

# CEE6501 — Lecture 3.2

## The Direct Stiffness Method (DSM) for Trusses

# Learning Objectives

By the end of this lecture, you will be able to:

- Construct the **local-to-global transformation** for a truss member
- Compute element stiffness in global coordinates:

$$\mathbf{k} = \mathbf{T}^T \mathbf{k}' \mathbf{T}$$

- Assemble the **global stiffness matrix  $\mathbf{K}$**  using scatter-add
- Explain why an **unsupported structure** leads to a **singular stiffness matrix**
- Apply boundary conditions through **DOF partitioning** and solve for displacements
- Recover **member axial forces** from global displacements for design

# Agenda

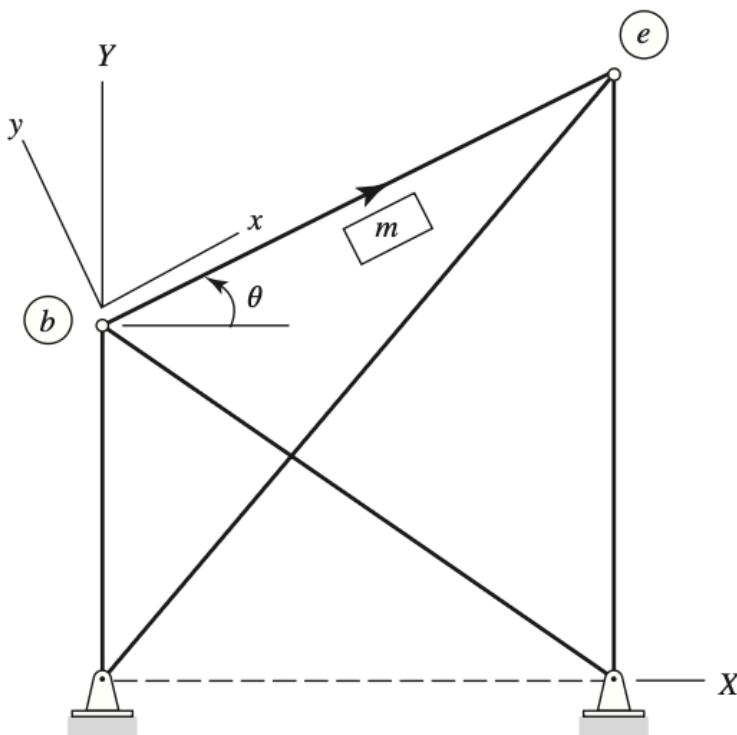
1. Local-to-global transformation using direction cosines
2. Member stiffness relations in the global coordinate system
3. Assembly of the global stiffness matrix  $\mathbf{K}$  (manual)
4. Assembly of the global stiffness matrix  $\mathbf{K}$  (DSM)
5. Constraints and supports (boundary conditions)
6. Partitioning into free and restrained DOFs
7. Solving for global displacements and reactions
8. Element force recovery
9. DSM summary

Big idea:

- A truss is a network of axial springs.
- Each element contributes stiffness to shared DOFs.
- Assembly is adding contributions into the right global rows/columns.

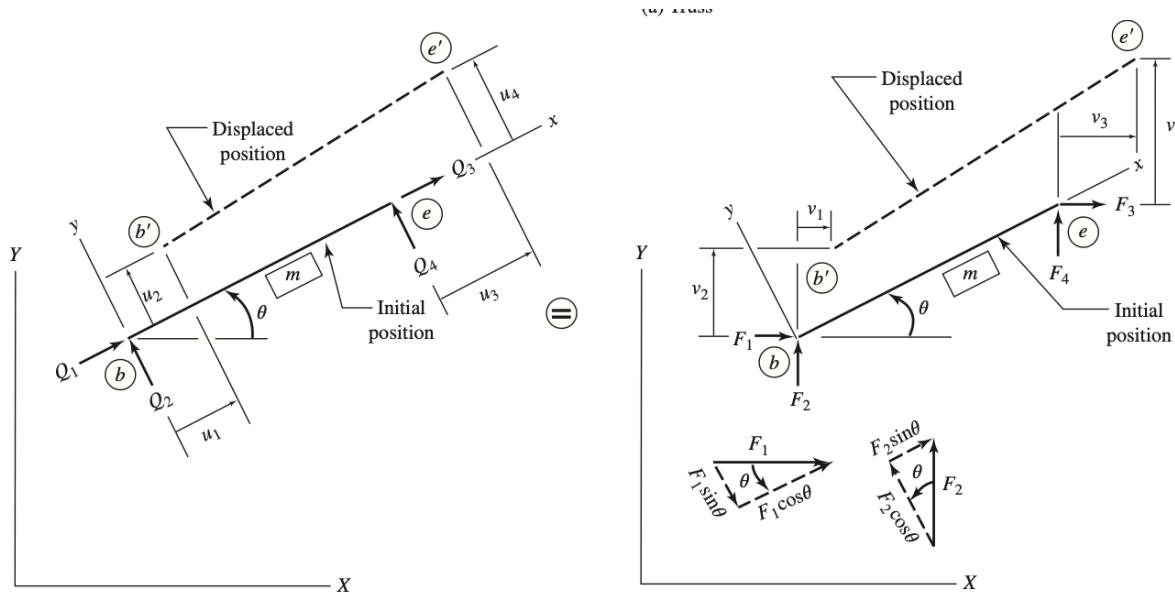
# Part 1 — Local to Global Transformation

# Generic Truss Element in a Structure



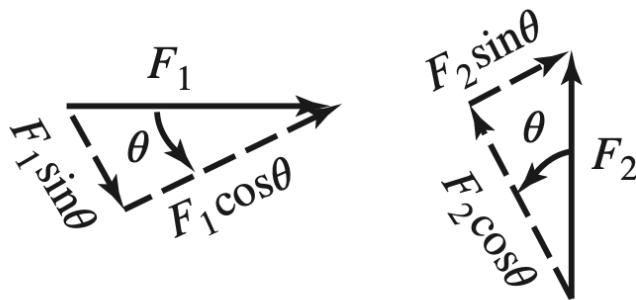
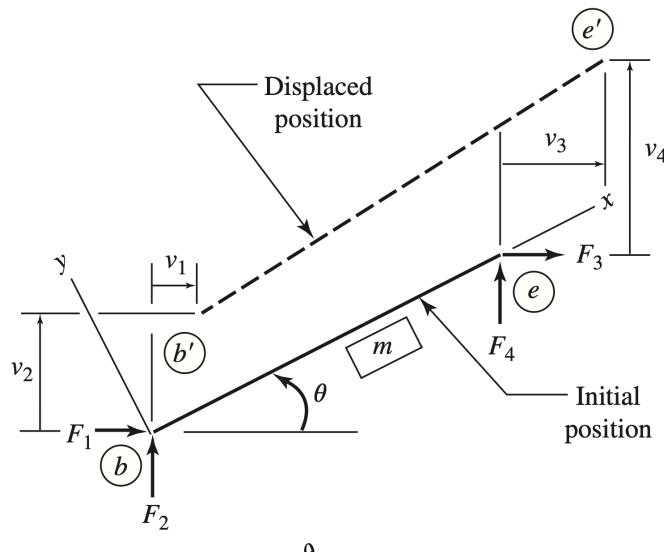
- A truss member is embedded in a **global coordinate system** ( $X, Y$ )
- The element stiffness was derived in a **local coordinate system** ( $x, y$ ) aligned with the member
- The member orientation is defined by an angle  $\theta$ , measured **counterclockwise** from global  $X$  to local  $x$
- Structural assembly requires **transforming forces and displacements** between local and global coordinates

# Local/Global Perspective



- **Local coordinate system (left):** forces  $\mathbf{Q}$ , displacements  $\mathbf{u}$
- **Global coordinate system (right):** forces  $\mathbf{F}$ , displacements  $\mathbf{v}$

# Global ( $\mathbf{F}$ ) → Local ( $\mathbf{Q}$ ) Force Transformation: Trigonometry



**At node  $b$  (start node):**

- $Q_1 = F_1 \cos \theta + F_2 \sin \theta$
- $Q_2 = -F_1 \sin \theta + F_2 \cos \theta$

**At node  $e$  (end node):**

- $Q_3 = F_3 \cos \theta + F_4 \sin \theta$
- $Q_4 = -F_3 \sin \theta + F_4 \cos \theta$

# Global ( $\mathbf{F}$ ) → Local ( $\mathbf{Q}$ ) Force Transformation: Matrix Form

- Local member forces  $\mathbf{Q}$  are obtained by **rotating** global nodal forces  $\mathbf{F}$  into the member's local coordinate system
- Each  $2 \times 2$  block applies a **rotation by  $\theta$**  at a node
- The transformation changes **direction only**, not force magnitude
- This operation is a **pure coordinate transformation**

$$\left\{ \begin{array}{c} Q_1 \\ Q_2 \\ Q_3 \\ Q_4 \end{array} \right\} = \underbrace{\begin{bmatrix} \cos \theta & \sin \theta & 0 & 0 \\ -\sin \theta & \cos \theta & 0 & 0 \\ 0 & 0 & \cos \theta & \sin \theta \\ 0 & 0 & -\sin \theta & \cos \theta \end{bmatrix}}_T \left\{ \begin{array}{c} F_1 \\ F_2 \\ F_3 \\ F_4 \end{array} \right\}$$

$$\mathbf{Q} = \mathbf{T}\mathbf{F}$$

## Direction Cosines: Rotation Terms

- Direction cosines define the **orientation of a truss member** in the global  $(X, Y)$  coordinate system
- The angle  $\theta$  is measured **countrerclockwise** from the global  $X$  axis to the local  $x$  axis
- Computed directly from the **nodal coordinates** of the element ( $b$  = start node,  $e$  = end node)

$$\cos \theta = \frac{X_e - X_b}{\sqrt{(X_e - X_b)^2 + (Y_e - Y_b)^2}}, \quad \sin \theta = \frac{Y_e - Y_b}{\sqrt{(X_e - X_b)^2 + (Y_e - Y_b)^2}}$$

- The denominator is the **member length  $L$**
- Once computed,  $\cos \theta$  and  $\sin \theta$  are **reused throughout the element formulation**

