UNIVERSITY OF BAGHDAD
COLLEGE OF ENGINEERING
CIVIL ENGINEERING DEPARTMENT

DESIGN OF REINFORCED CONCRETE STRUCTURES

THIRD YEAR COURSE (JUNIOR COURSE)

PREPARED BY

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CIVIL ENGINEERING DEPARTMENT
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Academic Year 2018-2019

Syllabus

First Semester

Part I: Introduction to Reinforced Concrete Structures

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1.3 Loads.
1.4 Design Codes and Specifications.
1.5 Design Criteria.
1.6 Design Philosophy.
1.7 Strength Versus Working-Stress Design Methods.
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1.9 Syllabus.
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4.9 Analysis of a Rectangular Beam with Tension and Compression Reinforcements (a Doubly Reinforced Beam).
4.10 Home Work of Article 4.9, Analysis of a Rectangular Section with Tension and Compression Reinforcements Only (a Doubly Reinforced Section).
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5. Shear and Diagonal Tension in Beams (1^{st}-31^{st} of January)

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5.7 Problems for Solution on Basic Shear Aspects.

[1] Asterisk, *, indicates more specialized articles that may be terminated without destroying the continuity of the course.

5.8* Shear Design Based on the More Detailed Relation for V_c.
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6.3 ACI Code Provisions for Development of Tension Reinforcement.
6.4 Anchorage of Tension Bars by Hooks.
6.5 Anchorage Requirements for Web Reinforcement.
6.6 Development of Bars in Compression.
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6.9 Development of Flexural Reinforcement.
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7. Serviceability (8th-14th of March)

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8. Analysis and Design for Torsion (15th – 31st of March)

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Part III: Design of Reinforced Columns

9. Design of Reinforced Concrete Columns (1st - 14th of April)

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9.2 ACI Analysis Procedure for a Short Column under an Axial Load (Small Eccentricity).
9.3 ACI Design Procedure for a Short Column under an Axial Load (Small Eccentricity).
9.4 Home Work: Analysis and Design of Axially Loaded Columns.
9.5 Analysis of a Column with Compression Load Plus Uniaxial Moment.
9.6 Design of A Column with Compression Load Plus Uniaxial Moment.
9.7 Home Work: Analysis and Design of a Column under Axial Load and Uniaxial Moment.
9.8 Column Analysis under a Compression Force and Biaxial Moments.
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10. Slender Concrete Columns (15th -31st of April)

10.1 Introduction and Basic Concepts.
10.2 ACI Strategies for Slender Columns.
10.3 ACI Criteria for Neglecting of Slenderness Effects.
10.4 ACI Criteria for Non-sway versus Sway Frames.
10.5 Summary of ACI Moment Magnifier Method for Non-sway Frames.
10.6 Summary of ACI Moment Magnifier Method for Sway Frames.
10.7* Computer Applications.

Part IV: Analysis of Indeterminate Beams and Frames

11. Analysis of Indeterminate Beams and Frames (1st -7th of May)

11.8 ACI Moment Coefficients.
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Part V: Design of Reinforced Concrete Slabs

12. Design of One-Way Slabs (8th -14th of May)

12.1 Basic Concepts of One-Way System
12.2 Analysis of One-Way Slab System
12.3 Design Examples of One-Way Slab Systems Including Analysis and Design of Continuous Supporting Beams.
12.4* Computer Applications.

13. <u>Design of Edge Supported Two-Way Slabs (15th to 31st of May)</u>
- 13.1 Basic Concepts
- 13.2 Design Example of an Edge Supported Two-way Solid Slab Including Analysis and Design of Supporting Continuous Beams.
- *13.3 Computer Applications.

Part VI: Design of Concrete Structural Systems (1st to 7th of July)
Project Oriented Design Examples.

Text Books

1. A. H. Nilson, D. Darwin, and C. W. Dolan, Design of Concrete Structures, 13th Edition, McGraw Hill, 2004.
2. D. Darwin, C. W. Dolan, and A. H. Nilson, Design of Concrete Structures, 15th Edition, McGraw Hill, 2015 (Metric Edition).
3. Building Code Requirements for Structural Concrete (ACI318M-14).

References

1. J. K. Wight and J. G. MacGregor, Reinforced Concrete: Mechanics and Design, 7th Edition, Person/Prentice Hall, 2016.
2. E. G. Nawy, Reinforced Concrete: A Fundamental Approach, 6th Edition, Prentice Hall, 2009.
3. C.K. Wang, C.G. Salmon and J. A. Pincheira, Reinforced Concrete Design, 7th Edition, John Wiley & Sons, 2007.
4. J.C. McCormac and R. H. Brown, Design of Reinforced Concrete, 9th Edition, John Wiley & Sons, 2014.
5. M. N. Hassoun, A. Al-Manaseer, Structural Concrete: Theory and Design, 6th Edition, Wiley, 2015.
6. G.F. Limbrunner and A.O. Aghayere, Reinforced Concrete Design, 7th Edition, Prentice Hall, 2010.
7. M. Setareh, and R. Darvas, Concrete Structure, Prentice Hall, 2007.
8. M. E. Kamara, B. G. Rabbat, Notes on ACI 318-05, 9th Edition, 2005.

CHAPTER 1
INTRODUCTION

1.1 STRUCTURAL DESIGN, STRUCTURAL ELEMENTS, AND STRUCTURAL FORMS

1.1.1 Structural Design

- The main objectives of the structural design are to prepare a structural system that transfers the applied loads from the points of application to the supporting soil safely and with an acceptable cost.
- The first step in the structural design is to select a structural system to be used in transferring the applied loads from the points of application to the supporting soil.

1.1.2 Structural Elements

To deal with an uncountable variety of the structures, they are usually broken into the following structural elements in their analysis and design.

1.1.2.1 Bar Element

- As indicated in ***Figure 1.1-1*** below, the bar element is the structural element that has two dimensions small when compared with the third one.

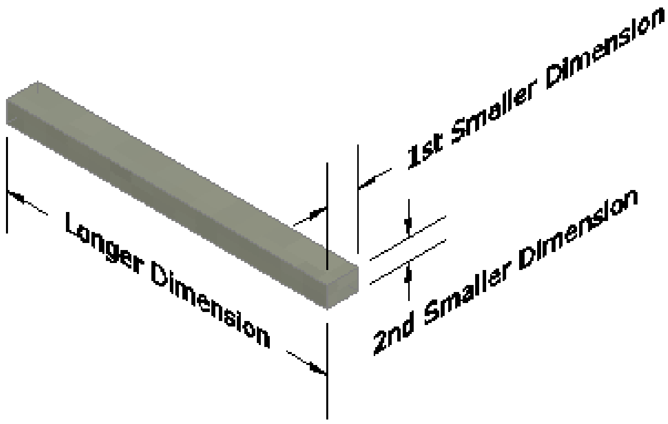


Figure 1.1-1: Bar element.

- A Bar element can be defined as a *beam* when the load is applied transversely to the element axis.

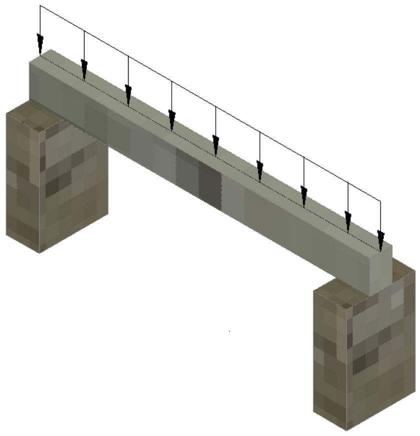


Figure 1.1-2: Beam element.

- A Column can be defined as the bar element that subjected to an axial load with or without bending moment.

Figure 1.1-3: Column element.

1.1.2.2 Plate Element

- The plate element is the structural element that has a one small dimension comparing with other its dimensions.

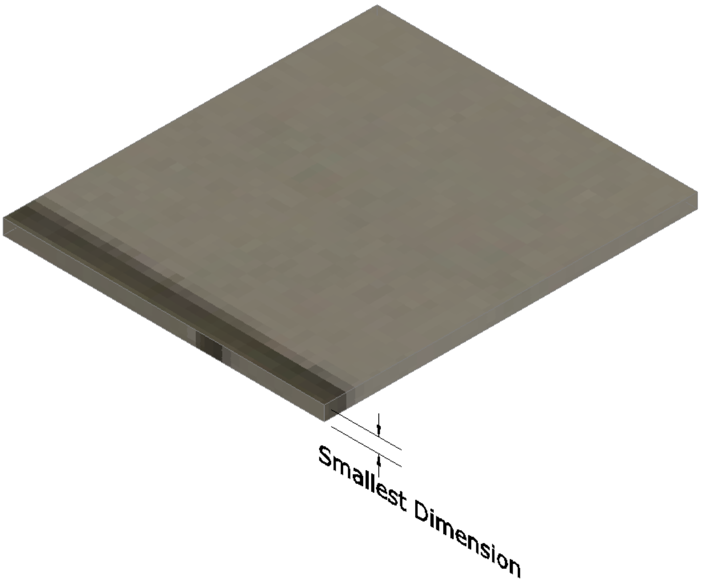


Figure 1.1-4: Plate element.

- The bearing wall is a plate element that subjected to an axial load.

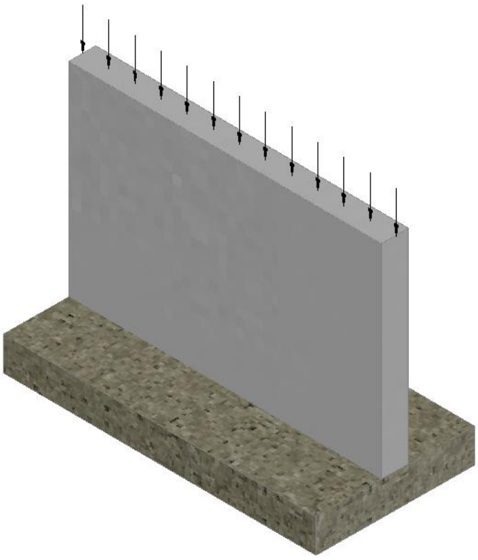


Figure 1.1-5: Bearing wall.

- The slab is a plate element that subjected to transverse loads.

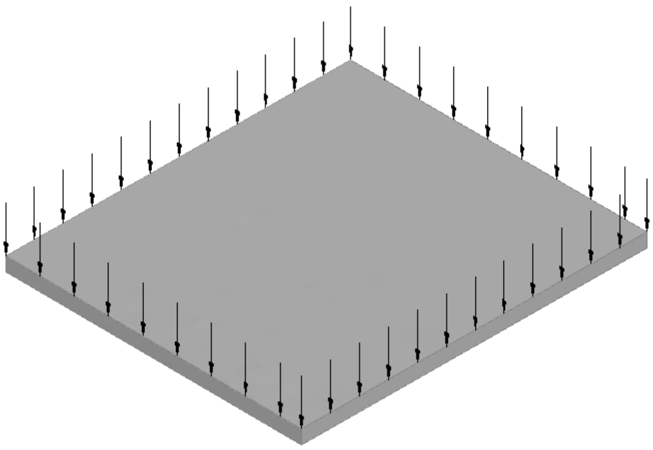


Figure 1.1-6: Structural slab.

1.1.2.3 Shell Element

- The shell element is a curved structural element; one of its dimensions is small when compared with the other two dimensions.
- It may take a form of the dome, or a form cylindrical shell.
- Shell element is out the scope of our course.

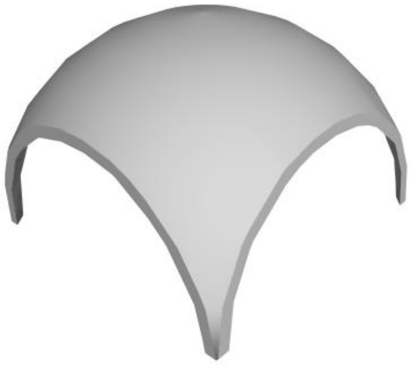


Figure 1.1-7: Dome Shell.

