General Principles of Seismic Design

Structural design is the control of phenomena by the derivation of structural systems with appropriate properties, such that the demands on the system (i.e. loads and climatic conditions) are just outweighed by the relevant capacities of the system. Therefore, it is important to be thoroughly familiar with the qualitative behaviour of the structural system under earthquake loading.

In the following, the typical behavior of buildings under earthquakes is discussed, using a framed building as an example, and from the perspective of the maximum design actions and displacement induced and which are of prime concern to the designer. The general principles apply to other types of building construction as well.

Typical Responses to Earthquakes for an Approximately Regular Building

The inertia force acting on a floor is applied through the CG of the floor (Fig 1). However, the CG of the floor does not coincide with the centre of stiffness or rigidity, CR, of the floor (Fig 2). Hence this causes torsion of the floor, in addition to the direct inertia forces (Fig 3).

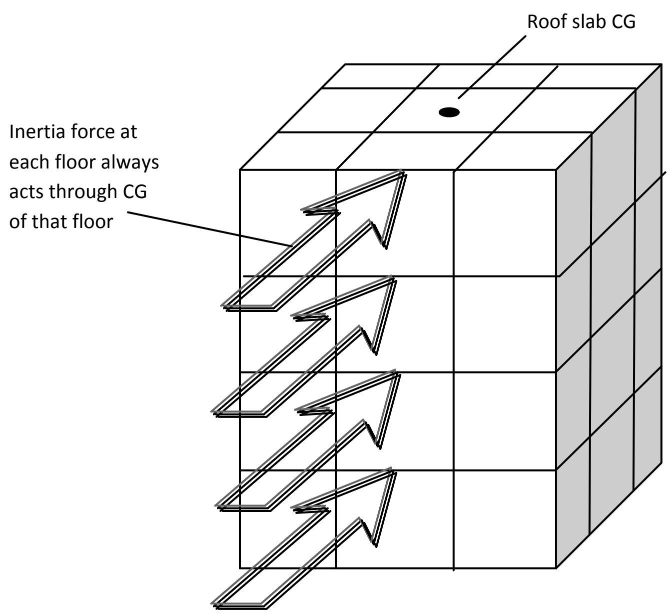
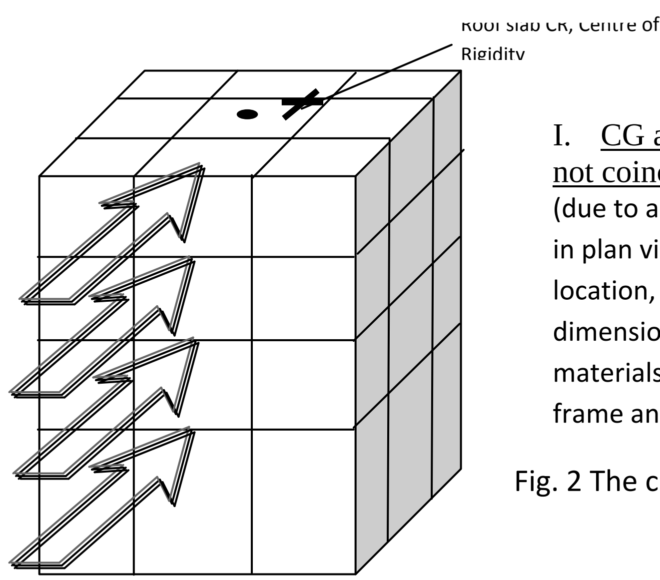


Fig. 1 Distribution of Inertia Forces to floors

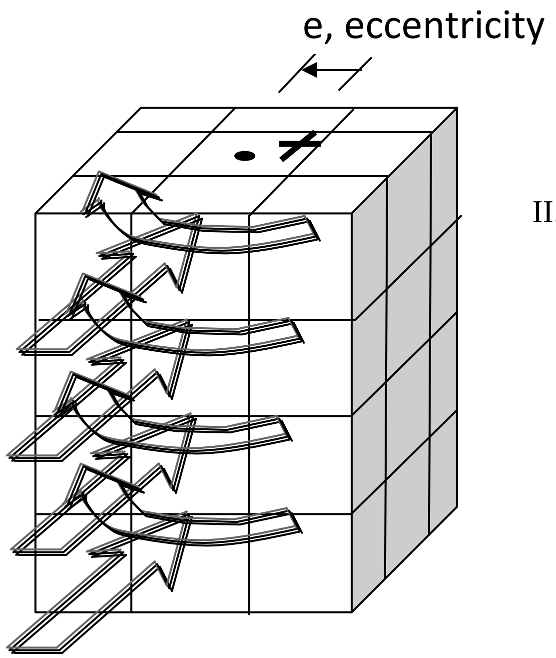


Each floor has a **centre of rigidity or stiffness , (CR)**, relative to a horizontal (lateral) load acting at right angles to the floor.

This is because each floor is supported by frames and/or walls that resist the lateral load.

I. CG and CR do not coincide (due to asymmetry, in plan view, of the location, dimensions, and materials of the frame and/or walls)

Fig. 2 The centre of rigidity concept



II. Inertia force plus torsion

Fig. 3 The generation of torsion on a floor

The building is typically modeled as a space frame, therefore to analyse the building, the inertia and torsion forces are converted to point loads on the frames (Fig.4).

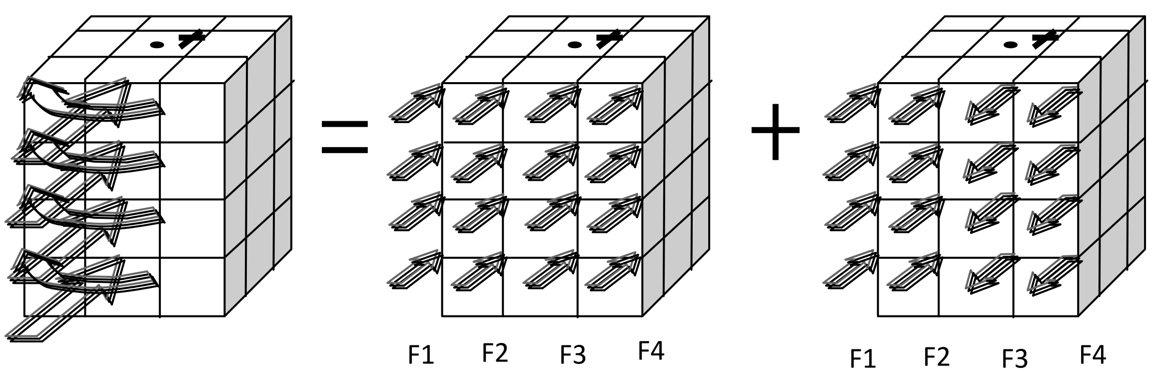


Fig. 4 Horizontal Distribution of Inertial and Torsional forces to Individual Frames or Walls

Analysis of a frame under the earthquake loads gives certain typical patterns of the design actions (moments, shears and axial loads) for the structural elements.

Figs. 5 to 7 shows typical bending moment and shearing force diagrams for beams. Fig 8 shows typical axial load and bending moment diagrams for columns.

