TUBULAR CONVEYOR GALLERY

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ABSTRACT

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TUBULAR CONVEYOR GALLERY

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A recent innovation in elevated conveyor galleries is the tubular conveyor gallery.

The tubular conveyor gallery consists of a thin-walled tube of large diameter, whose function is to support and house the conveyor and walkway.

This major technical report concerns itself with the description of tubular conveyor galleries, their advantages and disadvantages, and their structural analysis and design.

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NOTATIONS

NOTATION

The following list defines the principal symbols used in this major technical report. For convenience of reference, these are summarized for each chapter individually. Other symbols are defined in the text.

CHAPTER III

A	area of cross-section of tube
B.B.	braced bent
C_e	exposure factor
C_g	gust effect factor
C_n	wind force coefficient
c	corrosion allowance
D	inside diameter of the tube
D_o	outside diameter of the tube
F_y	yield strength of material
I_p	polar moment of inertia
L	span between inverted V-bent and single post support
M_v	moment caused from vertical loads
M_H	moment caused from horizontal loads
$n = (\frac{D}{D_o})$	ratio of inside diameter to outside diameter of tube
q	reference velocity pressure

R	inside radius of the tube
r	radius of gyration of cross-section of tube
S.P.	single post support
$S_x = S_y = S$	section modulus of cross-section of tube
t	thickness of the shell
V.B.	inverted V-bent support

CHAPTER IV

A	area of intermediate circumferential ring
M	circumferential bending moment
R	radius to the centroid of circumferential ring
S	section modulus of intermediate circumferential ring
T	axial circumferential force
V	circumferential shear
v	tangential shear

CHAPTER V

A	area of combined section
a	eccentricity of the support
b	width of ring girder
C	total effective width
M_T	reaction torque of tubular gallery over the inverted V-bent support

Q	vertical force reaction of tubular gallery over the support
R	radius to the centroid of combined section
S	section modulus of combined section
v	tangential shear
W	wind reaction (horizontal force) of tubular gallery over the inverted V-bent support

CHAPTER I

INTRODUCTION

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INTRODUCTION

A conveyor system is a continuous system of haulage of bulk materials. It enables us to haul anything from corn to iron ore in a continuous flow from one end of the plant to the other, feeding and reclaiming material through the different stages of process. Conveyors, besides being a major component of large material handling machines, and a vital part of almost all modern industrial plants, enable us to haul bulk material such as corn from one site to the other.

Up to now, railroads and trucks have been employed to fulfill this function, which is by nature, modular. But, if a continuous system (such as conveyors) can be employed (which, in addition, are electrically powered), the capacity and timing due to automation, will be improved and the ultimate ecological and environmental benefits will be provided, since the system is void of exhaust emissions associated with conventional means of transportation. The function would be similar to that of a pipeline.

1.1 SUPPORTING STRUCTURES OF BULK MATERIAL HANDLING SYSTEMS

It is axiomatic that in designing a bulk material handling system, the primary consideration is the functioning of the machines. The structure is secondary, its reasons for being are to support, enclose and provide access to the machine for servicing. This results in a wide variety of supporting structures, which can be classified into two main categories, depending on the height of the belt from grade :

(a) Supporting structures for a LEVEL CONVEYOR, where the belt line is three and a half to four feet above "grade". "Grade" may be a timber trestle, a steel floor, a concrete slab indoors, or in a tunnel, a series of concrete footings, or even railroad ties with or without ballast. Usually, standard channels are employed with the flanges turned in toward the center of the belt, supporting the idlers.

(b) Supporting structures for ELEVATED CONVEYORS where the height can range from five feet above "grade" to over one hundred. Spans may range from 30 ft to more than 500 ft, and spans of 1,000 ft have been considered. This category (elevated conveyor galleries) is the most important and interesting from the structural point of view.

1.2 STRUCTURAL ASPECTS OF ELEVATED CONVEYOR GALLERIES

There are several variations of supporting structures for conveyor galleries. The structural engineer of today, in choosing among those variations, has to pay considerable attention to some factors, whose influence in industry is of continuous growth and importance. Sharply rising costs of labor, materials and plant maintenance, compel the engineer to design plant systems and structures which will require a minimum of materials and labor for manufacture, installation and operation, and which will be maintenance-free to a maximum degree. Also, the aesthetic value of industrial complexes should be considered. Industrial communities, once conglomerations of functional structures, are now paying more attention to the aesthetic appeal of their structures.

Stricter regulations and laws for ecological and pollution control have also to be tackled.

Up to now, the most common type of conveyor supporting structure is the CONVENTIONAL ENCLOSED CONVEYOR GALLERY. It generally employs two vertical trusses to carry the dead and live loads of the conveyor and walkways and any vertical wind load due to uplift on the gallery. Bracing in the horizontal plane to take wind loads and other lateral forces is provided by horizontal trusses connected to the vertical trusses at their top and bottom chord.

A more recent innovation in conveyor galleries is the TUBULAR CONVEYOR GALLERY (of circular or elliptical cross-section). On the basis of the comparison made between the conventional enclosed gallery and the tubular gallery, for the support and housing of conveyor handling bulk material, the tubular gallery, supported alternately on inverted V-bents and single pipe posts, offers advantages over the conventional gallery and its supports.

On the other hand, the tubular conveyor gallery, by permitting simplified design and detailing, maximum shop fabrication using modern welding techniques, rapid field erection, and little maintenance, is in accord with the foregoing factors and requirements of modern industry which are the major reasons for the prevalence of this concept.

This report concerns itself with the description, stating the advantages and disadvantages, and the structural behavior of tubular conveyor galleries.

CHAPTER II

THE TUBULAR CONVEYOR GALLERY

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THE TUBULAR CONVEYOR GALLERY

2.1 DESCRIPTION

The tubular conveyor galleries (Figures 2.1 and 2.2) consist of a tube, shop fabricated of steel plates formed to the required radius and welded at longitudinal and circumferential joint sections in a staggered pattern. Structural grade steel, ASTM A-36 is used for most cases, although ASTM A-242 steel for corrosion and alloy steels for reduction in dead weight can be used when required.

For spans ranging from 60 ft to 90 ft and for ASTM A-36 steel, the usual thickness of plate is 1/4 inch. For this range of span, the diameter of the cross-section is not dictated mainly by structural reasons, but by the requirement of providing sufficient space to accommodate the belt conveyor and continuous walkway. Usually, for belt conveyors ranging in width from 24 inches to 54 inches, and a continuous walkway on one side of the belt of 2'-6", a diameter of 10 ft will be sufficient.

Inside the tube on the invert, lightweight concrete is usually applied to form a sluiceway down which spillage from the conveyor belt can be washed. Sluiceway concrete

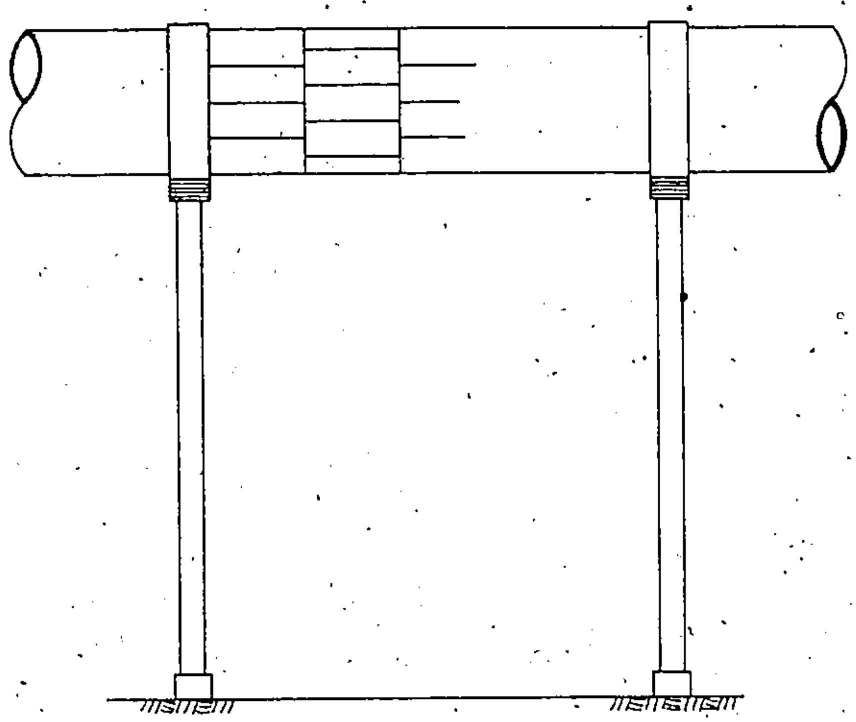


FIG. 2.1 Elevation of tubular conveyor gallery

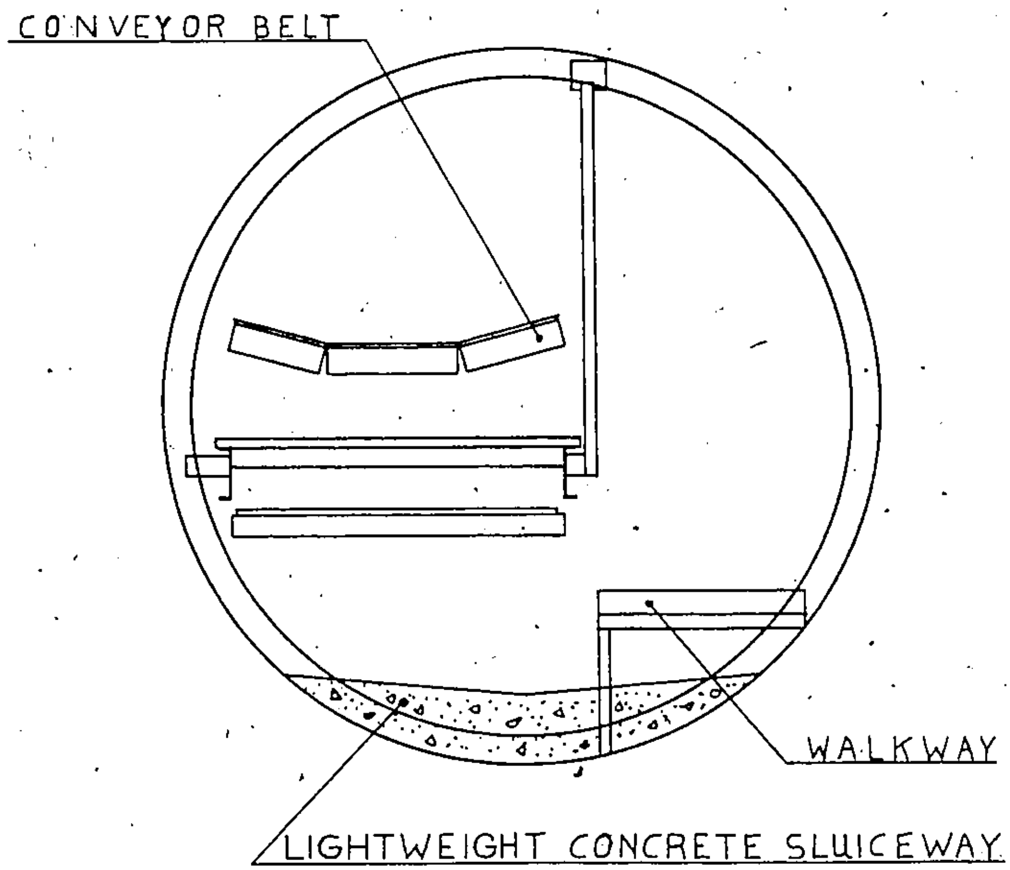


FIG. 2.2 Cross-section of tubular conveyor gallery

is to be applied after the gallery is in place, to avoid cracking resulting from handling stresses during erection. Epoxy compound is usually applied to the steel surfaces prior to placing the concrete, to insure adhesion of concrete to steel, and after the concrete has been cured, to the wearing surfaces of the concrete, to seal the surface and resist abrasion. An 8 inch to 12 inch drain pipe is provided at the lower end of the sluiceway to carry spillage slurry to disposal areas.

The walkway is located on one side of the conveyor, about 3'-6" below the belt-line, and a couple of feet above the sluiceway. The width usually is 2'-6" and a floor of grating, checkered plate, or expanded metal is used. Expanded metal turned in the right direction makes an excellent non-skid footing on sloping galleries. A channel (preferably C10) on the side of the walkway towards the conveyor, can serve both as support and kick-plate. Because of the small height (a couple of feet) of the walkway above the invert of the tube, no handrailing is necessary.

Thermal insulation is required for galleries housing conveyors for handling moist concentrate, at locations of very low winter temperatures. Blanket-type fiberglas insulation, 3 inches thick, can be strapped to the exterior surface of the gallery, and a 24-gauge galvanized metal protective sheathing then applied. Heaters are usually employed to blow hot air into the gallery at its lower end.

Thus, from the above description, we come to the conclusion that the tubular conveyor gallery, by its shape, fulfils at the same time, three functions.

(1) It acts as a structural member (as a beam of hollow structural section) carrying the conveyor above ground between supporting bents.

(2) It houses the conveyor and walkway to protect them against weather (no additional roofing or siding needed).

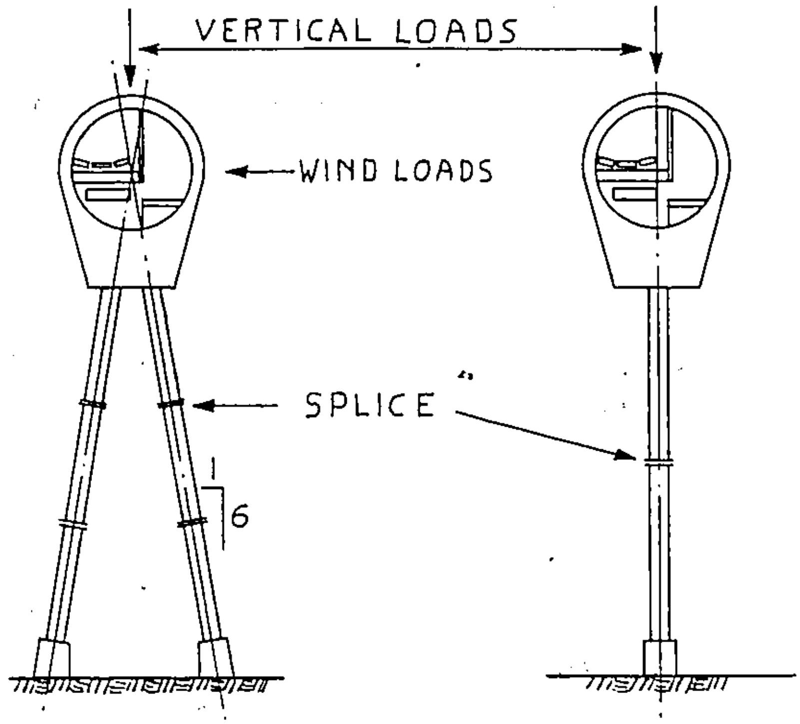
(3) It provides a sluiceway, by means of its concrete paved invert, down which spillage can be flushed (maintenance costs decreased).

2.2 SUPPORTS FOR TUBULAR GALLERIES

The superior wind shape factor and the uniform bending strength of the tubular gallery section against horizontal and vertical loads, permit the use of a design whereby two types of gallery support, both fabricated of steel pipe may be used. The first type of support is the inverted V-bent (Figure 2.3a) and is capable of taking both horizontal and vertical loads. The second type of support is a single pipe post (Figure 2.3b) taking vertical loads only. The V-bents are alternated with the single post supports. The spacing between alternate bents averages approximately 160 feet, and between adjacent supports, approximately 80 feet.

2.2.1 The Inverted V-Bent

Bents are normally made with two legs, generally battered with a slope of 1 in 6. The inverted V-bent is designed so that the axis of the two pipe legs extended will intersect at the point where the vertical dead and live loads of the gallery and the horizontal wind load intersect. (Figure 2.3a). All loads are thus transmitted via the pipe legs to concrete footings, as compression and tension forces. The pipe legs are subject to a minimum of bending imposed by the eccentrically located loads of the walkway, walkway traffic, conveyor and conveyor material, and uneven snow accumulations. The V-bents are shop-fabricated of pipe sections, with sizes ranging from 12 inches (49.56 pounds per foot) to 22 inches (72.3 pounds per foot) in lengths of up to 100 feet. They are delivered to the site with a base plate welded to the lower end of each pipe leg, for connection to anchor bolts embedded in concrete footings. (Figure 2.4) At the upper end, a seat weldment is provided, for the field-bolted connection to a weldment



(a) The inverted V-bent support

(b) The single post support

FIG. 2.3 Supports for tubular conveyor

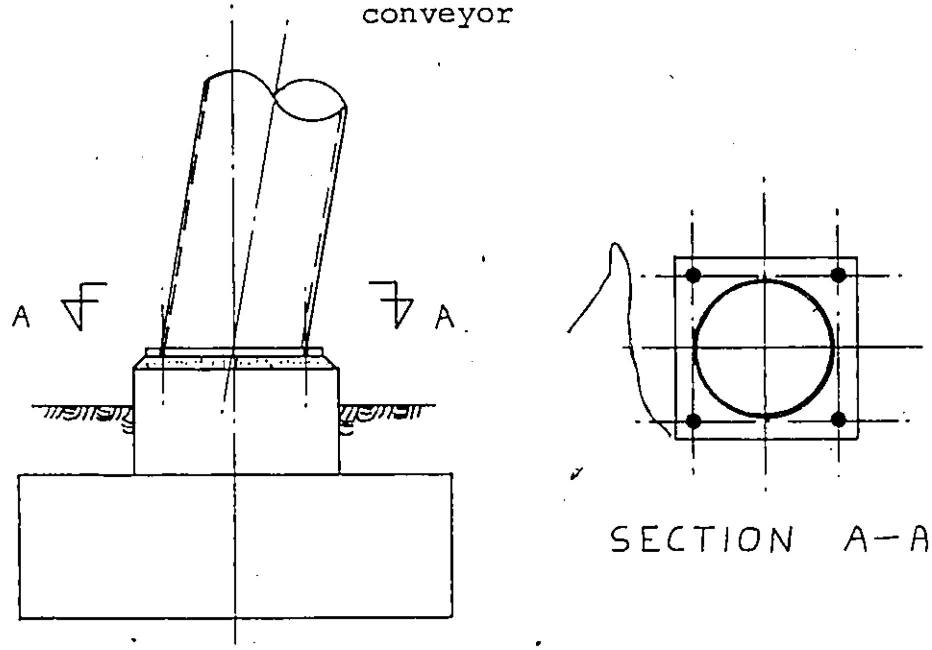


FIG. 2.4 Base plate details

on the underside of the tubular gallery section (Figure 2.5).

