PROGETTO DI STRUTTURE IN ACCIAIO

Corso di aggiornamento per ingegneri organizzato da APICE srl e prof. Aurelio Ghersi

col patrocinio di:

Ordine degli ingegneri della provincia di Perugia e Fondazione Promozione Acciaio

Problematiche costruttive, strutturali e funzionali di serbatoi e sili in acciaio

Parte 2: Progetto di serbatoi

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DESIGN OF CYLINDRICAL TANKS

General

Oil and oil products are most commonly stored in cylindrical steel tanks at atmospheric pressure or at low pressure. The tanks are flat bottomed and are provided with a roof which is of conical or domed shape.

Water is also sometimes stored in cylindrical steel tanks. When used to store potable water they are of a size suitable to act as a service reservoir for a local community; they have a roof to prevent contamination of the water. Cylindrical tanks are also used in sewage treatment works for settlement and holding tanks; they are usually without a roof.

The sizes of cylindrical tanks range from a modest 3m diameter up to about 100m diameter, and up to 25m in height. **They consist of three principal structural elements - bottom, shell and roof.**

For **petroleum** storage, the bottom is formed of steel sheets, laid on a prepared base. Some tanks for water storage use a reinforced concrete slab as the base of the tank, instead of steel sheets.

The shell, or cylindrical wall, is made up of steel sheets and is largely **unstiffened**.

The roof of the tank is usually fixed to the top of the shell, though floating roofs are provided in some circumstances. A fixed roof may be self supporting or partially supported through membrane action, though generally the roof plate is supported on radial beams or trusses.

Even though common standards are generally applicable to both oil and water tanks, it is the petroleum industry which has been responsible for the development of many of the design procedures and standards. The standards applied most widely are **British Standard BS 2654**, the **American Petroleum Institute Standard API 650** and the **Eurocode 3 (EN1993-4-2)**.

Main source ESDEP

Design Pressure and Temperature

Tanks designed for storage at nominally atmospheric pressure must be suitable for modest internal vacuum (**negative pressure**). Tanks may also be designed to work at relatively **small positive internal pressures** (in the order of 50 mbar (5,0 kN/m^2).

Non-refrigerated tanks are designed for a minimum metal temperature which is based on the lowest ambient air temperature (typically, ambient plus 10^{o}C) or the lowest temperature of the contents, whichever is the lower. No maximum service temperature is normally specified.

Material

Tanks are usually manufactured from **plain carbon steel** plate (traditionally referred to as mild steel) of grades S235 or S275 (to EN 10 025, or equivalent. Such material is readily weldable. The use of higher strength grades of low alloy steel (e.g. Grade S355) is less common, though its use is developing.

Notch ductility at the lowest service temperature is obtained for thicker materials (> 13 mm) by specifying minimum requirements for impact tests. This is normally achieved by specifying an appropriate sub-grade to EN 10 025.

Internally, oil tanks are normally **unpainted**. Water tanks may be given a coating (provided it is suitably inert, where the water is potable), or may be given cathodic protection. Externally, tanks are normally **protected**. Where any steel is used uncoated, an **allowance** must be made in the design for loss of thickness due to **corrosion**.

Wall coatings

Glass coating *has significantly lower coating maintenance costs compared to welded or painted bolted steel tanks. Glass formulation is inert and chemically bonds to the steel once the panels are fired at 850 °C. This gives the coating a great adherence (> 10 times the bond strength of field applied or baked on paint). The glass-fused-to-steel coating does not need to be re-applied vs. the periodic re-painting of other tanks. Also, it provides a water-resistant interior and a graffiti-resistant exterior.*

Epoxy coatings *are an excellent choice for hot and pure water applications as well other specialized storage needs. They have an interior liquid immersion coating system that combines good chemical resistance and physical properties and meets NSF Standard 61 requirements for potable water contact surfaces.*

Galvanized *tanks are hot dipped in compliance with relevant standards. Steel parts are pickled then dipped in molten zinc so both the interior and exterior is coated. These tanks are ideal in fire water, process water and wastewater applications.*

Stainless steel and aluminum alloy *tanks are offered in a variety of different grades. These tanks withstand the wider range of temperatures and PH factors encountered in industrial applications. Stainless steel and aluminum alloys tanks are virtually maintenance free and ideal for corrosive environments.*

ACTIONS ON TANKS

A tank is designed for the **most severe combination** of the various possible loadings.

Dead Load

The dead load is that due to the **weight** of all the parts of the tank.

Superimposed Load

A **minimum** general superimposed load in the order of 1,2 kN/m^2 (over the horizontal projected area) is applied to the roof of the tank. This load is sometimes mistaken for the **'snow load'**, but in fact represents, as well as a nominal snow load, any other imposed loads, such as maintenance equipment, which might be applied to the roof, and it includes the internal vacuum load. It is therefore applied even in locations where snow is not experienced.

Non-pressure tanks are often fitted with valves which do not open until the vacuum reaches a value of 2,5 mbar, to contain vapour losses. By the time a valve is fully open, a vacuum of 5 mbar (0,5 kN/m^2) may have developed. Even without valves a tank should be designed for a vacuum of 5 mbar (0,5 kN/m^2), to account for differential pressure under wind loads. In pressure tanks the valves may be set to 6 mbar vacuum, in which case a pressure difference of 8,5 mbar (0,85 kN/m^2) may develop.

Actual code-predicted snow load or other superimposed load, plus appropriate vacuum pressure, should be used when it is greater than the specified minimum.

Liquid induced loads

The weight and hydrostatic pressure of the contents, up to the full capacity of the tank, should be applied. Full capacity is usually determined by an overflow near the top of the tank; for a tank without any overflow, the contents should be taken to fill the tank to the top of the shell.

For oil and oil products, the relative density of the contents is less than 1.0, but tanks for such liquids are normally tested by filling with water. A density of 1000 kg/m^3 should therefore be taken as a minimum.

During operation, the load due to the contents, that is the weight of the product to be stored from maximum design liquid level to empty.

During test, the load due to the contents, that is the weight of the test medium from maximum test liquid level to empty.

Internal pressure loads

During operation, the internal pressure load due to the specified minimum and maximum values of the internal pressure.

During test, the internal pressure load due to the specified minimum and maximum values of the test internal pressure.

Distributed live load

Concentrated live load

Snow

Wind

Wind loads are determined on the basis of a design wind speed. Maximum wind speed depends on the area in which the tank is to be built; typical wind values are specified in the codes. Generally, a given value is taken as the design wind speed, representing the maximum gust speed which is exceeded, on average, only once every 50 years.

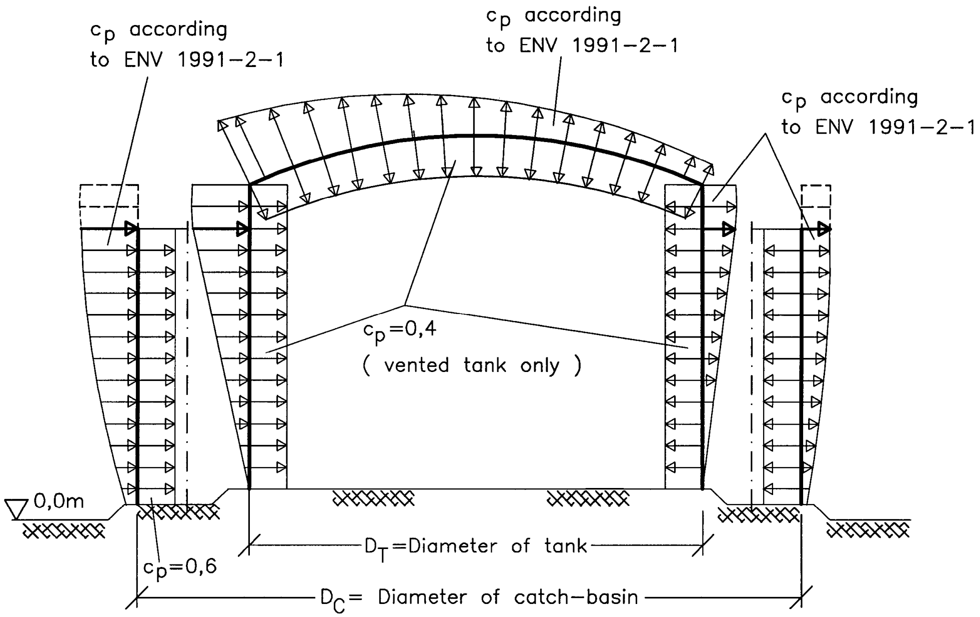
Wind load according to EC3

The following pressure coefficients may be used for circular cylindrical tanks:

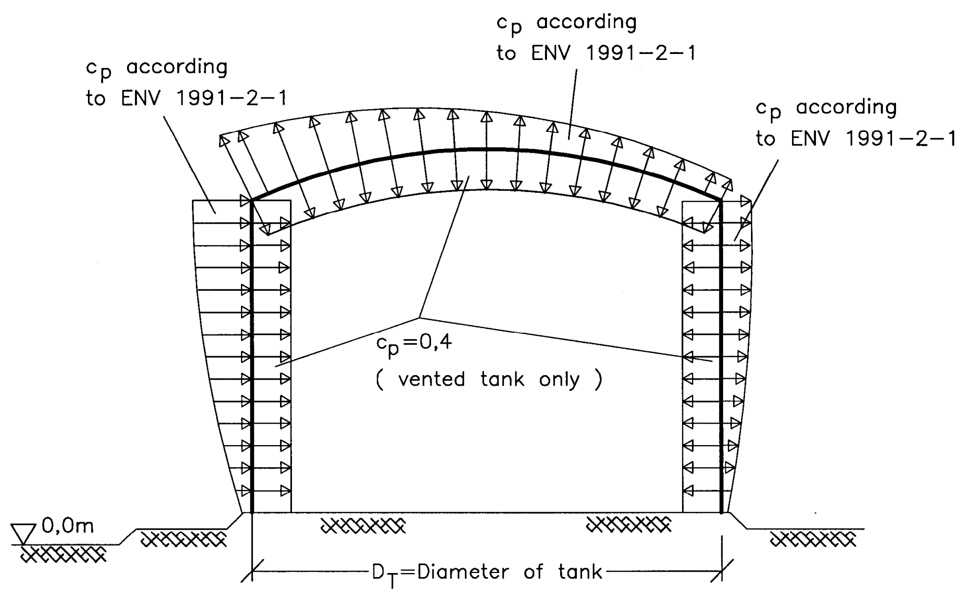
a) internal pressure of open top tanks and open top catch basin: c_p = -0,6.

b) internal pressure of vented tanks with small openings: c_p = -0,4.

c) where there is a catch basin, the external pressure on the tank shell may be assumed to reduce linearly with height.



a) Tank with catch basin



b) Tank without catch basin

Seismic loadings
In some areas, a tank must be designed to withstand seismic loads. Guidance is given in EC8 (EN1998-4)

Thermally induced loads
Stresses resulting from restraint of thermal expansion (they may be ignored if the number of load cycles due to thermal expansion is such that there is no risk of fatigue failure or cyclic plastic failure).

Insulation loads
The insulation loads resulting from the weight of the insulation.

Suction due to inadequate venting

Loads resulting from connections
Loads resulting from pipes, valves and other items connected to the tank and loads resulting from settlement of independent item supports relative to the tank foundation.

Loads resulting from uneven settlement
Settlement to be taken into account where uneven settlement can be expected during the lifetime of the tank.

Emergency loadings
Loadings from events such as external blast, impact, adjacent external fire, explosion, leakage of inner tank, roll over, overfill of inner tank (to be specified by the relevant authority or by the purchaser).

Reliability differentiation according to EC3

Different levels of reliability shall be adopted for tanks, depending on the possible economic and social consequences of their collapse. Three classes of reliability related to structural safety and serviceability shall be used (Reliability Classes 1, 2 and 3) to be agreed between the designer, the client and the relevant authority. The classification of tanks should be based on the following:

Reliability Class 3: Tanks storing liquids or liquefied gases with toxic or explosive potential and large size tanks with flammable or water-polluting liquids in urban areas. Emergency loadings should be taken into account for these structures where necessary.

Reliability Class 2: Medium size tanks with flammable or water-polluting liquids in urban areas.

Reliability Class 1: Agricultural tanks or tanks containing water

Simplified design procedure allowed in EC3

The simplified analysis of this section may be applied where all the following conditions are satisfied:

- the restrictions given in the code are met (see next slide);
- the tank structure is of the form shown in next slide;
- the only internal actions are liquid pressure and gas pressure above the liquid surface;
- the following loadings can all be neglected: thermally induced loads, seismic loadings, loads resulting from uneven settlement or connections and emergency loadings;
- no course is constructed with a thickness less than that of the course above it, except for the zone adjacent to the eaves ring;
- the design value of the circumferential stress in the tank shell is less than 435 N/mm^2;
- for a spherical roof, the radius of curvature is between 0,8 and 1,5 times the diameter of the tank;
- for a conical roof, the slope of the roof is 1 in 5 if self supported or 1 in 16 if column supported;
- the design gradient of the tank bottom is not greater than 1:100;
- the bottom is fully supported or supported by closely spaced parallel girders;
- the characteristic internal pressure is not below -8,5mbar and not greater than 60mbar;
- the number of load cycles is such that there is no risk of fatigue failure.

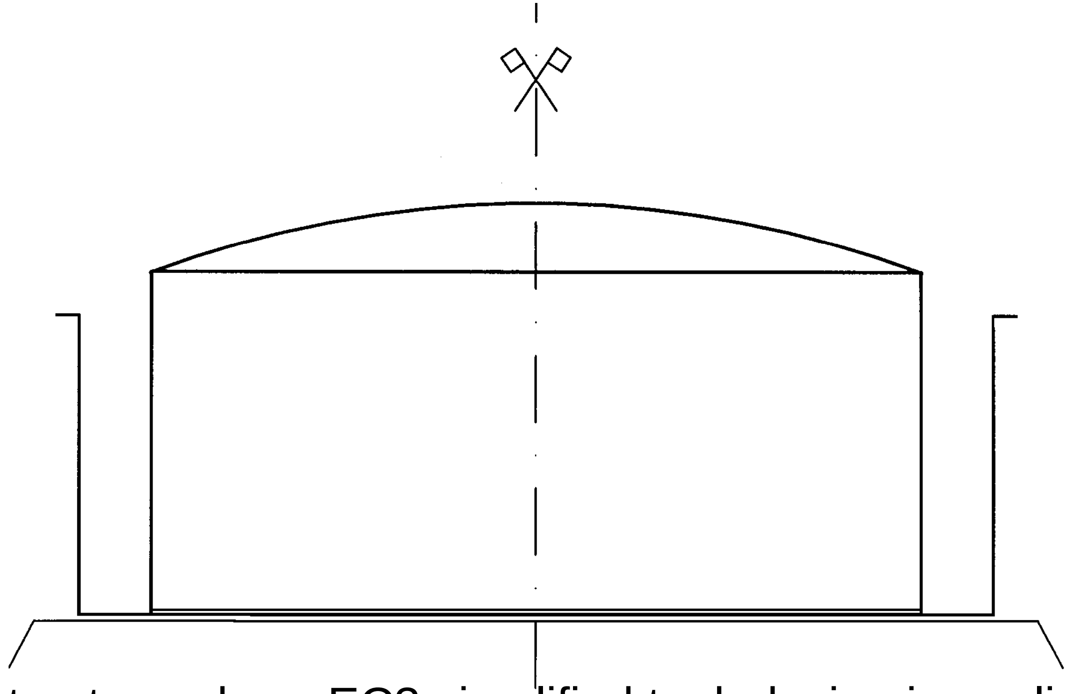
Restrictions given in EN1993-4-2 for applying the simplified procedure

Part 4.2 of Eurocode 3 provides principles and application rules for the structural design of vertical cylindrical above ground steel tanks for the storage of liquid products with the following characteristics:

a) characteristic internal pressures not less than -100mbar and not more than 500mbar ;

b) design metal temperature in the range of -50ºC to +300ºC; for tanks using austenitic stainless steel the design metal temperature may be in the range of -165ºC to +300ºC

c) maximum design liquid level not higher than the top of the cylindrical shell.



