Steel fibre reinforced concrete ground bearing slabs

Introduction

The design of ground bearing reinforced concrete floors has changed significantly in recent times; the major development being the increasingly common adoption of steel fibre reinforcement in concrete floor slabs. This was described in Technical Guidance Note No. 30 (Level 1) which introduced ground bearing slabs with respect to their construction and detailing.

This Technical Guidance Note describes how steel fibre reinforced concrete ground bearing slabs are designed. This is a relatively recent innovation that continues to evolve. As such, this note aims to motivate the design and development of steel fibre reinforced ground bearing slabs, based on the most up to date information available at the time of writing.

ICON LEGEND

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Design principles

Reinforced concrete ground bearing slabs

Ground bearing slabs can be found in a multitude of structures, be they commercial offices, residential, schools or hospitals. They are, however, most commonly found in large warehouses and out-of-town shopping complexes. More often than not, these buildings contain tall storage racks that subject the slab to concentrated point loads. Ground bearing slabs are laid directly onto a sub-base material, which deflects and heaves when placed under load. This is because ground bearing slabs are flexural elements that share load with the founding material they are sat upon. Therefore, they cannot be regarded as a raft because they are not stiff enough, and soil structure interaction is of paramount concern.

Noting the commonality of the use of ground bearing slabs in warehouses and similar buildings, this note focuses on the design of slabs that are subject to such loading conditions. The concentrated point loads from storage racks can induce punching shear failure in the slab, as well as the initiation of cracks within the slab as it deforms. Plant vehicles such as forklifts place dynamic point loads within discrete rectilinear shaped areas where they traverse.

Founding material

The key property of the founding material relevant to ground bearing slabs is the modulus of subgrade reaction k. This is typically determined using the results of a plate bearing test carried out on the soil, in accordance with BS EN 1997-2. Where such a test cannot be carried out it is possible to estimate the value of k from other known soil properties. This is not recommended, however, as it can produce inaccurate results. For similar reasons, k should not be derived using the Californian Bearing Ratio (CBR) of the soil.

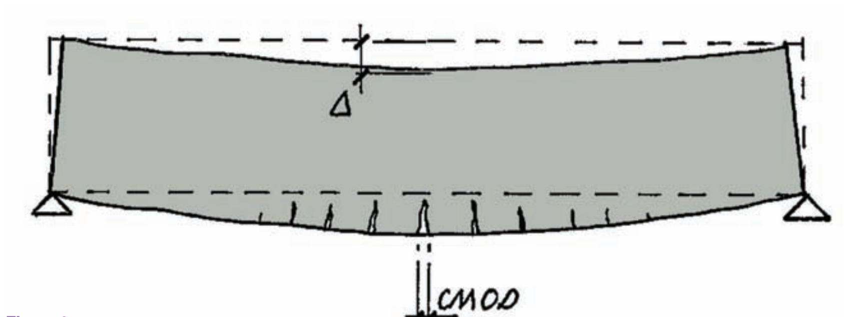


Figure 1
Crack mouth opening displacement in steel fibre reinforced concrete

The value of k typically ranges from 0.03 to 0.08N/mm²/mm. Advice on its magnitude should be sought from a suitably qualified and experienced geotechnical engineer. Further details on this can be sourced from Chapter 5 of TR34: *Concrete industrial ground floors* (4th ed.)

In addition to determining the value of k, any ground investigation should provide all other relevant data to aid the design of the ground bearing slab. In particular, what long term settlement could occur when the soil is subjected to imposed area loads, line loads and concentrated loads through the ground bearing slab?

On top of the founding material is a sub-base. This is taken to be a minimum of 150mm thick and is made from well compacted Type 1 and 2 granular materials as defined in the *Specification for Highway Works, Series 800, Road Pavements*, published by the UK's Highways Agency. For more information on the make-up of supporting material to ground bearing slabs, see Technical Guidance Note No. 30 (Level 1).

