# Techno India NJR Institute of Technology

## Academic Administration of Techno NJR Institute

## Syllabus Deployment

| | | |
|---|---|---|
| Name of Faculty | : Mr. Rakesh Yadav | Subject Code: 5CE4-21 |
| Subject | : Concrete Structures Design | |
| Department | : Civil Engineering | Sem: V |
| Total No. of Labs Planned: 13 | | |

COURSE OUTCOMES HERE (3 OUTCOMES)

At the end of this course students will be able to:

CO1: To design various components of the structures.

CO2: Study the development length and shear reinforcement.

CO3: To design the axially loaded column, isolated column footing.

| Lab No. | Exp. No. | Topic |
|---|---|---|
| 1 | 1 | Revision of Typical problems of BMD and SFD |
| 2 | 2 | Analysis and Design of singly reinforced rectangular beam section for Flexure, based on Working stress design philosophy. |
| 3 | 3 | Analysis and Design of singly reinforced rectangular beam section for Flexure, based on Limit State design philosophy |
| 4 | 4 | Analysis and Design of doubly reinforced rectangular beam section For flexure, based on Limit State design philosophy |
| 5 | 5 | Analysis and Design of flanged beam section for flexure, based on Limit State design philosophy |
| 6 | 6 | Problems on Limit state of serviceability for deflection as per codal Provisions of empirical coefficients. |
| 7 | 6 | Analysis and design of prismatic sections for shear using LSD |
| 8 | 8 | Problems on limit state of collapse in bond |
| 9 | 9 | Analysis and design of one way slabs using LSM |
| 10 | 10 | Analysis and design of two way slabs using LSM |

For Techno India NJR Institute of Technology
पंकज पोरवाल
Dr. Pankaj Kumar Porwal
(Principal)

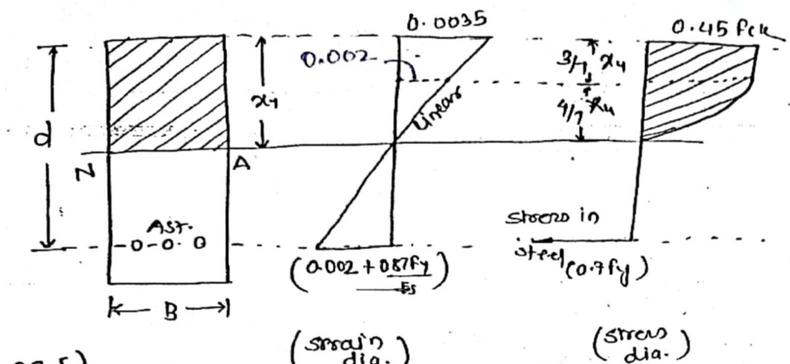
**5CE4-21: CONCRETE STRUCTURES DESIGN**

**1.** Analysis and Design of singly reinforced rectangular beam section for flexure, based on Limit stress design philosophy.

## Explanation

* Singly Reinforced section. by LSM

* Assumptions [38.1 (a to f). P-69, IS 456-2000]




"going to Movie Super 30" on Wednesday.

38.1)

(a) At any section a plane section before bending remains plane after bending.

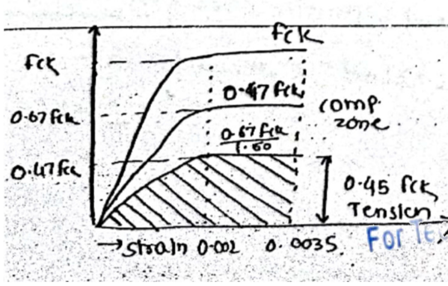
→ strain diagram is Linear.

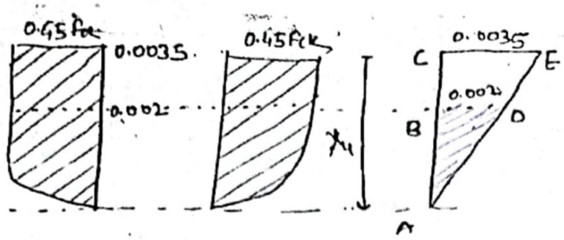
(b) The max^m strain at highly compressed extreme fiber in concrete in bending compression.

= 0.0035

(* At highly compressed fiber at failure.)

(c) An acceptable stress strain curve is given in (fig-21)






From similar $\Delta$le.

$$\frac{AB}{AC} = \frac{0.002}{0.0035} = \frac{4}{7}$$

$$AB = \frac{4}{7} AC$$

$$BC = x_u - \frac{4}{7} x_u = \frac{3}{7} x_u$$

* The compressive strength of concrete shall be considered 0.67 fck and partial factor of safety of $\gamma_{mc} = 1.50$ shall be applied in addition of this.

* 0.67 fck relation is due size ~~of~~ factor. The actual size of RCC member may differ from the size of cube (150 mm) used for testing.

assign stress on concrete

$$= \frac{0.67\, f_{ck}}{1.50} = 0.45\, f_{ck}.$$

④ All tensile stresses shall be taken by steel only none by concrete.

→ Stress in concrete in tension ~~member~~ becomes very high ; so concrete is considered cracked tensile strength of concrete is ignored. this theory is called cracked section theory.

ase-2 Design of beam

When size of beam is given the section is to be designed for a BM.

① calculate factored BM

$BM_u = 1.5\ BM.$

② calculate : $M_u \lim = Q \cdot Bd^2$

③ case-i) $BM_u < M_{ulim}$

→ design a under reinf^n section (singly)

case-ii) $BM_u > M_{ulim}$

→ design a doubly reinf^n section.

⑤ case (i) Design of a singly URS.

$x_u < x_{ulim}$

⑥ Equate

$BM_u$ = M.R. Formula.

$$\boxed{BM_u = 0.36\, f_{ck} \cdot B \cdot x_u \cdot (d - 0.42\, x_u)}$$

Solve = $x_u$

$BM_u = 0.87 \cdot f_y \cdot A_{st} \, (d - 0.42 x_u)$

$$A_{st} = \frac{BM_u}{0.87 F_y \cdot (d - 0.42 x_u)}$$

# Direct formula for $A_{st}$

For an under reinf.$^m$ section and Limiting section

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\, BM_u}{F_{ck} \cdot B \cdot d^2}}\right] \times B \cdot d.$$

Q.5 $B = 400$ mm. $d = 600$ mm $\frac{0.5 f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6\, BM_u}{f_{ck}\, bd^2}}\right) bd$

$D = 650$ mm M40 Fe 415.

design for 30 kN/m (L.L. + S.I. Load)

over span 6 m Use LSM.

$\Rightarrow$ $DL = 0.4 \times 0.65 \times 25 = 6.5$ kN/m

$L \cdot L + S \cdot I = 20$ kN/m

$26.5$ kN/m

i) Factored BM $= \frac{1.5 \times 26.5 \times 6^2}{8} = 178.2$ kN.m

$Mu\ lim = Q \cdot Bd^2$

$= \frac{5.51 \times 400 \times 600^2}{10^6}$

$Mu_{lim} = 793.44$ kN.m

③ $BM_u < BM_{ulim}$ (URS)

④ $$A_{st} = \frac{0.5 \times 40}{415} \times \left[1 - \sqrt{1 - \frac{4.6 \times 178.875 \times 10^6}{40 \times 400 \times 600^2}}\right] \times 400 \times 600$$

$$= \underline{857.95 \text{ mm}^2}$$

OR

⑤ $A_{st} = \frac{BM_u}{\quad}$

$BM_u = 0.36 f_{ck} B \times x_u (d - 0.42 x_u)$

$178.875 \times 10^6 = 0.36 \times 40 \times 400 \times x_u (600 - 0.42 \times x_u)$

$x_u = 53.78$ mm

⑥ $$A_{st} = \frac{BM_u}{0.87 \times f_y (d - 0.42 x_u)}$$

$$= \frac{178.875 \times 10^6}{0.87 \times 415 \times (600 - 0.42 \times 53.78)}$$

$$= \underline{858.01 \text{ mm}^2}$$

**2. Analysis and Design of doubly reinforced rectangular beam section for flexure, based on Limit State design philosophy.**

**<u>Explanation</u>**

## Design of doubly Reinforced Section
[illegible]/11/[illegible]

**Steps**

① calculate factored B.M.

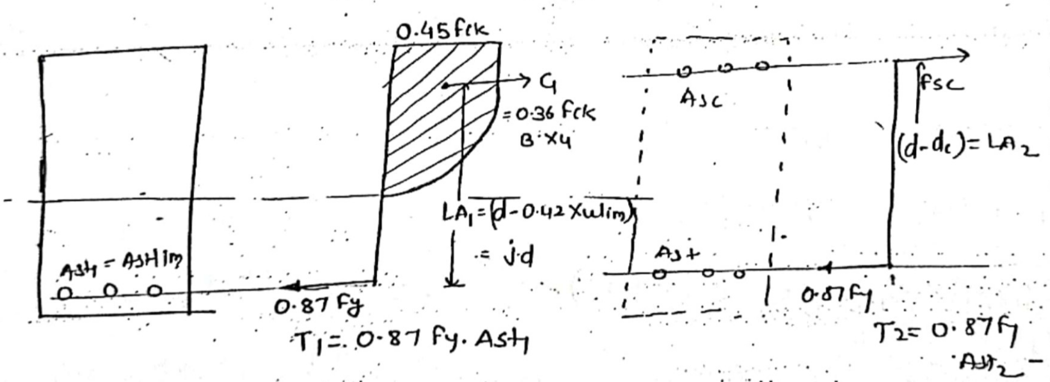
$$BM_u = 1.5BM = \left(\frac{W_u \cdot L^2}{k_1}\right)$$

② check $M_{ulim} = Q \cdot B \cdot d^2$

$$Q = 0.36\, f_{ck} \cdot j \cdot k.$$

③ if $BM_u > B_{ulim}$ (SR).

then we need a doubly reinforced section.




for $Mu_2 = BM_u - M_{ulim}$

④ $M_{ulim}(S.R) = T_1 \times 4 = 0.87 \times f_y \cdot A_{st_1} (j \cdot d)$

$$A_{st_1} = \frac{M_{ulim}(S.R.)}{0.87 f_y \cdot j \cdot d}.$$

⑤ calculate $Mu_2 = BM_u - M_{ulim}$ (S.R.)

