Chapter 33

BUILDING STRUCTURES

33.1 Introduction

33.1.1 Beam Column Connections

1 The connection between the beam and the column can be, Fig. 33.1:

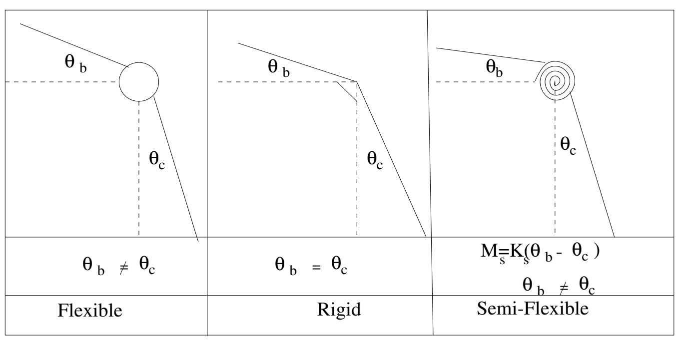

Figure 33.1: Flexible, Rigid, and Semi-Flexible Joints

Flexible that is a hinge which can transfer forces only. In this case we really have cantiliver action only. In a flexible connection the column and beam end moments are both equal to zero, $M_{\text{col}} = M_{\text{beam}} = 0$. The end rotation are not equal, $\theta_{\text{col}} \neq \theta_{\text{beam}}$.

Rigid: The connection is such that $\theta_{\text{beam}} = \theta_{\text{col}}$ and moment can be transmitted through the connection. In a rigid connection, the end moments and rotations are equal (unless there is an externally applied moment at the node), $M_{\text{col}} = M_{\text{beam}} \neq 0$, $\theta_{\text{col}} = \theta_{\text{beam}}$.

Semi-Rigid: The end moments are equal and not equal to zero, but the rotation are different. $\theta_{\text{beam}} \neq \theta_{\text{col}}$, $M_{\text{col}} = M_{\text{beam}} \neq 0$. Furthermore, the difference in rotation is resisted by the spring $M_{\text{spring}} = K_{\text{spring}}(\theta_{\text{col}} - \theta_{\text{beam}})$.

33.1.2 Behavior of Simple Frames

2 For vertical load across the beam rigid connection will reduce the maximum moment in the beam (at the expense of a negative moment at the ends which will in turn be transferred to

