CHAPTER eight

Structural Design for Special Applications

INTRODUCTION

Corrugated steel pipe has many uses other than for culvert and storm sewer applications. However, even these conventional applications have a myriad of fittings, steel manholes, etc. that in larger sizes and deeper cover applications need special design and reinforcement.

Nonstandard applications include using corrugated steel pipe as vertical shaft liners, standpipes, grouted in place reline (rehabilitation) structures, above ground aerial spans, and structural columns to name a few. When the pipe is not backfilled or if it is stood on end, structural considerations change. Standpipes, grouted in place pipes, and other applications, have hydrostatic buckling and floatation issues that must be recognized in both design and construction.

FITTINGS REINFORCEMENT

Standard and special fittings can be shop fabricated from corrugated steel pipe. Like the rest of a buried pipeline, the thrust in the fitting depends on its diameter and the loads acting upon it. As the fittings get larger or become more deeply buried, they reach a point where additional steel structural reinforcing members or tensile strips, must be used to reinforce the area where the mainline has been cut away to allow the branch hub to join it.

■ CSP reinforced fitting.

Table 8.1 provides a list of the largest fittings that do not require longitudinal reinforcement while carrying up to 10 feet of cover with H20 and H25 wheel loads. This is a simple check to cover smaller diameter applications. Reinforcement, when necessary, should

Table 8.1

Maximum CSP branch diameters that do not require longitudinal reinforcement under up to 10 ft cover and H20/H25 wheel loads**

2 2/3" x 1/2" CORRUGATIONS															
Wall Thickness	0.064"			0.079"			0.109"								
Maximum Cover	10'	20'	30'	10'	20'	30'	10'	20'	30'						
Main Diameter															
48"	48	36	24	48	42	30	48	48	36						
60"					54*	36*	24*	42	36						
3" x 1" and 5" x 1" CORRUGATIONS															
Wall Thickness	0.064"			0.079"			0.109"			0.138"			0.168"		
Maximum Cover	10'	20'	30'	10'	20'	30'	10'	20'	30'	10'	20'	30'	10'	20'	30'
Main Diameter															
60″	42	24	18	54	30	24		42	30						
72"	36	24	18	48	24	18	54	36	24						
84"	30	18	18	42	24	18	54	30	24		42	30			
96"	30	18	12	36	24	18	48	30	18	54	36	24			
108"				36	18	12	42	24	18	42	30	24			
120"				30	18	12	42	24	18	48	30	24	54	36	24
132"							36	24	18	42	30	18	48	36	24
144"										42	24	18	48	30	24
3/4" x 3/4" x 71/2" SPIRAL RIB PIPE															
Wall Thickness	0.064"			0.079"			0.109"			0.138"					
Maximum Cover	10'	20'	26'	10'	20'	29'	10'	20'	30'	10'	20	' 30'			
Main Diameter															
48"	42	30	24	48	30	24	48	42	30						
60"	36	24		48	30	24	60	36	24						
72"				42	24		54	30	24						
84"							48	30	24	60	42	30			
96"							42	24	18	54	36	30			
108"										48	36	30			
3/4" X 1" x 111/2" SPIRAL RIB PIPE															
Wall Thickness	0.064"			0.079"			0.109"								
Maximum Cover	10'	20'	26'	10'	20'	29'	10'	20'	30'						
Main Diameter															
48"	42	30	24	48	30	24	48	42	30						
60"	36	24		42	30	24	60	36	24						
72"				36	24		48	24	24						
84"				36	24		42	24	18						
96"							42	24	18						
108"							36	24							

Notes:

* 60" 16 gage main diameter not available. Use 54" main diameter.

** Branch diameters listed assume 90 degree tee connections to the mainline. For wyes and other conditions, increase the branch diameter to d/sinØ before entering the table. Ø is the acute angle of the pipe's intersection. d/sinØ is equal to the span of the main cutout.

† Blank entries indicate cases not investigated. For intermediate branch diameters, or intermediate covers, interpolate or select the lower branch diameter. For branch angles other than 90 degrees (but no less than 30 degrees), use the span (major dimension of opening cut in main pipe for branch pipe) rather than the branch diameter.

be done in accordance with ASTM A 998. In some cases, circumferential reinforcement is also required. The ASTM standard not only provides tabulated solutions but also a complete design method. A computer program based on this method, CSPFIT, is available from the National Corrugated Steel Pipe Association.

There are several ways to reinforce most fittings and even more ways to attach the reinforcements. Most fabricators have their own detail depending on their tooling and inventory items. It is most economical to allow the fabricator to select the reinforcement means, while the specifier insists on reinforcement to ASTM A998 requirements.

Where steel structures are designed for storage (such as detention, retention and recharge systems) rather than flow, reinforcement can often be avoided by cutting a man-way or access door through the fitting. For example, rather than cutting out the full opening from the branch into to the mainline to fabricate a tee, a narrow doorway is cut just large enough to provide for adequate flow and personnel access through the tee.

Like a doorway, man-ways are typically cut two and a half to three feet wide and extend to the invert. The man-way is cut as tall as necessary (6' – 8" where possible) to provide easy access. The man-way does not require longitudinal reinforcement as long as its width along the axis of the mainline pipe does not exceed the diameter of the largest fitting (tap-in) in Table 8.1. The need for circumferential reinforcement should also be checked.

