INTRODUCTION TO STRUCTURAL MATERIALS & METHODS

WITH REFERENCE TO CONCRETE, STEEL, MASONRY TIMBER & GLASS

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STRUCTURAL DESIGN FOR THE SMALL PRACTICE

DUCTILE & BRITTLE MATERIALS

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Steel
Timber
Concrete
Masonry
Glass

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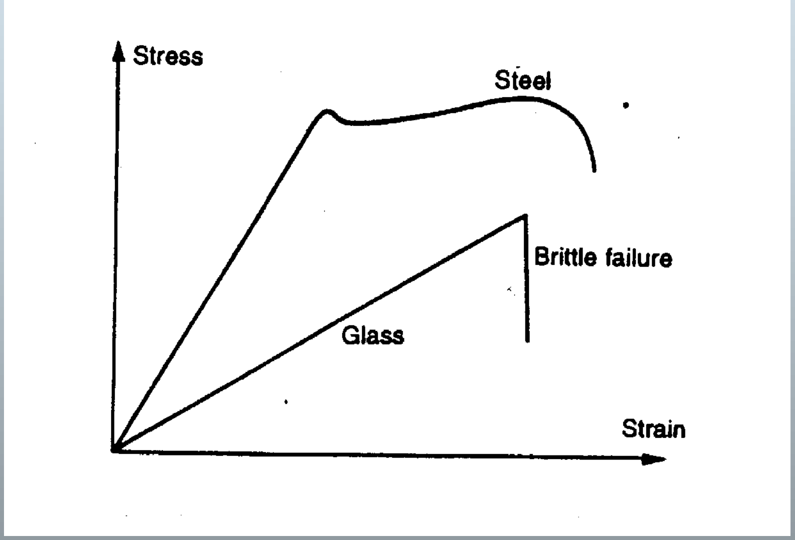
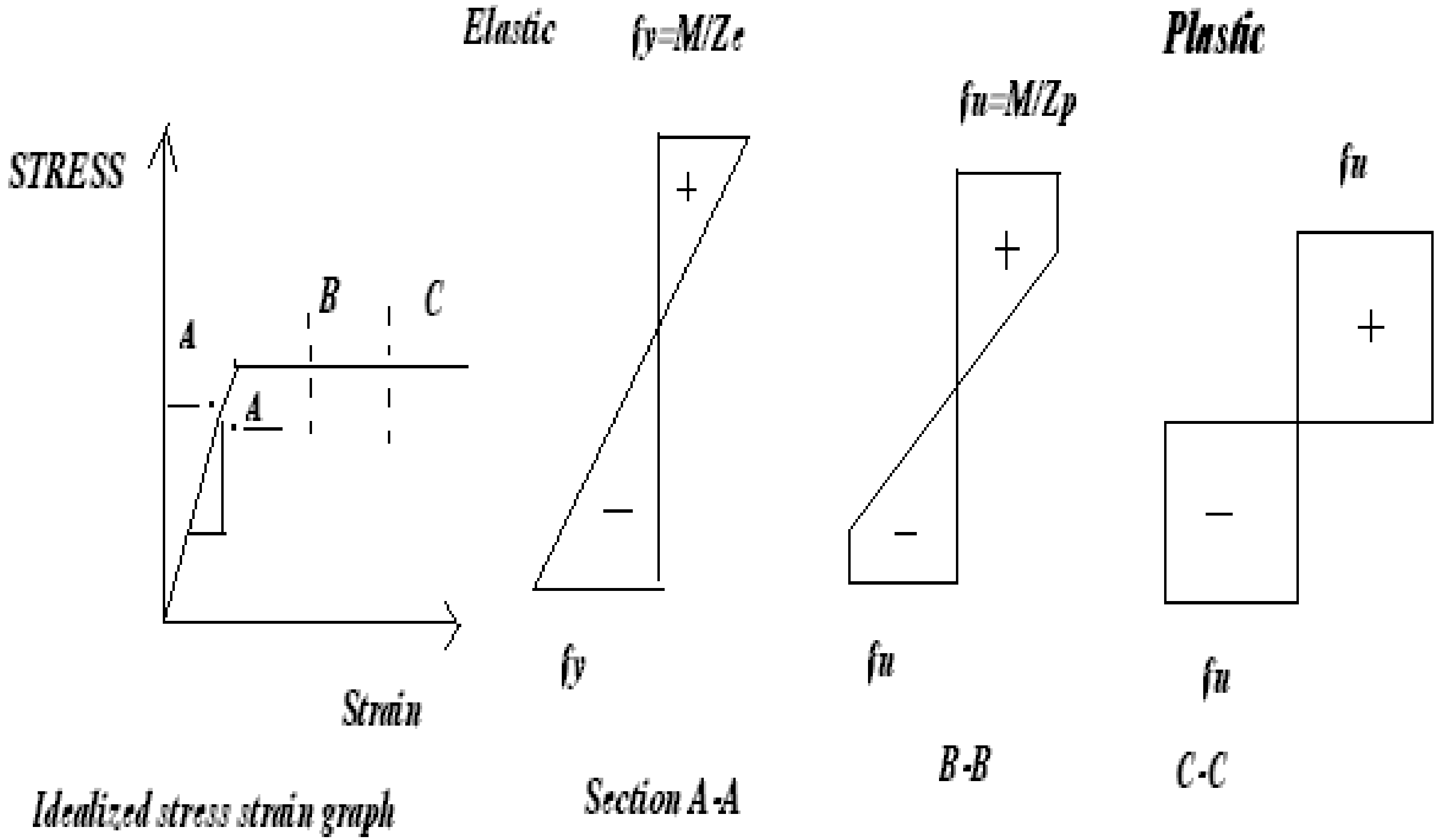


