

MECH0013 Topic Notes

UCL

HD

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Chapter 1

Basic Concepts of Structure Mechanics

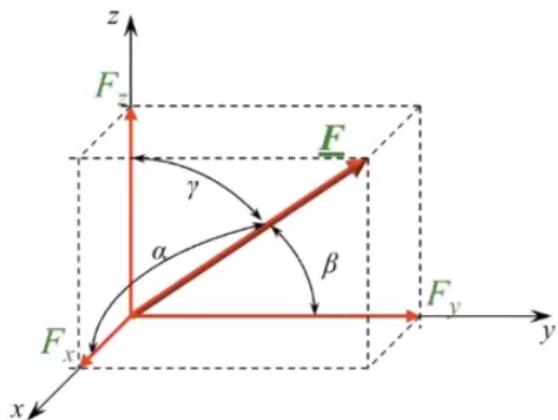
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1.1 Actions and Deformations

1.1.1 Vector Quantities

A vector is a quantity defined by **magnitude** and **direction**. Mechanical actions (forces and moments) can be represented as **vectors**.

Vector quantities can be decomposed in components, that can be conveniently oriented with the Cartesian reference system.

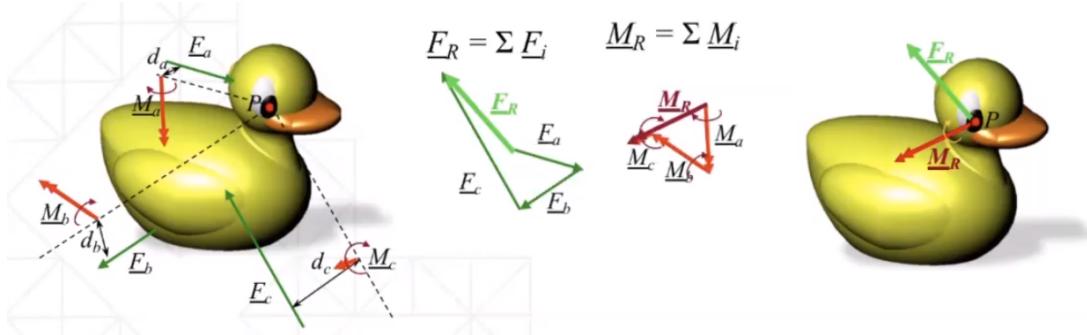


$$F_x = F \cdot \cos \alpha \quad M_x = M \cdot \cos \alpha \quad (1.1)$$

$$F_y = F \cdot \cos \beta \quad M_y = M \cdot \cos \beta \quad (1.2)$$

$$F_z = F \cdot \cos \gamma \quad M_z = M \cdot \cos \gamma \quad (1.3)$$

On the other hand, a set of vector forces can be composed in a resultant force applied to any point P, and the moment they produce about P.



$$\vec{F}_R = \sum \vec{F}_i \quad \vec{M}_R = \sum \vec{M}_i \quad (1.4)$$

$$F_R = \sqrt{(F_x)^2 + (F_y)^2 + (F_z)^2} \quad (1.5)$$

$$M_R = \sqrt{(M_x)^2 + (M_y)^2 + (M_z)^2} \quad (1.6)$$

1.1.2 Equilibrium State

If a configuration is in equilibrium, the resultant of all external forces and moments is zero. This can be expressed mathematically in the following 6 equations:

$$\begin{array}{lll} \sum F_x = 0 & \sum F_y = 0 & \sum F_z = 0 \\ \sum M_x = 0 & \sum M_y = 0 & \sum M_z = 0 \end{array}$$

These equations have to be valid for the entire body, and for any of its portions.

1.1.3 Deformations

Mechanical actions produce **deformations** in the body. These can be:

- Tension
- Compression
- Bending

- Twisting

These deformations translate into local strains and are opposed and balanced by internal reaction forces (and stresses), that guarantee the structural congruence of the body.

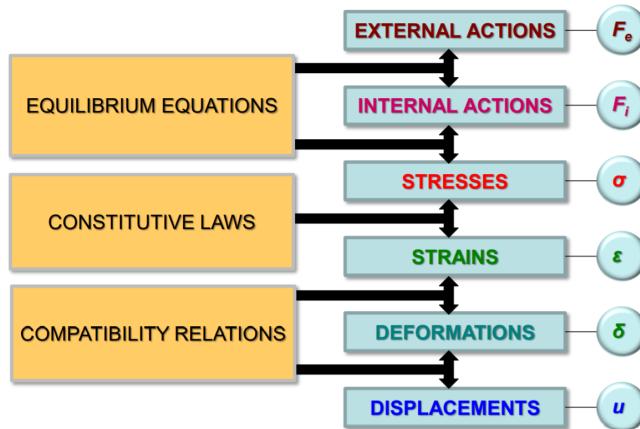
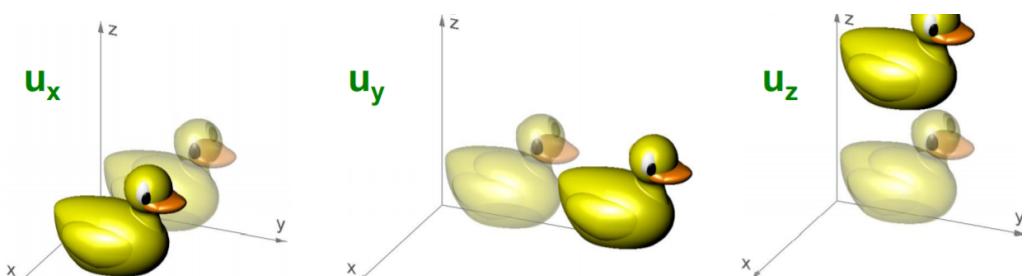


Figure 1.1: Solid Mechanics Equation: When dealing with mechanical action problems, the actions listed in the flowchart above occur, starting with external/internal forces and ending with displacements/deformations

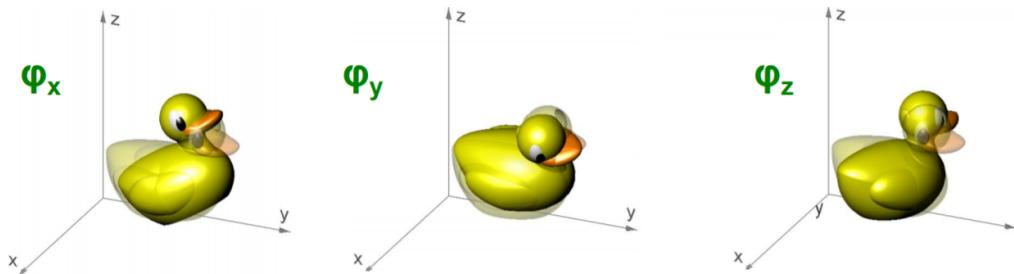
1.2 Degree of Freedom and Supports

We define **degree of freedom** of a system as all the basic kinematical parameters (or all the forms of movement) allowed. A rigid body in the space, in a coordinate system, has 6 degrees of freedom:

3 translations along the coordinate axes x , y and z



3 rotations about the coordinate axes x , y and z



The total translational and rotational movement of an object can be shown with the following expression:

$$\vec{u} = \begin{bmatrix} u_x \\ u_y \\ u_z \\ \phi_x \\ \phi_y \\ \phi_z \end{bmatrix}$$

In a 2D plane, the degree of freedom reduces to only 3 variables:

$$\vec{u} = \begin{bmatrix} u_x \\ u_y \\ \phi_z \end{bmatrix}$$

1.2.1 Constraint

We define **constraint** as a limitation of the degree of freedom of the system. The most common constraints are:

- Supports providing the required reacting forces to maintain overall equilibrium
- Connections providing reaction forces between two components of the system

The table below summarizes the different types of supports (constraints) that will be used throughout the course:

Fixed	Rotating	Roller	Sliding
$\vec{u} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$	$\vec{u} = \begin{bmatrix} 0 \\ 0 \\ \phi \end{bmatrix}$	$\vec{u} = \begin{bmatrix} u_x \\ 0 \\ \phi \end{bmatrix}$	$\vec{u} = \begin{bmatrix} u_x \\ 0 \\ 0 \end{bmatrix}$
$\vec{R} = \begin{bmatrix} R_x \\ R_y \\ M_z \end{bmatrix}$	$\vec{R} = \begin{bmatrix} R_x \\ R_y \\ 0 \end{bmatrix}$	$\vec{R} = \begin{bmatrix} 0 \\ R_y \\ 0 \end{bmatrix}$	$\vec{R} = \begin{bmatrix} 0 \\ R_y \\ M \end{bmatrix}$

1.3 Beams and Sign Conventions

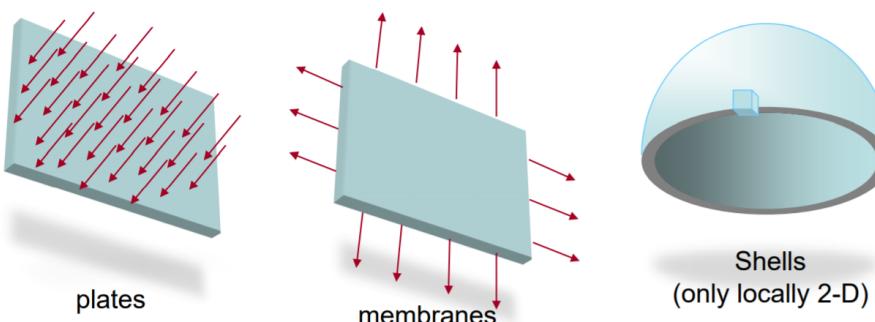
Structures are sets of solid bodies components with the function of carrying loads. All solid bodies are 3-dimensional, however, often it is possible to identify some dimension that is more relevant. Many structures can be analysed as bi-dimensional (2D) or mono-dimensional (1D).

1.3.1 Types of Structures

Bi-dimensional Structures

If one of the dimensions is negligible compared to the other two, the structure can be studied as bi-dimensional. Some examples are:

- Plates
- Membranes
- Shells (Only locally 2D)



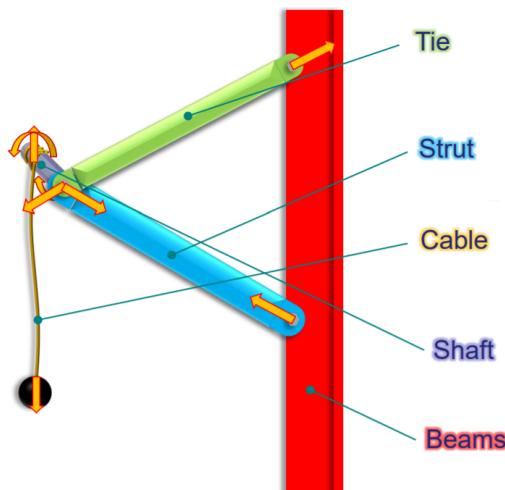
For shells to be considered bi-dimensional, they need to be looked at very closely where they can resemble a plate. The difference between plates and membranes is

the bending rigidity; force is required to bend plates while membranes are really floppy.

Mono-dimensional Structures

If two of the dimensions are negligible compared to the other one, the structure can be studied as mono-dimensional. Some examples are:

- Tie - Prevents two parts of the structure from moving away
- Strut - Prevents two parts of the structure from moving forward
- Cable - Flexible string that stands only tensile loads
- Shaft - Is used for the transmission of torque
- Beams - Can carry also transverse loads



1.3.2 Beams

The generic mono-dimensional components of structures, able to carry also transverse loads are called **beams**. Beams are between the most common and important components in structures. In order to study beams as mono-dimensional structures, all mechanical actions have to act on the **centre of gravity (CG)** of the beam section. If there is a case where a force isn't acting on the CG, it will be converted to act on it, so that the beam can be analysed in a simple manner.

