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I am submitting herewith a thesis written by Larry S. Johnston entitled "The Use of Welded Wire Fabric as Shear Reinforcement in Concrete Lintels." I recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Engineering, with a major in Civil Engineering.

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THE USE OF WELDED WIRE FABRIC AS SHEAR REINFORCEMENT
IN CONCRETE LINTELS

A Thesis
Presented for the
Master of Engineering
Degree
The University of Tennessee, Knoxville

Larry S. Johnston

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Finally, I am very grateful to my brother, Greg, for his long hours of assistance in conducting the actual load tests and to Brenda Smith for helping organize my initial rough drafts.

ABSTRACT

The purpose of this study was to evaluate design assumptions in the use of welded wire fabric as web reinforcement in precast concrete lintels. Casting processes have made the use of conventional shear reinforcement in form of stirrups not possible. Welded wire fabric placed vertically in the longitudinal direction has been employed to act as shear steel to accommodate fabrication procedures.

In the evaluation there was a total of sixteen full size precast lintels which were fabricated by Southern Cast Stone Company as representative of their normal casting process. In the sixteen lintels were three commonly produced cross sections. These cross sections varied in depth, so that, the test variables were amount of shear reinforcement and beam depth.

Concurrent with the load testing operation, deflections at midspan were measured to evaluate the flexural characteristics of the lintels as compared to calculated values. In certain lintels, electrical resistance gages were attached to the reinforcement to monitor stresses under loading.

The major finding of this research work was that the welded wire fabric did not increase the load capacity of lintels in two of the three cross sections. From these

results some additional questions have been raised that will require additional testing in this area to be answered.

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LIST OF SYMBOLS

A.C.I.	American Concrete Institute
A.S.T.M.	American Society for Testing and Materials
d	depth from top of beam to centerline of flexural steel
h	beam total depth
a	shear span
A_s	area of flexural reinforcing steel
b_w	beam width
w	uniform load (pounds per lineal foot)
V_n	nominal concrete beam shear strength
V_c	concrete shear strength alone
V_s	shear strength of web reinforcement
f'_c	concrete compressive strength
f_r	modulus of rupture
p.s.i.	pounds per square inch
ρ	percentage of flexural reinforcement ($\rho = \frac{A_s}{b_w d}$)
E_c	modulus of elasticity of concrete
I_g	moment of inertia of the uncracked section
I_e	effective moment of inertia of the cracked section

CHAPTER I

INTRODUCTION

Concrete masonry block has proven to be a very economical means of construction since its introduction in the early 1900's. The major reason for its economy is that it provides a fast means of constructing load bearing walls using manual labor only. Concrete block also has the advantage of serving as a finished wall surface after painting. To achieve a uniform texture in exposed masonry block walls with openings, there exists a need for beams made from the same type concrete. These concrete beams are commonly referred to as "lintels." The term lintel actually applies to any supporting member over a wall opening, but it will be used here to specifically describe precast concrete beams having the same surface texture as concrete block. Masonry block is made from a concrete mix having a very low water-cement ratio, which requires a special casting process. A similar casting method is required with lintels to reproduce the same surface texture. This style of casting will be discussed in more detail in Chapter II.

The earliest lintels may or may not have had detailed reinforcement design, a fact which probably had very little effect on their performance under service load conditions.

These lintels had such low service loads and short spans that they were functioning in most cases as uncracked sections. Very little knowledge, if any, was available to early concrete block pioneers about the design of reinforced concrete beams. Due to this limited knowledge of the load carrying capacity of lintels, not much change took place in accepted practices. If a certain size building had proven to work well, that same size structure was built over and over again.

More current times brought an age of increased research in the area of reinforced concrete by numerous load tests and mathematical analysis. This research yielded valuable knowledge of the strength and behavior of reinforced concrete beams which led to much sophistication in design. With concrete beam design a more exact science, new innovations were introduced in building construction. Longer spans, heavier allowable loads, and more slender members were just a few of the possibilities. These same trends were also incorporated in concrete lintel design. Lintels were no longer bound by accepted practices which had governed their use in previous times. Needed flexural capacity was achieved by producing deeper cross sections or by adding more reinforcing steel to standard sections. Also, with the increasing use of ultimate strength design, more high strength steel (A.S.T.M. A615, grade 60) was used. Due to the required method of casting, stirrups could not be added to the lintel

cross section. This left the shear strength of the concrete as the principle limiting design factor. The allowable working shear stress of $0.03f'_c$ which had been established in the 1940's "Recommended practice and standard specifications for concrete and reinforced concrete" was reduced in 1963 by the A.C.I. 318-63 building code to a value of $1.1 \sqrt{f'_c}$. This represented a reduction of 33 percent for 3000 p.s.i. concrete. With this further limitation on design capacity an increased need for shear reinforcement was apparent.

Southern Cast Stone Company introduced a solution to this need for shear reinforcement in the mid 1960's. Welded wire fabric (w.w.f.), better known as wire mesh, was placed in lintels in vertical layers, which extended longitudinally along the beam, in order to provide that reinforcement. Wire mesh is anchored in concrete by its transverse wires. For this reason w.w.f. seemed the perfect solution to developing a tension force across the shear cracks without using conventional stirrups. In 1971 the A.C.I. 318-71 building code was the first to approach the use of welded wire fabric as shear reinforcement. It recommended standard details, but in all cases showed the wire mesh bent around the main flexural reinforcement with development provided above the mid-depth of the beam. The use of vertical layers of wire mesh running longitudinally along the beam was not mentioned. This still gave no answer to whether this method used at Southern Cast Stone was valid or not. Using the wire mesh

in lintels as shear reinforcement has become a standard practice over the last 20 years without being justified by load tests. This investigation has been originated for the purpose of determining the contribution of welded wire fabric to the ultimate shear strength of concrete masonry lintels.

CHAPTER II

CONCRETE LINTELS

A clearer picture of the production procedure used in fabricating concrete lintels can be obtained by first understanding block production. A building material such as concrete masonry block must be capable of being produced by the thousands daily. This type mass production is achieved by use of automation and a very dry concrete mix, meaning a low water-cement ratio. Just enough water is added to the mix for hydration of the cement which leaves it after mixing with the consistency of a coarse sand that is moist and loose. This consistency is important because just as moist sand will hold a molded shape so will concrete with a low water-cement ratio. Using pressure and vibration to consolidate materials, the concrete is forced into a block die. This die is immediately removed leaving the concrete to hold its new molded shape. Another block is prepared in the same fashion while the first block is moved out by conveyor to be stacked and later taken to be cured. As can be seen this method of casting does lend itself quite well to mass production.

The casting process discussed above leaves a surface texture that is flat but porous due to the lack of cement

slurry that would normally fill in around the coarse aggregate in most mixes. To reproduce this same surface texture, concrete lintels are made using a similar concrete mix and casting process. The proportions of the mix are 300 pounds of lightweight aggregate, 100 pounds of mascot sand, and 100 pounds of Type I Portland Cement. The lightweight coarse aggregate used is an expanded shale processed by a rotary kiln method. Its gradation is shown in Table I. Water content in the aggregate is somewhat variable because of storage methods which makes the free water in the mix variable also. The man operating the mixer uses the mix consistency as an indicator of correct water content. The approximate amount of water added to the proportions given above is 3 1/2 gallons. This represents a water-cement ratio by weight of 0.29.

The quantity of lintels produced by Southern Cast Stone Company can in no way compare to that of its counterpart block. This simply means that the small volume of lintels produced does not justify automation; so they are produced manually by two man crews. These men receive a fabrication schedule for each lintel which shows dimensions of the cross section, length and reinforcement. It is their responsibility to set up a mold to the specified dimensions before casting begins. Next the reinforcing bars are coated with a cement and water slurry mixture. This procedure is used to insure that all the deformation on the reinforcement

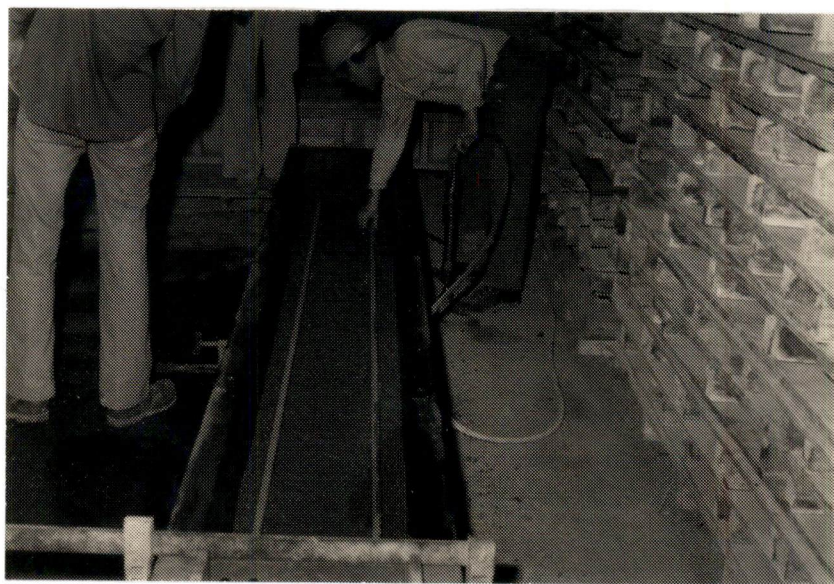
TABLE I
COARSE AGGREGATE GRADATION

<u>Sieve Size</u>	<u>Percent Passing by Weight</u>
3/8"	100
No. 4	85-95
No. 16	40-55
No. 50	20-30
No. 100	10-20

will interlock with the drier concrete. Just as the outside surface is textured and porous, so would be the interface between steel and concrete if not for the slurry mixture. After the bars are coated, the casting process begins with one of the men shoveling the first "lift" or layer of concrete into the mold. The second man follows behind the first compacting the loose concrete material with an air operated tamper. (See Figure 1a.) Each lift is approximately three to four inches deep and compresses down to about half its original loose depth. In most of the standard cross sections only four lifts are required to complete the fabrication. Reinforcement is added normally after the first and third lift so that a one and a half inch minimum clear distance is maintained. Equal amounts of top and bottom steel are used so that the member will function as designed even if it is set in place upside down. (See Figure 1b.) Upon completing fabrication each lintel is covered and left to cure overnight. The next morning it has enough strength to be lifted and set off in another area to open-air cure for the remaining days before it is shipped to its destination. The 28-day design strength for this mix is approximately 3000 p.s.i.; the strength at the time of shipment is probably on the order of 2000 p.s.i. The most common lintel sizes are produced in nominal cross sections of eight inch wide by eight inch deep ($7 \frac{5}{8}$ " x $7 \frac{5}{8}$ "), eight inch wide by twelve inch deep ($7 \frac{5}{8}$ " x



(a) Placement and compaction of concrete



(b) Top and bottom reinforcement

Figure 1: Fabrication process for concrete lintels

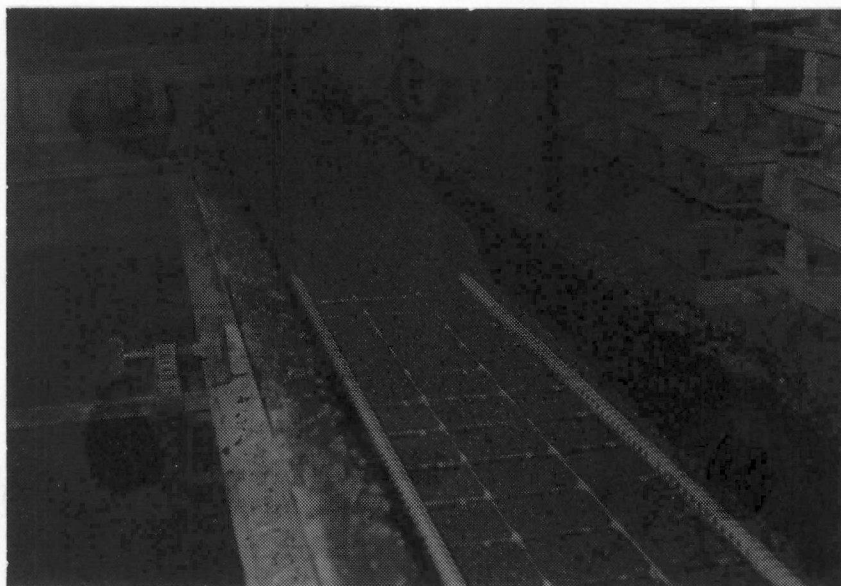
11 5/8"), and eight inch wide by sixteen inch deep (7 5/8" x 15 5/8").* These sections are set up in modules to accommodate the 3/8 inch mortar joints commonly used in concrete masonry block construction.