2.2.2 The Single Post Support

The single tubular posts, located at intermediate points between the V-bents, are designed to take vertical loads only and are provided with base plates and seat weldments in a manner similar to that described for the inverted V-bents. The sections are compatible to those of the inverted V-bents (Figure 2.3b).

2.3 ERECTION OF TUBULAR GALLERIES

Nearly all of the steel is shop-welded and field-bolted using high strength bolts. Normally, the work is shop assembled into the largest pieces which can be economically shipped and handled in the field.

Sections of the tubular gallery, of 60 ft to 80 ft in length can be shop assembled and conveniently shipped to the job site, together with the V-bents and single post supports which are also shop assembled. Lugs are provided on the exterior surfaces of the tube during fabrication, for convenience in lifting during erection. Usually, the tube is shipped in lengths sufficient to span the distance between supports. For larger spans, sections of the tubular gallery in lengths smaller than the span are shipped to the

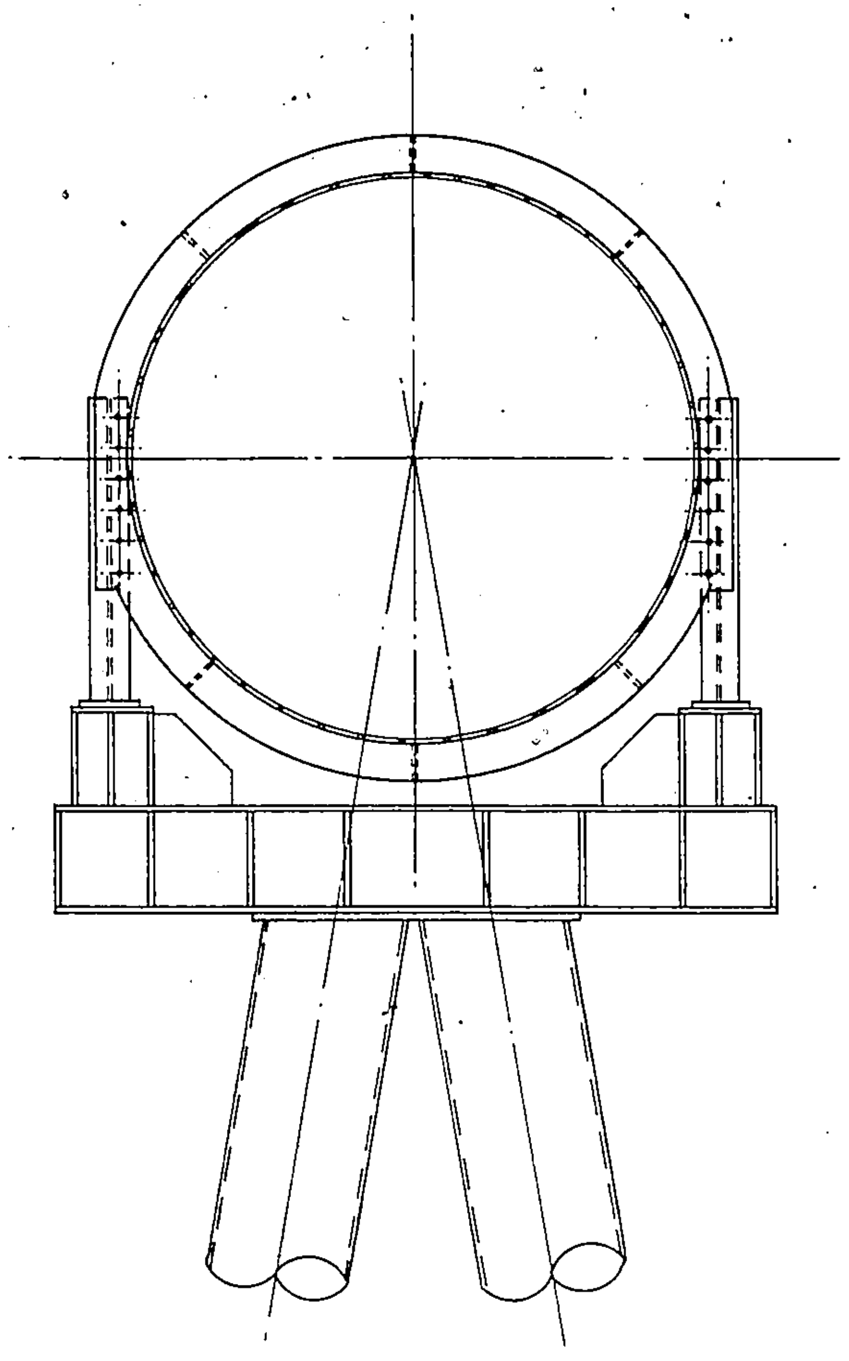


FIG. 2.5 Seat weldment over inverted V-bent

site, and there two or more sections are bolted together to form a span's length.

The conveyor components are field installed within the tube prior to erection.

In erecting the structure, V-bents and single pipe posts are lowered on and secured to anchor bolts embedded in the footings. Sections of the tubular gallery, complete with conveyor components mounted in place, of sufficient length to span the distance between a V-bent and adjacent single tubular post, are then lifted into place and secured to supports (seats) by bolts.

2.4 THE CONVENTIONAL ENCLOSED CONVEYOR GALLERY

For the sake of realizing the advantages of the tubular conveyor gallery over the conventional gallery and its supports, which is the most common type in industry (has been extensively tried and generally accepted up to now), a brief description of the conventional enclosed conveyor gallery will follow.

The conventional enclosed conveyor gallery (Figure 2.6) as usually constructed, consists of two vertical steel trusses installed parallel to each other to carry the gravity loads of the conveyor and walkway. The vertical trusses are connected at their top and bottom chords by framing, to form two horizontal truss systems designed to take the wind load.

The vertical trusses are installed a sufficient distance apart to accommodate the belt conveyor and a parallel walkway extending the entire length of the conveyor. The trusses are enclosed with sheet metal siding and roofing, with the roofing at sufficient height above the walkway to provide headroom and working space. In cold climates, bottom closure plates may be installed and the conveyor gallery enclosure may be insulated, to protect equipment and to prevent freezing and ensure the free flow of wet materials. Insulation may be of the blanket type sandwiched between metal panels.

The box-type gallery formed by this construction is supported by braced bents installed at intervals as required to transmit vertical and horizontal loads to concrete foundations.

The maintenance of the conveyor, including belt inspection and idler lubrication, deck clean-up, and idler replacement, as necessary, is performed from the continuous walkway. Spillage from the conveyor belt to the floor and walkway is returned to the belt manually.

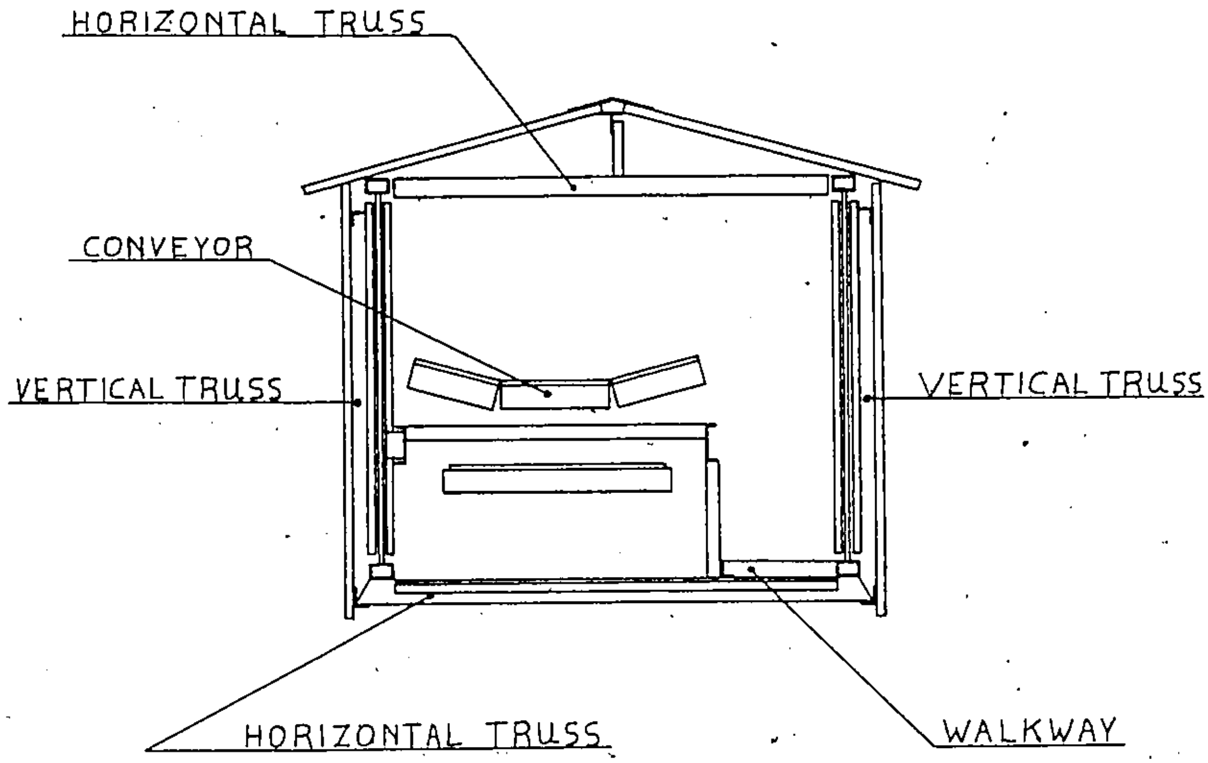


FIG. 2.6 The conventional enclosed conveyor gallery

2.5 ADVANTAGES OF THE TUBULAR GALLERY

2.5.1 Structural Advantages

The tubular cross-section has a superior wind shape factor as compared to that of the conventional rectangular section. As a result, the wind load per linear foot on the tubular section will be approximately 40% less than the wind load on a conventional enclosed gallery of dimensions to provide equivalent working space. This lesser wind load is a favorable factor in the design of gallery supports and foundations.

The tubular section has the same moment of inertia in all directions and provides equal resistance to both horizontal and vertical loads. This uniform moment of inertia, or bending strength, makes it possible to carry wind loads over long spans, using inverted V-bent and single post supports alternately.

The tubular section is an ideal section for resistance to the torsional stresses imposed by the eccentrically located loads of the walkway and conveyor.

The tubular section is an ideal column section for taking compression forces imposed by conveyor belt pull. For inclined galleries rising from underground structures and entering buildings, this element may be used to advantage by installing the conveyor head pulley (traction forces of the order of 30 kips or more) within the tubular section, instead of on building structures. All longitudinal forces are thus transmitted through the tubular gallery section to the foundation structure at grade, and the necessity for bracing building structures to take conveyor pull is avoided. In this design, a rolling or sliding bearing is provided between building structure and conveyor gallery to eliminate the transfer of stresses from one structure to the other. This method of transmitting belt pull to foundations was found to be especially advantageous in the design of industrial plants located in an earthquake zone. In this case, the building structures are usually of non-braced, rigid-frame design to provide the flexibility of structures desirable in earthquake zones.

The tubular gallery by nature (being a beam of hollow tubular cross-section, having a uniform and substantial bending strength), by employing inclined inverted V-bent supports, permits the use of cantilever construction (Figure 2.7), in order to eliminate interference with stockpile operations. Sometimes, the concrete foundations of the tubular gallery have to be located well outside of the live portion of the stockpile, or foundations of clinker storage. In this case, the tubular section can be canti-

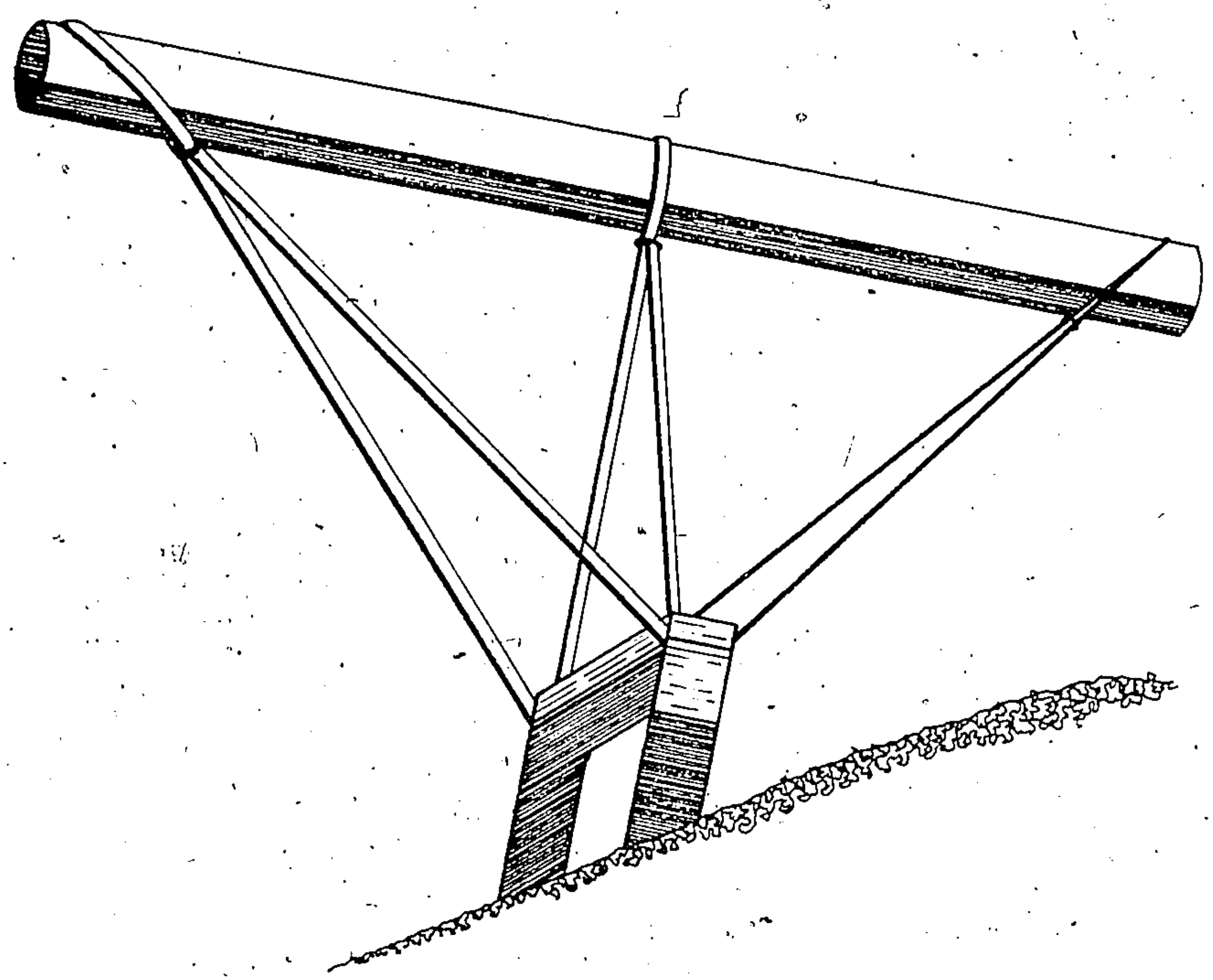


FIG.2.7 Cantilever construction

levered up to a considerable span (80 ft to 120 ft) beyond its concrete foundations.

In areas where snow loads influence design, snow buildup on the tubular gallery will be minimal, as compared with that on the rectangular gallery of the same width.

2.5.2 Advantages as an Enclosure

The tubular gallery functions as a very rugged enclosure for the conveyor. As compared with the conventional gallery with sheet metal roofing and siding, the tubular enclosure is far more resistant to damage by spillage of heavy ore lumps from the conveyor belt and from damage by mobile equipment.

