MAINE YANKEE ATOMIC POWER COMPANY •

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September 30, 1982
MN-83-190 JHG-82-178

United States Nuclear Regulatory Co[illegible]ssion
Washington, D. C. 20555

Attention: Office of Nuclear Reactor Regulation
Division of Licensing
Operating Reactor Branch #3
Mr. Robert A. Clark, Chief

References: (a) License No. DPR-36 (Docket No. 50-309)
(b) USNRC Letter to MYAPCo, dated August 26, 1982
(c) MYAPCo Letter to USNRC, Maine Yankee Final Safety Analysis Report, dated July 7, 1982 (MN-82-139)
(d) MYAPCo Letter to USNRC, dated November 4, 1980 (WMY-80-149)

Subject: I.E. Bulletin 80-11 - "Masonry Wall Design"

Dear Sir:

This letter transmits Maine Yankee Atomic Power Company's response to your request for additional information, Reference (b), regarding masonry walls at Maine Yankee. We have addressed each request and have provided additional information for your review.

REQUEST 1

Indicate whether the load combination mentioned in the response to Item 2.b.iii.a in Reference (d) is specified in the final safety analysis report (FSAR).

RESPONSE

The load combination mentioned in the response to Item 2.b.iii.a in Reference (d) is not specified in the FSAR as the specific design basis of masonry walls. However, this load combination was based upon the design loads listed in FSAR Section 5.1.1.2, Reference (c), and is intended to include all postulated loads.

A001

REQUEST 2

Indicate the boundary conditions used for analyzing the masonry walls and provide justification for those boundary conditions.

RESPONSE

Both fixed and pinned boundary conditions were investigated for each wall panel analyzed in order to bound panel frequencies and to calculate stresses due to applied loads. If a pinned joint was assumed, flexural shear capacity was checked at the joint. If moment fixity was assumed, both shear and bending capacities were checked.

Please note that Request 3 has been separated into 3 parts.

REQUEST 3a.

In Reference (d), the licensee indicates that arching analysis has been used to qualify some of the masonry walls. The NRC at present time does not accept the application of the method to masonry walls in nuclear power plants in the absence of conclusive evidence to justify this application. The licensee is requested to indicate the number of walls which have been analyzed by arching analysis.

RESPONSE

Arching analysis was used for the following walls:

1. Battery Room Walls at Elevation 35'-0".
2. Battery Room West Wall at Elevation 45'-6".
3. Cable Tray Room East Wall at Elevation 35'-0".

Arching analysis was employed in lieu of more refined analyses after the first conservative load combination resulted in a tensile stress, perpendicular to the bed joints, in excess of ACI allowables.

REQUEST 3b. and c.

The following areas need technical verification before any conclusion can be made about this technique:

Exp[illegible] how the arching theory handles cyclic loading, especially when the l[illegible] reversed.

Provide justification and test data (if available) to validate the applicability of the arching theory to the masonry structures at Maine Yankee Atomic Power Plant, with particular emphasis on the following areas:

1. Nature of Load
2. Boundary Conditions
3. Material Strength
4. Size of the Test Walls

RESPONSE

As discussed by Gabrielson and Kaplan, (See Enclosures A and A.1) arching theory handles cyclic loads by simply reversing the arch curvature and resisting inertia forces by shear and compressive strength. The shock tunnel tests discussed in Enclosure A apply to the masonry walls analyzed since the test specimens are of comparable size (3.5 feet high X 12 feet wide), strength (8 inch brick vs 12 inch block), and boundary conditions (vertically restrained). The authors cite examples of arching behavior under actual earthquake loadings, however, the test load was a dynamic blast pressure (as opposed to seismic shaking) where cracked test panels withstood many cycles of reverse loadings with maxima equivalent to accelarations greater than 1.0g.

REQUEST 3d.

If hinges are formed in the walls, the capability of the structures to resist an in-plane shear force is diminished, and shear failure might take place. This in-plane shear force would also reduce the out-of-plane stiffness. Explain how the effect of this phenomenon can be accurately determined.

RESPONSE

In-place shear forces are not a severe load since the inter-story displacements are small (approximately .003") and self-limiting and since block walls are not depended upon to resist these loads. (Also see response to REQUEST 7).

REQUEST 4

Provide sample calculations for block pullout analysis.

RESPONSE

Refer to Enclosure B.

REQUEST 5

Indicate whether the walls are stack bond or running bond. If any stack bond wall exists, provide sample calculations of the stresses for a typical wall.

RESPONSE

All mortared walls are running bond.

REQUEST 6

Indicate if cracking of sections was given proper consideration in the analysis.

RESPONSE

As discussed in our November 4, 1980 response, Reference (d), calculated wall frequencies were reduced 50% and the peak acceleration between F_{calc} and $F_{calc}/2$ was used for the analysis. Since our walls are not vertically reinforced, cracking, as defined by the calculated stress exceeding ACI allowables, was interpreted as the initiation of failure or of arching action and was then analyzed and/or modified accordingly.

REQUEST 7

Indicate how earthquake forces in three directions were considered in the analysis.

RESPONSE

Vertical acceleration was combined individually with each orthogonal horizontal acceleration as described in the Maine Yankee FSAR.

REQUEST 8

Provide the drawings of the deflection shield which was installed to prevent the stacked wall in the primary auxiliary building from falling into the surge drum area.

RESPONSE

Refer to Enclosure C.

REQUEST 9

Provide a description and the current status of the required modifications. Also, provide detailed drawings of sample modifications and a sample calculation to show that the modified walls will be qualified according to the working stress design method.

RESPONSE

All required modifications were installed by March 1, 1982. However, during final checking of the modification designs a math error was detected which will require additional strengthening of masonry wall number 7 (See Attachment 1, Reference (d)). This change will be completed by the end of the upcoming 1982 outage.

Please see Enclosure D for a sample calculation and modification details.

We trust that you will find this additional information satisfactory; however, should you have any questions, please feel free to contact us.

Very truly yours,

MAINE YANKEE ATOMIC POWER COMPANY

John H Garrity

John H. Garrity, Senior Director
Nuclear Engineering & Licensing

JHG:pjp

Enclosures

cc: Mr. Ronald C. Haynes
Mr. Paul A. Swetland

ENCLOSURE A

ARCHING IN MASONRY WALLS SUBJECTED TO OUT-OF-PLANE FORCES

Bernard L. Gabrielsen

San Jose State University

and

Kenneth Kaplan

Scientific Service, Inc.

Abstract. Non-reinforced masonry walls, confined between rigid supports that restrict in-plane motions and rotation of wall elements about the supports, can display very high resistance to out-of-plane forces by forming three-hinged arches after cracking in flexure. Analysis indicates that two different types of arching can occur depending on whether a wall is tightly fitted between supports (rigid arching), or is separated from one support by a small gap (gapped arching).

Special static tests were devised to investigate the kinds of loading that occur at the hinges of the arches (line loadings). These tests indicated that rigid arching walls can resist 6 to 8 times the loads that gapped arching walls can, although gapped arching walls are still considerably stronger than either cantilevered walls or walls mounted as simple beams.

An extensive dynamic test program involving full-scale walls, 8½ ft (2.6 m) high and 12 ft (3.7 m) wide, subjected to blast waves in a large shock tunnel, confirmed that brick walls undergoing rigid arching could withstand loadings as high as 19 psi (131 kN/m^2) equivalent to about 34 g. These walls cracked in flexure but did not fail, and then withstood many cycles of reversing loadings with maxima equivalent to accelerations greater than 1 g.

Examples of arching behavior under actual earthquake loadings were found in the San Fernando earthquake of 1971, and the Caracas earthquake of 1967.

A number of important espects of arching phenomena (e.g. plate or two-way arching, soft arching, effects of curtain walls) still require investigation.

Key Words. Arching; buildings; earthquakes; failure; infill walls; low-level fatigue; masonry; rigid frames; walls.

1. GENERAL

The response of non-reinforced masonry walls to out-of-plane loadings has been studied for approximately eight years as part of a program sponsored by both the Defense Civil Preparedness Agency and the Veterans Administration. While many different types of walls were studied, emphasis in this paper is on "arching walls" -- walls that acquire resistance to out-of-plane motions or forces by being confined in a frame that inhibits in-plane motions and rotation about the walls' supports.

2. FULL-SCALE WALL TEST PROGRAM

For the Civil Defense studies, a unique facility called a shock tunnel was used which enabled full-scale walls to be subjected to air blast loadings over one entire face. Fig. 1 is a cutaway view of the facility, a former coastal defense installation. The area occupied by the shock tunnel is in the foreground. It consisted of a 63-ft (19.2 m) long steel cylinder which served as a "compression" chamber, and a 100-ft (30.5 m) long passageway 8½ ft (2.6 m) high by 12 ft (3.7 m) wide which served as an "expansion" chamber, in which full scale walls were mounted. The expansion chamber opened on a large casemate area which once housed a 16-in. (40.6 cm) gun.

Fig. 1 Cutaway view of the shock tunnel test facility.

Fig. 2 is a closer cutaway view of the shock tunnel. In the domed steel cylinder, 60-ft (18.3 m) long strands of Primacord (detonating fuse) were strung. Upon detonation, the hot, high-pressure detonation products expanded rapidly in the compression chamber and acted like a piston to drive a shock (blast) wave out the open mouth of the cylinder. The cylinder, which experienced thrusts of up to 1,000,000 lbs (4.5×10^3 kN), is held in the tunnel solely by polyurethane foam. The shock wave, moving at a velocity somewhat greater than the speed of sound, proceeded down the expansion chamber, at the end of which it encountered a test wall, which thereupon experienced a generally uniform loading over one entire face.

A still closer cutaway view of the tunnel showing wall mounting details is sketched in Fig. 3. Heavy steel blocks were bolted to the tunnel wall to serve as anchors for steel girders which spanned the tunnel, simulating floor systems in actual buildings. Two additional vertical girders were used in connection with walls that were mounted as plates. Most of the walls were constructed outside the shock tunnel itself in light steel frames. After curing, the wall in its frame was moved into the tunnel and affixed to the girders.

Walls which were mounted as simple plates (with all edges pinned) in the manner shown in Fig. 3, acutally exhibited modified arching behavior once flexural failure began. Because of wall bonding to the frame members, the "picture frame" support structure became a perimeter restraining ring (to a rose petal type shell). As a result, arching thrusts developed as shown in Fig. 4, and the walls appeared to be approximately twice as strong as true simple plates would have been. In one impressive case [1], the result of this behavior was to neatly remove a section of wall, approximately 2.5 ft (0.75 m) on a side, from the center of a panel, leaving the remainder of the panel standing. This

