NOTES ON THE SPECIFICATION FOR LIGHTING COLUMNS

1. INTRODUCTION

This specification is to cover the performance, design, approval, fabrication, testing and installation of lighting columns of the direct ground planted and flange mounted types, for rigid and passively safe (frangible) types, with several options on the column materials and forms used.

The supply of passively safe (frangible) lighting columns has been encouraged in the interests of road safety, by allowing several alternative testing standards or procedures for impact testing.

2. GENERAL REQUIREMENTS

Dimensions of the lighting columns and parts used historically have been retained as much as possible.

To secure the base compartment door, a secure fastener system, such as security hex socket (pin-in-hex-socket) or security torx (pin-in-torx) is to be used.

Note that the ground stub for shear base columns is now required to have a minimum wall thickness of 6mm.

3. STRUCTURAL DESIGN REQUIREMENTS

3.1 General

This specification, by establishing that the structural design actions are to follow AS/NZS 1170 general procedures and principles using limit state design, requires that the lighting columns must be designed, and satisfy, New Zealand environmental conditions and loading.

3.2 Design Working Life

The design working life is now defined as either a minimum of 25 or 50 years. The longer period of 50 years is for lighting columns that are deemed to be critical in either their operation or location.

3.3 Design Loading

Although the design working life of 25 years under AS/NZS 1170 for a lighting column with an importance level of 2 requires a regional wind speed of 43 m/s for a return period of 250 years, a minimum site design wind speed of 45 m/s has been defined. This, along with the minimum (rural) terrain category of 2, ensures that all lighting columns nationwide shall have a minimum level of robustness.

3.4 Serviceability Limits

The deflection criteria have been kept as per the superseded NZTA M19 specification. The values remain in the middle of those from the range of BS, EN and AS/NZS 4676 standards for utility service poles.

3.5 Durability Considerations

It is assumed that lighting columns should be able to reach their intended design life without significant maintenance (e.g. reinstating the protective coating) but the level of protection required to achieve this will depend on both the specified intended life and the environment in which the material is located. This life to first maintenance of the protective coating is dependent on the atmospheric corrosivity classification for the local macro environment and while this is largely governed by distance from the sea, the local topography, wind strength and direction and average humidity levels also have their influence. In addition, microclimates can exist, on surfaces that are sheltered from rain washing, in crevices in which salts can concentrate due to capillary action and evaporation, and on internal surfaces where condensed moisture is unable to drain away.

3.6 Fatigue Considerations

Historically, fatigue in lighting columns has not been a large problem, but as materials are designed to their limits, the serviceability limit state for fatigue in the stress levels and critical detail types becomes more significant.

Lighting columns are relatively flexible and hence will experience a high number of stress cycles in the life. The number of cycles induced in the column and holding down bolts from natural wind gusts can be higher than those covered under NZS 3404, and so the procedure and strengths of the current edition of the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, has been followed. Also base welds are now required to comply with AS/NZS 1554.5 which has higher fatigue resistance.

The loadings from yearly mean wind speeds of 6.5 m/s and 5 m/s for the two wind regions of New Zealand are used to check the critical stress points. Note that these may be exceeded in some locations, e.g. where mounted on bridges over gorges with funnelling effects, when a site specific wind speed should be adopted.

For tall flange mounted columns, use of the fixing details and installation method given in VicRoads Design of Steel Cantilever and Portal Sign Structures and High-Mast Light Poles, (BTN 2010/001) is recommended.

3.7 Foundation Design

Foundations are to be designed in accordance with accepted principles of soil mechanics, taking into account the soil properties of the foundation material, along with the influence of the water table and sloping ground.

The typical strength reduction factors to be used for ultimate design should meet the following requirements unless specific investigations are undertaken to justify a different value are given in Table 3-1 below.

Table 3-1 Range of Strength Reduction Factors to be used for Ultimate Design

Load Direction	Method of Soil Assessment	Strength Factor Φ
Horizontal	Soil maps	0.4
Horizontal	Visually assessed soils from bore logs or test pits.	0.5
Horizontal	Geotechnical analysis of soils, including laboratory or in situ testing	0.6
Horizontal Earthquake		0.8
Vertical Bearing		0.8

The New Zealand Building Code Verification Method B1/VM4 for foundations as published by the Department of Building and Housing, can be used as a design method for the ultimate limit state design. This uses the Broms method as a basis for design of lateral resistance.

The ENA (Energy Networks Association, Australia) C(b)1 *"Guidelines for design and maintenance of overhead distribution and transmission lines"* which led to the Appendix L3 Foundation Design for Poles, in AS/NZS 7000 - Overhead line design, have good guidance on the use of the Brinch Hansen method of calculating for pole foundations. (The ENA Brinch Hansen program is currently available for download at http://ipowermation.com/download/bh/).

