1

Appealing Shell Structures

After reading this chapter you should be able to:

- List the main subsectors and components of a tank's design and analysis
- Explain the function of each element
- Identify the behaviour related to seismic and static performance

1.1 Beams and Arches

The structural design process has traditionally been, and still is, essentially carried out for elements subjected to bending actions (beams and slabs), generally controlled by a flexural behaviour, upon which the design is based. Once flexural resistance has been ensured, these members are verified to prevent excessive deformation or shear failure. In the case of members loaded by a combination of bending moments and axial compressive loads (columns or walls), the preliminary design is often based on the axial component only and the combination with flexural action is then verified (in the case of tanks, bending is sometimes considered just to check the possibility of buckling).

These simplified approaches assume the presence of structural components able to resist either tensile or compressive stresses, such as concrete and steel bars in reinforced concrete elements. The design is thus based on an estimate of the loads to be carried by each member, and subsequently on the design of sections where the maximum resulting bending moment is expected. As an example, consider a beam where the moment acting on a section is estimated as $M = \frac{ql^2}{\alpha}$, where q is the applied load per unit length, l is the length of the element and α is a coefficient that depends on the end constraints. This acting moment has to be balanced by a couple, estimated as the result of internal tensile and compressive actions, equal to each other and multiplied by the distance between their approximate points of application to compute a resisting moment.

It is thus quite understandable that in ancient times the material mainly used for roofing systems was timber, combined in boards, joists, beams, and girders with progressively increasing capacity.

The only viable, though more complex, alternative was to resort to an arch, which was able to cover long spans using materials able to resist only compressive actions, such as brick or stone masonry. Though

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widely applied in ancient times, it was Robert Hooke[1] in 1675 who first clearly expressed the basic concept that allows the design of a fully compressed arch. His statement was simple, though very comprehensive: a perfectly compressed arch shall have a shape in reverse and identical to that which a suspended cable would assume under the same load combination. For example, under a uniformly distributed load, the cable would assume a parabolic shape, with an upward concavity and that should be the geometry of a compressed arch, with the concavity oriented downward.

As stated, this conceptual solution does not offer a relevant clue as to how to design an arch when several different load configurations are considered, nor takes into account the effects of abutment constraints, etc. The problem is thus far more complex and, as common in the past (and present) building engineering practice, crucial simplifications were adopted for sizing and preliminary design, e.g. assuming that the horizontal reaction at the abutment (P_H) can be approximated by the equation applicable to a three-hinged arch case:

$$P_H = \frac{ql^2}{8h} \tag{1.1}$$

where q and l are the load per unit length and the span length, and h represents the height of the arch.

It is interesting to compare an arch and a beam used to span a similar length supporting similar weights. Consider thus a three-hinged arch, assume the horizontal forces are eliminated at the abutments by means of some horizontal tie, and compare it to a beam of equal span, simply supported at both ends, assuming that it is made by an elastic material with a similar behaviour in tension and in compression.

Immediately one notes that the same external moment induced at midspan by the applied loads ($M_m = \frac{ql^2}{8}$, assuming a uniformly distributed load per unit length), must be equilibrated by internal action couples characterized by quite different arms. For the case of an arch, the internal couple results from forces located in the centre of the mass of the arch (compression) and of the tie (tension), while for the case of a elastic beam, they are applied at points located at a distance of two-thirds of the beam height. When a fully plastic response is assumed, and thus a constant value is assumed for both tensile and compressive stresses, the distance between the resultant forces is one half of the section depth (d, Figure 1.1(a)). In this case, the beam's internal action would be:

$$P_t = P_c = \frac{ql^2}{4d} \tag{1.2}$$

Consequently, assuming an identical strength in compression and in tension, f_w, and a given width of the beam section, b_b, the required section depth (d_b) could be derived as::

$$A = \frac{b_b d_b}{2} = \frac{ql^2}{4d_b f_w} \qquad \text{and thus} \qquad d_b = \sqrt{\frac{ql^2}{2b_b f_w}} \tag{1.3}$$

For the sake of simplicity, assume now that the arch and the tie are also made with materials with the same compression capacity (the arch) and tensile capacity (the tie). Assuming that both compression and tensile forces will act at the centre of the corresponding element, each force can be derived from Equation (1.1), and consequently the required depth of arch (d_a) and tie (d_t) would be:

$$A_a = A_t = b_{a,t} d_{a,t} = \frac{ql^2}{8hf_w} \qquad \text{and thus} \qquad d_{a,t} = \frac{ql^2}{8b_{a,t} f_w} \tag{1.4}$$

1 Ut pendet continuum flexile, sic stabit contiguum rigidum inversum.

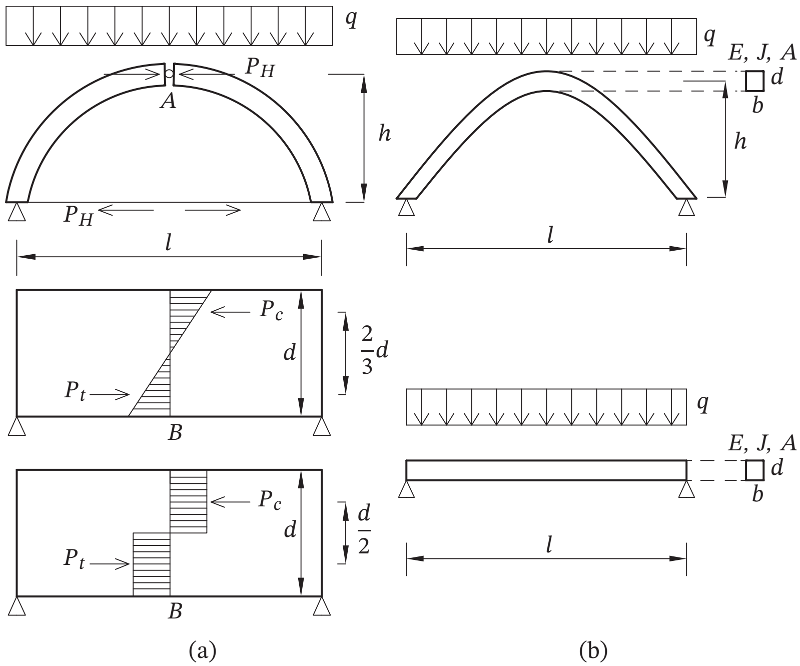


Figure 1.1 (a) Three-hinged arch with a uniform load on top; (b) two-hinged parabolic arch and simply supported beam.

Assuming that all considered elements have the same width ($b_a = b_t = b_b$), then the depth of the arch and the tie can be computed as a function of the depth of the beam, combining Equations (1.3) and (1.4):

$$d_{a,t} = \frac{d_b^2}{4h} \tag{1.5}$$

It can immediately be verified that for reasonable values of the rise of the arch compared to its span (l_a, e.g. $h = \frac{l_a}{4}$) and of the height of the beam, compared to its span (l_b, e.g. $d_b = \frac{l_b}{15}$), the depth required for the arch and the tie is at least 10 times less than the one required for the beam.

Applying the same uniform load on a two-hinged parabolic arch (as shown in Figure 1.1(b)) and on a simply supported beam with the same span (both with a rectangular section $A = bd$), the deflection of the arch at the keystone (Point A) and of the beam at midspan (Point B) can be calculated as follows:

$$w_A = 2.3 \cdot 10^{-5} \frac{ql^4}{EJ} \tag{1.6a}$$

$$w_B = \frac{5}{384} \frac{ql^4}{EJ} = 0.013 \frac{ql^4}{EJ} \approx 565 \cdot w_A \tag{1.6b}$$

The apparent overall stiffness differs by two or three orders of magnitude.

This rather trivial example is just a first case study in which the superiority of curved geometry structures is shown in terms of the required material to obtain similar strength or deformation capacities under gravity loads, when compared to similar structures based on straight geometry. The more complex case of cylindrical vs. rectangular tanks will offer more, possibly not as trivial, evidence.

1.2 Plates and Vaults

As already mentioned, a common technology to cover a rectangular area was based on the properties of timber, a material readily available, easy to work, and structurally attractive. This technology is made by a combination of linear elements, overlaying girders, beams and joists, until reasonable span dimensions are achieved to apply boards of reasonable thickness.

Covering the same area using a single plate would require some homogeneous material capable of carrying shear and bending moments in two directions. Clearly this is feasible, though impossible in practical terms, using a steel plate, but became a viable alternative only with the advent of reinforced concrete. Its potential for an isotropic (or rather orthotropic) behaviour, the possibility of shaping its geometry and tapering its thickness, the separation of the internal elements countering compression and tensile stresses, appear to be an ideal combination to build an efficient horizontal slab.

Consider first a simple comparison between a simply supported beam and a similar one-way slab of indefinite width. The bending moment will be expressed by the same equation, while the slab stiffness will increase because of the hindered transversal dilatation. This effect will be accounted for by a correction factor to be applied to the beam stiffness equal to $1 - \nu^2$, where ν is the Poisson coefficient, in the range of 0.15 for concrete. The correction will thus be in the range of 2% not as relevant.

A much more relevant effect will become evident if comparing a one-way and a two-way response, particularly when the two sides of the slab will not differ much.

Take the example of a simply supported square plate, with a uniform load p, and assume an isotropic elastic response (i.e. in the case of concrete, neglecting any cracking phenomenon). In the case of a two-way response, the maximum bending moment and the deflection at the centre of the plate will be calculated as:

$$m_{\max} = 0.04416\ pa^2 \tag{1.7a}$$

$$f = 0.00406\frac{pa^4}{B} = 0.04677\frac{pa^4}{Es^3} \tag{1.7b}$$

where a is the side of the plate, s its thickness and $B = \frac{Es^3}{12(1-\nu^2)}$ its flexural stiffness.

Considering the same geometry and the same load, but hinged supports on two opposite sides only, the bending moment and flexural deflection will be those of a simply supported beam (possibly with the minor stiffness correction mentioned above, not applied in the equations):

$$m_{\max} = \frac{1}{8}pa^2 = 0.125\ pa^2 \tag{1.8a}$$

$$f = \frac{5}{384}\frac{pa^4}{EJ} = 0.15625\frac{pa^4}{Es^3} \tag{1.8b}$$

The values of bending moments and deflection calculated for the beam are thus approximately three times those obtained for the bidirectional plate.

It is easy to observe that a barrel vault sustained by continuous supports on two sides can be regarded as a tri-dimensional transformation of an arch with the corresponding transversal section. Its basic structural behaviour under gravity loads can thus be derived from that of an arch (Figure 1.2(a)). Things are quite different when a barrel vault is not sustained along the support lines perpendicular to the arch section, but rather along the other two sides (Figure 1.2(b)) or even by punctual vertical support located

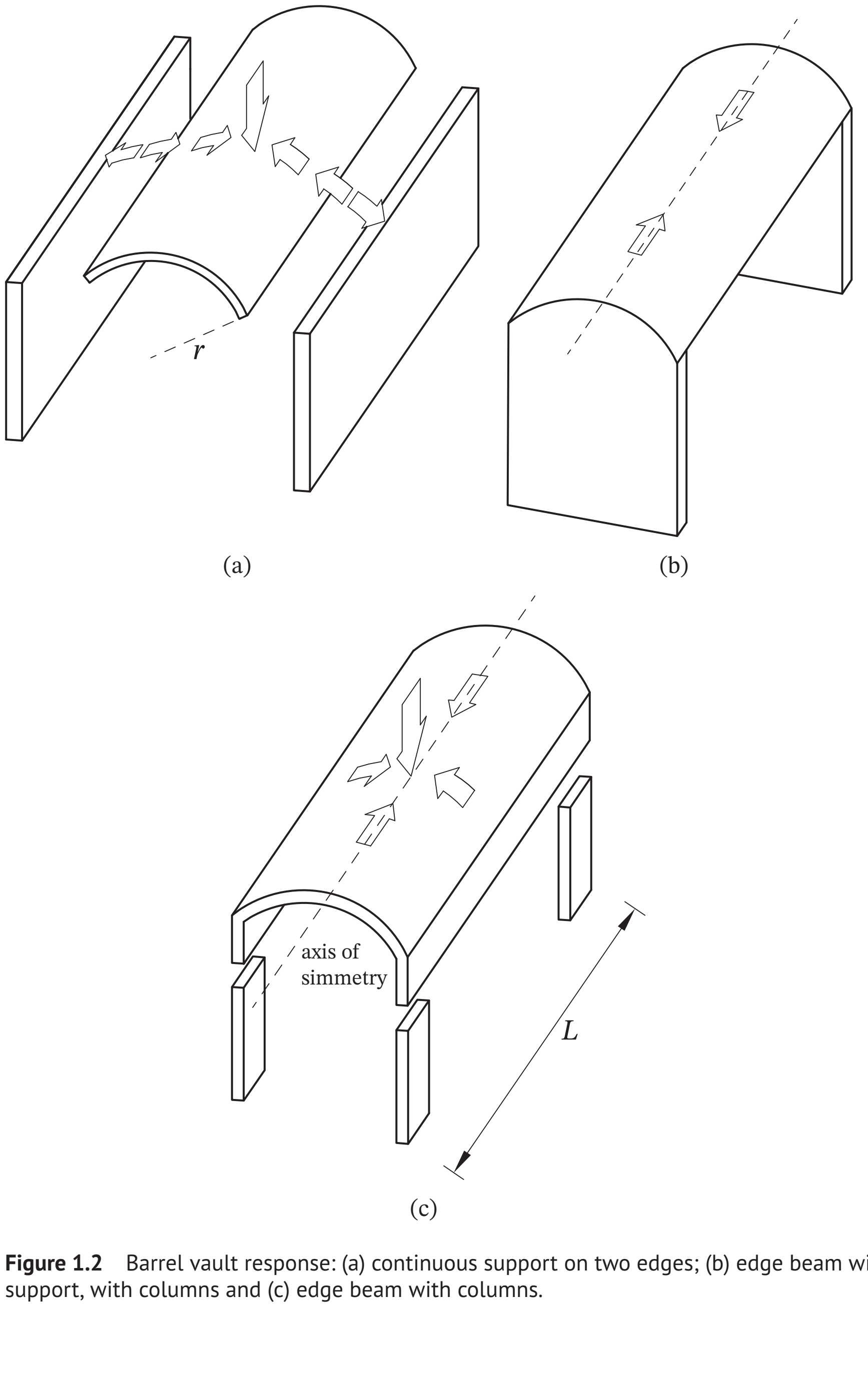


Figure 1.2 Barrel vault response: (a) continuous support on two edges; (b) edge beam without continuous lateral support, with columns and (c) edge beam with columns.