Plasticity demonstrated by flat portion
Brittle failure is sudden without a flat portion



Rectangular shapes $I = bd^3/12$

$Z_e = bd^2/6$ $\quad$ $Z_p = bd^2/4$

FIG 2

Table 1

Material	Ultimate Stress (N/mm^2)	Modulus of Elasticity (N/mm^2)	Density (KN/m^3)	Coeff of Thermal Expansion $*10^{-6}/^oC$	Embodied Energy MJ/kg (Embodied CO_2) (kg/t)	Material Factor of Safety γm
Mild steel	275	205000	70	10.8	35(2030)	1.0
High Yield steel	460	200000	70	10.8	35(2030)	1.0
Pre-stressing wire	1570	200000	70		35(2030)	1.15
Reinforced concrete	20-60	28000	24	10.8	8(203)	1.5
Timber: Softwood Hardwood	 10-30** 35-70**	 7000** 12000**	 6	 3.5** 3.5**	 2(1644) 3(2136)	 1.3***
Franka Masonry	7.5	17000	20	4.0	2(32)	2.5-3.5
Aluminium Alloy	255	70000	24	23.0	300(17000)	1.2
Glass fibre composite	250	20000	18		100(8070)	1.7
Float glass	7(28)*	70000	25	8.3	15(1130)	1.0
Toughened glass	50(56)*	70000	25	8.3	20(1130)	1.0

* Gust loading; ** Parallel to gram; ***EC5 - Timber

European Model Codes in the 60s and 70s

The principles of partial safety factors was proposed in 1927, by the Danish Moe.

An early example of the result of this work is in a British standard CP110. Any condition that a structure might attain, which contravened the basic requirement was designated a Limit State. The most important innovation in CP110 was the explicit use of probability theory in the selection of "characteristic" values of strength which – according to some notional or measured distribution – would be exceeded in at least 95% of standardised samples.

In 1978 the Nordic Committee on Building Regulations (1978) issued a report on Limit State Design containing "Recommendation for Loading and Safety Regulations of Structural Design" – NKB report No 36.

It introduces a concept of Structural Reliability dealing in safety and control class

LIMIT STATE DESIGN –

CHARACTERISTIC VALUE & DESIGN STRENGTH

CHARACTERISTIC STRENGTH OF A MATERIAL is the strength below which not more than 5% (or 1 in 20) samples will fail.

