SEISMIC DESIGN CONFIGURATIONS

BY KARZAN ZAHER SH.

ئەم بابەتە کۆکراوەی چەند سەرچاوەیەکە بە مەبەستی بەرزکردنەوەی پلەی ئەندازیاری لە ڕێگەپێدراوەو بۆ ڕاوێژکار

Karzanz1976@gmail.com

Introduction

The aim of this paper is to bring out the main contributing factors which lead to poor performance during the earthquake and to make recommendations which should be taken into account in designing the multistoried reinforced concrete buildings so as to achieve their adequate safe behavior under future earthquakes.
The content of this papers is based upon that view of an architect's role in seismic design. An architect should have the skills to conceive the structural configuration at the preliminary design stage that not only satisfies programmatic requirements and his or her design ideas, but is structurally sound especially with respect to seismic forces. Subsequent to this conception of structure, and ideally during that preliminary design process, structural engineering collaboration is indispensable. Ideally a structural engineer with specialist technical skills - and a sensitivity towards architectural aspirations - works alongside the architect to develop and refine the initial structural form. The engineer, designing well beyond the technical abilities of the architect then determines member sizes and attends to all the other structural details and issues.
Seismic resistant design is intended to achieve two objectives:

- Protect human lives, and
- Limit building damage.

The first objective is achieved primarily by the provision of adequate strength and ductility. This ensures that a building is protected from full or partial collapse during large earthquakes that occur infrequently.
The second objective limits building damage during lesser, more frequently occurring earthquakes, in order to minimize economic losses including loss of building functionality.

NATURE OF SEISMIC FORCES

Seismic forces are inertia forces. When any object, such as a building, experiences acceleration, inertia force is generated when its mass resists the acceleration.
Inertia forces act *within* a building. They are internal forces. As the ground under a building shakes sideways, horizontal accelerations transfer up through the superstructure of the building and generate inertia forces throughout it. Inertia forces act on every item and every component. Every square meter of construction, like a floor slab or wall, possesses weight and therefore mass. Just as gravity force that acts vertically is distributed over elements like floor slabs, so is seismic inertia force, except that it acts horizontally.

THE BASIC SEISMIC STRUCTURAL SYSTEMS

A building's structural system is directly related to its architectural configuration, which largely determines the size and location of structural elements such as walls, columns, horizontal beams, floors, and roof structure. Here, the term **structural/architectural configuration** is used to represent this relationship.

The Vertical Lateral Resistance Systems

Seismic designers have the choice of three basic alternative types of vertical lateral force–resisting systems, and as discussed later, the system must be selected at the outset of the architectural design process. Here, the intent is to demonstrate an optimum architectural/structural configuration for each of the three basic systems. The three alternatives are illustrated in Figure 1-1.

- Shear walls

  Shear walls are structural walls designed to resist horizontal force. The term 'shear wall' originally referred to a wall that had either failed or was expected to fail in shear during a damaging quake. Shear walls are designed to receive lateral forces from diaphragms and transmit them to the ground. The forces in these walls are predominantly shear forces in which the material fibers within the wall try to slide past one another. To be effective, shear walls must run from the top of the building to the foundation with no offsets and a minimum of openings. Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces lateral sway of the building and thereby reduces damage to structure and its contents, since shear wall carry large horizontal earthquake forces, the overturning effects on them are large. Thus design of their foundations requires special attention. Shear walls should be provided along preferably both length and width. However, if they are provided along only one direction, a proper grid of beams and columns in the vertical plane must be provided along the other direction to resist strong earthquake effects.

- Braced frames
  Braced frames act in the same way as shear walls; however, they generally provide less resistance but better ductility depending on their detailed design. They provide more architectural design freedom than shear walls. There are two general types of braced frame: conventional concentric and eccentric. In the concentric frame, the center lines of the bracing members meet the horizontal beam at a single point. In the eccentric braced frame, the braces are deliberately designed to meet the beam some distance apart from one another: the short piece of beam between the ends of the braces is called a link beam. The purpose of the link beam is to provide ductility to the system: under heavy seismic forces, the link beam will distort and dissipate the energy of the earthquake in a controlled way, thus protecting the remainder of the structure (Figure 1-2).
- Moment-resistant frames
  A moment resistant frame is the engineering term for a frame structure with no diagonal bracing in which the lateral forces are resisted primarily by bending in the beams and columns mobilized by strong joints between columns and beams. Moment-resistant frames provide the most architectural design freedom.
  These systems are, to some extent, alternatives, although designers sometimes mix systems, using one type in one direction and another type in the other. This must be done with care, however, mainly because the different systems are of varying stiffness (shear-wall systems are much stiffer than moment-resisting frame systems, and braced systems fall in between), and it is difficult to obtain balanced resistance when they are mixed. However, for high-performance structures,) there is now increasing use of dual systems. Examples of effective mixed systems are the use of a shear-wall core together with a perimeter moment-resistant frame or a perimeter steel-moment frame with interior eccentric-braced frames. Another variation is the use of shear walls combined with a moment-resistant frame in which the frames are designed to act as a fail-safe back-up in case of shear-wall failure.
  The framing system must be chosen at an early stage in the design because the different system characteristics have a considerable effect on the architectural design, both functionally and aesthetically, and because the seismic system plays the major role in determining the seismic performance of the building. For example, if shear walls are chosen as the seismic force-resisting system, the building planning must be able to accept a pattern of permanent structural walls with limited openings that run uninterrupted through every floor from roof to foundation.

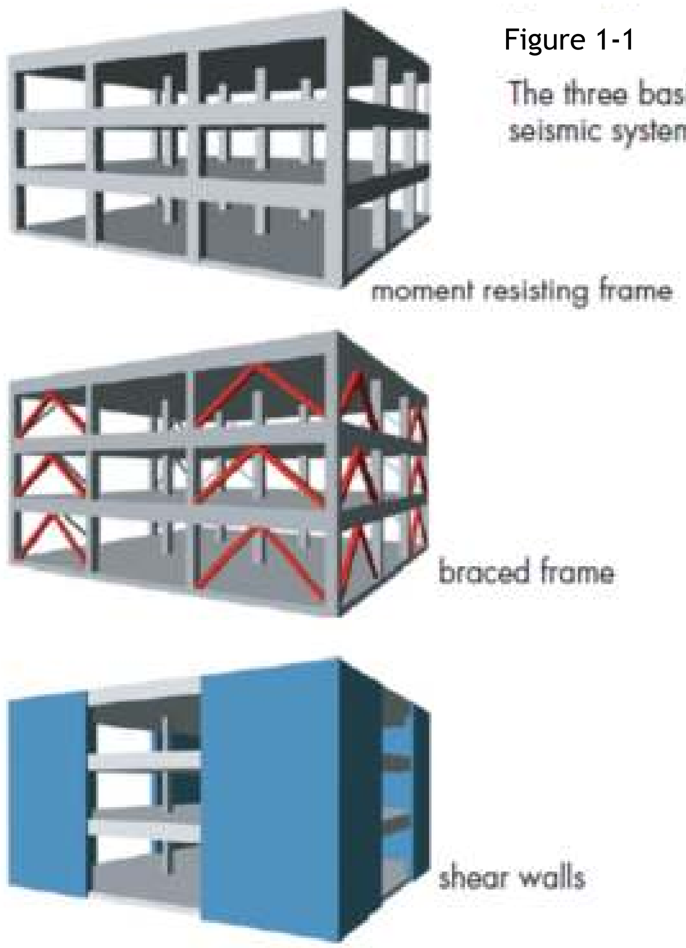
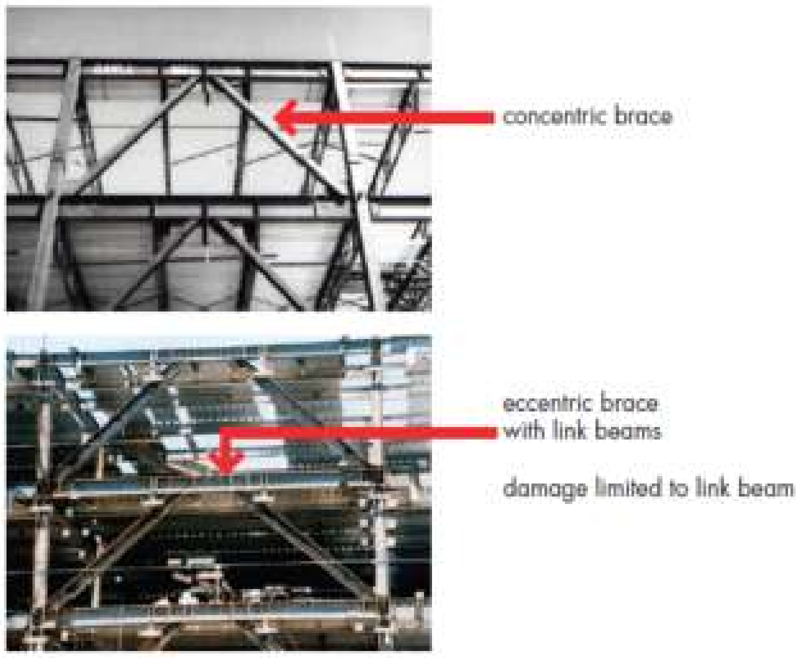


Figure 1-1

The three basic vertical seismic system alternatives.

Figure 1-2

Types of braced frames.



Diaphragms—the Horizontal Resistance System

The term “diaphragm” is used to identify horizontal-resistance members that transfer lateral forces between vertical-resistance elements (shear walls or frames).

The diaphragms are generally provided by the floor and roof elements of the building; sometimes, however, horizontal bracing systems independent of the roof or floor structure serve as diaphragms. The diaphragm is an important element in the entire seismic resistance system (Figure 1-3).

