# DESIGNING A CELLULAR RAFT FOUNDATION USING THE RIGID METHOD

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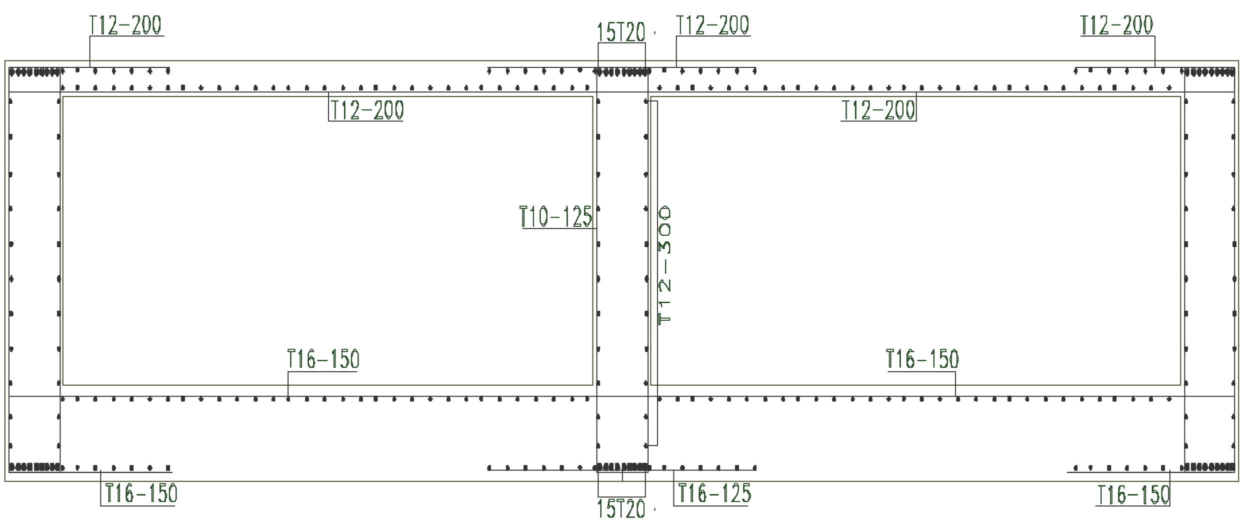


## Summary

*This article presents the design of cellular raft foundation, a spread foundation arrangement with two way interlocking ground beams with a ground bearing slab in contact with the soil and a suspended slab at the top surface. They're typically useful where the foundation is to support very heavy concentrated loads on relatively weak soil or where the bearing soil is susceptible to subsidence /seismic events, where deep foundations have been considered not to be an option.*

## 1.0 Introduction

Raft foundations belongs to the family of spread foundations – foundations that are cited on a shallow bearing stratum. They are necessitated by poor underlying stratum, or where the required depth to a suitable bearing stratum is excessive, where the load carrying capacity of the underlying soil strata deteriorates with depth or even where the loads from a superstructure is enormous relative to the capacity of the soil. Rafts are employed to spread the applied loads over a large base area, at least the building footprint, thereby reducing the contact bearing stress to acceptable limits.

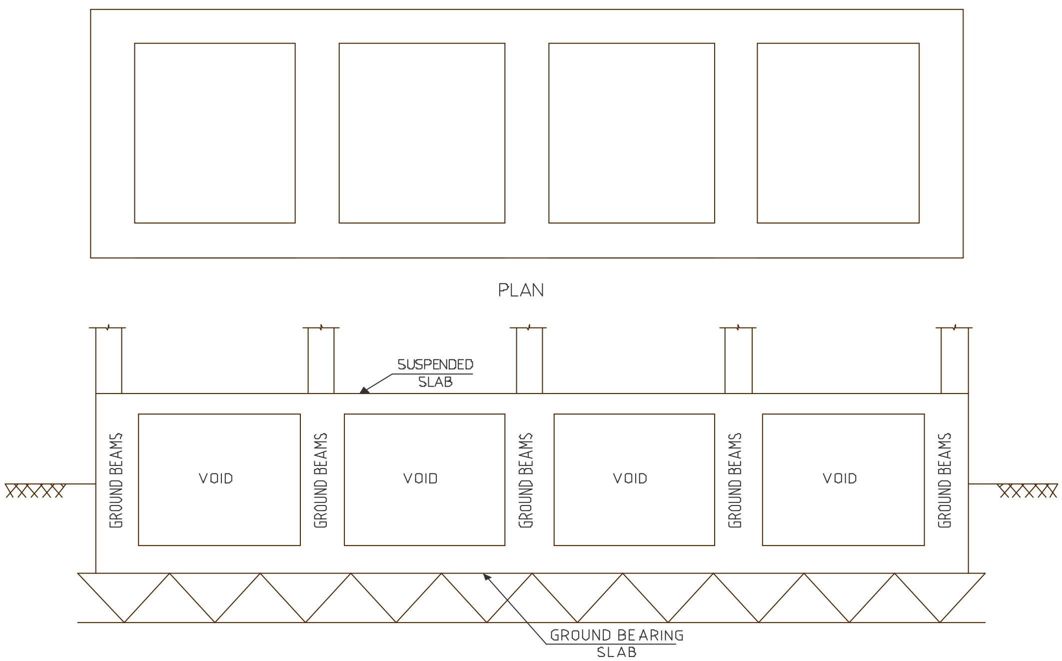


*Figure 1: Cellular Raft*

There are several types of raft foundation, this has been fully appraised in a previous post, (See: Foundation Types: Selection Criteria). However, the focus of this post is on the cellular raft foundation (*Figure 1*) which offers extra advantage when compared to the slab rafts or beam strip rafts. First, they are very suitable where very heavy concentrated forces are to be sustained on relatively weaker soils. Secondly, the removal of the overburden as a result of the hole created by the cellular invariably leads to an increase in bearing capacity.

