1a) a.) To design for Safety
b.) The design must be economical
c.) The deformation of the structure must not impose with the intensity of the structure.

1b Limit state designs considers the disadvantages of load factor design and other failure that can cause the structure to be structurally unfit while Elastic design is a method of analysis which the design of a structural member is based on a linear stress - strain relationship assuming that the working stress are only a fraction of the elastic limit of the material.

STAIR CASE DESIGN

$$\text{Slope factor} = \frac{\sqrt{R^2+T^2}}{T} = \frac{\sqrt{150^2+275^2}}{275} = 1.14$$

* Load Analysis

A. WAIST = $R \times 24kN/m^2$
$= 0.15 \times 24 = 3.6kN/m^2$

B. FINISHES = $1.2kN/m^2$

C. STEPS = $T \times \frac{1}{2} \times 24kN/m^2$
$= 0.275 \times 0.5 \times 24 = 3.3kN/m^2$

D. $G_k = ((A+B) \times S_F) + C$
$= (4.8 \times 1.14) + 3.3$
$= 8.77kN/m$

D.L.F $= 1.4G_k + 1.6Q_k$
$= 1.4(8.77) + 1.6(1.5)$
$= 14.68kN/m^2$

Span = Ttotal + 0.5(10+16) = (275x12) + 0.5(225+225) = 3.525

d = h - Cover - ½ϕ

= 150 - 25 - 6 = 119mm

$M = \frac{FL^2}{10} = \frac{14.68 \times 3.525^2}{10} = 18.24\,kNm$

$K = \frac{M}{bd^2 F_{cu}} = \frac{18.24 \times 10^6}{1000 \times 119^2 \times 25} = 0.052$

$I_a = 0.5 + \sqrt{0.25 - \frac{K}{0.9}} = 0.5 + \sqrt{0.25 - \frac{0.052}{0.9}} = 0.938$

$Z = I_a d = 0.938 \times 119 = 111.622\,mm$

$A_s = \frac{M}{0.95 f_y z} = \frac{18.24 \times 10^6}{0.95 \times 410 \times 111.622} = 419.53$

$A_{s\,pro} = 452\,mm$

provide T12 @ ~~250 c/c~~ 250 c/c ($A_{s\,pro} = 452mm$)

Deflection check

$F_s = \frac{2}{3} \times \frac{1}{3} \times \frac{A_{req}}{A_{s\,pro}} \times f_y V$

$F_s = \frac{2}{3} \times 1 \times \frac{419.53}{452} \times 250$

$= 154.69\ N/mm^2$

$M.F = 0.56 + \frac{477 - 154.69}{120\left(0.9 + \frac{18.24 \times 10^6}{1000 \times 119^2}\right)} = 1.78$

$d_{req} = \frac{span}{a_f \times e_{lr}} = \frac{3525}{1.78 \times 26} = 76.17mm$

Since $d_{req} < d$, Deflection is OK

2a $P_1 = P_2 = P_3 = \frac{4300}{4000} = 1.075 < 2 = $ 2way slab

$P_7 = P_8 = P_9 = \frac{4800}{4000} = 1.125 < 2 = $ 2way slab

$P_4 = P_5 = P_6 = \frac{4300}{400} = 1.075 > 2 = $ 1way slab

$P_{10} = P_{11} = P_{12} = \frac{4000}{1500} = 2.6777 = $ 1way slab

2b

Designing to P_2

$\frac{L_y}{L_x} = \frac{4300}{4000} = 1.075 \simeq 1.1$

shortspan coefficient $= \frac{\quad}{0.054}$

longspan coefficient $= \frac{0.058}{0.044}$

Assuming specification of slab thickness = 178mm

F_{cu} = 25N/mm²

F_y = 410N/mm²

$DL = 1.4 G_k + 1.6 Q_k$

G_k = weight of slab = 0.175 x 24

Partition = 1.0

Finishes = 1.2

6.4kN/m²

Assuming factory

D.L $(1.4 \times 6.4) + (1.6 \times 5)$

$= 16.96 \simeq 17\ kN/m^2$

Shotspan Coefficient $= 0.044$

0.033

longspan Coefficient $= 0.037$

0.028

Short span mid $= P$

$M = B \times wl^2 x = 0.044 \times 17 \times 4^2$

$= 11.968$

$d = h - cover - \frac{1}{2}\phi = 144$

$k = \frac{M}{bd^2 f_{cu}} = \frac{11.968 \times 10^6}{1000 \times 144^2 \times 25} = 0.023$

