2.2 Effects of earthquakes on concrete buildings

2.2.1 Global seismic response mechanisms

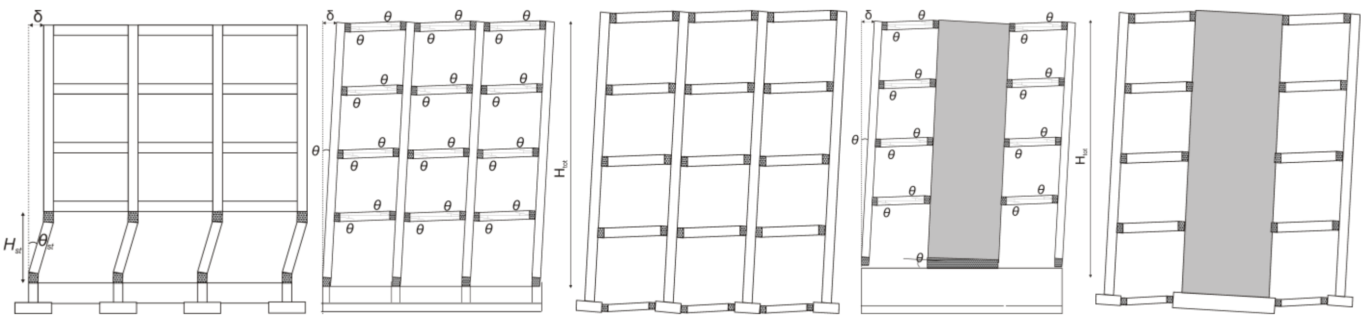
A structure supported on the ground follows its motion during an earthquake, developing, as a result, inertia forces. A typical concrete building is neither stiff enough to follow the ground motion as a rigid body, nor sufficiently flexible to stay in the same absolute position in space, while its base adheres to the shaking ground. As we will see in Sections 3.1.1, 3.1.2, 3.1.4, the building will respond to the seismic inertia forces by developing its own oscillatory motion. The amplitude, frequency content and duration of that motion depend, on one hand, on the corresponding characteristics of the ground shaking and, on the other, on the dynamic properties of the structure itself (see Section 3.1.1).

The base of the structure will follow all three translational and all three rotational components of the motion of the ground it is supported on; accordingly, its dynamic response will be in 3D, with displacements and rotations in all three directions. However, for a typical concrete building, only the structural effects of the two horizontal translational components of the ground motion are worth considering. The - by and large poorly known - rotational components are important only for very tall and slender structures, or those with twisting tendencies very uncommon in buildings designed for earthquake resistance. Concerning the vertical translational component, its effects are normally accommodated within the safety margin between the factored gravity loads (e.g., the "persistent and transient design situation" of the Eurocodes, where the nominal gravity loads enter amplified by the partial factors on actions) for which the building is designed anyway, and the quasi-permanent ones considered to act concurrently with the "design seismic action" (see Section 1.3.1). Important in this respect is the lack of large dynamic amplification of the vertical component by the vibratory properties of the building in the vertical direction.

As we will see in detail in Chapters 3 and 4, a concrete building is expected to respond to the horizontal components of the ground motion with inelastic displacements. It is allowed to do so, provided that it does not put at risk the safety of its users and occupants by collapsing. Very important for the possibility of collapse are the self-reinforcing second-order (P-Δ) effects produced by gravity loads acting through the lateral displacements of the building floors: if these displacements are large, the second-order moments (i.e., the overlying gravity loads times the lateral displacements) are large and may lead to collapse.

Because the major part of lateral structural displacements are inelastic and, besides, they tend to concentrate in the locations of the structural system where they first appeared, very important for the possibility of collapse is the "plastic mechanism" which may develop in the building under the horizontal components of the ground motion. Inelastic seismic deformations in concrete buildings are flexural; they concentrate as plastic rotations wherever members yield in flexure (normally at member ends). Once the yield moment is reached at such a location, a "plastic hinge" forms and starts developing plastic rotations with little increase in the acting moment. The "plastic hinges" may form at the appropriate locations and in sufficient number to turn the building structure into a "mechanism", which can sway laterally under practically constant lateral forces ("plastic mechanism"). The two extreme types of mechanism in concrete buildings are shown in Fig. 2.9. Of the two mechanisms, the one that can lead to collapse is the "column sway" or "soft-storey" mechanism in Fig. 2.9(a). If the ground storey has less masonry infills or other components with significant lateral stiffness and strength than the storeys above, a "soft-storey" mechanism is more likely to develop there.

