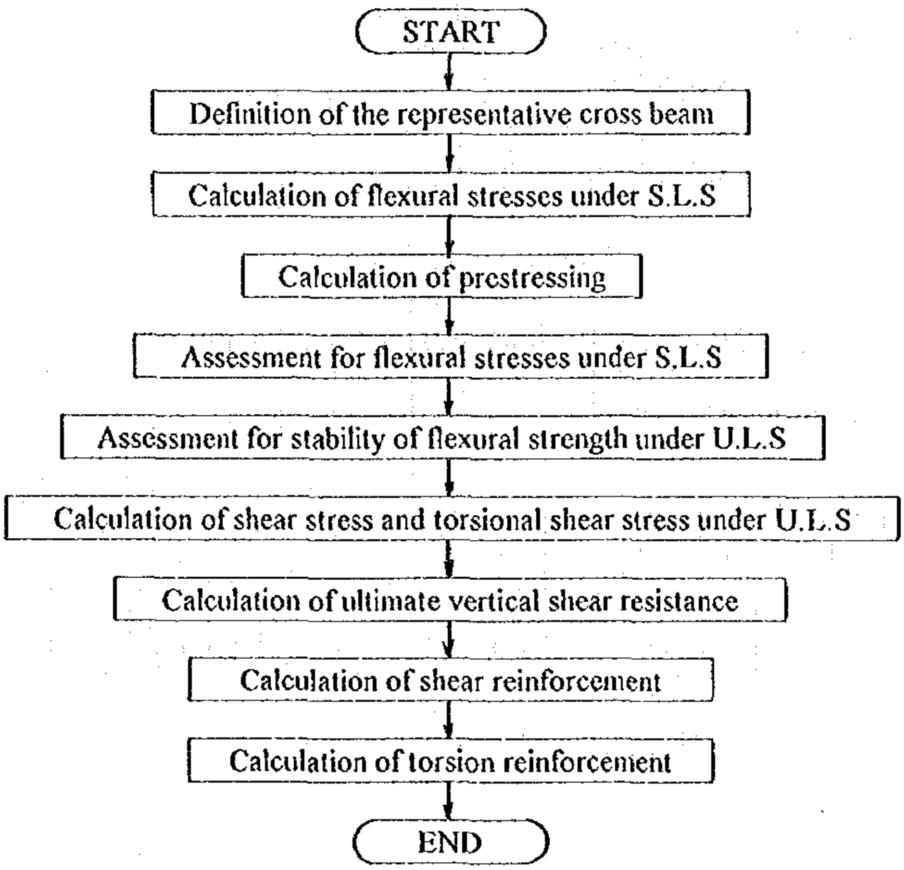
2.5 Design of Cross Beam

This section describes procedure of the design for cross beam which is shown in Fig. 2.2 "Flowchart for Superstructure Design Programme". The display's number for input data of design programme is shown in the title of each section as references. *The display is attached in "DIVISION IV."*

2.5.1 Design Flowchart

Design of cross beam shall be carried out with nearly the same flowchart as in the design of main beam.



2.5.2 Input Data [F.1 to F.4]

The input data shall be prepared the arrangement of reinforcement and prestressing tendon and relative prestressing data, such as effective prestress, nominal area and diameter of sheaths.

2.5.3 Calculation of Prestress

It is just a intricate problem to conduct a calculation of the losses of prestressing force of cross beam according to each discrete stage of erection, it is, therefore, decided to follow the practice in Japan as below.

That is, a simplified method was adopted by setting that:

- initial prestressing force is 0.7 x characteristic strength of PC tendon,
- effective factor for prestressing force is 0.8,
- then, effective force is (0.7 x 0.8=)0.56 x characteristic strength

2.5.4 Assessment of Flexural Stresses under Serviceability Limit State

This Clause is the same assessment as in the main beam design in clause 2.4.5 described before.

In the design for flexural moment of composite T-beam, the design for the positive flexural moment shall be carried out as prestressed concrete member reflecting the each discrete stage of erection, on the other hand the design for the negative flexural moment shall be carried out as reinforced concrete member to the composite T-section.

2.5.5 Assessment of Stability under Ultimate Limit State

This Clause is the same assessment as in the main beam design in clause 2.4.6 described before.

2.6 Design of Slab

2.6.1 General

Design of slab shall not be incorporated in the computerized design system in this Study, and standard slab thickness shall be determined by the Finite Element Method. With thorough consideration on safety, since slab supports directly the live load which are repeated loadings, and the calculation sheets shall be prepared separately to assure the safety of slab.

Accordingly, this section describes the cases of examination, the division of elements and the loads to be considered.

2.6.2 Examination Cases

	Slab Thickness (cm)	Space of Main Beam (m)	Straight or Skew	Objective Structure Type
Case 1	18	2.1	Straight	Post-tensioned T-beam
Case 2	20	2.1		
Case 3	22	2.1		
Case 4	20	2.1	Skew = 30 deg.	
Case 5	20	1.3	Straight	Pre-tensioned T-beam

2.6.3 Division of Elements

The division of Elements is shown in Fig. 2.12and Fig. 2.14 . The analysis of deck slab shall be aimed at the near support of beam and the middle, accordingly the elements of their part are divided finely.

2.6.4 Loads to be Considered

The loads shall be considered as below.

(1) Slab self-weight
(2) Premix load
(3) Parapet load
(4) Live load

As the live load, nominal HB wheel loads shall be considered to be uniformly distributed over a square contact area, assuming an effective pressure 1.1N/mm2. Loading cases of live load shall be set four cases aiming at the cantilever slab and the middle slab in the transversal direction as shown in Fig. 2.15 to Fig. 2.18 .

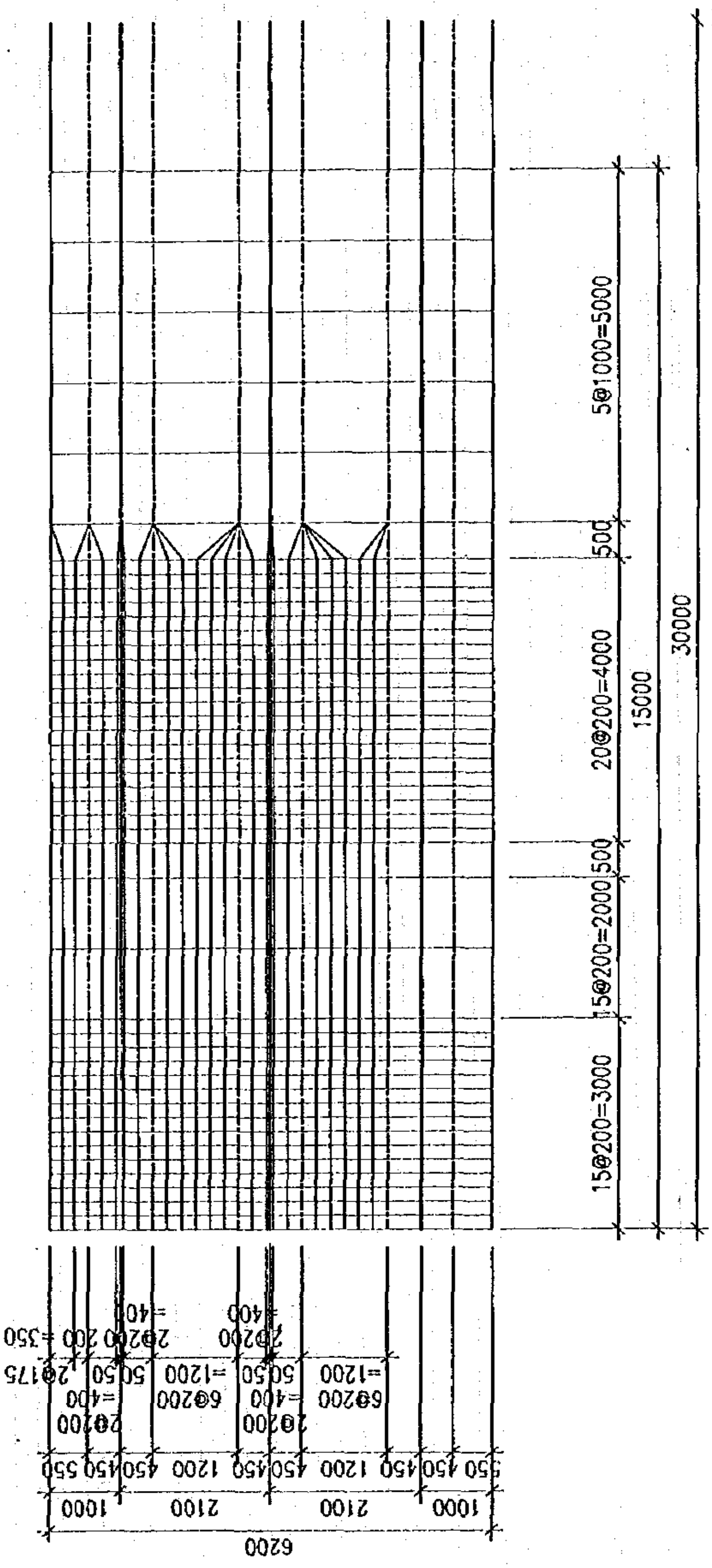


Fig. 2.12 Element of Slab (1)

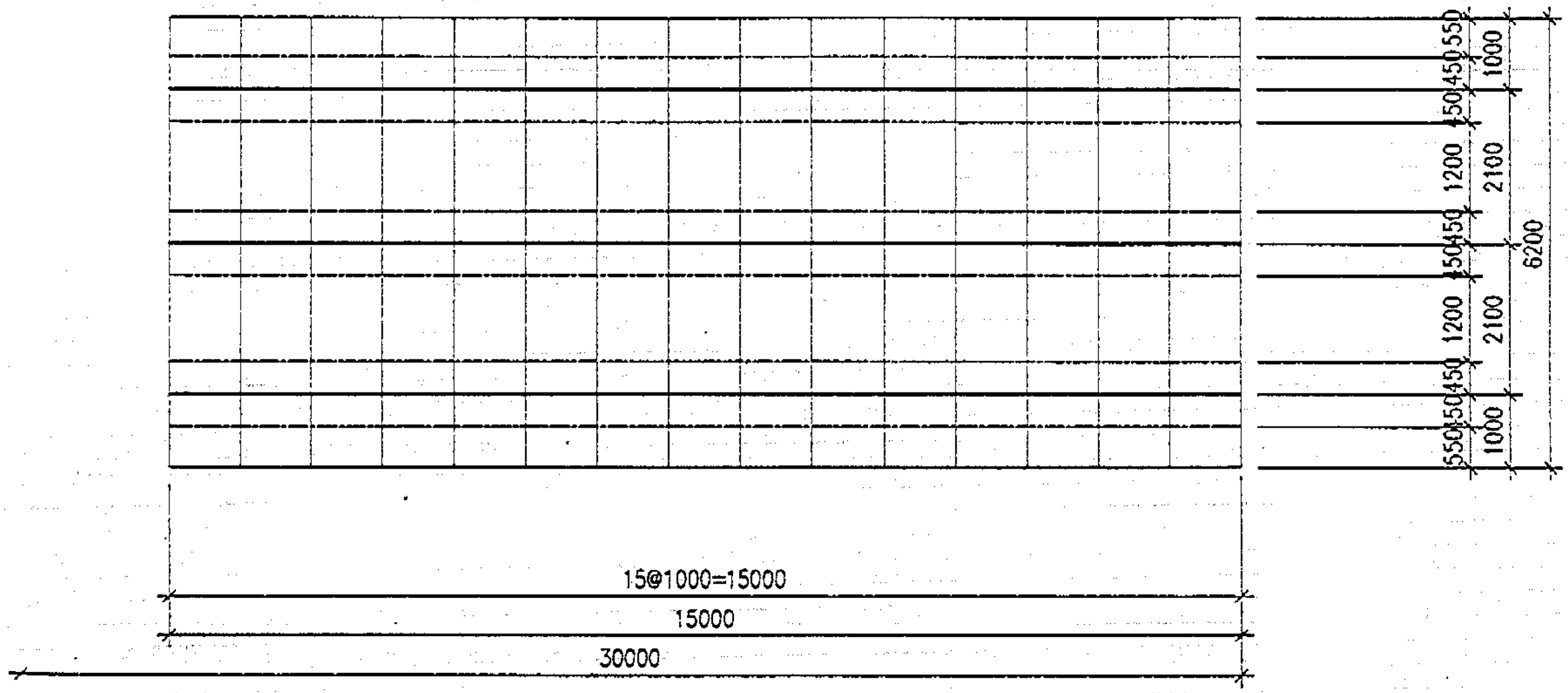


Fig. 2.13 Element of Slab (2)

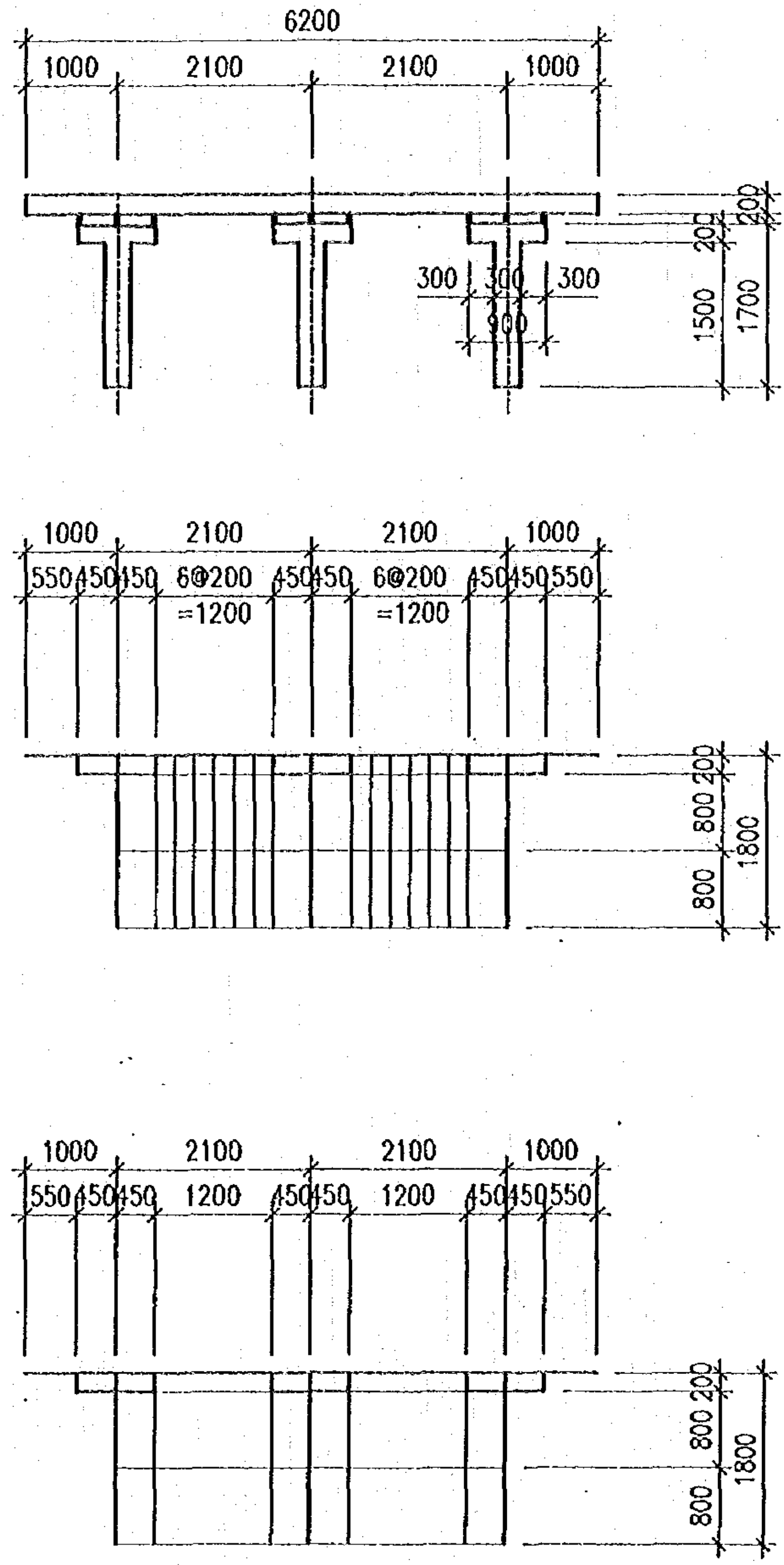


Fig. 2.14 Element of Beam

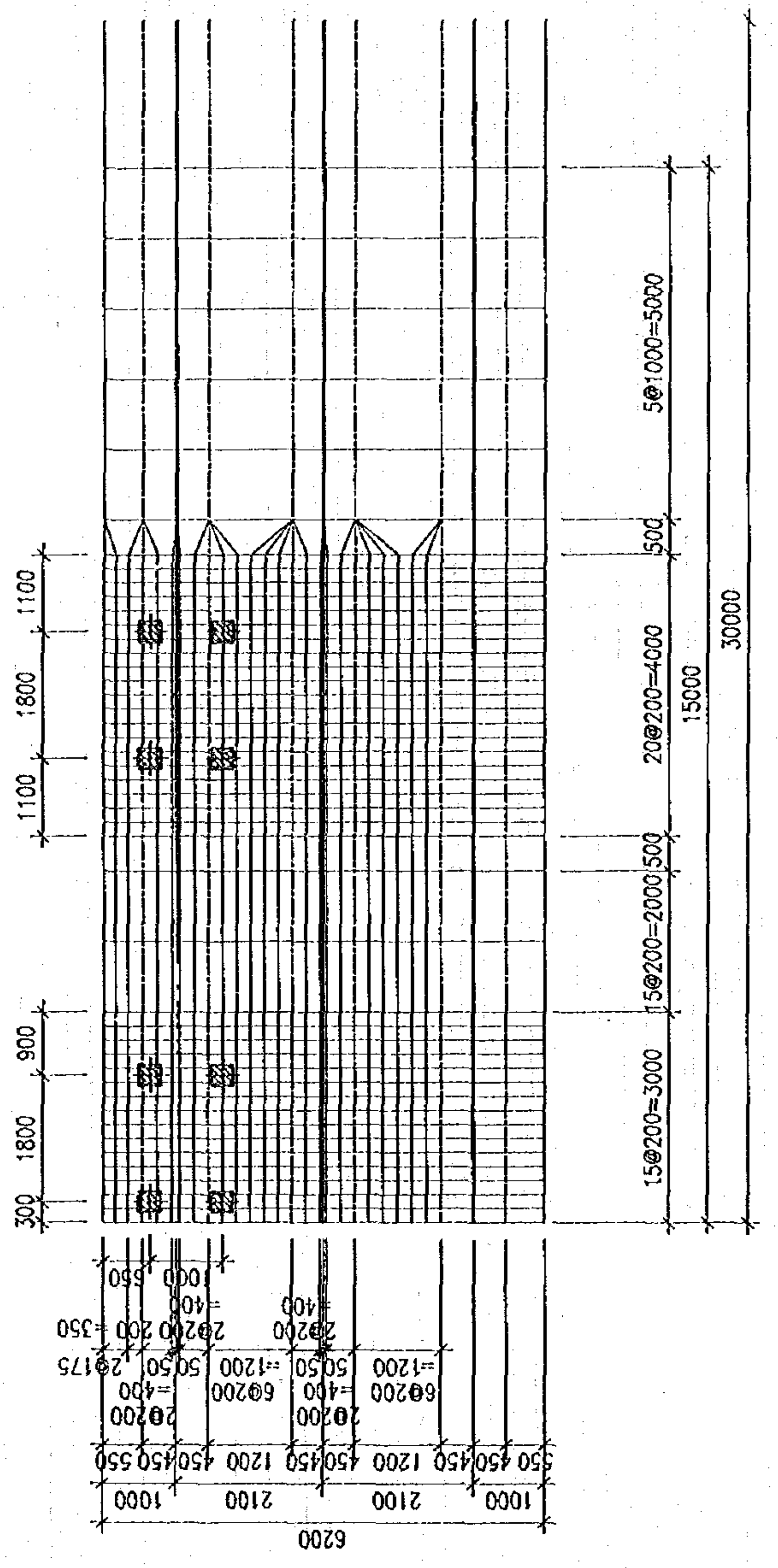


Fig. 2.15 Live Load Case (1)

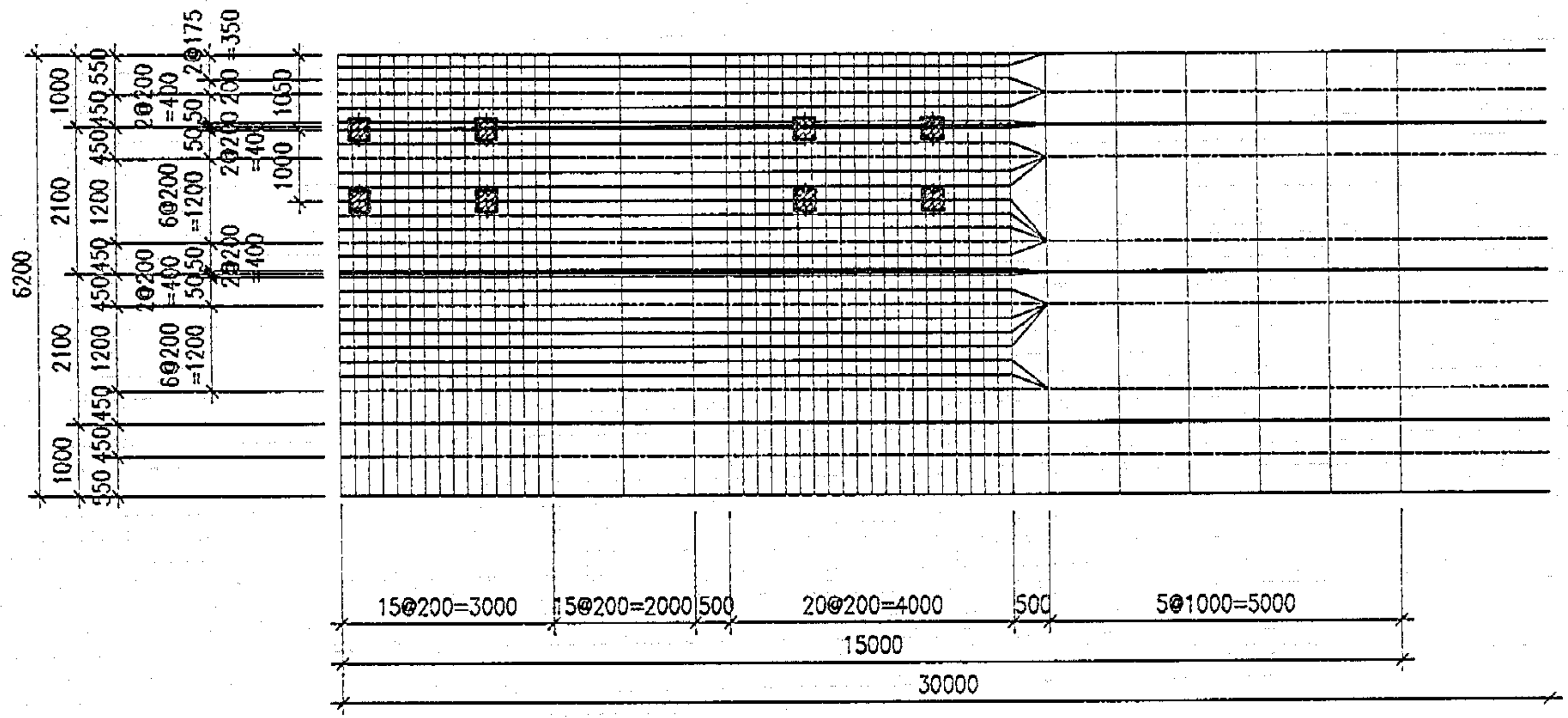


Fig. 2.16 Live Load Case (2)

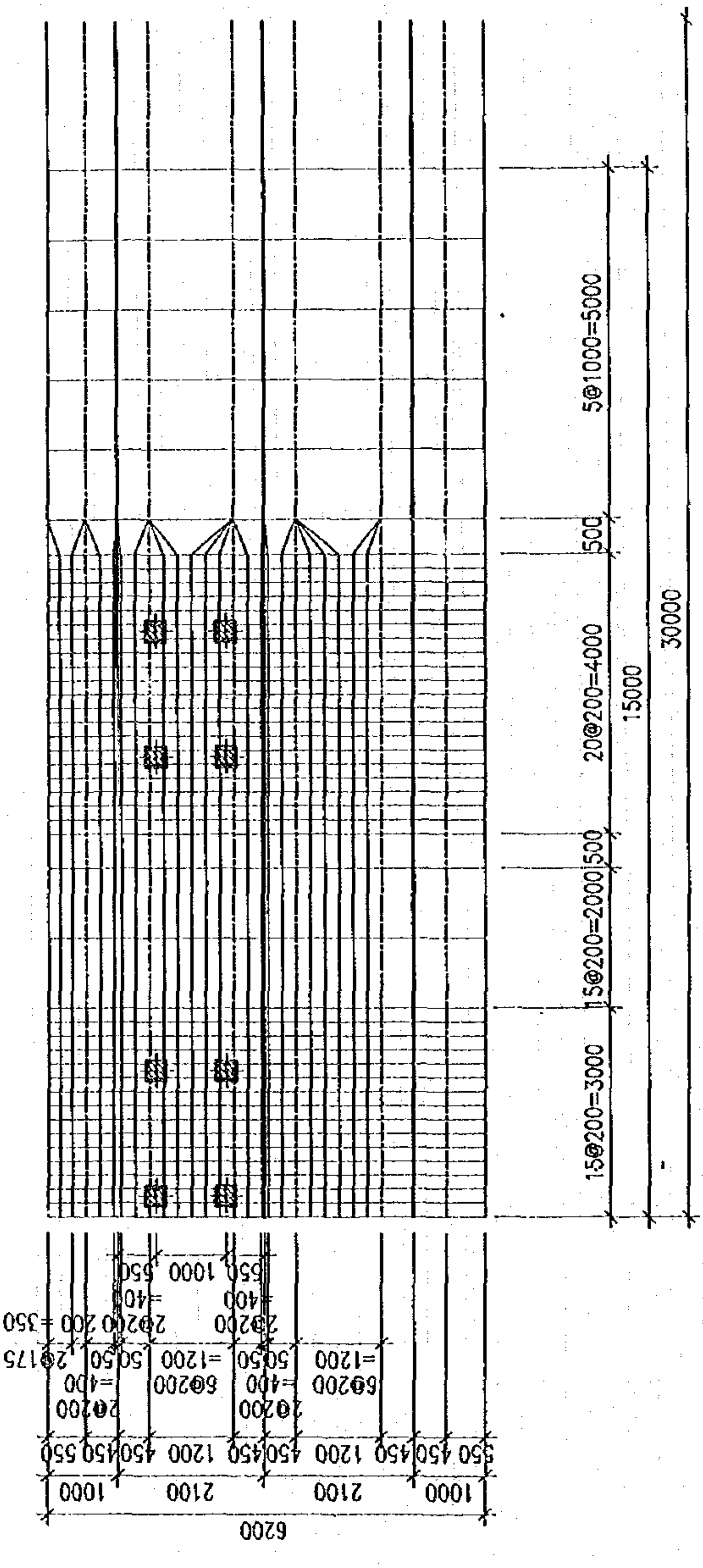


Fig. 2.17 Live Load Case (3)

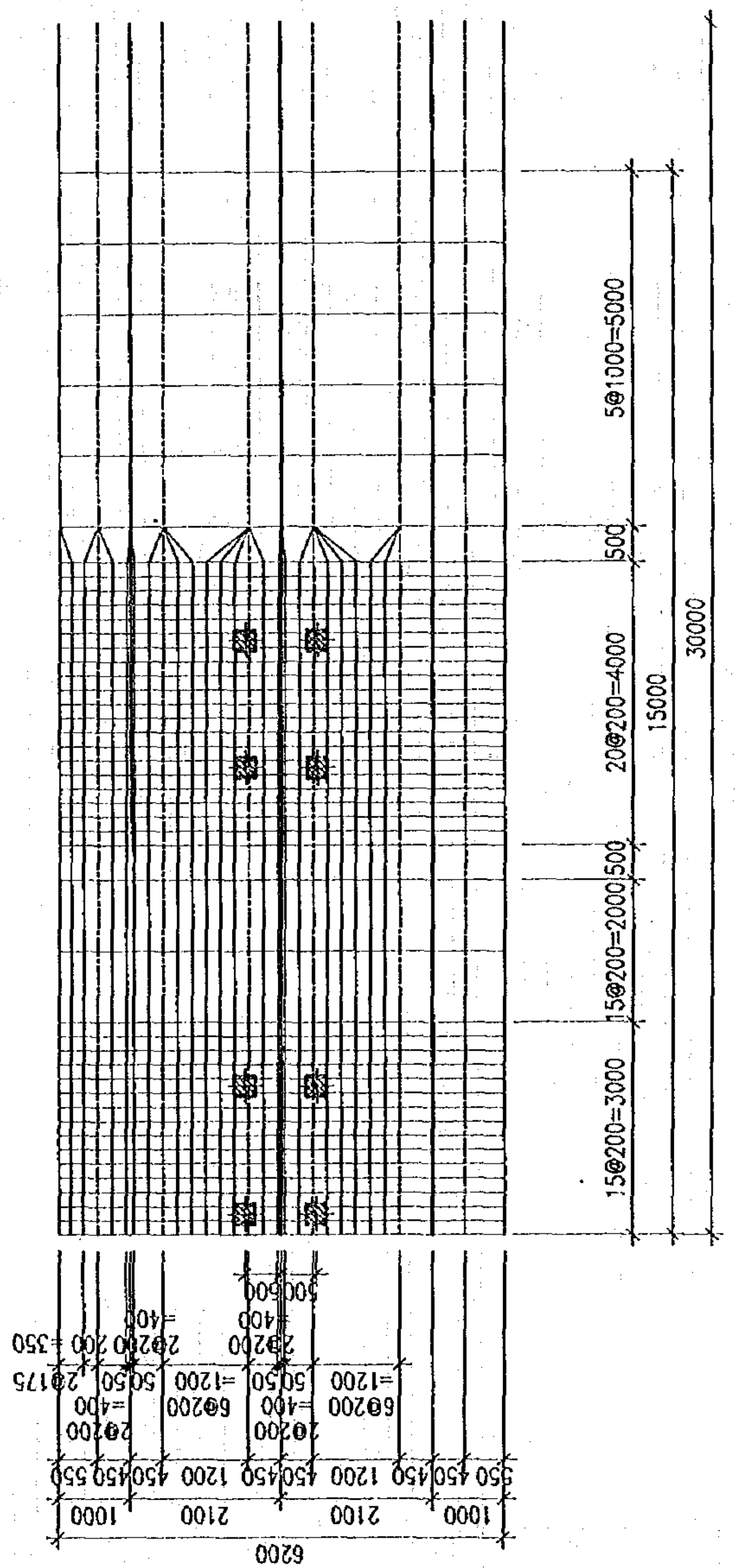


Fig. 2.18 Live Load Case (4)

2.7 Design of Bridge Accessories

2.7.1 Bridge Bearing

Laminated rubber bearing pad and strip rubber bearing using natural rubber shall be adopted for bridge bearings.

