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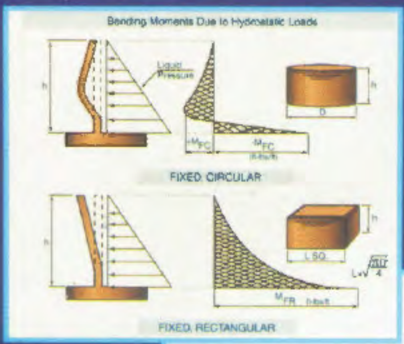
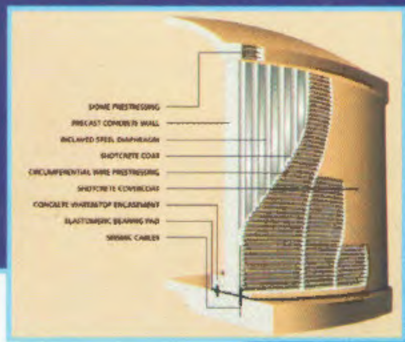
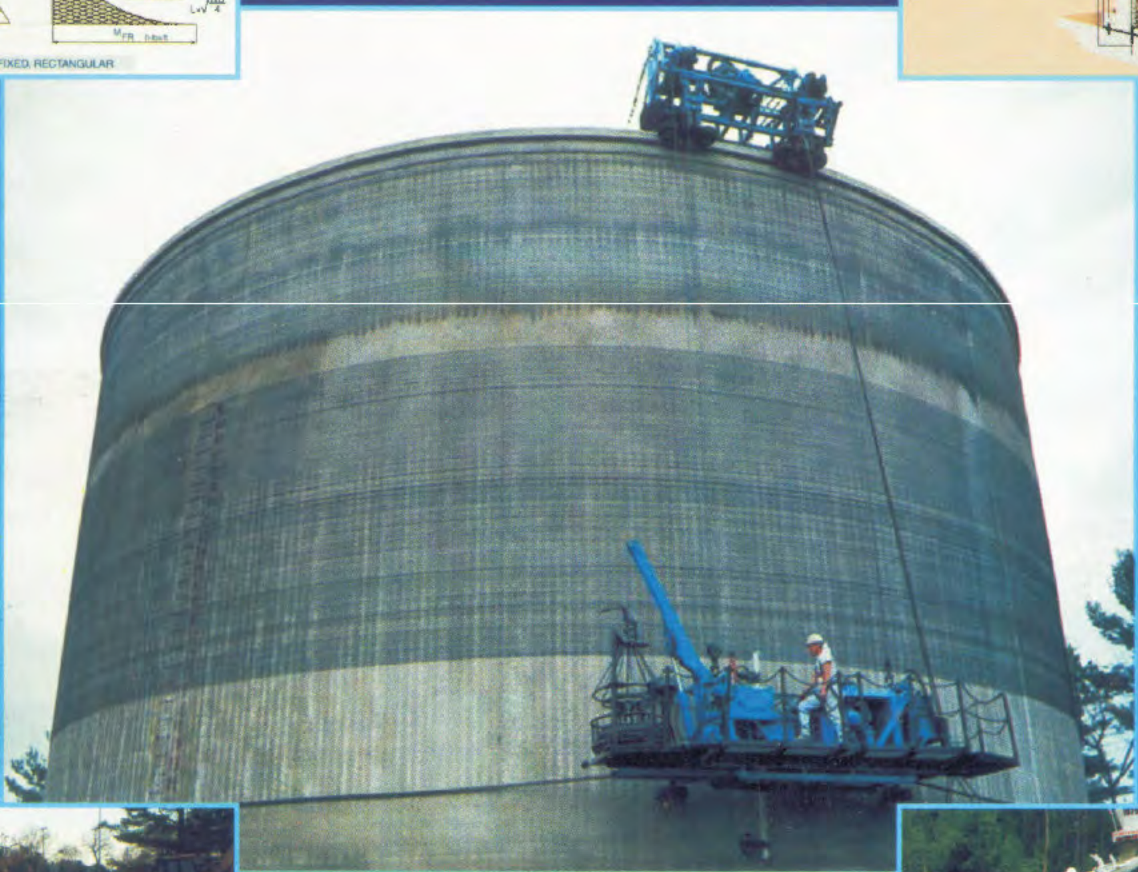
VOLUME 14-2

Apr-May-Jun-2012







Design and construction of wirewound circular precast prestressed concrete tank

(See page 10)

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Fraternity News

WELCOME TO NEW MEMBERS

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M- 1180 Rajnish Talakshi Boyad

M- 1181 Elizer Saure Abrea

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M- 1196 Nishant Yadav

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Patrons : 29	Organisation Member : 20	Sponsors : 8
Members : 1178	Junior Members : 11	

TOTAL STRENGTH 1246

FIELDS CONSIDERED AS ASPECTS OF STRUCTURAL ENGINEERING

- Structural Designing & Detailing
- Computer Software
- Materials Technology, Ferrocement
- Teaching, Research & Development
- Rehabilitation of Structures
- Construction Technology & Management
- Geo-Tech & Foundation Engineering
- Environmental Engineering
- Non Destructive Testing
- Bridge Engineering

& Other related branches

AIMS & OBJECTIVES

1. To restore the desired status to the Structural Engineer in construction industry and to create awareness about the profession.
2. To define Boundaries of Responsibilities of Structural Engineer, commensurate with remuneration.
3. To get easy registration with Governments, Corporations and similar organizations all over India, for our members.
4. To reformulate Certification policies adopted by various authorities, to remove anomalies.
5. To convince all Govt. & Semi Govt. bodies for directly engaging Structural Engineer for his services.
6. To disseminate information in various fields of Structural Engineering, to all members.

EFFECT OF GRADE OF STEEL ON LOAD CAPACITY OF COMPOSITE SLABS

Pragnya Shah,Brijesh Pandya & Harismita Pathak

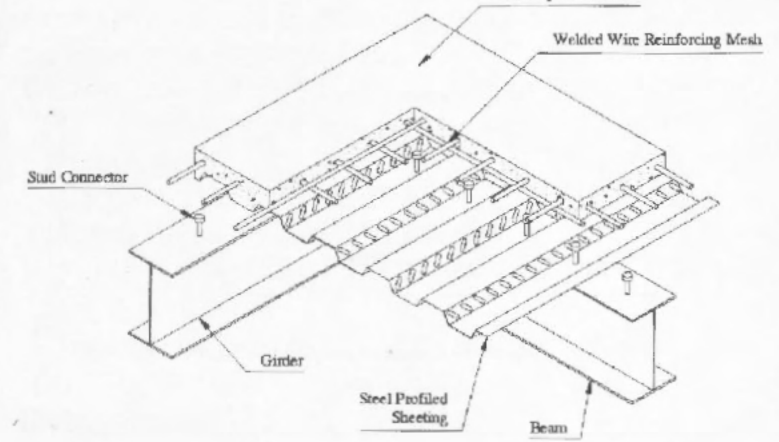
INTRODUCTION

The substantial increase in India's steel production has motivated the structural steel consumption in construction industry to use more technologically efficient construction Practices. With the demanding needs for infrastructure development in today's scenario, builders and designers in India are looking beyond the conventional methods and exploring Robust, Fast and Economical Practices to win over today's challenges.

Composite deck slab construction is one of the viable options. The light gauge profiled steel deck is manufactured by Cold forming process. It is available in various thicknesses ranging from 0.8 mm to 1.6 mm with galvanized surface coatings. The embossment is provided within the profile at the time of forming which will develop a shear bond with the concrete & resist sleep of concrete over the profile. The base metal is available in various grades having yield strength of 250, 345 & 550 MPA. It is used in composite reinforced concrete slabs as load carrying structural members in steel frame high rise buildings, Sky walks, Fly over, Parking sheds, Industrial floors etc.

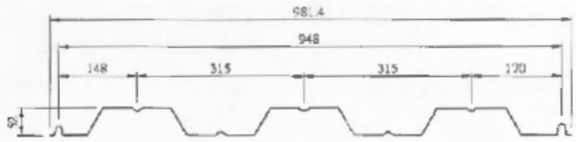
The composite slabs formed using profiled deck offers faster construction, serves as permanent formwork for concrete, a working platform and tensile reinforcement to a concrete slab.

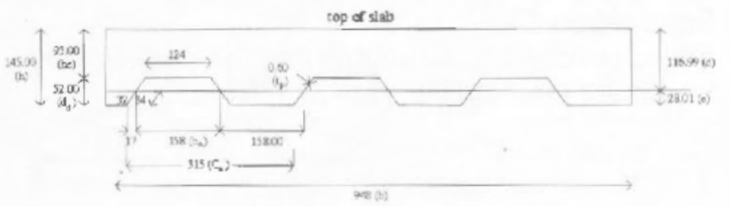
This paper aims in analyzing the effect of



Steel grades as mentioned earlier to the load carrying capacity of the composite slab in terms of safe span.

Met form Deck Profile





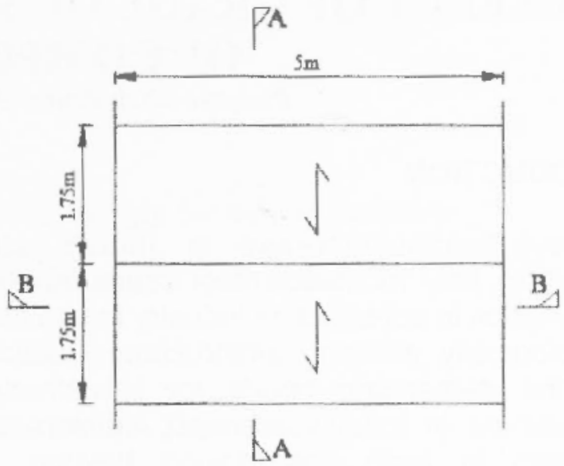
Cross Section of Composite Slab

Design Thickness (mm)	Profile Weight (kg/m)	Area of Steel (mm^2/m)	Height to neutral axis from bottom (mm)	Moment of Inertia (mm^4/m)
0.8	0.08	978	28.09	480000
1	0.1	1222	28.17	600000
1.2	0.12	1465	28.25	720000
1.6	0.16	1954	28.35	940000

PARAMETERS FOR THE ANALYSIS & DESIGN

- Type of sheet: Profiled Steel Sheet "52"
- Thickness of sheet: 0.8 mm, 1 mm, 1.2 mm, 1.6 mm
- Thickness of composite slab: 120 mm, 145 mm, 240 mm
- Grade of steel sheet: 250 N/mm^2, 350 N/mm^2, 550 N/mm^2

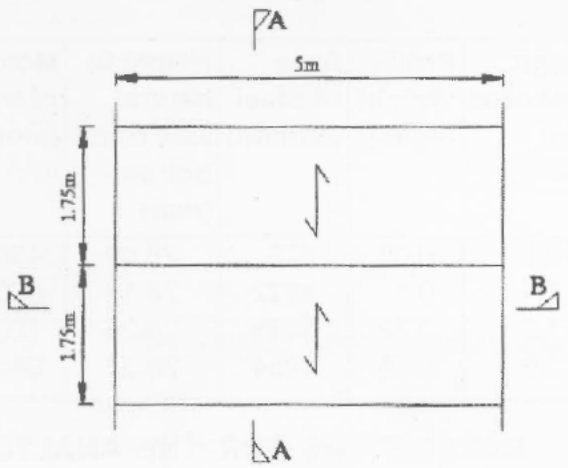
- Grade of concrete M25
- Shear Stud Details:25 mm diameter ;100 mm high
- Reinforcement Detail: 8 mm diameter @ 100 mm c/c – at supports
  8 mm diameter @ 315 mm c/c – at bottom transverse
- Span: Double- Un propped



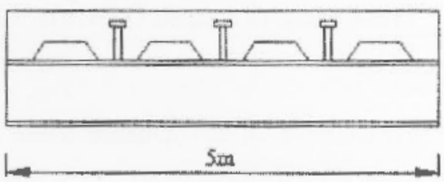
PLAN

DESIGN EXAMPLE

Design of Composite slab with profiled steel sheeting (As per BS 5950: part 4, Euro code 4 and IS 456:2000 wherever applicable)
Span of slab = 1.75 m
Span of beam = 5 m
The characteristic strengths and the partial safety factors for materials (γ_M) are as follows:



PLAN



SECTION B-B

Profiled steel sheet: f_{yp} = 250 N/mm^2, γa = 1.1
Concrete; cube strength: f_{ck} =25 N/mm^2, γc = 1.5
Reinforcement; yield strength: f_y=415 N/mm^2, γ_s = 1.15
Structural steel; yield strength: f_y=250 N/mm^2, γ_a = 1.1
Shear connector; 25-mm headed stud, 100 mm high; ultimate strength=540 N/mm^2, γv = 1.25
Resistance of the shear connector Q_k = 93.5 kN (IS: 11384-1985 Clause 9.3 Table 1)
(25 mm diameter shear connector 100 mm high and M25 grade concrete)
Dead loads
Floor finish = 1 kN/m^2
Live load = 2.0 kN/m^2
Design a continuous composite slab with profiled steel sheeting.

