DESIGN OF REINFORCED CONCRETE CONTAINMENTS

A. K. BANERJEE, K. A. CONDON, R. A. RETTIG, C. F. REEVES

Engineering Mechanics Division, Stone & Webster Engineering Corporation,
245 Summer Street, P.O. Box 2325, Boston, Massachusetts 02107, U.S.A.

SUMMARY

The practical design and construction features of reinforced concrete containments are presented. Full scale test results are compared with analysis results. The impact of ACI-359 code on containment design is discussed.

The membrane design of reinforced concrete cylindrical containments is well established. The internal pressure cracks the concrete shell biaxially and the entire membrane stress is assumed to be carried by the reinforcing steel acting as a network. There is no significant bond demand in reinforcement except at discontinuities and near the top of the dome where compatibility of the hoop and meridional reinforcement induce bond stresses. Radial reinforcement is required to prevent splitting in the dome and at discontinuities to carry shear.

Earthquake design of containments is complex. The dynamic analysis is performed once assuming the containment to be uncracked and again assuming the containment to be fully cracked. The shell stress resultants are conservatively assumed to be sine and cosine functions. The SRSS of the effects of the three independent earthquake components lead to a design in which the reinforcement has to withstand the simultaneous application of the maximum values of meridional, hoop and shear forces. The design will be more realistic if the three earthquake components are algebraically combined in the time domain. Studies show that the maximum horizontal response due to three earthquake components applied simultaneously is no more than 10 to 20 percent greater than the horizontal response due to one of those components. In this tress analysis the maximum membrane tension and shear are separated 90° along the circumference.

A revision to the ACI-359 code permits limited yielding of the diagonal reinforcement in one direction during an earthquake. The analysis indicates that this revision is reasonable and that after the initial yielding of the diagonal bars, no successive yielding occurs.

The most complicated portion of containment design is near the base and near hatches and penetrations. Proper stiffness representation of the shell, especially in the areas of discontinuities, is essential. Several design details of these areas are presented and construction advantages discussed.

The Structural Acceptance Tests of containments give useful data regarding the behavior of containments under pressure. Results of these tests compare well with analytical predictions which include the effect of concrete cracking, tensile strength of concrete and the steel liner. The crack pattern around the hatches is used in performing the detailed finite element analysis of the concrete and reinforcement. Full scale tests were made on a 6 inch thick circumferential segment of the containment base and the results indicate that the design is conservative.

Practical solutions for a complex shell design problem are presented. Tests and analytical studies indicate that the reinforced concrete containment design is conservative.

1.0 Introduction

The containment structure for a Pressurized Water Reactor (PWR) power plant consists of a steel lined concrete shell. The functions of the containment structure are to provide a leak tight barrier against an uncontrolled release of radioactive material to the environment, to assure that the containment design conditions are not exceeded for as long as postulated accident conditions require, and to protect the nuclear steam supply system from all design basis environmental conditions.

The containment boundary is a steel lined, conventionally reinforced concrete structure with cylindrical shell, hemispherical dome and a flat circular base mat. With reference to Fig. 1, the cylindrical portion of the shell is 130 to 150 ft. in diameter with a 4 to 5 ft. thick wall and 140 to 170 ft. in height from top of mat to the bend line. The mat is 10 to 12 ft. thick and 150 to 300 ft. in diameter depending upon whether it supports only the containment or the containment and other safety related structures. The hemispherical dome is usually 2 1/2 ft thick.

The cylindrical portion of the steel liner is 3/8 in. thick, and the dome liner is 1/2 in. thick to facilitate concrete pouring during construction. The liner covering the mat is 1/4 in. thick with the exception of areas where transfer of large loads may require additional thickness. The liner on the mat is protected from short duration thermal loads and internal missiles by a 2 ft. layer of concrete poured on and anchored to the structural mat. Typical ultimate strength of concrete and yield stress of reinforcement and liner used in the containment shell are:

Concrete	= 3000 to 4000 psi at 28 days
Reinforcing steel	= 40,000 psi (#11 bar and smaller)
	= 60,000 psi (#14 and #18 bars)
Liner	= 32,000 psi
Anchor studs	= 50,000 psi
Flat plate bars	= 50,000 psi

The actual materials used in containment shell depend on the structural and project requirements.