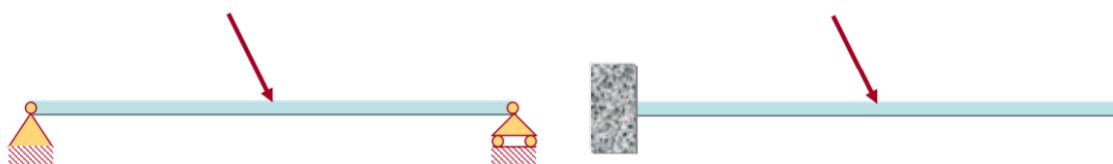
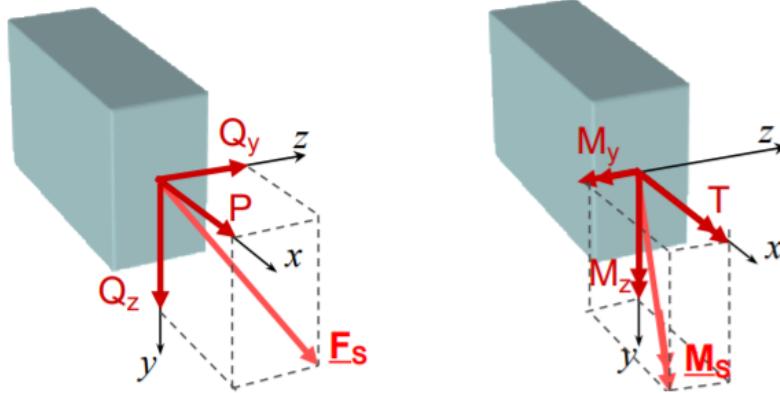


Figure 1.2: On the left: Simply supported beam (rotating support + roller support), On the right: Cantilever beam (with a fixed end)

1.3.3 Internal Forces

Each point of the beam is characterised by a specific set of internal forces. We consider a cross section of the beam to investigate these forces.

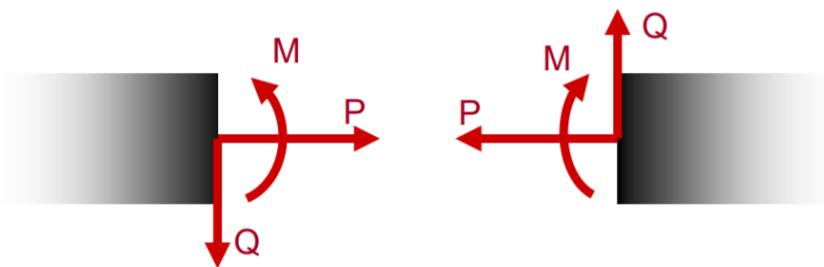


- P - Longitudinal force
- Q_y - y shear forces
- Q_z - z shear forces
- T - Torque
- M_y - y bending moment
- M_z - z bending moment

The internal forces at every point of the beam can be characterised with the following expression:

$$\vec{F} = \begin{bmatrix} P \\ Q_y \\ Q_z \\ T \\ M_y \\ M_z \end{bmatrix}$$

In a 2D plane, it simplifies to:



- P - Longitudinal force

- Q - Shear forces
- M - Bending moment

$$\vec{F} = \begin{bmatrix} P \\ Q \\ M \end{bmatrix}$$

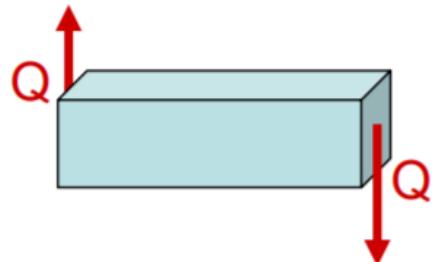
1.3.4 Sign Conventions

Longitudinal Force



The direction of pulling is considered to be positive.
The direction of compression is considered to be negative.

Shear Force



The left side pointing upwards and right side pointing downwards are taken as positive.
The left side pointing downwards and right side pointing upwards are taken as negative.

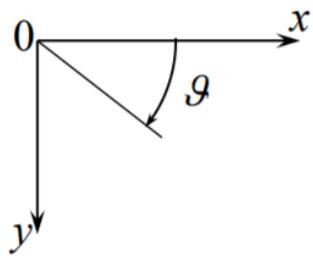
Bending Moment



If the bending moment is making the beam a upwards concave shape (like U), it is considered to be positive.

If the bending moment is making the beam a downwards concave shape, it is considered to be negative.

Reference System



x axis (horizontal direction) to the right is taken as positive.

y axis (vertical direction) downwards is taken as positive.

Chapter 2

Bending of Beams

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2.1 Internal Forces and Diagrams

2.1.1 Bending Moment

The **bending moment** is by far the most relevant of the internal forces, since it produces the largest levels of deformations and stress into the beam. Therefore, it is essential to be able to determine the distribution of the bending moment along the members, in order to assess the mechanical and functional safety of the structure.

2.1.2 Diagrams and Determination of Internal Forces

To determine the internal forces of a body, and draw the relevant diagrams, the following steps are followed:

Step 1

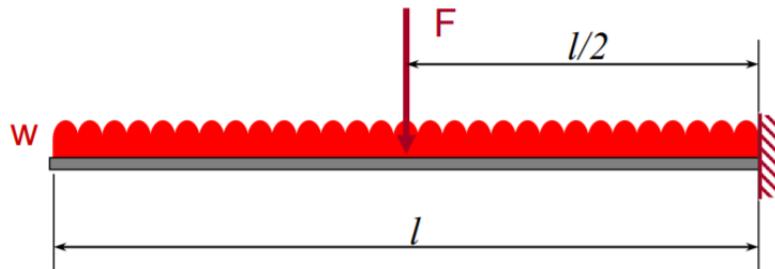
Apply to the body the force and moment equilibrium equations to find support reactions (it is possible only if the system is statically determinate)

Step 2

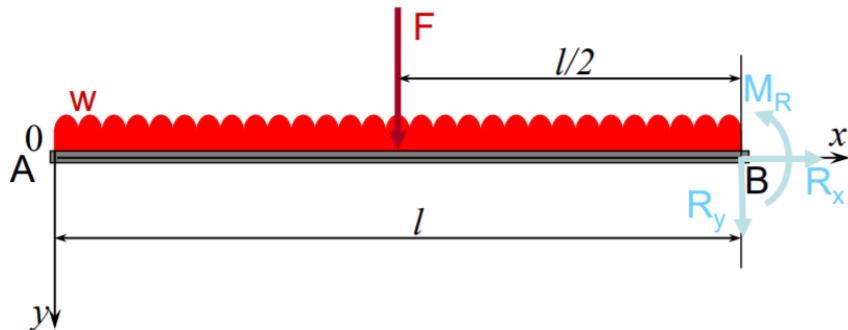
Imagine to cut the beam at a section at a distance x from the beam extreme. Balance the forces and moments of the rigid body by applying the required forces and moment at the cut section

- The **Shear Force** on any given section of a structural member is the algebraic sum of the forces to **one side only** of the section considered.
- The **Bending Moment** on any given section of a structural member is the algebraic sum of the moments of all the forces to **one side only** of the section, about the section
- The maximum value of bending moment occurs at the point where the Shear Force is zero

Example: Cantilever beam having combined concentrated and distributive loads



Determination of Support Reactions:



$$\vec{R}_B = \begin{bmatrix} R_x \\ R_y \\ M \end{bmatrix}$$

$$\sum F_x : R_x = 0 \quad (2.1)$$

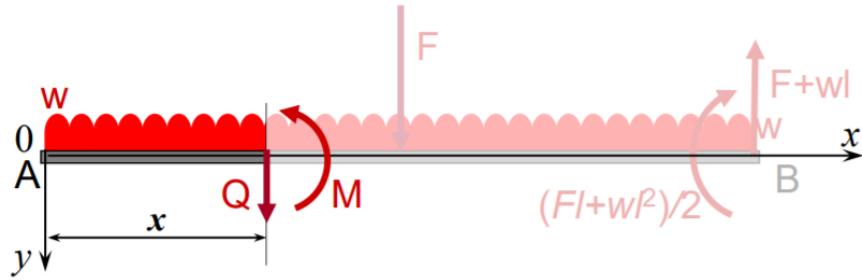
$$\sum F_y : R_y + F + wl = 0 \quad (2.2)$$

$$R_y = -(F + wl) \quad (2.3)$$

$$\sum M : M - F \frac{l}{2} - wl \frac{l}{2} = 0 \quad (2.4)$$

$$M = F \frac{l}{2} + wl \frac{l}{2} = \frac{Fl + wl^2}{2} \quad (2.5)$$

Determination of internal forces (from $x = 0$ to $x = \frac{l}{2}$)



$$\sum F_y : Q + wx = 0 \quad (2.6)$$

$$Q = -wx \quad (2.7)$$

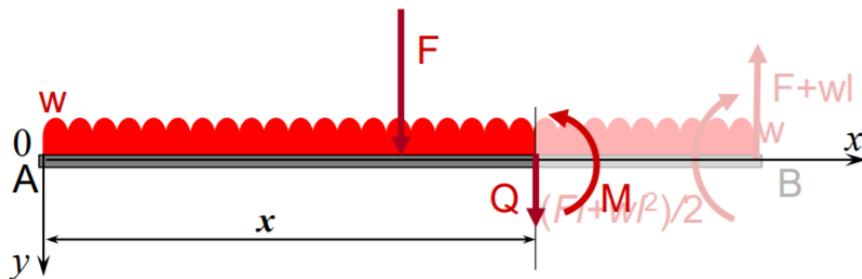
Q varies linearly: it is zero at $x = 0$ and is $\frac{-wl}{2}$ at $x = \frac{l}{2}$.

$$\sum M : M_x + wx \frac{x}{2} = 0 \quad (2.8)$$

$$M_x = \frac{-wx^2}{2} \quad (2.9)$$

M varies parabolically: it is zero at $x = 0$ and $\frac{-wl^2}{8}$ at $x = \frac{l}{2}$.

Determination of internal forces (from $x = \frac{l}{2}$ to $x = l$)



$$\sum F_y : Q + wx + F = 0 \quad (2.10)$$

$$Q = -wx - F \quad (2.11)$$

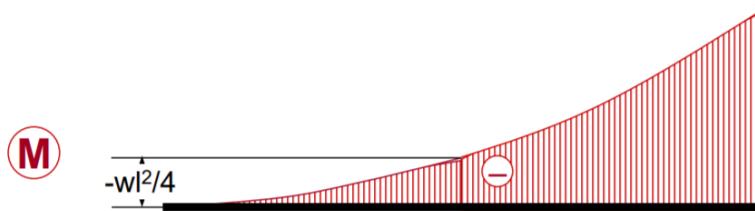
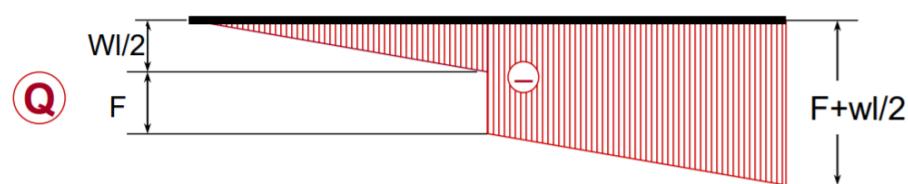
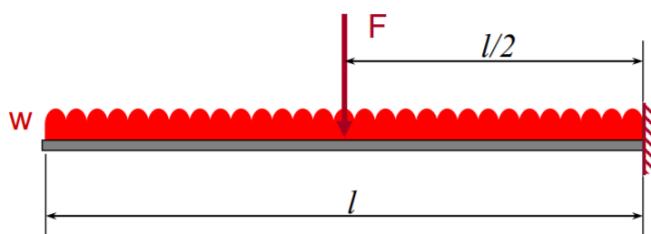
Q varies linearly between $x = \frac{l}{2}$ (where it is $-(\frac{wl}{2} + F)$) and $x = l$ (where it is $-(F + wl)$)

$$\sum M : M_x + wx \frac{x}{2} + F(x - \frac{l}{2}) = 0 \quad (2.12)$$

$$M_x = \frac{-wx^2}{2} - F(x - \frac{l}{2}) \quad (2.13)$$

At $x = \frac{l}{2}$, M distribution changes into a parabola with a steeper slope

