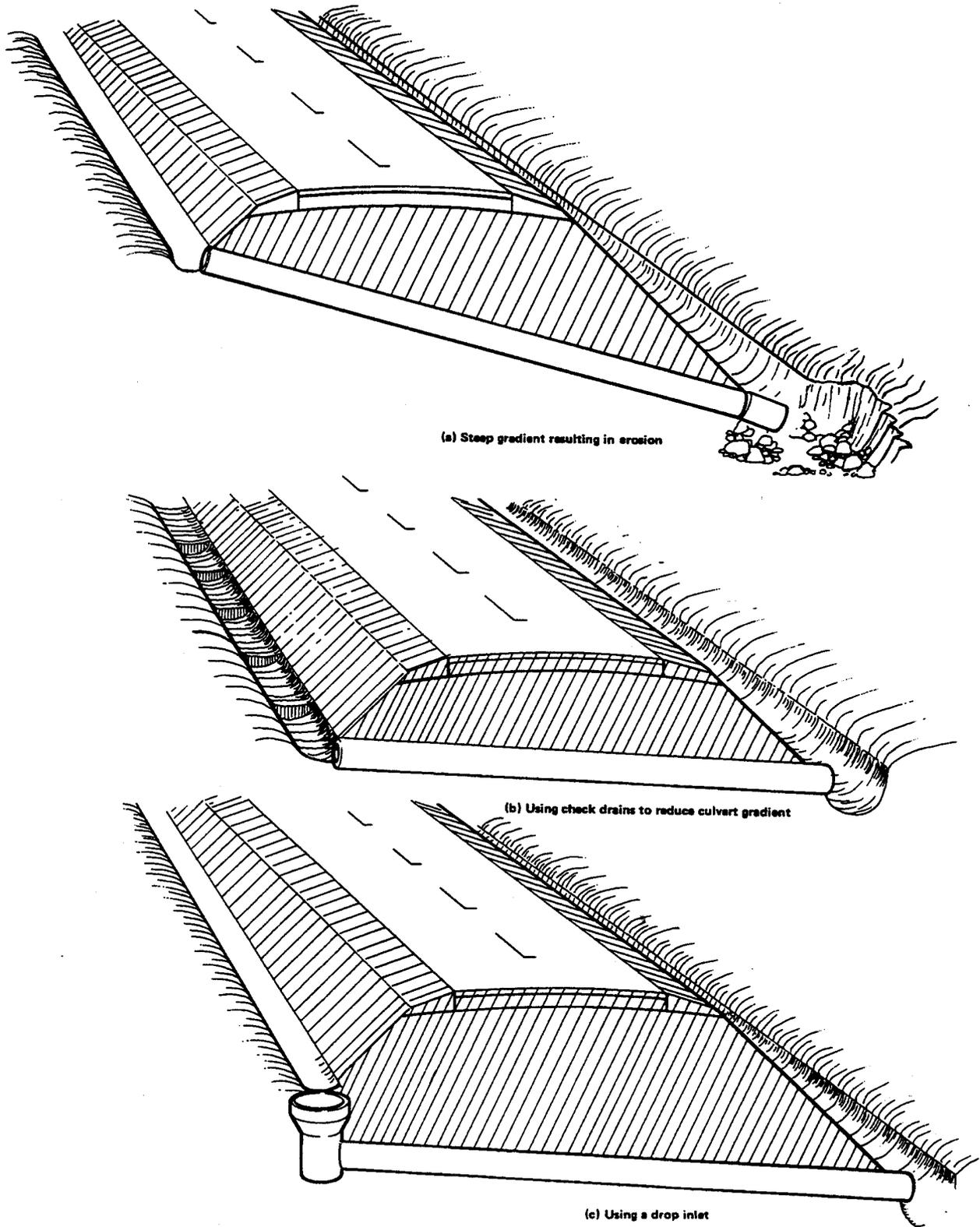


Figure 12.1 Reducing water velocity through a culvert

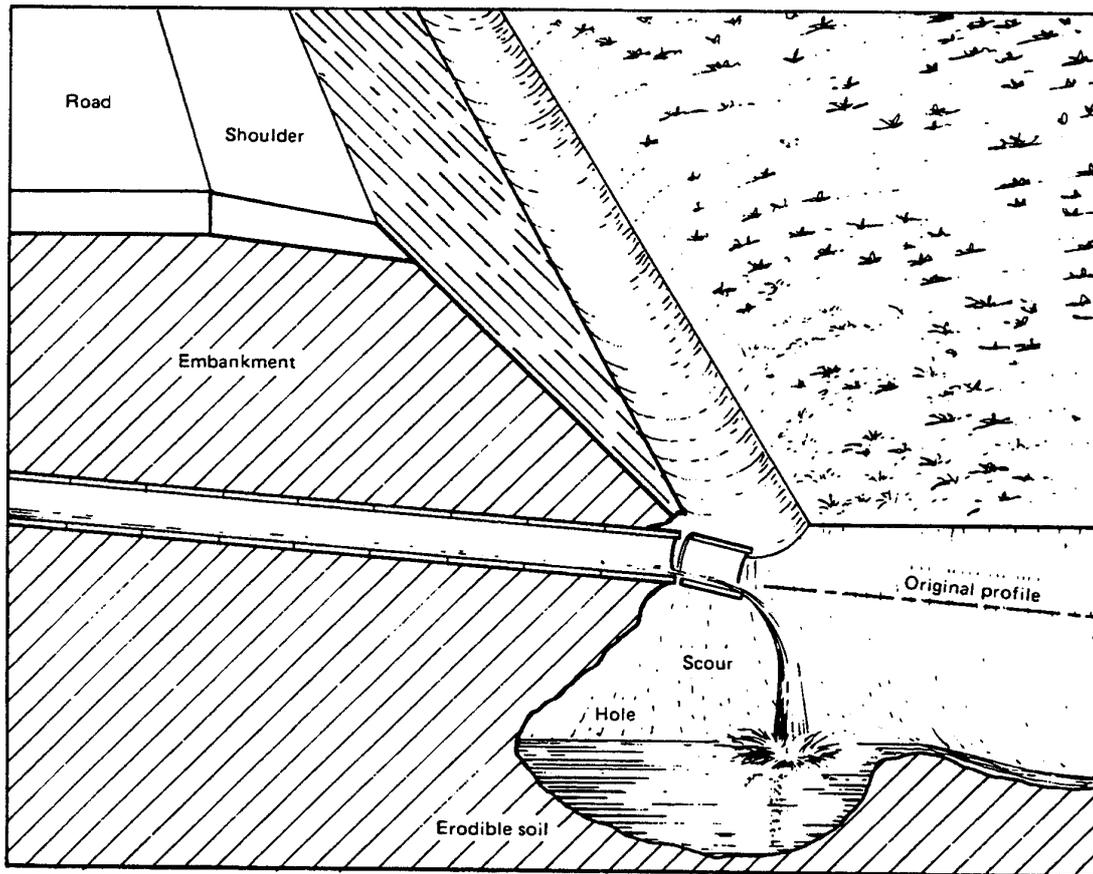


Figure 12.2 Typical erosion where a culvert discharges into erodible soil

- Where there is an established stream, the culvert should follow the existing alignment, unless the alignment can be improved.
- The gradient of the culvert should be the same as the gradient of the stream.
- Measures may be necessary to ensure that the watercourse does not move. This could cause severe damage and the consequent change of location of the culvert would be expensive.

As well as venting at the lowest point, it is good practice to install culverts for cross drainage at regular intervals down a long grade. This avoids the necessity of building a large culvert at the bottom of the grade and may also provide the opportunity for safer dispersal of water in smaller mitre drains on the lower slope. The appropriate frequency of these cross drains is best decided on the basis of local experience and depends on gradient, soil characteristics, intensity of rainfall and related factors. As a general rule, there should be at least one culvert every 300m, unless the road follows a ridge.

The gradient of the culvert is important because it affects future maintenance. If it is too steep, it will encourage erosion at the outlet. If it is less steep than

the stream, there is likely to be a sediment problem. A gradient of 2 to 4 per cent is advisable where silts are carried in the flow; a minimum of 0.5 per cent is recommended for clear water.

It is also important to set the culvert invert at the same level as the natural stream bed. Culverts are frequently set low to avoid humps in the road above, which results in silting and a consequent reduction in the waterway. Where the road embankment is not high enough to provide adequate clearance above the pipe, the ditch has to be lowered on both sides of the road or the embankment raised. This may necessitate widening the side slopes and demonstrates the importance of detailing drainage structures early in the design of a road.

Where an established stream is met at an angle to the road alignment, it is usually better to follow the line of the stream with a skewed culvert, even though the construction cost will be increased by the greater length. An abrupt change in the direction of flow at the inlet and outlet of a square crossing may be accommodated without problem for most of the year, but when heavy rains cause maximum flow the change of direction can result in severe erosion or wash-out.

It may also be necessary to stabilise a shifting channel or even move it to improve the geometry where stream and road interact. The principal objectives here are:

- to avoid a bend at either end of the culvert if possible;
- if a bend is unavoidable, to place it at the outlet.

Any change of stream channel must be constructed so that there is no possibility of the stream regaining its original course. Figure 12.3 taken from Armco (1938) illustrates these principles.

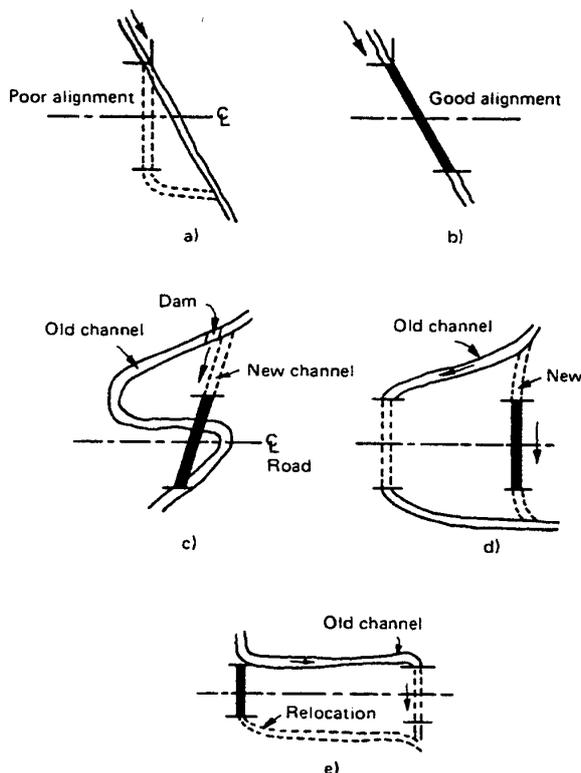


Figure 12.3 Preferred culvert alignments

On some minor roads, ditches cross small entrances and are blocked, either deliberately or by the action of traffic. Where pipe culverts are installed, they are often placed away from the road to reduce the length. It is better for the long-term performance of the road to install pipe culverts at the outset and locate them as shown in Figure 12A.

Where ditches are spaced away from the carriageway, culverts should be built from ditch to ditch, not just across the width of the pavement, so as to move the location of possible erosion damage away from the pavement. Building the culvert long enough to reach the base of the embankment also reduces the additional cost of building high headwalls.

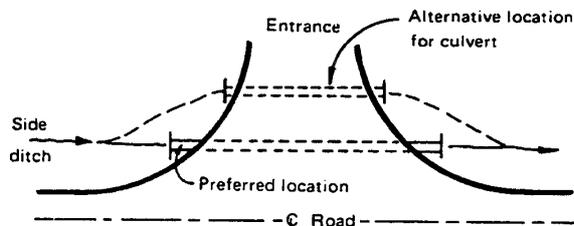


Figure 12.4 Culvert location in a side ditch

12.3 Headwalls

Headwalls retain and protect the embankment at the ends of a culvert and help to counteract the dislocation of jointed pipes due to the spreading forces at the base of large embankments. They also divert debris from falling into the waterway, reduce seepage of water into the embankment and prevent small animals from burrowing alongside the pipe. Where conditions favour erosion, headwalls should incorporate deep curtain walls and side slope protection in the form of gabions or hand-pitched stone.

Since large headwalls are expensive to build (Figure 12.5), it is advisable initially to make the culvert pipe long enough to reach the toe of the embankment and to monitor the effects for a year: a small headwall may be adequate, provided that side slopes are shallow enough not to need retaining, embankments are relatively low or one-piece pipes are employed, and there is no build-up of water.

Headwalls are usually made of masonry or concrete. Figures 12.6 and 12.7 show standard designs for large headwalls. Small walls may be built to similar proportions, leaving out the reinforcement. Matching retaining walls, where required, can be detailed using Figures 8.12 and 8.13 or Figures 8.16 and 8.17. Headwalls, side slopes and retaining walls are illustrated in Figures 7.5, 12.8 and 12.12. Sheet metal headwalls can be purchased with sheet metal pipes.

12.4 Concrete pipe culverts

On rural access earth roads, sections of concrete pipe are butted together and the embankment material is compacted around them. This is not the most permanent construction, but is cheap and often serves adequately for several years. A far better quality culvert is obtained by surrounding the pipe with concrete (Figure 12.8 and Figure 12.9). The concrete cladding resists the tensile forces that open the pipe joints and strengthens the pipe against bending. If the subgrade is likely to settle under the weight of the embankment, the concrete surround should be reinforced no matter what the depth of cover between pipe and road surface.

All soft clay material and rock should be excavated

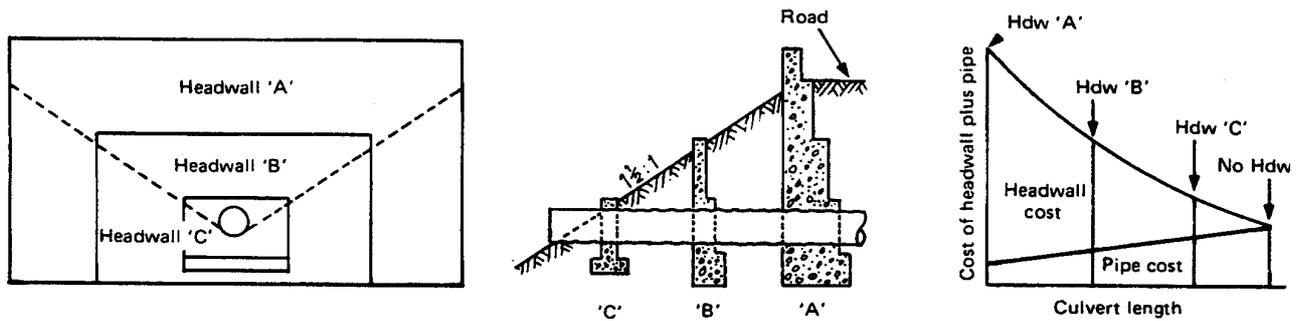


Figure 12.5 Illustration of headwall costs

from below the pipe to a depth of at least 200mm and replaced with good granular material, well compacted.

Sections of concrete pipe may be bought locally or manufactured in-house using steel moulds. The relevant British Standard Specification is BSI (1982) or, for pipes with flexible joints, BSI (1988).

12.5 Flexible steel culverts

These are made from pre-formed corrugated sections bolted together to form a circular or elliptical tube. They can accept a certain amount of longitudinal or radial deformation without sustaining damage and so are better suited to unstable conditions than rigid concrete structures. They are also easier to transport and faster to construct than the equivalent size of pre-cast pipe culvert, but require specialist manufacture.

Corrugated steel sections may also be used as permanent formwork for masonry arch structures.

12.5.1 Foundations

Although they are to some degree flexible, for optimum performance and service life corrugated steel pipes require a firm foundation that distributes the load evenly. Where settlement is likely to occur it will be greatest under the centre of the embankment: the engineer should therefore consider the provision of a longitudinal camber to compensate for any tendency for the pipe to sag over its length. Normal practice is to adopt a camber of 1 per cent of the pipe length. To avoid the possibility of upstream ponding, the upstream half of the pipe can be laid level and all the drop provided in the downstream half.

Uneven foundations

When the excavation crosses soft or hard spots, the foundation should be made as uniform as possible by excavating rocks, clay pockets, etc, below the proposed foundation level and replacing good selected material.

Soft foundations

All soft unstable material should be excavated and backfilled to foundation level with sand/gravel mixture, crushed stone or other suitable material.

Swampy foundations

Where deep unstable foundations are encountered which cannot be stabilised with granular material, timber fascines can be used to spread the load.

Rock foundations

Rock should be excavated to at least 250mm below the foundation level and wide enough to prevent the pipe resting directly on rock at any point. It should then be backfilled to provide a cushion for the pipe.

12.5.2 Backfill

The first stage is to settle the assembled pipe onto the foundation material and compact the fill under the haunches. At this stage it may be necessary to load the inside, in order to prevent the pipe being wedged upwards by the fill.

Backfilling is continued in layers of approximately 200mm, loading each side evenly and fully compacting each layer before placing the next. The inside shape may need to be protected by strutting as the fill is built up on each side and eventually above the pipe.

All the fill material should be of good quality, extending to a distance roughly equal to the pipe diameter each side and above, or up to the road base if this distance is less.

12.5.3 Design

Corrugated metal pipes are made by specialist manufacturers who will supply technical publications on the detailed design of pipes and culverts, including notes on hydraulic design. Manufacturers also offer a design service to the engineering industry. The names and addresses of two manufacturers are given in Section 12.7.

12.5.4 Durability and corrosion protection

The degree of protection against corrosion required to extend the working life of a corrugated steel pipe structure is determined on the basis of:

- the importance of the structure and its required working life;
- the corrosive nature of the environment in which it exists.

Durability studies indicate that metal loss associated with corrosion usually originates on the interior surface of the structure and progresses towards the exterior or buried face. The conclusion is that protective treatment to the exposed surfaces should generally be superior to that applied to the buried surfaces. All corrugated steel structures should be hot-dipped galvanised or treated with alu-zinc and hot coated with bitumen on the inside. Bituminous paving to the invert of the pipe will also help extend the life of the structure.

12.6 Reinforced concrete box culverts

Reinforced concrete box culverts may be constructed either as a monolithic box or as a U section with a simply supported top slab propping the side walls.

Twin or multiple cell box culverts are used where the horizontal opening is more than about 4m. These too may be constructed with monolithic top slab and walls or with a separately cast lid.

Culverts with separately constructed top slabs are simpler to construct because they require less complicated steel reinforcement and, though containing more material, they generally offer a more economical form of construction. For this reason monolithic boxes are not considered further. The side walls of the culverts detailed in this chapter are constructed monolithically with the base slab and are propped at the top by the simply supported roof slab. Owing to the reversal of the bending moment in the walls and bottom slab, reinforcement to both faces of these elements is required but the top slab of a single cell culvert requires only bottom reinforcement.

For a twin box culvert with a separately cast top slab, the outer walls are again propped apart and the centre wall may be positively located to the top slab by the provision of dowel bars. The top slab of a twin box culvert is designed as a two span continuous beam spanning over the central wall and requires top reinforcement in the region of the central wall.

Standard designs for single, twin and multiple box culverts are given in Figures 12.10 and 12.11, and Tables 12.1 to 12.20. Figure 12.12 shows typical details of single cell box culvert inlet and outlet.

Thorough compaction of the fill material on both sides of the concrete structure is essential to prevent settlement under traffic and to preserve the good vertical profile of the carriageway. If past experience suggests that there may be excessive settlement of the embankment on either side of a box culvert, the engineer should consider the inclusion of run-on slabs, as described in Section 8.8.

12.7 References

Armco (1938). *Handbook of culvert and drainage practice.* Armco Ltd, London.

BSI (1982). *Precast concrete pipes for drainage and sewerage Part 3, specification for pipes and fittings with ogee joints.* BS 5911:1982 Pt 3. British Standards Institution, London.

BSI (1988). *Precast concrete pipes for drainage and sewerage Part 100, unreinforced and reinforced pipes and fittings with flexible joints.* BS 5911:1988 Pt 100. British Standard Institution, London.

