PART I: THE DESIGN PROCESS

A) INTRODUCTION

B) DESIGN LOADS

C) CODES & SPECIFICATIONS

D) DESIGN FORMATS

1.1 READING

MacGregor	Chapters 1 & 2
Wang & Salmon:	Chapter 2
Ferguson, Breen, Jirsa:	Chapter 1
ACI 318 Building Code:	Chapter 2
	Chapter 9; Sections 9.1 - 9.4

1.2 OBJECTIVES

(1) General Design Concepts
(2) Loads
  (a) Determination of loads (gravity, wind, snow, earthquake)
  (b) How are loads carried from the floor slabs, to the joists and beams, to the columns, and then to the foundation. Important concepts: Load paths, tributary areas
(3) Design Formats and Limit States
  (a) Limit State (Ultimate loads)
  (b) Allowable Stress (Service loads)

1.3 THE DESIGN PROCESS (For Buildings)

1.3.1 GENERAL

(1) Background Course-work

Statics
Dynamics
Strength of Materials $\Rightarrow$ Design
Structural Analysis

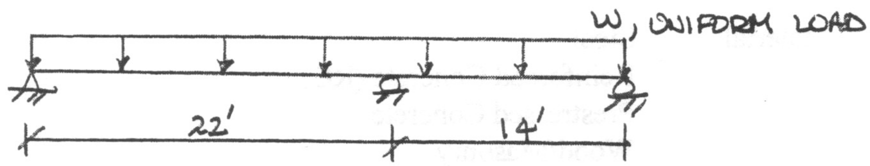
Combine intuitive feeling for behavior of a structure with a sound knowledge of principles of statics, dynamics, strength of materials, and structural analysis to produce a safe, economical structure which will serve its intended purpose.

Intutitive Feeling
- Is the answer realistic?
- Rough approximations to get "ball park" estimates
- How will the system/element fail

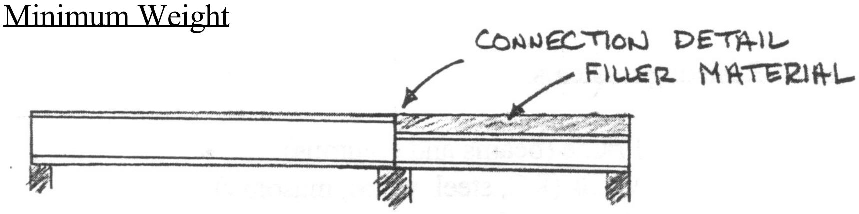
For a Safe, Economical Design: Design process should give an optimum solution. The criteria for the design can be:

Minimum Cost
Minimum Weight
Minimum Construction Time
Minimum Labor
Maximum Long Term Operating Efficiency
Safety (Nuclear)
Operation (hospital following an earthquake)

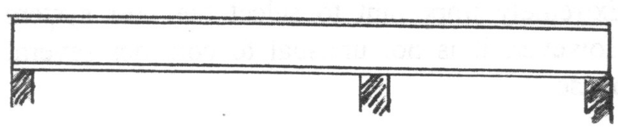
Best solution is more likely to be a combination of criteria. Consider a simple continuous beam.



Minimum Weight



Minimum Cost



Connection design and construction is costly; therefore, minimum weight design may not always be the same as minimum cost.

1.3.2 THE STRUCTURAL DESIGN PROCESS

(1) Functional Planning:

Establish form, shape and size requirements
- Adequate working area
- Stairs/elevators
- Architectural attractiveness

Identify unusual structural requirements
- Large loads, long spans

Review building code requirements
- Local, State, Federal, Client

(2) Select Trial Structural Systems

Material:
- Steel
- Reinforced Concrete (RC)
- Prestressed Concrete
- Wood/Masonry

Combination (composite construction)

Load Resisting Systems:
- Frame (beams and columns)
- Walls (RC, steel, wood, masonry)
- Frame-wall
- Braced frame (concentric, eccentric)
- Floor system (RC, prestressed, composite)

Extremely important to select the best system for a given application. However, it is not unusual to consider several in the preliminary design phase.

Structural Configuration
- General layout of structural system
- Special considerations due to functional requirements/loads

Foundation System
- Best types of foundations for a given building site

(3) Preliminary Design of Trial Systems

(a) Estimate Design Loads: dead, live, snow, impact, wind, pressure, temperature, earthquake, etc.

(b) Use simple, approximate methods of analysis and design.

(c) Establish approximate member sizes and general connection details.

(4) Evaluate Preliminary Designs

Review each trial system. Criteria for review:

Cost
Aesthetics
Local practice/preference
Etc.

Select a structural system for detailed analysis (may choose more than one).

(5) Analysis of Structural System(s)

Establish loads
Model structure (for hand or computer analysis)
Compute forces for members (P, V, M, T)
Compute deflections

The analysis is usually done with commercial software (STAAD, SAP, RISA, etc).

(6) Detailed Design of Structural System(s)

Design of members, connections, and foundation

(7) Evaluation of Detailed Design

Are the criteria met

Possible Redesign: Steps (2) - (6) Complete Redesign
Steps (5) - (6) Partial Redesign

This course will focus on all aspects of design; however, steps (5) and (6) will receive the most attention. The course project, if included, will invlove all facets of design, including oral presentations with visual aids.

(8) Prepare Drawings

Drafting
Project specifications

(9) Construction

Fabrication, Erection, and Inspection of the steel

1.3.3 RESPONSIBILITIES OF STRUCTURAL DESIGNER

Structure must be:

(1) Safe
(2) Serviceable, ie. it can be used for its intended purpose. No excessive deformations, vibrations, etc.
(3) Economical
(4) Constructable
(5) Maintainable
(6) Aesthetically pleasing (architect), especially for dominant building

Safety is, by far, the most important of these responsibilities. Safety depends on many factors, such as:

Quality of materials
Quality of labor
Degree of quality control and inspection
Maintenance (corrosion, fatigue)
Uncertainty of loads
Code requirements

Because safety is a matter of public concern it is not left completely to the judgment of the engineer.

Safety issues are dictated by codes, specs, and standards; however, it should be understood that codes and specifications provide only minimum requirements.

1.3.4 BUILDING CODES, SPECIFICATIONS, AND STANDARDS

(1) Building Codes

- Usually legal documents
- Purpose is to protect public safety
- Developed by engineers (professional organizations, such as ASCE, AISC, ACI)
- Codes typically specify good design practice for typical buildings and loadings; therefore, they may not be sufficient for all buildings.
- Types of loadings specified: live loads, wind loads, earthquake loads
- Detailed design rules for steel,concrete, etc.
- Other issues (fire protection)
- Examples: (International Building Code – IBC)

  Uniform Building Code (e.g., UBC-97)
  National Building Code
  Standard Building Code

  ACI 318 Building Code

  Major City Codes: Chicago, NY, LA, SF, etc

(2) Specifications

- Design guidelines and recommendations
- Published by recognized engineering societies
- Examples

  AISC: Steel Buildings
  LRFD Specifications
  ASD Specificatons

  ACI: Concrete Buildings
  Committee reports & Journals

  AASHTO: Bridges and roads

- Specifications are not legal documents; however, they are usually referenced in building codes. They may also contain more up to date information, and be of assistance for unusual design problems.

(3) Standards

- Cover many areas (material requirements)

- Example: (a) ASTM (American Society of Testing and Materials)

  Mechanical and Chemical properties of steel
  Dimensions of bolts
  Testing procedures to establish material properties

  (b) Recommended minimum design loads ASCE 7-98, Minimum Design loads for buildings and other structures.

- Not a legal document; however, they are often referenced in codes.