Mixed situations are very common, with plastic hinges forming at column ends at a number insufficient for a "soft storey" mechanism, and in fewer beams than in a full-fledged "beam-sway" mechanism. Strictly speaking, a mixed distribution of plastic hinges does not give a "mechanism" that kinematically allows sway of the building at little additional lateral force. Therefore, normally it does not lead to collapse nor to notable residual horizontal drifts. A full mechanism of the types shown in Fig. 2.9 (especially the one in Fig. 2.9(a)) may lead to collapse, or to demolition because of large, irreversible residual drifts.



(a) (b) (c) (d) (e)

Fig. 2.9 Side-sway plastic mechanisms in concrete buildings: (a) soft-storey mechanism in weak column/strong beam frame; (b), (c) beam-sway mechanisms in strong column/weak beam frame; (d), (e) beam-sway mechanisms in wall-frame system

2.2.2 Collapse

Collapses of "open ground storey" buildings are depicted in Figs. 2.10 and 2.11. Fig. 2.11 shows on the left a very common type of collapse in multi-storey concrete buildings: the so-called "pancake" collapse, with the floors falling on top of each other, trapping or killing the occupants.

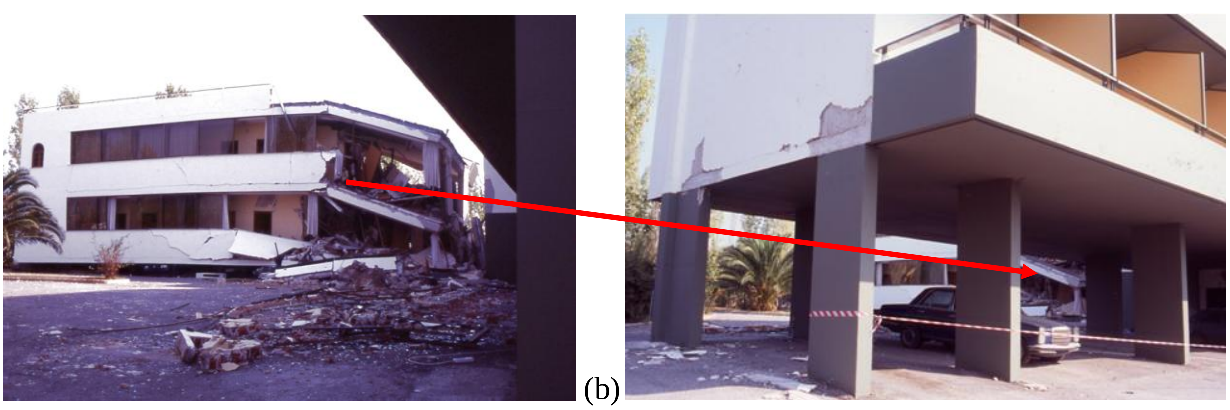
(a) (b)


Fig. 2.10 (a) Collapse of open ground storey building; (b) collapsed building shown at the background; similar building at the foreground is still standing with large ground storey drift.

(a)  (b) 

Fig. 2.11 Typical collapses of frame buildings with open ground storey; "pancake" type of collapse shown on the right.

As we will see in detail in Sections 4.5.2 and 5.4.2, a stiff vertical spine of strong columns or large concrete walls promotes "beam-sway" mechanisms of the type in Figs. 2.9(b) to (e) and helps avoid "soft-storey" ones per Fig. 2.9(a). Walls are quite effective in that respect: in Fig. 2.12(a) the walls in the middle of the lateral sides and at the corners with the back side have failed at the ground storey (one is shown inside a yellow frame), but have prevented the collapse of columns all along the front side from triggering "pancake" collapse; in Fig. 2.12(b) perimeter walls may have failed terminally, but have prevented collapse of the building.

