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BRIDGE CONSTRUCTION TECHNIQUES MANUAL

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TABLE OF CONTENTS

1. INTRODUCTION

1.1 SCOPE OF MANUAL

This Manual describes a number of matters relating to construction supervision of bridgeworks in Indonesia which most often produce problems for supervision staff. It should be used in conjunction with the Bridge Construction Supervision Manual.

The Manual describes aspects of construction which will provide the best opportunities for savings of time and money and which will lead to improved quality of construction by use of the most appropriate construction techniques and methods.

1.2 SUMMARY OF STANDARD BRIDGE TYPES

1.2.1 General

There is a large range of foundations and substructures used in Indonesia.

Cast in-situ concrete caissons, bored steel piles, driven steel piles and driven reinforced and prestressed concrete piles are all widely used.

Pile cap and single or multiple column pier and crosshead arrangements are used for substructures.

It is in the area of superstructures however, that the greatest degree of standardisation has taken place. A number of standard steel truss and girder systems have been developed and the use of precast concrete beams is also widespread.

Usually the design will be fixed and completed by the time the construction supervision team becomes involved in the project. Occasionally, there will be a need for a design review to be carried out at the commencement of the project and the supervision personnel should be aware of the advantages and disadvantages of the various design configurations with respect to their suitability to a particular site and the ease of construction.

1.2.2 Foundations

Foundations present a source of problems to Contractors due to the fact that details of the ground conditions are rarely known precisely.

Foundation types fall into two main types for construction. The first are those which may be constructed without requiring the use of specialised equipment. This type of construction includes spread footings and concrete caissons which are cast in-situ.

The second type includes piled foundations and precast concrete or steel shell caissons. Piled foundations may be either driven or bored and piles may be steel (sections or shells) or concrete.

1.2.3 Substructures

Abutments, generally of the concrete wall and beam type, provide a seating for the bridge bearings and retain the embankment behind the abutment. Where a spill-through abutment is used, the abutment acts as a cap and seating for the bearings.

Gravity abutments using stone masonry and concrete seating and backwall arrangements are also often used.

Piers may be either a trestle arrangement, i.e. a concrete cap acting as a cross beam with the pile heads embedded in the cap, or a column arrangement which has a separate pile cap, column and cross head system.

It is common practice in Indonesia to use the trestle arrangement for piled foundations.

This has the piles extending directly into the pier crosshead. The major advantages are cost, ease of construction and a reduction in the amount of potential scour in a river. The main disadvantage is that it is not pleasing in appearance, especially at times of low flow. Increased debris accumulation also increases loads on the piles and could have an adverse effect on afflux during times of flood. In addition, the pile cap is often located at a relatively high elevation above the water.

Where well foundations are used for a pier a concrete cap, column system and crosshead beam are used. The column system may have single or multiple columns or may in fact be a solid wall. Abutments with well foundations usually have the abutment structure seated directly on the well foundations.

This system is sometimes also used for piled foundations.

1.2.4 Superstructures

a. General

Superstructures are generally steel or concrete. Longer span bridges in Indonesia are of steel construction. Steel girders and trusses are generally of standard design and are supplied from a number of sources. Concrete girders of standard design, either reinforced concrete or prestressed concrete, are fabricated on site or in precasting factories located at several locations in Indonesia. These girders are usually supplied to the Contractor as part of the construction contract.

b. Steel Superstructures

i. Trusses

The steel trusses commonly in use in Indonesia are listed in Table 1.1.

The Callender Hamilton truss bridging from the United Kingdom is no longer being used.

The difference between the various trusses are in the section dimensions of the components.

Some Australian designed trusses have been manufactured in Indonesia by Trans Bakrie. These, the latest Special (H series) trusses and the Dutch and Austrian (permanent) trusses have steel trough decking with a concrete topping. The other trusses use timber formwork for the in-situ concrete deck.

The Australian Semi Permanent truss is usually constructed with a timber deck but has been designed to carry a concrete deck if required. The decking in this case consists of steel troughing and a concrete topping.

The C Class Dutch and the Austrian Semi Permanent bridges have no top bracing as the top chords provide sufficient lateral stiffness. The A and B class Dutch bridges have vertical members at each chord diagonal intersection point.

The Dutch bridges have a fence attached to the footway or kerb. The other trusses have handrails attached directly to the truss.

ii. Girders

Girder spans from Canada, Japan and Australia have been used in Indonesia.

The Canadian and Japanese girders are unlikely to be used for any new construction work in the future.

The Australian girder bridge is a Transfield-M.B.K. design with spans in the range 20 to 30 metres. The bridges are available in Class A, B, or C configuration which differ in roadway width and kerb/footway configuration. They are designed to have composite reinforced concrete decks. The bridging has been designed for assembly using hand tools only and all site connections are by bolting. Erection may be by lifting into place (girder by girder), assembly in place on falsework supports or rolled out after assembly on the bank making use of an anchor span and special link steel. Assembly on falsework is the most common method used in Indonesia due to the lack of cranes on many sites.

The weight of individual assembled girders is as follows:

20 metre Girder - 3.3 tonnes
25 metre Girder - 5.3 tonnes
30 metre Girder - 7.9 tonnes

All components for Australian girders are galvanised.

Table 1.1 - Truss Bridge Systems

TYPE	CLASS	WIDTH	SPANS	LOADING	ERECTION METHOD	COMMENTS
Austrian						
- Semi Permanent	C	4.1 m	15m to 35m in 5m increments	BM70	Cantilever	Timber Decking
- Permanent	A,B,C	varies	35m to 60m in 5m increments	BM100	Cantilever,Falsework	
Australian						
- Semi Permanent	C	3.5 m	30m to 60m in 5m increments	BM70	Cantilever, Falsework	M Series components
- Permanent	A	7m	L 50m ,55m ,60m	BM100	Cantilever, Falsework, Single Span Launching	
	B	6m	L 60m	BM100	ditto	
	A	7m	S 35m, 40m, 45m	BM100	ditto	
	B	6m	S 45m, 50m, 55m	BM100	ditto	
	C	4.5m	S 55m, 60m	BM100	ditto	
	B	6m	M 35m, 40m	BM100	ditto	
	C	4.5m	M 35m, 40m, 45m, 50m	BM100	ditto	
	A	7m	H 80m, 100m	BM100	Half Span Cantilever, Falsework	
	B	6m	H 80m, 100m	BM100	ditto	
- Trans Panel		4.1m	10m to 50m in 10m increments	BM70		Other systems also used, e.g. Bailey, Armco and Mabey
Dutch						
- Permanent	A,B,C	7, 6, 4.5m between kerbs	40 to 60m in 5m increments	BM ??		Hollandia Kloos (also 100 and 105 m spans)

c. Concrete Superstructures

i. General

Concrete superstructures include both reinforced concrete and prestressed concrete. Reinforced concrete (R.C.) girders are generally only used for shorter span bridges although spans up to 25 metres can be used. At this span however the use of prestressed concrete (P.S.C.) beams is much more economical. A number of different types of P.S.C. beams have been used in Indonesia.

Concrete decks are not usually prestressed and are cast in-situ. There are deck systems which are made up of prestressed planks, stressed together to form an integral deck system, which do not require an in-situ concrete deck. Other systems use precast concrete slabs as part of the permanent deck. These slabs also act as formwork for the cast in-situ concrete deck.

ii. Reinforced Concrete

R.C. girders may be either precast or cast in-situ. Small spans may be cast integrally with the deck. Sections are often rectangular. Longer spans may be cast in sections and joined in-situ. They are usually I-sections or inverted Tee-sections.

R.C. slab bridges are sometimes used for very small spans. These usually consist of a deck slab with (optionally) some type of beam cast integrally with the slab.

iii. Prestressed Reinforced Concrete

Prestressed units may be either pre-tensioned or post-tensioned.

Pre-tensioned units are usually slabs, often with voids to reduce the dead load. They may be placed side by side and post tensioned using a threaded bar or single cable at two or three points along the span to form a single structural unit. A concrete deck may be cast in-situ or asphalt laid directly on the units.

Typical spans for pretensioned units are 5 to 15 metres.

Post tensioned units are usually longer span units, typically from 20 to 45 metres. They may be Tee or I-sections and cast in a single length or in segments which are joined and post-tensioned in-situ prior to the application of dead load.

A cast-in-situ deck is usually used with I-sections but Tee-sections may have simple R.C. infills between the top flanges.

R.C. diaphragms are usually cast at each end (and possibly also at intermediate locations) of the beams to provide lateral stability.

1.3 SELECTION OF APPROPRIATE CONSTRUCTION METHOD

The type of structure will often dictate which method of construction for the foundation, substructure and superstructure is the most efficient in terms of time and money.

Site conditions also play an important part but often the contractor can carry out the construction work more efficiently by a suitable choice of construction method (for example the use of cantilever erection of a truss system over a very deep gorge rather than using falsework).

2. PILING TECHNIQUES

2.1 GENERAL

2.1.1 Transport

Transport of steel piles is not usually a problem in Indonesia. Steel pile shells are usually supplied in 6 metre lengths as they can be easily spliced on site by welding.

Concrete piles may be supplied in various lengths. The 15 metre lengths require the use of semi-trailers for transport as they must be supported at the quarter or fifth points. There is another series of piles which are supplied in 8 metre lengths as upper segments and lower segments. The upper segments usually have a steel plate for splicing to the lower segment. Refer also to Section 2.2.4 for details of handling and storage of concrete piles.

2.1.2 Preparation for piling installation

The area where piles are to be installed should be prepared as level as possible, especially when tracked cranes are used with pile leaders. The ground surface must be strong enough to take the load of the crane or other plant to be used for pitching and driving or boring of the piles.

Where piles are to be installed over water consideration should be given to constructing a falsework bridge with finger piers for pile driving. If suitably designed falsework is installed it may also be used during erection of the superstructure.

When pile driving is to be carried out from a floating platform it is essential that suitable anchor positions be established, either on the river bank or using anchors sunk in the water, to accurately control the position of the platform. In addition it will be necessary to have some means of positioning the leaders independent of the position of the platform. Allowance must also be made for any tidal effects, especially with the driving of raker piles.

2.1.3 Setting out of Piles

In setting out piled foundations on land, baselines should be established outside the area covered by the piles. They should be set out in such a manner as to allow for checking of the pile during driving. Centre lines within the pile area become suspect as soon as driving commences because of the likelihood of pegs being disturbed by movement of plant and heaving of the ground.

Driving piles over water presents different problems. If driving is carried out from previously driven falsework centrelines can be established on the falsework.

If piles are driven from a floating platform the positioning of piles is more difficult. It can be done by establishing a baseline on part of the structure or on the bank at right angles to the bridge centreline. The position of the piles can them be determined by using two steel tapes for the base line in the form of a right angled triangle. One tape is used to measure the chainage and the other to measure the hypotenuse. The intersecting point or apex of the triangle is the position of the pile to be driven.

It will often be necessary to use a surveyor, for example when the chaining distances are too great, because of obstructions or it is not possible to establish a baseline to work from. In these circumstances the pile location is determined using theodolite and EDM equipment or other method of chaining.

2.1.4 Handling and Pitching

After being prepared for driving, piles are transported to the positions in which they are to be driven and pitched by mobile crane or dragged into position by suitable tackle. When moving piles by pulling with wire ropes and winch, the ropes must be kept clear of any guys, the pile driving frame and other obstructions. The pile should be carefully checked for position in relation to the leaders; it should be lowered until it rests on and is fully supported by the natural surface. In the case of long piles it may be necessary to open up a hole in the ground to receive the toe of the pile to leave enough room at the head for the insertion and operation of the hammer. The hammer should then be placed in position in the leaders preparatory to driving. The hammer may be lifted by crane, hoist, winch or other suitable means. Care is necessary to avoid damage to the pile by striking it with the leaders. It is essential to see that the yoke, or other fixture for holding the hammer in the leaders, is securely bolted.

2.1.5 Test Loading

On site load tests are performed on piles to verify the carrying capacity. Load tests may also be carried out during the design phase on test piles to check the estimated capacities.

Both cohesive and cohesionless soils will have their properties altered by the installation of a driven pile. In clays the disturbance causes remoulding and loss of strength. With time much of the original strength is regained and hence load testing should be performed several weeks after the pile is installed. In sands a temporary condition is created where extra resistance is developed. Shortly after installation however the extra resistance is lost, usually several days after driving.

Piles may be test loaded by one of the following means:

- Dead load in the form of kentledge added directly to the pile
- Jacking against a dead load supported above the pile
- Jacking against a cross-head anchored to two adjacent piles
- Jacking against a cross-head which is anchored down to rock by prestressing cables grouted into the rock outside the pile

Two of these methods are shown in Figure 2.1.

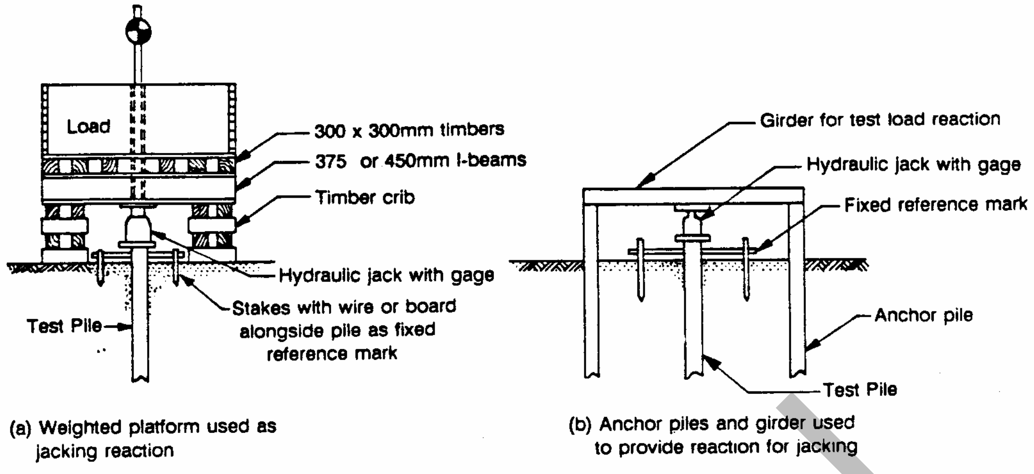


Figure 2.1 - Load Testing of Piles

Measurement of pile movement is related to a fixed reference mark. The support for reference marks needs to be located outside the soil zone that could be affected by pile movements. Distances of the greater of 5 pile diameters or 2.5 metres from the pile being tested are sometimes given as the minimum distance of the support from the pile. In any case the reference mark should be checked by independent levelling during the course of the load test.

Several different methods of load testing are in use. ASTM D 1143 describes the most common, the slow-maintained load test. With this procedure the test load is applied in eight equal increments until twice the design load is reached. Time-Settlement data are obtained for each load increment. Each increment is maintained until the rate of settlement becomes less than 2.5 mm per hour, or for 2 hours, whichever occurs first. The final load (double the design load) is maintained for 24 hours. Unloading also occurs in increments.

The ultimate pile load is taken as the load at which the slope of the load v settlement curve becomes nearly vertical, as shown in Figure 2.2.

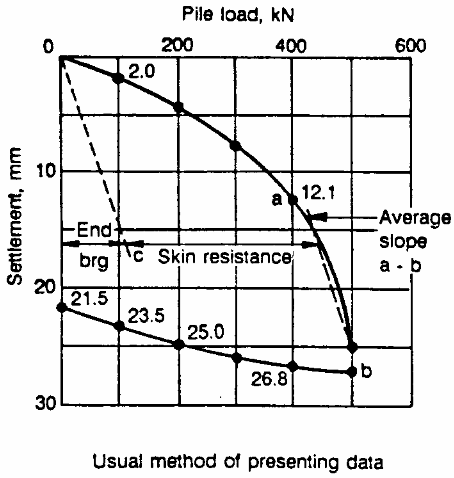


Figure 2.2 - Typical Load v Settlement curve

2.1.6 Capacity of Piles

One method of predicting the ultimate pile capacity using dynamic formulas is presented in the Specifications and in the Construction Supervision manual. As described in Section 2.1.5, load tests may be carried out to determine the ultimate capacity of the pile.

In all cases however, it is necessary to be able to relate the predicted ultimate capacity to the design loads in the pile. The value of the maximum design load in any pile should be given on the drawings. The Supervising Engineer must choose an appropriate factor of safety to apply to the ultimate capacity and check if it is greater than the design load.

The choice of a factor of safety depends on the type of dynamic formula used and the significance of the structure. Temporary structures may be able to be constructed with lower factors of safety than permanent structures.

Factors of safety of 3 to 6 are quoted for the Danish formula used in the Specifications. Some design codes require minimum values of 2.5 or 3.0 for dynamic formulas and 2.0 where a sufficient number of load tests has been carried out.

2.2 CONCRETE PILES

2.2.1 General

Concrete is adaptable for a wide range of pile types. It can be used in precast form in driven piles or added to bored piles. Dense, well-compacted concrete can withstand fairly hard driving and it is resistant to attack by aggressive substances in the ground or in water. However, concrete in precast piles is liable to damage (possibly unseen) in hard driving conditions. Weak concrete in cast in-situ piles is liable to disintegrate when aggressive substances are present in the ground or water.

Another disadvantage of concrete piles is that they are more difficult to splice than steel piles. On most projects, the required length of piles is not known until actual driving commences. Piles which are required to be lengthened usually cannot be completed until a new extension piece is cast and cured (at least 20 days) and the pile redriven.

2.2.2 Manufacture

Piles may be cast on beds using removable side forms of timber or steel. The type of bed and choice of material for the side forms will depend on the number of piles to be cast. The casting bed must be sited on firm ground to prevent bending of the piles during and after casting. A mass concrete bed is often used. This arrangement is shown in Figure 2.3.

The stop ends must be set truly square with the pile axis to ensure an even distribution of the hammer blow during driving. Vibrators are used to obtain thorough compaction of the concrete and the concrete between the upper steel bearers and the concrete should be worked with a 'slicing' tool to eliminate honeycombed patches.

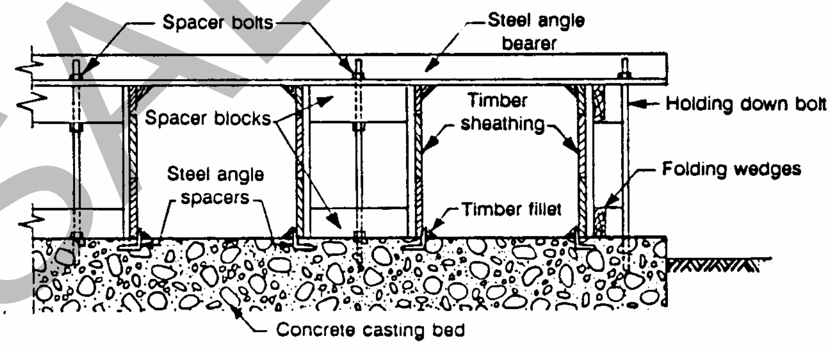


Figure 2.3 - Casting arrangement for concrete pile

When piles are cast with timber side forms the forms should be removed as soon as possible and wet curing using a water spray and hessian maintained for a seven day period. As soon as crushing tests on cubes show that the piles are strong enough to be lifted, they should be slightly canted by careful levering with a bar and packing with wedges to release the suction between the pile and the bed. The lifting slings or bolt inserts may then be fixed and the pile lifted for transport to the stacking area. The operation of first canting and lifting must be done with great care because the piles will still have only low strength and any cracks or incipient cracks formed at this stage will open under driving stresses.

The piles should be clearly marked with a reference number, length and date of casting at or before the time of lifting, to ensure that they are driven in the correct sequence.

They should be protected from the sun by covering the stack of piles with a tarpaulin or other sheeting.

2.2.3 Precast Prestressed Concrete Piles

Precast prestressed concrete piles are often used on bridge projects. The piles are normally pretensioned with an induced compressive stress at release of between 4 and 11 Mpa (40 - 110 kg/cm^2).

Standard lengths of these piles are from 6 metres to 20 metres in diameters up to 600mm. Splicing of these piles is carried out by means of a steel band plate at the end of the section to be spliced.

2.2.4 Handling and Storage

Reinforced concrete piles should be lifted or moved from the horizontal position by lifting at two points clearly marked in advance one-fifth of their length (or such other positions as may be shown on the Drawings) from each end. They must be carefully handled without shock or sudden movement. Special instructions may be supplied for the handling and stacking of unusually long (in excess of 15 metres) piles.

Piles should not be stacked more than three layers high and care is needed to ensure that settlement of the stack cannot take place especially during prolonged wet weather. The piles should be separated from each other by bearers placed truly vertical above those below.

The piles should be supported under the lifting points for transport and if piles are to be stacked one on another for transport (or storage) it is essential that the timber packers be placed vertically one directly above the other. This will eliminate tension cracking in the lower pile(s) due to bending, as shown in Figure 2.4.

Piles of different lengths should not be stacked together. Reinforced concrete piles must also be moved or stacked with the two opposite faces vertical or horizontal.Jinkers, mobile cranes or other suitable equipment are used for moving piles from a stack to the bridge site for pitching and driving, using two point support in transit as set out above.

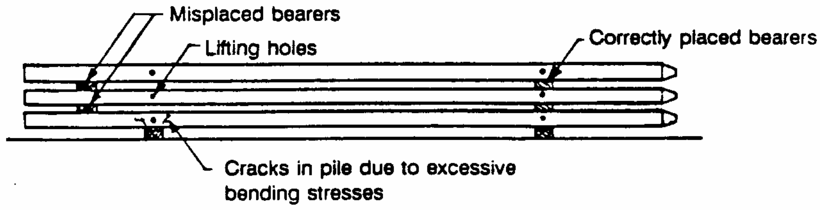


Figure 2.4 - Stacking of Concrete Piles

Before lifting, concrete piles are marked at 250 mm intervals. Short piles, up to 15 m in length, are lifted into position from the upper third point only, with the toe resting on the ground, care being required to ensure that the toe of the pile is not allowed to bump over obstructions or uneven ground. Long piles are lifted from both fifth points, unless otherwise instructed. The toe end is kept clear of the ground and gradually lowered until the pile approaches the vertical. A tail rope is used to avoid sudden collision with the pile frame or leaders. Final lifting should be carried out by means of a sling around the pile near the head. For very long piles, a suitably designed bridle may be necessary. In this case drawings will often be supplied by the Engineer.

2.2.5 Splicing and Extending

There are a number of different approaches to lengthening a concrete pile. Lengthening of a pile after driving is completed is the simplest because the splice will not be required to withstand the considerable stresses encountered during driving. Normal splice lengths for the reinforcement and the usual concrete practices will apply.

If the pile is to be driven further after splicing, the splice must be able to at least resist the compressive and torsional stresses imposed during driving and be capable of transmitting the moment in the pile across the joint. While a number of proprietary splices have been developed (for example the Hercules 'screw type' splice) these are not in common use in Indonesia. The most common method of splicing observed has been the use of a steel sleeve above and below the splice location. Some piles have steel plates embedded in the concrete to allow for easy splicing by welding the plates on the upper and lower segment of the pile. This is not common practice for piles fabricated on site. The advantage of the steel sleeve or the welded plates is that the pile can be driven within a short time of completing the splice. It is important that the two mating facings match each other as closely as possible in the same plane. The use of a steel sleeve and epoxy will compensate for lack of matching.

It is preferable (when using a steel sleeve) to insert and epoxy dowel bars into holes drilled in the upper and lower sections of the pile. This will enable a moment transfer to take place across the splice, as is usually assumed by the designer.

Figure 2.5, 2.6, 2.7, 2.8, 2.9 and 2.10 show different types of splices for concrete piles.

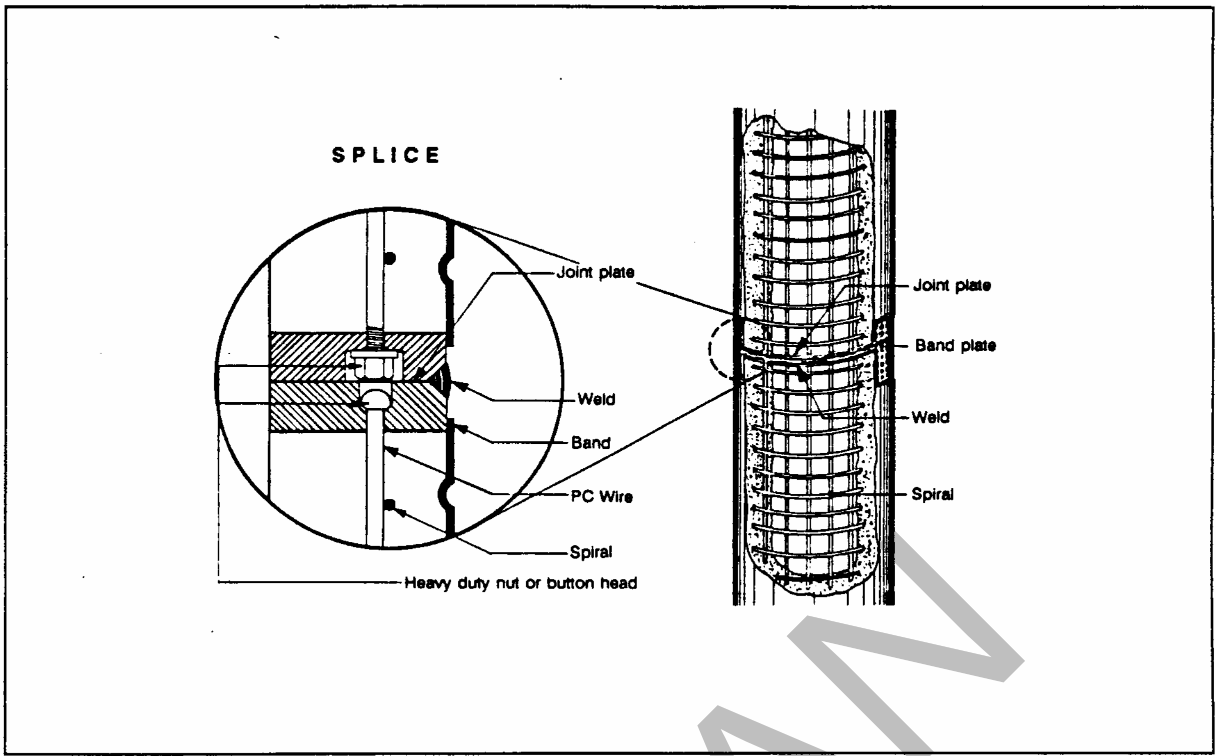


Figure 2.5 - Wika Pile Splice - typical detail

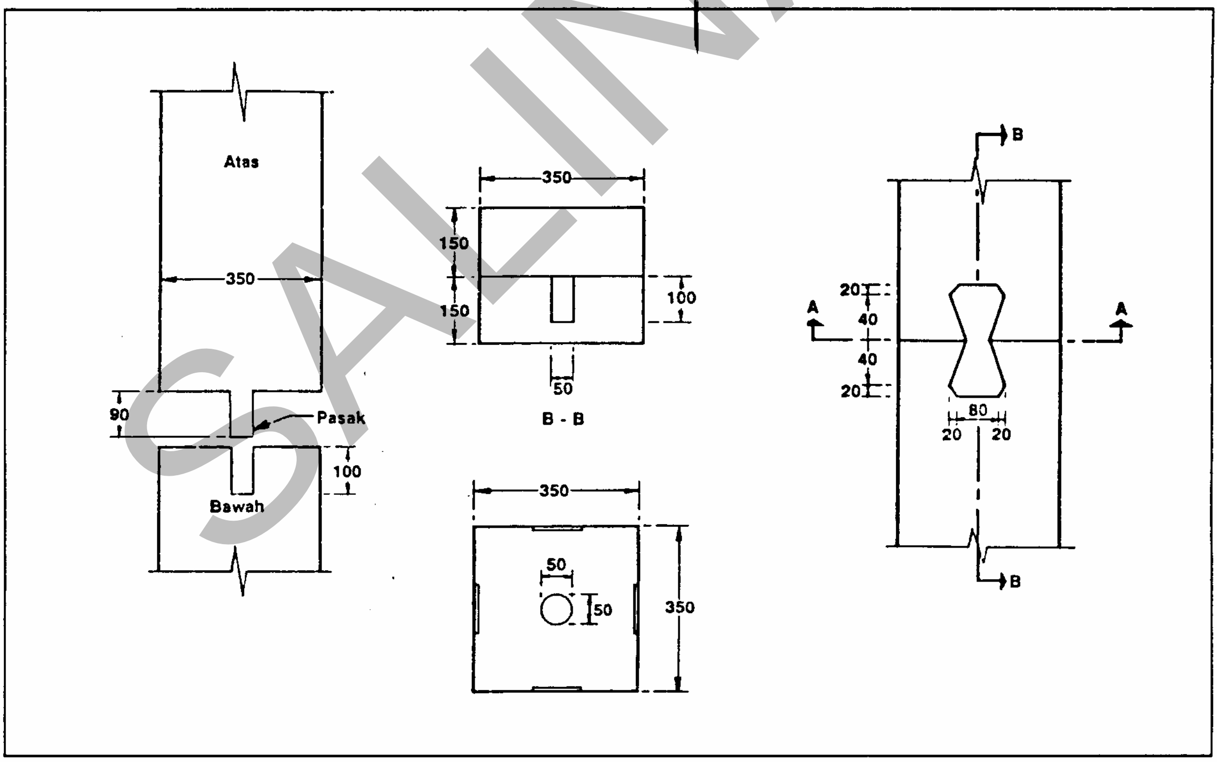


Figure 2.6 - JKS Pile Splice - typical detail

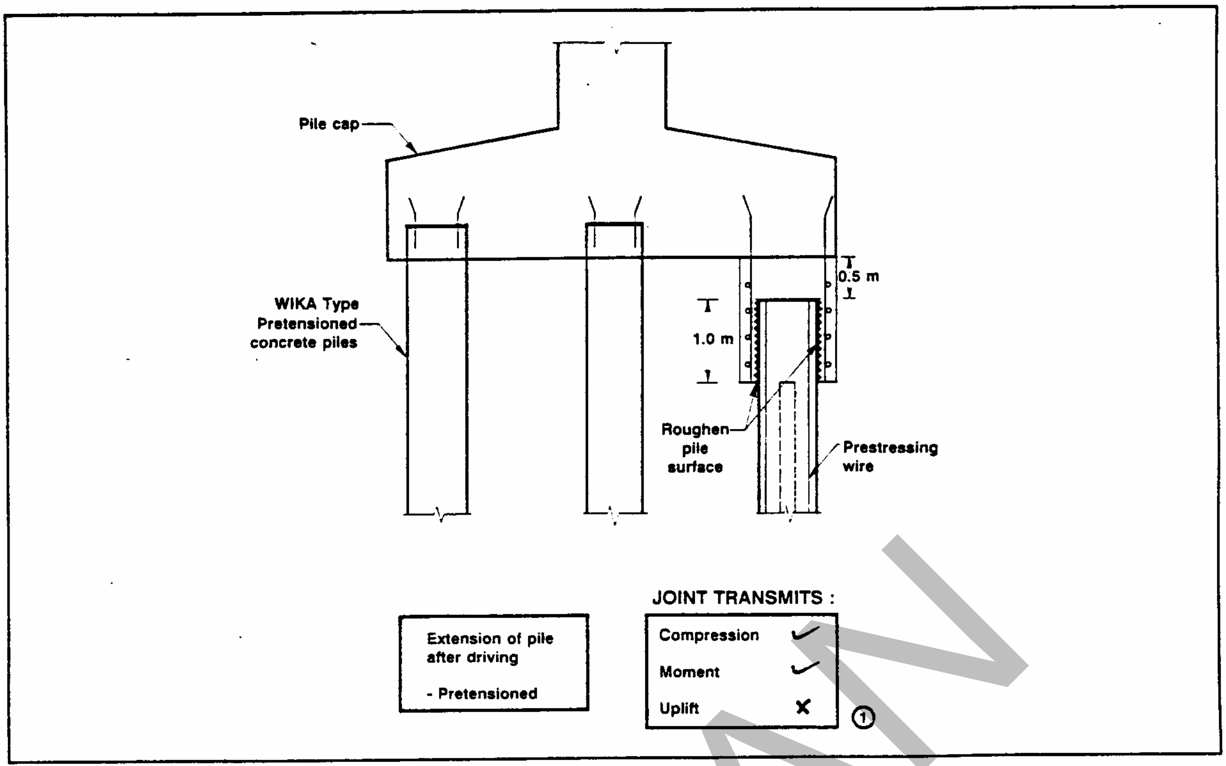


Figure 2.7 - Typical Concrete Pile Splice

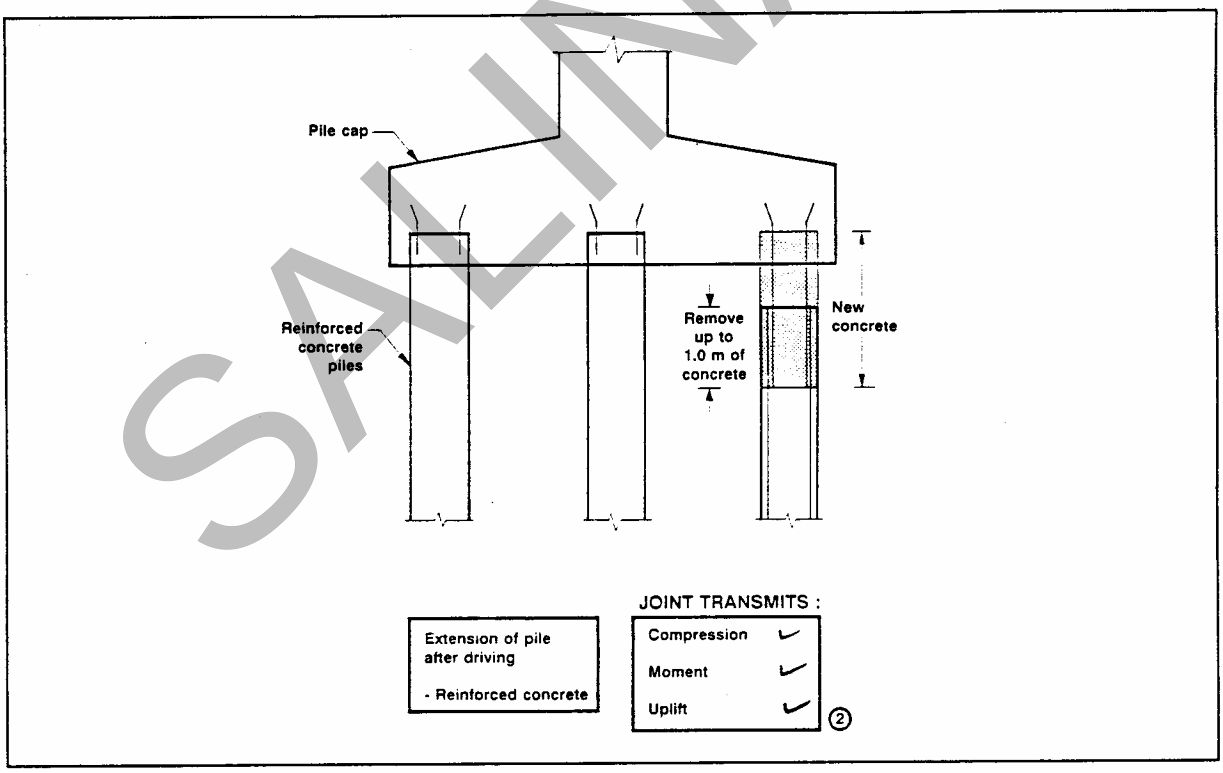


Figure 2.8 - Typical Concrete Pile Splice

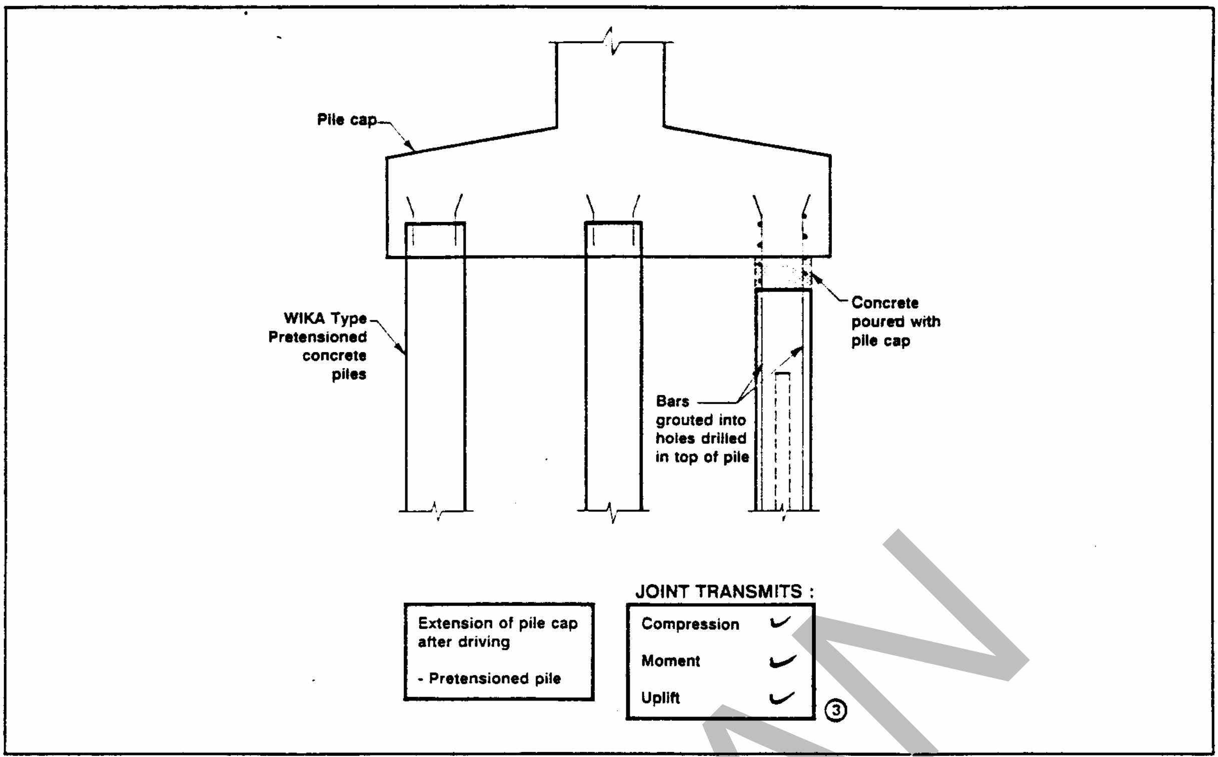


Figure 2.9 - Typical Concrete Pile Splice

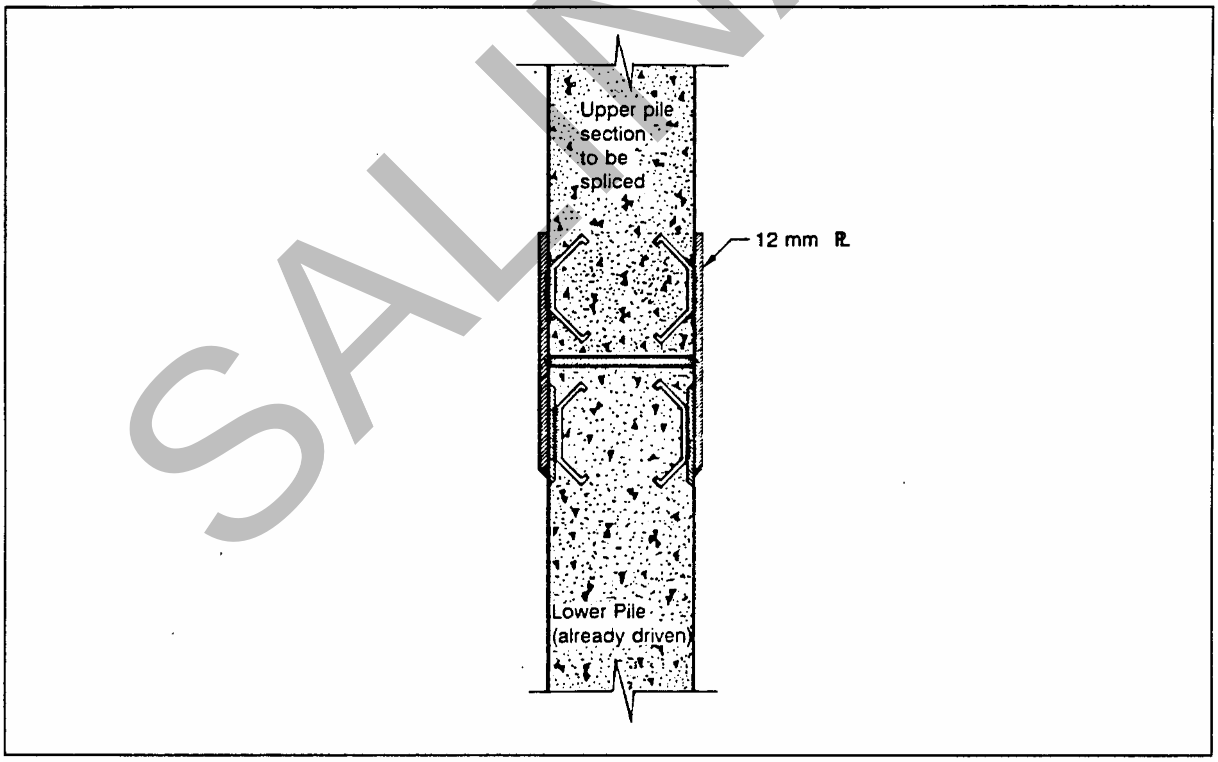


Figure 2.10 - Typical Concrete Pile Splice

Where splicing is to be carried out by breaking out concrete and splicing reinforcement from the upper to the lower section it is important that the area near the head of the pile segment which has been driven is carefully checked for damage. If significant cracking and spalling have occurred the concrete over the top 0.5 metres of the pile should be removed and the reinforcement cut back appropriately. In very hard driving the top 1.0 metre section may be found to be affected. The bars in each of the sections should be butt welded on the ends and a 12 or 16 mm diameter bar fillet welded to the upper and lower bars. The area between the piles is then formed up and concrete cast. The disadvantage of this system of splicing is that the upper section of the pile must be accurately and rigidly supported until the joint is sufficiently strong and that lengthened pile cannot be driven until the joint has the compressive strength required by the Specification.

The use of a single rod and recess for splicing is not recommended. These simple compression splices are unable to resist the tendency for one segment of the pile to wander off line if an obstruction is met and are unable to transmit any moment across the splice.

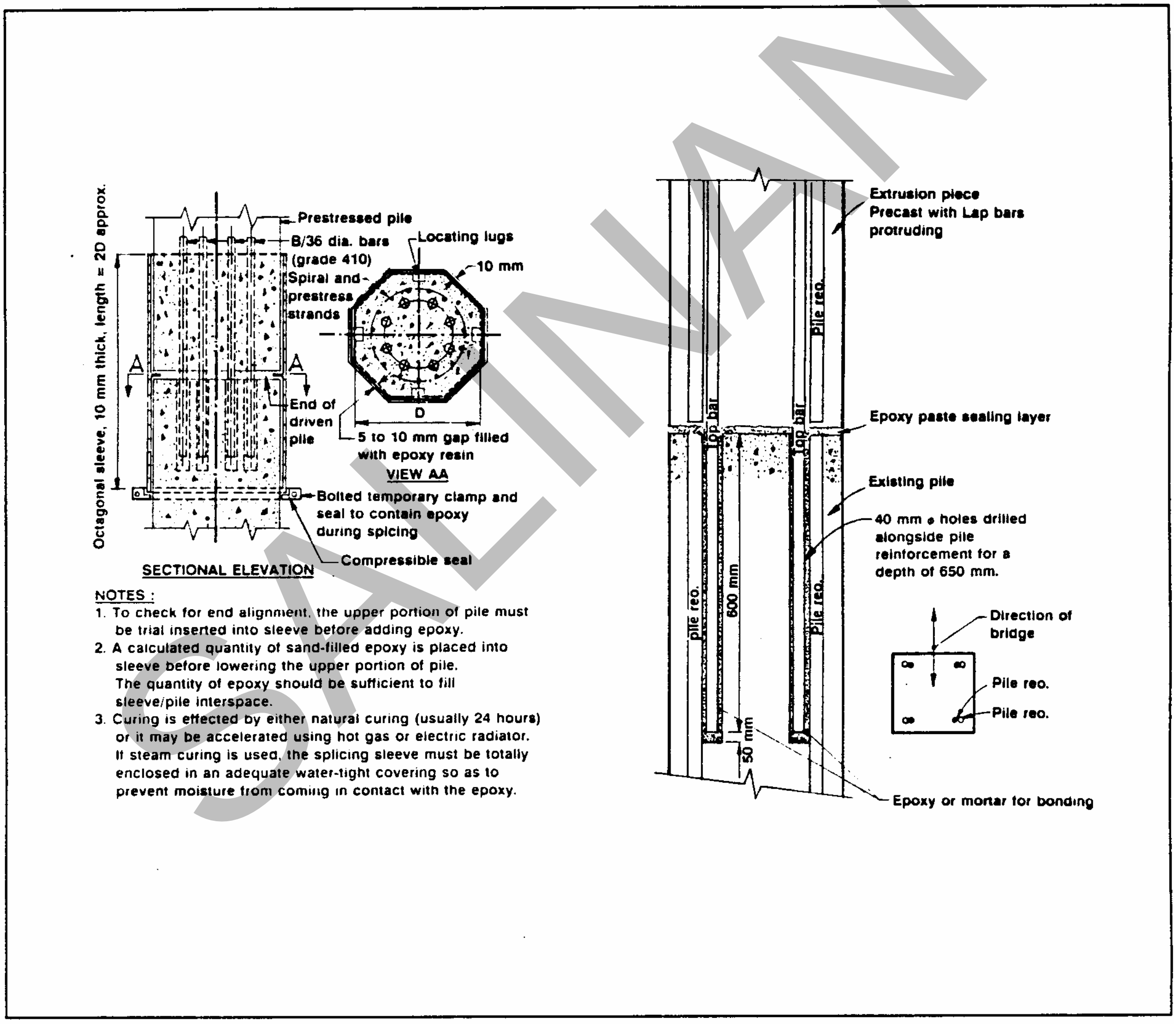


Figure 2.11 - Splicing Concrete Piles

2.3 STEEL PILES

2.3.1 General

Steel piles have the advantages of being robust, light to handle, capable of carrying high compressive loads when driven onto a hard stratum and capable of being driven hard to a deep penetration to reach a bearing stratum or to develop a high skin frictional resistance. Their cost per metre is usually higher than precast concrete piles. They can be readily cut down or extended to suit the variations in the bearing stratum level.

Hollow tubes may be driven open ended or closed ended. Piles which are required to carry high compressive loads are usually driven with a closed end. Open ended piles may have a reinforcing band added at the toe (either inside the shell or outside) when driving through resistant layers is expected.

Piles which are to be filled with concrete are installed with closed ends and concrete filling of steel tubes is carried out after driving has been completed. The steel tube is usually left in the ground as part of the permanent pile.

2.3.2 Fabrication

Steel tubes are usually supplied in built up form. Tubes may be either seamless, spirally welded or lap welded. Where necessary, lengths can be joined prior to driving and additional lengths can be easily spliced on as required.

To fabricate a closed ended pile the pile tube is often cut and welded to form a tip, as shown in Figure 2.12.

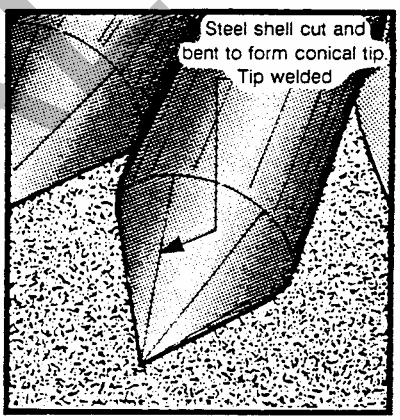


Figure 2.12 - Fabrication of closed-end pile

2.3.3 Splicing

Splicing of steel pile sections demands welding of a high standard and should only be undertaken by properly qualified welders.

The welds should be subject to visual and non destructive testing.

It is usually necessary to cut 300 mm to 500 mm off the top of the pile section driven, both to square up the end and to remove an area of work hardened steel which makes welding difficult.

Steel piles present no major problems in splicing. Welding is all that is required and the time taken to weld on an extension does not constitute a significant interruption to driving. However it is important to hold the extension in its correct alignment during welding and to maintain the correct gap. Suitable jigs and fixtures should be used to achieve this. Pieces of reinforcement bar (about 200 mm long) welded as lugs inside the tube will serve to locate the extension and the previously driven pile. The welded splice should be capable of transmitting the full moment in the pile and (for steel tubes) should normally be a full penetration butt weld around the periphery of the tube.

Two examples of typical splices for steel piles are shown in Figure 2.13.

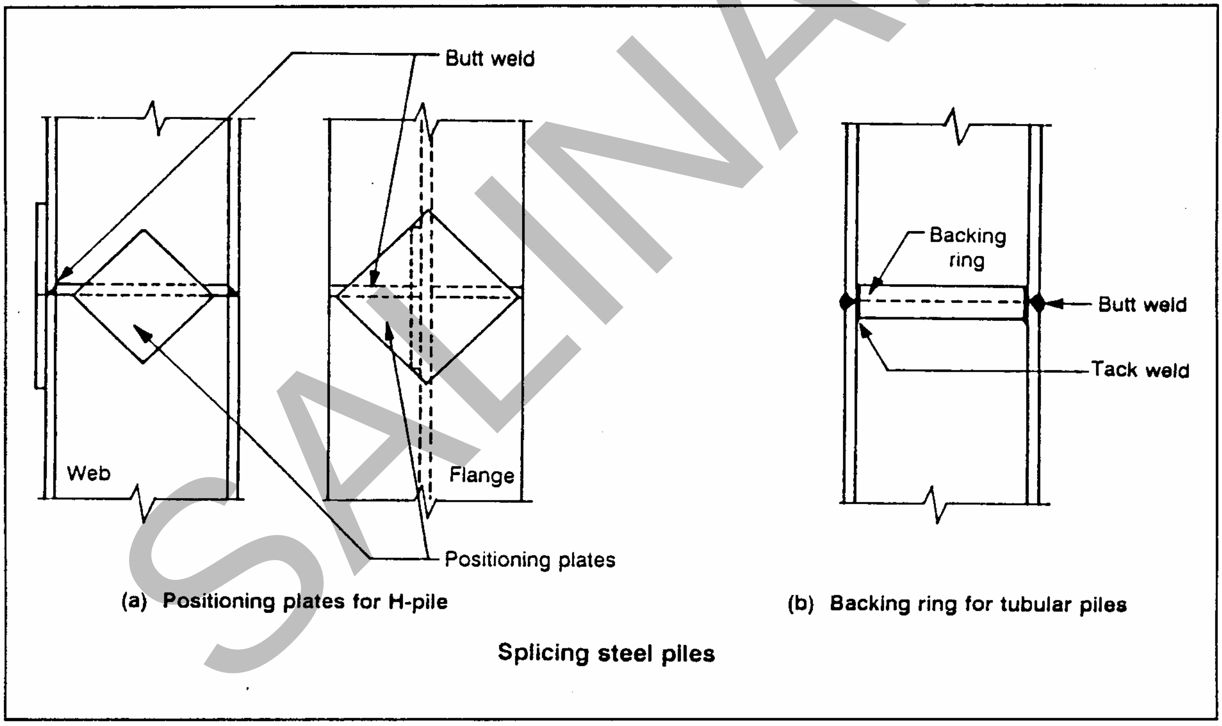


Figure 2.13 Typical Steel Pile Splices

2.3.4 Concrete in Piles

The majority of piling works on bridge projects are steel tubes driven into the ground and subsequently filled with concrete. A reinforcing cage is placed in the tube prior to concreting. The bars extend above the cutoff level of the pile and serve to tie the pile into the abutment or pier cap.

It is often not practical to compact by vibration concrete in the lower part of cast in place piles. The concrete in the top 2 or 3 metres should however be compacted using normal methods of vibration.

The reinforcing should be positioned centrally in the tube with the specified cover. This may be achieved by attaching suitable spacers to the outside of the reinforcing cage. Note that these spacers may tend to rotate as the cage is lowered into the pile. Spacers should be attached every 90° around the circumference of the reinforcing cage and should be spaced about every 2 or 2.5 metres longitudinally down the pile.

2.4 DRIVEN PILES

2.4.1 General

Both mobile and fixed plant for pile driving are in common use in Indonesia. By far the most common type of hammer is the diesel hammer. Drop hammers are seldom used for installing the permanent piles on bridge projects.

Abutment piles are often driven in advance of placing the fill for the approach embankment. This is very poor practice and can lead to a significant reduction in the bearing capacity of the pile.

Assuming that the embankment material can be properly compacted around the piles in the group (and this is extremely difficult to achieve, especially in the area under the abutment itself) the embankment material will settle around the piles. This settlement will cause a downdrag on the piles, additional to the vertical loads in the piles due to the loads imposed by the structure. Every effort should be made to ensure that the embankment material is placed as early as possible to avoid this problem. If a large depth of fill is to be placed at the abutment, consideration should be given to preloading the embankment with an additional height of material (2 or 3 metres would probably be sufficient). This material is then left in place for a period of several months to accelerate the settlement of the embankment and the additional material is then removed just prior to driving the piles.

2.4.2 Pitching

Other notes on handling and pitching of piles are contained in Section 2.1 and 2.2 of this Manual.

The Supervising Engineer should check the setout of the piles independently of the contractor.

It is a good idea to dig a shallow hole at the location of each pile to make positioning of the pile tip easier when pitching the pile.

When piles are to be driven on a steep rake the hammer will tend to overturn the pile frame during pitching as it will be high up on the leaders. In these cases the stability of the frame should be checked and, if necessary, weight added to the front of the frame (steel or concrete) or the frame chained down at the front.

If a system of hanging leaders is being used, that is the leaders are suspended from the hook of the crane, a set of guy wires and turfier type winches should be used to position the toe of the leaders correctly. The guy wires should be securely attached to suitable anchors. The ropes can be used to make minor corrections to the rake of the pile in the early stages of driving.

Concrete piles up to about 15 metres in length may be pitched using a single rope at the third point from the end.

After the pile is pitched and in position under the hammer the supervising engineer should check the pile for rake or verticality. This can best be done using a long spirit level and a plumb bob.

2.4.3 Driving Procedure

During the initial stages of driving of reinforced concrete piles, the blow of the hammer should be controlled to produce a penetration per blow not greater than 60 mm. Heavy blows in soft ground can produce tension cracking. As resistance increases, the blow should be increased keeping the penetration about 50 mm per blow until the maximum permitted blow is being used.

Driving is continued until the pile reaches the specified permissible set and until the toe has reached the designed level. Alternatively, as with test piles, driving is continued until the pile reaches nominal refusal. Under some specifications this is defined as a penetration of not more than 25 mm for 20 successive blows with the specified driving energy. To obtain firm seating on rock, refusal for steel piles is often accepted as a net penetration of 6 mm or less for the last 5 blows.

If a pile has been driven to within 1 metre of the ground and there is no indication that the required set will be achieved driving should cease and enough of the pile left protruding above the ground to allow for easy splicing. If the pile appears to be pulling up then it may be possible to complete driving with the use of a dolly or follower.

Frequently there is a reason to temporarily stop driving (for example to splice on extensions). When driving resumes, the penetration per blow may be much less than when driving ceased. Pore pressure, which has kept the pile "lubricated", has had time to dissipate and re-driving may become difficult. Under no circumstances should the size of the blow be increased above the maximum permissible. After a sufficient number of blows, the pile will normally start moving again. With diesel hammers the time-delay in overcoming such resistance is minimal.

Every attempt should be made to avoid stoppages during driving. Measurement of "set" should be done at the end of driving, and not following driving after a prolonged rest period.

2.4.4 Driving

Some additional notes on pile driving operation are provided below:

- Raker piles being driven from leaders tend to drift to a flatter slope than the pitched rake. If this does occur it is better to anticipate the drift and adjust the pitch before driving. Under no circumstances should concrete piles be pulled in an attempt to correct the deviation from the design rake. The pile will need to be extracted and redriven if corrective action is warranted.

- The drawings will indicate the plan location of the raker pile at the underside of the pile cap. The point at which the raker piles enter the ground is dependent on the difference in level between the underside of the pile cap and the ground surface, and the rake of the pile (see Figure 2.14).

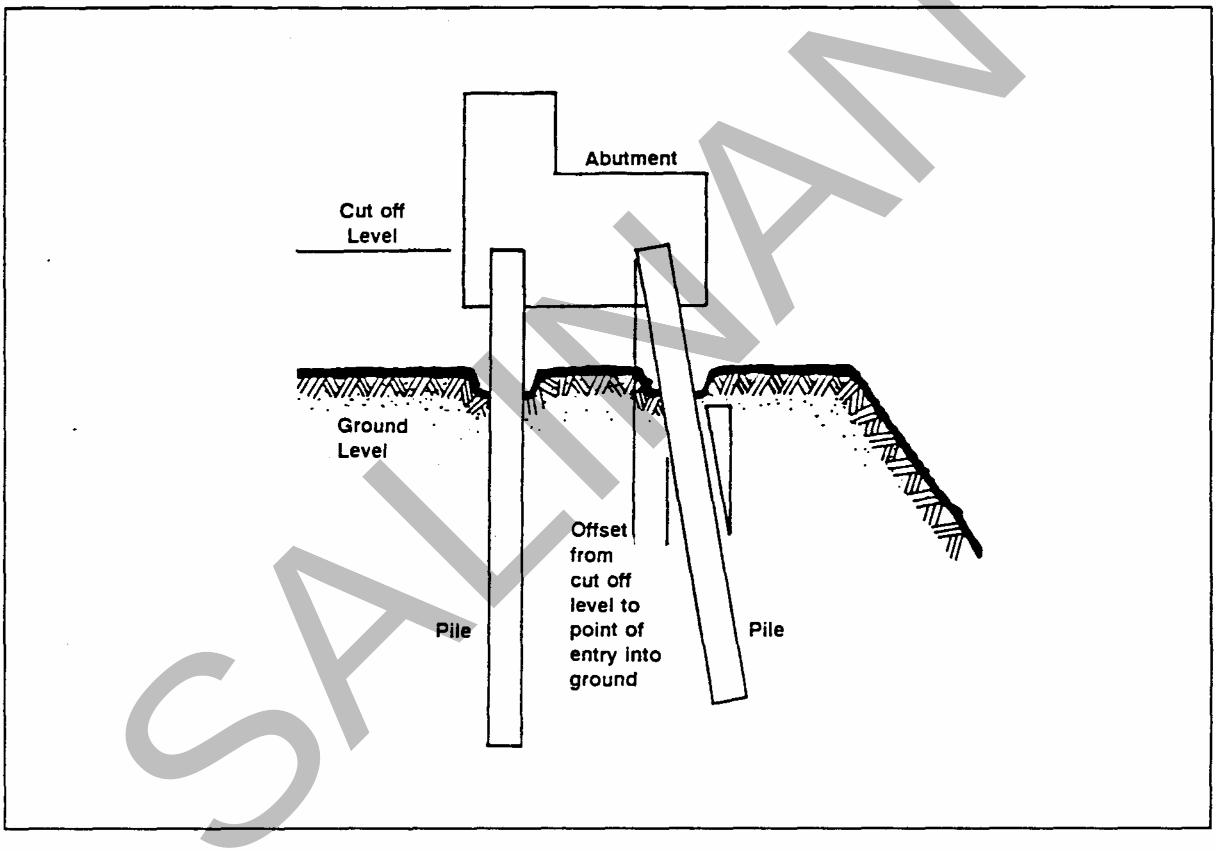


Figure 2.14 - Adjustment of pile level for raking pile

On dry ground it is usual to prepare a flat working area at the pier location. The point of entry of each pile can then be readily pegged. Similarly, for buried pile caps, it is desirable to excavate to the underside of the pile cap before commencement of driving. However, if this is not possible and the ground remains uneven, the points of entry can be pegged after determining the natural surface levels.

