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Loads for Design of Stacking Tubes for Granular Materials

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Abstract

As yet, there is inadequate published information for the structural design of stacking tubes for use with granular materials. This paper (the first of a two-paper series) suggests load conditions and load combinations that should be considered in structural design. Where the method of computing those loads is not familiar, the paper illustrates methods which may be used for their computation. Also given are the results of qualitative tests performed by the authors on two small model stacking tubes.

A second paper will address the design procedures for stacking tubes of steel or of reinforced concrete. Illustrative design examples will also be given.

1. Introduction

A stacking tube (also known as a "lowering tube" or "lowering tower") is a vertical tube of either reinforced concrete or metal, having openings in its walls at various elevations (Fig. 1). Pairs of diametrically opposite openings are usually used, each pair being staggered at 90° from the pair above and the pair below. The stacking tube is used to allow smooth stockpiling of granular materials without causing excessive dust. Material enters the top of the tube from an overhead conveyor and spills out of the tube by the lowest available opening that is not already surrounded by the conical stockpile of material outside. When the level of material covers an opening of the tube, discharge from that opening ceases and further discharge onto the stockpile continues from the next higher opening (or pair of openings).

Diameter and height of stacking tubes vary depending on the desired volume of the stockpile. For reinforced concrete stacking tubes, heights are as much as 150 ft (46 m) and diameters usually range from 6 to 15 ft (1.8 to 4.6 m).

Stacking tubes are sometimes built directly on top of a reclaiming tunnel (Fig. 2) containing a conveyor by which

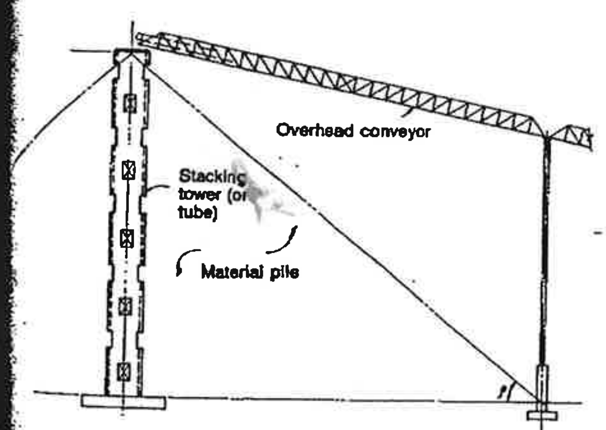

Fig. 1: General view of stacking tube

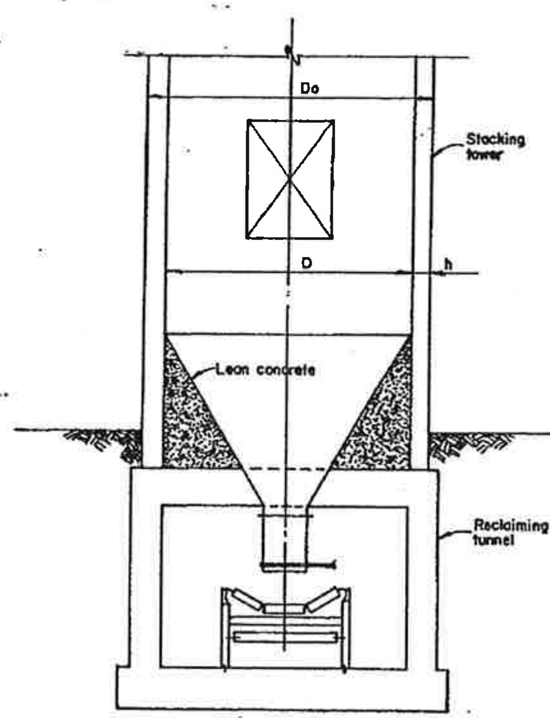

Fig. 2: Stacking tower on top of reclaiming tunnel

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stored material is withdrawn from the stockpile around the tube. In such cases, the tube will have not only side openings for discharge onto the stockpile, but also a bottom, hopper-shaped opening for direct discharge from the tube onto the tunnel conveyor. If the material to be handled is prone to deterioration due to long-term storage and a bottom reclaim tunnel is not provided, the tube bottom is sloped so that the tube is self-cleaning, the material flowing easily out of the side openings nearest the bottom. For materials that are not apt to deteriorate under long-term storage (limestone or sand, for example) this precaution is not necessary and the tube bottom may be flat.

2. Design

Two types of design are required:

1. Functional design, which involves choosing the diameter, height, method of operation, opening size and opening position, so that the tube will create the desired volume and shape of stockpile and so that material flow will be satisfactory.
2. Structural design, so that safety (and reasonable economy) are assured under all loading conditions.

This paper addresses only the latter concern — structural design.

To date, no United States code or standard gives structural design criteria specifically for stacking tubes. The most badly needed criteria are those concerning design loads. This paper suggests design loadings and shows qualitative experimental evidence to support some of them.

3. Suggested Loadings for Structural Design

Loadings which the authors believe should be considered in structural design of stacking tubes are the following:

A. Dead load
B. Live loads
   1. Conveyor loads
   2. Wind loads
   3. Stored and stacked material loads
   4. Seismic loads

3.1 Dead Loads

Dead loads to be considered for stacking-tube design include the weight of the tube itself and the dead-load portion of the reaction of the overhead conveyor system, if that system is attached to the top of the tube. For purposes of computing horizontal seismic forces on the tube, the weight of stored material contained within the tube should also be considered as dead load.

3.2 Live Loads

Stacking tube live loads are of four kinds: conveyor loads, wind loads, stored and stacked (outside) material loads, and seismic loads.

3.2.1 Conveyor Live Loads

Conveyor live loads that should be considered are:

1. Vertical reaction (at the top of the tower) to the weight of material carried by the conveyor.
2. Friction load, parallel to the conveyor and acting in either longitudinal direction. This would be the friction force due to expansion or contraction of the conveyor support structure. The authors feel that if the magnitude of this force is not specified by the owner, the structural designer should assume a force of at least 10% of the *total* (dead plus live) vertical reaction of the conveyor system on top of the tube.
3. Belt tension, if the conveyor is so constructed that the belt tension must be transferred to the tube, rather than being resisted by the conveyor support structure itself.

3.2.2 Wind Loads

If it is permitted by the applicable code or standard being observed, the allowable stress for load cases that include wind may be increased above that allowed for dead and other live loads alone. (Or, if the design is by strength methods, the combined factored loading may be reduced.) Wind loads acting on the tube vary according to the degree to which the tube is surrounded by the stockpiled material. Specific cases that the structural designer should consider are:

1. Static wind pressure on the projected area of the *entire height of the tube*, with the tube empty and no stockpiled material against the outside of the tube. Various codes and standards [1], [2] give design wind pressures for circular structures. If wind pressures for design are not specified by the applicable code or standard, the authors suggest using the American National Standard ANSI A58.1 [2]. This standard gives design wind pressures at various heights on the structure and considers structure shape, purpose and locality. In special locations, local records may indicate that the specified loads are too low. In these cases, the designer should seriously consider using wind pressures higher than the minimum specified. Wind force transferred to the top of the tube by the conveyor structure should be considered in combination with the force from wind on the tube itself.
2. Static wind pressure on the portion of the tube that is exposed above the level of stacked material. This pressure should be considered along with the lateral pressure applied by the stacked material. Perhaps the worst, but entirely possible condition would be wind on the exposed portion of the tube, with the stockpile on the leeward side being partially or completely absent. This eccentric condition will result when material is removed only from one side of the stockpile. This load combination may be critical for vertical compression on the stacking tube wall. Again, wind force from the conveyor structure above should be combined with forces of wind on the tube.
3. Variation of wind pressure around the circumference. This will cause horizontal bending moments in the tube wall. The varying radial pressures have a resultant in the direction of the wind, and this resultant is resisted by shearing forces in the wall, as shown in Fig. 3. Bending moments due to these varying pressures are more likely to be significant in steel stacking tubes than concrete ones.

3.2.3 Stored Material Loads

These should include loads both by material within the tube and by material stacked outside. Except as noted for seismic conditions under dead loads, above, all pressures and frictional forces due to stored materials should be treated as live

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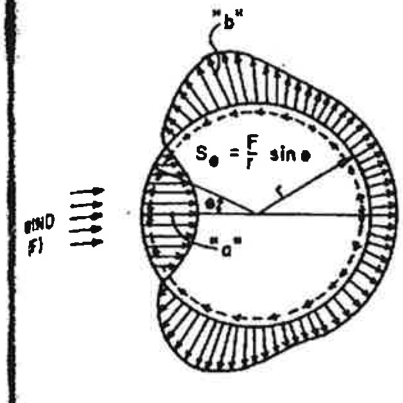


Fig. 3: Distribution of wind pressure on tube and shear stresses in wall

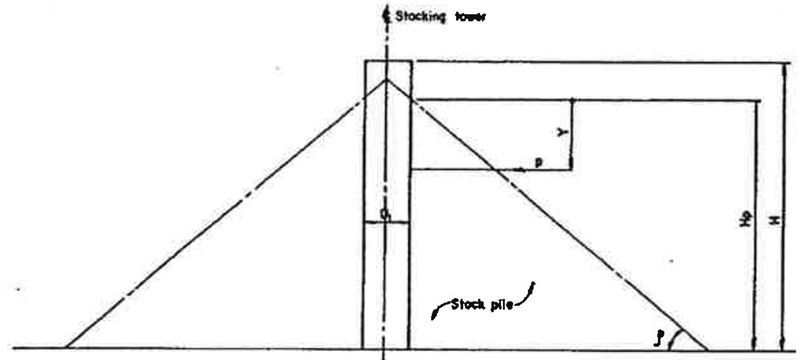


Fig. 4: Uniformly distributed stockpile, surface sloping at angle of repose φ

loads. The following stored-material load conditions are suggested, and these are based partly on the qualitative experimental studies reported later in this paper:

1. Stacking tube full, but the outside stockpile absent. This condition is, perhaps, only remotely possible, but it might occur if all the supposedly self-opening gates on the sides of the tube were frozen or rusted shut, or if the tube were filled with moist material that had become frozen or cemented together so that it would not flow as expected. For this load case, the radial outward pressures would be computed as for a silo, using, for example, either the Janssen or Reimbert equations for lateral pressure [5], [6], [7].
   For computing the vertical force in the tube wall, friction due to the stored material in the tube should also be considered.
2. The exact opposite of Load 1 above, with the stockpile completely in place all the way to the uppermost opening and uniformly distributed all around the outside of the tube, but with the tube itself empty. This would be possible only if the tube had a bottom discharge opening into the conveyor tunnel below. This load condition would cause a hoop compression in the tube wall.
3. The same as Load 2 above, but with the tube full also. This condition would cause lesser horizontal hoop compression or tension in the tube walls, but would result in the largest value of vertical load in the wall due to downward friction of both the inside and outside material.
   If the outer lateral pressure for this condition is larger than the silo pressure computed for the material stored within the tube, then the inside pressure resisting inward movement of the tube wall may be taken as high as the passive pressure for the inside material. The authors suggest a limit for this passive pressure equal to twice the active pressure value computed by the Janssen or Reimbert equations (Fig. 4).
4. Tube either full or empty (whichever is more serious) and one sixth of the outside stockpile removed in a 60° sector, as shown in Fig. 5. This lateral load condition, illustrated clearly by the qualitative experiments, causes an overturning moment. This moment, in turn, causes large vertical bending stresses which must be combined with the stresses due to the simultaneous vertical loads. (A method of analyzing for this condition is illustrated later.)

3.2.4 Seismic Loads

Seismic forces acting on a stacking tube are illustrated in Fig. 6. They consist of:

1. A lateral force E_1 due to seismic action on the weight of the conveyor and the material it carries.
2. A lateral force E_2 due to seismic action on the weight of the tower itself and on the weight of material stored in the tower. The authors recommend using the entire weight of the inside stored material to compute the seismic force, rather than the 80% used for silo design in ACI 313—R77 [5].

