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December 8, 1982

Mr. Darrell G. Eisenhut, Director
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Subject: Zion Station Units 1 and 2
I.E. Bulletin 80-11, Masonry Walls
NRC Docket Nos. 50-295 and 50-304

Reference (a): October 22, 1982, letter from
S. A. Varga to L. O. DelGeorge.

Dear Mr. Eisenhut:

In response to the NRC's request of reference (a), this is to provide additional information on masonry walls at Zion Station. The Attachment to this letter provides the requested information.

Please address questions regarding this matter to this office.

Very truly yours,

F. G. Lentine

F. G. Lentine
Nuclear Licensing Administrator

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Attachment

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COMMONWEALTH EDISON COMPANY

ZION STATION UNITS 1 and 2

Response to Request for Addition Information on

Masonry Walls (I.E. Bulletin 80-11)

5547N

Question: (1) Provide a general description of modification methods. Also confirm that all modified walls have been qualified by the working stress design method.

Response: All safety related masonry walls at Zion Station have been qualified by the working stress design method. When the allowable stresses for the "As Built" wall configurations were exceeded the modification methods described in Table 1 were used to bring the design stresses to within the working stress allowables.

TABLE 1

Type of Span	Type of Loading	Modifications to Reduce this Stress Type	Type of Modification
Vertical	Out of Plane	Tension Normal to Bed Joint	1) When the overstress was local in nature due to attachment loads, steel framing was added to carry these loads. 2) When the overstress was general in nature, vertical wide flange members which span floor to ceiling were added to provide lateral supports. The members were attached to the wall by thru bolting.
Horizontal	Out of Plane	Tension Parallel to Bed Joint	Vertical Posts consisting of wide flange members spanning floor to ceiling were added to reduce the horizontal span of the wall. The members were attached to the wall by thru bolting.
Vertical Cantilever	In Plane	Tension Normal to Bed Joint	Angle sections thru bolted to the wall and attached to the slab or beam above were added to transfer load and therefore reduce overturning stresses.

Question: (2a) Provide the types of horizontal reinforcement used in the horizontally spanned walls. Also provide verification to assure proper anchorage of the reinforcement at the boundary and proper bonding between the reinforcement and mortar. Provide the basis, including applicable test data which justifies the use of Dur-O-Wall type reinforcement as structural element (for example, NCMA has conducted some experiments to evaluate the structural role of joint reinforcement in Concrete Masonry, NCMA-TEK No. 99).

Response: Horizontal reinforcement which has been used for masonry wall construction at Zion station consists of continuous prefabricated 3/16" diameter truss or ladder type joint reinforcement conforming to ASTM A82 for cold drawn steel wire. The out-to-out spacing of longitudinal rods is 2" less than the nominal thickness of the wall. The prefabricated joint reinforcement has been fully embedded in mortar for its entire length with minimum mortar cover of 5/8" from the exterior wall face. The joint thickness of 3/8" has been used for the 3/16" diameter joint reinforcement.

These placement requirements for the horizontal joint reinforcement have been used for masonry wall construction at Zion Station and conform to the construction requirements of NCMA-1979, which is generally in agreement with Uniform Building Code 1979 and ACI 531-79.

The supporting test data reported for DUR-O-Wall products indicate that the joint reinforcement, when used as indicated above, provide adequate bond between mortar and the reinforcement and adequate protection against slippage of the reinforcement.

Masonry walls at Zion station have been designed based on simply supported horizontal spans and have been properly anchored to the supports to transfer shear due to lateral loads. Anchorage of horizontal joint reinforcement into the support is not necessary. A minimum lap length of 6" has been used for the joint reinforcement. Also at the corners, the prefabricated corner reinforcement has been used. This assures the continuity of the truss ars.

The following is a list of references in which test data regarding the effectiveness of joint reinforcement towards the structural strength of the masonry wall are reported.

1. "Investigation of Continuous Metal Ties as a Replacement for Brick Ties in Masonry Walls," DUR-O-WAL Technical Bulletin No. 67-5.

2. "Load Tests of Patterned Concrete Masonry Walls," by R. O. Hedstrom, Proceedings, American Concrete Institute, Vol. 57, p. 1265, 1961.

3. "Transverse strength of Concrete Block Wall," by F. W. Cox and J. L. Ennenga, Proceedings, ACI, Vol. 54, p. 951, 1958.

The following observations can be made from these references.

1. Horizontal joint reinforcement helps to control the cracks and to keep the wall together after it has cracked, and thus assuring a certain minimum strength of the wall.

2. The joint reinforcement is effective in increasing the ultimate strength of the wall.

Codes and Specifications such as ACI 531-79, NCMA-1979 and UBC-1979 permit an allowable tensile stress for joint reinforcement equal to 0.5fy but not greater than 30,000 psi, thus indicating joint reinforcement is effective in increasing the flexural strength of the wall. The stresses in truss bars at Zion do not exceed the allowable value.

Publications such as "ACI 531-Commentary," "Reinforced Masonry Design Book" by Robert R. Schneider and Walter L. Dickey, and "Reinforced Masonry Engineering Handbook by J. E. Amrhein of Masonry Institute of America, recommend joint reinforcement be used to resist lateral loads and increase the flexural tensile strength of the masonry wall.

TEST DATA FOR DUR-O-WALL PRODUCTS

BOND

Adequate bond between mortar and reinforcement is necessary to develop full value of the reinforcement. Test results in Table 9 show the increased bond strength of Dur-O-waL's patented deformation over smooth wire. The test results also provide the basis for Dur-O-waL's splice recommendation (6" lap) and the recommendation that Extra Heavy (3/16" side rods) be used only with ASTM type "S" or stronger mortar.