⑥ $A_{st_2}$ equating $M_{u_2} = T_2 \times LA_2$

$$M_{u_2} = 0.87 f_y \cdot A_{st_2} \cdot (d - d_c)$$

$$A_{st_2} = \frac{M_{u_2}}{0.87 f_y \cdot (d - d_c)}$$

⑦ calculate $\varepsilon_{sc}$

$$\varepsilon_{sc} = \frac{x_{ulim} - d_c}{x_{ulim}} \times 0.0035$$

⑧ calculate $f_{sc}$ based on $\varepsilon_{sc}$ from stress-strain table.

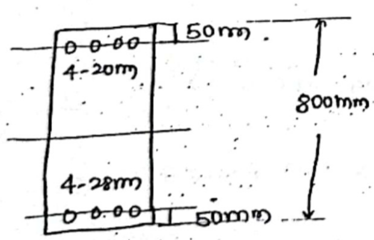
⑨ Equate

$$M_{u_2} = C_2 \cdot LA_2$$

$$M_{u_2} = (f_{sc} - 0.45 f_{ck}) \cdot A_{sc} \, (d - d_c)$$

$$A_{sc} = \frac{M_{u_2}}{(f_{sc} - 0.45 f_{ck})(d - d_c)}$$

Q.1] calculate M.R. of a beam as shown in fig. M30 concrete and Fe 415 steel bars are used.




Fe 415

| Strain | stress |
|---|---|
| 0.00144 | 288 |
| 0.00163 | 306 |
| 0.00192 | 324 |
| 0.00241 | 343 |
| 0.00276 | 352 |
| 0.00380 | 361 |

$\Rightarrow A_{sc} = 4 \times \frac{\pi}{4} \times 20^2 = 1256 \text{ mm}^2$

$A_{st} = 4 \times \frac{\pi}{4} \times 28^2 = 28^2 \times 3.14$

$= 2461.74 \text{ mm}^2$

$\Rightarrow x_u \lim = k \cdot d = 0.48 \times 750 = 360$

② Actual depth of N.A.

$$C_1 + C_2 = T$$

$$0.36 f_{ck} \cdot B \cdot x_u + (f_{sc} - 0.45 f_{ck}) A_{sc} = 0.87 f_y \cdot A_{st}$$

$$0.36 \times 30 \times 360 \times x_u + (f_{sc} - 0.45 f_{ck}) 1256 = 0.87 \times 415 \times 2963$$

$$x_u = \frac{90.6231 - 1.257 f_{sc}}{3888} \quad \text{— (A)}$$

**Trial - 1** Assume $f_{sc} = 350 \ N/mm^2$

From (A)

$$x_u = \frac{90.6231 - 1.257 \times 350}{3.888} = 119.9 \ mm \approx 120 \ mm$$

$$\varepsilon_{sc} = \frac{x_u - d_c}{x_u} \times 0.0035 = \frac{120 - 50}{120} = 0.00204$$

$$f_{sc} = 324 + \frac{343 - 324}{(0.00214 - 0.00192)} \times (0.00204 - 0.00192)$$

**trial - ②** $f_{sc}$. Consider. $329 \ N/mm^2$

$$x_u (\text{from } A) = \frac{906.231 - 125 \times 329}{3888}$$

$$= 126.74 \ mm$$

IS 456

consider $x_u = 126\ mm$

$$\varepsilon_{sc} = \frac{126-50}{126} \times 0.0035 = 0.00211$$

$$f_{sc} = 324 + \frac{343-324}{(0.0024-0.00192)} \times (0.00211 - 0.00192)$$

$= 331\ N/mm^2$

trial - ③ $f_{sc} = 331\ N/mm^2$

$x_u = 126.07$ say $\approx 126\ mm$

$\varepsilon_{sc} = 0.00211$

$f_{sc} = 331\ N/mm^2$ matched.

step - ⑤ MR (ultimate) $x_u < x_{ulim}$ - URS.

$$M_u = 0.36 f_{ck} \cdot B \cdot x_u (d - 0.42 x_u) + (f_{sc} - 0.45 f_{ck}) A_{sc} (d - d_c)$$

$$= 0.36 \times 30 \times 360 \times 126 \times (750 - 0.42 \times 126) + (331 - 0.45 \times 30) \times 1256 (750 - 50)$$

$= 620.63\ kN \cdot m$

## 3. Problems on limit state of collapse in bond.

**Explanation**

(tension) Reinf$^n$)

③ Shear reinf$^n$.

28 NOV. 2016

* Is code Method (for design for shear)

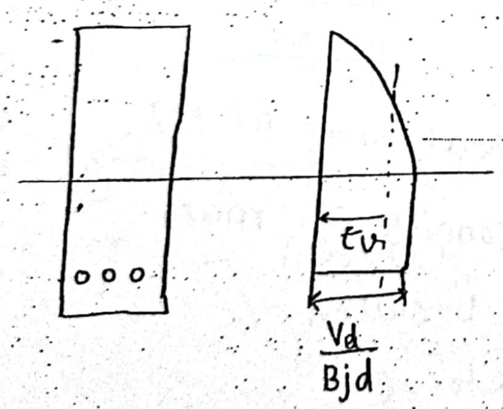
\# maximum shear force shall be calculate at critical section

– for example -- for a simply supported.

$$V = \frac{WL}{2} \quad || \quad V_u = \frac{W_u.L}{2}$$

① Nominal shear stress ($\tau_v$)

Avg. shear stress to be considered for design purpose.




$$\tau_v = \frac{V}{Bd} \text{ or } \frac{V_u}{Bd}$$

$\neq \tau_{max}$

If $\tau_v > \tau_{max}$

No other option - change the size of beam.

② $\tau_{max}$ [max$^m$ shear strength of concrete with shear reinf$^m$]

* This is the max$^m$ value of shear stress that an RCC beam take i.e. shear stress resisted by concrete. main reinforcement and shear reinf$^m$.

| | M15 | M20 | M25 | M30 | M35 | M40 & above | |
|---|---|---|---|---|---|---|---|
| WSM | 1.6 | 1.8 | 1.9 | 2.2 | 2.3 | 2.5 | - Tb - 24 |
| LSM | 2.5 | 2.8 | 3.1 | 3.4 | 3.7 | 4.0 | - Tb - 20 |

* $\tau_{max}$ limit is so that the concrete does not fail due to principle compression (a diagonal compression failure.

* If $\tau_v < \tau_{max}$ the beam will be safe in diagonal compression also

* The value of $\boxed{\tau_{max}{}^m}$ depends on only grade of concrete

③ $\tau_c$; (Shear strength of concrete without shear reinf$^n$)

The shear stress resisted by concrete i.e. main tension reinf$^n$ (Not compression reinf$^n$). is called design shear strength of concrete ($\tau_c$)

## steps for design of shear Reinf?

① calculate shear force.

$$V_u = \frac{W_u \cdot L}{2} \quad \left[\text{only for S.S. beam}\right]$$

② Nominal shear stress

$$\tau_v = \frac{V_u}{B.d} \not> \tau_{max}.$$

③ calculate % tension reinf$^m$

$$p_t \% = \frac{A_{st}}{B.d} \times 100$$

④ Read $\tau_c$ value from table-19 & 23

⑤ shear force resisted by concrete + Main Reinf$^m$

$$V_c = \tau_c \cdot B.d.$$

⑥ shear force to be resisted by shear reinf$^m$

$$V_s = V_u - V_c$$

⑦ Design of shear reinforcement.

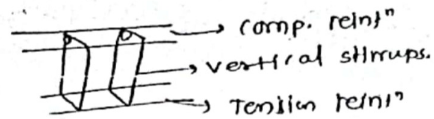
~~three type of shear reinf$^m$~~ can be used.

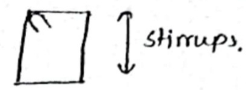
① Vertical shear reinf$^n$

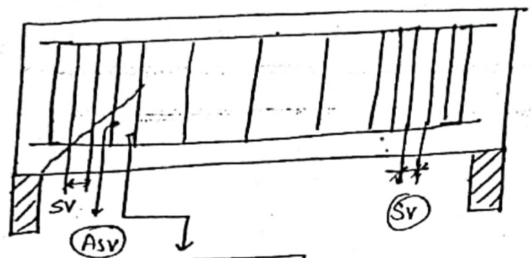
② inclined shear reinf$^m$

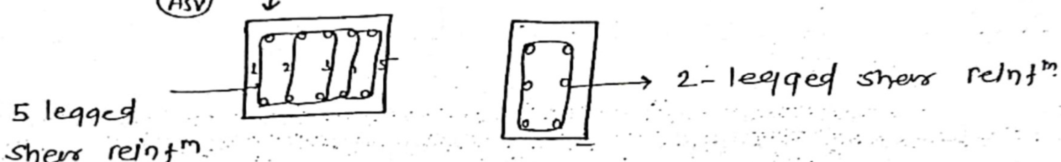
③ Bent-up bars. (along with vertical & inclined 'stirrups)

## (a) Vertical Shear Reinforcement




$$\boxed{A_{sv} = n \times \frac{\pi}{4}(\phi)^2}$$ — Area of shear reint^m at one location.