Tank structure where EC3 simplified tank design is applicable

FIXED ROOF DESIGN

General

Fixed roofs of cylindrical tanks are formed of steel plate and are of either **conical** or domed (**spherically** curved) configuration. The steel plates can be entirely **self supporting** (by **'membrane**' action), or they may rest on top of some form of support structure.

Membrane roofs are more difficult to erect - they require some temporary support during placing and welding (or bolt fastening) - and are usually found only on smaller tanks.

Permanent support steelwork for the roof plate may either span the complete diameter of the tank or may in turn be supported on columns inside the tank. The use of a single central column is particularly effective in relatively small tanks (15-20 m diameter), for example.

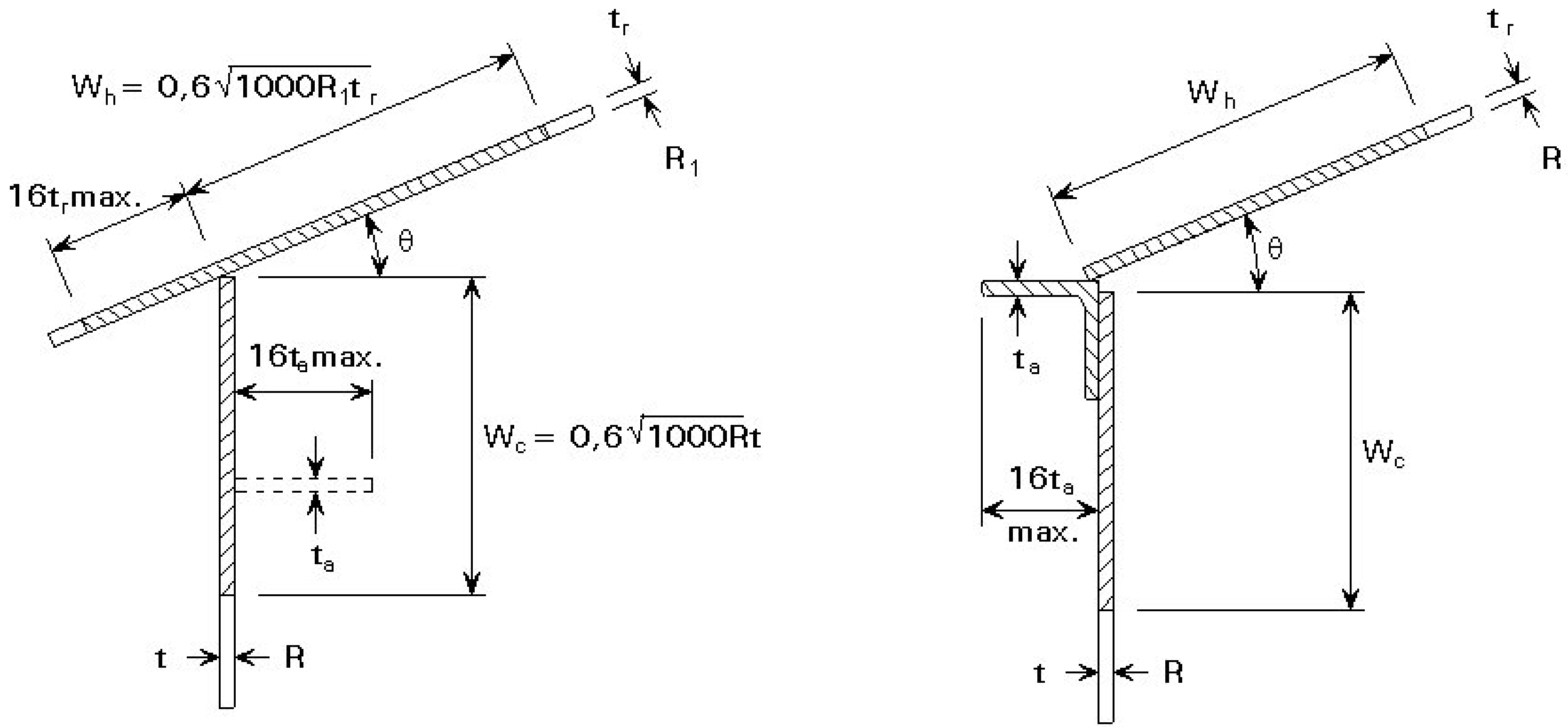
The main members of the support steelwork are, naturally, radial to the tank. They can be simple rolled beam sections or, for larger tanks, they can be fabricated trusses.

Roof plates are usually **lapped** and **fillet welded** to one another. For low pressure tanks, they do not need to be welded to any structure which supports them, but they must normally be welded to the top of the shell.

In a membrane roof, the forces from dead and imposed loads are resisted by **compressive radial stresses**. The net upward forces from internal pressure minus dead load are resisted by **tensile radial stresses**. For **downward loads**, the radial compression is complemented by **ring tension**.

For **upward loads**, i.e. under internal pressure, the radial tension has to be complemented by a **circumferential compression**. This compression can only be provided by the junction section between roof and shell (see next slide).

Conical roofs usually have a slope of 1:5. Spherical roofs usually have a radius of curvature between 0,8 and 1,5 times the diameter of the tank.



Key to symbols

R_1 is the radius of curvature of roof (m) (for conical roofs = R/sin θ)

R is the radius of tank shell (m)

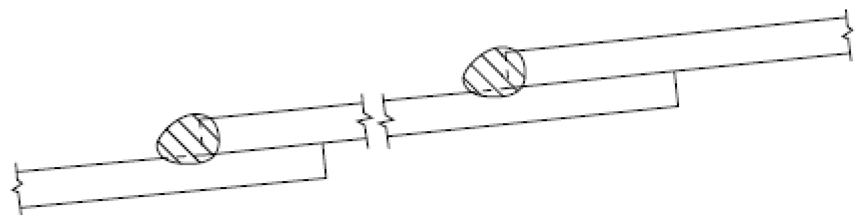
t is the thickness of shell (mm)

t_a is the thickness of angle stiffener (mm)

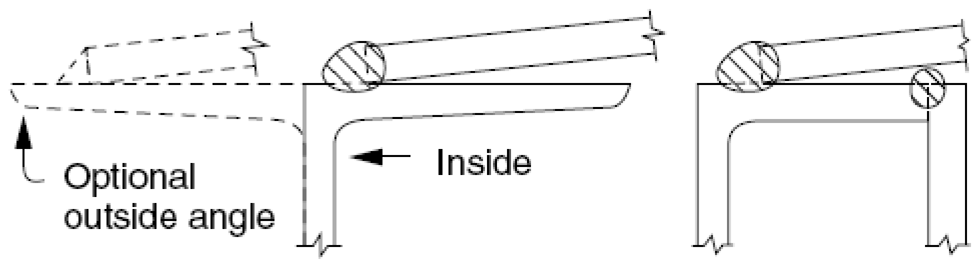
t_r is the thickness of roof plate at compression ring (mm)

W_h is the effective roof length (mm)

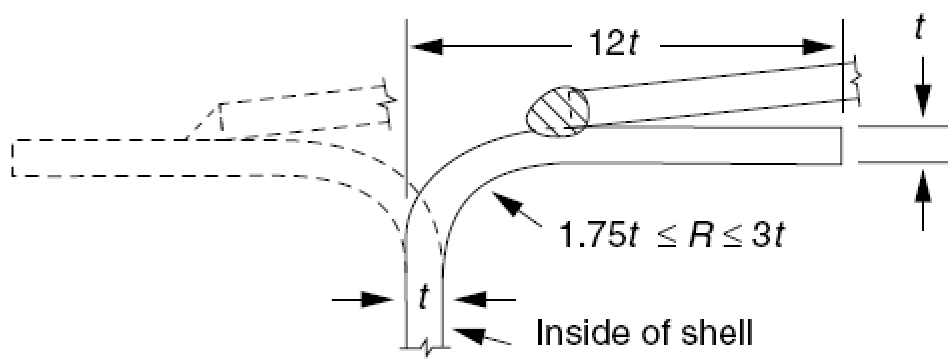
Effective shell roof compression areas (BS2654)



ROOF-PLATE JOINT



ROOF-TO-SHELL JOINTS



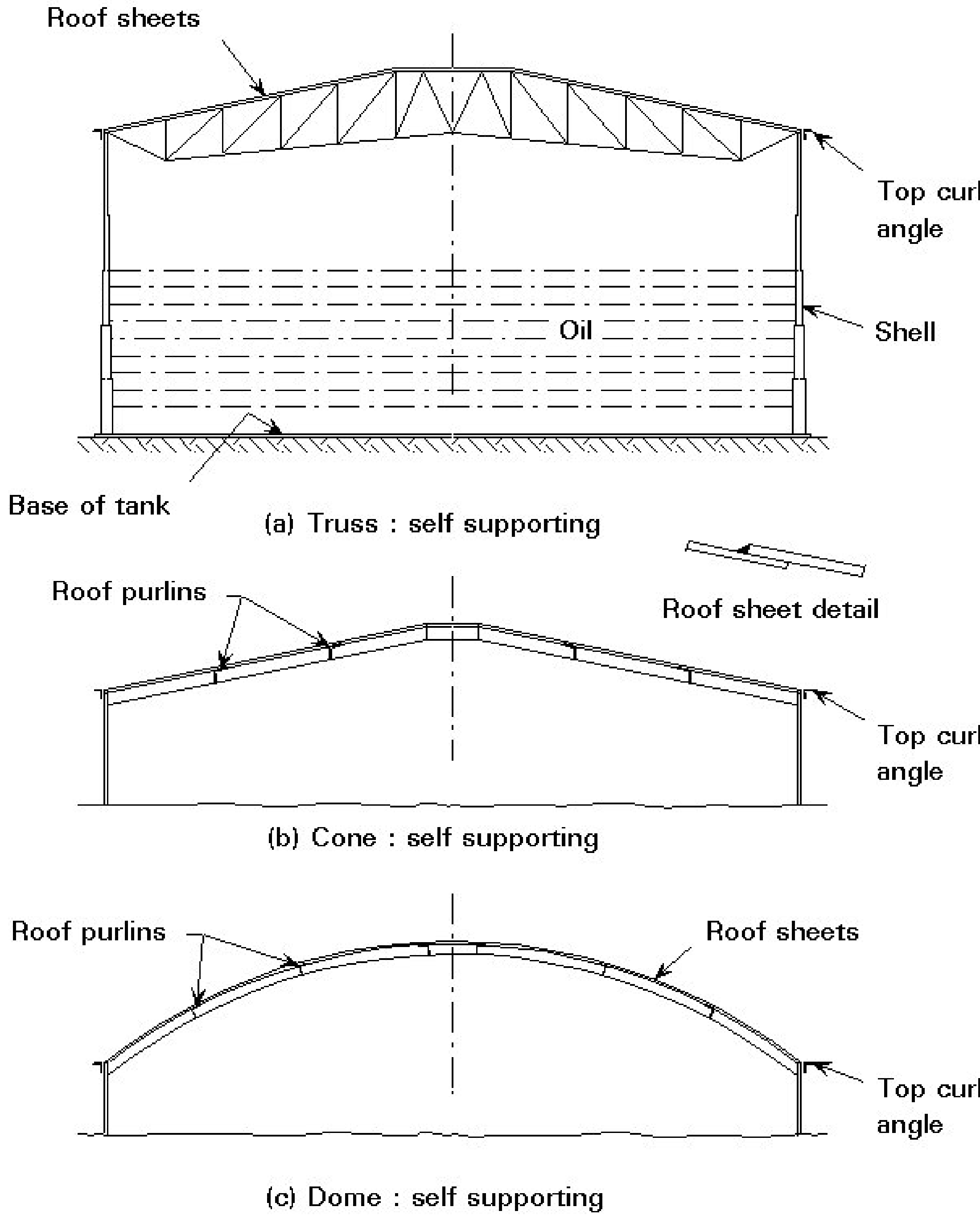
ALTERNATIVE ROOF-TO-SHELL JOINT

Supported Roofs

Radial members supporting the roof plate permit the **plate thickness** to be kept to a **minimum**. They greatly facilitate the construction of the roof. Radial beams are arranged such that the span of the plate between them is kept down to a minimum of about 2 m. This limit allows the use of 5 mm plate for the roof. The plate simply lies on the beams and is **not connected** to them.

Supported roofs are most commonly of conical shape, although spherical roofs can be used if the radial beams are curved.

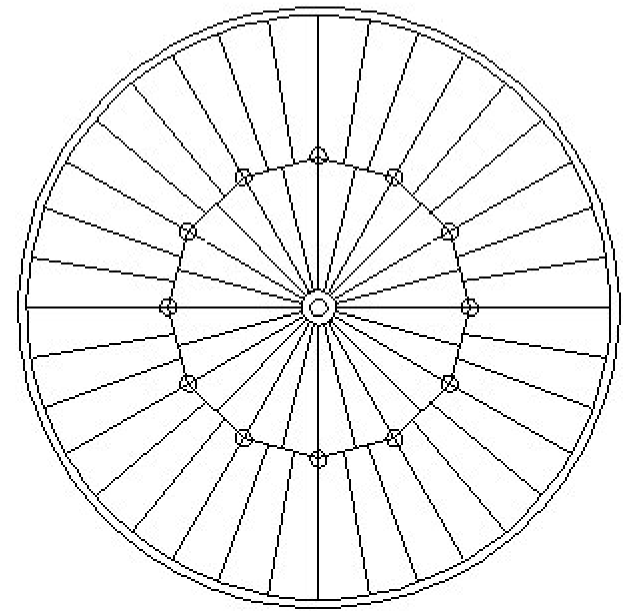
The roof support structure can either be **self supporting** or be supported on **internal columns**. Typical arrangements are shown in section in Figure. Self supporting roofs are essential when there is an internal floating cover.



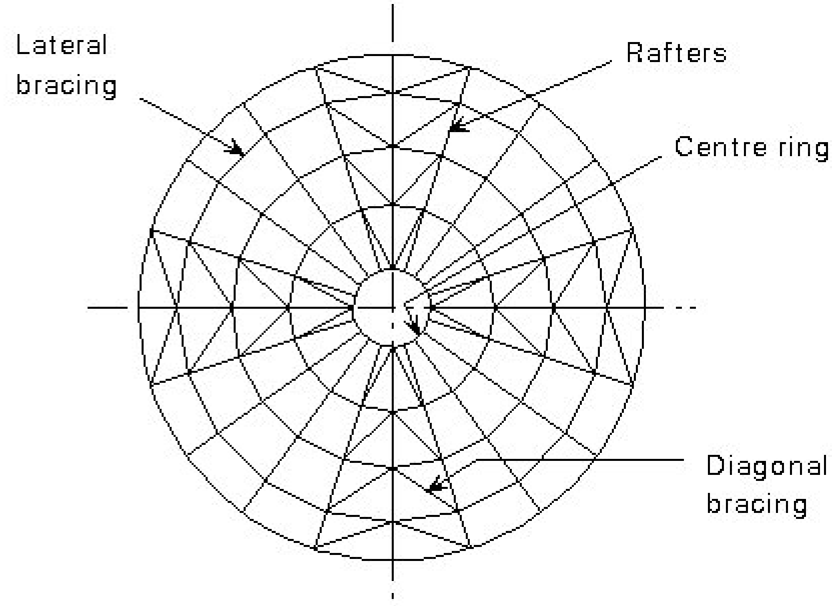
Self supporting fixed roofs

Main source ESDEP

When **columns** are used to support the roof, the slope may be as low as 1:16. When the roof is **self supporting** it may be more economic to use a steeper roof.
Not all radial members continue to the centre of the tank. Those doing that may be considered as **main support beams**; the secondary radial members may be considered as **rafters** - they are supported at their inner ends on **ring beams** between the main support members. Where internal columns are used they will be below the main support members. Typical plan arrangements are shown in the Figure.
The main support members need to be **restrained** at intervals to stabilise them against **lateral-torsional buckling**. Cross **bracing** is provided in selected bays.
In some codes it is permitted to assume that friction between the roof plate and the beam is adequate to restrain the compression flange of the secondary rafter beams, provided that they are not too deep; such restraint cannot be assumed for the main beams, however.
The main support members may be subject to **bending** and **axial load**. Where they are designed for axial thrust, the **central ring** must be designed as a **compression ring**; the top of the shell must be designed for the **hoop forces** associated with the axial forces in the support members.
Design of beams and support columns may generally follow conventional **building code rules**. The **shell/roof junction** zone must be designed for compression, in the same way as for membrane roofs.