Diagram of Internal Forces



2.2 Derivation of Bending Equations

2.2.1 Equilibrium – Beam Bending Equations

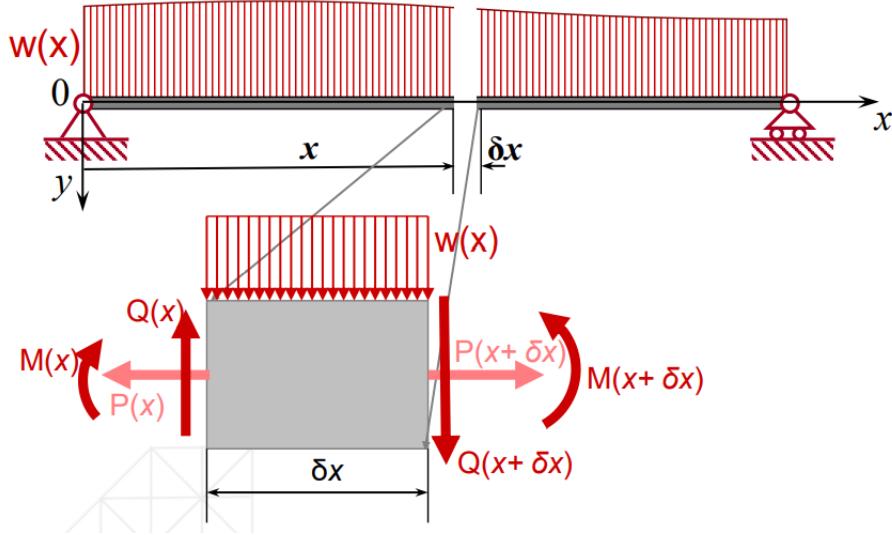


Figure 2.1: Horizontal beam with vertical distributed load. We focus a section of the beam with the very small length δx .

y-direction

$$Q(x + \delta x) - Q(x) + w(x)\delta x = 0 \quad (2.14)$$

x-direction

$$M(x + \delta x) - M(x) - Q(x)\delta x + w(x) \cdot \delta x \cdot \frac{\delta x}{2} = 0 \quad (2.15)$$

We assume that δx is so small that no matter the distribution of the load on the beam, $w(x)$ is constant along the investigated beam section.

Rearranging the terms in the y direction yields:

$$\frac{Q(x + \delta x) - Q(x)}{\delta x} = -w(x) \quad (2.16)$$

Taking δx to the smallest limit (0), the above equation can be written as:

$$\delta x = 0 \quad (2.17)$$

$$\frac{dQ}{dx} = -w(x) \quad (2.18)$$

$$w(x) = -\frac{dQ}{dx} \quad (2.19)$$

The load w in the y direction is the derivative of the shear force Q . Integrating equation (2.19):

$$Q = - \int w \, dx \quad (2.20)$$

We consider the bending moment at the distance δx (right-side for the example). Rearranging the terms in the x direction yields:

$$\frac{M(x + \delta x) - M(x)}{\delta x} = Q(x) - \frac{1}{2}w(x)\delta x \quad (2.21)$$

Taking δx to the smallest limit (0), the above equation can be written as:

$$\delta x = 0 \quad (2.22)$$

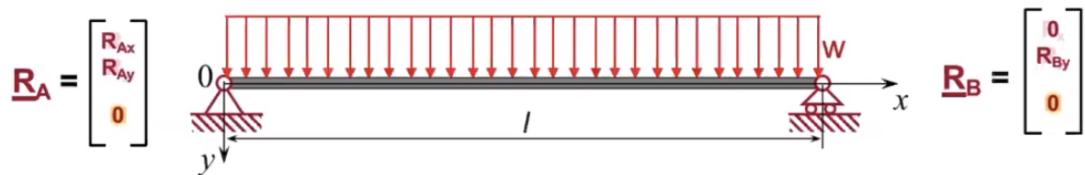
$$Q(x) = \frac{dM}{dx} \quad (2.23)$$

$$(2.24)$$

The shear force Q is the derivative of the bending Moment M . Integrating equation (2.24):

$$M = \int Q dx \quad (2.25)$$

Example:



$$Q = - \int w dx \quad (2.26)$$

$$= -wx + Q_0 \quad (2.27)$$

$$(2.28)$$

$$M = \int Q dx \quad (2.29)$$

$$= \int (-wx + Q_0) dx \quad (2.30)$$

$$= -\frac{1}{2}wx^2 + Q_0 \cdot x + M_0 \quad (2.31)$$

Applying the boundary conditions:

$$M(0) = 0 \rightarrow -\frac{1}{2}w \cdot 0^2 + Q_0 \cdot 0 + M_0 = 0 \quad (2.32)$$

$$M_0 = 0 \quad (2.33)$$

$$(2.34)$$

$$M(l) = 0 \rightarrow -\frac{1}{2}wl^2 + Q_0 \cdot l + 0 = 0 \quad (2.35)$$

$$Q_0 = \frac{1}{2}wl \quad (2.36)$$

Overall:

$$Q = -wx + \frac{1}{2}wl \quad (2.37)$$

$$M = -\frac{1}{2}wx^2 + \frac{1}{2}wlx \quad (2.38)$$

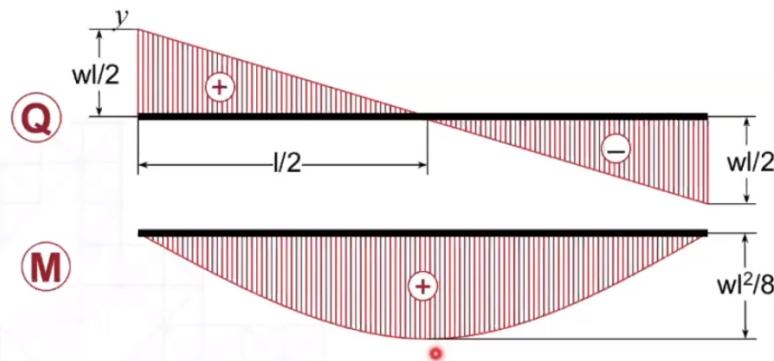


Figure 2.2: The shear force and bending moment varying along x

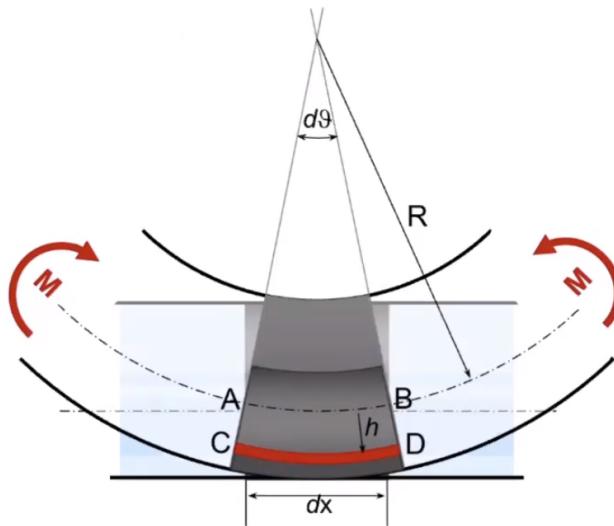
2.3 Differential Equations for Deflection

2.3.1 Theory of Pure Bending

- The beam is initially straight and unstressed;
- The beam material is perfectly homogeneous and isotropic;
- Plane cross-sections remain plane before and after bending;
- Every cross-section in the beam is symmetrical about the plane of bending;
- There is no resultant force perpendicular to any cross-section;
- The elastic limit is nowhere exceeded;
- Young's Modulus for the material is the same in tension and compression;

2.3.2 Compatibility - Strains in Pure Bending

Consider a portion of the length dx from the beam subject to uniform bending.



Lower fibres stretch and upper fibres shorten. Hence, since cross-sections remain plane, there must be a plane where the fibre elongation is zero. This is called **neutral plane** (it is a plane because section is symmetrical) and the intersection with the plane of bending is called **neutral axis**.

If we indicate with R the radius of curvature of the neutral axis, along the neutral plane:

$$\overline{AB} = \widehat{AB} \quad (2.39)$$

$$\widehat{AB} = dx = R d\theta \quad (2.40)$$

For a generic plane CD, distant h from N.A.:

$$\widehat{CD} = (R + h) d\theta \quad (2.41)$$

The longitudinal strain of CD is given by:

$$\epsilon(h) = \frac{\text{elongation}}{\text{initial length}} = \frac{\widehat{CD} - \overline{CD}}{\overline{CD}} \quad (2.42)$$

$$(2.43)$$

$$= \frac{(R + h) \cdot d\theta - R \cdot d\theta}{R \cdot d\theta} = \frac{h}{R} \quad (2.44)$$

$$(2.45)$$

$$\epsilon(h) = \frac{h}{R} \quad (2.46)$$

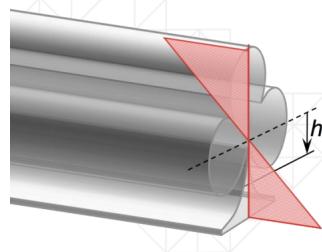


Figure 2.3: Strains are distributed linearly across the section

Stresses and strains can be associated with each other. For most materials, under small deformation:

$$\sigma = E \cdot \epsilon \quad (2.47)$$

Where:

- σ is the stress
- E is the Young Modulus
- ϵ is the strain

Therefore, from equations (2.46) and (2.47):

$$\sigma_x(h) = E \frac{h}{R} \quad (2.48)$$

2.3.3 Constitutive - Stress-Curvature Relation

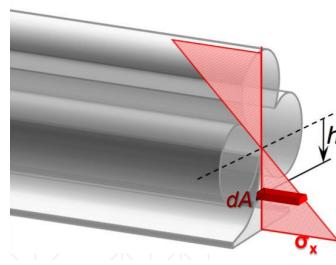
Stress is also distributed linearly across the section, being 0 at the neutral plane and maximum (in tension and compression) at the outer surfaces, where the distance from the neutral plane is maximum.

$$\sigma_{min} = E \frac{h_{min}}{R} \quad (2.49)$$

$$\sigma_{max} = E \frac{h_{max}}{R} \quad (2.50)$$

The minimum value is at the compression and the maximum value is at the tension. This just follows the convention; the tension is taken as positive and compression is taken as negative, in terms of the stress direction.

2.3.4 Equilibrium - Force-Stress Relation



Considering an elemental area dA , the force associated with bending stress is:

$$dF_x = \sigma_x \cdot dA \quad (2.51)$$

$$= E \frac{h}{R} \cdot dA \quad (2.52)$$

For the force equilibrium of the entire section:

$$\int_A dF_x = \int_A E \frac{h}{R} \cdot dA = 0 \quad (2.53)$$

$$(2.54)$$

E and R are constants, so they don't affect the integral and can be taken out. Hence the first moment of area is:

$$\int_A h \cdot dA = 0 \quad (2.55)$$

The first moment of area of a section is zero if it is calculated about the centroid.

The neutral axis corresponds to the centroid of the section.

2.3.5 Equilibrium - Bending-Stress Relation

Considering an elemental area dA , the internal moment produced by the bending stress is:

$$dM = \sigma_x \cdot h \cdot dA \quad (2.56)$$

$$= dF_x \cdot h \quad (2.57)$$

$$= E \frac{h}{R} \cdot h \cdot dA \quad (2.58)$$

Moment of the entire section:

$$M = \int_A dM = \int_A E \frac{h^2}{R} \cdot dA = \frac{E}{R} \int_A h^2 \cdot dA \quad (2.59)$$

The second moment of area is:

$$\int_A h^2 \cdot dA = I \quad (2.60)$$

The second moment of area represents how easy or difficult it is to bend a beam, depending on the shape of the cross section. The overall relation is:

$$M = \frac{EI}{R} \quad (2.61)$$

This defines how the cross section, elastic modulus, curvature, and bending moment are related to each other.