1. **Profiled steel sheeting as shuttering**

The profiled steel sheeting must resist the weight of wet concrete and the construction loads.
Assuming the following trapezoidal type of profiled sheet;

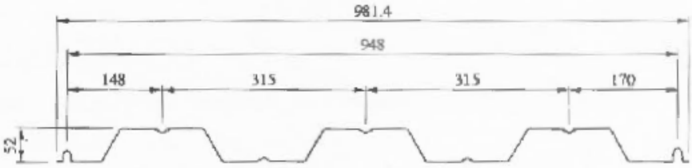
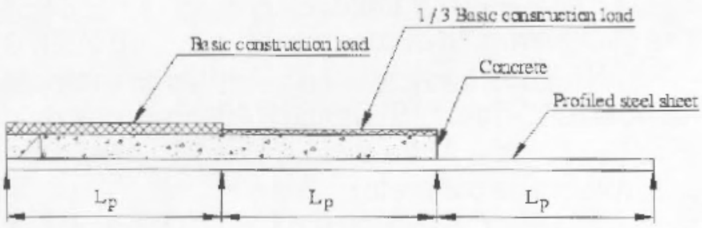


Figure 5 - Section of Profiled Steel Sheet

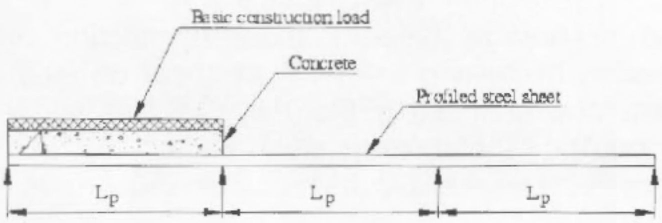
(Effective Span of profiled sheet/overall depth of slab = 35)
Assuming total thickness of slab as 120 mm
Loadings
Dead loads

- Self weight of sheet = 0.2 kN/m (assumed)
- Weight of concrete = 25 x (0.948 x 0.12-0.5x(124+191)/1000x52/1000x3) =2.23 kN/m

Live load
(BS 5950:4-1994-Clause 2.2.3.1)/ (EC4-Clause 9.3.2.1)
A construction load of 1.5 kN/m^2 is considered on the sheet. No allowance is made for



a) Arrangement for maximum negative moment



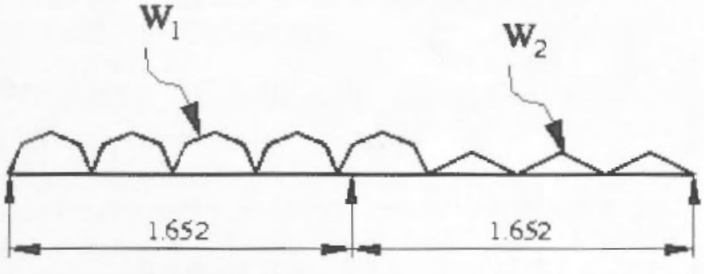
a) Arrangement for maximum positive moment

Figure 6-Arrangement of Construction loads
Arrangement of loads(BS5950:4-1994-Clause 6.6.3-Fig 1)

excessive impact or heaping of concrete or pipeline or pumping loads.
Design load = 1.5x (0.2+2.23+1.5) = 5.89 kN/m

Effective span
The top flanges of the supporting steel beams are assumed to be at least 0.15 m wide.
L_e = (1750-150) +52 =1652 mm or 1750 mm
Thus, L_e = 1652 mm
Span/depth=1652/52=31.76<35
(BS 5950:4-1994-Clause 6.6.3)
Depth of slab=1750/35 = 50 mm

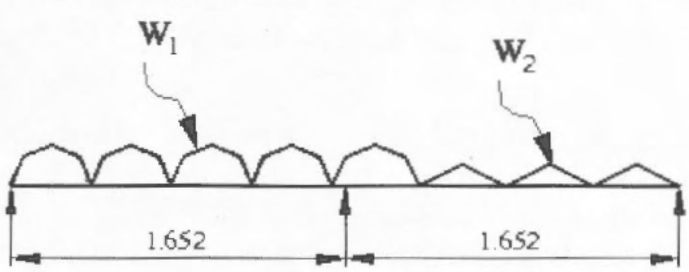
Case (a) Arrangement for maximum negative moment



w_1= weight of profiled sheet+ concrete+ basic construction load
w_2= weight of profiled sheet+ concrete+ 1/3 basic construction load
The maximum design bending moments are:

- Sagging M_{sd} =1.231 kNm/m
- Hogging M_{sd} = 1.755 kNm/m

Case (b) Arrangement for maximum positive moment



w_1= weight of profiled sheet+ concrete+ basic construction load
w_2= weight of profiled sheet+ concrete+ 1/3 basic construction load
The maximum design bending moments are:

- Sagging M_{sd} = 1.52 kNm/m
- Hogging M_{sd} = 1.038 kNm/m

Assuming thickness of sheet = 0.8 mm.
Self weight of sheet = 0.08 kN/m^2
Sectional properties of profiled steel sheet;
Effective area of cross-section, A_p= 978 mm^2/m
Second moment of area, I_p = 0.48 x 10^6 mm^4/m
Distance of centroid above base,
e = 28.09 mm
Section modulus,Z_b = 17.09 x 10^3 mm^3/m
Section modulus,Z_t= 20.07 x 10^3 mm^3/m
The design resistance to moment is;
M_R= 0.66 x f_y x Z_t
3.31 kNm/m>1.755 kNm/m
(Maximum of 1.231, 1.755, 1.52, 1.038 kNm/m)
Thus safe.
Deflection
The design serviceability load is (0.08+2.23+1.5)

= 3.81 kN/m^2. The maximum deflection in span AB, if BC is unloaded, is

$w L_e^4/185\,EI$= 1.597 mm

(Textbook-Composite structures of steel and concrete-R.P.Johnson)

Permissible deflection (effects of ponding not taken into account)

(BS5950:4-1994-Clause 5.3)

$L_e/130$ = 12.71 mm > 1.597 mm, thus safe.

Permissible deflection (ECG-Clause 0.6[2])

=Le/180=9.7 mm 1.597 mm, thus safe.

2 Composite slab- flexure and vertical shear

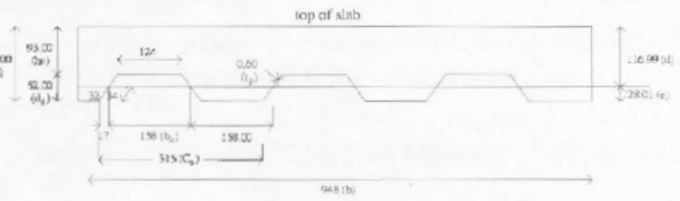


Figure 7 - Cross Section of Composite Slab

Effective span

L_e = clear span + depth of sheet or centre to centre of span; whichever is minimum

= (1750-150) + (120-28.09) = 1691.9 mm or 1750 mm

Thus,

L_e = 1691.9 mm

For vertical shear, the span is taken as 1.75m, so that the design loading for the beams includes the whole area of slab.

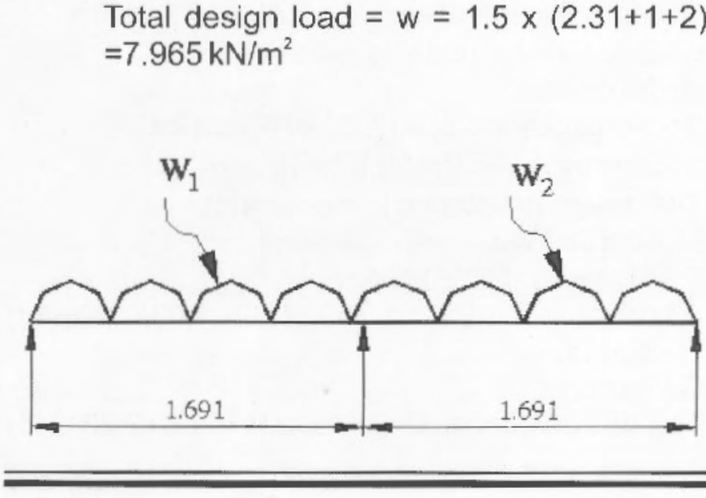
Loadings

Dead loads

- Self weight of composite slab = 2.23+0.08=2.31 kN/m
- Floor finish = 1 kN/m^2

Live load = 2 kN/m^2

Total design load = w = 1.5 x (2.31+1+2) =7.965 kN/m^2



$w_1=w_2$=7.965 kN/m^2

The maximum design bending moments are:

- Sagging M_{sd} = 1.599 kNm/m
- Hogging M_{sd} = 2.842 kNm/m

Resistance to bending moment is given by,

$M_{pRd} = 0.87 \times f_{yp} X A_p \times (d - 0.42 \times x_u)$

(IS 456:2000-Clause 38.1)

Depth of neutral axis is given by,

$x_u = 0.87 \times f_{yp} \times A_p/(0.36 \times f_{ck} \times b)$=24.17 mm

M_{pRd} =17.32 kNm/m >2.842 kNm/m, thus safe

Maximum vertical shear is

V_{sd} = 7.965 x 1.69 – 5.048 = 8.41 kN/m

Resistance to vertical shear is given by,

$V_{v.Rd} = b_o \times d \times \tau_c$

(BS5950:4-1994-Clause 6.5.1)

b_o = 158 mm, d = 91.91 mm.

The shear strength of concrete is

= 0.81 N/mm^2;

(IS 456:2000-Table 19 -Clauses 40)

(For 100 A_s/b d = 100 x 948/ (948 x 91.91) =1.088 and M25 grade concrete)

$V_{v.Rd}$ = 158 x 91.91 x 0.81 = 70.61 x 10^3 x 948/315 = 212.5 kN/m

= 212.5 kN/m > 8.41 kN/m, thus safe.

3. Composite slab – longitudinal shear

(BS5950:4-1994-Clause 6.4.3)

'End anchorage' type of shear connection is provided by resting the ends of sheet on steel beam and connecting them by welding studs through the sheeting to the steel flange.

(Constructional details as per BS5950:4-1994-Clause 4.8.1)

Shear capacity per unit width is given by;

$V_a = N \times P_a\,(d - x_c/2)/L_v$

N= number of shear connectors provided at the end of sheet per unit length of beam. 1 shear connector is assumed in each rib of profiled sheet thus 1000/315= 3 no's per unit length of beam.