Column supports



Alternative support for roofs

Main source ESDEP

Fixed roof design according to EC3

Unstiffened roof shell butt welded or with double lap weld

The strength of the roof under the design internal pressure $p_{o,d}$ should be verified using:

$$\frac{p_{o;d}\, R_s}{2t} \leq j\, f_{y,d} \qquad \frac{p_{o;d}\, R_c}{t} \leq j\, f_{y,d}$$

for spherical roofs and conical roofs, respectively.

in which:

R_c	$= r/\sin\alpha$ for a conical roof
j	is the joint efficiency factor;
$p_{o,d}$	is the radial outward component of the uniformly distributed design load on the roof;
r	is the radius of the tank cylindrical shell wall;
R_c	is the radius of curvature for the conical roof;
R_s	is the radius of curvature of the spherical roof;
t	is the roof plate thickness;
α	is the slope of the conical roof to the horizontal.

The joint efficiency factor should be taken as:

j = 1,00 for butt welds;
j = 0,50 for lapped joints with fillet welds on both sides.

The **stability** of the **roof** under the **design external pressure** $p_{i,d}$ should be verified using:

$$p_{i,d} \leq 0{,}05 \left\{ 1{,}21 E \left(\frac{t}{R_0} \right)^2 \right\}$$

in which:

$R_0 = R_s$ for a spherical roof;
$R_0 = R_c$ for a conical roof;

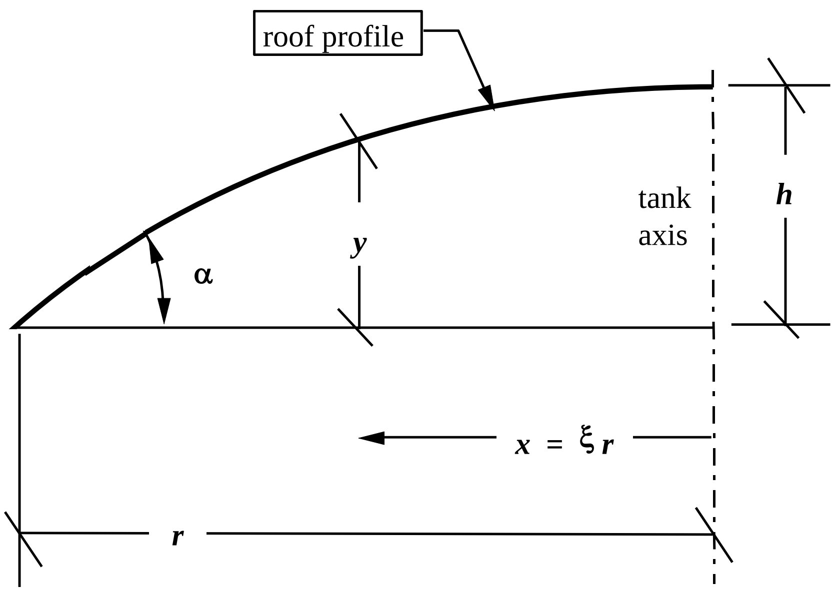
and $p_{i,d}$ is the radial inward component of the uniformly distributed design load on the roof.

Self supporting roof with roof structure

The specified **thickness** of all roof plating should be not less than 3mm for stainless steels and not less than 5mm for other steels.

The design of the roof supporting structure shall satisfy the requirements of EN 1993-1-1.

Provided that the distributed load does not deviate strongly from symmetry about the tank axis, the procedure in the next slide may be used.



Tank roof coordinates

For **spherical** roofs under the action of distributed loads arising from imposed load, snow load, wind load, permanent load and pressure, the maximum vertical component should be taken as the design value p_{vd} acting either upwards or downwards, with p_{vd} taken as negative if it acts upwards. The **total** design vertical force per **rafter** should be taken as:

$$P_d = \beta r^2 p_{vd}$$

in which:

β = π/n

n is the number of rafters;

r is the radius of the tank;

p_{vd} is the maximum vertical component of the design distributed load including the dead weight of the supporting structure (downward positive);

P_d is the total design vertical force per rafter.

The normal force N_d and bending moment M_d in each rafter for design according to ENV 1993-1-1 may be obtained from:

$$N_{Ed} = 0{,}375 \frac{r}{h} P_{Ed}$$

$$M_{Ed} = \frac{1}{3}\left(\frac{r}{1-\varepsilon}\right)\left\{1-\left(\frac{x}{r}\right)^3 - 1{,}10\left(\frac{y}{h}\right)\right\} P_{Ed}$$

in which h is the rise of the tank roof and:

$$\varepsilon = N_{Ed} \frac{(0{,}6r)^2}{\pi^2 EI_y}$$

For **column supported roof** the specified thickness of all roof plating should be not less than 3mm for stainless steels and not less than 5mm for other steels.
The design of the roof supporting structure shall satisfy the requirements of EN 1993-1-1.
If the roof plates are not connected to the rafters, **bracing** shall be used.
For roofs exceeding 15 m diameter, at least **two bays of bracing** should be provided (i.e. two pairs of adjacent rafters connected by truss members). The sets of braced bays should be evenly spaced around the tank circumference.
For braced roofs with diameter between 15 m and 25 m, an **additional circumferential ring** should be provided. For braced roofs with diameter over 25 m, **two additional circumferential rings** should be provided.
The bracing should be designed for a stabilising force equal to 1% of the sum of the normal forces in the stabilised members.
The force in the **effective edge ring** (area where the roof is connected to the shell) should be verified using:

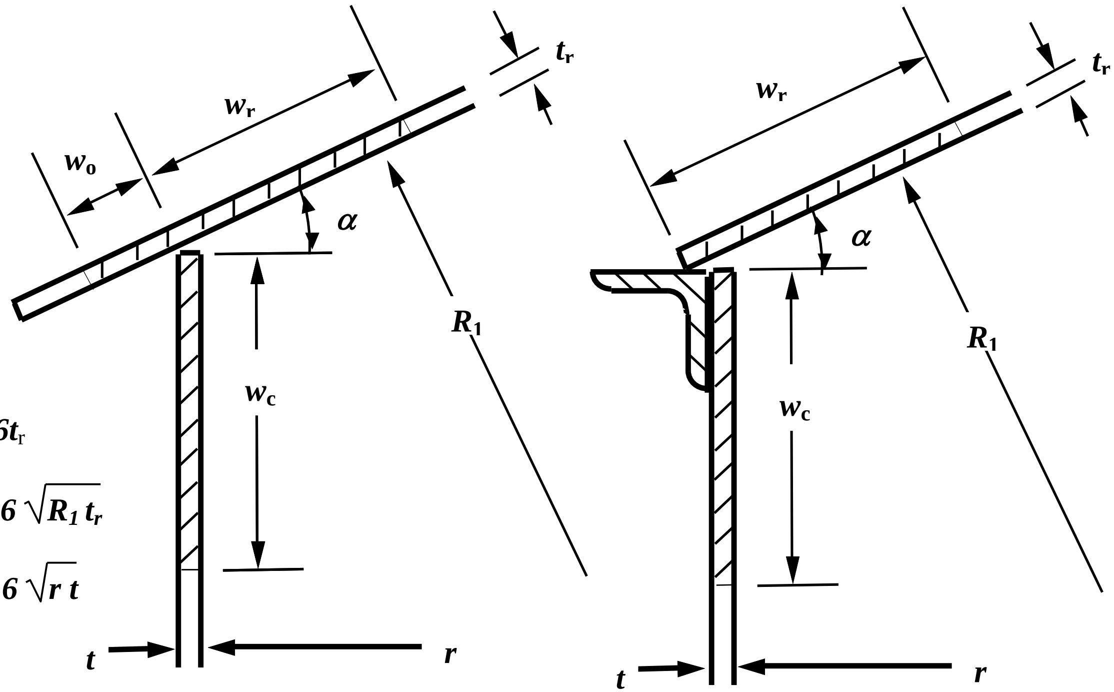
$$\frac{N_{\mathrm{d}}}{A_{\mathrm{eff}}} \leq f_{\mathrm{y,d}} \qquad \text{in which} \qquad N_{\mathrm{d}} = \frac{p_{\mathrm{vd}}\, r^2}{2 \tan\alpha}$$

where:

A_{eff} is the effective area of the edge ring;

α is the slope of the roof to the horizontal at the junction;

p_{vd} is the maximum vertical component of the design distributed load including the dead weight of the supporting structure (downward positive).



$w_o \leq 16t_r$

$w_r = 0,6\sqrt{R_1 t_r}$

$w_c = 0,6\sqrt{r\,t}$

Effective edge ring at the shell to roof junction

Where the separation between adjacent rafters at their points of connection to the edge ring does not exceed 3,25m, the **stability** of the edge ring need not be verified.

Where the design distributed load p_{vd} acts **upwards**, the bending moments in the edge ring may be ignored.

Where the separation between adjacent rafters at their points of connection to the edge ring does not exceed 3,25m, and the design distributed load p_{vd} acts **downwards**, the **bending moments** in the edge ring may be ignored.

Where the separation between adjacent rafters at their points of connection to the edge ring exceeds 3,25m, the **bending moments** in the edge ring about its vertical axis should be taken into account in addition to the **normal force** in the ring N_d.

The bending moments in the ring (**positive values inducing tensile stresses on the inside of the ring**) should be evaluated using the following expressions.

At the connection of the rafter:

$$M_{s,Ed} = -\left(\frac{p_{v,Ed} r^3}{2\tan\alpha}\right)\left(1 - \frac{\beta}{\tan\beta}\right)$$

At half span between the rafters:

$$M_{F,Ed} = -\left(\frac{p_{v,Ed} r^3}{2\tan\alpha}\right)\left(\frac{\beta}{\sin\beta} - 1\right)$$

SHELL DESIGN

Circumferential Stresses

Vertical cylinder tanks carry the **hydrostatic pressures** by **simple hoop tension**. No circumferential stiffening is needed for this action. The circumferential tension in the shell will vary directly, in a vertical direction, according to the head of fluid at any given level.
The circumferential normal stress due to liquid loads and internal pressure should be verified in each shell course using:

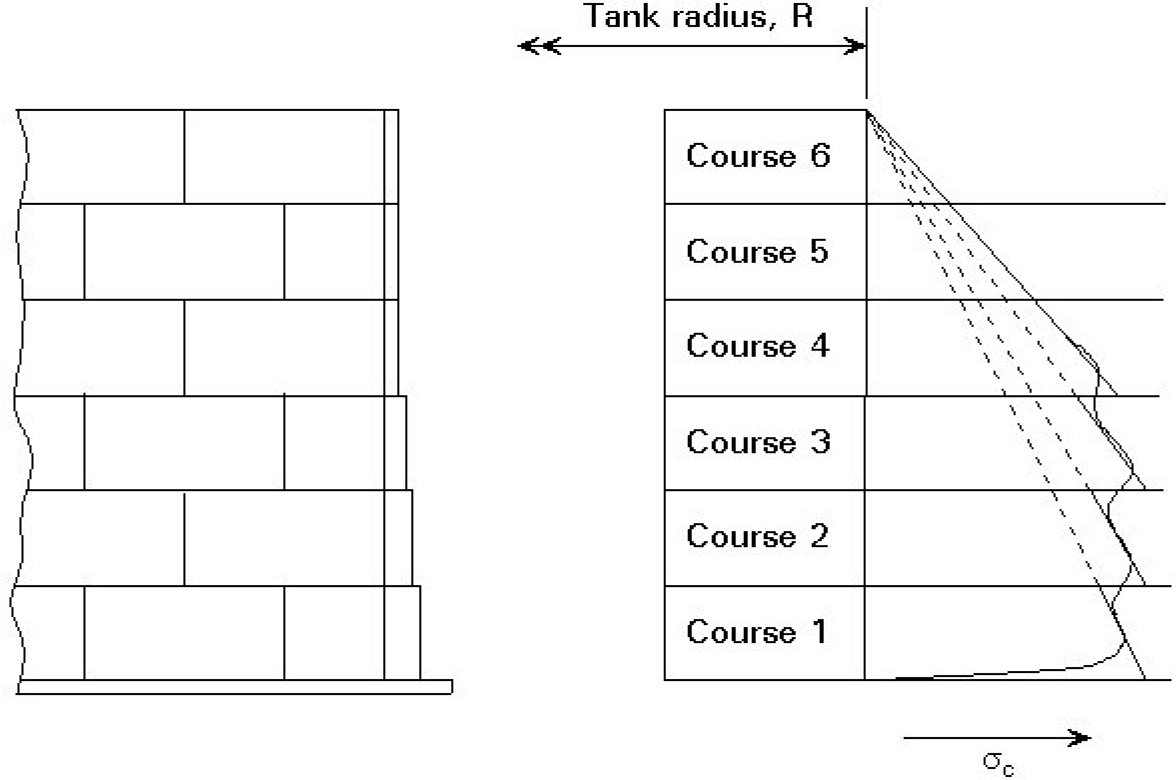
$$\left[\gamma_F \, \rho g H_{red} + p_d\right]\left(\frac{r}{t}\right) \leq f_{y,d}$$

where:

ρ	is the density of the contained liquid;
g	is the acceleration due to gravity;
$H_{red,j}$	(= H_j – 0,30m) is the vertical distance from the bottom of the j-th course to the liquid level;
p_d	is the design value of the pressure above the liquid level.

For practical reasons, it is necessary to build up the shell from a number of fairly **small rectangular pieces of plate**, connected together. Each piece will be cylindrically curved and it is convenient to build up the shell in a number of rings, or courses, one on top of the other. This technique provides, at least for deeper tanks, a convenient opportunity to use thicker plates in the lower rings and thinner plates in the upper rings.
The lowest course of plates is fully welded or bolted to the bottom plate of the tank providing **radial restraint** to the bottom edge of the plate. Similarly, the bottom edge of any course which sits on top of a thicker course is somewhat restrained because the thicker plate is stiffer. The effect of this on the hoop stresses is illustrated in next slide.

Because of these restraints, an **empirical adjustment** is introduced into the design rules which effectively requires that any course is simply designed for the **pressure 0,3m above the bottom edge** of the course, rather than the greater pressure at the bottom edge. (This is known as the **'one foot rule**' in API 650).



Stress variation in shell wall

Constructional issues
Each course is made of a number of plates, butt welded or bolted along the vertical join between the plates. Each course is joined to the course below along a circumferential line. Good geometry and weld procedures can minimise the distortions or deviations from the ideal flat or curved line of the surface across the weld, but some **imperfection** is inevitable, especially with thin material. Consequently the rules call for the vertical seams between panels to be **staggered** from one course to the next - at least one third of the length of the individual plates, if possible.

When the load due to internal pressure is taken into account an allowance for **corrosion** loss is introduced. An increased value of the wall thickness is determined according to the codes.