### total shear force resisted by stirrups

$$V_s = \begin{pmatrix}\text{no. of} \\ \text{stirrups}\end{pmatrix} \times \begin{pmatrix}\text{Area of} \\ \text{steel of} \\ \text{one stirrup}\end{pmatrix} \times (\text{shear strength})$$

$$= \frac{d}{S_v} \times A_{sv} \times (0.87 f_y) \rightarrow \text{LSM.}$$

or $\sigma_{sv} \longrightarrow$ WSM

### Spacing of vertical shear reint^m

$$S_v = \frac{A_{sv}(0.87 f_y).d}{V_s} \quad \text{— LSM}$$

$$= \frac{A_{sv}(\sigma_{sv}).d}{V_s} \quad \text{— WSM}$$

4. **Analysis and design of prismatic sections for shear using LSD.**

**Explanation**

1.I) A rectangular beam of size

$B = 380$ mm
$D = 600$ mm

Provide
$A_{st} = 4 - 25$ mm $\phi$
$A_{sc} = 4 - 20$ mm $\phi$

The beam is subjected to a live load of 46 kN/m over a s.s. clear span 7.60m design suitable shear reinf$^n$ for the beam M30 / Fe415 steel are used.

→ ① $DL = 0.38 \times 0.600 \times 1 \times 25 = 5.70$ kN/m.

$L.L = 46$ kN/m

$TL = 51.70$ kN/m

② Max$^m$ sf $= \frac{W_u . L}{2} = \frac{51.70 \times 7.60}{2} = 294.69$ kN $= V_u$

③ Nominal shear stress $= \tau_v = \frac{V_u}{Bd} = \frac{294.69 \times 10^3}{380 \times 550} = 1.41$ N/mm²

$\tau_{cmax} = 3.30$ N/mm² $\tau_v < \tau_{cmax}$ = OK

④ % of tension Reinf$^m$ $P_t\% = \frac{A_{st}}{Bd} \times 100 = \frac{1963.50}{380 \times 550} \times 100 = 0.94\%$

⑤ $\tau_c$ 0.75 % → 0.59

1.00 % → 0.66

$\tau_c = 0.59 + \frac{0.66 - 0.59}{0.25} \times (0.94 - 0.75)$

$= 0.64$ N/mm².

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⑥ $V_c = \tau_c \cdot B \cdot d$

$= \frac{0.64 \times 380 \times 550}{1000} = \underline{133.76 \text{ kN}}$

⑦ Remaining SF || $V_s = V_u - V_c$

$= 294.69 - 133.76$

$= \underline{160.93 \text{ kN}}$

⑧ spacing of 2L → 8mm ϕ stirrups.

$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53 \text{ mm}^2$

$S_v = \frac{A_{sv} \cdot 0.87 f_y \cdot d}{V_s} = \frac{100.53 \times 0.87 \times 415 \times 550}{160.93}$

$= \underline{124.04} \text{ mm}$

⑨ min. reinform check

$\frac{A_{sv}}{B \cdot S_v} \geq \frac{0.4}{0.87 f_y}$

spacing of same ~~to~~ 2-L 8mm ϕ stirrups.

$S_v = \frac{0.87 f_y \cdot A_{sv}}{0.4 \cdot B} = \frac{0.87 \times 415 \times 100.53}{0.4 \times 380}$

$= 238.6 \text{ mm}$

⑩ max^m spacing

(i) $0.75 \times 550 = 412.5 \text{mm}$

(ii) 300 mm

## 5. Problems on limit state of collapse in bond

**Explanation**

# BOND STRESS AND DEVELOPMENT LENGTH #

① Bond stress

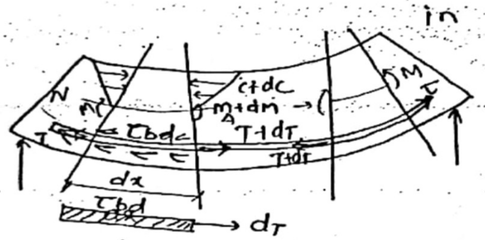
There are two type of bond.

① Flexural bond

② Anchorage bond.

ⓐ Flexural Bond - Bond stress developed b/w steel and concrete due to Bending effect of the section is called flexural bond.

* In an RCC beam as shown in Fig. consider $dx$ length of Reinf$^m$.




$\tau_{bd}$ called Flexural bond stress.

unbalanced tensile force over reinf$^n$.

$= d_T$ force $= (T+d_T) - T = d_T$

Bond stress Is developed due to this unbalanced $d_T$ force. If bond stress developed $= \tau_{bd}$

$$\therefore \quad d_T = \tau_{bd} \cdot (n \cdot \pi \cdot \phi) \cdot dx.$$

Bond stress =

$$\tau_{bd\,(dev)} = \frac{d_T}{n \cdot \pi \phi \cdot dx} = \frac{dM}{(n \pi \phi) \cdot j \cdot d \cdot dx}$$

$$M = C \cdot j \cdot d = T \cdot j \cdot d$$

$$M + dM = (C + dC) \cdot jd = (T + dT)\, j \cdot d$$

$$dM = dT \cdot j \cdot d.$$

$$dT = \frac{dM}{j \cdot d}$$

$$\tau_{bd} = \frac{V}{(\pi \cdot n\phi) \cdot j \cdot d}$$

$$\boxed{\tau_{bd} = \frac{V}{\Sigma O \cdot j \cdot d}}$$

$\Sigma O$ = sum of perimeter $(n\pi \cdot \phi)$

# The value of bond stress developed shall not be more than the permissible values.

for mild steel (in tension)

| $\tau_{bd\,(per)}$ | M20 | M25 | M30 | M35 | M40 | M45 | M50 |
|---|---|---|---|---|---|---|---|
| WSM | 0.8 | 0.9 | 1.0 | 1.1 | 1.2 | 1.3 | 1.4 |
| LSM | 1.2 | 1.4 | 1.5 | 1.7 | ← | 1.90 | → |

* For HYSD / CTD / TMT Bars ⇒ increase by 60%.
* For Bars in compression → increase by 25%.

Example

In LSM :- fc M30 / fe 415

① $\tau_{bd}$ (Per) for bars in tension

$$= 1.50 \times 1.60 = 2.40\ N/mm^2$$

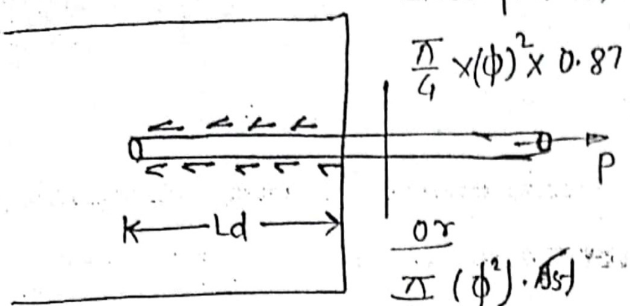
② $\tau_{bd}$ (per) for bars in comp.

$$= 1.50 \times 1.25 \times 1.60 = 3.0\ N/mm^2.$$

# Development length

Strength in Tension

$\frac{\pi}{4} \times (\phi)^2 \times 0.87\, f_y$.




or

$\frac{\pi}{4} (\phi^2) \cdot \sigma_{st}$

Strength in bond

$\tau_{bd} \cdot \pi \times \phi \cdot L_d$.

Development length is the minimum length of reinf$^m$ required to be kept within concrete; so that the strength of reinf$^m$ in bond is not less than the strength of reinf$^m$ in tension.