CHARACTERISTIC STRENGTH =

MEAN VALUE – 1.64 X Standard Deviation

DESIGN STRENGTH =

$$\frac{\text{CHARACTERISTIC STRENGTH}}{\text{MATERIAL FACTOR OF SAFETY}} \qquad \frac{f_u}{\gamma_m}$$

EXAMPLE:

Ten concrete cubes were prepared and tested by crushing in compression at 28 days. The following crushing strengths in N/mm^2 were obtained:

44.5 47.3 42.1 39.6 47.3 46.7 43.8 49.7 45.2 42.7

Mean strength $x_m = \frac{448.9}{10} = 44.9 N/mm^2$

Standard deviation $= \sqrt{[(x-x_m)^2/(n-1)]} = \sqrt{(80/0)}$

$= 2.98 N/mm^2$

Characteristic strength = 44.9 – (1.64 X 2.98)

= 40.0 N/mm^2

Design strength $= \frac{40.0}{\gamma_m} = \frac{40.0}{1.5}$

$= 26.7 N/mm^2$

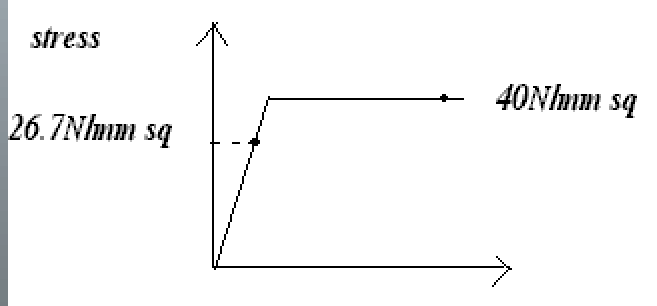


Fig 3

MATERIAL PROPERTIES
(Ref Ashby & Jones; Engineering Materials 1980)

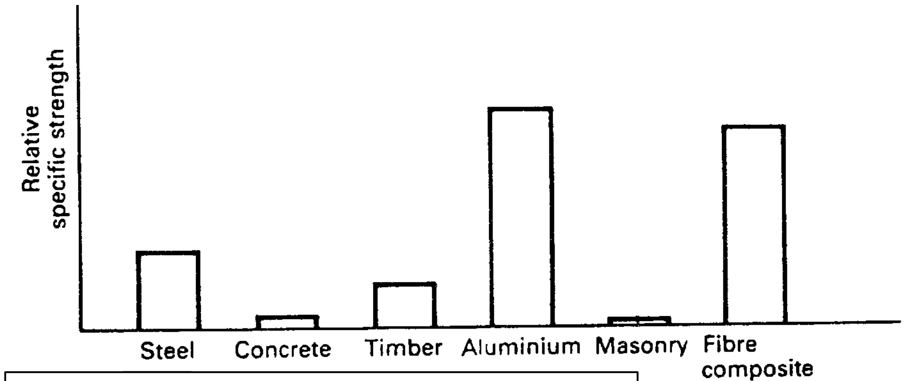


Figure 4
Design strength per unit weight for Structural materials
(Source : D. Seward (Understanding Structures)

The weight of a building is usually greater than its contents. If the structure is made lighter, structural members become smaller. Weight, however, can be useful to resist wind loads.

