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Earthquake Design Practice for Buildings

Fourth edition

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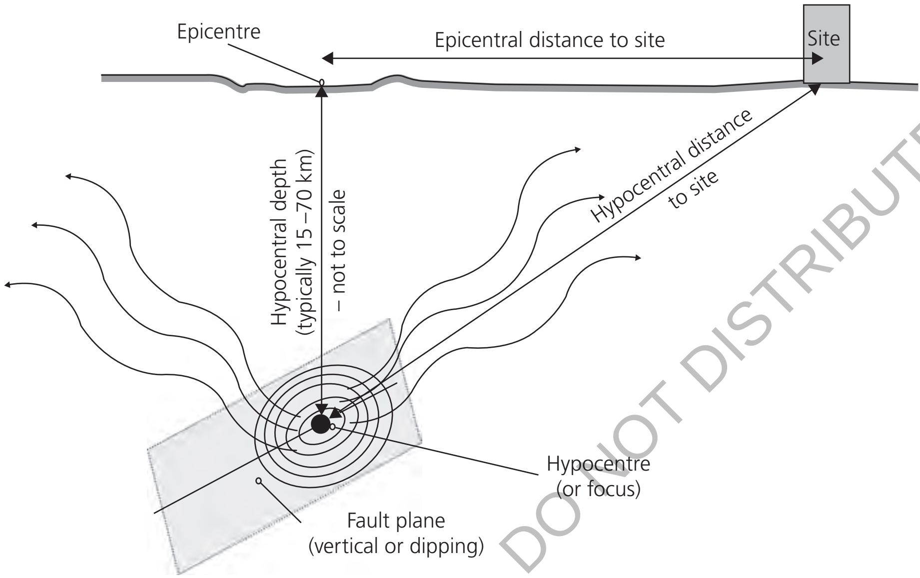
Figure 2.2 Describing an earthquake's source location



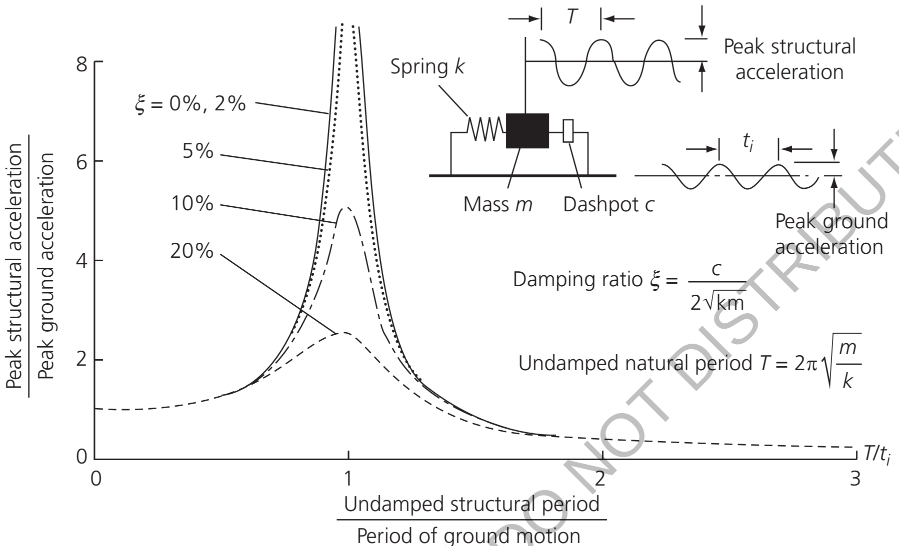
surface. Unlike magnitude, the intensity of a given earthquake depends on the location at which it is measured; in general, the larger the epicentral distance (see Figure 2.2) the lower the intensity. Thus, a given magnitude of earthquake will give rise to many different intensities in the region it affects. It is important to recognise this fundamental distinction between the two measures, which are discussed below in more detail.

2.2.2.2 Earthquake magnitude

A number of different magnitude scales exist. Three common scales are: (*a*) local, or 'Richter' magnitude, M_L; (*b*) the body wave magnitude, m_b (suitable for measuring smaller magnitude events); and (*c*) the surface wave magnitude, M_s (suitable for larger events). All three are measured by sensitive instruments (seismometers) that detect ground tremors at great distances from the earthquake source. Due to the way that ground motion scales with energy release, these scales 'saturate' for higher magnitude values (i.e. they are not able to discriminate between higher magnitude events). For this reason, a fourth scale, the moment magnitude, M_w, is now generally favoured. This is directly related to the estimated energy release at the earthquake source and is suitable for all sizes of event.