By virtue of its proper construction, the tubular gallery minimizes both the entrance of outside air and heat losses from within the structure. This is an advantage in cold climates.

2.5.3 Maintenance Advantages

A 4-inch thick layer of lightweight concrete, applied to the interior lower portion of the tube forms a sluiceway for flushing away material spilled from the conveyor belt. A drain line, located at the bottom of the sloping section of the gallery, will carry the slurry to a convenient disposal area, for periodic reclaiming by mechanical equipment. All reasonably fine materials spilled from belts may be removed periodically by this method. Large accumulations of spilled materials and coarser sizes not susceptible to removal by flushing, must be removed by manual methods.

The tubular section, free of inaccessible corners, angles, and projections, presents a minimum of surface area to be painted. The smooth exterior and interior surfaces of the gallery will remain cleaner during operation and will require far less surface preparation prior to repainting than the conventional gallery. The hollow structural section of the gallery, with its reduced surface area, free of pockets where dirt may collect, and closed shape, has a lower susceptibility to corrosion than the conventional sharp-edged section. The tube, therefore, can result in lower costs for painting, galvanizing and other corrosion protection methods.

Lugs may be provided on the exterior surfaces of the gallery during fabrication, for convenience of suspending painters' scaffolds.

2.5.4 Advantages in Detailing, Fabrication and Erection

Less time is required for the steel detailing for the tubular gallery, because of its uniform cross-section and the minimal number of connections required between adjacent components.

In a shop specializing in steel plate construction, fabrication of the tubular gallery is simple, using modern welding techniques. In keeping with current trends in the construction industry, the tubular construction lends itself to maximizing shop prefabrication and assembly so that expensive field labour can be minimized. Sections 60 ft to 100 ft in length can be shop assembled, and conveniently shipped to the field, ready for installation of conveyor components and subsequent erection.

Field erection time for the tubular gallery is less than that required for the conventional gallery.

The uniform cross-section, simplicity of construction, and durability of the tubular gallery, render it readily adaptable to modification at reuse.

2.5.5 Advantages in Appearance

In examining the Figures of the tubular gallery structures presented herewith, it is evident that the clean, simple lines of the tubular gallery and its supports present an appearance greatly superior to that of the conventional enclosed conveyor gallery. While this factor may not be of significance in some locations, it will be weighed carefully by management willing to create a favorable public image, particularly by those whose plants border on parkways, residential areas, and other areas in the public view.

2.5.6 Advantages of Inverted V-Bents and Single Post Supports

The advantages of the V-bent and single post support structure for conveyor galleries over the conventional type support structure include the following:

(a) The supports constitute a minimum of obstruction to plant operations. Mobile equipment and plant personnel may move with ease between the legs of V-bents and around single posts.

(b) The supports offer a minimum of obstruction to snow removal.

(c) The single post support requires a very small foundation.

(d) The heavy pipe sections used are less vulnerable to damage by mobile equipment than are supports with bracing of the conventional type.

(e) Less frequent painting maintenance is required for the supports for the same reasons given for the maintenance of the tubular gallery section.

2.6 DISADVANTAGES OF THE TUBULAR GALLERY

(a) While the conventional enclosed gallery may be easily daylighted by windows provided in the siding, (louvre type) and will require electric lighting during night hours only, the tubular gallery is not conveniently adaptable to this treatment and will require electric lighting 24 hours, a day.

(b) Conveyor belt take-up construction, if required to be located within the tubular gallery, will be more complicated than that required for the conventional gallery.

(c) A factor that must be taken into consideration in designing long-span tubular galleries is the effect of the sun's heat on the tube, one side of which may be in full sun while the other side is in full shade, with consequent distortion of the tube, to a degree where it may affect the conveyor operation. The temperature differential between the shaded and unshaded surfaces of the tube may be considerably reduced by the application of a paint selected for maximum reflective values.

2.7 COST COMPARISONS

The calculation of cost presented herewith compares the cost of the tubular gallery for bent conveyors, supported on V-bents and single posts, with the cost of the conventional conveyor gallery and supporting structure.

In the tabulation, the total cost of the tubular gallery structure is given a value of 100, with each major component assigned its relative value in making up this total. For the conventional gallery, relative values are assigned to each major component of the structure. The totals of these values may then be compared with the index value of 100 assigned to the tubular gallery, to determine the relative costs of the structures.

The values represent the cost of the structures, in place. The cost of conveyor components and belting is not included, since the conveyor costs would be the same, regardless of the type of supporting structure. The tabulation shows the cost of conventional galleries, both with and without a bottom closure plate.

TABLE 2.1 COST COMPARISON

Tubular Gallery versus Conventional Gallery			
	Tubular Gallery	Conventional Gallery Without Closure Plate	Conventional Gallery With Closure Plate
Steel for Gallery	76	59	71
Bents, Posts and Foundations	22	30	30
Siding and Roofing	--	14	14
Concrete for Sluiceway	2	--	--
TOTALS	100	103	115

CHAPTER III

ANALYSIS AND DESIGN RECOMMENDATIONS

CHAPTER III

ANALYSIS AND DESIGN RECOMMENDATIONS

3.1 LOADS

According to CSA Standard S 16 [19] the structure must be checked for various load combinations, such as:

(a) DL + LL

(b) 0.75 (DL + LL + W) or 0.75 (DL + LL + E)

(c) DL + W or DL + E.

Loading conditions vary in different projects, but the following general data must be considered.

3.1.1 Dead Load

(a) Own Weight of tubular structure = δ_t lbs/ft.

$$\delta_t = \pi D t \rho_s + \frac{NW_{st}}{L} \tag{3.1}$$

D = inside diameter of the tube, in.

t = thickness of the wall, in

ρ_s = unit weight of steel shell, 0.2833 lb.per in^3

N = number of stiffeners (in cross-section)

W_{st} = weight of one stiffener, lbs.

L = span between inverted V-Bent and Single Post, ft.

(b) Weight of mechanical Equipment = δ_e lbs/ft.

δ_e = belts + idlers + pulleys + drive + switches + wiring + other

(c) Weight of walkway = δ_w lbs/ft.

δ_w = checkered plate, grating, expanded metal + stringers + posts.

(d) Weight of lightweight concrete on the invert = δ_c lbs/ft.

$$\delta_c = 120\left(\frac{\pi a}{180} - \sin\alpha\right) \frac{D^2}{1152}$$

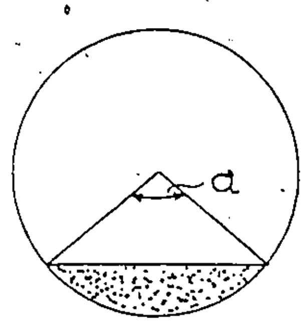


FIG. 3.1 Lightweight Concrete on the Invert

3.1.2 Live Load

(a) Live load due to the material on the belt = δ'_e lbs/ft.

No allowance is made for the impact and due to the bouncing of lumps as they move along the belt.

(b) Live load on walkway = δ'_w lbs/ft.

Normally, it is taken at 100 lbs/ft^2 locally, but 25 lbs/ft^2 for the span.

(c) Icing and snow (if applicable) = δ'_{i+s} lbs/ft.

(d) Belt pull during starting, stopping or running = P, lbs.

(P is a longitudinal force).

3.1.3 Wind Load or Earthquake Load

Wind pressure should be considered as acting in any direction. The design wind pressure W lbs/ft is specified by the Canadian Structural Design Manual, [14]. W is usually a horizontal force. The wind load per linear foot on the tubular section (because of the shape of the cross-section) will be less than the wind load on a conventional enclosed gallery of dimensions to provide equivalent working space. The Wind Force Coefficient, C_n, for tubular and conventional galleries (from Figures B-11 and B-6 of reference [14]) are presented schematically, in Figure 3.2.

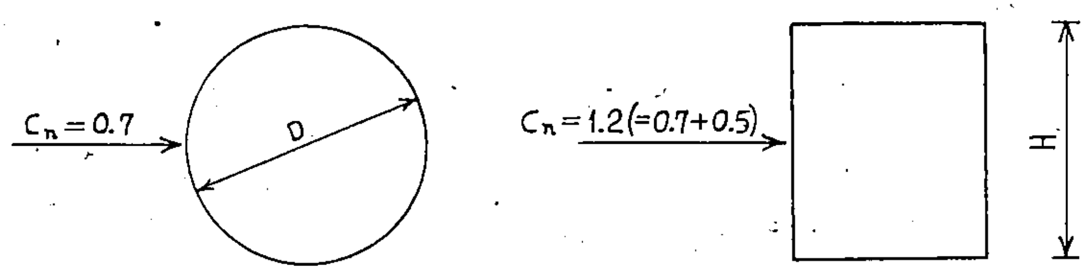


FIG. 3.2 Wind Force Coefficient, C_n for Tubular and Conventional Galleries

Total force per linear foot:

$$F = C_n \cdot q \cdot C_g \cdot C_e \cdot D, \quad \text{lbs/ft} \qquad (3.2)$$

Coefficients q, C_g, C_e are the same for both types of galleries.

Tubular gallery:

$$F = 0.7 \cdot q \cdot C_g \cdot C_e \cdot D, \quad \text{lbs/ft} \qquad (3.3)$$

Conventional Gallery:

$$F = 1.2 \cdot q \cdot C_g \cdot C_e \cdot H, \quad \text{lbs/ft} \qquad (3.4)$$

Earthquake forces should be considered according to seismic zones and following the provisions of the Canadian Structural Design Manual, [14].

3.2 STRUCTURAL BEHAVIOUR

As previously described, there are three different kinds of loads acting on the structure. The VERTICAL LOADS (due to gravity), the HORIZONTAL LOADS (due to wind or earthquake) and LONGITUDINAL LOADS (due to head pulley of conveyor if located inside the tube). Those loads which do not only differ in direction of application, but also in degree of importance and frequency of occurrence, cause different effects on the structure. The loads and their effects on the idealized structural system are presented schematically in Figure 3.3.

3.2.1 Vertical Loads

Vertical loads generally include the following components:

$$\delta_t,\ \delta_c,\ \delta_e,\ \delta_e',\ \delta_w,\ \delta_w',\ \delta_{i+s}' \quad \text{lbs/ft}$$

Vertical loads are the most important loads, because they prevail in magnitude and frequency of occurrence, (δ_t, δ_c, δ_e, δ_w are always there.) They are uniformly distributed along the length of the gallery. The tubular gallery