# For $L_D$

Strength in Bond $\geq$ Strength In Tension

$$\tau_{bd} \cdot \pi \cdot \phi \cdot L_d = \frac{\pi}{4} (\phi)^2 (0.87 f_y)$$

$$\boxed{L_d = \frac{\phi \cdot 0.87 f_y}{4 \cdot \tau_{bd}}}$$

here

$\tau_{bd}$ = Permissible

**6.** Analysis and design of one-way slabs using LSM.

## Explanation

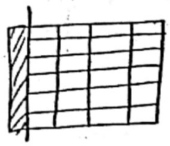
# Design of SLAB #

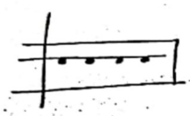
# Types

① one way

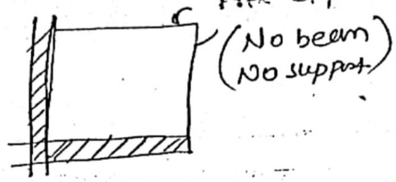
② Two way

③ cantilever slab



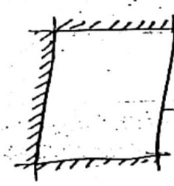


④




Two side supported slab
(adjucent side)

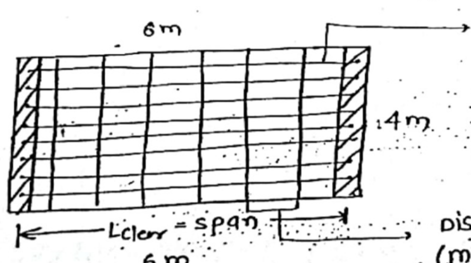
⑤



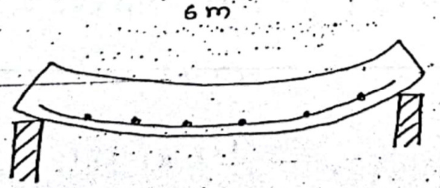

Three side support slab

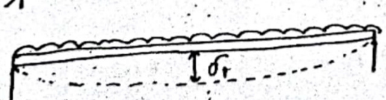
I] ONE WAY SLAB

Case-I




Distribution reint$^{m}$
(min. reint$^{m}$) = [0.12 or 0.15% of BD]

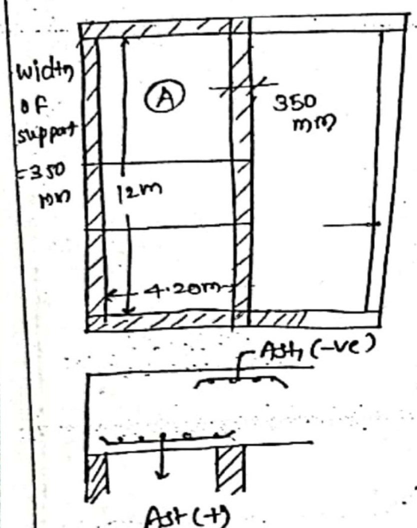




Q.] Design a slab as shown in fig. worked as

(A) Live load over slab is 8 kN/m² 60 mm flooring (marble) is provided over the slab (unit wt 24 kN/m) Use M30 concrete | Fe 415 steel use LSM.

29/Nov./2016




① LL = 8 kN/m²

Flooring = 40 mm

M30 – 415

$L_x$ short span = 4.20m

$L_y$ = 12 m

$\frac{L_y}{L_x} = \frac{12}{4.2} = 2.86 > 2$

∴ It is an one-way slab.

② depth of slab

Effective depth $d = \frac{span}{26} = \frac{4.200}{26} = 161.54$ mm.

consider $d = 170$ mm

Eff. cover $D = 30$ mm

$D = 200$ mm.

② Load over slab (over 1m × 1m)

1) DL self wt $= 0.20 \times 1.0 \times 1.0 \times 25 = 5.00$ kN/m

2) flooring $= 0.06 \times 1.0 \times 1.0 \times 24 = 1.44$ kN/m

1.44 kN/m

$W_u = 1.5 \times 6.44 = 9.66$ kN/m.

iii) Live load $= 8 \times 1 \times 1 = 8$ kN/m

$W_{Lu} = 1.5 \times 8$
$= 12.0$ kN/m

⑤ Effective span

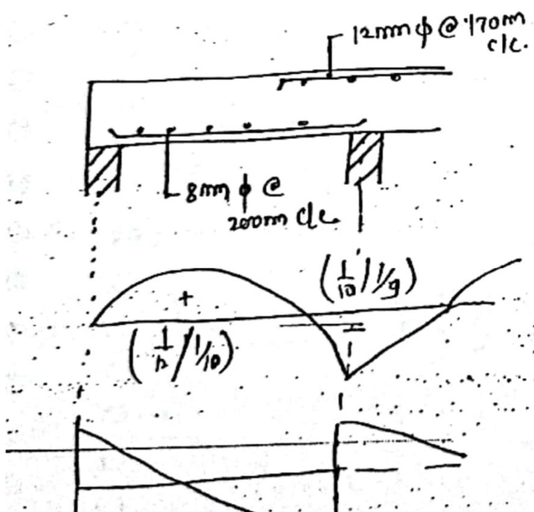
width of support $W = 350$ mm

$$\frac{L_{clear}}{12} = \frac{4200}{12} = 350 \text{ mm}$$

$\left(\frac{W}{12} = \frac{L_0}{12}\right)$ consider same as simply supported.

$L_{eff} = L_{clear} + d = 4.2 + 0.17 = 4.37$ m

$= L_{clear} + W = 4.2 + 0.35 = 4.55$ m




⑥ Bending moments

ⓐ $max^m$ -ve BM

$$M_{x(+)} = \frac{1}{12} W_{du} \cdot L_e^2 + \frac{1}{10} W_{Lu} \cdot L_e^2$$

$$= \frac{9.66 \times 4.37^2}{12} + \frac{12 \times 4.37^2}{10}$$

$= 38.30$ kN.m

ⓑ $max^m$ (-ve) Bond

$$M_{x(-)} = -\left(\frac{W_{du} \cdot L_x^2}{10} + \frac{W_{Lu} \cdot L_x^2}{9}\right)$$

$$= -\left[\frac{9.66 \times 4.37^2}{10} + \frac{12 \times 4.37^2}{9}\right] = 43.91 \text{ kN.m}$$

⑤ Depth Required.

$$d = \sqrt{\frac{M_{u.max}}{Q \cdot B}} = \sqrt{\frac{43.91 \times 10^6}{0.138 \times 30 \times 10^3}} = 103 \text{ mm}$$

< 170 mm provided

OK

keep D = 200 mm
d = 170 mm

⑥ Area of steel required

$$A_{st}(+) = \frac{0.5 \times 30}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 38 \times 90 \times 10^6}{30 \times 1000 \times 170^2}}\right] \times 1000 \times 170$$

$$= 660 \text{ mm}^2$$

$$A_{st}(-) = \frac{0.5 \times 30}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 43.91 \times 10^6}{30 \times 1000 \times 170^2}}\right] \times 1000 \times 170$$

$$= 764 \text{ mm}^2$$

⑦ spacing of main Reinf$^m$

no. of bars required $n = \frac{A_{st}}{\frac{\pi}{4} \times \phi^2}$

Spacing of bars $= \frac{1000}{n} = \frac{1000}{A_{st}} \times \frac{\pi}{4} \times \phi^2$

(+ve) moment Reinf$^m$

Spacing $(A_{st}(+)) = \frac{1000}{660} \times \frac{\pi}{4} \times (12)^2 = 171$ mm

Provide 12mm @ 170 c/c



Reinf$^m$

(-)ve moment reinf$^m$

$$\text{spacing} = \frac{1000}{764} \times \frac{\pi}{4} \times 12^2 = 148\text{mm}$$

provide 12mm $\phi$ @ 140mm c/c.

(8) Distribution Bars

$$A_{st\,min} = \frac{0.12}{100} \times BD$$

$$= \frac{0.12}{100} \times 1000 \times 200$$

$$= 240\text{mm}^2.$$

Spacing of 8mm $\phi$

$$= \frac{1000}{240} \times \frac{\pi}{4} \cdot (8)^2$$

$$= 209\text{ mm}$$

provide 8 mm @ 200mm c/c

9) Check for shear

Considering 50% reinf$^m$ are curtained and only 50% $A_{st}(+)$ are considered upto support.

(a) At simply supported edge

$$V_{u1} = 0.40 \times W_{du} \cdot L_e + 0.45 \times 12 \times 4.20$$

$$= 38.91\text{ kN}$$

$$\tau_{vu} = \frac{V_u}{Bd} = \frac{3891 \times 1000}{1000 \times 170} = 0.23$$

Check

$$p_t\% = \frac{A_{st}\ x(+)/2}{BD} \times 100 = \frac{660/2}{1000 \times 170} \times 100 = 0.19\%$$

$\tau_c = 0.15\% \quad \to 0.29$

$0.25\% \quad - 0.37$

$$\tau_c(0.19\%) = 0.29 + \left(\frac{0.37 - 0.29}{0.10}\right) \times (0.19 - 0.15)$$

$\tau_c = 0.32$

$k = 1.20$

$k \cdot \tau_c = 1.20 \times 0.32 = 0.38\ N/mm^2$

$\tau_{vu} < k \cdot \tau_c$ ... so safe

(b) At contineous edge.

$V_{u2} = 0.60 \times 4.66 \times 4.20 + 0.60 \times 12 \times 4.20$

$= 54.58$ kN

$$\tau_{vu} = \frac{54.58 \times 1000}{100 \times 170} = 0.32$$

$$p_t\% = \frac{A_{st}\ x(-ve)}{BD} \times 100 = \frac{764}{1000 \times 170} \times 100 = 0.45\%$$

$\tau_c \quad 0.25\% \longrightarrow 0.37$

$0.50\% \longrightarrow 0.50$

for :

$$0.49\% = 0.37 + \left(\frac{0.50 - 0.37}{0.25}\right) \times (0.45 - 0.25)$$

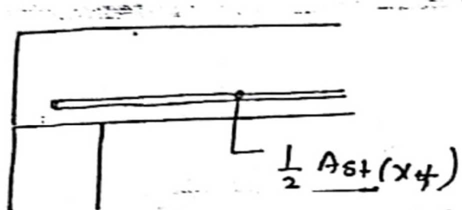
$= 0.474$

$k = 1.20$

$k \cdot \tau_c = 0.57 \ N/mm^2$

$(\tau v_u < k \cdot \tau_c)$ -- safe

(10) Check for Ld at simply supported edge.




12 mm @ 340 mm c/c.