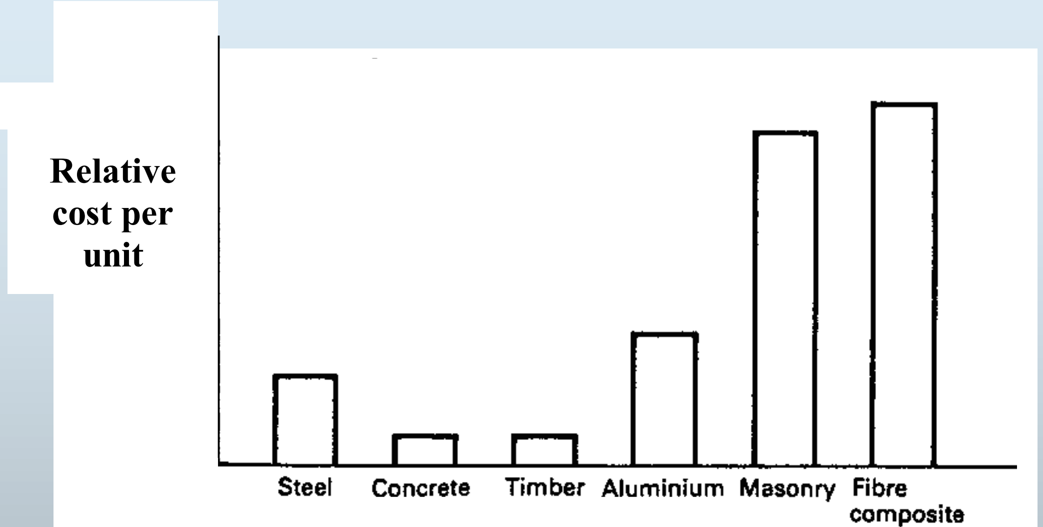
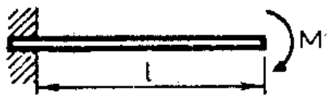
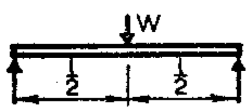


Figure 5
Relative cost of structural material per unit of stress carried
Source : D Seward (Understanding Structures)

Labour costs are ignored and some materials may require fire protection.

Table 2 – Slope and Deflexion Coefficients

BM_{max}	*Type of beam and loading*	*Max. slope*	*Max. deflexion*	*Factors*	
				Slope	*Deflexion*
M		1	$\frac{1}{2}$	$\frac{Ml}{EI}$	$\frac{Ml^2}{EI}$
M		$\frac{1}{2}$	$\frac{1}{8}$		
WL		$\frac{1}{2}$	$\frac{1}{3}$	$\frac{Wl^2}{EI}$	$\frac{Wl^3}{EI}$
$WL^2/2$		$\frac{1}{6}$	$\frac{1}{8}$		
WL/4		$\frac{1}{16}$	$\frac{1}{48}$		
$WL^2/8$		$\frac{1}{24}$	$\frac{5}{384}$		

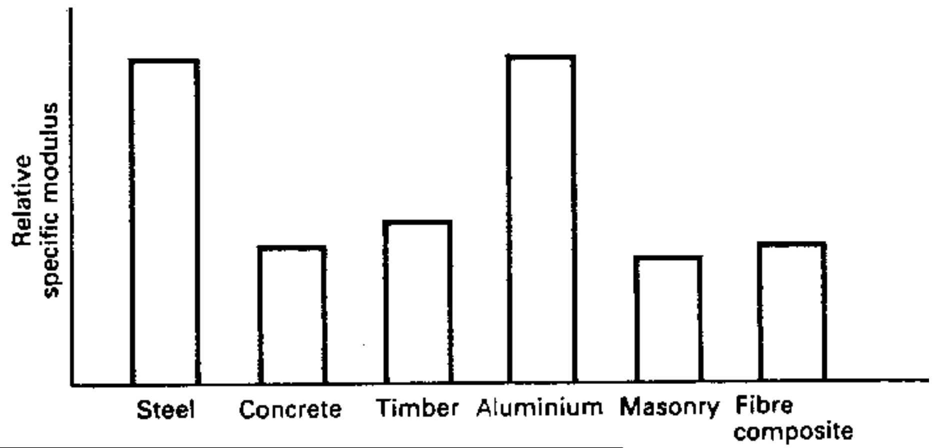


Fig 6
Modulus of elasticity per unit weight for structural materials
(Source: D. Seward (Understanding Structures)

With many structures, the design is limited by excessive deflections rather than strength, making **specific modulus** important

LOADS & LIMIT STATE DESIGN

G_k = characteristic dead load

Q_k = characteristic imposed load

W_k = characteristic wind load

Partial safety factors for loads, γ_f

Design load = characteristic load X γ_f

Table 3

Load Combination	Dead	Imposed	Wind
Dead and imposed	1.4* or 1.0	1.6*	-
Dead and wind	1.4 and 1.0	-	1.4
Dead and imposed and wind	1.2	1.2	1.2