$K = Z_a = 0.5 + \sqrt{0.25 - \frac{k}{0.9}} = 0.97 > 0.95 = 0.95$

$Z = Z_a \cdot d = 0.95 \times 144 = 136.8$

$A_s = \frac{M}{0.95 f_y z} = \frac{11.968 \times 10^6}{0.95 \times 410 \times 136.8} = 224.61$

Provide y 12 @ 377mm

$M = B x w l^2 x = 0.033 \times 17 \times 4^2 = 8.976$

$d = 144$

$k = \frac{M}{bd^2 f_{cu}} = \frac{8.976 \times 10^6}{1000 \times 144^2 \times 25} = 0.0173$

$Z_a = 0.5 + \sqrt{0.25 - \frac{k}{0.9}} = 0.5 + \sqrt{0.25 - \frac{0.0173}{0.9}} = 0.$

$z = \beta_a d = 0.83 \times 144 = 119.52$

$A_s = \frac{M}{0.95 f_y z} = \frac{8.776 \times 10^6}{0.95 \times 410 \times 119.52} = 192.81 mm$

Provide y12 @ 377mm

Longspan mid

$d = d(\text{Shortspan}) - \text{Steel thickness} = 144 - 12 = 132mm$

$M = \beta x w l^2 x = 0.037 \times 17 \times 4^2 = 10.064$

$k = \frac{M}{bd^2 f_{cu}} = \frac{10.064 \times 10^6}{1000 \times 132^2 \times 25} = 0.0231$

$\beta a = 0.5 + \sqrt{0.25 - \frac{k}{0.9}} = 0.97 > 0.95 = 0.95$

$z = \beta_a . d = 0.95 \times 132 = 125.4$

$A_s = \frac{M}{0.95 f_y z} = \frac{10.064 \times 10^3}{0.95 \times 125.4 \times 410} = 206.04$

Provide y12 at 377mm

Long span Continuous

$d = 132mm$

$M = \beta x w l^2 x = 0.028 \times 17 \times 4^2 = 7.616$

$k = \frac{M}{bd^2 f_{cu}} = \frac{7.616 \times 10^6}{1000 \times 132^2 \times 25} = 0.017$

$\beta_F = 0.5 + \sqrt{0.25 - \frac{k}{0.9}} = 0.98 > 0.95 = 0.95$

$z = \beta_a . d = 125.4$

$$A_s = \frac{M}{0.95 f_y z} = \frac{2.616 \times 10^6}{0.95 \times 410 \times 125.4} = 155.93$$

provide Y12 @ 377mm

Deflection Check

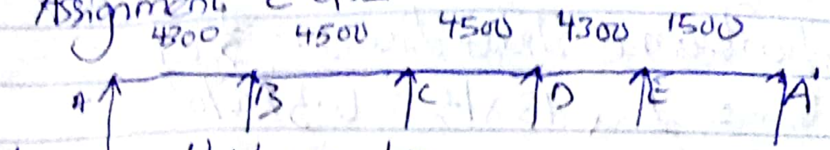
$$f_s = \frac{2}{3} f_y \, \beta \cdot \frac{A_{req}}{A_{s\,approved}}$$

$$f_s = \frac{2}{3} \times 250 \times 1 \times \frac{224.61}{377} = 99.3$$

$$m_f k = 0.55 + \frac{477 - 99.3}{120\left(0.9 + \frac{11.968 \times 10^6}{1000 \times 144^2}\right)} = 2.68 > 2$$