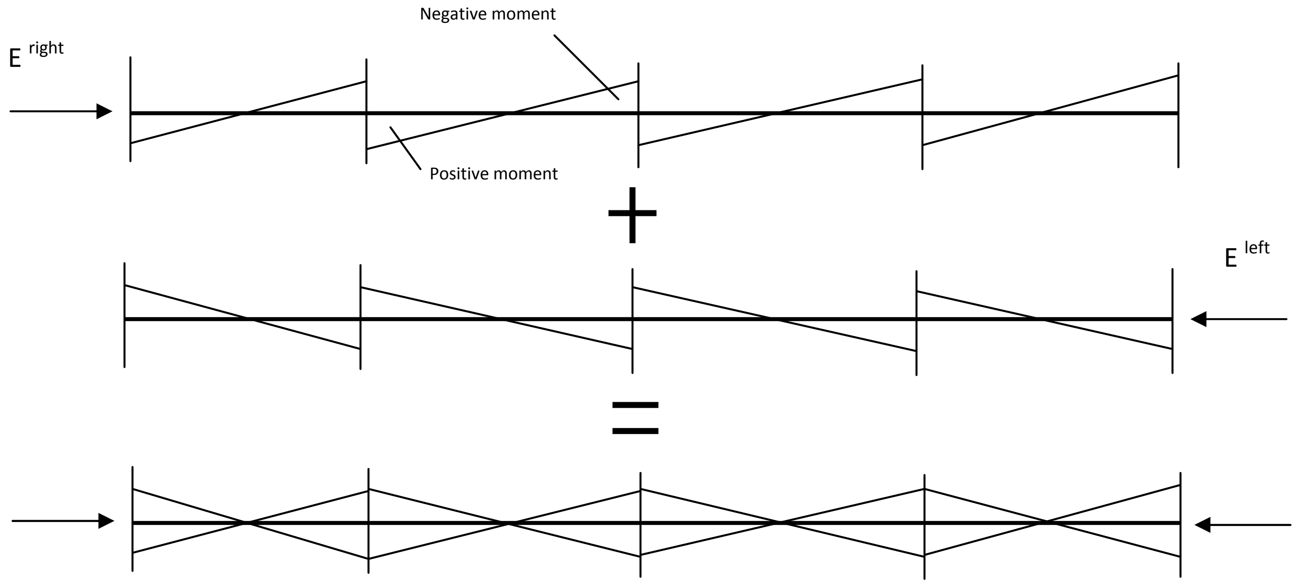


Fig. 5 Beam Moments Under E

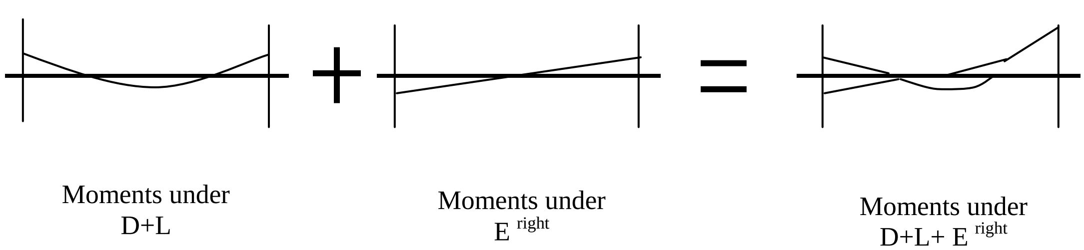


Fig. 6 Beam Moments Under D+L+E ^{right} at the same time

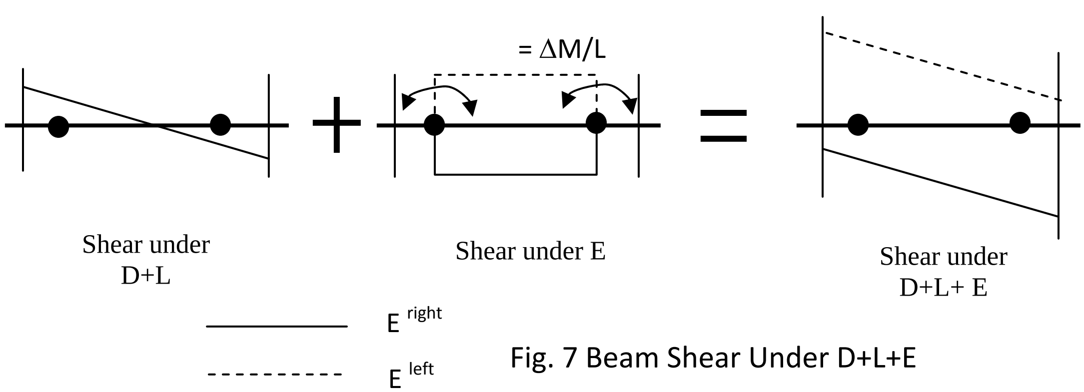


Fig. 7 Beam Shear Under D+L+E

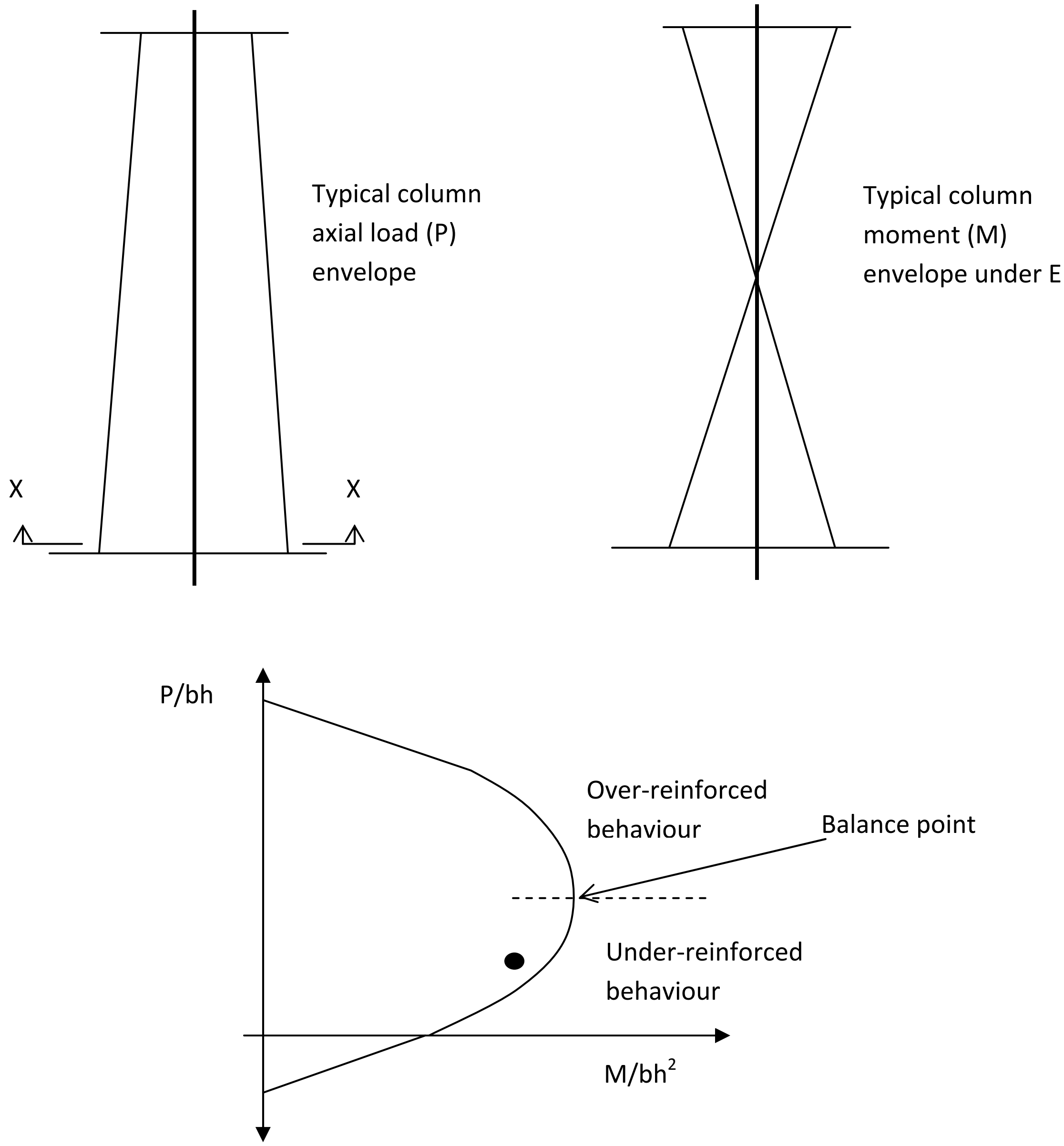


Fig. 8 Column strength interaction curve (for section x-x)

The Effect of Irregularity

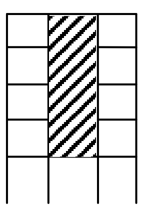
The above consideration of the behaviour of a framed structure under earthquake loads is with respect to an approximately regular building. A regular structure is one whose distribution of mass, stiffness, and strength, is uniform in each direction of the structure. A significant difference in any of these properties results in a significant change in the distribution of the applied earthquake load on the structure, and the magnitude of the resultant of the force as well.

The following is the ASCE 7-05 classification of irregularities. The ASCE 7-05 also quantifies the degree of severity of the irregularity in terms of what is allowed for design. Each type of irregularity is allowed only under certain conditions. The type and extent of the irregularity also affect the type of earthquake load analysis that is allowed for the structure. This is discussed in greater detail in subsequent sections.

Vertical Irregularities:

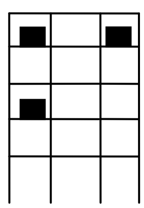
Type 1a,b

Soft Storey
and extreme soft
story



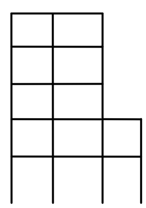
Type 2

Mass Irregularity



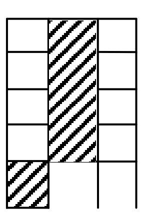
Type 3

Geometry Irregularity



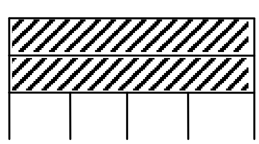
Type 4

In-plane Offset
of some element(s)



Type 5a, b

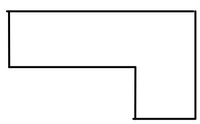
Weak Storey and
Extreme Weak Story



Plan Irregularities:

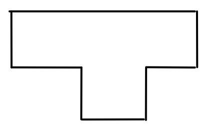
Type 1a,b

Torsional
Irregularity and
Extreme torsional irregularity



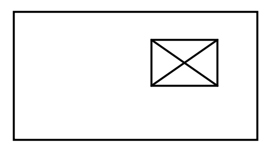
Type 2

Re-entrant Corners



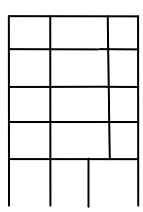
Type 3

Diaphragm Discontinuity



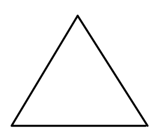
Type 4

Out-of-Plane Offset
(from floor to floor
hence for discontinuous
elements; side view)



Type 5

Non-Parallel Systems



Earthquake Resistant Design Philosophy

The general philosophy of earthquake resistant building design is that:

(a) For minor earthquakes – there should be no damage
(b) For moderate earthquakes – there may be minor, repairable, structural damage and some non-structural damage
(c) For major earthquakes - there may be major, unrepairable, structural and non-structural damage but without collapse of the building.

In terms of objective (c), though the building can be designed to remain in the elastic range of material behaviour, by international consensus it is agreed that allowing unrepairable damage is most economical for the majority of structures. This approach is called design by hysteretic damping. The objective is to allow the structure to enter the inelastic range at certain points, and maximise the energy absorbed by plastic flow. To achieve this, any type of possible brittle failure (e.g. shear failure, bond failure, slip, etc) must be suppressed as much as possible.