Notwithstanding, the cellular spaces created also offers the advantage for use as living spaces, storage and for service installations.

A typical cellular raft foundation consists of a series of deep reinforced concrete beams connecting a suspended slab with the bearing slab. The suspended slab can be equated with the ground floor slab in a typical structure, but in this case completely suspended from the ground, while the bearing slab below the suspended slab makes the contact with ground.

## Preliminary Sizing & Considerations

Having selected a cellular raft as a viable structural solution, at the preliminary design stage, the designer of a cellular raft must decide on the depth required. This depends on the depth of overburden required to be removed and the required flexural moment capacity of the cellular form. It is very common for the raft depth to be dictated by the estimated flexural moment to be induced, with the reduced overburden load being a bonus. For example, a building with span restrictions in any directions could lead to large shear forces and flexural moments thus, requiring very deep beams.

The design of a raft foundations can be sometimes complicated especially for beam strip rafts or cellular rafts. The designer must make a judicious choice between the flexible approach or the conventional rigid base method.

In the flexible approach, the raft is flexible relative to the supporting soil, the contact pressure is anything but uniform and the deflection of the raft slab will vary with location. This can be idealized using the beam on elastic foundation theory. The rigid method on the flip side assumes that the raft is stiff relative to the soil, hence the raft is idealized as fully rigid member without any consideration of the elastic properties of the soil and its interaction with the structure. In theory, either of the flexible or rigid approach can be used to design a raft, however, in reality no raft is wholly flexible nor is it totally rigid, these are merely reasonable assumption that lies completely within the remit of the designer.

The flexible approach would yield lower design values than the rigid method, but is only suitable with finite element powered software packages, while the rigid would otherwise give higher design values and it's practically suitable for hand calculations.

## Design Principle

As stated in the preceding section, the conventional rigid approach treats the raft as a member that is rigid relative to the soil without recurse to the elastic properties of the soil. The designer must start with estimating the design column loads and relating this to the overall plan of the building and ground pressures. The calculation for the ground pressure will be based on the centre of gravity of the loads while considering the relative stiffness of the raft itself.

The entire raft is divided into an idealized large beam member in both directions, and the row of column loads normal to the direction of the beam is summed up. With this column load applied on the idealized beam member, the eccentricities of the resultant loads from the centroid of the raft can be found and used to determine the ground pressures at the corners of the building (Figure 2). Assume N is the total axial action; A is the area covered by the raft; $e_y$ and $e_z$ are the eccentricities; My and $M_z$ are the induced moment in both direction due to eccentricities and $Z_y$ and $Z_z$ are the section modulus assuming a symmetrical plan. The ground pressure in each corner of the building can be estimated from the equation:

$$\sigma = \frac{N}{A} \pm \frac{M_y}{Z_y} \pm \frac{M_z}{Z_z}$$

Where: $M_y = N \times e_y$ & $M_z = N \times e_z$

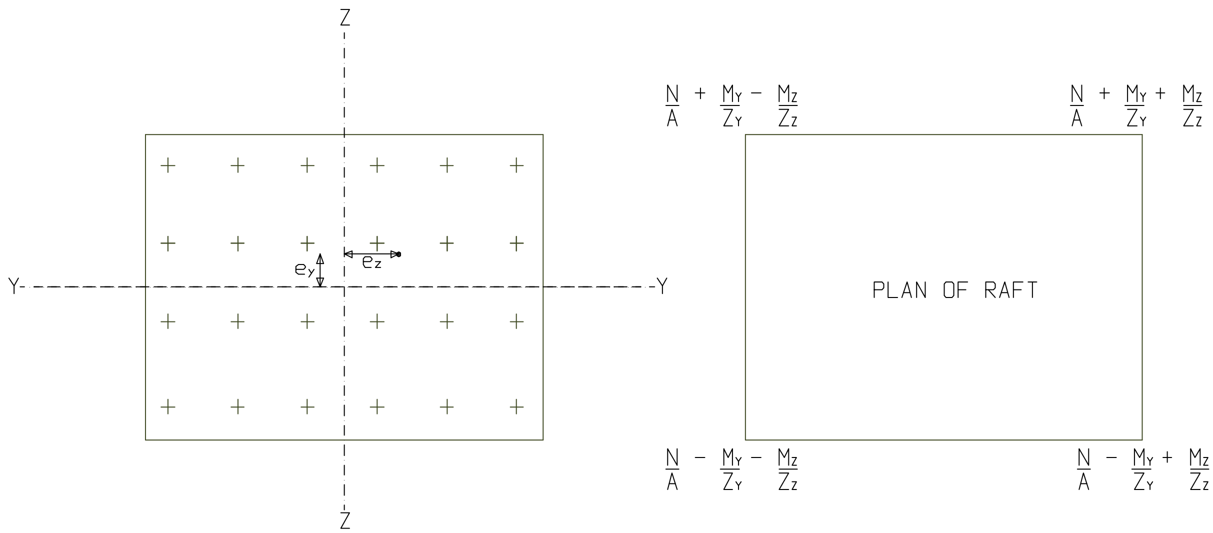


*Figure 2: Point of application of resultant & Corners Pressures in a stiff raft*