$$d_{req} = \frac{4 \times 1000}{2 \times 26} = 26.92 = Ok$$

Assignment 2 Q.1



Assuming thickness - 150mm

$F_{cu} = 25N/mm^2$

$F_y = 410N/mm^2$

Slab loading

Slab weight $= 0.15 \times 24 = 3.6 kN/m^2$

Partition $= 1.0 kN/m^2$

finishes $= 1.2 kN/m^2$

Total G.k $= 5.8 kN/m^2$

assuming for classroom

$D.L = 1.4(5.8) + 1.6(3.0) = 13 kN/m^2$

Beam loading

Self wt of beam $= 0.225 \times 0.6 \times 24 = 3.24 kN/m^2$

finishes $= 1.2 kN/m^2$

Wall load $= 3 \times 3.47 \simeq 10.41 kN/m^3$

Total G.k $= 14.85$

$DL = 1.4(14.85) = 20.79 kN/m^3$

Slab load on beam in longer direction $= \frac{1}{2} w l_x (1 - \frac{1}{3k^2})$

$k = \frac{l_y}{l_x}$

$\frac{4300}{4000} = 1.075$

$\frac{4500}{4000} = 1.125$

$\frac{1}{2} \times 13 \times 4.3 \left(1 - \frac{1}{3 \times (1.075)^2}\right) = 19.89 kN/m^2$

$\frac{1}{2} \times 13 \times 4.5 \left(1 - \frac{1}{3 \times (1.125)^2}\right) = 21.55 kN/m^2$

Slab load on beam in Shorter direction $= \frac{1}{3} wlx$

$= \frac{1}{3} \times 13 \times 1.5 = 6.5 kN/m^2$

Total load $= 19.89 + 20.79 = 40.68 kN/m$

$= 21.55 + 20.79 = 42.34 kN/m$

$= 6.5 + 20.79 = 27.29 kN/m$

Distribution factor

$KAB = 1$

$KBA = \frac{1/LBA}{1/LBA + 1/LBC} = \frac{\frac{1}{4.3}}{\frac{1}{4.2} + \frac{1}{4.5}} = 0.51$

$KBC = 1 - 0.51 = 0.49$

$KCB = \frac{1/4.5}{1/4.5 + \frac{1}{4.5}} = \frac{\frac{1}{4.5}}{} = 0.5$

$KCD = 1 - 0.5 = 0.5$

$KDC = 0.44$

$KDE = 0.51$

$KED = \frac{\frac{1}{4.3}}{\frac{1}{4.3} + \frac{1}{1.5}} = 0.26$

$KEA = 1 - 0.26 = 0.74$

$KAE = 1$

F.E.M

$U.D.L = \frac{WL^2}{12}$

1) $\frac{40.68 \times 4.3^2}{12} = 62.68 kN/m$

2) $\frac{42.34 \times 45^2}{12} = 71.45 kN/m$

3.) $\frac{27.29 \times 1.5^2}{12} = 5.1 \, kN/m$

	A		B		C		D		E		F
		AB	BA	BC	CB	CD	DC	DE	ED	FA.	AF	AF
DF	0	1	0.51	0.49	0.5	0.5	0.44	0.61	0.56	0.34	1	0
FEM		62.68	62.68	71.45	71.45	-71.45	71.45	62.8	62.68	5.1	5.1	
ORM		62.68		-8.77		0		8.77		57.58	-5.1	
BM		62.68		8.77		0		-8.77		-57.58	5.1	
DM	0	62.68	4.47	4.30	0	0	-4.30	-4.47	-14.77	-42.81	-5.1	0
TM		2.48	31.34	0	2.15	-2.15	0	-4.44	-2.24	-2.25	-21.505	
ORM		2.235		31.34		0		-7.44		-4.79	-21.305	
BM		-2.235		-31.34		0		7.44		4.79	21.305	
DM	0	-2.235	-15.78	-15.56	0	0	3.67	3.82	1.25	3.54	21.31	0
TM		-7.44	-1.12	0	-7.69	1.84	0	0.65	1.41	10.66	1.77	
ORM		-7.49		-1.12		5.85		0.63		12.57	1.77	
BM		7.49		1.12		5.53		-0.63		-12.57	-1.77	
DM	0	7.49	0.57	0.55	2.95	2.95	-0.31	-0.32	-3.22	-9.27	-1.77	0
TM		0.29	3.75	1.47	0.28	-0.10	1.47	-1.64	-0.16	-0.87	-4.65	
ORM		0.29		4.87		0.12		-0.12		-1.05	-4.65	
BM		-0.29		-4.87		-0.12		-0.12		1.05	4.65	
DM	0	-0.29	-2.48	-2.39	-0.06	-0.06	-0.08	-0.07	0.21	0.78	4.65	0
Σ =		0	88.88	-88.88	69.04	-69.05	72.06	-72.06	45.46	-45.46	0	