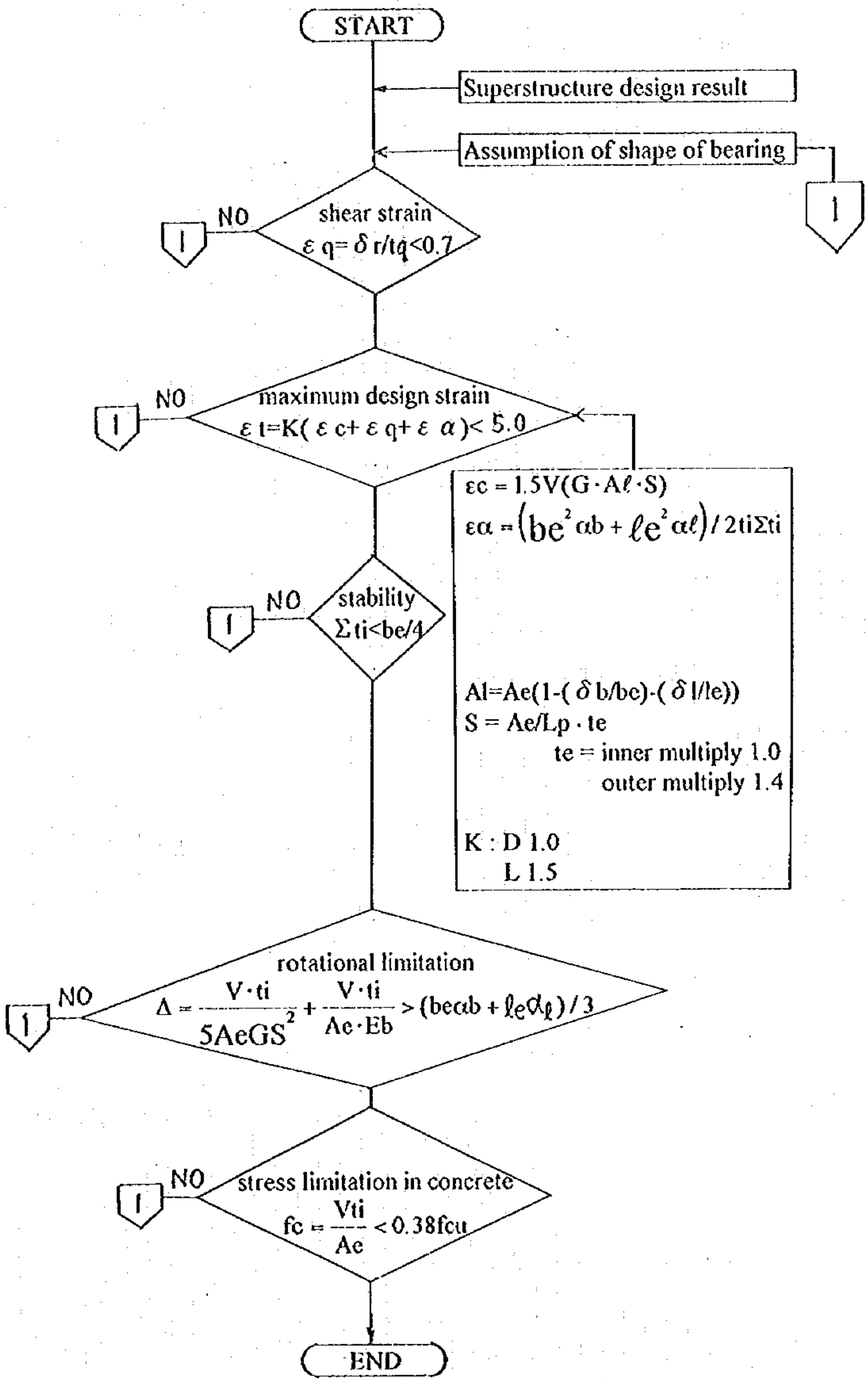
The design of laminated rubber bearing shall be carried out in accordance with the Clause 10, Part 9, BS 5400 as the following flowchart.

2.7.2 Expansion Joint

Expansion joints that are available locally are shown in the standard drawings as reference, in consideration to their structural view point, performance, cost and maintenance. A particular expansion joint will be selected from the horizontal movement depending on span length and the environmental condition of construction site.

2.7.3 Parapet

The New Jersey parapet, aluminum parapet and steel railing that are available locally are shown in the standard drawings as reference, in consideration to their performance, cost and maintenance.


START
Superstructure design result
Assumption of shape of bearing
1
NO
shear strain
ε q= δ r/tq<0.7
NO
maximum design strain
ε t=K(ε c+ ε q+ ε α)< 5.0
εc = 1.5V(G·Aℓ·S)
εα = (be² αb + ℓe² αℓ) / 2tiΣti
Al=Ae(1-(δ b/be)-(δ l/le))
S = Ae/Lp · te
te = inner multiply 1.0
outer multiply 1.4
K : D 1.0
L 1.5
NO
stability
Σti<be/4
NO
rotational limitation
Δ = V·ti / 5AeGS² + V·ti / Ae·Eb > (beαb + ℓeαℓ) / 3
NO
stress limitation in concrete
fc = Vti / Ae < 0.38fcu
END

CHAPTER 3 Analysis for Substructure

3.1 Explanation of Notations

The notations used are basically the same with BS 5400, so only important and new ones used in this Manual and the calculation sheets are shown in the following.

(1) Notations for Load

D	:	Superstructure dead load
SD1	:	Superimposed dead load except premix dead load
SD2	:	Premix dead load
HA	:	Normal live load
HB	:	Abnormal live load 30 units
HB*	:	Abnormal live load 45 units
HA*	:	Only 1 lane normal live load
DS	:	Substructure dead load
EPV	:	Earth pressure vertical load
EPH	:	Earth pressure non-vertical load
WL	:	Wind load
TC	:	Temperature change
SK	:	Accidental skidding load
LONG	:	Longitudinal force
COLL	:	Vehicle collision load
BUOY	:	Buoyancy load

(2) Notations for Sectional Force

M	:	Bending moment
Mg	:	Bending moment due to permanent load
Mq	:	Bending moment due to live load
Mu	:	Ultimate resistance moment
N	:	Axial force
S, V	:	Shear force

(3) Notations for Dimension

av	:	Distance between the line of action or point of application of the load and the critical section of supporting member
b, ba	:	Width or breadth of section
h	:	Overall depth of section
L	:	Span length

(4) Notations for Reinforcement

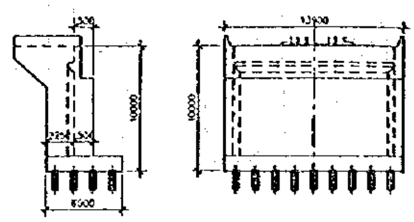
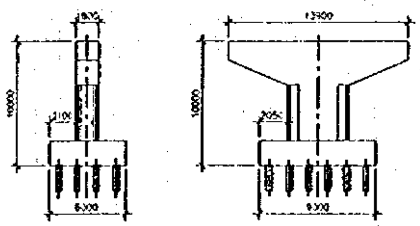
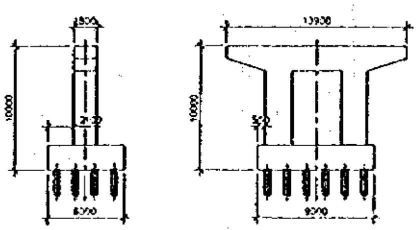
As	:	Area of reinforcement
C	:	Cover

d	:	Effective depth to tension reinforcement
do	:	Effective depth to tension reinforcement at supported point
fy, fyv	:	Characteristic strength of reinforcement
ϕ	:	Size of reinforcement

3.2 Design Conditions

3.2.1 Common Conditions

3.2.1.1 Structure Types

Type	Cross Section
Abutment	
T-type Pier	
Multiple Column Pier	

3.2.1.2 Soil Conditions

(1) Assumed Soil

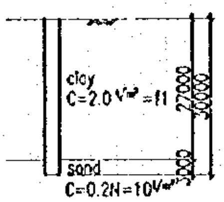
Before foundations are designed and a method of construction is determined, it is essential to carry out a site exploration to ascertain the character and variability of the strata underlying the site of the proposed structure. In particular, it is necessary to assess those properties which may affect the performance of the structure and the choice of method of construction.

This Study is for standard design, and a specific construction site has not been designated in the design stage. The standard design for foundation shall be applied mainly to the bridge construction on the coastal area in Malaysia.

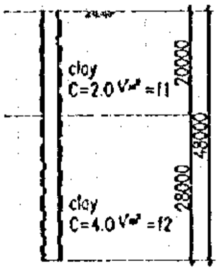
The soil on coastal area is composed of river alluvium as shown in Fig. 2.19, and it is very soft.

Based on the collected data from the actual borings, the hearing information from local consultants and discussions with JKR, the typical soil conditions were decided as follows for the 2 types of the pile foundation design.

- Model 1: This is for design of PC bearing piles used in the alluvium weak stratum, depositing 20 m or more.



- Model 2: This is for design of friction piles used in the deeper bearing layer, which has similar soil layer as in the Model 1.



(2) Soil Constant

The soil constant which may not cause a big difference between the actual designing of substructure is determined according to the results of soil test as shown in Table 2. .

The cohesion is determined as the value of 2.0 t/m2 according to the result of the vane shear test at the site, and the results of the unconfined compression test in the laboratory.

The modulus of deformation is determined according to the unconfined compression test, because N value is O.

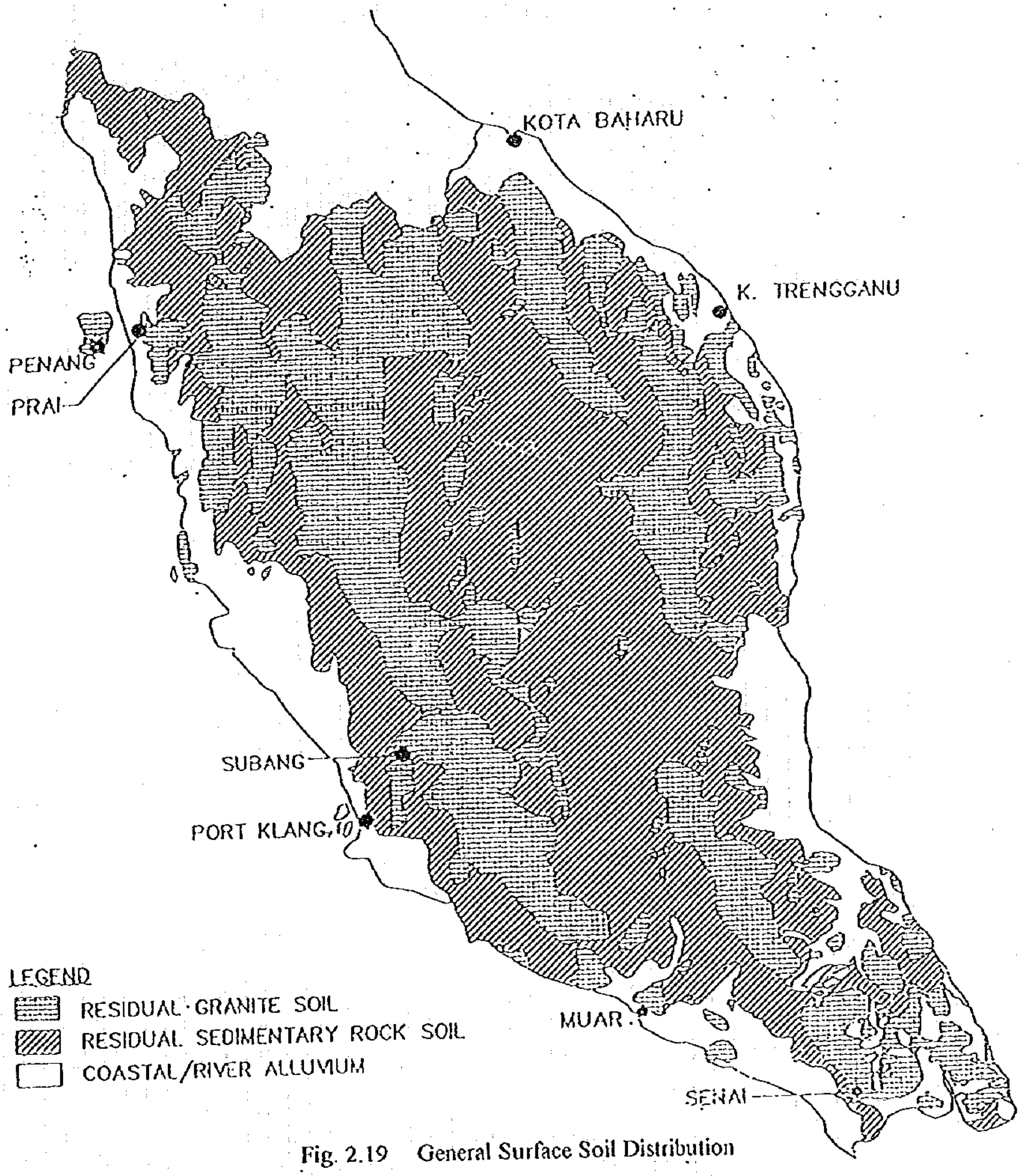


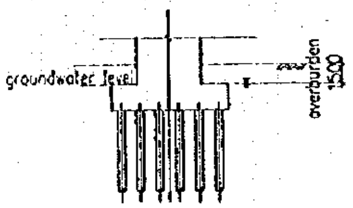
Fig. 2.19 General Surface Soil Distribution

Table 2.8 Soil Constants

Model	Soil	Depth (m)	Unit weight r (KN/m3)	N value	Modulus of deformation E0 (N/mm2)	Cohesion c (N/mm2)
Model 1	Alluvium clay	27.0	18.0	0	4.9	2.0
	Alluvium sand	3.0	19.0	50	98.0	10.0
Model 2	Alluvium clay	20.0	18.0	0	4.9	2.0
	Alluvium clay	28.0	18.0	0	9.8	4.0

(3) Foundation conditions

The groundwater level and depth of the overburden shall be adopted as below.



3.2.1.3 Other Conditions

(1) Environment Condition

In the discussion with JKR, the environment condition for cover and crack width of reinforced concrete member shall be adopted as below.

	Environmental Condition	Limitation
Cover	very severe environment	50mm
Crack width	severe environment	0.25mm

(2) Design Life

A design life of 120 years has been assumed throughout BS 5400 (unless otherwise stated)

3.2.2 Load Classification and Conditions

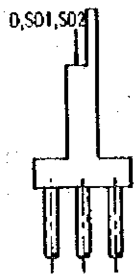
3.2.2.1 Loads to be Considered

D	:	Superstructure dead load
SD1	:	Superimposed dead load except premix dead load
SD2	:	Premix dead load
HA	:	Normal live load
HB	:	Abnormal live load 30 units
HB*	:	Abnormal live load 45 units
HA*	:	Only 1 lane normal live load
DS	:	Substructure dead load
EPV	:	Earth pressure vertical load
EPH	:	Earth pressure non-vertical load
WL	:	Wind load
TC	:	Temperature change
SK	:	Accidental skidding load
LONG	:	Longitudinal force
COLL	:	Vehicle collision load
BUOY	:	Buoyancy load

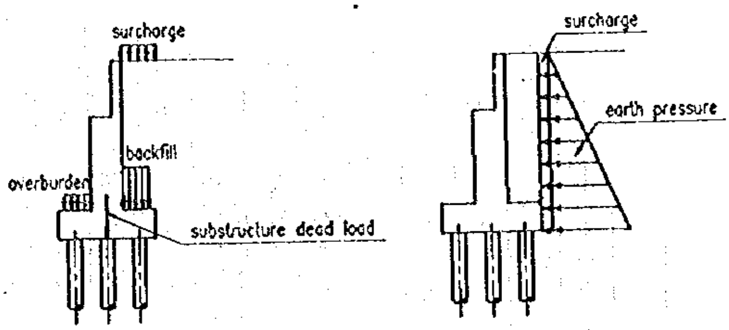
3.2.2.2 Loading Condition for Permanent Loads

(1) Abutment

(a) Loads due to Superstructure

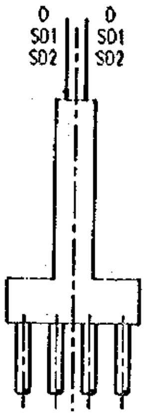


(b) Loads due to Substructure and Earth Pressure

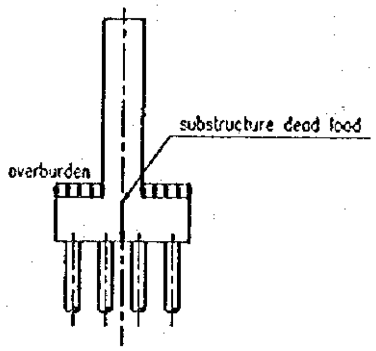


(2) Pier

(a) Loads due to Superstructure



(b) Loads due to Substructure

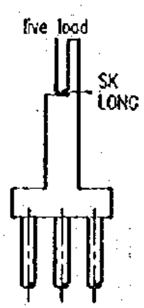


3.2.2.3 Loading Condition for Transient Loads

(1) Abutment

The live load, skidding load and longitudinal load shall be considered. But the wind or temperature load shall not be considered, because they are not critical factors in determination of the cross section.

The combination of the primary and secondary live load shall be determined as follows according to related regulations of the BD 37/88.



(a) Primary live load only
- The highest among (HA), (HA+HB), (HB*)

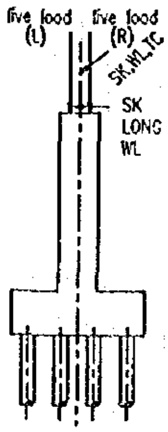
(b) Loading with the skidding load
- (HA)+(SK)

(c) Loading with the longitudinal load
- (HA or HB* whichever higher)+(LONG)

(2) Pier

All of the live, wind, temperature, skidding, longitudinal loads and collision load shall be considered, but the temperature load shall be considered only for the design of the transversal direction of the Multiple column pier.

Determination of the combination of the live load is the same way as for the abutment.



(a) Primary live load only
  - $(HA)_{(L)}+(HA+HB)_{(R)}$

(b) Loading with the wind load
  - $(HA)_{(L)}+(HA+HB)_{(R)}+WL$

(c) Loading only the wind load
  - (WL)

(d) Loading with the temperature load
  - $(HA)_{(L)}+(HA)_{(R)}+(TC)$

(e) Loading with the skidding load
  - $(HA)_{(L)}+(HA)_{(R)}+SK$

(f) Loading with the longitudinal load
  - $[\{(HA^*)_{(L)}+(HA^*)_{(R)}\}$ or (HB^*) whichever higher$]$ + $(LONG)$

(g) Collision load only
  - $(COLL)$

where, (L) : left side
(R) : right side

(3) Loading Method of Wind Load

According to the BD 37/88, the nominal transverse wind load Pt (in N) shall be derived from the following equation.

$Pt = q1 \times A1 \times C_D$
where, $q1 = 0.613 \times vc \times vc$
$vc = v \times K1 \times S1 \times S2$
A1:solid area
C_D: Drug coefficient

In this standard design, the various values for calculating the load Pt are determined as follows.

v: According to the Meteorological data (= 40 m/s).
vc is regulated to be 35 m/s when loading with the live load, and as the result, q is the constant value of 0.75.

S2, A1 and C_D should be determined by the projected height and projected length as shown in Fig.2.20, 21 and 22.

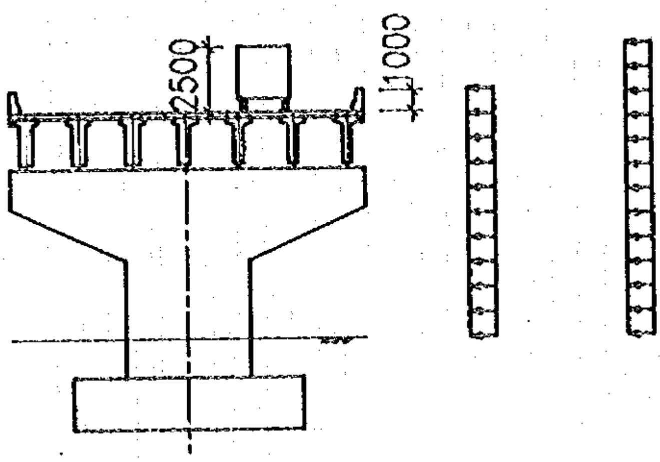


Fig.2.20 Projected Height

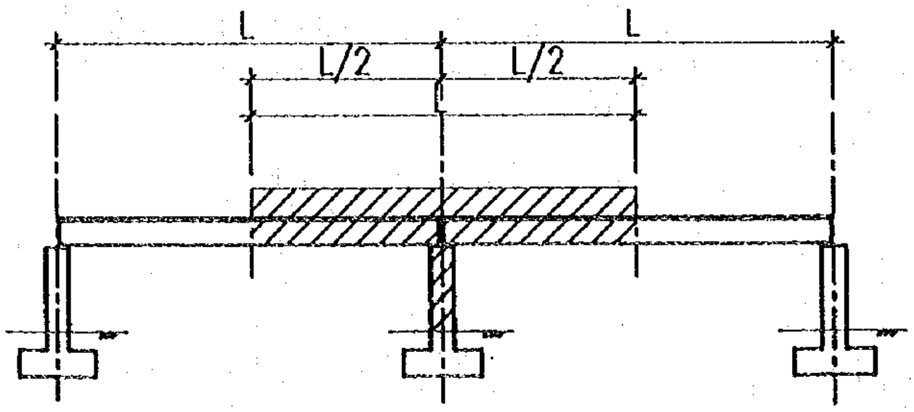


Fig.2.21 Projected Height and Area in the Longitudinal Direction

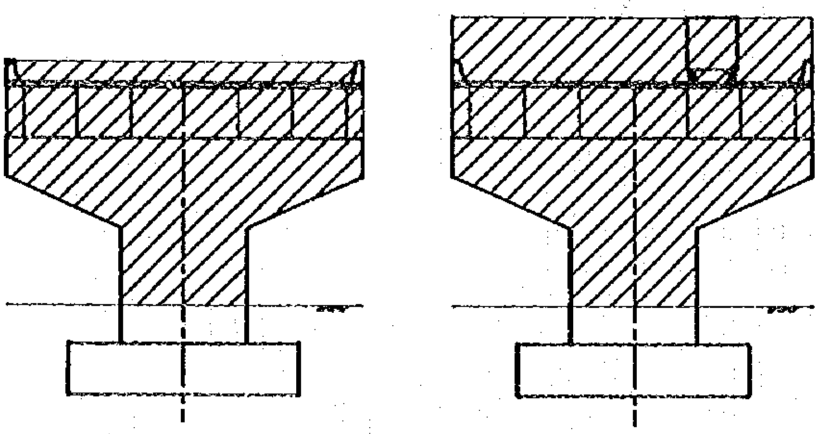


Fig.2.22 Projected Area in the Transversal Direction

3.2.2.4 Combinations of Loads

(1) Abutment

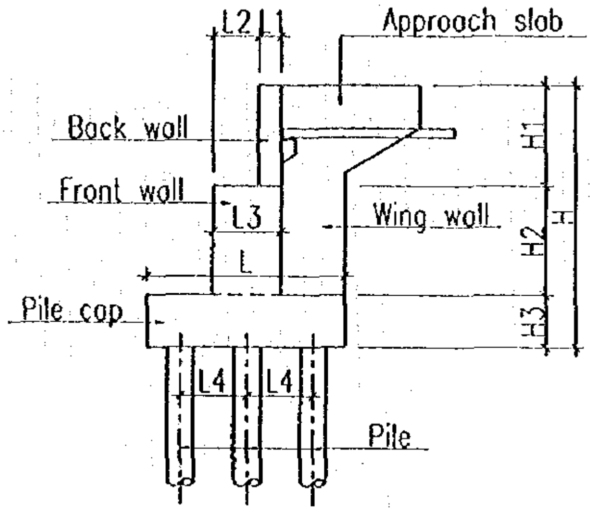
Load Combination Case		D	SD1	SD2	HA	HA+HB	HB*	DS	EPV	EPH	SK	LONG	BUOY
	1				-----	1.00	-----				-----	-----	(1.0)
Nominal	2	1.00	1.00	1.00	1.00	-----	-----	1.00	1.00	1.00	1.00	-----	(1.0)
	3				-----	-----	1.00				-----	1.00	(1.0)
	4				-----	1.10	-----				-----	-----	(1.0)
SLS	5	1.00	1.00	1.20	-----	-----	-----	1.00	1.00	1.00	-----	-----	(1.0)
	6				1.00	-----	-----				1.00	-----	(1.0)
	7				-----	-----	1.00				-----	1.00	(1.0)
	8				-----	1.30	-----				-----	-----	(1.2
ULS	9	1.20	1.20	1.75	1.25	-----	-----	1.20	1.20	1.50	1.25	-----	(1.2)
	10				-----	-----	1.10				-----	1.10	(1.2)

(2) Pier

Load Combination Case		D	SD1	SD2	HA	HA+HB	HA*	DS	WL	TC	SK	LONG	COLL
	1				1.00	1.00	-----		-----	-----	-----	-----	-----
	2				1.00	1.00	-----		1.00	-----	-----	-----	-----
Nominal	3	1.00	1.00	1.00	-----	-----	-----	1.00	1.00	-----	-----	-----	-----
	4				1.00	1.00	-----		-----	1.00	-----	-----	-----
	5				1.00	-----	-----		-----	-----	1.00	-----	-----
	6				-----	-----	1.00		-----	-----	-----	1.00	-----
	1				1.20	1.10	-----		-----	-----	-----	-----	-----
	2				1.00	1.00	-----		1.00	-----	-----	-----	-----
	3				-----	-----	-----		1.00	-----	-----	-----	-----
SLS	4	1.00	1.00	1.20	1.00	1.00	-----	1.00	-----	1.00	-----	-----	-----
	5				1.00	-----	-----		-----	-----	1.00	-----	-----
	6				-----	-----	1.00		-----	-----	-----	1.00	-----
	7				-----	-----	-----		-----	-----	-----	-----	-----
	1				1.50	1.30	-----		-----	-----	-----	-----	-----
	2				1.25	1.10	-----		1.10	-----	-----	-----	-----
	3				-----	-----	-----		1.40	-----	-----	-----	-----
ULS	4	1.20	1.20	1.75	1.25	1.10	-----	1.20	-----	1.30	-----	-----	-----
	5				1.25	-----	-----		-----	-----	1.25	-----	-----
	6				-----	-----	1.25		-----	-----	-----	1.25	-----
	7				-----	-----	-----		-----	-----	-----	-----	1.50

3.2.3 Dimensions

3.2.3.1 Abutment



The dimensions of each components are determined as described below.

(1) Back Wall

Thickness	L1	:	500mm
Height	H1	:	Varies according to beam height (BH), slab thickness (ST) and bearing and epoxy thickness (BT=100mm).

$$H1 = BH + ST + BT$$

(2) Front Wall

Thickness L3 : Varies according to span length.

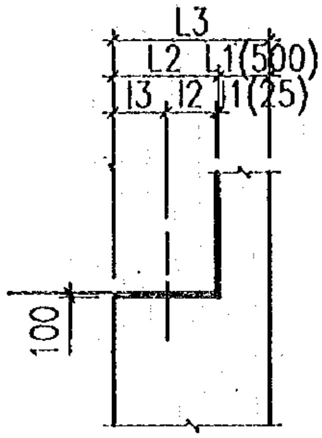
L3 = l1 + l2 + l3 + L1 (refer to Fig.)

Span length (m)	l 1 (mm)	l 2 (mm)	l 3 (mm)	L2' (mm)	L2 (mm)	L3 (mm)
10		250	250	525	700	1200
16		300	280	605		
22	25	350	310	685	1000	1500
28		350	340	715		
32		400	360	785		
35		400	375	800		
45		400	425	900		

The distance from the face of front wall to the center of bearing, l 3, is calculated from the following equation based on the Japanese Standard.

$l3 = \{ 0.5 \times (\text{span length (m)}) + 20 \}$ (cm)

Two kinds of thickness (L3) which are larger than required thickness (500+L2') are adopted as the standard design.



Height H2 : Varies according to parapet wall height and pile cap depth.

H2 = H - H1 - H3

(3) Pile Cap

Depth H3 : Varies according to the section analysis for the pile cap, but the minimum is 1200mm in consideration of the attachment of pile head to the cap.

Breath L : Varies according to arrangement of piles that designed by stability analysis.

(4) Pile

The prestressed concrete pile with 600mm diameter shall be adopted for the bearing and the friction pile.

Spacing L4 : 2.5 x (diameter) = 1500mm for the bearing pile
3.0 x (diameter) = 1800mm for the friction pile

(5) Approach Slab

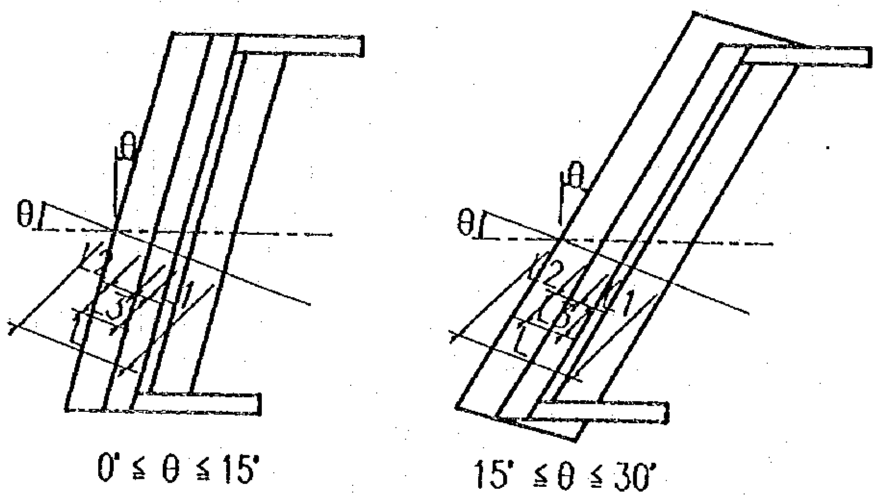
Thickness	: 350mm	according to section analysis
Length	: 5000mm	recommended length

(6) Wing Wall

Thickness	:	700mm for abutment with 8m and 10m height, and 500mm for 6m height according to section analysis.
Length	:	5000mm for abutment with 8m and 10m height and 3000mm for 6m height adopted in the Study

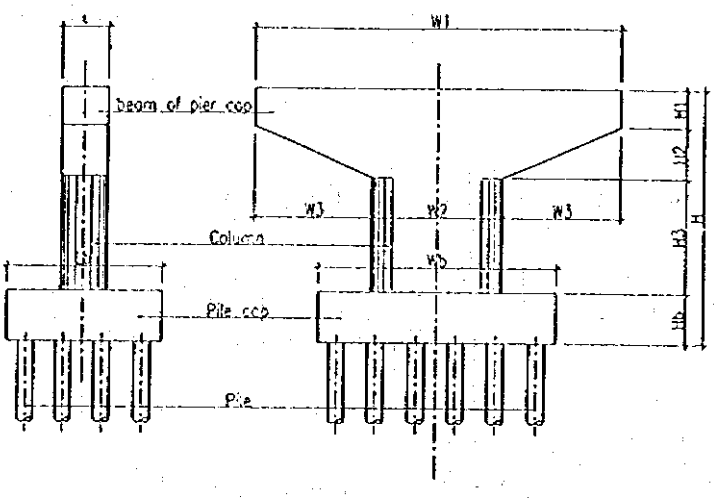
(7) Plane Dimension

The each dimensions predescribed are given in right direction to the skew angle, and pile cap shall not be inclined larger than 15 degrees based on the Japanese Standard as shown Fig. 2.25.