Fabric vs steel fibre reinforcement

In recent years, the use of steel fibre reinforcement has been increasingly employed as a method of providing flexural tension capacity to ground bearing slabs.

It reduces the amount of workmanship needed to install the slab as it only requires that metal steel fibres are placed into the concrete as it is mixed. This is in comparison to the more labour intensive installation of fabric reinforcement.

Structural properties of steel fibre reinforced concrete

The inclusion of steel fibre reinforcement within concrete provides an enhanced tension capacity throughout its cross-section. The effectiveness of the steel fibres to provide this tension capacity is affected by their form and spread throughout the mix of concrete. In recognition of this, the method of determining the structural properties of steel fibre reinforced concrete takes into account these variables, which are dependent on the manufacture of the steel fibres and how they are mixed into the concrete.

In order for steel fibres to be effective there must be a sufficient inclusion within the concrete mix so as to achieve a residual flexural strength f_R over and above its unreinforced state. In a similar way to how traditional reinforcement interacts with the concrete that surrounds it, the ultimate strength model assumes that the concrete cracks before the steel fibres engage as a reinforcing element. It therefore follows that the concrete's flexural strength is based on the magnitude of the crack mouth opening displacement (CMOD) of the concrete cracks when it deflects under load (Figure 1).

EN 14889 stipulates that manufacturers of steel fibres must specify the quantity needed in the concrete to achieve a flexural strength, which is dependent upon concrete's CMOD value.

Bending moment capacity

Ground bearing concrete slabs have two bending moment capacities. One is based on their reinforced properties M_u and the other on their unreinforced properties M_{un}.

Bending capacity of unreinforced concrete

The bending capacity of an unreinforced concrete slab of 1m width is defined as:

$$M_{un} = f_{ctd,fl}\left(\frac{h^2}{6}\right)$$

where:
h is the overall depth of the slab
$f_{ctd,fl}$ is the flexural strength of unreinforced concrete as defined in Clause 3.1.8 of BS EN 1992-1-1. Here $f_{ctd,fl}$ is defined as:

"A ground bearing slab can be subjected to three primary forms of applied load"

$$f_{ctd,fl} = f_{ctm} \times \left(1.6 - \frac{h}{1000}\right) / \gamma_m \geq \frac{f_{ctm}}{\gamma_m}$$

where:
f_{ctm} is the mean axial tensile strength of concrete that is drawn from Table 3.1 of BS EN 1992-1-1 and partially reproduced in Table 1
γ_m is the material factor of concrete, which is 1.5 as stated in Clause 2.4.2.4 of BS EN 1992-1-1

Bending capacity of steel fibre reinforced concrete

The bending capacity is based on its residual flexural tensile strength. It is only possible to determine this by laboratory testing. This provides the strength of the concrete via a series of experiments carried out on a dozen concrete beams made from steel fibre reinforced concrete that matches the properties of the mix that is to be used on a project. These beams are tested in accordance with BS EN 14651 (the test for notched beams). The effect that steel fibre reinforcement has on concrete is derived using the method described in BS EN 14845. The results of these tests are used to calculate the bending moment capacity per metre width of the steel fibre reinforced concrete f_R using the following formula:

$$f_R = \frac{3F_R l}{2bh_{sp}^2}$$

where:
f_R is the applied load on the beam
l is the span of the beam
b is the width of the beam
h_{sp} is the depth of the beam

This is calculated four times from a batch of three test samples and the mean value of the four results of flexural strength is used to design the concrete element. These four values are called $f_{R,1}$, $f_{R,2}$, $f_{R,3}$ and $f_{R,4}$. There are two critical values taken from these test results that are used to determine the bending capacity of steel fibre reinforced concrete. These are $f_{R,1}$, which can range from 1-6 N/mm², and $f_{R,4}$, which can lie between 0-4 N/mm². More information on this method can be read in Rilem document TC 162-TDF.