Moments

$M_a = 0 kNm$

$M_b = 82.88 kNm$

$M_c = 69.06 kNm$

$M_D = 72.06 kNm$

$M_E = 45.47 kNm$

$M_A = 0 kNm$

Fee moment

For U.D.L $= \frac{wl^2}{8}$

1.) $\frac{40.68 \times 4.5^2}{8} = 94.02\ kN/m^2$

2.) $\frac{42.34 \times 4.5^2}{8} = 107.17\ kN/m^2$

3.) $\frac{27.29 \times 1.5}{8} = 8.24\ kN/m^2$

Span moment

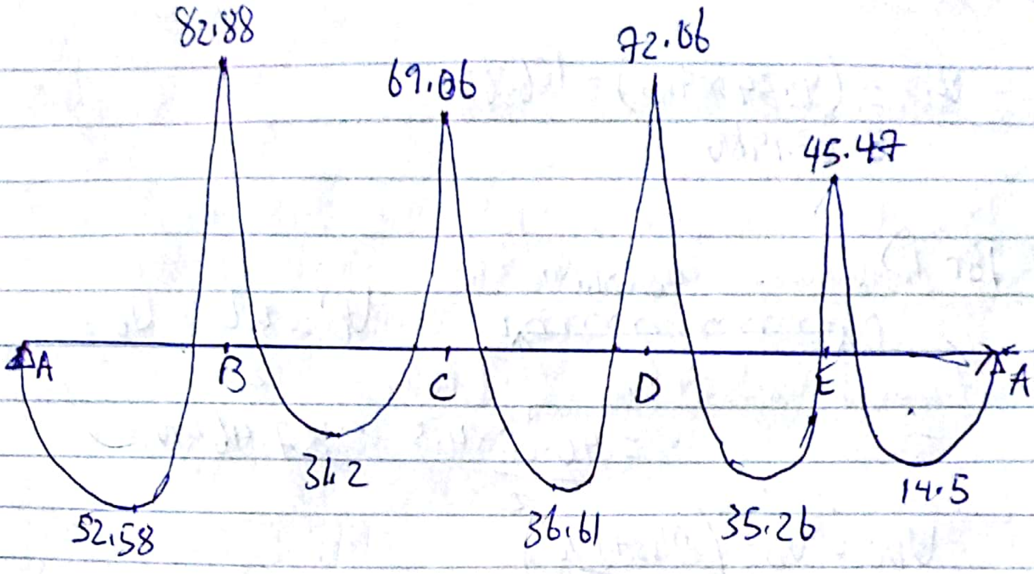
$M_{AB} = M^F - \left(\frac{M_A + M_B}{2}\right) = 94.02 - \left(\frac{0 + 82.88}{2}\right) = 52.58 kNm$

$M_{BC} = M^F - \left(\frac{M_B + M_C}{2}\right) = 107.17 - \left(\frac{82.88 + 69.06}{2}\right) = 31.2 kNm$

$M_{CD} = M^F - \left(\frac{M_C + M_D}{2}\right) = 107.12 - \left(\frac{69.06 + 12.06}{2}\right) = 36.61 kNm$

$M_{DE} = M^F - \left(\frac{M_D + M_E}{2}\right) = 94.02 - \left(\frac{72.06 + 45.47}{2}\right) = 35.26 kNm$

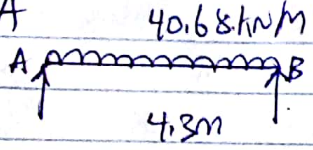
$M_{EA'} = M^F - \left(\frac{M_E + M_{A'}}{2}\right) = 8.24 - \left(\frac{45.47 + 0}{2}\right) = -14.5 kNm$