This approach is therefore based on the need for ductility in the structural system chosen to resist the earthquake. Some systems (materials plus geometric configuration) are naturally more ductile than others. Each system has a ductility capacity. This capacity is determined, in the case of reinforced concrete and reinforced masonry, by the arrangement of the reinforcement. In the case of structural steel the ductility capacity is determined by the arrangement of the connections, and selection of the section types. Since the ductility demand on the structure is typically not calculated, it is important that the detailing be consistent with the response modification factor used to determine the base shear.

Other approaches to earthquake resistant include base isolation, and the use of supplemental damping devices.

In terms of objectives (a) and (b), under more frequent but less severe earthquakes, excessive damage to the secondary elements such as partitions, is controlled by specifying limits on the horizontal displacement of the floors, called the inter-storey drift – the ratio of the story's inelastic lateral displacement to the story height. Controlling the lateral displacement is also needed to minimize instability due to the P-Δ effect.

<u>Earthquake Resisting Structural Systems</u>

An earthquake-resistant structural system is a structural system with properties and behaviour that are favourable towards the objective of adequately resisting earthquakes. Moment-resisting frames, braced frames, walls, and combinations of these (called dual and building frame systems), are typically used.

Given that the main earthquake-resistant design philosophy is the use of the phenomenon of hysteretic damping to resist earthquakes, this implies that a main desirable property of the system is high ductility. Systems comprised of certain materials and methods of construction, naturally possess better system ductilities than others. An overall measure of the system allowable or ultimate ductility is the IBC 2009 (ASCE 7-05) response modification factor, R.

The other desirable properties of earthquake-resistant structural systems which promote high ductility and overall favourable seismic response are:

- regularity - little change in stiffness, mass, and strength from floor to floor, and in the two dimensions in plan as well
- continuous load path – the absence of gaps between members so that the force is effectively transferred from each member to its successive member or members on its way from the diaphragms to the foundation
- short load path - small (if any) offsets of beams, columns and walls
- multiple load paths (i.e. redundancy) – the presence of several routes that the force can take on its way from the diaphragms to the foundation; in this way if a member is stressed to its capacity, the other members can be relied on to absorb the energy
- strong connections.- to ensure that the load path is not broken by excessive deformation or rupture of a connection.

When these factors are maximized, the sequence of the formation of the hinges and their distribution are such that the energy absorption in the system as a whole is maximized.

The following is an example of the desirable hinge mechanism for ductile moment-resisting frames.