Enquiries concerning sheet metal culverts may be made to:

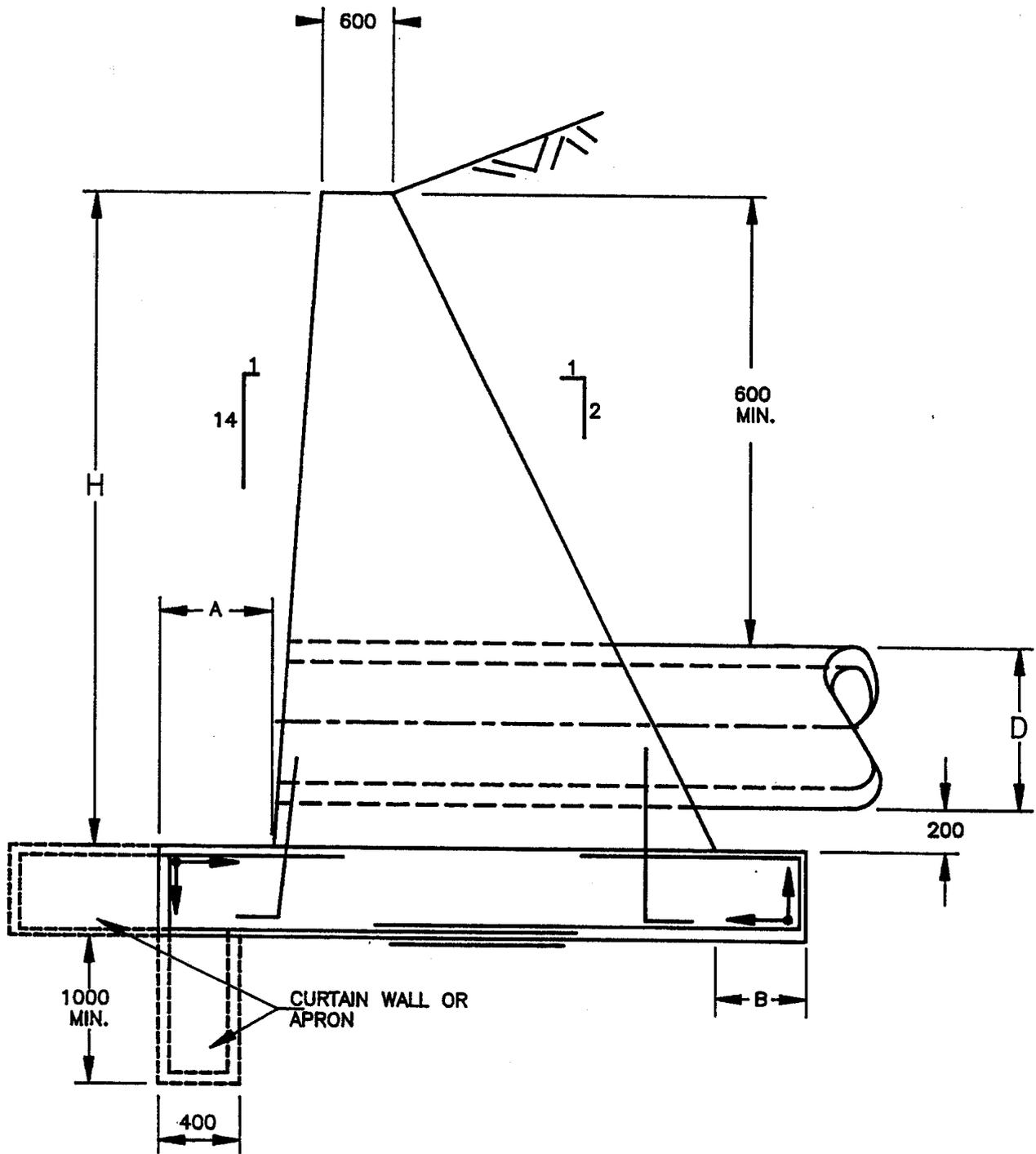
Armco Ltd. 76 Jermyn Street, London SW1Y 6NP, England.

Wells Spiral Tubes Ltd. Prospect Works, Airedale Road, Keighley, West Yorkshire BD21 4LW, England.

12.8 Standard drawings and tables for concrete box culverts

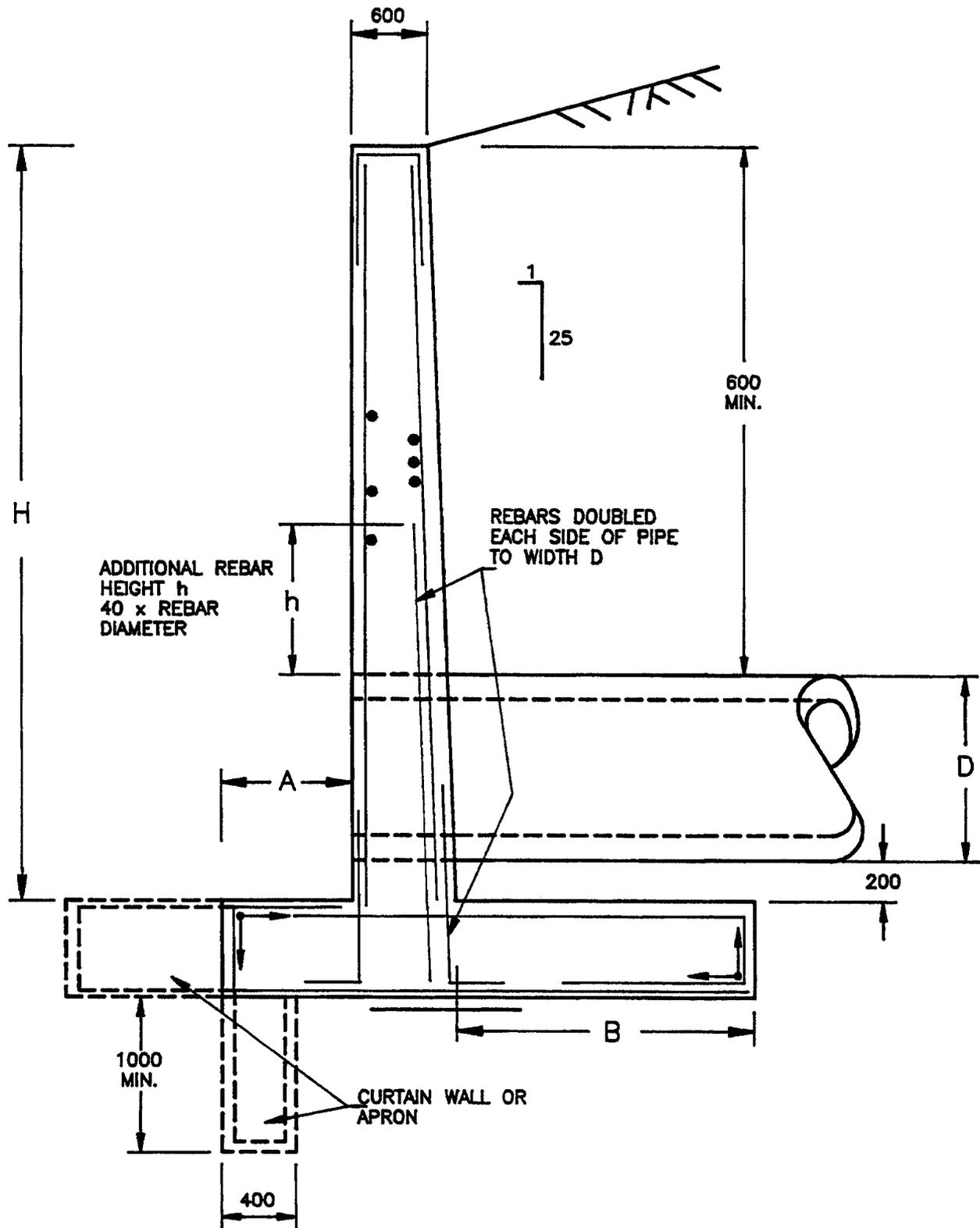
12.8.1 Index of drawings

<i>Title</i>	<i>Figure numbers</i>
Mass concrete headwall for a high embankment	12.6
Reinforced concrete headwall for a high embankment	12.7
Typical details of a pipe culvert with headwalls	12.8
Details of concrete pipe surrounds	12.9
Single box culvert - section	12.10
Twin box culvert - section	12.11
Typical details of single cell box culvert inlet and outlet	12.12



MINIMUM WIDTH $4 \times D$ OR SEE FIG.12.8
 FOR FOUNDATION SIZES AND REINFORCEMENT DETAILS SEE
 TABLES 8.5 AND 8.6, AND FIG.8.13.
 FOR CORRESPONDING RETAINING WALLS SEE FIG.8.13.
 ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 12.6 Mass concrete headwall for a high embankment



MINIMUM WIDTH $4 \times D$ OR SEE FIG 12.8
 FOR FOUNDATION SIZES AND REINFORCEMENT DETAILS
 SEE TABLES 8.11 TO 8.13 AND FIG.8.17.
 FOR CORRESPONDING RETAINING WALLS
 SEE FIG.8.17.
 ALL DIMENSIONS IN MILLIMETRES (mm).

Figure 12.7 Reinforced concrete headwall for high embankment

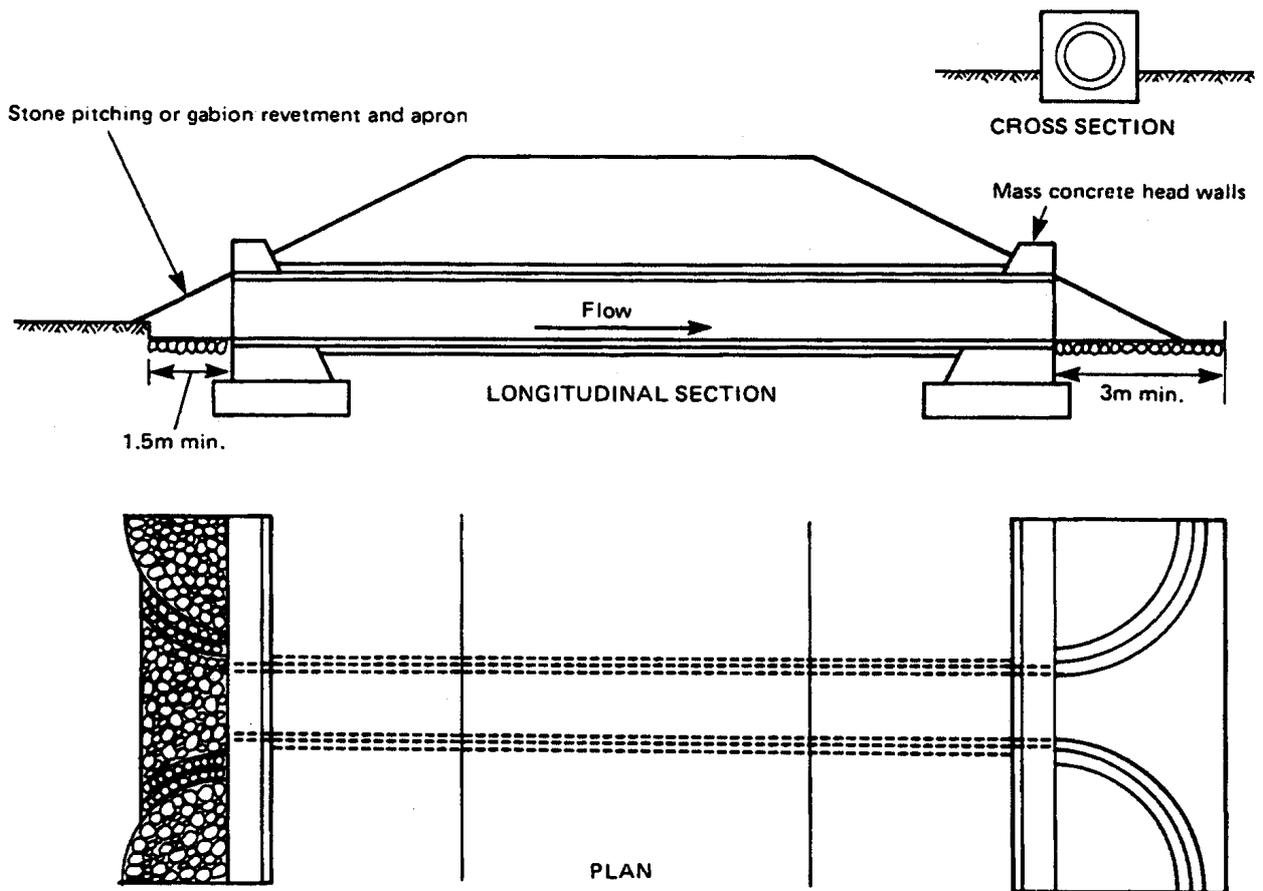
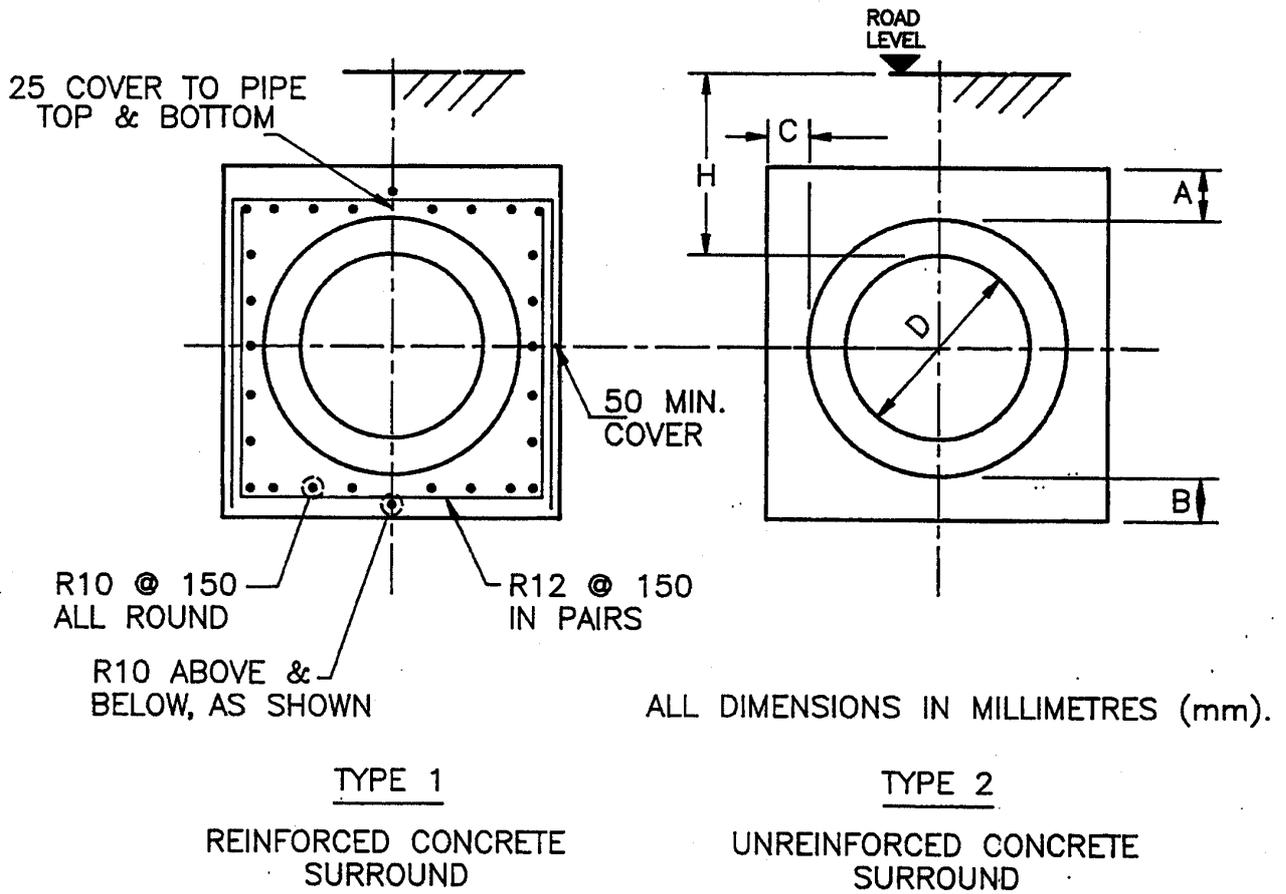


Figure 12.8 Typical details of a pipe culvert with headwalls



DIMENSIONS FOR REINFORCED & UNREINFORCED CONCRETE

D (m)	A (mm)	B (mm)	C (mm)
0.60	250	150	175
0.75	250	150	175
0.90	275	175	200
1.00	300	200	225
1.20	300	200	225
1.50	375	275	300
1.60	400	300	325

		SURROUND TYPE				
		0.5 (0-0.5)	4.0 (0.51-4.0)	5.0 (4.1-5.0)	6.0 (5.1-6.0)	6.5 (6.1-6.5)
H (m)	D (m)					
0.60	0.60	1	2	2	2	2
0.75	0.75	1	2	2	1	1
0.90	0.90	1	2	2	1	1
1.00	1.00	1	2	2	1	1
1.20	1.20	1	2	1	1	1
1.50	1.50	1	2	1	1	1
1.60	1.60	1	2	1	1	1