2.0 Containment Design for Axisymmetric Loads

PWR containment structures are presently designed for load factors and combinations listed in ASME Section III, Div. 2 code. The primary design axisymmetric load is the internal pressure due to the Design Basis Accident (DBA). Typical DBA pressures for atmospheric and subatmospheric structures are 45 to 50 psig.

The design of the shell structure for membrane forces due to axisymmetric loads is well established. The internal pressure cracks the concrete shell biaxially and the membrane forces are carried by continuous reinforcing steel (1). Therefore, most of the structure is represented as an orthotropic shell with properties equivalent to those of reinforcing steel only.

While the membrane forces are insensitive to the shell stiffness representation, bending moments and radial shear forces vary significantly with changes in shell stiffness caused by concrete cracking. Near the base, because of low hoop forces and large bending moments, the containment wall is not fully cracked. Partially cracked shell stiffness properties are used in this zone.

One advantage of having a hemispherical dome with thickness approximately equal to half the thickness of the cylindrical shell is to minimize discontinuity stresses at the dome cylinder junction. Thus, the reinforcement detail in this area is simple and construction is facilitated.

There is no significant bond demand on reinforcement in the entire membrane region of the shell except at discontinuities and near the top of the dome. The bond demand in the dome occurs because of the variation in spacing of meridional reinforcing bars which converge towards the apex of the dome.

The bond force (u) per unit length of reinforcing bar in the dome is:

$$u = \frac{d\,T_b}{dl} = \frac{1}{R}\,\frac{d\,T_b}{d\emptyset} \qquad (1)$$

$$\text{where}\quad T_b = (S)(T) = \frac{2\,R\,\sin\emptyset\,T}{N} \qquad (2)$$

R = radius of the dome

S = spacing of meridional rebars which varies with the angle $\emptyset$

$\emptyset$ = angle measured from apex along the meridian

N = total number of meridional rebars

T = meridional tension per unit length of shell normal to the bar

Neglecting dead load of the dome, for DBA internal pressure the force $T = pR/2$.

$$\text{Then } u = \frac{pR\cos\emptyset}{N} \qquad (3)$$

As can be seen from expression (3), the bond demand is the greatest near the apex of the dome. Radial shear reinforcement is provided in this region to prevent splitting of concrete due to high bond stresses since splitting would further reduce the bond capacity of the reinforcement in a region which is already subjected to biaxial tension.

3.0 Earthquake Design

Containment design for earthquake loads is complex because crack patterns for the DBA plus earthquake loading are not well defined. The dynamic analysis ~~is~~ performed first assuming the containment to be uncracked and again assuming the containment to be fully cracked. The shell is designed for the envelope of results obtained from the two assumptions (2). The shell properties are assumed to be axisymmetric which results in a conservative, sine and cosine, distribution of the shell stress resultants.

The design is based on the simultaneous application of the three orthogonal earthquake components which are algebraically combined in the time domain. Studies show that the maximum horizontal response due to three earthquake components applied simultaneously in the time domain is no more than 10 to 20 percent greater than the horizontal response due to one of the horizontal earthquake components. The maximum meridional tension and the membrane shear here are separated 90° along the circumference.

This approach is clearly superior to the method where the shell is analyzed for each of the three earthquake components separately and the stress resultants are summed as the square root sum of the squares (SRSS). In the SRSS method the maximum values of meridional tension and the membrane shear would have to be taken at the same place, resulting in a very conservative design.

The ASME Section III, Div. 2 Code permits tangential shear stress of 60 psi to be transmitted across a crack by means of concrete aggregate interlock. When this value is exceeded, a system of two-way inclined reinforcement is required to carry the excess. The inclined reinforcement will have strains compatible with the vertical and hoop reinforcement. Therefore, under internal pressure, the diagonal rebars will be near tensile yield with only a modest reserve to resist tangential earthquake shear forces without yielding in tension.