STEEL BULKHEADS

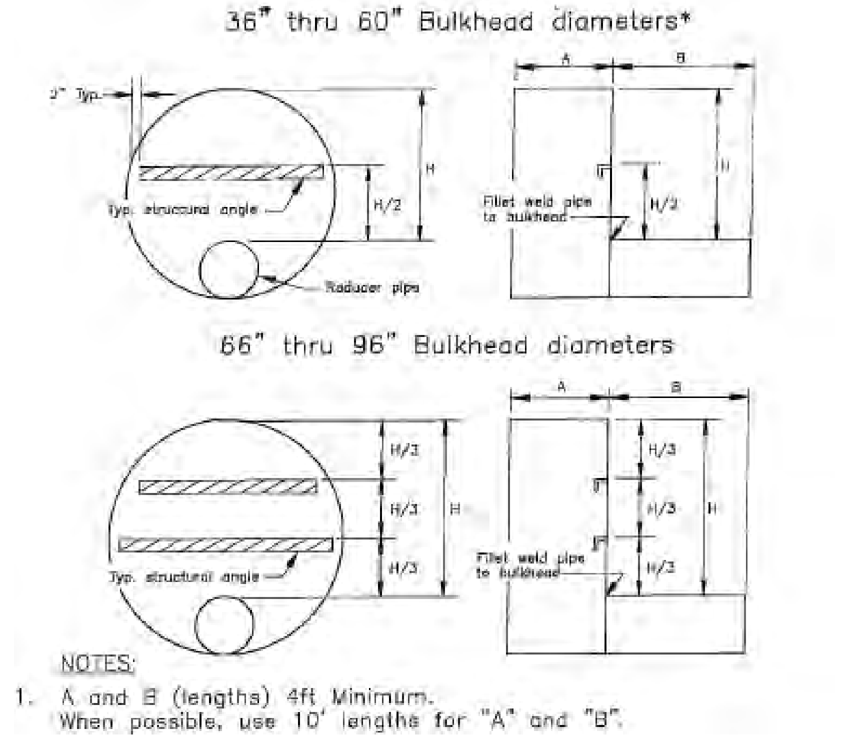
Steel bulkheads can be supplied with the pipe. They are widely used in detention and recharge systems and for transitions of smaller pipes into larger ones or as end plates. Typically they are continuously shop welded to CSP and either bolted or field welded to structural plate structures.

Shop attached CSP reinforced bulkheads.

The amount of reinforcement necessary varies with the diameter, depth of cover, and the thickness of the bulkhead plate itself. While it results in an overly conservative design, large diameter bulkheads can be handled traditionally by taking the reinforcements as a series of simple beams spanning the pipe end, with the bulkhead plate welded to them to develop composite action. In the opposite direction, the bulkhead plate itself can be analyzed as a continuous beam, spanning over the reinforcements.

A bulkhead welded to the pipe end more correctly acts as a fixed edge diaphragm. It may be designed using appropriate flat plate formulas from sources such as "Roark's Formulas For Stress and Strain", W.C. Young. Bending strength can be provided with a combination of the bulkhead plate thickness and steel reinforcements.

The basic equations to determine the necessary bending strength (required section modulus), reinforcement spacing and attachment welds, are provided below. The maximum reinforcement spacing depends on the bulkhead plate thickness. The spacing can be determined by taking the bulkhead plate as a second rectangular diaphragm with a width matching the spacing of the structurals. Welding requirements are provided that assure composite action of the plate and structural reinforcement.



■ **Figure 8.1** Bulkhead details.

Earth pressure $p = (h + 0.67\ D)g\ K_a / 144$

where:
P = design soil pressure on bulkhead, psi
h = height of cover, ft
D = diameter or rise of pipe, whichever is less, ft
g = soil density, typically taken as 120 pcf
K_a = active soil pressure coefficient (assume $K_a = 0.4$)

Bulkhead wall thickness $t_1 = [3w/(4\pi S_1)]^{1/2}$

where:
t_1 = required bulkhead wall thickness if an unreinforced, thick bulkhead plate were used, in.
$w = \pi\ (D/2)^2\ p$
$S_1 = F_y$ =36,000 psi (yield strength of steel reinforcement)

$$S_{req'd} = \frac{t_1^{\,2}}{6}$$

where: $S_{req'd}$ is the required section modulus of the composite, reinforced section used in lieu of a thick bulkhead plate (in^3 /ft of bulkhead width)

Max spacing, $b = [S_2 t^2/(\beta\ p)]^{1/2}$

Where:
S_2 = F_y = yield strength of the plate = 33,000 psi
β = diaphragm shape coefficient taken as 0.5
t = thickness of bulkhead plate chosen for use, in.

In fabricating these designs, the steel structural reinforcements must be located on the outside of the bulkhead. This insures that the flat plate will be in bending tension and will remain fully effective. To assure composite action, the bulkhead plate and reinforcement must be adequately welded together. Typically the reinforcements are welded to the plate with intermittent fillet welds sized to provide adequate shear flow between them. These attachment welds can be sized as follows.

Q = (A of one reinforcement) d

where: d is the distance between the neutral axis of the reinforcement and that of the completed composite section (reinforcement and plate)

$$q = VQ/I$$

where:

q = required weld strength of reinforcement, lbs/in.
V = maximum shear on the section (lbs/ft) which can be taken as 12 (span/2) p (in. lbs/ft)
I = moment of inertia of the welded, composite section (in^4) for one reinforced spacing

A specific weld strength and welding pattern can be selected by conventional means. However, limiting the maximum center to center weld spacing to 12 in., the following limits may be conservatively applied:

P_w = 700 L (lbs per weld) using an E70 electrode

where:

P_w = strength of a 1/16 in. fillet weld, lbs/in.
1/8 and 3/16 in. fillet welds respectively provide twice and three times this strength
L = length of each weld, in.

INTERNAL FLOW CONTROL STRUCTURES

It is sometimes necessary to incorporate flow control features within a drainage or detention system. In many cases, these flow controls are handled in the form of a weir plate that may have orifices or notches cut within it to limit the outflow at various elevations within the system. Corrugated steel pipes generally allow these features to be fabricated directly within the piping system, eliminating the need for transitioning to a junction chamber or other device to achieve the necessary flow controls.

In regards to the design methodology for internal weir plates is very similar to that described in the previous section for plate bulkheads. The primary difference is in the loads that need to be considered. Internal weir plates only need to be able to resist the hydrostatic loads of the water they are holding back within the pipe. Therefore, a conservative approach to determining the design pressure would be as follows:

Design pressure $p = (g \times H) / 144$

Where:

p = design pressure on the weir, psi
g = unit weight of water = 62.4 pcf
H = total height of weir plate, ft

Internal CSP reinforced weir.

Once this pressure is determined, it can be inserted into the same equation for determining the required plate thickness, required section modulus, allowable spacing for reinforcement, etc. as was given in the previous section for the design of bulkheads.

If a composite reinforced section is used, the reinforcement should be attached on the upstream side of the weir plate. Generally, the weir will be continuously welded to the pipe so water cannot leak through the seam.

CSP MANHOLES

Corrugated steel pipe makes an excellent, cost-effective manhole for use with CSP culverts, storm sewers and underground detention systems. The riser manhole and the shaft manhole are two common types that are used. These are shown below. The riser manhole is used where the mainline is a larger diameter than the manhole. The riser is typically aligned with the spring line of the main pipe rather than centered over the pipe. This not only transmits load more effectively, but also allows the ladder steps to transition smoothly to the floor.