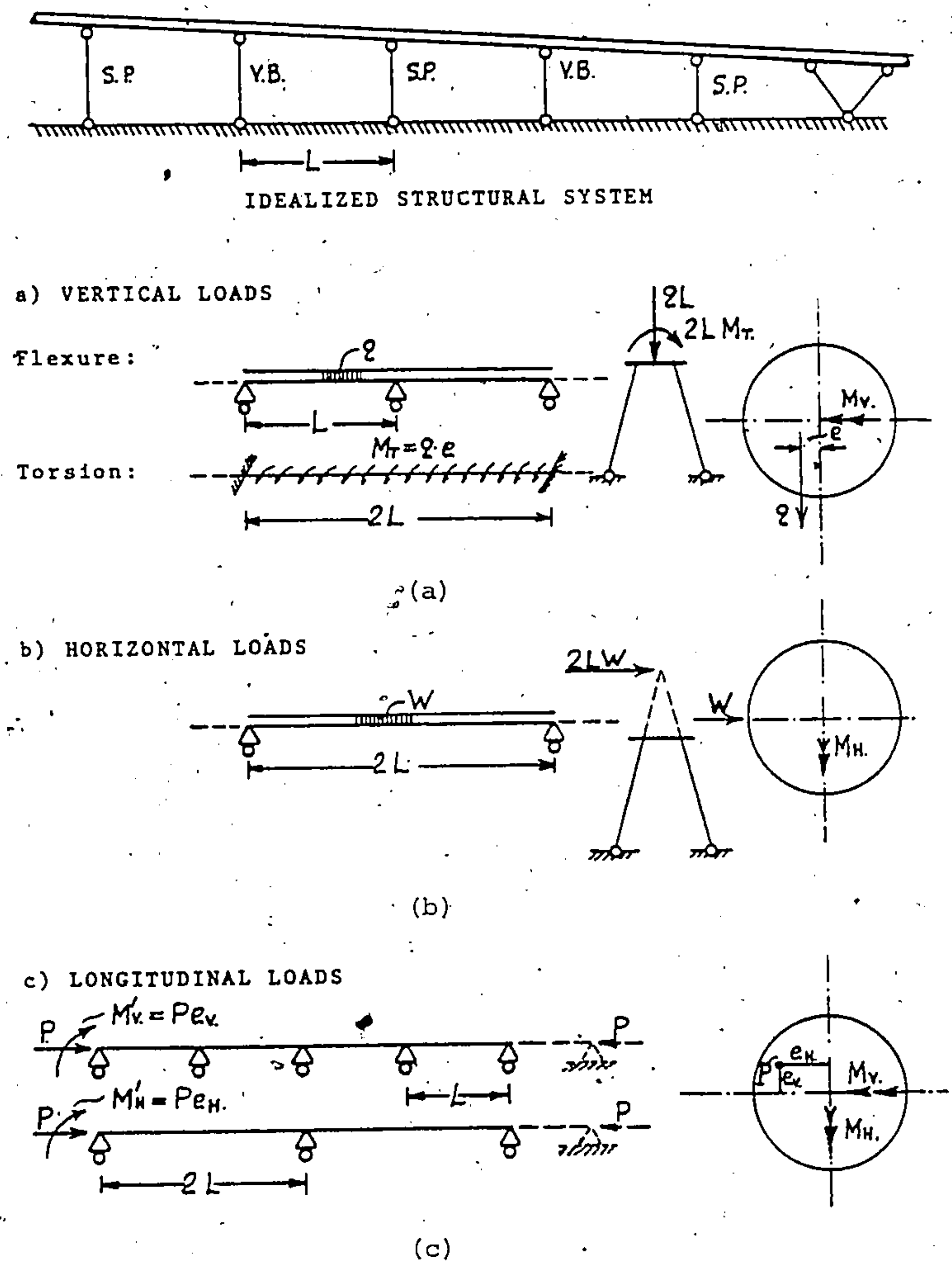


FIG.3.3 Vertical, horizontal and longitudinal loads

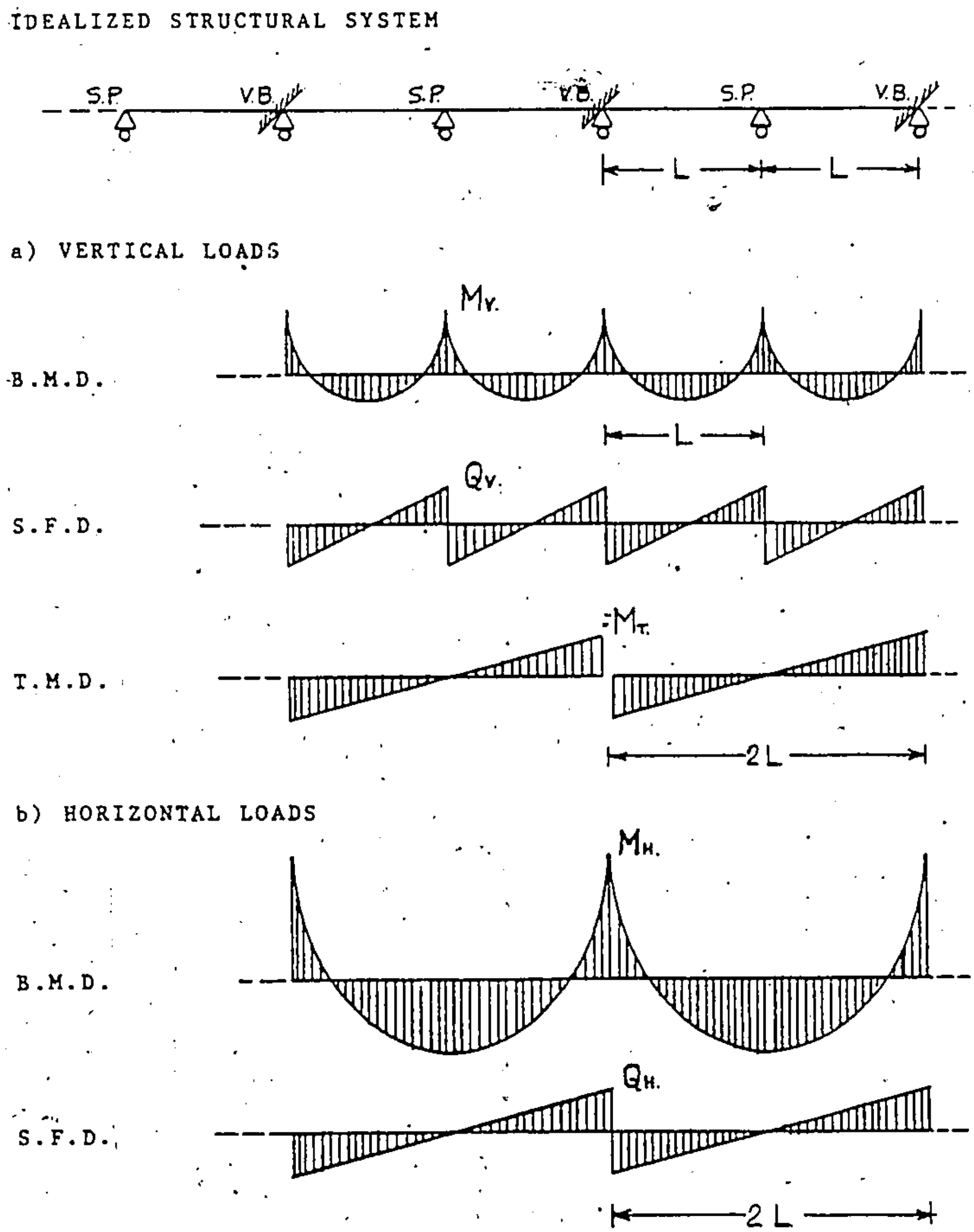


FIG.3.4 Effects of vertical and horizontal loads

c) LONGITUDINAL LOADS

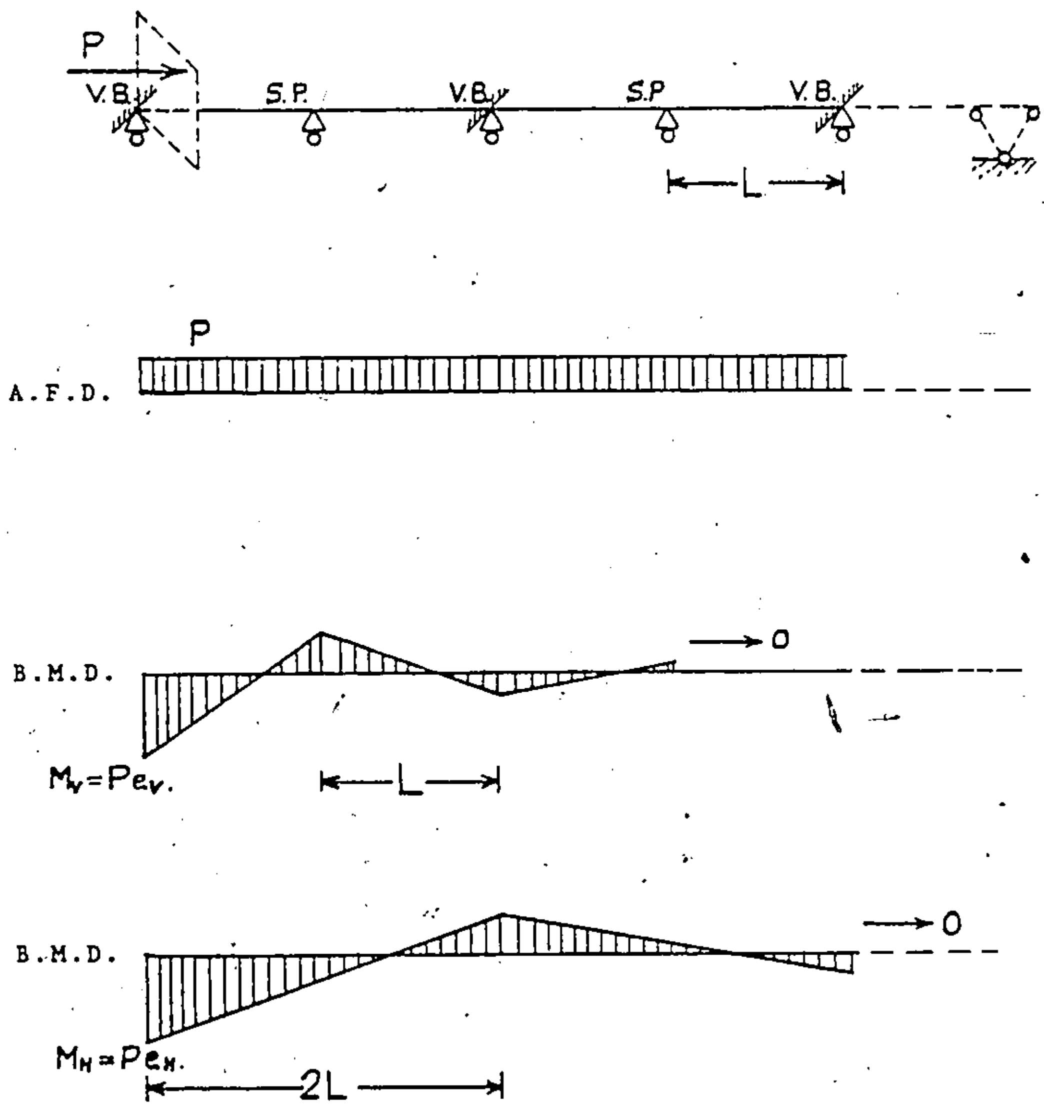


FIG.3.5 Effects of longitudinal loads

spanning over the inverted V-bents and single posts has to resist flexure, as a continuous beam of hollow circular cross-section over numerous supports (span L). In addition to flexure the vertical loads usually cause uniformly distributed torsional moment. The resultant of the vertical loads in a typical cross-section of the gallery does not usually coincide with the center of the tube. The eccentricity is often quite small and the tubular section is the ideal section to resist torque.

In order for the tube to act as a beam, some requirements have to be met. The ordinary theory of flexure can be applied in this case, only if the cross-section of the gallery remains circular at and between the supports. Rigid ring girders prevent the distortion of the tube at the location of supports, and allow the tube to act as a beam. Another function of the rigid ring girders is to resist torsion and transmit it through the inverted V-bents to the foundations. The single post supports are designed to take only vertical loads, Figure 3.3(a).

3.2.2 Horizontal Loads

The horizontal loads are wind pressure W lbs/ft′ or earthquake forces E lbs/ft. In this case, too, the tube spanning as a continuous beam (of span 2L) over the inverted

V-bents (the single posts cannot transmit the wind force to the foundation) resists the wind or earthquake in flexure. Figure 3.3(b).

3.2.3 Longitudinal Loads

Sometimes as explained before, it is profitable to locate the head pulley of the conveyor within the gallery (usually at one end, the highest of an inclined gallery). As a result, a longitudinal compressive force, originated by the head pulley of conveyor, has to be resisted (by column action) by the tube, and transmitted from one end of the gallery (the upper for inclined galleries) to the other (lower) which is held by braced bent against longitudinal translation. The unsupported length of the hollow tubular column is 2L since the single posts are considered as having zero flexural rigidity.

This compressive longitudinal force at one end of the conveyor gallery is usually eccentric with respect to the center of the tube. As a result, two moments are induced, one acting in a vertical, and one in a horizontal plane, at this end of the gallery. Usually, the designer tries to minimize this eccentricity and the magnitude of those moments is small. On the other hand, they are applied at the first support of a continuous beam. If those moments are small, they can be neglected altogether,otherwise they may be superimposed on the previous cases, since they act in horizontal and vertical planes, respectively. Figure 3.3(c).

3.2.4 Temperature Effects

The tubular gallery spanning over the inverted V-bents and single posts consists of a continuous beam of hollow circular cross-section.

The inverted V-bents and single posts,by not being fully fixed at their foundations, allow the tube to expand freely (by rotating) due to temperature changes, without transmitting any longitudinal forces due to restraints. (Effect similar to rollers of an idealized continuous beam). The lower end (usually) of the gallery is held against the longitudinal movement, and it is designed to take any longitudinal forces (conveyor pull, wind forces) which may occur. For relatively long conveyor galleries (more than 4 to 5 spans) it is better to break continuity of the tube in order to prevent eccessive rotation of the inverted V-bents and single posts. In this case, two or more braced bents must be provided to guard against longitudinal movement.

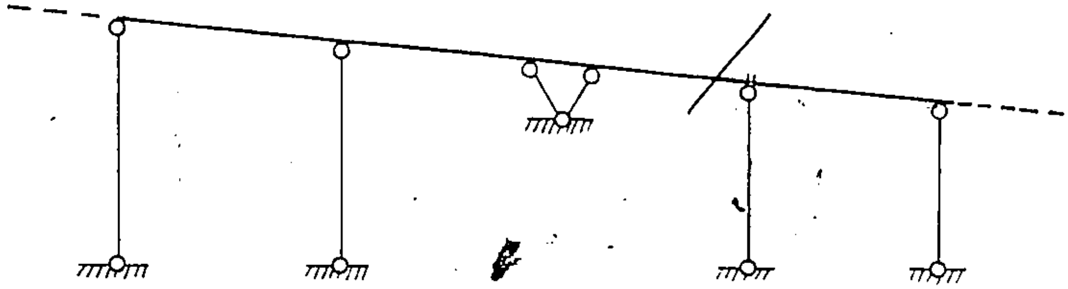


FIG. 3.6 Temperature Effects

3.2.5 Conclusion

Thus, the structural function of the tubular gallery is to resist vertical (gravity loads) and horizontal (wind or earthquake) forces as a CONTINUOUS BEAM, longitudinal compressive forces (if the conveyor's head pulley within the gallery) as a COLUMN, and torsion (because of eccentricities of the vertical forces) as a series of beams with both ends fixed against the torsion (span 2L), and transmit all those components of force safely to the foundations.

In Figures 3.3, 3.4, and 3.5, an effort to isolate the three categories of loads (vertical, horizontal, and longitudinal) and their effects on the structure (Moments, shear forces, axial forces) was made. In the case of all of them occurring simultaneously, by using the principle of superposition we can get their combined effect on the structure.

3.3 STRESSES

The vertical, horizontal and longitudinal loads induce stresses in the tube. Those stresses will be considered first for each case separately and second for their combined effect. Before that some geometric and material constants of the cross-section of the tube will be given:

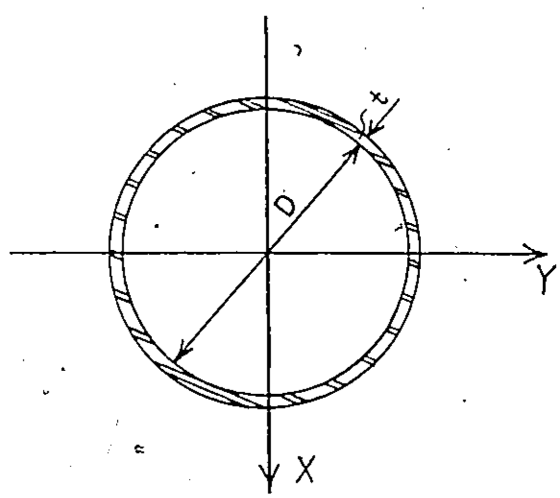


FIG. 3.7 Cross-section of Tube

The thickness t of the shell is very small and all the computations will be based on the inside diameter of the tube D where

D = inside diameter of the tube, in.

t = thickness of the wall, in.

c = corrosion allowance, 1/16 in.

D_o = outside diameter of the tube, in.

$E = 30{,}000$ ksi

$$G = \frac{E}{2(1+\mu)} = \frac{30 \times 10^3}{2(1+0.3)} = 11.5 \times 10^3 \text{ ksi}$$

$\mu = 0.3$

$F_y = 36$ ksi

$$A = \pi D(t-c), \text{ in}^2 \tag{3.5}$$

$$I_x = I_y = \pi\frac{D^3}{8}(t-c), \text{ in}^4 \tag{3.6}$$

$$S_x = S_y = \pi\frac{D^2}{4}(t-c), \text{ in}^3 \tag{3.7}$$

$$I_p = I_x + I_y = \pi\frac{D^3}{4}(t-c), \text{ in}^4 \tag{3.8}$$

3.3.1 Stresses due to vertical loads

The vertical loads induce bending moment, torsional moment and shear in the tube. In discussing the maximum stress produced in the shaft, it is necessary to consider:

(a) Shearing stresses due to the torque M_T.

$$\tau_{max} = \frac{M_T\ D_o}{2I_p} = \frac{16\ M_T}{\pi D_o^3 (1 - \frac{D^4}{D_o^4})} \qquad (3.9)$$

(b) Normal stresses due to the bending moment M_V.

$$(\sigma_V)_{max} = \frac{M_V}{S} = \frac{32\ D_o\ M_V}{\pi (D_o^4 - D^4)} \qquad (3.10)$$

In Equation (3.10), S is the exact section modulus.

$$S = \frac{\pi (D_o^4 - D^4)}{32\ D_o} = \frac{\pi D_o^3}{32} [1 - (\frac{D}{D_o})^4] \qquad (3.11)$$

(c) Shearing stresses due to the shearing force Q_V. The stress due to the shearing force is usually only of secondary importance. Its maximum value occurs at the neutral axis where the normal stress due to bending is zero. Hence the maximum combined stress usually occurs at the point where stresses given by equations (3.9) and (3.10) are a maximum. In this case, at the top and bottom surfaces of the tube.

The principal stresses due to the combination of (3.9) and (3.10) are:

$$\sigma_{max} = \frac{\sigma_V}{2} \pm \frac{1}{2}\sqrt{\sigma_V^2+4\tau^2} \qquad (3.12)$$

From Equations (3.9), (3.10) and (3.12), we obtain (where $n = \frac{D}{D_0}$):

$$\sigma_{max} = \frac{16}{\pi D_0^3(1-n^4)}\left(M_V + \sqrt{M_V^2+M_T^2}\right) = \frac{1}{2S}\left(M_V + \sqrt{M_V^2+M_T^2}\right) \qquad (3.13)$$

It should be noted that σ_{max} would have the same value for a case of simple bending in which the EQUIVALENT BENDING MOMENT is:

$$M_{equivalent} = \frac{1}{2}\left(M_V + \sqrt{M_V^2+M_T^2}\right) \qquad (3.14)$$

or

$$\sigma_{max} = \frac{M_{equivalent}}{S} \qquad (3.15)$$

where

S is given by Equation (3.7).

3.3.2 Stresses due to Horizontal Loads

$$(\sigma_H)_{max} = \frac{M_H}{S} \qquad (3.16)$$

where

S is from equation (3.7).

3.3.3 Stresses Due to Longitudinal Loads

$$\sigma = \frac{P}{A} \qquad (3.17)$$

where

A is from equation (3.5).

The stress due to bending moments caused by the eccentricity of longitudinal load will be computed, as above.

3.3.4 Summary and Combination of Loading Cases

(a) Vertical loads (M_V, M_T):

Compute:

$$M_{equiv.} = \tfrac{1}{2}(M_V + \sqrt{M_V^2 + M_T^2}) \qquad (3.14)$$

Find:

$$\sigma_{max} = \frac{M_{equiv.}}{S} \qquad (3.15)$$

(b) Vertical and Horizontal Loads (M_V, M_T, M_H):

Compute:

$$M_{Total} = \sqrt{(M_V)^2 + (M_H)^2} \qquad (3.16)$$

Compute:

$$M'_{equiv} = \frac{1}{2}(M_{Total} + \sqrt{(M_{total})^2 + (M_T)^2}) \quad (3.17)$$

Find:

$$\sigma_{max} = \frac{M'_{equiv}}{S} \quad (3.18)$$

(c) Vertical and Horizontal and Longitudinal Loads (M_V, M_T, M_H, P):

Find: $$\sigma_{max} = \frac{P}{A} + \frac{M'_{equiv}}{S} \quad (3.19)$$

In the case of absence of horizontal loads:

Find:

$$\sigma_{max.} = \frac{P}{A} + \frac{M_{equiv}}{S} \quad (3.20)$$

3.4 ALLOWABLE STRESSES

Tubular steel structures must be designed to satisfy STRENGTH and STABILITY requirements.

3.4.1 Strength Requirements

According to strength requirements, the maximum stress at a point of a cross-section, selected so that the moments and forces due to one loading case or a combination of loading cases are maximum, has to be smaller than the allowable stresses specified by CSA STANDARD S 16. [19]

By inspection of Figures 3.4 and 3.5, the maximum loaded section of the gallery occurs over the inverted V-bents (where the combination of loads is the most critical.) The maximum stresses (tensile and compressive) shall be computed as presented by equations (3.14) to (3.20) and checked against the allowable stresses given by the Code.

$$\sigma_{max,compressive} < \sigma_{allowable} (\simeq 0.60\ F_y) \quad (3.21)$$

$$\sigma_{max,tensile} < \sigma_{allowable} (\simeq 0.60\ F_y) \quad (3.22)$$

3.4.2 <u>Stability Requirements</u>

A thin-walled cylindrical shell subjected to compression, in the direction of its longitudinal axis may fail either by the instability of the shell as a whole, involving bending of the axis, or by the local instability of the wall of the shell, which may not at all involve lateral distortion of the axis. The former type of failure is called OVERALL or EULER BUCKLING, and the strength depends upon the ratio of the length to the radius of gyration of the shell (L/r_t). The latter type of failure is called LOCAL BUCKLING or WRINKLING, and the strength depends upon the ratio of thickness of the radius of the shell wall (t/R).

As in the case of steel columns, we can classify a cylinder under axial load into three categories.