Fig. 5: Determination of the design lateral loads (one sixth of stockpile removed)

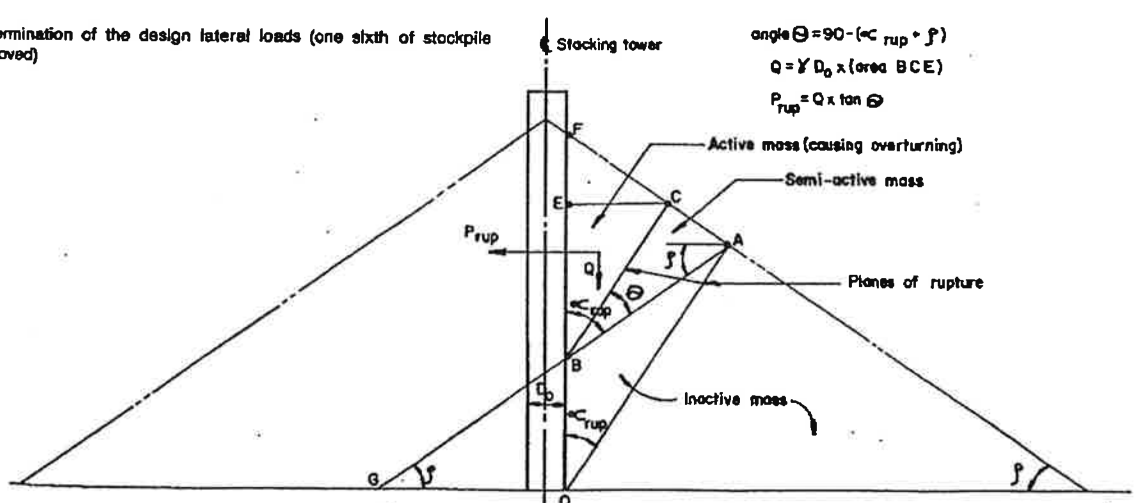


FIG. 5

3. A lateral force E_3 due to seismic action of the outside stockpile, computed as a product involving either Q_3 (Fig. 6) or Q (Fig. 5), whichever controls. For cases in which the stockpile is uniformly distributed all around the cylinder, the authors would assume that the seismic force from material stacked against one side is resisted by passive pressure of the material on the opposite side. However, for the condition in which one quadrant of material has been removed, this seismic force could be significant.

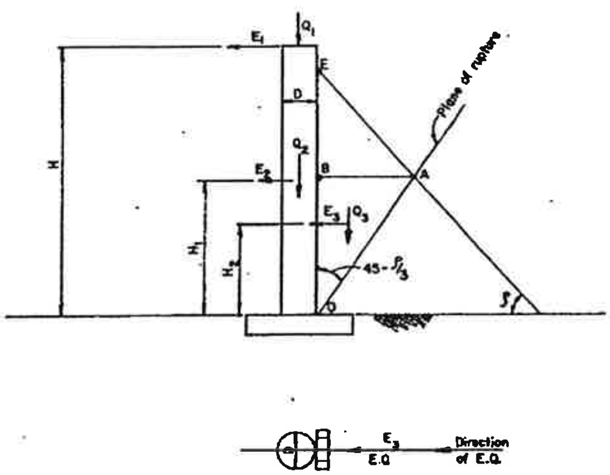


Fig. 6: Seismic forces acting on stacking tube

4. Load Combinations for Design

Load combinations that are reasonably possible should certainly be considered by the designer. However, such combinations as full wind and full earthquake force are extremely unlikely, and most designers would not consider them. Combinations the authors would consider are the following:

Combination A:
- Tube full
- External stockpile complete
- Conveyor live and dead loads
- Tube dead load
- Wind on conveyor
- Wind on exposed part of tube
- Longitudinal force from conveyor (if it causes higher tube stress).

Combination B:
Same as A, but without wind.

Combination C:
- Tube full
- Stockpile complete except for a 60° sector
- Tube dead load
- Conveyor dead load
- Conveyor live load (if it causes higher tube stress)
- Wind on conveyor
- Wind on exposed part of tube
- Longitudinal force from conveyor (if it causes higher tube stress).

Combination D:
Same as C, but without wind.

Combination E:
- Tube full
- Outside stockpile absent
- Wind on full height of tube
- Tube dead load
- Conveyor dead load
- Longitudinal force from conveyor (if it causes higher tube stress).

Combination F:
(if there is a bottom reclaim tunnel)
- Tube empty
- Outside stockpile complete.

Combination G:
Same as A, but with earthquake rather than wind.

Combination H:
Same as C, but with earthquake rather than wind.

Combination I:
Same as E, but with earthquake rather than wind.

5. Pressure and Force Computation

5.1 Lateral Pressure of Stacked Material

Lateral pressure applied to the walls by the outside stockpile may be computed using Reimbert's approach for retaining walls [8], assuming that the surface of the pile is inclined at the angle of repose for the material. By this approach, the active unit pressure at height Y per linear horizontal width of wall is

$$p_a = \gamma \cdot Y \frac{(1 - 2\,\varrho/\pi)^3}{(1 + 2\,\varrho/\pi)^2} \quad (1)$$

The total active pressure per unit width of wall at the bottom of the wall is

$$P_a = \frac{\gamma \cdot H_p^2}{2} \cdot \frac{(1 - 2\,\varrho/\pi)^3}{(1 + 2\,\varrho/\pi)^2} \quad (2)$$

The passive pressure of the stockpiled material (useful for load combinations that tend to cause overturning) is given by A. and M. Reimbert [8] as unit pressure at height Y equal to

$$p_p = \gamma \cdot Y \frac{(1 - 2\,\varrho/\pi)^2}{(1 + 2\,\varrho/\pi)} \quad (3)$$

Similarly, the total force due to passive pressure of the stockpiled material is

$$P_p = \frac{\gamma \cdot H_p^2}{2} \cdot \frac{(1 - 2\,\varrho/\pi)^2}{(1 + 2\,\varrho/\pi)} \quad (4)$$

5.2 Friction by Stockpile Material

According to the Reimbert experiments, friction forces on the outside of the tube (the downward components of the pressures from the stacked material) are independent of the magnitude of pressure against the walls. To compute these forces, the Reimberts suggest assuming the angle of friction of the material against the walls to be equal to the angle of repose. Thus, the outside frictional force per unit area at depth Y is

$$V = p \cdot \tan \varrho \tag{5}$$

and the total frictional force per unit width of wall at the bottom of the wall is

$$\Sigma V = P \cdot \tan \varrho \tag{6}$$

The total vertical force per unit width of wall will include the value given by Eq. (6) above as well as the vertical force due to dead load and other live loads.

5.3 Friction from Inside Stored Material

Vertical friction from the material stored inside the tube may be computed by either the Reimbert or Janssen methods [3], [7], [8] for tubes of 8 ft (2.4 m) diameter or larger. For tubes of smaller diameter, the authors suggest that the total downward friction force from the inside stored material be assumed at about 75% of the weight of the inside stored material.

5.4 Pressure and Overturning Moment from External Stockpile with 60° Sector Removed

Fig. 6 shows a method of computing this force and the overturning moment it causes. According to Reimbert [8], the angle of rupture is

$$\alpha_{rup} = (45 - \varrho/3) \tag{7}$$

(Traditionally, a similar equation is used, but with a "2" in the denominator rather than a "3".) In Fig. 5, the plane of rupture is drawn through point 0 at the tube centerline, intersecting with the stockpile surface at point A. Next draw line AB, its slope with the horizontal being equal to the angle of repose. Point B on this line is at the tube centerline. From point B, draw line BC, its slope from the vertical being equal to the angle of rupture. Point C is at the intersection of line BC with the stockpile surface.

The total weight of the trapezoidal mass of material (shown crosshatched) is

$$Q = \gamma \cdot D_0 \cdot Area\ BCE \tag{8}$$

where D is the outside diameter of the tube. The design value of force Q is

$$P_{rup} = Q \cdot \tan \theta$$

where

$$\theta = 90 - (\alpha_{rup} + \varrho) \tag{9}$$

This design force is assumed to act horizontally through the centroid of triangle BCE, and knowing the amount and location of the force, the overturning moment at any elevation may be computed. *Mass BCE* is assumed to be significant in computing the overturning moment. The semi-active *Mass ABC* is usually considered to have a negligible effect on the overturning moment.

6. Seismic Forces

Seismic forces for structural design may be estimated in several ways. A relatively simple approximation is shown here. More precise methods, if available and practical, should be used in lieu of the approximation. For reinforced concrete stacking tubes, Appendix A, Special Provisions for Seismic Design, of ACI 318—83, Building Code Requirements for Reinforced Concrete [3], should be observed where applicable.

The direction should be the same for all the seismic forces shown in Fig. 6; this is consistent with the assumption of first-mode action. Loads E_1 and E_2 should be determined following the method of the Uniform Building Code (UBC) [1]. These forces would then be given by

$$E_1 = Z \cdot 0.1\ Q_1 \tag{10}$$

$$E_2 = Z \cdot 0.1\ Q_2 \tag{11}$$

in which Z is the seismic zone factor. Values of Z are given by the UBC as 1.0, 0.75, 0.5 and 0.1875 for earthquake zones 4, 3, 2 and 1, respectively.

To determine load E_3 due to the material stockpile with a 60° sector removed, the authors suggest the following:

1. Assume that material in the shaded region shown in Fig. 6 (*Area ABO*) is effective and that the force E_3 acts at the centroid of triangle ABO. Similarly, assume that material in the *Area BCE* (Fig. 5) is effective and that the force E acts at the centroid of triangle *BCE*.
2. Consider the volume of effective material to be the *Area ABO* multiplied by the tube outside diameter D.
3. Assume that the magnitude of force E_3 is given by

$$E_3 = Z \cdot 0.1\ Q_3 \quad \text{(or } Q \text{ in Fig. 5)} \tag{12}$$

where Q_3 (or Q) is the weight of the effective material.

The total base shear due to seismic forces is

$$V = E_1 + E_2 + E_3 \tag{13}$$

and the overturning moment about the base is (Fig. 6)

$$M = E_1 H + E_2 H/2 + E_3 H_2 \tag{14}$$

If $E_3 H_2$ computed from Fig. 6 is less than that determined from Fig. 5, then the value obtained from Fig. 5 should be used instead.

Seismic forces and the overturning moment can act in any horizontal direction. Vertical seismic forces are also possible, but they are normally considered to be of less importance in design than the horizontal ones.

The authors would use the same approach with respect to seismic forces for stacking tubes of either material — steel or reinforced concrete. As for load combinations that include wind, design codes either permit higher stress levels or require lower load factors for the design of structures subject to combinations that include seismic loads. (The authors do not allow such stress increase for the design of anchor bolts, however.)

7. Qualitative Test of Model Tubes

To help establish suitable design criteria, the authors performed simple tests on two cardboard model stacking tubes (Fig. 7). These were loaded with sand to create a complete surrounding stockpile. The smaller tube was also loaded with powdered coal. In each case, the surrounding material was then removed from one 60° sector to create an unsymmetrical load condition.