Probably the most important phase of the casting process and also the most difficult to control is consolidation of the loose concrete. The difference between the block compaction process and the lintel tamping is that the block has confinement from all directions giving optimum results. The tamper used for lintels has a head diameter of approximately three inches which means localized compaction is good, but some minor disruptions are caused in adjoining areas which are not confined. The result is that the quality, meaning by that the strength and density, of each lintel is directly related to consolidation. The minor disruptions which are a part of normal production in lintels do not pose any problem for simply reinforced members. Simply reinforced implies two bars placed in the top and bottom only. Attempts at more complicated steel arrangements have adverse affects on the attainment of thorough consolidation. The vibrations caused by the tamper and the size of the tamper head prohibit the use of bars that project through lifts such as conventional stirrups. This is the dilemma

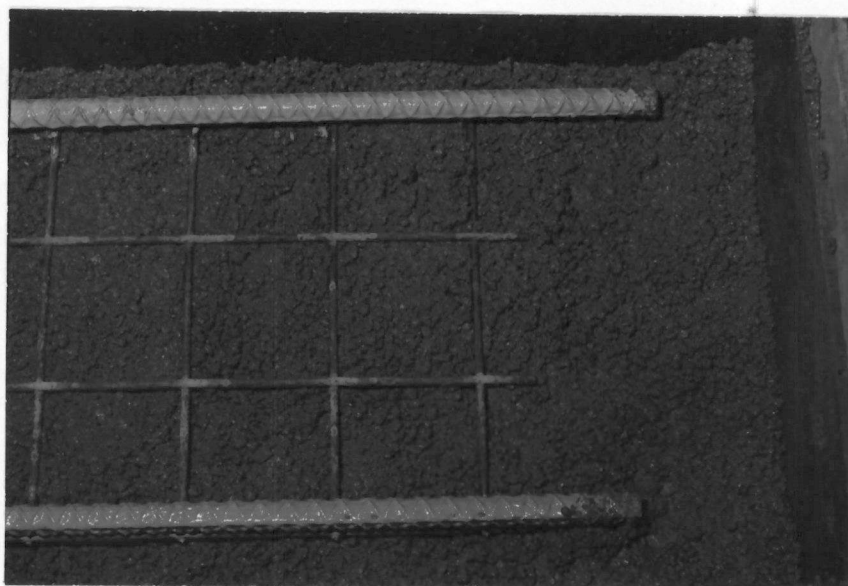
*Actual dimensions are given in parenthesis.

that prompted the use of w.w.f. as a means of shear reinforcement.

Lintels are made lying on their side so that a layer of w.w.f. could be placed between lifts just as the main flexural reinforcement. This is convenient and provides a minimum amount of additional disturbances in compaction. The w.w.f. used almost exclusively is designated as a 4 x 4 - W4.0 x W4.0 mesh (A.S.T.M. A185). This designation means the mesh is made from four gage wire ($A_s = 0.04 \text{ in.}^2$) in both directions at four inch centers. For placement in common lintel sizes the mesh is cut four inches wide with two longitudinal wires for the eight inch deep member, eight inches wide with three longitudinal wires for the twelve inch deep member, and twelve inches wide with four longitudinal wires for the sixteen inch deep member. This allows a clear distance of approximately two inches at the top and bottom. Normally, one layer of w.w.f. which runs the full length of the member is used if required by design, but if necessary a second layer can be added. These layers of w.w.f. are added in between the same lifts as the flexural reinforcement. (See Figure 2.) If still more shear strength is required for design, then the lintel depth is increased to the next appropriate size.



(a) Wire mesh placement in lintel cross section



(b) Wire placement in 16" lintel cross section

Figure 2: Fabrication of lintels using wire mesh

CHAPTER III

SHEAR ANALYSIS

While the primary purpose of this test series was to evaluate design assumptions in the use of w.w.f. as web reinforcement, there was a potential secondary benefit that would be quite valuable to a designer. This benefit was the verification of published values for the allowable shear strength of sand lightweight concrete as applied to this tamped mix. In trying to make a limited number of load tests provide both of these values much more consideration was given to creating load tests representative of actual field applications. The shear strength of w.w.f. could, most probably, be measured by using any one of many different loadings with only small variation in results. The shear strength of the concrete section alone, however, was quite sensitive to type of loading and was capable of large variations. One of the difficulties to be considered is that many times actual field situations cannot be effectively reproduced in a testing laboratory. The only reasonable alternative, then, is to use loading approximations to simulate the desired effects.

In the case of lintel design, just as other beam elements, the applied loads are in most cases uniformly

distributed over the span. Establishing the lintel's response to this type loading would yield a design concrete shear strength relevant to practical cases. The problem with this idea is that applying a uniform load as part of a controlled testing procedure is not practical for a small testing facility. For this reason a two point symmetric loading system was used to simulate the stresses created by a uniform load.

Most of the beam shear testing done prior to this work has been accomplished using a similar type, two point loading. The major concern in these previous tests was establishing the effects of certain parameters, such as concrete compressive strength, type of aggregate used, percentage of flexural reinforcement, and shear span; it was not in approximating a uniform load. What was found in these earlier investigations is that the shear span by far has the greatest effect on the shear strength of a concrete section without web reinforcement. Shear span is defined as the distance between load and reaction; it is commonly given as a multiple of the shear depth "d." Figure 3 that follows is taken from research done by Kani (2) and clearly shows the tremendous increase in shear strength as the load is moved closer to the support. The reason for this increase is due to load induced vertical compressive stresses which reduce diagonal tensile stresses (1).

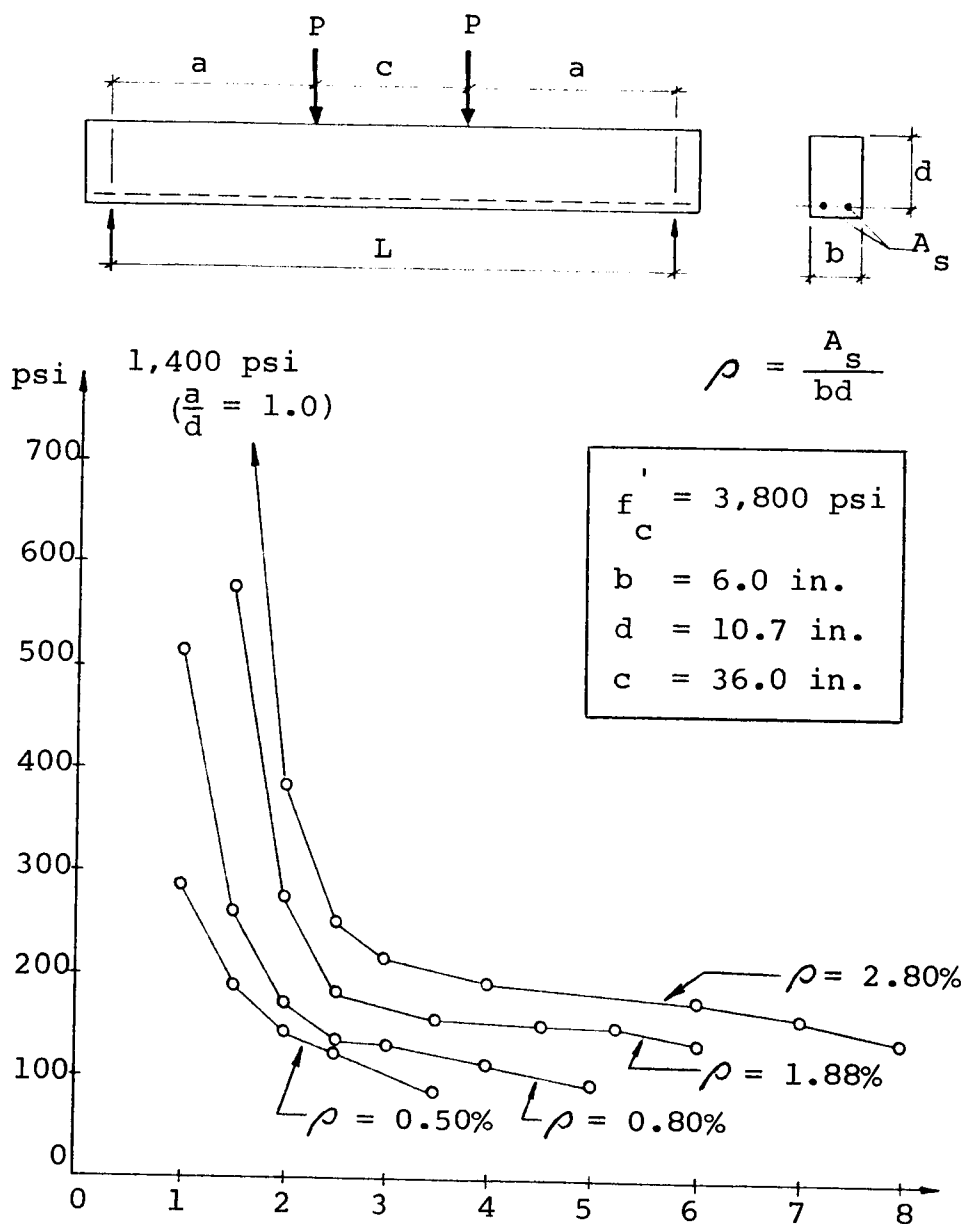
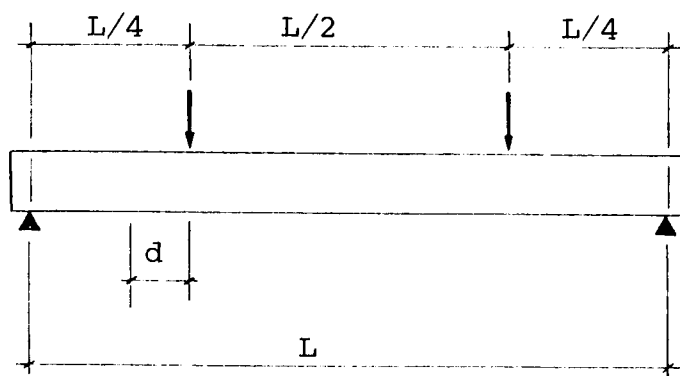


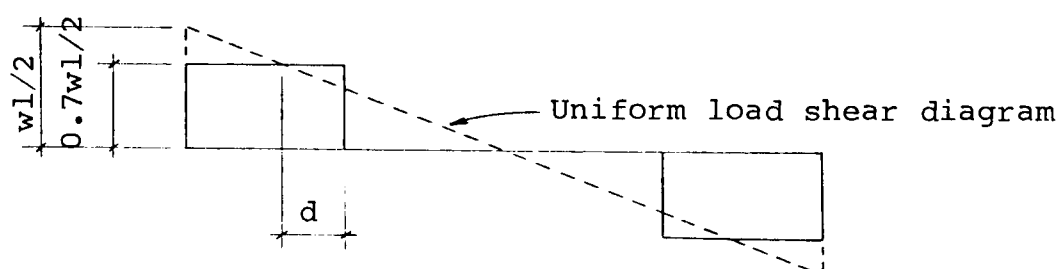
Figure 3: Shear stress at failure versus a/d
(from reference 2)

Shear span is not defined for a uniform load, but it represents an important consideration for these load tests. The minimum shear span used by Hanson (3) in his testing of lightweight concrete beams was $2.5d$; this value was also selected for this series of tests. By using the same shear span as this previous author, a point of reference was created for a possible comparative study. Ferguson noted that load effects had a rapidly reducing influence for shear spans of $2.5d$ and greater (1). This same reducing influence can clearly be seen as the point where a considerable slope change occurs in Figure 3.