The diaphragm can be visualized as a wide horizontal beam with components at its edges, termed **chords**, designed to resist tension and compression: chords are similar to the flanges of a vertical beam (Figure 1-3A)

A diaphragm that forms part of a resistant system may act either in a **flexible** or **rigid** manner, depending partly on its size (the area between enclosing resistance elements or stiffening beams) and also on its material.

The flexibility of the diaphragm, relative to the shear walls whose forces it is transmitting, also has a major influence on the nature and magnitude of those forces. With flexible diaphragms made of wood or steel decking without concrete, walls take loads according to tributary areas (if mass is evenly distributed). With rigid diaphragms (usually concrete slabs), walls share the loads in proportion to their stiffness (figure 1-3B).

Collectors, also called **drag struts** or **ties**, are diaphragm framing members that “collect” or “drag” diaphragm shear forces from laterally unsupported areas to vertical resisting elements (Figure 1-3C).

Floors and roofs have to be penetrated by staircases, elevator and duct shafts, skylights, The size and location of these penetrations are critical to the effectiveness of the diaphragm. The reason for this is not hard to see when the diaphragm is visualized as a beam. For example, it can be seen that openings cut in the tension flange of a beam will seriously weaken its load carrying capacity. In a vertical load-bearing situation, a penetration through a beam flange would occur in either a tensile or compressive region. In a lateral load system, the hole would be in a region of both tension and compression, since the loading alternates rapidly in direction (Figure 1-3D).

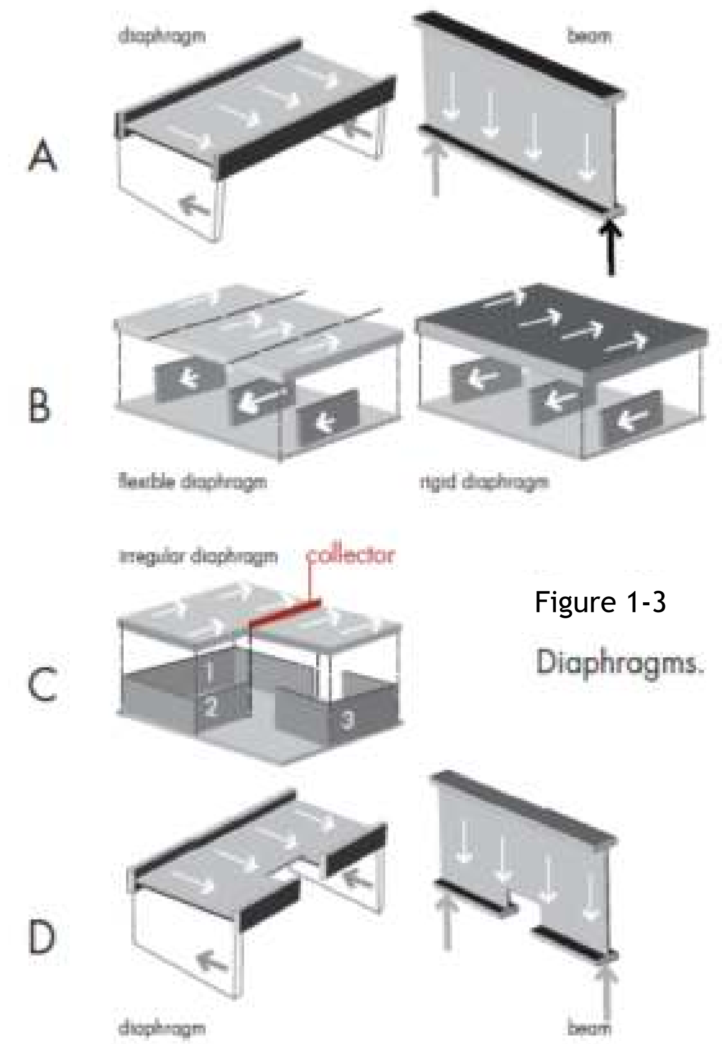


Figure 1-3

Diaphragms.

Optimizing the Structural/Architectural Configuration

Configuration' describes the layout of structure both in plan and elevation.
And how structure and building massing integrate to achieve seismic resistance. This and the following describe commonly occurring configuration challenges that architects face and suggest ways to overcome them without excessively compromising architectural design objectives.
Engineers approach configuration irregularities with the aim of minimizing or eliminating them. One point of potential conflict between the professions might be when an engineer refuses a commission where an architect is unwilling to agree to a more regular horizontal layout. No doubt the architect then shops around for another engineer willing to take a more creative or positive approach towards irregularity. Sadly, the architect may find an engineer less aware of the dangers of poor configuration during a quake.

1-HORIZONTAL CONFIGURATION

Codes provide definitions of irregularity. For the purpose of guiding structural engineers on how to approach the design of horizontally irregular structures, Codes lists and defines five types of horizontal irregularities in order to classify a building either regular or irregular:

- Torsional and extreme torsional
- Re-entrant corner
- Diaphragm discontinuity
- Out-of-plan offsets, and
- Non-parallel systems.

Irregularity means a far more time-consuming period of design and consequent increase in design costs. Whereas regular structures may be designed by simple and straight forward methods, irregular structures necessitate far more sophisticated approaches
Based on observations of quake-damaged buildings, experienced engineers acknowledge the performance of buildings with irregular horizontal configuration is unlikely to be as good as that of more regular structures.

TORSION

Building torsion occurs either where structural elements are not positioned symmetrically in plan or where the centre of rigidity or resistance (CoR) does not coincide with the center of mass (CoM).
In summary, if the Centre of Mass (CoM) of a building is not coincident with the Centre of Resistance (CoR) a torsional moment acts in the horizontal plane causing floor diaphragms to twist about the CoR (see Fig. 1-4). The rotation affects columns located furthermost from the CoR most severely. They are subject to large horizontal deflections, sometimes damaging them so seriously they collapse under the influence of their vertical gravity forces. Numerous torsion failures were observed during the 1994 Northridge and 1995 Kobe earthquakes (Fig. 1-5). Based upon post-earthquake observations of building failures, torsion is recognized as one of the most common and serious horizontal configuration problems.

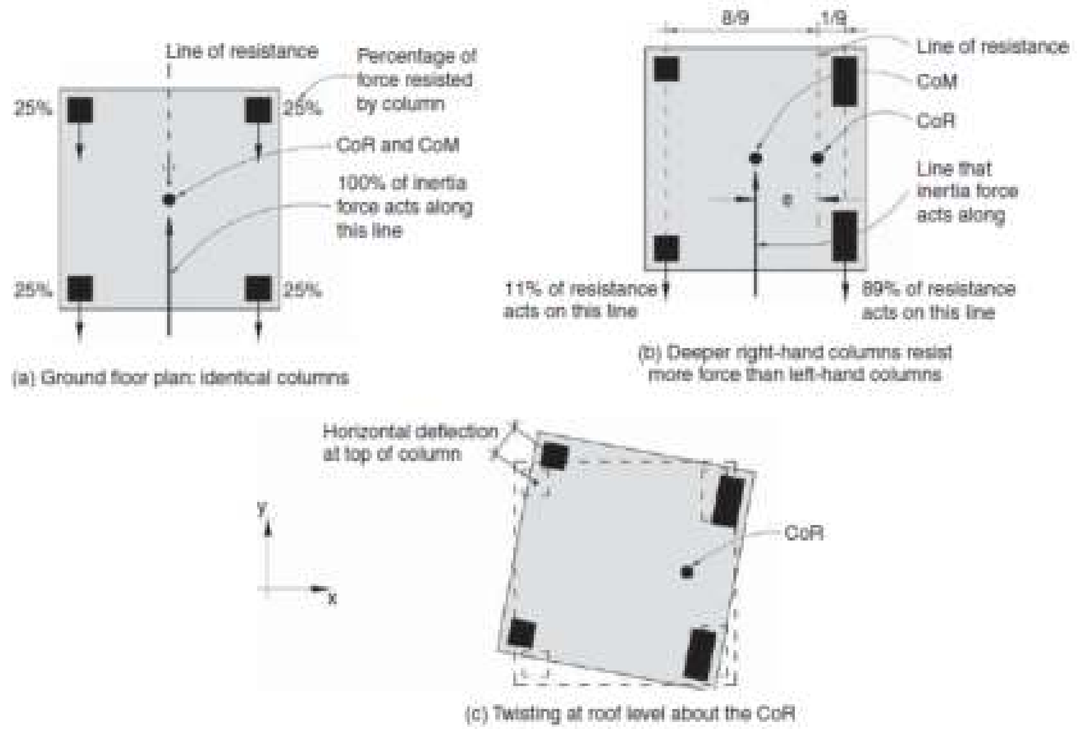


Figure 1-4

A symmetrical structure is modified to illustrate torsion and how it causes a building to twist. (Movement of the roof in the y direction is not shown.)

Figure 1-5 Collapse of a concrete frame building at ground floor level due to torsion, 1995 Kobe, Japan earthquake. (Reproduced with permission from EERI. David R. Bonneville, photographer).

Architects and structural engineers prevent building damage arising from torsion by using several approaches. Firstly, they minimize the distance in plan between the CoM and CoR. Remember that even with a perfectly symmetrical structural configuration some degree of torsion still occurs due to torsional motions within the ground shaking. Codes specify a minimum design eccentricity to account for this and unavoidable out-of-balance or asymmetrical distribution of gravity forces in a building with respect to the CoR.
Secondly, designers provide a minimum of two lines of vertical structure parallel to each of the main orthogonal axes of a building yet horizontally offset from each other. The horizontal off-set or lever-arm between each line of structure should be as large as possible to maximize both the latent torsion-resisting strength and stiffness. When the building in Fig. 1-6(a) twists in plan, its shear walls offer no significant resistance because they warp, flexing about their weak axes. In contrast, when the plan in Fig. 1-6(b) twists about the CoR which is centrally located, each of the four walls reacts along its line of strength against the horizontal deflection imposed upon it by the rotation of the floor diaphragm. Long lever-arms between pairs of walls provide the best possible resistance against torsion.

