IS : 9456 - 1980
(Reaffirmed 2003)
Edition 1.1
(1982-03)

Indian Standard

CODE OF PRACTICE FOR DESIGN AND CONSTRUCTION OF CONICAL AND HYPERBOLIC PARABOLOIDAL TYPES OF SHELL FOUNDATIONS

(Incorporating Amendment No. 1)

UDC 624.15.023.62 : 624.04 : 006.76

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

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Indian Standard

CODE OF PRACTICE FOR DESIGN AND CONSTRUCTION OF CONICAL AND HYPERBOLIC PARABOLOIDAL TYPES OF SHELL FOUNDATIONS

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Indian Standard

CODE OF PRACTICE FOR DESIGN AND CONSTRUCTION OF CONICAL AND HYPERBOLIC PARABOLOIDAL TYPES OF SHELL FOUNDATIONS

0. FOREWORD

0.1 This Indian Standard was adopted by the Indian Standards Institution on 18 March 1980, after the draft finalized by the Foundation Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Shells are structures which derive their strength from 'form' rather than 'mass'. The basic attribute of the shell which recommends its use in roofs is economy under conditions of large spans, apart from aesthetics, which, of course, is of no concern in the case of a buried structure like the foundation. It has been found in respect of foundations that in situations involving heavy column loads to be transmitted to weaker soils, adoption of shells can lead to substantial saving in concrete and steel.

0.2.1 Analysis has indicated that the economy with shell foundations normally increases with increase in column load and decreases in allowable bearing pressure of the soil, with greater sensitivity to the latter.

0.2.2 Attendent on the saving in valuable materials of constructions, is the fact that in all cases shell footings are substantially lighter than their plain counterparts. The attribute of lightness and the consequent ease for transportation indicate high scope for precasting these shell footings.

0.2.3 Since foundation shells bear directly on soil at their bottom and carry backfill on top, besides being deep and thick, the problem of elastic stability (buckling) is of lesser concern in foundation shells when compared to roof shells. However, cracking of concrete is a subject of greater concern, as with all foundations, particularly under deleterious ground environments, for fear of corrosion of the reinforcing steel. Hence sufficient cover requirements and other preventive measures are indicated. It may be noted here that design based on membrane theory usually results in nearly uncracked sections at working loads.

0.3 Even though a variety of shells such as cylinder, cone hyperbolic paraboloid, elliptic paraboloid and inverted dome, and also funicular shells, can be judiciously adapted in various foundation situations, this standard covers only conical and hyperbolic paraboloidal shells; these being of more frequent use in foundations.

0.3.1 Between cone and 'hypar' (common abbreviation for hyperbolic paraboloid), however, while the use of the former is limited to individual footings on account of its circular plan, the latter can be adopted for individual footings (square or rectangular), combined footings as well as for rafts.

0.4 The depth, thickness and boundary, as well as loading conditions of foundation shells are such that rigorous analyses involving them are necessarily much more complex than those of roof shells. The state of stress in foundation shells can be predicted to any reasonably high degree of accuracy only by a rigorous 'bending analysis' involving the above factors. Such an analysis being not easily amenable to practical use, the design of foundation shells is usually made on the basis of the much simpler. 'membrane analysis', which is based on a large number of radically simplifying assumptions with regard to the factors mentioned above. The membrane analysis is invariably a conservative design aid, and the approach to design based on it, with necessary modifications in the matter of detailing which will ensure the high ultimate strength (load carrying capacity) of these foundations, has been found to be sufficient for all practical purposes. Hence the same is recommended in this code.

NOTE — The provisions given in this standard have been explained in detail in the book 'Modern Foundation — An Introduction to Advanced Techniques: Part I Shell Foundation' by Dr Nainan P. Kurian.

0.5 This edition 1.1 incorporates Amendment No. 1 (March 1982). Side bar indicates modification of the text as the result of incorporation of the amendment.

0.6 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard covers the design and construction aspects pertaining to conical and hyperbolic paraboloidal types of shell foundations subjected to the action of isolated column loads.

*Rules for rounding off numerical values (*revised*).

2. TERMINOLOGY

2.1 For the purpose of this standard the definitions given in IS : 1904-1978*, IS : 6403-1971†, IS : 2210-1962‡, IS : 2204-1962§, and the following shall apply.

2.1.1 *Shell Foundation* — Foundations which incorporate structural shell elements in place of the plain element of ordinary shallow foundations.

3. NECESSARY INFORMATION

3.1 The information called as in IS : 1080-1962|| and IS : 2950 (Part I)-1973¶ are required for the purpose of this code. The additional information as indicated in **3.1.1** will also be necessary.

3.1.1 *Suitability of In-situ Soil for Core Preparation* (*see* Fig. 1 and 3) *Under Shell Foundations* — If *in-situ* soil is shrinkable, necessity for bringing non-swelling soil from elsewhere for this purpose is indicated. (This is necessary to allay the fear of partial loading of the shell brought about by a variable subsidence of the core soil).

4. PRELIMINARY DESIGN CONSIDERATIONS

4.0 The complete design of a shell foundation, consists of two parts, namely, 'soil design' and 'structural design'.

4.1 The aim of soil design is to proportion the foundation (that is, to determine its plan dimensions) so that the 'net loading intensity' (*see* IS : 6403-1971†) under actual field conditions does not exceed the 'allowable bearing pressure' (*see* IS : 6403-1971†), which is the lesser of (a) the 'safe net unit bearing capacity', and (b) soil pressure for a given permissible settlement. It may be noted in this connection that in case of sand the safe net unit bearing capacity increases and soil pressure for a given settlement decreases with increase in the foundation width, unlike the case of clay where the safe net unit bearing capacity is independent of the foundation dimensions.

NOTE — Width is the smaller of the plan dimensions, which alone influences these quantities.

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*Code of practice for structural safety of buildings : shallow foundations (*second revision*).

†Code of practice for determination of allowable bearing pressure on shallow foundations.

‡Criteria for the design of reinforced concrete shell structures and folded plates.

§Code of practice for construction of reinforced concrete shell roof.

||Code of practice for design and construction of simple spread foundations.

¶Code of practice for design and construction of raft foundations: Part I Design (*first revision*).

