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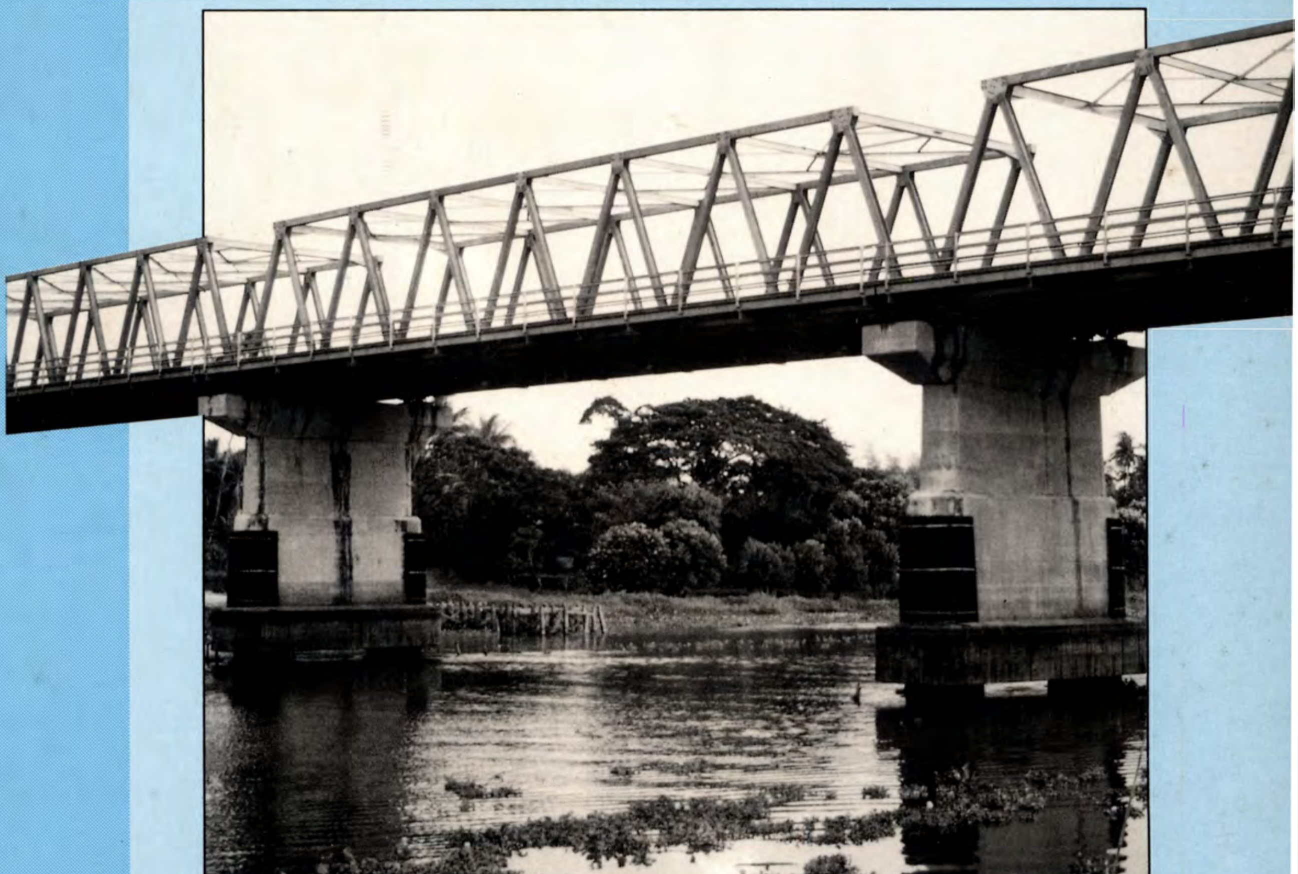
Yangon-Pathein Road Project, Myanmar

Construction and Design Manual

Bored Piles with Bentonite
Medium Span Steel Truss Bridges

M. Diggelmann, W. Osterwalder



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Medium Span Steel Truss Bridges

Consulting Association ITECO / TIF CONTRACTOR

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TIF CONTRACTOR

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1 Preface

In 1983 the Government of the Union of Myanmar submitted to Swiss Development Cooperation (SDC) a proposal for assistance in the construction of a road from Yangon, the capital, west across the delta of the Ayeyarwady river, to the important town of Pathein.

The road will serve an area with a population of approaching two million people and very poor existing communications. The project will assist the socio-economic development of this area, in particular agriculture and related processing industry. An estimated 4-5 million tonnes of rice, ground-nuts, maize and jute are produced annually and the road will make possible improved access for fertilizers, farm equipment and processing machinery and, in addition, enable surplus produce and cash crops to provide additional income. Improved access may also be expected to enhance education, health and other social services in the area.

By late 1984, agreement had been reached for SDC to assist, initially, with the introduction of new design and construction techniques for essential major river crossings along this road, including eight bridges with a total length of some 690 m. The first bridge was completed in June 1987, a further three during the period to July 1988; two were completed in 1989. The programme of medium span bridges was terminated in 1991 with the erection of the two last bridges. Works on the major crossing of the Bayintnaung bridge over the Hlaing river using similar techniques is progressing.

Whilst the primary aim of the project, to contribute towards the wider socio - economic development of the area, will take some time to become apparent, the more immediate aims have largely been accomplished. These include the passing on to Myanmar engineers of techniques and experience in deep-bored, bentonite piling and the erection of multiple-span, steel truss bridges. The project has included training in Switzerland and in-situ erection assistance/training over three years in Myanmar.

A final part of this work has been the preparation of this manual to record the design and construction principles and experience for further use. Although the piling and steel design theory is more widely applicable, the manual is specific both to bridge type and to the new techniques, structures or equipment introduced. For areas such as pier design and construction it introduces only factors specific to the bridge types constructed. The manual does not deal with the economics of this bridge type nor does it set out to make comparison of the benefits or otherwise of the use of either deep-bored, large diameter piles or the truss superstructures with the numerous alternatives either in use in Myanmar or elsewhere.

The project was implemented on behalf of Swiss Development Cooperation by the consulting association ITECO/TIF Contractor of Switzerland. Following tender, the steel parts supply and detail design work was awarded to Transfield PTY Ltd. of Australia and their consulting engineers McMillan Britton and Kell PTY Ltd. of Sidney. ITECO/TIF acknowledge their gratitude to these companies for their cooperation throughout and for assistance in drafting sections of this manual.

Foto 1: Completed Bridge

Foto 2: Drilling Rig in River

2 Units and Definitions

2.1 Abbreviations and Acronyms

AASHTO	American Association of State Highway and Traffic Officials
AS	Australian Standard
ASTM	American Society for Testing Material
SDC	Swiss Development Cooperation

2.2 Units

The "Systeme International d'Unites" (SI) is the generally accepted unit system. It consists of a few base units based on the metric system, such that all other physical quantities can be described or expressed algebraically in terms of base units or simply derived units.

Basic Civil Engineering Units:

length	metre	m
mass	kilogram	kg
time	second	s
angle	radian	rad

Frequently used 'derived' units:

force	Newton	N	$1\ N = kg\ m/s^2$
energy	Joule	J	$1\ J = 1\ Nm$
pressure, stress	Pascal	Pa	$1\ Pa = 1\ N/m^2$
temperature	degree Celsius	°C	

Multiples of SI-units:

Factor	Prefix	Symbol
10^6	mega	M
10^3	kilo	k
10^{-3}	milli	m

2.3 Conversion Table : Imperial System to SI Units

length	inch	1 in	=	0.0254	m		
	foot	1 ft	=	0.3048	m		
	yard	1 yd	=	3	ft	=	0.9144 m
	mile	1 mile	=	1.609	km		
area	square inch	1 in^2	=	6.452×10^{-4}	m^2		
	square foot	1 ft^2	=	0.0929	m^2		
	acre	1 acre	=	4,046.8	m^2		
	square mile	1 $mile^2$	=	2.590	km^2		
volume	cubic inch	1 in^3	=	1.639×10^{-5}	m^3		
	cubic foot	1 ft^3	=	0.028316	m^3		
	cubic yard	1 yd^3	=	0.766	m^3		
	gallon	1 gal	=	4.546	l		
	litre	1 l	=	1.0×10^{-3}	m^3		
velocity	foot per second	1 ft/s	=	0.3048	m/s		
	mile per hour	1 mph.	=	0.447	m/s		
	kilometre per hour	1 km/h	=	0.278	m/s		
mass	pound	1 lb	=	0.4536	kg		
	ton (long)	1 ton	=	2240	lb	=	1016.0 kg
	ton (short)	1 ton	=	2000	lb	=	907.2 kg
	metric ton	1 tonne	=	1000	kg		
density	pound per cubic foot	1 lb/ft^3	=	16.02	kg/m^3		
force	pound force	1 lbf	=	4.448	N		
	ton force	1 tonf	=	2240	lbf	=	9.96 kN
stress	pound force per square inch	1 lbf/in^2	=	6894.7	N/m^2		
	kilogram force per square centimetre	1 kg/cm^2	=	1.0×10^5	N/m^2		
moment/torque							
	pound force inch	1 lbf in	=	0.113	Nm		
	pound force foot	1 1bf ft	=	1.356	Nm		
	ton force foot	1 tonf ft	=	3.04	kNm		
electricity	kilo volt ampere	kVA					
temperature	Fahrenheit	deg F	=	deg Celsius x 2.25 + 32			

3 Survey and Site Investigation

3.1 General

The information required for bridge design requires both general and also specialised site investigations. This chapter provides general guidelines for optimum bridge site selection, includes investigations specific to this bridge type and provides an outline of the information more generally required. The initial definition of the bridge site within the larger context of the road feasibility and alignment studies, taking account of many wider technical and economic factors is assumed to be beyond the scope of this manual.

3.2 Site Selection

The project has introduced deep bored piles designed for conditions found in many areas of the Ayeyarwady delta. Few special constraints upon site selection exist there other than the poor soils which primarily consist of soft clay, silt or sand. The foundations of any large bridge in such an area will clearly form a disproportionate part of the overall bridge cost and site selection should ensure that this high cost is minimised. One danger introduced with a technique which permits rapid piling down to 50 m is that the economies of selecting an alternative site with possible shorter piles may be disregarded.

In addition to the choice of the least difficult soils, site selection should, as a part of the final consideration seek also a location where the construction equipment is able to operate without unnecessary constraints. In particular, survey information should be sufficient to permit selection of a location where, should the unifloat/drilling rig be required, the water is deep enough, throughout the dry construction season to permit to it to float freely whilst avoiding water so deep as to require special temporary guide casings to be fabricated.

Over a longer term, information should be sufficient to forsee possible movements of the river, either as a continuation of present processes or as a conseqence of channel constraints by the bridge itself. The modest additional cost of slightly longer approach roads to a safer site may, in the long term, prove more economic than continual works to retrain the river. At a number of sites throughout Myanmar, the lack of attention to this problem has ensured that underscouring of abutments, wing walls and protective aprons has become such a problem. Meandering rivers in the delta are always likely to change their present course and a study of all data available (maps, local observation, information from old residents) to foresee the development of the river is necessary.

It is of primary importance during site selection to leave permanent records of all possible alignments considered (see also setting out). This requires the establishment of sufficient known positions to permit replacement of lost, removed or vandalized stations without the unnecessary problem of trying to re-establish from a distance and often with different personnel, once the decision to proceed has been made.

3.3 Outline Investigation

Site and background survey should seek to provide full information on the following :

- access to the site, limitations upon width and weight consequent on ferries or existing bridges
- immediate access situation, length and type of temporary access road required
- availability of land for construction camp
- simple topographic survey of possible camp areas with specific attention to any special problems, e.g. areas liable to flood
- existing land-use and vegetation especially trees which may be useful shade or a problem to remove
- situation regarding services, availability of means of communication, electricity and water

And, in order to satisfactorily design and construct any reasonably sized bridge, the basic survey information will include special surveys to define conditions regarding:

- site topography, see 3.4
- hydrology, see 3.7
- soil conditions, see 3.5 and 5.1

and for deep bored pile foundations special attention to soil tests is required.

3.4 Topographic Survey

Topographic survey for bridge sites with deep bored piles requires contour or spot height plans, notes of any special features such as poor ground conditions and referencing the survey back to the national or known local reference levels.

In order to pre-assemble the temporary steel guide casing and ensure the unifloat/drill rig will float, good cross sectional profiles of the river bed are required. In addition, because the surface layers of the river bed may be too insubstantial to properly install the casing, the river profile should also indicate firm soil as well as the initial mud encountered on the river bed. While normal Construction Corporation standards should apply to topographic surveys, for the above river bed levels, an accuracy of +/- 0.2 m is adequate.

3.5 Soil Investigation

The soils found at most river crossings within the delta area require very deep foundations due to their large components of silt and clay. Thus sub-surface investigations are required to a depth at least as deep as any likely foundations. For many sites, this requires examining the soils to more than the maximum operating depth of the piling equipment available, that is approximately 50 metres (165 ft.) unless the design engineer or geologist has advised otherwise (see also section 5.2). It should become normal to investigate by first drilling one borehole, to 50 m if possible and to determine the soil properties for the full possible depth. The results should permit the design engineer to make a preliminary pile design and specify the soil sample testing requirement. If there is any doubt regarding the investigation it is best for the geologist or the design engineer to be on site throughout actual pile boring in order to judge the excavated material.

In normal circumstances, the information required will include :

- the depth and the thickness of each soil layer encountered
- classification of the soils of each layer using the legend in fig. 3.1
- laboratory tests of layers considered significant
- the level of ground water, if possible relating to the level of the river
- Standard Penetration Tests upon samples of all layers, this will normally arise as tests are carried out at small regular increases in depth, see example in table 5.1.

The layers at 30m down to 50m are likely to be particularly interesting, as they determine the pile head resistance. In addition to the rough visual classification and standard penetration testing, samples should be collected for laboratory testing.

Standard penetration with (SPT) split spoon sampling may be the routine and main test of the soils. A standard 50.8 mm diameter split spoon sampler is driven 450 mm into the base of the borehole at a number of depths as required. The spoon is driven by a standard weight falling a standard height onto the top of the column of normal drilling rods. The first 150mm is discounted as this part of the soil is likely to be disturbed by the drilling process and N is the number of blows to penetrate the underlying 300 mm . The spoon splits opens after lifting to reveal a soil sample of the section penetrated and these samples are used for classification and also for laboratory tests as required. Interpretation of SPT tests is empirical, the test having been widely used in many countries. Correlations of SPT numbers to standard soil properties are given in section 5.2. SPT results, particularly of tests at great depths, can be too high as a consequense of friction of the steel rod in the drill hole and are therefore somewhat doubtful. In addition, the samples taken with the split spoon are rather small and are always disturbed. The consequent laboratory test results (e.g. of triaxial shear tests) are, at best regarded as indicative, since properly undisturbed samples should be used to achieve representative test results. Better samples may be taken during pile or test pile boring with the bottom sampling attachment used in conjuction with the Kelly bar as these are relatively undisturbed.

Legend of Symbols used to indicate materials in profile of borehole:

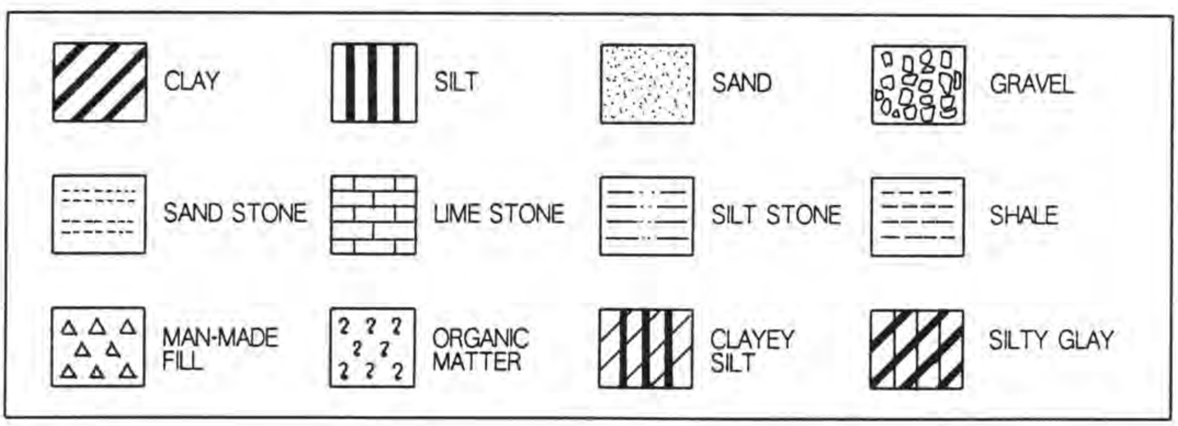


Fig 3.1

Reliable soil tests permit reduction in the length of piles and help avoid overdesign. For example, at Kyounggone Bridge a satisfactory bearing layer of 2.5 m minimum thickness was found at 38 m depth. The soil tests indicated a Standard Penetration Test value greater than 30 and sand content more than 50%. The angle of internal friction, phi, was greater than 25 degrees. Without these test results, it had been assumed that a further 10 m of pile was required. It will clearly require the use of discretion by the test engineer but because of the savings possible, each pier and abutment location should normally be proved by proper soil testing.

Terminology used to denote the percentage of each component by weight:

Descriptive Term	Range of Proportion
Trace	1 - 9 %
Some	10 - 19 %
Adjective (e.g. sandy, silty)	20 - 34 %
Major soil	greater than 35 %

Table 3.1

The design engineer or geologist must be able to determine from the initial drill results which samples should be submitted to further laboratory testing. Table 5.1 is an example of data collected during the soil investigation at Pathwe Bridge with a standard penetration hammer and split spoon sampler plus the results of the later laboratory tests upon samples..

These tests will include grain size distribution, to confirm the visual classification, unconfined compression tests (UCS) on cohesive soil samples (clay layers) and triaxial shear tests at lower, potentially bearing layers.

Dry density and water content tests have been made upon apply to all samples but, as these are almost certainly upon samples which are disturbed, their true value is open to much doubt. If possible, these should be confirmed by sampling during pile boring with the bottom sampler attached to the Kelly bar. The communication of soil test results should follow the existing legend, see table 5.1and in section 5.2.

It is important to correlate the field observations with the laboratory tests and a number of "rules of thumb" checks will apply. Section 5.2 and the table below indicate how SPT test results may be expected to relate to sample classification and to expected test results.

N value	Descriptive Term	Unconfined Compressive Strength (kN/m2)
> 2	very soft	below 25
2 - 4	soft	25 - 50
4 - 8	firm or medium stiff	50 - 100
8 - 15	stiff	100 - 200
15 - 30	very stiff	200 - 400
> 30	hard	above 400

Table 3.2

As stated above, the results of the soil investigations are of economic importance since they define the number and length of piles required. While the procedure of standard penetration tests and split spoon sampling is adaquate for preliminary investigations, uncertainties and diffficult interpretation of results remain a drawback for precise pile design since generally conservative assumptions will have to be made, see section 5.

3.6 Test Piles

Whilst in normal situations, tests on soils will be limited to SPT investigations for substantial foundations, with either a large number of or with deep piles, test piles are recommended to reveal effective load bearing capacities. The costs of such tests are generally paid back since the safety factors may be decreased due to extended knowledge of actual pile behaviour and design assumptions, both optimistic or pessimistic, corrected , resulting in more economic design.

Pile tests should be executed at an easily accessible location with average soil properties for the site. Soil investigations at pier and abutment locations will allow correlation of the test results.

It is obvious that one or more test piles will only be of value if they demonstrate characteristics similar to the proposed foundations. Accordingly, the locations and the depths must be selected with this in mind. A preliminary design must initially be made in accordance with section 5. For testing, a pile of smaller diameter may be bored as this is more economical and will permit testing with a more manageable test load.

As an example the test piles which were made at Bayintnaung bridge site, are described below. Two 0.8 m diameter piles one each side of the proposed centre line were bored after first having determined that the soils on each side of the river are similar to the likely pile depths. The drilling used the same pile drilling equipment as for the earlier 2.5 m piles.

A limited amount of special equipment was required:

- Strain gauges to be fixed at various levels down the length of the pile. Three low voltage displacement transducers at five levels have been used at Bayintnaung.
- Parallel plate micrometer attachment for a standard level together with invar scales to measure pile displacement
- hydraulic jacks and compressor with load measurement facility
- the use of concrete additives (Sikament FF and Sikagrout) may make it easier or quicker to fix a steel bearing plate accurately onto the pile head but is not essential
- steel or RC beams plus load distribution plates of appropriate dimensions to concentrate the loading of approx 5'000 kN of the test load onto the hydaulic jacks.

The procedure was the following:

1. Drilling and concreting of test pile. Before lowering the reinforcement cage, strain gauges were fitted at 5 reference levels. The pile was ready for testing after reaching a concrete strength of 25 N/mm2 which was checked by test cubes.

2. Preparation of pile head. The steel loading plates were added and the test load assembled.

3. Applying of test load. The test load was used as a reaction load against the jacks, loading steps were made at approx. 1000 and 2000 kN and at working load in accordance to the Swiss Norm 192 to relate settlement with additional loading.

4. Recording of pile load and settlement/strain at time intervals determined as sufficient to adequately record the relationships.

5. Removal of the load. The recovery was recorded in a similar manner.

5. Repetition of the test. The loading was continued until soil failure was reached.

6. Analysis of the results as base for detailed pile design.

An example of such load tests is illustrated in the following sketches, data and photos taken at the site of the first pile load test at Bayintnaung bridge site.

3.7 Hydrology

The important hydrological factors influencing bridge design are the highest flood levels , the maximum flow velocity, the required clearance for navigation and the history and control of channel movement. Although extensive records, dating back many years, have provided a basis for irrigation in the area, these can provide a sound design base for runoff but require reassessment before direct use to predict expected flood levels for the road embankment and for bridges.

From the river a minimum clearance of 3 m above high flood level will be sufficient to permit floating objects to clear the span. However, for navigation purposes it may be necessary to consider the costs and benefits of raising this to permit passage of large boats during normal and during peak flood periods.

Generally, in the delta area, river velocity will have little effect upon design. Unless the river is providing the only outlet to an extensive flood area, speeds are not normally sufficient for scour to be a serious feature required of a design. However, bridges are commonly constructed in conjunction with long road embankments and coordination with road design and associated irrigation works is required to anticipate consequent changed flows. This is not a problem unique to this bridge type.

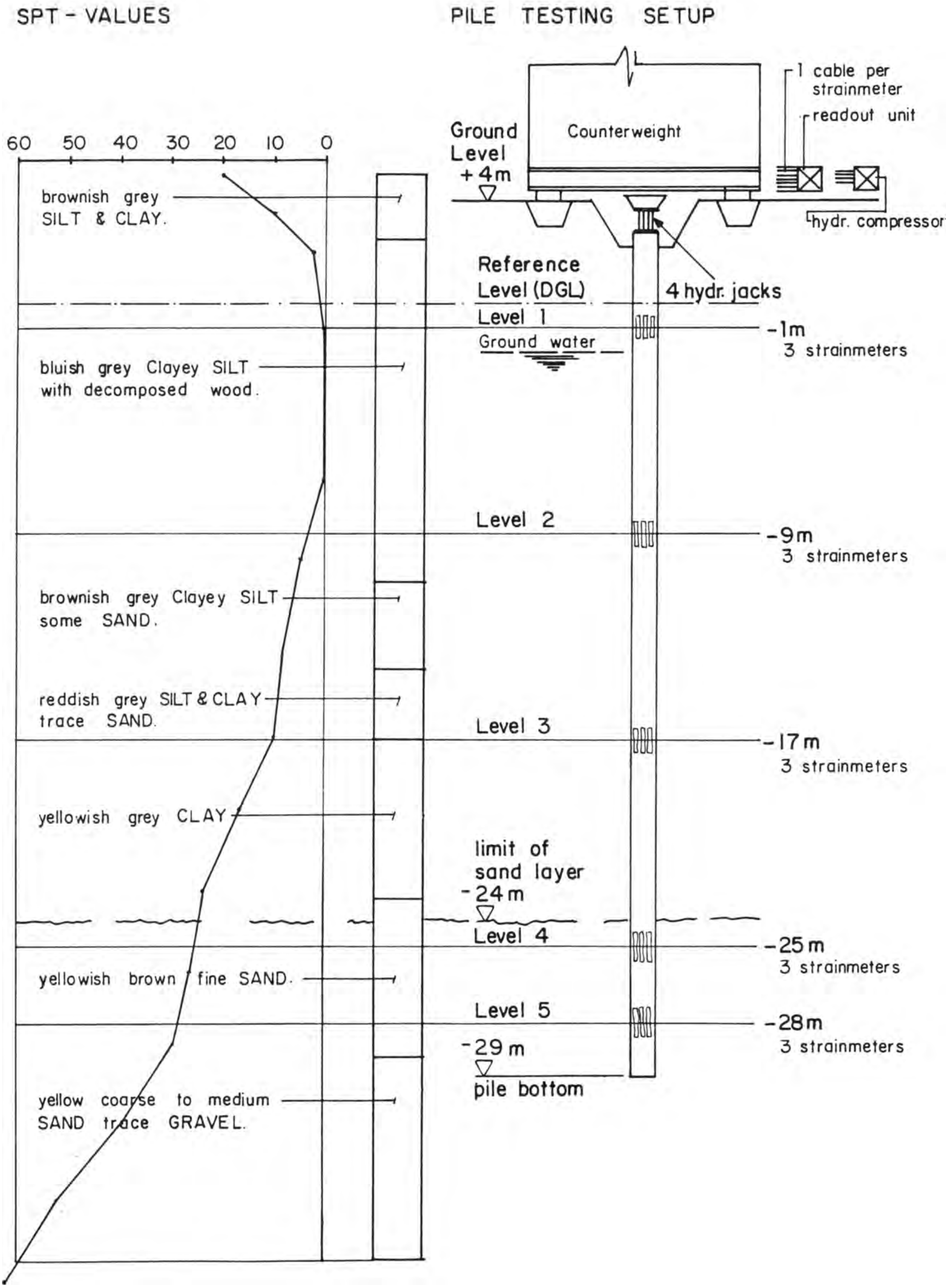
In the delta area, channel migration is likely to be more of a factor than velocity. In the long term all sites are metastable, but in the design life period of a bridge this is not the case. However, the design of any significant bridge should evaluate the possibility of both upstream channel diversion and also local movements of the channel, particularly as influenced by the constriction and the construction of the bridge. In the first case the bridge could become redundant, in the latter, expensive to maintain.

Foto 3: Pile Load Test

Assembled test load of approx. 5000 kN by using reinforcement steel. The test load is transmitted to the pile head by the centre concrete girders. The outer ones are only used for stabilizing the load.

Foto 4: Jacks on Test Pile

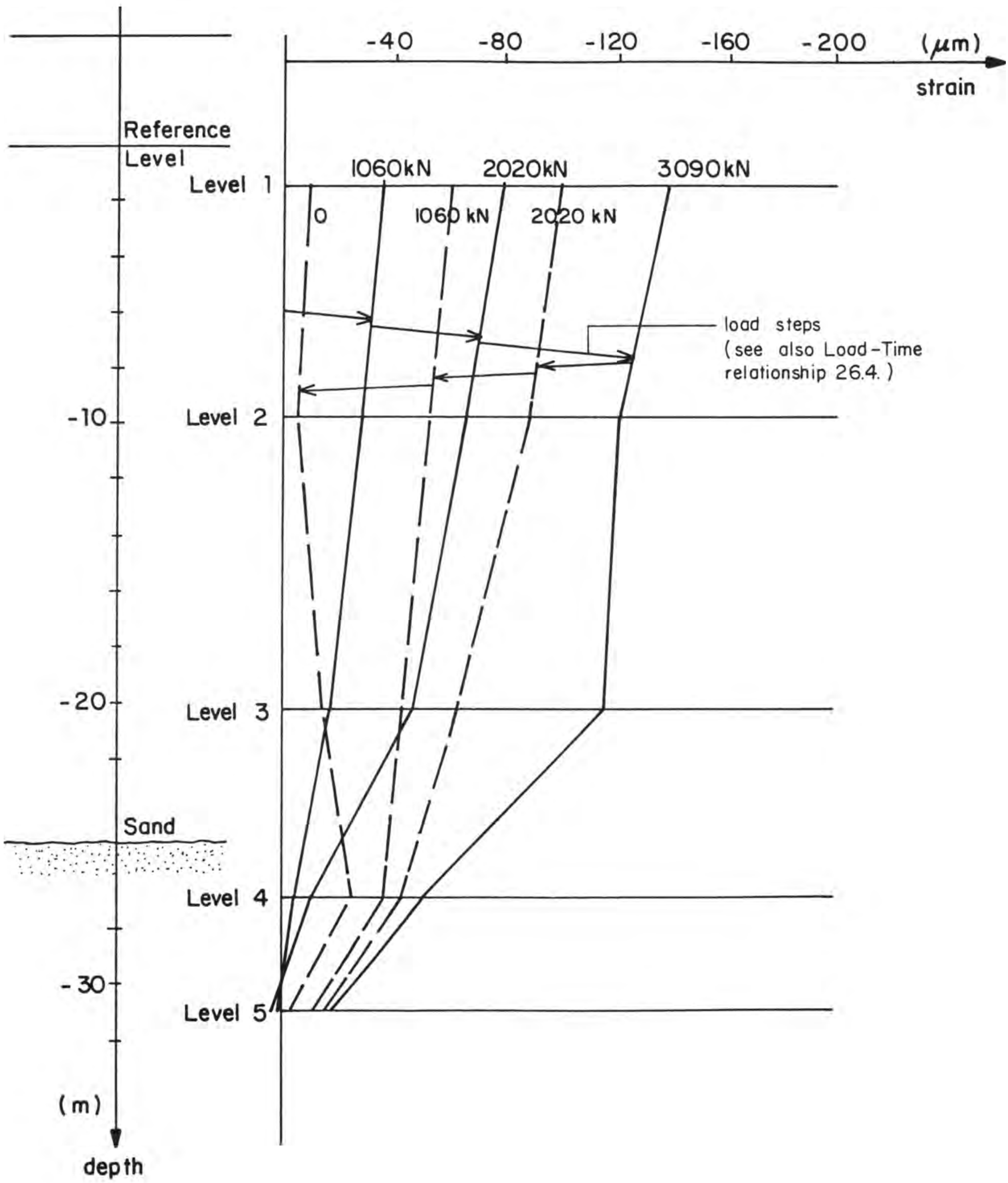
4 jacks transmit the test load from the centre girders to the top of the test pile. The pile head settlement is measured at 3 periphery locations (see cable of concrete strain meter).



SPT - VALUES
PILE TESTING SETUP
60 50 40 30 20 10 0
brownish grey SILT & CLAY.
bluish grey Clayey SILT with decomposed wood.
brownish grey Clayey SILT some SAND.
reddish grey SILT & CLAY trace SAND.
yellowish grey CLAY
yellowish brown fine SAND.
yellow coarse to medium SAND trace GRAVEL.
Counterweight
Ground Level +4m
1 cable per strainmeter
readout unit
hydr. compressor
Reference Level (DGL)
4 hydr. jacks
Level 1
-1m
3 strainmeters
Ground water
Level 2
-9m
3 strainmeters
Level 3
-17m
3 strainmeters
limit of sand layer
-24m
Level 4
-25m
3 strainmeters
Level 5
-28m
3 strainmeters
-29m
pile bottom

BAYINTNAUNG BRIDGE, YANGON

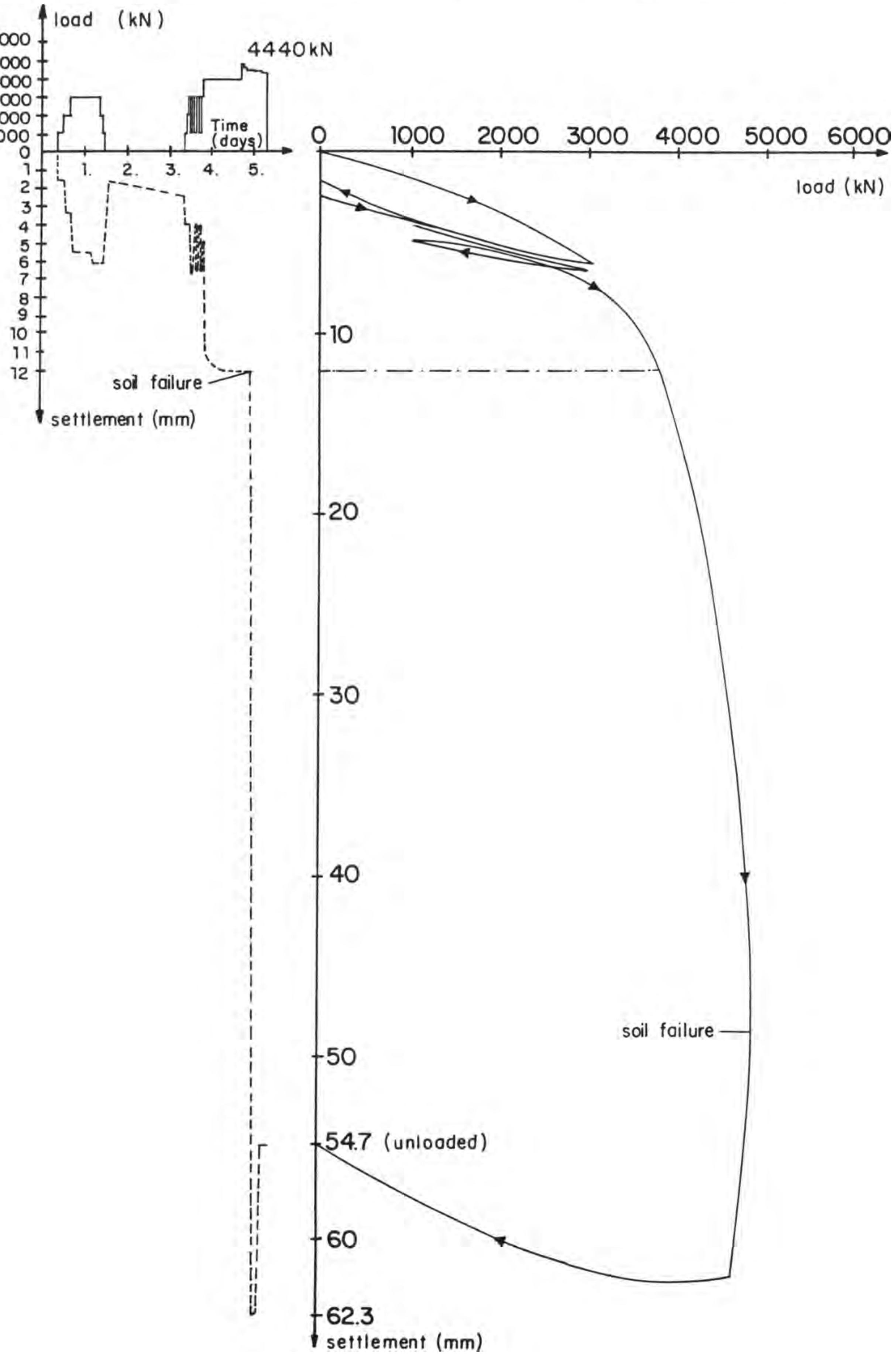
INSTRUMENTED TEST PILE NO. 1
Strain Measurements



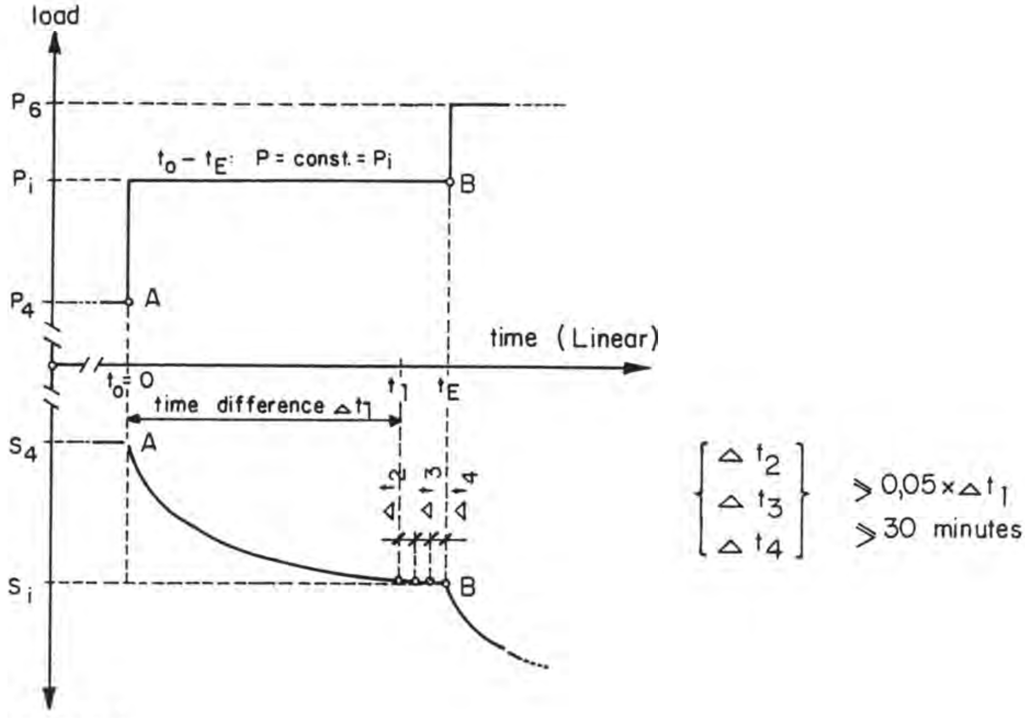
BAYINTNAUNG BRIDGE, YANGON

INSTRUMENTED TEST PILE NO. 1

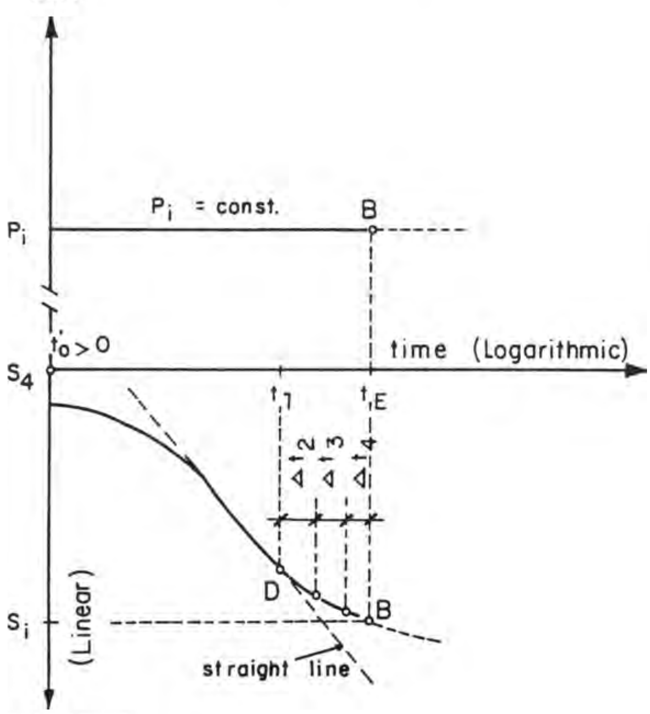
Load - Time - Settlement Relationship / Load - Settlement Relationship



1. Linear Time - Settlement Diagram



2. Halflogarithmic Time - Settlement Diagram



conditions for B :
settlements have finished under the load P_i

t_o : time for loading from P_4 to P_i.

t_1 : starting point from upwards deviation from straight line (logarithmic time scale)

t_E : settlement finished under constant load P_i

4 Design Basis

4.1 General

The designs demonstrated in the following sections show the bridge system introduced under the Yangon - Pathein Road Project. However, with minor ammendments, it is also applicable for other steel truss bridges of different spans and, in the case of foundations design, for any other bridge type. The following sections illustrate the approach for wide diameter, cast in situ reinforced concrete, deep bored piles, reinforced cement concrete abutments and piers and standard steel truss modules of 36.5 m. The system adapts for single or multiple span steel truss bridges.

Pile design and construction are for the difficult subsoil conditions of the delta area utilizing the mobile drilling rig supplied which is capable of operation either off - or on - shore. This equipment permits 2.5 m diameter piles to be bored with only a short steel guide casing and bentonite drilling mud borehole support down to depths of 50 m.

The truss bridge elements are designed to be assembled at site using only simple, precision made steel components small enough for the transport network of the area for cantilever erection with a mobile crane, without using falsework or closing the river.

Materials used for the abutments, piers and pilecaps and for the piles, with a very few exceptions, are indigenous. The bridge elements were ordered after international tender and , because of their precision nature, they are not currently able to be manufactured in Myanmar.

4.2 Design Specifications

For the bridge, the design loading specifications adopted were :

Traffic:	2 full lanes AASHTO Standard Vehicle Loading HS 20-44
Footways:	5.0 kN/m2 live load (where provided)
Railings:	1.0 kN pointed load
Wind:	1.0 kN/m
Seismic:	12% of dead load

based upon :

AASHTO Standard Specification for Highway Bridges (1983)
NAASRA Bridge Design Specification (1976)
AS 1250-SAA Steel Structures Code
AS 1511-SAA High Strength Bolting Code
and AS 1554-SAA Welding Code

The bridging elements supplied are complete with bearings, railings and deck joints. All steel components are galvanised in accordance with AS 1650.

4.3 Foundations

The design of the pile foundations for the bridge system is a direct consequence of their loading, with considerations including dead load, live load to AASHTO standards plus loading from wind, earthquake and impact. The basis of pile design is the transfer of vertical load to the soils by a combination of skin friction and soil bearing resistance at the pile foot and simple acceptable limits of subsequent settlement. The transfer of horizontal forces to the soil is also considered. The use of bentonite mud to support the pile during construction introduces no new features to the design process. The effects of negative friction, consolidation of soft layers, group effect and bentonite are considered. Concrete design is related to sand, aggregates, cement and steel as found in Myanmar and the design is based upon soil tests as currently possible including visual classification, standard penetration tests, unconfined compression and triaxial shear tests. For larger bridges, data from test piles may permit more economical design assumptions to be included.

4.4 Piers and Abutments

The design of piers and abutments by Public Works has not been significantly altered within the project. Designs have been checked as adaquate for the loads to be imposed upon them and the available construction techniques. The design required that the superstructure dead load, the live load as per the AASHTO and other specifications above, wind load, earthquake load and also possible ship collision load are considered.

4.5 Bridge Design

The initial bridge design was based upon a 36.5 m span, standard Warren truss configuration detailed for two lanes of AASHTO vehicle loading HS 20-44. The standard design allows for bridges with and without walkways and the span may be extended. A basic feature of the design is for erection using only hand tools and a light crane, no component weighs more than 1.5 tonne or is longer than 8.5 m.

The primary elements of the concrete deck are precast concrete panels. These span between steel stringers and act as formwork for the in-situ concrete topping slab. Thus the deck becomes a four span continuous slab, spanning one-way across the stringers. The stringer beams then act compositely with the concrete slab in the longitudinal direction. The deck will support its own self-weight and the weight of the bituminous wearing course. Live and earthquake loads are imposed as per the above standards.

Each span has one pair of fixed and one pair of moving bearings designed in accordance with AASHTO Standard Specification for Highway Bridges. Holding down bolts are designed for uplifts of 10% of the dead load in accordance with accepted practice for seismic conditions. Longitudinal forces generated by earthquake and live loads are transferred to the abutments via the fixed bearings. The steel parts are fabricated such that with dead load and average bolt slip deflection a camber of one eight hundredth of span will remain.

5 Design of Large Diameter Bored Piles

5.1 General

5.1.1 Introduction

This section provides guidelines and procedures for the design of bored piles by easily understandable procedures and tables to facilitate design work, covering most of the cases which are likely to be encountered when designing a pile foundation.

Pile design is based on empirical results to which the various existing theoretical approaches have been adapted. Most of the theories or approaches are applicable only for specific conditions, e.g. complete guidelines exist for piles in London clay, but their application on other subsoil conditions is doubtful without exact knowledge of the original London clay. The approach presented in this chapteris a summary of the results and theories presented in literature, in case of ambiguities conservative values are indicated. In order to take advantage of the available bearing capacity, it is recommended that in-situ pile tests are made. This is particularly relevant for bridgeworks requiring either long spans or a large number ofpiles (see Section 3.6).

Section 5 is confined to the design of large diameter bored piles. The design approach for driven piles is not considered. The following parameters would be different for driven piles:
- skin resistance (compared to bored piles, skin resistance is less for cohesionless soil and bigger for cohesive soils)
- negative friction (increased compared to bored piles)
- admissible stresses (generally the piles are pre-fabricated and the concrete quality will be higher)

Design limits, e.g. admissible stresses, depend on the codes applied. The limits indicated in this chapter are derived from Indian (which are based on British), German and Swiss codes and put into a general form where admissible stresses are expressed as a fraction of the ultimate material strengths.

Pile foundation design should ensure that all loads of the bridge are safely transferred to the ground. Vertical load is generally transferred by both point resistance (the soil resistance at the pile foot) and skin friction (adhesion along the pile shaft). Horizontal load is transferred by earth pressure.

In all design a vertical displacement of at least 10 mm must be allowed to activate skin friction. Point resistance is only fully activated after displacements of 20 to 50 mm or more.

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(3) "Elements of Foundation Design", Smith G.N. and Pole E.L.
(4) "Pile Foundations", Chellis
(5) "Bodenmechanik und Grundbau", Lang H.J. und Huder J., 1985 (in German)
(6) "Grundbau Taschenbuch: Pfähle", Franke E., 1982 (in German)
(7) "Biegemomente elastisch eingespannter Pfähle", H.Werner, Beton-und Stahlbau 2/1970 (in German)
(8) "Elastically Fixed Structures", Sherif, 1976

<u>Design and Construction Codes:</u>

Swiss: *SIA 162 (Concrete Structures), SIA 192 (Piles)*
German: *DIN 4014 Part 2 (Large Diameter Bored Piles), DIN 1054 (Bearing Capacitiy of Soil)*
DIN 4094 (Soil Investigations)
Indian: *IS 456 (Columns), IS 2911 (Piles)*

5.1.2 Summary of Design Criteria

A successful pile design has to fulfill the following conditions :

1) The bearing capacity of the pile must exceed the design load multiplied by a safety factor

2) Displacement under working loads shall not exceed specified limits which will depend upon the requirements of the superstructure

3) The stresses in the pile shall not exceed the admissible stresses of the materials used

Vertical bearing capacity of pile is defined by :

- admissible vertical load (see Section 5.3)
- admissible settlement of pile (see Section 5.4)
- admissible compressive stresses in pile (see Section 5.7)

Horizontal bearing capacity of pile is defined by :

- admissible passive earth pressure of soil (see Section 5.5)
- admissible bending moment in reinforced pile (in interaction with compressive forces) (see Section 5.7)
- admissible displacement of pile head (see Section 5.5)

The available construction equipment is an additional factor to be considered in any pile design. With the equipment introduced for the Yangon - Pathein Road Project the limitations are :

- pile diameter 0.80, 1.25, 1.80 or 2.50 m
- pile depth up to maximum of 50 m
- vertical piles only

5.1.3 Notations

The following symbols, subscripts and superscripts are used in this section:

A	- cross sectional area
c	- cohesion
d_u	- undrained shear strength
D	- pile diameter
d	- diameter of steel reinforcement
d_c	- compressive strength of soil
DGL	- design ground level (actual ground level minus 1.5 m)
E	- Young's modulus of elasticity
f_c	- strength of concrete
f_y	- yield strength of steel
GWL	- ground water level
H	- horizontal force
h	- height

I	-	moment of inertia
k	-	used for factors or coefficients
k_h	-	coefficient of horizontal subgrade reaction
L	-	total length of pile
L_o	-	characteristic length of pile
L_1	-	length of embedment in bearing layer
M	-	moment
N	-	number of blows (see SPT)
n	-	ratio of moduli of elasticity of reinforcement steel and concrete
n_h	-	horizontal bearing coefficient
N_q, N_c	-	bearing capacity factors
Q	-	vertical bearing capacity
q_v	-	vertical stress in soil (overburden)
q	-	unit bearing capacity (vertical)
R	-	resultant of external loads acting on pile
SPT	-	standard penetration test
s	-	safety factor
UCS	-	unconfined compression test
u	-	horizontal displacement (longitudinal direction)
V	-	vertical force
v	-	horizontal displacement (transverse direction)
W	-	weight
w	-	vertical displacement, settlement
z	-	depth below DGL
$\alpha, \beta, (\gamma)$	-	used for various factors
ε	-	strain
μ	-	reinforcement coefficient
$\varnothing$	-	angle of internal friction
θ	-	rotation angle
γ	-	unit weight
σ	-	stress (compression, tension)
τ	-	shear stress
adm	-	admissible

various sub-indices are also used :

c	-	concrete
eff	-	effective, actual
h	-	horizontal
i	-	individual (sample)
max	-	maximum
min	-	minimum
nf	-	negative friction
p	-	pile
pr	-	point resistance
sr	-	skin resistance
st	-	steel
u, ult	-	ultimate
x	-	longitudinal direction (bridge axis)
y	-	transverse direction , yield
z	-	vertical direction

5.1.4 Embedment in Bearing Stratum

A desirable solution to the design problem is to put the piles into a dense layer with high resistance (e.g. sand, gravel). Calculations of point resistance clearly show that the deeper this layer, the higher the point resistance. The following limits for the thickness of this layer have to be taken into account *(DIN 4014/2)* :

a) L_1, the embedment length should be greater than 3m

b) $\Delta L - L_1$, the thickness of the bearing stratum below the pile should be greater than the larger of 2m or 2D

Condition a) considers uncertainties in investigations and tolerances in construction. Condition b) avoids breakthrough of the pile through bearing stratum.

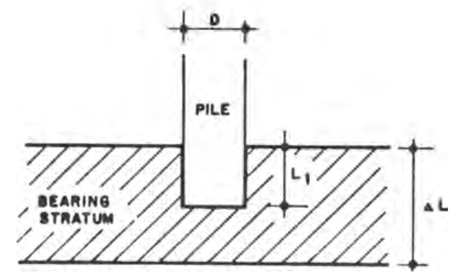


Fig 5.1

It is therefore clear that a good bearing stratum for large bored piles should have a thickness of at least $\Delta L = 5$ m.

5.1.5 Floating Piles

If the main part of the load (say more than two thirds) is transferred to the soil by friction, the pile is called "floating". Great care must be taken for the design of this type of pile, due to the fact that calculation of skin friction requires a sound knowledge of the characteristics and behaviour of the soil which is not always available.

In addition, negative friction may reduce the bearing capacity considerably (see Section 5.1.7). Floating piles should be avoided or, if this is not feasible, increased safety factors should be used (see Section 5.3.9).

5.1.6 Consolidation of Soft Layers

Layers of soft clay will consolidate under the induced load and thus move downwards so that the initial displacement between pile and soil will decrease. It is evident that all the layers above will undergo the same displacement. If the relative displacement between pile and soil is zero, no friction will be induced. The best solution to this problem is to disregard the skin friction of all the layers possibly moving downwards, otherwise consolidation effects have to be estimated.
An application of this methodology is given in the example at the end of this section.

5.1.7 Negative Friction

Consolidation due to additional surface loads or pumping of water from one of the layers can cause a downward movement of the soil relative to the pile shaft. In such situations, skin friction on the pile is acting downwards, hence the term negative friction. These effects are considerably larger for driven piles than for bored piles, mainly due to residual stresses of the driving operation. In any case, careful examination of negative friction effects is recommended (see Section 5.3.8). Battered piles (inclined to bear horizontal forces) should be avoided where negative friction is likely to occur.

5.1.8 Bentonite Piles

Bentonite piles are piles where the open borehole is supported by a bentonite drilling mud. Concrete is cast into the borehole, initially from the base, displacing the bentonite slurry upwards. This leaves a cake of fine material between the concrete and soil which, tests show, does not reduce skin friction *(Stocker M.: "The influence of bentonite on the bearing capacity of piles formed with temporary casing").*

The concrete is cast under the pressure of the bentonite mud and is pressed evenly to the sides of the borehole, ensuring good contact with the soil. As a result, shear failure at the ultimate load will not occur at the pile - soil interface but rather in the adjacent surrounding soil (yielding higher resistance). Accordingly, the usual approaches to compute the ultimate shear strength at the pile "skin" will give conservative design values.

Due to irregularities during execution, the pile diameter will not be even but will become bigger than specified as, inevitably, parts of the pile "wall" are damaged during excavation. For design purposes, the diameter specified is taken as design diameter.

5.2 Properties of Pile Foundations

As a basis for design calculations, a soil profile of the piling site must be established, containing the following data on the various soil layers :

- depth and thickness of each layer
- ground water level
- relevant soil properties
  (unit weight, angle of internal friction, cohesion, shear strength, modulus of elasticity etc.)

For this purpose, all the relevant data of the soil investigations described in Section 3 must be collected and discussed. This data may be compiled in a table utilising the following recommendations:

- combine layers with similar properties into one layer with average values, to reduce the amount of calculations
- if the drill hole of the investigation does not match the piling site exactly, the layers can be interpolated having regard for nearby drill holes and the general geological situation
- if several investigations for one pile group exist and do not differ much, compile one single design profile by interpolation

- complete missing values by comparing similar layers found at the same depth at site (or comparable site), or by deriving them from other measured soil characteristics

Note that the established profile determines the bearing capacities of the pile and thus should be compiled with great care. In case of uncertainties, conservative values should be used.

In order to determine soil characteristics, data from soil investigations is required. This will consist of the results of the following tests :

1) field tests:	- visual classification of the various soil layers - standard penetration test (SPT)
2) laboratory tests:	- grain size distribution (USCS - classification) - consolidation test - unconfined compression test (UCS) - triaxial shear test - compression test (odeometer) - standard physical properties, i.e. density, water content

For successful pile design, it is very important that the design engineer and the geologist determine the type of soil investigations together. It is best to drill one bore hole to the estimated depth of the pile, compute a preliminary design and adjust the soil investigation programme to its results (depth of bore holes, important layers, tests required etc.). It is problematic to design a pile where soil investigations do not cover the entire depth needed and, if at all possible, tests to beyond the estimated required depths should be insisted upon. Generally, the deeper layers are the most important since they determine point resistance and the embedment length.

The investigations should go down at least 5m deeper than the assumed pile depth (see also Section 5.1.4).

In order to obtain a realistic assessment of the soil profile, it is not necessary to make all tests on all samples. A typical example of a soil investigation is given in Table 5.1. The engineer has decided to perform UCS tests on the first cohesive soil samples and triaxial shear tests on approx. every third cohesionless soil sample (see also Section 3 for evaluation of soil properties).

Some correlations between test results in order to estimate soil characteristics for calculations are given below. Values obtained by these relationships are only of an indicative nature and do not replace test results.

Estimate of angle of internal friction *(e.g. Poulos (2))* :

$$\varnothing = 15 + 15\sqrt{N}$$, where N is the number of blows for 300 mm (1ft) of the Standard Penetration Test (SPT)

Estimate of undrained shear strength c_u:

For saturated clays the following correlation can be given for short-time loading :

$$c_u = d_c / 2$$,where d_c is the ultimate compression strength determined with UCS test

Typical soil properties of sand and clay are given in Table 5.2. Note that these values are average values and should only be used to compare similar layers or to check investigation results.

<u>Example:</u> The following is an example of data collected during the soil investigations at Pathwe Bridge . The investigation was done with a standard penetration hammer and split spoon sampler.

sample	depth below ground level (m)	type of soil	γ wet (kN/m^3)	N SPT (blows/ft.)	d_c UCS test (kN/m^2)	friction angle ø (degrees)	cohesion c (kN/m^2)
1	0 - 0.6	soft silt & clay	16.5	2			
2	1.5 - 2.1	"	16.5	2			
3	3.0 - 3.6	"	17.0	3	15.6		
4	4.6 - 5.2	soft sandy & clayey silt	16.5	5	21.4		
5	6.1 - 6.7	medium sand, some silt	18.0	7			
6	7.6 - 8.2	"	18.4	9		27	2.4
7	9.1 - 9.7	"	18.5	10			
8	10.7 - 11.3	"	18.2	11		28	2.4
9	12.2 - 12.8	"	18.2	12			
10	13.7 - 14.3	"	18.1	11			
11	15.2 - 15.8	"	18.4	13		28	2.0
12	16.8 - 17.4	"	18.5	14			
13	18.3 - 18.9	medium silty sand, trace clay	18.6	14			
14	19.8 - 20.4	"	18.6	12		29.5	2.7
15	21.4 - 22.0	medium silty sand, some clay	20.2	12			
16	22.9 - 23.5	soft silt & clay, trace sand	18.0	10			
17	24.4 - 25.0	"	18.6	9	26.8		
18	25.9 - 26.5	"	18.7	9			
19	27.5 - 28.1	medium sand & silt, some clay, trace gravel	20.0	13			
20	29.0 - 29.6	"	19.0	15		31.5	5.8
21	30.5 - 31.1	medium sandy silt, some clay	19.5	13			
22	32.0 - 32.6	"	19.8	17		24	3.7
23	33.6 - 34.2	"	19.8	21			
24	35.0 - 35.6	dense sandy silt, trace clay	20.6	27			
25	36.6 - 37.2	"	19.5	33		27	2.4
26	38.1 - 38.7	very dense sand, some silt, trace gravel	19.5	42			
27	39.6 - 40.2	"	19.7	43		32	3.2

Table 5.1

Typical soil properties of sandy soils:

SPT (blows/ft) N	consistency	internal angle of friction ø	modulus of elasticity E_s (10^3 kN/m^2)	horizontal bearing coefficient n_h (10^3 kN/m^3)	n^*_h (10^3 kN/m^3)
< 10	loose	30	25 - 50	2	1.2
10 - 20	medium	33	50 - 70	6	4
20 - 30		36			
30 - 40	dense	39	70 - 100	18	10
40 - 50		41			
50 - 60		43			
> 60		44			

* if the pile is submerged in ground water (high ground water level)

Table 5.2a

Typical soil properties for clayey soils :

SPT (blows/ft) N	consistency	compressive strength d_c (kN/m^2)	cohesion* c (kN/m^2)	modulus of elasticity E_s (10^3 kN/m^2)
< 2	very soft	0 - 25	0 - 12.5	
2 - 4	soft	25 - 50	12.5 - 25	4 - 5
4 - 8	medium	50 - 100	25 - 50	5 - 10
8 - 15	stiff	100 - 200	50 - 100	10 - 40
15 - 30	very stiff	200 - 400	100 - 200	40 - 70
> 30	hard	> 400	> 200	70

Table 5.2b

* equal to undrained shear strength for short durations

Above values can be found e.g. in Poulos (2), Terzaghi in Franke (6), Smith (3), IS 2911 and DIN 4094

5.3 Calculation of Vertical Bearing Capacity

The design rules given in this section are for long large diameter bored piles only. Generally, the same procedure can be used for short bored piles, with special care to the horizontal bearing capacity. The rules cannot be taken over as such for driven piles, since coefficients given in the tables would differ.

5.3.1 Pile Layout

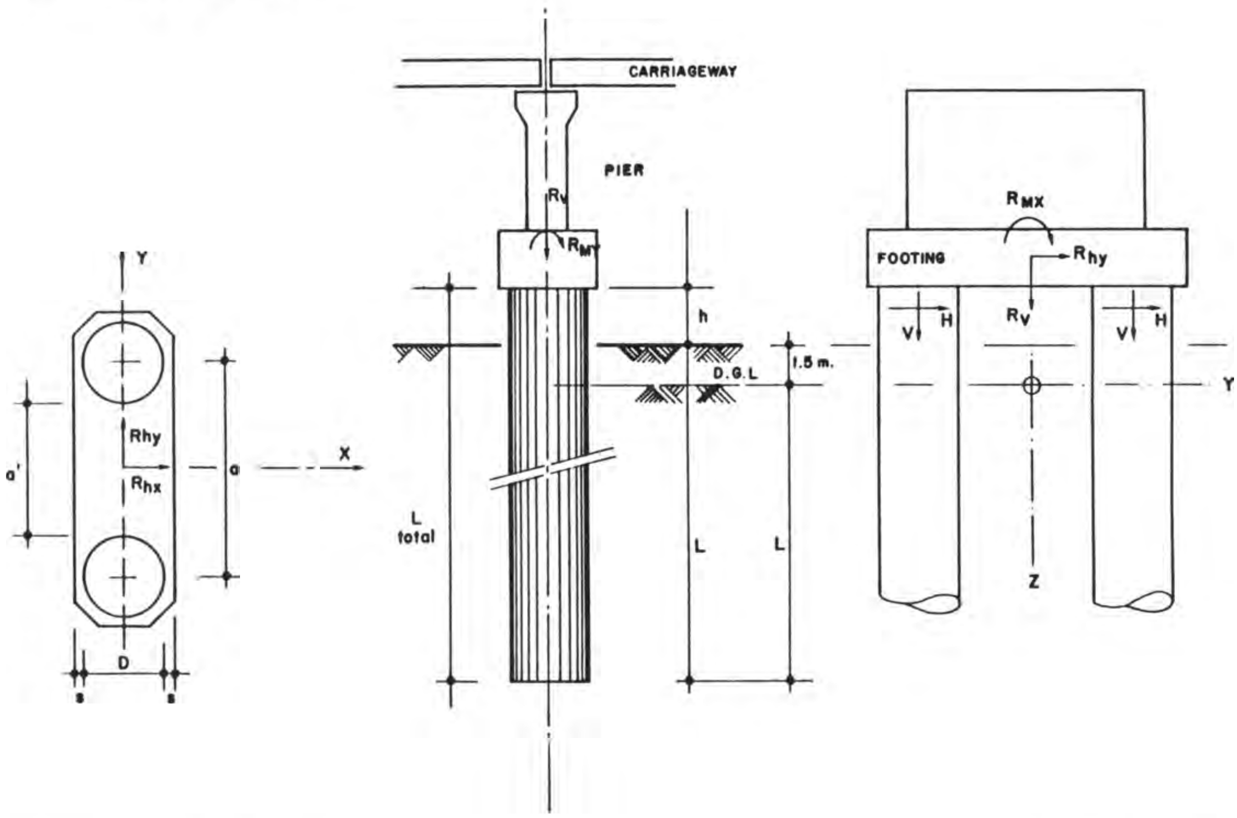


Fig 5.2

D = 0.80, 1.25, 1.80 or 2.5 m

The following limits of dimensions are recommended : a' > 1.5 D
(Vesic (1) and Franke (6)) a > 2.5 D
s > 0.3 m

Figure 5.2 shows an example of a simple layout with two piles (considered as minimum number), as used for Kyounggone or Pathwe bridge. Layouts with two or more piles in each direction are possible and necessary if big forces have to be transferred to the soil.

5.3.2 Loads

The loads on a pile are composites from a number of sources and depend on the superstructure of the bridge, this section sets out the designations used (see Sections 6, 7 and 8 for computation and estimates) .

The resultant of forces is designated R (see Figure 5.2) :

vertical forces :	R_v	
horizontal forces :	R_{hx}	: longitudinal direction
	R_{hy}	: transverse direction

The following forces are considered:

<u>Regular Loads</u>:

dead load	R_v	: weight of carriageway and steel structure : weight of pier and footing (+ weight of pile)
live load	R_v	: traffic load (without impact factor)
	R_{hx}	: braking forces
wind load	R_{hy}	: wind on carriageway and pier
temperature	R_{hx}	: elongation of superstructure

<u>Exceptional Loads</u>:

earthquake load	R_{hx}	: $R_{hx} = \alpha \cdot R_v$ (dead load of structure above ground level)
(static method)	R_{hy}	: $R_{hy} = \alpha \cdot R_v$ (dead load of structure above ground level)
impact loads	$R_{hx,hy}$	: impact due to truck or ship crash according to pier design

Where α is the ratio of horizontal to vertical forces required for the static method of earthquake calculations. The factors are given in the construction codes and depend on the seismicity of the region.

Example: for Pathwe Bridge: $\alpha = 0.12$

All these forces have to be reduced to the pile top. Note that moments R_{My}, R_{Mx} occur due to horizontal loads and due to eccentricity of vertical loads.

5.3.3 Load Cases

In general, the following load cases have to be considered to define maximum loads on piers and piles:

1)	Main load	: Dead + Live + Wind
2)	Maximum load; longitudinal direction	: Dead + Impact or Earthquake
3)	Maximum load; transverse direction	: Dead + Earthquake or Impact

However, all probable and possible combinations of the above loads should be checked to define the relevant design load cases.

5.3.4 Preliminary Design

For preliminary design only, average values of unit point resistance q_{pr} and skin friction q_{sr} for different soils are indicated in Table 5.3 *(DIN 4014/2)*.
Note that values given have to be divided by the safety factors indicated in Section 5.3.9 to achieve admissible loads.

Typical bearing capacities of bored piles -- diameter 0.8 to 2.5 m

Soil	SPT	q_{pr} (kN/m^2)	q_{sr} (kN/m^2)
gravel	N > 40	5000	100
sand	N > 30	3500	100
cohesive soil	N > 20	1200	50
"	20 > N > 10	1000	40
"	10 > N > 5	very low	25

(derived from DIN 4014/2) Table 5.3

5.3.5 Point Resistance

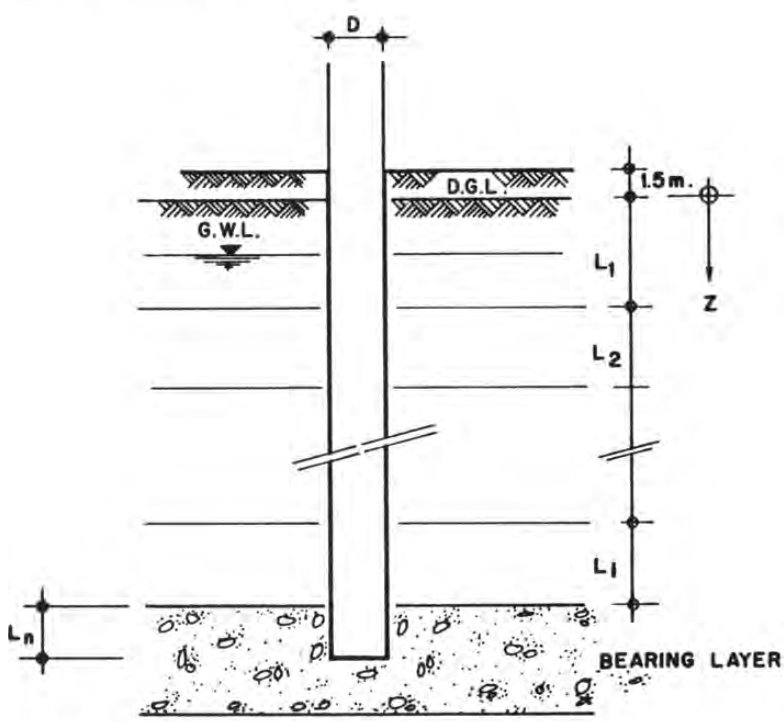


Fig 5.3

ΔL_i : length of individual bearing layer

n : total number of layers

Two approaches (theoretical and empirical) are given below. Both should be checked if possible and the lower value of the two methods applied for design.

a) Theoretical Approach

The same formulae as for the calculation of the bearing capacity of a flat foundation are used but the factors are adapted to meet the properties of the deep foundation *(e.g. Vesic (1) or Lang (5))*.

Bearing coefficients as functions of ø :

$$N_q = e^{\pi \tan \varnothing} \cdot \tan^2 (45° + \varnothing / 2)$$
$$N_c = (N_q - 1) / \tan \varnothing$$

The bearing coefficients may be derived from the diagram given in Figure 5.4.

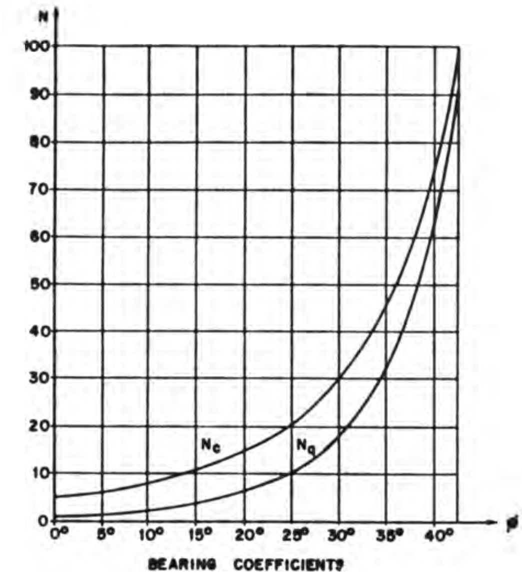


Fig 5.4

unit point resistance of pile :

- cohesionless soil (including silt) :

$$q_{pr1} = \beta (c N_c + q_v N_q)$$

where:

N_q, N_c are the bearing coefficients as described above

β is the form factor for structures with limited extension *(Lang (5))* : $\beta = 1 + (b_1 / b_2) \cdot \tan \varnothing$

where b_1 and b_2 are the widths (in x- and y-direction) of the pile: $b_1 / b_2 = D / D = 1$

therefore: $\beta = 1 + \tan \varnothing$

q_v is the vertical stress (overburden pressure) at the pile tip

$$q_v = \sum_{i=1}^{n} \gamma_i \cdot \Delta L_i$$

γ : effective unit weight of soil, below ground water level: $\gamma' = \gamma - \gamma_{water}$

- cohesive soil:

$$q_{pr1} = c N_c$$ *(Lang (5) and Franke (6))*

where:

$N_c = 9$ for $L/D > 5$
$N_c = 0.6\ L/D$ for $L/D < 5$

b) <u>Empirical approach</u> :

The unit point resistance is expressed as a function of SPT values:
(Vesic (1) and Meyerhof in Franke (6))

$$q_{pr2} = \alpha N'$$

where:

$N' = N$ for $N < 15$
$N' = 15 + (N - 15) / 2$ for $N > 15$

and values of α are set out below :

soil	α in kN/m^2
gravel and sand	400
silt	250
saturated clay	200

(derived from Vesic (1) and Franke (6)) Table 5.4

In summary : The ultimate point resistance for a single pile is the lesser of the calculated and the empirically derived point resistances.

$$q_{pr} = \min \left(q_{pr1}, q_{pr2} \right) \qquad \text{and} \qquad Q_{pr} = A_p \cdot q_{pr}$$

5.3.6 Skin Resistance

As a first step, all the layers relevant for skin friction have to be determined. Note that consolidation and downward movements of layers reduce skin resistance. (see Section 5.1.6 and 5.1.7).

a) Cohesionless soil (including silt)

For each individual soil layer, the skin friction increases with depth according to the following formula :

$$q_{sr,\,i} = \alpha \; q_{vi}$$

where:

q_{vi} is the vertical stress at the centre of the i'th layer

$$q_{vi} = \gamma_i \; \Delta L_i / 2 \; + \; \sum_{j=1}^{i-1} \gamma_j \; \Delta L_j$$

γ : effective unit weight of soil, see Section 5.3.5

α is a factor depending upon the soil type of the individual layer set out in the table below

soil type	N (SPT)	α
dense sand, gravel	> 30	0.8
medium sand	10 - 30	0.6
loose sand	< 10	0.4
silt		0.3

(derived from Lang (5)) Table 5.5

The values given are for long bored piles and can be regarded as conservative values for short piles. Experience shows that skin friction above 150 kN/m^2 is not realistic, so that higher computed values shall be reduced to:

$$q_{sr,\,i} < 150 \text{ kN/m}^2$$

b) Cohesive soil

The same approach as for cohesionless soil can be used for clays. The value of α is given in Table 5.6a as a function of the undrained shear strength (for correlation of the undrained shear strength to SPT values refer to Section 5.2).

$$q_{sr,\,i} = \alpha \; q_{vi}$$

clay type	undrained shear strength c_u (kN/m^2)	α
soft	< 25	0.2
medium	25 - 50	0.3
stiff	50 - 75	0.5
stiff	75 - 100	0.8
very stiff	100 - 125	1.2
very stiff to hard	> 125	1.5

(derived from Vesic (1) and Meyerhof in Franke (6)) Table 5.6a

In reality, the value of α decreases with depth and the skin resistance will eventually be almost constant. This effect can be considered by expressing the limit of the skin friction as a function of the undrained shear strength:

$$q_{sr,\,i} \;=\; \beta \cdot c_{u,\,i}$$

clay type	c_u (kN/m^2)	β
soft	< 25	1.0
medium	25 - 50	0.8
stiff	50 - 100	0.5
very stiff	100 - 200	0.4
hard	> 200	0.25

(derived from Meyerhof in Franke (6) and DIN 4014/2) Table 5.6b

The lesser value of the two computations will be relevant. Experience shows that skin friction above 100 kN/m^2 is not realistic *(DIN 4014/2)* , so that

$$q_{sr,\,i} = \min\ (\ \alpha \cdot q_{vi}\ ;\ \beta \cdot c_{u,\,i}\ ) \quad < 100\ \text{kN/m}^2$$

After determination of values of the skin friction for each layer relevant for skin resistance, the ultimate skin resistance for a single pile is given by :

$$Q_{sr} = \P \cdot D \sum_{i=1}^{n} q_{sr,i}\ \Delta L_i$$

5.3.7 Group Effect

The bearing capacity of a pile group is generally smaller than the sum of the capacities of the single piles, since the stresses in the soil originating from neighbouring piles have to be superimposed. Note that therefore pile separation in pile groups (spacing) should normally exceed 2.5 times the pile diameter (see Section 5.3.1). Observing this limit, the bearing capacity of a pile group is given by :

bearing capacity = group factor (g) · safety factor (s) · sum of capacities of single piles

- for point resistance of piles separated as above *(Vesic (1))* :

$$g_{pr} = 1, \text{ then } \quad Q_{pr} = \text{ sum of the individual } Q_{pr} \text{ of piles}$$

- for skin resistance:

in cohesionless soil:

$$g_{sr} = 1$$

in cohesive soil *(Vesic (1))* :

$$g_{sr} = \frac{2\,(b_1 + b_2)}{n \P D} \quad <= 1$$

where $2(b_1 + b_2)$ is the circumference of the pile group (see Figure 5.5)

and $n \P D$ is the sum of the circumferences of the single piles

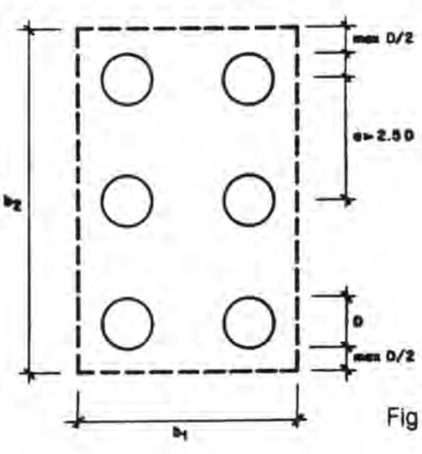


Fig 5.5

5.3.8 Negative Friction

As mentioned in Section 5.1.7, negative friction occurs where the soil moves downwards relative to the pile. It may be caused by:

1) additional load at the nearby ground surface
2) drainage of individual soil layers
3) consolidation of soft cohesive layers

The treatment of negative friction depends upon the situation:

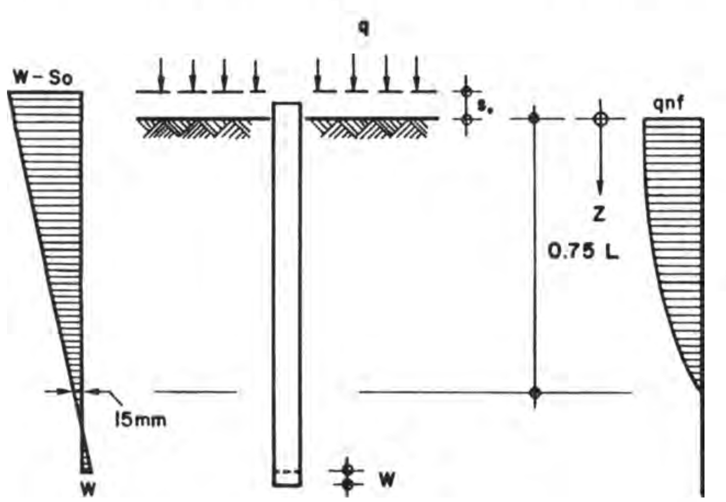
1) piles with high point resistance :

- layers which will consolidate should be omitted for calculation of skin resistance
- negative skin friction due to cause 1) or 2) can be estimated with the formulae given below and must be considered in the calculation of bearing capacity

2) floating piles :

- estimation of negative skin friction by the method given below is sufficient when negative friction is smaller than live load
- for greater values, extreme caution has to be taken and consolidation tests have to be performed to check the behaviour of the soil.
  A reduction of the bearing capacity according to test results is recommended in this case.