A recent revision of the code permits limited yielding of the diagonal reinforcement in one direction during an earthquake. After the initial yielding, no successive yielding occurs; i.e., there is no system of progressive collapse or increasing displacement with each earthquake cycle. The code requires a minimum amount of diagonal rebars of area equal to $(S-S_c)/A_D$. Where S is total earthquake shear per unit length, S_c is the portion of shear carried by concrete per unit length and A_D is the area of diagonal reinforcement per unit length normal to the diagonal. This guarantees that sufficient inclined reinforcement is provided to maintain overall equilibrium.

This can be demonstrated by neglecting the participation of concrete and designing the two-way inclined reinforcement to resist all of the membrane shear force at (+) and (-) 0.9 fy. The reinforcement is assumed to yield at the stress of 0.9 fy and the maximum permissible strain is twice the yield strain.

Let f_1 and f_2 = stresses in the two orthogonal diagonal rebars of equal area.

f_o = initial bar stress due to membrane forces other than shear.

S = Earthquake shear/unit length

A_D = Area of each diagonal rebar/unit length normal to the diagonal.

The rebars are sized such that neglecting all other loads

$$f_1 = -f_2 = 0.9fy = \frac{S}{A_D}$$

$e_1 = -e_2 = 0.9ey$ = strains in diagonal reinforcement

First Shear Application

$f_1 = f_o + f_s = 0.9fy$ (yielding of tensile inclined rebar)

where $f_s = 0.9fy - f_o$

S_1 = shear at which the first rebar yields = $A_D f_s < S$

The second rebar stress

$$\begin{aligned} f_2 &= f_o - (f_s + \frac{2(S-S_1)}{A_D}) \\ &= f_o - \frac{2S}{A_D} + f_s \\ &= 0.9fy - \frac{2S}{A_D} = -0.9fy \end{aligned} \qquad (4)$$

$e_2 = -0.9ey$

The second rebar does not yield.

Unloading to Zero Shear

No yielding occurs during unloading.

Thus the change in stress in both the diagonals = $\pm \frac{S}{A_D}$

$$\begin{aligned} f_1 &= 0.9fy - \frac{S}{A_D} \\ f_2 &= 0.9fy - \frac{S}{A_D} \end{aligned} \qquad (5)$$

The two rebar stresses are equal and opposite as required by equilibrium and they are different from the initial stress f_o before application of shear. This is the necessary consequence of the permanent plastic elongation of bar 1.

Shear Reversal

This cycle is also elastic

$$f_1 = 0.9fy - \frac{S}{A_D} - \frac{S}{A_D} = 0.9fy - \frac{2S}{A_D}$$

$$f_2 = 0.9fy - \frac{S}{A_D} + \frac{S}{A_D} = 0.9fy \quad (6)$$

Hence forth, each bar experiences elastic stress cycles from a maximum tension of +0.9fy to a maximum compression of $-(\frac{2S}{A_D} - 0.9fy)$.

The above stress analysis has an effect on hoop and meridional rebars which can easily be accounted for. The primary purpose of the example given above is to show that no successive yielding occurs through several earthquake loading cycles as only one diagonal rebar is allowed to yield.

A similar analysis is performed using the effect of uncracked concrete along the compression diagonal. One important design consideration is to place at least one layer of hoop reinforcement outside the diagonal bars to prevent buckling of the compression diagonal outward from the concrete shell. A recent revision to ASME, Section III, Div. 2 code requires that the inclined reinforcement be tied in order to prevent the compression diagonal from buckling.

4.0 Comments on ASME Section III, Div. 2 Design Philosophy

Even though factored load combinations are used in both the ACI-318-71 and the ASME code designs, unlike the ACI code, the ASME code does not require that the ultimate strength of shell wall be determined. The ACI code requires that the beam cross-section is under-reinforced such that the tensile reinforcement yields before the ultimate compressive strain of concrete (0.003) is reached. The ASME code design may be called a modified strength design method where design for flexure is elasto-plastic. The tensile reinforcement is not permitted to yield except for thermal loads and the non-linear stress strain curve is used for concrete. The concrete strain is not permitted to reach ultimate value, the maximum permissible strain being 0.002. The user of the ASME code must have clear understanding of this difference in design philosophy between the two codes.