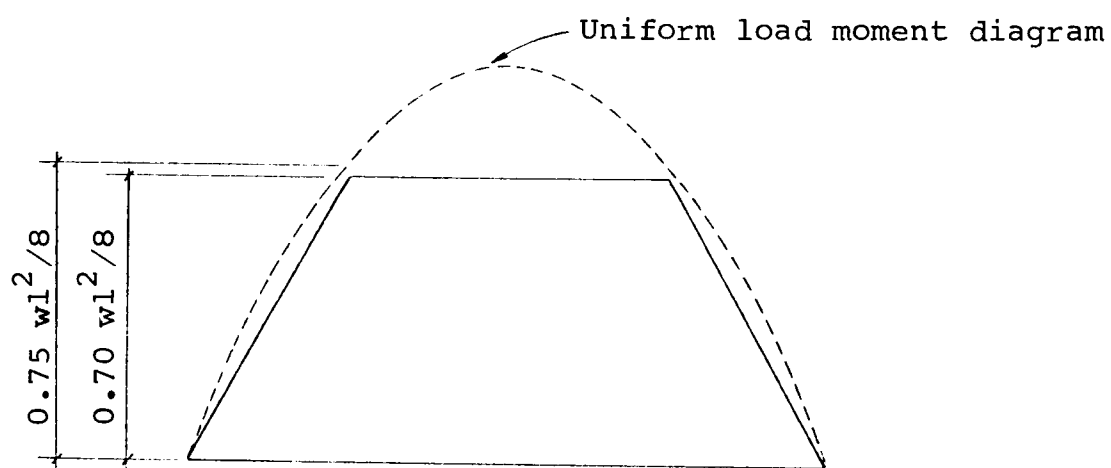
After deciding on the load placement with respect to the support, the decision of load placement within the span had to be considered. The location used as suggested by Kani (2) was the quarter points which appeared to be the best place for simulation of the moments and shears caused by a uniform load. The actual point where the moments and shears were critically compared was a distance " d " from the load in the direction of the support. This point was the anticipated location of the diagonal tension crack (3). In the range of shear spans greater than $2d$, the diagonal tension cracks start from the top of the last flexural cracks (1). In taking this into consideration, it seemed important for the quarter point load to give a good approximation of the slope for the parabolic moment diagram. In Figure 4 a comparison of the shear and moment diagrams of the two type



(a) Quarter point loading



(b) Shear diagram



(c) Moment diagram

Figure 4: Comparison of uniform versus quarter point loading

loads is given. The quarter point loading diagrams are superimposed on those for a uniform loading. In making this comparison the assumption is made that the shear stresses are equal at a distance "d" from the quarter point. This gives the concentrated quarter point load from which the moment diagram can be obtained. As can be seen the slope of the moment diagram for the quarter point loading gives a reasonable approximation of the parabolic curve for a uniform load.

Once these preliminary requirements of loading were determined, the individual member reinforcement designs were left as the final step. The approach to design was the same for each different depth lintel. Contrary to normal accepted practice, each of the lintels was designed to fail in shear. This was accomplished by first calculating the nominal shear strength of each lintel cross section with the maximum amount of web reinforcement (two layers of w.w.f.). The A.C.I. 318-77 code equation for that calculation is given as:

$$V_n = V_c + V_s$$

where,

$$V_c = (0.85) (2 \sqrt{f'_c}) (b_w d) \quad \text{for sand lightweight concrete}$$

$$V_s = \frac{A_v f_y d}{s}$$

This nominal shear strength is the load required to cause an ultimate shear failure assuming adequate moment capacity. The flexural reinforcement was next designed to insure that

shear failure would occur. For example, the sixteen inch deep lintels had an expected ultimate shear load of

$$V_n = (0.85)(2\sqrt{3000})(7.625)(13.5) + \frac{(0.08)(60,000)(13.5)}{4}$$

$$V_n = 25,785 \text{ lbs.},$$

for which the required moment capacity was 75,520 foot-pounds. The moment capacity supplied, based on using 2-#8 reinforcing bars (grade 60) in the bottom, was 80,480 foot-pounds. This same amount of flexural steel was used in all the sixteen inch deep lintels so that the amount of web reinforcement was the only thing varied. The same process was used for both the other sections and gave comparable amounts of steel. The percentage of reinforcement for each of three cross sections varied somewhat, but were all approximately 1.5 percent. The different lintel cross sectional dimensions with flexural steel locations are given in Table II.

TABLE II
TEST LINTEL DIMENSIONS AND FLEXURAL STEEL LOCATION

Lintel Size	Width b, in.	Height h, in.	Steel Depth d, in.	Shear Span a, in.	a/d	Span in.	A_s in. ²	ρ
8 x 8	7.625	7.625	5.5	13.75	2.5	55	0.62	.0147
8 x 12	7.625	11.625	10.0	25.00	2.5	100	1.20	.0157
8 x 16	7.625	15.625	13.5	33.75	2.5	135	1.58	.0153

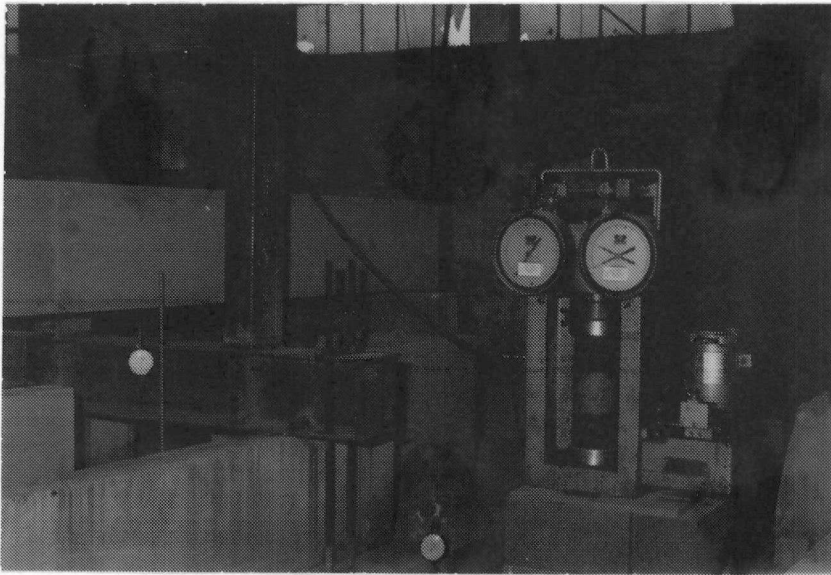
CHAPTER IV

TESTING

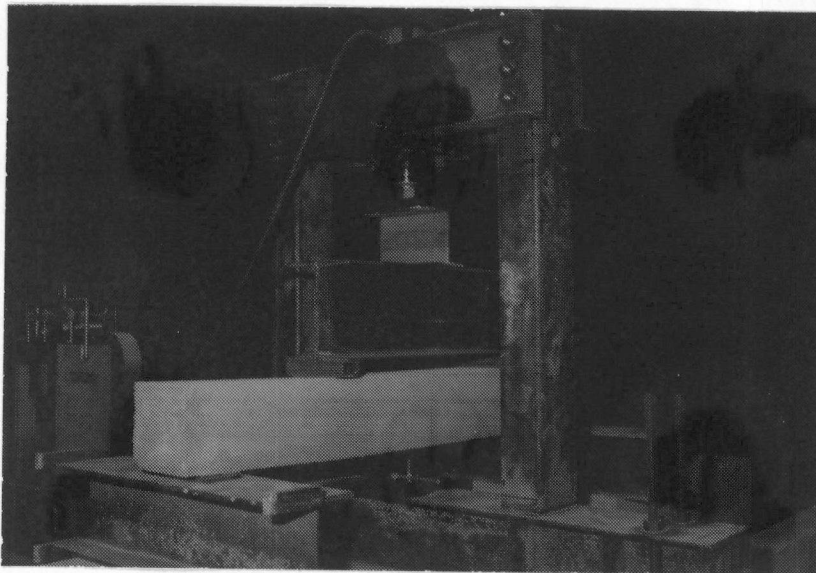
I. TESTING APPARATUS

Part of the preliminary work involved in testing full scale concrete beams was creating a means of applying a monitored load. The loading frame shown in Figure 5 was constructed for that purpose using a steel framework bolted to a concrete platform. This platform had been used in some previous testing and required only minor modifications. A six inch diameter hydraulic cylinder with maximum rated capacity of 84,000 pounds was mounted in the frame and linked to the hydraulic pumping system of a Forney FT-31-DR Portable Testing Machine. The Forney Testing Machine also contained a six inch diameter load cell so that its loading gages could give loads applied by the testing frame directly. To use these gages it was necessary to bypass the Forney load cell to allow the fluid pressure to drive the cylinder in the loading frame. The required loading was to be applied at two points, so steel wide flanges were used as spreader beams.

Another important phase of the loading test was monitoring the lintel centerline deflection. The measurement of these deflections was somewhat complicated by the flexibility of the loading frame's supporting platform. Two deflection



(a) Loading frame, deflection gages, and Forney Testing Machine



(b) Hydraulic loading cylinder mounting

Figure 5: Load testing apparatus

gages were used to take into account movements of the testing platform. As shown in Figure 5a, deflections were measured from the lintel to the platform and from the platform to the floor slab. This meant that the true lintel deflection was found by subtracting the platform movement from that measured for the lintel. The test results given in Appendix A lists the lintel deflection as Δ_1 and the platform deflection as Δ_2 . The term $\Delta_1 - \Delta_2$ is the actual lintel midspan deflection referenced throughout the following discussions.

II. CONCRETE PROPERTIES

In trying to evaluate the properties of this sand lightweight concrete, a different approach was taken. A typical compression test which is required almost always in concrete work was not used. There were two reasons for not running this type test. First, actual diagonal tensile strengths cannot be directly determined from compressive strength. It has been established in past research that the tensile strength for each particular type lightweight aggregate largely influences the concrete shear strength (3). Second, it was felt that tamping in a six by twelve inch cylinder would yield higher compressive strengths than were actually achieved in the lintels. The reasons for this assumption were that more lateral confinement could be provided by a cylinder mold during compaction; also, cylinders would be loaded in the same direction as compaction,

whereas the compressive load in lintels is applied perpendicular to the direction of compaction. For these reasons no attempt was made to fabricate cylinders for compression tests.