CSP riser manholes in underground detention system.

CSP shaft-type riser.

Typically a shaft type manhole is set, open bottomed in a freshly poured concrete base slab. The slab extends out, beyond the manhole outside diameter (OD) far enough to keep the manhole from floating and is designed to be strong enough to handle any other vertical loads. Other vertical loads can include a wheel load applied to the manhole cover, the dead load or weight of the riser, and for shaft type manholes, any soil drag down load that is allowed to occur.

Either type of manhole is reinforced as any other fitting, in accordance with ASTM A998. However the shaft type manhole is less likely to need much reinforcement. In this case, the pipeline taps the manhole itself. The vertical shaft only sees the active soil pressure rather than the soil prism load exerted on a typical storm sewer of culvert.

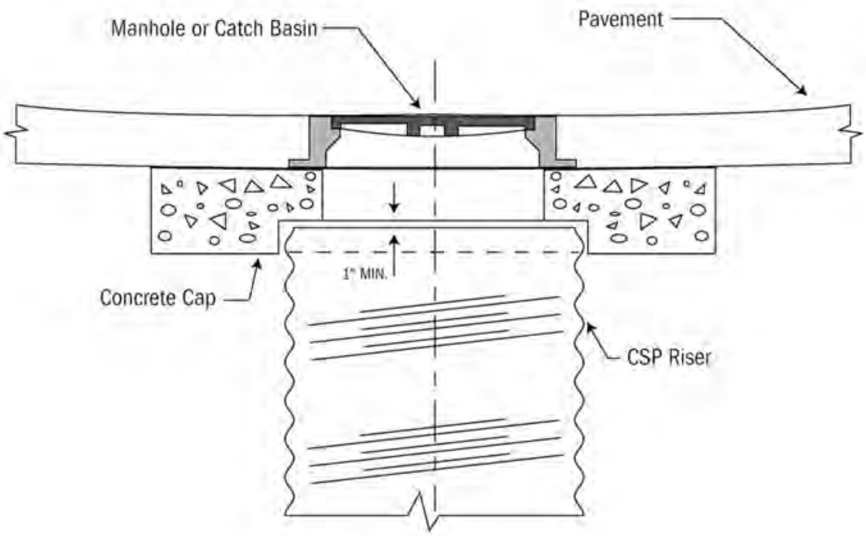
For riser type manholes, reinforcement of the connection to the mainline becomes an additional consideration. It is recommended that the manhole or catch basin inlet be supported at or near the surface by a concrete cap (actually a footing). The CSP riser is kept uncoupled from the cap so the cap floats, bearing down on the backfill and soil. This keeps the traffic load off the CSP manhole.

Drag-down loads are caused by soil settlement around the manhole riser. As the soil settles it attempts, through the friction force of the soil against the manhole, to drag the riser down with it. These loads can be very large. It is generally better to accommodate the movement (settlement) of the soil than try to design for loads of this magnitude.

Where necessary, the magnitude of the drag-down load can be estimated from the following equation:

$$Q = p_v \beta A_s$$

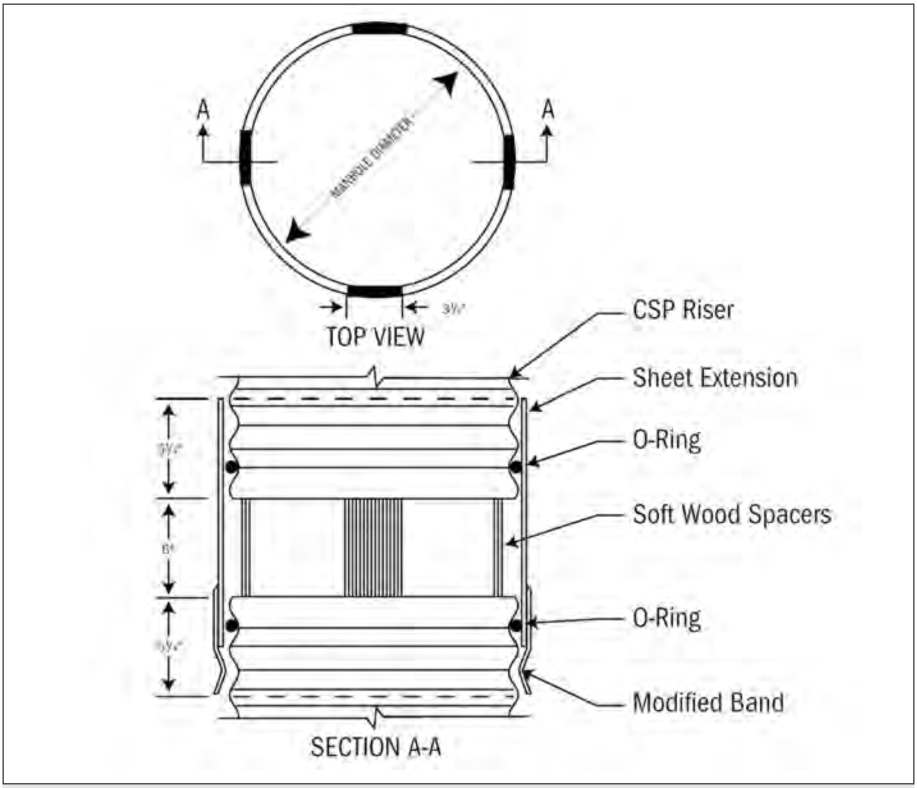
where: Q = dragdown force
p_v = average vertical soil pressure along height of riser, psf
β = 0. 20 to 0.025 for clay; 0.25 to 0.35 for silt; and 0.35 to 0.50 for sand.
A_s = surface area of the riser = DH
D = diameter, ft
H = height, ft



Notes:
1. A concrete cap shall be used on top of a riser when the riser is located within the area of vehicular traffic.
2. The concrete cap shall be sized and designed by others so that vehicular loads are transmitted to the soil and not to the riser.
3. The concrete cap shall be sized to provide an adequate bottom area based on the allowable bearing capacity of the soil.

Figure 8.2 Manhole cover detail.