* Eurocodes give these values as 1.35 and 1.5 respectively

Loads from liquids and earth pressure use the same factors as dead loads

IMPOSED LOADS

Table 4

Art galleries	4.0
Banking halls	3.0
Bars	5.0
Car parks	2.5
Classrooms	3.0
Churches	3.0
Computer rooms	3.5
Dance halls	5.0
Factory workshop	5.0
Foundries	20.0
Hotel bedrooms	2.0
Museums	4.0
Offices (general)	2.5
Offices (filing)	5.0
Private houses	1.5
Shops	4.0
Theatres (fixed seats)	4.0
Based on BS 6399: Part 1:1996	

Table 5 - Wind Pressure for the Maltese Islands in KN/m^2 for various building heights & terrains for a basic wind speed of 47m/s, where the greater horizontal or vertical dimension does not exceed 50m, as per CP3:ChV.

H – m	*Sea front with a long fetch*		*Countryside with scattered wind breaks*		*Outskirts of towns and villages*		*Town centers*	
		cladding		*cladding*		*cladding*		*cladding*
3 or less	1.05	1.12	0.90	0.97	0.81	0.86	0.70	0.76
5	1.12	1.19	1.00	1.07	0.88	0.95	0.74	0.81
10	1.28	1.35	1.19	1.26	1.00	1.05	0.84	0.90
15	1.34	1.39	1.28	1.35	1.12	1.19	0.93	1.00
20	1.36	1.43	1.32	1.39	1.22	1.28	1.01	1.07
30	1.42	1.47	1.39	1.44	1.31	1.36	1.15	1.21
40	1.46	1.51	1.43	1.48	1.36	1.42	1.26	1.31
50	1.49	1.54	1.46	1.49	1.40	1.46	1.32	1.38

For Structural Eurocodes, 90% of the above values to be used

LIMIT STATE DESIGN OF MASONRY COLUMN

DESIGN DEAD LOAD = 1.4*600KN = 840kN

DESIGN LIVE LOAD = 1.6*450KN = 720KN

TOTAL DESIGN LOAD = 1560KN

Characteristic Compressive strength of franka = 7.5N/mm^2

Design Stress = Characteristic value / γ_m

= 7.5N/mm^2/3 = 2.5N/mm^2

AREA OF COLUMN = 1560KN/2.5N/mm^2

= 0.625m^2

SERVICEABILITY LIMIT STATE

Loads factors taken as 1.0

- Deflection }
- Vibration } design checks
- Cracking – detailing
- Durability – specification
- Fire Resistance – the better the denser the material

DEFLECTION LIMITS TO STEELWORK EC 3 Table 6

Conditions	Limits δ_{max}	Limits δ_2
Roofs generally	L/250	L/250
Roofs frequently carrying personnel other than for maintenance	L/250	L/300
Floors generally	L/250	L/300
Floors supporting plaster or other brittle finish or non-flexible partitions	L/250	L.350
Floors supporting columns (unless the deflection has been included in global analysis for the ultimate limit state)	L/400	L/500
Where δ can impair the appearance of the building	L/250	

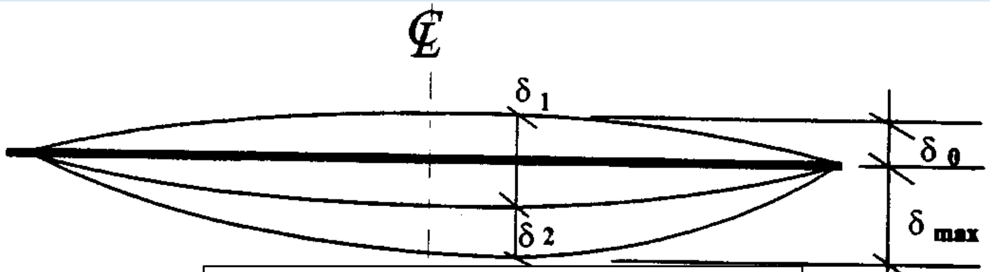


Fig 7 – Deflection limits

$\delta_o =$ deflection due to pre-camber

$\delta_1 =$ deflection due to dead load

$\delta_2 =$ deflection due to live load

Timber deflection on live load is to be limited to L/300

Concrete calculated on span/depth ratios

Vibration to EC3 (steelwork) & EC5 (timber)

(a) The fundamental frequency of floors in dwellings and offices (EC3) should not be less than 3 cycles/second. This may be deemed to be satisfied when $\delta_1 + \delta_2$ (see Fig7) < 28mm.