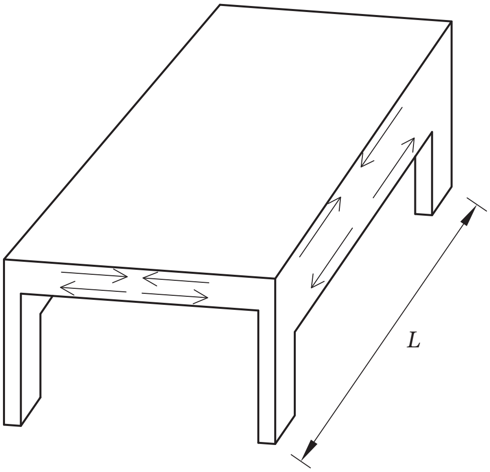


Figure 1.3 Flat roof plate on side beams.

at the corners (Figure 1.2(c)). The structural system becomes much more complex, with the barrel vaults forced to act somehow as a longitudinal beam, coupled with transversal arches. This response can be compared with that of a structure made of longitudinal beams and a transverse slab (Figure 1.3).

The structural response of the barrel vault can be decomposed into two interacting mechanisms: an arch action in the transversal direction and a beam action in the longitudinal direction.

While in the case of slab and beams the components' responses are essentially decoupled, the behaviour of the barrel vault is more complex, because the two systems interact between them, with a resulting combined response that depends mainly on the ratio between the longitudinal length of the vault (L) and its radius of curvature (r).

The traditional beam theory can be applied as a first approximation to calculate internal forces and deformation of the vault structure in the longitudinal direction. The applied bending moment can thus be readily computed and section equilibrium will allow the determination of stresses along the arch-shaped section of the beam. In general, if free rotation is assumed at the ends, the stress distribution will look like that shown in Figure 1.4(a). Unfortunately, one fundamental assumption of beam theory is that

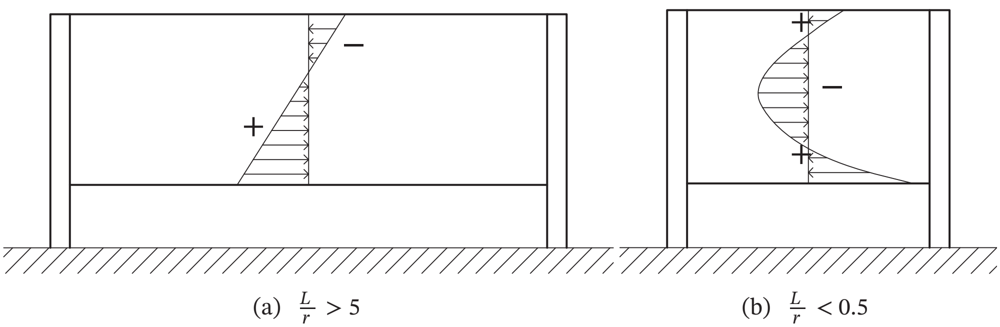


Figure 1.4 Qualitative distribution of internal stresses in a transversal arch section of a barrel vault, for (a) large or (b) small $\frac{L}{r}$ ratios.

the transversal plane section will remain plane in the deformed configuration and this is not always an acceptable approximation for barrel vaults. Actually, refined numeral analysis has shown that this assumption is valid only for relative values of the fundamental parameters mentioned above, i.e. essentially for $\frac{L}{r} > 5$. Obviously, this can be obtained also by taking measures to restrain section deformations, such as, for example, inserting transversal walls or ties [499]. It is clear that in these cases all the advantages of a deep beam will apply, with a large lever arm between compression and tension resultants and small displacements.

In the case of unrestrained relatively short vaults, the section deformation may differ significantly from a straight line and the related internal stress distribution will be affected (Figure 1.4(b)).

This problem is no different from that of any deep beam with a complex transversal section, which counts on shape more than on mass material to resist the applied forces. As pointed out, a main design problem in these cases is to take measures to control the section deformations. A second fundamental problem (in the case of reinforced concrete) is related to the high potential for significant cracking of the parts subjected to tension, which may not be adequately controlled by means of a proper reinforcement distribution. A possible viable solution may be found in a rational application of post-tensioning, which will not only be effective in controlling cracking, but also in reducing the deformation. Possible qualitative arrangements of cables are illustrated in Figure 1.5, as a function of the presence of beam-like elements at the edges of the vault. It is evident in Figure 1.5(a) that a rational longitudinal disposition of the cables in a vault will imply potentially significant effects on the transversal response since the sectional arch sections will be subjected to varying transverse forces and bending moments, not necessarily negligible nor necessarily favourable.

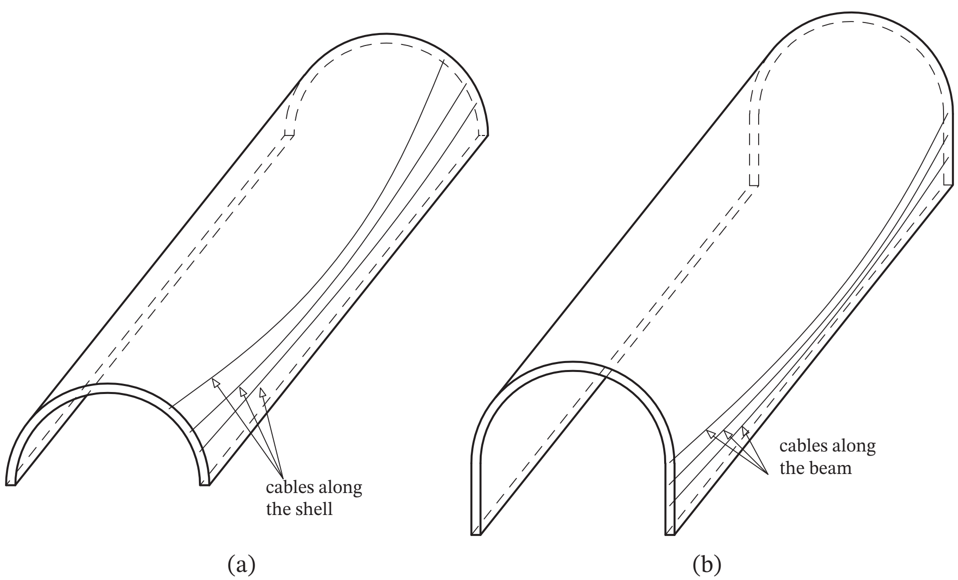


Figure 1.5 Qualitative disposition of post-tensioned cables (a) along the shell and (b) along the edge beams.

1.3 Rectangular and Cylindrical Tanks

The vertical walls of a rectangular tank can be regarded as a series of vertical plates, usually fixed on three sides and free, simply supported or fixed on the fourth one (the side shared with the roof). Each plate is generally subjected to its own weight (in plane) and to the forces generated by the contained material (mainly out of plane), which generate a variable pressure along the height. In the absence of internal friction, the maximum horizontal pressure at the base of each plate is $P_0 = \gamma H$, where γ is the weight per volume unit of the fluid and H is the height of the tank. This pressure varies linearly with the height, becoming zero at the top. As such, the pressure along the height can be evaluated as [184]:

$$p_z = -\gamma(H - y) \tag{1.9}$$

For squat and large tanks, the acting forces are mainly equilibrated by a vertical cantilever action while the contribution of the plate effect in the horizontal direction can be ignored. If this is the case, and it is further assumed that no constraint is provided at the top side, the maximum bending moment per unit length at the base is:

$$M_0 = -\frac{\gamma H^3}{6} = -\frac{1}{6}P_0H^2 = -0.1667P_0H^2 \tag{1.10}$$

With a progressive reduction of the horizontal measure of each plate with respect to their height, the contribution of the transverse reaction becomes progressively more important. For example, when the height is equal to the horizontal span (i.e. the tank assumes a cubic shape), the same maximum moment around the base line of of each plate is reduced to less than one-fifth, becoming:

$$M_0 = -0.0299P_0H^2 \tag{1.11}$$

The estimation of this maximum bending moment is one of the crucial issues when the flexural response controls the design of the structure and is often used as a basic preliminary parameter to define the required wall thickness at the base of the tank.

As an example, with the aim of getting some feeling about figures, consider the case of a cubic tank with side length of 20 m, with a consequent total capacity of 8000 m^3 of liquid, assumed to be water. The resulting bending moment M_0 at the base of each wall is:

$$M_0 = -0.0299P_0H^2 = -0.0299\gamma H^3 = -0.0299 \cdot 10 \cdot 20^3 = -2392 \text{ kNm/m} \tag{1.12}$$

Assuming that the design moment at the ultimate limit state is factorized to 1.3 times ($M_0^{slu} = -1.3 \cdot 2392 = -3110$ kNm/m) and that the neutral axis depth is approximately equal to $\xi = \frac{x}{d} = 0.35$, the required concrete thickness s and the corresponding amount of vertical reinforcement A_s can be estimated as:

$$d = \sqrt{\frac{|M_0|}{0.8\,\xi\, b(1 - 0.4\xi)f_{cd}}} = \sqrt{\frac{3110 \cdot 10^6}{0.8 \cdot 0.35 \cdot 1000(1 - 0.4 \cdot 0.35)17}} = 872 \text{ mm} \tag{1.13a}$$

$$s = d + 35 \approx 900 \text{ mm} \tag{1.13b}$$

$$A_s = \frac{|M_0|}{0.9df_{yd}} = 10276 \text{ mm}^2/\text{m} \tag{1.13c}$$

where f_{cd} is the concrete design strength, f_{yd} is the yield strength of steel, b is the unitary width of the wall, and d is the depth of the section excluding the concrete cover, assumed to be 35 mm. The resulting vertical reinforcement percentage is 1.14%. The acting bending moment reduces to about 1000 kNm/m at midheight, hence the section can be tapered, the reinforcement can be reduced, or both. A possible solution is to reduce the wall thickness to 550 mm and the vertical reinforcement ratio to about 1%. Considering a linear tapering, the wall depth at the top will be 200 mm. The bending moment acting at midheight around a vertical axis is about 1400 kNm/m and will require a horizontal reinforcement ratio approximately equal to 1.4%. Cracking width and distance at the base can be checked by applying one of the several formulations proposed in different codes of practice. Considering as an example the equations recommended by the Italian code, NTC 2018, the serviceability limit state bending moment (~2400 kN/m) will be used, estimating the maximum distance between cracks as $\Delta_{\max} = 162$ and their width as approximately 0.2 mm.

Simple calculations can estimate the total amount of concrete required to cast the four walls of the tank, of about 880 m^3. A proper evaluation of the required reinforcement will imply some assumptions about appropriate minimum percentages for the compression sides of the sections, for corner detailing, for low stress regions. A reasonable total amount will be around 2200 kN, which corresponds to an average steel weight of 2.5 kN/m^3 or to an average reinforcement percentage in both directions of about 1.6%.

A tank with a similar volume capacity can be designed as a cylindrical structure, with a diameter of 20 m and a height of 25 m. In order to complete some simplified design as done above in the case of a square tank, and thus be able to compare the required material quantities, it is necessary to anticipate the basic features of the response of cylindrical tanks. This can be regarded as one of the simplest cases of a shell structure, which can be used as a case study for the development of a more general theory, to be applied to more complex geometrical shapes.

For this purpose, a number of key assumptions have to be accepted (similar to those applied to the simplified theory of plates):

1) the thickness is small compared to the smallest radius of curvature;
2) the resulting displacements will be small with respect to the shell thickness;
3) the cross-sections normal to the midsurface of the shell will remain straight and normal to the midsurface after deformation;
4) the component of internal stress normal to the midsurface is negligible if compared to the tangential components;
5) the stress distribution along the shell thickness is linear, and its value is zero at mid-surface.

Under these assumptions, the response of a cylindrical tank can be simplified in the interaction of two different reaction systems: a flexural response of vertical cantilevers, and a retaining action of a system of horizontal rings. The solution of this redundant problem must obviously respect both equilibrium and compatibility and was ingeniously developed between the end of the nineteenth and the beginning of the twentieth century.

To solve the problem it was first assumed one should ignore any flexural reaction at the base of the tank, i.e. to allow the base of the cylinder to rotate and to move freely along the horizontal plane. In this case, the horizontal acting forces will have to be equilibrated by the ring action only, while the vertical action will be simply equilibrated by a vertical reaction component. This situation is defined as a *membrane solution*, since bending is excluded, and shear in the plane tangent to the shell must also be zero because of the axi-symmetric condition of both loads and structure. Considering a tank containing a liquid with unit weight γ with no internal friction or cohesion, with height with no internal friction or cohesion, with height H, radius r, and a total self weight (or total applied vertical load) R, the solution described can be expressed by the following equations, as a function of internal vertical (N'_y) and horizontal (N'_θ) actions per unit length:

$$N'_y = \frac{R}{2\pi r} \tag{1.14a}$$

$$N'_\theta = \gamma(H - y)r \tag{1.14b}$$

where y is the vertical coordinate, being equal to zero at the base and to H at the top.

The horizontal displacement (w) and rotation around the tangent horizontal line (ϕ'_y) required by compatibility in this specific case were also calculated, as:

$$w = -\frac{\gamma r^2}{Eh}(H - y) \tag{1.15a}$$

$$\phi'_y = \frac{\gamma r^2}{Eh} \tag{1.15b}$$

where h is the thickness of the shell and E is the Young modulus of its constituent material. Note that the rotation is constant along the height and the displacement, consistently, varies linearly, from a maximum value at the base to zero at the top. Assuming a concrete shell, with a Young modulus $E = 30000$ MPa, a thickness h of 200 mm and uncracked sections, the displacement at the base would be 4.2 mm and the constant rotation 16.7×10^{-5} rad, which will lead to zero displacement at the top. In the case of some flexural and shear constraint at the base that would not allow either relative displacement or rotation or both, some flexural action will occur, and the displacement and rotation resulting from this action will have to properly combine with the membrane results to assure compatibility.