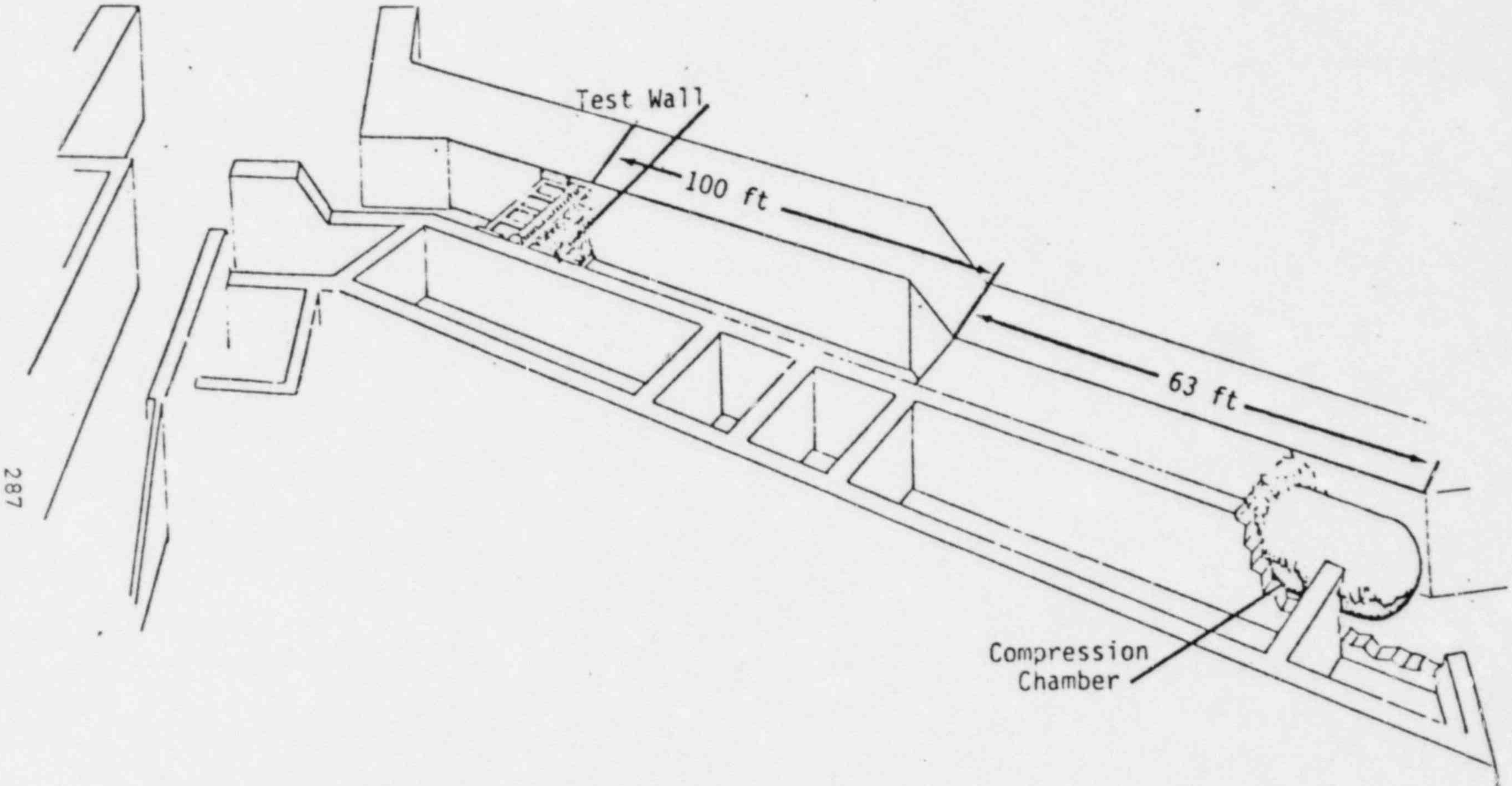

Fig. 2 Cutaway view of shock tunnel showing wall in place.

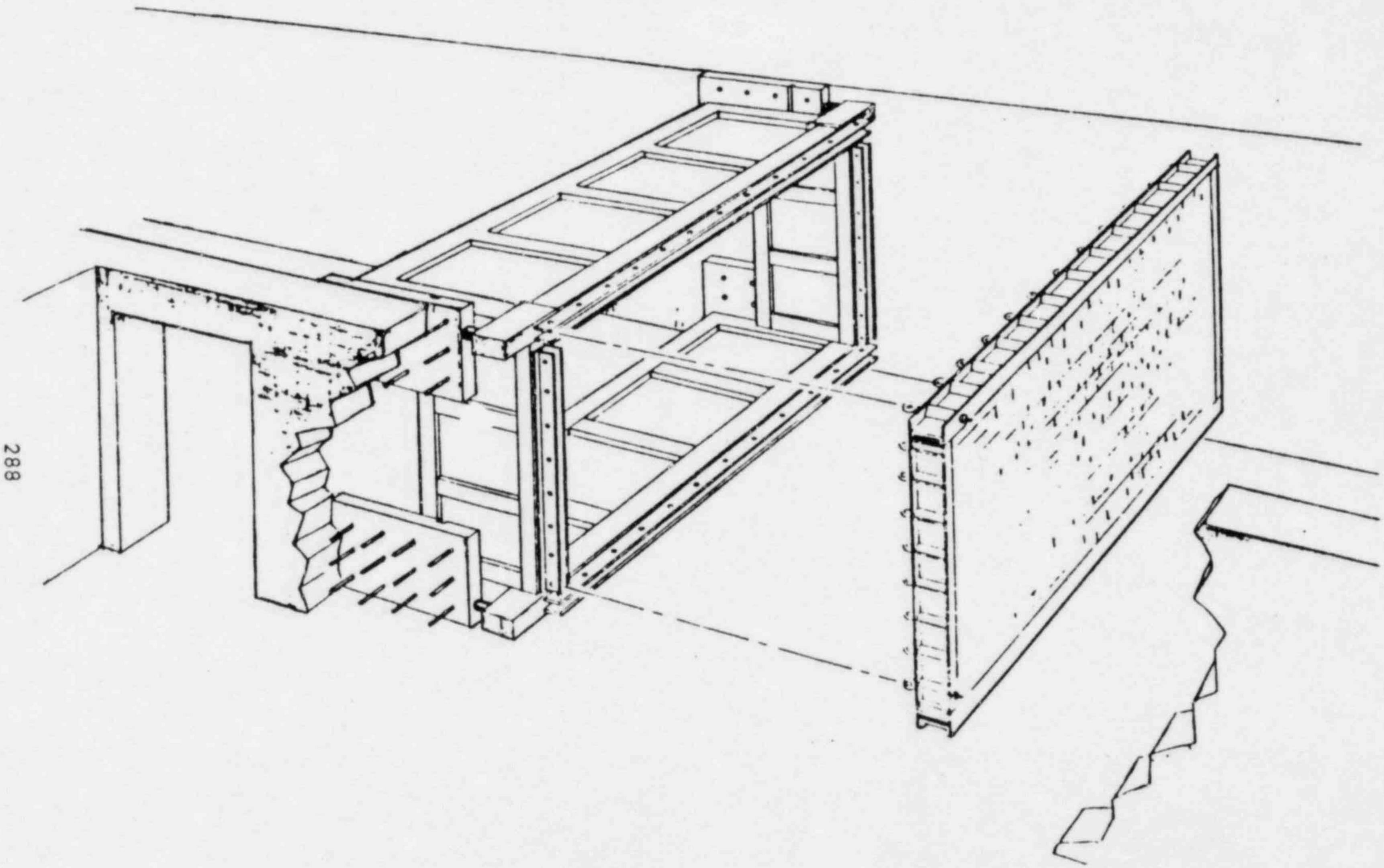

Fig. 3 Cutaway view of shock tunnel showing test panel and simple plate support condition hardware.

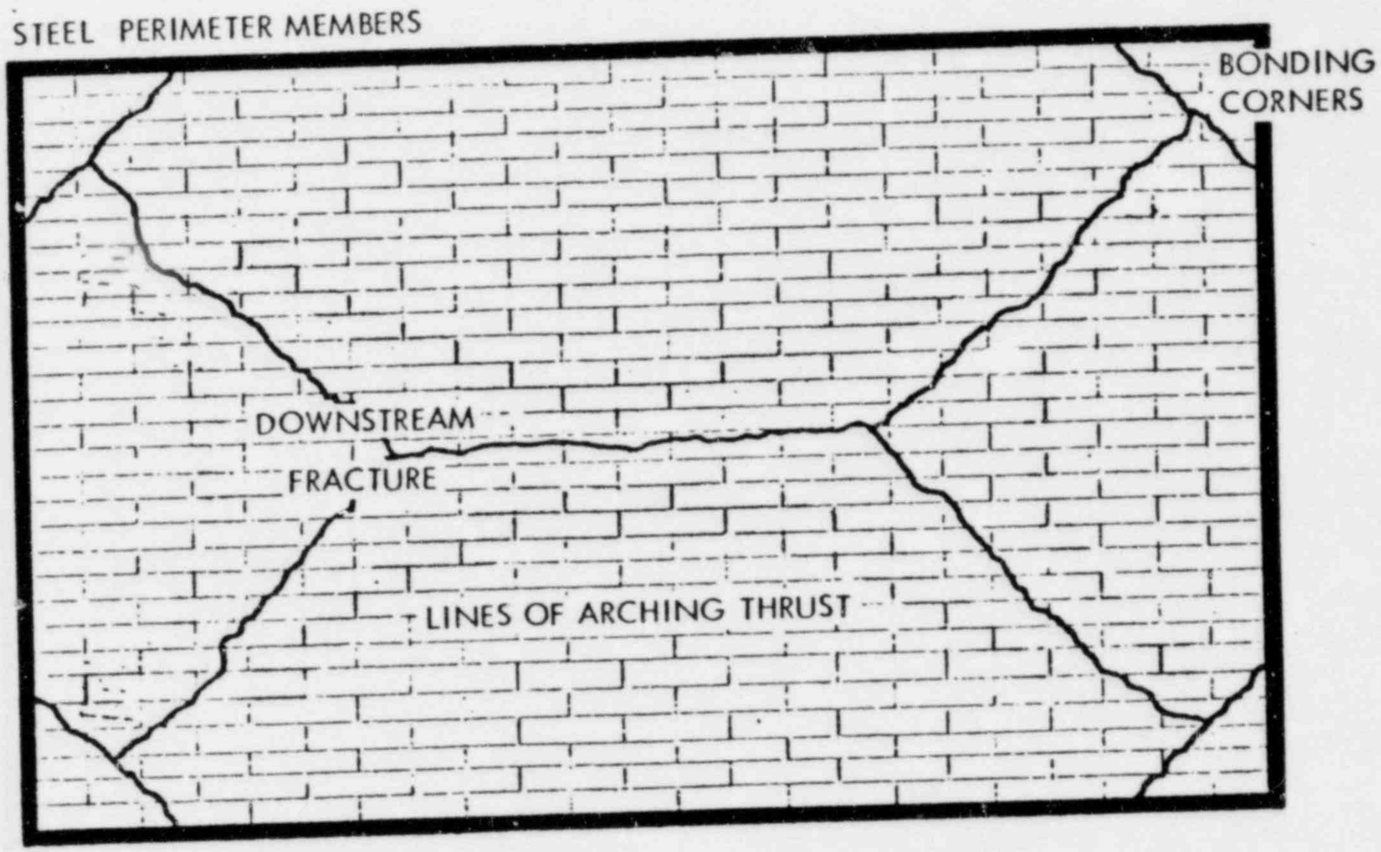

Fig. 4 Thrust lines for modified arching in plate mounted walls.

is shown in Fig. 5. Note the crack pattern on the downstream face of the wall extending from the corners of the hole.

The building of the walls was contracted out to a local bricklayer who was instructed to use normal practices while constructing them. At the same time, the contractor also fabricated many samples to be used in a variety of static tests of material properties and properties of the brick and mortar assemblies. These included samples for compression, shear, and tension tests, and others (beams) to be used in tests for tensile strength in flexure.

Failure statistics reflected a fairly large property variability, as can be seen in Fig. 6, an extreme probability plot of tensile stress at flexural failure [2]. Curve 1 is for specimens carefully constructed in the laboratory, and Curve 2 is for specimens constructed by the bricklayer. The data for both lines show stresses that differ by about a factor of three. There is good correlation between failure stresses in the field and laboratory samples.

3. RIGID AND GAPPED ARCHING

The resistance of walls tightly fitted into rigid frames to out-of-plane loadings has been studied for many years. Theories for walls rigidly restrained on two opposite edges were developed almost 20 years ago [3,4], and more recently refined [5,6]. These indicated that resistance of such walls to out-of-plane forces, brought about by what is termed "rigid arching", could be larger by factors of 10 or more than the resistance of similar walls mounted as simple beams (pinned on two opposite edges).