Figure 12.9 Details of concrete pipe surrounds

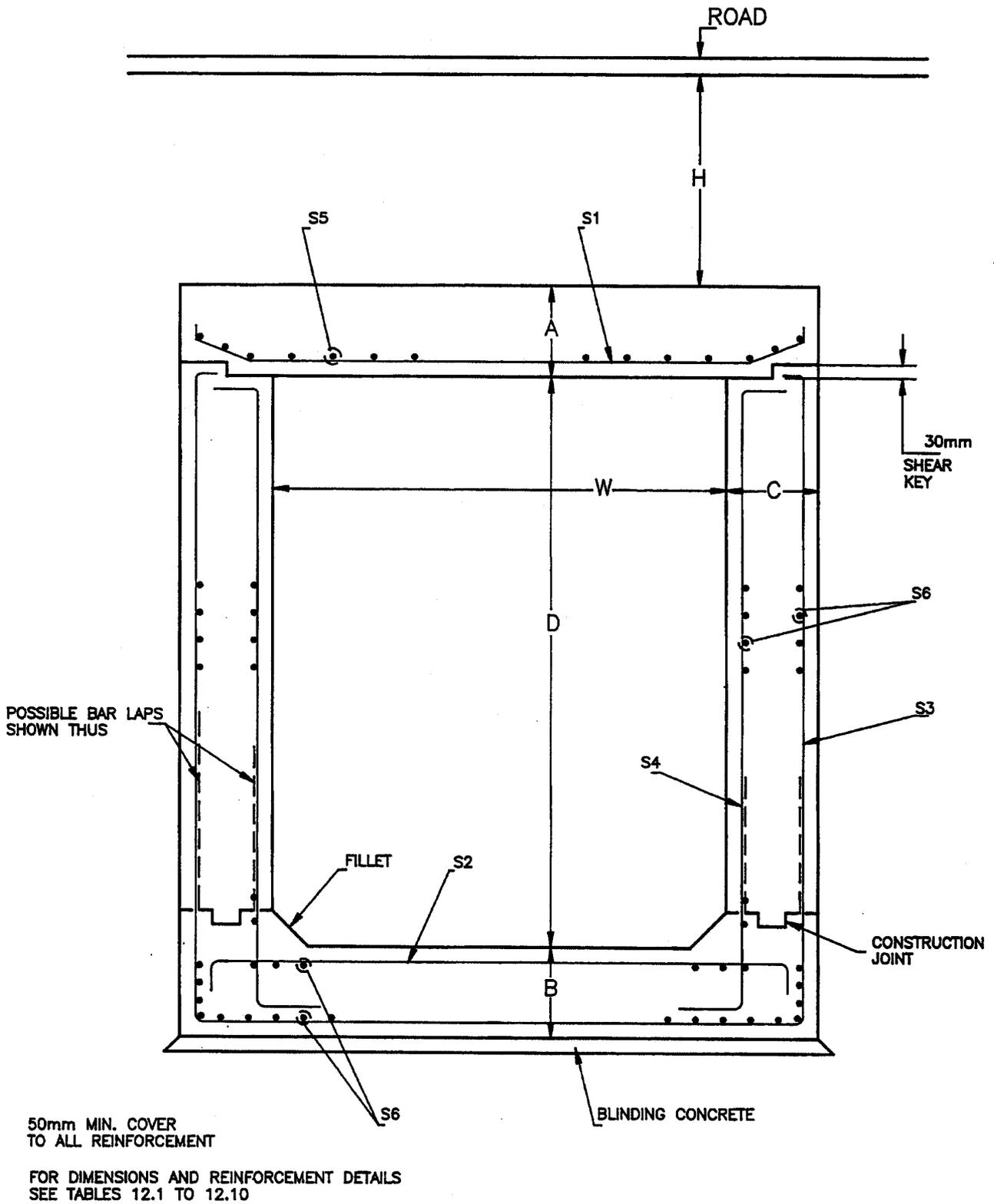


Figure 12.10 Single box culvert - section

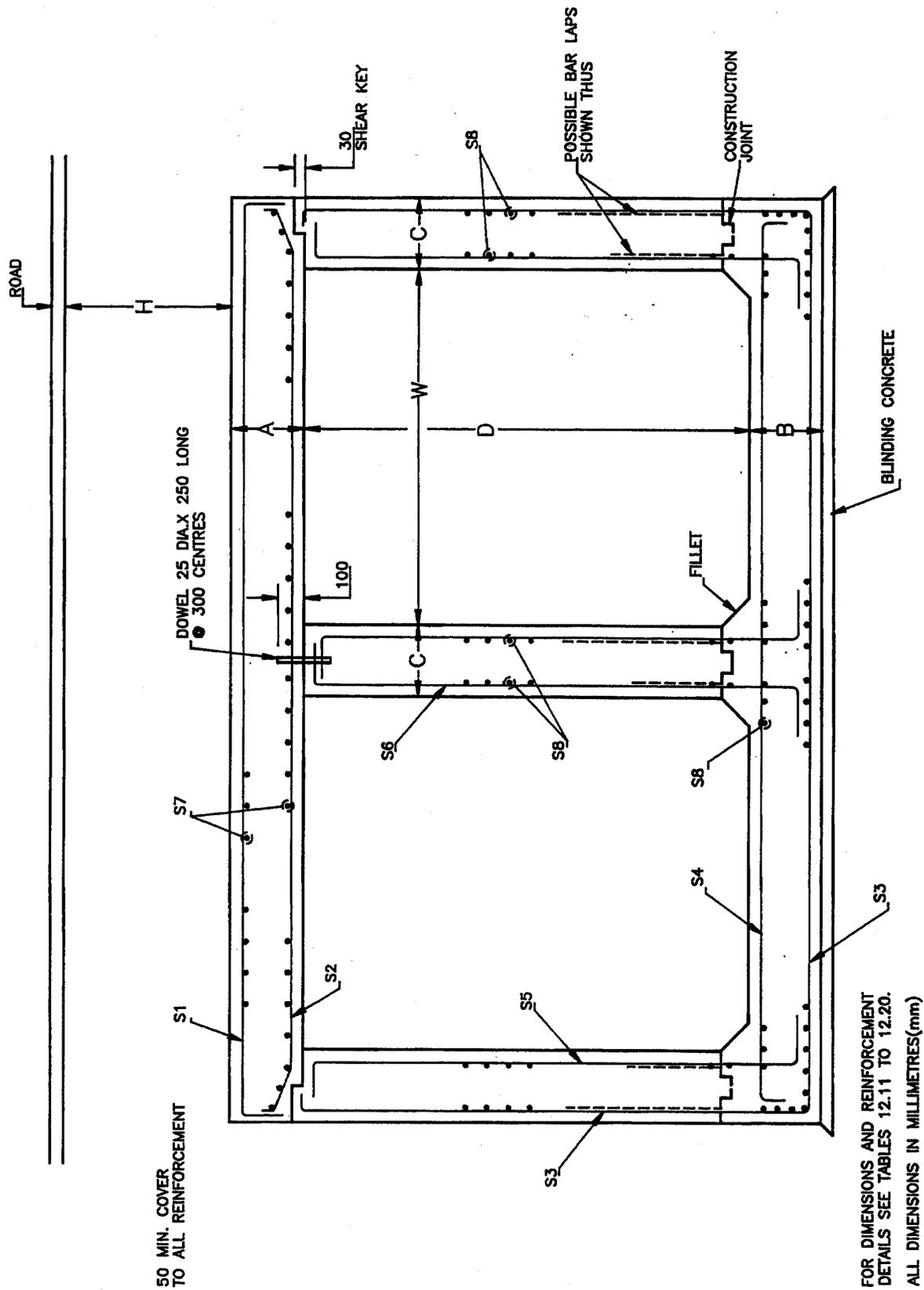


Figure 12.11 Twin box culvert - section

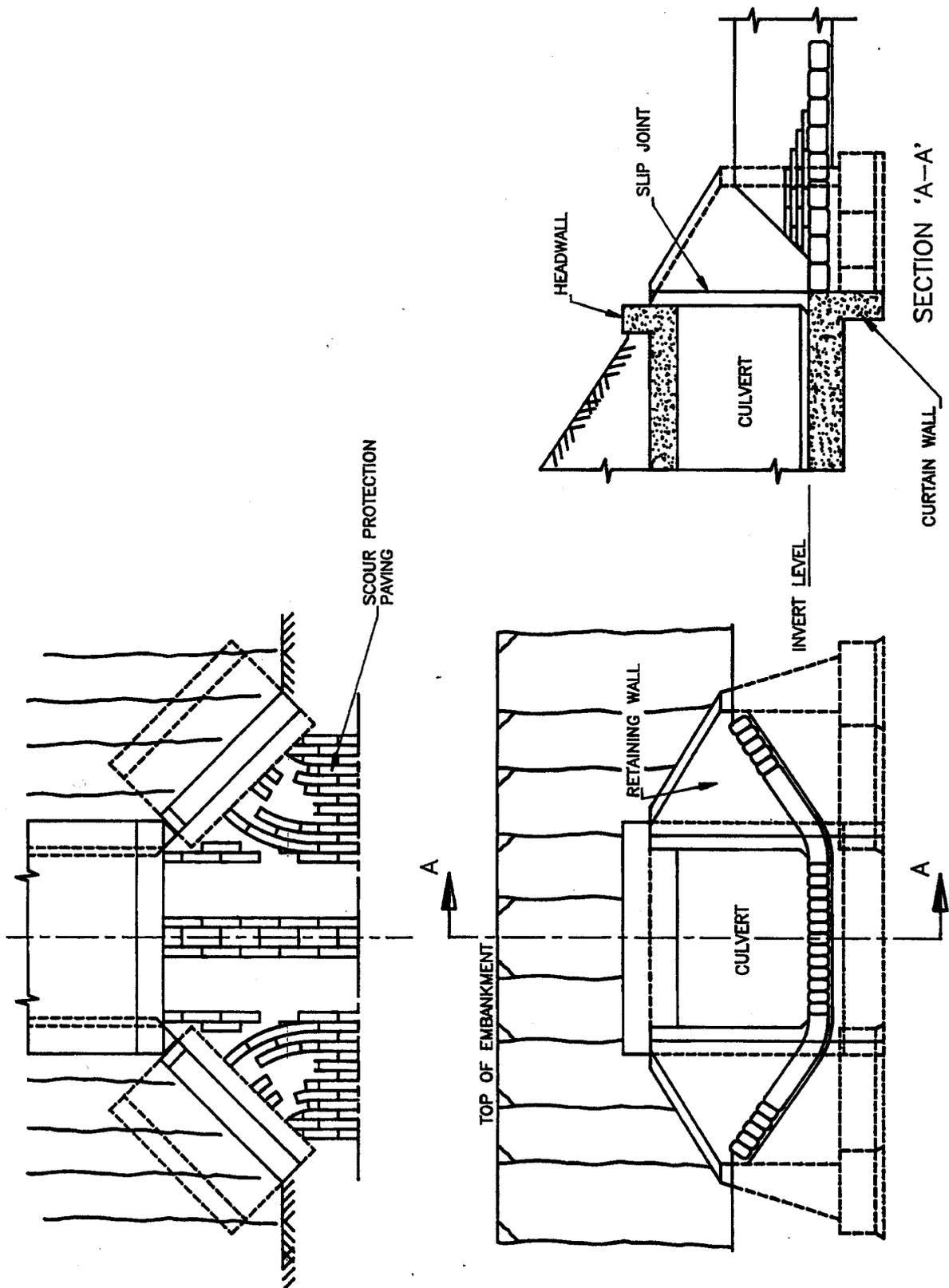


Figure 12.12 Typical details of single cell box culvert inlet and outlet

12.8.2 Index of tables

<i>Title</i>	<i>Height of fill</i>	<i>Table number</i>
Single box culvert - high yield steel	H = 0.5m	12.1
Single box culvert - high yield steel	H = 1.0m	12.2
Single box culvert - high yield steel	H = 2.0m	12.3
Single box culvert - high yield steel	H = 4.0m	12.4
Single box culvert - high yield steel	H = 6.0m	12.5
Single box culvert - mild steel	H = 0.5m	12.6
Single box culvert - mild steel	H = 1.0m	12.7
Single box culvert - mild steel	H = 2.0m	12.8
Single box culvert - mild steel	H = 4.0m	12.9
Single box culvert - mild steel	H = 6.0m	12.10
Twin box culvert - high yield steel	H = 0.5m	12.11
Twin box culvert - high yield steel	H = 1.0m	12.12
Twin box culvert - high yield steel	H = 2.0m	12.13
Twin box culvert - high yield steel	H = 4.0m	12.14
Twin box culvert - high yield steel	H = 6.0m	12.15
Twin box culvert - mild steel	H = 0.5m	12.16
Twin box culvert - mild steel	H = 1.0m	12.17
Twin box culvert - mild steel	H = 2.0m	12.18
Twin box culvert - mild steel	H = 4.0m	12.19
Twin box culvert - mild steel	H = 6.0m	12.20

Table 12.1 Single box culvert – high yield steel, H = 0.5m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	120	250	250	250	T16 @ 175	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
	2.5	126	250	250	250	T16 @ 175	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
	3.5	134	250	350	350	T16 @ 175	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
2	1.5	105	300	250	250	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
	2.5	111	300	300	300	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
	3.5	120	300	400	400	T16 @ 150	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 200	T10 @ 175
3	1.5	90	400	350	350	T16 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 175	T10 @ 200
	2.5	96	400	400	400	T16 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 175	T10 @ 175
	3.5	102	400	400	400	T16 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 175	T10 @ 175
4	1.5	83	500	400	400	T20 @ 150	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 200	T10 @ 175
	2.5	91	500	500	500	T20 @ 150	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 200	T12 @ 200
	3.5	98	500	550	550	T20 @ 150	T16 @ 175	T16 @ 175	T16 @ 175	T12 @ 200	T12 @ 175

Table 12.2 Single box culvert – high steel, H = 1.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	130	300	300	300	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 175
	2.5	137	300	400	400	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 175
	3.5	146	300	400	400	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 200	T10 @ 175
2	1.5	118	400	400	400	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 175
	2.5	125	400	400	400	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 175
	3.5	134	400	450	450	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 175	T10 @ 150
3	1.5	103	450	450	450	T16 @ 125	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 150
	2.5	109	450	450	450	T16 @ 125	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 150
	3.5	116	450	500	500	T16 @ 125	T16 @ 175	T16 @ 175	T16 @ 175	T10 @ 150	T10 @ 125
4	1.5	95	500	500	500	T20 @ 125	T16 @ 175	T16 @ 175	T16 @ 175	T10 @ 125	T10 @ 125
	2.5	100	500	500	500	T20 @ 125	T16 @ 150	T16 @ 150	T16 @ 150	T10 @ 125	T10 @ 125
	3.5	110	500	550	550	T20 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 125	T10 @ 125

Table 12.3 Single box culvert – high yield steel, H = 2.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	128	400	300	300	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 200
	2.5	135	400	300	300	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 175	T10 @ 200
	3.5	146	400	400	400	T16 @ 200	T16 @ 150	T20 @ 150	T16 @ 150	T10 @ 175	T10 @ 175
2	1.5	120	450	400	400	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 175
	2.5	129	450	450	450	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 150
	3.5	139	450	500	500	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 125
3	1.5	112	500	500	500	T16 @ 125	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	2.5	120	500	550	550	T16 @ 125	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	3.5	127	500	550	550	T16 @ 125	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
4	1.5	107	550	550	550	T20 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	2.5	114	550	600	600	T20 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 125	T12 @ 150
	3.5	120	550	600	600	T20 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 125	T12 @ 150

Table 12.4 Single box culvert – high yield steel, H = 4.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	167	450	300	300	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 200
	2.5	177	450	400	400	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 150	T10 @ 175
	3.5	191	450	550	550	T16 @ 200	T16 @ 200	T25 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
2	1.5	162	500	500	500	T16 @ 175	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	2.5	171	500	550	550	T16 @ 175	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	3.5	182	500	600	600	T16 @ 175	T16 @ 175	T20 @ 175	T16 @ 175	T10 @ 125	T12 @ 150
3	1.5	154	600	550	550	T16 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 150	T10 @ 125
	2.5	162	600	600	600	T16 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 150	T12 @ 150
	3.5	173	600	700	700	T16 @ 125	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 150	T12 @ 125
4	1.5	147	700	500	500	T25 @ 200	T16 @ 175	T25 @ 175	T16 @ 175	T12 @ 125	T10 @ 125
	2.5	156	700	600	600	T25 @ 200	T16 @ 200	T25 @ 200	T16 @ 200	T12 @ 125	T12 @ 150
	3.5	166	700	700	700	T25 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 125	T12 @ 125

Table 12.5 Single box culvert – high yield steel, H = 6.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	205	450	300	300	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 200
	2.5	216	450	450	450	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 150	T10 @ 150
	3.5	227	450	500	500	T16 @ 200	T16 @ 150	T25 @ 150	T20 @ 150	T10 @ 150	T10 @ 125
2	1.5	202	600	500	500	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 150	T12 @ 125
	2.5	213	600	600	600	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 150	T12 @ 150
	3.5	227	600	700	700	T16 @ 200	T16 @ 200	T25 @ 200	T20 @ 200	T12 @ 150	T12 @ 125
3	1.5	193	700	500	500	T20 @ 175	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 125	T12 @ 125
	2.5	203	700	600	600	T20 @ 175	T16 @ 200	T20 @ 200	T16 @ 200	T12 @ 125	T12 @ 150
	3.5	214	700	700	700	T20 @ 175	T16 @ 200	T25 @ 200	T20 @ 200	T12 @ 125	T12 @ 125
4	1.5	188	800	500	500	T20 @ 125	T16 @ 150	T25 @ 150	T16 @ 150	T16 @ 200	T12 @ 125
	2.5	197	800	600	600	T20 @ 125	T16 @ 200	T25 @ 200	T16 @ 200	T16 @ 200	T12 @ 150
	3.5	211	800	800	800	T20 @ 125	T16 @ 175	T20 @ 175	T16 @ 175	T16 @ 200	T16 @ 200

Table 12.6 Single box culvert – mild steel, H = 0.5m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	120	250	250	250	R16 @ 150	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	2.5	126	250	250	250	R16 @ 150	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	3.5	134	250	350	350	R16 @ 150	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R12 @ 200
2	1.5	105	300	250	250	R16 @ 125	R16 @ 175	R16 @ 175	R16 @ 175	R10 @ 200	R10 @ 200
	2.5	111	300	300	300	R16 @ 125	R16 @ 200	R20 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	3.5	120	300	400	400	R16 @ 125	R16 @ 200	R25 @ 200	R16 @ 200	R10 @ 200	R12 @ 200
3	1.5	90	400	350	350	R20 @ 150	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	2.5	96	400	400	400	R20 @ 150	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	3.5	102	400	400	400	R20 @ 150	R16 @ 200	R25 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
4	1.5	83	500	400	400	R20 @ 125	R16 @ 175	R25 @ 175	R16 @ 175	R12 @ 150	R12 @ 200
	2.5	91	500	500	500	R20 @ 125	R16 @ 200	R25 @ 200	R16 @ 200	R12 @ 150	R12 @ 150
	3.5	98	500	550	550	R20 @ 125	R16 @ 175	R20 @ 175	R16 @ 175	R12 @ 150	R12 @ 125

Table 12.7 Single box culvert – mild steel, H = 1.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	130	300	300	300	R16 @ 175	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	2.5	137	300	400	400	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R10 @ 200	R12 @ 200
	3.5	146	300	400	400	R16 @ 175	R16 @ 200	R25 @ 200	R20 @ 200	R10 @ 200	R12 @ 200
2	1.5	118	400	400	400	R16 @ 150	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	2.5	125	400	400	400	R16 @ 150	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	3.5	134	400	450	450	R16 @ 150	R16 @ 200	R25 @ 200	R20 @ 200	R12 @ 200	R12 @ 150
3	1.5	103	450	450	450	R20 @ 150	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 150	R12 @ 150
	2.5	109	450	450	450	R20 @ 150	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 150	R12 @ 150
	3.5	116	450	500	500	R20 @ 150	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 150	R12 @ 150
4	1.5	95	500	500	500	R25 @ 175	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 150	R12 @ 150
	2.5	100	500	500	500	R25 @ 175	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 150	R12 @ 150
	3.5	110	500	550	550	R25 @ 175	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 150	R12 @ 125

Table 12.8 Single box culvert – mild steel, H = 2.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	128	400	300	300	R16 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R10 @ 200
	2.5	135	400	300	300	R16 @ 200	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 200	R10 @ 200
	3.5	146	400	400	400	R16 @ 200	R16 @ 150	R25 @ 150	R20 @ 150	R12 @ 200	R12 @ 200
2	1.5	120	450	400	400	R16 @ 150	R16 @ 150	R16 @ 150	R16 @ 150	R12 @ 150	R12 @ 200
	2.5	129	450	450	450	R16 @ 150	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 150	R12 @ 150
	3.5	139	450	500	500	R16 @ 150	R16 @ 125	R20 @ 125	R16 @ 125	R12 @ 150	R12 @ 150
3	1.5	112	500	500	500	R20 @ 150	R16 @ 175	R20 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	2.5	120	500	550	550	R20 @ 150	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 150	R12 @ 125
	3.5	127	500	550	550	R20 @ 150	R16 @ 125	R20 @ 125	R16 @ 125	R12 @ 125	R12 @ 125
4	1.5	107	550	550	550	R25 @ 150	R16 @ 125	R20 @ 125	R16 @ 125	R12 @ 125	R12 @ 125
	2.5	114	550	600	600	R25 @ 150	R16 @ 125	R20 @ 125	R16 @ 125	R12 @ 125	R16 @ 200
	3.5	120	550	600	600	R25 @ 150	R20 @ 200	R25 @ 200	R20 @ 200	R16 @ 200	R16 @ 200

Table 12.9 Single box culvert – mild steel, H = 4.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	167	450	300	300	R16 @ 200	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R10 @ 200
	2.5	177	450	400	400	R16 @ 200	R16 @ 200	R25 @ 200	R20 @ 200	R12 @ 150	R12 @ 200
	3.5	191	450	550	550	R16 @ 200	R16 @ 150	R25 @ 150	R20 @ 150	R12 @ 150	R12 @ 125
2	1.5	162	500	500	500	R16 @ 125	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	2.5	171	500	550	550	R16 @ 125	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 150	R12 @ 125
	3.5	182	500	600	600	R16 @ 125	R20 @ 175	R25 @ 175	R20 @ 175	R12 @ 150	R12 @ 125
3	1.5	154	600	550	550	R20 @ 125	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 125	R12 @ 125
	2.5	162	600	600	600	R20 @ 125	R16 @ 125	R20 @ 125	R16 @ 125	R12 @ 125	R12 @ 125
	3.5	173	600	700	700	R20 @ 125	R16 @ 125	R20 @ 125	R16 @ 125	R12 @ 125	R16 @ 175
4	1.5	147	700	500	500	R25 @ 125	R20 @ 125	R25 @ 125	R16 @ 125	R16 @ 175	R12 @ 150
	2.5	156	700	600	600	R25 @ 125	R16 @ 125	R25 @ 125	R16 @ 125	R16 @ 175	R12 @ 125
	3.5	166	700	700	700	R25 @ 125	R20 @ 175	R25 @ 175	R20 @ 175	R16 @ 175	R16 @ 175