Since typical compressive tests were not used, more emphasis was placed on evaluation of the modulus of rupture. To accomplish this, eight small, unreinforced beams, six inch by six inch by twenty-four inch long, were fabricated and cured along with the test lintels. Each of these sample beams was loaded in flexure using a third point loading and an 18 inch span according to A.S.T.M. specification C78-64. The modulus of rupture was then calculated using the equation in that specification of:

$$f_r = \frac{PL}{bd^2},$$

in which P is the total load applied, b is the width of the rectangular beam, and d is the overall depth of the beam. The first half of the samples were broken at 60 days to correspond with the time of testing of the first lintels. The second half were broken at 155 days to evaluate the change of rupture strength with time. Approximate compressive strengths were obtained by using A.C.I. 318-77 code equation 9-9

$$f_r = (0.85)(7.5)\sqrt{f'_c} \quad \text{for sand lightweight concrete}$$

and solving for f'_c . The results of these beam tests are given in Table III. The average value of modulus of rupture for the 60 day tests was $f_r = 405.3$ p.s.i. with a corresponding value of compressive strength of $f'_c = 4057$ p.s.i. The 155 day tests yielded an average $f_r = 454.2$ p.s.i. which represents a 12 percent increase.

III. SHEAR TESTING

This series of load tests had as its main goal establishing the contribution of welded wire fabric to the ultimate shear capacity in each of three commonly produced cross sections. These cross sections, as noted earlier, are eight by eight inch, eight by twelve inch, and eight by sixteen inch. To keep the number of tests to a reasonable limit, the only element varied besides lintel depth was the amount of shear reinforcement. The percentage of reinforcing (ρ) and type of loading was held constant for all lintels, while the amount of w.w.f. was varied from one to two layers in the twelve and sixteen inch sections. The eight by eight inch cross section requires a maximum spacing of shear reinforcement of not more than three inches which is narrower than the wire spacing of the 4 x 4 - W4.0 x W4.0 mesh. For this reason the only amount of w.w.f. used in eight by eight lintels is two layers of mesh staggered to give vertical wires at two inch centers. For each of the twelve and sixteen inch deep sections there

TABLE III
MODULUS OF RUPTURE TESTS ON RECTANGULAR BEAMS

Beam Number	Age (days)	P* (lbs.)	Modulus of Rupture (psi)	Compressive** Strength (psi)
1	60	5275	439.6	4755
2	60	4620	385.0	3647
3	60	4550	379.2	3538
4	60	5010	417.5	4289
5	155	5560	463.3	5282
6	155	5800	483.3	5748
7	155	5170	430.8	4567
8	155	5270	439.2	4746

*Total concentrated load applied

**Calculated on basis of modulus of rupture test results

were to be three types tested: the first having no w.w.f., the second having one layer of w.w.f., and the third having two layers of w.w.f. The eight inch deep specimens would either have no w.w.f. or two layers staggered. To help improve the consistency of data, two of each type lintel mentioned above were fabricated for testing. This amounted to a total of sixteen lintels to be used in evaluating the mesh performance as web reinforcement.

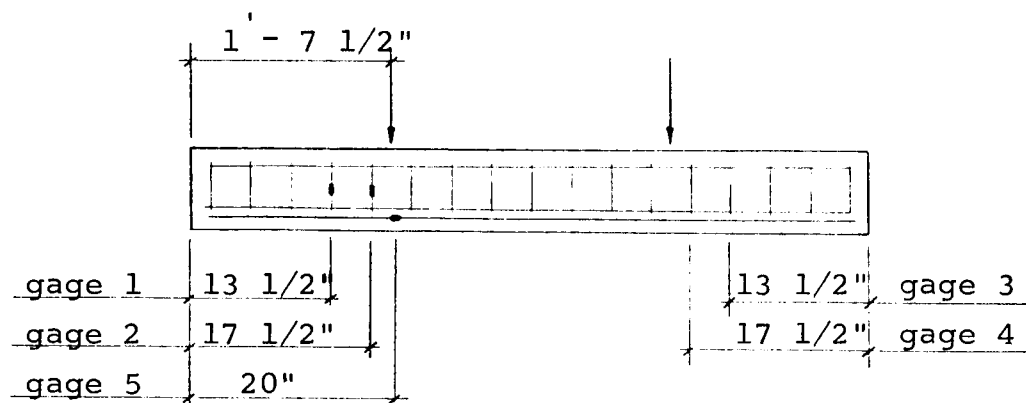
In order to make the discussion about any particular lintel as uncomplicated as possible, each is given a descriptive designation. This designation gives lintel depth, number of layers of w.w.f. used, and whether it is the first or second of that particular type tested. For example, the designation 2L12-0 indicates this is the second, twelve inch deep lintel with no w.w.f. Also, the lintels were divided into two groups for testing, with nine in the first group and seven in the second. The reason for this separation is given in the following discussion.

Of the nine lintels in the first series, six of those were fabricated without w.w.f. This left the majority of those with web reinforcement in the second group. Of the remaining three in the first group, two of those (1L12-1 and 1L16-1) were discarded due to production error. These two lintels were then scheduled to be remade after the first tests and allowed a minimum of twenty-eight days to cure. Since these two lintels became the last to be tested, some

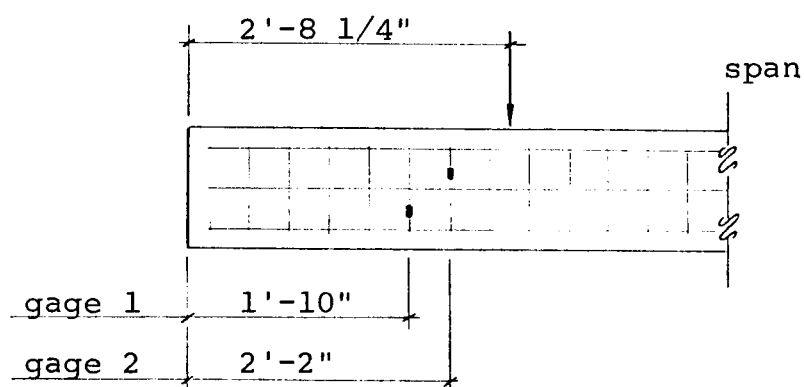
special support conditions were tried. Because of this, the results for these remakes are not included with the rest of the data. A discussion of their outcome will be included at the end of this chapter. Due to having these remakes, there were actually only seven lintels in group one. All of these were tested after sixty days of open-air curing.

The second series of lintels were prepared to have strain measurements taken in the w.w.f. and main flexural reinforcement using electrical resistance strain gages. For that reason, much more preparation time was required before the lintels could be fabricated. There was a total of fifteen foil gages attached to the reinforcement in five of the seven lintels. These five lintels and gage locations are shown in Figure 6. Strain readings were taken at each load increment using a Budd strain indicator; this resulted in tests requiring more time and attention than those of the first series. The final results for both series of tests have been placed in Table IV along with the predicted ultimate load from design.

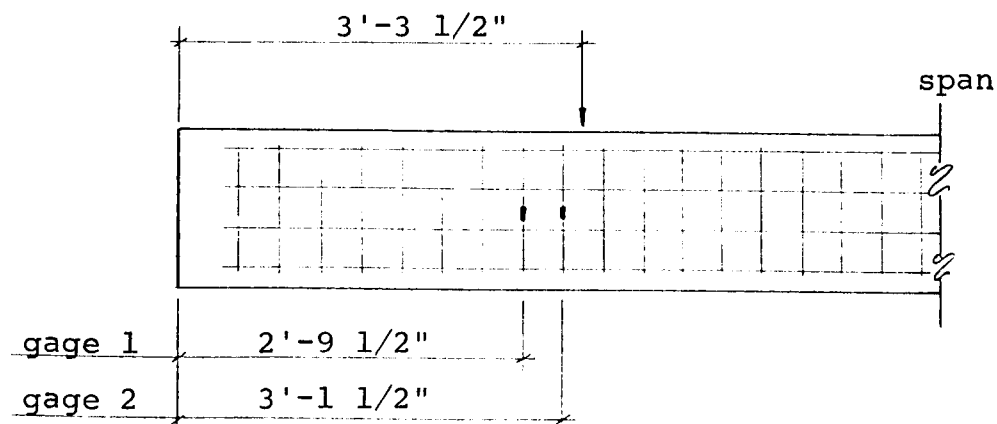
In all the load tests midspan deflections were recorded at each load increment. These data have been compiled and placed in Appendix A for reference. From these data, load deflection curves have been plotted in Figures 7 through 9. These curves should demonstrate the flexural behavior of each lintel as compared to the others of the same cross section. Also, a comparison of the actual deflections with



(a) Gage locations for lintel 2L8-2

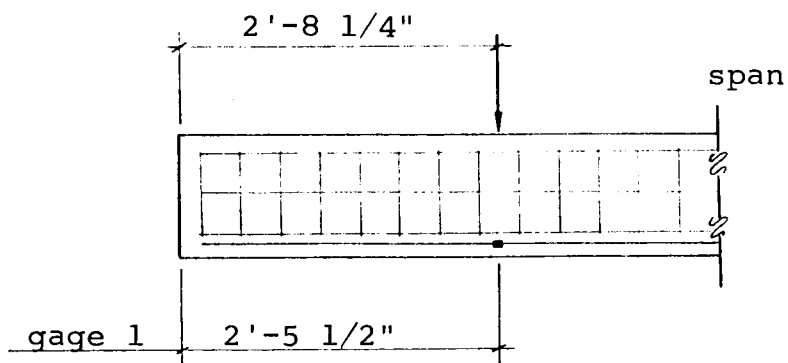


(b) Gage locations (symmetric) for lintel 2L12-1

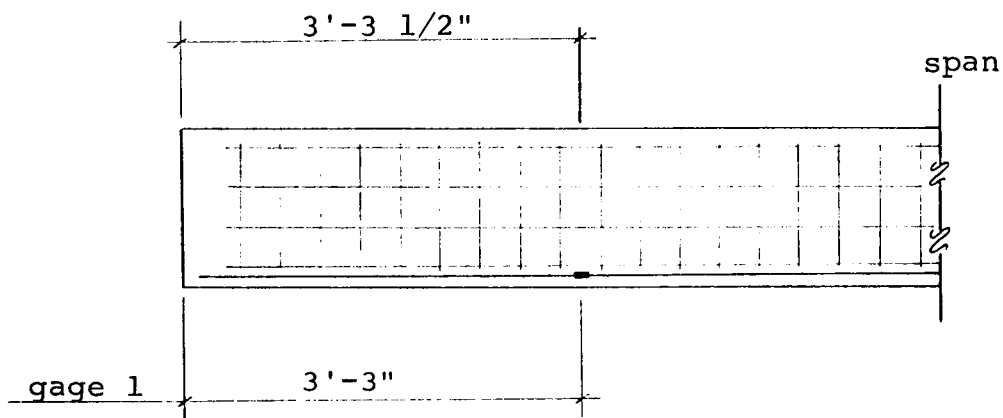


(c) Gage locations (symmetric) for lintel 2L16-1

Figure 6: Electrical resistance strain gage locations



(d) Gage location for lintel 2L12-2



(e) Gage location for lintel 2L16-2

TABLE IV
RESULTS FROM LINTEL SHEAR TESTS

Lintel Number	V_c (lbs.)	V_s (lbs.)	V_n^* (lbs.)	V_u^{**} (lbs.)	v_u (p.s.i.)
1L8-0	3,886	--	3,886	7,400	176.5
2L8-0	3,886	--	3,886	5,500	131.1
1L8-2	3,886	6,600	10,486	11,450	--
2L8-2	3,886	6,600	10,486	11,500	--
1L12-0	7,065	--	7,065	14,300	187.5
2L12-0	7,065	--	7,065	12,300	161.3
2L12-1	7,065	6,000	13,065	10,500	--
1L12-2	7,065	12,000	19,065	11,250	--
2L12-2	7,065	12,000	19,065	13,500	--
1L16-0	9,537	--	9,537	13,400	130.2
2L16-0	9,537	--	9,537	14,100	156.4
2L16-1	9,536	8,100	17,637	14,250	--
1L16-2	9,537	16,200	25,737	14,250	--