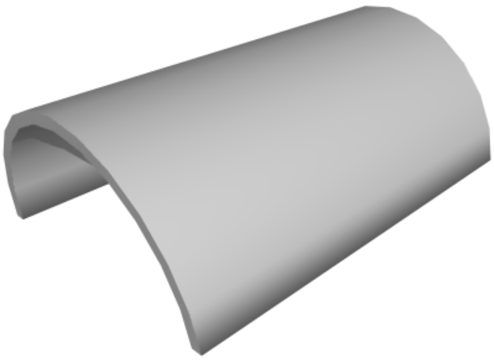


Figure 1.1-8: Cylindrical Shell.

1.2 FLOORING AND ROOFING SYSTEM

Reinforced–concrete floors can be classified into the following systems.

1.2.1 One-way Floor System

- In this system, the applied load acting on the slab is transferred in one direction to the supporting beams, then to the supporting columns.

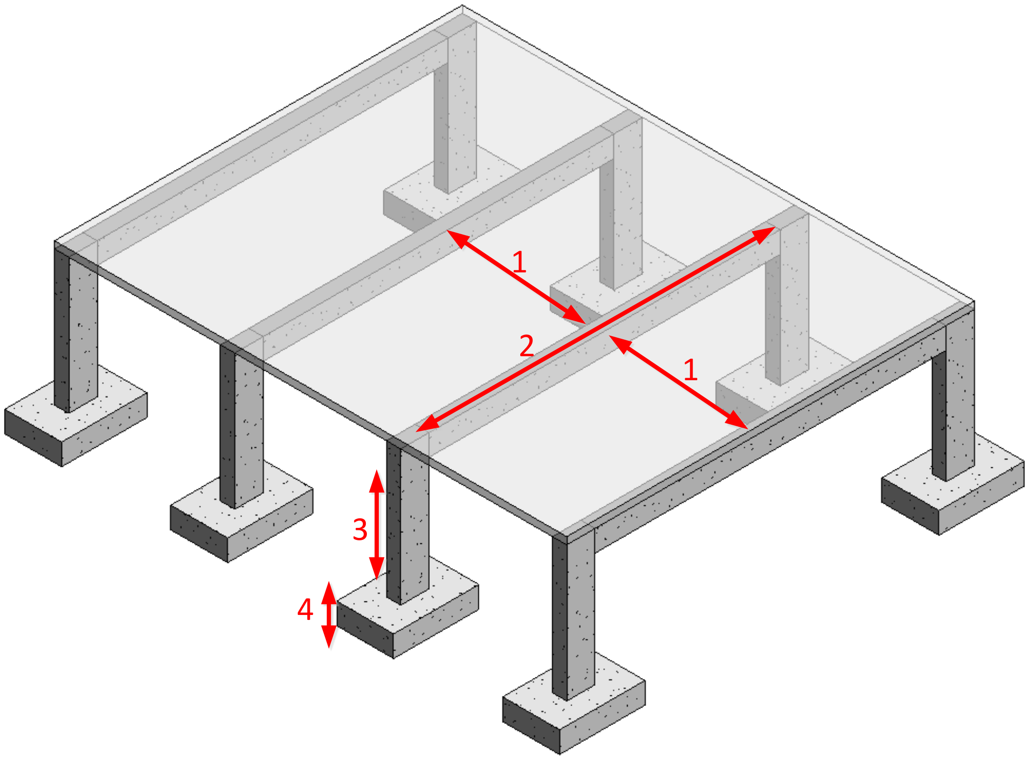


Figure 1.2-1: One-way floor system.

- For a large column spacing, the load may be transferred from the slab to the floor beams, then to larger beams (usually called the girders), and in turn to the supporting columns.

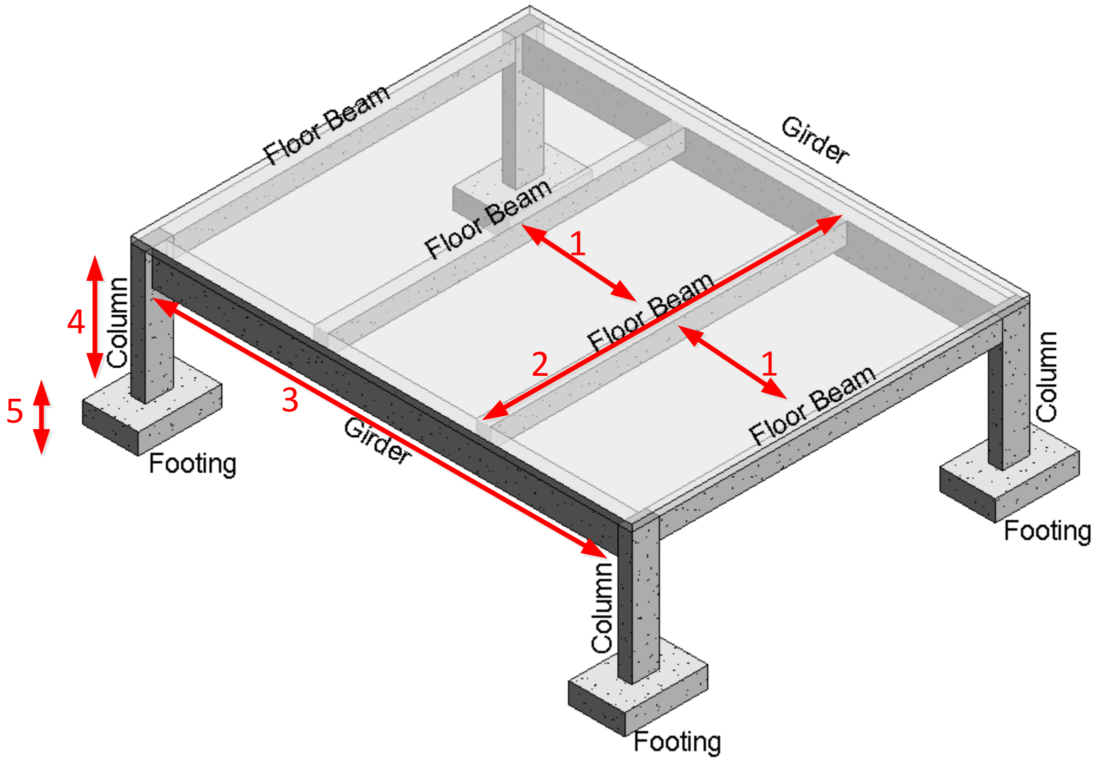


Figure 1.2-2: Slab-beam-girder one-way system.

1.2.2 Two-way Floor System with Beams

In this system, the applied load acting on the slab is transferred in two directions to supporting beams on the slab periphery, and in turn to the supporting columns.

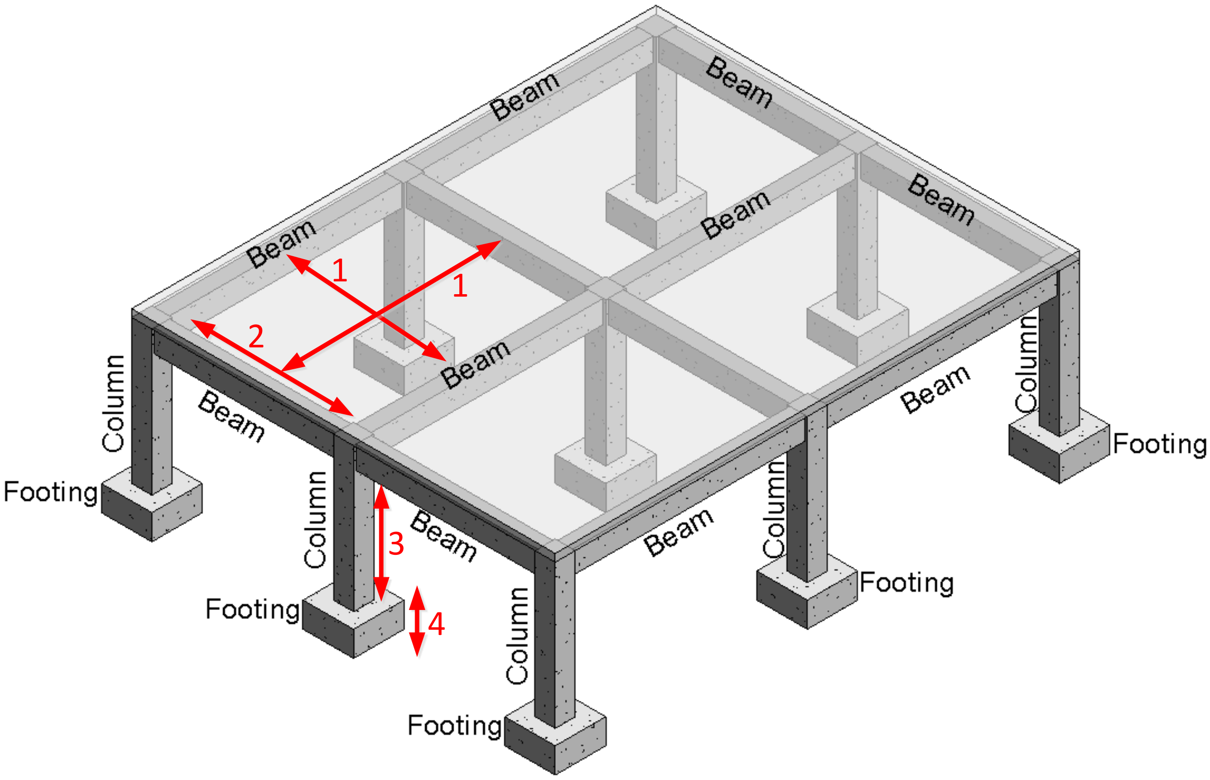


Figure 1.2-3: Two-way floor system with beams.

1.2.3 Two-way Floor System without Beams

- In this system, usually called ***flat plate system***, the slab is supported directly on the columns. Load transferred directly from the slab to the supporting columns.

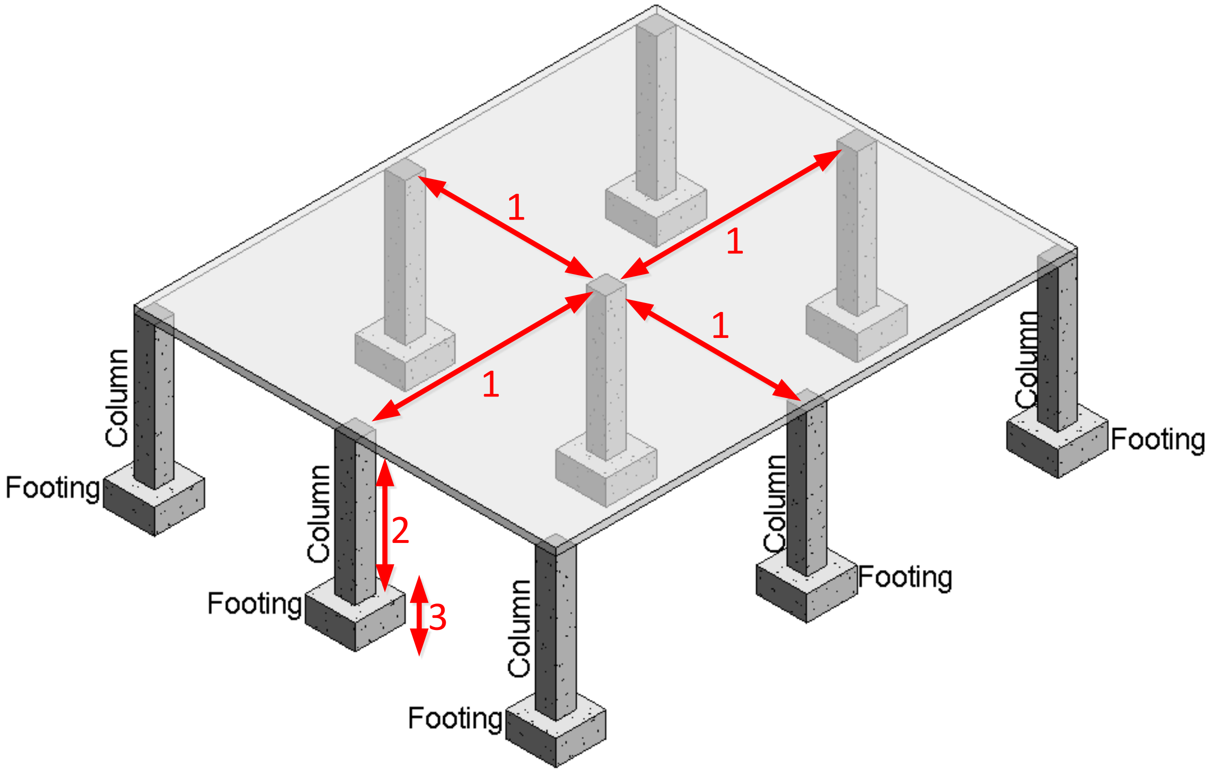


Figure 1.2-4: Flat plate floor system.