TABLE 9: EFFECT OF WIRE DEFORMATION ON BOND WITH MORTAR*

Mortar		Wire Size	Length of Embedment	Average Ultimate Load (lbs.)		Average Bond Stress (psi)	
ASTM Type	Compressive Strength (psi)			Smooth	Dur-O-waL Deformed	Smooth	Dur-O-waL Deformed
M	3330	9 gage	4"	505 2	1520 1	272	817
S	2300	9 gage	4"	—	1485 1	—	798
N	1280	9 gage	4"	430 2	1275 4	231	685
M	3330	9 gage	6"	—	1435 1	—	514
S	2300	9 gage	6"	—	1510 1	—	541
N	1280	9 gage	6"	—	1485 3	—	532
M	3330	3/16"	6"	—	2275 3	—	644
S	2300	3/16"	6"	—	2280 4	—	646
N	1280	3/16"	6"	—	2170 2	—	615

1 — Rods failed in tension before slipping in mortar in all tests.
2 — Rods slipped in mortar at load indicated in all tests.
3 — Rods failed in tension in majority of tests.
4 — Rods slipped in mortar in majority of tests.
*Tests conducted by Research Foundation, University of Toledo.

EFFECT OF WIRE FINISH ON BOND

Test results shown in Table 10 indicate that the wire finish has little effect on the bond with mortar. The amount of lap at splices is applicable to all finishes.

TABLE 10: EFFECT OF WIRE FINISH ON BOND OF DEFORMED WIRE

Wire Finish	Relative Bond Values*
Brite Basic	1.00
Mill Galvanized	1.02
Hot Dip Galvanized	.95

*Based on average results of pull-out tests of No. 9 gage and 3/16 in. wire embedded in ASTM Types M, S and N mortars.

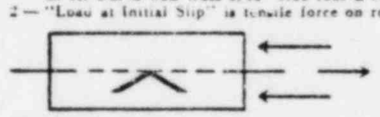
ANCHORAGE

In addition to bond strength of deformed side rods with mortar the anchorage value of the cross rods shown in Table 11 provide substantial protection against slippage of the reinforcement.

TABLE 11: ANCHORAGE VALUE OF WELDED JOINTS*

Mortar Type	Dur-O-waL Type 1	Average Load at Initial Slip 2 (lbs)
M	Standard	1115
	E.H.	1280
N	Standard	665

1 — Standard Dur-O-waL truss No. 9 gage side rods & No. 9 gage cross rods.
E. H. Dur-O-waL truss 3/16" side rods & No. 9 gage cross rods.
2 — "Load at Initial Slip" is tensile force on rod at instant unloaded end registered movement of .0010 inches.



Dur-O-waL Truss Joint Plan View

Cast in 3/8" mortar joints
4" x 8" concrete brick
Cross rods cut off 1/4" from side rods
Side rods neutralized with heavy coating of grease
*Tests conducted by Research Foundation University of Toledo

EFFECT OF DEFORMATION

Deformation of the side rods reduces their cross sectional area, however, the cold rolling deformation process also increases the tensile strength of the steel. The combined effect of this operation is shown in Table 12. Deformed side rods meet or exceed strength requirements of non-deformed wire.

TABLE 12: EFFECT OF DEFORMATION ON TENSILE STRENGTH*

Wire Size	Nominal Area, sq. in.	Required Minimum Tensile Strength lb.**	Test Values of Tensile Strength		Decrease Due to Deforming, percent	Strength Ratio, (Def. Wire / Req'd. Min.) percent
			Plain Wire, lb.	Deformed Wire, lb.		
No. 9	0.0173	1380	1490	1380	7.4	100
3/16 in.	0.0277	2220	2540	2450	3.5	110

*From tests by Prof. Edwin L. Saxer, University of Toledo.
**Nominal cross-sectional area (As) x 80,000.

Reference 1

Table V — TRANSVERSE LOAD DATA

Wall Thickness (inches)	Property	Header	Type of Tie	
			Special	Standard
8	average load, psf	150	157	143
	standard deviation, psf	26.5	23.1	11.5
	coefficient of variation, %	17.6	14.7	8.1
	average stress, psi	98.5	103.0	94.0
	modulus of elasticity, psi x 10^{-6}	2.26	1.34	1.30
12	average load, psf	198	171	177
	standard deviation, psf	2.9	16.2	18.8
	coefficient of variation, %	1.5	9.4	10.6
	average stress, psi	57.7	50.0	51.6
	modulus of elasticity, psi x 10^{-6}	1.23	1.66	1.97

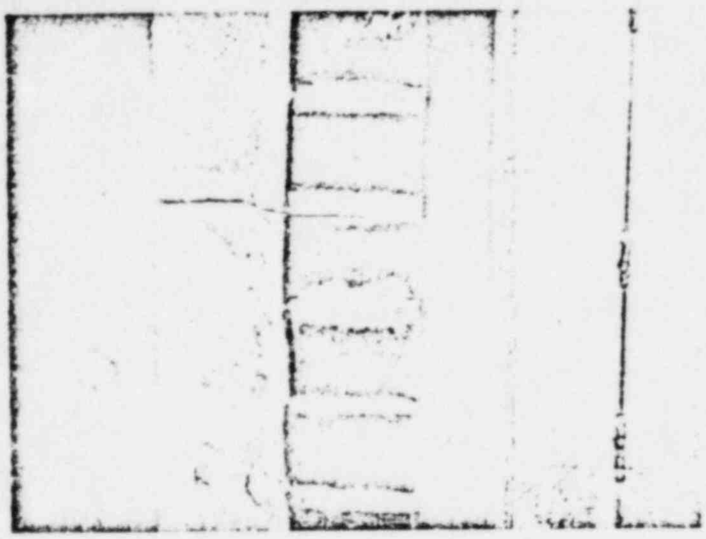


Figure 20 — Typical Transverse Crack.