Holes in the shell for inlet/outlet **nozzles** or access **manholes** cause a local increase in circumferential stresses. This increase is accounted for by requiring the provision of reinforcing plates. These plates may take the form of a circular doubling plate welded around the hole or of an inset piece of thicker plate. The **nozzle** provides some stiffening to the edge of the hole; it may also be made of sufficient size that shell reinforcement can be omitted.

Some codes dealing with tanks still follow the **allowable stress** approach. The allowable design stress in tension in the shell is generally taken to be a suitable **fraction** of the material yield stress. BS 2654 defines it as two-thirds of the yield stress, thus giving an overall factor of 1,5 on the plastic strength of the plate. API650 also uses two-thirds of the yield stress, but additionally limits the design stress to a smaller fraction of the ultimate strength; for higher strength steels, this is slightly more restrictive. Further, API650 allows a slightly higher stress during the hydrostatic test than the allowable design stress for service conditions when the relative density is less than 1,0.

Axial Stresses in the Shell

The cylindrical shell has to carry its weight, and the weight of the roof which it supports, as an axial stress. In addition, wind loading on the tank contributes tensile axial stress on one side of the tank and compressive stress on the other.
A thin-walled cylinder under a sufficient axial load will of course buckle locally, or wrinkle. The **critical** value of this stress, for a perfect cylinder, can be obtained from classical theory and, for steel, has the value:

$$\sigma_{cr} = 0{,}605E\frac{t}{R}$$

In practice, **imperfect** shells buckle at a **much lower stress**; a limit stress level in the order of as little as a tenth of the above is more realistic. However, in normal service the axial stresses in shells suitable to carry the circumferential loads for the size of tank used for oil and water storage are much smaller than even this level of stress. But under seismic conditions, larger stresses result because of the large overturning moment when the tank is full. In that case the axial stresses must be calculated. Axial stress due to overturning moment, M, is given simply by the expression:

$$\sigma_a = 4M/\pi tD^2$$

In BS 2654 the axial stress under seismic conditions is limited to 0.20Et/R, which is considered a reasonable value when the cylinder is also under internal hydrostatic pressure. API650 uses a similar value, provided that the internal pressure exceeds a value which depends on the tank size. **Buckling checks are carried out according to EN1993-1-6.**

Main source ESDEP

Primary Wind Girders

A tank with a **fixed roof** is considered to be **adequately restrained** in its cylindrical shape by the roof; no additional stiffening is needed at the top of the shell, except possibly as part of an effective compression ring. At the top of an **open tank** (or one with a floating roof), circumferential stiffening is needed to maintain the roundness of the tank when it is subject to wind load. This stiffening is particularly necessary when the tank is empty.

As the calculation of the stability of stiffened tanks is quite complex, empirical formulas, such as the **De Wit**'s one, can be easily applied in design. In **BS 2654** this formula is expressed as a **required minimum section** modulus given by $Z = 0{,}058\, D^2 H$, where Z is the (elastic) section modulus (cm^3) of the effective section of the ring girder, including a width of shell plate acting with the added stiffener, D is the tank diameter (m), H is the height of the tank (m). The formula presumes a design wind speed of 45 m/s. For other wind speeds it may be modified by multiplying by the ratio of the basic wind pressure at the design speed to that at 45 m/s, i.e. by $(V/45)^2$.

Main source ESDEP

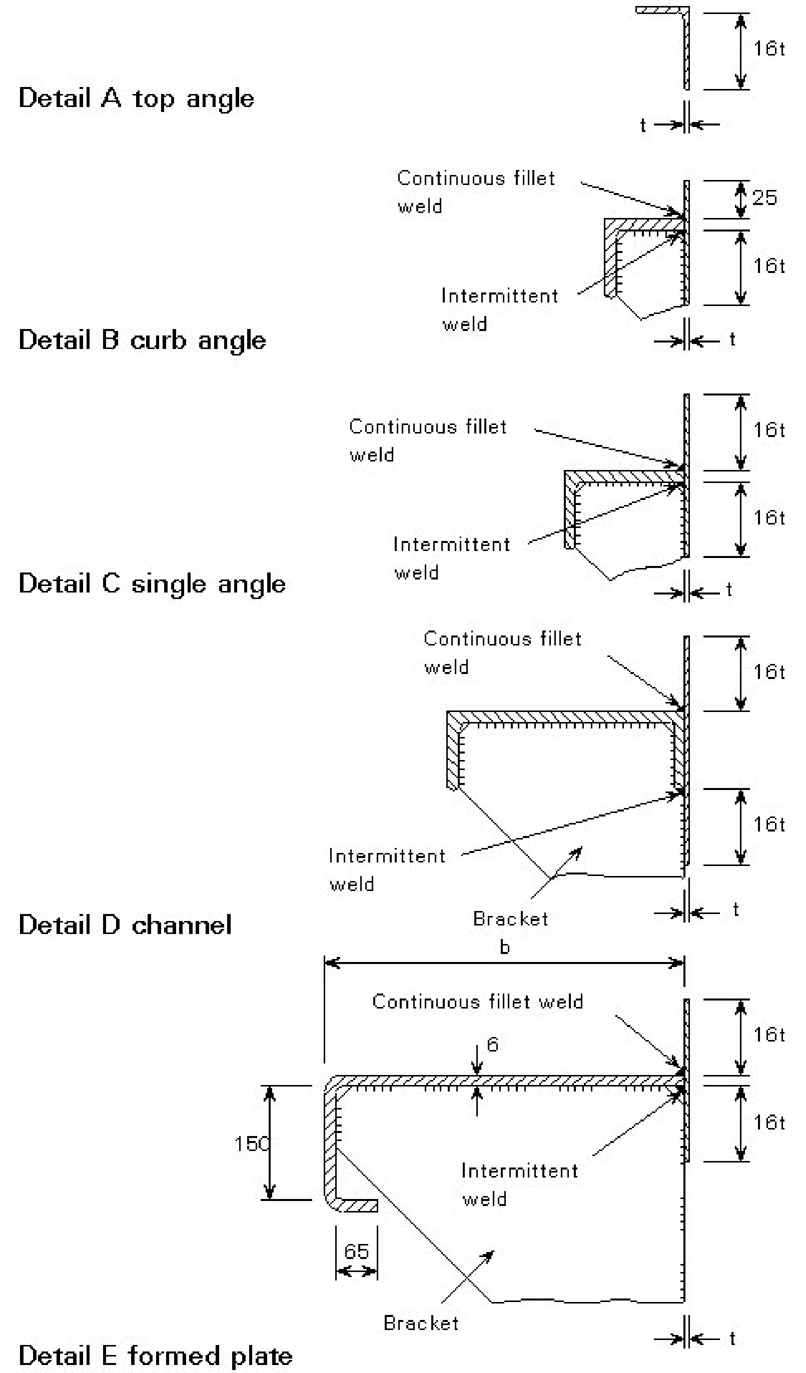
Wind girders are usually formed by welding or bolting an angle or a channel around the top edge of the shell. Note that continuous fillet welds should always be used on the upper edge of the connection, to avoid a corrosion trap.

It is recognised that application of the **De Wit's formula** to tanks over 60 m diameter leads to unnecessarily large wind girders, which is why the size can be limited to that needed for a 60 m tank.

Primary wind girders are normally **external** to the tank. Settlement tanks usually require a **gutter** around the inside edge of the tank, into which the water spills and passes to the outlet. A suitable gutter detail can participate as a primary wind girder, provided it is relatively close to the top of the tank.

Although the primary wind girder or the roof will stabilise the tank over its full height, **local buckling** can occur in empty tall tanks between the top of the tank and its base. To prevent this local buckling, **secondary wind girders** are introduced at intervals in the height of the tank. The determination of the number and position of these secondary wind girders is dealt with in codes, which set out procedures based on the determination of the length of tube for which, with the ends held circular, the elastic critical buckling will occur at a given uniform external pressure.

Main source ESDEP



Wind girders

Stiffening rings according to EC3

Fixed roof tanks with roof structures may be considered to be adequately stiffened at the top of the shell by the roof structure. **A primary ring needs not be used**.

Open top tanks should be provided with a primary ring which is located at or near the top of the uppermost course.

When stiffening rings are located **more than 600mm** below the top of the shell, the tank should be provided with a top curb angle with the following size:

60×60×5 where the top shell course has a thickness less than 6mm;

80×80×6 where the top shell course has a thickness of 6mm or more.

For either angle section, the horizontal leg should be not further than 25mm from the top edge of the shell.

The requirement for a **secondary ring** to prevent local buckling of the shell should be investigated using the following procedure. The height H_E over which buckling of the unstiffened shell can occur (measured from the top of the shell or the primary wind girder downwards) should be taken from:

$$H_E = \sum h \left(\frac{t_{min}}{t}\right)^{2,5}$$

where:

h is the height of each course in turn below the edge ring or the primary wind girder;

t is the thickness of each course in turn;

t_{min} is the thickness of the thinnest course.

The height that may be taken to be stable without a secondary ring should be taken from:

$$H_P = 0{,}46\left(\frac{E}{p_d}\right)\left(\frac{t_{min}}{r}\right)^{2{,}5} r\,K$$

in which:

$K = 1$ if the axial stress $\sigma_{x,Ed}$ is **tensile**

$$K = \left\{1-\left[2{,}67\left(\frac{\sigma_{x,Ed}}{E}\right)\left(\frac{r}{t}\right)\left(1+\frac{1}{54}\left(\frac{r}{t}\right)^{0{,}72}\right)^{1{,}25}\right]^{0{,}8}\right\}$$

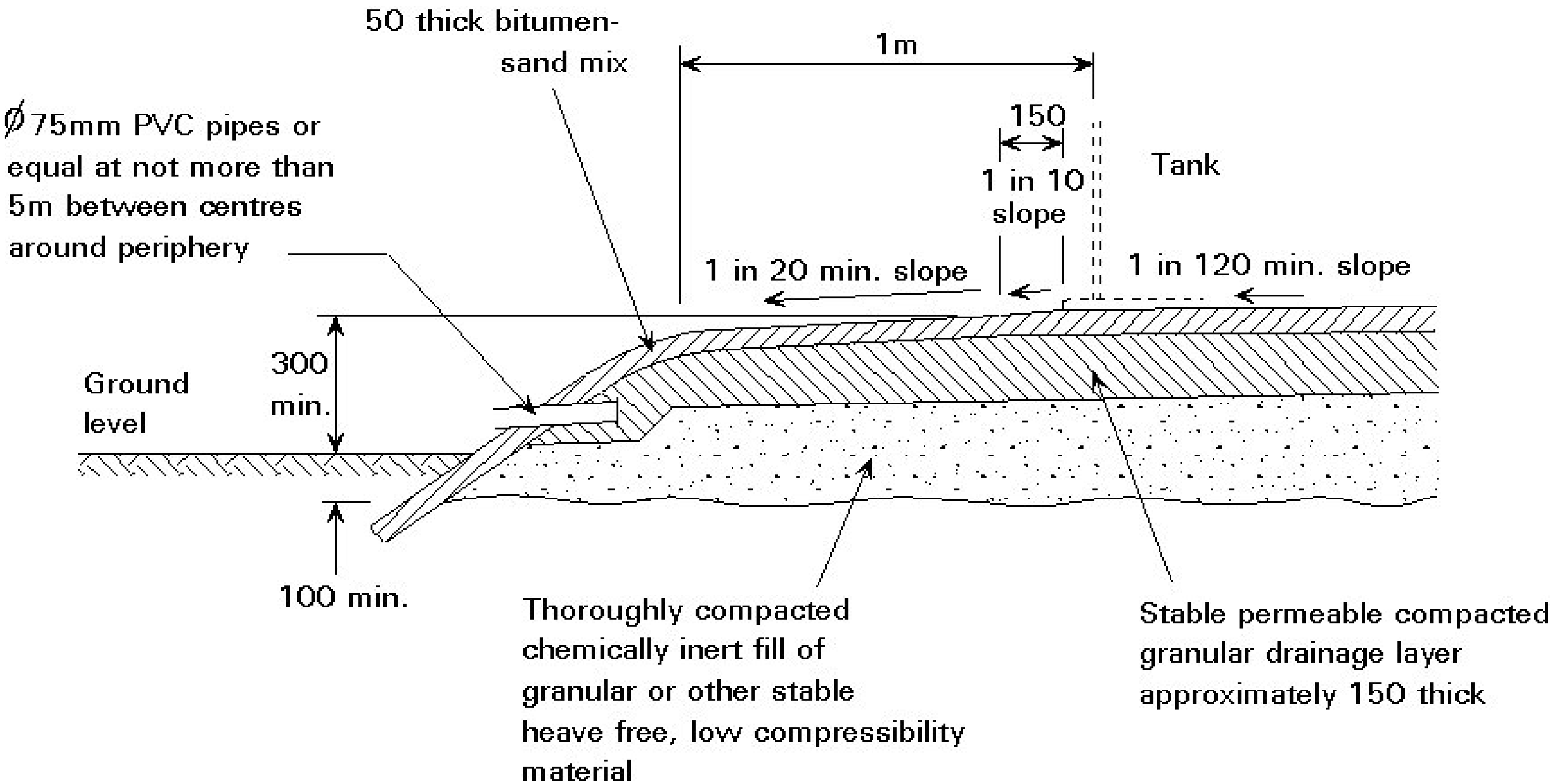
if the axial stress $\sigma_{x,Ed}$ is **compressive**, where p_d is the **maximum design value** of the **inward component** of the pressure on the shell wall (pressure on the outside, negative pressure on the inside) and (r/t) is taken at the same location as the design value $\sigma_{x,Ed}$ of the compressive axial membrane stress.

If $H_E \leq H_P$, a secondary ring need not be used.

If $H_E > H_P$, the height H_E should be subdivided by stiffening rings equally spaced at separations H_P or less to prevent buckling of the shell wall. If more than one stiffening ring is necessary, the value of K may be calculated separately for each bay between stiffening rings, to give different distances H_P between stiffening rings as above.