The procedure to estimate negative friction is set out below :



s - settlement of soil (decreasing with depth)

w - settlement of pile (constant)

q - additional load

Fig 5.6

Negative friction qnf occurs where the relative displacement pile - soil (w - s) is big enough (> 15mm) to develop skin friction *(Vesic (1))* . As a simplified approach, it can be assumed that this is the case for the upper three quarters of the pile (conservative value, see*Vesic (1))* . For this part, negative friction can be estimated according to the calculation of skin resistance, but with slightly different constants than for "normal" (positive) skin friction.
For a detailed computation of the negative skin friction, the settlement of the soil at various depths would have to be computed.

$$q_{nf,\,i} = \alpha' \cdot q_{vi}$$

where q is the vertical stress at the centre of the i'th layer (see Section 5.3.6)

The values of α' are given in the following table (they are slightly lower than for "positive" skin friction):

soil	α'
soft to medium clay	0.2
stiff clay	0.8
silt	0.25
loose sand	0.5
dense sand, gravel	0.8

(derived from Vesic (1) and Franke (6)) Table 5.7

Total negative friction of a single pile :

$$Q_{nf} = g_{sr} \cdot \sum_{z=0}^{0.75L} q_{nf,i} \cdot \Delta L_i$$ where g_{sr} is the group factor for skin resistance

For bored piles there are no feasible measures to reduce negative skin friction.To reduce this effect it has to be checked if the design could be changed to driven piles to bring down the values of α' to 0.01 - 0.05 *(see Vesic (1))* :

- installation of casing around pile shaft
- coating of pile shaft with bituminous layer

Negative skin friction of a pile group can be reduced with the group factors given for "positive" friction (see Section 5.3.7).

5.3.9 Admissible Load and Safety Factors

The ultimate vertical bearing resistance is

$$Q_u = Q_{pr} + g_{sr} \cdot Q_{sr}$$ where g_{sr} is the group effect factor

then the admissible load can be expressed as :

$$Q_{adm} = Q_{pr} / s_{pr} + g_{sr} \cdot Q_{sr} / s_{sr}$$ where s is the safety factor

The generally used safety factors are *(SIA 192)* : for point resistance , $s_{pr} = 2$
for skin friction, $s_{sr} = 3$

Consequently, the following design formula can be given :

$$Q_{eff} < Q_{adm}$$

where $Q_{eff} = V + W_p$
V : component of vertical forces, reduced to pile top
W_p : weight of pile
(note that generally the pile is submerged, so that weight of pile can be reduced by uplift pressure)

As stated in Chapter 5.1.5, safety factors should be increased for floating piles if the negative friction is not carefully assessed. In case of pile tests, the admissible load shall be determined by the minimum (in case of several tests) of the ultimate loads of the test piles, divided by an appropriate safety factor. This safety factor depends on the observed variations of the tested piles and on the transferability of the test pile conditions to the actual pile conditions. Should these conditions (soil characteristics, depth and water conditions etc.) be similar, a safety factor of 2 (in case of one single test) or 1.75 (in case of more than one test) is suggested *(DIN 1054)* . In any case, the ultimate tested load should be compared to the computed admissible load.

5.4 Vertical Displacement of Pile

5.4.1 Settlement of Single Pile

The prediction of settlement of a pile is dependent upon the estimation of soil characteristics. In practice, calculated settlement values may differ by as much as 50% from the actual settlement. The influence of greater displacements than computed upon the superstructure must therefore be examined. Two methods for calculating settlement of single piles under working load are given:

a) Graphic Method

This is a simple method but nevertheless very comprehensive. For large settlements, the results should be checked with the second method given or any other method found in literature. The principles involved for bored piles are:

- settlement of 10mm (cohesive soil) to 20mm (cohesionless soil) activates full skin friction
- point resistance is activated gradually as presented in the table below
  (for cohesionless soil, the smaller of 150 mm or 0.15 D shall be taken for computation)

	settlement w [mm]	settlement relative to pile diameter	fraction of Q_{pr} activated	fraction of Q_{sr} activated
with cohesionless soil	10		0.14	0.5
as bearing layer	20		0.23	1.0
(point resistance)	30		0.32	
	150	if > 0.15 D	1.0	
with cohesive soil	10			1.0
as bearing layer		0.01 D	0.42	
		0.015 D	0.58	
		0.05 D	1.0	

(derived from DIN 4014/2) Table 5.8

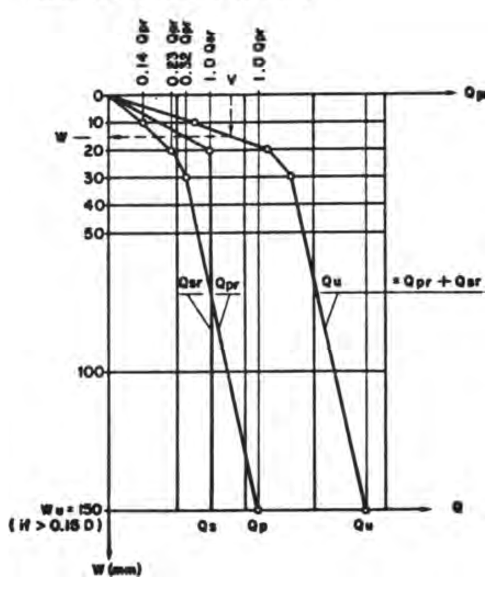


Fig. 5.7a Pile displacement with a bearing layer of cohesionless soil

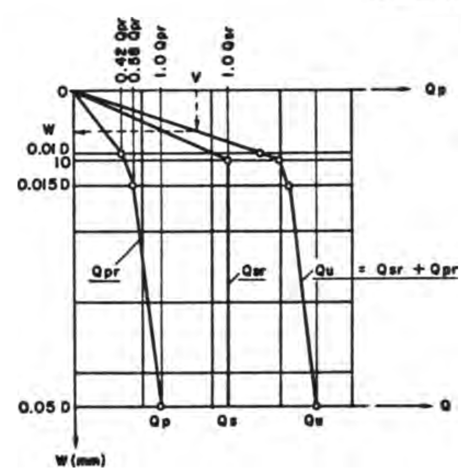


Fig 5.7b Pile displacement with a bearing layer of cohesive soil

The two load-settlement curves for point resistance and skin friction can be plotted on a diagram and added graphically to compute the load - settlement (Q - w) relation for a single pile. Actual settlements can then be estimated by readding the value w of the curve under working load V (not multiplied by safety factors) as shown in Figure 5.7.

For example, in the given diagram for cohesionless soil (Figure 5.7a) the ultimate skin resistance was calculated to be approx 70% of the ultimate point resistance of the pile. The working load was found to be approx 80% of the ultimate point resistance resulting in a settlement of approx 15mm.

b) <u>Method of Cassan</u> :

The formula has been developed for point resistance piles, where the pile is embedded in a bearing stratum *(Cassan in Lang (5))*.
Skin friction of the upper (softer) layers is neglected.
The formula can equally be applied to floating piles when taking L_1 equal to L.

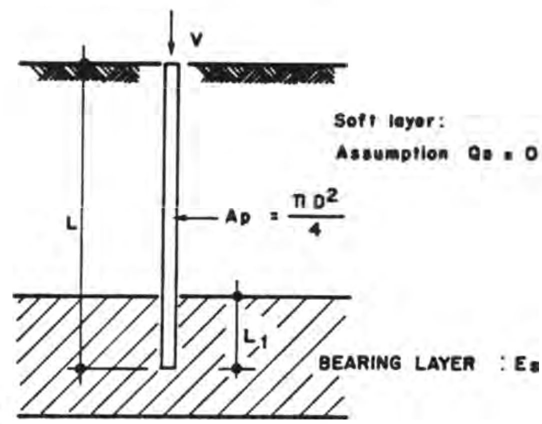


Fig. 5.8

$$w = \frac{Q_{eff} \cdot D}{A_p} \cdot \frac{1 + \frac{4.5 \cdot E_s}{f \cdot D \cdot E_p} \cdot \tanh(f \cdot L_1)}{4.5\, E_s + f \cdot D \cdot E_p \cdot \tanh(f \cdot L_1)}$$

where $\quad f = \sqrt{\frac{1.68\, E_s}{D \cdot E_p}}$

and $\quad L_1$ is the embedment length

for reinforced concrete piles, $E_p = 25 \cdot 10^6\ \mathrm{kN/m^2}$
for estimate of E_s refer to Section 5.2

<u>Simplified Method</u>

The formula of Cassan can be simplified if the following condition is fulfilled: $L_1 / D > 5$

$$w = V \cdot L \,/\, \mu \cdot A_p \cdot E_p$$

where : $\mu = 0.022\ L_1/D - 0.05$

Although the formula of Cassan has been developed for embedded piles, it can be used as well for floating piles by setting:

$L_1 = L$

Both approaches a) and b) should be used to compute the vertical displacements and the effect of the biggest computed displacement on the superstructure has to be checked.

5.4.2 Group Effect

The observed settlement of a pile group is usually greater than that of a single pile under corresponding load, since stresses in soil induced by the piles will be superposed, especially if skin friction is substantial.

This is expressed by : $\quad w_{group} = g \cdot w_{single\ pile}$

a) Piles on cohesionless bearing layers

$$g = \sqrt{\frac{b}{D}} \quad \textit{(Vesic (1))}$$

where $b = \dfrac{b_1 + b_2}{2}$ in metres

and b_1 and b_2 are the width and length of the pile group (see Figure 5.5)

This formula indicates that the influence of stress distribution in the soil outside the foundation area is vanishing with larger dimensions. Should the spacing of piles be large enough (a/D > 8), settlements of the pile group will not increase compared to the settlement of a single pile (g = 1).

b) Piles in cohesive soil

Instead of calculating a factor g , the settlement of the whole group can be estimated by considering the pile group as a flat foundation of depth z_0 *(Poulos (2))* :
(depending on the ratio of point to skin resistance)

$$Q_p / Q_s > 1 : \quad z_o = L$$
$$Q_p / Q_s < 1 : \quad z_o = 2/3\ L$$

The settlement of a flat foundation may be expressed using the theory of Boussinesq *(e.g. in Smith (3) or Lang (5))* to compute compressive stresses in soil under the relevant point A indicated in Figure 5.9.

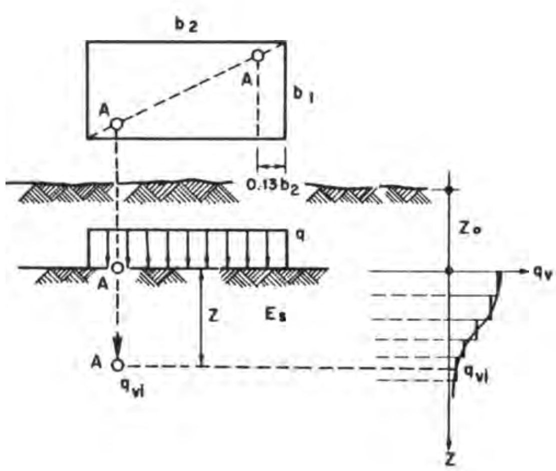


Fig. 5.9

Factor k (ratio vertical stress / load)

z / b1	b2 / b1		
	1	1.5	2
0	1.000	1.000	1.000
0.1	0.898	0.928	0.937
0.2	0.694	0.757	0.788
0.3	0.577	0.621	0.663
0.4	0.470	0.529	0.571
0.6	0.362	0.412	0.448
0.8	0.289	0.336	0.367
1.0	0.234	0.279	0.308
1.2	0.191	0.235	0.261
1.4	0.158	0.199	0.224
1.6	0.131	0.169	0.194
1.8	0.111	0.145	0.169
2.0	0.094	0.125	0.148

(from Lang (5)) Table 5.9

It is best practice to divide the soil into layers of thickness ΔLi as indicated in Fig 5.9, then:

$$w = \Sigma \frac{q_{vi} \cdot \Delta L_i}{E_{si}}$$

where $q_{vi} = k \cdot q$
k is a function of depth z from Table 5.9
q is the distributed vertical load acting on the foundation

5.4.3 Displacement at Pile Top

a) Effective displacement at the pile top

Let w_1 be the vertical displacement of the pile top

w_1 = settlement of pile (w) + elastic compression of pile

then $w_1 = w + \frac{V}{A_p \cdot E_p} \cdot L$

b) Admissible Displacements

Limits have to be established in order to ensure an acceptable performance of the superstructure under working loads. Generally, the stiffer the structure, the smaller the admissible displacement. Very often, the effect of differential settlements (e.g. of different pier foundation) is relevant for the behaviour of the structure, not only the absolute values.

$w_1 <= w_{adm}$

For a common structure of normal stiffness, w_{adm} = 20 mm represents a safe limit.

Again, it has to be emphasized that the computed displacements are only of indicative nature and that actual displacements can be significantly higher. The influence on the superstructure of displacements greater (50 to 100%) than computed has to be examined.

5.5 Horizontal Displacement

5.5.1 Pile/Pier Models for Computation

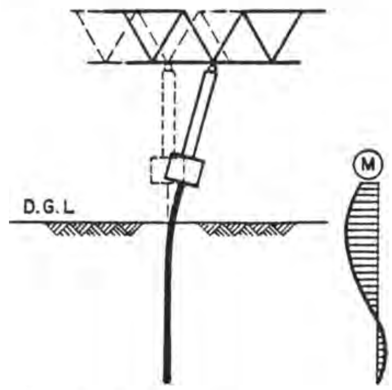
The model which serves as the basis for the computation of horizontal displacements, bending moments and shear forces depends on:

- the ratio of the moments of inertia of pile and pier
- the number of piles (in direction of horizontal forces)
- the boundary conditions for pier and pile at the pile cap (fixed/hinged), which depend mainly on the layout of the reinforcement

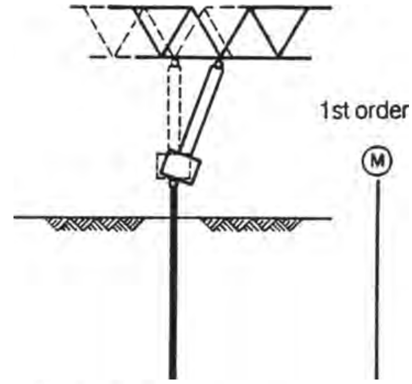
Possible layouts and the corresponding models (which will cover most of the cases encountered when designing a pile foundation) are indicated in Figure 5.10. It is evident that the choice of the model for computation has a large influence on the moments and forces and therefore has to be made with care. Different models will have to be used for the x- and y-direction if the relevant parameters (number of piles etc.) are not similar. In case of uncertainties about which model should be used, it is best to compute maximum and minimum boundary values with various model assumptions. As an example, it is clear that the displacements and moments of case b3) in Figure 5.10 must be within the values of case b1) and b2), which therefore can be considered as "extreme case" assumptions.

The computation of displacements, moments and shear forces is subdivided in three steps. In a first step, the values for the part of the pile which is below the design ground level are computed, then the values of the protruding part of the pile above the design ground level are computed, finally the values of the two parts are combined depending on the boundary conditions as described above.

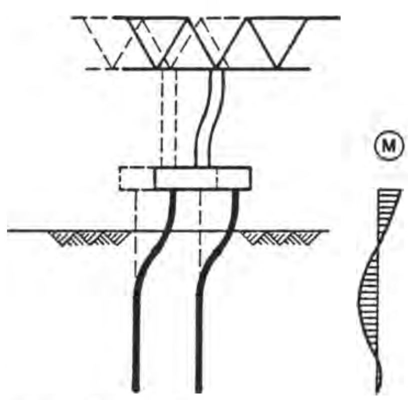
Pile/Pier Models for Horizontal Displacements:



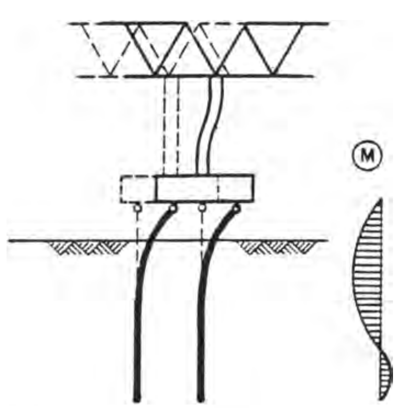
a1) single pile
fixed pile head
I pier » I pile



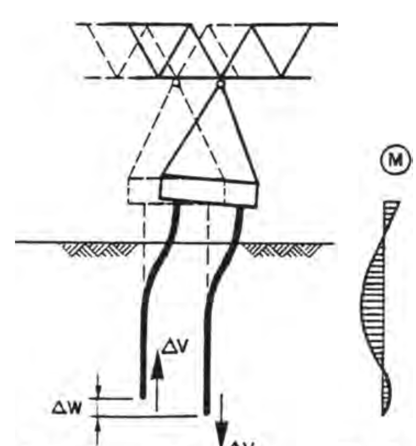
a2) single pile
hinged pile head
Attention: Stability !
(2nd order forces)



b1) several piles
fixed pile head



b2) several piles
fixed pile head
with rotation



b3) several piles
fixed pile head
I pier » I pile

Fig. 5.10

5.5.2. Single Pile

The total displacement of a single pile at the pile cap may be expressed as :

$$u = u_o + \theta_o \cdot h + u_1$$

and $\theta = \theta_o + \theta_1$

where u_o, θ_o : displacements at design ground level

u_1, θ_1 : displacements of protruding part

u, θ : displacements at pile top

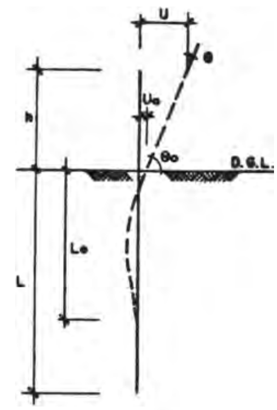


Fig. 5.11

Displacement at Design Ground Level

The pile is elastically embedded in the soil as described by the widely used theory of subgrade reaction *(e.g. in Poulos (2), Lang (5) or Smith (3))* .

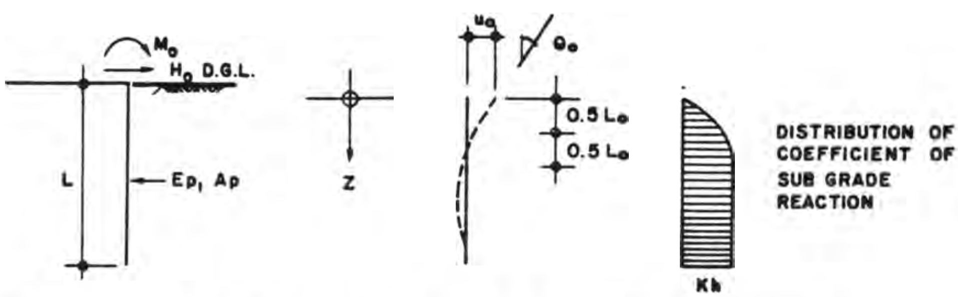


Fig. 5.12

The characteristic length of the pile is defined as follows for k_h = constant :

$$L_o = \sqrt[4]{\frac{4 \cdot E_p \cdot I_p}{D \cdot k_h}}$$

where $I_p = \pi \cdot D^4 / 64$ is the moment of inertia

and k_h is the coefficient of subgrade reaction as following *(Franke (6)* :

- for a cohesionless soil and soft clay: $k_h = n_h \cdot z / D$
  where n_h is the horizontal bearing coefficient as described in Chapter 5.2
- for cohesive soils: $k_h = E_s / D$

The coefficient of subgrade reaction is increasing linearly with depth in cohesionless soil but remains constant over the whole pile length for cohesive soils. Figure 5.12 shows a distribution of the coefficient of subgrade reaction in a soil with many layers. It can be assumed that the pile part which is below the characteristic length of the pile is strongly embedded (fixed) in the soil and does not move significantly. Furthermore, the upper soil layers generally are disturbed (pile head movement, construction process) and will not develop their full resistance. Horizontal pressures are taken mostly by the soil layers above the characteristic length of the pile, it is therefore advisable to consider the horizontal subgrade reaction of this section as relevant for the computation of the characteristic length.

average of k_h over pile length from $z = 0.5\,L_o$ to $z = L_o$

$$k_h = 1/L_o \sum_{0.5L_o}^{L_o} k_{hi}\, \Delta L_i$$ (low values are normally omitted for shorter lengths)

In the following, computation procedures are given based on the distribution of the horizontal subgrade reaction given in Figure 5.12, which will be valid for most of the cases encountered, and for long piles (pile length L greater than characteristic length). The procedure will also give good estimates if the soil conditions differ (e.g. piles only in sand).

For a detailed computation of the displacements and moments for short piles or in different soil conditions than indicated in Figure 5.12, literature should be consulted *(e.g. Werner (7) or Sherif (8))* to derive corresponding factors.

The displacement at design ground level DGL can be computed as following, using the coefficients α and β as given in Table 5.10:

$$u_o = \alpha_H \frac{H_o \cdot L_o}{E_p \cdot I_p} + \alpha_M \frac{M_o \cdot L_o}{E_p \cdot I_p}$$

$$\theta_o = \beta_H \frac{H_o \cdot L_o^2}{E_p \cdot I_p} + \beta_M \frac{M_o \cdot L_o}{E_p \cdot I_p}$$

L/L_o	1	1.5	2	3	4	>6
α_H	1.39	0.97	0.80	0.77	0.84	0.99
α_M	2.11	1.06	0.78	0.72	0.78	0.87
β_H	2.11	1.06	0.78	0.72	0.78	0.87
β_M	4.20	1.73	1.27	1.19	1.23	1.29

from Werner (7) Table 5.10

Displacement of Protruding Part of Pile:

Additional displacements due to the protruding part may be calculated by the linear-elastic static method. Figure 5.13 shows moments and deflection curves for a pile which is fixed at its bottom and for different boundary conditions at the pile head. Figure 5.14 and 5.15 show how the displacements of the pile in the soil and of the protruding part can be added. Long piles (longer than 3 times their characteristic length) can be regarded as fixed at the depth of the characteristic length, so that simplified formulae can be used in this case.

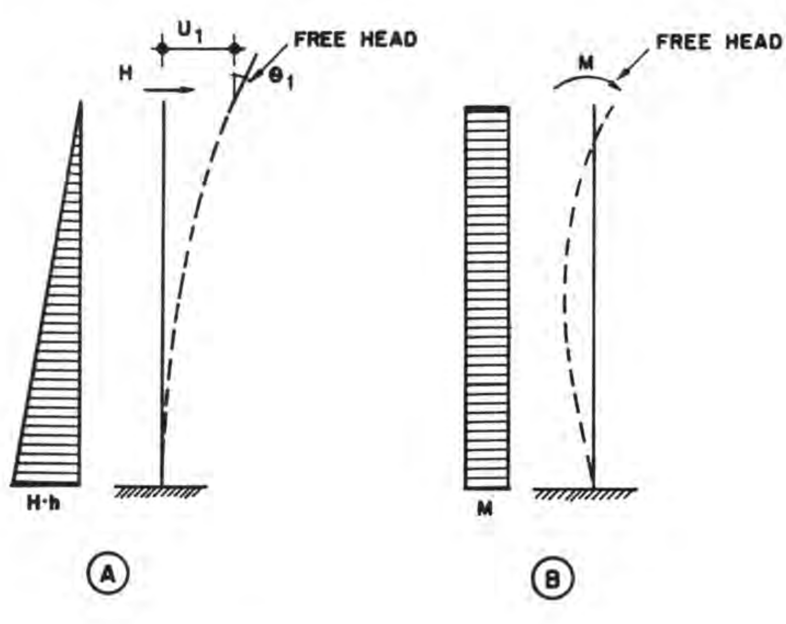


Fig. 5.13

$$u_1 = \frac{H \cdot h^3}{3 E_p \cdot I_p} \qquad u_1 = \frac{M \cdot h^2}{2 E_p \cdot I_p}$$

$$\theta_1 = \frac{H \cdot h^2}{2 E_p \cdot I_p} \qquad \theta = \frac{M \cdot h}{E_p \cdot I_p}$$

a) "Free" Pile Head

"Free" in this context means that the pile head will either follow turning movements of the pier footing (case a1) in Figure 5.10), in which case moments can be transferred from pier to pile, or that a hinge is formed at the pile head (case a2) and b2) in Figure 5.10), which is the case if the reinforcement does not guarantee a restraint at the pile head. The displacements at pile head can be computed by combining the displacements at design ground level and displacements of the protruding part as shown earlier in this chapter.

$$u = u_o + \theta_o \cdot h + u_1$$
$$\theta = \theta_o + \theta_1$$

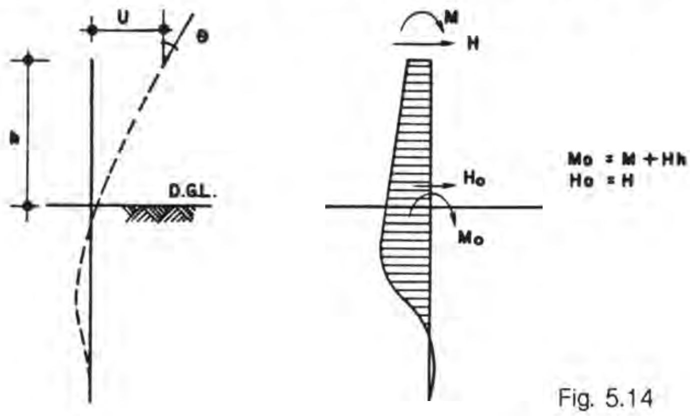


Fig. 5.14

Simplified formulae for long piles: $L / L_o > 3$ (assuming fixation of pile at depth L_o) *(Franke (6))*

$$u = \frac{(h + L_o)^3 H}{3 E_p \cdot I_p} + \frac{(h + L_o)^2 M}{2 E_p \cdot I_p}$$

$$\theta = \frac{(h + L_o)^2 H}{2 E_p \cdot I_p} + \frac{(h + L_o) M}{E_p \cdot I_p}$$

For pile tops with a hinge (case a1) and b2) of Fig. 5.10), the same formulae can be applied while setting the moment M at pile top equal to zero. Should the pile cap be at the level of the design ground level, the height h of the protruding part has to be set to zero, and above formulae can be used.

b) Fixed Pile Head (no rotation at pile head)

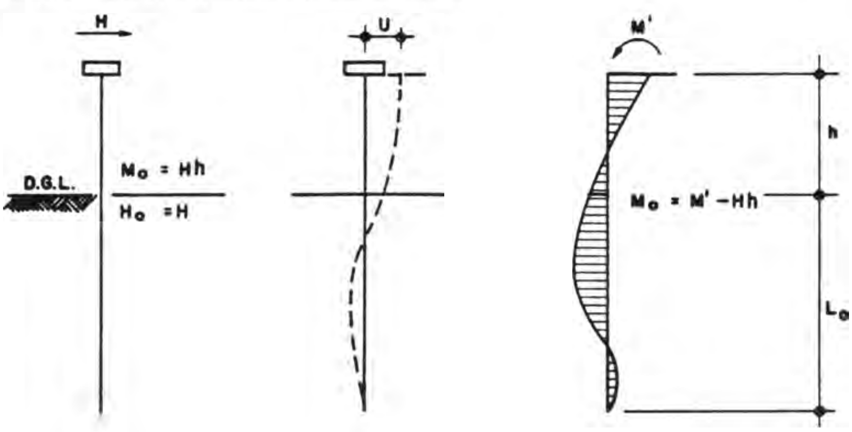


Fig. 5.15

The displacement u must be calculated in three steps :

1) calculate displacements u and θ at the pile head as a function of H as for a pile with "free" head (see Section a) of this chapter)
2) calculate head moment M' at fixed head with the geometric condition that the angle θ must be zero at the fixed head: $\theta(H) - \theta(M') = 0$

   The angle θ due to M' may be calculated with the formulae given for "free" head :

   $$\theta(M') = \theta_o(M') + \theta_1(M')$$

   $$\theta(M') = \beta_M \frac{M' \cdot L_o}{E_p \cdot I_p} + \frac{M' \cdot h}{E_p \cdot I_p} = \theta(H) \text{ ; then } M' = \frac{\theta(H) \cdot E_p \cdot I_p}{\beta_M \cdot L_o + h}$$

   (coefficient β_M from Table 5.10)

3) calculate displacement $u(M')$ due to M' (with formulae given for "free" head)

finally $u = u(H) - u(M')$

$\theta = \theta(H) - \theta(M') = 0$ (geometric condition)

Simplified formula for long piles : $L / L_o > 3$ (assuming fixation of piles at depth L_o) *(Franke (6))*

$$u = \frac{H \cdot (h + L_o)^3}{12\, E_p \cdot I_p} \quad ; \quad M' = H \cdot (h + L_o) / 2$$

Should the pile cap be at the level of the design ground level, the height h of the protruding part has to be set to zero, and above formulae can be used.

c) <u>Fixed Pile Head (rotation at pile head)</u>

For this case, the following approach can be applied:

1) compute the displacements and moments as for the case with fixed pile head and no rotation at pile head
2) distribute the external moment into vertical forces of the piles : $\Delta V = M / a$ (case b3) Fig. 5.10) calculate the differential settlements of the piles due to ΔV and derive the subsequent rotation angle θ of the pile top
3) add the displacements (rotation) at the pile top computed in step 1) and 2)
4) compute the moment M' which is inducing the rotation θ of the pile top as calculated in step 2)
5) add the moments at the pile top computed in step 1) and 4)

As seen in Figure 5.10, the displacements and moments are within the case b1) "free pile head" and case b2) "fixed pile head with no rotation" which can be regarded as extreme values for case b3). Note that the group effect according to Section 5.4.2 has to be taken into account, considering that only half of the pile cap is under relevant pressure and consequently adjusting the widths of the pile group.

5.5.3 Group Effect

If several piles are spaced in the direction of the lateral forces, the displacements increase since the stresses in the soil due to horizontal forces have to be superimposed. This effect can be taken into account by decreasing the coefficients of subgrade reaction, depending on the ratio pile spacing to pile diameter *(DIN 4014/2)* .

$a \leq 3D$: $k_h \text{(group)} = 0.25\, k_h$

$3D < a < 8D$: $k_h \text{(group)} = (0.25 + 0.15 \frac{a - 3D}{D})\, k_h$

(linear interpolation between 3D and 8D)

$a > 8D$: $k_h \text{(group)} = k_h$

where a is the spacing of the piles in the direction of the horizontal force (see Fig. 5.5)

This reduction of the coefficient of subgrade reaction is not as significant as it looks like since only the fourth root enters into the formula of the characteristic length of the pile. The characteristic length for the computation of displacements and moments will therefore only slightly increase for a pile group.

5.5.4 Impact Loads

The soil behaves more stiffly during short term excitation than for long term loads. For impact loads (like ship or truck crashes into piers), the coefficient of subgrade reaction can be slightly increased *(DIN 4014/2)*.

$$k_h \text{ (impact)} = 3\, k_h$$

5.5.5 Permanent Lateral Loads

Computation of displacements so far has been done for non-permanent loads (wind, earthquake, etc.). However, displacements increase under long term loads (e.g. shrinkage, temperature) due to consolidation and creeping of the soil *(Poulos (2))*.

$$k_h \text{ (long term)} = 0.8\, k_h$$

5.5.5 Admissible Horizontal Displacement

They are limited by the stiffness of the superstructure and the type of bearings used. They have to be checked for each project, but for preliminary design, horizontal displacements should generally not exceed 20 mm.

It should be noted that for larger displacements, the soil resistance will be the limiting factor. The horizontal stress in the soil should, in any case, be smaller than the passive earth pressure *(Franke (6))*

$$\sigma_{hs} < k_p \cdot \gamma_s$$

where $\sigma_{hs} = k_h \cdot u$

and $k_p = (1 + \sin \varnothing) / (1 - \sin \varnothing)$ (coefficient of passive earth pressure)

As already mentioned for vertical displacements, the available procedures for computation and the knowledge on soil parameters are limited, so that actual displacements can be significantly higher than computed. The influence of such greater displacements than computed (+ 50 - 100%) on the superstructure shall be checked.

5.6 Moments and Shear Forces in Piles

5.6.1 Moments due to Eccentric and Lateral Loads

As for the computation of displacements, the theory of subgrade reaction is used to calculate bending moments for a single pile. However, the distribution of the coefficient of subgrade reaction depends on the soil properties (see also Chapter 5.5.2). The approach used in this section is based on the distribution of the coefficient of subgrade reaction as given in Figure 5.12 (valid for soil with many layers) and is valid for long piles only (length of piles greater than characteristic length). The procedure will also give good estimates if the soil conditions differ (e.g. piles in soil only). For detailed computation of moments for short piles or different soil conditions, literature shall be consulted *(e.g. Werner (7) or Sherif (8))*.

Moments for pile in soil:

$$M = \alpha \cdot H_o \cdot L_o + \beta \cdot M_o$$

where L_o : characteristic length of pile

M_o, H_o : moments and lateral forces reduced to design ground level

α, β : coefficients according to Table 5.11 depending on pile length (L / L_o)

$$M(z/L) = \alpha(z/L) \cdot H_o \cdot L_o + \beta(z/L) \cdot M_o$$

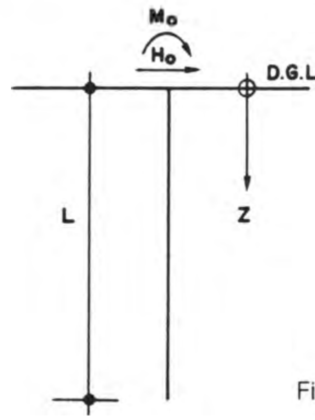


Fig. 5.16

L/L_o	1.0		2.0		3.0		4.0		5.0		6.0		8.0	
z/L	α	β	α	β	α	β	α	β	α	β	α	β	α	β
0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0	0	1.0
0.1	0.10	0.99	0.19	0.98	0.27	0.97	0.34	0.96	0.40	0.93	0.45	0.89	0.52	0.80
0.2	0.16	0.94	0.31	0.92	0.42	0.86	0.48	0.77	0.51	0.67	0.50	0.56	0.44	0.38
0.3	0.20	0.85	0.36	0.80	0.45	0.70	0.45	0.55	0.40	0.39	0.33	0.24	0.18	0.08
0.4	0.20	0.71	0.34	0.65	0.40	0.50	0.33	0.30	0.24	0.14	0.15	0.05	0.03	-0.03
0.5	0.17	0.55	0.29	0.49	0.31	0.33	0.21	0.16	0.12	0.04	0.04	-0.01	0	0
0.6	0.13	0.40	0.23	0.34	0.21	0.20	0.12	0.04	0.04	-0.02	0	-0.03	0	0
0.7	0.09	0.24	0.14	0.20	0.13	0.10	0.06	0.02	0	-0.03	-0.02	-0.03	0	0
0.8	0.05	0.12	0.07	0.10	0.05	0.05	0.01	0	-0.01	-0.01	-0.01	-0.01	0	0
0.9	0.02	0.04	0.02	0.02	0	0.01	0	0	0	0	0	0	0	0
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

derived from Werner (7) Table 5.11

Estimate Formulae

For preliminary design, the following formulae can be used to estimate the maximum moment *(Lang (5))*

a) "free" head: $M_{max} = 0.32 \cdot H_o \cdot L_o$

b) fixed head: $M_{max} = 0.5 \cdot H_o \cdot L_o + M_o$

Protruding Pile

a) "free head" : Moments for protruding part are given in Chapter 5.5.2 (Figure 5.15) and can be added to the moment M computed for the pile in soil.

b) fixed head : As for the computation of displacements, the moments are calculated in several steps (see Figure 5.17) :

1) compute moments and lateral forces due to H for pile in soil :
$M_o = H \cdot h$
$H_o = H$
2) calculate end moment M' for angle $\theta = 0$ at fixed head (see Section 5.5.2) and compute moments in pile due to M'
3) add moments computed in steps 1) and 2)

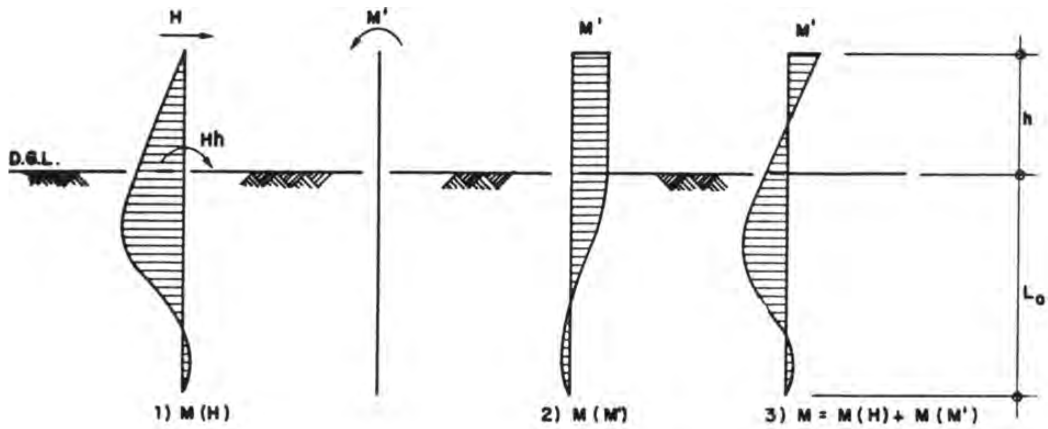


Fig. 5.17

5.6.2. Shear Forces

The shear forces can be computed as the first deviation of the moments. It is therefore convenient to determine the shear force S step by step by computing the differences of the moments as shown in Figure 5.18a. Figures 5.18b and 5.18c show the approximate distribution of shear forces for "free" (hinged) and fixed head. In general, the maximum shear force equals the lateral force H.

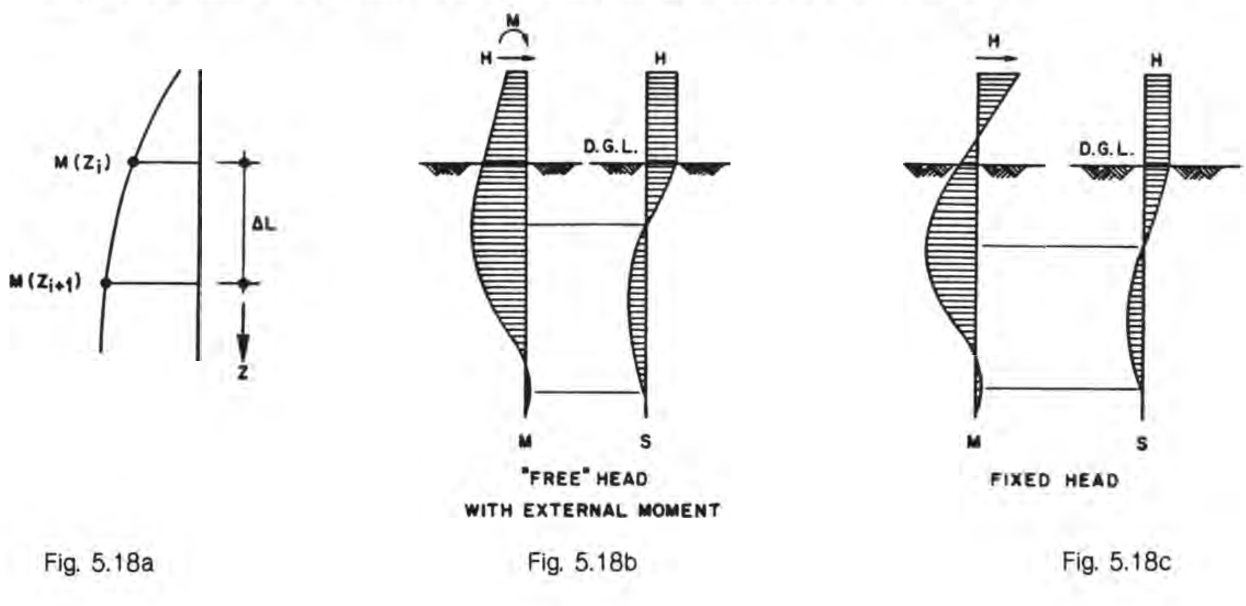


Fig. 5.18a Fig. 5.18b Fig. 5.18c

5.6.3 Group Effect

As for the computation of displacements, the modulus of subgrade reaction has to be reduced for spacings a < 8D in the direction of horizontal load (see Section 5.5.3).

5.6.4 Small Horizontal Load

If the ratio horizontal / vertical load H/V is smaller than 0.03 for main loads or H/V < 0.05 for maximum loads (load cases 2 and 3), then the resulting displacements are small and do not have to be calculated.

5.7 Stresses and Reinforcement

5.7.1 General Rules and Material Properties

Reinforcement crates for bored piles are fabricated on site and placed by crane. It is best to use equal fixed lengths of reinforcement steel for main bars (for example 12 or 6 m).

Some connections of binders/main bars may have to be welded to yield sufficient stiffness of the overall cage. Ensure that the steel specified can be welded.

The cross section area of steel is commonly expressed in relation to the concrete area:

$$\mu = \frac{A_{steel}}{A_{concrete}} = \frac{A_s}{\pi \cdot D^2/4}$$

The proportion of steel reinforcing bars in the concrete section is governed by the following recommended maximum and minimum limits:

Minimum reinforcement for large bored piles :

- main bars : μ_{min} = 0.4 % *(adapted from SIA 192 for large piles)*
- binders : minimum diameter => 12 mm , spacing a < 500 mm
  and a < 250 mm at pile top
  use additional stiffeners if necessary to increase the stability of the crate for lifting and lowering into the bore hole

Maximum reinforcement : μ_{max} = 6 % *(IS 2911 or SIA 162)*
(normally not relevant for large diameter piles)

Concrete cover : > 50 mm for section with standpipe
> 150 mm for section in soil without casing

Overlapping length for bars without hook : > 45 diameters *(SIA 162)*
> 60 diameters at pile head (tension due to bending)

Since the reinforcement bars will be concentrated when overlapping, check that there is enough space for the distribution of concrete during casting.The materials (reinforcement steel, concrete) which will be used shall be clearly marked on the drawings and the material strengths tested regularly during construction (see also the chapter on Bentonite Piling).

a) Material Strengths

The following values are generally used as the basis for the computation of the bearing capacity of concrete structures:

- Concrete : The minimum ultimate compressive strength $f_{c,min}$ (28 days cube or cylinder test). Note that $f_{c,min}$ is a minimum value (not an average value). This means that the strengths of tested concrete must be higher than this specified value. (see chapter on Concrete Tests of Bentonite Piling)
  Statistically $f_{c,min}$ represents the 2% fraction of the tested values.
- Steel : The yield stress f_y guaranteed by the manafacturer is taken as basis for computation.

b) Admissible Stresses

Construction codes of different countries indicate varying admissible stresses or do not give any value for bored piles at all. The values given below represent a conservative average of existing values (German, Swiss code etc.). Admissible stresses are given as fractions of the material strengths and shall not be exceeded by the maximum computed stresses under working loads.

$$\sigma_{c,max} < \sigma_{c,adm}$$

$$\sigma_{st,max} < \sigma_{st,adm}$$

Since concrete for piles is cast without vibrating and tolerances, e.g. for placing of reinforcement, are larger than for building construction, the admissible stresses for bored piles are generally lower than the ones given for standard concrete structures *(SIA 192: 30% reduction for axial stress, 45% reduction for bending stress)* .

- Concrete
  - admissible axial compressive stress: $\sigma_{c,adm} = 0.2\ f_{c,min}$
    - for $f_{c,min} = 25$ N/mm^2 ; $\sigma_{c,adm} = 5$ N/mm^2
    - for $f_{c,min} = 20$ N/mm^2 ; $\sigma_{c,adm} = 4$ N/mm^2
  - admissible compressive stress for bending:
    1) regular load: $\sigma_{c,adm} = 0.25\ f_{c,min}$
       - for $f_{c,min} = 25$ N/mm^2 ; $\sigma_{c,adm} = 6.25$ N/mm^2
       - for $f_{c,min} = 20$ N/mm^2 ; $\sigma_{c,adm} = 5$ N/mm^2
    2) exceptional load (incl. either earthquake or impact) : $\sigma_{c,adm} = 0.35\ f_{c,min}$
       - for $f_{c,min} = 25$ N/mm^2 ; $\sigma_{c,adm} = 8.75$ N/mm^2
       - for $f_{c,min} = 20$ N/mm^2 ; $\sigma_{c,adm} = 7$ N/mm^2
  - admissible shear stress (with shear reinforcement) : $\tau_{c,adm} = 0.1\ f_{c,min}$
    - for $f_{c,min} = 25$ N/mm^2 ; $\tau_{c,adm} = 2.5$ N/mm^2
    - for $f_{c,min} = 20$ N/mm^2 ; $\tau_{c,adm} = 2$ N/mm^2

The values of these admissible concrete stresses can be increased if the pile dimensions and quality are tested by an appropriate method (e.g. ultra-sonic sounding) and if measures are taken in case of insufficient test results (injections, additional piles etc.) to reach the specified design assumptions. In this case, above values can be increased by up to 30% to take into account the higher certitude pile quality.

- Reinforcement Steel
  - admissible tensile or compression stress:
    1) regular load: $\sigma_{st,adm} = 0.6\, f_y$
       for SD-24: $f_y = 240\ N/mm^2$; $\sigma_{st,adm} = 140\ N/mm^2$
       for SD-30: $f_y = 300\ N/mm^2$; $\sigma_{st,adm} = 180\ N/mm^2$
    2) exceptional load (incl. either earthquake or impact) : $\sigma_{st,adm} = 0.75\, f_y$
       for SD-24: $f_y = 240\ N/mm^2$; $\sigma_{st,adm} = 180\ N/mm^2$
       for SD-30: $f_y = 300\ N/mm^2$; $\sigma_{st,adm} = 225\ N/mm^2$

c) Limit State Computational Values

Values generally used for the computation of the bearing capacity by the limit state method are given below *(derived from SIA 162)*. These values are only of indicative nature since the limit state method is not treated any further in this manual. Should this method be used to determine the necessary reinforcement, the stresses in concrete and steel should also be checked by the linear-elastic method (as described in the following chapter) to guarantee the serviceability of the piles (limitation of cracks).

- concrete: $f_c = 0.7 \cdot 0.65\ f_{c,min} = 0.45\ f_{c,min}$

  The factor 0.7 accounts for the lower concrete quality and higher tolerances in piles compared to standard concrete structures *(SIA 192)*. As indicated above, the value of f_c can be increased by up to 30% if the pile dimensions and quality are tested.

- steel: f_y is the minimum yield stress specified by the manufacturer (see above).

The following safety factors are generally used when comparing the actual forces to the ultimate bearing resistance computed with the limit state method *(SIA 162 RL 34)*:

1) dead + live load: s = 1.8
2) dead + live + wind load: s = 1.5
3) dead + exceptional loads (earthquake or impact): s = 1.2

d) Elasticity of Materials

Young's moduli:

- steel: $E_{st} = 2.1 \cdot 10^5\ N/mm^2$
- concrete: $E_c = 2.1 \cdot 10^5\ N/mm^2$
- ratio: $n = E_{st} / E_c = 10$

5.7.2. Required Reinforcement for Bending

Core of the Section

Eccentricity of vertical force : $e = M/V$
The core of the section is defined as the part of the section where vertical forces do not create tensile stress and can be expressed as follows:

stress in point B: $\sigma(B) = \frac{V}{A_p} - \frac{e \cdot V}{I_p} \cdot D/2$

for $\sigma(B) = 0$; $\frac{V}{1/A_p - e \cdot D/2\,I_p} = 0$

then $e_o = \frac{2 \cdot I_p}{A_p \cdot D}$

for circular sections:

$A_p = \frac{\pi \cdot D^2}{4}$ and $I_p = \frac{\pi \cdot D^4}{64}$

then $e = \frac{2 \cdot \pi D^4 / 64}{\pi D^2 / 4D} = D/8$

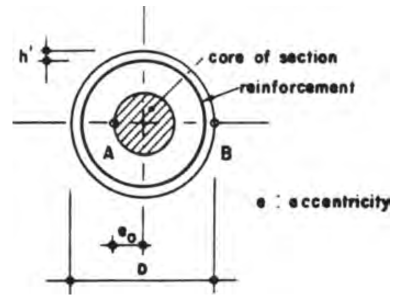


Fig. 5.19

The following two cases are handled seperately:
a) small eccentricity: compressive stress over whole section $e/_R < 0.30$
b) great eccentricity: tensile stress due to bending $e/_R > 0.30$

To simplify the formulae, the expressions of the "combined area" A' and "combined inertia" I' are used:

$A' = A_c + n \cdot A_{st}$

$I' = I_c + n \cdot I_{st}$

with $I_{st} = A_{st} / 4 \cdot (D/2 - h')^2$

$I_c = I_p = \pi \cdot D^4 / 64$

a) Small Eccentricity
(e < D / 8)

concrete: $\sigma_{c,max} = \frac{V}{A'} + \frac{V \cdot e}{I'} \cdot D/2$

steel: $\sigma_{st,max} = \frac{D/2 - h'}{D/2} \cdot n \cdot \sigma_{c,max}$

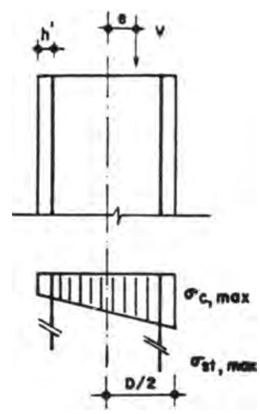


Fig. 5.20

In this case, the minimum required reinforcement normally is sufficient, which has to be confirmed by computing the stresses.

b) Great Eccentricity
(e › D / 8)

Internal forces applied at centre of gravity of respective cross section:

C_c : compression capacity concrete
C_{st} : reinforcement under compression
T_{st} : reinforcement under tension

equilibrium conditions:

$V = C_c + C_{st} - T_{st}$
$M = e \cdot V = C_c \cdot y + C_{st} \cdot y'$

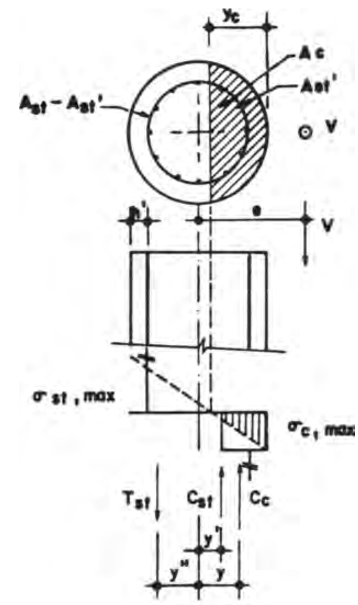


Fig. 5.21

In principle, the allowable eccentricity (or moment) for a given axial force and a given reinforcement can be calculated using above conditions on an iterative basis (estimating the neutral line). For speedy computation of stresses, an excerpt of tables for circular concrete sections *(from K. Hofacker: "Stahlbetontabellen", Leemann 1971)* is presented on the next pages with following designations:

$\sigma_c = c \cdot 4\,V / D^2$; $\sigma_{st} = \gamma \cdot \sigma_c$

The parameters c and γ are given for various ratios of e/R (where e is the eccentricity of the axial force e = M / V and R is the radius R = D/2) and ratios of 2h'/D .
Intermediate values can be interpolated.

Table 5.12 (on next pages):

Factors for computation of stresses in concrete and reinforcement for circular sections with evenly spaced reinforcement by linear- elastic method (see also Fig. 5.19):

a) 2h'/D = 0.90
b) 2h'/D = 0.85
c) 2h'/D = 0.80
d) 2h'/D = 0.75

where h' is the distance from pile shaft to reinforcement (centre of gravity of main bars)
and D is the pile diameter

Values between the parameters indicated can be interpolated linearly.

For calculation of reinforcement based on the limit state method, diagrams are given in Table 5.13 (from C. Menn "Stahlbetonbrücken", Springer 1986).

Table 5.12: Tables for Stress Calculation (from K. Hofacker)

μ%	f/R	0,30	0,40	0,50	0,60	0,70	0,80	0,90	1,00	1,20	1,40	1,60	1,80	2,00	3,00	4,00	5,00	6,00	8,00	10,0	20,0	30,0	40,0	50,0	∞	f/R	μ%
0,40	c	0,667	0,819	1,02	1,29	1,62	1,98	2,36	2,75	3,52	4,28	5,04	5,78	6,52	10,17	13,79	17,39	20,98	28,16	35,33	71,13	106,9	142,7	178,5	3,578	c	0,40
	y	0,30	2,26	4,57	7,20	9,94	12,52	14,80	16,75	19,84	22,12	23,87	25,25	26,37	29,78	31,53	32,60	33,32	34,22	34,77	35,88	36,17	36,37	36,55	37,02	y	
0,50	c	0,658	0,806	1,00	1,25	1,55	1,87	2,21	2,55	3,23	3,91	4,57	5,23	5,89	9,13	12,35	15,55	18,74	25,12	31,50	63,34	95,16	127,0	158,8	3,188	c	0,50
	y	0,27	2,20	4,42	6,89	9,38	11,69	13,71	15,45	18,20	20,24	21,82	23,07	24,08	27,18	28,78	29,76	30,42	31,25	31,75	32,77	33,05	33,24	33,39	33,82	y	
0,60	c	0,650	0,794	0,982	1,22	1,49	1,79	2,09	2,40	3,01	3,62	4,23	4,83	5,42	8,37	11,28	14,19	17,09	22,89	28,68	57,62	86,54	115,5	144,4	2,892	c	0,60
	y	0,24	2,14	4,29	6,62	8,92	11,03	12,87	14,45	16,95	18,83	20,28	21,43	22,36	25,24	26,73	27,64	28,25	29,03	29,50	30,45	30,78	30,96	31,03	31,42	y	
0,70	c	0,642	0,782	0,963	1,19	1,44	1,71	1,99	2,27	2,84	3,40	3,95	4,51	5,05	7,77	10,46	13,14	15,81	21,16	26,50	53,18	79,85	106,5	133,2	2,668	c	0,70
	y	0,21	2,08	4,17	6,38	8,54	10,49	12,19	13,65	15,97	17,71	19,06	20,14	21,01	23,71	25,11	25,97	26,55	27,28	27,73	28,63	29,03	29,18	29,18	29,55	y	
0,80	c	0,634	0,771	0,946	1,16	1,40	1,65	1,91	2,17	2,70	3,22	3,73	4,25	4,76	7,28	9,79	12,29	14,78	19,76	24,74	49,60	74,47	99,33	124,2	2,486	c	0,80
	y	0,18	2,03	4,06	6,18	8,21	10,04	11,63	12,99	15,16	16,80	18,07	19,08	19,91	22,47	23,80	24,61	25,17	25,86	26,29	27,15	27,49	27,62	27,67	28,03	y	
0,90	c	0,627	0,760	0,929	1,13	1,36	1,59	1,84	2,08	2,58	3,06	3,55	4,03	4,51	6,89	9,23	11,58	13,92	18,60	23,28	46,64	69,99	93,35	116,7	2,336	c	0,90
	y	0,16	1,98	3,96	5,99	7,92	9,65	11,15	12,43	14,49	16,04	17,25	18,21	18,99	21,43	22,70	23,49	24,01	24,68	25,09	25,91	26,23	26,35	26,42	26,76	y	
1,00	c	0,619	0,750	0,913	1,11	1,32	1,55	1,78	2,01	2,47	2,93	3,39	3,84	4,29	6,54	8,76	10,98	13,19	17,62	22,04	44,13	66,22	88,31	110,4	2,209	c	1,00
	y	0,13	1,93	3,86	5,82	7,67	9,31	10,74	11,96	13,91	15,39	16,54	17,46	18,21	20,55	21,77	22,53	23,03	23,68	24,07	24,86	25,17	25,29	25,35	25,68	y	
1,10	c	0,612	0,740	0,898	1,08	1,29	1,50	1,72	1,94	2,38	2,82	3,25	3,68	4,11	6,24	8,35	10,46	12,57	16,77	20,97	41,97	62,95	83,92	104,9	2,109	c	1,10
	y	0,11	1,89	3,78	5,67	7,44	9,01	10,37	11,54	13,41	14,83	15,93	16,81	17,54	19,79	20,97	21,70	22,19	22,81	23,19	23,96	24,29	24,40	24,43	24,75	y	
1,20	c	0,605	0,730	0,884	1,06	1,26	1,46	1,67	1,88	2,30	2,71	3,13	3,54	3,95	5,98	8,00	10,01	12,02	16,03	20,04	40,08	60,12	80,16	100,2	2,004	c	1,20
	y	0,08	1,84	3,69	5,53	7,24	8,75	10,05	11,17	12,97	14,33	15,40	16,25	16,95	19,13	20,27	20,97	21,45	22,05	22,42	23,17	23,46	23,57	23,62	23,94	y	
1,30	c	0,598	0,721	0,870	1,04	1,23	1,43	1,63	1,83	2,23	2,62	3,02	3,41	3,80	5,75	7,68	9,61	11,53	15,37	19,21	38,41	57,59	76,78	95,96	1,915	c	1,30
	y	0,06	1,80	3,62	5,41	7,06	8,51	9,77	10,84	12,58	13,89	14,92	15,75	16,42	18,54	19,65	20,33	20,79	21,38	21,74	22,47	22,79	22,89	22,91	23,22	y	
1,40	c	0,591	0,712	0,857	1,02	1,21	1,39	1,58	1,78	2,16	2,54	2,92	3,30	3,67	5,54	7,40	9,25	11,09	14,79	18,48	36,92	55,35	73,78	92,21	1,843	c	1,40
	y	0,03	1,76	3,55	5,29	6,89	8,29	9,51	10,55	12,23	13,50	14,50	15,30	15,96	18,01	19,09	19,76	20,21	20,78	21,13	21,84	22,19	22,29	22,28	22,57	y	
1,50	c	0,585	0,703	0,845	1,01	1,18	1,36	1,55	1,73	2,10	2,47	2,83	3,19	3,56	5,35	7,14	8,92	10,70	14,26	17,81	35,57	53,32	71,08	88,83	1,776	c	1,50
	y	0,01	1,72	3,48	5,18	6,73	8,10	9,27	10,28	11,91	13,15	14,12	14,90	15,53	17,54	18,59	19,24	19,68	20,25	20,59	21,28	21,55	21,65	21,71	22,00	y	
1,60	c	0,578	0,694	0,833	0,990	1,16	1,33	1,51	1,69	2,04	2,40	2,75	3,10	3,45	5,18	6,91	8,63	10,35	13,78	17,21	34,35	51,49	68,62	85,76	1,714	c	1,60
	y		1,69	3,41	5,08	6,59	7,92	9,06	10,04	11,62	12,82	13,77	14,53	15,15	17,11	18,14	18,78	19,21	19,76	20,09	20,77	21,07	21,16	21,19	21,47	y	
1,70	c	0,572	0,686	0,821	0,974	1,14	1,31	1,48	1,65	1,99	2,33	2,67	3,01	3,35	5,03	6,70	8,36	10,02	13,34	16,66	33,24	49,81	66,39	82,96	1,658	c	1,70
	y		1,65	3,35	4,98	6,46	7,75	8,86	9,82	11,36	12,53	13,45	14,20	14,80	16,72	17,73	18,35	18,78	19,31	19,64	20,31	20,60	20,69	20,72	21,00	y	
1,80	c	0,566	0,678	0,810	0,958	1,12	1,28	1,45	1,61	1,95	2,28	2,61	2,93	3,26	4,88	6,50	8,11	9,72	12,94	16,15	32,22	48,28	64,33	80,39	1,606	c	1,80
	y		1,62	3,30	4,89	6,34	7,60	8,68	9,61	11,11	12,26	13,16	13,89	14,49	16,36	17,35	17,96	18,38	18,91	19,23	19,89	20,19	20,28	20,29	20,56	y	
1,90	c	0,560	0,670	0,799	0,944	1,10	1,26	1,42	1,58	1,90	2,22	2,54	2,86	3,18	4,75	6,32	7,88	9,45	12,57	15,69	31,27	46,85	62,44	78,02	1,559	c	1,90
	y		1,59	3,24	4,81	6,22	7,45	8,51	9,42	10,89	12,01	12,90	13,61	14,19	16,03	17,00	17,61	18,01	18,53	18,85	19,50	19,78	19,87	19,89	20,16	y	
2,00	c	0,554	0,662	0,789	0,930	1,08	1,23	1,39	1,55	1,86	2,17	2,48	2,79	3,10	4,63	6,15	7,67	9,19	12,22	15,25	30,40	45,54	60,69	75,83	1,514	c	2,00
	y		1,56	3,19	4,73	6,12	7,32	8,36	9,25	10,68	11,78	12,65	13,35	13,92	15,73	16,68	17,27	17,68	18,19	18,50	19,13	19,38	19,46	19,52	19,79	y	
2,20	c	0,543	0,647	0,769	0,903	1,05	1,19	1,34	1,49	1,79	2,08	2,37	2,66	2,96	4,41	5,85	7,29	8,73	11,60	14,47	28,83	43,18	57,53	71,88	1,436	c	2,20
	y		1,50	3,09	4,59	5,92	7,08	8,07	8,93	10,31	11,37	12,21	12,88	13,43	15,18	16,11	16,68	17,07	17,57	17,87	18,49	18,72	18,80	18,87	19,13	y	
2,40	c	0,532	0,633	0,750	0,879	1,01	1,15	1,29	1,43	1,72	2,00	2,27	2,55	2,83	4,21	5,58	6,95	8,32	11,06	13,79	27,45	41,11	54,76	68,42	1,366	c	2,40
	y		1,44	3,00	4,45	5,74	6,86	7,82	8,65	9,99	11,01	11,82	12,47	13,01	14,71	15,61	16,17	16,55	17,03	17,33	17,93	18,15	18,23	18,30	18,55	y	
2,60	c	0,522	0,620	0,733	0,856	0,985	1,12	1,25	1,39	1,65	1,92	2,19	2,45	2,72	4,03	5,35	6,65	7,96	10,57	13,18	26,23	39,27	52,32	65,36	1,305	c	2,60
	y		1,39	2,92	4,34	5,59	6,67	7,60	8,40	9,70	10,69	11,48	12,11	12,64	14,29	15,17	15,71	16,08	16,56	16,85	17,43	17,65	17,73	17,79	18,04	y	
2,80	c	0,512	0,607	0,716	0,834	0,959	1,09	1,21	1,34	1,60	1,85	2,11	2,36	2,62	3,88	5,13	6,39	7,64	10,14	12,64	25,13	37,62	50,12	62,61	1,240	c	2,80
	y		1,34	2,84	4,23	5,45	6,50	7,40	8,18	9,44	10,41	11,17	11,79	12,30	13,92	14,78	15,31	15,67	16,14	16,42	16,99	17,23	17,30	17,35	17,59	y	
3,00	c	0,502	0,595	0,700	0,814	0,934	1,06	1,18	1,30	1,55	1,79	2,04	2,28	2,52	3,73	4,94	6,14	7,34	9,75	12,15	24,15	36,15	48,14	60,14	1,200	c	3,00
	y		1,29	2,77	4,13	5,32	6,34	7,22	7,98	9,21	10,16	10,90	11,51	12,00	13,59	14,43	14,95	15,30	15,76	16,04	16,60	16,82	16,89	16,95	17,19	y	

a) 2h'/D = 0.90 (above)

b) 2h'/D = 0.85 (below)

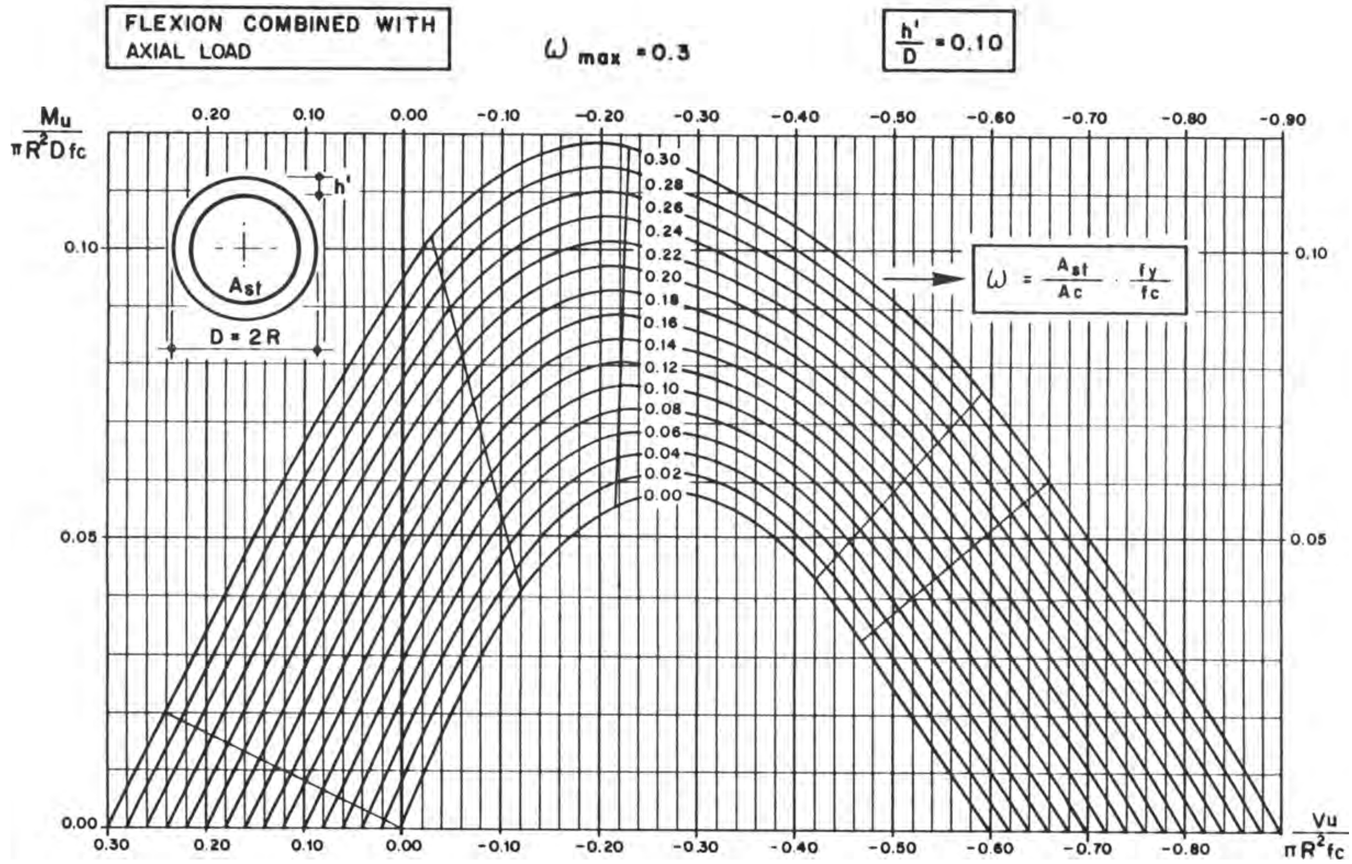
μ%	f/R	0,30	0,40	0,50	0,60	0,70	0,80	0,90	1,00	1,20	1,40	1,60	1,80	2,00	3,00	4,00	5,00	6,00	8,00	10,0	20,0	30,0	40,0	50,0	∞	f/R	μ%
0,40	c	0,670	0,824	1,03	1,31	1,65	2,03	2,43	2,84	3,64	4,44	5,23	6,01	6,79	10,62	14,42	18,20	21,98	29,52	37,05	74,69	112,3	149,9	187,5	3,761	c	0,40
	y	0,07	2,00	4,29	6,93	9,67	12,25	14,51	16,43	19,45	21,67	23,36	24,69	25,75	29,00	30,66	31,66	32,33	33,18	33,69	34,72	35,05	35,27	35,35	35,78	y	
0,50	c	0,662	0,812	1,01	1,27	1,58	1,92	2,28	2,63	3,35	4,06	4,76	5,45	6,14	9,55	12,93	16,30	19,67	26,38	33,10	66,63	100,2	133,7	167,2	3,351	c	0,50
	y	0,04	1,95	4,17	6,64	9,13	11,44	13,44	15,15	17,84	19,83	21,35	22,54	23,51	26,46	27,97	28,89	29,50	30,28	30,75	31,70	32,10	32,29	32,27	32,67	y	
0,60	c	0,654	0,801	0,995	1,24	1,53	1,84	2,16	2,48	3,13	3,77	4,40	5,03	5,66	8,76	11,84	14,90	17,96	24,08	30,18	60,70	91,20	121,7	152,2	3,051	c	0,60
	y	0,02	1,90	4,05	6,39	8,70	10,80	12,62	14,17	16,62	18,44	19,83	20,94	21,83	24,56	25,96	26,81	27,39	28,11	28,55	29,43	29,75	29,93	29,97	30,33	y	
0,70	c	0,646	0,790	0,978	1,21	1,48	1,76	2,06	2,36	2,95	3,54	4,12	4,71	5,28	8,14	10,98	13,81	16,64	22,29	27,93	56,11	84,27	112,4	140,6	2,818	c	0,70
	y	0,00	1,86	3,95	6,17	8,33	10,28	11,96	13,39	15,66	17,35	18,64	19,67	20,50	23,06	24,36	25,18	25,73	26,41	26,82	27,66	27,95	28,11	28,16	28,51	y	
0,80	c	0,639	0,780	0,961	1,18	1,43	1,70	1,98	2,25	2,81	3,35	3,90	4,44	4,98	7,65	10,29	12,93	15,57	20,84	26,10	52,41	78,71	105,0	131,3	2,630	c	0,80
	y		1,82	3,85	5,98	8,02	9,84	11,41	12,75	14,87	16,45	17,67	18,64	19,43	21,85	23,10	23,86	24,38	25,03	25,42	26,21	26,49	26,64	26,70	27,03	y	
0,90	c	0,632	0,769	0,945	1,16	1,40	1,65	1,90	2,16	2,68	3,20	3,71	4,22	4,72	7,23	9,72	12,21	14,69	19,64	24,59	49,34	74,06	98,78	123,5	2,474	c	0,90
	y		1,78	3,76	5,81	7,74	9,46	10,94	12,21	14,21	15,71	16,86	17,78	18,53	20,84	22,03	22,76	23,25	23,87	24,25	25,01	25,28	25,43	25,48	25,79	y	
1,00	c	0,624	0,760	0,930	1,13	1,36	1,60	1,84	2,09	2,58	3,06	3,55	4,03	4,50	6,88	9,23	11,59	13,93	18,63	23,31	46,74	70,16	93,58	117,0	2,342	c	1,00
	y		1,74	3,68	5,65	7,50	9,14	10,54	11,74	13,64	15,07	16,17	17,05	17,77	19,98	21,12	21,82	22,30	22,89	23,26	23,99	24,24	24,39	24,44	24,74	y	
1,10	c	0,618	0,750	0,916	1,11	1,33	1,55	1,79	2,02	2,48	2,94	3,40	3,86	4,31	6,57	8,81	11,05	13,28	17,75	22,21	44,49	66,76	89,03	111,3	2,228	c	1,10
	y		1,70	3,60	5,51	7,29	8,85	10,19	11,33	13,15	14,52	15,58	16,42	17,11	19,23	20,34	21,01	21,47	22,05	22,40	23,11	23,37	23,50	23,54	23,83	y	
1,20	c	0,611	0,741	0,902	1,09	1,30	1,52	1,74	1,96	2,40	2,84	3,28	3,71	4,15	6,30	8,45	10,58	12,72	16,98	21,24	42,53	63,82	85,11	106,4	2,129	c	1,20
	y		1,67	3,53	5,39	7,09	8,59	9,88	10,97	12,72	14,03	15,05	15,86	16,53	18,58	19,65	20,31	20,75	21,31	21,65	22,34	22,58	22,71	22,76	23,04	y	
1,30	c	0,604	0,732	0,889	1,07	1,27	1,48	1,69	1,90	2,33	2,75	3,17	3,58	4,00	6,07	8,12	10,17	12,21	16,30	20,39	40,80	61,20	81,60	102,0	2,037	c	1,30
	y		1,63	3,46	5,27	6,92	8,36	9,60	10,65	12,34	13,60	14,59	15,37	16,01	18,01	19,04	19,68	20,11	20,65	20,99	21,66	21,89	22,00	22,06	22,34	y	
1,40	c	0,598	0,723	0,877	1,05	1,25	1,45	1,65	1,85	2,26	2,66	3,07	3,47	3,87	5,85	7,83	9,80	11,76	15,69	19,62	39,25	58,87	78,50	98,12	1,963	c	1,40
	y		1,60	3,40	5,16	6,76	8,15	9,35	10,37	11,99	13,22	14,17	14,94	15,56	17,49	18,50	19,12	19,54	20,07	20,39	21,05	21,30	21,41	21,44	21,71	y	
1,50	c	0,592	0,715	0,865	1,04	1,22	1,41	1,61	1,81	2,20	2,59	2,97	3,36	3,75	5,66	7,56	9,46	11,36	15,15	18,93	37,86	56,78	75,69	94,61	1,892	c	1,50
	y		1,57	3,34	5,06	6,61	7,96	9,12	10,11	11,68	12,87	13,80	14,54	15,15	17,03	18,02	18,62	19,03	19,55	19,86	20,50	20,75	20,86	20,89	21,16	y	
1,60	c	0,585	0,707	0,853	1,02	1,20	1,39	1,57	1,76	2,14	2,52	2,89	3,26	3,64	5,48	7,32	9,16	10,99	14,65	18,31	36,59	54,87	73,14	91,42	1,828	c	1,60
	y		1,54	3,29	4,96	6,48	7,79	8,91	9,87	11,40	12,56	13,46	14,18	14,77	16,61	17,57	18,16	18,56	19,07	19,38	20,00	20,23	20,34	20,38	20,65	y	
1,70	c	0,579	0,699	0,842	1,00	1,18	1,36	1,54	1,72	2,09	2,45	2,81	3,18	3,53	5,32	7,10	8,88	10,65	14,19	17,73	35,43	53,12	70,82	88,51	1,770	c	1,70
	y		1,51	3,23	4,87	6,35	7,63	8,72	9,65	11,14	12,27	13,15	13,86	14,43	16,23	17,17	17,75	18,14	18,64	18,94	19,55	19,77	19,87	19,93	20,18	y	
1,80	c	0,574	0,691	0,831	0,989	1,16	1,33	1,51	1,69	2,04	2,39	2,74	3,09	3,44	5,17	6,90	8,62	10,34	13,77	17,21	34,37	51,52	68,68	85,83	1,716	c	1,80
	y		1,49	3,18	4,79	6,23	7,48	8,54	9,45	10,91	12,01	12,87	13,56	14,12	15,88	16,80	17,37	17,75	18,24	18,54	19,14	19,36	19,46	19,51	19,76	y	
1,90	c	0,568	0,683	0,820	0,974	1,14	1,31	1,48	1,65	1,99	2,34	2,68	3,02	3,35	5,04	6,71	8,38	10,05	13,39	16,72	33,39	50,05	66,70	83,36	1,666	c	1,90
	y		1,46	3,13	4,71	6,12	7,34	8,38	9,27	10,69	11,77	12,61	13,28	13,83	15,56	16,46	17,02	17,40	17,88	18,17	18,76	18,97	19,07	19,12	19,37	y	
2,00	c	0,562	0,676	0,810	0,960	1,12	1,28	1,45	1,62	1,95	2,28	2,62	2,94	3,27	4,91	6,54	8,16	9,79	13,03	16,27	32,48	48,68	64,88	81,08	1,620	c	2,00
	y		1,44	3,09	4,64	6,02	7,21	8,23	9,10	10,49	11,54	12,37	13,03	13,57	15,26	16,15	16,70	17,07	17,54	17,83	18,41	18,60	18,70	18,76	19,00	y	
2,20	c	0,551	0,661	0,791	0,934	1,09	1,24	1,40	1,56	1,87	2,19	2,50	2,82	3,13	4,68	6,22	7,77	9,31	12,38	15,46	30,84	46,21	61,59	76,96	1,538	c	2,20
	y		1,39	3,00	4,50	5,83	6,98	7,96	8,79	10,12	11,14	11,93	12,57	13,09	14,73	15,59	16,12	16,48	16,94	17,22	17,78	17,98	18,08	18,12	18,36	y	
2,40	c	0,541	0,648	0,772	0,910	1,06	1,20	1,35	1,50	1,80	2,10	2,40	2,70	3,00	4,48	5,95	7,42	8,88	11,82	14,75	29,40	44,05	58,70	73,35	1,465	c	2,40
	y		1,34	2,92	4,38	5,67	6,77	7,71	8,52	9,81	10,79	11,56	12,17	12,68	14,27	15,10	15,62	15,97	16,41	16,68	17,23	17,41	17,51	17,57	17,80	y	
2,60	c	0,531	0,635	0,755	0,887	1,03	1,17	1,31	1,45	1,74	2,03	2,31	2,60	2,88	4,29	5,70	7,11	8,51	11,31	14,12	28,12	42,13	56,13	70,14	1,401	c	2,60
	y		1,30	2,85	4,27	5,52	6,59	7,50	8,28	9,53	10,48	11,22	11,82	12,31	13,86	14,67	15,18	15,52	15,95	16,21	16,75	16,92	17,02	17,08	17,30	y	
2,80	c	0,521	0,622	0,739	0,866	0,999	1,14	1,27	1,41	1,69	1,96	2,23	2,51	2,78	4,13	5,48	6,83	8,17	10,86	13,55	26,98	40,41	53,84	67,27	1,344	c	2,80
	y		1,26	2,78	4,17	5,38	6,42	7,31	8,06	9,28	10,20	10,93	11,51	11,99	13,50	14,29	14,78	15,12	15,54	15,80	16,32	16,48	16,58	16,64	16,86	y	
3,00	c	0,512	0,610	0,723	0,846	0,974	1,10	1,24	1,37	1,63	1,90	2,16	2,42	2,68	3,98	5,28	6,57	7,87	10,45	13,03	25,95	38,86	51,77	64,68	1,292	c	3,00
	y		1,22	2,72	4,08	5,26	6,27	7,13	7,87	9,05	9,95	10,64	11,23	11,70	13,17	13,95	14,43	14,76	15,17	15,43	15,94	16,10	16,20	16,26	16,47	y	

