STRUCTURAL ENGINEERING

QUARTERLY JOURNAL OF

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

VOLUME 8-1, JAN.-FEB.-MARCH, 2006

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INNOVATIVE ENABLING STRUCTURE FOR P. S. C. BOX GIRDER BRIDGE OVER HIMALAYAN RIVER

(Eng. A. B. Karnik)

The case illustrated here is of a Bridge over river Mahananda at Dalkhola (West Bengal). This bridge has 12 spans each of 52.80 m c/c. The piers have well foundations. The superstructure over piers is of Prestressed Concrete Box Girder weighing 1200 T per span.

<u>A suitable system of enabling structure had to be evolved.</u>

Many ideas came up suggesting span by span staging erection between piers, complete the span work, dismantle the staging on floating barge and re-erect the same for the next span. This process was very laborious, time consuming and uneconomical. Constructing components of P.S.C. box on shore and erecting them in position by crane mounted on floating barge was also not workable. Novel structural system for this enabling work (ref. dwg. H) was developed. Main span of 52.80 m between permanent wells is further divided into four equal spans of 13.20 m and provided with three temporary groups of piles on either side of the bridge (six in number). Each pile group consists of three 500 mm dia. piles. Structural steelwork is used for the staging and centering. Shuttering material used is 12 mm thick plywood except for box soffit which is 3.15 mm mild steel. Shuttering is backed up by mild steel rolled sections. Complete centering and shuttering assembly in sequence of support system is as follows:

Rollers are provided on each of the pile groups and well caps (over brackets) with rolling top at + 37.56 level (ref.dwg. L). Track Girder TG1 is mounted over the above rollers to support staging. Five Portals (two over wells and three over pile groups spaced at 13.20 m centres) consisting of vertical frames F2, F2X and horizontal girder G2 (ref.dwg. J) are supported on TG1. Two Girders G1 span across each of the Portals to make up full length of 52.8 m. Girders G1 support the shuttering frames of concrete box soffit, web and cantilever deck by suspension (ref.dwg. K). Shuttering for inside of box web and deck is supported on the soffit slab. Sequence of operations is as follows: Concreting of box is done as per the specifications. After due age of concrete and as per specifications, the shuttering is cleared off the contact with the concrete box to allow freedom for prestressing. Only soffit support of the box is retained. After the concrete box is self-supporting as per the specifications, shuttering frames are made free of suspension from Girders G1. Two halves of the soffit shutters are rotated to vertical position and cantilever deck shutters also are rotated down to the extent they are free of the concrete deck. Shuttering frames are in the convenient widths of 4.88 m & 4.16 m and rotated over hinges mounted on the Portals and bracing girders in between. Staging is now ready for ROLLING to the next span. Soffit shutters are now vertical and hence, whole staging has clear passage across the piers. Full staging on ROLLERS has definite merit as regards accuracy in line and levels to be achieved as also as a great time saver. Rolling of whole assembly to next span takes two days and raising of soffit shutters in line and level take six days. Thereafter, reinforcement laying for the soffit slab starts. Earlier estimated time cycle of 100 days for one span thus got reduced to 60 days. Apart from this great economy in time and money, system is very safe and accurate.

A. B. Karnik

(Born 22.07.1935)

Graduate in Science (with Mathematics as principal subject) from University of Bombay (Year 1955); Graduate in Civil Engineering from University of Bombay (Year 1958)

Life Fellow of the Institution of Engineers (India) - the Indian Institution of Bridge Engineers - the Indian Geotechnical Society - the Association of Consulting Civil Engineers (India).

Life Member of the India Chapter of the American Concrete Institute – the International Society for Soil Mechanics and Geotechnical Engineering - the Indian Society of Structural Engineers.

Member of the Experts Delegation to China, Honkong, Malaysia and Singapore (1995), representing The Indian Institution of Bridge Engineers.

Practicing as a Consulting Engineer since 1962

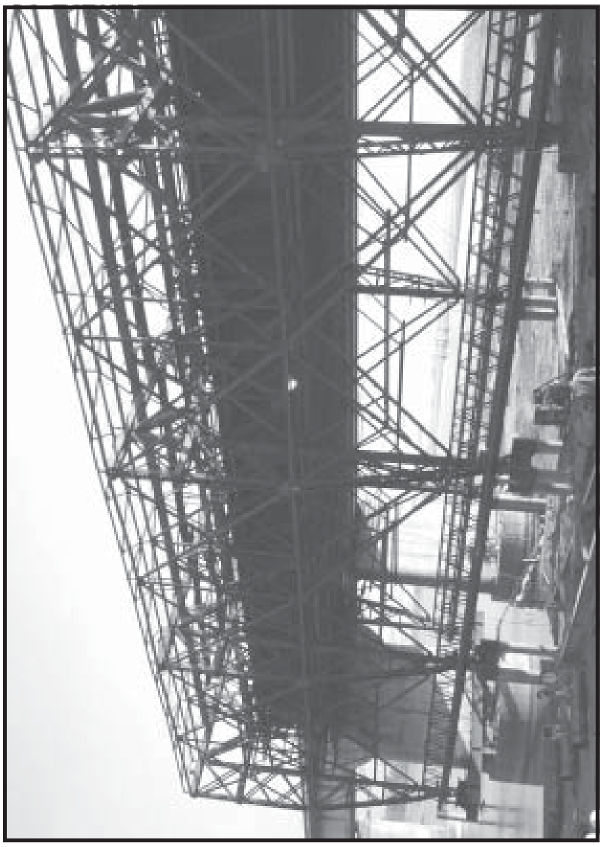
a) Partner of M/S N.N.Shrikhande & Co. 1962 to 1990

b) Jt. Managing Director of M/S Shrikhande Consultants Pvt. Ltd. 1978 to 1990

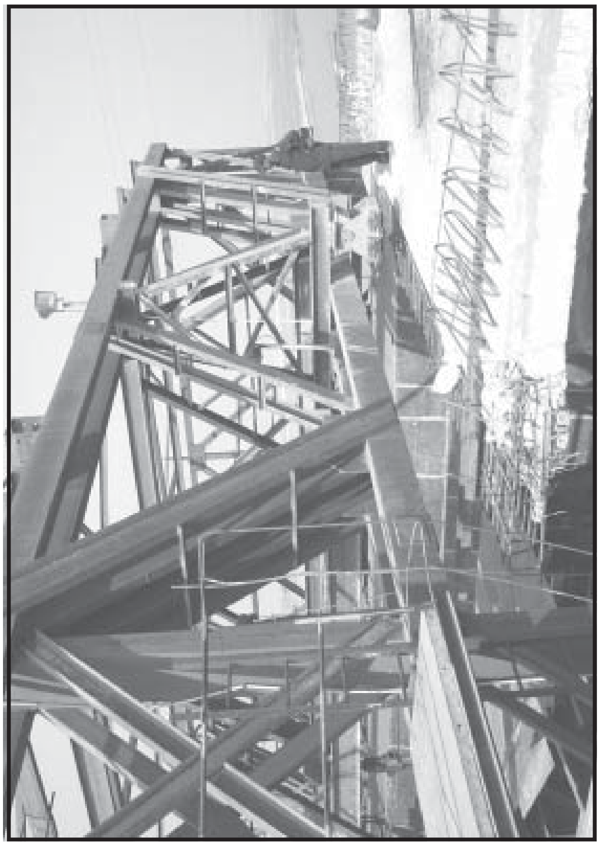
c) Proprietor of A.B.Karnik and Associates, May 1990 onwards

Recipient of S. B. Joshi Memorial Award for 1997 instituted by the Institution of Engineers (India) Maharashtra State Centre for "Outstanding Contribution in Spreading Engineering Knowledge".

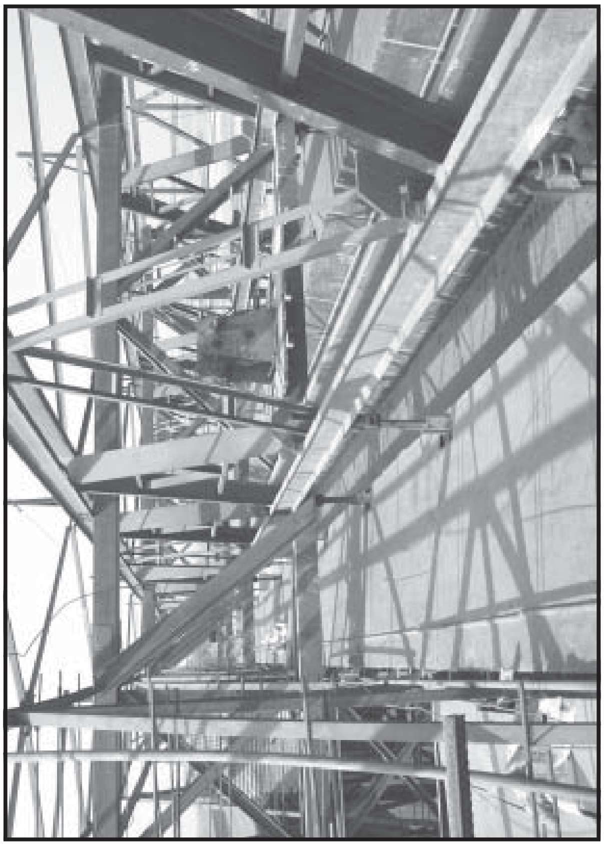
Recipient of ACCE SIMPLEX AWARD 1997 for "Innovative Design of Structures" instituted by Association of Consulting Civil Engineers (India)