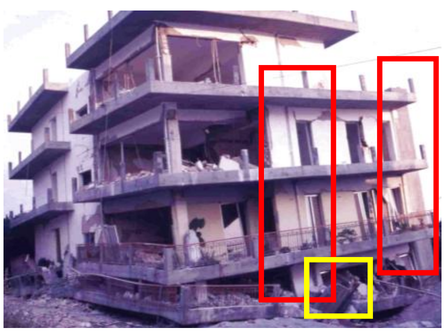
(a)  (b) 

Fig. 2.12 Role of walls in preventing pancake collapse of otherwise condemned buildings.

The dismal performance of walls in the earthquake of February 2010 in Chile has shown that walls are not a panacea. Wall buildings were a success story in past Latin American earthquakes, leading designers to extremes in their use in high-rise construction: in recent practice, very narrow, long walls, bearing the full gravity loads are used in tall buildings, in lieu of columns and non-load bearing partitions. These walls were subjected to very high axial stresses due to gravity loads and failed at the lowest level in flexure-cum-compression, sometimes with lateral instability. A typical case is that of the building on the cover of this book, depicted in more detail in Fig. 2.13.

Fig. 2.13 Collapse of Alto Rio wall building in Concepción, Chile; February 2010 earthquake (structural walls are shown in black in the framing plan).

In all the examples shown so far, as well as in Fig. 2.14, the ground storey was critical. Fig. 2.14(c) depicts the typical case of a concrete frame building with masonry infills, which have suffered heavy damage at the ground storey but may have saved the building from collapse. Figs. 2.10 to 2.14 may be contrasted to Fig. 2.15, where the top floors or an intermediate one have collapsed, but the underlying ones withstood both the earthquake and the collapse of the floors above. Such exceptions to the rule are most often due to an abrupt reduction in the lateral resistance of a floor, because that floor and those above were thought to be non-critical. Higher modes of vibration (see Section 3.1.4, 3.1.5), which are more taxing on certain intermediate floors than on the ground storey, may have played a role as well.

(a) 

(b)  (c) 

Fig. 2.14 Typical concentration of failures or damage in ground storey with role and damage to infills shown in (c).

(a)  (b) 

Fig. 2.15 Collapse of top floors in Mexico City (1985) or of an intermediate one in Kobe (1995)

Twisting of the building about a vertical axis is more often due to the horizontal eccentricity of the inertia forces with respect to the "centre of stiffness" of the floor(s) than to the rotational component of the motion itself about the vertical. In such cases, twisting takes place about a vertical axis passing through the "centre of stiffness" which is closer to the "stiff side" in plan and produces the maximum displacements and the most severe damage to the perimeter elements on the opposite, "flexible side". The example in Fig. 2.16 is typical of such a response and its consequences: twisting about the corner of the building plan where the stiff and strong elements were concentrated (including a wall around an elevator shaft, the staircase, etc) caused the failure of the elements of the "flexible side"; the seismic displacements on that side, as increased by twisting, exceeded the – otherwise ample – ultimate deformation of these columns.

(a)  (b) 

Fig. 2.16 Collapse of flexible sides in torsionally imbalanced building with stiffness concentrated near one corner.

The collapse of the strongly asymmetric one-storey building in Fig. 2.17 demonstrates the opposite effect: calling the side in Fig. 2.17(a) as front, the vertical elements of the back side were shear-critical "short columns", developing higher shear forces than the columns on the front, owing to their much larger stiffness and short length. However, they did not have sufficient shear strength to resist these forces. They collapsed, pushing out the columns of the front side as well.

The remark about "short columns" brings up the effects of earthquakes on typical concrete members: columns, beams, the connections between them ("joints") and walls.

(a) 

(b)  (c) 

Fig. 2.17 Shear failure of short columns on stiff side (inside red rectangle) causes collapse of flexible side as well.

2.2.3 Member behaviour and failure

Typical seismic damage or failures of columns, joints, beams and walls are shown in Figs. 2.18 to 2.23 and are commented in the following.