3.2.3.2 T-pier

The dimensions of each components are determined as described below.

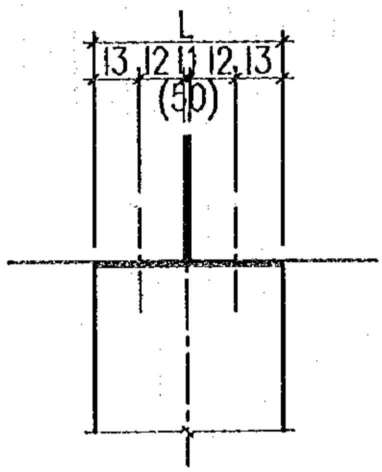


(1) Beam of Pier Cap

Shape of cantilever beam W3, H1, H2

: W3=4.5m, H1=1.5m, H2=2.0m according to the aesthetic and the structural view point in consideration of the balanced arrangement of reinforcement, such as diameter, spacing, and layer.

Thickness : 1.8m according to the same structural view point as in the determination of shape of cantilever beam, and it is sufficient to install superstructure.



(2) Column

Breadth W2 : Varies according to the overall (W1) and the fixed length of cantilever beam (W3)

$$W2 = W1 - 2 \times W3 = W1 - 9.0(m)$$

Height H3 : Varies according to the cantilever beam height and the pile cap depth.

$$H3 = H - H1 - H2 - Hb$$

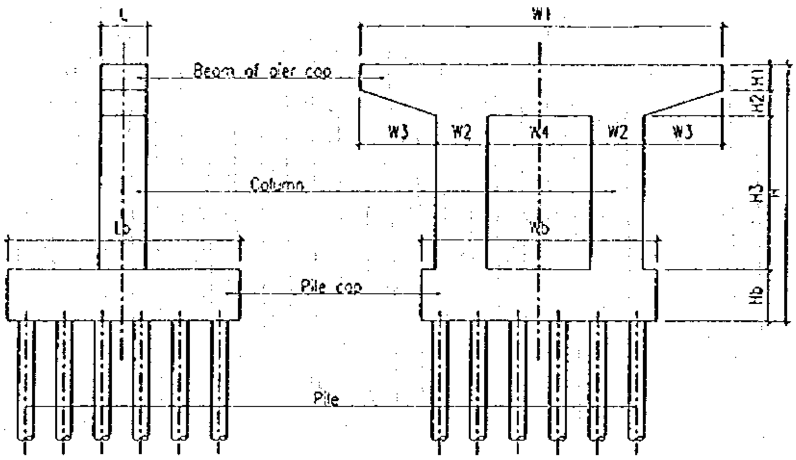
(3) Pile Cap

Breadth Lb, Wb: Varies according to the arrangement of piles designed by stability analysis.

Depth Hb : Varies according to the section analysis for the pile cap, but the minimum depth shall be 1200mm in consideration of the attachment of pile head to the cap.

3.2.3.3 Multiple pier

The dimensions of each components are determined as described below.

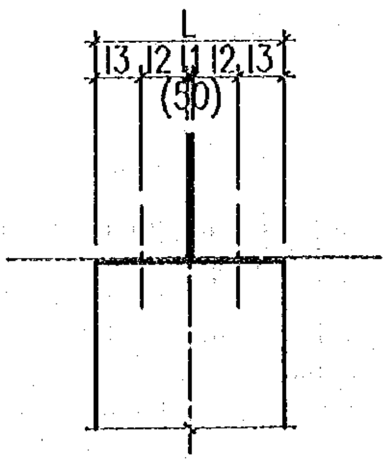


(1) Beam of Pier Cap

Shape of cantilever beam W3, H1, H2

: W3=3.0m, H1=1.0m, H2=1.0m according to the aesthetic view point in consideration of the balanced arrangement of reinforcement, such as diameter, spacing, and layer.

Thickness : 1.8m according to the same structural view point as in the determination of shape of cantilever beam, and it is sufficient thickness to install superstructure.



(2) Column

Breadth W2 : 2.0m according to the same view points as in the determination of shape of cantilever beam

Height H3 : Varies according to the cantilever beam height and the pile cap depth.

H3 = H - H1 - H2 - Hb

(3) Pile Cap

Breadth Lb, Wb: Varies according to the arrangement of piles designed by stability analysis.

Depth Hb : Varies according to the section analysis for the pile cap, but the minimum depth shall be 1200mm in consideration of the attachment of pile head to the cap.

3.2.4 Design Properties of Materials

3.2.4.1 Concrete

Item	Unit	Reinforced Concrete	Prestressed Concrete Spun Pile	Bored Pile
Grade	(N/mm2)	40	60	30
Characteristic strength	(N/mm2)	40	60	30 (25)*
Modulus of elasticity	(KN/mm2)	31.0	36.0	28.0
Compressive stress	(N/mm2)	16.0	24.0	12.0
Design crack width	(mm)	0.25	----	0.25
V, Vtu	(N/mm2)	4.75	4.75	4.75
Vtmin	(N/mm2)	0.42	0.42	0.42

( )* : Design Value

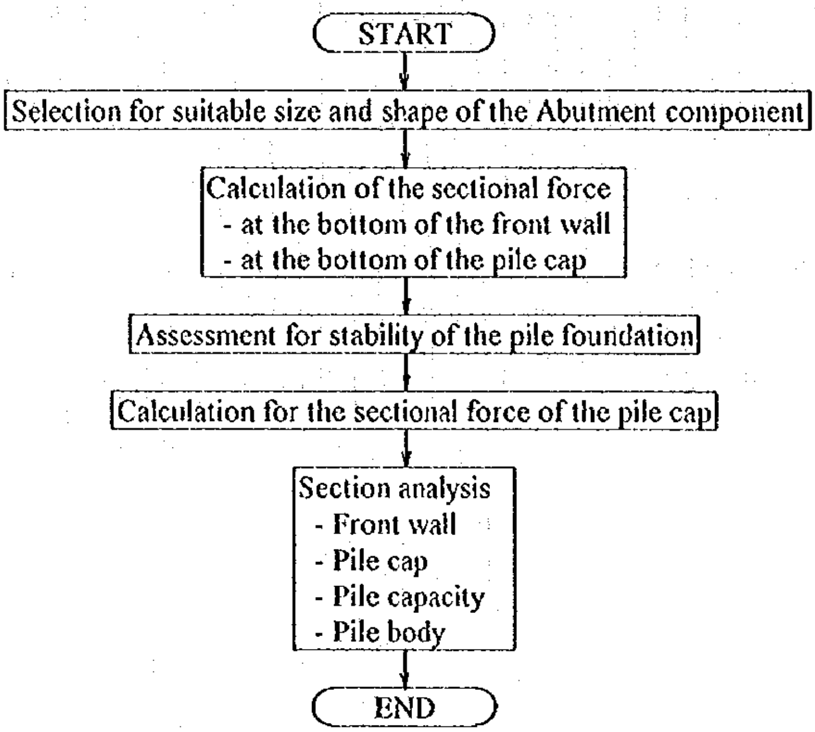
3.2.4.2 Reinforcement

Item	Unit	T460
Characteristic strength	(N/mm2)	460
Modulus of elasticity	(KN/mm2)	200
Compression	(N/mm2)	345
Tension	(N/mm2)	345

3.3 Design for Abutment

3.3.1 Design Flowchart

Design for abutment shall be carried out according to the design flow chart below.

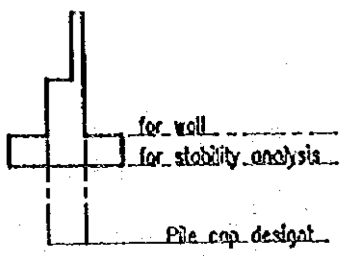


3.3.2 Selection for Suitable Size and Shape

The suitable size and shape of the Abutment components shall be selected as the first trial design from the various design conditions such as the reaction, beam height and length of the superstructure, the overall height of Abutment and the soil condition. (Refer to Clause 3.2.3.1.)

3.3.3 Design Section

The design sections for each components of Abutment shall be taken as below.



3.3.4 Calculation of Sectional Force

The sectional force shall be calculated at each design sections based on the load conditions in the clause 3.2.2.

The axial and shear force shall be calculated by summing up the magnitude of vertical and horizontal loads at the design section. The bending moment shall be calculated by summing up the each moment which is obtained from multiplying the vertical and horizontal loads by the lever arm from the design sections.

Accordingly, the each force shall be calculated as the cantilever beam supported on the design section, whereas sectional force for the pile cap shall be taken by the reaction of pile which is obtained from the design of stability analysis of foundation.

3.3.5 Assessment for Stability of Pile Foundation

3.3.5.1 Stability Analysis

The stability analysis shall be carried out by assessing the axial compressive bearing capacity and the horizontal displacement.

The axial compressive force and the amount of displacement shall be calculated, generally, by the analysis taking account of the displacement.

The programme FPDNN2 was developed for the stability analysis. The requisite design results shall be obtained automatically by giving the input data such as the section constants and arrangement of pile, modulus of deformation for soil and the sectional force in the pile cap.

The FPDNN2 is composed of as follows:

In Fig. 2.29, an arbitrary point 0 on the footing is taken as the origin of both coordinates. The external forces act on the point 0 as shown in the figure and the displacement x and y and the rotation of 0 are set in the direction shown in the figure.

The origin point 0 may be selected in an arbitrary position but it is better to be coincided with the center of the figure of the whole piles beneath the footing.

The displacement of the point 0 can be obtained by solving the ternary simultaneous equations (A) for the footing.

$$\left.\begin{aligned} A_{xx}\cdot\delta_x + A_{xy}\cdot\delta_y + A_{x\alpha}\cdot\alpha &= H_o \\ A_{yx}\cdot\delta_x + A_{yy}\cdot\delta_y + A_{y\alpha}\cdot\alpha &= V_o \\ A_{\alpha x}\cdot\delta_x + A_{\alpha y}\cdot\delta_y + A_{\alpha\alpha}\cdot\alpha &= M_o \end{aligned}\right\} \text{-(A)}$$

Assuming the tailside of the footing to be horizontal, these coefficients can be obtained by the following equations.

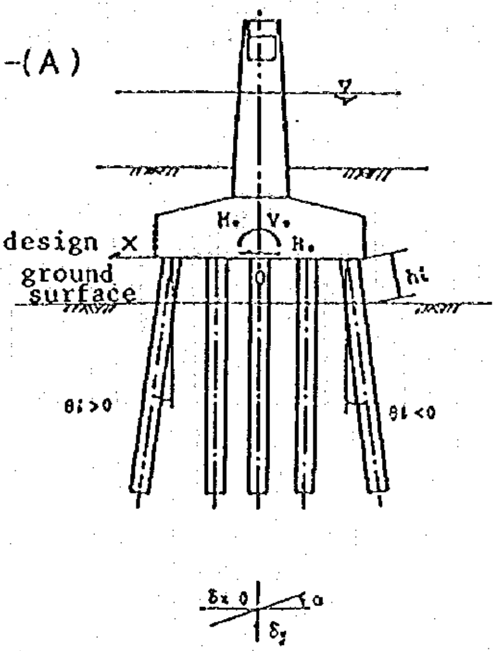


Fig. 2.29 The coordinates for calculation in using displacement method.

$$\left.\begin{aligned} A_{xx} &= \Sigma(K_1\cdot\cos^2\theta_i + K_v\cdot\sin^2\theta_i) \\ A_{xy} &= A_{yx} = \Sigma(K_v - K_1)\sin\theta_i\cdot\cos\theta_i \\ A_{x\alpha} &= A_{\alpha x} = \Sigma\left[(K_v - K_1)\,x_i\cdot\sin\theta_i\cdot\cos\theta_i - K_2\cos\theta_i\right] \\ A_{yy} &= \Sigma(K_v\cdot\cos^2\theta_i + K_1\cdot\sin^2\theta_i) \\ A_{y\alpha} &= A_{\alpha y} = \Sigma\left[(K_v\cdot\cos^2\theta_i + K_1\sin^2\theta_i)\,x_i + K_2\cdot\sin\theta_i\right] \\ A_{\alpha\alpha} &= \Sigma\left[(K_v\cdot\cos^2\theta_i + K_1\cdot\sin^2\theta_i)\,x_i^{\,2} + (K_2 + K_3)\,x_i\cdot\sin\theta_i + K_4\right] \end{aligned}\right\} \text{-(B)}$$

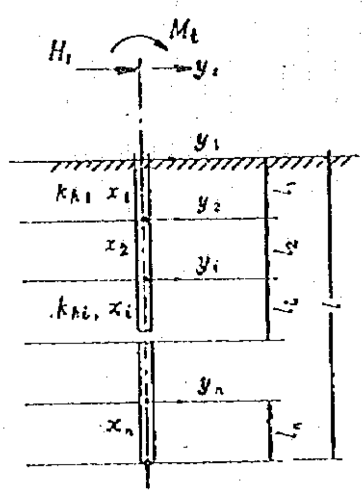
where,

Ho	:	Horizontal load acting above the tailside of footing (t)
Vo	:	Vertical load acting above the tailside of footing (t)
Mo	:	Moment of the external forces acting around the point 0 (t.m)
δx	:	Horizontal displacement of the point 0 (m)
δy	:	Vertical displacement of the point 0 (m)
α	:	Angle of rotation of footing (rad.)
Xi	:	x coordinate of the top of the i-th pile (m)
θi	:	The angle formed between the axis of the i-th pile and y coordinate (refer to Fig. 2.29) (degree)

It is practically permissible to use the value of β to be calculated as below to obtain the lateral spring constants K_1, K_2, K_3 and K_4 of pile when the horizontal coefficient of ground reaction k is constant irrespective of the depth and the penetration length is sufficiently long ($l > 3 / \beta$).

However, it is quite rare that ground consists of one kind of stratum. Therefore, analysis method which can be applied to various ground conditions for multi stratum shall be introduced, and pr gramme for it shall also be for multi stratum.

The multi stratum ("n" stratum) is shown below.



Equation for each stratum shall be analyzed by dividing "n" at an embedding part of a pile, considering following condition. "x = li" for "i" stratum and "x = 0" for "(i+1) stratum" at boundary layer have same displacement (value) (y.Q) and sectional force (M,H). Using this phenomena, equation for relation between head and base of pile shall be established.

Table 2.9 Lateral Spring Constant

	Rigidly connected pile heads		Hinge-connected pile heads	
	$h \neq 0$	$h = 0$	$h \neq 0$	$h = 0$
K_1	$\dfrac{12\ EI\ \beta^3}{(1+\beta h)^3 + 2}$	$4EI\beta^3$	$\dfrac{3\ EI\ \beta^3}{(1+\beta h)^3 + 0.5}$	$2EI\beta^3$
K_2, K_3	$K_1 \cdot \dfrac{\lambda}{2}$	$2EI\beta^2$	0	0
K_4	$\dfrac{4EI\beta}{1+\beta h} \cdot \dfrac{(1+\beta h)^3 + 0.5}{(1+\beta h)^3 + 2}$	$2EI\beta$	0	0

where,

β : Characteristic value or a pile $\beta = \sqrt[4]{\frac{kD}{4EI}} (m^{-1})$

λ : $h = \frac{1}{\beta}$

k : Coefficient of lateral subgrade reaction * (t/m3)

D : Diameter of pile (m)

EI : Flexural rigidity of a pile (t/m2)

h : Axial length of a pile above ground surface in designing (m)

Note)* When k has been obtained in kg/cm3, the value should be increased by thousand times.

From the displacements ($\delta x \cdot \delta y \cdot \alpha$) at the origin point on the footing, which have been obtained from the above calculations, the axial force P_{Ni}, the lateral force P_{Hi} and the moment M_{ti}, which act on the pile head can be obtained by the following equations.

$$\left.\begin{aligned} P_{Ni} &= K_v \, \delta'_{yi} \\ P_{Hi} &= K_1 \, \delta'_{xi} - K_2 \alpha \\ M_{ti} &= -K_3 \, \delta'_{xi} + K_4 \alpha \end{aligned}\right\} \text{-(C)}$$

$$\text{where} \quad \left.\begin{aligned} \delta'_{xi} &= \delta_x \cos \theta_i - (\delta_y + \alpha x_i) \sin \theta_i \\ \delta'_{yi} &= \delta_x \sin \theta_i = (\delta_y + \alpha x_i) \cos \theta_i \end{aligned}\right\} \text{-(D)}$$

$\delta'xi$: Lateral displacement of the i-th pile head (m)

$\delta'yi$: Axial displacement of the i-th pile head (m)

Kv : Axial force (axial spring constant) which causes a unit axial displacement to the pile head (t/m)

K_1, K_2, K_3, K_4 : Spring constants (See Table 2.8)

x_i : x coordinate of the i-th pile head (m)

θ_i : Angle which is formed between the axis of the i-th pile and y coordinate (degree)

P_{Ni} : Axial force of the i-th pile (t)

P_{Hi} : Lateral force of the i-th pile (t)

M_{ti} : Moment as an external force which acts on the i-th pile head (t.m)

M_{ti} of the aboves is an external moment distributed to the pile head and the bending moment M_{bi} as an internal force which is caused to the pile head is equal to $-M_{ti}$.

The vertical reaction V_i and horizontal reaction H_i at the pile head are given by

$$\left.\begin{aligned} V_i &= P_{Ni}\cdot\cos\,\theta_i - P_{Hi}\cdot\sin\,\theta_i \\ H_i &= P_{Ni}\cdot\sin\,\theta + P_{Hi}\cdot\cos\,\theta_i \end{aligned}\right\} -(E)$$

These equations are used in the calculation for the arrangement of reinforcement bars of the footing.

Since the following equations should hold, thereby it is possible to check to see if the calculations have been made correctly.

3.3.5.2 Allowable Axial Compressive Bearing Capacity

The allowable axial compressive bearing capacity of a pile, Ra, shall be calculated from the following equation:

$$Ra = \frac{1}{n}(Qd \cdot Ab + U \sum lifi)$$

where,

n	:	Safety factor for the bearing pile : n=2.5 for the friction pile : n=2 0
Qd	:	Ultimate bearing capacity of a pile =30 x N value ≦ 1,000t/m2
Ab	:	Area of concrete of a pile tip φ600: 0.158m2
U	:	Stratum depth with the skin friction φ600: 1.885m
li	:	Stratum depth with the skin friction
fi	:	Maximum skin friction of stratum

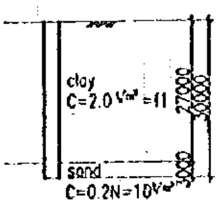
The maximum skin friction shall be taken as follows according to the Japanese Standard.

for clay : Cohesion, C or N Value
for sand : 0.2 x N Value

Ra for a bearing and friction pile shall be calculated respectively as follows:

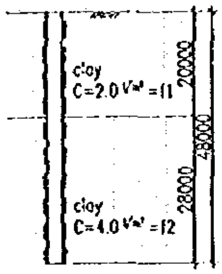
- For a bearing pile

Ra = (1 / 2.5) x {1,000 x 0.158 + 1.885 x (27.0 x 2.0 + 3.0 x 10.0)}
= (1 / 2.5) x (158.0 + 158.3)
= 126.5 t → 1240 KN



- For a friction pile

$$Ra = (1 / 2.0) \times \{1.885 \times (20.0 \times 2.0 + 28.0 \times 4.0)\}$$
$$= (1/ 2.0) \times (75.4 + 211.1)$$
$$= 143.2\ t \rightarrow 1404\ KN$$



3.3.5.3 Allowable Horizontal Displacement

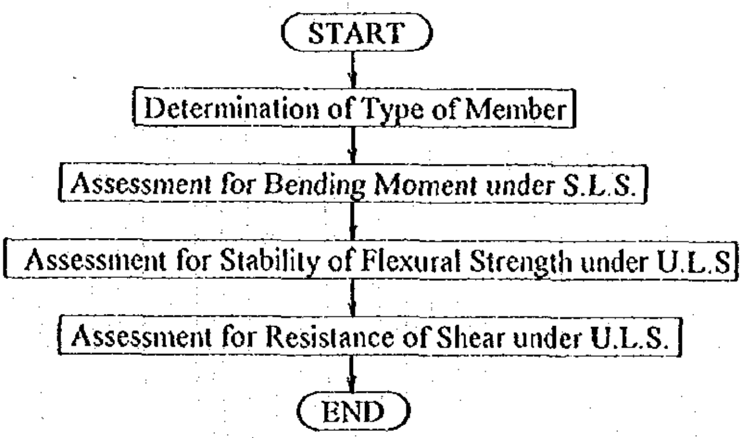
The allowable horizontal displacement of a pile shall be less than 3% of pile diameter or 20mm, whichever bigger in order, to assure the safety against the lateral force.

3.3.6 Assessment for Section Analysis

3.3.6.1 Design flowchart

In this clause, the section analysis for the front wall and for the pile cap shall be described, and the pile shall be assessed its capacity comparing with common piles used in Malaysia.

The section analysis shall be carried out according to the design flow chart below.



3.3.6.2 Determination of Type of Members

(1) Front Wall

The front wall shall be designed as "short reinforced concrete wall" based on the clause 5.6, Part 4, BS 5400 as below.

- Greater lateral dimension is more than four times lesser lateral dimension

  11.9m > 4 x 1.5m = 6.0 → Wall

- Ratio of effective height to thickness does not exceed 12.

  (10.0m / 1.2m) = 8.3 > 12 → Short

Mentioned in the above, the various values is taken as critical values in order to judge each regulation.

(2) Pile Cap

The pile cap shall be designed as "beam-and-slab" in accordance with the clause 5.3 "Beams", Part 4, BS 5400, and also designed by the bending theory.

3.3.6.3 Assessment for Bending Moment under S.L.S

The design for bending moment under S.L.S. shall be carried out by the assessment of the crack width, compressive concrete stress and reinforcement stress.

The programmee BSDANM was developed for section analysis. The requisite design results under S.L.S. and U.L.S. shall be taken automatically by giving the input data such as the material properties, reinforcement data, dimension and sectional force.

(1) Design Crack Width

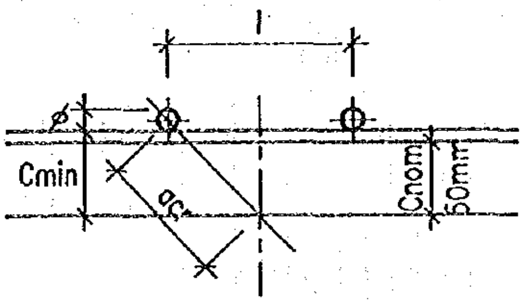
The design crack width shall be calculated by the following equation:

Design crack width = (3acr ε m) / (1+2(acr-Cmin) / (h-dc)

whereas,

ε m= ε 1 - { (3.8bth(a'-dc)) / ε sAs (h-dc)}{1- (Mq)/(Mg)}10^{-9}.

In the above, acr is the distance from the point considered to the surface of the nearest bar which controls the crack width.



The design crack width shall be limited to the value of 0.25mm.

(2) Compressive Concrete Stress

The compressive concrete stress shall be calculated from the following equation:

$$fcu = (Mg + Mq) \times dc / I$$

where,

$I = 1/3 \times bt / dc^3 + Es/Ec \times As \times (ds-dc)^2$.

$dc = \{-nAs + \sqrt{\ }\{(nAs) + 4 \times 1/2 \times bt \times nAs \times ds/bt\}$

$Ec = \{1 / (Mg+Mq)\}(Eg \times Mg + Ec \times Mq)$

$Eg = 1/2\ Ec$

The compressive stress shall not exceed the value of 16.0 N/mm2.

(3) Reinforcement Stress

The reinforcement stress shall be calculated by the following equation:

$$fs = Es / Ec \times (Mg + Mq) \times (ds - dc) / I$$

The reinforcement stress shall not exceed the value of 345 N/mm2.

3.3.6.4 Assessment for Stability of Flexural Strength under U.L.S

The resistance moment for the reinforced concrete member shall be calculated according to the clause 5.3.2, Part 4, BS 5400.

When analyzing a cross section to determine the ultimate strength, the following assumptions shall be made.

- Plane section remain plane
- The strain at the outermost compression fibre is taken as 0.0035.
- The tensile strength of the concrete is ignored.
- The stress strain curves for concrete and reinforcement are given in figure 1 and 2 in Part 4, BS 5400.

The ultimate moment of resistance shall be larger than 1.15 times to the required value, if it is less than 1.15 times, the section shall be proposioned such that the strain at the centroid of the tensile reinforcement is not less than:

$$0.002 + fy / Es\ \gamma m$$

3.3.6.5 Assessment for Resistance of Shear under U.L.S

Calculation for shear resistance is only required for the ultimate limit state.

(1) Calculation of Shear Stress

The shear stress, v, at any cross section shall be calculated from:

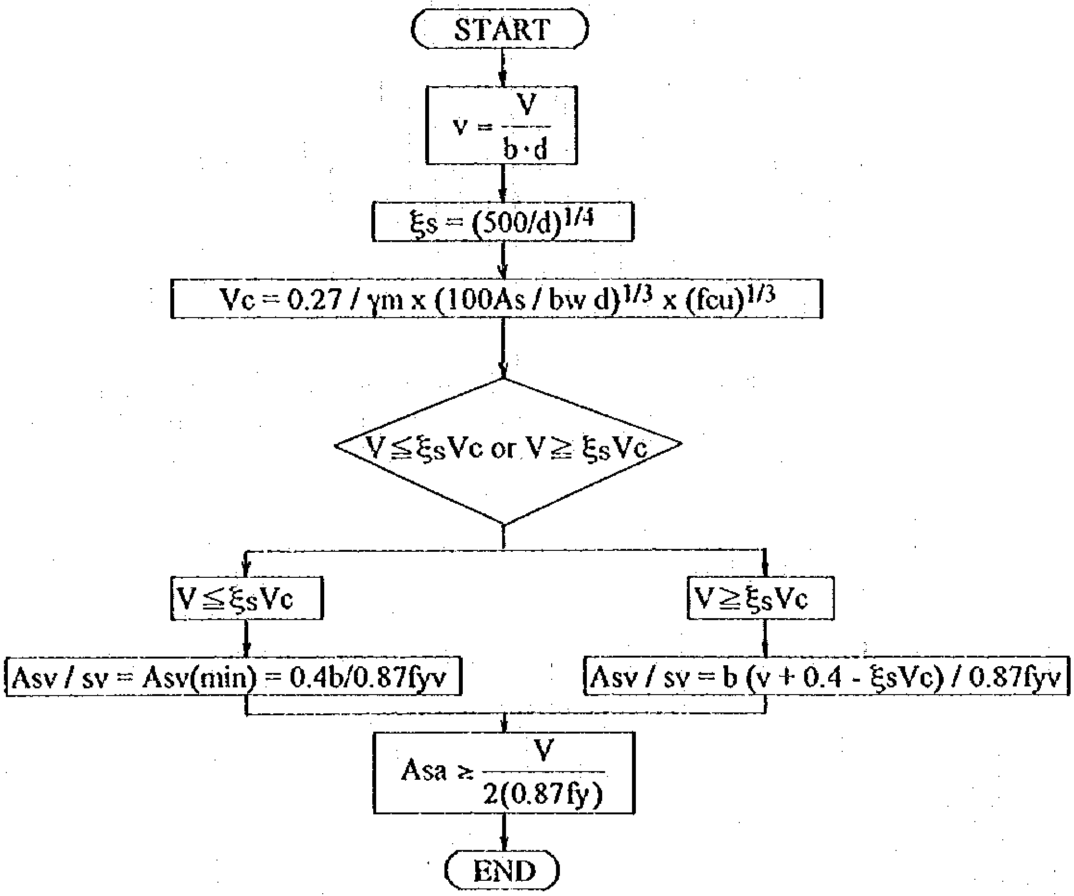
$$v = \frac{V}{bd}$$

where,

v : the shear force due to ultimate loads.
b : the breath of the section taken as the rib width, (bw).
d : the effective depth to the tension reinforcement taken as the depth from the extreme compression fiber to the centroid of the reinforcement.

(2) Shear Reinforcement

Shear reinforcement shall be calculated based on the Part 4, BS 5400 as undermentioned flow chart.



where,

$\gamma m = 1.25$, $fcu \leqq 40$

Arrangement of Minimum reinforcement, Asv(min), is not necessary for the wall and pile cap.

Shear reinforcement required shall be calculated as the area of one leg of a closed link at a section per 1.0m length.

$$\Sigma Av = Asv / 2$$

On the other hand, the area of additional longitudinal reinforcement required shall be calculated as the required amount for shear force in the tensile zone.

$$\Sigma Asa = Asa$$

The remained area of the required tensile reinforcement for the bending moment under the ultimate limit state shall be effective to the longitudinal reinforcement Σ Asa.

In the study, shear reinforcement Asv, shall be calculated the required cross-sectional area of all the legs of the links per 1.0m length.

(3) Assessment of Punching Shear Stress for Pile Cap

The assessment of punching shear shall be carried out at the corner pile of the cap, which causes highest stress in accordance with the clause 5.4.4, Part 4, BS 5400.

3.3.7 Design for Back Wall

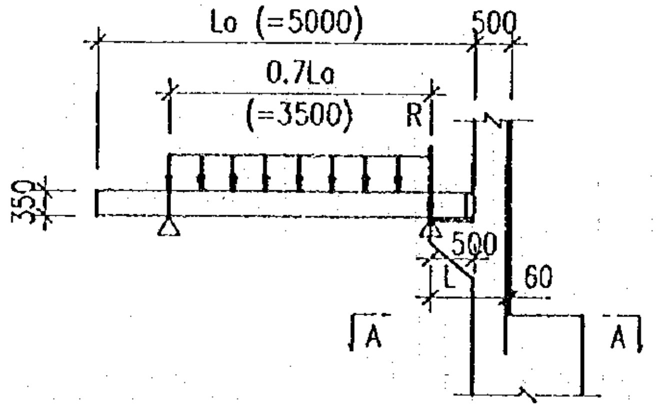
The loads and calculation of sectional force for design of back wall shall be described only, because the assessment of member, by using the programme BSDAM, shall be carried out by the same way as in other members.