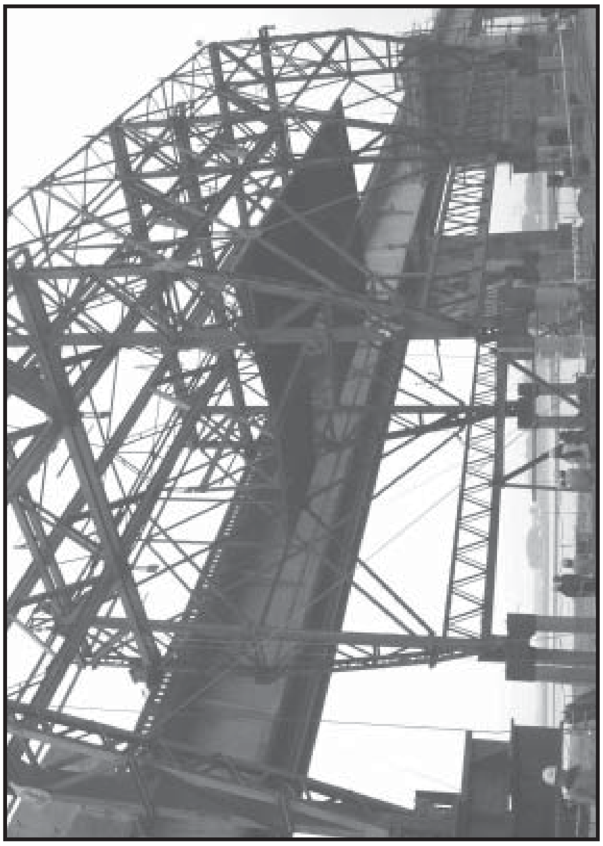
Perspective View of Gantry



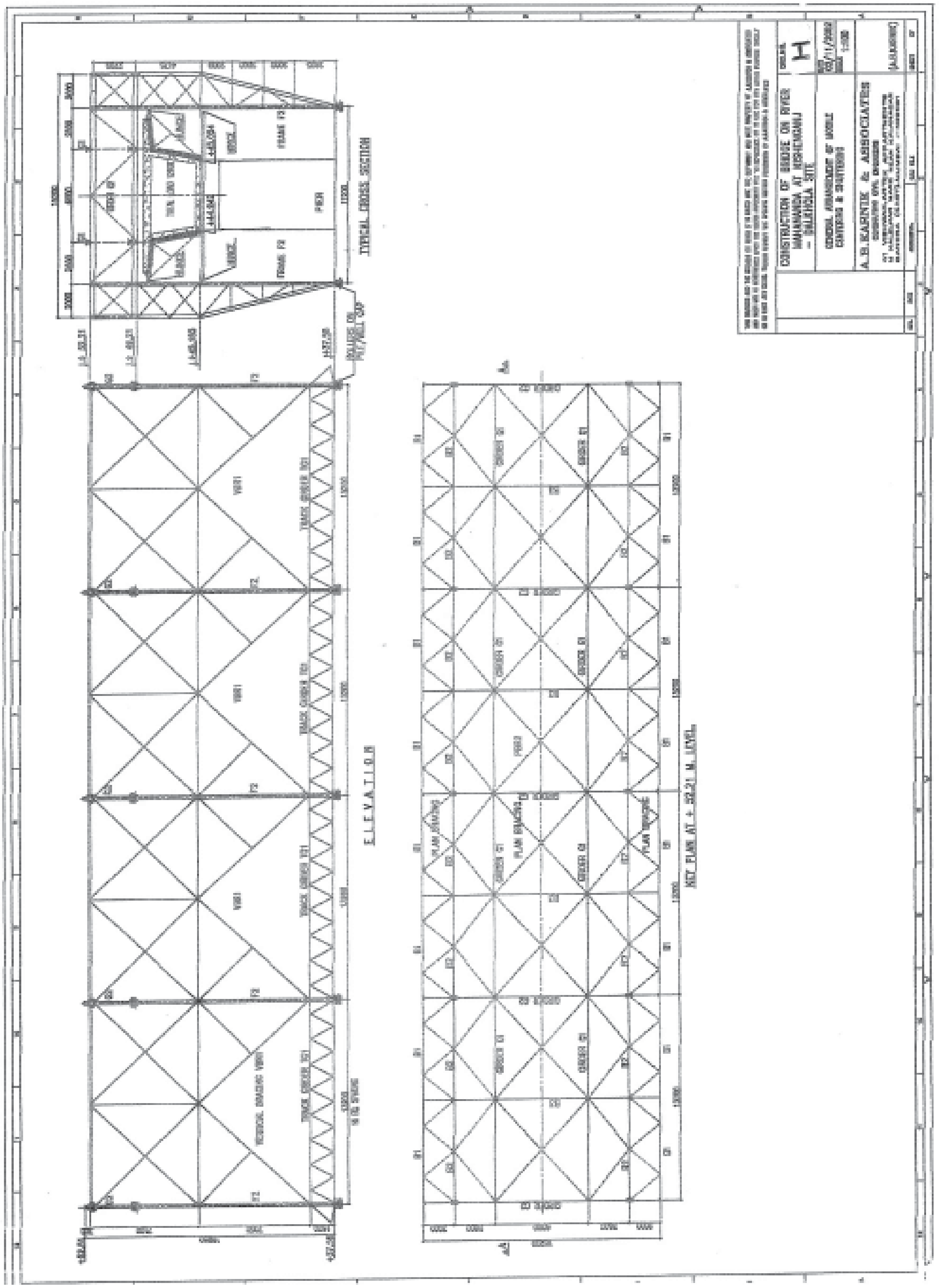
Top Chord of Portal Frame Attached with Rollers at Centre for Movement on Top of Completed Box Deck