P_a= end anchorage capacity per shear connector = 0.4 Q_k = 0.4 x 93.5 = 37.4 kN

Effective depth of slab; d = 120 -28.09 = 91.91 mm

x_c= depth of concrete in compression at mid-span= 20 mm

L_v=shear span = L/4 = 1750/4 =

V_a = 3 x 37.4 x 1000 x (91.91-20/2)/437.5

=21 KN/m >8.41 kN/m, thus safe

4. Composite slab-serviceability

Cracking of concrete above supporting beams

Continuity across the steel beams should be provided by reinforcement of area 0.4 % of the gross sectional area of concrete for propped construction. Hence,

$A_s = 0.004 \times 948 \times (120\text{-}52) = 258\ mm^2/m$

((EC4-Clause 9.8.1(2))

The detailing is best left until longitudinal shear in the beam and fire resistance have been considered.

Deflection

Maximum deflection is given by;

(BS5950:4-1994-Clause 6.6.2) & ((EC4-Clause 9.8.2(4))

Moment of inertia of unreinforced composite slab is given by;

$I = (I_{ucr} + I_{cr})/2$ in steel units

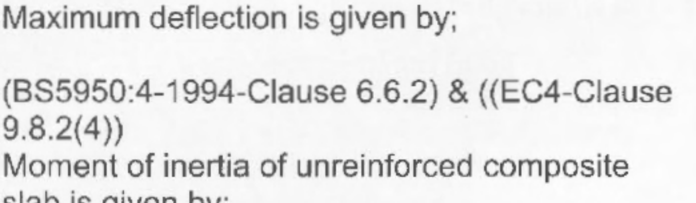


Figure 8 - Section of Composite Slab for Moment of inertia calculation

I_{ucr}=moment of inertia of uncracked section

$[bh_c^3 / 12 + bh_c(y_{uc}\text{-}0.5h_c)^2 + W_r d_d^3/12b/C_s + W_r d_d(h\text{-}y_{uc}\text{-}0.5d_d)^2 b/C_s + nI_s + nA_s(d\text{-}y_{uc})^2]/n$

y_{uc} = neutral axis of the uncracked section

$= 0.5bh_c^2 + nA_s d + W_r d_d(h\text{-}0.5d_d)b/C_s / [bh_c + nA_s + W_r d_d b/C_s]$

$= 54.23$ mm

$I_{uc} = 12.48 \times 10^6\ mm^4/m$

y_{cc}= neutral axis of the cracked section

$= d\ \{\sqrt{2A_s bd/n + (A_s n/bd)^2} - A_s/bdn\}$

$= 32.66$ mm

$I_{cr} = 5.031 \times 10^6\ mm^4/m$

$I = (12.48+5.031) \times 10^6$

$= 8.755 \times 10^6\ mm^4/m$

$\delta_{max} = 0.322$ mm

Permissible deflection = $L_e/250 = 1691/250 =$

6.766 mm > 0.322 mm, thus safe

(BS5950:4-1994-Clause 6.6.1) & ((EC4-Clause 9.8.2(3))

5. Transverse reinforcement

(BS5950:4-1994-Clause 6.9)

Minimum area of reinforcement = 0.1% of cross sectional area of concrete above ribs

$= 0.1/100 \times 948 \times (120\text{-}68) = 64.46\ mm^2/m$

Provide 8 mm bars @ 300 c/c (167.5 mm^2/m).

6. Composite slab-fire design

The slab is designed for a fire duration of 60 minutes.

C r i t e r i o n " I "

(EC4-part 1.2-Clause 4.3.1.2)

The effective thickness of slab is given by,

$h_e = h_1 + 0.5h_2(l_1 + l_2/l_1 + l_3)$

for $h_2/h_1 \geq 1.5$ and $h_1 > 40$ mm

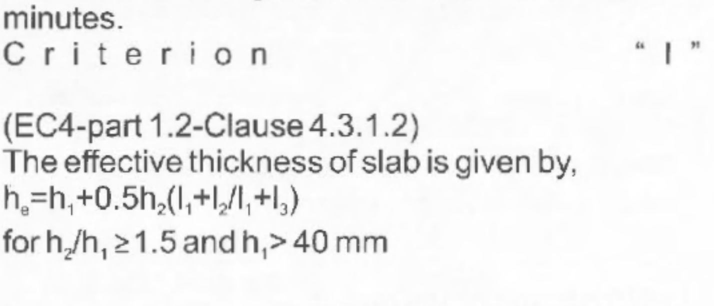


Figure 9 - Section of Composite Slab for Fire design

The dimensions are:

h_1=thickness of concrete above ribs =68mm,

h_2=depth of sheet=52 mm,

h_3=depth of screed layer above slab assumed =20 mm,

l_1=158 mm, l_2=124 mm, l_3=124 mm

$h_e = 68 + 0.5 \times 52\ ((158+124/158+24))$

=94 mm

The minimum thickness required is

(EC4-part 1.2-Table 8)

$h_e \geq (90\text{-}h_3) = (90\text{-}20) = 70$mm

So criterion I60 is easily satisfied.

C r i t e r i o n " R "

(EC4-part 1.2-Clause 4.3.1.3)

The moment to be resisted during fire is

$R_{fid} \geq \eta^* R_d$

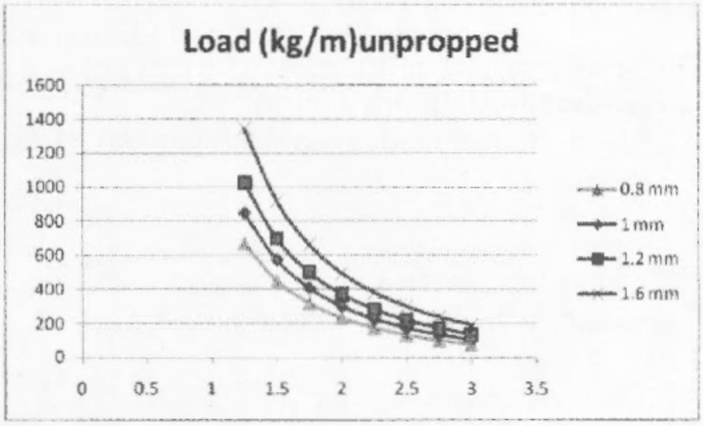
η^*= resistance ratio given by

$\eta^* = \eta_f E_d/R_d$

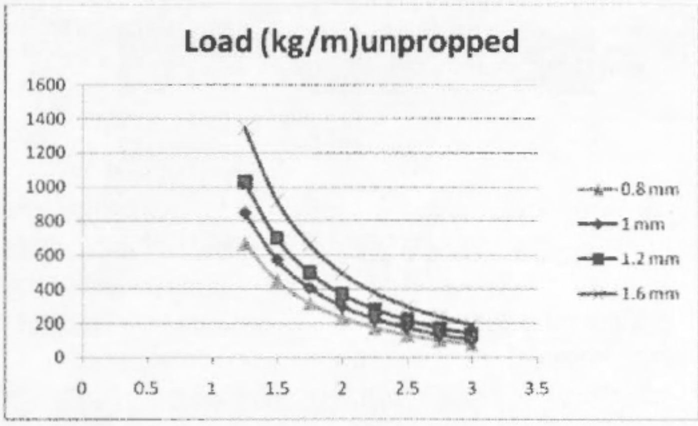
$\eta_f = 0.6$

E_d = maximum mid span bending moment = 2.842 kNm/m
R_d = moment of resistance = 17.32 kNm/m
$\eta^* = \eta_F E_d / R_d = 0.098$
$R_{fd} > 0.098 \text{ X } 17.32 = 1.705$ kNm/m < 17.32 kNm/m ,thus safe**Graph (Span Vs Capacity) with grades 250/350/550 MPA**