Rather than attempt to design for these loads, it is often better to install a slip joint near the bottom, just above the mainline pipe(s). This is generally done by using a special band and wooden blocking devices as shown in Figure 8.3. Excessive loads split the blocks, which allows the riser to move down with the settlement and relieve the loads. Manholes taller than 10 feet or those backfilled with other than well compacted, granular materials, should have a slip joint located about 2 feet above the mainline pipe(s). With very tall risers, it is best to install at least one additional slip joint, farther up the riser. An estimate of the soil modulus for the contractor-placed backfill around the riser and the soil load on it, can be used to estimate the settlement that will occur. There needs to be enough potential slippage to accommodate expected settlements.



■ **Figure 8.3** CSP riser slip joint.

Loads beyond these are primarily the horizontal pressures on the manhole. These include the active soil pressure, any ground water pressure, and lateral affects of live load pressures (near the surface). They are described as:

$$P_h = \gamma_w H_w + K_a\{\gamma H [1 - 0.33(H_w/H) + P_{LL}\}$$

where:

γ_w = density of water = 62.4 pcf
H_w = height of the ground water above the location being evaluate, ft
K_a = active soil pressure coefficient = $\tan^2(45-\Phi/2)$

where: Φ is the internal friction angle of soil (for clays, take Φ as 28 degrees)

γ = density of the soil of the moist soil, typically taken as 120 pcf
H = height of soil cover above the location being evaluated, ft
P_{LL} = live load pressure at the depth evaluated, psf

HYDROSTATIC BUCKLING

Conduits that are not buried in compacted soil while subjected to external hydrostatic pressure may be designed for buckling assuming they act as circular tubes under uniform, external pressure. No active or passive soil pressure is available for support in this condition and the pipe ring itself must resist instability, including the effects of bending moments resulting from out-of-roundness.

"Theory of Elastic Stability", Timoshenko and Gere, details methods of analysis for such thin tubes. However, no extensive correlation has been made between these buckling equations and corrugated pipe. Field experience and the few tests that have been done suggest that a modified form of the equations provides a conservative estimate of the collapse pressure of corrugated steel pipe.

The Timoshenko buckling equation is:

$$P_{cr} = \frac{3EI}{(1 - \mu^2)\, R^3}$$

Where: P_{cr} = critical pressure, psi
E = modulus of elasticity of pipe wall, psi
I = moment of inertia of pipe wall, in.^{4}/in.
μ = poisson's ratio = 0.3 for steel
R = radius of pipe, in.

To provide for slight imperfections and other variations from ideal conditions, design for the critical pressure divided by 2. This results in the following estimated design pressure limit:

$$P_e = \frac{3EI}{2\,(1 - \mu^2)\, R^3}$$

where: P_e = design pressure limit, psi, including a factor of safety of 2.0

These equations assume the pipe is round and surrounded by a uniform pressure fluid. However, no pipe is perfectly round and the strength reduction due to an out-of-round condition is significant as shown in Figure 8.4. Thus, if the deflection exceeds 1 or 2 percent, a further reduction in the design pressure may be in order.

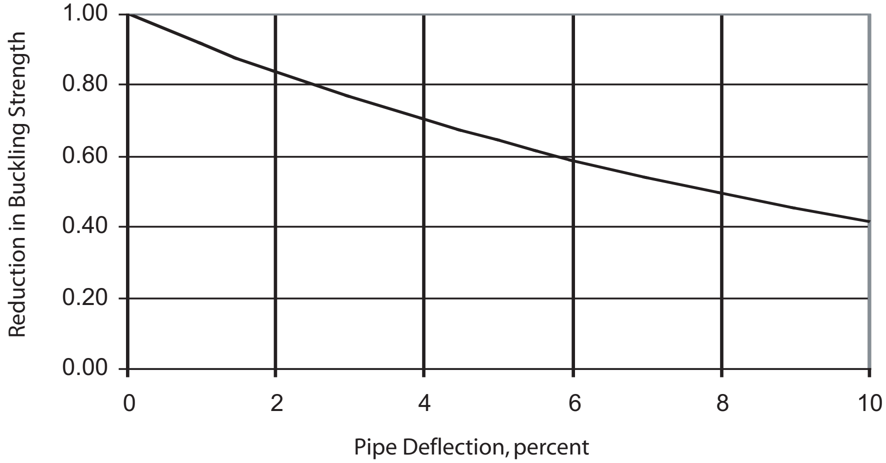


Figure 8.4 Reduction in hydrostatic buckling strength due to pipe deflection.

Many applications such as using CSP as a vertical standpipe in a lake, grouting it in as a vertical shaft liner, etc. do not subject the pipe to uniform pressure. Rather, the maximum pressure occurs at the lowest elevation, which is often imbedded in concrete for support. If the critical buckling pressure is reached near the level of the concrete, the pipe cannot buckle because it is supported by the immediately adjacent concrete. Additionally, a few feet above this point, the pipe is exposed to a lower pressure. To some degree, this portion of the pipe with less pressure provides additional support against buckling for the critically loaded section.

RELINE STRUCTURES

Corrugated steel pipes and structures are widely used for rehabilitation and reline applications as discussed in Chapter 12. The ability to manufacture CSP in any diameter necessary as well as to supply it with a hydraulically smooth interior adds to its appeal.

Rehabilitation methods typically involve grouting the annulus between the liner and the host structure. Grouting in this manner often stops the deterioration of the host pipe while it adds several inches of structural grout. Typically, reline structures are designed to carry the entire load imposed on them. This is inefficient in that an in-service host structure has at least a factor of safety of 1.0 under those same loads. However, design can conform to methods reviewed in Chapter 7, with the exception that the stated flexibility factors are not applicable.

Careful, low-pressure grouting places lower bending stresses on the pipe than placing and compacting traditional backfill. Also, the reline structure usually can be braced against the host structure to prevent flotation and improve buckling strength. The stiffness (flexibility factor limit) required in the liner becomes a matter of the grouting rate and technique involved. The Timoshenko buckling equation does not apply to grouting a horizontal structure. The pressure around it is not uniform. Rather the contractor must maintain a balance of grout depth, from side to side, much as would be done in using conventional backfill. The contractor must also handle buoyancy forces by bracing the reline structure off the host pipe.

For construction details, see Chapter 10.

VERTICAL SHAFT LININGS

Vertical shafts are used to construct inspection pits, deep foundations, insertion pits, etc. Often these are temporary structures that are used as forms or construction aids and not as permanent structures. In stable ground conditions, where a bored hole will stand for a short period, shaft linings are often picked up and inserted into the hole in a single piece. Alternatively, where ground conditions demand, shafts are excavated from the surface in stages with segmented 2-flange steel liner plate linings being erected in the hole as it advances. Corrugated steel pipe, structural plate, and steel liner plates, are typically the material of choice for shaft liners.