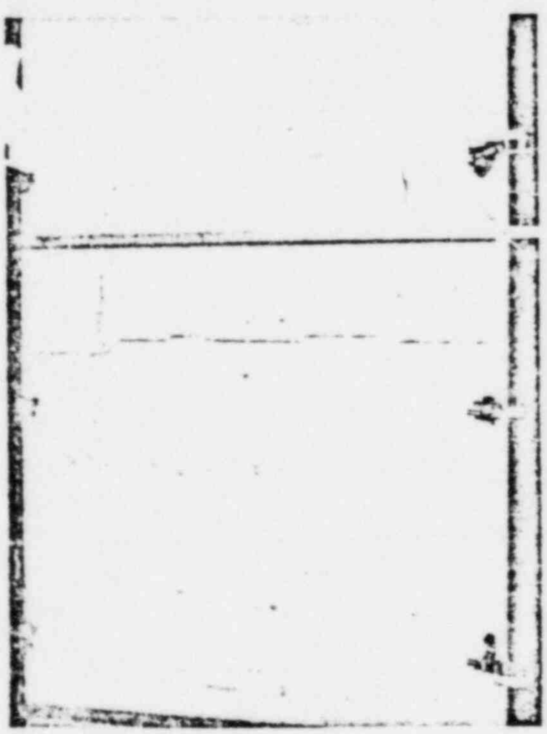


Figure 21 — Typical Failure in Wire-Tied Wall.

psi. Loads calculated from experimental data are greater than the code values of design loads for concrete block and brick masonry construction. The initial failure load value is used to calculate permissible wall strengths; the construction can no longer be considered safe beyond this point, although the walls will support greater loads before ultimate failure.

B. Transverse Strength

Table V lists the results of the transverse loading study of the 8-inch and 12-inch brick and block walls. The data shown in the table exhibits a random variation in all values for the various walls. However, as a group the transverse load carrying capacities are higher for the 12-inch than for the eight-inch walls. While this is to be expected, because of the greater cross section of the 12-inch walls, the increase in load capacity is well below that which would be calculated from the greater wall thickness. The section modules increases as the square of the thickness, thus the expected increase for a 12-inch wall would be 2.25 times the eight-inch wall. The actual increase is only 1.21 times the eight-inch wall load capacities. Other investigators also have noticed that the increase in load resistance for thicker wall specimens is below the theoretical value (ASTM Special Technical Publications No. 166, p. 37). There are two possible explanations for the reduction of strength below the theoretical, the effectiveness of the collar joint in transmitting shear stresses between the brick and the block wythes and the size effects. Young's elastic modulus calculations for these experiments also show random behavior. The average Young's modulus for the eight-inch specimens is 1.63×10^6 psi and that for the 12-inch specimens is 1.62×10^6 psi. This would indicate that the elastic properties measured from bending data are not dependent on size.

A statistical analysis of the data exhibits no real difference in transverse strength due to the type of wall tie. The strength differences shown in Table V are within the normal variation inherent in masonry construction. In order for there to be a significant

Ref 1

strength difference between the wall groups there would have to be a difference of at least 25.4 psi between the eight-inch wall groups and 8.4 psi between the 12-inch wall groups.

The failure patterns indicated uniform loading with failure occurring at a center course of the wire tied walls and at a header course near the center of the wall or the center course between two header courses. Failure occurred when the mortar-to-brick bond broke and the joint opened as illustrated in Figure 20. A typical transverse failure of a wire-tied wall is exhibited in Figure 21. The rupture of the wire-tied walls was random with respect to cracking in a reinforced or unreinforced joint, there being 12 walls of this type. The crack crossing the collar joint (Figure 20) demonstrates that the collar joint is effectively transmitting some shear between the brick and block wythes. A typical header failure is displayed in Figure 22, however, two of the six header samples failed in a block joint as shown in Figure 23.

The load-deflection curves for the transverse strength specimens are given in Figures 24 through 29. The dial gage deflection readings were uniform from side to side, the maximum variation between sides being 0.002-inch at the higher loads. Young's modulus of elasticity in bending was calculated from this data using the secant method since the curves are non-linear. A secant is drawn from the origin through the curve at one half the ultimate strength, the elastic modulus of the specimen is calculated for that load and deflection.

The laterial design load for these walls can be calculated using the criteria for allowable load presented

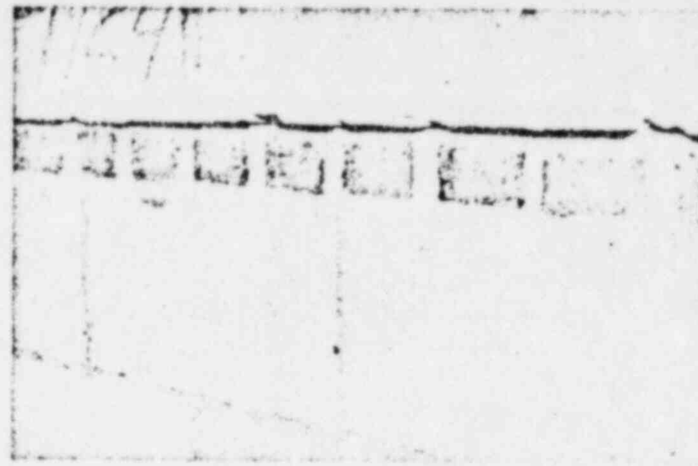


Figure 22 — Typical Failure in Header-Tied Wall.

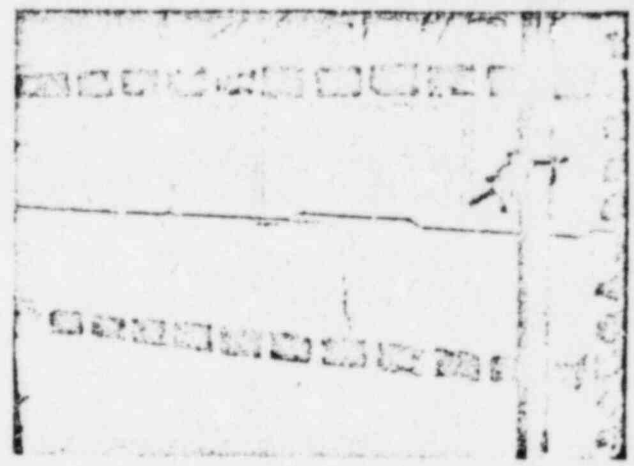


Figure 23 — Occasional Failure of Header-Tied Wall Occurred between Block Courses.