2.2.3.1 Columns

Columns may be damaged or fail in flexure, as shown in Fig. 2.18. Flexural damage or failure phenomena are concentrated in horizontal bands at the very top or bottom of a column in a storey (where the bending moments are at maximum). Such regions are the physical manifestation of flexural "plastic hinges", where the plastic rotations take place. It is clear from Fig. 2.18 that "plastic hinging", although essential for the seismic design of the building for ductility and energy dissipation (see Sections 3.2.2, 3.2.3 and 4.6.3), is not painless: it implies damage, normally reparable, but sometimes not (especially if it is accompanied by irreversible residual horizontal drifts). Flexural damage always includes a visible horizontal crack and loss of concrete cover, often accompanied by bar buckling, opening of stirrups or partial disintegration of the concrete core inside the cage of reinforcement; sometimes one or more vertical bars rupture or the concrete core completely disintegrates. The cyclic and reversed nature of the deformation imposed on concrete elements by the earthquake plays an important role on its response: the opposite sides of the element are cyclically subject to tension and compression; when in tension, transversal cracking occurs but, then, when the force changes to compression the crack closes and the concrete cover may be lost (if the compressive strain is too large). Additionally, if the lateral restraint of the longitudinal bars is insufficient, the bars on the compressed face may buckle outwards, rupturing the stirrups and accelerating the loss of the concrete cover. Note that the Bauschinger effect decreases very sharply the buckling resistance of bars that have yielded previously in tension.

(a)  (b)  (c) 

Fig. 2.18 Flexural damage (a) or failure (b, c) at column ends

A column may fail in shear anywhere between its two ends, the end regions included (since the shear force is essentially constant along the height of the column). The signature of a shear failure is a diagonal crack or failure zone (Fig. 2.19); sometimes such cracks or zones form in both diagonal directions and cross each other. If the column carries a low axial load relative to its cross-sectional area, the inclination of the shear failure plane to the horizontal is about 45^{o}; it is steeper, sometimes over 60^{o}, if the column is heavily loaded. In columns engaged in two-way frame action, the shear failure plane may be at an inclination to both transverse directions of the column. Stirrups intersected by the diagonal failure band(s) may open or break. The concrete may disintegrate all along the diagonal failure zone or across the full core inside the reinforcement cage (especially if failure is not due to one-way shear, parallel to a single transverse direction of the column). For shear, the cyclic and reversed nature of the earthquake effects on the elements is even more important than for flexure. In fact, as the direction of the shear alternates, two “families” of diagonal cracks form, intersecting each other and leading to a very fast disintegration of the concrete. Additionally, since the horizontal stirrups are in tension for both directions of shear, diagonal cracks do not close upon reversal of the force; hence, the cracks become wider ever more, causing a very fast degradation of the lateral stiffness and strength of the column, denoting a so called brittle failure.

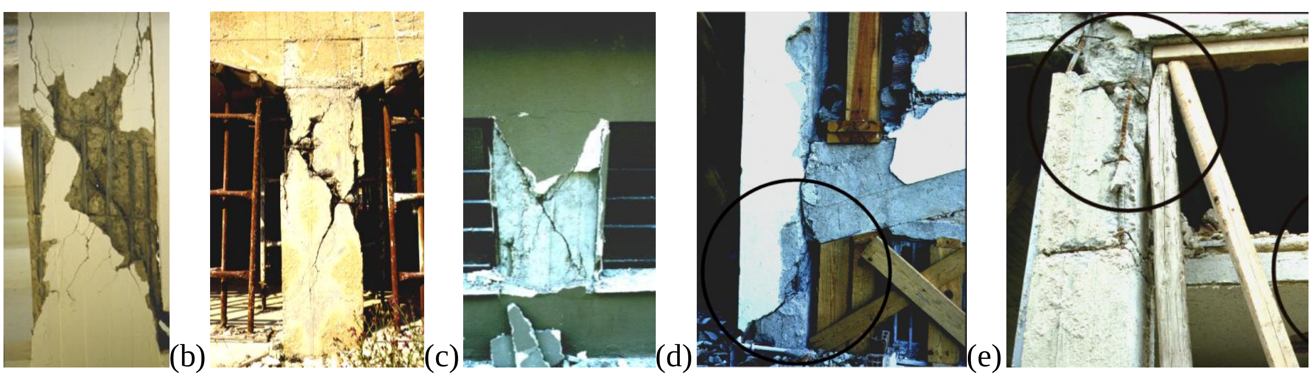

(a) (b) (c) (d) (e)

Fig. 2.19 Shear failure of columns, including a captive one between the basement perimeter wall and the beam (c) and short columns due to mid- storey constraint by a stair (d) or a landing (e) supported on the column.

