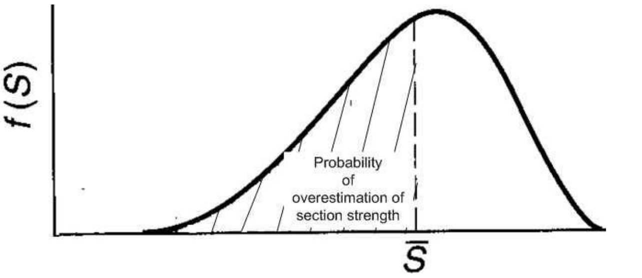


Figure 1.6-5: Adopting of $\bar{S}$ implicitly equivalent to acceptance of 50% of understrength.

3. Therefore, sectional nominal strength (S_n) should be reduced to a design strength (S_d) (see Figure 1.6-6 below).

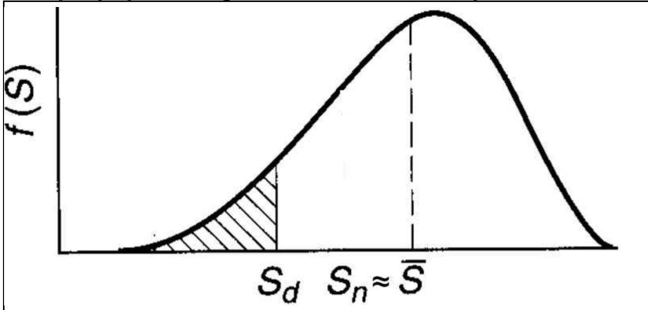


Figure 1.6-6: Factored strength with low probability of understrength.

4. Above reduction will be according to following relation:

   $S_d = \emptyset S_n$ **Eq. 1.6-9**

   where $\emptyset$ is the strength reduction factor that computed based on ACI 21.2.1 (See Table below)

Table 1.6-2: Strength reduction factors φ

Strength Condition	Strength Reduction Factor ϕ
Tension-controlled sections[a]	0.90
Compression-controlled sections[b]	
Members with spiral reinforcement	0.75
Other reinforced members	0.65
Shear and torsion	0.75
Bearing on concrete	0.65
Post-tensioned anchorage zones	0.85
Strut-and-tie models[c]	0.75

[a] Chapter 19 discusses reductions in ϕ for pretensioned members where strand embedment is less than the development length.
[b] Chapter 3 contains a discussion of the linear variation of ϕ between tension and compression-controlled sections. Chapter 8 discusses the conditions that allow an increase in ϕ for spirally reinforced columns.
[c] Strut-and-tie models are described in Chapter 10.

5. Final Design Relation:

   Based on above discussion, a section to be classified as adequate according to strength requirements of the ACI Code, it should satisfied the following relation:

   $\emptyset S_n \geq \gamma \bar{Q}$ **Eq. 1.6-10**

Example 1.6-1

Probability curves for flexural strength of precast beams fabricated by two different manufactures are presented in Figure 1.6-7 below.

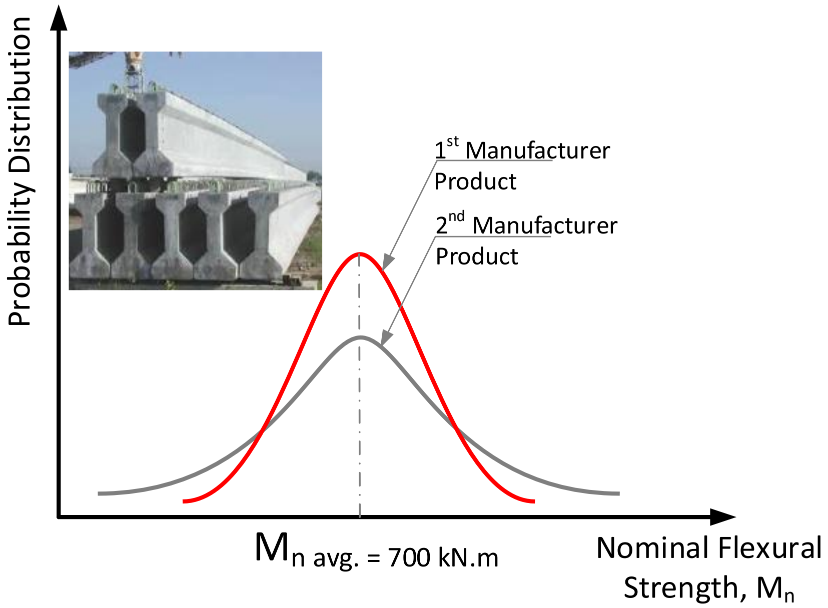


Figure 1.6-7: Probability distribution for precast beams of Example 1.6-1.

1. Which one of two product seems stronger?
2. Which one of two product seems more controlled?
3. Which one of two product needs a larger margin of safety?

Solution

1. In terms of mean strength value, both products have same strength, namely $M_{n\,avg.} = 700\ kN.m$.
2. First product is more controlled than second product as it has lower scatter, deviation, than second product.
3. Second product has a larger margin of safety as it has larger scatter than first product.

Example 1.6-2

Probability curves for two uniformly distributed loads are presented in Figure 1.6-8 below. According to current design philosophy:

- Which one of two loads has larger magnitude?
- Which one of two loads has larger scatter?
- Which curve may represent dead load and which one may represent live load when both loads have same mean value? Explain your answer.

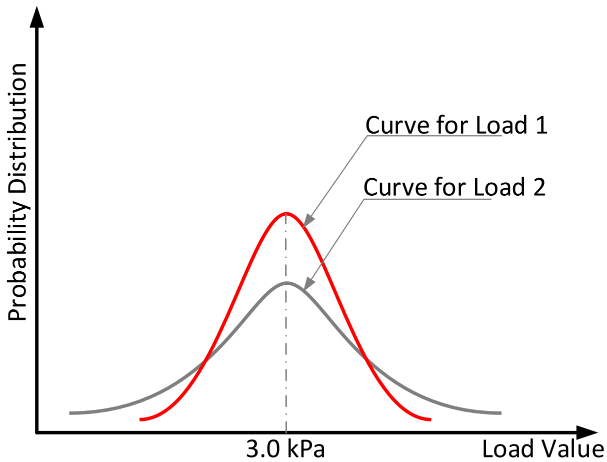


Figure 1.6-8: Probability distribution of uniformly distributed loads for Example 1.6-2.

Solution

- In term of mean value, both loads have same magnitude.
- To be a probability models, area under above curves should equal to one unit. Therefore, width of curve gives an indication on its scatter, its standard deviation, σ, and Load 2 is more scatter than Load 1.

Regarding to their scatters, Load 1 is more suitable to simulate dead load where scatter is smaller than live load.

Example 1.6-3

A factory producing two types of precast beams. According to quality control department, these beams have probability distribution curves shown in Figure 1.6-9 below:

- Which one of two types has larger flexure strength?
- Which one of two types is more controlled?
- Which one of two types need a larger margin of safety?

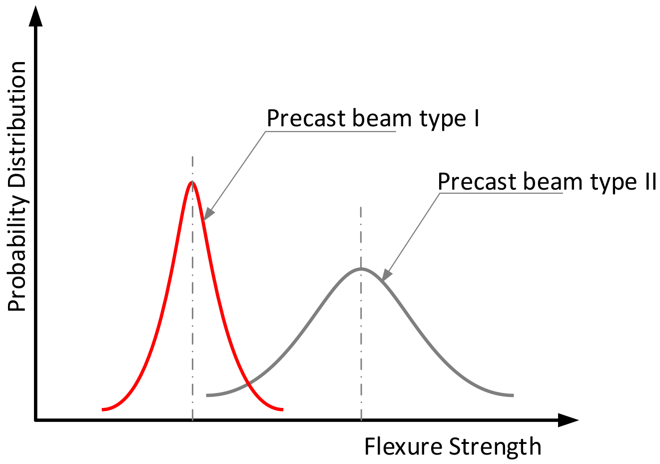


Figure 1.6-9: Probability distributions for flexural strength of precast beams of Example 1.6-3.

Solutions

- In term of mean strength, beam Type II is more strength than Type I.
- Width of curve base gives an indication on standard deviation of the design process. A wider base a less controlled process. Therefore, Type I is more controlled than Type II.

Larger margin of safety should be adopted for beam Type II.

Example 1.6-4

Check adequacy of a simply supported beam presented in Figure 1.6-10 below for flexural strength according to the requirements of ACI318M-14.

Given

$\mathrm{w_D} = 20\,\mathrm{kN/m}$ (Not including beam weight)
$\mathrm{w_L} = 20\,\mathrm{kN/m}$
$\gamma_{\mathrm{Concrete}} = 24\,\mathrm{kN/m^3}$

Nominal (theoretical) flexure strength $\mathrm{M_n} = 1000\,\mathrm{kN.m}$ (This will be computed in details in Chapter 3).

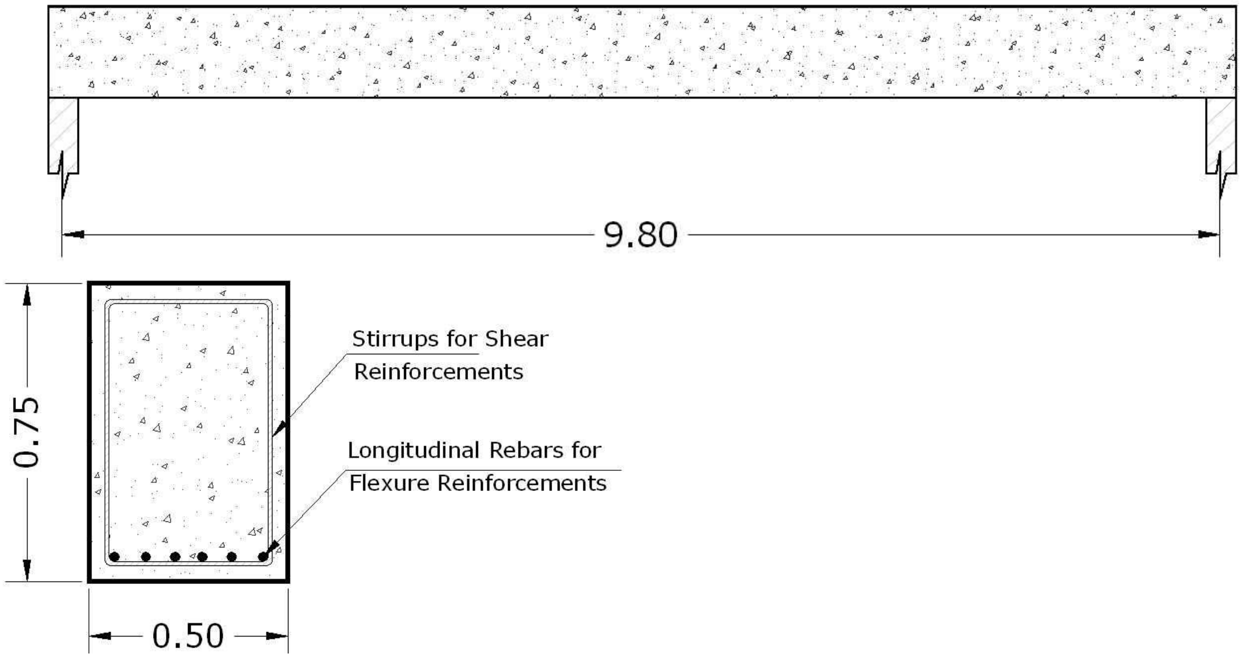


Figure 1.6-10: Simply supported beam for Example 1.6-4.

Solution

1. Compute the Factored Loads:
   Factored load W_u, i.e., the loads that increased to include the load uncertainty can be taken as the maximum of:
   $W_u = 1.4W_{Dead}$
   $W_u = 1.2W_{Dead} + 1.6W_{Live}$
   $w_{Self} = 24\ kN/m^3 \times 0.75m \times 0.5\,m = 9\,kN/m$
   $w_D = 20\,\mathrm{kN/m} + 9\ \mathrm{kN/m} = 29\ \mathrm{kN/m}$
   Then either:
   $$W_u = 1.4 \times 29 \frac{kN}{m} = 40.6 \frac{kN}{m}$$
   or:
   $$W_u = 1.2W_{Dead} + 1.6W_{Live} = 1.2 \times 29 \frac{kN}{m} + 1.6 \times 20 \frac{kN}{m} = 66.8 \frac{kN}{m}$$
   Therefore, the govern value of the factored load is:
   $$W_{u\,Govern} = 66.8 \frac{kN}{m}$$
2. Compute the required flexural strength:
   Bending moment diagram for simply supported beam subjected to uniformly distributed load shows that the maximum bending moment occurs at beam mid span and it has a value of:
   $$M_{u\,maximum} = \frac{W_u l^2}{8} = \frac{66.8 \times 9.8^2}{8} = 802\ kN.m$$
3. Compute the available design strength:
   $\emptyset M_n = 0.9 \times 1000 = 900\,\mathrm{kN.m} > \mathrm{M_u}\ \therefore$ ok.

Example 1.6-5

In addition to its own weight, a building supports loads indicated in Figure 1.6-11 below. Check if frame columns with dimensions of 400x400mm and with a nominal strength of, $P_n = 2800\ kN$, at their base, are adequate to support applied loads?
Consider following load combinations in checking,
$U = 1.4D$
$U = 1.2D + 1.6L$
In your solution assume a strength reduction factor, $\phi = 0.65$, and that footings behave as hinges.