$= 330 \ m^2$

$$L_d = \frac{\phi \cdot 0.87 f_y}{4 \cdot \tau_{bd}} =$$

$$= \frac{12 \times 0.87 \times 415}{4 \times 1.50 \times 1.60} = \underline{451.3 \ mm}$$

$\boxed{M_{u1}}$ : $$x_u = \frac{0.87 \times 415 \times 330}{0.36 \times 30 \times 1000} = 11.06 \ mm$$

$$M_u = 0.87 f_y \cdot A_{st} (d - 0.42 \ x_u)$$

$$= 0.87 \times 415 \times 330 \times [170 - 0.42 \times 11.06]$$

$$= \underline{19.70 \ kN \cdot m}$$

$V_u = V_{u1} = 38.9 \ kN.$

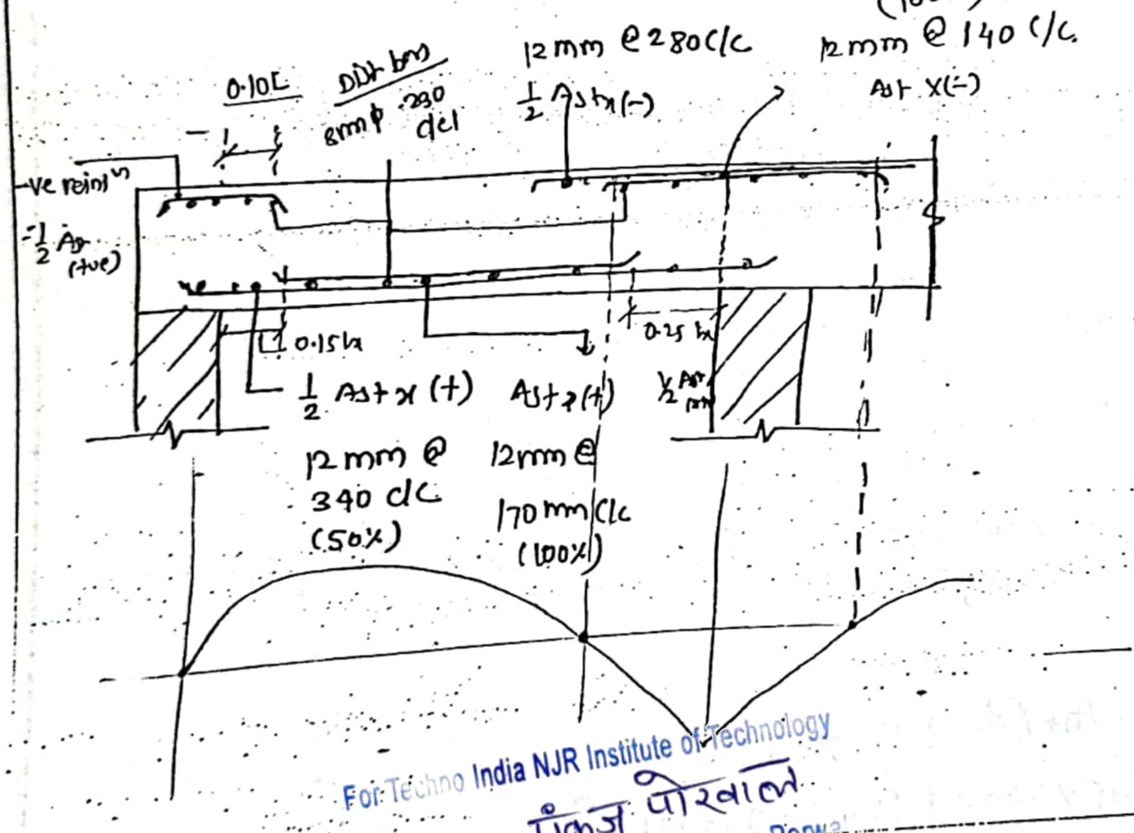
$$\frac{M_{u1}}{V_u} = \frac{19.70 \times 10^6}{38.9 \times 10^3} = 506.30\,mm$$

$\therefore$ safe.

$$L_0 = \frac{l_x}{2} - x' = \left(\frac{350}{2} - 30\right) = 145\,mm$$

$$\left(\frac{M_{u1}}{V_u} + L_0\right) = 506 + 145 = 651\,mm.$$

$$L_d < \left(\frac{M_{u1}}{V_u} + L_0\right) \text{ — safe}$$



# 7. Analysis and design of two-way slabs using LSM.

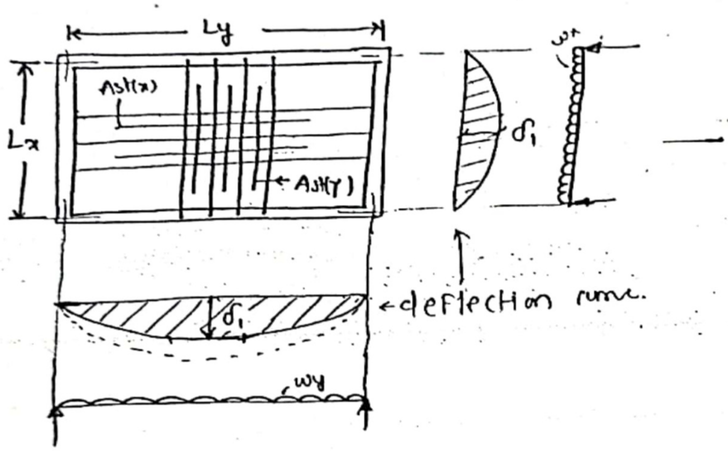
## Explanation

## TWO-WAY SLAB

# Where a slab is supported on all four sides and span ratio

$$\boxed{\frac{L_y}{L_x} \leq 2.0}$$

It is called two way slab.



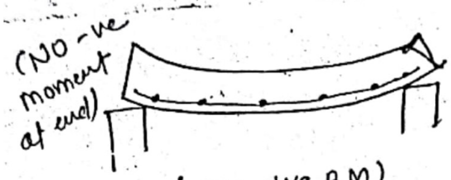

In this case moments are observed in ~~depth~~ both direction.

$$\left.\begin{array}{l} M_{ux} = ? \Rightarrow A_{st}(x) \\ M_{uy} = ? \rightarrow A_{st}(y) \end{array}\right\} \text{steel provide}$$

* There are different methods for design.

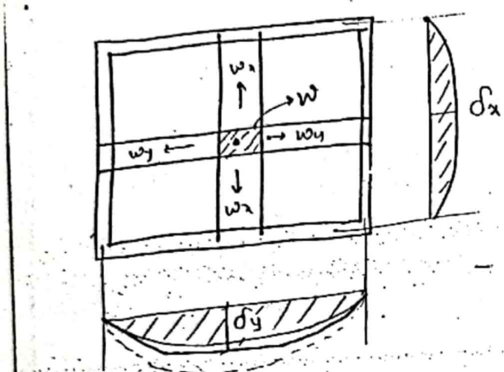
1. Rankine Grashoff Theory

(for simply supported slab with edges and corners free to lift) (NOT Restrained) (Not prevented from lifting)




* If edges are free to lift NO ~~get~~ negative BM are developed at edges and corner.
* Slab shall be designed for only +ve BM.

$\delta x$ -




$\delta_x = \delta_y$

$$\frac{5}{384}\cdot\frac{w_x \cdot L_x^4}{EI} = \frac{5}{384}\cdot\frac{w_y \cdot L_y^4}{EI}$$

$$\frac{w_x}{w_y} = \left(\frac{L_y}{L_x}\right)^4 = r^4$$

$$\therefore \frac{L_y}{L_x} = r$$

$$\frac{w_x}{w_y} = r^4$$

$$= \boxed{w_x = r^4 w_y}$$

## Total load

$$w = w_x + w_y$$
$$= r^4 w_y + w_y$$
$$= (r^4+1)\, w_y$$

$$w_y = \frac{1}{(1+r^4)}\, w$$

$$w_x = \left(\frac{r^4}{1+r^4}\right)\cdot w$$

## Moment formula

$$M_x = \frac{w_x \cdot L_x^2}{8} = \left(\frac{r^4}{1+r^4}\right)\cdot w \cdot \frac{L_x^2}{8} \quad \text{—(1)}$$

$$M_y = \frac{w_y \cdot L_y^2}{8} = \left(\frac{1}{1+r^4}\right) w \cdot \frac{L_y^2}{8} \quad \text{—(2)}$$

## III] I.S. Code Method

### (a) Design of simply supported slab

( edges and corners not prevented from lifting.) and adequate provision to resist torsion are not made.

moment $M_x = \alpha_x . w . L_x^2$

formula $M_y = \alpha_y . w . L_x^2$

value of $\alpha_x$ and $\alpha_y$ are to be read from T-27

| $l_y/l_x$ | 1.0 | 1.1 | 1.2 | 1.3 | 1.4 | 1.5 | 1.75 | 2.0 |
|---|---|---|---|---|---|---|---|---|
| $\alpha_x$ | 0.062 | 0.074 | 0.084 | | | | | |
| $\alpha_y$ | 0.062 | 0.061 | 0.059 | | | | | |

### (b) When corners and edges are prevented from lifting

The slab (any type) may be designed as per D.1.1 to D1.11

D.1.1 Moments for a two way slab

$$M_x = \alpha_x . w . L_x^2 \quad \text{———} \ (1)$$

$$M_y = \alpha_y . w . L_y^2 \quad \text{———} \ (2)$$

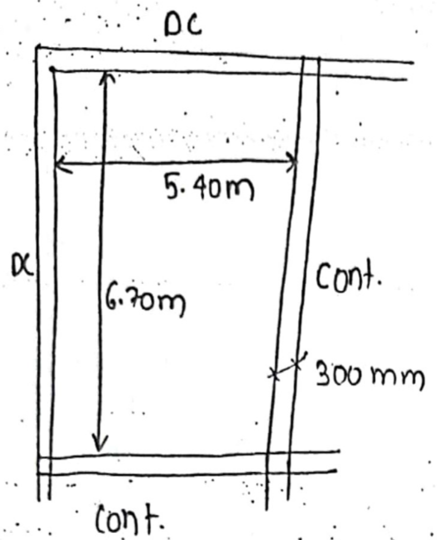
The value of $\alpha_x$ & $\alpha_y$ are to be read from ~~T-27~~ T-26

Q.3] Design a slab as shown in fig. subjected to

Live load - 6 kN/m²

SI. Dead load - 3 kN/m²

Flooring = 1.5 kN/m².