Cases (c) to (e) in Fig. 2.19 are "short columns", which develop very high shear force demands and are very vulnerable to shear; the one in (c) is made "short" by design: those in (d) and (e) unintentionally, as the secondary elements supported by the column between its two ends split its free height to two shorter ones. The back side columns in Fig. 2.17, whose failure triggered the global collapse of the building, were also short.

Except for the one in Fig. 2.18(a), all columns in Figs. 2.18, 2.19 have essentially lost their entire lateral resistance and stiffness: they will not contribute at all against an aftershock or any other future earthquake. However, except for the column in Fig. 2.18(c), they all retain a good part of their axial load capacity. Note that the "quasi-permanent" gravity loads normally exhaust only a small fraction of the expected actual value of the axial load capacity of the undamaged column. On the other hand, the overlying storeys, thanks, among others, to their masonry infills, can bridge over failed columns working as deep beams. So, buildings with many failed columns or a few key ones in a storey are often spared from collapse. For example, very few columns were left in the building of Fig. 2.20 with some axial load capacity. Another example are the six storeys above the failed corner column in Fig. 2.21(a), which survived by working as a 6-storey-deep multilayer-sandwich cantilever beam, with the concrete floors serving as tension/compression flanges or intermediate layers and the infills as the web connecting them.

Fig. 2.20 Despite complete failure of columns across the ground storey, their residual axial load capacity still supports gravity loads.

(a)  (b)  (c) 

Fig. 2.21 Shear failure of beam column joints

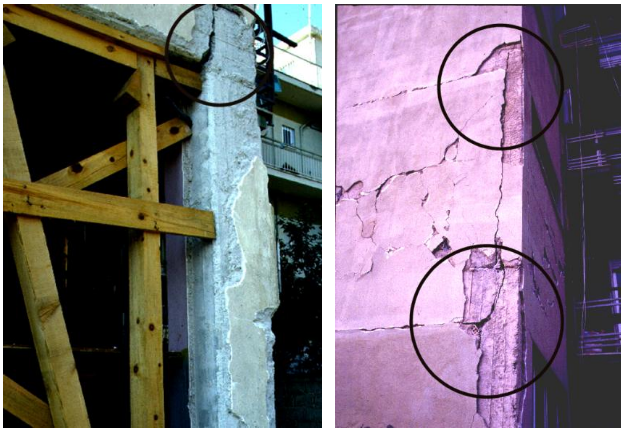
2.2.3.2 Beam-column joints

As explained in Section 4.4.3.1 with the help of Fig. 4.12, an earthquake introduces very high shear stresses to the core of a beam-column joint. These stresses are parallel to the plane of frame action. Effects of such shear stresses are shown in Fig. 2.21: in (a), complete diagonal failure of an unreinforced joint; in (b), (c), diagonal cracking in reinforced joints. These effects are clearly manifested in exterior joints, especially corner ones (Fig. 2.21(a), (b)). Interior joints profit from the confinement by the slab on all four sides and by the beams in any direction they frame into the joint.

The joints provide also the anchorage zone of beam bars, whether they terminate there (as in corner joints, see Fig. 2.22(a)), or continue into the next beam span across the joint. The next sub-section addresses this issue.

2.2.3.3 Beams

Beam bars with insufficient anchorage in a joint may pull out in an earthquake. Such a failure of bond and anchorage shows up at the end section as a crack through the full depth of the beam (Fig. 2.22(a)). A characteristic feature of a pull-out crack is its large width, well in excess of the residual crack width typical of yielding of the steel (which is a fraction of a mm or around 1 mm). The impact of this type of bond failure on the global behaviour is not dramatic: the beam cannot develop its full moment resistance at the end section and the force resistance and stiffness of the frame it belongs to drops accordingly. The damage is reparable, although the original deficiency, namely the poor anchorage of beam bars in the joint, cannot be corrected easily.