In driving raker piles from a barge, the location can first be determined by using a theodolite to establish line, and (preferably) electronic distance measuring (EDM) equipment for its location. If EDM equipment is not available a steel measuring tape or band may be used, with suitable corrections for temperature, sag etc. to ensure the highest degree of accuracy possible.

For piles raked in the direction of the longitudinal centreline the measured length must be calculated for the level at which measuring is carried out. For piles raked transversely to the centreline, the position of the theodolite on the baseline must be calculated for a predetermined level on the pile. When driving from a barge, the pile is first positioned with its weight supported by the barge. When the pile toe is lowered to the river bed the barge will rise due to the transfer of the pile load from the barge. The pile position at the surface should be checked prior to commencing driving.

The rise or fall of the barge with the tide may also alter the rake of the pile in early stages of driving. Therefore until the pile is properly embedded the rake must be checked regularly and corrected by moving the barge. At the same time the position of the piles should be checked by measurement.

- Piles on steep sloping banks often drift out towards the stream. If this does occur, piles should be pitched slightly raked to compensate for this drift.

- To ensure accurate driving, every possible care before and during the first blow of the hammer should be taken. The centroid of the hammer must travel in a line concentric with the centre of the pitched pile at all times. It is essential therefore that the pile frame is maintained in good condition and that the hammer guides be as straight as possible throughout their length.

- If a pile drifts off line in the early stages of driving it may be corrected by moving the pile frame slightly. With concrete piles this procedure should not be carried out after the first few blows of the hammer as they are easily cracked in this position. Some correction may be possible by the use of guys or slings in the appropriate direction but if the pile is significantly off line the usual procedure is to extract it, fill the hole with sand or similar material (compacting the material in layers) and attempt to drive it again at the correct angle. There is usually a tendency of the pile to follow the old hole.

- If the above method is not satisfactory or the pile is unable to be extracted (no equipment available for extraction or the pile has been driven too deep to allow extraction) usual practice is to drive another pile alongside the first and to extend the pile cap.

- In certain materials, usually when they are coarse grained, pervious and saturated, the pile will appear to pull up with the required resistance. After a short period of time (within 24 hours) however, the pile will lose up to 40 percent of its resistance to driving. This is caused by the material around the pile being compacted during driving, but on ceasing driving the material has a chance to absorb water and readjust itself. If this happens the pile must be redriven. It is therefore good practice when conditions such as this are suspected to cease driving and wait for 12 or 18 hours before retesting. If the pile does not move with the first few blows of the hammer it can be assumed that the above situation did not occur.

- If any doubt exists as to the load carrying ability of a pile the supervising engineer should call for a load test. This is especially important where no test piling has been carried out, either during the design phase or before installation of the permanent piles commenced. (see Section 2.1.5)

- In certain situations, the pile will be found to rebound a considerable distance, up to 300 mm or more. This can be caused by driving through an aquifer. In this situation the method of pile driving must be altered. One or two blows on the pile must be followed by a period of waiting for the pore pressure to dissipate, when another one or two blows can be given. This process must be continued until the toe of the pile has penetrated the lower limit of the aquifer.

- When driving concrete piles, the blow of the hammer during the early stages of driving should be such that the penetration per blow is not more than 50 mm. Heavy blows in soft ground will produce tension cracking. This can sometimes be recognised by puffs of dust about one third of the pile length from the top of the pile.

- Use suitable and adequate packing in the helmet and examine the packing before driving each pile. Replace the packing if necessary.

- If there are any signs of cracking of the pile, stop driving. If any doubt exists as to the severity of the cracking, it can be tested by a bucket of water thrown over the crack or cracks, while a few blows are applied to the pile. If water is forced out of the crack followed by a slurry this indicates that the crack is serious enough to warrant the cessation of driving.

- Observation of the pile, once it is pitched can be achieved by one of two methods. The simpler method is to sight past two plumb lines suspended nearby. This ensures that the pile remains vertical but does not check on lateral displacement. The more accurate method is to sight through two theodolites set up at right angles to each other. This will warn if the pile is moving from the vertical or if the pile is drifting laterally.

- Bad drifting of the pile may be corrected by jetting but in general it cannot be corrected without causing damage to the pile.

- Prestressed concrete and reinforced concrete piles in particular are difficult to drive through sand. The washing action of a jet of water from a pipe either embedded in the pile or forced down alongside the pile, eases the piles entry into the ground. However, jetting is ineffective in clay and clayey soils.

  Jetting can only be used where it is permissible to have large volume of water on the surface and it generally leads to a very "messy" site.

  The jetting must be even to prevent the toe running off line. Use a jet pipe on each side instead of a single line. The jet pipe should be 37.5 mm to 50 mm diameter terminating in a nozzle or fishtail of slightly smaller cross section.

  If pre-drilling jetting is used, ensure that the pile point is well seated with reasonable soil resistance at the point before using full driving energy.

  Do not drive and jet simultaneously.

- Safe working should be observed at all time during piling operations. In particular the following should be observed :

  a. Wear a safety helmet at all time during pilling operations. This is most important for the person recording the penetration and final set.

  b. Ensure properly spliced eye and thimble or a wedge socket for attaching the pile rope to the trip or hammer. **DO NOT USE ROPE CLIPS**. Wire the shackle pin so that it will not unscrew.

  c. Ensure that access ladders and platforms are in good order (condition)

  d. When hanging the hammer off in the leaders ensure that it is adequately supported by a beam or a wire rope sling or any other method set down by the manufacturer, even though the winch rope may still be attached to the hammer.

  e. When pitching piles remember that it is a heavy and rather awkward lift, and precautions must be taken to ensure the safety of the men at all times.

2.4.5 Test Piles

Test piles are piles which are driven before the permanent piles to find out how the permanent piles will behave during driving. Information from test piles can assist the designer in supplementing boring and sounding information. Test piles driven at the start of a contract can give the engineer and the contractor information on the required length of piles. This is especially important when concrete piles are to be cast on site. The test pile can determine the casting length of the rest of the piles and whether additional lengths for splicing will be required.

The test pile can often be the first of the permanent piles. The contractor will then be paid at the schedule rate for the pile driving and as an extra for any additional driving beyond the nominal design level and any splicing etc. which may be required.

If the test pile pulls up well above the predicted depth, considerable savings will be achieved in the manufacture and pitching of the remaining piles.

It the test pile goes well below the predicted depth, considerable time will be saved by eliminating the need to splice each pile as the piles can be cast in longer lengths (up to a maximum of about 15 metres as noted previously).

Obviously these savings are going to be achieved only if the test piles are correctly interpreted and the remaining piles behave in the same manner as the test pile.

Providing boring and sounding information is consistent across the site, Table 2.1 can be a guide to the interpretation of test piles. It should be remembered that for piles driven in groups the first will be the deepest and that the remainder will each pull up successively at shallower depths.

Table 2.1 - Interpretation of Test Piles

R.L. of Toe of Test Pile	**Distance from Toe to Contract R.L.**	**Recommended Action**
Well below Contract R.L.	More than 15 m	Reconsider boring information and type of foundation.
	5 to 15 m	Check boring information. If extra depth can be explained, adopt toe of test pile as new Contract R.L.
	1 to 5 m	Adopt toe of test pile as new Contract R.L.
Near Contract R.L.	1 m below to 1 m above	No change to Contract R.L.
	1 to 3 m above	Adopt toe of test pile as new Contract R.L.
Well above Contract R.L.	Minimal driving in sound material - 5 m	Adopt toe of test pile as new Contract R.L.
	Less than 5 m of driving in sound material	Check boring information. Reconsider type of foundation.

2.5 BORED PILES

2.5.1 General

Installation of bored piles requires specialised equipment and most contractors sub contract this work to a specialist piling contractor.

There are two main sources of problems with bored piles.

The first is installation in material which is not self supporting. This problem may be overcome by installing liners or by drilling under a drilling fluid such as bentonite. The former is more common in Indonesia.

The second is installation in material containing boulders. In this case the auger will not be able to penetrate the rock and some system of rock chisel will be required.

2.5.2 Boring

Various types of buckets are available for use with rotary augers. The standard type usually has scoop bladed openings fitted with projecting teeth. The rock bucket has a large opening designed to pick up rock broken by raising and lowering the chopping bit on the kelly. Often the drilling assembly is disconnected from the crane and a rocket shaped device with a heavy bit used to break the rock by simply dropping it against the surface of the rock. Alternatively a special auger with rock cutting teeth may be used. This latter is more expensive and requires a more powerful drive unit for the auger.

Although bored piles may be installed on a rake there is a problem caused by the auger dropping off line outside the end of the tube. This makes retraction of the auger quite difficult. Bored piles are usually installed vertically.

2.5.3 Excavation

Excavation of the bored pile is usually an integral part of the boring process as the auger drills into the ground. The soil is removed from a spiral plate auger by spinning it after withdrawal from the hole. Continuous flight augers will carry the soil from the tip of the auger to the ground surface without interruption to the boring process.

Excavation of rock is usually carried out by hand or using special attachments.

2.5.4 Enlarging of bases

Some bored pile systems allow for the base of the pile to be enlarged after boring has been completed. The enlarged base of the pile increases the bearing capacity of the pile on the founding material.

Enlargement may be carried out by hand digging or by the use of a special attachment on the boring machine which passes through the tube and expands in diameter as it digs below the level of the toe of the pile. It may then be retracted and extracted from the tube. The under-reamed base is usually filled with mass concrete of the same quality as the pile concrete.

2.5.5 Rock Socket

In hard formations, drilling using augers may not be feasible and special rock chisels may be required to achieve sufficient penetration. A rock socket provides high resistance to lateral forces and may be required in some instances (see Figure 2.15).

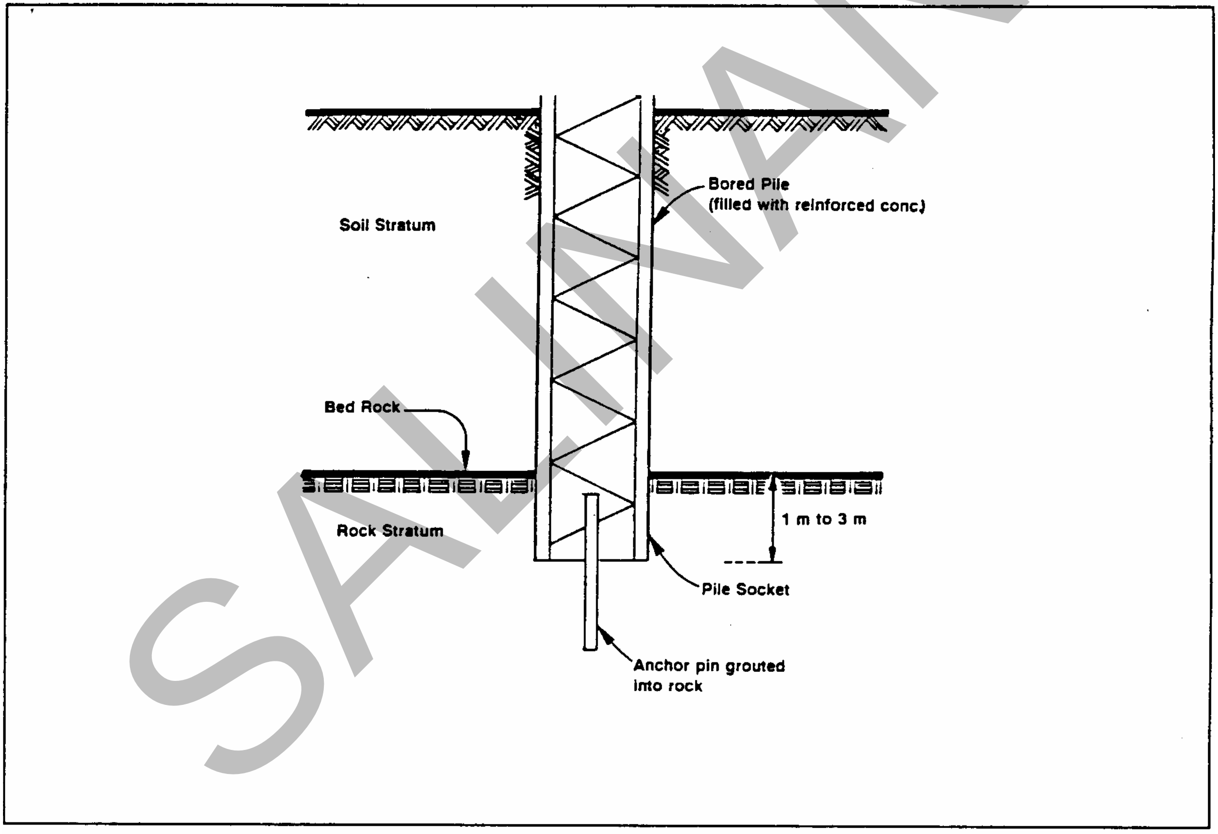


Figure 2.15 - Rock Socket for Bored Pile

3. CAISSON INSTALLATION

3.1 GENERAL

Notes on caissons and their installation are contained in Part II of the Construction Supervision Manual.

Most caissons in Indonesia are constructed of concrete cast in-situ in segments approximately 1.5 to 2.5 metres high, 2.5 metres outside diameter and sunk by progressively excavating beneath the base. The weight of the concrete in the caisson provides the vertical force required to overcome the friction between the side walls and the concrete and so sink the caisson.

3.2 CONCRETE - CAST IN SITU

3.2.1 Setting Out

Accurate setting out of a caisson is very important as the area occupied by a caisson is quite large. Consequently, errors in setting out, together with any tilting that occurs as the caisson is sunk, can result in the caisson finishing outside the abutment or pier section. This will mean additional work to enlarge the abutment or pier and will apply the vertical load from the superstructure into the substructure eccentrically.

The longitudinal centreline of the bridge and the transverse centreline of the caissons should be set out and offset a suitable distance to ensure that these reference points are not disturbed during construction of the caissons.

Care must be taken in setting out each segment to ensure that the new segment will have the correct alignment along the vertical axis. This is especially important when a segment is being added to a caisson which is off vertical. Ideally this tilt should be corrected prior to the addition of the next segment.

3.2.2 Preliminary Excavation

After the setting out work is complete preliminary excavation is carried out to provide the initial path along which the caisson will sink. The sides of this excavation should be as near as possible to vertical.

3.2.3 Casting of Concrete Segments

Sufficient concrete sections should be cast initially to allow the caisson to start sinking. Timbers should be placed radially under the cutting edge of the caisson and concrete segments added until the timbers start to lift near the centre of the caisson. A more complete description is contained in the Construction Supervision Manual.

The outside surface of the concrete should be made as smooth as possible.

See also Sections 4 and 5 of this Manual for further details on concrete construction.

3.2.4 Excavation

When a caisson is to be sunk on land, it is built in its correct location on suitable timber packing, which can be removed when sinking is to begin. The structure is constructed above ground until the supporting timbers (arranged radially) show signs of kicking up. The removal of the timbers is then effected in staggered sequence to avoid uneven settlement, care being necessary because the operation involves a certain amount of danger from possible sudden movement of the timbers. Excavation is then commenced by hand until the caisson is sinking evenly over its full area.

If the caisson is to be sunk through water, it must first be partly constructed, then launched and located over its correct position. Small caissons may be constructed on falsework at their sites, and lowered into the water by suspension tackle giving excellent control of the sinking procedure.

Excavation should proceed uniformly around the perimeter of the caisson. The Contractor should ensure that adequate provision has been made for removing the spoil from within the caisson. Suitable ramps and footways should be provided. It is preferable that the material be transported to a disposal area without double handling.

Excessive excavation under the edge of the caisson should be avoided. Only enough material to allow the caisson to sink is required.

The usual method of excavation is to use manual labour and a rope and bucket system to excavate the interior of the caisson. Pumping of water from the caisson is carried out using suitable pumps sited at the top of the caisson.

Verticality of the caisson must be watched constantly, particularly during the early stages - it is easier to keep it plumb the deeper it goes. If the caisson leans to one side the excavation should be concentrated on the high side until the axis is again plumb, but care must be taken to prevent the caissons from rolling from side to side because of over-correction of slight inclinations from the vertical. Correct alignment of the axis with each succeeding increase in height must be obtained by accurate setting-out.

Caissons are sometimes constructed by first excavating a hole to the required depth. The caisson is then formed on both sides and is used to contain the floor and cyclopean concrete. One problem associated with this approach is ensuring the stability of the sides of the excavation while men are working down in the hole.

If the hole is carefully excavated, so as to minimise overbreak, there should be no need to form the outside of the caisson and the concrete can be poured directly against the excavation. This method is considered preferable to forming the outside of the caisson and backfilling between the caisson and the hole.

A number of instances have been observed of contractors digging through hard rock for a caisson foundation. When such conditions are encountered consideration should be given to changing the foundation type. A spread footing, keyed into the rock with a shear key or by using dowel bars, will usually be considerably less expensive than a caisson where rock is encountered.

The Supervising Engineer should ensure that the base of the caisson is checked for level when excavation is completed and before the Contractor commences placing concrete.

3.2.5 Dewatering

Dewatering should be carried out using pumps in good condition. It is important that the point of discharge of the water be far enough away from the caisson to prevent erosion outside the caisson. A chute, pipe or preferably a flexible hose should be used to carry the water away. The Contractor should always have at least one pump in reserve in case of breakdowns.

3.2.6 Adding concrete segments

Additional segments are added when required, either to provide additional vertical load or when the caisson is nearly at existing ground level.

Reinforcement must be made continuous over the height of the caisson and sufficient length of bar left protruding out of the segment to allow the next bar to be properly spliced.

The construction joint between the segments should be formed so that there is no misalignment between the upper and lower surfaces. This will require a higher standard of formwork than is usually observed for caisson construction.

4. CONCRETE PRODUCTION

4.1 GENERAL

This Section and Section 5 cover the two main problem areas associated with the construction of bridges in Indonesia.

Better quality concrete work, particularly on superstructures, will lead to a significant reduction in the requirement for maintenance and repair of concrete in the early years of the life of the bridge.

This Section covers the production of concrete from raw materials using a suitable mix design and transporting the plastic concrete to the job site.

4.2 MIX DESIGN

4.2.1 General

Concrete mixes should be designed to give the most economical and practical combination of the available materials which will produce the necessary workability in the fresh concrete and the required properties of the hardened concrete.

The process of designing a concrete mix extends from the reading of the specification to the production of concrete of the required quality on the job.

All methods of mix design, although depending to some extent on theoretical considerations, are derived from empirical information. All mix designs follow essentially the same procedure no matter how complicated or different they may appear to be. Irrespective of the method used the first trial mix will usually require some modification.

There are a number of different methods of mix design. Most of the methods are similar and yield satisfactory concrete.

4.2.2 Design Method

a. General

This Section details one method for designing concrete mixes. A number of the terms used are defined below.

The *Characteristic Strength* of the various classes of concrete, in accordance with the Indonesian Concrete Code (PBI 71), is defined as being the strength below which only 5 percent of specimens fall for a minimum of 20 specimens tested.

The concrete mix is designed for a *target strength* in excess of the specified Characteristic Strength. The target strength is selected having regard to the degree of quality control which the Contractor can expect over the materials and handling of concrete in the field.

For water cured concrete the target strength shall not be less than T,

where $F'_c = T - 1.64\ s$

and F'_c is the specified Characteristic Strength at 28 days, and s is the standard deviation as defined below.

For other methods of curing the Contractor should submit the method of calculation of T.

The *mean strength* is the average compressive strength of a group of test results.

The *standard deviation* is a statistical measure of the spread or scatter of individual test results from the mean or average value. A number of compressive strength tests are performed during the project and the mean strength and standard deviation are calculated.

A suitable formula for the calculation of the standard deviation is :

$$s = \sqrt{\frac{\sum_{b=1}^{N}(\sigma_b - \sigma_{bm})^2}{N-1}} \quad \text{and} \tag{4.1}$$

$$\sigma_{bm} = \frac{\sum_{b=1}^{N}\sigma_b}{N}$$

where s = standard deviation
σ_b = Individual compressive strength test of concrete specimen
σ_{bm} = Mean of compressive strength test of concrete specimens
N = Number of test specimens

N should be greater than 10 for statistical accuracy.

This formula is taken from the Indonesian Concrete Code N.I. - 2 1971.

In the absence of previous test data an estimate must be made of the standard deviation.

For classes of concrete with Characteristic Strengths less than or equal to 35 MPa (350 kg/cm^2) the estimated standard deviation of the compressive strengths of the concrete produced should be not less than 3.5 MPa (35 kg/cm^2) nor greater than 7.5 MPa (75 kg/cm^2). For classes of concrete with Characteristic Strengths above 35 MPa (350 kg/cm^2) the estimated standard deviation of the compressive strengths of the concrete produced should be not less than 2.5 MPa (25 kg/cm^2) nor greater than 5.0 MPa (50 kg/cm^2).

The Contractor nominates the target strength for the Engineer's consent. The standard deviation is estimated for the concrete batch plant used and should allow for variability of materials, batching, mixing, sampling and delivery operations. The target strength nominated takes into account that the characteristic minimum compressive strength of concrete is based on the testing of samples taken at the point of use. Table 4.1 may be used as an initial guide in the determination of the estimated standard deviation.

Table 4.1 - Initial Estimate of Standard Deviation

Job	Standard of Control	Estimated Standard Deviation (MPa) [kg/cm²]		Margin by which target should exceed specified strength (Mpa) [kg/cm²]	
		F'c < 35 (MPa) (350 kg/cm²)	F'c > 35 (MPa) (350 kg/cm²)	F'c < 35 (MPa) (350 kg/cm²)	F'c > 35 (MPa) (350 kg/cm²)
Weigh batching of all materials, aggregate moisture and slump checks, uniform materials, very good methods of transport and placement and complete freedom from contamination of the concrete, constant supervision.	Excellent (automated control)	3.5 - 4.5 [35-45]	2.5 - 3.5 [25-35]	6.0 - 7.5 [60-75]	4.0 - 6.0 [40-60]
Weigh batching of all materials, slump checked, occasional changes in production and slump, good methods of transport and placing and regular supervision.	Very Good	4.5 - 5.5 [45-55]	3.5 - 5.0 [35-50]	7.5 - 9.0 [75-90]	6.0 - 8.0 [60-80]
Weigh batching of all materials or volume batching of aggregates plus allowance for moisture bulking, regular supervision of mixing and placing of concrete.	Fair	5.5 - 7.5 [55-75]	Not Applicable	9.0 - 12.0 [90-120]	Not Applicable

b. Design Procedure

i. General

The design method chosen for presentation in this manual is based on an English system. This system was chosen for its suitability to a wide range of aggregate types and ease of use.

Figure 4.1 is a form which can be used for mix design and follows the steps of the design method. It will be referred to in the next sections as the method is explained. Reference to this figure will be by the Item numbers which are shown on the left hand side of the Figure 4.1

The Characteristic Strength (Item 1.1) and the Standard Deviation (Item 1.2) are selected as discussed in Section 4.2.2.a above.

CONCRETE MIX DESIGN FORM

NO	ITEM	REFERENCE OR CALCULATION	VALUES
1.1	*Characteristic Strength*	*Specified*	______ MPa at ______ days Proportion Defective ______ percent
1.2	Standard Deviation	Table 4.1	______ MPa or no data ______ MPa
1.3	Margin	C1	(k = ______) ______ x ______ = ______ MPa
1.4	Target mean strength	C2	______ + ______ = ______ MPa
1.5	*Cement type*	*Specified*	OPC/SRPC/RHPC
1.6	Aggregate type:coarse Aggregate type:fine		______ ______
1.7	Free-water/cement ratio	Fig. 4.2	______ Use the lowest value
1.8	Water/cement ratio for durability	Table 4.3	______
1.9	*Maximum free-water/cement ratio*	*Specified*	______ ______
2.1	*Slump or V-B*	*Specified*	Slump ______ mm or V-B ______ s
2.2	*Maximum aggregate size*	*Specified*	______ mm
2.3	Free-water content	Fig. 4.3	______ kg/m³
3.1	Cement content	C3	______ / ______ = ______ kg/m³
3.2	*Maximum cement content*	*Specified*	______ kg/m³
3.3	*Minimum cement content*	*Specified*	______ kg/m³ Use if greater than item 3.1 and calculate item 3.4
3.4	Modified free water/cement ratio		______
4.1	Relative density of aggregate (SSD)		______ known/assumed
4.2	Concrete density	Fig. 4.4	______ kg/m³
4.3	Total aggregate content	C4	______ - ______ - ______ = ______ kg/m³
5.1	Grading of fine aggregate	BS 882	Zone (Fig. 4.5 or 4.6) ______
5.2	Proportion of fine aggregate	Figs. 4.7, 4.8 or 4.9	______ = ______ percent
5.3	Fine aggregate content		______ x ______ = ______ kg/m³
5.4	Coarse aggregate content	C5	______ x ______ = ______ kg/m³

Quantities (uncorrected for air or moisture in aggregates)	Cement (kg)	Water (kg or l)	Fine aggregate (kg)	Coarse aggregate (kg)
per m³ (to nearest 5 kg)	______	______	______	______

1) Figures in italic are optional limiting values that may be specified
2) OPC = Ordinary Portland Cement; SRPC = Sulphate Resisting Portland Cement; RHPC = Rapid Hardening Portland Cement
3) Relative density is specific gravity
4) SSD = based on a saturated surface-dry basis

Figure 4.1 - Concrete Mix Design Form

ITEM	DESCRIPTION	Cement (A)	Water (B)	Fine Aggregate (C)	Coarse Aggregate I (D)	Coarse Aggregate II (E)	TOTAL (F)
6.1	Weight Basic mix design (kg)						From bottom of Figure 4.1
6.2	Parts per part of cement	1					[6.1]/[A 6.1]
7.1	Relative Density		1.00				
7.2	Weight of materials per 40 kg bag of cement	40					[6.2] x 40
7.3	Volume of materials in litres						[6.1]/[7.1]
7.4	Air Content		%		Total volume including air		litres
7.5	Weight of materials for 1 m^3 concrete						$\frac{[6.1]\text{x}1000}{[F\ 7.3]\text{x}(1+[7.4])}$
8.1	Moisture Content (%)						
8.2	Absorption (%)						
8.3	Weight Oven Dry (kg)						$\frac{[7.5]}{(1+[8.2]/100)}$
8.4	Change in water (kg)						[8.3] x (1 + [8.1]/100) - [7.5]
8.5	Weight of materials corrected for moisture						[7.5] - [8.4]
9.1	Volume based on 8.5 above						[8.5] / [7.1]
9.2	Weight for 1 m^3 corrected for air content and moisture						[8.5] x $\frac{(1 - [7.4]/100)}{[F\ 9.1]}$
9.3	Volume of materials in litres						[9.2] / [7.1]
9.4	Weight of materials per 40 kg bag cement	40					[9.2] x 40/[A 9.2]
9.5	Volume / 40 kg bag cement	1 bag					[9.3] x 40/[A 9.3]
9.6	Weights for a trial mix of : m^3 (Z)						
9.6.1	kg						[9.2] x Z
9.6.2	litre						[9.6.1] / 7.1
	Note : [F 9.1] means the total of columns A to E in Row 9.1 [B 6.1] means the value in column B of row 6.1						

Figure 4.1 Concrete Mix Design Form (continued)

Note : The formulas in the TOTAL column on the right hand side of a row indicate how the entries for that row are calculated.

ii. Selection of Target Strength

The target strength (Item 1.4) may be calculated from the following:

Target strength = Characteristic strength + k x standard deviation

"k" is a statistical factor used to calculated the (usually in Bina Marga projects) 95 percent confidence limits necessary for the determination of characteristic strength. "k" is also dependent on sample size - values are shown in Table 4.2.

Where no tests are available for the grade of concrete an assumed value of standard deviation from Table 4.1 is used with "k" = 1.64.

Once test results are available from the laboratory for the particular grade of concrete, calculate the Standard Deviation and use that in the formula with the appropriate value of "k".

Table 4.2 Value of 'k' for Determination of Characteristic Strength

No. of Specimens in Test Sample	k
2	6.31
4	2.35
6	2.02
12	1.80
20	1.73
30 or more	1.64

iii. Selection of Water/Cement Ratio

The water/cement ratio is commonly expressed as a ratio by weight.

The selection of the water/cement ratio as a basis for designing a concrete mix involves consideration of the degree of exposure to which the concrete is to be subjected, whether it must be watertight, and the strength requirements of the structure. Because of the high strengths that are now obtained with portland cement, ample strength will be obtained if the requirements of exposure are properly met. For this reason the first step in designing a mix should be to select the water/cement ratio necessary to meet the degree of exposure. If the required strength is higher than can be expected from this water/cement ratio, then a ratio meeting this strength requirement should be chosen. The value to be used in the calculations will be the lowest of Items 1.7,1.8 and 1.9. The value for Item 1.9 is the specified maximum value of the water/cement ratio.

Water/cement ratio for durability and watertightness:

Table 4.3 gives recommended water/cement ratios (Item 1.8) on the basis of minimum curing for concrete to meet different degrees of exposure in different classes of structures.

The minimum curing, where portland cement is used, is the equivalent of seven days moist curing at a temperature of 20 °C.

Table 4.3 - Durability Requirements

Condition of Exposure	Maximum Water/Cement Ratio	
	Plain Concrete	Reinforced Concrete
a) Internal, subject to heavy condensation	-	0.60
b) Alternate wetting and drying	0.60	0.60
c) Sea water or salt spray	0.50	0.45
d) In water retaining structures	-	0.50

Water/cement ratios for strength

Where a water/cement ratio which gives adequate durability does not satisfy strength requirements, the water/cement ratio must be reduced to that which will produce the desired strength. Selection of the free water/cement ratio (Item 1.7) may be based on the data in Figure 4.2.

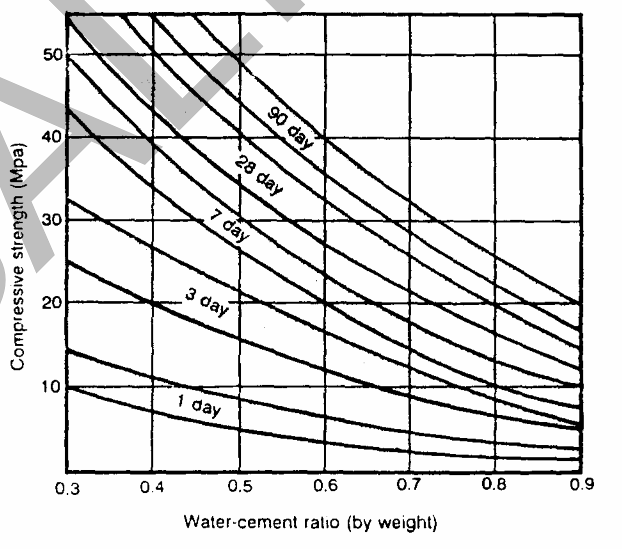


Figure 4.2 - Effect of Water/Cement Ratio on Compressive Strength

Thus, if a strength of 20 MPa is required, take an assumed value of Standard Deviation from Table 4.1 of 4.5 MPa and hence: Target strength = 20 + 1.64 x 4.5 = 27.4 MPa

From Figure 4.2 the water/cement ratio indicated for this strength at 28 days, using Type I cement, is 0.6 i.e. 24 kg of water per 40 kg bag of cement.

For maximum economy, tests for strength should be made with the actual materials to be used on the job, and under job conditions. A job curve similar to that shown in Figure 4.2 can be developed from such tests, and from this the water content appropriate to the required "target" strength can be selected.

iv. Consistency of Concrete

For a given volume of concrete the higher the water content the more fluid the mix - see Figure 4.3. Alternatively with a given amount of cement paste, more aggregate is used in stiff mixes than in fluid mixes. Consequently, stiff mixes are more economical in cost of materials than fluid mixes.

Stiff mixes make it more difficult to effectively compact the concrete and if the mix is too stiff, the cost of placing may offset any saving that is made in materials. Concrete mixes should always be of a consistency and workability suitable for the conditions of the job. Thus thin members and heavily reinforced members require more fluid mixes than large members containing little reinforcing.

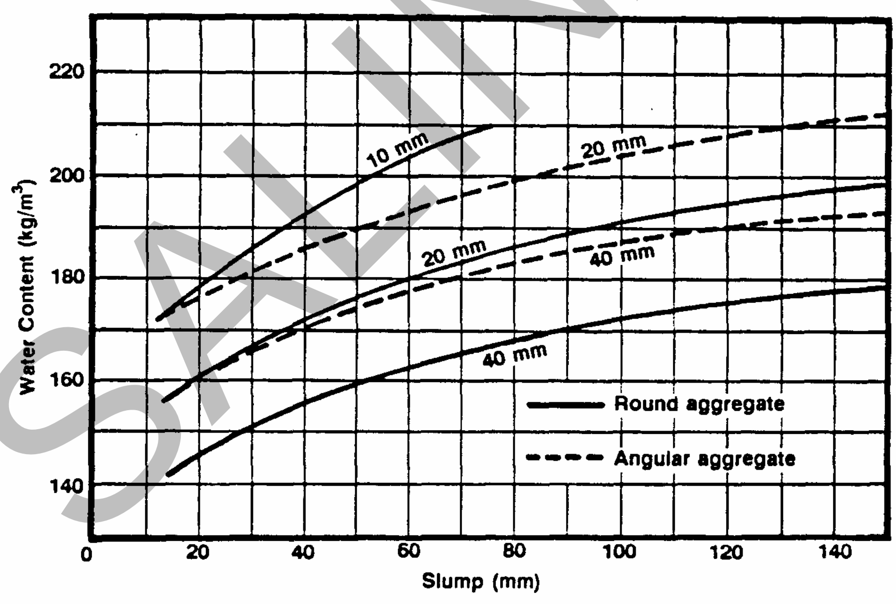


Figure 4.3 - Water Requirement

In describing the character of fresh concrete, three terms are often used *consistency, plasticity and workability*.

Consistency is a general term relating to the state of fluidity of the mix and embraces the entire range of fluidity from the driest to the wettest possible mixes. It requires a qualifying term for definition. The term **plasticity** is used to describe a consistency of concrete which can be readily moulded but which permits fresh concrete to change form or shape slowly if the mould is removed. A plastic mass does not crumble, but flows sluggishly without the segregation that occurs in wetter mixes. Thus, neither the very dry crumbly mixes nor the very fluid, watery mixes are regarded as being of plastic consistency. In this connection it should be pointed out that a low water/cement ratio does not necessarily mean a dry consistency.

Workability denotes the ease or difficulty of placing the concrete in a particular location. The conditions under which the concrete is to be placed - size and shape of member, spacing of reinforcing bars or other details interfering with the ready filling of forms - determine the degree of workability required.

It is obvious that a stiff plastic mix with large aggregate, which is workable in a large open form would not be workable in a thin wall with closely-spaced and complicated reinforcing.

An approximate measure of consistency is the *Slump Test*, which should be made in accordance with the appropriate standard test (for example AASHTO T119). This test is not an absolute measure of workability and it should not be used to compare mixes of wholly different proportions or of different kinds or sizes of aggregates. For mixes of the same design or with the same components, changes in consistency as indicated by the slump test are useful in indicating changes in the character of the materials, the proportions or the water content of the concrete.

To avoid mixes too stiff or too wet, slumps falling within the limits given in Table 4.4 are recommended. This will give the value(s) to be used in Item 2.1.

The slumps shown in Table 4.4 are for concrete with a maximum aggregate size of 20 mm. Equivalent workability can be obtained by lower slumps with smaller aggregate or higher slumps with larger aggregate. For Indonesian conditions, it is better to aim for a slump towards the upper limit due to the ambient temperature.

Given the slump range and the size and type of aggregate Figure 4.3 can be used to give an estimate of the free water content, Item 2.3 in Figure 4.1, the mix design form. This may in turn be used to calculate the cement content (Item 3.1). If this value is outside the range of the specified cement content (but note that the usual limit is the lower or minimum cement content) then the relevant specified limit should be used for Item 3.4.

Table 4.4 - Recommended Slump for Concrete - 20 mm maximum size Aggregate

Type of Construction	Recommended Slump (mm)	
	Minimum	Maximum
Heavy Mass Concrete	30	80
Plain Footings, caissons and substructure walls	50	80
Pavements and slabs	50	80
Beams	50	100
Reinforced Footings	50	100
Columns	50	100
Pumped Concrete	70	120
Reinforced thin walls	80	120
Tremie Concrete	120	200

v. Proportioning of Aggregates

The three essential ingredients of concrete are water, cement and aggregates. So far the ratio of water to cement has been fixed to give the required strength and durability. The next step in proportioning is to fix the exact quantity of each ingredient in a cubic metre of concrete.

The various methods of mix proportioning all take into account the required workability of the concrete, and the type and maximum size of aggregate being used. Workability is usually expressed in terms of the slump test, and reference to Table 4.4 gives an indication of how the required concrete slump might vary for various types of construction.

The mix designer must now refer to the appropriate design tables for the method of proportioning that he is using. Such tables indicate either the water content and the fine aggregate content or the aggregate/cement ratio that is necessary for a certain size and type of aggregate to produce concrete with any required slump.

Stage 1 of the mix design method determines the water/cement ratio, Stage 2 the free water content and Stage 3 the modified water/cement ratio. These are described in Section 4.2.2.b.iv above.

Stage 4 calculates the total aggregate content and Stage 5 completes the basic mix design process by calculating each of the fine and coarse aggregate proportions.

The relative density of the aggregates in a saturated and surface dry condition (see notes on Correction for Moisture in the example of mix design for a description of this term) is usually known from laboratory tests or can be estimated on the basis of past experience (Item 4.1).

The density of the wet compacted concrete can be estimated from Figure 4.4. The figure is entered with the relative density of the combined aggregate (on a saturated surface dry basis) and the free water content in kg/m^3. The wet density of fully compacted concrete is read off the left scale (Item 4.2). The total aggregate content (Item 4.3) is calculated from the density of concrete less the mass of water and cement in 1 cubic metre of concrete.

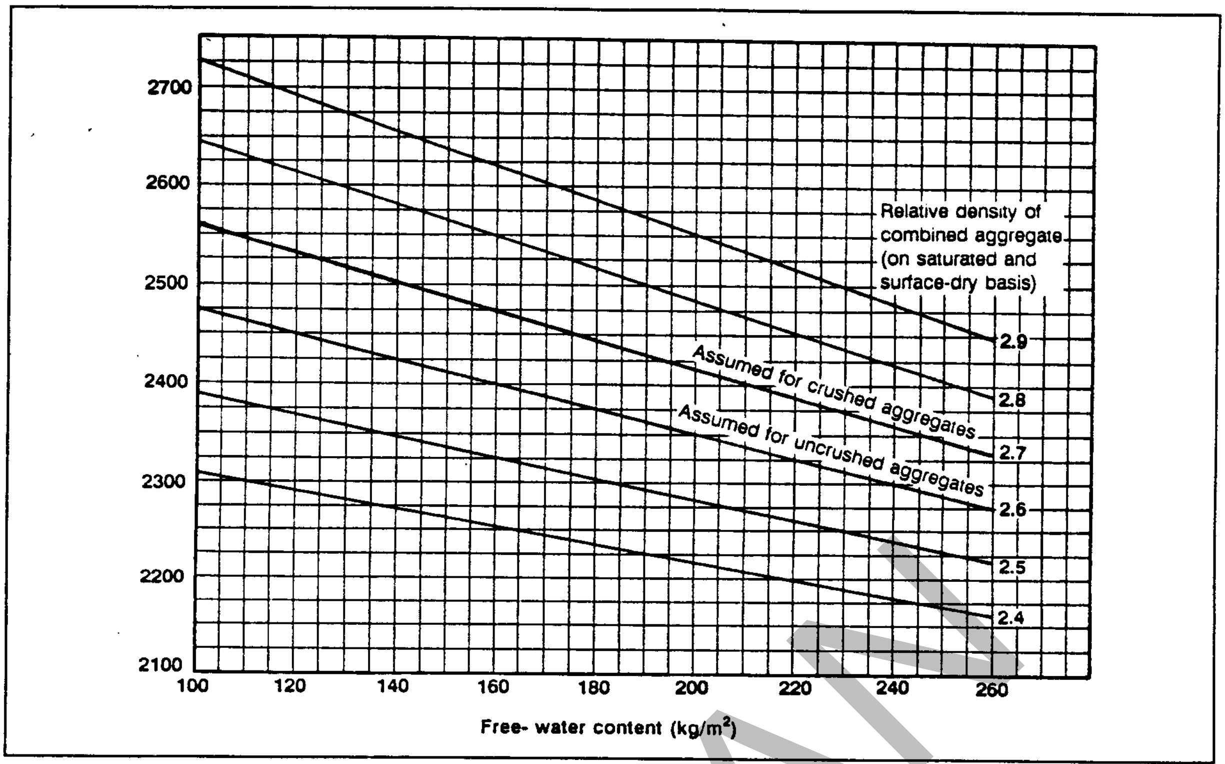


Figure 4.4 - Estimated Wet Density of Compacted Concrete

The proportions of coarse and fine aggregate are next calculated. The fine aggregate grading is compared with a number of standard gradings. Two (Zone 1 and 2) are shown in Figures 4.5 and 4.6, and used as the basis for reading the proportion of fine aggregate in the total aggregate (Item 5.2) as set out in Figures 4.7, 4.8 or 4.9. These three figures are for 10 mm, 20 mm and 40 mm nominal size aggregates respectively.

These graphs are plotted for a number of slump ranges and free water/cement ratios.

An average proportion is selected and this proportion (Item 5.2) used to calculate the weight of fine aggregate per cubic metre of concrete (Item 5.3). The balance of the aggregate is coarse aggregate (Item 5.4).

When two or more aggregates are available they are combined to give a grading which should approximate to one of those in Figures 4.10, 4.11 or 4.12.

Given the relative percentages of fine and coarse aggregates (Item 5.2) a combined grading may be calculated and compared with the grading curves of Figures 4.10, 4.11 or 4.12. If the grading falls too far outside the relevant curve the percentage of fine aggregate may need to be adjusted and a check on the mix design carried out.

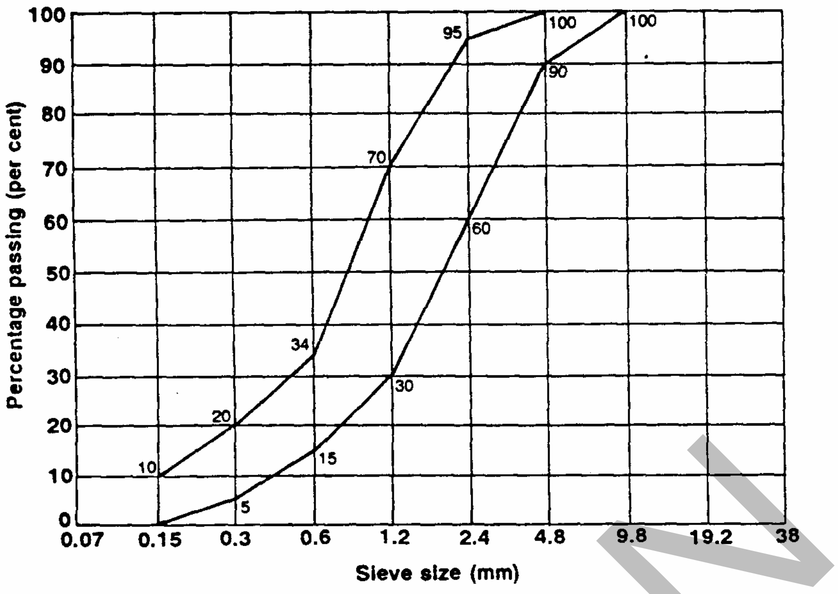


Figure 4.5 - Zone 1 - For Fine Aggregate

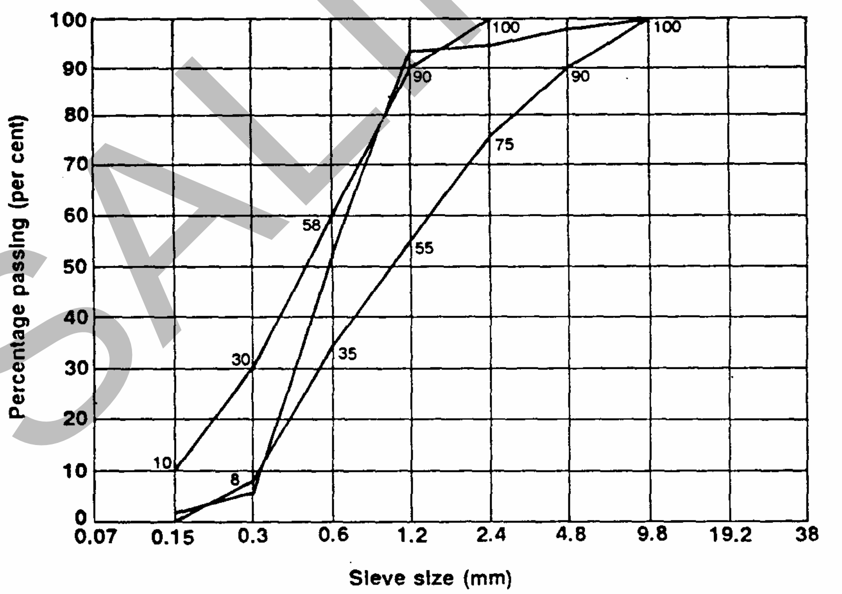


Figure 4.6 - Zone 2 - For Fine Aggregate

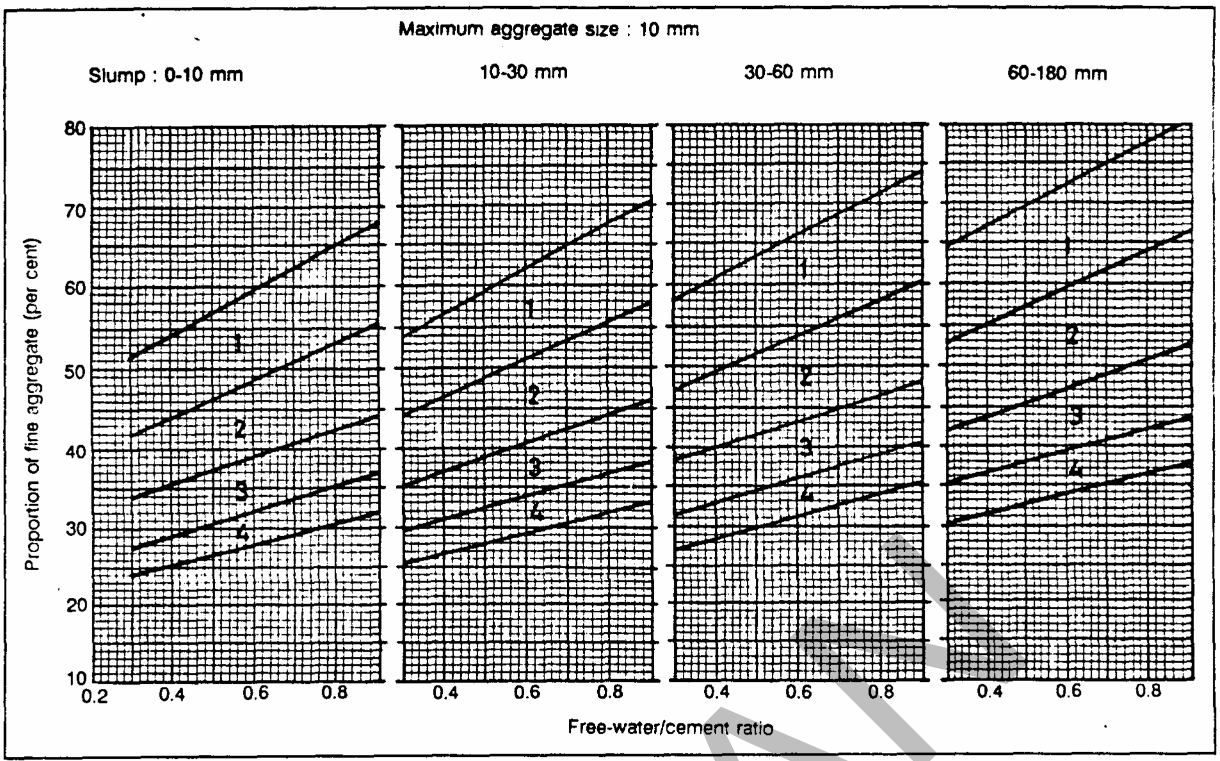


Figure 4.7 - Recommended fine aggregate proportions for 10 mm aggregate

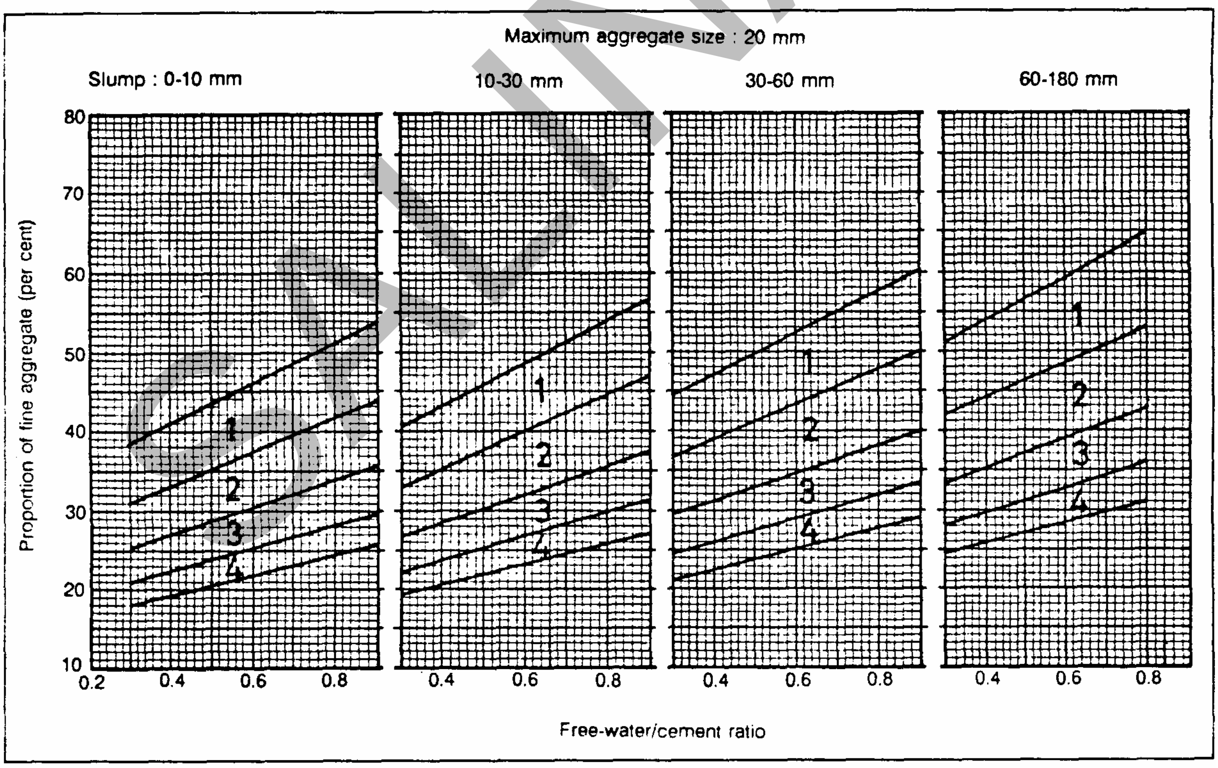


Figure 4.8 - Recommended fine aggregate proportions for 20 mm aggregate

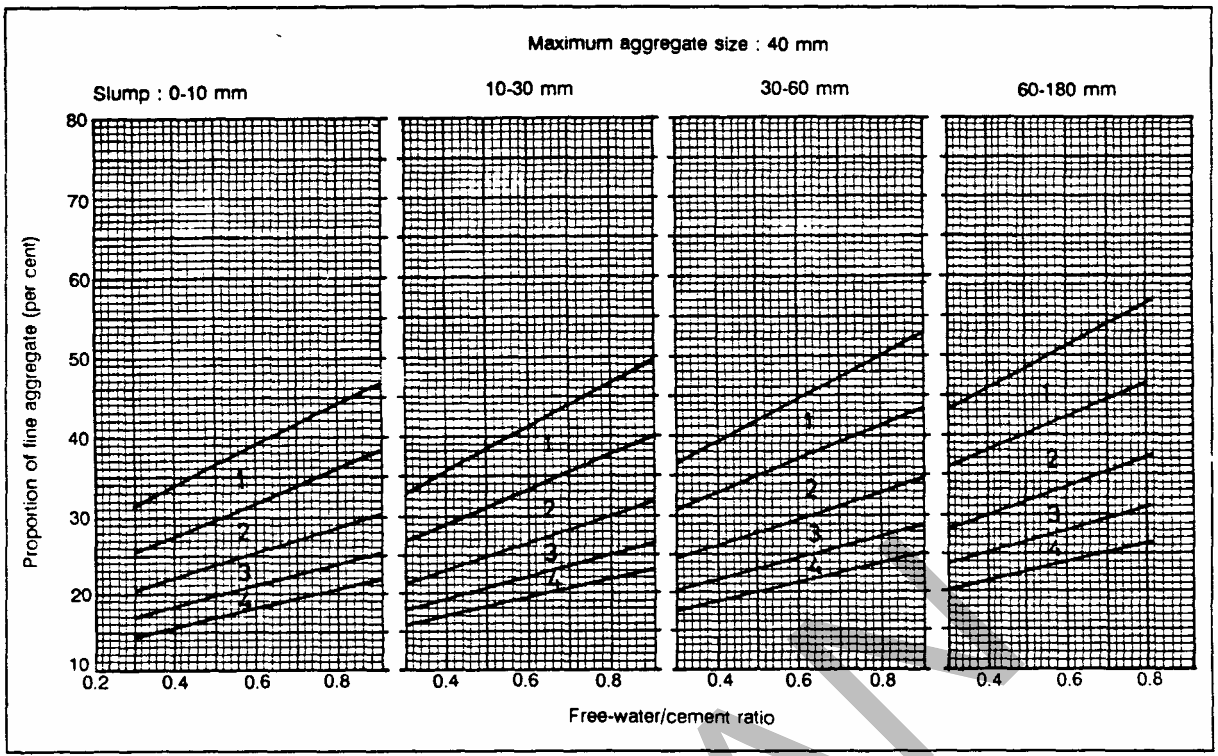


Figure 4.9 - Recommended fine aggregate proportions for 40 mm aggregate

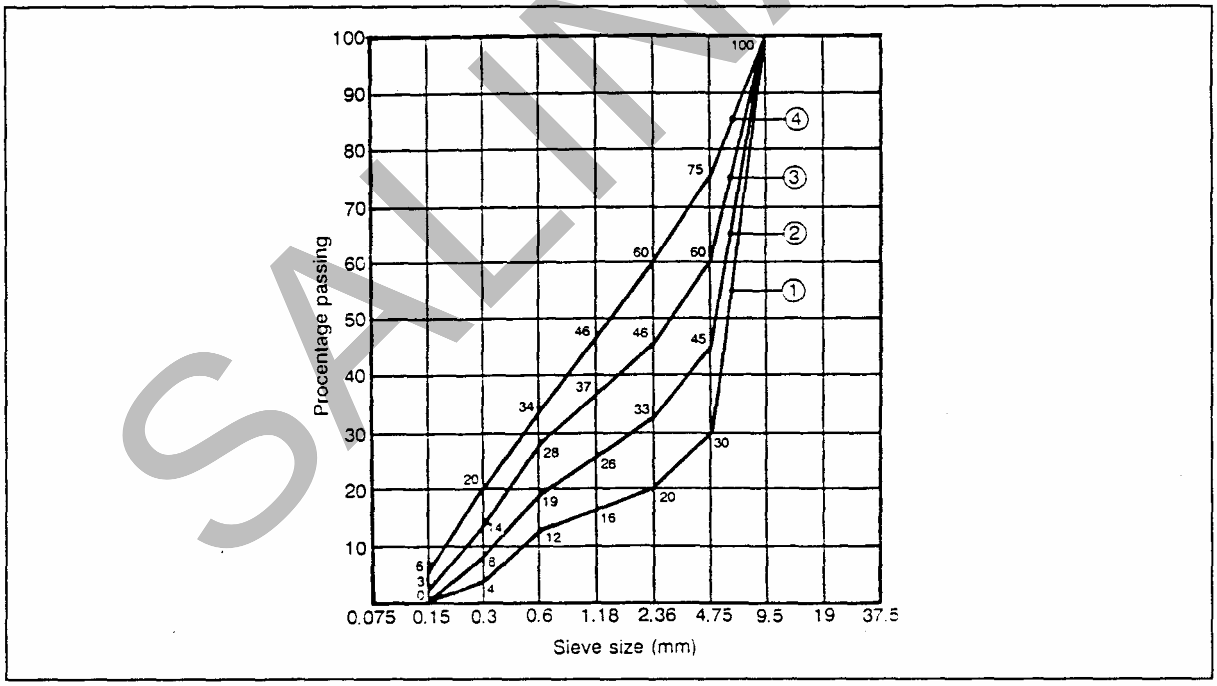


Figure 4.10 - Gradings for 10 mm Aggregate

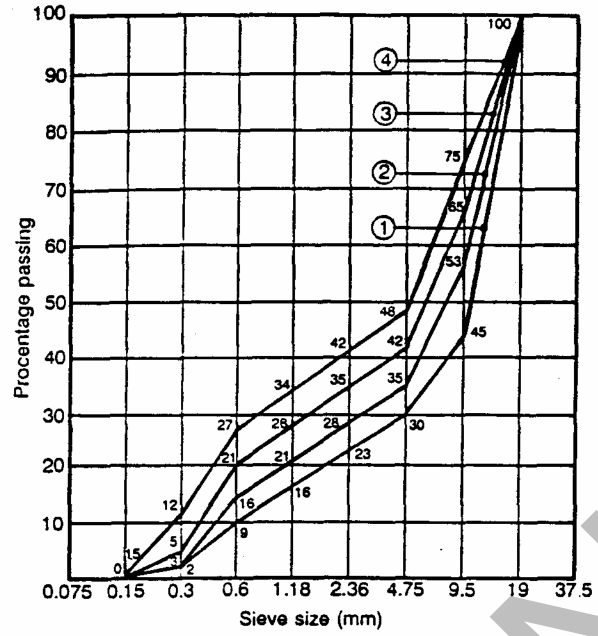


Figure 4.11 - Gradings for 20 mm Aggregate

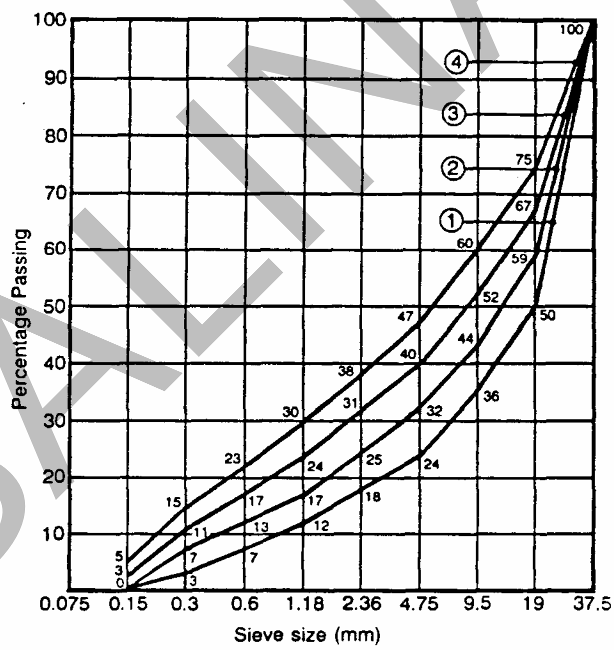


Figure 4.12 - Gradings for 40 mm Aggregate

vi. Example Mix Design

Concrete is required for a reinforced footing. The Design strength is 30 Mpa (cylinder strength) at 28 days.