This Brinch Hansen method is considered appropriate for the dimensional range and characteristics of lighting columns, is applicable for a wide variety of soil types, and provides consistent results, with the proviso that the method does not give indications of column rotation at nominal failure load, although ground line rotational displacements of 1 – 2 degrees may be expected.

4. LIGHTING COLUMN IMPACT PERFORMANCE

4.1 General

For new roads, a safe roadside is generally achieved by ensuring that sufficient space is provided immediately adjacent to the road that is both free of obstacles and designed so that drivers are able to regain control of their vehicles. For existing roads the provision of a safe roadside usually involves removing or treating hazards that may result in a crash or contribute to the severity of a crash.

Whilst it would seem desirable to provide a completely clear width adjacent to the carriageway that would allow all errant vehicles to recover, recent research suggests it is not feasible to provide sufficient width to allow errant vehicle speed to drop to a level where the resultant impact forces are survivable. Hence to design a roadside that has an acceptable level of risk there is a need to both manage vehicle speed and mitigate hazards.

Acceptable vehicle speeds may be achieved by local speed limit reductions. Keeping the roadside clear of hazards has obvious benefits, but for lighting columns, moving the column back from the side of the road inhibits the main function of the lighting column. Therefore, the main option of the risk mitigation of vehicle impact is to either use a road safety barrier or making the column passively safe (frangible).

In areas of high pedestrian activity where passively safe (frangible) lighting columns are not normally permitted and locations where cut/fill slope limitations mean the impact performance of slip base lighting columns cannot be guaranteed, barrier protection of ground planted columns may be required.

4.2 Passively Safe (Frangible) Lighting Columns

There is now a vast amount of literature and testing that has been published on the hazards posed by rigid lighting columns and on the alternatives that have been developed. Conventional rigid lighting columns, whether made of steel, concrete, or timber, pose one of the greatest dangers to motorists with a high accident severity for fixed object collisions. The high accident severity is due to the columns normally being rigid, with any vehicle impact causing a high "head" deceleration to the vehicle occupants.

The term "frangible road lighting column" is stated in AS/NZS 1158.1.2 as covering all types of lighting columns that are specially designed to break away, yield, or otherwise absorb the energy of an impacting vehicle, to the extent that the resultant deceleration forces on the vehicle and its occupants are reduced to within specified acceptable limits. Passively safe (frangible) lighting columns do not all behave in the same way in an impact.

There are two main principles with lighting columns of achieving passive safety and these are:

- Breaking away at the base
- Flattening, yielding, distorting or degrading as the vehicle hits, absorbing the impact energy from the vehicle and not bringing it to a violent stop.

There are basically two types of passively safe (frangible) lighting columns in use:

- Collapsible column: normally constructed from a light material, aluminium, fibreglass or light gauge steel, and which shears off at the base or bends and collapses on impact. A slip or shear base is a particular (heavier) type that slips off the base when the column is struck, releasing the column from its foundation. Breakaway couplings are another form.
- Energy (or impact) absorbing column: generally where the column does not separate from its base but deforms in response to the vehicle impact and entraps the vehicle within acceptable limits of deceleration.

The slip base and collapsible column types have been used widely over the world. Slip base designs have the advantage that after impact; often the base (ground stub) can be re-used.

With slip base and collapsible lighting columns that are designed to break free on impact, however there is the risk they will form a further hazard to pedestrians or other vehicles. The behaviour of the falling column is determined by the speed of impact. There is general sector acceptance that where the speed of the impacting vehicle is above 35 km/h, the impacted column rotates above its centre of inertia and falls in the direction of the impacting vehicle and parallel to the roadway. Only in instances of speeds less than 35km/h is a falling column more likely to encroach on the roadway.

4.3 Safety Evaluation Standards for Passively Safe (Frangible) Lighting Columns

AS/NZS 4676 design standard for utility service poles is the only New Zealand standard that refers to *NCHRP Report 350 – Recommended procedures for the safety performance evaluation of highway features*. NCHRP Report 350 sets out the testing standards and procedures governing impact on highway support structures. The European Standard BS EN 12767 also provides a common basis for the testing of vehicle impacts with items of road equipment and reporting. This standard was developed further from the earlier work set out in the NCHRP Report. NCHRP 350 has also now been overtaken by the AASHTO Manual for Assessing Safety Hardware (MASH), which increases both the small test vehicle mass from 820kg to 1100kg and the angle of impact from 20 to 25 degrees.

With various authorities around the world now using similar (but not identical) testing standards and procedures for impacts on passively safe (frangible) lighting columns, all trying to achieve reduction in injury to the impacting occupants, it has been deemed that multiple similar standards can be used to determine acceptable passively safe lighting column design criteria for New Zealand roads.

