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Design of a Common Base Plate, Anchor Bolt Group and RCC Pedestal for a Four-Column CHS Hoarding Mast under Large Overturning Moment

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Abstract: ***Hoarding structures carry a large advertisement panel on a slender mast, so wind rather than gravity governs the foundation design. Where the mast uses several columns on a single common base plate, the axial force couple between the columns produces an overturning moment far larger than the sum of the individual column moments, and the connection cannot be designed as a set of independent column bases. This paper presents the design of the base plate, the 60-bolt anchor group, the RCC pedestal and the isolated footing for a 19 m wide, 18 m tall hoarding carried on a four-column circular hollow section (CHS) mast. The four leg reactions are first combined into a single resultant force and moment system at the plate centroid, giving P = 611.8 kN and M = 5772.6 kN·m, in which the force-couple term alone accounts for 4446.8 kN·m, about 4.4 times the summed local leg moments. The resultant is then redistributed across the full bolt group by the elastic method. A 1900 mm × 1900 mm × 50 mm plate with 60 nos. M36 Grade 8.8 bolts, a 2200 mm × 2200 mm × 1200 mm M30 pedestal and a 6.7 m × 6.7 m isolated footing at 2.0 m embedment are shown to be adequate, with maximum bolt tension utilisation of 47.4 percent, combined shear-tension utilisation of 0.225 and pedestal moment utilisation of 34.5 percent. The governing constraint is found to be geometric rather than strength-based: the anchor bolt edge distance provides only 2 mm of margin over the code minimum.***

Keywords: Common base plate, anchor bolt group, RCC pedestal, hoarding structure, overturning moment, IS 800

1. Introduction

Hoarding structures are large outdoor advertisement panels mounted on a slender supporting mast, widely used along highways and urban roads. Because the panel presents a large wind catching area on a comparatively light frame, wind and not gravity is the governing action, and the design of the foundation is dominated by overturning rather than by bearing [1], [2].

Where the mast is formed of several columns sharing one common base plate, the connection behaves differently from a conventional single-column base. Under lateral load the windward columns go into compression while the leeward columns go into net uplift, and this axial force couple, acting over the column spacing, generates an overturning moment at the plate that is much larger than the bending moment carried by any individual column. Designing each column base in isolation would therefore substantially underestimate the demand on the anchorage.

This paper sets out the complete design of the foundation connection for such a structure: the common base plate, the anchor bolt group, the RCC pedestal beneath it, and the isolated footing. The reactions used are taken from the fixed-base envelope of the STAAD.Pro analysis reported previously by the authors [3], where the soil-structure interaction behaviour of the same hoarding was examined; the present paper is concerned with the design of the foundation elements themselves.

2. Structural Configuration and Design Basis

The structure considered is a hoarding sited in Pune, Maharashtra. The panel is 19 m wide and 8 m tall, arranged in nine bays and four rows, and is carried on an 8 m tall mast of four CHS columns of 609.6 mm outside diameter and 20 mm wall thickness, spaced at 1.0 m centres in both directions. A 2 m transition zone widens from the mast to the central panel bay, giving an overall height of about 18 m. Fig. 1 shows the general arrangement together with the foundation elements.

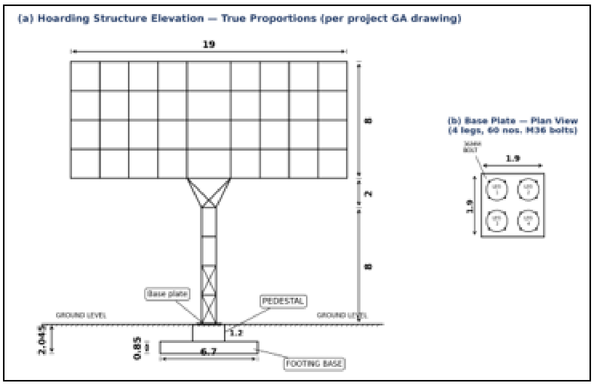

Figure 1: General arrangement of the hoarding structure showing panel, mast, base plate, pedestal and footing

The design follows IS 800:2007 for the steel base plate and anchor bolts, IS 456:2000 for the RCC pedestal and bearing on concrete, and IS 6403:1981 for the bearing capacity of the founding soil [4]–[6]. The governing load combination throughout is 1.5(DL + WL). Material properties adopted are given in Table 1.