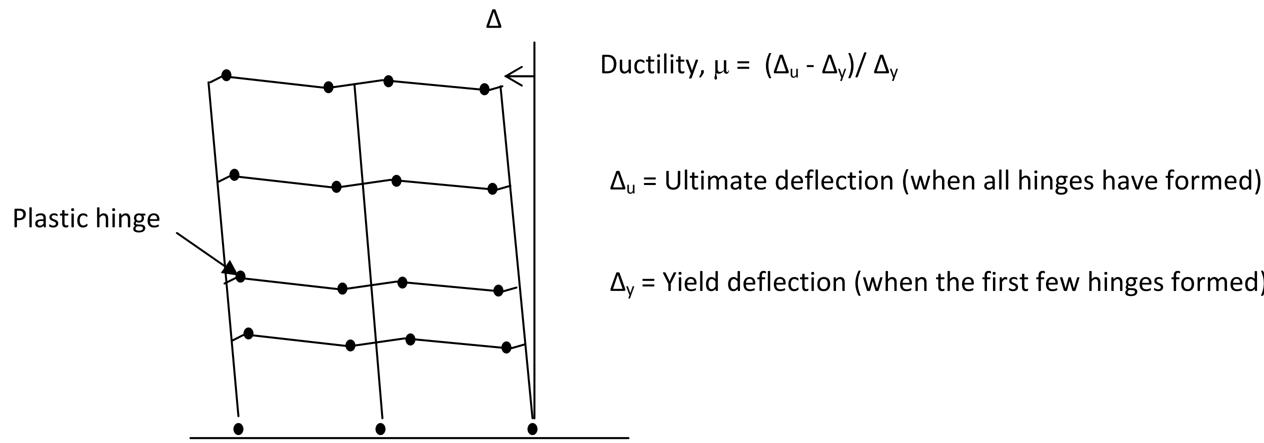
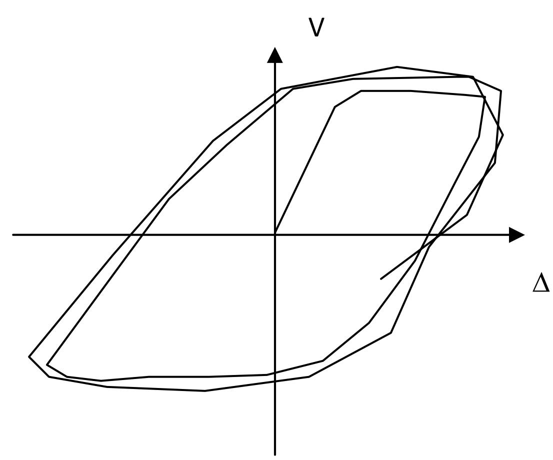


Fig. 9 Example of suitable plastic hinge formation in a frame

Though ductility is typically stated as the key requirement for earthquake resistant structural systems, it must be emphasized that this is due to the high energy absorption under dynamic conditions that results when the system has high ductility. Therefore to be more specific, the primary parameter is the energy absorption. This is displayed by the load-displacement hysteresis loops of the structural system as it undergoes the dynamic motion into the inelastic range. A typical example for regular structural steelwork ductile moment-resisting space frames is as follows. This system displays the maximum possible energy absorption of all present available earthquake resistant structural systems. The energy absorption is equivalent to the total area of all the hysteresis loops. The shape of the typical loop shown is called the "spindle" shape, and is considered the ideal loop shape. During the earthquake, the hysteresis loops of the plastic hinge change, and this affects the subsequent behaviour of the entire structure, during the same earthquake.



General Comments:

- All frame elements must be detailed so that they can respond to strong earthquakes in a ductile fashion. Any elements which are necessarily incapable of ductile behaviour must be designed to remain elastic at ultimate load conditions.

- Non-ductile modes such as shear and bond failures must be avoided. This implies that anchorage and splices of rebars should not be done in areas of high concrete stress, and a high resistance to shear should be provided.
- Rigid elements should be attached to the structure with ductile or flexible fixings.
- As many zones of energy-absorbing ductility as possible should be provided before a failure mechanism is created. For framed structures this means that the yielding should occur first in the beams, then in the columns (weak beam-strong column).
- Movement joints should be provided at discontinuities so that pounding is avoided.

Overall Process for the Seismic Design of a New Building

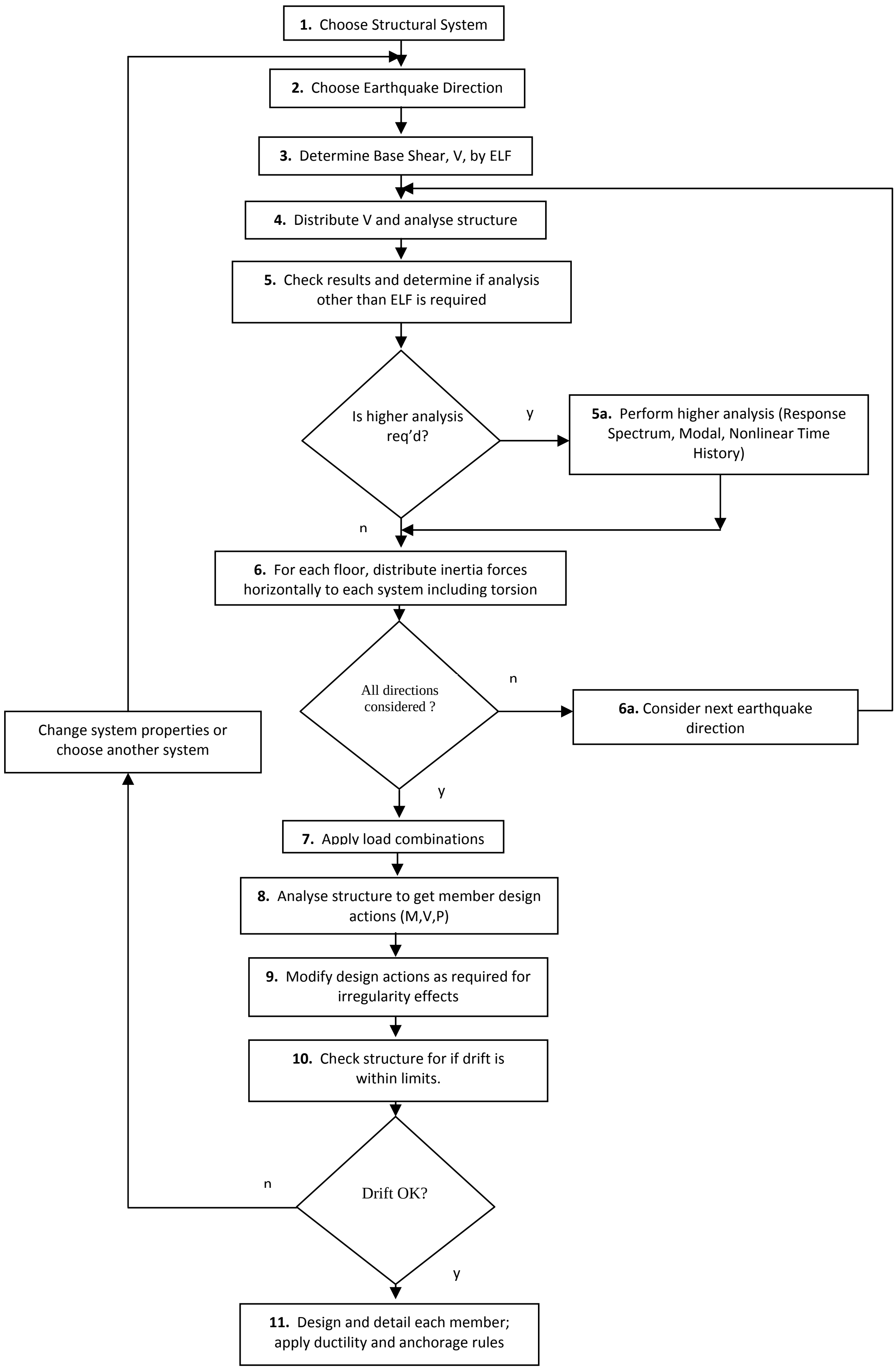
The following points summarize the phenomena associated with buildings under earthquake loading:

1. The loading is based on Newton's Second Law hence is inherently time-varying or dynamic. Engineers design the structure to accommodate the peak response quantities. However, the loading is also inherently random so there is considerable uncertainty in the quantities used for design.
2. The most common design approach is to induce plastic failure at pre-determined locations. However during the earthquake, the hysteresis loops of the plastic hinge change, and this affects subsequent behaviour during the same earthquake.
3. Therefore, the most important property of a seismic structural system for favourable performance is its overall dynamic displacement capacity.
4. Successful design (apart from economic and aesthetic considerations) requires that the structure not collapse under a severe earthquake, and that the partitions, glazing, ceilings, and other non-structural elements, not collapse under a moderate earthquake.

These phenomena and overall requirements make seismic design a distinct discipline. It cannot be considered an extension of wind load design with which it shares only the fact that both loads are predominantly lateral loads.

The overall process for the seismic design of a new building is as indicated on the following sheet. Of note is that the seismic loading must be estimated early on, as the base shear V, in order to determine if a higher type of analysis is required, and for determining if the structure is irregular. The determination of the base shear V by use of the code equation for V is called the equivalent lateral force (ELF) method and is only valid for regular or approximately regular buildings.

OVERALL PROCESS FOR THE SEISMIC DESIGN OF A NEW BUILDING



SEISMIC DESIGN OF REINFORCED CONCRETE BEAMS AND COLUMNS OF SPECIAL DUCTILE MOMENT RESISTING FRAMES

Factors Affecting Behaviour

For practical values of section size and reinforcement:

Section ductility capacity is increased for:

- An increase in the compression reinforcement
- An increase in concrete compressive strength
- An increase in ultimate concrete strain

Section ductility capacity is decreased for:

- An increase in tensile steel reinforcement
- An increase in steel yield strength
- An increase in axial load

General Materials Requirements

Concrete Quality:

- The minimum recommended characteristic cylinder crushing strength is 20 MPa but less than 27 MPa for lightweight concrete

Reinforcement Quality:

- Suitable quality must be ensured by both specification and testing.
- An adequate minimum yield stress may be ensured by specifying steel to an appropriate standard, such as BS4449 or ASTM A615 or A706.
- The actual yield stress should not exceed the minimum specified yield stress by more than 124 MPa.
- Grades of steel with characteristic strength in excess of 415 MPa should not be used.

Empirical Beam Design Rules (ACI 318-02 Ch. 21)

1. b/h shall not be less than 0.3 (b, total beam width; h, total beam depth).
2. b shall not be less than 250mm.
3. b shall not be greater the column width plus 0.75h on each side.
4. The minimum longitudinal steel content as a fraction of the gross cross-sectional area of the web shall be $1.4/f_y$ (N/mm^2) or $200/f_y$ (psi).
5. The maximum longitudinal steel content as a fraction of the gross cross-sectional area of the web shall be 0.025.
6. The positive moment strength at the beam-column joint face shall not be less than one-half of the negative moment strength provided.

7. At any section in the beam span, neither the negative nor the positive moment strength shall be less than a quarter of the maximum moment provided at the face of either beam-column joint.
8. Lap splices shall not be used: within joints; within 2h from the face of the beam-column joint, at locations of potential plastic hinging.
9. Lap splices where used shall be confined by hoops or spiral reinforcement with a maximum spacing or pitch of d/4 or 100mm (d, effective depth to main steel).
10. Transverse reinforcement in beams must satisfy requirements associated with their dual function as confinement reinforcement and shear reinforcement.
11. Confinement reinforcement in the form of hoops is required: over a distance 2d from the column face, over distances 2d on both sides of sections within the span where flexural yielding may occur due to earthquake loading.
12. The first hoop shall be 50mm from the column face and the maximum hoop spacing shall be the smallest of d/4; 8 times the diameter of the smallest longitudinal bar; 24 times the diameter of the hoop bar, or 300mm.
13. Where hoops are not required, the hoop spacing shall be less than d/2.
14. Shear reinforcement is to be provided so as to preclude shear failure prior to the development of plastic hinges at the beam ends. Design shears for determining shear reinforcement are to be based on a condition where plastic hinges occur at beam ends due to the combined effects of lateral displacement and factored gravity loads. The probable flexural strength associated with a plastic hinge is to be computed using a strength reduction factor of 1.0 and assuming a stress in the tensile reinforcement of 1.25 $f_{y.}$ Note that the hoop reinforcement may satisfy the shear steel requirements and vice versa.
15. In determining the required shear reinforcement, the contribution of the concrete is to be neglected if the shear associated with the probable flexural strengths at the beam ends is greater than one-half the total design shear, and the factored axial compressive force including earthquake effects, is less than $A_g f_c/20$.

Seismic RC Beam and Column Design Steps

Drift Limits

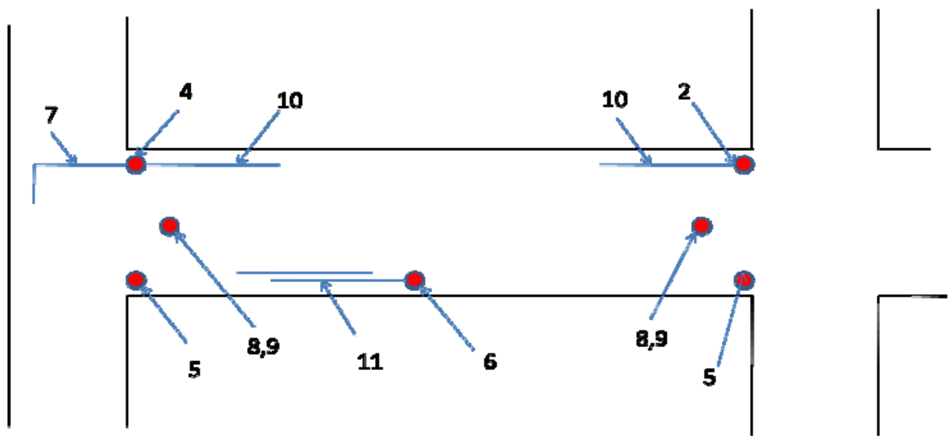
Limitation of the storey drift is a major design requirement as indicated in the flowchart. This is required in order to limit damage to non-structural components as well as reduce the likelihood of instability due to P-Δ effects.

