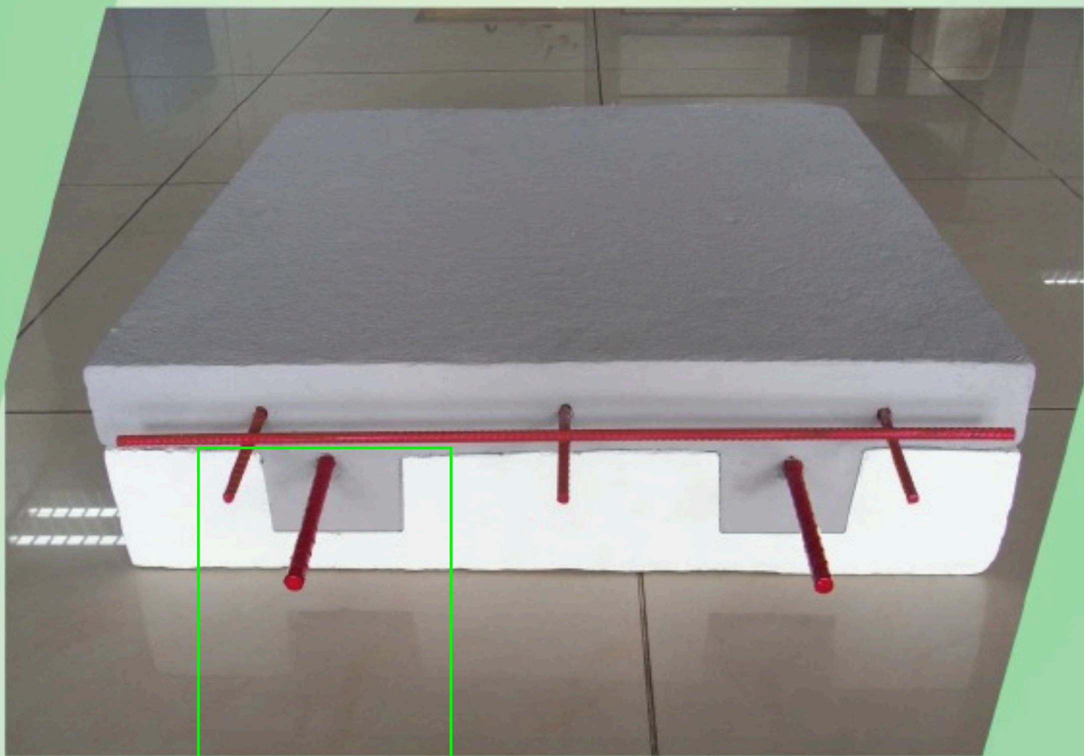
KOTO FLOOR DECK

Design and Detailing Manual



Malaysia

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CONTENTS

This manual has been prepared by Ron Marshall B.E., M.Eng.Sc. on behalf of Koto Corp for use by Building Designers. The information provided in this publication is intended as general guidance only and is not a substitute for the services of professional consultants on particular projects. The manual is subject to regular updates and designers should check they have the latest version.

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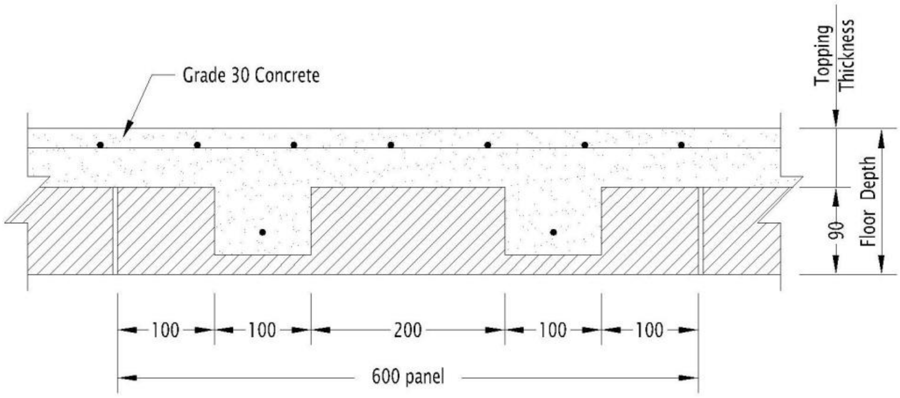
1. Introduction

1.1 Scope

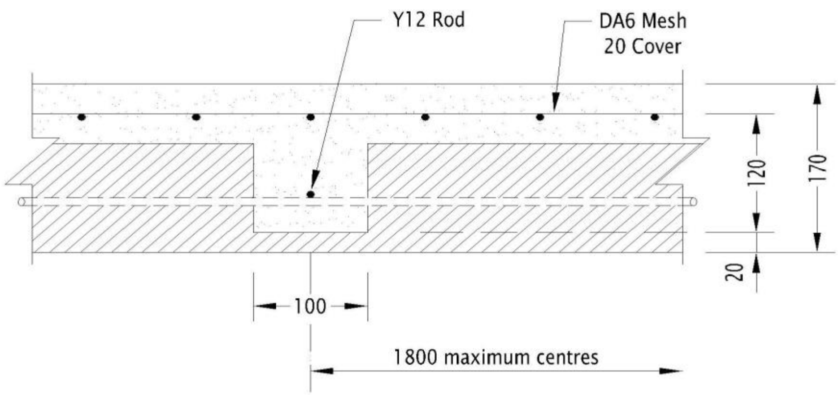
This design manual has been prepared by Koto Corp to assist in the design of concrete flooring in buildings using K-Form panels. While alternative design approaches may be applicable for specific applications, the methodology adopted in this manual has been developed to comply with English Standard *BS8110-1:1997*. The designer must also take into account all other detail requirements of *BS8110-1:1997* not specifically covered in this manual.