- To avoid slab punching due to column concentered forces, aforementioned system may be strengthened with drop panels and/or column capital.
- The resulting system is ***flat slab system***.
- Flat plate and flat slab systems are out the scope of our junior course.

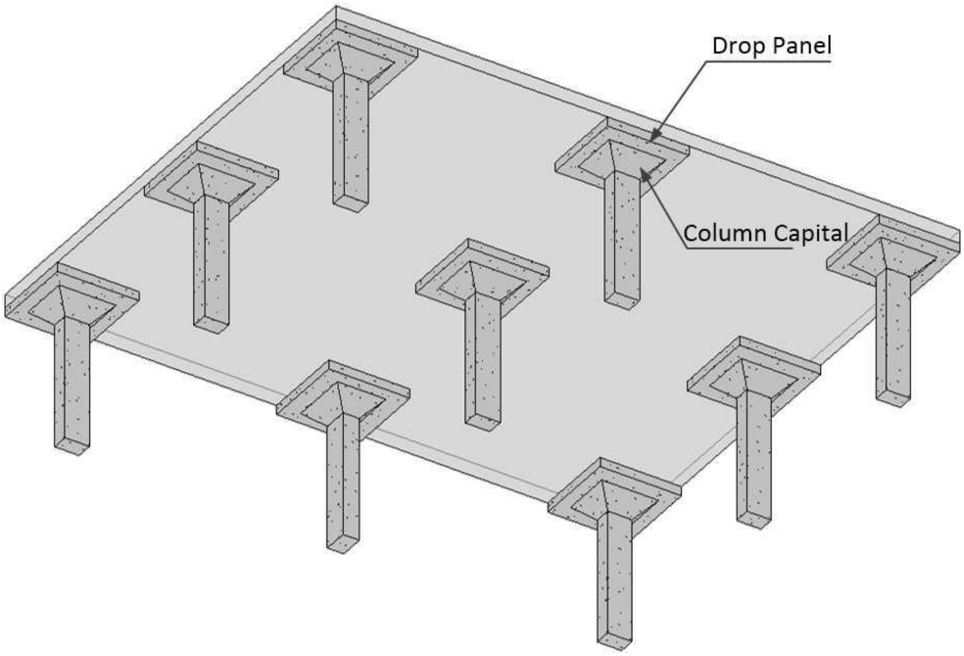


Figure 1.2-5: Flat slab system.

1.3 LOADS

Loads that act on the structures can be classified into three categories: dead loads, live loads, and environmental loads.

1.3.1 Dead Load

1. The major part of it is the weight of the structure itself.
2. It is constant in magnitude and fixed in location throughout the life of the structure.
3. It can be calculated with good accuracy from the dimensions of the structures and density of the materials.
4. Dead loads may be further classified into:
    - Selfweight, which represents own weight of the structural system.
    - Superimposed loads, which represents own weight of surfacing, mechanical, plumbing, and electrical fixtures.

1.3.2 Live Load

1.3.2.1 Floor Live Loads

- It consists of occupancy loads in buildings. According to ***section 5.3.4*** of the code, the live load, L, shall include, see Figure 1.3-1 through Figure 1.3-5.
    - Concentrated live loads,
    - Vehicular loads
    - Crane loads,
    - Loads on hand rails, guardrails, and vehicular barrier systems,
    - Impact effects,
    - Vibration effects.

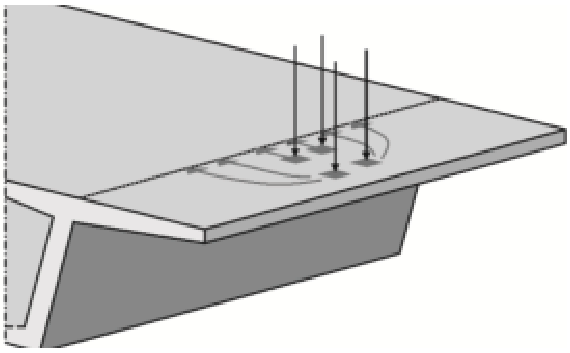


Figure 1.3-1: Concentrated live loads.

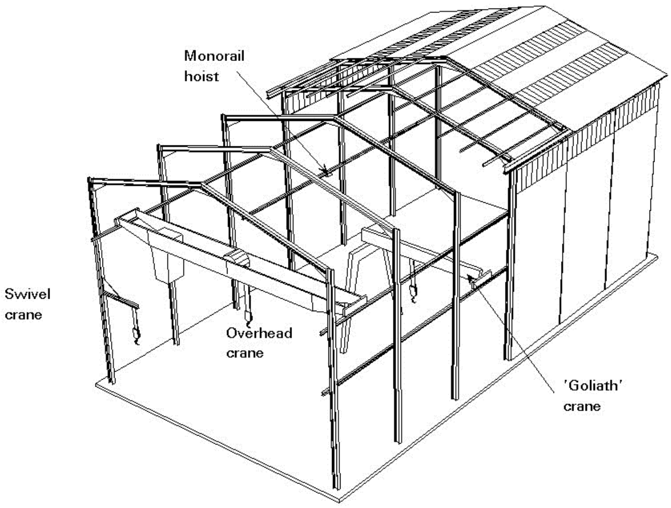


Figure 1.3-2: Crane loads on a building frame.

Figure 1.3-3: Handrail and vehicular guardrail or barrier.

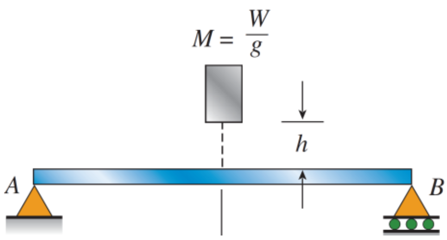


Figure 1.3-4: Impact effects.

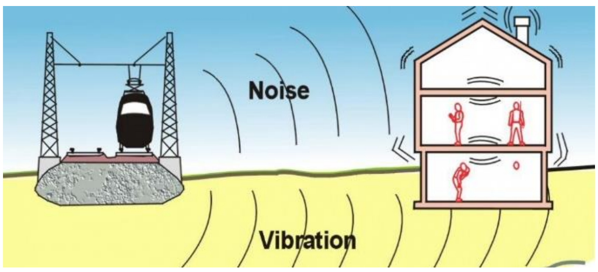


Figure 1.3-5: Vibration effects.

- It may be either fully or partially in place or may not percent at all.
- It may be changed in location.
- Its magnitude and distributions at any given time are uncertain and even their maximum intensities throughout the lifetime of the structures are not known with precision.
- The minimum live loads for which the floors and roof of a building to be designed are usually specified by the building code that governs at the site of constructions.
- Representative values of minimum live loads to be used in many locations including Iraq are presented in Table 1.3-1 below. These values are adopted from (ASCE/SEI 7–10)***, Minimum Design Loads for Buildings and Other Structures.***
- As can be seen from the table, in addition to the uniformly distributed loads, it is recommended that, as an alternative to the uniform loads, floors be designed to support certain concentrated loads if these produce a greater stress.

1.3.2.2 Reduction in Floor Live Load

- As it is improbable that a large floor area be fully loaded with live load at a same time, most of building codes offer relations to relate the value of live load supported by a structural member to the area which supported by this member.
- According to article 4.7.2 of (ASCE/SEI 7–10), reduced live load can be estimated based on following relation:

$$L = L_o(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}) \quad \textbf{Eq. 1.3-1}$$

where
Lo is unreduced design live load per m^2 of area supported by the member (see Table 1.3-1 below),
L is reduced design live load per m^2 of area supported by the member,
K_{LL} is live load element factor (see Table 1.3-2 below).
A_T is tributary area in m^2.
To be a large area where reduction in live load is permitted, the influence area, $K_{LL}A_T$, should be:
$K_{LL}A_T \geq 37.16\, m^2$

- L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.4L_o$ for members supporting two or more floors.
- Live loads that exceed $4.79\ kN/m^2$ shall not be reduced.

Table 1.3-1: **Minimum Uniformly Distributed Live Loads, and Minimum Concentrated Live Loads.**

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m^2)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dining rooms and restaurants	100 (4.79)[a]	
Dwellings (see Residential)		
Elevator machine room grating (on area of 2 in. by 2 in. (50 mm by 50 mm))		300 (1.33)
Finish light floor plate construction (on area of 1 in. by 1 in. (25 mm by 25 mm))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.5	
Garages		
Passenger vehicles only	40 (1.92)[a,b,c]	
Trucks and buses	[c]	
Handrails, guardrails, and grab bars	See Section 4.5	
Helipads	60 (2.87)[d,e] Nonreducible	[e,f,g]
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see Residential)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[a,h]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)[a]	2,000 (8.90)
Heavy	250 (11.97)[a]	3,000 (13.40)

Table 1.3-1: Minimum Uniformly Distributed Live Loads, and Minimum Concentrated Live Loads, Continued.

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	100 (4.79)	2,000 (8.90)
Offices	50 (2.40)	2,000 (8.90)
Corridors above first floor	80 (3.83)	2,000 (8.90)
Penal institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Recreational uses		
Bowling alleys, poolrooms, and similar uses	75 (3.59)[a]	
Dance halls and ballrooms	100 (4.79)[a]	
Gymnasiums	100 (4.79)[a]	
Reviewing stands, grandstands, and bleachers	100 (4.79)[a,k]	
Stadiums and arenas with fixed seats (fastened to the floor)	60 (2.87)[a,k]	
Residential		
One- and two-family dwellings		
Uninhabitable attics without storage	10 (0.48)[l]	
Uninhabitable attics with storage	20 (0.96)[m]	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs	40 (1.92)	
All other residential occupancies		
Private rooms and corridors serving them	40 (1.92)	
Public rooms[a] and corridors serving them	100 (4.79)	
Roofs		
Ordinary flat, pitched, and curved roofs	20 (0.96)[n]	
Roofs used for roof gardens	100 (4.79)	
Roofs used for assembly purposes	Same as occupancy served	
Roofs used for other occupancies	[o]	[o]
Awnings and canopies		
Fabric construction supported by a skeleton structure	5 (0.24) nonreducible	300 (1.33) applied to skeleton structure
Screen enclosure support frame	5 (0.24) nonreducible and applied to the roof frame members only, not the screen	200 (0.89) applied to supporting roof frame members only
All other construction	20 (0.96)	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000 (8.9)
All other primary roof members		300 (1.33)
All roof surfaces subject to maintenance workers		300 (1.33)
Schools		
Classrooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
First-floor corridors	100 (4.79)	1,000 (4.45)
Scuttles, skylight ribs, and accessible ceilings		200 (0.89)
Sidewalks, vehicular driveways, and yards subject to trucking	250 (11.97)[a,p]	8,000 (35.60)[q]
Stairs and exit ways	100 (4.79)	300[r]
One- and two-family dwellings only	40 (1.92)	300[r]

Table 1.3-1: Minimum Uniformly Distributed Live Loads, and Minimum Concentrated Live Loads, Continued.

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)[a]	
Heavy	250 (11.97)[a]	
Stores		
Retail		
First floor	100 (4.79)	1,000 (4.45)
Upper floors	75 (3.59)	1,000 (4.45)
Wholesale, all floors	125 (6.00)[a]	1,000 (4.45)
Vehicle barriers	See Section 4.5	
Walkways and elevated platforms (other than exit ways)	60 (2.87)	
Yards and terraces, pedestrian	100 (4.79)[a]	

[a]Live load reduction for this use is not permitted by Section 4.7 unless specific exceptions apply.

[b]Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 lb (13.35 kN) acting on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm); and (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 lb (10 kN) per wheel.

[c]Design for trucks and buses shall be per AASHTO LRFD Bridge Design Specifications; however, provisions for fatigue and dynamic load allowance are not required to be applied.