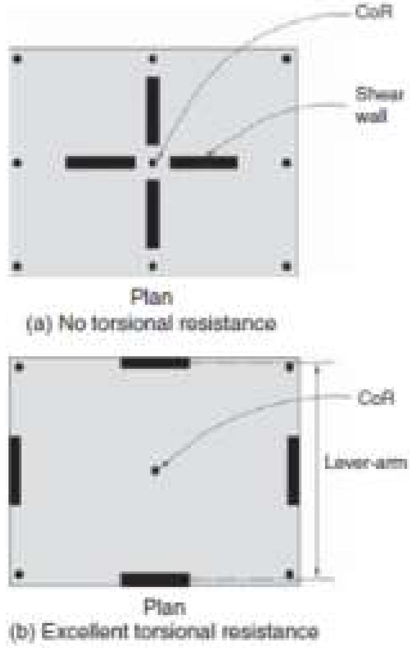


Figure 1-6 Two structural configurations, each with four shear walls, with contrasting abilities to resist torsion

How exactly does vertical structure resist torsion? Consider the building in Fig. 1-7 . It is very well configured structurally to resist torsion– two perimeter shear walls in each direction. Assuming a torsional eccentricity *e* between the resultant line of action of inertia forces acting in the *y* direction and the CoR, the building twists clockwise. Its diaphragm rotates as a rigid unit. A diaphragm is usually very stiff and strong in its plane, especially if constructed from reinforced concrete.
When twisting occurs about the CoR, which is the point through which the resistance from all the shear walls acts, the shear walls acting in the y direction deflect in opposite directions a small amount Δ*y*. These movements are additive to the shear wall deflections due to the *y* direction forces that are not shown. Each shear wall also twists a little. This source of torsional resistance is neglected because the twisting strength of an individual wall is so low. As each wall is pushed, it resists the imposed deflection in the direction of its strength (the *y* direction) and applies a reaction force. The value of these reaction forces multiplied by the lever arm between them represents a moment couple that partially resists the torsional moment causing diaphragm rotation.

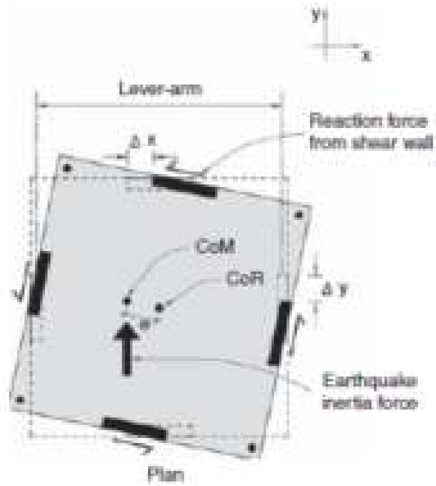


Figure 1-7 A building plan illustrating how vertical structure resists torsion. Most gravity-only structure and the movement of the building in the y direction is not shown.

Also due to the diaphragm rotation, the *x* direction shear walls deflect horizontally Δ*x* in opposite directions. Like the *y* direction shear walls, they react against the movement that deflects them. They apply equal and opposite reaction forces upon the diaphragm creating another moment couple. Even though no *x* direction seismic forces act on the building, because these two shear walls orientated parallel to the *x* axis are strongly connected to the diaphragm, they nonetheless participate in resisting torsion. The two torsion-resisting couples formed by the pairs of parallel shear walls combine to resist the torsional moment and provide torsional equilibrium. Any structural damage is unlikely since only minimal diaphragm rotation occurs.

The four extra shear walls added in plan (Fig. 1-8(a)) enhance torsional resistance slightly. Even if the new walls are identical to the perimeter walls because they are closer to the CoR they are subject to 50 per cent smaller displacements when the diaphragm twists and the lever arms between them are less. With a lesser resisting force (proportional to horizontal displacement) and half the lever arm their torsion-resisting contribution is only 25 per cent of that provided by the perimeter walls. If the perimeter walls are removed, and horizontal forces and torsion are now resisted by the inner walls alone, the two torsion-resisting couples must offer the same resistance as before since the value of the torsion moment is unchanged. We can neglect any torsional resistance from the slender perimeter columns. Since the lever-arms between the inner walls are half of the original lever-arms wall reaction forces double. This means that these walls will need to be considerably stronger and that the diaphragm will twist further. The structural configuration in Fig. 1-8(b) is therefore twice as torsional flexible as that in Fig. 1-8(b) ; but it might still be structurally adequate especially if the perimeter gravity-only columns can sustain the ensuing horizontal movements without damage.

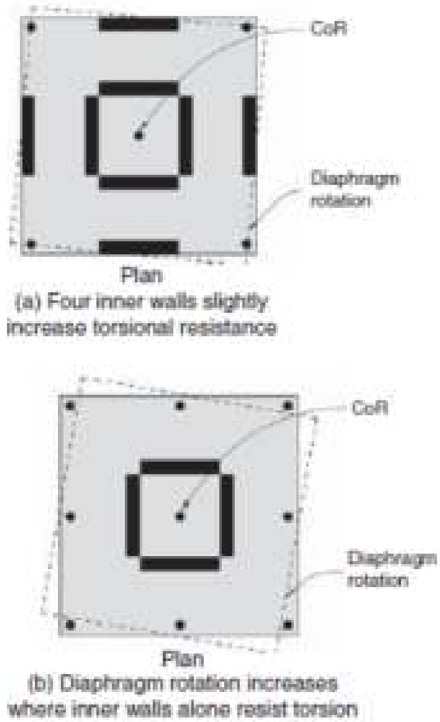


Figure 1-8 Structure located close to the CoR is less effective at resisting torsion.

Although the previous figures illustrate shear walls resisting seismic forces, moment and braced frames can also provide adequate torsion resistance. Replace the shear walls with one- or multi-bay moment frames and the principles outlined above still apply. The building will be less torsionally stiff due to the lesser stiffness of the frames but still perform adequately, especially if the frames are located on the perimeter of the building.

In the examples considered so far, a recommended torsion-resistant structure comprises a minimum of four vertical elements, like shear walls or moment frames, with two in each direction. However, in some situations the number of elements can be reduced to three (Fig. 1-9). Any *y* direction forces are resisted by one shear wall, albeit long and strong especially given an absence of redundancy, and *x* direction forces resisted by two walls. When torsion induces diaphragm rotation, the two *x* direction walls, in this case with a long lever-arm between them, form a moment couple. They provide torsional stability or equilibrium irrespective of the direction of loading – but only so long as they remain elastic. Most shear walls and frames are designed for relatively low seismic forces if they incorporate ductile detailing. So when one *x* direction wall yields as a result of inertia forces in the *x* direction as well as torsion it temporarily loses its stiffness and the COR migrates towards the stiffer end, increasing torsional eccentricity. The system becomes torsionally unstable.

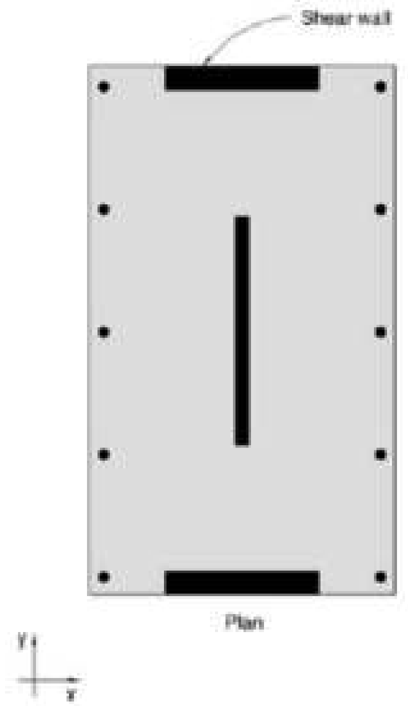


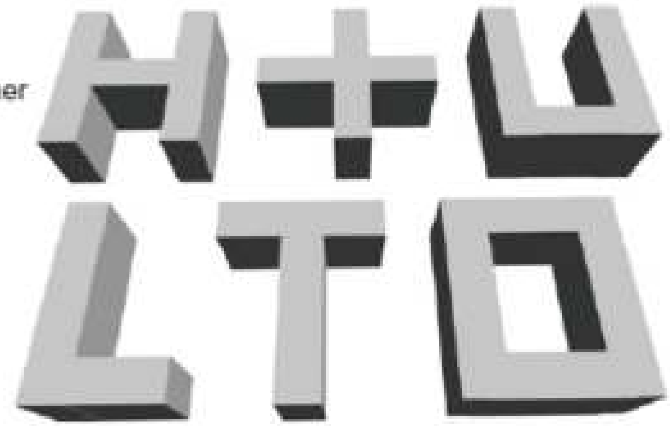
Figure 1-9 An example of a torsionally unbalanced system.

Re-entrant Corners

The re-entrant corner is the common characteristic of building forms that, in plan, assume the shape of an L, T, H, etc., or a combination of these shapes (Figure 1-10).

Figure 1-10

Re-entrant corner plan forms.



There are two problems created by these shapes. The first is that they tend to produce differential motions between different wings of the building that, because of stiff elements that tend to be located in this region, result in local stress concentrations at the re-entrant corner, or “notch”.

The second problem of this form is torsion. Which is caused because the center of mass and the center of rigidity in this form cannot geometrically coincide for all possible earthquake directions. The result is rotation. The resulting forces are very difficult to analyze and predict. Figure 1-12 shows the problems with the re-entrant-corner form. The stress concentration at the “notch” and the torsional effects are interrelated.