2.3.6 Solid Mechanics Equations

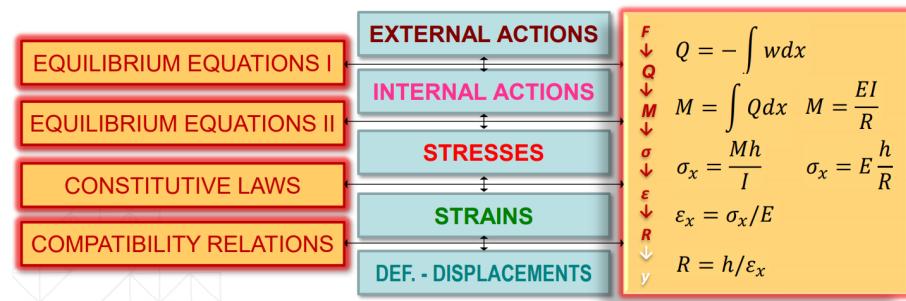


Figure 2.4: The relationships between Force, Shear Force, Bending Moment, Stress, Strain, Curvature

2.3.7 Geometric - Slope-Deflection Relation

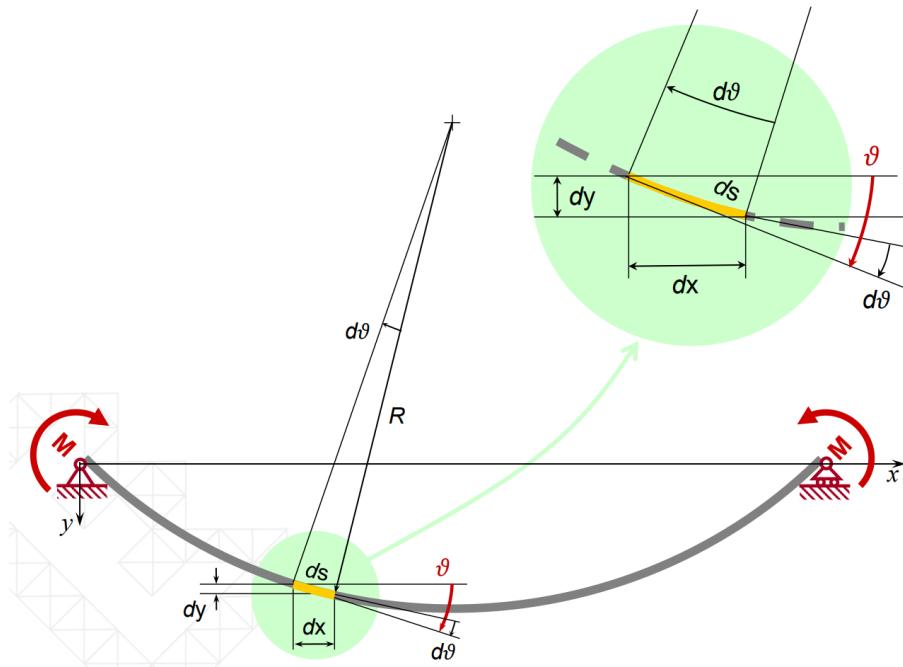


Figure 2.5: The geometry of the beam under bending. The direction of the angle θ is based on the "right-hand rule".

For infinitesimal deformations:

$$dx \approx ds = -R \cdot d\theta \quad (2.62)$$

$$\frac{1}{R} = -\frac{d\theta}{dx} \quad (2.63)$$

The negative sign comes from the situation that the x axis direction is towards right while the angle (θ) direction is to the left. Hence, they are opposite. Since:

$$M = \frac{EI}{R} \quad (2.64)$$

$$\rightarrow M = -EI \frac{d\theta}{dx} \quad (2.65)$$

Assuming angle θ is small:

$$\theta \approx \tan(\theta) = \frac{dy}{dx} \quad (2.66)$$

- The load w in the y direction is the derivative of the shear force Q .
- The shear force Q is the derivative of the bending moment M .
- The bending moment M is proportional to the derivative of the slope θ .
- The slope θ is the derivative of the deflection y : $\theta = \frac{dy}{dx}$

2.3.8 Stresses Due to Shear Force

The shear force also produces stresses into the section (shear stresses). However:

- They are much lower than bending stresses
- They are zero at surfaces, where bending stresses are maximum
- Their effect on the deformation is negligible compared to bending stresses

In general, neglecting the shear stresses due to the shear force is a good approximation for both the calculation of failure and deflections.

2.3.9 Summary of Beam Bending Equations

Deflection:

$$y \quad (2.67)$$

Slope:

$$\theta = \frac{dy}{dx} \quad (2.68)$$

Bending Moment:

$$M = -EI \frac{d\theta}{dx} = -EI \frac{d^2 y}{dx^2} \quad (2.69)$$

Shear Force:

$$Q = \frac{dM}{dx} = -EI \frac{d^3 y}{dx^3} \quad (2.70)$$

Load Distribution:

$$w = -\frac{dQ}{dx} = EI \frac{d^4 y}{dx^4} \quad (2.71)$$

The equations above give a scenario of being given deflection y , and eventually finding load distribution w . However, the inverse can also occur where load distribution w is given, and the y can be found through integration:

Load Distribution

$$w \quad (2.72)$$

Shear Force:

$$Q = - \int w \cdot dx \quad (2.73)$$

Bending Moment:

$$M = \int Q \cdot dx = - \int \int w \cdot dx \, dx \quad (2.74)$$

Slope:

$$\theta = -\frac{1}{EI} \int M \cdot dx = \frac{1}{EI} \int \int \int w \cdot dx \, dx \, dx \quad (2.75)$$

Deflection:

$$y = \int \theta \cdot dx = \frac{1}{EI} \int \int \int \int w \cdot dx \, dx \, dx \, dx \quad (2.76)$$

2.3.10 Direct Integration Method

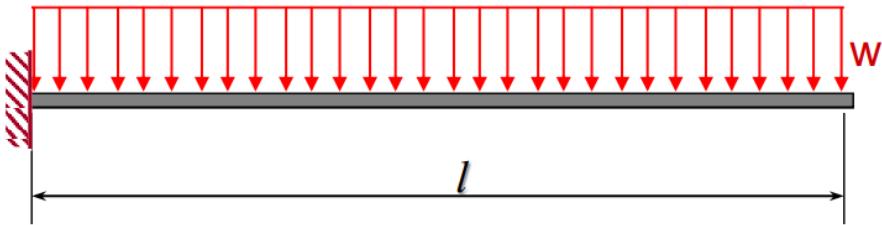
STEP 1: Determination of Support Reactions

Apply to the body the force and moment equilibrium equations to find support reactions (it is possible only if the system is statically determinate).

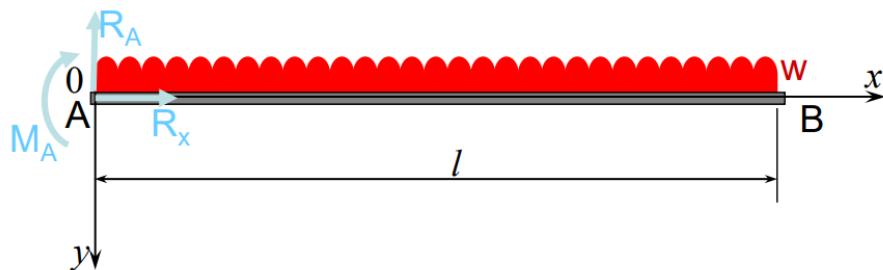
STEP 2: Determination of Deflection

1. Write bending moment expression
2. Use double integration on bending moment expression. This would result in 2 constants of integration.
3. Use boundary conditions to determine constant of integration.

Example: Uniformly Distributed Load on a Cantilever Beam



Determination of Support Reactions:



$$\vec{R}_A = \begin{bmatrix} R_x \\ R_y \\ M_A \end{bmatrix}$$

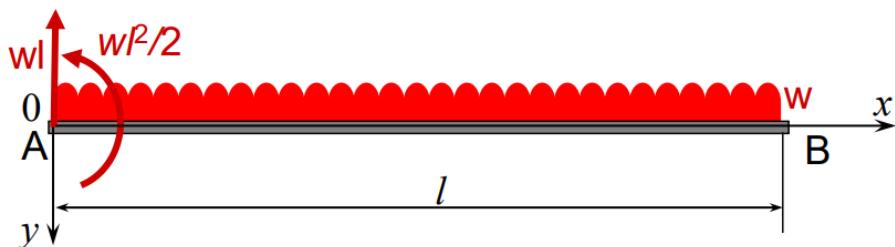
$$\sum F_x : R_x = 0 \quad (2.77)$$

$$\sum F_y : R_y = wl \quad (2.78)$$

$$\sum M : M_A + wl\frac{l}{2} = 0 \quad (2.79)$$

$$M_A = -\frac{wl^2}{2} \quad (2.80)$$

Determination of Deflection:



Direct Integration:

$$Q = - \int w \cdot dx = -wx + Q_0 \quad (2.81)$$

$$(2.82)$$

$$M = \int Q \cdot dx \quad (2.83)$$

$$= \int (-wx + Q_0) dx \quad (2.84)$$

$$= -\frac{1}{2}wx^2 + Q_0x + M_0 \quad (2.85)$$

Boundary Conditions:

$$Q(0) = R_y = wl \quad (2.86)$$

$$\rightarrow Q_0 = wl \quad (2.87)$$

$$M(0) = M_A = -\frac{1}{2}wl^2 \quad (2.88)$$

$$\rightarrow M_0 = -\frac{1}{2}wl^2 \quad (2.89)$$

Therefore:

$$Q = -w(x + l) \quad (2.90)$$

$$M = -\frac{1}{2}wx^2 + wlx - \frac{1}{2}wl^2 \quad (2.91)$$

Relating Deflection to the Bending Moment:

$$M = -\frac{1}{2}wx^2 + wlx - \frac{1}{2}wl^2 \quad (2.92)$$

$$\theta = -\frac{1}{EI} \int M \cdot dx = -\frac{1}{EI} \left(-\frac{1}{6}wx^3 + \frac{1}{2}w lx^2 - \frac{1}{2}wl^2 x \right) + \theta_0 \quad (2.93)$$

$$y = \int \theta \cdot dx = -\frac{1}{EI} \left(-\frac{1}{24}wx^4 + \frac{1}{6}w lx^3 - \frac{1}{4}wl^2 x^2 \right) + \theta_0 x + y_0 \quad (2.94)$$

$$(2.95)$$

Boundary Conditions:

$$\theta(0) = 0 \rightarrow \theta_0 = 0 \quad (2.96)$$

$$y(0) = 0 \rightarrow y_0 = 0 \quad (2.97)$$

Therefore:

$$y = \frac{1}{EI} \left(\frac{1}{24}wx^4 - \frac{1}{6}w lx^3 + \frac{1}{4}wl^2 x^2 \right) \quad (2.98)$$

$$y_{max} = y(l) = \frac{wl^4}{8EI} \quad (2.99)$$

2.3.11 Statically Determinate Beams

23/10/2020

Statically determinate (or isostatic) beams are characterised by the fact that all unknown supports reactions can be determined from the forces and equilibrium equations: the number of supports reactions is equal to the degree of freedom.

A beam in 2D plane has 3 DOF. If only lateral loads and moments act (horizontal actions and reactions = 0), the beam has only 2 degree of freedom.

Beams subjected to lateral loads are statically determinate if they have **only 2 unknown support reactions**: the two equilibrium equation for lateral forces and moments are sufficient to solve the system. If the supports produce **three or more unknown reactions**, the beams are **statically indeterminate**.