## Global ( $\mathbf{v}$ ) → Local ( $\mathbf{u}$ ) Displacements

- Nodal displacements are transformed using the **same rotation matrix** as forces
- Displacements and forces transform identically because they are defined along the **same directions**
- This is a **pure coordinate rotation**, not a change in deformation

$$\mathbf{u} = \mathbf{T}\mathbf{v}$$

- $\mathbf{v}$ : global displacement vector
- $\mathbf{u}$ : local displacement vector

## Local ( $\mathbf{Q}$ ) → Global ( $\mathbf{F}$ ) Force: Transformation Equations

- This is the **reverse of the global → local process**
- Local member forces are **rotated back** into the global ( $X, Y$ ) directions

**At node  $b$  (start node):**

$$F_1 = Q_1 \cos \theta - Q_2 \sin \theta, \quad F_2 = Q_1 \sin \theta + Q_2 \cos \theta$$

**At node  $e$  (end node):**

$$F_3 = Q_3 \cos \theta - Q_4 \sin \theta, \quad F_4 = Q_3 \sin \theta + Q_4 \cos \theta$$

# Local ( $\mathbf{Q}$ ) → Global ( $\mathbf{F}$ ) Force: Matrix Form

- Local member forces are mapped to global nodal forces using the **transpose** of the global→local transformation

$$\begin{Bmatrix} F_1 \\ F_2 \\ F_3 \\ F_4 \end{Bmatrix} = \underbrace{\begin{bmatrix} \cos \theta & -\sin \theta & 0 & 0 \\ \sin \theta & \cos \theta & 0 & 0 \\ 0 & 0 & \cos \theta & -\sin \theta \\ 0 & 0 & \sin \theta & \cos \theta \end{bmatrix}}_{\mathbf{T}^\top} \begin{Bmatrix} Q_1 \\ Q_2 \\ Q_3 \\ Q_4 \end{Bmatrix}$$

$$\mathbf{F} = \mathbf{T}^\top \mathbf{Q}$$

**Recall (Global → Local):**

$$\mathbf{T} = \begin{bmatrix} \cos \theta & \sin \theta & 0 & 0 \\ -\sin \theta & \cos \theta & 0 & 0 \\ 0 & 0 & \cos \theta & \sin \theta \\ 0 & 0 & -\sin \theta & \cos \theta \end{bmatrix}$$

## Properties of the Transformation Matrix

$$\mathbf{T}^{-1} = \mathbf{T}^T$$

The transformation matrix is **orthogonal**, which greatly simplifies operations involving the stiffness transformations.

# Summary — Local $\leftrightarrow$ Global Transformations

**Direction cosines (member orientation):**

$$\cos \theta = \frac{X_e - X_b}{L}, \quad \sin \theta = \frac{Y_e - Y_b}{L}$$

**Global  $\rightarrow$  Local (forces or displacements):**

$$\mathbf{Q} = \mathbf{T}\mathbf{F}, \quad \mathbf{u} = \mathbf{T}\mathbf{v}$$

**Local  $\rightarrow$  Global (forces or displacements):**

$$\mathbf{F} = \mathbf{T}^T \mathbf{Q}, \quad \mathbf{v} = \mathbf{T}^T \mathbf{u}$$

$$\mathbf{T} = \begin{bmatrix} \cos \theta & \sin \theta & 0 & 0 \\ -\sin \theta & \cos \theta & 0 & 0 \\ 0 & 0 & \cos \theta & \sin \theta \\ 0 & 0 & -\sin \theta & \cos \theta \end{bmatrix}$$

- $\mathbf{T}$  is a **pure rotation matrix** (no translation)
- $\mathbf{T}^{-1} = \mathbf{T}^T$  (orthogonal)

# Part 2 — Member Stiffness in the Global Coordinate System

## Goal

- We have derived the **local stiffness relation** (Lecture 3.1 today):

$$\mathbf{Q} = \mathbf{k}\mathbf{u}$$

- We also know how to **transform forces and displacements** between local and global systems
- Objective: express the **member stiffness relation entirely in global coordinates**

# Transformation Chain — From Local to Global (Step-by-Step)

## Step 1 — Local force–displacement relation

$$\mathbf{Q} = \mathbf{k} \mathbf{u}$$

The element stiffness matrix  $\mathbf{k}$  relates **local nodal displacements**  $\mathbf{u}$  to the corresponding **local nodal forces**  $\mathbf{Q}$ .

## Step 2 — Transform local forces to global forces

$$\mathbf{F} = \mathbf{T}^T \mathbf{Q}$$

Global nodal forces are obtained by rotating the local force vector into the global coordinate system.

Substitute the local stiffness relation from Step 1,  $\mathbf{Q} = \mathbf{k} \mathbf{u}$ , into the force transformation:

$$\mathbf{F} = \mathbf{T}^T \mathbf{Q} \implies \mathbf{F} = \mathbf{T}^T (\mathbf{k} \mathbf{u})$$

**Step 3 — Transform global displacements to local displacements**

$$\mathbf{u} = \mathbf{T} \mathbf{v}$$

Substitute the displacement transformation into the previous expression:

$$\mathbf{F} = \mathbf{T}^\top \mathbf{k} \mathbf{u} \implies \mathbf{F} = \mathbf{T}^\top \mathbf{k} (\mathbf{T} \mathbf{v})$$

**Step 4 — Rearrange into global stiffness form**

$$\mathbf{F} = (\mathbf{T}^\top \mathbf{k} \mathbf{T}) \mathbf{v}$$

## Step 5 — Final global stiffness relation

Define the global element stiffness matrix:

$$\mathbf{K} = \mathbf{T}^T \mathbf{k} \mathbf{T}$$

This is the element stiffness relation used directly in global assembly for the Direct Stiffness Method.

textbook notation:

$$\boxed{\mathbf{F} = \mathbf{K} \mathbf{v}}$$

our notation (interchangeable):

$$\boxed{\mathbf{f} = \mathbf{K} \mathbf{u}}$$

# Calculating the Global Stiffness Matrix, $\mathbf{K}$

The global element stiffness matrix is obtained by **rotating the local axial stiffness** into the global coordinate system:

$$\mathbf{K} = \begin{bmatrix} \cos \theta & -\sin \theta & 0 & 0 \\ \sin \theta & \cos \theta & 0 & 0 \\ 0 & 0 & \cos \theta & -\sin \theta \\ 0 & 0 & \sin \theta & \cos \theta \end{bmatrix} \cdot \frac{EA}{L} \begin{bmatrix} 1 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 \\ -1 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} \cdot \begin{bmatrix} \cos \theta & \sin \theta & 0 & 0 \\ -\sin \theta & \cos \theta & 0 & 0 \\ 0 & 0 & \cos \theta & \sin \theta \\ 0 & 0 & -\sin \theta & \cos \theta \end{bmatrix}$$

Carrying out the matrix multiplication yields the **closed-form global stiffness matrix**:

$$\mathbf{K} = \frac{EA}{L} \begin{bmatrix} \cos^2 \theta & \cos \theta \sin \theta & -\cos^2 \theta & -\cos \theta \sin \theta \\ \cos \theta \sin \theta & \sin^2 \theta & -\cos \theta \sin \theta & -\sin^2 \theta \\ -\cos^2 \theta & -\cos \theta \sin \theta & \cos^2 \theta & \cos \theta \sin \theta \\ -\cos \theta \sin \theta & -\sin^2 \theta & \cos \theta \sin \theta & \sin^2 \theta \end{bmatrix}$$

## Key Observations — Global Stiffness Matrix, $\mathbf{K}$

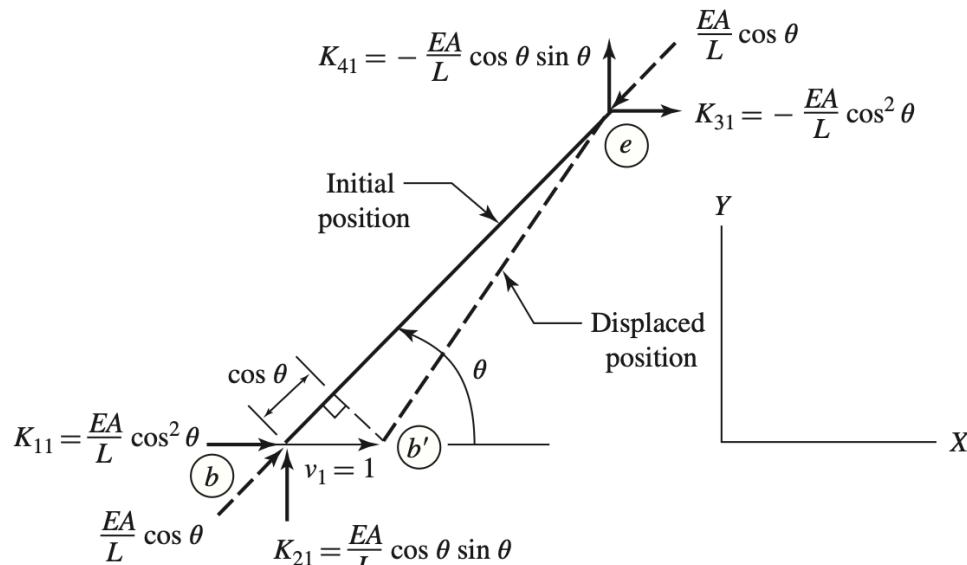
- The member global stiffness matrix  $\mathbf{K}$  is **symmetric**, just like the local stiffness matrix
- $\mathbf{K}$  represents the same physical behavior, but expressed in the **global ( $X, Y$ ) coordinate system**
- Each coefficient  $K_{ij}$  is the **force at global DOF  $i$**  required to produce a **unit displacement at global DOF  $j$** , with all other displacements fixed

## Direct Calculation of Global Stiffness Matrix, $\mathbf{K}$

- One could derive  $\mathbf{K}$  directly by:
  - Applying **unit global displacements** to a generic inclined truss member
  - Evaluating the **global end forces** required to produce each unit displacement in global coordinates
- The  $j$ th column of  $\mathbf{K}$  gives the global nodal force pattern caused by  $v_j = 1$
- This approach is **theoretically equivalent** to the transformation-based derivation and provides a clear physical interpretation of  $\mathbf{K}$ .
- However, it is **significantly more labor-intensive**, and is mainly useful as a **verification tool**, rather than for routine analysis.