Table 12.10 Single box culvert – mild steel, H = 6.0m

W (m)	D (m)	Maximum bearing pressure (kN/m ²)	A (mm)	B (mm)	C (mm)	S1	S2	S3	S4	S5	S6
1.5	1.5	205	450	300	300	R16 @ 150	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 150	R10 @ 200
	2.5	216	450	450	450	R16 @ 150	R16 @ 175	R25 @ 175	R20 @ 175	R12 @ 150	R12 @ 150
	3.5	227	450	500	500	R16 @ 150	R16 @ 150	R32 @ 150	R25 @ 150	R12 @ 150	R12 @ 125
2	1.5	202	600	500	500	R16 @ 125	R16 @ 175	R20 @ 175	R16 @ 175	R12 @ 125	R12 @ 150
	2.5	213	600	600	600	R16 @ 125	R20 @ 150	R20 @ 150	R20 @ 150	R12 @ 125	R12 @ 125
	3.5	227	600	700	700	R16 @ 125	R20 @ 150	R25 @ 150	R20 @ 150	R12 @ 125	R16 @ 175
3	1.5	193	700	500	500	R25 @ 175	R16 @ 125	R20 @ 125	R16 @ 125	R16 @ 175	R12 @ 150
	2.5	203	700	600	600	R25 @ 175	R16 @ 125	R20 @ 125	R16 @ 125	R16 @ 175	R12 @ 125
	3.5	214	700	700	700	R25 @ 175	R16 @ 125	R25 @ 125	R20 @ 125	R16 @ 175	R16 @ 175
4	1.5	188	800	500	500	R25 @ 125	R25 @ 175	R32 @ 175	R16 @ 175	R16 @ 150	R12 @ 150
	2.5	197	800	600	600	R25 @ 125	R20 @ 125	R25 @ 125	R20 @ 125	R16 @ 150	R12 @ 125
	3.5	211	800	800	800	R25 @ 125	R20 @ 150	R25 @ 150	R20 @ 150	R16 @ 150	R16 @ 150

Table 12.11 Twin box culvert – high yield steel, H = 0.5m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	84	250	250	250	T16 @ 200	T10 @ 200	T10 @ 200					
	2.5	89	250	250	250	T16 @ 200	T10 @ 200	T10 @ 200					
	3.5	99	250	350	350	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
2	1.5	75	300	250	250	T16 @ 150	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
	2.5	80	300	300	300	T16 @ 150	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 200
	3.5	90	300	400	400	T16 @ 150	T16 @ 150	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 175
3	1.5	68	400	350	350	T16 @ 150	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 200
	2.5	73	400	400	400	T16 @ 150	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 175
	3.5	78	400	400	400	T16 @ 150	T16 @ 150	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 175
4	1.5	65	500	400	400	T16 @ 125	T16 @ 125	T16 @ 175	T16 @ 175	T16 @ 175	T16 @ 175	T12 @ 200	T10 @ 175
	2.5	72	500	500	500	T16 @ 125	T16 @ 125	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 200	T12 @ 200
	3.5	78	500	550	550	T16 @ 125	T16 @ 125	T16 @ 175	T16 @ 175	T16 @ 175	T16 @ 175	T12 @ 200	T12 @ 175

Table 12.12 Twin box culvert - high yield steel, H = 1.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	95	300	300	300	T16 @ 200	T10 @ 200	T10 @ 200					
	2.5	97	300	400	400	T16 @ 200	T10 @ 200	T10 @ 175					
	3.5	111	300	400	400	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 200	T10 @ 175
2	1.5	90	400	400	400	T16 @ 200	T10 @ 175	T10 @ 175					
	2.5	96	400	400	400	T16 @ 200	T10 @ 175	T10 @ 175					
	3.5	104	400	450	450	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 150
3	1.5	81	450	450	450	T16 @ 150	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 150
	2.5	86	450	450	450	T16 @ 150	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 150
	3.5	93	450	500	500	T16 @ 150	T16 @ 150	T16 @ 175	T16 @ 175	T16 @ 175	T16 @ 175	T10 @ 150	T10 @ 125
4	1.5	77	500	500	500	T20 @ 150	T20 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	2.5	81	500	500	500	T20 @ 150	T20 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	3.5	87	500	550	550	T20 @ 150	T20 @ 150	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125

Table 12.13 Twin box culvert – high yield steel, H = 2.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	104	400	300	300	T16 @ 200	T10 @ 175	T10 @ 200					
	2.5	110	400	300	300	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 175	T10 @ 200
	3.5	121	400	400	400	T16 @ 200	T16 @ 200	T20 @ 150	T16 @ 150	T16 @ 150	T16 @ 150	T10 @ 175	T10 @ 175
2	1.5	101	450	400	400	T16 @ 200	T10 @ 150	T10 @ 175					
	2.5	107	450	450	450	T16 @ 200	T10 @ 150	T10 @ 150					
	3.5	117	450	500	500	T16 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 125
3	1.5	96	500	500	500	T20 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	2.5	103	500	550	550	T20 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	3.5	109	500	550	550	T20 @ 200	T20 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
4	1.5	94	550	550	550	T25 @ 200	T25 @ 200	T16 @ 175	T16 @ 175	T16 @ 175	T16 @ 175	T10 @ 125	T10 @ 125
	2.5	100	550	600	600	T25 @ 200	T25 @ 200	T16 @ 175	T16 @ 175	T16 @ 175	T16 @ 175	T10 @ 125	T12 @ 150
	3.5	105	550	600	600	T25 @ 200	T25 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T12 @ 150

Table 12.14 Twin box culvert – high yield steel, H = 4.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	144	450	300	300	T16 @ 175	T16 @ 175	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 200
	2.5	153	450	400	400	T16 @ 175	T16 @ 175	T25 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 175
	3.5	168	450	550	550	T16 @ 175	T16 @ 175	T25 @ 200	T16 @ 200	T20 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
2	1.5	143	500	500	500	T20 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	2.5	152	500	550	550	T20 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 125	T10 @ 125
	3.5	161	500	600	600	T20 @ 200	T20 @ 200	T20 @ 175	T16 @ 175	T16 @ 175	T16 @ 175	T10 @ 125	T12 @ 150
3	1.5	138	600	550	550	T25 @ 200	T25 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 150	T10 @ 125
	2.5	146	600	600	600	T25 @ 200	T25 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 150	T12 @ 150
	3.5	156	600	700	700	T25 @ 200	T25 @ 200	T20 @ 150	T16 @ 150	T16 @ 150	T16 @ 150	T12 @ 150	T12 @ 125
4	1.5	134	700	500	500	T25 @ 175	T25 @ 175	T25 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 125	T10 @ 125
	2.5	142	700	600	600	T25 @ 175	T25 @ 175	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 125	T12 @ 150
	3.5	151	700	700	700	T25 @ 175	T25 @ 175	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 125	T12 @ 125

Table 12.15 Twin box culvert – high yield steel, H = 6.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	182	450	300	300	T16 @ 150	T16 @ 150	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 200
	2.5	193	450	450	450	T16 @ 150	T16 @ 150	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T10 @ 150	T10 @ 150
	3.5	203	450	500	500	T16 @ 150	T16 @ 150	T25 @ 150	T16 @ 150	T20 @ 150	T16 @ 150	T10 @ 150	T10 @ 125
2	1.5	184	600	500	500	T20 @ 175	T20 @ 175	T16 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 150	T12 @ 125
	2.5	194	600	600	600	T20 @ 175	T20 @ 175	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 150	T12 @ 150
	3.5	206	600	700	700	T20 @ 175	T20 @ 175	T20 @ 150	T16 @ 150	T16 @ 150	T16 @ 150	T12 @ 150	T12 @ 125
3	1.5	177	700	500	500	T25 @ 200	T25 @ 200	T20 @ 200	T16 @ 200	T16 @ 200	T16 @ 200	T12 @ 125	T12 @ 125
	2.5	186	700	600	600	T25 @ 200	T25 @ 200	T16 @ 150	T16 @ 150	T16 @ 150	T16 @ 150	T12 @ 125	T12 @ 150
	3.5	196	700	700	700	T25 @ 200	T25 @ 200	T25 @ 200	T20 @ 200	T20 @ 200	T16 @ 200	T12 @ 125	T12 @ 125
4	1.5	175	800	500	500	T32 @ 200	T25 @ 200	T25 @ 175	T20 @ 175	T16 @ 175	T16 @ 175	T16 @ 200	T12 @ 125
	2.5	182	800	600	600	T32 @ 200	T25 @ 200	T20 @ 150	T16 @ 150	T16 @ 150	T16 @ 150	T16 @ 200	T12 @ 150
	3.5	195	800	800	800	T32 @ 200	T25 @ 200	T20 @ 175	T16 @ 175	T16 @ 175	T16 @ 175	T16 @ 200	T16 @ 200

Table 12.16 Twin box culvert – mild steel, H = 0.5m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	84	250	250	250	R16 @ 125	R16 @ 125	R16 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	2.5	89	250	250	250	R16 @ 125	R16 @ 125	R16 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	3.5	99	250	350	350	R16 @ 125	R16 @ 125	R25 @ 200	R16 @ 200	R20 @ 200	R16 @ 200	R10 @ 200	R12 @ 200
2	1.5	75	300	250	250	R20 @ 175	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R10 @ 200	R10 @ 200
	2.5	80	300	300	300	R20 @ 175	R20 @ 175	R20 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	3.5	90	300	400	400	R20 @ 175	R20 @ 175	R25 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R12 @ 200
3	1.5	68	400	350	350	R20 @ 150	R20 @ 150	R20 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	2.5	73	400	400	400	R20 @ 150	R20 @ 150	R20 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	3.5	78	400	400	400	R20 @ 150	R20 @ 150	R25 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
4	1.5	65	500	400	400	R20 @ 125	R20 @ 125	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 200
	2.5	72	500	500	500	R20 @ 125	R20 @ 125	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	3.5	78	500	550	550	R20 @ 125	R20 @ 125	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 125

Table 12.17 Twin box culvert – mild steel, H = 1.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	95	300	300	300	R20 @ 200	R20 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R10 @ 200
	2.5	97	300	400	400	R20 @ 200	R20 @ 200	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R10 @ 200	R12 @ 200
	3.5	111	300	400	400	R20 @ 200	R20 @ 200	R25 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R10 @ 200	R12 @ 200
2	1.5	90	400	400	400	R20 @ 200	R20 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	2.5	96	400	400	400	R20 @ 200	R20 @ 200	R20 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R12 @ 200
	3.5	104	400	450	450	R20 @ 200	R20 @ 200	R25 @ 200	R16 @ 200	R20 @ 200	R16 @ 200	R12 @ 200	R12 @ 150
3	1.5	81	450	450	450	R20 @ 150	R20 @ 150	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	2.5	86	450	450	450	R20 @ 150	R20 @ 150	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	3.5	93	450	500	500	R20 @ 150	R20 @ 150	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
4	1.5	77	500	500	500	R25 @ 175	R25 @ 175	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	2.5	81	500	500	500	R25 @ 175	R25 @ 175	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	3.5	87	500	550	550	R25 @ 175	R25 @ 175	R20 @ 150	R16 @ 150	R16 @ 150	R16 @ 150	R12 @ 150	R12 @ 125

Table 12.18 Twin box culvert – mild steel, H = 2.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	104	400	300	300	R16 @ 175	R16 @ 175	R16 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 200	R10 @ 200
	2.5	110	400	300	300	R16 @ 175	R16 @ 175	R20 @ 150	R16 @ 150	R16 @ 150	R16 @ 150	R12 @ 200	R10 @ 200
	3.5	121	400	400	400	R16 @ 175	R16 @ 175	R25 @ 150	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 200	R12 @ 200
2	1.5	101	450	400	400	R20 @ 175	R20 @ 175	R16 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 150	R12 @ 200
	2.5	107	450	450	450	R20 @ 175	R20 @ 175	R20 @ 200	R16 @ 200	R16 @ 200	R16 @ 200	R12 @ 150	R12 @ 150
	3.5	117	450	500	500	R20 @ 175	R20 @ 175	R25 @ 175	R16 @ 175	R20 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
3	1.5	96	500	500	500	R25 @ 175	R25 @ 175	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	2.5	103	500	550	550	R25 @ 175	R25 @ 175	R16 @ 150	R16 @ 150	R16 @ 150	R16 @ 150	R12 @ 150	R12 @ 125
	3.5	109	500	550	550	R25 @ 175	R25 @ 175	R25 @ 200	R20 @ 200	R20 @ 200	R20 @ 200	R12 @ 150	R12 @ 125
4	1.5	94	550	550	550	R32 @ 200	R32 @ 200	R20 @ 150	R16 @ 150	R16 @ 150	R16 @ 150	R12 @ 125	R12 @ 125
	2.5	100	550	600	600	R32 @ 200	R32 @ 200	R20 @ 175	R20 @ 175	R20 @ 175	R20 @ 175	R12 @ 125	R16 @ 200
	3.5	105	550	600	600	R32 @ 200	R32 @ 200	R25 @ 200	R20 @ 200	R20 @ 200	R20 @ 200	R12 @ 125	R16 @ 200

Table 12.19 Twin box culvert – mild steel, H = 4.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	144	450	300	300	R20 @ 175	R20 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R10 @ 200
	2.5	153	450	400	400	R20 @ 175	R20 @ 175	R25 @ 150	R16 @ 150	R20 @ 150	R16 @ 150	R12 @ 150	R12 @ 200
	3.5	168	450	550	550	R20 @ 175	R20 @ 175	R32 @ 200	R20 @ 200	R25 @ 200	R20 @ 200	R12 @ 150	R12 @ 125
2	1.5	143	500	500	500	R25 @ 200	R25 @ 200	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 150	R12 @ 150
	2.5	152	500	550	550	R25 @ 200	R25 @ 200	R20 @ 150	R16 @ 150	R16 @ 150	R16 @ 150	R12 @ 150	R12 @ 125
	3.5	161	500	600	600	R25 @ 200	R25 @ 200	R25 @ 175	R20 @ 175	R20 @ 175	R20 @ 175	R12 @ 150	R12 @ 125
3	1.5	138	600	550	550	R25 @ 125	R25 @ 125	R20 @ 125	R16 @ 125	R16 @ 125	R16 @ 125	R12 @ 125	R12 @ 125
	2.5	146	600	600	600	R25 @ 125	R25 @ 125	R25 @ 200	R20 @ 200	R20 @ 200	R20 @ 200	R12 @ 125	R12 @ 125
	3.5	156	600	700	700	R25 @ 125	R25 @ 125	R20 @ 125	R16 @ 125	R16 @ 125	R16 @ 125	R12 @ 125	R16 @ 175
4	1.5	134	700	500	500	R32 @ 175	R32 @ 175	R25 @ 150	R20 @ 150	R16 @ 150	R16 @ 150	R16 @ 175	R12 @ 150
	2.5	142	700	600	600	R32 @ 175	R32 @ 175	R25 @ 175	R20 @ 175	R20 @ 175	R20 @ 175	R16 @ 175	R12 @ 125
	3.5	151	700	700	700	R32 @ 175	R32 @ 175	R25 @ 175	R20 @ 175	R20 @ 175	R20 @ 175	R16 @ 175	R16 @ 175

Table 12.20 Twin box culvert – mild steel, H = 6.0m

W m	D m	Max bearing pressure (kN/m ²)	A mm	B mm	C mm	S1	S2	S3	S4	S5	S6	S7	S8
1.5	1.5	182	450	300	300	R25 @ 200	R25 @ 200	R20 @ 200	R20 @ 200	R16 @ 200	R16 @ 200	R12 @ 150	R10 @ 200
	2.5	193	450	450	450	R25 @ 200	R25 @ 200	R20 @ 125	R16 @ 125	R16 @ 125	R16 @ 125	R12 @ 150	R12 @ 150
	3.5	203	450	500	500	R25 @ 200	R25 @ 200	R32 @ 150	R16 @ 150	R25 @ 150	R16 @ 150	R12 @ 150	R12 @ 150
2	1.5	184	600	500	500	R25 @ 175	R25 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R16 @ 175	R12 @ 125	R12 @ 150
	2.5	194	600	600	600	R25 @ 175	R25 @ 175	R20 @ 125	R16 @ 125	R16 @ 175	R16 @ 125	R12 @ 125	R12 @ 125
	3.5	206	600	700	700	R25 @ 175	R25 @ 175	R25 @ 150	R20 @ 150	R20 @ 150	R20 @ 150	R12 @ 125	R16 @ 175
3	1.5	177	700	500	500	R32 @ 200	R32 @ 200	R25 @ 200	R20 @ 200	R20 @ 200	R20 @ 200	R16 @ 175	R12 @ 150
	2.5	186	700	600	600	R32 @ 200	R32 @ 200	R20 @ 150	R20 @ 150	R20 @ 150	R20 @ 150	R16 @ 175	R12 @ 125
	3.5	196	700	700	700	R32 @ 200	R32 @ 200	R25 @ 150	R20 @ 150	R20 @ 150	R20 @ 150	R16 @ 175	R16 @ 175
4	1.5	175	800	500	500	R40 @ 200	R32 @ 200	R32 @ 175	R25 @ 175	R16 @ 175	R16 @ 175	R16 @ 150	R12 @ 150
	2.5	182	800	600	600	R40 @ 200	R32 @ 200	R25 @ 125	R20 @ 125	R16 @ 125	R16 @ 125	R16 @ 150	R12 @ 125
	3.5	195	800	800	800	R40 @ 200	R32 @ 200	R25 @ 150	R20 @ 150	R20 @ 150	R20 @ 150	R16 @ 150	R16 @ 150

13 EMERGENCY AND TEMPORARY STRUCTURES

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13 Emergency and temporary structures

Temporary bridge structures are intended to provide rapid solutions to essentially short-term problems of access. Decisions about their construction have to balance the high expense of the fastest solutions against the likelihood that they will yield benefits for only a limited period, and hence the need to keep their costs as low as possible.

The principal reasons for requiring a temporary crossing are:

- damage to an existing bridge results in the need for a temporary structure on a nearby site while the disabled bridge is being repaired or replaced. (This could be an opportune time to consider upgrading the existing structure in terms of width, traffic capacity, height above the water and durability.)
- upgrading the road requires a wider, higher or stronger bridge and a temporary crossing is needed for a detour to carry traffic while the new road and bridge are built on the old alignment.
- a river crossing is required urgently and for the short term by an organisation other than a roads department, eg an aid organisation concerned with the delivery of emergency relief supplies.
- temporary access to a construction site is needed in advance of the construction of a permanent way.

13.1 The time factor

The time factor is central to all the decisions that have to be made about the crossing, influencing in particular the type of structure to be used. For example, a washed-out bridge on a strategic road will require substitution at the earliest possible moment, whereas a detour for a road upgrading project can be planned well in advance, enabling the cheapest solution to be prepared.

The length of time that the temporary structure will be in service also influences its design and cost. A low-level structure may be adequate for a short period, but if it is likely to be needed beyond the end of the dry season, a road or culvert may not suffice and a temporary bridge with associated bank protection may be required. A realistic estimate of the time that would be needed to build a permanent replacement is necessary, together with a worst-case view of the weather and the projected traffic loading, before a firm decision can be made about the type of temporary structure to be built.

13.2 Types of structures

Generally, the order of preference for the type of

structure will follow the order of increasing cost, unless specific skills or materials are unavailable. This order of preference is likely to be:

- fords;
- causeways;
- temporary beam bridges;
- prefabricated decks.

13.2.1 Fords

A ford may be no more than a prepared descent to and exit from a river bed, taking into account the traction requirements of known traffic on the slopes (Sections 7.1 and 7.2).

13.2.2 Causeways

A vented earth causeway can be built using pipes stocked in most road department depots, taking measures to prevent erosion of the fill around the pipes. A temporary surfacing may also be required. Sections 7.3 and 7.4 contain diagrams of the various forms of causeways.

Though a ford or causeway may not be serviceable for use by general traffic at times of high water, it might be acceptable in the short term; an existing crossing of this type might even be found on a nearby older alignment.

13.2.3 Temporary beam bridges

Timber may be the most readily available material in the locality. The design of timber bridge decks is detailed in Chapter 11. They do not need preservative treatment for temporary duty. If steel or concrete beams were available for short-term use, it would be necessary to ascertain their bending and shear characteristics and to employ them in a similar manner to timber beams (Section 11.2) with timber decking as outlined in Section 11.3. Chapter 10 considers only superstructures made of steel and concrete acting together in a composite manner, which is unsuitable for temporary structures. The steel beams detailed in Chapter 10 would be grossly overloaded without the composite concrete top slab.

13.2.4 Prefabricated bridge decks

Timber truss decks of the trestle type were popular in the nineteenth century but the skills required to build one quickly are no longer generally available. However, the Timber Research and Development Association (TRADA) in England has developed a design of bridge deck made from identical softwood timber panels, viable for spans up to about 24m. The panels may be stockpiled for use in various combinations, such as a

Bailey bridge is made. Figure 13.1 shows an illustration of the design. Details are available in Parry (1981) and TRADA (1985).