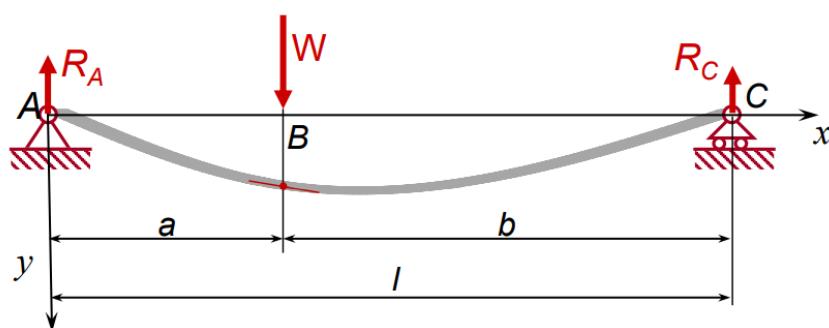
2.3.12 Double Integration Method

In more complex cases, such as discontinuous loads, the multiple integration method can still be used:

1. Write one bending moment expression for each part of the beam (e.g. AB and BC).
2. Use double integration on each bending moment expression. This would result in 2 constants of integration for each part of the beam (in this case, 4 constants).
3. Use boundary conditions to determine constant of integration.

The boundary conditions at the supports are generally not sufficient to obtain all the constants of integration (in our case we need 4 and they are 2) \therefore We need other boundary conditions.

Example: Concentrated Load on a Simply Supported Beam



We know that, though the load is discontinuous; the slope and deflection of the beam must be continuous from one section to the next. \therefore at C:

$$\theta_{AC} = \theta_{CB} \quad (2.100)$$

$$y_{AC} = y_{CB} \quad (2.101)$$

The 1st equation refers to continuous slope, while the 2nd equation refers to continuous deflection. Continuity of slope and deflection of the beam from adjacent sections provides the supplementary equations that allow us to determine all constant of integration. By solving the system of equations, the following can be obtained:

$$y_a = -\frac{x}{6EI} [R_A l(x^2 - l^2) + W(l - a)^3] \quad (2.102)$$

$$y_b = -\frac{1}{6EI} [R_A l x(x^2 - l^2) - W l(x - a)^3 + W x(l - a)^3] \quad (2.103)$$

Memorising or fully understanding these equations are not that important; it is more crucial to understand the whole process of double integration method.

If a system is composed of n parts:

- Determine n equations of the bending moment
- Solve $2n$ integrals
- Solve a system of $2n$ equations in $2n$ unknowns

2.4 Macaulay's Method

2.4.1 Step Function

We define the step function:

$$\langle x - a \rangle^n = \begin{cases} (x - a)^n & x - a \geq 0 \\ 0 & x - a < 0 \end{cases}$$

The step function can easily describe the common forms of bending moment distribution. The expression of the bending moment can be obtained, moving from the left extreme of the beam to the right, adding every time we find a new load the quantities:

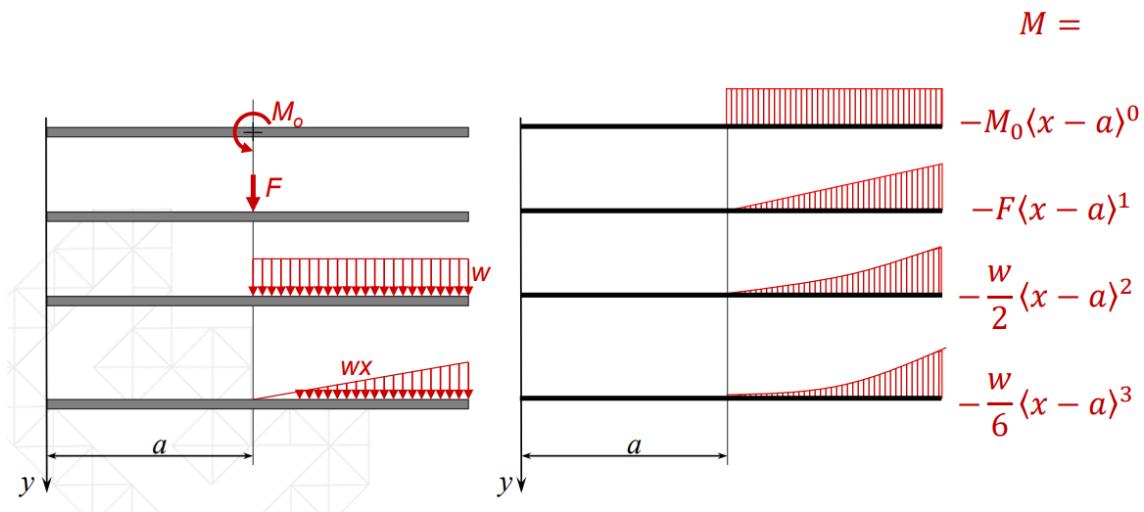


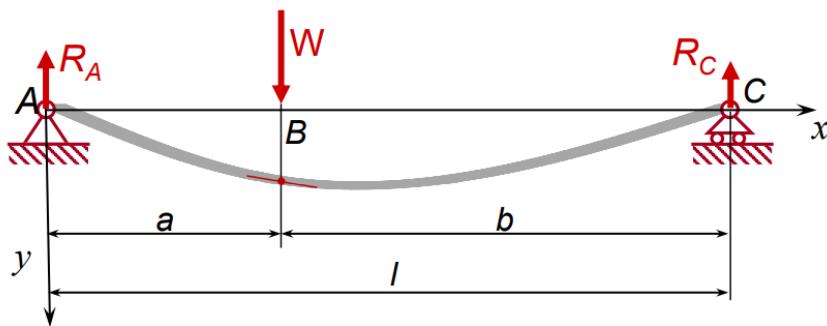
Figure 2.6: Different loads/moment applied to beams, their respective moment distributions and step functions

The crucial property of the step function is that it can be integrated just like an ordinary polynomial expression:

$$\int (x-a)^n \, dx = \frac{(x-a)^{n+1}}{n+1} + C \quad (2.104)$$

Using the step function, the expressions of the different parts of a beam (under discontinuous load) can be put together into one single expression.

Example: Simply Supported Beam with a Concentrated Load



The bending distribution in AB and BC respectively is:

$$M_a = R_A x \quad (2.105)$$

$$M_b = R_A x - W(x-a) \quad (2.106)$$

Using the step function, the bending distribution in the entire beam is:

$$M = R_A x - W \langle x - a \rangle \quad (2.107)$$

$$\theta = -\frac{1}{EI} \int M dx = -\frac{1}{EI} \left[R_A + \int x dx - W \int \langle x - a \rangle dx \right] \quad (2.108)$$

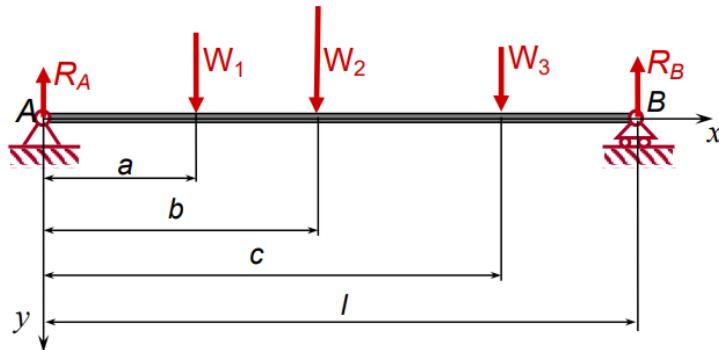
$$= -\frac{1}{EI} \left(\frac{R_A x^2}{2} - \frac{W \langle x - a \rangle^2}{2} \right) + \theta_0 \quad (2.109)$$

$$y = \int \theta dx = -\frac{1}{EI} \left[\frac{R_A x^3}{6} - \frac{W \langle x - a \rangle^3}{6} \right] + \theta_0 x + y_0 \quad (2.110)$$

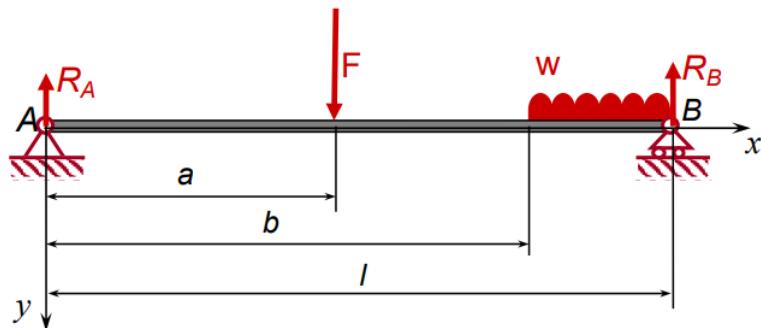
There are only 2 integration constants \therefore 2 boundary conditions are sufficient to solve the equation.

2.4.2 Examples

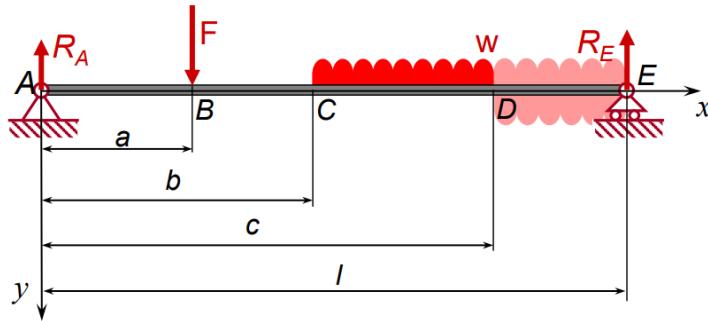
The following figures and equations are just examples of generic loads on simply supported beams, and what their step functions are. To find the deflection, the functions need to be integrated twice, and the boundary conditions must be substituted to find the integration constants.



$$M = R_A x - W_1 \langle x - a \rangle - W_2 \langle x - b \rangle - W_3 \langle x - c \rangle \quad (2.111)$$



$$M = R_Ax - F(x - a) - \frac{w}{2}(x - b)^2 \quad (2.112)$$



In this example, the distributed load is not acting all the way to the end point E ∵ A counteracting distributed load is added to cancel the contribution of the distributed load between DE.

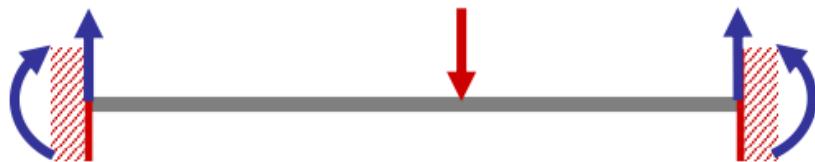
$$M = R_Ax - F(x - a) - \frac{w}{2}(x - b)^2 + \frac{w}{2}(x - c)^2 \quad (2.113)$$

2.5 Statically Indeterminate Beams

30/10/2020

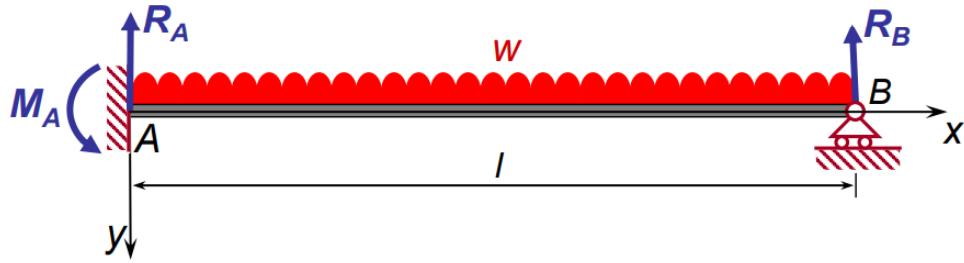
So far, we have analysed statically-determinate beams. These are characterised by only **two unknown support reactions**, that can be determined from the force and moment equilibrium equations.

If the supports produce **three or more unknown reactions**, the beam is **statically indeterminate**.