The experience in New Zealand for slip or shear base lighting columns has been varied, but generally acceptable. These bases perform very well if hit at an angle, and the slip base mounting is at the correct level. However, there have been numerous instances where the slip base has not worked as envisaged, due to being hit in line or due to re-levelling of the surrounding ground leaving the base (ground stub) either too low or, more often, too high. If the base (ground stub) is too high, the impacting vehicle will also damage the base requiring a complete and expensive foundation replacement. Maintenance is critical with correct bolt torque checks required on a regular basis in accordance with the manufacturer's instructions – generally every 6 months and more frequently in areas with frequent strong wind gusts.

The generation of ground planted tapered sectional steel lighting columns in New Zealand have performed very well in service despite not been recognised or having undergone formal testing as "frangible lighting columns". With full ground planting, the lighting columns do collapse on impact and, in most instances, in a controlled manner. When tapered sectional steel lighting columns have suffered obvious whiplash under impact (often indicated by the loss of the luminaire) or have impacted the ground, experience indicates that the whole column should be completely replaced rather than just replace the damaged sleeved section following inspection.

There remains the problem of control of assembly of tapered sectional steel lighting columns where the sleeves overlap in the critical lower section (approximately up to 1m above ground) which then provides a thickened (and stiffened) zone, reducing the column's ability to absorb the impact and provide a progressive energy absorbing collapse.

Existing lighting columns as approved by the NZTA under the M19 Specification and currently in use in New Zealand being formed or rolled from steel sheet (up to 3mm wall thickness) or aluminium, of tubular segmental construction, are deemed to satisfy the passive safety (frangibility) requirements for the following performance classes 70:LE and 70:HE (for ground planted type) and 100:NE (for slip or shear base type) of this Specification. Refer Appendix A for specific details of the type approvals.

5. TYPE APPROVAL PROCESS

5.1 General

While the Type Approval process option has been allowed for in the specification, its applicability will mainly lie in applications for new designs or suppliers of passively safe (frangible) column types. It allows for an approval process which will enable review of the documentation on the impact testing regime and evaluation criteria, it will also ensure that any overseas product will be designed for the environmental conditions in New Zealand, both for loading and fatigue, requiring an engineering design statement though the Producer Statement (PS1) or compliance certificate.

6. DURABILITY

6.1 General

It is assumed that lighting columns should be able to reach their intended design life without maintenance. The level of protection required to achieve this will depend on both the specified minimum life and the environment in which the material is located.

The durability of galvanized steel components above ground is discussed in AS 2309 (which updates the requirements given in the steel pole design standard AS/NZS 4676) and also AS/NZS 2312. These indicate that the life to first maintenance of the protective coating is dependent on the atmospheric corrosivity classification for the local macro environment. This is largely governed by distance from the sea, but also local topography, wind strength and direction and average humidity levels. In addition, microclimates can exist, on surfaces that are sheltered from rain washing, in crevices in which salts can concentrate due to capillary action and evaporation, and on internal surfaces where condensed moisture is unable to drain away.

The life of ground planted steel lighting columns has often been limited by “ring-bark” corrosion occurring at the air/soil or air/concrete interface. This had been recognised in the previous TNZ M19 specification, which had required a continuous non conductive barrier coating over a 500mm zone at the interface, using epoxy-mastic or similar, of at least 150 microns thickness. The absence of this protective coating was the cause of early failure of lighting columns in New Zealand. This requirement is therefore maintained and the past experience recognises that this thickness should be increased to a minimum of 350 microns. For greater than a 25 year life, the ground planted exterior of galvanized lighting columns should be coated with 700+ microns of 100% VS polyurethane or polyurea coating, or a field proven equivalent.

7. ELECTRICAL REQUIREMENTS

7.1 Disconnection / Isolation of Supply

For installation requirements, reference should also be made to the local electrical network or territorial authority requirements.

ANNEX A: Fatigue Considerations

To assist in the fatigue design of steel and aluminium lighting columns and connections, the following tables and figures have been abstracted from the current edition of AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signs. The approach is similar to NZS 3404 but has more fatigue-sensitive connection details of the type that are typically used in lighting columns and outreach arms.

The following are included in this Annex:

- Table 11-2 Fatigue Details listings (and Notes)
- Table 11-3 Constant-Amplitude Fatigue Limits for steel and aluminium materials
- Figure 11-1 Illustrative Connection Details

Table 11-2—Fatigue Details of Cantilevered and Noncantilevered Support Structures