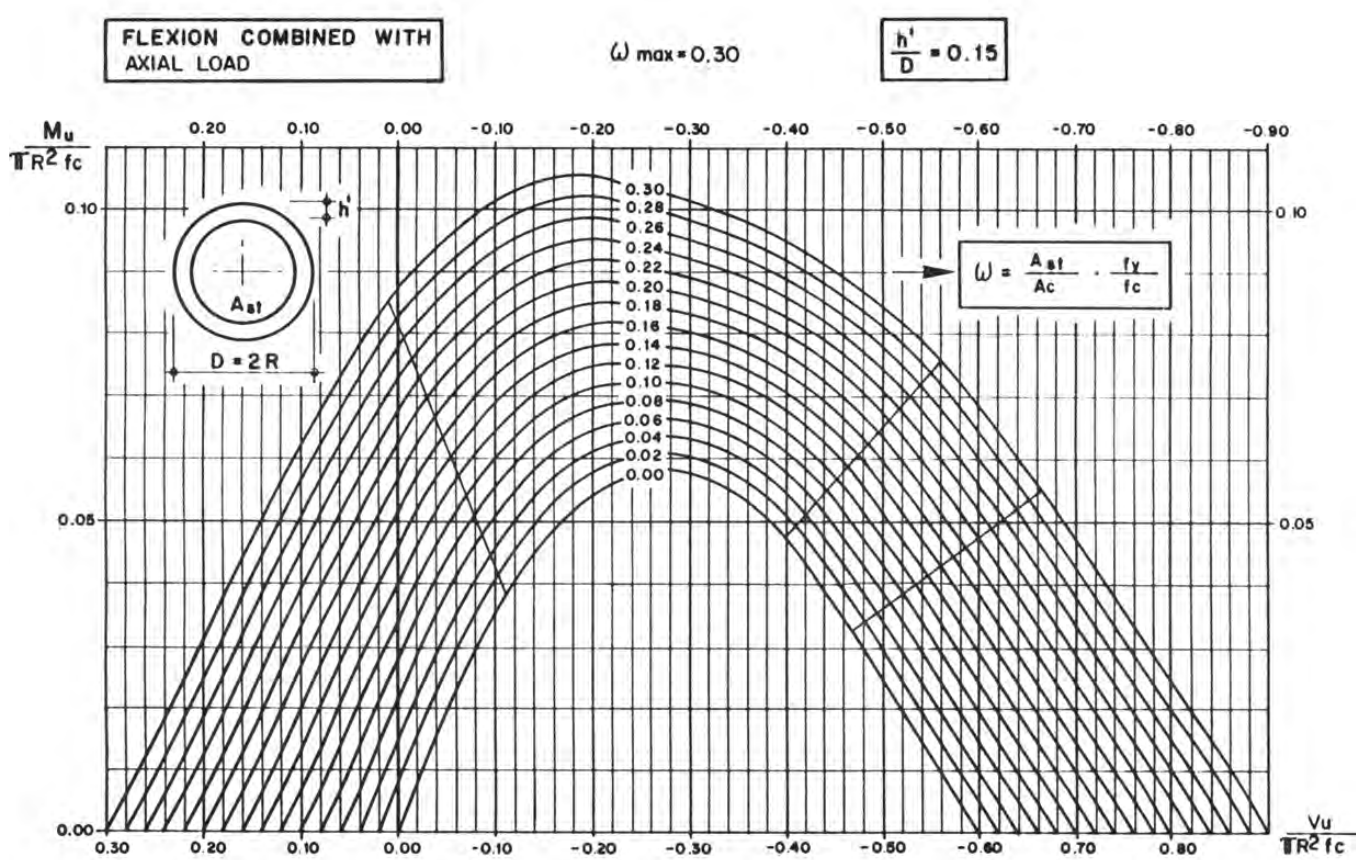
μ%	f/R	0,30	0,40	0,50	0,60	0,70	0,80	0,90	1,00	1,20	1,40	1,60	1,80	2,00	3,00	4,00	5,00	6,00	8,00	10,0	20,0	30,0	40,0	50,0	∞	f/R	μ%
0,40	c	0,672	0,829	1,04	1,33	1,69	2,08	2,50	2,92	3,77	4,60	5,43	6,25	7,06	11,08	15,07	19,04	23,01	30,93	38,85	78,39	117,9	157,5	197,0	3,952	c	0,40
	y		1,73	4,01	6,64	9,38	11,95	14,19	16,09	19,04	21,19	22,82	24,09	25,11	28,20	29,76	30,70	31,33	32,12	32,60	33,56	34,00	34,13	34,14	34,54	y	
0,50	c	0,665	0,818	1,03	1,30	1,62	1,98	2,35	2,72	3,47	4,21	4,95	5,68	6,40	9,99	13,54	17,09	20,63	27,70	34,76	70,06	105,3	140,6	175,9	3,528	c	0,50
	y		1,70	3,90	6,37	8,88	11,17	13,16	14,84	17,46	19,38	20,84	21,99	22,91	25,71	27,14	28,00	28,57	29,30	29,73	30,61	31,00	31,13	31,15	31,51	y	
0,60	c	0,657	0,808	1,01	1,26	1,56	1,89	2,23	2,57	3,25	3,92	4,59	5,25	5,91	9,17	12,41	15,65	18,87	25,32	31,76	63,94	96,09	128,2	160,4	3,217	c	0,60
	y		1,66	3,81	6,15	8,46	10,55	12,35	13,88	16,27	18,02	19,36	20,42	21,27	23,86	25,17	25,97	26,51	27,18	27,59	28,41	28,80	28,93	28,91	29,24	y	
0,70	c	0,650	0,798	0,992	1,24	1,52	1,82	2,13	2,44	3,07	3,69	4,30	4,91	5,52	8,54	11,54	14,53	17,51	23,47	29,43	59,19	88,96	118,7	148,5	2,977	c	0,70
	y		1,63	3,72	5,95	8,11	10,05	11,71	13,12	15,32	16,95	18,20	19,18	19,97	22,40	23,63	24,38	24,89	25,52	25,91	26,68	27,01	27,14	27,15	27,46	y	
0,80	c	0,643	0,788	0,977	1,21	1,47	1,76	2,04	2,34	2,92	3,50	4,07	4,64	5,21	8,03	10,83	13,62	16,41	21,98	27,55	55,36	83,17	111,0	138,8	2,781	c	0,80
	y		1,60	3,64	5,77	7,81	9,63	11,18	12,49	14,55	16,08	17,25	18,17	18,92	21,21	22,38	23,09	23,57	24,18	24,54	25,28	25,59	25,72	25,72	26,03	y	
0,90	c	0,636	0,779	0,962	1,19	1,44	1,70	1,97	2,25	2,79	3,34	3,88	4,41	4,95	7,60	10,24	12,87	15,49	20,74	25,99	52,19	78,39	104,6	130,8	2,620	c	0,90
	y		1,57	3,56	5,62	7,55	9,26	10,72	11,96	13,90	15,35	16,46	17,33	18,04	20,22	21,34	22,02	22,48	23,05	23,40	24,11	24,40	24,53	24,53	24,83	y	
1,00	c	0,630	0,770	0,948	1,16	1,40	1,65	1,91	2,17	2,69	3,20	3,71	4,22	4,73	7,24	9,74	12,23	14,72	19,69	24,66	49,50	74,33	99,17	124,0	2,484	c	1,00
	y		1,54	3,49	5,47	7,32	8,95	10,33	11,50	13,35	14,72	15,78	16,62	17,30	19,38	20,45	21,11	21,55	22,10	22,44	23,11	23,37	23,50	23,52	23,81	y	
1,10	c	0,623	0,761	0,934	1,14	1,37	1,61	1,85	2,10	2,59	3,08	3,57	4,05	4,53	6,92	9,30	11,68	14,05	18,79	23,52	47,18	70,82	94,46	118,1	2,365	c	1,10
	y		1,51	3,42	5,34	7,12	8,67	9,99	11,11	12,87	14,18	15,20	16,00	16,65	18,66	19,69	20,32	20,74	21,28	21,60	22,25	22,49	22,62	22,65	22,92	y	
1,20	c	0,617	0,752	0,921	1,12	1,34	1,57	1,80	2,04	2,51	2,97	3,44	3,90	4,36	6,65	8,92	11,19	13,46	17,99	22,52	45,15	67,77	90,38	113,0	2,263	c	1,20
	y		1,49	3,36	5,23	6,93	8,42	9,69	10,76	12,45	13,71	14,68	15,46	16,09	18,02	19,02	19,63	20,04	20,56	20,87	21,50	21,74	21,87	21,89	22,15	y	
1,30	c	0,611	0,743	0,909	1,10	1,31	1,53	1,76	1,98	2,43	2,88	3,32	3,77	4,21	6,40	8,59	10,77	12,94	17,29	21,64	43,35	65,07	86,78	108,5	2,171	c	1,30
	y		1,46	3,30	5,12	6,77	8,20	9,42	10,45	12,07	13,29	14,23	14,98	15,58	17,46	18,43	19,02	19,41	19,92	20,22	20,84	21,07	21,19	21,21	21,47	y	
1,40	c	0,605	0,735	0,897	1,08	1,29	1,50	1,72	1,93	2,36	2,79	3,22	3,65	4,07	6,18	8,29	10,38	12,48	16,66	20,84	41,75	62,67	83,58	104,5	2,090	c	1,40
	y		1,44	3,25	5,02	6,62	8,00	9,17	10,17	11,74	12,92	13,83	14,55	15,14	16,96	17,90	18,47	18,86	19,35	19,65	20,25	20,48	20,60	20,61	20,86	y	
1,50	c	0,599	0,727	0,885	1,07	1,26	1,47	1,68	1,88	2,30	2,72	3,13	3,54	3,95	5,99	8,01	10,03	12,06	16,09	20,13	40,30	60,47	80,63	100,8	2,017	c	1,50
	y		1,41	3,20	4,92	6,48	7,81	8,95	9,91	11,44	12,58	13,46	14,16	14,74	16,51	17,42	17,98	18,36	18,84	19,13	19,71	19,95	20,07	20,07	20,31	y	
1,60	c	0,593	0,719	0,874	1,05	1,24	1,44	1,64	1,84	2,24	2,64	3,04	3,44	3,84	5,80	7,76	9,72	11,67	15,58	19,48	38,99	58,49	77,99	97,49	1,951	c	1,60
	y		1,39	3,15	4,84	6,35	7,65	8,75	9,68	11,16	12,27	13,13	13,81	14,37	16,10	16,99	17,54	17,91	18,37	18,66	19,23	19,44	19,56	19,58	19,82	y	
1,70	c	0,587	0,712	0,863	1,04	1,22	1,41	1,61	1,80	2,19	2,58	2,96	3,35	3,73	5,64	7,54	9,43	11,32	15,11	18,89	37,79	56,68	75,58	94,47	1,890	c	1,70
	y		1,37	3,10	4,75	6,23	7,49	8,56	9,47	10,91	11,99	12,83	13,49	14,04	15,72	16,60	17,13	17,50	17,95	18,23	18,79	18,99	19,11	19,13	19,37	y	
1,80	c	0,581	0,704	0,853	1,02	1,20	1,39	1,57	1,76	2,14	2,52	2,89	3,26	3,64	5,49	7,33	9,17	11,00	14,67	18,34	36,68	55,02	73,35	91,69	1,834	c	1,80
	y		1,35	3,06	4,68	6,12	7,35	8,39	9,28	10,68	11,73	12,55	13,20	13,73	15,38	16,24	16,76	17,12	17,57	17,84	18,39	18,58	18,70	18,72	18,95	y	
1,90	c	0,576	0,697	0,843	1,01	1,18	1,36	1,55	1,73	2,09	2,46	2,82	3,19	3,55	5,34	7,13	8,92	10,71	14,27	17,84	35,66	53,48	71,30	89,12	1,782	c	1,90
	y		1,33	3,02	4,61	6,01	7,21	8,23	9,10	10,47	11,50	12,30	12,93	13,45	15,07	15,91	16,42	16,77	17,21	17,48	18,02	18,21	18,33	18,35	18,57	y	
2,00	c	0,570	0,690	0,833	0,993	1,16	1,34	1,52	1,70	2,05	2,41	2,76	3,11	3,46	5,21	6,95	8,69	10,43	13,90	17,37	34,72	52,06	69,40	86,74	1,734	c	2,00
	y		1,31	2,98	4,54	5,91	7,09	8,09	8,93	10,27	11,28	12,06	12,68	13,19	14,78	15,60	16,11	16,45	16,88	17,15	17,68	17,85	17,97	18,00	18,22	y	
2,20	c	0,560	0,676	0,814	0,967	1,13	1,30	1,47	1,63	1,97	2,31	2,64	2,98	3,31	4,97	6,63	8,28	9,93	13,23	16,53	33,01	49,49	65,98	82,46	1,649	c	2,20
	y		1,27	2,90	4,41	5,74	6,87	7,82	8,63	9,92	10,88	11,64	12,24	12,73	14,26	15,06	15,55	15,88	16,30	16,55	17,07	17,22	17,34	17,38	17,60	y	
2,40	c	0,549	0,663	0,796	0,943	1,10	1,26	1,42	1,58	1,90	2,22	2,54	2,86	3,18	4,76	6,34	7,92	9,49	12,64	15,79	31,52	47,24	62,97	78,69	1,573	c	2,40
	y		1,24	2,83	4,30	5,58	6,67	7,59	8,37	9,61	10,54	11,27	11,85	12,33	13,81	14,58	15,06	15,38	15,79	16,03	16,53	16,68	16,80	16,84	17,05	y	
2,60	c	0,550	0,650	0,779	0,921	1,07	1,22	1,38	1,53	1,84	2,14	2,45	2,76	3,06	4,58	6,09	7,60	9,10	12,12	15,13	30,19	45,24	60,30	75,35	1,506	c	2,60
	y		1,20	2,77	4,20	5,44	6,49	7,38	8,13	9,33	10,24	10,94	11,51	11,97	13,41	14,16	14,63	14,94	15,34	15,58	16,06	16,19	16,30	16,36	16,57	y	
2,80	c	0,540	0,638	0,763	0,899	1,04	1,19	1,34	1,48	1,78	2,07	2,37	2,66	2,95	4,41	5,86	7,31	8,75	11,65	14,54	28,99	43,44	57,90	72,35	1,446	c	2,80
	y		1,17	2,71	4,11	5,31	6,33	7,19	7,92	9,09	9,97	10,65	11,20	11,65	13,06	13,79	14,24	14,55	14,94	15,17	15,65	15,79	15,90	15,94	16,14	y	
3,00	c	0,531	0,626	0,749	0,879	1,02	1,16	1,30	1,44	1,73	2,01	2,29	2,57	2,85	4,25	5,65	7,04	8,44	11,22	14,00	27,91	41,82	55,73	69,64	1,391	c	3,00
	y		1,14	2,66	4,02	5,19	6,19	7,03	7,74	8,87	9,73	10,39	10,93	11,37	12,74	13,46	13,90	14,20	14,58	14,81	15,28	15,41	15,52	15,56	15,76	y	

c) 2h'/D = 0.80 (above) d) 2h'/D = 0.75 (below)

μ%	f/R	0,30	0,40	0,50	0,60	0,70	0,80	0,90	1,00	1,20	1,40	1,60	1,80	2,00	3,00	4,00	5,00	6,00	8,00	10,0	20,0	30,0	40,0	50,0	∞	f/R	μ%
0,40	c	0,674	0,834	1,05	1,35	1,72	2,14	2,57	3,02	3,90	4,77	5,64	6,50	7,35	11,57	15,75	19,92	24,08	32,40	40,71	82,22	123,7	165,2	206,7	4,149	c	0,40
	y		1,46	3,72	6,34	9,09	11,64	13,86	15,72	18,60	20,68	22,25	23,47	24,44	27,37	28,84	29,72	30,31	31,06	31,50	32,39	32,70	32,87	32,93	33,30	y	
0,50	c	0,668	0,824	1,04	1,32	1,66	2,03	2,42	2,81	3,60	4,37	5,14	5,91	6,67	10,44	14,18	17,91	21,63	29,07	36,50	73,63	110,8	147,9	185,0	3,712	c	0,50
	y		1,44	3,63	6,10	8,60	10,89	12,85	14,50	17,05	18,91	20,32	21,41	22,29	24,95	26,28	27,09	27,63	28,30	28,71	29,53	29,85	29,98	30,02	30,36	y	
0,60	c	0,661	0,815	1,02	1,29	1,60	1,94	2,30	2,65	3,37	4,08	4,78	5,47	6,17	9,61	13,02	16,42	19,82	26,62	33,40	67,32	101,2	135,1	169,0	3,391	c	0,60
	y		1,41	3,55	5,90	8,21	10,29	12,07	13,56	15,89	17,58	18,87	19,87	20,68	23,13	24,37	25,12	25,62	26,25	26,62	27,38	27,61	27,75	27,84	28,15	y	
0,70	c	0,654	0,805	1,01	1,26	1,56	1,87	2,20	2,53	3,19	3,84	4,49	5,13	5,77	8,96	12,12	15,27	18,42	24,71	31,00	62,42	93,85	125,3	156,7	3,142	c	0,70
	y		1,39	3,48	5,71	7,88	9,80	11,44	12,82	14,96	16,53	17,73	18,66	19,42	21,71	22,87	23,57	24,04	24,63	24,99	25,70	25,96	26,11	26,13	26,42	y	
0,80	c	0,648	0,797	0,993	1,24	1,51	1,81	2,12	2,42	3,04	3,65	4,25	4,85	5,45	8,43	11,39	14,34	17,29	23,18	29,06	58,47	87,88	117,3	146,7	2,941	c	0,80
	y		1,37	3,41	5,55	7,60	9,39	10,92	12,21	14,21	15,68	16,80	17,68	18,39	20,55	21,65	22,31	22,76	23,32	23,66	24,34	24,59	24,73	24,75	25,03	y	
0,90	c	0,641	0,788	0,979	1,21	1,48	1,76	2,04	2,33	2,91	3,48	4,05	4,62	5,19	7,99	10,78	13,57	16,35	21,90	27,45	55,19	82,93	110,7	138,4	2,780	c	0,90
	y		1,35	3,35	5,41	7,35	9,04	10,48	11,69	13,58	14,96	16,02	16,84	17,53	19,59	20,63	21,26	21,69	22,22	22,55	23,20	23,37	23,54	23,59	23,86	y	
1,00	c	0,635	0,779	0,966	1,19	1,44	1,71	1,98	2,26	2,80	3,35	3,89	4,42	4,96	7,62	10,27	12,91	15,55	20,82	26,09	52,42	78,75	105,1	131,4	2,628	c	1,00
	y		1,33	3,29	5,28	7,13	8,74	10,10	11,25	13,03	14,35	15,36	16,16	16,80	18,77	19,77	20,37	20,78	21,30	21,61	22,23	22,43	22,57	22,61	22,87	y	
1,10	c	0,629	0,771	0,953	1,17	1,41	1,67	1,93	2,19	2,71	3,22	3,74	4,25	4,76	7,30	9,82	12,34	14,86	19,88	24,91	50,02	75,11	100,2	125,3	2,511	c	1,10
	y		1,31	3,23	5,16	6,94	8,47	9,77	10,86	12,57	13,83	14,79	15,56	16,17	18,06	19,02	19,61	20,00	20,49	20,79	21,39	21,57	21,70	21,76	22,01	y	
1,20	c	0,623	0,763	0,941	1,15	1,38	1,63	1,87	2,12	2,62	3,12	3,61	4,10	4,59	7,01	9,43	11,84	14,25	19,07	23,88	47,93	71,99	96,04	120,1	2,405	c	1,20
	y		1,29	3,18	5,06	6,76	8,23	9,48	10,52	12,15	13,36	14,29	15,03	15,62	17,44	18,37	18,93	19,31	19,79	20,08	20,66	20,85	20,98	21,02	21,26	y	
1,30	c	0,617	0,755	0,929	1,13	1,36	1,59	1,83	2,07	2,54	3,02	3,49	3,96	4,43	6,76	9,08	11,40	13,72	18,34	22,96	46,07	69,18	92,29	115,4	2,310	c	1,30
	y		1,28	3,13	4,96	6,60	8,02	9,21	10,22	11,79	12,95	13,85	14,56	15,13	16,89	17,79	18,34	18,71	19,17	19,45	20,02	20,19	20,33	20,36	20,59	y	
1,40	c	0,611	0,747	0,918	1,12	1,33	1,56	1,79	2,02	2,48	2,93	3,39	3,84	4,29	6,54	8,77	11,01	13,24	17,69	22,15	44,41	66,67	88,94	111,2	2,226	c	1,40
	y		1,26	3,09	4,86	6,46	7,83	8,98	9,94	11,46	12,59	13,45	14,14	14,69	16,40	17,28	17,81	18,17	18,62	18,89	19,44	19,60	19,73	19,77	20,00	y	
1,50	c	0,606	0,740	0,907	1,10	1,31	1,53	1,75	1,97	2,41	2,85	3,29	3,73	4,16	6,33	8,49	10,65	12,80	17,11	21,41	42,91	64,41	85,90	107,4	2,150	c	1,50
	y		1,25	3,04	4,78	6,33	7,65	8,76	9,70	11,17	12,26	13,10	13,76	14,30	15,96	16,81	17,33	17,68	18,12	18,39	18,92	19,10	19,23	19,25	19,47	y	
1,60	c	0,600	0,732	0,896	1,08	1,29	1,50	1,71	1,93	2,35	2,78	3,20	3,63	4,05	6,15	8,24	10,32	12,41	16,58	20,74	41,55	62,37	83,18	104,0	2,081	c	1,60
	y		1,23	3,00	4,70	6,20	7,49	8,57	9,47	10,90	11,96	12,77	13,42	13,95	15,56	16,39	16,90	17,24	17,67	17,93	18,45	18,62	18,73	18,77	18,99	y	
1,70	c	0,594	0,725	0,886	1,07	1,27	1,47	1,68	1,88	2,30	2,71	3,12	3,53	3,94	5,98	8,00	10,03	12,05	16,09	20,13	40,31	60,51	80,70	100,9	2,019	c	1,70
	y		1,22	2,96	4,62	6,09	7,34	8,39	9,27	10,65	11,68	12,48	13,11	13,62	15,20	16,01	16,50	16,84	17,26	17,51	18,03	18,20	18,32	18,34	18,55	y	
1,80	c	0,589	0,718	0,876	1,06	1,25	1,45	1,65	1,85	2,25	2,65	3,05	3,45	3,85	5,82	7,79	9,75	11,72	15,64	19,56	39,17	58,77	78,37	97,97	1,961	c	1,80
	y		1,20	2,93	4,55	5,99	7,20	8,22	9,08	10,43	11,43	12,21	12,83	13,33	14,87	15,66	16,14	16,47	16,88	17,13	17,64	17,81	17,93	17,94	18,15	y	
1,90	c	0,584	0,711	0,866	1,04	1,23	1,42	1,62	1,81	2,20	2,59	2,98	3,37	3,75	5,67	7,59	9,50	11,41	15,23	19,04	38,11	57,18	76,24	95,31	1,907	c	1,90
	y		1,19	2,89	4,49	5,89	7,07	8,07	8,90	10,22	11,20	11,96	12,56	13,05	14,56	15,34	15,81	16,13	16,54	16,78	17,28	17,44	17,57	17,57	17,78	y	
2,00	c	0,579	0,704	0,856	1,03	1,21	1,40	1,59	1,78	2,16	2,54	2,92	3,29	3,67	5,54	7,40	9,26	11,12	14,84	18,56	37,13	55,70	74,27	92,84	1,857	c	2,00
	y		1,17	2,86	4,43	5,80	6,95	7,92	8,74	10,03	10,99	11,73	12,32	12,80	14,28	15,04	15,51	15,82	16,22	16,46	16,94	17,12	17,23	17,24	17,44	y	
2,20	c	0,569	0,691	0,838	1,00	1,18	1,36	1,54	1,72	2,08	2,44	2,80	3,16	3,51	5,29	7,07	8,84	10,61	14,15	17,69	35,37	53,04	70,72	88,39	1,768	c	2,20
	y		1,15	2,80	4,31	5,63	6,74	7,67	8,45	9,68	10,61	11,32	11,89	12,35	13,77	14,51	14,96	15,26	15,65	15,88	16,35	16,50	16,62	16,63	16,83	y	
2,40	c	0,559	0,679	0,821	0,979	1,15	1,32	1,49	1,66	2,01	2,35	2,69	3,03	3,38	5,08	6,77	8,46	10,16	13,54	16,92	33,81	50,70	67,59	84,48	1,689	c	2,40
	y		1,12	2,74	4,21	5,48	6,55	7,44	8,20	9,38	10,27	10,96	11,51	11,95	13,33	14,05	14,48	14,78	15,15	15,37	15,83	15,98	16,09	16,11	16,29	y	
2,60	c	0,549	0,666	0,804	0,956	1,12	1,28	1,44	1,61	1,94	2,27	2,60	2,93	3,25	4,88	6,51	8,13	9,75	12,99	16,23	32,43	48,62	64,81	81,00	1,620	c	2,60
	y		1,10	2,68	4,12	5,35	6,38	7,24	7,97	9,12	9,98	10,64	11,17	11,60	12,94	13,64	14,06	14,35	14,71	14,93	15,37	15,49	15,62	15,64	15,83	y	
2,80	c	0,540	0,655	0,789	0,936	1,09	1,25	1,41	1,56	1,88	2,20	2,51	2,83	3,14	4,71	6,27	7,83	9,39	12,50	15,62	31,19	46,75	62,32	77,88	1,557	c	2,80
	y		1,08	2,64	4,03	5,22	6,22	7,06	7,77	8,88	9,71	10,36	10,88	11,30	12,60	13,27	13,69	13,97	14,32	14,54	14,97	15,09	15,21	15,23	15,41	y	
3,00	c	0,532	0,643	0,774	0,916	1,06	1,22	1,37	1,52	1,83	2,13	2,43	2,74	3,04	4,55	6,05	7,56	9,06	12,04	15,06	30,06	45,06	60,06	75,06	1,500	c	3,00
	y		1,06	2,59	3,95	5,11	6,09	6,90	7,58	8,67	9,48	10,11	10,61	11,02	12,29	12,95	13,35	13,63	13,97	14,18	14,61	14,74	14,85	14,86	15,04	y	

Table 5.13: Diagramme for Limit State Method (from C. Menn)





5.7.3 Shear Reinforcement

Nominal Shear Stress:

When analysing the equilibrium of the forces ("truss analogy": compression force in concrete ; tensile force in reinforcement) and the continuity of these forces, it is evident that not the whole section can be considered when computing the relevant shear stress. It can be assumed that the relevant concrete section for shear consists of two strips located at the circumference of the pile section as shown in Figure 5.22. Considering the orientation of the shear forces, the relevant shear stress can be expressed as:

$$\tau = \frac{S}{b' (D - 2 h')}$$

where S is the shear force
(as computed in Chapter 5.6.2)
and $b' (D - 2 h')$ is the substituted shear section
with $b' = 400$ mm

The nominal shear stress has to be smaller than the admissible shear stress. If not, the pile section or the number of piles has to be increased.

$$\tau < \tau_{c,adm}$$

Calculation of Shear Reinforcement:

$$A_{st,b} = \frac{S \cdot a}{(D - 2h') \cdot \sigma_{st,adm}}$$

where a is the spacing of binders

$$A_{st,b} = 2 \cdot \pi \, d^2 / 4$$

Note that binders are "cut twice" when considering the equilibrium of inner (reinforcement) and outer (shear) forces.

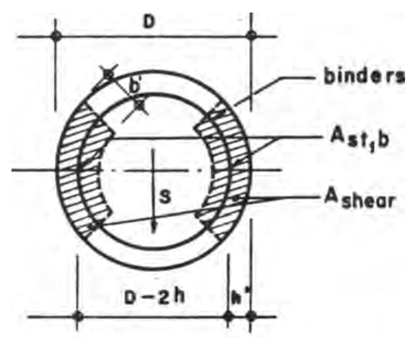


Fig. 5.22

The minimum reinforcement is indicated in Section 5.7.1:
Binders min. dia 12 mm with spacing $a > 500$ mm and $a > 250$ mm near the pile cap on a length minimum equal to the pile diameter.

5.8 Check of Results

- the required safety factors for bearing capacity are: - point resistance: s = 2; - skin resistance : s = 3
- admissible settlement : generally $w < 20$ mm
- admissible displacements: generally $w < 20$ mm (must be checked with superstructure)
- reinforcement limits (main bars) : $\mu_{min} = 0.4\%$; $\mu_{max} = 6\%$
- admissible stresses: depending on material and load case (see Section 5.7.1)

All important data, together with related drawings, should be presented in a finished form and in such a manner that the construction site may easily and with certainty understand the design requirements. An example of such a form is presented on the next page.

Construction Site:				**Pile No:** ....
Refe-rence Level 0.00 = D.G.L. (m)	**Depth below Ground Level (m)**	**Soil, Ground Water, Pile Remarks**	**Pile Reinforce-ment**	**Pile Data** D.G.L. (above see level) .... m Design diameter .... m Total length .... m Length below ground level .... m Embedment length in bearing layer .... m
				<u>Concrete</u>: Cement .... kg/m3 Water-Cement ratio .... Minimum strength .... N/mm2 (28 days cube or cylinder) <u>Reinforcement</u>: Type of steel .... Main bars Ø .... No. .... Binders Ø .... a = ... mm & at topmost Ø .... a = ... mm Overlapping .... m Free end at top .... m Concrete cover .... mm Spacers .... per cage <u>Piling Works</u>: Drilling diameter .... m Excavation (normal) .... m3 Filling (bentonite) .... m3 Concrete .... m3 Depth below G.L. .... m Piling equipment: .... Bentonite pumping equipment: .... Boring gear up to 44 m: ....
				Scale: 1: .... Date: .... Design Engineer: .... Related Drawings: ....

5.9 Example

In this section, the design of piles for the **Pathwe Bridge** is taken as an example.

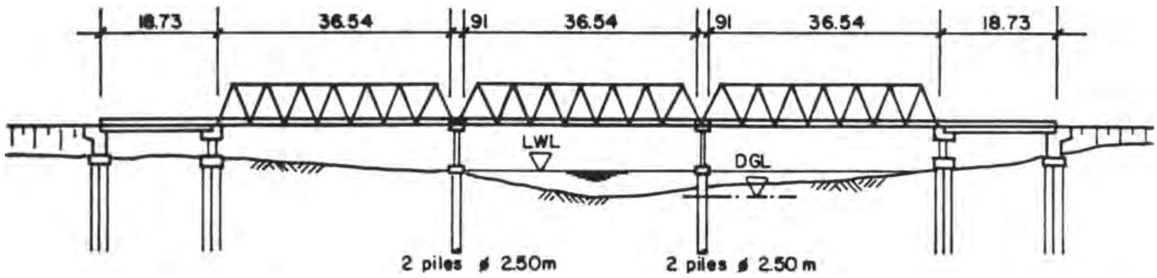


Fig. 5.23

5.9.1 Compilation and Consolidation of Data from Soil Investigation

The on-site soil investigation of the pile site provided a set of data upon the soils down to 40.2 m.. Note that, in addition to the on-site data, selected samples collected with the "split spoon" were submitted to laboratory tests for gradation, standard properties as moisture content and density, unconfined compression strength and triaxial shear strength. For the original survey results, see Section 5.2: Soil Investigation. (Table 5.1)

These test results are consolidated into bands of more or less consistent soils in order to simplify calculations. Depths have been rearranged to the Design Ground Level (D.G.L.).

Layer	Depth below DGL m	Thickness of Layer m	Soil Classification	γ kN/m^3	SPT N	Shear strength c_u kN/m^2	Cohesion c kN/m^2	Friction ø
1	0 - 3.7	3.7	soft silt & clay	16.5	3	16.0		
2	3.7 - 20.5	16.8	medium dense sand	18.0	12		2.4	28°
3	20.5 - 25.0	4.5	soft silt	18.5	9	26.8		
4	25.0 - 32.7	7.7	medium dense sand & silt	19.5	15		4.0	27°
5	32.7 - 35.7	3.0	dense sandy silt	20.0	30		2.4	27°
6	35.7 - 40.2	4.5	very dense sand	19.5	42		3.2	32°

Table 5.13

Assumption: 2 piles with a length of 40 m below ground level were chosen with diameter of 2.5 m.

Ground water level: 3.0 m below ground level (1.5 m below D.G.L.)
Embedment in layers No. 5 and No. 6: embedment length 5.8 m
Pile length above ground level: 2.5 m (4.0 m above D.G.L.)

5.9.2 Loads

Forces on Pier

Derived from load calculation of super- and substructure:

Load	Element	Direction	Force [kN]	Height above pile head [m]
Dead Load	superstructure substructure	vertical vertical	2504 2912	- -
Traffic Load (AASHTO HS 20-44)	superstructure superstructure	vertical hor., longitudinal	931 47	- 5.50
Wind Load (3.6 kN/m2)	superstructure substructure	hor., transverse hor., transverse	154 43	5.50 2.30
Earthquake (12% of DL)	superstructure substructure	hor., long. & transv. hor., long. & transv.	302 350	5.50 2.30

Table 5.14

Load Cases

After reducing the forces, the following max. loads are obtained at the pile cap:

Load Case	Load *	Direction		Force/ Moment
1) Main Load (transv. dir.)	DL+TL + Wind	vertical hor. transv. Moment (x-)	R_V R_{Hy} R_{Mx}	6347 kN 197 kN 946 kNm
2) Max. Load (long. dir.)	DL+EQ	vertical hor. long Moment (y-)	R_V R_{Hx} R_{My}	5416 kN 652 kN 2466 kNm
3) Max. Load (transv. dir.)	DL+EQ	vertical hor. transv. Moment (x-)	R_V R_{Hy} R_{Mx}	5416 kN 652 kN 2466 kNm

Table 5.15

* DL: Dead Load (excluding weight of piles)
TL: Traffic Load (AASHTO HS 20-44)
EQ: Earthquake (12% of DL)

Protruding part of pile (above D.G.L.): h = 4.00 m

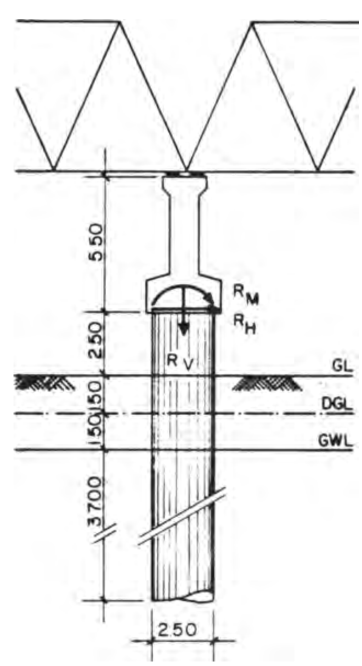


Fig. 5.24

5.9.3 Calculation of Bearing Capacity

Properties of Piles

Diameter (designed): $d = 2.50$ m

Area of pile: $A_p = 4.91\ m^2$

Distance between piles: $a = 7.10$ m (centre-centre)

Point resistance

$\varnothing = 32°$ (layer No. 6) => $N_q = 23;\ N_c = 35$

a) Theoretical Approach

Vertical stress in soil:

Layer	Length L_i (m)	Vertical Stress q_v (kN/m^2)	Unit Weight γ ' (kN/m^3)	
1	1.5	24.75	16.5	above ground water level G.W.L.
1	2.2	14.30	(16.5 - 10.0)	below G.W.L
2	16.8	134.40	(18.0 - 10.0)	
3	4.5	38.25	(18.5 - 10.0)	
4	7.7	73.15	(19.5 - 10.0)	
5	3.0	30.00	(20.0 - 10.0)	
6	2.8	26.60	(19.5 - 10.0)	
total	**38.5**	**341.45**		

Table 5.16

$$Q_{pr,1} = A_p\,(1 + \tan \varnothing) \cdot (c \cdot N_c + q_v \cdot N_q)$$
$$= 4.9\,(1 + \tan 32^{o}) \cdot (3.2 \cdot 35 + 341.45 \cdot 23) = 63'419 \text{ kN}$$

b) Empirical Approach

$$N' = 15 + (N - 15)/2 = 15 + (42 - 15)/2 = 28.5$$
$$q_{pr} = \alpha \cdot N' = 400 \cdot 28.5 = 11'400 \text{ kN} \qquad (\alpha \text{ from Table 5.4})$$
$$Q_{pr,2} = A_p \cdot q_{pr} = 4.9 \cdot 11'400 = 55'860 \text{ kN}$$

We therefore take the minimum of the theoretical and empirical approach:

$$Q_{pr} = Q_{pr,2} = \underline{55'860} \text{ kN}$$

Skin Resistance

Remark:

Layers 1) and 3) are susceptible to consolidation since they consist of soft silt. Layer 2) will follow the downward movement of layer 3). Thus, only layers 4) to 6) are taken into account for skin friction. As these layers are silty or sandy the approach for cohesionless soil is used.

Layer	Length L_i (m)	Vertical Stress q_{vi} (kN/m^2)	SPT N	α	$q_{sri} = \alpha \cdot q_{vi}$ (kN/m^2)	$L_i \cdot q_{sri}$ (kN/m)
4	7.7	248.3	15	0.6	149.0	1147.3
5	3.0	299.8	30	0.6	179.9 (use 150) *	450
6	2.8	328.2	42	0.8	262.5 (use 150) *	420
total						**2017.3**

Table 5.17

Friction coefficient α is taken from Table 5.5, emprical limits for skin friction (*) are indicated in Section 5.3.6. The vertical stresses in the centre of the soil layer were computed as follows:

$$q_{v4} = 211.7 + 73.15 / 2 = 248.3 \text{ kN/m}^2$$
$$q_{v5} = 284.85 + 30.0 / 2 = 299.3 \text{ kN/m}^2$$
$$q_{v6} = 314.85 + 26.6 / 2 = 328.2 \text{ kN/m}^2$$

Total skin resistance:

$$Q_{sr} = \pi \cdot D \cdot \Sigma\, q_{sri} \cdot L_i = 3.14 \cdot 2.5 \cdot 2017.3 = 15'844 \text{ kN}$$

For the considered cohesionless soil layers, the group factor is: $g = 1$
(Spacing between centres of piles: $a = 7.10$ m > 2.5 $D = 6.25$ m)

Total Bearing Capacity

Ultimate vertical bearing resistance: $Q_u = Q_{pr} + Q_{sr} = 55'860 + 15'844 = 71'704$ kN

Admissible load : $Q_{adm} = Q_{pr} / s_{pr} + Q_{sr} / s_{sr}$
$= 55'860 / 2 + 15'844 / 3 = 33'211$ kN

Total weight of pile : $W_p = A_p \cdot (L'' \cdot \gamma_c + L' \cdot \gamma_c')$
$= 4.9 \cdot (5.5 \cdot 24 + 37 \cdot (24 - 10)) = 3'185$ kN

Effective maximal load (case 1): $V_{eff} = R_v / 2 + W_p = 6'347 / 2 + 3'185 = 6'359$ kN

Check: $V_{eff} = 6'359$ kN $<$ $Q_{adm} = 33'211$ kN

----> The bearing capacity is largely sufficient for effective loads.

(The increase/decrease due to $\Delta V = +/- (M / a) = +/- (946 / 7.10) = +/- 133$ kN is neglected.)

5.9.4 Calculation of Vertical Displacement

Settlement of Single Pile

a) Graphic Method

- point resistance:

$0.14\ Q_{pr} = 0.14 \cdot 55'860 = 7'820$ kN ; w = 10 mm
$0.23\ Q_{pr} = 0.23 \cdot 55'860 = 12'850$ kN ; w = 20 mm
$0.32\ Q_{pr} = 0.32 \cdot 55'860 = 17'880$ kN ; w = 30 mm
$1.00\ Q_{pr} = 55'860$ kN ; w = 150 mm

- skin friction

$1.00\ Q_{sr} = 15'844$ kN ; w = 20 mm

from Figure 5.25:

$V_{eff} = 6'359$ kN ---> settlement w = 4 mm

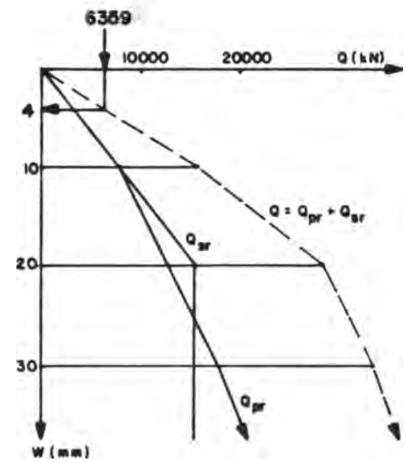


Fig. 5.25

b) Method of Cassan

- modulus of elasticity of soil: $E_s = 80 \cdot 10^3$ kN/m²
(estimated from soil properties and Table 5.2a)
- modulus of elasticity of pile (concrete) : $E_p = 25 \cdot 10^6$ kN/m²
- embedment length : $L_1 = 5.8$ m
(layers No. 5 and 6 considered)

$$f = \frac{1.68 \cdot E_s}{D \cdot E_p} = \frac{1.68 \cdot 80 \cdot 10^3}{2.50 \cdot 25 \cdot 10^6} = 0.046 \qquad \mathrm{tgh}\,(f \cdot L1) = 0.261$$

$$w = \frac{Q_{eff} \cdot D}{A} \cdot \frac{1 + \frac{4.5 \cdot E_s}{f \cdot D \cdot E_p} \cdot \mathrm{tgh}\,(f \cdot L_1)}{4.5 \cdot E_s + f \cdot D \cdot E_p \cdot \mathrm{tgh}\,(f \cdot L_1)} \cdot 10^3 = 3.0 \text{ mm (settlement)}$$

For the calculation of the total displacement the higher value of w = 4 mm is taken into account.

Group Effect

w (group) = g · w (single pile)

for cohesionless bearing layers $g = \sqrt{\frac{b}{D}}$

with a = 7.10 m, D = 2.50 M

and $b = \frac{b_1 + b_2}{2} = \frac{10.20 + 3.20}{2} = 6.70$ m

Thus $g = \sqrt{\frac{6.70}{2.50}} \cdot 1.64$

and w (group) = 1.64 · w (single pile)

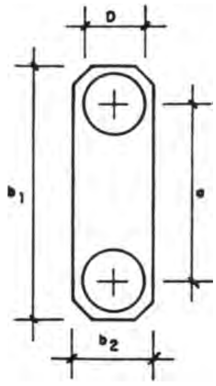


Fig. 5.26

Elastic Compression of Pile

$$W_{el} = \frac{V_{eff}}{A_p \cdot E_p} \cdot L \Rightarrow \frac{6'359}{4.91 \cdot 25 \cdot 100} \cdot 42'500 = 2.20 \text{ mm}$$

Total Displacement at Pile Foot

$$w_1 = (w + w_{el}) \cdot g = (4.0 + 2.2) \cdot 1.64 = 10 \text{ mm} < w_{adm} = 20 \text{ mm}$$

5.9.5 Calculation of Horizontal Displacement

Characteristic length of pile:

$$L_o = \sqrt[4]{\frac{4\, E_p \cdot I_p}{D \cdot k_h}}$$

where: $E_p = 25 \cdot 10^6$ N/mm^2

$$I_p = \frac{\pi \cdot D^4}{64} = 1.92 \text{ m}^4$$

Coefficient k_h of subgrade reaction at relevant depth $z = 0.75 \cdot L_o$

Estimation of L_o: approx 10 m $\Rightarrow$ $z = 7.50$ m

Horizontal bearing coefficient for layer N^o 2: $k_h = 3 \cdot 10^3$ kN/m^3

(medium dense sand, estimated from table 5.2 a)

Thus the coefficient k_h for a single pile is:

$$k_h = n_h \frac{z}{D} = 3 \cdot 10^3 \cdot \frac{2.50}{7.50} = 9.00 \cdot 10^3$$

resp. for a group of piles:

$k_h(\text{group}) = 0.25 \cdot k_h = 2.25 \cdot 10^3$ (see 5.5.3)

and the characteristic length of pile is:

in longitudinal (bridge) direction: $L_o = \sqrt[4]{\frac{4 \cdot 25 \cdot 10^6 \cdot 192}{2.50 \cdot 9.00 \cdot 10^3}} = 9.61$ m

in transverse (bridge) direction: $L_o = \sqrt[4]{\frac{4 \cdot 25 \cdot 10^6 \cdot 192}{2.50 \cdot 2.25 \cdot 10^3}} = 13.59$ m

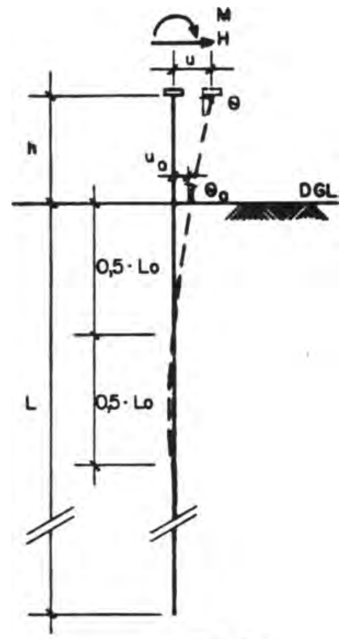


Fig. 5.27

Displacement in longitudinal direction

(case 2, DL + Earthquake)

Relevant pile model: Single pile with fixed head type a1 (Fig. 5.10)

$$H = \frac{R_{Hy}}{2} = \frac{652}{2} = 326 \text{ kN / pile}$$

$$M = \frac{R_{My}}{2} = \frac{2466}{2} = 1233 \text{ kNm / pile}$$

$$Ep \cdot Ip = 25 \cdot 10^6 \cdot 1.92 = 48 \cdot 10^6 \text{ kN / m}^2$$

$h = 4.00$ m

$L = 38.5$ m

Displacement at design ground level (D.G.L.)

With $L/Lo = \frac{38.5}{9.61} = 4.0$ the coefficients are :

$\alpha_H = 0.84$ $\quad \alpha M = 0.78$

$\beta_H = 0.78$ $\quad \beta M = 1.23$

$H_o = H = 326$ kN / pile

$M_o = H_o \cdot h + M = 326 \cdot 4.00 + 1233 = 2537$ kNm / pile

$$u_o = \alpha_H \frac{H_o \cdot L_o^3}{E_p \cdot I_p} + \alpha_M \frac{M_o \cdot L_o^2}{E_p \cdot I_p} = 0.84 \frac{326 \cdot 9.6^3}{48 \cdot 10^6} + 0.78 \frac{2537 \cdot 9.6^2}{48 \cdot 10^6} = 0.0088 \text{ m}$$

$$\theta_o = \beta_H \frac{H_o \cdot L_o^2}{E \cdot I_p} + \beta_M \frac{M_o \cdot L_o}{E_p \cdot I_p} = 0.78 \frac{326 \cdot 9.6^2}{48 \cdot 10^6} + 1.23 \frac{2537 \cdot 9.6}{48 \cdot 10^6} = 0.0011 \text{ rad}$$

Displacement of the protruding part (free head)

$H = 326$ kN / pile

$M = 1233$ kNm / pile

$$u = H \cdot \frac{h^3}{3 E_p \cdot I_p} + M \cdot \frac{h^2}{2 E_p \cdot I_p} = 326 \frac{4.00^3}{3 \cdot 48 \cdot 10^6} + 1233 \frac{4.00^2}{2 \cdot 48 \cdot 10^6} = 0.0004 \text{ m}$$

$$\theta = H \cdot \frac{h^2}{2 E_p \cdot I_p} + M \cdot \frac{h}{E_p \cdot I_p} = 326 \frac{4.00^2}{2 \cdot 48 \cdot 10^6} + 1233 \frac{4.00}{48 \cdot 10^6} = 0.0002 \text{ rad}$$

Total displacements

$u = u_o + \theta_o \cdot h + u_1 = 0.0088 + 0.0011 \cdot 4.00 + 0.0004 = 0.0136 \text{ m} \;\hat{=}\; 13.6 \text{ mm}$

$\theta = \theta_o + \theta_1 = 0.0011 + 0.0002 = 0.0013 \text{ rad} \;\hat{=}\; 1.3 \cdot 10^{-3} \text{ rad}$

Admissible displacement

$u_{adm} = 20 \text{ mm} > u_{eff} = 13.6 \text{ mm}$

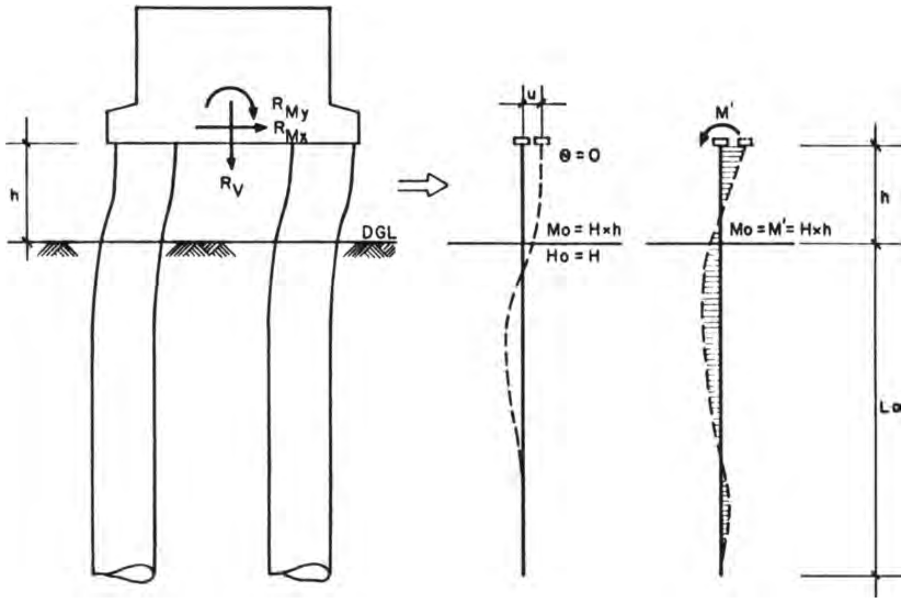


Fig. 5.28

<u>Displacement in transverse direction</u>

(case 3, Dead Load + Earthquake)

With $L/L_o = \frac{38.5}{13.59} = 2.83 \dashrightarrow 3.0$ the coefficients are

$\alpha_H = 0.77$ $\quad\quad$ $\alpha_M = 0.72$

$\beta_H = 0.72$ $\quad\quad$ $\beta_M = 1.19$

Step 1) : <u>for "free" head</u> : ($M_o = 326 \cdot 4 = 1304$ kNm, $H_o = 326$ kN)

$$u_o(H) = \alpha_H \frac{H_o \cdot L_o^3}{E_p \cdot I_p} + \alpha_M \frac{M_o \cdot L_o^2}{E_p \cdot I_p} = 0.77 \cdot \frac{326 \cdot 13.6^3}{48 \cdot 10^6} + 0.72 \frac{1304 \cdot 13.6^2}{48 \cdot 10^6} = 0.0168 \text{ m}$$

$$u(H) = H \cdot \frac{h^3}{3 E_p \cdot I_p} = 326 \cdot \frac{3 \cdot 4.0^3}{3 \cdot 48 \cdot 10^6} = 0.0001 \text{ m}$$

$$\theta(H) = \beta_H \frac{M_o \cdot L_o}{E_p \cdot I_p} + M_\beta \frac{M_o \cdot L_o}{E_p \cdot I_p} = 0.72 \frac{326 \cdot 13.6^2}{48 \cdot 10^6} + 1.19 \frac{1'304 \cdot 13.6}{48 \cdot 10^6} = 1.34 \cdot 10^{-4} \text{rad}$$

$$\theta_1(H) = H \frac{h^2}{2 E_p \cdot I_p} = 326 \frac{4.0^2}{2 \cdot 48 \cdot 10^6} = 0.54 \cdot 10^{-4} \text{rad}$$

Thus $\quad u(H) = u_o + \theta_o \cdot h + u_1 = 0.0168 + 1.34 \cdot 10^{-3} \cdot 4.00 + 0.0001 = 0.0223$ m

$\quad\quad\quad \theta(H) = \theta_o + \theta_1 = 1.34 \cdot 10^{-3} + 0.54 \cdot 10^{-4} = 1.39 \cdot 10^{-3}$ rad

Step 2): for fixed head with $\hat{\theta} = \theta(H) - \theta(M') = 0$:

$$\theta(M') = \theta_o (M') + \theta_1 (M') = \beta_M \frac{M' \cdot L_o}{E_p \cdot E_p} + \frac{M' \cdot h}{E_p \cdot I_p} = \theta(H)$$

$$\text{Thus } M' = \frac{\theta(H) \cdot E_p \cdot I_p}{\beta_M \cdot L_o + h} = \frac{1.39 \cdot 10^{-3} \cdot 48 \cdot 10^6}{1.19 \cdot 13.6 + 4.0} = 3'306 \text{ kNm}$$

Step 3) : Finally u = u (H) - u (M')

$$\text{with} \quad u(M') = u_o + \theta_o \cdot h + u_1 = \alpha_M \frac{M' \cdot L_o^2}{E_p \cdot I_p} + \beta_M \frac{M' \cdot L_o}{E_p \cdot I_p} h + \frac{M' \cdot h}{E_p \cdot I_p}$$

$$= 0.77 \frac{3306 \cdot 13.6^2}{48 \cdot 10^6} + 1{,}19 \frac{3300 \cdot 13.6}{48 \cdot 10^6} \cdot 4.0 + \frac{3306 \cdot 4.0}{48 \cdot 10^6} = 0.0145 \text{ m}$$

and $\quad u(H) = 0{,}0223 \text{ m} \Rightarrow u = 0.0223 - 0.0145 = 0.078 \text{ m} \hat{=} 7.8 \text{ mm} < u_{adm} = 20 \text{ mm}$

5.9.6 Moments in Piles

Relevant load cases: Dead Load + Earthquake (both directions)

Load case 2: Earthquake in longitudinal direction ("free" head)
(Fig. 5.10, a1)

There are two single piles with a characteristic length L_o= 9.61 m
With values α, β from Table 5.11 the distribution of moments is computed as follows:

L	=	38.50 m	L/L_o	4.0	=>	4
L_o	=	9.61 m				
H	=	326 kN	$H_o \cdot L_o = H \cdot L_o$		=	3133 kN
M	=	1233 kNm				
h	=	4.00 m	M_o	$= H_o \cdot h + M$	=	2537 kNm

$M(z) = \alpha\,(z/L) \times H_o \cdot L_o + \beta\,(z/L) \times M_o$

z =	0.00 m	z/L =	0.0	M(z) =	2537 kNm
	3.85 m		0.1		3501 kNm
	7.70 m		0.2		3437 kNm
	11.55 m		0.3		2805 kNm
	15.40 m		0.4		1795 kNm
	19.25 m		0.5		1064 kNm
	23.10 m		0.6		477 kNm
	26.95 m		0.7		239 kNm
	30.80 m		0.8		31 kNm
	34.65 m		0.9		0 kNm
	38.50 m		1.0		0 kNm

Table 5.19

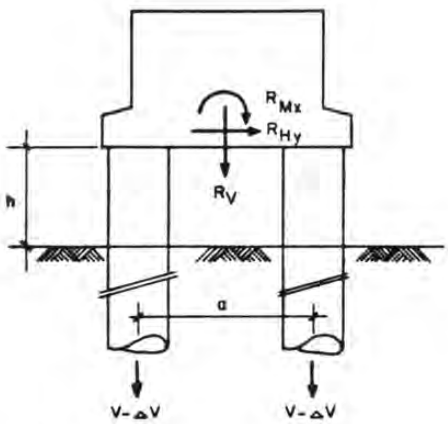
Load case 3: Earthquake in transverse direction (fixed head)

Here is a group of 2 piles with a characteristic length L_o = 13.59 m.

3 subcases are considered:

- piles with free head ---> upper extreme value (Fig. 5.10, b2)
- piles without rotation at fixed head ---> lower extreme value (Fig. 5.10, b1)
- piles with rotation at fixed head ---> most exact model (Fig. 5.10, b3)

Moment R_{Mx} is taken by additional vertical forces $\pm \Delta V$, which is no problem for vertical bearing capacity. Thus M = O kNm/pile.



$$V = \frac{R_V}{2} = \frac{5'416}{2} = 2708 \text{ kN/pile}$$

$$A_V = \pm \frac{R_{Mx}}{a} = \frac{2'466}{7.10} = \pm 347 \text{ kN/pile}$$

$$H_o = H = \frac{R_{Hy}}{2} = \frac{652}{2} = 326 \text{ kN/pile}$$

Fig. 5.29

a) Calculation of moments for "free head" (upper extreme value):

L	=	38.50 m	L/L_o	2.8	=>	3
L_o	=	13.59 m				
H	=	326 kN	$H_o \cdot L_o = H \cdot L_o$		=	4430 kN
M	=	1233 kNm				
h	=	4.00 m	$M_o = H_o \cdot h + M$		=	2537 kNm

$$M(z) = \alpha\,(z/L) \times H_o \cdot L_o + \beta\,(z/L) \times M_o$$

z =	0.00 m	z/L =	0.0	M(z) =	2537 kNm
	3.85 m		0.1		3657 kNm
	7.70 m		0.2		4043 kNm
	11.55 m		0.3		3770 kNm
	15.40 m		0.4		3041 kNm
	19.25 m		0.5		2211 kNm
	23.10 m		0.6		1438 kNm
	26.95 m		0.7		830 kNm
	30.80 m		0.8		348 kNm
	34.65 m		0.9		25 kNm
	38.50 m		1.0		0 kNm

Table 5.20

b) Calculation of moments without rotation at fixed head (lower extreme value):

Step 1) analogous to a) for "free" head

Step 2) Calculation of end moment M' for angle $\theta = 0$ at fixed head; computation of moments in pile due to M' (see Fig. 5.10, b1).

Angle at pile head due to H (M = 0):
(coefficients α and β see calculation of hor. displacement)

$$\theta(H) = \theta(H_o) + \theta(M_o)$$

$$\theta_o(Ho) = \beta_H \frac{H_o \cdot L_o^2}{E_p \cdot I_p} + \beta_M \frac{M_o \cdot L_o}{E_p \cdot I_p}$$

$$= 0.72 \frac{326 \cdot 13.6^2}{48 \cdot 10^6} + 1.19 \frac{326 \cdot 400 \cdot 13.6}{48 \cdot 10^6} = 1.34 \cdot 10^{-3} \text{ rad}$$

$$\theta_1(M_o) = \frac{H \cdot h^2}{2 E_p \cdot I_p} = \frac{326 \cdot 4.00^2}{2 \cdot 48 \cdot 10^6} = 0.54 \cdot 10^{-4} \text{ rad}$$

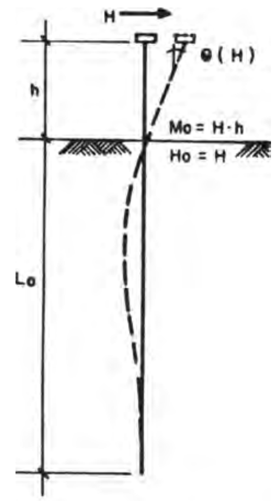


Fig. 5.30 a)

Head moment M' for $\theta = 0$:

$$\theta(M') = \theta_o(M') + \theta(M')$$

$$= \beta_M \frac{M' \cdot L}{E_p \cdot I_p} + \frac{M' \cdot h}{E_p \cdot I_p}$$

$$= 1.19 \cdot \frac{13.6}{48 \cdot 10^6} \cdot M' + \frac{4.00}{48 \cdot 10^6} \cdot M'$$

$$= 4.21 \cdot 10^{-7} \cdot M' \text{ [in kN/m]}$$

with $\theta(H) - \theta(M') = 0 \Rightarrow \theta(M') = \theta(H)$

$$4.21 \cdot 10^{-7} \cdot M' = 1.39 \cdot 10^{-3}$$

Therefore $M' = \dfrac{1.39 \cdot 10^{-3}}{4.21 \cdot 10^{-7}} = 3'306$ kNm/pile *

* compare with displacements in transverse direction.

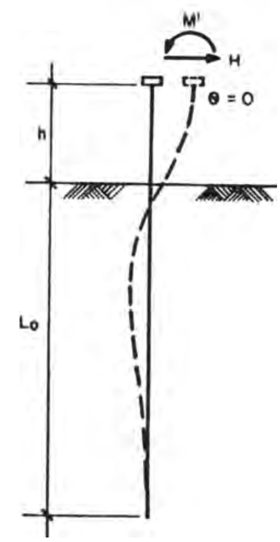


Fig. 5.30 b)

Step 3): Recomputation of moments of the pile in soil with above moments and forces (values α and β from Table 5.11):

L	=	38.50 m	L/L_o	2.8	=>	3
L_o	=	13.59 m				
H	=	326 kN	$H_o \cdot L_o = H \cdot L_o$		=	4430 kN
M	=	- 3306 kNm				
h	=	4.00 m	$M_o = H_o \cdot h + M$		=	- 2002 kNm

$M(z) = \alpha\ (z/L) \times H_o \cdot L_o + \beta\ (z/L) \times M_o$

z =	z/L =	M(z) =
0.00 m	0.0	- 2002 kNm
3.85 m	0.1	- 746 kNm
7.70 m	0.2	139 kNm
11.55 m	0.3	592 kNm
15.40 m	0.4	771 kNm
19.25 m	0.5	713 kNm
23.10 m	0.6	530 kNm
26.95 m	0.7	376 kNm
30.80 m	0.8	121 kNm
34.65 m	0.9	- 20 kNm
38.50 m	1.0	0 kNm

Table 5.21

c) Calculation with rotation at fixed pile head ($\theta \pm 0$) (different settlements, most exact model):

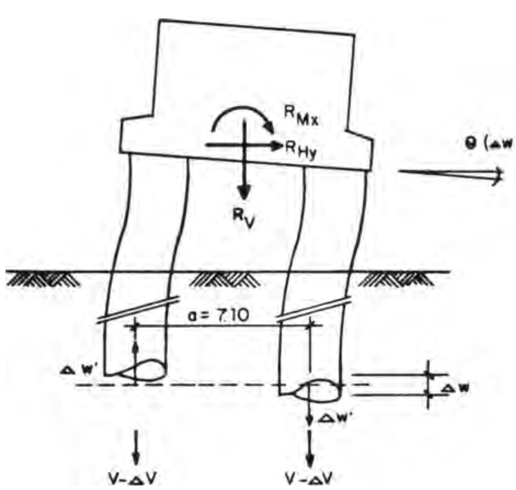


Fig. 5.31

Calculation of different settlements due to $\Delta V = \pm 347$kN/pile

Total displacement at pile foot (see Chapter 9.5.4)

$$\Delta w' = \frac{\Delta V}{V_{eff}} \cdot w\ (V_{eff}) = \frac{347}{6359} \cdot 6 = 0.33 \text{ mm}$$

$$\Delta w = 2 \cdot \Delta w' = 0.66 \text{ mm}$$

Angle of pile head rotation:

$$\theta(\Delta W) = \frac{0.66}{7100} = 9.30 \cdot 10^{-5} \text{ rad}$$

Head moment M" for $\theta(\Delta W) = 9.30 \cdot 10^{-5}$ rad (compare step 2 of b)

with $\theta(H) - \theta(M") = \theta(\Delta W)$

=> $\theta(M") = \theta(H) - \theta(\Delta W)$

$$4.21 \cdot 10^{-7} \cdot M" = 1.39 \cdot 10^{-3} - 9.30 \cdot 10^{-5}$$

$$M" = \frac{1.39 \cdot 10^{-3} - 9.30 \cdot 10^{-5}}{4.21 \cdot 10^{-7}} = 3084 \text{ kNm/pile}$$

Recomputation of moments of the pile in soil with moments and forces for case 3 with pile head rotation:

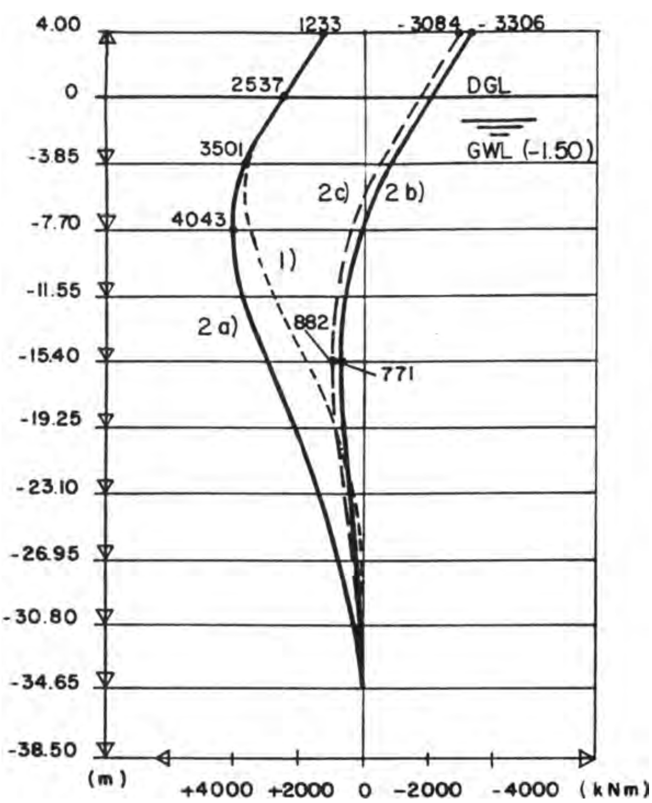
L = 38.50 m	L/L_o	2.8	=>	3
L_o = 13.59 m				
H = 326 kN	$H_o \cdot L_o = H \cdot L_o$		=	4430 kN
M = - 3084 kNm				
h = 4.00 m	M	$= H_o \cdot h + M_o$	=	- 1780 kNm

$M(z) = \alpha\ (z/L) \times H_o \cdot L_o + \beta\ (z/L) \times M_o$

z =	z/L =	M(z) =
0.00 m	0.0	- 1780 kNm
3.85 m	0.1	- 530 kNm
7.70 m	0.2	330 kNm
11.55 m	0.3	748 kNm
15.40 m	0.4	882 kNm
19.25 m	0.5	786 kNm
23.10 m	0.6	574 kNm
26.95 m	0.7	398 kNm
30.80 m	0.8	133 kNm
34.65 m	0.9	- 18 kNm
38.50 m	1.0	0 kNm

Table 5.22

d) Summary of Moment Distribution:



"free head"

=> 1a) relevant moments for longitudinal direction

(one single pile with fixed head)

=> 2a) upper extreme case for transverse direction (two piles with hinged head)

fixed head, no rotation of cap

=> 2b) lower extreme case for transverse direction (two piles with fixed head)

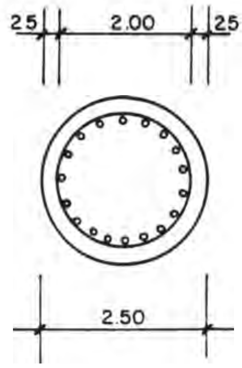
fixed head, with rotation of cap

=> 2c) most exact model for transverse direction (two piles with fixed head + different settlement)

Fig. 5.32

5.9.7 Stresses and Reinforcement

Geometry



Design diameter: $D = 2.50$ m

Distance between centre of main bars to pile edge: $h' = 0.25$ m

A large concrete cover has been chosen to ensure the minimum cover of 0.15 m even with the tolerances of execution.

Fig. 5.33

Material Properties and Admissible Stresses

- Concrete:

Cement 350 kg/m3

. ultimate strength:		$f_{c, min} = 25$ N/mm^2 (Concrete of samples)
. admissible stress:	- axial compression:	$\sigma_{c, adm} = 5.00$ N/mm^2
	- bending, regular loads:	$\sigma_{c, adm} = 6.25$ N/mm^2
	- bending, exceptional loads:	$\sigma_{c, adm} = 8.75$ N/mm^2
	- shear:	$\sigma_{c, adm} = 2.50$ N/mm^2

Because there was no test of the pile concrete quality, the admissible stresses cannot be increased.

- Steel:

Deformed bars SD-30 (Japanese Type)

- yield stress:		$f_y = 300$ N/mm^2
- admissible stress:	- bending, regular loads:	$\sigma_{s, adm} = 180$ N/mm^2
	- bending, exceptional loads:	$\sigma_{s, adm} = 225$ N/mm^2

Check of Bending Stress

Generally several cases must be checked because the worst one is not evident due to the interaction of moment and vertical force.

1) Case 2a: Dead Load + Earthquake in transverse direction, "free" head

Relevant depth: $t = -7.70$ m

$M_{max} = 4043$ kNm/pile

$V (t = -7.70 \text{ m}) = 2708 + 4.91 \cdot (24 \cdot 5.50 + 14 \cdot 6.20) = 3782$ kN/pile

$$e = \frac{M_{max}}{V} = \frac{4043}{3782} = 1.07 \text{ m}$$

$$\text{ratio} \quad e/R = \frac{1.07}{1.25} = 0.86 > 0.30 \quad (\text{---> great eccentricity})$$

$$\text{and} \quad d/D = \frac{2.00}{2.50} = 0.80$$

with minimum reinforcement of $u_{min} = 0.40$ % (19'640 mm^2)

The effective stresses can be computed with table 5.12 c) to:
(interpolation for e/R between 0.80 and 0.90)

$$\sigma_c = c \cdot \frac{V}{R^2} = 2.33 \cdot \frac{3782}{1.25^2} \cdot 10^{-3} = 5.7 \ N/mm^2 < \sigma_c, adm = 8.75 \ N/mm^2$$

$$\sigma_s = \gamma \cdot \sigma_c = 13.29 \cdot 5.7 = 76 \ N/mm^2 < \sigma_s, adm = 225 \ N/mm^2$$

2) Case 2b: Dead Load + Earthquake in transverse (no rotation)
Relevant depth: t = + 4.00 (at pile head)

$$M_{max} = - \ 3'306 \ kNm/pile$$

$$V (t = 0) = 2'708 \ kN/pile$$

$$e = \frac{M_{max}}{V} = \frac{3'306}{2'708} = 1.22$$

$$\text{ratio } {}^{e}/R = \frac{1.22}{1.25} = 0.98 > 0.30 \ (\text{---> great eccentricity})$$

$$\text{and } {}^{d}/D = \frac{2.00}{2.50} = 0.80$$

with minimum reinforcement of u_{min} = 0.4 % (19'640 mm^2)

The effective stress can be computed with table 5.12 c) to:
(interpolation for $^e/R$ between 0.90 and 1.00)

$$\sigma_c = c \cdot \frac{V}{R^2} = 2.84 \ \frac{2'708}{1.25^2} = 4.9 \ N/mm^2 < \sigma_c, adm = 8.75 \ N/mm^2$$

$$\sigma_s = \gamma \cdot \sigma_c = 15.71 \cdot 4.9 = 77 \ N/mm^2 < \sigma_s, adm = 225 \ N/mm^2$$

Thus the minimum reinforcement is sufficient to cover the moments of all 3 cases a), b) and c).
40 bars of a diameter of 25 mm provides total area of 19'640 mm^2.