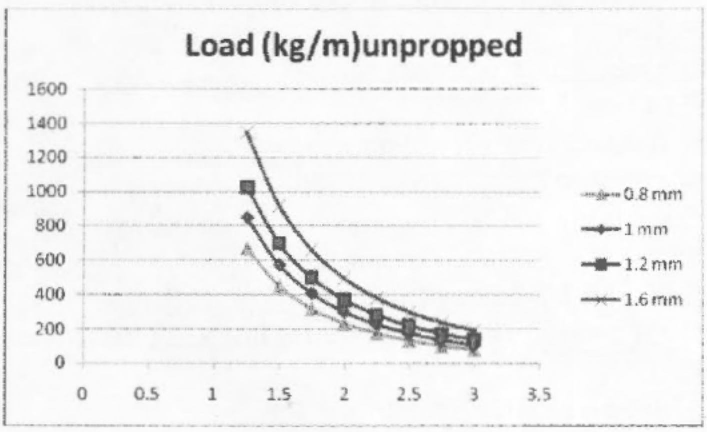
f_y=250 N/mm^2 ,h=120 mm



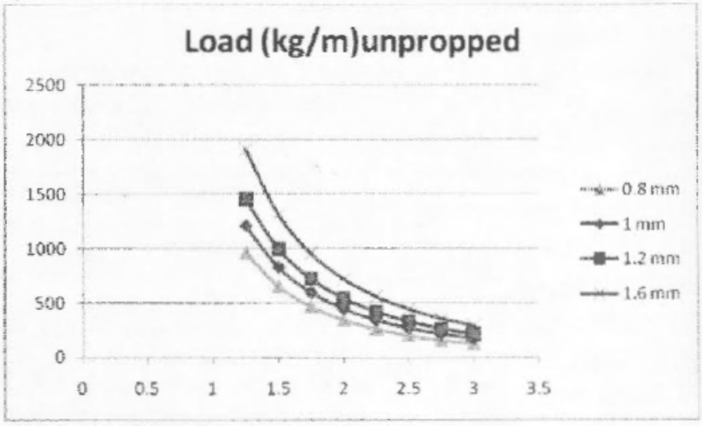
f_y=250 N/mm^2 ,h=145 mm



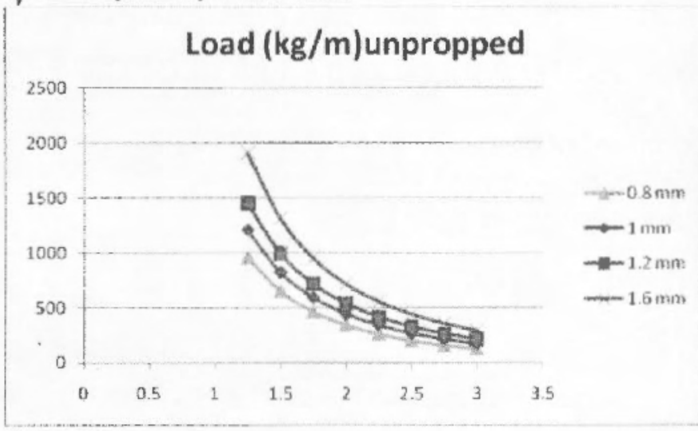
f_y=250 N/mm^2 ,h=240 mm



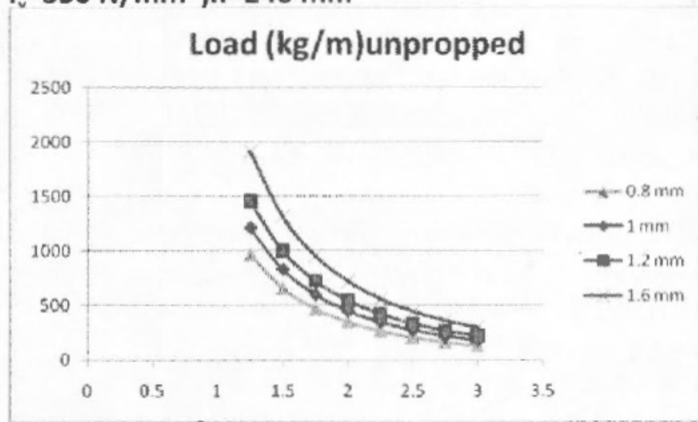
f_y=350 N/mm^2 ,h=120 mm



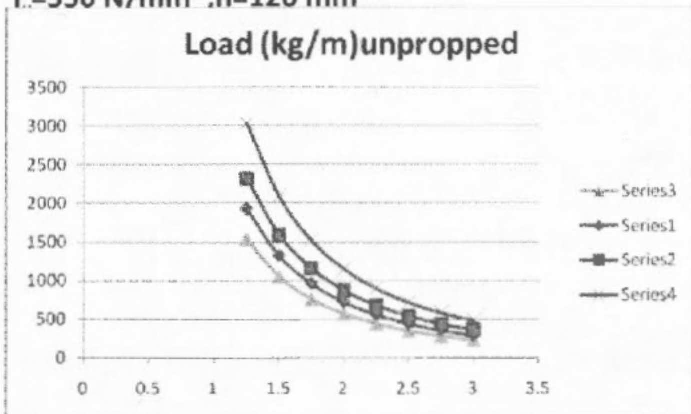
f_y=350 N/mm^2 ,h=145 mm



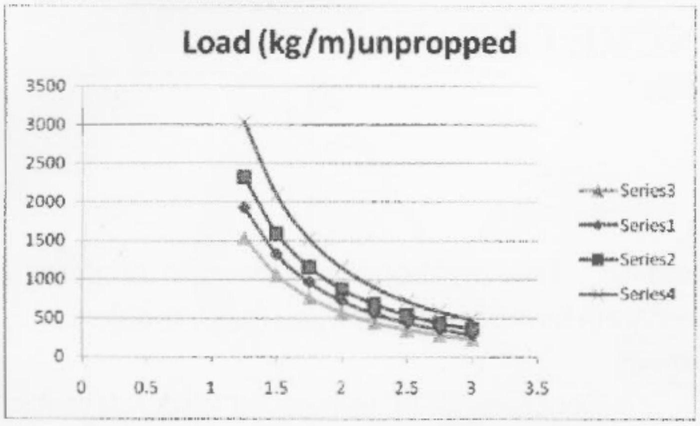
f_y=350 N/mm^2 ,h=240 mm



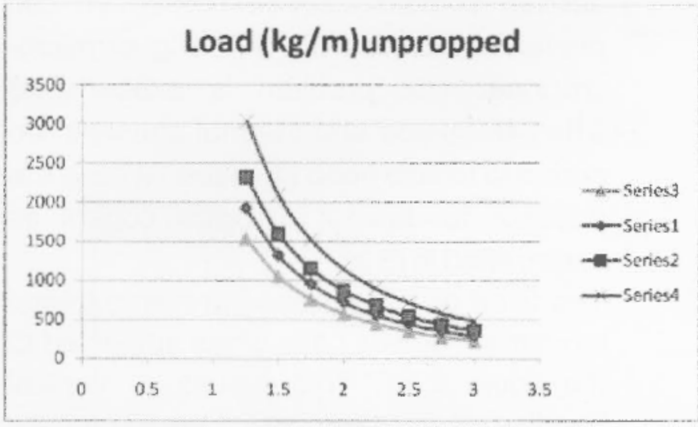
f_y=550 N/mm^2 ,h=120 mm



f_y=550 N/mm^2 ,h=145 mm



f_y=550 N/mm^2 ,h=240 mm



Interpretations
Span of slab = 2 m, Dead load on slab = 0.5 t/m^2
Live load on slab = 1 t/m^2 Total thickness of slab required = 240 mm
Total load on slab = 0.5+1=1.5 t/m2 = 1500 kg/m^2
Consider 1 m width of slab, total load on slab = 1500 kg/m
From Load v/s span graphs
For grade of sheet = 250 N/mm^2
Thickness of sheet required = 1.2 mm
For grade of sheet = 350 N/mm^2
Thickness of sheet required = 1 mm
For grade of sheet = 550 N/mm^2
Thickness of sheet required = 0.8 mm

Conclusions

With increase in grade of sheet;

1. Thickness of sheet reduces for the same capacity
2. Self weight of sheet decreases
3. Load capacity increases
4. Cost difference between grades of steel is only 5 to 7 %
5. Net savings in overall project by use of high grade steel is about 10 to 15%
6. Steel consumption decreases

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- **BS 5950-1994(Part 4):** Code of Practice for design of composite slabs with profiled steel sheeting
- **IS 11384-1985:** Code of Practice for Composite construction in Structural Steel and Concrete
- **IS 456-2000:** Plain and Reinforced Concrete Code of practice
- **IS 800-2007:** General Construction in Steel Code of practice
- **IS 1642:** Fire safety of buildings (General):Details of construction
- **Thakkar Group of Companies** Section Property manual

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DESIGN AND CONSTRUCTION OF WIREWOUND CIRCULAR PRECAST, PRESTRESSED CONCRETE TANKS

Sanjay Mehta

Abstract:

Circular precast, prestressed circular concrete tanks, although common in the North American Continent, are only now making their way in the Indian subcontinent. Circular concrete tanks, prestressed with continuous, spirally wrapped high strength steel wires offer superior performance, liquid tightness and service life as compared with the reinforced concrete tanks. Once hydro tested after initial construction, such tanks can be virtually maintenance free. Liquid tightness of such tanks is particularly appealing in terms of Indian context because of significant water loss in storage and distribution systems. Furthermore, replacing a system of Underground Sump Reservoir (USR) and Elevated Storage Reservior (ESR) with a single, tall prestressed concrete tank can result in significant savings in pumping electricity cos,t in addition to savings in land and excavation cost. Precasting operation has an added advantage of speeding up the construction. Tanks as large as 100m in diameter and 25m in height have been successfully designed and constructed across the United States. This article is written to explain salient features of design and construction of such tanks.

INTRODUCTION:

Circular reinforced concrete tank walls are designed and constructed to keep concrete hoop stress within modulus of rupture when subjected to liquid load. Amount and spacing of reinforcement is designed to control cracking within the tank wall as per the applicable provisions of IS 3370 and IS 456. Typically, base of the tank wall is either fixed or hinged to the footing. This traditional approach to reinforced concrete tank design has three major problems:

1. Concrete is in tension in circumferential direction when subjected to liquid load. No amount of mild steel reinforcement can prevent concrete from cracking or micro-cracking. The problem is exacerbated when shrinkage and thermal stresses are added to tensile hoop stresses. At best, it is possible to design for crack control as exemplified in IS 3370[1].
2. The fixed or hinged base prevents lateral movement of wall base when subjected to the liquid load. This results in vertical bending of the tank wall. Flexural tensile stress well in excess of modulus of rupture of concrete can develop due to this base restraint.
3. In case of a seismic event, the load path from tank wall, subjected to liquid convection and impulse, goes through the wall-footing joint. Although this joint can be designed and reinforced to provide the required ductility, it is not possible to control cracking in case of an extreme seismic event. The liquid tightness of the tank will be compromised during such a seismic event, precisely the time when water requirement is very critical to control post-earthquake fires.

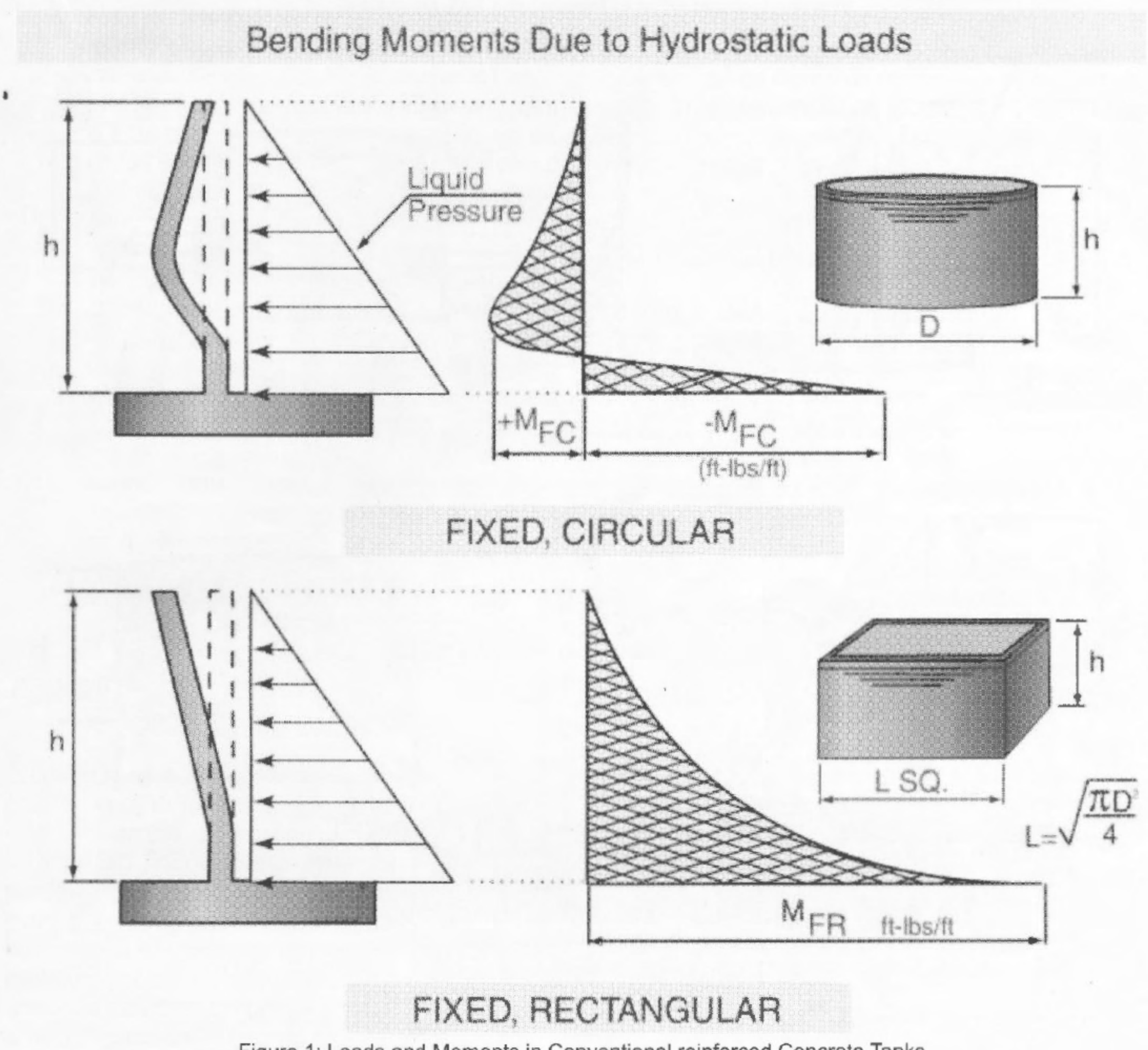


Figure 1: Loads and Moments in Conventional reinforced Concrete Tanks

Figure 1 illustrates various stages of loading and stresses in a circular reinforced concrete tank.

Figure 2 shows similar stages for a circular prestressed concrete tank. The problems mentioned above for a reinforced concrete tanks are alleviated in a spirally wound, prestressed concrete tank explained as follows:

1. The circular tank wall is prestressed in excess of the hoop tension caused by liquid load. Thus, even under full liquid pressure, concrete remains in compression.