(1) Load to be Considered

- Over burden
- Approach slab self-weight
- Live load surcharge with HA
- Live load surcharge with HB 45 units
- Earth pressure due to back fill

(2) Calculation of Sectional Force

The sectional force for back wall shall be calculated according to the Japanese practice as shown below. The approach slab is assumed as the simple beam.



where,

M : Nominal bending moment for parapet wall design at A-A section
R : Reaction due to loads acting 70% length of approach slab
L : Lever arm from main reinforcement of parapet wall to end of basement of approach slab

(3) Example of Calculation of Sectional Force per 1.0m Width

(a) Front

(i) Over burden

t = 0.95m, w0 = 23 kN/m3 → w1 = 21.85 kN/m2

M1 = R1 x L
= (21.85 x 3.5 x 1/2) x 0.94
= 35.9 kN · m/m

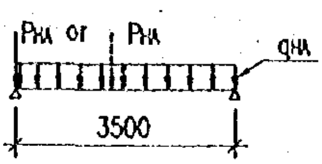
(ii) Approach slab

t = 0.35m, w0 = 25 kN/m3 → w2 = 8.75 kN/m2

M2 = R2 x L
= (8.75 x 3.5 x 1/2) x 0.94
= 14.4 kN · m/m

(iii) Live load surcharge with HA

HA shall be loaded as below

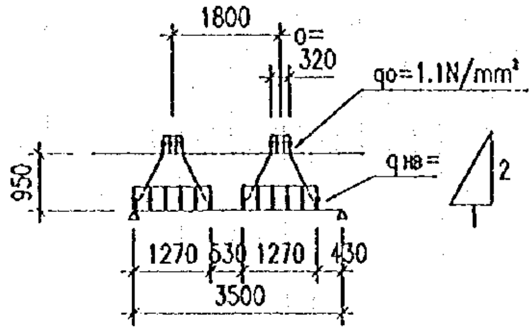


Carriage width ,bL= 3.25m , Lane factor = 0.274 bL = 0.891
, therefore
PHA = 0.891 x 120 / 3.25 = 32.898 kN/m
qHA = 0.891 x 336 x (1/3.5)0.67 / 3.25 = 39.793 kN/m2

R3 = 32.898 + 1/2 x 39.793 x 3.5 = 102.54 kN/m
M3 = R3 x L
= 102.54 x 0.94
= 96.4 kN · m/m

(iv) Live load surcharge with HB 45 units

HB shall be dispersed as below



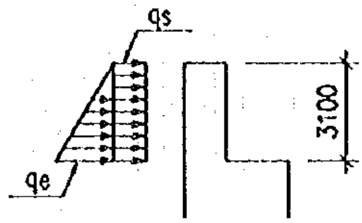
a = √ (2.5 x 1000 x 45 / 1.1 = 320 mm
l1= 320 + 950 = 1270 mm, therefore
qHB = 2.5 x 45 / 1.270 = 88.583 kN/m2

R4=112.5 x {(3.5 - 0.635) / 3.5 + (0.430 + 0.635) / 3.5}=126.32 kN/m
M4= R4 x L
= 126.38 x 0.94 = 118.7 kNm/m

(b) Rear

(i) Earth pressure due to back fill

Earth pressure shall be loaded as below



ka = 0.309, δ = φ / 3 = 10 (deg), γ = 19 kN/m3

M1 = 1/2 x 0.309 x 19 x 3.1 x 3.1 x 3.1 / 3 x cos 10 = 28.71 kNm/m

(ii) Live load surcharge with HA

qs =10 kN/m2
M2 = 0.309 x 10 x 3.1 x 3.1 x 1/2 x cos 10 = 14.62 kNm/m

(iii) Live load surcharge with HB 45 units

qs =20 kN/m2
M2 = 0.309 x 20 x 3.1 x 3.1 x 1/2 x cos 10 = 29.24 kNm/m

(d) Under S.L.S.

Front (DL) = 1.0 x (M1 + M2) = 1.0 x (35.9+14.4) = 50.3 kNm/m
Front (LL) = 1.2 x M3 = 1.2 x 96.4 = 115.9 kNm/m

Rear (DL) = 1.0 x M1 = 28.7 kNm/m
Rear (LL) = 1.0 x M2 = 1.0 x 14.62 = 14.6 kNm/m

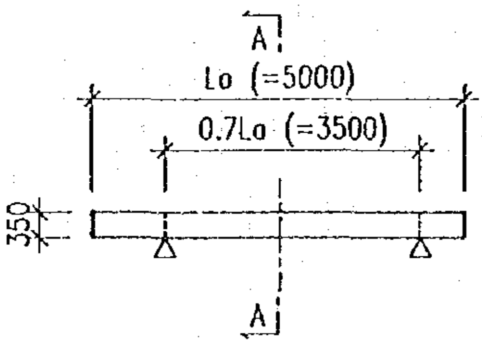
(e) Under S.L.S.

Front (M) = 1.2 x (M1 + M2) + 1.3 x M4
= 1.2 x (35.9+14.4) + 1.3 x 118.7 = 214.7 kNm/m

Rear (M) = 1.5 x (M1 + M3)
= 1.5 x (28.71 + 29.24) = 86.9 kNm/m

3.3.8 Design for Approach Slab

Similar to the design of parapet wall, the sectional force for approach slab shall be calculated according to the Japanese practice as shown below. (refer to the Design for Parapet wall).



Example of calculation of sectional force per 1.0m width.

(1) Over burden

M1 = 1/8 x 21.85 kN/m2 x 3.5 x 3.5 = 33.5 kN · m/m

(2) Approach Slab

M2 = 1/8 x 8.75 kN/m2 x 3.5 x 3.5 = 13.4 kN · m/m

(3) Live Load Surcharge with HA

M3 = 32.898 kN/m x 3.5/4 + 1/8 x 39.793 kN/m2 x 3.5 x 3.5 = 89.6 kN · m/m

(4) Live Load Surcharge with HB 45 units

M4 = 112.5 kN/m x 3.5/2 + 88.583 kN/m2 x 1.270 x 1/2 x (0.530 + 1.270)
= 95.6 kN · m/m

(4) Under S.L.S.

M (DL) = 1.0 x (M1 + M2) = 1.0 x (33.5+13.4) = 46.9 kNm/m
M (LL) = 1.2 x M3 = 1.2 x 89.6 = 107.5 kNm/m

(5) Under U.L.S.

M = 1.2 x (M1 + M2) + 1.5 x M3
= 1.2 x (33.5 + 13.4) + 1.5 x 89.6 = 190.7 kNm/m

3.3.9 Design for Wing Wall

(1) Load to be considered

- Earth pressure

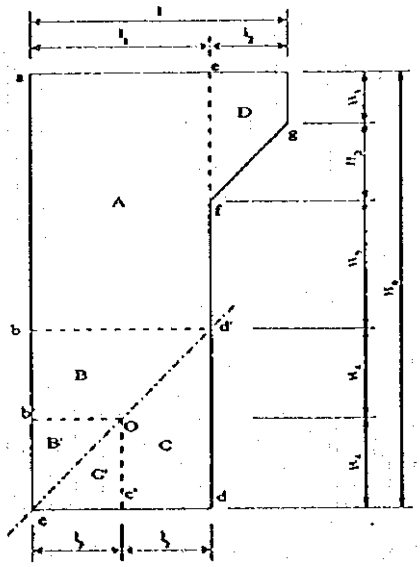
coefficient at rest : ko = 0.5
back fill weight : $\gamma 1$ = 1.9 kN/m3

- Live load surcharge

fo HB45units : pHB = 20 kN/m2
for HA : pHA = 10 kN/m2

(2) Design section

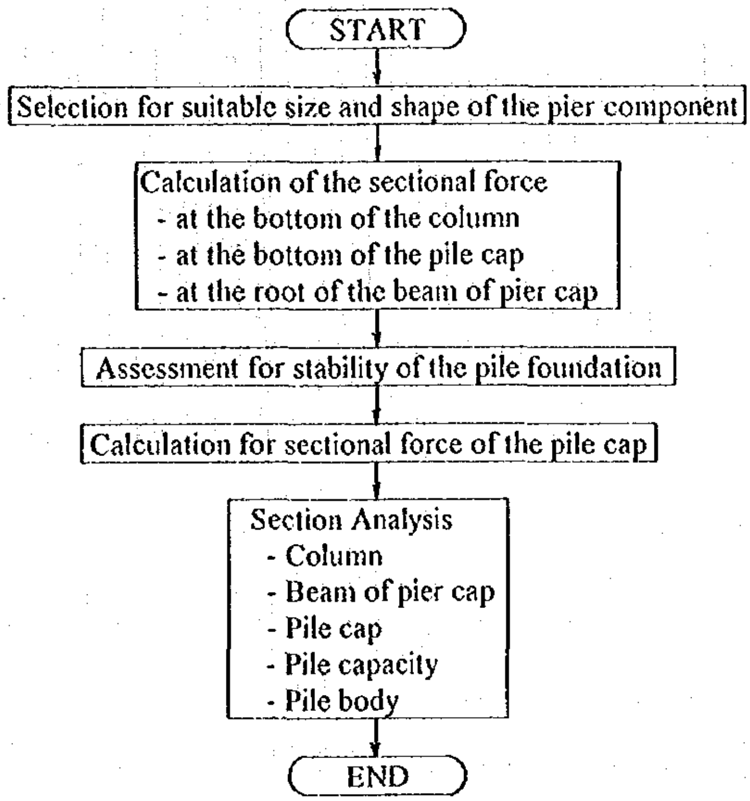
Design of wing wall shall be carried out at section of (e-f), (a-b), (b-b'), (b'-c), (c'-d), and (c-c') as shown below.



3.4 Design for T-type Pier

3.4.1 Design Flowchart

Design for T-pier shall be carried out according to the design flow chart below.

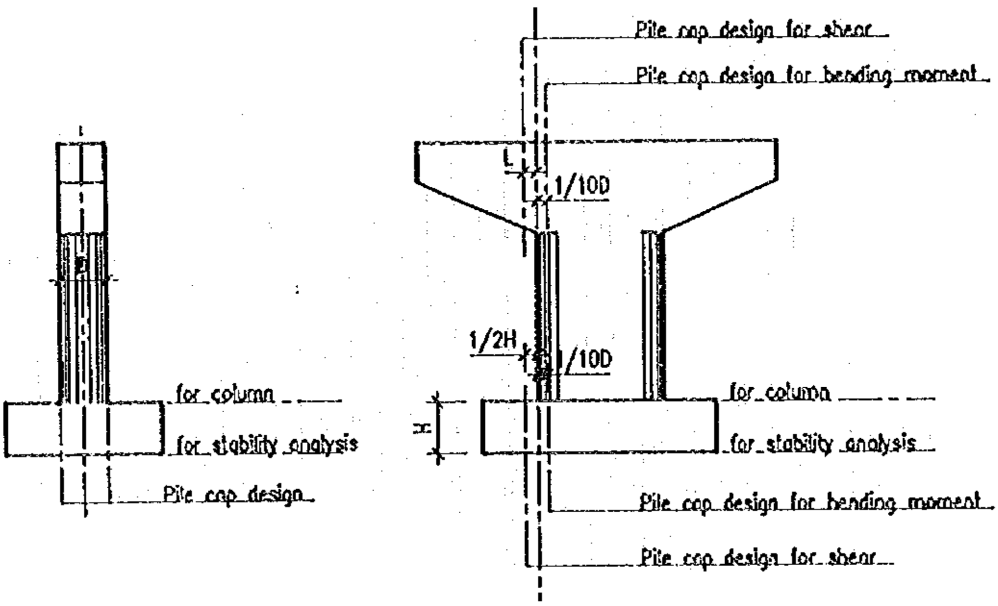


3.4.2 Selection for Suitable Size and Shape

The suitable size and shape of the T-pier components shall be selected, as the first trial design, from the various design conditions such as the reaction, beam height and length of the superstructure, the overall height of T-pier and the soil condition. (refer to Clause 3.2.3.2 in this Chapter)

3.4.3 Design Section

The design sections for each components of T-pier should be taken as below.



The distance of (1/10D) shall be adopted according to the Japanese Standard, and (L) shall be taken according to the clause 5.3.1.1, Part 4, BS 5400.

3.4.4 Calculation of Sectional Force

The sectional force shall be calculated at each design sections based on the load conditions in the clause 3.2.2.

The axial and shear force shall be calculated by summing up the magnitude of vertical and horizontal loads at the design section.

The bending moment shall be calculated by summing up each moment which is obtained from multiplying the vertical and horizontal loads by the lever arm from the design sections.

Accordingly, the each forces shall be calculated as the cantilever beam supported on the design section, whereas sectional force for the pile cap shall be taken by the reaction of pile which is obtained from the design of stability analysis of foundation..

3.4.5 Assessment for Stability of Pile Foundation

The procedure of assessment is the same as in the Abutment design, refer to the clause 3.3.5 in this Chapter.

3.4.6 Assessment for Section Analysis

The different point of section analysis for T-pier, compared with Abutment, is the treatment of type of structural member such as the beam of pier cap and the column. Other procedure of section analysis is same as in the Abutment. Refer to the clause 3.3.6.

(1) Beam of Pier Cap

The beam of pier cap shall be designed as "cantilever beam" supported on column.

(2) Column

The column shall be designed as "short column" based on the clause 5.5, Part 4, BS 5400 as below.

- Greater lateral dimension is less than or equal to four times lesser lateral dimension

  $7.05m < 4 \times 1.8m = 7.2m \rightarrow$ Column

- Ratio of effective height to depth does not exceed 12.

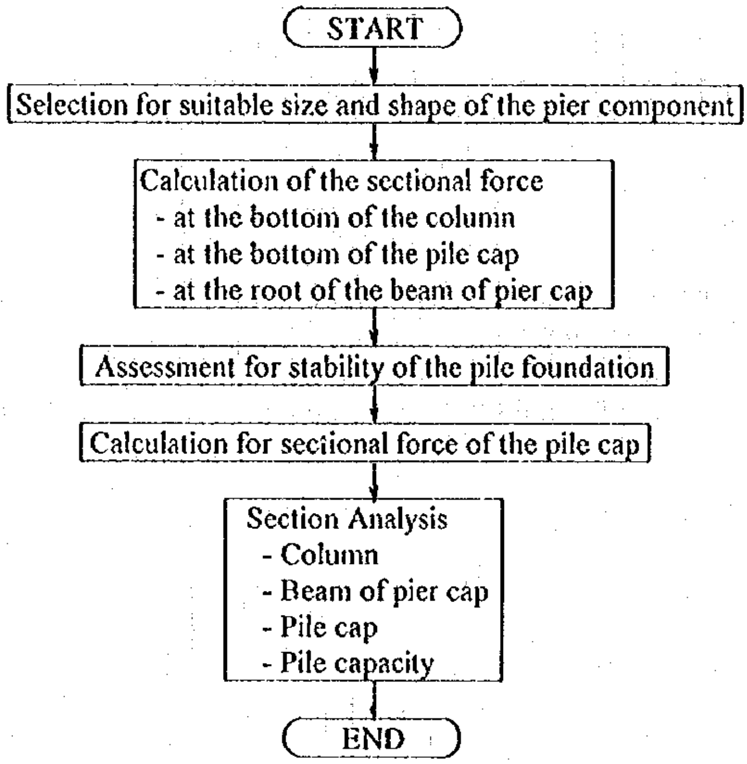
  $20.0m / 1.8m = 11.1 < 12 \rightarrow$ Short

Mentioned in the above the various values is taken as critical values in order to judge each regulation.

3.5 Design for Multiple Pier

3.5.1 Design Flowchart

Design for Multiple pier shall be carried out according to the design flowchart below.

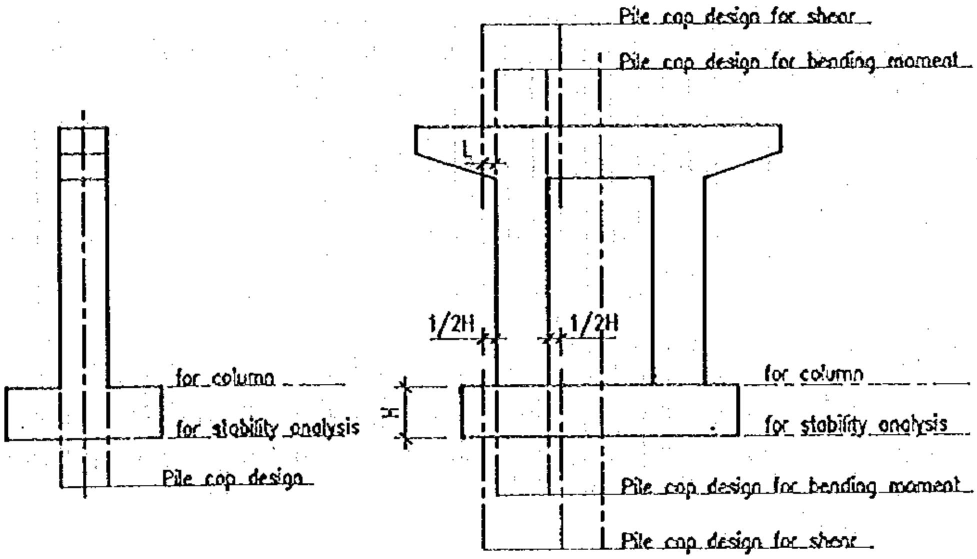


3.5.2 Selection for Suitable Size and Shape

The suitable size and shape of the Multiple pier shall be selected, as the first trial design, from considering the various design conditions such as the reaction, beam height and length of the superstructure, the overall height of Multiple pier and the soil condition. (refer to Clause 3.2.3.3 in this Chapter)

3.5.3 Design Section

The design sections for each components of Multiple pier shall be taken as below.

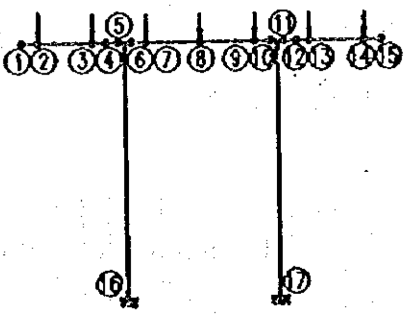


The distance of (1/2H) shall be adopted according to the Japanese Standard.

3.5.4 Calculation of Sectional Force

The sectional force shall be calculated at the each design sections based on the load conditions in the clause 3.2.2.

The axial force, shear and moment shall be calculated by the frame supported on the center of column as shown below.



The programmee TMATS was developed for the frame analysis by elastic method. The requisite design results shall be taken automatically by giving the input data such as the coordinate data, section constants, reactions due to superstructure, and load data required for substructure design.

Whereas, sectional force for the pile cap shall be calculated as the continuous beam supported on the center of column considering the reaction of pile which. is obtained from the design of stability analysis of foundation.

3.5.5 Assessment for Stability of Pile Foundation

The procedure of assessment is the same as in the Abutment design, refer to the clause 3.3.5 in this Chapter.

3.5.6 Assessment for Section Analysis

The different point of section analysis for Multiple pier, compared with Abutment, is the treatment of type of structural member such as the beam of pier cap and the column. Other procedure of section analysis is the same as in the Abutment. Refer to the clause 3.3.6.

(1) Beam of Pier Cap

The beam of pier cap shall be designed as "cantilever beam" supported on column.

(2) Column

The Column shall be designed as "short column" based on the clause 5.5, Part 4, BS 5400 as below.

- Greater lateral dimension is less than or equal to four times lesser lateral dimension

  7.05m < 4 x 1.8m = 7.2m → Column

- Ratio of effective height to depth does not exceed 12.

  20.0m / 1.8m = 11.1 < 12 → Short

Mentioned in the above, the various values shall be taken as critical values in order to judge each regulation.

3.6 Detailing

(1) Concrete Cover on Reinforcement

- Abutment and Wall : 50mm
- T-pier & Multiple Pier : 50mm
- Pile Cap : 70mm

(2) Minimum Reinforcement

The area of tension reinforcement shall be not less than undermentioned amount when using grade 460 reinforcement.

Table 2.9 Minimum Reinforcement

Structure	Component	Force	Minimum reinforcement	Type of member
Abutment	Front wall	Moment	0.004 b x d	short reiforced concrete wall
		Shear	0.4 x sv / (0.87 x fyv)	
	Pile cap	Moment	0.0015 b x d	beam
		Shear	0.4 x sv / (0.87 x fyv)	
T-pier & Multiple pier	Column	Moment	0.01 x Ac or 0.15 x N / fy whichever lesser	short column
		Shear	0.4 x sv / (0.87 x fyv)	
	Beam of pier cap	Moment	0.0015 b x d	cantilever beam
		Shear	0.4 x sv / (0.87 x fyv)	
	Pile cap	Moment	0.0015 b x d	beam
		Shear	0.4 x sv / (0.87 x fyv)	

(3) Lap Length

The lap length shall be adopted as 25 times bar size plus 250mm in reinforcement.

$$L = 25 \times \phi + 250 \text{ (mm)}$$

(4) Spacing

The normal spacing shall be adopted as 150mm, and the maximum spacing shall not exceed 300mm.

The detailing of substructure, such as concrete cover, minimum reinforcement, and lap length and spacing of reinforcement, shall be determined in accordance with the clause 5.8, Part , BS 5400.

3.7 Design of Spread Foundation

This section describes only for the stability analysis of base, since other design procedure is the same as pile foundation.

3.7.1 Stability Analysis

Design of the spread foundation shall be carried out by assessing the bearing, overturning and sliding.

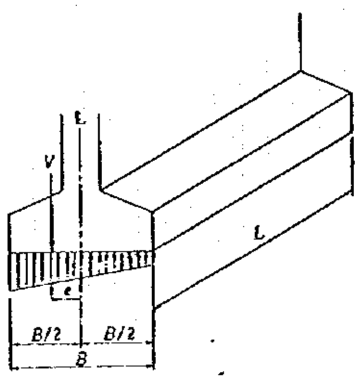
(1) Bearing Capacity

Maximum and minimum bearing capacity calculated from the following equation should be within the allowable bearing capacity.

$$q_{max,min} = \frac{V}{LB} \pm \frac{6M_B}{LB^2} < q_a$$

where,

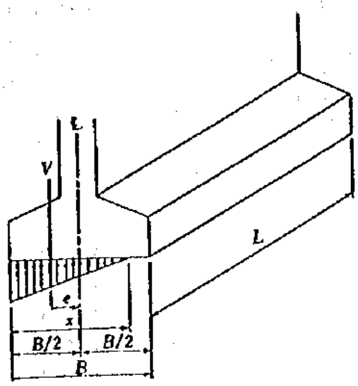
V	:	Vertical force at the bottom of base
M_B	:	Beading moment at the center of base
L	:	Width of base
B	:	Breadth of base
qa	:	Allowable bearing capacity



$$q_{max} = \frac{2V}{Lx}$$

where,

V	:	Vertical force at the bottom of base
L	:	Width of base
x	:	Working breadth of reaction at the bottom of base
		{x = 3 x (B/2 - e)}



(2) Overturning

The value of "e" shall be less than one-sixth the breadth of foundation "B".

$$e = \frac{M_B}{V} < B/6$$

(3) Sliding

$$Hu = CBA' + V\tan\phi_B > 1.5$$

where,

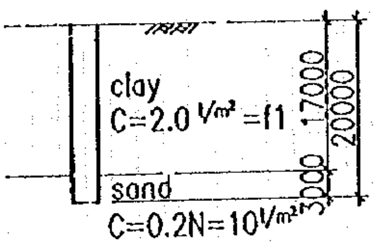
CB	:	Cohesion
A'	:	L x (B - 2e)
ϕ	:	Angle internal force

3.8 Design of Bored Pile Foundation

Bored pile with diameter ϕ800 shall be adopted in the Study.

This section describes only for the soil condition, bearing capacity and the sectional analysis of the bored pile body, since other design procedure is the same as spun pile foundation.

(1) Soil Condition



Therefore the allowable axial compressive bearing capacity ,Ra, shall be calculated based on the soil condition as below. (Refer to 3.3.5.2 in the Manual)

$$Ra = \frac{1}{n}(Qd \cdot Ab + U\sum lifi)$$

$$= (1 / 2.5) \times \{1{,}000 \times 0.503 + 2.513 \times (17.0 \times 5.0 + 3.0 \times 10.0)\}$$

$$= (1 / 2.5) \times (503.0 + 289.0)$$

$$= 316.8\ t \rightarrow 3105\ KN$$

where,

n	:	Safety factor : n=2.5
Ab	:	Area of concrete of a pile tip ϕ800: 0.503m2
U	:	Stratum depth with the skin friction ϕ800: 2.513m

(2) Sectional Analysis of Pile Body

Section analysis is only required for the ultimate limit state, and it shall be carried out by using circular section in the programme "BSDANM".

Input data are material, dimensions, reinforcement arrangement and sectional force obtained from the stability analysis of bored pile.

Required reinforcement for bending moment and shear force shall be determined, but minimum reinforcement as 0.5 % of cross section of pile should be arranged.

References

1. BS 5400: Steel, concrete and composite bridges, British Standards Institution, 1988

2. Departmental Standard BD37/88: Loads for highways bridges, The Department of Transport, London, 1989

3. BS 8004: Code of Practice for foundations, British Standards Institution, 1985

4. BS 8110: Structural use of concrete, British Standards Institution, 1985

5. Specifications for Highway Bridges, Japan Road Association, 1994

6. A Guide on Geometric Design of Roads, Jabatan Kerja Raya, 1986

DIVISION III

BRIDGE CONSTRUCTION PLAN AND COST ESTIMATE

DIVISION III BRIDGE CONSTRUCTION PLAN AND COST ESTIMATE

CONTENTS

DIVISION III BRIDGE CONSTRUCTION PLAN AND COST ESTIMATE

CHAPTER 1 GENERAL

This manual is prepared to assist engineers in understanding the construction plan/method and estimating procedure of standard bridge construction project.

The construction plan presents mainly current practices and problems on construction method and work approach of bridge construction, and it will relate to the cost estimate. However, materials and workmanship stipulated in the specifications are not described in this manual. Technical matter concerning concrete products and ready-mixed concrete are stipulated in Malaysian Standard (MS). Also, the following manual and guideline will be referred for site management and quality control of concrete.

- Construction Supervision Manual for Contract Roadworks (JKR, 1990)
- Guideline for Inspection and Testing of Roadworks (JKR, 1990)
- Manual on Quality Assurance in Concrete Works (JKR, 1991)

Bridge construction costs, even though the same in size, are varied depending on site condition, environment, construction method and location of the site. The accuracy of any cost estimate will depend on the amount of information known about the project. This cost estimate presents fundamental method and procedure which are universally applicable.

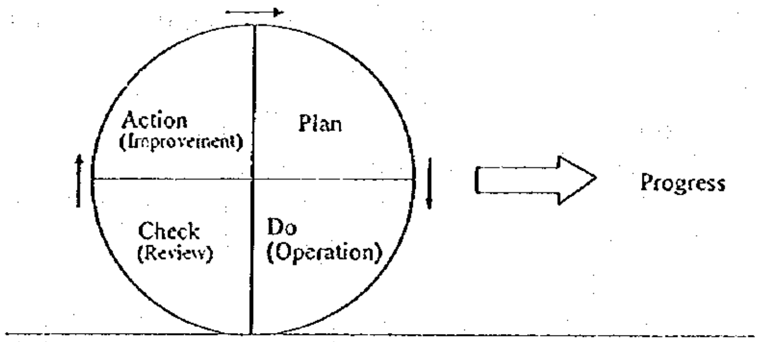
CHAPTER 2 CONSTRUCTION PLAN

2.1 Objective of Construction Plan

The construction plan is important and vital to the successful implementation of the works, and suitable construction plan will facilitate the construction.

Prior to starting actual construction, the construction plan should be studied for possible problems in order to complete the project in accordance with the Drawings and Specifications within the contract period and with sound engineering practice and government's budget.

The construction plan should be reviewed from time to time during construction, in accordance with the prevailing circumstances on the site, also if change in the contract or design are required during construction, the construction plan should be modified or corrected to meet the requirement. The cycle of construction plan is illustrated below.



2.2 Procedure of Construction Plan

2.2.1 Preliminary Study

The construction plan will be provided taking into account the scope and quantity of works, and site condition including environment.

(1) Scope and Quantity of Works

- Scope of works
- General drawings
- Major quantities

(2) Site Condition and Environment

-	Topography	Topographic condition Drainage condition
-	Geology	Geographic condition Soil condition and soil profile Assumed depth of foundation

- Climate — Annual rainfall and density
  Dry and wet season
- River — Highest and lowest water level
  Stream velocity
- Environment — Housing, hospital and school around the site
  Road traffic and railways
- Public utilities — Water, electricity, telephone, sewers, oil and gas

2.2.2 Items of Construction Plan

The following items are to be considered in planning decision to be made.

(1) Materials and Products

- Sources and availability
- Route of delivery

(2) Preliminary Works (Preparatory Works)

- Site office, storage and laboratory..........location and size
- Electricity and water supply
- Survey and laboratory equipment

(3) Construction Methods and Work Approach

- Temporary works ....Form and falseworks
  Cofferdams
  Temporary water control
  Temporary bridge and road
- Structure excavation and backfill
- Removal of existing structure
- Driving foundation piles .... Test pile of loading test
  Driving equipment
- Concrete.. Ready-mixed concrete or site mix
  Transportation and placing
- Prestressing...................................... Force and elongation of tendons
- Handling and launching beams.......... Launching method
- Miscellaneous works.......................... Bearing
  Handrail
  Expansion joint
  Drainage pipe

(4) Construction Plant and Equipment

- Selection and delivery to the site
- Kinds, numbers, period of use of equipment required
- Layout on the site
- Workshop

(5) Site Organization and Supervision

- Work assignment and their duties
- Nominated or specialist suppliers
- Nominated or specialist sub-contractors
- Committee and communication
- Scheduling of labour requirement

(6) Work Programme

- Sequence of works
- Production rate
- Effect of major public holidays and weather constraints
- Site traffic and traffic pattern

(7) Quality Control

- Technical inspection
- Quality control testing

(8) Construction Safety

- Cofferdams
- Handling and launching beams

(9) Public Relations
- Utilities..Diversion or protection
- Road traffic management
- Railways/highways.............................Construction operation of flyover

2.3 Flow Chart of Major Works

The flow charts of major works of bridge construction, substructure and superstructure (post-tensioned beam) are shown in Fig. 3.1 and Fig. 3.2 respectively.

PREPARATION WORKS

- Site Survey
- Temporary road and bridge
- Temporary water channel
- Relocation of public utilities

↓

STRUCTURE EXCAVATION

- Open excavation or cofferdam

↓

PILE DRIVING

- Test pile
- Pile driving
- Splicing of piles
- Pile cut-off

↓

FOOTING

- Leveling concrete
- Formwork
- Reinforcing steel
- Concrete placing
- Curing

↓

ABUTMENT AND PIER BODY

- Falsework and formwork
- Reinforcing steel
- Concrete placing
- Curing

↓

BACKFILL AND APPROACH ROAD

- Compaction of soil
- Road base and subgrade
- Slope protection

↓

RIVER BANK PROTECTION

- Stone masonry
- Concrete block
- Grouted riprap

Fig. 3.1 Work Flow of Substructure

PREPARATION WORKS

- Site Survey
- Manufacturing base for PC beams

↓

MANUFACTURING PC BEAM

- Formwork
- Reinforcing steel and ducts
- Concrete placing
- Curing

↓

PRESTRESSING

- Installing prestressing steel
- Calibration of jack
- Prestressing
- Grouting

↓

HANDLING AND LAUNCHING

- Setting bearings
- Lifting and transportation
- Launching and placing

↓

CROSS BEAM

- Formwork
- Reinforcing steel
- Tendons
- Concrete placing
- Curing
- Prestressing

↓

RC SLAB

- Formwork
- Reinforcing steel
- Concrete placing
- Curing

↓

MISCELLANEOUS WORKS

- Parapet and handrail
- Drainage pipes
- Expansion joints

↓

PAVEMENT

- Base course
- Wearing course

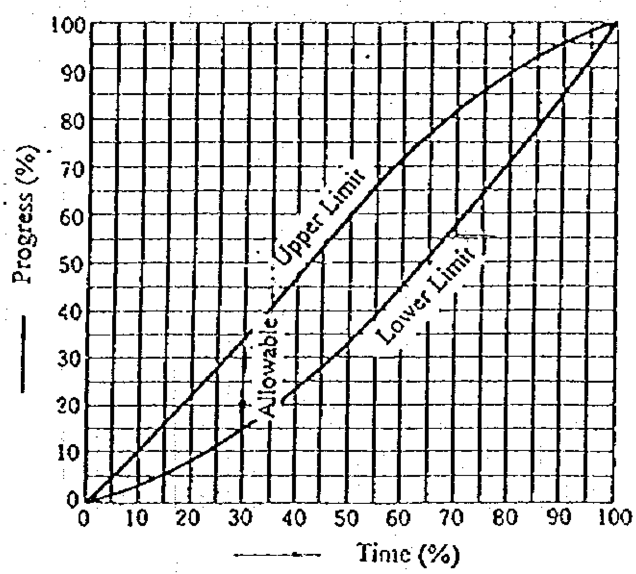
Fig. 3.2 Work Flow of Superstructure

2.4 Construction Programme

In order to estimate the programme of construction, the quantities of each works should be determined. It will estimated at the probable rate of the work to be performed considering available labour and equipment, allowing for estimated loss in time due to weather conditions, site condition or any other causes. From this information it will be possible to estimate the total time required to complete each work.