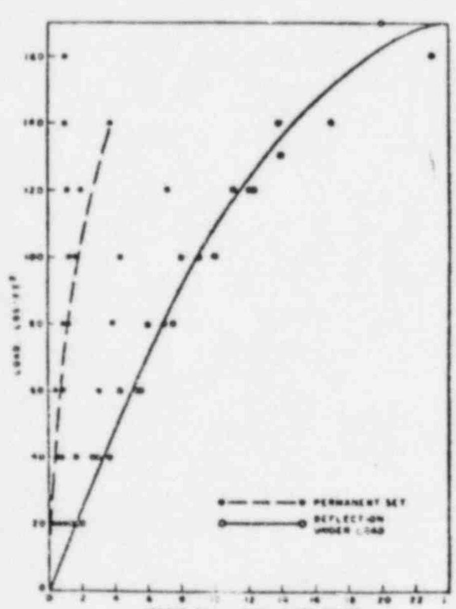


Figure 24
Transverse Load on 8-Inch Walls.
Header Brick Tie.

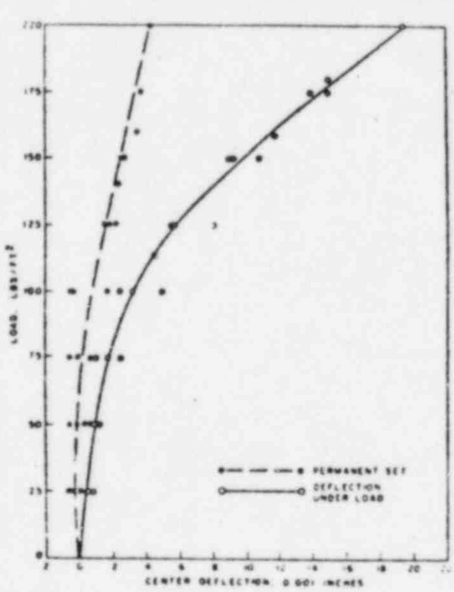


Figure 25
Transverse Load on 8-Inch Walls.
Special **DUR-O-WAL** Tab Tie

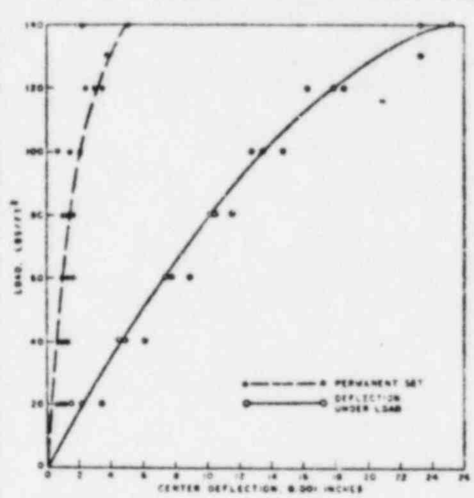


Figure 26
Transverse Load on 8-Inch Walls.
Standard **DUR-O-WAL** Truss Tie

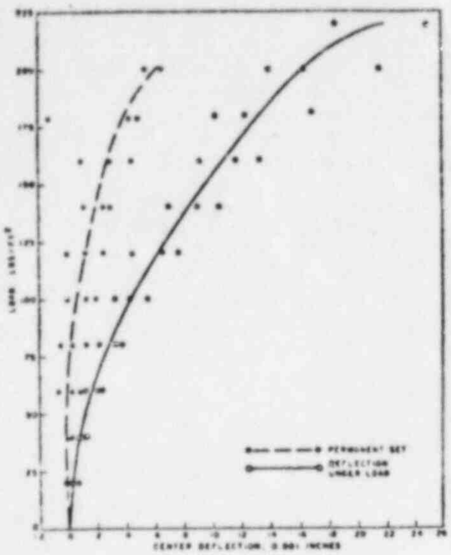


Figure 27
Transverse Load on 12-Inch Walls.
Header Brick Tie.

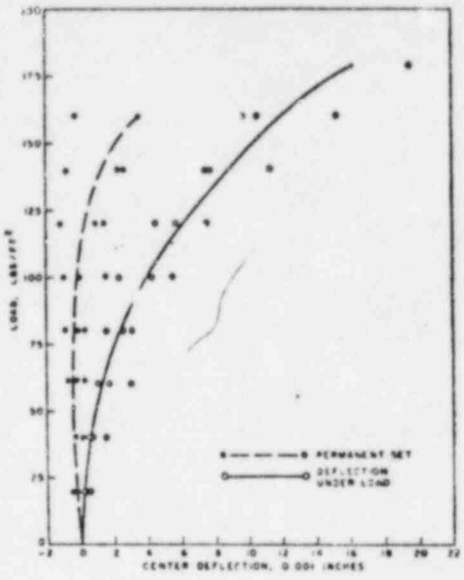


Figure 28
Transverse Load on 12-Inch Walls.
Special **DUR-O-WAL** Tab Tie

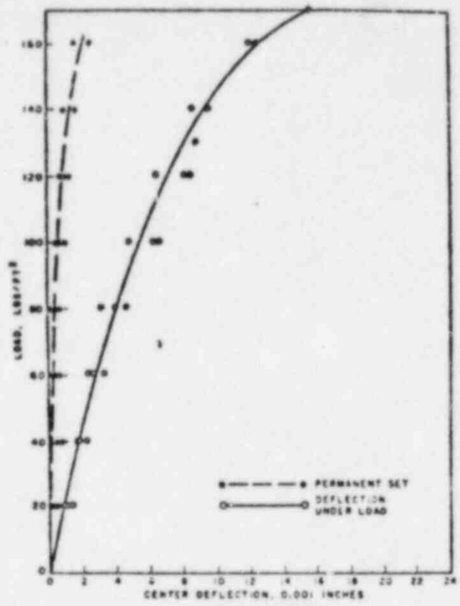


Figure 29
Transverse Load on 12-Inch Walls.
Standard **DUR-O-WAL** Truss Tie

in BSM Report 109, 40% of the least maximum load. The least load for the 8-inch walls is 130 psf and for the 12-inch walls is 152 psf. Therefore, the allowable design load for the 8-inch walls would be 52 psf and 61 psf for the 12-inch walls. These loads reduce to an allowable bond stress of 34.1 psi for the 8-inch walls and 17.8 psi for the 12-inch walls. The reduction in allowable stress is due to the size effects that have been found, from available experimental evidence, to be in operation.