B.M.D

Shear Force

for A



$V_A = \frac{WL}{2} = V_B$

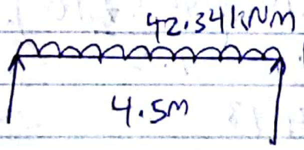
$= \frac{40.68 \times 4.3}{2} = 87.462\,kN$

$V_{AB} = V_A + \left(\frac{M_A - M_B}{L}\right) = 87.462 + \left(\frac{0 - 82.88}{4.3}\right) = 68.19$

$V_{BA} = WL - V_{AB}$

$= (40.68 \times 4.3) - 68.19 = 106.73\,kN$

For B



$V_{BC} \; \frac{WL}{2} = V_C$

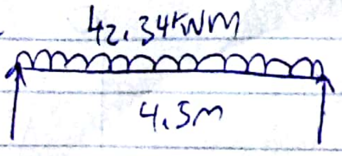
$= \frac{42.34 \times 4.5}{2} = 95.27\,kN$

$V_{BC} = V_B + \left(\frac{M_a + M_c}{L}\right) = 95.27 + \left(\frac{82.68 + 69.06}{4.5}\right) = 129.03\,kN$

$V_{CB} = (42.34 \times 4.5) - 129.03$

$= 61.5\,kN$

For C

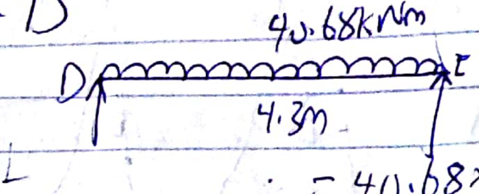


$V_C = \frac{WL}{2} = V_D$

$= \frac{42.34 \times 4.5}{2} = 95.27\,kN$

$V_{CD} = (42.34 \times 4.5) - 126.63$
$= 63.19kN$

For D



$V_D' = \frac{wl}{2} = V_E$

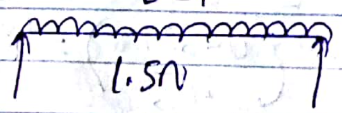
$= \frac{40.68 \times 4.3}{2} = 87.46kN$