Check of Shear Stress

Relevant load case: Dead load + Earthquake (in both directions)
From the moment distribution of the pile (Fig. 5.32) it is obvious that the maximum shear force occurs in the protruding part. Thus

$$S_{max} = H = 326 \ kN/pile$$

$$\tau = \frac{S}{b' (D\text{-}2h')} = \frac{326 \cdot 10^3}{400 \ (2.500\text{-}2 \cdot 250)} = 0.41 \ N/mm^2 < \tau_{c, adm} = 2.50 \ N/mm^2$$

Shear reinforcement:
With a spacing between bindes of a = 400 mm, the required steel area is computed as follows:

$$A_{s,B\ req} = \frac{S \cdot a}{(D - 2h') \cdot \sigma_s, adm} = \frac{326 \cdot 10^3 \cdot 400}{(2500 - 2 \cdot 250) \cdot 225} = 290 \ mm^2$$

With binders of a diameter 14 mm, the area is

$$A_{s,B\ req} = 2 \cdot \frac{14^2}{4} = 308 \ mm^2$$

At the pile head the spacing is reduced to a' = 200 m over a length of 2.50 m.

Computation with Limit State Method

This method is an alternative approach to design the reinforcement and to check the bearing capacity of the piles (see 5.7.1c).

The values or material properties for the calculation are:

concrete: $f_c = 0.7 \cdot 0.65 \cdot 25 = 11.4$ N/mm²

steel: $fy = 300$ N/mm²

required safety factor for earthquake: s = 1.20 for case 2b the factors are:

$$\frac{s \cdot M}{Ac \cdot D \cdot f_c} = \frac{1.2 \cdot 3306}{4.91 \cdot 2.5 \cdot 11.4} \cdot 10^{-3} = 0.028$$

$$\frac{s \cdot V}{Ac \cdot f_c} = \frac{1.2 \cdot 2708}{4.91 \cdot 11.4} \cdot 10^{-3} = 0.058$$

from diagram Table 5.13 for h'/D = 0.25/2.50 = 0.10 : w = 0.02

Therefore $A_s = w \cdot A_c \cdot \frac{f_c}{f_y} = 0.02 \cdot 4.91 \cdot \frac{11.4}{300} \cdot 10^6 = 3'732 \text{ mm}^2$

Thus $u_{min} = 0.4$ % with $A_s = 19'640 \text{ mm}^2$ is sufficient.

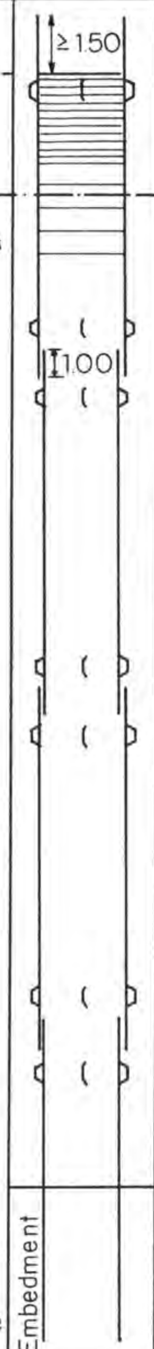
5.9.8 Representation of Results

As stated in chapter 5.8 important data and relevant results of pile design are compiled in standard from as per attached sheet.

Construction Site: Pathwe Bridge **Pile No:** 3

Reference Level 0.00 = D.G.L. (m)	Depth below Ground Level (m)	Soil, Ground Water, Pile Remarks
+4.00	+ 2.50	
+ 1.50	±0.00	
±0.00 D.G.L.	- 1.50	G.W.L.
-3.70	-5.20	soft silt & clay
-20.50	-22.00	medium dense sand
-25.00	-26.50	soft silt
- 32.70	-34.20	medium dense silt
- 35.70	-37.20	dense sandy silt
- 38.50	- 40.00	very dense sand
-40.20	- 41.70	

Pile Reinforcement



Pile Data

D.G.L. (above see level) .... m
Design diameter 2.50 m
Total length 42.50 m
Length below ground level 40.00 m
Embedment length in bearing layer 5.80 m

Concrete:
Cement 350 kg/m3
Water-Cement ratio <0.55
Minimum strength 25 N/mm2
(28 days cube or cylinder)

Reinforcement:
Type of steel SD 30
Main bars Ø 25 No. 40
Binders Ø 14 a = 400 mm
& at topmost Ø 14 a = 200 mm

Overlapping 1.00 m
Free end at top 1.50 m
Concrete cover 250 mm
Spacers 2x4 per cage

Piling Works:
Drilling diameter 2.5 m
Excavation (normal) 196 m3
Filling (bentonite) 196 m3
Concrete 209 m3
Depth below G.L. 40.0 m

Piling equipment:
Link - Belt LS 108 / Soilmec RT 3/C

Bentonite pumping equipment:
Jonio 100 pump / Soilmec BE - 10/ 50

Boring gear up to 44 m:
4 x 12 m Kelly bar

Scale: 1: 250
Date: 1.7.1986
Design Engineer: U Sann Win
Related Drawings: xxx
yyy

6 Design of Piers and Abutments

6.1 General

6.1.1 Introduction

The design principles for bored piles set out in the previous section include elements specific to the newly introduced technique. For piers and abutments no new elements have been brought in by the project and Public Works has, for many years, been designing adequate structures.. Accordingly, this section is included only as a design checklist including items specific to deep foundations with bored piles.

6.1.2 Checklist for Design Loads

Piers and, in many cases, abutments are monolithic with piles and many design parameters will therefore be common and interactive.

For design of bridges, each of the following loads should be considered:

a) Dead load of super- and substructure

b) Live load on the carriageway
   (i) on all spans simultaneously
   (ii) live load on one adjacent span

c) Wind load
   (i) on the superstructure
   (ii) on each pier

d) Earth pressure

e) Water pressure and uplift

f) Earthquake load, in this case a factor of dead load x 0.12 has been used.

g) Horizontal component of live load (braking forces)

h) Friction of bearings
   (i) Impact load of ship collision or truck crash

When compiling the load combinations to compute maximum forces on the structure, thought should be given to the probability of occurance of these combinations. For example, the combination of maximum wind and earthquake can be excluded whilst the combination of wind and impact load is quite probable.

6.2 Abutments

6.2.1 General

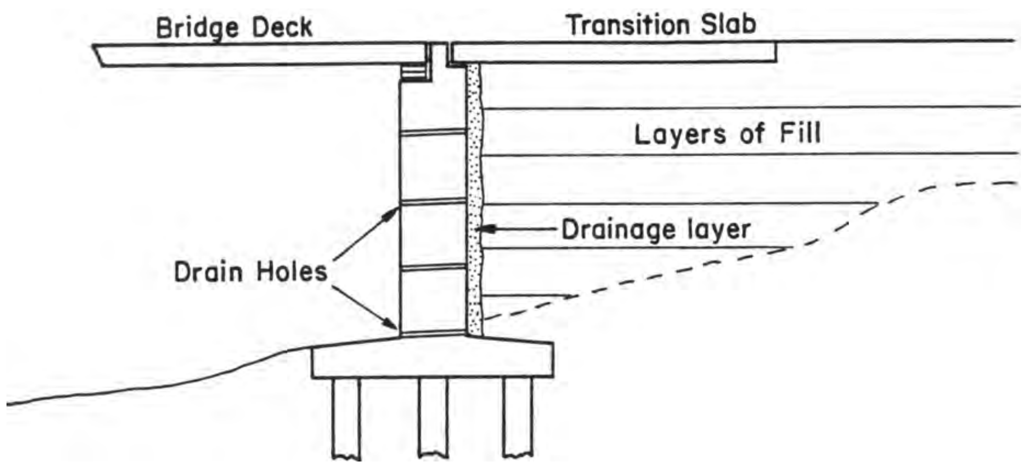


Fig. 6.1

Fig. 6.1 represents a typical abutment. There are three areas which have caused concern at bridges examined during the project and which are treated in more detail;

- drainage of retained material,
- compaction/settlement of fill
- stability of wing walls

6.2.2 Drainage

It is unnecessary to design an abutment to retain the earth pressure of wet backfill but design and construction must ensure a properly functioning drainage system is installed. This must cover both collection of surface and discharge of ground water.

Collection of water held in the soil pores of the backfill behind the abutment should not be difficult if the backfill material is granular. In areas of the delta, good backfill material may not be too easy to find and then even more care is required to ensure adequate drainage is installed. A thin layer of granular material or a few perforated pipes will almost never be acceptable and the design needs proper specification and then careful site supervision. The drainage system must work throughout the life of the bridge and migration of silt and clay particles must be prevented from blocking the percolation capacity of the layer. Two graded bands, installed carefully during backfilling will generally be required.

In a related way, the distribution of drain outlet holes must be sufficient to ensure that the flow path from any point of fill is short and related to the quality of the drainage layer installed. This must ensure that migration of fine material does not prevent the whole free draining.

6.2.3 Settlement of Backfill

The composition and settlement of fill near the abutment is clearly of concern for the road pavement and the bridge engineer. Any settlements of the fill adjacent to the abutment results in impact loading at the abutment. Such loading may be avoided through the construction of a transition slab and by proper fill compaction.

The problem of incomplete compaction is particulary difficult in clayey soils as found in the delta. It can be resolved only by special material selection and by subsequent supervision to ensure fill is laid and subsequently compacted in shallow layers. Maximum layer depths of 0.3 m of selected material, with appropriate compaction and with the correct and checked moisture content should enable real problems to be avoided.

The relatively simple alternative open to the bridge engineer not certain that abutment backfill will be adaquately compacted and who requires to avoid a step in the road surface due to differential settlements is to design a transition slab as an integral part of the bridge structure.

6.2.4 Wing Walls

The purpose of wing walls is to retain the end of the embankment near the abutment and to ensure that the abutment is not eroded from the side or rear. It may be observed that wing walls have, on these criteria, failed to function satisfactorily in numerous bridges and culverts in the delta.

Wing walls are properly regarded as an integral part of bridge design, requiring adequate foundations and to be permanently attached to the abutment. In some cases, this may require additional piling to bear the significant loads on the wing wall and may make minor construction difficulties to ensure adaquate bonding of elements constructed at different times.

There will be many occasions when the front face of the abutment is required very quickly and construction of the wing section can only follow. Experience shows that to retain the integral nature of the structure does not require significantly complicated detailing of reinforcing steel or even brickwork and neither does it delay the construction . For truss bridges on piles, where concrete abutments will be normal, simple key sections and lapped steel reinforcement as part of the construction joint should become standard features.

6.3 Piers

6.3.1 Pile and Pier Connection

The principles governing the design of pile to pier connection may be summarised as follows:

- minimum dimensions are as per the sketch below
- main bars of the pile must be anchored into the footing to a length of 65 x the diameter of the main pile reinforcement
- at the pile top the spacing of the binders must be small due to the main shear stress. That is a maximum spacing of 250 mm for a length of D below the footing - the concrete cover must not be less than 50 mm for section with standpipe and 150mm for sections in soil without casing.

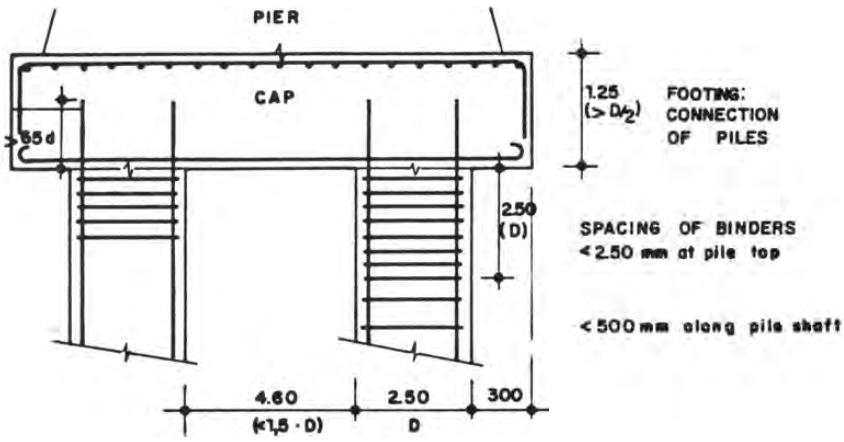


Fig 6.2

6.3.2 Calculation of Impact Loads

Generally the most severe case is a ship collision with a pier:

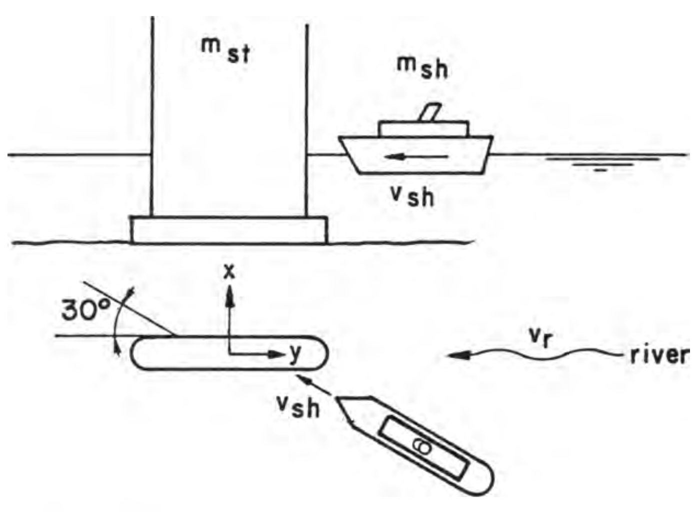


Fig 6.3

Where:

M_{sh} : mass of ship

M_{st} : mass of structure: $M_{st} = W_{st} / g$, wher g = 9.81 m / s and

W_{st} : dead load of pier and superstructure

v_{sh} : velocity of ship. For rivers, take $v = v_{sh} + v$ and note that both wind and tide may increase the relative velocity of the ship.

E J : stiffness of the structure

(Should the inertia of the pier and pile be different, the stiffer inertia can be taken to calculate a conservative value of H.)

Note that the moment of inertia must be calculated in the direction of the impact (eg 30°).

The crash force H can be calculated using an energy balance before and immediately after collision :

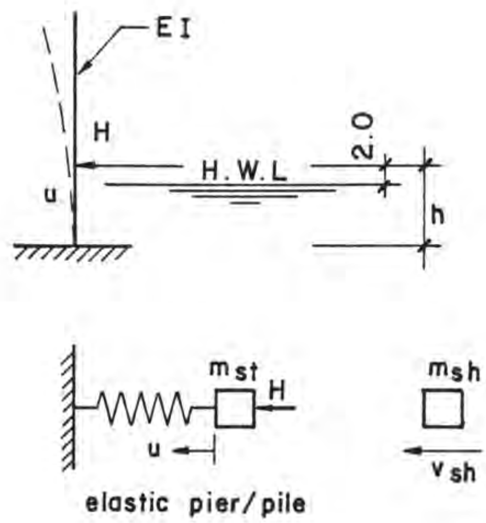


Fig 6.4

Before collision:
kinetic energy of the ship is
(considering the different masses of ship and pier)

$$E_k = \frac{M_{Sh} \cdot v_{Sh}^2}{2}$$

After collision:
with the displacement u the potential energy of the pier is

$$E_p = \frac{H \cdot u}{2}$$

where $u = \frac{H \cdot h^3}{3 \cdot EJ}$

Energy balance: $E_k = E_p = \frac{H^2 \cdot l^3}{6 \cdot EJ}$

Thus the crash force is $H = \sqrt{\frac{6\ E_k \cdot EJ}{h^3}}$

Note: - the height h of impact above foundation level is generally taken as 2 m above high water level although for large ships it may be more
- for flat foundations, h is the distance of the impact from the foundation (Fig. 6.5 a)
- for deep foundations with piles, it is assumed that the pile is fixed at the depth of its characteristic length L_o (Fig.6.5 b). For calculation of L_o see Section 5.5.1.

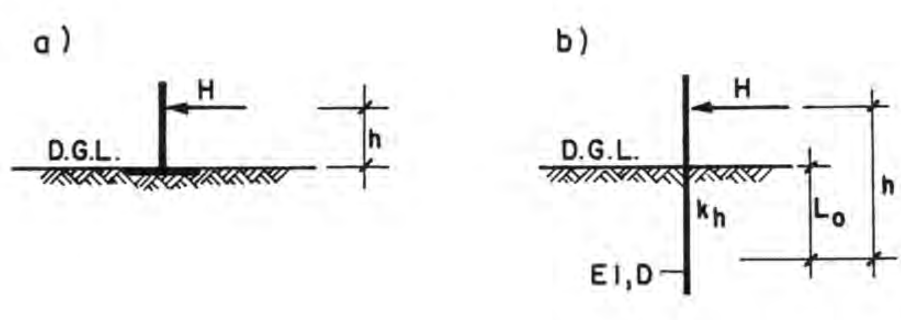


Fig. 6.5

The above permits the dynamic impact to be regarded as a pseudo static force able to be included in the pile and also in the pier design.

It is recommended to provide a flexible protection fence around the pier which is easy to repair. The deformation of the ship body and the fence is not considered. Thus the calculation of the crash force is on the safe side.

The same calculation method as indicated for ship collisions with a pier can be similarly applied for the determination of truck crash forces on piers.

Crash forces on truss members or rail guards attack approximatively 1.2 m above the road surface level in the direction of the impact which normally is parallel to the road axis. Some construction codes are directly indicating pseudo static loads, e.g. H = 1000 kN for heavy trucks.

7 Design of Bridge Steelwork

7.1 General

This part describes the design approach and summarises the calculation procedures followed in the design of the steelwork components of the bridge system. Stringers and cross girders included as part of the carriageway system are dealt with in section 8. Design has been carried out to the working stress (service load) provisions of the AASHTO Standard Specification for Highway Bridges, Thirteenth Edition 2 1983 (AASHTO). Steel grade used is Grade 50 to ASTM A572 (yield stress fy = 345 N/mm^2) except for minor members, e.g. bracings and railing which are Grade A36 to ASTM. M24 high strength bolts to ASTM A325, Type 1, in 26 mm diameter clearance holes are used for all member connections. Bolts are fully tensioned. The design considers bolts in bearing type mode.

The design is shown for the standard 36.5 m span used initially in the Yangon - Pathein Road Project. Emphasis in this section has been given to constructional details. The structural analysis, nowadays easily made by computer programme, is not provided in detail. Where design rules are given, they may be taken generally for the design of Warren type steel bridges with spans other than the standard from the project. AASHTO specifications used as the design basis are not generally reprinted unless necessary for clarification.

The main features to check in case the detailed design has been the responsibility of the steel bridge manufacturer, are:

- basic assumptions, e.g. all relevant load cases including erection, material strengths etc.
- analysis of layout including relevant details, e.g. space for erection tools
- structural analysis of truss

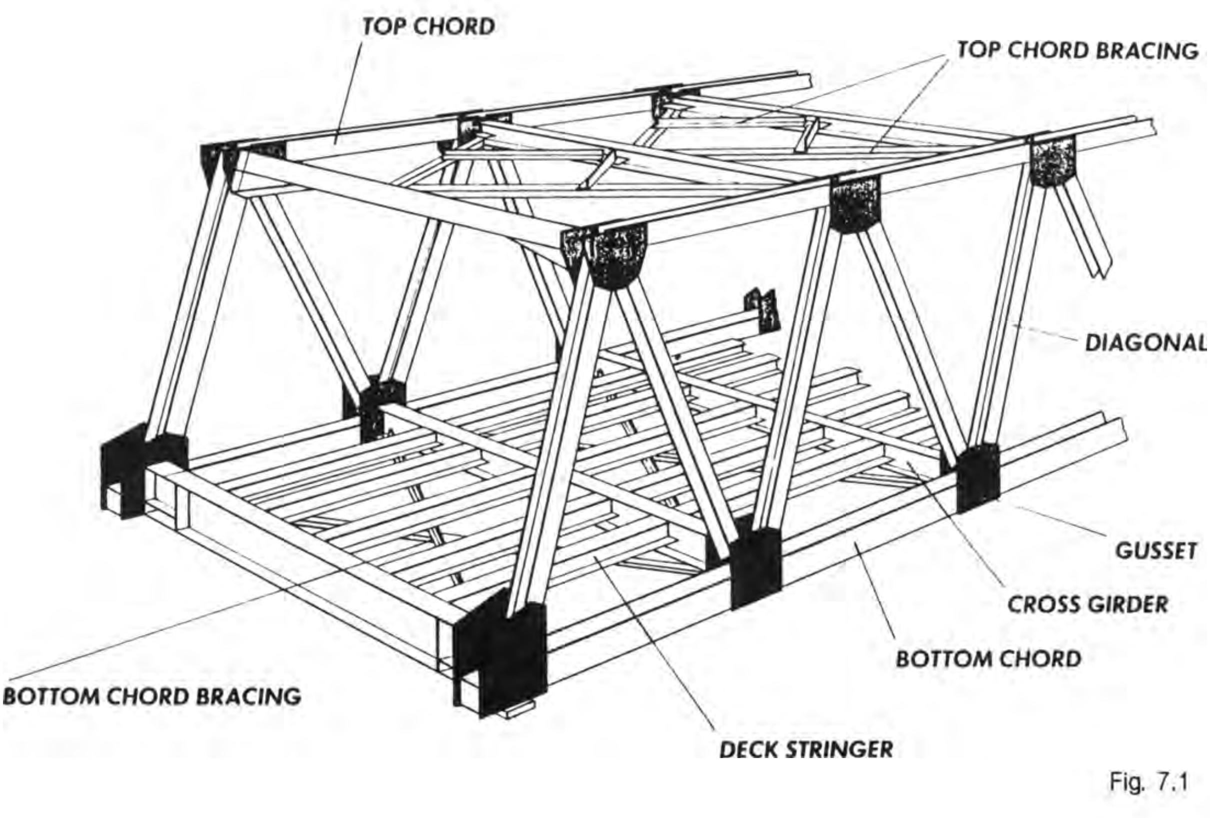
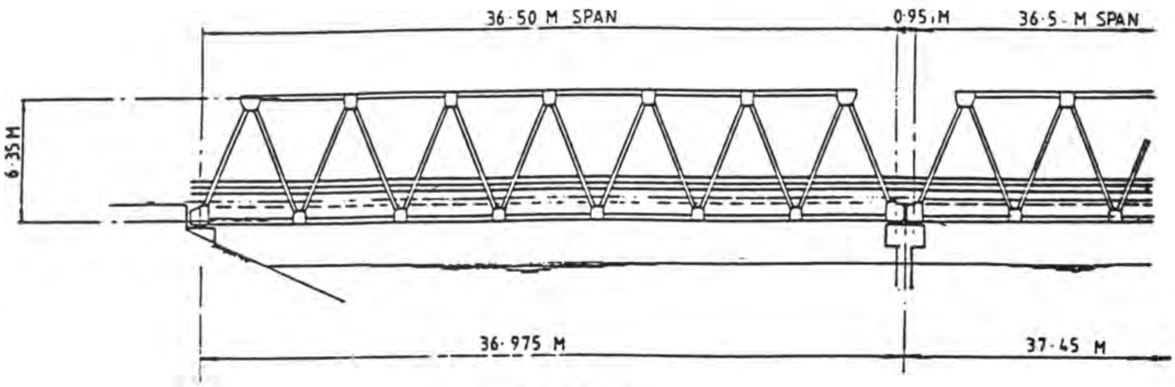
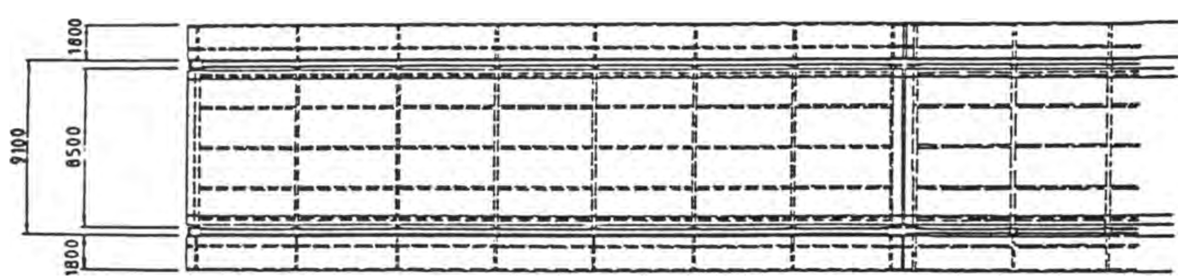


Fig. 7.1

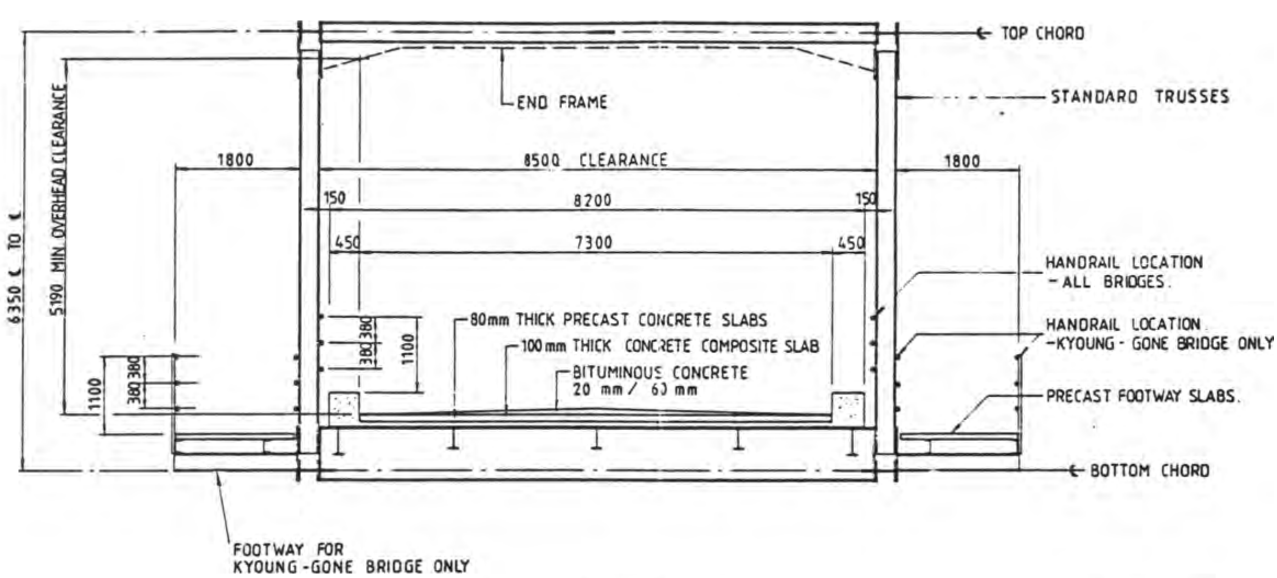
Example for General Arrangement of Steel Truss Bridge: Kyounggone Bridge with 4 Spans and Walkways



ELEVATION
Scale 1:200



PLAN
Scale 1·200



Typical Cross Section

Fig. 7.2

The bridges are initially detailed for 36.5 m span and 2 lanes of AASHTO vehicle loading HS20-44 in addition to the self weight of the steelwork and carriageway, see general arrangement of the bridges (Fig. 7.2). Allowance is made for 20% impact or live load in accordance with Article 3.8 of AASHTO and for falsework in the river. This method requires the use of a standard span as an anchor span and additional and earthquake loading equal to 12% of dead load. Provision is made for the spans to incorporate a cantilever footway each side of the trusses. Calculations described in this part are only for bridges without walkways, but may be extended by adding the dead load plus live load for the footways.

The design ensures that no component weighs more than 1.5 tonnes or exceeds 8.5 m in length. Assembly is designed to be by hand tools assisted by a light mobile crane in the store and at the bridge site.

Fig 7.1 illustrates the steel truss arrangement. The system has been designed for piece-by-piece cantilever erection avoiding the need for link steelwork between the anchor span and the permanent span being erected. Other methods of erection, such as part cantilever or erection on falsework, are feasible.

7.2 Trusses

The trusses are a standard Warren truss configuration simply supported over 36.5 m nominal span. Top and bottom chord bracing systems transfer lateral loads to each end of the bridge. End diagonals and cross beams form a rigid end portal. Four pot-type bearings, one at each corner of the bridge, transfer loads to the substructure.

All truss components are of fabricated H-form. This has been adopted to avoid the problems associated with the wide tolerances on rolled sections. H sections are also very robust and efficient and the use of high technology mass production techniques enables precise control of quality and accuracy of components. Consequently, a large degree of interchangeability has been achieved in bridge systems.

Main truss members all have flange widths of 300 mm and a web depth of 300 mm. Thickness of flanges and webs vary according to function of the truss members.

Member load capacities are calculated in accordance with the working stress provisions given in Part C of AASHTO Section 10, Structural Steel.

7.2.1 Structural Analysis

Analysis of trusses assume conventional pin jointed connections. Maximum member forces for top and bottom chords and diagonals under dead and live loads are determined taking account of the worst case location of partial lane loading and knife edge loads. The HS20-44 lane loading is considered for the truss analysis. Loads are applied to the truss through the bottom chord panel points in determining truss member forces.

Maximum axial loads for all members obtained from the various load combinations specified in AASHTO Article 3.22 are as tabulated.

Load Case	Load Combination	Permissible Unit Stress
Group I	D + L + I	100 %
Group IA	D + 2 (L + I)	150 %
Group VII	D + E	133 %
I (Impact) = 20 % of L (Live load)		
E (Earthquake) = 12 % of D (Dead load)		

Table 7.1

7.2.2 Diagonals

Diagonal members are subject to some bending coincident with axial load due to the cross-girder loading through the internal gusset plate at each bottom chord panel point.
The bending to the diagonals is taken as P · e / 2,

where P = cross girder panel point load and
e = eccentricity of the inner gusset with the centreline of the truss .

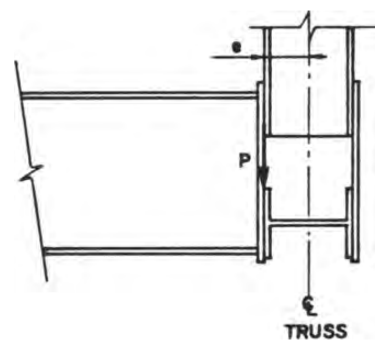


Fig. 7.3

The design considers both maximum axial load with corresponding bending and maximum bending with corresponding axial load in the determination of the diagonal member sizes. An effective length factor of 0.75 is similarly assumed for compression diagonals for both bending and compression.

End diagonals form part of the end portal frame and are considered separately in section 7.4

7.2.3 Top Chords

Top chord compression members are designed assuming an effective length factor of 0.75 for minor axis buckling and accounting for reduced effective section in compression due to plate buckling as given in AASHTO Article 10.35.2. A comprehensive buckling analysis of the top chord bracing system performed at the University of Sydney has previously verified that minor axis buckling of the top chord compression members is the critical buckling mode for design.

7.2.4 Bottom Chords

Bottom chords in tension consider reduced net areas accounting for bolt holes where applicable in accordance with AASHTO Article 10.18.4.

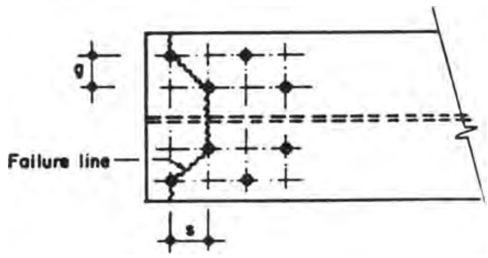


Fig 7.4

The capacities of internal diagonal members are calculated for both tension and compression combined with (P x e / 2) bending by considering the combined stress expressions provided in AASHTO Article 10.36.

For fatigue considerations, chord and diagonal members are classified as Category B in accordance with AASHTO Article 10.3. The fatigue design life of a million cycles then limits the permissible range of stress undergone by the truss members to 110 N/mm^2. This was found by calculation to be satisfactory in all cases.

7.2.5 Connections

Truss members are field bolted using inner and outer gusset plates at each panel point. In addition, top and bottom chord members have flange and web splice plates as required to achieve the required force transfer. The bolting arrangement for a typical truss bottom chord connection are as illustrated in Fig 7.6. Top chords are similar.

Gusset and splice plates are designed in accordance with AASHTO provisions for tension and compression members.

The number of bolts is determined by the forces to be transferred assuming bearing mode type failure of the bolts. It is noted that the bolts are tensioned for improved joint performance and to avoid the possibility of nuts working loose. Bolt capacities are determined in accordance with AASHTO Article 10.32.3 Bolt lengths have been determined such that threads are excluded from the shear plane for bolts in single shear. Bolts in double shear are assumed to have at least one shear plane with threads included.

M24 High Strength Bolts					
Shear (kN)		Bearing (kN)			
Threads included in shear plane	Threads excluded from shear plane	Plate thickness (mm) for f_y = 345 N/mm^2			
		6	8	10	12
61	84	87	117	146	175

Table 7.2

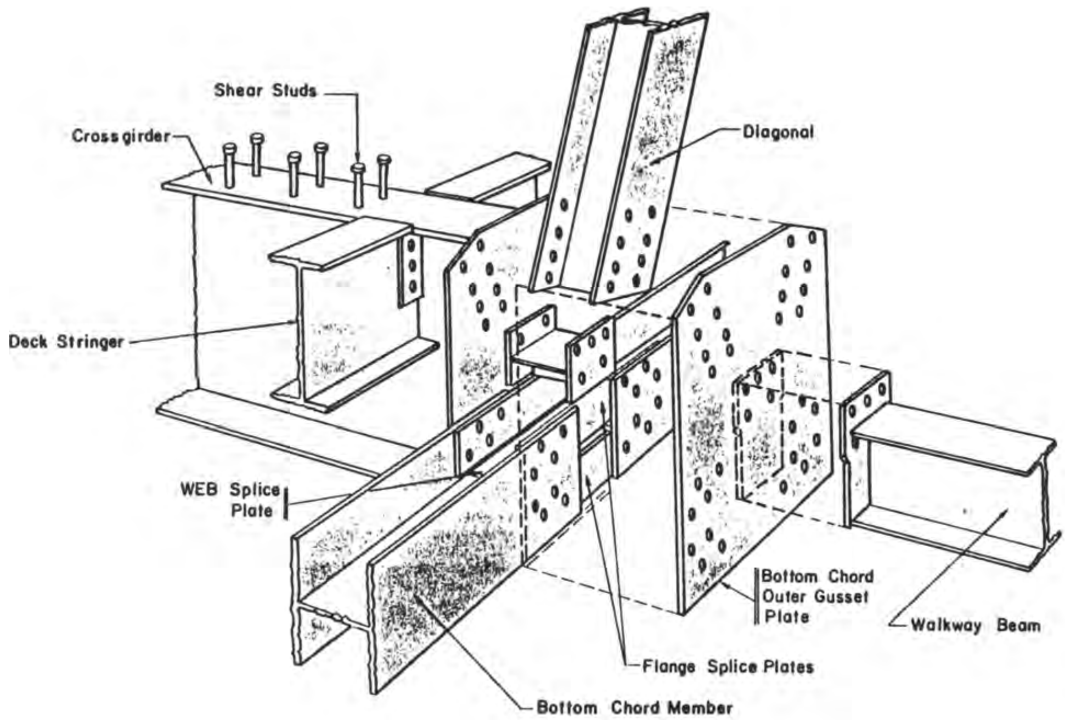


Fig. 7.5

End panel point connections to the end diagonals from part of the end frame system, and these are considered separately, refer to section 7.4

Camber is achieved by varying the spacing of the holes for the chord connection to the top and bottom gusset plates. All internal chord members are of the same length, top and bottom. Allowance for partial bolt slip or initial bolt displacement in the clearance holes during assembly of 0.5 mm per bolt has been included in the determination of deflections. The length of the truss members has been designed such that residual camber remaining "as fabricated" after construction of the carriageway system and allowing for 0.5 mm bolt displacement is calculated to be 56 mm, or span/650.

7.3 Top Chord Bracing

The top chord bracing system provides in-plane stiffness to the top chord of the bridge truss and extensive analysis has shown that buckling of the compression chords about the minor axis is the critical buckling mode for design. In addition, lateral loads from wind and earthquake conditions are transferred by the top chord bracing system to the end frames and subsequently to the substructure.

Wind forces are given by AASHTO Art. 3.15 for a base wind velocity of 100 mph. This specifies a design design pressure of 75 lb/sq.ft (5.6 kN/m2) acting on the exposed area of the bridge structure. A total horizontal load of 35 kN uniformly distributed to the top chord bracing is calculated. Earthquake forces are taken as 12% of the dead load of the top chord system acting horizontally in any direction.

AASHTO does not provide any guidance on the load or stiffness requirements to be adopted for design of the restraining system to the top compression chord. However, the Australian Standard AS1250, Steel Structures Code, recommends that the lateral restraining system for a compression member be designed for a force equal to 0.25 times the axial load in the member being restrained. Furthermore, AS1250 requires that the transverse deflection of the restraining system be limited to .0025 times the span of the restrained member.

The top chord bracing system is analysed as a truss spanning between the end frames for the loads given above. Maximum design forces are obtained from the load combinations set out in AASHTO Article 3.22.

Member sizes are determined in accordance with the design provisions of AASHTO for the design forces derived from the load combinations.

7.4 End Frame

The end frame comprising the end diagonals, end cross beams and bearing assemblies provides a rigid ring to the end of the bridge for lateral restraint of the top chord system and transfer of all lateral forces to the substructure. Top chord bracing loads are resisted by frame action and transferred to the bearings. All connections are considered to be rigid and designed for the moments and forces resulting from the frame analysis.

End cross-girders composite with the deck are checked for bending from frame action in addition to the bending from carriageway loads described in section 8.3.

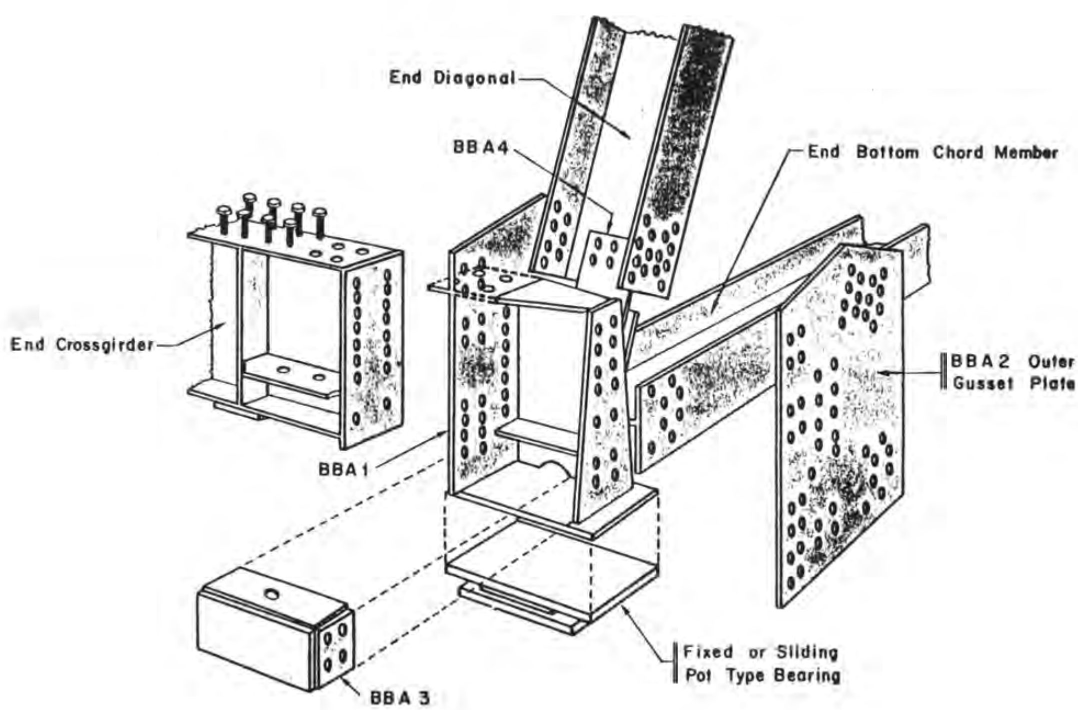


Fig. 7.6

End diagonals resist combined bending and compression. The top cross beam forms the rafter of the end frame in transferring top chord effects. All members are designed in accordance with the provisions of AASHTO Section 10, Part C, for their respective load effects.

The top chord end panel connection is illustrated and shows the detail used to achieve moment transfer between the top end cross beam and the end diagonal. The first internal diagonal is conservatively assumed not to make any contribution to the lateral stiffness at the end of the bridge.

The bearing assembly detail, Fig. 7.6, illustrates that all vertical and horizontal loads where applicable must be transferred by the assembly to the pot type bearing fixed to the underside and thence to the substructure. All the respective load actions have been considered in the design of the individual plates and bolt groups comprising the bearing assembly.

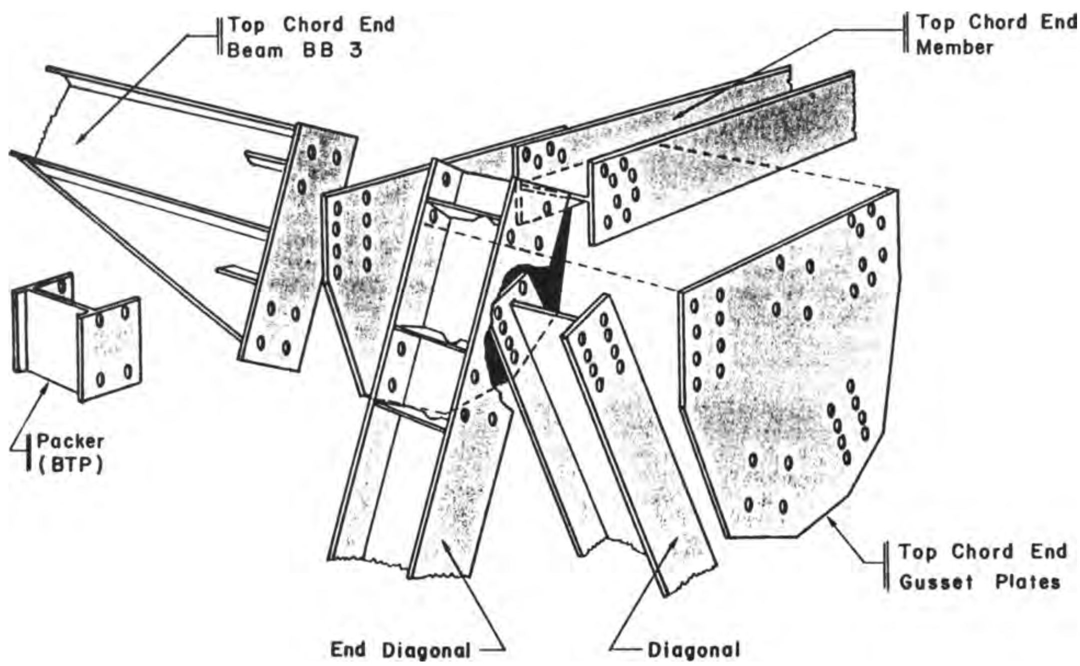


Fig. 7.7

7.5 Bottom Chord Bracing

In-plane stiffness to the bottom chord is provided by a fully cross-braced system at the level of the bottom chords. Wind and earthquake effects are assumed to be resisted by this system. In addition, lateral restraint to the bottom chord members in compression during cantilever erection is provided by the stiffness of the bottom chord bracing.

Upon completion of the deck concrete, the deck system comprising the deck slab in composite action with the internal stringers and cross-girders will in fact provide a rigid diaphragm stiffener to the bottom chord, and the angle bracing could be removed.

7.6 Cantilever Erection

During erection by cantilever, the top and bottom chord members undergo stress reversal from their designed conditions. Standard link steelwork must also be provided to connect the anchor truss assembled and the permanent span being erected. The various erection parts required to link the spans are as illustrated in Fig. 7.8:

- bottom chord link (gusset)
- top chord link (truss members)
- top chord bracings

The anchor span is erected with bolts snug tightened only. Similarly, link members should have bolts snug tight only to enable subsequent dismantling and re-use of the parts.

Analysis of the cantilever truss, anchor truss and link steelwork as a single unit provides the design loads for truss members and link steelwork. All steel sections gave to be checked such that their bearing capacity is also sufficient for this load case. For Kyounggone bridge the bottom chord members immediately adjacent to the forward support for both spans are found to require strengthening. This is achieved by the addition of reinforcing angles bolted each side of the bottom chord. The remainder of truss chords and diagonals are found to the satisfactory.

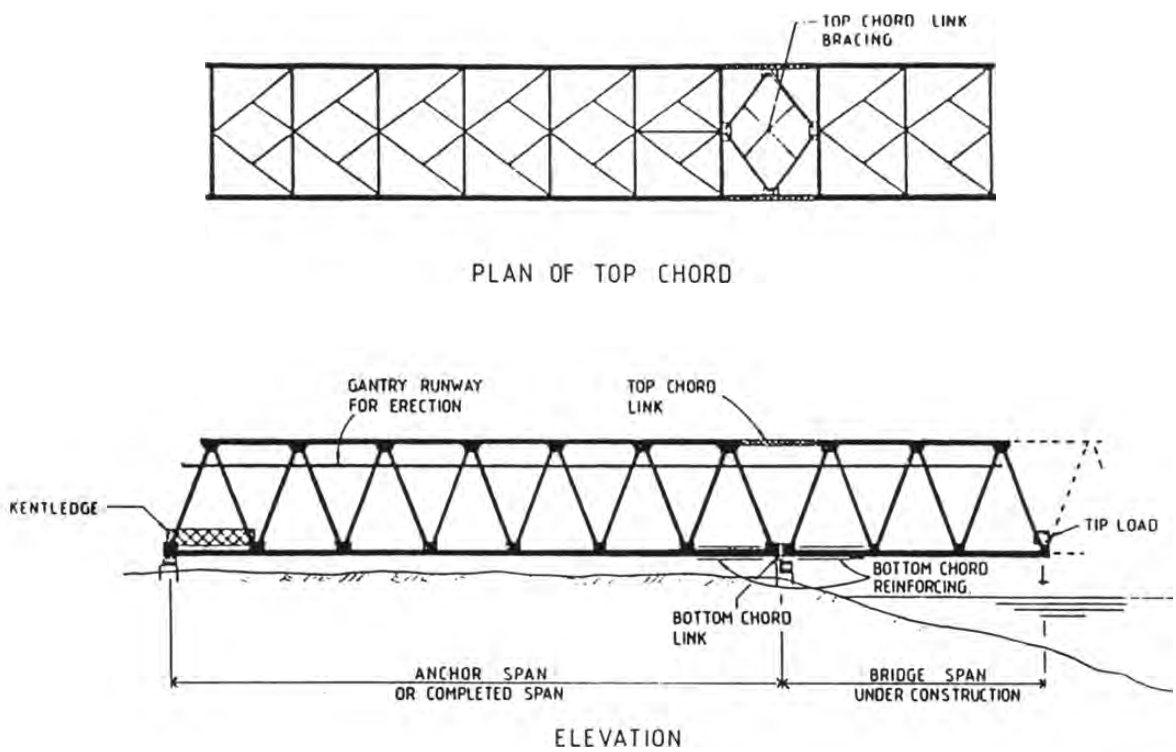


Fig. 7.8

Top and bottom chord links, top chord bracing link and bottom chord bracing link are all designed for the forces derived from the above analysis. The overall stability against overturning must be calculated considering both dead load and construction load , particularly loads at the tip arising from the erection equipment. A safety factor of 1.3 should be used. In order to fulfill this condition, it is normal to place a kentledge load at the rear end of the anchor span consisting of concrete panels or other easily available load.

A kentledge load of 8.5 tonnes stacked at the rear of the anchor span was calculated to satisfy overturning requirements at Kyounggone, including the factor of safety, but, in practice, the site supervision decided upon erection without installation of stringers would provide a safe and more efficient procedure in this particular case.

In addition it is necessary to calculate the deflection of the tip at full cantilever in order to define the required elevation at the launching pier, see also section 13.3.4. In the case of the first bridges erected, permissible tip deflection at full cantilever was calculated as 190 mm and allowance for this proved just acceptable during erection.

7.7 Example

The example shows the results calculated for a steel truss bridge without walkway, e.g. as erected at Pathwe or Inma.

7.7.1 Structural Analysis

Calculation of the load combinations as described in section 7.2.1 gives the results as indicated in Fig 7.9. Loads are given in kN and reduced to 100% basic unit stress, as given in Table 7.1.

Key: Figures in [kN] with " - " for compression and
"+" for tension

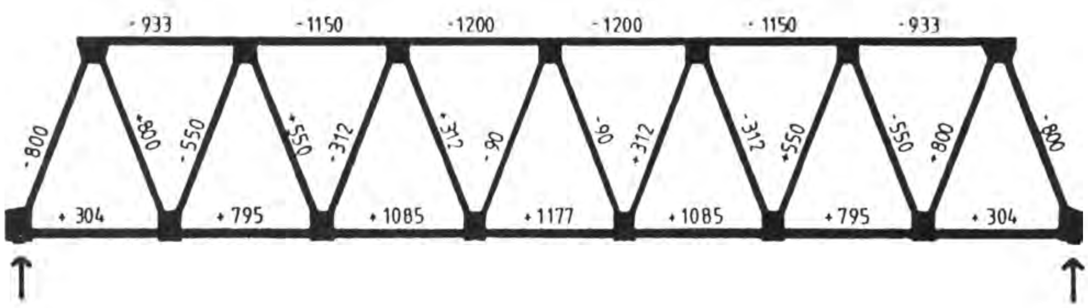


Fig. 7.9

7.7.2 Truss Members Load Capacities

Based upon the effective lengths calculated as outlined in 7.2.2 and 7.2.3, the following load capacities of the truss members have been calculated:

Truss Member Load Capacities		
Member		Axial capacity
Top chord	BC2A	- 980
	BC3	- 1740
Bottom	BC1	+ 1115
	BC2	+ 1400
Internal	BD1	- 540 / + 1165
Diagonals	BD2	- 775

Table 7.3

8 Design of Bridge Deck

8.1 General

The bridge deck system comprises precast, reinforced concrete panels bearing onto stringer beams and designed to act compositely with a cast-in-place topping slab. The precast panels are designed to support their own weight plus the in-situ kerb edging plus that of the topping slab and construction loads. Vehicular loads are carried by the composite slab.

Provision for roadway drainage is made through the inclusion of PVC pipes at intervals along the deck. Deck protection angles and joint seals are provided at each end of the deck.

8.1.1 Loading Assumptions

Design has been carried out to the working stress (service load) provisions of the AASHTO Standard Specification for Highway Bridges, Thirteenth Edition, 1983 (AASHTO). The design vehicle for the carriageway system is the Standard AASHTO HS 20-44 Truck Loading, see the copy from the AASHTO Standard (Fig. 8.1), for conversion of units see section 2.2. Allowance for impact and earthquake loading has been made.

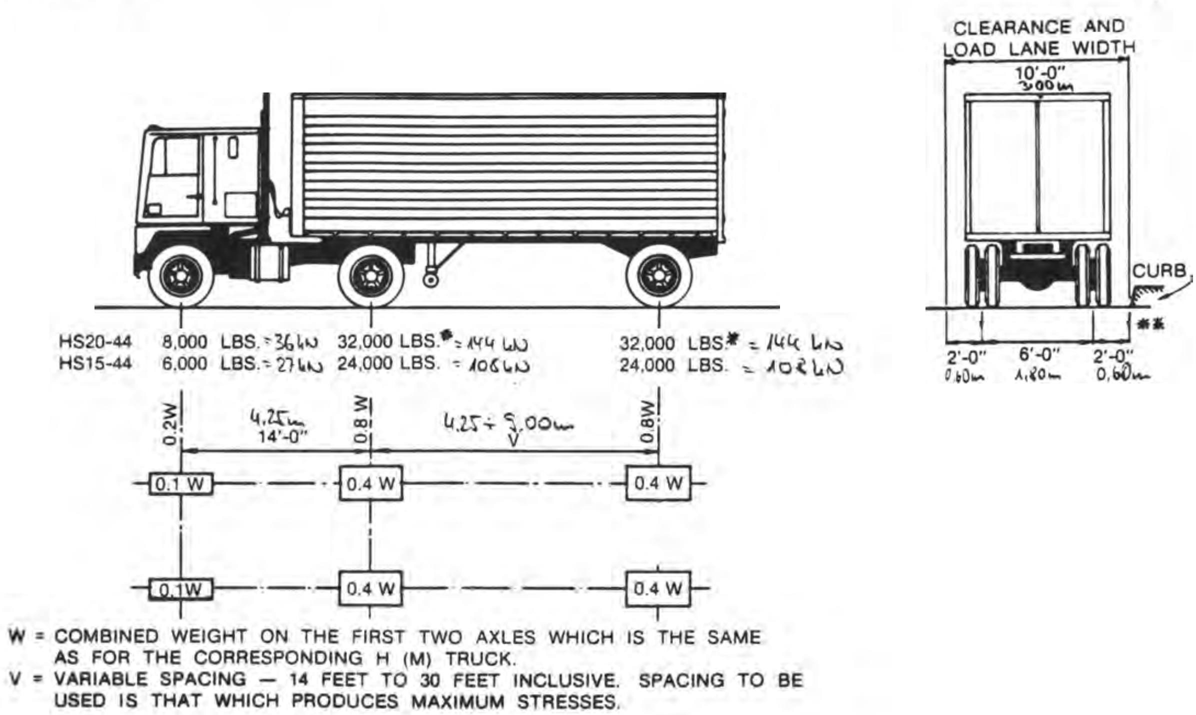


Figure 3.7.7A. Standard HS Trucks

*In the design of timber floors and orthotropic steel decks (excluding transverse beams) for HS 20 loading, one axle load of 24,000 pounds or two axle loads of 16,000 pounds each, spaced 4 feet apart may be used, whichever produces the greater stress, instead of the 32,000-pound axle shown.

**For slab design, the center line of wheels shall be assumed to be 1 foot from face of curb. (See Article 3.24.2).

Fig. 8.1

8.1.2 Material Properties

Material assumptions are the following :

Structural Steel (Grade 50, ASTM A572)	Yield stress	f_y = 345 N/mm^2
Reinforcement Steel (SD 30)	Yield stress	f_y = 275 N/mm^2
Concrete	Compressive strength	f_c = 25 N/mm^2
	Elastic Modulus	E_c = 22.2 kN/mm^2
	Density	γ_c = 23.5 kN/mm^3
Asphalt Concrete	Density	γ_a = 21.5 kN/mm^3

Table 8.1

8.2 Concrete Deck

8.2.1. Precast Panels

Reinforced precast panels span between stringers and act as formwork for the cast-in-place topping slab. They are also required to support construction loads prior to the slab behaving in a composite manner. Each panel is considered as simply supported between stringers. After fitting, the panels have to support construction loads including steelwork, concreting and other temporary weights.

The design loads for the initial bridges were :

Precast panels	80 mm thick	1.73 kN/m^2
Concrete top slab	100 mm thick	2.16 kN/m^2
Construction load		2.00 kN/m^2

All panels are reinforced with S16 bars and with 25 mm clear bottom cover. The outer panels are designed for a kerb edging to be cast with the slab.

It is considered that a 25% increase in permissible loads is acceptable under the combined action of the above loads for construction conditions. The effective span length is taken as the clear distance between supports plus the depth of panel, as given by AASHTO Article 8.8.

If the applied moment for the inner slab panel is less than the uncracked moment capacity of the section, provision of nominal reinforcement, central within the slab, is satisfactory. Stress in the reinforcement for the working load condition is negligible, however, the reinforcement is taken into account in the ultimate condition giving a load factor against collapse.

In similar manner, the outer panels are found to have a moment due to total load which is less critical for the internal panels.

The permanent deflection of the simply supported precast panels may be calculated from elastic theory since the sections remain uncracked. It is considered acceptable if the deflection is less than the span divided by 2400.

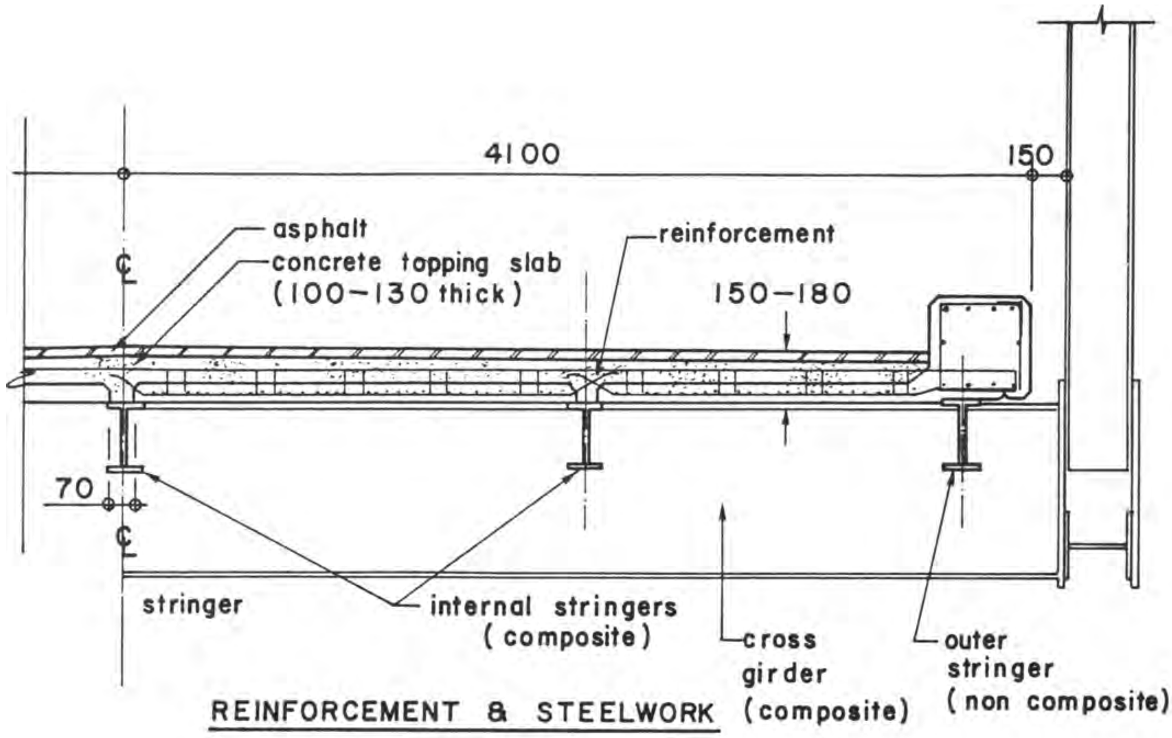


Fig. 8.2a

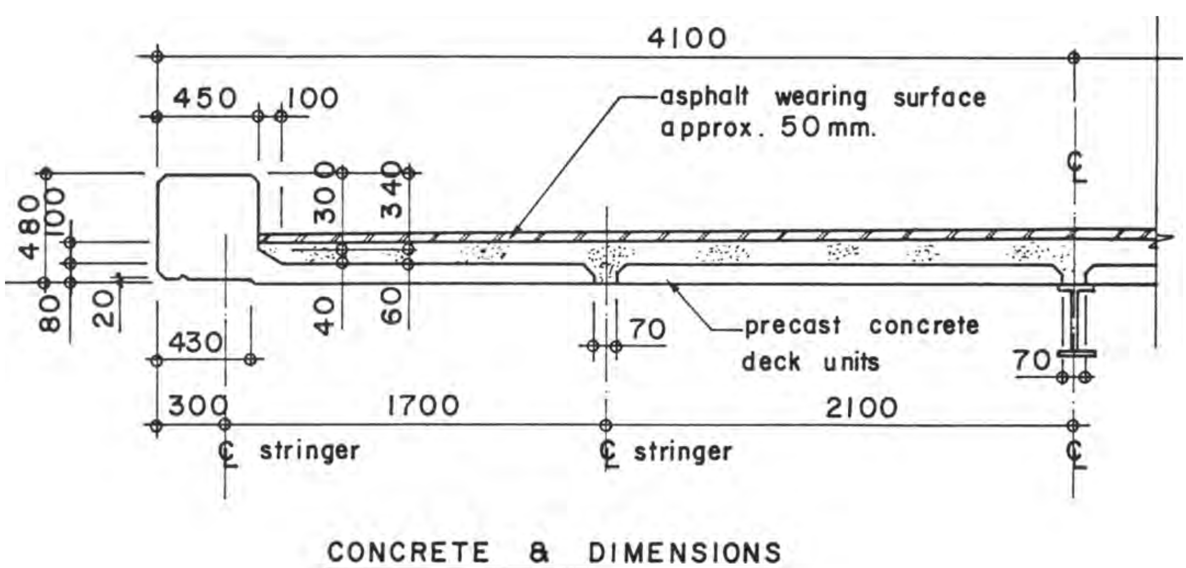


Fig. 8.2b

8.2.2 Composite Deck Slab

In addition to the self weight of the precast panels and topping slab carried by the panels, the composite deck slab is required to resist the imposed dead load of the asphalt surfacing and the vehicle live loads.

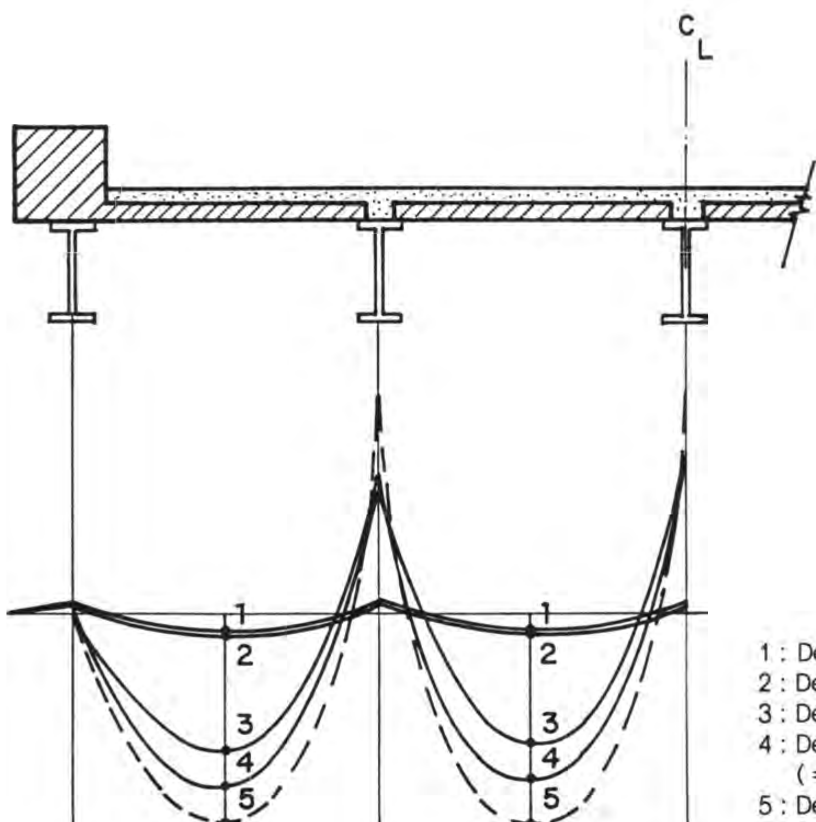
Design loads:
- Precast panels (80 mm)
- Topping slab (100 mm)
- Asphalt surfacing (55 mm max.)
- Earthquake load equal to 12% of total dead loads
- HS 20-44 Truck wheel load including 30% impact (Article 3.8 of AASHTO).

The relevant load combinations and allowable stresses (expressed as a percentage of basic unit stress) from Article 3.22 of AASHTO are the same as applied for the truss calculation (see Table 7.1).

The composite deck slab behaves as a continuous beam for imposed dead and live loads and earthquake loads.

Imposed dead load and earthquake effects are obtained from a continuous beam computer programme. The concrete self weight is resisted by the precast panels prior to the establishment of composite slab action and is not included in the continuous beam analysis.

Fig. 8.3 provides a qualitative summary of composite slab moments and combinations for deck bending :



1 : Dead Load
2 : Dead Load plus Earthquake
3 : Dead + Live Load
4 : Dead + Live Load + Impact
(= Group 1)
5 : Dead + (2x Live + Impact) /150%
(= Group 1A)

Fig. 8.3

Live load moments are derived using the load distribution formulae given in AASHTO Article 3.24 for concrete slabs with main reinforcement perpendicular to traffic; for impact a 30% increase is allowed. This is conservatively assumed to apply to each span for the design of top and bottom slab reinforcement.

Reinforcement requirements for these design moments are determined from the working stress provisions for reinforced concrete, Section 8 of AASHTO, resulting in the provision of S16 bars top and bottom. The bottom reinforcement is cast into the precast panels and projects at the ends for anchorage over the stringers to ensure continuity.

Note that dead load effects on the precast panels require nominal reinforcement only and the reinforcement determined above is considered satisfactory.

Article 3.24.4 notes that shear and bond in reinforced slabs designed using the live load distribution factors of Article 3.24 are deemed to be satisfactory.

Transfer of horizontal shear forces at the contact surface of the topping slab and precast panel is achieved through the provision of vertical tie bars cast in the precast panels. In addition, the upper surface of the precast panels is to be artificially roughened by brushing at the time of casting and is to be left free of laitance. Under these conditions AASTHO Article 8.15.5.5 limits shear stress to 1.10 N/mm2.

8.2.3 End of Deck

The leading edges of the carriageway at each end of the bridge extend beyond the end cross-girders. This section is designed to be fully cast-in-place as a reinforced concrete cantilever with conventional timber formwork.

Design is for self weight plus asphalt surfacing and HS 20-44 wheel loads plus impact. For wheel loadings, assume a contact area of 400 mm square and a further spread of 45 degrees to the centre line of the girder. The typical arrangement at the end of the deck is shown in Fig. 8.4:

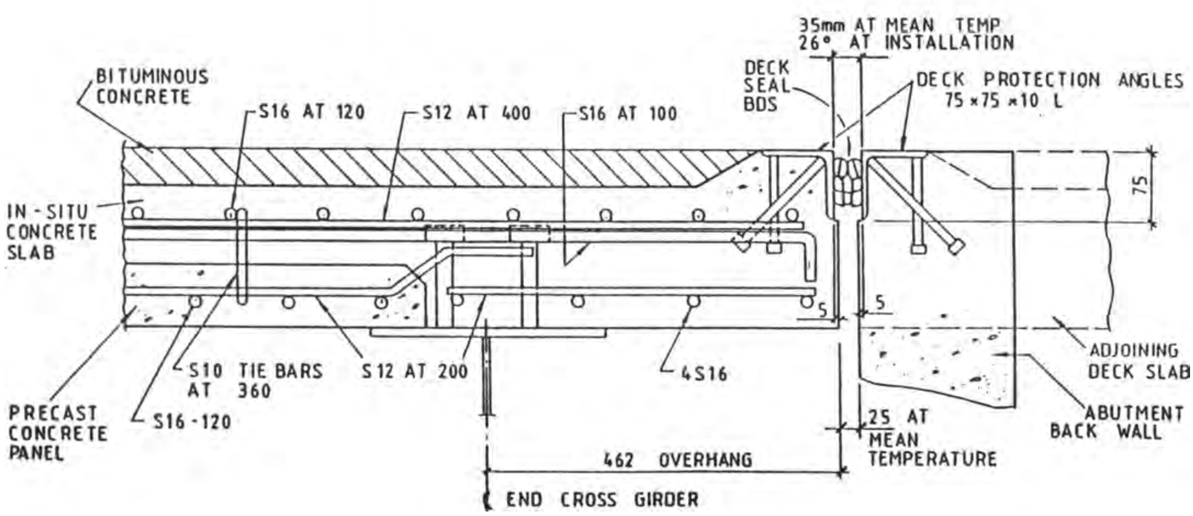


Fig. 8.4

The edges of the deck slab (and abutment backwalls) are provided with cast-in, mild steel protection angles, complete with stud anchors. Preformed cellular elastomeric deck seals are to be installed between the deck protection angles at the ends of each span to accommodate the range of movements due to load and temperature effects. The deck seals are installed at mean temperature and movements due to temperature variations of +/- 15 degrees have been taken into account.

In order to get the maximum and minimum expansion, the additional longitudinal movement at bearing corresponding to vertical deflection of span under load has to be considered (influence by concrete deck placement, live load, allowance for 0.5 mm per group bolt displacement).

Note that one end of the 36.5 m truss span is provided with fixed bearings and the full range of movement occurs at the sliding (free) end.

8.3 Stringer Beams

8.3.1 General

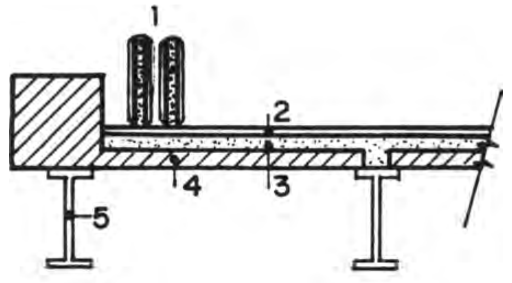
Stringer beams are initially required to support the precast panels plus topping slab and imposed contruction loads during concreting of the deck. Following completion of the concrete, further imposed dead and vehicular loads are required to be supported on the stringers assuming one-way action of the deck slab.

The AASHTO factors for the lateral distribution of wheel loads given in Article 3.23 are used to determine the proportion of the HS 20-44 axle load applied to each stringer. Impact allowance is determined in accordance with Article 3.8 to be 30%.

8.3.2 Outer Stringer Beam

The outer stringer beams are non-composite with the slab above and span, simply supported between cross girders.

From the slab analysis, the dead loads are determined for :



1 : live loads
2 : asphalt surfacing
3 : topping slab
4 : precast panel
5 : self weight of stringer

Fig. 8.5

Due to wheel loads, maximum positive moment is given by single wheel loads located at midspan, inclusive of impact load :

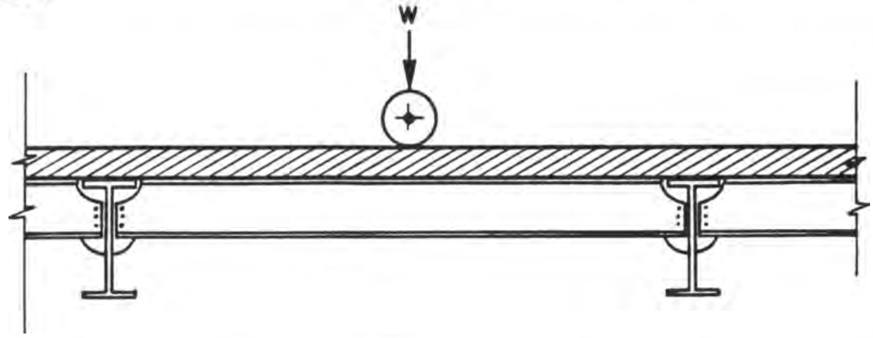


Fig. 8.6

The critical load combination is group 1 or 1A . According to AASHTO Article 10.32 the permissible bending stress is fy = 134 N/mm2.

The outer stringer is bolted to the cross girders each end through a web side plate type connection as illustrated.

Vertical shear at the support is taken by the coped stringer section of 350 x 8 mm.

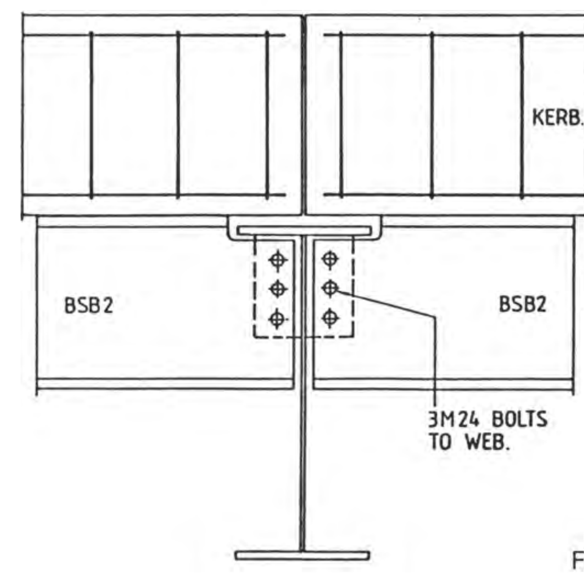


Fig. 8.7

The critical load combination for shear is Group 1 or 1A. According to AASHTO Article 10.34.3, the permissible shear stress is fy/3 = 115 N/mm2. For the connection, M24 bolts are used with a capacity in shear, threads excluded from the shear plane, of 84 kN and therefore 3 No. of bolts , as provided for, is satisfactory.

The deflection due to live load, including impact, is limited by AASHTO Article 10.6 to be not greater than span / 800.

Load effects during construction are not critical for outer stringer beams.

8.3.3 Internal Stringer Beams

Advantage is taken of the cast-in-place topping slab over the internal stringers to provide composite action and continuity over the cross-girders. Stud shear connections and transverse continuity reinforcement cast with the panels ensure longitudinal composite action.

Design loads are obtained in the same manner as for the outer stringers. However, precast panels and topping slab loads are taken on a steel stringer section alone, while additional imposed dead loads and vehicle loads are resisted by the composite section. Thus a two stage calculation of stresses is required. All three internal stringers have been made identical for maximum component interchangeability and the design is for the worst case of the central stringer. Load to steel section only is applied due to slab construction plus self-weight of beam.

After composite slab action and continuity of stringer is established, there are additional loads due to inposed dead loads, eg asphalt pavement as well as distributed live loads of 1.25 x (wheel loads) and increased by 30% for impact.

Computer analysis of a 7-span continuous beam, including moving wheel loads along the stringers, results in the maximum positive and negative moments:

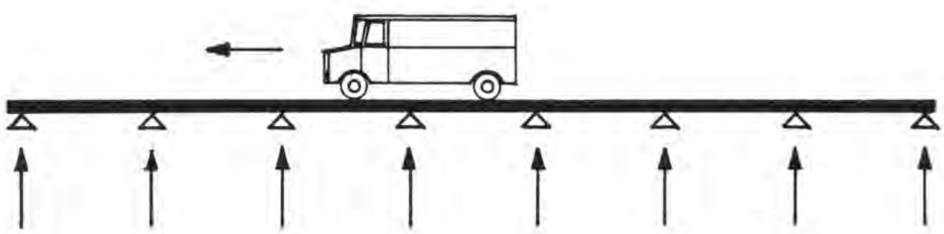


Fig. 8.8

Bending stresses are then calculated taking account of the appropriate section properties. For positive bending of the composite beam, an equivalent transformed section is considered, using a modular ratio Es/Ec equal to 9. Long term effects are accounted for by using a reduced transformed section assuming creep and shrinkage results in a long term elastic modulus of concrete equal to 1/3 the short term value. Refer to AASHTO Article 10.38.1 for design provisions.

The effective flange width of concrete slab contributing to the section is given by Article 10.38.3 to be span divided by 4. The maximum bending stressed for group 1A combination, reduced to 100% basic unit stress, are obtained for tension in the bottom flange and compression in the slab.

AASHTO limits steel tension to 190 N/mm^2 and concrete compression to 10 N/mm^2.

For negative bending over the cross girders, continuity is achieved through the provision of additional slab reinforcment and bolted bottom flange plate only.

The vertical end reactions are taken on the stiff bearing seat welded to the cross girder, however, 3 No. of M24 bolts are also provided to a web side plate connection.