2. Elastomeric bearing pads separate base of the tank wall from the ring footing. When subjected to prestress loading, the tank wall moves in with minimum restraint allowing compression to develop without significant vertical bending. Similarly, the tank wall moves out with minimum restraint when subjected to liquid pressure. However, total outward radial movement is

Bending Moments Due to Prestressing & Hydrostatic Load

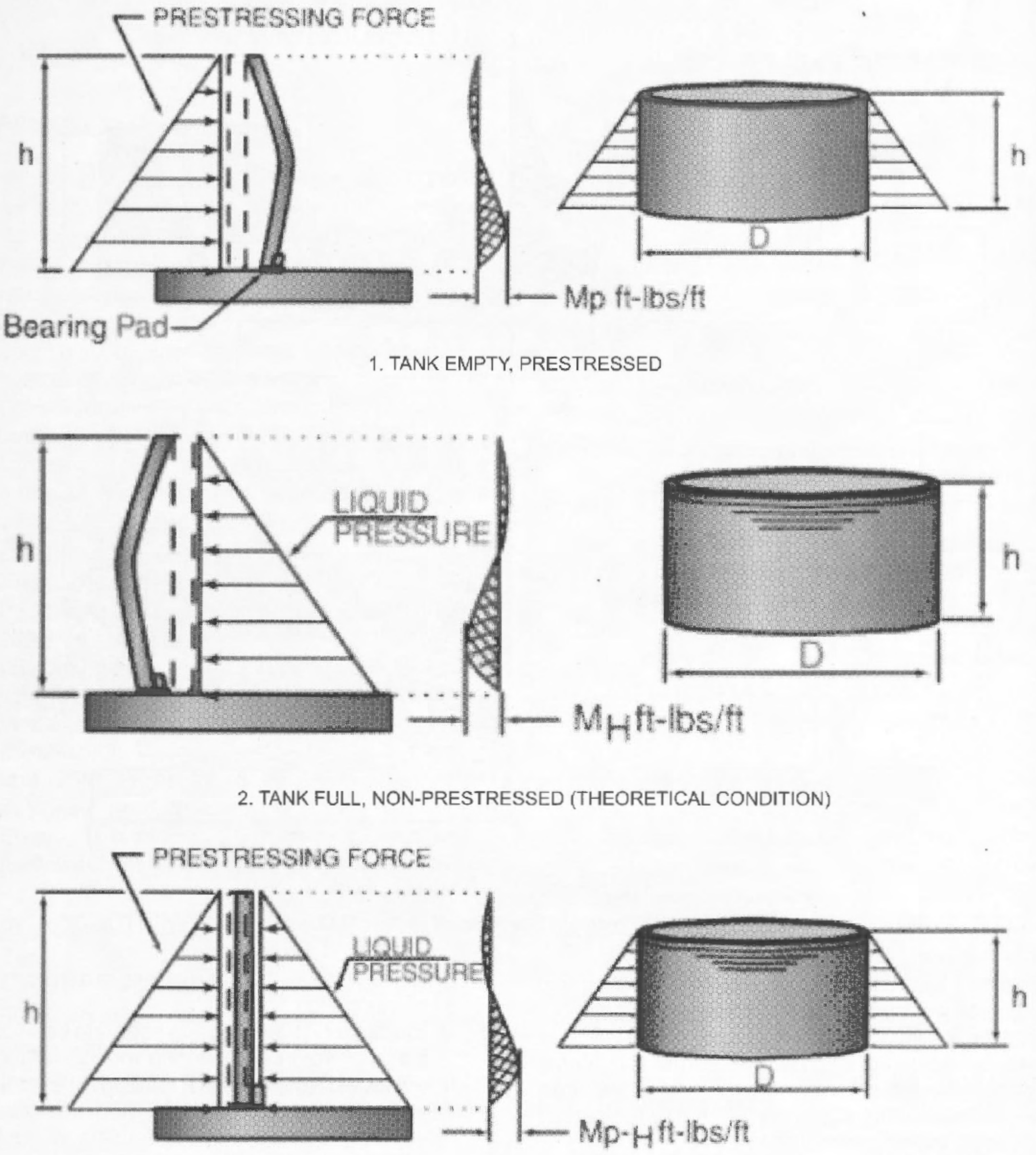


Figure 2: Loads and Moments in Circular Wirewound-Prestressed Concrete Tanks

always less than total inward radial movement, ensuring residual compression in the tank wall under full hydrostatic loads. The liquid tightness of the wall-footing joint is achieved by use of a specially designed PVC waterstop. This waterstop, about 275mm long, has ability to move radially by about 75mm without compromising liquid tightness of the tank.

3. Typically, seismic restraint cables are placed on the outside of the precast core walls. The cables are activated only when tank wall begins to slide either during earthquake or wind or any such lateral loads. Under normal operating conditions the restraint cables remain inactive and almost unstressed. Thus, lateral load resistance is separated from the vertical and hydrostatic load resistance, providing superior seismic performance.

BENEFITS OF PRECAST TECHNOLOGY:

It is well known that better concrete quality control can be exercised when concrete is poured under controlled condition. This is particularly true in case of tank walls because poor quality of concrete is the primary reason for leakage from liquid containing concrete structures. Furthermore, if it were possible to cast circular walls in sections, shrinkage stresses due to restraint could be drastically reduced. Small sections of walls are easier to pour and finish, especially when concrete pour is in horizontal direction. Since wall panels are cast in horizontal position, they have to be tilted up and placed in position on the circular ring footing. The tilt-up operation generates high flexural stresses in the wall panel. Thus, the strength and quality of concrete is tested even before the tank is placed in service, due this tilt-up operation. In contrast to cast-in-place walls, the precast wall casting operation can start simultaneously with the floor pour. This significantly reduces the construction time of the entire project since wall concreting is no longer along the critical path.

DISCUSSION OF CRITICAL DESIGN AND CONSTRUCTION CONCEPTS:

Figure 3 shows a typical section of wirewound precast, prestressed concrete tank. Accordingly, there are five critical elements in the design and construction (1) Floor and Footing (2) Wall Base Joint (3) Precast\Wall Panels and Joints (4) Wall Prestressing and Shotcreting and (5) Dome Construction and Prestressing. Each element is discussed as follows:

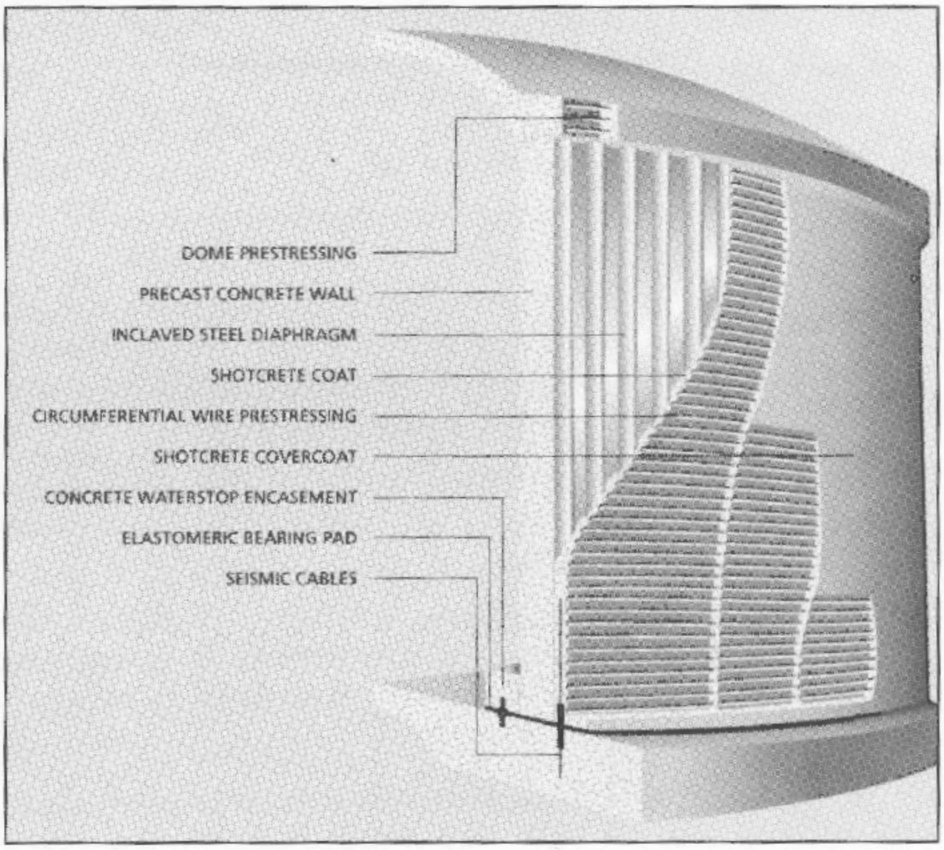


Figure 3: Typical Cross Section of Wirewound-Prestressed Concrete Tank

1) Floor and Footing

Typically, 0.3m deep footing is cast monolithically with 0.1m membrane slab. Figure 4 shows cross sectional details of footing-floor connection. As

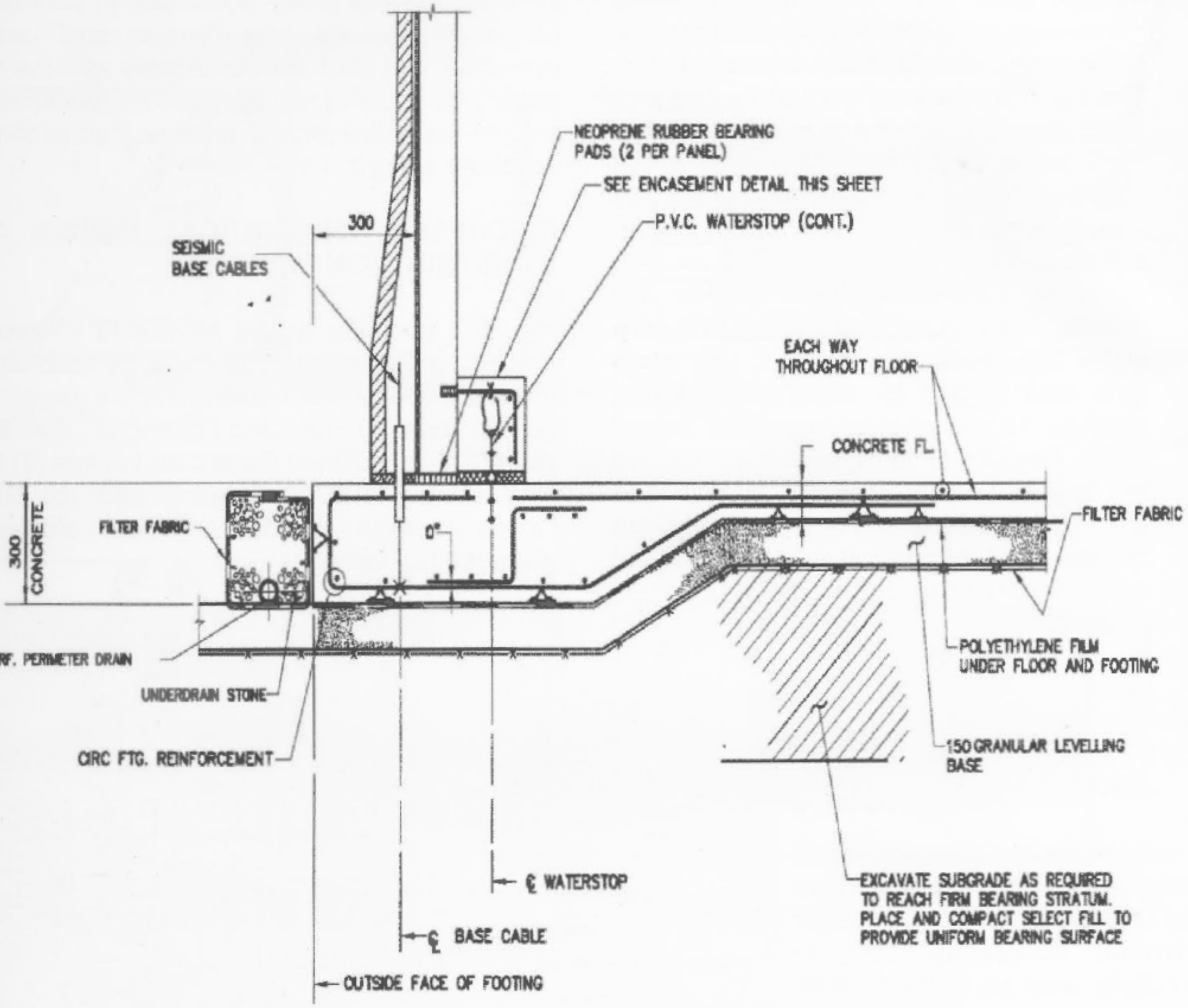


Figure 4: Wall-Floor-Footing Details

per ACI 350-06[2], Appendix G, floors upto 0.15m in thickness are classified as "membrane" floors. Membrane floors are designed with a single layer of reinforcement in each direction for creep and shrinkage of concrete. They have very little flexural capacity and liquid load is transferred "directly" to the supporting soil. That is, the slab is flexible enough to conform to the deformed shape of supporting soil. The key to the successful performance of a membrane floor lies in the uniformity and strength of subgrade. So long as supporting soil is compacted properly to avoid localized settlement, the membrane slab will deform in "dish-type" settlement shape. ACI 372R -03[3], Appendix A, Section A.3.2 describes deflection limits on various deformation modes of membrane slab-on-grade, along with design considerations.