The flexural solution of the problem is fully derived and explained in [66], here we will simply mention and use it, as expressed by the following fourth-order differential equation and its solution:

$$\frac{d^4w}{dy^4} + 4\beta^4 w = \frac{p_z}{B} \tag{1.16a}$$

$$w = e^{-\beta y}(C_3 \cos \beta y + C_4 \sin \beta y) + f(y) \tag{1.16b}$$

where p_z is the horizontal pressure, $B = \frac{Eh^3}{12(1-\nu^2)}$ is the flexural stiffness of a plate with thickness equal to that of the shell and $4\beta^4$ is the ratio between the horizontal stiffness of the rings (k_r) and the flexural stiffness of the plate, expressed by the following equations:

$$k_r = \frac{Eh}{r^2} \tag{1.17a}$$

$$4\beta^4 = \frac{k_r}{B} = \frac{12(1 - \nu^2)}{r^2h^2} \tag{1.17b}$$

In a mathematical context, the function $f(y)$ is defined as "particular integral" and can here be represented by the membrane solution found earlier, which is "particular" in the sense that corresponds to the specific situation described above with reference to the base constraints. Again, in mathematical terms, the first term of the solution is the "general integral" of the associated homogeneous equation (i.e. without any applied pressure). This part of the solution depends on the boundary conditions (in this case, from the base restraints, which will determine the values of the constant C_3 and C_4).

It is intuitive that the parameter $4\beta^4$, i.e. the ratio between the cantilever and the ring stiffness, determines which one of the two systems will dominate the structural response. Equation (1.17b) is thus indicating that large radius, thicker tanks are more affected by the flexural response, while small, thin shells are dominated by the ring's reaction. It is interesting to observe that this is not the only relevant role played by the parameter β.

Actually, the wavelength of the damped sinusoid expressed by the general integral is $\lambda = \frac{2\pi}{\beta}$ and the damping factor expressed by the negative exponent of e is a linear function of β. Both effects have a clear physical meaning: if the stiffness of the rings system prevails on the flexural response, β is higher and consequently the period of the sinusoid is shorter and its effect is damped more rapidly. The damping factor can be expressed as a function of the wavelength, replacing β with $\frac{2\pi}{\lambda}$, and obtaining $e^{-\frac{2\pi}{\lambda}y}$. This expression indicates the reduction factor to be applied to the sinusoid is around 5, a distance equal to one quarter of the wavelength (i.e. at $y = \frac{\lambda}{4}$), around 23 at a distance of half wavelength ($y = \frac{\lambda}{2}$), and around 535 at $y = \lambda$.

In the case study we were considering, it is immediately obvious that to calculate that $4\beta^4 = 2.87\ \mathrm{m}^{-4}$ and $\beta = 0.92\ \mathrm{m}^{-1}$ (a Poisson ratio $\nu = 0.2$ has been adopted), the wavelength of the sinusoid is thus around 6 metres, and the flexural effect can certainly be ignored at a distance of about 3 metres from the base. While the values of bending moment (M) and shear force (Q) along the height can be calculated by applying the usual relations between displacement, curvature, bending moment and shear, i.e.:

$$M = -B\frac{d^2w}{dy^2} \tag{1.18a}$$

$$Q = -B\frac{d^3w}{dy^3} \tag{1.18b}$$

It appears that only the maximum values of bending moment (Figure 1.6) and shear force are relevant to design the required amount of vertical reinforcement at the base, a reinforcement that will be needed for a few metres only.

As already pointed out, the combination of the membrane and the flexural solutions aims to obtain total displacement and rotation at the base compatible with the restraint of each specific case. For a fixed-base

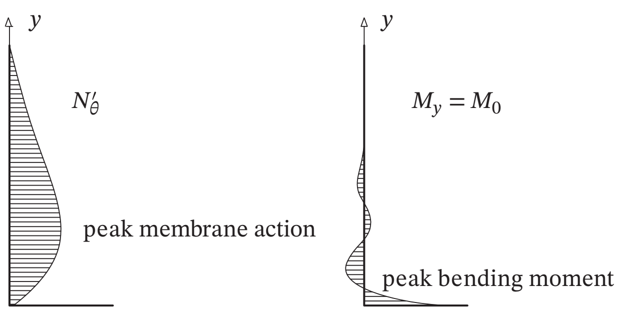


Figure 1.6 Qualitative variation of membrane action N'_θ (hoop force) and bending moment M_y or M_0 along the height of the container.

tank, the displacements and rotations generated by the flexural effect (M_0 and Q_0) must be equal and opposite to those generated by the membrane solution, i.e.:

$$-\frac{\gamma r^2}{Eh}H = \frac{1}{2\beta^3 B}(\beta M_0 + Q_0) \tag{1.19a}$$

$$\frac{\gamma r^2}{Eh} = -\frac{1}{2\beta^2 B}(2\beta M_0 + Q_0) \tag{1.19b}$$

In the case study, the base bending moment and shear, calculated by applying again a 1.3 protection factor, are $M_0 = 183$ kNm/m and $Q_0 = -345$ kN/m. These values are compatible with the assumed section depth (0.20 m), requiring a total amount of reinforcement of 3254 mm^2, or a geometrical steel percentage $\rho = 1.63\%$. At heights above 5 m from the base and on the compression side, the minimum allowed reinforcement percentage can be adopted (possibly 0.3%). The bending moment computed at the base of the cylindrical tank is about seventeen times lower than that obtained for the cubic tank, but unfortunately this is not the governing action to dimension the shell. Actually, when considering the circumferential membrane forces, it is immediately realized that a relevant crack pattern will be induced. For example, at a height of about 3 metres the membrane action is $N'_\theta = 1.3 \cdot 2300 = 3000$ kN/m (Figure 1.6), which would require about 7670 mm^2 of steel, corresponding to a very high geometrical percentage of 3.8%. The resulting theoretical crack opening width would be 0.4 mm with an average crack maximum distance of $\Delta_{max} = 315$ mm. This crack pattern would not be acceptable in most cases.

The most obvious correction of the preliminary design to redirect the result to performances similar to those obtained in the case of a rectangular tank with 900 mm thick walls will simply be to increase the shell thickness. In fact, increasing the shell depth from 200 to 400 mm will induce a modification of the coefficient $\beta = 0.65$ m^{-1}, thus increasing the flexural stiffness with respect to the membrane one. The previous calculations will be repeated, obtaining $M_0 = 358$ kNm/m, $Q_0 = -480$ kN/m and $N'_\theta = 1.3 \cdot 2000 = 2600$ kN/m. While bending moment and shear action will be easily absorbed by the increased resisting section, the theoretical mean crack width will now be 0.17 mm. The total required amounts of concrete and steel will still be around 30% lower than those estimated in the case of a rectangular tank, thus resulting in a competitive design.

A second, more sophisticated, design solution could be based on the insertion of post-tensioning cables to reduce the tensile membrane forces and displacement along the circumferential direction. A possible way of estimating an effective post-tensioning force could be to equilibrate the tensile circumferential resultant due to the internal fluid pressure $p = 1.3 \cdot 10 \cdot 25 \cdot 1 = 325$ kN/m acting on each ring of unitary height. In this case, a preliminary estimate of the required post-tensioning steel area (A_{sp}) can be obtained as follows:

$$A_{sp} = \frac{pr}{f'_{syd}} = \frac{325 \cdot 10}{1.1} = 2955 \text{ mm}^2$$

where f'_{syd} is the steel stress after prestress losses occurred, estimated around 65% of the steel cable yield strength. A reasonable corresponding concrete section is then obtained assuming that at yield conditions of the steel cables the concrete average stress is still acceptable, i.e., lower than the compression design strength:

$$A_c = \frac{A_s f_{syd}}{f_{cd}} = \frac{2955 \cdot 1452000}{17000} = 252352 \text{ mm}^2$$

For a strip of unitary height of the vertical wall, this corresponds to a thickness of about 25 cm. As a consequence, in this case the total amount of concrete required is 393 m^3, i.e. less than one-third of the amount estimated for the rectangular tank, with some 55 tons of steel reinforcement. These calculations are obviously preliminary estimates, since a more refined solution would require reasoning in terms of induced deformations rather than brutally equating forces.

An additional further alternative would be to modify the connection between the foundation and the cylinder, allowing relative displacement and rotation. In such conditions, the membrane boundary constraints would be respected and the flexural response would be eliminated. The consideration related to the circumferential stresses will still apply, with the consequent need of post-tensioning or adequate thickness. To allow rotation and displacement, a rubber pad should be designed and inserted, properly connecting it to both sides.

1.4 Seismic Behaviour of Tanks

A cylindrical tank is evidently a stiff structure, the simplest assumption to define its response to a ground motion is to consider it as a rigid body. As such, the tank will be possibly modelled as a mass, equal to a fraction of the total mass of structure and contents, rigidly connected to the ground. The mass relative displacement will be assumed as null, the maximum base shear (Q) equal to the mass multiplied by the peak ground acceleration (a_g) and the maximum overturning moment (M) equal to the base shear multiplied by some equivalent height.

In many cases, and particularly for empty tanks, this model will provide acceptable results, provided that reasonable values have been assumed for the fraction of the total mass and for the equivalent height at which the mass is lumped.

Considering first the case of an empty tank with a light roof, a possible choice is to consider the mass of the wall (m_w) only, to ignore the mass of the base plate (m_b) and to assume an equivalent height (H_{eq}), equal to one-half of the height of the tank:

$$Q = m_w a_g \tag{1.20a}$$

$$M = QH_{\mathrm{eq}} = m_w a_g H_{\mathrm{eq}} \tag{1.20b}$$

Considering once more the examples of the rectangular and cylindrical tanks previously analyzed (Section 1.4) and a peak ground acceleration $a_g = 0.35g$, with reference to Figure 1.7(a), the base shear (Q) and the total bending moment (M) at the base of the wall will be:

Cylindrical tank	Rectangular tank	
$Q = 2749$ kN	$Q = 10762$ kN	(1.21a)
$M = 34361$ kNm	$M = 107625$ kNm	(1.21b)

These values give an immediate impression of the significant difference between the action on the cylindrical and rectangular tank, but will be of little use, since no mention has been made of the action induced by the contents. In any case, even limiting our consideration to the empty case, these values apply to the walls and to their connection to the foundation. In order to design the foundation and to evaluate the soil response, including the possibility of rocking or sliding phenomena, the shear (Q') and moment (M')

generated by the foundation mass need to be added. In this case, an assumption of a rigid response is even more justified, thus Equations (1.20a) and (1.20b) can be applied, using the foundation mass (m_b) and an equivalent height H_{eq} equal to one half of the thickness of the foundation plate. This thickness is assumed to be equal to 0.40 m for the cylindrical tank and to 1.0 m for the rectangular one. The different thicknesses are justified by the similar depth of the walls to be connected to the plate and by the different values of shear and bending moment to be transmitted. The resulting actions will be:

Cylindrical tank	Rectangular tank	
$Q' = 3849$ kN	$Q' = 14262$ kN	(1.22a)
$M' = 34581$ kNm	$M' = 109375$ kNm	(1.22b)

As expected, considering the small equivalent height where the mass is lumped, the additional actions induced by the foundation have a significant effect on the shear values and a negligible one on the overturning moment.

In the case of the rectangular tank, the consequence of this simple model on the design of each wall would be to consider a plate, restrained on three or four sides (depending on the roof type and connection) and to apply to the plate a uniform load equal to 35% of its own weight.

In the case of the cylindrical tank, a uniformly distributed load p_s (Figure 1.7(b)) will be applied along the height of the cylinder, equal to its total mass per unit length, again multiplied by the peak ground acceleration $a_g = 0.35g$, i.e.:

$$p_s = a_g h \gamma_c \tag{1.23}$$

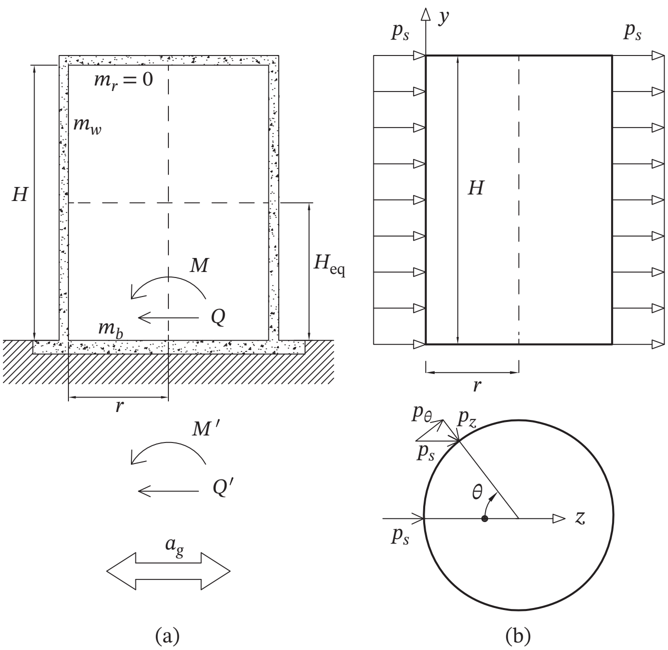


Figure 1.7 (a) Empty (cylindrical or rectangular) tank with the corresponding masses and heights and (b) an example of the seismic load p_s statically acting on the cylindrical tank's wall.

where h is the thickness of the tank and γ_c the unit weight of the structural material (concrete in our case).