The analysis of statically indeterminate structures requires the calculation of displacements to find a sufficient number of equations to solve for all unknown reactions.

The double integration method or the Macaulay's method can be used to find the support reactions and the solution in statically indeterminate problems.

Example: Propped Cantilever with Uniformly Distributed Load (Double Integration Method)


The equilibrium equations:

$$R_A - wl + R_B = 0 \quad (2.114)$$

$$R_A l - M_A - \frac{1}{2}wl^2 = 0 \quad (2.115)$$

Moment at the point distance x away from the left end:

$$M(x) = -M_A - \frac{1}{2}wx^2 + R_Ax \quad (2.116)$$

Integrating the moment equation:

$$\theta(x) = \frac{dy}{dx} = -\frac{1}{EI} \int \left(-M_A - \frac{1}{2}wx^2 + R_Ax \right) dx \quad (2.117)$$

$$= -\frac{1}{EI} \left(-M_Ax - \frac{1}{6}wx^3 + \frac{1}{2}R_Ax^2 \right) + \theta_0 \quad (2.118)$$

Boundary Condition: $\theta(0) = 0 \rightarrow \theta_0 = 0$

$$\therefore \theta(x) = -\frac{1}{EI} \left(-M_Ax - \frac{1}{6}wx^3 + \frac{1}{2}R_Ax^2 \right) \quad (2.119)$$

Integrating further:

$$y(x) = -\frac{1}{EI} \int \left(-M_Ax - \frac{1}{6}wx^3 + \frac{1}{2}R_Ax^2 \right) dx \quad (2.120)$$

$$= -\frac{1}{EI} \left(-\frac{1}{2}M_Ax^2 - \frac{1}{24}wx^4 + \frac{1}{6}R_Ax^3 \right) + y_0 \quad (2.121)$$

Boundary Condition: $y(0) = 0 \rightarrow y_0 = 0$

$$\therefore y(x) = -\frac{1}{EI} \left(-\frac{1}{2}M_Ax^2 - \frac{1}{24}wx^4 + \frac{1}{6}R_Ax^3 \right) \quad (2.122)$$

Applying another boundary condition $y(l) = 0$ (to consider the other end of the beam):

$$y(l) = -\frac{1}{EI} \left(-\frac{1}{2}M_Al^2 - \frac{1}{24}wl^4 + \frac{1}{6}R_Al^3 \right) = 0 \quad (2.123)$$

$$\rightarrow \frac{1}{2}M_Al^2 + \frac{1}{24}wl^4 - \frac{1}{6}R_Al^3 = 0 \quad (2.124)$$

$$\rightarrow 12M_A + wl^2 - 4R_Al = 0 \quad (2.125)$$

The R_A , R_B , and M_A are still **unknowns** at this point. Combining the equilibrium conditions with the double integration (equation (2.125)):

$$R_A - wl + R_B = 0 \quad (2.126)$$

$$R_A l - M_A - \frac{1}{2}wl^2 = 0 \quad (2.127)$$

$$4R_A l - 12M_A - wl^2 = 0 \quad (2.128)$$

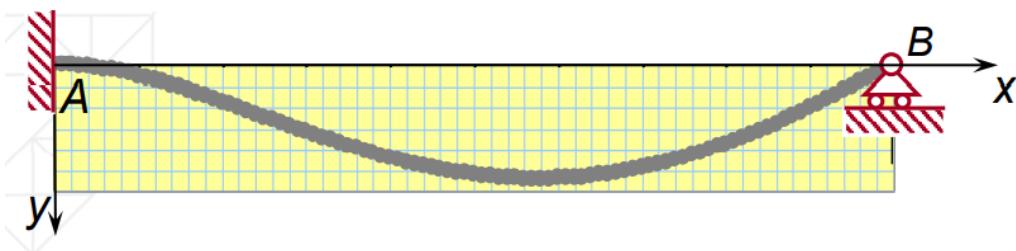
Solving for the above system of equations yields:

$$R_A = \frac{5}{8}wl \quad (2.129)$$

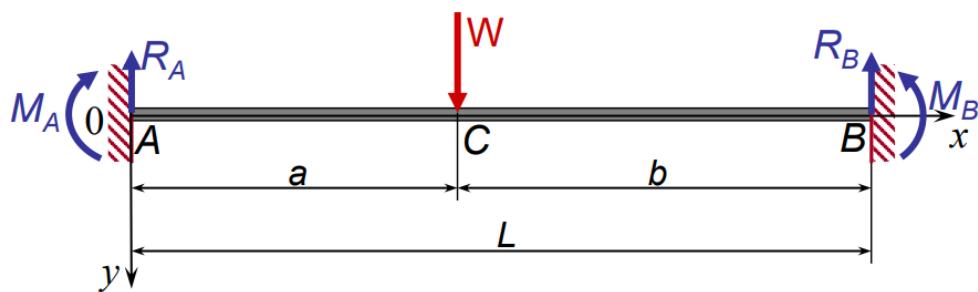
$$R_B = \frac{3}{8}wl \quad (2.130)$$

$$M_A = \frac{wl^2}{8} \quad (2.131)$$

Using these values in all previous equations, all quantities can be determined.



Example: Fixed Beam with Concentrated Load (Macaulay's Method)



The equilibrium conditions:

$$R_A - W + R_B = 0 \quad (2.132)$$

$$M_A - Wb + R_A L - M_B = 0 \quad (2.133)$$

Moment at the point distance x away from the left end:

$$M(x) = M_A + R_A x - W(x - a) \quad (2.134)$$

Integrating the moment equation:

$$\theta(x) = \frac{dy}{dx} = -\frac{1}{EI} \int (M_A + R_Ax - W(x-a)) dx \quad (2.135)$$

$$= -\frac{1}{EI} \left(M_Ax + \frac{1}{2}R_Ax^2 - \frac{1}{2}W(x-a)^2 \right) + \theta_0 \quad (2.136)$$

Boundary Condition: $\theta(0) = 0 \rightarrow \theta_0 = 0$

$$\therefore \theta(x) = -\frac{1}{EI} \left(M_Ax + \frac{1}{2}R_Ax^2 - \frac{1}{2}W(x-a)^2 \right) \quad (2.137)$$

Integrating further:

$$y(x) = -\frac{1}{EI} \int \left(M_Ax + \frac{1}{2}R_Ax^2 - \frac{1}{2}W(x-a)^2 \right) dx \quad (2.138)$$

$$= -\frac{1}{EI} \left(\frac{1}{2}M_Ax^2 + \frac{1}{6}R_Ax^3 - \frac{1}{6}W(x-a)^3 \right) + y_0 \quad (2.139)$$

Boundary Condition: $y(0) = 0 \rightarrow y_0 = 0$

$$\therefore y(x) = -\frac{1}{EI} \left(\frac{1}{2}M_Ax^2 + \frac{1}{6}R_Ax^3 - \frac{1}{6}W(x-a)^3 \right) \quad (2.140)$$

Applying other boundary conditions $\theta(L) = 0$, $y(L) = 0$ (to consider the other end of the beam):

$$\theta(L) = -\frac{1}{EI} \left(M_AL + \frac{1}{2}R_AL^2 - \frac{1}{2}W(L-a)^2 \right) = 0 \quad (2.141)$$

$$y(L) = -\frac{1}{EI} \left(\frac{1}{2}M_AL^2 + \frac{1}{6}R_AL^3 - \frac{1}{6}W(L-a)^3 \right) = 0 \quad (2.142)$$

Overall:
$$\begin{cases} R_A - W + R_B = 0 \\ M_A - Wb + R_AL - M_B = 0 \\ 2M_AL + R_AL^2 - W(L-a)^2 = 2M_AL + R_AL^2 - Wb^2 = 0 \\ 3M_AL^2 + R_AL^3 - W(L-a)^3 = 3M_AL^2 + R_AL^3 - Wb^3 = 0 \end{cases} \quad (2.143)$$

The R_A , R_B , M_A , and M_B are the 4 **unknowns** with 4 equations. Hence, they can be solved:

$$R_A = \frac{Wb^2}{L^3}(3a+b) \quad (2.144)$$

$$M_A = -\frac{Wab^2}{L^2} \quad (2.145)$$

$$R_B = \frac{Wa^2}{L^3}(3b+a) \quad (2.146)$$

$$M_B = -\frac{Wa^2b}{L^2} \quad (2.147)$$

The equation of deflection:

$$y = -\frac{W}{2EI} \left[\frac{b^2(3a+b)x^3}{3L^3} - \frac{ab^2x^2}{L^2} - \frac{(x-a)^3}{3} \right] \quad (2.148)$$

$$y(a) = \frac{Wa^3b^3}{3EIL^3} \quad (2.149)$$

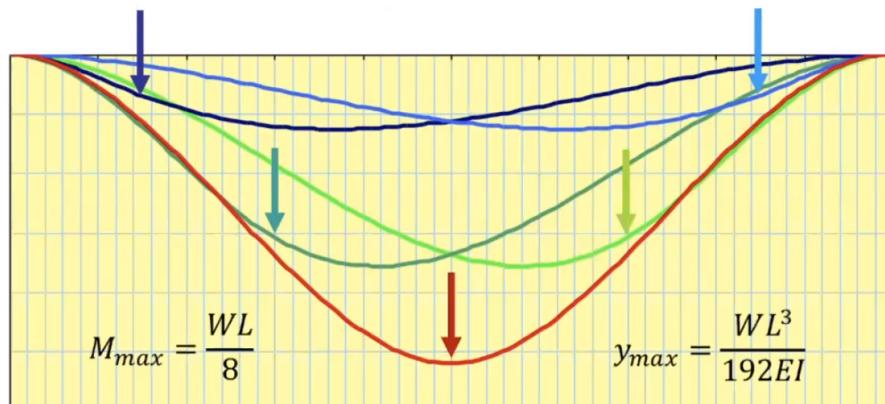
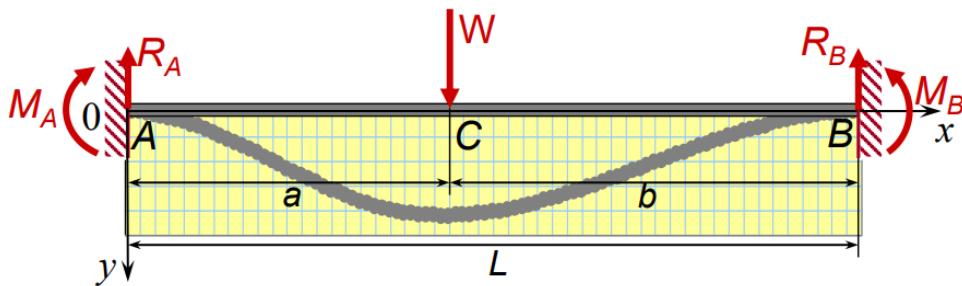


Figure 2.7: Depending on the loading, the profile of the deflection will be different. Maximum deflection occurs when $a = b = \frac{L}{2}$

Statically indeterminate structures produce **lower displacement and stress levels**, resulting in savings in material. **Failure** of a member in a statically determinate structure **leads to collapse**. In case of failure of a member, statically indeterminate structures can find **alternative load paths**, at least temporarily.

Chapter 3

Collapse Mechanism

20/11/2020

What you already know on bending is sufficient to enable the basic engineering design of static beam structures. Basic engineering design is primarily concerned with:

- Controlling elastic deformations
- Maintaining structures within their elastic range

However, for safety reasons, it is essential for engineers to be able to predict how a structure would fail if unexpected loading conditions occur.

3.1 Introduction

3.1.1 Elastic and Plastic Regimes

Stressing within the **elastic regime**:

- Material returns to original state upon removal of external actions
- Deformation depends solely upon stress and not upon load history

Exceeding the elastic regime, **plastic regime** is reached:

- Permanent distortions take place in the material
- Deformation depends upon stress and load history

Equations of solid mechanics apply to both cases.