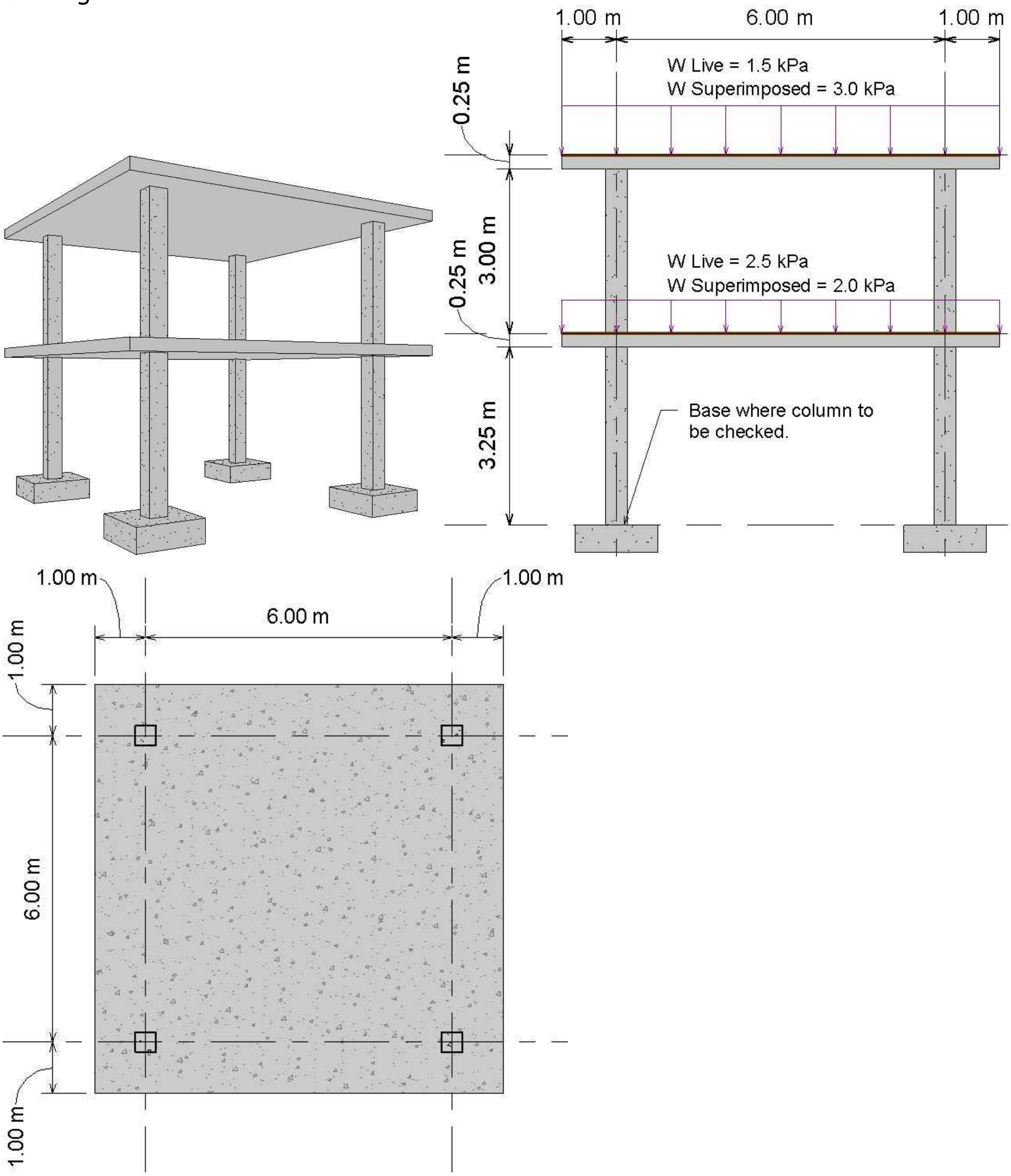
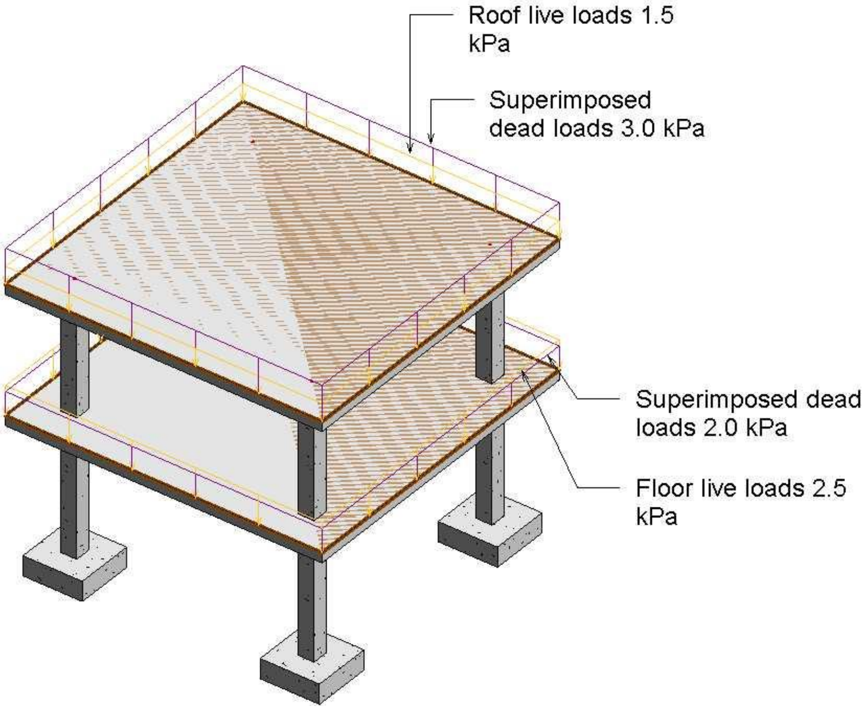


Figure 1.6-11: Frame for Example 1.6-5.

Solution

In 3D, loads acting on slab are as indicated in below:



Resultants for selfweight and superimposed dead loads are:

$R_{Slabs} = 2 \times (8 \times 8 \times 0.25 \times 24) = 768\ kN$

$R_{Columns} = \left((0.4^2 \times (3.25 + 3.00)) \times 24\right) \times 4 = 96\ kN$

$R_{Superimposed} = (3.0 + 2.0) \times 8^2 = 320\ kN$

$R_{Dead} = 768 + 96 + 320 = 1184\ kN$

As the columns are distributed in a symmetrical form, therefore share for each column would be:

$$P_{Dead} = \frac{1184}{4} = 296\ kN$$

In a similar approach column share due to live load would be:

$$P_{Live} = \left((1.5 + 2.5) \times 8^2\right) \times \frac{1}{4} = 64\ kN$$

The factored load, P_u, would be:

$P_u = maximum(1.4 \times 296\ or\ 1.2 \times 296 + 1.6 \times 64)$

$P_u = maximum(414.4\ or\ 457.6) \approx 458\ kN$

$P_u = 458\ kN < 0.65 \times 2800 = 1820\ kN \therefore Ok$

Example 1.6-6

Resolve Example 1.6-5 above, with considering the difference between floor live load, L, and roof live load, L_r, in your solution.

Solution

From previous solution:

$P_D = 296\ kN$

The axial force due to floor live is:

$$P_L = (2.5 \times 8^2) \times \frac{1}{4} = 40\ kN$$

While the axial force due to roof live load is:

$$P_{Lr} = (1.5 \times 8^2) \times \frac{1}{4} = 24\ kN$$

According Table 1.6-1 above, following load combinations should be considered:

$P_{u1} = 1.4P_D = 1.4 \times 296 = 414\ kN$

$P_{u2} = 1.2P_D + 1.6P_L + 0.5P_{Lr} = 1.2 \times 296 + 1.6 \times 40 + 0.5 \times 24 = 431\ kN$

$P_{u3} = 1.2P_D + 1.0P_L + 1.6P_{Lr} = 1.2 \times 296 + 1.0 \times 40 + 1.6 \times 24 = 434\ kN\ Govern$

$P_u = 434\ kN < 0.65 \times 2800 = 1820\ kN \therefore Ok$

Example 1.6-7

For the one-way structural system of the small apartment building indicated ***Figure 1.3-12***:

- Select an appropriate value for roof live load according to ASCE/SEI 7-10.
- Reduce the selected roof live load, if possible, to determine its resultant on a typical interior beam.
- Determined the floor live load according to ASCE/SEI 7-10. Assume that most of the floor area is for private rooms of more than two families.
- Reduce the selected floor live load, if possible, to determine its resultant on a typical interior beam.
- If a typical interior beam at the floor level is subjected to a dead load (selfweight plus superimposed) of 30kN/m determine the factored negative moment for the cantilever part of the beam.

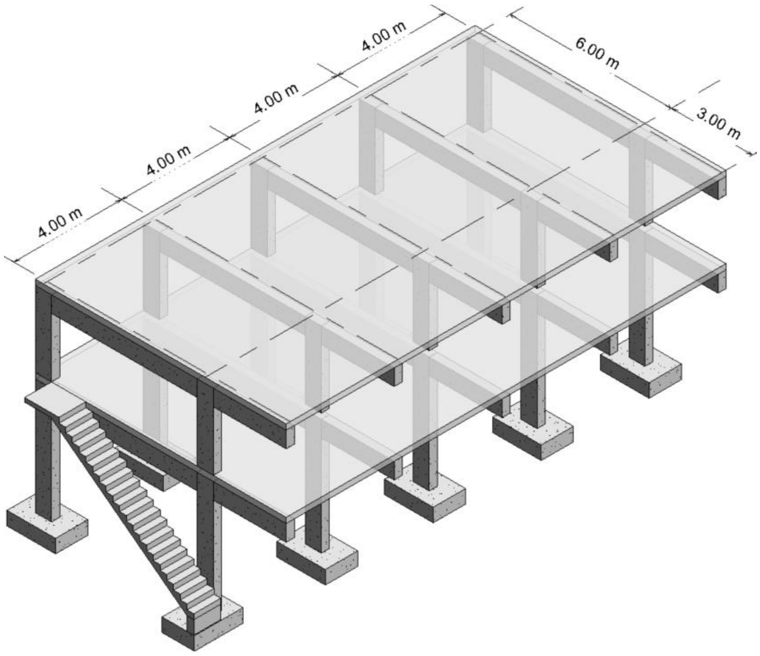


Figure 1.6-12: Structural system for a small apartment building.

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:
  The tributary area for a typical interior roof of floor beam is indicated in ***Figure 1.6-13***.
  $A_{T\,typical\,edge\,column} = 4 \times 9 = 36\,m^2 \Rightarrow$
  $\because\ 18.58\,m^2 < A_T < 55.74\,m^2$
  $\therefore R_1 = 1.2 - 0.011A_T = 1.2 - 0.011 \times 36$
  $= 0.804$

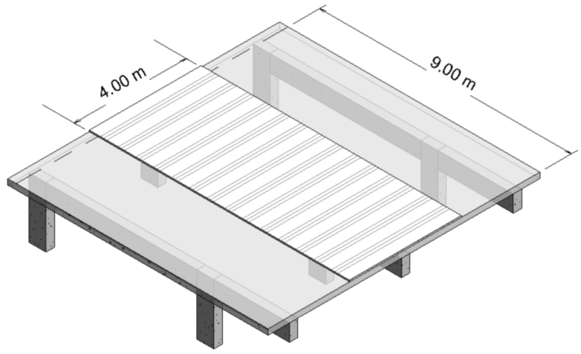


Figure 1.6-13: Tributary area for a typical interior beam.

  As the roof is flat, therefore the reduction factor R_2 is 1.0.
  $L_r = L_o R_1 R_2 = 0.96 \times 0.804 \times 1.0 \approx 0.772\,kPa > 0.58\,kPa \therefore Ok.$ ■
  Load share of the roof live load that is supported by a typical interior beam is:
  $W_{Lr} = L_r \times Spacing\ between\ the\ Beams = 0.772 \times 4.0 = 3.01\frac{kN}{m}$ ■
- Floor live load:
  The building is for residential purpose with floor area mostly for private rooms of more than two families. Therefore, the floor live load according to ASCE/SEI 7-10 is:
  $L = 1.92\,kPa$ ■
- Reduction of floor live load:
  As the live load is smaller than $4.79\,m^2$ and as the influence area is:
  $K_{LL}A_T = 2 \times (4 \times 9) = 72\,m^2 > 37.16\,m^2$
  therefore, the floor live load is reducible according to ASCE/SEI 7-10.
  $L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right) = L_o\left(0.25 + \frac{4.57}{\sqrt{72}}\right) = 0.788L_o > 0.5L_o \therefore Ok.$
  $L = 0.788L_o = 0.788 \times 1.92 = 1.51\,kPa$ ■
  Load share of the floor live load that is supported by a typical interior beam is:
  $W_L = L \times Spacing\ between\ the\ Beams = 1.51 \times 4.0 \approx 6.00\frac{kN}{m}$ ■

- Load Combinations

  For a typical beam at the floor level, the following load combinations have to be considered:

  $$W_u = \max(1.4W_D, 1.2W_D + 1.6W_L) = \max(1.4 \times 30, 1.2 \times 30 + 1.6 \times 6.00) = 45.6\frac{kN}{m}$$

  As nothing is mentioned about column dimensions, therefore the negative moment for cantilever will be conservatively determined at the center of the column:

  $$M_u = \frac{W_u \ell^2}{2} = \frac{45.6 \times 3.00^2}{2} = 205\ kN.m\ ■$$

1.7 STRENGTH (LRFD) VERSUS WORKING-STRESS DESIGN METHODS

As an alternative to the ***Strength Design Method***, members may be proportioned based on the ***Working-Stress Design Method***, where the stresses in the steel and concrete resulting from normal service loads (unfactored loads) should be within a specified limit known as the ***Allowable Stresses***.
Allowable Stresses, in practice, are set at about one-half the concrete compressive strength and one-half the yield stress of steel.
The following Table summarized the main differences between the Strength Design Method and Working-Stress Design Method.