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Table 1: Material properties adopted

Item	Grade	Property
Base plate	IS 2062 E250	fy = 250 MPa, fu = 410 MPa
CHS column	IS 1161, E250	fy = 250 MPa
Anchor bolts	Class 8.8	fyb = 640, fub = 800 MPa
Grout / bearing	M25	fck = 25 MPa
Pedestal	M30 / Fe500	fck = 30, fy = 500 MPa

Partial safety factors are taken as $\gamma m0 = 1.10$ for the plate section and $\gamma mb = 1.25$ for the bolts.

3. Combined Resultant Load at the Plate Centroid

The four leg reactions obtained from the analysis under the governing combination are listed in Table 2. Legs 1 and 2 are in compression and legs 3 and 4 in net uplift, the legs being located at ±500 mm from the plate centroid in both directions.

Table 2: Leg reactions under 1.5(DL + WL)

Leg	Fy (kN)	Fz (kN)	Mx (kN·m)
1	+2376.4	+108.6	+333.0
2	+2376.4	+108.6	+333.0
3	−2070.5	+99.3	+329.9
4	−2070.5	+99.3	+329.9

Because the plate is common to all four legs, the reactions are combined into a single resultant system at the plate centroid. The net vertical force is the algebraic sum of the leg axial forces:

$$P\ total = \Sigma\ Fy,i = 611.8\ kN \quad (net\ compression) \qquad (1)$$

The total overturning moment has two contributions: the sum of the legs' own local bending moments, and the moment generated by the axial force difference between the windward and leeward rows acting at the lever arm z i:

$$M\ total = \Sigma\ Mx,i + |\ \Sigma\ (Fy,i \times z\ i)\ | \qquad (2)$$

$$M\ total = 1325.8 + 4446.8 = 5772.6\ kN{\cdot}m \qquad (3)$$

The force-couple term of 4446.8 kN·m dominates: it is about 3.4 times the summed local leg moments and roughly 13 times the local moment of any single leg. This is the central point of the design. A designer treating each of the four columns as an independent base, and sizing its anchorage for its own 333 kN·m, would miss the greater part of the demand actually delivered into the anchor group. The corresponding moment terms about the orthogonal axis sum to approximately zero, confirming that the loading is symmetric in the transverse direction.

4. Common Base Plate and Anchor Bolt Group

4.1. Bolt group layout

A 1900 mm × 1900 mm square plate is adopted, with 60 nos. M36 Grade 8.8 anchor bolts forming a single group common to the whole plate rather than four independent per-leg rings. The layout, shown in Fig. 2, comprises a cluster of four bolts at each of the four plate corners (16 nos.), a cluster of eight bolts at each of the four edge-mid zones between adjacent legs (32 nos.), and a twelve-bolt ring at the plate centre shared by all four legs.

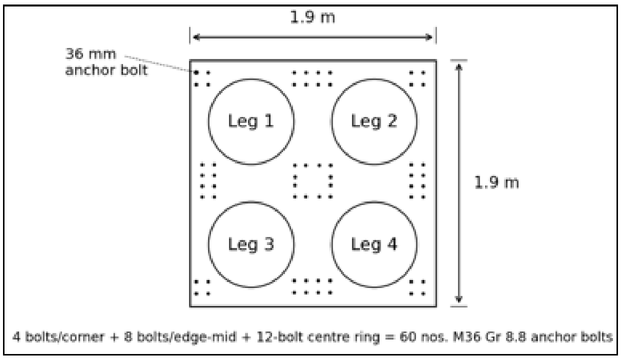

Figure 2: Common base plate — anchor bolt layout (60 nos. M36 Grade 8.8 bolts).

The corner bolt-cluster offset of 380 mm was selected specifically to satisfy the minimum edge distance of IS 800 Table 10. For a 40 mm hole,

$$e\ min = 1.7 \times 40 = 68\ mm\ ; \quad e\ actual = 450 - 380 = 70\ mm \qquad (4)$$

The check passes with only 2 mm to spare, and is the governing constraint of the whole connection. The maximum offset this plate size permits is about 382 mm, so there is very little geometric reserve; any change of hole size or plate dimension at fabrication stage requires this check to be repeated. The clearance between the CHS wall and the nearest bolt is 380 − 304.8 = 75.2 mm, which is a workable allowance for weld access and wrench engagement.