BOTTOM DESIGN

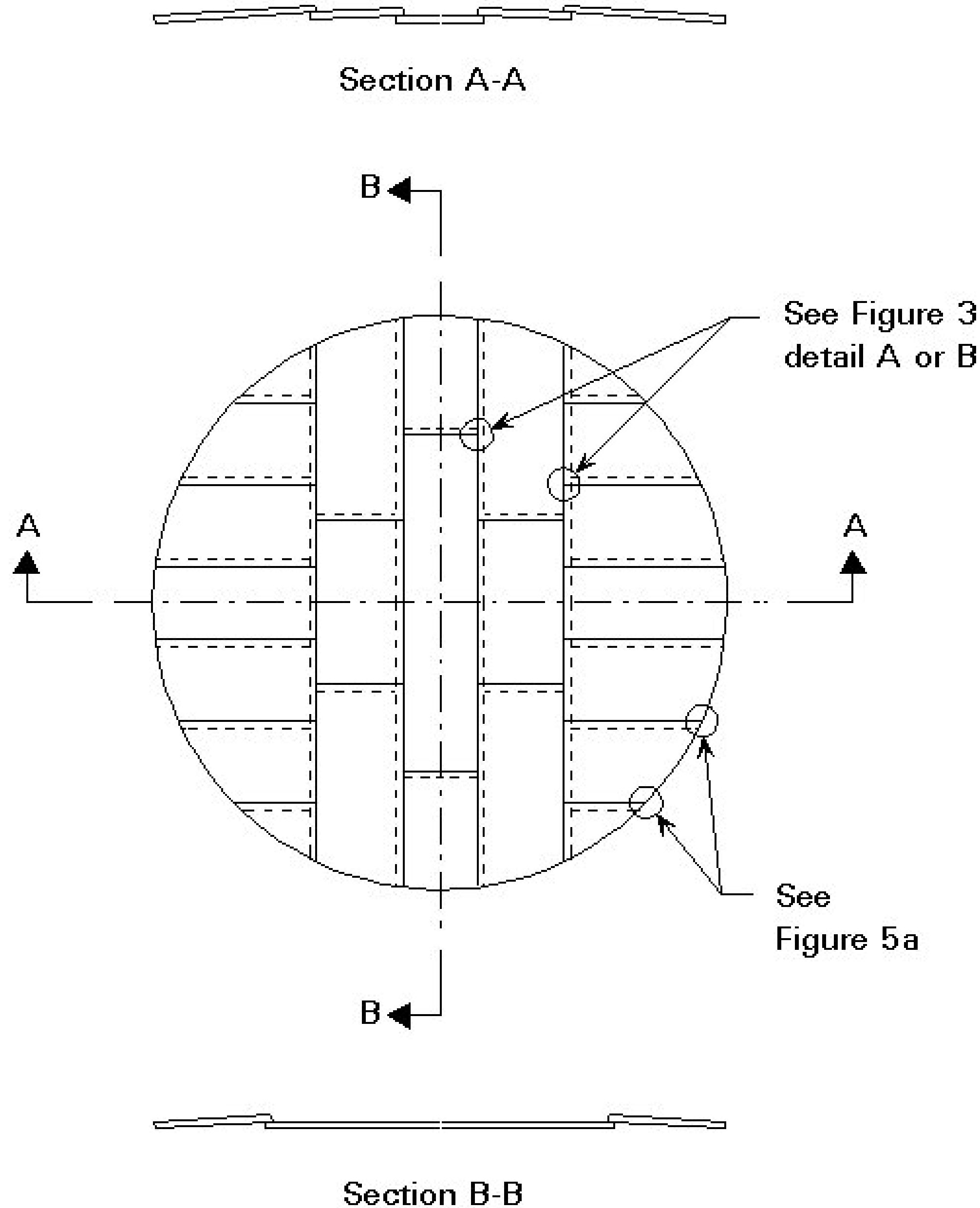
For petroleum storage tanks, steel bottom plates are specified, laid and fully supported on a prepared foundation. The steel plates are directly supported on a bitumen-sand layer on top of a foundation, usually of compacted fill or, if the subsoil is weak, possibly a reinforced concrete raft.



Typical tank foundation

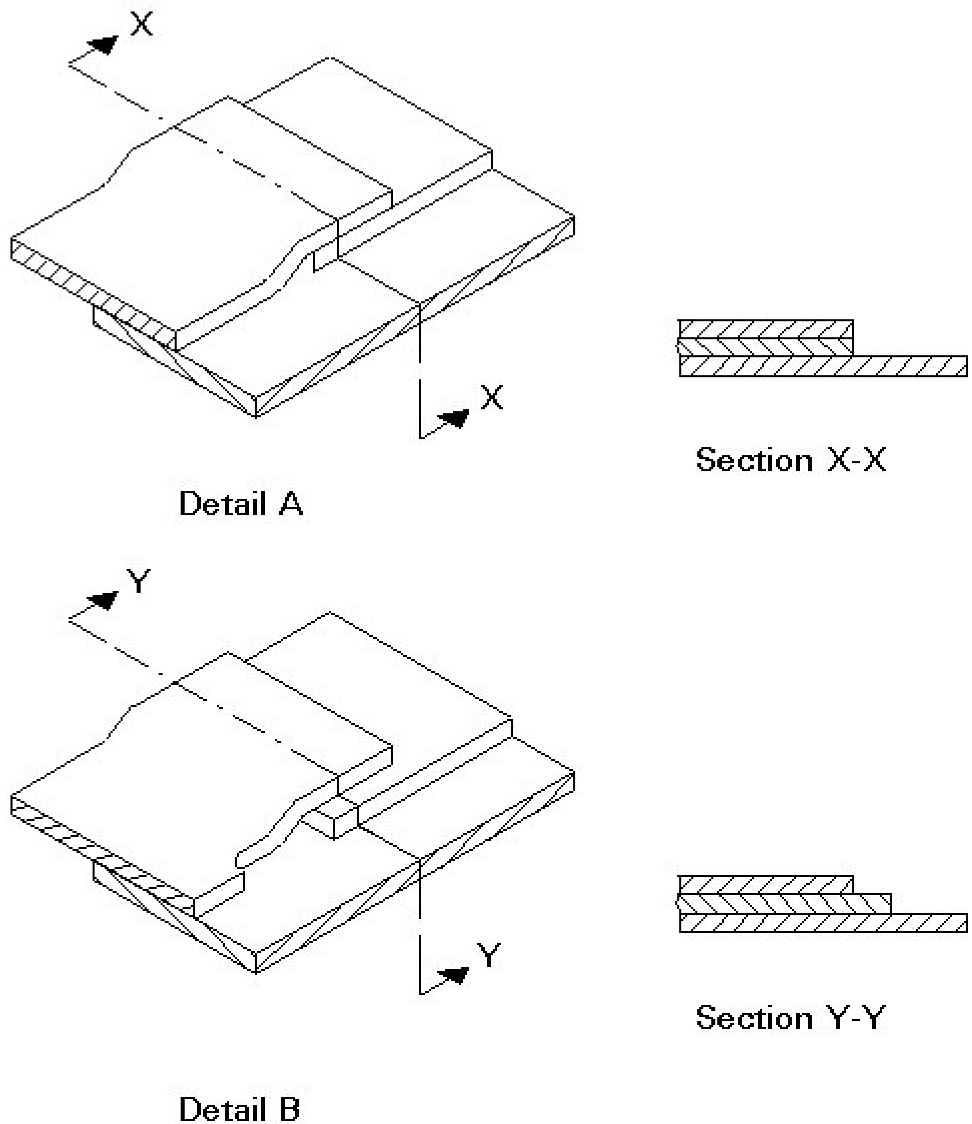
Main source ESDEP

The bottom is made up of a number of rectangular plates, surrounded by a set of shaped plates, called **sketch plates**, to give a circular shape, as shown in Figure. The plates slightly overlap each other and are pressed locally at the corners where three plates meet (see next slide). As an alternative to bolted joints, lapped and fillet welded joints are preferred to butt welded joints (which must be welded onto a backing strip below the joint) because they are easier and cheaper to make.



Typical bottom layout for tanks up to 12 m diameter

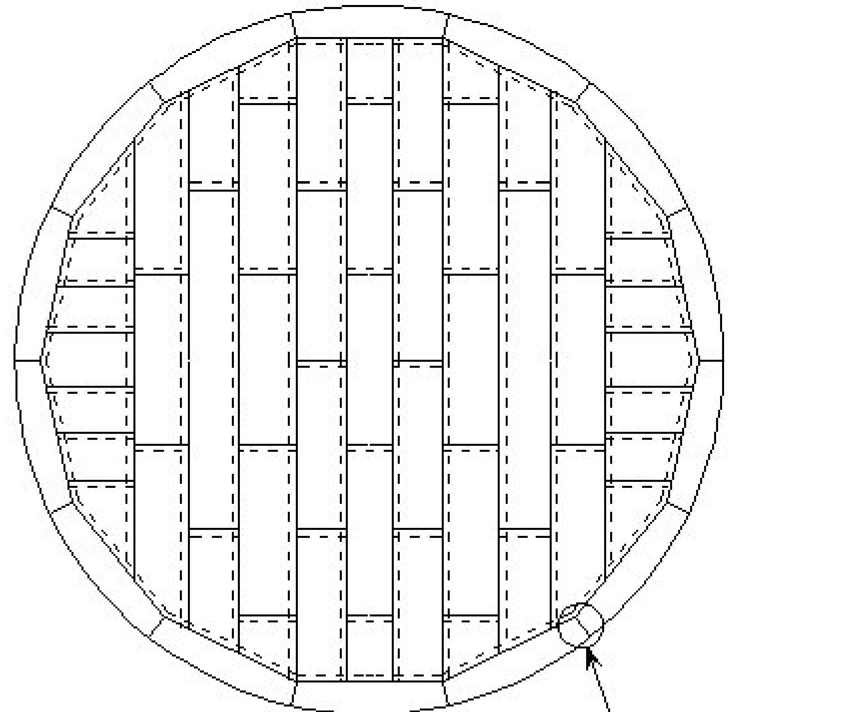
Main source ESDEP



Detail of cross joints in bottom plates

Main source ESDEP

For larger tanks (over 12 m diameter) a ring of **annular plates** is provided around the group of rectangular plates. The radial joints between the annular plates are **butt welded**, rather than lapped, because of the ring stiffening which the plates provide to the bottom of the shell.



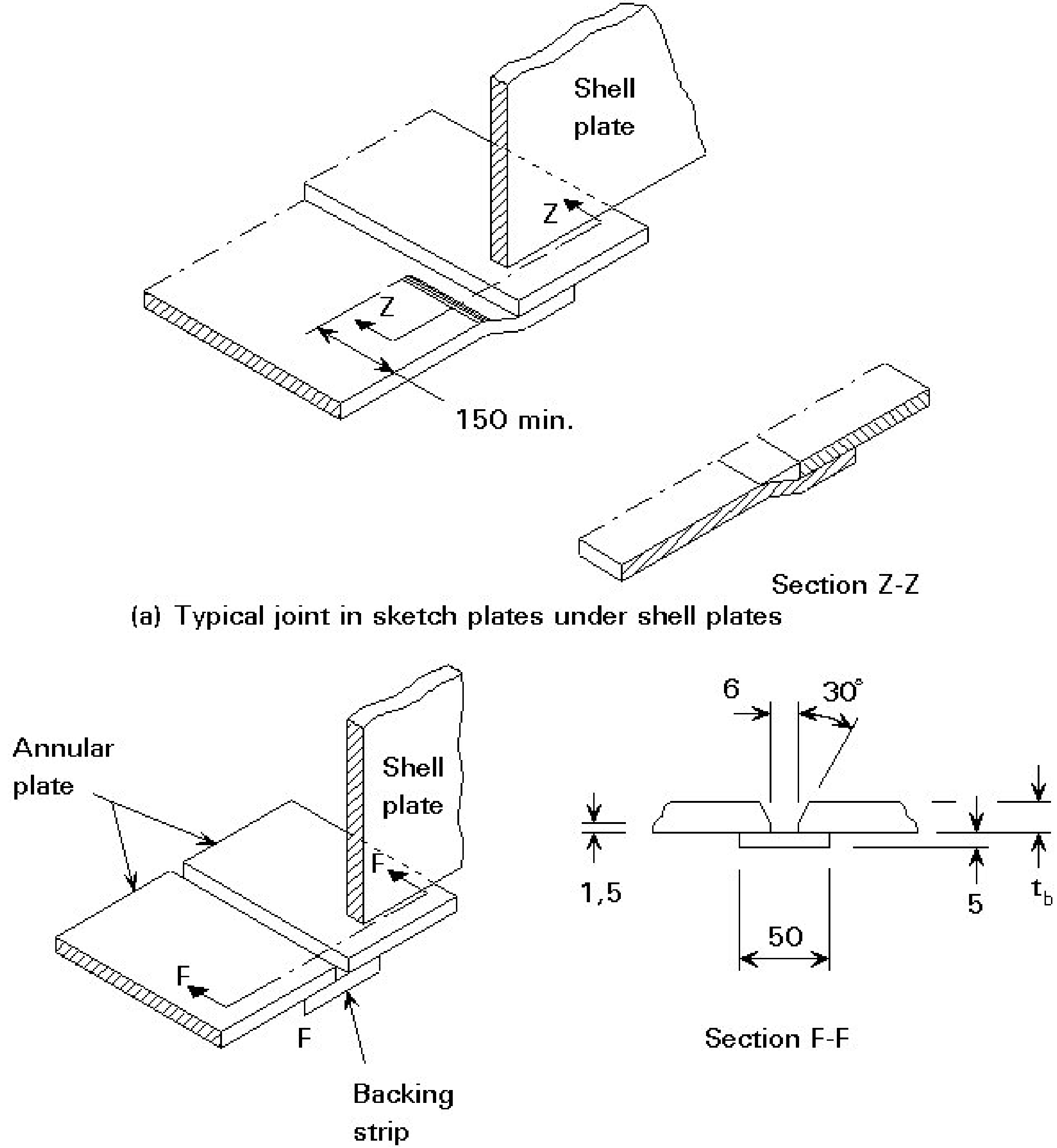
Typical bottom layout for tanks over12 m diameter

Main source ESDEP

The shell sits on the sketch or annular plates, just inside the perimeter and is fillet welded to them (see Figure).

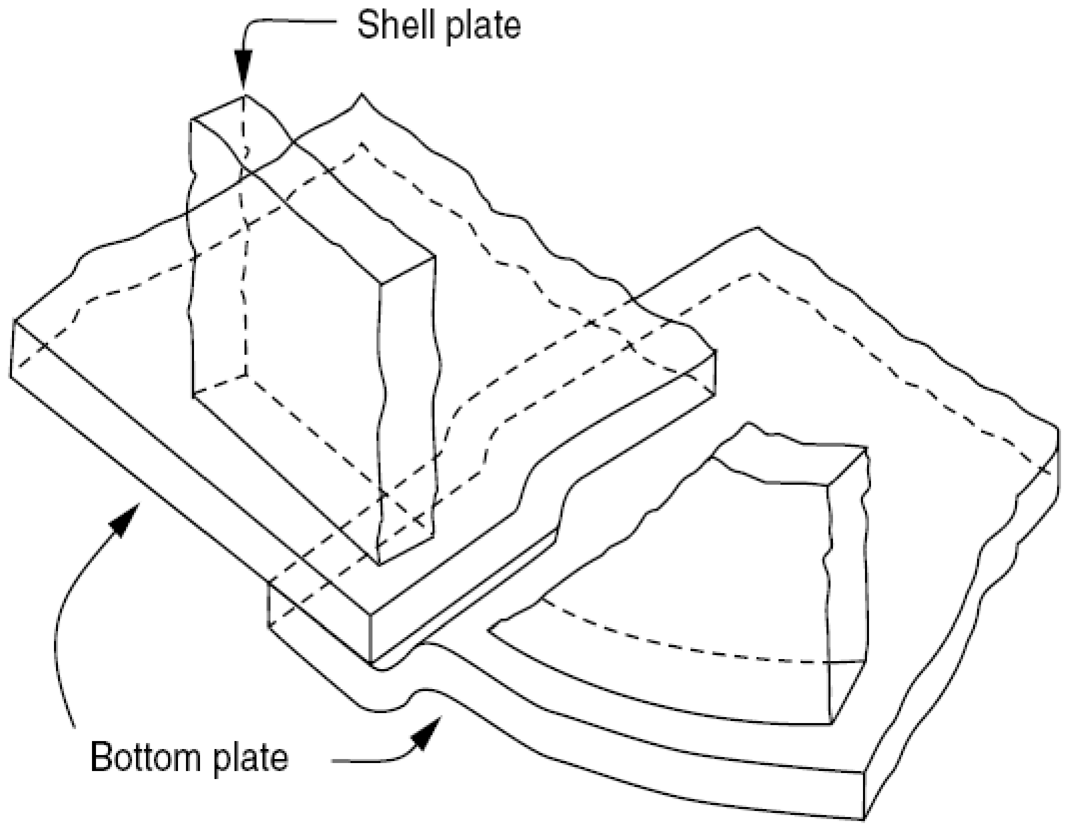
The bottom plates act principally as a seal to the tank. The only load they carry, apart from local stiffening to the bottom of the shell, is the pressure from the contents, which is then transmitted directly to the base. Stress calculations are not normally required for the base, though codes set out minimum thicknesses of plate depending on the size of the tank.

Water tanks may also have a steel bottom. In some circumstances a reinforced concrete slab is specified instead. There are no standard details for the connection between a shell and a concrete slab, though a simple arrangement of an angle welded to the bottom edge of the shell and bolted to the slab will usually suffice.