The ring footing is connected to the membrane floor using haunch and reinforcement as shown in Figure 4. The membrane floor acts as a "tie" which prevents footing rotation due to eccentricity of the vertical load. The design approach requires checking tie stress at floor-footing haunch to ensure that the tensile tie stress is well within the modulus of rupture of concrete to ensure liquid tightness. The width of the footing is selected such that stresses from vertical loads are within the allowable soil bearing capacity.

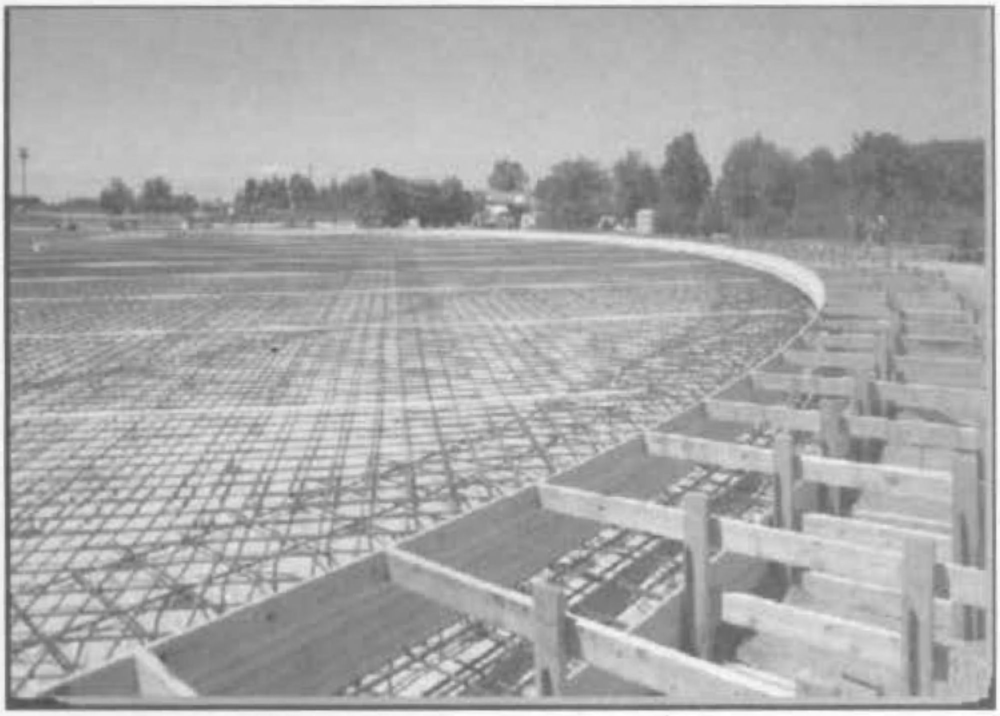


Figure 5: Floor-Footing Reinforcement

As far as possible, it is preferable to cast floor and footing in one pour to avoid construction joint. Construction joints in liquid containment structure are a maintenance problem. Special care is required to vibrate concrete around horizontal water-stop at the construction joint to prevent "honeycombing". Membrane floors as large as 60m have been cast in one pour. Figures 5 through 8 show various stages of membrane floor and ring footing construction.

2) Wall Base Joint

As mentioned earlier, unlike reinforced concrete tanks, circular prestressed concrete tanks are designed to move in and out in the radial direction. This is achieved by setting elastomeric bearing pads between top of the footing and wall base. Bearing pads allow radial tank movement with minimum resistance. The design approach requires calculation of bearing pad resistance and vertical moment in the wall due to this resistance. The hoop forces in the tank wall are computed assuming no reduction in stresses due to pad resistance. As can be seen in Figures 3 and 4, liquid tightness at the wall base joint is achieved by casting a special waterstop into the footing on the inside face of the wall. After wall panels are erected, the waterstop is encased in a concrete curb, as shown in Figures 9 and 10. The design approach requires computation of wall base joint displacement when subjected to combined effect of circumferential prestressing and liquid load. The wall base movement is restricted to about 40mm to ensure sufficient factor of safety on the displacement capacity of waterstop. In some situations, partial prestressing may be required. That is, curb may have to be cast after

Figure 6: Floor Concrete Pour

Figure 7: Screeding and Finishing Operation

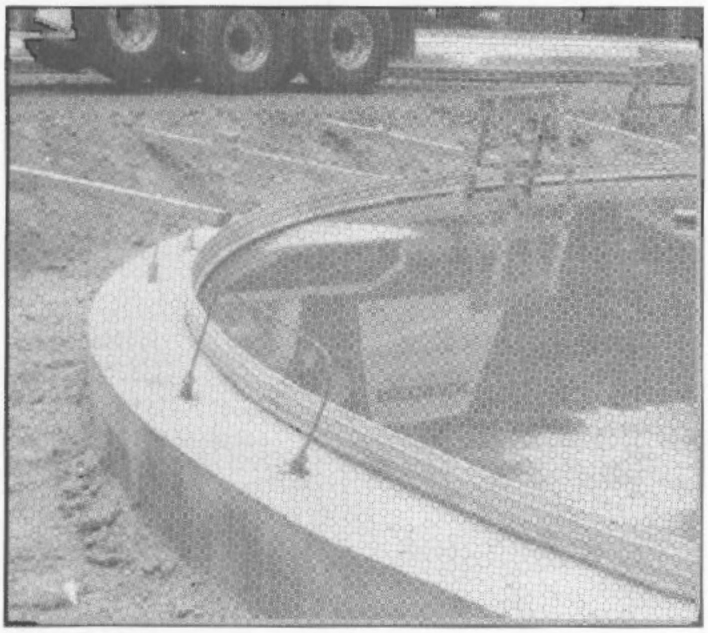


Figure 8: Flooding and Curing of Concrete Floor (Waterstop and Seismic Cables Embeded in Footing)

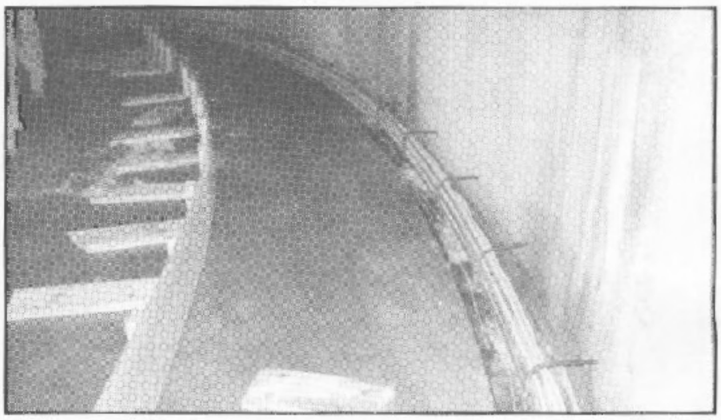


Figure 9: Waterstop Embeded in Footing

Figure 10: Concrete Encasement for Waterstop

prestressing the tank to some extent. The remaining prestressing will be applied after casting curb.

3) **Precast Wall Panels and Joints**

Typically, the wall panels are cast in 3m sections. The panel curvature is developed by computing ordinates for 3m section based on the tank radius. A sand bed is prepared on the ground with proper curvature. A thin steel sheet metal diaphragm (similar to corrugated steel sheet) is placed on the sand bed. It is this steel diaphragm that separates the concrete wall panels from shotcrete and

Figure 11: Diaphragm for Wall Panels

prestressing wires. The barrier provided by the steel diaphragm prevents the water from reaching the prestressing wires. Liquid tightness provided by the steel diaphragm, when combined with alkaline rich shotcrete encapsulating the prestressing wires, prevents corrosion of wires. Hence, it is possible to use plain prestressing wires without any need for galvanizing. Thousands of prestress concrete tanks using this approach have been designed and constructed in the United States using non-galvanized prestressing wires.

Figure 11 shows field preparation of steel diaphragm for casting 3m wide panels. Figure 12 shows concrete operation for panel casting. A wood screed cut to the curvature of the tank radius is used to "strike off" concrete.

AWWA Standard D110-04[4] restricts allowable stress in the steel diaphragm to 125MPa (18ksi) when counted as reinforcement in the vertical direction. The bond stress between plain steel metal diaphragm and concrete in the vertical direction is one of the reasons for restricting the allowable stress to 125MPa. This bond is fully tested during the panel pick up operation because of high flexural stresses developed in the precast wall panel. Frequently, the panel tilt up operation governs the reinforcing steel requirements in the panel. Figure 13 shows panel tilt up operation.

Figure 14 shows adjacent wall panels set on the top of the bearing pads over footing. As can be seen, there is vertical joint between wall panels, abut 178mm to 203mm wide. Horizontal reinforcement placed at 1200mm on center projects out from the panel ends. This reinforcement is welded to 18mm X 18mm X 6mm