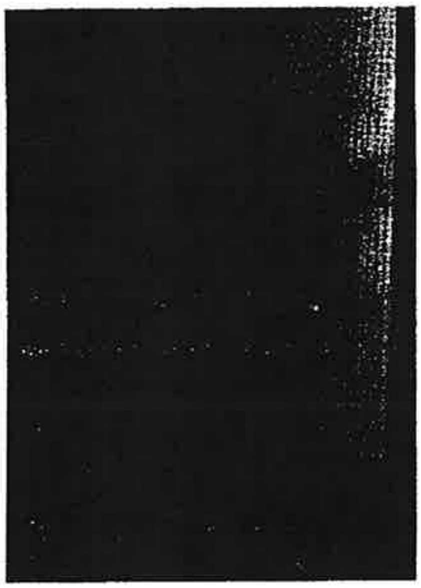


Fig. 7: Model stacking tubes A and B

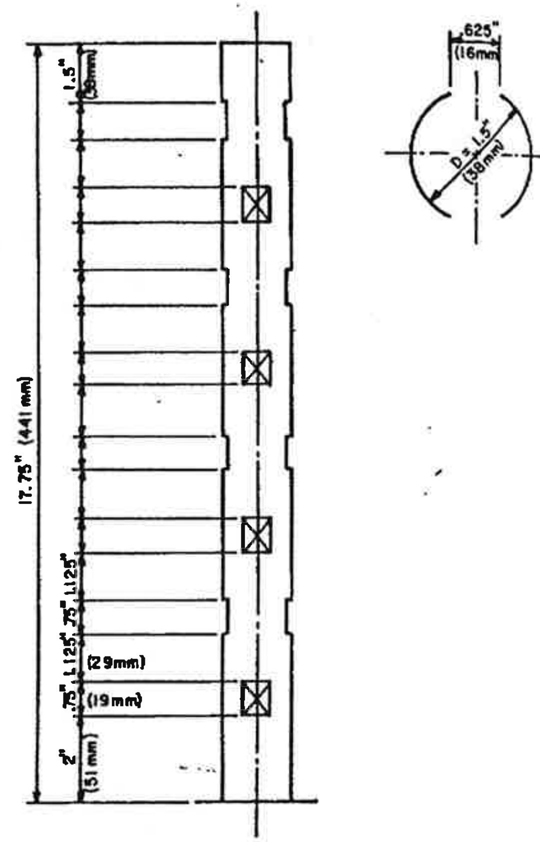
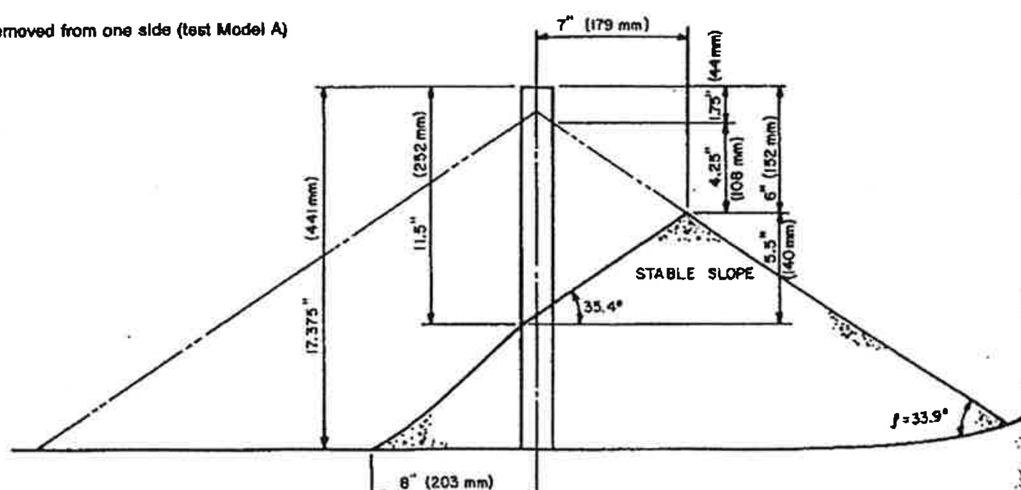


Fig. 8: Stacking tube (test Model A)

Fig. 8 shows the dimensions of Model A, the smaller model. This model was first loaded with dry coal of 1/8 inch maximum size. The coal was fed into the top of the tube and allowed to flow out of the side openings to form the surrounding cone of material. After the cone was complete to the height of the top of the upper openings, coal was gradually removed from one 60° sector around the tube to create the worst unsymmetrical load condition, as shown in Fig. 9. Material was then removed from the opposite side of the cone to create the worst condition causing horizontal bending (ovalling) of the tube. The same procedure was followed for the test with sand, the sand having a maximum grain size of 3/16 inch, but the majority being about 1/32 inch. The sand was partly dry and partly damp (Fig. 10).

Fig. 11 shows the dimensions of Model B, the larger model. This model was tested with sand only. Figs. 12 and 13 show Model B with sand removed from one sector and two opposite sectors, respectively. Measurements were made at two stages as sand was removed from one side. These measurements are shown in Fig. 14.

Fig. 9: Coal pile removed from one side (test Model A)



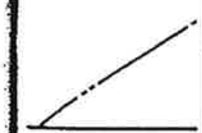


Fig. 10: Dry sand pile

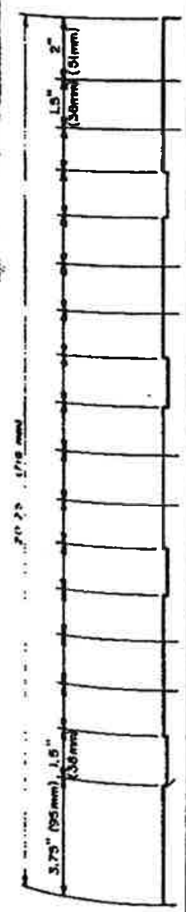
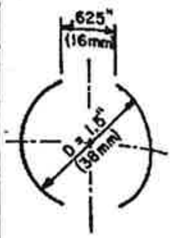


Fig. 11: Stacking tube



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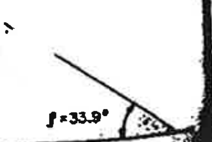

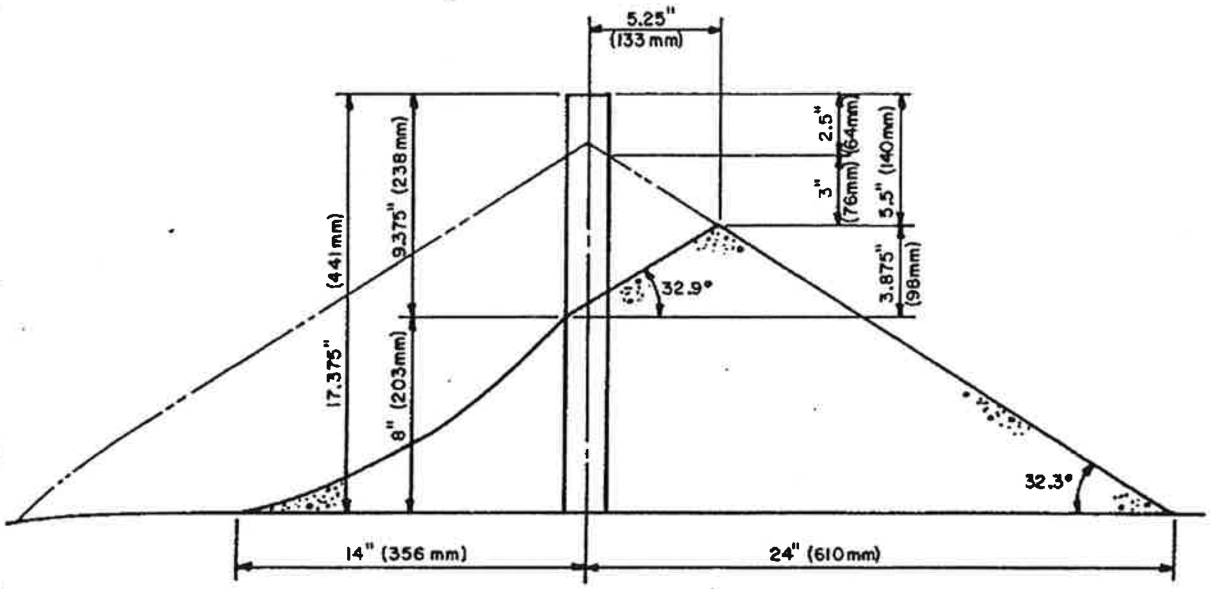

Fig. 10: Dry sand pile removed from one side (test Model A)

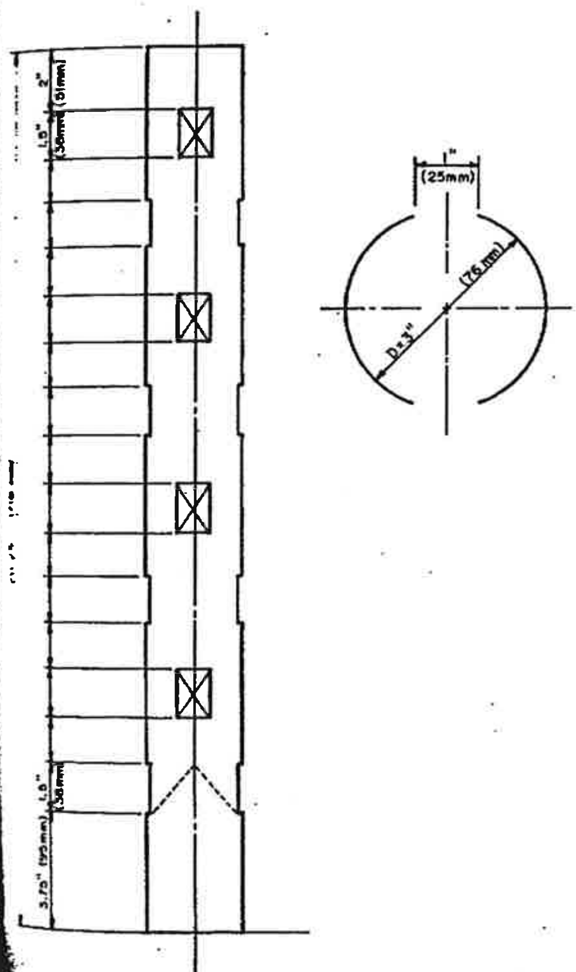

Fig. 11: Stacking tube (test Model B)

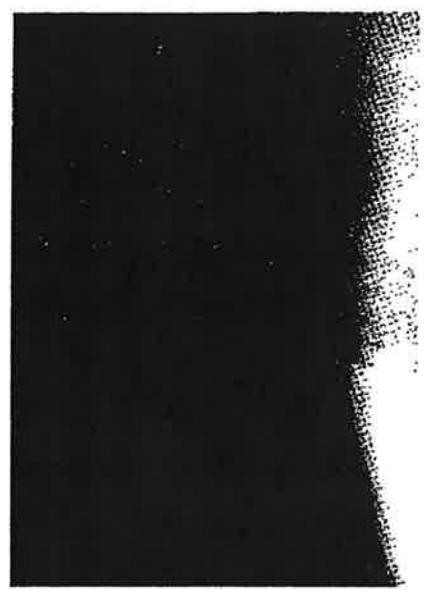

Fig. 12: Sand removed from one 90° quadrant (test Model B)

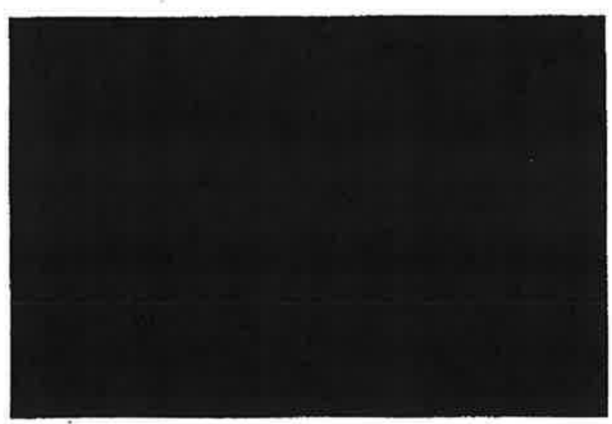

Fig. 13: Sand removed from opposite sides (test Model B)

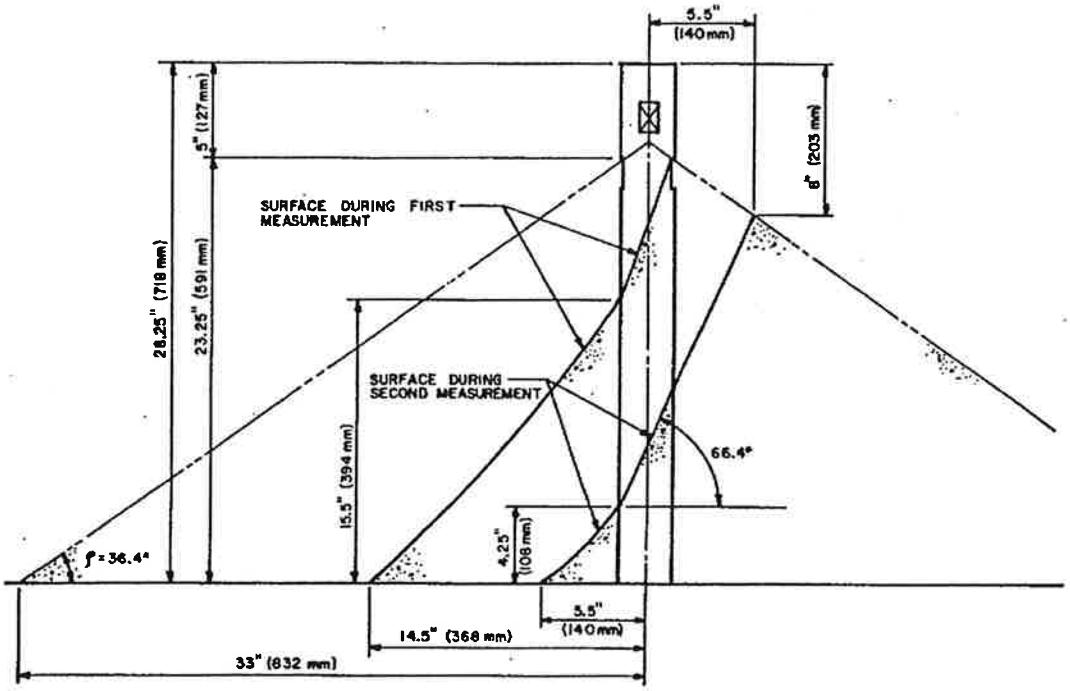

Fig. 14: Semi-wet sand pile removed from one side (test Model B)

8. Observations and Conclusions

When material was gradually removed from one 60° sector of the stockpile, a steep but unstable slope developed in the material at either side of the sector. The height and slope of the mass on either side decreased as material fell into the area from which material was being removed. Material directly opposite the region from which material was being removed fell in both directions around the tube (if the material was dry). This flow would continue until the surface slope was at the angle of repose for the material.