1.2 Product Description

The product consists of rigid panels, which provides permanent formwork to a ribbed reinforced concrete flooring system. The floor detailed in this manual is suitable for residential floor requiring a fire resistance of 90 minutes with spans up to 6.0 metres. This floor has a panel depth of 90 mm topped with 80 mm concrete giving a structural floor depth of 150 mm and an overall floor thickness of 170 mm. For other applications both floor depth and the topping thickness can be varied to suit specific requirements.



Panel Section



Cross Beam Section

2. Material Specification

Concrete

The design values in this manual have been based on a concrete compressive strength of 30 MPa. Other grades can be used with appropriate adjustment to the design values.

- All concrete must be vibrated.

- All slabs shall be properly cured. One method is to apply an approved PVA compound immediately after finishing.

- Standard topping thickness is 80 mm but can be varied to suit design application.

Reinforcement

- Cover to top reinforcement to be 20 mm. Note: It has been assumed that the floor slabs are inside buildings. If they are exposed to external environments this cover may need to be increased for durability purposes.

- Bottom reinforcement in ribs to be Y type bars (grade 460 MPa) with 20 mm cover. Other grade reinforcement may be used with an appropriate adjustment to the design values.

- A mesh of type DA6 is used throughout the topping. Where greater reinforcement area is required for negative bending over the main support walls, the mesh can be either increased in size or supplemented with bars. NOTE: Where supplementary bars are used to increase the negative bending capacity over internal supports, BS8110 requires that they be extend a distance equal to 0.25 of the span

- Where shear reinforcement is required, it can be provided using 6 mm wire stirrups of grade 400 MPa spaced at 100 mm centres.

3. Analysis Methodology

Koto ribbed slab construction is designed as a one-way spanning ribbed floor system. It is recommended that *Table 3.12 – Ultimate bending moments and shear forces in one-way spanning slabs* be used for determining the bending and shear coefficients in the direction of the ribs for continuous slabs. In summary the design coefficients given are:-

(a) End Span:-

End support -	Support Simple -	Moment	0	Shear	0.4
	Support continuous -	Moment	-0.04	Shear	0.46
Midspan -	Support Simple -	Moment	0.086		
	Support continuous -	Moment	-0.075		
First interior support -		Moment	-0.086	Shear	0.6

(b) Interior Spans:-

Supports -	Moment	-0.063	Shear	0.5
Midspan -	Moment	0.063		

Even though the floor will have adequate strength when designed as one way spanning, bending will occur in the cross direction generating positive moments in the mid span and negative moments over walls. Beams are provided in the cross direction with sufficient moment capacity to control cracking. As this cracking is a serviceability criterion, the imposed moments are calculated using serviceability load factors. *Clause 3.5.3.3 Simply-supported slabs* is an appropriate method to evaluate the moment coefficient in the cross direction. The coefficients in Table 4.1 have been extracted from *Table 3.14*.with the moments being calculated using the shortest length of the two directions.

Table 4.1 Cross Moment Coefficients for floors continuous in two directions

Ratio of span in direction of ribs to span in cross direction	<= 1.0	1.1	1.2	1.3	1.4	1.5	1.75	>= 2.0
Negative Moment Coefficient for slab continuous over side support wall	0.047	0.056	0.063	0.069	0.074	0.078	0.087	0.093
Positive Moment Coefficient	0.036	0.042	0.047	0.051	0.055	0.059	0.065	0.070

4. Strength and Deflection Calculation

The floor strength and deflection are calculated in accordance with AS3600 using the following formula:-

4.1 Bending Strength

Design Strength in Bending $$M_{design} = \frac{f_y}{\gamma_m} A_s\, d \left(1 - \frac{0.75\, A_s \frac{f_y}{\gamma_m}}{\frac{f_{cu}}{\gamma_m} b_{ef}\, d} \right)$$

Minimum Reinforcement $A_s^{\min.} = 0.18 b_w h$ For positive bending

$= 0.26 b_w h$ For negative bending

Maximum Reinforcement $A_s^{\max.} = 4.0 b_w h$

Where f_y = Characteristic strength of reinforcement

= 460 MPa

A_s = Area of tensile reinforcement

f_{cu} = Characteristic cube strength of concrete at 28 days

= 30 MPa

h = Overall depth of concrete section

d = Distance from compression face to centroid of reinforcement

b_{ef} = Effective width of compression face

= Width of flange for positive bending

= Width of web for negative bending

b_w = Width of web

γ_m = Partial safety factor for strength of materials

4.2 Shear Strength

Design Concrete Shear Stress:- $$v_c = \frac{0.79 \left(\frac{100 A_s}{b_v d} \frac{f_{cu}}{25} \right)^{1/3} \left(\frac{400}{d} \right)^{1/4}}{\gamma_m} x \left(\frac{f_{cu}}{25} \right)^{1/2}$$

Where minimum shear reinforcement is provided

Design Shear Stress:- $v = v_c + 0.4$

Where minimum shear reinforcement is NOT provided

Design Shear Stress:- $v = 0.5 v_c$

Design Shear Strength:- $V = v b_v d$

where

b_v = Width of web

d = Distance from compression face to tensile reinforcement

4.3 Deflection

Clause 3.4 6 Deflection of beams provides a method where the deflection of a beam will not be deemed excessive if the ratio of its span to its effective depth is not greater that the appropriate ratio. This limiting ratio is given as a basic ratio modified for tension reinforcement stress and compression reinforcement percentage.