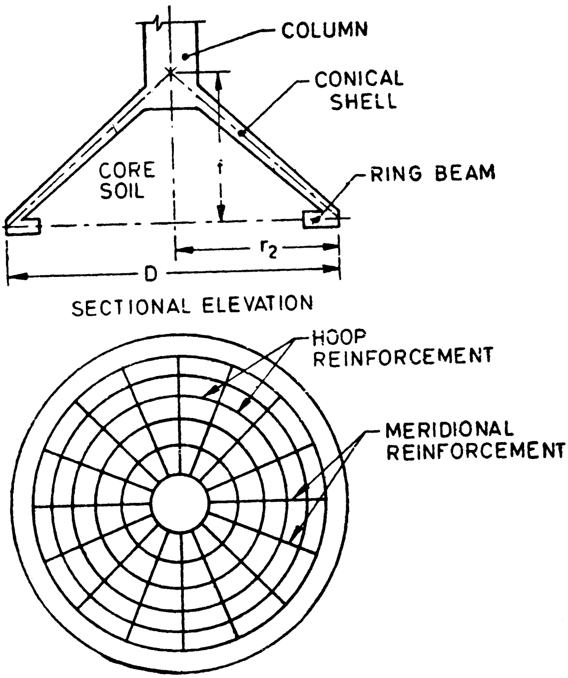


FIG. 1 PLAN SHOWING REINFORCEMENT OF CONICAL FOOTING

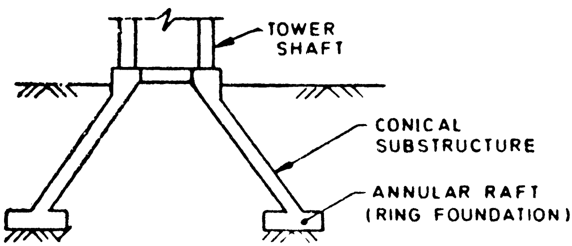


FIG. 2 CONICAL SUBSTRUCTURE FOR TOWERS

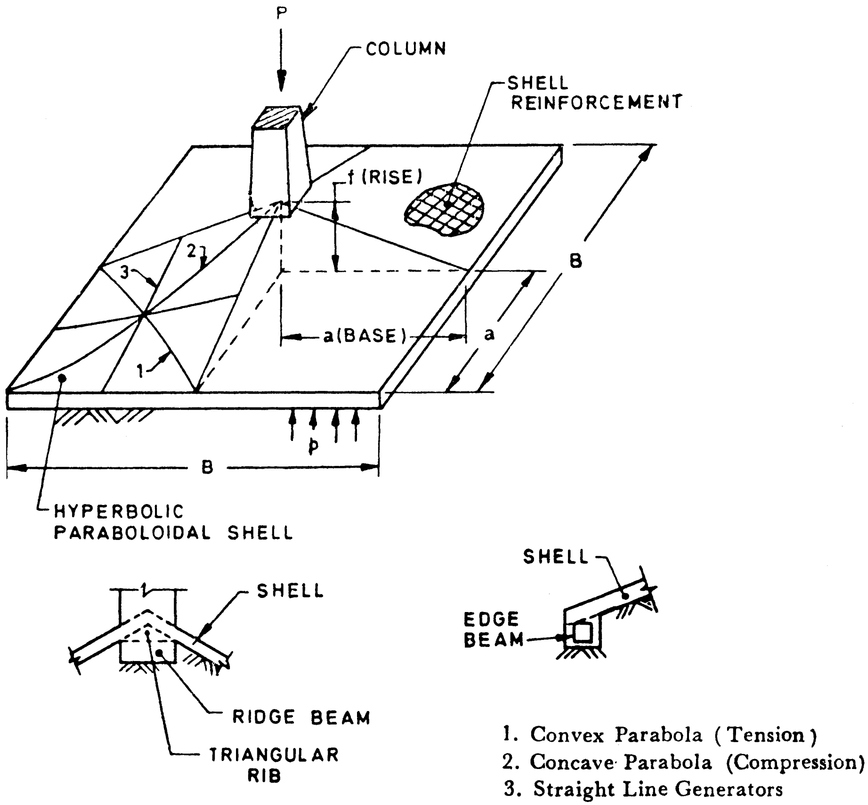


FIG. 3 HYPERBOLIC PARABOLOIDAL SHELL FOOTING

4.2 The net loading intensity and the allowable bearing pressure should be determined according to IS : 6403-1971*. The influence of the position of water-table on these quantities should be carefully ascertained and duly taken into acount.

4.3 If the soil filling the hollow space underneath the shells (core) (*see* Fig. 1) is assumed to be incompressible and act integrally with the foundation, the soil response below the shell foundation in terms of both bearing capacity and settlement will be modified to the extent of

*Code of practice for determination of allowable bearing pressure on shallow foundations.

the additional friction that will be mobilized at the bottom of the trench between soil and soil, than at the interface between foundation and soil as in the case of plain foundations. However, results of limited number of tests tend to indicate that this variation in soil response is marginal. Hence it is customary to ignore this difference and assume the bearing capacity and settlement under shell and plain foundations to be identical, under identical foundation conditions, for the purpose of soil design.

5. STRUCTURAL DESIGN

5.0 The structral design of the foundation should follow after proportioning the foundations in accordance with the requirements set out in **4**.

5.1 The conical footing shown in Fig. 1 is the simplest form in which a shell is made use of in foundation. The provision of radial and circumferential steel is as simple as for a circular plain raft (footing) while the construction is only a little more difficult.

NOTE — While this type of conical foundation is potentially suited for individual columns, chimney stacks and similar tower shaped structures, the majority of instances in which these shells have been adopted are for tall telecommunication towers (television, radio, telephone, etc) in reinforced concrete, where they serve not as regular foundations, but as substructures linking the tower shaft to the annular raft, or ring which is regular foundation bearing on soil (*see* Fig. 2). The space within this conical substructure is utilized for services. Very often these cones are stiffened internally, the stiffening taking various forms, to resist moments and shears due to wind effects, etc. Prestressing is indicated for the hoop reinforcement in the cone as well as the foundation ring, to prevent or limit the width of cracks in concrete. These conical shells being substructures, are beyond the scope of this standard.

5.2 While the cone is a singly curved shell, the hyperbolic paraboloid is a doubly curved anticlastic shell with its surface made up of two sets of parabolae having curvatures in opposite directions. The chief advantage of the hypar, however, is that just as the cone, it is also a ruled surface, (*see* Appendix A of IS : 2210-1962* for shell classification) consisting of two sets of straight line generators inclined at 45° to the parabolae, as shown in Fig. 3.

5.2.1 This straight line property of the cone and hyperbolic paraboloid are effectively exploited in profiling the core soil and the shell, besides preparing the reinforcement grills, and formwork for making precast shell footings.

5.2.2 The combination of hypar shell elements (square or rectangular) with set of edge and ridge beams shown in Fig. 3, is popularly known as the 'umbrella' footing, it being the natural offshoot of the well known 'inverted umbrella' shell used in the construction of roofs.

*Criteria for the design of reinforced concrete shell structures and folded plates.

5.3 In the dimensioning of the shell foundations, the ratio of rise to base radius (f/r_2) in the case of cone (*see* Fig. 1), and the rise to base ratio of the shell (f/a) in the case of hyperbolic paraboloid (*see* Fig. 3), shall vary from 0.5 to 1. From the point of view of ease of construction, values near the lower limit are more suitable. It may, however, be noted that membrane theory will not be adequate for design at very low values of rise.