[d]Uniform load shall be 40 psf (1.92 kN/m^2) where the design basis helicopter has a maximum take-off weight of 3,000 lbs (13.35 kN) or less. This load shall not be reduced.

[e]Labeling of helicopter capacity shall be as required by the authority having jurisdiction.

[f]Two single concentrated loads, 8 ft (2.44 m) apart shall be applied on the landing area (representing the helicopter's two main landing gear, whether skid type or wheeled type), each having a magnitude of 0.75 times the maximum take-off weight of the helicopter and located to produce the maximum load effect on the structural elements under consideration. The concentrated loads shall be applied over an area of 8 in. by 8 in. (200 mm by 200 mm) and shall not be concurrent with other uniform or concentrated live loads.

[g]A single concentrated load of 3,000 lbs (13.35 kN) shall be applied over an area 4.5 in. by 4.5 in. (114 mm by 114 mm), located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load need not be assumed to act concurrently with other uniform or concentrated live loads.

[h]The loading applies to stack room floors that support nonmobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 90 in. (2,290 mm); (2) the nominal shelf depth shall not exceed 12 in. (305 mm) for each face; and (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 36 in. (914 mm) wide.

[i]In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 24 lb per linear ft of seat applied in a direction parallel to each row of seats and 10 lb per linear ft of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

[j]Uninhabitable attic areas without storage are those where the maximum clear height between the joist and rafter is less than 42 in. (1,067 mm), or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 in. (1,067 mm) in height by 24 in. (610 mm) in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirement.

[k]Uninhabitable attic areas with storage are those where the maximum clear height between the joist and rafter is 42 in. (1,067 mm) or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 in. (1,067 mm) in height by 24 in. (610 mm) in width, or greater, within the plane of the trusses. At the trusses, the live load need only be applied to those portions of the bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 in. (508 mm) in width by 30 in. (762 mm) in length that is located where the clear height in the attic is a minimum of 30 in. (762 mm); and

ii. The slope of the truss bottom chord is no greater than 2 units vertical to 12 units horizontal (9.5% slope).

The remaining portions of the bottom chords shall be designed for a uniformly distributed nonconcurrent live load of not less than 10 lb/ft^2 (0.48 kN/m^2).

[l]Where uniform roof live loads are reduced to less than 20 lb/ft^2 (0.96 kN/m^2) in accordance with Section 4.8.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable load effect.

[m]Roofs used for other occupancies shall be designed for appropriate loads as approved by the authority having jurisdiction.

[n]Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

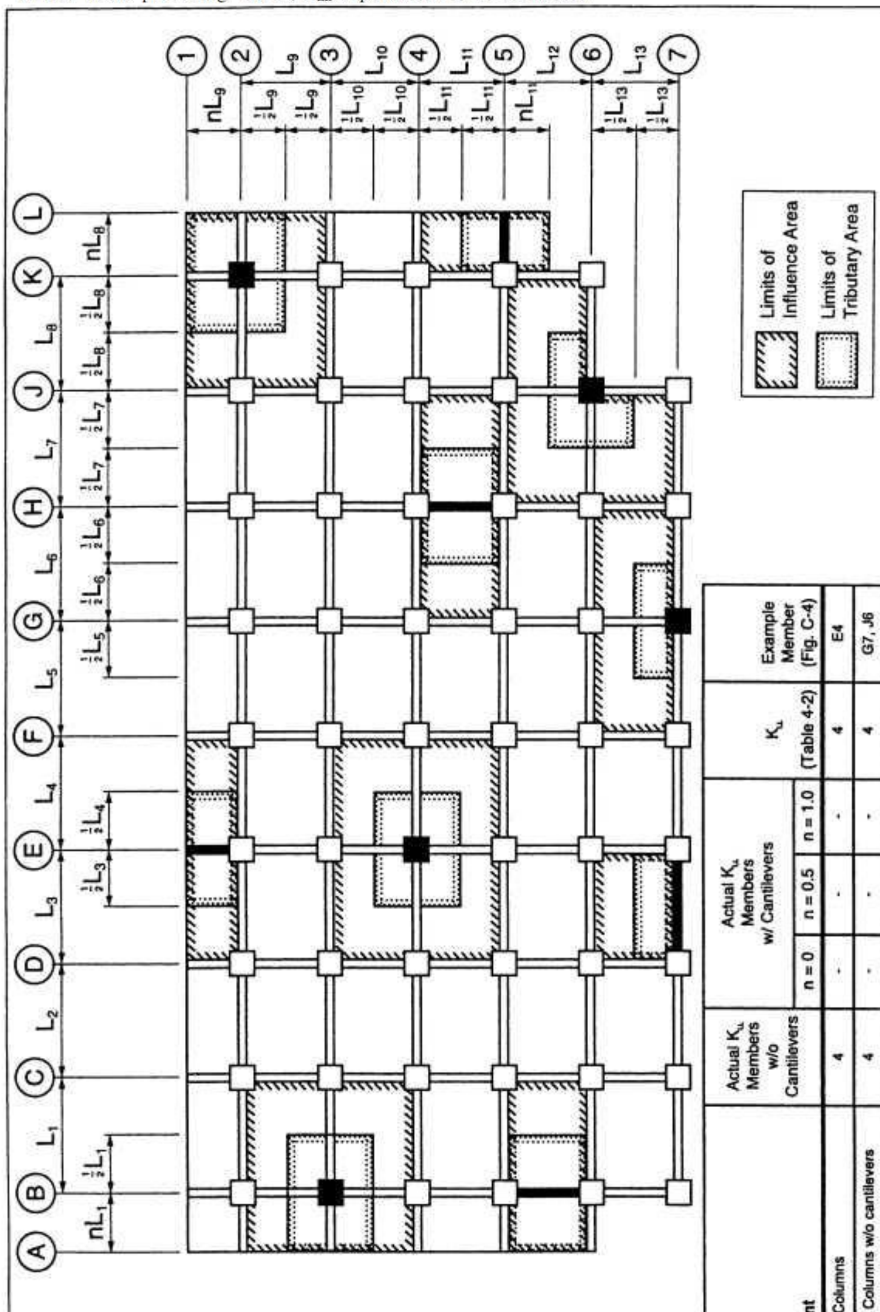
[o]The concentrated wheel load shall be applied on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm).

[p]Minimum concentrated load on stair treads (on area of 2 in. by 2 in. [50 mm by 50 mm]) is to be applied nonconcurrent with the uniform load.

Table 1.3-2: Live Load Element Factor, K_{LL}

Element	K_{LL}[a]
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified, including:	1
Edge beams with cantilever slabs	
Cantilever beams	
One-way slabs	
Two-way slabs	
Members without provisions for continuous shear transfer normal to their span	

[a]In lieu of the preceding values, K_{LL} is permitted to be calculated.



Element	Actual K_{LL} Members w/o Cantilevers	Actual K_{LL} Members w/ Cantilevers n = 0	n = 0.5	n = 1.0	K_{LL} (Table 4-2)	Example Member (Fig. C-4)
Interior Columns	4	-	-	-	4	E4
Exterior Columns w/o cantilevers	4	-	-	-	4	G7, J6
Edge Columns w/ cantilevers	-	4	3	2.67	3	B3
Corner Columns w/ cantilevers	-	4	2.25	1.78	2	K2
Edge Beams w/o cantilever slabs	2	-	-	-	2	D7 - E7
Interior Beams	2	-	-	-	2	H4 - H5
Edge Beams w/ cantilever slabs	-	2	1.5	1.33	1	B5 - B6
Cantilever Beams w/o cantilever slabs	2	-	-	-	1	E1 - E2
Cantilever Beams w/ cantilever slabs	-	2*	1.5*	1.33*	1	K5 - L5

* The value of n for member K5-L5 is used to calculate the distance nL_{11}

Example 1.3-1

Flat plate system indicated in ***Figure 1.3-6*** below is proposed for a school building. Almost all floors area are classes. For this building,

- According to requirements of ASCE 7-10, select an appropriate value for floor live load.
- Compute live load resultant acting on a typical interior column. Reduce floor live load if possible.

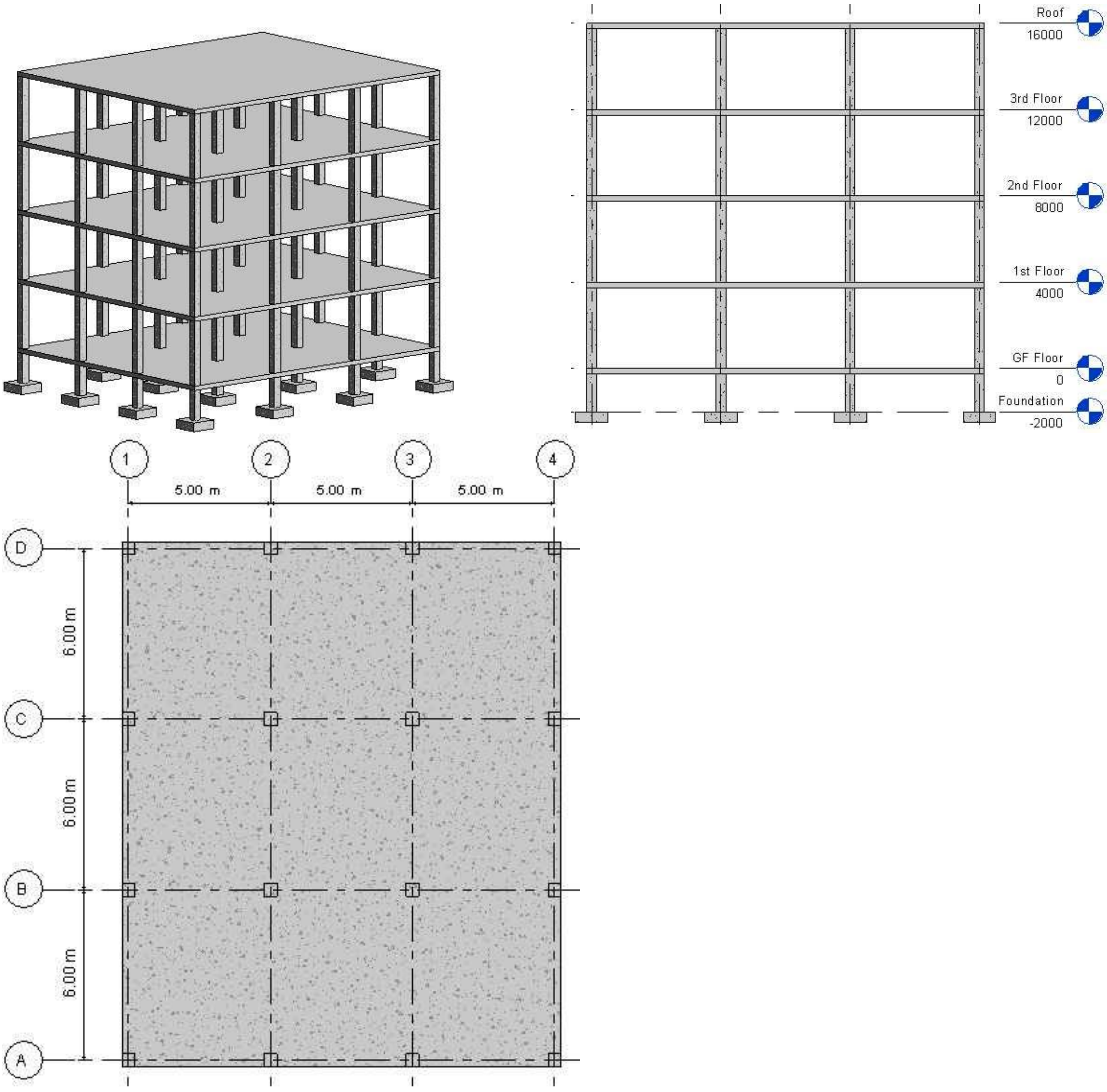


Figure 1.3-6 Flat plate building for Example 1.3-1.