For a continuous rib floor the basic ratio is given as:-

Span/ effective depth ratio = 20.8

Modification factor for tension reinforcement is given as:-

$$\text{Modification ratio} = 0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \leq 2.0$$

Modification factor for compression reinforcement is given as:-

$$\text{Modification ratio} = 1.0 + \frac{100A'_{s\,prov}}{bd} \Big/ \left(3 + \frac{100A'_{s\,prov}}{bd}\right) \leq 1.5$$

5. Design Process

Design Steps

A. Decide on which direction the ribs will span, usually in the shortest direction.

B. Select a trial floor section.

C. Calculate design loads.

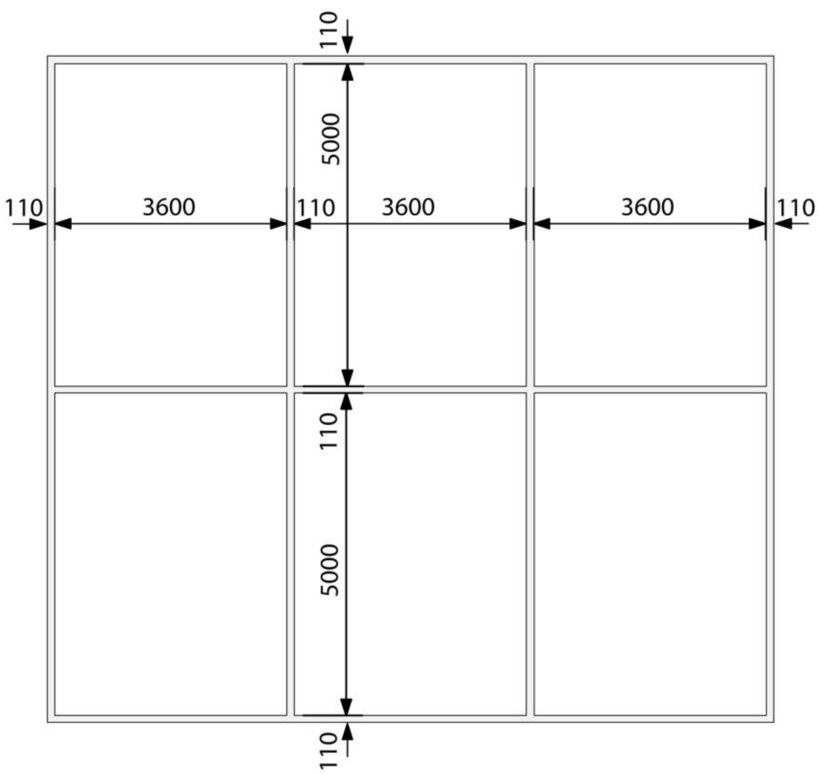
D. Design rib bottom reinforcement. The positive bending forces in the direction of the span determine reinforcement size. Check the positive shear strength of ribs and increase reinforcement if necessary.

E. Calculate the reinforcement required for negative bending over the bearing wall in the direction of ribs. Determine if supplementary reinforcing rods are needed in addition to the mesh to give the required strength. Check the negative shear strength and add shear reinforcement if required.

F. Check the deflection under serviceability loads.

G. Check that the cross ribs have sufficient for bending capacity to control cracking under serviceability loads.

This process is illustrated in the following example. A Koto ribbed floor is to be used in a building composed of 6 rooms each 3.6 m by 5.0 m with a required live load of 2.0 kPa.



Design Steps

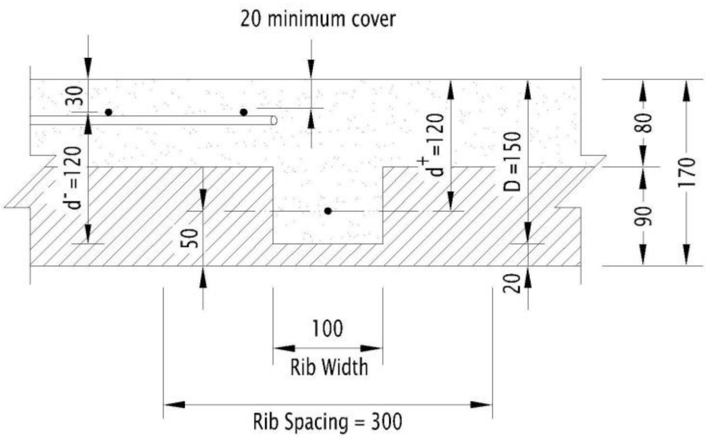
A **Direction of span**

The ribs usually span in the shortest direction. This floor will span the 3.6m direction.

Shortest effective span, L_x = 3.6 + 0.11 = 3.71

Longest effective span, L_y = 5.0 + 0.11 = 5.11

B. **Trial Section**



C. **Calculate Design Loads**

Imposed Load (IL) = 2.0 kPa (Note: Residential loads normally taken as 1.5 kPa)

Dead Load (DL) = 2.7 kPa

Design Load for Strength, $F_{strength}$ = LF*IL + LF*DL
= 1.6*2.0 + 1.4*2.7 = 7.0 kPa

Design Load for Serviceability, $F_{serviceability}$ = LF*IL + LF*DL
= 0.4*2.0 +1.0*2.7 = 3.5 kPa

D. **Design Rib Reinforcement**

Design for worst case in any span:-

Positive Moment Coefficient (end span) = 0.086

Design Positive Moment M_{design} $= 0.086\ F_{strength}\ L_x^2$
$= 0.086*7.0*3.71^2 =$ 8.3 kNm/m

Reinforcement limits:-

Minimum Reinforcement $A_s^{min.} = 0.18 b_w h$

$= 0.0018*100*150 = 27$ mm^2/rib

Maximum Reinforcement $A_s^{max.} = 0.040 b_w h$

$= 0.040*100*150 = 600$ mm^2/rib

Try 1-Y12 per rib A_{st} = 113 mm^2/rib = 113*1000/300 = 377 mm^2/m

Design Strength $$M_{design} = \frac{f_y}{\gamma_m} A_s\, d \left(1 - \frac{0.75\, A_s \frac{f_y}{\gamma_m}}{\frac{f_{cu}}{\gamma_m} b_{ef}\, d}\right)$$

$$= \frac{460}{1.05} * 377 * 10^{-6} * 120 * \left(1 - \frac{0.75 * 377 * \frac{460}{1.05}}{\frac{30}{1.5} * 1000 * 120}\right) = 18.7\ kNm/m$$

$M_{design} = 8.3 < 18.7$ *OK*

1-Y12 per rib satisfactory for Bending.