The basic concept of steel panels, assembled to make bridges of different length, width and load capacity, was developed by Colonel Bailey of the British Army and has been used all over the world for 50 years. Other well known versions of unitary construction bridges are the Acrow Panel bridge and the Callender Hamilton designs. Suppliers of these bridges are listed at the end of this chapter. They issue detailed design and assembly instructions and provide a design and advisory service for customers.

Parts of old Bailey bridges can be found in most countries, and great care should be taken to identify these parts correctly. Detail design modifications and changes to the material specifications have taken place over the years, resulting in greater load-carrying capacities. New and old panels should not be assembled in the same structure. Old components must be used only in accordance with the corresponding manuals.

Prefabricated steel bridges are unlikely to offer an economical solution to the need for temporary bridging, unless their facility for being dismantled and re-used can be properly exploited. However, their component parts are so easily transported that a stock held centrally by a roads department can be made available at short notice nationwide in most countries.

13.3 Abutments and piers

Most temporary bridge decks can be assembled on existing or temporary abutments. An exception is the prefabricated timber panel deck, which requires carefully made abutments at a precise distance apart, because the truss is located below the running surface and must have a length equal to a multiple of the standard frame length.

Existing abutments, if they are sound and located on an acceptable alignment, have three important advantages:

- they have a proven record of sustaining the dynamic and static applied loads;
- they have shown satisfactory resistance to attack by the river;
- their use avoids the time and expense of building new abutments.

Temporary abutments may be made from gabions or logs. However, they are highly susceptible to scour and erosion, and should be constructed with great care using ties and anchors where possible (Figure 13.2), because they can be destroyed by a single flood. The log bridges shown in Figures 13.3 and 13A served only until the volume and velocity of the water rose and the abutments failed owing to scour.

Bank seat abutments may be made in the form of log grillages as shown in Figure 13.5, if these can be built above the expected high water level and involve no interference to the waterway.



Figure 13.1 Timber panel bridge

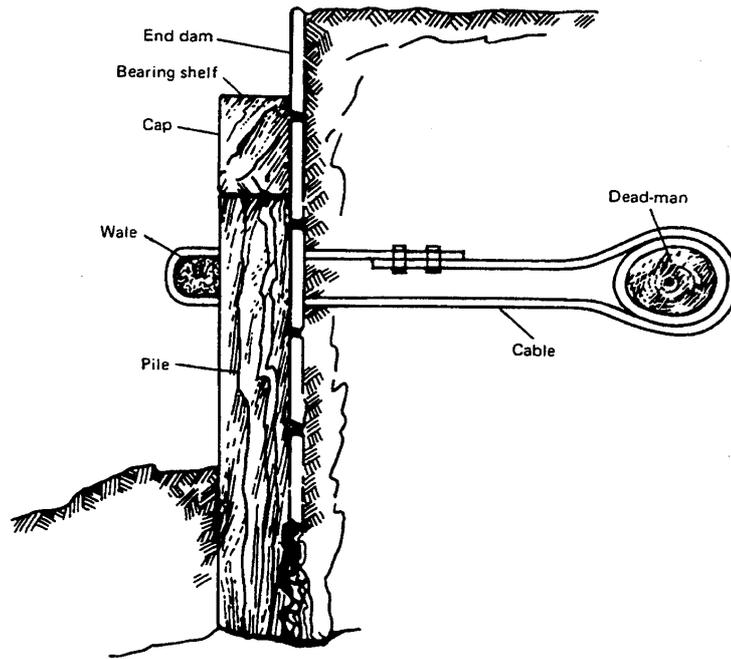


Figure 13.2 Timber abutment tied back to a buried log



Figure 13.3 Temporary bridge on gabion abutments

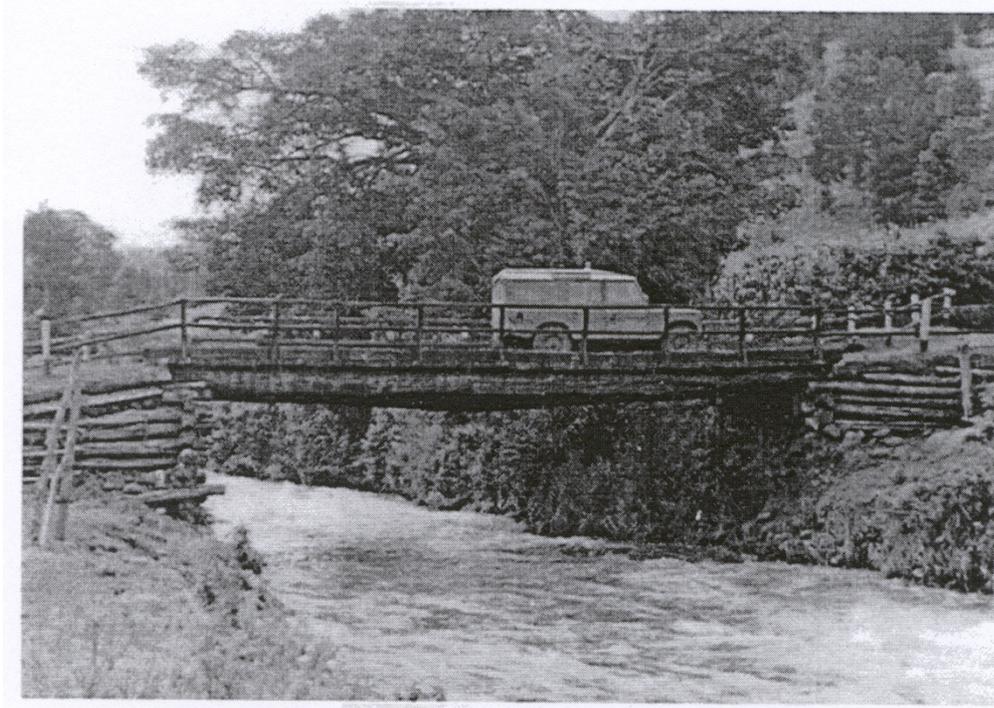


Figure 13.4 Log abutments with integral wing walls

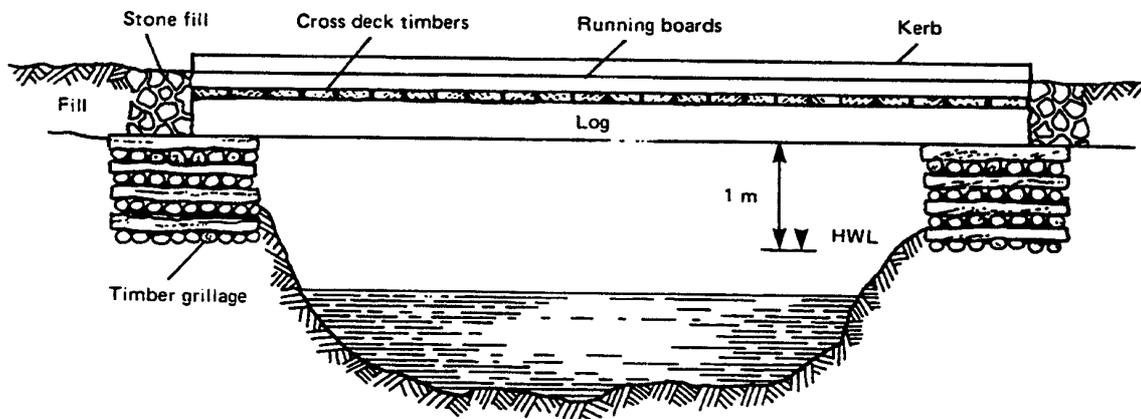


Figure 13.5 Bridge on timber grillage abutments

Where water flow is slow, timber piled abutments and piers have proved successful. The use of piers reduces the section of the road bearing beams, and a whole bridge like the one shown in Figure 13.6 can be built with timber no larger than 325 mm *in* diameter. Figure 13.7 shows a typical longitudinal section: its cross-section would be according to Figures 11.3 or 11.4.

If the engineer is satisfied that there is strong enough material at bed level or a little below, open caisson

piers may be constructed using pre-cast concrete rings (Figure 13.8). The first ring is placed in position and excavation takes place from inside it. Rings are added as the first progresses downwards until a firm base material is reached, then more rings are added until the required deck height is achieved. Lean concrete can be used to fill the caisson and a stronger concrete is used at the top to take anchor bolts for the transoms. A height to diameter ratio of 3:1 should not be exceeded without careful calculations of toppling moments and the possible



Figure 13.6 Timber deck on piled piers and abutments

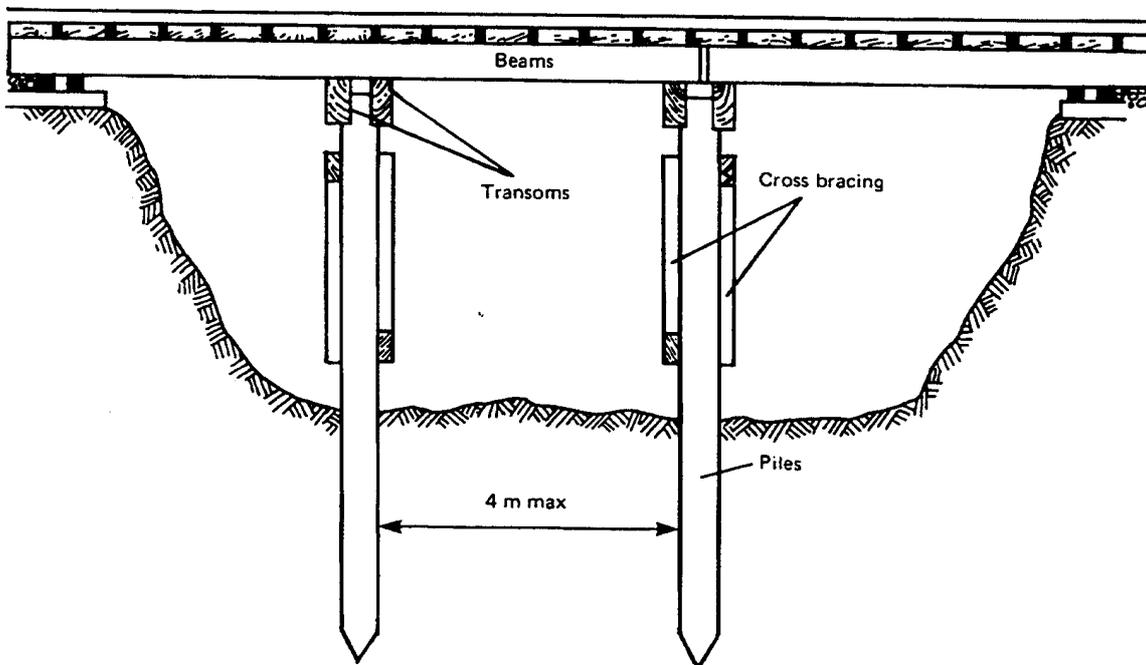


Figure 13.7 Longitudinal section through a bridge on timber pile piers

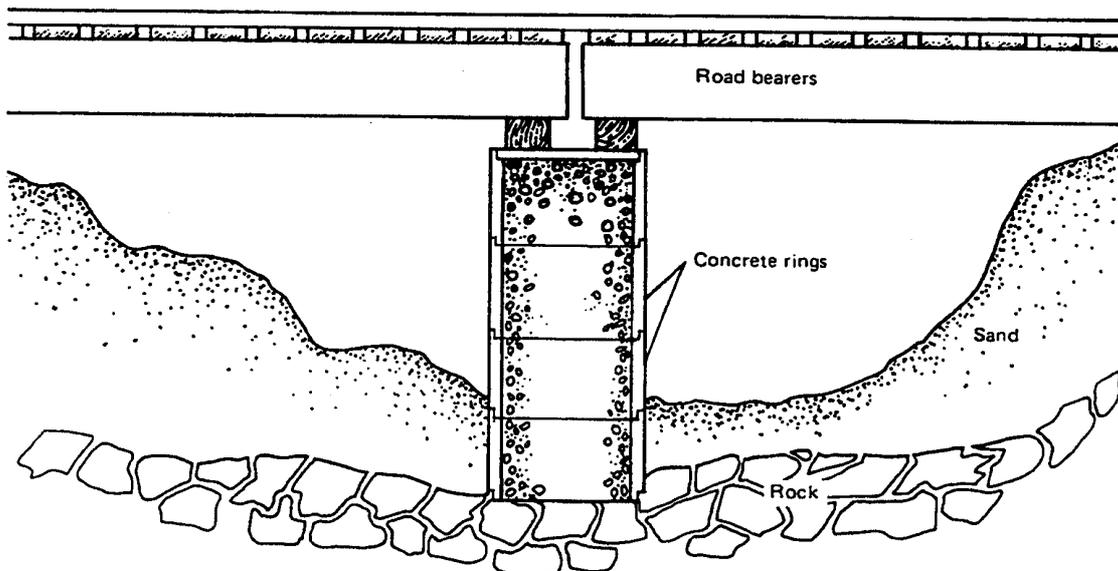


Figure 13.8 Bridge pier made of concrete rings

addition of steel reinforcement, which rather negates the concept of a temporary structure.

One other serious drawback with caisson piers is the obstruction to flow presented by the relatively large surface area. If there is the possibility of fast currents, these could well cause scour at the abutments and under the caissons themselves, unless they are founded deep or on rock. The problem is discussed in detail in Chapter 5.

13.4 Protection of temporary structures

Though temporary structures, by definition, are not expected to be as robust as permanent ones, they must nonetheless offer reliable service over the required period of use. The two key aspects of durability in temporary bridging are:

- load carrying adequacy;
- protection from water damage.

A temporary bridge may not be completely adequate for all vehicles that normally use the road. If the temporary structure has any limitation in load capacity, width or height, this must be clearly marked at the entrance to the road on which the bridge is located and repeated on the approach to the bridge. It may be possible to divert large vehicles to a prepared ford, while small ones are permitted to use the bridge. If possible, physical barriers should be erected to prevent drivers of large vehicles from infringing the temporary regulations in areas where drivers are known to ignore warning signs.

Section 13.3 noted the influence of the river on the design of temporary crossing to be used. Hydraulic

design is discussed at some length in Chapter 5, and Chapter 6 describes various methods of protecting structures over rivers. For most applications the two main precautions to be taken are:

- to allow adequate clearance between high water level and the temporary deck
- to build the sub-structure so that there is a minimum of interference to the flow.

13.5 References

Parry J D (1981). *The Kenyan low cost modular timber bridge*. Laboratory Report LR970. Transport Research Laboratory, Crowthorne.

TRADA (1985). *Prefabricated modular timber bridges*. Timber Research and Development Association, High Wycombe.

Bailey type bridges:

Mabey and Johnson Ltd. Floral Mile, Twyford, Reading RG10 9SQ, England.

The Callender Hamilton bridge:

Balfour Beatty Power Construction Ltd. 7 Mayday Road, Thornton Heath CR7 7XA, England

14 BRIDGE BUILDING MATERIALS

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14 Bridge building materials

It is assumed that the engineer is familiar with the structural use of steel, concrete and timber. The notes on building materials in this chapter are intended only to:

- specify the materials and terms employed in Chapters 8 to 13;
- clarify certain aspects of their use;
- draw attention to potential problems;
- provide a consolidated list of the numerous references.

14.1 Concrete

The designs presented in Chapters 8, 9 and 10 are based on the use of Class 30 concrete using 20mm coarse aggregate (30/20 concrete). HA (1986) defines ordinary structural concrete using ordinary Portland cement, Portland blast furnace cement, sulfate resisting Portland cement and Portland pulverised fuel ash cement. The relevant British Standard Specifications for these cement types are: BSI (1996a, 1996b, 1996c and 1996d).

Class 30 concrete has a characteristic cube strength of 30N/mm^2 at an age of 28 days, when tested according to BSI (1983a). This is generally interpreted as an average strength of at least 37.5N/mm^2 with no sample having a strength of less than 25.5N/mm^2 . The equivalent American standard ASTM (1986) specifies the testing of cylindrical specimens, which return a crushing strength approximately 80% that of the cube strength to BSI (1983a).

BSI (1983b) describes methods of sampling and testing concrete for strength and other characteristics; mix design is described in BRE (1988a) and the 'Man on the Job' leaflets published by C&CA (1979, 1980) are useful practical guides covering materials and work practices.

Aggregates used in making concrete should conform with BSI (1992) or a similar national code. Concrete for use with steel reinforcement is more likely to achieve the standard of compaction required if the maximum aggregate size is limited to 20mm.

Where there are significant amounts of sulphates in the soil or ground water, as described in Section 3.5 and BRE (1981), it is prudent to take precautions to prevent deterioration of the concrete, which is in contact with the ground. Breakdown of the concrete mass can result from reaction between sulfate and hydrates. Concrete made with less than 3.5 per cent tricalcium aluminate is considered resistant to sulfate attack BSI, (1996d). Cement mixtures including blast

furnace slag with alumina content of less than 15 per cent BSI, (1996a) and pulverised fuel ash/cement mixtures with a PFA content greater than 25 per cent BSI, (1996c) also resist attack by sulfates. However, the basic concept of making a dense, impermeable concrete with adequate cover to reinforcement is still the best defence against the penetration of pollutants which attack the concrete mass or the steel reinforcement.

The total chloride content of the concrete mix should not exceed 0.3 per cent of the cement, or cement plus pozzolan mixture, when steel reinforcement is used, or 0.2 per cent when using sulphate resisting Portland cement in mass or reinforced concrete.

In order to avoid detrimental alkali-silica reaction, it is better to use aggregate known to be non-reactive. The alternative is to restrict the total sodium oxide content of the cement to less than 3.0kg per cubic metre of concrete. This point is discussed more fully in HA (1986) and BRE (1988b).

Besides ensuring that the materials are of adequate quality, the single most important factor influencing both strength and durability of concrete is the water/cement ratio of the mix. When specifying concrete, the maximum free-water/cement ratio by weight should generally be limited to 0.5. In calculating the free water available in the concrete, it is important that any moisture on the surface of the aggregates is included (BRE, 1988a). In marine situations or near the coast, the free-water/cement ratio should be no more than 0.42 to avoid premature deterioration due to corrosion of reinforcement or damage to mass concrete by salt scaling or sulphate attack. Careful mix design is necessary to ensure ease of placing concrete without violating the maximum free-water/ cement ratio.

Cover to reinforcement and curing of concrete must also be carefully specified and supervised on site if the structure is to have an adequate service life. The cover of 50mm shown to reinforcement in the standard designs is intended as a minimum for a structure in a moderate environment. In a severe environment, such as a marine situation in a hot climate, cover may need to be increased to 75mm or even 100mm to achieve the required lifespan. To ensure that the cover is as impermeable as possible, the concrete must be moist cured with clean non-saline water as soon as surface finishing is complete for exposed surfaces, or as soon as forms can be safely stripped from formed faces (3 days for side forms, 14 days for structural supports or as advised by local engineers). This curing process should consist of covering the concrete with wet hessian and covering the hessian with polythene sheeting, which must be sealed around the edges to stop the hessian from drying out. The hessian and polythene should be kept in place until the concrete is at least 7 days old,

if possible 14 days. From time to time the hessian may need to be re-dampened, taking care to reinstate the polythene afterwards.

Special measures are necessary for concreting in hot weather and when there are drying winds and low humidity. Aggregates should be kept shaded, cool mix water should be used and the time between mixing and placing should be kept to a minimum. Detailed advice on how to minimise the harmful effects of hot weather on concrete is given in ACI (1991) and BRE (1971).

14.2 Steel reinforcement

The designs included in Chapters 8, 9, 10 and 12 provide for the use of two grades of steel reinforcement, as specified in BSI (1997a), ie plain round mild steel (given the symbol R in the figures) and deformed high yield steel (given the symbol T).

Table 14.1 gives their basic mechanical properties, which should be matched as closely as possible if the tables of reinforcement details given in Chapters 8, 9, 10 and 12 are not to require modification. Any deviation from the listed properties requires examination by a structural engineer. BSI (1989) specifies scheduling and bending, etc and HA (1986) gives guidance on cutting, fixing, lapping and joining. Reinforcing bars should not be welded unless the joints can be demonstrated to the satisfaction of the Resident Engineer to be acceptable in terms of location, fatigue life, durability and surface condition.