There was some question, however, whether a wall separated from one of its confining frame members by a gap (which could be caused by poor mortaring techniques, mortar shrinkage, or even by deliberate inclusion of a low-strength flexible seal

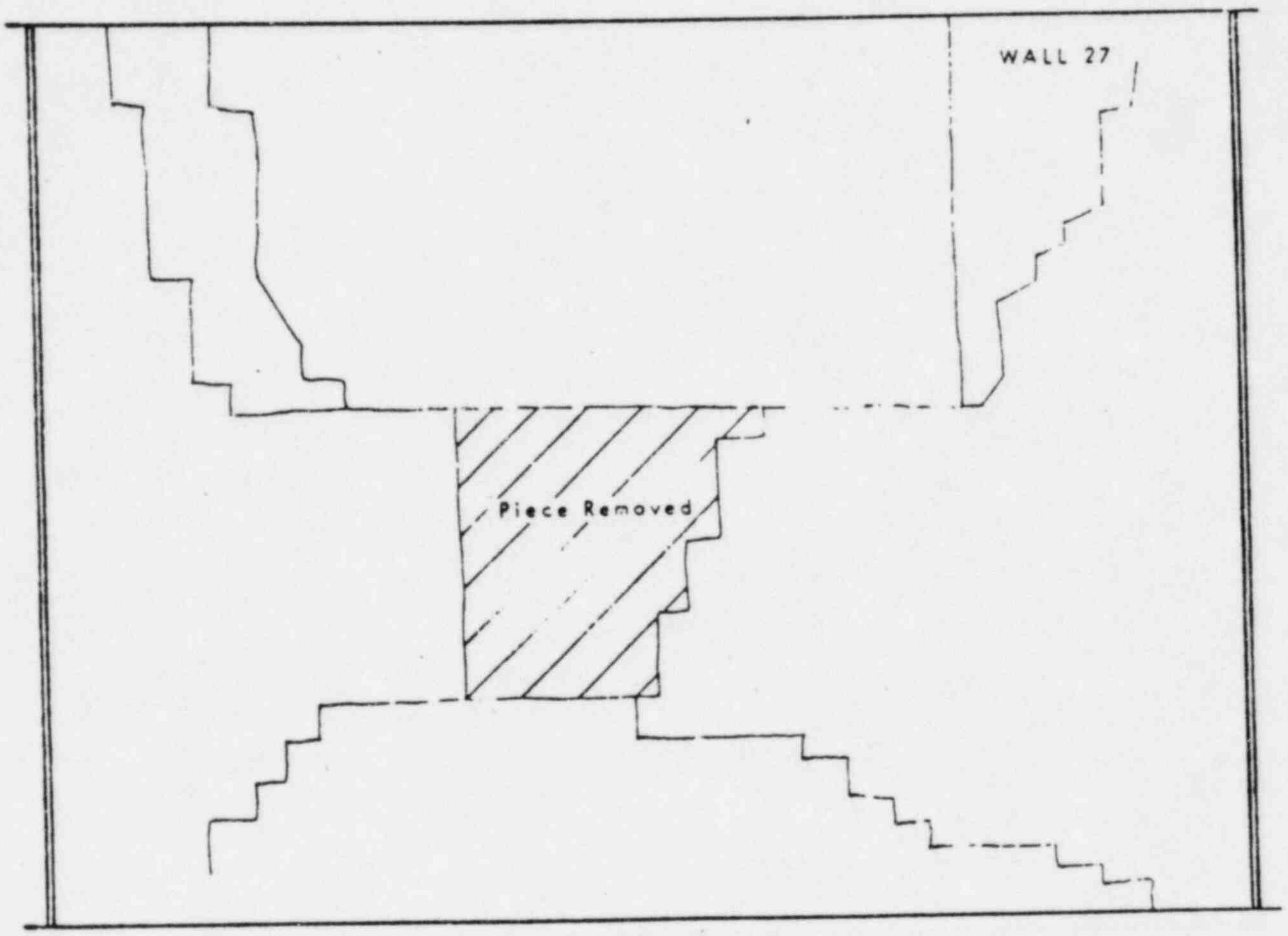

Fig. 5 Sketch of downstream face of plate-mounted wall after loading with over-pressure of 3.5 psi.

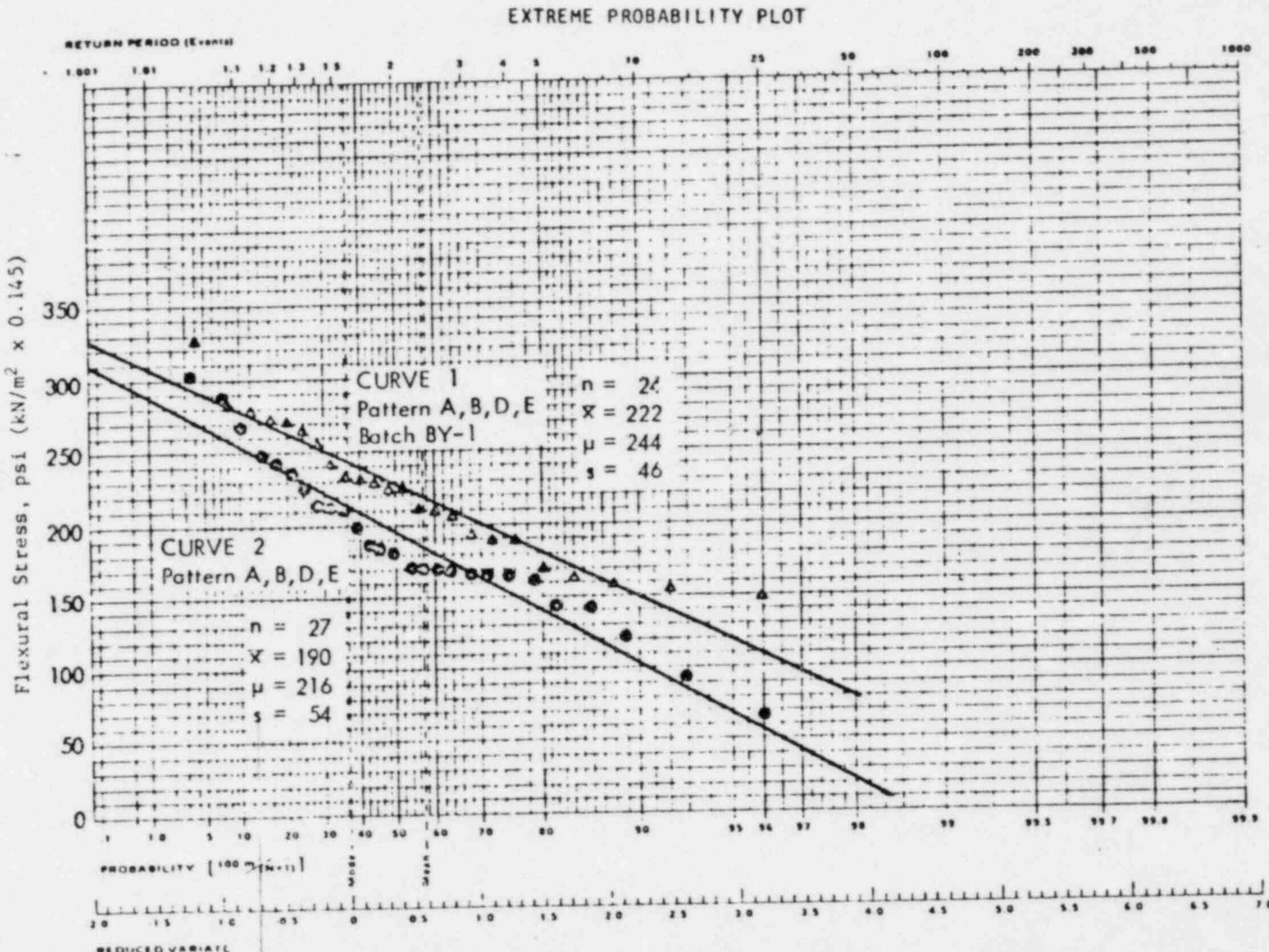

Fig. 6 Comparison of wall specimens and laboratory control specimens (patterns A, B, D, and E).

between wall and frame) would also exhibit arching behavior. It can be shown [7] that a modified form of arching (termed "gapped arching") could take place which would still afford increased resistance to out-of-plane forces over the walls mounted as simple beams, though not nearly as much as is afforded by rigid arching.

The essential differences between the two forms of arching are shown in Figs. 7 and 8. Fig. 7 shows, in exaggerated fashion, the motions that take place. In rigid arching, a symmetrical three-hinged arch is formed; in gapped arching, an unsymmetrical arch is formed. Fig. 8 contains free-body diagrams of the wall elements in the two cases. In rigid arching, forces at the arch hinge points are all directed into the wall (or parallel to its face). In gapped arching forces at two hinge points are directed away from the wall. Thus, in rigid arching, failure at the hinge points should be largely through crushing (compressive) forces. In gapped arching, failure would also take place through spalling (tensile) forces.

In both types of arching, the loads at the hinge points are applied along the hinges, that is along lines in the wall faces. To determine failure strengths under these line-loads, static tests were conducted using the specially designed test configurations shown in Fig. 9, modifications of standard compression test configurations. In 11 tests with rigid arching samples, an average line-load of 4500 lb/in. (7900 N/cm) was required to cause failure; in four tests with gapped arching samples, line loads of about 1000 lb/in. (1800 N/cm) caused failure [7]. Photographs of a rigid arching type of static line-load test are shown in Fig. 10. The sample is one from a wall of a V.A. hospital [8].

The measured values of line-load failure strengths led to static resistance

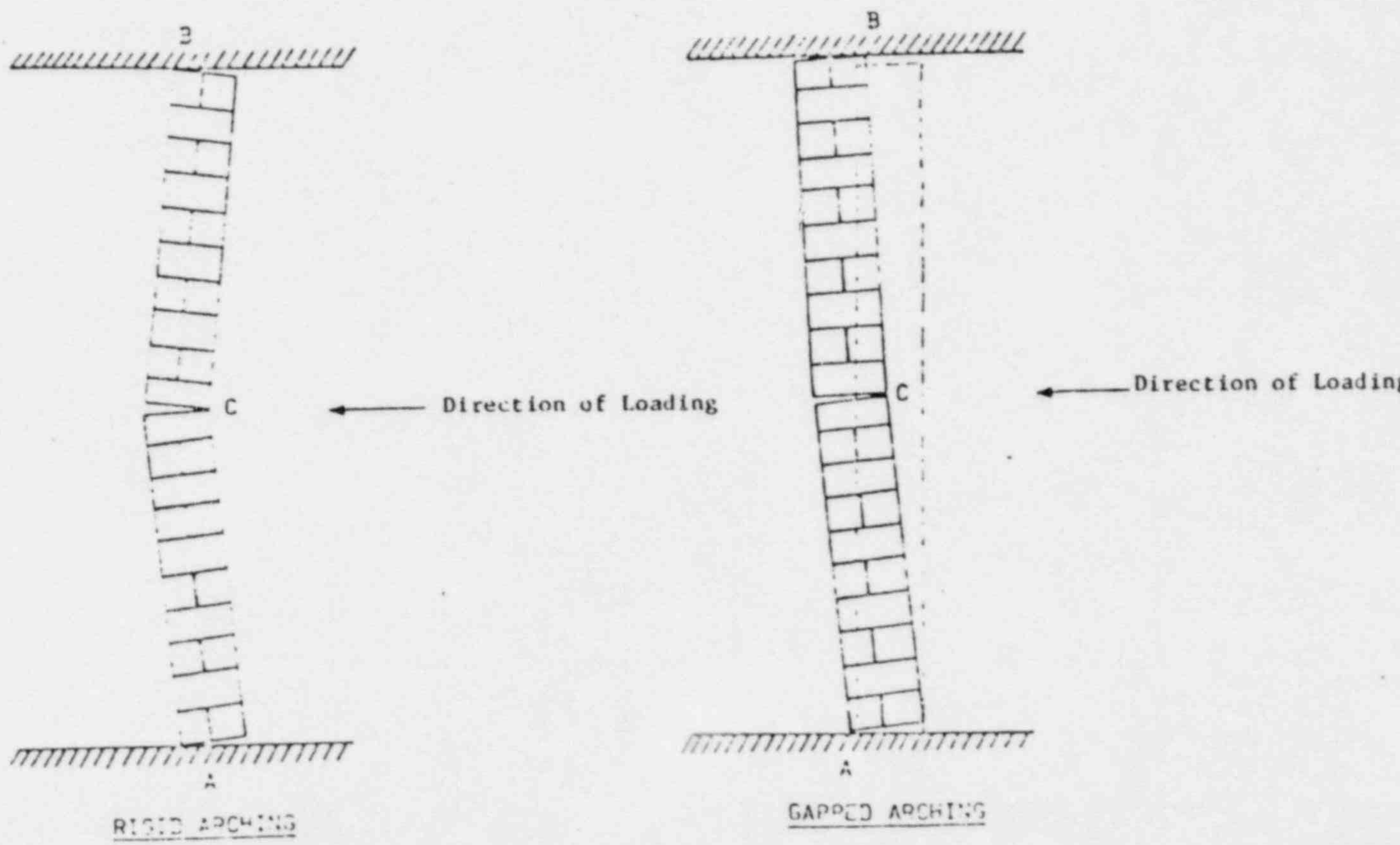

Fig. 7 Sketch illustrating the differences in motion between rigid and gapped arching.