(4) Use of Codes, Specifications, and Standards

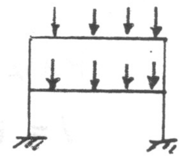
- Considerable overlap
  Often must be familiar with each

- They do not cover every situation; therefore, the engineer must have sound knowledge of behavior so that they are not improperly applied.

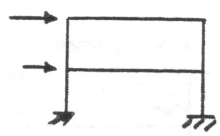
- The engineer is ultimately responsible for ensuring that the structure is both safe and functional (from a structural point of view).

1.4 LOADS

(1) Gravity Loads
- (a) Dead
- (b) Live
- (c) Snow



(2) Lateral Loads
- (a) Wind
- (b) Earthquake



(3) Others
- (a) Change in temperature
- (b) Current, waves, etc.
- (c) Blast
- (d) Vibrating machinery
- (e) Etc.

We will use the latest version of the Uniform Building Code (UBC) to establish loads for most cases. The UBC is revised every three years (91, 94, 97), and is widely used throughout the US. However, UBC-97 is the last version – the IBC will replace it.

1.4.1 DEAD LOADS

Fixed position gravity load - weight of structure and attached components (e.g. pipes,ducts, lighting fixtures, floor coverings, etc.). They can be predicted with reasonably accuracy.

STATIONARY AND CONSTANT

Often use published data in handbooks to estimate weights for preliminary design, for example, ASCE 7-98 (See Section 1.3.4), and ACI 318 (Section 9.1 - 9.3). Values may need to be revised for final design.

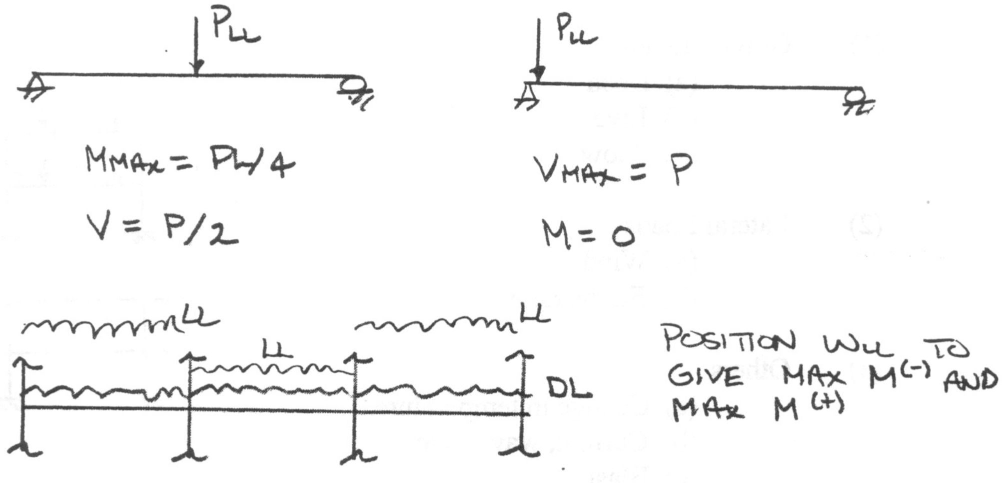
1.4.2 LIVE LOADS

Gravity loads acting when structure is in service (e.g. people, furniture, movable equipment, etc.)

NOT STATIONARY AND NOT CONSTANT

More difficult to predict than dead loads (more uncertainty). Minimum values are specified by codes and/or specs (eg., UBC Table 16-A, B, C) for typical construction. Also, see ASCE 7-98 (http://www.pubs.asce.org/).

Must position live load to give worst effect



May also have to consider impact loads for cranes and other machinery. For example, see LRFD 2nd Edition, Section A4.2, pg. 6-30 (lateral crane loads also apply).

Typical values for Live Loads (LL).

UBC-97 Table 16A		
	Residential	40 psf
	Offices	50 psf
	Storage	125 psf (light)
		250 psf (heavy)
UBC-97 Table 16C		
	Roofs	20 psf (minimum, may be more)
		snow must also be considered, as discussed later.

Live Load Reductions

Values given in UBC-97 are maximum values for a relatively small area. For large areas, a reduction is allowed. A reduction is allowed because it is unlikely that the maximum loads for a relatively small area will exist over a larger area. Example: humanoids in a room (except for large meeting areas). Exception: Storage areas.

Live Load reductions depend on the **Tributary Area** or **Influence Area** for the given structural member being considered for design. The tributary area is defined as the area of floor or roof (in plan) that causes loading on a particular structural element.

See UBC-97 Section 1606 and 1607 (UBC handout)

UBC-97 Section 1606 Requirements for Floors (for roofs, use Table 16-C):

If $A < 150\ \text{ft}^2$ No live load reduction

If $A \geq 150\ \text{ft}^2$ Then

$R_1 = r\,(A - 150\ \text{ft}^2)$, where $r = 0.08$

$R_2 = 23.1\,(1 + D/L)$

$R_3 \leq 40\%$ if loads on a given structural member are all from one level. This requirement typically applies to joists and beams, as well as to columns supporting roofs.

$R_4 \leq 60\%$ if loads on a given structural member are from more than one level. This requirement typically applied to columns other than those at the roof level.

$$R_{max} = \begin{cases} \text{Minimum value of } \{R_1, R_2, R_3\} \text{ for one level} \\ \text{Minimum value of } \{R_1, R_2, R_4\} \text{ for more than one level} \end{cases} \qquad \textbf{(Eq. 1-1)}$$

where: R = reduction in %
r = 0.08
A = tributary area (ft^2)
D = dead load (unfactored)
L = live load (unfactored)

R_{max} = the maximum allowed live load reduction

Other Requirements (UBC-97 Section 1607.5)

No live load reductions are allowed for places of public assembly with live loads greater than 100 psf.

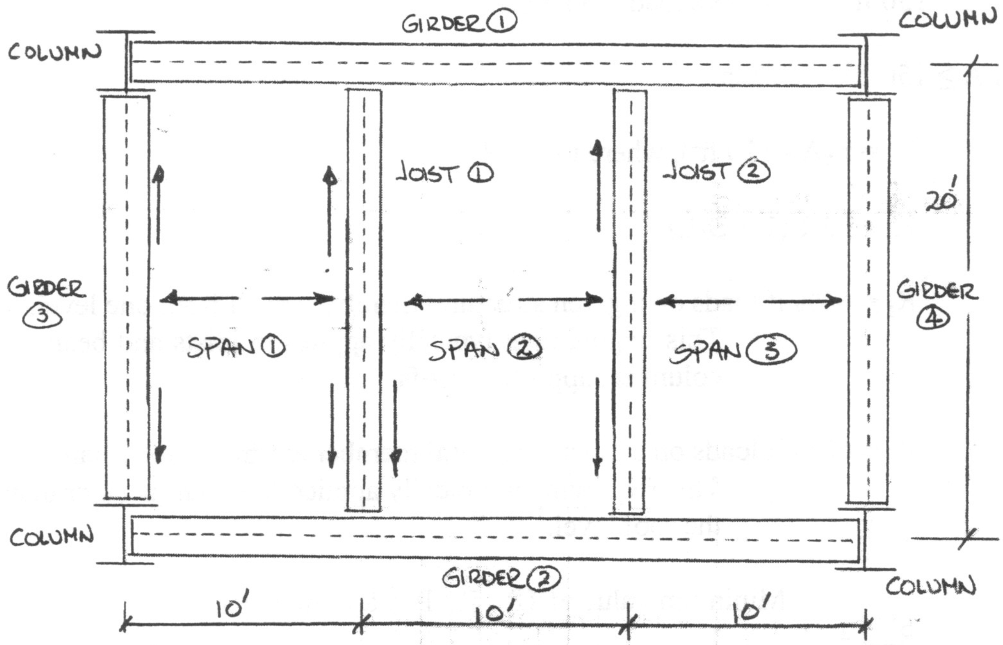
Storage facialities: LL > 100 psf

No LL reductions except columns may be reduced by a maximum of 20%. The reduction is computed based on the equations given above.