View Showing Suspension of Shuttering from Overhead Girder



Detals of Gantry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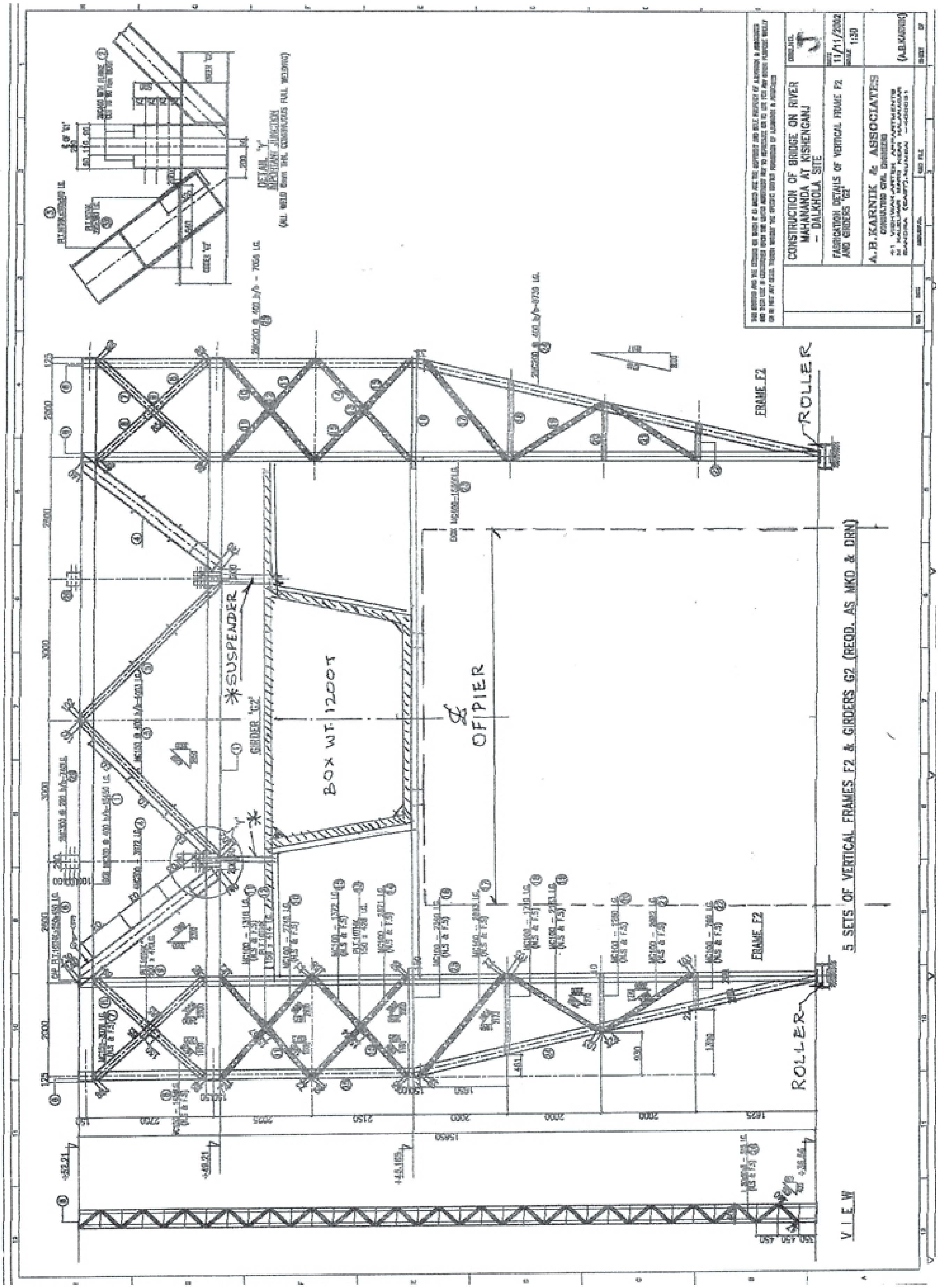
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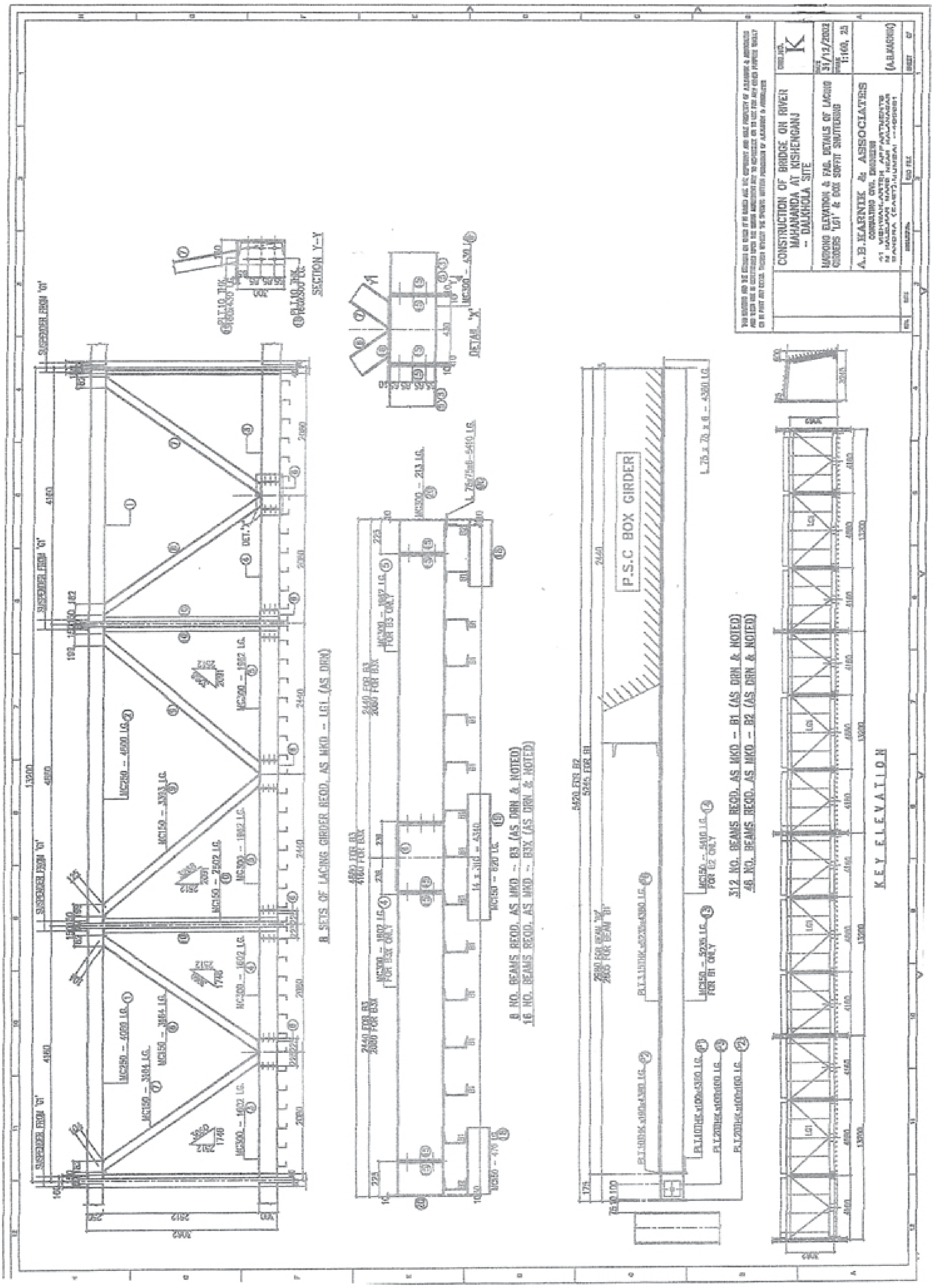
BOX WT. 1200T
OF PIER
FRAME F2
ROLLER
5 SETS OF VERTICAL FRAMES F2 & GIRDERS G2 (REQD. AS MKD & DRN)
VIEW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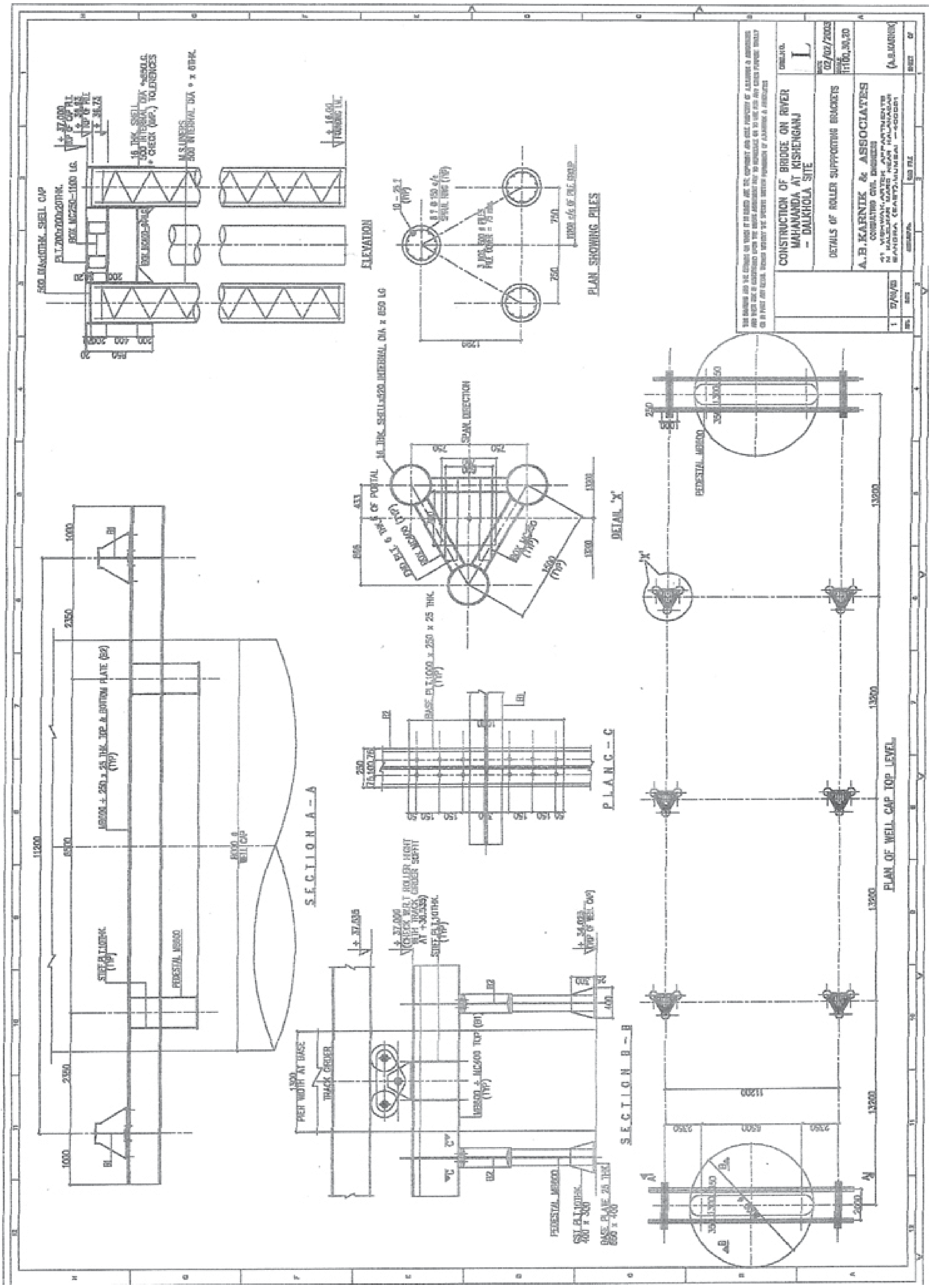
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A.B.KARNIK & ASSOCIATES

ANCHORED RCC DIAPHRAGM WALL FOR BASEMENT NEAR SEA-SHORE AT MUMBAI.

(ENG. D. J. KETKAR)

1. INTRODUCTION :-

A 24- Storeyed building was proposed at Mumbai, on a reclaimed land near the sea shore. During the execution of the work it was anticipated that infiltration of the subsoil water may take place into the basement excavation and subsoil slopes would be unstable. Hence, it was proposed to construct an anchored Diaphragm wall, to act as a retaining wall, around the basement excavation,

General ground level at the site was at R.L. 31.8 metres. Basement was 50 metres x 50 metres in plan and its depth was about 8 metres below ground level. The R.L. at bottom of basement was 23.6 M.

The peripheral stone bund for the reclaimed land was just 10 metres away from the site. Beyond the bund there was Arabian sea.

2. SUB-SURFACE CONDITIONS :-

The soil investigation at the site was carried out prior to this work. Typical subsurface profile, as indicated by the borelog, is presented below.

	Layer	Thickness in metres
a)	Fill-boulders, brick bats, murrum	4 to 5
b)	Bluish grey marine clay (soft to stiff)	1 to 6.5 m
c)	Basalt	Below 6 to 12.5 metres
d)	Ground water table	3.15 to 4.4 m BGL. (which varied with tide)

3. DETAILS OF BASEMENT, DIAPHRAGM WALL AND CABLE ANCHORS

a) Basement : The sectional elevation through basement is shown in figure 1 :

The details are :

Top of basement	:	31.85 metres R.L.
Total depth of excavation	:	9 to 10 m
Dimensions		50 m x 50 m in plan with a sump of 5 m x 5 m towards North –East.