5.4 The bottom rig beam in the case of cone and the edge and ridge beams in the case of hyperbolic paraboloid are to be provided within the shell dimension as shown in Fig. 1 and Fig. 3 respectively, so as to keep the plan dimensions arrived at by soil design intact.

5.4.1 In the case of the cone, the ring beams at the bottom are found to contribute to the stiffness of the footing at lower rises ($f/r_2 < 0.5$), without any marked contribution at higher rises.

5.4.2 In the case of hyperbolic paraboloid, footings have been designed without ridge beams but with thick edge beams, and alternatively, with heavy ridge beams but without any edge beams. However, footings with both edge and ridge beams should be able to adapt themselves better to irregular distribution of soil reaction and accidental eccentricities in load that are bound to occur in practice. As such footings of this kind are to be recommended in normal cases.

5.4.3 As far as the positioning of the beams is concerned, downstanding beams as shown in Fig. 3 are preferred as they are easier to construct and structurally more efficient from the point of view of possible bending.

5.4.4 When feasible, the width of the ridge beam may be made equal to the width of the column base (*see* Fig. 3). Where possible, in place of the projecting ridge beams, it may be more expedient, from the point of view of construction both by *in-situ* and precast methods, and also economy to provide triangular ribs at the ridge, with its rise decreasing from a maximum at the column and vanishing at the joint with edge beams.

5.4.5 The ring beam in the case of the cone and edge beams in the case of hyperbolic paraboloid, in addition to improving the stiffness, delay cracking of the shell and also contribute substantially to the ultimate load carrying capacity of these foundations by providing substantial reserve of strength, leading thereby to higher load factors. From the point of view of cracking, strong scope also exists for prestressing these beams.

5.5 Where cover requirements, and not stresses, govern the foundation, shell shall have a minimum thickness of 15 cm. (In precast construction this can be reduced to 12 cm.)

5.6 On the basis of the assumption that the weight of the core, mud mat, backfill and the self weight of the shell foundation, are directly transmitted to the soil below in such a manner as will not induce any substantial stresses in the shell foundation, the structural design of the shell foundation may be carried out for the maximum load transmitted at the foot of the column to the foundation, as done in respect of ordinary plain foundations.

5.7 When the above load is divided by the plan area of the foundation (A_p) which has been already finalized at the end of the 'soil design' (*see* 4), the average intensity of the soil pressure $p_v = \frac{P}{A_p}$ for the structural design of the footing, is obtained. This pressure may be assumed to be uniform for the purpose of structural design.

5.8 At every point of contact between the shell (and also beams) and soil, the soil reaction or 'contact pressure' can have normal and tangential components. The distribution of the actual resultant contact pressure is highly indeterminate, because of the elastic nature of the soil support, and the complex shell-beam-soil interaction. In the case of soft clay where no tangential frictional contact pressure components can be sustained because. of the negligible wall friction, the resultant soil pressure may be taken to be normal to the shell. However, in the case of sand, since tangential pressures of considerable magnitude can be mobilized due to the availability of higher contact friction, the resultant contact pressure can show a substantial shift from the normal to the vertical. As a general rule, it may be safer to design for the condition giving rise to higher stresses in each case. It may be noted in this connection that the intensity of the normal contact pressure (when tangential components are absent) is also obtained as P/A_p if the latter is also assumed to be uniform, which is the same as p_v, the intensity of vertical pressure, where A_p is the projected area of the foundation in plan (*see also* Appendix A).

5.9 Under a uniform contact pressure, normal or vertical, the conical shell is subjected to hoop tension decreasing upwards from a maximum at the base and a meridional compression decreasing downwards from a maximum at top (*see* Appendix A). Hoop steel is to be provided to take up the full tension, with preferably varying spacing, to match the variation in hoop tension. The horizontal sections which are in compression may be designed as short columns with steel not exceeding 5 percent. The steel so designed should be placed at the middle plan of the shell. It may further be ensured that sections are provided with minimum nominal steel of 0.5 percent.

5.9.1 The thickness of the cone may be varied from a maximum at the top to a minimum at the bottom. The maximum tensile hoop stress in the equivalent concrete section may be checked according to IS : 456-1978* and the thickness finalized (*see* **5.5**).

5.9.2 The ring beam at the bottom of the cone is optional. However, when the frustrum of a cone is used as foundation for a tower shaft, a ring beam at the top is essential to balance the horizontal component of the meridional compression at the top edge of the cone, which produces hoop compression in the latter.

5.9.3 The cracking strength of the above membrane design is normally higher than the load given by the membrane theory. The ultimate strength may be worked out by any suitable theory (*see* Appendix A) and the load factor ascertained. It may be mentioned here that with the onset of peripheral cracking, the soil pressure shows a tendency for shift from edges to the centre, which incidentally helps to increase the ultimate strength.

5.9.4 A cone may also be used in the inverted position as foundation for structures such as guyed masts (*see* Fig 4). In this case the loading (soil pressure) on the cone reverses sign subjecting the cone to meridional tension and hoop compression. Use of cone in this manner has the disadvantage of heavy meridional tension, for design, at the bottom sections of the cone.

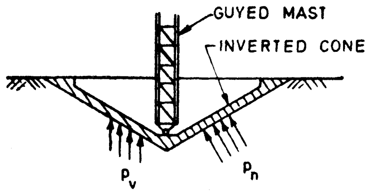


FIG. 4 INVERTED CONE AS FOUNDATION FOR GUYED MAST

5.10 The hypar footing shown in Fig. 3 is designed on the basis of the membrane theory used in the design of the corresponding inverted umbrella roof. According to this theory, under a uniform vertical soil pressure, the shell membrane is subjected to a state of pure shear of constant magnitude unaccompanied by normal stresses. This membrane shear, produces tension and compression of equivalent magnitude as the shear along the diagonally orthogonal convex and concave parabolic arches respectively (*see* Appendix A). Since the design of the shell is governed by this tension, the full requirement thereof is to be provided in terms of steel. However to avoid the

*Code of practice for plain and reinforced concrete (*third revision*).

necessity of bending bars to different parabolic profiles, it will be more expedient from the point of view of facility of grilwork, to detail the steel in the shell as straight rods along directions parallel to the edges in such a manner that its effective area along the diagonal is sufficient to withstand the full tension. Since this arrangement produces the same effective steel along the directions of the compressive arches also, the presence of concrete makes the compressive arches also stronger than the tensile arches, thereby leading to a slightly unbalanced, but at the sametime, safer design. It may be ensured that the steel thus provided does not fall below a value of 0.5 percent. According to the membrane theory, this steel should lie at the middle plane of the shell. In most instances concrete will be needed only as a cover for steel. Checking the tensile stress in the equivalent concrete section in accordance with IS : 456-1978* will usually reveal very low stresses, thereby ensuring practically uncracked sections at working loads.