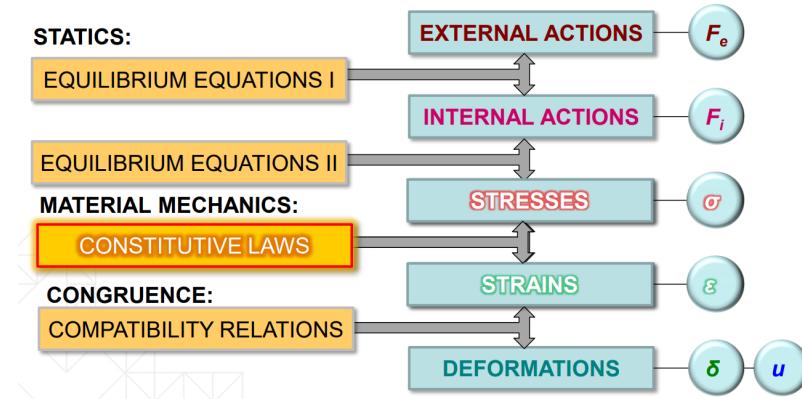


Figure 3.1: Solid Mechanics Equations: The relationship between actions, stresses, strains and deformations

3.1.2 Stress-Strain Plastic Relationship

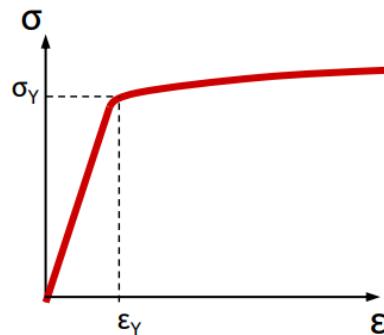


Figure 3.2: Typical elasto-plastic material

Typical elasto-plastic material:

- Region 1 - linear elastic behaviour up to yield stress σ_y
- Region 2 – non linear development with strain-hardening

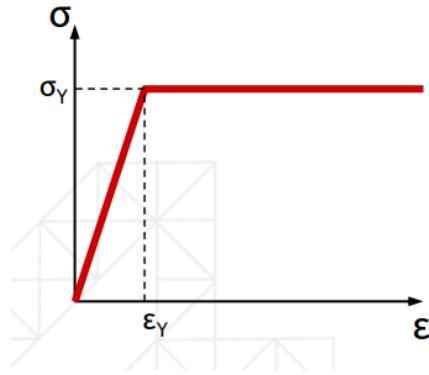


Figure 3.3: Perfectly elasto-plastic material

Perfectly elasto-plastic material:

- Region 1 - linear elastic behaviour up to yield stress σ_y
- Region 2 – strain increases at constant stress

Structural steels are elastoplastic materials and can be modelled as perfectly elasto-plastic (neglecting the strain-hardening is conservative for safety).

3.2 Plastic Theory of Collapse

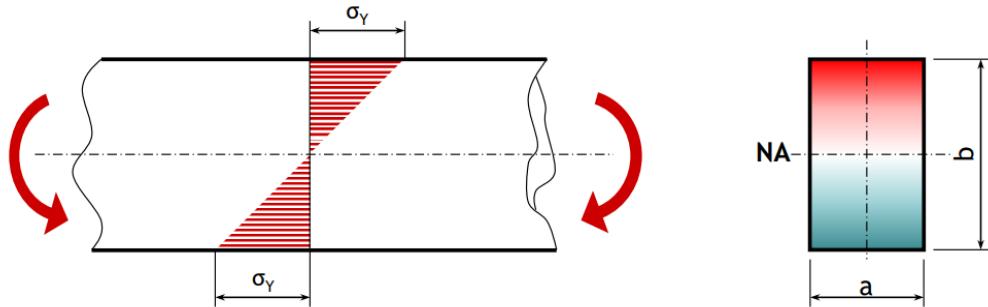
3.2.1 Bending and Plastic Collapse

Bending moment is by far the most relevant of the internal forces, since it produces the largest levels of deformations and stress into the beam. Therefore, **plastic collapse** of beam structures is commonly associated with **plastic bending**. Assumptions:

- Material is perfectly elasto-plastic. (in the plastic region stress will be constant)
- Yield stress is the same in tension and compression
- Transverse cross-sections remain plane (strain is proportional to the distance from the NA)
- When a cross-section is fully plastic (plastic hinge), its resisting moment remains constant until collapse of the whole structure
- Loads increase monotonically

3.2.2 Elastic Bending Moment

Consider a beam of rectangular cross-section, subjected to pure bending.



Maximum stress reaches the elastic limit σ_y when the bending moment is equal to:

$$M = \frac{EI}{R} \quad (3.1)$$

$$\sigma = \frac{Eb}{2R} \quad (3.2)$$

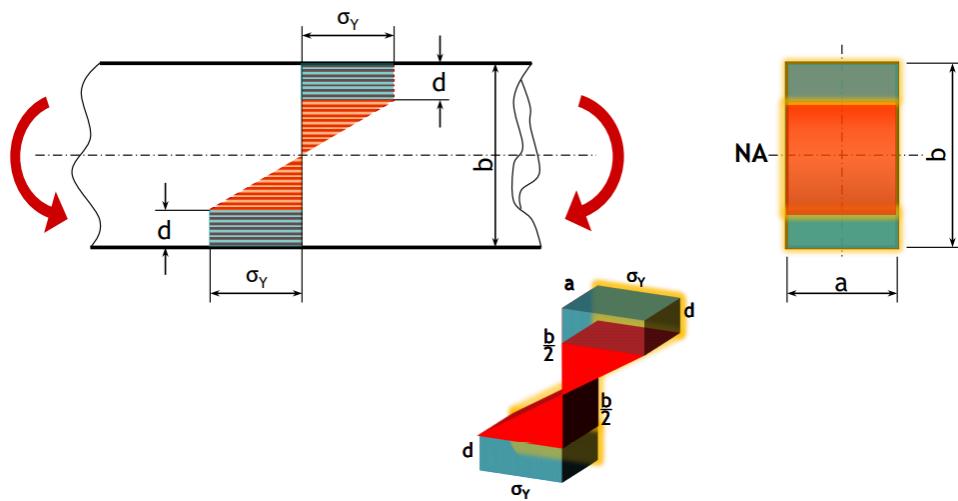
$$I = \frac{ab^3}{12} \quad (3.3)$$

$$\therefore \text{The yield bending moment : } M_Y = \sigma_y \frac{ab^2}{6} \quad (3.4)$$

At the elastic moment, all fibres are still in the elastic condition. If moment increases further, external fibres exceed the elastic limit and yield: deformation increases but stress keeps constant and equal to σ_y . With increasing bending moment, the plastic region penetrates deeper toward the Neutral Axis (NA).

3.2.3 Elasto-Plastic Bending Moment

Consider a beam of rectangular cross-section, subjected to pure bending.



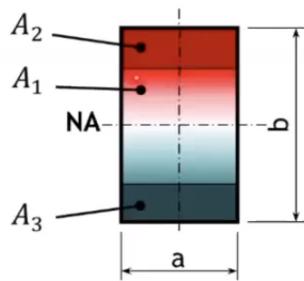
The moment is given by:

$$M = \frac{\sigma I}{h} \quad (3.5)$$

$$I = ? \quad (3.6)$$

$$\therefore \text{Elasto-plastic bending moment : } M = \frac{\sigma_Y ab^2}{6} \left[1 + 2\frac{d}{b} \left(1 - \frac{d}{b} \right) \right] \quad (3.7)$$

The derivation of the bending moment:



The cross-section is divided into 3 parts:

1. Elastic part in the middle (A_1)
2. Plastic part at the top (A_2)
3. Plastic part at the bottom (A_3)

$$M = \int_A dM = \int_A h \cdot \sigma dA \quad (3.8)$$

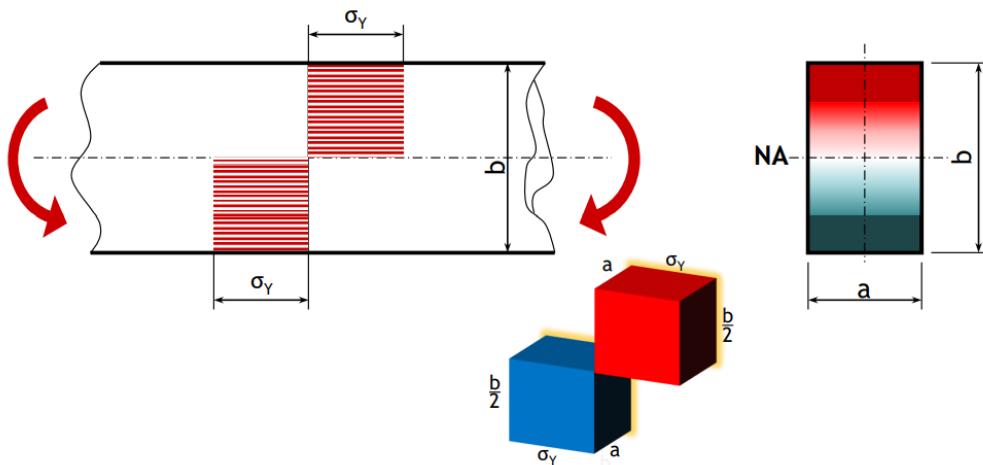
$$= \int_{A_1} \frac{\sigma_Y h}{b/2 - d} h dA + \int_{A_2} \sigma_Y h dA + \int_{A_3} \sigma_Y h dA \quad (3.9)$$

$$= \int_{-b/2+d}^{b/2-d} \frac{\sigma_Y a}{b/2 - d} h^2 dh + \int_{b/2-d}^{b/2} \sigma_Y ah dh + \int_{-b/2}^{-b/2+d} \sigma_Y ah dh \quad (3.10)$$

$$= \frac{\sigma_Y ab^2}{6} \left[1 + 2\frac{d}{b} \left(1 - \frac{d}{b} \right) \right] \quad (3.11)$$

3.2.4 Plastic Bending Moment

Consider a beam of rectangular cross-section, subjected to pure bending.



If bending moment keeps increasing, the plastic region propagate to the NA. In this case the moment is equal to:

$$\text{Plastic bending moment : } M_P = \sigma_Y \frac{ab^2}{4} \quad (3.12)$$

In practice, the plastic moment is given by the product of the yield stress by (the 1st moment of area (of the cross section) above the plastic NA + the 1st moment of area below the plastics NA).

3.2.5 Shape Factor

The yielding and plastic bending moments are different. In this case (rectangular cross-section) they are:

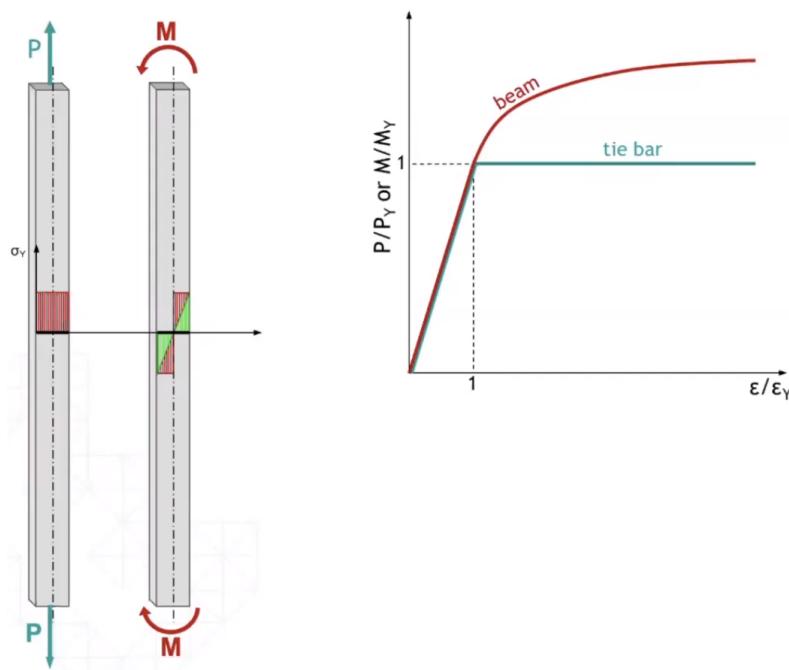
$$M_Y = \sigma_Y \frac{ab^2}{6} \quad M_P = \sigma_Y \frac{ab^2}{4} \quad (3.13)$$

The ratio $f = \frac{M_P}{M_Y}$ between the yielding bending moment and the plastic bending moment is solely a function of the shape of the cross-section, and it is called **shape factor**. In this case (rectangular cross-section) it is:

$$f = \frac{M_P}{M_Y} = \frac{\sigma_Y ab^2}{4} \frac{6}{\sigma_Y ab^2} = 1.5 \quad (3.14)$$

The shape factor is an indicator of the reserve strength that the beam can offer after yielding first begins.