Construction	Detail	Stress Category	Application	Example
Plain Members	1. With rolled or cleaned surfaces. Flame-cut edges with ANSI/AASHTO/AWS D5.1 (Article 3.2.2) smoothness of 1000 μ-in. or less.	*A*	—	—
	2. Slip-joint splice where *L* is greater than or equal to 1.5 diameters.	*B*	High-level lighting poles.	1
Mechanically Fastened Connections	3. Net section of fully tightened, high-strength (ASTM A 325, A 490) bolted connections.	*B*	Bolted joints.	2
	4. Net section of other mechanically fastened connections: a. Steel: b. Aluminum:	 *D* *E*	—	3
	5. Anchor bolts or other fasteners in tension; stress range based on the tensile stress area. Misalignments of less than 1:40 with firm contact existing between anchor bolt nuts, washers, and base plate.	*D*	Anchor bolts. Bolted mast-arm-to-column connections.	8, 16
	6. Connection of members or attachment of miscellaneous signs, traffic signals, etc. with clamps or U-bolts.	*D*	—	—
Holes and Cutouts	7. Net section of holes and cutouts.	*D*	Wire outlet holes. Drainage holes. Unreinforced handholes.	5

Table 11-2 continued on next page

Construction	Detail	Stress Category	Application	Example
Groove Welded Connections	8. Tubes with continuous full- or partial-penetration groove welds parallel to the direction of the applied stress.	*B'*	Longitudinal seam welds.	6
	9. Full-penetration groove-welded splices with welds ground to provide a smooth transition between members (with or without backing ring removed).	*D*	Column or mast arm butt-splices.	4
	10. Full-penetration groove-welded splices with weld reinforcement not removed (with or without backing ring removed).	*E*	Column or mast arm butt-splices.	4
	11. Full-penetration groove-welded tube-to-transverse plate connections with the backing ring attached to the plate with a full-penetration weld, or with a continuous fillet weld around interior face of backing ring. The thickness of the backing ring shall not exceed 10 mm (0.375 in.) when a fillet weld attachment to plate is used. Full-penetration groove-welded tube-to-transverse plate connections welded from both sides with backgouging (without backing ring).	*E*	Column-to-base-plate connections. Mast-arm-to-flange-plate connections.	5
	12. Full-penetration groove-welded tube-to-transverse plate connections with the backing ring not attached to the plate with a continuous full-penetration weld, or with a continuous interior fillet weld.	*E'*	Column-to-base-plate connections. Mast-arm-to-flange-plate connections.	5
Fillet-Welded Connections	13. Fillet-welded lap splices.	*E*	Column or mast arm lap splices.	3
	14. Members with axial and bending loads with fillet-welded end connections without notches perpendicular to the applied stress. Welds distributed around the axis of the member so as to balance weld stresses.	*E*	Angle-to-gusset connections with welds terminated short of plate edge. Slotted tube-to-gusset connections with coped holes (see note e).	2, 6
	15. Members with axial and bending loads with fillet-welded end connections with notches perpendicular to the applied stress. Welds distributed around the axis of the member so as to balance weld stresses.	*E'*	Angle-to-gusset connections. Slotted tube-to-gusset connections without coped holes.	2, 6
	16. Fillet-welded tube-to-transverse plate connections (see note j).	*E'*	Column-to-base-plate or mast-arm-to-flange-plate socket connections.	7, 8, 16
	17. Fillet-welded connections with one-sided welds normal to the direction of the applied stress.	*E'*	Built-up box mast-arm-to-column connections.	8, 16
	18. Fillet-welded mast-arm-to-column pass-through connections.	*E'* (See note f)	Mast-arm-to-column pass-through connections.	9

Construction	Detail	Stress Category	Application	Example
	19. Fillet-welded T-, Y-, and K-tube-to-tube, angle-to-tube, or plate-to-tube connections.	(See notes a and b)	Chord-to-vertical or chord-to-diagonal truss connections (see note a). Mast-arm directly welded to column (see note b). Built-up box connection (see note b).	8, 10, 11
	25. Fillet-welded ring-stiffened box-to-tube connection.	(See note g)	Ring-stiffened built-up box connections.	16
Attachments	20. Longitudinal attachments with partial- or full-penetration groove welds, or fillet welds, in which the main member is subjected to longitudinal loading: $L < 51$ mm (2 in.): 51 mm (2 in.) $\leq L \leq 12t$ and 102 mm (4 in.): $L > 12t$ or 102 mm (4 in.) when $t \leq 25$ mm (1 in.):	 C D E	Reinforcement at handholes.	13
	21. Longitudinal attachments with partial- or full-penetration groove welds, or fillet welds in which the main member is subjected to longitudinal loading.	E'	Weld terminations at ends of longitudinal stiffeners (see notes h and i).	12, 14
	22. Detail 22 has been intentionally removed.			
	23. Transverse load-bearing fillet-welded attachments where $t \leq 13$ mm (0.5 in.) and the main member is subjected to minimal axial and/or flexural loads. (When $t > 13$ mm [0.5 in.], see note d.)	C	Longitudinal stiffeners welded to base plates.	12, 14
	24. Transverse load-bearing longitudinal attachments with partial- or full-penetration groove welds or fillet welds, in which the nontubular main member is subjected to longitudinal loading and the weld termination embodies a transition radius that is ground smooth: $R > 51$ mm (2 in.) $R \leq 51$ mm (2 in.)	 D E (See note c)	Gusset-plate-to-chord attachments.	15

Notes:

[a] Stress Category ET with respect to stress in branching member provided that $r/t \leq 24$ for the chord member. When $r/t > 24$, then the fatigue strength equals:

$$(F)_n = (\Delta F)_n^{ET} \times \left(\frac{24}{\frac{r}{t}} \right)^{0.7}$$

where:

$$(\Delta F)_n^{ET}$$

is the CAFL for Category ET.
Stress Category E with respect to stress in chord.