$$*V_n = V_c + V_s$$

$**V_u$ is actual failure load

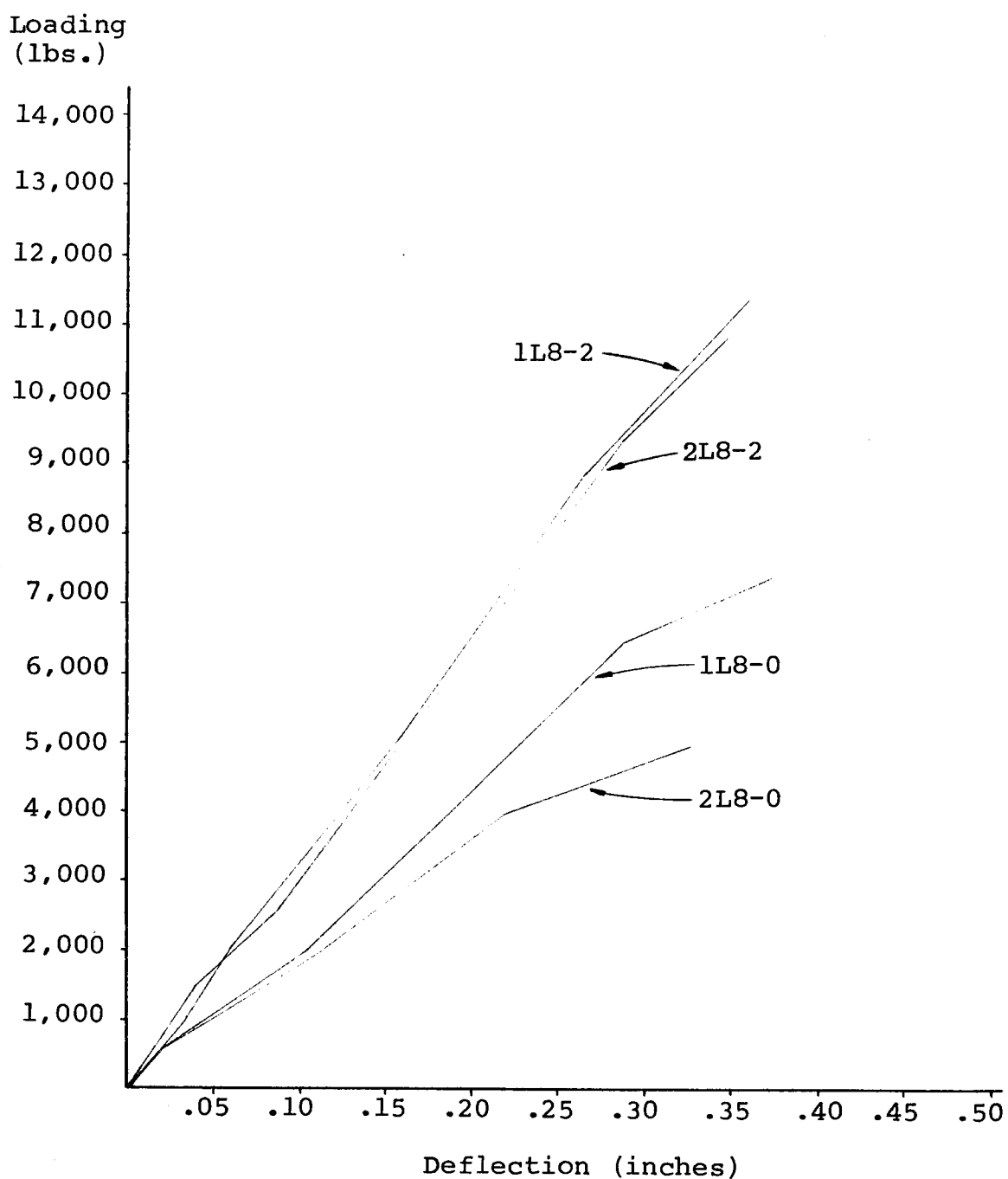


Figure 7: Load deflection curve for 8" x 8" lintels

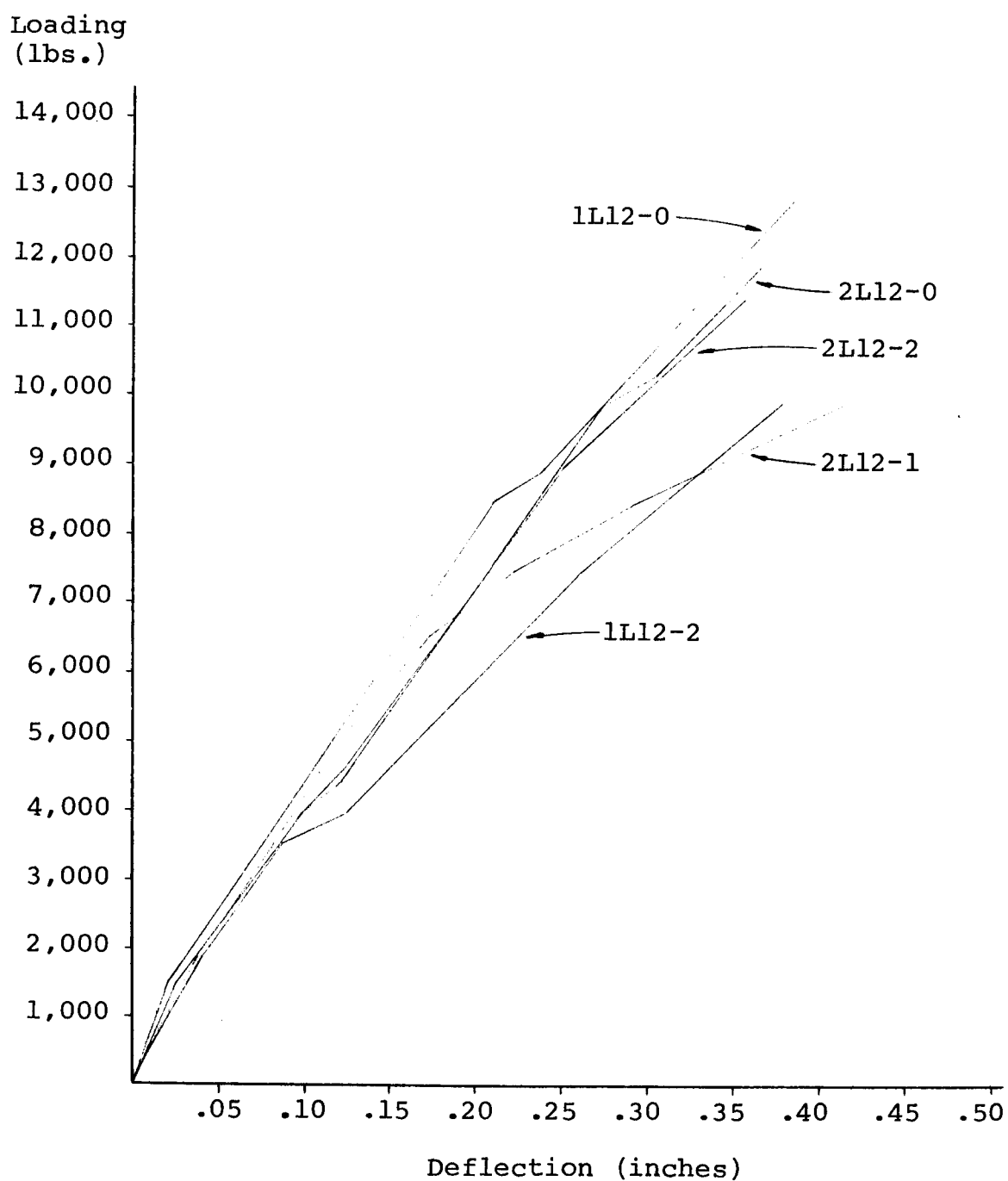


Figure 8: Load deflection curve for 8" x 12" lintels

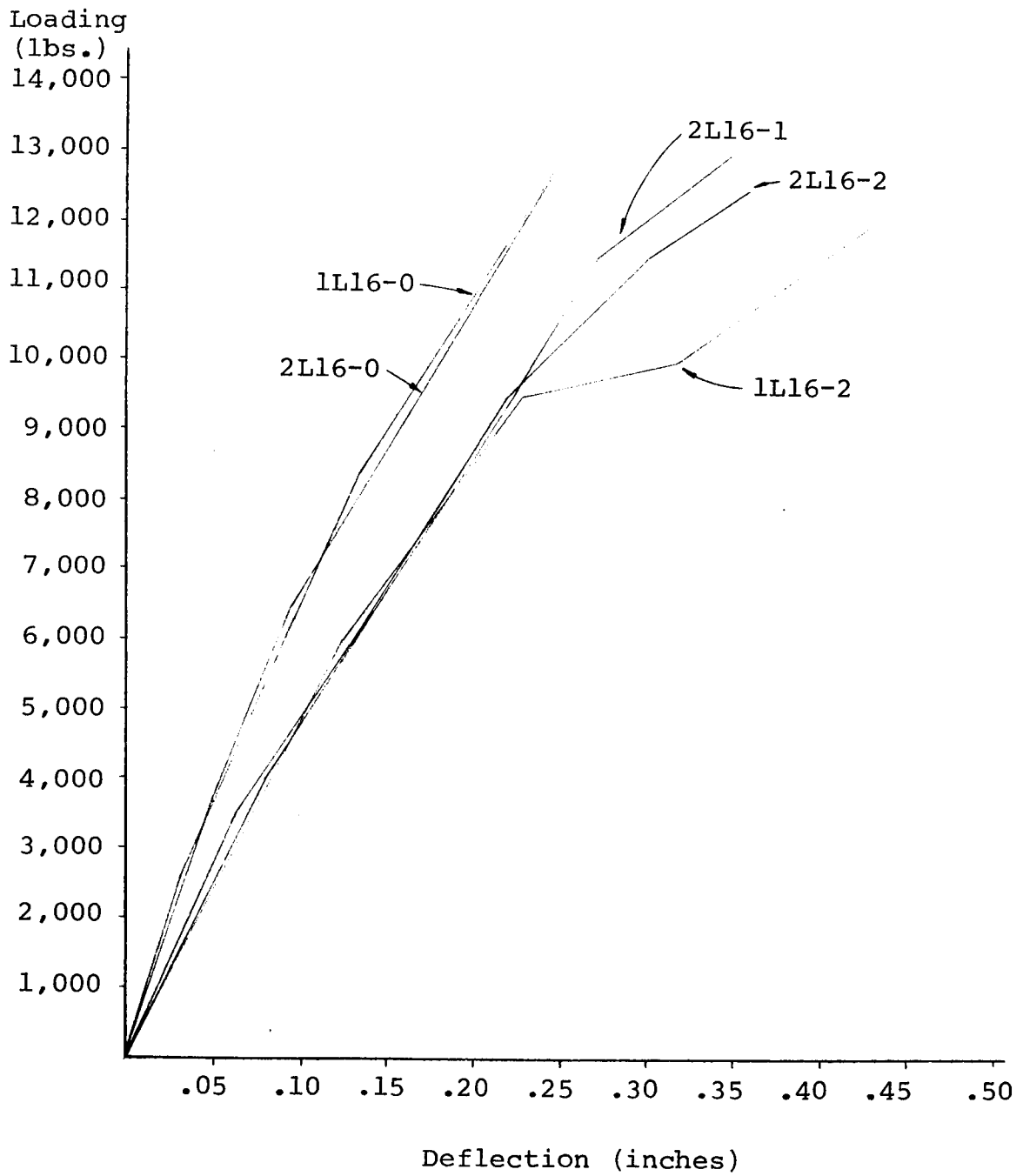


Figure 9: Load deflection curve for 8" x 16" lintels

the calculated deflections is given in Table V. These deflections were calculated using the gross section properties up to the cracking load and the transformed section as prescribed in the A.C.I. 318-77 code beyond that load. The cracking load " P_{cr} " which is also given in Table V was calculated using the average value for modulus of rupture obtained in the small beam tests discussed earlier.