From this test data the bending capacity of the reinforced concrete M_u can be calculated. There is a rigorous method that requires a significant amount of iteration but alternatively, a simplified yet conservative method, based on key assumptions about how the concrete behaves when in bending, can be used. These assumptions are:

- A failure of a concrete element occurs when the compressive strain within it reaches 0.0035
- As the concrete element approaches its ultimate bending moment capacity, its maximum tensile and compressive strains are reached. Due to the fact that the compressive strength of the concrete is far greater than the tensile strength of the steel fibres', equilibrium is not achieved. This is because the compressive force in the concrete exceeds the tensile force in the steel fibres as it approaches its ultimate bending moment capacity
- It follows from the second assumption that the neutral axis of the concrete element is a constant multiple of its depth.

From these assumptions Chapter 6.3.5 of TR34 describes how the following equation can be derived to calculate bending capacity of steel fibre reinforced concrete slabs based on 1m width:

$$M_u = \frac{h^2}{\gamma_m}(0.29\sigma_{r4} + 0.16\sigma_{r1})$$

where:
h is the overall depth of the concrete element (mm)
σ_{r4} is the stress for corresponding residual flexural tensile strength 0.45$f_{R,4}$
σ_{r1} is the stress for corresponding residual flexural tensile strength 0.37$f_{R,1}$

Table 1: Mean tensile strength of concrete vs. strength class

Concrete strength class	C25/30	C28/35	C30/37	C32/40	C35/45	C40/50
f_{ctm} (N/mm²)	2.6	2.8	2.9	3.0	3.2	3.5

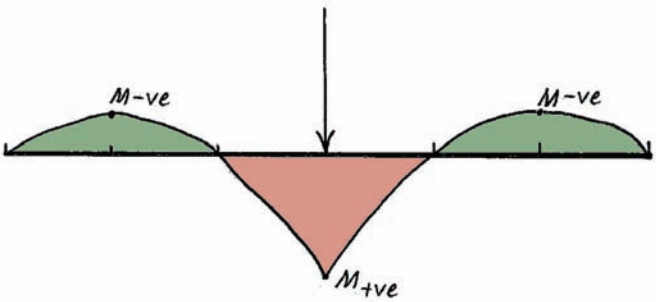


Figure 2
Positive (sagging) and negative (hogging) moments in a ground bearing floor slab due to concentrated point loads from storage racks

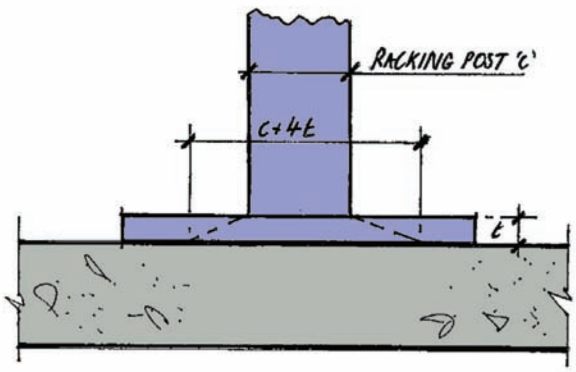


Figure 3
Radius of contact area a of vertical elements

Shear capacity

Steel fibre reinforcement does not provide enhancement to the shear capacity v_f of the concrete. The shear capacity is taken to be 1.5% of its flexural strength capacity. This value is not currently supported by sufficient empirical data and is therefore reduced by 50%, as follows:

$$\nu_f = \frac{0.015\,(f_{R,1}+f_{R,2}+f_{R,3}+f_{R,4})}{2}$$

where:
$f_{R,1-4}$ are the four residual flexural tensile strength values drawn from test data.

Applied actions

A ground bearing slab can be subjected to three primary forms of applied load. These are: concentrated point loads from storage racks and vehicles, line loads from partition walls and imposed area loads. Of greatest concern are the concentrated point loads and line loads as they induce the highest level of bending stress in the slab. Another form of action relates to thermal stresses, which are alleviated by either including joints within the slab or designing the slab to resist the effects of thermally induced movement.