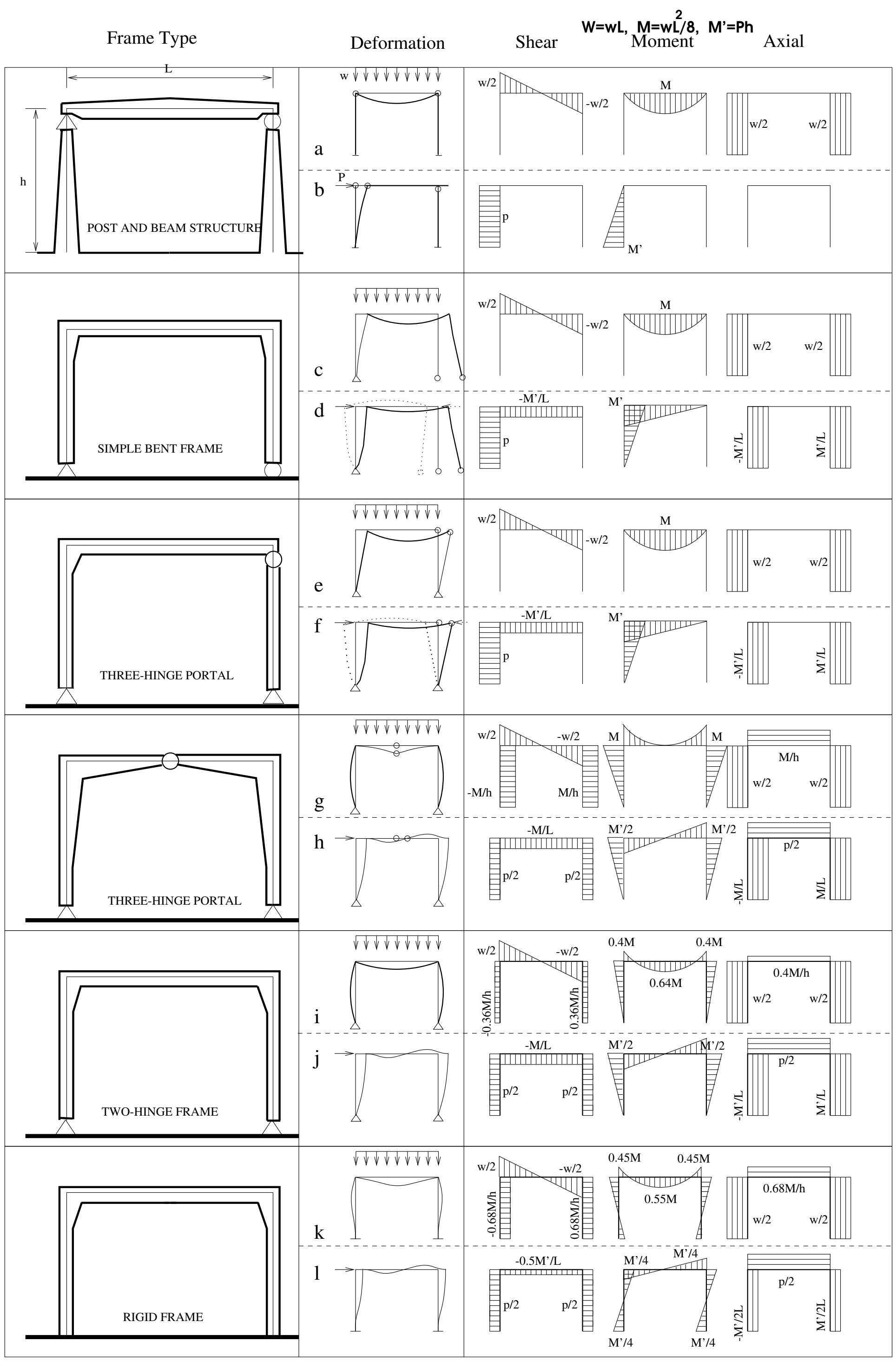

Figure 33.3: Deformation, Shear, Moment, and Axial Diagrams for Various Types of Portal Frames Subjected to Vertical and Horizontal Loads

33.2 Buildings Structures

11 There are three primary types of building systems:

Wall Subsytem: in which very rigid walls made up of solid masonry, paneled or braced timber, or steel trusses constitute a rigid subsystem. This is only adequate for small rise buildings.

Vertical Shafts: made up of four solid or trussed walls forming a tubular space structure. The tubular structure may be interior (housing elevators, staircases) and/or exterior. Most efficient for very high rise buildings.

Rigid Frame: which consists of linear vertical components (columns) rigidly connected to stiff horizontal ones (beams and girders). This is not a very efficient structural form to resist lateral (wind/earthquake) loads.

33.2.1 Wall Subsystems

12 Whereas exterior wall provide enclosure and interior ones separation, both of them can also have a structural role in trnsfering vertical and horizontal loads.

13 Walls are constructed out of masonry, timber concrete or steel.

14 If the wall is braced by floors, then it can provide an excellent resitance to horizontal load in the plane of the wall (but not orthogonal to it).

15 When shear-walls subsytems are used, it is best if the center of orthogonal shear resistance is close to the centroid of lateral loads as applied. If this is not the case, then there will be torsional design problems.