Again, these theoretical pressures do not reflect the reality and would not necessarily be achieved on site. A difficulty arises when the designer tries to assess the actual ground pressures. These pressures would depend on the sub-strata the flexibility of the raft and the loads occurring at any given time at which these pressures are being determined.

None of these can be estimated accurately and it's not necessary to do so. The true design of a raft is somewhere between the flexibility and rigid method.

## Element Design

The cellular rafts technically consist of three elements, the upper slab the lower slab and the adjoining beams. The upper slab is designed as a purely suspended slab subjected to the design actions at ground floor level. The load on the ground floor is then added to the loads coming from other parts of superstructure and taken down to the slab below to determine the contact pressure at ultimate limit state. This contact pressure is used to design the lower slab by considering it as an inverted suspended slab. The actions applied on the lower slab is then distributed amongst beams and used for the beam design.

## Worked Example

The plan shown in figure 3 is the column loads at the base of 8 a storey concrete frame which is to be founded on a ground where future mining activity is to be anticipated. In other to deal with the future likely subsidence wave, a cellular raft has been selected as a very viable solution considering the geotechnical and structural factors. The soil investigation report has indicated that a net allowable bearing pressure of $p_a$ = 75kN/m² is required to keep differential settlement in check. Carry out sufficient calculation to establish the size of the foundation and reinforcement required in the elements

*Presumed bearing resistance =75kN/m²*

*Soil Unit weight = 20 kN/m³*

*Imposed load (ground floor) = 5kN/m²*

*Concrete C30/35; $F_y$ = 460Mpa*

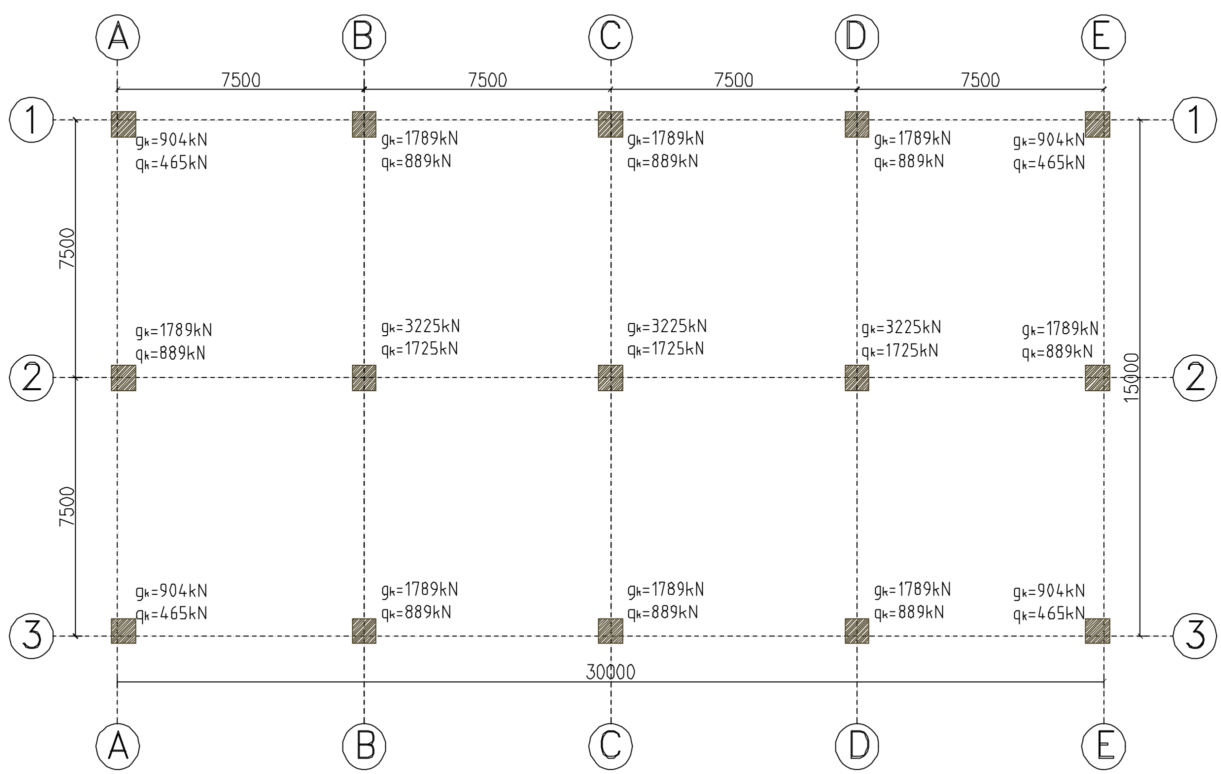


*Figure 3: Point of application of columns at ground level*

By inspection, the load is symmetrical about both axes of the building, hence the point of application of the resultant will coincide with centroid of the base and thus, there are no eccentricities and the bearing pressure at the corners and anywhere in the raft will be the same.