On the basis of the seven feet six inch heights of these walls, the allowable lateral loads are in excess of the usual code requirements of 20 to 30 psf.

WATER PERMEABILITY STUDIES

Sketches were made of the area of water penetration on each block course after the walls were dismantled. In Fig. 30 and 31, the block courses are numbered according to their position in the wall, the lowest course being No. 1. The courses containing the header brick or the wire reinforcement are so indicated. The notched corners indicate the actual shape of the wall cross section since the brick wythe was somewhat wider than the block wythe. Fig. 30 and 31 indicate that penetration was much less for wire-tied. than for header-tied specimens,

No water came through the block face of the WT 1 wall until the air pressure was raised from 20 to 35 lb. per sq. ft. after 8 days exposure. Leakage did not occur in the WT 2 wall until after 5 days exposure. A wet area appeared on the block side of the WT 3 wall after 2 days exposure. Shortly afterward, considerable moisture began to leak through the lower portion of this last specimen, indicating faulty wall construction. The appearance of this wet area was different from the normal leaks. Typical behavior for wire-tied walls was the appearance of moisture in the collar joint after about 8 hr. exposure. The wet areas in the collar joint would continue to spread until, after 5 or 6 days, wet areas would appear on the block face. Movement of the moisture appeared to be caused by slow absorption into the mortar.

For the header-tied walls (HT 1, 2, 3,) moisture appeared on the back of the specimens within the first hour of exposure. Within 2 or 3 days, the ends of the header bricks showed considerable wet areas and moisture was beginning to appear in the block joints. A close inspection reveals that the brick-mortar interface acts as a duct to transfer moisture through the wall. Although water tended to collect in the collar joint the same as in the wire-tied walls, the headers acted as a bridge to carry the moisture across this barrier. At the end of the exposure period, the header walls showed areas similar in appearance to the wire-tied wall with the construction flaw. All of the header-tied walls accumulated considerable water in the joints and cores of both the brick and the block. This was not true of the wire-tied walls where most of the moisture was confined to the collar joint.

The permeability sketches (Fig. 30 and 31) exhibit numerous areas of moisture penetration close to but not through the block face. The joints on the block side of the walls were struck off with a jointing tool while those on the brick face were cut off flush using a trowel. The slightly greater density of the struck joint probably acted as a moisture barrier.

Ref. 2

TABLE 3—TRANSVERSE STRENGTH OF MASONRY WALLS

Wall pattern	Mortar type	Vertical span transverse strength, lb per sq ft: No compressive load, lb per sq ft	Vertical span: No compressive load, Percent of standard	Vertical span: No compressive load, Average*	Vertical span: Compressive load, 85 psi	Horizontal span transverse strength: Unreinforced, lb per sq ft	Horizontal span: Unreinforced, Percent of standard	Horizontal span: Unreinforced, Average*	Horizontal span: Horizontal reinforcement, lb per sq ft, 16 in. c-c	Horizontal span: Horizontal reinforcement, lb per sq ft, 8 in. c-c
8 in. running bond (standard)	M	60.0	100		425.0	127.0			149.4	203.0
				100		136.0	100			
						127.0				
	S	33.7				123.0	100	100	149.9	202.3
		30.2	100							
Horizontal stacked bond	M	85.0	141		405.0	47.7	37		130.0	191.2
	S	39.9	120	130		29.2	24	28	131.3	190.0
Vertical stacked bond	M	60.6	101		357.5					
	S	24.6	74	87						
Diagonal basket weave	M	89.0	148		410.5	69.5	56	60		
						76.9	67			
	S	51.7	155	151		65.8	57			
Diagonal running bond	M	103.0	172		429.0					
	S	48.1	144	158						
Basket weave A	M	69.9	117		400.0					
	S	30.7	92	105						
Basket weave B	M	42.9	72	72						
		44.5	73							
	S	24.4	73							
Coursed ashlar	M	43.3	72							
		55.5	93	83						
	S	26.3	79							
		34.7	104							
4 in. running bond	M	65.0	108			169.4	129		160.0	
						173.0	133	130	193.0	196.0
	S	31.3	94	101		156.0	128		186.6	194.7

*Average for Type M and S mortars

strength of the walls. This continuity of the vertical mortar joints also stiffened the walls in the compressive tests, as shown by the reduced transverse deflections of the horizontal stacked bond pattern as compared with the running bond. The difference between the bond strength and tensile strength of the mortars was also demonstrated in the failure pattern of most of the walls tested. With the running bond pattern made with both the 4 in. and 8 in. high units, and the coursed ashlar patterns, the location of horizontal bond failures was between mortar and units on the side of the mortar joint opposite a vertical joint.

For walls with straight horizontal joints, closer spacing of the joints developed more uniform bending and higher flexural strengths. Although there was little difference in the flexural strength of walls with joint spacings of 4 or 8 in., there was an appreciable loss in strength when joint spacing was increased to 16 in. Combinations of the effects of horizontal joint spacing and continuity of vertical mortar joints greatly account for the wide range of recorded data.

For the running bond walls, the values of transverse strength (lb per sq ft) recorded in Table 3, are, for the test setup and span used, nearly equal to the computed extreme fiber stresses, and hence to the bond strength of the mortar (psi). These values indicate that the bond strength of the Type M and S mortars were about 56 psi and 32 psi, respectively. These compare favorably with corresponding bond strengths of 58 psi and 26 psi recorded in Table 1 for two-block piers.