# Example: First Column of $\mathbf{K}$

$$u_a = v_1 \cos \theta = 1 \cos \theta = \cos \theta$$



(a) First Column of  $\mathbf{K}$  ( $v_1 = 1, v_2 = v_3 = v_4 = 0$ )

- Impose  $v_1 = 1$ , all other global displacements zero
- Project the resulting axial deformation onto the member axis
- Resolve the axial force back into global components

You recover the **first column of  $\mathbf{K}$** , which should exactly match  $\mathbf{T}^\top \mathbf{kT}$ .

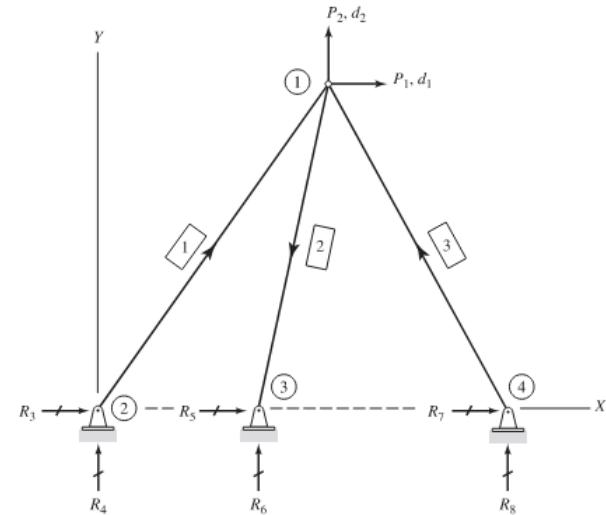
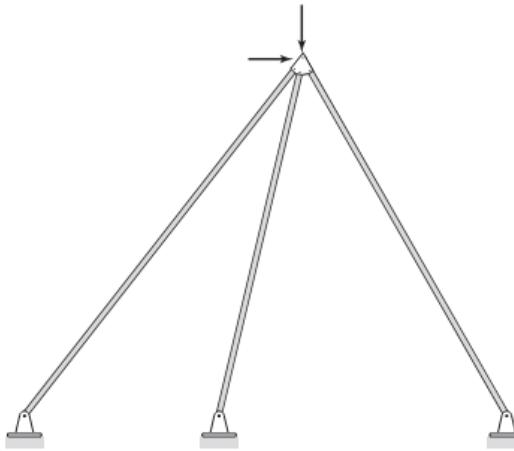
# Part 3 — Assembling the Global Structure Stiffness Matrix

*Manual Method*

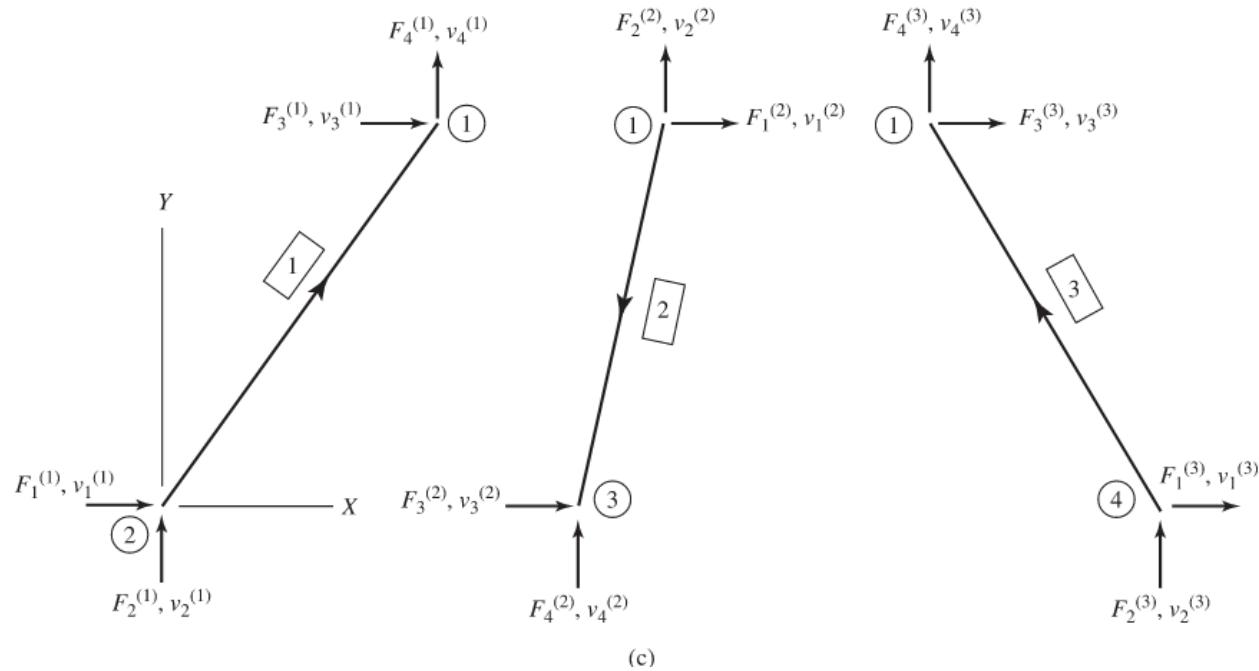
# Example Structure

We now move from **element-level** to a **complete truss structure** behavior.

- Structure composed of **3 axial truss elements**
- **4 nodes**, each with  $(X, Y)$  translational DOFs, node 1 at the top
- Total system size: **8 global degrees of freedom**
  - node 1: DOF (1,2)
  - node 2: DOF (3,4)
  - node 3: DOF (5,6)
  - node 4: DOF (7,8)



# Element-Level View of the Structure



- Nodes **2, 3, and 4** are **pinned** (no displacement)
- **Node 1** is free to move
- All three elements are connected at **node 1**
- A displacement at node 1 induces forces in **all connected members**

# Element Forces and Notation

Local forces are defined **per element**:

- Superscript ( $e$ ) → element number
- Subscript ( $i$ ) → local DOF index

Examples:

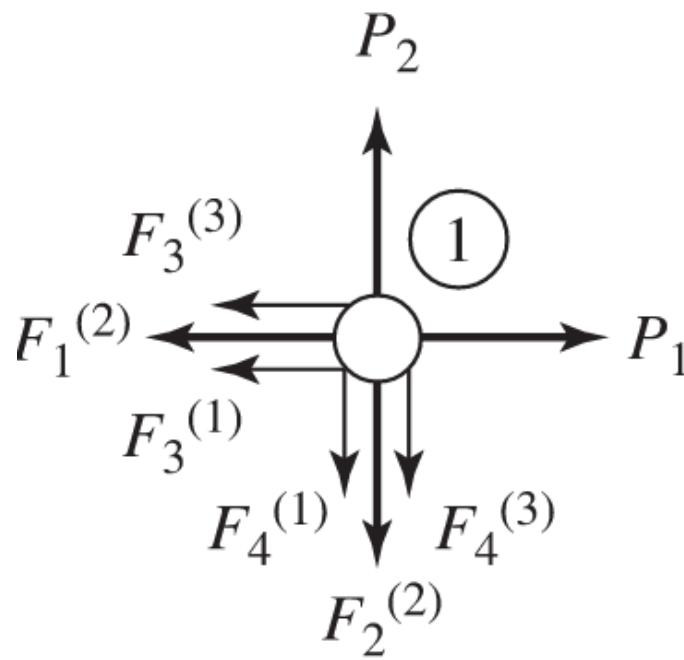
- $F_3^{(1)}$  → force at local DOF 3 in **element 1**
- $F_1^{(2)}$  → force at local DOF 1 in **element 2**

# Equilibrium Equations at Node 1

Because all elements share **node 1**, equilibrium at this node couples the response of all members.

$P$  = external applied loaded

$F$  = internal forces

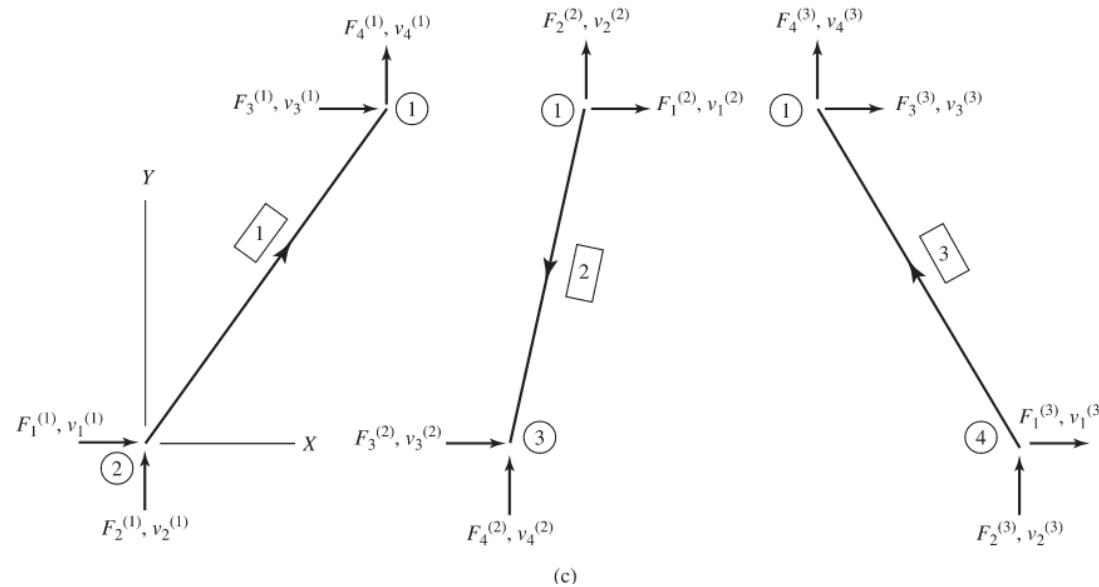


**Force equilibrium at node 1:**

$$P_1 = F_3^{(1)} + F_1^{(2)} + F_3^{(3)}$$

$$P_2 = F_4^{(1)} + F_2^{(2)} + F_4^{(3)}$$

# Compatibility Equations



$d_1$  and  $d_2$  are global displacements in X and Y at node 1

$$\text{Member (1)} : \quad v_1^{(1)} = v_2^{(1)} = 0, \quad v_3^{(1)} = d_1, \quad v_4^{(1)} = d_2$$

$$\text{Member (2)} : \quad v_1^{(2)} = d_1, \quad v_2^{(2)} = d_2, \quad v_3^{(2)} = v_4^{(2)} = 0$$

$$\text{Member (3)} : \quad v_1^{(3)} = v_2^{(3)} = 0, \quad v_3^{(3)} = d_1, \quad v_4^{(3)} = d_2$$