Important Comment:

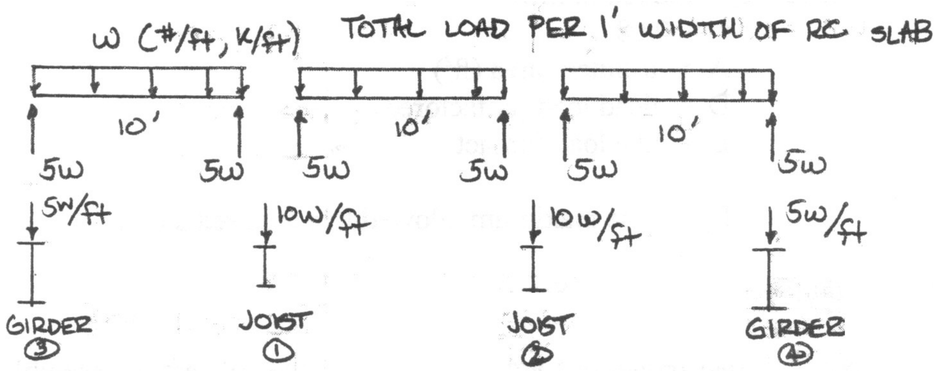
(1) **Under no circumstance can Dead loads be reduced**
This is a common error for homework and exams

1.4.3 LOAD TRANSFER or LOAD PATHS

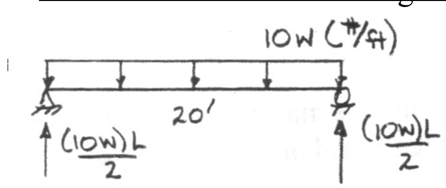


PLAN VIEW OF BUILDING

1.) A Floor slab (usually RC) is typically supported on floor joists.



2.) Joist loads are transferred to girders (As example, consider Joist 1)



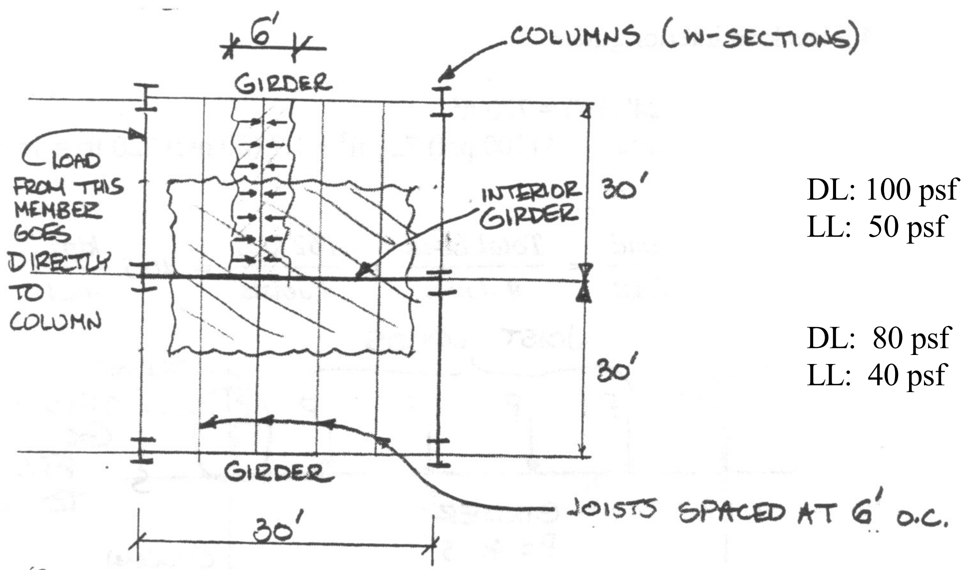
Girder 1 Girder 2

1/2 Load to Girder 1

1/2 Load to Girder 2

3.) Girder loads are transferred to column ⇒ Foundation

4.) Consider an example -- For now, neglect live load reductions



DL: 100 psf Floor
LL: 50 psf

DL: 80 psf Roof
LL: 40 psf

a) Compute Tributary Area

TRIBUTARY WIDTH
JOIST SPAN

Joist (Interior, Floor): ATRIB = 6' (30') = 180 ft^2

Girder (Interior, Floor): ATRIB = 24' (30') = 720 ft^2

Note: The load path for the 3' width of slab (at each end) is from the slab directly to the member that frames between the two columns, and then directly into the column. It does not transfer any load to the girder that the other joists are supported on.

Now we need to determine shear, moment, and axial load diagrams for each structural member.

b.) Joists at floor level

Concept of tributary width is useful for joist design:

w_{TRIB} = 6' : DL = 100 psf LL = 50 psf

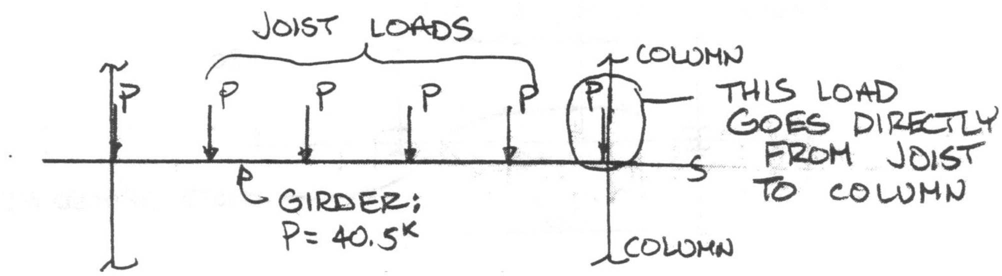
w_{DL} = 100 psf (6') = 600 #/ft; w_{LL} = 50 psf (6') = 300 #/ft
w_u = 1.4(600) + 1.7(300) = 1.35 kip/ft where 1.4 is the Dead load, "load factor," and 1.7 is the live load, "load factor" - See ACI Section 9.2.

c.) Girder (Interior) at floor level

4 joists load interior girder

Tributary Area = 24' (30') = 720 ft^2
Total (factored) Load = 1.4 (100 psf) 720 ft^2 + 1.7 (50 psf) 720 ft^2 = 162 kips

$$\frac{Load}{Joist} = \frac{Total\ Load}{\#\ Joists} = \frac{162\ kips}{4\ Joists} = 40.5\ \frac{kips}{joist}$$