Check shear in positive moment zone:-

Shear Coefficient (end span) = 0.4

Design Positive Shear $V_{design} = 0.4 F_d\, L_x$

$= 0.4 * 7.0 * 3.71 = 10.4$ kN/m

Concrete Shear Strength (without stirrups)

$$v = 0.5\ \frac{0.79\left(\frac{100 A_s}{b_v d}\ \frac{f_{cu}}{25}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4}}{\gamma_m}\ x\left(\frac{f_{cu}}{25}\right)^{1/2}$$

$$= \frac{0.79\left(\frac{100*113}{100*120}\ \frac{30}{25}\right)^{1/3}\left(\frac{400}{120}\right)^{1/4}}{2*1.25}\ x\left(\frac{f_{cu}}{25}\right)^{1/2} = 0.48\ Mpa$$

Maximum Design Shear without stirrups $V = v\ b_v d$

$= 0.48 * 0.100 * 0.12 * 10\text{^}3$

$= 5.8\ \ kN/rib$

$= 19.2\ \ kN/m$

$V_{design} = 10.4 \leq 19.2$ *OK*

1–Y12 satisfactory for Shear without stirrups.

E. Design Negative Bending:-

Negative Moment Coefficient (continuous floor) = -0.086

Design Negative Moment M_{design} $= 0.086\, F_d\, L_x^2$

$= 0.086*7.0*3.71^2 =$ 8.3 kNm/m

Reinforcement limits:-

Minimum Reinforcement $A_s^{min.} = 0.26 b_w h$

$= 0.0026*100*150 = 27$ mm²/rib

$= 27/0.300 = 90$ mm²/m

Maximum Reinforcement $A_s^{max.} = 4.0 b_w h$

$= 0.040*100*150/0.300 = 2000$ mm²/m

Check strength with the DA6 mesh, $A_s = 283$ mm²/m > $A_{s.min}$ OK

Design Bending Strength $$M_{design} = \frac{f_y}{\gamma_m} A_s\, d \left(1 - \frac{0.75\, A_s \frac{f_y}{\gamma_m}}{\frac{f_{cu}}{\gamma_m} b_{ef}\, d} \right)$$

$$= \frac{455}{1.05} * 283 * 10^{-6} * 120 * \left(1 - \frac{0.75 * 283 * \frac{455}{1.05}}{\frac{30}{1.5} * \frac{100}{0.300} * 120} \right) = 13.0\ kNm/m$$

$M_{design} = 8.3 < 13.0$ *OK*

DA6 mesh is satisfactory for Negative Bending

Check shear in negative bending Zone **:**

Negative Shear Coefficient = 0.6

Design negative Shear $V_{design} = 0.6\, F_d\, L_x$

$= 0.6 * 7.0 * 3.71 = 15.6$ kN/m

Concrete Shear Stress without stirrups

$$v = 0.5\, \frac{0.79 \left(\frac{100 A_s}{b_v d} \frac{f_{cu}}{25} \right)^{1/3} \left(\frac{400}{d} \right)^{1/4}}{\gamma_m}\, x \left(\frac{f_{cu}}{25} \right)^{1/2}$$

$$= 0.5\, \frac{0.79 \left(\frac{100*85}{100*120} \frac{30}{25} \right)^{1/3} \left(\frac{400}{120} \right)^{1/4}}{1.25}\, x \left(\frac{30}{25} \right)^{1/2} = 0.44\ Mpa$$

Maximum Design Shear without stirrups $V = v b_v d$

$= 0.44 * 0.100 * 0.12 * 10\hat{}3$

$= 5.3\ \ kN/rib$

$= 5.3/0.300 = 17.5\ \ kN/m$

$V_{design} = 15.6 \leq 17.5$ *OK*

DA6 Mesh satisfactory for Shear without stirrups.

F. Check Deflection Limits

For a continuous rib floor the basic ratio is given as 20.8

Modification factor for tension reinforcement:-

Serviceability moment = $0.086F_{serviceability}\,L_x^{\;2}$ = 0.086*3.5*3.71 = 4.2 kNm/m

Design service stress $f_s = \dfrac{2f_y A_{s\ req}}{3A_{s\ prov}} x \dfrac{1}{\beta_b} = \dfrac{2*455*4.2}{3*18.5} x \dfrac{1}{0.8} = 86\ MPa$

$$\text{Modification ratio} = 0.55 + \frac{(477 - f_s)}{120\left(0.9 + \dfrac{M}{bd^2}\right)} \leq 2.0$$

$$= 0.55 + \frac{(477 - 86)}{120\left(0.9 + \dfrac{4.2*10\text{^}3}{100/300*120^2}\right)} = 2.38 \leq 2.00$$

Modification factor for compression reinforcement is given as:-

$$\text{Modification ratio} = 1.0 + \frac{100A'_{s\ prov}}{bd} \Bigg/ \left(3 + \frac{100A'_{s\ prov}}{bd}\right) \leq 1.5$$