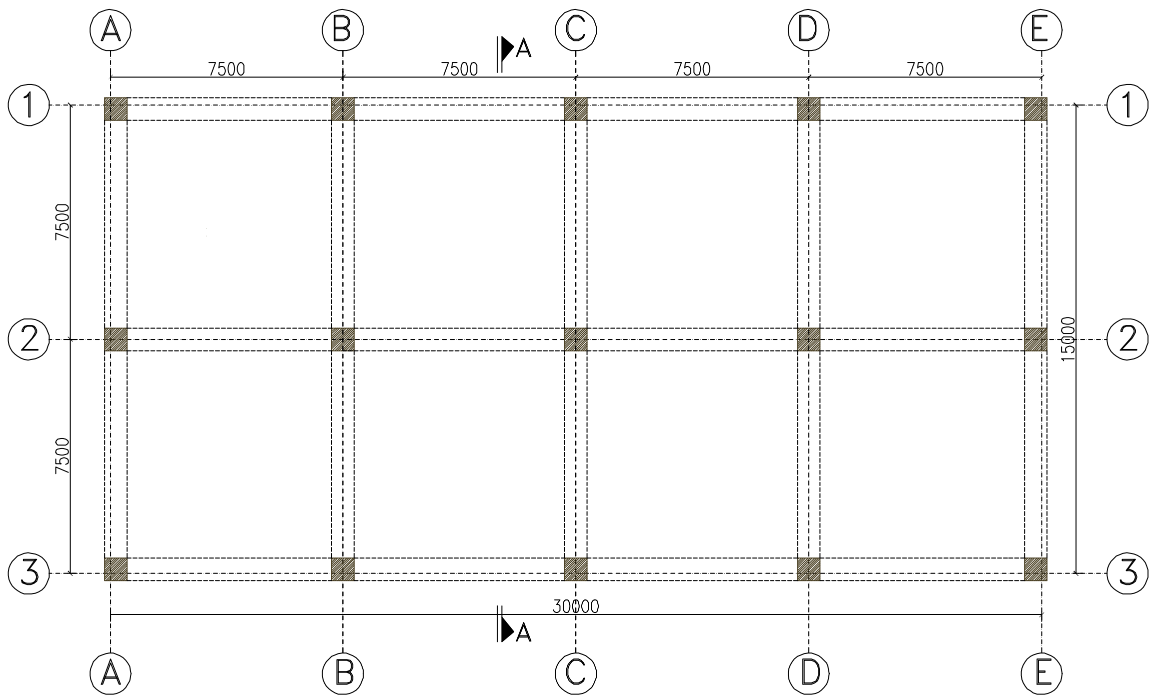


*Figure 4: Plan of Cellular Foundation*

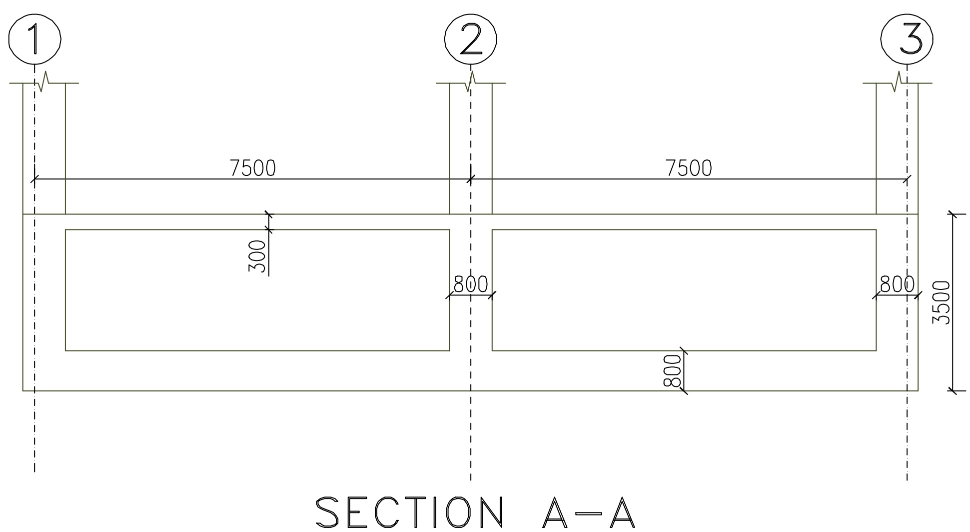


*Figure 5: Section through Cellular Raft*

**A) Serviceability Limit State**

**Actions**

a) Superstructure Actions

$\text{Permanent Actions; G} = 4(904) + 8(1789) + 3(3225) = 27{,}603\text{kN}$

$\text{Variable Actions; Q} = 4(465) + 8(889) + 3(1725) = 14{,}147\text{kN}$

$\text{Superstructure loads} = 27{,}603 + 14{,}147 = 41{,}750\text{kN}$

b) Foundation loads

Permanent actions:

$\text{Upper slab} = 0.3\text{m} \times 15\text{m} \times 30\text{m} \times 25 = 3375\text{kN}$

$\text{Lower slab} = 0.8\text{m} \times 15\text{m} \times 30\text{m} \times 25 = 9000\text{kN}$

$\text{Beam webs} = 0.8\text{m} \times 2.4\text{m} \times 165\text{m} \times 25 = 7920\text{kN}$

$\text{Permanent Actions; G} = 3375 + 9000 + 7920 = 20{,}295\text{kN}$

Variable Actions

$\text{Imposed loading(ground floor slab)} = 15\text{m} \times 30m \times 5 = 2250kN$

$\text{Foundation loads} = 20{,}295 + 2250 = 22{,}545\text{kN}$

$$\text{Total loads @ SLS} = 41{,}750 + 22{,}545 = \mathbf{64{,}295kN}$$

**Bearing Pressure Check**

$$\sigma = \frac{N}{A} \pm \frac{M_y}{Z_y} \pm \frac{M_z}{Z_z} = \frac{64295}{(15\text{m} \times 30\text{m})} = \mathbf{142.8kN/m^2}$$

If the net pressure at the formation level of 3.5m is, $P_a = 75\text{kN/m}^2$, the excess overburden pressure would be given as

$$P_o = 20 \times 3.5 = 70kN/m^2$$

Therefore, the total allowable pressure at the 3.5m depth is given as:

$$P_T = P_a + P_o = 75 + 70 = \mathbf{145kN/m^2}$$

$$Since\ \ (\sigma = 142.85kN/m^2) < (P_T = 145kN/m^2)\ \ O.k$$

**B) Ultimate Limit State**

**Actions**

Superstructure loads $= (1.35 \times 27{,}603) + (1.5 \times 14{,}147) = 58{,}484.6\text{kN}$

Foundation loads $= 1.35(3375 + 7920) + (1.5 \times 2250) = 18{,}623.25\text{kN}$

$$\text{Total loads @ ULS} = 58{,}484.6 + 18{,}623.25 = \mathbf{77{,}107.85kN}$$

**Bearing Pressure**

$$Bearing\ pressure\ @\ ULS, \qquad \sigma_{max} = \frac{\text{N}}{\text{A}} = \frac{77{,}107.85}{(15\text{m} \times 30\text{m})} = \mathbf{171.35kN/m^2}$$

**Designing the Bottom Slab**

Having determined the pressure at the ultimate limit state, the bottom slab is designed as a two-way spanning slab for this pressure (**171.35kN/m²**) in accordance with code provisions. It should be noted that in this case, the load is acting upwards, hence the tensile reinforcement required in the span would be positioned at the top while those required at the supports are fixed at the bottom.

$$\frac{L_y}{L_x} = \frac{7500}{7500} = 1.0$$

Coefficients from table for two ways slabs with two adjacent sides discontinuous is the most critical hence would be used to size the quantity of reinforcement required in the slab

$Coefficients = -0.047\ \&\ 0.036$

**Flexural Design**

<u>Negative Moment at Continuous Supports</u>

$$M = -0.047\sigma_{max} l_x^2 = -0.047 \times 171.35 \times 7.5^2 = -453.0kN.m/m$$

Assuming cover to reinforcement of 50mm, 16mm bars

$d = h - \left(c_{nom} + links + \frac{\emptyset}{2}\right) = 800 - \left(50 + \frac{16}{2}\right) = 742mm;\ b = 1000mm$

$$k = \frac{M_{Ed}}{bd^2 f_{ck}} = \frac{453 \times 10^6}{1000 \times 742^2 \times 30} = 0.027$$

$z = d\left[0.5 + \sqrt{0.25 - 0.882k}\right] \leq 0.95d$

$= d\left[0.5 + \sqrt{0.25 - 0.882(0.027)}\right] \leq 0.95d$

$= 0.95d = 0.95 \times 742 = 704.9mm$

$$A_s = \frac{M_{Ed}}{0.87 f_{yk} z} = \frac{453 \times 10^6}{0.87 \times 460 \times 704.9} = 1605.8mm^2/m$$

Try T16mm bars @ 125mm Centres Bottom ($A_{s, prov} = 1608mm^2$) Both ways

Positive Moment in Spans

$$M = 0.036\sigma_{max} l_x^2 = 0.036 \times 171.35 \times 7.5^2 = 347kN.m/m$$

Assuming cover to reinforcement of 50mm, 16mm bars

$d = h - \left(c_{nom} + links + \frac{\emptyset}{2}\right) = 800 - \left(50 + \frac{16}{2}\right) = 742mm;\ b = 1000mm$

$$k = \frac{M_{Ed}}{bd^2 f_{ck}} = \frac{347 \times 10^6}{1000 \times 742^2 \times 30} = 0.021$$

$z = d\left[0.5 + \sqrt{0.25 - 0.882k}\right] \leq 0.95d$

$= d\left[0.5 + \sqrt{0.25 - 0.882(0.021)}\right] \leq 0.95d$

$= 0.95d = 0.95 \times 742 = 704.9mm$

$$A_s = \frac{M_{Ed}}{0.87 f_{yk} z} = \frac{347 \times 10^6}{0.87 \times 460 \times 704.9} = 1230.06mm^2/m$$

Try T16mm bars @ 150mm Centres Top ($A_{s, prov} = 1340mm^2$)

**Detailing Checks.**

The minimum area of steel required in panel:

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_w d \geq 0.0013bd$$

$f_{ctm} = 0.30f_{ck}^{\frac{2}{3}} = 0.3 \times 30^{\frac{2}{3}} = 2.9\text{Mpa}$

$A_{s,min} = 0.26 \times \frac{2.9}{460} \times 1000 \times 742 \geq 0.0013 \times 1000 \times 742$

$= 1216.23\text{mm}^2$. By observation it is not critical anywhere in slab. Hence adopt all steel bars.