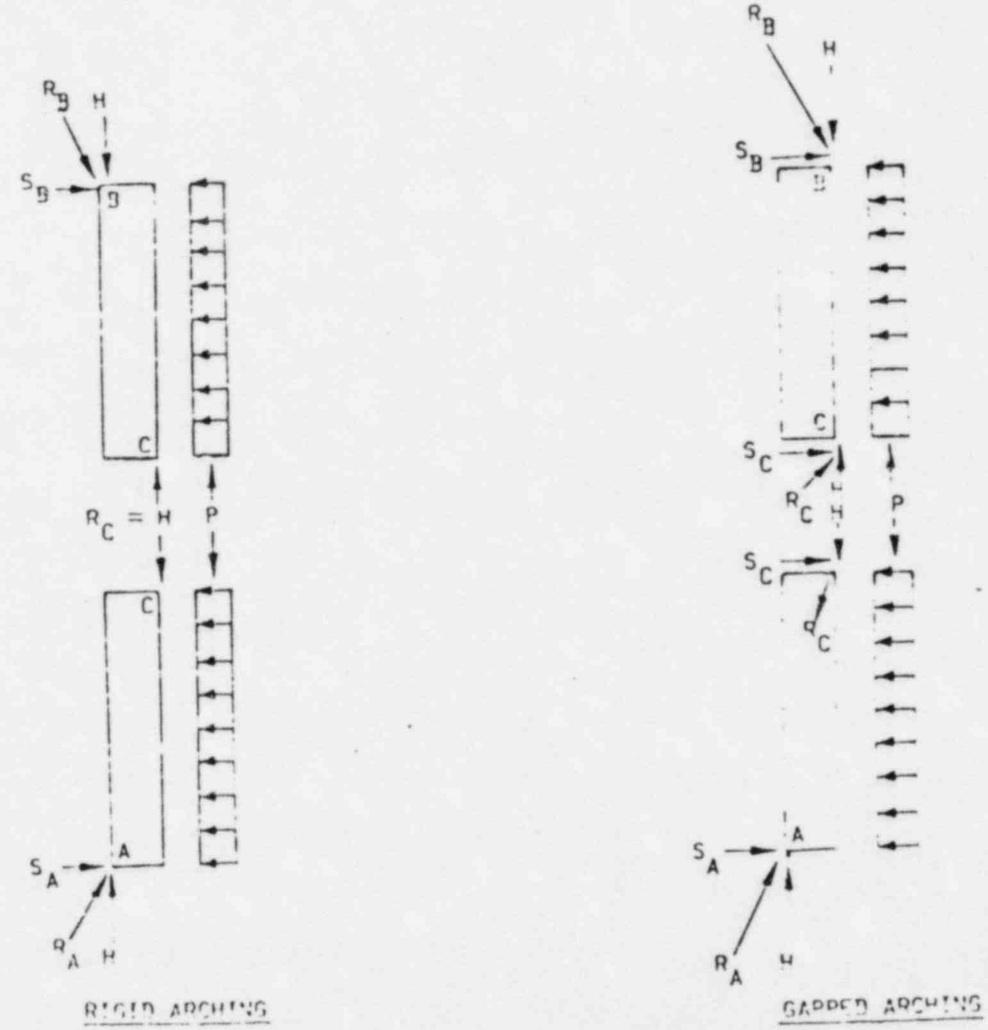

Fig. 8 Free body diagram showing forces in rigid and gapped arching.

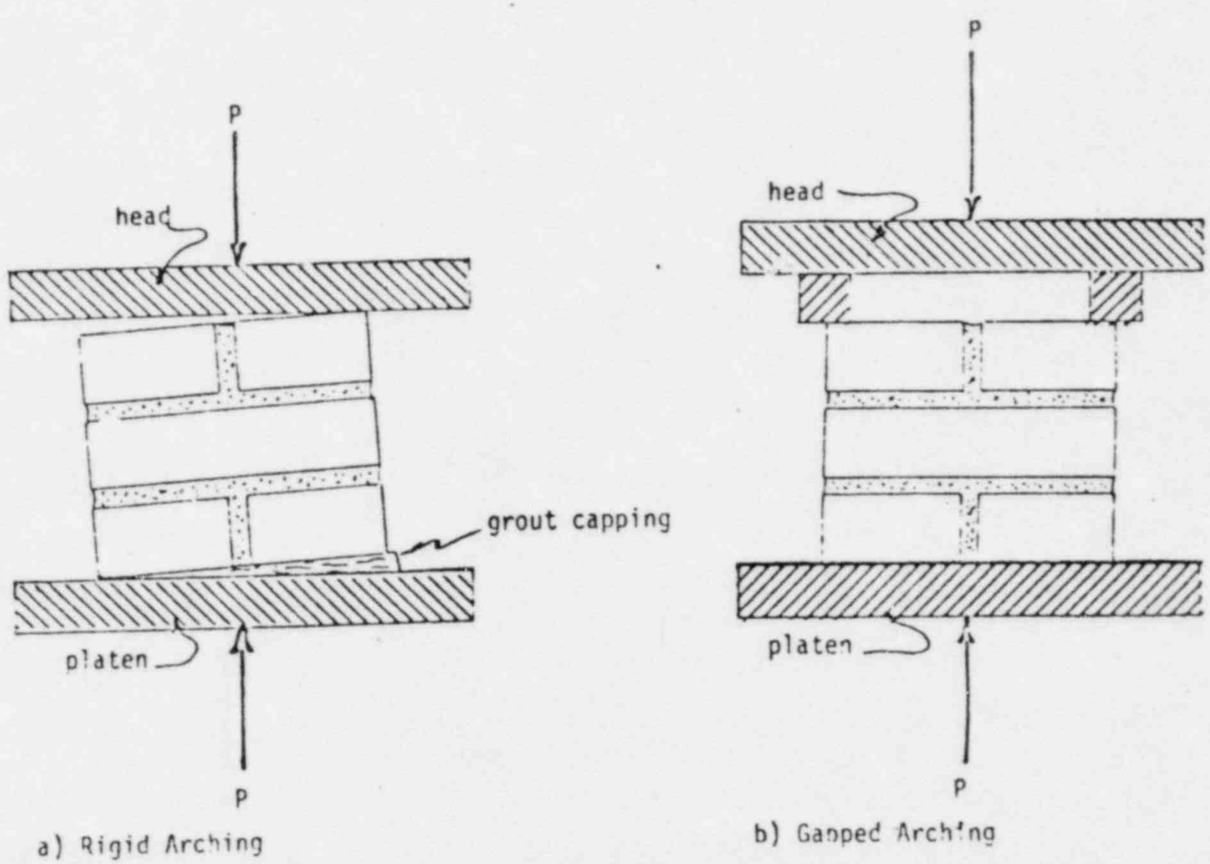

Fig. 9 Configurations for rigid and gapped arching line load tests.

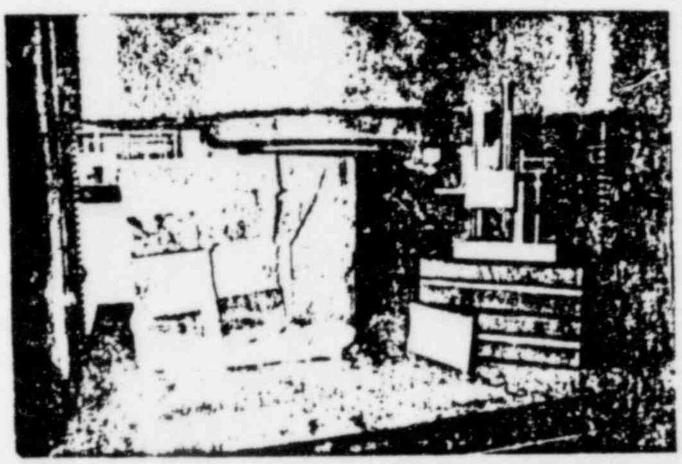

Fig. 10 Pre-, and post-test photographs of a rigid arching type of line-load strength test.

functions for walls (resistance of a wall to uniform loadings normal to one face) such as those shown in Fig. 11. Clearly, the resistance of gapped arching walls is much less than that of rigid arching walls. It should be appreciated, however, that both are still considerably larger than the resistance of non-arching walls. Consider, for example, the four types of walls shown in Fig. 12, on which tests and analysis were conducted, and which resemble each other superficially. Pressures to cause wall failure, and the relative energy absorbed by the walls to the point of their failure (i.e., when they become unstable and would collapse under gravity alone) are shown in Table 1.

Table 1

WALL FAILURE FACTORS

Wall Type	Approximate Failure Pressure psi (kN/m^2 x 0.145)	Energy Absorbed (arbitrary units)
Cantilever	0.2	1
Simple Beam	0.9	3
Gapped Arching	2.5	27
Rigid Arching	16.0	350

Theory and experiment for rigid arching walls compare quite well. In Fig. 13 are shown records of motion at the centerline of an 8-in. thick brick wall, tightly fitted between floor and ceiling of the tunnel so as to undergo rigid arching. The wall was tested three times. It was first exposed to a blast loading of 13 psi (90 kN/m^2). It cracked in the center, oscillated, but recovered. It was then exposed to a loading of 15 psi (100 kN/m^2). Again, it oscillated, but recovered. Finally, it was loaded at 20 psi (140 kN/m^2) well above its predicted failure point, and it did fail. Fig. 14 shows a comparison between centerline motion measured during the second test, and predictions of this