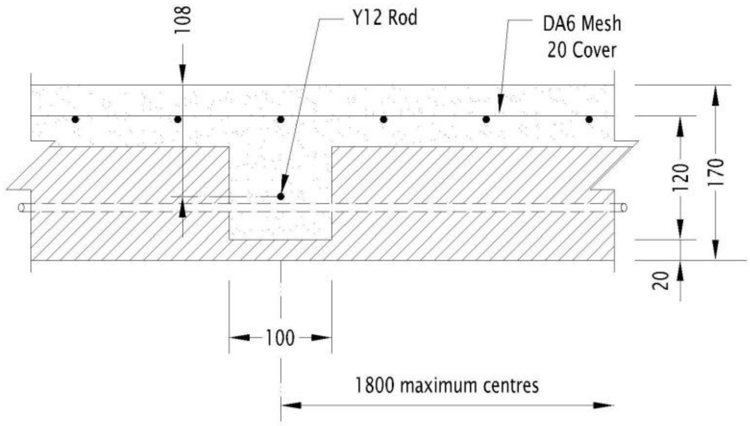
$$= 1.0 + \frac{100*283}{100/0.300*120} \Bigg/ \left(3 + \frac{100*283}{100/300*120}\right) = 1.19 \leq 1.5$$

Modified Span/ effective depth ratio = 20.8*2.00*1.19 = 49.5

Actual Span/ effective depth ratio = 3.71/0.150 = 24.7 < 49.5 OK

Floor Depth deemed satisfactory for Deflection.

G. Check Cross Rib Bending



Ratio Lx to Ly = 3.71/5.11 = 0.73 <1.0

Effective width of flange $b_{ef} = b_w + 0.2 * 0.7L_x$
$= 100 + 0.2*0.7*3.71 = 600$ mm

Design Load (for serviceability) = 3.5 kPa

Midfloor Bending

Positive moment coefficient $\beta_y = 0.035$ from Table 4.1

Design Positive Moment $M^* = 0.035\, F_d\, L_x^2$
$= 0.035 * 3.5 * 3.71^2$
$= 1.7$ kNm/m

Reinforcement limits:-

Minimum Reinforcement $A_s^{min.} = 0.18 b_w h$
$= 0.0018*100*150 = 27$ mm^2/rib

Maximum Reinforcement $A_s^{max.} = 4.0 b_w h$
$= 0.040*100*150 = 600$ mm^2/rib

Try 1-Y12 per rib $A_{st} = 113$ mm^2/rib

Design Strength
$$M_{design} = \frac{f_y}{\gamma_m} A_s\, d \left(1 - \frac{0.75 A_s \frac{f_y}{\gamma_m}}{\frac{f_{cu}}{\gamma_m} b_{ef}\, d} \right)$$

$$= \frac{455}{1.05} * 113 * 10^{-6} * 108 * \left(1 - \frac{0.75 * 113 * \frac{455}{1.05}}{\frac{30}{1.5} * 600 * 108} \right) = 5.1\ kNm/m$$

Maximum Rib Spacing $= M_{design} / M_{load} = 5.1/1.7 = 3.0\ m$

Cross Rib Spacing of 1.8 m is OK

Bending over side support

Negative Moment Coefficient β_y = 0.047 from Table 4.1

Design Negative Moment M^* $= 0.047\ F_d\ L_x^2$
$= 0.047 * 3.5 * 3.71^2$
$= 2.3$ kNm/m

Reinforcement limits:-

Minimum Reinforcement $A_s^{\min.} = 0.18 b_w h$
$= 0.0018*100*150 = 27$ mm^2/rib

Maximum Reinforcement $A_s^{\max.} = 4.0 b_w h$
$= 0.040*100*150 = 600$ mm^2/rib

Check strength with the DA6 mesh, A_{st} = 283 mm^2/m
= 283*600/1000
= 170 mm^2/cross rib

Design Strength $$M_{design} = \frac{f_y}{\gamma_m} A_s\, d \left(1 - \frac{0.75\, A_s \frac{f_y}{\gamma_m}}{\frac{f_{cu}}{\gamma_m} b_{ef}\, d} \right)$$

$$= \frac{455}{1.05} * 170 * 10^{-6} * 120 * \left(1 - \frac{0.75*170*\frac{455}{1.05}}{\frac{30}{1.5}*100*120} \right) = 6.8\ kNm/rib$$

Maximum Rib Spacing $= M_{design} / M_{load} = 6.8/2.3 = 2.9\ m$

Cross Rib Spacing of 1.8 m is OK

Design Solution

Use 160 mm depth Koto floor spanning continuously in the 3.6 m direction with 80 mm topping thickness, 1-Y12 rod in each rib, DA6 mesh throughout the topping slab and two 100 mm cross beam spaced at 1.2m centres reinforced with 1-Y12 rod.

6. Design Aids

6.1 Strength Properties In Direction Of Ribs

Bending and shear properties for Koto floors in the direction of the ribs are given Table 6.2 for Positive Bending and Table 6.3 for Negative Bending.