Damp material did not flow around the tube so readily, and the pile of material opposite the area of removal remained higher. This would suggest that the overturning moment on the tube would be higher for cohesive materials than for non-cohesive. Since stacking tubes are used outdoors, it is reasonable to expect that the stacked material will be damp and that its cohesiveness will be high. Thus, the designer should be conservative in estimating the overturning moment due to unsymmetrical loading from the stacked material.

The tests showed that the total lateral pressures were lowered when material was removed from two opposite sides of the tube, so that this load condition was not so serious as that with material removed from one side only. The difference in material level next to the tube was so low as to suggest that the tendency to cause horizontal bending (ovalling) was negligible.

9. Structural Design

A future paper will address the actual structural design of stacking tubes, and design examples will show features of design for both steel tubes and reinforced concrete tubes.

Acknowledgements

The authors gratefully acknowledge the cooperation of their staff members at SMH Engineering, Inc., Lakewood, CO and the University of Colorado, Denver, CO.

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Design of Stacking Tubes for Granular Materials

Part II

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Summary

A previous paper by the authors [1] concerned loads and load combinations for use in the design of stacking tubes and reported on qualitative tests performed to observe the behavior of stacked material during withdrawal. This second paper addresses the structural design proper for both steel and reinforced concrete tubes. Design examples are given following the load conditions suggested in the earlier paper.

1. Introduction

The structural design of a stacking tube may either follow or be carried out simultaneously with the functional design that is needed to determine the proper height and diameter and the proper size and location of the discharge openings. The structural design includes checking the stability of the tube under all reasonable loading conditions and selecting the tube material, thickness and reinforcement or stiffening. ...ough the design principles are the same for tubes made of concrete as for those made of steel, factors which are of great importance for the design with one material may be much less important for another. For this reason, steel tube design and reinforced concrete tube design are dealt with separately.

In any case, the design criteria and methods presented here are those which the authors would use; they are not requirements spelled out by codes or standards written especially for stacking tube design.

2. Design of Reinforced Concrete Stacking Tubes

Stacking tube walls may have different thicknesses at different heights, which can complicate the construction. Therefore, the walls are usually selected to have the same thickness for the full height. The authors believe that the minimum acceptable wall thickness is 6 inches (150 mm).

Because of potential damage to the wall by the power equipment used to remove the material stacked outside, an additional protective concrete cover (beyond the normally required cover) should be provided above the reinforcing bars. The authors recommend an additional one inch of cover for the outside of the lower 10 to 15 ft (3 to 4.6 m) of the wall, which may be provided by making the wall thicker from the base to the bottom of the first opening, with the additional thickness being on the outside.

Walls 8 inches or thicker should be reinforced with two layers of reinforcement in each direction, vertical and horizontal. One of the layers should be near the outside face and one near the inside face of the wall. The area of reinforcement in each direction should in no case be less than 0.003 times the gross wall area per unit width.

The bottom of the walls should be dowelled into the foundation, the dowels being at least equal in size, number and location to the vertical bars required for the wall itself at foundation level. The dowels should have a sufficient lap length with the vertical bars above and a sufficient embedment in the foundation below to develop the full tensile strength of the dowels. If the tube rests on the roof of a reclaim tunnel, the dowels must be developed by anchorage to the tunnel roof structure, and the tunnel structure itself must be strengthened to resist the forces applied to it by the tube.

The critical level of stress in the tube proper is usually either at the bottom of the tube or at the bottom of the lowest opening. Both locations must be investigated. Other potential critical locations are just above elevations where the wall thickness or the reinforcing steel is reduced and at the bottom of the first opening above those locations. At openings, the net properties of the tube cross sections must be used. If staggered pairs of openings are closely spaced vertically, the designer should consider whether to assume the complete group of four holes to be effective in reducing the area and the section modulus of the tube at any level. (If St. Venant's idea is applied, the effect of holes probably does not dissipate in a distance less than one-half of the tube circumference. If this is true, the possible overlapping effects of vertically adjacent pairs of openings should be questioned whenever the clear vertical distance between pairs of holes is less than one-half of the tube circumference.)

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...I of this paper, Loads for Design of Stacking Tubes for Granular Mate... was published in bulk solids handling Vol. 5(1985) No. 2, pp. 349—356.

2.1 Working Stress Analysis

For working stress design (ACI 318 refers to it as "the alternate method"), the following equations (taken from Ref. [2]) may be used to compute the stresses in the concrete and in the vertical reinforcement:

$$f_{c1} = \frac{W(\cos\beta - \cos\alpha)}{2rh(1-p)(\sin\alpha - \alpha\cos\alpha) - (1-p+np)(\sin\beta - \beta\cos\alpha) - np\pi\cos\alpha} \quad (1)$$

$$f_c = f_{c1}\left[1 + \frac{h}{2r\cos\beta\,(\cos\beta - \cos\alpha)}\right] \quad (2)$$

$$f_s = nf_{c1}\left[\frac{1+\cos\alpha}{\cos\beta - \cos\alpha}\right] \quad (3)$$

Where:

- W — total vertical load applied to the horizontal cross section in question, including any load above that section due to both internal and external friction due to the stored or stacked material
- r — mean radius of the tube
- h — wall thickness
- p — ratio of total area of vertical steel to total area of concrete at that cross section (vertical steel should include only that which can be fully stressed at that location; where the cross section includes an opening, vertical steel above and below the opening should not be included)
- n — ratio, modulus of elasticity of steel divided by that of concrete
- α — one-half the central angle subtended by the neutral axis as a chord on the circle of mean radius r
- β — one-half the central angle subtended by the opening as a chord on the circle of mean radius r
- M — vertical bending moment
- e — ratio M/W.

Angle α is determined from the following equation:

e/r

$$c = \frac{(1-p)(\alpha - \sin\alpha\cos\alpha) - (1-p+np)(\beta + \sin\beta\cos\beta - 2\sin\beta\cos\alpha) + np\pi}{2\,[(1-p)(\sin\alpha - \alpha\cos\alpha) - (1-p+np)(\sin\beta - \beta\cos\alpha) - np\pi\cos\alpha]} \quad (4)$$

Runman [2] states that the above equations were derived for a case with one opening, but are satisfactory to use for two opposite openings as well.

For cases with no openings, the equations are simpler, since angle β is taken as zero.

2.2 Discharge Openings

Discharge openings in concrete walls usually have structural steel frames anchored into the wall concrete. Frames may be as simple as shown in Fig. 1 or as complex as shown in Fig. 2. The top and bottom members of the frames must be bent to match the curvature of the tube walls.

Openings may have dust flaps. When used, they may be of Armorite fabric or of neoprene hung from the top of the opening. Flaps may also be made of metal, hinged at the top. Stainless steel and abrasion-resistant (AR) steel plate are often used. The hinge allows the flap to swing freely to the outside under pressure from the material inside the tube.

If vertical or horizontal steel walls are interrupted by openings, any reinforcing interrupted should be replaced by 1.2 times as much added steel around the opening. One-half of the added vertical steel should be placed on each side of the opening. One-half of the added horizontal steel should be placed above the opening and one-half below. The authors suggest that the added horizontal steel be continued all around the tube with adequate tension laps. The added vertical steel should be extended both above and below the opening for a distance equal to at least the development length (in tension).

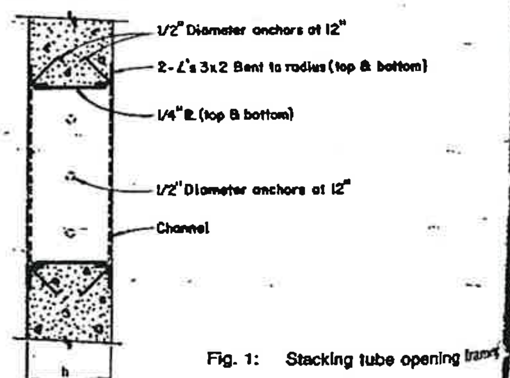

Fig. 1: Stacking tube opening frame

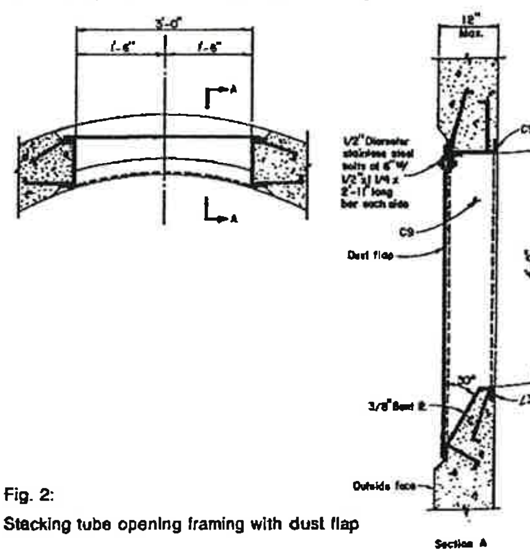

Fig. 2: Stacking tube opening framing with dust flap

2.3 Example 1

Fig. 3 shows the dimensions of a circular reinforced concrete stacking tube for coal, supported by a reclaim tunnel and located in seismic zone 2 (see Ref. [12]). The tube walls were designed using the following data:

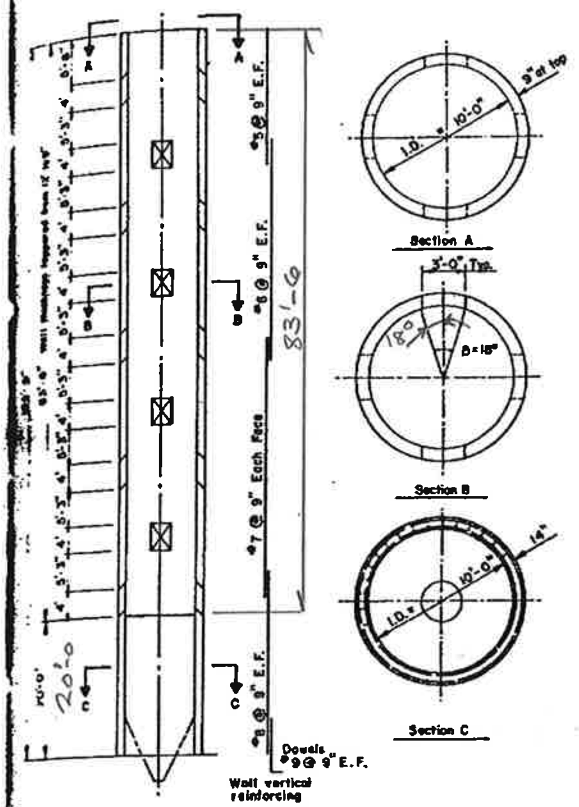

Fig. 3: Concrete stacking tube dimensions (Example 1)

1. *Coal properties:*
   - unit weight 55 lb/ft³ (pcf)
   - angle of repose 35°
   - coefficient μ' of friction against concrete 0.5 — 0.7
2. *Loads:*
   - wind loads see Fig. 4
   - conveyor loads:
     - gravity load at support 24,000 lb
     - longitudinal friction at support (due to expansion and contraction of conveyor assembly) 3,000 lb
3. *Construction material properties:*
   - 28-day strength of the concrete 4,000 lb/inch² (psi)
   - yield strength of the reinforcing steel 60,000 lb/inch²

To limit the length of this paper, only three of many possible load combinations will be considered, and only one will be solved. A complete design, of course, would require that many load combinations be considered and that stresses be checked at several elevations in the stacking tube. Only load combinations C, D and H are considered here, and stresses are computed only at the base and at the bottom of the lowest openings.