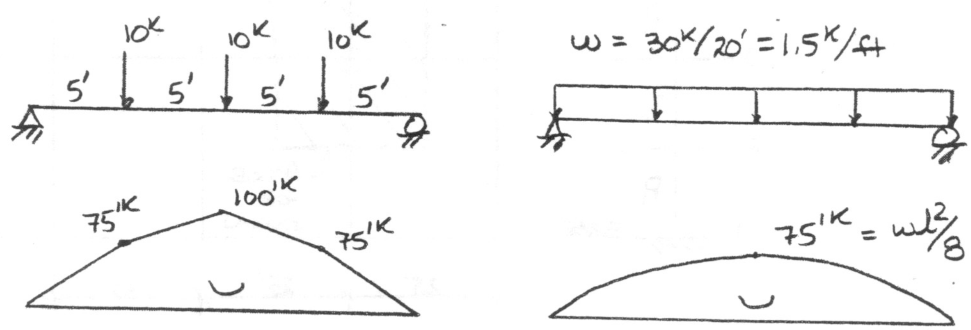
ELEVATION VIEW

d.) Exterior Girder at floor level

A_{TRIB} = 24' (15') = 360 ft^2

P_u = 20.25 kips (one-half of the load for an interior girder) e.) NOTE

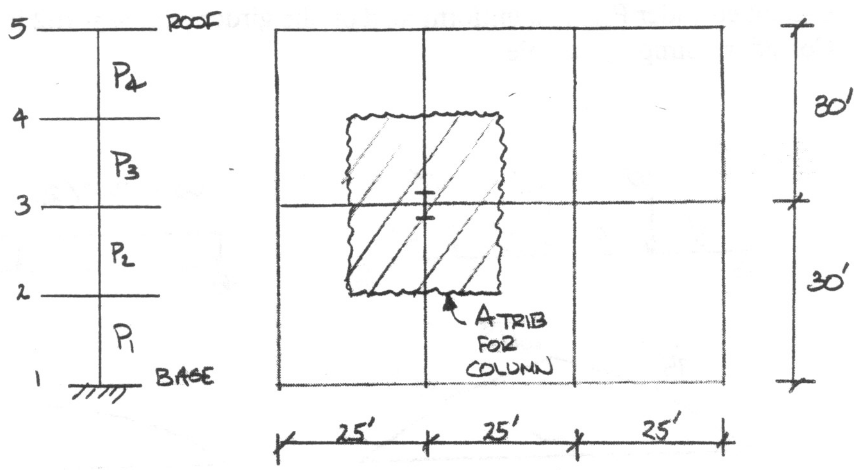
Cannot consider P_{total} as a uniform load on the girder (e.g. w = 162 k/30' = 5.4 k/ft). Why? Consider a simple example.



Moment Diagrams

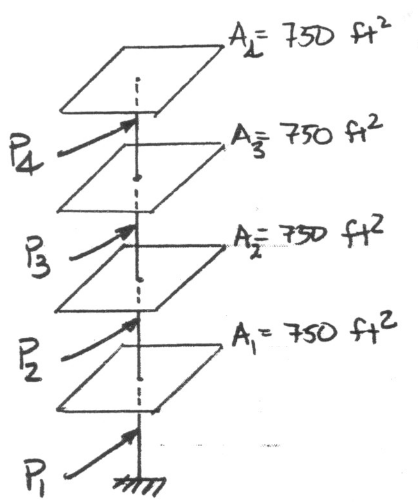
RESULT: The uniform loading (5.4 k/ft) under estimates moment (not safe). However, as more joists are used (try the same example with 5 joists), the two values for maximum moment get closer, but are still different.

f.) Column Assume 4-story Building, Consider an interior column:



ELEVATION PLAN VIEW

To compute column loads, consider each story individually. Note that once the loads are computed for the roof and third floor column, the remaining column loads can be computed directly; however, this may not be the case where live loads reductions are considered, or where loads vary from floor to floor.



For P_4: (Axial load in 4th-story column)

$A_{TRIB} = A_4 = 750\ ft^2$
$P_{4,DL} = 750\ ft^2\ (80\ psf) = 60\ kips$
$P_{4,LL} = 750\ ft^2\ (40\ psf) = 30\ kips$
$P_{4,u} = 1.4(60\ k) + 1.7\ (30\ k) = 135\ kips$

For P_3: (Axial load in 3rd-story column)

$A_{TRIB} = A_4 + A_3 = 1500\ ft^2$
$P_{3,DL} = 60\ kips + 750\ ft^2\ (100\ psf) = 135\ kips$
$P_{3,LL} = 30\ kips + 750\ ft^2\ (50\ psf) = 67.5\ kips$
$P_{3,u} = 1.4(135\ k) + 1.7(67.5\ k) = 303.75\ kips$

For P_2: (Axial load in 2nd-story column)

$P_{2,u} = 303.75^k + 168.75^k = 472.5^k$

For P_1: (Axial load in 1st-story column)
$P_{1,u} = 472.5^k + 168.75^k = 641.25^k$

5.) Consider the Same Example and Include Live Load Reductions

Joist (Interior, Floor): ATRIB = 6' (30') = 180 ft^2

R_1 = 0.08 (180-150) = 2.4%
R_2 ≤ 23.1 (1 + 80/40) = 69.3% ⇒ R = 2.4%
R_3 < 40% (because all loads from one level)

w_{DL} = 600 #/ft (no change from previous example)
w_{LL} = 50 psf (1-0.024) (6 ft) = 292.8 #/ft
w_u = 1.4(600) + 1.7(292.8) = 1337.76 #/ft

which is only slightly less than the previous example since the LL reduction is very small (2.4%).

Girder (interior, floor): ATRIB = 24' (30') = 720 ft^2

R_1 = 0.08 (720-150) = 45.6%
R_2 ≤ 23.1 (1 + 100/50) = 69.3% ⇒ R = 40%
R_3 < 40% (because all loads from one level)

P_{DL} = 100 psf (720 ft^2) = 72 kips
P_{LL} = 50 psf (1 - 0.40) (720 ft^2) = 21.6 kips
Total (factored) load = 1.4 (72) + 1.7 (21.6) = 137.52 kips
P_u (joist) = 137.52/4 = 34.38 kips

which is somewhat less than the 40.5 kip load (162 kips for four joists) when LL reductions were neglected.

Girder (Exterior, floor): ATRIB = 24' (15') = 360 ft^2

R_1 = 0.08 (360-150) = 16.8%
R_2 ≤ 23.1 (1 + 100/50) = 69.3% ⇒ R = 16.8%
R_3 < 40% (because all loads from one level)

P_{DL} = 100 psf (360 ft^2) = 36 kips
P_{LL} = 50 psf (1 - 0.168) (360 ft^2) = 15.0 kips
Total (factored) load = 1.4 (36) + 1.7 (15.0) = 75.9 kips
P_u (joist) = 75.9/4 = 18.975 kips

Note that for the joist loads on the exterior girder are not one-half of those for an interior girder (34.38 k/2 = 17.19^k) because the LL reductions are different (tributary areas are different).

Column (interior):

For P_4: (Axial load in 4^{th}-story column)

$A = A_4 = 750\ ft^2$
$R_1 = 0.08\ (750 - 150) = 48\%$
$R_2 \leq 23.1\ (1 + 80/40) = 69.3\%$ $\Rightarrow R = 40\%$
$R_3 < 40\%$ (because all loads from one level)

$P_{4,DL} = 750\ ft^2\ (80\ psf) = 60\ kips$
$P_{4,LL} = 750\ ft^2\ (40\ psf)\ (1\text{-}0.4) = 18\ kips$
$P_{4,u} = 1.4(60\ k) + 1.7\ (18\ k) = 114.6\ kips$

For P_3: (Axial load in 3^{rd}-story column)

$A = A_4 + A_3 = 1500\ ft^2$
$R_1 = 0.08\ (1500 - 150) = 108\%$
$R_2 \leq 23.1\ (1 + 100/50) = 69.3\%$ $\Rightarrow R = 60\%$
$R_3 < 60\%$ (because loads from two levels)

$P_{3,DL} = 60\ kips + 750\ ft^2\ (100\ psf) = 135\ kips$
$P_{3,LL} = [(750\ ft^2)\ (40\ psf) + (750\ ft^2\ (50\ psf)]\ (1\text{-}0.6) = 27\ kips$
$P_{3,u} = 1.4(135\ k) + 1.7\ (27\ k) = 234.9\ kips$

For P_2: (Axial load in 2^{nd}-story column)
$R = 60\%$
$P_{2,u} = 234.9\ k + [1.4\ (75\ k) + 1.7\ (15\ k)] = 365.4\ k$ [ ] = 130.5 kips

For P_1: (Axial load in 1^{st}-story column)
$R = 60\%$
$P_{1,u} = 365.4\ k + 130.5\ k = 495.9\ k$

It is clear from this example that live load reductions may have a significant affect on column loads, especially for columns near the base of the building. For example, at the base, the reduced factored load is only 77.3% (495.9/641.25 = 0.773) of the unreduced load.

