

# **INTRODUCTION TO STRUCTURAL MATERIALS & METHODS**

**WITH REFERENCE TO CONCRETE, STEEL,  
MASONRY TIMBER & GLASS**

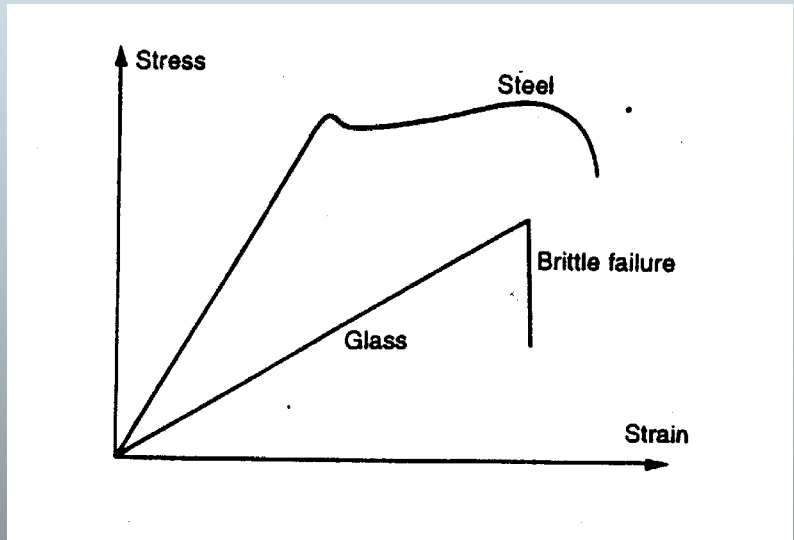
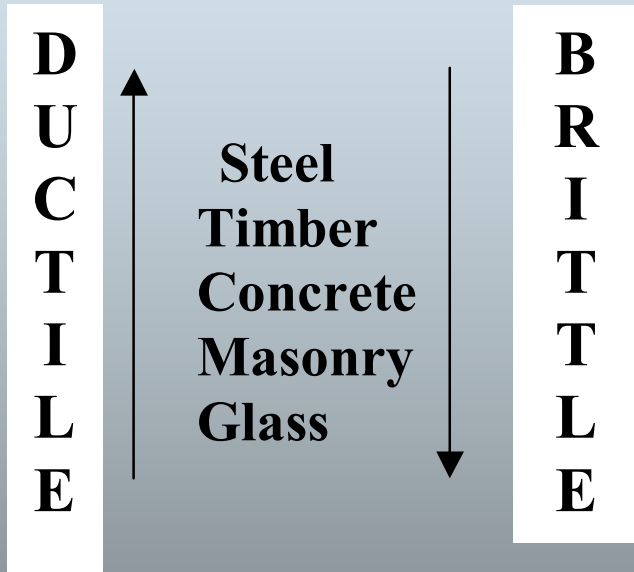
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**STRUCTURAL DESIGN FOR THE SMALL PRACTICE**