Single concentrated point loads

Concentrated point loads generate both positive and negative bending moments due to the deflected shape of the slab as it is placed under load. The magnitude of the concentrated point load the steel fibre reinforced slab can support, is determined based on the bending strength of the slab. This is calculated from the positive and negative bending moment capacities of the slab (Figure 2).

The positive bending moments M_{+ve} are derived from the ultimate bending moment capacity of steel fibre reinforced concrete M_u and are based on the cracked concrete model. The negative bending moments M_{-ve} are derived from the unreinforced concrete moment bending capacity M_{un}, as it is assumed there is to be a crack-free surface to the slab.

As such, the positive bending moment becomes static in terms of its magnitude, and any increase in applied load leads to moment redistribution and an increase to the negative bending moment. This will progress under increasing load until cracking occurs in the top surface, some distance away from the location of the point load.

The location of the applied loads, relative to the edge of the slab and their proximity to other loads, also has an impact on the applied bending moments. There are two key variables that need to be calculated before any assessment is made on what the effects might be. These variables are the radius of the contact area of the vertical element that is applying the force a, and the radius of relative stiffness of the slab l. The contact area is dependent upon the dimensions of the baseplate and its thickness (Figure 3). From this figure the value of a is calculated as follows, but must not be larger than the baseplate:

$$a_{\max} = \sqrt{\frac{(c+4t)^2}{\pi}}$$

where:
c is the width of the racking post
t is the thickness of the baseplate

When no precise data is known about the baseplate dimensions, it can be reasonably assumed to be the equivalent of a 100mm × 100mm area. For rectangular contact areas, a circle of equivalent area can be assumed in determining a. The spread of load in the baseplate can be taken as 1:2.

The relative stiffness radius is based on the likely crack pattern that develops, as a point load is applied to the slab. It is derived using the following equation:

$$l = \left(\frac{E_{cm}h^3 \times 10^6}{(12(1-\nu^2)k)}\right)^{\frac{1}{4}}$$

where:
E_{cm} is the secant modulus of elasticity for concrete and values are given in Table 3.1 of BS EN 1992-1-1, which is partially reproduced in Table 2
ν is Poisson's ratio for concrete, which is 0.2 according to BS EN 1992-1-1
k is the modulus of subgrade reaction as previously described

Once the spread of the force through the slab is determined, the impact of location can be assessed. Typically, concentrated point loads from vertical elements fall into three categories:

Internal - Where the point load is not less than $a+l$ away from the edge of the slab.
Edge - Where the point load is less than $a+l$ from one edge of the slab but not less than $a+l$ from a corner
Corner - Where the point load is less than $a+l$ from two edges of the slab

Once the location of the point load is determined, the capacity of the slab P_u can

Table 2: Secant modulus of concrete vs. strength class

Concrete strength class	C25/30	C28/35	C30/37	C32/40	C35/45	C40/50
E_{cm} (N/mm²)	31000	32000	33000	33000	34000	35000

be calculated. It is dependent upon the value of $\left(\frac{a}{l}\right)$ ratio. (For the purposes of the following formulae, values of $\left(\frac{a}{l}\right) \approx 0$ to two significant figures e.g. 0.01 can be considered to be 0):

Internal point load:

$$\text{If } \frac{a}{l} = 0 \text{ then } P_{u,0} = 2\pi\left(M_{+ve} + M_{-ve}\right)$$

$$\text{If } \frac{a}{l} \geq 0.2 \text{ then } P_{u,0.2} = 4\pi\frac{\left(M_{+ve} + M_{-ve}\right)}{\left[1 - \frac{a}{3l}\right]}$$

Edge point load:

$$\text{If } \frac{a}{l} = 0 \text{ then } P_{u,0} = \left[\pi\frac{\left(M_{+ve} + M_{-ve}\right)}{2}\right] + 2M_{-ve}$$

$$\text{If } \frac{a}{l} \geq 0.2 \text{ then } P_{u,0.2} = \frac{\left[\pi\left(M_{+ve} + M_{-ve}\right) + 4M_{-ve}\right]}{\left[1 - \frac{2a}{3l}\right]}$$