Control of concrete production is assumed to be good to very good.

1. **Select Materials**

   Use available materials

   a. Type I portland cement
   b. Medium grain sand - 5 mm nominal maximal size
   c. Crushed aggregate - 20 mm nominal maximum size

2. **Target Strength** Assume no relevant information is available

   From Table 4.1 - Assume Standard Deviation = 5.0 Mpa
   From Table 4.2 - Standard Deviation is assumed so use k = 1.64

   Therefore target strength
   = F'c + 1.64 x Standard Deviation
   = 30.0 + 1.64 x 5.0
   = 38.2 Mpa

3. **Water/cement (W/C) Ratio for Strength**

   From Figure 4.2 for type I cement and 28 day strength = 38.2 Mpa.
   W/C = 0.45 to 0.53 (Note that if using cube strengths a reduction factor of 0.8 (approximately) must be used to convert to the equivalent cylinder strength).
   assume W/C = 0.50

4. **Water/cement Ratio for Durability**

   Assume exposed structure - medium severity. From Table 4.3 a W/C ratio of 0.50 would be satisfactory for all but the most severe conditions.

5. **Design water/cement Ratio**

   A water/cement ratio of 0.50 (rounded up from 0.49) will satisfy both conditions for strength and durability.

6. **Choice of Slump** should be tailored for the situation.

Assume placing will be easy, so select a slump in the range 50 mm to 80 mm (65 mm average).

The free water content for this slump and a maximum aggregate size of 20 mm is 195 kg/m^3 (Figure 4.3).

The cement content is thus 195 / 0.50 = 390 kg/m^3. (Item 3.1)

The minimum cement content is 300 kg/m^3 so there is no need to change the water/cement ratio.

The relative density of the combined aggregate is assumed to be 2.65 (SSD) and the concrete density (Item 4.2) is obtained from Figure 4.4 as 2385 kg/m^3. The total aggregate content is then found (by subtraction) as 1800 kg/m^3.

The grading of the sand conforms to that of the Zone 2 grading (see Figure 4.6) and so the proportion of fine aggregate is read from Figure 4.8 as the range 37 % to 46 % (Slump range 60 - 180 mm, w/c = 0.5), say an average of 42 %. The fine aggregate content (Item 5.3) is thus 1800 x 0.42 = 756 kg/m^3 and the coarse aggregate content 1044 kg/m^3 (Item 5.4).

The grading of the combined aggregate can now be calculated and checked against the curves shown in Figures 4.10, 4.11 and 4.12. These grading curves represent aggregate gradings which will produce satisfactory concrete. If the combined grading curve falls outside the envelope for the relevant size of aggregate a new ratio of fine to coarse aggregate must be selected and back checked in Figure 4.8 (for 20 mm aggregate).

7. **Proportions**

From the figures above the calculated proportions are (Item 6.2):

0.5 parts water
1.0 parts cement
1.94 part sand
2.68 parts coarse aggregate

For a one (40 kg) bag of cement mix, these proportions will become (Item 7.2):

Water	0.5 x 40	= 20	kg
Cement	1.0 x 40	= 40	kg
Sand	1.94 x 40	= 78	kg
20 mm Gravel	2.68 x 40	= 107	kg
T o t a l		= 245	kg

The volume occupied by the mixed materials can be determined by dividing the mass of each material by its specific gravity. In the case of the aggregates the specific gravity is usually the density of the particle in a saturated dry condition (S.S.D.) see step 8.

The volume occupied by the materials of item 6.1 is then (Item 7.3):

Water	$\frac{195}{1.0}$	=	195 litres
Cement	$\frac{390}{3.15}$	=	124 litres
Sand	$\frac{756}{2.65}$	=	285 litres
20mm gravel	$\frac{1044}{2.70}$	=	387 litres
	Total		<u>101,7 litre</u>

Mixes of concrete usually contain entrapped air, see Table 4.5. With 2 percent entrapped air (typical of 20 mm aggregate concrete) the volume of the mix now becomes (Item 7.4): 991 x 1.02 = 1011 litres

Therefore the basic mix will yield 1011 litres of concrete. To arrive at proportions for one cubic metre of concrete the proportions will have to be multiplied by:

$$\frac{1000}{1011} = 0.990$$

Table 4.5 Entrapped Air

Coarse Aggregate Size	Non Air-Entrained Concrete	Air Entrained Concrete
10 mm	3	8
20 mm	2	6
40 mm	1	4.5
70 mm	0.3	3.5

Mix quantities for 1 m^3 of concrete are thus (Item 7.5):

Water	195 x 0.990	=	193 kg
Cement	390 x 0.990	=	386 kg
Sand	756 x 0.990	=	748 kg
Gravel	1044 x 0.990	=	1033 kg
	Total	=	2360 kg

This is not strictly accurate as an allowance must also be made for free water in the aggregates.

8. Correction for Moisture

Up to this stage all calculations have been based on the aggregates being in a saturated surface dry (S.S.D.) conditions. This condition occurs when the aggregate contains no free moisture but only absorbed moisture.

Table 4.6 lists criteria for estimating the moisture content of sands in the field. More accurate testing is required for final calculations.

Table 4.6 - Estimation of Sand Moisture Content

Moisture Content (%)	General Appearance of Sand
0	Very dry, dusty and free running - rarely found in practice.
1	As for 0% but sand slightly darker - rarely found in practice.
2	No dust, looks fairly dry, runs freely.
3	Moist in appearance - will not retain shape when squeezed in hand. Runs freely.
4	Slight tendency to retain shape when squeezed in hand - runs fairly freely.
5 - 6	Just retains shape when squeezed in hand. Does not run freely, tends to move in lumps. Hangs in small lumps to equipment.
7 - 10	Very "sticky" hangs to equipment. When squeezed no free moisture appears on surface.
10 - 20	As for 7 -10 but obviously wet and lumpy. Water settles out when undisturbed - obviously heavy on the shovel.

On site, aggregates will usually be in a different condition, therefore corrections will have to be made to the batch weights.

Assume the sand contains 8 percent moisture and the coarse aggregate has a 2 percent moisture content and each an absorption of 1 percent.

- **Sand**

If the moisture content is 8 percent and the absorption is 2 percent then an additional 6 percent of free moisture is present in the sand.

The oven dry weight (Item 8.3) of the 748 kg is 748/1.02 = 733 kg.

733 kg plus 8 % moisture = (1.08 x 733) or 792 kg.
There is thus 44 kg free water (792 - 748) (Item 8.4)

- **Coarse Aggregate**

If the moisture content is 2 percent and the absorption is 1 percent then an additional 1 percent of free moisture is present in the coarse aggregate.

The oven dry weight of the 1033 kg is 1033/1.01 = 1023 kg.

1023 kg plus 2 % moisture = (1.02 x 1023) or 1043 kg.
There is thus 10 kg free water (1043 - 1033)

Moisture correction calculation

Total free water in the aggregates is thus (44 + 10) = 54 kg. Added water for one cubic metre of concrete will have to be reduced by 54 kg ie. 193 - 54 = 139 kg.

As the relative densities of the aggregates and water are different and the relative amounts of water and aggregates have been changed, the adjusted weights given below will not give one cubic metre of concrete but some lesser amount

Water	139 kg	0.139 m^3
Cement	386 kg	0.123 m^3
Sand (8 % mc)	792 kg	0.299 m^3
Gravel (2 % mc)	1043 kg	0.386 m^3
	--------	------------
	2360 kg	0.947 m^3

There will be 2 % entrapped air and hence proportions for one cubic metre of concrete must be based on the above figures x 0.98 / 0.947 that is (Item 9.2)

Water	144 kg
Cement	400 kg
Sand (8 % mc)	820 kg
Gravel (2 % mc)	1081 kg

	2445 kg

These proportions can be used to prepare the first trial mix as described in Section 4.2.3.

Volume Batching

If the contractor is to use volume batching the weights calculated above must be converted into volumes.

Take the specific gravity of the cement as 3.15 and assume that tests on the sand and gravel gave specific gravities of 2.65 and 2.70 respectively.

The total volume of concrete also includes an amount of entrapped air as referred to above.

Relating the volumes of aggregates and water to a single 40 kg bag of cement the following was calculated (Item 9.5):

Water	0.014 m^3
Cement	1 bag
Sand (8 % mc)	0.031 m^3
Gravel (2 % mc)	0.040 m^3

Suitable gauge boxes should be constructed for each of the aggregates and a suitably calibrated container used for the water.

CONCRETE MIX DESIGN FORM - Example of Section 4.2.2.b.vi

NO	ITEM	REFERENCE OR CALCULATION	VALUES					
1.1	*Characteristic Strength*	*Specified*			30	MPa at	28	days
				Proportion Defective			5	percent
1.2	Standard Deviation	Table 4.1				MPa or no data	5.0	MPa
1.3	Margin	C1	(k = 1.64)		1.64 x	5.0 =	8.2	MPa
1.4	Target mean strength	C2			30.0 +	8.2 =	38.2	MPa
1.5	*Cement type*	*Specified*	OPC/SRPC/RHPC					
1.6	Aggregate type:coarse Aggregate type:fine		Crushed Natural					
1.7	Free-water/cement ratio	Fig. 4.2	0.49	Use the lowest value				
1.8	Water/cement ratio for durability	Table 4.3	0.50					
1.9	*Maximum free-water/cement ratio*	*Specified*	-			0.49 but say	0.50	
2.1	*Slump or V-B*	*Specified*		Slump	65 (avg)	mm or V-B		s
2.2	*Maximum aggregate size*	*Specified*					20	mm
2.3	Free-water content	Fig. 4.3					195	kg/m^3
3.1	Cement content	C3			195 /	0.50 =	390	kg/m^3
3.2	*Maximum cement content*	*Specified*					-	kg/m^3
3.3	*Minimum cement content*	*Specified*	360		kg/m^3	Use if greater than item 3.1 and calculate item 3.4		
3.4	Modified free water/cement ratio						0.50	
4.1	Relative density of aggregate (SSD)		2.65			known/assumed		
4.2	Concrete density	Fig. 4.4					2385	kg/m^3
4.3	Total aggregate content	C4	2385		195 -	390 =	1800	kg/m^3
5.1	Grading of fine aggregate	BS 882		Zone (Fig. 4.5 or 4.6)	2			
5.2	Proportion of fine aggregate	Figs. 4.7, 4.8 or 4.9			37 - 46	=	42	percent
5.3	Fine aggregate content				1800 x	0.42 =	756	kg/m^3
5.4	Coarse aggregate content	C5			1800 x	0.58 =	1044	kg/m^3

Quantities (uncorrected for air or moisture in aggregates)	Cement (kg)	Water (kg or l)	Fine aggregate (kg)	Coarse aggregate (kg)
per m^3 (to nearest 5 kg)	390	195	756	1044

1) Figures in italic are optional limiting values that may be specified
2) OPC = Ordinary Portland Cement; SRPC = Sulphate Resisting Portland Cement; RHPC = Rapid Hardening Portland Cement
3) Relative density is specific gravity
4) SSD = based on a saturated surface-dry basis

Figure 4.13 Concrete Mix Design Example

ITEM	DESCRIPTION	Cement (A)	Water (B)	Fine Aggregate (C)	Coarse Aggregate I (D)	Coarse Aggregate II (E)	TOTAL (F)
6.1	Weight Basic mix design (kg)	390	195	756	1044		From bottom of Figure 4.1
6.2	Parts per part of cement	1	0.50	1.94	2.68		[6.1]/[A 6.1]
7.1	Relative Density	3.15	1.00	2.65	2.70		
7.2	Weight of materials per 40 kg bag of cement	40	20	77.5	107.1		[6.2] x 40
7.3	Volume of materials in litres	123.8	195.0	285.3	386.7		[6.1]/[7.1] {991 l}
7.4	Air Content	2.0 %			Total volume including air		1011 litres
7.5	Weight of materials for 1 m³ concrete	385.9	193.0	748.1	1033.1		$\frac{[6.1]\text{x}1000}{[F\ 7.3]\ \text{x}\ (1+[7.4])}$
8.1	Moisture Content (%)			8.0	2.0		
8.2	Absorption (%)			2.0	1.0		
8.3	Weight Oven Dry (kg)			733.4	1022.8		$\frac{[7.5]}{(1+[8.2]/100)}$
8.4	Change in water (kg)		-54	44	10		[8.3] x (1+[8.1]/100) - [7.5]
8.5	Weight of materials corrected for moisture	386	139	792	1043		[7.5] - [8.4] {2 360 kg}
9.1	Volume based on 8.5 above	123	139	299	386		[8.5] / [7.1] {947 l}
9.2	Weight for 1 m³ corrected for air content and moisture	400	144	820	1081		[8.5] x $\frac{(1-[7.4]/100)}{[F\ 9.1]}$ { 2 445 kg}
9.3	Volume of materials in litres	126.9	143.7	309.6	400.2		[9.2] / [7.1] { 980 l} + 2% air
9.4	Weight of materials per 40 kg bag cement	40	14.4	82.1	108.1		[9.2] x 40 / [A9.2]
9.5	Volume / 40 kg bag cement	13.0	14.0	31.0	40.0		[9.3] x 40 / [A 9.3]
9.6	Weights for a trial mix of : 0.032 m³ (Z)						
9.6.1	kg	12.8	4.6	26.3	34.6		[9.2] x Z
9.6.2	litre	4.1	4.6	9.9	12.8		[9.6.1] / 7.1
	Note : [F 9.1] means the total of columns A to E in Row 9.1 [B 6.1] means the value in column B of row 6.1						

Figure 4.14 Concrete Mix Design Example (continued)

Note : The formulas in the TOTAL column on the right hand side of a row indicate how the entries for that row are calculated.

4.2.3 Trial Mixes

Having proportioned the concrete materials to give certain desired properties, it is then necessary to batch a small trial mix, say 0.1 m^3 of concrete, to ascertain whether or not the assumptions made in the mix design were correct. This trial mix must be tested for compressive strength, slump, and any other properties required by the designer to see if in fact these properties are obtained with the estimated proportions of materials.

It will often be found that some slight adjustments have to be made to the proportions as a result of testing the trial batch of concrete. Such adjustments should be made on the following basis:

1. **Adjustments for strength or durability:**

   Adjust the water/cement ratio in accordance with the strength-water/cement ratio relationship, i.e. to increase the strength or improve durability, reduce the water/cement ratio.

2. **Adjustment for slump, workability or cohesiveness:**

   All such adjustments must be made without altering the water/cement ratio, as this would alter the strength and durability of the concrete. Adjustment can be made to either the aggregate/cement ratio or to the grading of the aggregate. As a guide it should be remembered that a decrease in the aggregate/cement ratio (i.e. a mix richer in cement) will increase the slump and improve the workability of a concrete even though the water/cement ratio is not altered.

The following is an extract from the specifications:

"Prior to consent being given to a mix proposed by the Contractor its compressive strength and shrinkage at 28 days will be checked from the trial mixes.

A minimum of 20 specimens should be cast for the purpose of ascertaining the compressive strength of the trial mix.

In the case of urgency or for mixes which contain special admixtures, or are steam cured the Engineer may give a provisional consent based on tests at an earlier age than 28 days, but tests at age 28 days are to be the basis of final consent.

After the Engineer has consented to the use of a certain mix design for a particular class of concrete this mix shall be used for the work. In the event of changes in either properties or sources of materials or in their relative proportions the Engineer may require changes in the proportion of the materials and further testing."

Because of the delay in gathering data on compressive strength it may be necessary to use accelerated methods of curing and testing.

After a suitable laboratory mix has been determined it can then be used in the field. Alternatively mixes can be developed in the field, with the trial being used in unimportant work such as paths, temporary foundations for huts etc. As the job proceeds and test results become available it will be possible to check the Standard Deviation and compare it with the assumed Standard Deviation. If the results are better than assumed a lower target strength may be selected with resulting economics in materials. Mixes may also have to be varied to accommodate changes in the weather or variation in formwork and reinforcement congestion (see Section 4.2.4).

4.2.4 Control of Mix During Contract

The following is an extract from the specifications for concrete.

"In order to determine any need for mix adjustment throughout the progress of the work, a statistical check may be made of the compressive strength of concrete, using consecutive 28 day test results representing concrete placed in the work, and making separate checks of each mix.

For each separate class of concrete, the concrete mix and its method of production will be considered satisfactory should the following requirements be met:

(i) Not more than one specimen from a group of twenty (20) consecutive specimens shall have a compressive strength at 28 days less than the Characteristic Strength for that class of concrete.

(ii) The average of the compressive strength at 28 days of any four (4) consecutive specimens shall not be less than the Characteristic Strength for that class of concrete plus 0.82 times the standard deviation as defined below.

(iii) The difference in the values of the compressive strength at 28 days between the highest and lowest value of any four (4) consecutive specimens shall be less than 4.3 times the standard deviation defined below.

The standard deviation shall be taken as the initial estimate until 20 specimens from concrete in the structure have been tested. At this stage the value of the standard deviation shall be calculated from the results of the 20 strength tests. This review process shall be repeated after every successive 20 test results and the requirements (i), (ii) and (iii) above applied to succeeding batches of concrete.

In any case the standard deviation shall not exceed 8.5 MPa (85 kg/cm^2) for classes of concrete with Characteristic Strengths less than or equal to 35 MPa (350 kg/cm^2) or 5.0 MPa (50 kg/cm^2) for classes of concrete with Characteristic Strengths above 35 MPa (350 kg/cm^2).

Notwithstanding consent given by the Engineer to a proposed mix, the Contractor shall be solely responsible for producing concrete which satisfies the requirements of this Specification."

4.3 BATCH METHODS

4.3.1 General

This Section covers aspects of materials handling and batching which are specific to projects in Indonesia.

Before batching commences, the mixer drum should be thoroughly wetted with clean water and any surplus water discharged. Prior to charging the mixer with the first batch of concrete materials the mixer must be flushed with a suitable mixture of fine aggregate, cement and water, mixed for a minimum period of two minutes and then the resultant slurry discharged to waste. All the slurry and any cleaning water should be completely discharged from the mixer prior to the introduction of any concrete materials. This will ensure that all the cement paste from the batch forms part of the concrete and does not adhere to the otherwise dry walls of the mixer. The aggregate, cement and correct quantity of water, making due allowance for aggregate moisture content, is then added to the mixing drum and mixed for the prescribed period.

4.3.2 Handling of Materials

The following points are noted:

- Cement must be stored under suitable weatherproof covers. Any cement which has been affected by water or contains significant hard lumps must be rejected as unsuitable for use. Cement which is older than the age specified in the Specifications (usually between 10 and 16 weeks) should only be used after careful checking.

- Aggregates, especially fine aggregate, should be tested regularly for moisture content. The moisture content of the aggregates directly affects the amount of mixing water required to be added to the batched materials. Coarse aggregates should be stockpiled on a free draining base so that water will not be trapped in the stockpile.

4.3.3 Volume Batching

This is the more common method in use on bridge projects in Indonesia. It is the simpler method to use, but unfortunately leads to the greater problems.

The mix design will usually give the proportions of the constituents by weight and a conversion must be made from weight to volume if volume batching is to be carried out.

This conversion assumes that the aggregate weights are based on compacted unit weight at saturated surface dry conditions. Further adjustment will be required to take into account the moisture content and the bulking of the sand.

The moisture content of sand has a marked effect on its volume and must be taken into account when measuring to avoid significant inaccuracies in proportioning concrete and mortar. The volume of a given weight of sand is increased by moisture far out of proportion to the quantity of moisture present and the effect varies with the nature of the sand. Some sands may increase in volume by as much as 40 percent due to moisture.

The bulking effect is shown in Figure 4.15 for sands covering the range ordinarily used in concrete.

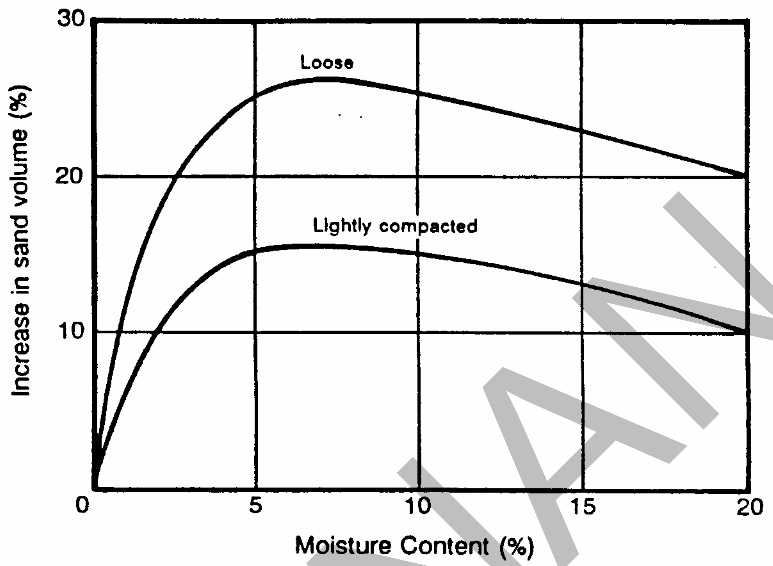


Figure 4.15 - Effect of moisture on bulking of sand

It will be observed that the maximum effect occurs at about 5 per cent moisture content which is commonly the moisture content encountered in the field.

Failure to allow for this bulking increases the cost of the concrete and often results in under-sanded mixes which are harsh and difficult to place.

For example :

If the medium sand shown in Figure 4.15 is used and it contains 5 per cent moisture it can be seen that the bulking is about 29 per cent. If the mix was 1 : 2 : 4 by volume and measured as such without correction, instead of 2 cubic metres of sand per 1 cubic metre of cement, the actual dry sand measured would be 2 / 1.29 = 1.55 cubic metres. The mix would then be 1 : 1.55 : 4 in terms of dry sand. This reduction in the ratio of sand causes a reduction in the amount of concrete produced with each bag of cement and in most cases there would be insufficient fine material for the amount of coarse material to give a workable mixture.

To make allowances for the bulking in this example, 1.29 x 2 = 2.58 cubic metres of the damp sand should be used for each 1 cubic metre of cement. The volume of dry sand in this quantity of damp sand will be 2 cubic metres.

Harsh mixes due to under-sanding make finishing difficult and therefore more expensive. Such mixes often result in honeycombing or stone pockets which require repairs that add further to the cost of the concrete.

Gauge Boxes

Volume batching should be carried out using gauge boxes. These should be not too shallow and their internal dimensions should be accurate. They should be loose filled with the material being measured and then struck off level with a straight edge. Measures such as the *shovelful* or shallow gauge boxes heaped up with material should not be used as no two measurements will be exactly alike.

It is preferable to arrange proportions so that whole bags (40 kg) of cement are used as significant bulking of cement occurs when cement is tipped from the bag into a gauge box.

4.3.4 Weigh Batching

Concrete for major works is preferably batched by weight and is recommended as the preferred method of batching for producing consistently good quality concrete.

Weigh batching eliminates uncertainty caused by bulking and allowances for moisture in the aggregates may be easily made.

Equipment for weigh batching may be as simple as a set of scales and a runway over which wheelbarrows are run and weighed. With a little experience, the labour force will be able to estimate fairly closely the amount of each type of material needed in the barrows and little addition or removal of material will be required. Material from the barrow is then tipped directly into the batching plant.

Larger batch plants use hoppers with some form of weighing apparatus but these are in general beyond the scope of most bridge construction projects in Indonesia due to the size of the projects, the average amount of concrete in a bridge being less than 400 cubic metres, divided into a number of relatively small pours.

4.4 MIX METHODS

4.4.1 Mix Records

It is important that good records are kept of all concrete mixed and incorporated in the structure. The inspection report of the batch and mixing plant should verify and document:

- Storage details of cement and aggregate
- Sufficient quantities available for each placement
- that the batch is released for placement
- the adjustment made for moisture content fine and coarse aggregates
- temperature of materials
- mixing time to ensure that uniformity requirements are met
- total water used compared with that allowable to maintain required water-cement ratio

The daily summary of inspection of the concrete plant should include at least the following information:

- Date
- Total number of cubic metres of each class of concrete batched
- Placement identification
- Brand and type of cement and dates when shipments were received and used
- Moisture contents of aggregates
- Temperatures of materials
- Mixing times for central mixer

When transit mixers are used for mixing, reports should include the following inspection results compared with the allowable limits:

- Mixing and agitating revolutions
- Time concrete delivery completed after batching
- Total water, including added water

Sample batch plant inspection forms are shown in Figures 4.16 and 4.17. These may be used as the basis for inspection forms and modified to suit individual needs.

BATCH PLANT INSPECTION REPORT
Part A - Summary

BRIDGE	CONTRACTOR	CONTRACT NO
PROVINCE	REPORT NO	DATE

BATCH NUMBERS	TOTAL CU. METRES.	DELIVERY TICKET NUMBERS	

TIME BATCHING STARTED: ____________ TIME BATCHING COMPLETED: ____________

WEATHER: ____________

PLACEMENT LOCATION: ____________

____

BATCH PLANT DELAYS: ____________

REMARKS: ____________

____

____

____

____________ Inspector

____/____/____ Date

Figure 4.16 - Batch Plant Inspection Form - Part A

BATCH PLANT INSPECTION REPORT
Part B Details of Batch Plant Operation

INSPECTION ITEMS	CHECK (✓) APPLICABLE RATING				
	EXCELLENT	GOOD	FAIR	POOR	SEE REMARKS
Admix Storage Facilities					
Condition of Cement Silos					
Condition of Aggregate Stockpiles					
Condition of Delivery Trucks					
Reliability of Printout					
Hot/Cold Weather Provisions					
Overall Plant Performance					
Housekeeping					

TIME (See Note 1)	DELIVERY TICKET NUMBER (See Note 2)	AIR TEMP (°C)	WATER TEMP (°C)	CON-CRETE TEMP (°C)	FINE AGG MOIST CONTENT (%)	COARSE AGGREGATE MOISTURE CONTENT (%)				Allowable Water kg or litres
AM PM										
AM PM										
AM PM										
AM PM										
AM PM										
AM PM										
AM PM										
AM PM										
AM PM										
AM PM										
AM PM										

NOTES:

1. Time sample taken for moisture test
2. No. of first delivery ticket using information from this moisture test.

Figure 4.17 - Batch Plant Inspection Form - Part B

4.4.2 Ready Mixed Concrete

Very few projects have access to ready mixed concrete. Some sites which are close to major centres may be able to utilise these facilities.

Ready mixed plants fall into one of three types.

- Central Mixing plants completely mix the concrete which is then transported to the site in a truck agitator or truck mixer.
- Stage mixed plants partially mix the concrete (15 to 30 seconds) and mixing is then completed in the truck mixer. This method minimises the problems associated with the extra bulk of separate materials.
- Truck mixers completely mix concrete in the truck. The separate materials are usually dry batched at a central batching plant. Water may be added at the plant, from the truck or at the site.

When such plants are available a number of points should be noted.

- For thorough mixing of truck or stage mixed concrete the acceptable number of drum revolutions at the manufacturers' designated mixing speed is usually between 55 and 100
- Discharge should be completed within 45 minutes of the commencement of mixing. This time may need to be reduced to take the effects of hot weather into account
- The volume of concrete mixed in a truck mixer should not exceed 63 per cent of the gross internal volume of the drum
- The volume of concrete centrally mixed and transported in a transit mixer should not exceed 80 per cent of the gross internal volume of the drum

A sample Ready mixed plant inspection form is shown in Figure 4.18.

PLANT INSPECTOR'S CERTIFICATE*
READY-MIXED CONCRETE

Plant: Date:

Docket No: Volume of Mix: cu.m.

Time of batching a.m./p.m.

MIX Cement Content kg/cu.m.

Nominal Size mm Nominal Slump mm.

MATERIALS	Design Mass	Adjusted Design Mass	Adjusted Design Mass (kg) cumulative		Actual Batch Mass (kg) cumulative		M.C.
	kg/cu.m.	kg/cu.m.	1 cu.m.	... cu.m.	... cu.m.	... cu.m.	%
40 mm							
20 mm							
13 mm							
C. Sand							
F. Sand							
Cement							
Water (litres)							

Total water added at plant + = litres

AMOUNT OF WATER WHICH MAY BE ADDED ON SITE [ ] litres

Signed: Signed:

Supplier's Representative Plant Inspector

THE SLUMP OF THIS BATCH MUST BE MEASURED AT SITE TO CHECK COMPLIANCE WITH REQUIREMENTS OF SPECIFICATION

SITE USE ONLY Job No: Time cast: a.m./p.m.

Part of Structure

Measured Slump mm. No. of cylinders

Water may only be added on site prior to the commencement of discharge of the concrete and at the discretion of the manufacturer and then shall not exceed the quantity shown above. If water is added on site the mixer shall be operated at mixing speed until the uniformity of the concrete is within the required limits.

AMOUNT OF WATER WHICH MAY BE ADDED ON SITE [ ] litres

Signed:

* This certificate is to be signed by the plant inspector, and also the person on site who has been delegated authority by the supervising engineer. It is to be kept on site until compressive strength results have been obtained.

Figure 4.18 - Readymix Concrete Plant Inspection Form

4.4.3 Site Mixing

This section covers the mixing of concrete in mixers at the job site, probably the most common method in use on bridge construction sites in Indonesia.

Mixers are usually only small, of the order of 0.25 cubic metres. This size is actually too small for bridge concrete works, even though several mixers are used whenever a larger pour, usually the concrete deck, is carried out.

Many problems in concrete works arise as a result of using small mixers. The output of such mixers is low and in hot weather and with too few mixers in operation the surface of the concrete is likely to have hardened before the next layer of concrete is placed. This gives rise to a series of 'cold' joints which are quite evident in the concrete.

The use of hand mixing should be prohibited except in the case of a real emergency, and then only to mix sufficient concrete to form up to a suitable construction joint. Contractors often do not form construction joints but rather let the concrete flow at the end of most pours. This should not be allowed and the contractor should be instructed to comply with the specifications in this regard.

4.4.4 Transport of Concrete

The transport of the freshly mixed concrete to the point of discharge or placement may be carried out using a number of different methods.

Irrespective of the method used, consideration must be given to minimising:

- the delay before placing
- drying out of the concrete, and
- segregation of the coarse aggregate from the rest of the concrete

Some general notes on these points are given in the Construction Supervision Manual.

Additional notes are given below:

- The degree of stiffening that occurs in the first 30 minutes after mixing is not significant.

- Concrete with a low water/cement ratio will stiffen more quickly than one with a higher water/cement ratio

- If drying out of the mix is likely to occur a richer, more workable mix should be used and protection from the sun and the wind provided during transport and placement of the concrete

The methods of transport usually adopted in Indonesia are discussed below.

- **Chutes**

  This is the most common system in use on bridge projects. Chutes are constructed out of timber from the point of mixing to the point of placement. The main problem with the chutes is that the concrete is allowed to discharge directly from the end of chute into the form (and so cause segregation) rather than vertically through a baffle and opening arrangement, as shown in Figure 4.19. The slope of the chute must be sufficiently steep to allow gravity flow of the concrete at the lowest slump to be used. Angles of 25 to 30 degrees are usually sufficient.

  Long chutes should preferably be covered to protect the concrete from the sun.

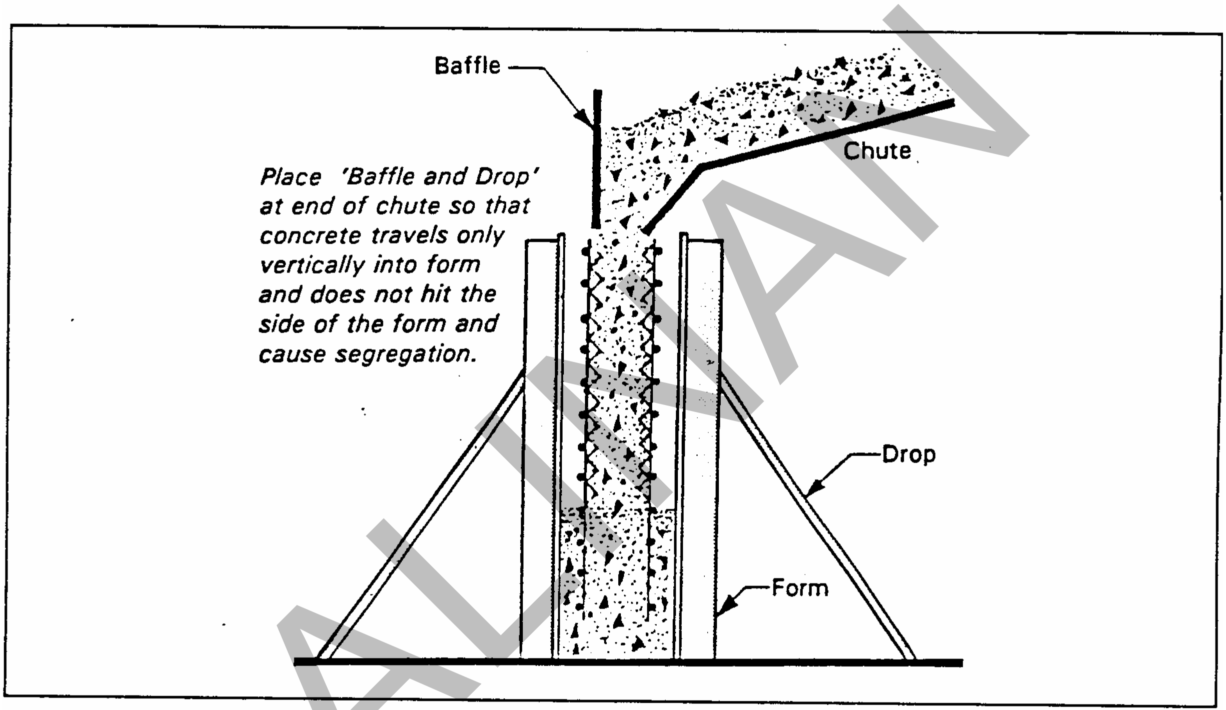


Figure 4.19 - Discharge of Concrete from a chute

- **Barrows and Handcarts**

  These are common in Indonesia as they require no specialised plant. It is essential that the runways used are properly supported and that both forward and return paths are provided to prevent congestion, especially near the mixer(s).

- **Dump Buggies**

  These are simply a mechanised form of barrows, used for horizontal transport, which normally come in sizes up to about 1 cubic metre.

- **Crane and bucket**

  This system is a common form of transport when the concrete is required to be transported through large vertical distances. It assumes the presence of a crane on site which means that it is only feasible on the larger projects. Buckets are round or square in cross section and should be capable of discharging a portion of their load at a time, using a swinging gate arrangement at the base of the bucket.

- **Rail cars**

  This system is sometimes used for deck or long walls and is a variation on the barrow system.

- **Pump**

  Specialised pumping equipment will allow large quantities of concrete to be transported through horizontal and vertical distances much faster than the other methods outlined above. As concrete pumps are expensive, only the larger contractors have this facility and their use is more common on building sites than bridge sites.

4.4.5 Retempering of Concrete

Retempering of concrete is the process of adding water to concrete which has stiffened due to time or temperature effects. It is almost always prohibited by the specifications. This process should not be confused with water which is added on arrival at site when **both** the slump is less than that specified **and** the water/cement ratio is less than the design value.

If a typical (small) site mixer system is used this problem should not occur. Given the batch quantities being mixed at any given time, concrete which has lost its workability should be simply discarded and not used.

The following points should be noted:

- If the concrete has stiffened to the extent that it cannot be placed or adequately compacted, workability can often be restored by remixing. This may be successful to 1 hour or so after mixing in the usual temperatures in Indonesia.

- The addition of cement and water (in the correct proportions) may assist with the remixing. Addition of water alone to restore workability is not to be permitted.

4.5 CONTROL OF CONCRETE PRODUCTION

4.5.1 General

Control of concrete production during the course of a project is a relatively straightforward matter. The Supervising Engineer should ensure that proper records are kept of materials used, batching operations, wet concrete properties, concrete placement and curing and the compressive strength of the test specimens taken.

All of this information will build up a complete picture of concrete production over a period of time. The Specification will set out control limits for acceptance and rejection but the Supervising Engineer should be able to determine trends in quality reduction before they become a question of absolute rejection. If aggregate testing and batch checks are carried out regularly, a correlation between say 7-day strengths and material properties can be established. In addition, a good correlation between 7 and 28 day (or some other age) concrete strengths can also be established.

4.5.2 Consistency of Concrete

Consistency of concrete is usually monitored by using a slump test. An extract from AASHTO T 119 is included in Appendix 4-I for reference.

It is customary to maintain consistency of the concrete relatively constant for a given type of construction (see Table 4.4 for recommended maximum slump for different types of concrete construction). This is done primarily to simplify transporting, placing, compaction and finishing of the concrete. If the aggregate supplies vary in quality, grading or moisture content, or if different slumps are required for various parts of the work, adjustments will be required in batch quantities. The Supervising Engineer should observe the consistency of the fresh concrete in the mixer, in transporting devices, and in the forms during placing and compaction. He should judge the nearest practical slump value for the final requirements at the form. The general tendency of untrained operators is to make the concrete as wet as possible, under the mistaken impression that wet concrete reduces the labour in placing. The importance of maintaining the required w/c ratio and the need to increase the cement content if water is to be increased (to make a wetter concrete) are frequently not realised.

The greater probability for wetter concrete to segregate, especially with leaner (that is lower cement content) mixes, is not adequately recognised. The mix should be just wet enough to ensure placement and full compaction, without honeycombing occurring, and not more.

The mixer operator usually regulates the water to be added at the mixer, based on measured slumps in previous batches. If aggregates are uniform in moisture content and in quality, there will be little need to vary the quantity of water to be added at the mixer. Hence the water content should be varied only to suit the variations in moisture content in aggregates. Because of this necessary adjustment, the water-measuring device (where used) should not be locked at any one fixed quantity. Finish-screening at the batching plant will help to control gradation and water requirements of the concrete mix.

Although the specifications generally provide for control of consistency by the slump test or other suitable test, the supervisor should place strong reliance on his assessment of the concrete at the forms, and he should determine a consistency that can be satisfactorily placed, compacted and finished.

It should be noted that the last stage at which water can be added to the concrete is at the mixer before delivery, after which the concrete must be mixed thoroughly to ensure uniformity of the product. On no account may water be added later, even if it is found that the concrete, after being placed in the forms, could not be compacted satisfactorily by vibration before hardening. There are often some difficulties in the first few batches, but in all major works the system goes smoothly, so long as the supervisor is thorough and systematic in his inspections, and watches for departures from established routine procedures and variations in uniformity of concrete at the forms.

Appendix 4-1

Standard Method of Test for

Slump of Portland Cement Concrete

AASHTO DESIGNATION : T 119-82 (1986)
(ASTM DESIGNATION : C 143-78)

1. SCOPE

1.1 The method covers determinations of slump of concrete, both in the laboratory and in the field.

NOTE 1 - This method is considered applicable to plastic concrete having coarse aggregate up to 1½ in (38 mm) in size. If the coarse aggregate is larger than 1½ in. in size, the method is applicable when it is made on the fraction of concrete passing a 1½ in sieve with the larger aggregate being removed in accordance with Section 4 of T 141 'Sampling Fresh Concrete'[2]. This method is not considered applicable to non-plastic and non-cohesive concrete.

NOTE 2 - The values stated in U.S. customary units are to be regarded as the standard. The metric equivalents of U.S. customary units may be approximate.

2. APPARATUS

2.1 *Mould* - The test specimen shall be formed in a mould made of metal not readily attached by the cement paste. The metal shall not be thinner than No. 16 gauge (BWG) and if formed by the spinning process, there shall be no point on the mould at which the thickness is less than 0.045 in. (1.14 mm). The mould shall be in the form of the lateral surface of the frustum of a cone with the base 8 in. (203 mm) in diameter, the top 4 in. (102 mm) in diameter, and the height 12 in. (305 mm). Individual diameters and heights shall be within ± ⅛ in. (3.2 mm) of the prescribed dimensions. The base and the top shall be open and parallel to each other and at right angles to the axis of the cone. The mould shall be provided with foot pieces and handles similar to those shown in Figure 1. The mould may be constructed either with or without a seam. When a seam is required, it should be essentially as shown in Figure 1. The interior of the mould shall be relatively smooth and free from projections such as protruding rivets. The mould shall be free from dents. A mould which clamps to a non-absorbent base plate is acceptable instead of the one illustrated provided the clamping arrangement is such that it can be fully released without movement of the mould.

2.2 *Tamping Rod* - The tamping rod shall be a round, straight steel rod ⅝ in. (16 mm) in diameter and approximately 24 in. (600 mm) in length, having the tamping end rounded to a hemispheric tip the diameter of which is ⅝ in.

5. REPORT

5.1 Record the slump in terms of inches (millimetres) to the nearest ¼ in. (6mm) of subsidence of the specimen during the test as follows :

Slump = 12 - inches of height after subsidence

6. PRECISION

6.1 Data are being complied and developed that will be suitable for use in developing precision statements for this method.

Metric Equivalent

in	1/16	⅛	½	1	1½	3	3⅛	4	8	12
mm	1.6	3.2	12.7	25.4	38.1	76.2	79.4	102	203	305

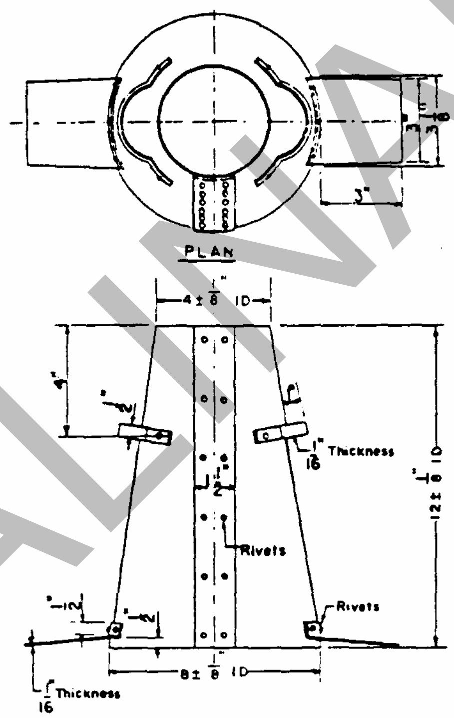


Figure 1 - Mould for Slump Test

5. CONCRETE CONSTRUCTION

5.1 GENERAL

This Section covers aspects of concrete from preparation of the formwork and placement of the reinforcement to the placing and curing of concrete in the forms.

5.2 FORMWORK AND FALSEWORK

5.2.1 General

The plasticity of fresh concrete allows it to be moulded into any desired structural shape. Any inaccuracy or defects in the formwork will be reproduced in the concrete in the structure. It is therefore essential that forms be designed and constructed accurately so that the desired size, shape, position and finish of the cast concrete structure are obtained.

Forms themselves are temporary structures in that they are required to carry, in addition to their self load, loads and pressures from freshly placed concrete as well as construction loads such as materials, equipment and workmen. Formwork must be designed and constructed to carry all these loads without fear of collapse or excessive deflection.

5.2.2 Formwork

a. Design

Section 23 of the Construction Supervision Manual describes the loads to be considered when formwork is to be designed.

Once the loads have been calculated the forms are designed using conventional timber design methods. If the timber properties are unknown estimates of allowable stresses, Elastic modulus etc. must be made. Unfortunately, this is likely to be highly inaccurate unless the species of timber is known.

Design of formwork should be carried out in accordance with the Bridge Design Code or NI-5 (PKKI 1961) 'Peraturan Konstruksi Kayu Indonesia'.

b. Materials

The quality of formwork materials will determine to a large degree the quality of the finished concrete as to shape, surface finish etc.

Formwork for exposed surfaces (front faces of abutments, kerbs, outside of wing walls and the like) should be plywood-lined.

Timber used for studs and wales varies widely in quality and is often too small in section to prevent excessive deflection.

Contractors often do not use a system of form ties (to resist the horizontal forces in the forms) but rely on external props.

2. Planning - the Contractor should plan the work so as to achieve an efficient cyclic programme of assembly, concreting, removal and reshoring and the supervising engineer should check the Contractor's proposals.

3. Safety - the Supervisor must ensure that the Contractor takes adequate safety precautions to protect workmen. Some of the deficiencies which can lead to form failures are:

(i) Premature removal of forms or props.
(ii) Inadequate bracing.
(iii) Failure to control rate of placing concrete in deep forms.
(iv) Failure to regulate properly the placing of concrete on horizontal forms to prevent unbalanced loadings.
(v) Failure to check adequacy of footings for falsework to prevent settlement in unstable ground.
(vi) Failure to inspect formwork during concreting to detect any abnormal deflections or signs of imminent failure.
(vii) Failure to provide adequately for lateral pressure on formwork.
(viii) Props not plumb.
(ix) Inadequate provision against uplift.
(x) Damaged threads on ties or props.
(xi) Failure to check that the drawings are being interpreted correctly.
(xii) Under-design.

4. Workmanship - besides general dimensional accuracy and safety, some of the points on workmanship which warrant attention are:

(i) Joints or splices in sheathing, plywood panels and bracing should be staggered.
(ii) There should be the proper number and location of tie rods or clamps.
(iii) Tie rods or clamps should be properly tightened as concrete vibrators loosen threaded connections very easily.
(iv) The connections of props and stays to joists, stringers and wales must be adequate to resist any uplift or twisting at joints.
(v) Form coatings should be applied before placing of reinforcement and should not be used in such quantities as to run onto bars.
(vi) Bulkheads for joints should preferably be made by splitting the bulkhead along the lines of reinforcement which pass through it so that each portion of the bulkhead may be positioned and removed separately.
(vii) Tapered inserts to form keyways at contraction joints should be left undisturbed when forms are stripped, and removed only after the concrete has been sufficiently cured.
(viii) Wood inserts for architectural treatment should be partially split by sawing to permit swelling without applying pressure to the concrete.
(ix) The loading of new slabs should be avoided in the first few days after concreting.
(x) Formwork must not be treated roughly or overloaded if re-use is desired.
(xi) To facilitate removal, the taper on inserts should be at least 1 in 10.

Steel Forms

Form oils that are satisfactory on wood may not always be suitable for steel forms. Paraffin based form oils, and petroleum based oils blended with a synthetic oil, silicones or graphite, have been successfully used.

Application of Coatings

Surface coatings should be applied to smooth, clean surfaces by methods such as roller, brush, spray, wiping, etc. depending on the type of coating. Coverage must be complete and uniform for good stripping and appearance. There should be no excess coating to stain the concrete. Very thin form oils should not be used in hot weather on vertical impermeable forms such as steel as they trend to run, causing adhesion at the top and excessive oil on the lower sections.

Whenever possible forms should be coated before erection. When this is not possible, the application of coating should precede steel placement to keep the steel free of the coating material. The concrete surface at construction joints must also be kept free of form oils.

f. Adjustment of Formwork

Before Concreting

1. Devices should be installed on supported forms and elsewhere as required, to facilitate detection and measurement of formwork movements during concreting.

2. Wedges used for final alignment before concreting should be secured in position (for example by nailing) after the final check.

3. Positive means of adjustments (wedges or jacks) should be provided to permit realignment or readjustment of falsework if excessive settlement or displacement occurs.

In most circumstances, it will be virtually impossible to realign forms containing concrete.

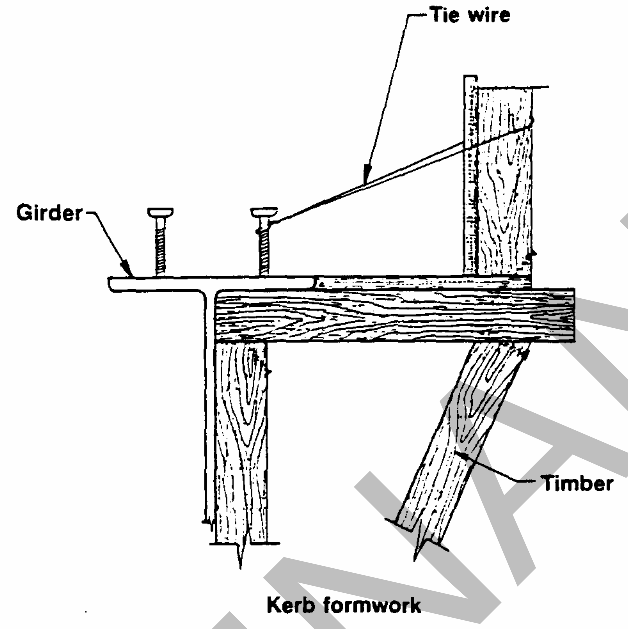
During and after Concreting

The level, camber and plumbness of formwork should be checked constantly and appropriate adjustments made promptly. During concreting it is especially important to have at least one carpenter watching the forms constantly; tightening wedges, adjusting braces and looking for weak spots. He can take prompt action if an emergency arises.

If, during concreting, a weakness develops and falsework shows excessive settlement or distortion before the weakness can be corrected, the work should be stopped, the affected construction removed if permanently damaged, and the falsework strengthened.

A common but unsatisfactory method of forming the kerb overhang on steel girder bridges is shown in Figure 5.2.

This method should not be used as the tie wire, which extends from the surface of the concrete to the shear connector, serves as a path for the entry of a moisture into the concrete and will lead to carbonation of the concrete and corrosion of the reinforcing steel in the deck.



5.2 - Unacceptable Kerb Form System

i. Materials for Formwork

(a) Exterior grade waterproof plywood is the best timber form material. It is usually plastic faced to give a smooth, durable finish. With care, it may be reused up to 20 times. Plywood should be aligned with the grain in the outer plies running perpendicular to the studs or joints for maximum strength and stiffness.

(b) Timber boarding is also used as a form surface but does not give a good off the form finish to the surface of the concrete. The boards used in Indonesia are often of non uniform width and are difficult to seal.

(c) Lined timber forms - Non-structural form facings may be used over a structural timber backing. Such lining types include timbered hardboard (6mm) and waterproof ply nailed to the backing with flathead nails. This is a practical alternative to (a) above when timber boards are used for deck slab formwork.

k. Form Ties

Factors influencing the choice of ties are :

- strength requirements
- anticipated re-use
- ease and speed of assembly and disassembly
- effect of surface finish.

Form ties must be completely removed or, alternatively, they should leave no residual metal part within 40mm of the surface or within the specified cover, whichever is the larger. Typical form ties are shown in Figure 5.3.

On large surfaces such as retaining walls or piers, ties should be regularly spaced to provide a symmetrical appearance.

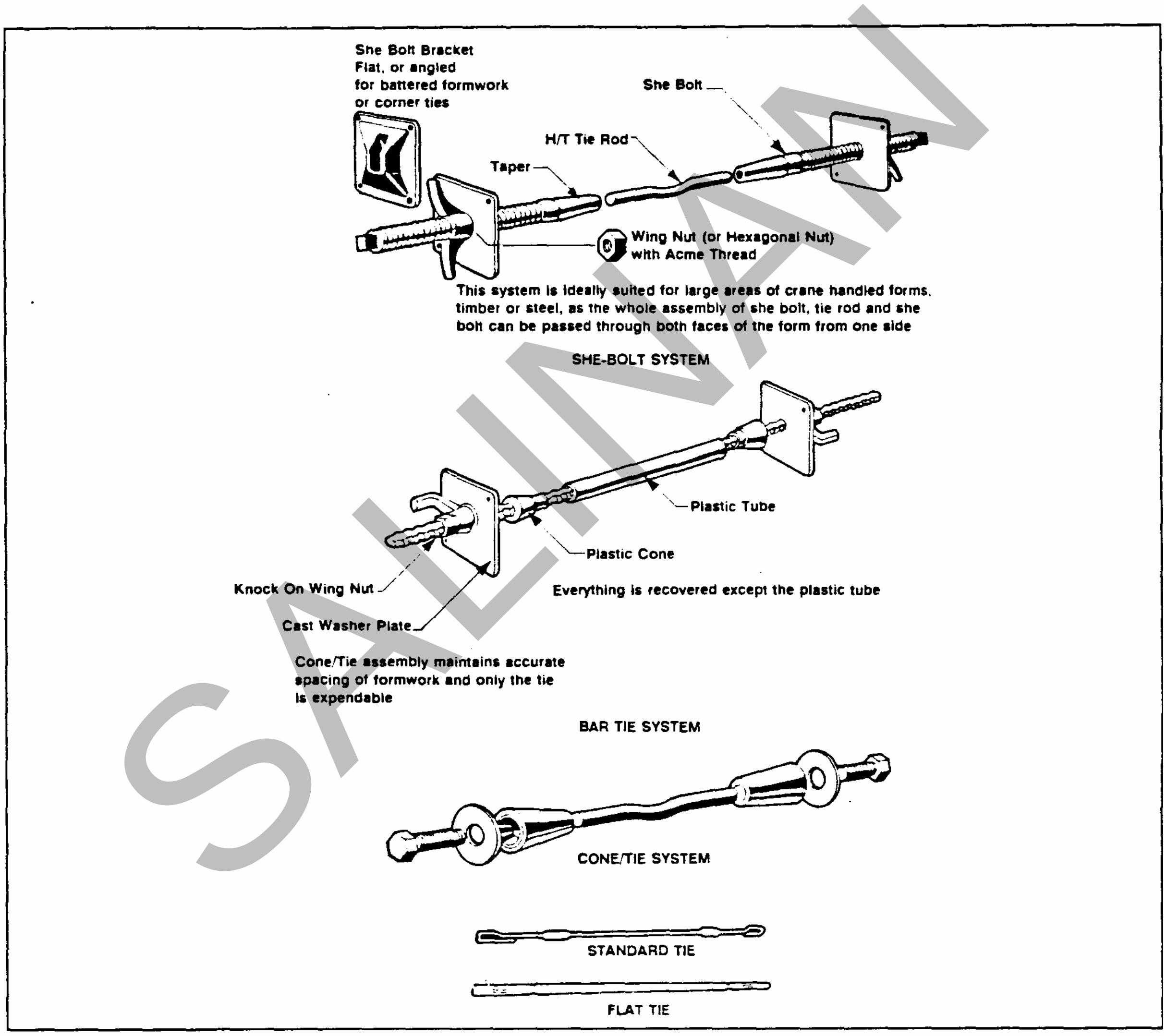


Figure 5.3 - Typical Form Ties

g. Off-Form Finishes

It is usually economic for the formwork structural materials to also provide the surface finish. Where special characteristics such as smoothness, pattern, texture, intricate detail, etc, are required in the finish, extra care must be taken in the selection of form materials and in form construction.

The contact surface is vitally important in off-form finishes. The lining or sheathing material used determines the surface characteristics.

Smooth Surfaces

Most sheathing and lining materials are available in a grade smooth enough to form a blemish-free concrete surface. The proper choice of form oil or coating is most important in achieving the desired smoothness.