[b] Stress Category ET with respect to stress in branching member.
Stress Category K_2 with respect to stress in main member (column) provided that: $r/t_c \leq 24$ for the main member.

Notes continued on next page

When $r/t_c > 24$, then the fatigue strength equals:

$$(F)_n = (\Delta F)_n^{K2} \times \left(\frac{24}{\frac{r}{t_c}} \right)^{0.7}$$

where:

$(\Delta F)_n^{K2}$

is the CAFL for Category K_2.

The nominal stress range in the main member equals $(S_R)_{main\ member} = (S_R)_{branching\ member} (t_b/t_c)\ \alpha$
where t_b is the wall thickness of the branching member, t_c is the wall thickness of the main member (column), and α is the ovalizing parameter for the main member equal to 0.67 for in-plane bending and equal to 1.5 for out-of-plane bending in the main member. $(S_R)_{branching\ member}$ is the calculated nominal stress range in the branching member induced by fatigue design loads. (See commentary of Article 11.5.)

The main member shall also be designed for Stress Category E using the elastic section of the main member and moment just below the connection of the branching member.

c First check with respect to the longitudinal stress range in the main member per the requirements for longitudinal attachments. The attachment must then be separately checked with respect to the transverse stress range in the attachment per the requirements for transverse load-bearing longitudinal attachments.

d When $t > 13\ mm$ (0.5 in.), the fatigue strength shall be the lesser of Category C or the following:

$$(\Delta F) = (\Delta F)_n^c \times \left(\frac{0.094 + 1.23\frac{H}{t_p}}{t_p^{\frac{1}{6}}} \right) (\text{MPa}) \qquad (\Delta F) = (\Delta F)_n^c \times \left(\frac{0.0055 + 0.72\frac{H}{t_p}}{t_p^{\frac{1}{6}}} \right) (\text{ksi})$$

where $(\Delta F)_n^c$

is the CAFL for Category C, H is the effective weld throat (mm, in.), and t_p is the attachment plate thickness (mm, in.).

e The diameter of coped holes shall be the greater of 25 mm (1 in.), twice the gusset plate thickness, or twice the tube thickness.

f In addition to checking the branching member (mast arm), the main member (column) shall be designed for Stress Category E using the elastic section of the main member and moment just below the connection of the branching member (mast arm).

g Stress Category E′ with respect to stress in branching member (ring-stiffened built-up box connection). The main member shall be designed for Stress Category E using the elastic section of the main member and moment just below the connection of the branching member.

h Only longitudinal stiffeners with lengths greater than 102 mm (4 in.) are applicable for Detail 21. On column-to-base-plate or mast-arm-to-flange plate socket connections having a wall thickness greater than 6 mm (0.25 in.) that have exhibited satisfactory field performance, the use of stiffeners having a transition radius or taper with the weld termination ground smooth may be designed at a higher stress category with the approval of the Owner. Under this exception, the Owner shall establish the stress category to which the detail shall be designed. See commentary for Article 11.5.

i Nondestructive weld inspection should be used in the vicinity of the weld termination of longitudinal stiffeners. Grinding of weld terminations to a smooth transition with the tube face is not allowed in areas with fillet welds or partial-penetration welds connecting the stiffener to the tube. Full-penetration welds shall be used in areas where grinding may occur. See commentary for Article 11.5.

j Fillet welds for socket connections (Detail 16) shall be unequal leg welds, with the long leg of the fillet weld along the column or mast arm. The termination of the longer weld leg should contact the shaft's surface at approximately a 30° angle.

Table 11-3 Constant-Amplitude Fatigue L imits

Detail Stress Category	Steel (MPa)	Aluminium (MPa)
A	165	10.2
B	110	6.0
B’	83	4.6
C	69	4.0
D	48	2.5
E	31	1.9
E’	18	1.0
ET	8	0.44
K_2	7	0.38