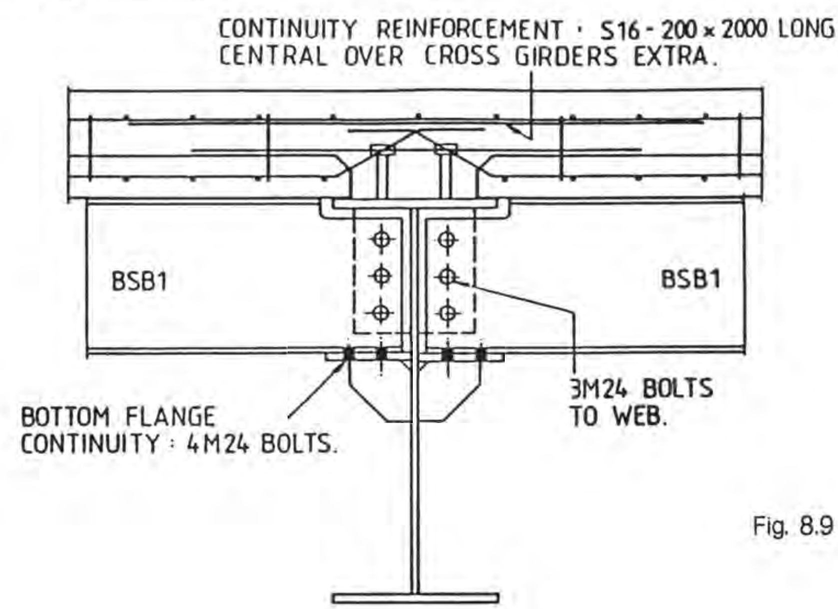


Fig. 8.9

The negative moment of this section follows due to imposed dead load and to live load plus impact (group 1A loading). For the connection of the bottom flange plate 4 No. of M24 bolts are required .

The vertical shear in the internal stringer is similarly applied at different stages. Note that stringer to cross girder connection, as illustrated, provides a rigid seat for the transfer of vertical loads to the supporting cross girder, see Fig. 8.10.

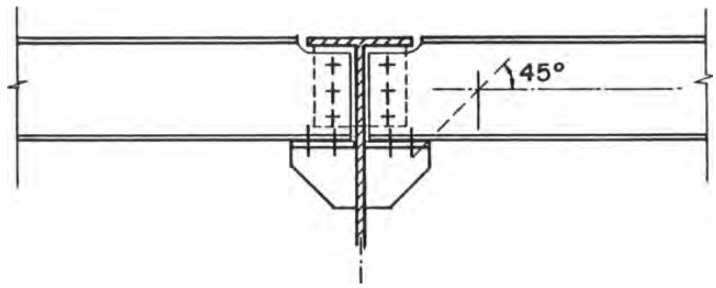


Fig. 8.10

Horizontal shear capacity between the steel to concrete interface is provided by shear studs welded to the top flange of the stringer. Design is in accordance with AASHTO Article 10.38.5.

The deflection due to live load plus impact is obtained by computer analysis using an average inertia of the stringer section to account for cracking of the slab over the cross-girders.

Maximum deflection is determined to be 4 mm. This is equal to that experienced by the outer, non-composite, stringer and ensures transverse compatibility of the deck system.

Construction loads act on the steel section alone before composite action is achieved. A check of this load condition is found to be satisfactory.

8.4 Cross Girders

8.4.1 General

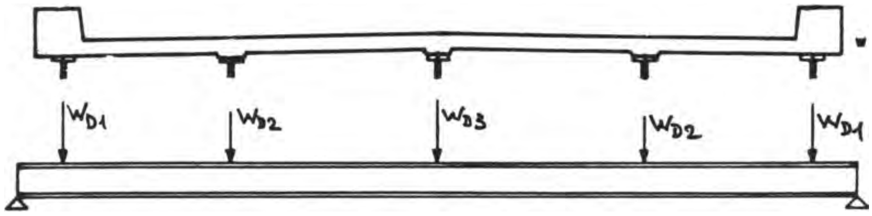
Cross-girders are located at each of the bottom chord panel points and span between the end gusset plates of the steel truss. The design assumes simple support and the use of composite action with the slab for imposed dead and vehicle live loads. Two cross-girder sections are provided, i.e. internal and end cross-girders.

Dead loads from the slab system and asphalt road surfacing are assumed to be applied directly through the stringer beam connections. Vehicle wheel loads are located transversely on the deck for worst effect to the cross-girder with a full axle located directly over the cross-girder. As for the internal stringer beams, loads are assumed to apply at two separate stages and total stresses need to be summed for the effects of each stage.

8.4.2 Internal Cross Girders

The reactions fo r dead loads to steel section prior to composite action are given from the stringers. They include the weight of concrete and the stringers as well as the self weight of the cross girders. The asphalt surfacing and live loads are carried by composite sections.

Dead loads:



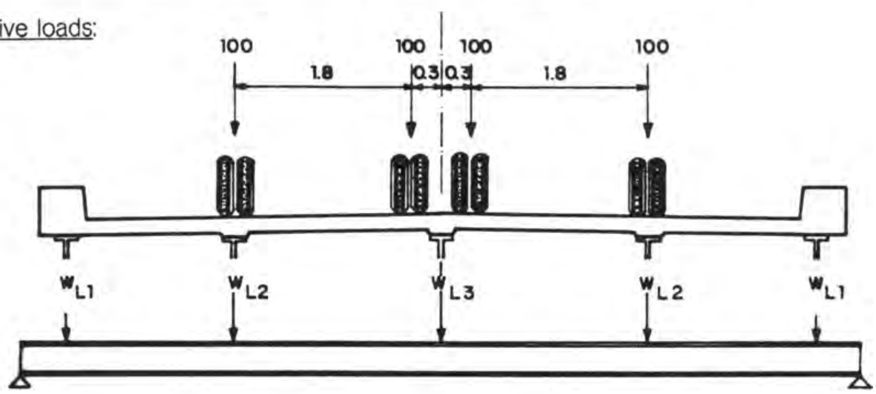
W_D : Asphalt, Concrete & Stringer

w : Cross girder

Fig. 8.11

Live load effects are obtained by considering two full HS 20-44 vehicles plus the allowance of 30% for impact. Wheel loads are taken to be applied through the stringers. Therefore, for maximum positive bending, distribution to stringers for a unit load is as follows :

Live loads:



$W_{L\ 1,2,3}$: Reactions from live load on cross girder

Fig. 8.12

From the internal stringer calculations, analysis of the deck as continuous over the internal cross girders results in the following maximum loading of the first internal cross girder for axle loads located as shown :

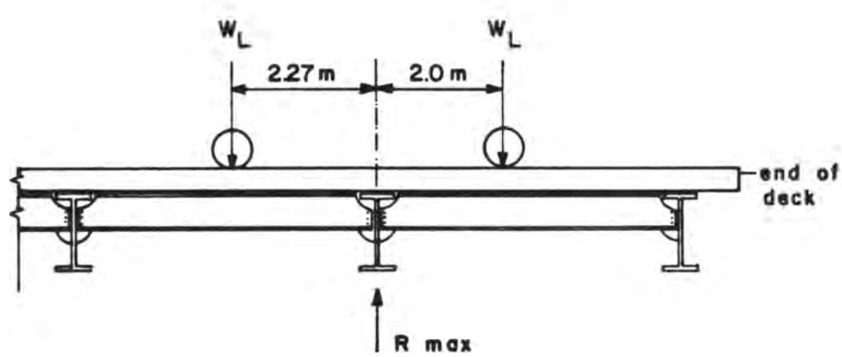


Fig. 8.13

Maximum stresses for Group 1 and 1A combination, reduced to 100% basic unit stresses, have to be limited to the AASHTO permissible stresses of :

tension in bottom flange : 190 N/mm^2
compression in slab : 10 N/mm^2

The internal cross girders are fully welded to gusset plates to each end to form the panel point connection with truss members. Shear is assumed to be resisted by the girder section alone. Design vehicles are positioned side by side against the kerb of the carriageway to give the maximum reaction. For a unit wheel load, stringer loads are shown in Fig. 8.14 :

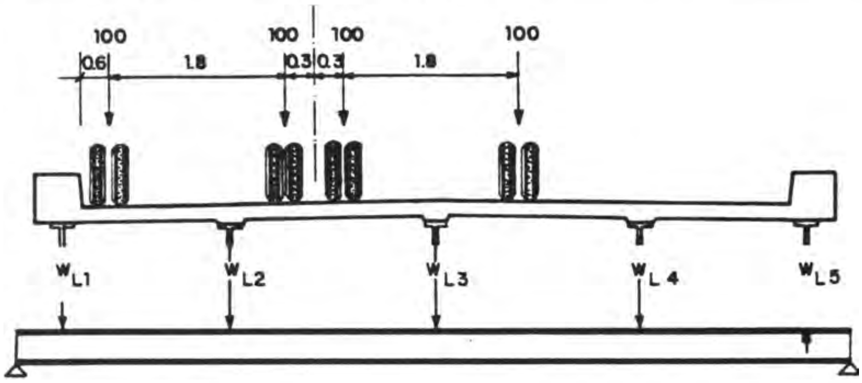


Fig. 8.14

In the critical section X - X, shear stress due to dead and live loads including impact has to be limited to :

$$f_y/3 = 115 \text{ N/mm}^2$$

Shear stud connectors are provided in pairs along the length of the top flange for transfer of horizontal shear along the concrete slab to steel girder interface.

8.4.3 End Cross Girder

In addition to supporting the deck system, the end cross girders form part of the rigid end frame transferring top chord lateral loads to the bearings. The design of the end frame is considered in section 7.4. The distribution of dead and live load in the transverse direction is similar to the internal cross girders. In the longitudinal direction the deck overhang at each end as well as the extreme position of the live load has to be taken into account.

For the continuous beam analysis of internal stringers, the maximum reaction on end cross girder is given in Fig. 8.15:

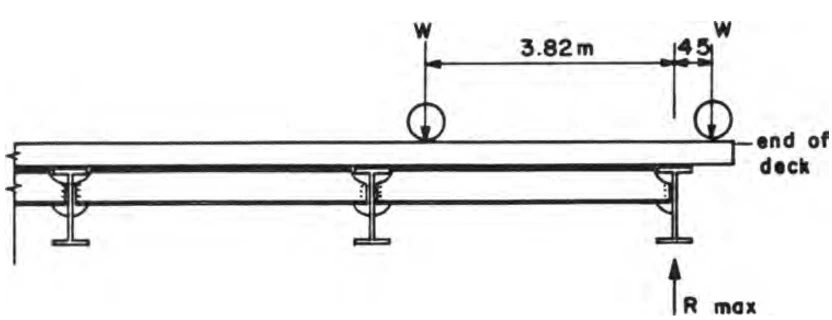


Fig. 8.15

End cross girders are bolted to the bearing assembly through a full depth end plate. Shear is assumed to be resisted by the girder section alone.

For maximum shear stress, design vehicles have to be positioned towards the kerb as shown for the internal cross girders. Shear studs are provided to transfer horizontal shear loads between the steel and concrete interfaces.

Calculated deflection of the end cross girder, assuming short term composite section properties, due to live load plus impact is 6 mm, i.e. span / 1420. Further checks on the end cross-girders under the combined load effects due to frame action for stability loads, earthquake and wind forces are found to be satisfactory in all cases (see Section 7).

8.5 Walkways

Where required, as in the case of the Kyounggone Bridge, walkways are fixed each side, outside trusses. These walkways comprise precast concrete deck slabs bolted to steel stringers which are supported on beams cantilevered from the bottom chord panel points. These beams are bolted through the truss chord outer gusset plate to the ends of the cross-girders behind. This is illustrated with a typical section. Walkways, floors, stringers and immediate supports are designed for a live load of 5.0 kN/m2 .

Concrete strength for precast panels is 25 N/mm2 and for supporting steelwork is Grade ASTM A36, fy = 250 N /mm2 . Precast slabs span simply supported on longitudinal stringers. Design moments are kept below the slab cracking moment and nominal reinforcement provided. Slabs are bolted to stringers over a strip of bituminous felt to avoid stress concentrations. Bolt holes for fixing and rebates to suit the railing posts are cast in and tops finished with a broomed non-slip texture.

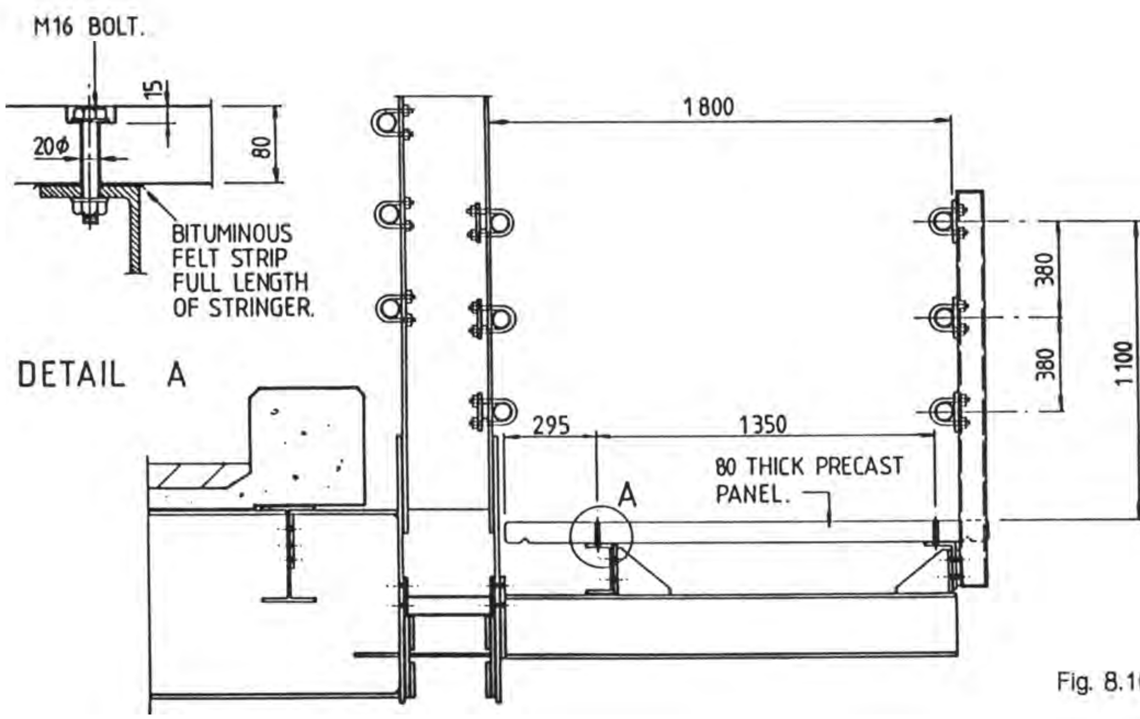


Fig. 8.16

Longitudinal stringers span simply supported between cantilever beams at each of the bottom chord panel points. Standard rolled section channels are used.

The cantilever support beam is bolted to the outer gusset plate at each panel point. A connecting bracket is provided between the inner and outer gusset plates of the truss panel point to link the support beam to the internal cross-girder beyond.

Handrails each side of the walkway are designed for a load of 75 kg/m acting vertically and horizontally on each handrail simultaneously. Posts are provided at approximately 2.6 m centres, fixed to the face of the outer walkway stringers and are designed for a transverse load of 200 kg acting at the centre line of the top rail. Footway cover plates bolted on each end of the precast panels complete the walkway.

9 Construction Planning

9.1 General

Neither bored, bentonite piling nor the Warren steel truss bridge require special construction planning. Whilst the programme for a series of bridges clearly needs to relate to overall road construction planning for individual bridge sites there does not appear to be any reason to illustrate such planning thought in any way more complicated than bar charts, as operations are largely sequential. However, project experience has highlighted the need to look to basics: personnel, equipment and materials have each proved problematic at different times.

9.2 Time and Progress Plans

Bridge planning is aimed at ensuring that, from initial survey through to opening, activities connected with the construction process are effectively coordinated. As a minimum, the following should be required:

- plant and equipment utilization programme
- materials supply plan
- labour/supervision requirement plans

Should certain periods look congested, permit critical path analysis should be worked through.

For example, at Kyounggone bridge, physical planning was set out in two charts: the construction programme which illustrated physical progress along the bridge and bar charts for installation and construction.

The illustrated "Construction Programme" breaks operations down to seven related activities and highlights the sequential nature of parts of the operation. The "Construction Programme" is special only in that it sub-divided and separated activities and all key component operations.

9.3 Plant and Equipment

Plant and equipment are key elements in most major civil engineering projects and, especially where replacements are not easily found, a clear plan identifying times when each different item of plant and equipment is required at a particular site and also times when such items are under pressure for near capacity use will prove beneficial. The rational allocation of available construction equipment through such simple planning should ensure maximum resource utilization and avoid having valuable equipment lying unused in remote places.

For managing an individual bridge site, a detailed plan should show up where individual items are in excess demand, e.g. concrete plant for bridge deck and piles and must provide a guide to periods when maintenance time is available without interruption of construction progress.

KYOUNG · GONE - BRIDGE : CONSTRUCTION PROGRAMME

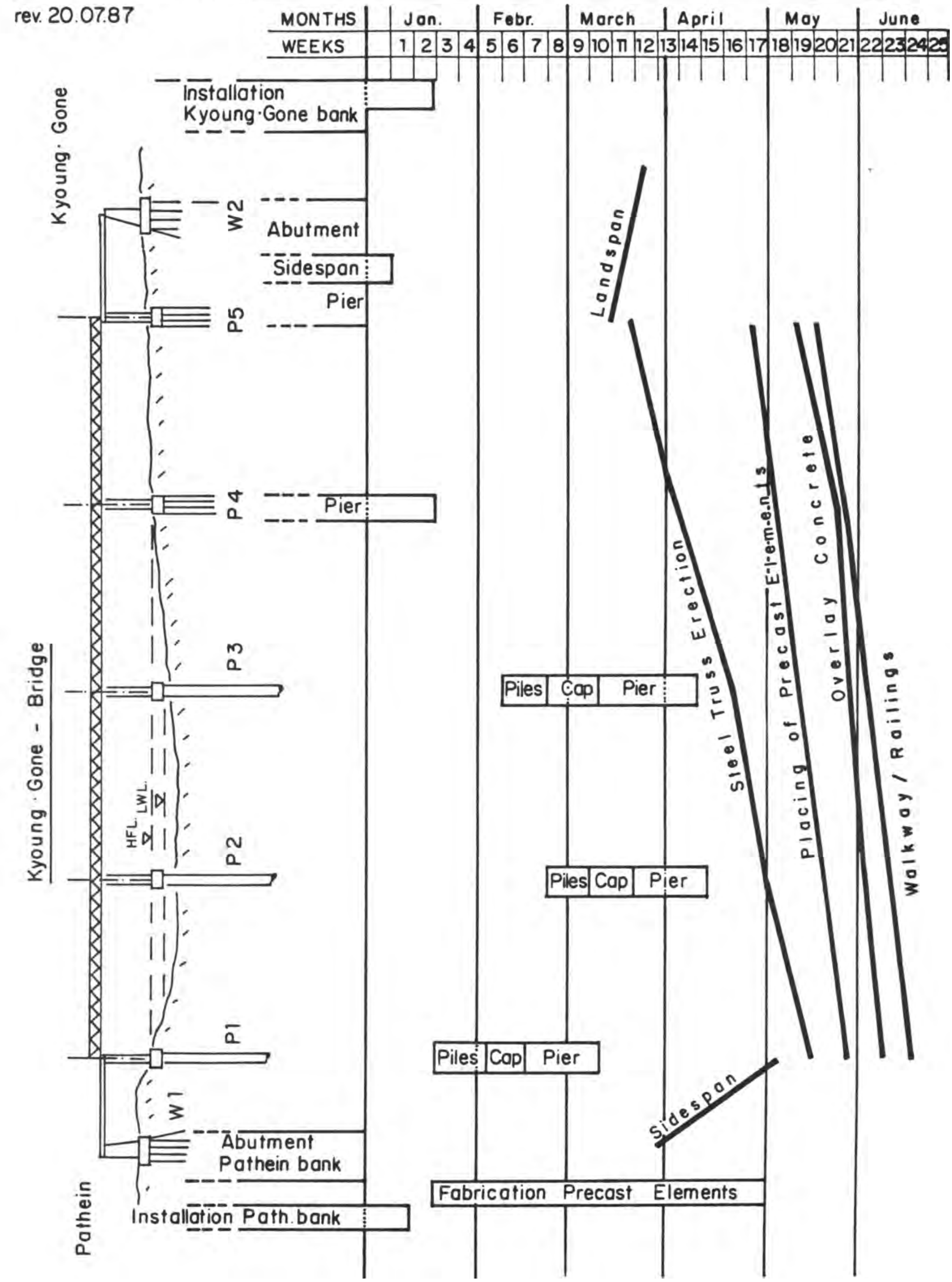


Fig. 9.1

9.4 Personnel

A simple plan for personnel is necessary such that management may fully incorporate availability of specialists, holidays, festivals, busy agricultural seasons, training courses and periods of difficult weather into the overall construction plan. This is of particular concern with deep bored piling in which there are a few key items of equipment with very few experienced and capable operators. A schedule of working periods should be agreed with the boring master and consideration given to other specialist posts, for example, who tests the bentonite?, who can operate the concrete plant? and so on.

9.5 Materials

Despite a basic construction progress plan at Kyounggone, an element which has caused serious problems has been supply of materials. Key materials, e.g. aggregates, cement, reinforcing steel and bentonite require monthly procurement plans to ensure requirements are identified, all orders for materials are placed with appropriate periods left for processing, purchasing and transportation, and that such plans are viable with respect to the available finances. While no plan can identify the many administrative and physical infrastructure problems which may arise, the reality of such plans will be reflected in how much is anticipated or accommodated while retaining the basic timetable.

These plans must be fully detailed and should be subdivided by each material type, including the various grades and qualities. The plans should include details such as suppliers and transport mechanisms with their lead times and crucial elements, e.g. Z-craft, barge or truck requirements, clearly identified.

9.6 Financial Plan

For a private contractor, the financial plan may well be the pivot about which other plans will be adapted. It will seek to plan cash flow and its origins, identify cost centres and will identify the profitable elements of the job and their timing. This is less likely to be crucial for government agencies where the essential element of the financial plan is to relate physical progress to a project budget, identify actual expenditures and the comparison with those projected and ensure that wider budgets, often poorly coordinated to construction cycles, are achieved. These are all necessary and may be crucial to the progress of a project, particularly a large one extending over several years. They are, however, of little relevence to the on-site progress of individual bridges.

10 Site Installation

10.1 Outline

The works required to establish a significant construction site from green field to fully operational working is best regarded as a full sub-project which is crucial to the successful undertaking of the main, bridge, project. For this sub-project the balance of cost; to time; to quality balance, upon which normal project planning depends, is biased very strongly towards time. If the overall bridge construction project is to start and proceed according to plan, it is evident that it must start as scheduled with full operational possibilities.

10.2 Elements

There are some twenty different separate areas (see Table 10.1 and Fig. 10.1) of the construction site which need to be properly established before starting the first piling or abutment work. For effective construction activities to start, it is essential that all these are properly installed and working. It should not be acceptable that installation work is continuing whilst the principal works progress as the one inevitably will distract from the other. It similarly should not be acceptable to make partial provision on the argument that the works are only temporary. Proper establishment of the site will permit overall savings by ensuring that construction proceeds effectively and is not delayed: proper stores to ensure parts are not lost, site roads that all machines may travel over without delay or damage, loading ramps that permit quick loading and unloading, especially for the many boats and pontoons involved and accommodation which provides staff and labour with proper water, electricity, cooking and sanitary facilities etc.

As for the overall construction, a programme for this vital sub-project should be made to ensure it may be effective and on time. A theoretical example is included, see 10.3. This identifies clearly the key role throughout the installation cycle of the ground clearing and levelling equipment and the need to get sufficient skilled labour onto the site to bring the landing platform, accommodation and the workshop quickly into use. Even with the tight programme indicated, a minimum of 14 weeks would be required and, for sites seeking to follow a post monsoon construction start, this requires that the installation procedures must start before the monsoon to make full use of the dry season.

With the tentative programme outlined, there would be an eight week period in which a build-up of construction material could arrive with the proper reception and storage facilities available. This is believed sufficient, even for the bulk materials like aggregates. The proposed plan, however, only allows for one weeks flexibility, this may not be enough on sites with especial transport uncertainties where the case for an early start to installation is correspondingly stronger.

For bridge sites in the delta, there are significant problems peculiar to the area, communications are difficult, access is a problem for major equipment and materials supply, soil conditions are poor and much of the area is subject to annual flooding. The process will also, on some sites, be complicated by the necessity to work with a secondary site on the opposite bank. This may be required for a variety of reasons, perhaps to suit the road construction programme or, if the river cannot be closed to permit concrete pumping across the water as an alternative concrete source. There is also likely to be the need to make allowance for facilities which may be supplied by or required for the implementation of the associated roadworks and a number, for example the workshop, which, though mutually beneficial ensures more complex planning.

Site Area	Space Required	Other Facilities
Concrete Batching Plant	Minimum area of 30m x 30m, better 40m x 40m for plant and associated storage. See Drg. of concrete bases Close to bridge line to reduce concrete pumping distance to piles Require space at concrete silo for seiving cement before loading into silo.	Water, electricity and space adjacent to the loading ramp for min. aggregate movements.
Cement Store	Possible use of 2000 bags per day. Store for 5000 bags required. Min 10m x 20m covered store with dry floor. Stacking max 10 - 12 bags high with circulation space for efficient import and output.	Electricity and direct access to batching plant and to landing platform (or road if to be supplied by truck).
Sand and Aggregate Area	See also Batching Plant. 40m x 40m area will store approx 400 m3. Main store area requires separate sand/ 5-12mm/12-25mm aggregates. Approx 500m2 per pile x 1m high for wheel loader operations. Total volume dependent upon reliablility of resupply	Direct access to incoming supply point Directly adjacent to Batching Plant
Precast Panel Casting Area	20m x 20m for casting. Stockpiles or secondary storage area required.	Direct access to aggregates. Road access to panel store.
Precast Panel Storage Area	Relate to number of bridge sections (at least twice casting area required).	Direct road access to end of bridge. Water for curing.
Reinforcement Fabrication Area	25m x 25m min, better 45m x 30m plus areas outside for storage of completed crates.	Close to landing area for steel supply with road for carriage of 12m long crates to crate storage.
Pontoon Bridge	See Drg. Required to carry concrete, bentonite pipes and personnel to drill.	Close to line of bridge to reduce pumping distance and to batching plant and road.
Bentonite Tank	See Drg. Cut and bund formation to 300m3 capacity for 50m deep piles.	Pipe access to pipe bridge.
Bentonite Mixing	10m x 10m.	Water, electricity. Direct pipe required to storage/curing tank.
Bentonite Store	After initial mixing daily use likely to be only 100kg per day. Use cement store initially and small separate store later.	

Table 10.1

Workshop and Spare Parts Store	Min 15m x 15m building for small site and 20m x 20m for larger site in 45m x 40 - 100m compound. Building with high roof with overhang. Wall only to spare part store and office areas.	Electricity, water and road Away from cement/bentonite. Logical centre for fuel store and electricity generators.
Fuel Store	Diesel, petrol and oils store. elevate tanks onto concrete or wood structures to gravity feed machines.	Road access and link to workshop. Security fencing.
Electricity Generator/ Distribution	Logically the load centre for all electric power. Away from accommodation areas.	Link to workshop for control and maintenance.
Tools Store	Covered area with separation for various types of tools and major separation of mass use tools (e.g. shovels), pans from more specialist (e.g. vibrators), pulling machines and small tools (e.g. spanners, jacks, drifts).	Near to other stores for control. Possible combine with workshop store, if not too far or bridge/small parts stores.
Bridge Parts Store Small Items Store	Store gusset plates, splice plates, end assemblies, bolts, washers, bearings. Size depends upon number of spans. Min 5m x 10m building, with raised floor and separate partitioned areas with bridge items and small consumables (e.g. nails) separated.	Close to truss assembly area.
Bridge Members	Level dry area min 20m x 20m. For stack and protection see Drg. in 14.5.	Road access with easy way to landing platform.
Site Offices / Meeting Room	Min. two rooms large enough for drawing tables and desks plus space for meetings areas. Ideally with view to site.	Away from noise and dirty. Road access plus electricity and sanitation and water.
Accommodation	Size depends upon bridge and area. Kyounggone had min 250 labour and 17 staff.	Water, electricity and sanitation with close road access.
Landing Platform	5m wide, approx. 1.5m above LWL. Length sufficient to ensure 2.5m min water depth. Wood structure with bearing capacity to carry heaviest plant (for mobile crane and drilling rig access onto pontoons).	Focal point for concrete and bridge elements.

Table 10.1

10.3 Theoretical Installation Programme

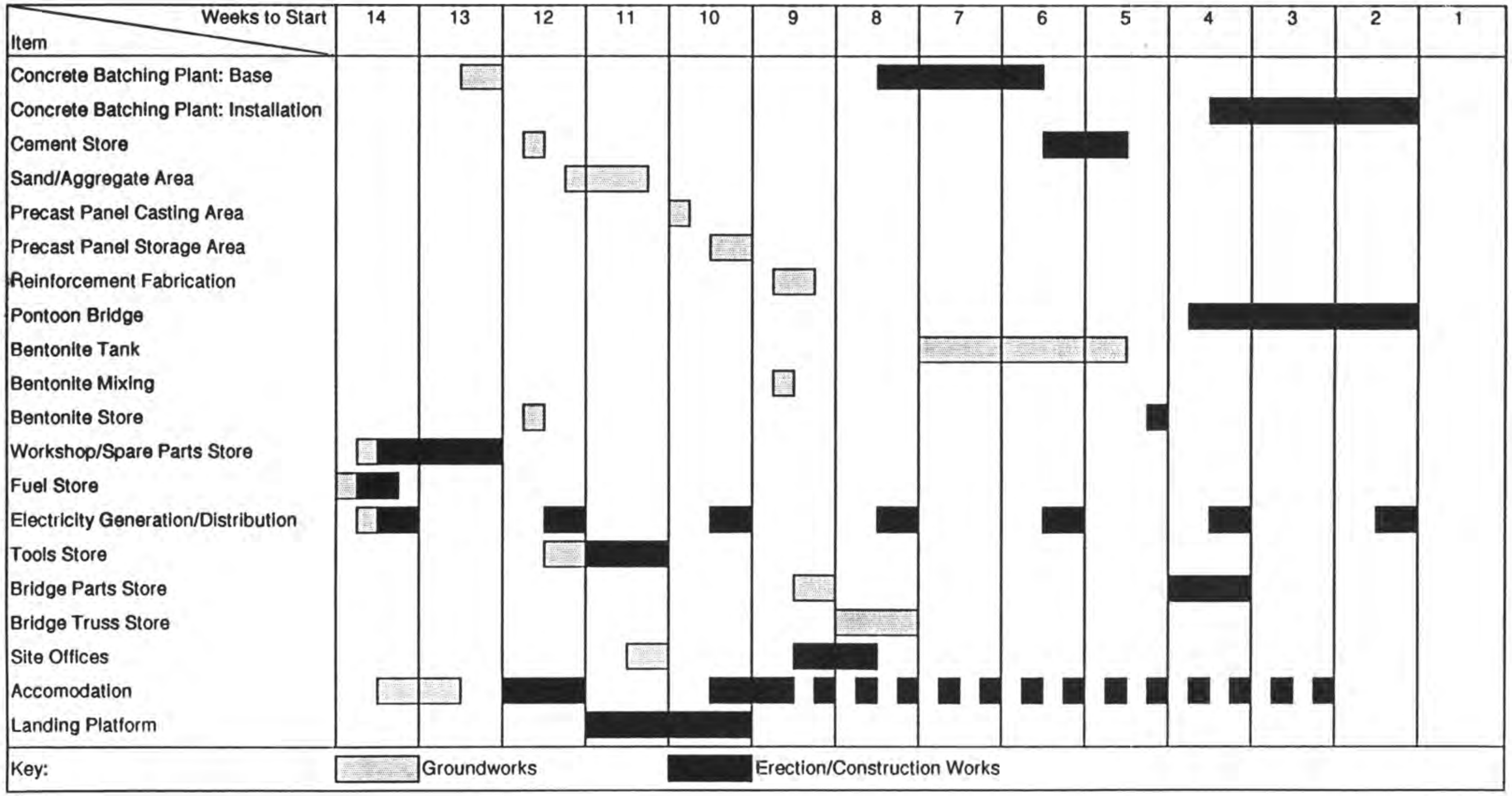


Fig. 10.1

10.4 Layout of Main Site Facilities

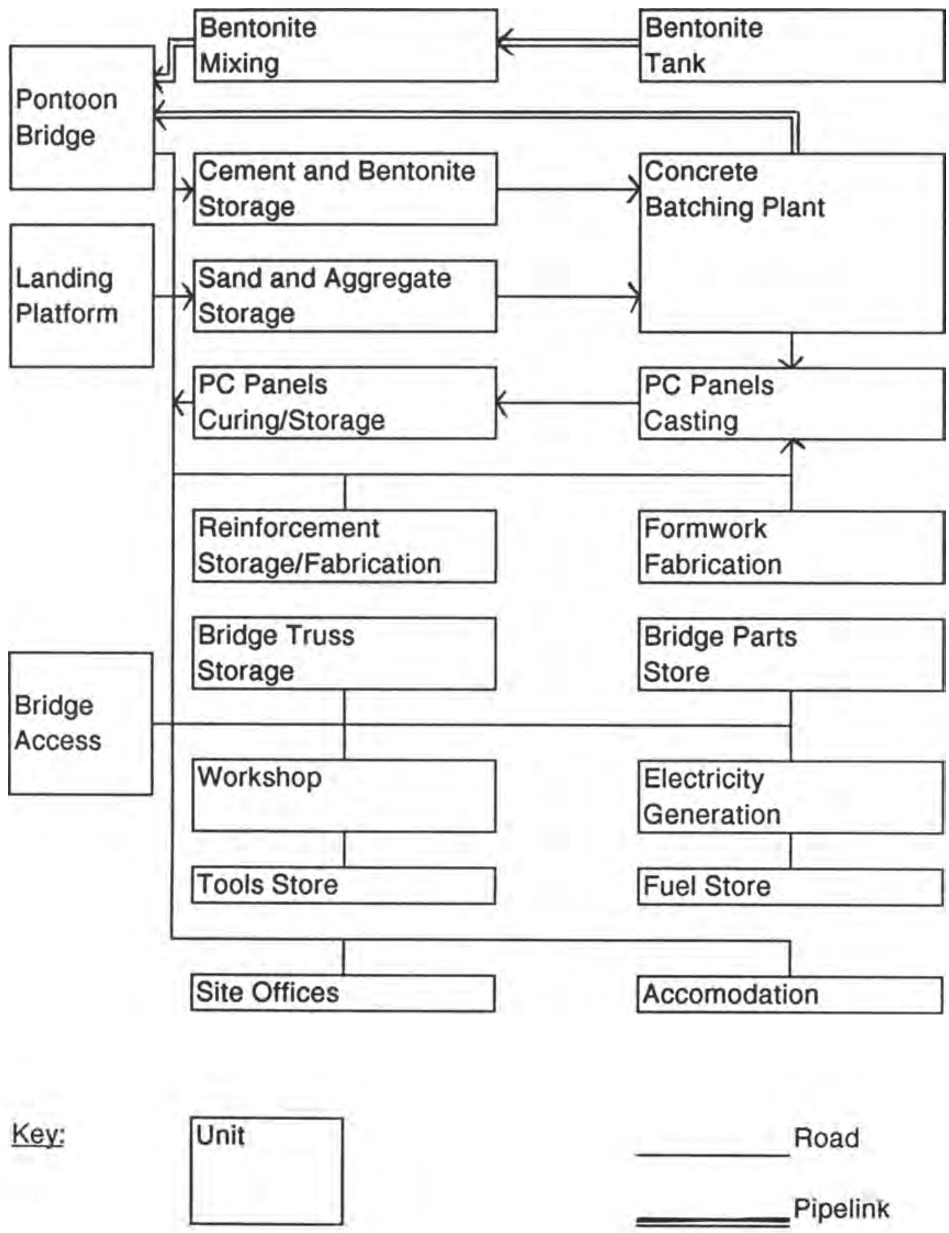


Fig. 10.2

Fig. 10.3: Layout of Bentonite Liquid Storage Tank

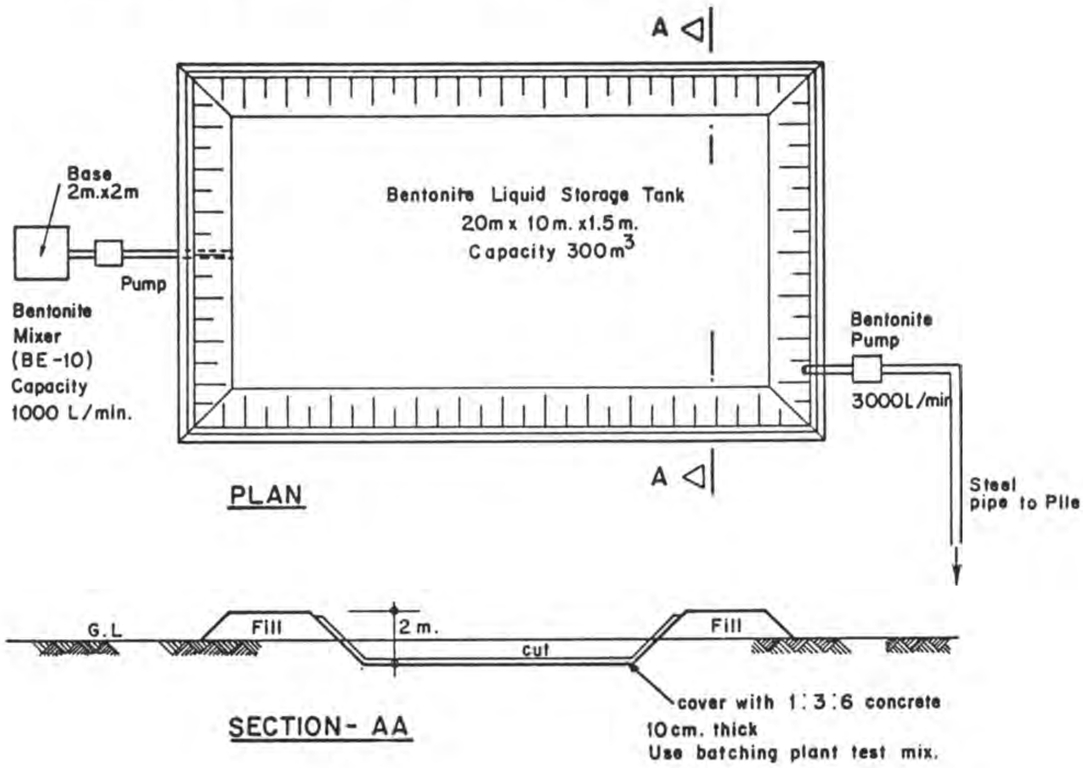


Fig. 10.4: Layout of Reinforcement Yard and Truss Stacking Place

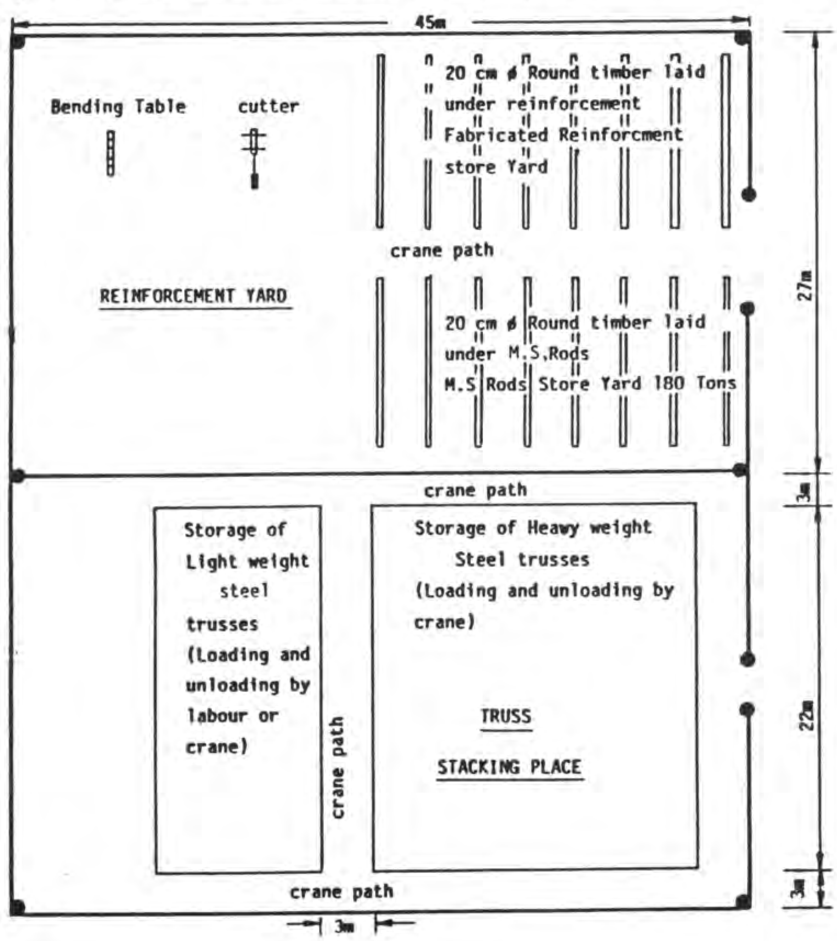
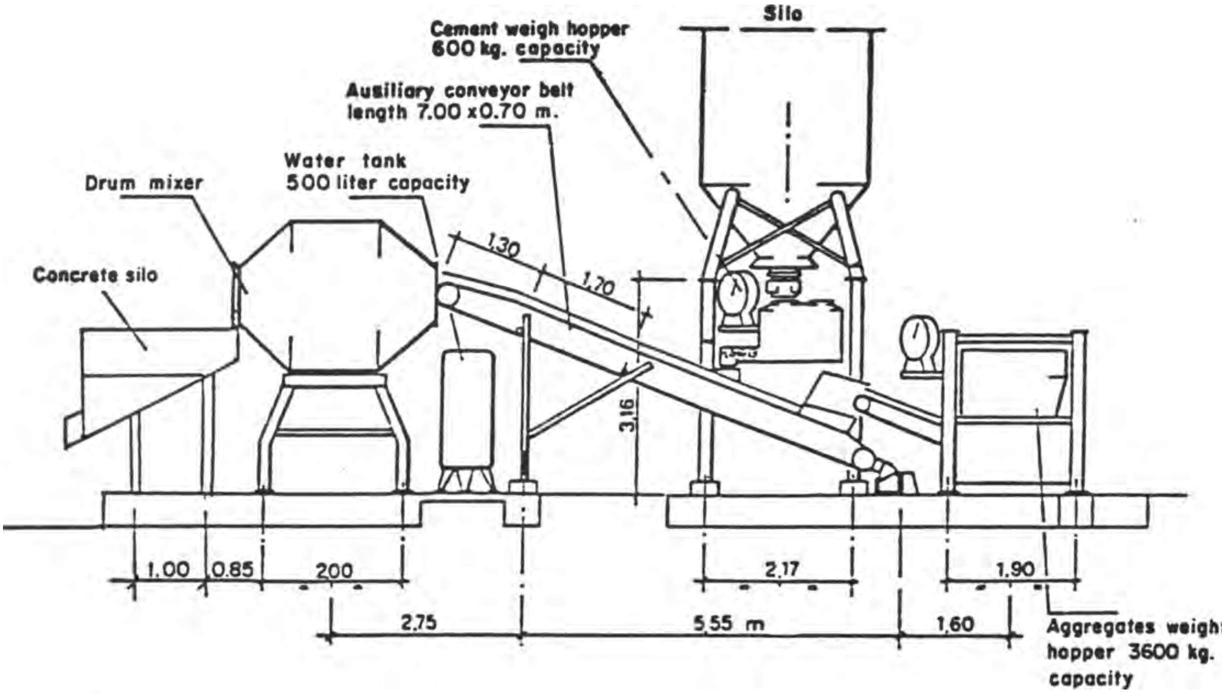
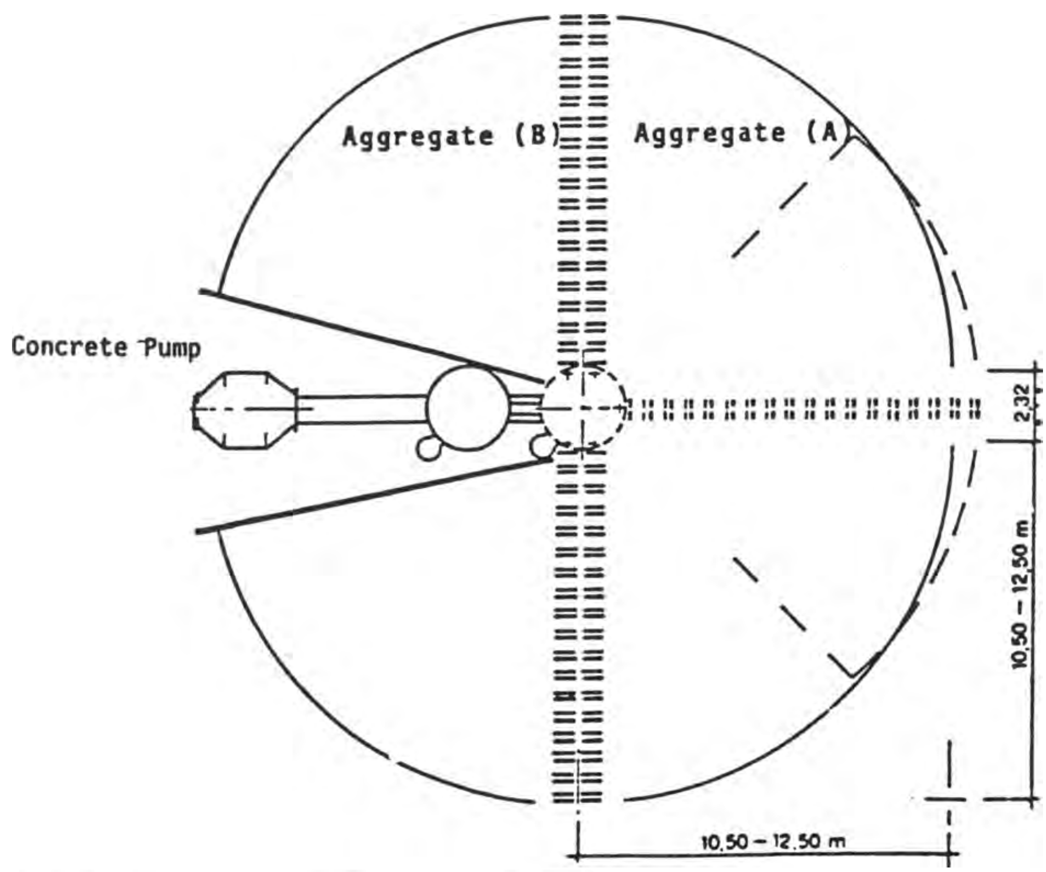


Fig. 10.5: Layout of Concrete Batching Plant



Side View



Plan View (total storage capacity of aggregates: 400 m3)

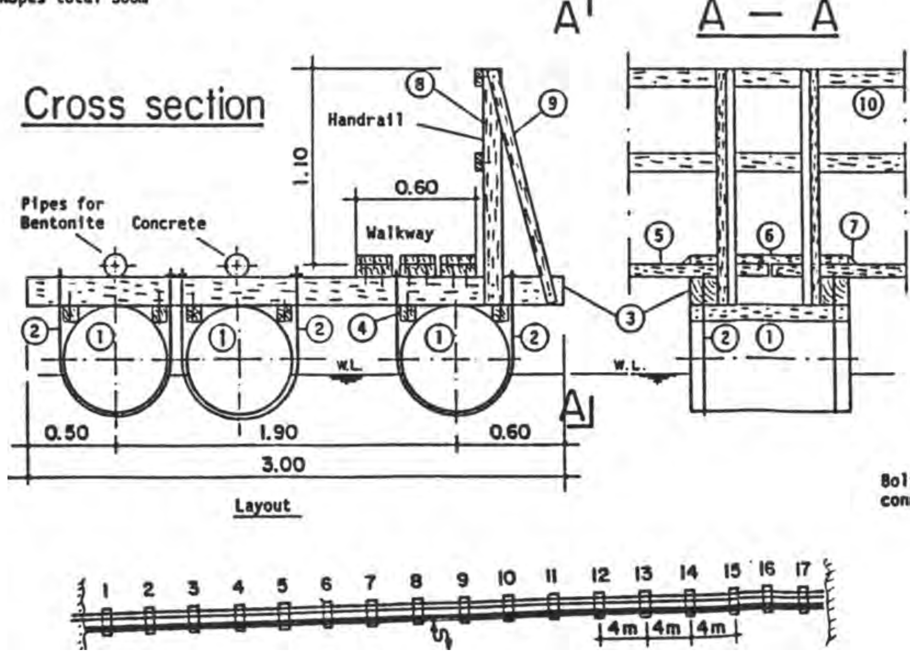
Temporary floating platform

List of material

Description	Dimensions in	Sketch	Numbers	Position
Oil barrels	diameter length			1
Round bars	diameter ≥	10 80 thread		2
Squared timber	16/16/300		68	3
Squared timber	9/9/90		68	4
Plank (Walkway)	20/7/400		51	5
Plank "	20/3,5/90		51	6
Triangular Fillet	3,5/3,5/60		34	7
Board (Railing)	10/5/130		34	8
Board "	5/5/135		34	9
Board (Handrail)	10/5/ total length 180m			10
Board (Bracing)	10/5/440		68	11

Bolts, Nuts, Washers, Nails

Ropes total 300m



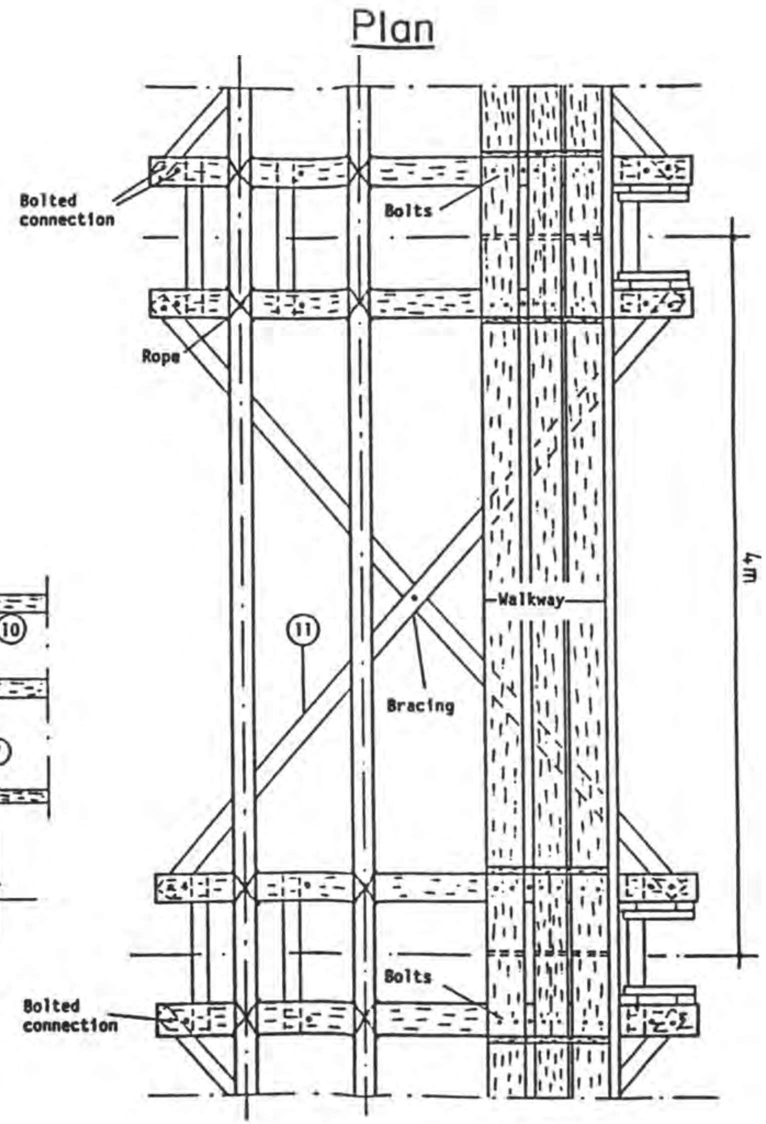


Fig. 10.6: Temporary Floating Platform

11 Setting Out

11.1 General

Setting out for bridges varies greatly with the size and complexity of the particular bridge and the site conditions. For the bored pile and truss bridges introduced by the project the conditions are relatively simple. This section sets out some of the basics and details only the introductions made by the project, in particular the distomat and its use in setting out pile positions. Thus, the section includes:

- tolerances for the bridge type
- level and horizontal angles, standard rules
- distomat in detail.

11.2 Tolerances and Checks

In order that all parts of the bridge will fit together, the initial setting out and subsequent construction checks must take account of the following tolerances:

Position:			
	- pier and abutments	- within 20 mm	measured along line of bearings
	- holding down bolts	- within 5 mm	
	- centre to centre bearings	- within 10 mm	longitudinal in final position
		- within 5 mm	cross spacing at one pier/abutment
Level:			
	- top of pier or abutment	- within 20 mm	
	- top of plinth for bearings	- within 5 mm	

11.3 Preservation of Basic Setting Out

The setting out for a bridge has certain complications prior to construction starting, but these are negligible compared with the difficulties of working once machinery becomes operational and position and level marks are both obstructed and destroyed. It is essential that any such basic points are able to be replaced accurately, offsets established and that, during construction, basic marks are protected and replaced if damaged.

To retain these control stations the following simple measures are required:

- physical protection
- offset markers
- triangulation.

Physical protection is simple and, if possible, should be very visible. However, accident, casual abuse or necessary removal require that some points will necessarily need to be restored. If it is possible, replacement with only measuring tapes is best, using simple triangulation measurements from well marked and unlikely to be disturbed points or from offset marks specially sited to avoid disturbances and to be easy to use. Such offsets and triangulation points also require protection.

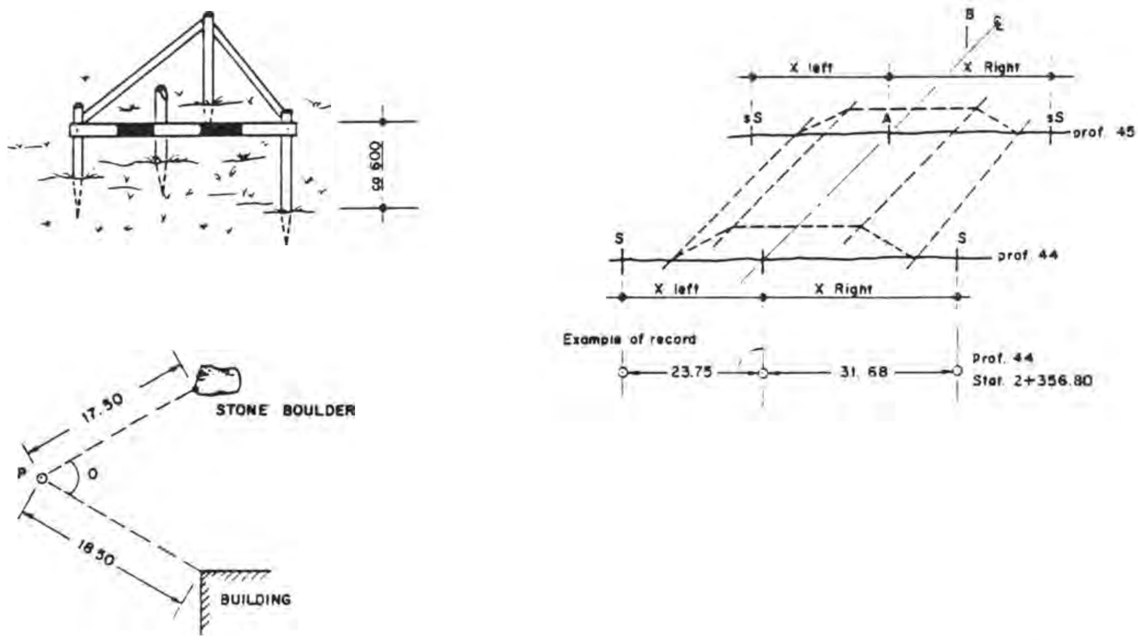


Fig. 11.1

11.4 Levelling

In order to reduce the possibility and size of incorrect level readings, the following simple measures should be rigorously applied:

- i) The measuring staff needs to be vertical. This should require that the person holding it should understand why this is important and how it may be checked (spirit bubble).
- ii) Results of levels and also angles, distances etc should always be calculated as readings are taken, never at some later time.
- iii) Calculated levels should be compared with known levels and expected or design levels.
- iv) Tie in from known levels to other known levels to form closed circuits; do not use loose ends for important levels.
- v) Check that both the adjusting and the fixed, top screws of the tripod are tight
- vi) A level should only be used after the engineer or surveyor has personally checked that it is able to read accurately and it should be checked before setting any new base points or critical levels, e.g. bearing levels.

The simplest means of checking a level before use, i.e. testing its horizontality, is as follows:

In a flat area, select a test bay, between 45 and 60 m long (about 150 to 200 ft.) Divide this into three equal sections (of length d, see Fig. 11.2) . A levelling staff is set up at each of the intermediate points, B and C, respectively and readings are then taken with the instrument set up, in turn over the terminal points, A and D respectively. With the instrument carefully levelled up at A, readings a1 and a2 are taken to the levelling staves held at B and C and then, with the instrument at D, readings a3 (to C) and a4 (to B) are taken. If the line of sight of the level is absolutely horizontal, then

$$a4 - a1 = a3 - a2$$

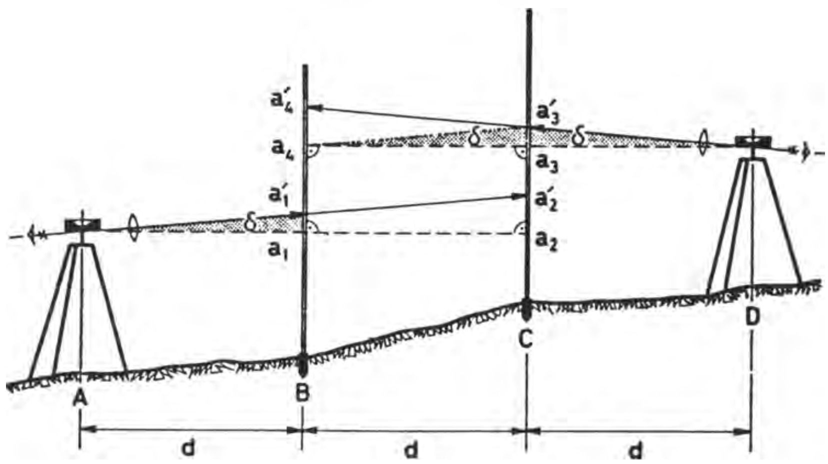


Fig. 11.2

If the actual readings do not prove equal then first the whole procedure should be repeated and, if the difference is confirmed, then the line of sight of the level must be adjusted.

11.5 Horizontal Angles

Before setting out base points, always test the instrument using both faces:

If the vertical circle is Left of the eyepiece the instrument is said to be in Face Left position. If the circle is Right of the eyepiece it is in Face Right. If full use is to be made of the theodolite's accuracy, then measurement of the angles in both Face Left and Face Right is necessary. An exception is with the subtense bar, when it is sufficient to measure the parallactic angle in one face only as both targets are at the same distance and height.

The measurement of a single horizontal angle between two targets is the most simple form of angle measurement and is used in traversing. As only two targets are sighted, measuring time is short, thus systematic errors from residual changes in the verticality of the standing axis and twisting of the tripod can practically be avoided. For accurate measurements it is normal to observe in both face left and face right and take the mean. After setting the reading to the R. O. (Reference Object) to zero, or to the known bearing, the observing sequence is:

Face left, swing clockwise, observe left target (R.O.), swing clockwise, observe right target.
Face right, observe right target, swing anticlockwise, observe left target (R.O.).

If more accuracy is needed additional sets are observed. If two sets are observed the setting to the R.O. for the second set is changed by 90 degrees. If four sets are observed the settings to the R.O. should be approximately 0 degrees, 45 degrees, 90 degrees, 135 degrees.

In triangulation and polar coordinate work (radiation) it is common to observe to several targets from one station. The routine is the same as just described except that more targets are observed. A distinct distant target should be chosen as the R.O. (reference object). After setting the reading to the R.O. to zero, or to the known bearing, the observing sequence to comprise one set is :

Face left, swing clockwise, observe to targets 1 (R.O.), 2, 3, 4, 5...n.
Face right, swing anticlockwise, observe in reverse order to targets n...5, 4, 3, 2,1

If more accuracy is needed, additional sets are observed with different circle settings as described above.

It may be desirable to close each half set to the R.O. In this case the first half set, face left and swinging clockwise, will be 1(R.O.), 2, 3, 4, 5 ...n, 1(R.O.). The second half set, face right swinging anticlockwise will be 1(R.O.), n ... 5, 4, 3, 2, 1(R.O.). The values are meaned and any difference between the R.O. readings can be adjusted through the set.

If the plate level changes during the measurements, re-adjustment can be made at the end of a half set, but never during a half set.

11.6 Distance

For small distances steel measuring tapes are sufficiently accurate. For larger distances it is now standard to use a Distomat. For this project a Wild DI 1000 Distomat has been provided; the following instructions (section 11.8) explain its use.

11.7 Procedure for Setting Out and Checking the Position of the Pile

Setting-out the position of a pile proceeds from two known positions to the establishment of a third. At its least certain, this requires starting from one exactly located point and lining up to the second, but it is better to have distances from each of the two known points to the new point checked.

As an example, setting out from Pier 4 to Pier 3, Kyounggone Bridge.

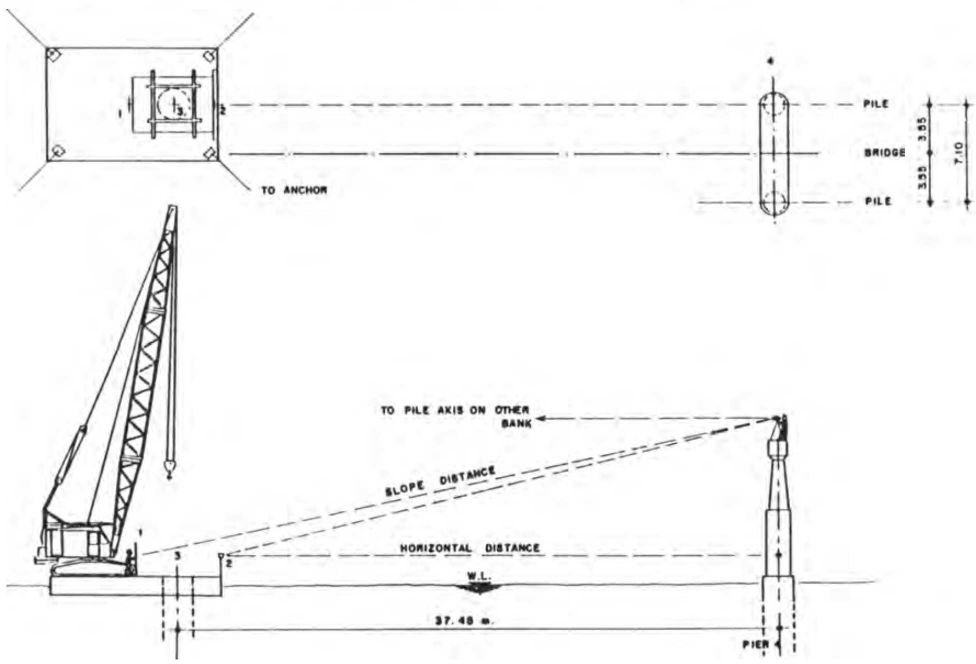


Fig. 11.3

- Establish the centre of Pier 4 by line and distance back to initial setting-out points, i.e. confirm the bridge axis and the pier centre.

- Set-up the theodolite over the pile by establishing the point 3.55 m at 90 degrees to the bridge axis.

- Thedolite sight along the pile axis, i.e. to a line parallel to the bridge axis but 3.55 m off it.

- Set an approximate centre for the pile in the river bed by alignment and measuring tape and mark with a bamboo pole.

- Position the unifloat at the bamboo pole and fix by anchor/winches.

- Mark and fix the pile axis exactly at points 1 and 2 on the float.

- Measure the slope distances to point 1 and to point 2 using the distomat. N.B. the vertical angles to 1 and 2 must be entered (see section, Wild Distomat).

- The horizontal distances to points 1 and 2 from the centre of Pier 4 are displayed from which the difference to the required 37.45 m to the pile centre may be calculated. The tolerance for position is maximum 0.20 m and it will be normal to be within 0.05m.

- Weld guide-beams to the unifloat as per outside diameter of steel casing plus 0.05 m.

- Check the axis and distance to the welded guide beams.

- Drive in the casing about a metre.

- Check the axis and distance direct to the casing.

This process establishes and checks the position but before drilling through the casing the guide beams must be removed.

11.8 Use of the Wild DI 1000 Distomat

The Wild DI 1000 Distomat is an electronic distance measuring instrument designed to fit onto an adapter plate on the telescope of the Wild optical theodolite.

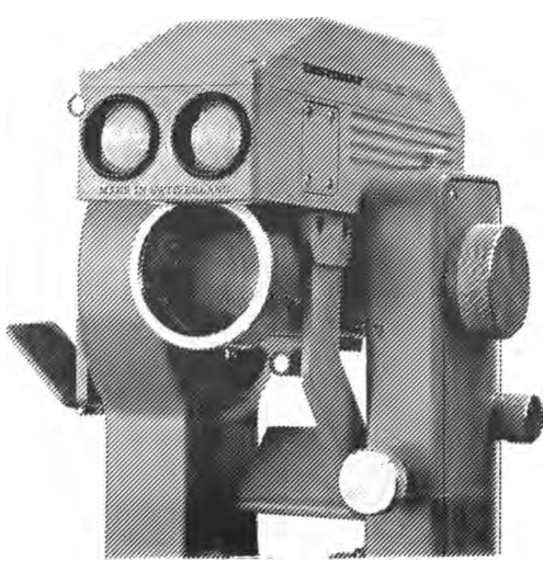


Fig. 11.4

The distomat is powered by a 12v battery permitting up to 2000 measurements. It is robust but requires to be kept clean and wet equipment must be dried before storing. In hot sun, shade the instrument and for long range use it is best to shade the reflector prism, too.

The DI 1000 fits onto an adapter plate on the theodolite telescope. It may be operated with or without keyboard. Without optional keyboard the DI 1000 is operated with the three control keys above the liquid crystal display.

The distomat measures 500 m normally within an accuracy of 5mm. The prism holder GPH1A should be used to point the DI 1000 to the reflector and, to obtain the best results, it is essential that the infra-red beam of the DI 1000 and the line-of-sight of the theodolite telescope are parallel.

With the DI 1000 adjusted correctly to the telescope, only a single pointing to the reflector is needed for both distance measurement and angle measurement. Point with the telescope cross hairs, as shown, to the yellow targets beneath the prism.

For detailed instructions regarding use, checking, correction and care, the reader is referred to the Wild Distomat DI 1000 Operators Manual.

12 Bentonite Piling

12.1 General

Bored piling for pier and abutment foundations requires limited specialised equipment, plus knowledge, experience and care, with the bentonite drilling mud. It also demands equipment and organisation capable of ensuring a reliable and steady flow of concrete through a period of up to ten hours.

Bentonite piling depends upon good planning to ensure that equipment is available as required and in reliable working condition, that sufficient materials are forthcoming and the site management is able to organise labour and the various other elements to come together throughout the work period.

12.2 Equipment

The equipment used for deep bored piles will vary with the site, the depth, pile diameter etc. The details below refer to that used for 2.5 m diameter piles at Kyoung-gone bridge.

12.2.1 Drilling

- Track mounted crane, 20 m jib, 30-60 tonne capacity (as supplied: Link-Belt LS 108 with 65 HP, 914 mm tracks and 15 m boom plus 2 extensions, total 24.5 m).
- Rotary drilling rig with telescopic kelly bar and crane mountings (as supplied: Soilmec RT 3/5 with 4 x 12 m Kelly bar).
- Drilling tools: clay auger and sand/silt bucket (as supplied: 2.5, 1.8, 1.2 and 0.8 m diameter)
- Crane support mats (see Plant and Maintenance, Section 15).
- Floating pontoon with 100 tonne capacity (as used: unifloat with mooring cable, anchor, winches).
- Steel guide casing (2.65 and 1.35 m diameter, 5 - 10 m long)
- Vibrohammer with fitting attachments to drive and lift the casing.

12.2.2 Bentonite

- Mixer, capacity 1000 litre (as supplied: Soilmec type BE-10)
- Desander unit, capacity 50 m3/hour (as supplied: Soilmec BE-50)
- Pumps and pipes (as supplied: Jonio 100 pumps and pipes 6 m long and 100 mm diameter with quick couplings, bends and plugs)
- Floating walkway and pipeline supports (see site installation).
- Testing equipment.

12.2.3 Concrete

- Batching plant, capacity 50 m3/hour (as supplied Autoformula 1561 with 50 tonne silo, 1500 litre drum mixer, screen feed cement hopper, conveyor,two aggregate conveyors and water pump with electric weighing).
- Concrete pump (on Hino truck mounted used)
- Concrete tremie pipes (260 mm diameter with receiving hopper, holding fork and pipe handling swivel).
- Cubes, slump cones etc. for quality control.

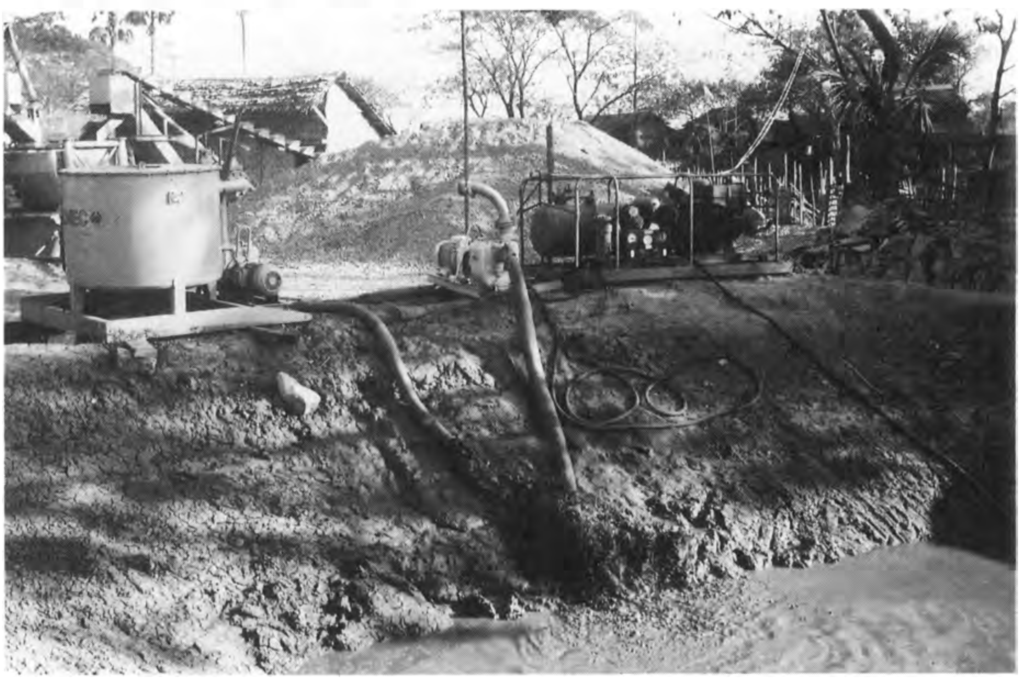
12.2.4 Miscellaneous

- Electric generator (150 kVA with distribution board 380/220 V)
- Mobile air compressor (4.5 m3 used for 40 m deep piles)
- Two small riverboats (approximately 6 m long with outboard motor)
- Dump truck
- Payloader and dozer
- Z-craft (for transport and ease of working)

The above list is general; all other normal large construction site facilities are assumed (see installation) including, and most importantly, a properly set-up spare part store and workshop to ensure the required reliable operation of all the equipment.

Foto 5: Concrete Batching Plant

Foto 6: Bentonite Tank and Pipes



12.3. Pile Wall Support with Bentonite Muds

12.3.1 General

The most economic system to keep a bore-hole open while drilling is in progress and for casting a pile involves the use of a fluid, the drilling mud, to oppose:

- the pressure of outside water
- the earth pressure into the hollow section.

Widespread experience in deep piling has resolved that the use of bentonite based drilling mud is more or less essential. The drilling mud or slurry, consists of water mixed with powder to form a colloidal substance based upon bentonite, a montmorillonitic clay, whose chemical formula is:

$$(Mg\ Ca)\ O \times Al_2O_3 \times 5\ SiO_2 \times n\ H_2O$$

However, bentonite is effective in stabilising soils only if the following conditions are satisfied :

a) the hydrostatic pressure of the mud in the hole must exceed the pressure exerted from the water table.
b) the soil granule size must be such as to prevent dispersion of the slurry, i.e. it is not appropriate for coarse sand and gravel.

If these two conditions are not satisfied, ground water will penetrate the hole, pull in solid particles and cause the walls to collapse.

The bearing resistance of piles made with bentonite has been tested and, both laboratory experiment and practical experience, has demonstrated that full account may be taken of lateral friction between the pile and soil. The bentonite layer or cake does not reduce this friction.

12.3.2 Properties of Bentonite Muds

The bentonite mud must possess the following properties:

- screening: oversize particles through 10000 mesh/cm2 screen	1 %
- Marsh 1500/1000 viscosity of 6% suspension in distilled water	40"
- settling of 6% suspension in 24 hours	2%
- water separated by pressurised filtration or 450 ccm of the 6 % suspension in 30 minutes at a pressure of 7 kg/cm 3	18 cm
- pH of filtered water	7 - 9
- cake thickness on filter press filter	2.5 mm
- density of normal mud	1.03 - 1.05 kg/l

The density of the mud, in exceptional circumstances, may be raised to 1.30 - 1.60 kg/l by mixing in barium sulphate whose density is more than 4.2.

When bentonite mud is left undisturbed it changes from a colloidal to a gelatinous state, a property known as thixotropy. This property is best measured by experience, by detecting the increased viscosity after extended undisturbed periods (1/ 2 / 5 hours).

In a permeable soil the mud infiltrates into the surrounding soil and these small, separated cells of mud act differently to the mud in the borehole which is kept agitated. The separated mud becomes still within the soil pores and, with its thixotropic property, the colloidal solution soon becomes a gel which binds the enclosing soil particles. This will apply for a distance of several centimetres into the side of the borehole and, in this way, the wall bonds together to form a so called "cake" which prevents the side of the borehole collapsing inwards (see also Fig. 12.5).