# DUCTILE & BRITTLE MATERIALS



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

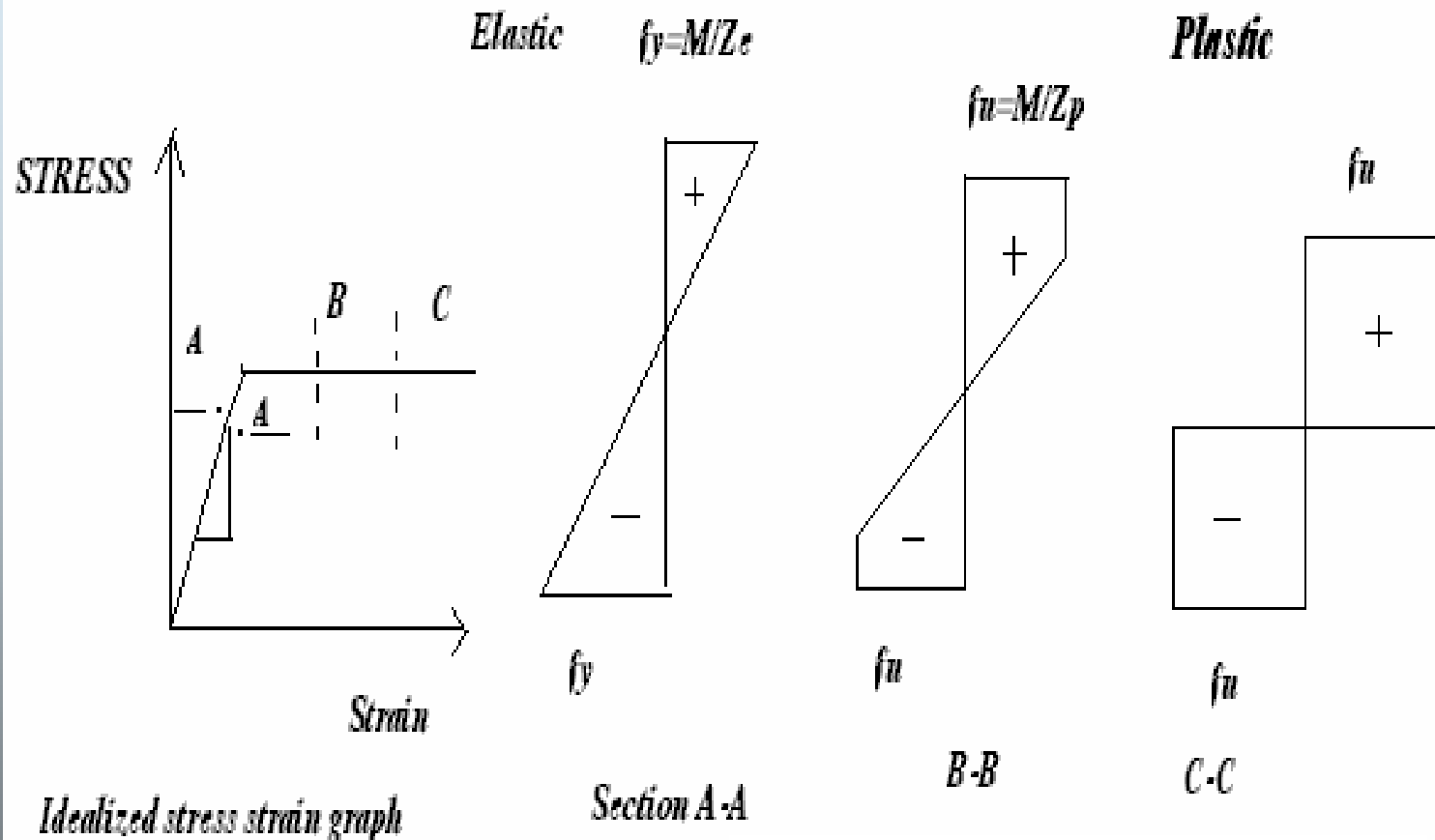


FIG 2

**Rectangular shapes  $I = bd^3/12$**

$$Z_e = bd^2/6$$

$$Z_p = bd^2/4$$

# Table 1

Material	Ultimate Stress (N/mm <sup>2</sup> )	Modulus of Elasticity (N/mm <sup>2</sup> )	Density (KN/m <sup>3</sup> )	Coeff of Thermal Expansion *10 <sup>-6</sup> /°C	Embodied Energy MJ/kg (Embodied CO <sub>2</sub> ) (kg/t)	Material Factor of Safety γ <sub>m</sub>
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	10-30**	7000**	6	3.5**	2(1644)	1.3***
Hardwood	35-70**	12000**		3.5**	3(2136)	
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 grain;    \*\*\*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 =**  
$$\frac{\text{CHARACTERISTIC STRENGTH}}{\text{MATERIAL FACTOR OF SAFETY}} \quad \frac{f_u}{\gamma_m}$$

## EXAMPLE:

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

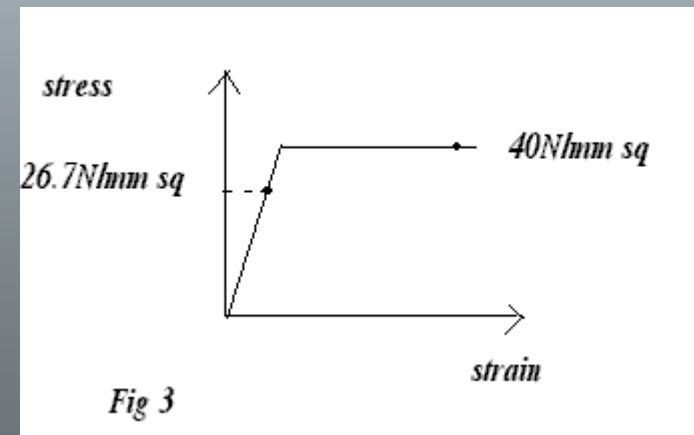
44.5 47.3 42.1 39.6 47.3 46.7 43.8 49.7 45.2 42.7