**Designing the Top Slab**

The top slab is designed as any other suspended slab in the superstructure– designed as two-way spanning slab for the applied permanent and variable actions.

**Actions**

Permanent Actions**:**

i. Self weight of slab $= 0.3 \times 25 = 7.5\text{kN/m}^2$

Total Permanent Actions $= g_k = 7.5kN/m^2$

Variable Actions

i. Floor Imposed Loading $= q_k = 5.0kN/m^2$

*Design Value of Actions***:**

Design Actions $n_s = 1.35g_k + 1.5q_k$ =(1.35 × 7.5) + (1.5 × 5) = $\mathbf{17.625kNm^2/m}$

**Flexural Design**

Negative Moment at Continuous Supports

$M = -0.047\sigma_{max}l_x^2 = -0.047 \times 17.625 \times 7.5^2 = -46.6kN.m/m$

Assuming cover to reinforcement of 50mm, 16mm bars

$d = h - \left(c_{nom} + \text{links} + \frac{\emptyset}{2}\right) = 300 - \left(25 + \frac{12}{2}\right) = 269\text{mm};\ b = 1000\text{mm}$

$k = \frac{M_{Ed}}{bd^2f_{ck}} = \frac{46.6 \times 10^6}{1000 \times 269^2 \times 30} = 0.021$

$z = d\left[0.5 + \sqrt{0.25 - 0.882k}\right] \leq 0.95d$

$=d[0.5 + \sqrt{0.25 - 0.882(0.027)}] \leq 0.95d$

$=0.95d = 0.95 \times 269 = 255.6mm$

$$A_s = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{46.6 \times 10^6}{0.87 \times 460 \times 255.6} = 455.6mm^2/m$$

Try T12mm bars @ 200mm Centres Top ($A_{s,\,prov} = 565mm^2$) Both ways

Positive Moment in Spans

$$M = 0.036\sigma_{max}l_x^2 = 0.036 \times 17.625 \times 7.5^2 = 35.7kN.m/m$$

Assuming cover to reinforcement of 50mm, 16mm bars

$$d = h - \left(c_{nom} + links + \frac{\emptyset}{2}\right) = 300 - \left(25 + \frac{12}{2}\right) = 269mm;\ b = 1000mm$$

$$k = \frac{M_{Ed}}{bd^2f_{ck}} = \frac{35.7 \times 10^6}{1000 \times 269^2 \times 30} = 0.016$$

$$z = d[0.5 + \sqrt{0.25 - 0.882k}] \leq 0.95d$$

$=d[0.5 + \sqrt{0.25 - 0.882(0.016)}] \leq 0.95d$

$=0.95d = 0.95 \times 269 = 255.6mm$

$$A_s = \frac{M_{Ed}}{0.87f_{yk}z} = \frac{35.7 \times 10^6}{0.87 \times 460 \times 255.6} = 349mm^2/m$$

Try T12mm bars @ 200mm Centres Bottom ($A_{s,\,prov} = 565mm^2$)

**Deflection Verification**

Deflection verification can be carried using either of the two alternative methods provided in section 7.4 of Eurocode 2 (Part 1). By inspection, deflection is not critical here, the slab has been sized conservatively.

**Detailing Checks.**

The minimum area of steel required in panel:

$$A_{s,min} = 0.26\frac{f_{ctm}}{f_{yk}}b_w d \geq 0.0013bd$$

$f_{ctm} = 0.30f_{ck}^{\frac{2}{3}} = 0.3 \times 30^{\frac{2}{3}} = 2.9\text{Mpa}$

$A_{s,min} = 0.26 \times \frac{2.9}{460} \times 1000 \times 269 \geq 0.0013 \times 1000 \times 269$

$= 440.9\text{mm}^2$. By observation it is not critical anywhere in slab. Hence adopt all steel bars.

**Designing the Beam**

The beams are deep concrete beams, designed for the actions transferred from the bottom slab to the beams. They are designed as I concrete sections. Under normal conditions, the depth of beam involved would mostly be conservative relative to the span, hence the corresponding reinforcement would be light.

**Actions on Beam**

$$\text{Equivalent Udl on each span } w = 2 \cdot \frac{\sigma_{max} l}{3} = 2 \times 171.35 \times \frac{7.5}{3} = \mathbf{856.8kN/m}$$

**Analysis of Beam**

Since the geometry of the beam is equal and the actions on the beams are uniform, hence simple coefficient can be used to obtain the flexural moments and shear forces in the beam.