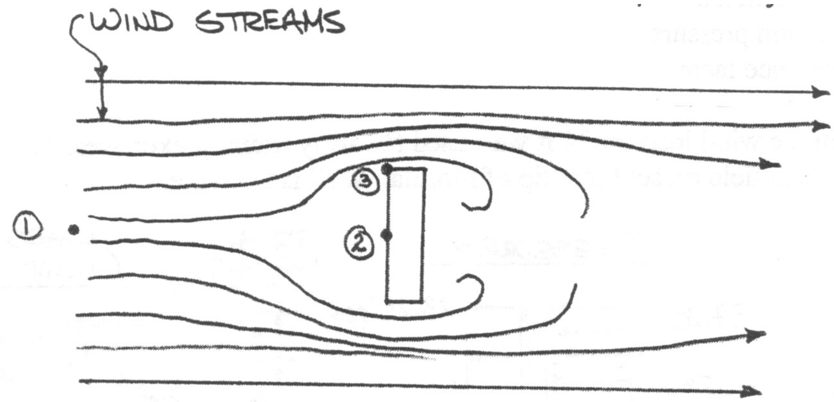
Note that this problem is a little more complicated, if floor loads are different. For this case, compute R based on the total tributary area for the given member, and then apply this R value to the loads at each floor level (for example, in the previous example, the R value for the base column is 60% (unless the loads changed such that R_2 was less than 60%), then apply this 60% reduction to the loads at all levels). Use judgement in calculating R_2 if the ratio of the dead and live loads at each level are different. For example, use a weighted average or consider each level independently.

1.4.4 Wind Loads

(a) Based on Bernoulli's Theorem for flow along a stream (incompressible fluid).

$p + 0.5\,\rho V^2 = \text{constant} = C$

p = pressure
V = velocity
ρ = density



Points 1 and 2:

$p_1 + 0.5\,\rho V^2{}_1 = p_2$
$p_2 - p_1 = 0.5\,\rho V_1^2 = q_s$

q_s = stagnation pressure

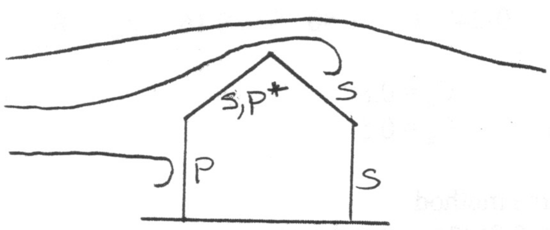
Points 1 and 3:

$p_1 + 0.5\,\rho V_1^2 = p_3 + 0.5V_3^2$
$p_3 - p_1 = 0.5\,\rho\,(V_1^2 - V_3^2)$

But $v_3 > v_1$, due to smaller smaller flow area

$\therefore\ p_3 - p_1$ = suction

(b) For Buildings



s = suction
p = pressure

* Steep Roof □ Pressure

Flat Roof
or Slight Slope ⇒ Suction

UBC: $q_s = 1/2\ (0.0765\ \#/ft^3/32.2\ ft/sec^2)\ (5280/3600)^2\ V^2 = 0.0026\ V^2$

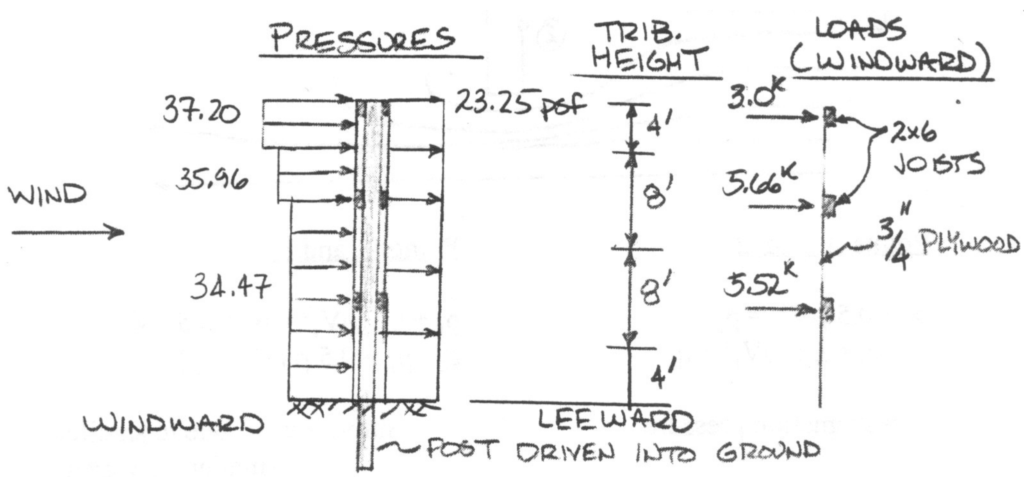
V in mph, from Table 16-F of UBC-97

(c) UBC Wind Loads: Section 1613 -- 1623

p (pressure) = $C_e\ C_q\ q_s\ I_w$ **(Eq. 1-2)**

C_e: commbines height, gust, and exposure
C_q: pressure coefficient
q_s: stagnation pressure
I_w: importance factor

Example: Compute wind load on 24 ft wall used for army training exercises. The wall is in an open field on southern tip of Florida. Wall is 20' wide.



* I = 1.0
* Use Table 16-F and Fig. No. 16-1 V = 110 MPH, q_s = 31 psf
* Table 16-G

C_e (exposure D):	windward:	0-15'	C_e = 1.39
		15-20'	C_e = 1.45
		20-25'	C_e = 1.50
	leeward:	0-24'	C_e = 1.50 See S 1622 pg. 2-8

* Table 16-H

C_q (normal force):	windwall	C_q = 0.8
	leeward wall	C_q = 0.5

* Compute pressures: using normal force method

0-15':	windward:	p = 1.39 (0.8) (31 psf) = 34.47 psf
	leeward:	p = 1.50 (0.5) (31 psf) = 23.25 psf
15-20':	windward:	p = 1.45 (0.8) (31) = 35.96 psf
	leeward:	p = 1.50 (0.5) (31) = 23.25 psf
20-24':	windward	p = 1.50 (0.8) (31) = 37.20 psf
	leeward:	p = 1.50 (0.5) (31) = 23.25 psf

* Forces: $P_{24'}$ = (37.2 psf) 4' (20') = 3.0 k
$P_{16'}$ = (35.96) (5')(20') + 34.47(3')(20') = 5.66 k
$P_{8'}$ = (34.47) (8') (20') = 5.52 k
windward forces, then add these to the leeward forces

1.4.4 Earthquake Loads

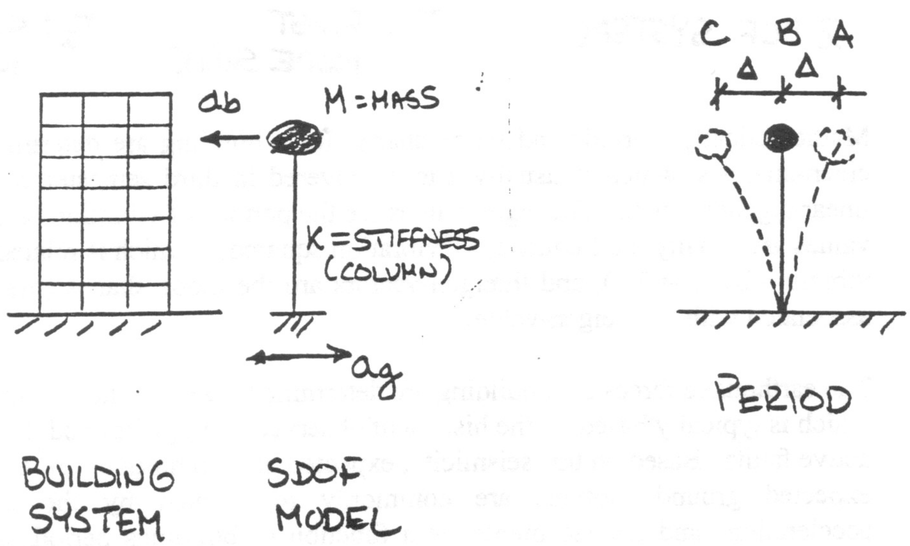
(a) Seismicity

California	San Andreas, Hayward, Inglewood faults
St. Louis	New Madrid (1811, 1812)
Charleston	South Carolina (1886)
Quebec City	1988
Earthquakes:	Northridge (1994), Kobe (1995), Turkey (1999), Taiwan (1999)

(b) Codes - UBC, ANSI; most developed using information from the SEAOC Blue Book (Structural Engineers Association Of California).