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

$$\begin{aligned} \text{Standard deviation} &= \sqrt{[(x-x_m)^2/(n-1)]} = \sqrt{(80/9)} \\ &= 2.98 \text{ N/mm}^2 \end{aligned}$$

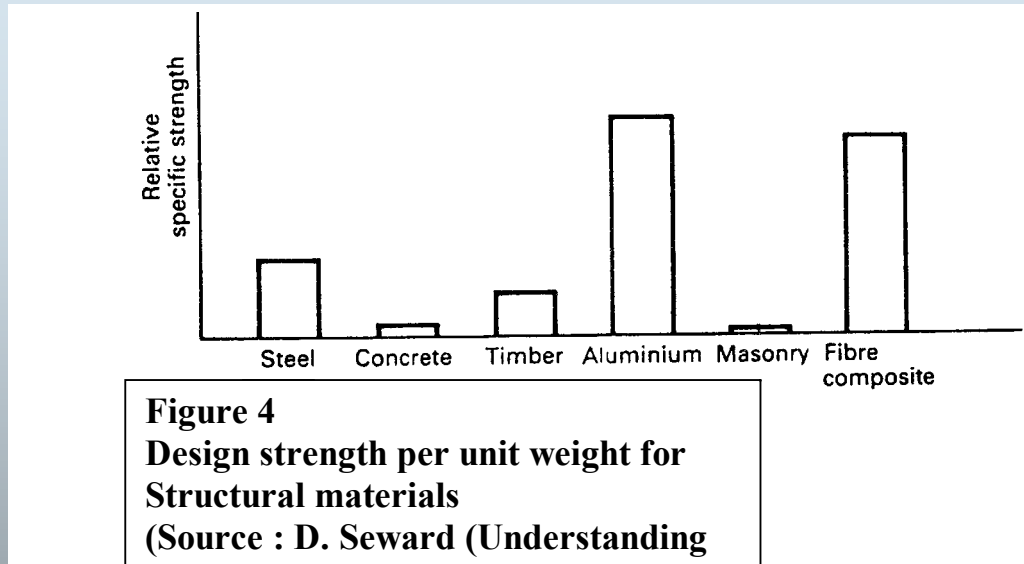
$$\begin{aligned} \text{Characteristic strength} &= 44.9 - (1.64 \times 2.98) \\ &= 40.0 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} \text{Design strength} &= \frac{40.0}{\gamma_m} = \frac{40.0}{1.5} \\ &= 26.7 \text{ N/mm}^2 \end{aligned}$$