Table 14.1 Grades of steel reinforcement

Designation	Nominal grade	Characteristic strength N/mm ²	Minimum elongation %
Mild steel	250	250	22
High yield steel	460	460	12

14.3 Structural steel beams and steel fittings

The load-bearing beams listed in Chapter 10 conform to BSI (1993a) dimensionally and are rolled from steel conforming to BSI (1993b), grade 43c. If beams made to other specifications are to be used, the engineer must ensure that the following properties are acceptable:

- elastic section modulus about the beam major axis;
- total cross sectional area;
- web cross sectional area (overall depth times thickness);

- radius of gyration about the beam minor axis;
- beam overall depth.

Other steel items, such as the parapets in Chapter 9, should be made from steel conforming with to BSI (1993b), the grades as specified on the drawings.

14.4 Structural timber

The durability of a timber bridge will be conditioned by climate, biological factors and design. The most important climatic factor is moisture, since large seasonal or daily variations in the moisture content of the timber will cause it to shrink and swell, which may lead to cracking. The two most important biological factors are wood-destroying fungi and insects.

Hardwoods are generally more durable than softwoods, especially against termite attack. A design life ranging from 5 years for untreated softwood to 20 years or more for hardwood heart timber can reasonably be assumed. Treatment with chemical preservatives to protect the timber against insect attack and fungi decay can extend this design life considerably.

The need for treatment is determined by the type of timber, the sizes of the timber sections, and the design life required. A few species have a high natural resistance to insect attack and decay, and their heartwood may be used without treatment. However, treatment is almost always needed for framed structures, since any deterioration of their light members is likely to cause a significant loss of strength.

Durability can be enhanced by good design. When detailing, the engineer should:

- provide for drainage of the timber superstructure to prevent retention of water;
- provide for drainage of the approaches to prevent water running onto the deck;
- keep the ends of logs, beams or trusses away from soil and water;
- avoid soil and debris from the deck being trapped in pockets or against the timber superstructure;
- space timbers and design joints to accommodate movement from shrinkage or swelling.

14.4.1 Preservative treatments

To be effective in the long term, chemical treatment of timber should take place in a pressurised tank, because protection is short lived unless the preservative penetrates the timber adequately. If

boring or shaping of timbers takes place after treatment, the exposed surfaces should be flooded with preservative before final assembly. The active chemicals are applied diluted in water or in a spirit.

Generally speaking, the hard dense heart of hardwoods does not allow the entry of preserving chemicals, nor does it need them, but where the hardwood to be used includes some sapwood, treatment should be applied. All softwoods should be treated.

Newly felled timber should be given a preliminary coating of preservative to prevent early contamination by the spores of fungi, and should then be seasoned before pressure treatment.

There are many commercial products available for the preservation of timber, but BSI (1997b) recommends only pressure treatment with creosote or copper-chrome-arsenite (CCA). Table 14.2 gives recommended quantities. Where pressure tanks for treatment are available the techniques will be known and are not detailed here.

If pressure impregnation is not available, the hot and cold tank treatment using creosote is a useful alternative. The timber is immersed in a tank of cold preservative which is then heated to about 85-90°C (Figure 14.1). This temperature is maintained for at least an hour and the preservative is allowed to cool down before the wood is removed. The process is described in Tack (1979), which is one of a very useful series of Overseas Building Notes, all of which are listed in this reference, and BSI (1997c).
Brush or

Table 14.2 Recommended quantities of timber preservative

	Creosote kg/m ³	CCA kg/m ³
Timber in contact with the ground	160	10
Timber not in contact with the ground	100	5.3

spray-applied preservative treatments give only slight protection when compared to the methods referred to above and should, where possible, be used only for retreating cut or worked surfaces.

Since there is a risk of corrosion of mild steel or galvanized steel in contact with salt type preservatives such as CCA, it is important not to apply metal fasteners until the fixation of the preservative within the wood is complete. This usually takes about 7 days. If the wood is treated with a preservative containing copper, aluminium fixings should not be used.

All preservatives are toxic and those which are dissolved in white spirit will be highly inflammable. The specification should therefore include instructions to the effect that workers are required to use overalls, rubber gloves and eye shields.

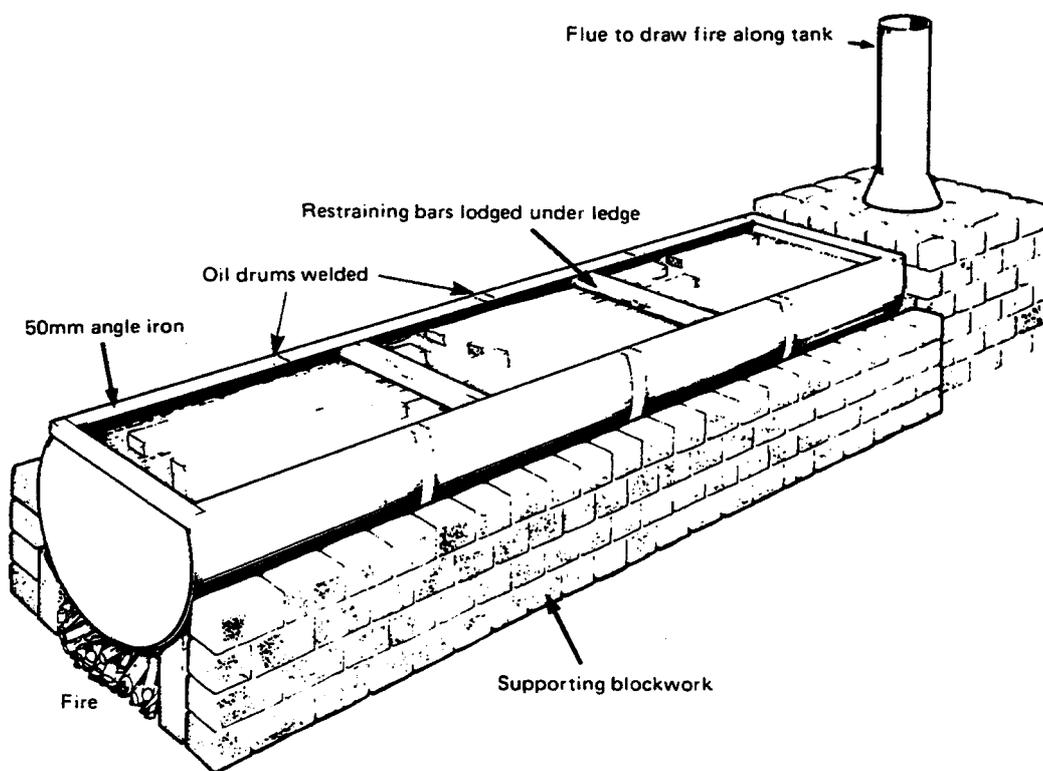


Figure 14.1 Hot and cold timber treatment tank

14.4.2 Classification of timber for bridge engineering

Timber is commonly classified into two basic groups hardwood and softwood. Normally hardwood is stronger, more durable and therefore more suitable for bridge building, but this is not always the case. Cutting, drilling or nailing some of the denser hardwoods may cause problems, while species such as camphor are known to have a sap which reacts with steel, corroding nails, screws and fittings.

A useful guide to the strength of wood is given by its density, which in turn is related to fineness of grain and rate of growth, indicated by the distance between the annual growth rings. In general, species that have a high density are strong; but the strength of any individual piece of timber will depend on the number and size of knots, fissures and other faults within it.

In this manual, to provide standard log or beam bridge designs, as shown in Figures 11.3 and 11.4, timber has been categorised into three groups: heavy hardwood, lighter hardwood, and softwood.

Hardwoods with a density greater than 650kg/m^3 after drying to a moisture content of 18% are considered heavy. Methods of measuring timber moisture content are given in BSI (1997c) Appendix A. The timber species set out in Table 14.3 as examples of the three groups are all considered suitable for bridge works. The lists are by no means comprehensive. The bridge engineer is expected to use knowledge of local timber and practice when judging whether a particular species is suitable for bridge construction, and which group to place it in.

14.5 Masonry

Stone or brick masonry may be used in river bed protection, culvert headwalls, curtain walls for fords and raft foundations, bridge abutments or the barrels and spandrels of arch bridges. This is dealt with more fully in Chapter 15.

14.6 Rubber bearings

Guidance on the selection of material and installation may be found in BSI (1983c). If purpose-made rubber bearings are not readily available, natural rubber sheet of hardness between 45 and 75 IRHD to BSI (1995) may be used. This is more successful when installed in discrete lengths of about 1m.

Rubber bearings may be laid directly onto the bearing shelf concrete, provided the surface is flat and smooth. If a bedding pad of mortar is used, this should be kept thin enough not to affect the specified protruding length of the steel dowels. Both the bedding pad and bearing shelf should be constructed so as to drain water away from the bearing and avoid trapping debris that can harbour moisture.

Where bearings are installed before casting an in situ concrete deck, formwork around the bearings should be carefully sealed to prevent grout leakage.

Before assembly onto the rubber pad, the bearing flanges of steel beams should be protected from corrosion with a thin layer of resin mortar, or a similar material.

14.7 Joint sealant

The expansion gap between a concrete bridge deck and the ballast wall on the abutment is best sealed with a cold applied proprietary material, even when covered with a layer of asphalt surfacing as shown in Figures 9.3 and 10.12. The methods of application and use of joint sealants are described in BSI (1990b).

Should suitable sealing material not be available, the expansion gap may be left open, but if this alternative is adopted minor variations to the abutments and deck are required (Figures 14.2, 14.3 and 14.4). The most important points are to arrange good drainage for the water penetrating the joint, and to make it possible to clean the bearing shelf of small stones and other debris. In practice it is much easier to provide slopes for drainage and access for cleaning on a bearing shelf supporting a composite deck than on one supporting a reinforced concrete deck.

The vertical alignment and crossfall of the approaches to all bridges should be designed to drain surface water into side ditches and not permit any water to flow onto the deck. This is even more important when the expansion gaps are not sealed.

14.8 Construction joint seals

When it is not possible to cast the concrete slab of a bridge deck in one continuous pass, a construction joint is made at the junction of the two parts.

If suitable proprietary joint sealants for application in the horizontal and vertical plane are available, the engineer is advised to consider these for application at construction joint locations. Correctly applied, these sealants can prevent moisture seepage at construction joint interfaces and any resultant corrosion of the steel reinforcement. It is generally required that a rebate is built at the construction joint and filled with the sealant. However, should the engineer be aware that this type of joint is unlikely to be adequately maintained, he should not specify the groove or sealant, but simply call for the two concrete faces to be butted together.

If proprietary internal and external water bars are available, these should also be considered for installation at construction joints to promote watertightness of the detail.

Table 14.3 Timber species suitable for bridge beams

Table 14.3(a)	<p>Group A – Heavy hardwoods Density >650kg/m³ when dried to 18% moisture content</p> <hr/> <p>Some of the common species in this group are:</p> <table border="0"> <tr> <td>Afrormosia</td> <td>(Pericopsis elata)</td> </tr> <tr> <td>Ekki</td> <td>(Lophira elata)</td> </tr> <tr> <td>Greenheart</td> <td>(Ocotea rodiaei)</td> </tr> <tr> <td>Iroko</td> <td>(Chlorophora excelsa, regia)</td> </tr> <tr> <td>Jarrah</td> <td>(Eucalyptus marginata)</td> </tr> <tr> <td>Karri</td> <td>(Eucalyptus diversicolor)</td> </tr> <tr> <td>Keruing (gurjun)</td> <td>(Dipterocarpus spp)</td> </tr> <tr> <td>Opepe</td> <td>(Nauclea diderrichii)</td> </tr> <tr> <td>Sapele</td> <td>(Entandrophragima cylindricum)</td> </tr> <tr> <td>Teak</td> <td>(Tectona grandis)</td> </tr> </table>	Afrormosia	(Pericopsis elata)	Ekki	(Lophira elata)	Greenheart	(Ocotea rodiaei)	Iroko	(Chlorophora excelsa, regia)	Jarrah	(Eucalyptus marginata)	Karri	(Eucalyptus diversicolor)	Keruing (gurjun)	(Dipterocarpus spp)	Opepe	(Nauclea diderrichii)	Sapele	(Entandrophragima cylindricum)	Teak	(Tectona grandis)
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Sapele	(Entandrophragima cylindricum)																				
Teak	(Tectona grandis)																				
Table 14.3(b)	<p>Group B – Lighter Hardwoods Density <650kg/m³ when dried to 18% moisture content</p> <hr/> <p>Some of the common species in this group are:</p> <table border="0"> <tr> <td>African Mahogany</td> <td>(Khaya ivorensis, anthotheca)</td> </tr> <tr> <td>Afzelia</td> <td>(Afzelia spp.)</td> </tr> <tr> <td>Dahoma</td> <td>(Piptadeniastrum africanum)</td> </tr> <tr> <td>Gum</td> <td>(Eucalyptus saligna)</td> </tr> <tr> <td>Jacareuba</td> <td>(Calophyllum brasiliense)</td> </tr> <tr> <td>Meranti</td> <td>(Shorea spp.)</td> </tr> <tr> <td>Muminga</td> <td>(Pterocarpus anyolensis)</td> </tr> </table>	African Mahogany	(Khaya ivorensis, anthotheca)	Afzelia	(Afzelia spp.)	Dahoma	(Piptadeniastrum africanum)	Gum	(Eucalyptus saligna)	Jacareuba	(Calophyllum brasiliense)	Meranti	(Shorea spp.)	Muminga	(Pterocarpus anyolensis)						
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Meranti	(Shorea spp.)																				
Muminga	(Pterocarpus anyolensis)																				
Table 14.3(c)	<p>Group C – Softwoods. Softwoods suitable for bridge construction should generally have: Density >420kg/m³ when moisture content = 18%</p> <hr/> <p>Some of the common species in this group are:</p> <table border="0"> <tr> <td>Cedar</td> <td>(Cedrus spp.)</td> </tr> <tr> <td>Cypress</td> <td>(Cuppressus spp.)</td> </tr> <tr> <td>Douglas fir</td> <td>(Pseudotsuga taxiflora)</td> </tr> <tr> <td>Kauri, East African</td> <td>(Agathis alba)</td> </tr> <tr> <td>Parana Pine</td> <td>(Araucaria angustifolia)</td> </tr> <tr> <td>Pine, Caribbean Pitch</td> <td>(Pinus Caribaea)</td> </tr> <tr> <td>Pine, Scots or Redwood</td> <td>(Pinus sylvestries)</td> </tr> </table>	Cedar	(Cedrus spp.)	Cypress	(Cuppressus spp.)	Douglas fir	(Pseudotsuga taxiflora)	Kauri, East African	(Agathis alba)	Parana Pine	(Araucaria angustifolia)	Pine, Caribbean Pitch	(Pinus Caribaea)	Pine, Scots or Redwood	(Pinus sylvestries)						
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Pine, Caribbean Pitch	(Pinus Caribaea)																				
Pine, Scots or Redwood	(Pinus sylvestries)																				

Table 14.4 may be used to classify local species not found in Table 14.3.

Table 14.4 Permissible short-term stresses

	GROUP A: Heavy Hardwoods N/mm²	GROUP B: Lighter Hardwoods N/mm²	GROUP C: Softwoods N/mm²
Bending	15.1	8.6	5.4
Tension	9.0	5.0	3.2
Compression parallel to the grain	11.3	6.8	5.0
Compression perpendicular to the grain	2.2	1.8	1.5
Shear parallel to the grain	2.2	1.1	0.9

14.9 References

ACI (1991) *Hot weather concreting*. Manual of concrete practice A CI3OSR-89. American Concrete Institute, Detroit.

ASTM (1986). *Standard test methods for compressive strength of cylindrical concrete specimens*. C39. American Society for Testing of Materials, Philadelphia.

BRE (1971). *Problems of concrete production in arid climates*. Overseas Building Notes No.139. Building Research Establishment, Watford.

BRE (1975). *The hot and cold open tank process of impregnating timber*. Technical Note No 42. Building Research Establishment, Watford.

BRE (1981). *Concrete in sulphate-bearing soils and ground waters*. BRE Digest 250. Building Research Establishment, Watford.

BRE (1988a). *Design of normal concrete mixes*. Building Research Establishment, Watford.

BRE (1988b). *Alkali aggregate reactions in concrete*. BRE Digest 330. Building Research Establishment, Watford.

BSI (1983a). *Methods of testing concrete Part 116. Method for determination of compressive strength of concrete cubes*. BS 1881: 1983 Pt 116. British Standards Institution, London.

BSI (1983b). *Methods of testing concrete Parts 101 to 127*. BS 1881.1983. British Standards Institution, London.

BSI (1983c). *Steel concrete and composite bridges, Section 9.2. Specification for materials, manufacture and installation of bridge bearings*. BS 5400 Section 9.2:1983. British Standards Institution, London.

BSI (1989). *Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete*. BS 4466: 1989. British Standards Institution, London.

BSI (1990b). *Cold applied joint sealant systems for concrete pavements Part 2. Code of practice for the application and use of joint sealants*. BS 5212 Pt 2:1990. British Standards Institution, London.

BSI (1992). *Aggregates from natural sources for concrete*. BS 882:1992. British Standards Institution, London.

BSI (1993a). *Structural steel sections Part 1. Specification for hot-rolled sections*. BS 4:1993 Pt 1. British Standards Institution, London.

BSI (1993b) *Weldable structural steels*. BS EN 10025: 1993. British Standards Institution, London

BSI (1995). *Methods of testing vulcanised rubber Part A 26. Determination of hardness*. BS 903: 1995 Pt A 26. British Standards Institution, London.

BSI (1996a). *Portland blast furnace cement Part 2. Metric units*. 135146: 1996 Pt2. British Standards Institution, London.

BSI (1996b). *Portland cements*. BS 12:1996. British Standards Institution, London.

BSI (1996c). *Portland pulverised fuel ash cement*. BS 658& 1996. British Standards Institution, London.

BSI (1996d). *Sulphate resisting Portland cement*. 135 4027.1996. British Standards Institution, London.

BSI (1997a). *Carbon steel bars for the reinforcement of concrete*. 1354449:1997. British Standards Institution, London.

BSI (1997b). *Structural use of timber Part 5. Code of practice for the preservative treatment of structural timber*. 135 526&~1 997. Pt 5. British Standards Institution, London.

BSI (1997c). *Wood preservation using coal tar creosotes Part 2. Methods for timber treatment*. 135 144: 1997. British Standards Institution, London.

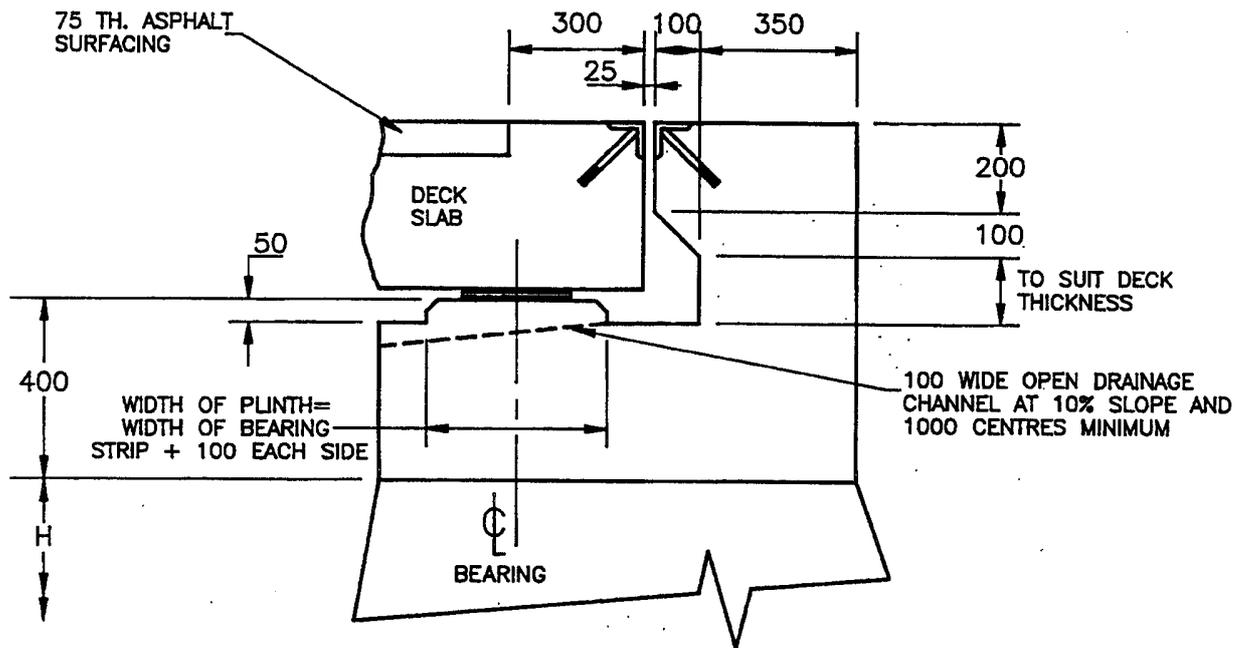
C&CA (1979, 1980). *Man on the job leaflets*. Nos 45.101 to 45.116. Cement and Concrete Association, Slough.