For concrete framed structures the drift of each storey shall not exceed $0.025h_{sx}$, $0.020h_{sx}$, $0.015h_{sx}$, for structures in Occupancy Categories I and II, III, and IV respectively, where h_{sx} is the storey height below level x.

The design story drift for a storey is calculated as the difference of the deflections at the top and bottom of the storey under consideration. The deflection is determined from the results of a structural analysis of the model under the design seismic forces, but increased to cater for the amplification of the ground motion. The stiffness properties of reinforced concrete elements shall consider the effects of cracked sections. The specific values are not stated in the relevant U.S documents but the following is recommended. Consider the effect of cracking as a reduced moment of inertia of the section such that for beams, use a value of $0.35I_g$ and for columns, use a value of $0.5I_g$, where I_g is the moment of inertia of the gross section.

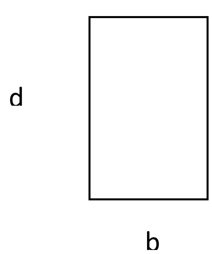
The deflection at level x at the centre of the mass, $\delta_x = C_d\ \delta_{xe} / I$, where δ_{xe} is the deflection determined by an elastic analysis, and I is the importance factor for the structure.

Beam Design Procedure



The following procedure is for the end span of a continuous beam, but is also applicable to internal spans (with some steps unnecessary). The numbers in sketch above refer to the step numbers below.

1. Check proposed beam dimensions limits:



- $b/d \geq 0.3$
- $b > 250$mm
- $b <$ column width + 1.5d

2. Calculate $^{-}A_s$ (Top Main Rebar) at Internal Supports (i.e. Section at Inner end of Beam Span):

   $a = A_s\, f_y / (0.85\, f_c'\, b)$;a = depth of compression block; f_y = rebar yield stress; f_c' = 28-day compressive strength of concrete cylinder

   $M_u < \phi M_n = \phi\, A_s\, f_y\, (d - a/2)$

   M_u = applied negative moment from structural analysis moment envelope; ϕ = strength reduction or construction quality factor = 0.9; d = effective depth to rebar (or centroid of a group of rebars).

You may substitute "a" in "M_u" and solve the quadratic equation for A_s.

3. Check Limits on the A_s calculated in step 2:

   $\rho = A_s/bd > \rho_{min} = 200/f_y$; f_y in psi units
   $\rho > 3\sqrt{f_c'}/f_y$
   $\rho < 0.025$

4. Calculate , ${}^{-}A_s$ at External Supports and Check Limits:

   Same as steps 2 and 3 but with M_u for external supports.

5. Calculate the Minimum Positive Moment Strengths that must exist at the Internal and External Supports:

   In each case, substitute the A_s provided from steps 2 and 4, in the moment equation of step 2 to get the negative moment strength.

   In each case, the minimum positive moment strength that must exist at the section, ${}^{+}M_n = \phi M_n/2$

6. Calculate ${}^{+}A_s$ (Bottom Main Rebar) Near Beam's Mid-Span:

   Same as steps 2 and 3 but with M_u as applied positive moment from the structural analysis results. Substitute the A_s provided in the moment equation to get the positive moment strength near mid-span, ${}^{+}M_{n,\ mid\text{-}span}$.

   Compare with results of step 5; if ${}^{+}M_{n,\ mid\text{-}span}$ > the moments calculated in step 5, then simply extend the ${}^{+}A_s$ (botton rebar) at mid-span to the internal and external supports. If not, additional bottom rebar will be needed at any support that has too small a ${}^{+}M_n$.

7. Calculate and Check the Required Anchorage Lengths for the Main Rebars that End in the External Columns, l_{dh}:

   For standard 90 deg hooks, and for both the top and bottom bars, l_{dh} is the larger of:

   $l_{dh} > f_y d_b/(65\sqrt{f_c'})$;use imperial units; d_b is the rebar diameter
   $l_{dh} > 8d_b$
   $l_{dh} > 150mm$

   Check the length of the rebar in the proposed beam-column joint zone (from inside edge of column to end of bar including curved part) and ensure that this length > l_{dh}.

8. Calculate Seismic Shear Rebar:
   The seismic shear is the shear in the hinges and is typically higher than the

elastic shear of the structural analysis results.

- Calculate the gravity load when the earthquake occurs, $w_u = 1.2w_D + 1.6w_L$
  where w_D is the dead load in force per unit length that the beam carries, and
  w_L is the live load (in force per unit length).

- Calculate the seismic shear:

  $V_u = w_u l/2 \pm (M_{pL} + M_{pR})/l$

  M_{pL}, M_{pR} is the relevant worst-case pair of ϕM_n strengths at the left and right ends of the beam considering when the earthquake is pushing to the right then to the left. They are calculated as done before but with $\phi = 1$ and f_y is 25% higher that the one used before, along with the actual A_s provided. l is the beam's length.

-Calculate V_c:

  If $(M_{pL} + M_{pR})/l > V_u/2$ then $V_c = 0$

  If $(M_{pL} + M_{pR})/l \leq V_u/2$ then $V_c = 2\sqrt{f_c'}\, b_w d$; b_w is the width of the beam's web.

-Calculate ϕV_s:

  $\phi V_s = V_u - \phi V_c$

-Calculate the stirrup spacing, s:

  - Select a stirrup rebar size and f_y and calculate its cross-sectional area for 2 legs, A_v
  - $s = A_v f_y d / V_s$

9. Check Stirrup Spacing:
   In the confinement zone (2h from the column face and where the seismic shear steel must be placed):

   $s < s_{max}$ which is the smallest of:

   = d/4
   = 8 x diameter of smallest main rebar
   = 24 x stirrup diameter
   = 300mm

   Outside the confinement zone, $< s_{max} = d/2$

10. Calculate the Cut-Off Points for the Top Steel:
    Do this for some of the internal and external supports rebars.

    In each case, develop an equation for the variation of the negative moment with distance from the support. Equate this to the moment strength of the section after the rebars are cut. Solve the equation for the distance, then add the development length for the rebar or rebars.

11. Calculate Splice Lengths:
    In case splices are required their length (Class B) is 1.3 l_d , where l_d is given by ACI 318-02 clause 12.15.2.

Empirical Column Design Rules

1. The shorter cross-sectional dimension shall be greater than or equal to 300mm.
2. The ratio of shorter dimension to the perpendicular dimension shall be greater than or equal to 0.4.
3. The maximum and minimum longitudinal steel content as a fraction of the gross cross-sectional area shall be 0.06 and 0.01.
4. For all members framing into a beam column joint, the sum of the flexural strength of the columns (for the relevant axial load level) must be greater than 1.2 times the sum of the flexural strength of the beams.
5. Lap splices are to be used only in the middle half of the column.
6. As in beams, transverse reinforcement in columns must be provide confinement to the concrete core and lateral support to the longitudinal bars, as well as shear resistance. In columns however, the transverse reinforcement must all be in the form of closed hoops or continuous spiral reinforcement. Sufficient reinforcement should be provided to satisfy the requirements for confinement or shear, whichever is larger.
7. Confinement requirements:

   For spiral reinforcement or circular hoop reinforcement, the volumetric ratio must be greater than,

   $0.12\, f_c^{'} / f_{yh}$ or

   $0.45[(A_g/A_{ch}) - 1]\, (f_c^{'} / f_{yh})$

   f_{yh} is the specified yield strength of transverse reinforcement, A_{ch} is the core area of column section measured to the outside of transverse the transverse reinforcement in in^2.