Corner point load:

$$\text{If } \frac{a}{l} = 0 \text{ then } P_{u,0} = 2M_{-ve}$$

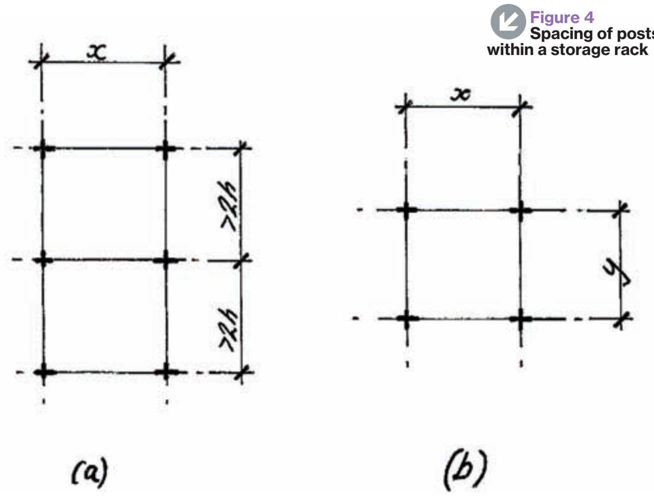
$$\text{If } \frac{a}{l} \geq 0.2 \text{ then } P_{u,0.2} = \frac{\left[4M_{-ve}\right]}{\left[1 - \frac{a}{l}\right]}$$

There are no expressions for $\left(\frac{a}{l}\right)$ values that are more than 0 but less than 0.2. In all such instances (including those that follow) it is acceptable to interpolate between the two values.

Grouped concentrated point loads

In instances where the dimension between storage rack posts is less than 2h, a combined point load is developed. It is therefore acceptable to use the rules for a single point load described previously.

Figure 4 Spacing of posts within a storage rack



Where point loads are spaced at greater than 2h apart (Figure 4a) the following expressions are to be used, to determine the internal point load capacity on the floor slab:

$$\text{If } \frac{a}{l} = 0 \text{ then } P_{u,0} = \left[2\pi\frac{1.8x}{l}\right]\left(M_{+ve} + M_{-ve}\right)$$

$$\text{If } \frac{a}{l} \geq 0.2 \text{ then } P_{u,0.2} = \left[\frac{4\pi}{1 - \frac{a}{3l}} + \frac{1.8x}{l - \frac{a}{2}}\right]\left(M_{+ve} + M_{-ve}\right)$$

where:
x is the distance between the centrelines of the storage rack posts.

As the distance x increases, the capacity of the floor slab tends toward that for single point loads. Where pairs of loads are near edges, the internal load capacity can be multiplied by the ratio of the internal point load and the edge point load capacities.

Where there are a set of four point loads that are placed x and y apart (Figure 4b), and where both dimensions are greater than 2h, the applied loading on the floor slab is typically less than for single point loads. For such groups, the following expressions should be used to determine the point load capacity of the slab:

$$\text{If } \frac{a}{l} = 0 \text{ then } P_{u,0} = \left[2\pi\frac{1.8\left(x+y\right)}{l}\right]\left(M_{+ve} + M_{-ve}\right)$$

$$\text{If } \frac{a}{l} \geq 0.2 \text{ then } P_{u,0.2} = \left[\frac{4\pi}{1 - \frac{a}{3l}} + \frac{1.8\left(x+y\right)}{l - \frac{a}{2}}\right]\left(M_{+ve} + M_{-ve}\right)$$

where:
y is the orthogonal distance between the centreline of the storage rack posts that are perpendicular to x.

When any of the distances between posts are greater than 3.5l, the single point load capacity described previously, applies.