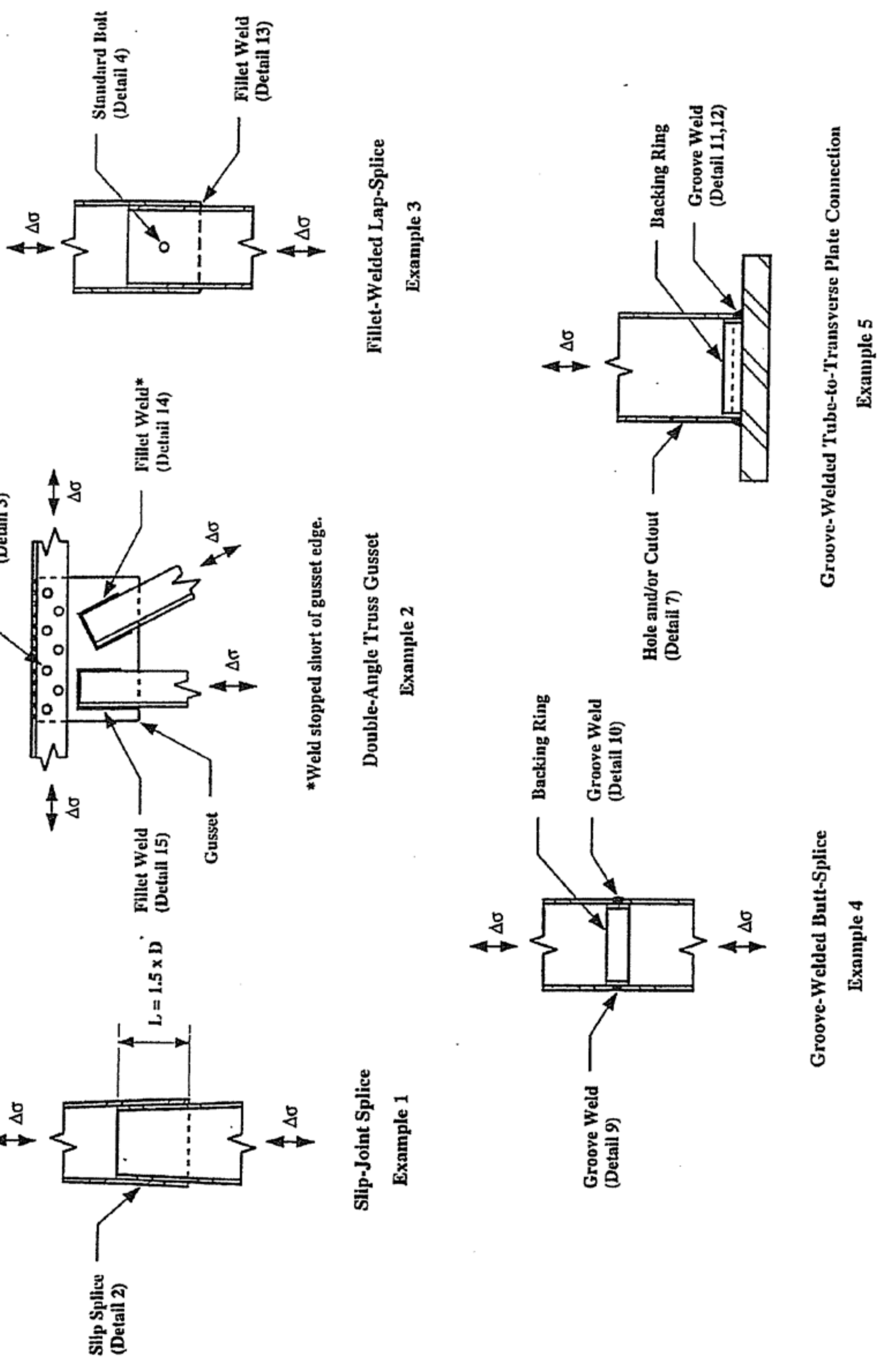
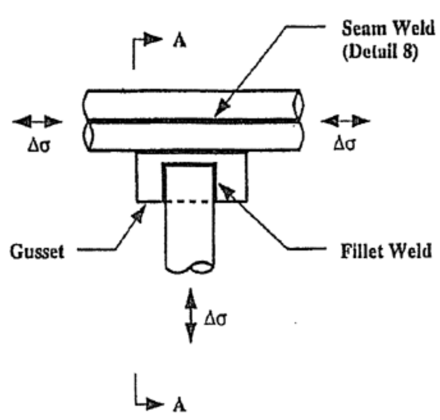