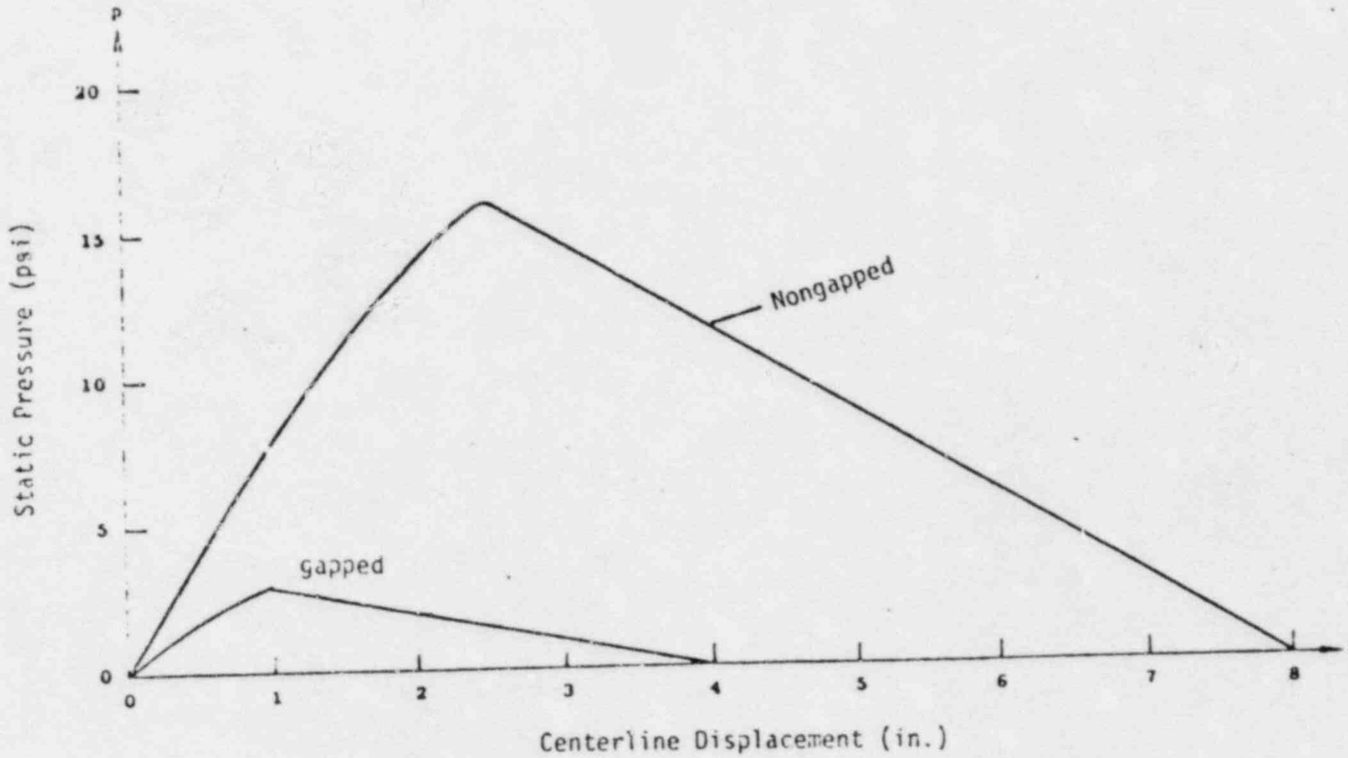

Fig. 11 Arched brick wall, static resistance.

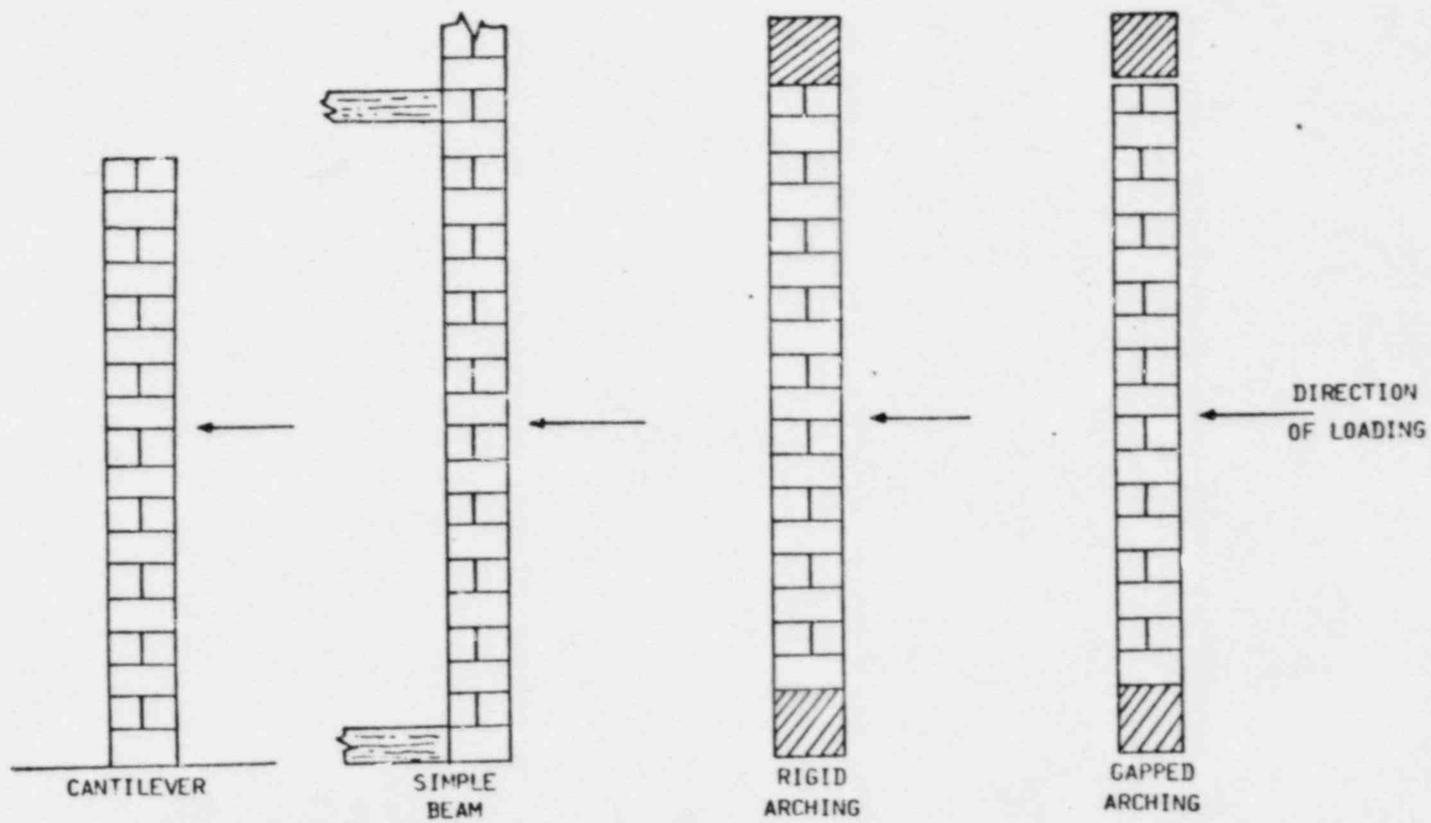

Fig. 12 Types of walls.

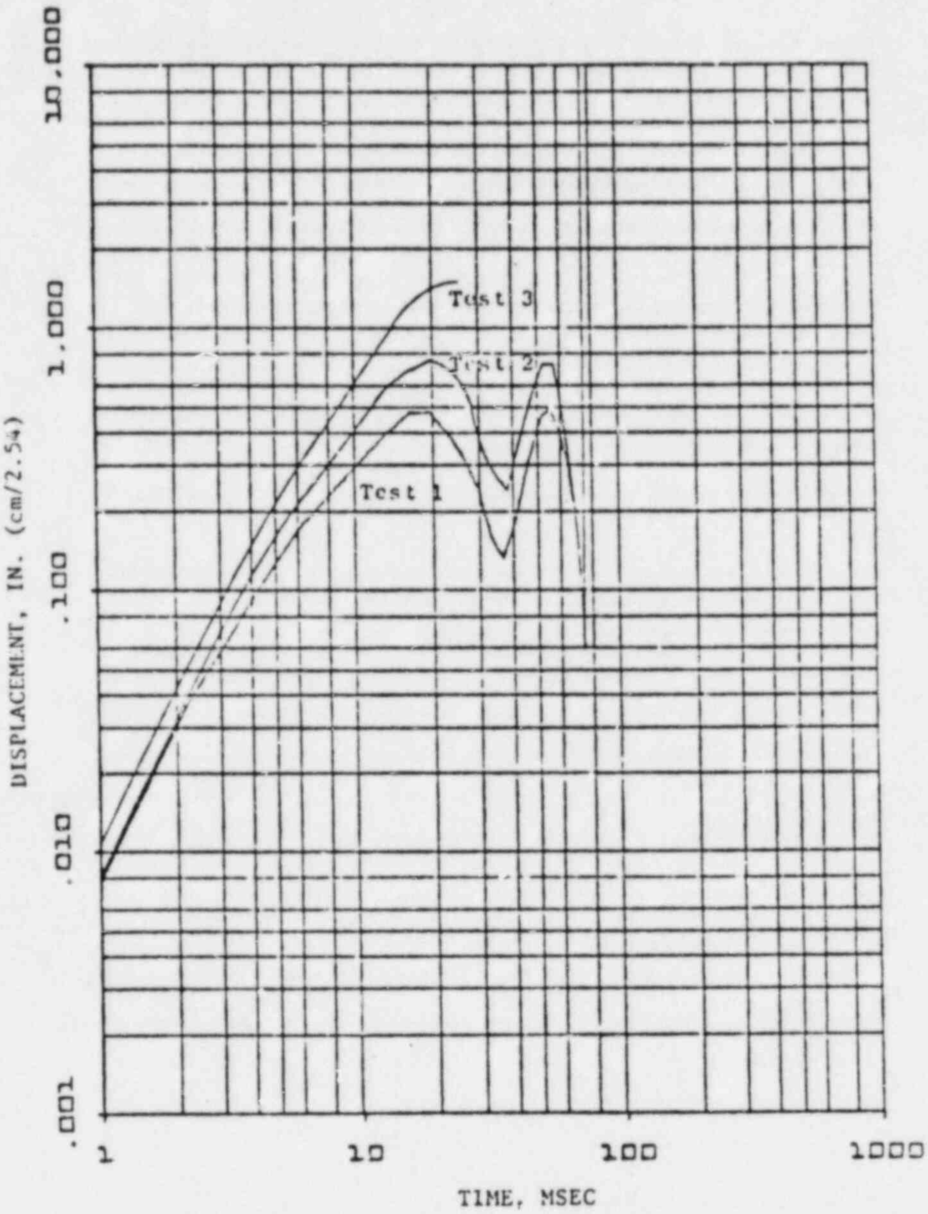

Fig. 13 Displacement as a function of time, wall no. 87, Test 1, 13 psi (90 kN/m^2); Test 2, 15 psi (100 kN/m^2); Test 3, 20 psi (140 kN/m^2).