To estimate the membrane actions, by analogy to what has been done in the case of the horizontal action induced by the pressure of the content, it is necessary to derive the tangent and radial components of p_s (the vertical component p_y is obviously null):

$$p_\theta = p_s \sin\theta \tag{1.24a}$$

$$p_z = p_s \cos\theta \tag{1.24b}$$

The solution of the problem is not as simple as in the case of an internal pressure, since the load is now not axisymmetric, and is discussed in detail in [39, 40]. From the integration of the equations which govern the membrane equilibrium, presented in [39, 40, 66], applying the boundary conditions at the unrestrained top of the tanks (i.e.: $N'_{y\theta} = N'_y = 0$ in $y = H$), results in:

$$N'_\theta = -p_s r \cos\theta \tag{1.25a}$$

$$N'_{y\theta} = 2p_s(H-y)\sin\theta \tag{1.25b}$$

$$N'_y = \frac{p_s}{r}(H-y)^2 \cos\theta \tag{1.25c}$$

In these equations, N'_θ and N'_y have the same meaning as in Equations (1.14a) and (1.14b), i.e. N'_θ is the horizontal reaction force per unit length tangent to the tank surface and N'_y is the vertical reaction force per unit length. $N'_{y\theta}$ is the in-plane shear force per unit length, which was null in the case of internal pressure because of the axisymmetric nature of the problem. The formulation of the equations is showing that:

- the membrane ring force (N'_θ) is constant over the height and varies along a ring from zero in the elements parallel to the acting force to a maximum compression on the side of the action to a maximum tension on the opposite side;
- the membrane vertical force (N'_y) varies quadratically along the height, from zero at the top to a maximum at the base, and varies along each ring similarly to N'_θ;
- the membrane shear ($N'_{y\theta}$) varies linearly along the height, from zero at the top to a maximum at the base, while along each ring it shows the opposite trend with respect to N'_θ and N'_y, being null in the elements perpendicular to the applied force and maximum in the parallel elements.

These trends are shown graphically in Figure 1.8.

The distributions of the axial and the shear force per unit length (N'_y and $N'_{y\theta}$) seem to indicate some similarities with the internal axial (σ) and shear (τ) stresses of a cantilever beam fixed at the base and subjected to the same horizontal applied load. In the case of the beam, the internal stresses at any height (y) will be expressed as functions of the bending moment (M_y) and shear (Q_y) acting at the corresponding section as:

$$\sigma = \frac{M_y}{J}x \qquad \text{and} \qquad \tau = \frac{Q_y S}{bJ} \tag{1.26}$$

where J is the moment of inertia of the section, x is the distance between the fibre considered within the section and the neutral axis, S is the first moment of the area above the considered fibre, b is the

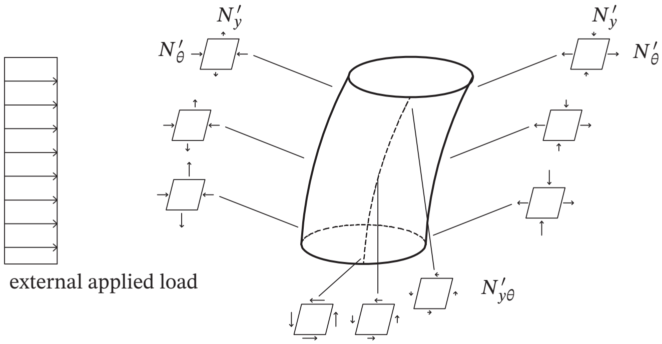


Figure 1.8 Qualitative distribution of N'_θ, N'_y and $N'_{y\theta}$.

width of the section at the fibre. Considering the case of the cylindrical tank, all the variables appearing in Equation (1.26) can be derived as follows (H is the height of the cylinder, h the shell thickness, r the radius of the generating ring):

$$Q_y = 4\int_y^H \int_0^{\pi/2} p_s\, r\, d\theta\, dy = 2p_s\pi r(H-y) \quad \text{Shear force at height } y \tag{1.27a}$$

$$M_y = Q\frac{H-y}{2} = p_s\pi r(H-y)^2 \quad \text{Bending moment at height } y \tag{1.27b}$$

$$S = 2\int_0^\theta hr^2\cos\theta\, d\theta = 2hr^2\sin\theta \quad \text{First moment of area of a ring sector subtended by an angle } 2\theta \tag{1.27c}$$

$$J = 4\int_0^{\pi/2} hr^3\cos^2\theta\, d\theta = \pi hr^3 \quad \text{Moment of inertia of a ring} \tag{1.27d}$$

$$b = 2h \qquad x = r\cos\theta \quad \text{Width and distance from the neutral axis} \tag{1.27e}$$

The internal stresses at any point of the cylindrical tank can be related to the membrane forces of Equations (1.25b) and (1.25c), assuming that these are constant over the thickness (h), in which case it results in:

$$N'_y = \sigma \cdot h \qquad \text{and} \qquad N'_{y\theta} = \tau \cdot h \tag{1.28}$$

Substituting Equations (1.27a)–(1.27e) into (1.26) and multiplying by h to obtain the membrane forces, the following equations are obtained:

$$N'_{y\theta} = \frac{2hp_s\pi r(H-y)2hr^2\sin\theta}{2h\pi hr^3} = 2p_s(H-y)\sin\theta \tag{1.29a}$$

$$N'_y = \frac{hp_s\pi r(H-y)^2 r\cos\theta}{\pi hr^3} = \frac{p_s}{r}(H-y)^2\cos\theta \tag{1.29b}$$

These results are identical to those expressed in Equations (1.25b) and (1.25c), indicating that the pure membrane theory applied to a cylindrical tank subjected to a uniform horizontal load and the beam theory applied to a vertical cantilever with the same section subjected to the same load will produce identical results. It is known that one of the fundamental assumptions for the application of the beam theory is that horizontal sections remain plane and undeformed. As a consequence, the application of this analogy is possible if the diameter is small compared to the height.

All the considerations and calculations developed up to this point have been related to empty tanks, and this appears rather limiting, and possibly useless, when it is observed that the mass of the contained liquid is normally several times larger than the mass of the containing structure. In the cylindrical tank considered as an example, the mass of the liquid is likely to be of the order of 7–8 times the mass of the concrete shell. On the other hand, it is immediately verified that considering the entire mass of the liquid as rigidly connected to the shell will result in unreasonably large actions.

Actually, a careful observation of the response of the liquid contained in a cylindrical tank to a horizontal acceleration (which could easily be simulated by moving a glass of water on a table), will lead one to note that the upper part of the liquid will respond with a kind of a vertical wave, as shown in Figure 1.9. This periodic variation of the height of the liquid at the opposite sides of the tank will induce a variation of the pressure on the walls and on the base plate. While the response of the shell was assumed to be characterized by a very short period of vibration (as a consequence of its high stiffness), this kind of motion, sometimes called "sloshing mode", is characterized by a long period of vibration, in the range of several seconds for normal size tanks.

In addition, it appears that only part of the liquid will participate in this mode of vibration, in the upper part of the tank, while the remaining lower part will essentially participate in the rigid response discussed and assumed for the shell structure. The problem is thus how to define the fraction of liquid participating in each mode and to estimate a reasonable period of vibration of the sloshing mode. The solution to this problem will easily allow an estimate of the induced actions, since the very significant difference of the periods of vibration of the two modes will facilitate a combination of their effects. The details of this problem are presented in Chapter 2, however, it is here anticipated that the fundamental parameter to determine the percentage of fluid participating in either mode is the ratio of the height of the fluid in the tank over its radius, $\gamma = \frac{H}{R}$. For the same ground motion intensity, a higher mass participating in the sloshing mode (i.e. for smaller values of γ) also implies a higher convective wave, and consequently higher values of the actions induced by the corresponding mode. This effect can be again intuitively verified by moving with similar input two different glasses of water over a table, one wide and low (like a highball), the other one taller and narrower (like a tumbler). The water will more easily overflow from the highball, because a greater fraction of the fluid will participate in the convective motion and the wave will be higher. The problem described above is graphically represented in Figure 1.10.

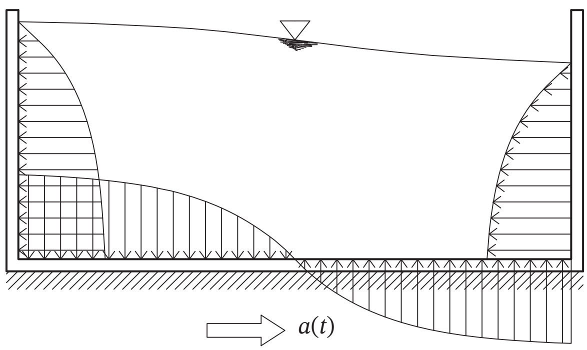


Figure 1.9 Qualitative trend of the convective wave and variation of the dynamic pressure on walls and base plate induced by a horizontal ground motion in a tank full of liquid. Source: Adapted from [412].

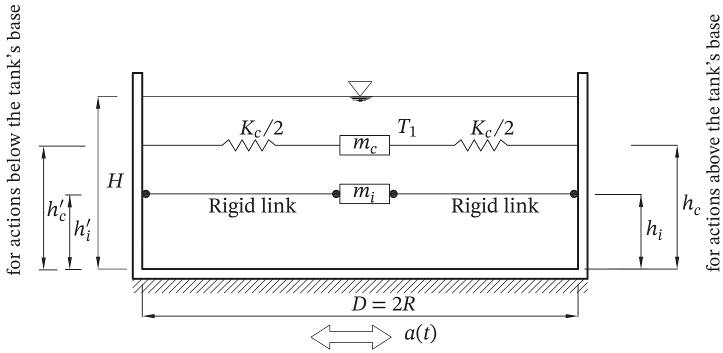


Figure 1.10 Equivalent mechanical model for a tank full of water under seismic excitation. Source: From [150]/American Society of Civil Engineers.

A lower mass is rigidly linked to the shell structure, while a higher mass is linked to the walls with elastic links, whose stiffness (together with the associated mass fraction) will define the period of vibration of the sloshing mode. In general, in tall and narrow tanks, characterized by a value of γ greater than 3, the fraction of liquid participating in the convective motion is less than 15%. As a consequence, in these cases, it is acceptable to compute the actions considering the whole liquid mass as rigidly linked to the walls. The solution will thus be identical to that discussed above, only the total mass located at mid-height of the tank and subjected to the ground acceleration will be larger. In the case of low and wide tanks, with values of γ smaller than 0.3, more than 80% of the mass of the liquid will likely participate in the convective motion.

As anticipated, the analytical solution to the problem of estimating the mass fractions participating in either mode and their height of application is provided in Chapter 2, however, the basic resulting equations are depicted in a graphical form in Figure 1.11. It is immediately noted that the mass participating in the two modes varies rapidly with γ, while the equivalent height of application of the two masses is much less sensitive to the geometry of the tank.

In Chapter 2 how the convective component of the response motion is actually composed of several modes, with distinct participating mass and different periods of vibration will be discussed, however, it will be clear that most of the mass will participate in the first mode, and on this we will now focus attention. The period of vibration of the first sloshing mode can be estimated using the following expression:

$$T_1 = \frac{2\pi}{\omega_{c1}} = \frac{2\pi\sqrt{\frac{R}{g}}}{\sqrt{\lambda_1 \tanh\left(\frac{\lambda_1 H}{R}\right)}} \tag{1.30}$$

where, as usual, r is the radius and H the height of the fluid in the tank, g is the constant of gravity and λ_1 is a numerical parameter equal to 1.8412. The resulting period of vibration T_1 is thus varying between 2.3 and 6.7 seconds for any tank with height and diameter between 5 and 25 m. In the example case considered above, with a diameter equal to 20 m and a height equal to 25 m, the period of vibration of the sloshing mode will be estimated as $T_1 = 4.7$ sec.

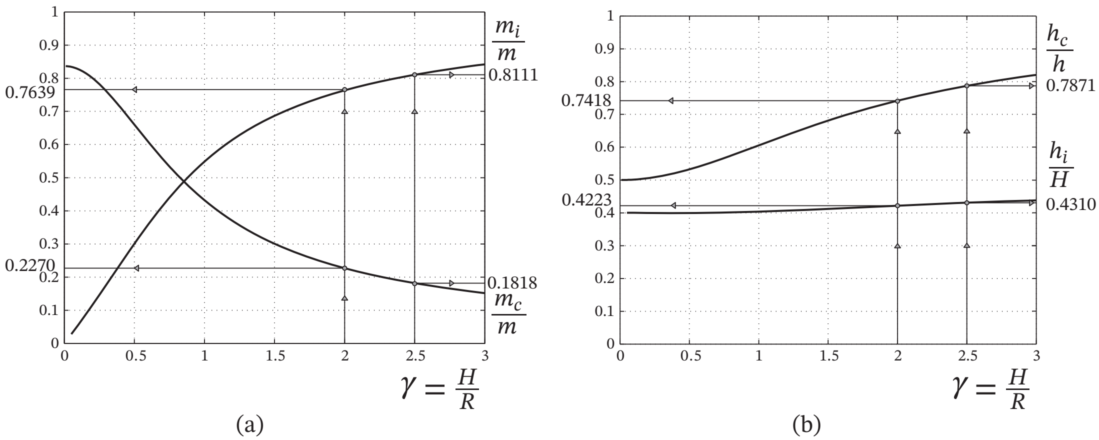


Figure 1.11 New Zealand Recommendation – NZSEE-09: (a) percentages of total mass for impulsive (m_i) and convective (m_c) components; (b) equivalent height of application of the impulsive (h_i) and convective (h_c) mass.

It is thus confirmed that the convective period of vibration is quite high, and certainly much greater than the period of the tank. The two motions will therefore be completely uncoupled and the actions resulting from each one can be computed separately and simply added together. The theory derived to treat the sloshing modes of cylindrical tanks can be adapted to rectangular tanks by introducing minor modifications (see Chapter 2), to estimate the following masses and equivalent heights of application for the two example used in this introduction, as follows:

Cylindrical tank $H = 25$ m, $r = 10$ m	Rectangular tank $H = 2B = 20$ m	
$m_i = 6361$ ton	$m_i = 6104$ ton	(1.31a)
$m_c = 1429$ ton	$m_c = 1816$ ton	(1.31b)
$h_i = 10.8$ m	$h_i = 8.4$ m	(1.31c)
$h_c = 19.7$ m	$h_c = 14.8$ m	(1.31d)

These values can be approximately derived from the plots in Figure 1.11. In both cases the impulsive mass is about 80% of the total liquid mass. For the cylindrical tank, this corresponds to almost eight times the mass of the shell, while for the cubic tank the liquid mass is only approximately two times the mass of the walls.