Bentonite mud is more viscous than water and holds in suspension particles of soil either not picked up by the drilling bucket or lost in the lifting process. In water the particles would drop to the bottom, with mud they are held by viscosity, deposition into the borehole is reduced and the particles are cleared from the mud by the desanding process, see section 12.4.5. However, viscosity, i.e. the internal friciton of mud, possesses negative as well as positive effects because, when too high it requires an ever increasing power of pumps used for conveyance and increasing periods for settlement in the storage tank.

Viscosity is tested at construction sites using a Marsh cone viscometer but whilst the actual unit of density is the centipoise (mud rises to around 20 cP), the usual measure employed at construction sites is a measurement of time.

12.3.3 Preparing of Bentonite Muds

Bentonite powder cannot properly be mixed with water through agitation alone and mixing is performed by turbine mixers which break down the bentonite lumps and cause solid particles to be intimately blended with water. Only with such dispersion may the clay particles absorb all the moisture necessary for swelling and thus acquire colloidal properties.

A good drilling mud should be mixed such that the percentage of bentonite is from 5% to 8% (of water), but it greatly depends on the quality and characteristics of the bentonite available. The maximum possible mud production that can be obtained with the Soilmec BE-10 mixer and a 1m tank is 12 m3/hour. Initially, 500 litres of water is fed to the mixer and then bentonite slowly added through the screen; when this is sufficiently mixed additional water is added to reach a total of 1000 litres of mud. Finally, the remaining bentonite powder is added and mixed in.

Bentonite drilling mud requires at least six hours to develop its colloidal and other desirable properties. A tank is required into which the newly mixed mud should be pumped where it should best remain and be kept circulating by a pump (see below) for at least 24 hours before use. With some bentonite powder, longer maturing periods are essential as, if the development of colloidal properties is inadaquate, the sides of the drill hole may not be properly supported and collapse. The tank ideally holds about 2 times as much mud as required to fill the pile and will normally be formed by cut and bank filling, perhaps to 2 m depth (see Section 10).

The bentonite mud must be kept circulating to prevent separation of bentonite from water . For this purpose it is advisable to disperse the flow around the tank and keep all the mud in motion. A pump connected to a 2" diameter pipe and with several different nozzles appears to work well. The pump should operate at regular intervals (30 minutes in action and 30 out, throughout maturing).

The mud recirculated back from a borehole will contain varying amounts of sand particles and, in order to recover this mud for reuse, a settling tank is required. In practice, a common settling/maturing tank is often used. This tank must be of sufficient size for the mud to lie quietly and the sand to settle to the floor. Mud for recycling is drawn off at the surface or, if this still contains an amount of sand above 3%, it will be necessary to pass the mud through the centrifugal de-sander before its return to the storage tank. At intervals, according to the amount of settlement from the mud, it may be necessary to clean the sand from he bottom of the tank by using a grab.

12.3.4 Checking and Testing of Bentonite Muds

Before and during drilling work, various properties of the mud should be monitored, i.e.: viscosity, density, pH, percentage content of sand and the presence of any contaminating substances. The frequency of testing and the method and procedure of sampling should be agreed prior to the commencement of the work depending upon the past performance in similar conditions.

The tests should be carried out at a new site or with new bentonite, with account being taken of the mixing process, any blending of freshly mixed bentonite and previously used bentonite mud and the process which may have been used to remove impurities from reused mud. In the event of a change in the established working pattern, tests for pH value should be required.

Property to be measured	Acceptable range of results	Test method at 20 degrees C
Density	Less than 1.10 kg/l	Mud balance
Viscosity	30 - 90 sec	Marsh cone method
Sand Content	Less the 3%	Baroid test tube
pH	9.5 - 12	Paper or electrical

Table 12.1

Special attention should be paid to measuring the density of the mud during drilling. If this is found to be greater than 1.3, it is a sign that a large quantity of soil material is present in the suspension. This material must be eliminated by the use of desander.

Bentonite kg/m3 of mud	Bentonite %	Density kg/l	Viscosity sec.	sand %
nil	0	1.000	27	1.0
20	2	1.010	28	1.1
30	3	1.020	30	2.2
40	4	1.025	35	3.7
50	5	1.035	40	6.6
60	6	1.035	40	12.0
70	7	1.040	45	19.0
80	8	1.045	55	35.0
90	9	1.070	60	68.0
100	10	1.075	70	92.0

Table 12.2

For measuring density relatively simple scales can be used. These scales should be clean and dry. The mud has to be drawn by dipping the container into a fully agitated point and the outside of the container must be cleaned and dried and the weight then taken. The density can be calculated knowing the weight and volume of the container.

For measuring the sand content a set is used consisting of:

- One Baroid test tube of glass graduated by percentage, ending in a cone at its bottom section
- One 2 1/2" diameter plastic cylinder equiped with a 200 mesh screen in its middle section
- One plastic funnel which can fit either end of the above cylinder.

To use the Baroid set the following practice should be followed :

1. The bentonite mud should be poured into the test tube until it reaches the "MUD" line.

2. The test tube should be filled with water up to the "WATER" line. The top section of the test tube should be covered with the thumb. The tube must be shaken thoroughly for a few seconds.

3. The content of the test tube should be poured into the screen, the funnel should be fitted to the side from which the mud was poured. The test tube should be turned by 180 degrees to introduce the end of the funnel into the test tube. A thin stream of water should be directed over the screen until the test tube is filled again as far as the "WATER" line, ensuring that no sand residues remain. With this operation all sand deposited on the screen should drop in to the test tube.

4. This operation should be repeated again twice in order to segregate the sand from mud accurately. At the third operation, after a pause of 15 minutes, the amount of sand contained in the mud can be read on the bottom cone of the test tube.

The sand content is expressed by percentage and should be between 1% and 3%. For values above 3% it is necessary to use the desander.

12.3.5 Mud Pollution

Drilling mud can be polluted to such a point as to negate its special properties. Mixing waters will not generally be a problem except in saline water but a check should be made when starting the works and repeated if any change of source is made or problems develop.

Concrete pollution is different and is a consequence of the process. Even with the use of a Tremie pipe to pour the concrete, drilling mud comes into contact with the rising fresh, concrete surface and pollution from Calcium ions in the cement is inevitable. For this reason, mud from the last two or three metres of casting should not be recovered and returned to the storage vat, but always discharged as waste.

12.4 Piling Procedure

12.4.1 Outline

The basics of the pile drilling process are illustrated in Fig. 12.1 to 12.10. The equipment used to drill the hole consists of a crane with a diesel powered drilliing head attached, which drives and turns a telescopic kelly-bar. To the kelly-bar is attached a drilling tool (an auger or, more commonly, a drilling bucket). The whole is mounted upon a stable base, either a sufficiently large (100 tonne) pontoon to work in the river or wooden crane support mats (Section 10) for soft ground conditions.

When the machine has been positioned accurately, with the drilling tool over the pile position, the motor of the drilling rig turns the kelly which rotates the drilling bucket. This is lowered to the ground where teeth on the underside of the bucket cut into the soil gradually filling the bucket. When it is full, the kelly lifts the bucket, turns away from the borehole and the bucket is opened and empties. This sequence of operations, shown in figs 12.2, 12.3 and 12.4, is repeated until the required depth has been reached. During this process the cut material must be logged in order to check that the strata are similar to that for which the pile design was made.

In order to drill with bentonite, a short steel guide casing is used (Fig. 12.1 and 12.2) such that the top level of this casing is, at all times, more than 1 m above the river or ground water level. Thus the bentonite is contained at a level such that at all times during drilling, the pressure exerted by the bentonite exceeds the pressures exerted by the soil and external water. The level of bentonite in the borehole must be continuously checked and maintained by topping up as the pile is cut deeper and as mud is lost with excavated material.

In the event of rapid loss of bentonite from the pile, as may occur through encountering a void or a lens of course gravel, the borehole shall be backfilled without delay, if necessary with weak (50 - 75 kg/m3) mix concrete. The engineer in charge must be consulted before excavation at that location is resumed.

When the design depth of the pile has been reached (Fig. 12.4 and 12.5), the pile must be cleared of all soil particles by desanding (Fig. 12.6 and Section 12.4.5). Then reinforcement is added (Fig. 12.7). After putting the sectional tremie pipe and hopper (Fig. 12.8) concreting is made by filling the pile from the bottom upwards with sections of tremie being removed. The bentonite mud which is displaced out by the concrete is discharged back to the settling tank for reuse (Fig 12.9.)

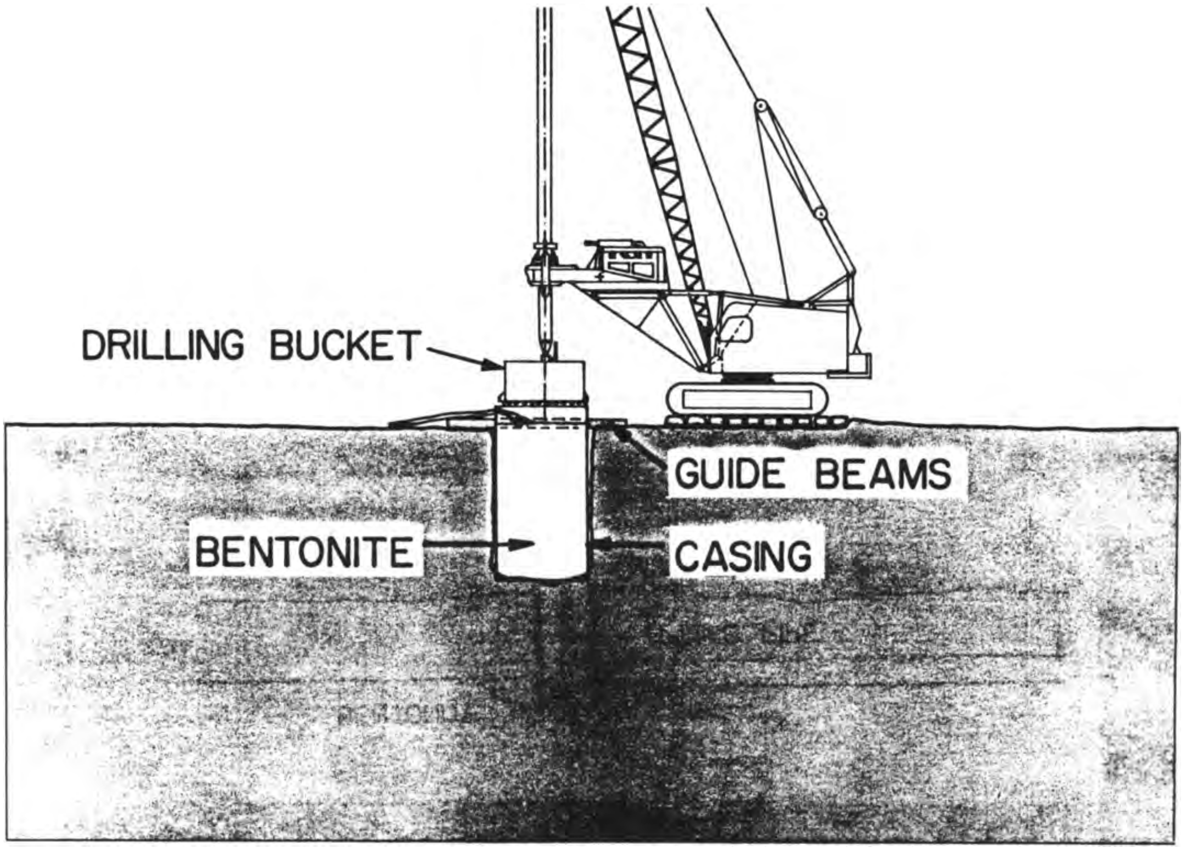
12.4.2 Installing the Guide Casing

On-shore it is normal to use a casing approximately 3 - 5 m long. The casing is best positioned by direct measuring from offset pegs placed earlier and then driven in perhaps 1 m. At this time, its verticality and position should be checked before completing the vibration of the casing, leaving its top some 0.5 m or more above the proposed pile level.

Off-shore the procedure differs slightly (Fig. 11.3). As part of the setting-out process, the drilling pontoon is positioned and steel beams are fixed by welding to guide the casing. With the position checked, the casing is lowered into place and, as it reaches the river bottom, its verticality must be checked and then rechecked as it is driven into the river bed. The casing is generally driven at least 2 m into the river bed

Foto 7: Driving of Temporary Casing

Foto 8: Lowering of Steel Reinforcement


DRILLING BUCKET
GUIDE BEAMS
BENTONITE
CASING

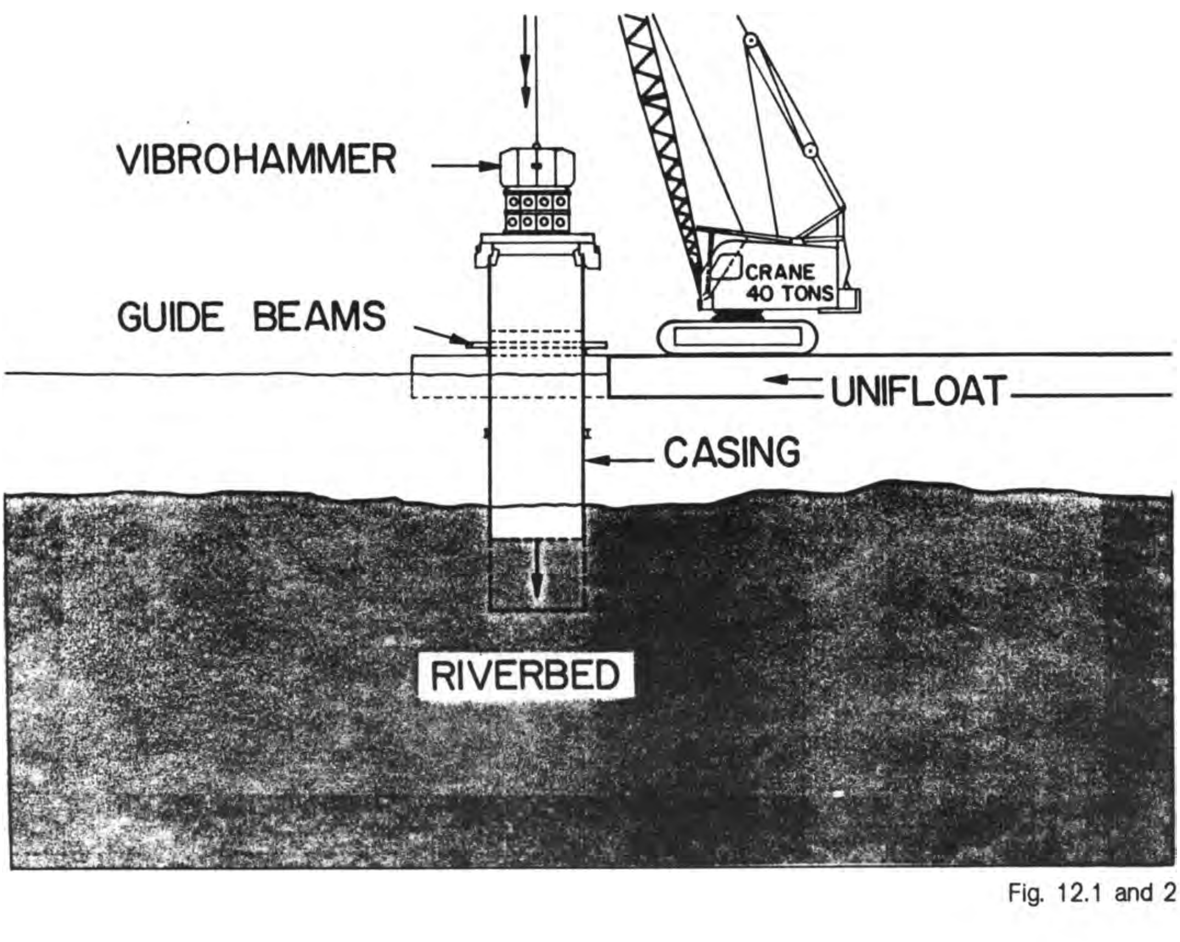
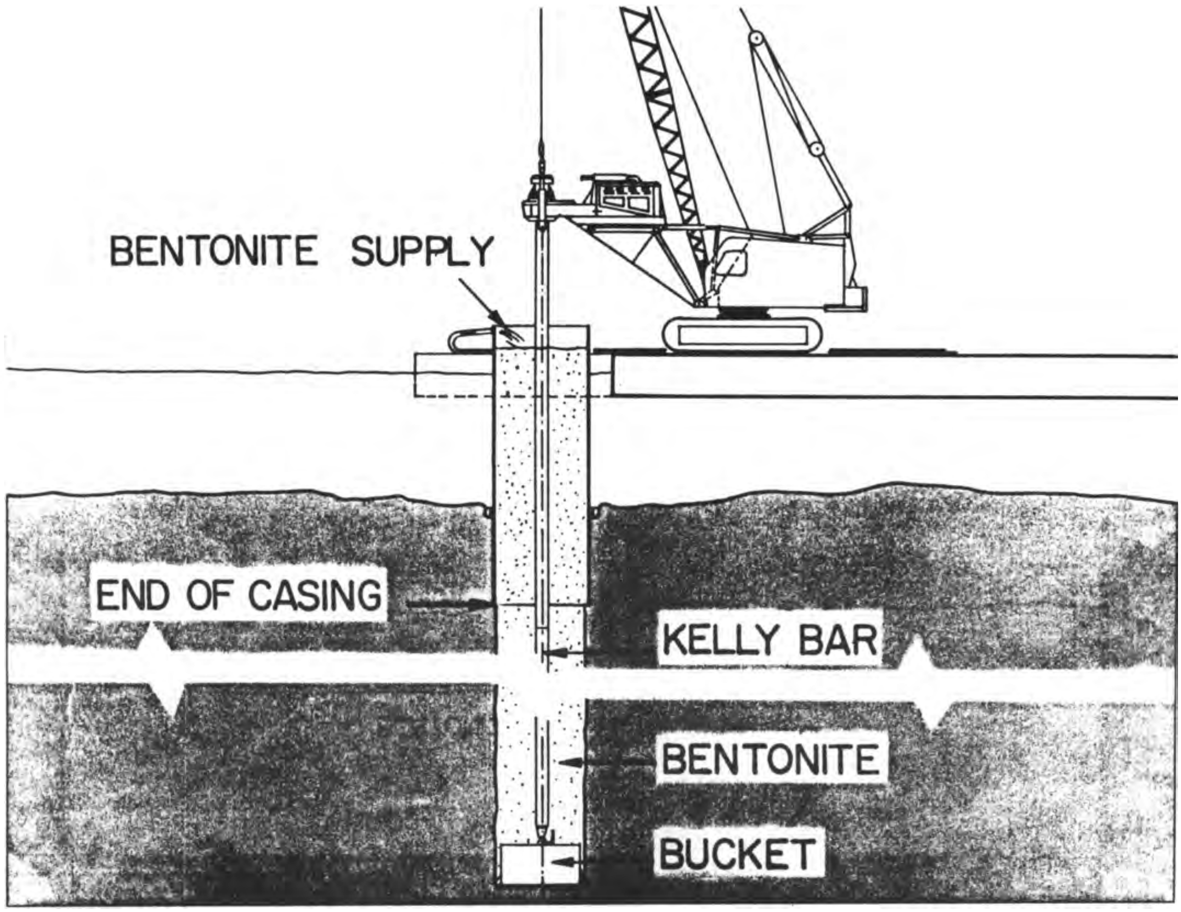

VIBROHAMMER
GUIDE BEAMS
CRANE
40 TONS
UNIFLOAT
CASING
RIVERBED

Fig. 12.1 and 2


BENTONITE SUPPLY
END OF CASING
KELLY BAR
BENTONITE
BUCKET

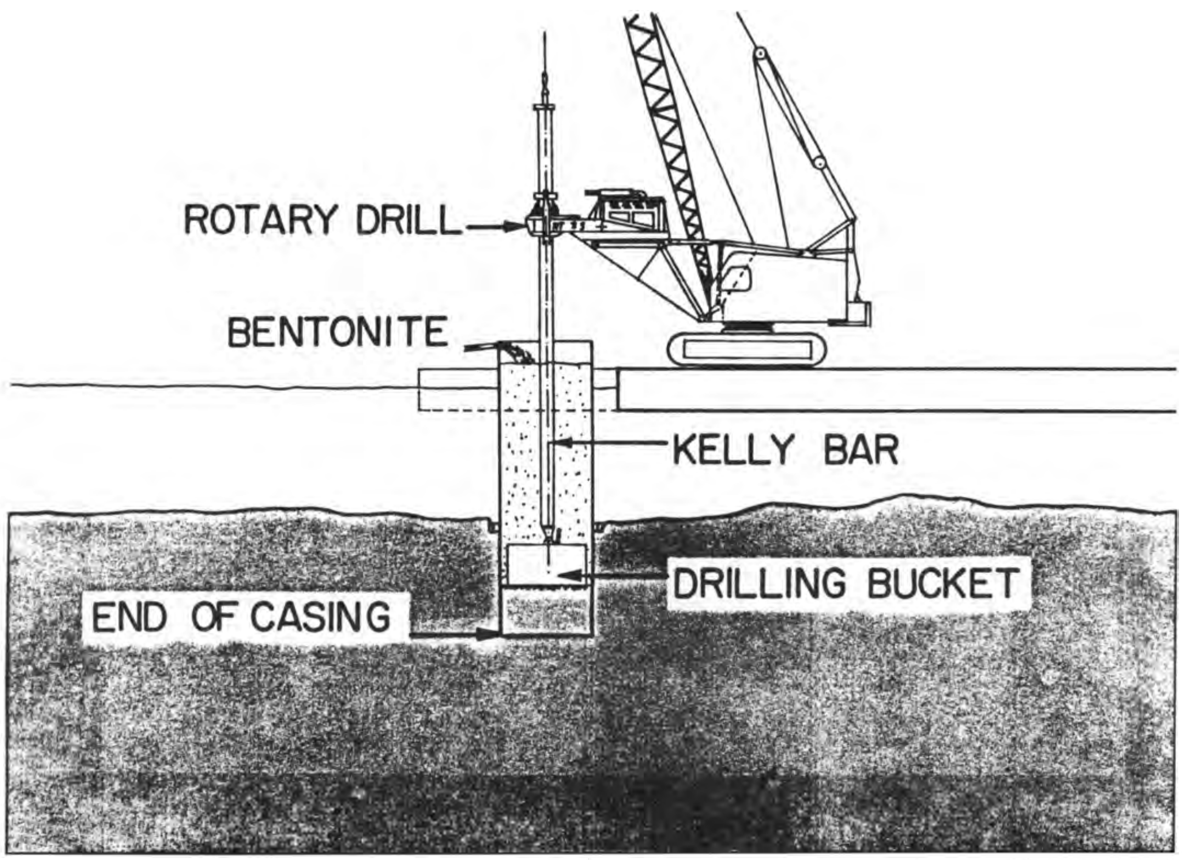

ROTARY DRILL
BENTONITE
KELLY BAR
DRILLING BUCKET
END OF CASING

Fig. 12.3 and 4

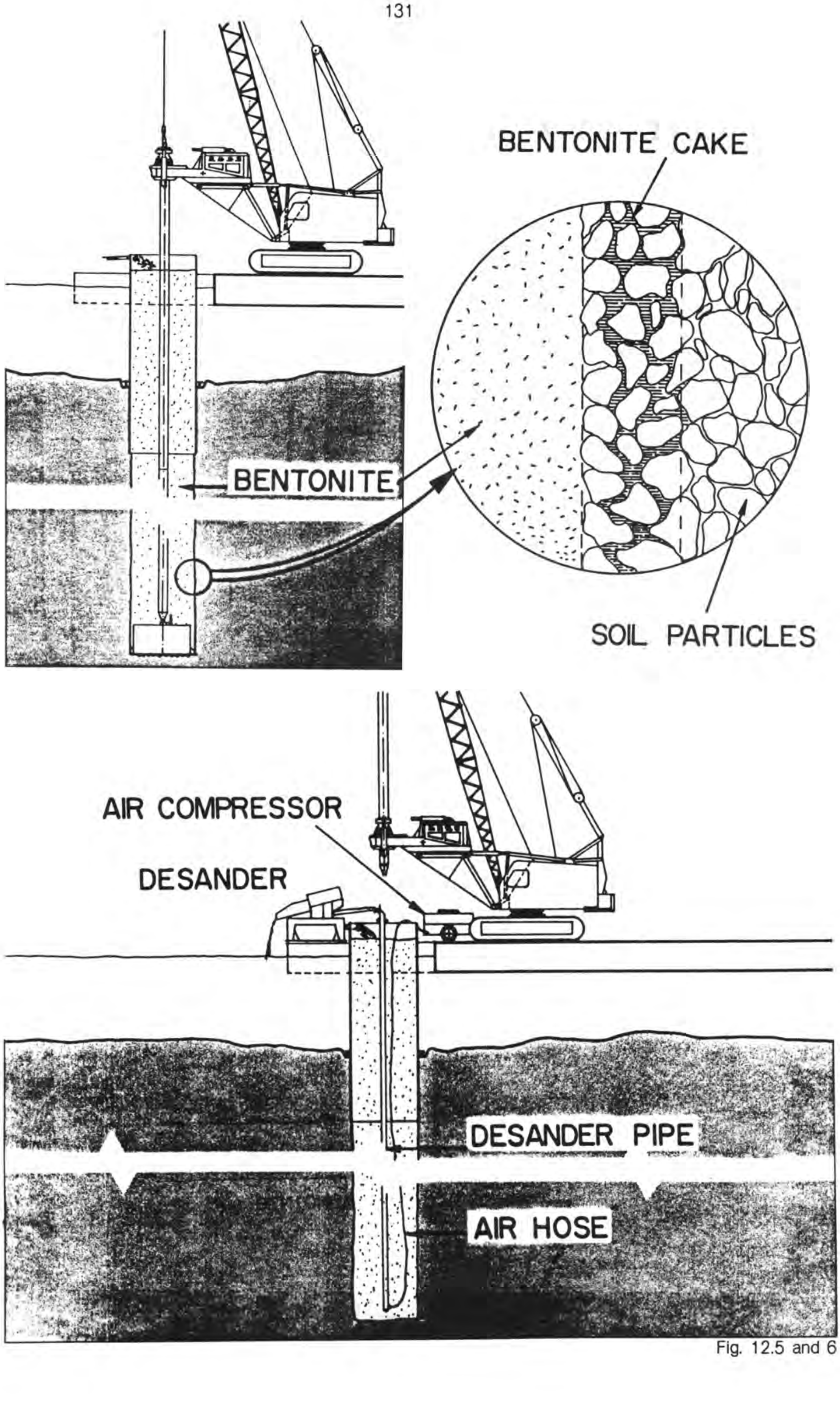
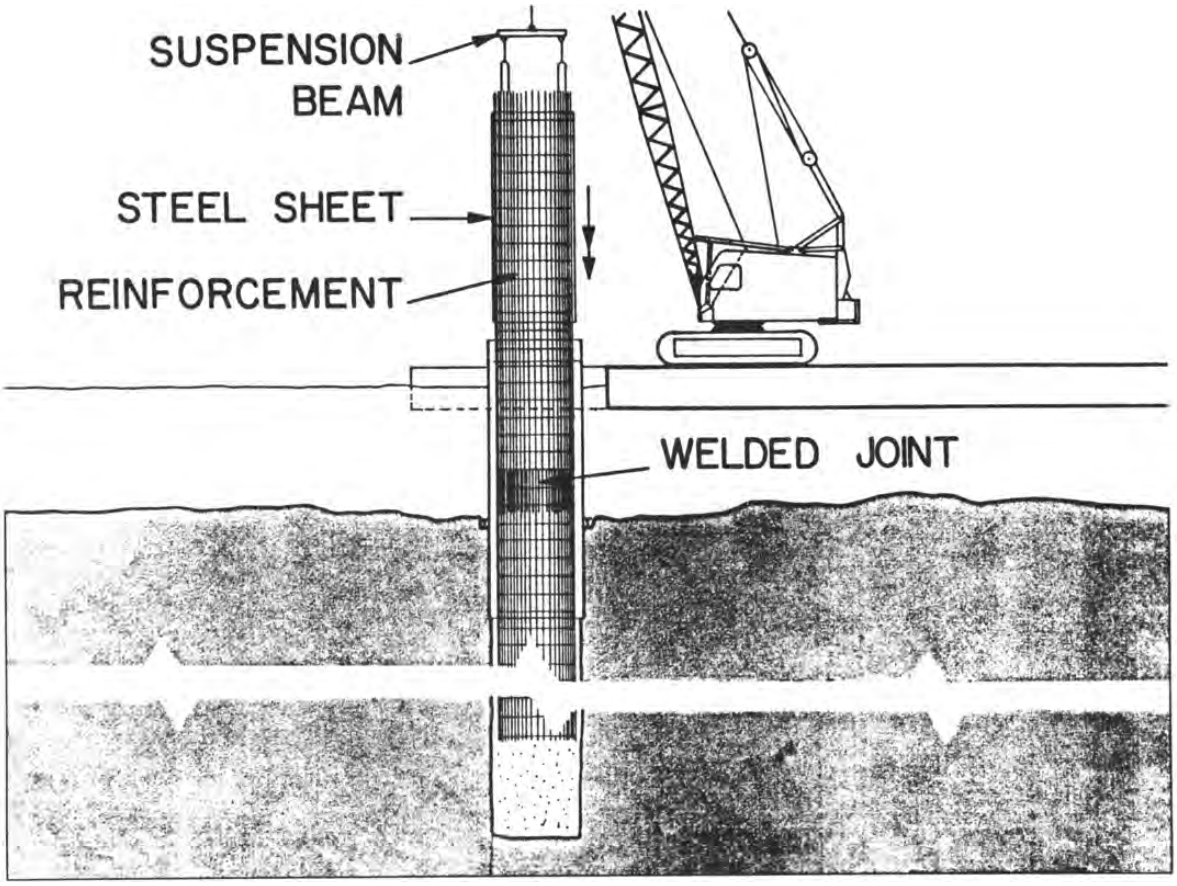

BENTONITE CAKE
BENTONITE
SOIL PARTICLES
AIR COMPRESSOR
DESANDER
DESANDER PIPE
AIR HOSE

Fig. 12.5 and 6


SUSPENSION
BEAM
STEEL SHEET
REINFORCEMENT
WELDED JOINT

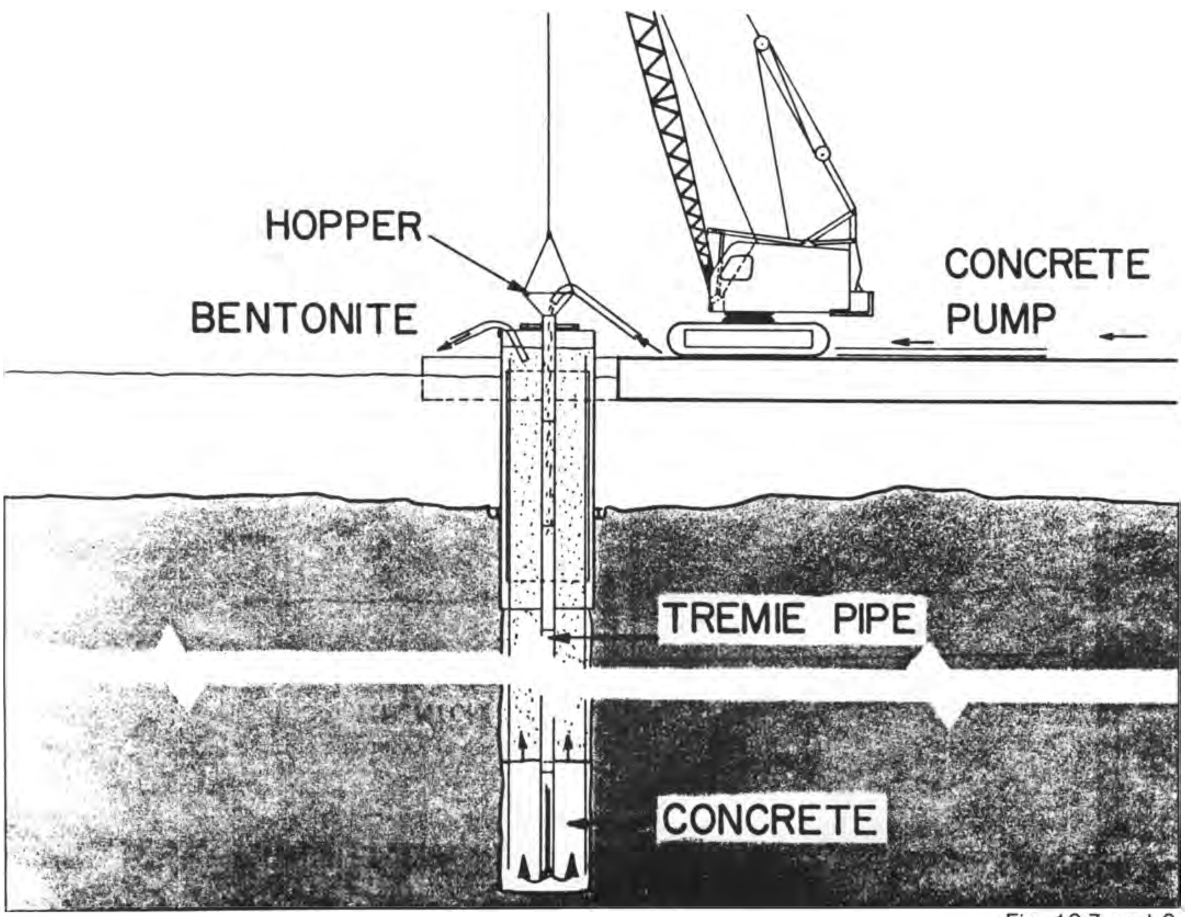

HOPPER
BENTONITE
CONCRETE
PUMP
TREMIE PIPE
CONCRETE

Fig. 12.7 and 8

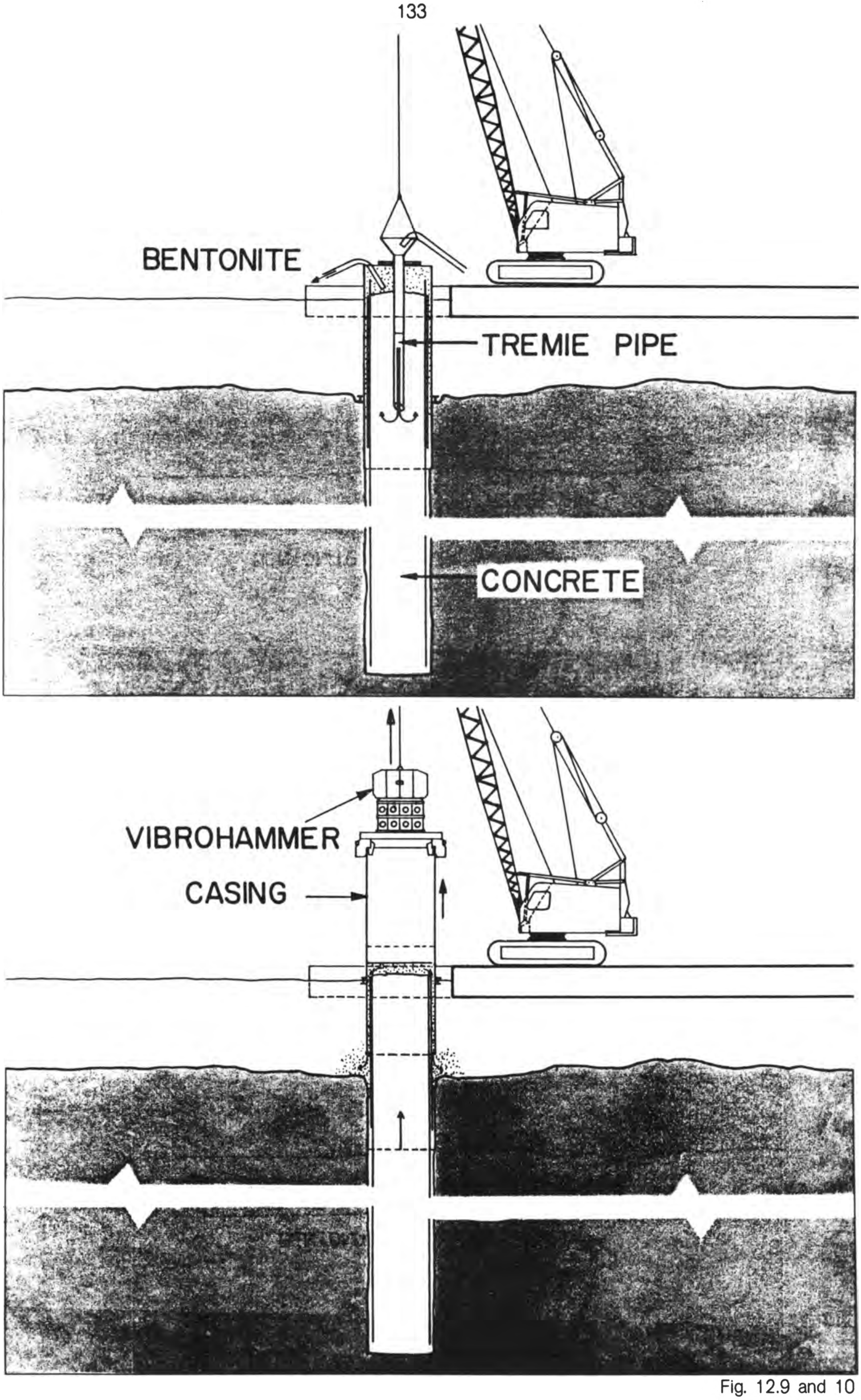


Fig. 12.9 and 10

and this should be increased if very soft ground is encountered. In some cases it has been found that the casing subsides further into the river bed during drilling. To prevent this, short sections of a H- beam are welded, transversely, to the outside of the casing at the length required down to the river bed plus an allowance for some sinking into the bed (at Kyoung-gone 0.2-0.5 m was required). This should prevent excessive movement into the river bed.

The top of the driven casing must be at least 1 m above water level (check 1 m above high level if tidal) and also a minimum of 0.5 m above that of the pile top. The guide beams should then be removed. The pontoon position must be checked and moved if necessary to ensure it does not touch the casing as the drill rig causes the pontoon to move during drilling work. Finally, before drilling, the position of the casing should be checked for both line and distance. The crane/drilling rig is then moved to a position ensuring ease of spoil disposal and the kelly bar made vertical using the hydraulic ram adjustment of the drilling rig. Bentonite mud must now be pumped into the casing to at least a metre above water level and drilling may commence.

12.4.3 Boring Log

During drilling, a log of boring progress is required (see also Section 16 and the sample from Inma bridge, Fig. 12.11). The operator of the driling rig should be responsible for keeping this record as he is necessarily involved from start to finish of a pile. The site engineer should check that it is being produced and take notice of the soils excavated especially as the pile reaches realistic depths where decisions to utilise different bearing layers may be made. A comparison of the 'as found' soil conditions with the 'as designed' pile should provide the basis for the engineers decision. The log will record time, depth of pile, soil types and any special problems encountered and should also include bentonite conditions. Measurement of depth will most conveniently be with the specially marked measuring rope and description of the excavated material should be as Section 3.4.

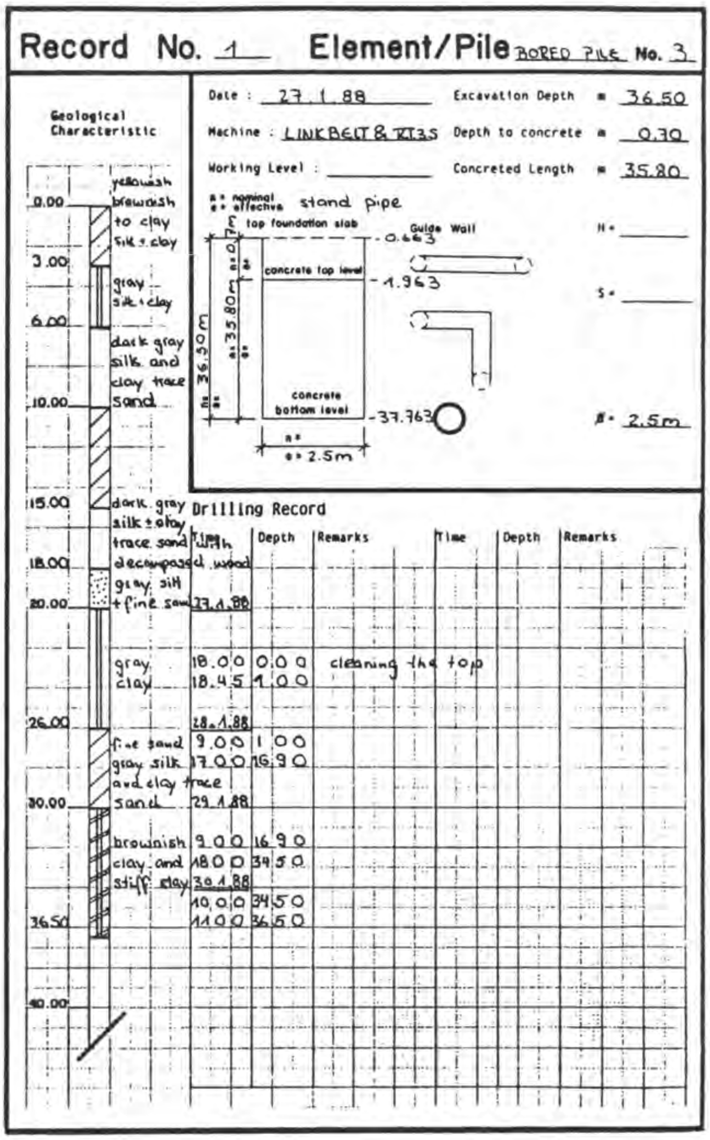


Fig. 12.11

12.4.4 Tolerances for Positioning the Drill Rig

The tolerance required will vary with the design of both the pile and the pilecap/superstructure but, for 2.5 m diameter piles used for the initial bridges, positional errors of 0.2 m and verticality errors of less than 0.7° have been established.

For setting-out the positions of piles on-shore no special measures are required other than offset pegs to permit checking after initial installation of the casing. For off-shore positioning see section 11, Setting-out.

12.4.5 Desanding

Once the design or other agreed pile depth is achieved, the borehole must be cleaned. Throughout the drilling process, small soil lumps and particles cut out, drop into the bentonite and then slowly sink to the base of the hole. These must be removed before concreting.

The drilling bucket is removed from the rig by using both cables of the crane. The bentonite desander and the air compressor are installed at the borehole and a bentonite return pipe with attached air hose is lowered section by section to the foot of the drilling. When all connections are made, compressed air is pumped to the foot of the borehole to return up through the desanding pipe. As it rises, it carries drilling mud mixed with loose particles up from the bottom of the drilling to the desander. The desander is a centrifuge in which the heavy particles are spun out of the drilling mud. It is operated to ensure the bentonite separates from the sand and small impurities. These are discharged to the river whilst the clean bentonite is returned to the borehole. The process is continued until the sand content of the mud is below 3%. Immediately before installation of the reinforcement, the drill bucket is again fitted and lowered to the foot of the borehole and turned to clean any further material which has settled off the base of the hole.

The centrifugal desanding unit consists of a cylindrical-conical steel body lined with a special rubber to withstand abrasion. The mud returned from the pile to the unit is spun around and sand and soil particles segregate out to the cylinder wall and drop to the base of the cone as the rotation is gradually reduced. The clean mud returns out at the top whilst the sand collects at the bottom outlet from where it is discharged to waste.

In the light of present experience and knowledge, the recommended flow rate of mud back to the desander should be between 300 and 600 litres per minute. This depends on the quantity of sand and soil in the mud, and its particle size and density. The speed of rotation is, however, critical: too slow and separation will not occur, too fast and the unit will eject mud as well as sand.

12.4.6 Reinforcement

Reinforcement cages are prepared at site in sections, generally to a maximum length of 12 m, in accord with design drawings. In general, four or five cages will be necessary, requiring joining together of steel sections during lowering into the pile casing. To minimize damage to cages during reinforcement fitting it is necessary to follow the procedure indicated in Fig. 12.12:

1) use of a purpose-made lifting beam
2) welding of 25 mm steel lifting lugs to the top of the cage, and
3) welding of at least 3 binders within the cage to each main bar.

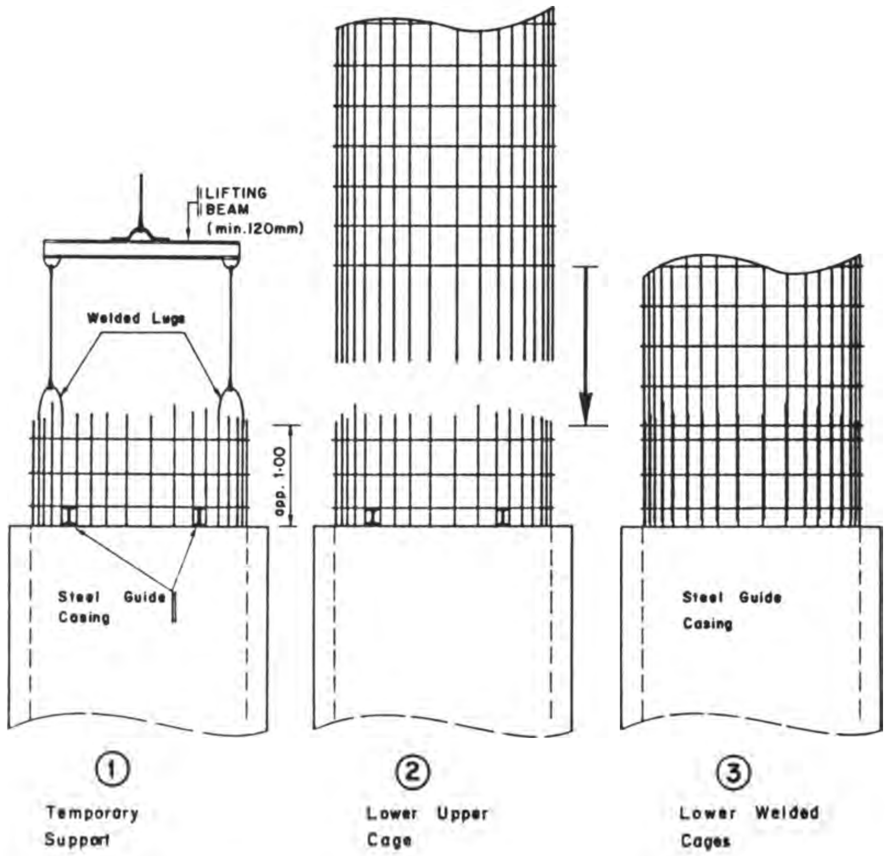


Fig. 12.12

The reinforcement cages should be prepared as shown in Fig. 12.13:

- Check that spacing bars are sufficient to ensure minimum concrete cover to steel and that the cage is correctly dimensioned (100 mm cover was selected for the project).
- Lower the cage until its top steel is approximately 1 m above the top of the casing.
- Put two short (3 m length) steel H - beams through the reinforcement beneath a binder hoop (which must be welded to each main bar) of the cage to rest across the top of the steel casing.
- Lower the cage onto the H - beams and ensure that the support is strong and secure.
- Remove the lifting beam and cut off the lifting lugs from the installed cage.
- Lower the next cage onto the installed cage.
- Ensure overlap is 45 times the main bar diameter.
- Weld together a minimum of 10 main (longitudinal) bars, each 200 mm length.
- Lift joined caged sections to take out the supporting H - beams.
- Lower to correct level or to 1 metre below the top of the next cage for next installing sequence.

The top of the pile will be cast within the steel guide casing and a lining to this casing is required to ensure that it may be removed after concreting without damage to the fresh concrete in this critical, exposed section of the pile. The lining is formed of 2 mm sheet steel welded to the spacer bars of the reinforcement cage as shown. It must extend from the level of the base of the casing up to the level of the top of the pile, normally only a few metres but possibly up to around 15 m if piling is made in deep water.

The reinforcing cage must be measured as it is lowered in order to know its position. When all the steel is installed the whole should be adjusted by raising or lowering to ensure the top reinforcement is at the correct level to leave sufficient lap steel to accord with the design for the pilecap. The cage should not rest on the base of the hole and will generally be 1m or more above it .

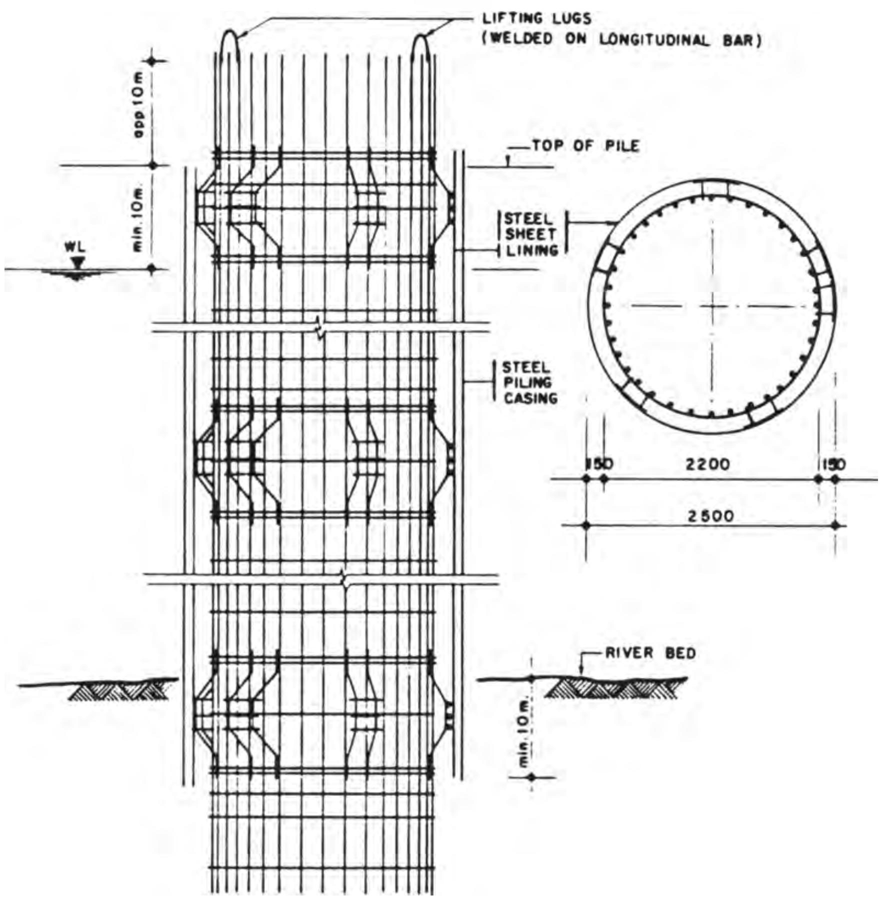


Fig. 12.12

After installation of the reinforcing cage, the 260 mm diameter tremie pipes are installed. First the holding fork is fitted and then, one by one, the pipes screwed together and the extended pipe lowered. These should be permanently numbered, their standard length known, and the numbering sequence as they are installed recorded. It is important at all times during concreting that the level of the foot of the tremie pipe is known such that the outlet is always below the level of the new concrete. To this end, the volume of concrete pumped and the depth to the new concrete level should regularly be measured. In addition, immediately before concreting it must be set to not more than 0.5 m above the bottom of the borehole and the hopper and feed pipes from the concrete pump installed. During this period, the level of bentonite must be maintained and the concrete and bentonite outflow pipes fitted.

12.5 Concrete for Large Bored Piles

12.5.1 General

In order to supply the required volume of concrete used for the large bored piles concrete pumping from the batching plant was decided upon. This involves some modification of the concrete mix in order to ensure it is liquid enough to be pumped. After tests with the cement, aggregates and water, it was found that a mix made with 300 kg of cement for each cubic metre of aggregate would meet the design strength of 25 N/mm2. In Myanmar, the cement used is frequently as high as 350/375 kg/m3 metre but this was considered excessive. The cement normally used is Portland 325 type, but in some cases, chiefly in presence of salt water, either "Pozzolanico" type or "Alto Forno" type (always the 325 range) is used (see also Section 5.7). In Myanmar it is not normal to use admixtures to increase workability, thus the water/cement ratio will have to be increased to enable the concrete to be pumped.

12.5.2 Concrete Casting

Following the desanding of the bentonite, the process of installing steel reinforcement, together with the associated tremie pipework preparatory to concreting may be expected to occupy at least 12 hours. For successful concreting, the following steps should be taken:

- The first batch of concrete must be isolated from the bentonite in the tremie pipe by a plug. This will normally be a plug of paper approximately 20 cm thick.
- The concrete should be poured without interuption at a rate of at least 15 to 20 m3/hr. The rate of feed should not be too slow, as this can produce a "plug" of set concrete in the pipe which would have to be withdrawn to remove this.

The bottom of the tremie pipe must always be submerged to a depth of at least 1 m in the concrete that has already been poured as, if the depth is less than 1 metre, backflow of bentonite may occur. If the pipe is inserted too deeply results may be equally undesirable in that the fresh concrete may rise in passageways through the concrete already cast, or the concrete will barely flow down the tremie pipe. As this is continuously changing, care with checking the depth and removing sections of the tremie is necessary. The limits to the depth of submerging are between a half and a maximum of, normally, six metres.

During casting, as the borehole fills with concrete the bentonite drilling mud will be displaced upwards. The new concrete surface should rise evenly from the bottom of the hole to the surface and fully fill irregularities in the sides of the borehole. The placing of concrete with the tremie should aim to produce the least mixing of bentonite into the concrete .

The used bentonite is pumped back to the storage tank for reuse with care that its level does not drop below water level plus one metre. This applies until the concrete is almost up to its designed level. The last two or three metres or so of bentonite will be badly polluted with cement and is no longer useful and it should be allowed to flow to waste.

Concrete must be poured to above the required level for the top of the pile. The last 0.5 m of concrete in contact with the bentonite mud will be of poor quality and it is usual to pick it away as soon as this is reasonable, or in any event within two or three hours, before it becomes hard and difficult to remove.

Upon completion of concreting to the finished level, the last tremie pipe should be withdrawn and all pipes and pumps moved and cleaned thoroughly. The final step in the concreting process is the withdrawal of the steel guide casing installed to permit drilling. The sheet-steel sleeve installed with the last section of reinforcement ensures this should neither be difficult nor should it disturb the new concrete.

12.5.3 Concrete Mix and Water / Cement Ratio

The water/cement ratio must be carefully controlled as too much water is the primary factor in reducing concrete strength. An important responsibility of the site engineers is to exercise control during concrete mixing to control the water-cement ratio ! The quantity of water added to the mix must allow for the water carried by the surface of the aggregates used. This water content should be determined by tests and adjustments in the quantity of aggregates and water shall be made on the basis of the test results involving both slump and strength tests on the actual cement, aggregates and water to be used. In the absence of exact data, the amount of water may be estimated from the following values :

very wet sand :	120	l/m3
moderately wet sand :	80	l/m3
moist sand :	40	l/m3
moist gravel or crushed rock :	20-40	l/m3

The optimal consistency of concrete for pile-casting with concrete pumping was found with:

Proportion of cement :	300	kg/m3
Water/cement ratio :	0.48 - 0.50	
Slump, Abrams cone :	110 - 150	mm

Table 12.1

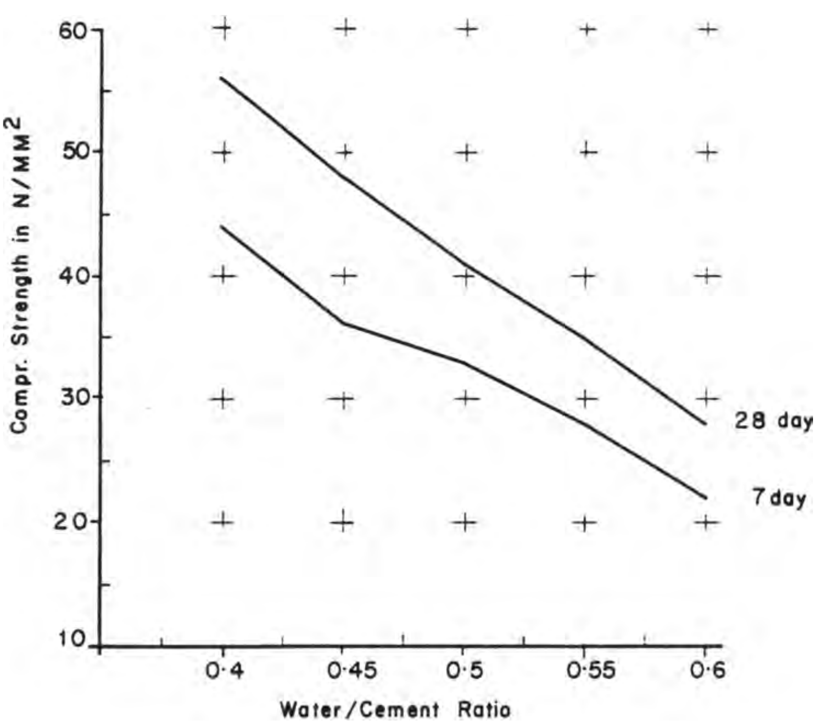


Fig 12.13

The cube strength of the concrete 28 days after the date of casting should be above 25 N/ mm2 and experience in Myanmar has indicated that for piling, concrete of the required strengtht may be pumped up to 150 m before the liquidity has to be increased to such a degree as to make the resulting concrete too weak.

It is rare to use retarders in concrete for large bore piles. Experience has shown that even if a pile-casting operation takes 12 to 18 hours, using concrete with properties as above, ensures that there is no danger of setting occuring provided the casting is carried out without interuption. A 15 to 20 minute break can remove concrete fluidity and both its ease of casting and pile wall penetration may be impaired. Additionally, piping at the tremie outlet may introduce a weak layer into the pile. A longer break, caused perhaps by a breakdown of the mixing plant will always be very serious . If it is within reach of the surface it may be possible to pump out the bentonite, physically go into the pile and break out the top half metre or so of weak, bentonite spoiled concrete. If the discontinuity is lower, a decision to abandon or to accept a weak pile may only be made by the senior project engineer.

12.5.4 Concrete Tests

Testing the crushing strength of trial mixes and then at least one representative sample of three cubes from each pile is the easiest and most reliable means of ensuring that a continued good quality of concrete is being made. Trial mixes should be used to test the cement and aggregates available for the pile and the repeat tests are the means to prevent a loss of quality during operations .

For standard cubes, the following may be assumed :

Age in days	7	14	28
Development of Compressive Strength	80%	90%	100%

Table 12.2

13 Erection of Bridge Steel Works

13.1 General

13.1.1 Procedure

The following description applies to those steel truss bridges of the Yangon Pathein road supplied by Transfield (for design see section 7). For other manufacturers, and for other steel truss bridge types, the principles will be similar but many of the details will, necessarily, be different. For simplicity the truss members are designed to be assembled in a particular sequence according to the order of assembly. It is assumed, in the manual, that the bridge structure will be erected by the piece-by-piece method with cantilever spans although the structure could also be erected on falsework. The erection procedure requires an initial anchor span to be assembled on land, on the approach behind an abutment. Special link set members will be fitted to start the cantilever assembly out from there . Truss members will be added until the span lands at the far side or intermediate pier. Then link members and anchor span will be removed. For multi-span bridges the process is repeated. A gantry runway and a lifting frame at the truss tip are temporary devices during erection.

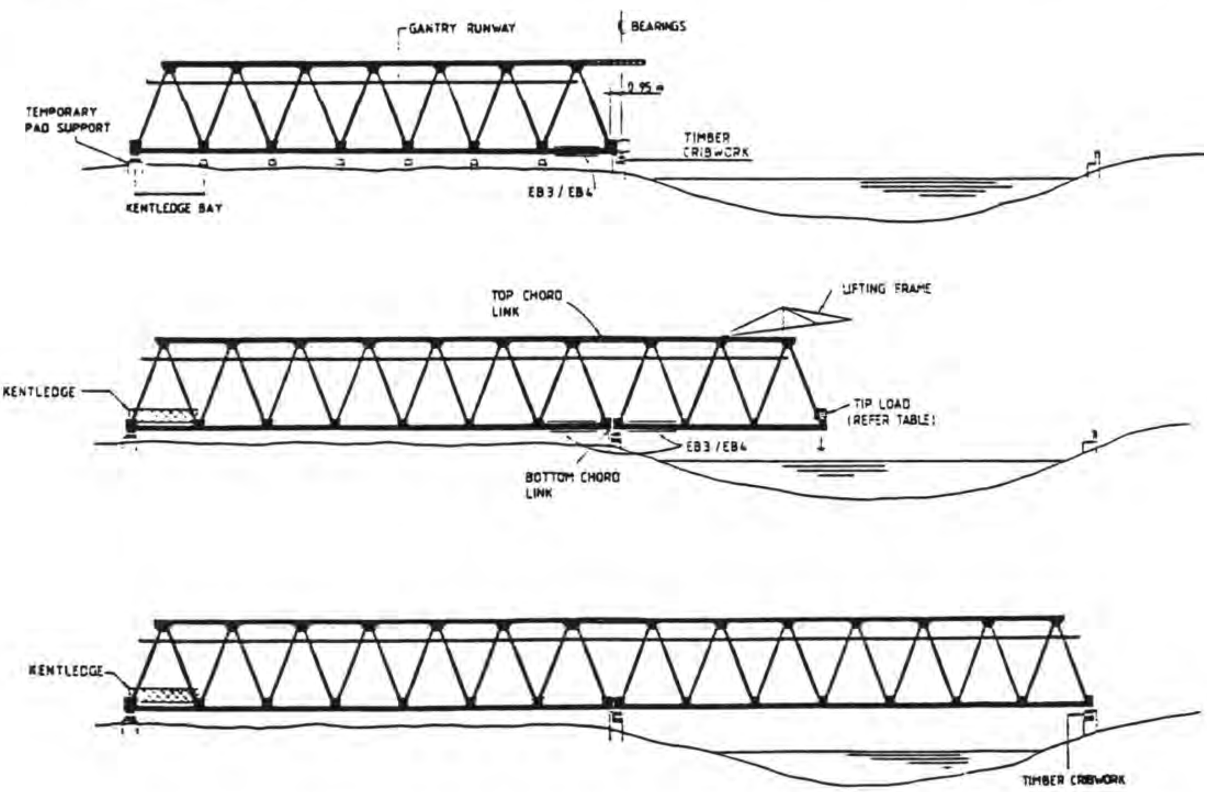


Fig. 13.1

As soon as bridge parts are delivered to site all elements should be checked and repaired if required. Prior to assembly, parts should be again inspected and any damage, including damage to the galvanising treatment must be rectified.

13.1.2 Fitting of Components

The steelwork is designed for field splices, all connections are made with 24 mm high tensile strength bolts except those for the walkways. No welding of steel components is required nor should be permitted on site.

With in-place, piece-by-piece cantilever erection each joint of the truss members must be completed, fully and permanently bolted and tensioned as soon as all members in the joint are installed, before proceeding with the next bay. Excepted are the leading end chords of the anchor span and trailing end of the permanent span, which must be reinforced. Generally, there is no requirement for the removal of bolts, plates, etc. or the addition of members after initial assembly and erection.

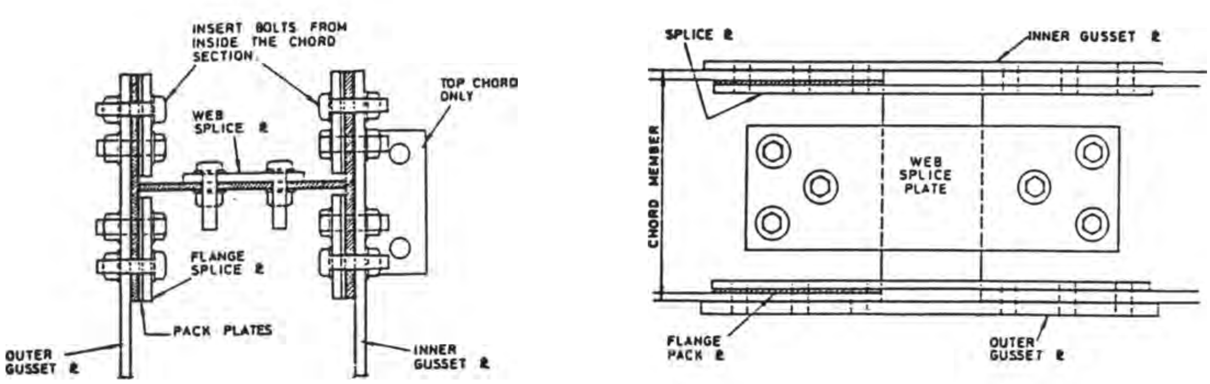
Drifts are supplied with the erection gear and are to be used to align and bring the various parts into place, but excessive hammering, leading to distortion, should be avoided. When a component is fitted into a joint it should be accurately located and held by use of the supplied drifts, in order to centralise all components before bolting.

Where a truss member is to be installed between two gusset plates which are already bolted to other components, it is important that the member is inserted one end at a time, with good control to avoid jamming. It has been found that this is normally achieved without difficulty because of the end clearances provided between members within truss joints. However, if necessary, the gussets may be eased apart using a timber block to assist in the insertion of the member.

13.1.3. Truss Chord Splices

Top and bottom chord gusset and splice plates have been fabricated to particular dimensions such that the required camber is achieved in the assembled span. A common and interchangeable gusset is used throughout the bottom chord connections, and similarly to top chords. Gussets are not interchangeable bottom with top however (Marks differ).

The inner and outer gusset plates at top chord connections differ (in that a bracing cleat is welded to the inner plate) and are not interchangeable, see Fig. 13.2. The inner (roadway side) gusset plates to the top chord are of two types Mark BG2L and BG2R differing in the arrangement of the bracing cleat. All gussets on one side are to be of the same Mark. This arrangement is indicated on the truss marking plans (Fig. 13.3).



Section Plan Fig. 13.2

Foto 9: Truss Splices

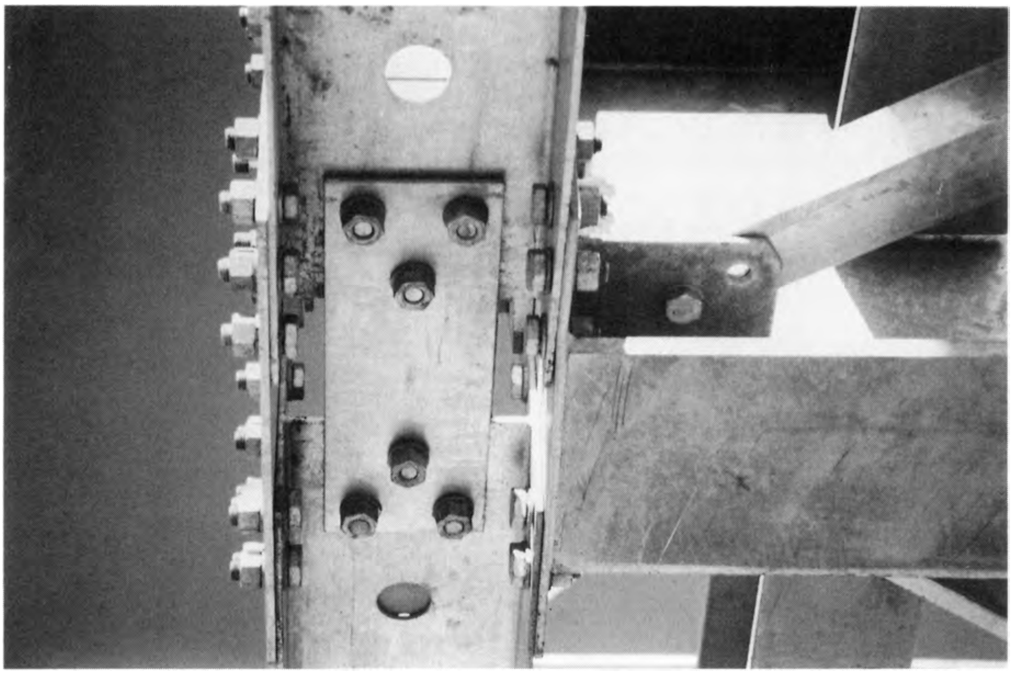
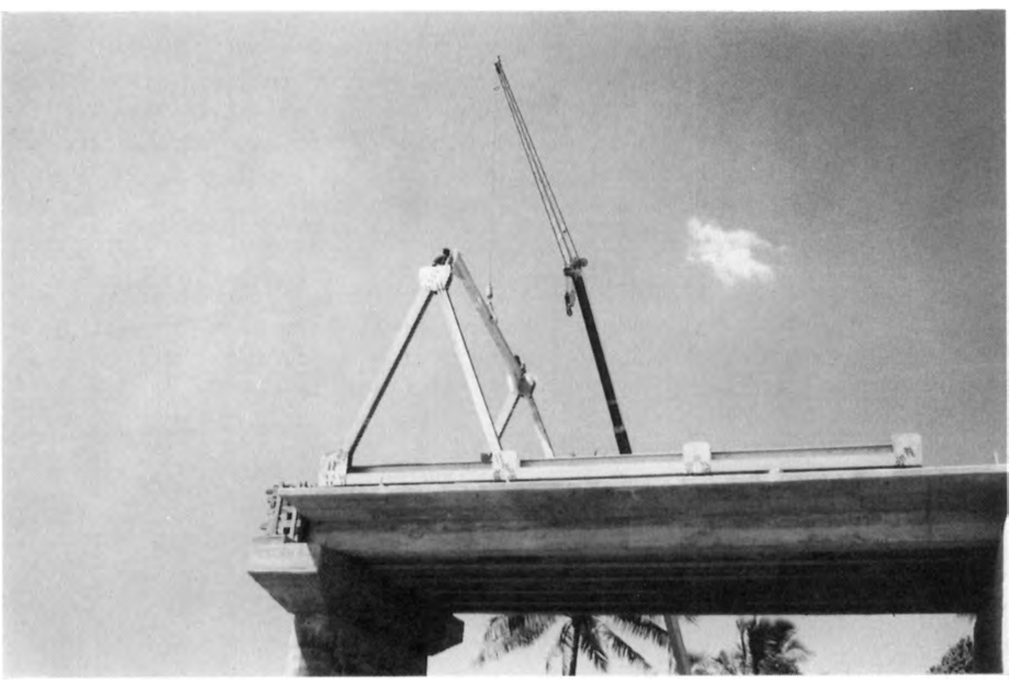


Foto 10: First Elements of Anchor Span



The bottom chord gusset plates are of two thicknesses to suit the "with" and "without " walkway designs. The two thicknesses of plates are designated by a different Mark indicated on the drawings. It is critical that on a bridge with a walkway that the thick plate is used.

Where the flange plates of end-for-end abutting chord members are of different thicknesses, then shim or pack plates should be provided under the splice plate to make up the difference in thickness. A correct combination of plates is required as indicated on the marking plans. Where a single splice plate is detailed to be provided to the chord web, this is to be located on the upper surface of the web.

Bolts to flanges or side plates of chords are for installation from inside, that is with the bolt head on the inside of the section (Fig. 13.2). Bolts of the correct length shall be used as described in section 13.6.

13.2 Anchor Spans

13.2.1 Arrangement

Erection of the truss bridges by cantilever requires the use of an anchor span to serve as a balance for the weight of the cantilever span. The anchor span will most conveniently be a standard 36.5 m truss span, which may be either a second span for the same bridge or from another site or store. For a single span bridge or the first span of a multi-span bridge this will be a specially erected span, for second and subsequent spans the initial span, linked through the special link set members will act as the anchor.

The anchor span will comprise all the components of a standard span. It is best assembled in the sequence and arrangement (as detailed in 13.2.3) as if it were a permanent span, the only exceptions being the addition of chord reinforcing. Because the anchor span is temporary and is to be dismantled, the bolts are to be tightened firmly , but are not to be fully tensioned as for a permanent span. A standard hardened washer is to be provided to the head and nut. On dismantling, the bolts, nuts and washers should be cleaned, oiled, sorted and packed in appropriate bags for reuse in other spans, except that damaged bolts are to be discarded and replaced.

The anchor span must be assembled and, in order to give the correct levels to the cantilever span, is best set on temporary supports under the bearing plates at the trailing end and timber cribwork located under the end cross-girder at the leading end. Unless special precautions are taken the anchor span will always require kentledge as a counterweight to prevent the overturning of the cantilever.

13.2.2 Base for Erection

The assembly of a temporary anchor span on one bank requires that the approach on the side selected should be cleared and graded approximately to the correct level over an area extending back from the abutment for a minimum length of 50 m and for a width of 18 m.

The surface of this area should be graded such that it will drain. The ground should be compacted and stable batters provided to filled embankments in accordance with normal earthworks practice.

The bases are to be designed for not less than the specified load, taking into account the nature of the supporting soil. It is suggested that a lower than normal safety factor (leading to a higher than normal bearing pressure), can be adopted, considering the temporary nature of the load , that it is a construction load, and that it is possible to pack under the rollers. As a guide it is suggested that a bearing pressure of 0.5 N/mm2 could be adopted on sound clay or roadway filling compacted with rollers.

It is envisaged that for bases, timber cribwork will be utilised (see also 13.7.2). The centre of these bases must be located in the position detailed within a tolerance of 100 mm, with finished level within 25 mm.

For a bridge with concrete sidespans, the anchor span will be constructed on the concrete slab, approximately 1.5 m above its permanent level. After completion of the first span and removing the link set, this first span lying on the temporary timber supports can be lowered to a height of approximately 20 cm above bearing level with the procedure described in section 13.4.

13.2.3 Anchor Span Assembly

The following sequence applies for the anchor span, or for any span erected other than by piece-by-piece cantilever erection. Labels refer to Marking Plan, Fig. 13.3.

1. Bolt bearing assembly (BBA) to end cross-girders (BX2)

2. Erect bottom chords (BC1A etc.), install inner splice plates (BS5 etc) and pack plates (BS7) at each panel connection, and bolt together.

3. Erect and bolt into place the bottom chord diagonal bracing members (BB1) and stringers (BSB1) to the full span, thus creating a rigid bottom chord platform.

4. Bolt the end diagonal (BD4) each side into the end bearing assemblies (BBAL, BBAR) and the first internal diagonal (BD1 etc). to the cross-girder end plate. Then bolt the top outer end gusset plate (BG4) to connect the two diagonals.

5. Erect the top chord end beam (BB3) together with the inner gusset plate (BG4) and bolt to the end diagonals (BD4).

6. Assemble the internal diagonals (BD1 etc.) together in pairs, bolted to the top chord gusset plates (BG1, BG2L, BG2R). Ensure that the flange holes for railings are on the inside (roadway side). Lift these A-shaped assemblies into position and bolt them at the bottom to the end plates of the cross-girders (maximum lift 0.8 tonne).

7. Install the bottom chord outer gusset plates (BG3 or BG6), together with flange splice plates (BS4) and pack plates (BS7), and bolt into place to bottom chords and diagonals.

8. Erect top chord members (BC1A etc.) between top chord gusset plates (refer to 6 above). Install chord flange splice plates (BS3 etc.) and pack plates (BS9) and bolt together.