**Flexural Design**

*Interior Supports*

$M_{Ed} = 0.11wl^2 = 0.11 \times 856.8 \times 7.5^2 = 5302kN.m$

Assuming cover to reinforcement of 50mm, 16mm tensile bars, 16mm compression bars & 8mm links

$d' = \left(c_{nom} + \text{links} + \frac{\emptyset}{2}\right) = 50 + 10 + \frac{20}{2} = 70mm$

$b = b_{eff} = b_w + b_{eff,1} + b_{eff,2} \leq b$

$b_{eff,1} = b_{eff,2} = 0.2b + 0.1l_o \leq 0.2l_o$

$$b = \left(\frac{7500 - 400 - 400}{2}\right) = 3350mm$$

$$l_o = 0.85l = 0.85 \times 7500 = 6375mm$$

$$\mathrm{b_{eff,1}} = \mathrm{b_{eff,2}} = (0.2 \times 3350) + (0.1 \times 6375) \leq (0.2 \times 6375) = 1275mm$$

$$\mathrm{b_{eff}} = 800 + 1275 + 1275 = 3350\mathrm{mm} \leq 3350\mathrm{mm}$$

$$\mathrm{d} = \mathrm{h} - \left(\mathrm{c_{nom}} + \mathrm{links} + \frac{\emptyset}{2}\right) = 3500 - \left(50 + 10 + \frac{20}{2}\right) = 3430\mathrm{mm};\ \mathrm{b} = 800\mathrm{mm}$$

$$\mathrm{k} = \frac{\mathrm{M_{Ed}}}{\mathrm{bd^2f_{ck}}} = \frac{5302 \times 10^6}{3350 \times 3430^2 \times 30} = 0.0045 < 0.168 \text{ (Section is s reinforced)}$$

$$\mathrm{z} = \mathrm{d}\left[0.5 + \sqrt{0.25 - 0.882\mathrm{k}}\right] \leq 0.95\mathrm{d}$$

$$= \mathrm{d}\left[0.5 + \sqrt{0.25 - 0.882(0.0045)}\right] \leq 0.95\mathrm{d}$$

$$= 0.95\mathrm{d} = 0.95 \times 3430 = 3258.5\mathrm{mm}$$

$$\mathrm{A_s} = \frac{\mathrm{M_{Ed}}}{0.87\mathrm{f_{yk}z}} = \frac{5302 \times 10^6}{0.87 \times 460 \times 3258.5} = 4065.8\mathrm{mm^2}$$

Try 14T20mm bars Bottom ($A_{s,\ prov} = 4396mm^2$)

*Spans*

$$M_{Ed} = 0.09wl^2 = 0.09 \times 856.8 \times 7.5^2 = 4338kN.m$$

Assuming cover to reinforcement of 50mm, 16mm tensile bars, 16mm compression bars & 8mm links

$$\mathrm{d'} = \left(\mathrm{c_{nom}} + \mathrm{links} + \frac{\emptyset}{2}\right) = 50 + 10 + \frac{20}{2} = 70mm$$

$$\mathrm{b} = \mathrm{b_{eff}} = \mathrm{b_w} + \mathrm{b_{eff,1}} + \mathrm{b_{eff,2}} \leq \mathrm{b}$$

$$\mathrm{b_{eff,1}} = \mathrm{b_{eff,2}} = 0.2\mathrm{b} + 0.1l_o \leq 0.2l_o$$

$$b = \left(\frac{7500 - 400 - 400}{2}\right) = 3350mm$$

$$l_o = 0.85l = 0.85 \times 7500 = 6375mm$$

$$\mathrm{b_{eff,1}} = \mathrm{b_{eff,2}} = (0.2 \times 3350) + (0.1 \times 6375) \leq (0.2 \times 6375) = 1275mm$$

$$\mathrm{b_{eff}} = 800 + 1275 + 1275 = 3350\mathrm{mm} \leq 3350\mathrm{mm}$$

$$\mathrm{d} = \mathrm{h} - \left(\mathrm{c_{nom}} + \mathrm{links} + \frac{\emptyset}{2}\right) = 3500 - \left(50 + 10 + \frac{20}{2}\right) = 3430\mathrm{mm};\ \mathrm{b} = 800\mathrm{mm}$$

$$k = \frac{M_{Ed}}{bd^2 f_{ck}} = \frac{4338 \times 10^6}{3350 \times 3430^2 \times 30} = 0.0036 < 0.168 \text{ (Section is s reinforced)}$$

$$z = d\left[0.5 + \sqrt{0.25 - 0.882k}\right] \leq 0.95d$$

$$=d\left[0.5 + \sqrt{0.25 - 0.882(0.0036)}\right] \leq 0.95d$$

$$=0.95d = 0.95 \times 3430 = 3258.5mm$$

$$A_s = \frac{M_{Ed}}{0.87 f_{yk} z} = \frac{4338 \times 10^6}{0.87 \times 460 \times 3258.5} = 3326.6mm^2$$

<u>Try 12T20mm bars Bottom ($A_{s,\ prov}$ = 3768mm$^2$)</u>

<u>Detailing Checks</u>

*Minimum Area of Steel*

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_w d \geq 0.0013bd$$

$$f_{ctm} = 0.30 f_{ck}^{\frac{2}{3}} = 0.3 \times 30^{\frac{2}{3}} = 2.9Mpa$$

$$A_{s,min} = 0.26 \times \frac{2.9}{460} \times 800 \times 3432 \geq 0.0013 \times 300 \times 702.5$$