Consider :

M25 concrete

fe 500 steel

Use - IS code Method

T-27

① $\frac{Ly_0}{Lx_0} = \frac{6.70}{5.40} = 1.24 < 2$

→ Two way slab.

Two way slab

$\frac{\text{span}}{\text{overall depth}}$

| | Mild steel | HYSD |
|---|---|---|
| Simply supported | 35 | 28 |
| continuous | 40 | 32 |

Assume depth

overall depth - $\frac{\text{span}}{32}$

$D = \frac{5400}{32} = 168.7$

say 170 mm

Effective depth = $d = 170 - 30 = 140$ mm

2] Load calculation

$DL = 0.17 \times 1 \times 1 \times 25 = 4.25$ kN/m$^2$

$LL = 6 \times 1 \times 1 = 6.$

$S.F.PL = 3 \times 1 \times 1 = 3$

Flooring $= 1.5 \times 1 \times 1 = 1.50$

total $= 14.75$ kN/m$^2$

$W_u = 1.5 \times 14.75 = 22.125$ kN/m$^2$.

3] Effective span

width of support $W = 300$ mm

$$\frac{L_o}{12} = \frac{5400}{12} = 450 \text{ mm}$$

$\left(W < \frac{L_o}{12}\right) \rightarrow$ same as simply supported.

$L_x = L_{xc} + d = 5.4 + .17 = 5.54$ m $\}$ min$^m$ = 5.54 m

or $L_{xc} + W = 5.4 + 0.30 = 5.7$ m

$L_y = L_{x} + d = 6.7 + 0.14 = 6.84$ m.

$$r = \frac{L_y}{L_x} = \frac{6.84}{5.54} = 1.23$$

| moment | $\alpha$ | moments $\alpha \cdot W \cdot L_x^2$ | d | Ast | spacing dia | required | spacing dia | provi. |
|---|---|---|---|---|---|---|---|---|
| $M_{ux}(-)$ | $\alpha_x(-) = 0.062$ | 42.10 | 140 mm | 778 | 10 | 100.95 | 10 | 100 |
| $M_{ux}(+)$ | $\alpha_x(+) = 0.046$ | 31.24 | 140 mm | 558 | 10 | 140.75 | 10 | 140 |
| $M_{uy}(-)$ | $\alpha_y(-) = 0.047$ | 31.92 | 140 mm | 571 | 10 | 137 | 10 | 130 |
| $M_{uy}(+)$ | $\alpha_y(+)$ 0.035 | 23.77 | 130 mm | 452 | 10 | 173 | 10 | 170 |

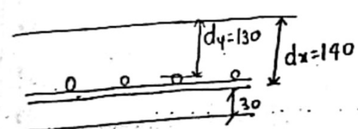
4] moments

$$M = \alpha \cdot W \cdot L_x^2$$

$$= \cancel{\alpha \times 222}$$

$$= \alpha \times 22.125 \times 5.54^2$$

$$= 679.05\,\alpha$$




5] Depth required

$$d = \sqrt{\frac{M_{u\,max}}{Q \cdot B}}$$

$$= \sqrt{\frac{42.10 \times 10^6}{0.134 \times 25 \times 1000}}$$

$$= 112.1\ mm < 140\ mm$$

6] $A_{st} = \dfrac{0.5 \times 25}{500} \times \left[1 - \sqrt{1 - \dfrac{4.6 \times 4210 \times 10^6}{25 \times 1000 \times 140^2}}\right] \times 1000 \times 140$

$$= 778\ mm^2$$

=

7] Spacing

$$= \frac{1000}{A_{st}} \times \frac{\pi}{4} \times \phi^2 = \frac{1000}{778} \times \frac{\pi}{4} \times 10^2 = 100.95$$

8] Distribution bars $= \frac{0.12}{100} \times 1000 \times 170 = 204 \text{ mm}^2$

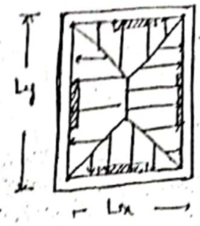
spacing of 8mm $= \frac{1000}{204} \times \frac{\pi}{4} \times 8^2 = 246.4$ mm

provide 8 mm @ 240 mm c/c.

9] Check for shear

$V_{u_2} = \left(\frac{W_u \cdot L_x}{3}\right)$ ... for $\frac{1}{2}$ Asty (t)

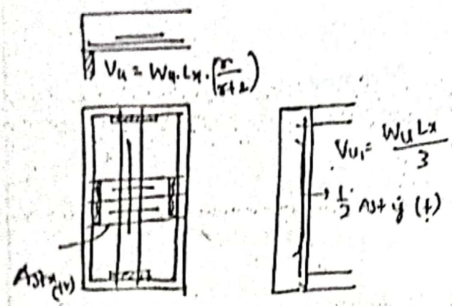
$p_t = \frac{\frac{1}{2} Asty(t)}{Bd} \times 100$




$V_{u_1} = W_u L_x \left(\frac{r}{2+r}\right)$

for $\frac{1}{2}$ Astx (t)

$p_t = \frac{\frac{1}{2} Astx(t)}{Bd} \times 100$




(a) for x-direction

$V_{u_1} = W_u . L_x . \frac{r}{2+r} = 22.125 \times 5.40 \times \frac{1.23}{2+1.23} = 45.5$ kN

$\tau_{vu} = \frac{45.5 \times 1000}{1000 \times 140} = 0.325$ N/mm$^2$

$P_{xt} = \frac{Astx(t)/2}{Bd} \times 100$

$= \frac{558/2}{1000 \times 140} \times 100$

$= 0.20$ %

$\tau_c = 0.15 \rightarrow 0.29$

$0.25 \rightarrow 0.36$ $\quad\quad$ $\tau_c = 0.325$

$k = 1.25$

$k.\tau_c = 1.25 \times 0.325 = 0.41\ N/mm^2$ $\quad \therefore$ safe.

$\tau_v < k.\tau_c$.

ⓑ For y-direction

$$V_{u_2} = \frac{W_u . L_x}{3} = \frac{22.125 \times 540}{3} = 39.83\ kN.$$

$$\tau_v = \frac{39.83 \times 1000}{1000 \times 130} = \frac{V_{u2}}{B.d_y} = 0.31\ N/mm^2.$$

$$P_t\% = \frac{A_{sty}(+)/2}{B.d_y} \times 100 = \frac{452/2}{1000 \times 130} \times 100 = 0.17\%.$$

$$\tau_c = 0.29 + \frac{0.36 - 0.29}{0.10} \times 0.002$$

$$= 0.304\ N/mm^2.$$

$K = 1.25$

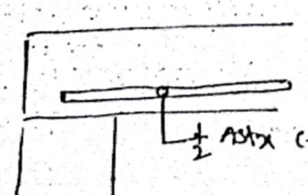
$k.\tau_c = 1.25 \times 0.304 = 0.38\ N/mm^2$

$\therefore$ safe

$\tau_v < k.\tau_c$.

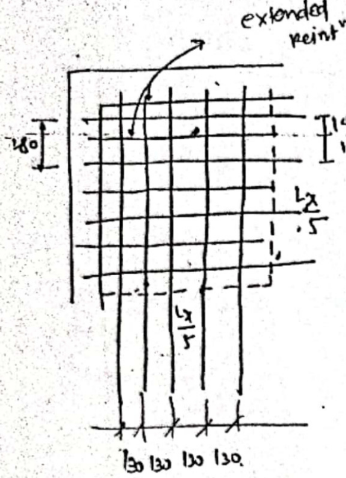
10.] check for development length.

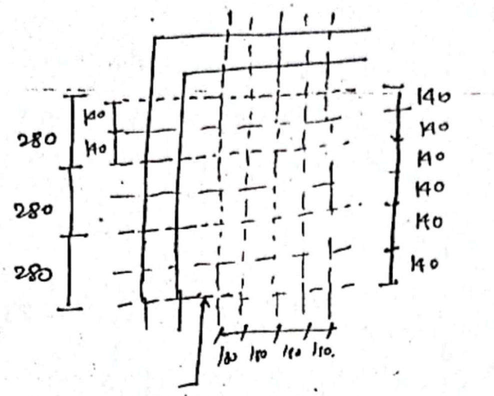
ⓐ for x-direction Reinf$^m$



$\frac{1}{2} A_{stx}(+) = 553/2$ $\quad V_u = 455\ kN.$

$$\text{spacing} = \frac{1000}{209.25} \times \frac{\pi}{5} \times 10^2 = 375 \text{ mm.}$$



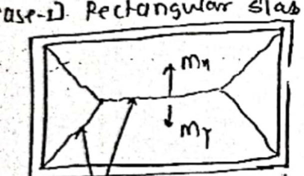

$\frac{1}{2}$ Ast x (+)

$= \frac{1}{2} \times 558$

$= 2$

# Yield Line Theory

Case-I Rectangular slab

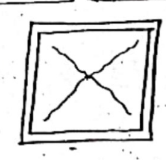



Yield Lines

(1st crack developed and its movement)

Imaginary cracks developed in a slab perpendicular to max^m moment direction are called Yield stress.