It is now possible to estimate the approximate value of the seismic actions, recurring to an acceleration spectrum, which will be here assumed to have the typical shape adopted for stiff soil properties, as depicted in Figure 1.12. To enter the spectrum and thus define the multipliers to be applied to the effective masses of each mode, it is necessary to estimate the period of vibration of each mode, as follows:

- as already discussed, the impulsive mode of the cylindrical tank will be assumed to be infinitely rigid, the corresponding period of vibration will be thus assumed to be zero ($T_{\mathrm{cyl},i} = 0$) and the corresponding spectral acceleration will be identical to the peak ground acceleration ($S_e(T_{\mathrm{cyl},i}) = a_g = 0.35g$);

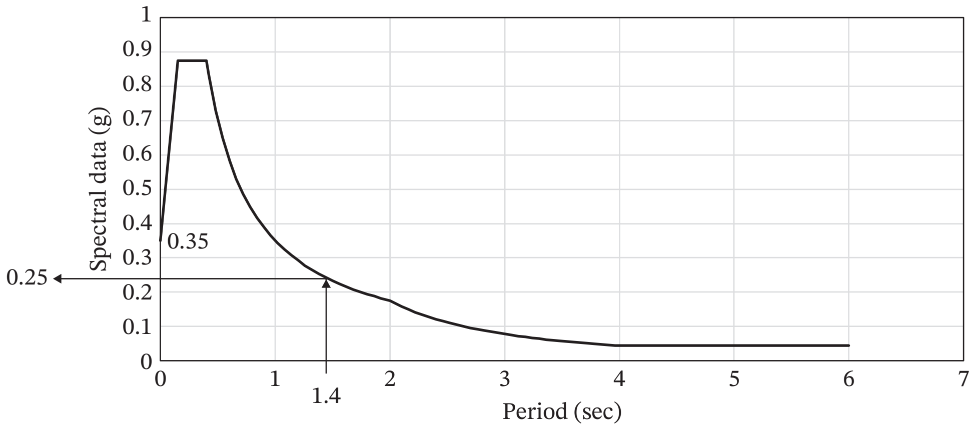


Figure 1.12 Acceleration spectrum having the typical shape adopted for stiff soil properties: soil A, spectrum Type 1, $S = 1.0$, $T_B = 0.15$ sec, $T_C = 0.4$ sec and $T_D = 2.0$ sec, $a_g = 0.35g$. Source: Point 3.2.2.2 in UNI EN 1998-1:2004.

- the impulsive action on one of the walls of the rectangular tanks should not be assumed to be acting at zero period, since it should account for the flexibility of a 20 by 20 m plate, 0.9 m thick, restrained along three sides. The fundamental period of vibration of such a plate can be estimated as around 1.4 sec and the corresponding spectral acceleration approximately equal to: $S_e(T_{\text{rec},i}) = 0.25g$ (see Figure 1.12);
- the convective (sloshing) motion in the cylindrical and rectangular tanks will be characterized by slightly different periods of vibration, equal to $T_{\text{cyl},c} = 4.7$ sec and $T_{\text{rec},c} = 5.1$ sec and by a conventionally lower damping ratio, taken as 0.5%. The corresponding spectral acceleration, including an amplification factor $\eta = 1.34$, to account for the lower damping, will be low and similar: $S_e(T_{\text{cyl},c}) = 0.04g$ and $S_e(T_{\text{rec},c}) = 0.03\text{g}$.

The shear force and bending moment induced at the base of the tanks by the liquid response can be calculated by applying the spectral acceleration to the corresponding equivalent mass, obtaining the results reported below. It is immediately observed that the actions induced by the sloshing modes are negligible with respect to those generated by the impulsive modes. Also noticeable is the very favourable effect resulting from the flexibility of the walls of the rectangular tank:

Cylindrical tank	Rectangular tank	
$Q_i = 21818$ kN	$Q_i = 15260$ kN	(1.32a)
$Q_c = 572$ kN	$Q_c = 545$ kN	(1.32b)
$M_i = 235637$ kNm	$M_i = 128184$ kNm	(1.32c)
$M_c = 11261$ kNm	$M_c = 8063$ kNm	(1.32d)

The total impulsive actions at the base of the tanks can be calculated considering the contributions of the structure (Equation (1.21)) simply added to those of the impulsive liquid response:

Cylindrical tank	Rectangular tank	
$Q_b = 24567$ kN	$Q_b = 26022$ kN	(1.33a)
$M_b = 269998$ kNm	$M_b = 235809$ kNm	(1.33b)

Note that to obtain the maximum values of the action on the foundation plate (and on the soil, obviously), the contribution of the bending moment due to the height variation of the sloshing mode has to be combined with the total impulsive term (same for the shear contribution).

It is thus of interest to evaluate the height of the convective wave, also to estimate the possibility of liquid spilling or the potential pressure on the roof. To gain a feeling of the wave height, the following approximate expression can be used, applicable to the case of a cylindrical tank:

$$d_{\max} = 0.84 r S_e(T_1) \tag{1.34}$$

where, as usual, r is the radius of the tank, $S_e(T_1)$ is the ordinate of the appropriate acceleration response spectrum, evaluated for the period of vibration of the first convective mode, expressed as a fraction of g. In the case considered above, the wave height would be $d_{\max} = 0.34$ m, while applying a similar equation to the case of the rectangular tank the wave height would result in $d_{\max} = 0.31$ m. This wave will induce an additional bending moment on the foundation and possibly pressure on the wet area of the roof, with a consequent increment of the impulsive mass due to the roof weight acting on the liquid. These problems are discussed in Section 2.2.8.

1.5 Field Observation of Damage to Tanks Induced by Seismic Events

The damage to and possible failure of a storage tank during an earthquake may induce consequences, and namely losses, far exceeding those connected to the repair or re-construction of the structure and the proper value of its contents [67]. A tank could be the primary source of local public water and its loss could result in disruption of water use both for domestic consumption and for post-earthquake recovery. Fire services may be impaired when they will be most needed. Alternatively, a tank could contain products that are toxic and dangerous for the environment [573], possibly contaminating the atmosphere and the groundwater layer or resulting in conflagration determining an immediate flash of vapor and a toxic gas cloud (e.g. Figure 1.13).

Failure modes induced by ground motion can be quite diverse (e.g. Figure 1.14) and are usually grouped into several categories [328, 419, 469, 508]:

Figure 1.13 Burned tanks following the Izmit earthquake in the Kocaeli Province of Turkey (17 August 1999, Magnitude 7.4). Source: From [573]/Taylor & Francis.

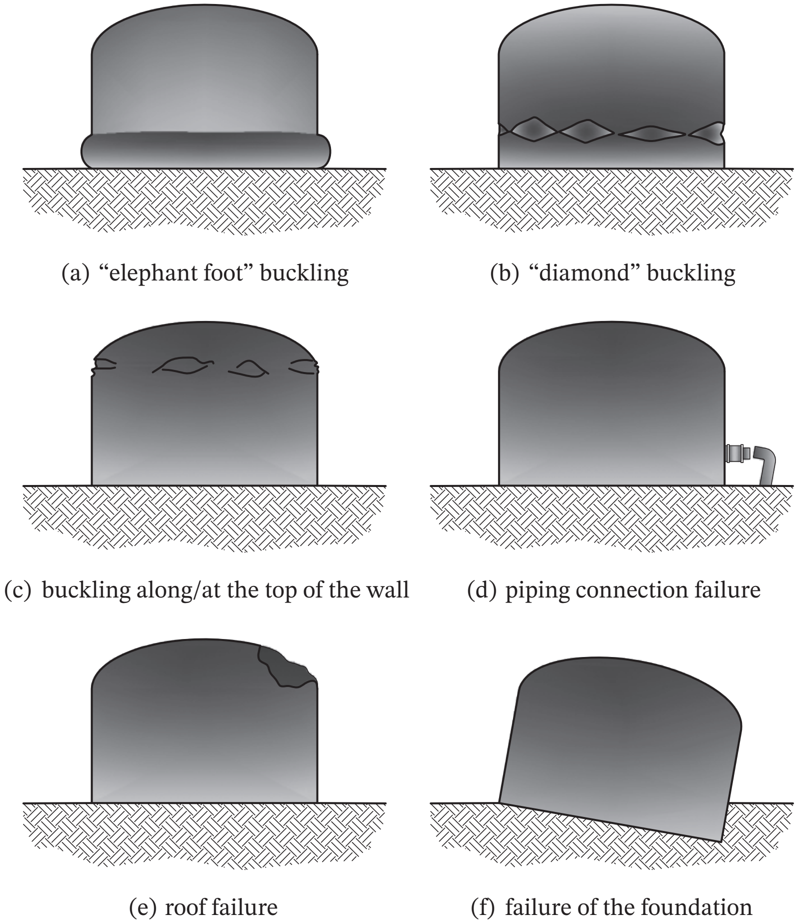


Figure 1.14 Global overview of observed damage in tanks in past earthquakes.

- the most common collapse mechanism, observed in steel tanks, is a form of elastopastic buckling [481, 488, 489, 491, 493] commonly known as “elephant’s foot” buckling (Figure 1.14(a)). The characteristic outward bulge above the base of the tank (Figure 1.15(a)), extending partly or completely around the circumference, results from the combination of the large circumferential tensile stresses induced by the internal hydrostatic and hydrodynamic pressures and by the axial compression due to the overturning moment caused by the horizontal seismic loads. At the attainment of the tensile yield strength in the annular ring strip, the applied compression vertical stress will easily induce buckling, even for small stress increments, due to the reduced elastic modulus. This kind of elastoplastic instability tends to assume an axisymmetrical shape due to the cyclic nature of the action and is generally associated with large tanks, with a relatively low height-to-radius ratio (say $\frac{H}{r} \leq 1$) [328].
- a second common failure mode, again related to instability problems, is known as “diamond-shaped” buckling. It is induced by an elastic form of instability, again in the proximity of the base, where the stresses are higher (Figure 1.14(b)). This damage mode (Figure 1.15(b)) takes place at relatively low

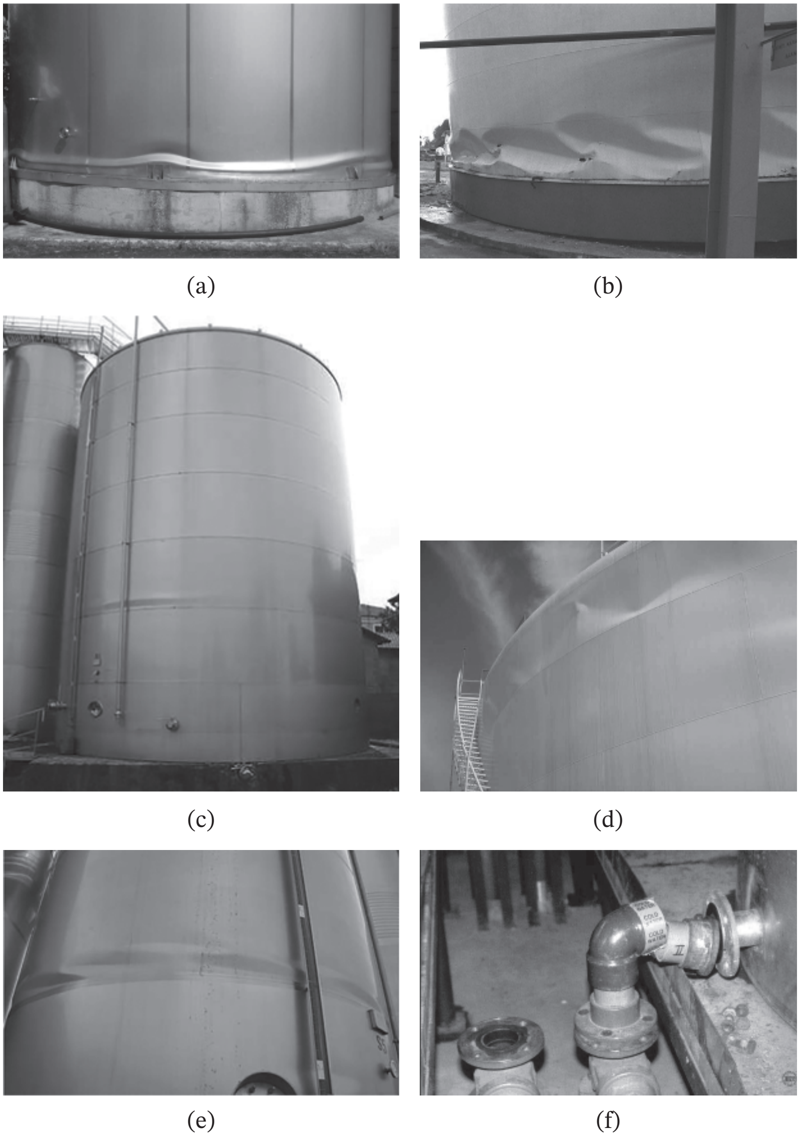


Figure 1.15 Observed damage to tanks in past earthquakes: (a), (c), (e) Emilia earthquake, Italy (20 and 29 May 2012, Magnitude 6.1 and 5.9 respectively); (b) Izmit earthquake, Turkey (17 August 1999, Magnitude 7.4) (from Izmit Collection, University of California, Berkeley); (d) San Fernando earthquake, California (9 February 1971, Magnitudo 6.6) (from Karl V. Steinbrugge Collection, University of California, Berkeley); (f) Edgecumbe earthquake, New Zealand (2 March 1987, Magnitude 6.3) (from Edgecumbe Collection, University of California, Berkeley).

values of tensile ring stresses and it is mainly caused by high vertical compression. It typically takes place in more slender tanks, with a height-to-radius ratio greater than one [328]. Elastic buckling considerations may govern the design of cylindrical shells with a relatively low internal pressure [481].