The loads on vertical shaft linings are quite different than those on normal buried pipes. Soil loads typically are limited to the active soil pressure acting on the shaft. These can be as little as a third or half of the soil prism load carried by a buried pipeline at the same depth. Once a shaft penetrates the water table, the liner must carry both the full hydrostatic load of the water and the active pressure of the buoyant soil.

While the design example that follows suggest a factor of safety of 2, the actual requirement for temporary structures depends on how well the ground conditions (and resulting loads) are known and the necessary degree of safety to protect the bore and any workers involved. In some instances, lower factors of safety may be acceptable. Design for the imposed loadings is identical to those for manhole shafts, but drag-down loads can be ignored when the bore is in stable soil that will not settle.

The necessary installation stiffness must be addressed. Vertical shafts can typically be more flexible than a buried pipeline since the installation loads are less severe. However, if the liner is to be back grouted rapidly, the resulting fluid grout (hydrostatic) pressure may dictate the necessary stiffness. The Timoshenko buckling equation and discussion, should be reviewed. With the many corrugated steel alternatives, the contractor can select an appropriate minimum stiffness to meet the installation requirements of the site and construction sequence.

A 16 foot diameter structural plate shaft liner is picked up to be inserted in a bored shaft.

Inserting the structural plate shaft liner.

Unlike backfilling a conventional pipeline or grouting a reline structure, grouting the annulus of a vertical shaft induces uniform, radial loads around the liner. Thus, the compaction and unbalanced fill loads induced during conventional pipeline installation are avoided. Unlike a conventional (horizontal) tunnel liner or reline structure, the vertical shaft does not need to support an unbalanced, side-to-side, grout loading during construction.

Other construction loads can come from a slough-in or other soil failures and surface loads from construction equipment, etc.

Safety is the major consideration. Even where workers are not in the hole, the cost of losing the bore is a consideration the contractor must address. Increasing the stiffness of the liner can provide an added measure of safety if a slough-in occurs. The effectiveness of additional stiffness depends on the specific site conditions, construction practices and other factors difficult to predict.

Typically, the stiffness requirements for shaft and tunnel liners is expressed as:

Minimum Stiffness = $EI/D^2 \geq$ Stiffness Factor

The contractor or his engineer should provide stiffness limits. Where they are not provided, suggested stiffness factors for vertical shafts are summarized in Table 8.2. Lower stiffness liners may be used, depending on ground conditions and construction practices.

Table 8.2

Typical stiffness factors for vertical shaft liners

Corrugation Depth (in.)	Stiffness Factor (lb/in.)
1/2, 3/4 & 1	17
2	33
2-flange liner plate	33
4–flange liner plate	74

Design Example

Given: Shaft diameter = 12 ft
Excavation depth = 38 ft
Water table depth = 25 ft
Soil Unit Weight, γ = 120 lbs/ft^3
Buoyant unit weight γ' = 72 lbs/ ft^3
ϕ = 30 degrees
Active earth pressure, K_a = Tan (45-ϕ/2) = 0.333
Liner will be grouted in place.

Solution:

1. Design Pressure:
   Earth pressure at 25 feet = γK_a 25 = 999 psf
   Buoyant earth pressure at 38 ft = $\gamma' k_a$ (38-25) = 312 psf
   Water pressure at 38 feet = 62.4 (38-25) = 811 psf
   Total pressure P = 2122 psf

2. Ring Compression:
   C = P x S/2
   Where S = span, ft
   Then C = (2122) x (12/2) = 12,732 lbs/ft

3. Allowable Wall Stress
   For 12 ga 2-flange liner plate
   $S = 144 < (r/k)\ [24E/F_u]^{.5} = 366$ in.
   where:
   A wall = 1.62 in^2/ft, I=0.049 $in.^4$/in., r = .602,
   SS = seam strength= 30,000 lbs/ft
   K = 0.22 assumed for grout backfill
   E = 30,000,000 psi
   F_u = 42,000 psi
   S is less than 366 in., therefore:
   $F_b = F_u - [(F_u^2/48E)]\ (kS/r)^2 = 38{,}607$ psi
   $F_b > F_y$, Therefore: F_c= 28,000 psi and $f_c = F_c/2$= 14,000

4. Required Wall Area
   $A = C/f_c$ = 12,732/ 14,000 = 0.909 in^2/ft < 1.62
   12-ga. 2-flange steel liner plate **OK**

5. Seam Strength
   $SS \geq 2C$ = 2(12,732) = 25,464 lbs/ft < 30,000 lbs/ft
   12-ga. 2-flange steel liner plate **OK**

6. Minimum Stiffness
   Min. Stiff = EI/S^2 = 70.8 > 33
   12-ga. 2-flange steel liner plate is **OK**

7. Ultimate Buckling Pressure due to fluid grout
   $P_{cr} = 3EI/[(1-\mu^2)R^3] = 3(30000000)\ 0.049/[(1 - 0.3^2)\ 72^3] = 13.0$ psi

   where:
   P_{cr} = critical buckling pressure, psi
   μ = Poisson's ratio = 0.3 for steel
   R = pipe radius, in.

Equivalent feet of fluid grout = 13.0(144)/140= 13.4 ft (A suitable factor of safety needs to be applied).

TUNNEL LINERS

Loads

When steel structures are installed by jacking or tunneling, the soil load on the structures may be considerably less than indicated by the load factors, K, discussed in Chapter 7. In sound soils, the jacking or tunneling processes can produce a soil bridging effect that keeps most of the load off the structure. However, in the case of plastic clays, the entire soil overburden load is likely to come to rest on the structure at some point in time.

During the first half of the 20th century, Anston Marston participated in numerous load studies and developed the soil load theory on buried structures that is still widely used today. It forms the basis for the current loading charts for steel tunnel liner plates. The design pressure acting at the top of the tunnel or jacked pipe structure can be taken as:

$$P_d = C_d (\gamma H + PL)$$

Where: C_d = soil load coefficient from Figure 8.5
C_d = 1.0 if inadequate information is available to describe the soil at the level of the structure.
γ = soil density, typically taken as 120 pcf
PL = live load pressure (from Table 7.7) taken at the crown elevation of the structure.