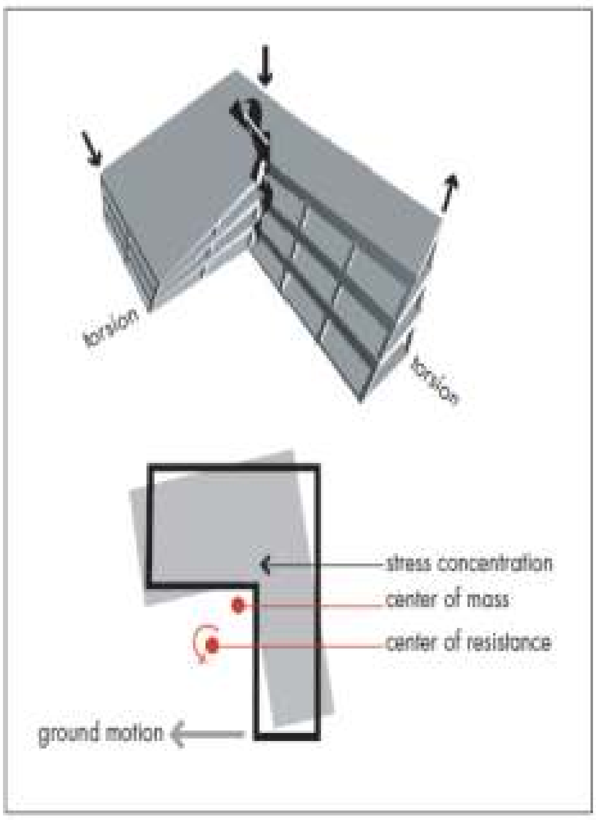


Figure 1-11

Re-entrant corner plan forms.

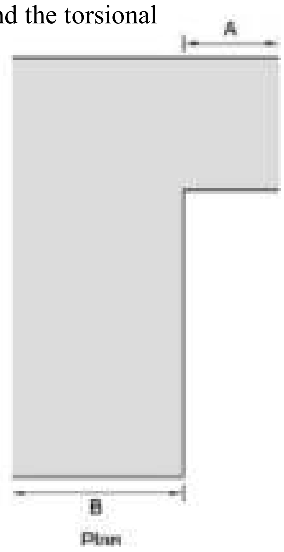


Figure 1-12 A typical definition of an irregular re-entrant configuration is where A > 0.15B.

Figure 1-13 West Anchorage High School, Alaska earthquake, 1964. Stress concentration at the notch of this shallow L-shaped building damaged the concrete roof diaphragm.

Re-entrant corner plan forms are a most useful set of building shapes for urban sites, particularly for residential apartments and hotels, which enable large plan areas to be accommodated in relatively compact form, yet still provide a high percentage of perimeter rooms with access to air and light.

- Solutions

There are two basic alternative approaches to the problem of re-entrant-corner forms: structurally to separate the building into simpler shapes, or to tie the building together more strongly with elements positioned to provide a more balanced resistance (Figure 1-14). The latter solution applies only to smaller buildings.

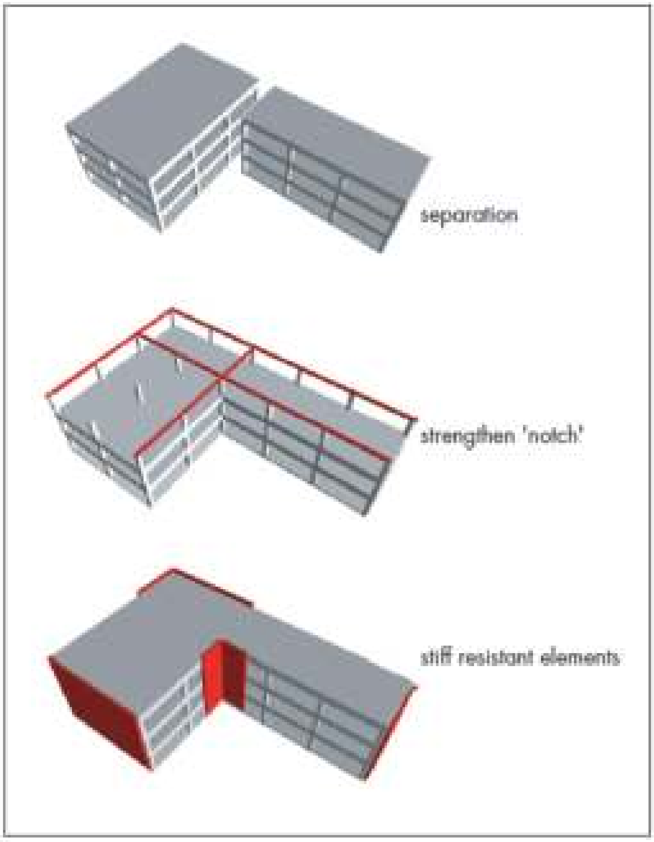


Figure 1-14

Solutions for the re-entrant-corner condition.

Once the decision is made to use separation joints, they must be designed and constructed correctly to achieve the original intent. Structurally separated entities of a building must be fully capable of resisting vertical and lateral forces on their own, and their individual configurations must be balanced horizontally and vertically.

To design a separation joint, the maximum drift of the two units must be calculated by the structural consultant. The worst case is when the two individual structures would lean toward each other simultaneously; and hence the sum of the dimension of the separation space must allow for the sum of the building deflections.

DIAPHRAGM DISCONTINUITIES

In the ideal world of the structural engineer, diaphragms in buildings are not penetrated by anything larger than say a 300 mm diameter pipe. Diaphragms are also planar and level over the whole floor plan.
However, the real world of architecture is quite different, because in most buildings quite large penetrations are required for vertical circulation such as stairways and elevators. Building services, including air ducts and pipes also need to pass through floor slabs and in the process introduce potential weaknesses into diaphragms.

The size of a penetration can be large enough to ruin the structural integrity of a diaphragm altogether. Consider the case of a simple rectangular diaphragm spanning between two shear walls that act in the *y* direction (Fig. 1-15). What are the structural options if a full-width slot is required? The slot destroys the ability of the diaphragm to span to the right-hand wall. If the purpose of the slot is to introduce light or services through the diaphragm one option is to bridge the slot by introducing a section of steel bracing (Fig. 1-15(a)). If designed and connected strongly enough it restores the original spanning capability of the diaphragm. Alternatively, if the geometry of diagonal members isn't acceptable aesthetically a horizontal vierendeel frame, with its far larger member sizes, can be inserted to restore structural function
(Fig. 1-15(b)). In both solutions, light and services can pass between structural members.
If the intention of the penetration in Fig. 1-16 is to provide a staircase, then both previous options are unacceptable. It is now impossible for the diaphragm to transfer forces to the right-hand shear wall. The only option is to no longer consider that wall as a shear wall but to provide a new shear wall to the left of the penetration. Now a shortened diaphragm spans satisfactorily between shear walls. The force path has been restored. All that remains to complete the design is to stabilize the right-hand wall for *x* direction forces by tying it back to the newly down-sized diaphragm (Fig. 1-17). The two new ties may also have to act as horizontal cantilever beams or members of a vierendeel frame. This will transfer seismic forces from the now non-structural wall to the diaphragm if there is insufficient bracing in the wall to deal with its own inertia forces.

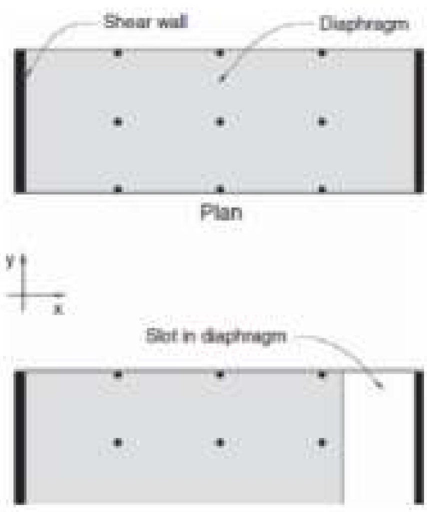


Figure 1-115

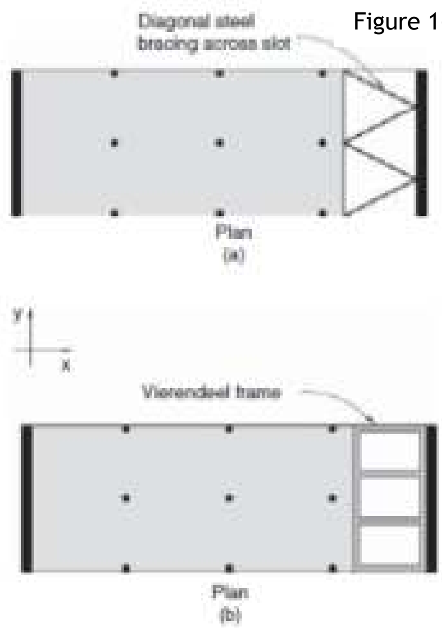


Figure 1-16 Plans of two diaphragms where structural integrity across a slot is restored by steel bracing and frame-action.

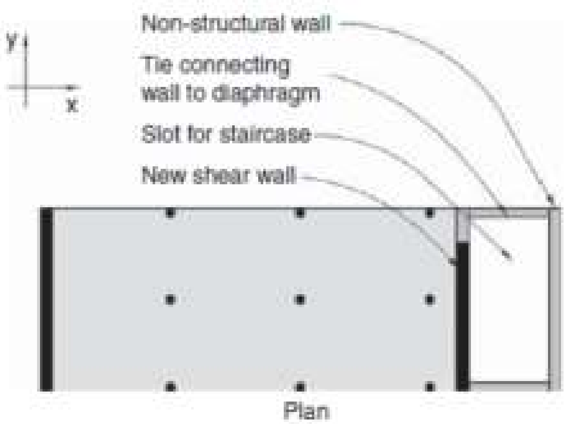


Figure 1-17 A new shear wall enables the right-hand wall to become non-structural.

Figure 1-18 considers a more difficult scenario. Now a penetration is required near the middle of a diaphragm, also spanning between two walls. If the insertion of any horizontal structure like the diagonal bracing of Fig. 1-18(a) is impossible due to architectural requirements the only option is to physically separate the two portions of the building. Although perhaps perceived as one building with penetrated diaphragms, each section now becomes an independent structure. The end shear walls need to be replaced by moment frames to minimize torsion (Fig. 1-18(b)) . All non-structural connections bridging the gap, such as walls and roof, are detailed to accommodate the relative seismic movements between the two structures.