Strength Design Method	**Working-Stress Design Method**
1. Individual load factors may be adjusted to represent different degree of certainty for the various types of loads, and reduction factors likewise may be adjusted to precision with which various types of strength are calculated.	1. All types of loads are treated the same no matter how different in their individual uncertainty.
2. Strength is calculated with explicit regards for inelastic action.	2. Stresses are calculated on the elastic basis.
3. Serviceability with respect to deflection and cracking is considered explicitly.	3. Serviceability with respect to deflection and cracking is considered only implicitly through limits on service loads stress.

Because of these differences, the Strength Design Method has largely displaced the older Allowable Stresses Design Method. *Prior to 2002, Appendix A of the ACI Code allowed design of concrete structures either by Strength Design Method or by Working-Stress Design Method. In 2002, this appendix was deleted.*

1.8 FUNDAMENTAL ASSUMPTIONS FOR REINFORCED CONCRETE BEHAVIOR

As was discussed in the previous article, the uncertainties in the design process have been treated based on the *Strength Reduction Factor* φ and *Load Factors* γ. Therefore, remaining of the design process is based mainly on the *Structural Mechanics* to dealing with the following ***deterministic*** aspects:

1. Compute the theoretical stresses and internal forces (shear, moment, torsion, and axial forces).
2. Compute the theoretical or nominal strength (e.g. M_n, V_n, and P_n).
3. Compute the theoretical deformations and deflections.

The fundamental assumptions on which the *Mechanics of Reinforced Concrete* is based can be summarized as follows:

1.8.1 Assumptions that Related to Equilibrium

All reactions, internal forces, internal stresses and deformations satisfy the equations of equilibrium.

1.8.2 Assumptions that Related to the Compatibility

1. Prefect bonding:
   $\epsilon_{Steel} = \epsilon_{of\ Surrounding\ Concrete}$
2. Cross section, which was plane prior to loading, continues to be plane after loading.

1.8.3 Assumptions that Related to the Constitutive Law

1. Neglecting the tensile strength of concrete.
2. Theory is based on the actual stress-strain relationships and strength properties of the two constituent materials.

1.9 SYLLABUS

1. Based on previous discussions, one can consider any structure like a chain that consists of many rings. Each ring receives loads from previous rings and submits it to the next ones.
2. Syllabus for this course is presented in terms of schematic chain indicated in ***Figure 1.9-1*** below.

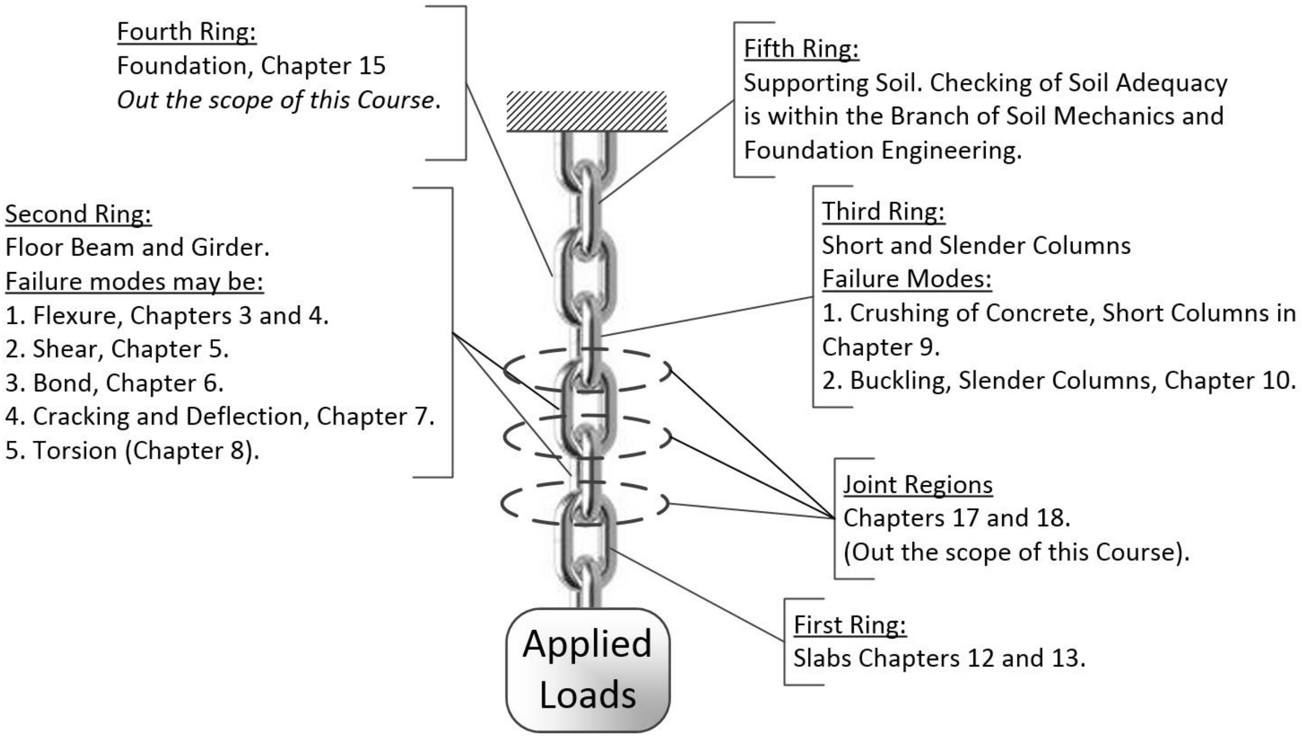


Figure 1.9-1: The chain concept for presenting the syllabus of the course.

1.10 SI UNITS

1.10.1 Metric System in North America

- The American Concrete Institute adopted an SI version of the 1983 ACI Code and continues with the current 2014 SI version.
- In Canada, formal conversion to SI was accomplished with the adoption of the 1977 Canadian Concrete Code.

1.10.2 Different Types of Metric System

- The metric system, although not SI, is used in ***nearly all western hemisphere countries*** (other than the USA and Canada).
- In these countries, the ***MKS*** (meter-kilogram-second) system is used, where instead of the kilogram (kg) as a unit of mass, as in SI, the kilogram (***kgf***) is used as a unit of force.

1.10.3 Soft Metric

- A "***soft conversion***" involves changing a measurement from inch-pound units to equivalent metric units within what the DoD calls "***acceptable measurement tolerances***."
- This is done to ***merely convert the imperial measurements to metric without physically changing the item*** and it is typically used to specify a requirement.
- For example, a ½-in. rebar diameter would be converted to either 12.7 or 13 mm using soft conversion. Although this is not a standard metric rebar size, it expresses the requirement.

1.10.4 Hard Metric

- A "***hard conversion***" involves a change in measurement units that results in a "***physical configuration change***."
- Using the rebar example, this would be ***analogous to changing the diameter of the rebar from ½ in. to an M12 (12-mm) or M14 (14-mm) rebar diameter***.
- Either one of the two new metric rebar diameters would be outside an "***acceptable measurement tolerance***". The new rebar would be considered a "***hard metric***" item.
- This is size substitution, which is one method of using hard conversion; the other method is adaptive conversion, where imperial and metric units are reasonably equivalent, but not exact conversions of each other.

1.11 GENERAL PROBLEMS

Problem 1.11-1

Check adequacy of the beam shown in Figure 1.11-1 below for bending and shear according to the requirements of ACI 318M-14. Assume that $M_n = 400kN.m$ and $V_n = 280kN$. Beam selfweight is not included in the dead load shown. Assume $\phi_{flexure} = 0.9$.

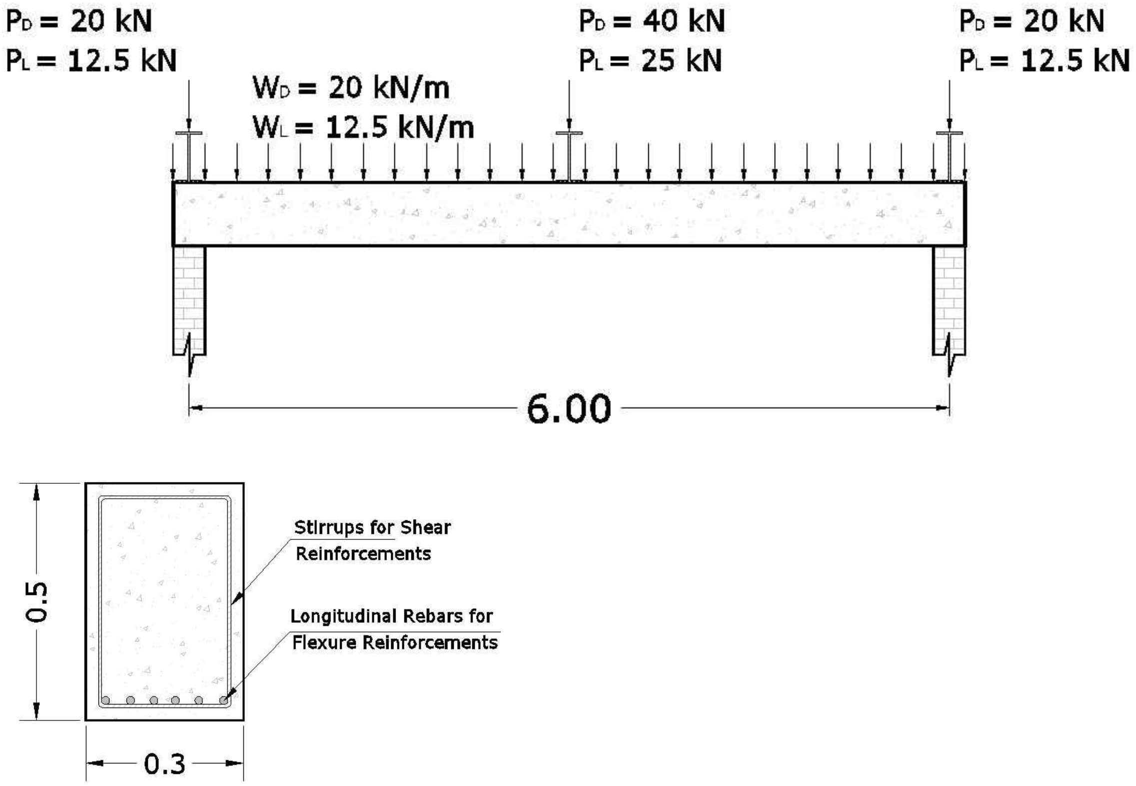


Figure 1.11-1: Beam for Problem 1.11-1.

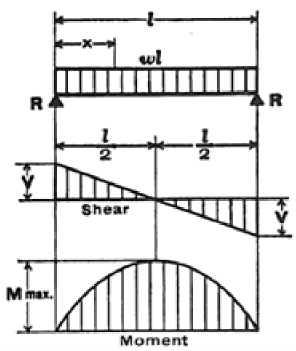
Notes:

1. It is useful to notice that the concentrated loads on girders in civil engineering applications usually resulted from reactions of the floor beams that extend normal to the girder plane, see Figure 1.2-2***.***

2. Designers usually compute the required design internal forces (bending moment, shear force, and axial force) based on the principle of superposition (i.e. compute the effect of each load separately then sum the effects of all loads to obtain the required result), instead of drawing of shear force, bending moment, and axial force diagrams assuming that all loads acting simultaneously.

Design handbooks usually contain shear force, bending moment, and axial force diagrams for beams and simple frames and for typical load conditions. Figures below show the beam diagrams for common load cases in engineering practice:

SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . $= wl$

R = V . . . . $= \frac{wl}{2}$

V_x . . . . $= w\left(\frac{l}{2} - x\right)$

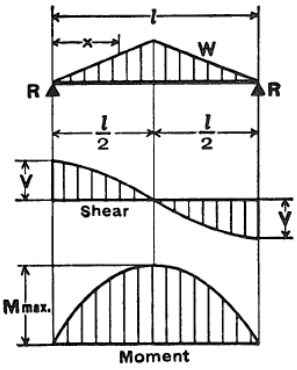
M max. (at center) . . . . $= \frac{wl^2}{8}$

M_x . . . . $= \frac{wx}{2}(l - x)$

Δmax. (at center) . . . . $= \frac{5wl^4}{384EI}$

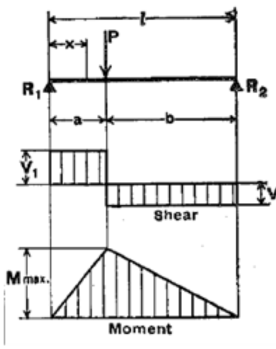
Δx . . . . $= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO CENTER



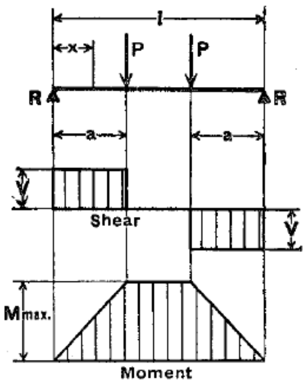
Total Equiv. Uniform Load		$= \frac{4W}{3}$
$R = V$		$= \frac{W}{2}$
V_x	$\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{W}{2l^2}(l^2 - 4x^2)$
M max.	(at center)	$= \frac{Wl}{6}$
M_x	$\left(\text{when } x < \frac{l}{2}\right)$	$= Wx\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right)$
Δmax.	(at center)	$= \frac{Wl^3}{60EI}$
Δx	$\left(\text{when } x < \frac{l}{2}\right)$	$= \frac{Wx}{480\,EI\,l^2}(5l^2 - 4x^2)^2$