Design considerations

Once the design pressure (P_d) is determined, it is used to calculate the thrust in the structure and design checks are otherwise performed in accordance with Chapter 7. The soil stiffness factor K used in buckling calculations depends on the backfill or grout immediately around the structure. For a liner backfilled with clay, K, is taken as 0.44 while sand backfilling produces a design value of 0.22. Typically, a grouted annulus is checked using a K of 0.22 even though this can be very conservative.

Having determined the actual load on the tunnel liner, the remainder of design follows principles in Chapter 7, to achieve a minimum factor of safety of 2.0. However, construction stiffness can become an issue. The stiffness of liner plate is calculated as the reciprocal of its flexibility factor. That is, the minimum stiffness for a horizontal, 2-flange tunnel is:

$$\text{Minimum Stiffness} = EI/D^2 \geq 50 \text{ lb/in.}$$

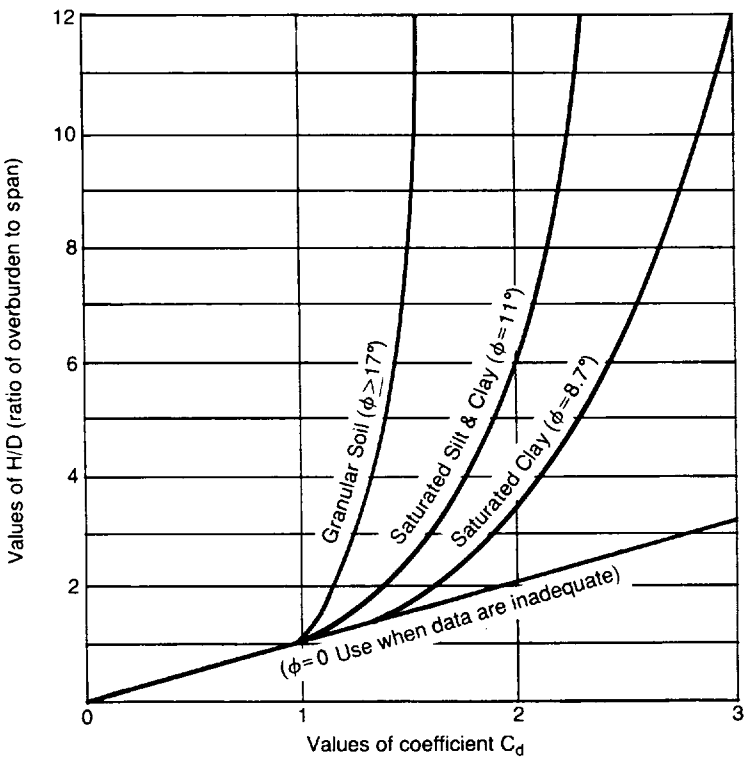


Figure 8.5 Diagram for coefficient C_d for tunnels in soil (ϕ = friction angle)

Where: E = young's modulus for steel = 30,000,000 psi
I = tunnel liner moment of inertia (from Table 8.3)
D = diameter of the tunnel liner, in.

A minimum stiffness value of 50 lb/in. is equivalent to a flexibility factor limit of 0.020 inches per pound used for an embankment installation of a 2 inch deep corrugated steel structural plate. It provides adequate stiffness for backfilling (done by compacting the void between the liner and bore full of sand) or grouting.

Unlike a vertical shaft liner, a tunnel liner is often selected to provide a higher stiffness than needed for simple backfilling. Until it is supported by grout or backfill, the liner is protecting the crew from collapse of the tunnel bore. In poor soils or instances when the

Table 8.3

Structural properties for 2-flange liner plate*

Specified Thickness (in.)	Uncoated Thickness (in.)	Effective Wall Area ($in.^2/ft$)	Effective Moment of Inertia ($in.^4/in.$)	Ultimate Seam Strength (lb/ ft)
0.079	0.0747	1.152	0.034	20,000
0.111	0.1046	1.620	0.049	30,000
0.140	0.1345	2.088	0.064	47,000
0.170	0.1644	2.556	0.079	55,000
0.188	0.1793	2.796	0.087	62,000
0.218	0.2092	3.264	0.103	87,000
0.249	0.2391	3.740	0.118	92,000

Notes:
* Steel per ASTM A 1011
  Tensile strength = 42,000 psi
  Yield strength = 28,000 psi
  Minimum Elongation (2 in.) = 30%

actual soil conditions have not been determined, one often selects a heavier gage steel tunnel liner to obtain two or three times the minimum stiffness requirement to provide added protection for the crew.

AERIAL SPANS

Should the need arise to run water or sewers, etc. above ground or under bridges, CSP aerial sewers supported on bents afford an economical solution. Table 8.4 provides allowable spans for this purpose. The table is for pipes flowing full of water, including the weight of an asphalt-coated pipe. The bending moments were calculated on the basis of a simple span and with the pipe bending strength determined by limited testing.

Consideration must be given to the design of the pipe support system. Small diameter pipe with short spans can often be placed directly on bents. Larger diameter pipe should be supported by shaped, 120-degree concrete cradles or by a ring girder. The importance of the support requirements increases with diameter and span. Design methods used for smooth steel water pipe systems can be adapted to investigate these requirements.

COLUMN OR END LOADS

Tests were conducted as early as 1930 at the University of North Carolina and the University of Illinois to determine the strength of 2-2/3 x 1/2 inch corrugated steel pipe for carrying compression end loads. These results are useful in determining the necessary strength of circumferential seam connections, maximum jacking loads and its strength for use as bridge piers, caissons, vertical shaft liners and other columns used in construction.

Corrugated steel pipe aerial sewer.

Table 8.4

Allowable span in feet for CSP flowing full

Diameter of Pipe, in.	Specified Steel Thickness, in.								
	0.064	0.079	0.109	0.138	0.168	0.188	0.218	0.249	0.280
2-2/3 x 1/2 in. Corrugation									
24	13	15	20						
36	12	15	20	25					
48	11	14	19	25	30				
60		14	19	24	29				
72			18	24	29				
84				23	28				
96					27				
5 x 1 in. or 3 x 1 in. Corrugation									
36	9	11							
48	9	11	15						
60	8	10	14	18					
72	8	10	14	18	22				
84	8	10	14	18	22				
96		10	14	18	22				
108			14	18	21				
120				17	21				
6 x 2 in. Corrugation									
			0.111	0.140	0.170				
72			12	15	17	19	22		
84			11	14	17	19	22	24	27
120			11	14	16	18	21	24	27
144			11	13	16	18	21	24	27
168			10	13	16	18	21	23	26
192			10	13	16	17	20	23	26
216				12	15	17	20	23	26
240					15	17	20	22	25

Subsequent tests at the Ohio State University (1965) confirmed that these short column results are conservative for both annular and helically corrugated steel pipe. Ultimate short column or compression block values for the 2-2/3 x 1/2 inch corrugation are provided in Figure 8.6.