TABLE 6.1 Positive Bending In Direction Of Ribs

Total floor Depth (mm)	Floor self weight (kPa)	Reinforcement In Each Rib	Positive Design Capacities		
			Maximum Design Moment (kNm/m)	Maximum Design Shear (kN/m)	
				No Shear Reinforcement	With Shear Reinforcement
170	2.7	1-Y12	18	19	55
		1-Y16	31	23	63
		1-Y20	46	27	70

TABLE 6.2 Negative Bending in Direction of Ribs

Total Floor Depth (mm)	Size of Reinforcement mesh in Topping	Topping Reinforcement				
		Mesh only		Mesh plus Y12 bars @ 300 mm spacing		
		Maximum Design Moment (kNm/m)	Maximum Design Shear (kN/m)	Maximum Design Moment (kN.m/m)	Maximum Design Shear (kN/m)	
					With No Shear Reinforcement	With Shear Reinforcement in each rib
170	DA6	13	17	25	23	62

6.3 Strength Properties Cross Beams

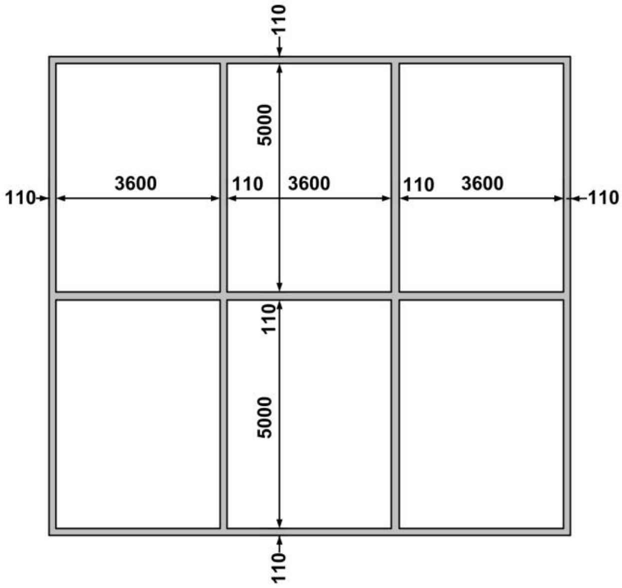
Table 6.4 gives bending properties for cross beams in Koto floors. If the moment capacity with the beams at the standard 1.8 m spacing is not adequate, then the spacing can be reduced.

TABLE 6.3 Moment Capacities of Cross Beams

Total Floor Depth (mm)	Size of Reinforcement mesh in Topping	Maximum Negative Design Moment with mesh only in topping		Maximum Positive Design Moment with 1-Y12 rod in cross rib	
		Moment per Rib (kNm/Rib)	Moment with Ribs @ 1.8 m spacing (kNm/m)	Moment per Rib (kNm/Rib)	Moment with Ribs @ 1.8 m spacing (kNm/m)
170	DA6	6.8	3.7	5.1	2.8

6.4 Design Using Strength Property Tables

NOTE: The designer must satisfy himself of the accuracy of the Tables before using them.



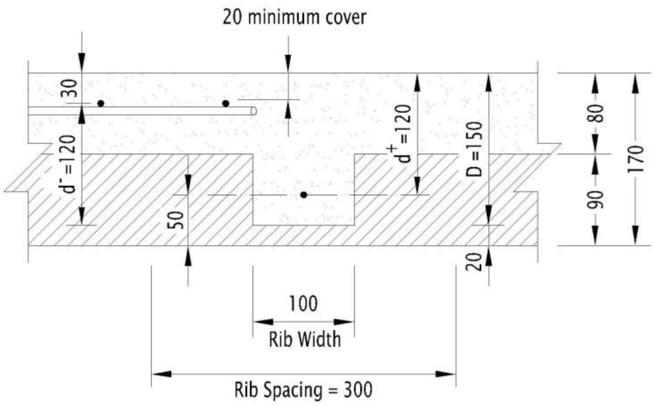
DIRECTION OF SPAN

Span the floor in the 3.6 m direction.

Shortest effective span, L_x = 3.71

Longest effective span, L_y = 5.11

TRIAL STANDARD 170 MM KOTO FLOOR



DESIGN LOADS

Imposed Load (IL) = 2.0 kPa
Dead Load (DL) = 2.7 kPa

Design Load for Strength, $F_{strength}$ = LF*IL + LF*DL
= 1.6*2.0 + 1.4*2.7 = 7.0 kPa

Design Load for Serviceability, $F_{serviceability}$ = LF*IL + LF*DL

= 0.4*2.0 +1.0*2.7 = 3.5 kPa

RIB REINFORCEMENT

CHECK POSITIVE BENDING:-

Positive Moment Coefficient (end span) = 0.086

Design Positive Moment M_{design} $= 0.086\ F_{strength}\ L_x^2$
$= 0.086*7.0*3.71^2 =$ 8.3 kNm/m

Positive Shear Coefficient = 0.4
Design Positive Shear V_{design} $= 0.4 F_d\ L_x$
= 0.4 * 7.0 * 3.71 = 10.4 kN/m

From Table 6.1, for a total floor thickness of 170 mm with 1-Y12 rod in each rib,
Maximum Positive Moment Capacity = 18 kNm/m > 8.3 *OK*
Maximum Design Shear Capacity = 19 kN/m > 10.4 *OK*
Ribs satisfactory for Positive Bending.

CHECK NEGATIVE BENDING:-
Negative Moment Coefficient (continuous floor) = -0.086
Design Negative Moment M_{design} $= 0.086\ F_d\ L_x^2$
$= 0.086*7.0*3.71^2 =$ 8.3 kNm/m

Negative Shear coefficient = 0.6
Design Transverse Shear $= 0.6\ F_d\ L_x$
= 0.6 * 7.0 * 3.71 = 15.6 kN/m

From Table 6.2, for a total floor thickness of 160 mm with DA6 mesh in topping,
Maximum Negative Moment Capacity = 13 kNm/m > 7.5 *OK*
Maximum Design shear Capacity = 17 kN/m > 15.6 *OK*
Floor satisfactory for Negative Bending.