In several areas the ASME code is very conservative because of lack of available test results. Punching shear strength of reinforced concrete under biaxial tension, allowable bond stress and anchorage requirement of rebars in biaxial tension zones, effect of simultaneous application of three orthogonal components of earthquake on cracking patterns of containment shell are cases in point. Further research is recommended in these areas.

5.0 Design and Construction Features

The containment mat thickness of 10 to 12 ft is chosen for both a well balanced design and ease of construction. The twelve foot thickness approaches the maximum suitable for pouring the entire mat thickness in one lift. Mats much thicker than 12 ft. must be poured in two lifts requiring horizontal construction joints and shear keys. A typical pour size is generally 1600 to 2000 cu.yd. The 10 ft. mat thickness allows the vertical dowels from the containment shell and the containment internal structure to be installed straight without hooks at the ends. The congestion of mat reinforcing is an important design consideration. The reinforcement at the top of the mat is placed in a radial grid for ease in the installation of the dowels. The reinforcement at the bottom of the mat is in either a rectangular pattern or a radial/circumferential pattern. The mat pours are preferably pie shaped wedges where the reinforcement is radial/circumferential. The bulkheads for concrete pours are made from expanded metal meshes to facilitate construction. Typical reinforcement in the containment mat and shell wall are shown in Figs. 2, 3 and 4

The loads from the containment internal structure are carried through the mat liner into the mat concrete by means of either bridging bars or through cadweld sleeves. As shown in Fig. 5, in the bridging bar detail, the 1/4" thick plate is welded to a 6" x 4" bar which runs parallel to the faces of the concrete walls of the internal structures. Wall reinforcing bars are cadwelded to the top of the bridging bar and the mat dowels are welded to the bottom of the bridging bars, thus transfering the loads directly to the mat without affecting the liner. In the through cadweld concept, a long solid plug double cadweld sleeve is passed through the liner which is welded to the sleeve. Sections of liner plate with several sleeves are made up in the shop.

The base of the containment requires a large amount of radial shear reinforcement. Instead of placing individual shear bars, ladders are made by welding plates which act as shear reinforcement between the inside and outside face vertical rebars. These welded assemblies simplify installation. All penetrations are kept at least 20 ft. above the mat to eliminate interferences with the base reinforcement.

The meridional reinforcement in the dome is welded to a ring at the apex. The ring provides anchorage and also acts as a vent opening during construction.

The Nuclear Steam Supply equipment is brought in though a large construction opening in the cylindrical portion of the shell and installed by using the containment polar crane which is supported on the containment internal structure. This procedure allows construction of the shell to be off the critical path.

The studs on the containment liner are tied back to the containment shell reinforcing at 2 ft spacing each way. This permits the liner to be used as a form; the concrete is poured in 6 ft. high lifts.

6.0 Tests

Tests have been undertaken to verify certain aspects of the design. Full scale tests of shear truss assemblies for the shell base were made to obtain information regarding splitting of concrete due to close spacing of rebars, load transfer mechanism, cracking behavior and overall performance (3).

A section of wall 4'-6" wide, 5'-0" high and 6 inches thick including a full size shear truss assembly was tested to the loads consistent with the factored DBA pressure condition. The tests demonstrated the capability of the assembly to carry the imposed loads without excessive deformation and cracking. The angle of cracks was 15° to 20° from the horizontal, the maximum crack width was 0.02". Studs up to 4'-0" long were welded to the liner to reduce splitting of concrete between the bars on the tension face. These tests led to the use of long studs near the base of the containment shell.

The shear tests also indicate low stresses in the shear plates. This suggests that concrete, even when fully cracked, can transfer significant shear (4). In containment base design, however, the radial shear strength of cracked concrete is typically neglected.