$= 4500.4mm^2$. By observation, this is critical everywhere, hence minimum area of steel controls the design

<u>Provide 15T20mm bars Top & Bottom ($A_{s,\ prov}$ = 4710mm$^2$)</u>

**Shear Design**

$$V_{Ed} = 0.6wl = 0.6 \times 856.8 \times 7.5 = 3855.6kN$$

$$V_{Rd,c} = \left(\frac{0.18}{\gamma_c}\right) k(100\rho_1 f_{ck})^{\frac{1}{3}} b_w d \ \geq \ 0.035 k^{\frac{3}{2}} \sqrt{f_{ck}} b_w d$$

$$k = 1 + \sqrt{\frac{200}{3430}} = 1 + \sqrt{\frac{200}{3430}} = 1.24 < 2$$

$$A_s = 4710mm^2$$

$$b_w = 800mm$$

$$\rho_1 = \frac{A_s}{b_w d} = \frac{4710}{800 \times 3430} = 0.0017$$

$$V_{Rd,c} = \left(\frac{0.18}{1.5}\right) \times 1.24 \times (100 \times 0.0017 \times 30)^{\frac{1}{3}} \cdot 800 \times 3432$$

$$\geq 0.035 \times 1.24^{\frac{3}{2}} \times \sqrt{30} \times 800 \times 3432 = 726.8\text{kN}$$

Since $V_{Ed} > V_{Rd,c}$ $(3855.6kN > 726.8kN)$ therefore shear reinforcement is required.

$$\theta = 0.5sin^{-1}\left(\frac{5.56V_{Ed}}{b_w d(1 - \frac{f_{ck}}{250})f_{ck}}\right) = 0.5sin^{-1}\left(\frac{5.56 \times 3855.6 \times 10^3}{800 \times 3430\left(1 - \frac{30}{250}\right)30}\right) = 8.60^{\circ}$$

$\cot\theta = \cot 17.66 = 8.60 > 2.5$ Hence take $\cot\theta = 2.5$

$\frac{A_{sv}}{S_v} \geq \frac{V_{Ed}}{\text{zcot} \quad {}_{ywd}}$ where $z = 0.9d = 0.9 \times 3430 = 3087\text{mm}$

$$\frac{A_{sv}}{S_v} \geq \frac{3855.6 \times 10^3}{3087 \times 2.5 \times 460} = 1.09$$

$\text{max spacing} = 0.75d = 0.75 \times 3430 = 2572.5\text{mm}$

$$\frac{A_{sv,min}}{S_v} = \frac{0.08\sqrt{f_{ck}}b_w}{f_{yk}} = \frac{0.08 \times \sqrt{30} \times 800}{460} = 0.77$$

Use T10 @ 125mm centres (1.26)

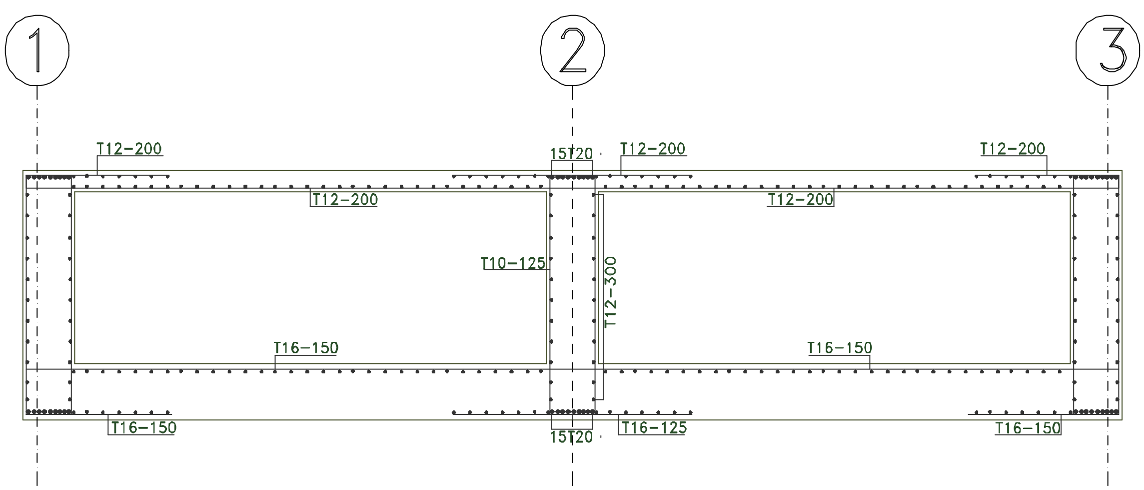


*Figure 6: Section through cellular foundation details*

**Detailing Consideration**

Since the beams are deep, the detailing side bars must be provided to ensure the beam does not fail from lateral buckling. The advice given in Eurocode 2 for the side faces of deep beams may be followed. The UK National Annex recommends that 0.2% is provided in each face. The distance between bars should not exceed the lesser of twice the beam depth or 300 mm.