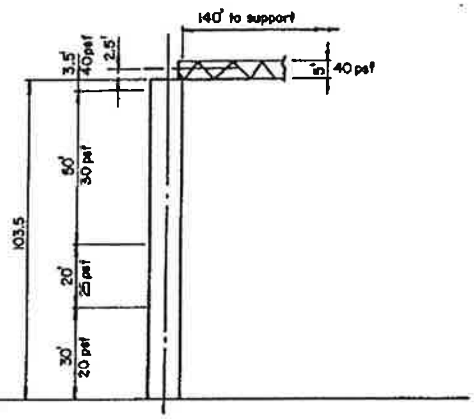

Fig. 4: Wind loads (Example 1)

Conveyor and conveyor supports	24 klb (kip)
Stacking tube (top to bottom of lowest openings	414 klb
	438 klb
Openings (deduct concrete weight)	—27.6 klb
Total dead load above a depth of Y = 83.5 ft	410.4 klb
Wall concrete from Y = 83.5 to bottom	123.2 klb
Total dead load above the base	533.6 klb

Properties of tube cross section:

— assumed thickness	14 inches
— outside radius	6.17 ft
— inside radius	5 ft
— gross area	41.06 ft²
— net area (two openings deducted)	34.04 ft²

Trial reinforcing:

— base dowels	# 9 at 9 inches each face (EF)
— vertical:	
— 0 to 25 ft (from bottom)	# 8 at 9 inches EF
— 25 to 56 ft	# 7 at 9 inches EF
— 56 to 86 ft	# 6 at 9 inches EF
— 86 to top	# 5 at 9 inches EF
— horizontal	# 4 at 12 inches EF

2.3.1 *Compute Pressures Due to Stored and Stacked Materials*

The inside loads are determined using Janssen's method with a coefficient of friction of 0.7. (This will give the maximum vertical load on the wall. Since the critical conditions for design of the tube wall include vertical bending as well as direct vertical compression, loads using the lower limit of a coefficient of friction of 0.5, or perhaps even zero friction, should also be analyzed). The Janssen equation for static lateral pressure is:

$$p_{st} = \frac{\gamma R}{\mu'}\left[1 - e^{-\mu' k Y/R}\right] \qquad (5)$$

where:

γ — unit weight of stored material, γ = 55 lb/ft³
R — hydraulic radius, R = 2.5 ft
μ' — coefficient of friction against concrete, μ' = 0.7
k = $(1 - \sin p)/(1 + \sin p)$, k = 0.271 for p = 35°.

At the bottom of the lowest openings, the storage depth is 83.5 ft. At this depth, Janssen's equation gives a static lateral pressure of 196 lb/ft² (psf) and a vertical pressure (equal to the lateral pressure divided by k) of 724 lb/ft².

The downward frictional load on the walls is:

$$V = (\gamma Y - q_{st}) \cdot R = 9{,}670 \text{ lb/ft}$$

(Verify that the total computed downward friction does not exceed the weight of coal in the tube. Multiplication by the inside perimeter, the total computed friction force above $Y = 83.5$ ft, is 303,800 lb, whereas the total weight of coal above that elevation is 360,700 lb. In further calculation, use the computed friction value.)

For calculation of the outside material pressure, the active pressure at a depth of 83.5 ft is determined as:

$$p_a = [55 \cdot 83.5\,(1 - 2 \cdot 35/180)^3/(1 + 2 \cdot 35/180)^2] \text{ lb/ft}^2$$
$$= 543 \text{ lb/ft}^2$$

and the total active force per unit horizontal width of the wall above 83.5 ft is:

$$P_a = (543 \cdot 83.5/2) \text{ lb/ft} = 22{,}670 \text{ lb/ft}$$

The (downward) friction load due to outside material at depth $Y = 83.5$ ft is:

$$V_{outside} = P_a \cdot \tan p = 22{,}670 \cdot \tan 35° = 15.87 \text{ k/ft}$$

The total outside friction can be calculated from the product of the outside perimeter and $V_{outside}$ as:

614.9 klb

The corresponding values computed at the base of the tube are:

Lateral inside pressure	196 lb/ft²
Vertical inside pressure	725 lb/ft²
Inside friction V	12,420 lb/ft
Outside lateral pressure	673 lb/ft²
P_a	34,800 lb/ft
Outside friction	24.36 k/ft
Total outside friction	943.9 klb

The friction loads are summarized as follows:

	At 83.5 ft depth	At the base
Inside friction load in klb	303.8	390.2
Outside friction load in klb	614.9	943.9

Whether or not to include the friction forces in the load combinations depends on the effect they will have. In some cases, higher stresses result when the friction is included, while in others, the more serious stresses occur if friction is absent. Certainly, when a 60° sector of material is removed, the outside friction is considerably reduced. For this condition the authors would use about one-half of the total outside friction load calculated. (In this example, the authors computed the stresses caused by combination D, both with friction and without friction; the larger stresses in both reinforcing steel and concrete resulted when friction was omitted from the loading.)

2.3.2 Compute Wind Forces and Overturning Moments

Table 1 summarizes the wind forces and the overturning m

Similarly, the base shear and the base moment at 83.5 ft depth are 29.3 klb and 1,773 ft-k, respectively.

The overturning moment due to a conveyor force of 3 klb is

$$3 \cdot 103.5 \text{ kft} = 311 \text{ ft-k}$$

2.3.3 Seismic Loads

The seismic loads at the two elevations used in the example are computed as shown in Tables 2 and 3 (see also Figs 5 and 6).

The loads computed are summarized in Table 4.

2.3.4 Stress Computation

The stresses are computed in the following way:

At depth $Y = 83.5$ ft (bottom of the lowest openings), $W = 410.4$ klb, $M = 7{,}392$ ft-k, $e = M/W = 18.0$ ft, $e/r = 3.225$ and the steel ratio $p = 0.01254$ (# 8 at 9 inches c/c).

ρ = 5.58

By trial and error, using Eq. (4), the angle α is found to be 65.5°. By substituting 65.5° for α and 18° for β in Eqs. (1), (2) and (3), the following values are obtained:

$$f_{c1} = 1.115 \text{ klb/inch}^2 \text{ (ksi)}$$
$$f_c = 1.331 \text{ klb/inch}^2$$
$$f_s = 23.4 \text{ klb/inch}^2$$

These values are lower than the allowable stresses of $0.45\,f_c' = 1.80$ klb/inch² for concrete and 24 klb/inch² for grade-60 steel, respectively, so that the design is satisfactory at depth $Y = 83.5$ ft for combination D (without friction).

At the base, similar computations were made, with angle β taken as zero (no openings). Also, similar computations were made at both depths for combination D including friction. The results of these computations are summarized in Table 5.

All concrete and steel stresses computed are below the allowable values of 1.80 klb/inch² and 24 klb/inch², respectively. Thus, the design is satisfactory for load combination D, but other combinations should also be investigated of course, e.g., combination H (which includes earthquake forces and moments), for which the results should be compared with 4/3 times the allowable stresses given above.

The tube is mounted above a reclaim tunnel, so that it is possible to have a full outside pressure of 673 lb/ft² while the tube is empty. By inspection, it is obvious that buckling due to external pressure will not occur.

2.3.5 Extra Reinforcing around Openings

The openings are 3 ft wide and 4 ft high. Following the requirement of ACI 313-77 [7], steel of an area 1.2 times that of the vertical and horizontal steel interrupted by the openings should be added:

— Vertical steel interrupted, 8 bars per opening: Addition of 1.2 · 8 bars = 9.6 bars, say 12 bars, or 6 bars on each side (3 per face, per side of opening) turning Moments

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Table 1: Wind forces and overturning moments (entire tube exposed)

	Pressure lb/ft²	0.6 · Area ft²	Force k	Arm ft	Moment about Base ft-k
On tube	20	219	4.4	15	66
	25	145	3.6	40	144
	30	356.3	10.7	75	803
	40	24.7	1.0	102	102
On conveyor and support	40	350	14.0	106	1,484
Total			33.7*		2,599**

19.7 k

1115 1-k

* base shear
** base moment

Table 2: Seismic load computation at the base

Source	Weight k	Force = 0.1 · 0.375 · Weight k	Arm ft	Moment ft-k
Tube dead load	533.6	20.0	51.75	1,035
Coal in tube	447.1	16.8	51.75	869
Conveyor	24	0.9	103.5	93
Stacked coal (Fig. 6)	958	35.9	66 · 2/3	1,580
Total				3,577

Table 3: Seismic load computation at depth Y = 83.5 ft

Source	Weight k	Force = 0.1 · 0.375 · Weight k	Arm ft	Moment ft-k
Tube dead load (above 83.5 ft)	410.4	15.4	41.75	643
Coal in tube	360.7	13.6	41.75	568
Conveyor	24	0.9	83.5	75
Stacked coal (Q3 in Fig. 6)	958	35.9	44 − 20	862*
or (Q in Fig. 5)	374 ✓	14.0	44	616
Total				2,148

* used for calculation of the total moment

Table 4: Summary of the wall loads for combinations C, D, and H

Source	At Y = 83.5 ft Vertical Load k	At Y = 83.5 ft Moment ft-k	At the Base Vertical Load k	At the Base Moment ft-k
Dead load	410.4		533.6	
Friction	802.4		1,171.5	
Outside stack with sector removed and conveyor force		6,634		8,467
Wind on exposed tube		758		972
Wind on conveyor		1,204		1,485
Earthquake		2,148		3,577
Totals:				
— comb. C, with wind	1,213	8,596	1,705	10,924
— comb. D, no wind	1,213	7,392	1,705	9,439
— comb. H, with EQ	1,213	8,782	1,705	12,044

w/o CNVR 6384

7392

w/o CNVR 8156

$P_{RUP} = 149^k$

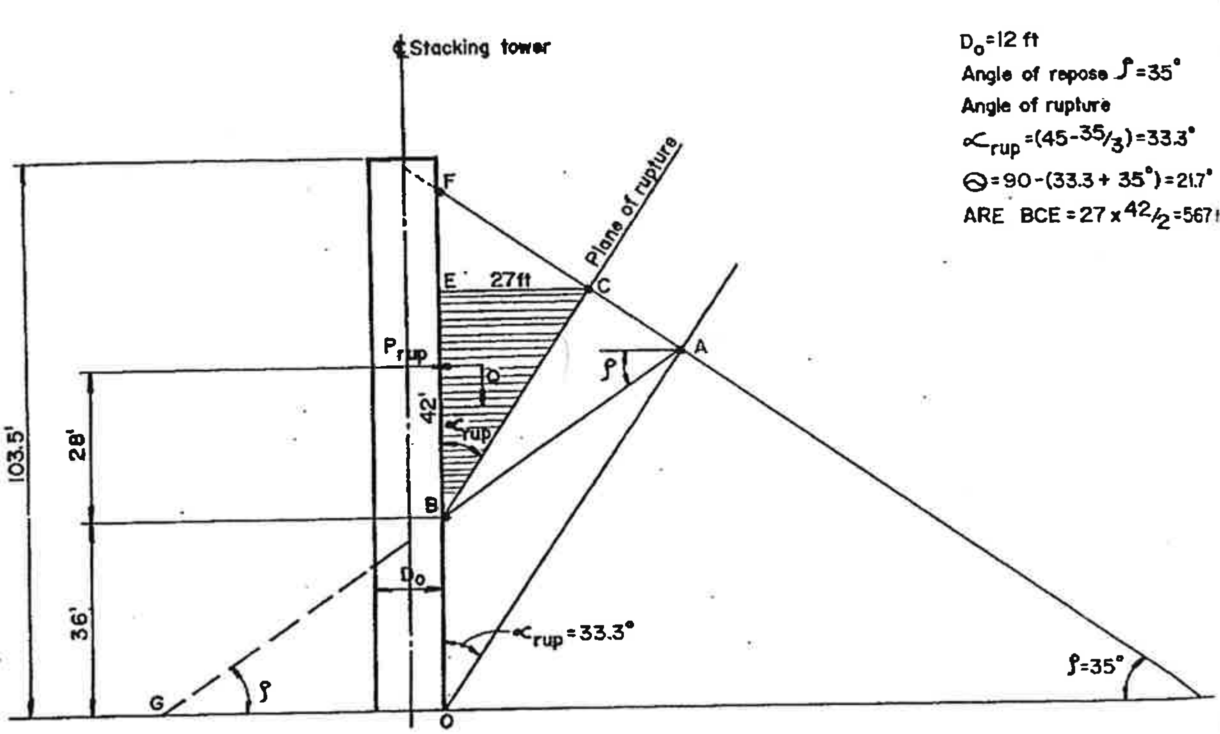

Fig. 5: Design lateral loads (Example 1)