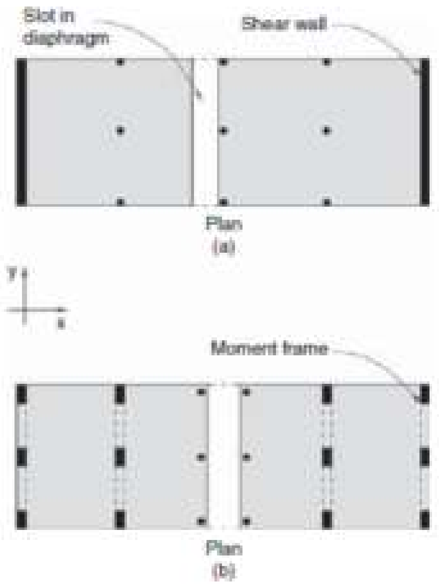


Figure 1-18 A diaphragm slotted near the middle leads to the formation of two separated structures (a). To avoid serious torsional eccentricities, the shear walls are substituted by moment frames (b). The torsional configuration of each structure can be improved if the inner two frames are moved closer to the gap. (X direction structure not shown.)

Another equally serious diaphragm discontinuity occurs where a potential floor diaphragm consists of more than one level. If a relatively small area is raised or lowered it can be treated, as far as seismic behavior is concerned, as if it were a penetration. But consider the situation where a step is introduced across a diaphragm near the middle of its span (Fig. 1-19). The diaphragm is now kinked, and just as a beam kinked in plan is unable to transfer force neither can a kinked diaphragm (Fig. 1-20). If you are skeptical, model a simple straight beam from cardboard. Note how it withstands reasonable force where spanning a short distance. Now introduce a kink. Observe how you have destroyed the integrity of the beam.

The other problem caused by the step is to prevent *x* direction inertia forces from the right-hand end of the building being transferred into the two shear walls acting in that direction (Fig. 1-21(a)). Two ways to overcome these problems are; firstly, to fully separate the building into two structures as discussed previously; or secondly, to introduce a shear wall or frame along the line of the step (Fig. 1-21(b)) and provide *x* direction shear walls at each end of the building. Now there are two diaphragms.

Both span independently between their original perimeter lines now braced by moment frames and a new frame along the line of the step.

Frames have replaced the walls to allow for circulation between both halves of the floor plan. If the step is higher than several hundred millimeters, one diaphragm will apply *y* direction forces directly to the columns of the center frame. This could lead to their premature failure and so the best approach would be to separate the diaphragms and their supporting members into two independent structures.

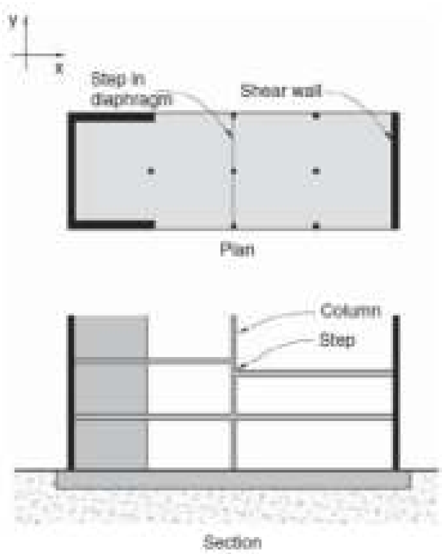


Figure 1-19 A stepped diaphragm.

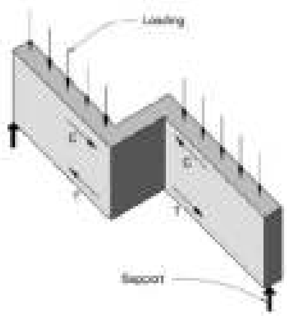


Figure 1-20 A kinked beam showing internal compression and tension forces that can not be achieved. The beam is structurally unsound.

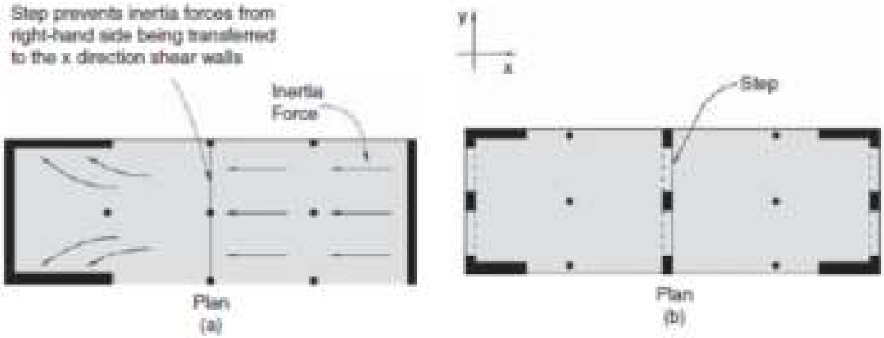


Figure 1-21 The structural difficulty posed by the diaphragm step (a) is solved by increasing the number of shear walls effective in the x direction to four and connecting two to each diaphragm section (b). Moment frames replace y direction shear walls to avoid a mixed system once a moment frame is introduced along the step. Had a shear wall been introduced along the step, the original shear walls in the y direction could have remained.

NON-PARALLEL SYSTEMS

Figure 1-22 illustrates two non-parallel systems. In each case the directions of strength of the vertical structures are angled with respect to any sets of orthogonal axes. The ability of each configuration to resist horizontal forces and torsion is understood by considering the length of each vertical system as a strength vector. A vector can be resolved into components parallel to, and normal to, a set of axes (Fig. 1-23). But what is less apparent is that when these systems resist horizontal force their skewed orientation leads to unexpected secondary forces that are required to maintain equilibrium. In this symmetrically configured building, as the shear walls resist *y* direction forces, the diaphragms must provide tension and compression forces to keep the system stable. When the configuration of non-parallel systems is asymmetrical the distribution of these internal forces becomes far more complex. For this reason codes insist that structural engineers model non-parallel systems in 3–D in order to capture these effects and design for them.

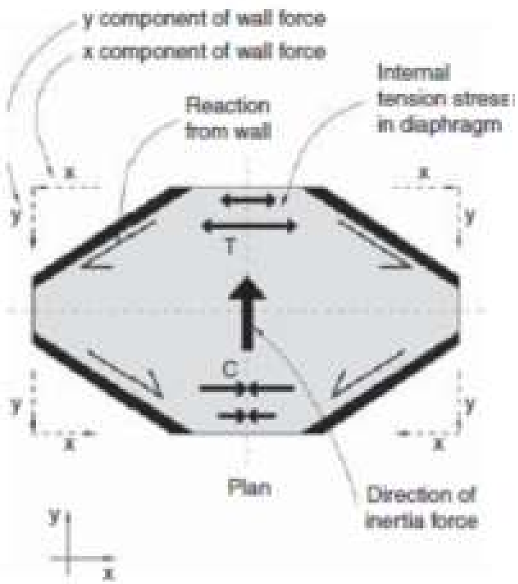


Figure 1-23 A non-parallel system showing the orthogonal force components of each wall and secondary diaphragm stresses for a y direction force.

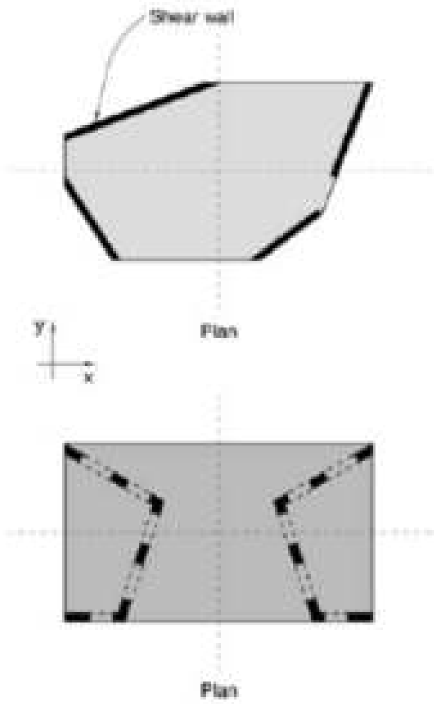


Figure 1-22 Two examples of non-parallel systems. Gravity-only structure not shown.

2-VERTICAL CONFIGURATION

The vertical configuration of a building encompasses two aspects of architectural form – the building envelop profiles in elevation and the elevation of the vertical structural systems in both orthogonal directions.
The best possible seismic performance is achieved where both the 3–D massing and vertical structure of a building are regular. This means an *absence* of the following vertical irregularities repeatedly observed after earthquakes to have initiated severe damage:

- A floor significantly heavier than an adjacent floor
- Vertical structure of one storey more flexible and/or weaker than that above it
- Short columns
- Discontinuous and off-set structural walls, and
- An abrupt change of floor plan dimension up the height of a building.

The irregularities listed above so seriously affect the seismic performance of a building they should be avoided at all cost.

Soft and Weak Stories

The problem and the types of condition
The most prominent of the problems caused by severe stress concentration is that of the "soft" story. The term has commonly been applied to buildings whose ground-level story is less stiff than those above. The building code distinguishes between "soft" and "weak" stories. Soft stories are less stiff, or more flexible, than the story above; weak stories have less strength. A soft or weak story at any height creates a problem, but since the cumulative loads are greatest towards the base of the building, a discontinuity between the first and second floor tends to result in the most serious condition.