# Member 1 — Force–Displacement Relations

Compatibility (Member 1):  $v_1^{(1)} = v_2^{(1)} = 0$ ,  $v_3^{(1)} = d_1$ ,  $v_4^{(1)} = d_2$

Element-level global stiffness expression:

$$\left\{ \begin{array}{c} F_1^{(1)} \\ F_2^{(1)} \\ F_3^{(1)} \\ F_4^{(1)} \end{array} \right\} = \left[ \begin{array}{cccc} K_{11}^{(1)} & K_{12}^{(1)} & K_{13}^{(1)} & K_{14}^{(1)} \\ K_{21}^{(1)} & K_{22}^{(1)} & K_{23}^{(1)} & K_{24}^{(1)} \\ K_{31}^{(1)} & K_{32}^{(1)} & \boxed{K_{33}^{(1)}} & \boxed{K_{34}^{(1)}} \\ K_{41}^{(1)} & K_{42}^{(1)} & \boxed{K_{43}^{(1)}} & \boxed{K_{44}^{(1)}} \end{array} \right] \left\{ \begin{array}{c} v_1^{(1)} = 0 \\ v_2^{(1)} = 0 \\ v_3^{(1)} = d_1 \\ v_4^{(1)} = d_2 \end{array} \right\}$$

The forces acting at **end node of member 1** (Global node 1) are:

$$F_3^{(1)} = K_{33}^{(1)}d_1 + K_{34}^{(1)}d_2$$

$$F_4^{(1)} = K_{43}^{(1)}d_1 + K_{44}^{(1)}d_2$$

## Member 2 — Force–Displacement Relations

Compatibility (Member 2):  $v_1^{(2)} = d_1$ ,  $v_2^{(2)} = d_2$ ,  $v_3^{(2)} = v_4^{(2)} = 0$

Element-level global stiffness expression:

$$\begin{Bmatrix} F_1^{(2)} \\ F_2^{(2)} \\ F_3^{(2)} \\ F_4^{(2)} \end{Bmatrix} = \begin{bmatrix} K_{11}^{(2)} & K_{12}^{(2)} & K_{13}^{(2)} & K_{14}^{(2)} \\ K_{21}^{(2)} & K_{22}^{(2)} & K_{23}^{(2)} & K_{24}^{(2)} \\ K_{31}^{(2)} & K_{32}^{(2)} & K_{33}^{(2)} & K_{34}^{(2)} \\ K_{41}^{(2)} & K_{42}^{(2)} & K_{43}^{(2)} & K_{44}^{(2)} \end{bmatrix} \begin{Bmatrix} v_1^{(2)} = d_1 \\ v_2^{(2)} = d_2 \\ v_3^{(2)} = 0 \\ v_4^{(2)} = 0 \end{Bmatrix}$$

The forces acting at **start node of member 2** (Global node 1) are:

$$F_1^{(2)} = K_{11}^{(2)}d_1 + K_{12}^{(2)}d_2$$

$$F_2^{(2)} = K_{21}^{(2)}d_1 + K_{22}^{(2)}d_2$$

## Member 3 — Force–Displacement Relations

Compatibility (Member 3):  $v_1^{(3)} = v_2^{(3)} = 0$ ,  $v_3^{(3)} = d_1$ ,  $v_4^{(3)} = d_2$

Element-level global stiffness expression:

$$\left\{ \begin{array}{c} F_1^{(3)} \\ F_2^{(3)} \\ F_3^{(3)} \\ F_4^{(3)} \end{array} \right\} = \left[ \begin{array}{cccc} K_{11}^{(3)} & K_{12}^{(3)} & K_{13}^{(3)} & K_{14}^{(3)} \\ K_{21}^{(3)} & K_{22}^{(3)} & K_{23}^{(3)} & K_{24}^{(3)} \\ K_{31}^{(3)} & K_{32}^{(3)} & \boxed{K_{33}^{(3)}} & \boxed{K_{34}^{(3)}} \\ K_{41}^{(3)} & K_{42}^{(3)} & \boxed{K_{43}^{(3)}} & \boxed{K_{44}^{(3)}} \end{array} \right] \left\{ \begin{array}{l} v_1^{(3)} = 0 \\ v_2^{(3)} = 0 \\ v_3^{(3)} = d_1 \\ v_4^{(3)} = d_2 \end{array} \right\}$$

The forces acting at **end node of member 3** (Global node 1) are:

$$F_3^{(3)} = K_{33}^{(3)}d_1 + K_{34}^{(3)}d_2$$

$$F_4^{(3)} = K_{43}^{(3)}d_1 + K_{44}^{(3)}d_2$$

# Global Stiffness Matrix for Free DOFs

$$\begin{aligned}P_1 &= F_3^{(1)} + F_1^{(2)} + F_3^{(3)} \\P_2 &= F_4^{(1)} + F_2^{(2)} + F_4^{(3)}\end{aligned}$$

Substituting element-level force expressions (from last 3 slides) into the node 1 force equilibrium gives:

$$P_1 = (K_{33}^{(1)} + K_{11}^{(2)} + K_{33}^{(3)})d_1 + (K_{34}^{(1)} + K_{12}^{(2)} + K_{34}^{(3)})d_2$$

$$P_2 = (K_{43}^{(1)} + K_{21}^{(2)} + K_{43}^{(3)})d_1 + (K_{44}^{(1)} + K_{22}^{(2)} + K_{44}^{(3)})d_2$$

These equations can be written compactly as:

$$\left\{ \begin{array}{l} P_1 \\ P_2 \end{array} \right\} = \underbrace{\begin{bmatrix} K_{33}^{(1)} + K_{11}^{(2)} + K_{33}^{(3)} & K_{34}^{(1)} + K_{12}^{(2)} + K_{34}^{(3)} \\ K_{43}^{(1)} + K_{21}^{(2)} + K_{43}^{(3)} & K_{44}^{(1)} + K_{22}^{(2)} + K_{44}^{(3)} \end{bmatrix}}_{\mathbf{K}_s} \left\{ \begin{array}{l} d_1 \\ d_2 \end{array} \right\}$$

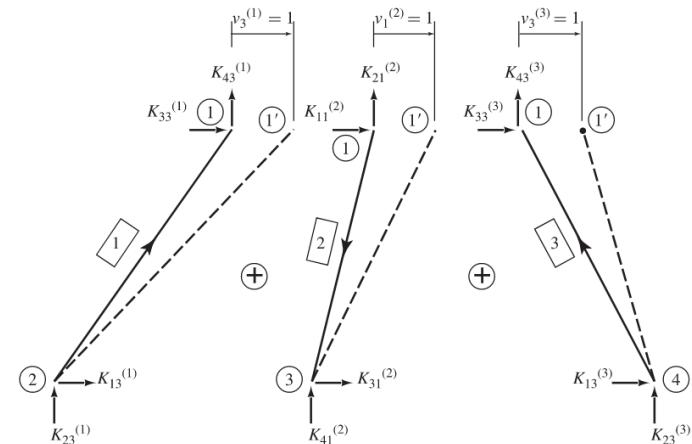
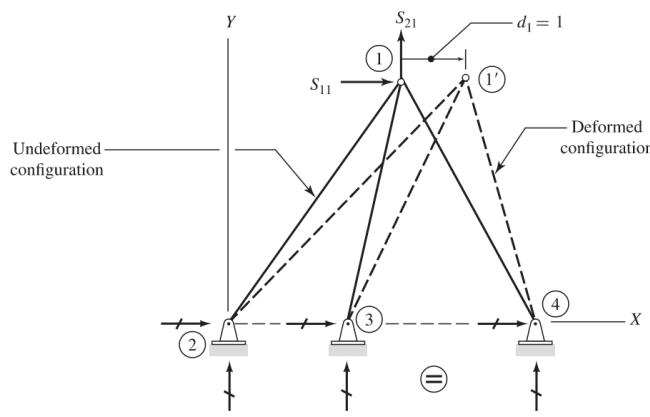
$$\boxed{\mathbf{P} = \mathbf{K}_s \mathbf{d}}$$

## Physical Interpretation of the Structure Stiffness Matrix

- The **structure stiffness matrix**  $\mathbf{K}_s$  has the same physical meaning as an element stiffness matrix, but at the **structure (joint) level**
- A stiffness coefficient  $K_{s,ij}$  represents:
  - The **joint force** at DOF  $i$
  - Required to cause a **unit displacement** at DOF  $j$
  - While **all other joint displacements are zero**
- Equivalently:
  - Each **column** of  $\mathbf{K}_s$  corresponds to a unit displacement pattern
  - The column entries are the **resulting joint forces** needed to enforce that displacement

# Example — First Column of $\mathbf{K}_s$

- To obtain the **first column** of  $\mathbf{K}_s$ :
    - Impose a **unit displacement** at the first free DOF:
- $$d_1 = 1, \quad d_2 = 0$$
- All other joint displacements are held fixed
  - The resulting joint forces (DOF 1, 2) define the **first column** of  $\mathbf{K}_s$



# Part 4 — Assembling the Global Structure Stiffness Matrix

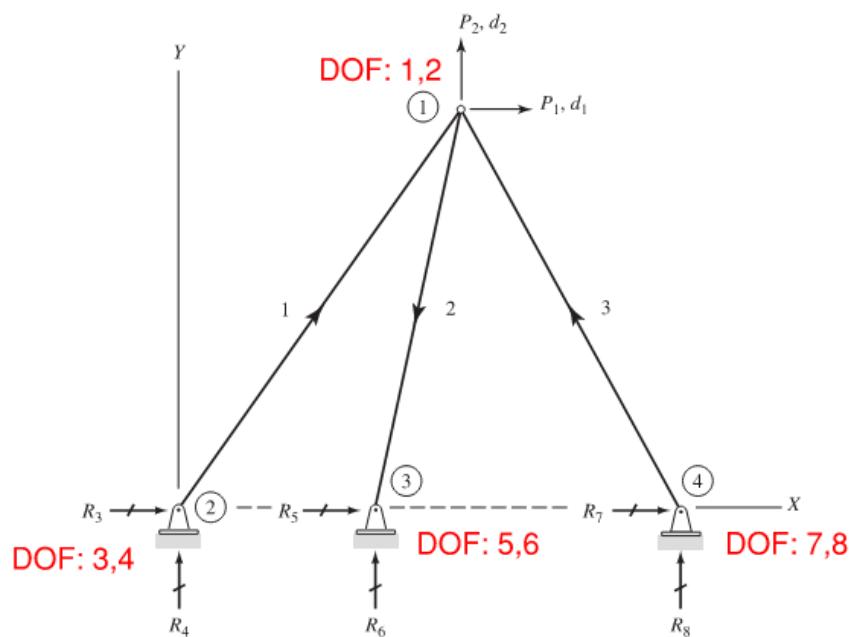
*Direct Stiffness Method*

## Why Not the Manual (Equilibrium + Compatibility) Approach?

- Becomes **tedious** very quickly
- Requires manually tracking:
  - Free vs constrained DOFs
  - Compatibility relationships
- Hard to implement **programmatically**
- Does **not scale** to larger or changing structures