- local bucking has also been observed in the upper areas of the shell (Figures 1.15(c) and 1.15(e)), even close to the top (Figure 1.14(c)) [419, 420]. This latter failure (Figure 1.15(d)) is believed to have been caused by some rupture close to the base followed by a rapid loss of the tank contents and consequently resulting in a partial vacuum in the upper part, leading to a pressure decrease or even to a change of sign of the ring stress. Note that near the top the tank shell can be rather thin, since the theoretical acting forces and stress are low [469].
- an uncommon damage mechanism has been observed in the Emilia earthquake sequence (Italy, 2012, main shocks magnitudes 6.1 and 5.9). A diamond-shape elastic buckling was observed in the proximity of the circumferential welding, at an intermediate height (Figure 1.15(e)). The location of buckling is certainly related to the presence of imperfections induced by the welding and by the variation of shell thickness above and below the welding line.
- relative displacements between tank and connected piping and equipment have induced damage and failure to connections (Figures 1.15(f) and 1.14(d)). This cannot be regarded as proper tank damage, but it is undoubtedly inducing an operational break and the consequent economic loss. This kind of problem can easily be solved by providing some adequate deformation capacity at all connections.
- tanks are usually designed to allow sloshing waves to move within the freeboard between the still water surface and the roof, even if this solution has undesirable side effects on the system capacity and corrosion prevention and, ultimately, in the case of potable water, on water quality. As shown in Figure 1.16(a), an insufficient freeboard may cause an upward pressure on the tank roof that will influence the vertical force in the shell and increase the impulsive mass, modifying the impulsive and convective periods of vibration and the hydrodynamic pressure against the shell. Sloshing waves impacting on the roof (Figure 1.13(a)) have sometimes resulted in roof failure or in damage to the shell-roof joint or seam (Figure 1.14(e)). Similar problems have resulted from internal pressure induced by increased temperature related to burning of nearby tanks (Figure 1.16(b), [573]).
- in tanks containing inflammable material, an explosion can be caused by bouncing of the floating roof against the shell, producing sparks that may ignite the inflammable material [573].
- tanks are often constructed in areas close to the sea, with possibly loose sand soil and high ground water layer. These conditions may be prone to liquefaction (Figure 1.16(c)) and to consequent global or or partial settlement of the foundation (Figure 1.14(f)). Sliding (Figure 1.16(d)) or walk-off of foundations due to progressive rocking has also been observed.
- some examples of damage to the connection between the tank and the foundation are shown in Figures 1.17(a)–1.17(c). In some cases, this damage is associated with some shell damage, as shown Figure 1.17(a), where some diamond-shaped buckling of the tank wall is clearly visible. Often, excessive tensile demand causes the fracture of anchor bolts or their extraction from the concrete foundation. Spalling of concrete, induced by insufficient distance between the anchor bolt and the edge of the foundation and low resistance of the concrete, is shown in Figure 1.17(b). A flexural failure that occurred in the anchor plates is depicted in Figure 1.17(c). All these damage modes are clearly not pertaining to the shell structure, but are related to the actions induced by the tank response and to poor dimensioning and steel reinforcement, anchor bolts and plates, and foundation concrete.
- leg-supported tanks, usually smaller than flat-bottomed systems, have often evidenced problems in the legs (Figure 1.17(d), with little consequence on the functionality of the tank) or in their connections

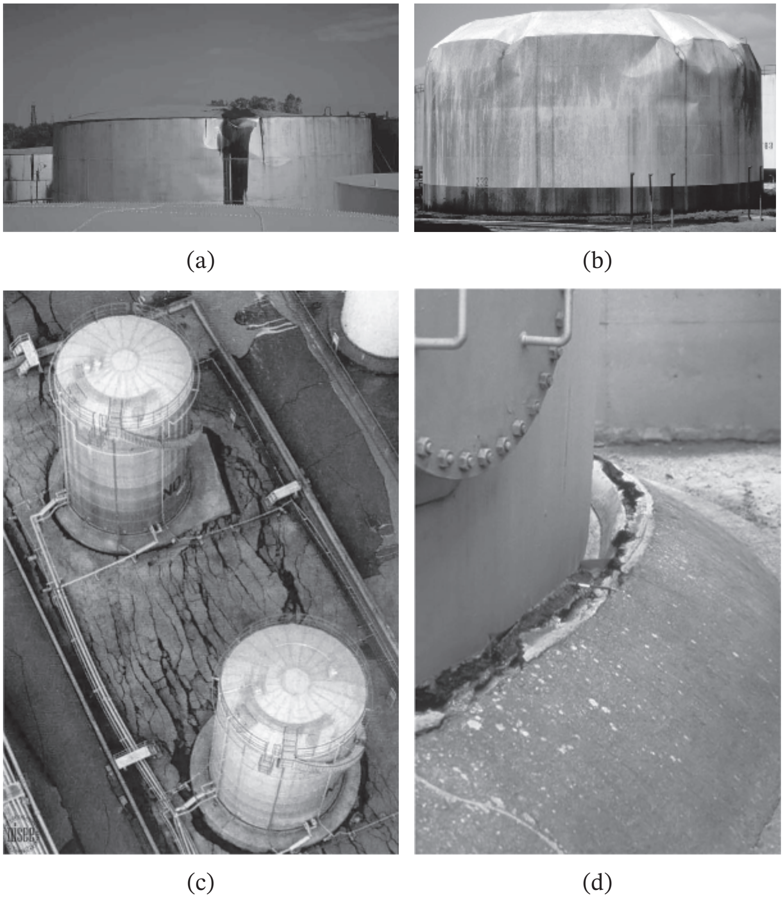
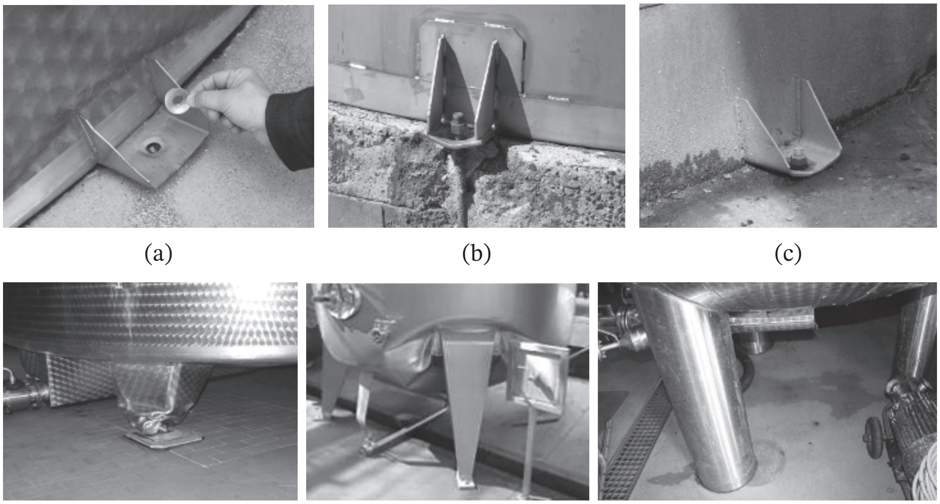


Figure 1.16 Observed damage to tanks in past earthquakes: (a) Kern County, California earthquake (21 July 1952, Magnitude 7.7) (from Karl V. Steinbrugge Collection, University of California, Berkeley); (b) Tupras refinery, Izmit earthquake, Turkey (17 August 1999, Magnitude 7.4) (Izmit Collection: IZT-174, Courtesy of the National Information Service for Earthquake Engineering, EERC, University of California, Berkeley; photo by Andrew S. Whittaker); (c) and (d) Kobe earthquake, Japan (17 January 1995, Magnitude 6.7) (Peter W. Clark Kobe Collection, University of California, Berkeley).

(Figure 1.17(e), where a relatively stiff leg is directly welded to a thin shell, leading to an undesirable "strong leg-weak tank wall" mechanism [187]). In some cases they have also been stepping away (see in Figure 1.17(f) the footprint of the former position of the leg). It is noticeable how unanchored legged systems often performed considerably better (like the case in Figure 1.17(f)), with no damage to the tank or the foundation. In these cases, the possibility of a combination of rocking and sliding has limited the transmitted shear force and bending moment and the piping system was flexible enough to accommodate the relative displacement demand. This effect can be regarded as a sort of not-designed base-isolation response.

- in elevated tanks, damage and failure are often concentrated in the supporting steel reinforced concrete structure, as shown in Figures 1.18(a)–1.18(g).



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

Figure 1.17 Observed damage to tanks in past earthquakes: (a)–(d) and (f) Emilia earthquake, Italy (20 and 29 May 2012, Magnitude 6.1 and 5.9 respectively); (e) Maule Earthquake, Chile (2 February 2010, Magnitude 8.8). Source: From [187]/Elsevier.

Certainly, it is important to highlight that codes/standards and recommendations (and the design therein suggested as highlighted in Section 1.6) cannot prevent extreme events from occurring, but they can provide good practice in order to offer a measure of protection against the earthquake. The following main options can be followed to avoid or reduce the damage to an acceptable level [16, 581]:

- a fixed geometry of the tank (radius R, height H, slenderness) may be maintained by increasing the wall thickness and by appropriately dimensioning the anchors at the base; it is possible also to add dissipative steel anchors at the base as suggested by Malhotra in [369]; dampers mainly control the moment transmitted to the foundation and, in the case of unanchored tanks, they consistently reduce the hoop compressive stress in the wall;
- depending on the boundary conditions, it may be possible to upgrade the geometry of the tank by changing its slenderness $\frac{H}{R}$ or by replacing a lower shell course, causing the tank to meet the seismic requirements. The annular plate could also be added or thickened, which can increase the seismic resistance or the tank's wall stiffened be using vertical strings;
- in order to reduce the hydrodynamic loading on the shell, tanks in seismic regions can be mounted on multilayered elastomeric bearing isolators as suggested by many authors [73, 74, 93, 304, 344, 366, 367, 378, 545]. When fluid containers are located at the top of a supporting structure or in the upper stories of a building, base isolation is still very favourable [531, 544]. The elevated tank can be isolated by two techniques, namely, by placing the bearings between the base of supporting structure and the foundation [531], and by placing the bearings between the bottom of the liquid container and the top of the supporting structure [544]. As an alternative to elastomeric isolators, curved surface sliding bearings, which are usually called a friction pendulum system (FPS) are commonly used for base isolation of liquid storage tanks since the period of the isolation system is independent of the storage level [1, 65, 93, 344, 436, 540, 660]. It is important to highlight that sloshing in tanks commonly

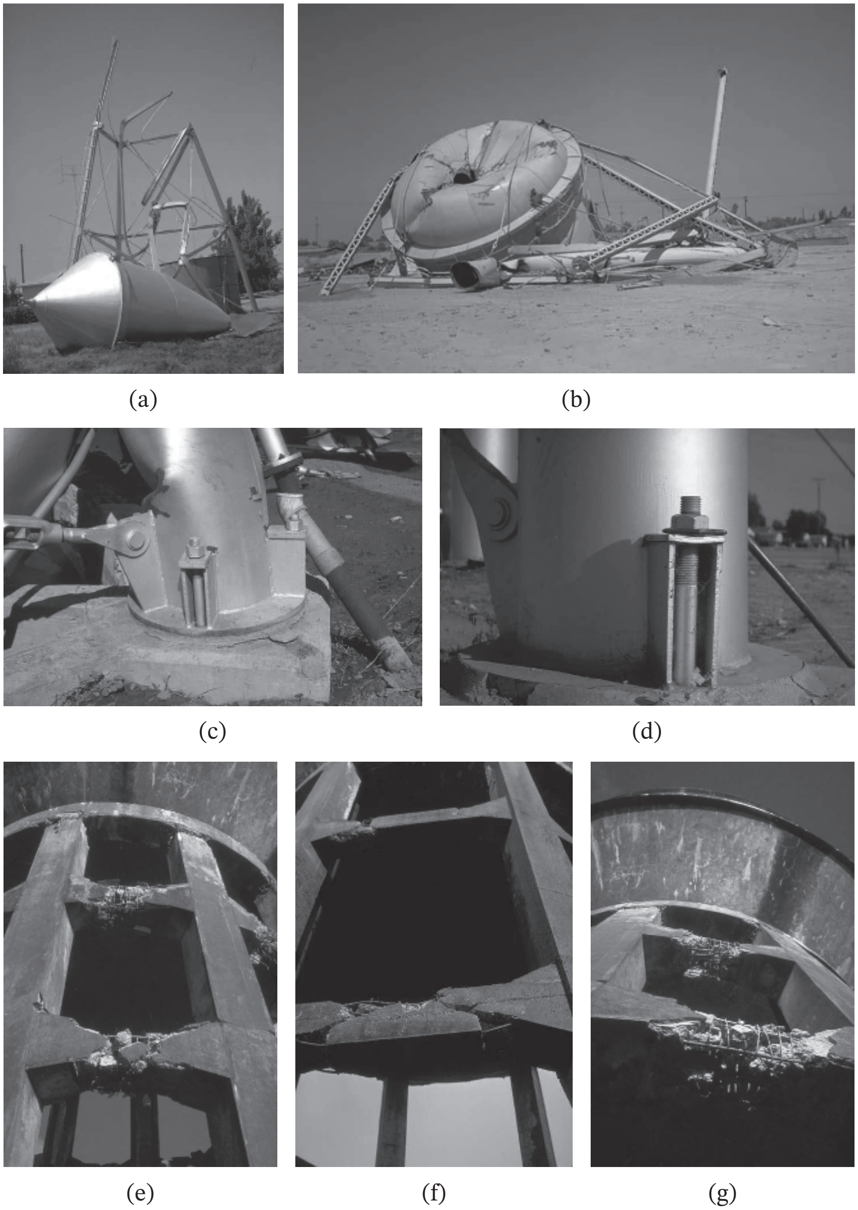


Figure 1.18 (a), (b) and (d) Kern County, California earthquake (21 July 1952, Magnitude 7.7); (c) Imperial Valley earthquake (21 October 1979, Magnitude 7.0); (e)–(g) Chile earthquake (22 May 1960, Magnitude 8.5). ((a)–(f) from Karl V. Steinbrugge Collection, University of California, Berkeley.)

used in engineering practice is a phenomenon of relatively low frequency and for this reason base isolation might increase it, causing a slightly higher convective pressure component and water elevation response. However, this problem can easily be solved by appropriate selection of the base isolation system or by adding dampers at the location of the maximum water particle velocities, as suggested in Section 6.3. So, generally, base isolation has more effect on the impulsive component, thus leading to the reduction of the impulsive pressure component of the hydrodynamic motion.