Tests were also conducted to determine the failure mode of a 3/8" thick liner plate attached to concrete with 5/8" and 3/4" studs. The surface between the liner and the concrete was subjected to movement. The 5/8" stud failed in shear without excessive strain in the liner, whereas, the liner with the 3/4" stud failed by tearing. As a result of these tests, in the containment, the liner is attached to the concrete with 5/8" studs.

7.0 Structural Acceptance Test

All containment structures are tested at 115 percent of the design pressure before an operating license is granted. The containment is pressurized in four or more equal increments and depressurized in the same increments. At each pressure level, the pressure is held constant for at least one hour before measurements are taken and the cracks are plotted.

After full pressure has been applied, the vertical crack spacings average 18 in. and the crack widths average 0.01 in. The horizontal cracks usually occur at construction joints which are 4 to 6 ft. apart. Above the dome spring line the crack spacing is nearly equal in orthogonal directions because the section is thinner and forces are more nearly equal in both directions. The cracking patterns near large openings such as hatches are similar to the typical shell cracks except that in the hatch ring beams the cracks are radial with respect to the hatch center line. Discontinuity moment near the base causes compression on the outside face of the shell. No cracks are visible in this area.

The measured radial deflections at mid-height and at the springline reach 80 percent of the analytically predicted values. The vertical displacements are about 60 percent of the predicted values. The theoretical values are calculated assuming no tensile strength for concrete. In reality, concrete does affect shell stiffness between cracks. When the concrete stiffness between cracks is accounted for, the analytical and test results match more closely than those reported above.

8.0 Conclusion

Practical design and construction aspects of containment shell design have been presented. Included were: a treatment of bond demand in dome reinforcement, designing of earthquake reinforcement in the containment wall, comments on the ACI-318 & ASME Section II Div. 2 codes, the design of containment mat and wall base details, and a discussion of cracking patterns. Tests and analytical studies give confidence in the performance of reinforced concrete containment shells.

References

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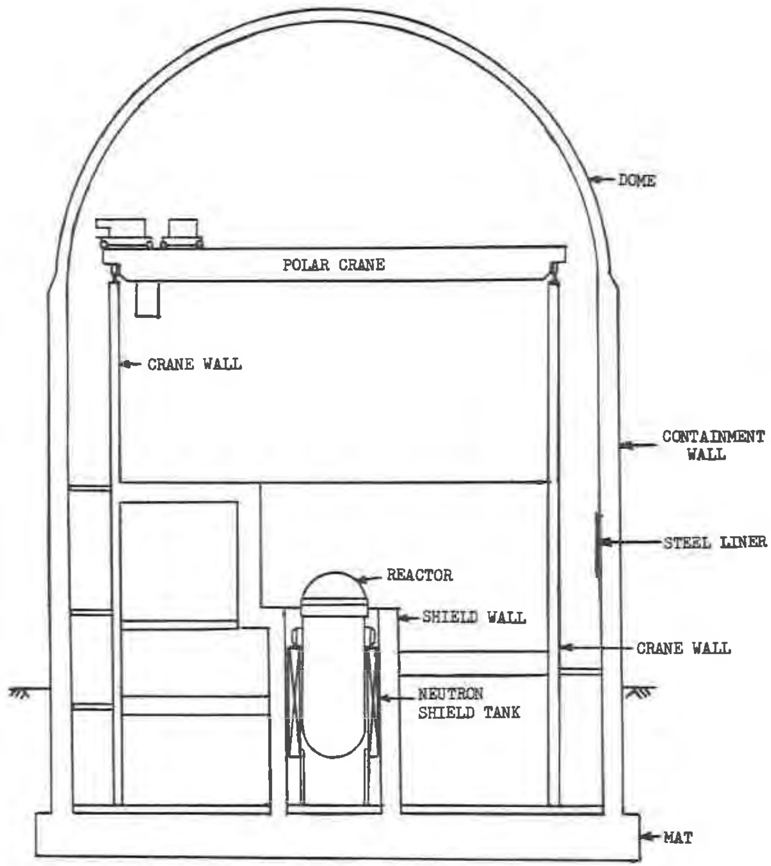