(b) The fundamental frequency o floors used for dancing and gymnasia EC3 should not be less than 5 cycles/second. This may be deemed to be satisfied when $\delta_1 + \delta_2$ (see Fig 7) < 10mm.

(c) For domestic timber floors (EC5), the fundamental frequency is to lie between 8Hz<f<40Hz, may be deemed to be satisfied when $\delta_1 + \delta_2 < 14$mm (see Fig 7).

DESIGN THEORY

Inexact design theory leads to a wider spread in the failure loads and an even higher mean weight.

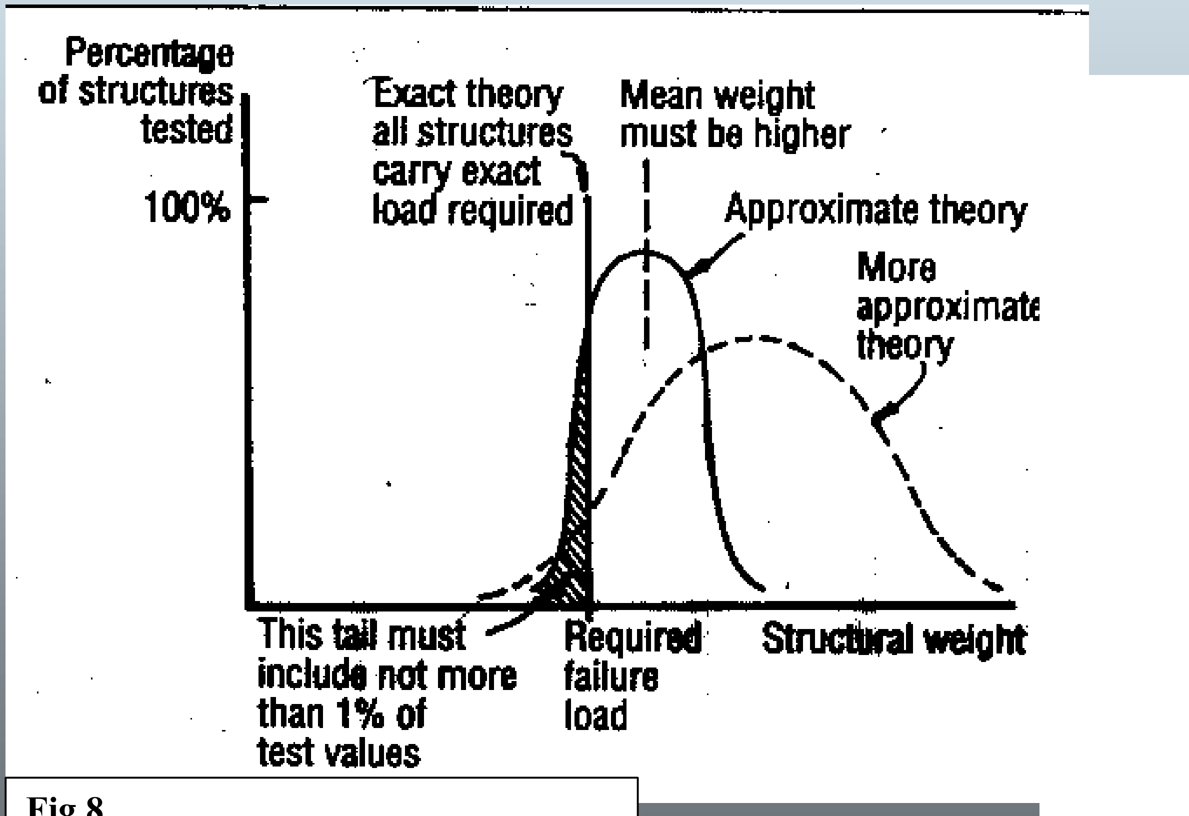


Fig 8
Statistical effect of design inaccuracy
Source: Bolton :Design Codes 2002

MOMENT DISTRIBUTION – HARDY CROSS METHOD