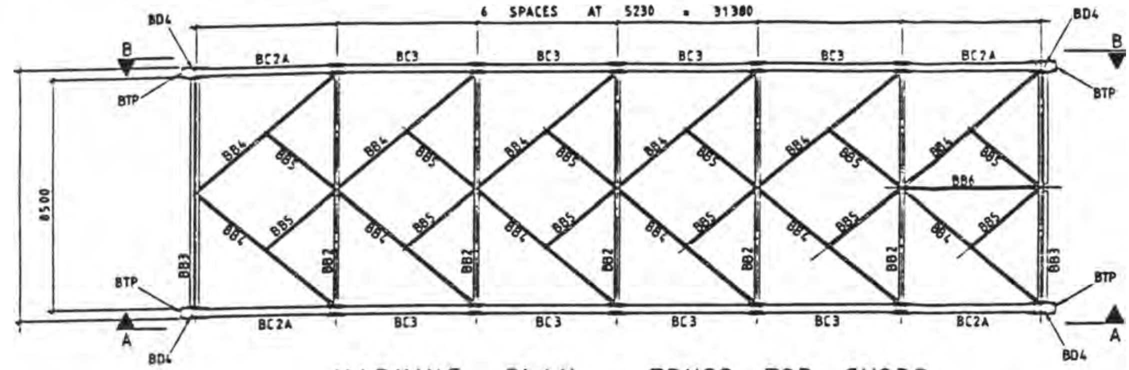
9. Progressively, with 8, erect top chord bracing members (BB2, BB4, BB5, BB6) and bolt into place.

10. Check for camber, line and fit.

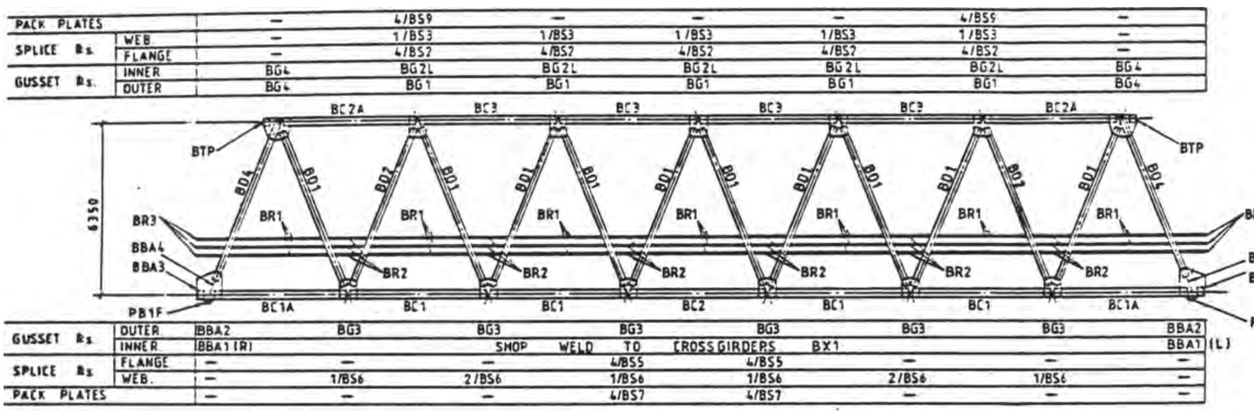
11. Install web splice plates (BS3 top chord) and (BS6 bottom chord) to all chord splices.

12. Insert and tighten remainder of bolts. This can be done progressively as erection proceeds.

Fig. 13.3: Marking Plan of a Standard Truss Span, Supplied by Transfield



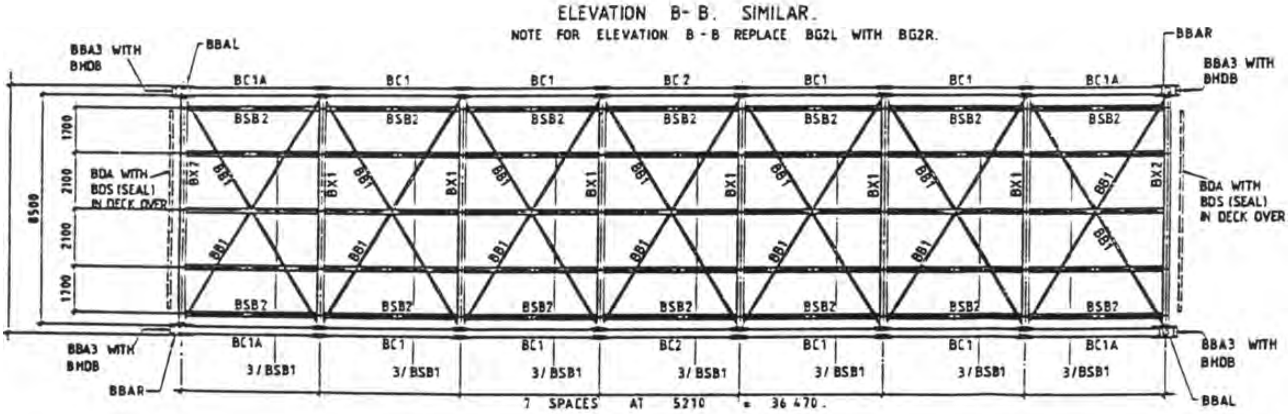
MARKING PLAN - TRUSS TOP CHORD.



ELEVATION A-A.

ELEVATION B-B. SIMILAR.

NOTE FOR ELEVATION B-B REPLACE BG2L WITH BG2R.



MARKING PLAN - TRUSS BOTTOM CHORD.

MARK	Nº OFF	WEIGHT PER ITEM Kg	TOTAL WEIGHT Kg
CHORDS			
BC 1	8	278	2224
BC 1A	4	269	1076
BC 2	2	346	697
BC 2A	4	335	1340
BC 3	8	483	3864
DIAGONALS			
BD1	20	301	6020
BD2	4	375	1500
BD4	4	916	3664
CROSS GIRDERS			
BX1	6	1436	8616
BX2	2	1217	2434

MARK	Nº OFF	WEIGHT PER ITEM Kg	TOTAL WEIGHT Kg
DECK PARTS			
BDA	2	87	174
BDS	1	-	-
EXTRA FOR ABUTMENTS			
BDA	2	87	174
BDS	1	-	-
BOLTS			
B 6	1570		
B 90	1270		
B 120	35		

MARK	Nº OFF	WEIGHT PER ITEM Kg	TOTAL WEIGHT Kg
BRACING			
BB1	14	114	1596
BB2	5	267	1335
BB3	2	511	1022
BB4	12	117	1404
BB5	12	34	408
BB6	1	88	88
STRINGERS			
BSB1	21	230	4830
BSB2	14	463	6482
GUSSET PLATES			
BG1	10	38	380
BG2R	5	43	215
BG2L	5	43	215
BG3	12	41	492
BG4	8	61	488

MARK	Nº OFF	WEIGHT PER ITEM Kg	TOTAL WEIGHT Kg
SPLICE PLATES			
BS 2	40	7	280
BS 3	10	8	80
BS 5	48	7	336
BS 6	16	6	96
BS 7	16	0·5	8
BS 9	16	1	16
ASSEMBLIES			
BBAR	2	370	740
BBAL	2	370	740
BTP	4	19	76
BHDB	4	9	36
HANDRAILS			
BR1	42	34	1428
BR2	36	0·7	25
BR3	12	0·7	7
BR4	84	0·3	25·2

13.3 Piece-by-Piece Cantilever Erection

13.3.1 Erection Link Set

The bridge system includes an erection link set which has been designed to link the cantilever span being erected to a trailing anchor span over either piers or abutments. The set consists of the following components:

- Bottom chord links, (EB6 and EB7) connected into the truss bearing assemblies
- Permanent truss end cross-girder (BX2)
- Top chord links (EB1) bolted into the end of the anchor span
- Permanent truss end diagonal (BD4) and top cross-beam (BB3)
- Link-bracing-top (EB2) and bottom (EB5)
- Bottom chord reinforcing angles (EB3, EB4) fitted to the end bay of the anchor span

The link set members are assembled together with the end of the permanent span being erected in accordance with Fig. 13.4. The high strength bolts provided must be used throughout. These are to be "snug" tightened, i.e. with firm effort with a hand spanner, but not fully tensioned in the link steelwork. When fitting the link members to the top chord connection or to the end bearing assembly of the anchor span, it may be necessary to loosen bolts to permit adjustment. This should be done with great care and under the instruction of the engineer-in-charge.

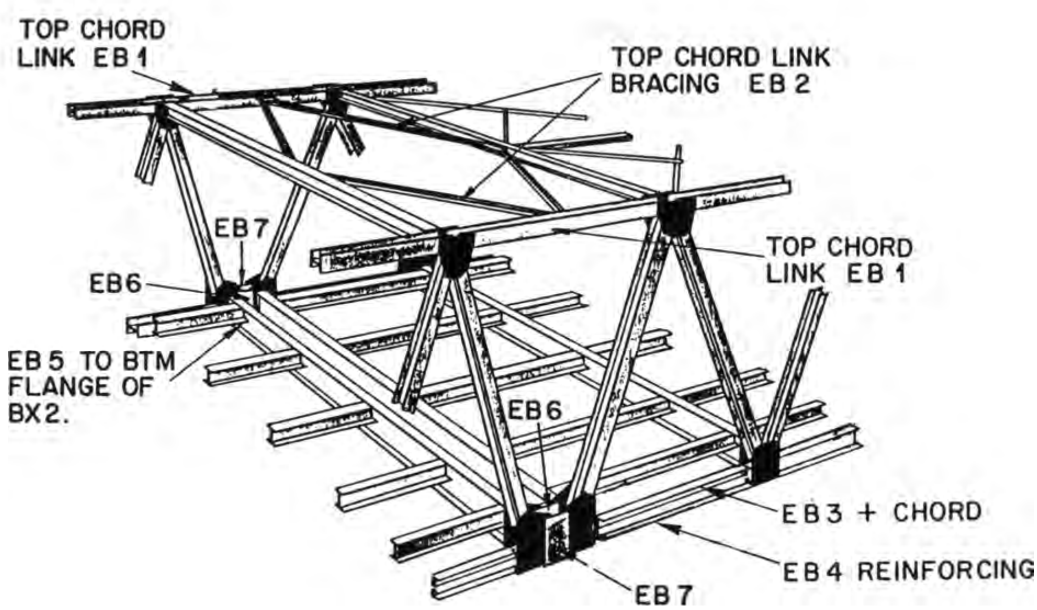


Fig. 13.4

The bottom chords at the ends of both the anchor span and the permanent span adjacent to the link must be reinforced to sustain the forces involved. This temporary reinforcing comprises angle-members EB3 and EB4 fixed to the chords outside the connecting gusset plates. In cases where the anchor span becomes a permanent one, the chord reinforcing shall be removed according to section 13.3.5.

Foto 11: Lower Link Members

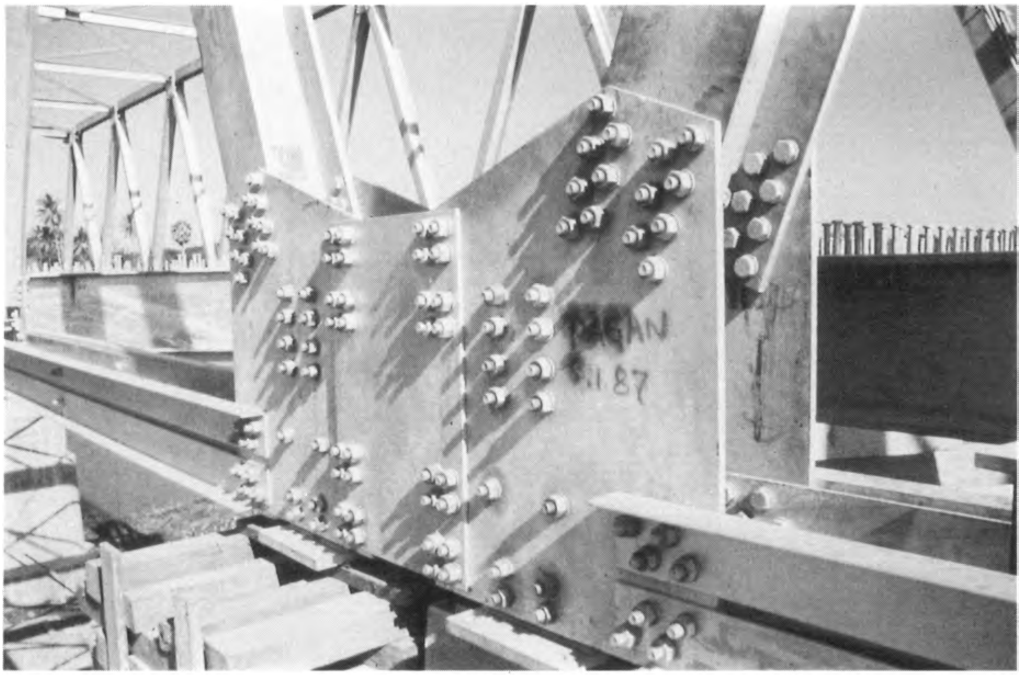
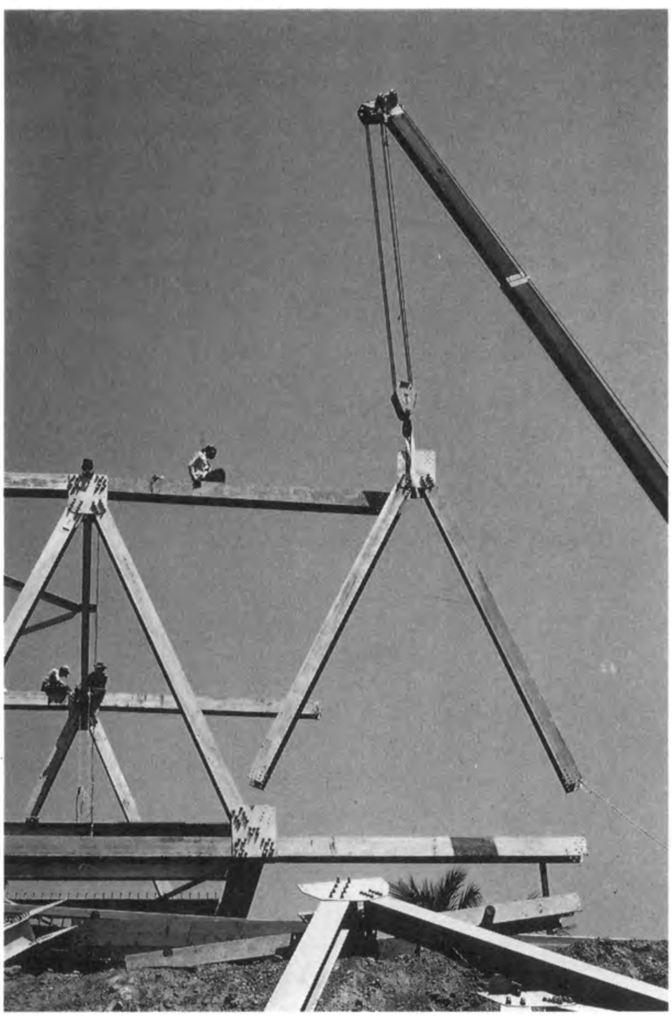


Foto 12: Diagonal Fitting



13.3.2 Piece-by-Piece Assembly

The erection of truss spans by the piece-by-piece (P x P) cantilever method follows a different sequence of assembly to the anchor span. This is set out in an order planned to simplify the process and to avoid the need to remove and replace gusset and splice plates. The key is to leave off the upper flange splice plates BS2 at initial assembly of the individual chord connection. This simplifies the fitting of the next chord member into the joint. Referring to the sequence diagram of Fig.13.5.

1) Erect top chord member (1) by lowering directly between the gussets and lower splice plate of the partially completed joint (J1). Insert packing plates (where designated on the Marking Plan) and the top splice plate, and complete Joint (J1) by fully bolting. Bolts are to be fully tensioned.

2) Bolt the next 2 diagonal members (2) and the 2 top chord gussets (BG1), (BG2) together to form a A assembly. Note that the flange holes for hand-rail fixing are to be on the inside of the span. Haul out and lift vertically and insert the bottom trailing end (of the A) between the bottom chord gussets at joint (J2). Pin with drift and pivot to fit the top gussets over the end of the top chord (1) at (J3). Fit the lower splice and pack plates at (J3) and bolt up part (J3) and (J2), i.e. lower half only.

3) Erect the bottom chord member (3), feeding in between the gussets of the partially completed connection (J2). Fit packing plates and upper splice plates as specified on the Marking Plan and fully bolt. At the leading end of the bottom chord member, fit the outer gusset and lower splice plate and bolt to the chord and diagonal (J4).

4) Erect the top chord cross-beam (4) and bracing (BB2), (BB4) and (BB5) connecting into joint (J 3) and back to the previous cross-beam.

5) Where the gantry is being used to transport the steel, erect the next pair of runway beams after first removing stop brackets (BLS). Refix the brackets.

6) Erect the cross-girder (5). Bolt the cross-girder to the forward diagonal of the assembly (2), fit the lower pack and splice plates and bolt through to bottom chord at (J4). In the case of a bridge with walkways, the brackets (BWC) are to be bolted to the cross-girder end plates before erection of the latter.

7) Erect the two bottom chord braces (BB1) in the preceding 5.21 m bay and fully bolt.

8) Erect the next top chord member (6) as (1) above, and complete the top chord joint (J3). The top chord web splice plates may be fixed one panel behind the general assembly.

9) Erect the next pair of diagonals and repeat the erection cycle.

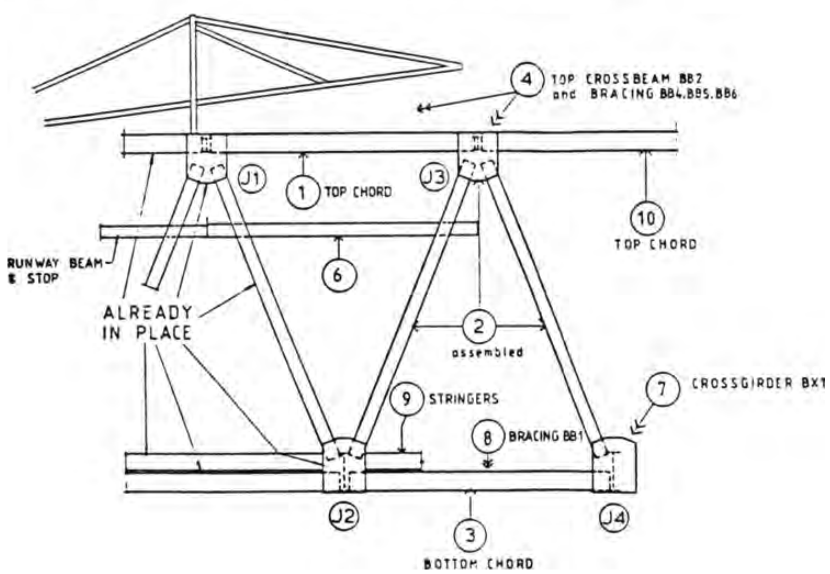


Fig. 13.5

Foto 13: End Bearing Assembly

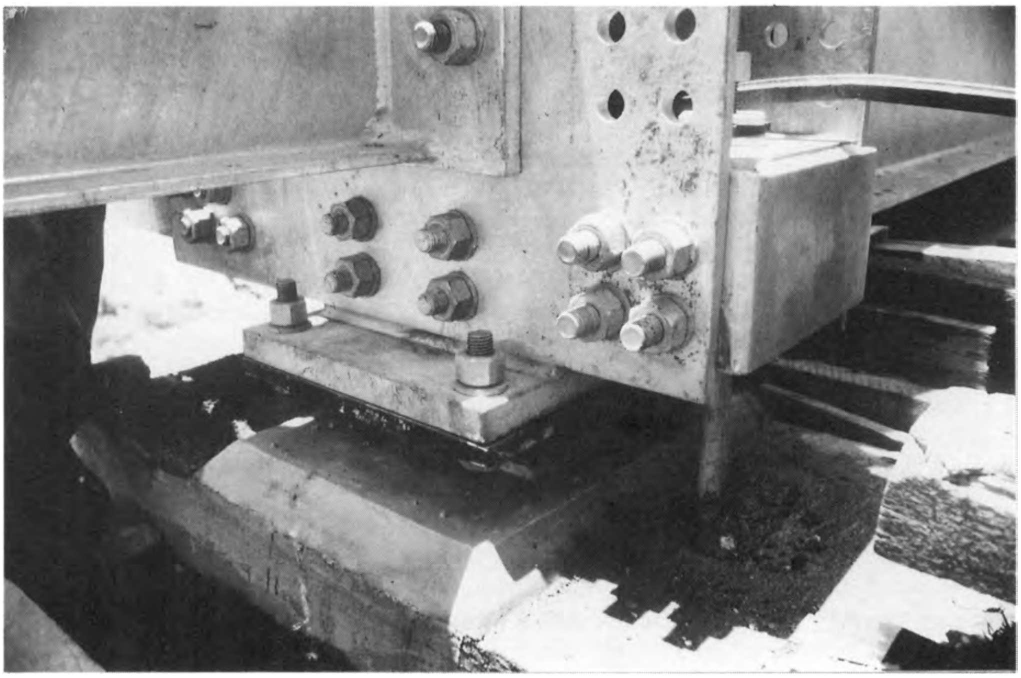
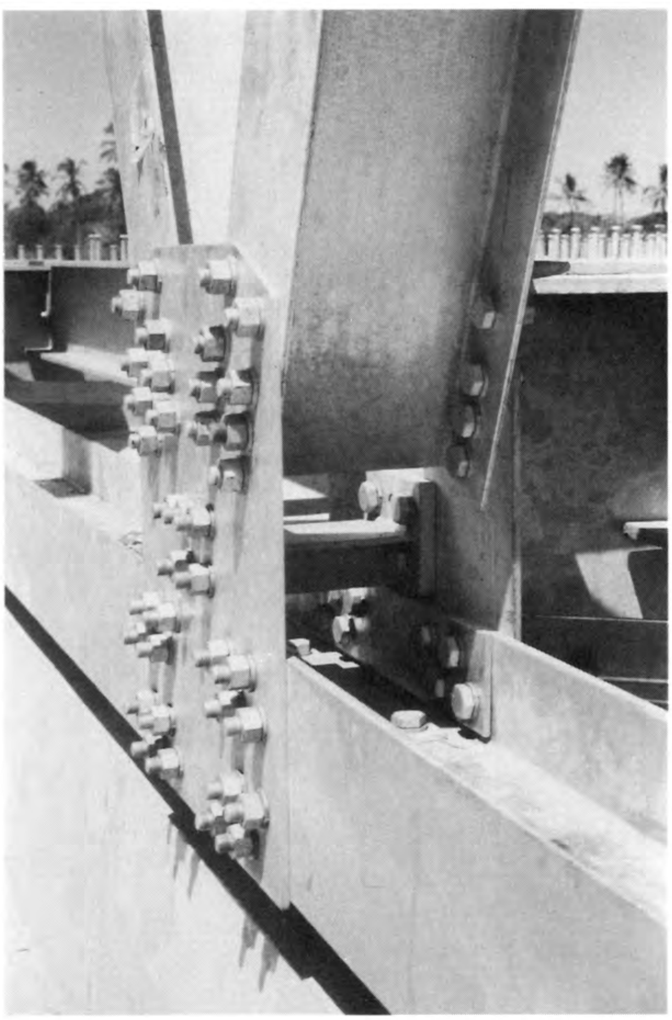


Foto 14: Bottom Chord Detail



The stringer beams are best omitted during cantilever erection and installed only after landing of the span. This procedure can permit erection without the need for kentledge but may only be undertaken under the strict supervision of the engineer-in-charge.

13.3.3 Cantilever Deflection, Camber and Tip Load

During P x P cantilever erection, both the cantilever truss span and the anchor span will deflect combining to create the overall downward deflection at the cantilever tip. The estimated maximum deflection at full cantilever is 190mm (see Fig. 13.6). This is for elastic deflection and for an applied load of 500 kg at the tip.

The steelwork is fabricated such that the required longitudinal camber will be automatically achieved on assembly with all bolts centrally located in the holes provided. No special attention or subsequent adjustments are required during cantilever construction. The cantilevering effect will, if anything, lead to a small increase in the positive camber of the final span. Because of the spacing of bolt holes in the gusset plates, it is important that as soon as all members are connected into any one joint, the bolts in that joint should be fully tensioned. The holes in components are precision drilled and the specially shaped drifts provided with the tools should be utilised to align members and gusset and splice plates at panel points and splices.

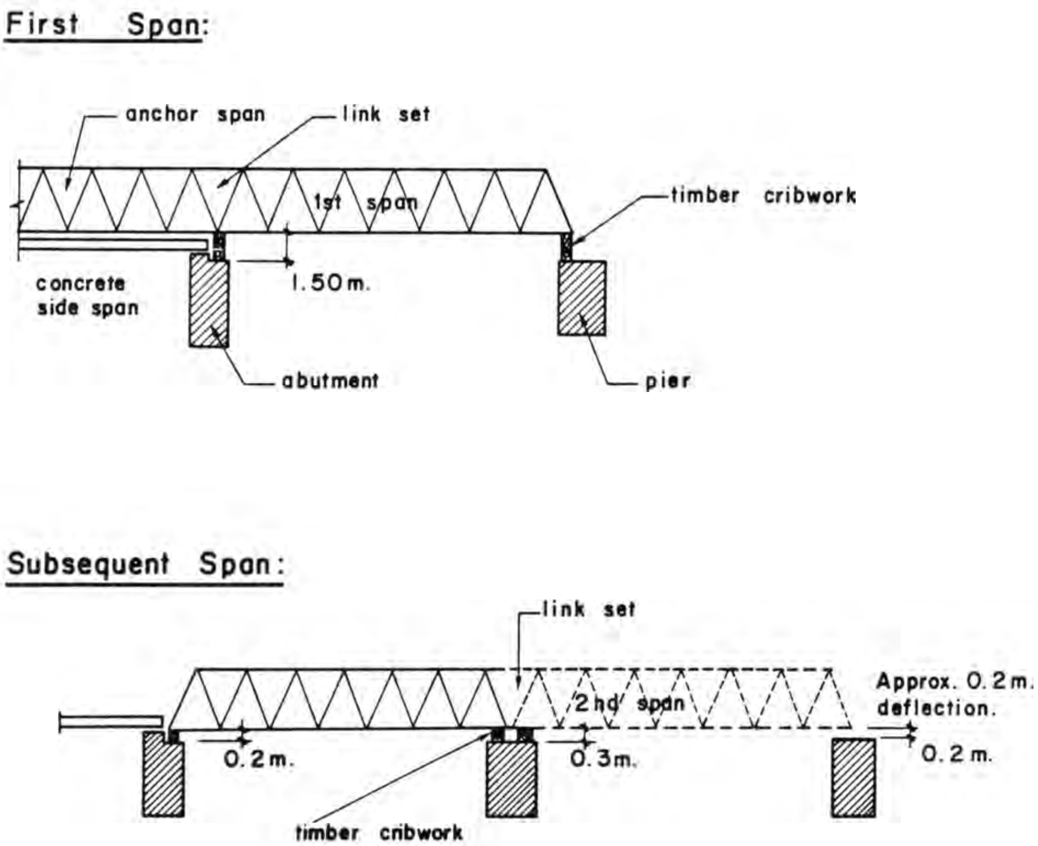
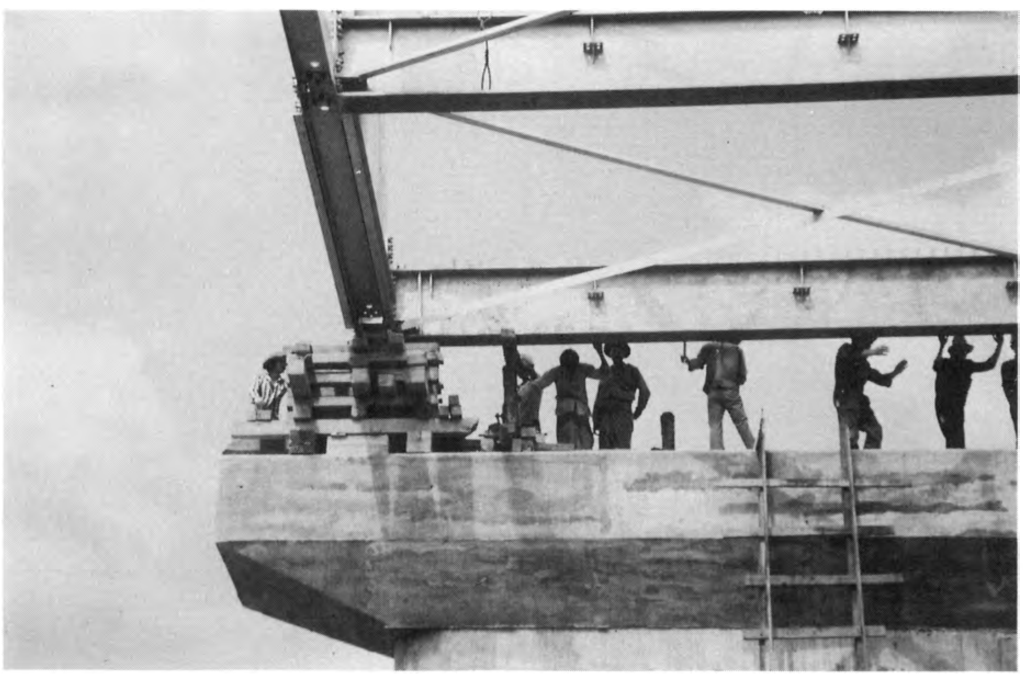


Fig. 13.6

Foto 15: Landing Span

Foto 16: Cribwork Support



It is necessary to erect the steel starting from levels which makes allowance for the deflection. The temporary levels of both the leading and the trailing edge of the anchor span need to be adjusted to ensure that proper cantilever landing will be achieved, such that there is enough space to put the jacks below the truss for lifting the cantilever. To save cribwork, the completed span serving as anchor should normally be lifted approximately 100 mm before installing the link set, thus compensating the cantilever deflection.

As the span being constructed approaches full cantilever any additional loads to the span other than the components being installed should be minimised. Refer to indications by the manufacturer for the limitations on the load at the tip of the cantilever.

13.3.4 Multi-Span Bridges

In a multi-span case, it is suggested that Span 2 is used as an anchor whilst erecting Span 1. Once Span 1 is in position, securely supported on timber cribwork directly above its correct location, the link shall be dismantled and transferred to the front of Span 1 over the pier and the anchor span (Span 2) shall be dismantled. Span 2 can then be re-erected in its correct position, again using the link and the piece-by-piece cantilever method.

In the case where the 1st permanent span becomes the anchor span for the 2nd span and the 2nd for the 3rd, etc. the procedures described above apply, with the following variations:

i) The bottom chord reinforcing (EB3, EB4) to the anchor span will be installed during erection of the anchor span (The bolts are not tensioned).

ii) The temporary cribwork support under the leading end bearing plates of the anchor span will remain in place. Support must also be provided under the trailing end bearing plates of the span being erected to prevent excessive sag.

iii) The link steelwork (EB7) and (EB1) should be fitted to the anchor span at the time of assembly of that span.

13.3.5 Temporary Support and Removal of Anchor Span and Link Members

When the leading end of the cantilevering span is landed at the far abutment or pier, it should be a clearance above the concrete notwithstanding the deflection of the span (see 13.3.5). If this is not the case, the trailing end of the anchor span must be jacked down with the span anchored against longitudinal movement. It should immediately be supported to the underside of the truss bearing plate at each side of the leading edge by timber cribwork based upon the abutment or pier. Before this, the area of landing must be cleared to permit the cribwork to be bedded on mortar applied over a hessian or polyethylene sheet (to allow subsequent removal).

With the span temporarily supported at the 4 bearing plates, the anchor span should be separately supported and the link members unbolted. This should take place immediately on completion of the erection and before starting with the next span.

Where the anchor span is temporary, it should be removed in the following sequence:

1) Remove kentledge - if utilised.

2) Support the anchor span each side under the first panel point back from the forward end, lift using the timber blocking and wedges.

3) Unbolt the top and bottom chord connecting link members. It may be necessary to jack the trailing end of the anchor span to free the bolts.

4) Check the supports under each bottom chord panel point or cross-girder are wedged tightly into position.

5) Unbolt and remove all outer gusset plates.

6) Unbolt and remove the top chord bracing and top chords, then diagonals, bottom chords, etc.

7) Clean and repair, if necessary, all parts and return to store.

Where the anchor span is formed by the preceding permanent span of the bridge, then it will be necessary to remove any kentledge, the chord reinforcing and the link member only. The anchor span must be supported on accurately placed and substantial timber cribwork under each end cross-girder and then to jack the trailing end up or down to free the bolts and permit unbolting of the link set.

The removal of the chord reinforcing angle members (EB3) and (EB4) is a simple part of the normal course of disassembly when the anchor span is temporary. However, when the anchor span is preceding a permanent span of a multi-span bridge, the chord reinforcing must be removed in the following sequence to prevent collapse of the span:

1) There are four reinforcing angles fitted to each chord. Remove one at a time leaving the others fully bolted.

2) Remove nuts and washers from the bolts securing the first reinforcing angle, remove it leaving the bolts in place. Replace the nuts and washers through the chords and gussets and fully tension.

3) Repeat (2) for the 2nd, 3rd and 4th reinforcing angles in turn.

This same careful procedure must be followed for the removal of the chord reinforcing from the trailing end of the span being erected.

13.4 Completion of Spans

13.4.1 Lowering of Span

After a single span bridge is landed the link members and anchor span is dismantled. For a multi-span bridge, two spans are "working" together, the span being erected and its preceeding, anchor span. Only at three or more span bridges earlier spans will be lowered individually onto their permanent bearings as soon as the link members are removed. In these cases, the lowering of the span(s) will involve a height of about 1.4 m, and this will require a staged lowering procedure.

The designed length of each span is 36.50 m. However, due to the inbuilt camber, and prior to the installation and hence the dead load of the deck slabs, the length of the landed steelwork is actually 36.47 m. The link set will forward the difference of 30 mm to the adjacent span, resulting in the addition of this difference in length. The front end of a multi-span bridge would become increasingly in error and fall short of the designed bearing. Accordingly, the spans must be moved longitudinally after removing the link sets.

This longitudinal and lateral adjustment may be achieved by jacking between the plinths and the steelwork. Jacking points selected must correspond with the chord/cross-girder connection and the forces required should be minimised by adjusting any packing, by applying grease to steel/steel surfaces and using steel rollers.

Lowering procedure, jacking positions and loads are as illustrated in Fig. 13.7 below.

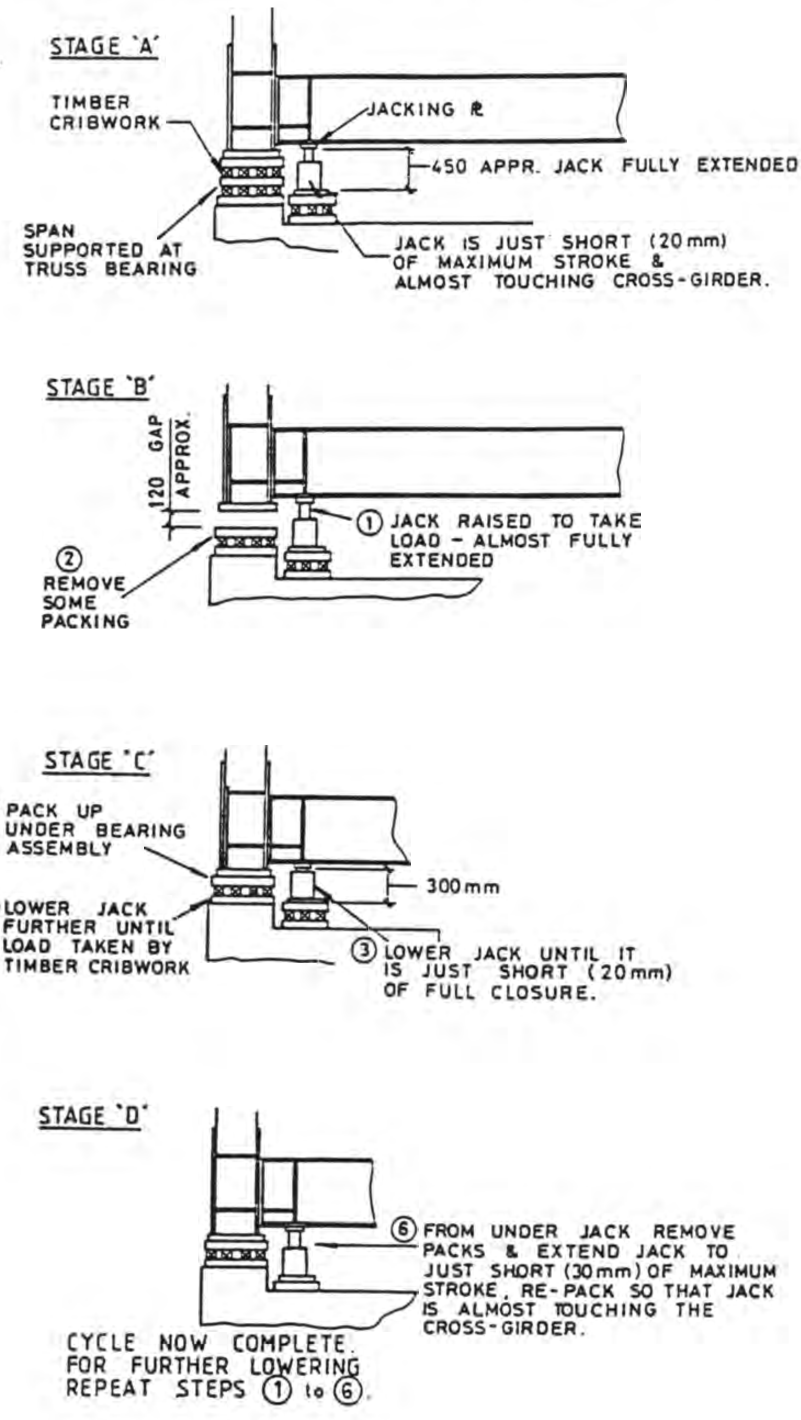


Fig. 13.7

The dead load of steelwork per bearing (4 No.) is 13.6 tonnes or 16.8 tonnes with walkway. For maximum extension of jacks see their specifications. Ensure that the jacks are properly supported by hardwood or steel and, to prevent damage of the steel structure, put a steel or hardwood plate between the jack and cross-girder. For timber cribwork under jacks, the same specifications as for bearings shall be applied. Check that the upper supports of the jack and the bridge are able to be moved independently in the "step" process of lowering. Both jacks have to work in unison.

Each end of the span should be lowered in turn so that the difference in level between the two ends does not exceed 600 mm at any stage. When the clearance has been reduced to approximately 400 mm under all 4 bearing plates, the permanent bearings are inserted and, after their accurate levelling, the span is lowered onto these.

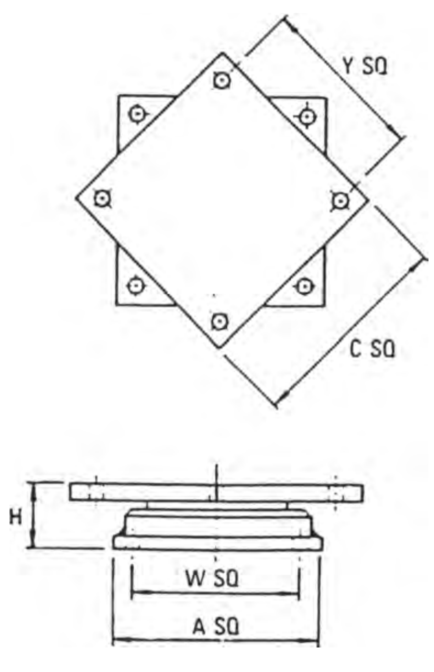
13.4.2 Bearings

The bearings are "pot" type bearings. They are detailed on the manufacturer's drawings and are supplied, for shipping, with temporary fixing and protection.

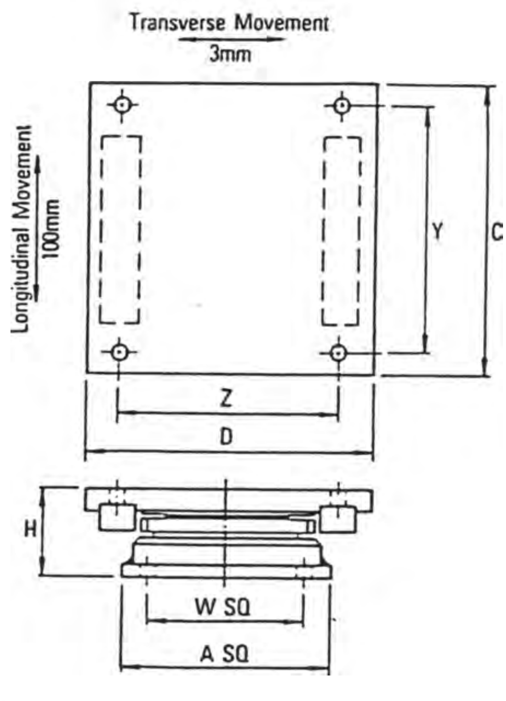
Each span has two types of bearings, fixed and sliding guided (or expansion). The bearings are to be installed on the piers and abutments to the set-out and levels within the following tolerances :

Span length (36.50 m)	+/- 5 mm
Cross spacing	+/- 3 mm
Level (any bearing and 2 bearings one end)	+/- 3 mm
Slope	1.0 %

Table 13.1



Fixed Bearing



Sliding Bearing

Fig. 13.8

Bearing Type	Vertical Load (kN)		Horizontal Load (kN)	Installation Dimensions							Max. Bolt Size
	Normal DL + LL	Max. DL		A	C	D	H	W	Y	Z	
PG 100	1000	667	180	295	397	369	103	235	337	293	20
PG 150	1500	1000	220	352	437	413	107	282	377	337	20
PF 100	1000		180	295	302		74	236	242		20
PF 150	1500		220	352	352		78	282	292		20

Table 13.2

The following procedure indicates the best means of installation of the bearings within the small tolerances given:

1) Lower the span to approximately 300-400 mm as described above.

2) Hard steel industrial rollers are fitted under the bearing plates at the bearing plinths and supported on steel plates. The truss can then be lowered onto the rollers and longitudinal adjustment made by jacking. The jacking points must correspond with the chord/cross-girder connection. During adjustment, the span should be anchored against excessive movement.

3) Once the span is located within the tolerances given, it may be lifted to remove the rollers. The bearings assembled with straps can then be bolted to the truss bearing plate and the span lowered to the level indicated in the drawings.

4) The above procedure must first be carried out at the fixed bearings and only then at the sliding bearings. Note that positioning of the base plate of the sliding bearing will result in a longitudinal offset of approximately 20 to 30 mm between upper and lower section if lowering before placing deck slabs and deck casting.

5) Remove timber cribwork only after the installation of all 4 bearings of one span.

13.4.3 Holding Down Bolts

Holding down bolts are supplied with the bearings and are to be set into the concrete of the substructure, in the locations and with the projections detailed in Fig 13.9.

Holes should be drilled into the concrete in the appropriate locations to permit the truss span bolts BHDB to be inserted to the correct depth after the span has been landed and the permanent bearing set in place. These holes must be thoroughly cleaned by air blast and/or water jet aided by scraping prior to setting in the bolts and the projection of bolts is to be carefully checked before grouting. The anchor bolts must fit through the end stop BBA3. Large clearance holes are provided in BBA3 and the anchor bolts are provided with a plate washer to fit over the hole, together with a rubber washer, a second plate washer and the nut. A special tube spanner should be used for tightening these bolts.

The bolt may be grouted only after having been set in the hole and blocked firmly into position and levelled. The grout would best be a proprietory brand "liquid" grout approved by the engineer, but alternatively a 1 : 3 cement - sand mix is acceptable. The socket hole is filled with grout up to 30 mm beyond the base plate. Once the grout has partially set, the space under the base plate should be packed with 1 : 3 cement - sand mortar extending 30 mm beyond the bearings and neatly finished to shed water.

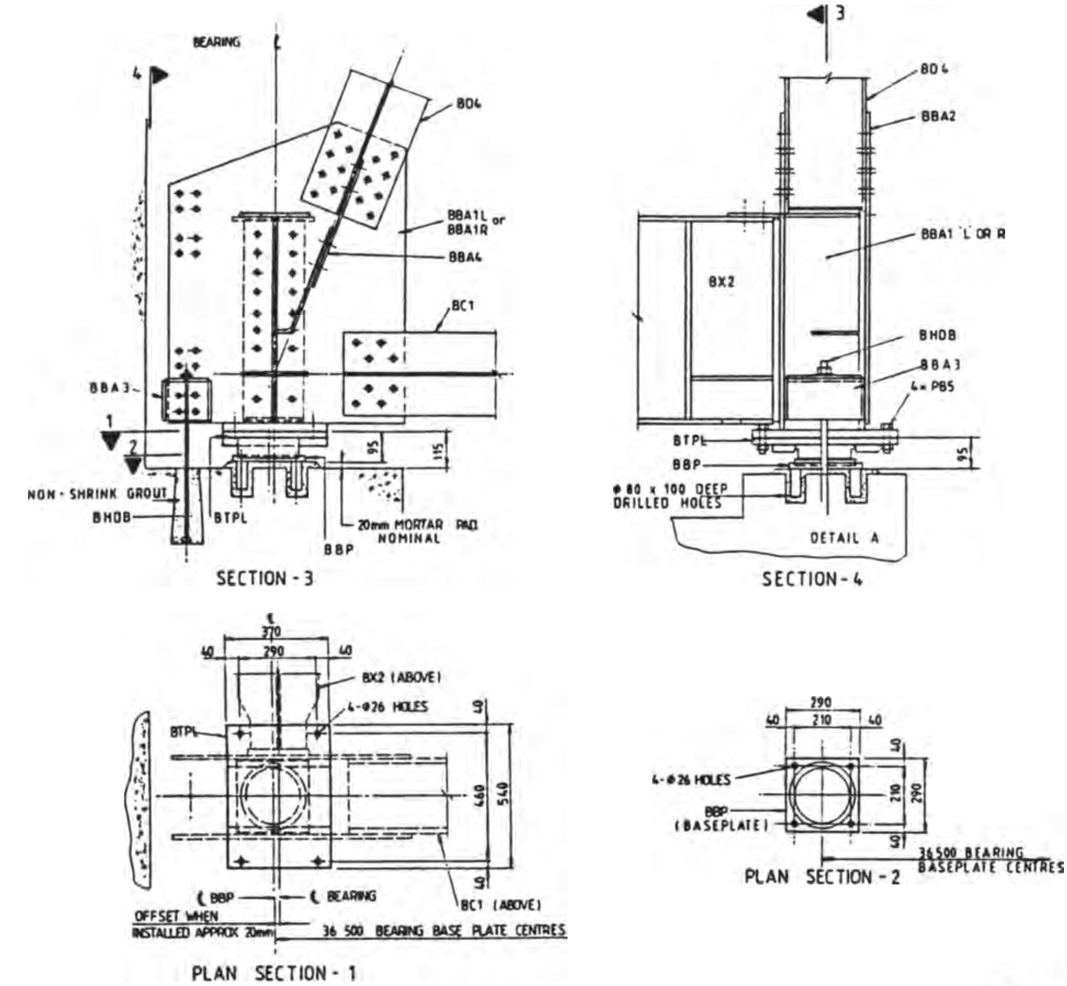


Fig. 13.9

13.4.4 Spacers in End Assemblies

The end gussets BBA2 and BG4, to the bottom and top chords of trusses respectively , have been extended to accommodate the link steel connections for cantilever erection. Following completion of the erection and removal of link steel, spacers BBA3 (bottom) and BTP (top) are to be fitted between the gussets and bolted thereto. Eight M24 bolts are required for each, and these are to be fully tensioned.

13.4.5 Railings

Railings comprise standard 76 mm OD galvanised steel pipe, and are supplied in 5.22 m nominal lengths threaded at both ends. These lengths are to be coupled on site to form a continuous railing. Standard pipe couplers will not clash with the connecting bolts if arranged as detailed in the drawing.

The ends of each side of the two rails are to be trimmed after erection with an even extension beyond the end panel point. This overhang will be arranged on site to suit abutment detailing. The cut ends should be cleaned and painted and the force fit pipe cap supplied with the railings, driven into position.

Three forms of railing connection to the steelwork are involved:

i) To the roadway side of the diagonals to all spans, with U-bolts through the diagonal flanges. Note that 4 holes have been provided and only 2 are to be used, with the U-bolt vertical in all cases.

ii) Using U-bolts and special clamping plates to the footway side of the diagonals to bridges with footways, e.g. Kyoung-gone.

iii) To the posts on the outer side of the footway, with U-bolts as for (i). The U-bolts (BR4, BR20) are provided with spring washers and are to be tightened using hand spanners. The thread is to be punched in 2 locations against the nut after final alignment and tightening to prevent removal.

13.4.6 Walkway Steelwork

Walkways comprise precast concrete deck slabs bolted to steel stringers which are supported on cantilever beams. These beams are bolted to the truss chord outer gusset plates. Walkway steel, if it is to be fitted, should be erected after the spans have been lowered onto their permanent bearings.

Where a walkway is required, a connecting bracket BWC is fixed to join the inner and outer gusset plates of the bottom chords. The brackets BWC are fully bolted to the end plates of the cross-girders at the initial stage of assembly. The outer gussets BG6 are installed and fully bolted to the chords and diagonals as part of the span assembly but not to the brackets.

When the span has been completed, the walkway beams BW1, BW2 are placed and held, working from the inside of the span, and bolted through the gussets BBA2 and BG6 and brackets BWC with B120 bolts, nuts on the walkway side. Space is restricted at this connection and tensioning is carried out with an open ended spanner with pipe extension. With the cantilever beams in place, the remainder of the walkway steelwork may be erected and bolted into place

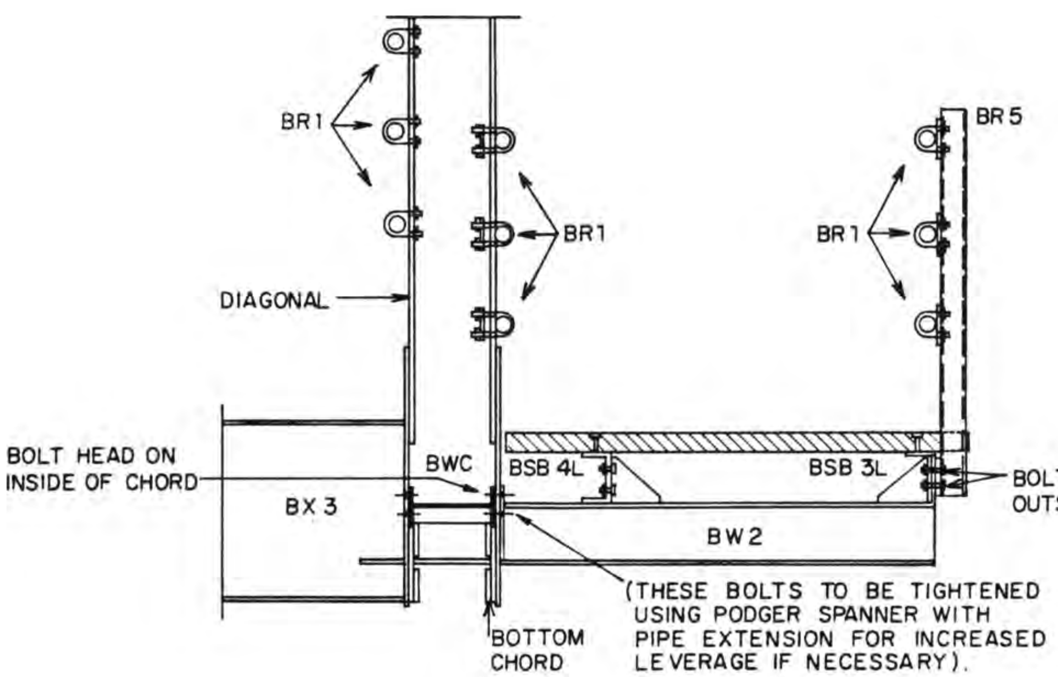


Fig. 13.10

Foto 17: Walkway Steel

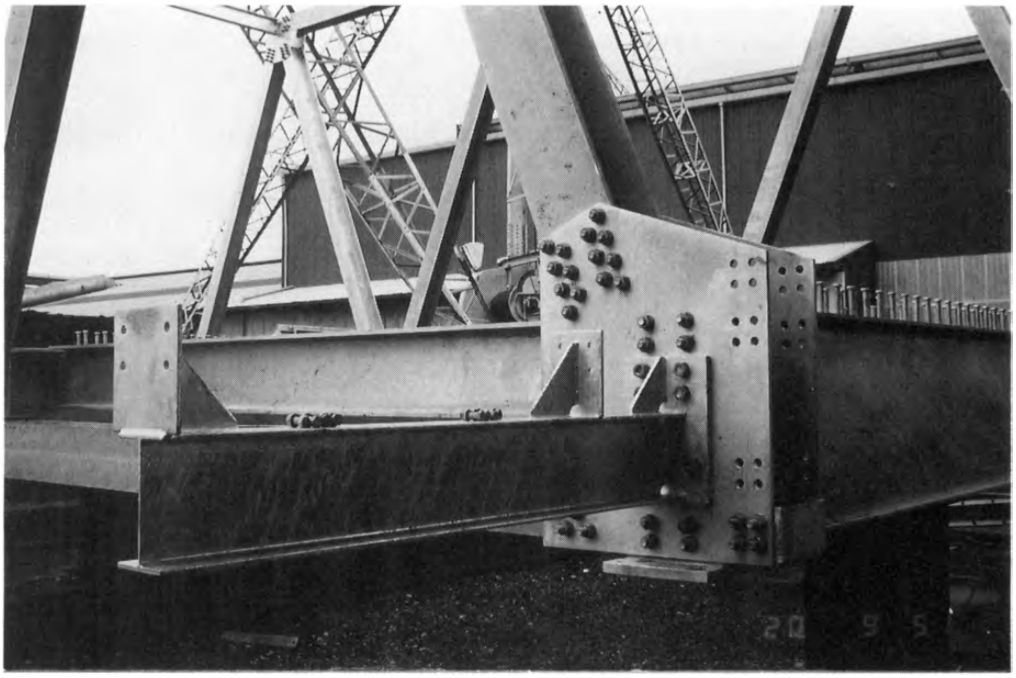
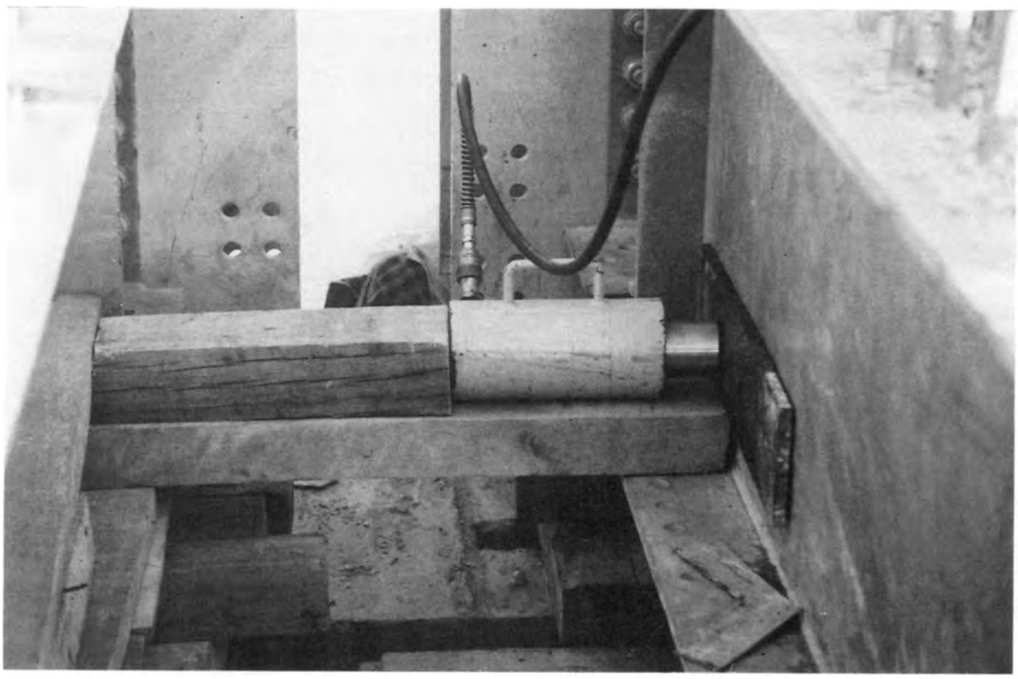


Foto 18: Jacking into Position



13.5 Care of Steelwork

Site management and those involved in transporting steelwork should seek to ensure that steel members are handled, lifted and stored so as to avoid damage, overstressing or damage to surface treatment.

The galvanised surface treatment is of high quality and in normal circumstances repairs should not be necessary. Fine scratches need not be treated. Larger areas, associated with rectification of steel for instance, are to be cleaned back to bright metal by power grinding and treated with an approved inorganic zinc priming paint as described in section 16, Maintenance.

Most members may be lifted by a single central sling. However, the cross-girders should be lifted using 2 slings for proper control.

Truss spans utilised as anchor spans and erection steelwork should be handled and protected as described for the permanent steelwork. Shear studs bent in transit may be straightened to clear precast slabs but otherwise should be left bent.

Members should be stacked on site on timber packing, level and clear above ground. Bolts and bearings and similar parts must also be stored under cover to avoid contamination. The typical stacking arrangement is as illustrated over. H-shaped members may be stacked with webs either vertically or horizontally for transport but not in store.

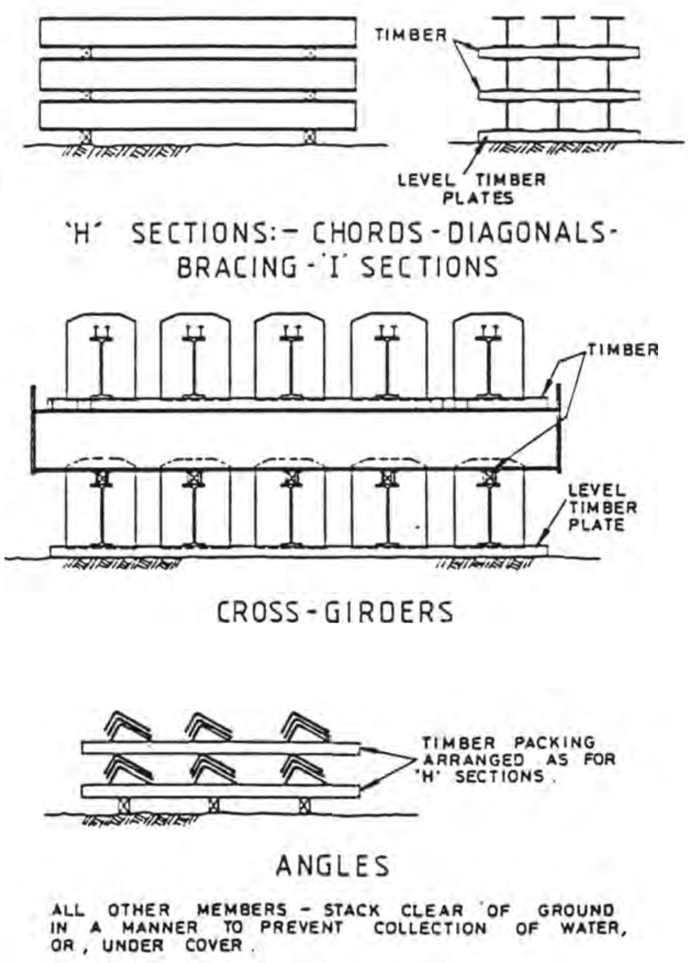


Fig. 13.11

13.6 Bolts

13.6.1 Type and Supply

Structural field connections use 24 mm diameter structural high strength bolts conforming to Australian Standard AS1252. Any replacement bolts including nuts and washers must be of equivalent standard (Note: Proof load not less that 212 kN).

The bolts are supplied, complete with nut and high strength flat washer, in bags, packed in steel drums. The contents of the drums should be marked on the outside. The number of bolts to be supplied will be equal to the number required for the span plus 5% excess (of each length) to allow for reasonable loss or damage during assembly. Additional bolts have been supplied for erection of anchor spans. These high strength bolts are to be re-used and should be carefully removed during dismantling, oiled and repacked in the containers provided. Any bolts exhibiting visible damage must be discarded unless otherwise directed.

13.6.2 Protection

The bolts, nuts and washers will be supplied galvanised. They should be stored on the site in their original bags under cover in a dry, airy and secure place. They are supplied with a coating of light oil on the threaded portion. They should be inspected prior to use and any dirt, contamination or heavy oxidation on the thread cleaned by hand using a brush. To permit proper tightening where the lubricant has been removed, the threads should be brushed with lubricant prior to assembly.

Nuts and bolts with insufficient lubrication cause higher friction forces than correctly lubricated bolts. Applying the specified torque will result in lower prestressing forces (and in consequence lower safety factors) for badly lubricated assemblies. Inspect all components before using. Return parts to store which are discoloured, damaged or badly galvanised, or which are marked incorrectly.

13.6.3 Bolt Assembly

All bolts to chord and diagonal splices are to be inserted from the inside of the chord and diagonal section. They are always to be assembled with one hardened washer under the nut. The nut and not the bolt shall be turned for tightening. In some locations one washer is also required under the head of the bolt. These locations are indicated in the drawings.

The holes to all steelwork should be properly aligned before inserting the bolt. This should be achieved using the drifts supplied, with light hammering if necessary, but such that the edges of the holes are not damaged or burred. If greater force is found to be necessary, all bolts should be loosened or removed and the intersecting members checked for alignment and location. When inserting the bolts any force or hammering that would damage the thread must not be used . Before snug tightening check that the bolt is square to the steel surface.

13.6.4 Bolt Lengths

Three lengths of 24 mm high strength bolts for structural connections are required for the permanent spans and these are designated on the drawings and in the bolt lists by the prefix "B" followed by the length in millimetres (measured exclusive of the head):

B 65	(65 mm)	for bracing member connections and connections of thinnest plates,
B 90	(90 mm)	for general use in truss spans,
B 120	(120 mm)	for top connections to end diagonals and for splices to chord members in truss spans with walkways.

Bolt Type	Bolt Diameter mm	Shank Length mm	Unthreaded Shank mm	Thread Run Out mm	Location
C 60	16	60	17 +/-1.5	5	Walkway stringers and handrail post supports
C 110	16	110	70 +/-2.0	5	PC walkway slabs
B 65	24	65	17 +/-1.5	5	Top and bottom chord bracing, chord webs, diagonals, stringers
B 90	24	90	23.5 +/- 1.5	5	Main connections of chords, end diagonals cross girders
B 120	24	120	56 +/- 3.0	7	End diagonals, walkway beams

Table 13.3

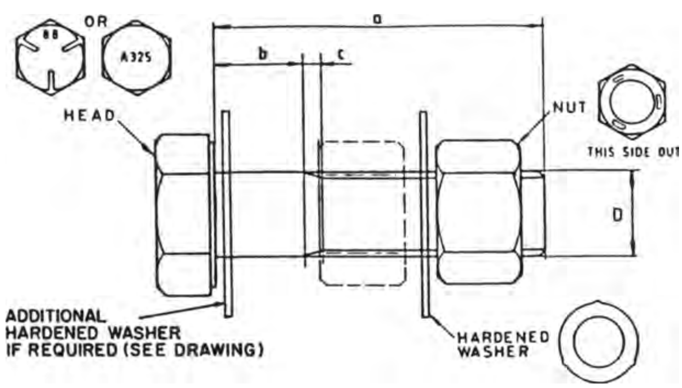


Fig. 13.12

Bolts of 16 mm diameter are supplied for lesser connections associated with the walkways. These are designated on the drawings and by the prefix "C" followed by the length in millimetres:

C60	(60 mm)	for walkway steel connections
C110	(110 mm)	for walkway precast concrete panel connections (not tensioned)
C120	(120 mm)	countersunk head bolt for connection of footway plates (not tensioned)

13.6.5 Tightening of Bolts

The bolts are to be initially snug tightened using a ring or open ended spanner of the correct size. No final tightening shall be undertaken until the whole of the connection has been approved with respect to camber, fit and member arrangements, and all bolts have been snug tightened. Finally, the bolts are to be tensioned using the torque wrenches provided. In any case tightening is to be carried out by turning the nut and not the head. Spanners of correct size must always be used. Tension should be applied evenly and completely to each bolt in turn. Tightening shall commence at the centre of any group of bolts and continue to the outside.

Based on tests on bolts and nuts, carried out by the Swiss Federal Institute of Technology, the following method of tightening should be used:

1st step: Tighten the bolt with a moment of M = 300 Nm, using the torque wrenches provided.

2nd step: Put the torque multiplier and the tightening angle indicator (set to 0 degrees) between torque wrench and socket and turn the nut an additional angle of 60 degrees.

All bolt locations should be accessible with the torque wrench supplied. However, bracing members may be removed temporarily to gain access to bolts, in order to permit proper tightening of main members.

13.7 Equipment for Piece by Piece Cantilever Erection

13.7.1 Erection Gear Equipment

The following erection equipment is available with the main steelwork for in-place cantilever erection:

- Anchor truss span, comprising a standard span
- Erection link set
- Gantry with hoist and runway beams
- Hydraulic jacks, 50 tonne capacity (min. 2 pcs.)
- Industrial hard steel rollers (Serva Technics)
- Tyrfor 2.5 tonne pulling machine with 15 m rope and hook (2 pcs.)
- Selson torque multipliers and accessories R36 and M24 (manually operated with incorporated planetary drive gearing 8 : 1)
- Small ratchet wrenches and accessories M20 and M16
- Tube, open and combination spanners, drifts, hammers, steel measuring tape etc.

In addition to the above equipment, it will be necessary to have available the following items :

- Timber cribwork as temporary supports
- Mobile crane to handle and lift steel components from store, during part assembly and up to the bridge structure.
- Jacks, jacking plates and timber packing for use in the span lowering operation.
- Suitable boats or pontoons to carry out the mobile crane and also the partially assembled units
- Unless the engineer-in-charge is able to confirm that it may be dispensed with, kentledge at the trailing end of the anchor span and a platform to support it above the shear studs of stringers and cross-girders .

13.7.2 Timber Cribwork

Timber cribwork is proposed for both temporary anchor span support and for the landing and subsequent movement and jacking down of permanent spans during cantilever erection. In this last case, the cribwork will be built on the abutment or pier head.

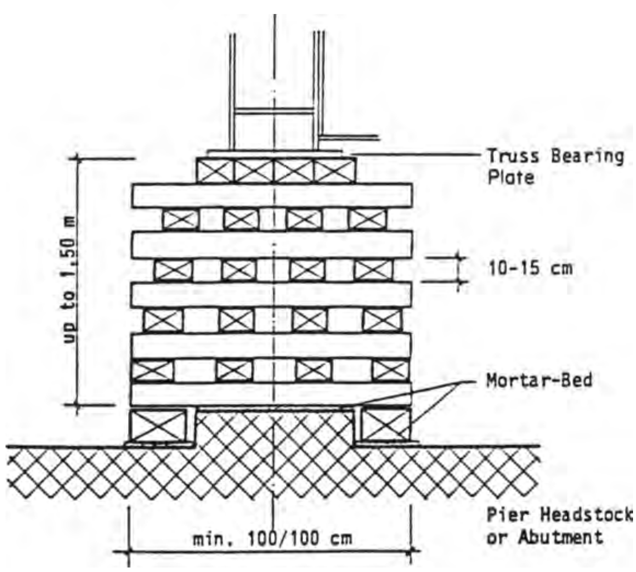


Fig. 13.13

Such cribwork will be subjected to high vertical loads of not less than 10 tonnes, and possible horizontal forces arising during assembly from wind loading on the bridge structure. It should incorporate wedges to permit adjustment of height to suit the camber of the bridge within a tolerance of +/- 5 mm and to move thereby free members slightly during dismantling . Timber must be hardwood; rotten, broken, or uneven timber cannot be used. All timber must be placed with its larger dimension horizontal to ensure stability.

The bottom elements must be levelled and bedded on 1: 3 cement-sand mortar. Each base size must not be less than 1.0 m x 1.0 m. Alternate layers should be laid across the previous layer and be trimmed carefully to fit so that each layer is firm, level and square. The truss bearing plate should bear on timber over its full area.

13.7.3 Erection Gantry

A 2 tonne capacity, manually operated travelling gantry is provided, together with runway beams which are bolted to each side of the truss diagonals. This gantry is designed to transport components along the span being erected.

The gantry comprises the following items:

- a single top mounted beam
- end carriages, top running and operated either by hand-chain through a geared drive or pulled along by hand
- a 2 tonne capacity hand-chain operated, geared travelling trolley (crab)
- a 2 tonne capacity chain block hoist
- runway beams of RHS section, which incorporate the crane wheel track and fixing brackets and stops (BRB1, BRB2, BRB3R and BRB3L). The beams are to be fixed to the inside flange of the truss diagonal.

For gantry assembly, the beam and two end carriages are fitted into the top mounted gantry crane as indicated in the drawings. This has to be done before lifting it onto the runway rails and the following procedure is recommended:

1) Support the end carriages on ground level at the correct spacing; place the beam over and then bolt it to the top of the end carriage with 20 mm bolts. Bolts should be snug tightened.

2) Fit the drive shaft to the outriggers (with the chain fitted), adjust gears to mesh on the carriage drives and bolt the covers to the blocks on the end carriages and outriggers, using the studs provided.

3) Unbolt the 2 keep bolts from the travelling trolley, fit the trolley to the beam, lower flange and rebolt.

4) Lift the assembled gantry unit, place it on each side of the rails, using the chain hoist provided with the gantry kit. The weight of the assembled gantry unit is about 700 kg. The tolerance on fit in overall width is only 3 mm, so it is important to check and adjust the cross dimension of the runway beams before attempting to install the crane. If necessary, packing 2mm steel plate between the flange and the beam fixing plates is possible before tightening the Lindaptors.

5) Hook the chain block hoist onto the travelling trolley.

6) Check everything for free operation.

Note: The first gantry beams BRB1 should be fixed with great care according to the given specifications. Differences in height will result in such difficulties in adjusting the following beams to the extent that the first beams will have to be repositioned.

The beams are to be installed such that the top of the rail is nominally 1465 mm below the centre of the top chord, but this dimension may be varied by +/- 20 mm to achieve a smooth even profile without discontinuities in elevation which would preclude the operation of the crane. The beam will follow the cambered profile of the truss spans.

The beam fixing brackets are bolted to the inner flanges of the diagonals via "Lindaptor" clamping bolts. There are two lengths of Lindaptors :

- M20/90 plus "Long tail": for internal diagonals of trusses
- M20/220 plus "Short tail": for end diagonals of trusses

The Lindapters are to be set square to the edge of the diagonal flange and tightened to snug tight. Exact positioning of the first beam is required as later adjustment is difficult and time consuming.

The ends of two adjacent runway beams are to be connected with the cover plates provided, bolted through the webs. The gap at these joints should not exceed 10 mm.

Four longitudinal stop brackets are required. One pair is permanently fixed to the trailing end of the runway throughout the operation. The second pair is to be fixed to the leading end of the two runway beams as each new section of beam is fixed, and before operating the gantry. The two leading end brackets must be removed just before installing the next pair of runway beams.

For a bridge of multiple spans, the runway will be continuous through all permanent spans for the steel erection phase and the placing of precast elements. Therefore the link beam BRB2 is designed as a variable length beam. The runway beams may be fixed, initially, continuously from the temporary anchor span to the cantilevered span under construction.

It is also possible, as a help to the mobile crane, that with runway beams erected progressively with the span, that steel components may be transported to the end of each cantilevering span in turn ready for their lifting with the crane.

The normal mode of "free" longitudinal travel requires that the gears on the drive shaft are to be slid along the shaft to a position over the wheels, clear of the hand drive gear, after first loosening the worm screws. Then, normal travel will be "free", however, proper care and control must be exercised to prevent the load "running away". Forward and back haul ropes should be connected to each side of the end carriage, be manned during travel and be fixed when the gantry is at rest. If any problem is encountered, the longitudinal chain drive must be reconnected to control the travel. The stop brackets, BLS, must be fitted at all times during operation of the crane.

Lateral movement across the span will normally be by hand-chain operating through the gears in the trolley.

Hoisting will be by the chain block. The limit of 2 tonnes must be strictly observed.

For general use, all components of the gantry have "sealed for life" bearings which require no maintenance. The gantry, including chain block, should be protected from damage, corrosion or contamination and should be covered when not in use. The chain drives are to be lightly oiled each day of use. Generally, the runway beams, when not in use, shall be stored as for steelwork. Bolts, nuts, washers and Lindaptors shall be cleaned, lightly oiled and stored in containers for re-use. Any damaged parts shall be replaced.

13.7.4 Kentledge

Kentledge or cantilever counterweight is generally required to augment the weight of the anchor span However, in case the sequence of erection can be strictly controlled, the engineer-in-charge may certify that kentledge is not necessary if:

- he considers the link set as part of the anchor span
- the span is lowered onto its bearings or at least onto cribwork before either its stringers are installed or any other significant load is applied
- the anchor span is complete with stringers.

Is a kentledge required, it should be installed as soon as the anchor span is completed, i.e. before commencing the permanent span. Prior to placing the kentledge, the stringers and protection timbers must be installed throughout the anchor span.

The required mass of the kentledge is indicated in the manufacturers instructions. It may be comprised of concrete deck slabs, steel sections or plates or boxes containing earth. In any case, the mass of the load must be known within an accuracy of +10% / -5% and it must be capable of being stacked in a completely stable arrangement. A substantial platform designed to span between the end bay stringers and to spread the kentledge load is required. When constructing a second span, kentledge would again be needed on the first one as an anchor span unless similar precautions, as above, are applied. When constructing the third span, the first and second span can remain linked, acting together as an anchor.

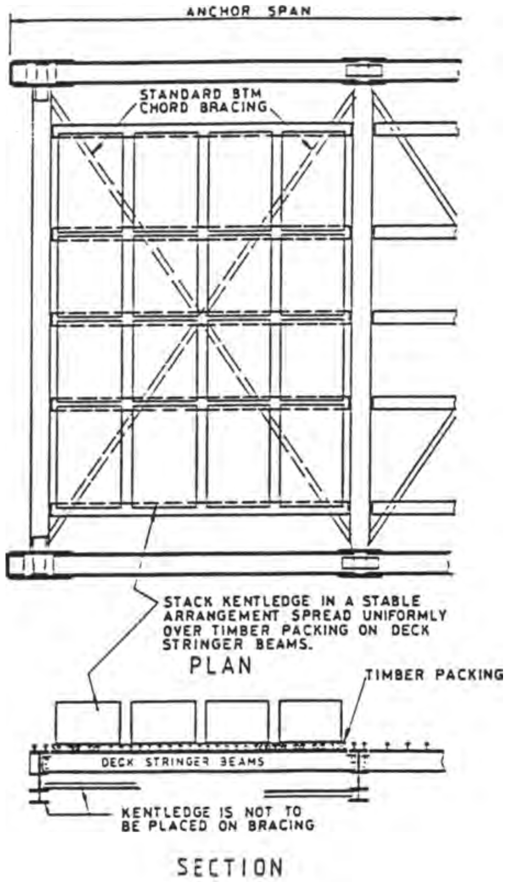


Fig. 13.14

14 Carriageway

14.1 General

The bridge deck consists of a concrete topping slab cast onto panels of precast, reinforced concrete. The composite slab bears onto longitudinal stringer beams which are designed as continuous spans over the 5.21 metres between truss cross-girders. These internal stringers are provided with shear connectors to achieve composite action with the slab . The cross-girders are designed as simply supported, spanning 8.5 metres between the trusses.