The estimated starting date and completion date for each work will be determined. In scheduling the activities it will consider the desirable sequential relationship between the activities. For example, in constructing a concrete foundation, it will be necessary to complete the excavation before concrete is placed, and concrete can not be placed until the forms have been erected and the reinforcing steel has been placed. Work programme will be presented usually in Bar Charts or Critical Path Method. Typical work programme of construction of pre-tensioned and post-tensioned concrete bridge are shown in Fig. 3.3.

The schedule control curve which is called S-cures or Bell-curves for road construction was produced by the statistical analysis in the California Division of Highway, USA is shown below.

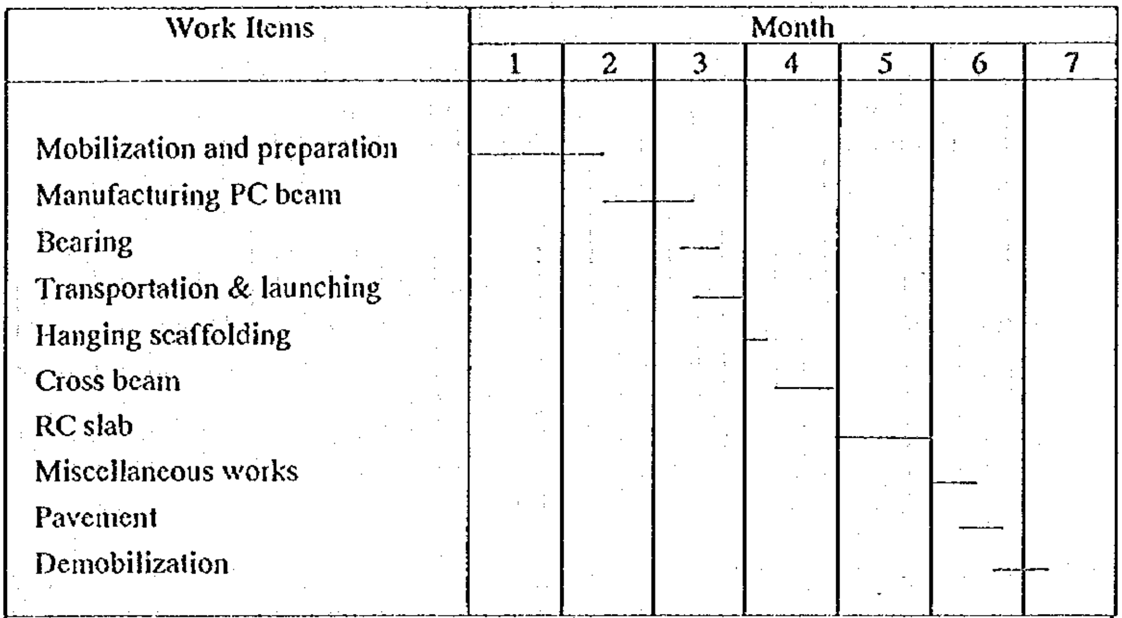


The lower line in this figure gives allowable progress rate, i.e. at 30% of time passing, an allowable rate of progress is 16-35%. If the progress of work is under 16%, it is necessary to take measures to recover from the critical condition, and if the progress is over 35%, present incorrect work programme is to be reviewed.

An example of work programme for road construction is shown in Fig. 3.4.

1. Pre-tensioned Concrete Bridge

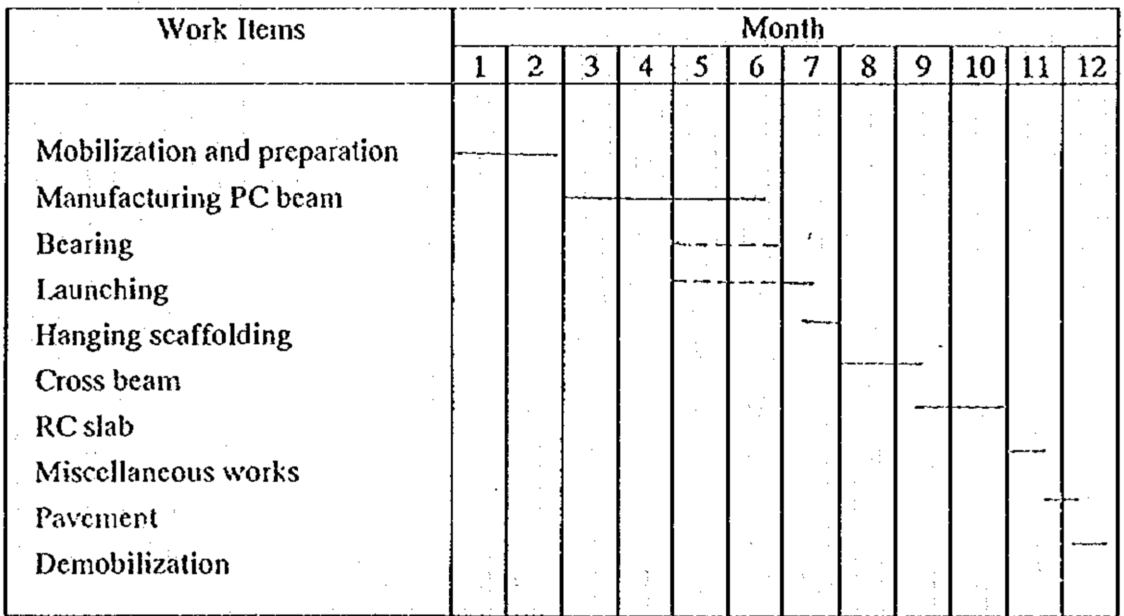
Bridge length : 3-span x 22.7m = 68.1m
No. of beam : 3 x 11 = 33 nos.



Remarks : Manufacturing PC beams in factory : 30 days

2. Post-tensioned Concrete Bridge

Bridge length : 3-span x 28.7m = 86.1m
No. of beam : 3 x 7 = 21 nos.



Remarks : Manufacturing PC beams on site
(16 days x 21 nos.) /3 - manufacturing bases = 112 days

Fig. 3.3 Work Programme of PC Bridge

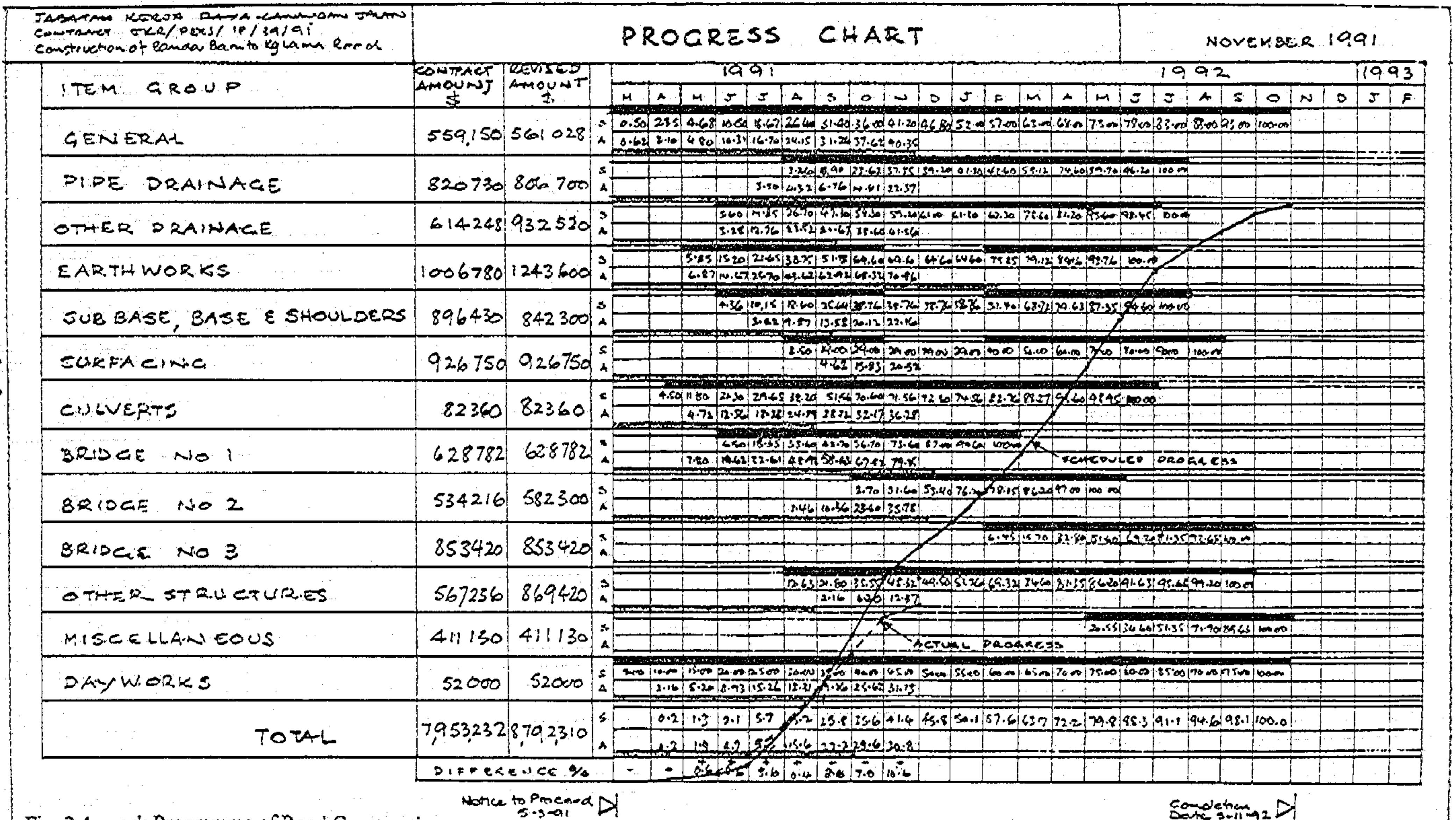


Fig. 3.4 work Programme of Road Construction

	M	A	M	J	J	A	S	O	N	D	J	F	M	A	M	J	J	A	S	O	N	D	J	F
SCHEDULED VALUE OF COMPLETED WORK $ x 10³	-	32	117	193	501	1225	2264	3129	3636	4027	4405	5064	5601	6352	7016	7764	8012	8321	8623	8792				
ACTUAL VALUE OF COMPLETED WORK $ x 10³	-	19	166	409	819	1672	1948	2512	2707															

2.5 Construction of Substructures

2.5.1 Structure Excavation

Foundation of substructures are constructed in open excavation if ground-water level is low, and if necessary, a cofferdam will be constructed to exclude earth and water from an area in order that work may be performed there under reasonable dry condition. Temporary water control consists of dikes, by-pass channels and flumes are carried out specially in dry season.

(1) Open Excavation

The following critical slope grade of excavated surface and depth may be maintained during excavation works, without any cofferdam and falseworks, if the ground-water level is low.

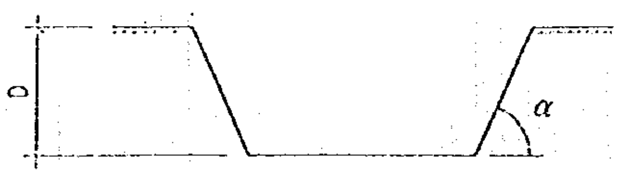
Table 3.1 Critical Slope Grade and Depth During Excavation

Type of Ground	Depth of Excavation (D)	Slope Grade (α)
Rock or Hard Clay	$D < 5m$ $D \geqq 5m$	$\alpha < 90°$ $\alpha < 75°$
Other Soil	$D < 2m$ $2m \leqq D < 5m$ $D \geqq 5m$	$\alpha < 90°$ $\alpha < 75°$ $\alpha < 60°$
Sand	$D < 5m$	$\alpha < 35°$

Generally, the depth of open excavation is taken from the following equation if ground-water level is low:

$$D = 2c/\gamma \cdot \tan(45° + \phi/2)$$

where; D : Depth of open excavation
C : Cohesion of soil (t/m2)
γ : Unit weight of soil (t/m3)
ϕ : Coefficient of internal friction (°)



(2) Temporary Cofferdam

Cofferdam should be safety designed and constructed, and be made as water-tight as it is necessary for the proper performance of the work which must be done inside. It is usually cheaper to permit some flow of water into the working area wherein water can be removed with pumps than to make the cofferdam watertight.

The interior dimensions of cofferdam should have sufficient clearance for the construction of forms for inspections and to permit pumping from outside the forms. Types of cofferdam are as follows:

Land Cofferdam	-	Wooden plate
	-	H-beam pile with wood plate
	-	Steel-sheet pile
Water Cofferdam	-	Earth or rock fill
	-	Steel-sheet pile

There are cantilever type and supported with crib type for wooden plate, H-beam and steel-sheet pile, according to depth of excavation, kind of soil and water level.

As a single wall of steel-sheet pile has limited strength in resisting the horizontal pressure of water or earth, it is necessary to provide an internal system of bracing to resist the pressure. If the dimensions across a cofferdams are not too great, rows of wales and cross braces will be satisfactory and economical.

The paths followed by particles of water flowing under a sheet-piling cofferdam are illustrated by lines 1, 2, 3, and 4 in Fig.3.5. As line 1 offers the shortest path, the velocity of flow will be the greatest along this line. If the velocity of the water entering the cofferdam is sufficiently high, it may agitate the sand and cause boils at the bottom of the pit, as indicated.

If an impervious blanket is placed outside and a berm of earth is placed inside a sheet-pile cofferdam as illustrated in Fig.3.5., the increased flow distance will reduce the value of the hydraulic gradient which will reduce the velocity and quantity of water flowing under the cofferdam. In addition to reducing the flow of water, the berm adds horizontal stability to the cofferdam.

2.5.2 Falsework and Form

(1) Loads.

Falseworks and forms are to be designed to have sufficient strength and stiffness for the forces which are likely to act on them.

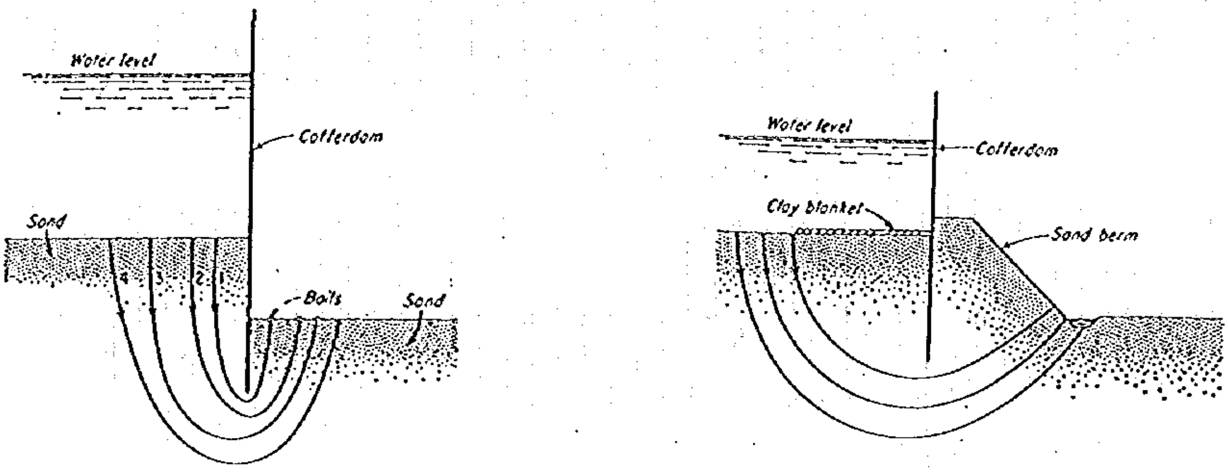


Fig.3.5 Water Flowing under Sheet Pile

(a) Vertical Forces

Weight of forms, concrete, workmen and equipment (including their impact) from concrete placing, temporary installations, and others.

(b) Horizontal Forces

Compaction of concrete, eccentric load and others.
Load act for falsework is generally assumed as 5% of the sum of dead loads.

When manufactured devices are employed, the design where in loads on such devices do not exceed the load ratings recommended by the manufacturer.

(2) Falsework

Timber or manufactured devices which are steel in frames, pipe supports, I or H shaped steel beam and truss beams are used for falsework.

Falseworks are to be constructed so as to transmit acting loads to the foundation, to allow its anticipated settlement and deflection and set camber for the vertical alignment. To adjust falsework with setting camber strips should use between falsework beams and soffit forms or screw jacks should use at each posts.

Foundation for falsework

- Concrete (thickness is 30 to 40cm)	Anticipated settlement
- Timber sleeper	to be 10 to 30 m/m
- Pile	depend on soil condition
- On footing	no settlement

Settlement of falsework itself is to be desirable within 10m/m. Displacement of joints of falsework is as follows:

Steel support - Steel support	0.5 to 1.0m/m
Steel support - Wood support	1.0 to 2.0m/m
Wood support - Wood support	1.0 to 2.0m/m

(3) Forms

Forms to be made of wood, plywood, steel or other materials shall be mortar tight and of sufficient rigidity to prevent distortion due to pressure during concrete operations.

Frequency of use (repeating use) of forms depend on handling and maintenance degree, however it is usually as follows:

Kind of Forms	Frequency of Use
Wood	3 to 5
Plywood	5 to 10
Steel	40
Aluminum alloy	100

(4) Lateral Concrete Pressure on Forms

The pressure on a vertical surface is less than hydrostatic pressure of the concrete. This may be due to the stiffening of the concrete as the cement hydrates and to the arching of the concrete against opposite vertical surfaces.

For design of forms, concrete pressure is to be determined as follows:

(a) In columns

$$P=0.8+(80R)/(T+20) \quad ; \quad \text{Maximum } 15t/m2 \text{ or } 2.4h\ t/m2$$

(b) In walls, with rate of placement not exceeding 2m/hr

$$P=0.8+(80R)/(T+20) \quad ; \quad \text{Maximum } 10t/m2 \text{ or } 2.4h\ t/m2$$

(c) In walls, with rate of placement greater than 2m/hr

$$P=0.8+(120+25R)/(T+20)$$

If the revibration, the external vibration of forms, the retarding admixture, the pozzolanic additions are used, appropriate adjustment for increased pressure can be made.

where;

P : lateral pressure of concrete (t/m2)
R : rate of concrete placing (m/hr)
T : temperature of concrete in the forms (deg. C)

h : height of fresh concrete above point considered (m)

Relationship between rate of placement and lateral concrete pressure is shown in Fig3.6.

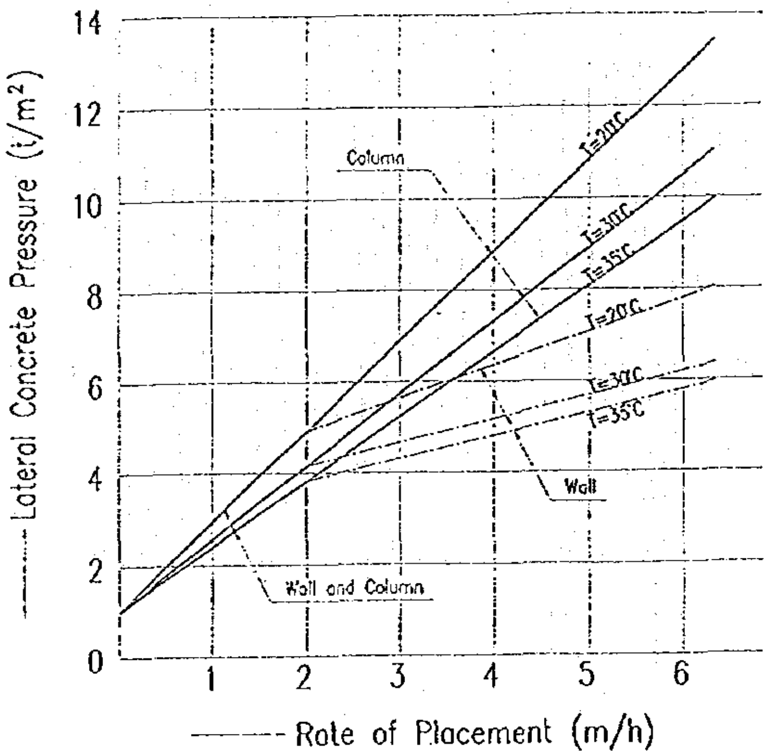


Fig.3.6 Lateral Concrete Pressure

2.5.3 Concrete

(1) Mixing Concrete

Concrete may be mixed at the site of construction, at a central plant, or wholly or part in truck mixers. When concrete will be mixed at the construction site, the mixers may be of the tilting-revolving drum or non-tilting-revolving drum type.

Table 3.2 presents the maximum output of various concrete mixers which may be useful to estimate the number of mixers needed at the site and the duration of concrete production.

(2) Transportation of Concrete

Once the mixed concrete arrives or batch mixing is finished at site, the fresh concrete shall be handled in such a manner that will maintain plasticity and avoid segregation. The transportation may be accomplished in several ways, depending upon the distance, elevation and other constraints imposed, as shown in Table 3.3.

Table. 3.2 Concrete Mixer

Wet Batch Capacity (m3)	Not-Tilting Type		Tilting Type	
	Power (kW)	Output per hour (m3)	Power (kW)	Output per hour (m3)
0.10	-	-	1	2
0.15	-	-	2.5	4.5
0.20	5	6	4	6
0.30	7	7.5	6	7.5
0.40	11	12	9	9
0.75	18.5	23	15	18
1.50	30	42	23	36
2.30	45	61	30	55
3.00	56	80	37	73

Table 3.3 Transportation Method

	Direction	Capacity (m3)	Maximum Distance (m)	Power	Remarks
Truck	Horizontal	1 to 10/no.	10,000	Engine	Access road
Fork lift	Horizontal	0.5 to 1.0/ no.	200	Motor	Access road
Agitator	Horizontal	1 to 10/no.	20,000	Engine	Long distance
Buggy	Horizontal	0.05 to 0.2/no.	50	Manual, Engine	Runway
Bucket	Vertical Horizontal	0.5 to 1.0/no.	50 25	With Crane	Crane
Pump	Horizontal Vertical	1.5 to 80/hr.	200 40	Engine	Bulk volume

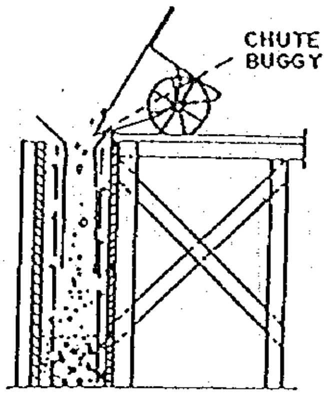
(3) Placing Concrete

To avoid segregation of the concrete materials and the displacement of the reinforcement, the long troughs, chutes, and pipes for conveying the concrete from the mixer or bucket to the forms will be used for placing concrete.

Fig. 3.7 demonstrate correct and incorrect handling method, and also show how to avoid segregation in placing concrete.

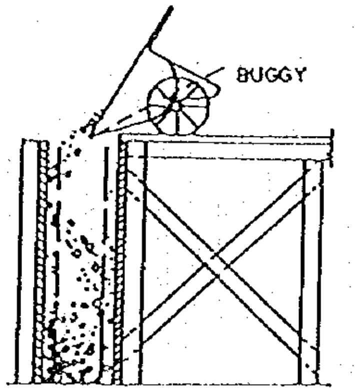
(4) Compaction of Concrete

The vibrators shall be inserted and withdrawn out of the concrete slowly and be of sufficient duration and intensity to thoroughly consolidate the concrete, but shall not be over vibrated so as to cause segregation. Fig. 3.8 demonstrate correct and incorrect compaction of concrete method.



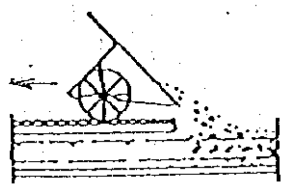
CORRECT

Separation is avoided discharging concrete into hopper feeding into drop chute. This arrangement also keeps forms and steel clean until concrete covers them.



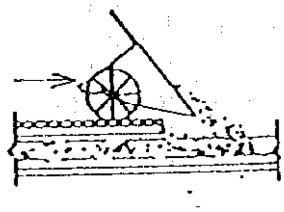
INCORRECT

permitting concrete from chute or buggy to strike against from and ricochet on bars and form faces causes separation and honeycomb at the bottom.



CORRECT

Concrete should be dumped into face of previously placed concrete.



INCORRECT

Dumping concrete away from previously placed concrete causes separation.

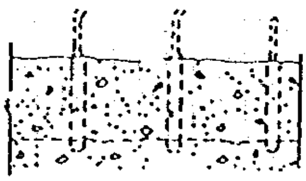
Fig. 3.7 Methods of Placing Concrete
(from ACI Manual of Concrete Practice)

CORRECT

Start placing at bottom of slope so that compaction is increased by weight of newly added concrete. Vibration consolidates the concrete.

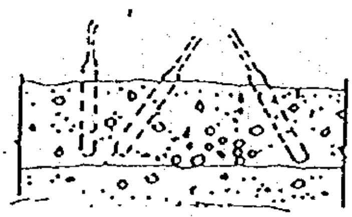
INCORRECT

When placing is begun at top of slope the upper concrete tends to pull apart especially when vibrated below as this starts flow and removes support from concrete above



CORRECT

Vertical penetration of vibrator a few inches into previous lift (which should not yet be rigid) at systematic regular intervals will give adequate consolidation.



INCORRECT

Haphazard random penetration of the vibrator at all angles and spacing without sufficient depth will not assure intimate combination of the two layers.

Fig. 3.8 Methods of Compaction of Concrete
(from ACI Manual of Concrete Practice)

2.5.4 Pile Driving

(1) Pile Driving Method

a) Percussion Driving Method

Many new pile driving methods have been developed but this method continues to be the most popular due to its performance, economy and efficiency. Piles may be driven with a drop hammer, an air/steam hammer or diesel hammer.

b) Vibration Method

The force is applied by vibro hammer continuously to overcome the skin friction of the pile and thus obtain penetration. This method was developed for driving sheet piles but, in the case of concrete piles, this method is applied when the required driving force becomes extreme by depending on the soil or when the piles are excessively long.

c) Excavation Method

This is one of the methods that have been developed to prevent the pollution of vibration, noise, etc. These methods include pre-boring methods and inside boring methods. In the pre-boring method, hoes are drilled ahead of time at the locations where piles are to be driven. Piles are lowered into these holes and then driven to refusal in order to obtain bearing strength. Bentonite or cement milk are often used to prevent cave-in of the hole where the ground is sandy or soft.

Compared with the percussion method, piles are sometimes bent or broken when the hole is not straight. Thus, great care is required in drilling holes. Control of the penetration depth is also difficult so a soil survey is required before starting the work.

d) Hydraulic Method

Machine manufacturers, etc., are developing low-noise hydraulic hammers having the same driving performance as diesel hammer. The noise level of all of these hydraulic hammers is below 80 phones (A) at a distance of 30m from the pile driving machine. A hydraulic hammer consists of a hammer, a power unit and a sound-proof cap. It lifts the ram by hydraulic pressure or supplied by the power unit and drops it freely to hit the pile and penetrate it.

(2) Selection of Hammer

Pile driving hammers are to be selected according to the soil conditions, kind of piles and size of pile. Standard hammer selection charts for precast concrete piles and steel pile are shown in Fig. 3.9.

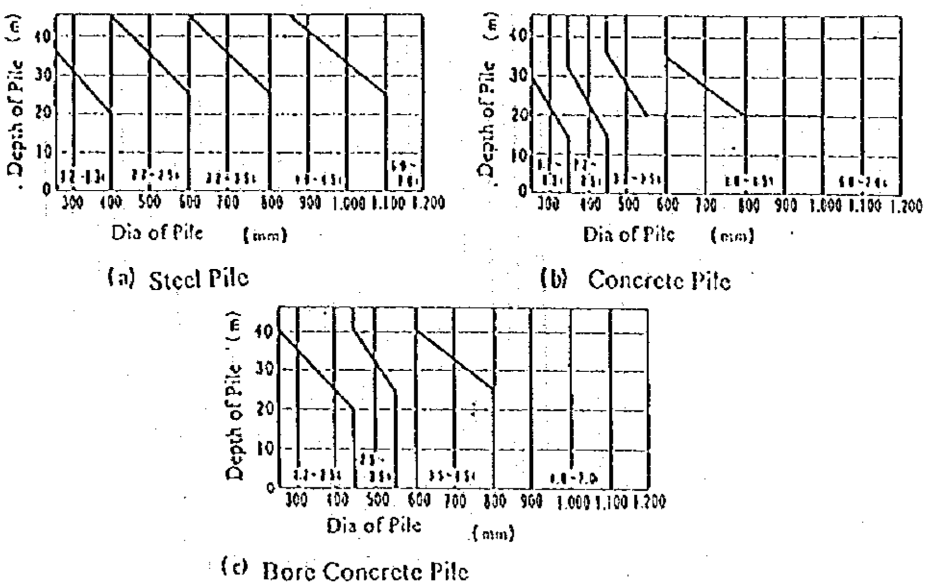


Fig.3.9 Hammer Selection Chart

(3) Sequence of Pile Driving

Driving resistance will be gradually increased due to compacted soil by driving piles. Therefore, pile driving work should be started from one side of pile group to the other side or from the center of pile group to outside.

(4) Determination of Bearing Capacity

When the driving method is adopted for precast pile, the bearing capacity can be estimated from the hammer conditions, also the pile penetration depth and the rebound can be measured. In general, the length of test pile will be greater than the estimated length of pile in order to provide variation in soil condition.