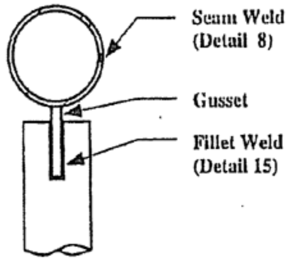
Figure 11-1—Illustrative Examples

Continued on next page



Slotted Tube-to-Gusset Connection

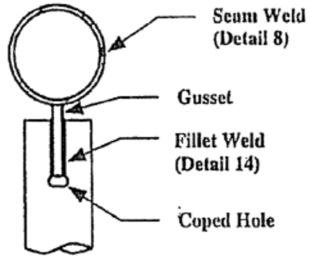
Example 6



Section A-A

Slotted Tube-to-Gusset Connection

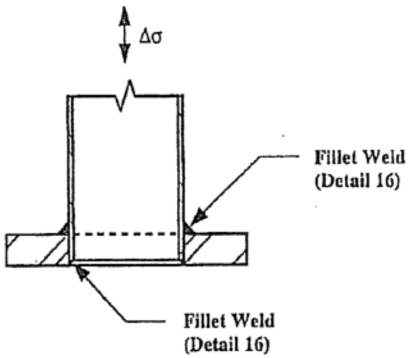
Example 6



Section A-A

Slotted Tube-to-Gusset Connection

Example 6

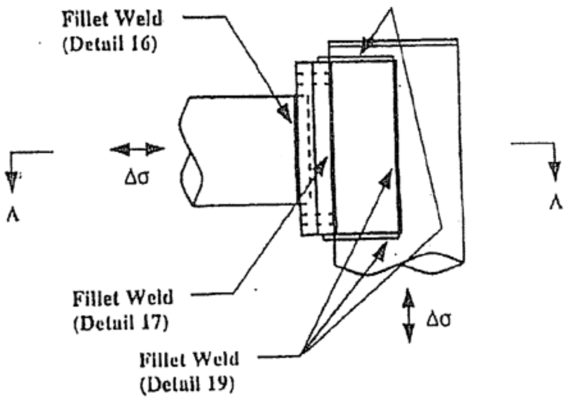


Fillet-Welded Socket Connection

Example 7

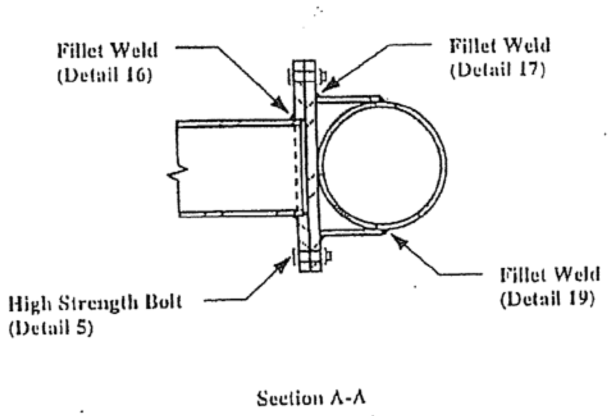
Figure 11-1—Illustrative Examples—*Continued*

Continued on next page



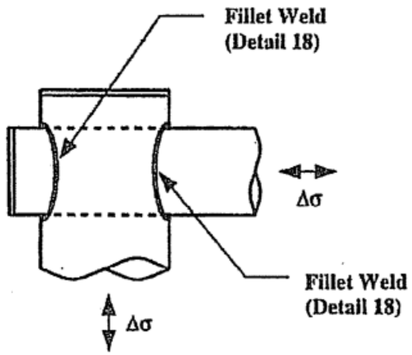
Fillet-Welded Mast-Arm-to-Column Connection
(Built-Up Box)

Example 8



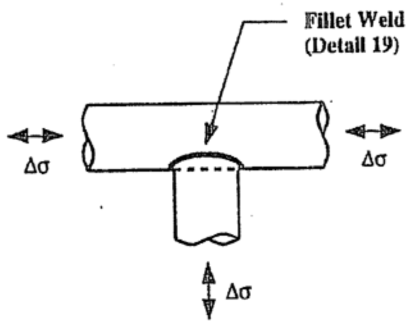
Fillet-Welded Mast-Arm-to-Column Connection
(Built-Up Box)

Example 8

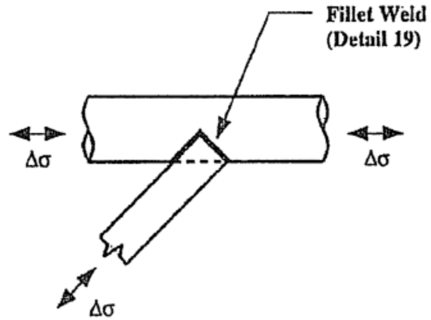


**Fillet-Welded Tube-to-Tube Column
Pass-Through Connection**

Example 9



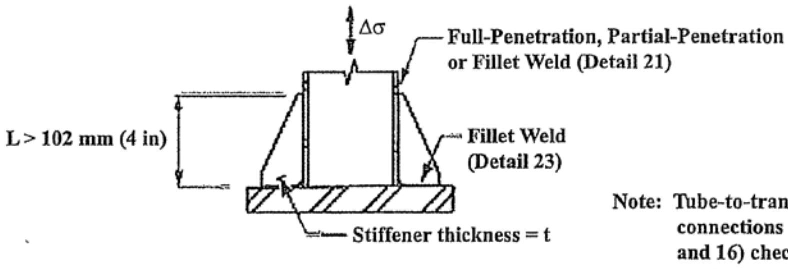
Fillet-Welded Tube-to-Tube Connection
Example 10