Solution

- Floor Live Load

According to ASCE 7-10, live load for classrooms is:

$$L_L = 1.92 \frac{kN}{m^2} = 1.92\ kPa\ \blacksquare$$

- Resultant of an Interior Column

As live load is less than $4.79\ kPa$, therefore it is reducible according to ASCE 7-10. Regarding to live load acting on a typical interior column, its useful to note that in regular system with equal spans, interior column is assumed to support a tributary area bounded by centerlines of adjacent panels, see Figure 1.3-7 below.

$$A_{T\ Supported\ by\ an\ Interior\ Column} = (5 \times 6) \times 4 = 120\ m^2$$

According to Table 1.3-2 above:

$K_{LL} = 4$,

The influence area is:

$K_{LL}A_T = 4 \times 120 = 480\ m^2 > 37.16\ m^2$

Therefore, the reduction in live load is permitted.

$L = L_o\left(0.25 + \frac{4.57}{\sqrt{4 \times 120}}\right) = 0.458\ L_o > 0.4L_O \therefore Ok.$

The resultant of live load acting on a typical interior column is:

$P_L = 0.458 \times 1.92 \times 120 = 106\ kN$ ■

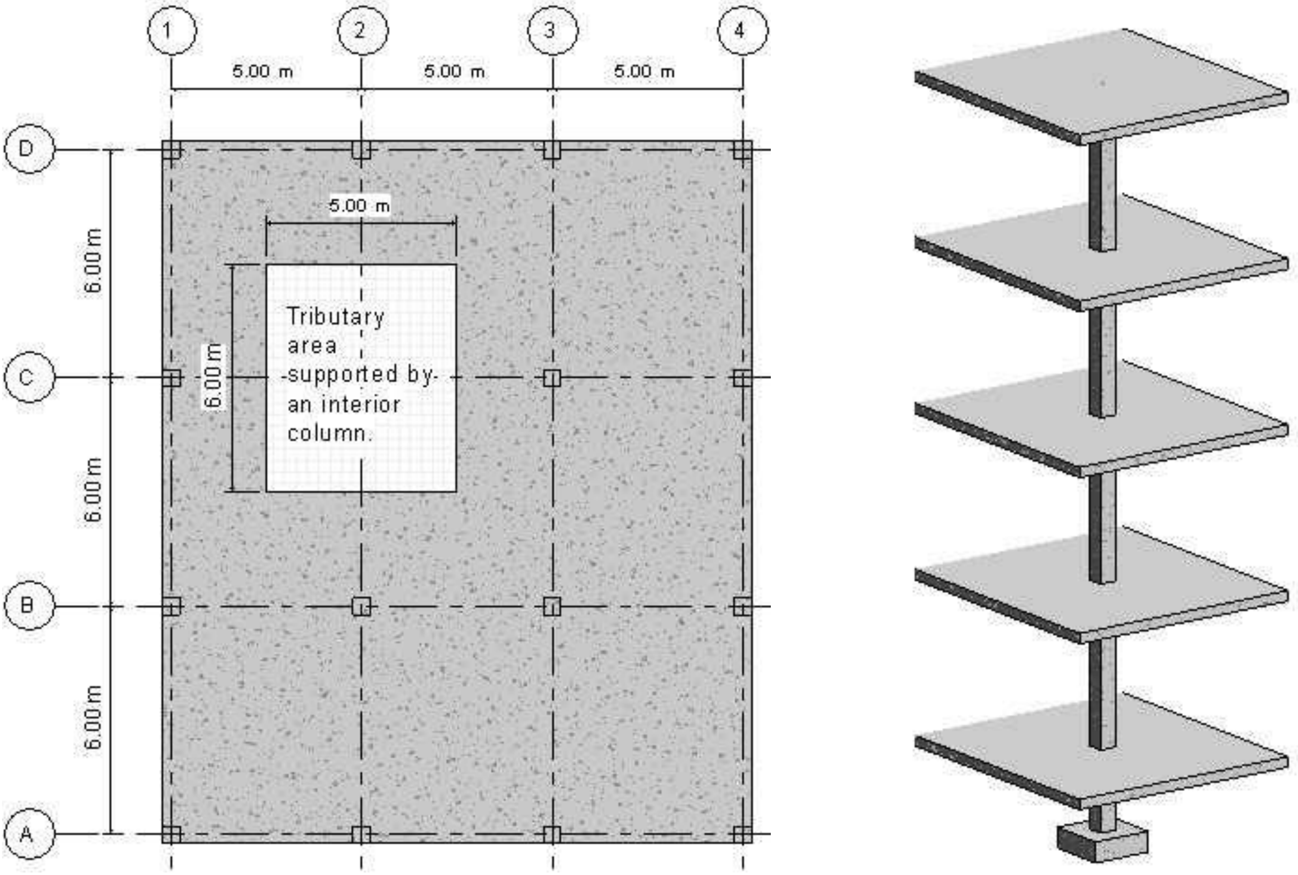


Figure 1.3-7: Tributary area supported by a typical interior column.

Example 1.3-2

Floor system presented in Figure 1.3-8 below is proposed for patient rooms in a hospital building. According to ASCE 7-10:

- Proposed a suitable floor live to be adopted for this floor system,
- Reduce floor live load for a typical interior floor beam,

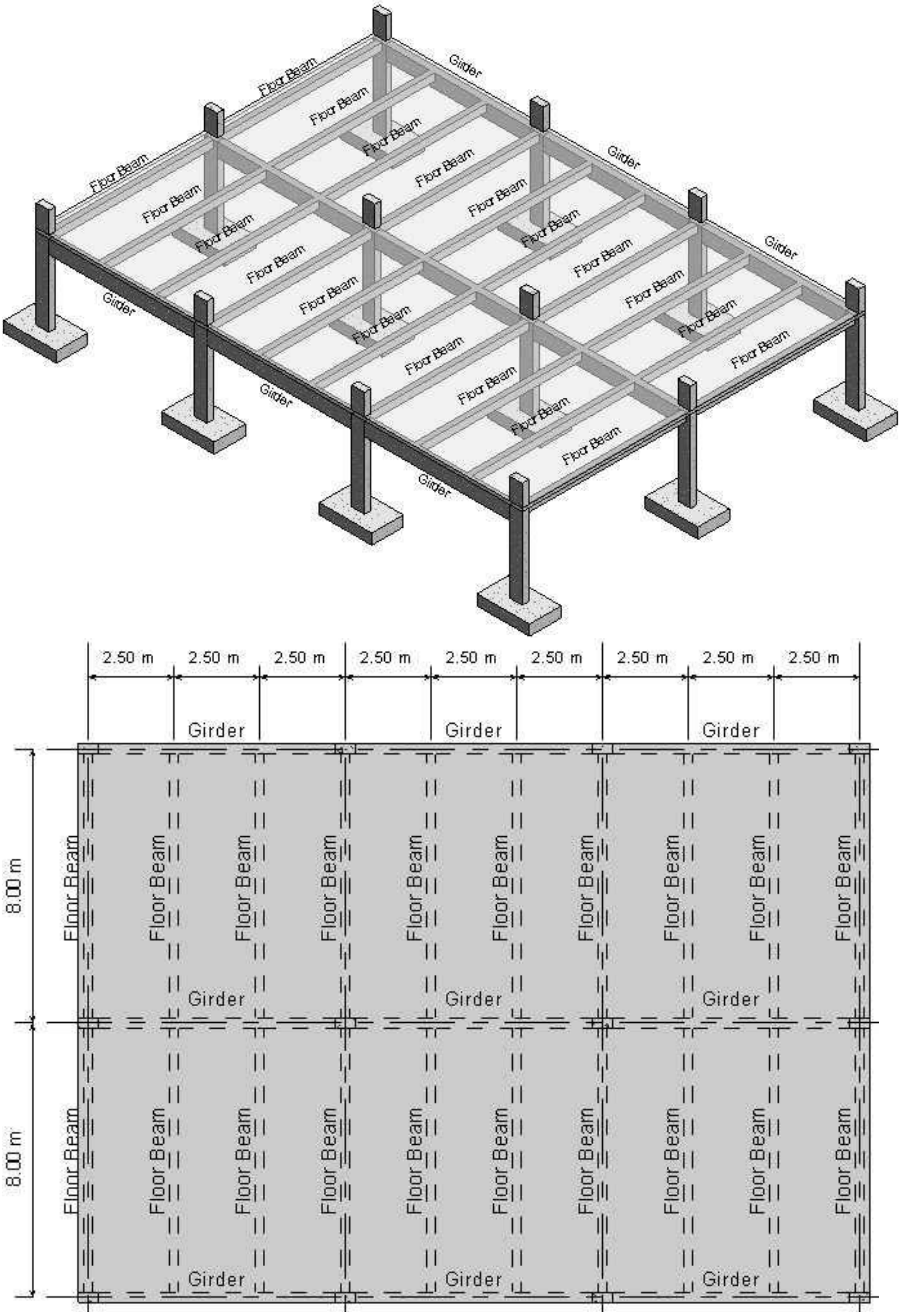


Figure 1.3-8: Floor system for Example 1.3-2.

Solution

According to Table 1.3-1 above, live load for patient rooms in hospital buildings is:

$L_{o\ \text{for patient rooms}} = 1.92\ kPa$ ■

In a one-way floor system, the tributary area supported by a typical interior beam is indicated in Figure 1.3-9 below.

$\therefore A_{T\ for\ typical\ interior\ floor\ beam} = 2.5 \times 8 \times 2 = 40\ m^2$

With K_{LL} factor of 2 according to Table 1.3-2 above, influence area for a typical interior floor beam would be:

$K_{LL}A_T = 2 \times 40 = 80\ m^2 > 37.16\ m^2$

$\because L_o = 1.92\ kPa < 4.79\ kPa$

Therefore, live load of a typical floor beam is reducible and can be estimated from relation below:

$$L = L_o\left(0.25 + \frac{4.57}{\sqrt{80}}\right) = 0.76\ L_O$$

As floor beams contribute in supporting their own story only, therefore reduced live load should be limited by $0.5L_o$.

$L = 0.76L_o = 0.76 \times 1.92 = 1.46\ kPa$ ■

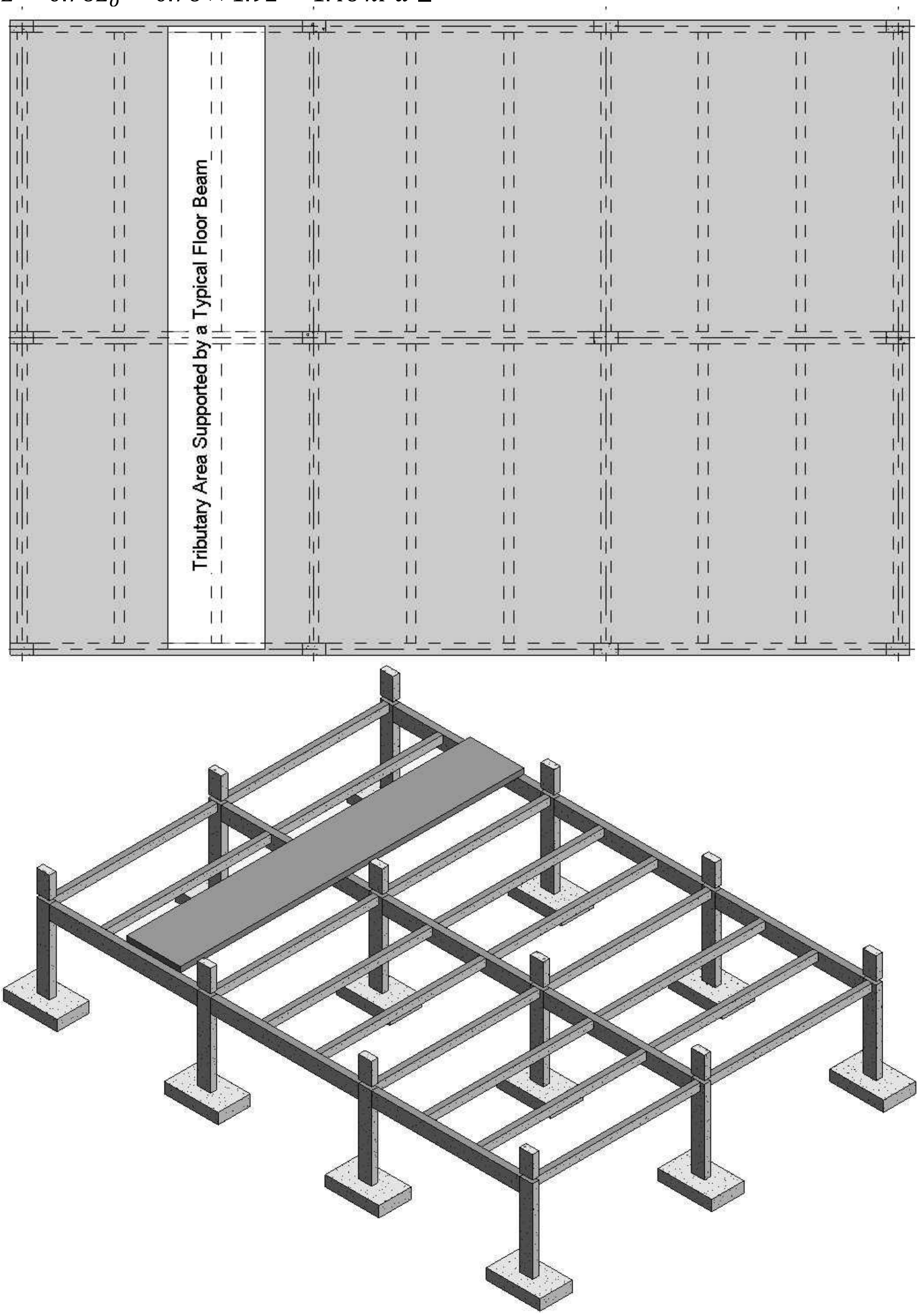


Figure 1.3-9: Tributary area supported by a typical interior floor beam.