These method is usually attach to a recording sheet on the pile and hold a pencil as shown in Fig. 3.10. The bearing capacity of piles will be determined using following methods.

a) Hiley's formula

$$Ru = (ef \cdot Wh \cdot H)/(S+1/2K) \qquad Ra = 1/3\ Ru$$

where; Ru : Ultimate bearing load (t)
Ra : Allowable bearing load (t)
ef : Efficiency of hammer (diesel hammer 0.5)

(drop hammer 0.7)

Wh	:	Weight of hammer or ram (t)
H	:	Height of hammer (cm)(diesel hammer 2H)
S	:	Permanent settlement (cm)
K	:	Rebound value (cm)

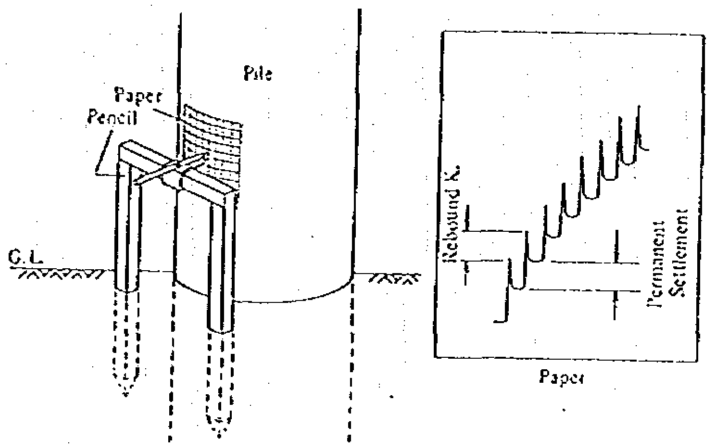


Fig.3.10 Measuring Penetration and Rebound per Blow

b) Wave equation formula

The equation is recommended in "Road Bridge Guideline", Japan

$$Ru = (A \cdot E \cdot K)/(eo \cdot l) + (N \cdot u \cdot l/ef) \qquad Ra = 1/3\ Ru$$

Pile driving work can be stopped at 2 to 10m/m of penetration values, if driving work continue under 2mm of penetration value, pile will be damaged.

where;

Ru	:	Ultimate bearing load (t)
Ra	:	Allowable bearing load (t)
A	:	Cross sectional area of pile (m2)
E	:	Young's modulus of pile (t/m2)
l	:	Pile length (m)
u	:	Distance around of pile (m)
N	:	Average standard penetration value of soil around pile
K	:	Rebound value (m)
eo,ef	:	Coefficient as follows:

Kind of pile	eo	ef
Concrete (PC,RC)	2 · WH/WP	2.5
Bored Concrete (PC)	4 · WH/WP	10
Steel	1.5 · WH/WP	2.5

WH : Weight of hammer
WP : Weight of pile

(5) Troubles During Pile Driving

Depending on types of soil layers, such a phenomena will possibly occur that furthermore driving becomes impossible befoie a pile reaches an intended bearing layer of finishing requirements are not met even when the pile is driven to a specified depth, as described below. In such cases, design and construction conditions shall be fully reviewed to cope with the events.

a) If it is impossible to drive a pile through an intermediate layer before it reaches a bearing one, or if further, driving is difficult due to excessively large compacting effect of earth:

   It seems that bearing requirements are met as far as axial bearing capacity is concerned. If depth of intermediate layer is more than 5m, it is considered as bearing stratum. However, the pile's embedded depth may have been determined at the design stage by scouring, lateral resistance, consolidated settlement of cohesive layers, and other requirements. It is necessary, therefore, to take such measures as using a larger hammer, apply a pile-boring method in addition to the pile-driving method, or mount a friction cutter at the pile tip.

b) If it is found difficult to make further driving to reach a specified depth, because in intended bearing layer is located at a shallower depth:

   This case in which it is found difficult to make further driving because an intended bearing layer is located at a shallower depth than expected are often seen when a construction area extends over a wide area or when construction is effected at a catchment basin of a dried river. Although, it is difficult to identify these cases from those caused by other factors, comprehensive judgment shall be made by increasing a number of boring tests and comparing with driving depth of other piles.

c) If finishing requirements are not met even by driving a pile to a specified depth:

   To cope with this case, an additional pile is often joined and driven. If it is impossible to splice/joint additional piles due to properties of materials, other measures shall be adopted by taking design conditions into account.

   It is impossible to unconditionally define penetration for finishing hammer and an appropriate hammering frequency to drive a pile, by ignoring differences in types, lengths, and shapes of piles and soil layer conditions. By referring to past data, however, it is concluded that penetration for finishing hammer shall be 2 to 10mm.

2.5.5 Pile Loading Test

(1) Purpose of Loading Test

Pile capacity can be estimated by using various dynamic or static formulas based on site investigation results. However, due to several inherent uncertaines and inaccuracies in these methods, it is a usual practice to verify the load carrying capacity of piles by loading tests.

Generally the main objectives of loading tests are:

- To determine the load-settlement characteristics of piles
- To check the ultimate capacity
- To check the structural soundness of pile foundation

(2) Type of Loading Test

Three common types of compressive loading test on piles are:

- Constant Rate of Penetration (CRP) Test, in which the compressive force is progressively increased to cause the pile to penetrate the soil at a constant rate.
- Maintained Load (ML) Test, in which the load is increased up to twice the design load in stages to some multiple, - settlement curve recorded at each stage of loading and unloading.

- Pile dinamic analysis (PDA) Test.

CRP and ML tests use the same type of loading arrangements and pile preparation. Suitable loading arrangements for applying the load to the pile by a hydraulic jack using as the reaction, either kentledge blocks.

(3) Times before Load Test

As long as possible, sufficient time shall be allowed before carrying out the load test after installation because the skin friction recover in early stage. "Construction Standard", Japan recommended that pile in clay should be tested more than 14 days after driving piles and for piles in sand 5 days.

(4) Evaluation of Loading Test Results

a) Ultimate Load

- The load at which settlement continues to increase without any further increase of load.
- The load causing gross settlement of 10% of the pile width(Terzaghi).

b) Allowable Load

A criteria for determination of the allowable working load from loading test results are:-

- Less than allowable stress of pile; or
- Yield load x (1/2 to 1/3); or
- Ultimate load x (1/3 to 1/4) whichever is the lowest value

The test results are summarized in the form of load-settlement curve for the CRP test and a load-time-settlement curve for the ML test.

Yield load can be judged from log P-log S curve, S-log t curve and $\triangle S/\triangle \log t$ - P curve.

2.6 Construction of Superstructure

2.6.1 Manufacturing Pre-tensioned Beam

In order to reduce beam sections or dead load of beam, the partial debonded strand method and deflected strand method are employed for hollow slab and composite T-beam respectively.

It can avoid tensile stresses at the top at supports by preventing bond for some of the tendons at a computed length near the ends by covering the strands with plastic tube. The deflected strand method needs some additional investment on the plant to provide for hold-downs and special equipment for raising the strands.

Both methods and anchorage for deflected strand are shown in Fig.3.11 and Fig.3.12.

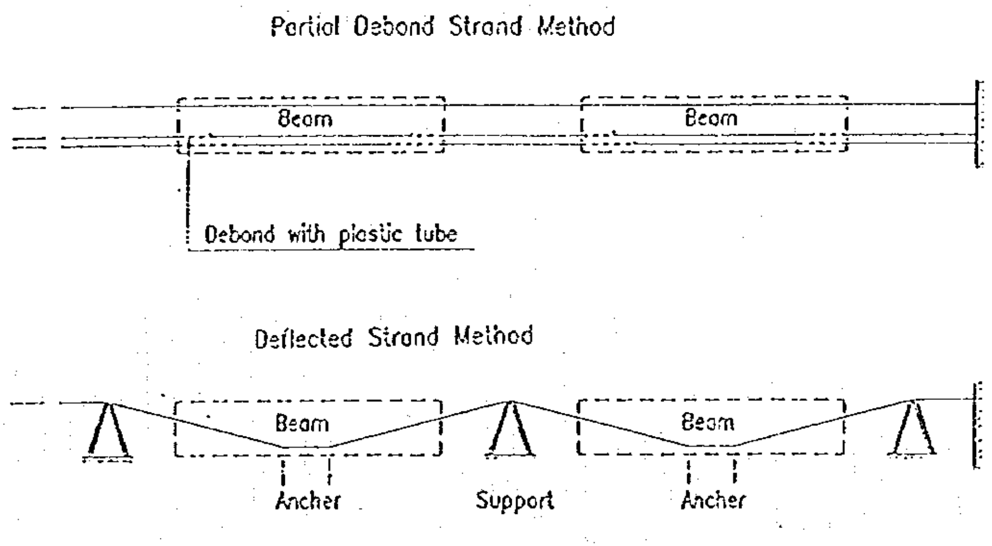


Fig.3.11 Partial Debond and Deflected Strand Method

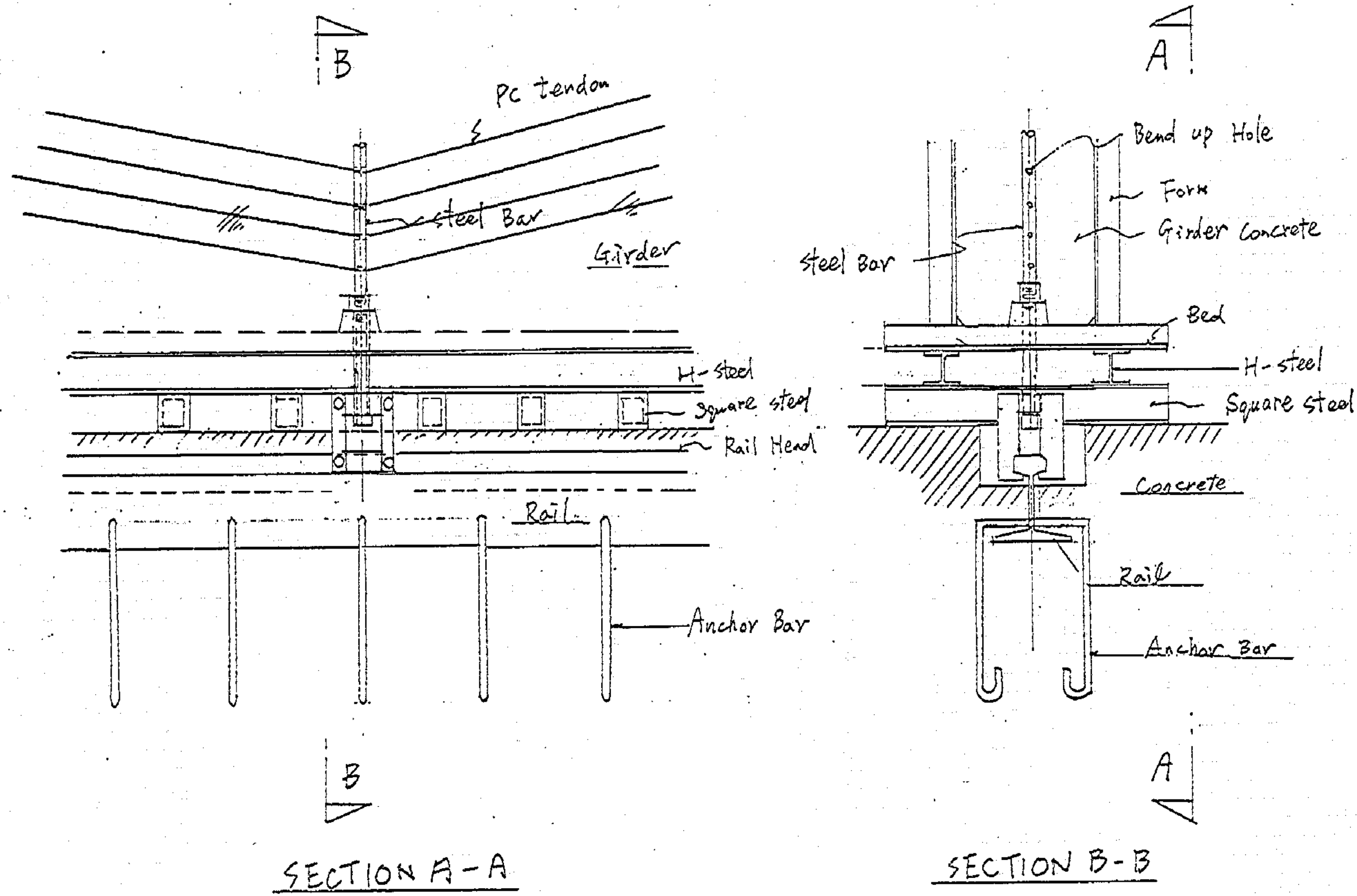


Fig. 3.12 Anchorage of Deflected Strand Method

2.6.2 Fabrication Yard for Post-tensioned Beam

The site of the precasting plant should be of adequate size including transport vehicles and handling equipment, and allow satisfactory manufacture and storage space of girders. The plant site should be flat and trafficable areas, and good drainage in the whole site is necessary. The energy for operating the plant and water supply for concrete mixing and curing should be available. The plant site is usually selected on the bridge approach road or at the river bed under the planned bridge.

The following facilities are provided on the plant site and their layout is shown in Fig. 3.13.

- Girder production yard
- Girder stock yard
- Concrete plant (if ready mixed concrete is not available)
- Material
- Office and storage
- Handling and transport

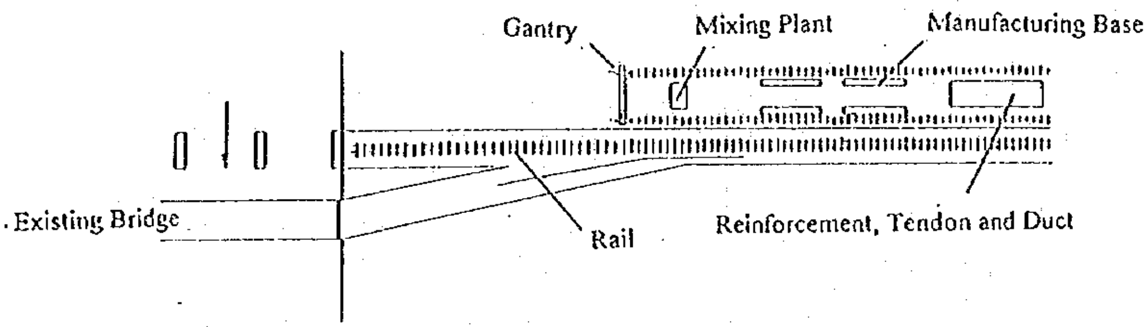


Fig.3.13 Fablication Yard for Post-tensioned Beam

2.6.3 Form

(1) Preparation of Form

Depending on the frequency of use (number of use of girder), the following types of formwork can suitably be used for post-tensioned T-beam.

Frequency of Use (number of girders)	Plywood Form	Steel Form
n < 10	1 set	
10 < n < 20	2 sets	
20 < n < 30		1 set
30 < n < 60		2 sets

The thickness of form usually used are as follows:

Plywood t = 12 to 21 m/m
Steel t = 2.0 ± 0.2 m/m
Aluminum alloy t = 3.5 ± 0.2 m/m

Since the upper flange and web width of beams are in same size, a steel form with height adjusting devices may be used for beams of varied height. Sample of the forms are shown in Fig. 3.14.

(2) Base of Form

During prestressing, the uniform reaction on base move to both end of T-beam, and at the same time shortening of beam occur (elastic deformation). Therefore, the form shall be so designed that it does not restrain the shrinkage movement and possible shortening due to prestressing and that the foundation around the end of beam should be strong enough for settlement against reaction. Structure of base around end of bearing is shown in Fig. 3.15.

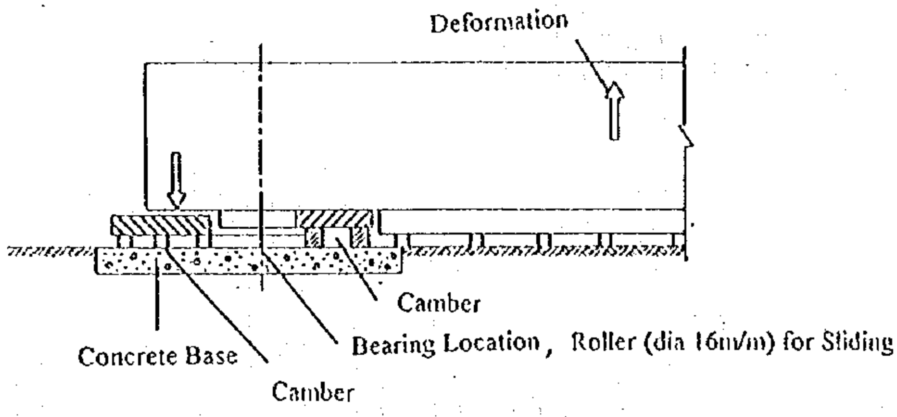
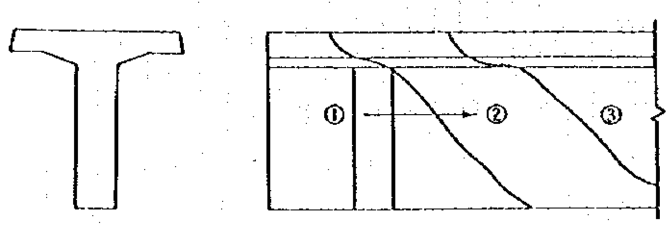


Fig.3.15 Structure of Base of Beam End

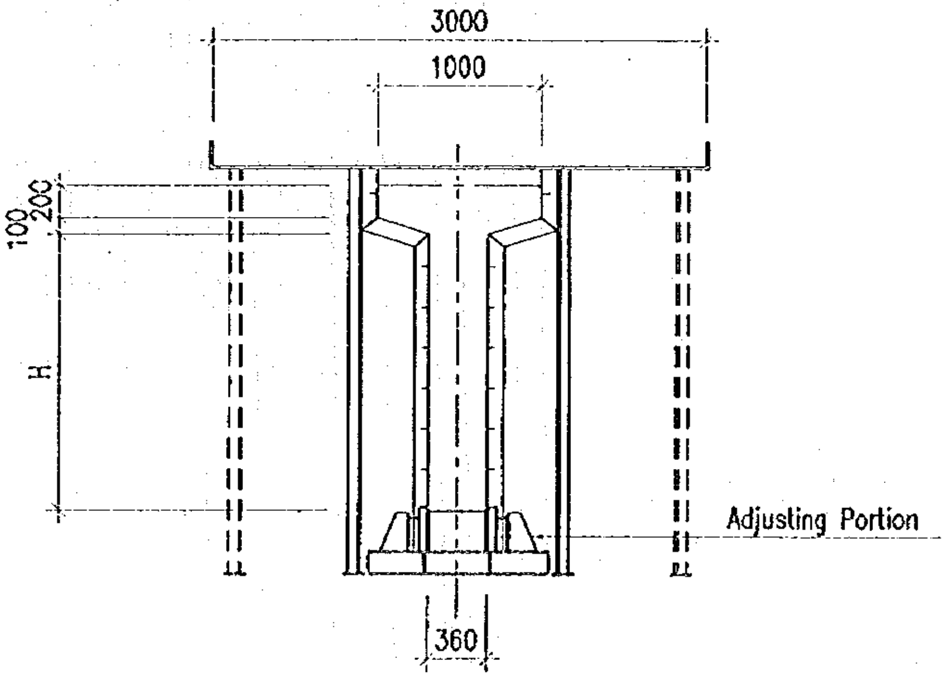
2.6.4 Concrete

Stirrups and link bars should be tied together with tie wire at the correct position to avoid displacement during concreting with heavy vibration, because ducts are supported and fixed at proper location with stirrups. Deformation of reinforcement and duct is shown in Fig. 3.16. Ducts shall be in smooth alignment without sudden changes in direction.

Concrete for T-beam whose depth is less than 2m may be placed in one continuous operation, however, for depth of 2m or more, may be placed in two separate operation; first, web of beam, and second, the flange of beam.



Adjusting Height by Sliding Form



Adjusting Height by Additional Form

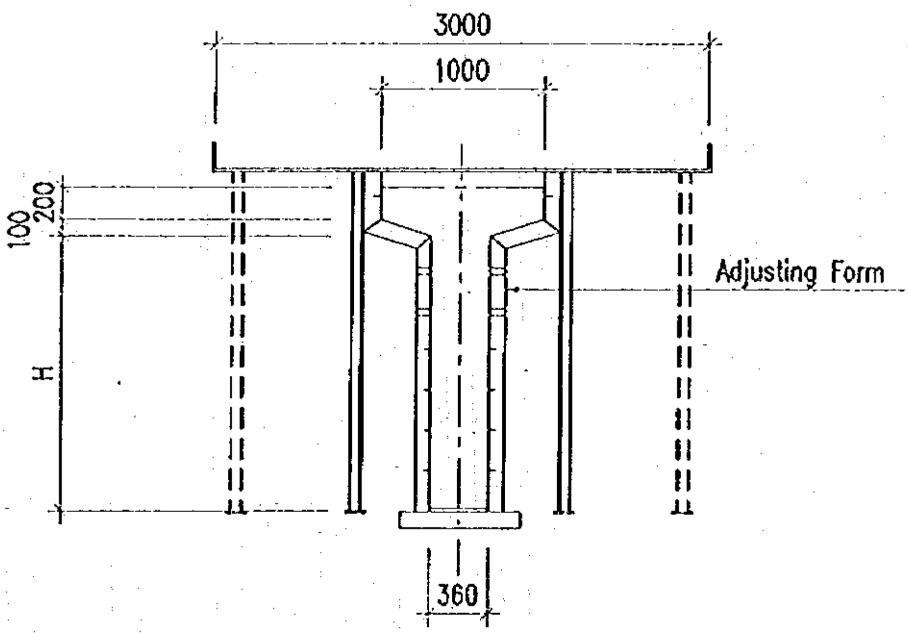


Fig. 3.14 Sample of Steel Form

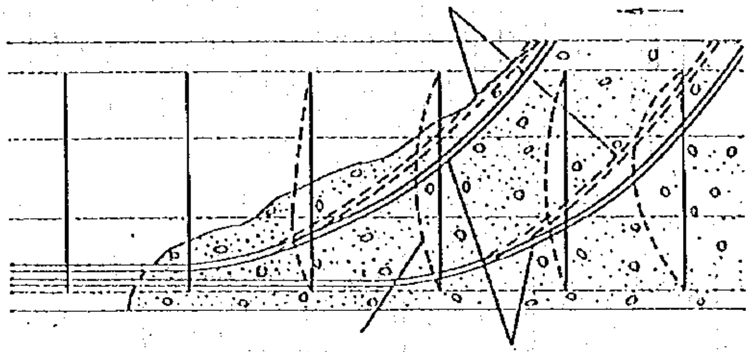


Fig3.16 Deformation of Duct and Stirrup

2.6.5 Prestressing

(1) Site Tensioning Operation

The tensioning operation will be carried out by suitably experienced supervisors and operators (specialist sub-contractor).

The instructions controlling the tensioning operations are based on the design of the structure and may be modified by the engineer as a result of the information obtained from site tensioning records.

The instructions are as follows:-

- Minimum concrete strength at the time of tensioning
- Whether tendon is stressed from one or both sides
- Tensioning sequence
- Tensioning forces and elongation

Tensioning of tendon should be done in sequence to introduce a minimum eccentric force in the beam section. In calculation of anticipated elongation of each tendons, cable friction, jack and anchorage friction and elastic deformation of the concrete beam are to be taken into account.

Prior to the tensioning operation, a specialist shall ensure the following matters:

(a) Understanding on Design Calculations

The relation between design and construction work of the prestressed concrete is much close than that of other ordinary reinforced concrete. And particularly, with respect to the design requirements and working drawings, prestressed concrete members are to be prestressed, the basic design contents shall be fully understood.

(b) Equipment

In carrying out the work, the tensioning equipment should be appropriate and operated in accordance with the system manufacture's instructions.

(c) Force Measuring

The pressure gauge of jacks are required to calibrated by dynamometer in order that the magnitude of the tensioning forces can be adequately controlled. This calibration must be done periodically since they tend to easily become inaccurate.

(d) Safety

Tensioning works can be dangerous. If a tendon breaks or slips there is a sudden and large release of energy which is likely to cause the tendon to be projected from the duct with a considerable force. Precautions must be taken to ensure that stressing is carried out safely.

(2) Control of Stressing Operations

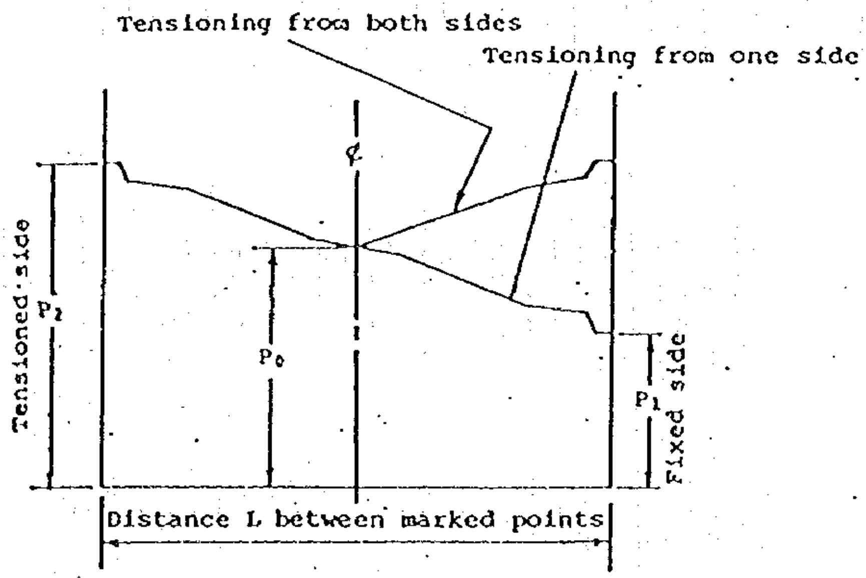
The recorded force and elongation on the site should be compared with the calculated value. If measured and calculated values are in close agreement, the friction losses assumed in the design are confirmed. Where tolerance has not been specified, a realistic value is normally ±6%, based on BS and FIP specification.

Valuation of forces and elongation may be arised from:

- Variation in internal friction in the jack
- Error in measuring elongation
- Variation in cross-sectional area in modulus of elasticity of tendon
- Difference between design and actual coefficients of friction
- Variation in friction in the anchorage

(3) Friction Tests

Two jacks are used for measuring actual friction, the force is applied via one end and the other registering the passive force at the other side of the tendon. The distribution of tension along the duct during prestressing and the relation between the forces at the tensioned side and fixed side in the friction measurement are shown below.



(4) Failures during Tensioning

Troubles and accidents during prestressing occur mainly on concrete, ducts, steel materials to be tensioned, anchoring devices and connecting devices, and the causes are the defective materials, design and construction, or a combination of these.

The cause of troubles or accidents during tensioning are sometimes clear (inevitable) and sometimes not clear (accidental). The inevitability occurs due to errors or carelessness during design and construction work, while the accidental troubles occur due to irresistible force which is not known by theory or calculations. However, such irresistible force seems to act when various kinds of causes with potential problems are overlapped.

(a) Broken Wire during Tensioning

During tensioning of strand, individual wire may be broken and pulled out, mostly caused by the improper installation of ducts and female cone not aligned after pouring of concrete.

In case of the area of broken wires does not exceed 2% (based on the AASHTO) of the total area of the prestressing steel in the member, the failures may be normally accepted by the Engineer. If there are some allowance in tension, additional tension equal to the force should be applied to the remaining wires as an emergency measure.

(b) Broken of Anchored Portion

Broken of anchored portion, such as abnormal setting, accident of wedge and depression of female cone may be occurred during tensioning. If troubles occur, the wedge or female cone should be taken out and replaced with new one.

(c) Cracks in Concrete

These cracks are occurred near anchoring portion during or after tensioning. Cracks are generally caused by insufficient amount of reinforcing bars, incorrect prestressing (sequence, tension, etc.) incorrect concrete work (mix, pouring method, curing, timbering, etc.), and combination of these, but it is extremely difficult to detect the actual cause of the cracks in many cases.

In order to avoid the trouble, grid-shaped reinforcing bars shall be increased around anchor portion.

(d) Leak of Mortar in Ducts

During tensioning from both sides, the operation of pumps is adjusted to assure the same elongation at both sides. If there is a great difference in elongation even after such adjustment, or the total elongation is extremely small, a leak of mortar and plugged duct shall be suspected.

Cause of such troubles are improperly wound tape at the joint of ducts, damaged duct due to improper handling, break of duct due to contact of bar-type vibrator during placement of concrete.

Measures for this trouble as follows:

- Ducts should be placed properly so as not to damage and connecting of ducts should be carried out perfectly.

- Immediately after placing the concrete, the plugging can be detected by moving the steel cable back and forth or by introducing air to the ducts from an air compressor.

- If plugged duct is found after tensioning, work must be stopped since occasionally prestress cannot be given, and the engineer in charge should take proper measures in removing the mortar.

  For example, if the mortar cannot be removed even after repeating the tensioning, probable location of mortar should be detected by inserting the steel wire of ϕ 2.9 from both ends of beam, and then duct should be cut out to remove the mortar.

In this case, full precautions are needed for not damaging steel wires of the cable. After removal of mortar, this portion should be covered with new duct and cleaned before pouring concrete.

2.6.6 Handling and Launching

(1) Launching Method

The precast prestressed concrete beams are usually launched in position using truck cranes or launching steel girders with gantry crane or bent. The following matters are significant for proper selection of launching method for the precast concrete beams.

- Geography of the site
- Soil conditions
- Condition of fabrication yard (location, width and length)
- Weight of beam
- Availability of specialized and/or heavy equipment
- Accessibility of the work areas
- Obstacles above and below the beam to be launched
- Safety of handling and launching

For selecting launching method, the methods in the Table 3.4 are marked ◎, ○and △ in order of their frequency of employment.

(2) Launching by Truck Crane

This method is suitable for bridges where the truck crane can either be stationed closely to the bridge abutments or piers, or between the bridge pier. The required capacities of cranes depend on applying of both length and working radius of crane booms.

(a) Launching by Single Truck Crane

A truck crane can lift beam and launch them on position and it is suitable for small scale bridge as pre-tensioned concrete beams for weights under 15 tons. In case of truck crane stationed between bridge piers, the lifting capacity of crane required is about 2 to 3 times of the weight of beams (30 to 45 tons truck crane). In the case of the truck crane stationed just behind of bridge abutment, the lifting capacity of crane required is about 6 to 8 times of the weight of the beams (90 to 120 tons truck crane).

Launching method is illustrated in Fig. 3.17.

Table 3.4 Adaptability of Launching Method of Precast Beam

Condition	Launching Method	Launching steel girder: Two steel girders	Hanging from one steel girder	One steel girder and two portal cranes	Two steel girders and shifting	One steel girder and shifting	Carrying method	Crane: Single truck crane	Two truck cranes	Floating crane	Gantry crane: Fixed gantry crane	Self-traveling gantry crane	Bent: Bent-type	Traveling bent
Fabrication yard	On approach road or on viaduct	◎	◎	◎			○	△	○			△	○	○
	Under or near the bridge				◎	◎	○	◎	◎		○	◎		
Space over and/or under the bridge	Obstacle over the bridge	◎	○	○	◎	◎	○		△		△	△	◎	◎
	River under the bridge	◎	◎	◎			○							
	Rail way under the bridge	◎	◎	○			○		△					
	Road under the bridge	◎	◎	○	○	○	○	○	○		△			
Weight of PC beam	Light (under 10t)	○	○	○	○	○	○	◎	○	○	○	○	○	○
	Heavy	◎	◎	◎	◎	◎	◎	△	◎	○	○	○	○	○
	Very heavy (over 100t)	◎	○	○	◎	○	△		○	○	◎	○		◎
Schedule	Necessary to be in stock	◎	○	○	◎	◎	◎	◎	◎	◎	○	◎	△	△
	Launching without stock	○	○	○	○	○	○				○	○	○	○
	Immediately launching after casting the bridge slab	△	△	△	◎	◎	◎	◎	◎	◎	◎	◎	△	△
Size and figure of pier	No space for support (transverse)	○	○	○	○	○	○	◎	◎	○	◎	◎		
	No space for support (longitudinal)	◎	○	○	○	○	○	○	○	○	◎	◎		
Curvature and slope	Curvature	◎	○	○	◎	○	○	◎	◎	◎	◎	◎	○	○
	Large slope	○	△	○	○	○		○	○	○	○	○	○	○
Others	Large skew	◎	△	○	◎	△	◎	◎	◎	◎	◎	◎	○	△
	Over river or sea	○			◎	◎	○			◎				

Note: ◎ Frequent
○ Sometime
△ Seldom

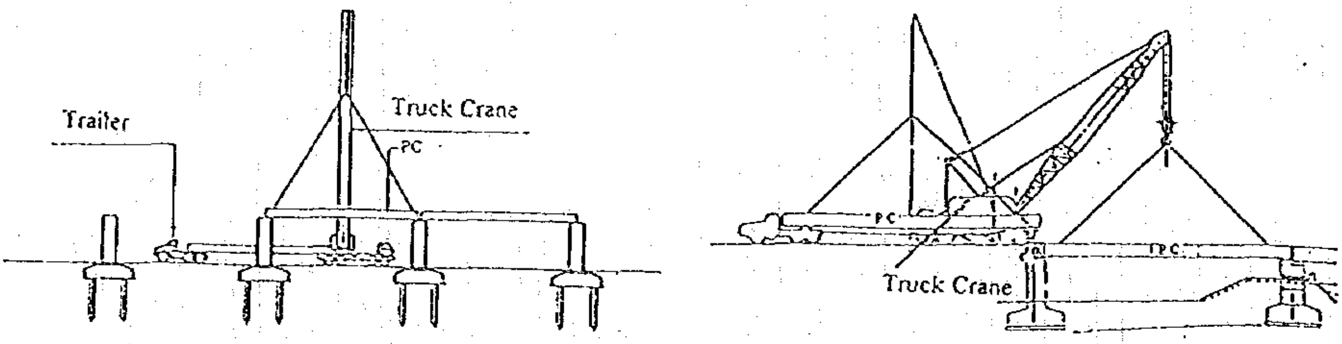


Fig.3.17 Launching by Single Truck Crane

(b) Launching by Two Track Crane

In this method, the launching girder is usually placed between the bridge span on which precast bridge beams are carried and placed on abutments or piers by 2 truck cranes. The lifting capacity of crane usually required is about 1.5 to 2.0 times of the weight of beam. Taking availability of heavy truck cranes and safety of launching works into consideration, this method is applicable for weights under 80 tons precast bridge beams. This method is usually applied for weight of 15 to 70 tons precast beams, and is used for over 70 tons as in special case. Applicable truck cranes for launching precast bridge beams are shown in Table 3.5 and launching method is illustrated in Fig. 3.18.