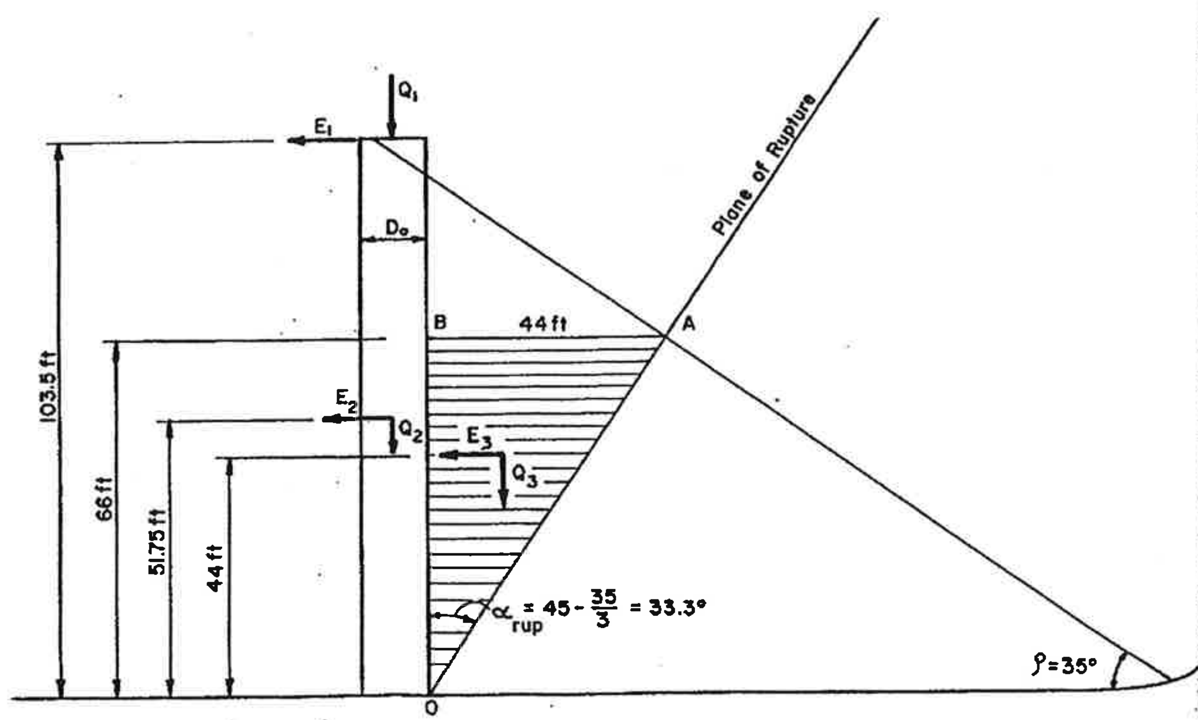

Fig. 6: Seismic loads (Example 1)

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3.1 Stresses in Cyli

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tubes of 10 ft (3 m)

Table 5: Stress computation for combination D

	At Y = 83.5 ft		At the Base	
	With Friction	Without Friction	With Friction	Without Friction
W in klb	1,013	410.4	1,392	533.6
M in ft-k	7,392	7,392	9,439	9,439
$e = M/W$ in ft	7.87	18.0	6.78	17.69
e/r	1.307	3.225	1.214	3.167
Steel ratio p	0.01254		0.015873	
Opening angle β in °	18		0	
Solved angle α in °	78	65.5	80	61
Stresses in klb/inch²:				
— f_{c1} for concrete	1.131	1.115	0.928	0.920
— f_c for concrete	1.297	1.331	1.048	1.107
— f_s for steel	14.65	23.4	10.57	21.20

– Horizontal steel interrupted, 8 bars per opening: Area of steel interrupted 8 bars · 0.2 inch²/bar = 1.6 inch². Addition of 1.2 · 1.6 inch² = 1.92 inch², or 0.96 inch² at the top and 0.96 inch² at the bottom; this could be provided by 4 added # 5 bars at the top and a like number at the bottom of each opening.

3. Design of Steel Stacking Tubes

Structural steel plates may be used to form a circular cylindrical stacking tube. Such tubes are much more flexible than reinforced concrete tubes; therefore, both vertical and horizontal buckling are important design considerations. Because of the buckling potential, some designers are reluctant to design stacking tubes in steel. The authors feel that, with proper design attention to the problems brought about by thin walls, a steel stacking tube should perform satisfactorily. The choice between steel and reinforced concrete will probably be made on the basis of economy, but durability should also be considered.

The main steps in the design of steel stacking tubes are:

1. Select wall thickness, which is usually based on the vertical wall stress and involves consideration of buckling
2. Design bottom ring-beam and anchor bolts
3. Design stiffeners for wall openings.

Types of steel ordinarily used for stacking tubes in the United States are:

- A-275 Grade C, yield point 30 klb/inch² (ksi)
- A-283 Grade C or D, yield points 30 and 33 klb/inch², respectively
- A-36, yield point 36 klb/inch².

3.1 Stresses in Cylindrical Steel Walls

3.1.1 Hoop Tension

The hoop tension alone does not determine the required thickness of a steel stacking tube wall. However, for stacking tubes of 10 ft (3 m) diameter or more, it may control the design. The horizontal hoop-tension force per unit height of wall is computed by:

$$F = p_{des} \cdot r \tag{6}$$

where p_{des} is the inside pressure of the stored material, computed by methods such as those developed by Janssen or Reimbert [4], [5], [6], and tension F is the force per unit width (if the pressure is measured in lb/ft² and the radius r in ft, the unit of F is lb/ft of plate width).

The American Petroleum Institute [8] recommends that the allowable tension stress be limited to 21 klb/inch², and that this be reduced by a "joint efficiency" factor of 0.95 for full penetration butt welds. DIN 419 [9] recommends an allowable tension stress equal to 2/3 the yield strength, with a joint efficiency factor of 0.8.

3.1.2 Vertical Compression

Vertical compression stress may result either from direct vertical load or from vertical load and bending. The wall of a steel stacking tube may fail either by overall, column-type buckling or by local buckling, or by interaction of the two. If the ratio of height to radius exceeds a critical value of:

$$L/r = 0.95 \sqrt{r/t} \tag{7}$$

the tube wall will probably fail by a combination of column-type and local buckling [10]. If the ratio H/r is less than the critical value, local buckling will probably control. Overall buckling as a column can be checked using the AISC equation for intermediate-length columns (Eq. 1.5—1 in Ref. [11]), substituting f_{cr} for F_y and using an effective-length factor k of 2.1, as recommended by AISC for an cantilever column. So modified, the AISC column formula is:

$$F_a = \frac{\left[1 - \frac{(2.1\ L/r)^2}{2\ C_c^2}\right] f_{cr}}{\frac{5}{3} + \frac{3\ (2.1\ L/r)}{8\ C_c} - \frac{(2.1\ L/r)^3}{8\ C_c^3}} \tag{8}$$

where L is the tube height, r is the radius of gyration, and C_c can be determined by:

$$C_c = \sqrt{\frac{2\ \pi^2\ E}{F_y}} \tag{9}$$

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The wall thickness will probably be controlled by the vertical compressive stress. Even though uniform pressure all around the tube from either inside or outside material tends to increase the resistance to vertical buckling, the authors believe that this effect should not be depended upon in selecting the tube wall thickness. Furthermore, next to openings, the lateral pressure from one side only may worsen the situation.

Several equations have been developed to predict the resistance of the tube wall to vertical buckling. Among them is the Boardman equation [5], [10]. This equation, wherein a safety factor of 2 against buckling is assumed, is given by:

$$F_c = 2{,}000\,(t/r)\,(1 - 100\,t/3\,r) \qquad (10)$$

where t is the thickness of the wall plate, and r is the radius of the tube.

The Boardman equation (Eq. (10)) is for use only with units of the U.S. system, giving the allowable stress for steel in lb/inch² (psi). For ratios of radius to thickness r/t less than 500, the Boardman equation gives unrealistically low allowable stresses. For these ratios, it is probably better to use a method given by the European Convention for Constructional Steelwork (ECCS), which recommends to compute the critical (ultimate) vertical compressive stress by [10]:

$$f_{cr} = CEt/r \qquad (11)$$

where C is given as:

$$C = \frac{0.374}{\sqrt{1 + 0.01\,r/t}} \qquad \text{for } r/t \leq 212 \qquad (12)$$

and as:

$$C = \frac{0.315}{\sqrt{1 + 0.01\,r/t}} \qquad \text{for } r/t \geq 212 \qquad (13)$$

However, if substitution of C into Eq. (11) gives a critical stress larger than 3/8 of the yield strength F_y, failure will probably occur due to inelastic buckling, given by ECCS as:

$$f_{cr(i)} = F_y \left[1 - 0.347 \left(\frac{F_y}{f_{cr}}\right)^{0.6}\right] \qquad (14)$$

The ECCS equations given above include a factor of 1.33 to consider initial imperfection of the shell, but no additional factor for uncertainty of load. Therefore, the values of f_{cr} must be considered as ultimate stresses, and they must be divided by the normal factor of safety for use in design. ECCS suggests that an additional safety factor of 1.5 be applied to obtain the (allowable) working stress. Thus, the total safety factor with respect to a perfect tube would be 2.00.

The ECCS equations are based on numerous tests of cylinders without side openings. For tubes with openings, the critical stress will be less, and the actual stress to which it must be compared should be computed considering the properties of the net section of the tube. Edge stiffening around the openings should at least restore the area of shell removed by the opening and at least the entire gross moment of the inertia of the cylindrical shell.

The authors think that the lower value of 15,000 lb/inch² (recommended by the American Water Works Association for water tank construction) should be used if the allowable vertical stress, computed by the Boardman or the ECCS equations with a safety factor, exceeds 15,000 lb/inch².

The critical location for local buckling is probably at the edge of an opening, where the vertical compressive stress is fairly high. Combined with the vertical compressive stress at that location, there may be either inward or outward pressure from the stored or stacked material, which will cause vertical bending, necessitating stiffening of the tube wall at the edge of an opening to resist this combination of stresses. In the absence of any more reliable design criteria, the authors prefer to design the side stiffener and a narrow strip of wall plate to act as a column with bending due to lateral load. However, if this is done, there is still the question of what to use for the effective length of the column. For this, the authors suggest using the height of the opening.

The resistance to vertical compression should be checked at the bottom, at the bottom of the lowest opening, at each height where the wall thickness is reduced and at the bottom of the first opening above points where the wall thickness is reduced.

3.1.3 *Horizontal Compression*

For stacking tubes above a reclaim tunnel, it is possible for the external stack to be fully in place while the tube itself is empty. This will cause maximum horizontal compression in the walls. The horizontal portion of the wall between vertically adjacent pairs of openings must act as a horizontal arch to resist this pressure. Assuming a portion of the wall to act as a pin-ended arch, the critical buckling pressure for a unit width of such an arch can be calculated by the following equation for a perfectly circular-shaped arch [14]:

$$p_{cr} = \frac{Et^3\left(\frac{\pi^2}{\alpha^2} - 1\right)}{12\,r^3\,(1 - \nu^2)} \qquad (15)$$

where α is one-half of the total angle subtended by the arch, t is the wall thickness, ν is the Poisson ratio, usually assumed to be 0.3 for steel, and E is the modulus of elasticity.