FIGURE 1 - Typical PWR Containment

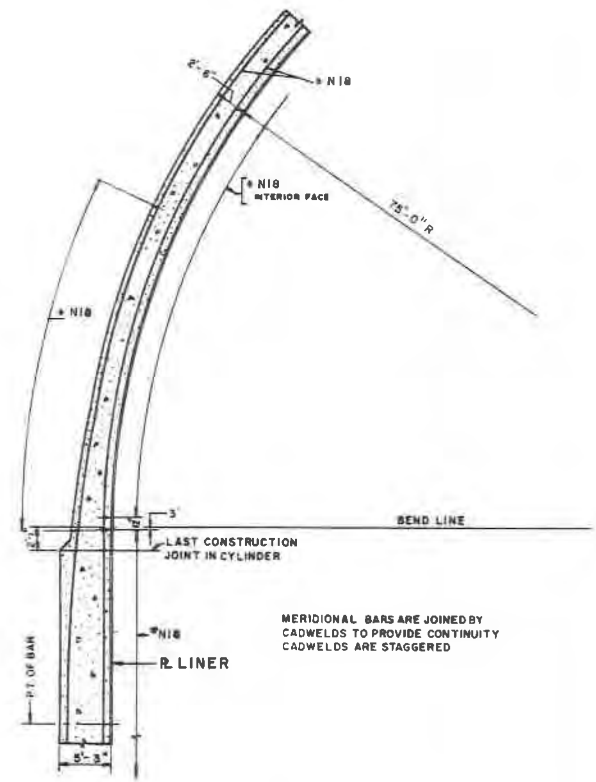

FIGURE 2 - Intersection of Cylindrical Shell and Dome

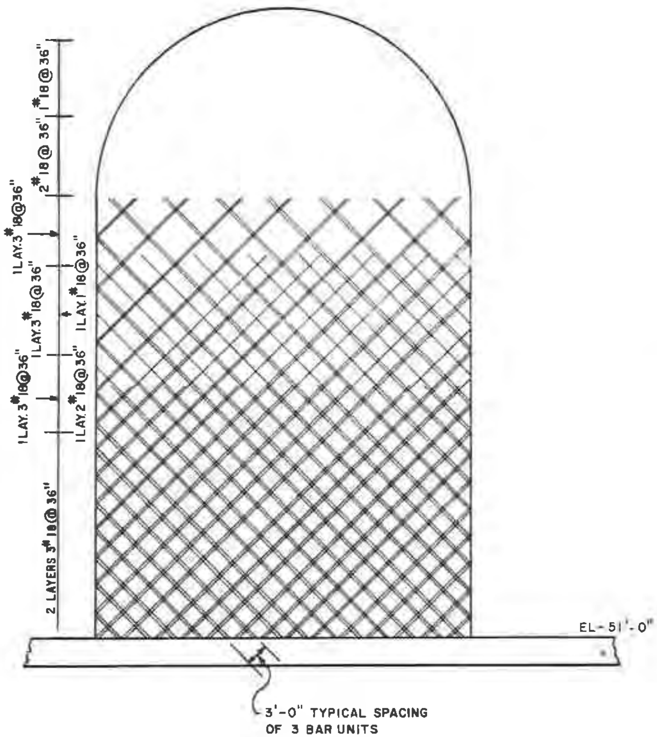

FIGURE 4 - Earthquake Reinforcement Pattern