CHECK CROSS RIB BENDING

Ratio Lx to Ly = 3.71/5.11 = 0.73 <1.0
Serviceability Design Load = 3.5 kPa

POSITIVE BENDING:-
Positive Moment Coefficient β_y = 0.035 from Table 4.1

Design Positive Moment M^* $= 0.035\ F_d\ L_x^2$
$= 0.035 * 3.5 * 3.71^2$
= 1.7 kNm/m

From Table 6.3, for a total floor thickness of 160 mm with 1-Y12 rod in rib,
Maximum Positive moment capacity = 2.4 kNm/m > 1.4 *OK*
Cross Rib Spacing of 1.8 m is OK

NEGATIVE BENDING:-

Negative Moment Coefficient β_y = 0.047 … from Table 4.1

Design Negative Moment M^* $= 0.047\ F_d\ L_x^2$
$= 0.047 * 3.5 * 3.71^2$
$= 2.3$ kNm/m

From Table 6.3, for a total floor thickness of 170 mm with DA6 mesh in topping

Maximum Positive moment capacity = 3.8 kNm/m > 1.7 *OK*

Maximum Negative Moment Capacity = 4.5 kNm/m > 2.3 *OK*

Cross Rib Spacing of 1.8 m is OK

Cross ribs satisfactory at 1.8 m spacing for serviceability load.

Standard 160 mm depth Koto floor spanning continuously in the 3.6 m direction with 1-Y12 rod in each rib, 1-Y12 rod in cross rib and DA6 mesh throughout the top slab is satisfactory.

7. Simplified Design

7.1 Load/Span Chart

The superimposed load (before the application of load factors) that the standard 170 mm Koto continuous span floor can be designed to carry is shown in Chart 7.1 for a selection of reinforcement.

NOTE: The designer must satisfy himself of the accuracy of this chart before relying on it.

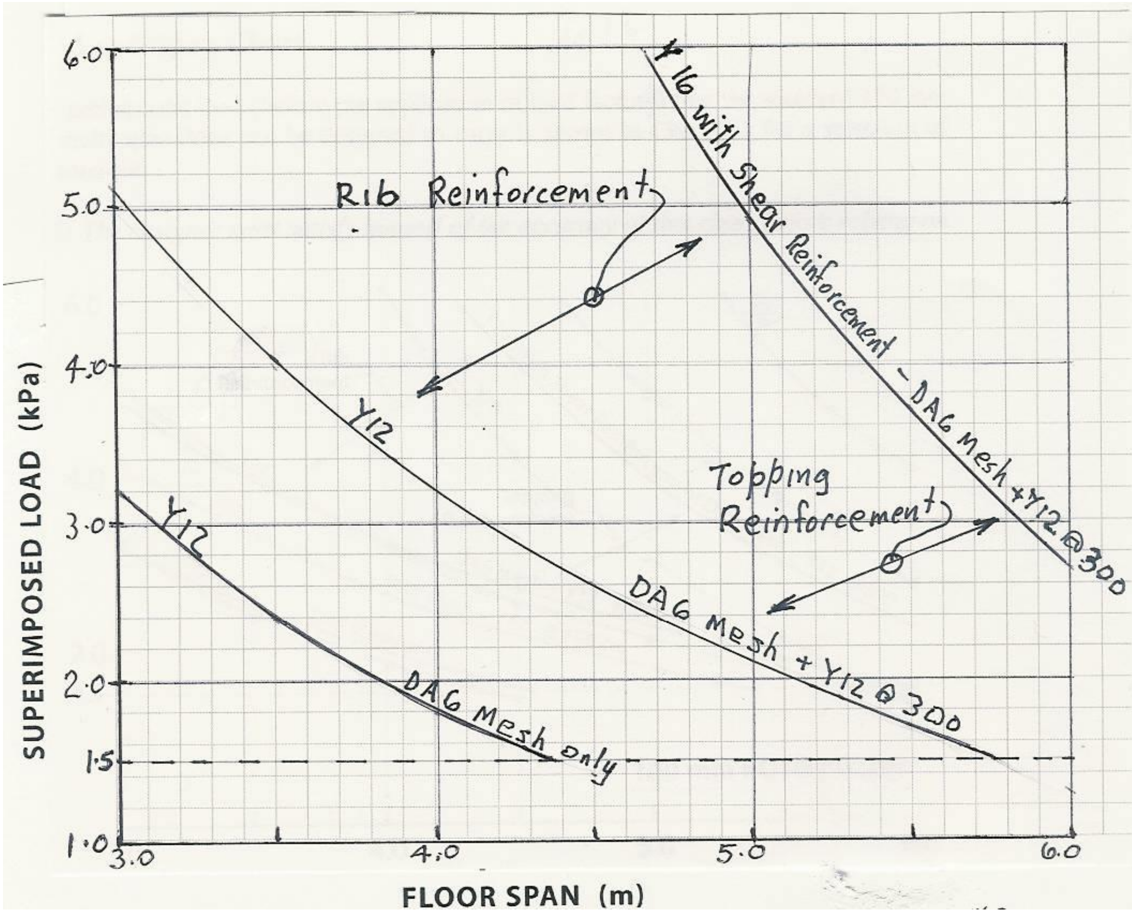


Chart 7.1 Load capacity of standard 170 mm KOTO floor

7.2 Design Using Load/Span Chart

Requirement: A floor is required to multi-span 5.0 m with a design load of 1.5 kPa.

Design: From chart a 170 Koto floor reinforced with 1-Y12 bottom in each rib and DA6 mesh in the topping supplemented with Y12 rods at 600 crs over the support wall, will carry a load of 2.1 kPa. - *Use*

Design Solution: Use a 170 Koto floor reinforced with 1-Y12 bottom in each rib and DA6 mesh in the topping supplemented with Y12 rods at 300 crs extending 1250 mm either side of centre support walls.