(c) Most codes are based on using a SDOF model and an equivalent static or a response spectrum analysis. For a response spectrum analysis, a building is modeled as an equivalent single-degree-of-freedom (SDOF) system to estimate design loads.

Mass and stiffness (flexibility) properties of the structure are used to develope the equivalent SDOF model. The building mass is calculated based on known (or estimated) material weights. The stiffness of the building is calculated based on fundamental principles of structural analysis. Based on the mass and stiffness of the building, the fundamental period of vibration can be calculated, and is then used to estimate earthquake design loads.

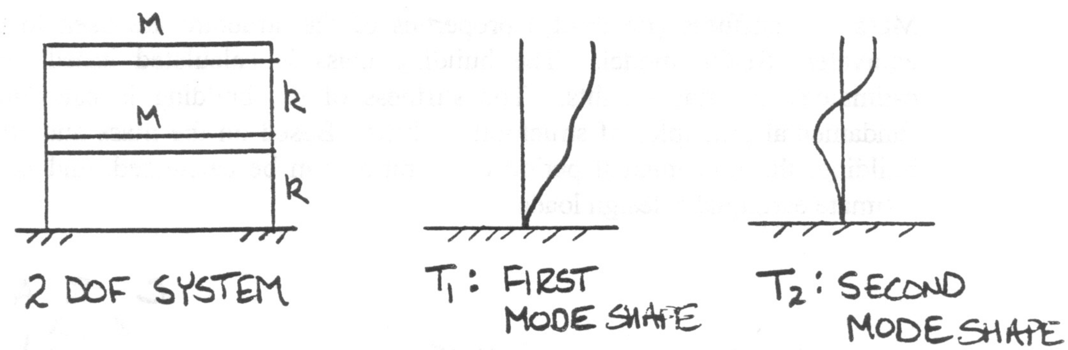


$$T = 2\pi\sqrt{\frac{M}{K}} \qquad \textbf{(Eq. 1-3)}$$

If the mass of a SDOF system (mass on a stick) is displaced Δ (position A) and then released, then the period of the SDOF system can defined as the time it takes the mass to return to its original position (A-B-C-B-A, See Figure on previous page). Buildings with different mass and stiffness properties will have different periods. For example, in general, taller buildings have longer periods than shorter buildings.

The fundamental period of a building plays a critical role in assessing the expected behavior of the building in an earthquake; therefore, considerable emphasis is placed on it in building codes (for example, the Uniform Building Code).

Although the fundamental period of a building is emphasized in building codes, it is important to note that typical buildings have more than one period of vibration. For example the two DOF system shown below has two periods of vibration, and a different displaced shaped associated with each period of vibration. Although the fundamental period and displaced shape (referred to as the first mode shape) will typically dominate the behavior of the building in an earthquake, for some buildings the second period and mode shape (referred to as a "higher" mode) may also play a critical role.

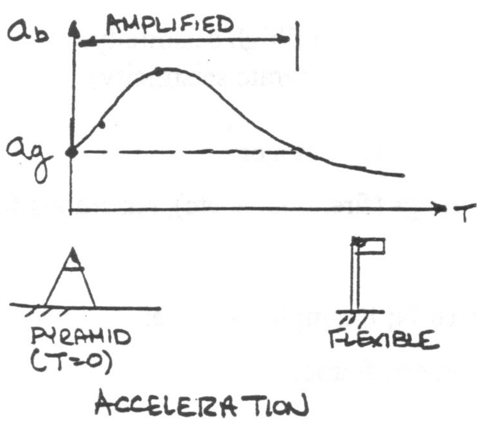


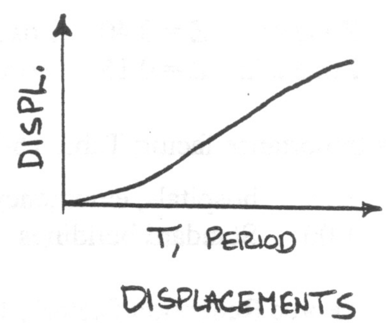
Mathematically, periods and mode shapes for a building are determined by doing an eigen-analysis, which is usually a topic covered in third semester calculus class or a linear algebra course. The eigen-values are the periods of vibration (actually, the eigen-values are usually the frequency of vibration squared, ω^2, which is related to the period of vibration by $T = 2\pi/\omega$), and the eigen-vectors are the mode shapes (the displaced shape associated with each eigen-value).

(d) The earthquake forces on a building are determined based on the seismicity of a region, which is typically based on the history of observed earthquakes and the identification of active faults. Based on this seismicity, expected ground motions are estimated. The expected ground motions are commonly represented by the expected ground accelerations and displacements, as a function of buildings period, as shown on the figure on the next page. For a very stiff structure (such as the pyramid), the maximum acceleration experienced by the structure is equal to the maximum ground acceleration. For moderately stiff buildings, the maximum acceleration experienced by the structure is greater than the maximum ground acceleration. For flexible structure (such as the flag pole), the maximum building acceleration may be less than the maximum ground acceleration. Maximum building displacements generallyincrease with increasing building period.

(e) Design acceleration and displacement for a building can be estimated by computing the periods of vibration and mode shapes. Forces on the building are computed from the design acceleration using Newton's Second Law: Force = mass x acceleration. An analysis of this type is somewhat complex and often requires a basic course in Structural Dymanics. Fortunately, building codes provide simplified approaches.

(f) Relations for acceleration and displacement versus period, based on plotting average relations for many ground motions.





a_b = building acceleration
a_g = ground acceleration

(g) UBC Provisions for Seismic Analysis

UBC provisons are developed based on the concept of **Base Shear**. The **Base Shear** for a building is the horizontal reaction at the base of the building required to balance the inertia forces (F=ma) that develop over the height of the building due to the earthquake.