   For rectangular hoop reinforcement total cross-sectional area within spacing s, must be greater than,

   $0.09sh_c\, f_c^{'} / f_{yh}$ or

   $0.3\, sh_c\, [(A_g/A_{ch}) - 1]\, (f_c^{'} / f_{yh})$,

   h_c = cross-sectional dimension of column core, measured centre-to-centre of confining reinforcement.

8. The maximum hoop spacing shall be the smallest of: quarter the smaller cross-sectional dimension, or 100mm.
9. The hoop reinforcement is to be provided over a length l_0 from each joint face, where l_0 is the largest of: d, one-sixth the clear span of the member, or 450mm
10. Transverse reinforcement for shear in columns is to be based on the shear associated with the largest nominal moment strengths at the column ends (using f_y and $\phi = 1$) corresponding to the factored axial compressive force resulting from the largest moment strengths.
11. Generally, it will be necessary to provide multiple stirrups, or stirrups and cross-ties, in order to give satisfactory confinement and restraint to main column reinforcement. Generally, overlapping hoops are to be preferred. In either case, one stirrup should surround the whole of the main

reinforcement. Where restrained bars are less than 200mm apart, it is not necessary to restrain intermediate bars.

<u>Column Design Procedure</u>

1. Calculate the column's main rebar as usual, using the actions (M, V, P) from the structural analysis of the structure including the earthquake load combinations.

   If the factored axial force in the column, $P_{max} > A_g f_c'/10$ then the member is classified as a column and the following seismic requirements are necessary (A_g is the gross cross-sectional area). If not, then the member should be designed as a beam.

2. Check the Limits on the column's dimensions and on the Main Rebar:
   b is the smallest cross-sectional dimension measured on a straight line passing through the geometric centroid of the section, and h is the dimension perpendicular to b.

   - b and h > 300mm
   - b/h > 0.4

   - $\rho = A_s/A_g > 1\%$
   - $\rho < 6\%$

3. Check for Strong-Column-Weak-Beam Behaviour:

   In each direction at the column's joints with the beams:

   $\sum M_c > (6/5) \sum M_b$

   Where M_c is a column's moment strength, and M_b is a beam's moment strength.
   A column can have 2 beams at the joint in the case of an internal column, or 1 beam as in the case of a side column.

   The column strength interaction curve can be used to determine the column's moment strength at the section in question.

4. Calculate the Transverse Rebar for Confinement Hoops:
   This requirement is based on the fact that a concrete core confined by rebar increases the ultimate compressive strain of the concrete, hence increasing the ductility of the section.

   - Determine the length of the confinement zone, l_0 from the bottom of the column where it meets the slab, and from the top of the column but beginning from the beam's soffit. If the length of the column between these sections (i.e. the clear height) is h,

     l_0 is the larger of h/6, the depth of the member at the joint face, or 450mm.

     Carry the confinement rebar through the beam region and the beam-column joint.

   - Calculate the cross-sectional area of confinement rebar required:

If rectangular confinement rebar of total cross-sectional area, A_{sh} , is to be used then for each direction of the column:

Assume a bar size and longitudinal spacing, s. Check that it is within the following limits:

s_{max} is the smallest of:

s_{max} = b/4

= 6 x diameter of smallest longitudinal rebar

s_x = 100 + ((350 – h_x) /3)   100mm < s_x < 150mm

where h_x is the maximum horizontal spacing of hoop or cross-tie legs on all faces of the column.

Try different combinations of confinement rebar size and longitudinal spacing, s, until the following equations are satisfied. Note that additional cross-ties are typically required to satisfy the equations.

Use the larger of:

$A_{sh} = 0.3\ s\ h_c\ ((A_g/A_{ch}) – 1)\ \ (f_c'/f_{yh})$

$A_{sh} = 0.09\ s\ h_c\ (f_c'/f_{yh})$

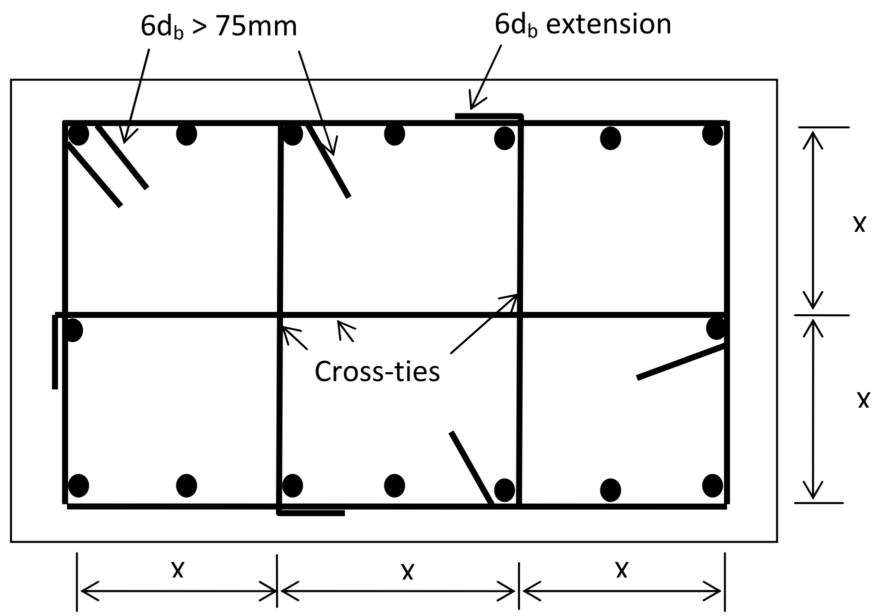
where,

A_{ch} = cross-sectional area of the column core measured out-to-out of the transverse reinforcement

h_c = cross-sectional dimension of the column core measured centre-to-centre of the confining reinforcement

f_{yh} = yield strength of the confinement rebar

Place the confinement rebar is accordance with the following where X < 350mm.



5. Calculate the Transverse Rebar for Shear:

- Calculate the ultimate seismic shear in the hinge, V_u as

$V_u = 2M_u/l$

Where l is the column's clear span. Use the column's strength interaction chart and determine M_u as the moment strength at the balance point.

M_u need not exceed the beam moments transferred to the column given the equilibrium of the beam-column joint.

-Calculate ϕV_s:

$\phi V_s = V_u - \phi V_c$

where $V_c = 2\sqrt{f_c'}\, bd$

-Calculate the stirrup spacing, s:

- Select a stirrup rebar size and f_y and calculate its cross-sectional area for 2 legs, A_v
- $s = A_v f_y d / V_s$

The same confinement rebar typically exceeds the shear resistance requirement. But areas outside the confinement may require checking.

6. Calculate the Rebar Splice Lengths.

SEISMIC DESIGN OF STEEL SPECIAL DUCTILE MOMENT RESISTING FRAMES

The following calculations are suitable in the case where:-
(1) The beams and columns are of hot-rolled I-section members
(2) The columns are laterally braced by beams with end plates
(3) The sections are uniform along the beam's length
(4) There are not any column or similar loads within the beam's span

Step 1. Select Beam Sizes:

This step is typically controlled by the drift (i.e. stiffness) requirements. Nevertheless, also compare against sizes obtained as you would in the non-seismic case but including the M and V from the seismic load cases. Use the larger of the two.