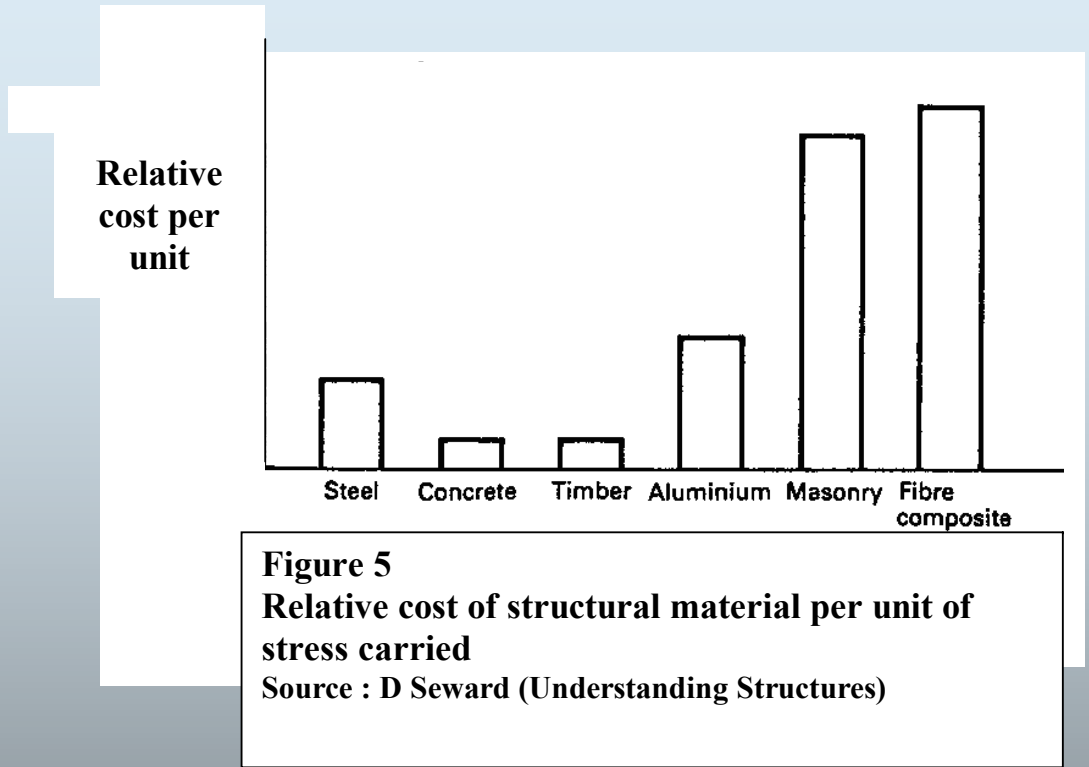
# MATERIAL PROPERTIES

(Ref Ashby & Jones; Engineering Materials 1980)



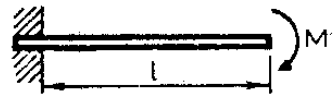
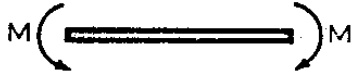
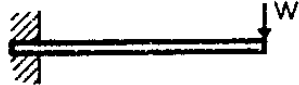
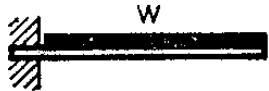
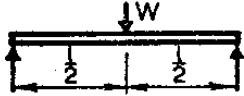

**Figure 4**  
**Design strength per unit weight for**  
**Structural materials**  
(Source : D. Seward (Understanding Structures))

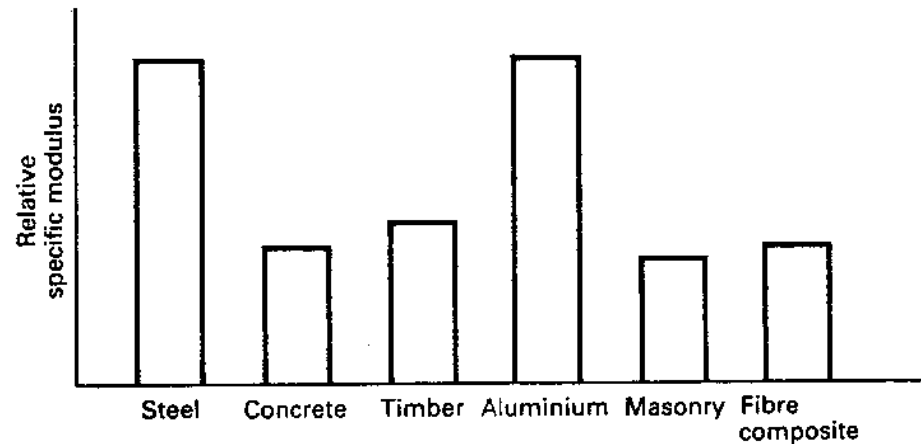
The weight of a building is usually greater than its contents. If the structure is made lighter, structural members become smaller. Weight, however, can be useful to resist wind loads.



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

# Table 2 – Slope and Deflection Coefficients

BM <sub>max</sub>	Type of beam and loading	Max. slope	Max. deflection	Factors	
				Slope	Deflection
M		1	$\frac{1}{2}$	$\frac{MI}{EI}$	$\frac{MI^2}{EI}$
M		$\frac{1}{2}$	$\frac{1}{8}$		
WL		$\frac{1}{2}$	$\frac{1}{3}$	$\frac{WI^2}{EI}$	$\frac{WI^3}{EI}$
$WL^2/2$		$\frac{1}{6}$	$\frac{1}{8}$		
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,  $\gamma_f$

Design load = characteristic load X  $\gamma_f$

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

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

**Table 5 - Wind Pressure for the Maltese Islands in KN/m<sup>2</sup> 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.**