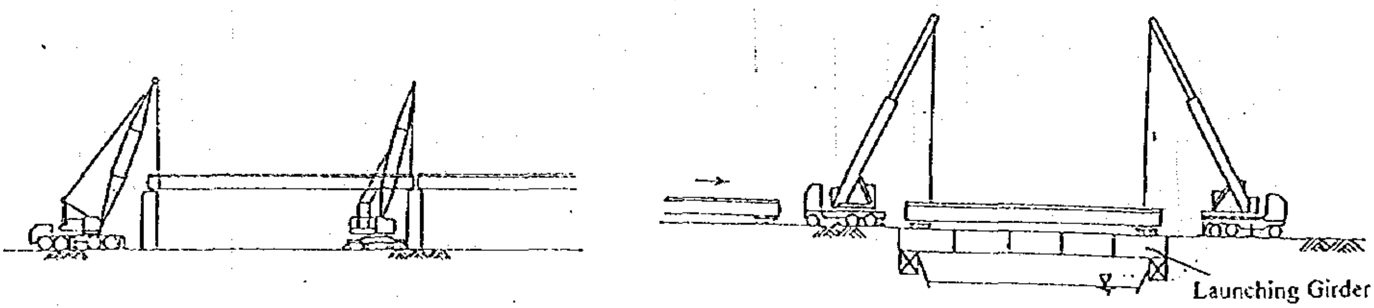


Fig.3.18 Launching by Two Track Crane

Table 3.5 Applicable Truck Crane for Launching Beams

Length of PC Beam	Weight of PC Beam	Applicable Lifting Capacities of Truck Crane
15 - 18m	15 - 20 tons	(30-40 tons) * 2 cranes
20 - 22m	25 - 30 tons	(45-60 tons) * 2 cranes
25 - 30m	50 - 70 tons	(90-130 tons) * 2 cranes

(3) Launching by Specialized Launching Girder

This method is suitable for launching post-tensioned beams with lengths of 25m up to 45m (weight of 50 - 135 tons) and for specialized launching girders in succession over several spans. Launching is not influenced by the height of the piers or the existence of a waterway, and there is no restriction on the ground condition under the launching girders. Transportation, assembling and disassembling costs for steel girder are necessary, therefore, it is not economical for short span beams or a few spans bridge.

Two types of specialized launching girder are shown in Fig. 3.19 and Fig. 3.20.

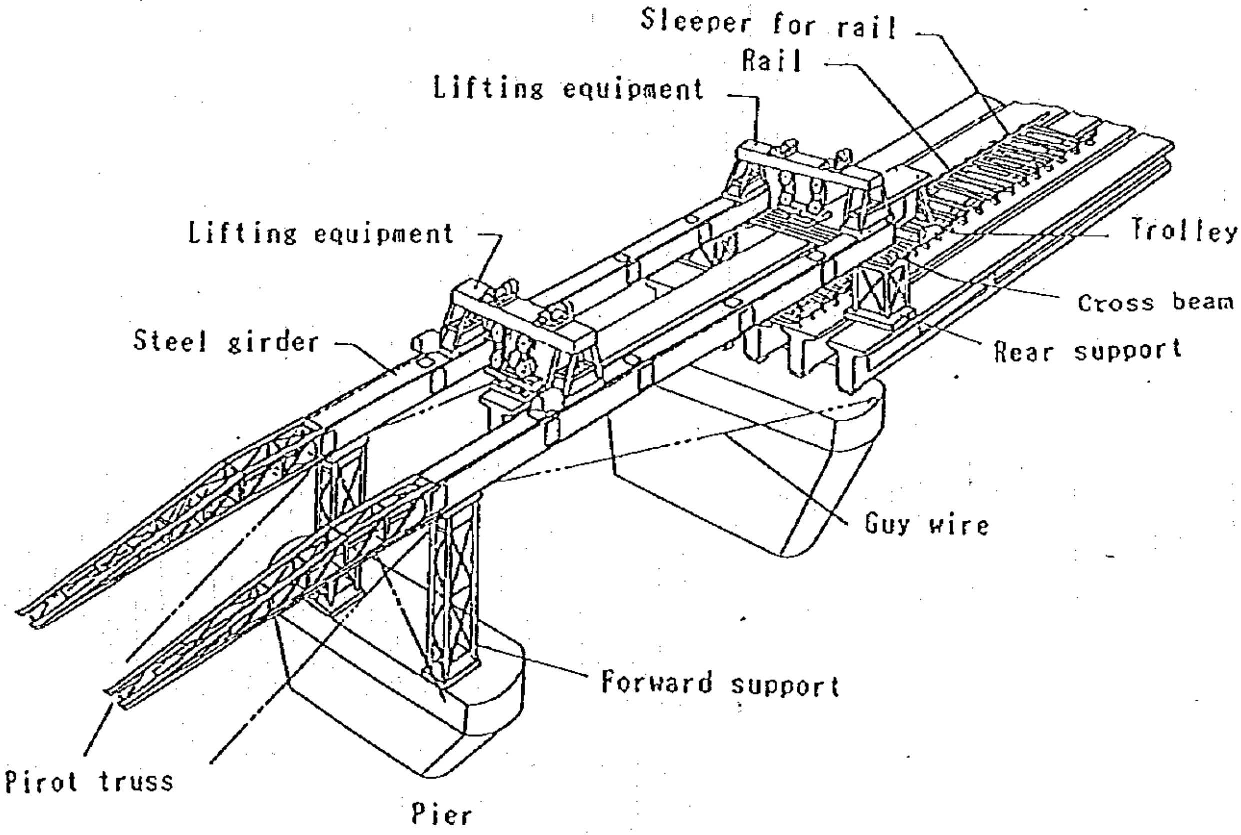


Fig.3.19 Launching by Two Steel Girders

(4) Launching by Gantry Crane

In this method, a pair of gantry crane straddling projected two piers lifts up and places into position the precast beams which have been delivered by a rail or a trailer truck running alongside of the projected span.

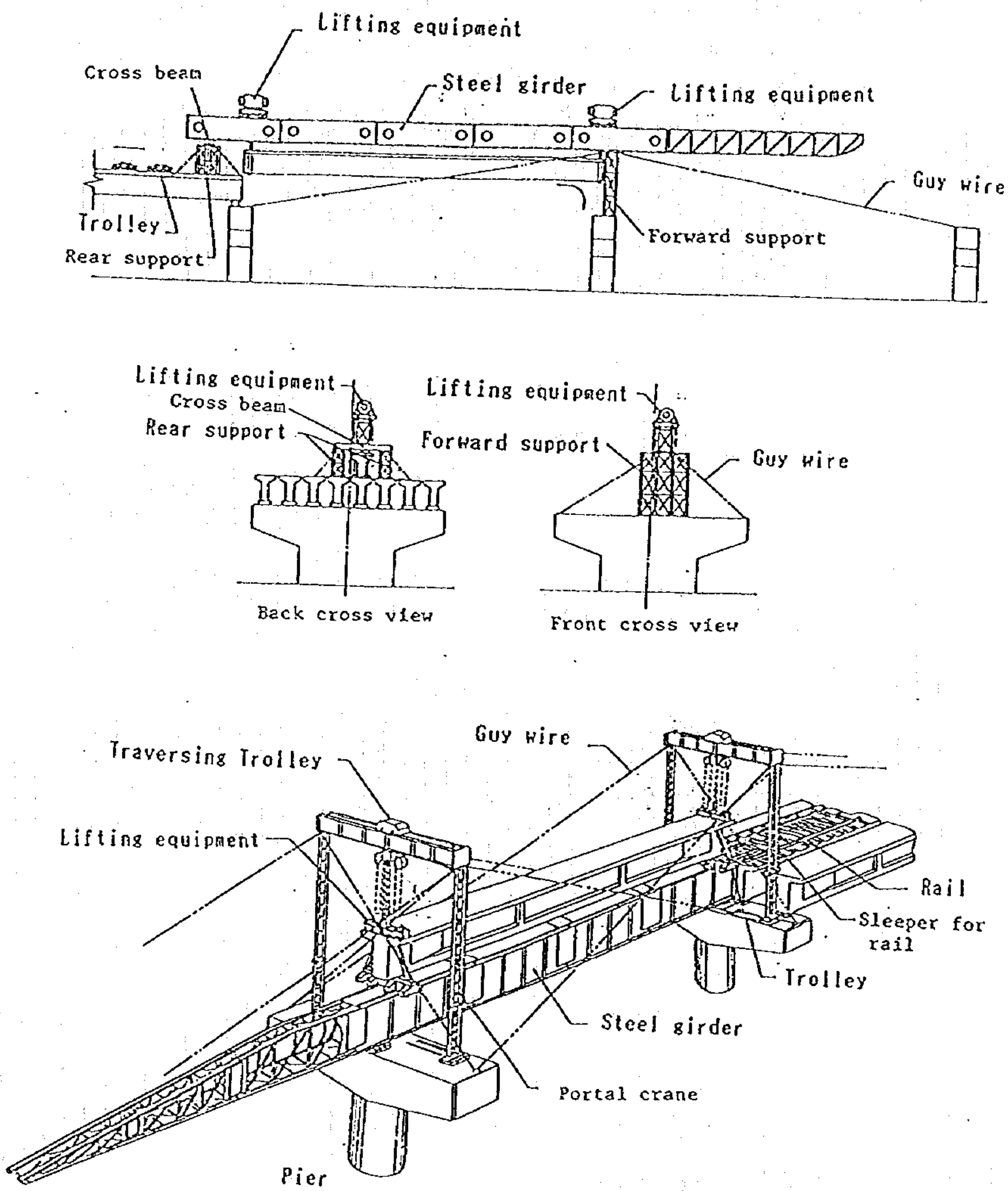


Fig. 3.20 Launching by Single Steel Girder

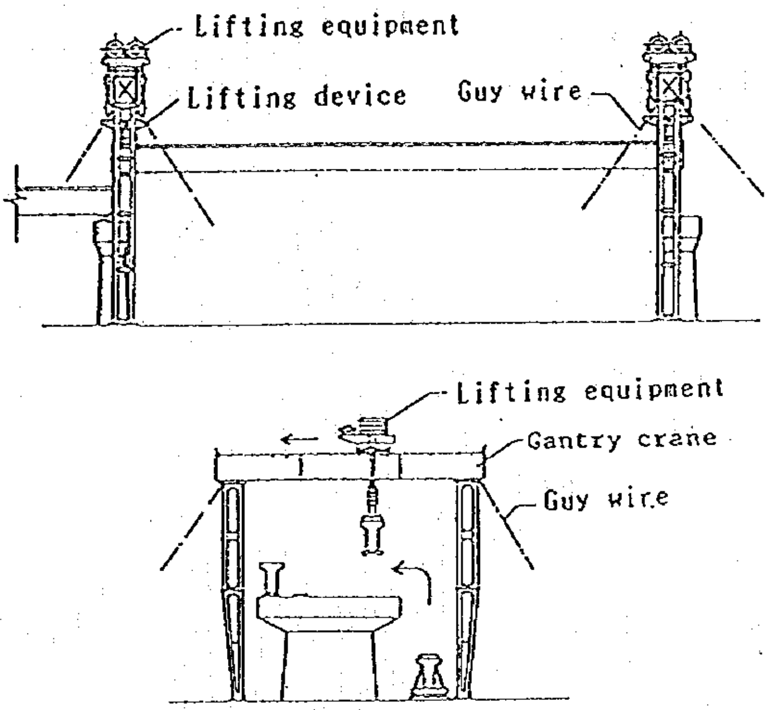


Fig. 3.21 Fixed Gantry Crane Method

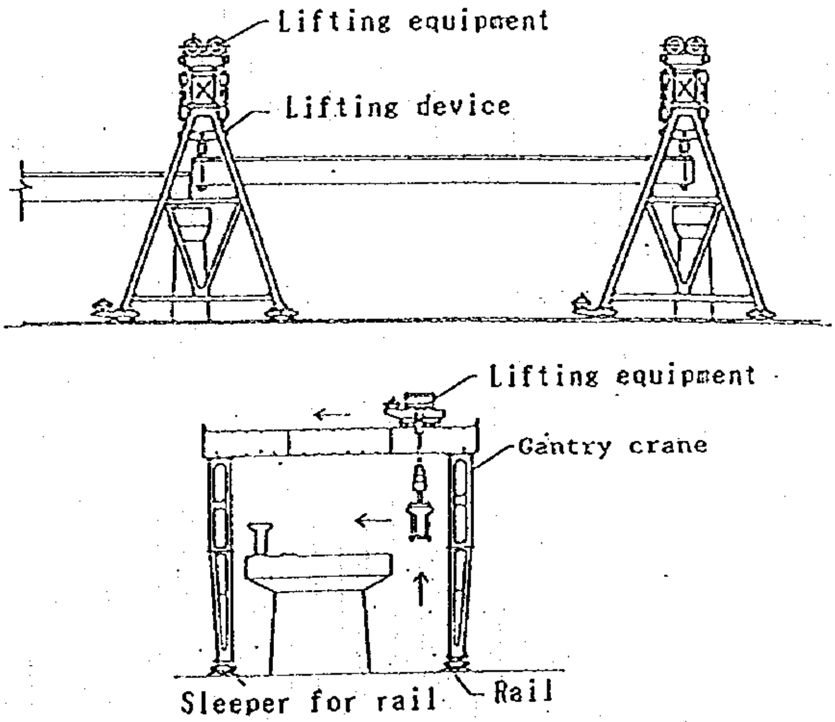
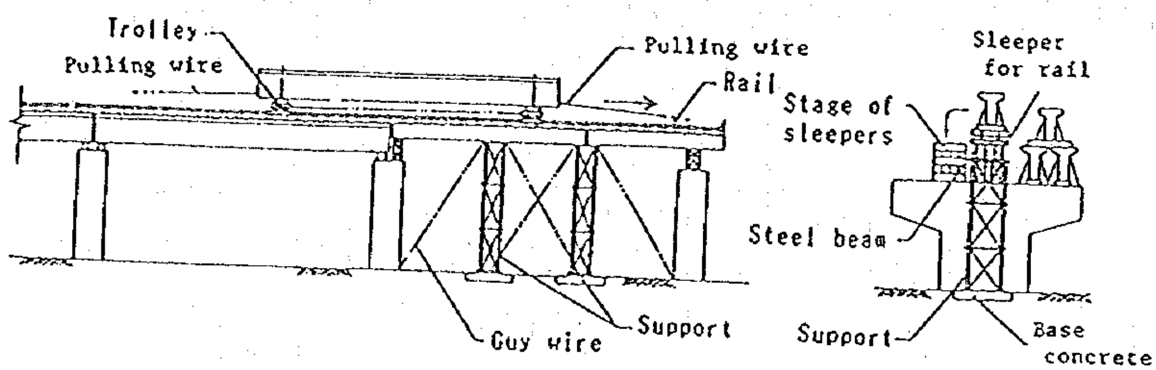


Fig. 3.22 Self Traveling Gantry Crane Method

A Carring and down method



B Carrying and shifting method

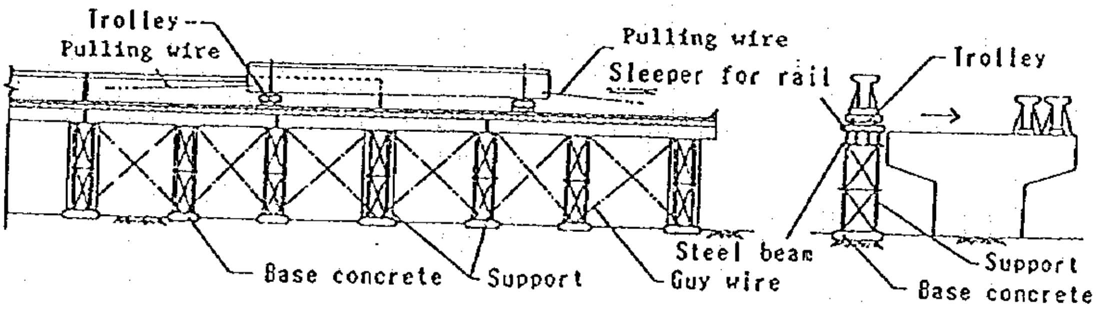


Fig. 3.23 Launching by Bent

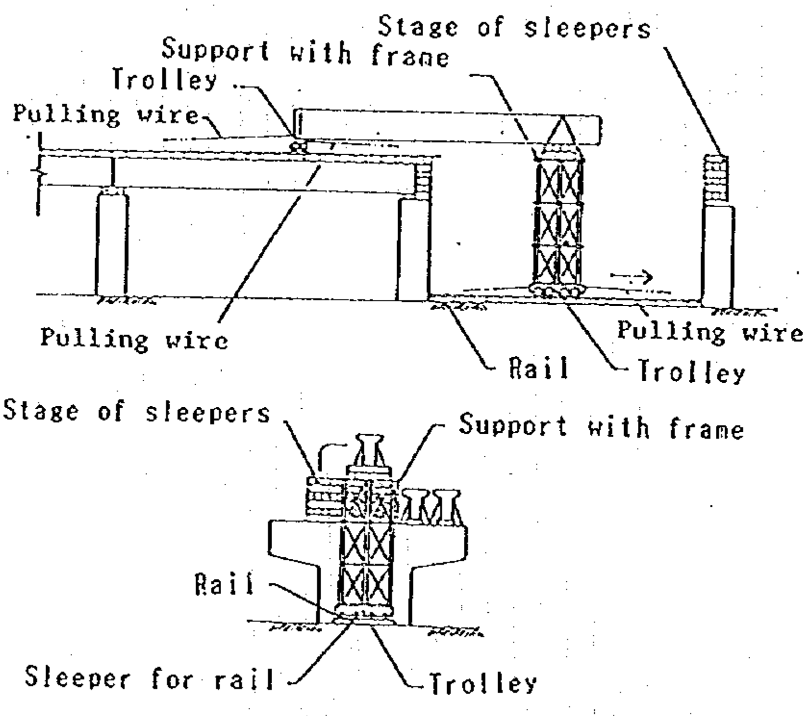


Fig. 3.24 Launching by Traveling Bent

(a) Fixed gantry crane method.
In this method, a pair of gantry crane is fixed by its legs to the ground so that it can lift and place concrete beams in position.

(b) Self-traveling gantry crane method
In this method, a pair of gantry crane is positioned on a track laid on the ground along the bridge spans so that it can freely travel over the bridge spans.

Both methods are shown in Fig. 3.21 and Fig. 3.22 respectively.

(5) Launching by Bent

In this method, steel supports are built either partially or entirely between the bridge spans, on which precast beams are carried to be laid them in place.

(a) Bent-type method
In this method, supports which are assembled between the entire bridge spans are utilized, on which the precast concrete beams can be pulled out.

(b) Traveling bent method
In this method, the end of a precast concrete beam is received on the travelling bent running along the ground rail, and launching can be accomplished by pulling out the traveling bent.

Both methods are shown in Fig. 3.23 and Fig. 3.24 respectively.

Characteristics of this method:

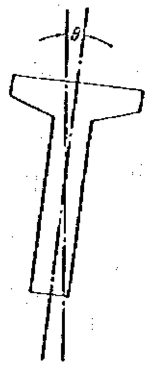
- Sufficient space for the bent to be placed and for it to travel over the span is necessary.

- When the pier height is low and the ground under the bent is firm, heavier beams can be launched.

(6) Safety Measures

Since, during transportation, the prestressed concrete beams are subjected only to the combined stress corresponding to the prestressing and the bending due to the dead load, the tensile stress at the top fiber of the beam is 10 kg/cm2.

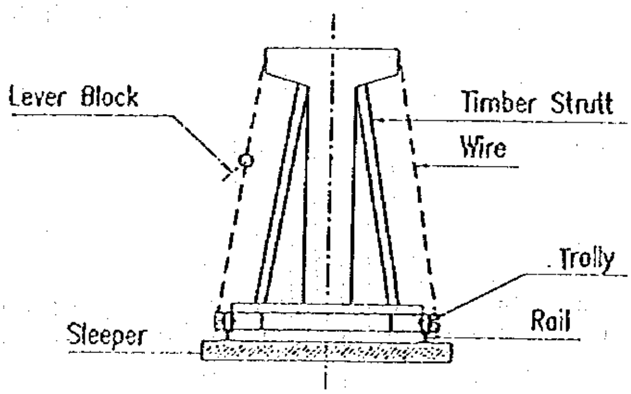
If the beam is inclined left or right, the bending moment acts laterally due to its dead load and the tensile stresses are induced on a side and at the top fiber of the beam.



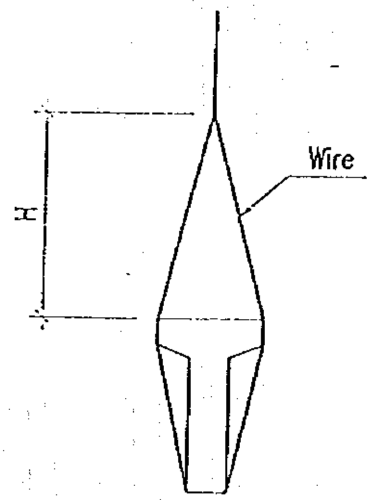
Therefore, appropriate handling and launching study shall be undertaken specially for long and slender girders, to ensure against lateral buckling or cracking during various stages of handling and launching.

- Position of temporary support
  Prestressed beams are only allowed to be suspended or supported firmly at their bearing points

- Inclination of beams
  Long and slender beams shall be safeguarded against tilting by means of auxiliary supports and temporary strutting or bracing.



Transpontatin of Beam



Launching by Truck Crane

H ≥ 1.5m for Pre-tensioned Beam

H ≥ 2.0m Post-tensioned Beam

Allowable Inclination of Beam

The following equation to be satisfied

$$\delta ct' \times \eta + \delta do' \times \{1-(Zc')/(Zh) \times \sin\theta\} \geqq \delta cat''$$

therefore, $\theta = \sin^{-1}\{(\delta cat'' - \delta ct' \times \eta - \delta do') / \delta do' \times (Zh/Zc')\}$

where;		
θ	:	Allowable inclination angle
$\delta cat''$	:	Allowable tensile stress due to temporary load during handling and launching = -25kg/cm²
$\delta ct'$	:	Tensile stress at top fibre due to prestressing
$\delta do'$	:	Compressive stress at top fibre due to girder weight
Zc'	:	Section modulus of X-Xaxis at top fibre
Zh	:	Section modulus of Y-Y axis at flange end fibre
η	:	Effective coefficient of prestressing due to loss = 0.95

Example calculation

Post-tensioned beam PTT-32 (span:32m)

$Zc' = 442 \times 10^3\ cm^3$

$Zh = 53 \times 10^3\ cm^3$ $\qquad \delta cat'' = -25\ kg/cm^2$

$\delta ct' = -79\ kg/cm^2$ $\qquad \eta = 0.95$

$\delta do' = 69\ kg/cm^2$

$$\theta = \sin^{-1}\{(-25+79\times0.95-69)/69\} \times (53\times10^3)/(442\times10^3)$$
$$= 1° -53'$$

2.6.7 Public Relations

Cooperation should be obtained from the government concerned and private agencies whose utilities or facilities affect the works, and also the inconvenience and danger to the public are to be avoided.

(1) Utilities

Involved utilities are water, electricity, telephone, sewers and oil/gas pipelines. These utilities are usually required diversion or protection by the government concerned or authorities and any damage to the utilities are to be avoided.

(2) Road Traffic

Where temporary diversion bridge or half-width construction is necessary, they shall be made safe and suitable, and also traffic management, during that period, is very important so as to minimize inconvenience to the public.

(3) Railway

Special care needs to be taken at intersections with railway tracks, or at any locations where work has to be undertaken close to railway tracks. It is essential that very close cooperation with the railway authority and its regulations are strictly taken and observed. Construction operation method and schedule shall be carefully planned, especially for lifting and launching operations of prestressed concrete beams.

2.7 Quality Control

2.7.1 Importance of Quality Control

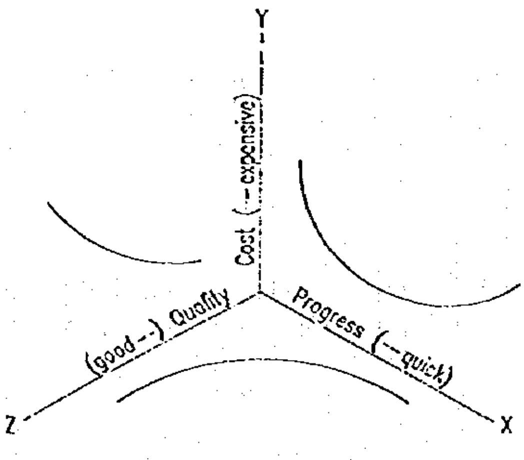
The objective of quality control is to ensure and verify the quality of the materials and the execution of works shall be in accordance with the contract documents and conforms good workmanship.

Two functions of quality control for materials and workmanship are as follows:

- Technical inspection
- Quality control testing

The technical inspection and quality control testing to be performed appropriately by suitably experienced engineers, inspectors and technicians, and thoroughly familiar with the relevant standards and corrected technical procedures. The construction quality control is a system which involves the joint but independent efforts on the Government, the Contractor and Manufacturer to achieve the level of quality in accordance with the Specifications. When the quality of completed structure does not meet required quality, the structure should be replaced and reconstructed. That is very uneconomical and brings loss not only to the Contractor but also to the Government.

In the generality of cases, quality and cost or progress is disagreed to each other, however, quality control is the most important matter than others.



2.7.2 Quality Control Checks

Flow chart of quality control is shown in Fig. 3.25. Quality control shall obtain the level of quality desired by the Specifications, such as Standard Specifications for Road Works, JKR, MS and BS.

Quality Control Checks for concreting works is referred "Manual on Quality Assurance in Concreting Works" - JKR 20700-0035-91. Quality control for tensioning works and grouting works are as follows:

(1) Tensioning Works

(a) Materials

- Identification of the steel and checking and filing of the documents (test reports, dispatch notes) sent by the supplier
- Checking anchorages and accessories
  Condition of the ducts.
  Geometric tolerances of anchorages and accessories.

(b) Before Tensioning Stage

- Placing the formwork
  Checking the condition of the tendons after storage and handling.
  Checking to ensure that the positioning tolerances have been complied with.
  Checking the number and rigidity of fasteners.

- Availability of full instructions issued by the designer
  Sequence of tensioning the tendons.
  Tensioning procedure (from one end or both ends, increments in the tensioning force to be applied, maximum pressure at the jack, corresponding theoretical tendon extension, tolerances on such extension).

- Availability and proper condition of the necessary equipment
  Calibration of the acks.

- Quality of the concrete (information check test results) and other materials concerned, (e.g. jointing materials)

- Degree of freedom
  To enable the prestressed concrete member to deform when the prestress is applied.
  To enable the cables to move in their ducts.

(c) Tensioning Stage

- Measurements for determining transmission coefficients, if necessary.

- Records of the forces and extensions corresponding to each increment of the tensioning force. (plotted in a diagram).

- Supervision of any re-tensioning operations and of the installation of anchorages and accessories.

- Measurements of slip of cable in relation to the anchorages.

(d) After Tensioning

- Visual inspection of the concrete of the structure.

(2) Grouting Works

(a) Before Grouting Stage

- Results of preliminary and suitability tests.

- Continuity of the ducts (no breaks in them), liquid-tightness of couplings, availability of unobstructed bent holes and weep holes.

- Making the grout (mix proportions and tolerances in the proportions, sequence of adding the constituents, mixing time, speed of the agitator, etc.).

- Grouting (adapted to the climatic conditions, blowing through with compressed air or flushing the ducts and tendons with water, grouting pressures, closure of bent holes).

(b) Grouting Stage

- Observing the pressure and detection of any leakage.

- Checking the volumes: quantity of grout and time that elapses before it emerges from the bent holes.

- Taking samples for quality control check.

(c) After Grouting Stage

- Checking that all bent holes and other openings have been properly sealed and that the permanent anchorages are suitably protected.