Line loads

When considering line loads, their location in the slab affects their capacity in a similar way to concentrated point loads. The key to the derivation of the slab's capacity to resist a line load, is what is described as the characteristic of the floor slab system λ and is defined as follows:

$$\lambda = \left(\frac{3k}{E_{cm}h^3}\right)^{\frac{1}{4}}$$

Once λ has been calculated, the line load capacity P_{lin} can be determined using the following expression:

$$P_{lin} = 4\lambda M_{un}$$

When the line load is less than 3/λ to an edge of a floor slab the expression alters to:

$$P_{lin} = 3\lambda M_{un}$$

It should be noted, however, that the line load capacity shall be compared to the unfactored line load, as relevant partial factors have already been applied to the derivation of the slab's line load capacity.

Imposed area loads

Area loads are typically of major concern when items are stacked using palettes. As with line loads, the characteristic system of the slab becomes important. The capacity of a slab to support an area load q is defined as follows:

$$q = 5.95\lambda^2 M_{un}$$

The value of q is typically given in terms of kN/m²

The area over which these loads can be applied is defined as π/l and they must be at least $\pi/2\lambda$ apart. This distance is often referred to as the 'critical aisle width' as it defines where palettes can be stacked within a warehouse. Similarly to line loads, the applied unfactored load should be compared against the capacity, due to the inclusion of partial factors in its derivation.

Punching shear

Punching shear failure is quite possible in ground bearing slabs, especially when very tall storage racks are placed upon them. It is therefore important to carry out punching shear checks on ground bearing slabs during their design. Fortunately, the fact that they are supported directly off of a subgrade provides them with some enhancement against punching shear failure.

The critical perimeter along which the slab must be checked is 2h away from the face of the baseplate to the storage rack posts. The corners of the critical shear perimeter should also be rounded (Figure 5).

Provided the ratio a/l is less than 0.2, a ground bearing slab has an enhanced capacity to resist punching shear due to applied design point loads P. To address this, reactions R_p and R_{cp} (for mid-span and edge locations respectively) can be calculated, which are deducted from the applied design point load. This reduced force is then used to calculate the applied punching shear to the slab. The following formulae can be used to determine these reactions:

Internal point load reduction R_p

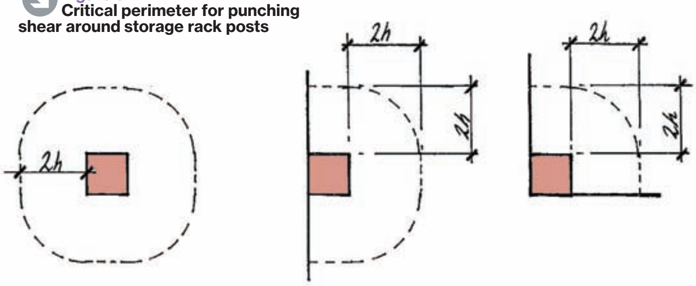
$$R_p = 1.4\left(\frac{h}{l}\right)^2 P + 0.47\,(b+d)\,\frac{dP}{l^2}$$

Edge and corner point load reduction R_p

$$R_{cp} = 2.4\left(\frac{h}{l}\right)^2 P + 0.8\,(2b+d)\,\frac{dP}{l^2}$$

where:
P is the unfactored point load
b is the width of the baseplate to the storage rack post
d is the length of the baseplate to the storage rack post

Figure 5
Critical perimeter for punching shear around storage rack posts



Worked example

A 150mm thick ground bearing steel fibre reinforced concrete ground bearing floor slab for a storage warehouse is to be made, based on the following material properties:

Concrete strength class of C32/40

EN 14651 Notched beam test data results:

$f_{R,1}$ = 2.2 N/mm²
$f_{R,2}$ = 2.0 N/mm²
$f_{R,3}$ = 1.8 N/mm²
$f_{R,4}$ = 1.5 N/mm²

The soil has a k value of 0.05N/mm²/mm.