If the surface is to be free from marks made by the sheathing material, the joints between boards or panels must be filled or covered. It is virtually impossible to eliminate marks created by the joining of smooth panels of formwork. If forms or liners can span the whole distance between control joints, joints between the units will be less noticeable.

h. Deck Formwork for Steel Girders

An acceptable method is to use a form tie system where the component (She bolt or Cone) nearest the concrete surface is removed and the hole filled with concrete, as shown in Figure 5.1

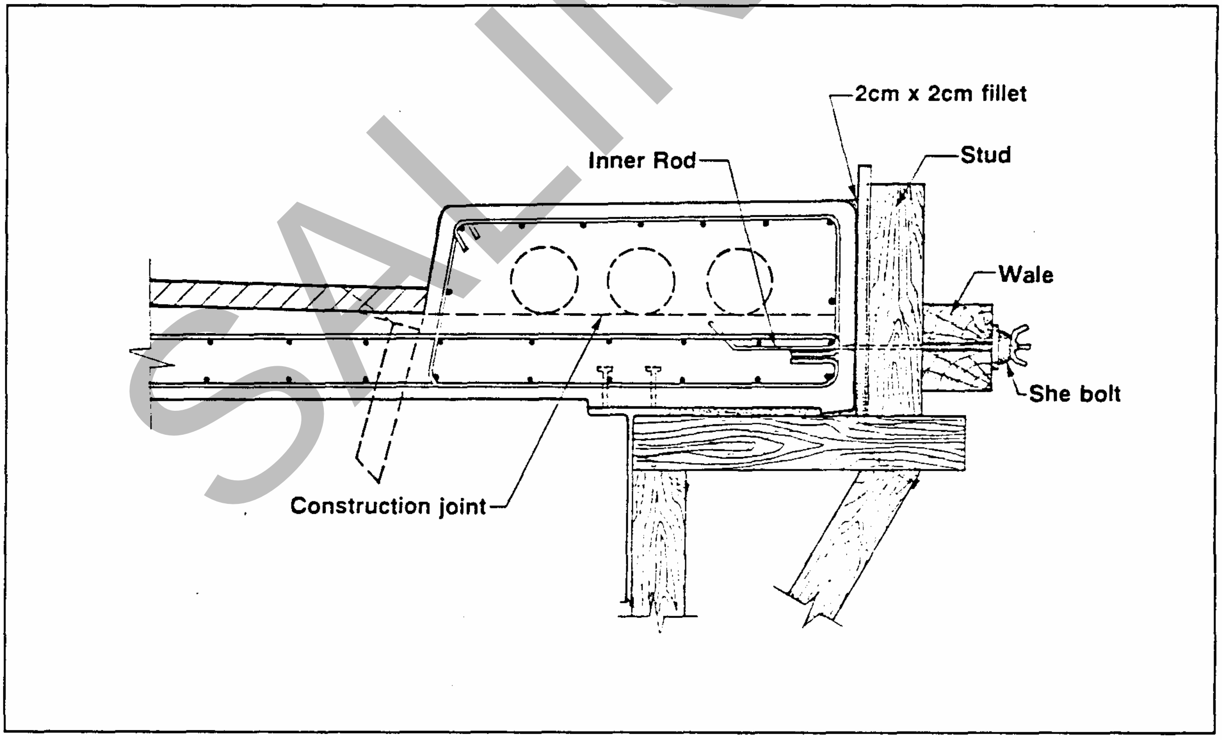
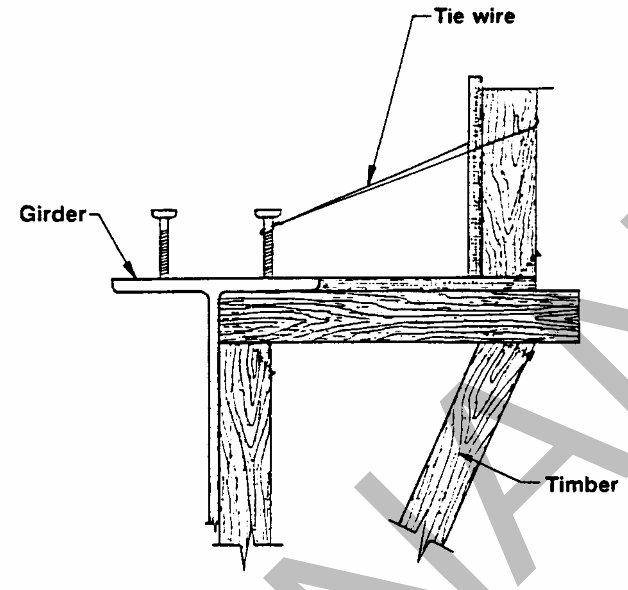


Figure 5.1 - Acceptable Kerb Form System

A common but unsatisfactory method of forming the kerb overhang on steel girder bridges is shown in Figure 5.2.

This method should not be used as the tie wire, which extends from the surface of the concrete to the shear connector, serves as a path for the entry of a moisture into the concrete and will lead to carbonation of the concrete and corrosion of the reinforcing steel in the deck.



5.2 - Unacceptable Kerb Form System

i. Materials for Formwork

(a) Exterior grade waterproof plywood is the best timber form material. It is usually plastic faced to give a smooth, durable finish. With care, it may be reused up to 20 times. Plywood should be aligned with the grain in the outer plies running perpendicular to the studs or joints for maximum strength and stiffness.

(b) Timber boarding is also used as a form surface but does not give a good off the form finish to the surface of the concrete. The boards used in Indonesia are often of non uniform width and are difficult to seal.

(c) Lined timber forms - Non-structural form facings may be used over a structural timber backing. Such lining types include timbered hardboard (6mm) and waterproof ply nailed to the backing with flathead nails. This is a practical alternative to (a) above when timber boards are used for deck slab formwork.

(d) Waterproof Particle Board (200mm). This material is cheaper than form ply but has a very limited number of reuses. It should not be used for forming exposed surfaces where a smooth surface finish is required.

(e) Steel Forms - Steel forms are often purpose built where a large number of reuses is anticipated, up to 200 or more, and the cost is justified. If the surface finish is important, the steel should be lightly blasted and treated to prevent rust.

Steel or plate for formwork appears to retain some of its deflection which is aggravated by constant re-use. Steel reflects with amazing clarity such factors as stud spacings, tack welds, plate welds, figures painted or pencilled on the surface and after some reuses even the pattern of mill scale will appear on the concrete. These effects are particularly pronounced where the light is oblique to the concrete surface or reflected from water.

To obtain good results, the following practical details should be observed :

- a clear span of more than 300mm for sheeting is not recommended for quality work
- keep welding to a minimum
- all welds should be with a very light gauge rod
- weld in an order that minimises distortion
- remember steel sheet or plate is never perfectly flat
- sealing between steel forms is difficult. It may be necessary to use rubber sealing strips
- use a release agent which will inhibit rust.

Several types of multipurpose steel panels are available commercially. One type is an all purpose, wall and slab form panel, consisting of replaceable plywood inserts in a steel frame. This system has been used extensively in the construction of retaining walls and is suitable where large, flat, uninterrupted areas have to be formed and where the joints is not considered objectionable.

j. Provision for Stripping and Reuse

Forms should preferably be built in panels for ease of handling. The forms should be detailed with adequate wedges and wrecking strips to allow stripping without damage to the concrete. Where possible, forms should be self-aligning on re-assembly.

k. Form Ties

Factors influencing the choice of ties are :

- strength requirements
- anticipated re-use
- ease and speed of assembly and disassembly
- effect of surface finish.

Form ties must be completely removed or, alternatively, they should leave no residual metal part within 40mm of the surface or within the specified cover, whichever is the larger. Typical form ties are shown in Figure 5.3.

On large surfaces such as retaining walls or piers, ties should be regularly spaced to provide a symmetrical appearance.

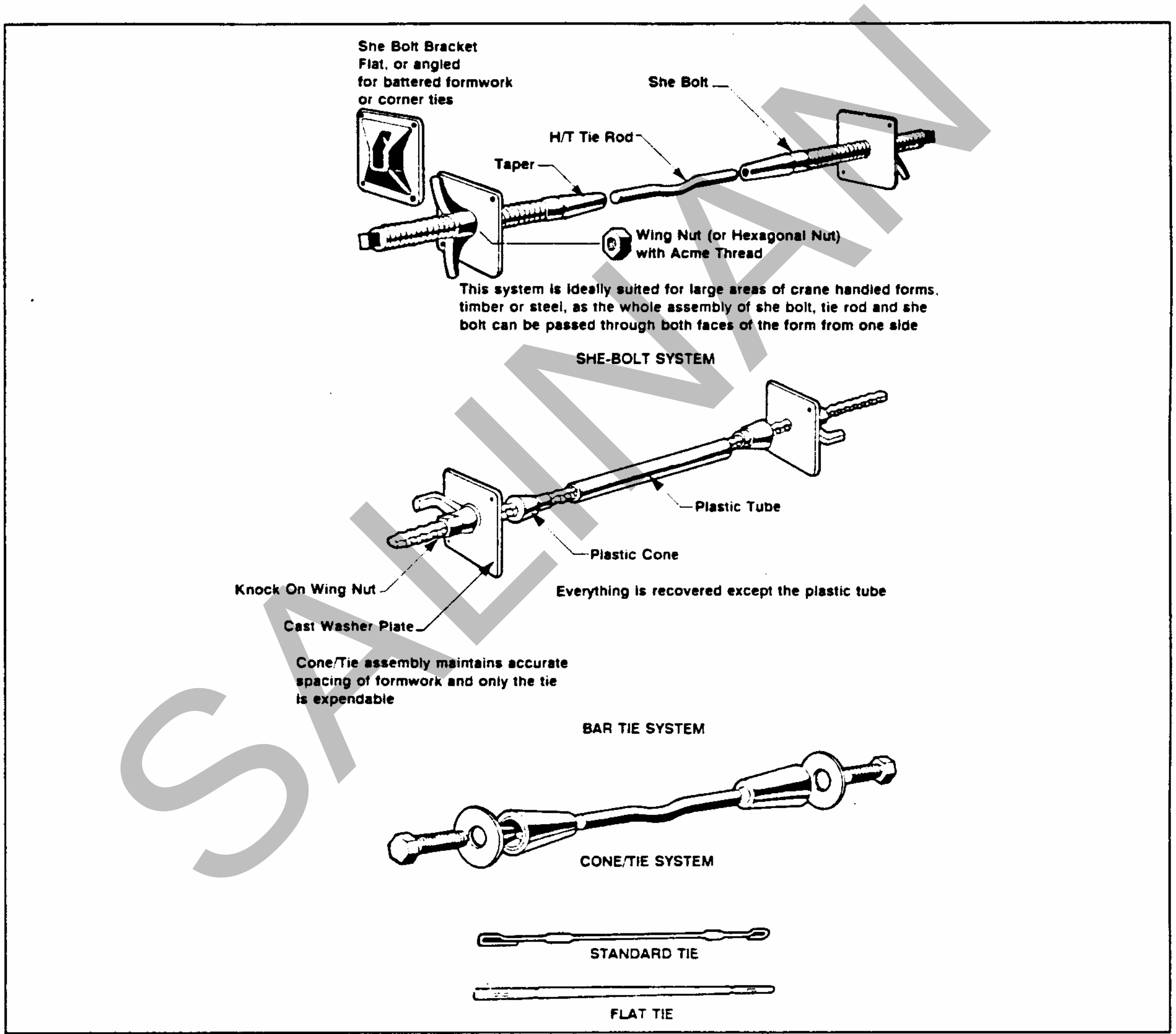


Figure 5.3 - Typical Form Ties

l. Release Agents

Forms should be lightly coated with a release agent either before or after assembly but, in any case, before the reinforcement is placed. Form oil must not be allowed to coat reinforcement or the concrete face at construction joints. Only oils and greases approved as suitable agents should be used for concrete forms. Most general purpose oils and greases are unsatisfactory as they contain components deleterious to concrete.

There is a need to guard against the application of too much form oil, leading to an accumulation of oil in the forms in some locations.

m. Stripping and Cleaning

Formwork should not be stripped until the concrete has achieved the specified strength, as verified by testing or, alternatively, for a specified period of time.

Forms should always be cleaned, oiled and carefully stacked between uses. Similarly, form fittings should be inspected for damage, oiled and stored in drums.

5.2.3 Joints

Joints between formwork panels must be watertight. If not loss of moisture is likely to occur resulting in honeycombing of the concrete surface.

The design of the joint should take into account the following :

(a) sealing the plywood edges with sealant and tightly clamping joints together.

(b) the use of joint filler tapes or suitable proprietary products that are compressed tightly in the formwork assembly and thus take up any minor irregularities in the interface.

(c) the movement of the formwork structure under the pressures of freshly placed concrete should not tend to open any joints.

(d) the edges of abutting panels should be evenly stiffened so that one side of the joint does not deflect more than the other during concreting.

Filler tapes should be non-absorbent with closed cell foams. Tapes can have adhesive on one face with a paper or plastic cover strip. Typical filler tape installation is shown in Figure 5.4.

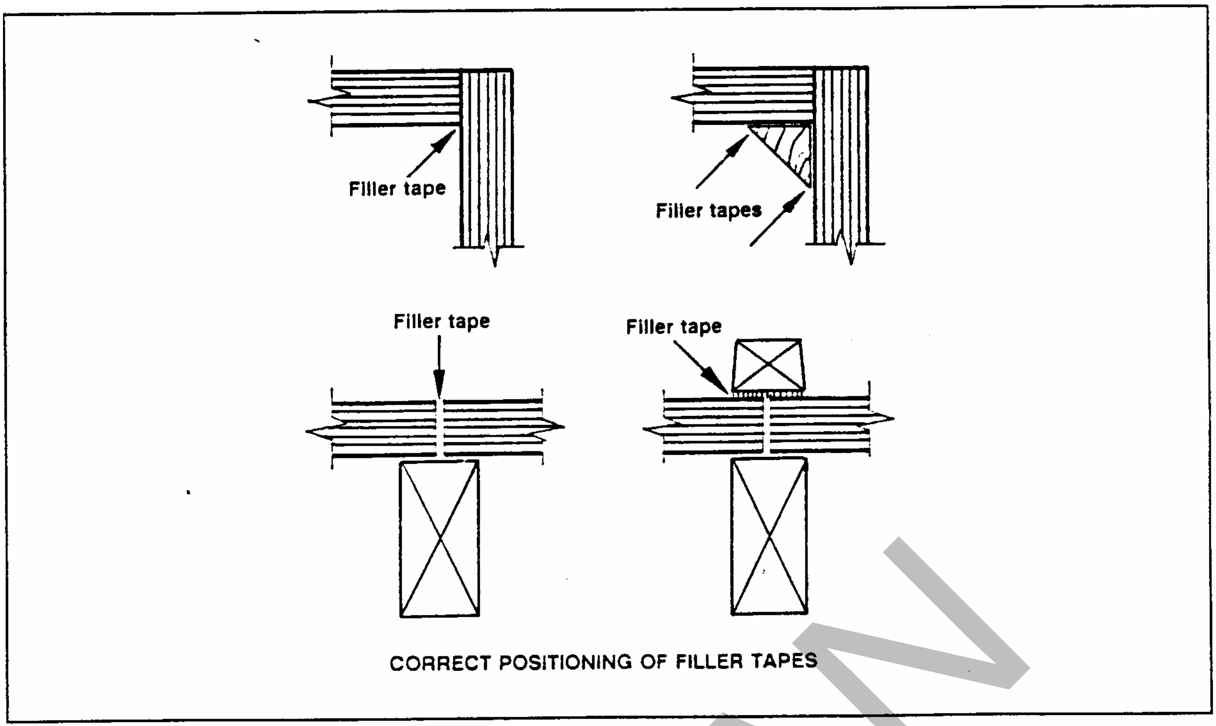


Figure 5.4 - Typical Filler Tape Installation

5.2.4 Falsework

Procedures for falsework design are described in the Construction Supervision Manual.

Problems (deficiencies in falsework design) are often related to foundation problems. The Supervising Engineer should ensure that the Contractor has clearly detailed on his drawing(s) of the proposed falsework how the loads from the falsework will be transferred into the ground.

Falsework on river silts must be constructed so that the bearing capacity of the silt is not exceeded. This may require the use of large mat foundations or even piled foundations. The Contractor should be encouraged to think about how he will construct the falsework at an early stage in the project so that he can take advantage of any equipment brought to site for abutment or pier pile installation.

Excessive deflection of falsework is common and the Supervising Engineer should ensure that the size and spacing of falsework members have been adequately checked. He should ensure that the Contractor complies with the Specification in this regard.

The responsibility clearly remains with the Contractor for the proper design and installation of the falsework but the Supervising Engineer can assist by carrying out checks on the Contractor's proposals.

5.3 REINFORCEMENT

5.3.1 General

The Construction Supervision Manual describes the general requirements for steel reinforcement in reinforced concrete.

5.3.2 Materials

Reinforcement for bridges is usually supplied to conform to the requirements of AASHTO M 31M (ASTM A 615).

Other reinforcement is supplied in accordance with the requirements of the appropriate following standards -

AASHTO M 225 (ASTM A 496) Deformed Steel Wire for Concrete Reinforcement
AASHTO M 32 (ASTM A 82) Cold Drawn Steel Wire for Concrete Reinforcement
AASHTO M 55 (ASTM A 185) Welded Steel Wire Fabric for Concrete Reinforcement

Steel reinforcement must be supplied free from loose mill scale, mortar, loose or thick rust, or any other coating.

Although deformed bar is preferable to plain bar for reinforcement (see Section 5.3.7) most projects in Indonesia use plain bar for all reinforcement.

The use of plain bars for sizes up to and including 10 mm in diameter is acceptable.

5.3.3 Delivery of Steel

Before accepting delivery, the reinforcement should be checked for :

- Correct diameter, shape, quantity of each type, and material type.
- Damage to bars during handling and transport.
- Cleanliness and rust condition.

5.3.4 Stacking on Site

All reinforcement must be stacked clear of the ground on timbers or racks with sufficient supports to avoid bending and kinking. Mud, oil and paint and other contaminants must be prevented from despoiling the reinforcement. If possible, stacking should be arranged by size and length with all similar bars tagged and bundled together.

5.3.5 Bending on Site

Bending should be performed cold with a slow and regular movement. Heating of bars to assist bending should be allowed only with the approval of the Engineer. Specified dimensions within tolerance must be achieved.

5.3.6 Cleaning Prior to Placing in Forms

Very light surface rusting or shallow pitting is generally not a problem with regard to bonding to concrete. However, heavy flaky surface rust, such as can be caused by stacking on the ground for long periods, must be removed before use if rusting has not progressed far enough for the steel to be rejected outright. Badly corroded and deeply pitted bars should not be used.

5.3.7 Bond, Anchorage and Splicing

The effectiveness of reinforced concrete depends upon adequate bond developing between the concrete and steel reinforcement, so that stresses can be transferred from the concrete to the steel. Good bond can be achieved by thoroughly compacting concrete around the clean reinforcing bars. Higher strength concretes generally have better bond to steel, and deformations in the shape of the steel rods, (these bars are called deformed bars) improve bond. Deformed bars can develop approximately twice the bond strength of plain bars and consequently the majority of reinforcing steel used should be deformed bar.

The hard drawn wire used in fabric is very smooth but bond with this type of reinforcement is seldom critical due to the small lengths between cross wires - usually a maximum of 200 mm.

It is usual to extend reinforcing bars beyond the region of tensile stress in a structural member to ensure that the bar has sufficient contact with the concrete beyond the region of stress, so as to develop satisfactory bond strength. Where it is difficult to extend the length of a bar, a bend or hook is used to develop bond with the concrete.

It is usual in reinforced concrete that bars must be joined or spliced to ensure continuity through the structure. This allows changes in bar size or changes in direction to be made and still ensure transfer of tensile forces into the reinforcing. In high walls it is usual to splice bars so as to avoid long lengths of unsupported vertical bars. These can be awkward to handle when constructing the footing and lower parts of the wall.

In general, bars should be placed in such a manner so as to avoid splices occurring at points of maximum stress. Splices should be staggered where possible.

Bars forming a lapped splice must overlap a certain distance. The specification will usually state that the lap length should be a certain number of bar diameters, about 40 bar diameters is normal. If the bars are hooked the lap distance may be reduced.

The lapped bars should be wired together using ordinary tie wire, about 1.6 mm diameter.

Bars may also be joined using mechanical couplings which have the advantage of reducing the congestion of steel at points of overlap. These are relatively expensive and not often used in Indonesia.

5.3.8 Cover to Reinforcement

It cannot be emphasised too strongly that adequate cover of concrete over the reinforcement is of vital importance to the long term structural integrity of the bridge. Corroded steel reinforcement is often seen on the bottom of deck slabs of relatively new bridges.

The concrete outside the reinforcement protects the steel from either rusting or chemical attack. The amount of cover required depends on the conditions of exposure and the nature of the structural element.

In the absence of any other information, cover for footings should be at least 50 mm, beams at least 40 mm and slabs at least 30 mm. Where other values are shown on the Drawings those values should of course be used.

The simplest and cheapest method of ensuring adequate cover is to use concrete spacer blocks. These can be made with excess concrete and tie wire and can be tied to either horizontal steel or vertical steel (see Figure 5.5)

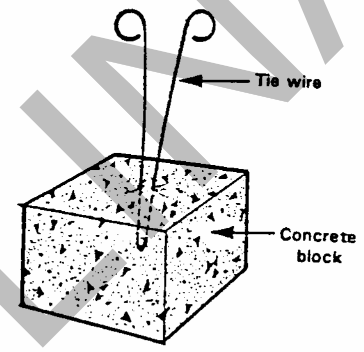


Figure 5.5 - Concrete Spacer Blocks

The responsibility for ensuring that adequate cover to the steel has been achieved rests with the Supervising Engineer. The Specifications are quite clear in this regard and the supervision team must inspect this aspect of concrete construction.

The Contractor should not be permitted to pour any concrete until a complete pre-placement check has been carried out, defects identified and rectified and the consent of the Engineer obtained for concrete placement to proceed.

A form for use in pre-placement checks is included in the Construction Supervision Manual.

5.3.9 Positioning and Tying

Reinforcement must be positioned and tied so that :

- the minimum specified concrete cover is achieved on all faces;
- bars will not be displaced by workmen walking over the steel or by the concrete placing and compaction operations;
- bars will not be displaced by flotation of void formers.
- the spacing and position of bars is achieved.

Tie wire should be approximately 1.6 mm in diameter. It is not usually necessary to tie every intersection point of reinforcing, every second intersection point is normally sufficient.

To obtain the correct cover, the appropriately sized spacer or "bar chair" should be wired securely in place. The chairs may be made of plastic (see Figure 5.6) or dense, high strength concrete blocks, precast with tie wire projecting for secure tying. The tying method must ensure that under vibration, the spacers when placed against vertical forms, cannot rotate on the bar to which they are fixed. Circular type spacers may prevent this occurrence in certain situations.

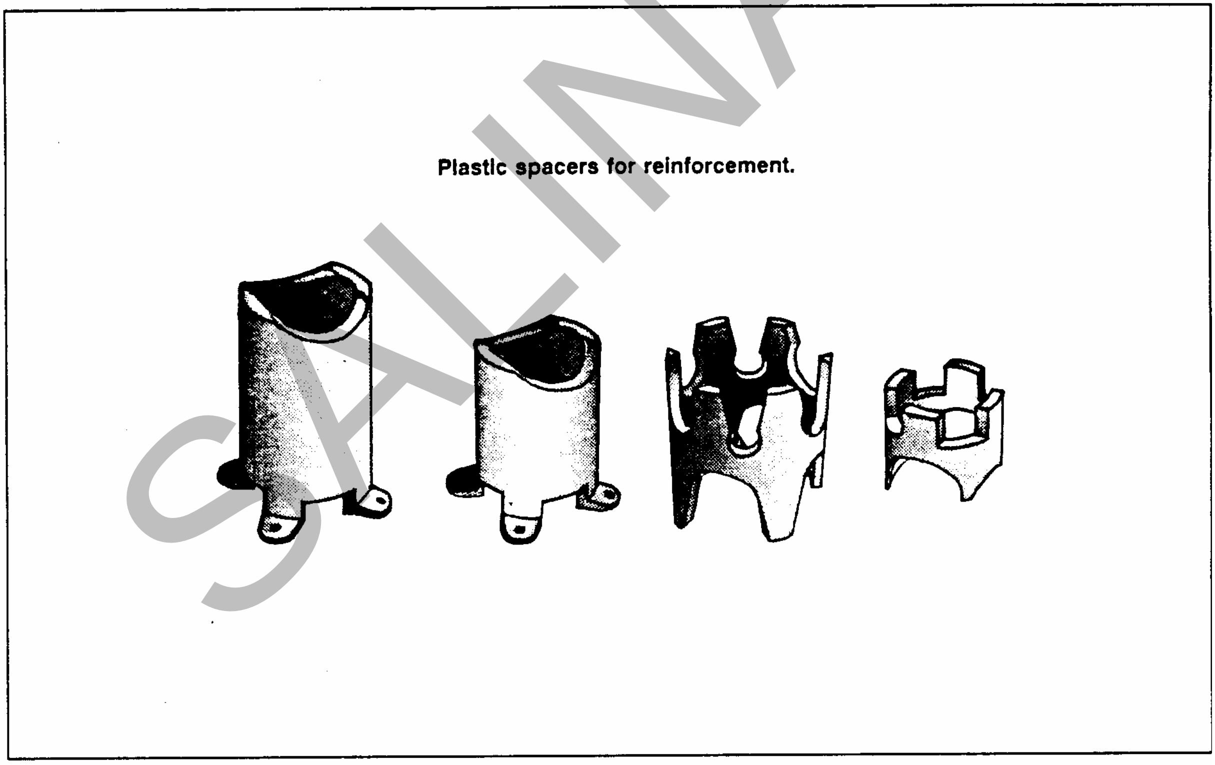


Figure 5.6 - Plastic Bar Chair for Reinforcement

Capped or uncapped steel or wire bar chairs must only be used against previously cast concrete and not against surfaces subsequently exposed to the weather or ground-water. They are best avoided, if possible.

Some plastic bar chairs of closed cylindrical shape inhibit complete compaction of concrete in and around the chair itself and should be avoided.

The practice of extending tie wire from the reinforcement or shear connectors to the outside of the form is widespread but is clearly not permitted by the Specifications. The tie wire will rust and permit water to enter the concrete and expedite rusting of the reinforcement and consequent spalling of the concrete.

If reinforcement is to be partially embedded in concrete the Contractor must ensure that sufficient space is left around the bar or bars to be embedded in future pours to allow the concrete to completely encase the bar. This is particularly important in transverse reinforcement and is often a problem in wall and kerb pours.

5.3.10 Tack Welding of Reinforcement

Use of tack welding to secure reinforcement should be kept to a minimum or preferably avoided altogether. It should not be used without the approval of the Engineer.

Tack welding can, however, facilitate assembly in many instances, for example in the prefabrication of large reinforcement cages. In such cases - if the use of welding is approved - welds should be located in low stress regions of the bars away from bends, carried out by qualified welding operators, and performed in accordance with the requirements of ANSI/AWS D1.4 Structural Welding Code - Reinforcing Steel.

5.4 PLACEMENT OF CONCRETE

5.4.1 General

Placement of concrete generally is covered in the Construction Supervision Manual.

This Section covers a number of areas in greater detail and presents some ideas which will assist to improve the quality of concrete work.

5.4.2 Placement methods

a. General

Precautions must be taken when placing concrete to ensure that:

1. Formwork and reinforcement are not damaged or dislodged, and
2. The concrete does not segregate.

Some commonly used but incorrect placing procedures, together with correct methods are illustrated in Figures 5.7, 5.8 and 5.9. These all tend to cause segregation in the concrete.

The following is a summary of some of the more important points of good placing practice:

(i) Concrete should be placed vertically, and as near as possible to its final position. If spreading is necessary it should be done with shovels and not by causing the concrete to flow.

(ii) Concrete should not be dropped into the forms from an excessive height as it can cause damage and segregation. The height of fall should be kept to a minimum, and where it exceeds 2 metres, a drop chute may be necessary.

(iii) The placing of concrete should start from the corners of formwork and from the lowest level if the surface is sloping.

(iv) Each load of concrete should be placed into the face of the previously deposited concrete, not away from it.

(v) Concrete should be deposited in horizontal layers, and each layer compacted before the next is placed. Each layer should be placed in one continuous operation and before the previous layer has hardened.

The thickness of each layer will depend upon the size and shape of the section, the spacing of reinforcement, the consistency of the concrete and the means of compaction. In reinforced concrete work, the layers will generally be 300 mm thick, and for mass concrete work, 500 mm thick.

(vi) Where a layer of concrete cannot be placed before the previous layer hardens, as on the morning after an overnight stop, a construction joint should be formed.

(vii) Concrete should not be placed in heavy rain without overhead shelter, otherwise the rain may wash cement from the surface.

(viii) In long wall pours where horizontal layers create cold joints, the concrete should be placed to full depth with a sloping face.

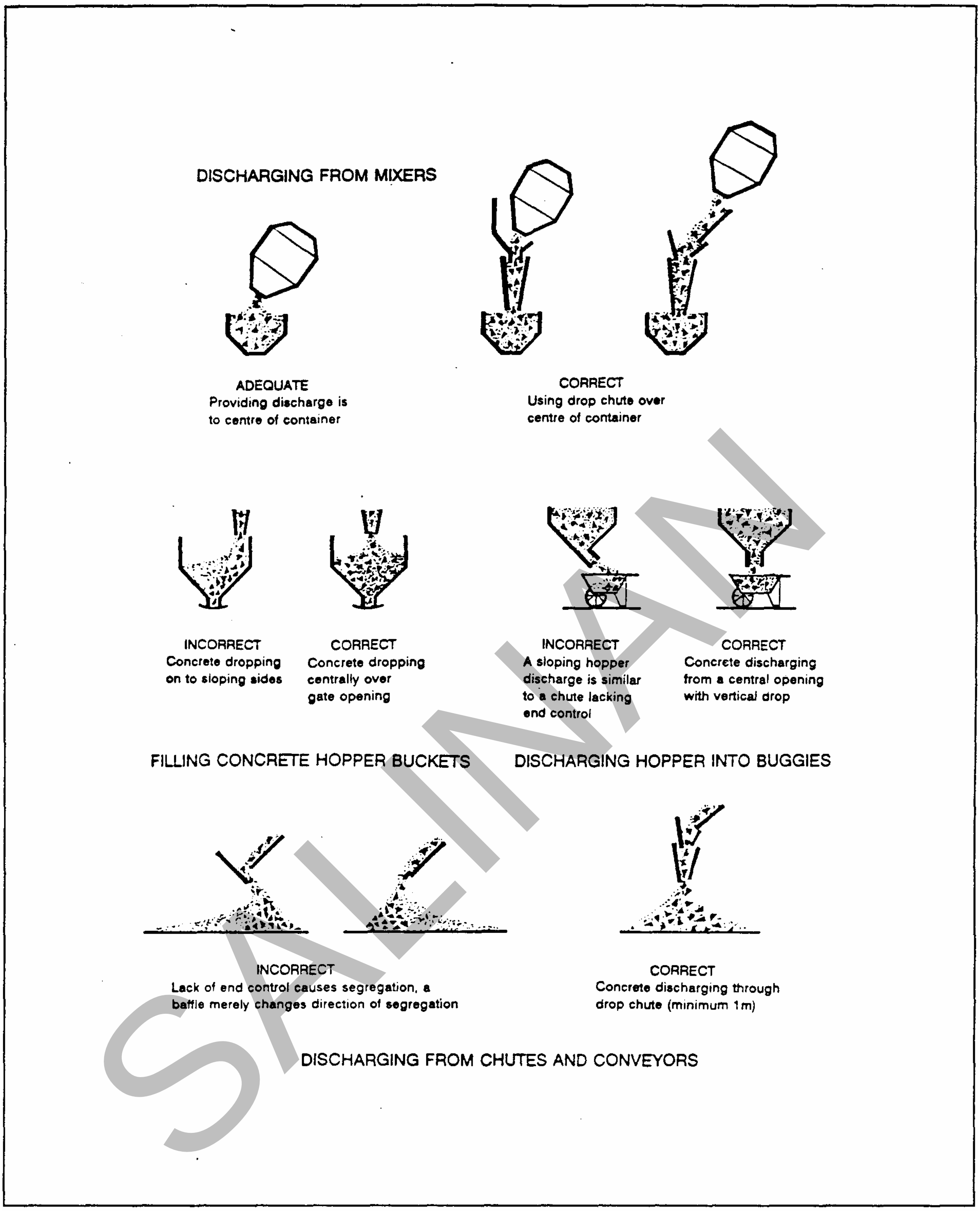


Figure 5.7 - Handling and Transporting Concrete to Avoid Segregation

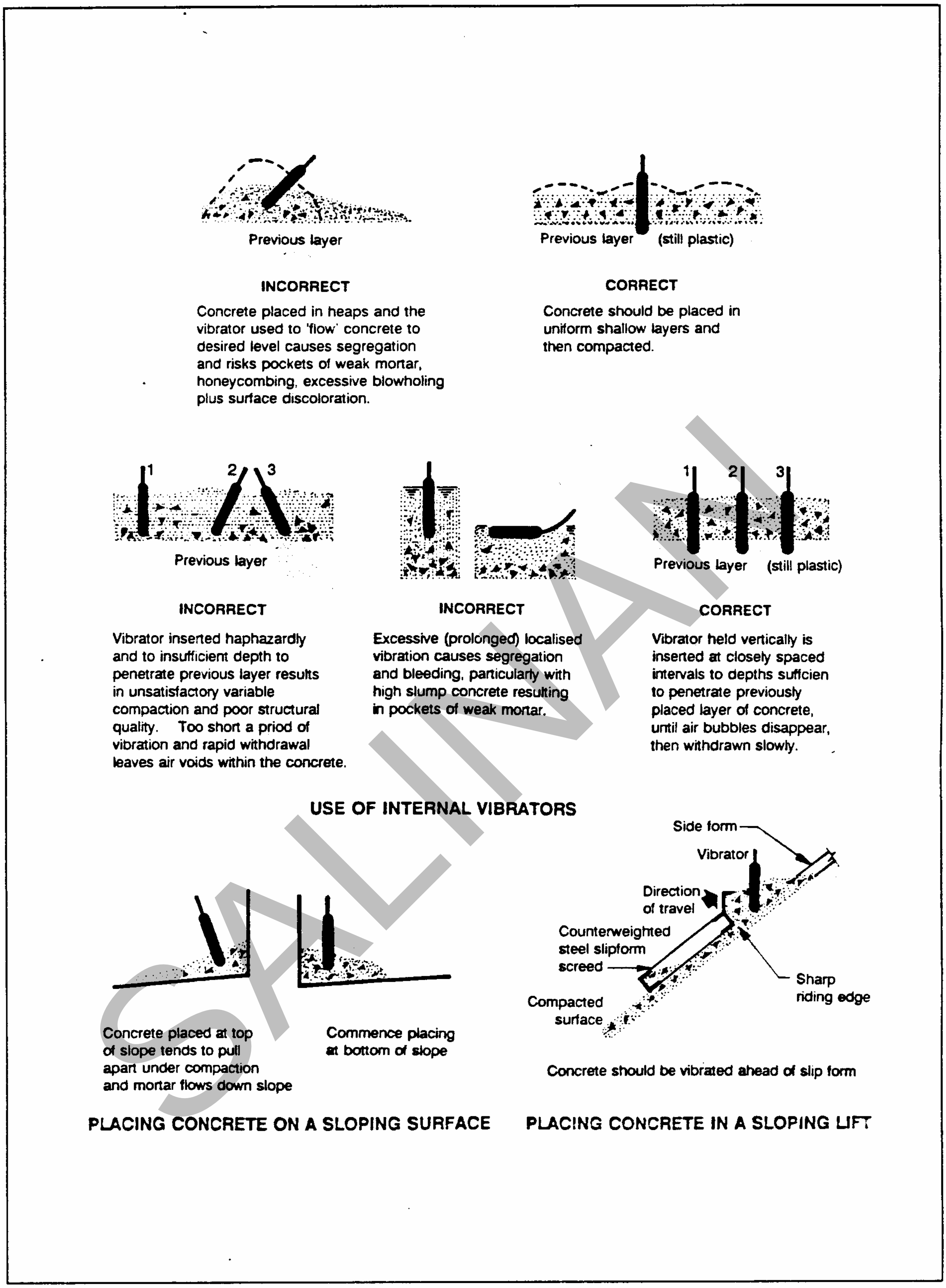


Figure 5.8 - Placing Concrete

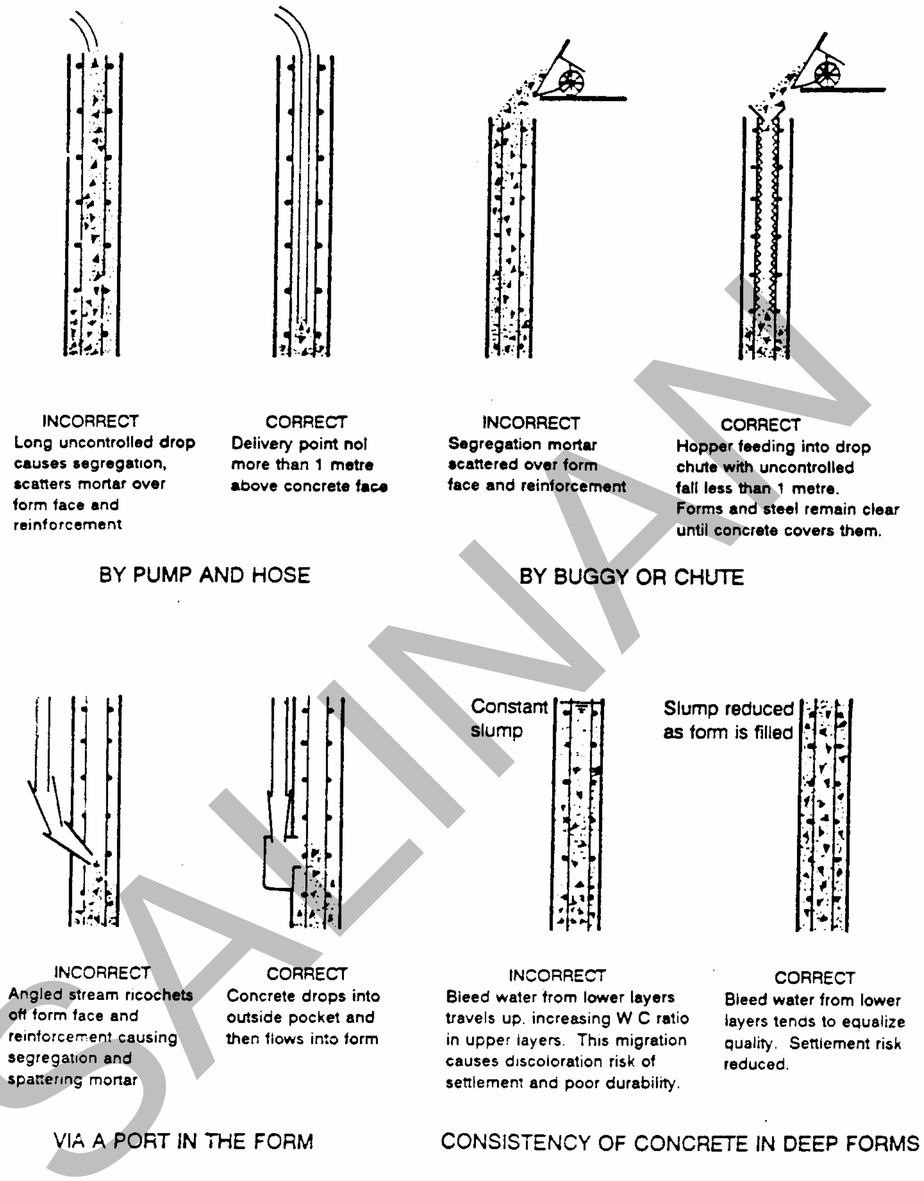


Figure 5.9 - Placing Concrete in Deep Narrow Forms

5.4.3 Placing concrete under water

Placement of concrete under water using the tremie method is described in the Construction Supervision Manual.

5.4.4 Compaction of Concrete

a. General

Compaction of concrete is described in the Construction Supervision Manual. Some additional points are included below.

Only experienced workers should be used for the operation of vibrators. It may be necessary to provide specific training for the operators. Close supervision and clear instructions should be given.

The following points should be emphasised :

- Select the size of vibrator appropriate to the task. Too small may be ineffective; too large may prevent effective penetration in zones of closely spaced reinforcement.
- The "radius of influence" of a 60 mm diameter vibrator in good working order is only about 300 mm. Hence it must be inserted at spacings of less than 600 mm to ensure complete compaction.
- Avoid damage to timber forms. Vibrators can be fitted with rubber caps to minimise damage to forms but the best means of prevention is to employ operators experienced in vibration techniques.
- Vibration loosens formwork ties and other threaded devices. Locknuts and securing pins are used to prevent this occurrence. During placement of concrete one or two workers should continually check the formwork for signs of distress, movement, leaks etc. Small petrol-driven vibrator motors must be secured against overturning by tying or other methods.
- Vibrator heads should be inserted vertically, held for 10 to 20 seconds until air bubbles cease, and then withdrawn slowly.
- When placing concrete in layers, the vibrator should penetrate approximately 150 mm into the previous layer to ensure a good join between the adjacent layers.
- Standby vibrators, in working condition, should be kept in readiness for immediate use should a vibrator breakdown.

b. Hand Compaction

Ordinary hand methods of compaction consist of roding, tamping and spading with suitable tools. This method of compaction is generally inferior to that obtained by the use of vibration.

c. Vibration

Although hand compaction may produce satisfactory results for some purposes, the use of vibration allows the use of drier mixes, resulting in higher strength and reduce shrinkage for given mix proportions.

Immersion Vibrators

This type of vibrator may be mechanically, electrically or pneumatically driven. Pneumatic vibrators have a safe and flexible drive, but since compressed air motors are relatively inefficient and expensive to maintain they may not be economic unless the compressor is otherwise being used on the job. Electric motors operate at constant speed and are conveniently portable, but require a reliable electricity supply.

Immersion vibrators (sometimes referred to as internal or poker vibrators) are probably the most efficient type of vibrator as they vibrate the concrete directly. They are available with heads ranging from 25 mm to 150 mm diameter, the 25 mm diameter head being suitable for small heavily reinforced sections while the 60-70 mm head is the most common general purpose type.

Vibrations are caused by an eccentric shaft which rotates within the vibrator head. Vibrators should be checked regularly either with special instruments or by comparing their effectiveness in concrete alongside a vibrator which is known to be satisfactory.

The vibration of concrete should be done systematically. The concrete should be placed in shallow layers and the vibrator allowed to penetrate each layer fully. The vibrator head should be inserted vertically at points 500 mm apart and then slowly withdrawn to close up the hole left by the vibrator. Vibration at any point should not be prolonged beyond the point at which mortar commences to collect on the surface, usually 5 to 15 seconds. As a general rule, the vibrator should not come nearer than 100 mm to the forms in order to obtain a uniform appearance. If it touches the forms a sand streak can result and the form could be damaged. In shallow sections some consolidation can be obtained by using the vibrator in a sloping or horizontal position.

Form Vibrators

Form vibrators, or external vibrators, are rigidly attached to the outside of forms by means of clamps, and impart oscillations or shaking motion to the forms. This form of vibrator is suitable for small members or narrow and heavily reinforced sections into which it is difficult to insert immersion vibrators. They are often used in conjunction with poker vibrators for a high degree of compaction and good dense surface finish.

Form vibrators are more power-consuming than immersion vibrators, as energy is absorbed by the formwork.

The formwork must be very rigid to withstand the oscillations and corners must be especially tight to prevent loss of cement mortar. The use of form vibrators is usually limited to steel forms.

Concrete should be placed continuously in shallow layers (say 500 mm deep) while the forms are kept vibrating. In this way air holes are removed as the concrete builds up. To ensure the concrete makes proper contact with the side forms towards the top of a lift, it is advisable to use immersion vibrators for the top 500 mm if space allows.

5.4.5 Surface Finish of Concrete

a. General

The efficiency of the inspection process will be judged by the condition and tolerances of the finished surface, which should be free from surface cracking and not have noticeable differences in texture and color.

Provision of a satisfactory unformed concrete surface requires:

- properly proportioned concrete,
- adequate mixing handling and placing methods which will minimise segregation,
- adequate compaction,
- controlled finishing techniques,
- adequate curing.

The concrete mix should be such that there are just sufficient fines (cement and sand) to allow a mortar to be worked to the surface with vibration and a little tooling effort. Too many fines will make finishing easier but will probably lead to surface crazing, as well as being more expensive than a well proportioned mix. Too much water in the mix (high slump) will create delays in finishing, as well as producing a weak surface layer of mortar, resulting in a dusty and crazed surface with low resistance to wear and abrasion.

b. Finishing

Irrespective of the type of surface finish required the essential requirements are:

- Initial finishing should be completed as soon as possible after placing and vibration.
- Final finishing, floating and trowelling should be delayed until the surface is ready - the final work should also be kept to the minimum necessary to produce the required surface.

Finishing operations should not be performed where there is free surface water.

c. Initial Finishing

Immediately after placing and vibration, a screed board (straightedge) is used to quickly level the concrete. The screed board is moved forwards with a sawing motion and in such a manner that a small amount of concrete is always pushed ahead of the screed. Concrete is shovelled up to or away from the front of the screed as is necessary.

After the initial screed, the area should be immediately rechecked for level with a straightedge or template. High and low spots are corrected immediately. High spots by cutting off the surface with a trowel or similar tool and minor low spots screened with mortar taken from fresh concrete. Over-working the surface must be avoided and every effort should be made to ensure the initial finish produces surface levels within specified tolerances. Subsequent finishing operations are used to remove minor imperfections and not to correct levels.

Concrete finishers use a wide variety of special tools some of which are proprietary lines but many are 'home made'. They are all essentially strike-offs, straightedges and floats. To avoid tearing the surface during final finishing, a perforated roller or plate can be used during initial finishing to push the larger aggregate below the surface. This unit should be used sparingly and not on high slump concrete otherwise a mortar-rich surface layer will be created which will be liable to craze and dust.

In many cases the initial finish will be all that is required.

d. Final Finishing

The finishing operations of edging, jointing, floating, trowelling and booming should be delayed as long as possible. Working the surface too soon will create a weak surface and produce laitance. Working the surface too late however will require considerably more finishing effort and may cause crumbling of the concrete surface.

While the correct delay is most important to the production of a quality finish, it is difficult to nominate the specific time as it depends on many variables. Some of these are concrete temperature and age, type of cement, admixture type and the quantities of water, cement and admixtures used. The delay also depends on weather conditions, depth of pour, type of aggregate, type of subgrade, etc.

Generally, finishing begins when the sheen has left the surface (in the case of air entrained concrete there may be little bleed water and no visible sheen and it may be possible to finish this type of concrete after a short delay). Normally when the sheen has left the surface, the concrete will support the weight of a man. He will, however, cause indentations of 5 mm or more, so that finishers must use foot and knee pads to distribute their weight. Where power equipment is used the delay period can be increased so that the concrete can support the weight of a man with little marking.

Note:

1. Finishing should **NOT** be attempted in any area where there is free surface water.

2. Cement should **NOT** be used to dry up surface moisture as this will cause surface cracking later on.

Both these practices will produce dusty, crazed and scabby surfaces with very poor abrasion and wear resistance. In cases of hot, dry and windy weather, evaporation and setting can be too rapid for satisfactory finishing. This applies particularly to high strength mixes or relatively old concrete. To prevent cracking and the difficulties in finishing hardened concrete, precautions will have to be taken (refer to Section 5.4.6d).

e. Floating

After the necessary delay, the surface is floated, normally with a wood float. Floating is the operation of smoothing irregularities in the surface following screeding. Its purpose is to:

- embed large aggregate below the surface
- remove imperfections in the surface
- provide a denser and in some cases a smoother surface
- prepare the surface for another finishing operation eg. trowelling, booming or decorative type finishes
- close minor surface cracks which can occur as the surface dries.

Hand floats are usually made of wood. The wood float produces a rough texture and which can often be the final finish. To further improve skid resistance, hessian or a wire broom can be lightly dragged across the surface.

The hand float is held flat on the surface and moved in a sweeping arc to fill in holes, cut off lumps and smooth ridges.

In some cases it may be necessary to float the surface a second time after some hardening has taken place to impart the desired final texture to the concrete. A more even texture may also be obtained by following up the wood float with a sponge float.

f. Trowelling

Steel trowelling is used to provide a smooth dense and hard surface. This type of surface is durable and easy to clean but can be slippery when wet.

Power floats will reduce labour requirements and finishing time. Power floating is carried out with a rotating steel disc or a power trowel fitted with steel floats.

Following a delay after the completion of floating, steel trowelling can commence. For the first trowelling (whether by hand or power) the trowel blade should be flat on the surface - tilting creates ripples. The hand trowel is used in a sweeping arc motion each pass overlapping one half of the previous pass. The first trowelling may produce a sufficiently fine surface but additional trowelling may be used to increase smoothness and hardness. There should be delays between trowellings with the final pass (usually second or third pass) being made with a narrow trowel. Pressure is exerted on the trowel to compact the paste and form a dense hard surface. This final pass should make a ringing sound as the blade passes over the hardening surface.

The first power trowelling should be followed by hand trowelling to remove small irregularities and to 'touch up' areas in corners or close to obstructions.

If necessary tooled edges and joints should be rerun after trowelling to maintain uniformity and true lines.

Although there are many special hand and power tools designed to improve the speed and quality of the concrete finisher's work, it nevertheless requires a degree of strength, skill and experience to provide quality finishes.

5.4.6 Curing of Concrete

a. General

Curing is covered in the Construction Supervision Manual. Some additional points are included in this Section.

b. Curing Methods

Ponding

On flat surfaces such as pavements and deck slabs, concrete can be cured by ponding. Earth or clay dikes around the perimeter of the concrete surface retain a pond of water within the enclosed area. Ponding is an efficient method for preventing loss of moisture from the concrete and is also effective for maintaining a uniform temperature in the concrete.

Sprinkling

Continuous sprinkling with water is also an excellent method of curing. If sprinkling is done at intervals, care must be taken to prevent the concrete from drying between applications of water. A fine spray of water applied continuously through a system of nozzles provides a constant supply of moisture. This prevents the possibility of 'crazing' or cracking caused by alternative cycles of wetting and drying. Disadvantages of sprinkling may be its cost, the necessity for a drainage system, and possibility of uncomfortable working conditions. The method also requires an adequate supply of water and careful supervision.

Wet Coverings

Wet coverings such as hessian or other moisture-retaining fabrics are extensively used for curing concrete. Such coverings should be placed as soon as the concrete has hardened sufficiently to prevent surface damage. Care should be taken to cover the entire surface, including the edges of slabs such as pavements and footpaths. The covering should be kept continuously moist so that a film of water remains on the concrete surface throughout the curing period.

Wet coverings of earth or sand are effective for curing but in recent years have been largely discontinued due to their high cost and possible discolouration of the concrete. The method is often useful on small jobs. Moist earth or sand should be evenly distributed over the surface of the concrete in a layer about 50 mm thick. It must be kept continuously wet.

Impermeable Coverings

Waterproof paper and plastic sheet are an efficient means of curing horizontal surfaces and structural concrete of relatively simple shapes.

They ensure suitable continuing hydration of cement by preventing loss of moisture from the concrete. They should be applied as soon as the concrete has hardened sufficiently to prevent surface damage. Edges of adjacent sheets should be overlapped at least 100 mm and tightly sealed.

Covering also provides some protection to the concrete against damage from subsequent construction activity.

In some cases plastic sheets may cause discolouration of hardened concrete. Where this is unacceptable some other curing method is advisable.

Curing Compounds

Liquid membrane curing compounds limit evaporation of moisture from the concrete.

They can be effective curing materials when used correctly. They are suitable for curing not only fresh concrete, but may also be used for further curing of concrete after removal of forms or after initial moist curing.

Clear or translucent compounds may contain a dye which fades out after application. The colour ensures coverage of the exposed concrete surface. During hot sunny days, white-pigmented compounds are most effective since they reflect the sun's rays, thereby reducing the concrete temperature.

Curing compounds are applied by hand-operated or power-driven spray equipment. The concrete surface to be cured should be moist when the coating is applied. Normally only one coat is applied in a smooth, even texture, but two coats may be necessary to ensure complete coverage. A second coat, when used, should be applied at right angles to the first.

Curing compounds can be used to prevent bond between hardened and fresh concrete, consequently they should not be used if bond is necessary.

Forms Left in Place

Forms provide satisfactory protection against loss of moisture provided that top exposed concrete surfaces are kept wet. Use of a soak-hose is an excellent means of doing this. Under these conditions forms should be left on the concrete as long as practicable. Wood forms left in place should be kept moist by sprinkling, especially during hot, dry weather. Unless wood forms are kept moist, they should be removed as soon as practicable and other methods of curing started without delay. Where fine off-form finishes are required, forms should be stripped as early as practicable and other (non-staining) curing methods used.

Steam Curing

Steam curing is normally only used for precast concrete. It is a useful means of providing excess moisture for curing and elevated temperature for accelerated strength development.

c. Length of Curing Period

The length of time that concrete should be protected against loss of moisture is dependent upon the type of cement, mix proportions, required strength, size and shape of the concrete mass, weather and future exposure conditions. This period may be a month or longer for lean concrete mixes used in structures such as dams; conversely, it may be only a few days for richer mixes, especially if high early strength cement is used. Since all the desirable properties of concrete are improved by curing, the curing period should be as long as practicable in all cases.

Since the rate of hydration is influenced by cement composition and fineness, the curing period should be prolonged for concretes made with cements of slow strength gain characteristics.

For most structural uses, the curing period for cast-in-place concrete is usually three days to two weeks, depending on conditions, eg. temperatures, cement type, mix proportions. The most commonly specified period is 7 days.

d. Prevention of Plastic Shrinkage Cracking

Cracking that sometimes occurs in the surface of fresh concrete soon after it has been placed and while it is still plastic is called 'plastic shrinkage cracking'. These cracks appear mostly on horizontal surfaces and may be practically eliminated if appropriate measures are taken to minimise the causes.

The simple precautions listed below can minimise the occurrence of plastic shrinkage cracking:

1. Dampen the subgrade and forms but ensure any excess moisture is removed before concreting commences.
2. Erect sunshades to reduce concrete surface temperatures.
3. Protect the concrete with temporary wet coverings during any delay between placing and finishing.
4. Reduce time between placing and start of curing by improved construction procedures. Evaporation in this period can be reduced by spraying a film of a special compound over the concrete surface. These compounds are aliphatic alcohols such as cetyl alcohol and are available under a number of trade names.

5. Protect the concrete during the first few hours after placing and finishing to minimise evaporation. This is most important to avoid checking and cracking. Application of moisture to the surface, using a fog spray nozzle, is an effective means of preventing evaporation from the concrete. This should be used until a suitable curing material such as a curing compound, wet hessian, or curing paper can be applied.

6. If unavoidable conditions create rapid generation of plastic cracks before the concrete has hardened revibration and refloating the surface will close up the cracks provided this work is followed by adequate curing. Revibration and refloating are normally carried out at about the time of initial set - say 1.5 hours for 27°C concrete temperature.

5.4.7 Quality of Concrete

This section discusses some issues relating to quality control of concrete.

Testing of concrete is described in the Construction Supervision Manual and control of concrete mix design is covered in Section 4 of this Manual.

Cubes and Cylinders

It should be noted that while the specifications allow the use of cubes for checking compressive strength of concrete specimens, the AASHTO standard T 23 is based on 150 mm x 300 mm standard cylinders. If cubes are used, it is better to use the British Standard BS 1881 Part 116: Method for determination of compressive strength of concrete cubes.

The use of cylinders is considered to be a more accurate measure of the strength of the concrete as the height of the cubes (150 mm) means that any end restraint or eccentricity can influence the load at which the concrete crushes. The cylinders are 300 mm high and the central zone of the concrete specimen is relatively independent of these effects.

Curing of Test Specimens

It is essential that the test specimens be properly taken and then cured in such a manner as to reflect the state of the concrete in the field. There is a tendency to take the specimens and then simply leave them in the shade without the appropriate curing.

Use of Rebound Hammer

The use of an impact device such as the Schmidt rebound hammer for routine testing is not recommended. This device is not as reliable as taking test specimens of the actual concrete cast, followed by proper curing and testing at the appropriate time. The principle of the rebound hammer is to give a measure of the stiffness of the concrete and relate that property to strength. There is a need to calibrate the hammer for each particular concrete mix as mixes with different stiffnesses (and hence different rebound test results) may be of the same strength and mixes with the same stiffness (and hence the same rebound test result) may be of different strength.

Accordingly the rebound hammer should only be used after suitable calibration with compressive strength test results on the concrete being tested. It should not be used to compare the strengths of concretes of different mix designs.

5.4.8 Joints

a. General

The magnitude of most concrete construction is such that interruptions will inevitably occur in the placing of concrete. If the interruption is of such a duration that it allows the concrete to stiffen to the extent that it cannot be worked, then a joint must be formed. There will also be occasions where for structural reasons it is considered necessary to purposely break the continuity of concrete placing and to construct a joint.

Joints can be of two general types:

1. Those which allow no relative movement of the concrete on either side of them

2. Those which do allow relative movement.

The first type of joint aims at bonding the new concrete to the hardened concrete in such a manner that the hardened concrete appears to be monolithic and homogeneous across the joint. This is called a *Construction Joint*. In practice it is very difficult to obtain complete adhesion, with the result that there is usually a plane of weakness at construction joints. Whenever possible, construction joints should be positioned at locations where contraction or other type of joint is required.

Joints which allow relative movement of the concrete on either side of them are named according to the type of movement they allow:

1. **Contraction Joints** allow the concrete to shrink away from the plane of the joint while restraining relative movement in other directions.

2. **Expansion Joints** separate the two mating concrete faces sufficiently to allow expansion towards the plane of the joint. This type of joint also allows contraction, but prevents movement in other directions.

3. **Isolation Joints** completely separate the two mating concrete faces and allow complete freedom of relative movement.

Careful consideration must be given to the need for joints in all types of concrete structures. The positioning of joints and the type of joint is usually governed by structural requirements. In some structures the need for making watertight joints is a primary consideration. To ensure that joints behave in the desired manner careful attention must be paid to the details of their design and construction.

b. Construction Joints

A construction joint is a concrete-to-concrete joint made in such a manner that the faces of the new and old concrete adhere sufficiently to prevent any relative movement across the joint.

While unscheduled interruptions will usually occur during the placing of concrete necessitating construction joints, some breaks in the continuity of concrete placing can be foreseen either during the design stage or just prior to the beginning of construction, thus allowing the position of many joints to be planned. Good planning will aim to interrupt concreting at a suitable location to form a contraction or other type of joint, thus eliminating the construction joint. Where this cannot be done, construction joints should be planned for positions in the structure where the presence of a plane of weakness will have least structural effect. A faulty joint can weaken a structure or allow the entry of water which may disfigure the concrete with unsightly efflorescence, as well as causing dampness and possible rusting of reinforcement steel.

Location of Joints

Where construction joints are to be made in structural members, their position must be approved by the Engineer. These joints are usually located where shear forces are a minimum. Areas of maximum bending moment should be avoided. In beams and slabs, the minimum shear is usually in the middle third of the span and so joints should be located close to the third points of the span (within one sixth of the span).

Construction joints should only be made as indicated on the drawings. Approval from the Engineer must always be obtained before changing joint locations or adding extra joints.

As a general rule, horizontal joints are never allowed in slabs and joints and never allowed near beam supports or over any other beam, column or wall. Shearing stresses in these locations are generally high.

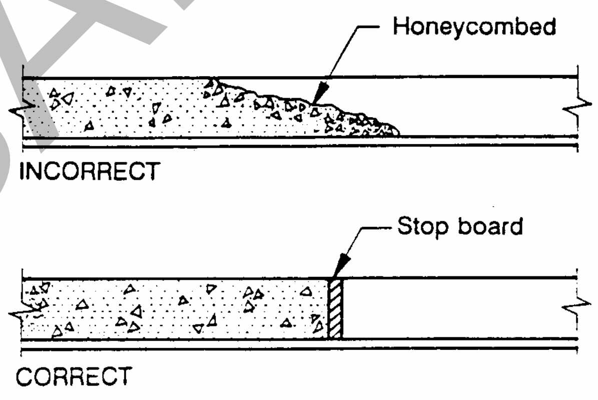


Figure 5.10 - Vertical Construction Joints In Slabs

Making Vertical Joints

When a construction joint has to be made in a beam or slab, a stop-end or bulkhead (see Figure 5.10) should be used to ensure that a vertical construction joint is formed. If the concrete is left free, it will adopt its natural angle of repose and will be impossible to compact thoroughly. This will result in a weak, porous joint. To assist the transfer of load across a vertical construction joint, dowels or a keyway to aid mechanical bonding can be placed at about mid-depth in the joint. Such devices are advisable in sections over 150 mm deep. Reinforcement must not be cut at a construction joint, and so the stop-end boards must either be built in segments or slotted to cater for reinforcement to pass through them without allowing loss of mortar.