Details of welded joints in bottom plates below shell plates

Main source ESDEP



Shell to bottom plate connection as
suggested by API 650

Bottom design according to EC3

The design of the bottom plate shall take corrosion into account.

Bottom plates should be lap welded or butt welded. For welding details see EN265001.

The **specified thickness** of the bottom plates should not be less than specified in table below excluding corrosion allowance. Larger values should be used if required to resist uplift due to the internal negative pressure, unless a minimum guaranteed residual liquid level is used to assist in resisting this uplift.

Bottom plates supported by **parallel girders** (elevated bottoms) may be designed as continuous beams according to small deflection theory. If the deformation of the cross section of the supporting girders due to the lateral load is negligible (e.g. concrete beams, hollow sections, beams with heavy flanges), the span of the continuous beam representing the plate may be taken as the **distance between adjacent edges** of these supporting members, instead of the distance between the centre-lines of the supporting members.

Bottoms for tanks greater than 12,5m diameter, should have a **ring of annular plates** that satisfies the strength and toughness requirements on the shell course to which they are attached. The bottom ring should have a minimum nominal thickness t_a excluding corrosion allowance obtained from $t_a = t_s/3 + 3\text{mm}$, but not less than 6mm, where t_s is the thickness of the attached shell course.

Minimum nominal bottom plate thickness

Material	Lap welded bottoms	Butt welded bottoms
Carbon steels	6 mm	5 mm
Stainless steels	5 mm	3 mm

The bottom ring should not have an **exposed width** w less than the limiting value w_a, obtained from:

$$w_a = 1{,}5 \left[\frac{f_y t_a^2}{\rho g H} \right]^{1/2}$$ but not less than 500mm

where:

H is the maximum design liquid height in metres;

w_a is the minimum exposed width (distance from the edge of the attached bottom plate to the inner edge of the attached shell plate) in millimetres.

t_a is the thickness of the attached shell course, taking account of the corrosion allowance, in millimetres.

The radial seams connecting annular plates to each other should be **full penetration butt welded**. For welding details, see EN 265001.

The distance from the outer edge of the shell plate to the outer edge of the bottom plates or annular plates should not be less than 50mm.

The attachment of the lowest course of the shell plate to the annular plates or bottom sketch plates should be continuous fillet welds on both sides of the shell plate.

The throat thickness for each fillet weld should be greater than or equal to the thickness of the annular plate or of the sketch plate, except that they should not exceed 10mm and where the shell plate thickness is less than the sketch plate or annular plate thickness, they should not exceed the appropriate value given in the code.

Anchorage design according to EC3

Tank anchorage should be provided for fixed roof tanks, if any of the following conditions can cause the cylindrical shell wall and the bottom plate close to it to lift off its foundations:

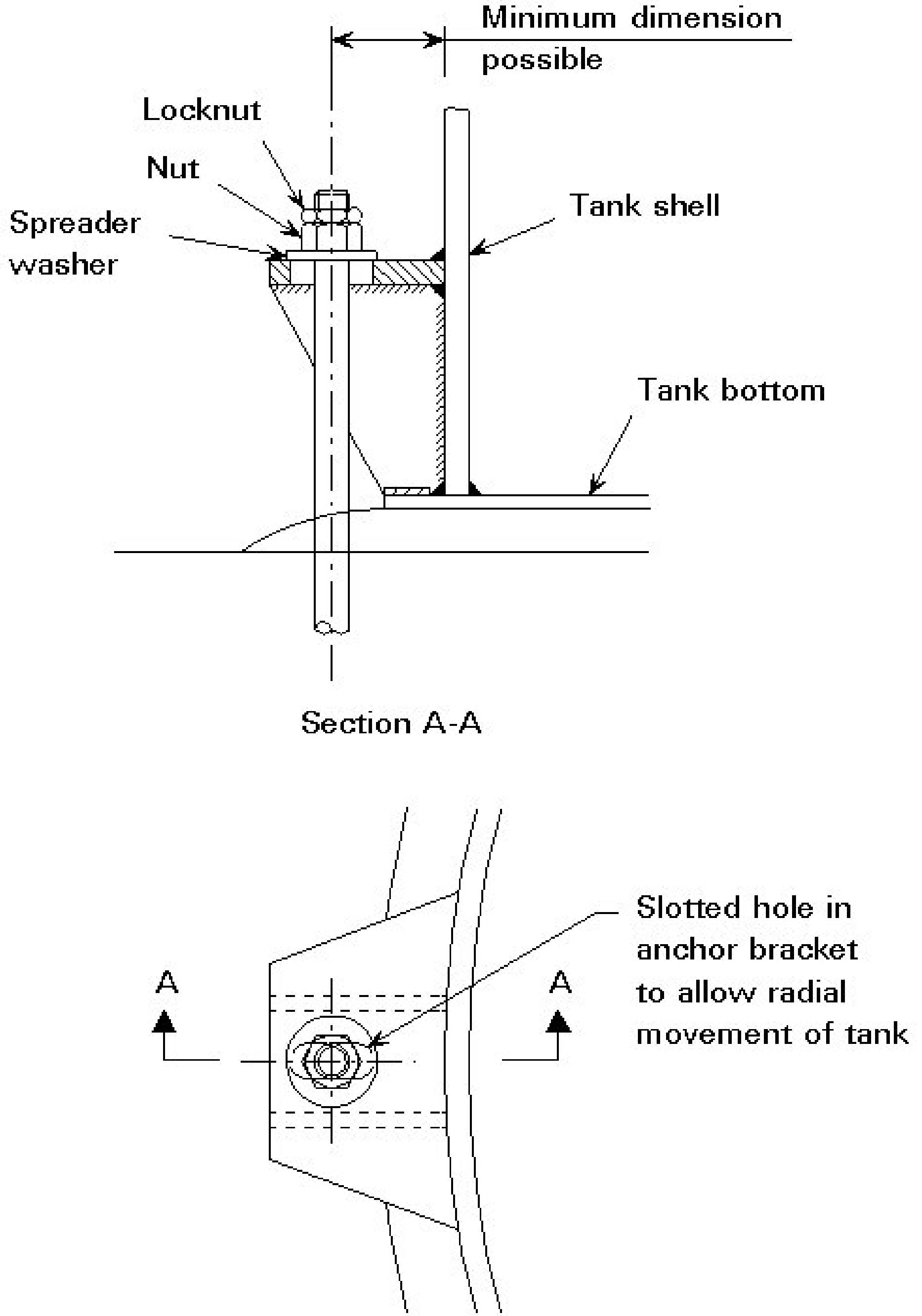
- Uplift of an empty tank due to internal design pressure counteracted by the effective corroded weight of roof, shell and permanent attachments;
- Uplift due to internal design pressure in combination with wind loading counteracted by the effective corroded weight of roof, shell and permanent attachments plus the effective weight of the product always present in the tank as agreed between the designer, the client and the relevant authority.
- Uplift of an empty tank due to wind loading counteracted by the effective corroded weight of roof, shell and permanent attachments;
- Uplift of an empty tank due to external liquid caused by flooding. In such cases it is necessary to consider the effects upon the tank bottom, tank shell etc. as well as the anchorage design.

For this check, the uplift forces due to the wind load may be calculated using the assumption that the tank shell has a rigid cross section (beam theory). This assumption implies that local uplift can occur. In cases where no local uplift can be allowed, a more sophisticated analysis is required.

Anchorage points should be spaced evenly around the circumference of the tank, insofar as this is possible.

The design of the holding down bolts or straps should meet the requirements of EN1993-1-1. The minimum cross-sectional area for the holding down bolts or straps should be $500 mm^2$. If corrosion is anticipated, a minimum corrosion allowance of 1mm should be added.

The anchorage shall be principally attached to the shell wall. It shall not be attached to the bottom plate alone.
The design of the anchorage shall accommodate movements of the tank due to thermal changes and hydrostatic pressure and minimise any stresses induced in the shell.
The design of the shell for local anchorage forces and bending moments resulting from the anchorage should meet the requirements of EN1993-4-1.
No initial tension should be applied to the holding down bolt or strap, to ensure that will become effective only if an uplift force develops in the shell of the tank.

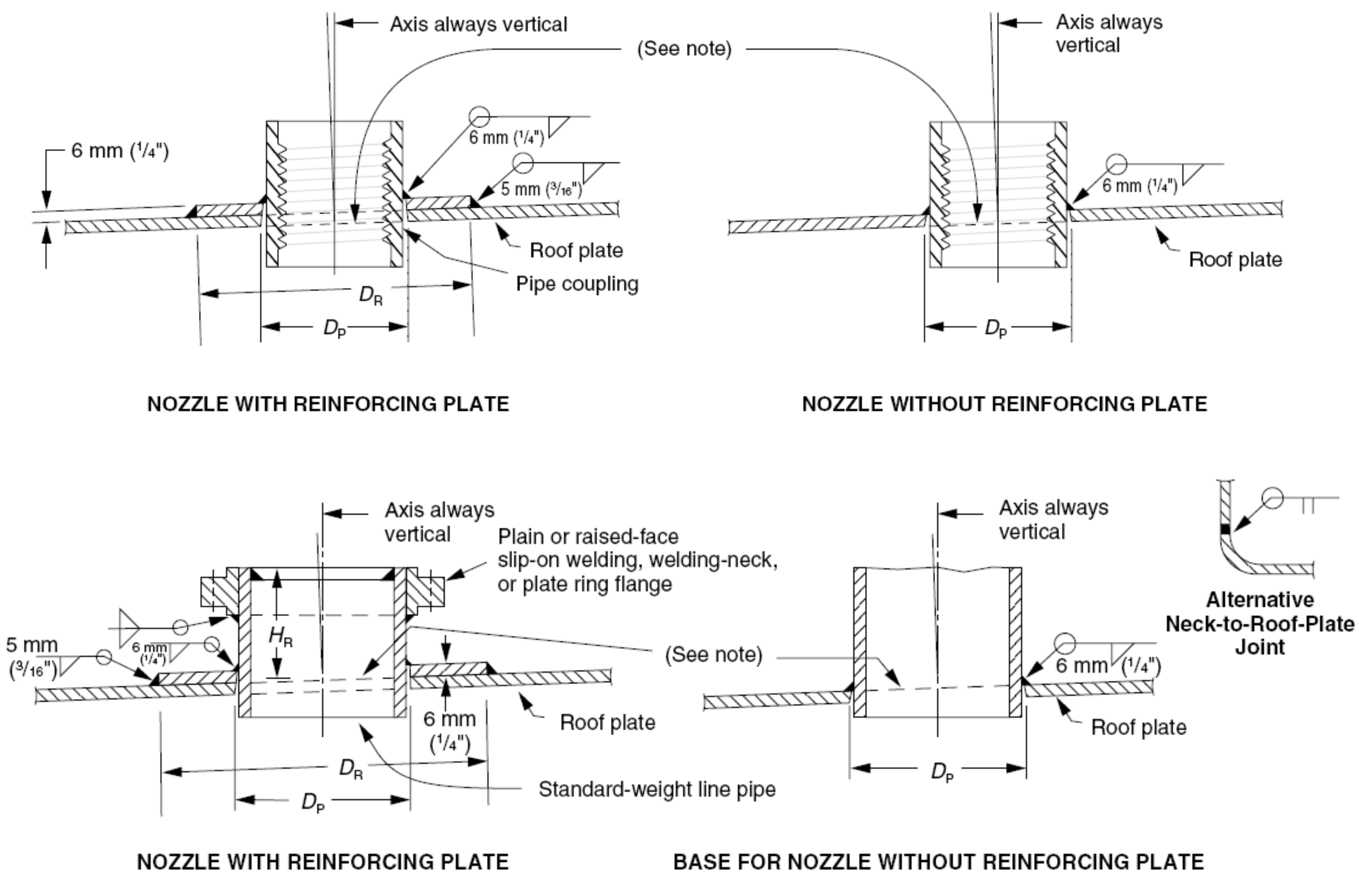


Typical tank anchorage detail

Main source ESDEP

Venting

Venting has to be provided to allow for movement of the contents into and out of the tank and for temperature change of the air in the tank. Venting can be provided by pressure relief valves or by open vents. For storage of petroleum products, emergency pressure relief has to be provided to cater for heating due to an external fire. Pressure relief can be achieved either by additional emergency venting or by designing the roof to shell joint as frangible (this means, principally, that the size of the fillet weld between the roof and the shell is limited in size - a limit of 5 mm is typical).



Note: When the roof nozzle is used for venting, the neck shall be trimmed flush with the roofline.

DESIGN OF FLOATING ROOFS AND COVERS

Use of Floating Roofs and Covers

As mentioned above, tanks need to be **vented** to allow for the expansion and contraction of the air. In petroleum tanks, the free space above the contents contains an **air/vapour mixture**. When the mixture expands in the heat of the day, venting expels some of this vapour. At night, when the temperature drops, fresh air is drawn in and more of the contents evaporates to saturate the air. The continued breathing can result in substantial evaporation losses. Measures are needed to minimise these losses; floating roofs and covers are commonly used for this purpose.

Floating Roofs

A floating roof is sometimes provided instead of a fixed roof. The shell is then effectively open at the top and is designed accordingly.

During service, a floating roof is completely supported on the liquid and must therefore be sufficiently buoyant; buoyancy is achieved by providing liquid-tight compartments in one of two forms of roof - pontoon type and double deck type.

A pontoon roof has an annular compartment, divided by bulkheads, and a central single skin diaphragm. The central diaphragm may need to be stiffened by radial beams.

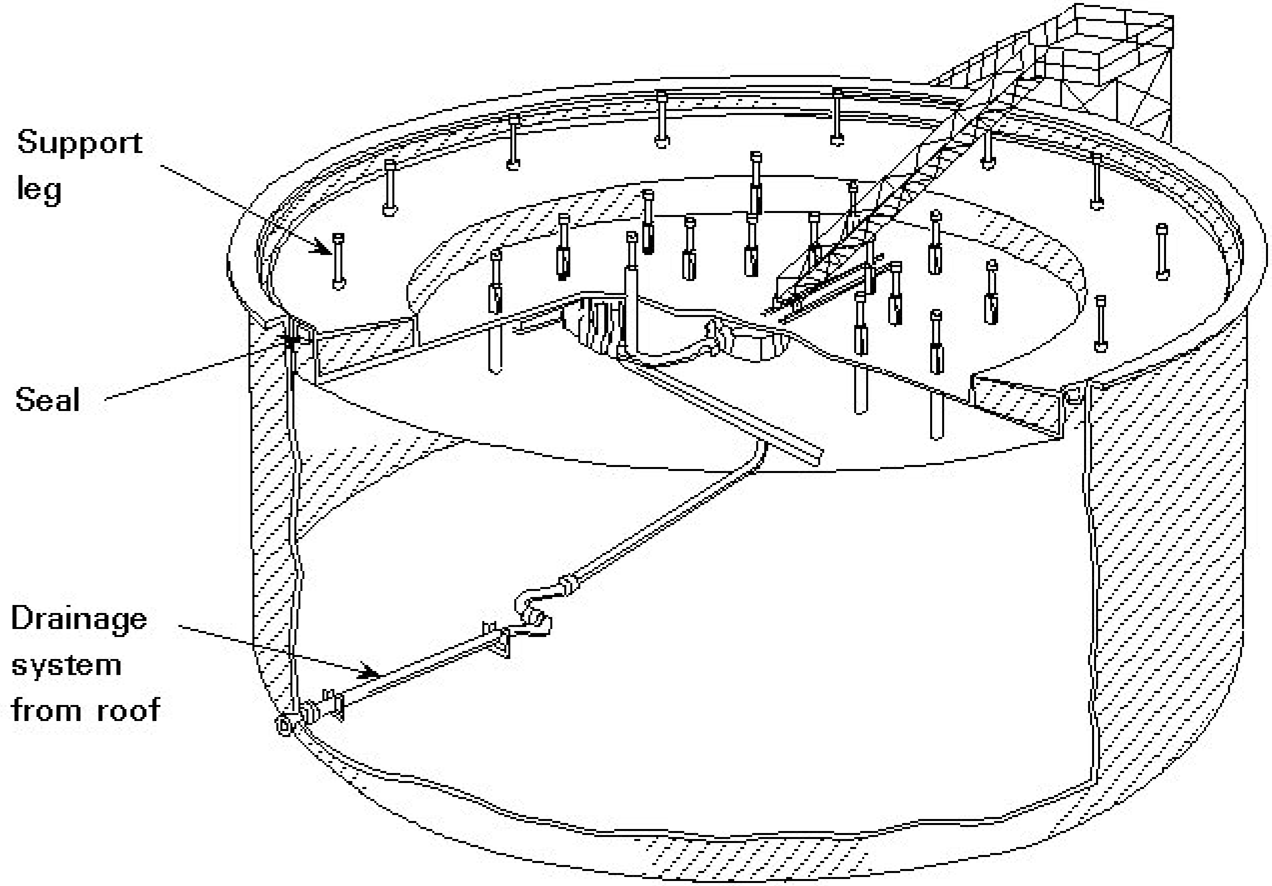
A double deck roof is effectively a complete set of compartments over the whole diameter of the tank; two circular skins are joined to circumferential plates and bulkheads to form a disk or piston.