4.2 Bolt capacity and force distribution

The design tension and shear capacities per bolt, using the tensile stress area Anb = 817 mm² and shank area Asb = 1018 mm², are:

$$T\ db = 0.9\ fub\ Anb\ /\ \gamma mb = 470.6\ kN \qquad (5)$$

$$V\ dsb = (fub\ /\ \sqrt{3}) \times (Asb\ /\ \gamma mb) = 376.5\ kN \qquad (6)$$

The resultant is redistributed across the bolt group by the elastic method, in which the force in any bolt is the sum of a uniform axial component and a component proportional to its distance from the neutral axis:

$$F\ i = P\ total\ /\ n - M\ total \cdot z\ i\ /\ I\ zz \qquad (7)$$

with n = 60 and the bolt group second moment of area $I\ zz = \Sigma\ z^2 = 21.79\ m^4$. The results are summarised in Table 3.

Table 3: Elastic bolt force distribution

Result	Value
Max bolt tension	223.0 kN
Max compression-side force	243.3 kN
Bolts in net tension	30 of 60
Total tension / compression	3869.6 / 4481.4 kN
Equilibrium check	611.8 kN ✓

The maximum bolt tension utilisation is therefore 223.0/ 470.6 = 47.4 percent.

4.3. Combined shear and tension

The total horizontal shear at the plate is ΣFz = 415.8 kN, the orthogonal component cancelling by symmetry, giving 6.93

kN per bolt. The interaction check of IS 800 Cl. 10.3.6 gives:

$$(6.93/376.5)^2 + (223.0/470.6)^2 = 0.225 \leq 1.0 \qquad (8)$$

Shear is negligible in comparison with tension; the anchorage is almost entirely tension-governed, which is characteristic of a wind-dominated overturning case.

4.4. Bearing and plate thickness

Taking the compression side as resisted by bearing on the grout, the average bearing pressure is 4481.4 / (1.9 × 0.95) = 2.48 MPa against a permissible 0.45 fck = 11.25 MPa, a utilisation of 22 percent. The plate thickness is obtained by treating the plate as a cantilever strip fixed at the CHS wall and loaded by the maximum bolt tension at the 75.2 mm clearance:

$$t = \sqrt{(4\, Mu\, \gamma m0 \,/\, fy\, b)} = 41.1 \text{ mm} \qquad (9)$$

A 50 mm plate is adopted. Given the magnitude of load transferred, a complete joint penetration groove weld is specified around the full CHS circumference with matching E410 electrode and 100 percent ultrasonic and dye penetrant testing, rather than a sized fillet weld.

5. RCC Pedestal Design

The pedestal transfers the same combined resultant system into the footing, together with its own self weight. A 2200 mm × 2200 mm section of 1200 mm height in M30 concrete with Fe500 reinforcement is adopted. Because the base plate was analysed as a single rigid assembly, the pedestal is designed for the same P total, M total and V total rather than for individual leg actions.

Design by strain compatibility gives a moment capacity of 18,169.9 kN·m against a design moment of 6271.6 kN·m, a utilisation of 34.5 percent. The section is governed by the code minimum reinforcement rather than by demand, and 50 nos. 32 mm diameter bars distributed around the perimeter are provided, giving Ast = 40,210 mm² or 0.831 percent. Shear stress is 0.089 MPa and is not critical. Transverse ties of 10 mm diameter at 300 mm centres are provided, with additional ties local to the anchor bolt clusters. Bearing of the base plate on the pedestal is 2.48 MPa against 13.5 MPa permissible, a utilisation of 18.4 percent. An anchor plate of 150 × 150 × 20 mm is provided at each bolt, with an indicative embedment of about 450 mm.

The manual design was verified independently in RCDC, with the model linked directly from the analysis file. The two converge closely on capacity utilisation, 34.5 percent against 34 percent, because both are governed by minimum reinforcement rather than by applied moment. The reinforcement detailing produced by RCDC is reproduced in Fig. 3, which shows the 2200 mm square section with perimeter-distributed bars of 36 nos. T32 plus 20 nos. T25 and T8 links at 150 mm centres. It should be noted that the RCDC run was executed on M25 concrete against the M30 assumed in the manual calculation; the close agreement in utilisation arises because both are limited by the code minimum area of steel rather than by the applied moment.