Methods of preparing the old joint surface to receive the new concrete depend on the age and condition of the old concrete:

1. Where the forms are removed sufficiently quickly to allow the construction joint to be made within about four hours of placing the initial concrete at the joint, the only preparation the joint surface needs is wire brushing to roughen, it, followed by the removal of all loose material before new concrete is placed.

2. Where the joint surface is older than four hours at the time of making the construction joint, sand blasting, high pressure water jets, scabbling or similar may have to be used in lieu of wire brushing to expose the coarse aggregate. The severe methods such as scabbling should only be used where the concrete has gained sufficient strength to resist loosening of the coarse aggregate. All loose material should then be washed or blown away. The new concrete should be well vibrated against the joint.

   Note: The use of a cement grout layer to improve bond in vertical or horizontal construction joints is not recommended. The strength of a construction joint depends primarily on the preparation of the old surface. A proprietary *wet-to-dry* epoxy may be used on the face of the old concrete.

Making Horizontal Joints

During the placing and compaction of concrete, a film of laitance and a layer of porous concrete directly beneath it tends to form on the top horizontal surface of the fresh concrete. This weak surface material must be removed before a sound construction joint can be made. Typical wall construction joints are shown in Figure 5.11.

As with vertical joints the method of preparing the surface will depend on its age and condition:

1. Where the joint is not more than four hours old at the time of making the joint, the removal of the old surface to a sufficient depth to reveal sound concrete is the only preparation required. The new concrete should be just sufficiently plastic to flow sluggishly into position when vibrated. Mixes which are too dry will not bond thoroughly, and mixes which are too wet may segregate and form an excessive amount of laitance.

2. Where the joint is being made to concrete which is older than four hours, the surface layer must be removed as before. If the concrete is less than three days old this can be relative easy. The surface should then be wire brushed, bush hammered or sand blasted to lightly expose the surface of aggregate without undercutting it. Before concreting, the surface should be washed clean of loose material or any further laitance which develops.

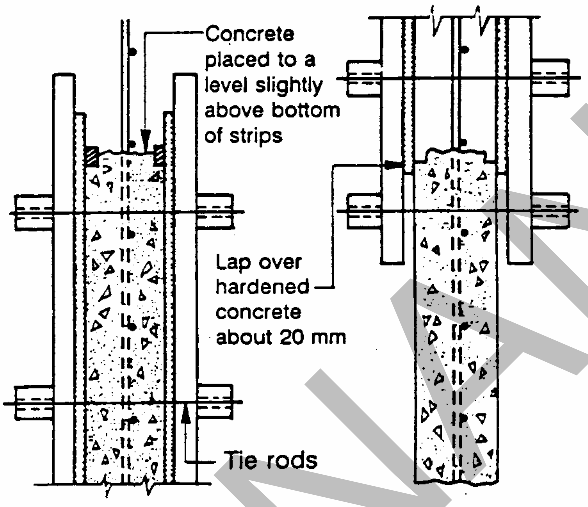


Figure 5.11 - Wall Construction Joints

c. Contraction Joints

A contraction joint is a concrete-to-concrete joint, made in such a manner that the concrete is free to shrink away from the plane of the joint while all other relative movement across the joint face is prevented.

Construction of Formed Contraction Joints

Contraction joints are made by purposely creating a vertical plane of weakness in the slab or wall. This joint is sometimes formed as a keyed joint to control differential movement across the plane of the joint, although dowels, with one end coated so as to be free to slide, are often used as an additional shear control. The bond between the new and existing concrete at a contraction joint must be broken. This can be done painting the joint surface with a curing compound, bituminous emulsion, form oil or similar bond-breaking material.

d. Control Joints

A control joint or dummy contraction joint is a plane of weakness which is built into the structure by means of a groove. This joint functions as a contraction joint in that it serves to concentrate shrinkage stresses at the weakened section, and thereby localises shrinkage cracks to beneath the groove.

Mechanical interlock across the irregular crack serves to transfer loads across the joint and to prevent relative movement in the plane of the joints.

Construction of Control Joints

Control joints can be made at any one of the three different stages during construction.

1. They can be made while the concrete is being placed by inserting a premoulded strip to form the groove.

2. After the concrete has been placed and is being finished the joint can be made with a suitable grooving tool. Such joints will have rounded edges and extend into the slab for one sixth to one quarter of the slab depth.

3. After the concrete has hardened, a sawn control joint can be made. The joint should be made as early as possible prior to drying shrinkage.

e. Expansion Joints

An expansion joint creates a gap between two mating concrete surfaces so as to allow expansion of the concrete into the gap. The gap is usually filled with a compressible filler such as rubber, plastic, cork or mastic. All relative movement in the plane of the joint is prevented.

Expansion joints are probably the most expensive type of joint to make. Designers should give careful consideration to the need for expansion joints and their spacing.

An increase in the concrete temperature will generally increase the concrete length e.g. a temperature rise of 10°C in a 10 metre length of unrestrained concrete will result in an expansion of about 1 mm.

Construction of Expansion Joints

Simple expansion joints, by definition, allow only expansion and contraction of the concrete. Accordingly, provision must be made to prevent movements in the plane of the joint. Thus some means must be used to transfer loads across an expansion joint. This could be done by forming a keyed joint, but the key would make the insertion of an expansion joint filler difficult.

Loads are usually transferred across the joint by means of dowel bars. Half the length of each bar is embedded in the concrete which is placed first at the joint. The other half is treated so as to prevent it bonding to the new concrete at the joint. Some methods used are to grease or bitumen coat half of each bar. The end of the treated half of the bar is then capped to make a socket into which the bar can move when expansion of the concrete occurs.

f. Isolation Joints

An isolation joint creates a gap between mating concrete surfaces so as to allow complete freedom of independent movement on either side of the joint. The gap is usually filled with a pliable filler such as fibreboard, cork, mastic, plastic or rubber.

Many expansion joints on bridges are in fact isolation joints as well.

Some typical joints are shown in Figure 5.12.

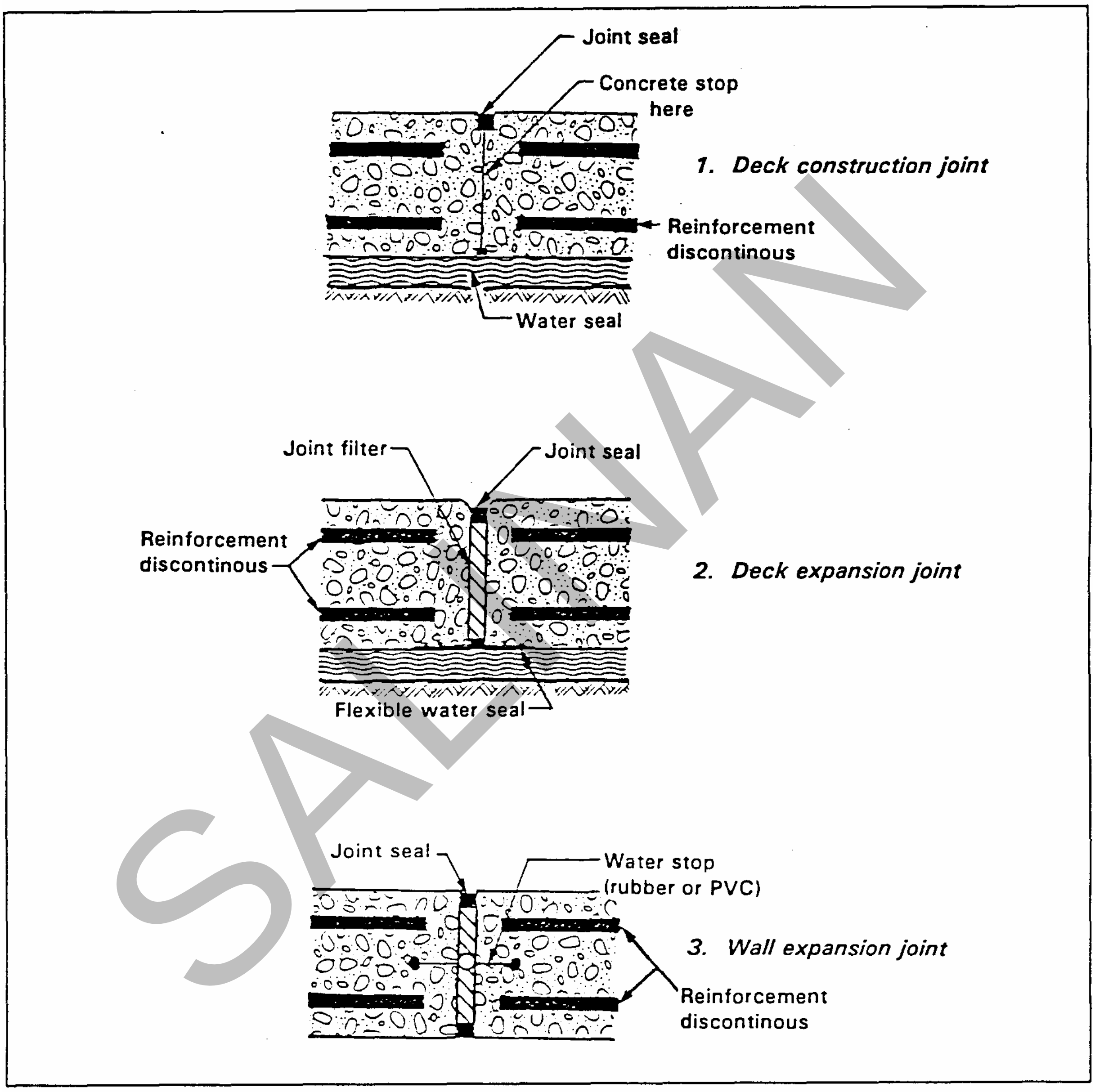


Figure 5.12 Typical Concrete Joints

5.5 PRESTRESSED CONCRETE

5.5.1 General

Section 16 of the Construction Supervision Manual, the I.A.C.B.P. *Prestressed Concrete Training Course Manual* and the NAASRA booklet *Prestressed Concrete Inspection Practice* should also be consulted for additional information on Prestressed Concrete Construction methods.

5.5.2 Ducting for Prestressing Tendons

The various forms of ducting used for prestressing tendons are usually proprietary items, and may be specified or described on the Drawings or be a part of the tensioning system. The ducting is often made of very light gauge steel to provide flexibility and for economy, and is easily damaged during handling, storage, fixing or during the concreting process.

The accurate location of ducts is critical. They must be accurately positioned and fixed to the reinforcement at close intervals, usually with tie-wire, tightly enough to prevent movement but not enough to deform the ducting. Ducting will float in wet concrete, so that it must be tied against upward movement as well as being supported from underneath.

The reinforcement may include saddles or locating bars to ensure accuracy. Ducts should be stiffened either by placing the stressing tendons in the ducts or by other suitable means (such as PVC or steel pipes) to minimise distortion of, or damage to, the ducting.

The duct joints must be carefully sealed to prevent slurry from the vibrated concrete entering the ducts.

Workmen operating internal vibrators must be carefully instructed and supervised, as the ducting can be easily damaged by the impact of the vibrator head.

Metal ducting is usually galvanised. An internal lead coating is sometimes provided if it is necessary to reduce friction losses in areas of acute tendon curvature.

Vents need to be provided at regular intervals in all ducts and specifically at all high and low points. The vents are usually about 20 mm in diameter and should be provided with a plug with which the vent is closed after air-free grout begins to flow. The vents must extend for some distance (around 300 mm is sufficient) beyond the surface of the concrete.

Vents are also necessary at both ends of each duct for grouting. Each vent must have a plug valve which will withstand 700 kPa for at least one minute without allowing either water or air to bleed away.

5.5.3 Tendons and Anchorages

Tendons for prestressing may consist of high tensile wire, strand or bar. Drawings and specifications may be drawn up to suit a particular prestressing system. Alternative systems may be permitted subject to approval by the Engineer and provided that all details of the alternative system are submitted by the Contractor at the time of bid.

Materials and equipment are often supplied by a subcontractor who may also stress and grout the member if required. Test information and samples of the wires, strands or bars are taken and checked. Load-extension graphs provided by the manufacturer or the testing authority for each batch are used for comparing actual and theoretical forces in the strands and elongations during stressing. It is important that tendons in multi-strand or wire systems be made up of strands or wires from the same batch or from batches with the same Young's Modulus.

It is essential that tendons be kept clean and safe from damage, kinks or bends. A very small nick caused by careless storage or handling can result in stress concentrations which may lead to a wire breaking during stressing or after installation is complete. Welding and flame cutting near tendons must be forbidden as this, too, can cause a tendon to break due to a stray arc or a drop of molten metal. Stressing material should never be dragged along the ground, walked on, run over by site plant or stored in situations where it may come into contact with grease, mud, paint or other coatings.

Anchorages must be inspected closely for general quality, finish and damage before they are installed.

Where tendons are pretensioned, the Drawings will show the location and details of saddles or other devices, if necessary, to maintain the tendons in their correct position until the concrete has set. The devices must be accurately positioned and be adequate to resist the calculated loads.

Tendons must be kept clean during installation and a solvent-soaked rag can be used to remove form oil or other undesirable coatings. If parts of the tendons are to be debonded a plastic sheath may be used with the ends sealed by tape or a proprietary tape may be wrapped around the debonded sections, usually in two layers, each wound in opposite directions. It is good practice to cast concrete as soon as possible after stressing.

The individual strands of post-tensioned tendons should not twist within the cable and for mono-strand systems spacers (at 1 m centres) must be used to achieve this.

Where tendons have been placed within ducts before concreting they should be pulled backwards and forwards about 300 mm each way following concreting to ensure that they are free and to break the bond with any slurry which may have leaked into the duct. This should generally be done as soon as possible after the concrete has taken its initial set, but may be done earlier in the case of insitu joints between precast segments. If there is any reason to believe that there has been any leakage into the ducts during concreting, they should be flushed out with water, then blown out with oil-free compressed air.

If a dead anchor system is used for the tendons it will not be possible to move the tendons after concreting. When a dead anchor system is used it is important that concrete be cast as soon as possible after placing the tendons to avoid undue exposure which could initiate rusting of the tendons in the area beyond the duct.

Anchorages must be set square to the line of the tendons. Templates have been found to be useful for locating and checking the position and alignment of the anchorage before and after concreting.

5.5.4 Stressing

a. General

The stressing of high tensile steel tendons is a very important operation which can, at times, be complex. It can also be potentially dangerous. It is essential therefore that the supervisors and the operators are properly experienced and have equipment which is reliable and well-maintained. Strict safety precautions must be taken during the stressing operation. The jacks must be appropriate for the anchorage system used, positioned centrally over the line of tensioning and sit squarely in the anchorage, and operated within their specified capacity.

Before stressing, the equipment should be checked for current calibration certificates from an acceptable laboratory. The ends of the wires, cables or bars must be cleaned of any material which might affect the grip of anchorage devices, which must themselves be clean.

In post-tensioning work the cable must be free to move within the duct, which should have been blown out with clean oil-free compressed air before placing the cable. Check that the anchorage head is centred accurately over the cast-in anchor plate. Stressing of cables should follow as soon as possible after placing the cable in the ducts. A delay of two weeks or more will necessitate the removal of the cable to check for contamination or rust.

The drawings and specifications set out the required prestressing loads and the order in which they should be applied. Proposed deviations should be referred to the Engineer to ensure that the structure does not suffer unacceptable loads. In the same way the instructions or guidelines provided by the proprietors of the stressing system being used must be carefully followed by the operators.

The concrete strength of the member must be checked before prestressing for post-tensioned members or before transfer of prestress for pre-tensioned members to ensure that the concrete has gained the required strength.

b. Stressing Procedure

i. General

Load extension graphs are used to calculate a theoretical elongation which for deflected pre-tensioned strands stressed in the deflected position and post-tensioned tendons should include allowance for friction losses. The losses should be confirmed by site test where possible.

The tendon load is generally measured by a dynamometer or a calibrated jack and gauge system and checked by comparing the actual elongations with the calculated values. Prestressing loads must be applied in the order shown and once started it is advisable that loading should proceed without delay until the member is completely prestressed. An initial load should be applied to all tendons to remove slack before tensioning. Allowance for this load can be made either by drawing a zero correction graph or by estimating and comparing elongations between the initial load and final load only. If the actual elongations differ by more than 5% from those calculated, check both equipment and material before releasing and re-applying load. When reloading, remember that the load-extension performance of the stressing material will not be the same as for first loading. If it is considered that friction losses are too great, tendons may be lubricated using water soluble oils only, or the loading may be applied from both ends.

All stressing should be recorded on the appropriate stressing records sheet together with all relevant information about the tendons, grout etc.

ii. Tensioning

The Contractor should supply details of the gauge pressure to used during tensioning, the extensions calculated for the tendons of particular coils, and the allowances he has made for losses in the anchorages, hold-ups, hold-downs and at the splice connectors.

The Supervising Engineer should ensure that the correct tensioning equipment is to be used for prestressing. In particular all jacks and gauges should be examined and their serial numbers noted, because similar types of jacks and gauges can vary considerably in performance.

Before tensioning is started, all jacks should be warmed up by pumping the ram in and out several times. Each tendon is to be numbered and the numbered tendon pattern sketched on the tensioning records. When the tendon is first pulled through to hold-up, hold-downs, and headstock billets, it will be slack and sagging between them. It is necessary therefore, to apply force to the tendon to take up the slack before the main tensioning activity commences. This operation is called 'sag pull up", and the pressure recorded on the gauge when this done is referred to as the "sag pull up pressure" or "S.P.U". The value of this pressure should be determined by observation of the tendons when tensioning occurs, and will vary with the prestressing arrangements and the length of the prestressing bed. However, in most cases, a gauge pressure of approximately 7 Mpa is adequate.

iii. Tensioning Procedure

The first tendon should be tensioned up to the sag pull up pressure, as indicated by the pressure gauge, and the tendon marked '1' at the tensioning end, as shown in Figure 5.13. At the same time marking is done at all splices and at the ends of the tendons, as illustrated in Figures 5.14 and 5.15. These marks are used for later reference in calculating the true measured extension. It is essential that the sag pull up pressure be read accurately. Any inaccuracy in observing this pressure will introduce an error into the extension required at full load.

The tendon should then be tensioned up to the specified jacking pressure, using the pressure gauge, and the tendon marked '2' at the tensioning end, as shown in Figure 5.13. The jack pressure is then released to allow the tendons to be gripped by the wedges at the headstock. The reduction in the extension from that at full jacking pressure is due to losses in the headstock anchorages after lock-off. This loss at the anchorages must be recorded and compared with estimated values. A loss in the extension of the tendon at the headstock anchorage, when the tendon is gripped by the wedges is referred to as the loss in the anchorage, and is a combination of anchorage slip and draw-in. The tensioning process should be repeated until all tendons have been tensioned. The first two tendons are then re-tensioned to determine the lift-off pressure when the cone lifts off the billet plate. It may be necessary to use a detensioning bridge to satisfactorily determine this lift-off pressure. The lift-off pressure of the tendons must be at last equal to the calculated pressure specified. If the lift-off pressure is less than that specified, this is an indication that the prestressing bed has shortened, or that anchorage slip has occurred, and must be reported to the Engineer. After the tensioning has been completed, a check of the end forms and the steel reinforcement should be made, to ensure that the tendons have not fouled them.

iv. Extensions

The "true" measured extension of the tendon is the measured extension between the marks '1' and '2' of Figure 5.13 less the following:

(i) Measured lock off in the anchorage in the headstock-Figure 5.13.

(ii) Slippage in the anchorage at the dead end-Figure 5.15.

(iii) Total slippage in the wedges at the splices-Figure 5.15.

(iv) Shortening of the casting bed.

(v) Local movement of the dead end sandwich plates and the reference point used for measuring extensions at the tensioning end.

Items (iv) and (v) may be negligible and are often omitted. However, they should always be checked to determine if they have any significant effects on extensions, particularly with regard to a prestressing bed fabricated from steel sections. The true measured extension and anchorage losses at the headstock are to be compared with calculated or estimated values, and must not vary from these values by more than that allowed by the Specifications. A check method of determining the true extension is to mark off a 4 m length of tendon and carefully measure this length before and after tensioning. Possible causes of variations between true measured and calculated extensions are:

(i) the wrong sag pull-up pressure may have been used.

(ii) the final jacking pressure may have been incorrectly read.

(iii) Calibration of the jacking system may be incorrect.

(iv) Tendons could be fouling reinforcement or end forms.

(v) Incorrect measurements may have been taken

(vi) Anchorage slip and draw-in may be different from that assumed.

(vii) Friction due to hold-downs and hold-ups may be different from that assumed.

(viii) Unexpected slippage of tendons could have occurred.

(ix) An unrepresentative sample may have been tested.

(x) the supplier's strand certificate may be incorrect.

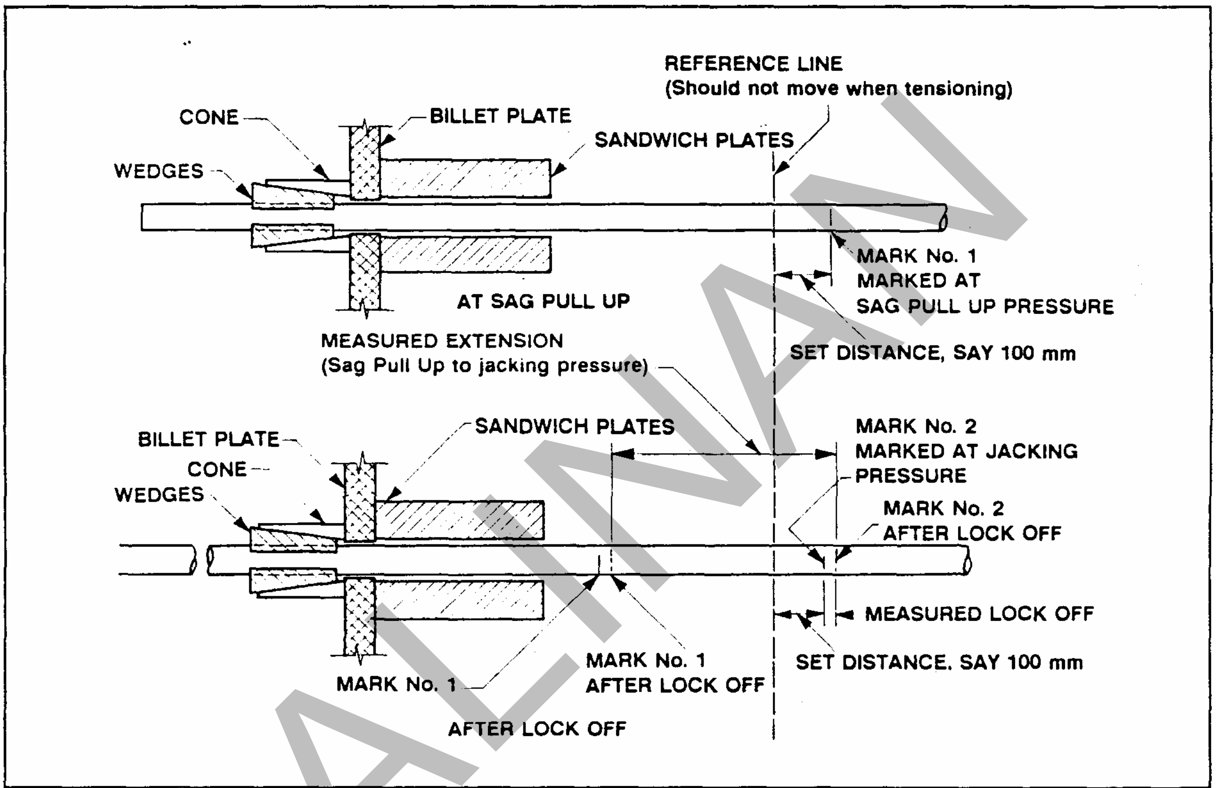


Figure 5.13 - Measured Extension

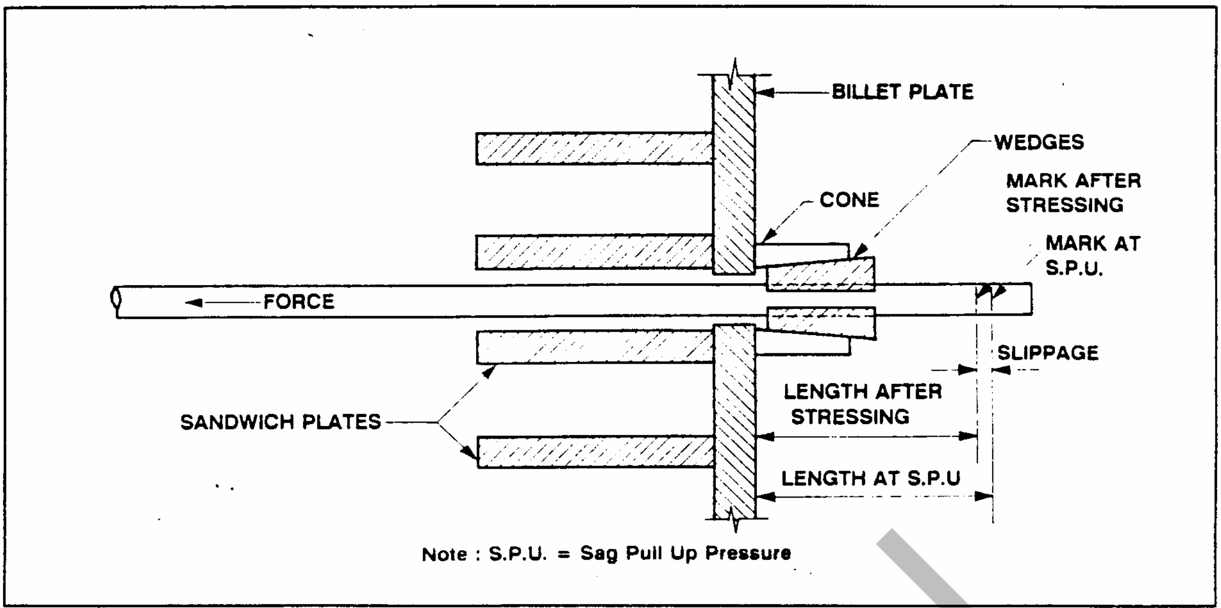


Figure 5.14 - Slippage at Dead End

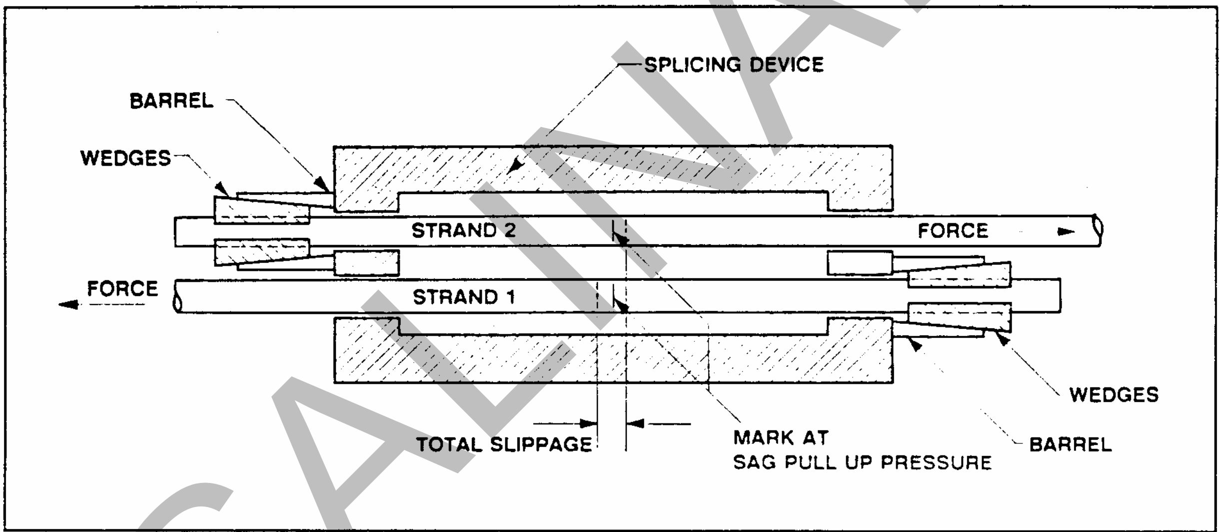


Figure 5.15 - Slippage at Splice

v. Tendon Failure

Tendon failure can occur because of worn grips or wedges, local tendon failure due to faulty material, corrosion, physical damage such as kinking, overstressing, or heat being applied to the tendon. As a safety precaution, the exposed tendons must be covered with tarpaulins or held down with toggles to prevent whipping of tendons if a failure occurs. Should a breakage occur it should be investigated to determine the cause of the failure before continuing with the work.

A tendon will usually slip through the wedges rather than break. If this occurs, it may shoot out at the other end of the prestressing bed in a straight line, until it is stopped by a barrier or a deflector. For this reason, it is important to keep areas behind the anchorages free from any objects, and not to allow anyone to stand behind the anchorages when the tendons are tensioned and exposed. Wedges should be inspected to ascertain that they are the correct size for the particular strand, are not cracked, the teeth are not blunted or worn, and that they are clean and free of grease and dirt. If excessive slipping occurs, the machining, tolerances and hardness of the wedges and anchor heads should be checked. Wedges normally used in post-tensioning operations are not to be used in pre-tensioning operations because the teeth are too fine. The Supervising Engineer's primary duty is to ensure that all safety precautions are observed in the precasting yards, and particularly, that safety signs are erected when tensioning is in progress. If a Contractor fails to comply with safety precautions, then work must be stopped until he does.

c. Transfer of Prestress

For pre-tensioned work the transfer of prestress to the concrete should be achieved slowly and uniformly using jacks to release the force in all tendons at the one time. Mechanical cutting of strands under load is not permissible because of the impact effect of the sudden release on the completed units. If a specially designed headstock to allow for the detensioning of all strands at the one time is not available, load transfer may be carried out by the partial jacking off of individual tendons in a prescribed pattern or by heat relaxation. For the transfer of load by heating to be acceptable the heat must be applied over a sufficient length of tendon and over a sufficient time to allow gradual relaxation before final failure. Relaxation of strand should proceed simultaneously at both ends of the stressing bed to prevent sudden movement of the unit. The concrete must be protected both from radiated heat from the flame, and from heat conducted through the tendons, by keeping the flame remote (at least 300 mm) from the units. Where pretensioned strands are deflected individual strands and hold down devices must be released in an order predetermined by the designer to avoid unacceptable loading patterns in the concrete.
After stress transfer, tendons must be cut off flush with the end of the member or the anchorage. Flame cutting should not be used for this purpose to avoid damage to the concrete. The exposed ends of tendons are then protected against corrosion by the application of a sealing compound such as tar epoxy or an epoxy resin.

d. Records

Information such as concrete strengths, hogs, bows, details of stressing equipment used, coil numbers used in fabrication of the cable, and load and elongations must be recorded, preferably on a standard form.

5.5.5 Safety Precautions

First and foremost, no one should stand behind a jack or anchorage during stressing operations.

Everybody not actively involved with the stressing operation and its supervision should be kept well away from the work. The supervisory staff must be competent and experienced. It is preferable that the operators themselves are also experienced with the stressing system in use.

The condition of all equipment should be carefully checked before starting, especially the gripping devices that are required to be used more than once. Establish that the equipment is in satisfactory condition. Cleanliness is essential. Components showing signs of wear or fatigue should be replaced, and the condition of hoses must not be overlooked.

Coils of high tensile wire must be handled with great care as they tend to recoil suddenly if the ends are not secured. If the unit being stressed or grouted is elevated, traffic passing underneath should be diverted or protected both from the effects of broken wires or cables and from leaking grout.

The jack or jacks must be secured against recoil, preferably with chains, in case there is a sudden failure of stressing material or equipment. Heavy barricades should be erected behind the jacks, and the space between the jacks and the barricade should be fenced off. Prominent signs must be displayed, warning workmen and passers-by to keep clear. Rolls of hessian or heavy plastic, and timbers can be laid over prestressing wires which are not contained within substantial formwork or reinforcement. Do not leave the jacking system under pressure. If stressing cannot be completed within a short time, unload the jack and start again when the problem has been solved, making any necessary adjustment to the load and extension.

Do not weld or flame-cut near stressing material or equipment, and do not hit with a hammer or otherwise jar the equipment once it has started to apply the load.

Check jack position and alignment and fixings at both ends of the unit after the initial load is applied. An experienced operator should keep the non-jacking end under observation during loading.

During grouting, operators should keep clear of duct bleeders as a temporary blockage may be followed by an explosive clearance.

5.5.6 Grouting

a. General

Grouting gives long term protection against corrosion to prestressing tendons, helps to spread superimposed loads throughout the unit, and helps to protect the unit against possible failure caused by shedding of load by one or more strands in a stressed cable. Grouting is advisable therefore immediately after stressing of a unit is completed, and desirably not later than two days after completion. Where exceptional circumstances occur grouting may be delayed; however consideration must be given to protecting the tendons continuously against corrosion during this time.

b. Materials and Mixing

Grout is a mixture of cement and water and an additive if approved. The design of the mix should contain only enough water to enable the mixture to run freely and penetrate voids. Plain cement and water grouts bleed and shrink and expansive additives or gel type additives or plasticisers may be approved to remedy these deficiencies. Standard tumble-action mixers are inadequate for grout mixing and high speed rotary mixers are more suitable, the water always being put in first. The grout is fed from the mixer by way of a hopper and strainer to a suitable pump which operates continuously and has a recirculating facility which will keep the mix moving if grouting is temporarily held up. Good practice requires that sufficient grout be mixed for only one duct. Any surplus should not be reused, and in the event of delay, grout more than 30 minutes old should not be used.

c. Procedure

Ducts are flushed out by first using a good supply of running water and then blown out by clean oil-free compressed air. Any water remaining in the duct will be forced out through the vents by the incoming grout. The grout supply is connected to the lowest vent. The remaining vents are progressively closed off as grout, completely free from air and water, flows from them. When the duct is full the pump continues to apply pressure, usually of the order of 700kPa, to the closed system for about a minute. If considered necessary the grout consistency can be checked by a hydrometer.

It is important that the system, especially all joints, be leak-free and that the equipment is clean and maintained in good order. If a leak appears during grouting, which cannot be stopped, the grout in the duct should be flushed out with water and the operation recommenced when the leak has been repaired. If there is blockage it may be possible to fill the entire duct by transferring the mixing and pumping operation to the other end of the unit, otherwise the blockage must be cleared using water and compressed air. Where there is a risk of cross bleeding of grout into an adjacent duct, also to be grouted, it is sometimes advisable to grout both ducts simultaneously.

Personnel working near units must be aware of the possibility of sudden spurts of air-water-grout mixture. Generally, workmen must keep clear of cables until the grout has set. The unit should not be moved until the grout has had 7 days to gain strength. Where units are grouted in their final location in the bridge they must not be subjected to heavy construction of traffic loads for 7 days following grouting.

Equipment, procedures and the characteristics of the grout-mix should be tested before and during the operation and samples may be taken for strength testing. A grout strength of 30 Mpa (300 kg/cm^2) is an appropriate 28 day strength.

When grouting is complete all projecting vent pipes are cut off flush and are made good.

5.5.7 Handling and Storage of Precast Prestressed Girders and Deck Units

Post-tensioned girders may be designed with sufficient reinforcement to allow them to be lifted from the casting bed soon after casting and before post-tensioning. Other designs may provide for partial stressing so that the unit can be moved away from the casting bed for completion of stressing and subsequent grouting. Others require units to be fully stressed before they may be moved. It is therefore essential that the casting yard supervisor understand clearly the permissible method of handling prestressed units, that the top is marked, and that they are moved, loaded, transported and unloaded only under close supervision.

Lifting points must be provided in the precast members. Their location is normally specified by the Drawings. The prestressed member must only be lifted and supported at these locations.

If a girder is being lifted without a spreader a conservative rule of thumb is that the slings should be at least 60^0 from the horizontal, although the Drawings may vary this. Very long, flexible girders may need side supports to prevent sideways buckling resulting from axial lifting loads imposed by the slings.

The stacking site must provide a flat, firm, tidy, well-drained area. Heavy full-width timbers, preferably hard-wood, support the girders close to each bearing position, and the ground between the supports must be cleared to ensure that if the main supports allow the girder to subside after heavy rain it will not receive any support from anything in this area. The girders must be kept vertical at all times and never be allowed to turn or fall on their sides.

It is prudent to provide each with independent side supports in case the bearers move. Units must be sufficiently far apart to allow for regular inspections during storage. Stacking of large members is not advisable, but smaller units such as deck planks or piles may be stacked, in which case the bearers must be vertically above each other to avoid inducing bending loads.

Some types of deck units are cast upside down for convenience. These members need support near mid-span while upside down, but near the ends after turning over. The designer should approve the design of the turning over equipment before it is used. A gradual smooth rotation is required.

5.5.8 Practical Details

a. General

A number of points relating to each of pretensioning and post-tensioning are noted in the following sections. They relate to practical details to which the supervision team should pay attention, in order to ensure that a high standard of workmanship and material quality is achieved.

b. Pretensioning

i. General

- Before any tensioning operations commence it is necessary for the Contractor to submit a schedule of stressing data for approval by the Engineer.

  The schedule should contain:

  - a detailed sketch of the longitudinal tendon pattern for the length of bed with the length per tendon to be tensioned clearly indicated

  - the tensioning force per tendon to be applied by the jack and allowances for friction along the length of bed especially in the case of deflection strand patterns

  - the estimated elongation per tendon, including allowances for slippage of the gripping devices at either or both ends of the bed

- The stressing bed should be checked to ensure that the base is flat and even.

ii. Tendons

- The tendons should have been sampled and tested in accordance with the specifications

- Care must be taken to ensure that the tensioning force is kept within the absolute limit of 85 percent of the ultimate tensile strength of the tendon

- The splicing of tendons within the length of a member is not to be permitted. Splicing by means of couplers may be permitted outside the member. Where couplers are used outside the member they must be observed during stressing for rotation or spin (which cause relaxation of the tendon and loss of elongation). If rotation or spin occurs immediate steps must be taken to have the couplers modified or permission to splice withdrawn.

iii. Tensioning Draped or Deflected Tendons

There are three general methods of tensioning deflected pattern tendons and special allowances in elongation and jacking force have to be made in the stressing schedule prepared by the Contractor.

The methods are as follows:

- Tension with each tendon held in its required position by means of low friction rollers or pins. In this case the extension for each tendon is calculated on the basis of its exact length with due allowance for friction at the rollers or pins.
- Assemble the deflected tendons in the low position, tension in a horizontal plane and then lift to the upper fixed pins. The difference between the initial and final tension is the tension induced by the added movement of the strand.
- Assemble the deflected tendons in the high position, tension in a horizontal plane and then deflect to the lower fixed pins. The difference between the initial and final tension is the tension induced by the added movement of the strand.

iv. Transfer of Prestress

- Strands should be heated in such a manner that failure of the first wire in each strand will occur after the torch has been applied for a minimum of five seconds or preferably longer .

  The sequence used for heating the strands should be according to an approved schedule that keeps the stresses nearly symmetrical about the axis of the members

- When hold downs have been installed the Contractor should supply details of how he proposes to release the hold down forces. This is particularly important when the weight of the concrete member is less than about twice as great as the total of the hold down forces. In this case weights or vertical restraints will need to be added directly over the hold down points.

v. Concreting

- Forms for internal ducts or voids must be positively anchored against movement or flotation during placing or vibration of concrete. The formers should be made of material that will not change shape during handling or placing of concrete.
- Care should be taken to ensure that form oil is not allowed to come into contact with tendons.

- An adequate number of test specimens should be moulded in order to provide for early testing of specimens for release and stripping. It is suggested that at least 3 pairs of cubes or cylinders for release are moulded per line of members cast.

- The underside of pretensioned units should be inspected by the supervising engineer as soon as the members are lifted from the bed

vi. Acceptance of Prestress Work

The acceptance of prestress work is the responsibility of the Engineer. There are however several features that the supervision team should observe and note in order to assist with the assessment of the completed work. These features are:

- Satisfactory stressing results where actual tendon forces agree with required tendon force within certain limits nominated the Engineer.

    - Maximum jacking force must not exceed 85 % of the specified minimum ultimate strength of the tendon

    - The actual force for an individual tendon is permitted to be within ± 5 percent of the required force provided that the force for the member as a whole is within ± 2 percent of the total required force

- The satisfactory transfer of prestress including visual inspection of concrete for cracks both prior to and after transfer. All cracks should be marked with crayon and their location and extent noted on a free hand sketch.

- The satisfactory compaction of concrete, that is the member does not have any honeycombing, voids or shrinkage cracks. Honeycombing is generally the result of inadequate vibration. Whether areas of honeycombing are allowed to be patched depends on their location and extent in the member. Members with extensive honeycombing, honeycombing in the bottom soffit, over bearing points or deep enough to expose tendons would not generally be accepted.

- That all measurements of the finished member conform to the tolerances permitted by the specification. Dimensional tolerances for cross section and length should be rigidly enforced but excessive dimensions in 'hog' (profile in vertical plane) or 'bow' (profile in horizontal plane) are sometimes permitted by the Engineer.

c. Post-Tensioning

i. Tendons

- All coils or bundles of tendons are to be sampled, tested and approved in accordance with the specifications prior to the commencement of work, irrespective of the presence of manufacturer's certificates.

- An adequate number of test specimens should be moulded in order to provide for early testing of specimens for release and stripping. It is suggested that at least 3 pairs of cubes or cylinders for release are moulded per line of members cast.

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c. Post-Tensioning

i. Tendons

- All coils or bundles of tendons are to be sampled, tested and approved in accordance with the specifications prior to the commencement of work, irrespective of the presence of manufacturer's certificates.

- The anchorage must be set exactly at right angles in all directions to the axes of the tendons

- Concrete behind the anchorages must be completely compacted.

iii. Placing of Tendons

- In in-situ type construction or in the casting of complete members the tendons should be placed in the ducts prior to the placement of concrete. The tendons assist in rigidly holding the ducts in their true position during the placement of concrete.

- As soon as possible after the placement of concrete the tendons should be moved backward and forward a few times in order to ensure that they are free from mortar intrusion.

- Where the VSL system of dead end anchors is being used care must be taken to protect the exposed strand (at the anchor end) from rusting prior to concreting. In addition special care is necessary to ensure that the ducts are sealed as the strands cannot be moved back and forward in the duct after concreting as they can in a normal post tensioned beam. There is thus no way of checking whether leaks have occurred which might cause problems when grouting is carried out.

iv. Tensioning Operations

- The anchorages and equipment must be inspected before commencement of tensioning. Check also that the member is free to move longitudinally.

- If the tendons have been placed in the ducts after the member has been cast it will be necessary for the ducts to be first flushed out with clean water and then blown out with compressed air to remove all foreign matter.

- If the gauge pressure is less than the expected pressure it means that less friction is being encountered than expected. If it is more then more friction is present. If the gauge pressure is considerably less it is recommended that the elongation calculations be checked before anchoring off.

- Note that tensioning is measured by elongation and that gauges, dynamometers and load cells are for checking purposes only.

- If the required elongation has not been attained when the pressure gauge indicates that the tension load has reached 85 percent of the ultimate tensile strength of the tendon, the tendon should be de-tensioned and the trouble investigated.

- Satisfactory stressing results are where actual tendon forces agree with required tendon force within certain limits nominated the Engineer. These limits are generally as follows:

  - Maximum jacking force <u>must not</u> exceed 85 % of the specified minimum ultimate strength of the tendon

  - The actual force for an individual tendon is permitted to be within ± 5 percent of the required force provided that the force for the member as a whole is within ± 2 percent of the total required force

v. Grouting

- The ducts should be pressure grouted with a suitable approved grout mixture within 48 hours of the completion of the stressing operation, unless otherwise specified or permitted by the Engineer.

- Immediately prior to grouting the ducts should be thoroughly flushed with clean water and then all surplus water removed using compressed air.

- Grout must always be applied by pumping towards an open vent. It is applied continuously under moderate pressure at one end of the duct until all entrapped air is forced out the open vent at the opposite end of the duct. This is continued until a steady solid stream of grout is discharging. The open vent is then closed while the pressure is maintained. The grouting pressure is gradually increased to a minimum of 700 kPa and held at this pressure for about 1 minute. The grouting entrance vent is then closed.

- On long beams a central vent hole is often incorporated, with a plastic tube extended through the web of the beam to provide for easier filling with grout.

5.6 DEFECTIVE CONCRETE

5.6.1 General

This section deals with repairs to concrete that has defects after stripping of the formwork.

The Specifications generally provide for repair of defects and other remedial work such as filling holes left by form fittings etc.

5.6.2 Repair methods

a. General

Four different repair methods are mentioned in the Specifications and discussed here.

Whichever method is used, it is important to recognise that the preparation of concrete for repair is as important, if not more important, than the actual repairing process.

The Supervising Engineer should ensure that clear and detailed instructions are given to the Contractor to ensure that the repair is properly carried out.

Where inspection after stripping of formwork reveals the need for repairs, it is necessary to carry out the work as soon as possible and preferably within 24 hours. While the repairs are in progress, the supervisor should ensure that curing is not interrupted at other locations in the member.

The concrete to be repaired must be clearly marked and a series of inspections carried out to determine the extent or concrete removal and repair.

Removal of condemned material is usually done by hand chiselling and must be supervised carefully to ensure that this does not affect the adjacent sound concrete.

The essential features for the correct removal of concrete prior to beginning repair work are shown in Figure 5.17.

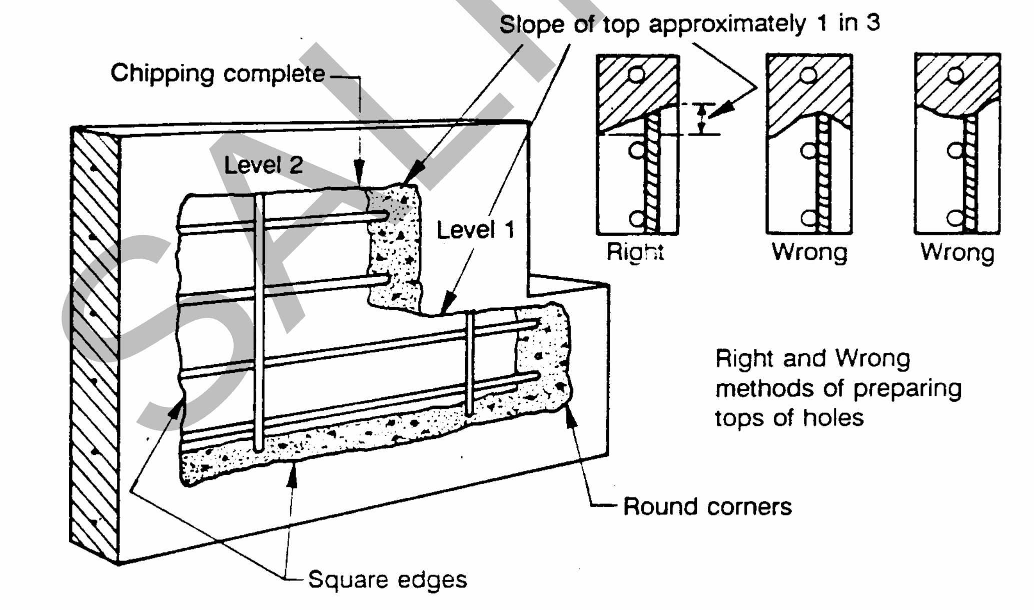


Figure 5.17 - Proper Removal of Defective Concrete Prior to Repair

b. Repair with Dry Pack

The dry pack method is used for relatively deep holes, having a depth equal to or greater than the least surface dimension and where lateral restraint can be obtained.

Where lateral restraint cannot be obtained the mortar replacement method may be more suitable. For filling behind considerable lengths of exposed reinforcement, or for filling holes right through walls or beams, concrete replacement will be a better method.

In preparing a patch for dry pack it is essential not only that the holes are sharp and square at the surface edges, but also that corners within the hole are rounded. The interior surfaces should be roughened to develop effective bond. Holes should be shaped so that the minimum depth for dry packing is about 25 mm.

The filling operation should be started after the surface has been washed clean and dried, and after inspection by the supervisor. The surface is first brushed with a stiff mortar or grout (barely wet enough to adhere to the surface) the mix being usually 1 of cement to 1 of fine sand with a consistence of thick cream. This bonding coat should not be wet enough nor applied so thickly as to affect the dry-pack material, which is applied promptly before the bonding coat dries out. Sometimes dry cement is dusted on the surface after the application of the bonding coat to absorb excess moisture; any surplus cement in the hole is then removed by brush before any packing begins.

Dry-pack is usually a mix of 1 part cement to 2.5 of sand passing a 1 mm sieve, these proportions being varied to ensure that the colour of the patch matches that of the adjoining area. Sometimes a small amount of white cement is used for this purpose.

For packing bolt holes a leaner mix of 1 to 3 or 1 to 3.5 will be sufficiently strong and may blend better with the colour of the adjacent concrete. Only sufficient mixing water is used so that the mortar will stick together on being moulded into a ball by a slight pressure of the hands, and will not exude water but will leave the hands damp.

Placing of the material is done in layers of about 10 mm thick and packing is done with wooden sticks of about 25 mm diameter by 200 to 250 mm long and a hammer. If rubberiness develops under this tamping further placement of layers should be delayed. The holes should be finished level with the adjoining surface and there should be no excess water. The supervisor should ensure that iron tools are not used for compaction as these will tend to darken the colour of the filling. When the repair is complete water curing should follow.

c. Concrete Replacement

The concrete replacement method is suitable for filling holes right through a concrete section, or when holes in unreinforced concrete are more than 1.0 m^2 in area and extend beyond the reinforcement.

When replacing concrete cast in formwork, or when replacing concrete on the side of a structure, the construction and setting of forms for the replacement work is important. Front forms for repairs of concrete walls more than 450 mm high should be constructed in horizontal sections so that concrete can be placed not more than 300 mm in depth, sections of formwork being set as concreting progresses. Typical formwork details for concrete replacement in such walls are shown in Figure 5.18. The forms must be mortar tight at all joints and tie-bolt holes, especially when pressure is applied during the final stages of concrete placement.

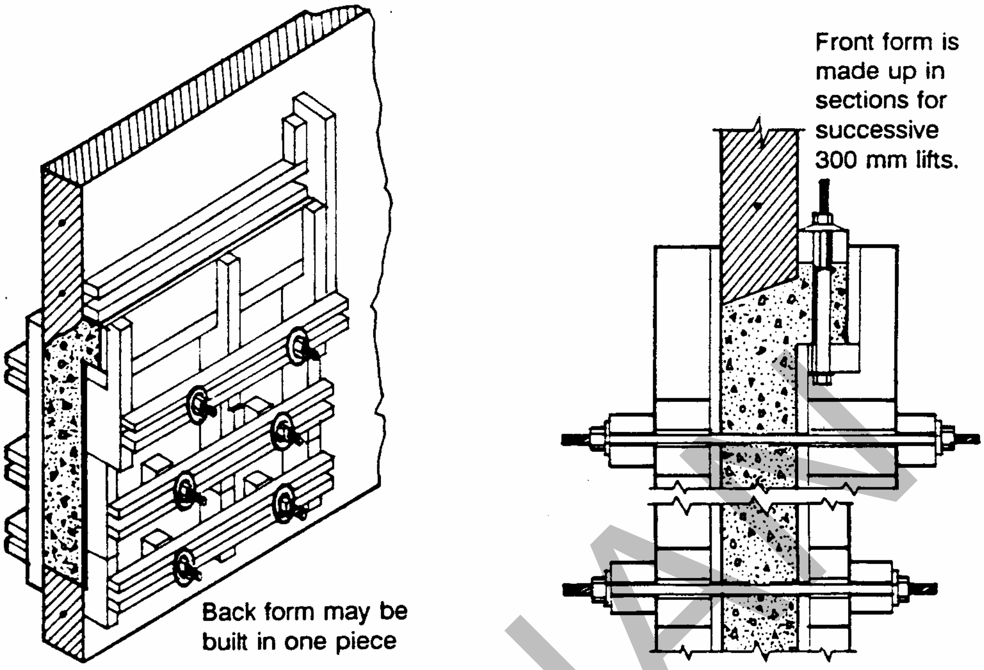


Figure 5.18 - Formwork Detail for Concrete Replacement

In preparing a patch for concrete replacement, unless otherwise specified, the supervisor should ensure that:

(a) Holes should have a minimum depth of 100 mm in new concrete and 150 mm in old concrete, and the minimum area for repair is 0.05 m^2 in reinforced and 0.1 m^2 in unreinforced concrete.

(b) Reinforcement bars are left partially embedded, and a clearance of at least 25 mm is available around each exposed bar. Loose-tie wire should be removed, and exposed reinforcement cleaned (preferably by sand blasting).

(c) The top edge of the hole at the face of the structure should be cut to a nearly horizontal line. If necessary, the top of the cut may be stepped down. The upper surface of the cut should be on a 1 to 3 upward slope from the back toward the face of the wall from which the concrete will be placed (see Figure 5.17).

(d) Holes in the walls should be kept wet by packing with burlap continuously kept wet until final cleaning prior to filling.

(e) Before placement of filling, the holes must be cleaned once again so as to leave the surface free of chipping dust, dried grout and other foreign matter. This is often done on large jobs by wet sand blasting, followed by air-water jets and lastly by an air jet. Removal of free surface moisture at the bonding surface or any other foreign material is essential before placement of filling material.

The back form is usually placed and fixed as soon as the removal of defective concrete is completed. The front form in not installed until after final cleaning, it should be then promptly fixed, and followed by application of a thin layer of grout or mortar about 3 mm thick to coat the concrete surfaces in the hole. This mortar should have the same composition and w/c ratio as the concrete mix used for replacement.

After this preparation of concrete surface filling begins immediately. Usually air-entrained concrete is used for this purposes and where colour uniformity with adjoining concrete is desired, the colour of the cement is selected carefully or a mixture with white cement adopted. To minimise shrinkage, the concrete should be as cool as possible when placed, and when placing concrete in lifts the work should not be continuous. For the lowest lift slumps of about 60 mm may be adopted, but for higher lifts concrete with lesser slump is used. The fresh concrete should be vibrated to ensure satisfactory compaction with form vibrators normally being used for this purpose. During placing and compaction, sufficient pressure should be exerted in the forms to maintain the desired shape of the replaced concrete. In a few cases expanding admixtures have been used to ensure that concrete fills the space adequately and under the given restraint develops sufficient strength. Expanding admixtures should be used with considerable care.

Forms for concrete replacement repairs may usually be removed the day after casting, unless this would cause damage to the fresh concrete. Any projections left are removed by trimming carefully without breaking back into the repaired portion, and any rough areas resulting from trimming are dressed carefully to match with the adjoining surface. Adequate curing then follows.

d. Mortar Replacement

Repair by the mortar replacement method is generally suitable for shallow holes, too wide for the dry-pack method and too shallow for concrete replacement, and also for all comparatively shallow depressions (large or small) which do not go beyond the reinforcement nearest the surface. Honeycombed areas and shallow imperfections apparent on removal of the formwork may be repaired by this method while the concrete is still green.

After the areas are prepared by removal of all defective concrete and thorough cleaning, the mortar should be applied immediately. No initial application of cement, mortar grout or wet mortar is necessary. Where hand placed mortar is to be used the edges of the chipped out areas should be squared leaving no feather edges. Where a pneumatic gun is used, comparatively shallow holes should be flared outwardly at about 1 to 1 slope (45 degrees) to avoid the inclusion of rebound, and the corners should be rounded. Where old concrete is to be repaired it should be removed to at least 25 mm deep.

Usually mortar replacement is done with a pneumatic gun, the type of equipment used depending on the size of the job. Small sized equipment is available for small concrete repairs.

The water content and a suitable mix for the mortar depend on the type of equipment used but fine sand passing a 1 mm screen is generally adopted. If repairs are more than 25 mm deep, the mortar should be applied in layers of about 20 mm to avoid sagging and loss of bond. After every layer there should be a time lapse of about 20 minutes before the next layer is placed, but the earlier mortar layer should not be allowed to dry. In completing the repair the mortar should be finished slightly more than the full level required and then trimmed off after the material has hardened a little, without damaging the filled portion. Adequate curing is essential.

The most popular forms of this technique using pneumatic equipment are shotcrete and gunite.

e. Epoxy Resins

Whatever the size of repair required, epoxy repairs should be carried out with the advice of a specialist. The term 'epoxy' refers to a thermosetting plastic which can be used as a bonding medium and is suitable for use in locations where long term curing cannot be carried out. Epoxy mortars consisting of fine sand and epoxy resin are used for dealing with thin patches where reuse of the area is desired promptly, so that moist curing cannot be done in that location. Epoxy resins have a short pot life and therefore should be used as soon as possible after mixing. Epoxy compounds have 3 to 5 times the coefficient of thermal expansion of ordinary concrete so they should only be used in areas where this difference will not give rise to difficulties.

Before starting repairs with epoxy compounds the work must be prepared as for other methods. A suitable formulation of epoxy with a suitable additive for curing is thoroughly mixed and immediately applied on the surface to be bonded by brushing to about 10 to 15 mm in thickness. The supervisor should check that this application is made within the pot life of the mix and using the appropriate techniques. Thinning or dilution with solvents to extend the pot life of the epoxies should not be permitted.

Where epoxy mortar is to be used the supervisor should ensure that the mortar is prepared using clean, dry and, where necessary, graded aggregate (usually sand) with the proper proportion of epoxy. For thin patches, mortar comprising 1 part of epoxy and 2 to 3 parts of sand may be suitable. For deeper patches, appropriately larger aggregates and leaner mixes, with 1 part of epoxy and 5 to 6 parts of graded aggregate with a maximum size up to 10 mm have been used. Where forms are used to support thicker patches the forms should be coated with a suitable release agent such as silicone.

Epoxy bonded concrete normally requires no special curing procedures, other than water curing. Epoxy mortars usually require no curing, temperatures of 20°C to 30°C for 1 to 3 days being all that is required.

At present the techniques for filling cracks by epoxies may be classified as follows:

- penetration by gravity
- use of natural capillary forces in narrow cracks
- positive injection using high or low pressures

These methods are described in more detail in the Construction Supervision Manual and also in the Bridge Maintenance and Rehabilitation Manual.