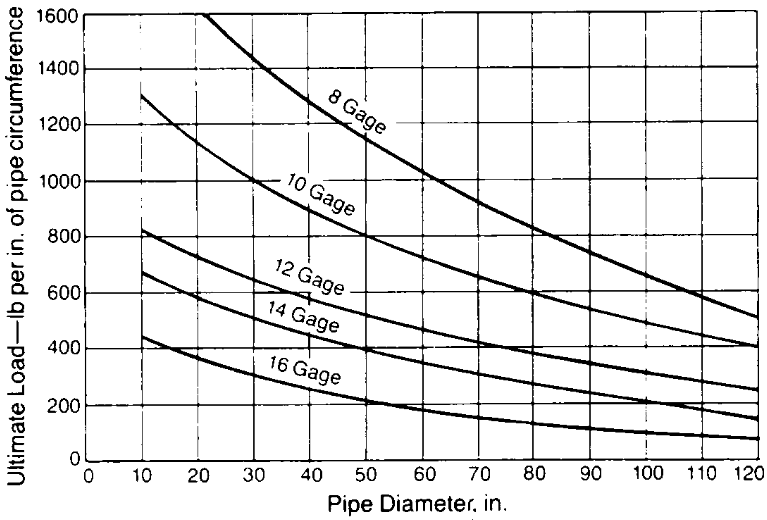


Figure 8.6 Ultimate unit compressive strength of short 2-2/3 x 1/2 inch corrugated steel pipe columns as determined at University of Illinois.

Recent strength testing, comparing the strength of other corrugations with 2-2/3 x 1/2 extends this earlier work to a broader range of corrugations. Being shallow, the 2-2/3 x 1/2 corrugation has more column load capacity than the deeper corrugations. Values tabulated in Table 8.5 are from testing done to develop ASTM A 998 and are expressed as a multiplier to reduce the values for 2-2/3 x 1/2 end loads from the figure.

Table 8.5

Column or end load strength

Corrugation Depth (in.)	Factor
3/4 (rib)	0.30
1	0.44
2	0.30

Example

Given: Determine the end load strength of a 54 in. diameter, 0.064 in. thick, 3 x 1 in. pipe

Solution

From Table 8.5 the end load capacity of 3 x 1 in. corrugated steel pipe is 44% that of a 2-2/3 x 1/2 in. pipe.

From Figure 8.6 the end load strength of 54 in. diameter, 0.064 in. thick (16 gage) 2-2/3 x 1/2 in. = 200 lbs per circumferential in.

Thus the end load capacity of 54 in. diameter 3 x 1 in. = 0.44 (200)= 88 lbs per circumferential in.

RECLAIM (CONVEYOR) TUNNELS

Reclaim tunnels and conveyor enclosures are nearly horizontal, buried pipe applications. However, they have special features that typically include a varying dead load caused by a rising and falling ore pile, as well as ore hoppers and conveyor bents that are often hung off the structure. At the same time, when the storage piles are run up, most of these structures are near their maximum safe cover limit, while they are often at minimum cover when the last of the ore is being bladed into the hoppers.

Because of their critical nature, they must be properly designed and the best quality backfill is needed. Yet a load reduction factor ($K<1.0$) does not apply to these structures because the loading is cyclical. Although load relief occurs the first time the ore pile is built up, additional deflection would need to occur each subsequent time in order to continually achieve load relief. In fact, these structures become locked in to the backfill and do not continue to deflect with each load application.

When the ore is drawn down it is not uncommon to use a front-end loader to push the last of the ore into the hopper. Compacted structural backfill needs to continue up a distance of span/8 above the structure, but if the working axle load of the loader exceeds H20, additional cover will be required (see Construction Loading, Chapter 10).

The hopper openings in the structure must be reinforced. While procedures are similar to reinforcing fittings in pipe, the openings here do not have a fitting stub welded integrally into the opening. Therefore, the reinforcement becomes more significant than that outlined in ASTM A 998. The fabricator should provide the reinforcement details, which are typically sized to carry the bending moment developed in the longitudinal reinforcements due to the thrust in the pipe. Assuming a simple beam, this results in a bending moment, M, in.-lb, of:

$$M = (C\, l^2)/8$$

Where: C = ring compression in the pipe (lbs/in.)
L = unsupported length of the reinforcement (in.)

Finally, the ore hoppers and conveyors are often hung directly off the structure. These loads not only put extra thrust in the structure, they typically are point loads applied to the structure asymmetrically. The hoppers and conveyor loads should not be hung directly. Rather stiff, curved, ring beams are typically applied to the outside of the structures so they bear, much like a saddle, on the corrugated steel structure. Ring formulas can be used to evaluate their necessary length and stiffness.

Alternatively, light crane loads as well as ore hopper and conveyor loadings can be supported by using stiff longitudinal beams to spread the loads sufficiently along the length of the structure. The longitudinal beams should be much stiffer than the longitudinal stiffness of the structure so the beams spread the point loads over enough attachment points for the structure to carry the loads.

MINIMUM COVER EVALUATIONS

Minimum heights of cover are difficult to calculate directly. Minimum cover levels have been determined through experience with the primary concern being that of maintaining the pavement, not collapsing the pipe.

To date, the best analytical approach to minimum cover requirements has been developed by Dr. J. M. Duncan. This method accounts for the size of the axle load, the plastic moment strength of the pipe and the stiffness of the backfill. However, it is not completely calibrated and if the entire method is applied it may not provide results that agree with the accepted, experience-based minimum cover limits.