1.3.2.3 Roof Live Load

1.3.2.3.1 Basic Value of Roof Live Load

The minimum uniformly distributed roof live loads, L_o, can be estimated from values presnted in Table 1.3-1 above.

1.3.2.3.2 Reduction of Roof Live Load

According to (ASCE/SEI 7–10), roof live load, L_o, can be reduced according to following relation:

$$L_r = L_o R_1 R_2 \qquad 0.58\,kPa \le L_r \le 0.96\,kPa \qquad \textbf{Eq. 1.3-2}$$

where

L_r is reduced roof live load per m^2 of horizontal projection supported by the member,
L_o is unreduced design roof live load per m^2 of horizontal projection supported by the member, (see Table 1.3-1 above).

The reduction factor R_1 simulates reduction of roof live load as a function of loaded area and it can be estimated from following relation:

$$R_1 = \begin{matrix} 1 & for\ A_T \le 18.58\,m^2 \\ 1.2 - 0.011A_T & for\ 18.58\,m^2 < A_T < 55.74\,m^2 \\ 0.6 & for\ A_T \ge 55.74\,m^2 \end{matrix}$$

where A_T is tributary area in m^2 supported by the member.

While the reduction factor, R_2, simulates reduction in roof live load with increasing in roof slope and it can be estimated from relation below:

$$R_2 = \begin{matrix} 1 & for\ F \le 4 \\ 1.2 - 0.05F & for\ 4 < F < 12 \\ 0.6 & for\ F \ge 12 \end{matrix}$$

where, for a pitched roof, F = 0.12 × slope, with slope expressed in percentage points.

Example 1.3-3

For ***Example 1.3-1*** above, select an appropriate value for the roof live load and compute the force resultant that supported by an interior column. In your computation, reduce roof live loads if possible.

Solution

As nothing is mentioned in the example statement about the nature of the roof, therefore an ordinary roof has been assumed. The roof live load is:

$L_o = 0.96\,kPa$ ■

Assuming that an interior column supports a tributary area bounded by centerlines of adjacent panels, see Figure 1.3-10,

$A_T = 5 \times 6 = 30\,m^2$

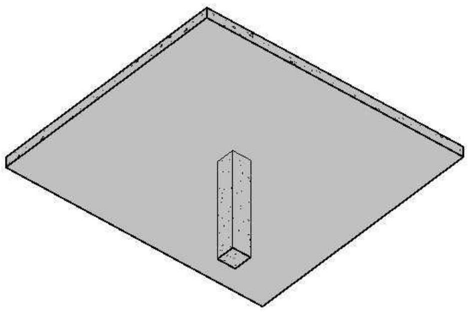


Figure 1.3-10: Roof area supported by a typical interior column for Example 1.3-3.

The reduction factor, R_1, is:

$\because 18.58\,m^2 < A_T < 55.74\,m^2$

$\therefore R_1 = 1.2 - 0.011A_T = 1.2 - 0.011 \times 30 = 0.87$

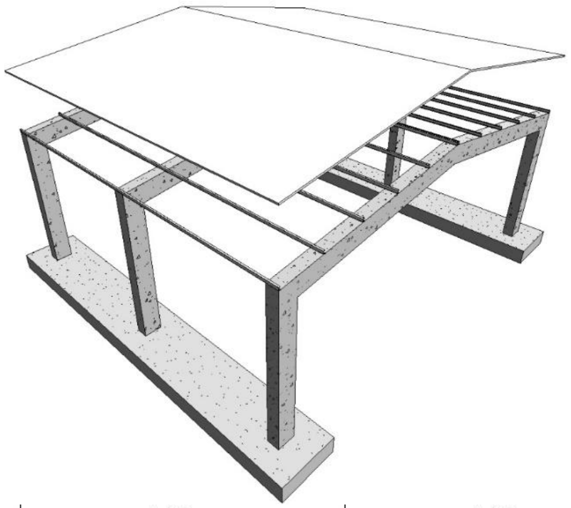
For flat roof,

$R_2 = 1.0$

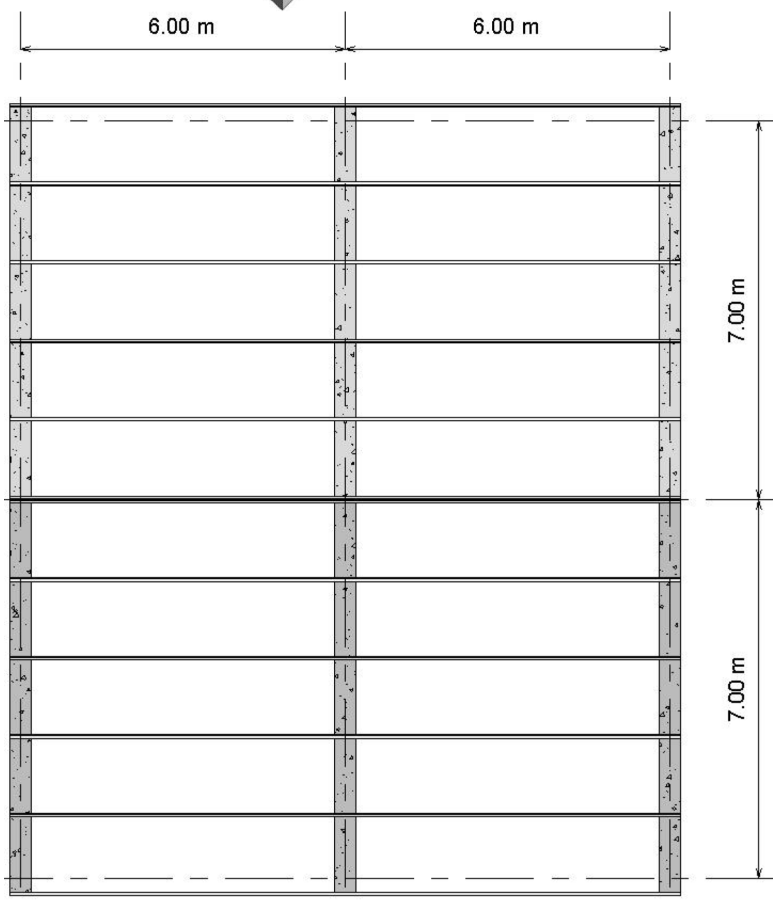
$$P_{Due\ to\ R_o} = \left(0.96\frac{kN}{m^2} \times 30\,m^2\right) \times 0.87 \times 1.0 = 25.1\,kN \ ■$$

Example 1.3-4

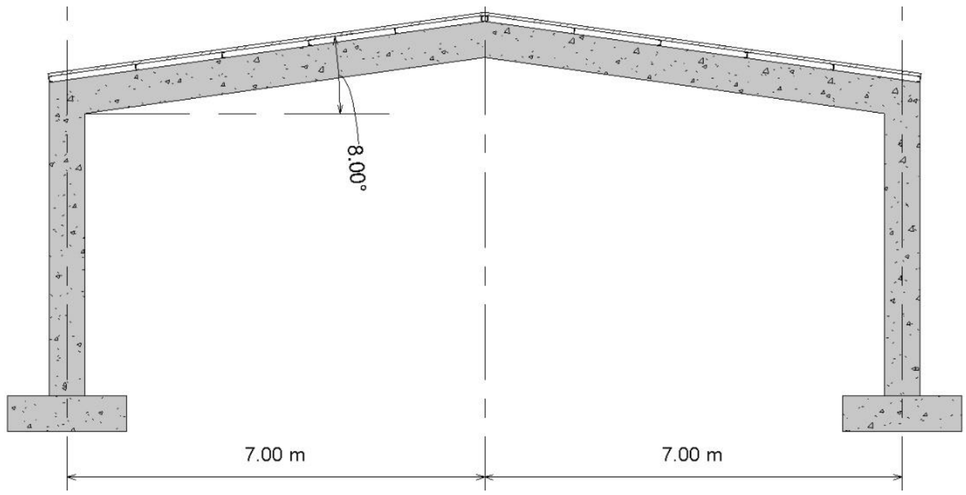
For gable frame presented in ***Figure 1.3-11*** below, select a suitable roof live load then determine the reduced live load that supported by the interior frame.



3D view.



Plan view.



Sectional view.

Figure 1.3-11: Gable frame for Example 1.3-4.

Solution

Assuming an ordinary pitched roof, roof live load according to ***Table 1.3-1*** above would be:

$L_r = 0.96\ kPa$ ■

According to ***Eq. 1.3-2*** above, the reduced roof live load is:

$L_r = L_o R_1 R_2 \qquad 0.58\ kPa\ \leq L_r \leq 0.96\ kPa$

As this live load is acting on the inclined surface, therefore the tributary area would be:

$$A_T = \left(\frac{7}{cos8} \times 2\right) \times \left(\frac{6}{2} \times 2\right) = 84.8\ m^2$$

$\because A_T \geq 55.74\ m^2\ \Rightarrow\ R_1 = 0.6$

In computing R_2, the F factor is determined as follows:

$F = 0.12 \times Slop_{Expressed\ in\ percentage\ units} = 0.12 \times (tan8) \times 100 = 1.69$

$\because F \leq 4 \Rightarrow \therefore R_2 = 1.0$

Hence, the reduced roof live load that supported by the interior frame would be:

$L_r = L_o R_1 R_2 = 0.96 \times 0.6 \times 1.0 \approx 0.58\ kPa = Lower\ bound\ \therefore Ok.$ ■

Example 1.3-5

For industrial building indicated in ***Figure 1.3-12*** below:

- Select an appropriate value for roof live load. The roof has a slope of 10^o.
- Reduce the selected roof live load, if possible, to determine its resultant on the indicated typical edge column.
- If the building floor is proposed for a light manufacturing process, determined the live load that should be adopted according to ASCE/SEI 7-10.
- Is the selected live load reducible or not? Explain your answer.
- Determine live load resultant on the indicated typical edge column.