(a) Very short cylinders:

$$L < 1.72\sqrt{Rt}$$

$$N_{cr.} = \frac{\pi^2 E(t^3/12)}{(1-\mu^2)L^2} \tag{3.23}$$

(b) Intermediate length cylinders:

$$1.72\sqrt{Rt} \leq L \leq 2.85\sqrt{R^3/t}$$

$$\left.\begin{aligned} N_{cr.} &= \frac{Et^2}{R\sqrt{3(1-\mu^2)}} \quad \text{classical buckling load} \\ \sigma_{cr.} &= \frac{N_{cr}}{t} = \frac{Et}{R\sqrt{3(1-\mu^2)}} \quad \text{classical buckling stress} \end{aligned}\right\} \tag{3.24}$$

(c) Long cylinders:

$$L \geq 2.85\sqrt{R^3/t}$$

$$\left.\begin{aligned} N_{cr.} &= \frac{Et}{2}\left(\frac{\pi R}{L}\right)^2 \\ P_{cr.} &= 2\pi R\, N_{cr.} = \frac{\pi^2 E}{L^2}(\pi R^3 t) = \frac{\pi^2 EI}{L^2} \end{aligned}\right\} \tag{3.25}$$

(Euler critical load)

The mode of buckling of the intermediate length cylinders is characterized by the formation of surface buckles in both the longitudinal and circumferential direction, without involvement of lateral distortion of the axis. Thus in this case LOCAL BUCKLING OR WRINKLING governs. Long cylinders buckle elastically as ordinary Euler columns by bending of the longitudinal axis in one-half sine wave and without local surface distortions (OVERALL BUCKLING).

For ordinary tubular gallery dimensions (span between inverted V-bents and single posts up to 100 feet, shell thickness 1/4 inch, diameter 10 feet) the LOCAL BUCKLING is prevailing.

For $L_u = 2L = 2 \times 100 = 200$ ft (in the horizontal direction the single post provides no flexural rigidity.)

$D = 10$ ft, $R = 5$ ft, $t = 1/4$ in, $R/t = 240$, $L_u/R = 40$

and

$$1.72 \sqrt{t/R} \leq \frac{L_u}{R} \leq 2.85 \sqrt{R/t}$$

becomes

$$0.111 \leq \frac{L_u}{R} \leq 44.152.$$

Therefore, the limits of applicability of the theory of Local BUCKLING to the tubular conveyor galleries of average length are comparatively great.

The critical length for which OVERALL BUCKLING will occur is:

$$L_{cr.} \geq 2.85 R\sqrt{R/t} \qquad (3.26)$$

and for the above dimensions:

$$L_{cr.} \geq 220.76 \text{ ft}$$

Critical lengths, L_{cr}, (above which overall buckling governs) for cylinders with different ratios of radius-to-shell thickness, R/t, are presented in Table 3.1, on the following page.

3.4.3 Local Buckling or Wrinkling

Local buckling is the governing consideration in the design of tubular galleries of moderate length. Failure of this type is due to the formation of characteristic wrinkles or bulges, circular or lobed in shape. Wrinkling is local in nature and depends upon the COMBINED COMPRESSIVE STRESSES AT THE POINT under consideration. That means that the maximum combined compressive stress which occurs at a point of the most critically loaded cross-section has to be smaller than the allowable buckling stress.

In studying thin-walled tubular structures, two considerations are of importance. First, local buckling should be prevented at stresses below yield strength; secondly, a more severe restriction is that the tendency to buckle locally should not reduce the general buckling

TABLE 3.1 CRITICAL LENGTH FOR OVERALL BUCKLING

$L_{cr} = 2.85\ R\sqrt{R/t}$				
		t=1/4 inch	t=3/8 inch	t=1/2 inch
D = 8 ft	R/t	192	128	96
	$L_{cr.}$	158 ft	129 ft	112 ft
D = 9 ft	R/t	216	144	108
	$L_{cr.}$	188 ft	154 ft	133 ft
D = 10 ft	R/t	240	160	120
	$L_{cr.}$	221 ft	180 ft	156 ft
D = 11 ft	R/t	264	176	132
	$L_{cr.}$	255 ft	208 ft	180
D = 12 ft	R/t	288	192	144
	$L_{cr.}$	290 ft	237 ft	205 ft.

load of a whole structure.

There is a serious disagreement between the results of classical equation (3.24) and experimental stress for the buckling of isotropic cylindrical shells under AXIAL COMPRESSION. Similar discrepancies can be observed for other loading conditions (bending, torsion). These experiments have indicated critical stress levels in the order of 1/3 of those given by the classical linear theory. This is due mainly to the nonlinear nature of the buckling process, the initial imperfections of such shapes and the edge effects. These facts were taken into consideration by introducing into the recommended design formulae and allowable stresses certain CORRELATION FACTORS, γ, resulting from the experimental tests.

The allowable buckling stresses which will be presented hereafter are for elastic buckling.

3.4.3.1 Axial compression

By Plantema

$$\sigma_{cr.} = \frac{662}{D/t} + 0.399\ F_y \text{ ksi} \qquad (3.27)$$

The ratio D/t is valid for

$$\frac{3,300}{F_y} \leq \frac{D}{t} \leq \frac{13,000}{F_y}$$

Equation (3.27) is recommended by the American Iron and Steel Institute.

By Wilson and Newmark

$$\sigma_{cr.} = \frac{8,000}{D/t} \text{ ksi} \tag{3.28}$$

or by assuming a factor of safety of 1.5

$$\sigma_{cr.} = \frac{5,333}{D/t} \text{ ksi} \tag{3.29}$$

By Baker et al

$$\sigma_{cr.} = 0.6\ \gamma_1\ E(\frac{t}{R}) \text{ ksi} \tag{3.30}$$

Valid for moderately long cylinders:

$$\gamma_1\ Z > \frac{\pi^2}{2\sqrt{3}}$$

where

$$Z = \frac{L^2}{Rt}\sqrt{1-\mu^2}$$

γ_1 = correlation factor obtained from Figure 3.9.

$$N_{cr} = \sigma_{cr}t = 0.6\ \gamma_1 E(\frac{t^2}{R})$$

$$P_{cr} = \sigma_{cr}2\pi Rt = 1.2\ \gamma_1 E\pi t^2 \tag{3.31}$$

The recommended allowable stresses for local buckling in the function of D/t are shown in Figure 3.8.

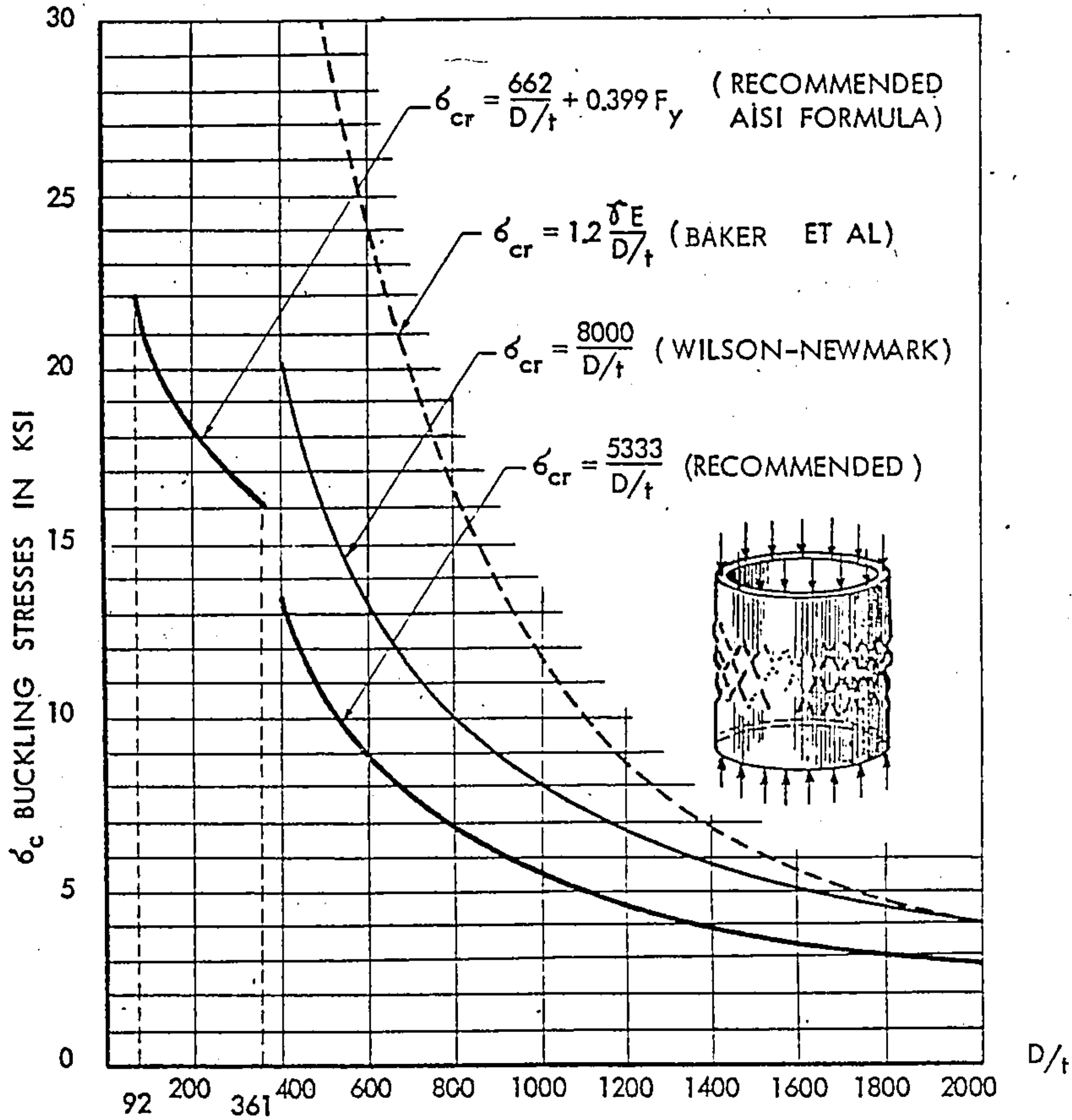


FIG.3.8 Axial compression. Recommended allowable buckling stresses.

3.4.3.2 Bending

Buckling tests on cylinders similar to those tested in axial compression indicate that buckling occurs over the compression side of the cylinder in the same wave form, with approximately the same wave-length, as in axially loaded cylinders. A comparison of axial compression and pure-bending tests results show exactly the same decrease of axial load P with an increase of the ratio R/t, as shown by axially-loaded cylinders.

The influence of initial imperfections in a shell, considering bending, should not be as great as in the axial compression. On the other hand, during the bending of shells, there occurs a certain flattening of the cross-section, which leads to an increase of maximum stress. Timoshenko found that the theoretical maximum bending stress that would cause the buckling of a circular cylindrical shell was equal to 1.3 times the axial compressive buckling stress.

The design-allowable buckling stress for a thin-walled circular cylinder subjected to bending is:

By Baker et al:

$$\sigma_{cr} = 0.6\ \gamma_2 \frac{Et}{R} \qquad (3.32)$$

Valid for moderately long cylinders.

γ_2 = correlation factor obtained from Figure 3.10.

For elastic stresses the allowable moment is:

$$M_{cr} = \pi R^2 t \sigma_{cr} = 0.6 \gamma_2 \, E \pi R t^2 \qquad (3.33)$$

By Brazier:

$$\sigma_{cr} = 0.34 \, E \, \frac{t}{R} \qquad (3.34)$$

3.4.3.3 Shear or Torsion

By Baker et al:

$$\tau_{cr} = C_s \, \frac{Et}{RZ^{0.25}} \qquad (3.35)$$

Valid for cylinders of moderate length:

$$100 < Z < 78(R/t)^2(1-\mu^2)$$

For long cylinders:

$$Z > 78(R/t)^2(1-\mu^2)$$

and the design-allowable buckling stress is:

$$\tau_{cr} = \frac{0.261 C_s E}{(1-\mu^2)^{0.75}} \left(\frac{t}{R}\right)^{1.5} \qquad (3.36)$$

The coefficient C_s is given in Figure 3.11.

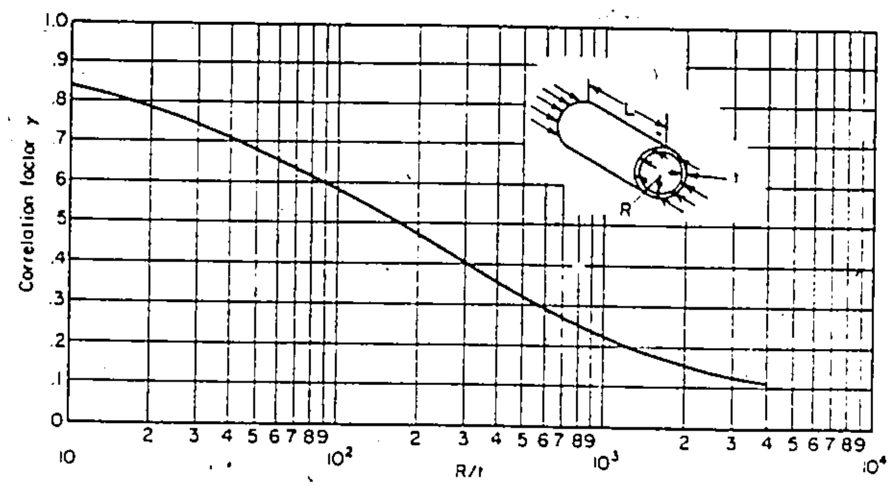


FIG.3.9 Correlation factors for unstiffened circular cylinders subjected to axial compression

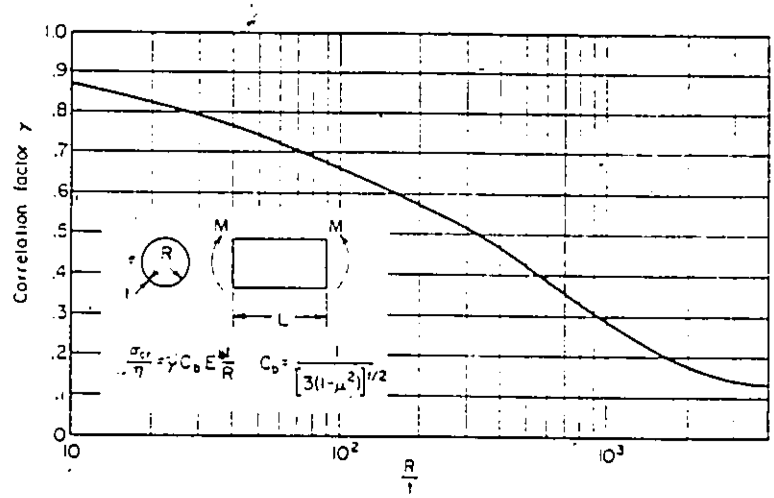


FIG.3.10 Correlation factors for unstiffened circular cylinders subjected to bending

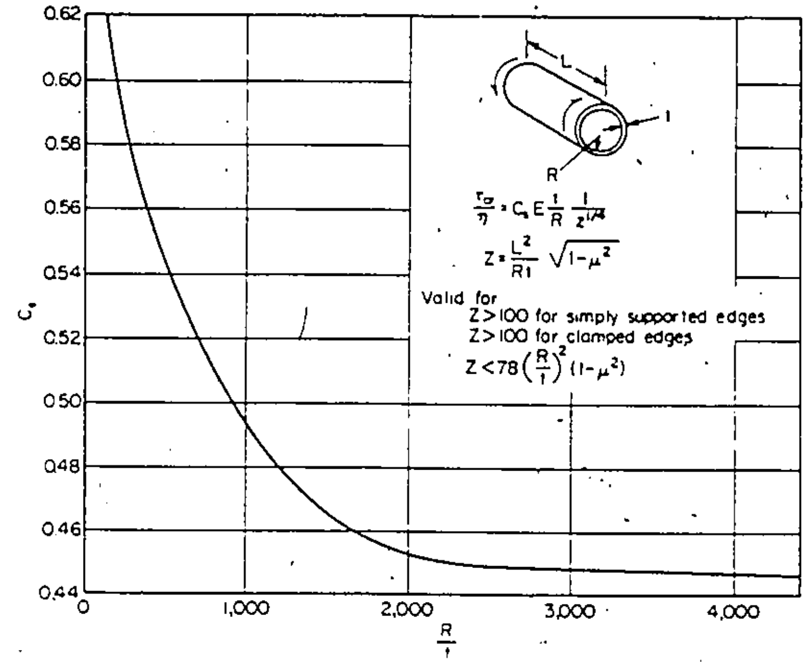


FIG.3.11 Buckling stress coefficient, C_s, for unstiffened circular cylinders subjected to torsion

3.4.3.4 Combined loading

The criterion for structural failure of a member under combined loading is frequently expressed in terms of a stress-ratio equation

$$R_1^x + R_2^y + R_3^z = 1 \qquad (3.37)$$

In general, the stress ratio R is the ratio of the allowable value of the stress caused by a particular kind of load in a combined-loading condition to the allowable stress for the same kind of load when it is acting alone. The subscripts denote the stress due to a particular kind of loading (compression, shear, etc.), and the exponents (usually empirical) express the general relationship of the quantities for failure of the member. A curve drawn from such a stress-ratio equation is termed a STRESS-RATIO INTERACTION CURVE. In simple loadings, the term "stress ratio" is used to denote the ratio of applied to allowable stress (R_a) and if the equation

$$R_{a_1}^x + R_{a_2}^y + R_{a_3}^z < 1 \qquad (3.38)$$

is satisfied (or the point falls on the left-side of the stress-ratio interaction curve) the structure is safe.