Basement was founded on one metre thick raft. Below the raft 75 mm thick water proofing material was placed.

b) Diaphragm Wall :-

Diaphragm wall was constructed along the periphery of the basement. The details of it are shown in Figure 1. They are as follows.

Total length of wall	:	210 metres
No. of Panels	:	42 nos.
Length of each panel	:	5 metres
Thickness of Wall	:	60 cms
Permissible tolerance	:	8 cms in plan 1 in 80 from verticality.

Depth of the wall depended on the level of founding rock below ground level and varied from 8 to 10 metres.

c) (i) Anchors :

The anchors were made up of H.T. Wires using single wire stressing system. They were installed at angles not exceeding 45^0 to the vertical. Initially, anchors were proposed only in one row with the diaphragm wall extending upto 8.50 m in ground. But rock was encountered at higher level in several panels. Basement excavation was to be carried out deeper than the level of the hard rock. Therefore it was decided that, for the panels which do not go below raft levels, two rows of anchors will be provided i.e. top anchors and bottom anchors. Details of the anchors are given in the table.

4) DESIGN AND CONSTRUCTION OF DIAPHRAGM WALL :-

a) Design :-

i) The design of the RCC diaphragm wall was done as per the load data provided. The horizontal pressure, due to soil and water, acting on the diaphragm wall is shown in figure 1 and is as follows.

Pressure at top of wall : 1.5 t/m^2

Pressure at 8 metres below top of wall = 12.166 t/m^2

No fixity at the bottom end of diaphragm wall was assumed.

ii) To provide lateral support, cable anchors were installed in the diaphragm wall. The capacity and spacing of the anchors was governed by the depth of hard rock below G.L. The anchors were made upto of 7 mm dia H.T.wires. They were inclined at 45 degrees to the vertical and were placed in two rows i.e as top anchors and bottom anchors.

b) CONSTRUCTION PROCEDURE :-

(i) Guide Wall :-

To maintain the alignment of the trenches, guide walls are constructed. They are two reinforced concrete walls having a clear distance the inside faces as 600 mm. Each guide wall is 250 mm thick. The guide wall was constructed upto a depth of 1.5 m.

(ii) Trenching :

A trench to the required depth was first excavated by a specially developed grab, operated from a standard crane by means of a purpose designed attachment including a Kelly Bar and Kelly-Bar guide. The use of Kelly Bar, ensures the correct alignment and vertically of construction. During the excavation trench was kept filled with bentonite suspension, which stabilized the trench walls, preventing caving in. Bentonite suspension was mixed on site and fed into the trench as digging proceeded and its density and viscosity were carefully checked during excavation and concreting.

(iii) **Chiselling and Rock Penetration** : Chiseling was resorted to :

(i) Overcome local obstructions, such as boulders, concrete pieces, etc and

(ii) achieve desired penetration in the founding rock.

Chiselling was carried out by 1.5 tonnes weight chisel having a free fall of one metre. Chiseling time was counted as the time elapsed between commencement of chiseling and stoppage of chiseling, i.e. excluding the period required for enaging of the grab by the chisel or the chisel by the grab. After chiseling the panel was cleared off by grabbing. Penetration at about 4 points along the length of the panel was noted. Average value of penetration was got after averaging out the penetration of 4 points. Average penetration was restricted to twice the value of minimum penetration.

Chiselling was carried out upto the level of hard rock. Hard rock was defined as the stratum where rate of trenching was less than 0.5 m^2 (10 cms. penetration of 5 metres long panel) for 30 minutes of chiselling. Required penetration of diaphragm well in the hard rock was 125 cms. However, this was restricted to about 10 cm. during actual execution. If the diaphragm wall was required to be stopped at higher elevation on account of encountering the hard rock, and if excavation for the basement was to be continued further below, then additional anchors were to be provided at a lower level in the diaphragm wall.

Reinforcement : Reinforcement as required by design was made and lowered in the form of a cage. Additional bars were used as diagonal chairs, stiffeners etc. to impart stiffness to cage. Rolling spacers made of mortar were used to allow clear cover of 50mm to the cage. Before the reinforcement cage was lowered, a circular steel stop- end pipe was placed at the end of panel. The pipe was gradually removed after the concrete had partially hardened, leaving a semi – circular shape in the concrete to form a joint between the panels.

For the trench depths of 7 meters, 8 m 9 m and 10 m, reinforcement cages of 6 m, 7 m, 8 m, and 9 m respectively we provided. But if the trench depth exceeded the above value by 0.3 metres, then the next higher length of cage was provided ; i.e. for 7.3 metres trench depth reinforcement cage of 7 m length was used.

Concreting : Concrete was poured in a continuous fashion through a tremmie pipe introduced initially upto the bottom of the trench. Only one tremmie pipe was used to avoid entrapping of bentonite slurry. The mix was designed to flow easily and without any tendency to segregate, the slump being 18 cms. The concrete mix was $1 : 1\frac{1}{2} : 3$ and was volume batched M:20. The cement consumption was 400 kg / m^3. Maximum size of the coarse aggregate used was 20mm to ensure better flow of concrete.

5) Sequence of Construction of cable Anchors with respect to excavation for basement :

a) After the construction for the diaphragm wall panels the anchors were to be installed at 45^0 through pre-formed holes in RCC Diaphragm Wall. For installing the anchors excavation of the inside soil of basement was to

		Top Anchors				Bottom Anchors			
Case	**Depth to rock (m)**	**Capacity**	**Hori. Spacing (t)**	**Dist. Below Wall Top (m)**	**Length In Rock (m)**	**Capacity**	**Hori. Spacing (t)**	**Dist. Below Wall Top (m)**	**Length In Rock (m)**
I	5 to 6	50	2.5	1.25	11.00	130	2.50	3.75	16.50
II	6 to 7	60	2.5	1.25	11.00	155	2.50	4.75	17.00
III	7 to 8	75	2.5	1.25	11.00	185	2.50	6.00	17.00
IV	8 to 9	90	2.5	1.25	11.00	144	1.66	7.10	17.00

be carried out. Sufficient open area of about 10 metres. width was maintained inside the perimeter of the disphragm wall. Therefore, the work was generally carried out from outside the diaphragm wall line. Initial excavation inside the face of diaphragm wall was commenced after keeping 5 metres margin for installation of anchors. Along with progressive construction of diaphragm wall this earth from inside the wall was excavated up to a depth of the top anchor level. The anchors were then installed and stressed upto 50% load. Subsequent excavation was commenced after 7 days of stressing the anchors.

b) Sequence of Operation for installation of Cable Anchors :

(1) Take up top anchors, for each pound.

- 1 . 1 Fix up cable head plate, with pins wherever required.
- 1 . 2 Drill through the cable head pipe.
- 1 . 3 Drill and case the hole through overburden.
- 1 . 4 Extend drilling in the rock, to the specified depth.
- 1 . 5 Place grout in the hole to stabilize.
- 1 .6 Clear the hole.
- 1 .7 Fabricate and lower the cable in the hole and put the bottom anchorage grout.
- 1 . 8 Fix top cable head.
- 1 . 9 Fix the stressing head.
- 1 . 10 Stress to 50% of design load.
- 1 . 11 For a given panel, finally grout the cable holes, after the bottom anchors are stressed.

(2) Take up bottom anchors, for the respective pound

- 2 . 1 Same as 1.1 to 1.9 above
- 2 . 2 Stress to 100% design load.
- 2 . 3 Finally grout the cable hole

(c) Insert for anchor heads with stressing plate :

The anchor top heads provided were of two types : (1) with insert plates. (2) with shear pins

(Typical data for 75 T caoacity anchor)

(i) **With Insert plates** : (Refer Fig. 5)

Along with the reinforcement cage of the diaphragm wall, 2 Nos. 16 Bars about 1100 mm long were provided as shown in Figure 6. A. 12 mm thick insert plate was welded to these bars. For placing the anchor head, insert plate was exposed by chipping off the concrete upto that level. A skew plate was riveted to the insert plate and care was taken to see that the holes for both insert and skew plate were drilled. Anchor top head consisted of :

(1) The stressing plate – 275 mm x 275mm x 50 mm thick.

(2) Bearing plate 375 mm x 375 mm x 12 mm thick. It has a 150 mm diameter pipe at the centre and 5 numbers stiffeners 100 mm 16 mm thick.

(3) 2 Anchor Blocks.

(4) Skew plate 375 mm x 525 mm x 12 mm thick with 150 mm dia. pipe at the centre and 5 nos. stiffeners as above.

After the skew plate is fixed in position, anchor blocks, bearing plate and stressing plate were welded as shown in the Figure 6.The cable consisted of 21 nos. H. T. Wires. One Wire was passed through one hole (of 8 mm dia) of the stressing plate. Barrels and wedges were used to hold the stressed wires. Vertical support to the stressing plate was given by the stiffeners.

(ii) Anchor top head with pins :- (Refer Fig. 6)

Wherever it was not possible to locate the insert plates, skew plates with shear pins were provided. Generally, four shear pins (two at top and two at bottom) were provided in one skew plate.

The diameter of the shear pins depended on the capacity of the anchor, as shown in Figure 7. Procedure for fixing anchor top head with pins :

(1) Drill 50 to 75 mm dia. hole to 450 mm depth using template.

(2) Insert cement grout at w /c ratio less than 0.5.

(3) Locate the skew plate .

(4) Insert dowel bars.

(5) Top up the grout .