In Eq. (15), consistent units must be used. If t and E are determined in inches, the unit of the critical pressure P will be lb/inch² (psi). Since Eq. (15) is for a geometrically perfect arch and for perfectly uniform material pressure around the circumference, the designer should be conservative in selecting a safety factor. On the other hand, it is conservative to assume a pin-ended arch, since the wall stiffness is certainly providing some end moment. The authors suggest a safety factor of not less than 2.0.

The final thickness of the wall plate used should be about 1/8 inch (3.2 mm) more than needed for strength alone; this is to allow for possible reduction by corrosion of either the inside or the outside surfaces. The thickness may also be increased to allow for reduction by abrasion of the stored material.

3.1.3.1 Bottom Ring-Beam

At the bottom of a steel stacking tube, a steel ring-beam is usually provided, serving several purposes: it stiffens the bottom edge to prevent damage from irregular horizontal load which might dent the tube; it spreads the vertical load to avoid overstressing the foundation concrete; and it provides for connection of the tube to the foundation by means of a circle of anchor bolts. The design of the circle of anchor bolts and the ring-beam proper may be found in Ref. [5] and is also demonstrated in Example 2.

3.1.3.2 Vertical Stiffeners

To product the lower portion of the tube from possible damage by the unloading machines, a series of vertical stiffeners is advisable. The stiffeners may consist of steel angles or beam sections welded to the tube wall plates.

3.2 Example 2

Fig. 7 shows the dimensions of an 8-ft outside diameter circular steel stacking tube for coal, located in earthquake zone 0 [12]. The tube walls, made of A-36 structural steel, were designed using the following data:

1. Coal properties:

– unit weight	55 lb/ft³ (pcf)
– angle of repose	30°
– coefficient of friction against steel	0.3

2. Loads:

– wind pressures:	
– 0— 30 ft above ground	22 lb/ft² (psf)
– 30— 50 ft above ground	27 lb/ft²
– 50—100 ft above ground	35 lb/ft²
– conveyor and headhouse loads:	
– gravity load at top of tube	18 klb (kip)
– longitudinal friction force (from expansion and contraction of conveyor structure)	2 klb
– headouse weight	4 klb

3. Construction material properties:

– yield strength of structural steel and anchor bolts, A-36 steel	36 klb/inch²
– 28-day compressive strength of foundation concrete	3,000 lb/inch²

3.2.1 Load Computation

To calculate the dead loads, reasonable steel thicknesses have to be assumed, as shown by Fig. 7. These thicknesses have an added 1/8 inch to allow for 1/16 inch corrosion on each face. Using these total thicknesses, the total weight of the structure above the tube base is:

Tube	89.74 klb
Conveyor and supports	20.0 klb
Headhouse	4.0 klb
Total	113.7 klb, say 114 klb

Weight of the coal inside the tube 244 klb

In Table 6, the wind forces and the overturning moments at the base are summarized.

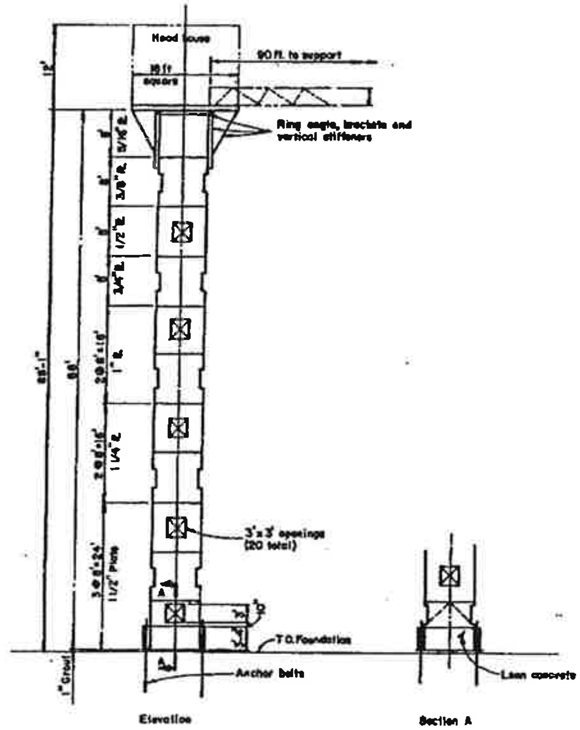

Fig. 7: Steel stacking tube dimensions (Example 2)

Table 6: Wind forces and overturning moments at the base

	Pressure lb/ft²	0.6 · Area ft²	Force k	Arm ft	Moment ft-k
On tube	22	30 · 8 · 0.6 = 144	3.2	15	48
	27	20 · 8 · 0.6 = 96	2.6	40	104
	35	38 · 8 · 0.6 = 182.4	6.4	69	442
On conveyor	35	4 (45-4) = 164	5.7	90	513
On headhouse	35	12 · 16 = 192	6.7	94	630
Total			24.6*		1,737**

* base shear
** base moment

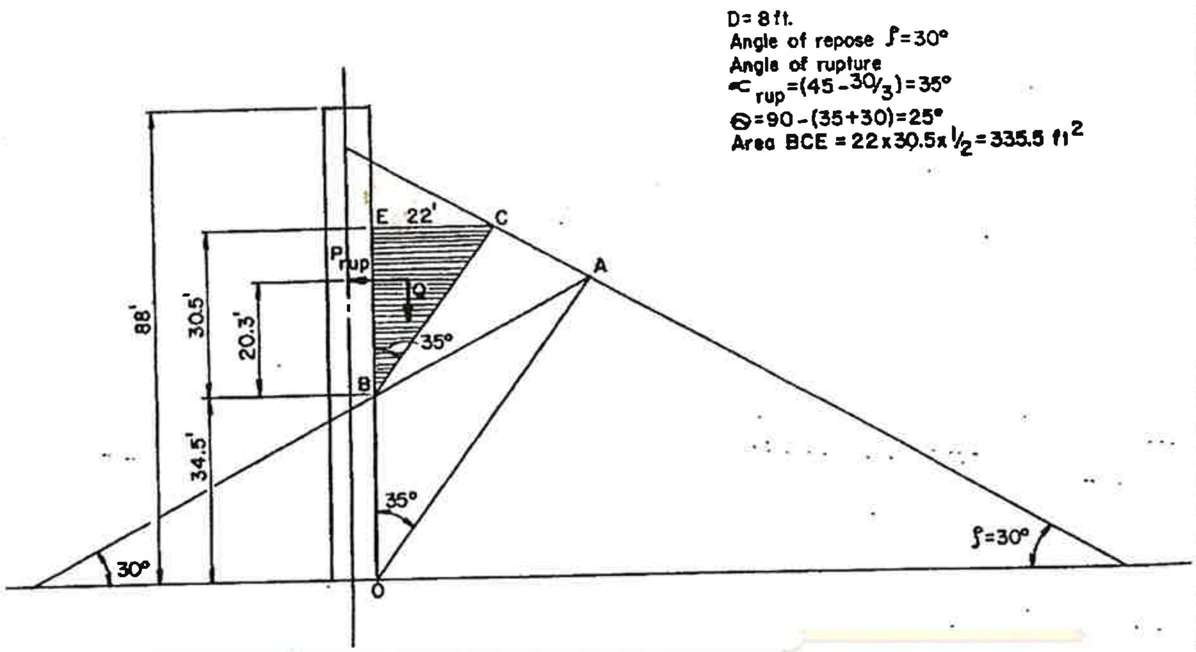

Fig. 8: Design lateral loads (Example 2)

Similarly, the base shear and the base moment at the bottom of the lowest openings are 24.2 k and 1,640 ft-k, respectively.

The wind on the exposed part of the tube with an eccentric external stack, as shown in Fig. 8, causes a bending moment of M = 921 ft-k at point B (34.5 ft above base) and M = 1,541 ft-k at the base.

3.2.2 Section Properties

For a tube thickness of t = 1.5 inch (from base to 24 ft above base), the effective thickness is $t_{eff} = t$—1/8 inch = 1.375 inch.

The gross cross-sectional area may be used in place of the net area, since the reinforcement to be placed around the openings will more than offset the area of the plate interrupted by the openings. Thus, the area (net or gross) is 414.7 inch².

The net moment of inertia about the axis through the center of the circle, but parallel to the faces of the opening, is:

$$I_{net} = [\pi\,(48^3)\,(1.375) - 2\,(48^2)\,(36 \cdot 1.375)]\ \text{inch}^4$$

$$= [477{,}723 - 228{,}096]\ \text{inch}^4 = 249{,}627\ \text{inch}^4$$

and the radius of gyration is 24.53 inch.

Similarly, for t = 1 inch, t_{eff} = 7/8 inch; area (net or gross) = 263.9 inch²; and I_{net} = 158,854 inch⁴.

The values of I_{net} can be increased, if desired, by adding for the effect of the stiffeners added around the openings.

3.2.3 Computation of the Pressure from Inside Stored Material

Using the Janssen method, the static lateral pressure at the tube bottom elevation is computed as 355 lb/ft² and the allowable passive pressure as 2 · 355 lb/ft² = 710 lb/ft².

3.2.4 Computation of the Force from Outside Stacked Material

The same method illustrated for the concrete tube gives, a depth of 88 ft (at the base), an active pressure of 546 lb/ft² a total force of 22,932 lb/ft width of the wall; and, thus, design pressure = (546 active — up to 710 passive) = zero. Therefore, the outside pressure is of no concern when the tube is full.

3.2.5 Friction Load on the Outside of the Wall above the Base

According to Reimbert [6], V is the active pressure per ft of wall width multiplied by the tangent of the angle of repose

$$V = (22.932 \cdot \tan 30°)\ \text{k/ft} = 13.24\ \text{k/ft}$$

The total friction load is equal to the sum of the outside and the inside force:

$$[13.24 + 244/(8 \cdot 3.1416)]\ \text{k/ft} = 22.95\ \text{k/ft}$$

Division by the net perimeter, so as to account for the openings, gives the total active pressure:

$$V_{total} = 25.99\ \text{k/ft}$$

3.2.6 Computation of the Vertical Compressive Stresses

The following stresses are computed for the loads at the bottom of the lowest openings (for this example, the loads are essentially the same as those at the base, but stresses are computed using the net properties of the cross section):

— Dead load:
P/A = (114/414.7) klb/inch² = 0.275 klb/inch²

— Wind on empty tube:
Mc/I = (12 · 1,640 · 4 · 12/249,627) klb/inch² = 3.78 klb/inch²

– Wind on tube exposed above point B (Fig. 8):
  – at point B:
  $Mc/I = (12 \cdot 921 \cdot 4 \cdot 12/158{,}854)$ klb/inch²
  $= 3.340$ klb/inch²
  – At the base:
  $Mc/I = (12 \cdot 1{,}541 \cdot 4 \cdot 12/249{,}627)$ klb/inch
  $= 3.558$ klb/inch²

– Internal friction (the entire weight of stored material assumed to be carried by friction against the tube wall): (243.3/414.7) klb/inch² = 0.587 klb/inch²

– External friction:
[13.24/12 · 1.375)] klb/inch² = 0.802 klb/inch²

3.2.7 Determination of the Wall Stresses Due to Eccentric External Stack

Force Q:

$$Q = [0.5\,(22 \cdot 30.5) \cdot 8 \cdot 0.055]\ \text{klb} = 147.6\ \text{klb}$$

Force P_{rup}:

$$P_{rup} = (147.6 \cdot \tan 25°)\ \text{klb} = 68.84\ \text{klb}$$

Overturning moment at point B due to eccentric pile loading (Fig. 8):

$M = (68.84 \cdot 20.3)$ ft-k = 1,398 ft-k
$Mc/I = (12 \cdot 1{,}398 \cdot 4 \cdot 12/158{,}854)$ klb/inch²
$= 5.069$ klb/inch²

Total force due to passive pressure on the tube below point B:

$$P_p = \frac{\gamma H_p^2\,(1 - 2\pi p)^2}{2\,(1 + 2\pi p)^2} \qquad (16)$$

$$P_p = \frac{0.055 \cdot 32.2^2 \left(1 - \frac{2 \cdot 30}{180}\right)^2}{2\left(1 + \frac{2 \cdot 30}{180}\right)^2}\ \text{klb} = 57.1\ \text{klb}$$

Overturning moment at the base:

$M = [68.84\,(20.3 + 34.5) - 57.1\,(32.2/3)]$ ft-k = 3,160 ft-k
$Mc/I = (12 \cdot 3{,}160 \cdot 4 \cdot 12/249{,}627)$ klb/inch² = 7.292 klb/inch²

3.2.8 Determination of Overturning Moment and Bending Stresses Due to Conveyor Force

Overturning moments:

– at point B:
$M = [2\,(88 - 34.5)]$ ft-k = 107 ft-k
$Mc/I = (12 \cdot 107 \cdot 4 \cdot 12/158{,}854)$ klb/inch²
$= 0.388$ klb/inch²

– at the base:
$M = (2 \cdot 88)$ ft-k = 176 ft-k
$Mc/I = (12 \cdot 176 \cdot 4 \cdot 12/249{,}627)$ klb/inch²
$= 0.406$ klb/inch²

Vertical loads above point B:

– headhouse	4 klb
– conveyor	20 klb
– tube weight (above point B only)	32.2 klb
– Total (ignoring friction)	56.3 klb

$M/A = (56.3/258.6)$ klb/inch² = 0.218 klb/inch²

The stresses due to vertical load and overturning moments are summarized in Table 7.