(a) Combined torsion and axial loading:

A semi-empirical interaction curve for circular cylinders is given in Figure 3.12.

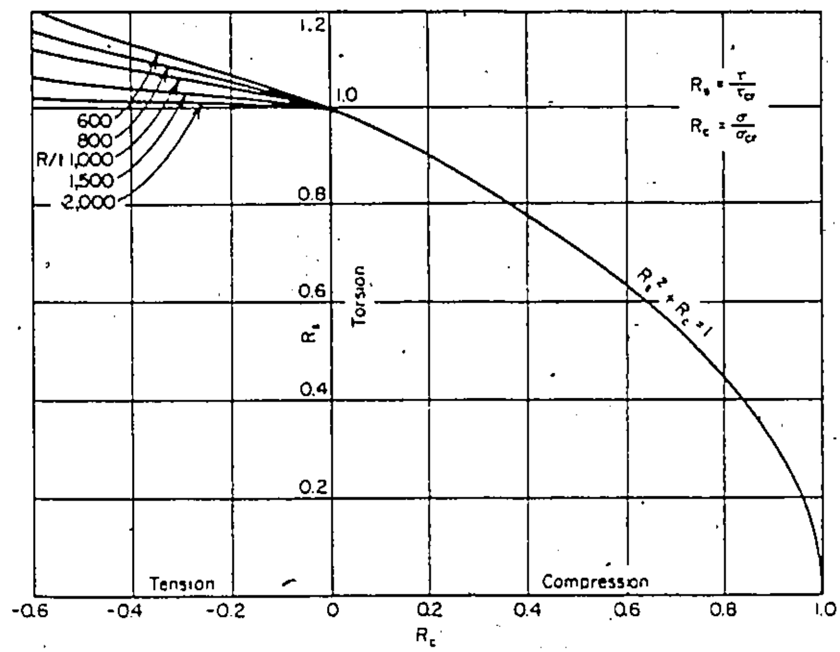


FIG.3.12 Buckling-stress interaction curve for unstiffened circular cylinders under combined torsion and axial loading

σ^c_{cr} is found using Figure 3.9 and τ_{cr} is found using Figure 3.11.

$$\frac{\sigma^c_{applied}}{\sigma^c_{cr}} + \left(\frac{\tau_{applied}}{\tau_{cr.}}\right)^2 < 1 \qquad (3.39)$$

(b) Bending and torsion:

An empirical interaction curve is given in Figure 3.13.

σ^b_{cr} is found from Figure 3.10 and τ_{cr} from Figure 3.11.

$$\frac{\sigma^b_{applied}}{\sigma^b_{cr}} + \left(\frac{\tau_{applied}}{\tau_{cr}}\right)^2 < 1 \qquad (3.40)$$

(c) Axial compression and bending:

The interaction curve is given in Figure 3.14.

σ^b_{cr} is found from Figure 3.10 and σ^c_{cr} from Figure 3.9.

$$\frac{\sigma^c_{applied}}{\sigma^c_{cr}} + \frac{\sigma^b_{applied}}{\sigma^b_{cr}} < 1 \qquad (3.41)$$

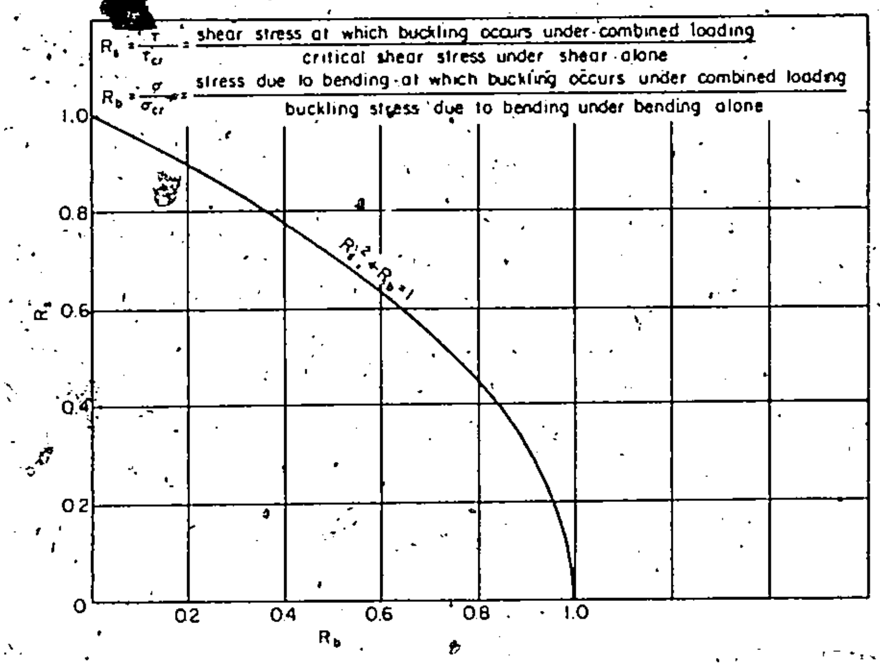


FIG.3.13 Buckling-stress interaction curve for unstiffened circular cylinders under combined bending and torsion

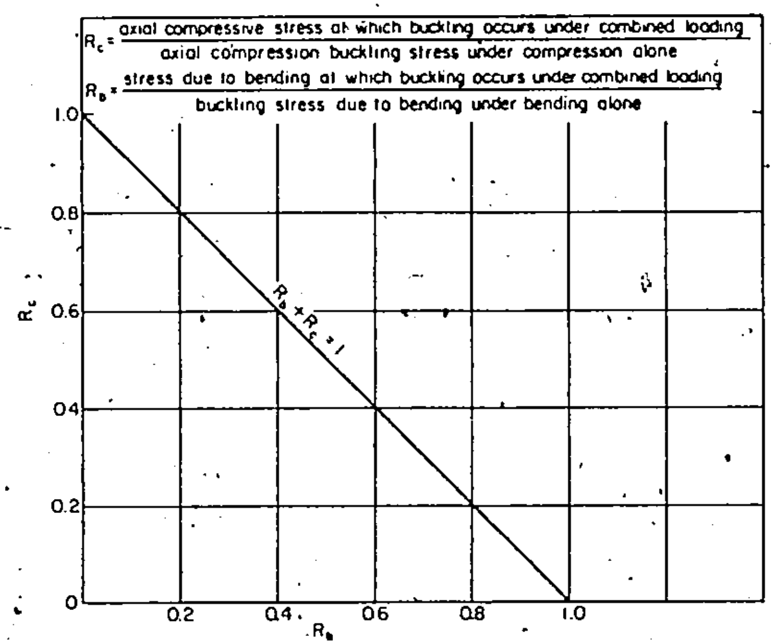


FIG.3.14 Buckling-stress interaction curve for unstiffened circular cylinders under combined axial compression and bending

(d) Axial compression, bending and torsion:

Although a slightly different interaction equation is proposed for each case, Schilling [10] suggests that a single formula can be conservatively applied to all these combinations. He proposes the relation:

$$\frac{\sigma^{c}_{applied}}{\sigma^{c}_{cr}} + \frac{\sigma^{b}_{applied}}{\sigma^{b}_{cr}} + \left(\frac{\tau_{applied}}{\tau_{cr}}\right)^{2} < 1 \quad (3.42)$$

3.4.4 Overall Buckling or Column Type Buckling

Although (as it is shown in Table 3.1) the local buckling prevails for axially loaded tubes of moderate length and diameter (L = 180 to 200 ft, span between inverted V-bents, D = 10 ft, t = 1/4 inch), since the span between the two inverted V-bents does not exceed the critical buckling length, small variations of radius-to-thickness ratio and span may cause the structure to fail in OVERALL BUCKLING. In addition, in several cases when the D/t ratio of a member is such that LOCAL BUCKLING is probable, the KL/r ratio of the member may be such that column buckling is also an important consideration. According to AISI's "Specification for the Design of Cold-Formed Steel Structural Members" purely column-type buckling of tubular members may occur with no LOCAL BUCKLING as long as

$$D/t \leq \frac{3{,}300}{F_Y}$$

where

F_Y = yield strength of material, ksi.

This formula is very conservative compared with the limits defined by Equation (3.26).

3.4.4.1 Tube under axial compression

The formulae which follow are valid for both unstiffened and stiffened shells (by longitudinal stringers and circumferential ring stiffeners.) In the case of stiffened shells, the stringers must be included in the calculation of the radius-of-gyration "r" of the tube cross-section, while circumferential stiffeners can be neglected since they have no direct effect on the overall column-buckling mode.

A sketch of the stiffened cylindrical shell is shown in Figure 3.15.

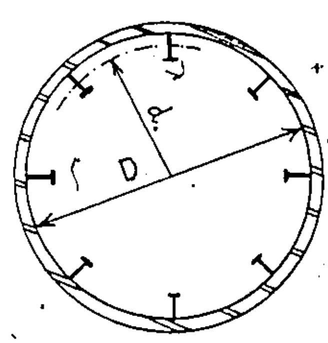


FIG. 3.15 Stiffened cylindrical shell

where

Radius of gyration $r = \sqrt{\frac{I_p}{2A}}$, in (3.43)

Area of combined section

$$A = \pi D(t-c) + NA_{st}, \text{ in}^2 \quad (3.44)$$

Polar moment of inertia of combined section

$$I_p = I_x + I_y = \pi \frac{D^3}{4}(t-c) + NA_{st}d^2, \text{in}^4 \quad (3.45)$$

where

A_{st} = area of one stiffener, in^2

N = number of longitudinal stiffeners

d = distance from center of tube to centroid of stiffener, in.

The tubes under axial compression, are classified as in the case of ordinary steel columns in two categories.

(a) Intermediate tubes

(b) Slender tubes

where

KL = effective length of circular tube for column-type buckling (in this case K = 1 conservative)

F_y = yield strength of material, ksi

E = 30,000 Young modulus for steel, ksi

$$\lambda = \frac{KL}{r}\frac{1}{\pi}\sqrt{\frac{F_Y}{E}} = \text{non-dimensional slenderness ratio} \quad (3.46)$$

F_a = allowable critical stress, ksi

σ_{cr} = theoretical critical axial stress, ksi

F.S = factor of safety

The CRS Column Strength Curve can be expressed by

(a) Intermediate Columns, $\lambda \leq \sqrt{2}$:

$$\left.\begin{aligned} F_a &= \frac{\sigma_{cr}}{F.S} \\ \sigma_{cr} &= (1-0.25\ \lambda^2)F_Y \\ F.S &= 1.67 + 0.265\lambda - 0.044\lambda^3 \end{aligned}\right\} \quad (3.47)$$

(b) Slender Columns, $\lambda > \sqrt{2}$:

$$\left.\begin{aligned} F_a &= \frac{\sigma_{cr}}{F.S} \\ \sigma_{cr} &= \frac{F_Y}{\lambda^2} \\ F.S &= 1.92 \end{aligned}\right\} \quad (3.48)$$

The above equations of the critical buckling allowable stress are identical to those of the wide flange columns.

Alternately, based on the results of tests with electric resistance welded tubes of carbon steels having yield strength of 45 and 55 ksi and for which stub-column tests found proportional limits as low as 50% of the yield strength, Wolford and Rebholz [10] proposed a more conservative column curve, which is recommended for use in cases of tubular cold formed structures.

(a) Intermediate Columns, $\lambda \leq \sqrt{3}$

$$\left.\begin{aligned} F_a &= \frac{\sigma_{cr}}{F.S.} \\ \sigma_{cr} &= [1.0 - \frac{2}{3\sqrt{3}}\lambda]F_y \\ F.S. &= 1.67 + 0.145\lambda \end{aligned}\right\} \quad (3.49)$$

(b) Slender Columns, $\lambda > \sqrt{3}$

$$\left.\begin{aligned} F_a &= \frac{\sigma_{cr}}{F.S.} \\ \sigma_{cr} &= \frac{F_y}{\lambda^2} \\ F.S. &= 1.92 \end{aligned}\right\} \quad (3.50)$$

A structural cylinder fabricated from plate will typically have high circumferential residual stresses caused by cold-forming processes, as well as from the circumferential and longitudinal residual stresses due to welding. Because the exact nature of these stresses and their effect on column strength are at this time, still a matter of conjecture, it is usually conservative to use Equations (3.49) and (3.50).

3.4.4.2 Tube under combined loading

In this case the usual interaction-type formula for beam-columns in compression and biaxial bending will be employed. Allowable-stress interaction equations for design are essentially empirical. The factor -of-safety depends in part upon the expressions chosen to define the allowable stresses, F_a (axial) and F_b (bending). In the discussion following, one has to keep in mind that LOCAL BUCKLING is local in effect, (limits the strength of the structure at a point of a certain cross-section), while OVERALL BUCKLING has to do with the stability of the structure, as a whole.

The interaction equation is:

$$\frac{P}{P_u} + \frac{C_{mv} M_{vt}}{M_{uv}(1-\frac{P}{P_{ev}})} + \frac{C_{mH} M_{Ht}}{M_{uH}(1-\frac{P}{P_{eH}})} < 1 \qquad (3.51)$$

or, in another form:

$$\frac{f_a}{F_a} + \frac{C_{mv} f_{bv}}{F_{bv}[1-\frac{f_a}{F'_{ev}}]} + \frac{C_{mH} f_{bH}}{F_{bH}[1-\frac{f_a}{F'_{eH}}]} < 1 \qquad (3.52)$$

One has to observe that the denominator of the second and third term of equation (3.51) represents the strength in bending at a particular point of a cross-section in vertical and horizontal direction, respectively, magnified by a factor $(1-\frac{P}{P_e})$ to guard against overall buckling, where

P = applied longitudinal compressive force, kips

C_{mv}, C_{mH} = reduction factors depending on the edge conditions, transverse loading, and curvature of bent, given in the CISC Handbook for Steel Columns

$M_{Vt} = \begin{cases} M_V = \text{bending moment due to vertical loads, Fig.3.4} \\ \frac{1}{2}(M_V + \sqrt{M_V^2+M_T^2}) = \text{equivalent bending moment due to vertical loads and torque } M_T \text{ Equation (3.14)} \end{cases}$

$M_{Ht} = \begin{cases} M_H \quad \text{bending moment due to horizongal loads, Fig.3.4} \\ \frac{1}{2}(M_H + \sqrt{M_H^2+M_T^2}) = \text{equivalent bending moment due to horizontal loads and torque } M_T \end{cases}$

$$f_a = \frac{P}{A} \qquad (3.53)$$

$$f_{bv} = \frac{M_{vt}}{S} \qquad (3.54)$$

$$f_{bH} = \frac{M_{Ht}}{S} \tag{3.55}$$

P_e is the elastic buckling strength of the member

$$P_e = \sigma_{cr}A = \frac{F_Y\ A}{1.92(\frac{KL_u}{r})^2\ \frac{1}{\pi^2}\ \frac{F_Y}{E}} = \frac{\pi^2 E}{1.92(\frac{KL_u}{r})^2} A =$$

$$= \frac{149{,}000}{(\frac{KL_u}{r})^2} A \tag{3.56}$$

$$F'_{ev} = \frac{P_e}{A} = \frac{149{,}000}{(\frac{KL_u}{r})^2} = \frac{149{,}000}{(\frac{L}{r})^2} \tag{3.57a}$$

$$F'_{eH} = \frac{P_e}{A} = \frac{149{,}000}{(\frac{KL_u}{r})^2} = \frac{149{,}000}{(\frac{2L}{r})^2} \tag{3.57b}$$

where

L = span between V-bent and single post

$2L$ = span between two V-bents

L_u = unsupported length

The loss of stability due to OVERALL BUCKLING will take place in the horizontal direction, where

$$L_u = 2L$$

In the vertical direction

$$L_u = L$$

(a) Overall buckling

In the case of

$$L_u = 2L >> L_{cr} = 2.85\ R\sqrt{R/t},$$

overall buckling will occur before local buckling. Since local buckling is not present, the full bending strength of the section can be considered.

$$F_{bv} = F_{bH} = 0.60\ F_Y \tag{3.58}$$

F_a will be taken from Equations (3.49) or (3.50).

The interaction equation (3.52) must be satisfied.

(b) Local buckling

In the case of

$$1.72\sqrt{Rt} < L_u = 2L < 2.85\ R\sqrt{R/t}$$

local buckling will occur before overall buckling. The appropriate interaction equation for local buckling [equations (3.39) to (3.42)] must be satisfied first.

Because of the loss of cross-sectional strength under bending due to local buckling, overall buckling may occur, and interaction equation (3.52) must be satisfied, too, where

$F_a = 0.60\ F_y$, or by equations (3.49), (3.50), (the smaller)

$$F_{bv} = F_{bH} = \sigma_{cr} = 0.6\gamma_2\ \frac{Et}{R} \qquad (3.59a)$$

or

$$F_{bv} = F_{bH} = \sigma_{cr} = 0.34\ E(\frac{t}{R}) \qquad (3.59b)$$

(c) <u>Local and overall buckling</u>

For many practical applications, when the D/t ratio of a member is such that local buckling is probable, the KL/r ratio of the member may be such that the column buckl-ling is also an important consideration. In other words

$$L_u = 2L \simeq 2.85\ R\sqrt{R/t}$$

In this case, both local buckling (associated with strength locally) and overall buckling (associated with overall stability of structure) must be checked. The requirements are:

(1) The appropriate local buckling interaction equation must be satisfied. Equations (3.39) to (3.42).