3.2.6 Plastic Collapse Under Bending

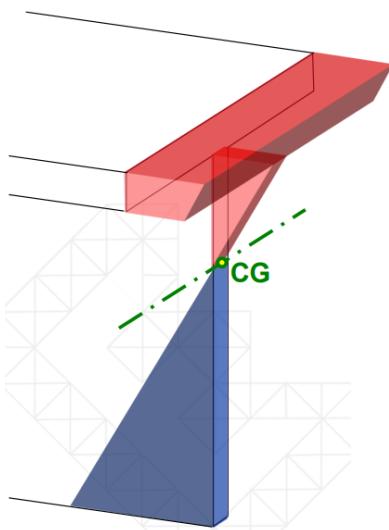


Contrary to the case of normal load, the plastic collapse of a beam under pure bending occurs at a moment greater than the yield moment.

For a rectangular cross section, it will occur at a moment 50% higher than the one that initiates yielding.

3.2.7 Neutral Axis with Asymmetry

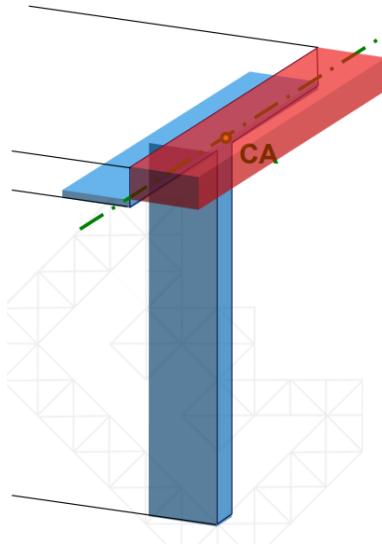
In the case of asymmetrical sections (though still singly symmetric), the analysis becomes more complex:



Equilibrium of forces in the ELASTIC case:

$$F = \int_A \sigma \cdot dA = \int_A \frac{M}{I} h \cdot dA = \frac{M}{I} \int_A h \cdot dA = 0 \quad (3.15)$$

The $\int_A h \cdot dA$ is the 1st moment of area. The **elastic neutral axis** corresponds to the centroid of the section (centre of gravity).



Equilibrium of forces in the PLASTIC case:

$$F = \int_A \sigma_Y \cdot dA = \int_{A_T} \sigma_Y \cdot dA - \int_{A_C} \sigma_Y \cdot dA = 0 \quad (3.16)$$

$$\therefore \sigma_Y A_T = \sigma_Y A_C \quad (3.17)$$

$$\therefore A_T = A_C \quad (3.18)$$

The **plastic neutral axis** corresponds to the centre of area (belongs to the line that divides the section into 2 equal areas).

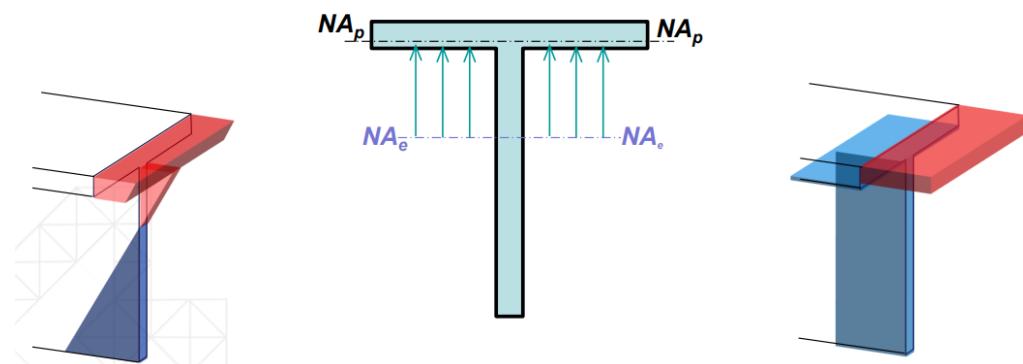
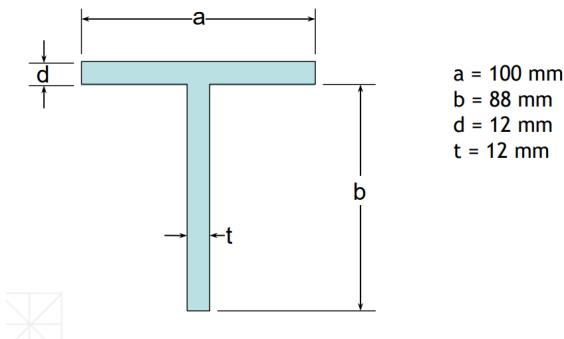


Figure 3.4: In the elasto-plastic phase, the neutral axis shifts from the centroid (centre of gravity) to the centre of area of the section

3.2.8 Example: Shape factor for T-beam

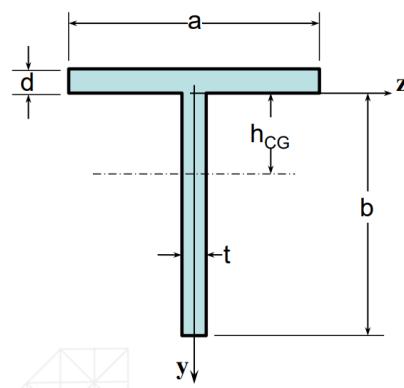


Yield moment M_Y and plastic moment M_P need to be calculated:

$$M_Y = \frac{\sigma_Y \cdot I}{h_{max}} \quad (3.19)$$

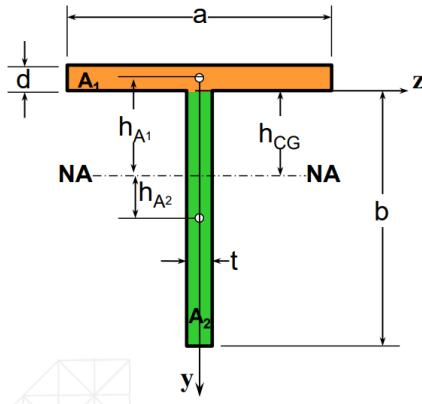
$$M_P = \sigma_Y(Q_1 + Q_2) \quad (3.20)$$

Elastic Neutral Axis:



$$\text{Centre of Gravity} = h_{CG} = \frac{\int_A h \cdot dA}{A} \quad (3.21)$$

2nd Moment of Area:



$$h_{A1} = h_{CG} + \frac{d}{2} = 23.4\text{mm} \quad (3.22)$$

$$h_{A2} = \frac{b}{2} - h_{CG} = -26.6\text{mm} \quad (3.23)$$

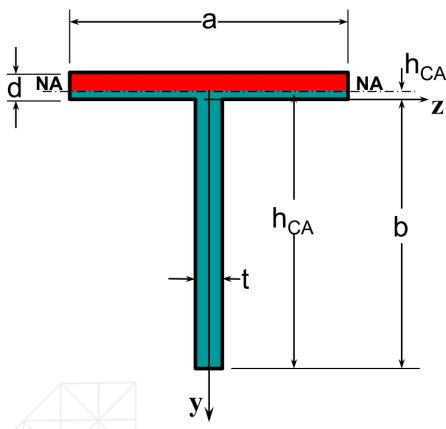
2nd moment of area (parallel axis theorem):

$$I_{zz} = (I_{A_1} + A_1 \cdot h_{A_1}^2) + (I_{A_2} + A_2 \cdot h_{A_2}^2) = 2.10 \cdot 10^6 \text{mm}^4 \quad (3.24)$$

Yield Moment:

$$M_Y = \frac{\sigma_Y \cdot I_{zz}}{h_{CG}} = 29745 \cdot \sigma_Y \quad (3.25)$$

Plastic Neutral Axis:

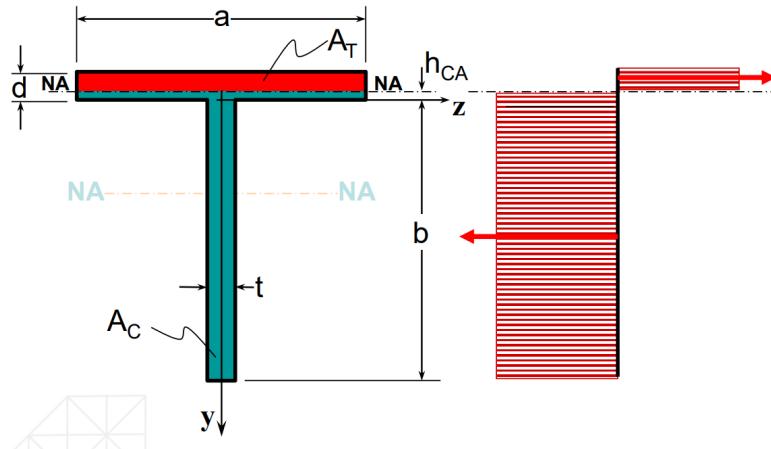


$$A_T = A_C = \frac{A_{total}}{2} \quad (3.26)$$

$$\text{Centre of Area} = h_{CA} = 0.72\text{mm} \quad (3.27)$$

Where A_T and A_C represent the areas of the top and bottom parts.

1st Moment of Area:



1st moment of the areas in tension & compression:

$$Q_{A_T} = \int_{A_T} h \cdot dA = 6362 \text{ mm}^3 \quad (3.28)$$

$$Q_{A_C} = \int_{A_C} h \cdot dA = 46846 \text{ mm}^3 \quad (3.29)$$

Plastic Moment:

$$M_P = \sigma_Y (Q_{A_T} + Q_{A_C}) = \sigma_Y \cdot 6362 + \sigma_Y \cdot 46846 = 53208 \cdot \sigma_Y \quad (3.30)$$

Shape Factor:

Calculated yield and plastic moments:

$$M_Y = \frac{\sigma_Y \cdot I_{zz}}{y_{CG}} = 29745 \cdot \sigma_Y \quad (3.31)$$

$$M_P = \sigma_Y (Q_{A_T} + Q_{A_C}) = 53208 \cdot \sigma_Y \quad (3.32)$$

$$\therefore \text{Shape Factor: } f = \frac{M_P}{M_Y} = 1.79 \quad (3.33)$$

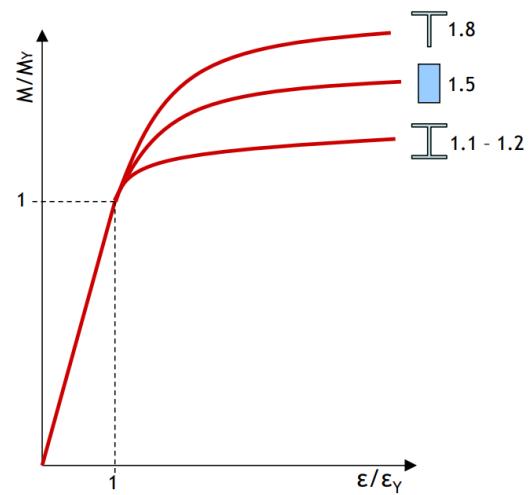


Figure 3.5: The shape factors of Rectangular, T-shaped and I-shaped cross sections