In broad terms, an earthquake with a magnitude less than 4 on any of the scales is unlikely to cause significant damage, while magnitudes larger than 8 are rare events affecting very large areas. Because of the logarithmic nature of the scale, a 1-point increase in magnitude represents a 30-fold increase in energy release and a 10-fold increase in the amplitude of seismic waves, although the violence of the accelerations close to the epicentre increases much more slowly with magnitude. It is the spatial extent and long duration of large earthquakes that make

Figure 5.2 Steady-state response to sinusoidal ground motion



damping ratio (e.g. for 5% damping the peak is 10 times the input amplitude, and for 0.5% it is 100 times). This is clearly very important for harmonically loaded, lightly damped structures, such as some building floor systems responding to foot traffic.

Away from the resonant peak, the dynamic amplification is much less dramatic. Very rigid systems with low periods track the ground motion closely. The normalised response therefore tends to unity as the system period tends to zero, or (equivalently) as the ground motion period becomes very long in comparison to the period of the structure. Very flexible springs, on the other hand, act to isolate their masses from the input motion, and so the response tends to zero when the period of the structure is very long compared with that of the ground motion. This is the principle behind, for example, isolation mounts for rotating machinery and also seismic isolation systems for earthquake-resistant buildings (as discussed in Section 13.3).

5.3.3 Response of single degree of freedom systems to earthquake ground motion

The free vibration and harmonic excitation cases described in the previous sections can be solved in closed form, and the problem is fully defined by just a few parameters. The response of an SDOF system to an earthquake motion at its base is a function not just of the structural properties (period and damping, as described previously), but also the ground acceleration at all points in time. It must therefore be calculated numerically.

Various time integration algorithms are available for carrying out ‘response history analysis’ (also referred to as ‘time history analysis’), wherein the response of the SDOF system (displacements or accelerations) is calculated across time. The ground motion is broken up into short discrete intervals (time steps), and the response is calculated step by step. The time step

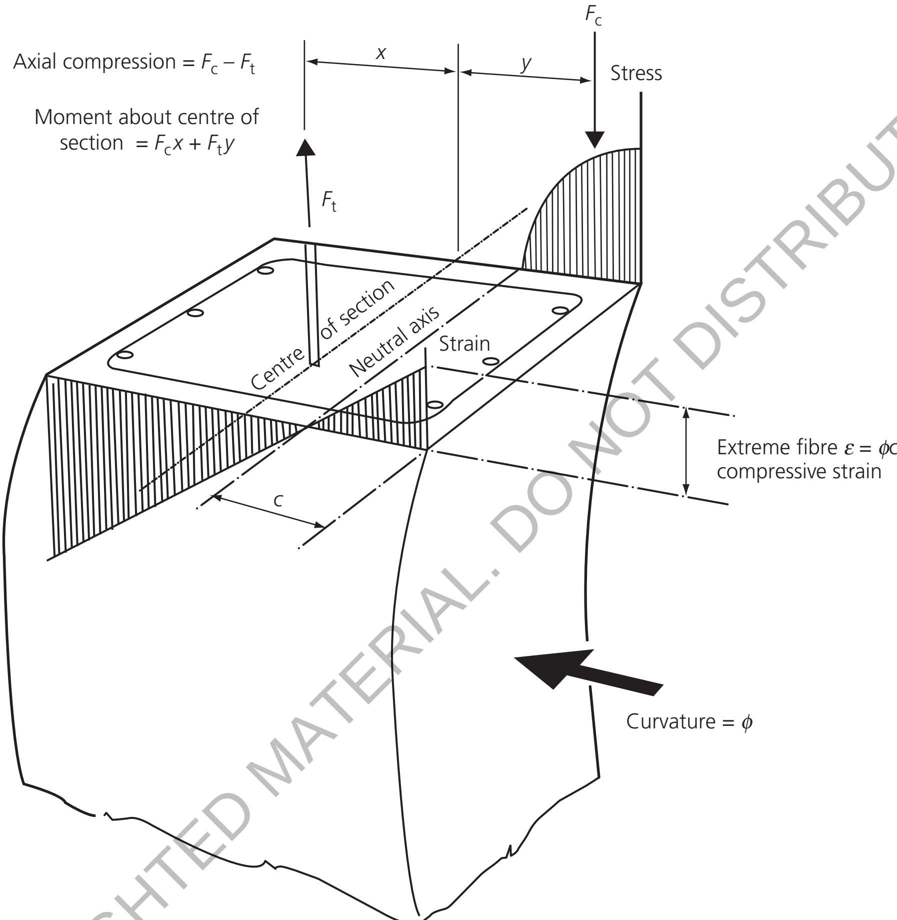
Figure 8.5 (a) Intermediate and (b) upper storey collapse of a multi-storey reinforced concrete structure in Mexico City, Mexico, 1985. ((a) © E. Booth; (b) photograph by M. Winney)