(2) The overall stability interaction equation (3.52) must be satisfied.

In the second case, in order to establish the allowable stresses (F_a,F_b) the approach suggested by Marshall [10] will be followed in a more general form. In this approach, the theoretical local-buckling strength is substituted for F_y in the appropriate beam-column formula. That is, that since the local buckling limits the strength locally, and occurs simultaneously with overall buckling, we can assume an equivalent material having a yield strength equal to the theoretical local buckling strength.

$$F_y' = \sigma_{cr} = 0.6\ E\ \frac{t}{R} \tag{3.60}$$

which is identical to the formulae suggested by Baker [9] without the correlation factors γ_1 and γ_2.

$$F_{bv} = F_{bH} = 0.60\ F_y' = 0.36\ E\ \frac{t}{R} \tag{3.61}$$

which is almost identical to the formula suggested by Brazier for pure bending (Equation (3.34))

$$F_a = \frac{F'_y}{(F.S.)\lambda^2} = \frac{0.6\ E(t/R)}{1.92\ \lambda^2} \qquad (3.62)$$

which is derived from Equation (3.50).

From Table 3.1, we see that the ratio R/t changes from between 100 and 300. By considering Figures 3.9 and 3.10, giving the correlation factors γ_1 and γ_2 versus the R/t ratio we notice that $0.70 < \gamma_1 < 0.90$.

The correlation factors γ_1 and γ_2 are smaller than the FS of 0.60 and $\frac{1}{1.92\ \lambda^2}$, employed in this case. In conclusion, the interaction Equation (3.52) yields conservative results.

CHAPTER IV

INTERMEDIATE CIRCUMFERENTIAL RINGS

CHAPTER IV

INTERMEDIATE CIRCUMFERENTIAL RINGS

4.1 DESCRIPTION

Along the length of a tubular gallery (for both unstiffened and stiffened tubes) at equal intervals of 10 to 20 ft., circumferential stiffeners are located inside the tube. Although their spacing is normally such that they should be considered discretely spaced, and have no direct effect on the LOCAL and OVERALL buckling mode, they serve a dual structural purpose:

1) To receive the posts and hangers, which support at equal intervals, the idlers of the conveyor and stringers of the walkway, see Figure 2.2.

2) To maintain the cross-section of the tube between the supports circular, and thus to allow the tube to act as a beam.

The circumferential stiffeners consist of angles or T-sections, continuously welded on the inside surface of the shell of the tube. Welded plates may form the section of the stiffeners (T-section or angle) or the usual structural rolled sections can be employed. If structural rolled sections are to be used, they are formed to the required radius at lengths of one quarter of the tube's circumference, and shop-welded to the shell of the tubular gallery.

4.2 DESIGN OF THE CIRCUMFERENTIAL RINGS

The stiffening rings have to be designed to meet the STRENGTH REQUIREMENTS of CSA STANDARD S 16. Therefore, stresses due to bending moments and shears at a section of the ring have to be smaller than the allowable stresses specified in the standard.

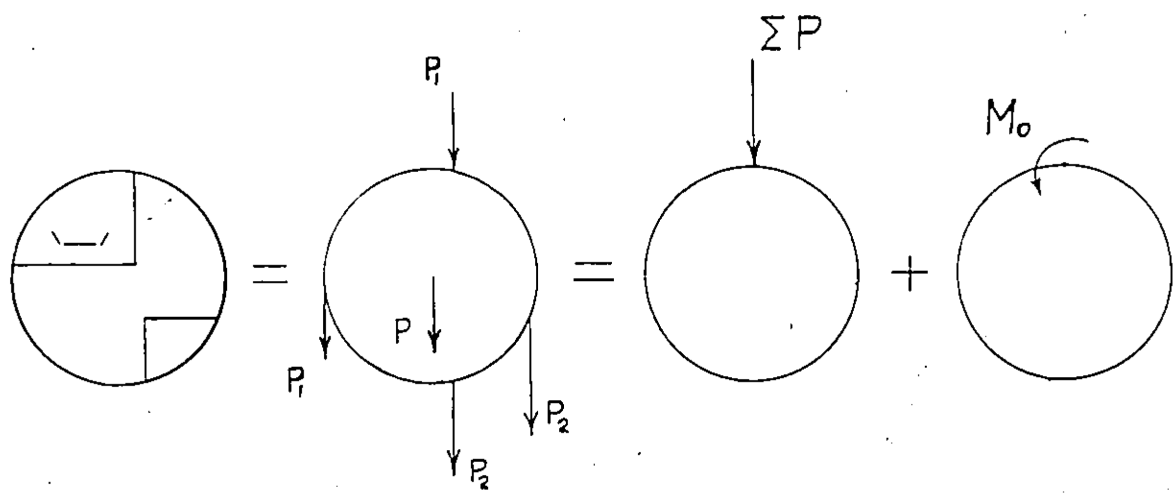


FIG. 4.1 Resolving of Forces

The forces which are applied on the ring due to conveyor, walkway, and self-weight, are shown above, schematically. In the design of the ring, the LL on the walkway is taken at 100 lbs/ft^2. The critical loading condition occurs for DL plus full LL on the conveyor and walkways. Those loads can be resolved in a vertical resultant ΣP (through the center of the ring) and a moment M_o. The analysis of the ring under these loading cases is presented in Table 4.1.

In the design, the maximum value of the stress which results from the combination of those loads, should be considered, (the maximum stress occurs at point A of the ring.)

S = section modulus of ring, in^3

A = area of ring, in^2

R = radius to the center of gravity of the ring, in.

due to bending moment

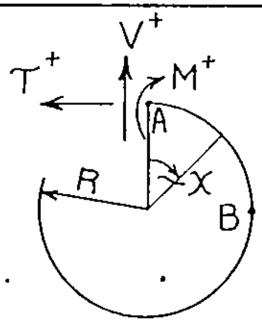
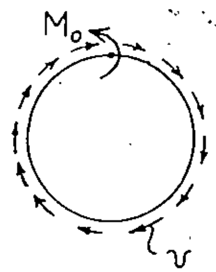
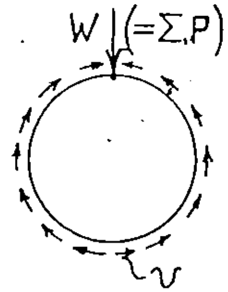
$$f_1 = \frac{0.50\ M_o + 0.24\ R\Sigma P}{S}\ \text{ksi} \quad (4.1)$$

due to axial force

$$f_2 = \frac{0.24\ \Sigma P}{A}\ \text{ksi} \quad (4.2)$$

The maximum compressive stress at the section is:

TABLE 4.1 ANALYSIS OF CIRCUMFERENTIAL RING

	 CASE I Ring loaded by bending moment M_o, in-Kips, and supported by tangential shear.	 CASE II Ring loaded by concentrated load W, Kips, and supported by tangential shear.	Superposition of Cases I, II, at Point A: $I_A + II_A$
Bending Moment M, in-Kips	$MaxM = 0.5000\ M_o$ at Point A	$MaxM = 0.2387\ WR$ at Point A	$MaxM = 0.5000M_o +$ $+0.2400WR$
Axial Circum-ferencial Force T, Kips	T=0, at Point A. $MaxT = \frac{0.3183M_o}{R}$, at Point B	$MaxT = -\ 0.2387W$ at Point A	$T = -\ 0.24W$
Circumferential Shear, V, Kips	$MaxV = \frac{3M_o}{2\pi R} = 0.4775\frac{M_o}{R}$ at Point A.	V=0	$V = 0.48\frac{M_o}{R}$
Tangential Shear v, k/inch.	$v = \frac{M_o}{2\pi R^2}$	$v = \frac{W\sin X}{\pi R}$ For $x = 90^o$ $maxv = \frac{W}{\pi R}$	$v = \frac{M_o}{2\pi R^2} + \frac{W}{\pi R}$

$$f = f_1 + f_2 \leq f_{allowable} \quad (4.3)$$

$$f_{allowable} = \begin{cases} 0.60 \; F_y \text{ for non-compact sections} \\ 0.66 \; F_y \text{ for compact sections} \end{cases}$$

The maximum shear stress is:

$$\tau = \frac{0.48 \, M_o}{AR} \text{ ksi} \quad (4.4)$$

The thickness of the shell has to be checked, if it is adequate to undertake the tangential shear. The maximum tangential shear occurs at point B of the ring.

$$v = \frac{M_o}{2\pi R^2} + \frac{\Sigma P}{\pi R} \text{, kips/in} \quad (4.5)$$

The maximum shear stress in the shell is:

$$\tau = \frac{\frac{M_o}{2\pi R^2} + \frac{\Sigma P}{\pi R}}{2(t-c)} \text{, ksi} \quad (4.6)$$

t = thickness of shell, in

c = corrosion allowance, 1/16".

CHAPTER V

RING GIRDERS AT SUPPORTS

CHAPTER V

RING GIRDERS AT SUPPORTS

5.1 DESCRIPTION

It has been found that rigid ring girders provide very effective supports for tubular structures. These girders prevent the distortion of the tube at the location of supports and thus maintain its ability to act as beams.

Typical ring girders are shown in Figures 5.1, and 5.2. In Figure 5.1, the ring girder consists of a plate girder stiffened by radially located stiffeners at equal intervals. In Figure 5.2, the girder consists of two stiffener rings continuously welded to the tube on both sides, and tied together with diaphragm plates welded between the two rings. Two short columns consisting of wide-flange I-beams are bolted between the rings to carry the load to the seat weldments.

In computing the section modulus of the ring girder, a portion of the adjacent shell may be considered as acting with the girder. [8] The total length of the shell thus acting is:

$$C = 1.56\sqrt{Rt} + b, \text{ in} \tag{5.1}$$

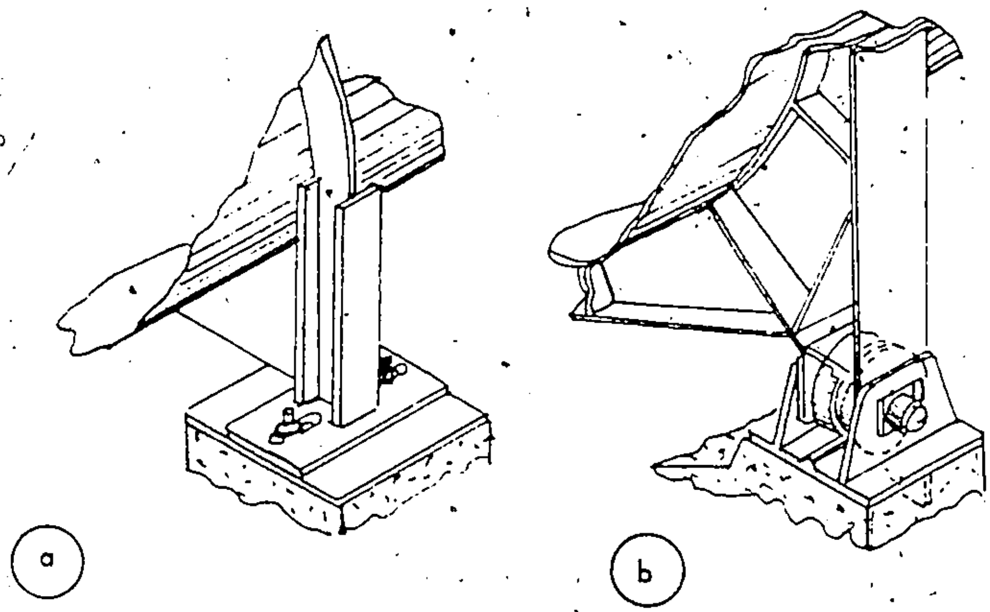


FIG. 5.1 Ring girder

(a) sliding support

(b) roller support

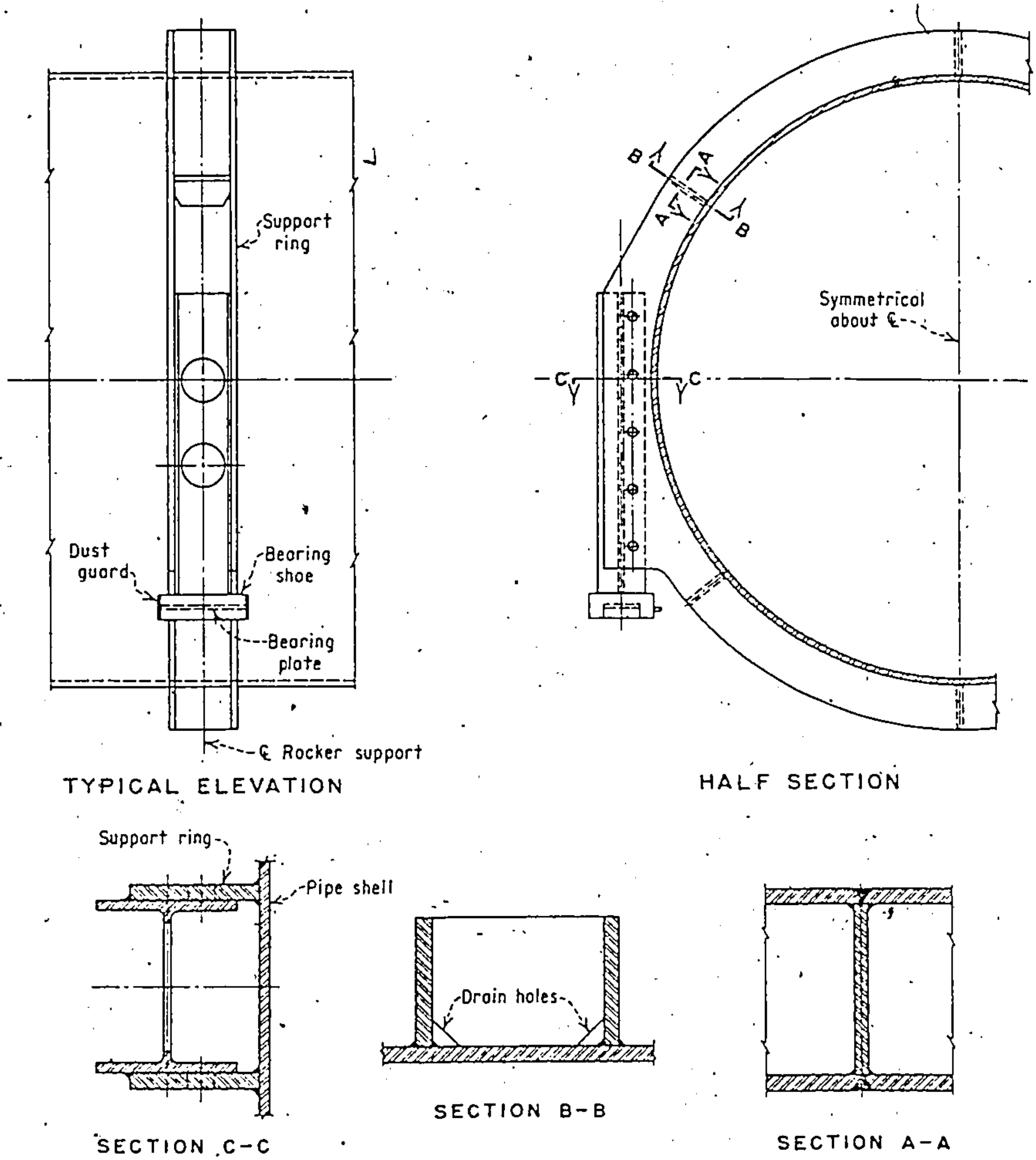


FIG.5.2 Ring girder and column support

where

b = the width or ring girder, in. See Fig. 5.3.

5.2 ANALYSIS AND DESIGN OF RING GIRDERS

The loads are transmitted from the shell of the tube to the rings by tangential shear. A design for ring girder construction, based on elastic theory, was developed by Schorer [7].

5.2.1 Vertical Loads

The shell transmits by circumferential shear to the ring, a concentric vertical load Q (reaction of a continuous beam) and a torque M_T (reaction torque of a both-ends fixed beam). This is the typical case for rings over the inverted V-bents. (Fig. 5.4).

For rings over the single post supports only Figure 5.4(a) should be considered.