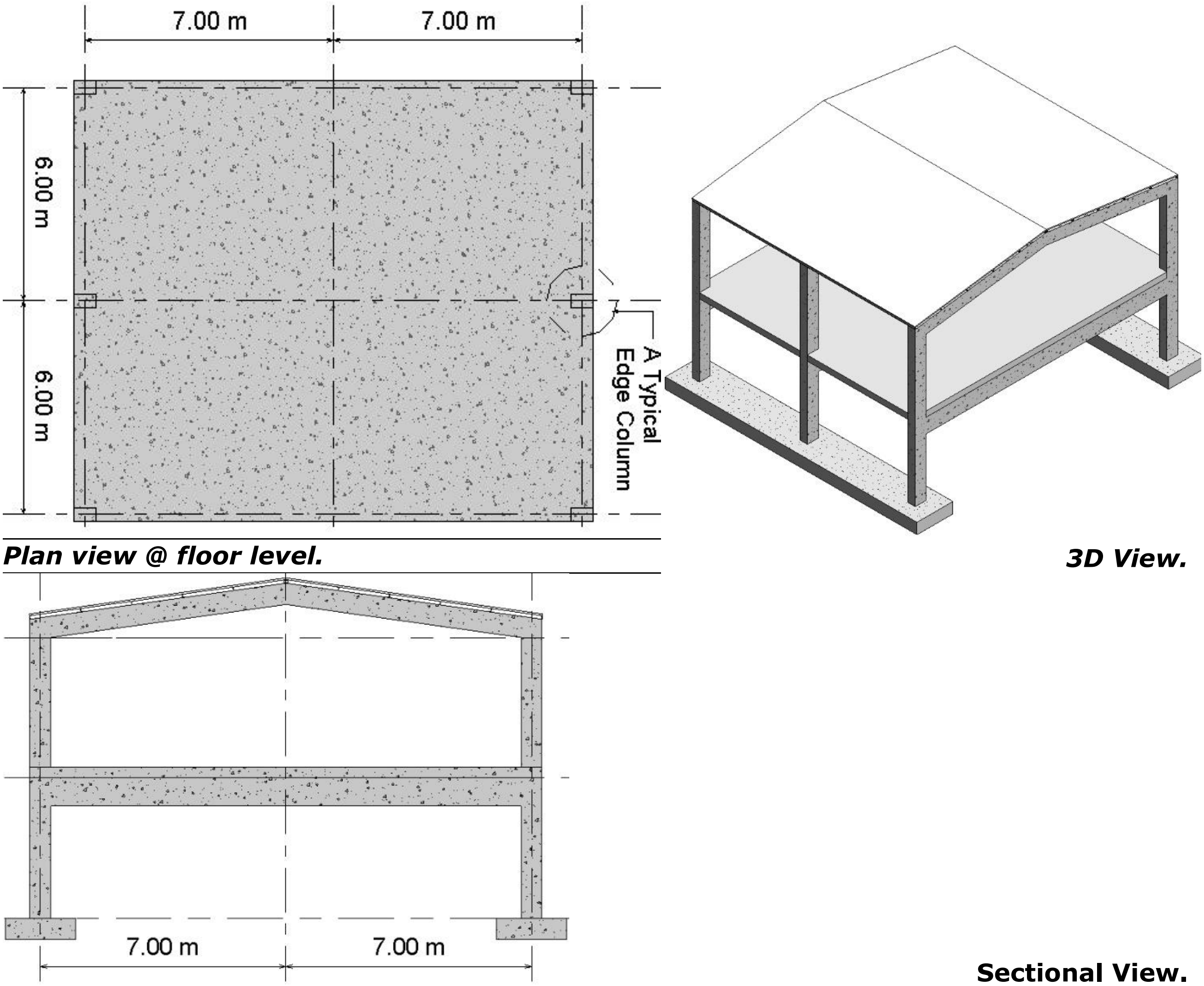


Figure 1.3-12: Structural system for the industrial building Example 1.3-5.

Solutions

- Appropriate value for roof live load:
  According to ASCE 7-10, live load for ordinary flat roof is:
  $L_r = 0.96\ kPa$
- Reduce of roof live load:
  $A_{T\ typical\ edge\ column} = \frac{7}{cos10} \times 6 = 42.6\ m^2 \Rightarrow \because\ 18.58\ m^2 < A_T < 55.74\ m^2$
  $\therefore R_1 = 1.2 - 0.011A_T = 1.2 - 0.011 \times 42.6 = 0.731$
  $F = 0.12 \times Slop_{Expressed\ in\ percentage\ units} = 0.12 \times (tan10) \times 100 = 2.12 \Rightarrow \because F \leq 4 \Rightarrow \therefore R_2 = 1.0$
  $L_r = L_o R_1 R_2 = 0.96 \times 0.731 \times 1.0 \approx 0.702\ kPa > 0.58\ kPa \therefore Ok.$ ■
- Floor live load:
  Assuming a light manufacturing process, the floor live load according to ASCE/SEI 7-10 is:
  $L = 6.0\ kPa$
- Reduction of floor live load:
  Floor live of $6.00\ kPa$ is irreducible according to ASCE/SEI 7-10 as it is greater than $4.79\ kPa$.
- Live load resultant on the indicated edge column:
  $P_L \approx 6\ kPa \times 7m \times 6m = 252\ kN$ ■

1.3.3 Environmental Loads

Environmental loads can be sub-classified into the following types:

1. Wind Loads.
2. Earthquake Loads.
3. Soil Pressure Loads.
4. Snow Loads.
5. Rain Loads.
6. Force caused by a differential temperature.

Like live loads, environmental loads at any given time are uncertain both in magnitude and in distribution. Therefore, their values and the distribution must be determined based on the codes and specifications like the *International Building Code* or *Minimum Design Loads for Buildings and Other Structures*.

1.3.3.1 Wind Loads

1.4 DESIGN CODES AND SPECIFICATIONS

After selection of a suitable structural system based on the functional and/or architectural requirements, the structural design process can be summarized by following three steps:

Prediction of the applied loads → Make a structural analysis to compute the member internal forces and the structural deformations that related to the applied loads. → Compute the member strength and then assessment the member adequacy based on the comparison between the applied loads and the member strength.

As was shown in the previous article on the loads and as will be shown in the next articles, each one of the above steps contains some kind of uncertainty. To deal with these uncertainties in the design process, the engineers must base their design decision not only on the theoretical aspects but also on the previous experience that usually written in the form of codes or specifications which edited by professional groups and technical institutes.

Following list states most important professional groups and technical institutes:

1.4.1 American Society of Civil Engineers (ASCE)

Produce a document titled "*Minimum Design Loads for Buildings and Other Structures*, *ASCE 7-10*" that is usually used in the definition of loads magnitude, distribution, and load combinations that should be considered in the structural design.

1.4.2 American Concrete Institute (ACI)

Produce documents that including provisions for the concrete design and construction. The "*Buildings Code Requirements for the Structural Concrete (ACI 318M-14)*" that related to the design and construction concrete buildings is an example of these documents.

1.4.3 American Association of State Highway and Transportation Officials (AASHTO)

Produce documents that related to the design and construction of the highway projects and highway bridges.

1.4.4 American Railway Engineering Association (Area)

Produce the documents that related to the design and construction of the railway projects.

1.5 DESIGN CRITERIA

Following criteria are usually adopted in design and assessment of different structural elements:

1.5.1 Criteria for Beams Design

Design and assessment of a reinforced concrete beam are based on the following criteria:

1.5.1.1 Strength criterion

Including checking or design for flexure strength, shear strength, torsion strength, and bond strength of the reinforced concrete beam.

1.5.1.2 Serviceability criterion

Including checking for adequacy of reinforced concrete beams for deflection, crack width, and vibration (vibration is out the scope of this course).

1.5.1.3 Stability criterion

As stated in theory of structure, a plane structure is stable when supported by three reactions or more that neither all parallel nor all concurrent at a single point.

Due to rough nature of surfaces in concrete and masonry structure, most of reinforced concrete beams are stable in nature. Consider for example the reinforced beam indicated in Figure 1.5-1(a) below due to surface roughness, a beam to wall connection can be simulated as a hinge. With two hinge supports indicated in Figure 1.5-1(b), a membrane force develops in the beam in addition to shear force and bending moment. In traditional reinforced concrete design, this membrane force is usually neglected and the beam is simulated as presented Figure 1.5-1(c).

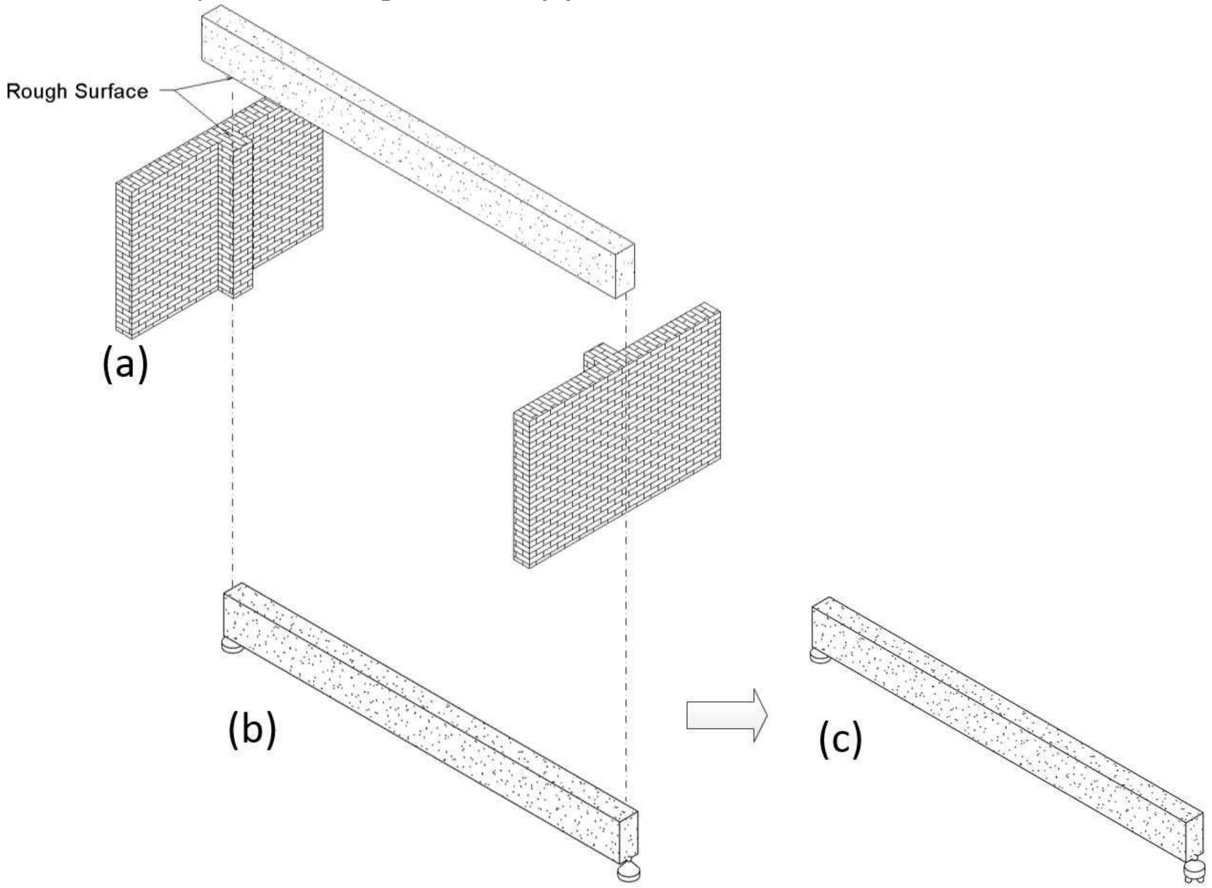


Figure 1.5-1: A simply supported reinforced concrete beam.

1.5.2 Criteria for Slabs Design

Design and assessment of the one-way slabs or two-way slabs are generally based on the following criteria:

1.5.2.1 Strength criterion

Including checking or design for flexure strength, shear strength, and bond strength of the reinforced concrete slab.

1.5.2.2 Serviceability criterion

Including checking for adequacy of reinforced concrete slabs for deflection, crack width, and vibration (vibration out the scope of our course).

1.5.2.3 Stability criterion

As discussed in stability criterion for beams, reinforced concrete slabs are stable in nature due to surface roughness.

1.5.3 Criteria for Columns Design

Design and assessment of the reinforced concrete columns are based on the following criteria.

1.5.3.1 Strength criterion

Including checking for flexure and axial strength of reinforced concrete columns.

1.5.3.2 Stability criterion

In additional to general stability criteria that related to number and arrangement of reactions, stability of some columns, called slender columns, is a function of axial load. For a specific level of axial forces, called column critical load or Euler load, the column is unstable in a sense that it cannot return to its equilibrium position after a small lateral disturbance, see Figure 1.5-2 below.