Table 7: Stresses (in klb/inch²) due to vertical load and overturning moment

Source	At Bottom of the Lowest Openings	At Elevation of Point B
Dead load	0.275	0.218
Internal friction	0.587	0.357
External friction	0.802	—
Conveyor force	0.406	0.388
Eccentric stack	7.292	5.069
Wind on entire tube	3.780	3.558
Wind on exposed tube	3.558	3.340
Totals:		
— Dead load, wind, conveyor force	4.461	4.164
— Dead load, full tube, eccentric stack, conveyor force	9.362	6.032
— Same, but with wind on exposed tube	12.920	9.372

3.2.9 Comparison of the Computed to the Allowable Vertical Stresses

Computed and allowable vertical stresses are compared for one location and loading only. Local buckling may be checked by various equations, but, since the value of the ratio r/t is far below 500, the Boardman equation (Eq. (10)) gives a ridiculously low answer and should not be used. For $r/t = 48.69/1.375 = 35.4$, the ECCS Eq. (11) is used to compute $C = 0.321$. Substituting this value of C and using 29,000 klb/inch² for the modulus of elasticity E, Eq. (11) gives a critical stress of 263 klb/inch², which is much larger than 3/8 of the yield strength of 36 klb/inch². Therefore, failure will occur due to inelastic buckling, and Eq. (14) is the correct one to use. Substituting the 263-klb/inch² value for f_{cr} and 36 klb/inch² for F_y, Eq. (14) gives $f_{cr(i)} = 32.2$ klb/inch². This is the ultimate value, so that the allowable stress can be determined by dividing by the desired safety factor. Using a safety factor of 2.0, the allowable vertical stress is (32.2/2) klb/inch² = 16.1 klb/inch².

For the case analyzed here, wind is a contributing cause of stress, so that the allowable vertical stress becomes 4/3 times 15.0 klb/inch², or 20.0 klb/inch². Since this value exceeds the computed actual stress of 12.92 klb/inch², the tube is satisfactory at the two elevations and for the loading considered. Other locations and other loadings must, of course, be considered in a complete design.

At the elevation of the bottom of the lowest openings, ratio $H/r = 91.8/4 = 23$, which exceeds $0.95\sqrt{4 \cdot 12/0.875} = 7$, necessitating checking of the overall buckling of the tube. This may be done using Eq. (8) [11] for intermediate-length columns, but with the critical stress for local buckling (computed by Eqs. (11) or (14)) substituted for the yield strength F_y, resulting in:

$$kL/r = 2.1 \cdot 12 \cdot 91.8/24.53 = 94.3$$

and an allowable stress of:

$$F_a = 12.24 \text{ klb/inch}^2$$

or, for a loading that includes wind:

$$F_a = (12.24 \cdot 4/3) \text{ klb/inch}^2 = 16.31 \text{ klb/inch}^2$$

Again, the allowable exceeds the computed actual stress, and, thus, the design is satisfactory.

The above types of stress check should be carried out for all relevant load combinations, at elevations where the steel plate thickness is reduced and at the bottom of openings above such elevations. Limited space precludes showing all the necessary calculations.

3.2.10 Check of Horizontal Bending Due to Wind

Horizontal bending due to wind will be most serious at the top, where the wind pressure is the highest and the tube thickness is the least. According to Ref. [2], the horizontal bending moment may be estimated by:

$$M = 0.317\, wr^2$$

Thus:

$$M = (0.317 \cdot 35 \cdot 4^2) \text{ ft-lb/ft} = 177.5 \text{ ft-lb/ft}$$

For a thickness of 5/16 inch and assuming corrosion at this elevation on the inside only, the effective thickness becomes 0.25 inch.

Thus, the stress is:

$$M/S = [177.5 \cdot 12/(12 \cdot 0.25^2/6)] \text{ lb/inch}^2 = 1{,}420 \text{ lb/inch}^2$$

which is less than 15,000 lb/inch² and thus satisfactory.

3.2.11 Check of the Arched Section of the Tube at an Opening at the Level of Point B

Effective thickness:

$$[1.25 - 2\,(1/16)] \text{ inch} = 1.125 \text{ inch}$$

The angle subtended by the arch is 158°; angle "a" is one-half of this, or 79°. By Eq. (15):

$$p_{cr} = \frac{29 \cdot 10^6\,(1.125^3)\,(\pi^2/1.379^2 - 1)}{12 \cdot 4 \cdot 12\,(1 - 0.3^2)} \text{ lb/inch}^2$$

$$= 330{,}000 \text{ lb/inch}^2 = 47{,}530 \text{ klb/ft}^2$$

Obviously, even with a generous safety factor, this value is more than adequate, since it exceeds the computed actual pressure of 546 lb/ft² by a wide margin.

3.2.12 Reinforcement around Openings

Area of tube shell interrupted:

$$3 \cdot 12t = 36t$$

Area to be added:

$$1.2 \cdot 36t$$

For reinforcement around openings, 2-inch plates should be used, as shown in Fig. 9 (see also Table 8).

3.2.13 Design of the Base Ring-Beam and Its Connection

Assuming that all load combinations have been checked at all critical locations, the design of the base ring-beam and its connection is the remaining step. At the base, the overturning moment is:

— due to eccentric stack (neglecting effects of passive pressure):

$$M = (68.84 \cdot 65) \text{ ft-k} = 4{,}475 \text{ ft-k}$$

— due to longitudinal load from the conveyor:

$$M = (2.0 \cdot 88) \text{ ft-k} = \underline{176 \text{ ft-k}}$$

— total at the base:

$$M = 4{,}651 \text{ ft-k}$$

The section modulus (line) for the ring is:

$$S = (\pi \cdot 8^2/4) \text{ ft}^2 = 50.3 \text{ ft}^2$$

Thus, the stress is:

$$\pm M/S = \pm(4{,}651/50.3) \text{ k/ft} = \pm 92.47 \text{ k/ft}$$

The ring base-plate width must be chosen to avoid overstressing the concrete foundation. By computations similar to those shown above for the tube, the maximum vertical load per ft of ring-beam is found to be 106.7 k/ft. Factored for concrete design by the strength method [13]:

$$P = (1.7 \cdot 106.7) \text{ k/ft} = 181.4 \text{ k/ft}$$

Using a strength reduction factor of 0.7, the minimum required base plate width is:

$$[181.4/(0.7 \cdot 0.85 \cdot 3 \cdot 12)] \text{ inch} = 8.5 \text{ inch}$$

Thus, the 16-inch plate width shown in the detail of Fig. 9 will be satisfactory.

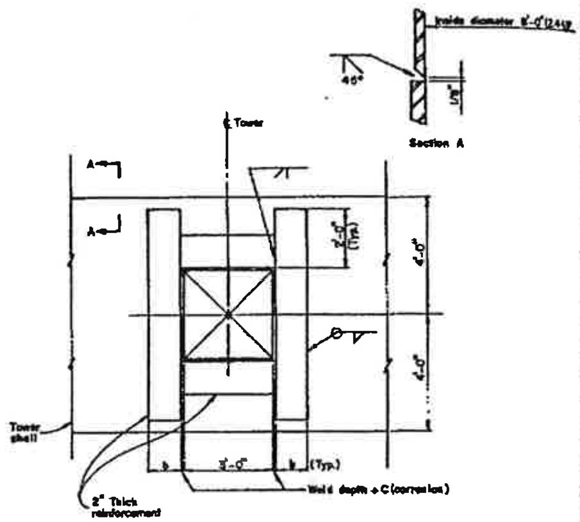

Fig. 9: Typical opening reinforcement (Example 2)

Table 8: Shell thicknesses, plate widths and fillet weld sizes

Shell Thickness inch	Plate Width inch	Fillet Weld Size inch
1—1/2	16	3/4
1—1/4	13 1/2	3/4
1	11	1/2
3/4	8	3/8
1/2	5 1/2	3/8
3/8	4	3/8

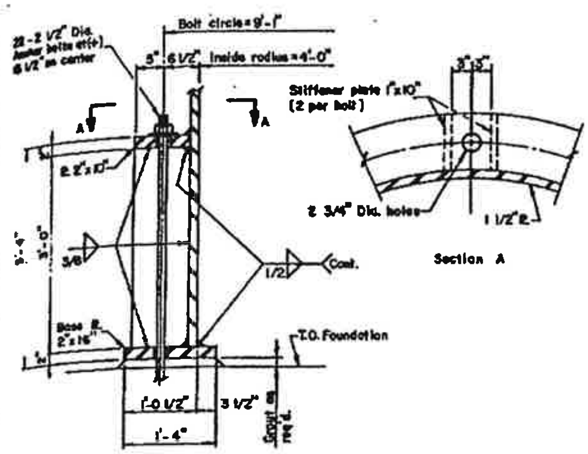

Fig. 10: Tube base ring-beam detail (Example 2)

3.2.14 Selection of Anchor Bolts

Using the method shown in Ref. [5], the tension on one anchor bolt is given by:

$$T = 2M/Nr - W/N \qquad (17)$$

where M is the overturning moment; W is the total vertical load; N is the number of anchor bolts in the ring; and r is the radius of the bolt circle, in this case 4.542 ft.

The solution of Eq. (17) gives a tension of 87.82 klb per bolt. The allowable tensile load for 2-1/2-inch diameter threaded A-36 steel [11] is the product of the bolt gross area times 0.33 times the ultimate strength of the material:

$$(4.91 \text{ inch}^2 \cdot 0.33 \cdot 58 \text{ klb/inch}^2)/\text{anchor bolt}$$
$$= 93.98 \text{ klb/anchor bolt}$$

The thickness of the top flange of the ring-beam must be sufficient to resist bending due to the 87.82 klb load from the bolt. Treating the flange plate as a simple beam between stiffeners, the plate bending moment is (conservatively):

$$M = (87.82 \cdot 7/4) \text{ in-klb} = 153.7 \text{ in-klb}$$

$S_{required}$ is:

$$(153.7/27) \text{ inch}^3 = 5.69 \text{ inch}^3$$

Use of a top flange plate of 2 inch x 10 inch gives:

$$S = 6.67 \text{ inch}^3$$

Each of the 1 inch x 10 inch stiffeners for the ring-beam carries a vertical load equal to one-half the bolt tension, or 43.9 klb, and satisfies all requirements.

4. Foundation Design

The design of the foundation for a stacking tube is not much different from the design of the foundation for any other structure subject to both vertical and lateral loads. The designer should select the worst loading conditions and be certain that stability against overturning is provided. A safety factor against overturning of at least 1.75 is suggested, which is about 17% higher than the safety factors normally used for other structures. The higher factor is advisable since precise determination of loads causing overturning is not possible. (Suggestions for computing overturning loads are given in the previous paper by the authors [1].)

When designing the foundation to resist vertical load, the designer should remember to include the large additional soil load caused by the weight of stacked material supported by the soil around the stacking tube. Also, the foundation will extend beyond the perimeter of the tube, and a large depth of stacked material will bear directly on this extension of the foundation or on the soil above it.

If the stacking tube is supported by the structure of a reclaim tunnel, the tunnel will have to be reinforced to withstand the forces applied by the tube. These will include forces due to wind, friction forces, dead load, eccentric material pressures and possible seismic loads.

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