5.10.1 According to the membrane theory, the edge beams are subjected to uniformly varying tension with zero value at corners and maximum value at the centres of edges (*see* Appendix A). Therefore, these central sections may be designed on the same lines as the shell. The section of the edge beam may be determined on the basis of limiting tension according to IS : 456-1978* and the edge steel detailed ensuring proper cover requirements. Notwithstanding the reduction in tension, however, the same section is normally provided throughout the edge. As far as the ridge beams are concerned, they are subjected to axial compression with zero value at the base and maximum value at the apex (*see* Appendix A). The section of the ridge beam may be designed as a short column with steel not exceeding 5 percent and detailed ensuring proper cover requirements. Irrespective of the variation in compression, the same section may be provided throughout the ridge as done in the case of edge beams. The design is complete with stirrups (nominal according to membrane theory) provided both in the ridge and edge beams.

5.10.2 Further detailing practices necessary to ensure the full ultimate strength of the hypar foundation are given in Appendix B.

5.10.3 Footings designed on the above lines crack at loads higher than those given by the membrane theory. The full ultimate strength of the footing may be determined by a suitable theory (*see* Appendix A) for ascertaining the load factor.

5.11 Since hyperbolic paraboloidal combined footings and rafts (Fig. 5 and 6) are essensially multiple units of the individual footing, these are designed on the same basis, except that valley beams which are

*Code of practice for plain and reinforced concrete (*third revision*).

edge beams common to two shells on either side, should be designed for the combined tension. Depending upon the area requirement of the foundation (soil design), the spacing of the columns, and the difference in column loads, different sets of square or rectangular shells will result in the design. The same applies to individual rectangular footings (*see* Fig. 7). However, where the column loads are unequal, it will be profitable to ensure that the resultant column load passes through the centre of gravity of the area of contact between the foundation and the soil in plan, so that the soil pressure on the foundation will be uniform throughout.

5.11.1 Where soil conditions permit (in terms of the requirements on plan area), the inverted hipped hyperbolic paraboloid (*see* Fig. 8) normally used in roofs, may suggest itself as a possible alternative for use as foundation. While this combination has the structural advantage that both the sets of beams are in compression, notwithstanding the necessity for tie beams between columns, its chief drawback in foundation is the difficulty of providing effective soil support below the triangular edges. Hence this type cannot be recommended for foundations in normal cases.

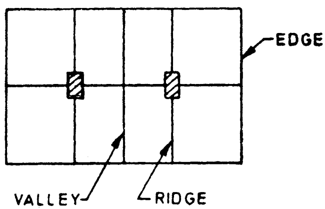


FIG. 5 COMBINED HYPAR FOOTING

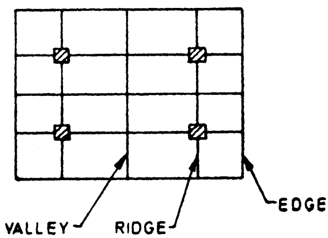


FIG. 6 HYPAR RAFT

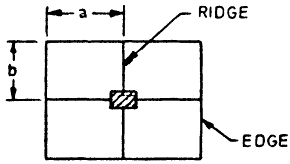


FIG. 7 RECTANGULAR HYPAR FOOTING

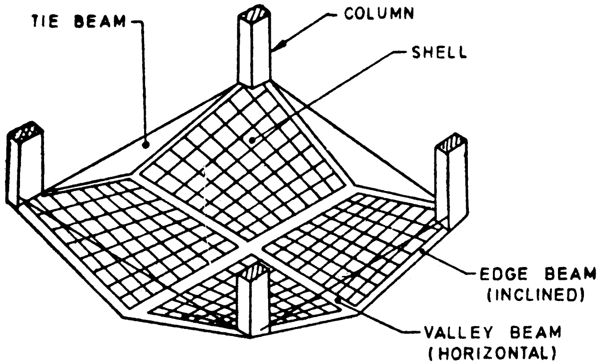


FIG. 8 INVERTED HIPPED HYPERBOLIC PARABOLOIDAL RAFT

5.12 As with other foundations, shell foundations also may be called upon to resist horizontal loads and moments at the level of its base, as a result of horizontal loads or couples or both transmitted from above or due to eccentricities of column loads. As for horizontal loads, shell foundations have the advantage of higher capacities to the extent of the increased friction (soil to soil contact) at the base even though its self-weight may be less than that of its plain counterpart. As regards moments, the same may be treated as in the case of plain foundations, as resulting in a linearly varying soil pressure distribution. Under such circumstances, the individual shell elements may be designed for the maximum soil pressure occurring under it due to the combined effect of vertical load and moment, to be on the safer side. However, where membrane solutions are available for the anti-symmetrical soil pressure produced by moment the stress resultants the latter may be superimposed on the stress resultants produced by the symmetrical soil pressure due to the vertical load for the purpose of the designs.

6. CONSTRUCTION

6.1 The concrete for shell foundations should be of grade not less than M20.

6.2 Shell foundations may be cast *in-situ* or precast. Even though these foundations are generally laid *in-situ*, the advantages of the shell in terms of lightness and transportability is best exploited in precasting. Because of this basic attribute of lightness, it should be noted that even large-sized footings of this kind are amenable to precasting. To this must be added the possibility of higher strength for the same mix (*see* **6.1**) on account of the better control that can be exercised during prefabrication.

6.3 In the *in-situ* method of construction, the shell foundation is cast at site on the soil core which has been cut to the correct profile of the shell. The straight line property of the shell enables this profiling to be simply achieved by rotating a template about a central axis in the case of the cone (*see* Fig. 9), and by moving a straight edge after establishing the ridge and base lines in the case of the hyperbolic paraboloid (*see* Fig. 10). A thin layer of lean cement mortar (mix not higher than 1 : 3) is then placed over the soil core (*see* Fig. 11). This is done to facilitate grillwork (bending and tying of reinforcements) and subsequent casting. Even when the foundations are moderately steep, formwork is needed only at the edges.

6.3.1 In the case of expansive soils, the core on which the footing is to be laid, should be prepared by cutting a trench to level bottom and filling it with non-swelling, or if possible with stabilized, soil. The soil is then compacted and profiled as described in **6.3** (*see* Fig. 12). This will prevent the chances of subsidence of the core brought about by a possible shrinkage. At any rate this will give rise to conditions at the base level of the shell foundation similar to those under plain foundation. To this must be added precautions normally taken in respect of plain shallow foundations in shrinkable soils.