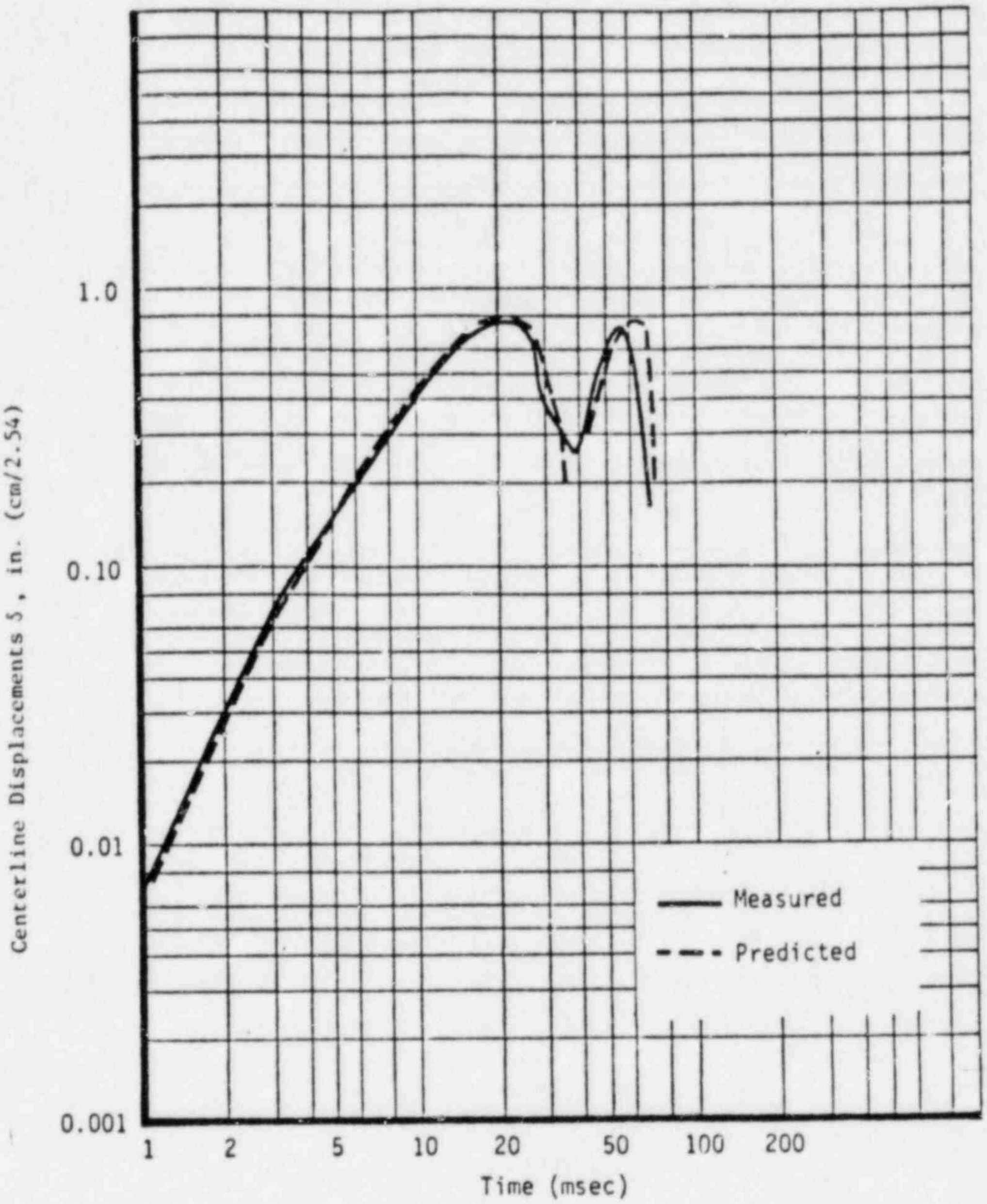

Fig. 14 Predicted and measured wall centerline displacements for a rigidly arching wall vs time from initial blast loading, wall no. 87, second test. Loading pressure = 15 psi (103.5 kN/m^2)

motion from a dynamic analysis [6].

As far as resistance to earthquake induced forces is concerned, the uniform blast load converts to an earthquake acceleration by multiplying the load by a factor of 1.8. (This assumes, of course, that a structure and frame can transmit a uniform acceleration to a wall.) Thus, a 15 psi (100 kN/m^2) uniform blast loading, which the wall withstood after being cracked by a 13 psi (90 kN/m^2) loading, is approximately equivalent to an earthquake acceleration of 27 g. Clearly, arching walls can provide substantial resistance to out-of-plane loadings even after they are damaged.

A special characteristic of the test facility emphasizes how strong arching walls can be even after they crack in flexure. If a wall (even one with a window or doorway opening) struck by a blast wave did not fail, the wave reflected from the wall returned to its source area, (see Fig. 2) then re-reflected to strike the wall again, about 0.3 sec after the first loading. On a second test, if the wall again did not fail, the process was repeated, so that some walls were loaded and reloaded many times. The pulses themselves had a positive loading pulse about 0.1 sec long, followed by a negative loading phase af about the same duration, but of much lower intensity. Thus, these tests provided information on "low-level fatigue", or the ability of walls to withstand a number of reversing loading cycles.

On a single test with a solid wall, each of the many pulses after the first loaded the wall with a maximum pressure about 2/3 the maximum of the preceding pulse. On a test with a wall containing a window opening, each pulse after the first had a maximum pressure about 1/3 of the preceding pulse. But some of the initial loads -- and therefore the succeeding loads as well -- were extremely high in terms of normal earthquake loadings. Furthermore, the walls that were tested a second time at high initial loadings and still did not fail, were again subjected to intense multiple pulses.

The type of pressure loadings experienced by a solid wall that did not fail is illustrated in Fig. 15, which clearly shows the loading reversals (the troughs of the trace where pressure is negative).

Table 2 gives the loading peaks, with pressures converted to equivalent acceleration in "g" units, experienced by seven brick walls, three of which were built with window openings. In each case, the wall cracked on first loading, but then withstood subsequent load pulses equivalent to very high earthquake accelerations.

4. ARCHING WALLS AFTER EARTHQUAKES

Field evidence confirms that non-reinforced, masonry, in-fill walls can provide increased resistance to out-of-plane loads over non-arching walls. Fig. 16 is a photograph of a building on the grounds of the V.A. hospital at Sylmar, which was exposed to an earthquake in February, 1971. Note that the upper walls facing the observer (which run north-south) are intact while the lower walls have fallen. The east-west walls of this building showed diagonal cracking from shear forces, strongly suggesting that the major direction of the earthquake was in the plane of these walls, that is, normal to the walls shown in Fig. 16.

Fig. 17 is a photograph of the first floor area of the same building where the north-south walls had been. Note the spalling and concrete failure at the top of the column in the photograph, which suggests that the frame did not behave as a rigid member as required for rigid arching to occur, i.e., the lower wall was not adequately restrained, and the upper one was. Fig. 18 is another example

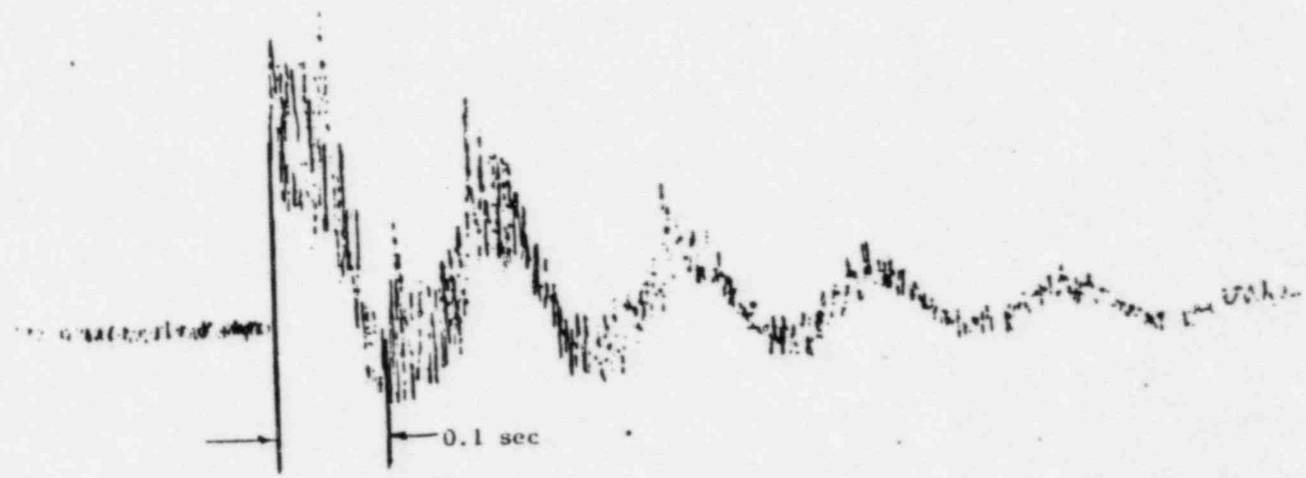

Fig. 15 Shock tunnel pressure gauge traces. Gauges mounted in nonfailing wall (peak reflected overpressure, first peak 5 psi.)

Table 2

"g" Loadings on Walls Experiencing Low Level Fatigue

Wall Number	Peak of Loading Cycles
	Solid Walls
71	7.4, 4.9, 3.2, 2.2, 1.5, 11.5, 7.7, 5.2, 3.4, 2.3, 1.6, 1.0, 16.6 wall cracked; cracks enlarged; wall failed
74	6.3, 4.1, 2.9, 2.0, 1.3, 23.2 wall cracked; wall failed
87	24.1, 16.2, 10.8, 7.2, 4.8, 3.2, 2.1, 25.6, 17.1, 11.4, 7.6, 5.1, 3.4, 2.3, 1.5, 36.0 wall cracked; cracks enlarged; wall failed
88	34.2, 22.8, 15.2, 10.1, 6.8, 4.5, 3.0, 2.0, 1.3, 14.4, 9.6, 6.4, 4.3, 2.9, 1.9, 1.3 wall cracked; cracks enlarged
	Walls With Window Openings
80	23.4, 7.8, 2.6, 27.0 wall cracked; wall failed
84	27.0, 9.0, 3.0, 1.0, 34.2 wall cracked; wall failed
85	26.1, 8.7, 2.9, 25.2, 8.4, 2.8, 32.4, 10.8, 3.6, 1.2, 43.2 wall cracked; cracks enlarged; additional cracking; wall failed

(Underlined loadings identify the first pulse maximum of a series.)

Fig. 16 Masonry infill wall and frame building at Sylmar, California.

Fig. 17 Detail of Sylmar building showing spalling at top of column.

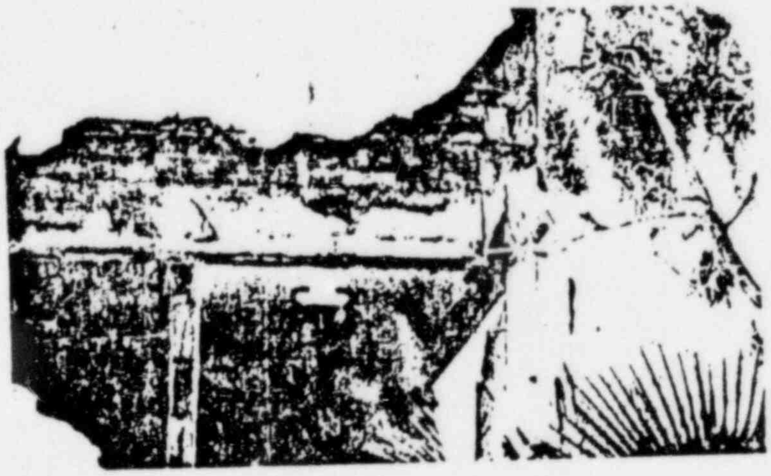

Fig. 18 Detail of Sylmar building showing failure at corner column.

of frame failure in the same building that prevented arching from occurri. and caused failure of the first-floor walls.

A photograph of another building in the same area as that shown previously, and of similar construction, is shown in Fig. 19. The parapet wall (a cantilever wall) failed, but the other exterior walls, which could have arched in their frames, did not fail.

A building in Caracas whose walls show evidence of severe loading is shown in Fig. 20. Walls in both the face of the building in the photograph, and in the hidden side of the building were identified as being infill walls in rigid frames [9]. None failed.

5. NEEDS OF A DESIGNER

Considerable effort is needed before the foregoing analyses can have a firm basis for practical application. The notion of a line-load which results in failure is relatively new, and the statistics of material strengths under such loadings are virtually non-existent. Many more tests involving the rigid arching type of failure (crushing) are needed, as is another method for evaluating the gapped arching type of failure (spalling). The test configuration shown in Fig. 9 only crudely approximates the actual mode of failure. Materials other than brick must also be studied.

The gapped arching analyses of [7] has not been adequately verified experimentally. In addition, while some preliminary work [10] has been done on cracked walls (rather than walls with a gap) more analyses and experiments are in order.