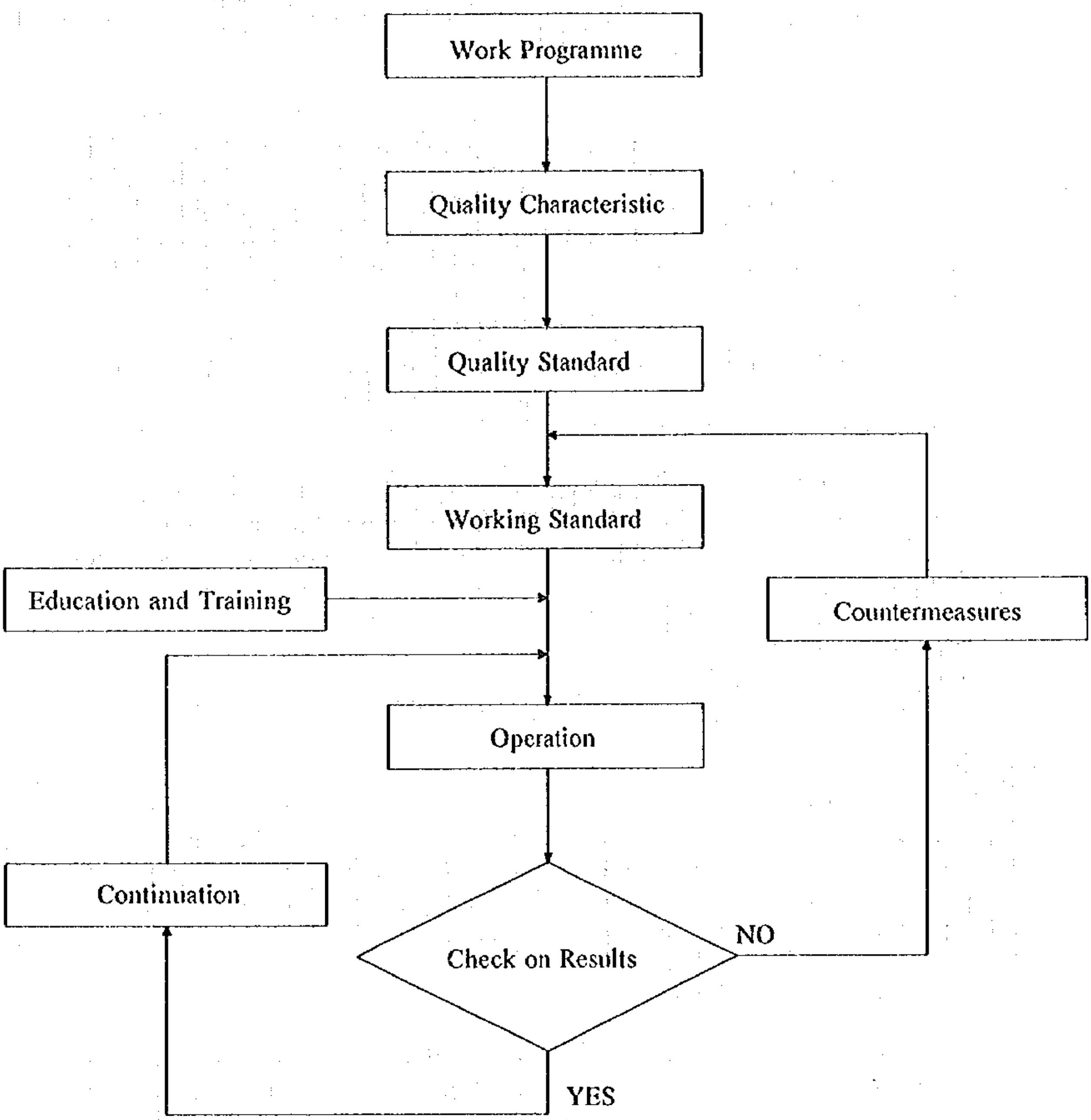


Fig.3.25 Flow Chart of Quality Control

CHAPTER 3 COST ESTIMATE

3.1 Objective of Cost Estimate

The purpose of cost estimate is to determine the forecast cost required to complete a project in accordance with the Specifications and preliminary construction plan. There are so many variations and factors that can influence the cost of construction, such as geology, topography, site environment, workmanship and construction difficulties. The construction cost should be estimated according to site condition, construction plan and construction time which were clarified in plan and design stage.

The estimated cost shall be reasonably enough to allow the contractor to complete the project with a reasonable profit, yet low enough to be within the Government budget.

Because construction estimates are prepared before a project is constructed, an estimate is to be close approximation of the actual cost. The true construction cost will not be known until the project has been completed and all costs have been recorded.

3.2 Type of Cost Estimate

Cost estimates may be divided into at least two different types, depending on the purpose as approximate estimates (preliminary estimate) and detailed estimates.

Early in a project, prior to the design, the Government may wish to know the approximate cost of a project. At the detailed design stage, the Consultants will estimate the cost in detail in order to finalize the design to satisfy the Government's budget. The Contractor must know the costs required to perform the works in accordance with the final contract documents.

3.2.1 Approximate Cost Estimate

Approximate cost may be estimated according with the statistical contract rate of the similar projects. It may be estimated to multiply the number of square meter of bridge surface area for superstructure, and the concrete volume for substructure excluding pile.

The unit cost to be obtained from a weighting of the data that emphasizes the average value yet it account for the maximum and minimum values as shown by the equation below.

$$UC = (A+4B+C)/6$$

where; UC : Proposed unit cost per m2
A : Minimum unit cost of previous bridge

B : Average unit cost of previous bridge
C : Maximum unit cost of previous bridge

It is necessary to adjust the cost information from previous completed bridge, such as year built, location and bridge size.

The adjustment should be present the relative inflation or deflation of costs with respect to year built, and relative costs of materials, equipment, and labour with respect to the geographic location of bridge under construction. Although the total construction cost of bridge will increase with size of bridge, however, the unit cost per square meter of bridge surface area may decrease.

Proposed cost = Previous cost x fy x fl x fs x fc

where; fy : Adjusting factor of year built
fl : Adjusting factor of location

fs : Adjusting factor of bridge size, such as span length and number of span
fc : Adjusting factor of construction difficulty

3.2.2 Detailed Cost Estimate

A detailed estimate is prepared by determining the cost of the materials, labour, equipment, subcontract work, overhead, profit, and some contingencies. The process of estimate begins with a thorough review of the contract document - contract requirements, drawings and technical specifications. It is necessary to visit the project site to clarify factors that can influence the cost of construction, such as site conditions, control of traffic, security, and existing underground utilities.

There are two distinct tasks in estimation; to determine the probable real cost and to determine the probable real time to construct the project. It is required to provide production rates, crew size, equipment and the estimated time to perform various individual work items for planning and scheduling of the project. This information concerning costs can cover an integration of the estimating and scheduling function of construction project management.

Flow chart of cost estimate and work schedule are shown in Fig.3.26 and Fig. 3.27 respectively.

3.3 Structure of Total Project Cost

Total project cost consists of construction cost and project cost, such as administration cost, engineering cost, land acquisition and compensation cost, and contingencies.

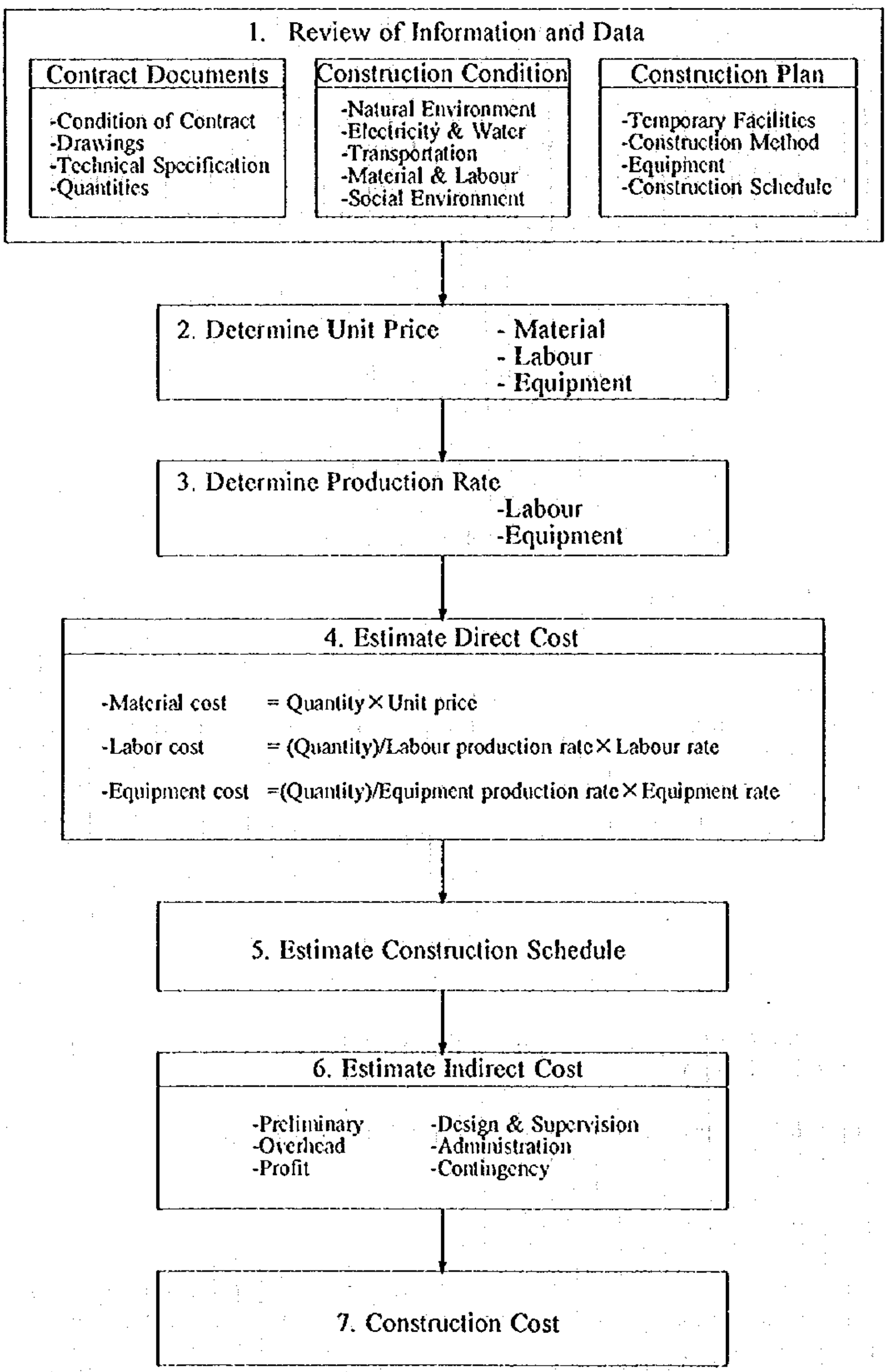


Fig. 3.26 Flow Chart of Cost Estimate

1. Sequence of Main Works

2. Work Items and Elements

3. Operation Days of Work Items

= (Quantity of Work Item)
/ (Average Daily Work Quantity)

4. No Working Days

-Daily weather patherns and seasonal weather contraints
-Sunday and public holidays
-Other factors

5. Necessary Operation Day of Work Items

= Item 3 x (Calender days) / (Calender days - No working days)

6. Mobilization and Demobilization Period

7. Whole work Schedule arranging Item 1 to Item 6

Fig. 3.27 Flow Chart of Work Schedule

Construction cost is subdivided into direct cost and indirect cost. Direct cost consists of labour cost, material cost and equipment cost. Indirect cost comprises of general cost, preliminary cost and, contractor's overhead and profit.

Structure of total project cost is charted in Fig. 3.28.

- Total Project Cost
  - Construction Cost
    - Direct Cost
      - Material Cost
      - Labour Cost
      - Equipment Cost
    - Indirect Cost
      - General Cost
      - Preliminary Cost
      - Overhead
      - Profit
  - Project Cost
    - Administration Cost
    - Engineering Cost
    - Contingency
    - Land Acquisition and Compensation Cost

Fig.3.28 Structure of Total Project Cost

3.3.1 Direct Cost

(1) Labour Cost

Labour Cost is calculated as follows:

Labour Cost = Number of Labour x Labour Rate
= Quantity of work item x Production rate x Labour rate

Labour rate includes wages, income tax, insurance and all fringe benefits, such as vacation, sick leave, medicare and workmen's compensation. The labour rate is estimated on the basis of data researched from the market investigation.

Production rate is the number of unit of work produced by a person in a specified time, usually an hour or a day. Production rate may also specify the time in labour-hours or labour-days required to produce some unit of works.

Such data may be obtained by keeping accurate records of the production of labour on projects during the progress of construction.

(2) Material Cost

Material cost is calculated as follows:

Material Cost = Quantity x Unit price
= Designed quantity x (1 + Rate of loss)
x (Material unit price + Transportation cost)

The quantity of material can be taken from the drawings, however, loss of material due to storage and construction should be considered.

Construction materials are delivered by the supplier or producer directly to the project site in trucks, however, some materials may be obtained by the Contractor at the storage yard of the supplier.

(3) Equipment Cost

There are two types of equipment costs: ownership costs and operating costs. Ownership costs refer to the costs incurred even if the machine is not working. They include depreciation, interest, taxes, insurance, and maintenance and repair. Operation costs are the costs incurred in operating the machine. They include costs for repair, fuel, lubricants, tires, consumable parts and operator's wages. Composition of equipment cost is shown in Fig. 3.29.

- Equipment Costs
  - Ownership Costs
    - Depreciation
    - Maintenance and Repair
    - Interest, Insurance, Taxes
  - Operating Costs
    - Fuel
    - Lubricants, Grease, Filters
    - Tires
    - Repairs
    - Consumable Parts
    - Operator's Wages
  - Transportation Costs

Fig.3.29 Composition of Equipment Cost

The equipment cost is calculated in unit of hourly cost (M$/hour) which is suitable for use on any project.

(a) Depreciation

In general, depreciation is a tax term referring to the legally permitted decline in value from the original purchase price of equipment for the depreciation periods from the view point of economical life of equipment.

Depreciation = (Net depreciation value)
/ (Depreciation period in hours)

$$= \text{(Purchase price - Salvage value)} / \text{(Depreciation period in hours)}$$

(b) Interest, Insurance and Taxes

Whether or not purchased equipment is actually in operation, its owner must pay interest, insurance and taxes. Interest refers to the interest on the investment, when the investment is covered by the owner's own fund or to the interest on the debt, when the investment is covered by a debt. In either case, the interest will be an equal amount.

Calculation is made of the average value of the residual value at the beginning of each year within the depreciation period, and interest, insurance and taxes are imposed on this value. By dividing this value by the number of hours and expects to operate the machine in one year, the hourly value can be calculated.

This can be calculated by using the following formula.

$$\text{Interest, insurance, tax} = \text{(Factor x Delivered price x Annual rates)} / \text{(Annual use in hours)}$$

The factor can be calculated by the following formula.

$$\text{Factor} = 1 - \{(n-1)(1-r)\} / 2n$$

where; n : Depreciation period in year

r : Salvage value rate

= (Machine worth at trade-in or resale time) / (Delivered price)

(c) Revision of Equipment Cost

The hourly cost of ownership and operating construction equipment will vary with the condition where the equipment is operated. If bulldozers are operated on soft rocks surface, equipment cost will be higher than when the same bulldozers are operated on common soil surface. Therefore, the equipment costs will be adjusted according to the site condition where the equipment is to be operated.

3.3.2 Indirect Cost

(1) General Cost

General cost includes the following items, which depend on site condition and construction requirement:

- Construction of temporary bridge and road for public
- Demolition of existing bridge
- Relocation of existing public utilities
- Temporary diversion of river/waterway

(2) Preliminary Cost

Preliminary cost includes the following items and it is assumed to be 10 to 15% of the direct cost depending on period of the project and amount of the contract.

- Site office and laboratory
- Waterhouse
- Survey
- Laboratory equipment
- Facilities for electricity and water supply
- Safety measures
- Transportation
- Engineering

Preliminary costs of PC bridge are;

Pre-tensioned bridge	:	(Direct cost- Manufacturing cost)x(10 to 15%)
Post-tensioned bridge	:	Direct cost x (10 to 15%)

(3) Overhead and Profit

The overhead cost is divided into two categories: job overhead and general overhead.

The job overhead can be specifically charged to a project and general overhead can be charged to the general office of the company.

Job Overhead

- Salaries of project staff
- Site office supplies
- Communication
- Rent
- Travel expenses and allowance
- Insurance and tax

General Overhead

- Salaries
- Office supplies
- Office rent
- Welfare
- Travel expenses and allowance
- Insurance and tax

The amount of profit is depending on the risk involved, desire of the Contractor to get the job and other factors.

The actual amount of overhead and profit may be ranged between 20 to 30% of the direct cost depending on the risk involved, desire of the contractor to get the work and others.

3.3.3 Project Cost

(1) Administration Cost

Administration cost is an expense of the Goverment arising from implementation of the project cost and is assumed to be 3 % of construction cost.

(2) Engineering Cost

Engineering cost consists of detailed design (Geological and topographical survey, and detailed design) and construction supervision and it is assumed to be 5 to 10% (for example 2% for the detailed design and 6% for supervision) of the construction cost.

(3) Contingency

Contingency is divided into physical contingency and price contingency as described below:

- Physical contingency is mainly to cover unforseeable or unavoidable matters during construction, such as temporary land acquisition and some variations. The contingency is usually considered 5 to 10% of the construction cost.

- Price contingency allows for future price escalation and fluctuation of exchange rates.

(4) Land Acquisition and Compensation Cost

A considerable amount of land acquisition and compensation costs are required for new bridge construction.

3.4 Major Work Classification

3.4.1 Clearing and Grubbling

Prior to staring structure excavation operations, all surface objects and all trees, stumps, roots and other obstructions shall be cleared and/or grubbed. This work will be operated by bulldozers. Measurement will be by area basis (hectares) or lump-sum basis or individual removal of trees (each).

3.4.2 Structure Excavation and Backfill

This work consist of the necessary excavation for foundations of substructures and placing backfill with free draining granular material in excavated area around structures.

This work may include necessary diverting of water streams, bailing, pumping, draining, and the necessary construction of cofferdams or gribs, with all sheeting and bracing involved.

All rock or other hard foundation material should be cleaned of all loose material and cut to a firm surface, either level or stepped. Blinding stone, such as cobble stone or crushed rock for use as a foundation to be provided.

(1) Operation of Excavation and Backfill

The operations of this work are as follows:

Operation	Equipment
Excavation	Power shovel (0.35 to 0.6m3) Bulldozer (15 tons)
Backfill	Power shovel (0.35 to 0.6m3)
Compaction	Bulldozer (15 tons) Vibration roller (0.8 to 1.1 tons) Compactor (60 to 100kg)
Transportation	Dump truck (11 tons)

(2) Temporary Works

	Operation	Material	Equipment
Cofferdam	Diving Removing Draining	Wood plate sheet-pile or Steel Sheet-pile or H-shaped pile	Diesel hammer Truck crane (20 to 22 tons) Vibro-hammer (30 to 40 kW) Pump
Diversion of Water Stream	Construction of Dikes or By-pass channels		Bulldozer Power shovel

3.4.3 Concrete

This work consist of the construction of portion of structures of portland cement concreted which is a mixture of cement, water, admixer, and coarse and fine aggregates, and may include temporary works.

(1) Use of Concrete

The uses of concrete are as follows:

- Post-tensioned concrete girders
- Cross beam
- RC slab
- Abutments and piers
- Curb and parapet
- Approach slabs
- Leveling concrete

(2) Operation of Concrete Work

Operation	Equipment
- Trial mix	Mixing plant
- Mixing concrete	
- Transport	Truck
- Handling	Cart
- Placing	Truck crane (15 to 20 tons)
	Concrete pump
	Hopper and chute
- Compacting	Vibrator
- Finishing	
- Curing	

(3) Temporary Works

This work include falsework, formwork and base for manufacturing post-tensioned beam.

Work Item	Use of Concrete	Operation	Material	Equipment
Falsework	Abutment Pier RC slab bridges	Piling or compaction Erection Fixing Removing	Foundation Pile or concrete or timber Scaffoldings Timber or Steel frame or Steel pipe	Truck crane (15 t 16 tons)
Formwork	Abutment Pier RC slab bridges	Fabrication Erection Fixing Removing	Plywood or steel or aluminum form Form tie Form oil	Truck crane (15 to 16 tons)
Base for Manufacturing Beam	Post-tensioned beam	Compaction	Concrete or timber Briding stone	

3.4.4 *Reinforcing Steel*

This work consist of furnishing, fabricating (cuttin, bending), splicing, and placing (erecting, fastening) reinforcing steel bars of the type and size conformed with Drawings. Tie wires and separators and other materials used in fastening the reinforcing steel bars are included in this work.

3.4.5 Furnishing Pre-tensioned Beam and Spun Concrete Pile

Pre-tensioned beams and spun concrete piles are manufactured and supplied by manufacturer. The cost will depend upon the type, length and number of purchase, and the distance from the manufacture yard of the supplier to the job site. The cost estimate and other information will be obtained from the manufacture suppliers.

3.4.6 Pile Driving

The work may consist of spun concrete pile driving, cut and spliced, and loading test. Operation of the work is as follows:

Work Item	*Operation*	*Equipment*
Pile Driving	Handling Driving Method - Percussion - Vibration - Excavation Splicing Cut off	Crawler driving machine Truck crane Vibro hammer Earth auger machine
Loading Test	Maintained load (ML) test or Constant-rate of penetration (CRP) test	Hydraulic jack Steel I-beam Timber crib Kentledge block

3.4.7 Prestressing

This work may consist of furnishing, placing and tensioning of prestressing steel, and grouting for post-tensioned concrete beams.

Operating of prestressing and grouting are as follows:

Work Item	Operation	Material	Equipment
Prestressing	Placing ducts Placing strand wire Prestressing	Duct Strand wire Anchorage Supporter of duct Grid	Cutter for strand wire Tensioning jack Tensioning pump
Grouting	Mixing grout Injecting grout	Cement Admixer	Grouting mixer Grouting pump

3.4.8 Handling and Launching

This work includes lifting, handling, transporting, launching and placing in position of pre-tensioned or post-tensioned beams.

Operation of the work are as follows:

Work Item	Operation	Equipment
Handling	Lifting Transportation	Truck crane Gantry crane Jack Rail Trolley Winch
Launching	Launching Shifting Placing	Track crane Launching steel girder Gantry crane Bent and rail

3.5 Unit Cost of Major Item of Work

3.5.1 Unit Rates and Prices

The unit rates and prices of labour, material and equipment in or around Kuala Lumpur and based on 1995 figures are derived from market investigation, local contractors and JKR.

These unit rates and prices are shown in Table 3.6.

3.5.2 Unit Cost of Major Item of Work

To determine the time required to perform a given quantity of work, it is necessary to estimate the probable rates of production of the labour and equipment. It is, however, difficult to estimate the accurate production rate due to lack of records and data in Malaysia.
Therefore, the production rate being used in Japan is modified taking into account labour's skillfulness, efficiency of equipment, operator's capability in Malaysia as well as based on those in similar projects in ASEAN countries. Finally, the unit cost of each work item is calcalated using the caliblated production rate.

Unit costs of major items of work are shown in Table 3.7.

Table 3.6 Unit Rates and Prices

(The cost shall be calculated after researching current conditions of the costs.)

1. Labour Unit Rate (M$)		
Description	Unit	Unit Rate
Foremen , Specialist	Day	70.00
Skilled Labour	Day	60.00
Semi-Skilled Labour	Day	55.00
Common Labour	Day	30.00
Carpenter	Day	60.00
Steel Bar Bender and Fixer	Day	60.00
Concrete Worker	Day	50.00
Welder	Day	60.00
Painter	Day	60.00
Mason	Day	60.00
Mechanic	Day	60.00
Electrician	Day	60.00
Pavement Worker	Day	40.00
Plant Operator	Day	60.00
Driver	Day	45.00
2. Material Unit Price (M$)		
Description	Unit	Unit Rate
Portland Cement	Kg	0.25
Reinforcing Steel	Kg	1.20
Bituminous Material	Ton	400.00
Gasoline	Litre	1.20
Diesel Oil	Litre	0.70
Lubricant	Litre	4.50
Fine Aggregate	M3	20.00
Coarse Aggregate	M3	45.00
Boulders	M3	20.00
Plywood t = 1/2"	M2	15.00
Timber	M3	500.00
H-Shaped Steel	Kg	1.80
Angle Steel	Kg	1.50
Steel Sheet Pile	Kg	1.90
Steel Pipe Pile	Kg	2.20

(The cost shall be calculated after researching current conditions of the costs.)

Form Tie		No	0.40
Separator		No	0.20
Spacer		No	0.20
PC Strand Wire		Kg	6.80
PC Anchor 12-RT15.5		No	400.00
Duct dia. 65mm		M	8.00
Ready Mixed Concrete			
G30		M3	132.00
G40		M3	154.00
G50		M3	177.00
Spun Concrete Pile			
dia. 350m/m		M	60.00
dia. 400m/m		M	75.00
dia. 500m/m		M	110.00
dia. 600m/m		M	140.00
Pre-tensioned Tee Beam			
L = 7.6m, W = 2.1 ton		Each	781.00
L = 9.5m, W = 2.6 ton		Each	906.00
L = 12.5m, W = 3.5 ton		Each	1,361.00
L = 16.7m, W = 6.6 ton		Each	2.565.00
L = 18.9m, W = 8.1 ton		Each	3.270.00
Elastomeric Bearing Pad			
230x200x26 2-steel plate		Each	71.00
400x280x47 3-steel plate		Each	216.00
510x375x69 3-steel plate		Each	485.00
3. Equipment Unit Rate (M$)			
Equipment Description	Specific	Unit	Unit Rate
Bulldozer (D4)	6 Ton	Hour	65.00
Bulldozer (D6)	15 Ton	Hour	80.00
Bulldozer (D7)	21 Ton	Hour	90.00
Bulldozer (D8)	38 Ton	Hour	100.00
Wheel Loader	1.2 sq.m	Hour	50.00
Wheel Loader	1.4 sq.m	Hour	50.00
Wheel Loader	1.7 sq.m	Hour	55.00

(The cost shall be calculated after researching current conditions of the costs.)

Wheel Loader	2.1 sq.m	Hour	60.00
Backhoe	0.3 sq.m	Hour	30.00
Backhoe	0.7 sq.m	Hour	30.00
Backhoe	1.2 sq.m	Hour	30.00
Crowler Crane	35 Ton	Hour	100.00
Crowler Crane	50 Ton	Hour	130.00
Crowler Crane	100 Ton	Hour	450.00
Truck Crane	5 Ton	Hour	45.50
Truck Crane	10 Ton	Hour	58.00
Truck Crane	15 Ton	Hour	65.00
Truck Crane	20 Ton	Hour	71.50
Truck Crane	35 Ton	Hour	100.00
Dump Truck	10 Ton	Hour	40.00
Cargo Truck	6 Ton	Hour	30.00
Cargo Truck	10 Ton	Hour	40.00
Trial Truck	30 Ton	Hour	650.00
Portable Compactor		Hour	10.40
Grader	2.5m blade	Hour	40.00
Motor Grader	3.7m blade	Hour	75.00
Agigator Truck	2.0 cu.m/hr	Hour	20.00
Concrete Truck Mixer	5.0 cu.m	Hour	45.50
Concrete Mixer	2.0 cu.m/hr	Hour	19.50
Concrete Mixer	0.3 cm.m/hr	Hour	9.10
Vibrator	30 m/m	Day	10.00
Concrete Pump	30 cu.m/hr	Hour	150.00
Diesel Hammer	2.5 Ton	Hour	140.00
Vibrating Hammer	60 kw	Hour	250.00
Pick Hammer (Jack Hammer)	7 kg	Day	35.00
Pile Driver	35 Ton	Day	1,680.00
Bar Bender	Max. 25mm	Day	35.00
Bar Cutter		Day	35.00
Winch		Purchase	800.00
Tamper	60 - 80 kg	Day	35.00

Remarks: These data are based on market research in 1995.

Table 3.7 Unit Cost of Major Item of Work (The cost shall be calculated after researching current conditions of the costs.)

No. 1 Concrete Work (grade 40) per 10 cu.m					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Skilled labour	man	0.3	60	18	
Semi-skilled labour	man	1.2	55	66	
Labour	man	1.4	30	42	
Truck crane	hr	2.3	65	150	15 to 16 ton
Ready-mixed concrete	cu.m	10.2	160	1,632	loss 2%, incl. transporation
Others	L.S.	1	30	30	
Total				1,938	
Per cu.m				193.8	

No. 2 Concrete Work (grade 50) per 10 cu.m					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Skilled labour	man	0.3	60	18	
Semi-skilled labour	man	1.2	55	66	
Labour	man	1.4	30	42	
Truck crane	hr	2.3	65	150	15 to 16 ton
Ready-mixed concrete	cu.m	10.2	184	1,877	loss 2%, incl. transportation
Others	L.S	1	30	30	
Total				2,183	
Per cu.m				218.3	

(The cost shall be calculated after researching current conditions of the costs.)

No. 3 Reinforcing Steel per 1.0 ton					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Skilled labour	man	0.8	60	48	
Semi-skilled labour	man	3.6	55	198	
Labour	man	2.9	30	89	
Reinforcing steel	ton	1.03	1,200	1,236	loss 3%
Others	L.S	1	30	30	
Total				1,601	

No. 4 Formwork per 100 sq.m					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Carpenter	man	29.4	60	1,764	
Labour	man	20.5	30	615	
Plywood	L.S	1	1,000	1,000	
Others	L.S	1	40	40	
Total				3.419	
Per sq.m				34.2	

(The cost shall be calculated after researching current conditions of the costs.)

No. 5 Falsework per 100 cu.m					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Skilled labour	man	14	60	840	
Labour	man	6.6	30	198	
Track crane	hr	4	65	260	15 to 16 ton
Scaffolding	L.S	1	600	600	frame type
Total				1,898	
Per cu.m				19.0	
Foundation of falsework					
Per sq.m				15.0	Cobble or crushed stone
* Area = scaffolding area					

No. 6 Spun Concrte Pile (600m/m x 30m) per 10 piles					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Skilled labour	man	7.7	60	462	
Semi-skilled labour	man	15.3	55	842	
Labour	man	7.7	30	231	
Welder	man	7.7	60	462	
Spun concrete pile	No.	30	1,400	42,000	600m/m x 10m
Earth auger	hr	32	230	7,360	
Crowler crane	hr	19	115	2,185	
Backhoe	hr	10	30	300	
Others	L.S	1	4,000	4,000	7.5% of above
Total				57,842	
Per pile				5,784	

(The cost shall be calculated after researching current conditions of the costs.)

No. 7 PC Cable (12 - dia.12.7 m/m) per 100m

Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Specialist	man	13.5	70	945	
Labour	man	9.8	30	294	
PC strand	m	106	63.2	6,699	loss 6%
Duct	m	106	8.5	901	loss 6%
Grout	cu.m	0.33	210	69	loss 50%
Others	L.S	1	50	50	
Total				8,958	
Per m				89.6	

No. 8 Prestressing (12 - dia.12.7 m/m) per 10 cable

Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Specialist	man	7.0	70	490	
Labour	man	7.2	30	216	
Anchorage	set	20	400	8,000	
Tensioning jack	set · day	3	100	300	
Grouting pump	set · day	4	10	40	
Others	L.S	1		100	
Total				9,146	
Per cable				914.6	

(The cost shall be calculated after researching current conditions of the costs.)

No. 9 Launching by 35 ton Single Truck Crane per day					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Foreman	man	1.5	70	105	
Specialist	man	9.0	70	630	
Labour	man	6.0	30	180	
Truck crane	day	1	1,000	1,000	35 ton x 1
Others	L.S	1		500	
Total				2,415	
* Average launching ; 50 ton/day					

No. 10 Launching by 55 ton Two Truck Crane per day					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Foreman	man	1.5	70	105	
Specialist	man	12.0	70	840	
Labour	man	7.5	30	225	
Truck crane	day	1	4,000	4,000	55 ton x 2
Others	L.S	1		800	
Total				5,970	
* Average launching; 100 ton/day					

(The cost shall be calculated after researching current conditions of the costs.)

No. 11 Launching by 120 ton Two Truck Crane per day					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Foreman	man	1.5	70	105	
Specialist	man	12.0	70	840	
Labour	man	7.5	30	225	
Truck crane	day	1	8,000	8,000	120 ton x 2
Others	L.S	1	1,100	1,100	
Total				10,270	
* Average launching; 160 ton/day					

No. 12 Transversal Prestressing (4 - dia.15.2) per 10 cable					
Breakdown of Item	Unit	Qty	Rate (M$)	Amount (M$)	Remarks
Foreman	man	3.8	70	266	
Specialist	man	10.5	70	735	
Labour	man	5.8	30	174	
PC strand	ton	0.636	6,800	4,325	loss 6%
Duct	m	136	6	816	
Anchorage	No.	20	120	2,400	
Grout	cu.m	0.23	210	48	loss 50%
Tensioning jack	set · day	1	35	35	
Grouting pump	set · day	3	10	30	
Others	L.S	1	50	50	
Total				8,879	
Per cable				888	

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