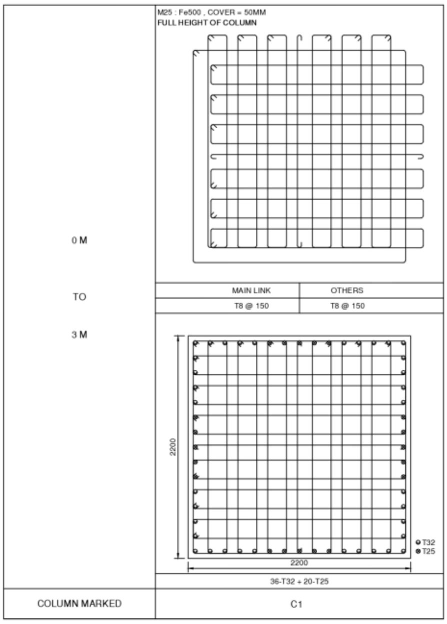

Figure 3: RCC pedestal reinforcement detailing (2200 mm × 2200 mm section, Column C1)

6. Footing Size and Bearing Check

The pedestal is carried on a 6.7 m × 6.7 m isolated footing, 0.85 m deep, founded at 2.0 m below ground level, giving a total embedment of 2.045 m to the underside of the base as shown in Fig. 4. The founding stratum is a silty sand with a governing SPT N value of 13 at founding level, and the water table coincides with the founding level.

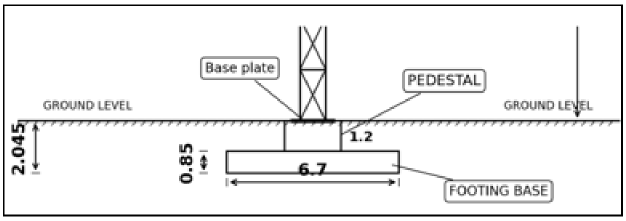

Figure 4: Foundation detail- base plate, pedestal and isolated footing (dimensions in metres).

The bearing capacity was assessed to IS 6403:1981 using Terzaghi's formulation for cohesionless soil, with the field N value corrected for overburden pressure and for dilatancy, and correlated to an angle of internal friction of 33 degrees. This gives a net ultimate bearing capacity of 2160.5 kN/m², a net safe bearing capacity of 864.2 kN/m² with a factor of safety of 2.5, and a gross safe bearing capacity of 900.2 kN/m².

The footing plan size is governed not by the vertical load, which is modest at 611.8 kN, but by the overturning moment of 5772.6 kN·m. The eccentricity e = M/P is large relative to the footing dimension, so the plan size is set by the requirement to keep the resultant within a workable proportion of the base and to limit the edge pressure, rather than by the average pressure. This is why a structure whose total vertical load would be carried comfortably by a footing of around 1 m square nonetheless requires a 6.7 m square base.

Table 4: Summary of foundation elements

Element	Size	Utilisation
Base plate	1900 × 1900 × 50 mm	—
Anchor bolts	60 nos. M36 Gr 8.8	47.4 % tension
Bolt shear-tension	—	0.225 ≤ 1.0
Plate bearing	—	22.0 %
Pedestal	2200 × 2200 × 1200 mm	34.5 % moment
Pedestal bearing	—	18.4 %
Footing	6.7 × 6.7 × 0.85 m	SBC 900.2 kN/m²

7. Results and Discussion

Table 5 collects the principal results of the design. Three observations follow from them.

Table 5: Summary of results

Quantity	Value	Check
Net vertical load, P total	611.8 kN	—
Total overturning moment, M total	5772.6 kN·m	—
from axial force couple	4446.8 kN·m (77 %)	governs
from local leg moments	1325.8 kN·m (23 %)	—
Max bolt tension	223.0 / 470.6 kN	47.4 % OK
Combined shear-tension	0.225	≤ 1.0 OK
Plate bearing pressure	2.48 / 11.25 MPa	22.0 % OK
Plate thickness required	41.1 mm (50 adopted)	OK
Pedestal moment	6271.6 / 18169.9 kN·m	34.5 % OK
Pedestal bearing	2.48 / 13.5 MPa	18.4 % OK
Gross safe bearing capacity	900.2 kN/m²	OK
Anchor bolt edge distance	70 mm vs 68 mm min	governs, 2 mm

First, the demand on the anchorage of a multi-column common base plate is dominated by the axial force couple between the columns, not by the columns' own bending. Here the couple contributes 77 percent of the total overturning moment. Any design approach that treats the four column bases independently will under-estimate the anchorage demand by a wide margin.