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

$$\text{DESIGN DEAD LOAD} = 1.4 * 600\text{KN} = 840\text{kN}$$

$$\text{DESIGN LIVE LOAD} = 1.6 * 450\text{KN} = \underline{720\text{KN}}$$

$$\text{TOTAL DESIGN LOAD} = 1560\text{KN}$$

$$\text{Characteristic Compressive strength of franka} = 7.5\text{N/mm}^2$$

$$\begin{aligned} \text{Design Stress} &= \text{Characteristic value} / \gamma_m \\ &= 7.5\text{N/mm}^2 / 3 = 2.5\text{N/mm}^2 \end{aligned}$$

$$\begin{aligned} \text{AREA OF COLUMN} &= 1560\text{KN} / 2.5\text{N/mm}^2 \\ &= 0.625\text{m}^2 \end{aligned}$$

# SERVICEABILITY LIMIT STATE

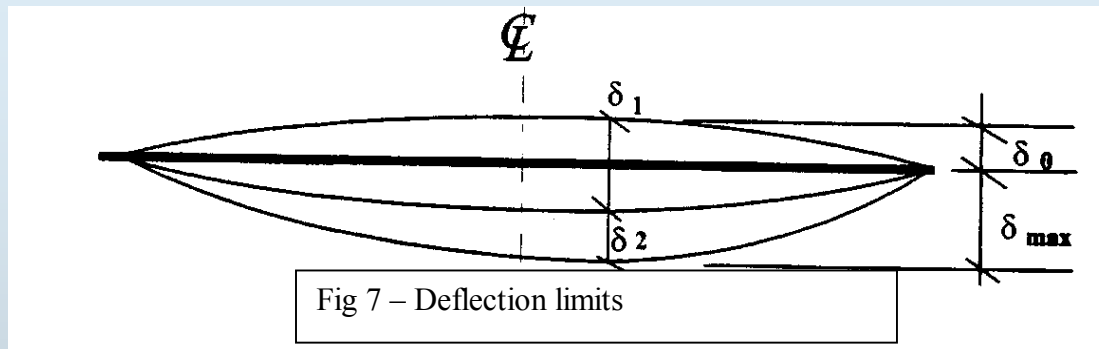
Loads factors taken as 1.0

- 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	
	$\delta_{\max}$	$\delta_2$
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 $\delta$ can impair the appearance of the building	L/250	



$\delta_0$  = 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)  $< 28\text{mm}$ .
- (b) The fundamental frequency of 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)  $< 10\text{mm}$ .
- (c) For domestic timber floors (EC5), the fundamental frequency is to lie between  $8\text{Hz} < f < 40\text{Hz}$ , may be deemed to be satisfied when  $\delta_1 + \delta_2 < 14\text{mm}$  (see Fig 7).