Both types of roof must remain buoyant even if some compartments are punctured (typically two compartments). The central deck of a pontoon roof should also be presumed to be punctured for this design condition.

Because the roof is open to the environment, it catches rain, which must be drained off. Drainage is achieved by a system on the roof which connects to flexible pipework inside the tank and thence through the shell or bottom plates to a discharge. The design is required to ensure that the roof continues to float in the event of a block in the drainage system which results in a surcharge of water on the roof (usually 250 mm of water).

When the tank is emptied, the roof cannot normally be allowed to fall to the bottom of the tank, because there is internal pipework; the roof is therefore fitted with legs which keep it clear of the bottom. At this stage the roof must be able to carry a superimposed load (1,2 kN/m^2) plus any accumulated rainwater.

For **maintenance** of the **drainage** system and for access to **nozzles** through the roof for various purposes, maintenance personnel need **access** from the top of the shell to the roof whatever the level of contents in the tank. Access is usually achieved by a movable ladder or **stairway**, pinned to the shell and resting on the roof. For maintenance of the tank when it is empty, an access **manhole** must be provided through the roof.



Typical arrangement of a pontoon type roof

Main source ESDEP

Floating Covers

Where a cover to the contents is provided **inside a fixed roof tank**, to reduce evaporation or ingress of contaminants (e.g. water or sand), a much lighter cover or screen can be provided. Such a cover is likely to be manufactured from lighter materials than steel, though a shallow steel pan can sometimes be provided. The cover does not need to be provided with access ladders, nor to be designed for surcharge. It does have to be designed to be supported at low level when the tank is empty and to carry a small live load in that condition.

MANHOLES, NOZZLES AND OPENINGS

Manholes

Access is required inside fixed roof tanks for maintenance and inspection purposes. Such access can be provided through the **roof** or through the **shell** wall. **Manholes** through the roof have the advantage that they are always accessible, even when the tank is full. Access through the shell wall is more convenient for cleaning out (some access holes are D-shaped and flush with the bottom for cleaning out purposes).

A manhole through a roof should be at least 500 mm diameter. Stiffening arrangements around the hole in the roof plate, and the type of cover, depend on the design of the roof. Access to the roof manhole must be provided by ladders, with suitable handrails and walkways on the roof.

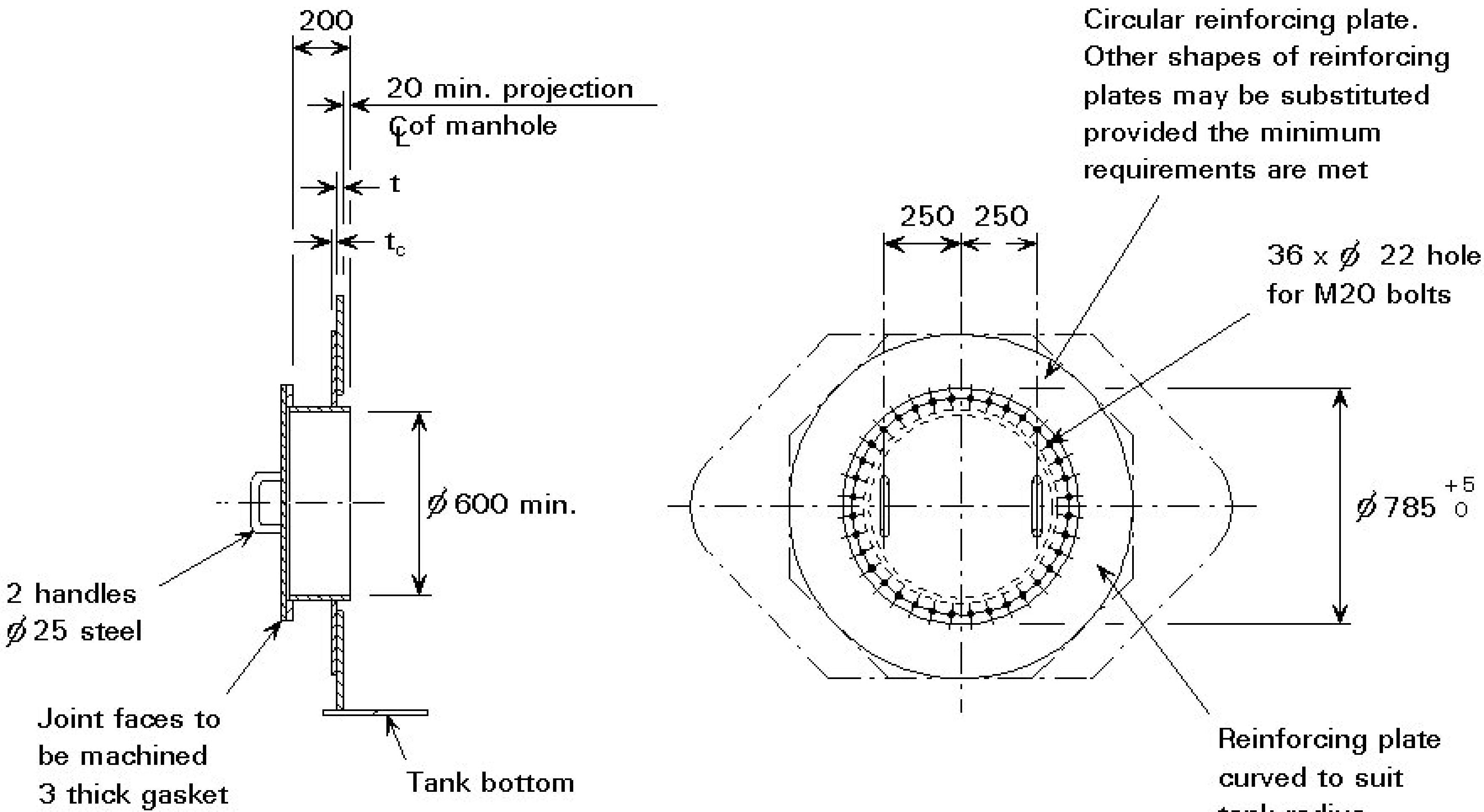
A manhole through the shell wall should be at least 600 mm diameter and is normally positioned just above the bottom of the tank.

Clearly, the cutting of an opening in the shell interferes with the structural action of the shell. The loss of section of shell plate is compensated by providing additional cross-section area equal to 75% of that lost. The area must be provided within a circular region around the hole, though the actual reinforcement should extend beyond that region. Reinforcement can be provided in one of three ways:

(1) a reinforcing plate welded onto the shell plate (similar to the section in next slide)

(2) an insert of thicker plate locally (in which the manhole is cut)

(3) a thicker shell plate than that required for that course of the shell



Typical manhole detail

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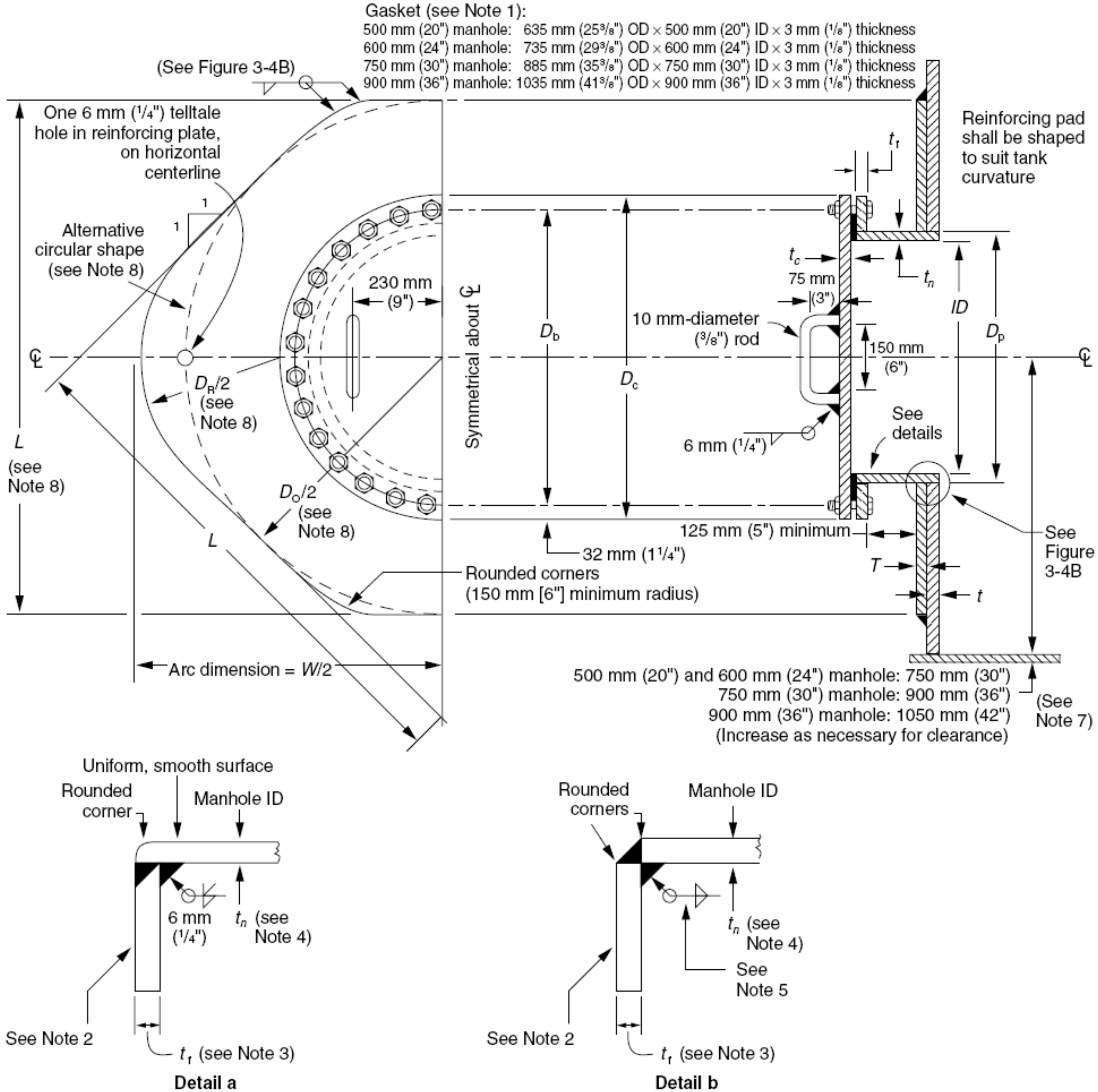
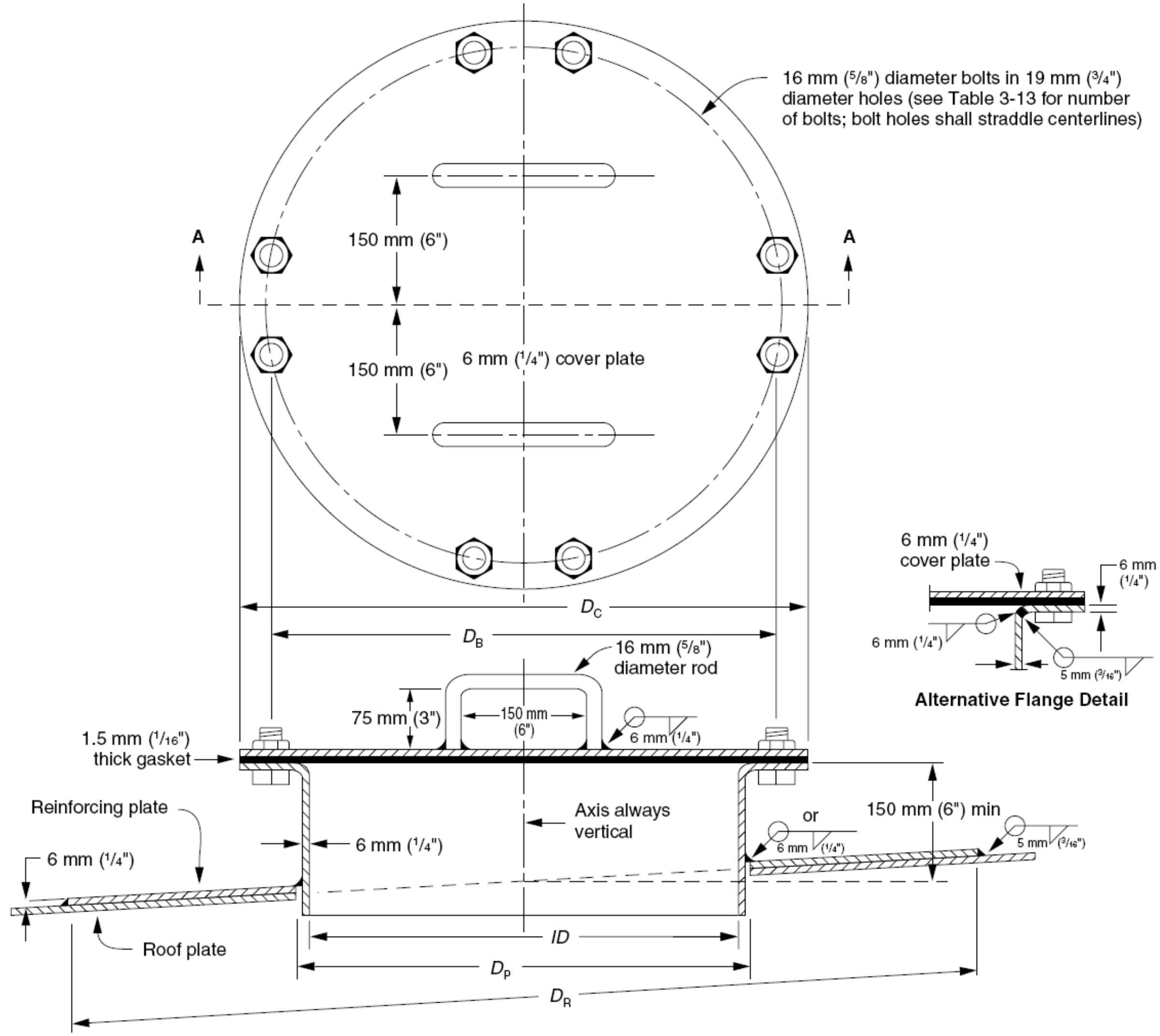
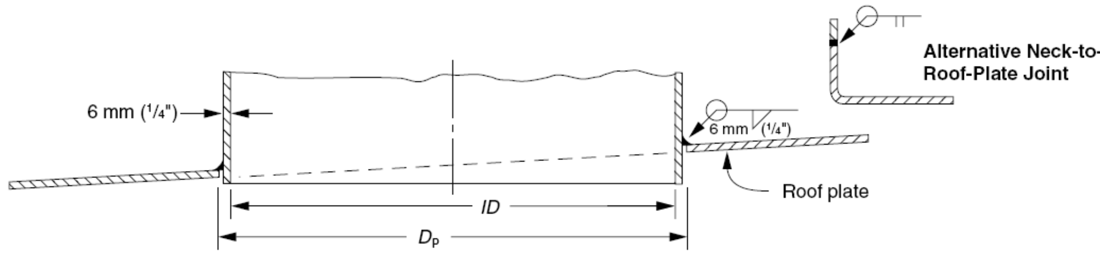