$$K_{BA} = \frac{0.75I}{3} \qquad K_{bc} = \frac{I}{4}$$

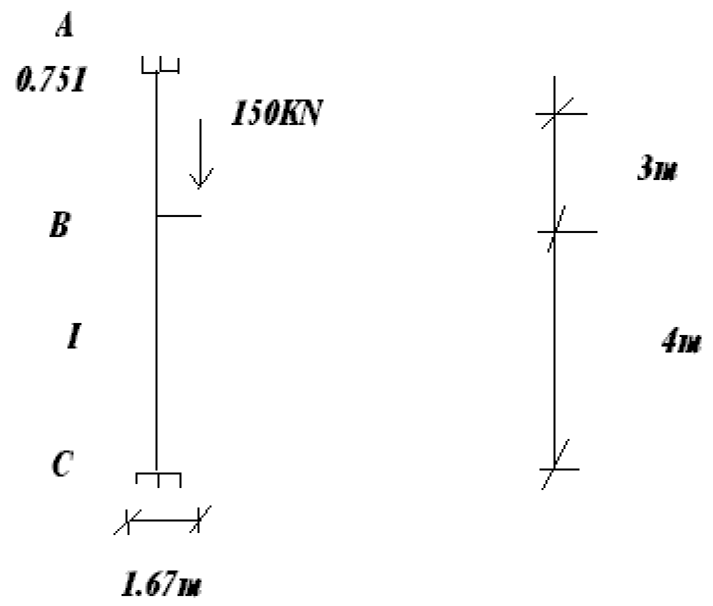
$$\Sigma K = \frac{I}{2}$$

$$DF_{BA} = \left(\frac{0.75I}{3}\right) / \frac{I}{2} = 0.5$$

$$DF_{BC} = \frac{I}{4} / \frac{I}{2} =$$

$$M_B = 150KN.1.67 = 250KN\text{-}m$$

Fig 9



MOMENT DISTRIBUTION - continued

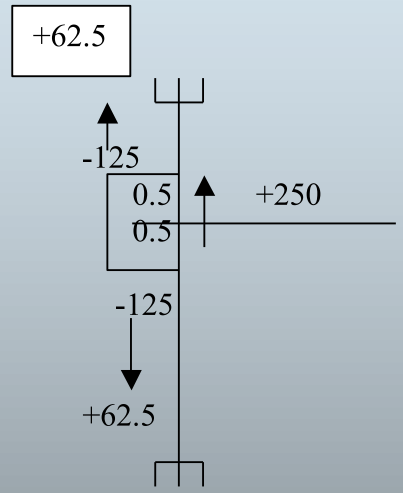
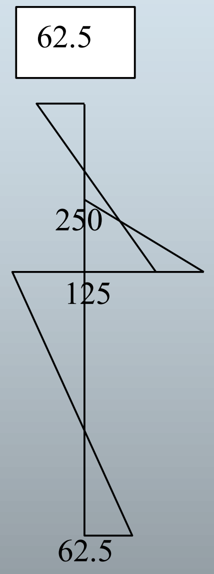
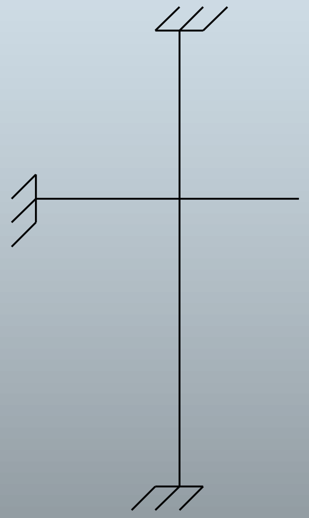


FIG 10



BM – diagram



further sub-frames

PRINCIPLES OF GLASS DESIGN

Glass in panes can deflect by more than its own thickness. This takes designers into the realm of large deflection theory, when the pane deflects by more than ½ its thickness