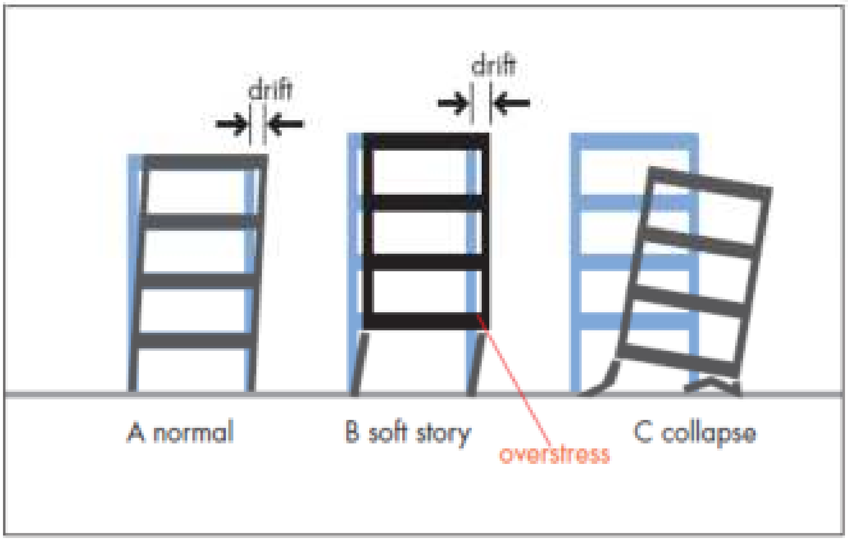
Figure 2-1 A soft storey ground floor has disappeared in this three-storey apartment block. 1994 Northridge, California earthquake. (Reitherman, R.K. Courtesy of the National Information Service for Earthquake Engineering, EERC, University of California, Berkeley)

Figure 2-2 Soft storey ground floor collapse of a four-storey building. 1995 Kobe earthquake. (Bertero, V.V. Courtesy of the National Information Service for Earthquake Engineering, EERC, University of California, Berkeley)

The way in which severe stress concentration is caused at the top of the first floor is shown in the diagram sequence in Figure 2-3 Normal drift under earthquake forces that is distributed equally among the upper floors is shown in Figure 2-3A. With a soft story, almost all the drift occurs in the first floor, and stress concentrates at the second-floor connections (Figure 2-3B). This concentration overstresses the joints along the second floor line, leading to distortion or collapse (Figure 2-3C).

Figure 2-3

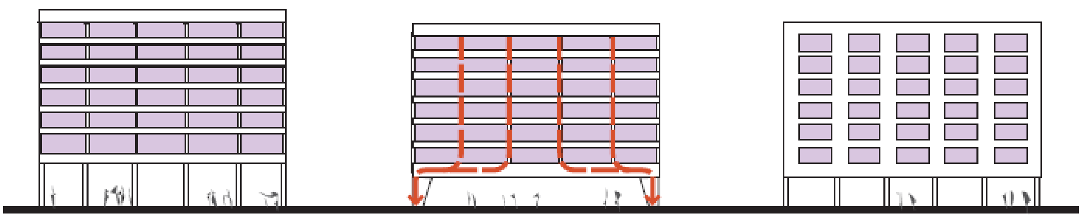
The soft first story failure mechanism.



Three typical conditions create a soft first story (Figure 2-4). The first condition (Figure 2-4A) is where the vertical structure between the first and second floor is significantly more flexible than that of the upper floors. This discontinuity most commonly occurs in a frame structure in which the first floor height is significantly taller than those above.

The second form of soft story (Figure 2-4B) is created by a common design concept in which some of the vertical framing elements do not continue to the foundation, but rather are terminated at the second floor to increase the openness at ground level. This condition creates a discontinuous load path that results in an abrupt change in stiffness and strength at the plane of change.

Finally, the soft story may be created by an open first floor that supports heavy structural or nonstructural walls above (Figure 2-4C). This situation is most serious when the walls above are shear walls acting as major lateral force-resisting elements.



A flexible first floor B discontinuity: indirect load path C heavy superstructure

Figure 2-4 Three types of soft first story.

- Solutions

The best solution to the soft and weak story problem is to avoid the discontinuity through architectural design. There may, however, be good programmatic reasons why the first floor should be more open or higher than the upper floors. In these cases, careful architectural/structural design must be employed to reduce the discontinuity. Some conceptual methods for doing this are shown in Figure 2-5.

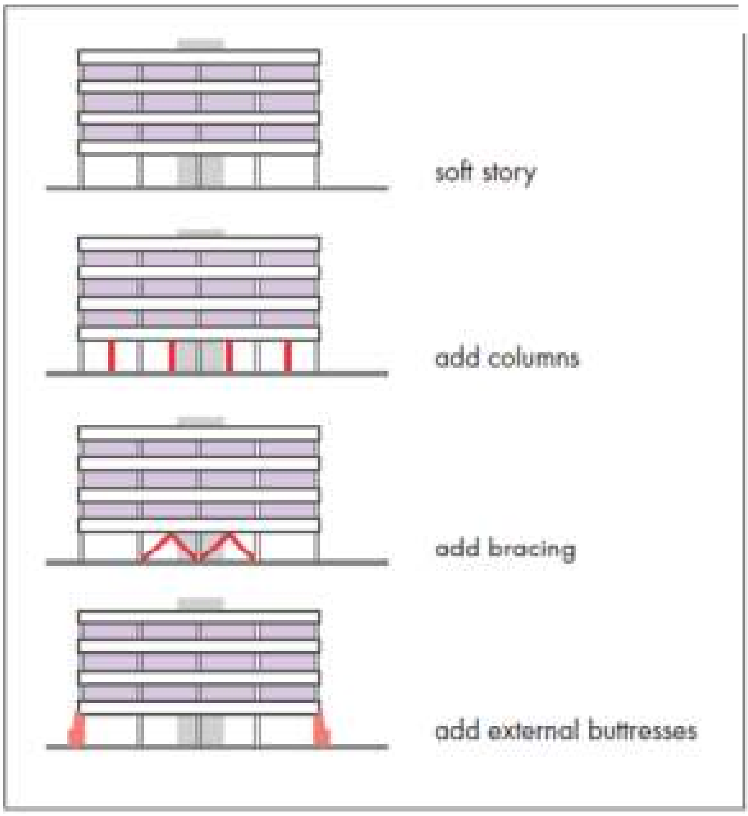


Figure 2-5

Some conceptual solutions to the soft first story.

Designers sometimes create a soft-story condition in the effort to create a delicate, elegant appearance at the base of a building. Skillful structural/architectural design can achieve this effect without compromising the structure, as shown in Figure 2-6. The building shown is a 21-story apartment house on the beach in Vina del Mar, Chile. This building was unscathed in the strong Chilean earthquake of 1985.

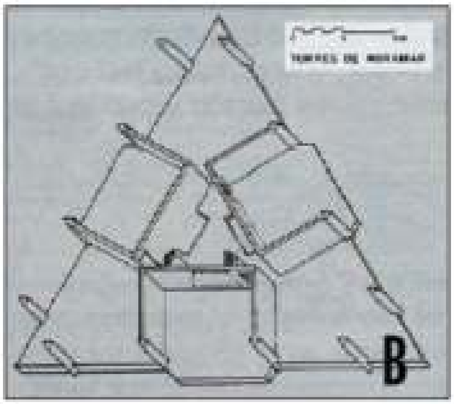


Figure 2-6 This apartment house appears to have a soft first story Figure 2-6A but the lateral force-resisting system is a strong internal shear wall box. in which the shear walls act as party walls between the dwelling units Figure 2-6B The architect achieved a light and elegant appearance, and the engineer enjoyed an optimum and economical structure.

Sometimes a soft story is created somewhere at mid-height of the multi-story building, for using the space as restaurant or gathering purposes, *see 2-6C*. Such soft story in building also collapsed in Kutch and Kobe earthquakes. For such a case also, the story columns should be designed for the higher forces OR a few shear walls introduced to make up for the reduced stiffness of the story.

Figure 2-6 C Collapse of soft middle storey in a building at Bhuj.

SHORT COLUMNS

There are two types of short column problems; firstly, where some columns are shorter than others in a moment frame, and secondly, where columns are so short they are inherently brittle. The short columns of the second group are usually normal length columns that are prevented from flexing and undergoing horizontal drift over most of their height by partial-height infill walls or very deep spandrel beams. Figure 2-7 shows examples where columns, some shorter than others in the same frame, cause seismic problems. The structural difficulty arising from these configurations is illustrated in Fig. 2-8. Two columns together, one that is half the height of the other, resist a horizontal force. The stiffness of a column against a horizontal force is extremely sensitive to its length; the shorter column is therefore eight times stiffer than the other, so it tries to resist almost eight times as much force as the longer column. It is unlikely to be strong enough to resist such a large proportion of the horizontal force and may fail.

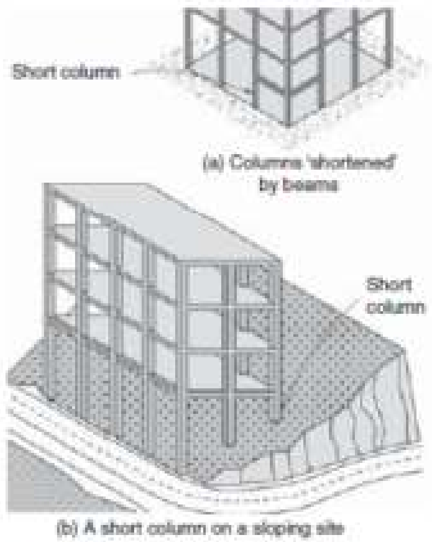


Figure 2-7 Examples of short columns among longer columns of moment frames.

In some situations the column is surrounded by walls on both sides such as up to the window sills and then in the spandrel portion above the windows but it remains exposed in the height of the windows. Such a column behaves as a short column under lateral earthquake loading where the shear stresses become much higher than normal length columns and fail in shear. (See fig. 14)

Recommendation:
To safe guard against this brittle shear failure in such columns the special confining stirrups should be provided throughout the height of the column at short spacing as required near the ends of the columns.