There are to be stud partitions and there is no information available on what storage rack system is to be adopted. Determine the single point, line load and area load capacity of the slab at internal, edge and corner locations.

DETERMINE BENDING MOMENT CAPACITY

REINFORCED CONCRETE : $M_u = \frac{h^2}{\gamma_m}\left(0.29\sigma_{r4} + 0.16\sigma_{r,1}\right)$

$= \frac{150^2}{1.5}\left((0.29\times0.45\times1.5)+(0.16\times0.37\times2.2)\right)$

$= 4889.9$ Nmm/m $= 4.9$ kNm/m

UNREINFORCED CONCRETE : $M_{un} = f_{ctd,fl}\left(\frac{h^2}{6}\right)$

$f_{ctd,fl} = f_{ctm} \times \left(1.6 - \frac{h}{1000}\right)/\gamma_m$

$= 3.0\text{N/mm}^2 \times \left(1.6 - \frac{150}{1000}\right)/1.5$

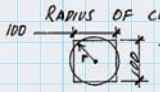
$= 2.9$ N/mm²

$\therefore M_{un} = 2.9\text{N/mm}^2 \times \left(\frac{150^2}{6}\right) = 10875$ Nm/m

$= 10.9$ kNm/m

DETERMINE RADIUS OF CONTACT AREA AND RELATIVE STIFFNESS

RADIUS OF CONTACT : ASSUME 100mm x 100mm CONTACT AREA.



$\therefore$ RADIUS $a = \sqrt{\frac{100\text{mm}\times100\text{mm}}{\pi}} = 56.4$mm

RELATIVE STIFFNESS RADIUS $l = \left(\frac{E_{cm}h^3\times10^6}{(12(1-\nu^2)k)}\right)^{\frac{1}{4}} = \left(\frac{33000\text{N/mm}^2\times150^3\text{mm}\times10^6}{(12\times(1-0.2^2)\times0.05\text{N/mm}^2/\text{mm}}\right)^{\frac{1}{4}}$

$l = 20970$mm

DETERMINE RATIOS OF RADII

$$\frac{a}{\lambda} = \frac{56.4\,mm}{20970\,mm} = 0.003 \approx 0$$

POINT LOAD CAPACITIES:

INTERNAL $P_{u,0} = 2\pi(M_{+ve} + M_{-ve}) = 2\pi(4889.7 + 10875)$
$= 99053\,N = 99.05\,KN$

EDGE $P_{u,0} = \left[\pi \frac{M_{+ve} + M_{-ve}}{2}\right] + 2M_{-ve} = \left(\pi\left(\frac{4889.7 + 10875}{2}\right)\right)$
$+ 2 \times 10875$
$= 46513\,N = 46.51\,KN$

CORNER $P_{u,0} = 2M_{-ve} = 2 \times 10875 = 21750\,N = 21.75\,KN$

LINE LOAD CAPACITY:

$$\lambda = \left(\frac{3k}{E_{cm} h^3}\right)^{\frac{1}{4}} = \left(\frac{3 \times 0.05}{33000 \times 150^3}\right)^{\frac{1}{4}} = 1.08 \times 10^{-3}$$

INTERNAL $P_{lin} = 4\lambda M_{un} = 4 \times 1.08 \times 10^{-3} \times 10875 = 46.98\,N/mm$

EDGE $P_{lin} = 3\lambda M_{un} = 3 \times 1.08 \times 10^{-3} \times 10875 = 35.24\,N/mm$

AREA LOAD CAPACITY

AREA LOAD CAPACITY $= 5.95\lambda^2 M_{un} = 5.95 \times (1.08 \times 10^{-3})^2 \times 10875$
$= 0.075\,N/mm^2$
$= 75\,KN/m^2$