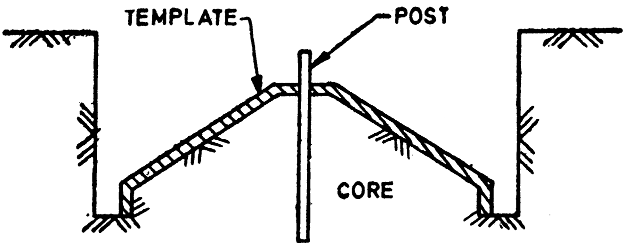


FIG. 9 CORE PROFILING FOR CONE

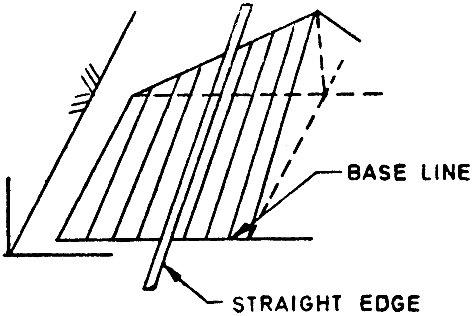


FIG. 10 CORE PROFILING FOR HYPAR

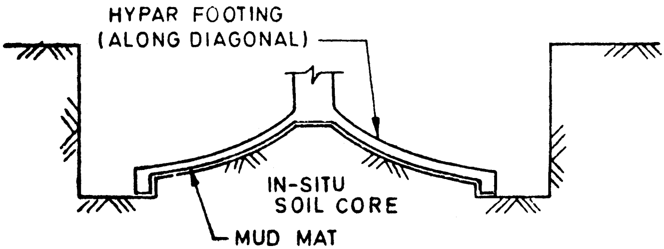


FIG. 11 *In-situ* CONSTRUCTION (SECTION ALONG DIAGONAL)

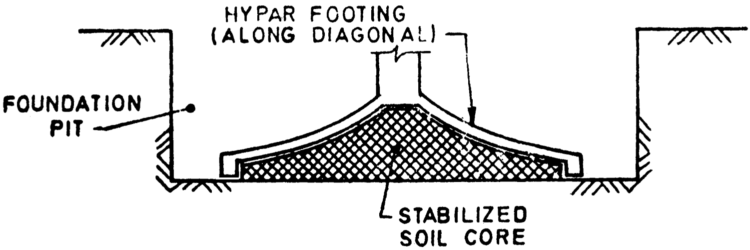


FIG. 12 *In-situ* CONSTRUCTION ON STABILIZED SOIL CORE (SECTION ALONG DIAGONAL)

6.3.2 In any case, whether the construction is *in-situ* or precast, it is very important to ensure that there is no loss of contact anywhere between the footing and the soil, since partial contact will lead to concentration of loads (soil pressure) on the shell, which can vitiate the performance of the shell itself, and precipitate premature collapse.

6.4 Precast cone and hypar footings may be cast in inverted wooden mould which helps easier removal of the footing from the mould facilitated by shrinkage. The moulds may be easily formed by cutting and nailing plywood strips along the directions of the straight line generators into a frame (*see* Fig. 13). An alternative technique which may be simpler and certainly more advantageous in terms of the number of units that can be turned out from each mould would be to make a mould in concrete itself (*see* Fig. 14). This can be done by making a box with wooden sides to conform to the edges and filling, the inside with lean concrete, profiling the same by template or straight edge as the case may be and finishing it smooth with cement paste.

6.4.1 In precast construction, however, it will not be expedient to out the soil to the required profile first as done in the case of *in-situ* construction, and then place the footings on it, since in doing so full contact between the footing and the soil core cannot be ensured under all circumstances. Instead, it would be more expedient to install the precast footing in a trench cut to level bottom. After centering and levelling the footing, dry sand may be poured into the hollow space below the footing through a hole in the column base provided at the time of casting. The sand thus poured is to be compacted to high and uniform densities. In the case of steep conical footings this space is accessible for compaction by manual tamping through the hole. However, in the case

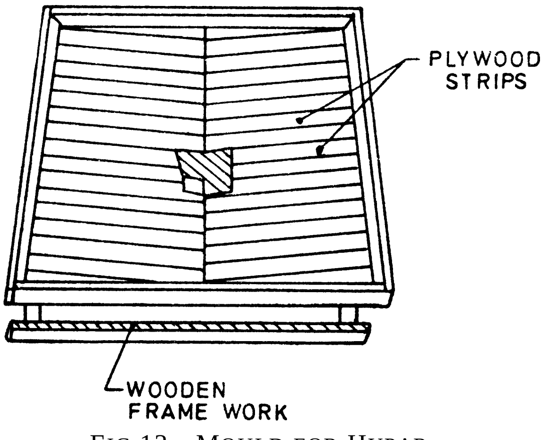


FIG 13 MOULD FOR HYPAR

of shallow conical footings, and hypar footings whose corners are substantially flat and therefore inaccessible even when the shells are deep, this sand is to be compacted by some remote technique, so as to form a sound core under the shell to receive the load. Such a simple but highly efficient technique of remote compaction is described in Appendix C. For connections with steel columns, bolts may be embedded in the column base at the time of casting which will engage the holes in the base plate of the column (*see* Fig. 15). Incorporation of a neoprene pad between steel column and base plate will serve as a hinge preventing the transmission of any moment to the footing. Connection with concrete columns may be effected through dowels protruding from the column base for continuous casting, or a socket arrangement in the column base into which a precast column is grouted as shown in Fig. 16.