Transverse loads across a horizontal span

This series of tests included four wall patterns with three mortars and with various amounts of reinforcement in the horizontal joints. The horizontal stacked bond was chosen as representative of patterns depending entirely on mortar bond for transverse strength. Other patterns included the diagonal basket weave and the 4 and 8 in. running bonds which developed lateral strength through shear strength in the joints between interlocking units. The transverse strengths of all walls are recorded in Table 3.

From a comparison of the curves of Fig. 10 and transverse strengths recorded in Table 3, it is evident that of the unreinforced walls, those with the running bond pattern were much stronger than the others, and that the unreinforced running bond walls built with the two mortars showed little difference in transverse load resistance across the horizontal span. The transverse strength of the horizontal stacked bond and diagonal basket weave patterns were about 30 and 60 percent of the standard, respectively. The running bond wall with 4-in. units was about 30 percent stronger than the standard wall with 8-in. units.

The addition of reinforcement steel in the 8 in. running bond walls, either in every course or in every other course, had little effect on the stiffness, or transverse deflections, for loads up to about 80 lb per

Ref 3

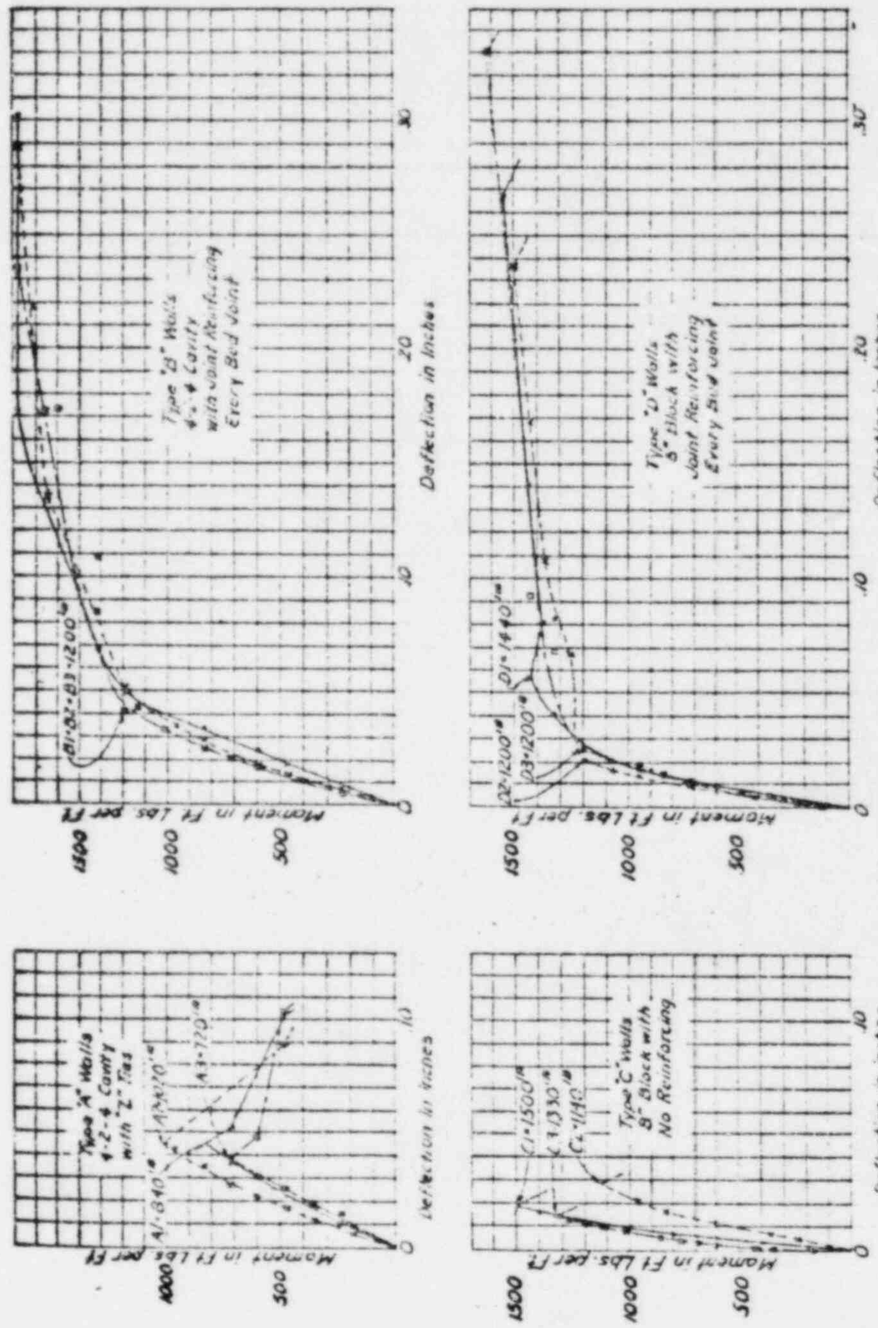


Fig. 6—Moment-deflection diagrams for block walls

ACI -531 COMMENTARY

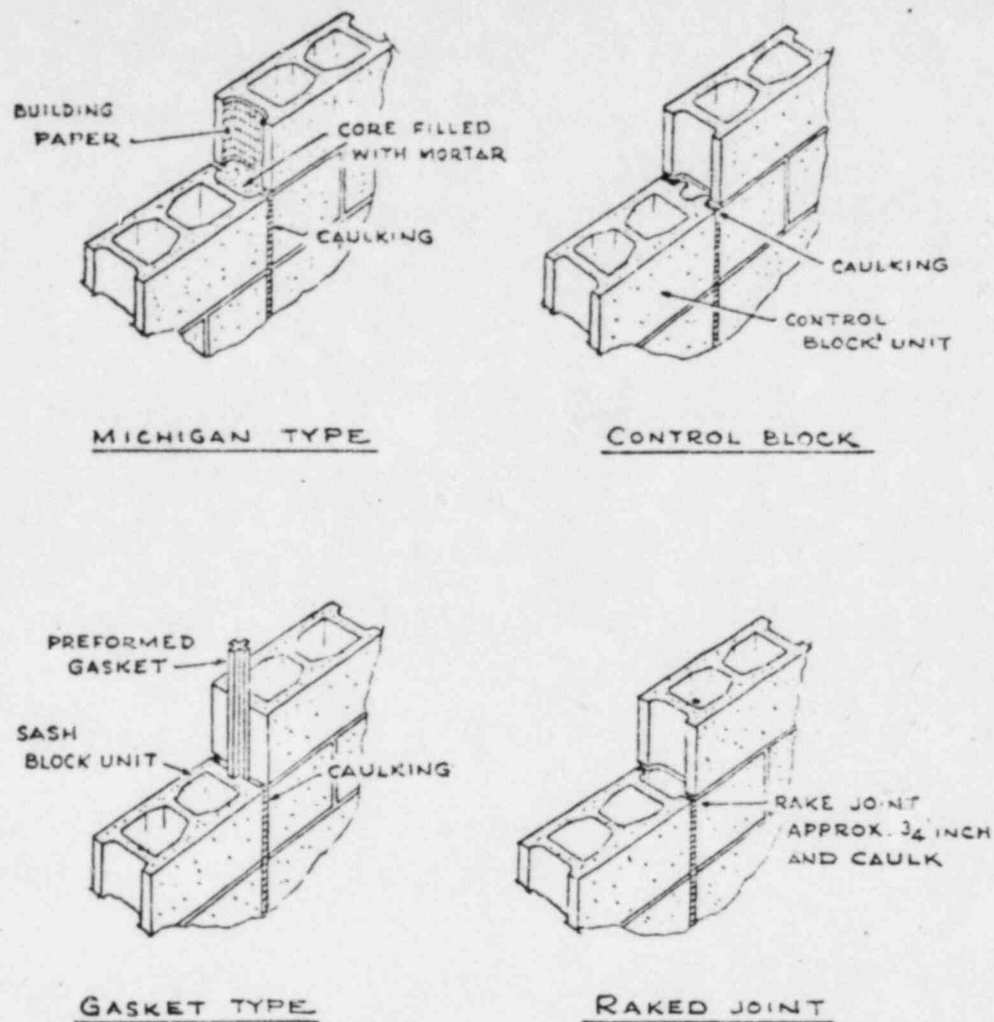


Fig. 6.7a—Control joints—Shrinkage

6.7—Control joints

The location of control joints, bond beams, and joint reinforcement should be clearly shown on the plans.

Control joints are continuous joints, usually vertical, built into concrete masonry walls where stresses might concentrate, to aid in controlling wall movements. They are usually located at vertical mortar joints to minimize any cutting of units. The joints should permit free movement, but have sufficient strength to resist required loads. They should be weathertight when located in exterior walls. Some types of control joints and their spacing are shown in Fig. 6.7a and 6.7b. Where there are control joints in the foundation, veneers and other contiguous construction, they should correspond to the control joints in the wall.

A bond beam is a masonry course which is generally constructed of special shaped units which are filled with concrete or grout and reinforcement. It may serve both as a structural element and as a means of crack control. The reinforcement should be at least two #4 bars.