6. STEEL SUPERSTRUCTURE ERECTION TECHNIQUES

6.1 GENERAL

This Section describes the steel superstructure types in use in Indonesia. A brief summary was presented in Section 1 of this Manual.

In addition to describing the different span configurations and widths, the various components and the differences between earlier and later versions are described.

The methods of erection of each type of bridge are described and advantages and disadvantages of each discussed.

The use of A, B or C class descriptions to indicate the width of the structure is common to all types of trusses.

- *A* class bridges are two lanes with a carriageway of 7.0 metres width with a 1.0 metre footway on either side;
- *B* class bridges are two lanes with a carriageway of 6.0 metres with a 0.5 metre kerb on either side but no separate footway;
- *C* class bridges have a 4.5 metre carriageway with a 0.5 metre kerb on either side but no footway.

6.2 AUSTRALIAN TRUSS

6.2.1 Permanent Truss

a. General

The Australian (Transfield or Transbakrie) system of truss bridging comprises precision-made standard steel components which are assembled by bolting together to form bridge spans in the range 35 to 60 metres of *through-truss* design.

The permanent spans are supplied in three classes - A, B and C - which differ in roadway width and kerb/footway configuration. Spans in all classes have composite reinforced concrete decks. Refer to Figures 6.1, 6.2, 6.3 for A-Class, B-Class and C-Class cross-sections respectively.

This bridging is supplied complete with bearings, seismic lateral stops and buffers, railings, deck angles and tools, equipment to be used in the assembly of the components into bridge spans, and an Erection Manual.

Components are clearly marked to permit assembly in the sequence shown on the Erection Drawings. Components with the same mark are interchangeable. No component weighs more than 1.5 tonnes and assembly is by hand tools provided with the bridge spans.

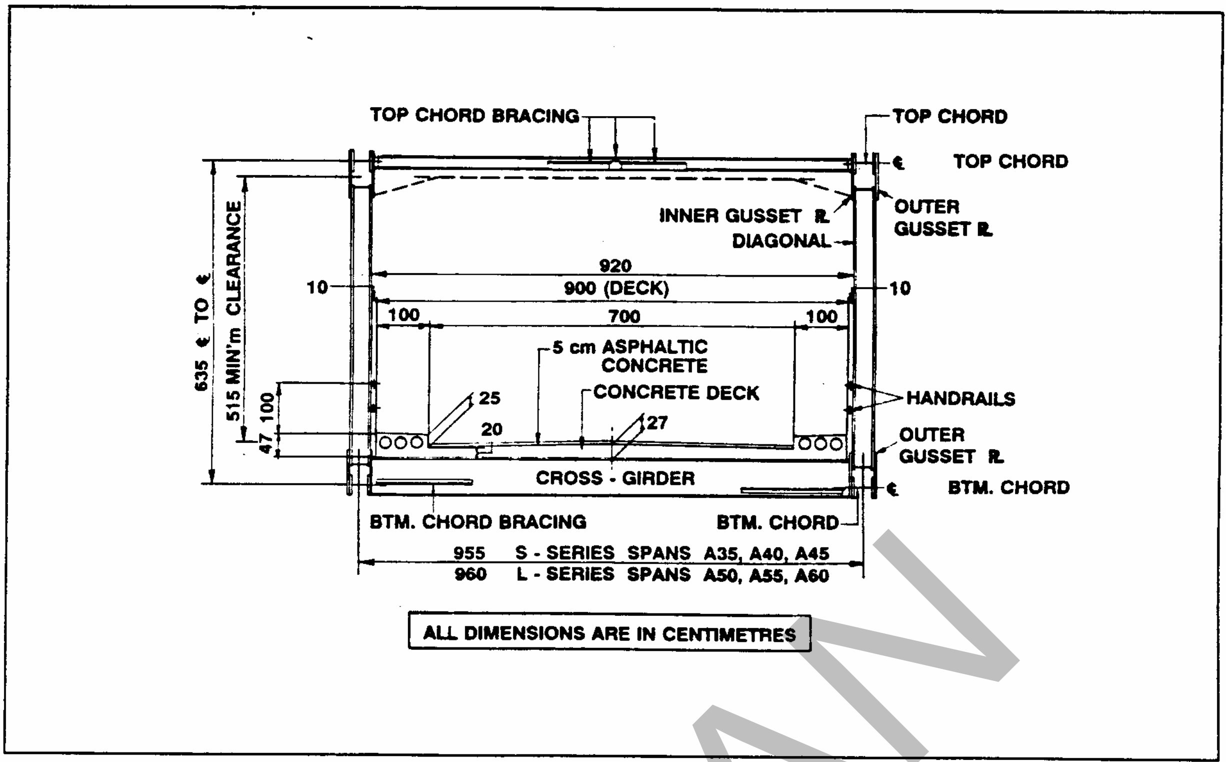
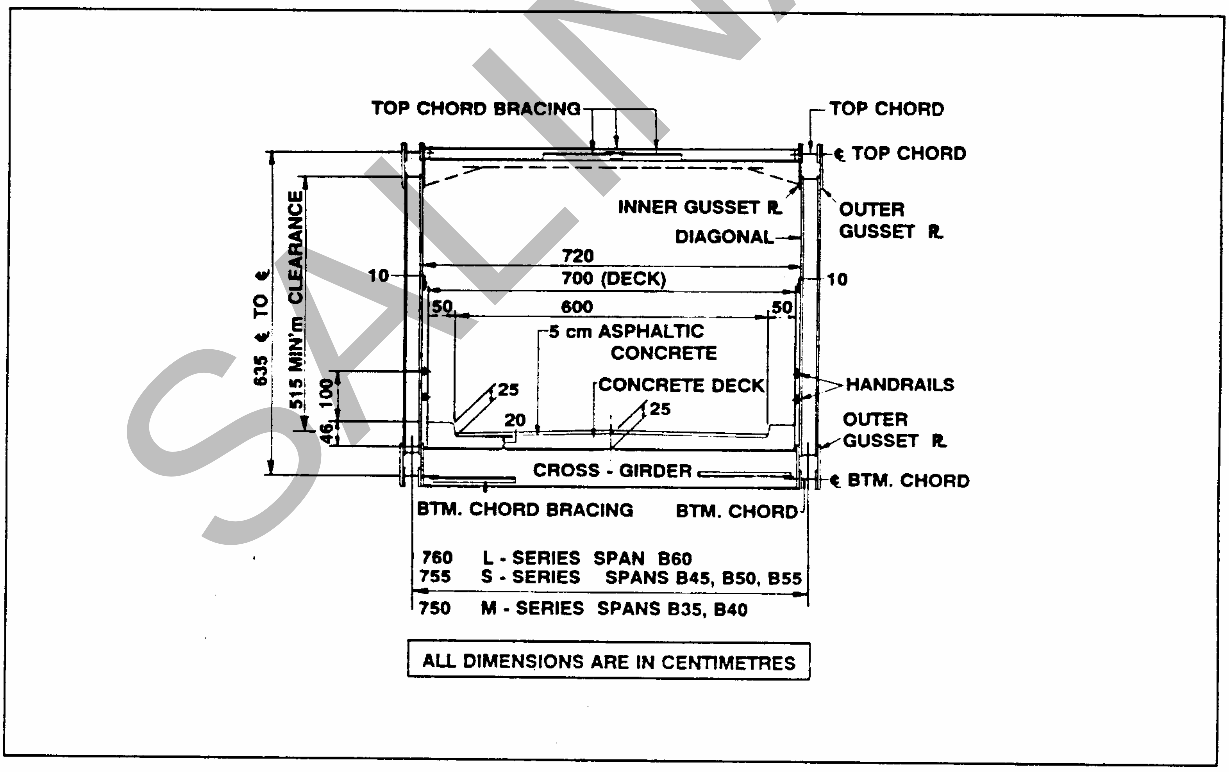


Figure 6.1 - Australian Permanent Truss - A Class



6.2 - Australian Permanent Truss - B Class

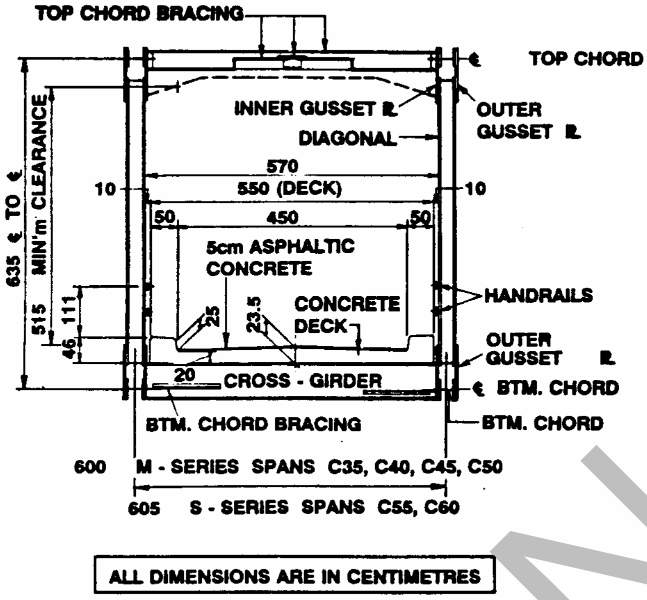


Figure 6.3 - Australian Permanent Truss - C Class

The system has been designed to permit progressive assembly in place by cantilever working from one bank, or rolling out by the single span launch (SSL) method without the use of falsework in the river. These two methods for truss span erection are described in the Erection Manual. Both require the use of a standard span as an anchor span and linking steelwork which is provided with the system. The SSL method requires special launching equipment as well as link steel.

Other methods of assembly and erection such as part-cantilever or erection on falsework are feasible. The principles laid down for the methods described in the Erection Manual will apply in these cases.

The construction of the concrete deck and the installation of the bearings and seismic lateral stops and buffers are also described in the Erection Manual.

This bridging system is planned to be of low maintenance characteristics. To this end all steelwork and bolts are galvanised and bearings are elastomeric. Basic maintenance procedures are described in the Erection Manual.

Design Criteria

Loading:	Loading Specifications for Highways Bridges No. 12/1970 (revised 1988) Direktorat Jenderal Bina Marga, Indonesia
Traffic:	A and B Class - two full lanes plus part lane, D-loading (plus impact) or T-loading (100%)

	C Class - one full lane, D-loading (plus impact) or T-loading (100%)
Footways:	A-Class 500 kg/m^2 one metre wide each side B-Class and C-Class - nil.
Railings:	100 kg/m
Wind:	100 kg/m^2
Seismic:	Region 1 as Specification 12/1988 (C=0.3)
Stream:	Superstructure assumed clear above flood level.
Temperature:	± 15°C

Design Specifications

NAASRA Bridge Design Specification 1976.

AASHTO Standard Specification for Highways Bridges 1983.

Abutments, Piers

The abutments and piers are to be designed for the forces arising from the steel spans and other effects, and constructed to suit the bearing and span dimensions. Relevant forces and details are given in the Erection Manual.

b. Components

There are four different series of components used for Australian truss bridging. One, the 'H' series is only used for the permanent special truss bridges, see Section 6.2.2. The other three, 'L', 'S' and 'M' series components, are used for different configurations of permanent truss bridging. There are some spans using 'MM' components, supplied under contracts from P.T. Trans-Bakrie.

The main differences in the L/M/S series of components is in the section properties of the main members. The size increases from 'M' to 'S' to 'L'. There are different components for each of the different series and, in general, the components are not interchangeable between series.

Table 6.1 lists the spans in which each series is used.

Table 6.1 Spans for Truss Series

Series	Spans
L - Series	A50, A55, A60, B60
S - Series	A35, A40, A45
	B45, B50, B55
	C55, C60
M - Series	B35, B40
.	C35, C40, C45, C50

In most cases the component code is prefixed by M, L or S to indicate the series of components. Note however that this is not the case with any of the EP (erection) components which are differentiated by number, for example the Bottom Chord link for M to M series trusses is EP48 while the same component for M to L series trusses is EP57.

In addition, the components which are identical for all series of trusses, for example FSB (formwork support beam) or THDB (holding down bolt) are not prefixed with M, L or S.

Table 6.2 Naming System for Truss Components

Code	Description	Prefixed
C	Longitudinal chord	*
X	Cross Girder	*
D	Diagonal	*
B	Bracing, cross beams (top)	
G	Gusset plate	*
S	Splice Plate	*
BA	Bearing assembly (left and right)	*
RB	Rubber bearing or buffer	
DA	Deck protection angle	
FP	Footway plate	
FSB	Formwork support beam	
R	Railing	
THDB	Holding down bolt	
LS	Lateral stop	
TP	Packer	
SP	Packer	
EP	Erection Piece (covers all temporary components)	

The third column of Table 6.2, 6.3 indicates (with a '*') which components are prefixed.

c. Erection Methods

i. General

This Section describes the various erection methods for the permanent truss bridging. These methods are basically described in the Erection Manual. Much of the following information has been based on the 'Bridge Inspectors Handbook' prepared for the Directorate General of Highway (DGH) on the Indonesian Australian Steel Bridge Project.

The choice of erection method should be carefully considered.

ii. Falsework

This method is probably the most common and may be used for single or multi-span structures. Temporary supports are used while the superstructure is being assembled. They are placed in the river bed between the substructures as shown in Figure 6.4.

The falsework is removed after completion of erection and before pouring the concrete deck. This allows the superstructure to deflect as designed when the deck is poured.

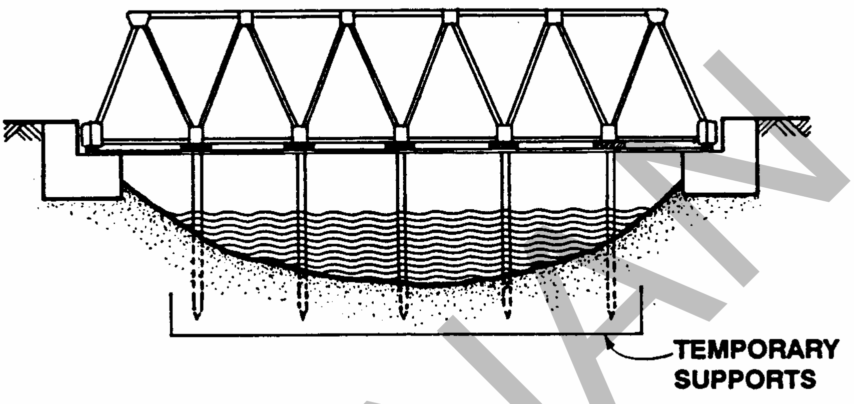


Figure 6.4 - Erection on Falsework

This method has a number of advantages for most sites. The biggest advantage is that there is no need for the additional anchor spans, link set and kentledge (counterweight) which the launching or piece-by-piece cantilever methods employ.

In addition, there is no need for heavy lifting equipment as the heaviest component is only 1.5 tonnes in weight. It is a labour-intensive method with a minimum of lifting equipment required.

On many sites, the existing bridge can be used as the basis for the falsework support and hence the cost is reduced.

One disadvantage is that a falsework bridge is usually required to be constructed across the river, presenting an obstacle to boats navigating the river. In general, a falsework pier or trestle is set up under each cross girder at spacing of about 5 metres.

In addition, there is a possibility of the falsework settling under the load of the truss if it is not properly supported. A falsework pier for an A class bridge must support about 10 tonnes dead load for the steel truss.

The installation of falsework across a river immediately before or during the wet season should be carefully considered as a river flow could demolish the falsework and the partially completed truss.

Erection Equipment Required

The following erection equipment is required with the main steelwork :

1. Erection Manual.
2. Construction Drawings.
3. Hydraulic jacks 25, 100 and 150 tonne capacities.
4. Tool kit (for assembly of all steel work and link kit).

In addition to the above equipment, the Contractor must supply and install the following items :

a. Material for the falsework supports
b. A minimum of 2 chain blocks for lifting the components into position
c. Equipment for hauling steel components from the bank onto the falsework
d. Jacking plates and timber packing for use in lowering the span
e. Temporary timber bearings

iii. Piece-by-Piece Cantilever Method

Piece-by-piece cantilever erection consists of the progressive assembly of a truss span, outwards from one abutment or pier to the opposite abutment or pier, by the addition and fixing of individual components (or small assemblies) into place at the forward end of the partially-completed span as it cantilevers over the crossing. This static cantilever procedure requires an anchor span and Link steel.

No falsework is needed and access can be gained to the end where the component is to be fixed via the previously erected steelwork.

The cantilever erection system has the advantages of simple erection equipment with no moving parts, and the space required for assembly on the bank is limited to the length of the anchor span. On the other hand, it is necessary to provide means of hauling or winching the components out across the river and lifting and supporting them in place over the water. This is a method of erection which requires only a minor amount of mechanical equipment such as hand winches, gin poles, pulley blocks and tackles. Cranes can be used to speed up erection time if a pontoon is available.

The general method of erection is pictured in Figure 6.5.

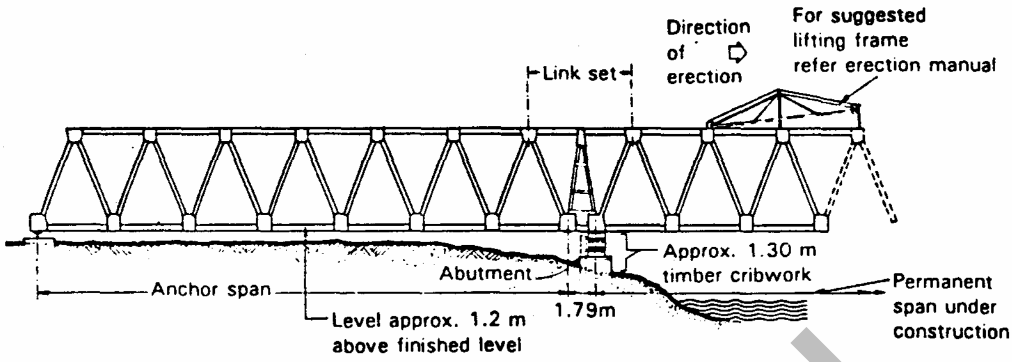


Figure 6.5 - Piece-by-Piece Cantilever Construction

Area Required

The clear area required behind the abutment for the erection of the steel work should be big enough to accommodate the anchor span which can range from 30.0 metres to 60.0 metres in length in 5.0 metre increments.

The area required can be limited to the length of the anchor span plus the working area surrounding it.

As a guide, the working area should be about 3.0 metres wider than the anchor span being used and 10 metres longer than the anchor span length.

The area should be graded and levelled to at least the top of the abutment and no higher than the finished roadway level.

Temporary Bearing Support

Substantial timber cribwork is to be used under each bearing for the support of the anchor end of the cantilever span at the abutment or pier during erection. The timber support is erected directly above the final bearing position.

Support to Anchor Span

The trailing end of the anchor span requires support at the two rear bearings on timber cribwork or temporary concrete pads designed to suit the soil conditions.

Anchor Span Link

The anchor span will comprise a standard truss span which is linked to the permanent span for erection via the Universal A-frame Erection Link Set, see Figures 6.6 and 6.7.

Depending on the length of the span being built and the length of the anchor span, it will be necessary to add kentledge (counterweight) to counteract the overturning effect of the cantilevering span. Details of kentledge weights are given in the Erection Drawings.

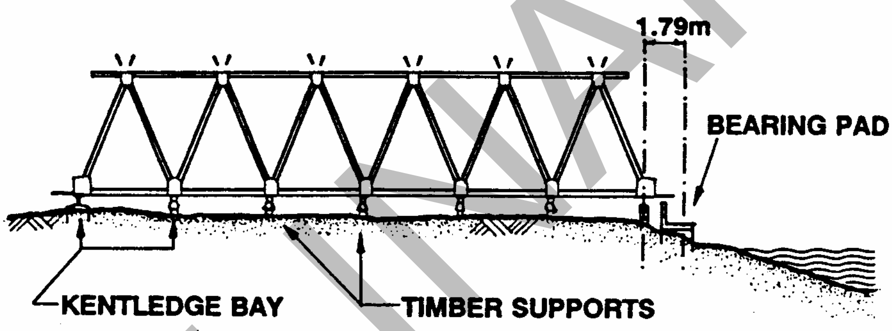


Figure 6.6 - Anchor Span For Cantilever Erection

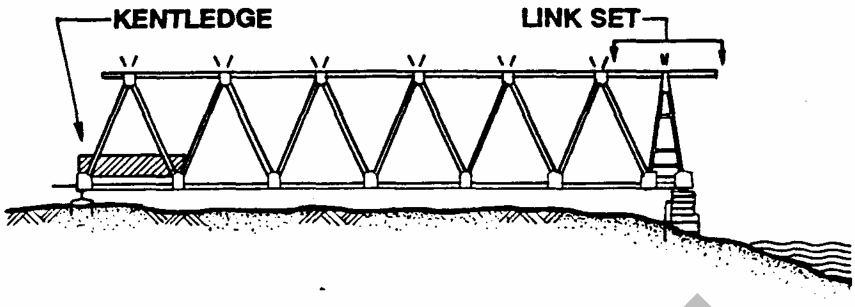


Figure 6.7 - Anchor Span and Linkset

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Anchor truss span
4. Erection Link set (linking steel) including reinforcing for chords where required.
5. Kentledge brace kit.
6. Hydraulic jacks 25, 100 and 150 tonne capacities.
7. Tool kit (for assembly of all steel work and Link set).

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Support framework or timber cribwork as temporary support to the permanent span bearing plates.
b. Kentledge platform to trailing end of anchor truss span.
c. Suitable material for kentledge (counterweight). For example sand packed in bags, concrete blocks, steel components, rocks etc. but whatever is used, the weight must be known.
d. Jacking plates and timber packing for use in lowering the span.
e. Equipment for hauling steel components from the bank across the stream and lifting and supporting in position.
f. Temporary timber bearings.

Hauling Out, Lifting

It will be necessary to lift and haul the components out from the storage area on the bank to the point of connection into the span as the erection progresses. This may be done by various means depending on site circumstances. Methods which have been used include:

- Access from an adjacent existing bridge using a light crane.
- Rafts made up of 200 litre drums.
- Flying fox cables suspended between the top bracing of the span.
- Skidding the components along the partially completed deck steelwork on a temporary timber platform. Rollers must be used to avoid damage to the components.

When it is in line with its final position, the component is hoisted into position. Various methods are available, including a derrick pole braced from the end of the partially erected truss.

However, it recommended that two simple lifting frames be fabricated from light steel sections and fixed to the end of the top chord each side by bolting through the drainage holes in the web. Used in conjunction with chainblocks or hand winches, these frames are simple to operate and can be moved along the span as erection proceeds.

iv. Single Span Launching

With this method of erection, the truss span is completely assembled on one bank and rolled out into position using an anchor span and kentledge (counterweight). No falsework is required within the crossing since the span is designed to fully cantilever. The general concept is shown in Figures 6.8 and 6.9.

This method is suitable for a single span or the first span of a multi-span bridge. It is particularly suited to bridge sites of one span which cannot be erected on falsework.

Not all bridge sites are suitable for this system because a longer assembly area is required on the bank from which the launching process is carried out, compared to the piece-by-piece cantilever method where no assembly area is needed on the river bank other than what has been specified previously for the assembly of the anchor span. The additional area required for S.S.L. is due to a need for rolling tracks which must be constructed to accommodate both the main span and anchor span.

The area required on the river bank will depend on the length of main span and anchor span plus the work area surrounding the spans.

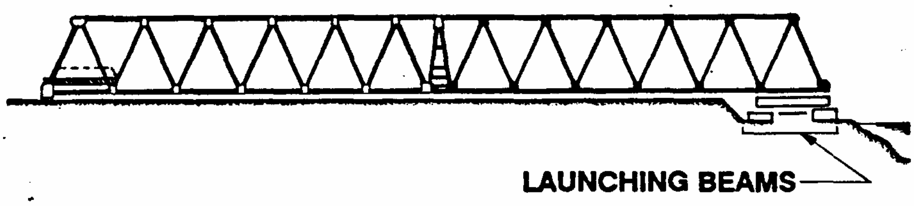


Figure 6.8 - Construction For Single Span Launching

Depending on the length of the span being built and the length of the anchor span, it may be necessary to add kentledge (counterweight) to counteract the overturning of the cantilevering span.

Erection Equipment Required

The following erection equipment is required with the main steel work :

1. Erection Manual.
2. Construction Drawings.
3. Anchor truss span,
4. Erection link kit (linking steel).
5. Kentledge brace kit (This does not include the actual counterweight)
6. Launching beams with front and rear rollers.
7. Tool kit (for assembly of all steel work).

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Rolling tracks constructed on either concrete or steel beams for the rear end of the span to roll on.
b. Concrete jacking pads behind the abutment.
c. Outhaul and backhaul winches.
d. Kentledge platform to trailing end of anchor truss span.

e. Suitable material for kentledge (counterweight). For example sand packed in bags, concrete blocks, steel components, rocks etc. but whatever is used the given weight must be known.

f. Jacking plates and timber packing for use in lowering the span.

g. Equipment for hauling steel components from the bank across the stream and lifting and supporting in position.

h. Temporary timber bearings.

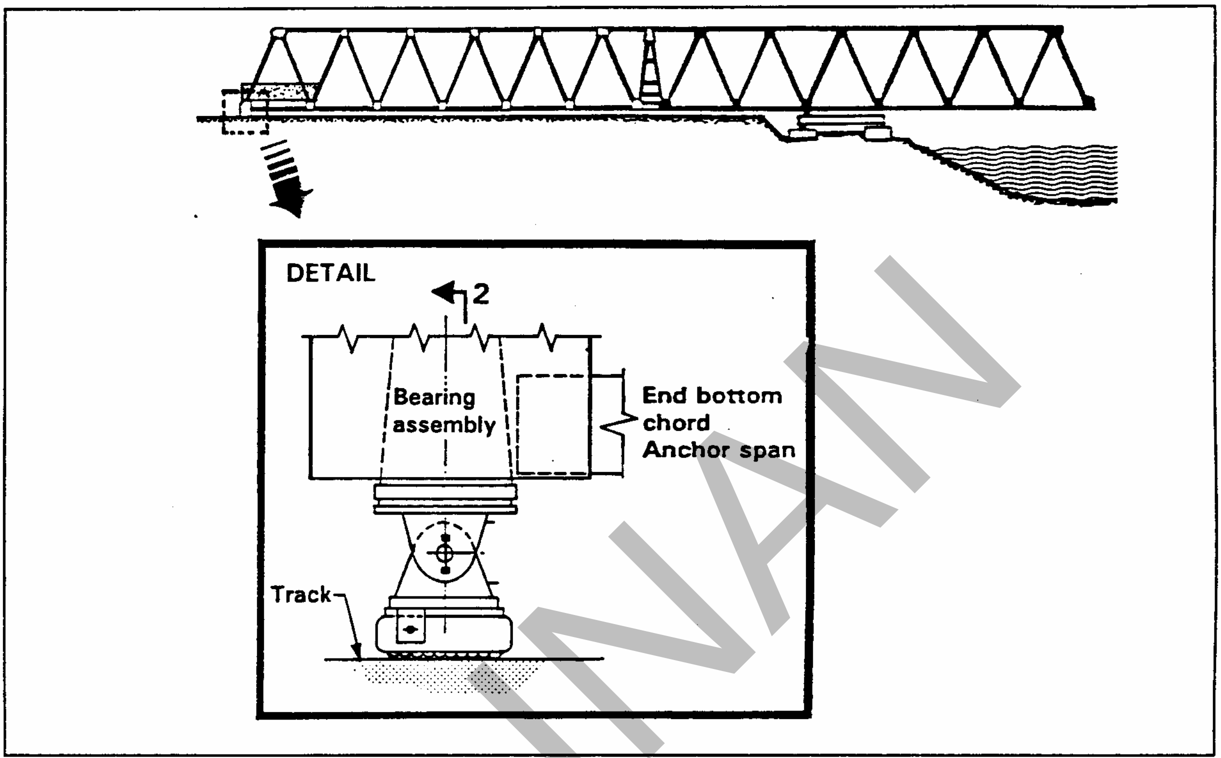


Figure 6.9 - Single Span Launching

v. Combination Methods

There are several alternative combinations of these erection methods possible, although these are seldom used.

It is possible to construct part of the span on falsework and then cantilever the remaining part of the span, using kentledge to maintain stability. It is also possible to part-launch and part-construct using piece-by-piece cantilever construction.

d. Variations

The standard components available in the various Australian bridging systems may be adapted for a wide range of bridging forms and design criteria, such as alternative loading specification, timber decks or continuous spans.

The only option which is available in the Permanent Steel Truss bridging is that PVC service ducts up to 3 150 mm diameter (maximum) may be installed in the kerb of Class A bridges.

Another "option" is that the MM series bridges (35, 40 and 45 metre spans) produced by Trans Bakrie had profiled steel trough decking and a 100 mm thick reinforced concrete topping slab instead of the more usual 270 mm (Class A), 250 mm (Class B) or 230 mm (Class C) (nominal) thickness reinforced concrete deck which acts compositely with the steel truss. As noted, these MM series spans have been almost all utilised in bridges already.

e. General Problems

The following problems have been noted in the construction of steel trusses:

Tightening of Bolts

It is absolutely essential that all bolt-tightening is completed before the concrete deck is poured. Failure to do so will result in a loss of camber of the structure. Permission to pour the deck should not be given until the bolt tightness certificate has been issued.

The gap as shown by the Load Indicating Washers must be between 0.15 mm and 0.25mm. If the bolt is tightened to a gap of less than 0.15 mm there is a possibility that the bolt will snap. See Figure 6.10 for detail.

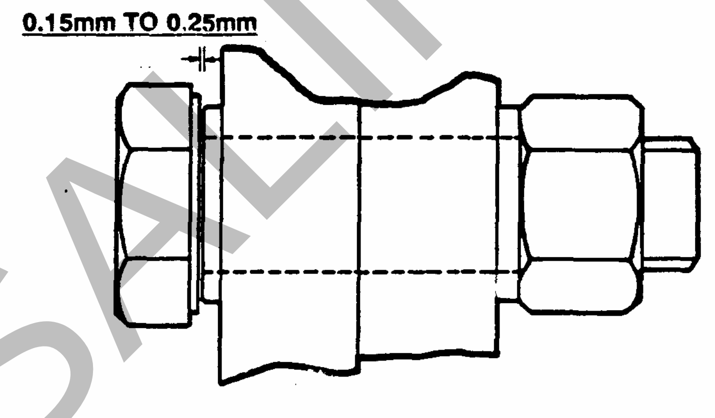


Figure 6.10 - Installed bolt after tightening

Tightening should commence at the centre of any group of bolts and proceed outwards as shown in Figure 6.11.

As tightening proceeds the head of each bolt should be marked to show that it has been tensioned correctly.

Any bolt that has been tensioned must not be reused and the bolt, nut and load indicating washer must be disposed of and replaced from the spares. The practice of reusing bolts must be prohibited.

Tension wrenches are not to be used to tighten friction grip bolts in this series of bridging as there is no correlation between a torque setting and the gap as shown by the Load indicating washers.

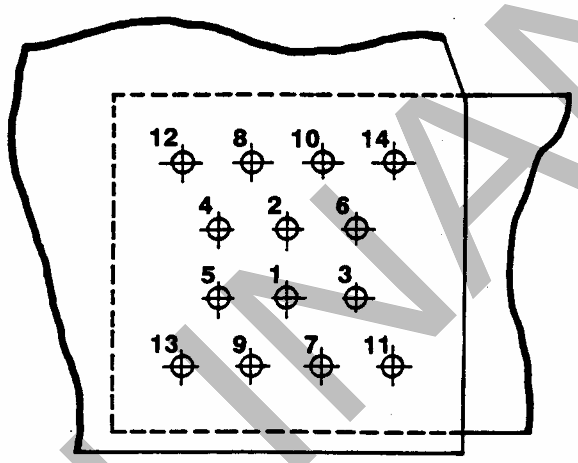


Figure 6.11 - Typical Bolt Tightening Sequence

Setting out of bearing centres

The longitudinal horizontal centre to centre distance of the bearings from abutment to abutment or abutment to pier should be checked against one of the following drawings:

TD01	A Class S Series Truss
TD02	A Class L Series Truss
TD03	B Class M Series Truss
TD04	B Class S Series Truss
TD05	B Class L Series Truss
TD06	C Class M Series Truss
TD07	C Class S Series Truss

It has been found in many cases that the nominal span length or the horizontal distances shown on the cross girder setout drawings has been incorrectly specified for the distance between bearings.

If this dimension is not correct it will have serious results in any multi-span configuration as the space between the superstructures will be too large and the linking steel will not fit in the gap between the two trusses.

Construction of bearing plinths on piers and abutments

The bearing plinth levels should not be less than those shown on the Bearing and Seismic Buffer Details Drawing, otherwise there may be a problem fitting the hydraulic jacks under the span.

Quality of concrete in deck slab

The Australian truss bridging has been designed to have a composite reinforced concrete deck. The deck slab is connected to the steel truss via the shear studs welded to the cross girders. It is very important that the quality of concrete in the deck slab is at least as good as that assumed by the designers.

Quality of concrete in lateral stops

It is important that both seismic and lateral stops are constructed as indicated on the Drawings because these bearings are vital to the structure if an earthquake should occur. The concrete used in these stops must be of good quality.

Clearances between the face of the rubber and the concrete must comply with the requirements of the Drawings, as shown in Figure 6.12.

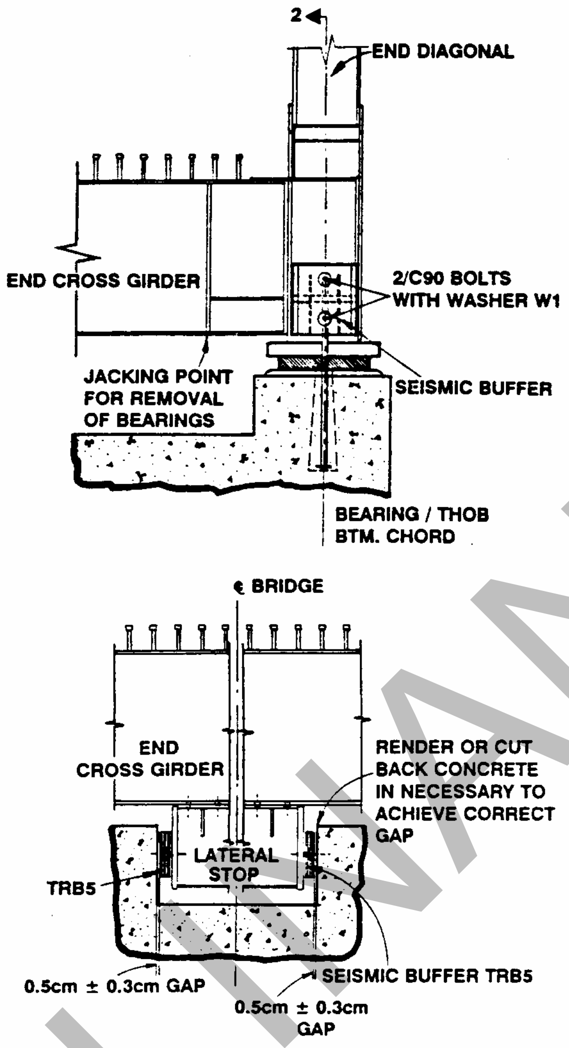


Figure 6.12 - Clearances at lateral and seismic stops

Omission of sections of concrete until after completion of truss erection

It is recommended that the back walls of the abutments or piers should not be completed to the full height until the deck has been poured and the truss set onto its permanent bearings.

The reinforcement which protrudes from the lower part of the wall should not be bent so as to cause sharp kinks which may become potential weak points.

Note that the height of the backwall has been designed based on a 50 mm thick asphaltic concrete layer. Any variation in the thickness of this layer should be reflected in a corresponding change in the height of the backwall.

Damage caused to components by poor storage and handling

Lost or damaged components usually must be replaced by spares which can take a long time. In many cases, the work on the site will come to a standstill. Accordingly, careful inspection is required of all components when they are received on site to determine if there are any missing or damaged components.

Steel members are to be handled, lifted and stored so as to avoid damage to the member and overstressing or damage to the protective treatment.

Prior to the arrival of steel components on site, an area of suitable size (to contain all the steel) should be prepared to receive all the components as they arrive at the site. The area should be as close as possible to the bridge site to avoid unnecessary double-handling of the material.

All members are to be stacked, on timber packing, level and clear of the ground. Members of a H-section are to be stored with webs vertical. Refer to the Erection Manual for typical stacking details.

Smaller components, such as gusset plates and splice plates, should be stacked in neat bundles above ground level on a timber platform and not loose on the ground.

Hand rail pipe should be stacked on timber packing and supported in such a manner that the pipe will not be bent.

Bolts, bearings and deck seals should be stored under cover, in a small shed if possible. If bolts, nuts and washers are left loose on the ground they will soon disappear. Note that all bolts, nuts and washers are to be kept dry up to the time of installing the bolts. This is to prevent the lubricating wax being washed off.

All the tools supplied on loan for the duration of the project should be stored in a secure place in the containers provided.

Components are often stored off site and brought in smaller quantities to the site during erection. The requirements for temporary storage adjacent to the bridge site are identical to those for the main storage area. The Contractor should not be permitted to stack components in such a way as to cause damage to the components or their protective coatings.

Use of falsework when piece-by-piece cantilever is a more appropriate method

Contractors often choose the falsework method when the cantilever method of erection would actually be better, easier and faster.

The use of falsework over deep river valleys or across busy rivers cannot usually be justified.

If the cantilever method is to be used, the Contractor will need to obtain sufficient components for a suitable anchor span. If no separate anchor span is available, a permanent span could be temporarily used as an anchor span, even if it is intended for a separate contract. Note that bolts on an anchor span do not have Load Indicating washers installed and are to be only snug-tightened.

If a small crane unit can be obtained for cantilever erection, the process can be considerably speeded up. Some sled-mounted lifting units have been used and are a good compromise, as small cranes are often difficult to obtain.

Jacking down onto permanent bearings

The Australian trusses have a different bearing system to the Dutch trusses and the truss **must not** be jacked down onto the permanent bearings until after the concrete deck has been poured. This is not the case with the Dutch trusses.

6.2.2 Permanent Special Truss

a. General

The permanent special truss spans are supplied in two classes - A and B - which differ in roadway width and kerb/footway configuration. Spans in both classes have concrete decks comprising profile steel sheeting with a concrete wearing surface. Refer to Figures 6.13 and 6.14 for A-Class and B-Class cross-sections respectively.

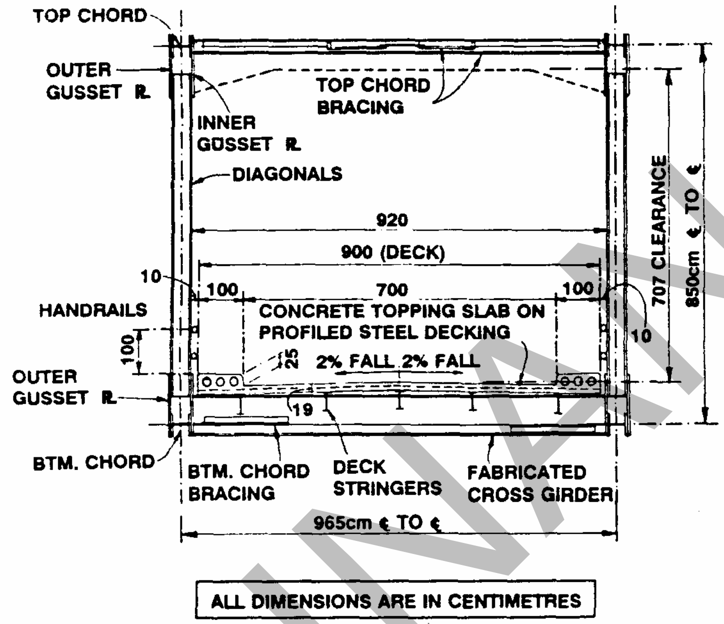


Figure 6.13 - Class A Permanent Special Truss

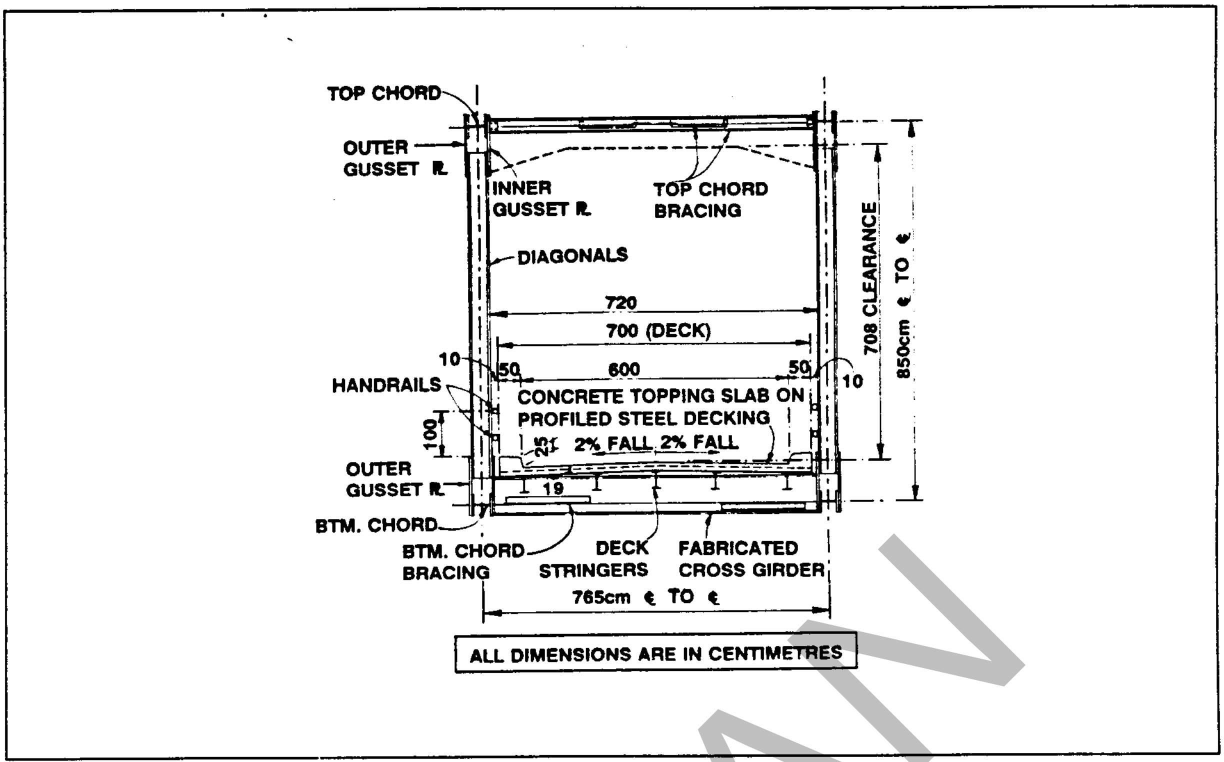


Figure 6.14 - Class B Permanent Special Truss

This bridging is supplied complete with bearings, seismics lateral stops and buffers, railings, profile steel decking and deck angles, tools and equipment to be used in the assembly of the components and an Erection Manual.

Only 80m and 100m spans of each class are available at present.

The spans in which permanent special trusses are available can vary in 6.67m increments.

Components are clearly marked to permit assembly in the sequence shown on the Drawings. Components with the same mark are interchangeable. No component weight more than 3.0 tonnes and assembly is by hand tools provided with the bridge spans.

The system has been designed to permit erection of the spans by half-span cantilever construction. This involves the piece-by-piece of cantilever construction of the two halves of the span from opposite river banks or from two adjacent piers in a multi-span bridge, and linking of the two halves at midspan. Each half requires an anchor span of the same class constructed from L-series components. Link steel, capable of linking L-series truss spans, is provided with the system. Adjustable erection bearings to permit alignment of the half-spans for connection are also supplied.

Although only the above method is described in the Erection Manual, other methods of assembly and erection such as erection on falsework are feasible. The principles laid down for the method described in the Erection Manual will apply in these cases.

The construction of the deck and the installation of the bearings and seismic lateral stops and buffers are also described in the Erection Manual.

This bridging system is planned to have low maintenance characteristics. To this end all steelwork and bolts are galvanised and bearings are elastomeric. Nevertheless, basic maintenance procedures are described in the Erection Manual.

The Erection Manual includes Marking Plans and Parts Schedules for all spans A80 (A100, B80 and B100) together with descriptions of materials and parts, bolting and assembly, and the half-cantilever erection method.

Where standard truss spans are used as approaches to the Special Truss spans, reference should be made to the separate manual covering construction.

Design Criteria

Loading:	Loading Specifications for Highways Bridges No. 12/1970 (revised 1988) Direktorat Jenderal Bina Marga, Indonesia
Traffic:	A and B Class - two full lanes plus part lane, D-loading (plus impact) or T-loading (100%)
Footways:	A-Class 500 kg/m^2 one metre wide each side B-Class - nil.
Railings:	100 kg/m
Wind:	100 kg/m^2
Seismic:	Region 1 as Specification 12/1988 (C=0.3)
Stream:	Superstructure assumed clear above flood level.
Temperature:	± 15°C

Design Specifications

NAASRA Bridge Design Specification 1976.

AASHTO Standard Specification for Highways Bridges 1983.

Abutments, Piers

The abutments and piers are to be designed for the forces arising from the steel spans and other effects, and constructed to suit the bearing and span dimensions. Relevant forces and details are given for the various spans in drawings bound in the Erection Manual.

b. Components

All four Special Truss types are assembled from a range of components known as the H-series components. These components have been specially designed to suit the high forces occurring in these long spans.

c. Erection Methods

i. General

This Section covers the various erection methods for the permanent Special Truss bridging. These methods are basically described in the Erection Manual.

It should be noted that it is not possible to use piece-by-piece cantilever construction from one end only with the Special Truss. This is because of the heavier sections used in the 'H' series components and the consequent excessive stresses which would be produced at the end of the truss due to the dead load of the truss.

The choice of erection method should be carefully considered.

ii. Falsework

Refer to Section 6.2.1.c ii for general details of falsework erection of truss bridging. It is expected that this will be the method of erection used for all single span and most two span structures. The main difference between the permanent truss and the permanent Special Truss bridging when using falsework is that the loads in the falsework trestles are much higher. Typical dead load is about 27 tonnes on a falsework support (for A100 span) and the Supervising Engineer must ensure that the falsework is designed and constructed to support loads of this size.

iii. Piece-by-Piece Cantilever

The permanent Special Truss spans are often used as part of a multi-span bridge built across wide and heavily-trafficked rivers, where the use of falsework to construct the main span is not possible.

The Special Trusses have been designed to be erected using the piece-by-piece cantilever method as shown in Figure 6.15. As noted above, the erection method is different to that used for the shorter permanent truss spans. The dead load of the heavier H series components makes it impossible to cantilever only from one end. Accordingly, the truss is erected by half-span cantilevering, that is by progressing simultaneously from each end of the span and meeting at mid span. Each half-span is linked back into an L series truss, either as an anchor span or as part of the permanent structure. Note that M or S series spans cannot be used as anchor spans for this type of bridge as the sections are not heavy enough to withstand the stresses imposed during construction. The general concept is shown in Figure 6.16.

When the spans meet in the middle, the trailing ends of the half spans are jacked up (and sideways if necessary) to allow the middle chord and diagonal assembly to be installed at the mid-span joint.

The span is then jacked down onto temporary supports until the decking system and concrete deck are in place when the span is placed on the permanent bearings.

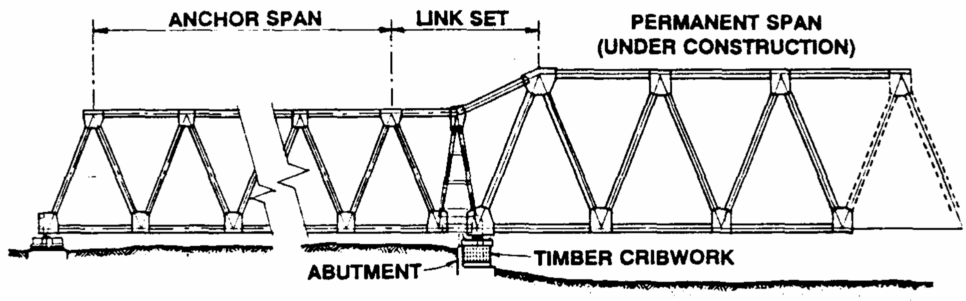


Figure 6.15 - Piece-by-Piece Cantilever Construction

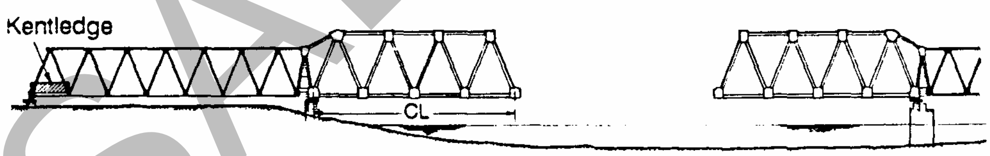


Figure 6.16 - Half Span Cantilever Construction

iv. Combination Methods

A combination of falsework and piece-by-piece cantilever methods, working from both ends can could be used.

d. General Problems with Erection of Special Trusses

The general problems described in Section 6.2.1.e are also applicable to Special Trusses.

In addition there are a number of points which are specific to the longer Special Truss spans.

- The need to accommodate a central expansion joint for the deck system means that the stringers at the expansion joint and internal cross girders generally are **not** reversible.

- Stringers for A class bridges must only be installed after the midspan truss joint has been completed, the bridge is supported at all four bearing points and the anchor span links have been removed. Stringers for B class bridges may be installed as erection proceeds.

- Inner and outer gusset plates at the top chord connections are not interchangeable

- Prior to installation of the special erection bearings they should be checked to ensure adequate freedom of movement of the bearings on the neoprene pad. Figure 6.17 shows for details of this bearing.

- Care must be taken to protect the neoprene pad and teflon and stainless steel surfaces from dirt and scratching as the surfaces are fragile and damage to any of these makes the whole unit unusable.

- All erection bearings must be installed so that the demountable arm points inwards toward the centre of the roadway.

- All erection bearings must have their transport clamps installed when they are not in use.

- The limit of movement of the erection bearings is ± 50 mm laterally and ± 75 mm longitudinally. Movement in excess of this will require resetting of the bearing.

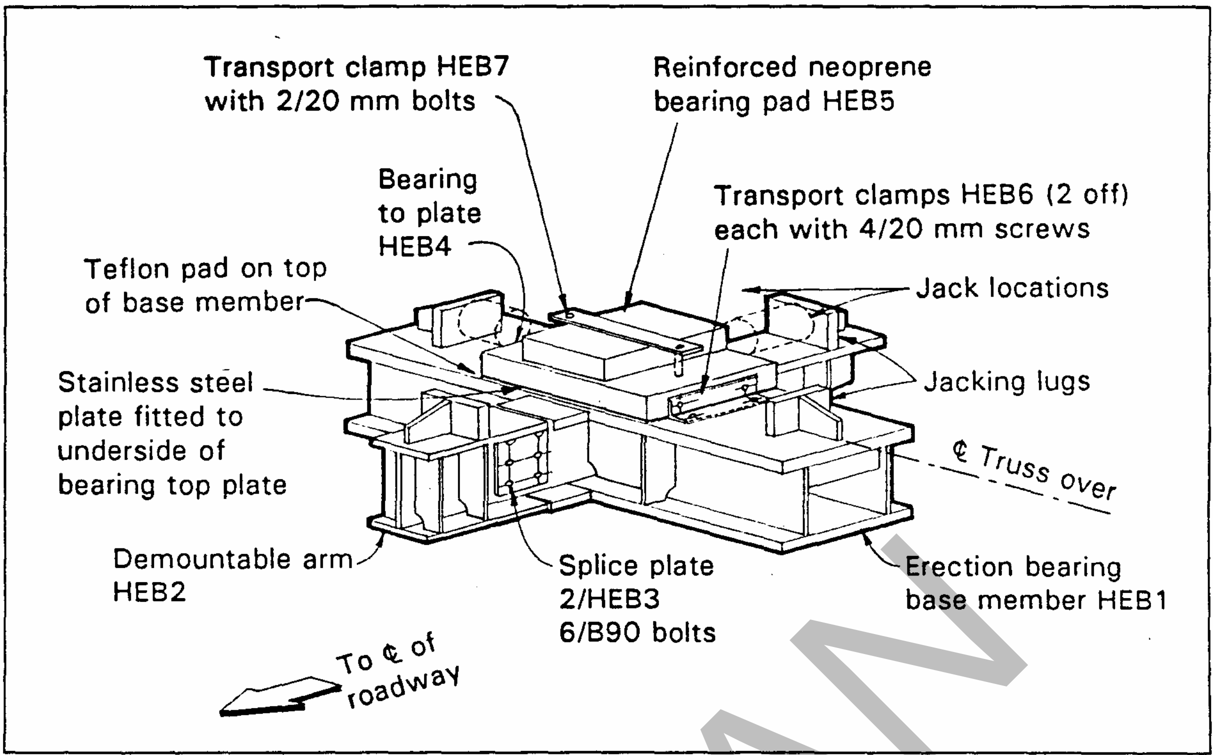


Figure 6.17 - Erection Bearings - Special Truss

- If a 35 metre L series anchor span is used, extra kentledge is required as set out in the Drawings.

- Lowering of the span involves a height of up to 1.4 metres and a staged lowering procedure is required. Jacking must proceed slowly and under careful control at all times.

- Construction joints in the concrete deck must be formed across the deck directly above a rib of the steel decking approximately 1.8 metres from the cross-girder. This joint must be formed and the use of an unformed construction joint must be permitted.

- Special attention must be paid to the details of the deck infill at piers because this detail is quite different from that normally used with permanent truss bridges. The deck infill is pictured in Figure 6.18.

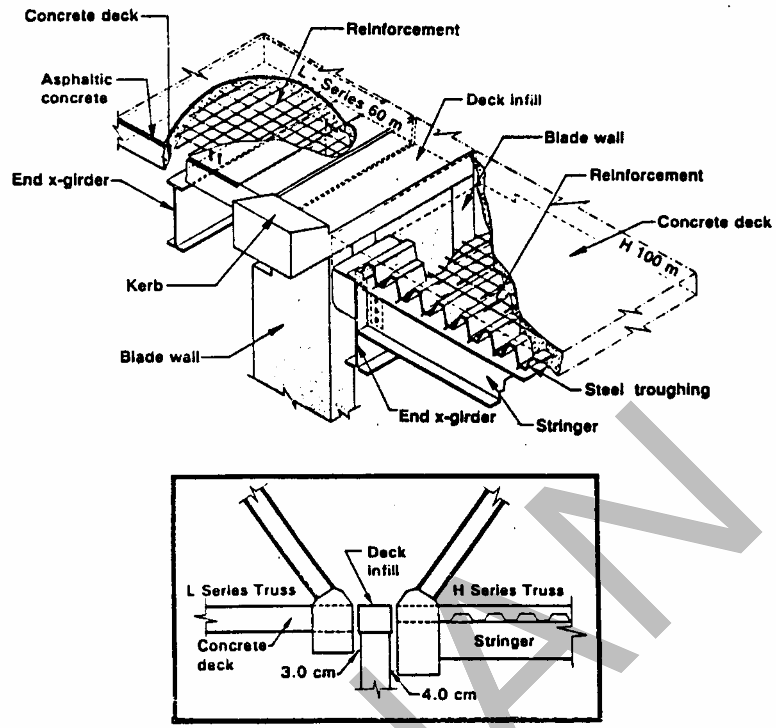


Figure 6.18 - Deck Infill

- There are many more bolts in an H series bridge than in an L series bridge: 60A:7600; 100A:18800.

- Very accurate setting out of bearings is required typically ± 1 mm.

- Jacking under approach spans should be carried out with jacks located under the stiffener plate, otherwise damage may be caused to the flanges of the chord.

- None of the components in the closure bay of the truss should be tightened until all the components are in position because some small adjustment with jacks may be required.

- Erection bearings must be locked after the spans are set up for closure, using timber packs and locking off the jacks. Placing components in the closure (mid-span) bay may cause the bridge to move on the erection bearings because the teflon bearing has very low friction.

- H series bridges are not designed to have an asphalt surface. This will increase the dead load (and correspondingly decrease the live load capacity). In addition, the joints will need to be set up differently.

- Camber offsets given in the Manual are approximate only, because of variations in camber (eg. the Contractor may not have fully tightened all bolts in the span, causing the camber to vary from that given). The closure offset figure may be more appropriate - refer to Drawings for details.

- The seal in the midspan deck joint (see Figure 6.19) is very important and must not be omitted. It is not supplied with the steel work from Australia.

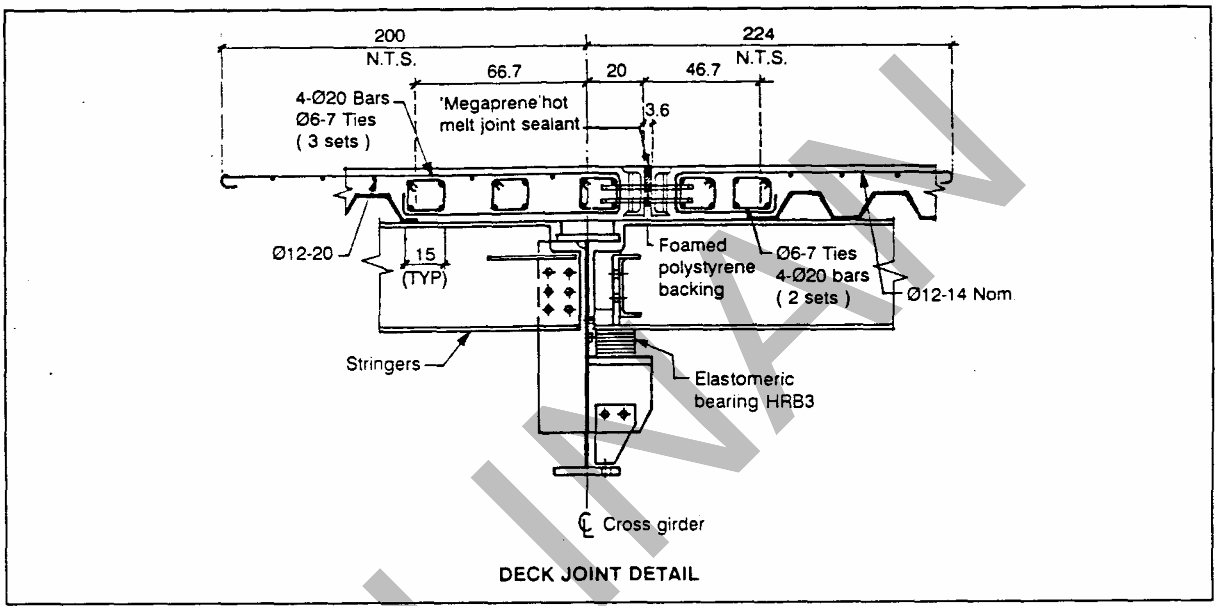


Figure 6.19 - Midspan Deck Joint

- The deck channels are structural components and must not be omitted (see Figure 6.19).