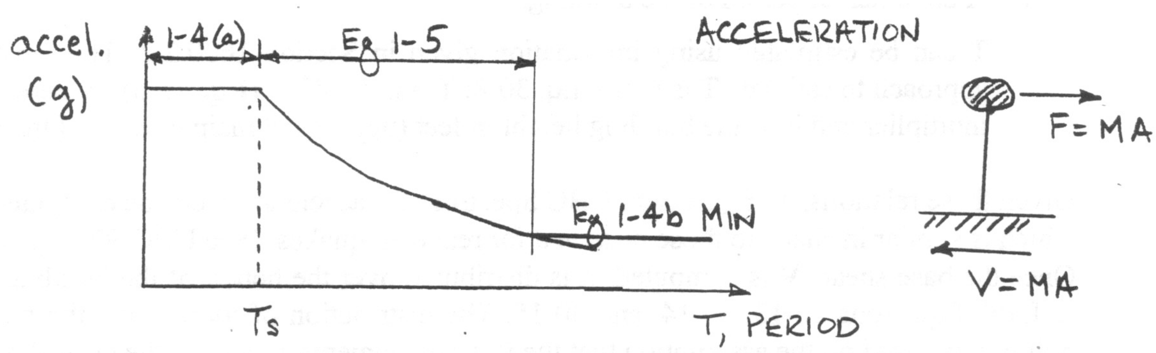
UBC-97 Equation 30-4 and 30-5:

$$V = \left[\frac{C_v I}{R T}\right] W \leq \left[\frac{2.5 C_a I}{R}\right] W$$

$$V \geq \left[0.11 C_a I\right] W \quad \text{and} \quad V \geq \left[\frac{0.8 Z N_v I}{R}\right] W \ \text{ for Zone 4} \qquad \textbf{(Eq. 1-4)}$$

The term in the brackets (for any of the terms) can be manipulated as:

$$V = \left[\frac{C_v I}{R T}\right] W = \left[\frac{C_v I}{R T}\right] M g = \left[\frac{C_v I}{R T} g\right] M \qquad \textbf{(Eq. 1-5)}$$



V = M [ ] = M [acceleration]

Therefore, the term in the brackets for Eq. 1-5 can be considered the building acceleration for design, as a fraction of g, the acceleration due to gravity. The other terms are:

Z = zone factor; Table 16-I Fig. 16-2:

Zone 4:	Z = 0.40	Los Angeles (region of high seismicity)
Zone 2A:	Z = 0.15	Boston (region of moderate seismicity)

I = Importance factor; Table 16-K: Occupancy Category

1.25	hospitals, emergency buildings (fire, police, etc), hazardous facilities
1.00	Standard buildings

R = "Force Reduction Factor"; Table 16-N; Examples include:

8.5	Steel EBF (eccentrically braced frame)
5.5	Concrete shear walls
8.5	Concrete Special Moment-Resisting Frame SMRF (requires special detailing)

The force reduction factor is established based on expected building performance, and has been “derived” based on observed building performance in earthquakes, as well as other factors (analytical and experimental research). In a very general sense, R is a measure of the ability of the building to bend (flex) without collapsing.

C_v = Seismic coefficient (for the velocity controlled region : Table 16-R
C_a = Seismic coefficient (for the acceleration controlled region : Table 16-Q

Values for C_v and C_a depend on the seimic zone factor (Z) and the soil profile type as defined in Table 16-J. Six soil profiles are defined, from S_A to S_F.

In the highest seismic regions (Zone 4), Values for C_v and C_a depend on the seismic source type (Table 16U). Values for C_v and C_a for Zone 4 are multiplied by N_v (Table T) and N_a (Table 16-S), depending on the sesimic source type. Seismic source type is a function of the earthquake magnitude expected for a given fault and the slip rate for the fault. The terms N_v and N_a are referred to as “Near-Source Factors”, and account for the higher ground accelerations expected in regions close to the fault rupture zone.

T = Fundamental period of the building.

T can be estimated using information given in Section 1630.2.2. The most common approach to estimate T is to use Eq. 30-8: $T = C_t (h_n)^{3/4}$ **(Eq. 1-6)** where C_t is a multiplier and h_n is the building height in feet (uppermost main portion of the building).

Given these relations, an Equivalent UBC Spectrum for acceleration can be computed, which which is similar in shape to those observed for real earthquakes. (See UBC-97 Fig. 16-3)
Once the base shear **V** is computed, it is distributed over the height of the building according to UBC Equations 30-13, 30-14, and 30-15. The distribution of forces over the height of the building is based on the assumption that the first (fundamental) mode is the critical mode.

$$V = F_T + \sum_{i=1}^{n} F_i$$

$$F_x = \frac{\left(V - F_T\right) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \textbf{(Eq. 1-7)}$$

$$F_T = 0.07TV \leq 0.25V \qquad T > 0.7\text{ sec}$$
$$F_T = 0.0 \qquad T \leq 0.7\text{ sec}$$

w = weight of story i or x
h = height from the base of the building to story i or x
n = total number of stories
i = story i
x = story x
V = base shear
F_x = force to be applied at story x
F_T = accounts for higher mode effects, and is computed as:

In general, the procedure for determining earthquake forces (base shear and story forces) can be outlined as:

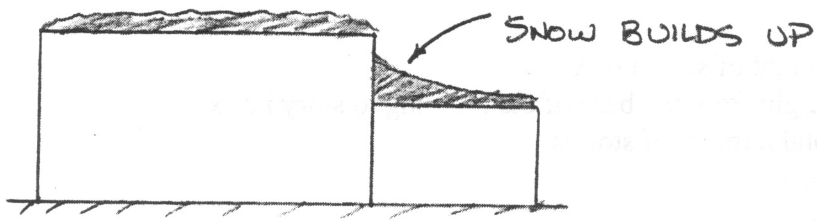
1) Compute w_i, the seismic dead weight for each floor and the roof (all levels). Typically, the seismic dead weight includes only the unfactored dead load; however, in some cases a portion of the unfactored live load may also be included (See Section 1630.1.1, at the bottom of the second column on page 2-13). Add the story values to obtain the total seismic dead load for the building.
2) Compute the base shear, V (based on the computed value of W, and values for Z, I, C_a and C_v, R, S, and T).
3) Compute F_T based on Eq. 30-14.
4) Compute $\sum w_i h_i$, where i goes from 1 to the number of stories. This value is a constant for a building, and is constant for all F_x
5) Compute F_x, the story forces using Eq. 30-15. It is often convenient to make a table or use a spreadsheet for these calculations.
6) Using the story forces, conduct an analysis to determine design loads for each structural element using hand or computer methods.

1.4.5 Snow Loads

Usually specified by local building codes due to significant local variations.

Vary from:	0	psf	(parts of Florida and Texas)
	40-50	psf	(Typical value for Northeast)
	70	psf	(Northern Maine)

Must also consider non-uniform loading effects due to drifting:



Snow loads may be reduced for sloped roofs. UBC gives the following relation (Section 1614):

$$R_s = \frac{S}{40} - \frac{1}{2} \qquad \textbf{(Eq. 1-8)}$$

S = total snow load, psf
R_s = snow load reduction in psf per degree of pitch over over 20^o.

For example, if S = 40 psf, R_s is 0.5. If the roof has a pitch of 30 degrees, then the reduction is (30-20)(0.5) = 5 psf

1.4.6 Comments on Loads

Code specified loads are given for the actual loads expected on a structure, and are typically referred to as "service" loads, that is, the structure will have to serve (function, be usable) under these loads. These loads may need to be increased by using "load factors" to provide a safety margin.

The engineer must use judgement in applying code specified loads. It may be conservative to use code specified loads in some cases, while in other cases, it may by unsafe.

There is uncertainty associated with the magnitude of design loads (eg, gravity loads), and also how loads combine (eg, gravity and wind loads).

The design process must somehow account for this uncertainty.

1.5 Uncertainties in Structural Design

1.5.1) Fundamental Design Objective

Loading Effects (Forces, Deformaitons, etc)	<	Resistance to Load (Strength, stiffness)
Q		R

There are uncertainties in both Q and R

Uncertainties in Q:

(1) Loading, also more uncertainty with some types of loads, ie, we can estimate dead load better than live wind, or snow loads. Even more uncertainty with Earthquake loads.

(2) Loading combinations, e.g., dead load combined with live load and wind load. It is not likely that all three will be maximum at the same time.

(3) Models used for analysis to determine member forces/deformations.