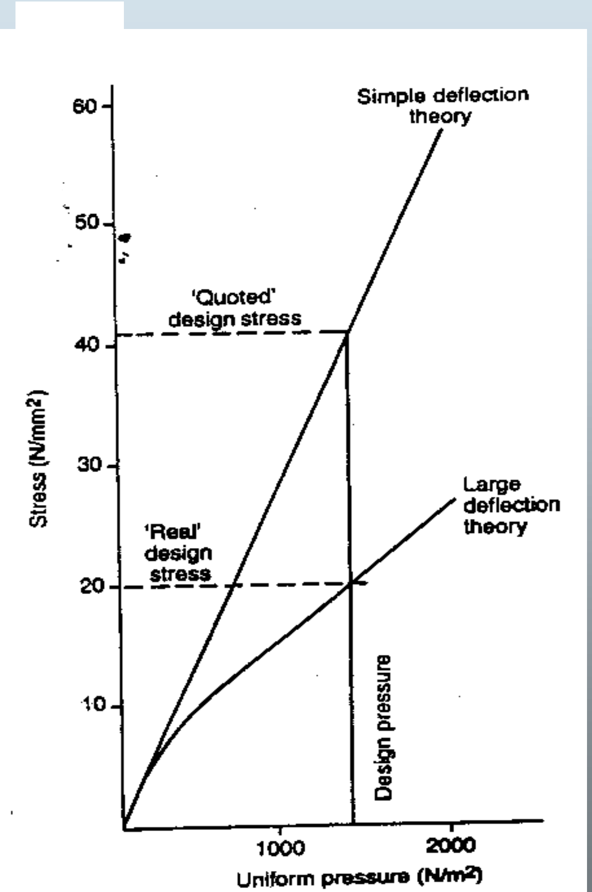


Fig 11 A comparison of small and large deflection Theory

Table 7 - ULTIMATE GLASS DESIGN STRESSES N/mm^2			
LOADING	PERMANENT	MEDIUM	SHORT
FLOAT	7	17	28
TOUGHENED	50	53	56

DESIGN EXAMPLE OF A FLOOR GLASS PANEL

The panel is 2.0m X 0.75m SS on 4 edges on a neoprene bedding on a steel angle. Assume a 19mm sheet of annealed glass subjected to a

LL of $4KN/m^2$ X 1.6 = $6.4KN/m^2$

DL of glass = 0.019mm X $25KN/m^2$ X 1.4= $0.665KN/m^2$

Ratio of sides = 2/0.75 2.67 from which α_{sx}=0.122 (Table 7)

$BM_{xx} = \alpha_{sx} wl_x^2$ $BM_{yy} = \alpha_{sy} Wl^2_x$

Table 8

Bending moment coefficients for slabs spanning in two directions at right angles, simply supported on four sides

l_y/l_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α_{sx}	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_{sy}	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

DESIGN EXAMPLE OF A FLOOR GLASS PANEL (continued)

$BM_{DL} = 0.122 \text{ X } 0.665 \text{ X } 0.75^2 = 0.033 \text{KN - m/m}$

$BM_{LL} = 0.122 \text{ X } 6.4 \text{ X } 0.75^2 = 0.44 \text{ KN – m/m}$

$f_{max} = BM/Z \ (Z = bd^2/6)$

$f_{DL} = 6 \text{ X } 0.033/0.019^2 = 548 \text{KN/m}^2 \ (0.548 \text{N/mm}^2) < 7 \text{N/mm}^2$

$f_{LL} = 6 \text{ X } 0.44 \ /0.019^2 = 7313 \text{KN/m}^2 \ (7.313 \text{N/mm}^2) < 17 \text{N/mm}^2$

Deflection Check

$\triangle = 5wL^4/384EI$ (where $I = bh^3/12$)

$\triangle = 5 \text{ X } 4 \text{ X } 750^4 \text{ X } 12/384 \text{ X } 70 \text{ X } 10^6 \text{ X } 9^3 = 0.41 \text{mm}$

This is significantly less than ½ the plate thickness, so simple bending theory is appropriate

$L/\triangle = 750/0.4 = 1875 > 175$