SIMPLE BEAM—CONCENTRATED LOAD AT ANY POINT



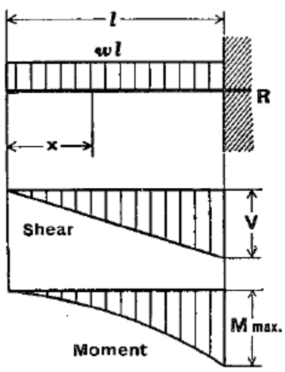
Total Equiv. Uniform Load		$= \frac{8\,Pab}{l^2}$
$R_1 = V_1$	(max. when $a < b$)	$= \frac{Pb}{l}$
$R_2 = V_2$	(max. when $a > b$)	$= \frac{Pa}{l}$
M max.	(at point of load)	$= \frac{Pab}{l}$
M_x	(when $x < a$)	$= \frac{Pbx}{l}$
Δmax.	$\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right)$	$= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27\,EI\,l}$
Δa	(at point of load)	$= \frac{Pa^2b^2}{3EI\,l}$
Δx	(when $x < a$)	$= \frac{Pbx}{6EI\,l}(l^2 - b^2 - x^2)$

SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



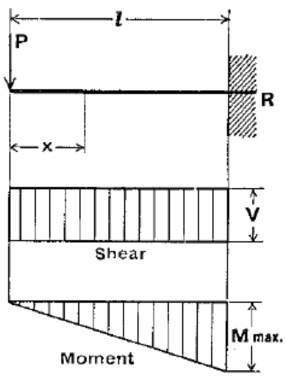
Total Equiv. Uniform Load		$= \frac{8\,Pa}{l}$
$R = V$		$= P$
M max.	(between loads)	$= Pa$
M_x	(when $x < a$)	$= Px$
Δmax.	(at center)	$= \frac{Pa}{24EI}(3l^2 - 4a^2)$
Δx	(when $x < a$)	$= \frac{Px}{6EI}(3la - 3a^2 - x^2)$
Δx	(when $x > a$ and $< (l - a)$)	$= \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$

CANTILEVER BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load		$= 4wl$
$R = V$		$= wl$
V_x		$= wx$
M max.	(at fixed end)	$= \frac{wl^2}{2}$
M_x		$= \frac{wx^2}{2}$
Δmax.	(at free end)	$= \frac{wl^4}{8EI}$
Δx		$= \frac{w}{24EI}(x^4 - 4l^3x + 3l^4)$

CANTILEVER BEAM—CONCENTRATED LOAD AT FREE END



Total Equiv. Uniform Load . . . . . = 8P

R = V . . . . = P

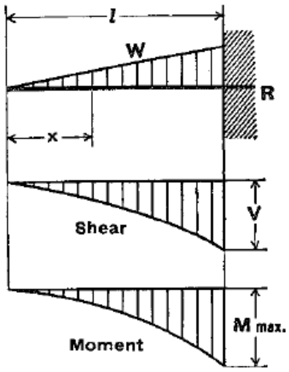
M max. (at fixed end) . . . . . . = Pl

M_x . . . . = Px

Δmax. (at free end) . . . . . $= \dfrac{Pl^3}{3EI}$

Δ_x . . . . $= \dfrac{P}{6EI}(2l^3 - 3l^2x + x^3)$

CANTILEVER BEAM—LOAD INCREASING UNIFORMLY TO FIXED END



Total Equiv. Uniform Load . . . . . $= \dfrac{8}{3}W$

R = V . . . . = W

V_x . . . . $= W\dfrac{x^2}{l^2}$

M max. (at fixed end) . . . . . . $= \dfrac{Wl}{3}$

M_x . . . . $= \dfrac{Wx^3}{3l^2}$

Δmax. (at free end) . . . . . . $= \dfrac{Wl^3}{15EI}$

Δ_x . . . . $= \dfrac{W}{60EIl^2}(x^5 - 5l^4x + 4l^5)$

3. ***Point loads act directly above supports, do not produce shear force nor bending moment in the beam and should be considered only in the design of supports.***

ANSWERS

$M_{u\ \text{due to 1.2DL + 1.6LL}} = 349\ \text{kN.m},\ V_{\text{due to 1.2DL + 1.6LL}} = 189\ \text{kN}$
Section is adequate for flexure and for shear.

Problem 1.11-2

Use bending moment diagrams presented in Problem 1.11-1 above, to compute the bending moment at centerline of the beam presented in Figure 1.11-2 below.

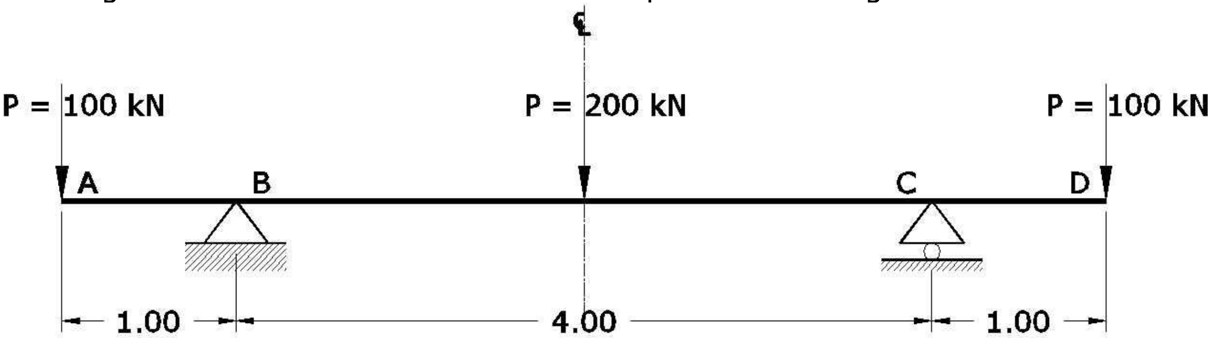
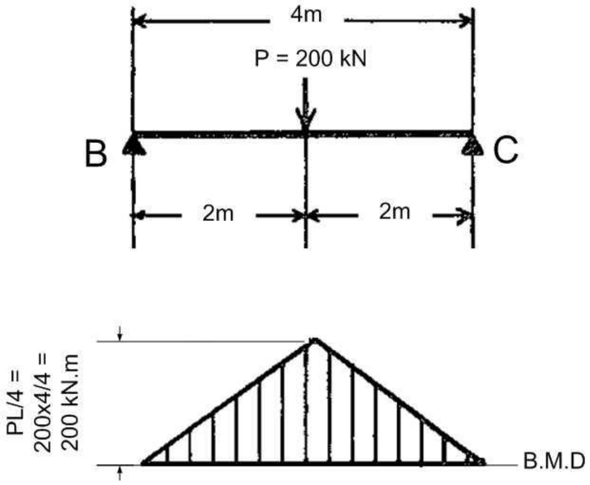


Figure 1.11-2: Overhang beam for Problem 1.11-2.

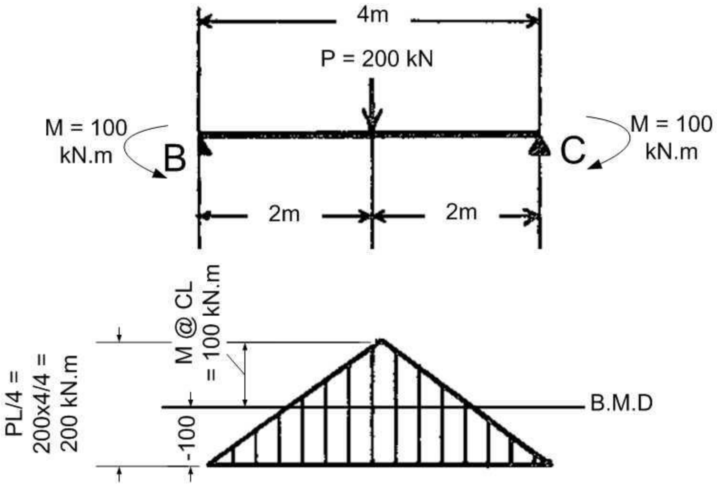
SOLUTION

This problem aims to show that documented diagrams that had been prepared to simple span can be used for problems with overhangs beams and continuous beams. This can be explained as follows:

1. ***Assume that the span BC is simple span, then the bending moment at beam centerline is:***



2. ***It is useful to note that the effects of end moments on moment diagram is that the moment diagram is shifted below for negative end moments and shifted above for positive end moment. For our case with negative end moments of (*$100\text{kN} \times 1.0\text{m} = 100\ \text{kN.m}$*), the bending moment diagram will be shifted with 100 kN.m in downward direction.***



Problem 1.11-3

Check the adequacy of the beam shown in Figure 1.11-3 below for bending and shear according to the requirements of ACI 318M-14. Assume that $M_n = 650kN.m$ and $V_n = 450kN.m$. Beam selfweight is not included in the dead load shown. Assume $\phi_{flexure} = 0.9$.

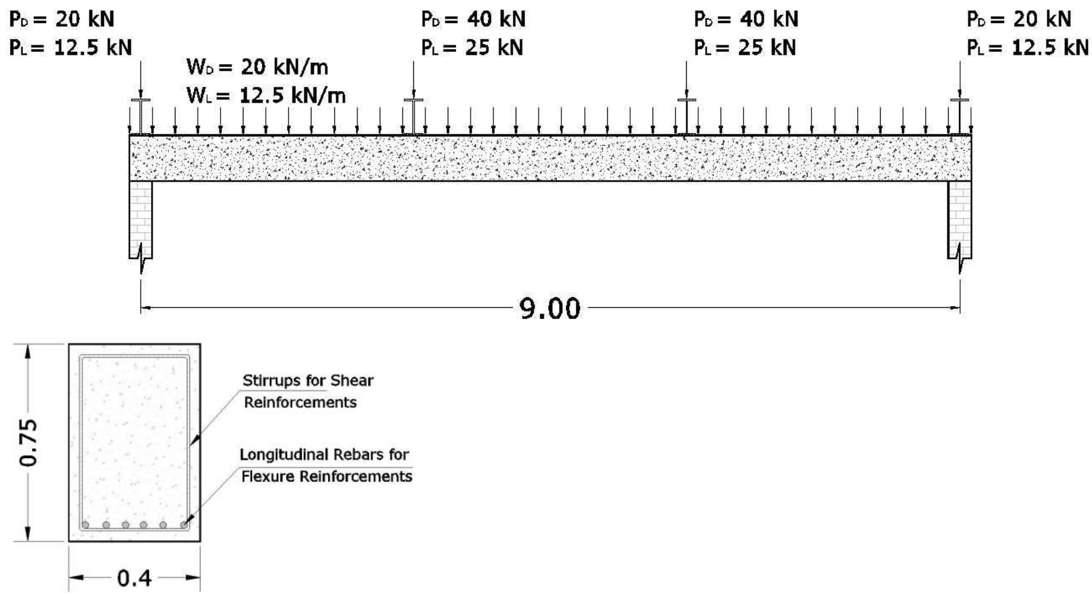


Figure 1.11-3: Simply supported beam for Problem 1.11-3**.**

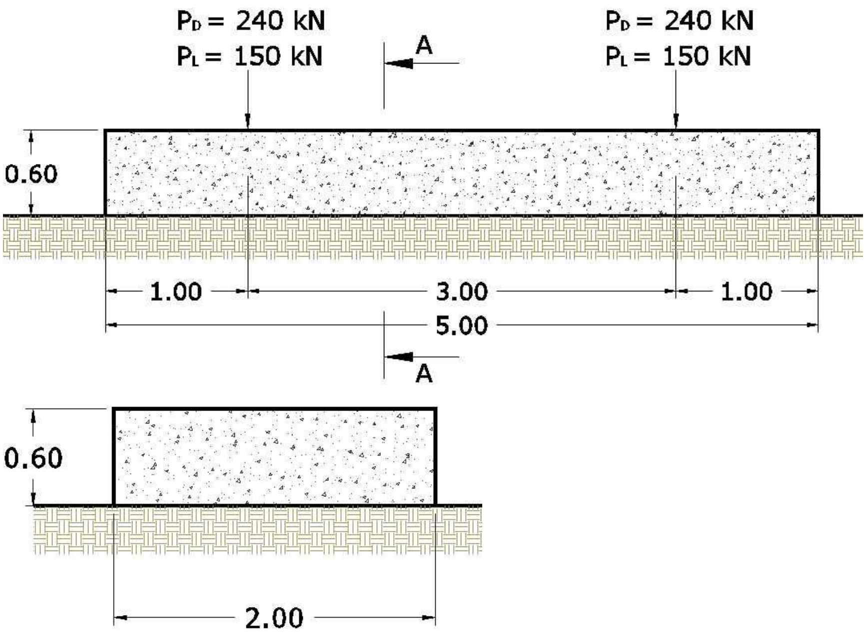
ANSWERS

$\mathrm{M_{u\ due\ to\ 1.2DL\ +\ 1.6LL}} = 797\ \mathrm{kN.m}$, $\mathrm{V_{due\ to\ 1.2DL\ +\ 1.6LL}} = 325\ \mathrm{kN}$

Section is inadequate for flexure and adequate for shear.

Problem 1.11-4

Check the adequacy of the foundation shown in Figure 1.11-4 below for bending and shear according to the requirements of ACI 318M-14. Assume that $M_n = 200kN.m$ and $V_n = 400kN.m$. Beam selfweight is not included in the dead load shown. Assume $\phi_{flexure} = 0.9$.



Section A-A

Figure 1.11-4: Combined footing for Problem 1.11-4**.**

ANSWERS

$\mathrm{M_{u\ due\ to\ 1.2DL\ +\ 1.6LL}} = 132\ \mathrm{kN.m}$, $\mathrm{V_{due\ to\ 1.2DL\ +\ 1.6LL}} = 317\ \mathrm{kN}$

Section is adequate for flexure and in adequate for shear.