6. STRUTS IN CORNER POCKET INSTEAD OF CABLE ANCHORS :-

As shown in figure 1, there is a sump of 5 m x 5 m in the North-East direction. Due to limited space it was not possible to drill for anchor in the opposite walls of sump. Therefore, it was decided to provide temporary struts for panels forming the sump chamber. Initially it was decided to provide RCC struting but later structural steel struting was preferred as it is faster and cleaner. Total 4 struts were fixed, two at top anchor position and two at bottom anchor position as shown in fig 7. Total work involved in fixing one strut in sump area was :

1) 2 Base Plates were fixed on either side by breaking concrete and welding it to main reinforcement.

2) 2 End plates (500 mm x 500 mm x 12 mm thick) fixing and welding to base plate.

3) End plates to be welded on all sides on either ends of 2 ISMC (250 mm x 250 mm 80 mm).

4) Two ISM channels to be welded face to face plus weld gusset plates (6 nos. 100 mm x 260 mm 12 mm thick with 40 mm ole) at quarter points on both sides of the joints.

5) Weld hook plates (4 Nos. 100 mm x 180 mm x 12 mm thick with 40 mm hole) at the end of ISMC.

It was carefully noted that only after the struts were fixed in the line of top anchor position, the excavation below top row of cable anchors was taken up.

7. QUANTITIES OF WORK :-

	Description	Quantity
(i)	Construction of nominal 55 cms thick RCC Diaphragm Wall upto max. 9 m. depth or Upto weathered rock whichever occurs earlier	1372 m^2

(ii) Inclined anchors

Anchors capacity	Total Anchor length
50 T (18 nos.)	331 M
60 T (37 nos.)	701 M
75 T (18 nos.)	362 M
90 T (2 nos.)	43 M

(iii)	150 T capacity anchors inclined at 45^0 (57 NOS)	1178 M
(iv)	200 T capacity anchors inclined at 45^0 (18 nos)	370 M
(v)	Provide and fix struts in sump.	4nos.

8. OBSERVATIONS AND PERFORMANCE :-

a) **Surface of Diaphragm Wall** :-

The surface of the wall was exposed after the excavation for the basement was complete. The exposed surface presented an excellent quality of concrete which was dense and uniform. Joints between the two panels were also straight and uniform. Bulging of concrete towards the upper side of the wall was seen at some places on the exposed surface. Bulges of concrete which were outside the tolerance limit were chipped off. No depressions were seen on the surface.

b) Contact of Diaphragm Wall with Rock :-

The contact of diaphragm wall with rock was exposed because the excavation for the basement proceeded much below the base of the wall. Blasting was carried out to loosen the hard rock in the basement. Exposed surface presented a good bond between rock and concrete. No foreign matter was found in the contact zone.

c) Water inflow from the Diaphragm Wall and rock contact :-

The wall was water-proof, as inflow of water from the panel joints or from the contact zone of the Diaphragm Wall with rock was practically nil and was prevented because of good alignment of the panels, good quality of concrete and a good bond between the rock and wall.

(d) Overbreak and excess concrete in the Diaphragm Wall :

Bulges of concrete above the designed cut off level (RL 29.2 m) and beyond the limit of tolerance on the sides of the Diaphragm wall were chipped off. Weak concrete mixed with bentonite or loose concrete was also chipped off. Breaking of bulges was carried out with two pneumatic breakers. Amount of concrete braking required =21.75 cu-m

Amount of concrete build up required for true geometry = 6.45 cu-m

Due to excessive chiseling the over-breaking was high, which resulted in extra cement consumption of around 15 %. Also due to repeated chiseling (to found the panels on hard rock), the progress dropped down considerably.

9. CONCLUSIONS :-

The diaphragm wall constructed offered excellent protection against sliding of overburden fill and ingress of water during subsequent deep excavation through recently reclaimed ground and very close to sea.

The anchored diaphragm wall facilitated basement construction in a clear space.

****

D. J. Ketkar

(Born 16-11-1936)

Academic Qualification :

Bachelor of Engineering (B.E.), Civil, University of Bombay, 1959.

Master of Technology (M.Tech), Soils Indian Institute of Technology, Bombay, 1964.

Diploma in Business Administration, Bombay, 1972.

Membership :

F.I.E., F.I.G.S., M.ISEG, M.I.IBE, M.IGGS.

Work Experience :

A) Over 35 years in M/s. Cementation / Ceminidia / Trafalgar House Construction Company Ltd.

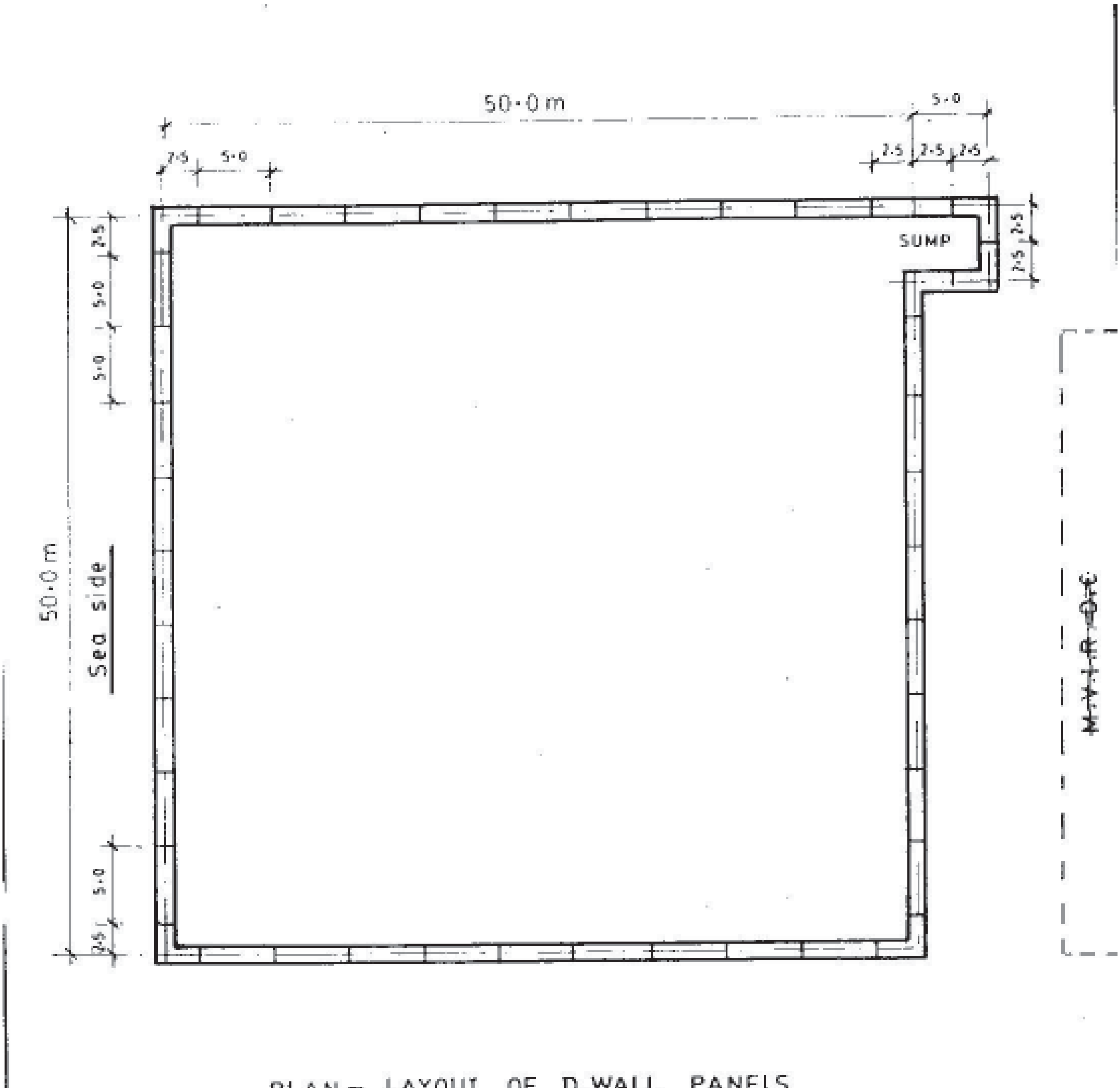
B) Experienced in design and Construction of Foundation Engineering Projects involving Land / Marine Soils Investigation, Drilling-Grouting-Anchorage's Underpinning, Piling, Diaphragm Walls, Chemical Grouting, Ground Improvement (Sandvicks, Sand-drains, Vibroflotation, Stone-columns, Anchor bars), Slope stabilisation Underwater concrete construction, Rehabilitation of Dams etc.

C) Head of Research and Development for over 10 years.

D) Worked as Manager Corporate Development since 1985 and retired as Dy. General Manager, Corporate Development and R & D, in 1995.