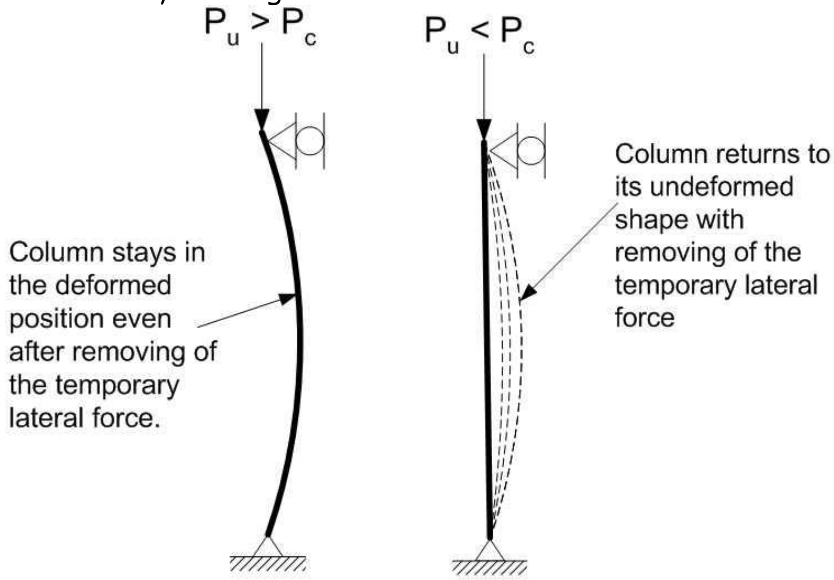


Figure 1.5-2: Physical interpretation of critical load.

In addition to stability aspect, equilibrium equations for a slender column should be formulated in terms of deformed shape instead of undeformed shape to take into account the effects of secondary moments, see Figure 1.5-3 below.

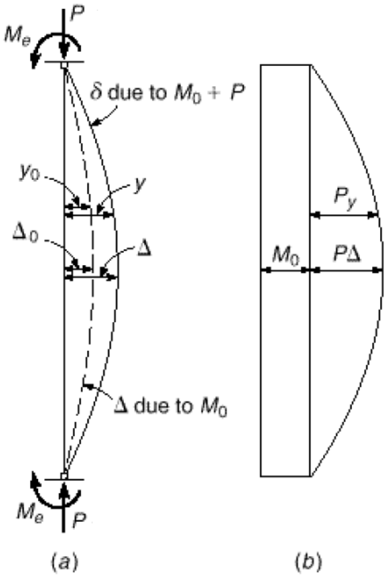


Figure 1.5-3: Secondary moment effects in column analysis.

1.5.3.3 Serviceability

As indicated in Figure 1.5-4 below, generally, axial deformation of columns produce rigid body motion in beams and floor systems and can be disregarded in serviceability checking.

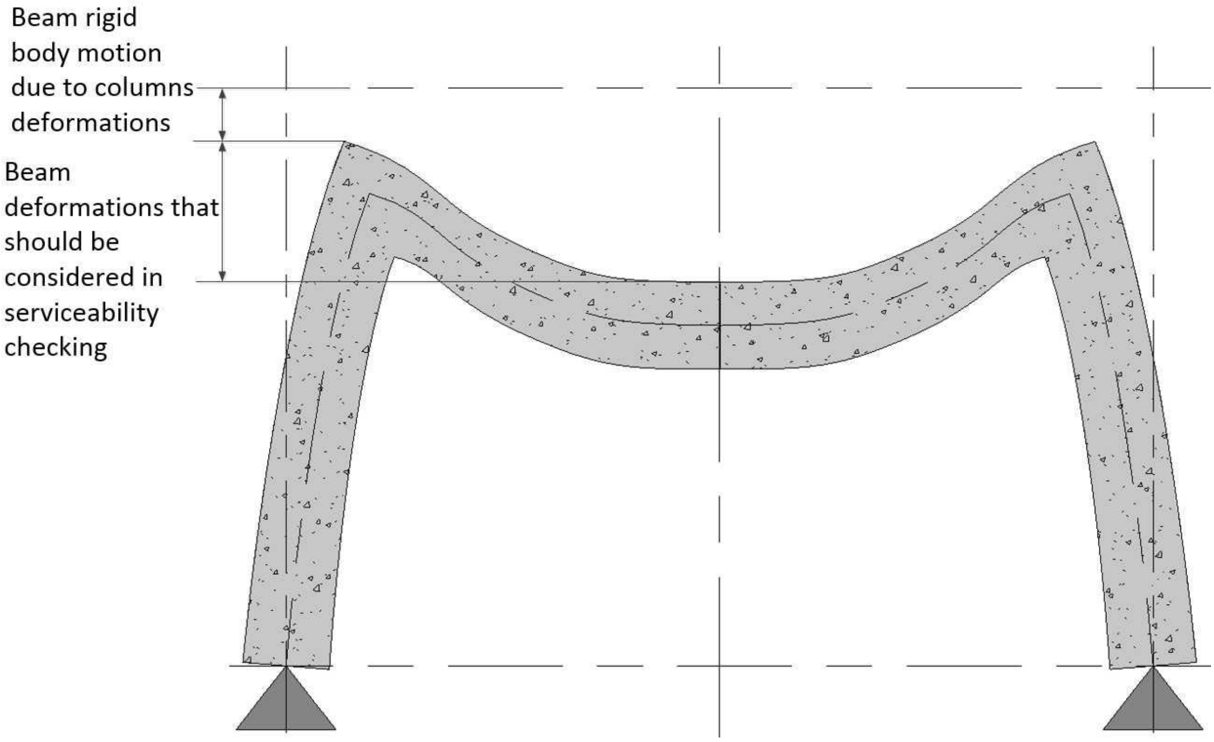


Figure 1.5-4: Rigid body motion and deformation of beams.

1.6 DESIGN PHILOSOPHY

Uncertainties in the analysis, design, and construction of reinforced concrete structures can be summarized in the diagram below:

Uncertainty in the Analysis and Design of Reinforced Concrete Structure can be Classified into:

Uncertainty in Prediction the Applied Load: This is due to
- Usually, there is a difference between the actual and predicted loads.
- Generally, the actual loads have a distribution that differ from the assumed one.

Uncertainty in the Computed Theoretical or Nominal Strength: This is due to
- Actual member dimensions are different from that shown in the drawings and assumed in the calculations.
- Actual materials strength are not exactly the same to that assumed in the design calculations.

1.6.1.1 Load Uncertainty

- Based on statistical data obtained from large-scale survey, load uncertainty can be described in terms of the probability model show below:

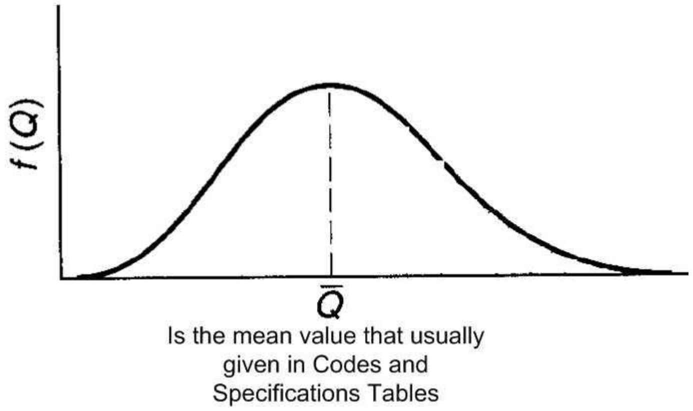


Figure 1.6-1: Frequency curve for load.

- Based on structural type and design code, a designer can select a design load ($\bar{Q}$) from related load Table (e.g. Table 1.3-1).
- If the designer use ($\bar{Q}$) value as a design load, then the designer implicitly accepts a probability of over load in the range of 50% (shaded area in the Figure 1.6-2 below).

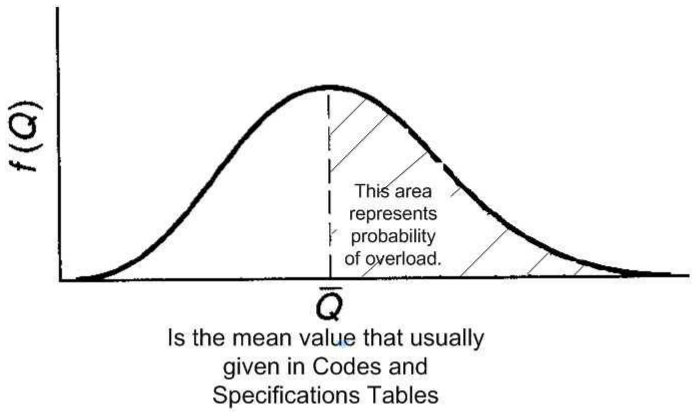


Figure 1.6-2: Adopting of $\bar{Q}$ implicitly equivalent to acceptance a probability of 50% of over load.

- As this probability for over load is so large to be accepted in a design process, the designer should increase the mean value ($\bar{Q}$) to a design value (Q_d) (See Figure below):

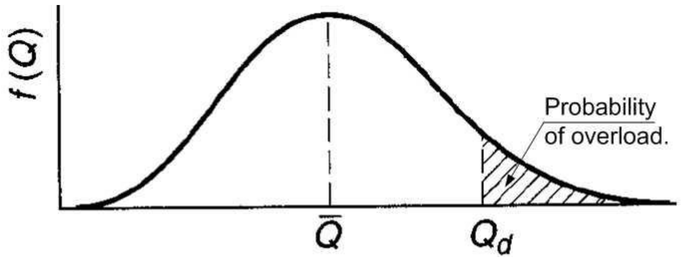


Figure 1.6-3: Factored load with low probability of overload.

- Above increasing or magnification is done based on following relation:
  $Q_d = \gamma \bar{Q}$ **Eq. 1.6-1**
  where
  Q_d is the factored load that will be used in structural design or assessment.
  $\bar{Q}$ is the mean value that usually given in Load Tables or computed theoretically.
  γ Load Factor. It is computed according to ACI 5.3.1 (See Table below):

Table 1.6-1: Load combinations

Load Combination	Equation Number according to this course	Equation Number according to the ACI code	Primary load
$\boldsymbol{U = 1.4D}$	Eq. 1.6-2	5.3.1a	D
$\boldsymbol{U = 1.2D + 1.6L + 0.5(L_r or S or R)}$	Eq. 1.6-3	5.3.1b	L
$\boldsymbol{U = 1.2D + 1.6(L_r or S or R) + (1.0L\ or\ 0.5W)}$	Eq. 1.6-4	5.3.1c	L_r or S or R
$\boldsymbol{U = 1.2D + 1.0W + 1.0L + 0.5(L_r or S or R)}$	Eq. 1.6-5	5.3.1d	W
$\boldsymbol{U = 1.2D + 1.0E + 1.0L + 0.2S}$	Eq. 1.6-6	5.3.1e	E
$\boldsymbol{U = 0.9D + 1.0W}$	Eq. 1.6-7	5.3.1f	W
$\boldsymbol{U = 0.9D + 1.0E}$	Eq. 1.6-8	5.3.1g	E

- Notes on Wind Load Combinations:
  - In the version of 2010, the ASCE-7 has converted wind loads to ***strength level*** and ***reduced the wind load factor to 1.0***.
  - The Code requires use of the previous ***load factor for wind loads***, ***1.6***, when ***service-level wind loads are used*** as in the case of ***Iraq wind maps***.
  - For serviceability checks, the commentary to Appendix C of ASCE/SEI 7 provides ***service-level wind loads***, W_a.

1.6.1.2 Strength Uncertainty

1. As all of section dimensions and material strength are changed randomly, then if we compute a theoretical or nominal strength of a section (S_n) based on ideal values for design parameters (section dimensions and material strengths), then if a large samples of this sections are test, probability density function of section strength will be as shown in Figure 1.6-4 below:

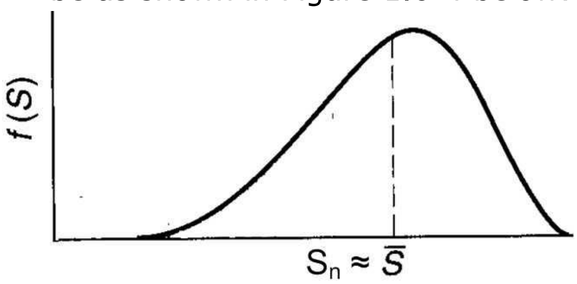


Figure 1.6-4: Frequency curve for strength.

2. If the designer use theoretical or nominal strength of section as a basis for design, he implicitly accept a probability of approximately 50% for overestimation of section strength (see Figure below):