Highways Agency (1986). *Specification for Highway Works*. Part 5, series 1700 Structural concrete. Stationery Office, London.

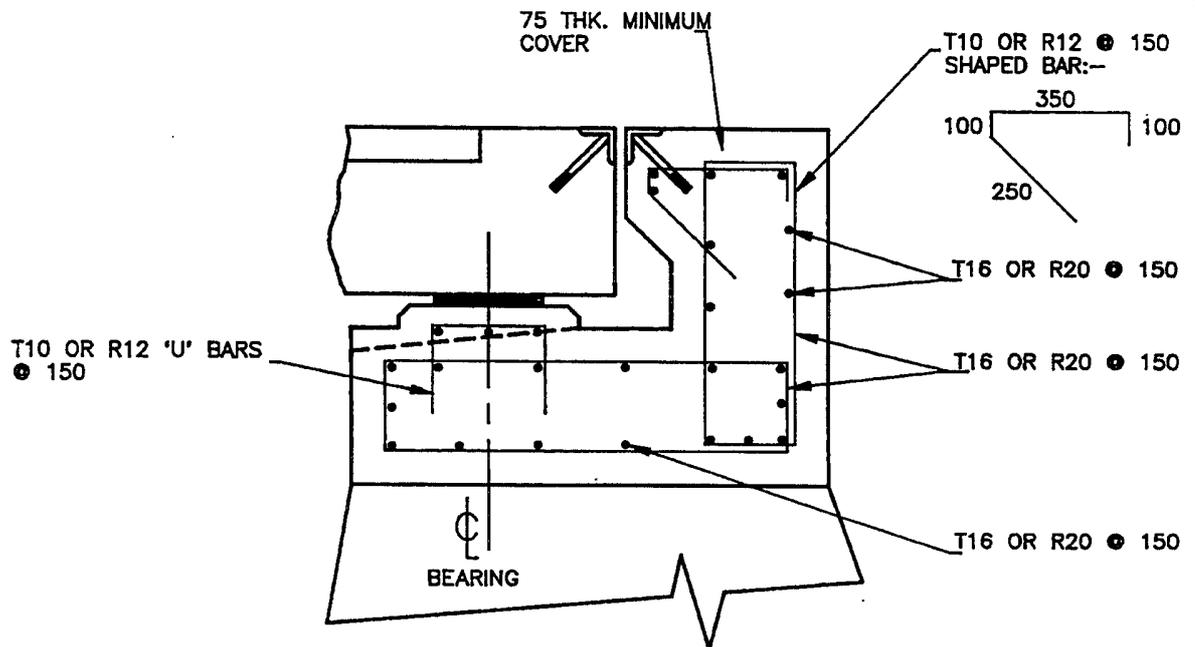
Tack C H (1979). *Preservation of timber for tropical building*. Overseas building notes, No 183. Building Research Establishment, Watford.

14.10 Standard drawings for unsealed expansion joints

<i>Title</i>	<i>Figure number</i>
Alternative expansion joint on a concrete deck	14.2
Alternative expansion joint on a composite deck	14.3
Alternative expansion joint at a pier	14.4



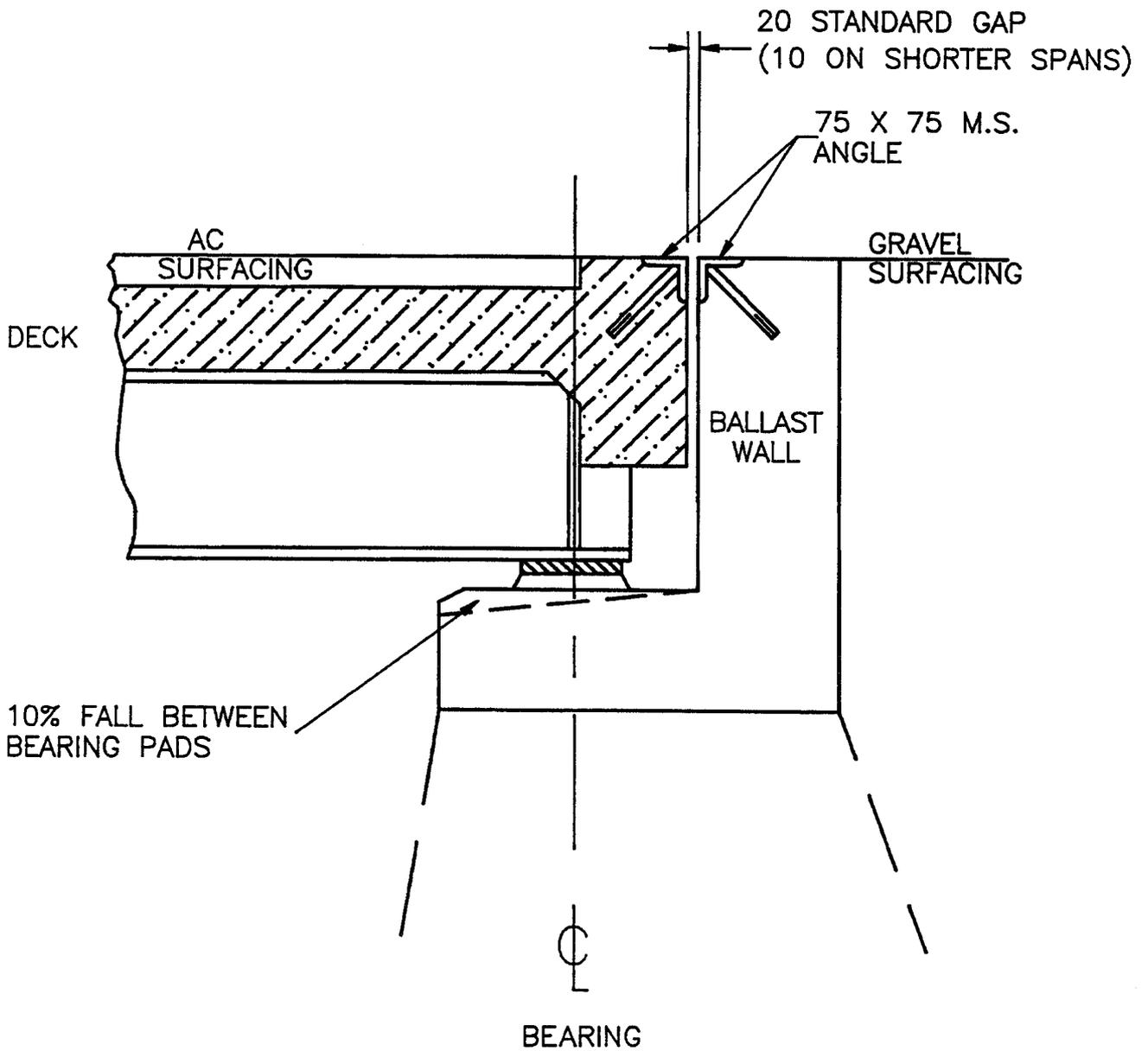
NOTE: BEARING PLINTH AND BEARING STRIP ARE NOT CONTINUOUS AT 100 WIDE DRAINAGE CHANNELS.



REINFORCEMENT DETAILS

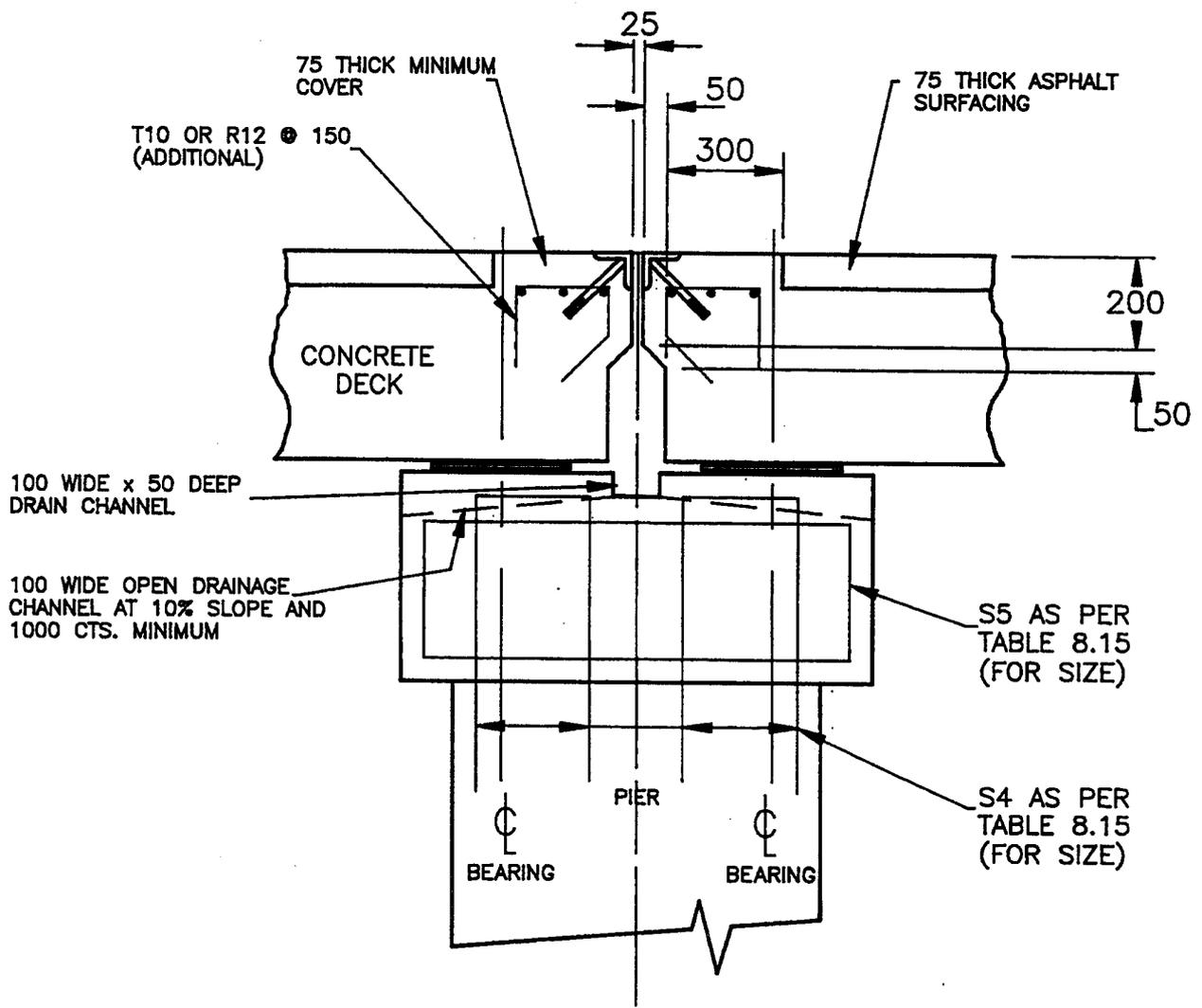
ALL DIMENSIONS IN MILLIMETRES(mm)

Figure 14.2 Alternative open expansion joint for a reinforced concrete deck and bearing shelf



ALL DIMENSIONS IN MILLIMETRES(mm)

Figure 14.3 Alternative expansion joint on a composite deck



ALL DIMENSIONS IN MILLIMETRES(mm)

Figure 14.4 Alternative open expansion joint at a pier

15 MASONRY

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15 Masonry

Masonry is the term used to cover all construction work involving the laying of rough or dressed stones, bricks and cement blocks. Masonry technology usually makes use of locally available resources such as stones or bricks, sand and labour skills. Smaller masonry structures can meet the necessary design standards at a significantly lower cost than concrete ones, especially for the secondary and tertiary road network. Careful planning, however, is still required for this kind of structure and skilled labour is essential for the construction of quality masonry work. An example of a masonry structure is given in Figure 15.1.

15.1 Use of stone and brick masonry for road works

History:

Stone and brick masonry have been used to build walls since ancient times. Nearly all of the most famous and largest buildings and structures that have survived from those days have been built using masonry technology. Some of these structures are still in use today. Bridges for roads, waterways and railways were constructed with masonry arch methods to cross rivers or valleys of any size before concrete technology was developed.

Use of masonry today:

Stone or brick masonry is very useful for the

construction of low cost structures, such as bridge abutments, retaining walls, culvert head walls and wing walls, river bed protection, drifts and the barrels and spandrels of arch bridges. If the two component materials, ie, stone / brick units and cement mortar, are both of sufficient quality to withstand load and abrasion, then masonry may be used instead of mass concrete where it is specified elsewhere in this guide.

15.2 Design guidelines

These guidelines are applicable for minor works only, typically for wall heights up to 4m and spans up to 6 m. For major masonry structures detailed engineering design is required. The adoption of a simple construction method does not dispense with the need for an adequate survey and design.

15.2.1 Foundations

Foundations for load-bearing walls (mainly abutments) are usually made of reinforced concrete and must be appropriately designed (see Chapter 8: Substructures and Foundations). This requirement is essential for abutments supporting rigid superstructures.

For walls up to 1.5m in height, which do not support additional imposed loads, no special concrete foundation is required; however, the following guidelines should be followed:

- The base of the foundation should be a minimum of 400mm below ground level on uniformly firm ground.

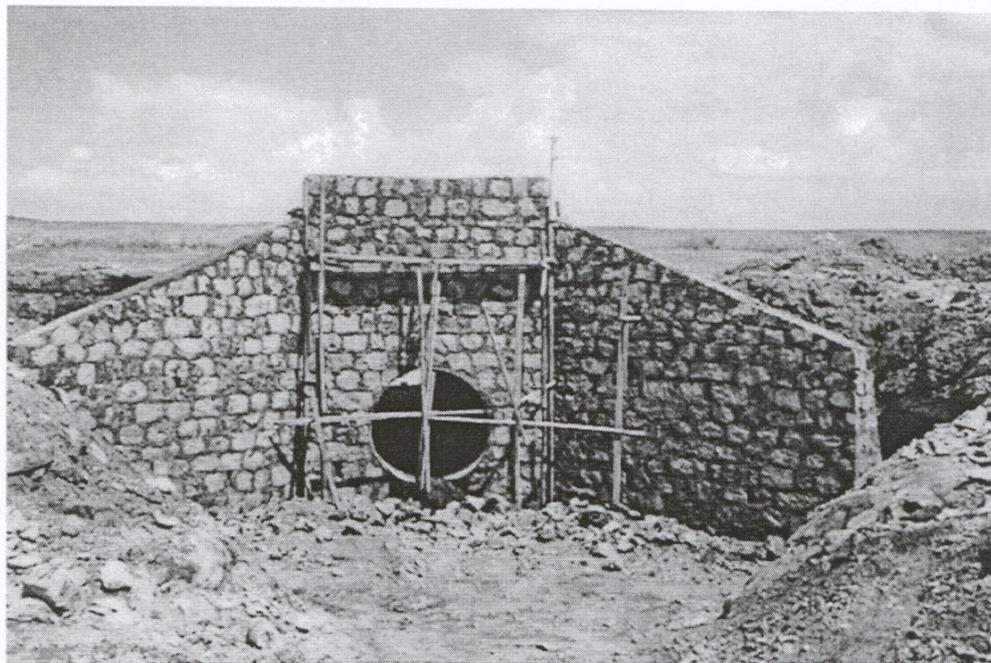


Figure 15.1 Culvert with masonry headwall and wingwalls

- A 50mm lean concrete layer (1:4:8, ie, cement: sand: coarse aggregate, by volume), is required as a clean base for the foundation.
- The first course (footing) should be laid with the largest and straightest stones, because the stability of the wall depends largely on the bearing of the stones on the ground.
- Where the ground has a doubtful bearing capacity, a masonry foundation is required. Chapter 8 discusses the bearing capacity of several types of soil. If there is doubt about the strength of the supporting soil then a plate bearing test can be carried out according to Section 8.3.3, or values can be used from Section 8.3.4.
- If a foundation is required, the width of this foundation should be double the thickness of the wall, with a minimum of 800mm. For retaining walls, foundation sizes can be taken from Table 8.5.

Figure 15.2 shows typical non-bearing walls and the foundations needed.

15.2.2 Walls

The minimum thickness for structural walls up to a height of 1.5m built using stones or bricks is 400mm

The thickness for load-bearing walls or those higher than 1.5, which also have to withstand earth pressure, should be adequately dimensioned (see Chapter 8: Substructures and Foundations, Section 8.4.1 Mass concrete abutments).

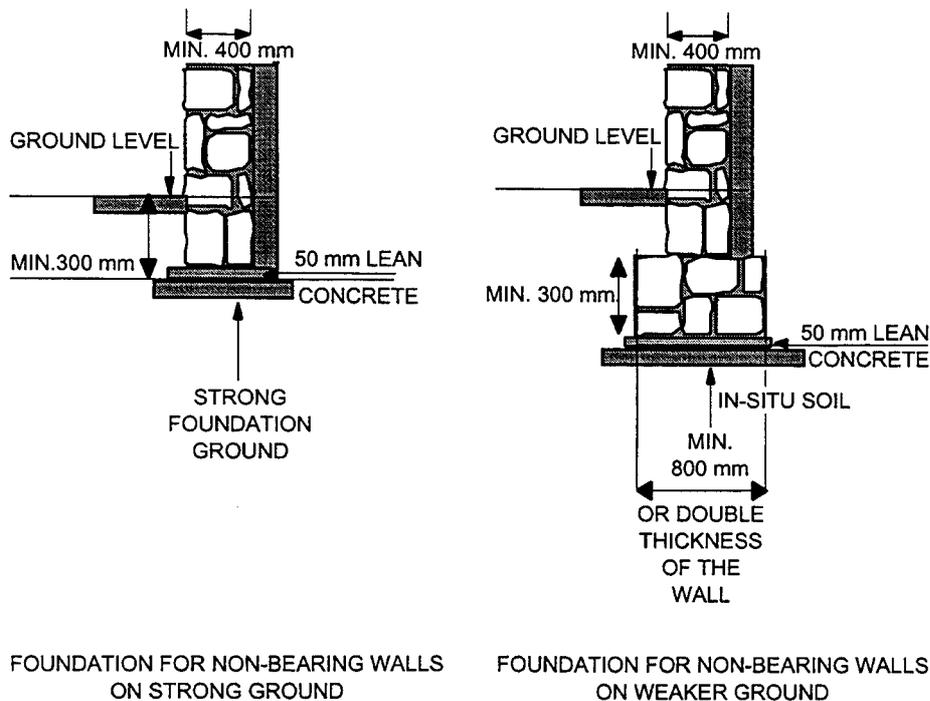


Figure 15.2 Foundations for non-load bearing walls

Attention should be given to the correct construction of the top of the wall. This is known as wall capping or the wall crown. A slope towards the outer side of the wall should be provided to allow any water to drain effectively and not to enter the wall structure. Capping should be completed as the last layer of masonry is laid.

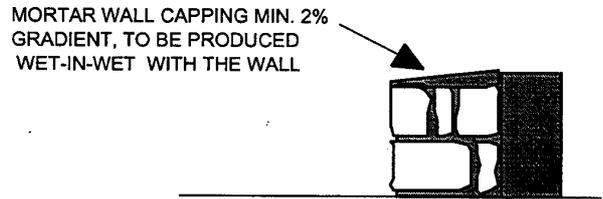


Figure 15.3 Mortar capping

15.2.3 Retaining walls

All retaining walls should be backed with a layer of free-draining material to prevent the build up of water pressure. Long walls over 2m high will also need weep holes, at least every 2m, as shown in the drawings in Chapter 8. Dimensions for retaining walls can be taken from Figures 8.12 and 8.13.

15.3 Materials and quantities

15.3.1 Mortar

The mortar used for masonry structures is a mixture of cement, sand and water. Lime should not be used for road structures. The strength of the bond will

depend on factors such as:

- the amount of cement used;
- the amount of water used;
- the type and quality of the sand;
- the surface characteristics of the stones;
- the quality of the workmanship.