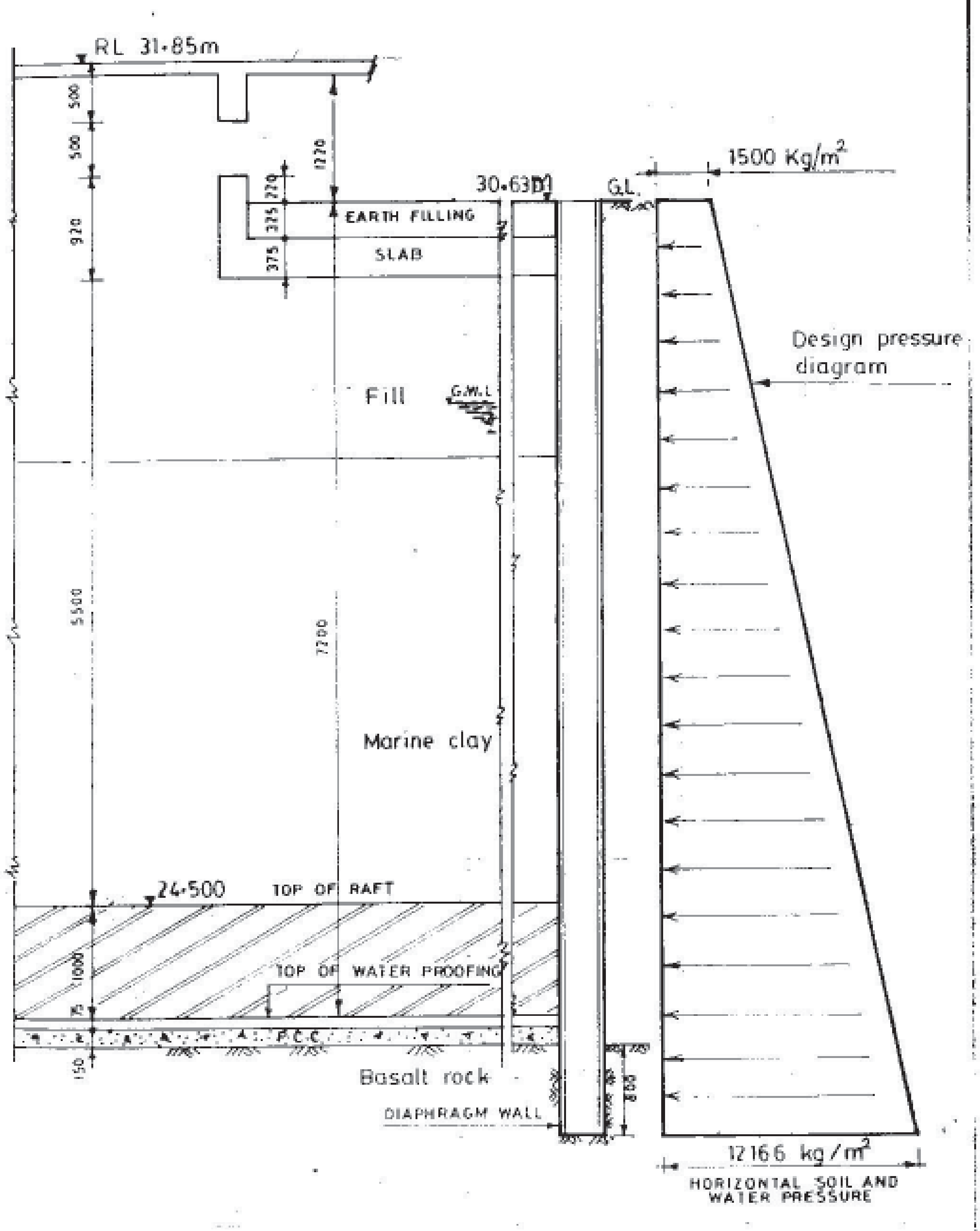
OVERSEAS EXPOSURE :

4 months training in the U.K., Projects-In-Charge for consultancy in soil investigation in Algeria, handled soils contracts in Sri Lanka, Baharain and Iraq.



PLAN – LAYOUT OF D.WALL PANELS

Figure: 1



DIAPHRAGM WALL PROPOSAL

Figure 2

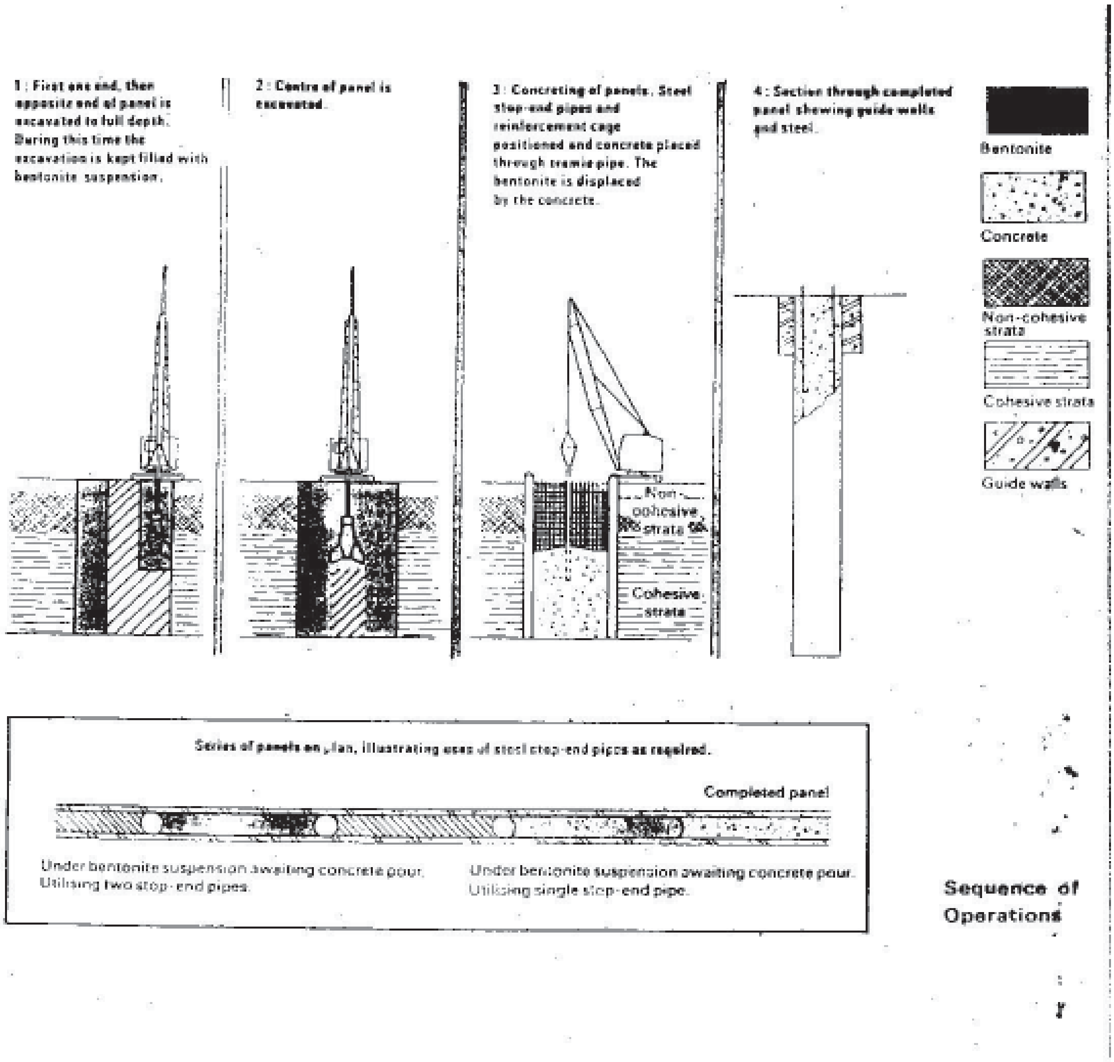
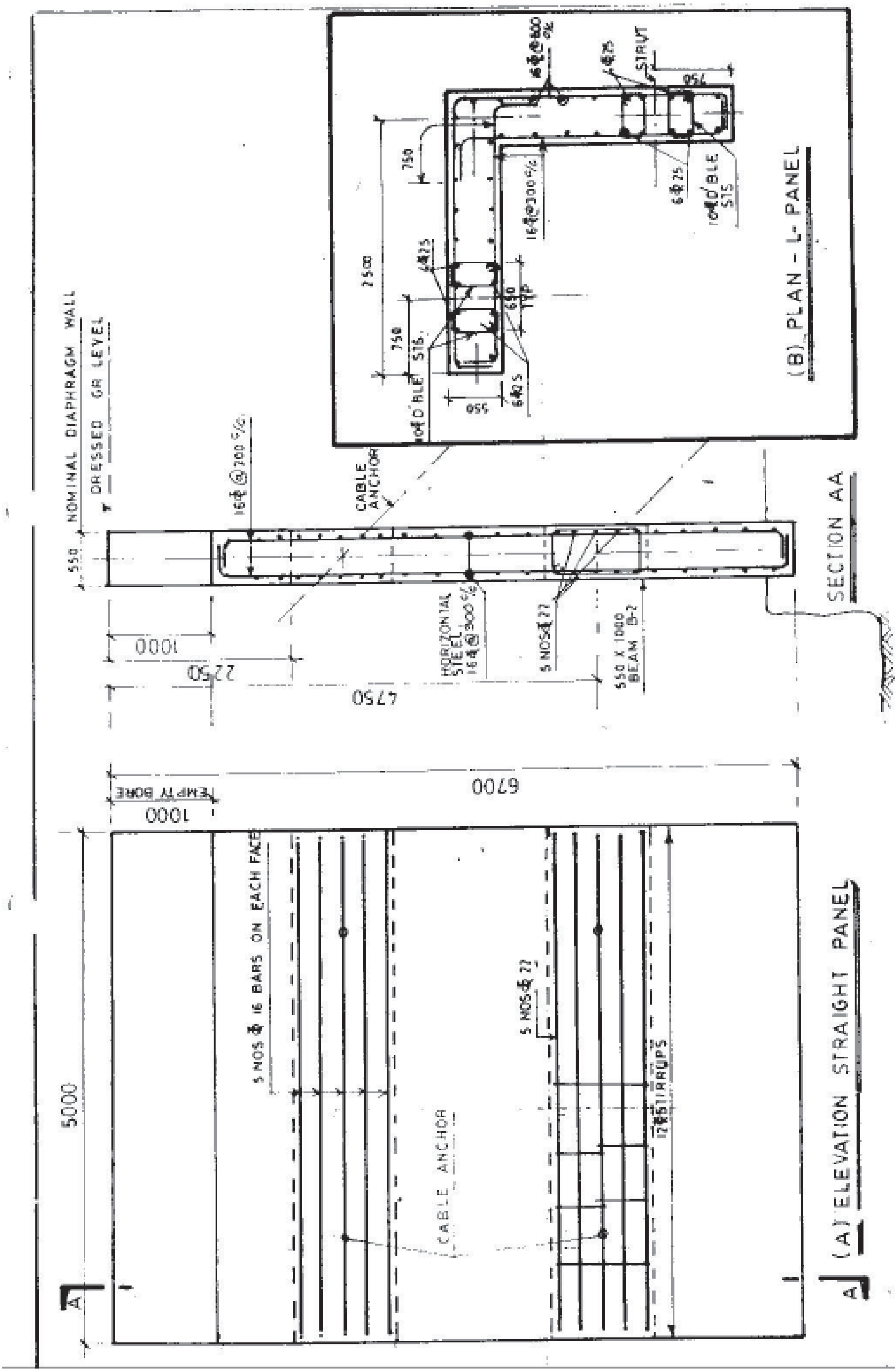