**Key idea:** We want a method that is systematic, scalable, and algorithmic.

## Reminder — Global Degrees of Freedom



- Each node contributes two DOFs:  $(X, Y)$
- 4 nodes  $\rightarrow$  **8 global DOFs**
- Consistent numbering is essential for assembly

# Map Element DOFs to Global DOFs

$$\mathbf{K}_1 = \begin{bmatrix} K_{11}^{(1)} & K_{12}^{(1)} & K_{13}^{(1)} & K_{14}^{(1)} \\ K_{21}^{(1)} & K_{22}^{(1)} & K_{23}^{(1)} & K_{24}^{(1)} \\ K_{31}^{(1)} & K_{32}^{(1)} & K_{33}^{(1)} & K_{34}^{(1)} \\ K_{41}^{(1)} & K_{42}^{(1)} & K_{43}^{(1)} & K_{44}^{(1)} \end{bmatrix} \begin{array}{|c|} \hline 3 \\ \hline 4 \\ \hline 1 \\ \hline 2 \\ \hline \end{array}$$

$$\mathbf{K}_2 = \begin{bmatrix} K_{11}^{(2)} & K_{12}^{(2)} & K_{13}^{(2)} & K_{14}^{(2)} \\ K_{21}^{(2)} & K_{22}^{(2)} & K_{23}^{(2)} & K_{24}^{(2)} \\ K_{31}^{(2)} & K_{32}^{(2)} & K_{33}^{(2)} & K_{34}^{(2)} \\ K_{41}^{(2)} & K_{42}^{(2)} & K_{43}^{(2)} & K_{44}^{(2)} \end{bmatrix} \begin{array}{|c|} \hline 1 \\ \hline 2 \\ \hline 5 \\ \hline 6 \\ \hline \end{array}$$

$$\mathbf{K}_3 = \begin{bmatrix} K_{11}^{(3)} & K_{12}^{(3)} & K_{13}^{(3)} & K_{14}^{(3)} \\ K_{21}^{(3)} & K_{22}^{(3)} & K_{23}^{(3)} & K_{24}^{(3)} \\ K_{31}^{(3)} & K_{32}^{(3)} & K_{33}^{(3)} & K_{34}^{(3)} \\ K_{41}^{(3)} & K_{42}^{(3)} & K_{43}^{(3)} & K_{44}^{(3)} \end{bmatrix} \begin{array}{|c|} \hline 7 \\ \hline 8 \\ \hline 1 \\ \hline 2 \\ \hline \end{array}$$

Each element has **4 local DOFs** (1, 2, 3, 4), which must be mapped to the correct **global DOFs** before assembly.

For this structure:

- **Element 1:** local (1, 2, 3, 4) → global (3, 4, 1, 2)
- **Element 2:** local (1, 2, 3, 4) → global (1, 2, 5, 6)
- **Element 3:** local (1, 2, 3, 4) → global (7, 8, 1, 2)

This mapping determines **which rows and columns** of the global stiffness matrix each element stiffness contributes to.

# Method 1: Assemble a Structure Matrix for the Free DOFs

$$\mathbf{K}_1 = \begin{bmatrix} 3 & 4 & 1 & 2 \\ K_{11}^{(1)} & K_{12}^{(1)} & K_{13}^{(1)} & K_{14}^{(1)} \\ K_{21}^{(1)} & K_{22}^{(1)} & K_{23}^{(1)} & K_{24}^{(1)} \\ K_{31}^{(1)} & K_{32}^{(1)} & K_{33}^{(1)} & K_{34}^{(1)} \\ K_{41}^{(1)} & K_{42}^{(1)} & K_{43}^{(1)} & K_{44}^{(1)} \end{bmatrix}$$

$$\mathbf{K}_2 = \begin{bmatrix} 1 & 2 & 5 & 6 \\ K_{11}^{(2)} & K_{12}^{(2)} & K_{13}^{(2)} & K_{14}^{(2)} \\ K_{21}^{(2)} & K_{22}^{(2)} & K_{23}^{(2)} & K_{24}^{(2)} \\ K_{31}^{(2)} & K_{32}^{(2)} & K_{33}^{(2)} & K_{34}^{(2)} \\ K_{41}^{(2)} & K_{42}^{(2)} & K_{43}^{(2)} & K_{44}^{(2)} \end{bmatrix}$$

$$\mathbf{K}_3 = \begin{bmatrix} 7 & 8 & 1 & 2 \\ K_{11}^{(3)} & K_{12}^{(3)} & K_{13}^{(3)} & K_{14}^{(3)} \\ K_{21}^{(3)} & K_{22}^{(3)} & K_{23}^{(3)} & K_{24}^{(3)} \\ K_{31}^{(3)} & K_{32}^{(3)} & K_{33}^{(3)} & K_{34}^{(3)} \\ K_{41}^{(3)} & K_{42}^{(3)} & K_{43}^{(3)} & K_{44}^{(3)} \end{bmatrix}$$

$$\mathbf{S} = \begin{bmatrix} 1 & 2 \\ K_{33}^{(1)} + K_{11}^{(2)} + K_{33}^{(3)} & K_{34}^{(1)} + K_{12}^{(2)} + K_{34}^{(3)} \\ K_{43}^{(1)} + K_{21}^{(2)} + K_{43}^{(3)} & K_{44}^{(1)} + K_{22}^{(2)} + K_{44}^{(3)} \end{bmatrix}$$

In this example, only **global DOFs 1 and 2** are free; all other DOFs are constrained.

- Contributions from **each element** that map to DOFs 1 and 2 are **summed**
- This produces a reduced **structure stiffness matrix** for the free DOFs
- The result matches the stiffness matrix obtained earlier using the **manual equilibrium and compatibility** approach

For example, the (1, 1) entry of the structure stiffness matrix is (matches what we did before):

$$K_{s,11} = K_{33}^{(1)} + K_{11}^{(2)} + K_{33}^{(3)}$$

## Method 2: Assemble the Full Global Stiffness Matrix

When assembling the **global structure stiffness matrix**, do **not** worry yet about which DOFs are constrained or free.

- **Assemble all DOFs first** into a single global matrix
- **Apply boundary conditions later** (supports, prescribed displacements)
- This separation is what makes the method **general, systematic, and scalable**

For the structure shown, there are **8 global DOFs**, so  $8 \times 8$  stiffness matrix:

$$\mathbf{K} = \begin{bmatrix} K_{11} & K_{12} & K_{13} & K_{14} & K_{15} & K_{16} & K_{17} & K_{18} \\ K_{21} & K_{22} & K_{23} & K_{24} & K_{25} & K_{26} & K_{27} & K_{28} \\ K_{31} & K_{32} & K_{33} & K_{34} & K_{35} & K_{36} & K_{37} & K_{38} \\ K_{41} & K_{42} & K_{43} & K_{44} & K_{45} & K_{46} & K_{47} & K_{48} \\ K_{51} & K_{52} & K_{53} & K_{54} & K_{55} & K_{56} & K_{57} & K_{58} \\ K_{61} & K_{62} & K_{63} & K_{64} & K_{65} & K_{66} & K_{67} & K_{68} \\ K_{71} & K_{72} & K_{73} & K_{74} & K_{75} & K_{76} & K_{77} & K_{78} \\ K_{81} & K_{82} & K_{83} & K_{84} & K_{85} & K_{86} & K_{87} & K_{88} \end{bmatrix}$$

Each entry  $K_{ij}$  represents the **combined stiffness contribution** from all elements that connect global DOF  $j$  to global DOF  $i$ .



# Element 1 — Contribution to Global Stiffness Matrix

**Local → Global DOF mapping (Element 1):**

$$(1, 2, 3, 4) \rightarrow (3, 4, 1, 2)$$

$$\mathbf{K}_1 = \begin{bmatrix} 3 & 4 & 1 & 2 \\ K_{11}^{(1)} & K_{12}^{(1)} & K_{13}^{(1)} & K_{14}^{(1)} \\ K_{21}^{(1)} & K_{22}^{(1)} & K_{23}^{(1)} & K_{24}^{(1)} \\ K_{31}^{(1)} & K_{32}^{(1)} & K_{33}^{(1)} & K_{34}^{(1)} \\ K_{41}^{(1)} & K_{42}^{(1)} & K_{43}^{(1)} & K_{44}^{(1)} \end{bmatrix} \quad \begin{matrix} 3 \\ 4 \\ 1 \\ 2 \end{matrix}$$
  

$$\mathbf{K} = \begin{bmatrix} K_{11} & K_{12} & K_{13} & K_{14} & K_{15} & K_{16} & K_{17} & K_{18} \\ K_{21} & K_{22} & K_{23} & K_{24} & K_{25} & K_{26} & K_{27} & K_{28} \\ K_{31} & K_{32} & K_{33} & K_{34} & K_{35} & K_{36} & K_{37} & K_{38} \\ K_{41} & K_{42} & K_{43} & K_{44} & K_{45} & K_{46} & K_{47} & K_{48} \\ K_{51} & K_{52} & K_{53} & K_{54} & K_{55} & K_{56} & K_{57} & K_{58} \\ K_{61} & K_{62} & K_{63} & K_{64} & K_{65} & K_{66} & K_{67} & K_{68} \\ K_{71} & K_{72} & K_{73} & K_{74} & K_{75} & K_{76} & K_{77} & K_{78} \\ K_{81} & K_{82} & K_{83} & K_{84} & K_{85} & K_{86} & K_{87} & K_{88} \end{bmatrix}$$