Figure 12: Panel Casting and Finishing

Figure 13: Tilt Up Operation for Precast wall Panel

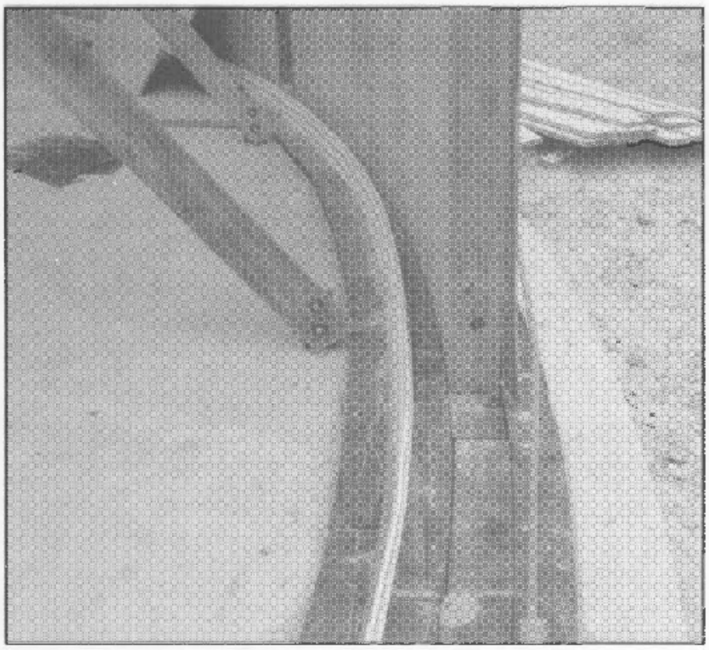


Figure 14a: Wall Panel Set on Bearing Pads

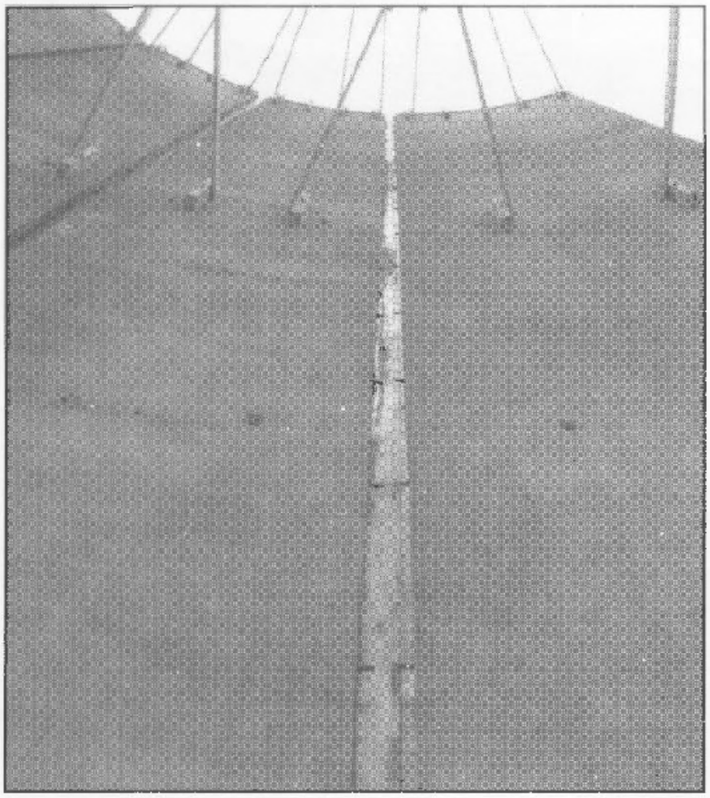


Figure 14b: Vertical Joints between Wall Panels

Figure 15: Shotcreting of Wall Panel Joints

thick steel angles, 150mm long. Welding of reinforcement to steel angles provides stability to wall panels during construction and prestressing operation. This joint is then filled with shotcrete applied against the projecting ends of the diaphragm sheets. Figure 15 shows joint shotcreteing operation.

The design approach requires that the wall panel joints remain in compression under full hydrostatic and hydrodynamic fluid pressure. Minimum prestressing is provided even above the waterline to ensure residual compression in the wall panel joints.

4) Wall Presrtessing and Shotcreting

After shooting all vertical wall panel joints, 12mm thick coat of shotcrete is applied over the steel diaphragm. Figure 16 shows steel diaphragm on the outside of the tank wall. Figure 17 shows similar view after shooting shotcrete over diaphragm. The prestressing operation starts after shotcrete coat has gained the required strength. Typically, 5mm diameter high strength steel wire (1517Mpa) is pulled through the die using a prestressing machine. Pulling high strength steel wire through die reduces the wire diameter before it is placed on the wall. The reduction in the wire diameter corresponds to prestressing force applied on the wall. This method of "wirewinding" or prestressing ensures continuous and uniform prestress along the tank circumference at a particular elevation from the tank floor. This is the most distinguishing feature of

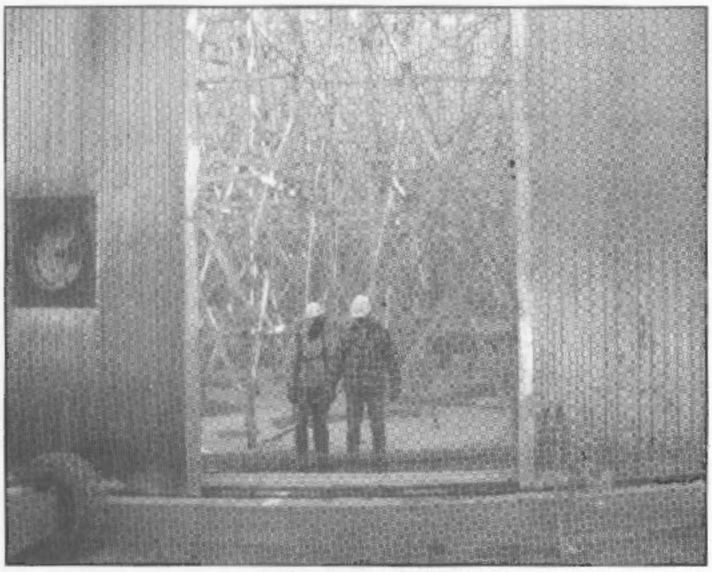


Figure 16: View Showing Steel Diaphragm on Outside of Wall Panels

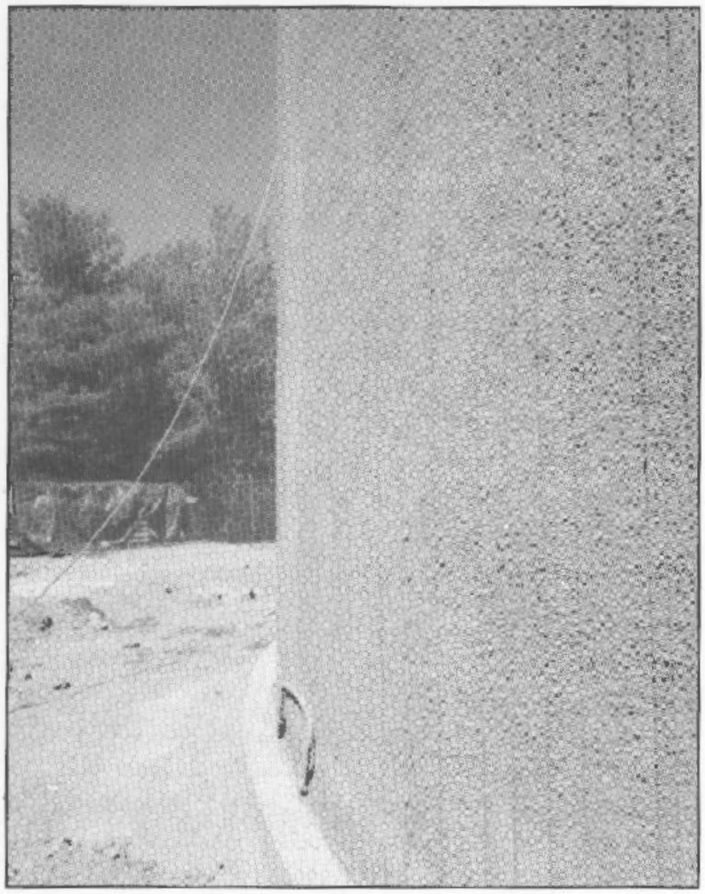


Figure 17: Tank Exterior After Shotcrete Coat Over Diaphragm

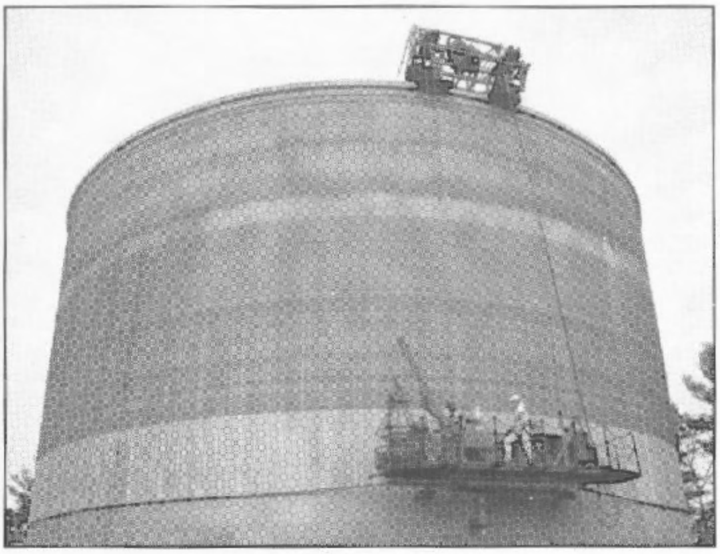


Figure 18: Wirewinding Machine for Circular Prestressing

wirewound tanks as compared to the tendon prestressed tanks, in which case (1) The circumferential prestressing is discontinuous and (2) Prestres, at a given height from the tank floor, is not uniform along the circumference due to curvature and friction losses. Figures 19 and 20 show wirewinidng operation.

Design and detailing requirements dictate that wires be spaced at least 1 wire diameter apart along the height of the wall so as to completely

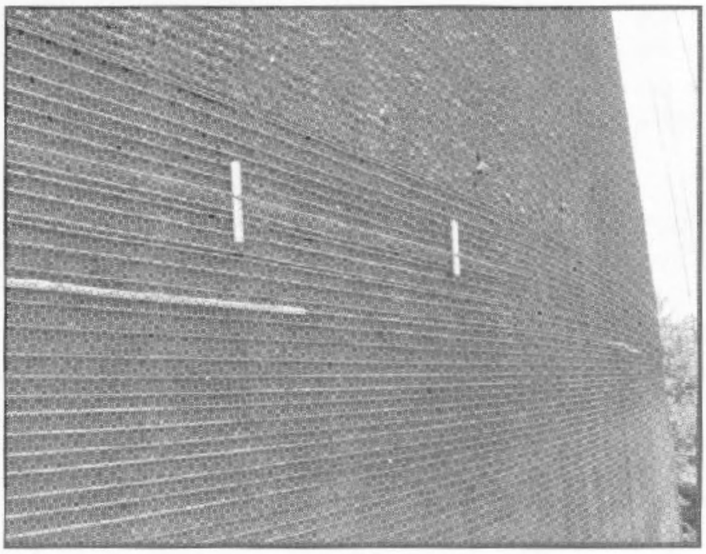


Figure 19a: Prestressed Wires on Wall

encapsulate them in shotcrete wire coat. This requirement restricts number of wires to approximately 22 per 300mm height of wall. Thus, many "layers" of wires would be required at the base of the wall to counteract higher liquid pressure as compared to only one "layer" of wires at the top of the wall. In terms of construction, the first layer of wires is applied along the full length of the wall over diaphragm shotcrete coat. This layer of wire is covered with shotcrete to provide minimum 6mm cover coat over wires. The next layer of wire is placed on the wall after shooting shotcrete over the first layer of wires. As many as 15 layers of wires may be required for bottom 2m of wall in case of tall, large diameter tanks.

At least 25mm of shotcrete cover coat is applied over the final layer of prestressing wire to provide adequate cover protection. The normal practice is to coat the wall shotcrete with "breathable" paint after applying final shotcrete cover coat.

5) Dome Construction and Prestressing

The most popular method of roof over potable water tanks is spherical dome. Clear spanning domes are aesthetically pleasing and provide tank interior that is free of any obstruction for cleaning and maintenance purposes. This is particularly important in case of digesters because rotating equipment inside the tank will function only when there are no obstructions. Furthermore, as compared with flat slabs, domes require less concrete and steel and hence, are economically attractive.

Typically, concrete domes are cast in-situ due to double curvature. However, many precast domes have also been built mainly for small diameter tanks. Seismic performance of cast-in-place domes is far superior to precast domes and many high seismic regions in United States precast domes are not preferred.

Construction of cast-in-place domes requires false work erection and formwork as shown in Figures 20 and 21. The dome concrete is poured in "pie" sections. Typically, 6 to 8 "pie" sections are required to complete the dome pour.