(a) Intermediate floor

(b) Top floor

Figure 8.6 Total collapse of a multi-storey reinforced concrete building in Baguio, Philippines, 1990. (© E. Booth)

Figure 8.29 Calculation of the moment curvature relationship under uniaxial bending, assuming that plane sections remain plane

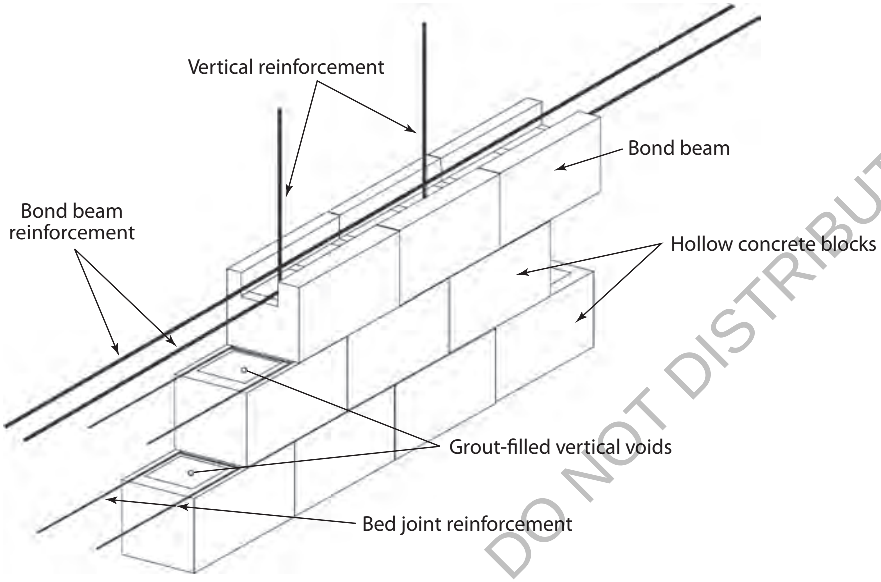


fairly readily programmed even in a spreadsheet). The result is a moment–curvature relationship, as shown in Figure 8.27.

In a purely mechanics-based approach, the moments would then be translated along the member (defined by equilibrium) into curvatures (from the moment–curvature relationship), and integrated to obtain the total chord rotations. This, however, does not give good agreement with experimental results, as a number of aspects of the real behaviour of reinforced concrete members (tension shift, shear deformation, strain penetration past the 'fixed' end of the member, and local jumps in curvature at crack locations) have been ignored (Priestley *et al.*, 2018).

Therefore, for reinforced concrete members, it is much more common (and justified) to make the assumption of a discrete plastic hinge at the ends of the member (of length L_{pl}) with

Figure 10.4 Typical reinforced hollow concrete blocks



In critical regions (at the base of walls where plastic hinges are expected to develop), allowing special boundary zone detailing (as required for reinforced concrete structural walls; see Section 8.7.5.3) may enhance the ductility capacity; this is not required or recognised in EC8, but is referred to in US and New Zealand practice. In the New Zealand masonry standard NZS 4230 (NZS, 2004), provisions are also included for the use of confining plates (steel plates placed horizontally in bed joints) in critical regions, to allow higher levels of compression strain to be achieved where this is required to develop a flexural plastic hinge.

Additional horizontal steel may also be required in continuous peripheral 'bond beams' (see Figure 10.4), although EC8 does not specify these; the beams are placed at floor levels and possibly also at intermediate levels when shear demands are high. Lintel beams over door and window openings (a universal requirement) may be formed in a similar way.

Figure 10.4 shows a wall with full grouting, in which all the voids are filled with grout, whether or not they have vertical reinforcing bars; this is required to achieve higher shear strength in the plastic hinge regions of ductile shear walls. For designs in which a limited ductility demand is envisaged, partially grouted walls in which the unreinforced voids are not grouted are commonly used; however, this practice results in a lower shear strength, and it may be difficult to prevent some of the grout leaking into the unreinforced voids. North American codes restrict the use of partial grouting to low-rise buildings; in New Zealand, full grouting is required in critical regions of 'limited ductile' shear walls, and everywhere in 'ductile walls'. Where horizontal reinforcement is placed in voids in masonry units rather than in the bed joints, full grouting is also needed to protect and anchor the horizontal bars.