Uncertainties in R

(1) Variation of Member Properties

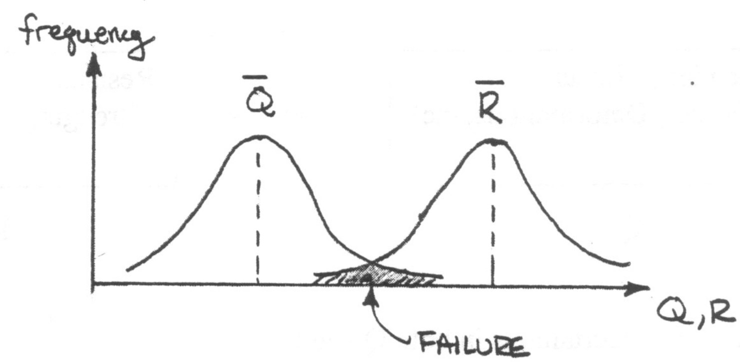
(a) Material properties
(b) Dimensions/imperfections
(c) Residual stresses

(2) Predicting Actual Member Strength

(a) Flexural strength (bending)
(b) Axial load strength (comp and tension)
(c) Torsional strength
(d) Combined flexure and axial load

The design process must take into account these uncertainties.

Graphically, we can depict design uncertainties as:



Objective: $Q < R$

Question: How much larger should $\overline{\boldsymbol{R}}$ be than $\overline{\boldsymbol{Q}}$?

We must have some margin of safety in design, but what is an "acceptable" failure level. It is uneconomical to provide 0% of failure.

1.5.2 Design Formats: ASD vs LRFD

The primary difference between ASD (Allowable Stress Design) and LRFD (Load and Resistance Factor Design) is how the "Margin of Safety" is incorporated into the design procedures.

ASD: For ASD, the "Margin of Safety" is commonly referred to as the "factor of Safety", and is applied to the Resistance, R.

$$f_{calc} \leq F_{allow} \quad ; \quad \text{where } F_{allow} = F_{limit} / F.S. \qquad \textbf{(Eq. 1-9)}$$

$$Q \leq R$$

f_{calc} = calculated stress in member under service loads

F_{allow} = allowable stress in member

F_{limit} = limiting stress (usually F_{yield})

F.S. = Factor of Safety

Design Requirements are given in ACI 318 Appendix A, Alternative Design Method.

LRFD: For LRFD, the "Margin of Safety" is accounted for by applying **load factors** and **resistance factors**, based on the uncertainty associated with each aspect of design.

$$\boxed{\sum \gamma_i Q_i \leq \phi R_n} \qquad \textbf{(Eq. 1-10)}$$

For Loads: See ACI 318, Section 9.1 - 9.3 (and Section 1.5.3 of these notes.)

Q_i = calculated load effect (P,V,M) under service load i (i = dead load, live load, wind load, etc.).

γ_i = "Load Factor". Accounts for uncertainities in Q_i. γ_i depends on i (the type of load) and on the load combination.

For Resistance: (See Section 1.5.4 of these notes).

R_n = Nominal strength or resistance to a particular loading

ϕ = "Resistance Factor". Accounts for uncertainty in R_n. ϕ varies with type of behavior. (flexure, shear, and axial load resistance).

LRFD: Margin of safety is (γ_i and ϕ), and thus is applied to both Q and R.

Read Section 9.3 of ACE 318 and Section 2.1 - 2.6 of MacGregor.

Why Use LRFD

For concrete design, an LRFD approach has been used for some time (available since 1956 in ACI 318 and referred to as Ultimate Strength Design - USD); therefore, there is no significant debate on use of an LRFD approach for reinforced concrete (called USD, or Ultimate Strength Design for concrete). The change from ASD to LRFD for structural steel is currently underway (picked up considerably in the early 90's). You may want to read Section 1.10 of the Salmon and Johnson text for steel design to get additional information of ASD vs LRFD have a better grasp on the margin of safety with a uniform design approach.

Primary incentive to use an LRFD approach is that it provides a more rational approach to design and yields more uniform reliability under different load conditions.

Unusual loads can be dealt with by modifying load factors.

Ultimately there will be a uniform design approach for steel, concrete, masonry, and wood.

1.5.3 Load Factors

Specific load factors and load combinations are given in ACI 318 Section 9.2. However, before applying the loads cases including earthquake, the earthquake force E determined using UBC-97 should be divided by 1.43. This step is needed because ACI load factors are based on service loads and UBC-97 earthquake loads are already factored (Note the load factor of 1.0 used for E in UBC-97, 1612.2.1)

In general, five load combinations must be considered for combined gravity (dead and live), wind and earthquake loads:

ACI 318 Eq.	Load Combination	Load at its Lifetime (50 year) Maximum
(9-1)	1.4D + 1.7 L	Dead load D & live load L
(9-2) (9-3)	0.75(1.4D + 1.7L + 1.7W) 0.9D + 1.3W	Wind Load W
(9-2) (9-3)	0.75(1.4D + 1.7L + 1.87E) 0.9D + 1.43E	Earthquake load E (1.1E for W in Eq. 9-3)
UBC-97: 1612.2.1	Use of ACI load cases for earthquake with additional multiplier of 1.1. See exception 2. Also, in UBC-97, $E=\rho E_h + E_v$, where $E_v = 0.5C_aID$ and ρ is a term to account for redundancy (in this class we will assume $\rho = 1.0$ for all cases). E_v accounts for vertical accelerations, and is grouped with "D" in load cases from Eq. (9-2) and (9-3). For example: $0.9D + 1.43(\rho E_h \pm E_v) =$ $(0.9 \pm 1.43 * 0.5C_aI)D + 1.43(\pm \rho E_h)$	

Load combinations assume that when a particular load is at its lifetime maximum, the other loads will not be at their maximum. Rather, the other loads will be at an "arbitrary point in time value." Snow load S is considered as a roof live load.

Notes:

(1) All five load combinations must be checked

(2) Position L, S for worst effect

(3) W, E can act in any direction (which effectively adds two more load cases)

1.5.4 Resistance Factors

ϕ Depends on type of behavior (failure mode)

$\phi = 0.90$ Flexure, without axial load

$\phi = 0.90$ Axial tension, axial tension and flexure

$\phi = 0.75$ Axial compression with spiral reinforcement

$\phi = 0.70$ Axial compression for other reinforced members

$\phi = 0.85$ Shear and Torsion

1.5.5 LRFD Vocabulary

$$\boxed{\sum \gamma_i Q_i \leq \phi R_n}$$

$\sum\gamma_i Q_i \Rightarrow$ Required strength (Mu, Pu, Vu, etc.)
Maximum member forces under factored loads

$R_n \Rightarrow$ Nominal strength (Mn, Pn, Vn, etc.)

$\phi R_n \Rightarrow$ Design strength (ϕMn, ϕPn, ϕ Vn, etc.)

LRFD Basic Design Equations:

$M_u \leq \phi M_n$

$P_u \leq \phi P_n$ **(Eq. 1-11)**

$V_u \leq \phi V_n$

etc.

1.5.6 Determination or γ_i's and ϕ's

Statistical Analyses

Judgment/Experience

1.5.7 Limit States for Design

A "Limit State" provides a general framework for design

There can be more than one limit state

(1) A limit of structural usefulness (may not be failure)

(2) Failure mode

Design such that no limit state is exceeded with an appropriate margin of safety. Typically, we consider two limit states

(1) Strength (ultimate) limit states

(a) Related to structural safety and maximum load carrying capacity (eg. buckling, fracture)

(b) Use factored loads

(2) Serviceability limit states

(a) Not safety related, but related to use (or function) of the structure (e.g. excessive deflections, vibrations)

(b) Use service (unfactored) loads.