- Linking steel bolts in the top chord (B120 type) are too long and require an additional four (4) hardened steel washers to ensure the load is carried on the shank of the bolt and not the thread.

- On the 100m span Class-A bridges only, the end top gusset plate (LG4Y) of the L series approach span truss must be reinforced with additional gusset plates. Extra gusset plates are supplied for this purpose, and they must not be removed after erection.

6.2.3 Semi-Permanent Truss

a. General

This system of truss bridging comprises precision-made standard steel components which are assembled by bolting together to form bridge spans of through-truss design in the range of 30 to 60 metres.

The bridging is supplied complete with bearings, seismic lateral stops and buffers, railings, tools and equipment to be used in the assembly of the components into bridge spans, and with an Erection Manual.

The spans described in the Erection Manual are SP-class (semi-permanent). These are of single lane, and in spans 30 m, 35 m, 40 m, 45 m, 50 m, 55 m, and 60 m. They are designed in all cases for timber decks but with provision for concrete decks. Refer Figures 6.20 and 6.21 for details of the cross-sections.

Components are clearly marked to permit assembly in the sequence shown on the drawings. Components of the same mark are interchangeable. No component weighs more than 0.55 tonnes, and assembly is by hand tools provided with the bridge spans.

The system has been designed to permit progressive assembly by cantilever working from one bank, without the use of falsework in the river. This method is described in the Erection Manual, and it requires the use of a standard span as a anchor span and link steelwork which is provided with the system.

Two other methods of cantilever construction viz., single span launch (SSL) and multi-span launch (MSL) are available with these bridge systems. Both require special launching equipment as well as link steel, and permit assembly on the bank and launching across the river. Advice should be requested from the design consultants in these cases.

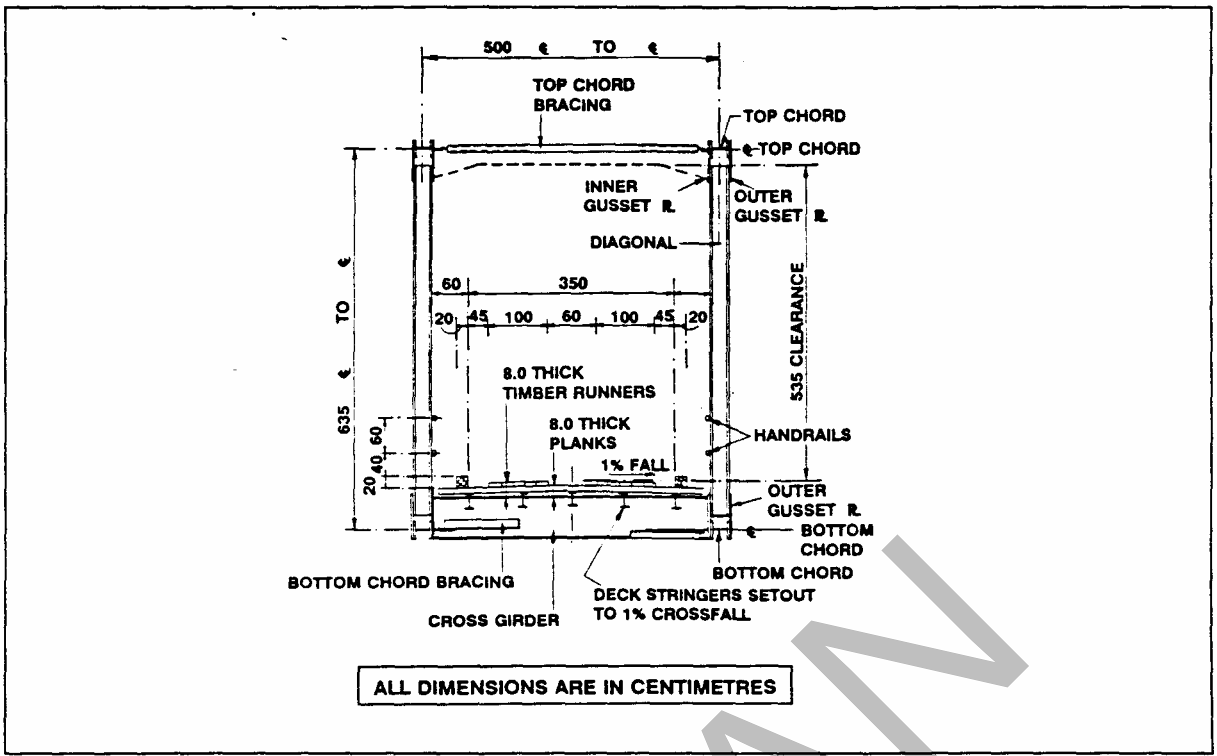


Figure 6.20 - Semi Permanent Truss with Timber Deck

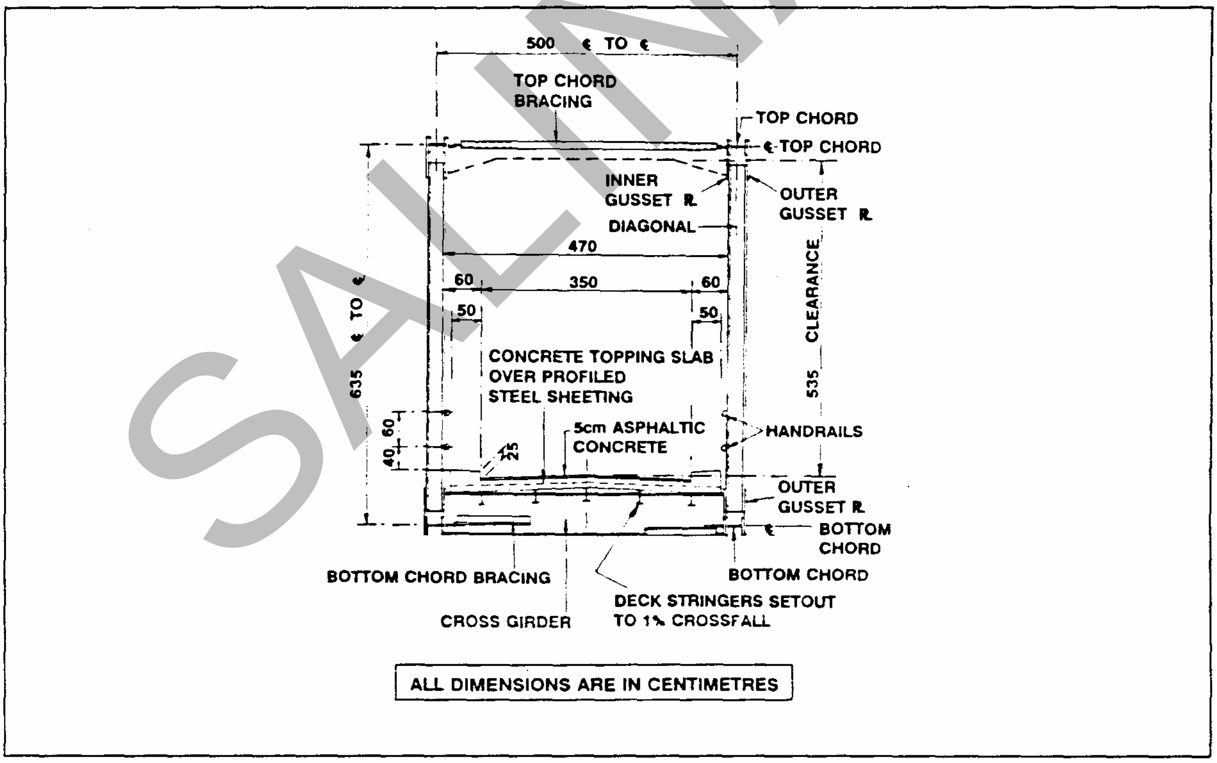


Figure 6.21 - Semi Permanent Truss with Concrete Deck

Other methods of assembly and erection such as part cantilever or erection on falsework are feasible. The principles laid down for the method described in the Erection Manual will also apply in these cases.

The construction of both the timber deck and the concrete deck on profiled steel sheeting including fixing to the steelwork are described and detailed. The installation of the bearings and lateral stops and buffers are also described in the Erection Manual.

This bridging system is planned to be of low maintenance. To this end all steelwork and bolts are galvanised and bearings are elastomeric. Nevertheless, basic maintenance procedures are described in the Erection Manual.

The Erection Manual includes Marking Plans and Parts Schedules together with descriptions of materials and parts, bolting and assembly and the cantilever erection method.

Design Criteria

Loading:	Loading Specifications for Highways Bridges No. 12/1970 (revised 1988) Direktorat Jenderal Bina Marga, Indonesia
Traffic:	One full lane of 70% D-loading (plus impact) or 70% T-loading
Footways:	None
Railings:	100 kg/m
Wind:	100 kg/m^2
Seismic:	Region 1 as Specification 12/1988 ($C = 0.3$)
Stream:	Superstructure assumed clear above flood level.
Temperature:	$\pm$ 15°C

Design Specifications

NAASRA Bridge Design Specification 1976.

AASHTO Standard Specification for Highways Bridges 1983.

Abutments, Piers

The abutments and piers are to be designed for the forces arising from the steel spans and other effects, and constructed to suit the bearing and span dimensions. Relevant forces and details are given for the various spans in drawings bound in the Erection Manual.

b. Components

The truss components for all SP-Class spans are based on M-series components (see Section 6.2.1.b).

c. Erection Methods

i. General

This Section describes the erection methods which are possible for the semi permanent truss bridging. These methods are basically described in the Erection Manual.

The choice of erection method should be carefully considered.

ii. Falsework

Refer to Section 6.2.1.c ii for general details of erection of truss bridging on falsework.

iii. Piece by Piece Cantilever

Refer to Section 6.2.1.c iii for general details of erection of truss bridging by the piece-by-piece cantilever method.

iv. Launching

Refer to Section 6.2.1.c iv for general details of erection of truss bridging by single span launching.

v. Combination Methods

There are a number of combinations of erection methods possible, but these are seldom used.

It is possible to construct part of the span on falsework and then cantilever the remaining part of the span, using kentledge as required to maintain stability. It is also possible to part-launch and part-construct using piece-by-piece cantilever construction.

d. Options

The semi permanent truss bridging may be constructed with either a timber deck or a reinforced concrete deck.

i. Timber Deck

The timber deck comprises timber planks spanning transversely between the longitudinal stringers with timber running strips and kerbs.

The timber deck is installed after the span is completed and is seated on the temporary timber supports.

The timber for the deck is not supplied with the steelwork but is provided by the Contractor. Bolts for the connection of the timber deck are supplied. The steel stringers and cross-girders are supplied already drilled to suit the bolting of the deck planks.

The timber planks must be seasoned hardwood and must be treated with creosote or other suitable timber preservative.

ii. Concrete Deck

The alternative deck treatment for semi-permanent truss bridging comprises profiled steel decking designed to span transversely between the longitudinal stringers, with a reinforced concrete slab to distribute wheel loads to the decking.

The components for the alternative decking (steel decking panels, panel fixing bolts, kerb plates, deck protection angles and scupper pipes) are supplied to the Contractor.

Drawings showing details of the reinforcement in the concrete deck are also supplied.

Construction joints in the concrete deck must be formed across the deck directly above a rib of the steel decking approximately 1.2 metres from the cross-girder. This joint must be formed and the use of an unformed construction joint should not to be permitted.

The steel decking is installed and the concrete deck poured whilst the span is seated on temporary timber supports before the permanent bearings are installed.

e. General Problems

The general problems as described in Section 6.2.1.e are also applicable to this series of bridging.

6.3 AUSTRALIAN GIRDER

6.3.1 General

The Australian girder bridging comprises precision-made standard steel components which are assembled by bolting together to form bridge spans of composite steel plate girder design in the range 20 to 30 metres.

The girders are supplied in three classes - A, B and C - for different roadway width and kerb/footway configurations. Spans in all classes have composite reinforced concrete decks. Refer to Figures 6.22, 6.23 and 6.24 for A-Class, B-Class and C-Class cross-sections respectively.

This bridging is supplied complete with bearings, seismic lateral stops and buffers, railings, deck angles, tools and equipment to be used in the assembly of the components into bridge spans, and with an Erection Manual.

Components are clearly marked to permit assembly in the sequence shown on the Drawings. Components of the same mark are interchangeable. No component weighs more than 2.4 tonnes, and assembly is by hand tools. All site connections are made by bolting.

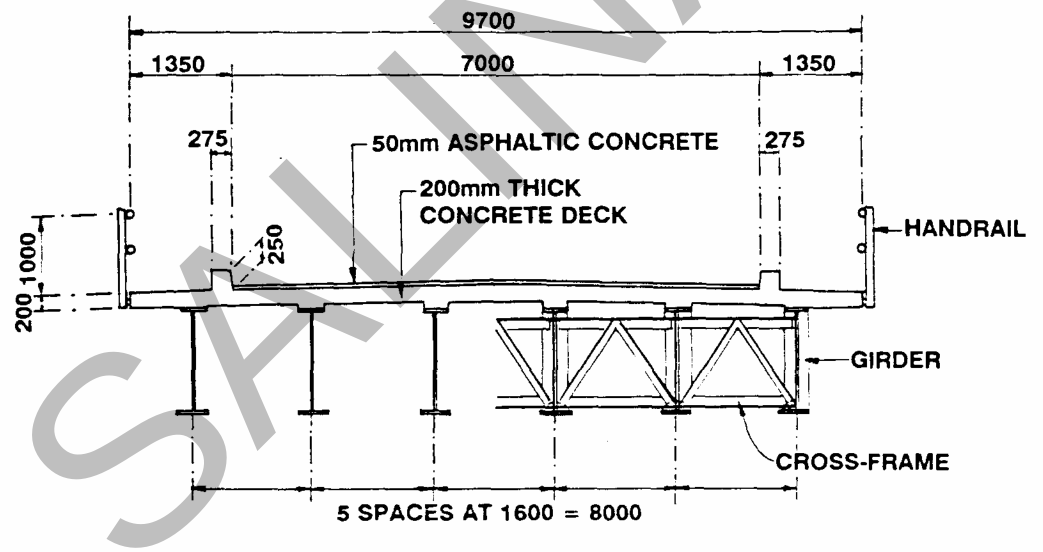


Figure 6.22 - Class A Permanent Girder

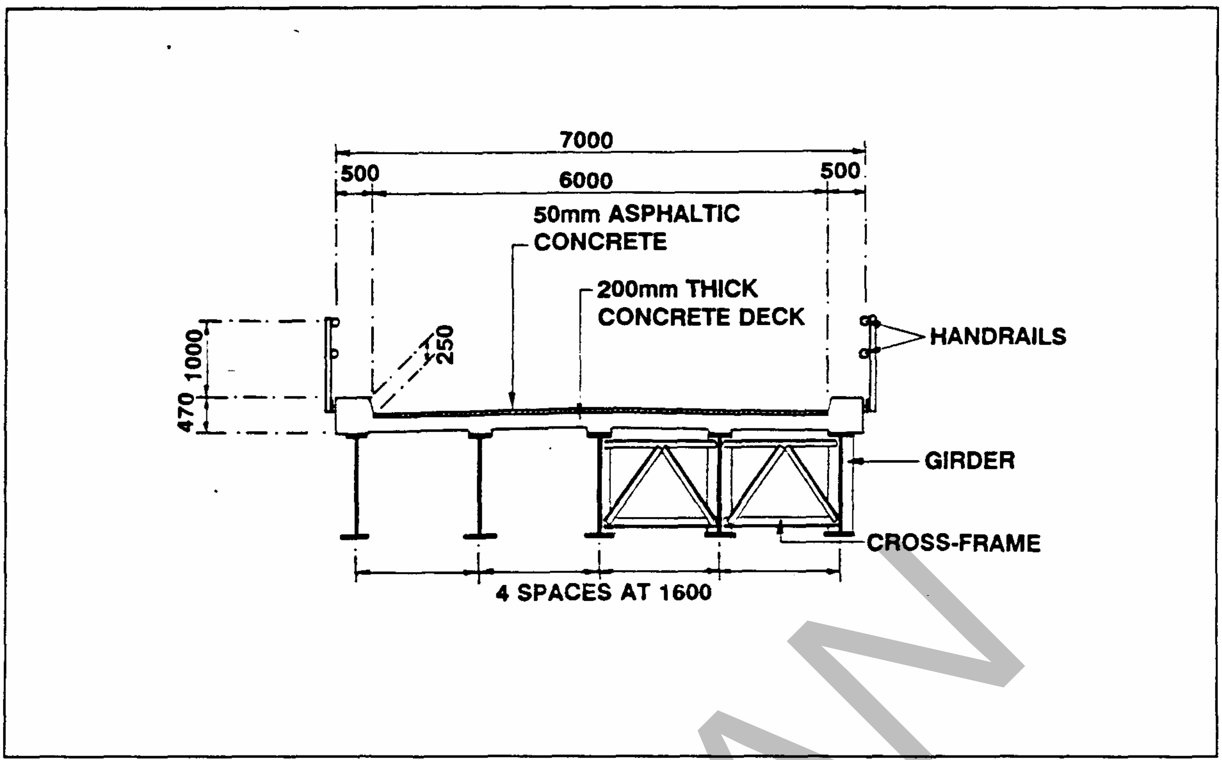


Figure 6.23 - Class B Permanent Girder

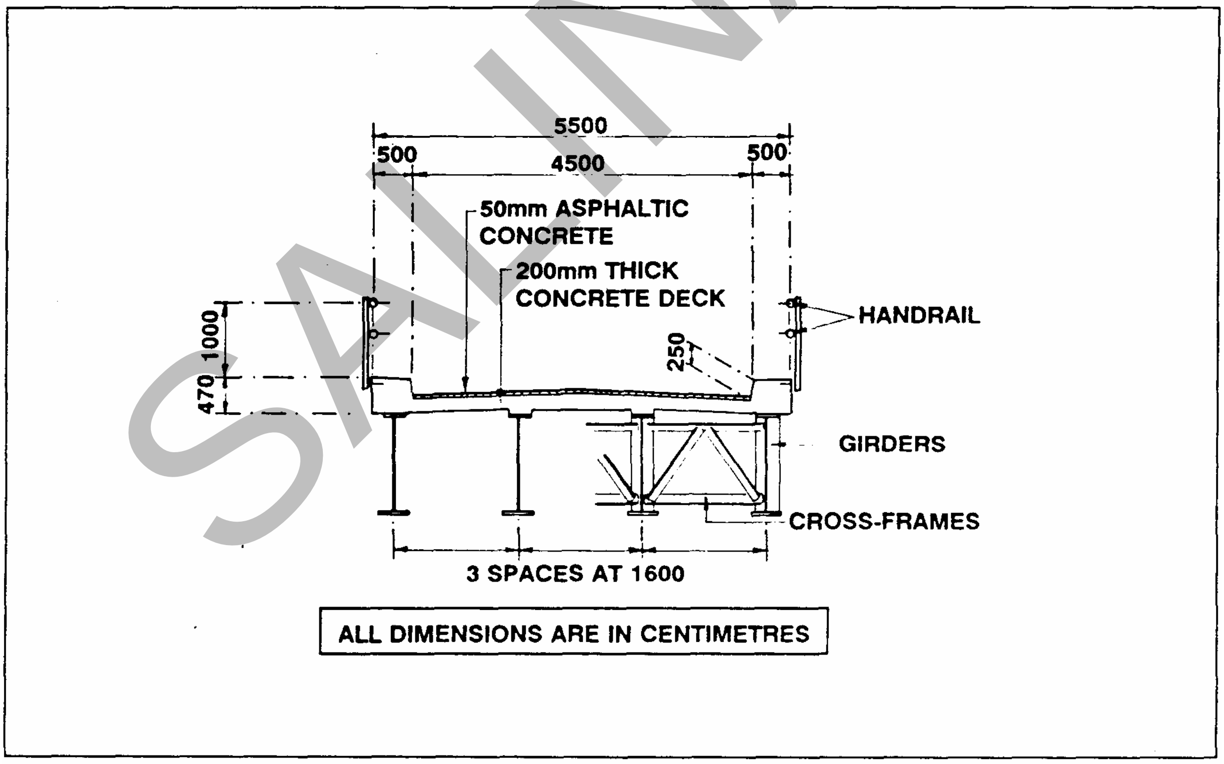


Figure 6.24 - Class C Permanent Girder

This Australian girder bridging system is planned to be of low maintenance. To this end all steelwork and bolts are galvanised and bearings are elastomeric. Nevertheless, basic maintenance procedures are described in the Erection Manual.

The Erection Manual includes Marking Plans and Parts Schedules together with descriptions of materials and parts, bolting and assembly and erection methods.

The standard components available may be adapted for a wide range of bridging forms and design criteria, such as: A, B or C Classes and alternative loading specification.

Design Criteria

Loading:	Loading Specifications for Highways Bridges No. 12/1970 (revised 1988) Direktorat Jenderal Bina Marga, Indonesia
Traffic:	A and B Class - two full lanes plus part lane, D-loading (plus impact) or T-loading (100%)
	C Class - one full lane, D-loading (plus impact) or T loading (100%)
Footways:	A-Class 500 kg/m^2 one metre wide each side B- and C-Class - nil.
Railings:	100 kg/m
Wind:	100 kg/m^2
Seismic:	Region 1 as Specification 12/1988 (C=0.3)
Stream:	Superstructure assumed clear above flood level.
Temperature:	± 15°C

Design Specifications

NAASRA Bridge Design Specification 1976.

AASHTO Standard Specification for Highways Bridges 1983.

6.3.2 Components

The numbering system used for the Australian girders is a similar philosophy to that used for the truss bridging. Table 6.3 list the Codes and their meanings. Note that there are no distinguishing letters denoting the class of bridging.

Table 6.3 Naming System for Girder Components

Code	Description
G	Girder Segments
F	Bracing Frames
GS	Splice Plates
GR	Handrails
GRB	Bearings
GHDB	Holding Down Bolt
SB	Lateral Stop
GDA	Deck Angle
GFP	Footway Plate
SP	Scupper Pipe

The Erection Manual contains a list of each component required for assembly and erection of the girder spans. The girder segments (G) used for the 25, 30 and 35 metre spans have suffixes of 1, 2 and 3 respectively. The other components are not as easy to distinguish and reference must be made to the parts list.

Tools in a tool box and 50 and 100 tonne jacks are supplied to the Contractor for the erection of each bridge.

6.3.3 Erection Methods

a. General

This Section covers the erection methods for the permanent girder bridging. These methods are described in the Erection Manual.

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Hydraulic jacks 50 or 100 tonne capacities (100t only required for A25, A30 and B30 spans).
4. Tool kit (for assembly of steel work).

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Material for the falsework supports
b. A minimum of 2 chain blocks for lifting the components into position
c. Method for hauling steel components from the bank onto the falsework
d. Jacking plates and timber packing for use in lowering the span
e. Temporary timber bearings

b. Falsework

This method is probably the most common one used. It may be used for single or multi-span structures. Temporary supports are used while the superstructure is being assembled. They are placed at locations to support each of the girder segments.

After completion of erection and before pouring the concrete deck the falsework is removed. This allows the superstructure to deflect as designed when the deck is poured. The formwork for the concrete deck is not to be supported on any of the falsework.

On many sites the existing bridge can be used as the basis for the falsework support and hence the cost is reduced.

One disadvantage is that a falsework bridge is required to be constructed across the river, thus presenting an obstacle to any boats navigating the river.

In addition there is a possibility of the falsework settling under the load of the girder if it is not properly supported. A falsework pier for a 30 metre span A class girder bridge must support about 9 tonnes dead load if trestles are installed near the end of each girder segment.

The installation of falsework across a river immediately before or during the wet season should be carefully considered as a flow in the river could demolish the falsework and the partially completed girder spans.

Components are often dragged along the ground out onto the falsework and the Supervising Engineer should ensure that this operation is carried out in such a manner so as not to cause damage to the protective coating of the girder elements.

The practice of erecting the girders and supporting them on temporary supports above the level of the bearings (before construction of the abutments) is not considered good practice. The presence of the girders makes it very difficult to properly construct the bearing plinths of the abutments and the safety aspects of supporting the girders for a considerable period of time need to be carefully considered.

c. Lifting

Contractors with access to cranes are able to assemble the girder segments into a completed girder and to lift the girder directly into position. The weights of Class A 20, 25 and 30 metre girders are 3.3t, 5.3t and 7.9t respectively.

The girders are only to be lifted at the lifting points as shown in the Erection Manual. The Supervising Engineer should ensure that only suitable lifting equipment (shackles, slings etc.) is used.

The first girder is to be supported under the top flanges against overturning until the second girder is placed and the bracing frames installed. Successive girders are lifted into position and the bracing frames fitted.

d. Launching

If the girders are erected by launching, two girders are assembled on one bank and launched over the crossing as a braced pair without any support within the crossing. The second pair of girders may be used as an anchor whilst launching.

This erection method requires the use of rollers and linking plates which must be fabricated and supplied by the Contractor. The Contractor must prepare and submit for approval detailed drawings of these components and the method proposed to be used.

This method is not commonly used.

6.3.4 General Problems

The following problems have been noted in the construction of steel girders. Refer to Section 6.2 ie. also.

Bolt tightening

It is absolutely essential that all bolt-tightening is completed before the concrete deck is poured. Failure to do so will result in a loss of camber of the structure. Permission to pour the deck should not be given until the bolt tightness certificate has been issued.

The gap as shown by the Load Indicating Washers must be between 0.15 mm and 0.25 mm. If the bolt is tightened to a gap of less than 0.15 mm there is a chance that the bolt will snap.

As tightening proceeds the head of each bolt should be marked to show that it has been tensioned correctly.

Any bolt that has been tensioned must not be reused and the bolt, nut and load indicating washer must be disposed of and replaced from the spares. The practice of reusing bolts must be prohibited.

Tension wrenches are not to be used to tighten friction grip bolts in this series of bridging as there is no correlation between a torque setting and the gap as shown by the Load Indicating washers.

Setting out of bearing centres

The longitudinal horizontal centre to centre distance of the bearings from abutment to abutment or abutment to pier should be checked against the Drawings

Construction of bearing plinths on piers and abutments

The bearing plinth levels should not be less than those shown on the Bearing and Seismic Buffer Details Drawing otherwise there may be a problem fitting the hydraulic jacks under the span.

Concrete quality in deck slab

The Australian girder bridging has been designed to have a composite reinforced concrete deck. The deck slab is connected to the steel girder via the shear studs welded to the girder. It is very important that the quality of concrete in the deck slab is at least as good as that assumed by the designers.

Refer to the sections on concrete production and construction for details of this work.

Poor quality of concrete in lateral stops

It is important that both seismic and lateral stops are constructed as indicated on the Drawings because these bearings are vital to the structure if an earthquake should occur. The concrete used in these stops must be of good quality.

Clearances between the face of the rubber and the concrete must comply with the requirements of the Drawings.

Omission of sections of concrete until after completion of girder erection

It is recommended that the back walls of the abutments or piers should not be completed to the full height until the deck has been poured and the girder set onto its permanent bearings.

The reinforcement which protrudes from the lower part of the wall should not be bent so as to cause sharp kinks which may become potential weak points.

Note that the height of the backwall has been designed based on a 50 mm thick asphaltic concrete layer. Any variation in the thickness of this layer should be reflected in a corresponding change in the height of the backwall.

Damage caused to components by poor storage and handling

Lost or damaged components usually must be replaced by spares which can take a long time. In many cases, the work on the site will come to a standstill. Accordingly, careful inspection is required of all components when they are received on site to determine if there are any missing or damaged components.

Steel members are to be handled, lifted and stored so as to avoid damage to the member and overstressing or damage to the protective treatment.

Prior to the arrival of steel components on site, an area of suitable size (to contain all the steel) should be prepared to receive all the components as they arrive at the site. The area should be as close as possible to the bridge site to avoid unnecessary double-handling of the material.

All members are to be stacked on site, on timber packing, level and clear of the ground. Refer to the Erection Manual for typical stacking details.

Smaller components, such as gusset plates and splice plates, should be stacked in neat bundles above ground level on a timber platform and not loose on the ground.

Handrail pipe should be stacked on timber packing and supported in such a manner that the pipe will not be bent.

Bolts, bearings and deck seals should be stored under cover, in a small shed if possible. If bolts, nuts and washers are left loose on the ground they will soon disappear. Note that all bolts, nuts and washers are to be kept dry up to the time of installing the bolts. This is to prevent the lubricating wax being washed off.

All the tools supplied on loan for the duration of the project should be stored in a secure place in the containers provided.

Components are often stored off site and brought in smaller quantities to the site during erection. The requirements for temporary storage adjacent to the bridge site are identical to those for the main storage area. The Contractor should not be permitted to stack components in such a way as to cause damage to the components or their protective coatings.

6.4 DUTCH TRUSS (HOLLANDIA KLOOS)

6.4.1 General

The Hollandia Kloos system of truss bridging comprises precision-made standard steel components which are assembled by bolting together to form bridge spans of through truss design in the range 40 to 105 metres.

The permanent spans are supplied in three classes - A, B and C - which differ in roadway width and kerb/footway configuration. Spans in all classes have composite reinforced concrete decks. Refer to Figure 6.25 for details of the typical cross-section.

This bridging is supplied complete with bearings, railings, road edges and tools and equipment to be used in the assembly of the components into bridge spans.

Components are clearly marked to permit assembly in the sequence shown on the Drawings. Components of the same mark are interchangeable. No component weighs more than 1.8 tonnes for bridges up to and including 60 metre spans. Assembly is by hand tools provided with the bridge spans.

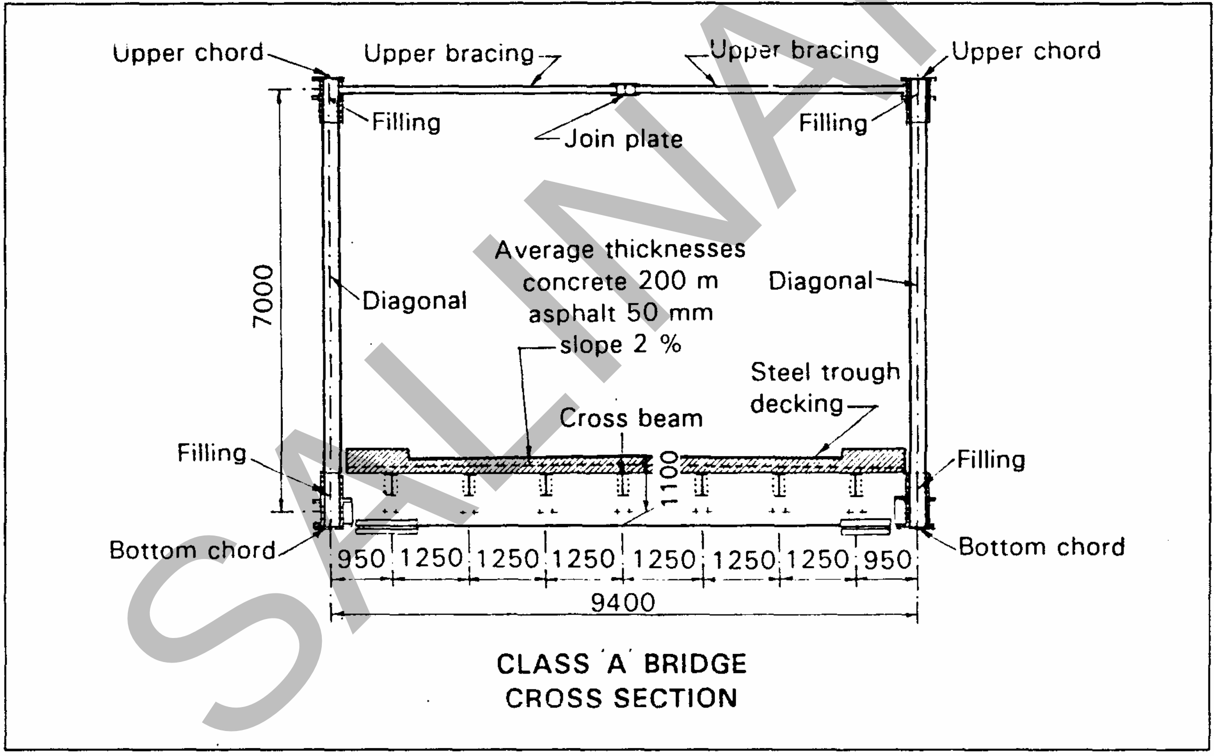


Figure 6.25 - Hollandia Kloos Truss

The system has been designed to permit the progressive assembly by cantilever working from one bank without the use of falsework in the river. This method of erection is described in the Erection Manual. It requires the use of a standard span as an anchor span and link steelwork which is provided with the system.

Other methods of assembly and erection such as erection on falsework are feasible. The principles laid down for the methods described in the Erection Manual will apply in these cases.

The construction of the concrete deck and the installation of the bearings are also described in the Erection Manual.

This bridging system is planned to be of low maintenance characteristics. To this end all steelwork and bolts are galvanised and bearings are elastomeric.

Design Criteria

Loading:	Loading Specifications for Highways Bridges No. 12/1970 (revised 1988) Direktorat Jenderal Bina Marga, Indonesia
Traffic:	A and B Class - two full lanes plus part lane, D-loading (plus impact) or T-loading (100%)
	C Class - one full lane, D-loading (plus impact) or T-loading (100%)
Footways:	A- and B- Class 500 kg/m^2 one metre wide each side C-Class - nil.
Railings:	100 kg/m
Wind:	100 kg/m^2
Seismic:	Region 1 as Specification 12/1988
Stream:	Superstructure assumed clear above flood level.
Temperature:	± 15°C

Design Specifications

Design Specifications for steel bridges, Draft 1978, Bina Marga

AASHTO Standard Specification for Highways Bridges 1983

The bridge superstructure is considered as a freely supported structure designed by elastic methods and sufficiently cambered in order to compensate for 150% of total dead load deflections.

The bolted connections are designed as friction type connections but are also able to be checked as bearing type connections. All bolts used in these connections are quality marked, type 1, high strength bolts in accordance with ASTM designation A 325.

6.4.2 Components

The component numbering system for the Dutch bridging is based on a numerical system.

The components cannot be readily identified from the component part number other than by a suffix to denote Left or Right . The numbering system is set up so as to generally follow the erection sequence, that is erection commences with '1'- Cross Beam and progresses more or less sequentially.

There are a number of components required as part of the linking system and these are all identified as a '500' series, for example 509 - Upper Chord.

6.4.3 Erection Methods

a. Piece-by-Piece Cantilever

This method of erection is illustrated in Figure 6.26.

The anchor span used is normally a 50 or 60 metre span. The anchor span has ballast added to the trailing end such that the restoring moment is greater than the overturning moment by at least 25 percent when the permanent truss is fully cantilevered.

As an example, a B60 anchor span needs 24 tonnes of ballast to allow cantilevering of an A 60 permanent span. For the purposes of calculation, erection loads of 2 tonnes are placed near the end of the cantilevered span (for example - 11 m from the end of an A60 span).

If the longitudinal girders are omitted from the anchor span the amount of ballast must be correspondingly increased.

The anchor span is first constructed on temporary supports behind the abutment and the ballast added.

The linking diagonal and top chord are added to the anchor span.

The first lower triangular bay is constructed. This includes the cross bracing in the floor and the longitudinal girders.

The upper triangular bay is then constructed, including the first set of top bracing.

The second lower triangular bay is constructed. The truss is then 20 metres from the abutment.

This process is repeated until 10 metres from the far abutment. The lower chords, cross girder and bracing are then placed onto temporary supports on the far abutment and connected to the cantilevered truss.

The final bay of the truss is then completed (on the far abutment). The span is jacked up and the linking steel and anchor span are removed.

The bridge is jacked down onto the permanent bearings and the decking, concrete and railings etc are constructed.

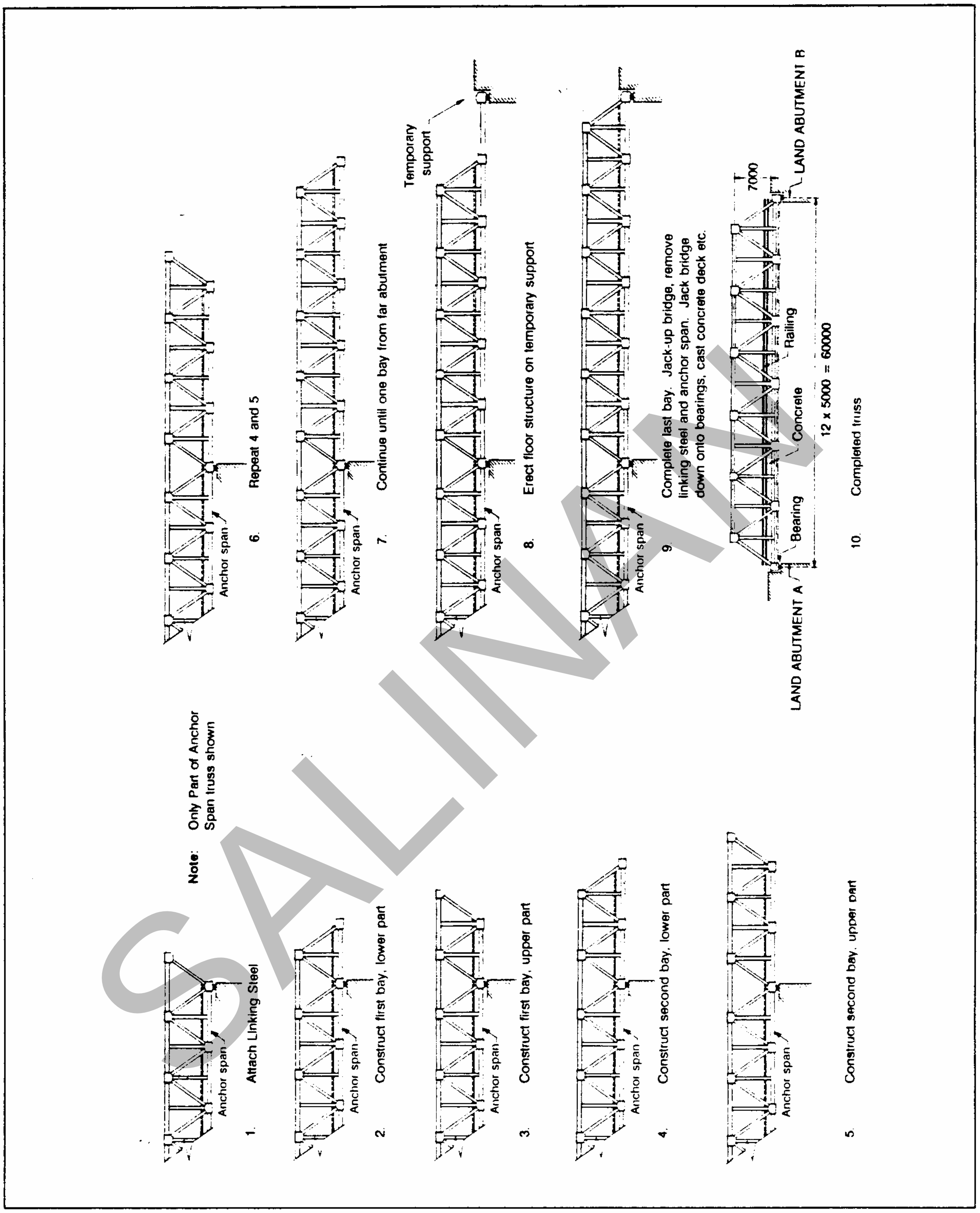


Figure 6.26 - Dutch Truss - Erection by Cantilever

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Anchor truss span
4. Erection link kit (linking steel).
5. Hydraulic jacks 75 tonne capacities.
6. Tool kit (for assembly of all steel work and link kit).

In addition to the above equipment the Contractor needs to supply and install the following items:

a. Support framework or timber cribwork as temporary support to the permanent span first cross girder.
b. Suitable material for kentledge (counterweight). For example sand packed in bags, concrete blocks, steel components, rocks etc. but whatever is used the weight must be known.
c. Jacking plates and timber packing for use in lowering the span.
d. Equipment for hauling steel components from the bank across the stream and lifting and supporting in position.
e. Temporary timber bearings.

b. Falsework

Temporary supports are used while the superstructure is being assembled. They are placed in the river bed between the substructures as shown in Figure 6.27.

After completion of erection and before pouring the concrete deck, the falsework is removed. This allows the superstructure to deflect as designed when the deck is poured.

The biggest advantage of this method is that there is no need for additional anchor spans, Linking kit or kentledge (counterweight) which the piece-by-piece cantilever method requires.

In addition, there is no need for heavy lifting equipment as the heaviest component is only 1.74 tonnes in weight. It is a labour intensive method with a minimum of lifting equipment required.

On many sites the existing bridge can be used as the basic for the falsework support and hence the cost is reduced.

One disadvantage is that a falsework bridge is required to be constructed across the river, presenting an obstacle to boats navigating the river. In general, a falsework pier or trestle is set up under each cross girder at spacing of about 5 metres.

In addition, there is a possibility of the falsework settling under the load of the truss if it is not properly supported. A falsework pier for an A-class bridge must support about 12 tonnes dead load for the steel truss. The erection method described in the Erection Manual shows two piles per pier driven at 5 metre centres longitudinally and 9.4 metres apart transversely.

The sequence of erection is described in detail in the Erection Manual.

The truss floor sections are assembled on the falsework across the span and connected together. The bridge is fixed in the transverse direction at the abutments.

The first two bays of diagonals and verticals are assembled and the top chord and associated vertical added, followed by the top bracing to complete two bays of the truss.

The next bay is then constructed with diagonals and vertical, followed by top chord and vertical and the cross bracing. The other bays are constructed in a similar manner. This process is illustrated in Figure 6.28.

The truss is then jacked up, piles removed, the bridge jacked down onto the bearings, and the deck and railings etc. constructed.

The installation of falsework across a river immediately before or during the wet season should be carefully considered as a flow in the river could demolish the falsework and the partially-completed truss.

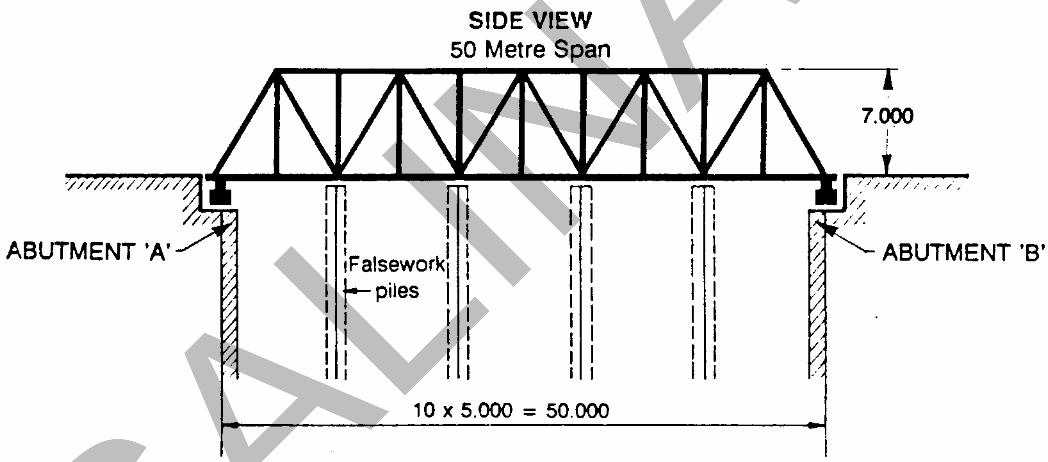


Figure 6.27 - Erection on Falsework

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Hydraulic jacks 75 tonne capacity.
4. Tool kit (for assembly of all steel work and link kit).

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Material for the falsework supports
b. A minimum of 2 chain blocks for lifting the components into position
c. Equipment for hauling steel components from the bank onto the falsework
d. Jacking plates and timber packing for use in lowering the span
e. Temporary timber bearings

A variation of this method is to assemble the superstructure on falsework and to move it laterally onto the abutments using rollers.

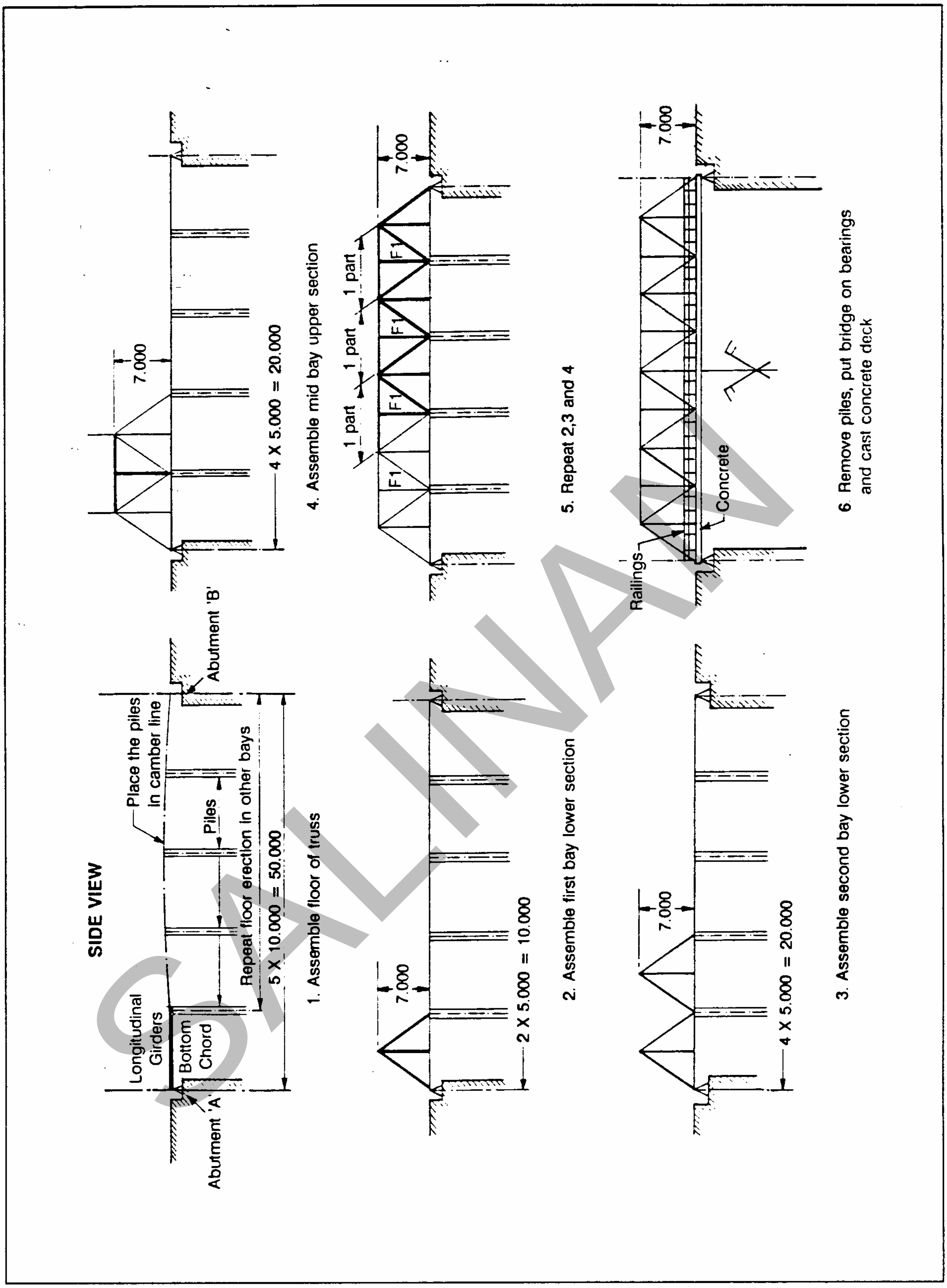


Figure 6.28 - Erection on Falsework - Dutch Truss

c. Semi-Cantilever Erection

This method is a combination of the other two methods and is shown in Figure 6.29.

Erection Equipment Required

The following erection equipment is required with the main steelwork and is to be requested from PALAN JAKARTA if not already available.

1. Erection Manual.
2. Construction Drawings.
3. Anchor truss span
4. Erection Link Set (linking steel).
5. Hydraulic jacks 75 tonne capacities.
6. Tool kit (for assembly of all steel work and link kit).

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Support framework or timber cribwork as temporary support to the permanent span first cross girder.
b. Temporary falsework pier support.
c. Suitable material for kentledge (counterweight). For example sand packed in bags, concrete blocks, steel components, rocks etc. but whatever is used the given weight must be known.
d. Jacking plates and timber packing for use in lowering the span
e. Equipment for hauling steel components from the bank across the stream and lifting and supporting in position.
f. Temporary timber bearings.

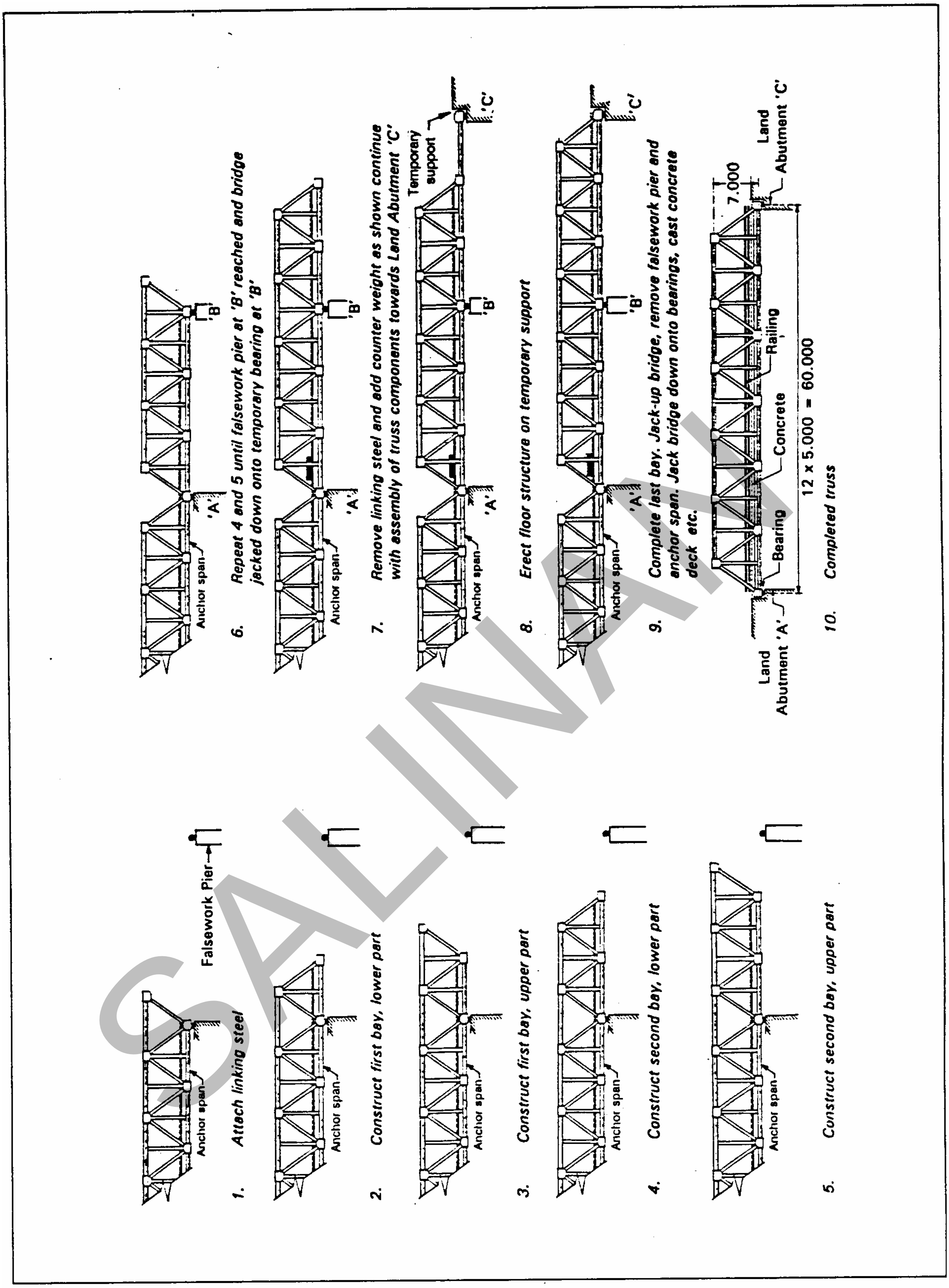


Figure 6.29 - Erection by Semi-Cantilever Method - Dutch Truss

6.4.4 General Problems

- The use of stay plates to make up the chord sections from channels requires the use of many more bolted connections than other styles of truss. For example, a 50 metre B-Class Dutch truss will require about 8 700 bolted connections compared with about 5 800 for an Australian truss of the same class and span.

- The use of a torque wrench for friction-grip bolted connections (especially using galvanised bolts and nuts) is unreliable. To improve the level of confidence, a check should be made of the manual torque wrench and the corresponding actual tension in the bolt at the setting given in the Erection Manual. These checks should be carried out at the beginning of each day's tightening and whenever the bolt diameter is changed. The average tension of at least three bolts should be calculated.

- The Dutch truss bridges have a different bearing system to the Australian trusses and it should be noted that the bridge is jacked down onto the permanent bearings **prior** to installation of the decking and construction of the concrete deck.

6.5 AUSTRIAN TRUSS

6.5.1 Permanent Truss

a. General

This Austrian (Wagner-Biro) system of truss bridging comprises precision-made standard steel components which are assembled by bolting together to form bridge spans of through truss design in the range 35 to 60 metres.

The spans are A-Class, B-Class and C-Class which differ in roadway width and kerb/footway configuration. Spans in all classes have concrete decking supported by corrugated trapezoidal steel sheets, supplied as part of the bridge system.

The bridges are supplied complete with bearings, seismic lateral stops and buffers, railings and tools and equipment to be used in the assembly of the components into bridge spans.

Components are clearly marked to permit assembly in the sequence shown in the drawings. Components of the same mark are interchangeable. No component weighs more than 1.5 tonnes.

The system has been designed to permit progressive assembly by cantilever working from one bank, without the use of falsework in the river. This method for truss span erection is described in the Erection Manual. It requires the use of another span as an anchor span and link steelwork which is provided with the system.

Other methods of assembly and erection such as part-cantilever or erection on falsework are feasible.

The installation of the bearings and seismic lateral stops and buffers are also described in the Erection Manual.

The Austrian bridge system is planned to be low maintenance, so all steelwork and bolts are galvanized.

The Erection Manual includes Marking Plans and Parts Schedules for all spans, together with descriptions of materials and parts, bolting and assembly and the cantilever erection method.

Attention is drawn to the specified method of bolt tightening specified using a torque wrench, and the method of testing the tightness of the bolts, as described in the Erection Manual.

The Austrian truss is jacked onto the permanent bearings like the Dutch truss, prior to the concrete deck being poured.

Design Criteria

Loading: Loading Specifications for Highway Bridges No. 12/1970 (Revised 1988) Direktorat Jenderal Bina Marga, Indonesia:

Traffic: A and B-Class -

C-Class - one full lane, D-loading (100% plus impact) and T-loading (100%) for corrugated steel sheet resp. stringers and crossgirders.

Wind: 100 kg/m^2

Seismic: Earthquake coefficient 0,2.

Stream: Superstructure assumed clear above flood level

Temperature: ± 15°C

Structural Design:

Structural analysis and design is based on elastic methods to the allowable stresses for appropriate materials, in accordance with "DIN-Standards".

Cross Section:

The cross-section of the truss bridge is illustrated in Figure 6.30. Roadway width, horizontal clearance and basic dimensions of the concrete deck are shown in Figure 6.31 (Class C bridge shown as example).