Step 2. Check Beam for Local Buckling Stability:

Beam flanges b/t, max $\lambda_{ps} = 0.3\sqrt{(E/F_y)}$; beams web $h/t_{w,}$ max $\lambda_{ps} = 2.45\sqrt{(E/F_y)}$. Note: $b/t = b_{flange}/(2t_{flange})$; re-do step 1 if check fails.

Step 3. Check Unbraced Length of Beam Flanges:

The unbraced length of the beam must be $\leq 0.086 r_y E/F_y$.
Note: It is typical to use composite deck flooring in which case the unbraced beam length is the spacing of the secondary beams supporting the floor.

Step 4. Determine Minimum Required Column Size:

Use factored loads or analysis results for the D+L+E load case.
Assume location of beam plastic hinge is 1.6 times beam total depth from the column center-line, ie.$x=1.6d_b$

$\sum M^*_{pc}/\sum M^*_{pb} \geq 1$ (1)
$\sum M^*_{pb} = \sum (1.1R_yM_p + M_v)$ (2)

R_y = 1.5 for ASTM A36M but 1.1 for ASTM A572M; Mp = plastic moment of beam = ZF_y
$M_v = V_px$
$V_p = 2M_p/(L-2x) + w(L-2x)/2$, where L is the beam's length to the columns' center lines, and w is the factored gravity load on the beam (force per unit length).
$\sum M^*_{pc} = \sum (Z_cF_{yc})$, where "c" refers to the column. (3)

With Z_c as the unknown, substitute (3) and (2) into (1) and solve for Z_c; select an appropriate section from the table.

Step 5. Check the column for local bucking stability:

Column flanges b/t, max $\lambda_{ps} = 0.3\sqrt{(E/F_y)}$; column web $h/t_{w,}$ max $\lambda_{ps} = 61.06$

Step 6. Check column strength under each of factored D+L+E, and D+L load cases:

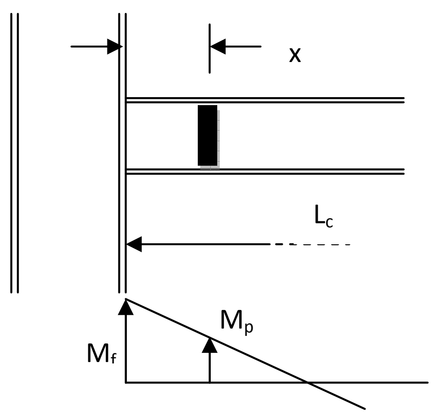
Typically, the beam hence column sizes are controlled by the drift requirements rather than the demand-to-capacity ratios for the design actions M, P, V. Nevertheless, still check the unity equations at critical sections. $\phi_c = 0.85$; $\phi_b = 0.90$

Calculate $\lambda_c = 1.1L\sqrt{(F_y/E)}/(\pi\, r_y)$, L is the column length between the beams, and let $Y = \lambda_c^2$.
Calculate $F_{cr} = 0.658^Y \times F_y$
$\phi_c P_n = \phi_c F_{cr} A$
$P/\phi_c P_n \geq 0.4$, then in the following, use the overstrength factor Ω_0 in the determination of the design P.
If $P/\phi_c P_n < 0.2$:
Check $P/(2\phi_c P_n) + (M_z/\phi_b M_{nz}) + (M_y/\phi_b M_{ny})$ which must be ≤ 1. If not, choose a column section with larger Z.
If $P/\phi P_n \geq 0.2$
Check $P/(\phi_c P_n) + 8/9[(M_z/\phi_b M_{nz}) + (M_y/\phi_b M_{ny})]$ which must be ≤ 1

In these equations P is the applied factored axial load, M_z is the applied factored moment in the column major-axis direction, and M_{nz} is the column moment strength = $Z_c F_{yc}$.

Step 7. Check the beam-column panel zone:

Assuming that the column axial load, $P \leq 0.75P_c$ ($=F_yAg_g$), calculate the panel zone nominal shear strength, R_v as (imperial units)



$R_v = 0.6F_yd_ct_p[1 + 3b_{cf}t_{cf}^2/(d_bd_ct_p)]$ where d_c is the overall column depth, t_p is the thickness of the panel zone including doubler plates, b_{cf} is the width of the column flange, t_{cf} is the thickness of the column flange, and d_b is the overall beam depth.

Select t_p so that
$\phi R_v = R_u$ where R_u is the ultimate applied panel shear determined as follows:

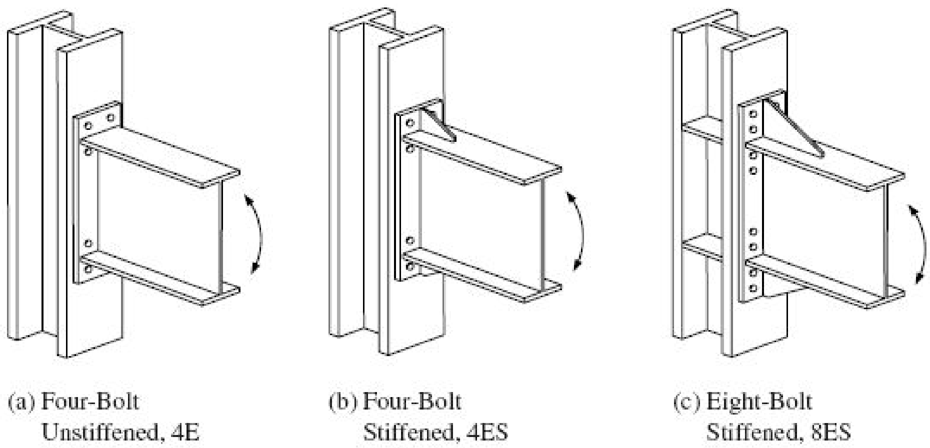
$R_u = SM_f/(d_b-t_{fb})$ where M_f is the moment at the column face determined by projecting the expected moment at the plastic hinge points to the column faces. Hence $M_f = R_yF_yZ[L_c/(L_c - 2x)]$

If a doubler plate is required in order to achieve the calculated required panel zone thickness, t_p, then the plate(s) must be plug welded to the column web if, for either the plate or web,

$t > (d_z + w_z)/90$ where d_z is the depth of the panel zone between the continuity plates, and w_z is the width of the panel zone between the column flanges

This is needed to avoid diagonal compression buckling of the plate.

Note on Bolted Seismic Connection Configuration



Beam-to-column connections used in the seismic lateral load resisting system shall satisfy the following 3 requirements:-

1. The connection shall be capable of sustaining an interstory drift angle (interstory lateral displacement divided by story height) of 0.04 radians.
2. The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.8M_p$ of the connected beam at an interstory drift angle of 0.04 radians.
3. The required shear strength of the connection shall be determined using the following quantity for the earthquake load effect: $E = 2(1.1R_yM_p/L_h)$; R_y is the ratio of expected to minimum specified yield stress, and L_h is the distance between the beam's plastic hinges.

The following is a free website with comprehensive data on American steel sections - http://www.structural-drafting-net-expert.com/steel-beam.html.