Of all the members tested the eight inch deep sections with web reinforcement displayed the most favorable results. These were the only lintels tested that went to a failure load equal to or greater than the calculated ultimate capacity. After these tests, inspection of the w.w.f. showed no yielding of the vertical wires, yet a considerable influence on the load carrying capacity had been experienced. Unfortunately the strain gages in 2L8-2 apparently did not function properly because their results showed very little stress increase in the steel. The other eight inch lintels without w.w.f. exhibited very poor load deflection characteristics due to poor bond development. They failed finally in flexural bond, but still carried shear loads in excess of what is allowed in the A.C.I. code.

As can be seen in Figures 8 and 9, the twelve and sixteen inch lintels acted quite differently from the eight inch members. Those sections without w.w.f. were as stiff or stiffer than the sections with w.w.f. which is just the opposite of how the eight inch lintels performed. The

TABLE V
COMPUTED VS. MEASURED MIDSPAN DEFLECTIONS

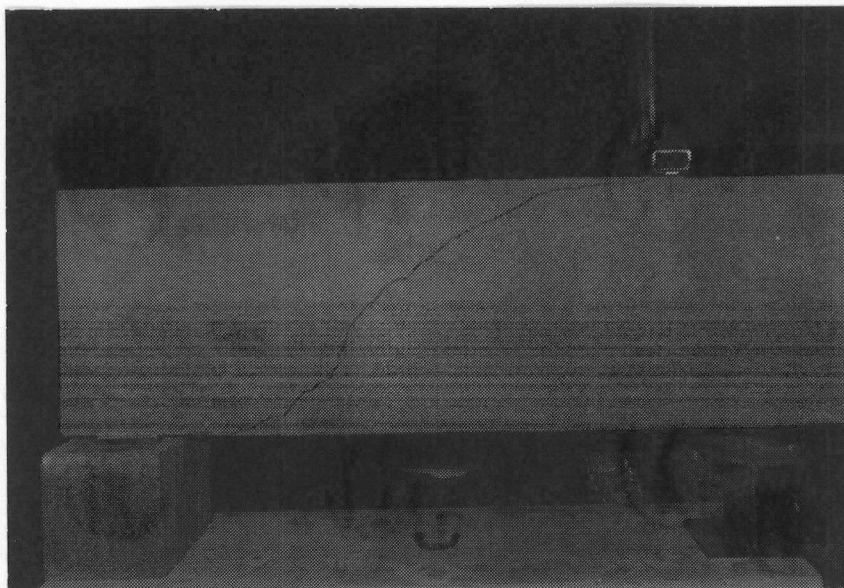
Lintel	E_c 10 ⁶ psi	I_g in. ⁴	P_{cr} lbs.	Δ_c^* in.	Δ_m^{**} in.	$P_{applied}$ lbs.	I_e in. ⁴	Δ_c^* in.	Δ_m^{**} in.
1L8-0	2.4	282	1,966	.014	.102	5,000	118	.103	.221
2L8-0	2.4	282	1,966	.014	.116	5,000	118	.103	.334
1L802	2.4	282	1,966	.014	.061	11,000	109	.245	.352
2L8-2	2.4	282	1,966	.014	.058	11,000	109	.245	.333
1L12-0	2.4	998	2,626	.031	.051	10,000	679	.200	0.271
2L12-0	2.4	998	2,626	.031	.056	10,000	679	.200	0.272
2L12-1	2.4	998	2,626	.031	.050	10,000	679	.200	0.416
1L12-2	2.4	998	2,626	.031	.058	10,000	679	.200	0.380
2L12-2	2.4	998	2,626	.031	.061	10,000	679	.200	0.277
1L16-0	2.4	2,424	3,567	.043	.046	13,000	1,648	.258	0.247
2L16-0	2.4	2,424	3,567	.043	.047	13,000	1,648	.258	0.252
2L16-1	2.4	2,424	3,567	.043	.062	13,000	1,648	.258	0.348
1L16-2	2.4	2,424	3,567	.043	.069	13,000	1,648	.258	0.501
2L16-2	2.4	2,424	3,567	.043	.070	13,000	1,648	.258	0.488

*Computer deflection at given load

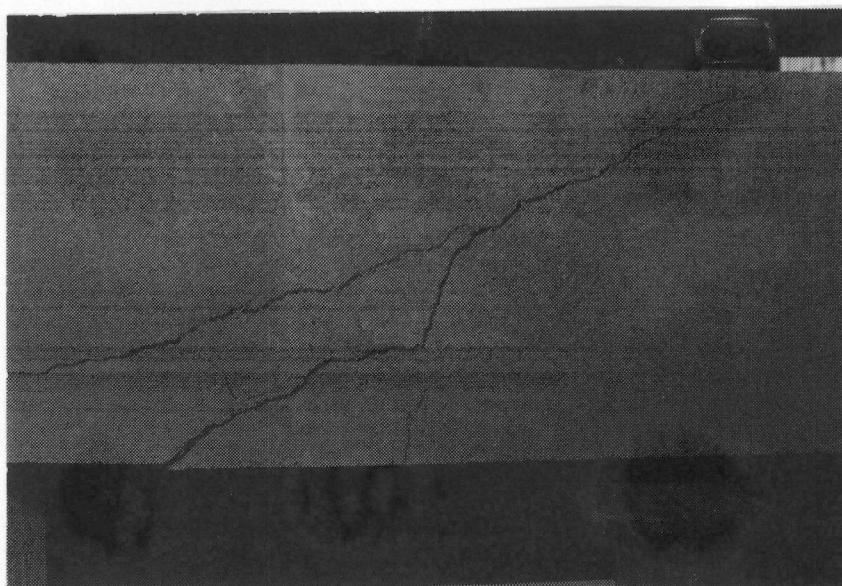
**Measured deflection at given load

sixteen inch sections without w.w.f., in particular, gave deflections almost identical to the calculated values. These members showed no sign of distress during loading and failed suddenly without warning. The members with web reinforcement developed visible shear cracks at loads well below failure. These initial shear cracks originated as extensions of flexural cracks inclined at approximately 45 degrees toward the load. As the load increased the shear cracks penetrated deeper into the compression zone at a very flat angle, and small splitting cracks along the flexural reinforcement became visible. The contrast in these two types of shear failure is shown in Figure 10. At the maximum load the sections with w.w.f. failed in either shear or bond. In most of the cases, as is seen in Figure 11, the concrete splitting along the flexural reinforcement caused a final bond failure.

Attempts of monitoring reinforcement strains in the twelve and sixteen inch lintels proved successful. Three of the four lintels with gages (2L12-1, 2L12-2, and 2L16-2) gave good results. It is felt that the fourth lintel (2L16-1) had poor results due to an error in the gage locations. As can be seen in Figure 6c, page 29, those gages would completely miss a probable diagonal tension crack. Strain measurements from the first three lintels have been converted into corresponding stresses and placed in Appendix B. The results for the mesh in lintel 2L12-1 indicated that a tensile

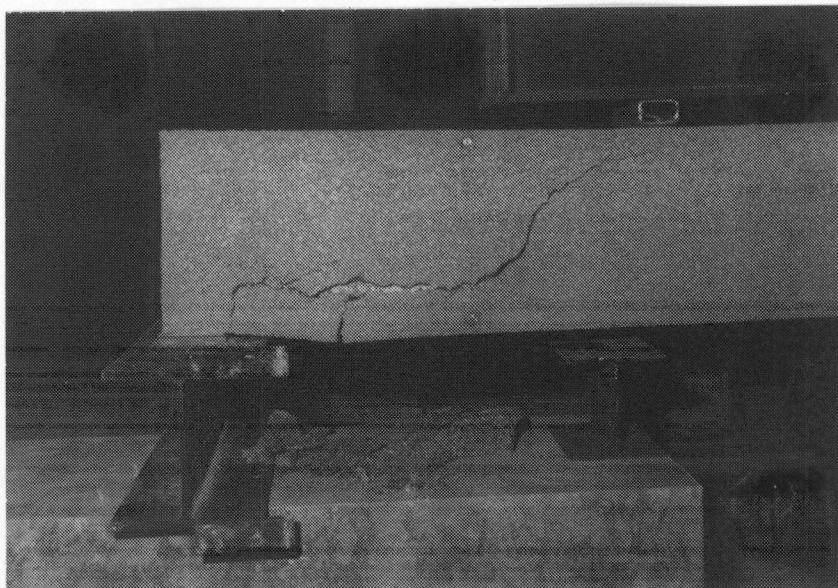


(a) Shear failure of 16" deep lintel without w.w.f.

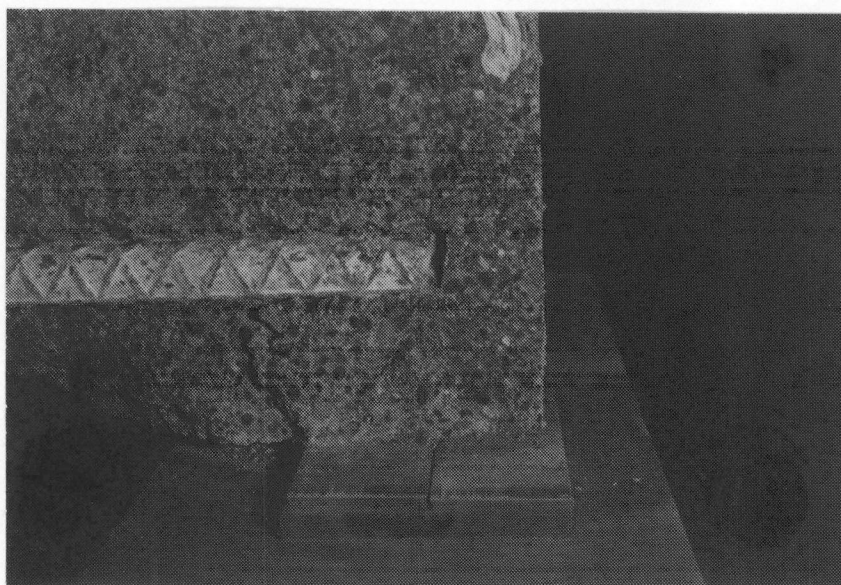


(b) Shear failure of 12" deep lintel with w.w.f.

Figure 10: Typical shear failures



(a) Horizontal splitting at reinforcing caused final bond failure



(b) Section through the lintel at the support showing bar slip

Figure 11: Typical shear-bond failures

stress as high as 55,680 p.s.i. was reached at failure. This showed that the mesh is capable of developing stresses near yield across the shear crack, but no actual increased load carrying capacity was experienced. The other twelve and sixteen inch lintels also showed no increased capacity with the addition of w.w.f.