Bond beams are generally located at lintels, sills, floors, roofs, top of wall, and as needed for wall stiffness. When serving as a means of crack control only, they should be discontinuous at control joints. Where structural requirements make bond beams continuous at control joints a dummy joint should be provided to control the location of the anticipated crack.

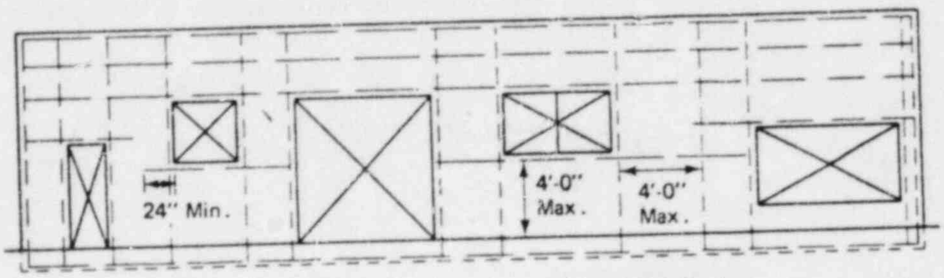
Horizontal joint reinforcement may be used in the wall to increase the tensile resistance and as a means of crack control. Typical spacing of joint reinforcement is shown in Table 6-2. Longitudinal wires should be a minimum of two No. 9 gage.

TABLE 6-2—MAXIMUM SPACING OF CONTROL JOINTS IN NONREINFORCED MASONRY*

	Vertical spacing of joint reinforcement			
	None	24 in. on center	16 in. on center	8 in. on center
Maximum L/H	2	2½	3	4
Maximum L	40 ft	45 ft	50 ft	60 ft

*By use of units of lower drying shrinkage and/or lower moisture content, and by consideration of additional factors such as tensile strength, extensibility, dead load, and mortar qualities, it may be possible to safely exceed these limits.

Reinforced Masonry Design Book
By
Robert R Schneider & Walter L Dickey



Arbitrary Reinforcing:
0.0007bt min. either direction
0.002bt min. total both directions

FIGURE 4-2. Typical wall reinforcing requirements for reinforced masonry.

Types of reinforcement

The type of steel used to reinforce masonry is the same as that used in reinforced concrete (i.e., the bars must comply with ASTM Standard A615-Grade 40, 50, or 60). The allowable design stress for grades 40 and 50 is 20,000 lb/in.^{2} in flexural tension (walls and beams), increasing to 24,000 lb/in.^{2} for steel with a yield of 60,000 lb/in.^{2} or more. The allowable axial compressive stress in columns is set at 0.4 of the minimum yield strength, with a 24,000 lb/in.^{2} maximum. The unknown factor of the relationship between the moduli of elasticity of the masonry and reinforcing ($n = E_s/E_m$), plus the fact that higher steel stresses, accompanied by greater elongations, might result in undesirable cracking in the masonry, tend to discourage the use of the higher-strength steels. Generally grade 40 is recommended for its greater ductility. However, in special circumstances where there are very heavy loads on high-rise bearing walls or masonry columns, a high-strength steel (A615-grade 60) might be used. Maximum size reinforcing must be limited to No. 11 bars. Sizes are specified in terms of the number of eighths of inch of bar diameter.