Figure : 3

APPENDIX

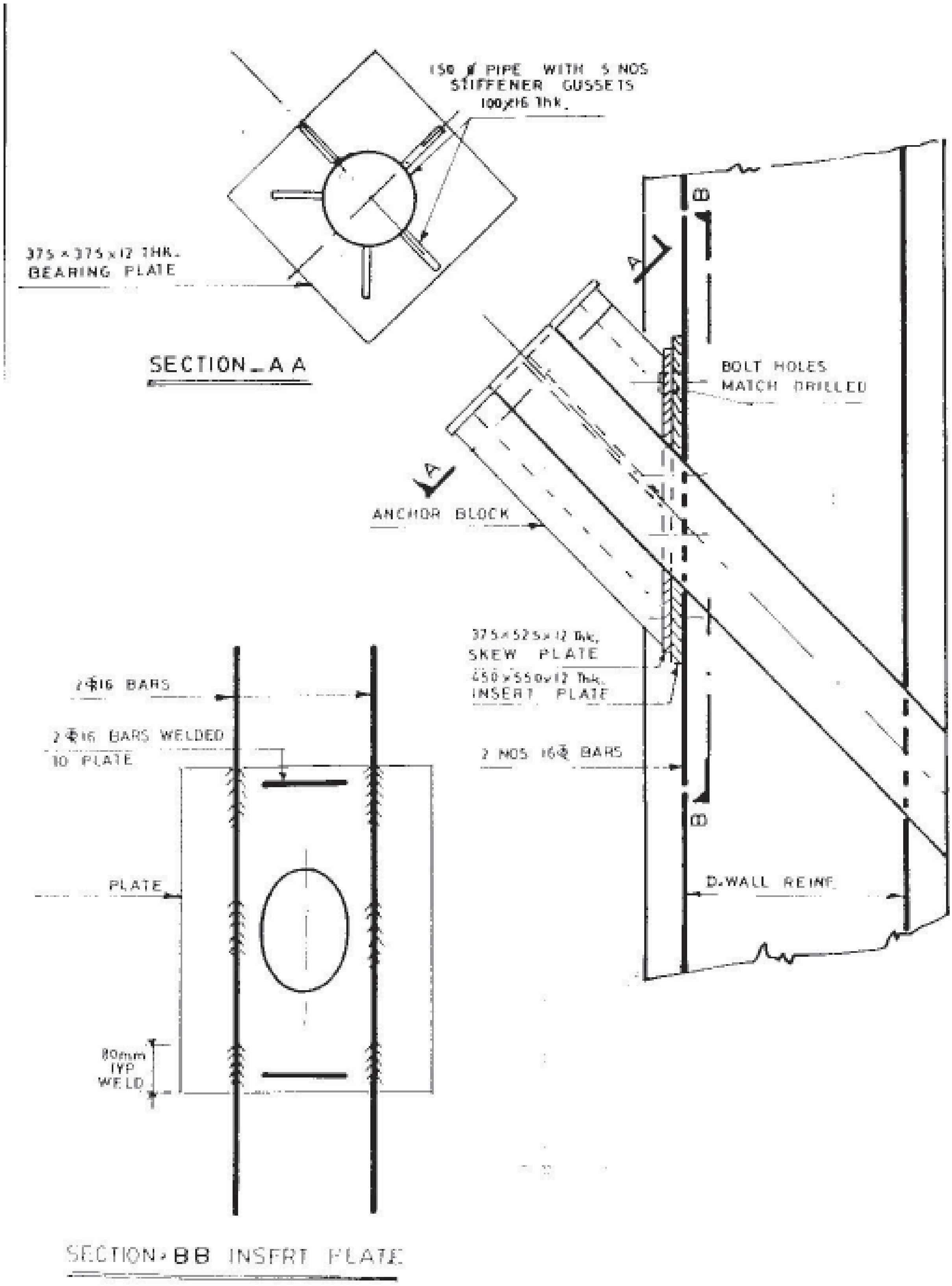
SIMILARLY FOR OTHER ANCHORS

Anchor Capacity	No. of H.T. Wires	Ancharage Length (mm)	Stress brg. Plate			Dimeter of stand Pipe (mm)
			L (mm)	D (mm)	T (mm)	
50	14	10.8	250	250	40	150
60	17	10.8	250	250	40	150
75	21	10.8	275	275	50	150
90	25	11.0	275	275	50	150
130	33	17.0	300	300	60	150
155	42	17.0	300	300	62	150
185	150	17.0	300	300	62	150
144	36	17.0	300	300	60	150