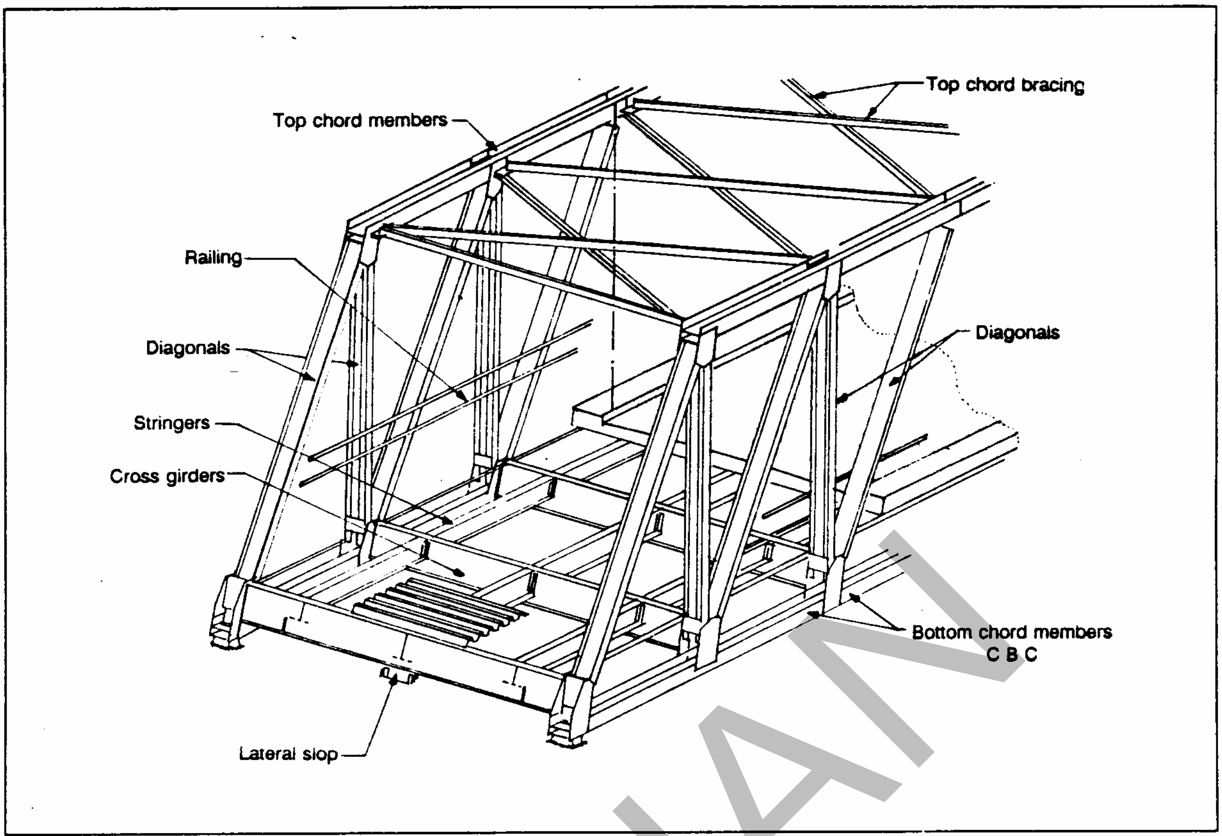


Figure 6.30 - Austrian Permanent Truss - Typical Cross Section

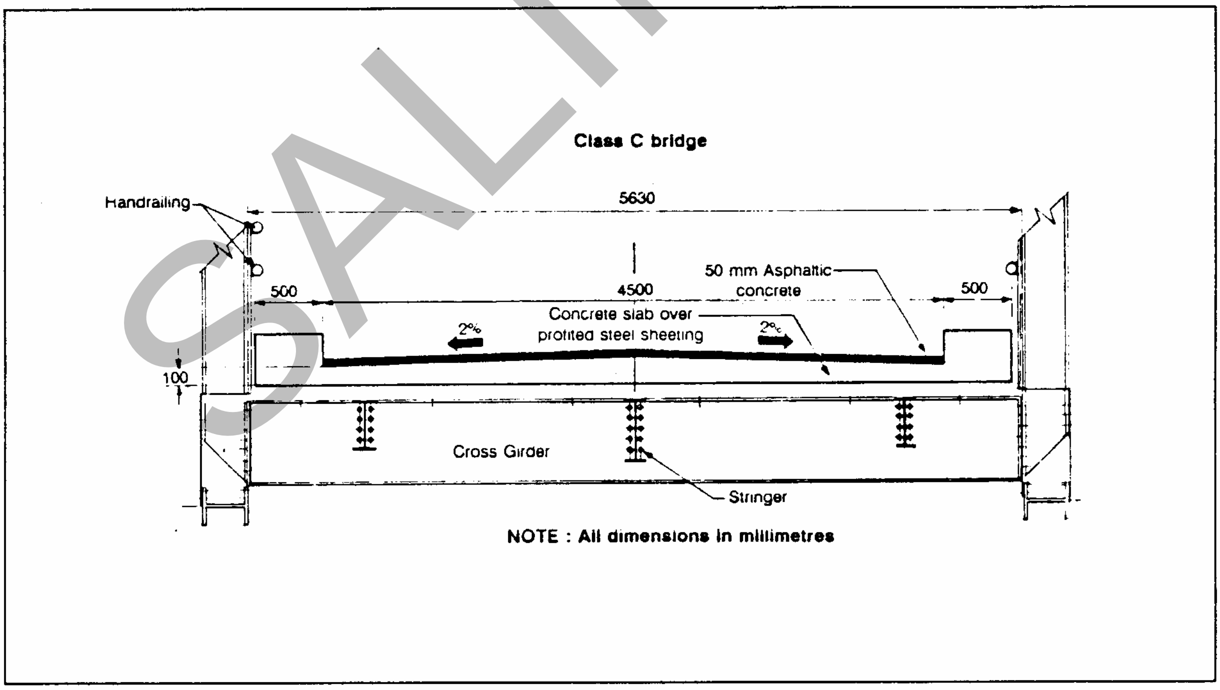


Figure 6.31 - Austrian Permanent Truss - Concrete Deck

b. Components

The components used for the Austrian Bridges are designated by a Class Prefix (A, B or C) with an identification Code and a mark number eg. CTC11 = Class C top chord mark 11.

Components such as hand railing which are independent of the class of bridge are not prefixed with A, B or C.

Codes for the main components used in the Austrian truss bridging are given in Table 6.4.

Table 6.4 Naming System for Austrian Truss Components

Code	Description
TC	Top chord
BC	Bottom Chord
D	Diagonal
CG	Cross Girder
S	Stringer
GP	Gusset plate
WB	Wind Bracing
TS	Trapezoidal Steel Sheeting
RNTB	Reinforced Neoprene Type bearing
HR	Hand Railing

The erection components are also prefixed with the Class of bridge for which they are intended.

c. Erection Methods

i. General

This Section covers the erection methods for the permanent truss bridging. These methods are described in the Erection Manual.

The choice of erection method at each different site should be carefully considered.

ii. Piece-by-Piece Cantilever Erection

This method of erection is shown in Figure 6.32.

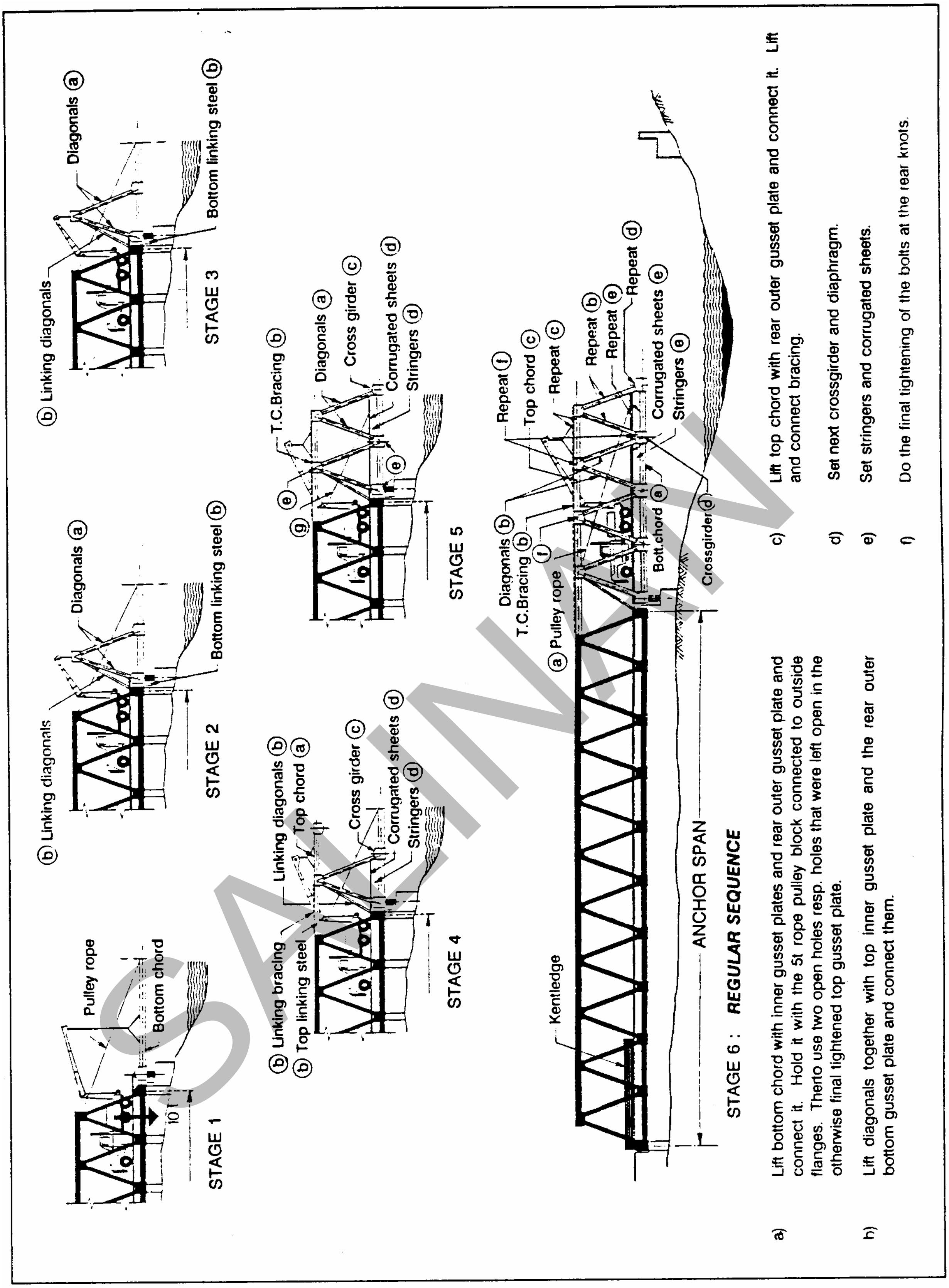


Figure 6.32 - Austrian Permanent Truss - Erection by Cantilever Method

The anchor span used is normally the same length as the permanent span. The anchor span must have ballast added to the trailing end as shown on the erection drawings.

For example, a C 60 anchor span needs 16 tonnes of ballast to allow cantilevering of a C 60 permanent span.

The anchor span is first constructed on temporary supports behind the abutment and the ballast added.

The cross girder and linking stringers are placed and connected to the anchor span.

The bottom chord is connected to the first cross girder and held in position by the use of a 5 tonne rope pulley block attached to the end diagonal of the anchor span.

The diagonals are added and the linking steel components (chord, diagonals and bracing) are installed.

Erection proceeds as set out in the Construction Procedure Drawing. The permanent span is jacked onto the permanent bearings and the anchor span removed. The concrete deck is then poured.

Note that there are small exterior stringers to support the outside of the steel decking.

If a crane is used, timbers must be placed over the steel trapezoidal sheeting to provide access. A minimum size of 500 mm wide by 50 mm thick is recommended for each wheelpath.

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Anchor truss span
4. Erection link kit (linking steel).
5. Hydraulic jacks 50 tonne capacities.
6. Tool kit (for assembly of all steel work and link kit).

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Support framework or timber cribwork as temporary support to the permanent span first cross girder.
b. Suitable material for kentledge (counterweight). For example sand packed in bags, concrete blocks, steel components, rocks etc. but whatever is used the weight must be known.
c. Two 5 tonne rope pulley blocks.
d. Jacking plates and timber packing for use in lowering the span
e. Equipment for hauling steel components from the bank across the stream and lifting and supporting in position.
f. Temporary timber bearings.

iii. Falsework

Temporary supports are used while the superstructure is being assembled. They are placed in the river bed between the substructures as shown in Figure 6.33.

After completion of erection and before pouring the concrete deck, the falsework is removed. This allows the superstructure to deflect as designed when the deck is poured.

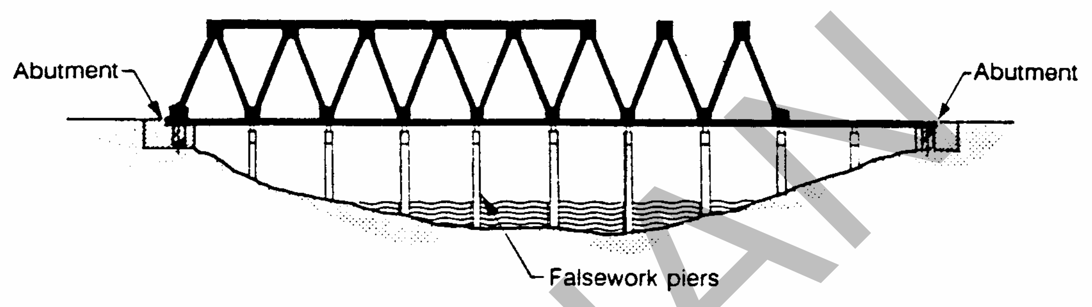


Figure 6.33 - Austrian Permanent Truss - Erection on Falsework

The biggest advantage of this method is that there is no need for the additional anchor spans, Linking kit or kentledge (counterweight) which the piece by piece cantilever method employs.

In addition there is no need for heavy lifting equipment as the heaviest component is only 1.5 tonnes in weight. It is a labour intensive method with a minimum of lifting equipment required.

On many sites the existing bridge can be used as the basis for the falsework support and hence the cost to the Contractor is reduced.

One disadvantage is that a falsework bridge is required to be constructed across the river, thus presenting an obstacle to any boats navigating the river. In general a falsework pier or trestle is set up under each cross girder, a spacing of about 5 metres.

In addition there is a possibility of the falsework settling under the load of the truss if it is not properly supported. A falsework pier for an A class bridge must support about 12 tonnes dead load for the steel truss. The erection method described in the Erection Manual shows two piles per pier driven at 5 metre centres longitudinally. The transverse distance depends on the class of bridge being constructed.

The sequence of erection is described in detail in the Erection Manual Drawings.

Basically the truss floor sections are assembled on the falsework across the span and connected together. The diagonals, top chords and the top chord bracing are assembled across the span.

The truss is then jacked up, the piles removed, the bridge jacked down onto the bearings and the deck, railings etc. constructed.

The installation of falsework across a river immediately before or during the wet season should be carefully considered as a large flood could demolish the falsework and the partially completed truss.

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Hydraulic jacks 50 tonne capacity.
4. Tool kit (for assembly of all steel work and link kit).

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Material for the falsework supports
b. A minimum of 2 chain blocks for lifting the components into position
c. Equipment for hauling steel components from the bank onto the falsework
d. Jacking plates and timber packing for use in lowering the span
e. Temporary timber bearings

6.5.2 Semi-Permanent Truss

a. General

This system of bridging comprises precision-made standard steel components which are assembled by bolting together to form bridge spans of through truss design in the range of 15 to 35 metres. The bridging is supplied complete with bearings, railings, tools and jacks to be used in the assembly of the components into bridge span.

Components are clearly marked to permit assembly in the sequence shown in the drawings. Components of the same mark are interchangeable. No component weighs more than 335 kg and assembly may be carried out using hand tools only.

The spans are designed to support timber decking and are designed for highway bridge loading BM 70.

The system has been designed to permit complete assembly by the cantilever method using an anchor span. This method requires the use of a standard span as an anchor span and linking steelwork which is provided. Construction on falsework is also feasible.

Design Criteria

Loading:	Loading Specifications for Highway Bridges No. 12/1970 (Revised 1988) Direktorat Jenderal Bina Marga, Indonesia:
Traffic:	**One full lane plus part lane :** 70% D-Loading (plus impact) and 70% T-Loading (timber deck only)
Wind:	100 kg/m^2
Seismic:	Earthquake coefficient 0,2.
Temperature:	± 15°C

Structural Design:

Structural analysis and design is based on elastic methods to the allowable stresses for appropriate materials, in accordance with "DIN-Standards".

Cross section:

The cross-section of the truss bridge is illustrated in Figure 6.34. Roadway width and horizontal clearance is indicated, basic dimensions of the timber decks are also shown.

b. Components

The identification codes set out in Section 6.5.1.b are also used for the semi permanent truss components. There is no class prefix ie. top chords are referred to as TC.

All structural steel components in the truss spans are fabricated or rolled from ST 52 and/or ST 37 complying with DIN-Standards.

Bolts for all structural connections are high strength, Grade 10.9 in accordance with DIN-Standards and with nuts and washers of equivalent hardened type.

All bolts, nuts and washers and all structural components are supplied galvanized in accordance with DIN-Standards with an average weight of coating not less than 610 gr/m^2.

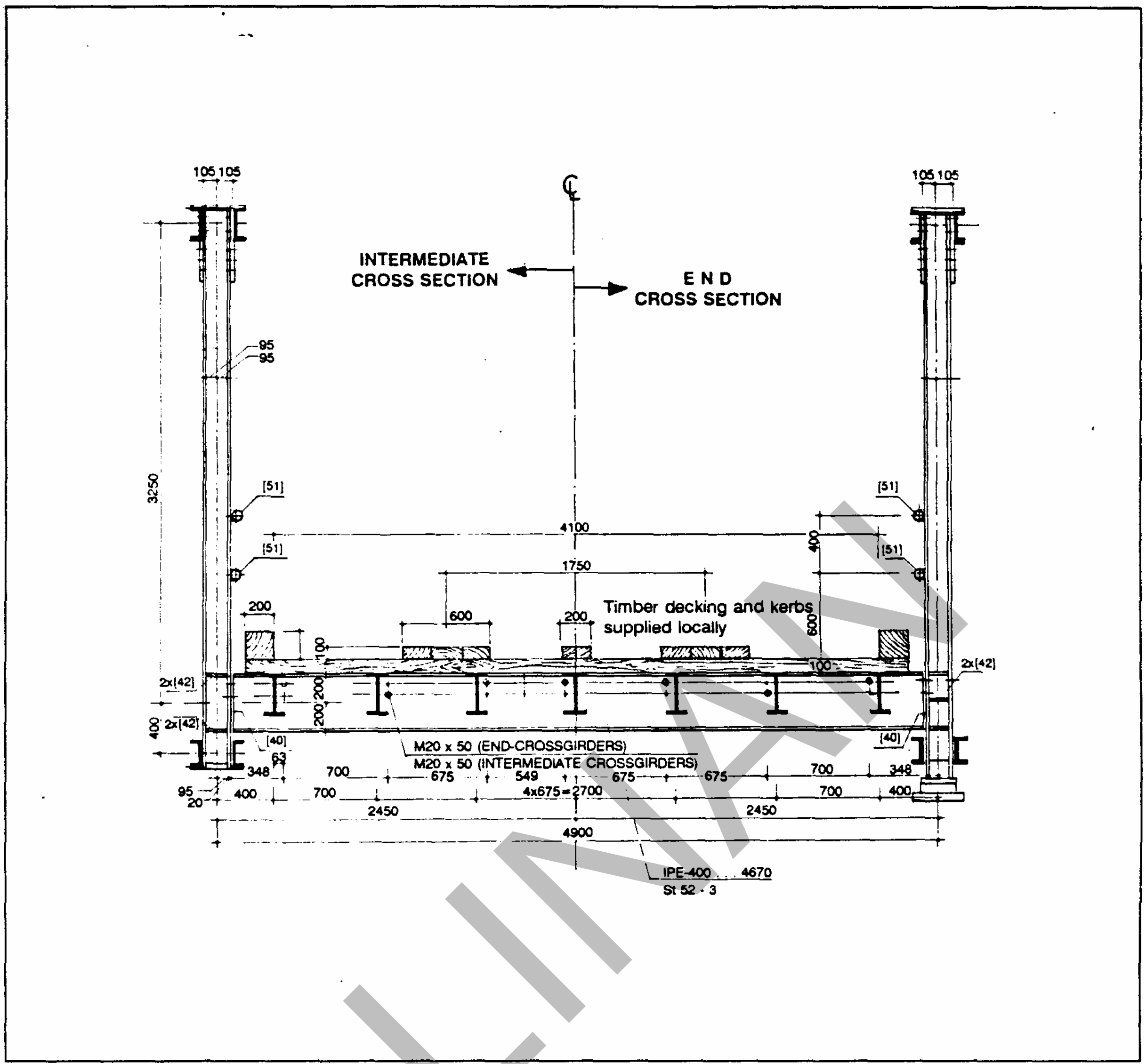


Figure 6.34 - Austrian Semi Permanent Truss - Cross Section

c. Erection Methods

The methods and comments in Section 6.5.1.c for Austrian permanent truss bridging are also applicable to the semi permanent bridging.

6.6 COMPARISON OF AUSTRALIAN, DUTCH AND AUSTRIAN PERMANENT TRUSS BRIDGING

FEATURE	AUSTRALIAN	DUTCH	AUSTRIAN
Spans	35 to 60 in 5m, 80, 100	40 to 60 in 5m, 100, 105	35 to 60 in 5m
Concrete Deck	Composite slab (exc 80,100 topping slab)	Topping slab	Topping Slab
Steel Decking	No (35 m - 60 m) Yes (80 and 100m)	Yes	Yes
Bolts - tightening system	Load Indicating Washers	Torque Wrench	Torque Wrench
Erection System	FW PxP CANTILEVER (80,100 M 1/2 SPAN) Semi Cantilever SSL MSL	FW PxP CANTILEVER Semi Cantilever	FW PxP CANTILEVER Semi Cantilever
Jacking down onto permanent bearings	After deck poured	Before deck poured	Before deck poured
Number of bolts in a B 50 span	5800	8700	
Weight of a B 50 span	74 ton (C50 = 54 ton)	96 ton	69 ton (C50)
Maximum Component Weight	1.5 t (H series 3.0 t)	1.8 t	1.5 t

6.7 TEMPORARY BRIDGING (TRANSPANEL AND MABEY COMPACT)

6.7.1 General

Panel bridges are single lane temporary bridges which can be erected in a short time using existing substructures or temporary substructures if required. They are all based on steel truss panels which are connected together using high strength pins and bolts.

The panels are arranged to form side trusses of various capacities to suit the span (panels may be used in pairs side by side, stacked vertically or additional reinforcing members used to increase the span and load capacity). The panels are fabricated to achieve a vertical camber to compensate for the dead load deflection. The deck comprises transoms and stringer units which support timber cross planks and running boards. Kerbs are also of timber. Some systems offer alternative steel decking.

The usual range of spans is 10 to 50 metres. Multiple spans are possible either as continuous spans or by forming (broken-backed) simply supported spans by removal of the top panel chord pin or a proprietary system (for example the Mabey span junction-post system).

Spans up to 80 metres are possible with some systems (e.g. Mabey DDR1H or DDR2).

The systems are designed to be assembled rapidly with the aid of light lifting equipment on one bank and progressively launched into position by rolling out over the crossing. A cantilever launching nose, assembled from standard components, is used for this purpose.

Other methods of assembly and erection, such as erection on falsework, are feasible. If suitable craneage is available the complete structure (except the timber decking) may be assembled on the bank and lifted into position. The weight of an undecked 20 metre span is about 25 tonnes.

All recent panel bridge components are galvanised, earlier Bailey bridge components were painted.

6.7.2 Australian Transpanel Bridge

a. General

This type of bridging is suitable for temporary or semi-permanent applications because assembly is rapid and foundation requirements are minimal. It comprises standardised steel components which are assembled in a specified arrangement and sequence and connected by high strength pins and bolts to form through-truss bridge spans in the range 10 to 50 metres.

The basic load-carrying component is the truss panel. Panels are arranged in particular combinations and reinforced as necessary to form side trusses of various capacities to suit the span. The panels are fabricated such that a vertical camber in the side trusses is achieved automatically. The camber is to compensate for deflection caused by the self-weight of the span. The deck comprises transoms and stringer units which support timber cross planks and running boards. Figure 6.35 shows a typical cross section.

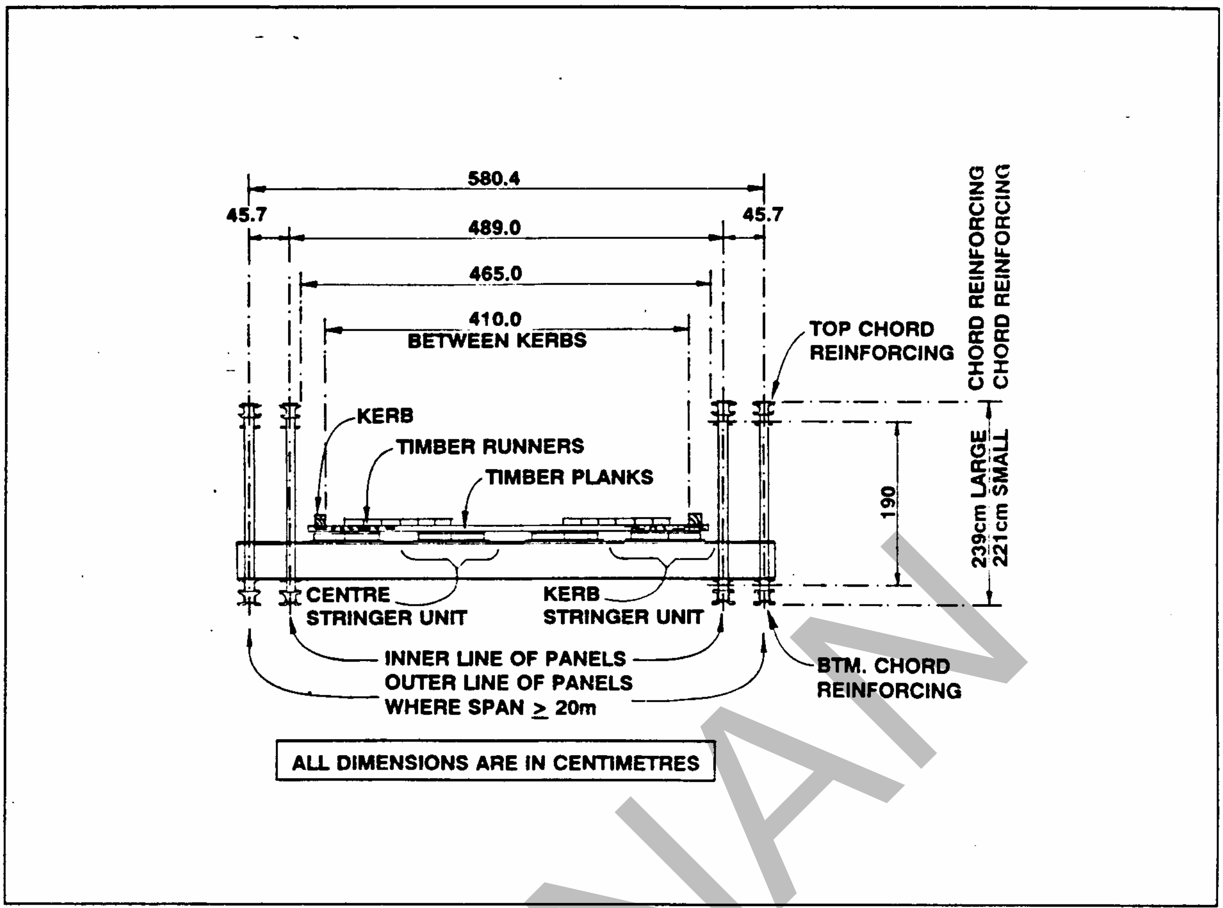


Figure 6.35 - Transpanel Bridging - Cross Section

All steelwork components including bearings and base plates are supplied together with hand tools and special erection components to be used in the assembly. Timber for the deck is supplied by the Contractor.

Components are clearly marked to permit assembly in the sequence as shown in the Drawings in the Erection Manual. Components of the same mark are interchangeable, and component weighs more than 440 kg.

The system is designed to be assembled rapidly with the aid of light lifting equipment on one bank and progressively launched into position by rolling out over the crossing. A cantilever launching nose, assembled from standard components is used for this purpose.

Other methods of assembly and erection, such as erection on falsework, are feasible.

The system is designed to be of low maintenance. All steelwork and bolts are galvanised and pins are of stainless steel. Basic maintenance procedures are described in the Erection Manual.

The Transpanel spans may be founded directly on the ground or on concrete structures.

Design Criteria

Loading:	Loading Specifications for Highway Bridges No. 12/1970 (Revised 1988) Direktorat Jenderal Bina Marga, Indonesia:
Traffic:	70% T-Loading (one vehicle only)
Footways:	Nil
Wind:	100 kg/m^2
Seismic:	Region 1 as Specification 12/1988 (C = 0.3)
Stream:	Superstructure assumed clear above flood level
Temperature:	± 15°C

Design Specifications

NAASRA Bridge Design Specification 1976

AASHTO Standard Specification for Highway Bridges 1983

b. Components

Each component of the Transpanel system is identified by a TP prefix and a number. Components which are only used for erection have the prefix TPE. The numbering system is not component-specific.

The main components are fabricated from steel plate and rolled sections of Grade 350 steel. The panel pins are made from high strength stainless steel to ASTM A564-630.

c. Erection Methods

The standard erection method for Transpanel bridges is by rolling out over the crossing with the aid of a launching nose. The length and configuration of the launching nose depends on the span being erected. Marking plans showing the configuration of launching noses are included in the Erection Manual.

Transpanel bridging has been designed to be compatible with standard Bailey Rocking and Plain Rollers which are used for rolling out. The rollers are laid out in a specified pattern on the *launching* and *receiving* banks of the river.

For spans 30 metres and less, the whole of the span and launching nose are assembled before rolling out. For longer spans, assembly is in two stages.

A sketch of the general arrangement of launching is shown in Figure 6.36.

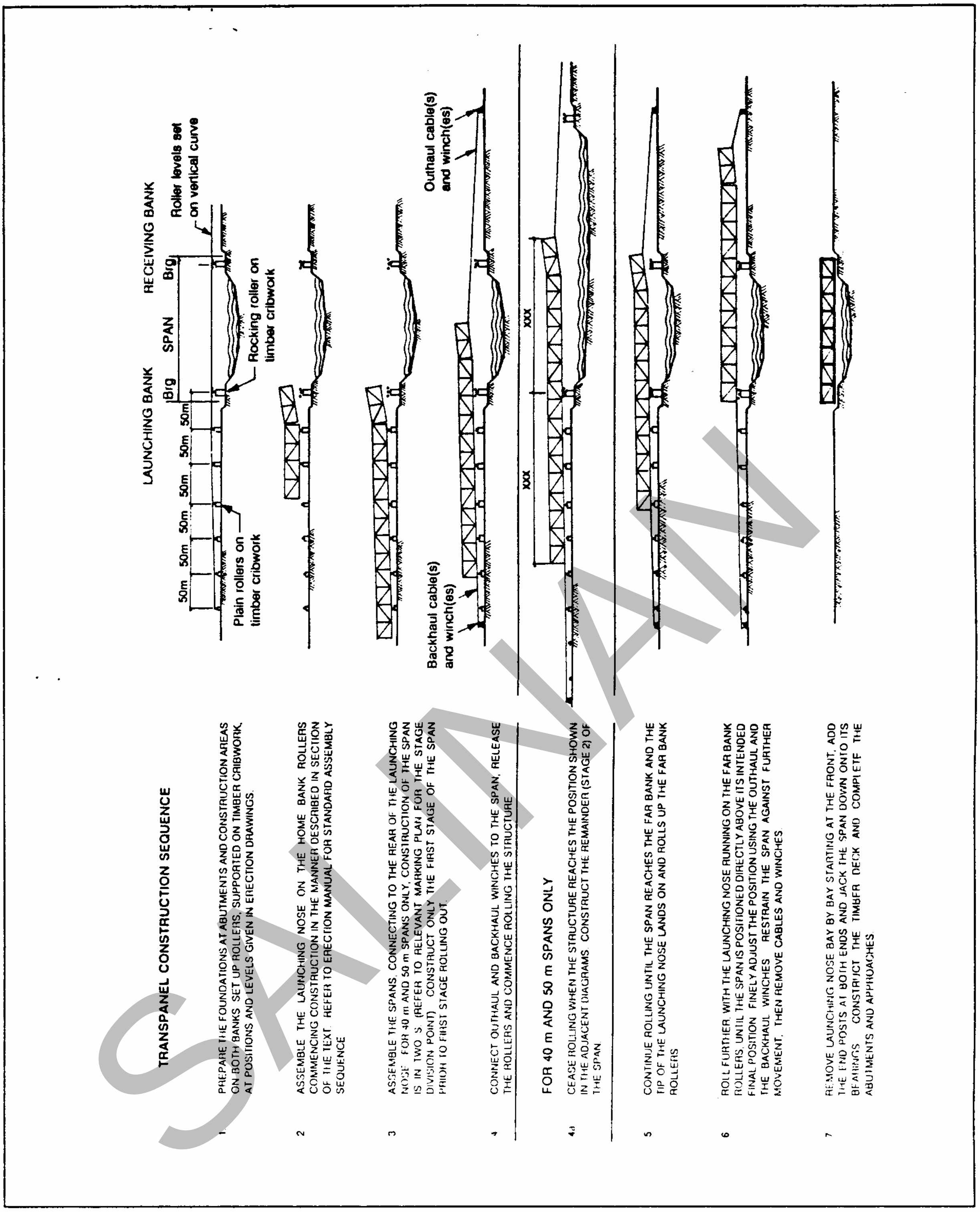


Figure 6.36 - Launching of Transpanel bridge - General Arrangement

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Hydraulic jacks 50 tonne capacity.
4. Tool kit (for assembly of all steel work).
5. All necessary roller components.

In addition to the above equipment, the Contractor needs to supply and install the following items:

a. Timber for decking.
b. Equipment for hauling bridge out across opening (winches for outhaul and backhaul, haulage ropes).
c. Jacking plates and timber packing.
d. Temporary timber bearings.

d. Options

The strength of the side trusses may be increased by adding reinforcement to the chords of the standard panel or by using two rows of panels on each side of the bridge. The chord reinforcement comes in two sizes. The possible truss configurations and the appropriate span length are shown in Table 6.5.

Table 6.5 Configurations for Transpanel Bridging

Span	Configuration		
	Construction	Reinforcing	Reinforcing Size
10	Single	Unreinforced	-
20	Single	Reinforced	Small
30	Double	Reinforced	Small
40	Double	Reinforced	Large
50	Double	Reinforced	Large

e. General Problems

The following points should be noted:

- Launching nose length varies with span length
- The longitudinal slope on a Transpanel bridge must not exceed 10 %. When a bridge is to be launched on a slope, it must be always launched uphill.

- Construction areas on the launching bank must be at least 9 metres wide and long enough to fit the span and the launching nose. Construction area on the receiving bank should be long enough for the launching nose plus space for the outhaul winch

- The camber of the Transpanel bridging is set on a circular arc of 1900 metres radius. The erection rollers must also be set up on a curve of this radius

- Proper set-up of rollers will prevent damage to components due to unequal load sharing and will also remove a common cause of the span running off the rollers during launching

- All bolts are to be snug-tightened. Snug-tightening is the tightening obtained by the full effort of a man using a standard podgier spanner

- Every roller is to be checked during rolling-out to ensure that the span does not move laterally. Adjustment to the alignment of the bridge must only be carried out in accordance with the procedures laid down in the Erection Manual

6.7.3 Mabey Johnson Compact Bailey

a. General

Mabey Johnson makes a number of basically similar panel bridging systems. The system in use in Indonesia is the Compact Bailey 200 series.

This form of bridging is suitable for temporary or semi-permanent applications as assembly is rapid and foundation requirements are minimal. It comprises standardised steel components which are assembled in a specified arrangement and sequence and connected by high strength pins and bolts to form through-truss bridge spans in the range 10 to 50 metres.

The basic load-carrying component is the truss panel, 2.13 metres high by 3.05 metres long. Panels are arranged in particular combinations vertically and horizontally, and/or reinforced as necessary to form side trusses of various capacities to suit the span. The deck comprises transoms and stringer units which support timber cross planks and running boards. Figure 6.37 shows a typical cross section of the panel system (without decking). An alternative steel deck is available but is not used in Indonesia.

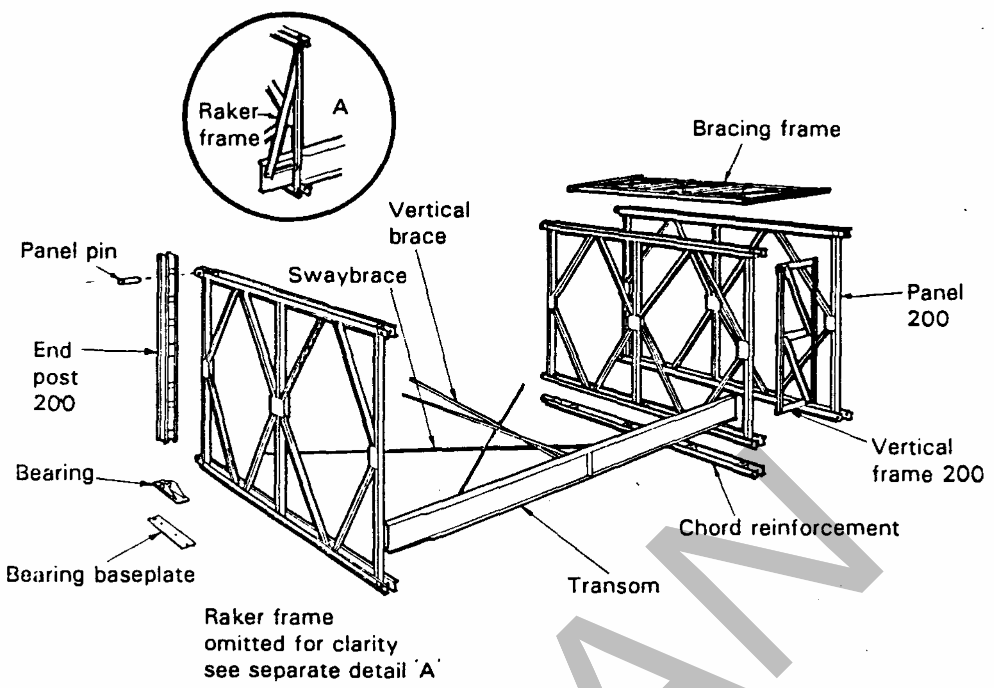


Figure 6.37 - Mabey Johnson Compact 200 Bridging - Typical Cross Section

All steelwork components including bearings and base plates are supplied together with hand tools and special erection components to be used in the assembly. Timber for the deck is supplied by the Contractor.

Components are clearly marked to permit assembly in the sequence shown in the Drawings in the Erection Manual. Components of the same mark are interchangeable, and no component weighs more than 450 kg.

The system is designed to be assembled rapidly with the aid of light lifting equipment on one bank and progressively launched into position by rolling-out over the crossing. A cantilever launching nose, assembled from standard components, is used for this purpose.

Other methods of assembly and erection, such as erection on falsework, are feasible.

Unlike earlier Bailey systems, only one transom per panel is required and the transom clamps have been replaced by transom bolts. There is only one diameter of bolt used and there are 4 different lengths of bolt.

Standard and heavy chord reinforcement is available to increase the load capacity of a standard span.

If timber decking is used, the stringers are secured to the transoms and the decking is located by holes in the outer stringers and held down with steel angles. This ensures a much quieter bridge than the standard Bailey configuration.

Broken-span bridges are suitable where settlement of piers may occur or where intermediate piers are at different elevations. They are constructed using special span junction posts at each intermediate support, which are designed to be locked during launching of the bridge and released when the bridge is jacked down into its final location.

Where piers are level and there is no possibility of settlement, continuous bridges may be built. In this case, bridge girders are supported at the piers on distributing beams.

Design Criteria

Loading: Loading Specifications for Highway Bridges No. 12/1970 (Revised 1988) Direktorat Jenderal Bina Marga, Indonesia:

b. Components

Mabey Compact Bailey components are generally prefixed with MC. The identification code is a number, for example MC1 is a standard 3 metre panel.

The system is designed to be of low maintenance. All steelwork and bolts are galvanised and pins are of stainless steel. Basic maintenance procedures are described in the Erection Manual.

Many standard Bailey components can be used with the Mabey Compact 200 series. However, some can not and the supervisor should check with the designer if there is any doubt as to the interchangeability of the components.

c. Erection Methods

The Bailey type of bridging is designed to be completely erected on rollers on one side of the gap to be bridged, and then launched without any temporary supports in the gap.

This is achieved by building a *launching nose* onto the front of the bridge which is constructed from the same type of parts.

The nose is built of such length that when the whole structure is rolled forward, the tip of the nose lands on rollers on the far bank before the centre of gravity passes the launching rollers. In general, the length of the launching nose is half the number of panels of the bridge plus one and is usually constructed in Single-Single configuration. Longer bridges may require heavier sections of launching nose where it attaches to the main part of the bridge.

After the bridge is in position across the gap, the launching nose is dismantled and the bridge is jacked up off the rollers and lowered onto its permanent bearings on the abutments.

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Loading: Loading Specifications for Highway Bridges No. 12/1970 (Revised 1988) Direktorat Jenderal Bina Marga, Indonesia:

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After the bridge is in position across the gap, the launching nose is dismantled and the bridge is jacked up off the rollers and lowered onto its permanent bearings on the abutments.

Alternatively, where adequate craneage is available, the bridge can be lifted as a complete unit or built up in-situ on falsework.

The general arrangement for launching is shown in Figure 6.38.

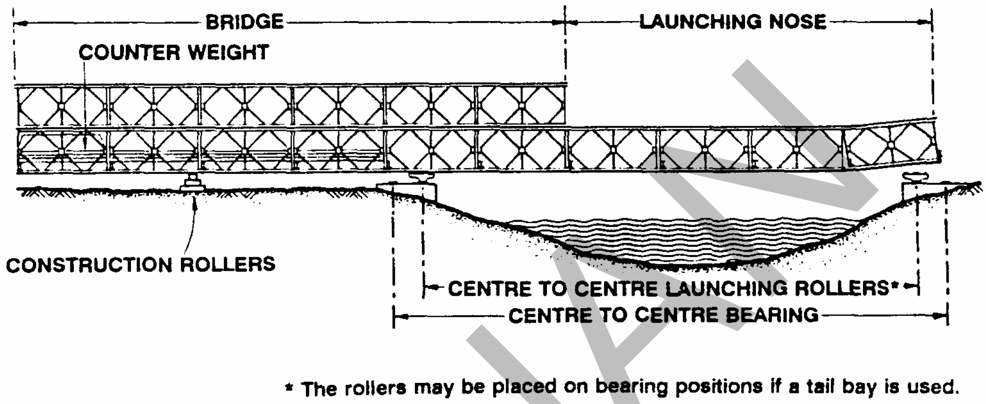

Figure 6.38 - Erection of Bailey Bridge

Erection Equipment Required

The following erection equipment is required with the main steelwork.

1. Erection Manual.
2. Construction Drawings.
3. Hydraulic jacks 50 tonne capacity.
4. Tool kit (for assembly of all steel work).
5. All necessary roller components.

In addition to the above equipment the Contractor needs to supply and install the following items:

a. Timber for decking
b. Equipment for hauling the bridge across the gap (winches for outhaul and backhaul, haulage ropes).
c. Jacking plates and timber packing.
d. Temporary timber bearings.

d. Options

The strength of the side trusses can be increased by adding reinforcement to the chords of the standard panel, or by using a two or three rows of panels and up to a maximum of 4 panels on each side of the bridge. The chord reinforcement is available in two sizes. Possible truss configurations are shown in Table 6.6 for spans up to 45 metres and vehicle loads up to 40 tonnes.

The decking may be steel or timber.

Multiple spans may be simply supported or continuous.

Table 6.6 Configurations for Transpanel Bridging

Span		Single Vehicle Loadings in tonnes		
Bays	**Metres**	**20**	**30**	**40**
3	9.1	SS	SS	SS
4	12.2	SS	SS	SS
5	15.2	SS	SS	SSR
6	18.3	SS	SSR	DS
7	21.3	SSR	SSR	DS
8	24.4	SSR	SSR	DSR1
9	27.4	SSR	DSR1	DSR1
10	30.5	SSR	DSR1	DSR1
11	33.5	SSRH	DSR1	DSR2
12	36.6	DSR1	DSR1H	DSR2
13	39.6	DSR1	DSR2	DSR2H
14	42.7	DSR2	DSR2	TSR2H
15	45.7	DD	DDR1	DDR1

e. General Problems

The following points should be :

- Launching nose length varies with span length and the position of the launching link, which provides the additional slope to the nose, may be located up to 4 bays from the tip of the nose. These details are given in the Erection Manual.

- The longitudinal slope should not exceed 10 %. When a bridge is to be launched on a slope it should be always launched uphill. The bearing plates must be installed horizontal, even if the bridge is on a slope.

- Construction areas on the launching bank must be at least 9 metres wide and long enough to fit the span and the launching nose. Construction area on the receiving bank should be long enough for the launching nose plus space for the outhaul winch.

- Proper set-up of rollers will prevent damage to components due to unequal load sharing and will also remove a common cause of the span running off the rollers during launching.

- All bolts are to be snug-tightened. Snug-tightening is the tightening obtained by the full effort of a man using a standard podgier spanner.

- Not all the decking is to be installed prior to launching. Check how much is needed.

- Check the amount of ballast needed for a counterweight.

- Every roller is to be checked during rolling out to ensure that the span does not move laterally. Adjustment to the alignment of the bridge must only be carried out in accordance with the procedures laid down in the Erection Manual.

- Never have both ends of the bridge supported on jacks at the same time.

- Use jack *catch packs*, which are packs adjacent to a jack so that if the jack should fail or sink, load is immediately taken by the catch pack and the structure cannot fall. The catch pack needs to be continuously adjusted for height during the jacking process.

- Avoid transverse slopes during jacking by working at both trusses simultaneously.

7. BEARINGS AND JOINTS

7.1 GENERAL

All the truss type bridges and girder type bridges in Indonesia use steel-reinforced elastomeric bearings, made of natural rubber or similar material.

The Australian Transpanel bridges and the other variants use a proprietary steel rocker bearing.

Smaller concrete girders are often designed with plain elastomeric strips or pads.

A variety of expansion joints is in use on bridges. Many bridges have open joints, with or without a cover plate. Others have a compressible rubber strip inserted in the joint. The H series Australian trusses (80 and 100 metre spans) are designed to have a central joint in the concrete deck which is filled with a polystyrene packer and sealant.

7.2 BEARINGS

The following points about installation of bearings should be noted:

- Elastomeric bearings are designed to have no horizontal displacement or shearing under dead load. A bearing which is distorted by shearing when the dead load is jacked down should be unloaded and re-centered. If shearing still occurs, the bearing may be faulty and should be replaced

- Elastomeric bearings which bulge or split under dead load should be replaced

- Pot-type bearings (used as erection bearings) should be left assembled until just prior to use because damage to the P.T.F.E. (Teflon) or the polished stainless steel surface may result

- Care should be taken with elastomeric bearings to avoid damage caused by dropping of sharp objects onto the bearing

- The mortar surface on the top of the abutment or pier must be horizontal

- Ensure that there is clearance between the rubber seismic buffers and the concrete surface

- Proper bearings are required for Transpanel bridges if the bridge is to function correctly. The bridge should not be left supported on timber packers

7.3 JOINTS

The main problem with deck joints is that the concrete beneath the steel protection angle is not placed correctly. If care is not taken, air can be entrapped under the angle while concrete is poured in the deck adjacent to the protection angle. To avoid this, it is recommended that the concrete under the angle be placed **first** and then screeded away from the protection angle, rather than screeded towards it (see Figure 7.1).

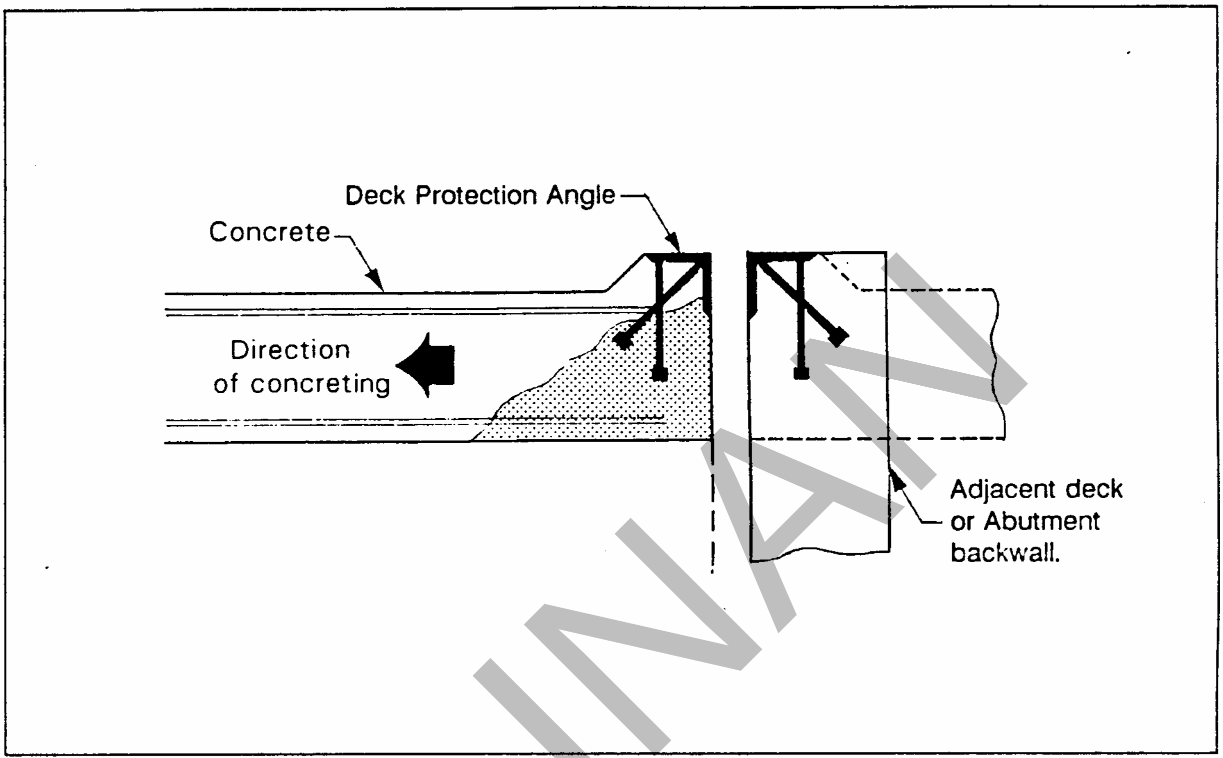

Figure 7.1 - Compaction of Concrete under Protection Angle

Omitting the concrete in the vicinity of the deck protection angle at the time of pouring the main deck should be prohibited.

Where a pre-formed expansion joint is to be inserted in a joint, it is essential that an appropriate installation tool is used to avoid causing damage to the rubber seal.

LIST OF REFERENCES

1. *Bridge Construction Practice*
Austroads, Draft 1989

2. *Construction Planning, Equipment and Methods*
Peurifoy and Ledbetter, 1987

3. *Civil Engineering Construction*
Antill and Ryan, 1972

4. *Training Manual for Bridge Foremen (Volumes 1 - 4)*
Department of Main Roads NSW, Australia, 1973

5. *Construction Manual for Highway Bridges and Incidental Structures*
AASHTO, 1973

6. *Australian Concrete Inspection Manual*
Nagarajan and Antill, 1978

7. *Basic Guide to Concrete Construction*
Cement and Concrete Association of Australia, 1975

8. *Prestressed Concrete Inspection Practice*
NAASRA, 1983

9. *Manual of Concrete Inspection*
American Concrete Institute, SP-2, 1988

10. *Recommended Practice for Concrete Formwork*
ACI 347, 1978

11. *Guide for Measuring, Mixing, Transporting and Placing Concrete*
ACI 304R, 1985

12. *Standard Practice for selection proportions for Normal, Heavyweight and Mass Concrete*
ACI 211.1, 1981

13. *Formwork for Concrete*
American Concrete Institute, SP-4, 1988

14. *Properties of Concrete*
Neville, 1981

15. *Foundation Analysis and Design*
Bowles, 1989

16. *Pile Design, Construction and Practice*
Tomlinson, 1989

17. *Pile Foundations*
Chellis, 1961

18. *Wika Piles*
PT Wijaya Karya, Brochure

19. *Construction and Geotechnical Methods in Foundation Engineering*
Koerner, 1988

20. *Site Instructions for the Assembly and Erection of Gabions*
Maccaferri Technical Brochure

21. *Project Management Techniques*
SMEC, 1985

22. *Australian Permanent Standard Truss Spans - Manual for Assembly and Erection of Steel Spans*
Transfield, 1990

23. *Australian Permanent Special Truss Spans - Manual for Assembly and Erection of Steel Spans*
Transfield, 1991

24. *Australian Girder Spans - Manual for Assembly and Erection of Steel Spans*
Transfield, 1989

25. *Australian Semi Permanent Truss Spans - Manual for Assembly and Erection of Steel Spans*
Transfield, 1990

26. *Australian Transpanel Demountable Bridging - Manual for Assembly and Erection of Steel Spans*
Transfield, 1991

27. *Austrian Permanent Truss Type Bridges Manual for Assembly and Erection of Steel Spans*
Waagner Biro, 1991

28. *Austrian Semi Permanent Truss Type Bridges Manual for Assembly and Erection of Steel Spans*
Waagner Biro, 1987

29. *Hollandia Kloos Permanent Truss Spans Manual for Assembly and Erection of Prefabricated Steel Bridges*
Hollandia Kloos, 1989

30. *Mabey Compact Bailey Manual*
Mabey Johnson, 1987

8. WATERWAY AND EMBANKMENT PROTECTION

8.1 GENERAL

This Section describes some aspects of embankment and scour protection.

It is essential that a structure intended to provide protection from scour should be founded below the scour level for the design flow in the river. This depth should be shown on the Drawings but if not, a conservative figure of 800 - 1000 mm below the stream bed should be adopted. The possibility of scour around the end of the structure should also be considered and some protection (rip rap, filter fabric) may need to be installed.

8.2 GABIONS

A gabion system is designed to act as a homogeneous and monolithic structure which can be designed to resist all the forces involved, not as a system of separate wire cages placed side by side.

Accordingly, it is important that the gabion structure is constructed exactly as designed and the following points should be noted:

- Ensure that creases are in correct position when the gabion is folded out, one at the edge of each end panel and each diaphragm
- When folding the box, ensure that the tops of all four sides of the box are level before wiring the top corners
- Use double loops at 100 mm intervals for the binding wire
- Ensure the ground beneath gabions is as level as possible before commencing placement of stones
- Place gabions front to front and back to back along a row so that pairs of facing lids can be wired down in one operation
- Anchor end of first gabion using rods driven into the ground through the two corners
- The height of the anchorages must be at least the height of the gabion
- Ensure the opposite end is kept stretched until the box has been filled. This can be done using bars and a stake attached to the gabions underneath.
- Check that the anchoring has not pulled apart any of the wiring of the box
- Use filling material not larger than 250 mm and not smaller than the holes in the mesh

If insufficient material of the correct size is available, use smaller rocks inside the gabion and at least 250 mm of larger rock on each outside face

- Ensure stone is tightly-packed and voids are minimised
- One metre high gabions need wire cross bracing at the one-third and two-thirds points of the box height
- Fill the gabion to about 25 mm or 50 mm higher than top of box to allow for settlement
- Avoid overstretching lids when wiring lids down

Reference should also be made to the relevant manufacturer's handbook for aspects of construction particular to that type of gabion.

8.3 STONE PITCHING

The following points should be noted:

- stone used for pitching must comply with the specification for
  - minimum weight
  - minimum dimensions
  - shape
  - soundness
- the grading of rock should be such as to minimise the amount of voids
- where the bank material is likely to wash out behind the pitching, a suitable bedding layer should be used, either a graded gravel or a filter fabric
- the toe of the pitching must extend below the anticipated scour level
- provision for protection against end scouring should be made
- where mortared stone pitching is required, suitable weepholes should be installed to provide drainage of the embankment.

8.4 SHEET PILING

Sheet pile walls are often used to provide protection against scour of the embankment of a bridge. The walls are designed to be self-supporting and it is essential that the sheet piles are driven with the interlocks coupled. If this is not done, the wall will not act as an integral unit and will probably fail (by slumping forward at the top), requiring the installation of tiebacks or walers.

The interlocks should be well greased before installation to ensure that they can slide freely as driving progresses.

9. ROAD APPROACHES

9.1 GENERAL

Almost all bridge contracts include some road approach construction. This is generally carried out late in the contract and is an area which is often overlooked in the overall quality control process.

Material often used for filling behind the abutment is either highly plastic clay or very coarse stone, both of which are difficult to compact.

Many bridges have abutments designed with an approach slab as shown in Figure 9.1.

The approach slab is designed to reduce the effects of settlement of the road embankment immediately behind the abutment. The approach slabs are usually constructed about 0.5 metres below the finished surface level.

The material which is to be placed over the concrete approach slab is to be pavement material, usually 250 mm of sub base, 150 mm of base and some form of surface treatment, often about 50 mm thick.

The quality and compaction of the pavement material are critical to the traffic ride on the immediate approach to the bridge. Poor quality pavement construction is common in many bridges. It usually leads to increased impact loading on the structure due to the settlement of the pavement and may cause other problems as a result of water entering the pavement if the pavement cracks.

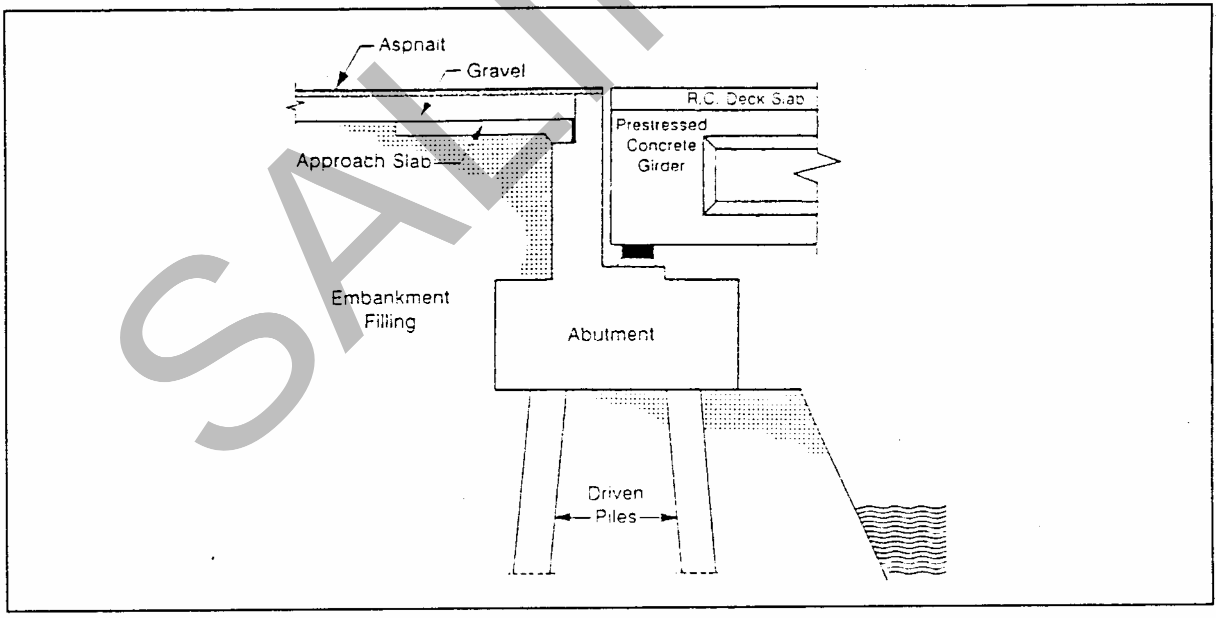

Figure 9.1 Approach Slab behind Abutment

9.2 MATERIALS

The Specifications set out the requirements for the materials to be used in the embankment and the pavement.

Quality control checks must be made on all pavement materials.

9.3 COMPACTION

The pavement materials must be compacted with suitable compaction equipment. This can range from large vibratory or static rollers to small vibrating plates. It is most important that the material is properly compacted if settlement of the pavement layer due to compaction under traffic and/or ingress of water is to be avoided. Proper compaction cannot be achieved if the material is too wet or too dry. Sufficient water must be added and mixed to enable the material to be compacted to the level specified.

9.4 BITUMINOUS SURFACING

Bituminous surfacing on bridge decks is usually a layer of asphalt.

The following points should be noted:

- application of the tack coat should not be carried out too far in front of the area where asphalt is to be applied as it will be worn away by traffic
- a longitudinal edge board (of a height equal to the specified thickness of asphalt) should be placed along the centreline of the road when the first side is being placed
- Wooden 'floats' on handles should be used for spreading the asphalt to an even surface
- the thickness of the uncompacted asphalt should be about 10 percent greater than the compacted thickness
- rolling and compaction should commence as soon as possible after initial laying and screeding
- all spreading equipment should be kept thoroughly clean