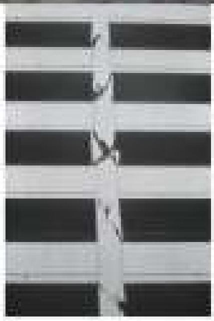


Fig 14 - Damage to buildings due to short column effect on columns

Continuing a short length of masonry up the sides of columns so that diagonal compression struts can act at the beam-column joint and thereby avoid short column failure (Fig. 2-9). Reduction in the width of an opening above a partial-height masonry or concrete infill to prevent a short column failure. The raised lengths of infill enable a compression strut to transfer force directly to the top of the column and avoids the need for the column to bend.

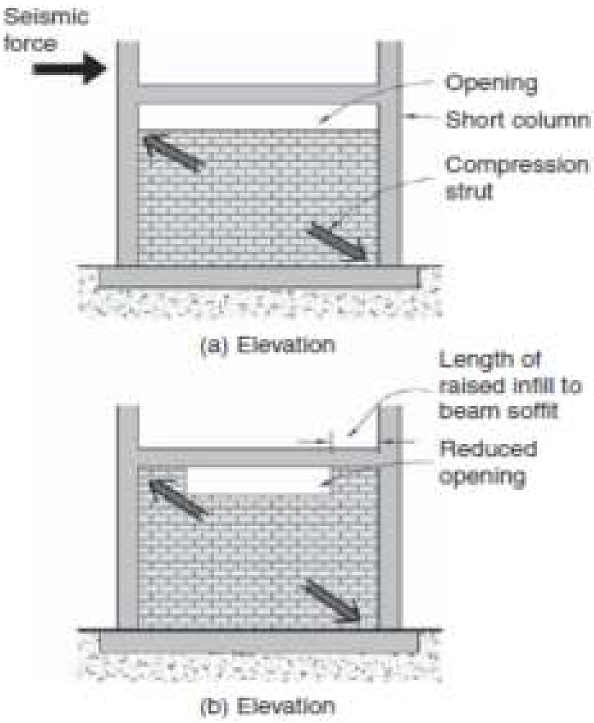


Figure 2-9 Reduction in the width of an opening above a partial-height masonry or concrete infill to prevent a short column failure. The raised lengths of infill enable a compression strut to transfer force directly to the top of the column and avoids the need for the column to bend.

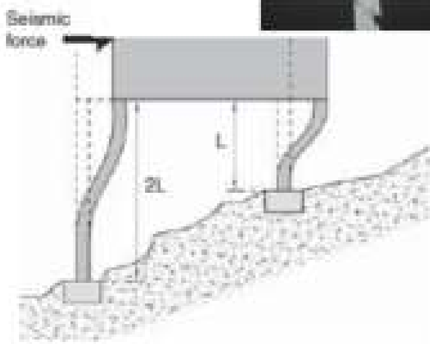


Figure 2-8 Two unequal height columns resisting seismic force.

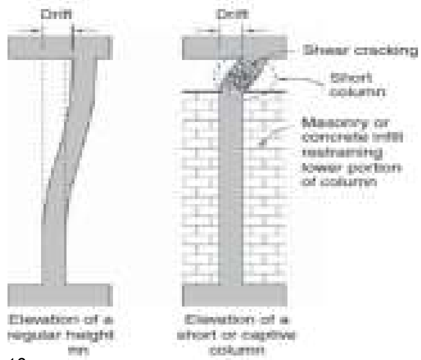


Figure 2-10 Comparison between a regular and a short column subject to horizontal drift

Figure 2-11 Typical short column damage. 1994 Northridge, California earthquake.
(Reproduced with permission from Andrew B. King)

Figure 2-12 Short column failure. 2007 Pisco, Peru earthquake.
(Reproduced with permission from Darin Bell)

Strong Beam, Weak Column

Structures are commonly designed so that under severe shaking, the beams will fail before the columns. This reduces the possibility of complete collapse. The short-column effect, discussed before, is analogous to a weak-column strong-beam condition, which is sometimes produced inadvertently when strong or stiff nonstructural spandrel members are inserted between columns. The parking structure shown in Figure 2-14 suffered strong-beam weak-column failure in the Whittier, California, earthquake of 1987.

Figure 2-14 Damaged parking structure, Whittier Narrows (Los Angeles) earthquake, 1987. The deep spandrels create a strong beam, weak column condition.

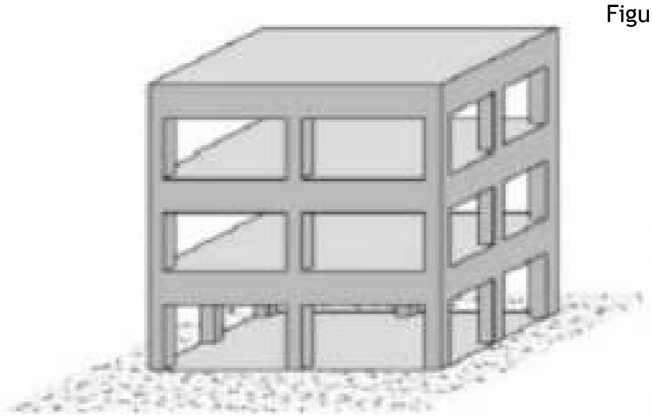


Figure 2-13 A weak column-strong beam structure develops a soft storey at ground level once columns are damaged.

DISCONTINUOUS AND OFF-SET WALLS

When shear walls form the main lateral resistant elements of a structure, and there is not a continuous load path through the walls from roof to foundation, the result can be serious overstressing at the points of dis-continuity. This discontinuous shear wall condition represents a special, but common, case of the "soft" first-story problem.

The discontinuous shear wall is a fundamental design contradiction: the purpose of a shear wall is to collect diaphragm loads at each floor and transmit them as directly and efficiently as possible to the foundation. To interrupt this load path is undesirable; to interrupt it at its base, where the shear forces are greatest, is a major error. Thus the discontinuous shear wall that terminates at the second floor represents a "worst case" of the soft first-floor condition. A discontinuity in vertical stiffness and strength leads to a concentration of stresses, and the story that must hold up all the rest of the stories in a building should be the last, rather than the first, element to be sacrificed.

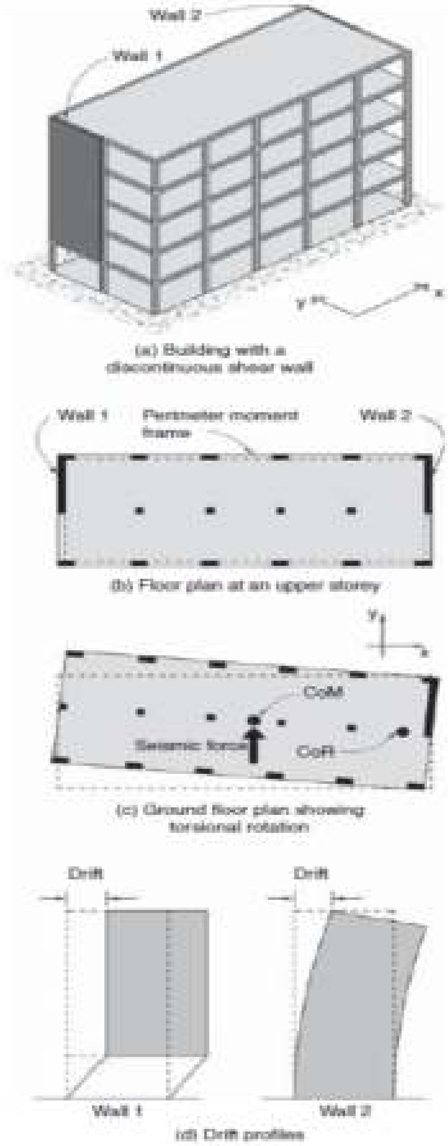


Figure 2-15 A discontinuous wall and its torsion-inducing influence on a building.

Consider the building in Fig. 2-15 . At its upper levels *y* direction forces are resisted by shear walls at each end, but at ground floor level the left-hand wall, Wall 1, is discontinuous. Two perimeter moment frames resist *x* direction forces. When struck by a quake in the *y* direction, the ground pulses will distort the ground floor columns under Wall 1. Their 'softness' prevents Wall 1 from providing the seismic resistance perhaps expected of it and exemplifies the worst possible case of a soft storey. At the other end of the building the base of Wall 2, which is continuous, moves with the ground motion. Due to the more substantial overall strength and stiffness of Wall 2, as compared to Wall 1, Wall 2 tends to resist the inertia forces from the whole building. The two different wall drift profiles are shown in Fig. 2-15(d) . Since Wall 1 resists almost no inertia force due to its discontinuity, yet Wall 2 is fully functional the building experiences serious torsion. To some degree, but limited by the modest lever-arm between them and their inherent flexibility, the two *x* direction moment frames try to resist the torsion. As the building twists about its CoR located close to Wall 2, the columns furthest away from the CoR are subject to large drifts and severe damage (Fig. 2-16).

Figure 2-16 Ground floor damage caused by a discontinuous wall. 1980 El Asnam, Algeria earthquake. (Bertero, V.V. Courtesy of the National Information Service for Earthquake Engineering, EERC, University of California, Berkeley)

What are the solutions to this problem? Probably the best option is to make both walls non-structural. Form them from either light-weight materials or use non-structural cladding panels to achieve the required architectural characteristics. Using the same approach as the building of Fig. 2-15, provide new moment frames behind the non-structural walls (Fig. 2-17(a)). Another possibility is to introduce an off-set single-storey wall back from Wall 1 (Fig.2-17(b)). As explained below, this solution, which introduces many architectural and engineering complexities, is best avoided. This situation applies to Wall 1. Two strong columns, one at each end of Wall1 must withstand vertical tension and compression forces to prevent it overturning under the influence of floor diaphragm forces feeding into it up its height.