Case-II Square

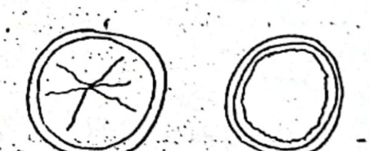


max^m moment capacity as per yield line theory

$$\boxed{M_u = \frac{W_u \cdot L^2}{24}}$$

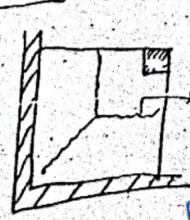
max^m moment

(case-iii)

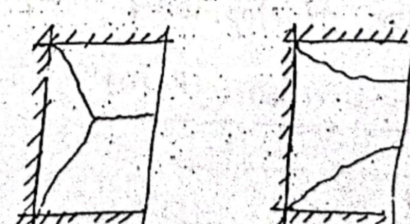


Bottom face  Top face

$\boxed{\text{case - IV}}$




Case-IV



Three side supported slab

Top face

**8. Analysis and design of short axially loaded columns.**

**Explanation**

20/11/2016

# Design of column by LSM

39.1 | p-70 | IS-code - 456

Assumptions

all assumption 38.1 ⓐ to ⓔ are also valied for. column (compression Member.)

38.1]

ⓐ strain diagram is linear.

ⓑ The max$^m$ strain in concrete. in bending compression at extreme fiber = 0.0035.

ⓒ stress in concrete (design) = 0.45 fck

ⓓ Tensile strength of concrete ignored.

ⓔ The stresses in reinforc$^m$ to be read from stress-strain curve partial F.O.S = 1.15

tension steel = 0.87 fy.

compression steel = Fsc.

from table

In addition following assumption.

39.1 ⓐ. The max$^m$ strain in concrete in direct compression = 0.002.

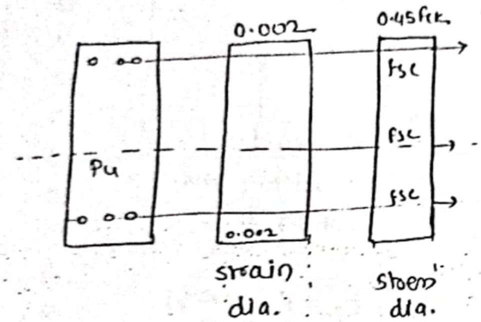
ⓑ the max$^m$ strain at highly compressed extreme fibre. in case the section is ~~be~~ subjected to axial comp. and bending and when there is no tension in section.

For Techno India NJR Institute of Technology
पंकज पोरवाल
Dr. Pankaj Kumar Porwal
(Principal)

= 0.0035 - 0.75 times strain at least compressed extreme fiber.

$$\boxed{\varepsilon_{hc} = 0.0035 - 0.75\ \varepsilon_{c}}$$

case-i) Axial compression only (NA at infinity)




column subjected to axial comp. only

strain in concrete = 0.002

stress in concrete = 0.45 fck

* Load taken by concrete:

$C_1 = 0.45\ f_{ck} \times A_c$

$C_1 = 0.45\ f_{ck}\ (BD - A_{sc})$

* Strain in steel = 0.002

* Stress in steel

mild steel = $0.87 f_y$ = 217.5 N/mm²

* fe415 $f_{sc}$ = 330 nearly = $0.79 f_y$. } $0.75 f_y$.

fe 500 $f_{sc}$ - 375 nearly = $0.75 f_y$ }

$$\boxed{f_{sc} = 0.75\ f_y}$$

- Load taken by steel

$C_2 = 0.75 f_y \cdot A_{sc}$

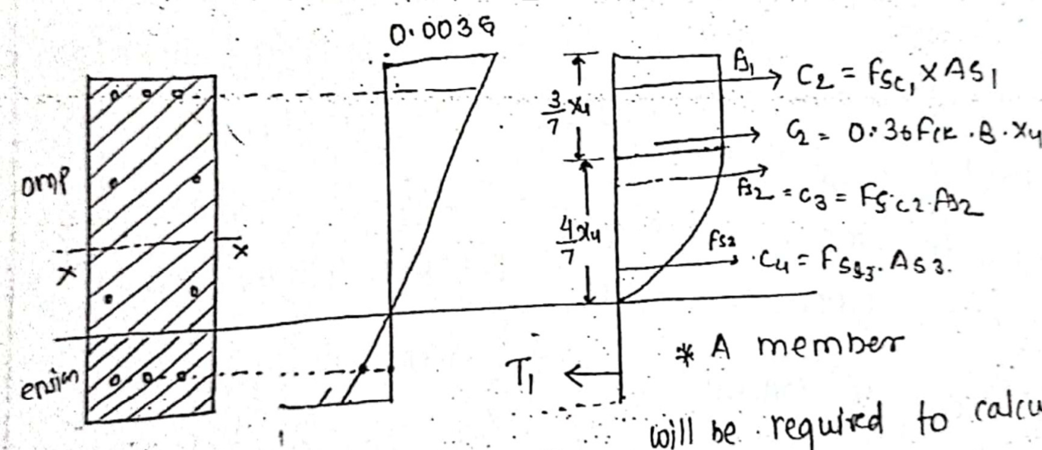
Ultimate ($max^m$) load carrying capacity of column (Axial)

$C_1 = C_1 + C_2$

$$\boxed{P_{u2} = 0.45 f_{ck} \cdot A_c + 0.75 f_y \cdot A_{sc}.}$$

** If No moment effect is considered. **

Case-2 - column subjected to axial load + Bending (When tension is developed). (NA within the section)




* A member will be required to calculate $\varepsilon_{sc}$ and $f_{sc}$ values for diff. steel.

$\boxed{\text{Solution is lengthy.}}$

## 9. Analysis and design of footing

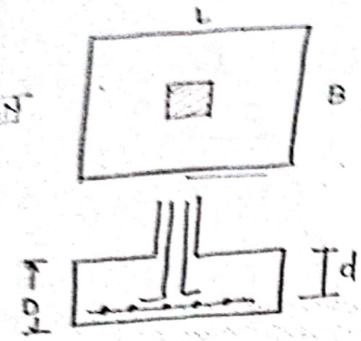
# FOUNDATION

⇒ Types

i) Isolated footing

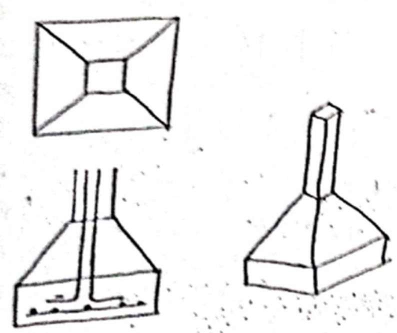
→ isolated (shape)
- → Rectangular
- → square
- → circular

PLAN




→ Thickness
- (a) uniform thickness
- (b) sloped footing.

b) sloped footing



# size of footing

Load from column = P

Weight of footing = 0.2 P

( 10% to 20% of P )

$\boxed{\text{total load } P_T = 1.2\,P}$

B = 3 m

length $L = \frac{15}{3} = 5$ m

* total soil pressure $= \frac{P_T}{A} = \frac{1800}{15} = 120$ kN/m$^2$

* pressure balanced $= \frac{0.2P}{A} = \frac{0.2 \times 1500}{15} = 20$ kN/m

* Net soil pressure —

= ~~p0~~ — total soil pressure − pressure balanced

= 120 − 20

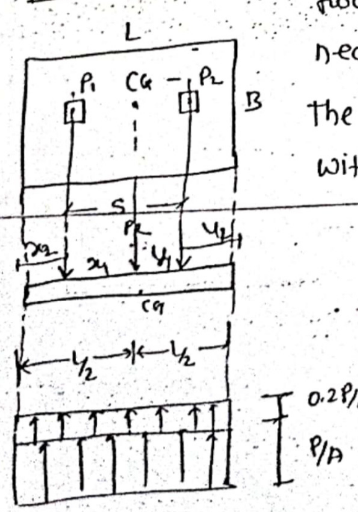
= 100 kN.

01/12/2016

2) Combined footing →

if $\boxed{P_2 > P_1}$

A combined footing is provided for two columns. (if $P_2 > P_1$). Projection near column are adjusted such that the C.G. of footing is matching with the CG of Load on column.



* CG of Footing is matched with CG of Load to keep the soil pressure uniform below footing

taking moments of loads

$$P_1 \cdot 0 + P_2(x_1+y_1) = P_R \cdot x_1$$

$$x_1 = \frac{P_2(x_1+y_1)}{P_R}$$

$$x_1 = \frac{P_2(x_1+y_1)}{(P_1+P_2)}$$

$$y_1 = (s - x_1)$$

projection near footing

$$x_2 = \frac{l}{2} - x_1$$

$$y_2 = \frac{l}{2} - y_1$$

#) Area of Footing

Load from column $= (P_1+P_2) = P_R$

Wt of Footing $= 0.2\, P_R$

Total $= 1.2\, P_R$

$$\text{Area} = \frac{P_T}{q_0} = \frac{1.20\, P_R}{q_0} = \frac{1.20\,(P_1+P_2)}{q_0}$$

$\boxed{\text{Length} = \frac{A}{B}}$ $\boxed{A = L \times B}$

Q-3] Design a rectangular footing for a column of size 300mm x 600mm subjected to a load of 1200kN. safe BC of Soil is 90 kN/m². Consider width of footing = 3.40m along shorter size of column use M30 | Fe 415 steel Use LSM.