Second, every strength check in the connection sits below 50 percent utilisation, with the highest being bolt tension at 47.4 percent. The design is not strength-critical. What governs instead is geometry: the anchor bolt edge distance clears the code minimum by 2 mm out of 70 mm. In a connection of this type the plate dimension, bolt count and cluster offset are mutually constrained, and the practical limit is reached through the plate geometry well before the bolts reach their capacity.

Third, the low utilisations should not be read as excess conservatism that could simply be designed out. The bolt count cannot be reduced without increasing the peak bolt tension, and the plate cannot be made smaller without violating the edge distance. The reserve is a consequence of a discrete, geometry-limited layout rather than of a deliberately conservative choice of section.

A limitation should be noted. The bearing check uses the standard elastic approximation, in which the compression side is taken by bearing and the tension side by the bolts, rather than a full neutral-axis iteration. Given the wide margin found here a full iteration is unlikely to alter the conclusion, but it is recommended where the exact bearing distribution is required. Similarly, the anchor bolts have been checked individually; a group anchor breakout check to ACI 318 Chapter 17 is recommended for the closely spaced 60-bolt cluster and is not covered here.

8. Conclusion

The foundation connection for a 19 m wide, 18 m tall hoarding on a four-column CHS mast has been designed as a single shared assembly. Combining the four leg reactions at the plate centroid gives a resultant of 611.8 kN with an overturning moment of 5772.6 kN·m, of which the axial force couple between the legs contributes 4446.8 kN·m. A 1900 mm × 1900 mm × 50 mm base plate with 60 nos. M36 Grade 8.8 anchor bolts, a 2200 mm × 2200 mm × 1200 mm M30 pedestal and a 6.7 m × 6.7 m isolated footing at 2.0 m embedment are adequate for this loading, with a maximum bolt tension utilisation of 47.4 percent, a combined shear-tension utilisation of 0.225 and a pedestal moment utilisation of 34.5 percent.

The principal finding for design practice is that a multi-column common base plate must be analysed as one shared rigid assembly. The governing check is the anchor bolt edge distance, which passes by 2 mm, and not any strength check; the connection is limited by plate geometry rather than by material capacity.

References

[1] IS 875 (Part 3):2015, Design Loads (Other than Earthquake) for Buildings and Structures — Code of Practice, Part 3: Wind Loads, Bureau of Indian Standards, New Delhi, 2015.
[2] N. Cook, The Designer's Guide to Wind Loading of Building Structures, Butterworths, London, 1985.
[3] S. A. Shaikh and G. V. Joshi, "Effect of Wind on Soil-Structure Interaction: A Study on Hoarding Structure," International Journal of Innovative Research in Technology, Vol. 13, Issue 3, pp. 1–8, 2026.
[4] IS 800:2007, General Construction in Steel — Code of Practice, Bureau of Indian Standards, New Delhi, 2007.
[5] IS 456:2000, Plain and Reinforced Concrete — Code of Practice, Bureau of Indian Standards, New Delhi, 2000.
[6] IS 6403:1981, Code of Practice for Determination of Bearing Capacity of Shallow Foundations, Bureau of Indian Standards, New Delhi, 1981.
[7] SP:6-1, Handbook for Structural Engineers — Structural Steel Sections, Bureau of Indian Standards, New Delhi.
[8] K. Terzaghi, Theoretical Soil Mechanics, John Wiley & Sons, New York, 1943.
[9] R. B. Peck, W. E. Hanson and T. H. Thornburn, Foundation Engineering, 2nd ed., John Wiley & Sons, New York, 1974.
[10] ACI 318-19, Building Code Requirements for Structural Concrete, American Concrete Institute, Farmington Hills, MI, 2019.