One more suggestion should be added to the above options, that the tank should be situated within a fluid tight bund [414] and, in order that it can be inspected externally for corrosion or leaks, a minimum distance between the tank's wall and the bund wall is recommended where possible [387].

1.6 Design Considerations

Storage containments are facilities used as a source of supply for essential lifelines such as potable water, fuel, and sewage disposal. Hence, the seismic performance of storage tanks is a matter of special importance in earthquake-prone countries beyond the economic value of the tanks themselves. As clearly highlighted in past earthquakes (Sections 1.5 and 5.5), severe damage or collapse, causing uncontrolled fires, spillage of toxic chemicals or liquefied gasses or loss of drinking water supplies, may cause substantially more damage than the effects on structures caused by the earthquake itself [431, 459]. In order to reach a level of structural safety of containments, according to Point 2.1.6(2), Eurocode 8, Part 4, the tank should be conceptually designed in such a way that it is able to sustain all the actions calculated using structural elements detailed for local ductility and constructed from ductile materials. Once the hydrodynamic pressures, exerted against the tank wall and bottom, are known (Chapters 2 and 3 for above-ground anchored and unanchored rigid tanks respectively; Chapter 4 for elevated tanks and Chapter 5 for flexible tanks; Section 6.5 for underground rigid tanks), they can be integrated so the tanks' designer can find the critical variables [414]:

- shear and bending moments, hoop tension, axial compression and tension, in the tank's wall, upper shell courses included;
- sloshing wave effects and eventual impact on the roof;
- shear and bending moments (overturning moments) transmitted to the foundation and subsequently to the soil.

The design according to the above quantities should also take into consideration, in order to mitigate earthquake effects, four main requirements (Point 2.1.6(2), Eurocode 8, Part 4): (1) redundancy of the system; (2) absence of interaction between mechanical/electrical and structural components; (3) easy access for inspection, maintenance and repair; and (4) quality control of all the components. A further consideration in the design of tanks is to establish the appropriate codes and standards to be used (mainly dependent on the tank supplier/designer in agreement with the buyer of the tank itself and the country of installation) and the material which is more effective and acceptable: steel or concrete (usually the supplier should take into account factors such as allowable stresses, welded or bolted connection cost compared to ordinary reinforced concrete or precast, availability as well as purchasing terms and mill delivery) [414]. Depending on the nature and amount of the contents and associated potential danger (as well as the combination of the actions) (Point 2.1.1(1), Eurocode 8, Part 4), the functional requirements during and after an earthquake, the environmental conditions, the materials used to build the tank and its corresponding typology, two main limit states are defined:

- the ultimate limit state (ULS) corresponds to structural failure or in the case of complete collapse would entail severe consequences, the ultimate limit state is defined as the one corresponding to a state prior to structural collapse. The design seismic action, a_{Ed}, for which the ultimate limit state may not be exceeded shall be expressed in terms of the reference seismic action, a_{Ek}, associated with a reference probability of exceedance,[2] P_{NCR}, in 50 years or a reference return period, T_{NCR} (Points 2.1(1) and 3.2.1, Eurocode 8, Part 1). Furthermore, the design seismic action, a_{Ed}, shall be expressed in terms of the importance factor γ_I which takes into account the reliability differentiation (Points 2.1(2), (3) and (4), Eurocode 8, Part 1):

$$a_{Ed} = \gamma_I a_{Ek} \tag{1.35}$$

from which can also be derived the following relation:

$$a_g = \gamma_I a_{gR} \tag{1.36}$$

where the value of the reference peak ground acceleration on type A soil,[3] a_{gR} (Point 3.2.1(2), Eurocode 8, Part 1), corresponds to the reference return period T_{NCR} of the seismic action for the no-collapse requirements; and a_g is the design ground acceleration on type A ground. For tanks it is appropriate to consider four main different importance classes, depending on the potential loss of life due to the failure of the containment structure and on the economic and social consequences of failure (Table 1.1 and Points 2.1.4(4)–(8), Eurocode 8, Part 4):

1) Class I refers to situations where the risk to life is low and the economic and social consequences of failure are small or negligible.
2) Class II refers to situations with medium risk to life and to local economic and social life.

Table 1.1 Importance Classes I to IV depending on the use and contents of the facility and the implications for public safety. Here are suggested values of γ_I, but it may be different for the various seismic zones of the country considered, depending on the seismic hazard conditions and on the public safety considerations.

Class	γ_I	Risk to life	Economic/social consequences
I	0.8	Low	Small/Negligible
II	1.0	Medium	Medium
III	1.2	High	Great
IV	1.6	Exceptionl	Extreme

2 The recommended values are $P_{\mathrm{NCR}} = 10\%$ and $T_{\mathrm{NCR}} = 475$ years.
3 Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface, with a value of the average shear wave velocity $v_{s,30} > 800$ m/s (Point 3.1 and Table 3.1 in Eurocode 8, Part 1).

3) Class III refers to situations with a high risk to life and great economic and social consequences of failure.
4) Class IV refers to situations with exceptional risk to life and extreme economic and social consequences of failure.

- the damage limit state (DLS) (or serviceability limit state (SLS) as called by Greiner in [193]) corresponds to requirements such as integrity and minimum operating level (Point 2.1.3, Eurocode 8, Part 4). The integrity requirements of the tank are satisfied when the system itself, including a specified set of accessory elements integrated with it (floating roofs, discharge devices, attached pipings, etc.), remains fully serviceable and leakproof under the relevant seismic action. Furthermore, the minimum operating level requirements of the containment are satisfied when the extent and amount of damage of the system itself, including some of its main components, are limited, so that, after the operations for damage checking and control have been carried out, the capacity of the tank can be restored up to a predefined level of operation. The relevant seismic action for which the damage limit state may not be exceeded shall have a probability of exceedance,[4] P_{DLR}, in 10 years and a return period, T_{DLR} (Point 2.1.3(5), Eurocode 8, Part 4).

It should be noted that the above preliminary considerations have to be specified and further explained in case of steel and concrete tanks.

In accordance with Point 2.1, Eurocode 3, Part 1.6, a steel tank should be designed in such a way that it will sustain all actions and satisfy the following requirements:

- overall equilibrium such as sliding, uplifting, and overturning. These three verifications may be found at Point 13.5.4, AWWA D100 (2011) and at Point 14.3.4, AWWA D103 (2009) in case of welded and factory-coated bolted steel tanks respectively. Hydrodynamic actions of the tank's liquid contents (subdivided into a portion of the liquid that acts as a solid in the bottom part and a portion of the fluid near the surface creating waves) produce forces and moments that want to tip the tank over, called an overturning behaviour. The response of the tank to this overturning action is to develop a pressure acting on the wall and on the bottom slab. With a sufficient acceleration due to the earthquake motion, a portion of the tank starts to uplift from the foundation or to slide. Usually, as well documented in [414], overturning, uplift, or sliding is not damaging the tank so much but the attached piping or equipment.
- equilibrium between actions and internal forces and moments, that should be checked with reference to the plastic (LS1) and buckling (LS3) limit state. As clearly defined by Rotter in [490] the plastic limit state (LS1) represents the state of extensive rupture or unacceptably large plastic deformation in tension or compression, when stability phenomena do not intervene. The simplest way to evaluate this limit state is to derive the plastic collapse load obtained from a mechanism based on small displacement theory (Point 4.1.1, Eurocode 3, Part 1.6). It is important to emphasize that the plastic limit state covers two main conditions in the membrane and bending shell design: (1) tensile rupture or compressive yield through the thickness (membrane); and (2) plastic collapse mechanism (involving bending). The limit state of buckling should be taken as the condition in which all or part of the structure suddenly develops large displacements normal to the shell surface, caused by loss of stability under compressive

4 The recommended values are $P_{\mathrm{DLR}} = 10\%$ and $T_{\mathrm{DLR}} = 95$ years.

membrane or shear membrane stresses in the shell wall, leading to inability to sustain any increase in the stress resultants, possibly causing total collapse of the tank (Point 4.1.3(1), Eurocode 3, Part 1.6). The tank's behaviour exhibited in the wall of a containment structure before, during, and after buckling appears depends mainly on the geometry, imperfections, boundary conditions, quality of construction, and loading pattern.

- limitation of cracks due to cyclic plastification, or better called the cyclic plasticity (LS2) limit state. The cyclic plasticity limit state, following the definition given by Rotter in [485], is concerned with a few cycles of loading that induce large strains (reversing in sign) and then producing plastic damage in both directions. Furthermore, according to Point 4.1.2(1), Eurocode 3, Part 1.6, this limit state should be taken as the condition in which repeated cycles of loading and unloading produce yielding in tension and in compression at the same point, thus causing plastic work to be repeatedly done on the tank's wall, eventually leading to local cracking by exhaustion of the energy absorption capacity of the material.
- limitation of cracks due to fatigue (LS4 or fatigue limit state). The limit state of fatigue should be taken as the condition in which repeated cycles of increasing and decreasing stress lead to the development of a fatigue crack (Point 4.1.4(1), Eurocode 3, Part 1.6).

Furthermore, API 620, API 650, API 12B, and UNI EN 14015 cover, in more detail, the design requirements for steel storage tanks in various standard sizes and capacities for different internal pressures, shop/field erected, product stored, and operating temperatures. Particularly, in order to assure structural stability and integrity, they establish minimum requirements for the design of tank sidewalls, roofs (fixed or floating) and bottoms, bolted or welded joints, foundation anchorage, openings (manhole or nozzle), and stiffening rings.

The two limit states for concrete tanks are the ultimate limit state (ULS) and the serviceability (or damage) limit states (SLS) as highlighted at Points 3.5.2.2(b) and 3.5.1 in EC8-4, respectively. Reinforced concrete or precast containments, including water and waste water treatment structures, belong to the category of reinforced concrete systems where minimal cracking is a prime requisite [307]. This means that the serviceability limit states consistently govern the design and usually leakage (Point 7, Eurocode 2, Part 3, where four tightness classes are introduced to classify liquid-retaining structures) and durability (Point 4 in Eurocode 2, Part 3) and govern deflection limitations [471]. In the seismic design situation relevant to the damage limitation state, tanks should be checked to satisfy the relevant serviceability limit state verifications required by Eurocode 2, Part 3, which covers additional rules to those in Part 1. According to Point 1.4.9, ACI 318-19 does not govern design and construction of concrete tanks and reservoirs, but indicates ACI 350-06 "Code Requirements for Environmental Engineering Concrete Structures" as the reference to be used in the design (and construction) of reinforced concrete reservoirs and other structures commonly used in water and waste treatment works, where dense, impermeable concrete with high resistance to chemical attack is required. In the case of concrete pedestal, elevated water storage tanks, ACI 371.R-08 describes how to apply ACI 318-19 and ACI 350-06 in designing concrete elements, including foundations, geotechnical requirements, appurtenances, and accessories. Over the years also the Portland Cement Association (PCA) has published many reports [128, 411, 412] on the design of rectangular and cylindrical reinforced concrete tanks, providing tables that assist practitioners in structural analysis of plates and shells. The prevention of the formation of through cracks due to tensile stresses can be done by following the prestressing philosophy [580]. The existing design approaches can be found in:

- ACI 372R-13 and AWWA D110-04 where three types of core walls (cast-in-place, shotcrete and precast) are considered with circumferential prestressing, using wrapped wire or strand systems, and vertical prestressing (or a vertically fluted steel diaphragm), using single or multiple high strength strands, bars, or wires;
- ACI 373R.97 and AWWA D115-06 for tanks prestressed with circumferential post-tensioned tendons placed within or on the external surface of the wall and vertical prestressed reinforcement.

As for steel tanks, the maximum action effects induced in the seismic design situation should be less or equal to the resistance of the shell evaluated as in the persistent or transient design situations. For concrete shells, the calculation of resistances and the verifications should be carried out in accordance with the ultimate limit state in bending with axial force and the ultimate limit state in shear for in-plane or radial shear (Point 3.5.2.2, EC8-4).

1.7 A Simplified Description of the Seismic Response of Tanks

The dynamic fluid pressures developed during an earthquake are important in the design of containment structures, such as tanks and silos. The analysis of a tank–fluid system requires preliminary considerations of the motions of both the structure and the contained liquid. As shown in Figure 1.19 when the base of a tank containing liquid is subjected to a lateral acceleration $a(t)$, the hydrodynamic pressures exerted by the fluid on the wall and base change in intensity and distribution from those corresponding to a state of hydrostatic equilibrium [214]. For a horizontally excited upright containment structure, a portion of the fluid close to the upper free surface tends not to displace laterally with the tank walls, experiencing vertical sloshing or rocking oscillations about a horizontal axis normal to the direction of the excitation (essentially antisymmetric modes) [66]. Nearer the floor base and along the walls at the bottom, the fluid is unable to move out of the way as the tank displaces and moves synchronously with the wall acting as an added mass rigidly attached to the wall [626]. The motion of the fluid in a storage container may be considered to be composed of two main parts: the first that moves in unison with the tank and is known as the *rigid impulsive* mass component, and the second undergoing sloshing motion that is known as

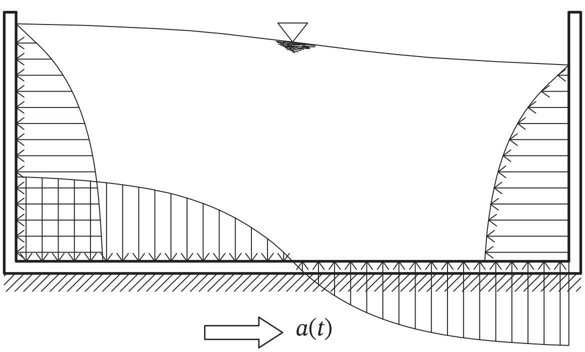


Figure 1.19 Hydrodynamic pressure distribution on the wall and base of a upright tank induced by ground acceleration $a(t)$. Source: Adapted from [412].

the *convective* mass component. The proportions of the liquid mass that participate in the two types of motion depend on the geometry of the tank: height of the free surface of the fluid H, diameter $D = 2R$ and radius R. The portion of the fluid that acts in a convective fashion decreases as the tank aspect ratio $\gamma = \frac{H}{R}$ increases, with the rigid mode becoming more predominant. In the case of tall/slender tanks, the pressure induced by the impulsive mass component is maximum near the tank base and is associated with high–frequency oscillations. Conversely, for tanks of very low aspect ratio, such as shallow/squat/broad tanks, only about 30% of the total fluid acts with the walls, with the remainder responding in sloshing modes [250]. In this case, the pressure induced by the convective component is maximum at the liquid surface and is associated with low–frequency oscillations.