Very little work has been done on some other important aspects of the general arching problem. For example:

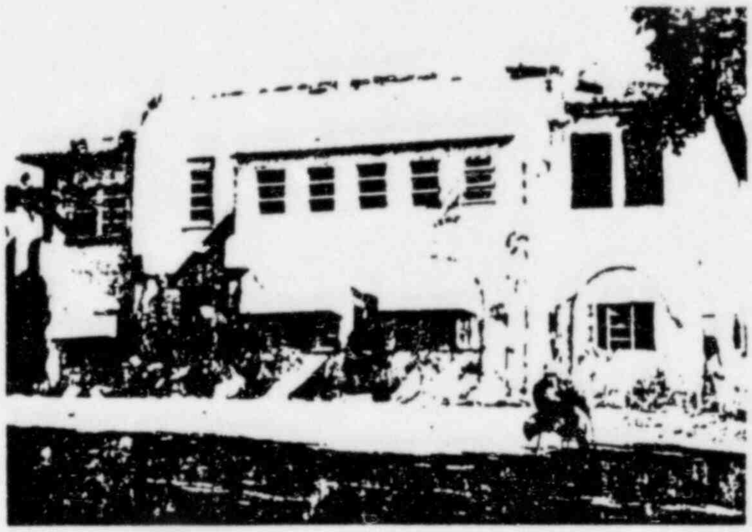

Fig. 19 Building at Sylmar, California in which parapet wall failed, but other walls did not fail.

Fig. 20 Building at Caracas, Venezuela showing cracked infill walls.

- The problem of "two way" or "plate" arching (where in-plane motions are restricted by rigid supports on all four edges) has received some consideration [11, 12], but the existing approaches do not consider any effects of gaps between wall and frame, or of cracks in masonry.
- Consideration has been given [7] to the effect of full-wall-height doorway openings on arching, but no analysis has been made of effects of window or partial-wall-height doorway openings.
- The changes in wall resistance that result from supports that are not perfectly rigid (termed here "soft arching") have been considered [13], but again more work is needed, as the existing analysis is an extension of the earlier rigid arching approaches.
- Little work has been done on determining wall response to forces that are directed neither normal to, nor parallel to, infill walls in a frame building. A fair amount of effort has gone into evaluating shear capacity of walls under in-plane loading [14, 15] and some has been done to evaluate the decrease in shear capacity due to cracks [15, 16, 17]. However, the effect on resistance to out-of-plane motions of both cornering and in-plane motions and their subsequent damage to the walls, has by no means received adequate attention.
- One problem that appears not to have been addressed at all involves a very common structural class: structures with curtain walls separated from the frame and its infill walls by a cavity. The curtain walls will most assuredly fail at lower loadings than will the infill walls if these have any tendency to arch at all. Whether a curtain wall failure can adversely affect an infill wall's capacity to arch is unresolved.
- The response of wall systems rather than individual walls requires consideration. It is easily conceivable that failure of a single wall in

a building could lead to failure of other walls that would otherwise withstand the forces imposed on them, because the first wall's failure altered the other walls' support and loading conditions.

It should also be appreciated that, while many of the problems associated with wall resistance to out-of-plane loadings, described in the preceding material, lend themselves to computer analysis, relatively little has been done in that regard.

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ENCLOSURE A.1

Section 2

ARCHING AND REINFORCED BRICK WALLS

BACKGROUND ON ARCHING

Prior analytical and experimental effort initially concentrated on walls of brittle materials (brick, clay tile, and concrete block) that acted as simple beams (the walls were supported on two edges, with the other two edges free), or as simple plates (the walls were supported on all four edges). It was found that where this type of support did not include resistance to forces parallel to the faces of the walls, or where this resistance was limited to preload values of the order of the weight of a few stories of walls, resistance to blast loadings was quite small. All such walls with openings (windows or doors) would fail at incident blast overpressures of four psi or less; all such solid walls (with no openings) would fail at overpressures of two psi or less.

The reason for this was simple: blast wave pressures applied normal to the upstream face of a wall supported at its edges would cause the wall to flex and induce tension in the downstream face of the wall. Tensile strengths of brick and mortar composites (or of similar brittle materials) are quite low; thus tensile cracks would form in the downstream face of the wall. Since there would be essentially no resistance -- other than the wall's inertia -- to the out-of-plane forces still being imposed by the blast wave, the wall would fail.

More recently, attention has been paid to conditions under which resistance to blast forces, even after a wall has cracked, can be very large. This can occur where the edges of a wall are enclosed within a rigid frame that does not permit in-plane movement, that is, movement of the wall parallel to its face. When such a wall is loaded normal to its face, it resists downstream motion because the elimination of in-plane motions does not allow individual wall elements to rotate freely about the wall's edges. In other words, the wall forms an arch between its rigid supports.

The potential importance of arching was recognized many years ago during the era of above-ground weapons testing in Nevada. Arching theory developed at that time -- supported by experiment both in Nevada and more recently in the shock tunnel -- indicated that resistance to out-of-plane wall motions could increase by factors of 10 or more if in-plane motions were totally prohibited.

During the course of the current program, however, a question arose about the strengh of walls which, though located within members that would prohibit in-plane motions, were separated from these members by a gap. The question had pertinence for two reasons: because some manuals of construction practice indicate that the inclusion of such a gap (or equivalent, a low-strength, flexible seal) between an infill wall and a frame is good building practice (it permits design frame action to occur); and because even where infill walls are carefully grouted into framing elements, mortar shrinkage is likely to cause the small gaps to form between wall and frame.

Analysis of this problem, subsequently supported by experiment, indicated that a form of arching could still occur where there was a small gap between wall and frame (gapped arching). However, it was of a different kind than the arching that occurred where there was no gap ("rigid arching"). Most importantly, the increase in resistance to blast pressures caused by gapped arching was found to be substantially less than that caused by rigid arching.

SUMMARY OF BASIC RIGID AND GAPPED ARCHING THEORY *

Fig. 2-1 and 2-2 illustrate the important difference between rigid and gapped arching. (Only "one-way" arching in which the wall is restrained on only two edges, is discussed**). Fig. 2-1 contains sketches showing how walls behave in the two cases (exaggerated for clarity). In rigid arching on the left, blast induced forces (or actually any force normal to the face of the wall) push the wall to the left. However, the wall is prevented from rotating about its top and bottom supports. Tensile (flexural) cracks form at the top, bottom, and center (where tensile stress is highest), but the wall elements cannot move downstream. A three-hinged arch forms with loadings along lines at the <u>downstream</u> edges of the top and bottom of the wall, and at the <u>upstream</u> edge of the crack.

* Summarized from Ref. 1 and Ref. 5

** Only limited work was done on "two-way"arching (where a wall is restrained on all four edges), preliminary calculations indicated that such walls should be about 1.5 times as strong as one-way arched walls.

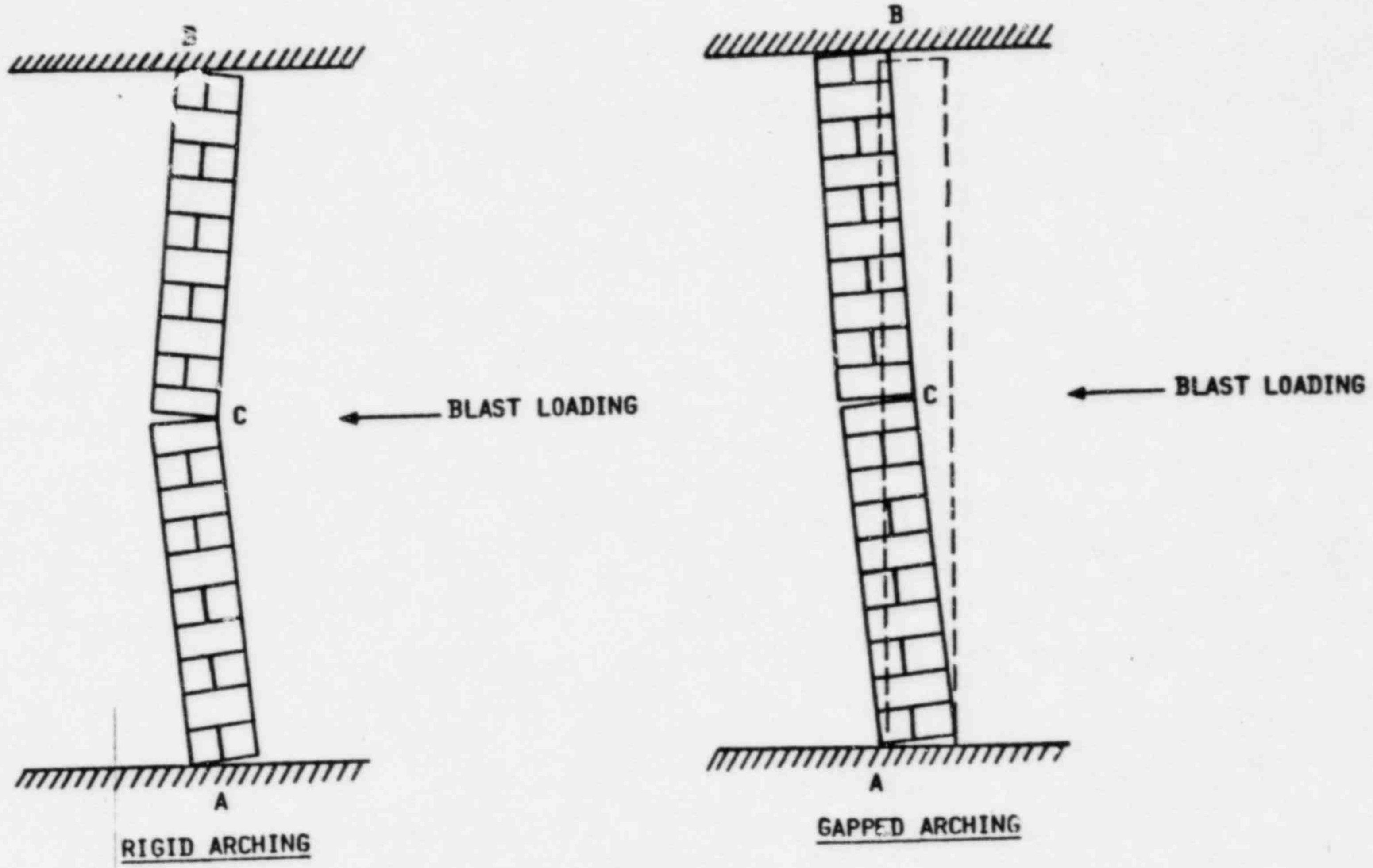


Fig. 2-1. Sketch Illustrating the Differences in Motion Between Rigid and Gapped Arching.

In gapped arching on the right, in Fig. 2-1, the gap permits the wall to move, either by bending or because of a crack at its base until the <u>upstream</u> edge at the top of the wall contacts the upper restraint. The original position of the wall is shown by the dashed line. A tensile crack forms at the wall center where maximum tensile stresses still occur. Again a three-hinged arch forms, but one which is not as symmetrical as in the rigid case. Loadings are at the <u>downstream</u> edge of the wall bottom, at the <u>upstream</u> edges of the tensile crack and the top of the wall. Note that the gap can be <u>very</u> small; (as shown in Ref. 1, it can be as little as about 0.01 in. - one hundreth of an inch - for an 8-in. thick, 96-in. high wall) the only requirement is that the <u>upstream</u> edge at the top of the wall contact the restraint.