PUNCHING SHEAR CHECK

SHEAR CAPACITY $= v_f = \frac{0.015(f_{R,1} + f_{R,2} + f_{R,3} + f_{R,4})}{2}$
$= \frac{0.015(2.2\,N/mm^2 + 2.0\,N/mm^2 + 1.8\,N/mm^2 + 1.5\,N/mm^2)}{2}$
$= 0.056\,N/mm^2$

INTERNAL POINT LOAD:

PERIMETER $= 4 \times 100\,mm + \pi \times 2 \times 2 \times 150\,mm = 2285\,mm$
$\therefore$ AREA RESISTING SHEAR $= 150\,mm \times 2285\,mm$
$= 342.74 \times 10^3\,mm^2$
$\therefore$ SHEAR RESISTANCE $= 0.056\,N/mm^2 \times 342.74 \times 10^3\,mm^2$
$= 19194\,N$
$= 19.2\,KN$

EDGE POINT LOAD:

PERIMETER: $3 \times 100\,mm + \left(\frac{\pi \times 2 \times 2 \times 150\,mm}{2}\right) = 1243\,mm$
$\therefore$ AREA RESISTING SHEAR $= 150\,mm \times 1243\,mm$
$= 186.45 \times 10^3\,mm^2$
$\therefore$ SHEAR RESISTANCE $= 0.056\,N/mm^2 \times 186.45 \times 10^3\,mm^2$
$= 10441\,N$
$= 10.4\,KN$

CORNER POINT LOAD:

PERIMETER: $2 \times 100\,mm + \left(\frac{\pi \times 2 \times 2 \times 150\,mm}{4}\right) = 671\,mm$
$\therefore$ AREA RESISTING SHEAR $= 150\,mm \times 671\,mm$
$= 100.69 \times 10^3\,mm^2$
$\therefore$ SHEAR RESISTANCE $= 0.056\,N/mm^2 \times 100.69 \times 10^3\,mm^2$
$= 5638\,N$
$= 5.64\,KN$

Applied practice

BS EN 1997-2 Eurocode 7: Geotechnical design. Ground investigation and testing

BS EN 1990-1 Eurocode 0: Basis of Design

BS EN 1992-1-1 Eurocode 2: Design of Concrete Structures – Part 1-1: General Rules for Buildings

BS EN 1992-1-1 UK National Annex to Eurocode 2: Design of Concrete Structures – Part 1-1: General Rules for Buildings

BS EN 14651:2005+A1:2007: Test method for metallic fibre concrete. Measuring the flexural tensile strength (limit of proportionality ((LOP)), residual)

BS EN 14845-1:2007: Test methods for fibres in concrete Reference concretes

BS EN 14889-1:2006: Fibres for concrete Steel fibres. Definitions, specifications and conformity

Glossary and further reading

Ground supported floor – concrete slab that is cast directly onto a sub-base material that, in turn, is placed on top of the ground. It is assumed that such a slab is uniformly supported.

Racking – Storage shelving that is very high and can only be accessed via plant such as forklifts and/or cranes.

Steel fibres – small fragments of metal poured into concrete prior to its placement to act as tension reinforcement.

Further Reading

The Concrete Society (2013) *Technical Report 34: Concrete industrial ground floors: A design guide to construction* (4th ed.), Camberley, UK: The Concrete Society

UK Highways Agency (2009) Specification for Highway Works, Series 800: *Road Pavements – Unbound, cement and other hydraulically bound mixtures* [Online] Available at: www.dft.gov.uk/ha/standards/mchw/vol1/pdfs/series_0800.pdf (Accessed: April 2014)

Vandewalle L. (1995) *Design method for steel fiber reinforced concrete proposed by Rilem TC162-TDF* [Online] Available at: www.rilem.net/images/publis/pro015-004.pdf (Accessed: April 2014)

The Institution of Structural Engineers (2013) Technical Guidance Note 30 (Level 1), *The Structural Engineer*, 91 (8), pp. 38-40

Web resources

The Concrete Centre: http://www.concretecentre.com/

The Institution of Structural Engineers library:
http://www.istructe.org/resources-centre/library