A simplified approach based on Duncan's work can be used to see the effects of increasing axle loads, heavier than normal steel thicknesses, or improved backfill. A constant C_3, below, is calculated based on corrugation, axle load and soil stiffness. The C_3 value can be used to provide a ratio of the necessary minimum cover depths or plastic moment strength (M_p) requirements from the AASHTO Span / 8 (Span / 4 for Spiral rib) rule for the specific pipe involved. Consider the following:

$$M_p = K_3\left(\frac{S}{H}\right)^2 \quad \text{or} \quad H = S/(M_p/C_3)^{0.5}$$

where:

S = span of the structure (in.)
H = actual minimum cover (in.)
C_3 = constant such that:
= 69 AL/(32C)

$$K_3 = \frac{AL\ d\ F_p}{C}$$

C = 69 for 90% standard Proctor compaction, or
C = 115 for 95% standard Proctor compaction
AL = Axle load (kips)
M_p = plastic moment strength of the pipe wall (ft-kips/ft)

From the suggested value of C_3 it is seen that the calculation for M_p assumes a 32 kip axle load along with 90% Proctor density backfill. Thus it can be seen that doubling the axle load either doubles the plastic moment strength required in the pipe or increases the necessary depth of cover by a factor of $(2)^{0.5}$.Using 95% standard Proctor density backfill in lieu of 90% Proctor reduces the necessary plastic moment strength to 60% (69/115 = 0.6) of that originally required. A similar increase in density of the backfill can reduce the required minimum cover height to about 80% of the AASHTO level ($0.6^{0.5}$). This method will provide a ratio to change the standard fill height requirement to an actual minimum cover needed for heavier axle loads by accounting for the benefit of stiffer backfill or a heavier gage (higher M) pipe. It is conservative to use a heavier axle load if that larger load leads to a wider footprint than the 20 square in. commonly assumed for an H 20/H 25 dual tire load. However, it is not recommended that a ratio be used for the minimum cover required for a lower axle load when the footprint is reduced.

To account for increasing the soil density from 90 to 95% Proctor, it is most reliable if a crushed rock backfill or a clean A1 material (course sand or gravel) per AASHTO M 145, is specified and its density field tested. Using select backfill materials help obtain the necessary soil stiffness.

This approach is not intended to replace the standard, experience-based, minimum covers from Chapter 7. However, it can be used to evaluate field conditions where grade elevations do not meet design requirements.

One use of the ratio allows accounting for too little minimum cover by increasing the material thickness and its plastic moment strength (M_P). In doing so, however one must recognize that a minimum cover of span / 10 requires a higher theoretical factor of safety than span / 8. It is suggested the required Mp be increased by an additional 10% as cover decreases to a minimum of span / 10.

For the sake of conservatism, changes should be limited to no more than one or two parameters in the design. Generally, minimum covers should not drop below span /10 (span / 5 for spiral rib pipe) or the 12 inch minimum not reduced to less than 9 or 10 inches.

Beyond moment strength considerations, high shear strength backfill materials such as crushed rock, cement stabilized sand or cement slurries have long been used to reduce the required cover on a steel pipe installation. Incorporating these materials can reduce the required minimum cover to 67% of the original requirement. Double reinforced structural concrete has been used as a load relief slab or saddle to reduce the required minimum cover by as much as 50%.

Table 8.6

Plastic moment strengths "M_p" of corrugated steel pipe (k-ft/ft)

Wall Thickness (in.)	2-2/3 x 1/2	3 x 1 / 5 x 1	6 x 2	15 x 5-1/2**	16 x 6**
0.064	0.39	0.79			
0.079	0.49	0.99			
0.109 /0.111*	0.69	1.40	2.66		
0.138/ 0.140*	0.90	1.82	3.44	14.43	
0.168/ 0.170*	1.11	2.24	4.22	17.66	18.46
0.188			4.73	19.75	
0.197					21.70
0.218			5.54	23.07	
0.236					26.34
0.249			6.36	26.39	
0.276					30.78
0.280			7.18	29.72	
0.315					35.00

Notes:
* Where two thicknesses are provided, the first is for pipe and the second for structural plate.
** M_p values for 15 x 5-1/2 and 16 x 6 corrugation are based on a yield strength (Fy) of 44 ksi.

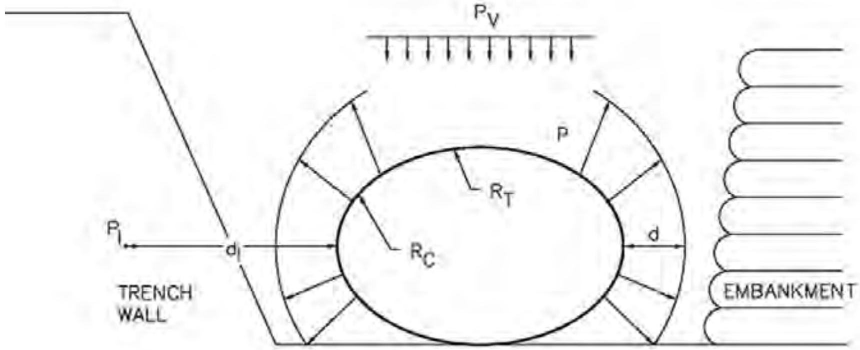
BEARING PRESSURE EVALUATIONS

Chapter 7 provides a means of evaluating bearing pressures at the tight radius corners of pipe arch, ellipse and underpass shapes. Typically, bearing pressures are calculated at the surface of the steel structure and a nominal width of backfill is provided. However, under high covers or in soft soil conditions, it becomes desirable to evaluate these pressures at various distances from the pipe to determine the effect of a wider backfill zone or the necessary strength of the embankment or trench wall.

For example, the designer may want to limit the horizontal compression strain of the backfill and embankment outside of it to 1% of the pipe diameter. Theoretically at least, this would result in 2% increase in span due to compression in the soil on both sides of the pipe. To make this evaluation, the designer may elect to calculate the pressure at the center of the backfill zone and at one or two distances out into the embankment or trench wall beyond. At a distance of one span from a round pipe the pressure in the soil has generally returned to its at rest pressure, even with the pipe in place.

Forces acting radially off the small radius corner arc of the structure at a distance d_1 from the plate surface may be taken as:

$$P_1 = T / (R_c + d_1)$$



■ **Figure 8.7** Bearing pressure evaluations.

Where: P_1 = horizontal pressure from the structure at distance d_1 (psf)
d_1 = distance from the structure (ft)
T = total dead load and live load trust (lb/ft)
R_c = corner radius of the structure (ft)

The required backfill envelope width, d, to limit strain in the trench wall or embankment is:

$$d = (T/P_{brg}) - R_c$$

Where:
d = required backfill envelop width (ft)
P_{brg} = allowable bearing pressure to limit the compressive strain in the trench wall or embankment to a suitable level (psf)

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