It is generally recommended that the mortar should be no stronger than the bricks or blocks, so that any cracks that develop will be in the mortar. Cracking through the blocks is more difficult to repair. Recommended mixing proportions for mortar are given in Table 15.1.

Table 15.1 Recommended mixing proportions for mortar

Construction type	Mix (Cement : Sand) by volume
Non load-bearing walls: stone lining, minor walls	1 : 7
Small retaining walls: culvert head and wing walls, retaining walls up to 1 m high	1 : 6
Load-bearing walls: walls for structures, abutments, retaining walls higher than 1 m	1 : 4

15.3.2 Cement

Ordinary Portland Cement (OPC) is recommended for use in the mortar. This type of cement should be widely available. When calculating mix proportions by volume, note that a 50kg bag will have an approximate volume of 0.036m³ (36 litres).

15.3.3 Sand

There are two main types of sand: soft sand, which has rounded particles, and sharp sand which has angular particles and is often used in concrete. For stone or brickwork mortar, soft sand is preferred because it makes a smooth, easily workable mortar. Sharp sand can be used, and it will produce a stronger mortar, but it makes a harsher, less workable mix.

Only clean sand should be used and if in doubt, this can be tested with a sediment bottle test. The test consists of a jar with straight sides being half-filled with the sample. Clean water is then added until the jar is almost three-quarters full, the lid is fastened and the jar is vigorously shaken. The sand should quickly settle but any silt or clay can take up to several hours to settle. Any fine material present will

settle on the sand forming a visible layer. The height of this layer (f) and the sand layer (s) can be measured and compared. If the sand has a fines content (ie, 100f/[f+s]) of more than 5 percent, then it is not clean sand and should not be used.

15.3.4 Water

The mixing water used in the mortar must be clean. It can be taken from taps, rivers, lakes or wells. Salt water from the sea or a lake, surface run-off water and water with other chemical or organic impurities must not be used. If no other water is available, then dirty water with organic particles can be used if it is left in a drum or a similar container until the particles have settled at the bottom. Use only the clean upper part of the water.

15.3.5 Stones

It is important to choose only good strong stones to build walls. The following stones should not be used:

- weathered stones;
- cracked or spalling stones;
- small stones (less than 200 mm diameter).

The length of any stone should not be greater than three times its height. Always choose stones whose shape is as near as possible to a rectangular prism (ie, brick shaped). They must be free of dust and dirt. It is therefore advisable to wash them and, if necessary, clean them with a brush. If mortar is to be used then the stones should be wetted, but surface dry before use to ensure a good bond with the mortar.

15.3.6 Bricks

Bricks should be uniformly burnt and of similar size (maximum tolerance 5 mm). The following bricks should not be used:

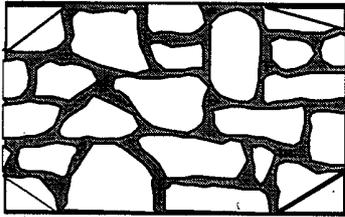
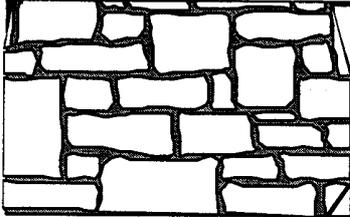
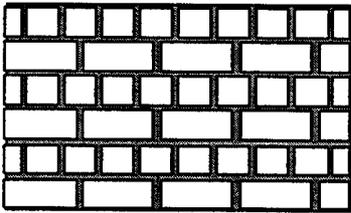
- cracked or spalling bricks;
- unevenly burnt bricks;
- 'bent' bricks.

15.4 Masonry work

The most common types of masonry wall are shown in Table 15.2, which also gives approximate material quantities.

All stratified stone that has bedding planes should be laid with the natural bed as near as possible at right angles to the direction of the load. In the case of arch rings the natural bed should be radial.

Table 15.2 Types of masonry wall and material quantities

Masonry type	Approximate width of joints	Requirements for 1 m ³ of finished wall
	Rubble stone masonry The stones are not specifically cut or shaped. To build a wall with proper bonding using rubble stones requires the skills of an experienced mason.	1 to 4 cm Stones: Approx. 1.3 to 1.5 m ³ (includes waste) Mortar: 300 to 400 litres.
	Shaped stone masonry The stones are shaped to a rectangular prism. Using these stones it is easier to produce a wall with proper bonding and uniform surface.	1 to 2.5 cm Stones: approx. 1.2 m ³ (includes waste) Mortar: 200 to 300 litres.
	Bricks Bricks of adequate quality and uniform size allow for high quality masonry work with a relatively smooth surface. Various bonding patterns are possible.	1.2 to 1.5 cm Bricks: approx. 1.1 m ³ (depends on the size of the bricks, includes waste) Mortar: 250 to 300 litres.

15.4.1 Joints and Pointing

For mortar bonded masonry, it is important that no stone or brick should touch another but each one should be fully bedded into mortar.

For road structures the joints are usually finished as 'flush joints'. The mortar between the stones is trowelled to a smooth surface flush with the face of the stone or brickwork using a mortar trowel or a pointing trowel. 'Ribbon' pointing, where the mortar stands out from the face of the stones, should be avoided in climates where frost is likely. Any water sitting on 'ribbon' mortar can damage the mortar if it freezes (BSI, 1976).

15.4.2 Bond for stone masonry

The bond should allow a minimum overlap of 1/4 length of each stone. Most of the stones are laid as stretchers, ie along the length of the wall. Header stones (also called through stones) should be laid at regular intervals across the width of the wall to bond the two faces of the wall together. The header stones should cover at least 2/3 of the wall thickness and their overlap should not be less than 100mm.

15.4.3 Mortar mixing

The mixing of mortar for small structure works is usually carried out on site by hand. The quantity of mortar to be mixed should not be more than a mason can finish using within one hour of mixing or half an hour if it is in very hot weather or strong sun. Hand-mixed batches should not exceed 0.5 m³. The mixing should never be done on the bare ground, as this results in contamination of the mix. A mixing platform of about 4m by 4m should be built with boards, metal sheets or lean concrete.

Procedure for mixing by hand

- 1) Measure the required amount of sand and cement using a gauge box of 36 litres (400mm x 300mm x 300mm).
- 2) Spread the cement and sand in alternating layers on the platform.
- 3) Mix the dry materials into a separate heap at least three times. This is best done with two persons, one on each side of the heap, who can shovel the heap to one side by turning the material in the process. This operation should be repeated, with the heap being thrown back to its original position and then back again, until the colour of the dry mix is a uniform grey.

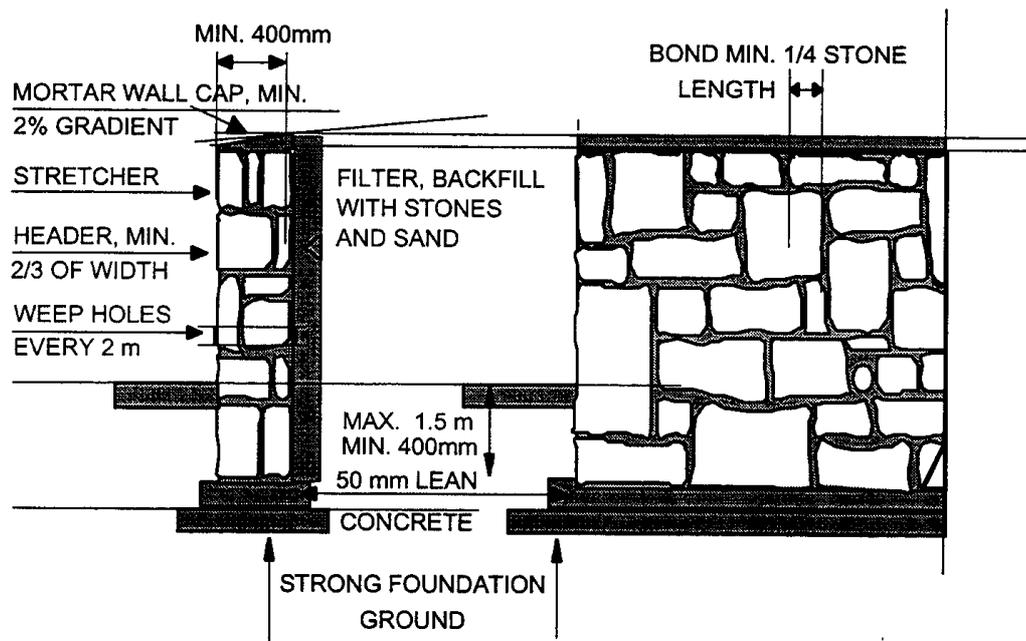


Figure 15.4 Masonry bonding

- 4) Add water. This is best done using a watering can so that the water is spread evenly while the material is mixed again. Only the correct amount of water should be added (see guideline below). The wet mixing must be continued, turning at least three times, until the mortar is uniformly wet and has reached the required consistency.

The water-cement ratio should be approximately 0.4 to 0.5, which is equal to 20 to 25 litres of water per 50kg bag of cement, or 16 to 20 litres for a 40kg bag. Trial and error is required to get the water content right, because this will depend upon factors such as the moisture content of the sand and the size of the sand particles. when the water content is right:

- the consistency should be such that the mortar does not flow off the trowel;
- the mortar can be kneaded in the hand and retains its form.

15.4.4 Rules for good quality mortar-stone masonry construction:

- 1) Construct a proper foundation.
- 2) Use only stones which are not cracked or weathered.
- 3) Clean all the stones with water and a brush.
- 4) Stones and bricks should be wetted before laying to ensure a good bond with the mortar.
- 5) Use the largest stones for the bottom layer and the corners of the wall, to ensure stability.

- 6) Use mortar of the correct mixture and consistency.
- 7) Ensure proper bonding and joints.

15.5 Arches

Arches can be used for both culverts and bridges. Their purpose is to transmit the load above to the abutments or piers on either side of the opening. Semi-circular and semi-elliptical arches are the most commonly used shapes for road structures. They can consist of single or multiple arches.

For larger structures, careful construction using wedge shaped stones or bricks is required to ensure that each stone or brick transmits load to the next stone or brick. Strong formwork will be needed to support the arch during construction.

The base and foundations of any arch structure are essential for stability. The ground must be excavated down to firm material and brought back to the required level with lean concrete or good hardcore, topped with 50mm of concrete. For culverts the base should be laid to the required gradient, as discussed in Section 12.2. This gradient is usually a minimum of 0.5% for clear water and 2 to 4 % where sediment is carried in the flow.

The formwork used for constructing arches can be made from strong timber, old oil drums or old car or lorry tyres laid side by side. Tyres and drums must be well matched in size and can be placed in a row on a stack of stones, bricks or a layer of compacted soil or sand to the required height. An example of arch culvert construction using wooden formwork is given in Figure 15.5.

The walls are then constructed up to the level where the arch begins (springing points). When building the



Figure 15.5 Masonry arch culvert with simple wooden formwork

arch, it is important to build both sides evenly, so that the formwork does not become distorted by the weight of the stones or bricks. The arch should be left to cure for at least 2 to 3 days before the formwork is removed. The stones, bricks or soil supporting the drums or tyres should be removed first. Tyres are flexible and therefore should be easy to remove.

When the formwork has been removed, the base should be cleaned of all loose material. It should then be rendered with a strong mortar screed about 50mm thick and finished slightly concave to keep small water flows away from the culvert walls.

With all culverts, it is very important to ensure that the backfill down the sides and over the arch is well compacted in thin layers (150 to 200mm) and that both sides are built up and compacted to the same level at each compaction. The fill above the arch must not be less than 500mm.

The example in Figure 15.6 shows a 'rough brick arch' constructed with normal bricks. The mortar joints are the wedges in this case. It is therefore essential to ensure that only good quality mortar (1: 4) is used and that all the joints are properly filled with mortar. It is also important to achieve proper bonding the longitudinal direction.

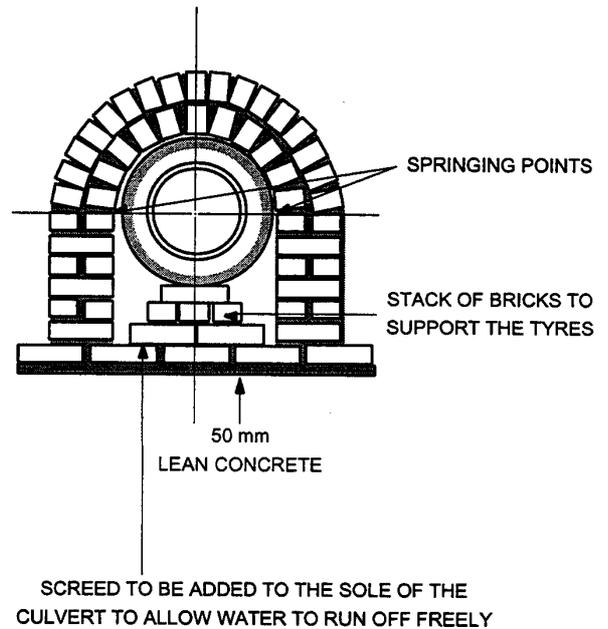


Figure 15.6 Rough brick arch

For larger structural openings the bricks need to be shaped as wedges and sized to appropriate dimensions, like the stones forming the arch in Figure 15.7.

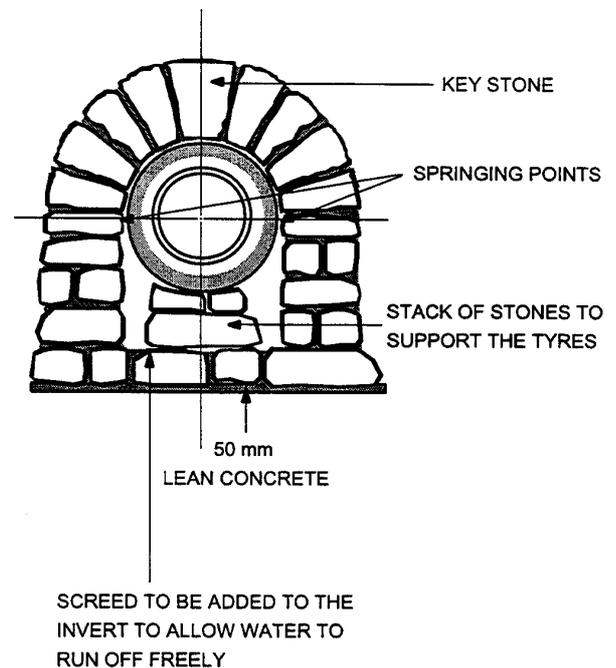


Figure 15.7 Gauged stone arch

Stone arches should be built as 'gauged stone arches' with tapered and sized stones as shown in Figure 15.7. As with brick arches, it is important to construct both sides simultaneously to avoid deformation of the formwork, eg tyres. The joints need to be fully filled with mortar of a mix of 1 :4.

15.6 Dry stone masonry

Dry stone masonry is suitable for walls which do not have to carry loads. As there is no mortar, the stones are laid to fit as tightly as possible and wedge-shaped pieces of stone are driven into the larger gaps to hold the stones firmly in place. Careful shaping, laying and bonding of the stones is essential and only skilled and experienced stonemasons should be allowed to carry out this work. Figure 15.8 shows part of a typical dry stone wall.

15.7 Masonry for river bed or slope protection

River bed protection should be laid according to Section 7.3: Bed Level Causeways.

As with other forms of river slope protection, any form of rigid cladding will be damaged by the water flow, unless it is laid on very firm material. If the river bed consists of alluvial material, a flexible cover of large stones (rip rap) or large stones in wire baskets (reno mattresses) will usually be more effective in preventing scour of the river bed and last longer than rigid cladding.

As for bed level causeways, river bed protection beneath a bridge should be constructed with curtain walls, as shown in Figure 7.4. The top surface should be flush with the level of the natural river bed.

15.8 References

BSI (1976). *BS 5390: 1976: Code of practice for stone masonry.* British Standards Institution.

BSI (1992a). *BS S62& Parts 1-3: 1992. Code of practice for use of masonry.* British Standards Institution.

Curtin W G, Shaw G, Beck J K and Bray WA (1987). *Structural masonry designer's manual. 2nd Edition.* BSP Professional Books, Oxford.

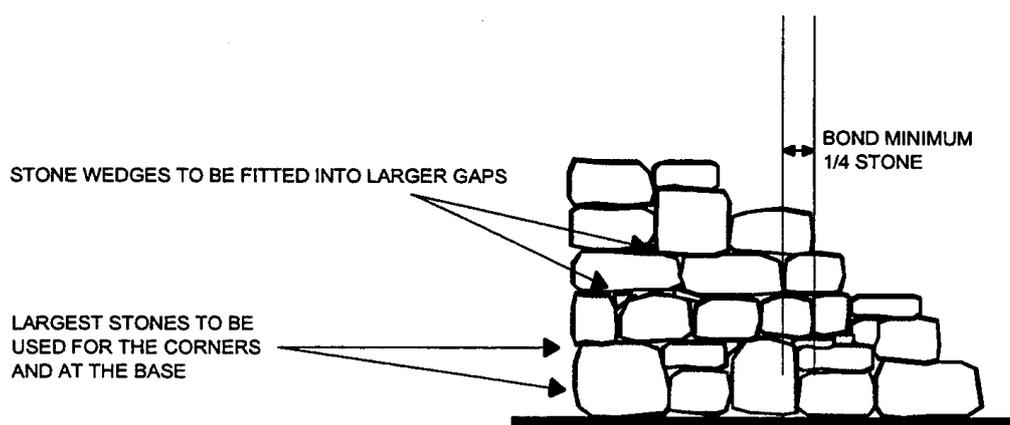


Figure 15.8 Part of a dry stone wall

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16 Drawings and specifications

When the designs for foundations, substructures, superstructure and river works have been selected on the basis of the preceding chapters of this manual, and all the necessary modifications to suit local requirements and conditions have been made the engineer must prepare drawings in sufficient detail and with all necessary dimensions to enable the structure to be built by direct labour or an outside contractor.

Since the drawings contain all the information required to build the structure, there should be no need for the builder to consult the designer. In practice, however, liaison between design office and construction site usually benefits both parties and is particularly helpful when unforeseen conditions are met. It is usual for the Resident Engineer, in consultation with the designer, to agree modifications with the contractor, with the aim of saving unnecessary costs, compensating for poor soils or using different materials to those specified.

A complete set of drawings and calculations is also required for the bridge inventory. This set should include any modifications that may be introduced during construction and is to be updated whenever repairs or strengthening, etc take place on the structure or river works.

The application of these guidelines will result in a complete set of information. Since it is helpful also to follow local practice, the following notes should be treated as advisory; their object is to provide all the necessary information in a clear and simple form.

16.1 Drawing number 1: site plan and longitudinal section

16.1.1 *The site plan, drawn to a scale of about 1:500*

- contains a north point and shows the direction of the nearest town;
- shows contours or spot levels of the river bed and the surrounding ground in the area of the bridge site;
- details the bench marks and levels established during the survey;
- shows the locations of the bore holes and trial pits put down during the site investigation;
- indicates the limits of the design flood and high flood, with direction of flow;
- specifies the horizontal alignment of the road approaches and the bridge.

16.1.2 *The longitudinal section, drawn to the same horizontal scale*

- contains relevant details of the subsoil conditions obtained from bore holes and trial pits;
- shows the design flood level, the high flood level and the low water level with dates;
- specifies the vertical alignment of the road approaches and the bridge.

16.2 Drawing number 2: bridge plan and sections

This drawing contains a plan, longitudinal section and cross section of the structure. It is drawn to a scale of about 1:100 and specifies the following major dimensions:

- abutment and pier width, height, bearing shelf levels and foundation levels;
- superstructure span, width and height to the underside;
- wing wall lengths, heights and foundation levels;
- finished road surface levels over the bridge.

16.3 Drawing number 3: substructure details

On this drawing the elevations, plans and sections selected from Chapter 8 give all the dimensions and levels required for the setting out and construction of the abutments, piers and wing walls. Concrete and reinforcement specifications should also be included.

16.4 Drawing number 4: superstructure main details

This drawing contains plans and sections detailing the bridge deck, selected from Chapters 9, 10, 11 or 13. It includes material specifications and bending schedules for reinforcement.

16.5 Further detailing

Further drawings are prepared as required in order to detail and specify materials for parapets, bearings, joints and drainage, etc, on the bridge, as well as river training, embankment protection and scour protection measures in the river.

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