(a)  (b) 

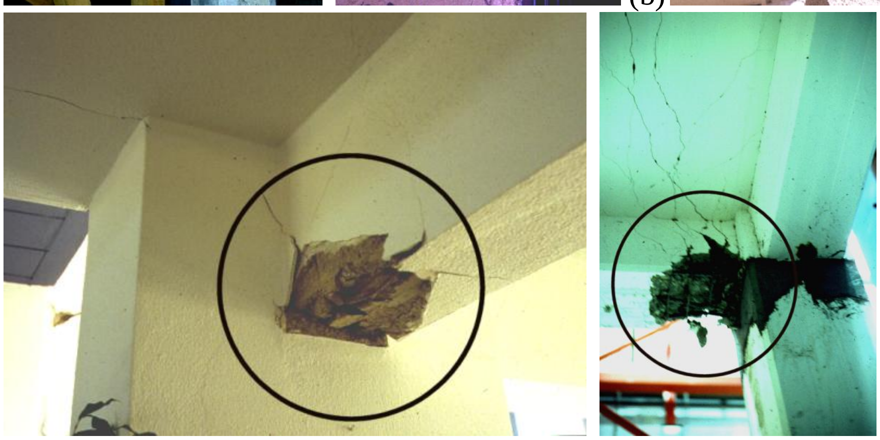
(c) 

Fig. 2.22 Typical features of beam behaviour: (a) pullout of beam bars from narrow corner column, due to short straight anchorage there; (b) wide crack in slab at right angles to the beam at the connection with the columns shows the large participation of the slab as effective flange width in tension; (c) failure, with concrete crushing and bar buckling at bottom flange next to the column.

Beams are designed to develop flexural plastic hinges at the ends and are expected to do so in an earthquake. The loss of beam anchorage highlighted above is part of such flexural action (although it prevents a proper plastic hinge from forming). A standard feature of a flexural plastic hinge in a beam is its through-depth crack at the face of the supporting beam or column, with a residual width indicative of yielding of the beam bars; that crack often extends into the slab and travels a good distance at right angles to the beam, sometimes joining up with a similar crack from a parallel beam (Fig. 2.22(b)). The length and the sizeable residual crack width of such an extension show that the slab fully participates in the flexural action with its bars which are parallel to the beam, serving as a very wide tension flange. Flexural damage is mostly associated with cracking and spalling of concrete and yielding of the reinforcement. By contrast, flexural failure comes with disintegration of concrete beyond the cover, often with buckling (or even rupture) of bars. Such effects (demonstrated in Fig. 2.22(c)) happen only at the bottom flange of a beam, because the slab provides the top flange with abundant cross-sectional areas of concrete and steel reinforcement. Larger amount of top reinforcement at the supports also result from the design for the hogging moments due to the factored gravity loads (the "persistent and transient design situation" of EN 1990). Note that a bottom reinforcement smaller than the top one, is unable to close the crack at the top face (as it is unable to yield the top reinforcement in compression): the vertical crack at the face of the support, across the full depth of the beam, tends to remain open and increase in width for each cycle of deformation; bottom bars may buckle and then rupture under the large cyclic excursions of strain across the open crack.

2.2.3.4 Concrete walls

Flexural or shear damage and failure phenomena in walls (Fig. 2.23(a) and (b)) are similar to those in columns, but take place almost exclusively above the base of the wall, and very rarely in storeys higher up. One difference concerning flexure is that spalling and disintegration of concrete are normally limited to the edges of the wall section (Fig. 2.23(a)). Owing to the light axial loading of the wall section by gravity loads, diagonal planes of shear failure are normally at about 45^{o} to the horizontal (Fig. 2.23(b)). Walls have lower friction resistance than columns, owing to their lower axial stress level and vertical reinforcement ratio; so, they may slide at their through-cracked base section, which happens to coincide with a construction joint (Fig. 2.23(c)).

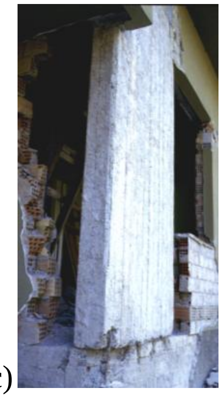
(a)  (b)  (c) 

Fig. 2.23 Typical failures of concrete walls: (a) flexural, with damage in shear; (b) in shear; (c) by sliding shear.