# Element 2 — Contribution to Global Stiffness Matrix

**Local → Global DOF mapping (Element 2):**

$$(1, 2, 3, 4) \rightarrow (1, 2, 5, 6)$$

$$\mathbf{K}_2 = \begin{bmatrix} 1 & 2 & 5 & 6 \\ K_{11}^{(2)} & K_{12}^{(2)} & K_{13}^{(2)} & K_{14}^{(2)} \\ K_{21}^{(2)} & K_{22}^{(2)} & K_{23}^{(2)} & K_{24}^{(2)} \\ K_{31}^{(2)} & K_{32}^{(2)} & K_{33}^{(2)} & K_{34}^{(2)} \\ K_{41}^{(2)} & K_{42}^{(2)} & K_{43}^{(2)} & K_{44}^{(2)} \end{bmatrix} \begin{matrix} 1 \\ 2 \\ 5 \\ 6 \end{matrix}$$
  

$$\mathbf{K} = \begin{bmatrix} K_{11} & K_{12} & K_{13} & K_{14} & K_{15} & K_{16} & K_{17} & K_{18} \\ K_{21} & K_{22} & K_{23} & K_{24} & K_{25} & K_{26} & K_{27} & K_{28} \\ K_{31} & K_{32} & K_{33} & K_{34} & K_{35} & K_{36} & K_{37} & K_{38} \\ K_{41} & K_{42} & K_{43} & K_{44} & K_{45} & K_{46} & K_{47} & K_{48} \\ K_{51} & K_{52} & K_{53} & K_{54} & K_{55} & K_{56} & K_{57} & K_{58} \\ K_{61} & K_{62} & K_{63} & K_{64} & K_{65} & K_{66} & K_{67} & K_{68} \\ K_{71} & K_{72} & K_{73} & K_{74} & K_{75} & K_{76} & K_{77} & K_{78} \\ K_{81} & K_{82} & K_{83} & K_{84} & K_{85} & K_{86} & K_{87} & K_{88} \end{bmatrix}$$

# Element 3 — Contribution to Global Stiffness Matrix

**Local → Global DOF mapping (Element 3):**

$$(1, 2, 3, 4) \rightarrow (7, 8, 1, 2)$$

$$\mathbf{K}_3 = \begin{bmatrix} 7 & 8 & 1 & 2 \\ K_{11}^{(3)} & K_{12}^{(3)} & K_{13}^{(3)} & K_{14}^{(3)} \\ K_{21}^{(3)} & K_{22}^{(3)} & K_{23}^{(3)} & K_{24}^{(3)} \\ K_{31}^{(3)} & K_{32}^{(3)} & K_{33}^{(3)} & K_{34}^{(3)} \\ K_{41}^{(3)} & K_{42}^{(3)} & K_{43}^{(3)} & K_{44}^{(3)} \end{bmatrix}$$

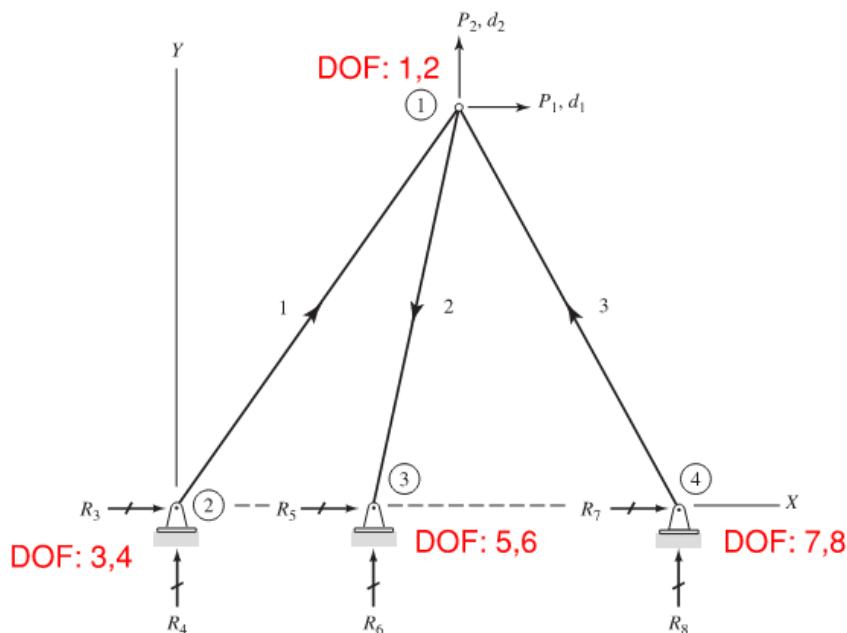
$$\mathbf{K} = \begin{bmatrix} K_{11} & K_{12} & K_{13} & K_{14} & K_{15} & K_{16} & K_{17} & K_{18} \\ K_{21} & K_{22} & K_{23} & K_{24} & K_{25} & K_{26} & K_{27} & K_{28} \\ K_{31} & K_{32} & K_{33} & K_{34} & K_{35} & K_{36} & K_{37} & K_{38} \\ K_{41} & K_{42} & K_{43} & K_{44} & K_{45} & K_{46} & K_{47} & K_{48} \\ K_{51} & K_{52} & K_{53} & K_{54} & K_{55} & K_{56} & K_{57} & K_{58} \\ K_{61} & K_{62} & K_{63} & K_{64} & K_{65} & K_{66} & K_{67} & K_{68} \\ K_{71} & K_{72} & K_{73} & K_{74} & K_{75} & K_{76} & K_{77} & K_{78} \\ K_{81} & K_{82} & K_{83} & K_{84} & K_{85} & K_{86} & K_{87} & K_{88} \end{bmatrix}$$

# Final Global Structure Stiffness Matrix

- Each  $K_{ij}$  may include **multiple element contributions**
- The matrix is **symmetric**
- Diagonal terms,  $K_{ii}$ , must be non-zero and positive for stability

$$\mathbf{K} = \begin{bmatrix} K_{11} & K_{12} & K_{13} & K_{14} & K_{15} & K_{16} & K_{17} & K_{18} \\ K_{21} & K_{22} & K_{23} & K_{24} & K_{25} & K_{26} & K_{27} & K_{28} \\ K_{31} & K_{32} & K_{33} & K_{34} & 0 & 0 & 0 & 0 \\ K_{41} & K_{42} & K_{43} & K_{44} & 0 & 0 & 0 & 0 \\ K_{51} & K_{52} & 0 & 0 & K_{55} & K_{56} & 0 & 0 \\ K_{61} & K_{62} & 0 & 0 & K_{65} & K_{66} & 0 & 0 \\ K_{71} & K_{72} & 0 & 0 & 0 & 0 & K_{77} & K_{78} \\ K_{81} & K_{82} & 0 & 0 & 0 & 0 & K_{87} & K_{88} \end{bmatrix}$$

# Interpreting Zeros in the Stiffness Matrix



- Zero entry  $\rightarrow$  no stiffness coupling
- Example:  $K_{37} = 0$
- DOF 3 and DOF 7 share no connecting element

# Part 5 — Constraints and Supports

## Inverting the Global Stiffness Matrix

You might think: *we have  $\mathbf{K}$ , so we can just solve*

$$\mathbf{K} \mathbf{u} = \mathbf{f}$$

and we're done.

However, for a **free (unconstrained) structure**, this step is **not yet valid**.

At this stage—before applying supports or boundary conditions—the global stiffness matrix typically contains **rigid-body modes** (motions that produce no strain energy). As a result:

- $\mathbf{K}$  is **singular** (not invertible)
- the structure still has **rigid-body motion** because we haven't applied supports / boundary conditions yet

**Important note (about our 8 DOF example):**

In our  $8 \times 8$  formulation, the ability to invert  $\mathbf{K}$  is an **artifact of how the matrix is set up**. The assembly already embeds enough *support-like stiffness*—intentionally or unintentionally—to suppress rigid-body motion.

As a result, the structure represented by this  $\mathbf{K}$  is **not truly free**, even though we have not yet explicitly applied boundary conditions.

In general, however, a global stiffness matrix assembled for an unconstrained structure **will be singular**.

This is why the proper and robust approach is always to apply supports explicitly (via partitioning) before solving.

In [1]:

```
import numpy as np

# Construct a singular 6x6 matrix
# Last row = sum of first two rows → linear dependence
K = np.array([
    [ 2, -1,  0,  0,  0,  0],
    [-1,  2, -1,  0,  0,  0],
    [ 0, -1,  2, -1,  0,  0],
    [ 0,  0, -1,  2, -1,  0],
    [ 0,  0,  0, -1,  2, -1],
    [ 1,  1, -1,  0,  0,  0] # dependent row
], dtype=float)

try:
    np.linalg.inv(K)
except np.linalg.LinAlgError as e:
    print("Inversion failed:", e)
```

Inversion failed: Singular matrix

## Why do we get a singular stiffness matrix?

Mathematically, an unconstrained structure admits non-zero displacement vectors  $\mathbf{u} \neq \mathbf{0}$  such that

$$\mathbf{K}\mathbf{u} = \mathbf{0}$$

These displacement patterns are **rigid-body (zero-energy) modes**, meaning  $\mathbf{K}$  has a **non-trivial nullspace** and is therefore **singular**.

## What does this mean in practice?

As a result, the structure can move without inducing member deformation:

- **Rigid translation in  $X$**
- **Rigid translation in  $Y$**
- **Rigid-body rotation** (geometry-dependent)

These motions are **rigid-body modes**, not structural deformations.

Because rigid-body modes produce zero internal force,  $\mathbf{K}$  cannot uniquely map applied loads to displacements. **Supports (boundary conditions)** remove these modes and render  $\mathbf{K}$  invertible.

## Boundary conditions: how supports enter the model

Supports and constraints are enforced by specifying **known displacements** (a.k.a. prescribed DOFs).

### Common idealized supports (2D truss):

- **Pin support:** Prescribes both translations:  $u_{ix} = 0$  and  $u_{iy} = 0$
- **Roller support:** Prescribes one translation and allows the other:

### Prescribed displacement (settlement / actuation):

- You can prescribe a non-zero value, e.g.,  $u_{iy} = -2$  mm at a support.

Why this fixes singularity:

- Constraints remove rigid-body modes by preventing global translations/rotations.
- The remaining free DOFs correspond to true structural deformation.

## What we do next (workflow)

We do **not** modify the element --> global matrix formulation to handle supports.