The thickness of domes is in the range of 75mm to 125mm and hence domes are classified as "thin shell structures". As is well known, the behavior of thin shell structures is governed by buckling. AWWA D110-04 provides an equation to compute dome thickness based on the buckling resistance when subjected to vertical dead and live loads. This equation accounts for the fact that there

Figure 20: Falsework for Dome Construction

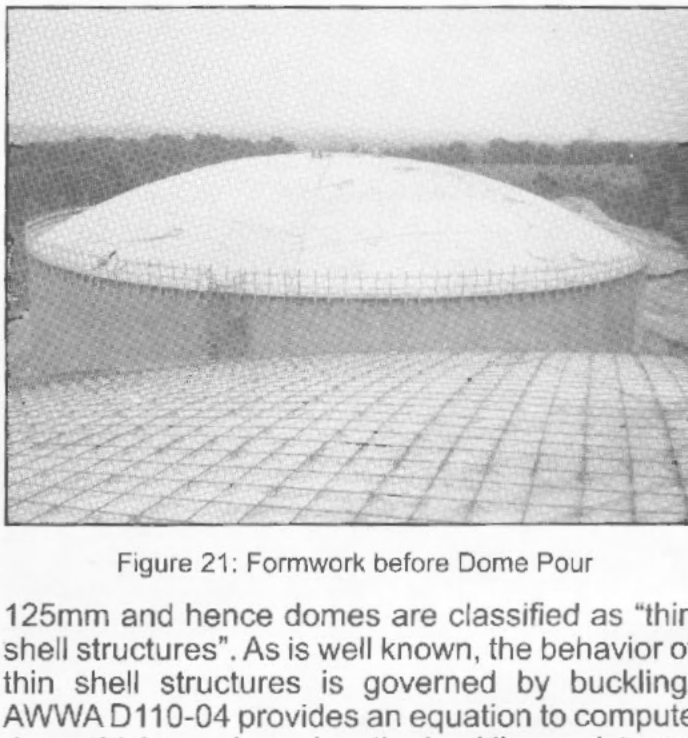


Figure 21: Formwork before Dome Pour

could be flat spots on the dome surface because of falsework fabrication and erection tolerance.

The design approach requires computing flexural stresses in the end regions of the spherical dome due to the "edge effect". The dome tension ring is prestressed to counteract the dead load and live load thrust. Typical dome design requires "hinged" connection between dome edge and top of the wall. Detailed discussion of all design aspects is beyond the scope of this introductory paper. However Billington[5], Timoshenko[6] and T.Y. Lin[7] have covered this topic in great detail.

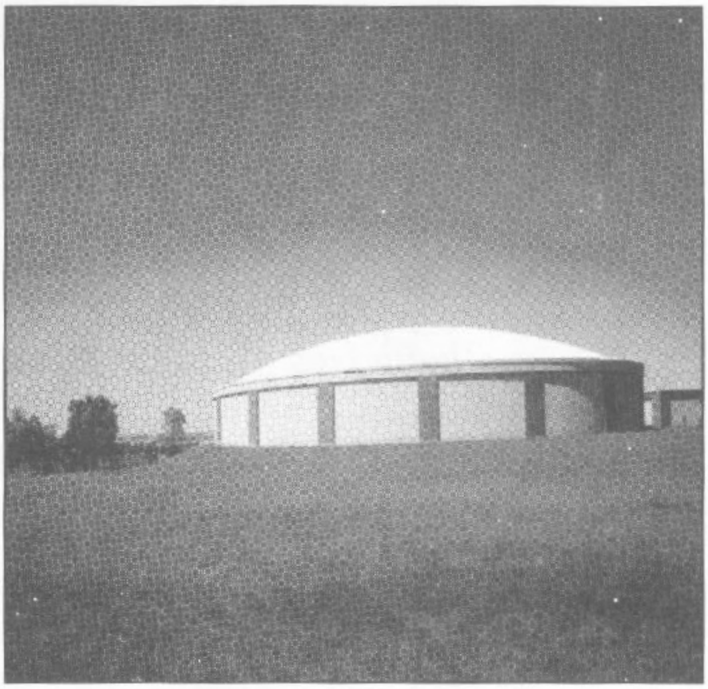
Dome prestressing operation begins after dome concrete has gained the required strength. Typically, dome is prestressed for both dead and live loads. Most of the times, dome "lifts" off the formwork support after application of full prestress because full live load is never applied on the dome surface. This "lifting" up of dome facilitates falsework and formwork removal operation.

HYDROTSTING OF TANK:

Most of the design and construction Specifications require that prestressed concrete tank be hydro tested to demonstrate the liquid tightness. Depending on the intended use of the tank, the extent of "liquid tightness" varies from bottle tight to measurable loss within the specified limit. As per AWWA D110-04, an ANSI standard for potable water tank, the tank is filled with potable water to the maximum level for 24 hours before starting the hydro test. The liquid drop is measured over the next 72 hours to determine the liquid volume loss for comparison with the allowable leakage. The tank is accepted if the net liquid loss for a period of 24 hours does not exceed ½ of 1% of the tank capacity. Furthermore, wet spots on the exterior of tank walls and flowing water at the base of the wall footing joint are not permitted. Necessary repairs are carried out if tank does not pass the hydro test at no expense to the owner.

PAINTING AND ARCHITECTURAL TREATMENT:

The exposed surface of dome and wall is painted to improve aesthetics and durability of the tank. In addition, various architectural treatment, as shown in Figures 22 through 23 can be applied on the tank exterior to improve aesthetics.



23: Architectural Treatment- Brick Pilasters

SUMMARY:

Circular wirewound, prestressed concrete tanks offer superior performance and durability as compared to reinforced concrete tanks because it eliminates all the problem areas associated with reinforced concrete tanks. In addition, it is possible to construct ground supported tanks as tall as 25m which can replace a system of USER and ESR. Replacing two tanks (USE and ESR) with one prestressed concrete tank provides significant savings in pumping electricity cost, increased storage capacity and savings in land as well as construction costs. The superior water tightness of prestressed concrete tanks, demonstrated by hydro testing after completion of construction, is particularly important in the Indian context because of significant water loss in storage and distribution system.

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Figure 21: Formwork before Dome Pour

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REPORT ON INAUGURATION OF ISSE AURANGABAD CENTRE 22 JUNE 2012

Hemant Vadalkar

Auranagabad centre of Indian Society of Structural Engineers was inaugurated on 22nd June 2012, at Aurangabad Gymkhana Club. The first body of elected office bearers was sworn in. Kamal Rao has been elected the Chairperson of the local chapter, Ravindra Bansode the Secretary, Bhushan Joshi theJoint Secretary and Amol Joshi the Treasurer.

The ceremony started with welcoming the guests and traditional lighting of lamps. The dignitaries were invited to the dais and the programme began with technical presentations of the main sponsor, Rajuri Steel. The presentation depicted the care and efforts taken by Rajuri Steel in manufacturing quality steel.

Ravindra Bansode, Secretary, Aurangabad centre, spoke about the inception of the chapter and a brief account of the activities carried out so far.

Hemant S. Vadalkar, Advisory Trustee of ISSE mentioned about the various publications that are regularly published. He also spoke on various aspects of the ISSE Web site.

P.B. Dandekar, Secretary, ISSE high lighted the activities carried out by the Society at the National level.

K.L.Savla, former Secretary, ISSE emphasiged the need and method of setting up new chapters at various centres of activities in the country.

Prof. G. B. Chaudhari, President ISSE enlightened all about the vision and need of having an institution like that of ISSE. He pointed at the technical marvels of Ajanta and Ellora, the grace and beauty of the caves and the highly specialised technical craftsmanship required in carving out the Kailash temple at Ellora, on the rock mass.

Kamal Rao as Chairperson accepted the challenges and responsibilities entrusted on the office bearers. She spoke about the challenges that have to be faced by her team boldly, bravely and squarely.

Senior Structural Consultants, N.R. Varma and Ravindra Babulay and Senior professors, Prof. Achwal, Prof. J.D. Koparkar and Prof. J.D. Koparkar were felicitated on this occasion.
Structural consultant N.R. Varma and N.G. Karkhane addressed the junior engineers and encouraged them to work to the best of their capacities for the profession.

The key note speaker at the ceremony was Dr. Sidharth Ghosh from IIT Powai. He spoke on the advantages of steel as a construction material. He gave special emphasis to the use of light gauge steel frames and tubes in residential structures. He also mentioned about the need to revise academic curriculum to meet the demands of industry

VISUALS FROM INAUGURAL FUNCTION OF AURANGABAD CENTRE OF ISSE

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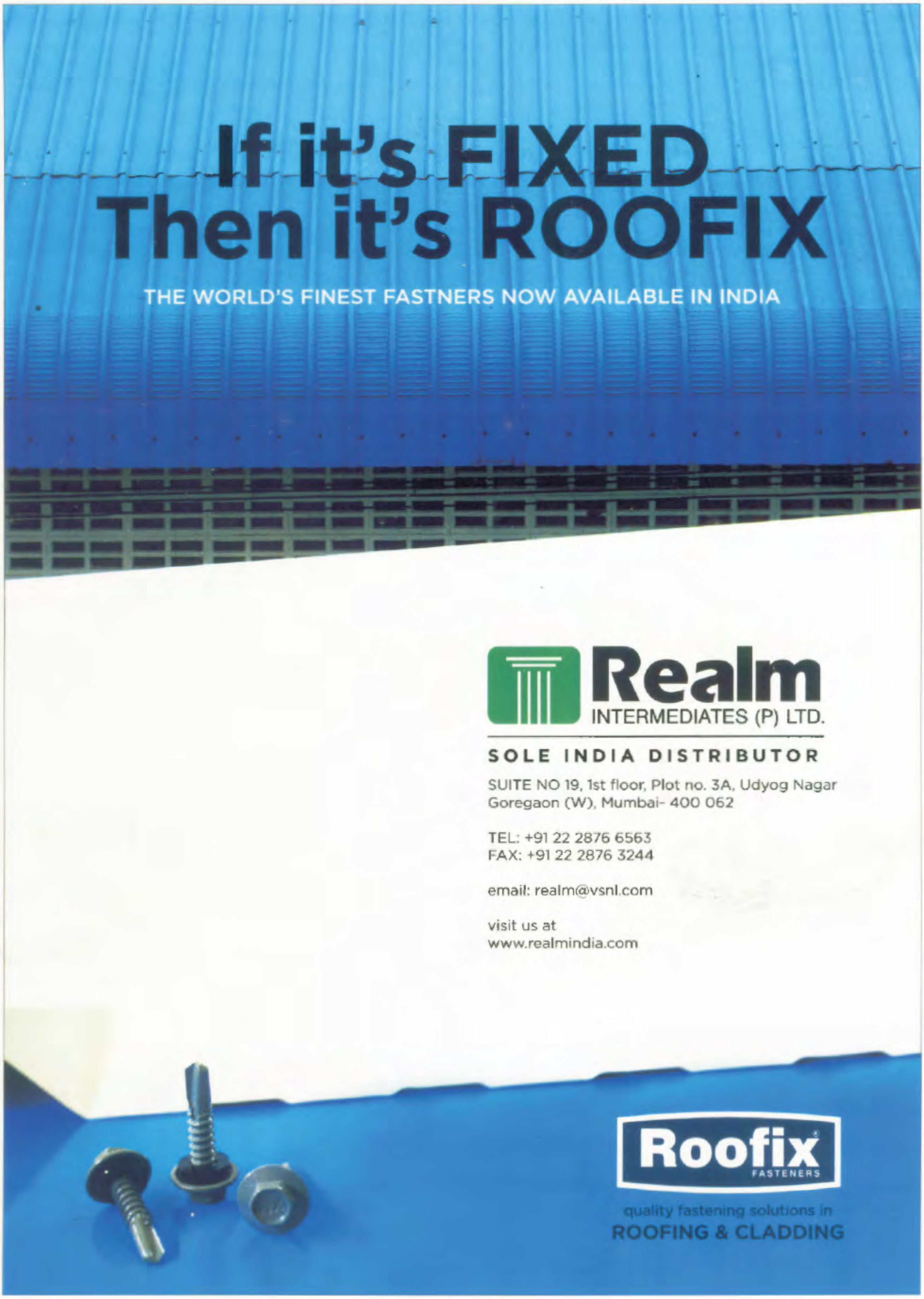
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