# DESIGN THEORY

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

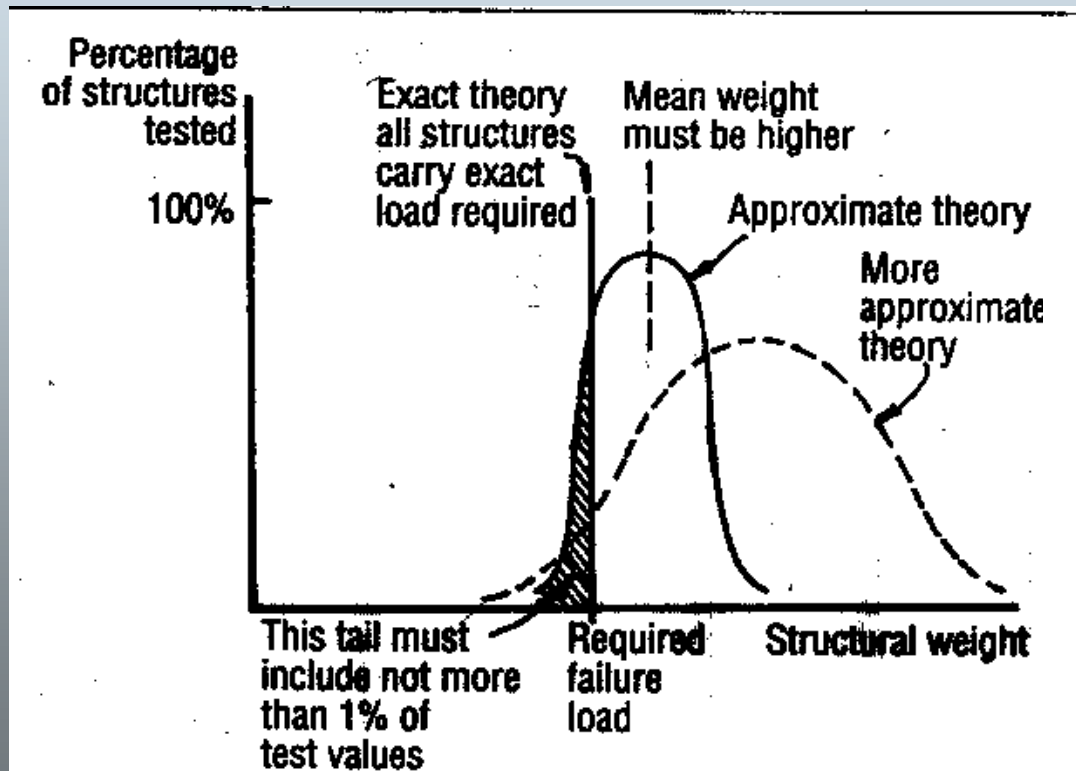


Fig 8  
Statistical effect of design inaccuracy  
Source: Bolton :Design Codes 2002

# MOMENT DISTRIBUTION – HARDY CROSS METHOD

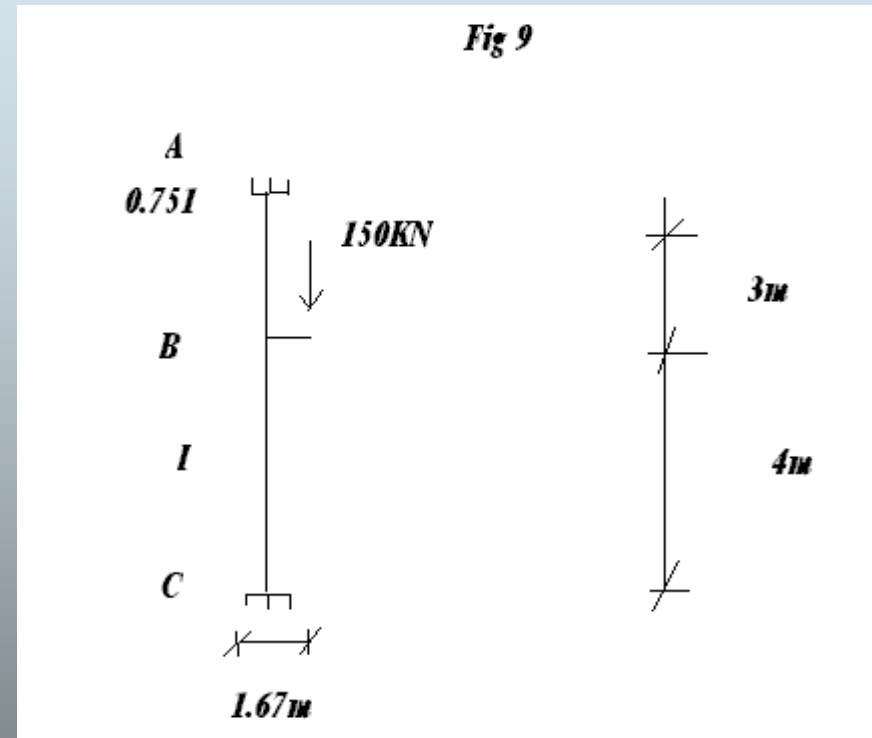
$$K_{BA} = \frac{0.75I}{3} \quad K_{bc} = \frac{I}{4}$$