Problem 1.11-5

For building presented in ***Figure 1.11-5*** below, check adequacy of column (C3) for axial load condition according to the requirements of ACI 318M-14. Assume that:

- Superimposed dead load is $4.0\frac{kN}{m^2}$.
- Live Load is $3\frac{kN}{m^2}$, roof live load is $1\frac{kN}{m^2}$.
- Slab thickness is $200mm$.
- All columns are $400mm \times 400mm$.
- $P_n = 2\ 800kN$.
- Strength reduction factor for column is $\emptyset = 0.65$.
- Your checking must be based on following load conditions:
  - (1.4D), (1.2D + 1.6L+0.5L_r),
  - (1.2D + 1.0L+1.6L_r).

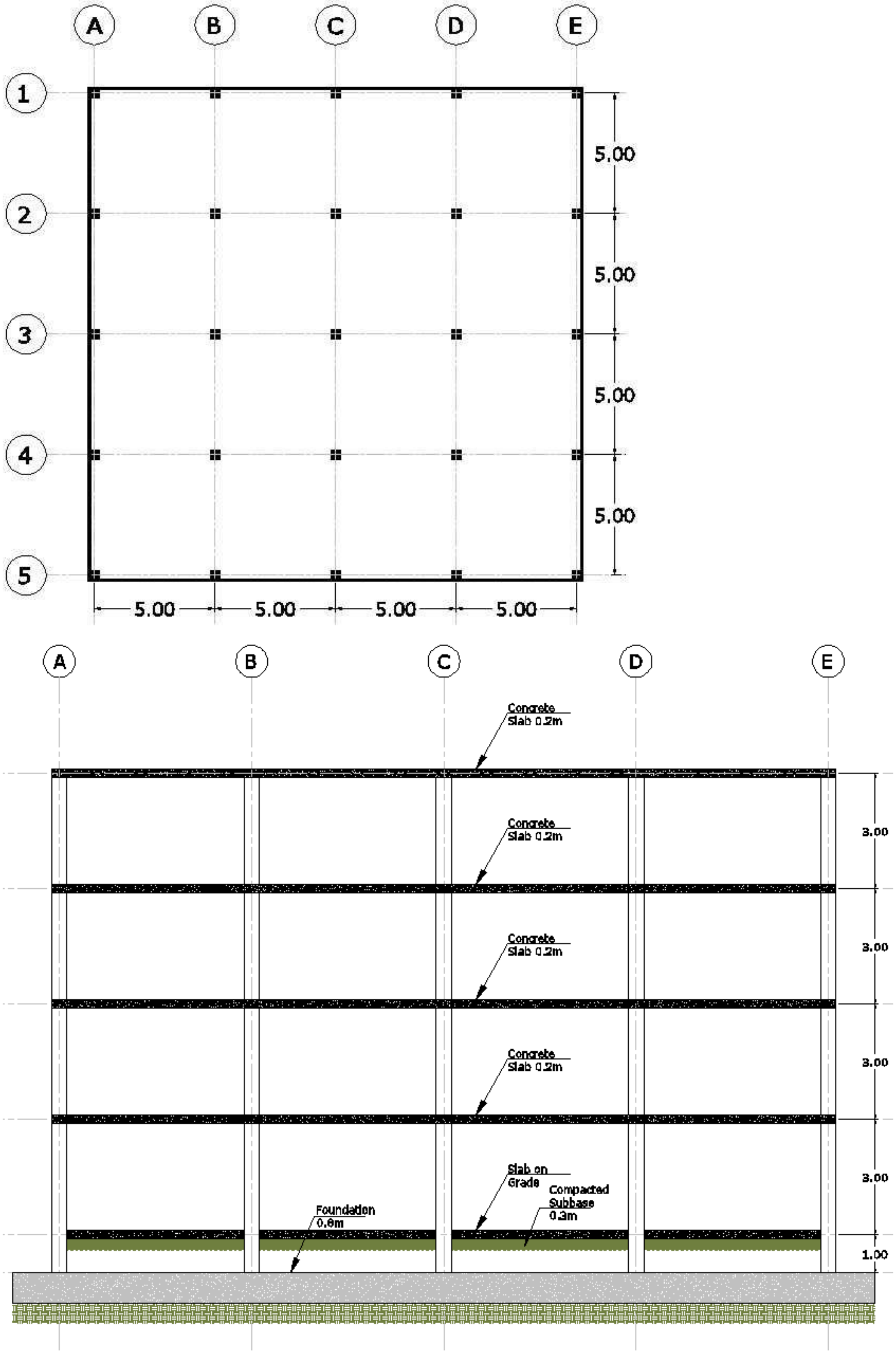


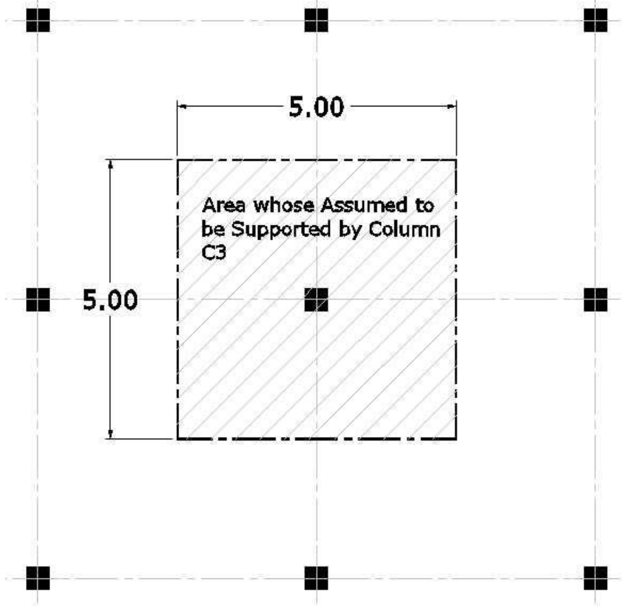
Figure 1.11-5: Building for Problem 1.11-5**.**

Note:

Ground floor is either supported on columns as presented in Example 1.3-1 *or it is supported directly on the underneath soil as assumed in this problem. When supported directly on soil, the slab is usually called as a slab on grade.*

SOLUTION

1. Compute of Basic Load Cases:
   As discussed in Example 1.3-1, it is a common in engineering practice to assume that interior columns are subjected to axial forces only. These forces are computed based on the assumption that an interior column is responsible on supporting an area bounded by centerlines of the four adjacent panels.



$$P_D = \left[\left(0.2m \times 24\frac{kN}{m^3} + 4.0\right)\frac{kN}{m^2} \times 25m^2\right]_{Slab\ Weight} \times 4_{No.of\ Slabs}$$
$$+(0.4^2 m^2 \times 13m)_{Volume\ of\ the\ Column} \times 24\frac{kN}{m^3}$$
$$P_D = 880\ kN + 49.9\ kN = 930\ kN$$
$$P_L = \left(3\frac{kN}{m^2} \times 25\ m^2\right) \times 3_{No.of\ Floors} = 225\ kN$$
$$P_{Lr} = \left(1.0\frac{kN}{m^2} \times 25\ m^2\right) \times 1_{No.of\ Roof} = 25\ kN$$

2. Compute Required Load Combinations:
   $P_{u\ due\ to\ 1.4D} = 1\,302\ kN$
   $P_{u\ due\ to\ 1.2D\ +\ 1.6L\ +\ 0.5\ Lr} = 1\,489\ kN$
   $P_{u\ due\ to\ 1.2D\ +\ 1.0L\ +\ 1.6Lr} = 1\,381\ kN$
   Then the maximum factored load is 1 489 kN due to 1.2D + 1.6L + 0.5L_r.
3. Column Check:
   $\emptyset P_n = 0.65 \times 2\,800\ kN = 1\,820\ kN\ \ ?\ P_{u\ due\ to\ 1.2D\ +\ 1.6L\ +\ 0.5\ Lr} = 1\,489\ kN$
   $\emptyset P_n = 1\,820\ kN\ \ >\ P_{u\ due\ to\ 1.2D\ +\ 1.6L\ +\ 0.5\ Lr} = 1\,489\ kN\ \ OK.$

Problem 1.11-6

For the elevated reinforced concrete water tank shown Figure 1.11-6 below:

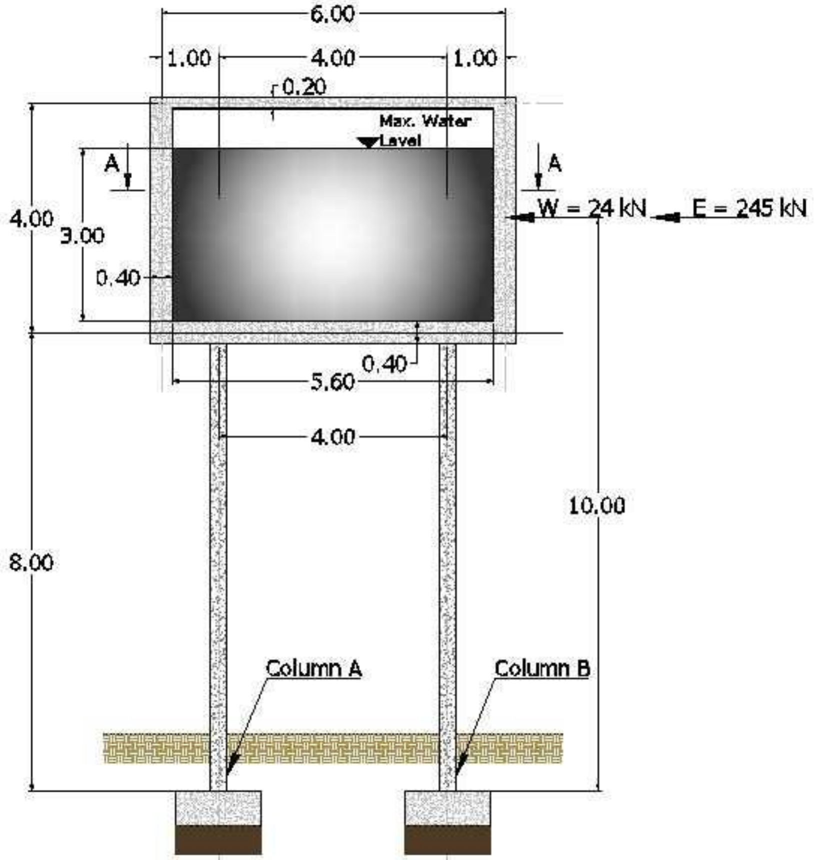
- Check the adequacy of ***Column A*** for axial load condition according to the ***strength requirements*** of ACI 318M-14. Base your strength checking on the following load cases:
  1.4 (D+F), (1.2D + 1.6W), and (1.2D + 1.0E).
- Check the adequacy of column B according to the ***stability requirements***[1] of ACI 318M-14. Base your stability checking on the following load cases:

[1] As most of foundation systems in civil engineering applications are incapable to resist a direct tensile force, then one of the most critical checking is to check that the foundation is not under direct tension. The load combinations with 0.9D are specified for the case where dead load reduce the direct tension effects due to other load conditions (e. g. the direct tension effects of wind and seismic forces on columns B in our Example).

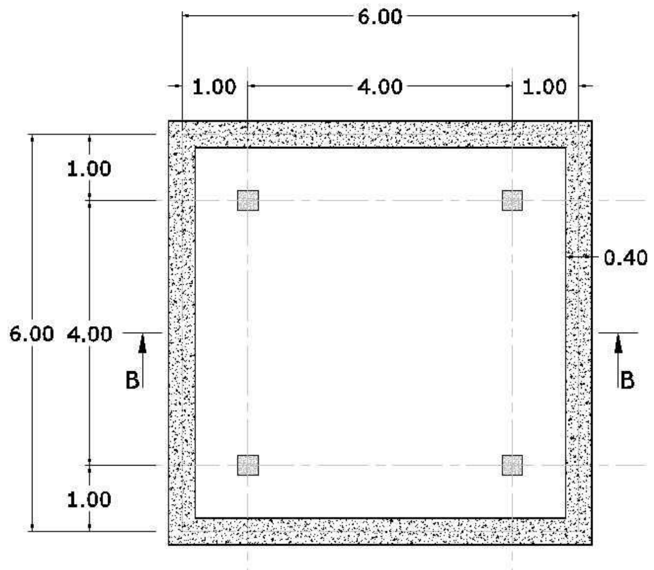
(0.9D + 1.6W), and (0.9D + 1.0E).

In your solution, assume that:

- All columns are $300mm \times 300mm$,
- $P_n = 1\ 700kN$,
- Strength reduction for column is $\emptyset = 0.65$.



Section B-B



Section A-A

Figure 1.11-6: Elevated tank for Problem 1.11-6.

Notes:

One may note that load factor for wind forces adopted in this problem differs from that adopted in Article 1.6.1.1. This difference can be explained as follows, in American codes before (ASCE/SEI 7–10), wind induced forces were having an explicit load factor of 1.6 to simulate uncertainty in wind induced forces. In (ASCE/SEI 7–10), this load factor has been included implicitly through modifying maps for wind speed. Therefore, for wind maps defined by other agencies, the load factor of 1.6 should be included explicitly in definition of wind-induced forces.