Prefabricated joint reinforcing (ASTM A-82) can be used in the masonry bed joints, either as a part of the required minimum horizontal reinforcing or as flexural tensile reinforcing. The allowable stress may be taken as 50% of the minimum yield, with a 30,000 lb/in.^{2} maximum. The longitudinal wires in the ladder type are joined with intermittent perpendicular cross wires called "spacers" (Figure 4-3). Another type has diagonal cross members forming a sort of truss.

Joint reinforcing possesses certain advantages. Since it has a greater surface area, it will develop a better bond with the masonry than will the larger reinforcing bars. Further, since it is closer to the outer fibers, it will begin to function much earlier in the loading process, with less cracking of the masonry taking place.

QUESTIONS

4-1. When mortar was originally used, what purposes did it serve?

4-2. What are some of the modern functions of mortar?

4-3. Why do you add lime to the mortar mix?

4-4. What

4-5. Desc

4-6. What

4-7. Why

4-8. What

4-9. What

4-10. What

4-11. What

4-12. Desc

4-13. Exp

4-14. Wh

4-15. Ma

4-16. W

4-17. W

4-18. W

4-19. W

4-20. W

4-21. N

e. All cells containing reinforcement shall be filled solidly with grout. Grout shall be poured in lifts of 4 feet maximum height. All grout shall be consolidated at time of pouring by puddling or vibrating and then reconsolidated by again puddling later, before plasticity is lost.
   When total grout pour exceeds 8 feet in height the grout shall be placed in 4-foot lifts and special inspection during grouting shall be required. Minimum cell dimension shall be 3 inches.

f. When the grouting is stopped for one hour or longer, horizontal construction joints shall be formed by stopping the pour of grout not less than ½ inch below the top of the uppermost unit grouted. Horizontal steel shall be fully embedded by grout in an uninterrupted pour.

1.10 Reinforcing Steel

The reinforcing steel generally used in reinforced masonry structures is Intermediate Grade ASTM A615 Grade 40 with an allowable stress of 20,000 psi (137.9 MPa). However, it is becoming very common to specify A615 Grade 60 which has an allowable stress of 24,000 psi (165.5 MPa) where high overturning forces or highly loaded columns are required.

Maximum size of reinforcing steel in masonry should be No. 10 bars for columns and a maximum of No. 11 bars for tension due to overturning moment.

Prefabricated joint reinforcing used in the horizontal masonry joints can be considered as part of the minimum required reinforcing steel. It may be used as structural reinforcing to resist lateral forces and increase the structural strength of the wall. Joint reinforcing in 9 gage (3.8 mm), 8 gage (4.1 mm), 3/16 inch (4.7 mm), ¼ inch (6.4 mm) and 5/16 (7.9 mm) diameter wire is fabricated from high strength cold drawn wire ASTM A82, which has an allowable stress of 30,000 psi (206.8 MPa). Joint reinforcing regularly spaced in the wall is influential in controlling shrinkage cracks.

1.11 Stresses, Allowable Increases

All allowable stresses may be increased one-third for the temporary short term loading due to wind or earthquake forces provided the size of member and the amount of reinforcing steel thus determined is not less than that required for normal dead and live loads alone.

1.12 Minimum Reinforcement

The 1976 Uniform Building Code requires a minimum of total steel in the wall, $A_s = 0.002bt$. The minimum steel, $A_s = 0.0007bt$, may be either vertical or horizontal.

Excerpt from 1976 UBC Sec. 2418(j)3.

3. **Reinforcement.** All walls using stresses permitted for reinforced masonry shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall and the minimum area of reinforcement in either direction shall be not less than 0.0007 times the gross cross-sectional area of the wall. The reinforcement shall be limited to a maximum spacing of 4 feet on center. The minimum diameter of reinforcement shall be ⅜ inch except that joint reinforcement may be considered as part of the required minimum reinforcement.

It is also recommended that minimum steel for flexural beam members, not walls, be not less than $p = 80/f_y$. Therefore for intermediate grade steel, $f_y = 40{,}000$ psi (275.8 MPa), minimum $p = \frac{80}{40{,}000} =$ 0.0020. The amount of minimum reinforcement may be less if the amount provided is at least one-third greater than that required by analysis. The Denver, Colorado, masonry code specifies minimum steel as $p = 52/f_y$. Therefore for $f_y = 40{,}000$ psi (275.8 MPa) minimum $p = 52/40{,}000 = 0.0013$.

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By
J.E. Amrhein

Question: (2b) Provide the number of walls which were qualified relying on the strength of the horizontal reinforcement.

Response: The number of walls which were qualified using horizontal reinforcement was 18.

Question: (2c) Provide the number of walls which were qualified relying on the strength of the vertical reinforcement.

Response: The number of walls which were qualified using vertical reinforcement was 33.

Question: (2d) Since no documentation of inspections for masonry work was found the Licensee is requested to confirm that the horizontal and vertical reinforcement exists in the walls as specified in the design and discuss the basis for this confirmation.

Response: The original construction of all masonry walls at Zion was performed in accordance with Sargent & Lundy Design Specification X-2259 and Standard 1727. The Specification requires full written approval of the contractor's Quality Assurance Program by both Commonwealth Edison and Sargent & Lundy prior to commencement of any construction. The Standard requires all work to be performed according to fully approved shop drawings. Commonwealth Edison Quality Assurance and Quality Control personnel were involved in all phases of the construction. This involvement included inspections of work in the plant as well as review of the contractor's quality control records.

Additional work on masonry walls is now conducted under an updated Specification and Standard. Quality Assurance and Quality Control requirements continue to apply, in a manner similar to that of original construction.

In our judgment, the controls provided by the Quality Assurance Programs of Commonwealth Edison, Sargent & Lundy, and the masonry wall contractor provide assurance that wall reinforcement is installed as designed.