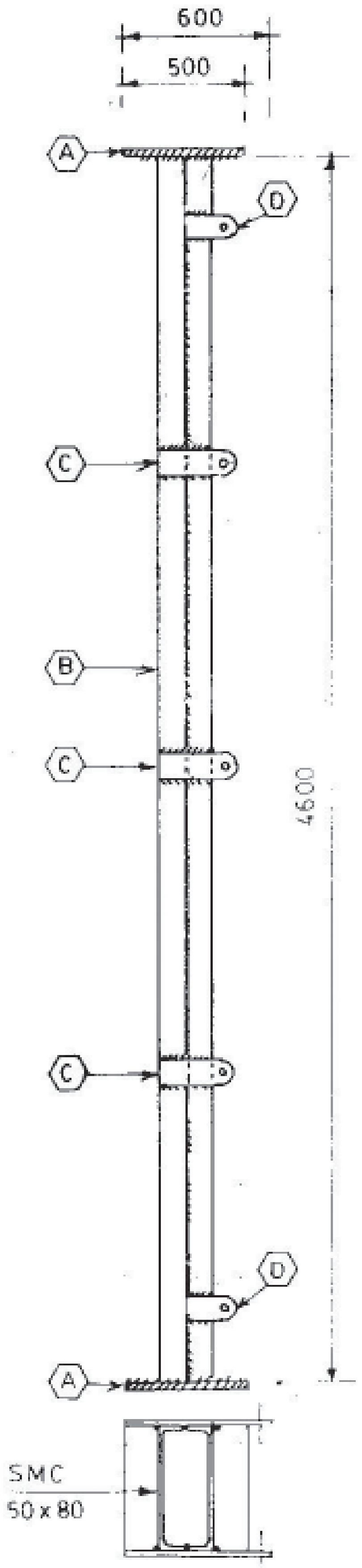
DETAILS OF REINFORCEMENT CAGE IN DIAPHRAGM WALL PANELS

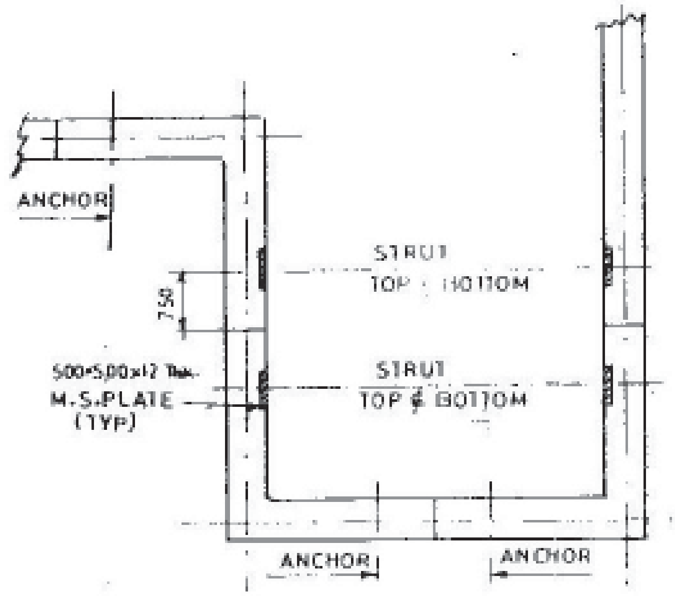
Figure - 4



TYPICAL ANCHOR DETAIL

Figure: 5


600
500
A
D
C
B
C
4600
C
D
A
SMC
50 x 80


ANCHOR
STRUT
TOP & BOTTOM
750
500×500×12 Thk.
M.S.PLATE
(TYP)
STRUT
TOP & BOTTOM
ANCHOR
ANCHOR

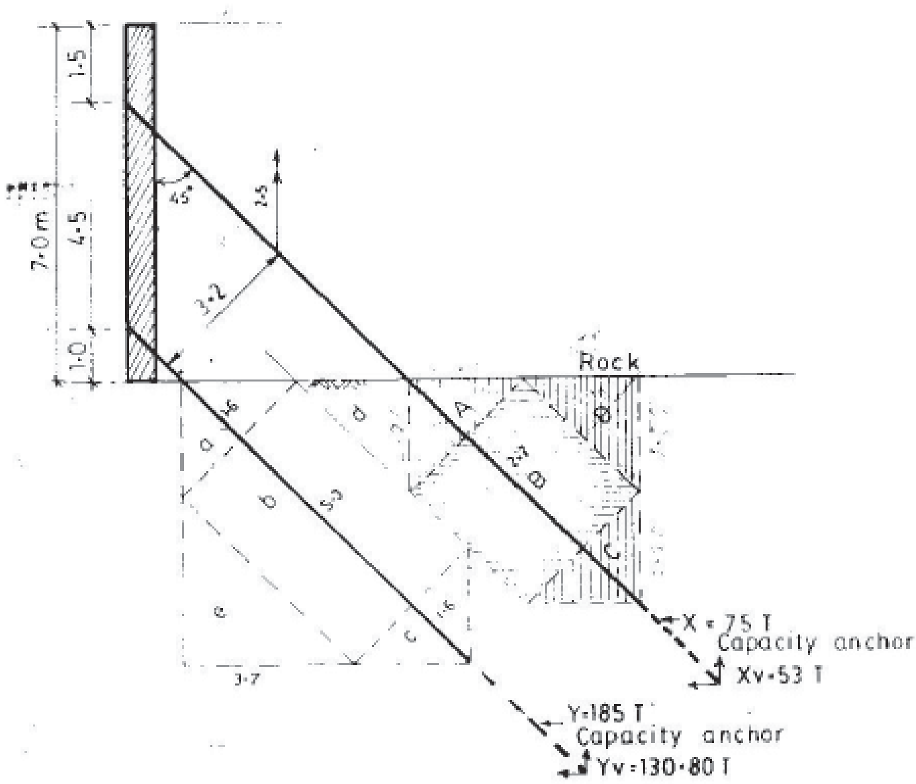

1.5
7.0m
4.5
1.0
45°
2.5
3.2
Rock
A
B
C
a
b
c
d
e
3.7
X = 75 T
Capacity anchor
Xv = 53 T
Y = 185 T
Capacity anchor
Yv = 130.80 T

Figure: 8

Typical Anchor Design Calculations

Assumptions :

(1) Shear strength of rock is neglected.

(2) Weight of overburden is neglected.

For 75 T Anchor :

Volume of A + B + C + D = 29.03 m^3

Total weight of rock = 29.03 x 2.6 = 75.48 T

Now : Vertical anchor load = Tv = 53 T

∴ S. F. = 75.48 / 53.00 > 1.25

∴ O. K. (cl. 5.2.2. IS 10270)

∴ Anchor length in rock = (2.2 + 3.2) X 2 = 10.8 m

For 185 T Anchor :

Volume of a + b + c + d + e = 65.10 m^3

Total weight of rock = 65.10 x 2.6 = 169.26 T

Now : Vertical anchor load = Tv = 130.80 T

∴ S. F. = 169.26 / 130.80 > 1.25

∴ O. K.

∴ Anchor length in rock = (5.3 + 3.2) X 2 = 17.0 m

Notes :

(A) Bearing (End) Plates 500 x 500 x 12 THK = 8 Nos.

(B) ISMC 250 (250 x 80 m) = 8 Nos.

(C) Guseat Plates 100 x 260 x 12 THK with 40 Hole = 24 Nos.

(D) Hook Plates 100 x 180 x 12 THK with 40 Hole = 16 Nos.

(A) To be welded on all sides on either ends of (B).

(B) Two ISM Channels to be welded face to face plus weld 100 x 260 x 12 m guest plates at quarter points on both sides of the joint.

Design of Cable Anchor and Stressing Plates

Typical Design for 75 T Cable Anchor :

(1) Number of H. T. Wires regerired :

Basis :	Diameter of wire	=	7 m
	UTS of wire	=	150 kg/mm^2
	Working Stress	=	70% of UTS

Wires can be stressed upto

150 x 0.7 = 105 kg/mm^2

∴ Capacity of each H. T. Wire.

= π / 4 x (7)2 x 105 = 4040 kgs = 4 T say

∴ For 75 T capacity anchor

No. of wires required = 75 / 4 = 19 (say)

provide 21 nos.

(2) Anchorage Length : (Figure 8)

Volume of rock for 75 T Anchor :

A + C	= (3.2 x 2.5 x 1/3 x 1.6)2	=	8.53 m^3
B	= (3.2 x 2.5) 2.2	=	17.60 m^3
D	= (1.98 x 2.26 x 2.5) 1/4	=	2.90m^3
			29.03m^3

Density of rock = 2.6 T/m3

Total weight of mass of rock = 75.48 T

Anchor Capacity = 75.00 T

∴ Vertical load = 75 x 0.707 = 53 T

∴ S. F. = 75.48 / 53 = 1.42 > 1.25 ∴ O.K. (cl. 5.2.2. IS 10270)

∴ Anchor length in rock = (2.2 + 3.2) x 2 = 10.8 m

Calculations for 185 T anchors :

Volume of rock :

a + c	= 8.53 m^3		
b	= (3.2 x 2.5) x 5.3	=	42.4 m^3
d	= 2.9 m^3		
e	= (5.3 x 3.7 x 2.3)	=	11.27 m3

∴ Total volume of rock = 65.10 m3

∴ Total weight of rock = 169.26 T

Anchor capacity = 185 T

Vertical load = 185 x 0.707 = 130.80 T

∴ S. F. = 169.26 / 130.80 = 1.29 > 1.25 ∴ O.K.

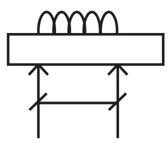
∴ Anchor length in rock = (5.3 + 3.2) 2 = 17 m

(3) Stressing Plate :

(Preference : Theory of Plates and Shells Timeshenko and Woinowsky Krieger, International Student Edition II 1983)

Case : Uniformly loaded circular plate with supported edge (page 56-57)

P = 75 T



Za = 15 cm

πa^2 q = p

= 0.3

Mr = Mt = $(3 + \eta)\ q\ a^2 / (16)$

= $(3 + \eta) / (16) \times P / \pi$

= 4921.9 kg-cm / cm.

now M = f n = fbd2 / b

f bending = 1575 kg / cm^2

For b = 1 cm

d = $\sqrt{\theta 1 / bf}$ = 4.33 cms = 43.3 mm

∴ Provide 50 mm thick Flate.

CIRCULAR FROM B.M.C.

Sub. : Applicability of I.S. Code 13920-1993 for the Buildings under development in Municipal Limits.

It is observed that the I.O.D. Condition enforcing the provisions of I.S. Codes like I.S. 456-2000, 13920-1993, 4326-1993 and 1893 of 2002 is already in operation and the Building Regulations and Bye Laws of M.C.G.M. are still in force. But it is observed that these codes and regulations are not put into practice by the Architects / Structural Engineers.

Hence, with Immediate effect the following requirements shall be fulfilled.

1. The external filler and/or non-load bearing walls, if in brick masonry, shall be of minimum 9" (230 mm) thick. These can be of minimum 150 mm thick autoclaved cellular light weight concrete blocks OR solid concrete blocks. In case of centrally reinforced R.C.C. walls, the minimum 4" (100 mm) thickness in M : 25 grade of concrete shall be provided.

2. a) The minimum thickness of Main framing Beams, Columns including Shear walls shall not be less than 230 mm. thick. Where any special structural system for design and/or construction is adopted, the minimum dimensional parameters as above shall not be applicable and they shall be as per relevant Indian Standard Codes.

   b) In an event of any such special structural system and construction system for better and/or speedy construction are proposed; the sectional dimensions to be adopted will be in conformity with relevant Indian Standard Codes, and/or other relevant and substantive International Codes. In such situation the structural calculations shall be exhaustive for Analysis, Design and Detailing relevant to such codes.

3. For parts of structure being "Cantilever Projections", forming "Vertical Projections" like Towers, Tanks, Smoke Stacks and other vertical projections attached to buildings, cantilever projections above the roof, shall be designed and checked for stability for five times the design horizontal seismic co-efficient i.e. 5 x Ah as per clause 7.12.2.1 of IS : 1893 (Part - I) : 2002 and "Horizontal Projections" like cornices and balconies, shall be designed and checked for stability, in accordance with clause 7.12.2.2 for five times the vertical seismic co-efficient i.e. 2/3 x 5 x Ah.

This circular supercedes the earlier circulars under No. CE / PD / 12387 / I of 17-03-2005, No. CE / 559 of 15-04-1974, and No. CE / 32066 / II of 23-12-1975.

All concerned are requested to take the note of this Circular.

03-01-06	05-01-06	12-01-06
Dy. C. E. (P & D) City	**City Engineer**	**Dir. (E. S. & P.)**

Announcement from A.C.I., India Chapter

Available Concrete Construction - Books

"India Chapter" offers flexible options with set of books on themes of current interest - Non Destructive Testing, Earthquake Resistant Structures, Ready Mix Concrete, High Volume Fly Ash Concrete, for 21st Century, Waterproofing, Sir Adam Neville's Point of View of Concrete, Forensic Engineering and lot more. Each category has a set of books to choose from. It is Users household approach for knowledge propagation with Incentives for professional development. Stock is limited - check through our website : www.icaci.com

or contact on e-mail : icaci@vsnl.net Phone : (091-22) 2446 9175 / 3095 7023.

Fax : (091-22) 2446 0760