In all the load tests mentioned so far, there was six inches allowed at each end for bearing. This is considerably less than what is used in actual field applications. The bearing length provided for sixteen inch lintels is usually sixteen inches and twelve inches for the twelve inch lintels. These lengths are required to keep the supporting masonry block walls at their allowable bearing stresses. For the testing of the remade lintels 1L12-1 and 1L16-1, these bearing distances just mentioned were used. The loads were still placed at $2.5d$ from the support, but the load location within the span was no longer at the quarter points. The reason for using this bearing detail was to see if having the main flexural steel developed beyond the edge of the support would increase the load carrying capacity. So many of the failures earlier appeared to be bond oriented after horizontal splitting that it prompted these changes. The failure in both these cases was observed to have occurred at horizontal splitting, but the main reinforcement did not lose bond beyond the support. The w.w.f.

in lintel 1L16-1 did yield at failure, but again no increased load carrying was experienced in either member.

CHAPTER V

DICUSSION

A great deal of research has been done on how concrete beams carry shear, with most of that work being done on beams without web reinforcement. In trying to understand the results from these sixteen lintels, it is important to first consider how shear is carried in a concrete beam. The approach to this understanding will begin by looking at the phases a concrete beam without web reinforcement goes through under an applied shear load.

I. BEAMS WITHOUT SHEAR STEEL

Shear stress on an uncracked concrete cross section is considered to have a parabolic distribution with a maximum value at the mid-depth. This shear stress distribution changes once flexural cracking begins which, in beams of common depth to span ratios, precedes any diagonal tension cracking (1). Once diagonal tension cracks form, the concept of a shear stress no longer has any real validity even though design ultimate shear strength is given as an allowable stress times the beam cross sectional area ($b_w d$). Formation of these diagonal tension cracks is not necessarily the point of failure even though the remaining uncracked

portion of the beam may be reduced to as much as one-fourth of its original depth. It has been proven that the shear strength after diagonal cracking is made up of three components: the shear strength of the compression zone, the dowel action of the flexural reinforcement, and aggregate interlocking forces at the rough interface of the shear crack (6, 7, 8, 9). These three components make up the shear strength in varying proportions, but for a normal weight concrete are in the ranges of (6):

compression zone	20-40%
dowel action	15-25%
aggregate interlock	33-50%

Aggregate interlock forces which represent a large percentage of the shear resistance are made possible by the tensile cracks occurring in the cement paste. This leaves projecting aggregate in the crack interface which resist lateral displacement along the crack. For concretes with lightweight aggregate this is not the case because tensile failures occur through both paste and aggregate which leaves a much smoother crack interface. In normal weight and lightweight concretes, the final diagonal tension failure occurs with a horizontal splitting at the level of the flexural steel (5). This horizontal splitting represents the loss of concrete tensile strength supporting the dowel action of the main reinforcement. With a loss of dowel action a progressive failure of the other two components follows. In some cases extensive

dowel splitting will cause bond failure prior to shear failure. These two types of failure can sometimes be hard to distinguish.

II. BEAMS WITH SHEAR STEEL

The next area of consideration is the design concepts involved with using web reinforcement to increase a beam's ultimate shear capacity. The nominal shear strength of a concrete beam with web reinforcement, as was given in Chapter III, is considered as the sum of the contributions of the concrete and shear steel. It is known that concrete can carry shear after web cracking, as was just discussed, but the presence of web reinforcement in the form of stirrups changes the relationships between dowel action, aggregate interlock, and the compression zone. Stirrups enclose the main flexural reinforcement and develop tensile forces by preventing further dowel displacements after initial horizontal splits occur. At ultimate load the contribution of dowel action is negligible due to support from the stirrups, so the concrete shear strength is derived from the remaining aggregate interlock and compression zone. These two remaining components could not exist at this stage of loading were it not for the stirrups. There is a interdependence between " V_c " and " V_s " which cannot really be seen by using the nominal shear strength equation given in the A.C.I. 318-77. The presence of stirrups up to their yield prevents significant

vertical separation of the diagonal tension crack which is critical for aggregate interlock. Also, stirrups support the main flexural reinforcement and inhibit progressive dowel splitting, which in turn reduces the possibility of bond slippage (7). By keeping the flexural steel fully bonded there are no excessive rotations of the cracked section which would also have a tendency to reduce the amount of aggregate interlock.

III. TEST RESULTS

There are two important points from the preceding sections which should be re-emphasized before continuing this evaluation. The first is that lightweight concretes tend to have a low percentage of shear carried by aggregate interlock, and second, there is an interdependent relationship between concrete shear strength and web reinforcement at the calculated ultimate load. Both of these play a part in explaining the overall poor performance of the w.w.f. as a shear reinforcement.

It has been established in work done by Acharya and Kemp (5) that a maximum of 40 percent of the concrete shear strength in beams can be contributed to the compression zone at ultimate load. With the aggregate interlock contribution reduced, a direct increase in the percentage of shear carried in dowel action must be realized. Thus, for lightweight aggregate concrete without web reinforcement, the percentage

of shear strength lost at dowel splitting is no longer 15 to 25 percent, but more on the order of 50 percent. This increase loss of shear carrying capacity after diagonal tensile cracking simply had the effect of reducing the nominal shear strength in the cases of lintels without w.w.f. Results from Hanson's (3) testing in lightweight concrete beams with the same shear span and approximate steel percentage showed an average shear stress at failure of 140 p.s.i. which compares quite well with the calculated stresses for these tests given in Table IV, page 31.

This increased loss of concrete shear strength at dowel splitting was more significant for lintels with w.w.f. The term " V_c " in the nominal shear strength equation would be made up almost entirely by the shear capacity of the remaining compression zone. The lack of aggregate interlock would represent a sizeable reduction which should be considered in design. Another difficulty was that after loss of dowel action there were no stirrups to inhibit progressive horizontal splitting, which led to eventual bond loss. It was observed in several of the lintels after testing that bond slippage of the main reinforcement caused the lower chords of the w.w.f. to actually yield by functioning in flexure. Failure in most of the members with w.w.f. should be attributed to flexural bond loss since no yielding of vertical wires was found in the initial eight lintels tested. For these cases the failure load would have to be given as

that which caused dowel splitting. This definition of failure load is further supported by the fact that the lintels with w.w.f. failed at loads comparable to those members without w.w.f. In the case of the remade 1L16-1, vertical mesh yielding did indicate a shear failure. This failure occurred immediately after dowel splitting at a load of 14,200 pounds for which the calculated ultimate should have been 17,637 pounds. If the concrete contribution " V_c " which was calculated to be 9,536 pounds, had been reduced by say 40 percent, then the predicted ultimate would have been quite close to the actual failure load.

Generally the load-deflection behavior of all the lintels except 1L16-0 and 2L16-0 were very poor. This problem seems to be rooted in inadequate bond between the flexural steel and the tamped concrete. Improper coating of the reinforcing with cement paste could have been the cause of this problem.

CHAPTER VI

CONCLUSION

The outcome of this test series has proven to be very informative; unfortunately, the results have not given a complete answer to the initial question of the performance of w.w.f. as a web reinforcement. All of the tests, except in a few cases, were plagued with bond imperfections as reflected in poor load deflection behaviors and progressive dowel splitting which led to bond failures. These kinds of complicating elements have to make one skeptical of the results. This is exemplified in the cases of the twelve and sixteen inch lintels that actually lost shear capacity by having w.w.f. added to their cross section.

Shear strengths obtained in cross sections without w.w.f. ran much higher than design allowables given in the A.C.I. building code and compared quite well with prior tests conducted by Hanson (3). The concrete shear strengths for the lintels with w.w.f. could not have been as high. Results from the strain measurements in the lintel 2L12-1 indicated that the mesh was capable of carrying some of the shear across the cracked section. Since failure still occurred right after dowel splitting and at approximately the same load as those without w.w.f., it was apparent that there was less

shear carried by the concrete. It is felt that the key reason for not obtaining those high concrete shear strengths can be attributed to flaws related to inadequate consolidation during production. These flaws were apparently introduced by having the w.w.f. placed in the lintel cross section.

As a direct result of these tests, many questions have been raised about production methods that were assumed to be adequate. The two major concerns, as just mentioned, are the bonding characteristics of deformed reinforcing steel in dry tamped concrete and the extent of imperfections caused during the compaction process. These two concerns could very well be interrelated. Another important point to consider is that w.w.f. does not inhibit progressive dowel splitting. To ever achieve design shear strengths full bonding of the flexural reinforcement must be insured beyond the support.

Due to the apparent negligible effect of vertical layers of w.w.f. as shear reinforcement in concrete lintels, it is recommended that future design be based entirely on the allowable concrete shear strength given by the A.C.I. 318-77 building code. Also, more testing should be conducted to explore more fully the results of the remade lintel 1L16-1. This could possibly be the answer to obtaining increased shear capacities for precast concrete lintels.

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LIST OF REFERENCES

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APPENDIXES

APPENDIX A

TABLE VI
LOAD DEFLECTION DATA FOR LINTEL 1L8-0

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.012	.000	.012
2,000	.043	.000	.043
3,000	.068	.000	.068
4,000	.105	.001	.104
5,000	.124	.002	.122
6,000	.143	.003	.140
7,000	.165	.003	.162
8,000	.187	.003	.184
9,000	.207	.004	.203
10,000	.226	.005	.221
11,000	.247	.005	.242
12,000	.270	.006	.264
13,000	.294	.007	.287
14,000	.327	.008	.319
14,800	.380	.009	.371

TABLE VII
LOAD DEFLECTION DATA FOR LINTEL 2L8-0

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.009	.000	.009
2,000	.043	.001	.042
3,000	.079	.001	.078
4,000	.120	.002	.118
5,000	.143	.003	.140
6,000	.176	.004	.172
7,000	.197	.004	.193
8,000	.219	.005	.214
9,000	.291	.005	.286
10,000	.340	.006	.334
11,000	.486	.008	.478

TABLE VIII
LOAD DEFLECTION DATA FOR LINTEL 1L8-2

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.011	.000	.011
2,000	.023	.000	.023
3,000	.040	.001	.039
4,000	.064	.001	.063
5,000	.083	.001	.082
6,000	.100	.002	.098
7,000	.115	.003	.112
8,000	.129	.003	.126
9,000	.146	.004	.142
10,000	.160	.005	.155
11,000	.176	.006	.170
12,000	.190	.006	.184
13,000	.206	.007	.199
14,000	.226	.008	.218
15,000	.239	.009	.230
16,000	.255	.010	.245
17,000	.271	.011	.260
18,000	.287	.012	.275
19,000	.303	.013	.290
20,000	.323	.014	.309
21,000	.340	.015	.325
22,000	.369	.017	.352
22,900	---	---	---

TABLE IX
LOAD DEFLECTION DATA FOR LINTEL 2L8-2

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.020	.000	.020
2,000	.036	.001	.035
3,000	.049	.001	.048
4,000	.061	.002	.059
5,000	.079	.002	.077
6,000	.094	.003	.091
7,000	.112	.003	.109
8,000	.128	.004	.124
9,000	.145	.005	.140
10,000	.160	.005	.155
11,000	.176	.006	.170
12,000	.190	.007	.183
13,000	.204	.008	.196
14,000	.220	.009	.211
15,000	.235	.010	.225
16,000	.250	.012	.238
17,000	.265	.013	.252
18,000	.281	.014	.267
19,000	.297	.015	.282
20,000	.315	.017	.298
21,000	.335	.019	.316
22,000	.354	.021	.333
23,000	.382	.024	.358