8. Detailing and Construction

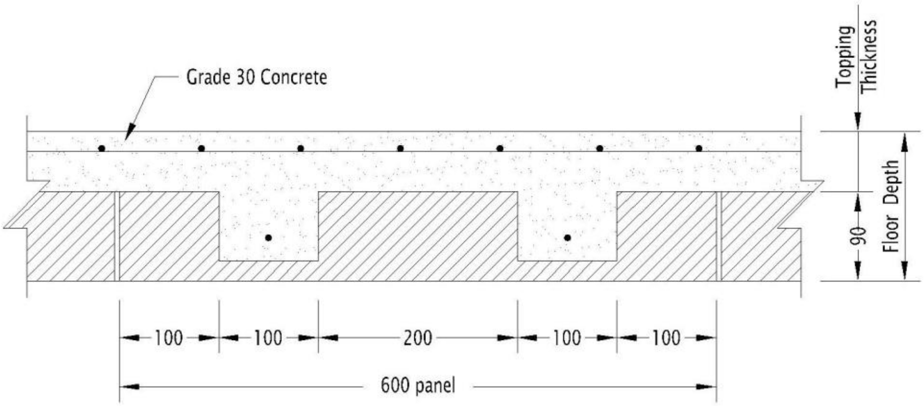


Diagram 8.1 Panel Section

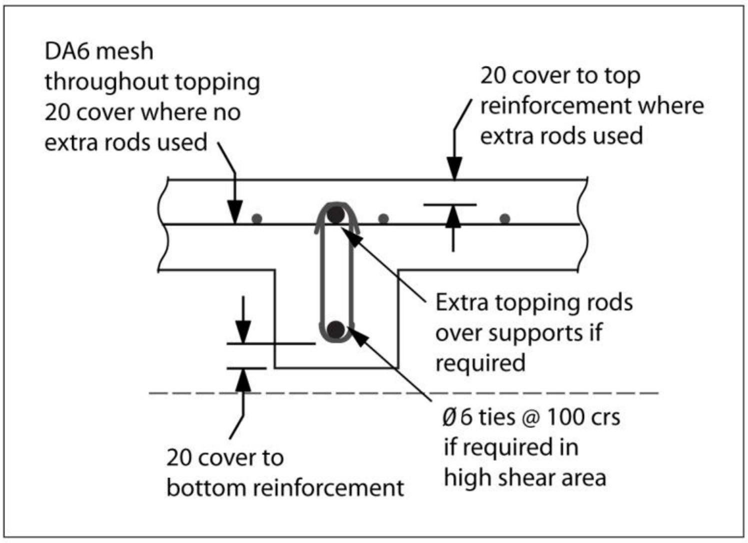


Diagram 8.2 Arrangement of Reinforcement

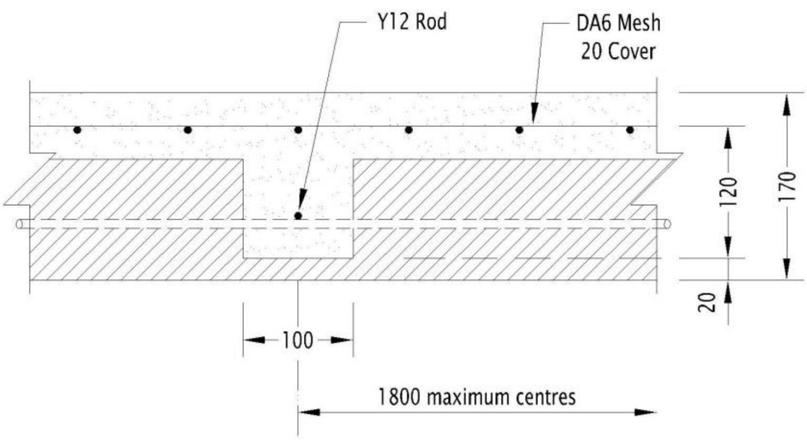


Diagram 8.3 Cross Beam Section

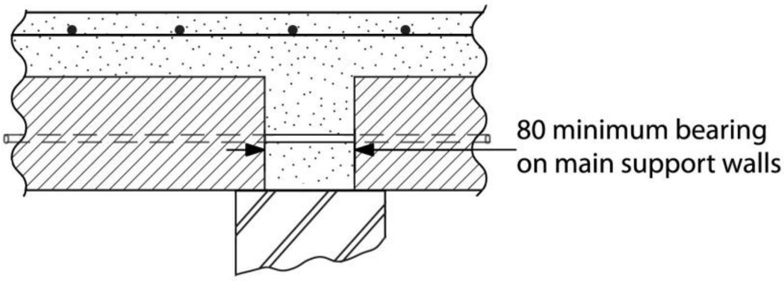


Diagram 8.4 Main Wall Bearing

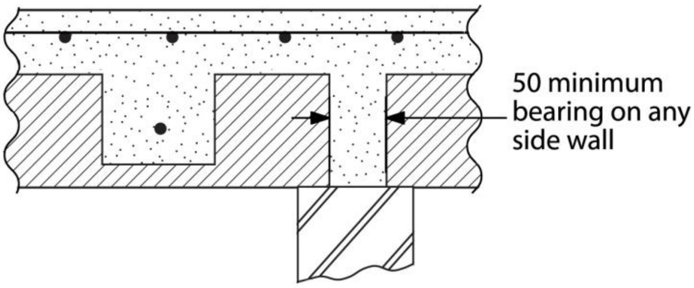


Diagram 8.5 Side Wall Bearing