33.2.1.1 Example: Concrete Shear Wall

`From (Lin and Stotesbury 1981)`

16 We consider a reinforced concrete wall 20 ft wide, 1 ft thick, and 120 ft high with a vertical load of 400 k acting on it at the base. As a result of wind, we assume a uniform horizontal force of 0.8 kip/ft of vertical height acting on the wall. It is required to compute the flexural stresses and the shearing stresses in the wall to resist the wind load, Fig. 33.5.

1. Maximum shear force and bending moment at the base

$$V_{max} = wL = (0.8)\text{ k.ft}(120)\text{ ft} = 96\text{ k} \tag{33.8-a}$$

$$M_{max} = \frac{wL^2}{2} = \frac{(0.8)\text{ k.ft}(120)^2\text{ ft}^2}{2} = 5,760\text{ k.ft} \tag{33.8-b}$$

2. The resulting eccentricity is

$$e_{\text{Actual}} = \frac{M}{P} = \frac{(5,760)\text{ k.ft}}{(400)\text{ k}} = 14.4\text{ ft} \tag{33.9}$$

3. The critical eccentricity is

$$e_{cr} = \frac{L}{6} = \frac{(20)\text{ ft}}{6} = 3.3\text{ ft} < e_{\text{Actual}} N.G. \tag{33.10}$$

   thus there will be tension at the base.

9. The compressive stress of 740 psi can easily be sustained by concrete, as to the tensile stress of 460 psi, it would have to be resisted by some steel reinforcement.

10. Given that those stresses are *service* stresses and not factored ones, we adopt the WSD approach, and use an allowable stress of 20 ksi, which in turn will be increased by 4/3 for seismic and wind load,
$$\sigma_{all} = \frac{4}{3}(20) = 26.7 \text{ ksi} \tag{33.16}$$

11. The stress distribution is linear, compression at one end, and tension at the other. The length of the tension area is given by (similar triangles)
$$\frac{x}{460} = \frac{20}{460+740} \Rightarrow x = \frac{460}{460+740}(20) = 7.7 \text{ ft} \tag{33.17}$$

12. The total tensile force inside this triangular stress block is
$$T = \frac{1}{2}(460) \text{ ksi}(7.7 \times 12) \text{ in} \underbrace{(12) \text{ in}}_{\text{width}} = 250 \text{ k} \tag{33.18}$$

13. The total amount of steel reinforcement needed is
$$A_s = \frac{(250) \text{ k}}{(26.7) \text{ ksi}} = \boxed{9.4 \text{ in}^2} \tag{33.19}$$

This amount of reinforcement should be provided at both ends of the wall since the wind or eartquake can act in any direction. In addition, the foundations should be designed to resist tensile uplift forces (possibly using piles).

33.2.1.2 Example: Trussed Shear Wall

`From (Lin and Stotesbury 1981)`

17 We consider the same problem previously analysed, but use a trussed shear wall instead of a concrete one, Fig. 33.6.

1. Using the maximum moment of 5,760 kip-ft (Eq. 33.8-b), we can compute the compression and tension in the columns for a lever arm of 20 ft.
$$F = \pm\frac{(5,760) \text{ k.ft}}{(20) \text{ ft}} = \pm 288 \text{ k} \tag{33.20}$$

2. If we now add the effect of the 400 kip vertical load, the forces would be
$$C = -\frac{(400) \text{ k}}{2} - 288 = \boxed{-488 \text{ k}} \tag{33.21-a}$$
$$T = -\frac{(400) \text{ k}}{2} + 288 = \boxed{88 \text{ k}} \tag{33.21-b}$$

3. The force in the diagonal which must resist a base shear of 96 kip is (similar triangles)
$$\frac{F}{96} = \frac{\sqrt{(20)^2 + (24)^2}}{20} \Rightarrow F = \frac{\sqrt{(20)^2 + (24)^2}}{20}(96) = \boxed{154 \text{ k}} \tag{33.22}$$

4. The design could be modified to have no tensile forces in the columns by increasing the width of the base (currently at 20 ft).