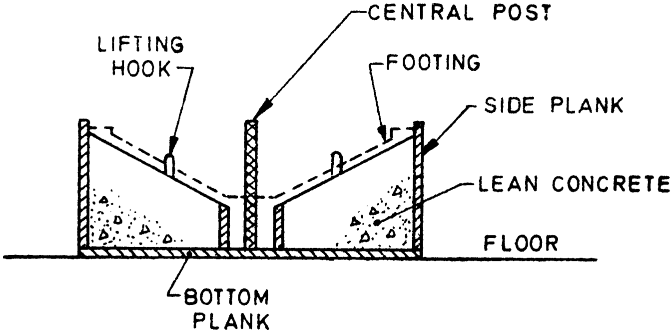


FIG. 14 INVERTED CONCRETE MOULD FOR CONE

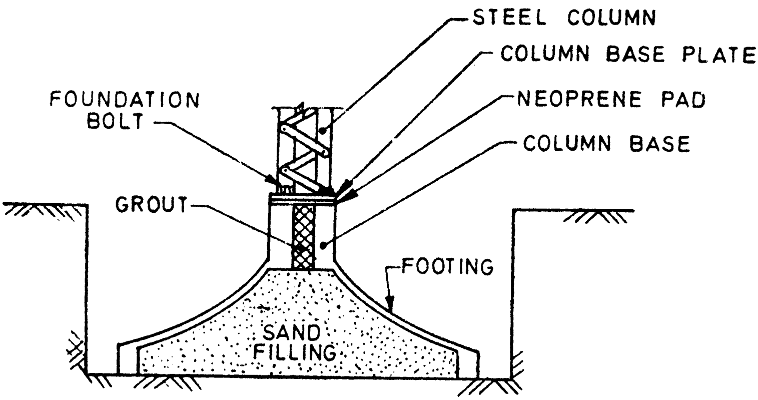


FIG. 15 FIXING OF STEEL COLUMN TO PRECAST HYPAR FOOTING

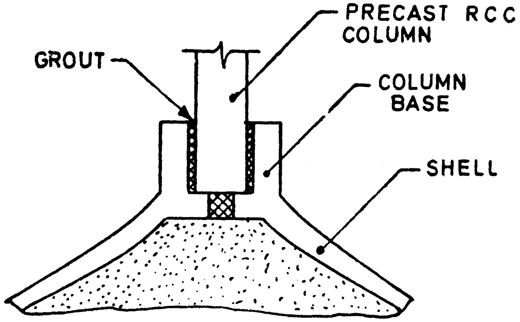


FIG. 16 SOCKET CONNECTION FOR R C COLUMN WITH PRECAST HYPAR FOOTING

APPENDIX A

(*Clauses* 5.8, 5.9, 5.9.3, 5.10, 5.10.1 *and* 5.10.3)

FORMULAE FOR THE DESIGN OF CONICAL AND HYPERBOLIC PARABOLOIDAL SHELL FOUNDATIONS

A-1. CONE

A-1.1 Membrane stress resultants per unit width of the shell due to vertical load and moment, are given below.

A-1.1.1 *Stress Resultants Under Vertical Soil Pressure* (*see* Fig. 17)

$$N_r = \frac{p_v}{2s} \tan\alpha \, (s^2 - s_2^2)$$

$$N_\theta = p_v s \frac{\sin^3\alpha}{\cos\alpha}$$

$$N_{r\theta} = 0$$

where p_v is the intensity of vertical soil pressure

$$p_v = \frac{P}{A_p}\text{, where } P = \text{column load, and}$$

A_p = plan area of the footing (= $\pi s_2^2 \sin^2 \alpha$ for full cone)

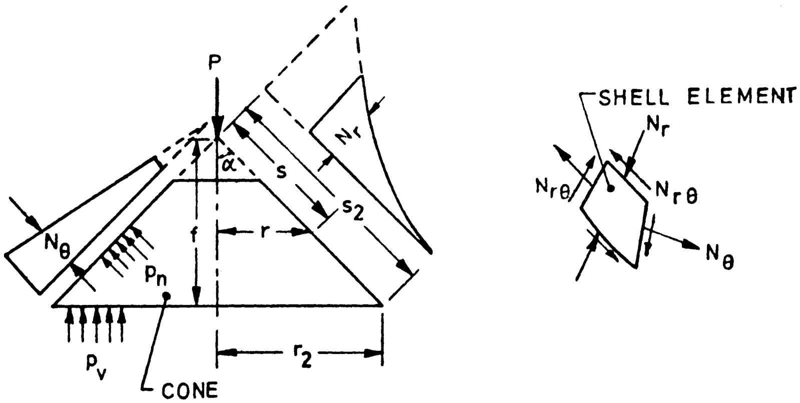


FIG. 17 MEMBRANE STRESS RESULTANTS IN CONICAL FOOTING

A-1.1.2 *Stress Resultants Under Normal Soil Pressure* (*see* Fig. 17)

$$N_r = \frac{p'_n}{2s} \tan\alpha \, [\, s_2^2 - s^2 \,]$$

$$N_\theta = p_n \, s \tan\alpha$$

$$N_{r\theta} = 0$$

where p_n is the intensity of normal soil pressure, and P/A_p is same as given under **A-1.1.1**.

The variation of the above stress resultants with the ratio of rise to base radius (f/r_2) is shown in Fig. 18.

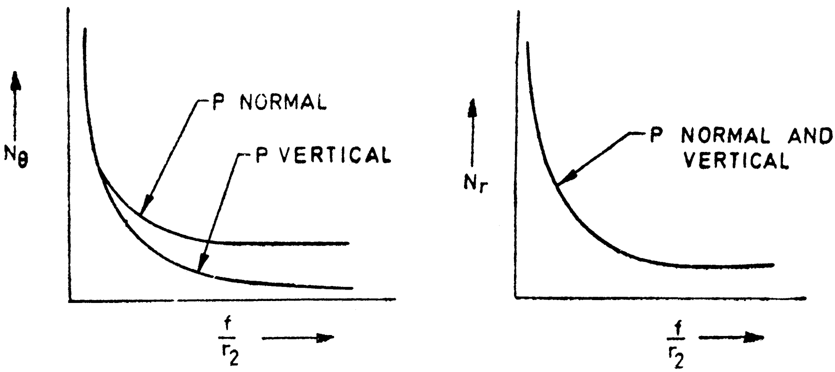


FIG. 18 VARIATION OF N_θ AND N_r WITH f/r_2 RATIO

A-1.1.3 *Stress Resultants Under Anti-symmetrical Soil Pressure Due to Moment Assuming the Soil Pressure to be Normal* (*see* Fig. 19)

$$N'_r = \frac{2p'_n}{s_2 \sin 2\alpha}\left[\frac{s_2^4 - s^4}{4s^2} - \frac{s_2^3 - s^3}{3s}\cos^2\alpha\right]\cos\theta$$

$$N'_\theta = \frac{p'_n}{s_2} s^2 \tan\alpha \cos\theta$$

$$N'_{r\theta} = \frac{p'_n}{4 s_2 s^2}\frac{(s_2^4 - s^4)}{\cos\alpha}\sin\theta$$

in which $p'_n = \dfrac{4M}{\pi s_2^3 \sin^3\alpha}$, where M is the moment producing the maximum anti-symmetrical soil pressure p'_n.

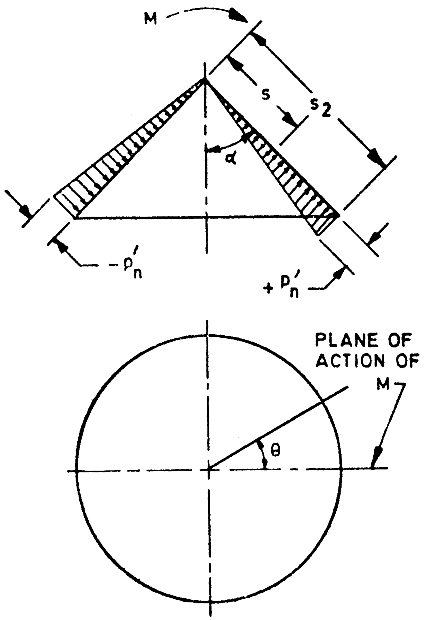


FIG. 19 CONICAL FOOTING UNDER MOMENT

A-1.2 The ultimate strength (value of soil pressure at which the footing fails structurally) P_u for uniform normal soil pressure, under assumptions of fixity at the upper edge, and a lower edge which is either free or provided with a ring beam, and assuming constant spacing of hoop steel, are given in **A-1.2.1** to **A-1.2.2** (*see also* Fig. 20).