→ Column size = 300mm x 600mm

① Size of footing

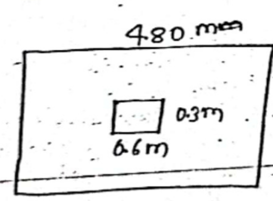
Load from column (P) = 1200 kN.

foundation wt $= 0.20 \times P = 0.20 \times 1200 = 240$ kN

Total = 1440 kN.

Area $A = \frac{P_T}{q_o} = \frac{1440}{90} = 16\ m^2$

$W = 3.4m$ so $L = \frac{A}{W} = \frac{16}{3.4} = 4.71m.$ say 4.8m




Soil pressure

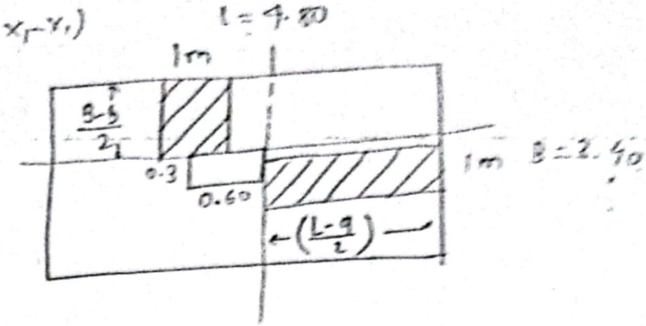
$w_o = \frac{1200}{4.8 \times 3.4} = 73.53\ kN/m^3$

For LSM

$w_{uo} = 1.5 \times w_o = 1.5 \times 73.53 = 110.29\ kN/m^2$.

2] Check for Bending moment

(a) moment Mux (about $x_1$-$x_1$)




$$M_{uy} = W_{uo} \times 1m \times \frac{(B-b)^2}{8}$$

$$= 110.3 \times 1m \times \frac{(3.40-0.30)^2}{8}$$

$$= 132.50 \text{ kN.m}$$

(b) moment Muy (about $Y_1$-$Y_1$)

$$M_{uy} = W_{uo} \times 1m \times \frac{(L-a)^2}{8}$$

$$= 110.3 \times 1m \times \frac{(4.80-0.60)^2}{8}$$

$$= 243.2 \text{ kN.m}$$

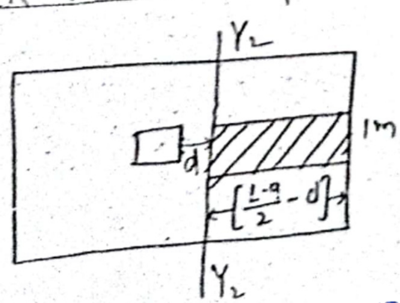
(c) depth of footing moment

$$d = \sqrt{\frac{M_{u\,max}}{Q \cdot B_1}} = \sqrt{\frac{243.2 \times 10^6}{138 \times 30 \times 1000}} = 242.37 \text{ mm}$$

say = 250 mm

$$d = 0.25 \text{ m}$$

3] Check for one way shear




max^m shear force at $Y_2$-$Y_2$

$$V_u = W_{uo} \times 1m \left(\frac{L-a}{2} - d\right)$$

$$= 110.3 \times 1 \left(\frac{4.8-0.6}{2} - 0.25\right)$$

$= 110.3 \times (2.10 - 0.25)$

$= \underline{204.055 \text{ kN}}$

$\tau_{vu} = \dfrac{V_u}{B.d} = \dfrac{204.055 \times 10^3}{1000 \times 250} = 0.81$

$> k.\tau_c$

$> 1.0 \times 0.29 \rightarrow$ failed

<u>consider</u> $d = 600\text{mm}$

$V_u = 110.3 \times (2.10 - 0.6)$

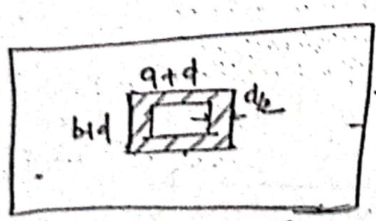
$= 165.45 \text{ kN}$

$\tau_{vu} = \dfrac{V_u}{B.d} = \dfrac{165.45 \times 10^3}{10^3 \times 600} = 0.275 \text{ N/mm}^2$

$< 0.29 \text{ N/mm}^2 \ldots \underline{\text{OK}}$

safe.

IV] <u>check for punching shear</u> →




<u>Punching shear stress developed</u>

$\tau_{vp(dev)} = \dfrac{P_u - w_{uo} \times (a+d)(b+d)}{2 \times [(a+d) + (b+d)] \times d} \text{ N/mm}^2$

$= \dfrac{1.5 \times 1200 - 110.29 \times (0.60 + 0.60)(0.3 + 0.6)}{2 \times [(0.6 + 0.6) + (0.3 + 0.6)] \times 0.6}$

$= 0.67 \text{ N/mm}^2$

$\tau_{vp(per)} = k_p \times 0.25\sqrt{f_{ck}}$

$k_p = (0.5 + \frac{b}{a}) \times 0. \quad (0.5 + \frac{300}{600}) = 0.5 + 0.5 = 1.0$

$\tau_{vp(per)} = 1.0 \times 0.25\sqrt{30} = 1.37 \ N/mm^2$.

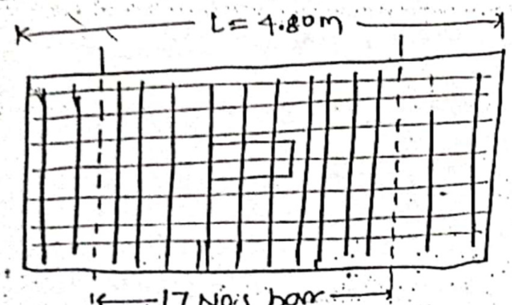
$\tau_{vp(dev)} < \tau_{vp(perm)}$ — safe.

provide $d = 600\,mm$

cover = $80\,mm$

$D = 680\,mm$

V] ⓐ For Mux [Shorter Bars]




i) Area of steel required for 1m width

$$A_{st} = \frac{0.5 \times 30}{415} \times \left[1 - \sqrt{1 - \frac{4.6 \times 132 \times 10^6}{30 \times 1000 \times 600^2}}\right] \times 1000 \times 600$$

$= 621 \ mm^2$.

$Ast_{min} = \frac{0.12}{100} \times BD = \frac{0.12}{100} \times 1000 \times 680 = 816 \ m^2$.

ii) For total width - $l = 4.80m$

Area of steel $= 4.80 \times 816 = 3917 \ mm^2$

iii) No. of bars 16 mm $\phi$.

$$= \frac{3917}{\frac{\pi}{4}(16)^2} = 19.50 \quad \text{say } 20 \text{ Nos} - 16 \text{ mm}\phi.$$

iv) No. of bars req. in central Band.

$$n_c = n_t \times \frac{2}{(1 + L/B)}$$

$$= 20 \times \frac{2}{\left(1 + \frac{4.8}{3.4}\right)} = 16.6 \quad \text{say } 17 \text{ bars.}$$

v) No. of bars in side band

$$n_s = \frac{20-17}{2} = 1.5 \quad \text{say } 2 \text{ bars.}$$

b] <u>For Muy</u>

$Muy = 243.5 \ kN \cdot m$

i) Area of steel for 1m.

$$A_{st} = \frac{0.5 \times 30}{415} \times \left[1 - \sqrt{1 - \frac{4.6 \times 243.5 \times 10^6}{30 \times 1000 \times 600^2}}\right] \times 1000 \times 600$$

$= 1154 \ mm^2 > A_{st\,min}$ <u>ok</u>

ii) for total width $B = 3.4m$

Area of steel $= 3.4 \times 1$[illegible]$24 \ mm^2$.

iii) No. of 16 mm $\phi$ = $\dfrac{3924}{\frac{\pi}{4}(16)^2}$ = 19.5 ≈ 20 Nos. bars.

vi] check for Bearing.

$$f_b = \frac{P_u}{a \times b} \not> 0.45\, f_{ck} \sqrt{\frac{A_1}{A_2}}$$

$$= \frac{1.5 \times 1200}{(0.6 \times 0.3) \times 10^3} \qquad 0.45 \times 30 \times \sqrt{\frac{A_1}{A_2}}$$

$$\not> 13.5\ N/mm^2 \times \sqrt{\frac{A_1}{A_2}}$$

$$= 10\ N/mm^2$$

$A_1 = L_1' \times B' = (a+4d)\ (b+4d)$

$= ((0.6 + \underbrace{4 \times 0.6}_{2.4})\ (0.3 + \underbrace{4 \times 0.6}_{2.4}) \times 10^3$

$A_2 = a \times b = 0.6 \times 0.3 \times 10^3$

$13.5 \times \sqrt{\frac{3 \times 2.7 \times 10^3}{0.6 \times 0.3 \times 10^3}}$

$13.5 \times 5.32$