$V_{DE} = V_D - \left(\frac{M_D + M_E}{L}\right)$

$= 87.46 - \left(\frac{72.06 + 45.47}{4.3}\right) = 60.13kN$

$V_{ED} = wl - V_{DE}$
$= (40.68 \times 4.3) - 60.13$
$= 114.74kN$

For E

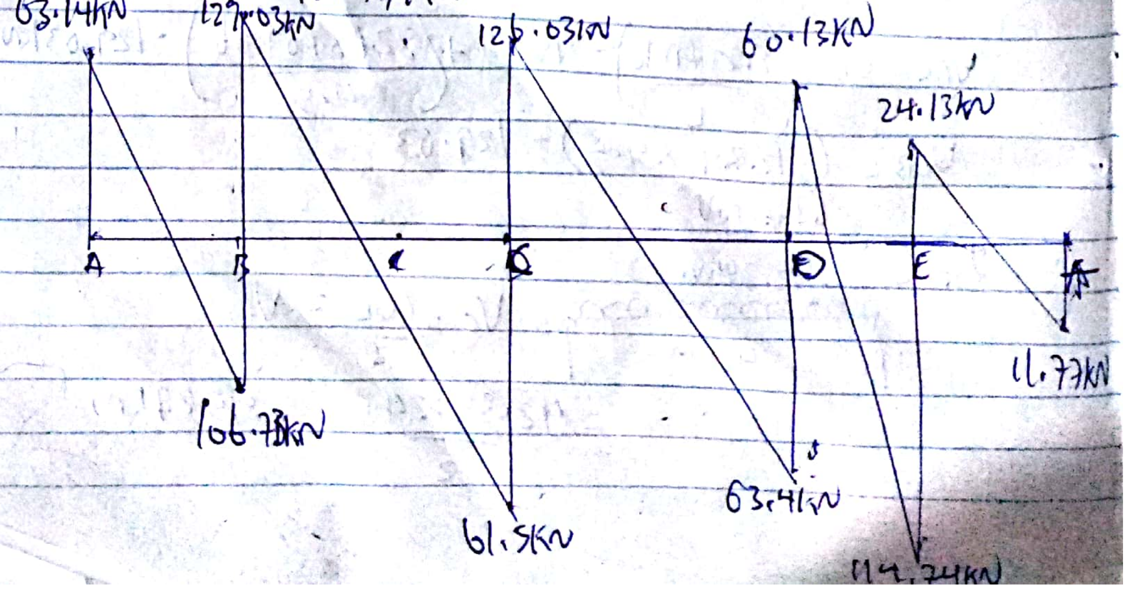


$V_E' - \frac{wl}{2} = V_D'$

$= \frac{8.24 \times 1.5}{2} = 6.18kN$

$V_{EA'} = V_E - \left(\frac{M_B + M_A}{L}\right) = 6.18 - \left(\frac{45.47 + 0}{1.5}\right) = 24.13kN$

$V_{A'E} = (8.24 \times 1.5) - 24.13$
$= 11.77kN$



Assignment 2 No 2

Base design

N = 1200kN

Strength = 25 - 410N/mm²

Fb = 150kN/m²

Area of base req = $\frac{N \times 1.1}{\lambda \times Fb}$ $\lambda = 1.40$

$\frac{1200 \times 1.1}{1.46 \times 150} = 6.027 m^2$

$\sqrt{6.027} = 2.45m \approx 2.5$

Net pressure, $F_{net} = \frac{N \times 1.1}{B}$

$\frac{1200 \times 1.1}{2.5} = 24 \times 0.660 \times 1.4$

$= 505.824 kNm$

Moment, $m = \frac{f_{net} L^2}{2}$

where $L = \frac{1}{2}(B - h)$ & = depth of base = 660

$L = \frac{1}{2}(2.5 - 0.225) = 1.138 \approx 1.14m$

$M = \frac{505.824 \times 1.14^2}{2} = 328.68 kNm$

$d = h - \text{Cover} - \frac{1}{2}\phi$

$= 660 - 50 - 10 = 600mm$

$K = \frac{M}{bd^2 f_{cu}} = \frac{328.68 \times 10^6}{1000 \times 600^2 \times 25} = 0.037$

$R_u = 0.5 + \sqrt{0.25 + \frac{0.037}{0.9}} = 0.96 > 0.95$

Scanned by CamScanner

$Z = Rnd = 0.95 \times 600 = 570mm$

$A_s = \frac{M}{0.95 f_y z} = \frac{328.68 \times 10^6}{0.95 \times 410 \times 570} = 1480.44mm^2$

Provide Y12.5 @ 300 c/c (1640)

Punching Shear
Column Size = 225 X 450mm
fcu, fy = 25 - 410mm
Area Footing = 6.027m²
Size of Footing = 2500 + 2800
q_n = Net pressure = 508.824 KNM
depth = 600
Critical Section, $\frac{d}{2}$ = 300

300 + 300 + 225 = 825mm
300 + 300 + 450 = 1050mm

Shear force $V_a = q_n \times \{\text{Area of footing} - (0.3 + d)^2\}$

$= 505.824 [2.5 \times 2.5 - (0.3 + 0.6)^2]$

$V_A = 2751.68N$

Nominal shear stress $T_v = V_n / bd$

b = perimeter of critical section
d = effective span / depth

$T_v = \frac{2751.68 \times 10^3}{((2 \times (825) + 2(1050)) \times 600)}$

$T_v = 1.223 N/mm^2$

Permissible Shear Stress

$\tau_c' = k_s \times \tau_c$

$k_s = (0.5 + \beta_c)$ but not greater than 1

β_c = Ratio of shorter to large side of columns

$\tau_c = 0.25\sqrt{f_{ck}}$

$k_s = 1$

$\tau_c = 0.25\sqrt{25} = 1.25 N/mm^2$

$\tau_v = 1.223 N/mm^2$

$\tau_v \leq \tau_v'$

Hence depth assumed is ok ✓

Checking for fb with actual size of footing

Unit weight of concrete = 24kN/mm²

unit weight of soil = 1.091×10^{-6} kN/mm²

Actual pressure footing below

$q = (1200 \times 2.5 \times 2.5) + (24 \times 0.660)$

$+ (1.091 \times 10^{-6} \times 0.66)$

$q = 214.94 kN/m^2$