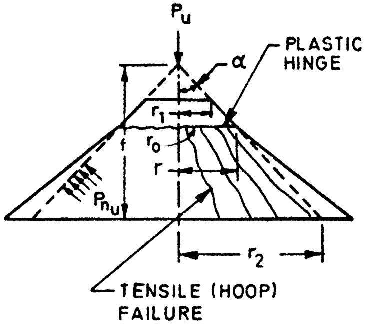


FIG. 20 ULTIMATE FAILURE OF CONICAL FOOTING

A-1.2.1 *Ultimate Normal Soil Pressure for Fixed Upper Edge and Free Lower Edge*

$$p_{nu} = 6 \left[\frac{\dfrac{\cos\alpha}{2}(1-R_o)^2 + \dfrac{M\sin^2\alpha}{N\ r_2}\ R_o}{R_o^3 - 3\ R_o + 2} \right] \frac{N}{r_2}$$

where

N = ultimate capacity of the shell per unit width in direct tension in the hoop direction (constant),

$R_o = \dfrac{r_o}{r_2}$, where r_o is the radius corresponding to the location of the plastic hinge.

M = moment capacity of the plastic hinge per unit width (r_o may be taken r_1 for all practical purposes).

A-1.2.2 *Ultimate Normal Soil Pressure for Lower Edge With Ring Beam*

$$p_{nu} = 6 \left[\frac{N\ \cos\ \alpha\ (1-R_o)^2}{2\ r_2(\ R_o^3 - 3\ R_o + 2)} + \frac{M\ \sin^2\alpha}{r_2^2}\ \frac{R_o}{R_o^3 - 3R_o + 2} + \frac{N_b\ \cos\ \alpha\ \sin\ \alpha\ (1-R_o)}{r_2^2\ (\ R_o^3\ - 3\ R_o + 2)} \right]$$

where N_b = ultimate capacity of the ring beam in direct tension

$$P_u = p_{nu} \times A_p$$

A-2. HYPERBOLIC PARABOLOID

A-2.1 The membrane stress resultants per unit width of the shell against vertical and normal soil reactions, together with the forces in beams, are given below (*see* Fig. 21)

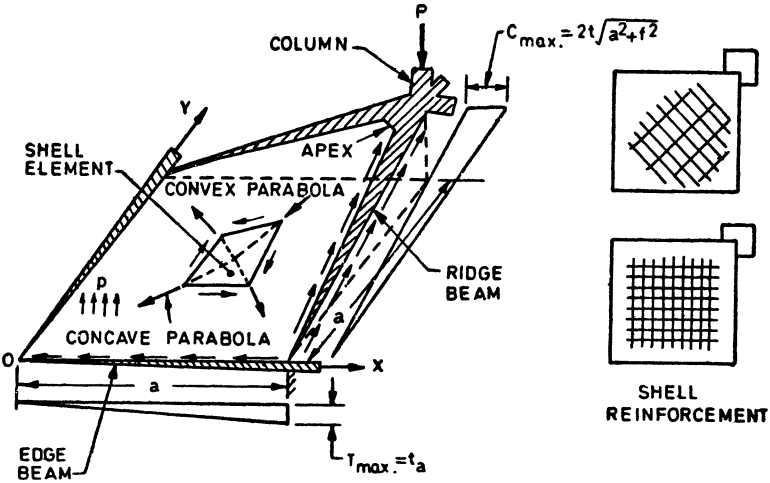


FIG. 21 MEMBRANE STRESSES IN HYPAR FOOTING

A-2.1.1 *Stress Resultants Under Vertical Soil Pressure*

$$N_x = N_y = 0$$

$$N_{xy} = t = \frac{p_V}{2k}$$

(N_x, N_y and N_{xy} are the membrane stress resultants. 't' is the equivalent tension per unit width developing in the convex parabolae).

where, $k = f/ab$ in which a and b are the plan dimensions of the rectangular hyperbolic paraboloidal shell quadrant

('k' is called 'warp' of the shell).

For a square shell ($a = b$)

$k = f/a^2$

For the square hypar footing, $T = t \, . \, a$

where T is the maximum direct tension in the edge beam, at the centre, and

$$C = 2t\sqrt{a^2 + f^2}$$

where C is the maximum direct compression in the ridge beam, at the apex.

A-2.1.2 *Stress Resultants Under Normal Soil Pressure*

$$N_x = 2\,p_n\,z\sqrt{\frac{1+k^2y^2}{1+k^2x^2}}$$

$$N_y = 2\,p_n\,z\sqrt{\frac{1+k^2x^2}{1+k^2y^2}}$$

where

$z = k.x\,y$, is the co-ordinate of the point (*see* Fig. 21).

[N_x and N_y are tensile]

$$N_{xy} = \frac{p_n}{2k}\,.\,(\,1 + k^2x^2 + k^2y^2\,)$$

(N_x and N_y are tensile)

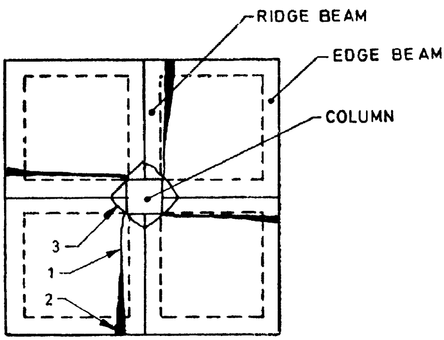
where $u = \sqrt{1/\mathrm{k}^2 + y^2}$

and $v = \sqrt{1/\mathrm{k}^2 + x^2}$

$$N_{xy} = t = \frac{p_n}{2k}\sqrt{1 + k^2x^2 + k^2y^2}$$

T and C are obtained as before.

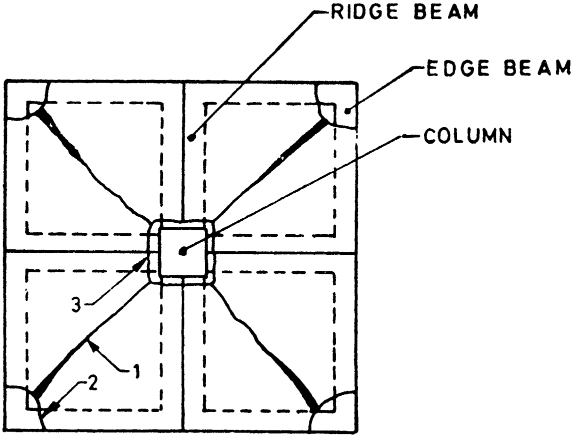
A-2.2 Rigorous and simplified expressions for the ultimate strength P_u (column load at failure) of square hypar footings under vertical soil pressure for both 'ridge' and 'diagonal' failures are given in **A-2.2.1** and **A-2.2.2** (*see* Fig. 22).