It is often advantageous to replace the liquid, conceptually, by an equivalent mechanical model [251] based on a system of rigid bodies, spring masses, and stiffness [119, 646]. Work by Graham and Rodriguez [189], Westergaard [664], Housner [240–242], Werner and Sundquist [663], Jacobsen, Hoskins and Ayre [238, 263, 265] has resulted in the simple mechanical analogy illustrated in Figure 1.20 in the case of a rigid tank wall. As discussed in Section 5.1, with flexible walls, the deformability of the shell is such that the natural cantilever period of the tank, acting with the fluid inertia mass, is significant and furthermore the flexibility of the tank walls also interacts with the sloshing behaviour, modifying the response. Usually the mechanical model parameters, between the equivalent and the actual/real system, are determined and based on the following conditions [251]:

- the equivalent masses and moments of inertia must be preserved between the equivalent and the actual/real system;
- the center of gravity must be the same for small oscillations;
- the equivalent system must possess the same modes of vibration and produce the same damping forces if compared with the real system;
- the forces (shear) and moments under a certain excitation at the base of the equivalent system must be equivalent to that produced by the actual system.

When the walls of the tank in Figure 1.20 accelerate back and forth, a certain fraction of the liquid, usually called the impulsive mass m_0 or m_i, is forced to participate in this motion and exerts a reactive force on the tank. The mass m_0 is rigidly linked to the wall at a height h_0 or h_i so that the horizontal force exerted by it is collinear with the resultant force exerted by the equivalent impulsive fluid [242]. A complete mechanical analogy for transverse sloshing must include an infinite number of oscillating

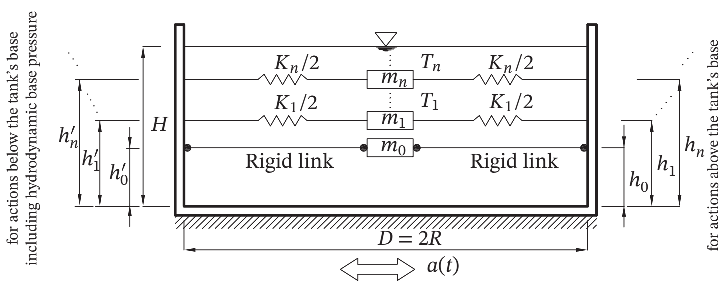


Figure 1.20 Equivalent mechanical model for response of contents of a rigid tank: impulsive and convective motions. Source: From [150]/American Society of Civil Engineers.

masses $m_1, m_2, \dots, m_n, \dots$, one for each of the infinity of normal sloshing modes, connected to the tank walls by springs of stiffness [88, 100, 646]:

$$K_n = 4\pi^2 \frac{m_n}{T_n^2} \tag{1.37}$$

The flexibility of the attached masses represents the various antisymmetric slosh frequencies of the fluid in the tank associated with vertical displacements. The sum of the impulsive and convective masses $(m_0, m_1, m_2, \dots, m_n, \dots)$ is equal to the total mass of the liquid m. As shown by Malhotra in [370], for most tanks in the range $0.3 < \gamma = \frac{H}{R} < 3$ (from shallow to very slender containers), the impulsive mode m_0 and the first convective mode m_1 together account for 85–98% of the total liquid mass in the tank. The results obtained using only the first impulsive and first convective modes are considered satisfactory in most cases. However, due to the fact that the sloshing liquid determines the freeboard requirements, to estimate fluid displacements it has been shown that higher modes should be considered as highlighted in Section 2.2.8 [377]. The behaviour of tanks containing fine granular material, such as cement or grains, may be expected to have very limited convective response. Furthermore many silos containing fine granular materials have high aspect ratios (usually $\frac{H}{R} > 2$) and for these containers the behaviour can be well approximated to that of similar tanks containing fluid of the same density as the granular material, since the fluid response of such tanks is dominated by impulsive motions (Point 1.4, NZSEE, 2009 Edition).

1.8 Discussion of the Existing Codes

Table 1.2 lists various international codes [659], standards, guidelines and recommendations, in order to highlight the main details on the types of tanks considered in each of the following documents [252, 270]:

- UNI EN 1998-4:2006 *Design of structures for earthquake resistance. Part 4: silos, tanks and pipelines* in the following abbreviated as a EC8-4. This standard specifies principles, analyses, and design rules for the seismic evaluation of the structural aspects of industrial facilities composed of above-ground and buried pipelines and of storage tanks and silos of different types and uses. The standard includes criteria and rules without restriction on sizes, structural materials (reinforced concrete, prestressed concrete, steel), types (above-ground, underground, elevated, rigid, flexible, cylindrical and rectangular, anchored and unanchored/uplifting), stored materials (either fluid or granular) and soil–structure interaction. Furthermore, UNI EN 14015:2006 is a specification for the design and manufacture of site-built, vertical, cylindrical, flat-bottomed, above-ground, welded, steel tanks for the storage of liquids at ambient temperature and above (mainly for static loads). Annex G of the UNI EN 14015:2006 gives recommendations for the seismic design of storage tanks and is mainly based on the requirements of Appendix E of API 650.
- *Seismic Design of Storage Tanks, Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering*, first edition, December 1986 by Prof. M.J.N. Priestley and then revised in November 2009 by Prof. D. Whittaker. The recommendation, for the sake of brevity denoted as NZSEE-09, has been revised to bring it in line with the New Zealand loading code NZS 1170.5 (2004) "Structural design actions" [667]. The outline of the procedure proposed by the study group is given by Whittaker and Jury in [666]. Herein the same tanks described in EC8-4 are considered and deeply investigated.

Table 1.2 Main categories of tanks considered in earthquake codes and standards.

Document	Year	Nation	Support[a)]	Material[b)]	Tank's type[c)]	Shape[d)]
EC8-4	2006	Europe	A, U	S, RC, P	1, 2, 3, 4, 5	R, C
UNI EN 14015	2006	Europe	A, U	S	1	C
NZSEE	2009	New Zealand	A, U	S, RC, P	1, 2, 3, 4, 5	R, C
ACI 350.3	2006	USA	A, U	RC, P	1, 2, 3	R, C
ACI 371.R	1998	USA	A	S	3	C
API 650	2012	USA	A, U	S	1, 5	C
API 650	2012	USA	A, U	Al	1, 5	C
API 620	2012	USA	A, U	S	1, 5	C
ASCE 7	2010	USA	A, U	S, RC, P	1, 2, 3, 4	R, C
AWWA D100	2011	USA	A, U	S	1, 3, 4	C
AWWA D103	2009	USA	A, U	S	1	C
AWWA D110	2004	USA	A, U	P	1, 2	C
AWWA D115	2006	USA	A, U	P	1, 2	C
IITK-GSDMA	2007	India	A, U	S, RC, P	1, 2, 3, 4	R, C
COVENIN 3623	2000	Venezuela	A, U	S	1	C
AIJ	2010	Japan	A, U	S, RC, P	1, 2, 3, 4	C

a) A = anchored or mechanically-anchored, U = unanchored or self-anchored or uplifting.
b) S = steel, RC = reinforced concrete, P = prestressed concrete, Al = Aluminium or aluminium alloys.
c) 1 = above-ground rigid tanks, 2 = underground rigid tanks, 3 = elevated tanks on shaft-type tower, 4 = elevated tanks on frame-type tower, 5 = flexible tanks.
d) C = cylindrical, R = rectangular.

- ACI (American Concrete Institute), *Seismic Design of Liquid-Containing Concrete Structures and Commentary*, (ACI 350.3-06) prescribes procedures for the seismic analysis and design of liquid-containing concrete structures: rectangular and circular, on-grade and below grade. Concrete pedestal-mounted structures and composite-style elevated water tanks, that consist of a steel water storage tank supported by a cylindrical reinforced concrete pedestal, are covered by ACI 371R-98, *Guide for the Analysis, Design and Construction of Concrete-Pedestal Water Towers*.
- API 650 2012 Edition: "Welded tanks for oil storage", American Petroleum Institute Standards, Washington, DC. This standard establishes minimum requirements for material, design, fabrication, erection, and testing for vertical, cylindrical, above-ground, closed- and open-top, welded storage tanks, used in the storage of petroleum and petroleum products, in various sizes and capacities for internal pressures approximating atmospheric pressure (internal pressure not exceeding the weight of the roof plates). Appendix E provides requirements for the design of welded on-grade steel flat-bottom tanks that may be subject to seismic ground motion. The Annex G of the European Standard UNI EN 14015:2006 gives recommendations for the seismic design of storage tanks and is mainly based on the requirements of Appendix E of API 650; API 620 2012 Edition covers a higher

pressure (not more than 15 lbf/in^2) and temperature (not greater than 250°F) rating than API 650 (atmospheric storage tanks). Appendix L provides minimum requirements for the design of welded storage tanks that may be subject to seismic action according to Appendix E of API 650. API 12B 2009 covers material, design, fabrication, and testing requirements for vertical, cylindrical, above-ground, closed- and open-top, bolted steel storage tanks in various sizes and capacities for internal pressures approximately atmospheric.

- AWWA D100 2011 Edition: "Welded carbon steel tanks for water storage" and AWWA D103 Edition 2009 "Factory-coated bolted carbon steel tanks for water storage", American Water Works Association. They provide minimum requirements for the design, construction, inspection, and testing of new cylindrical, welded and factory-coated bolted steel tanks for the storage of water at atmospheric pressure. The AWWA D115 2006 Edition "Tendon-prestressed concrete water tanks" describes recommended practice for the design, construction, and field observation of concrete tanks using tendons for prestressing. Instead AWWA D110 Edition 2004 "Wire and strand-wound, circular, prestressed concrete water tanks" applies to circular, prestressed concrete water-containing structures with the following four types of core walls: (1) cast-in-place concrete with vertical prestressed reinforcement (2) shotcrete with a steel diaphragm acting as a water barrier and assuring water tightness (3) precast concrete with a steel diaphragm; and (4) cast-in-place concrete with a steel diaphragm.
- ASCE 7 2010 Edition includes nonbuilding structures that may be required to resist the effects of an earthquake. Chapter 15 focuses on tanks and vessels: reinforced concrete, prestressed concrete, steel, aluminium, and fibre-reinforced plastic materials.
- IITK-GSDMA (2007) "Guidelines for Seismic Design of Liquid Storage Tanks, Previsions with Commentary and Explanatory Examples" freely downloaded from NICEE, National Information Centre of Earthquake Engineering (Indian Institute of Technology, Kanpur and Gujarat State Disaster Management Authority).[5] Two main documents can also be obtained entitled "Review of Code Provisions on Design Seismic Forces for Liquid Storage Tanks" and "Review of Code Provisions on Seismic Analysis of Liquid Storage Tanks" where IBC, ACI (371R and 350.3), AWWA (D100, D103, D110, D115), API 650, Eurocode 8, Part 4, and NZSEE are reviewed and compared in order to highlight similarities, discrepancies, and limitations in design, analysis, and modelling of tanks.
- COVENIN 3623 2000 Edition, "Diseño sismorresistente de tanques metálicos", is a Natinal Code from Venezuela. This standard establishes minimum requirements for design and analysis of vertical and cylindrical steel storage tanks with internal pressures approximating atmospheric pressure.
- NCh2369 2003 Edition, "Diseño sismico de estructuras e instalaciones (industriales)" (Seismic design for industrial structures and facilities), is a National Code from Chile. This standard at Chapter 11.7 and 11.8 focuses on above-ground and elevated tanks by referring to the American standard introduced above.
- AIJ (Architectural Institute of Japan), *Design Recommendation for Storage Tanks and their Supports with Emphasis on Seismic Design*, 2010 Edition. This Design Recommendation is applied to the structural design, mainly the seismic design, of water storage tanks, silos, spherical storage tanks (pressure vessels), flat-bottomed, cylindrical, above-ground storage tanks, and underground storage tanks.

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5 All the documents may be obtained from http://www.nicee.org/IITK-GSDMA_Codes.php from the National Information Center of Earthquake Engineering in India.

Though the seismic analyses of liquid-containing tanks are retained by various codes of practices, their implementation strategy is rather varied, leading to significantly different design forces in some cases. Nine documents (IBC 2006 Edition, Eurocode 8, Part 4 - 1998 Edition, NZSEE, ACI 350.3 2001 Edition, ACI 371 1998 Edition, AWWA D-100-05, AWWA D-110-95, AWWA D-115-95, and API 650 - 2005 Edition) have been reviewed and compared by Jaiswal et al. [270].

1.9 Structure of the Book

The response quantities related to the equivalent model depicted in Figure 1.20 [250, 451] and the seismic design codes compared in Table 1.2 are investigated corresponding to the following main typologies [200] (Point 4.2, HSE Research Report RR527):

1) above-ground anchored rigid tanks: cylindrical (Section 2.2) and rectangular (Section 2.3);
2) above-ground unanchored tanks: cylindrical (Section 3.2) and rectangular (Section 3.3);
3) underground rigid tanks (Section 6.5);
4) elevated tanks on a shaft and frame-type tower (Section 4.1);
5) flexible tanks (Section 5.1).

Particular sections will be introduced in order to highlight some important aspects regarding:

1) effects of liquid viscosity (Section 2.2.6);
2) effects of inhomogeneous liquids (Section 2.2.7);
3) effects of soil-structure interaction (Section 6.2);
4) flow-dampening device systems (Section 6.3);
5) isolation systems (Section 6.4).