Fillet-Welded Angle-to-Tube Connection
Example 11

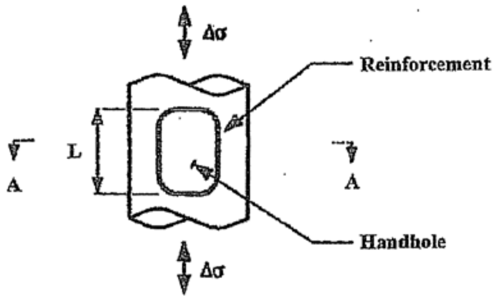
Continued on next page

Figure 11-1—Illustrative Examples—*Continued*



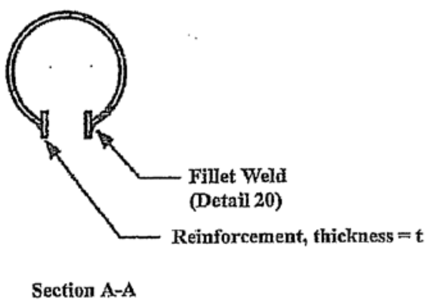
Note: Tube-to-transverse plate connections (Details 11, 12, and 16) checked using combined moment of inertia of tube and stiffeners

Longitudinal Attachment
Example 12



Reinforced Handhole

Example 13



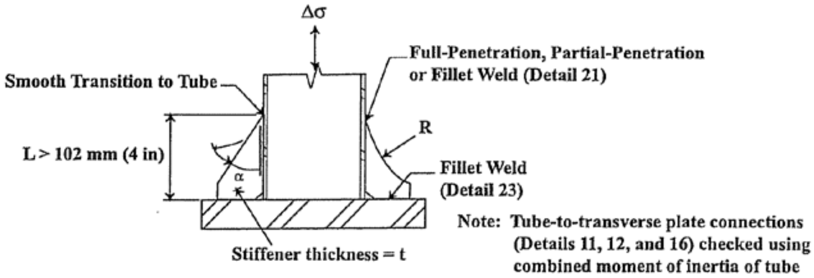
Section A-A

Reinforced Handhole

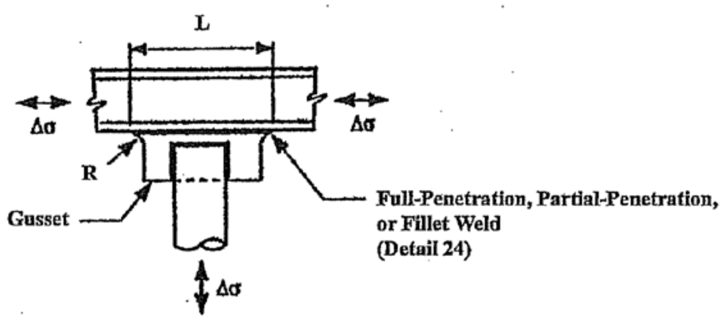
Example 13

Continued on next page

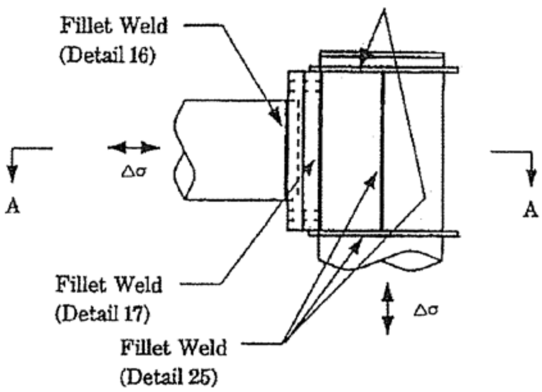
Figure 11-1—Illustrative Examples—*Continued*



Longitudinal Attachment
Example 14

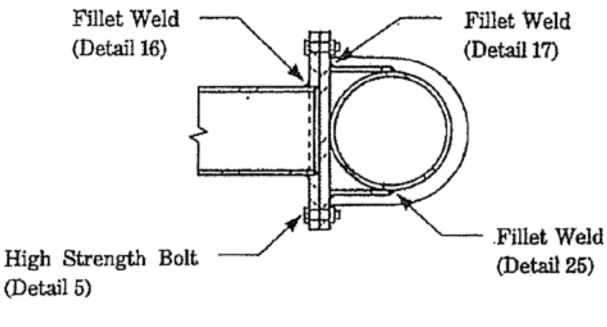


Transverse Load-Bearing Longitudinal Attachment
Example 15



Fillet-Welded Mast-Arm-to-Column Connection
(Ring-Stiffened Built-Up Box)

Example 16



Section A-A

Fillet-Welded Mast-Arm-to-Column Connection
(Ring-Stiffened Built-Up Box)

Example 16

Figure 11-1—Illustrative Examples—*Continued*