1. Principal ridge cracking in shell
2. Yielding section of edge beam
3. Plastic hinge at column face

22A A Ridge Failure

FIG. 22 FAILURE MECHANISMS OF HYPAR FOOTING — *Contd.*



1. Principal diagonal cracking in shell
2. Corner yielding
3. Plastic hinge at column face

22B Diagonal Failure

FIG. 22 FAILURE MECHANISMS OF HYPAR FOOTING

A-2.2.1 *Diagonal Failure*

$$P_u = 12\ Nf\left[\left\{\frac{a}{f}+\frac{1}{2}\left(\frac{a}{f}\right)^3\right\}\left\{\log_e\left(\frac{f}{a}\right)+\sqrt{1+\left(\frac{f}{a}\right)^2}\right\}\right.$$

$$\left.-\frac{1}{2}\left(\frac{a}{f}\right)^3\left\{\frac{f}{a}\times\sqrt{1+\left(\frac{f}{a}\right)^2}\right\}\right]+12\ N_b\,\frac{f}{a}+6\,\frac{M_r}{a}$$

where

N = the ultimate tensile capacity of the shell section per unit width,

N_b = ultimate tensile capacity of the edge beam, and

M_r = ultimate moment capacity of the ridge section.

A simplified form of the above expression which is sufficient for all practical purposes is:

$$P_u = 8\ Nf + 12\ N_b\left(\frac{f}{a}\right)+6\frac{M_r}{a}$$

A-2.2.2 *Ridge Failure* — The corresponding simplified expression for ultimate strength by ridge failure is:

$$P_u = 4\,Nf + 8\,N_b\left(\frac{f}{a}\right) + \frac{8}{\sqrt{2}}\,\frac{M'_r}{a}$$

where M'_r is the ultimate moment capacity of the failing ridge section.

APPENDIX B

(*Clause* 5.10.2)

DETAILING OF REINFORCEMENT AT CRITICAL SECTIONS OF THE HYPERBOLIC PARABOLOIDAL FOOTING TO ENSURE ITS FULL ULTIMATE STRENGTH

B-1. DETAILING

B-1.0 The critical sections of the hypar footing shown in Fig. 23 shall be detailed as given in **B-1.1** to **B-1.3** which will substantially ensure the development of its full ultimate strength.

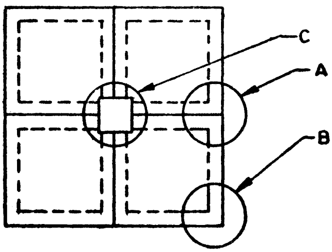


FIG. 23 CRITICAL SECTIONS OF HYPAR FOOTING

B-1.1 Centres of Edge Beams — In the interest of preventing a ridge failure, and ensuring ultimate strength by diagonal failure, the ridge steel may be continued into the edge beams, bending in opposite directions and properly anchored with hooks, as shown in Fig. 24A. The total percentage of steel in the central section of the edge beam, including such steel, shall not exceed 5 percent.

B-1.2 Corners of Edge Beams — To realise the full reserve strength from the edge beam in diagonal failure, the corners may be strengthened by extra diagonal steel properly anchored into fillets as shown in Fig. 24B.

B-1.3 Column Base-Ridge Joint — Even though the chances of failure of column by punching shear are remote on account of the transmission of column load to the ridge beams essentially in direct compression, as an extra measure of precaution against column shear, fillets may be provided at the column base-ridge joint, as shown in Fig. 24C, particularly where triangular ribs alone are provided without the projecting ridge beams.

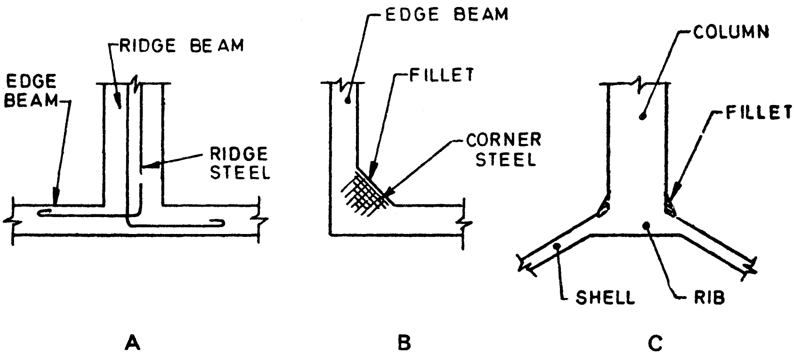


FIG. 24 EXTRA PROVISIONS AT CRITICAL SECTIONS (ORIGINAL REINFORCEMENT NOT SHOWN)

APPENDIX C

(*Clause* 6.4.1)

REMOTE TECHNIQUE FOR INFILLING PRECAST SHELL FOOTINGS

This technique is called 'Centrifugal Blast Compaction' and is effected by means of a centrifugal vane rotor, consisting of a rotating spindle carrying falling blades, designed as a simple attachment to an ordinary needle vibrator used for compacting concrete.

In this technique of compaction, after pouring a batch of dry sand the rotor is inserted into the hollow space through the hole in the column base (*see* Fig. 25). When the motor is, switched on, the vanes open out automatically due to centrifugal action and start rotating at high speeds. This high speed rotation of the vanes creates a heavy blast in the hollow space, under the influence of which, the sand particles become quickly air-borne and start moving radially outwards with high velocities. These particles collide against the inner surfaces of the footing, collapse and settle down to positions of maximum density due to the blast. As this process continues, the entire space gets progressively filled up from the periphery inwards. The work can be stopped on reaching the central portion which is directly accessible for manual compaction through the hole. Density indices (relative density) of the order of 80 to 90 percent can be obtained by this technique of compaction.

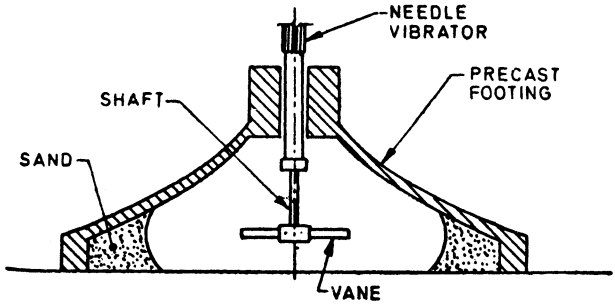


FIG. 25 CORE PREPARATION BY CENTRIFUGAL BLAST COMPACTION

(*Continued from page* 2)

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Amendments Issued Since Publication

Amend No.	Date of Issue
Amd. No. 1	March 1982

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