Gasket (see Note 1):
500 mm (20") manhole: 635 mm (25³/₈") OD × 500 mm (20") ID × 3 mm (¹/₈") thickness
600 mm (24") manhole: 735 mm (29³/₈") OD × 600 mm (24") ID × 3 mm (¹/₈") thickness
750 mm (30") manhole: 885 mm (35³/₈") OD × 750 mm (30") ID × 3 mm (¹/₈") thickness
900 mm (36") manhole: 1035 mm (41³/₈") OD × 900 mm (36") ID × 3 mm (¹/₈") thickness
(See Figure 3-4B)
One 6 mm (¹/₄") telltale hole in reinforcing plate, on horizontal centerline
Alternative circular shape (see Note 8)
Reinforcing pad shall be shaped to suit tank curvature
t_f
t_c
t_n
75 mm (3")
230 mm (9")
Symmetrical about ℄
D_b
D_c
ID
D_p
10 mm-diameter (³/₈") rod
150 mm (6")
℄
$D_R/2$ (see Note 8)
6 mm (¹/₄")
See details
L (see Note 8)
$D_O/2$ (see Note 8)
L
125 mm (5") minimum
32 mm (1¹/₄")
T
t
See Figure 3-4B
Rounded corners (150 mm [6"] minimum radius)
Arc dimension = $W/2$
500 mm (20") and 600 mm (24") manhole: 750 mm (30")
750 mm (30") manhole: 900 mm (36")
900 mm (36") manhole: 1050 mm (42")
(Increase as necessary for clearance)
(See Note 7)
Uniform, smooth surface
Rounded corner
Manhole ID
6 mm (¹/₄")
t_n (see Note 4)
See Note 2
t_f (see Note 3)
Detail a
Rounded corners
Manhole ID
t_n (see Note 4)
See Note 5
See Note 2
t_f (see Note 3)
Detail b



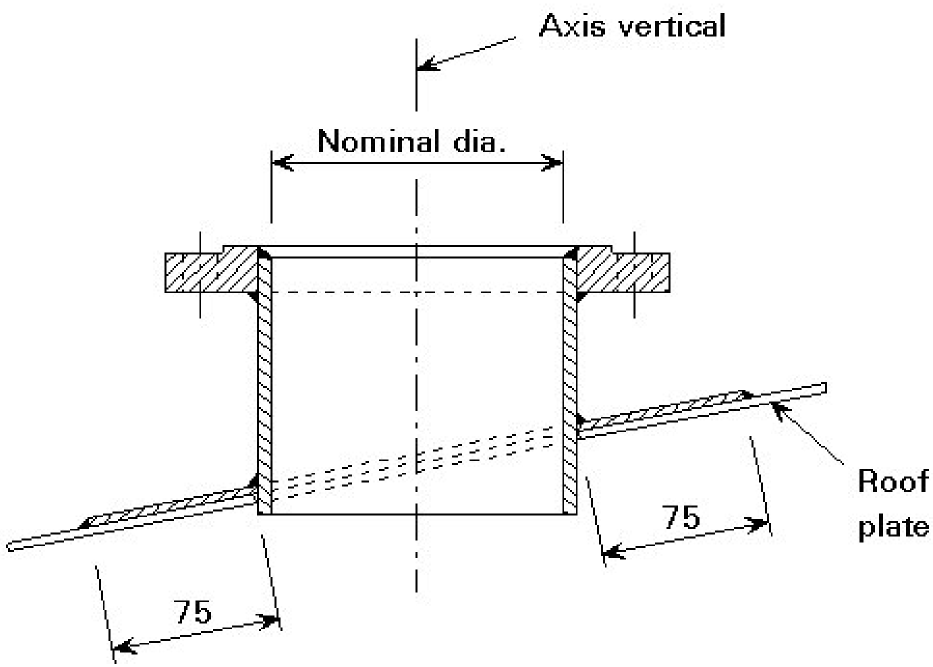
SECTION A-A—ROOF MANHOLE WITH REINFORCING PLATE



BASE FOR ROOF MANHOLE WITHOUT REINFORCING PLATE

Nozzles

As well as manholes for access and cleaning out, **nozzles** are required through the shell roof and bottom for **inlet, outlet, and drainage pipes, and for vents in the roof**. Venting has to be provided to allow for movement of the contents into and out of the tank and for temperature change of the air in the tank. Venting can be provided by **pressure relief valves** or by open vents. For storage of petroleum products, emergency pressure relief has to be provided to cater for heating due to an external fire. Pressure relief can be achieved either by additional emergency venting or by designing the roof to shell joint as frangible (this means, principally, that the size of the fillet weld between the roof and the shell is limited in size - a limit of 5 mm is typical). Nozzles are normally made by **welding a cylindrical section** of plate into a circular hole in the structural plate. For small nozzles, no reinforcement is necessary, the extra material is considered sufficient. Larger holes must be reinforced in the same way as manholes.



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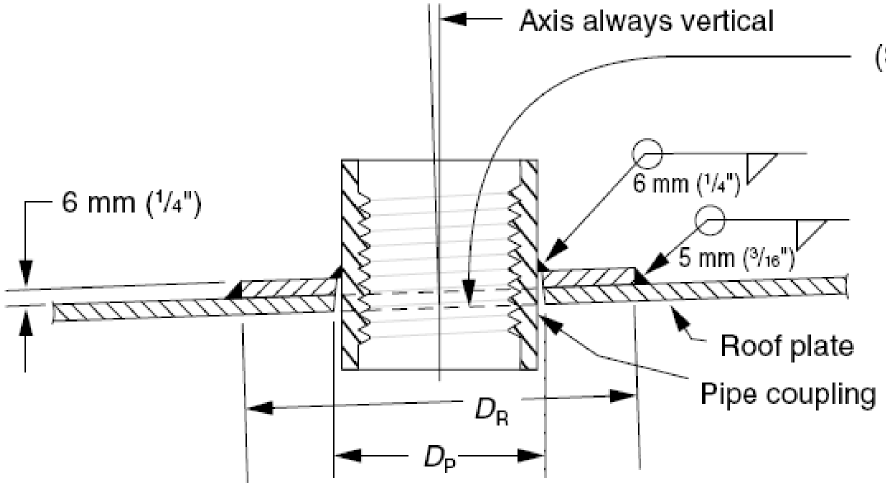
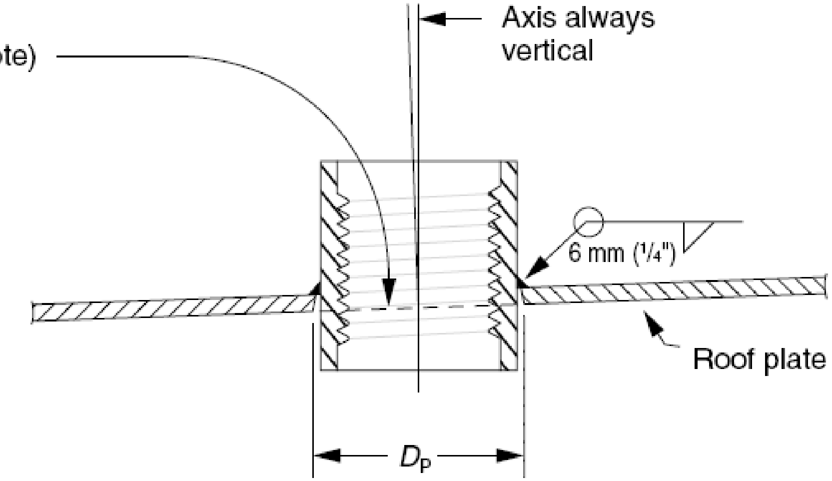
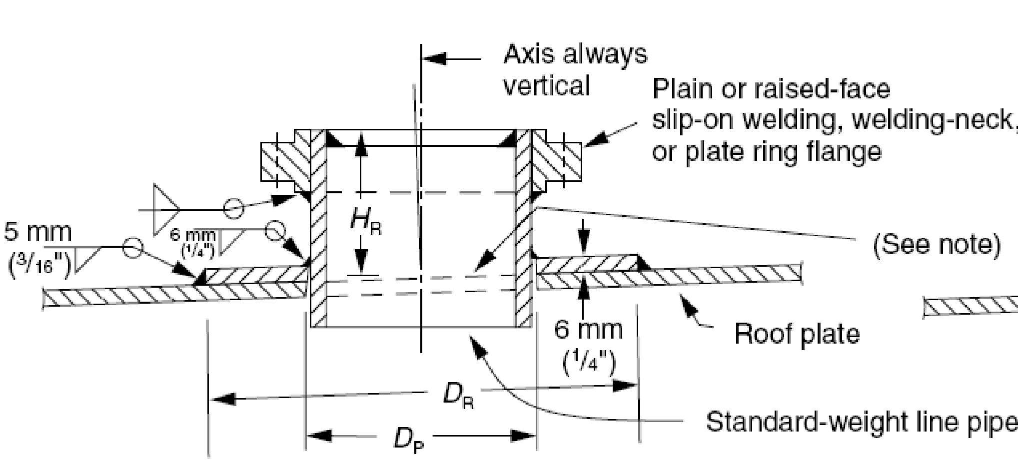


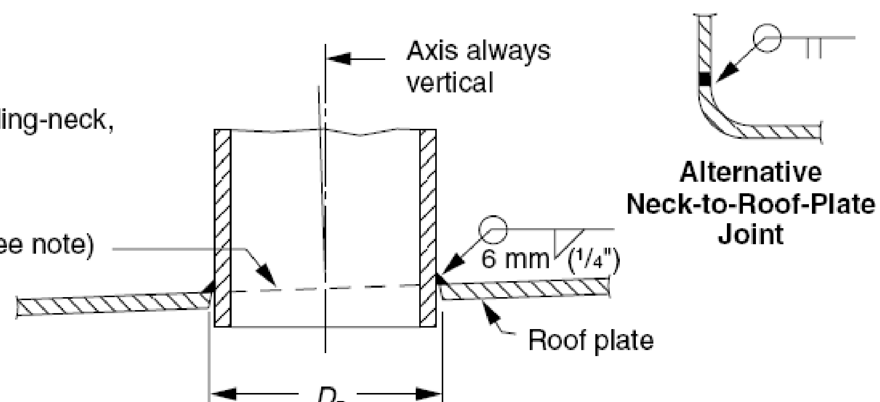
NOZZLE WITH REINFORCING PLATE



NOZZLE WITHOUT REINFORCING PLATE

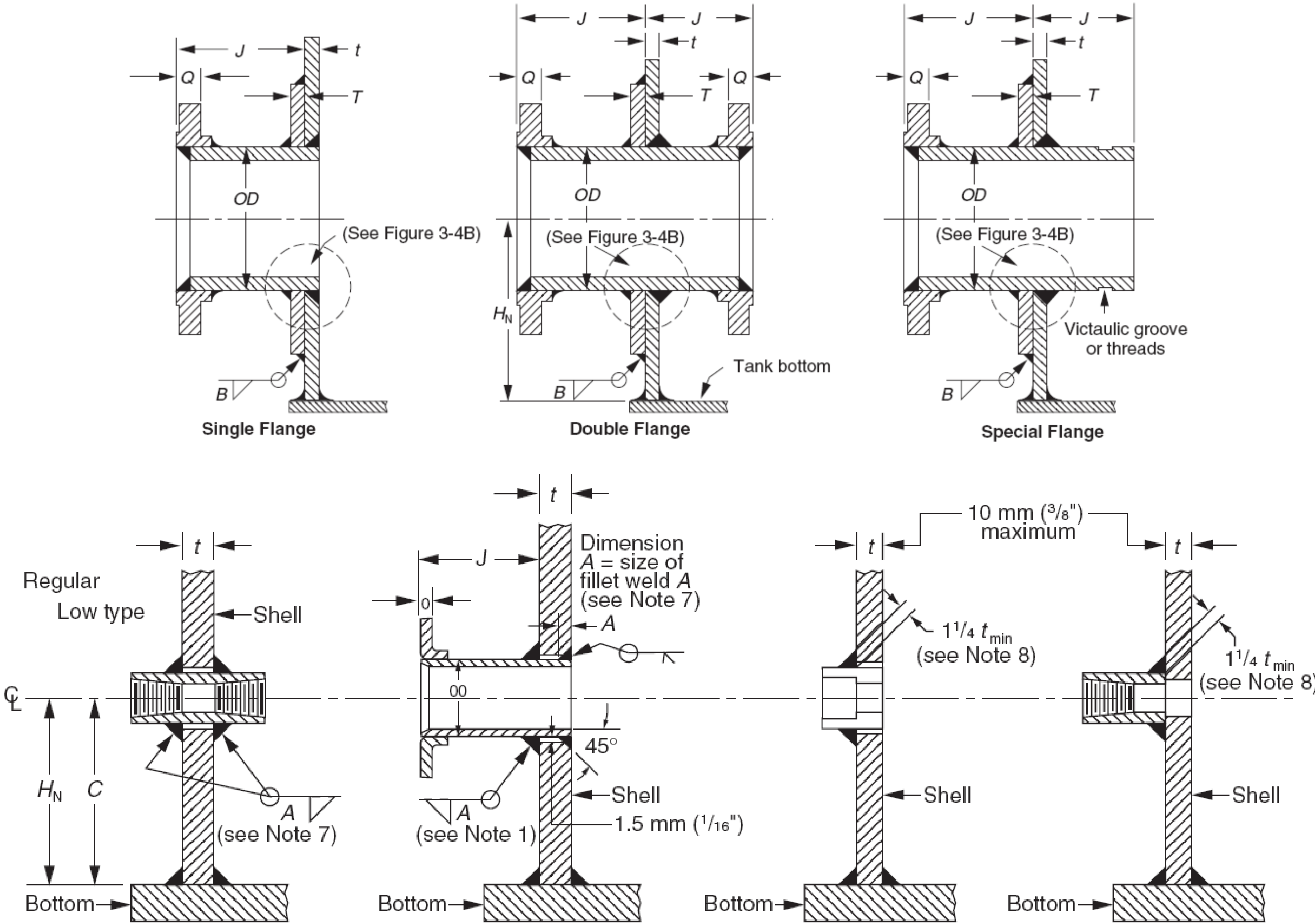


NOZZLE WITH REINFORCING PLATE



BASE FOR NOZZLE WITHOUT REINFORCING PLATE

Note: When the roof nozzle is used for venting, the neck shall be trimmed flush with the roofline.



J
t
Q
T
OD
(See Figure 3-4B)
B
Single Flange
J
J
t
Q
Q
T
OD
(See Figure 3-4B)
HN
B
Tank bottom
Double Flange
J
J
t
Q
T
OD
(See Figure 3-4B)
Victaulic groove
or threads
B
Special Flange
Regular
Low type
t
Shell
CL
HN
C
A
(see Note 7)
Bottom
t
J
0
Dimension
A = size of
fillet weld A
(see Note 7)
A
00
45°
Shell
A
(see Note 1)
1.5 mm (1/16")
Bottom
10 mm (3/8")
maximum
t
1 1/4 tmin
(see Note 8)
Shell
Bottom
t
1 1/4 tmin
(see Note 8)
Shell
Bottom

API 650

TANK F.E.M. DESIGN