Instead, the Direct Stiffness Method proceeds like this:

1. Assemble the **full** global matrix  $\mathbf{K}$  and global load vector  $\mathbf{F}$
2. Identify which DOFs are **free** vs **restrained**
3. Enforce boundary conditions by **partitioning** (or equivalently, by eliminating restrained DOFs)
4. Solve only for the unknown (free) displacements
5. Recover reactions at restrained DOFs

So supports are handled cleanly at the **system equation** level — via partitioning (next section).

# Part 6 — Partitioning the Matrix

*Free vs Restrained DOFs*

## Core idea — Partition DOFs (free vs restrained)

Separate the global degrees of freedom into two sets:

- **Free DOFs  $f$ :** unknown displacements (to be solved)
- **Restrained DOFs  $r$ :** prescribed displacements (from supports or constraints)

Reorder the global displacement and force vectors so that all free DOFs come first:

$$\boldsymbol{u} = \begin{bmatrix} \boldsymbol{u}_f \\ \boldsymbol{u}_r \end{bmatrix}, \quad \boldsymbol{F} = \begin{bmatrix} \boldsymbol{F}_f \\ \boldsymbol{F}_r \end{bmatrix}$$

Apply the **same reordering** to the global stiffness matrix to obtain a partitioned system:

$$\begin{bmatrix} \boldsymbol{K}_{ff} & \boldsymbol{K}_{fr} \\ \boldsymbol{K}_{rf} & \boldsymbol{K}_{rr} \end{bmatrix} \begin{bmatrix} \boldsymbol{u}_f \\ \boldsymbol{u}_r \end{bmatrix} = \begin{bmatrix} \boldsymbol{F}_f \\ \boldsymbol{F}_r \end{bmatrix}$$

**Note:** In our **8x8 truss example**, the system already appears partitioned because the two free global DOFs were labeled **1 and 2**, placing them in the correct positions from the start.

To illustrate the **general case**, the next slides use a separate **6x6 example** in which free and restrained DOFs are initially **intermixed**. The goal there is to show how the system is **reordered** so that all free DOFs appear first.

# Step 0 — Original ordering (free DOFs = 3, 5)

$$\boldsymbol{u} = \begin{bmatrix} u_1 \\ u_2 \\ \boxed{u_3} \\ u_4 \\ \boxed{u_5} \\ u_6 \end{bmatrix}, \quad \boldsymbol{F} = \begin{bmatrix} F_1 \\ F_2 \\ \boxed{F_3} \\ F_4 \\ \boxed{F_5} \\ F_6 \end{bmatrix}$$

The corresponding global stiffness matrix has rows and columns associated with the same DOFs. Rows and columns for free DOFs (3 and 5) are highlighted.

$$\boldsymbol{K} = \begin{bmatrix} K_{11} & K_{12} & \boxed{K_{13}} & K_{14} & \boxed{K_{15}} & K_{16} \\ K_{21} & K_{22} & \boxed{K_{23}} & K_{24} & \boxed{K_{25}} & K_{26} \\ \boxed{K_{31}} & \boxed{K_{32}} & \boxed{K_{33}} & \boxed{K_{34}} & \boxed{K_{35}} & \boxed{K_{36}} \\ K_{41} & K_{42} & \boxed{K_{43}} & K_{44} & \boxed{K_{45}} & K_{46} \\ \boxed{K_{51}} & \boxed{K_{52}} & \boxed{K_{53}} & \boxed{K_{54}} & \boxed{K_{55}} & \boxed{K_{56}} \\ K_{61} & K_{62} & \boxed{K_{63}} & K_{64} & \boxed{K_{65}} & K_{66} \end{bmatrix}$$



# Step 1 — Bring DOF 3 to the top (swap DOFs 1 and 3)

## Displacement and force vectors

$$\mathbf{u} = \begin{bmatrix} u_1 \\ u_2 \\ \boxed{u_3} \\ u_4 \\ \boxed{u_5} \\ u_6 \end{bmatrix} \longrightarrow \begin{bmatrix} \boxed{u_3} \\ u_2 \\ u_1 \\ u_4 \\ \boxed{u_5} \\ u_6 \end{bmatrix}, \quad \mathbf{F} = \begin{bmatrix} F_1 \\ F_2 \\ \boxed{F_3} \\ F_4 \\ \boxed{F_5} \\ F_6 \end{bmatrix} \longrightarrow \begin{bmatrix} \boxed{F_3} \\ F_2 \\ F_1 \\ F_4 \\ \boxed{F_5} \\ F_6 \end{bmatrix}$$

## Stiffness matrix (rows 1 $\leftrightarrow$ 3, columns 1 $\leftrightarrow$ 3)

$$\mathbf{K}^{(1)} = \begin{bmatrix} K_{33} & K_{32} & K_{31} & K_{34} & K_{35} & K_{36} \\ \boxed{K_{23}} & K_{22} & K_{21} & K_{24} & \boxed{K_{25}} & K_{26} \\ \boxed{K_{13}} & K_{12} & K_{11} & K_{14} & \boxed{K_{15}} & K_{16} \\ \boxed{K_{43}} & K_{42} & K_{41} & K_{44} & \boxed{K_{45}} & K_{46} \\ \boxed{K_{53}} & \boxed{K_{52}} & \boxed{K_{51}} & \boxed{K_{54}} & \boxed{K_{55}} & \boxed{K_{56}} \\ \boxed{K_{63}} & K_{62} & K_{61} & K_{64} & \boxed{K_{65}} & K_{66} \end{bmatrix}$$



# Step 2 — Bring DOF 5 to position 2 (swap DOFs 2 and 5)

## Displacement and force vectors

$$\mathbf{u} = \begin{bmatrix} u_3 \\ u_2 \\ u_1 \\ u_4 \\ \boxed{u_5} \\ u_6 \end{bmatrix} \longrightarrow \begin{bmatrix} u_3 \\ \boxed{u_5} \\ u_1 \\ u_4 \\ u_2 \\ u_6 \end{bmatrix}, \quad \mathbf{F} = \begin{bmatrix} F_3 \\ F_2 \\ F_1 \\ F_4 \\ \boxed{F_5} \\ F_6 \end{bmatrix} \longrightarrow \begin{bmatrix} F_3 \\ \boxed{F_5} \\ F_1 \\ F_4 \\ F_2 \\ F_6 \end{bmatrix}$$

## Stiffness matrix (rows 2 $\leftrightarrow$ 5, columns 2 $\leftrightarrow$ 5)

$$\mathbf{K}^{(2)} = \begin{bmatrix} K_{33} & K_{35} & K_{31} & K_{34} & K_{32} & K_{36} \\ K_{53} & K_{55} & K_{51} & K_{54} & K_{52} & K_{56} \\ K_{13} & K_{15} & K_{11} & K_{14} & K_{12} & K_{16} \\ K_{43} & K_{45} & K_{41} & K_{44} & K_{42} & K_{46} \\ K_{23} & K_{25} & K_{21} & K_{24} & K_{22} & K_{26} \\ K_{63} & K_{65} & K_{61} & K_{64} & K_{62} & K_{66} \end{bmatrix}$$



## Partitioned form (read directly)

After reordering, the free DOFs come first. For our example, the free set is  $f = \{3, 5\}$  and the restrained set is  $r = \{1, 4, 2, 6\}$ , so

$$\mathbf{u} = \begin{bmatrix} \mathbf{u}_f \\ \mathbf{u}_r \end{bmatrix} = \begin{bmatrix} u_3 \\ u_5 \\ \hline u_1 \\ u_4 \\ u_2 \\ u_6 \end{bmatrix}, \quad \mathbf{F} = \begin{bmatrix} \mathbf{F}_f \\ \mathbf{F}_r \end{bmatrix} = \begin{bmatrix} F_3 \\ F_5 \\ \hline F_1 \\ F_4 \\ F_2 \\ F_6 \end{bmatrix}$$

$$\mathbf{K} = \left[ \begin{array}{c|c} \mathbf{K}_{ff} & \mathbf{K}_{fr} \\ \hline \mathbf{K}_{rf} & \mathbf{K}_{rr} \end{array} \right] = \left[ \begin{array}{cc|ccccc} K_{33} & K_{35} & K_{31} & K_{34} & K_{32} & K_{36} \\ K_{53} & K_{55} & K_{51} & K_{54} & K_{52} & K_{56} \\ \hline K_{13} & K_{15} & K_{11} & K_{14} & K_{12} & K_{16} \\ K_{43} & K_{45} & K_{41} & K_{44} & K_{42} & K_{46} \\ K_{23} & K_{25} & K_{21} & K_{24} & K_{22} & K_{26} \\ K_{63} & K_{65} & K_{61} & K_{64} & K_{62} & K_{66} \end{array} \right]$$

Here the dashed line shows the split between free and restrained DOFs:

- top-left block:  $\mathbf{K}_{ff}$  (free–free)
- top-right block:  $\mathbf{K}_{fr}$  (free–restrained)
- bottom-left block:  $\mathbf{K}_{rf}$  (restrained–free)
- bottom-right block:  $\mathbf{K}_{rr}$  (restrained–restrained)



# Part 7 — Solving Global Displacements and Forces

*free node displacements and restraint forces*

# Expand the Partitioned System

Starting from the block system

$$\begin{bmatrix} \mathbf{K}_{ff} & \mathbf{K}_{fr} \\ \mathbf{K}_{rf} & \mathbf{K}_{rr} \end{bmatrix} \begin{bmatrix} \mathbf{u}_f \\ \mathbf{u}_r \end{bmatrix} = \begin{bmatrix} \mathbf{F}_f \\ \mathbf{F}_r \end{bmatrix}$$

expanding the matrix–vector multiplication gives two coupled equations:

$$\mathbf{K}_{ff}\mathbf{u}_f + \mathbf{K}_{fr}\mathbf{u}_r = \mathbf{F}_f \tag{1}$$

$$\mathbf{K}_{rf}\mathbf{u}_f + \mathbf{K}_{rr}\mathbf{u}_r = \mathbf{F}_r \tag{2}$$

The first equation governs equilibrium at the **free DOFs**,  
and the second governs equilibrium at the **restrained DOFs**.

# Solving Equation (1) for the Free DOFs

We are interested in solving for the **unknown displacements** at the free DOFs,  $\mathbf{u}_f$ .