5.2.1.1 Concentric vertical load Q

The bending moment diagram for an eccentricity of the support equal to 0,04R is shown in Figure 5.6. Figure 5.5 shows the absolute values of M_o, M_i and M_a plotted in the function of the ratio, a/R. The eccentricity, a, is taken

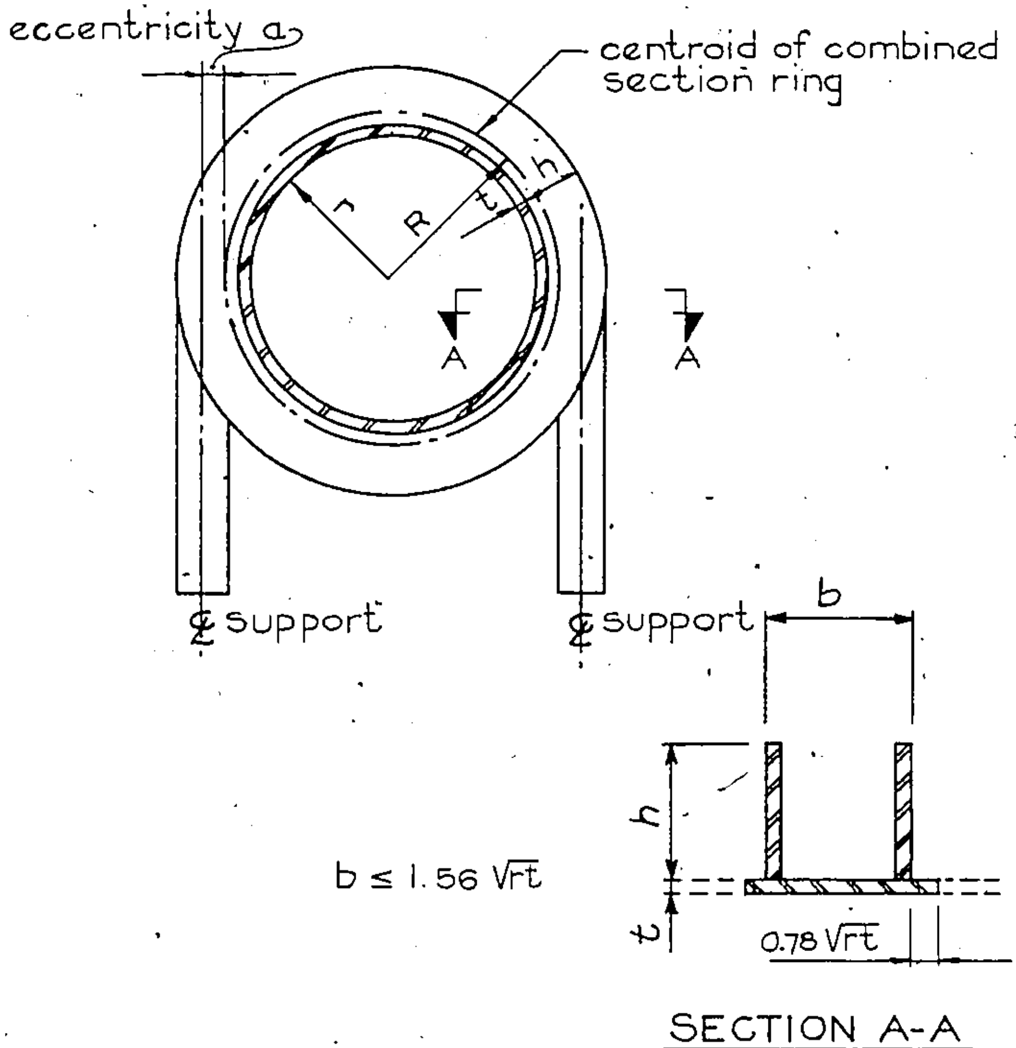


FIG. 5.3   Effective width of ring girder

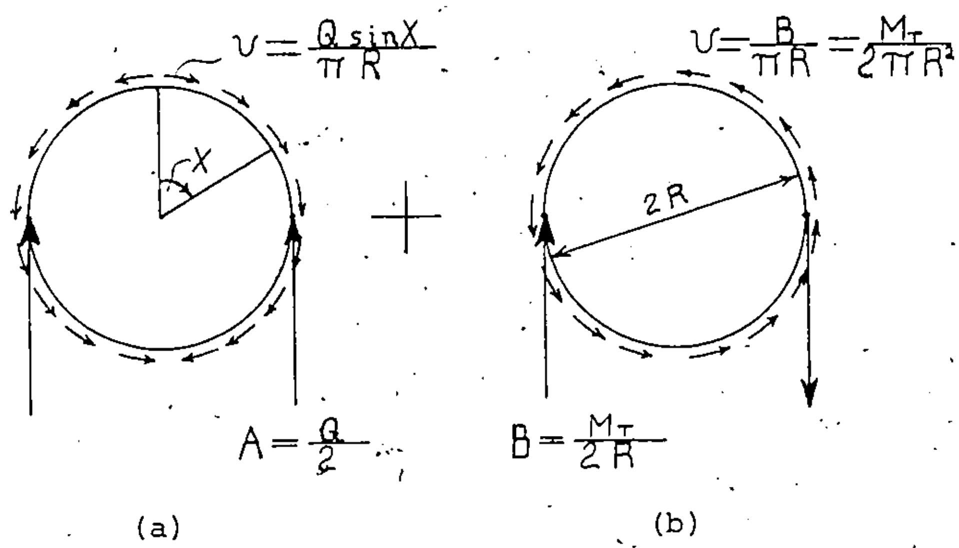


FIG. 5.4 Effects of Vertical Loads on Ring Girders

as positive when the reactions are applied outside the neutral axis of the supporting ring. As we see from Figure 5.5, the minimum possible ring moment is obtained for $a/R = +0.04$, in which case:

$$M_i = M_a = 0.010QR \qquad (5.2)$$

The maximum normal ring force (axial force), due to the circumferential shear, occurs at $X = \pi/2$ and is:

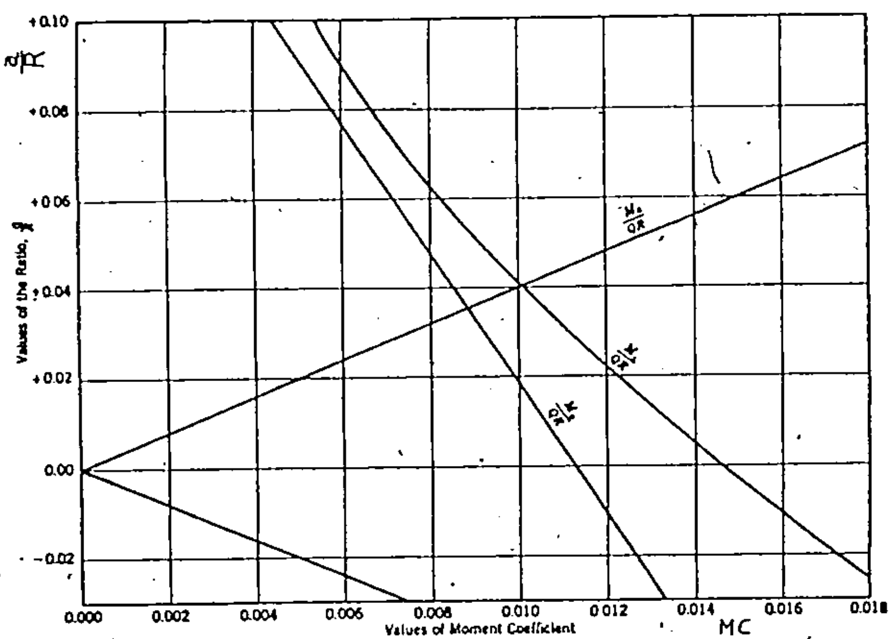


FIG.5.5 Bending moments in supporting ring

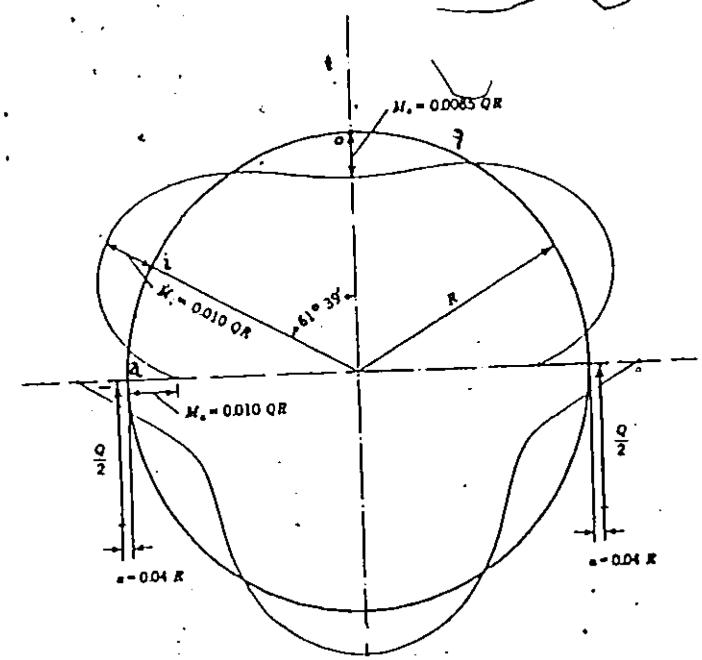


FIG.5.6 Bending moment diagram in supporting ring for

$$\frac{a}{R} = 0.04$$

$$T_a = \frac{Q}{4} \tag{5.3}$$

The force exerts compression just above, and tension just below, the horizontal diameter. Since, for $a/R \geq +0.04$, the maximum bending stress is also found at this point, a somewhat smaller combined stress will be obtained with $+0.04 > a/R > 0.0$, although it is hard to be achieved because of fabrication reasons.

Figure 5.5 can be used readily, and for a given ratio of support eccentricity to radius, a/R, the values of moment coefficient, M.C. can be obtained, at points, a,i,o.

$$M = (MC)QR \tag{5.4}$$

The maximum combined stresses in the ring for $a/R \geq 0.04$, occur at the horizontal diameter, where

$$f_1 = f_{bend.} + f_{axial} = \frac{(MC)QR}{S} + \frac{Q}{4A} \tag{5.5}$$

and

S = section modulus of combined section, in^3 (Figure 5.3)

A = area of combined section, in^2. (Figure 5.3)

a = eccentricity of the support, in. (Figure 5.3)

R = radius to center of gravity of ring, in.

And for a/R = +0.04, the maximum combined stress is:

$$f_1 = \frac{0.010\ QR}{S} + \frac{Q}{4A} \qquad (5.6)$$

5.2.1.2 Torque due to vertical loads

For the analysis of this case, (Figure 5.4(b)), use of the tables presented in Reference [18] was made.

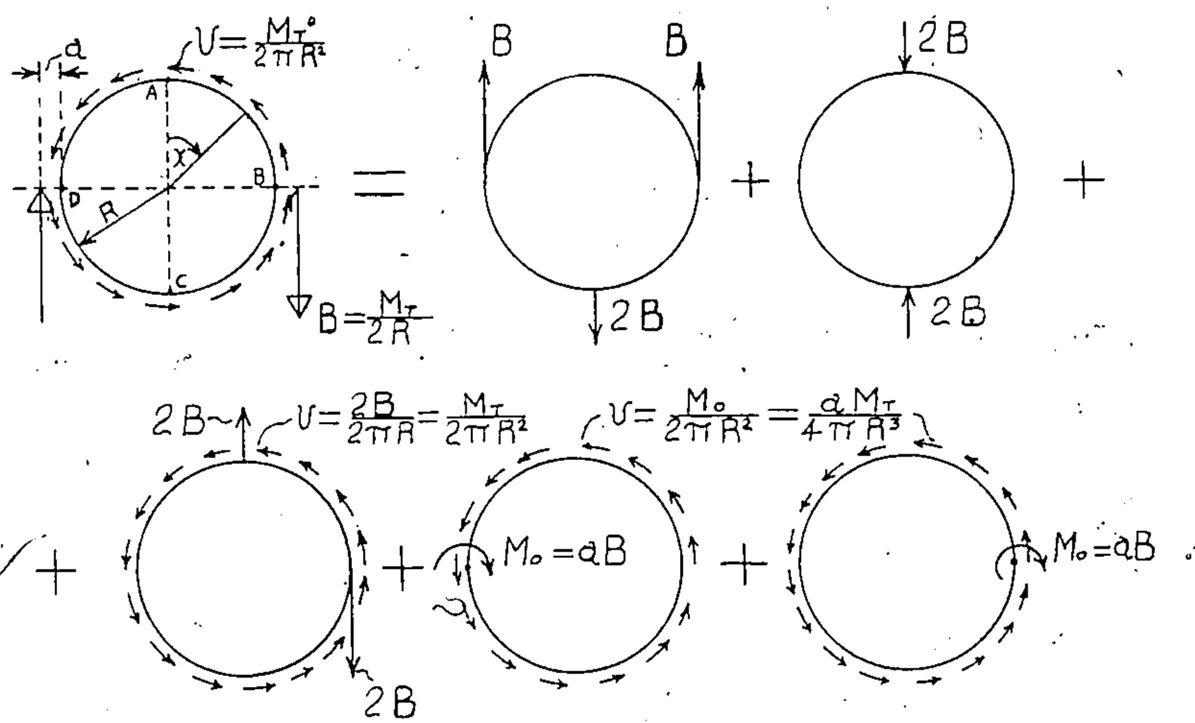


FIG. 5.7 Ring Girder Under Torque M_T

The maximum bending moment due to the combination of those cases occurs at the horizontal diameter:

$$\max M = \tfrac{1}{2} M_o = \frac{a}{4R} M_T \tag{5.7}$$

The maximum axial force occurs at the same point:

$$\max T = \frac{B}{2} = \frac{M_T}{4R} \tag{5.8}$$

The maximum combined stress at the horizontal diameter is:

$$f_2 = f_{bend.} + f_{axial} = \frac{aM_T}{4RS} + \frac{M_T}{4RA} \tag{5.9}$$

5.2.2 Horizontal Loads

The wind reaction, W, (over the inverted V-bents), is transmitted from the shell of the tube to the ring girders by tangential shear. For the analysis of the ring girder under horizontal loads the tables presented in reference [18] will be employed, which are presented schematically in Figure 5.8.

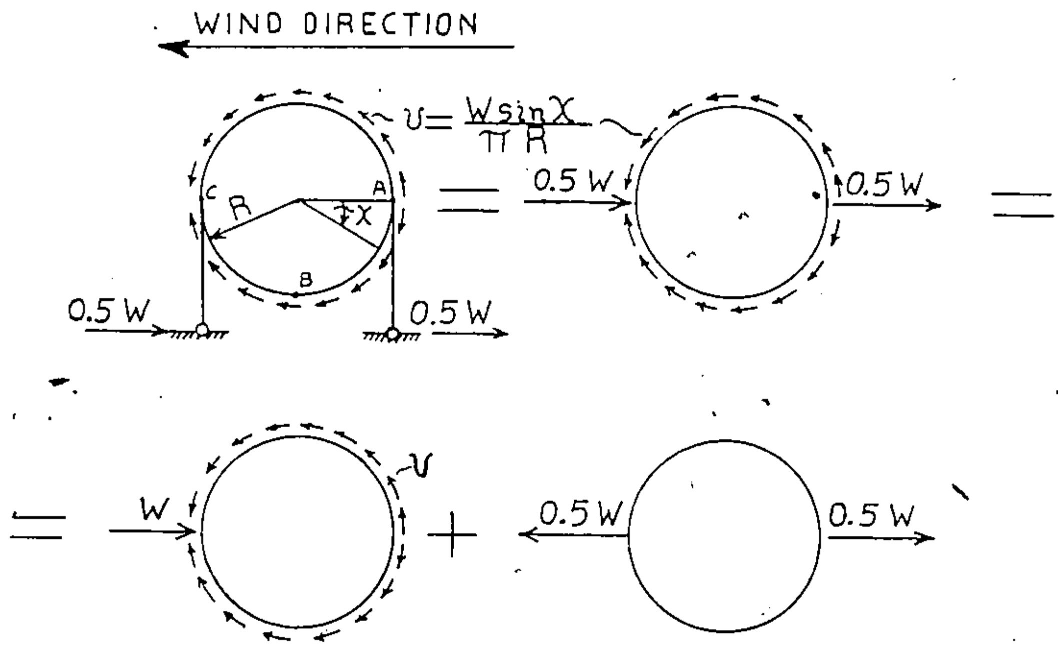


FIG. 5.8 Ring Girder Under Horizontal Load

$$M_C = -M_A = 0.0796\ WR \tag{5.10}$$

$$T_A = -T_C = 0.2387\ W \tag{5.11}$$

$$V_A = 0.0 \tag{5.12}$$

$$V_C = \frac{W}{2} \tag{5.13}$$

The maximum combined stress occurs at the horizontal diameter:

$$f_3 = \frac{\pm 0.08\ WR}{S} + \frac{\pm 0.24\ W}{A} \qquad (5.14)$$

5.2.3 Summary

a) Vertical loads

Ring girder above:

(1) Single post support:

$$f_1 = \frac{0.01\ QR}{S} + \frac{Q}{4A} \leq f_{allow.} \qquad (5.15)$$

(2) Inverted V-bent:

$$f = f_1 + f_2 = \frac{0.01\ QR}{S} + \frac{aM_T}{4RS} + \frac{Q}{4A} + \frac{M_T}{4RA} < f_{all.} \qquad (5.16)$$

b) Vertical and horizontal loads

Ring girder above:

(1) Single post support: Eq. (5.15).

(2) Inverted V-bent support:

$$f = f_1 + f_2 + f_3 = \frac{0.01\ QR}{S} + \frac{aM_T}{4RS} +$$

$$+ \frac{0.08\ WR}{S} + \frac{Q}{4A} + \frac{M_T}{4RA} + \frac{0.24\ W}{A} \qquad (5.17)$$

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