ANSWERS

- BASIC LOAD CONDITIONS:
  $\mathrm{P}_{@\text{ Column A \& B due to D}} = 377\text{ kN}$, $\mathrm{P}_{@\text{ Column A \& B due to F}} = 235\text{ kN}$,
  $\mathrm{P}_{@\text{ Column A due to W}} = 30\text{ kN}$, $\mathrm{P}_{@\text{ Column B due to W}} = -30\text{ kN}$, $\mathrm{P}_{@\text{ Column A due to E}} = 306\text{ kN}$, and
  $\mathrm{P}_{@\text{ Column B due to E}} = -306\text{ kN}$.
- Checking Strength for Column A:
  $P_{u\ due\ to\ 1.4\ (D+F)} = 857\ kN$
  $P_{u\ due\ to\ (1.2\mathrm{D}\ +\ 1.6\mathrm{W})} = 500\ kN$
  $P_{u\ due\ to\ (1.2\mathrm{D}\ +\ 1.0\mathrm{E})} = 758\ kN$
  $\therefore P_{u\ Maximum} = 857\ kN < \emptyset P_n = 0.65 \times 1\,700\ kN = 1\,105 kN \quad \therefore Ok.$
- Checking Stability for Column B:
  $P_{u\ due\ to\ (0.9\mathrm{D}\ +\ 1.6\mathrm{W})} = 291\ kN$
  $P_{u\ due\ to\ (0.9\mathrm{D}\ +\ 1.0\mathrm{E})} = 33.3\ kN$
  $\because P_{u\ Minimum} = 33.3\ kN > 0, \therefore Column\ B\ is\ Stable.$

1.12 ADDITIONAL EXAMPLES

Additional Example 1.12-1

Check the adequacy of the reinforced concrete column of the frame shown in Figure 1.12-1 below according to the requirements of ACI Code. In your checking, consider the following load cases:

$U = 1.4D$

$U = 1.2D + 1.6L$

Assume that the column has dimensions of 450mm x 450mm and has a nominal strength of $P_n = 3\,500\,kN$, and a strength reduction factor of 0.65.

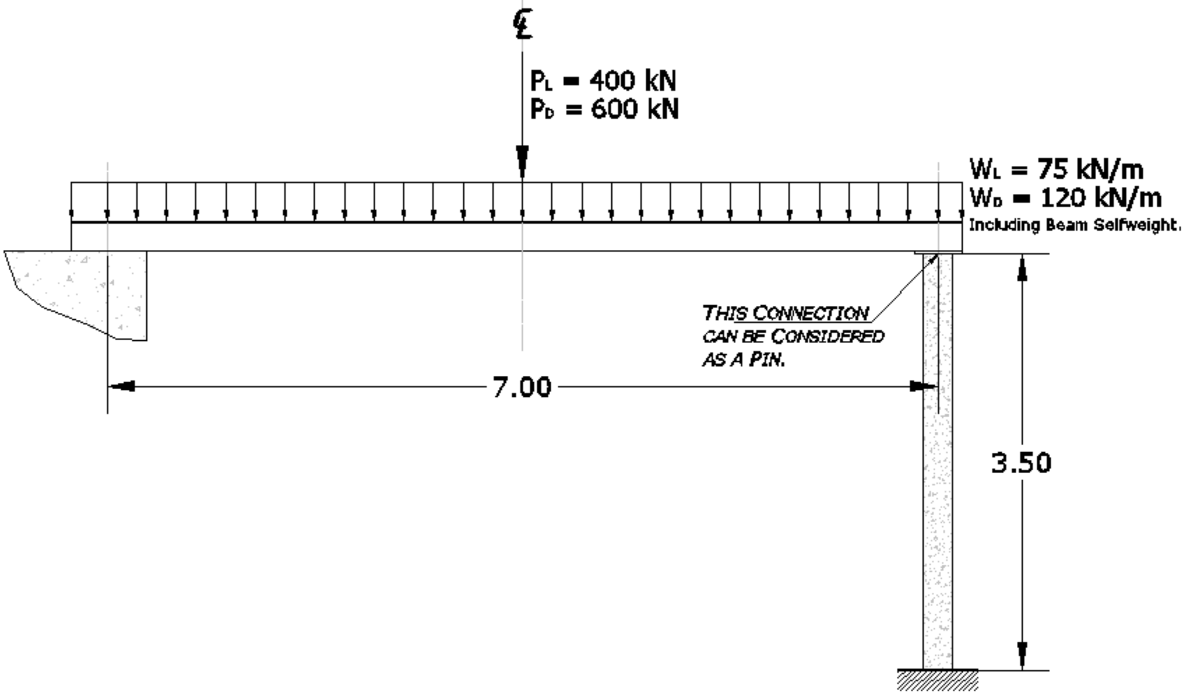


Figure 1.12-1 Frame for Additional Example 1.12-1.

Solution

1. Basic Load Cases:
    a. Compute $P_{Due\ to\ Dead\ Loads}$:

$$P_{Due\ to\ Dead\ Loads} = \frac{\left(120\frac{kN}{m} \times 7.0m\right)}{2} + \frac{600}{2} + (0.45^2 \times 3.5 \times 24) = 737\ kN$$

    b. Compute $P_{Due\ to\ Roof\ Live\ Load}$:

$$P_{Due\ to\ Roof\ Live\ Load} = \frac{75\frac{kN}{m} \times 7.0m}{2} + \frac{400}{2} = 462\ kN$$

2. Load Combinations:

$P_{u\ due\ to\ (1.4D)} = 1.4 \times 737\ kN = 1\,032\ kN$

$P_{u\ due\ to\ [1.2D\ +\ 1.6(L_r or\ S\ or\ R) +\ (1.0L\ or\ 0.8W)]} = 1.2 \times 737 + 1.6 \times 462 = 1\,624\ kN$

3. Check Columns Adequacy:

$P_u\ ?\ \emptyset P_n$

$P_u = 1\,624\ kN < \emptyset P_n = 0.65\ \times 3\,500\ kN = 2\,275\ kN\ \ Ok.$

Therefore, the columns are adequate according to strength requirements of ACI Code.

Additional Example 1.12-2

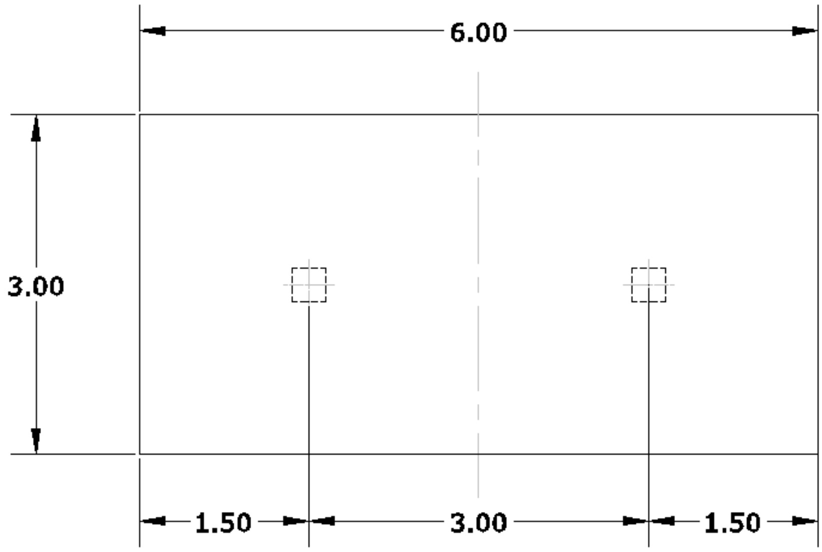
Check columns adequacy for the sunshade with plan and elevation shown in Figure 1.12-2 below according to requirements of ACI Code. Your checking must include following load combination:

$U = 1.4D$

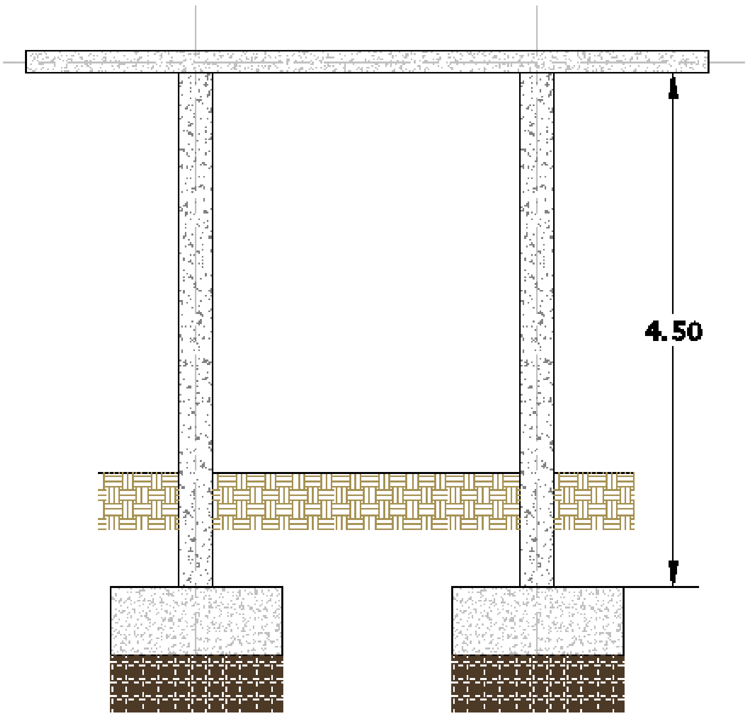
$U = 1.2D + 1.6(L_r or\ S\ or\ R) + (1.0L\ or\ 0.8W)$

Assume that the roof is subjected to a superimposed dead load of 2.0 kPa, roof live load of 1.0 kPa and snow load of 0.75 kPa. Also, assume that wind load effect can be neglected. Concrete slab has a thickness of 200mm.

Assume that all columns have dimensions of 250mm x 250mm and have a nominal strength of $P_n = 1100\ kN$, and a strength reduction factor of 0.65.



A Plan View



An Elevation View

Figure 1.12-2: Shade for Additional Example 1.12-2.

Solution

1. Basic Load Cases:
   a. Compute $P_{Due\ to\ Dead\ Loads}$:

$$P_{Due\ to\ Dead\ Loads} = \frac{\left(0.2m\ \times 24\frac{kN}{m^3} + 2.0\ kPa\right) \times 18m^2}{2} + (0.25^2 \times 4.5)m^3 \times 24\frac{kN}{m^3}$$

$$= 67.9\ kN$$

   b. Compute $P_{Due\ to\ Roof\ Live\ Load}$:

$$P_{Due\ to\ Roof\ Live\ Load} = \frac{1.0\ kPa\ \times 18\ m^2}{2} = 9.0\ kN$$

As snow load is less than roof live load, then it can be neglected in our checking.

2. Load Combinations:

$P_{u\ due\ to\ (1.4D)} = \ 1.4 \times 67.9\ kN = \ 95.1\ kN$

$P_{u\ due\ to\ [1.2D\ +\ 1.6(L_r or\ S\ or\ R) +\ (1.0L\ or\ 0.8W)]} = 1.2 \times 67.9 + 1.6 \times 9.0 = 95.9\ kN$

3. Check Columns Adequacy:

$P_u\ ?\ \emptyset P_n$

$P_u = \ 95.9\ kN < \ \emptyset P_n = 0.65\ \times 1100\ kN = 715\ kN\ \ Ok.$

Then the columns are adequate according to strength requirements of ACI Code.

Additional Example 1.12-3

Check the adequacy and stability for reinforced concrete columns of the high-elevated cylindrical tank shown in Figure 1.12-3 below according to the requirements of the ACI Code.

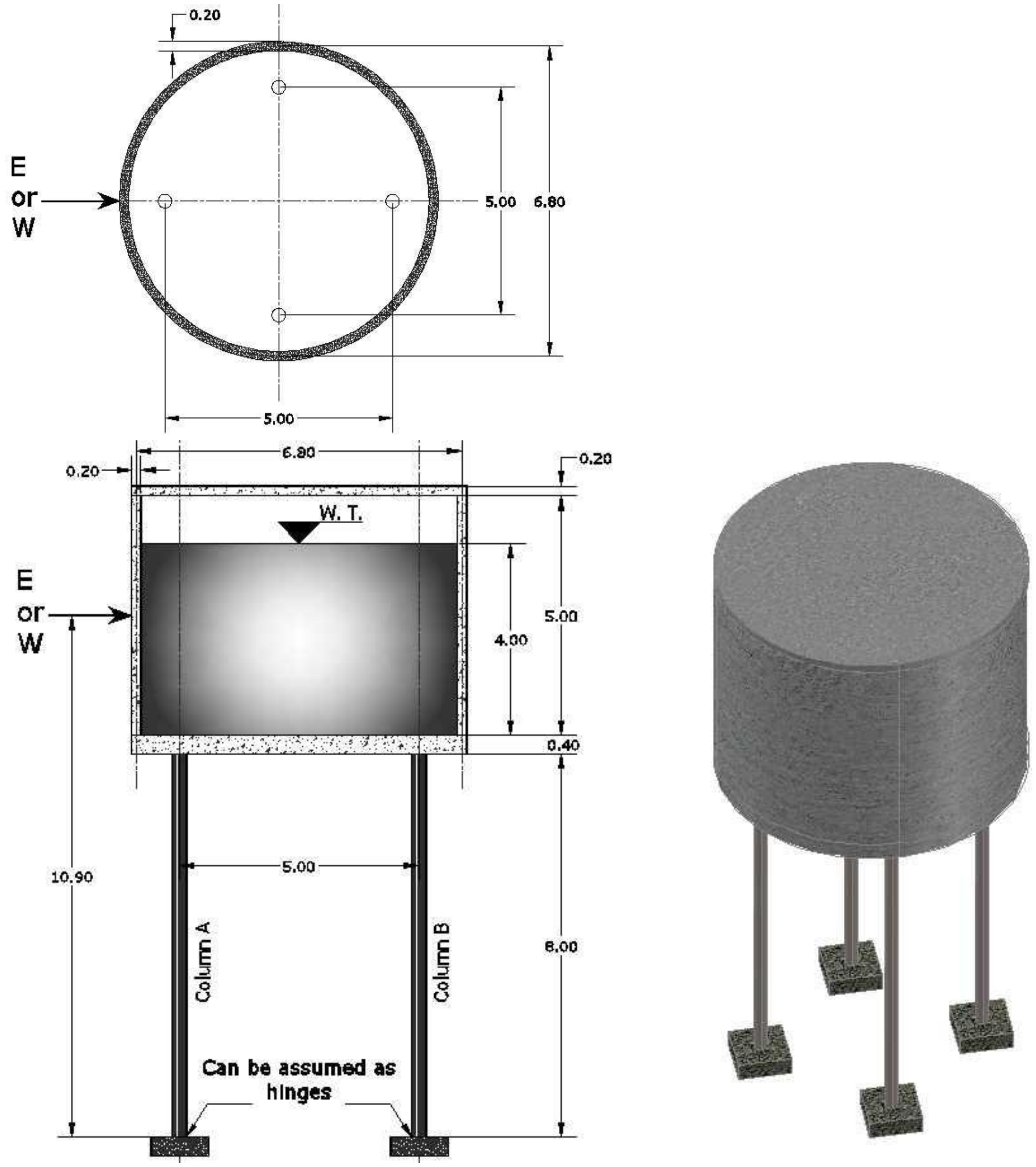


Figure 1.12-3: Elevated concrete tank for Additional Example 1.12-3.