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

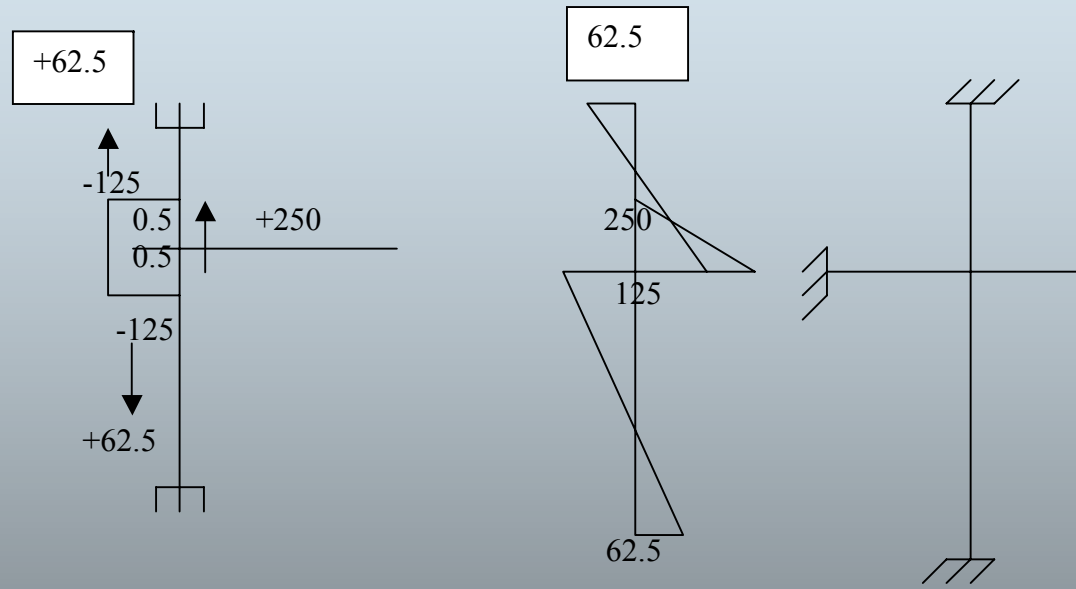
$$DF_{BA} = \frac{(0.75I) / 3}{(0.75I) / 3 + I / 4} = 0.5$$

$$DF_{BC} = \frac{I / 4}{(0.75I) / 3 + I / 4} =$$

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



# 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  $\frac{1}{2}$  its thickness

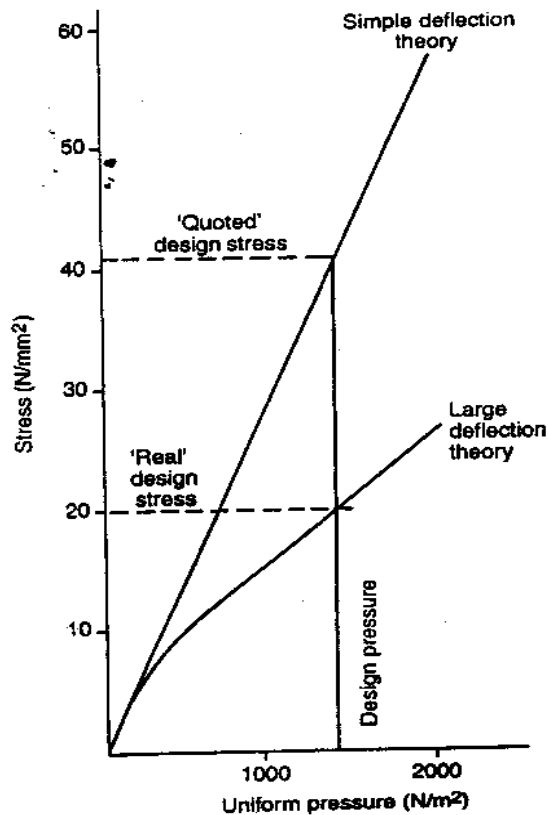


Fig 11 A comparison of small and large deflection Theory

Table 7 - ULTIMATE GLASS DESIGN STRESSES N/m m <sup>2</sup>			
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

$$\text{LL of } 4\text{KN/m}^2 \times 1.6 = 6.4\text{KN/m}^2$$

$$\text{DL of glass} = 0.019\text{m} \times 25\text{KN/m}^2 \times 1.4 = 0.665\text{KN/m}^2$$

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

$$\text{BM}_{xx} = \alpha_{sx} w l_x^2 \quad \text{BM}_{yy} = \alpha_{sy} w l_y^2$$

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
$\alpha_{sx}$	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
$\alpha_{sy}$	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)

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

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

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

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

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

Deflection Check

$$\Delta = 5wL^4/384EI \quad (\text{where } I = bh^3/12)$$

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

This is significantly less than  $\frac{1}{2}$  the plate thickness, so simple bending theory is appropriate

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