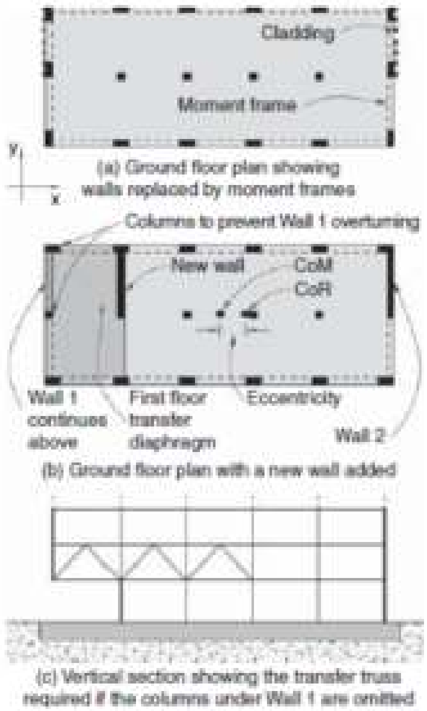


Figure 2-17 Alternatives to a discontinuous wall.

NON-STRUCTURAL ELEMENTS: THOSE LIKELY TO CAUSE STRUCTURAL DAMAGE

Non-structural elements are, by definition, not intended to resist any seismic forces other than those resulting from their own mass. They are also, in the main, elements that structural engineers do not design and for which architects, and mechanical or electrical engineers take primary responsibility. The diverse types of non-structural elements can be divided into three groups:

- Architectural elements such as cladding panels, ceilings, glazing and partition walls
- Mechanical and electrical components like elevators, air conditioning equipment, boilers and plumbing, and
- Building contents, including bookcases, office equipment, refrigerators and everything else a building contains.

INFILL WALLS

Infill walls are non-structural walls constructed between columns.
Where located on the exterior of a building as part of the cladding system, infill walls usually are bounded by structure; columns on either side, floor surfaces below and beams above. A beam may not necessarily be present but most infill walls abut columns. The description of most infill walls as ‘non-structural’ is misleading to say the least.
Infill walls can helpfully resist seismic forces in buildings, but only in certain situations. These include where there is no other seismic resisting system provided; the building is low-rise; the masonry panels are continuous from foundation to roof; there are enough panels in each plan orthogonal direction to adequately brace the building; the infills are not heavily penetrated; and finally, where infill walls are placed reasonably symmetrically in plan. Most infill walls do not satisfy these criteria and may introduce configuration deficiencies.

Problems associated with infill walls

Infill walls stiffen a building against horizontal forces, additional stiffness reduces the natural period of vibration, which in turn leads to increased accelerations and inertia forces (Fig. 2-18). As the level of seismic force increases, the greater the likelihood of non-structural as well as structural damage. To some degree, the force increase can be compensated for by the strength of the infills provided they are correctly designed to function as structural elements.

Secondly, an infill wall prevents a structural frame from freely deflecting sideways. In the process the infill suffers damage and may damage the surrounding frame. The in-plane stiffness of a masonry infill wall is usually far greater than that of its surrounding moment frame – by up to five to ten times! Without infill walls a bare frame deflects under horizontal forces by bending in its columns and beams. However, a masonry infill dominates the structural behavior (Fig. 2-19). Rather than seismic forces being resisted by frame members, a diagonal compression strut forms within the plane of the infill, effectively transforming it into a compression bracing member. Simultaneously, a parallel diagonal tension crack opens up between the same two corners of the frame because of the tensile elongation along the opposite diagonal and the low tensile strength of the infill material. The infill panel geometry deforms into a parallelogram. After reversed cycles of earthquake force, 'X' pattern cracking occurs (Fig. 2-20). The strength of the compression strut and the intensity of force it attracts concentrates forces at the junction of frame members. Shear failure may occur at the top of a column just under the beam soffit (Fig. 2-21). Such a failure is brittle and leads to partial building collapse.

Figure 2-20 Typical infill wall diagonal crack pattern. 1999 Chi-chi, Taiwan earthquake. (Reproduced with permission from Geoff Sidwell).

Figure 2-21 Damage to the tops of several columns due to infill wall compressive strut action. Mexico City, 1985 Mexico earthquake. (Reproduced with permission from R.B. Shephard).

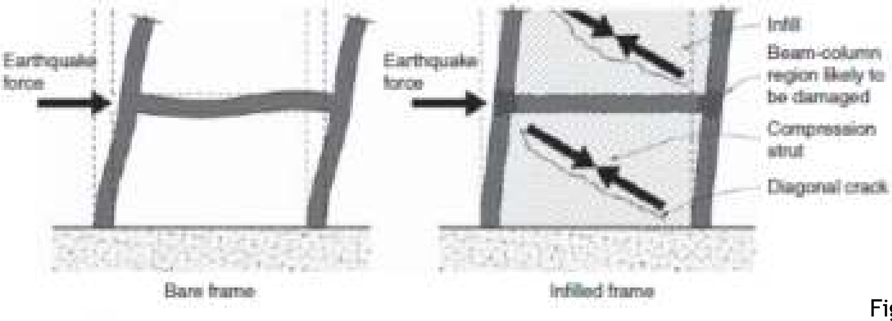


Figure 2-19 Whereas a bare frame deflects horizontally by columns and beams bending, the stiffness of a masonry infill limits horizontal movement. A diagonal compression strut forms together with a diagonal tension crack caused by elongation along the other diagonal.

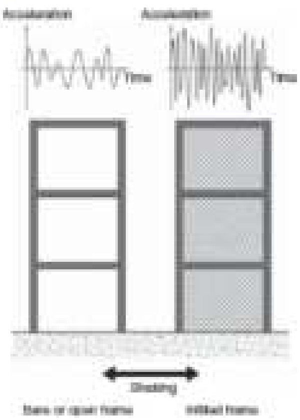


Figure 2-18 A comparison of roof-top accelerations of a bare or open frame with an infilled frame. Note the shorter periods of vibration and higher accelerations of the infill frame.

Even if infill walls are continuous vertically from the foundations to roof, once ground floor infill walls are damaged a soft story failure is possible.

Another danger facing a heavily cracked infill is its increased vulnerability to out-of-plane forces (Fig. 2-22). The wall may become disconnected from surrounding structural members and collapse under out-of-plane forces. Due to their weight, infill walls pose a potential hazard to people unless intentionally and adequately restrained.

The final problem associated with the seismic performance and influence of infill walls is that of torsion. Unless infill walls are symmetrically placed in plan their high stiffness against seismic force changes the location of the Centre of Resistance (CoR). In Fig. 2-23(a) the CoR and Centre of Mass (CoM) are coincident; no significant torsion occurs. If infill walls are located as in Fig. 2-23(b) , the CoR moves to the right and the subsequent large torsional eccentricity causes the building to twist when forced along the *y* axis (Fig. 2-23(c)). As one floor twists about the CoR relative to the floor beneath the columns furthest away from the CoR sustain large interstorey drifts and damage.

If the drifts are too large, those columns are unable to continue to support their gravity forces and their damage leads to that area of the building collapsing. In this example, the infill walls cause torsion during *y* direction shaking only.

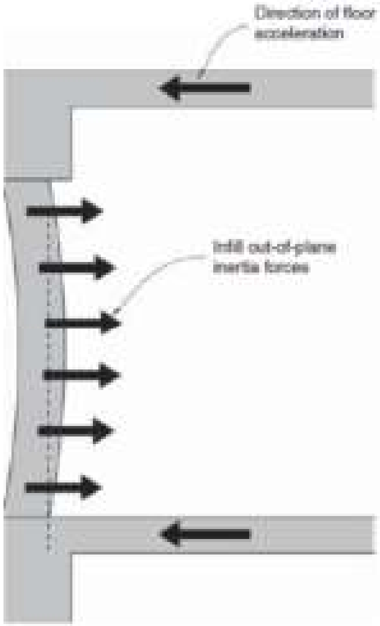


Figure 2-22 A section through two floors and an infill wall. Out-of-plane forces act on the infill which spans vertically between floors.

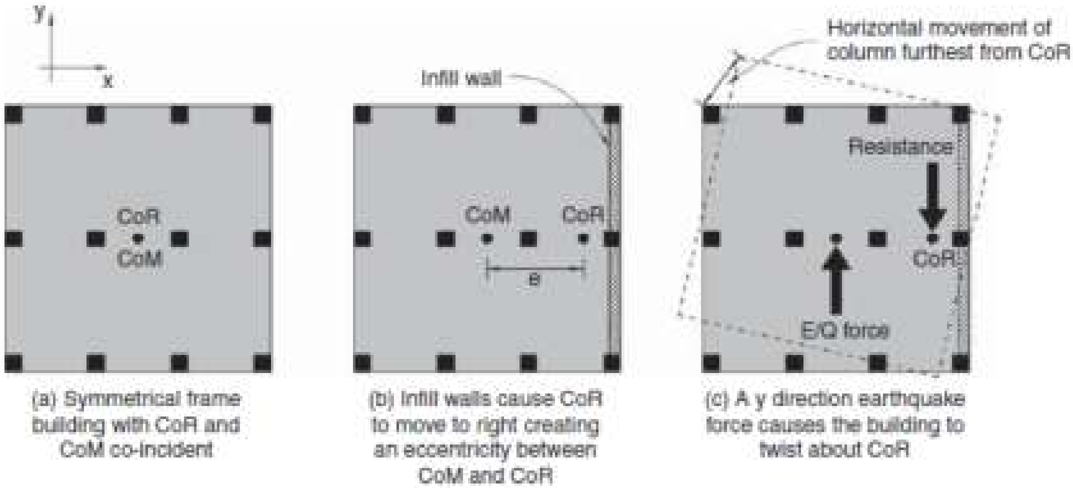


Figure 2-23 Asymmetrically placed infill walls cause building torsion that damages columns distant from the CoR.

References

- Seismic design for architect- OUTWITTING THE QUAKE by Andrew Charleson-2008.
- 12_Fundamentals_for_seismic_design_of_RCC_buildings_by_Prof_Arya
- Designing for Earthquakes A Manual for Architects FEMA 454 / December 2006