In your solution, assume that:

$W\ = \ 34\ kN$

$E = 0.2D$

Also assume that the column has a diameter of 300mm and has a nominal strength of $P_n = 1\,979\ kN$, and a strength reduction factor of 0.75.

Solution

1. Basic Load Cases:

a. Dead Loads:

$$W_{DRoof} = \frac{(6.8+0.2)^2 \times \pi}{4} \times 0.2 \times 24$$

$$W_{DRoof} = \frac{294\,\pi}{5} = 185\ kN$$

$$W_{DWall} = (6.8 \times \pi \times 0.2 \times 5) \times 24$$

$$W_{DWall} = \frac{816\,\pi}{5} = 512\ kN$$

$$W_{DFloor} = \frac{(6.8+0.2)^2 \times \pi}{4} \times 0.4 \times 24$$

$$W_{DFloor} = \frac{588\,\pi}{5} = 369\ kN$$

$$W_{DColumns} = \left(\frac{0.3^2 \times \pi}{4} \times 8.0\right) \times 24 \times 4$$

$$W_{DColumns} = \frac{432\,\pi}{25} = 54\ kN$$

$$W_D = (185 + 512 + 369 + 54)$$

$$W_D = 1120\ kN$$

$$P_D = 1120 \times \frac{1}{4}$$

$$P_D = 280\ kN$$

b. Fluid Weight:

$$W_{Fluid} = \left(\frac{(6.8-0.2)^2 \times \pi}{4} \times 4\right) \times 10$$

$$W_{Fluid} = \frac{2178\,\pi}{5} = 1368\ kN$$

$$P_{Fluid} = \frac{1368}{4} = 342\ kN$$

c. Wind Loads:

$$P_{Wind} = \pm\left(34 \times \frac{10.9}{5}\right)$$

$$P_{Wind} = \pm\, 74\ kN$$

d. Seismic Loads:

$$P_E = (1120 \times 0.2) \times \frac{10.9}{5}$$

$$P_E = \pm\, 488\ \mathrm{kN}$$

2. Checking of Column Strength:

$$\mathrm{U = 1.4(D + F)}$$

$$\mathrm{P_{u1} = 1.4(280 + 342)}$$

$$\mathrm{P_{u1} = 871\ kN}$$

$$\mathrm{U = 1.2D + 1.6W}$$

$$\mathrm{P_{u2} = 1.2 \times 280 + 1.6 \times 74}$$

$$P_{u2} = \frac{2272}{5} = 454\ \mathrm{kN}$$

$$\mathrm{U = 1.2D + 1.0E}$$

$$\mathrm{P_{u3} = 1.2 \times 280 + 1.0 \times 488}$$

$$P_{u3} = 824\ kN$$

$$\mathrm{P_u = Maximum\ (871\ or\ 454\ or\ 824\ )}$$

$$\mathrm{P_u = 871\ kN < \phi P_n = 0.75 \times 1979 = 1484\ kN\ \ Ok.} \blacksquare$$

Therefore, the columns are adequate according to strength requirements of ACI Code.

3. Checking of Columns Stability:

$$U = 0.9D + 1.6W$$

$$P_{u1} = 0.9 \times 280 - 1.6 \times 74$$

$$P_{u1} = \frac{668}{5} = 134\ kN > 0 \therefore\ Ok.$$
$$U\ =\ 0.9D\ +\ 1.0E$$
$$P_{u2} =\ 0.9 \times 280 - 1.0 \times 488$$
$$P_{u2} = -236\ kN\ < 0\ \therefore\ Not\ Ok. \blacksquare$$

Then the columns are instable according to stability requirements of ACI Code.

Additional Example 1.12-4

Check the adequacy and stability of reinforced concrete columns for the high-elevated cylindrical tank shown in Figure 1.12-4 below according to the requirements of ACI Code.

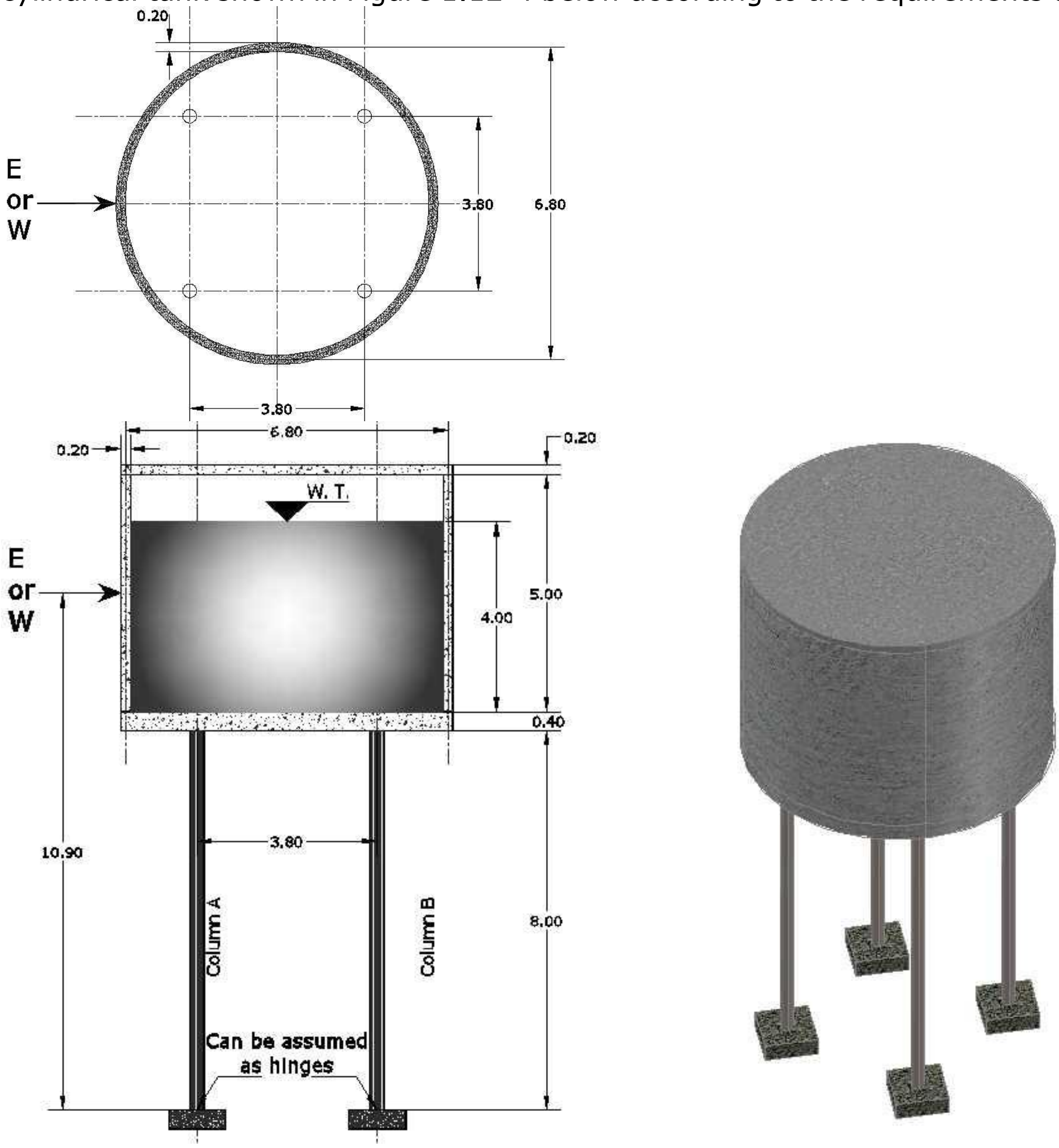


Figure 1.12-4: Elevated tank for Additional Example 1.12-4.

In your solution, assume that:

W = 40 kN

E = 0.25 D

Also assume that the column has a diameter of 300mm and has a nominal strength of P_n = 1979 kN, and a strength reduction factor of 0.75.

Solution

1. Basic Load Cases:

a. Dead Loads:

$$W_{DRoof} = \frac{(6.8 + 0.2)^2 \times \pi}{4} \times 0.2 \times 24$$
$$W_{DRoof} = \frac{294\,\pi}{5} = 185\ kN$$
$$W_{DWall} = (6.8 \times \pi \times 0.2 \times 5) \times 24$$
$$W_{DWall} = \frac{816\,\pi}{5} = 512\ kN$$
$$W_{DFloor} = \frac{(6.8 + 0.2)^2 \times \pi}{4} \times 0.4 \times 24$$
$$W_{DFloor} = \frac{588\,\pi}{5} = 369\ kN$$
$$W_{DColumns} = \left(\frac{0.3^2 \times \pi}{4} \times 8.0\right) \times 24 \times 4$$
$$W_{DColumns} = \frac{432\,\pi}{25} = 54\ kN$$
$$W_D = (185 + 512 + 369 + 54)$$
$$W_D = 1120\ kN$$
$$P_D = 1120 \times \frac{1}{4}$$
$$P_D = 280\ kN$$

b. Fluid Weight:

$$W_{Fluid} = \left(\frac{(6.8 - 0.2)^2 \times \pi}{4} \times 4\right) \times 10$$
$$W_{Fluid} = \frac{2178\,\pi}{5} = 1368\ kN$$
$$P_{Fluid} = \frac{1368}{4} = 342\ kN$$

c. Wind Loads:

$$P_{Wind} = \pm\left(40 \times \frac{10.9}{3.8}\right) \times \frac{1}{2}$$
$$P_{Wind} = \pm 57\ kN$$

d. Seismic Loads:

$$P_E = (1120 \times 0.25) \times \frac{10.9}{3.8} \times \frac{1}{2}$$
$$P_E = \pm 402\ \mathrm{kN}$$

2. Checking of Column Strength:

$$\mathrm{U} = 1.4(\mathrm{D} + \mathrm{F})$$
$$\mathrm{P_{u1}} = 1.4(280 + 342)$$
$$\mathrm{P_{u1}} = 871\ \mathrm{kN}$$
$$\mathrm{U} = 1.2\mathrm{D} + 1.6\mathrm{W}$$
$$\mathrm{P_{u2}} = 1.2 \times 280 + 1.6 \times 57$$
$$P_{u2} = 427\ \mathrm{kN}$$
$$\mathrm{U} = 1.2\mathrm{D} + 1.0\mathrm{E}$$
$$\mathrm{P_{u3}} = 1.2 \times 280 + 1.0 \times 402$$
$$P_{u3} = 738\ kN$$
$$\mathrm{P_u} = \text{Maximum } (871 \text{ or } 427 \text{ or } 738)$$
$$\mathrm{P_u} = 871\ \mathrm{kN} < \phi \mathrm{P_n} = 0.75 \times 1979 = 1484\ \mathrm{kN}\ \ \mathrm{Ok.} \blacksquare$$

Therefore, the columns are adequate according to strength requirements of ACI Code.

3. Checking of Columns Stability:

$$U = 0.9D + 1.6W$$
$$P_{u1} = 0.9 \times 280 - 1.6 \times 57$$
$$P_{u1} = 161\ kN > 0 \therefore\ Ok.$$
$$U = 0.9D + 1.0E$$
$$P_{u2} = 0.9 \times 280 - 1.0 \times 402$$
$$P_{u2} = -150\ kN < 0 \therefore\ Not\ Ok. \blacksquare$$

Therefore, the columns are instable according to stability requirements of ACI Code.

Additional Example 1.12-5

For foundation of pedestrian bridge presented in Figure 1.12-5 below, section A-A has been designed with M_n = 800 kN.m and φ = 0.9. Is this section adequate according to ACI Code requirement?