TABLE X
LOAD DEFLECTION DATA FOR LINTEL 1L12-0

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.007	.001	.006
2,000	.016	.002	.014
3,000	.023	.003	.020
4,000	.037	.004	.033
5,000	.025	.005	.047
6,000	.065	.007	.058
7,000	.080	.008	.072
8,000	.095	.009	.086
9,000	.111	.011	.100
10,000	.125	.012	.113
11,000	.139	.014	.125
12,000	.157	.015	.142
13,000	.173	.018	.155
14,000	.187	.020	.167
15,000	.202	.022	.180
16,000	.217	.025	.192
17,000	.233	.027	.206
18,000	.265	.031	.234
19,000	.286	.034	.252
20,000	.308	.037	.271
21,000	.330	.040	.290
22,000	.350	.042	.308
23,000	.376	.046	.330
24,000	.389	.048	.341
25,000	.410	.051	.359
26,000	.434	.053	.381
27,000	.460	.056	.404
28,000	.489	.058	.431
28,600	.525	---	---

TABLE XI
LOAD DEFLECTION DATA FOR LINTEL 2L12-0

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.009	.001	.008
2,000	.018	.002	.016
3,000	.029	.003	.026
4,000	.043	.004	.039
5,000	.058	.005	.053
6,000	.070	.006	.064
7,000	.087	.007	.080
8,000	.104	.008	.096
9,000	.127	.009	.118
10,000	.143	.010	.133
11,000	.160	.012	.148
12,000	.175	.013	.162
13,000	.190	.015	.175
14,000	.206	.016	.190
15,000	.222	.018	.204
16,000	.237	.020	.217
17,000	.253	.022	.231
18,000	.268	.023	.245
19,000	.285	.025	.260
20,000	.300	.028	.272
21,000	.333	.030	.303
22,000	.353	.033	.320
23,000	.380	.035	.345
24,000	.400	.037	.363
24,600	---	---	---

TABLE XII
LOAD DEFLECTION DATA FOR LINTEL 2L12-1

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.010	.001	.009
2,000	.021	.002	.019
3,000	.031	.003	.028
4,000	.041	.004	.037
5,000	.053	.006	.047
6,000	.067	.007	.060
7,000	.084	.009	.075
8,000	.100	.010	.090
9,000	.117	.012	.105
10,000	.133	.014	.119
11,000	.155	.016	.139
12,000	.175	.019	.156
13,000	.194	.022	.172
14,000	.225	.025	.200
15,000	.252	.028	.224
16,000	.286	.032	.254
17,000	.325	.035	.290
18,000	.360	.039	.321
19,000	.408	.043	.365
20,000	.464	.048	.416
21,000	.550	---	---

TABLE XIII
LOAD DEFLECTION DATA FOR LINTEL 1L12-2

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.010	.000	.010
2,000	.023	.001	.022
3,000	.033	.002	.031
4,000	.044	.003	.041
5,000	.059	.004	.055
6,000	.073	.005	.068
7,000	.093	.007	.086
8,000	.136	.009	.127
9,000	.158	.011	.147
10,000	.178	.012	.166
11,000	.201	.014	.187
12,000	.221	.016	.205
13,000	.240	.018	.222
14,000	.260	.020	.240
15,000	.280	.022	.258
16,000	.305	.024	.281
17,000	.333 .	.027	.306
18,000	.360	.030	.330
19,000	.390	.033	.357
20,000	.415	.035	.380
21,000	.483	.039	.444
22,000	.576	.043	.533
22,500	.605	---	---

TABLE XIV
LOAD DEFLECTION DATA FOR LINTEL 2L12-2

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.011	.001	.010
2,000	.023	.002	.021
3,000	.034	.003	.031
4,000	.048	.004	.044
5,000	.063	.005	.058
6,000	.078	.006	.072
7,000	.093	.007	.086
8,000	.108	.009	.099
9,000	.128	.010	.118
10,000	.144	.012	.132
11,000	.160	.014	.146
12,000	.176	.016	.160
13,000	.191	.018	.173
14,000	.208	.020	.188
15,000	.225	.023	.202
16,000	.242	.026	.216
17,000	.258	.029	.229
18,000	.275	.032	.243
19,000	.293	.034	.259
20,000	.314	.037	.277
21,000	.332	.040	.292
22,000	.367	.044	.323
23,000	.392	.048	.344
24,000	.316	.051	.363
25,000	.441	.054	.387
26,000	.470	.058	.412
27,000	.498	.061	.467

TABLE XV
LOAD DEFLECTION DATA FOR LINTEL 1L16-0

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.005	.000	.005
2,000	.012	.001	.011
3,000	.019	.002	.017
4,000	.027	.004	.023
5,000	.035	.005	.030
6,000	.046	.007	.039
7,000	.055	.009	.046
8,000	.064	.010	.054
9,000	.075	.012	.063
10,000	.085	.013	.072
11,000	.095	.015	.080
12,000	.105	.017	.088
13,000	.116	.020	.096
14,000	.129	.022	.107
15,000	.141	.024	.117
16,000	.155	.027	.128
17,000	.170	.029	.141
18,000	.183	.031	.152
19,000	.198	.034	.164
20,000	.212	.036	.176
21,000	.224	.039	.185
22,000	.239	.041	.198
23,000	.253	.044	.209
24,000	.265	.046	.219
25,000	.284	.048	.236
26,000	.297	.050	.247
26,800	.320	---	---

TABLE XVI
LOAD DEFLECTION DATA FOR LINTEL 2L16-0

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.007	.002	.005
2,000	.015	.003	.012
3,000	.023	.005	.018
4,000	.030	.006	.024
5,000	.037	.007	.030
6,000	.046	.009	.037
7,000	.057	.010	.047
8,000	.066	.012	.054
9,000	.078	.014	.064
10,000	.088	.016	.072
11,000	.098	.018	.080
12,000	.111	.020	.091
13,000	.125	.022	.103
14,000	.136	.025	.111
15,000	.150	.028	.122
16,000	.163	.030	.133
17,000	.176	.033	.143
18,000	.190	.035	.155
19,000	.208	.038	.170
20,000	.219	.040	.179
21,000	.233	.043	.190
22,000	.247	.045	.202
23,000	.266	.048	.218
24,000	.276	.050	.226
25,000	.290	.052	.238
26,000	.306	.054	.252
26,900	.320	---	---
28,200	---	---	---

TABLE XVII
LOAD DEFLECTION DATA FOR LINTEL 2L16-1

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in
1,000	.007	.000	.007
2,000	.018	.001	.017
3,000	.029	.003	.026
4,000	.038	.005	.033
5,000	.049	.006	.043
6,000	.059	.008	.051
7,000	.072	.010	.062
8,000	.086	.012	.074
9,000	.100	.014	.086
10,000	.115	.016	.099
11,000	.131	.019	.112
12,000	.148	.022	.126
13,000	.164	.025	.139
14,000	.182	.028	.154
15,000	.196	.031	.165
16,000	.215	.034	.181
17,000	.232	.037	.195
18,000	.248	.041	.207
19,000	.263	.045	.218
20,000	.279	.048	.231
21,000	.295	.051	.244
22,000	.310	.054	.256
23,000	.329	.058	.271
24,000	.366	.062	.304
25,000	.394	.066	.328
26,000	.418	.070	.348
27,000	.445	.074	.371
28,000	.474	.077	.397
28,500	.605	---	---

TABLE XVIII
LOAD DEFLECTION DATA FOR LINTEL 1L16-2

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.009	.001	.008
2,000	.020	.002	.018
3,000	.031	.003	.028
4,000	.043	.005	.038
5,000	.056	.007	.049
6,000	.067	.008	.059
7,000	.079	.010	.069
8,000	.091	.012	.079
9,000	.194	.014	.090
10,000	.118	.017	.101
11,000	.133	.020	.113
12,000	.148	.023	.125
13,000	.168	.026	.142
14,000	.185	.029	.156
15,000	.204	.033	.171
16,000	.222	.036	.186
17,000	.239	.039	.200
18,000	.256	.043	.213
19,000	.276	.046	.230
20,000	.375	.055	.320
21,000	.398	.058	.340
22,000	.434	.062	.372
23,000	.464	.066	.398
24,000	.493	.070	.423
25,000	.536	.074	.462
26,000	.579	.078	.501
27,000	.633	.081	.552
28,000	.685	.084	.601
28,500	.800	---	---

TABLE XIX
LOAD DEFLECTION DATA FOR LINTEL 2L16-2

Total Load lbs.	Δ_1 in.	Δ_2 in.	$\Delta_1 - \Delta_2$ in.
1,000	.009	.001	.008
2,000	.021	.003	.018
3,000	.032	.004	.028
4,000	.044	.005	.039
5,000	.056	.006	.050
6,000	.068	.008	.060
7,000	.080	.010	.070
8,000	.094	.012	.082
9,000	.108	.014	.094
10,000	.124	.016	.108
11,000	.140	.020	.120
12,000	.157	.023	.134
13,000	.174	.026	.148
14,000	.189	.030	.159
15,000	.205	.033	.172
16,000	.220	.036	.184
17,000	.238	.040	.198
18,000	.254	.043	.211
19,000	.272	.046	.226
20,000	.295	.050	.245
21,000	.318	.054	.264
22,000	.337	.058	.279
23,000	.362	.061	.301
24,000	.399	.065	.334
25,000	.426	.069	.357
26,000	.564	.076	.488
27,000	.639	.081	.558
27,900	1.090	.087	1.003

APPENDIX B

TABLE XX
STRESSES FROM LOAD TEST ON LINTEL 2L12-1

Total Load lbs.	Gage 1 p.s.i.	Gage 2 p.s.i.	Gage 3 p.s.i.	Gage 4 p.s.i.
1,000	---	---	---	---
2,000	---	---	---	---
3,000	---	---	---	---
4,000	---	---	---	---
5,000	---	---	---	---
6,000	---	---	---	---
7,000	---	---	---	---
8,000	---	---	---	---
9,000	---	---	---	---
10,000	---	---	---	---
11,000	---	---	---	4,930
12,000	---	870	---	6,670
13,000	---	2,320	580	11,310
14,000	580	5,800	1,160	20,010
15,000	11,890	8,410	6,380	25,520
16,000	20,590	9,280	8,700	28,420
17,000	29,290	8,120	11,020	29,580
18,000	37,990	4,640	12,760	29,870
19,000	46,400	3,480	14,210	30,450
20,000	53,650	1,740	16,530	33,350
21,000	55,680	---	16,820	---

TABLE XXI
STRESSES FOR LOAD TESTS ON LINTELS
2L12-2 AND 2L16-2

Total Load lbs.	2L12-2 p.s.i.	2L16-2 p.s.i.
1,000	870	290
2,000	1,160	870
3,000	1,740	1,160
4,000	2,610	2,030
5,000	5,220	2,610
6,000	6,090	3,190
7,000	7,830	3,770
8,000	9,280	4,640
9,000	11,020	5,510
10,000	12,760	6,670
11,000	14,210	7,830
12,000	15,660	8,700
13,000	17,110	9,860
14,000	18,560	11,020
15,000	20,010	11,890
16,000	21,460	12,760
17,000	22,620	13,630
18,000	24,070	14,500
19,000	25,520	15,660
20,000	26,970	15,950
21,000	28,420	16,240
22,000	29,580	16,820
23,000	31,030	17,110
24,000	31,900	17,690
25,000	33,350	---
26,000	34,800	17,980
27,000	35,670	18,270

VITA

Larry Stephen Johnston was born in Memphis, Tennessee, on March 18, 1953. He attended elementary schools in that city and graduated from Sheffield High School in June 1971. The following September he entered the University of Tennessee at Martin, Tennessee, where he attended for two years. In September of 1973 he transferred to Knoxville where he finished his undergraduate degree in Civil Engineering in June of 1975.

Upon graduation he accepted a position as structural engineer with Southern Cast Stone Company in Knoxville, Tennessee. In the fall of 1975 he began graduate studies by attending night classes at the University of Tennessee. His Master of Engineering degree was received in June 1979.