Starting from Equation (1):  $\mathbf{K}_{ff}\mathbf{u}_f + \mathbf{K}_{fr}\mathbf{u}_r = \mathbf{F}_f$

Rearrange to isolate the unknowns:

$$\mathbf{K}_{ff}\mathbf{u}_f = \mathbf{F}_f - \mathbf{K}_{fr}\mathbf{u}_r$$

Provided that  $\mathbf{K}_{ff}$  is invertible, the solution is:

$$\boxed{\mathbf{u}_f = \mathbf{K}_{ff}^{-1}(\mathbf{F}_f - \mathbf{K}_{fr}\mathbf{u}_r)}$$

Typically, the restrained displacements  $\mathbf{u}_r$  are **known** from boundary conditions (often  $\mathbf{u}_r = \mathbf{0}$ ). In that common case, the expression simplifies to:

$$\boxed{\mathbf{u}_f = \mathbf{K}_{ff}^{-1}\mathbf{F}_f}$$

Solve in practice to obtain the global displacements at the free DOFs.

## Solving Equation (2) for the Restrained Forces

Once the free displacements  $\mathbf{u}_f$  have been computed, we can determine the **forces at the restrained DOFs** (support reactions).

Starting from Equation (2):  $\mathbf{K}_{rf}\mathbf{u}_f + \mathbf{K}_{rr}\mathbf{u}_r = \mathbf{F}_r$

At the restrained DOFs, the displacements  $\mathbf{u}_r$  are **known** from the boundary conditions (often  $\mathbf{u}_r = \mathbf{0}$ ). Substituting these known values gives a direct expression for the reaction forces:

$$\boxed{\mathbf{F}_r = \mathbf{K}_{rf}\mathbf{u}_f + \mathbf{K}_{rr}\mathbf{u}_r}$$

In the common case where the supports are fixed and  $\mathbf{u}_r = \mathbf{0}$ , this simplifies to:

$$\boxed{\mathbf{F}_r = \mathbf{K}_{rf}\mathbf{u}_f}$$

This step **does not require solving a linear system** (i.e., no inverse needed).

The reaction forces are obtained directly by evaluating this expression after  $\mathbf{u}_f$  is known.

## Summary — What We Have Solved

At this point, the global system has been fully resolved.

We have determined the two previously unknown quantities:

- **Free nodal displacements,  $\mathbf{u}_f$**   
(obtained by solving the reduced system)
- **Support reaction forces,  $\mathbf{F}_r$**   
(computed directly once  $\mathbf{u}_f$  is known)

Together, these results provide the complete displacement and force state of the structure, satisfying equilibrium, compatibility, and the imposed boundary conditions.

# Sanity checks after solving

## 1) Stability

- If the structure is properly constrained,  $\mathbf{K}_{ff}$  should be non-singular
- If it is still singular, you likely have:
  - insufficient supports (rigid-body mode remains), or
  - a disconnected node / element connectivity error

## 2) Equilibrium

- Reconstruct the full displacement vector  $\mathbf{u}$  by inserting  $\mathbf{u}_f$  (just calculated) and  $\mathbf{u}_r$
- Compute  $\mathbf{K}\mathbf{u}$  and compare to  $\mathbf{F}$
- Differences should be near numerical tolerance (floating point error)

## 3) Reactions

- Check reaction directions and magnitudes against intuition
- Sum of reactions should balance applied loads (global equilibrium)

# Part 8 — Recover Element Forces

*Back to Local*

Up to this point, we have solved for the **global nodal displacements** of the structure. While this information is essential for equilibrium, it is **not** what we design members for.

Structural design is performed at the **element level**, using:

- axial forces
- stresses
- internal force resultants

expressed in each element's **local coordinate system**.

This step bridges analysis and design.

## Why Go Back to the Element Level?

- Global displacements describe how the **structure moves**
- Local element forces describe how **members carry load**
- Design checks (strength, buckling, serviceability) are based on **element forces**, not global DOFs

Therefore, we must recover **element-level displacements and forces** from the global solution.

# Element Force Recovery — Overview

For each element, we perform the following steps:

- 1. Extract element global displacements**

Assemble the element displacement vector from the global solution.

- 2. Transform to local coordinates**

Convert global displacements to the element's local axis.

- 3. Compute local end forces**

Use the element stiffness matrix to compute internal forces.

- 4. Evaluate axial force**

Extract the axial force used directly in design.

## Step 1 — Extract Element Global Displacements

From the global displacement vector  $\mathbf{u}$ , collect the DOFs associated with element  $e$ :

$$\mathbf{u}_e = \begin{bmatrix} u_{i,x} \\ u_{i,y} \\ u_{j,x} \\ u_{j,y} \end{bmatrix}$$

This vector contains the **global displacements** of the element's end nodes.

## Step 2 — Transform to Local Coordinates

Transform the element displacement vector to the local coordinate system:

$$\mathbf{u}'_e = \mathbf{T}_e \mathbf{u}_e$$

where  $\mathbf{T}_e$  is the **element transformation matrix**, for that specific element defined by the element orientation.

This step aligns the displacements with the element's axial direction.

## Step 3 — Compute Local End Forces

Use the local stiffness matrix to compute the internal end forces:

$$\mathbf{f}'_e = \mathbf{k}'_e \mathbf{u}'_e$$

These are the **local element forces** (axial forces at each end). For a truss element you should get the axial force in the element

## Step 4 — Compute Axial Stress

Once the axial force is known, the **axial stress** follows as:

$$\sigma = \frac{N}{A}$$

**This is what you design for:**

- axial stress limits
- tension vs. compression
- buckling checks for compression members

## Step 5 (Optional) — Member End Forces in Global Coordinates

This step is **not required for design**, but it can be useful for interpretation and post-processing.

So far, member forces have been computed in the **local coordinate system**, which is where axial behavior is defined.

In some cases, however, it is helpful to express the **member end forces in the global ( $X, Y$ ) frame**, for example:

- to visualize force flow in the structure
- to check nodal equilibrium in global directions
- to compare with externally applied loads

The local member force vector can be rotated back to global coordinates as:

$$\mathbf{f}_e = \mathbf{T}_e^T \mathbf{f}'_e$$

For a **perfectly horizontal member**, the local and global axes coincide, and this transformation leaves the forces unchanged.

# Part 9 — DSM Summary

# Forward Pass — Structural Analysis

## 1. Define geometry

- Nodes, members, connectivity

## 2. Number global DOFs

- Assign consistent global displacement indices

## 3. For each member

- Compute geometry:  $L, \theta, c, s$
- Build transformation matrix  $\mathbf{T}$
- Compute global element stiffness:

$$\mathbf{k} = \mathbf{T}^\top \mathbf{k}' \mathbf{T}$$

## 4. Assemble global stiffness matrix

- Scatter-add element contributions into  $\mathbf{K}$

# Forward Pass — Structural Analysis, cont...

## 5. Apply boundary conditions

- Partition DOFs into free ( $f$ ) and restrained ( $r$ )

## 6. Solve for unknown displacements

$$\mathbf{u}_f = \mathbf{K}_{ff}^{-1} (\mathbf{F}_f - \mathbf{K}_{fr} \mathbf{u}_r)$$

## 7. Recover support reactions

$$\mathbf{F}_r = \mathbf{K}_{rf} \mathbf{u}_f + \mathbf{K}_{rr} \mathbf{u}_r$$

# Backward Pass — Element Recovery and Design

## 8. Extract element global displacement vectors

- For each member, collect the relevant entries from  $\mathbf{u}$  to form  $\mathbf{u}_e$

## 9. Transform displacements to local coordinates

$$\mathbf{u}' = \mathbf{T} \mathbf{u}_e$$

## 10. Compute local element end forces

$$\mathbf{f}' = \mathbf{k}' \mathbf{u}'$$

## 11. Compute axial force and stress (design quantities)

$$\sigma = \frac{N}{A}$$

## 12. (Optional) Express element forces in global coordinates

$$\mathbf{f} = \mathbf{T}^T \mathbf{f}'$$

# Why DSM Is Powerful

- **Static indeterminacy is not an issue**
  - Any number of members can be handled
- **Equilibrium is enforced automatically**
  - Through the global stiffness equations
- **Compatibility is built in**
  - Via shared nodal displacements
- The method scales cleanly to:
  - large structures
  - complex geometries
  - numerical implementation

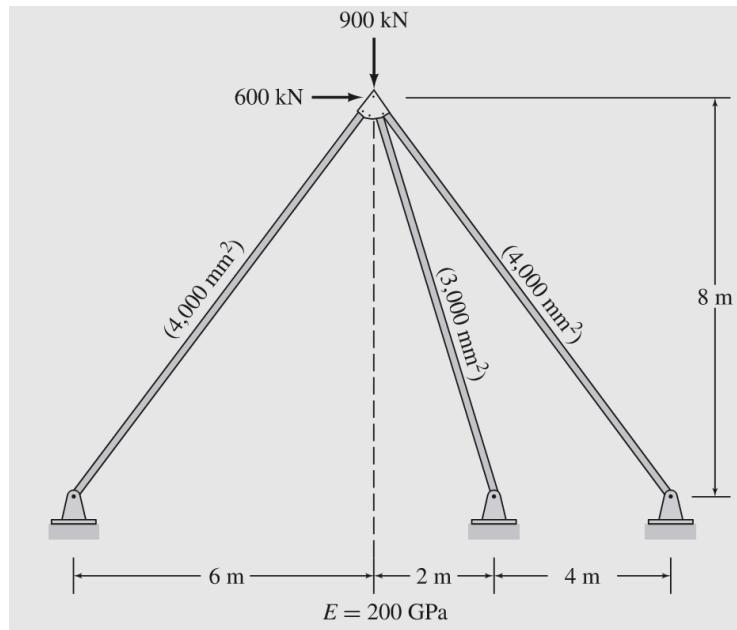
*DSM replaces manual equilibrium and compatibility equations with a single, systematic matrix framework.*

# Class Problem (Time Permitting)

This problem will form the basis of **this week's homework**.

Begin by working through the setup **by hand**:

- define the geometry and connectivity,
- number the global degrees of freedom,
- setup the local element stiffness matrices (calculate transformation matrices).



## Node numbering:

- Node 1: bottom-left support
- Node 2: middle support
- Node 3: top node
- Node 4: right support

# Wrap-Up

Today you built the DSM pipeline for trusses:

- local bar stiffness → transformation → global element stiffness
- assembly → constraints → solve → member force recovery

Next Lecture: implement DSM in Python for a worked truss example and discuss efficiency (sparsity/bandedness).