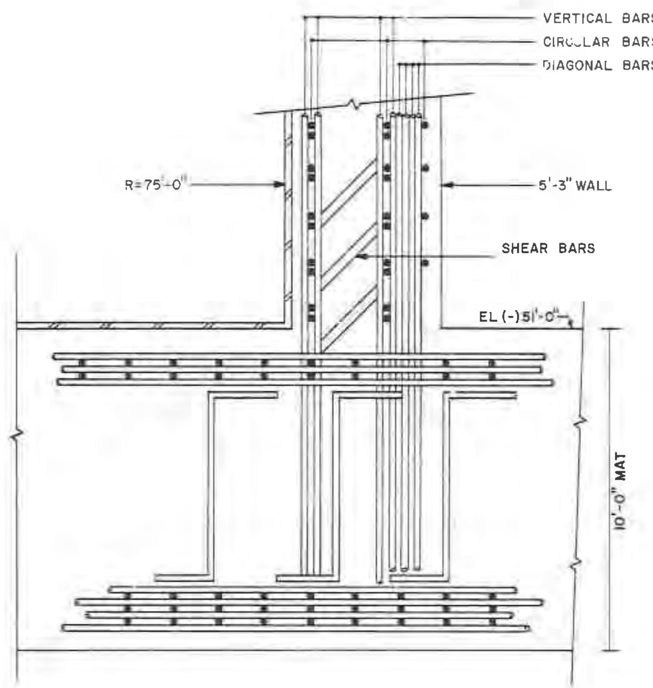

FIGURE 3 - Typical Detail of Containment Wall to Base Mat Junction

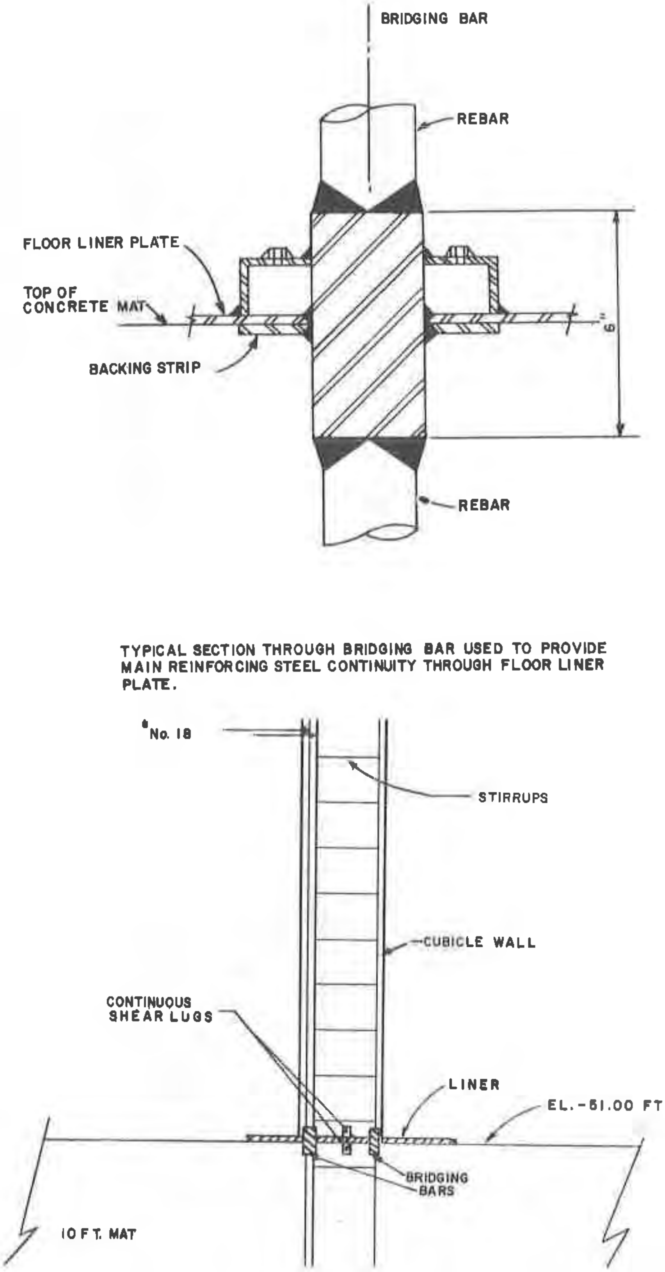

FIGURE 5 - Typical Bridging Bar Detail