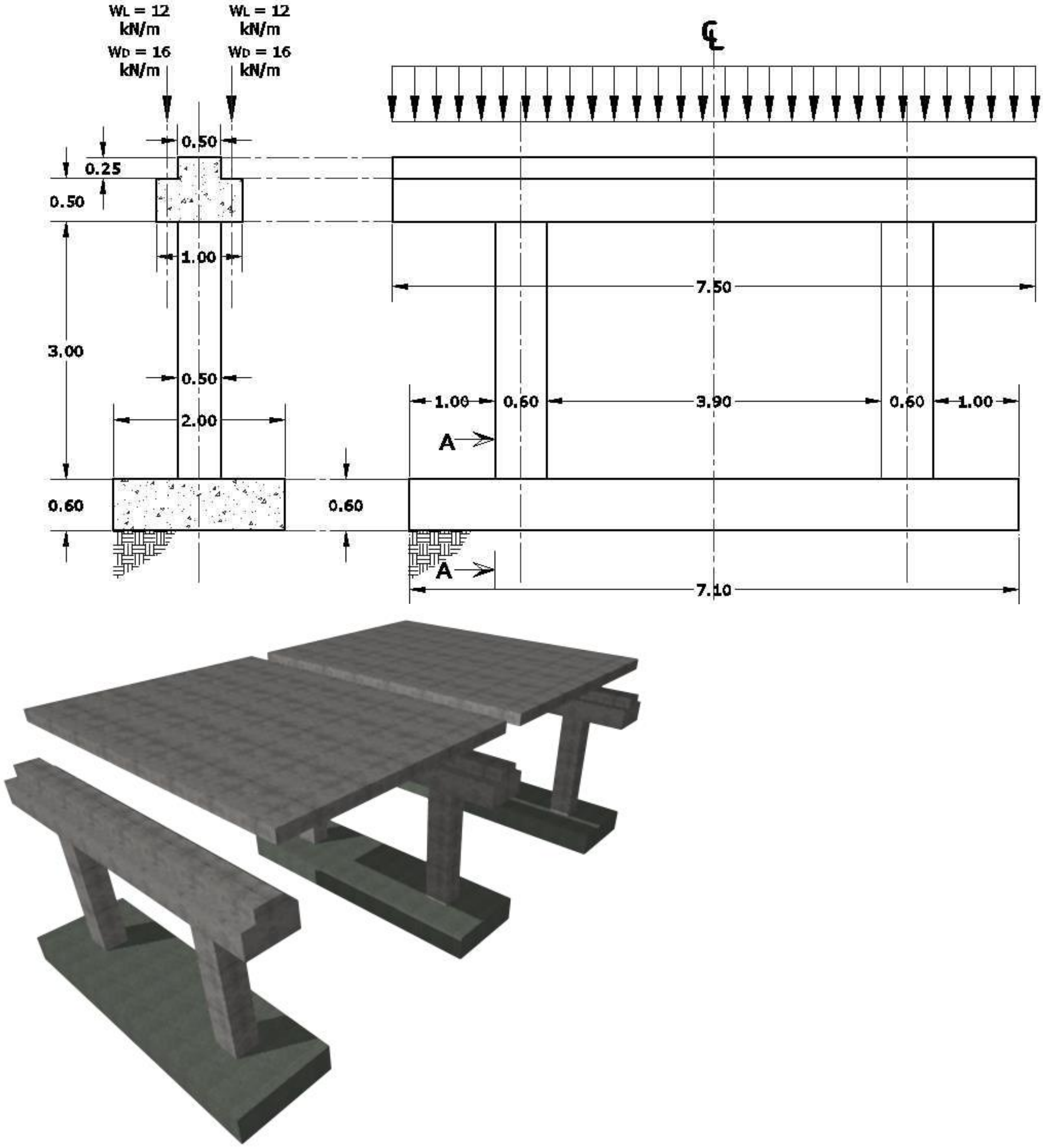


Figure 1.12-5: Pedestrian bridge for Additional Example 1.12-5.

In your Strength Checking, consider following Load Combinations:

- $1.4\,D$,
- $1.2D + 1.6L$.

In your solution, assume that:

- $\gamma_{concrete} = 24\,kN/m^3$,
- Uniform subgrade reaction.

Solution

Dead Load:

$$Volume\ of\ Concrete = [(0.5 \times 0.25 + 0.5 \times 1.0)m^2 \times 7.5m]_{Vol\ of\ cross\ beam} + [(0.5 \times 0.6)m^2 \times 3m \times 2]_{Vol.of\ columns}$$

$$Volume\ of\ Concrete\ = [4.69\ m^3]_{Vol\ of\ cross\ beam} + [1.8\ m^3]_{Vol.of\ columns}$$

$$Selfweight\ of\ Concrete\ = \left(4.69m^3 \times 24\frac{kN}{m^3}\right)_{weight\ of\ beam} + \left(1.8\ m^3 \times 24\frac{kN}{m^3}\right)_{weight\ of\ columns}$$

$$Selfweight\ of\ Concrete\ = 112.6 + 43.2 = 156\ kN$$

$$R_D = \left(16\frac{kN}{m} \times 2 \times 7.5m\right) + 156\ kN =$$

$$R_D = 240\ kN + 156\ kN = 396\ kN$$

Live Load:

$$R_{Live} = 12\frac{kN}{m} \times 2 \times 7.5\ m = 180\ kN$$

Subgrade Reactions:

$$W_D = \frac{396\ kN}{7.1\ m} = 55.8\frac{kN}{m}$$

$$W_L = \frac{180\ kN}{7.1\ m} = 25.4\frac{kN}{m}$$

Factored Load:

$$W_u = maximum\ (1.4 \times 55.8\ kN, 1.2 \times 55.8\ kN + 1.6 \times 25.4\ kN)$$

$$W_u = maximum\ \left(78.1\frac{kN}{m}, 108\frac{kN}{m}\right) = 108\frac{kN}{m}$$

Factored Moment:

$$M_u = 108\frac{kN}{m} \times 1.0m \times \frac{1}{2}m = 54\ kN.m$$

Checking of Section Adequacy:

$M_u\ ?\ \phi M_n$

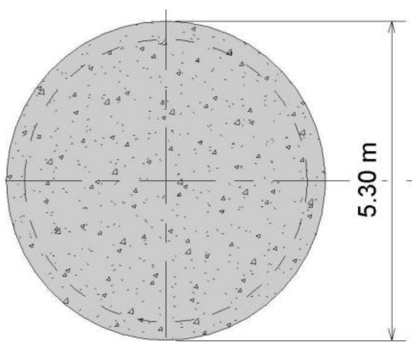
$54\ kN.m\ ?\ 0.9 \times 800\ kN.m$

$54\ kN.m < 720\ kN.m\ Ok.$ ■

Additional Example 1.12-6

Due to weak soil conditions, water tank shown in Figure 1.12-6 below is supported on a piled foundation. With this foundation, tank stability is ensured.

- What are ACI load combinations that should be considered in checking strength adequacy of the supporting column?
- If the column has a nominal axial strength, P_n, of 1200 kN and a design flexural strength, ϕM_n, of 300 kN.m, is it adequate from strength point of view? In your solution, assume reduction strength factor, ϕ, of 0.75.



Top View

3D View

Figure 1.12-6: Tank for Additional Example 1.12-6.

Solution

- Load Combinations for Strength Checking

  As stability is ensured with the piled foundation, then only strength combinations should be considered in the solution.

  $U_{DF} = 1.4(D + F)$

  $U_{DW} = 1.2D + 1.6W$

  $U_{DE} = 1.2D + 1.0E$

- Strength Checking of the Column

Basic Loads

Dead Load

$$P_{D\ Roof} = P_{D\ Floor} = \frac{\pi \times 5.3^2}{4} \times 0.3 \times 24 = 159\ kN$$

$$P_{D\ Wall} = (4.7 + 0.3) \times \pi \times 0.3 \times 3.4 \times 24 = 385\ kN$$

$$P_{D\ Column} = \frac{\pi \times 0.5^2}{4} \times 6.00 \times 24 = 28.3$$

$$P_D = 159 \times 2 + 385 + 28.3 = 731\ kN$$

Fluid

$$P_F = \frac{\pi \times 4.7^2}{4} \times 2.82 \times 10 = 489\ kN$$

Wind

$$M_W = 32 \times 8.0 = 256\ kN.m$$

Seismic

$$M_E = 0.2 \times 731 \times 8.00 = 1170\ kN.m$$

Load Combinations

Load Combination of 1.4(D+F)

$$P_u = 1.4 \times (731 + 489) = 1708\ kN > 0.75 \times 1200 = 900\ kN \therefore Not\ Ok.$$

Load Combination of (1.2D +1.6W)

$$P_u = 1.2 \times 731 = 877\ kN < 0.75 \times 1200 = 900\ kN \therefore Ok.$$

$$M_u = 1.6 \times 256 = 410\ kN.m > 300\ kN.m \therefore Not\ Ok.$$

Load Combination of (1.2D +E)

$$P_u = 1.2 \times 731 = 877\ kN < 0.75 \times 1200 = 900\ kN \therefore Ok.$$

$$M_u = 1.0 \times 1170 = 1170\ kN.m > 300\ kN.m \therefore Not\ Ok.$$

Therefore, the proposed is inadequate.

Important Notes

It will be shown in Chapter 8, Short Columns, that a column has different design strength, ϕP_n and ϕM_n, for different combinations of P_u and M_u. Therefore, using same design strength for different load combinations, as done in this example, is not accurate and has been adopted only to present the probabilistic nature of current ACI design philosophy.

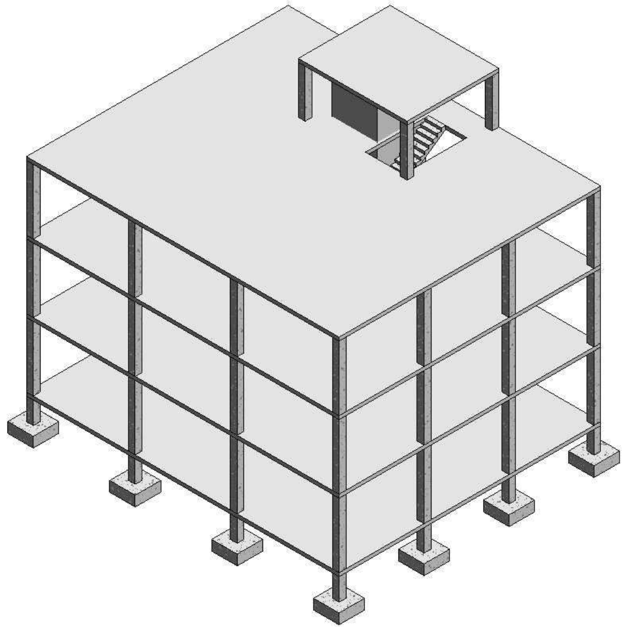
Additional Example 1.12-7

For a hotel building indicated in ***Figure 1.12-7*** below that has flat plate slabs ***200mm*** in thickness, columns of ***400mm by 400mm***, and it is subjected to a floor superimposed dead load of ***2.0 kPa*** and to a roof superimposed dead load of ***3.0 kPa***:

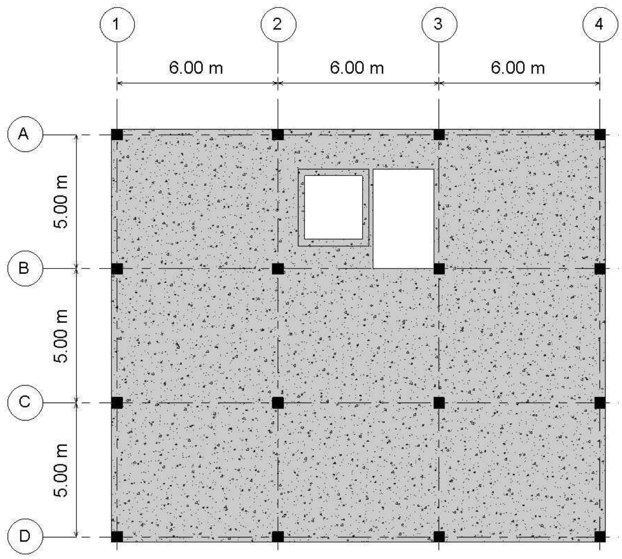
- Select an appropriate value for roof live load,
- Select an appropriate value for floor live load. Most of floor area is proposed for private rooms,
- For a column located at grid lines D-1:
  - What is column axial force at foundation level due to selfweight, i.e. P_{Self}?
  - What is column axial force at foundation level due to superimposed dead load, i.e. $P_{Superimposed\ Dead}$?
  - With adopting reduced live load, if possible, what is column axial force at foundation level due to floor live load, i.e. P_L?
  - With adopting reduced roof live load, if possible, what is column axial force at foundation level due to roof live load, i.e. P_{L_r}?

- What is column maximum ultimate axial load, P_u, due to following load combinations:
  $U = 1.2D + 1.6L + 0.5L_r$
  $U = 1.2D + 1.0L + 1.6L_r$

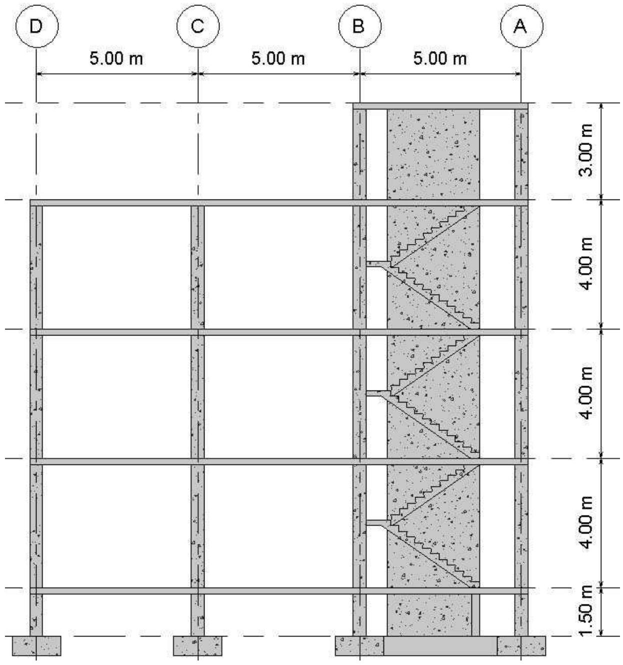
Is the column adequate to support aforementioned axial force, P_u, if it has an axial capacity of P_n of 3550 kN and has a strength reduction factor, ϕ, of 0.65?



3D view.



Plan view.



Elevation view.

Figure 1.12-7: Structural system for a hotel building for Additional Example 1.12-7.

Solution

- Appropriate value for roof live load:
  According to ASCE 7-10, live load for ordinary flat roof is:
  $L_r = 0.96\ kPa$
- Appropriate value for floor live load:
  According to ASCE 7-10, for private rooms in residential or hotel building, a live load of:
  $L = 1.92\ kPa$
  can be adopted.
- Axial force at foundation level due to selfweight:
  Assuming that the corner column located at grid line D-1 supports a tributary of:
  $A_{Tributary} = \frac{5 \times 6}{4} = 7.5\ m^2$