The precast panels span between stringers and act as formwork for the cast-in-place topping slab. They are designed to support their own weight plus the weight of wet concrete on the topping slab plus a live construction load of 2.0 kN/m2. They must not be subjected to concentrated or impact loading.

Vehicular loads are carried by the composite slab and the bridges have been designed to suit a bituminous wearing course. The cross-fall in the finished deck surface arises from falls built into the topping slab.

14.2 Precast Concrete Deck Panels

14.2.1 General

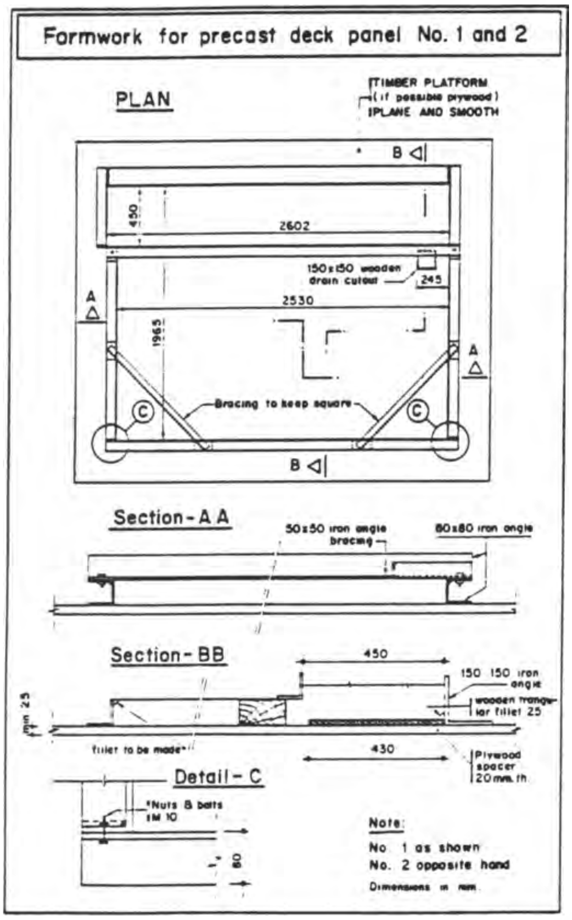
Each span (36.50 m) requires the fabrication of 56 precast concrete panels of three types . It is important, especially for a multi-span bridge, that these are available to provide temporary deck access as the bridge extends. For bridges including a walkway an additional set of panels is required.

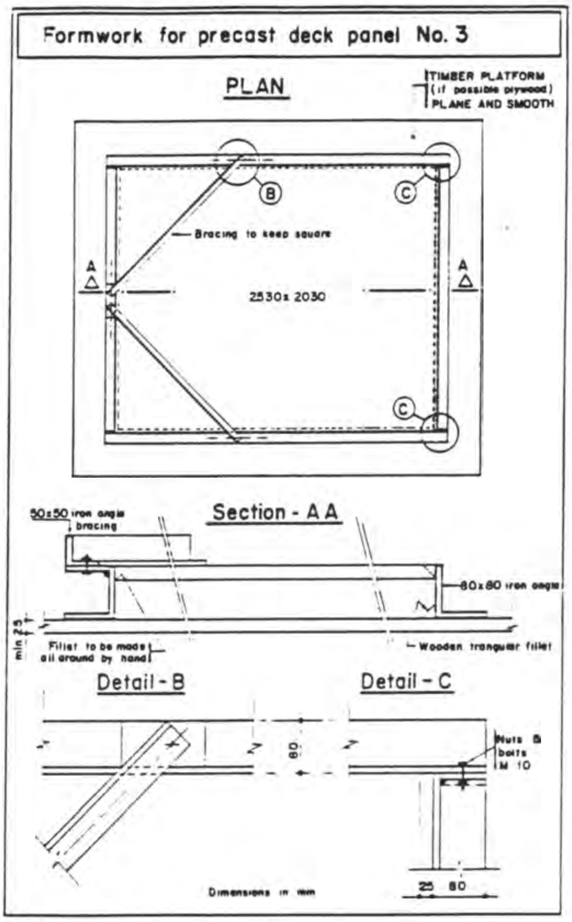
As any thin section structural concrete, the precast panels must be accurately made and close to the design size, regarding thickness, positioning of reinforcement and concrete quality.

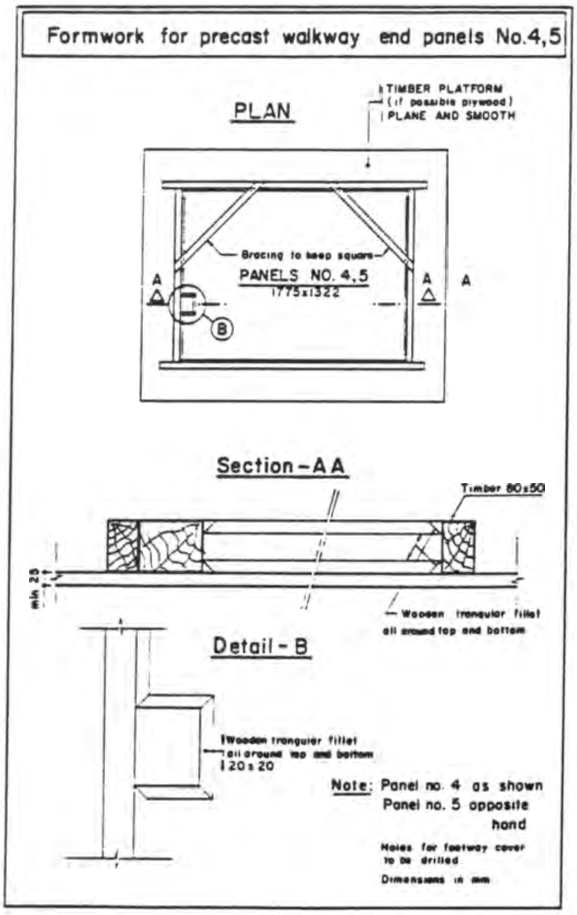
14.2.2 Formwork for Precast Panels

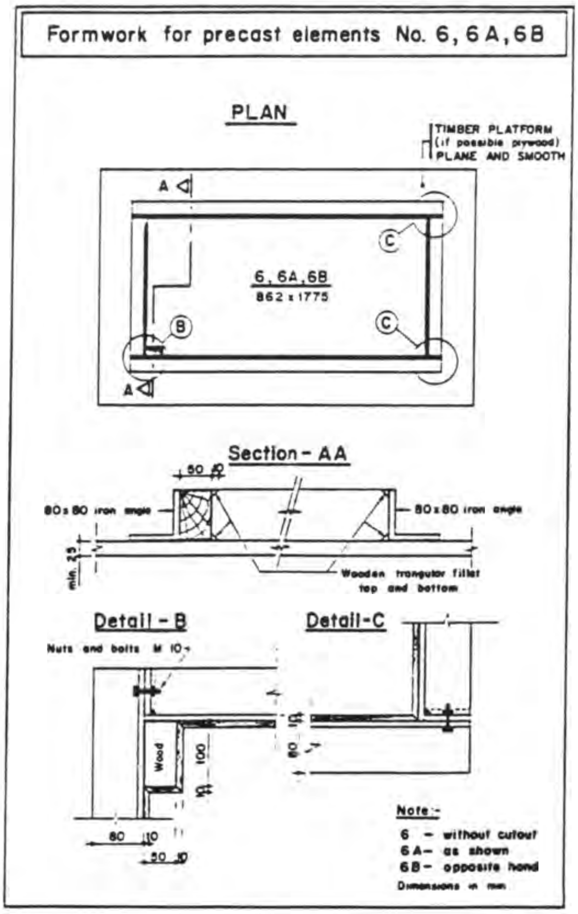
Formwork drawings (Fig. 14.1 - 14.4) detail the forms required to cast the three main types of precast panels for the main deck, together with three panel types for use when a walkway is also included. Edge formwork is a 80 mm steel angle with further angle added as bracings to ensure units are cast square. In all cases, it is very important that the base board is adequately strutted and supported from beneath to ensure that the completed panels are not twisted, because panels out of plane will not bed properly onto the supporting steel stringers.

Fig. 14.1 - 14.4: Formwork Drawings No. 1 - 6 for Precast Deck Panels









Because of the thin section, it is important to avoid any leakage of cement laitance through the forms. The base board needs to be made planar and to rest firmly on the ground. The steel form edging must sit properly on the base and, should this prove difficult, the steel angle forming the edges must be drilled and bolted to the base or fixing clips should be fitted. It is important that stripping should be simple and without damage and that reassembly should not require serious supervision. Steel angle pieces are best permanently marked and kept as sets. Should accidental bending and subsequent leaking occur, the affected part must be replaced.

14.2.3 Casting of Panels

The 80 mm thick panels are designed for a minimum concrete strength of 25 N/mm2 (Details of design see Section 8.2.1). The concrete needs to be in accordance with normal RC as per Table 14.1 with a maximum aggregate size of 32 mm. The concrete must be made with a low water/ cement ratio of maximal 0.50 (compare also Section 12.5.2). During casting, proper vibration is required to ensure that the lower surface and particularly the corners and bearing surfaces are without voids. The surfaces of the formwork must be thoroughly cleaned and properly coated with a release oil to permit easy removal from the forms without damage. The use of old engine oil is not allowed, because it will permanently damage the surface and encourage water absorption causing subsequent corrosion of the reinforcing steel.

Curing of the panels with water should be started three hours after casting and then continued by keeping wet for at least ten days.

14.2.4 Finishing of Panels

As the panels form part of the composite deck, good bonding to subsequent in-situ concrete is necessary. The upper surface must be roughly brushed one hour after concreting. Slabs are to be cast with projecting reinforcement, consisting of tie bars and edge continuity bars, with edge fillets and drainage holes. The outer panels are provided with recessed seating to bed exactly onto the outer stringers and with drip channels, both requiring careful stripping to keep a high quality finish. The position of drain holes requires advance planning and careful execution to avoid discharge of water onto the metal parts below.

14.2.5 Storage and Transporting

After casting, two days are required for setting before stripping off the form from the panels. The elements may then be moved carefully using polypropylene slings. Curing on adequate support to prevent cracking must be continued in a properly organised storage yard where the layout permits easy movement of panels by the mobile crane. A system of marking should also be developed to ensure panels are cured for the correct length of time and not brought out to the bridge until sufficient strength has developed.

TIME SCHEDULE FOR PRECAST ELEMENTS (Pathwe Bridge)

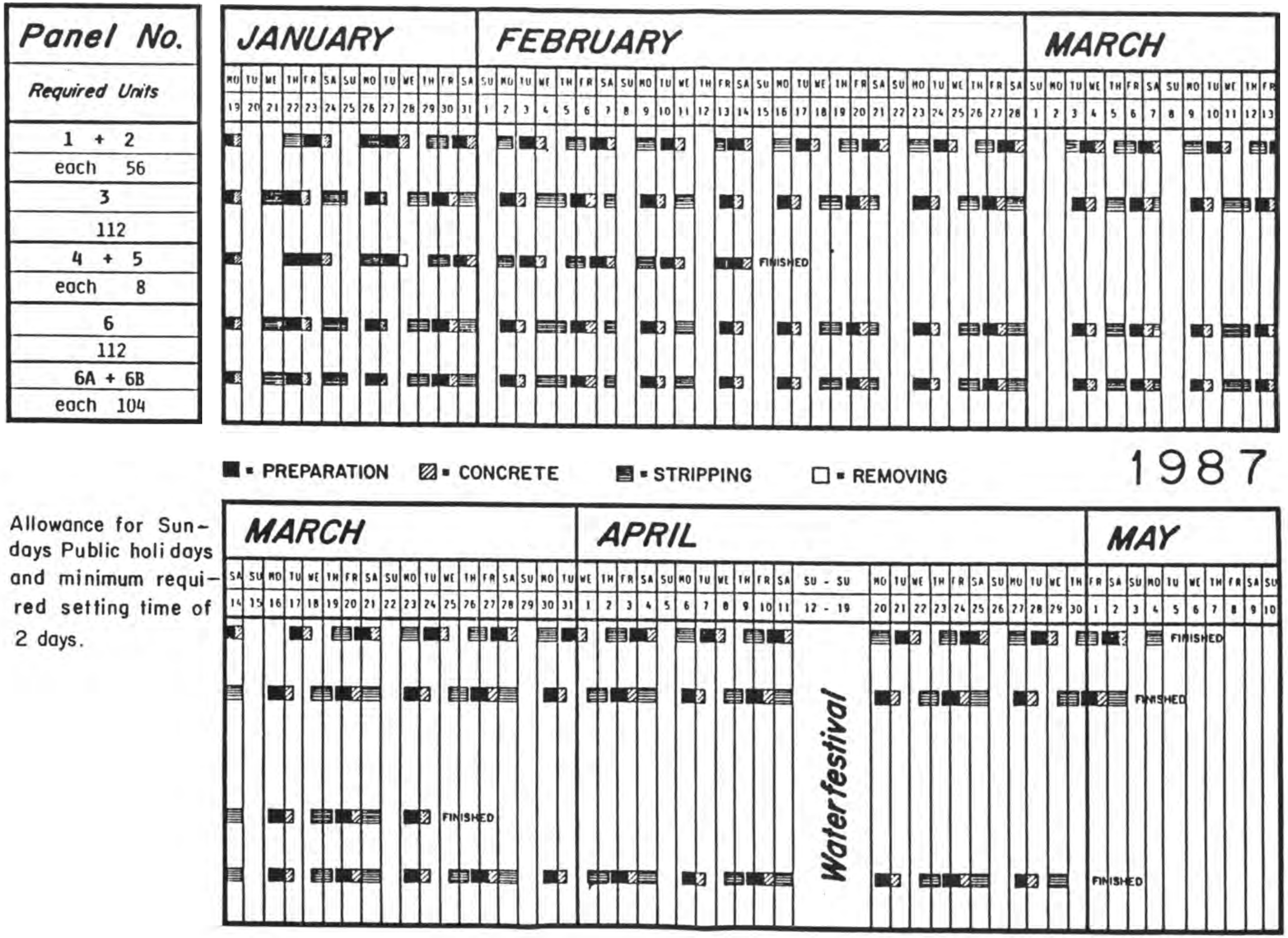


Fig. 14.5: Time Schedule for Precast Deck Panels

Foto 19: Precast Concrete Panels

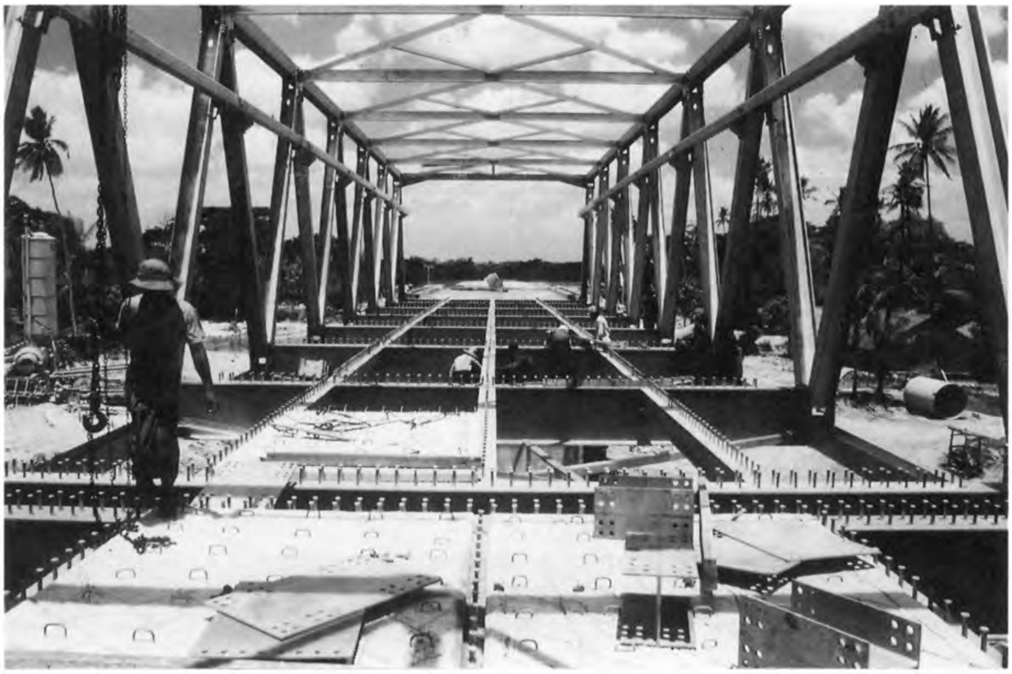
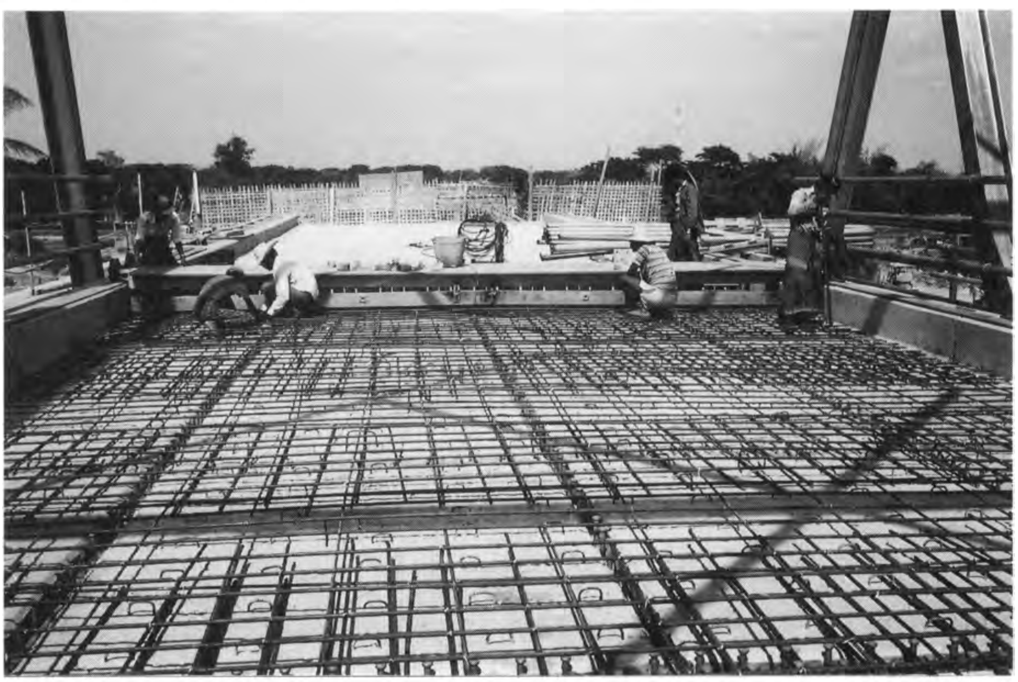


Foto 20: Concrete Platform and Deck Reinforcement



The panels may easily be transported into position along the bridge using hand labour and the overhead gantry crane. Each panel is placed directly on the supporting steelwork, abutting the adjoining panel. Any gap between supporting steel and abutting concrete which could cause the panel to rock, or gaps where slurry could be lost during concreting, should be packed with a stiff 1/3 cement-sand mortar.

14.2.6 Panel Casting Programme

For a single bridge span, without walkway, 56 panels are required. For multi-span bridges, a multiple number of panels is needed cured and ready to be used as access way to the already erected truss spans. As a minimum, each panel will require a day for formwork preparation, reinforcement fixing and casting, two days to initial curing such that movement to store is possible and then at least a further eight days for curing in the store. If the minimum of four sets of forms have been made then for a single span a minimum of 42 days would be required with no allowance for damage replacement, materials shortfalls or other problems and without any time period for festivals or even a weekly day rest.

A minimum time devoted for producing a programme for panel production is advisable, see example in Fig. 14.5. Panel production cannot be quickly accelerated and the working of overtime cannot, e.g. unlike steel erection, make up for delays to the programme. Care is also required during the period of pile casting that labour and concrete requirements do not conflict. "Planning" cannot prevent such double requirements but, by programming reasonable spare time into the programme the undesirable consequences may be avoided.

14.3 Reinforcement of Deck Concrete

Steel reinforcement, as specified by grade, diameters, lengths and shapes indicated on the drawing, should be fixed into position. The concrete deck is designed to provide a minimum dead load and hence is a near to a minimum thickness. Therefore, it is important that reinforcing steel is fixed at the correct spacing to concrete surfaces . Although plastic chairs are preferred, 1:3 mortar blocks carefully made to the correct size are also usable.

Reinforcing steel, in both in-situ deck and precast panel casting, is designed to take up tension stress, including cracking. It should be free of grease and loose rust scale and the formwork clean of paper and other loose materials. Joints in reinforcing bars should be in areas of low stress loading. Overlapping of bars should be 45 times the bar diameter and, where reinforcement bars are bent aside at construction joints and later bent back to their original position, care should be taken to ensure that at no time the radius of the bend is less than 6 times the bar diameter. If high tensile steel is used, the steel must be bent cold.

The following convention has been used:

1. Main Steel
2. Distribution Steel
3. Stirrup
4. Spacer

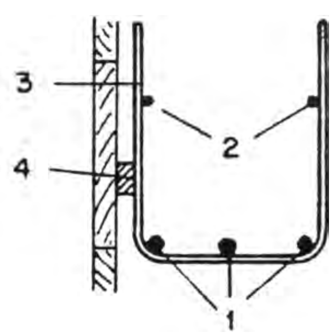


Fig. 14.6

All sections are thin and, before concreting, it is important that the cover is very carefully checked. Generally this should be a check upon the adequate installation of spacer blocks for the cover below. For the deck, top cover is also critical and the concreting platform should be set up with its angle guides correctly fixed. The platform may then be moved slowly over the section; the top cover must be checked by its clearance.

14.4 Kerb Formwork

It has been found advisable to cast kerbs in-situ and not, as originally proposed , as an integral part of the precast slabs. In order to create a regular flowing shape over the bridge camber, the height of the kerbs must be set at each joint (480 mm above the cross-girder) . These points are connected directly, creating a top of kerb curve comprised of a series of straight lines. At each cross-girder, kerbs require an expansion joint, formed using 5 mm thick plywood panel. Bays are cast alternately, using the formwork as detailled in Fig. 14.7.

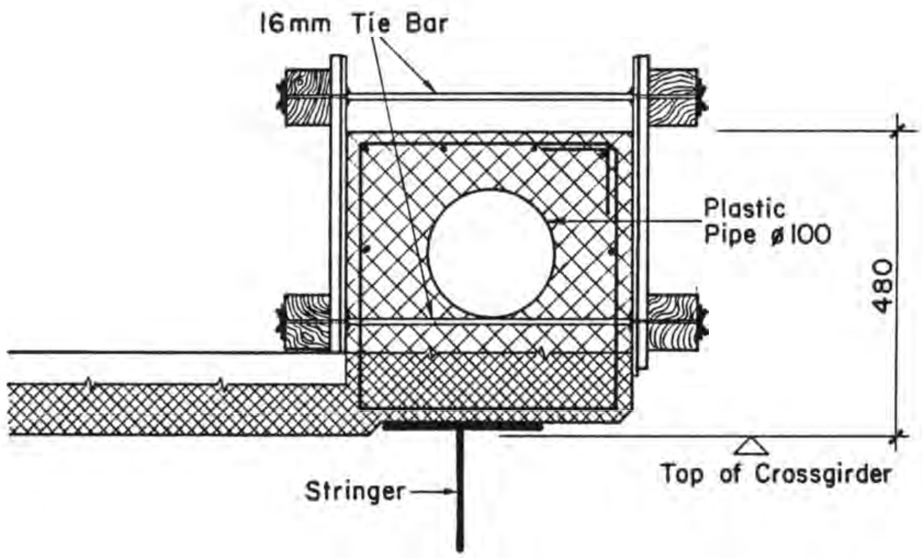


Fig. 14.7

Because of the very obvious and unpleasing impression created by irregularities, great care is required with kerb formwork in order to keep the horizontal alignment straight and the vertical alignment curved. Slight errors or bending of the formworks between bolts creates curves in the finished kerb which look much larger than errors in other places, giving the impression of careless work. To create a good line, the alignment should be exactly checked, with adjustments made along the whole section, before concreting.

14.5 Deck Concreting

14.5.1 General

The design of the concrete deck requires that the concrete will act compositely with the precast panels and the stringers and cross-girders, so it is most important that the surfaces of precast slabs are roughened during casting and are clean. The reinforcement must be fixed within specified tolerances, and the in-place concrete thickness and quality have to be maintained.

The end section of each span includes a 462 mm overhang to be cast without precast panels. Formwork support is given by either the pier or abutment. Fitting the carriageway protection angle to the ultimate road profile should be to an accuracy of 3mm and requires care (Details see Fig. 14.11).

14.5.2 Concrete Consistency

The in-situ concrete required to form the composite deck provides only a thin overlay to the precast panels. This makes particular demands of the mix and the placing of the concrete. It is important that the consistency is appropriate. There are four widely accepted descriptions of consistency for concrete :

DRY EARTH :		Concrete like damp earth, rolled or rammed only (not vibratable). Not for structural use.
STIFFLY PLASTICISED :		Concrete like sticky lumps, vibratable. For all RCC with high demands on compressive strength.
SOFTLY PLASTICISED :		Rather liquid concrete. For normal RCC.
CHUTED CONCRETE :		Liquid concrete. For hand working or pumping.

Fig. 14.8

The use of pumped concrete is limited by its resistance to flowing. Experience with the cements and aggregates available (e.g. Kyoung-gone bridge) shows that despite benefits in time and organisation with pumped concrete, 150 m is a maximum distance before the liquidity of concrete required for pumping is such that the water/cement ratio becomes unacceptably high. Where longer distances are unavoidable, part of the placing may have to be with traditional methods.

The following table provides a guide, such that the mix may be designed in such a way that the characteristic compressive strength at 28 days will not be less than the values given.

Use	Consistency (W/C Ratio)	Compressive Strength		Cement	Sand 0-4	Gravel 4-16	Gravel 16-32	Tot. Aggregate	Water
		N/mm2	PSI	kg/m3	kg/m3	kg/m3	kg/m3	kg/m3	l/m3
Not vibrated. As lean- or fill concrete for haunching of pipes etc.	Earth dry " " "	5 7 10 12	746 1045 1493 1791	75 100 150 200	720 710 690 680	720 710 690 680	610 610 600 590	2050 2030 1980 1950	30 40 50 70
High grade RC, vibrated. For elements with high demands on compessive strength.	Stiffly Plasticised (0.4 -0.55)	25 30	3731 4478	300 350	670 660	670 660	580 570	1920 1890	150 170
Normal RC, vibrated. With medium strength.	Soft Plasticised (0.55 - 0.7)	15 20 25	2239 2985 3731	250 300 350	680 645 610	680 645 610	590 560 530	1950 1850 1750	150 180 210
Pump concrete, few or nil vibration. For low grade RC only.	Chuted or Pumped (0.7 - 0.8)	20 25	2985 3731	300 350	625 595	625 595	540 510	1790 1700	225 260

Table 14.1

14.5.3 Concreting Platform

In order to permit the placing of the deck concrete to be efficient and accurate a moving platform has been included, as part of the construction equipment, to provide access during the in-situ concrete work. The platform spans across the deck, is 1.0 m wide and runs approximately 240 mm clear above the concrete surface. It consists of a steelframe, plywood decking and rubber tired wheels designed to run on the inside edge of the top of the kerbs. The falls required for the final road surface are cast into the topping slab using a modification to the platform. Adjustable steel angles bolted beneath are designed to define the required surface profile with falls of approx. 40 mm each way from the centre of the deck to the kerbs. This permits the asphalt surface to be simply laid and later repaired or replaced without the complication of wedge shaped layers.

However, the primary purpose of the platform is to provide access and working space for placing and finishing the concrete slab. The carrying capacity is 250 kg, or 4 workmen plus equipment. The platform is pushed ahead of the concrete work and this should be done evenly and under control to prevent jamming.

Foto 21: Concreting of Deck Overlay

Foto 22: Curing of Deck Concrete

14.5.4 Concreting Times

Generally, in Myanmar, exposed concrete should not be placed during the hottest time of day without any shadow. The best time for concreting work is in the late afternoon as the sun temperature is dropping or early in the morning with everything prepared the day before. Casting of thin structural elements like deck concrete overlay should be done in the late afternoon and onto well wetted precast slabs. Staff should be prepared for night-time work.

14.5.5 Curing

Both the precast panels and the in-situ deck concrete are cast as thin sections which could quickly dry out with cracking resulting in a serious reduction of compressive strength and durability. Curing must start immediately when initial setting becomes apparent (approximately 3 hours after casting) and continue throughout the following ten days. Because of high night-time temperatures in Myanmar, it is essential that arrangements are made for curing to be continued around the clock.

All surfaces of panel and in-situ decking are required to be rough, so curing may be commenced without fear of damage to the surface finish. Curing to concrete kerbing, which has fair top and side faces, should commence once surface setting has occured (after approximately 4 hours) but with a hessian cover to prevent direct flow of curing water onto the surface. After stripping curing, the hessian must be replaced and the kerbs watered for ten days.

14.6 Walkway Deck

External walkways are provided on some bridges (e.g. Kyoung-gone). These comprise precast, reinforced concrete slabs bolted to a cantilevered steel frame installed only after landing the span. Channel sections are provided as stringers spanning between cantilever support members at each truss section. The stringer provide a bearing for the precast concrete walkway panels (see Fig. 13.10).

The 80 mm thick slabs for walkways are to be cast with bolt holes and rebates to suit the railing posts. They must be installed over a strip of bituminous felt, holed to suit the fixing bolts. The panels are attached to the stringers with M16 bolts with provision previously having been carefully made in the panels to fit with the predrilled holes in the stringers. Washers are to be installed under both the nut and the bolthead and, once the position and level is checked, bolts should be firmly tightened and the thread spoiled by punching to prevent any possible loosening of the nut (Fig. 14.9).

The panels are designed such that no further topping is required and for a nominal gap between sections of 6 mm. Care is essential to both install the panels to matching levels and to carefully form the bevels at each edge to prevent a potentially dangerous step being created.

The junction of the last panel of the walkway and the abutment requires the installation of a special cover plate, bolted to the panel (Fig. 14.10).

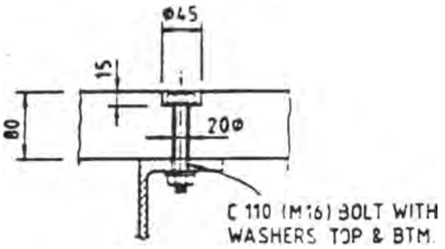


Fig. 14.9

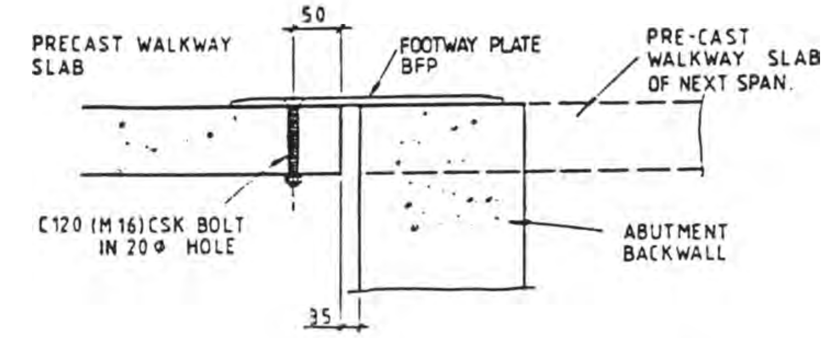


Fig. 14.10

14.7 Drainage

Longitudinal falls exist as a consequence of the camber of each bridge section. This will vary from approx. 1: 100 to be levelled at the centre section. A 40 mm two way cross fall is foreseen for the topping slab and the combination of these two falls will carry water to the kerbs and then to the 75 mm diameter PVC scupper pipes which are to be cast into each side of all bridge sections. It is necessary to plan boxed out sections to be left out in some of the panels in positions that the scupper pipes will discharge away from the steel parts below.

14.8 Deck Seals and Protection Angles

Preformed cellular elastomeric deck seals are to be installed between the deck protection angles at the ends of the span and the abutment or the adjacent spans (see Fig. 14.11).

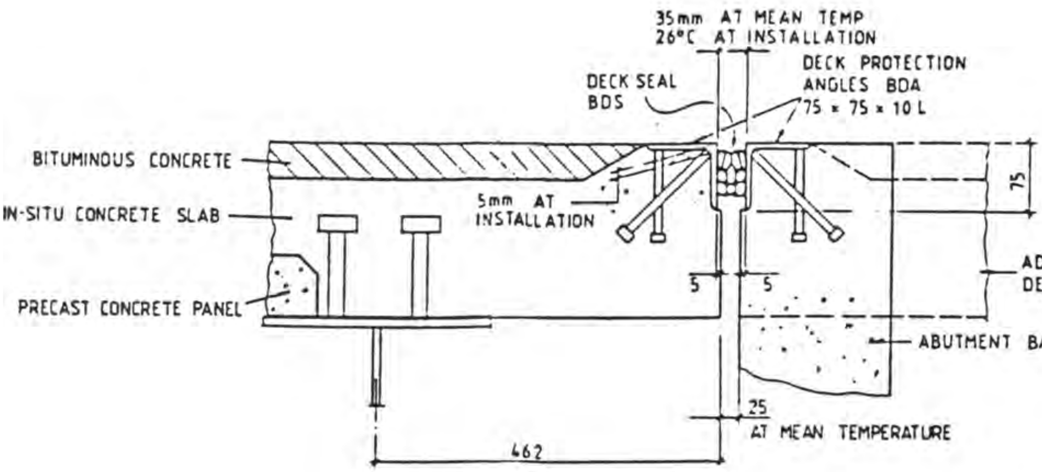


Fig. 14.11

The protection angles are set to the cross-fall of the deck, to finish flush with the bitumen surfacing, and will therefore project above the top surface of the in-place slab. The space under the angle is to be packed with concrete. The angles must be spaced very accurately to suit the seals and, the seals are glued in place using the lubricant adhesive provided. For these reasons, the protection angles must be securely braced together and fastened by tie - wire or, better, by bolts, to the deck formwork before concreting such that they will not move. Holes are provided for this purpose.

Protection angles are supplied with the required steelparts. The angles BDA are cast into the concrete to the following tolerances :

Level	< 3 mm
Out of square	1 part in 200
Joint spacing	3 mm

Table 14.2

Installation should be as follows:

step 1: - Thorough cleaning of steel surfaces of the protection angle,

step 2: - Application of lubricant one metre ahead of installation,

step 3: - Starting with a turn up at the kerb at one end, inserting the seal by squeezing into the joint using a flat steel plate,

step 4: - Forcing down the seal to its final position by progressive pressure. Cutting off leaving turn up at kerb.

The seal is to finish approx. 5 mm below the top of the angles.

15 Maintenance of Plant and Equipment

15.1 Maintenance Planning

In addition to operational and safety requirements common to all construction sites, deep bore piling makes special demands upon equipment, in particular, that it should work continuously throughout long periods without stoppage for maintenance or breakdown. In particular, if the concrete batching plant, cranes, concrete or bentonite pumps fail during piling works, it is likely that the pile would not be usable and expensive alternations to the foundations have to be designed. Whilst the situation is unlikely to be as problematical for other construction activities a programme of plant maintenance should be planned from the outset to ensure essential maintenance work will not hold up other essential site work.

An example for a " Service Overview" (see Fig. 15.4) indicates a simple programme extending over a six month period. This programme aims to reconcile the plant requirements of the primary construction programme with the objectives of the plant manager seeking to ensure all plant is in perfect working order for 100% of the time. It is not acceptable in any rationally planned operation to run plant until it breaks down or it is not withdrawn for service when its mechanical integrity demands that maintenance work is required. At the start of construction, some simple principles should be worked out, for example, the critical times for plant availability and the times when the cycle of , for example, piling work when key equipment may most easily be withdrawn from use for servicing. The principles could also consider phased servicing of duplicated items, e.g. water pumps and dozers and the extent to which non working periods, at night or on weekly free days may be used. Clearly the whole must also relate to the capacities of the workshop staff, the realistic maintenance requirements of the various items to be kept operational and the importance of building-in flexibility to deal with unforseen problems such as plant breakdown and staff sickness etc.

The main service overview will be supported by several other documents, three examples of which are enclosed. These should include regular periodic reappraisal of the actual service requirements of the various plant items (Fig. 15.1), personnel based plans which are short term work plans for the workshop staff (Fig. 15.2) and short term plans for the servicing of each item (Fig. 15.3). The objective in all cases is similar, to relate available service and maintenance capacity to best serve the aims of the construction programme.

PLAN AHEAD

for Equipment to be serviced

Page: ....

Date: ....

Site: ....

			Today km milage hours	next Service			estimated time	last Service			Remarks
Unit made	Asset	Location		aprox. Date	km/h	Type		Date	km/h	Type	

Fig. 15.1

Fig. 15.2: Personnel Based Plan

SERVICE TIME-TABLE FOR ....

Month: .... 19..

Day	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
Asset Comp. No.																
Type of Service																

Day	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
Asset Comp. No.																
Type of Service																

Fig. 15.3: Short Term Plan

Diary / Working shedule **Weekly Programme**

Monday Date:	Tuesday Date:	Wednesday Date:	Thursday Date:	Friday Date:	Saturday Date:	Sunday Date: ....
Noon / Lunchtime						
Remarks:						

Fig. 15.4: Example for Service Overview

RANGOON-BASSEIN ROAD PROJECT — SERVICE OVERVIEW — JAN - JULY 1988

			JANUARY				FEBRUARY				MARCH				APRIL				MAY				JUNE			
			1	2	3	4	5	6	7	8	9	10	11	12/13	13/14	15	16	17	18	19	20	21	22	23	24/25	26
MACHINE/VEHICLE	ASSET NUMBER	LOCATION	LAST SERVICE																							
LINK BELT LS 108 BHD												B 8.													C 15.	
ROTARY DRILL RT 3-5								A 8.																	D 23.	
GEN SET IVECO 150KVA								B 10.				D 15														D 27.
GEN SET IVECO 80KVA							D 7.								B 2.		C 21.			B 10.		D 21.			B 21.	
BATCH. PLANT AF 1561					A 18.									B 23.												
POWER PACK 2V-200S					A 18.											A 7.									D 25.	
VIBRATOR VS-8					A 18.											A 7.									D 25.	
BENTONITE MIXER BE10													A 19.													B 27.
do DESANDER BE 50													A 19.													B 27.
VIB. ROLLER RAMMAX							A 2.																	B 6.		
TRASHPUMP PT3B										A 27.																
BENTONITE PUMP JONID (1)											A 7.															
BENTONITE PUMP JONID (2)											A 7.															
GEN SET BOSCH 2.15 KVA						C 30.		B 12.		B 25.			B 22.					C 27.								
CONVERTER FU 4/200																										
INTERNAL VIB. IREK 1.64																										
INTERNAL VIB. IREK 1.64																										
AIR HAMMER LHB5 (1)													A 20.													
AIR HAMMER LHB5 (2)													A 20.													
BENTONITE PUMP LOMBARDINI									A 20.																	
TOYOTA HILUX SINGLE CAB												A 6.	B 15.	A 29			B 29.				A 21.			C 10.	A 24.	
TOYOTA HILUX DOUBLE CAB												A 6.	B 16.	A 31					B 9.		A 19.		C 1.			A 21.
MOBILE CRANE RT 58 B	2Z.055																					C 24.				
MOBILE CRANE TADANO	2Z.052										C 1.											B 29.				
CRETE PUMP HINO	2PU.001							B 12.						B 24												
AIR COMPR. BROOM WADE	2L.171						B 5.																			
AIR COMP.																										
GEN SET PUMA 30KVA	21.100											B 15.														
HYDR. EXCAVATOR 225																										
CAT D3B DOZER	1D099																						D 4.			
CAT D3B DOZER	1D 112																					D 29.				
CAT D3B BACK HOE																										
HINO TIPPER	R0-5023																									
HINO TIPPER	R0-5025																									
UNIFLOAT HARBOURMAST	2Z3.003																									
do HARBOURMASTER	2Z3.005																									
CAT 966C PAYLOADER	16.094																									
Z-CRAFT																										

Service activity (6-month)

Name: U SAW OO
Date: 26.6.88
Signature:

15.2 Maintenance Priorities

The nature of almost any construction site decides that some items of plant and equipment are close to indispensible and others, though important or useful, are of lesser priority for maintenance. For this bridge type the following became the established order of priority (detailed list of equipment see Section 12.2 and 13.7):

First Priority Maintenance	Second Priority Maintenance
Excavator Drilling rig Batching plant Concrete pump Bentonite pumps Bentonite desander Centrifugal trash pump Converter Mobile air compressor	Bentonite Mixer Vibrahammer with power unit Generators Mobile cranes Vibrators Stationary air compressor Compressed air hammers Hydraulic excavator Trucks Pay Loader and dozers Pick-up and cars Unifloat and Z-craft

Table 15.1

The above is based upon a plant and equipment overview for the construction of Pathwe bridge. At other bridge sites it may differ. This resulted in an overall maintenance requirement of:

21 units	Bridge Construction Equipment	High Priority Maintenance
12 units	Workshop Equipment	High Priority Maintenance
14 units	Outside Construction Corporation Plant / Equipment	To be maintained in collaboration with Public Works
totally 47 units	Site and Workshop Equipment	

Table 15.2

N.B. In addition to site equipment and plant maintenance, maintenance also has to include workshop machinery tools and equipment, which, because all other items are dependent upon them, must be accorded to a high priority in maintenance.

15.3 Workshop Layout

In view of the requirement to keep all plant in first class working order, the establishment of an adequate workshop, together with trained mechanics, is a high priority for early installation at a bridge construction site. In addition, it is essential that construction planning takes proper account of the stress upon equipment and that proper periods are built in to the programme to ensure that, at the very minimum, time is set aside to enable maintenance of the key, first priority items of concrete and drilling equipment.

The principles involved in designing a workshop for a bridge site are not too complicated. The area is separated to keep the store section separate from the working areas and similarly, welding and the oil store kept apart. The open working area must be high enough for all the equipment to travel inside and , of course, the separation of the main supports must be similarly dimensioned. The roof must be installed with at least 1 m of overhang and it is then possible to leave two sides of the working area open which provides at least some chance of cooling air movement whilst leaving the main equipment items protected.

Within the store area it is important to keep spare parts for the different types of machine separated and clearly labelled. This is particularly important for unique parts which, if mislaid may not be available elsewhere in the country.

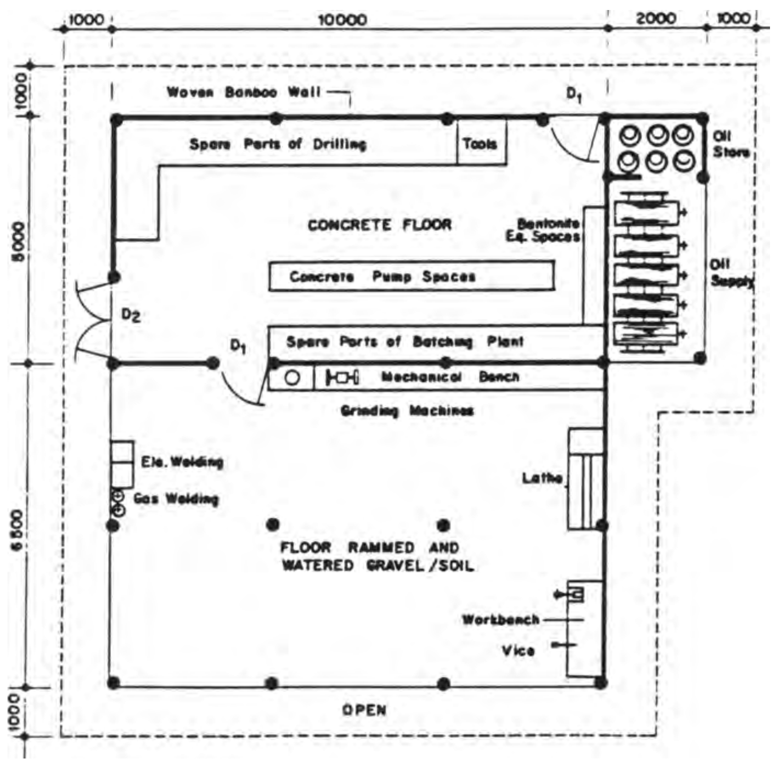


Fig. 15.5

15.4 Workshop Equipment

It is essential to have an adaquate workshop available at site from as early as possible. The very best arrangement is to have it operational to receive and check all incoming equipment and ideally it should be the first site building.

Foto 23: Inside the Workshop

Foto 24: Spare Parts Storage

The following workshop machinery, tools and equipment were utilised at Pathwe and there is little reason to believe a comparable site should operate with any less:

Drilling machine (portable)	DUAX D32 II	1 pc
Drilling machine on stand	BIMAK	1 pc
Electric welding units		2 pcs
Autogen welding units		2 pcs
Circular saw	BS500, AT 55E	2 pcs
Chain saw	Jonsered 630	1 pc
Welding Transformer	SIRIO 160a	1 pc
Drilling machine	DUAX ZS 8 II	1 pc

Table 15.3

Once the workshop building is finished and installation of workshop equipment can begin attention must be paid to the following:

1. All bare and shiny metal parts should be covered with a thin film of oil to prevent the metal from oxidizing.

2. For each kind of machine, spare parts have to be sorted out and placed in the correct section of the spare parts store. Keep parts in their original boxes. Don't mix rubber with lubricants. Parts should not be stored on the bare floor.

3. Store spare parts and workshop material in the spare parts store, but in different places as they do not belong together. Ensure that there is air circulation in the spare parts store.

4 The oilbar section outside the spare parts store must be kept clear of rubber, tires and gas welding bottles.

5. For safety reasons, keep one empty and top opened oil drum filled with water and one bucket filled with sand in the workshop in case of fire.

15.5 Equipment Service and Maintenance Records

During the project, a start was made to establish a system for recording various facts relating to all the equipment introduced with the aim of keeping the machines operating efficiently. These are as set out below:

a) Vehicle Identification Document (Fig. 15.6):

   This is intended to record all details of the machine which would be required to permit correct ordering of spare parts, it sets out a proposed service timetable and includes basic physical details important for shipping.

b) Daily Machine Report (Fig. 15.7):

   This report relates the output of each machine with the inputs provided. It provides not only a link between the maintenance staff and the work the machine has undertaken but for construction planning provides a record of the useful utilisation of each machine and the inoperative or poorly utilised periods .

RANGOON BASSEIN ROAD PROJECT

VEHICLE IDENTIFICATION SHEET

Asset No.: .... Company No.: .... Date of Purchase: 18. 8. 1986

Vehicle / machine kind: CRANE /EXCAVATOR LINK BELT

Supplier of Vehicle/Machine: Piazzi Alcide + C. s.n.c. 41018 S.Cesario sul Pan.

Agent abroad / Burma: Telex 222128 PIAZZI I Tel.(059)923313

Identification — Made: Link Belt; Model: LS 108 B HD; Chassis No.: 9LKG1732; Type: ....; Year of production: ....

Identification of Engine — Made: General Motors; Model: ....; Displacement: 6.99 ltr; Type: GM 6/71N; Performance: 165 HP; Serial No.: 6A 40300 -LC; Rev. per Minute: ....; Arrg. No.: ....; No. of Zylinder: 6

Battery: 12V/145Ah | Tyres and size: .... | ....: ....

Identifications: ....

Gearbox Torque Converter	Hydraulic Pump	Hydraulic Motor
Made: Allison	Made:	Made: ....
Type: TCDOA 475	Type:	Type: ....
Serial No.: 6880983	Serial No.:	Serial No.: ....

Weigts: - Standard weight: 27125 kg - With load: 62590 kg

- ~~with~~ attachments: 48365 kg - Pressure of kg/cm2: 0,91

Type of Service and Interval:
Ⓐ Type every 60 hours or weekly
Ⓑ Type every 250 hours or monthly
Ⓒ Type every 500 hours or every 6 months.
Ⓓ Type every 1000 hours or every year.

Remarks: Date of inventory: .... Milage/km/hours: 325 h

ORDER No BU-860717

(More details overleaf yes ☒ no ☐)

Fig. 15.6: Vehicle Identification Report

DITECO THE CONTRACTOR

SERVICE REPORT

SHEET

Asset No.: .... Company No.: ....

Vehicle / Machine made: Crane /Excavator LINK BELT

Chassis No.: 9LKG 1732 Type: LS 108 B HD

Date of Service	Milage km/hours	Type of Service	Hours of work	Name of Mechanic	Remarks Observations	Signature
Jan 87	60 h			D. Cassoli	With all Filters	
June 87	265 h			D. Cassoli	With all Filters	
2.3.88	431 h	Ⓑ	3/6	U Hlay Lwin	With all Filters	
15.6.88	513 h	Ⓒ	27/81	Hlaing Win	Acc. Service Manual LB/GM	

Fig. 15.8: Service Report

Daily Machine Report

Nr. 02529

Date

Construction Site

Construction Site No.

Type of Machine

Inventory Nr.

Diesel (fuel) | Diesel (cleaning) | Operating time (Hours) | Waiting Time (Hours)

Type of Work	Item	Output Transports, m³	Operating hours

Diesel (fuel)	Ltr.		Total Operating Time	
Diesel (cleaning)	Ltr.		Waiting Time because of	hrs
Petrol	Ltr.		Machine Repair	hrs
Oil/Grease	Ltr./kg		Works without Machine	hrs
Km and Hour reading			Total Hours	

Remarks:

Operator

Fig. 15.7: Daily Machine Report

c) Service Report (Fig. 15.8):

This follows from the timetable as set out in the identity document and provides a continuous record of the required regular services through the life of any plant item .

d) Repair and Maintenance Report (Fig.15.9):

This is the equivalent document to the Service Report but is intended to record non-routine works, breakages, special services and examinations and should provide an accurate account of the problems a machine provides through its life.

e) Condition Report (Fig.15.10):

This is a special document which is especially appropriate to plant which is used non-continuously. In Myanmar, it will be compiled as a machine is taken out of service before storing for the monsoon and will seek to ensure that when the machine is next required, a record of where any stored assessories or removable parts are stored, will record the results of a more than normal check of all wearing or vulnerable parts and will record exactly which spare parts are required to restore the machine to its best working order. It may also be used as a training / familiarisation record as it is generally produced at a time when normal siteworks demands upon machines should not be present and there is time for more experienced mechanics to pass on some of their knowledge as others learn about additional equipment .

Fig. 15.9: Repair and Maintenance Report

ITECO
THE CONTRACTOR

REPAIR AND MAINTENANCE
REPORT SHEET

Asset No.: .... Company No.: ....

Vehicle / Machine made: LINK BELT CRANE
Chassis No.: 9.4KG 1732 Typ: LS 108 HD

Description of work carried out (Foreman: FM, Mecanic: M1, Helper: H1)

Date		FM	M1	M2	H1	H2
15.3.88	Search for Pressure Coil, Replace Accu: Assy	10	10		10	10
14.6.88	Rotating Joint Replacement 5 No (L-U, F-1, R-1, R-U, F-1)	1	1		1	
"	Repair to clutch Cyl. Assy, Right Upper front 1	1	1		1	
16.6.88	Examination for worn-out parts and General check-up	15	15		15	15
	Total hours work					

Quantity	Meter Size	Description	Spare Parts Number	Price per item	Price
15.3.88		Accumulator Assembly	JC 2598		
14.6.88	1	Filter, Hyd Drain Assy	PX 0468		
"	3	Rotating Joint Assy, clutch	1J 0673		
"	1	Oil Seal and Ring.	1J 1919		
"	2	O-Ring, Back-up	PX 0309		
"	2	O-Ring, clutch Cyl. Assy	PX 0308		
11.6.88	1	Oil Filter Full Flow (GM 6-71N)	5 573 014		
11.6.88	1	Fuel Filter Element	5 574 508		
11.6.88	1	Fuel Filter Strainer Element	5 575 032		
15.6.88	29 lt	Hydraulic Oil HV 46			
"	21 lt	Engine Oil, S.A.E 15W-40.			
"	5 lt	Air cleaner Oil, SAE 15W-40			
"	40 lt	Torque Converter Oil, DONAX TM			
"	8 Kg.	Multipurpose Grease			
"	6	Grease Nipple			
		Total value of Spare Parts other Materials			

Fig. 15.10: Condition Report

CONDITION REPORT OF

Machine: BATCHING PLANT, ORU Model: Autoforculor 1561

Chassis :

Type :

Working hours appox. : 190 hrs.

1- Repair and Rebond damage parts. specially to scoops steel sheet and Swing element pipes
2- Clean steel and metal parts from concrete reminder
3- Used old-oil apply with brushes or rags to the bare metal where they are not covered by paint.
4- Apply sufficient grease to the motion parts.
5- Apply anticorrosion "Knorrostin" where ever parts have to be protected from corrosion.

Measures to the electrical parts storage in Bandi workshop as:

1- Electric water Pump	2 Nos	6 - Geared Motor	3 Nos
2 - Air Compressor w/Motor	1 No	7 - Hyd: Motor	2 Nos
3 - Valve w/ Motor	1 No	8 - Vibrator Motor	1 No
4 - Remote Control	1 No	9 - Control pannel	1 No
5 - Dial head	2 Nos	10 - Guide Roller	4 Nos

Dated and place of report 30-6-88 Bandi / Pantanaw.

Swiss Mech Eng
Swiss team
P. Hausmann

Mech Eng
C.C
U TIN NU

Mech Eng
C.C
U SAW OO

15.6 Staff Requirements

Experience and investigation indicate that, for the maintenance of one construction plant and equipment unit of any size, a minimum of 0.9 hours/day are required. For Kyoung-gone there were 47 units, thus:

47 x 0.9 hours = 42.3 hours/day.

The daily working hours per person are 9 hours, thus:

42.3 / 9 hours = 4.7 => 5 labourers.

Further experience has shown that the relationship of qualified mechanics to the required number of unqualified persons or helpers should be one to one and therefore are required:

3 qualified, competent plant mechanics plus 2 helpers.

Whilst the number of items in the plant and equipment may vary slightly from site to site, the above method of fixing the maintenance establishment should be regarded as a minimum. Even with this level of staffing, there will be periods when no leave or days off will be possible.

15.7 Monsoon Storage

Care of the plant and equipment in Myanmar requires special measures as works cease during the monsoon season. This is a period of opportunity for major overhaul but also one of the problems related to water and plant or mould growth.

In general, equipment should be stored under cover but with a free circulation of air as much as possible. All engine exhaust pipes should be closed; electric motors should be cleaned and removed to a dry, or well covered place. Control levers on the plant should also be sealed from water access.

The greatest care should be taken to protect electrical parts. If there is any doubt about protection from rain, the part should be removed to store with a careful record made of the disconnected cables. All batteries should be removed and terminals cleaned. Batteries should be recharged at 3 week intervals.

Any bare metal parts, e.g. hydraulic cylinders, should be greased or lightly oiled to prevent corrosion.

Details of the procedures adopted for this special storage should be recorded on the condition report, see above, which will be produced at this time.

15.8 Spare Parts and Workshop Store

An essential part of any site workshop is the associated spare parts store. The establishment of this facility will in large part be guided by the past service records of the plant expected to operate at the site the level of reliability expected and the restocking time for the various spares and consumable items. The

stock level will vary very widely between parts available on the open market, parts available but only in a central store or distant capital and parts specific to a particular machine which are available only in the country where it was manufactured. At the level of a particular store, the most important feature is that all parts supplied are able to be stored in appropriate conditions to ensure the parts are useful upon demand. This is in part physical conditions which prevent damage through moisture or other prevailing conditions but is also appropriate marking to both find the part and to be secure that it is exactly the part as specified by the manual. The layout and construction of stores as well as the proper management are each important, to have spares supplied but then lost or damaged before use is both common and a sure sign of something seriously wrong with the system. It is especially a problem where the spare parts are unique to a machine and not easily replaced as, in this instance, very expensive plant may be put out of action for want of parts which cost almost nothing.

There are again a few principles which should guide the operation of the system whether wonderfully computerised or maintained on card (Fig. 15.11 and 12). Each part must be recorded in a way that the record obviously related to the particular machine or manufacturer and also to the repair and service manuals available. The record should include information upon its correct or normally used name, the date and source of supply and the actual and agreed resupply level of stock for the part. Depending upon the system details of cost may also be included. The system should seek to ensure that each part is uniquely labelled such that similar parts for different machines are not confused, the physical storage system should prevent any mixing and that parts may be found by some logical process not depending upon any one persons wonderful memory. The system should also include some overview which will generally take the form of periodic summary stock cards from which deficiencies should be apparent very easily by comparison with the designed or specified stock levels.

The above principles should apply equally to fast moving consumable items, e.g. oils and gasoline as to very specialised and only periodically required replacement parts (Fig. 15.13).

STOCK CARD	Spare Parts Number:	Storage place: .... Costs of item: .... : ....
Name of unit:	Weight per item: .... Measures volume/item:	Supplier's item and/or order No.: ....

Name of item	Date		Number of items	Balance	Signature	Asset No. unit utilized Spare Parts for
	In	Out				

Fig. 15.11: Stock card

Fig. 15.12: Summary of Stock Cards

SUMMARY

STOCK CARD

Asset No.: .... Company No.: ....

Name of unit: .... Made: .... Model: .... Type: ....

Name of item	Spare Parts Number	Measures Weight per item		Date / Number			Balance	Signature	Asset No. unit utilized Spare-Parts for	Costs of item	Supplier item and/or order no	Storage place
		m3	kg	in	out	No.						

RANGOON-BASSEIN ROAD PROJECT

MAIN STORE

OIL BALANCE SHEET BALANCE REPORT 28.6.88

Date	Particular	Received	Issued	Balanced	Indent	Take-out	Remark
		Lt	Lt	Lt		name	
Jan. 88	BU-870825	800					
	Used until 28.6.88		583				
28.6.88	Engine oil SAE 15W-40						
	BP Vanellus C3			217			
Jan. 88	BU-870825	600					
	Used until 23.6.88		380				
28.6.88	Hydraulic oil						
	BP Bartran HV-46			220			
Jan. 88	Remainder of BU-860827	45					
	Used up until 19.3.88		45				
28.6.88	Gearbox oil						
	Shell Spirax SAE 90			0			
Jan. 88	Remainder of BU-860827	108					
	Used until 13.6.88		100				
28.6.88	Converter oil						
	Shell Donax TM			8			
Jan. 88	BU-870825	100 Kg					
	Used until 27.6.88		87 Kg				
28.6.88	Multipurpose Grease						
	BP Energrease LS-EP2			13 Kg			

Balances according detailed balance sheets.
Report made and confirmed. 28.6.88

~~Store Mech Eng.~~ ~~Mech Eng~~
C.C
U SAN OO

Fig. 15.13: Oil Balance

16 Bridge Maintenance

16.1 General

The maintenance of bridges is an integral part of ensuring continued serviceability, reliability and safety of the whole highway system. During the early years of the lifetime of a bridge maintenance will be very limited but, as the bridge ages, compromises between convenience, safety and economy will have to be made.

The basis for all decisions regarding the desirability for maintenance is the availability of realistic information. This requires the establishment and operation of a system ensuring regular effective inspection and evaluation of the condition of bridges. This must start with regular physical inspection by personnel who are both motivated and adequately trained and follow through to a sufficient data recording system which ensures that records are kept for each bridge and, chronologically record all inspections and works carried out during the life of the structure. Only then it is possible for economical and technically efficient measures to be considered.

16.2 Economic Background

In any realistic maintenance system, resources are finite, and not all desirable works can be immediately authorized. The making of decisions as to where to allocate available resources will, despite the appearance of being technical, be an economic one, either calculated or, more commonly, intuitive. For most situations the crucial questions are what delay is possible and what are the likely consequences of *extra costs* incurred by such delaying, e.g. postponing repair due to collision damage possibly requiring the replacement of an entire bridge element.

Whilst the problems arising as a bridge ages are likely to be apparent to the inspecting engineer, there is also a need to fully understand the benefits in delay. The comparison between expenditures for two possible maintenance schedules include - what is the real cost of delay, i.e. the forseeable cost after postponement plus the additional "real" cost of spending the money now rather than later with this "money cost", in reality from 3% upwards, independent of inflation, with 10% often used for economic calculations. In order to compare events, some of which are possible at a time in the future, it is common to calculate the Net Present Value of the cost. Costs for spending at different times in the future converted to NPV are then able to be compared. It is clear that even a delay of one year makes the cost of a repair effectively 10% cheaper and maintenance funding for different works at different times may be compared on a similar basis.

16.3 Bridge Inspection

16.3.1 Type of Inspection

Four types of bridge inspection may be differentiated:

- routine:	annual inpections for general condition and defects
- major:	regular inspections at intervals depending upon the type of bridge. Steel truss bridges should have a major inspection at two year intervals.
- special:	inspection related to specific problem, e.g. scouring at an abutment
- informal:	reports received from staff, 'in passing'.

This section will deal only with routine and major inspections.

16.3.2 Equipment

In order to properly examine the bridge for deterioration and/or damage, the inspector requires a selection of normal plus some special equipment. It is clear that he requires a measuring tape, string line, torch, waterproof marker, a wire brush and a large knife for vegetation clearing. The following equipment should also be available:

- extension ladder (extending to 7 m), plus planks etc. for access top chord level,
- binoculars,
- paint film gauge,
- knife, chisels and hammers and scrapers for checking paint,
- torque wrench to check bolt tightness.

The inspector should also have a standard bridge inspection form to ensure that the information is collected methodically and in a format useful to Public Works.

16.3.3 Piers and Abutments

The engineer should methodically look for:

- cracks and deterioration of concrete (for example spalling or disintegration)
- exposed reinforcement steel or signs of rust
- cracks in masonry joints, cavities in stonework and deterioration of the stone
- check of verticality where tilt or movement is suspected
- check of steel and concrete piles in the zone of water movement and,as far as it is practical, below the water surface for cracks, corrosion and deterioration.
- check that weepholes aren't blocked by foreign matter and for water leaking at other locations

16.3.4 Steel Beams and Girders

Examination all members looking for:

- cracking, particularly of welds
- corrosion (especially along top flanges and at the supported ends)
- damage or misalignment of flanges
- unusual vibration or excessive deflections under passage of heavy loads
- check for accumulation of dirt or debris, particularly on webs of chord members
- clean out any such debris during the inspection if possible. Drain holes in chord members should be cleared
- check that all holes have bolts, and check for looseness of nuts by tapping with a hammer
- check for bolt tightness on samples:
    1) mark location of nut
    2) loosen the bolt by 1/6 turn (60 degrees) with torque wrench (approx. 75% of the required torque should be enough)
    3) retighten bolt to original mark with required torque of 750 Nm for M24-bolts
- steelwork should be inspected for damage due to vehicle impact
- inspect protection system for corrosion, and determine the cause for deterioration.

Every connection and member of the bridge should be checked from above and, although often difficult, from below. Pay particular attention to the following possible problem areas:

- edges of surfaces in contact with concrete
- horizontal surfaces where moisture may be held
- welds, nuts, bolts, washers.

16.3.5 Bearings

Inspection should check and record:

- loose anchor bolts
- accumulation of dirt and debris
- proper seating, pads must be seated evenly over complete area
- cracks (especially edge cracks), spalls, deterioration at bearing support
- bearings to be checked for any distortion or excessive horizontal movement exceeding 30mm.

16.3.6 Expansion Joints

The inspector must look for:

- sound joint seals. Seals are intended to keep out sand, dirt, stones and must be immediately cleaned and then repaired if damaged
- adequate space for movement in expansion joints
- signs of distress in structural members at joints
- damage to concrete or asphalt surface and any sign of looseness of deck protection angles.

16.3.7 Asphalt Road Surfacing

Inspection should record the extent and position of:

- pot-holes, cracks and distortions
- weathering and breaking up
- functioning of drainage, surface slopes, gutters and drain pipes.

16.3.8 Signs and Reflectors

Checks should include all bridge signs, including advance warning signs (narrow bridge, vehicle weight limit, speed limit) and record that all is:

- in their proper location
- in good condition
- that all lettering is clear and legible.

Reflectors on end parapets or posts are especially important at night to highlight bridge structure limits. They must be visible to approaching traffic and should be checked for theft, damage and kept clean.

16.3.9 Concrete Walkways

Examination should include checks for:

- cracks , settlement, spalling
- scaling at surface, edges and adjacent to support steelwork
- condition at expansion and panel joints
- any hazards for pedestrians.

16.3.10 Bridge Railings

Railings must be checked for:

- insecure, loose sections
- condition of connectors
- damage, especially from traffic
- corrosion, especially at joints and around the steel posts
- loose fixing bolts and nuts.

16.3.11 Waterway

Examination of the waterway requires a degree of judgement to determine what are changes which may be either simple temporary consequences of recent rains and what are small changes which could be expensive to amend if not understood, e.g. channel changes. The inspector should, in particular, look for:

- erosion of stream banks and bed of waterway
- erosion or scour around abutments, wingwalls and piers
- scour adjacent to protective apron, spurs etc.
- formation of sand and mud banks but also record or arrange to have cleared
- accumulation of debris and junk obstructing the channel
- vegetation, obstructing the free flow of flood water.

16.3.12 Approaches

Bridge approaches must be inspected for:

- unevenness of pavement surface which can cause excess vehicle impact on bridge
- differential settlement, especially at joints between approach pavement and bridge
- poor drainage or erosion at abutments and wing walls
- shoulder wear, erosion, settlement
- slope erosion, slips
- damaged or missing guardrails
- vegetation (reduced sight distance).

16.4 Categories of Maintenance

The maintenance work which follows such a programme of inspection may be of different frequencies:

Routine works involve operations such as cleaning drains, patching of the wearing surface and minor repairs to protective gabions. Such works should be part of a continuous process involving the entire road length and do not require specialized bridge knowledge to carry out.

Periodic works will include regular examination and for example, would involve thorough checking of bolts and galvanizing, and at longer intervals the planned replacement of the bitumen wearing layer.

Extraordinary, or emergency maintenance is clearly the most problematic as accident damage or monsoon necessitated river training are impossible to predict, yet must be quickly responded to.

16.5 Repairs

16.5.1 Steelwork Repairs

Minor distortions in steel plates may be straightened by use of jacks and strongbacks assisted by hammering. Major components which are damaged in service, e.g. by collision, should be repaired, strengthened or replaced with the specific direction of the responsible bridge engineer.

Heat must not be applied to any members under load, i.e. parts of a finished truss, except under specific direction of the engineer. If it is found necessary to replace a truss member or apply heat to repair a damaged truss member while it is under load, some form of additional support will be required. Railings and bracings may be removed and repaired by heating provided proper restoration of anti-corrosive coatings is made, see below.

All steelwork is hot-dip galvanised at the time of fabrication. This coating will provide long-term protection without any additional protective coating. Where the zinc coating is scratched or penetrated it maintains cathodic protection to the underlying steel and need not be treated. Where a larger area of the coating is removed, e.g. during repair or by damage, the affected area should be treated as follows:

i) Clean back to bright metal using scrapers and power grinder.
ii) Wipe the surface clean and dry.
iii) Apply 2 coats of approved inorganic zinc silicate paint by brush or spray according to the manufacturer's directions. Finish coating thickness 80 micron.

If, after some years, it becomes evident that a topcoat system is required, the following procedure must be applied:

- Remove any oil or grease or mould growth using suitable solvent or detergent. Sand and / or brush the entire surface thoroughly to remove all dirt and oxidation "bloom" on the surface, but leaving the zinc coating intact.
- Repair any damaged or rusted areas, as above for repairs, by applying anorganic zinc priming paint.
- Finally wash the surface thoroughly with clean water. This should be done in sections ahead of the painting. Allow time for drying before painting.
- Paint over the zinc coating with a universal metal primer, suitable for use over a galvanised surface.
- Apply an approved micaceous iron oxide chlorinated rubber paint, mixed and applied according to the manufacturer's directions.
- This paint coating should be worked thoroughly into edges of plate interfaces and around nuts and bolts. The paint should be applied by spray (preferably) or brush in a single coating of even thickness, avoiding runs or blistering.

16.5.2 Bearing Repair and Replacement

The pot bearings provided are virtually free of maintenance and have an indefinite life in normal conditions. Any problem, see checks above, is more likely to be in the surrounding concrete. Bearings exhibiting movement exceeding 30 mm should be recentred. Bearings which have become damaged or otherwise deteriorated should be removed and replaced.

In such an event, the end of the span is to be jacked up evenly, such that the affected bearing and the other bearing at the same end of the span are just cleared. The faulty bearing may then be moved and the procedures of 13.4.2 followed in respect to the installation of the replacement bearing. Note: The bearing specifications are indicated in the drawings to permit the required replacement bearing to be ordered. Jacking loads for a completed span are indicated below.

Mass of span incl. concrete carriageway	(tonnes)
Bridges with walkway (e.g.Kyoung-gone)	260
Other bridges without walkway	222

Table 16.1

If a completed span is to be raised, jacks are to be located adjacent to the bearings under the end cross-girder (Fig. 16.1). The jacking forces can be derived by dividing the mass by 4.

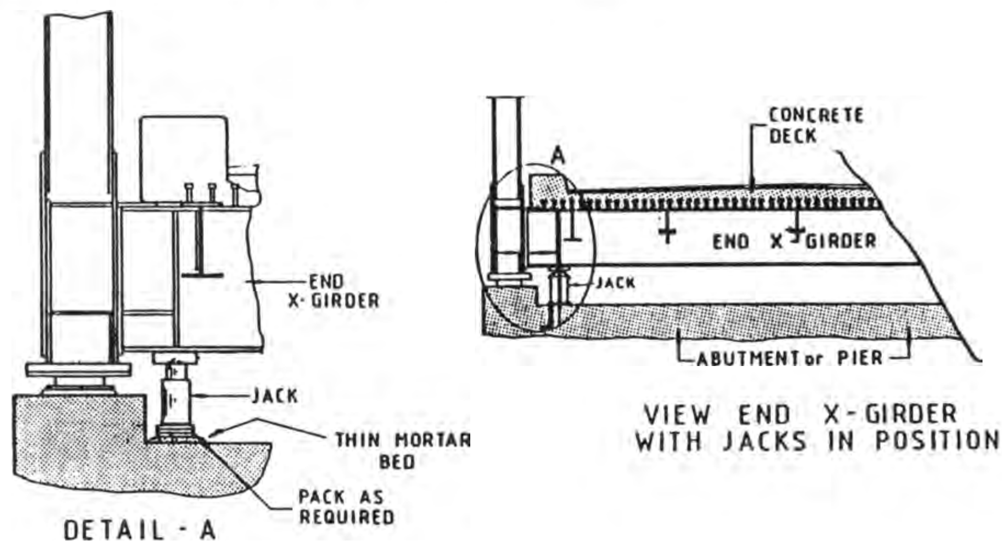


Fig. 16.1

16.5.3 Repairs to Expansion Joints

Joints between spans are sealed with elastomeric deck seals (Section 14.8). Provided they are properly fitted their life should be indefinite. Periodic checks ensure that the seals are not damaged; the most likely causes are intrusive materials from above and any displacement which each require speedy correction.

Care is required during repair or maintenance of the wearing course in order not to cover over the joint or to force down the deck seal with stones.

Should a span have to be lifted, e.g. in the unlikely event of bearing renewal, then renewal of the seal should also be anticipated. In such cases, the gap should not be left without a seal.

16.5.4 Repair of Concrete Works

Concrete repairs are common but, traditionally have been carried out using cement mortar, generally with poor results. In order to be effective, surface repairs, in particular, require either materials which penetrate into the concrete or adhere strongly to it as well as materials with resistance to climatic influence and to ageing.

Current practice indicates that, for permanent repair, several steps are required. Initially, remove of all deficient or damaged concrete down to the level of sound material, together with any oil or other external material. Then, remove loose rust from any exposed reinforcing bars. For reshaping the surface, a suitable resin based primer is required to get an acceptable adhesion. Finally, for durable repair, a high quality cement mortar with very low water/cement ratio is needed; to increase the workability a liquifier should be added. In special cases an epoxy plasticiser is recommended. Conventional sand/cement mortars, the most commonly used materials in the past, have a short life and, by retaining infiltrating water, may encourage further deterioration of the reinforcement.

16.5.5 Asphalt Repair

The most likely form of damages to the asphalt wearing course are slippage cracks. These are sometimes crescent-shaped cracks that point in the direction of the thrust of wheels on the pavement surface. For example, a braking vehicle approaching the bridge end, the thrust of the wheels is reversed and slippage in this circumstance will result in cracks pointing contra the flow.

The only proper way to repair a slippage crack is to remove the surface layer from around the crack to the point where good bond between the layers is found. Then the area is patched with plant-mixed asphalt material. The procedure is the following:

a) Removal of slipping areas and at least 0.3 m (one foot) into the surrounding well bonded pavement. Straight and vertical cutting, probably by a power pavement saw.

b) Cleaning of the surface of the exposed underlying layer with brooms and compressed air.

c) Application of a light tack coat.

d) Placing of enough hot plant-mixed asphalt material in the cut-out area to bring the surface to the same grade as the surrounding pavement when it is compacted.

e) Careful levelling of the mixture to prevent segregation.

f) Check of the riding quality of the patch with a straightedge or a stringline.

g) Compaction with a vibrating plate compactor or a steel-wheeled roller.

16.6 River Bank Protection

A common problem immediately after construction is bank and bed erosion , caused by the construction and the possibility of initiating a change of the channel course. With slow flowing rivers and deep piles significant bed erosion is unlikely, but eroded banks that threaten abutments or permit a change of the river course must be restored.

Various measures may be attempted to stop or slow down the erosion. For flexibility and economy, gabions and mattress works may be expected to be the best solution. Local settlement and underscouring cause excessive damage to most surface protection measures in which, despite having limited flexibility, gaps appear and great damage results. Stone filled gabions are quickly laid and durable; they are easily adaptable to existing groundforms. Once a problem arises, they are highly flexible and therefore much more accommodating to settlements, which are inevitable during the early years after construction, than other, initially good looking but less flexible solutions.

Indicative Thicknesses of Mattress and Gabion Revetments (from Maccaferri, 1985):

Type	Thickness (m)	Rock Fill size (mm)	Critical velocity (m/s)	Limit velocity (m/s)
Mattress	0.15 - 0.17	70 - 100 70 - 150	3.5 4.2	4.2 4.5
Mattress	0.23 - 0.25	70 - 100 70 - 150	3.6 4.5	5.5 6.1
Mattress	0.30	70 - 120 100 - 150	4.2 5.0	5.5 6.4
Gabions	0.50	100 - 200 120 - 250	5.8 6.4	7.6 8.0

Table 16.2

Where the revetment has to be placed under water, the thickness of a mattress remains the same since it can be launched from a pontoon which permits emergency repair works during flood periods and continuity of work through the monsoon season. Mattresses are widely used on sloping surfaces.

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