Fig. 2-2 shows the directions of forces on the two parts of each wall. The general formulas for these forces per unit length of wall and the initial magnitudes of these forces for a wall with a height (ℓ) of 96 in. and a thickness, (t) of 8 in. are given in Table 2-1 for a uniform pressure (p). Fig. 2-2 shows that in rigid arching, the resultant forces are either directed into the wall (R_A and R_B) or are parallel to the wall face (at C). In the gapped arching case, however, there are forces (R_B and R_C) directed <u>away</u> from the upper part of the wall, and both the force directed into the wall (R_A) and the in-plane force (H) are about twice as large as they are in the rigid arching case. Forces directed into the wall would tend to cause compressive (crushing) failures; those directed away from the wall would tend to cause tensile (spalling) failures which occur at much lower stress values. Thus, the upper part of the wall in gapped arching should fail essentially in ten-

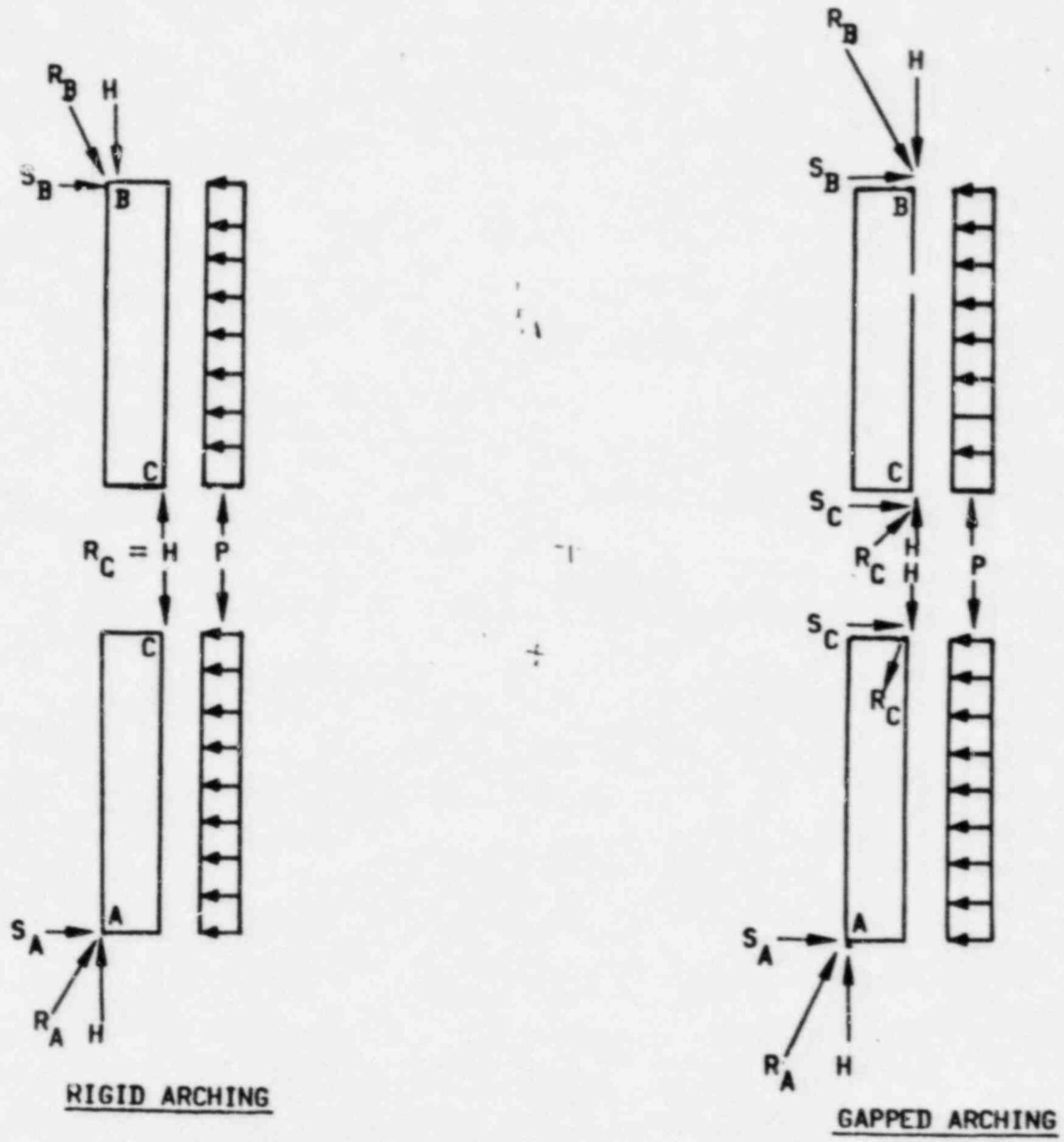


Fig. 2-2. Free Body Diagrams Showing Forces in Rigid and Gapped Arching. (See Table 2-1 for force values.)

sion at far lower incident pressures than those that would cause failure in rigid arching, since tensile strengths of masonry composites are lower than compressive strengths. Furthermore, since R_B and R_C are equal, the upper part of the wall should tend to move downstream without rotation. In other words, there is no tendency for a gapped arching mode of failure to convert to a rigid arching mode of failure, even when the gapped arching mode is instituted by a very small gap.

TABLE 2-1 FORCES IN RIGID AND GAPPED ARCHING

(See Fig. 2-2)

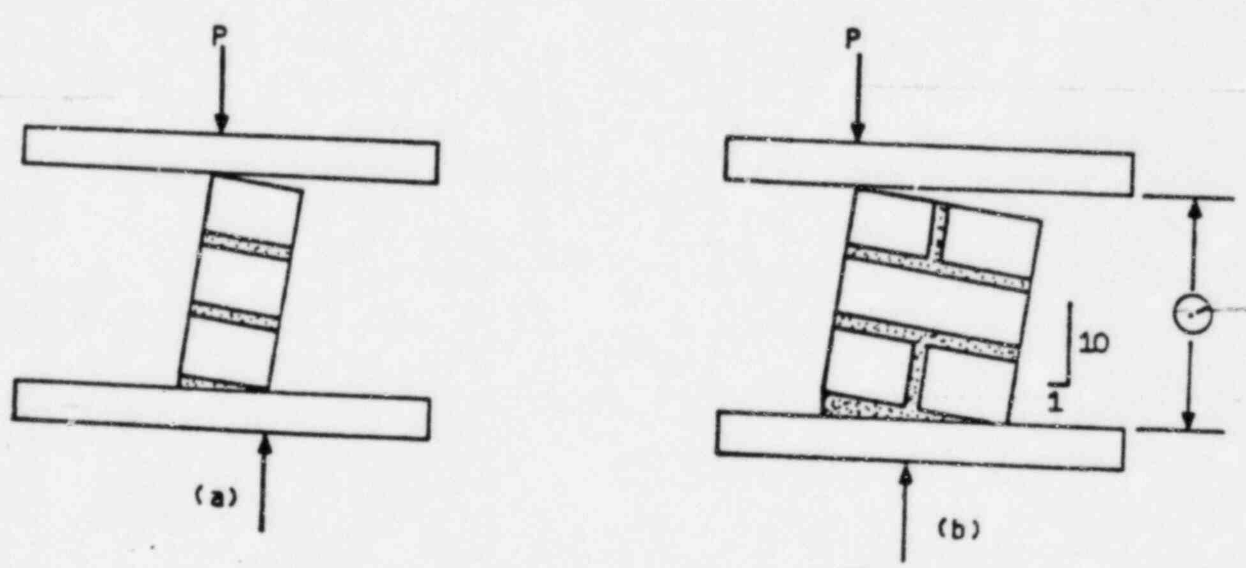
Force	Rigid Arching		Gapped Arching	
	General Formula	Value for ℓ = 96 in. t = 8 in. (lb/in)	General Formula	Value for ℓ = 96 in. t = 8 in. (lb/in)
H	$p\ell^2/8t$	144p	$p\ell^2/4t$	288p
S_B	$p\ell/2$	48p	$p\ell/4$	24p
R_B	$(H/\ell)\sqrt{\ell^2 + (4t)^2}$	152p	$(H/\ell)\sqrt{\ell^2 + t^2}$	289p
S_c	-	0	$p\ell/4$	24p
R_c	H	144p	$(H/\ell)\sqrt{\ell^2 + t^2}$	289p
S_A	$p\ell/2$	48p	$3p\ell/4$	72p
R_A	$(H/\ell)\sqrt{\ell^2 + (4t)^2}$	152p	$(H/\ell)\sqrt{\ell^2 + (3t)^2}$	297p

STRENGTH CONSIDERATIONS

In order to determine the resistance of both rigid and gapped arching walls, information is needed on the strength of wall materials subjected to the kind of forces that would cause either crushing or spalling failures along a line (that is, at the "hinges" of the arches). A variety of special static tests have been used to determine such "line load" strengths, and during this reporting period, techniques have been developed for predicting compressive line-load strength from standard compression tests.

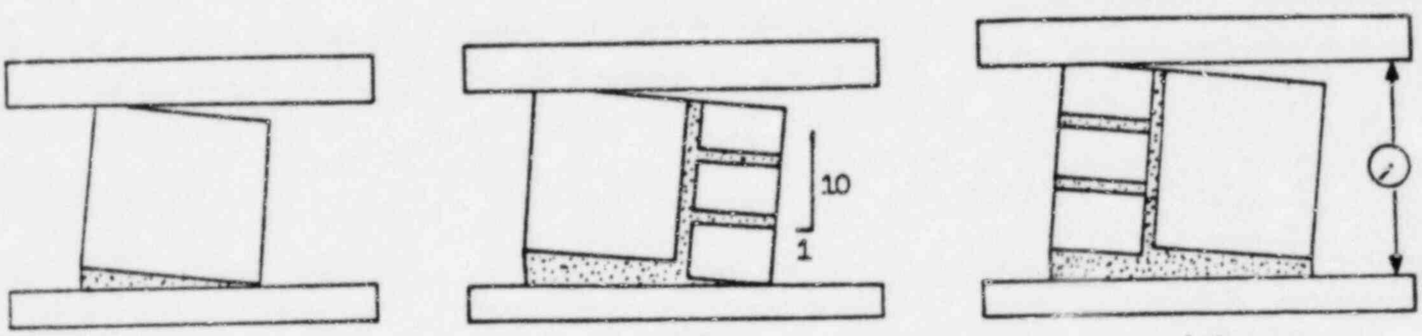
Summary of Static Tests

One series of tests were made using tilted brick-mortar composite specimens to determine the strength of wall composites under compressive (crushing) line loads. The basic test geometry for these static tests is sketched below.



Results from 11 tests with test setup (a) showed that failure occurred at an average total load (p) of 28,100 lbs. (Values ranged from 18,000 to 35,000 lbs. as shown in Table B-5 in Appendix B). The average line load at failure f_ℓ for the approximately 8 in. long specimens used was 3300 lb/in. (range 2100 to 4000 lb/in.). In the test setup (b) results from 11 tests gave an average p value of 43,100 lbs. (range: 35,000 to 52,000 lb) and an average f_ℓ value of 5100 lb/in. (range 4100 to 6000 lb/in.) again using approximately 8-in. long specimens.

Fewer line loading tests were made with concrete block, and brick-concrete block mortar composites. Test set ups are sketched below:

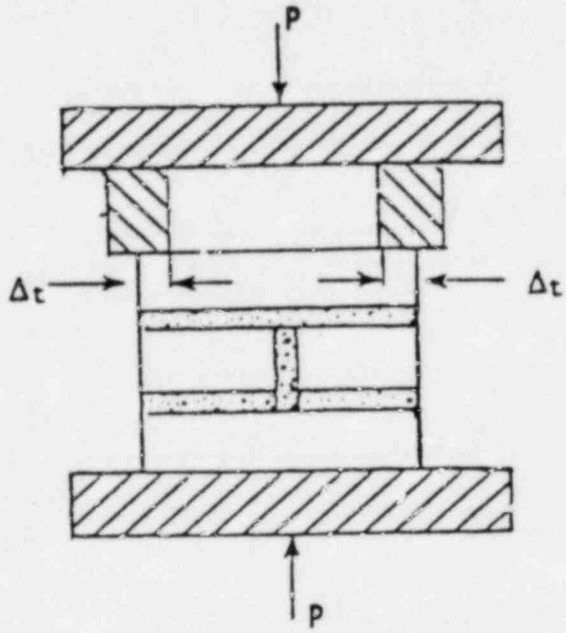


(c) (d) (e)

Average results were as follows:

Specimen	Number of tests	average p (lb)	average f_ℓ lb/in.
(c)	4	30,200	4,000
(d)	3	39,000	4,600
(e)	4	23,200	3,000

Additional tests were conducted to develop an approximation of strength under the tensile (spalling type) of line loading that could occur in the gapped arching case at points B or C in Fig. 2-1 and 2-2. That test set up is shown in the sketch below.



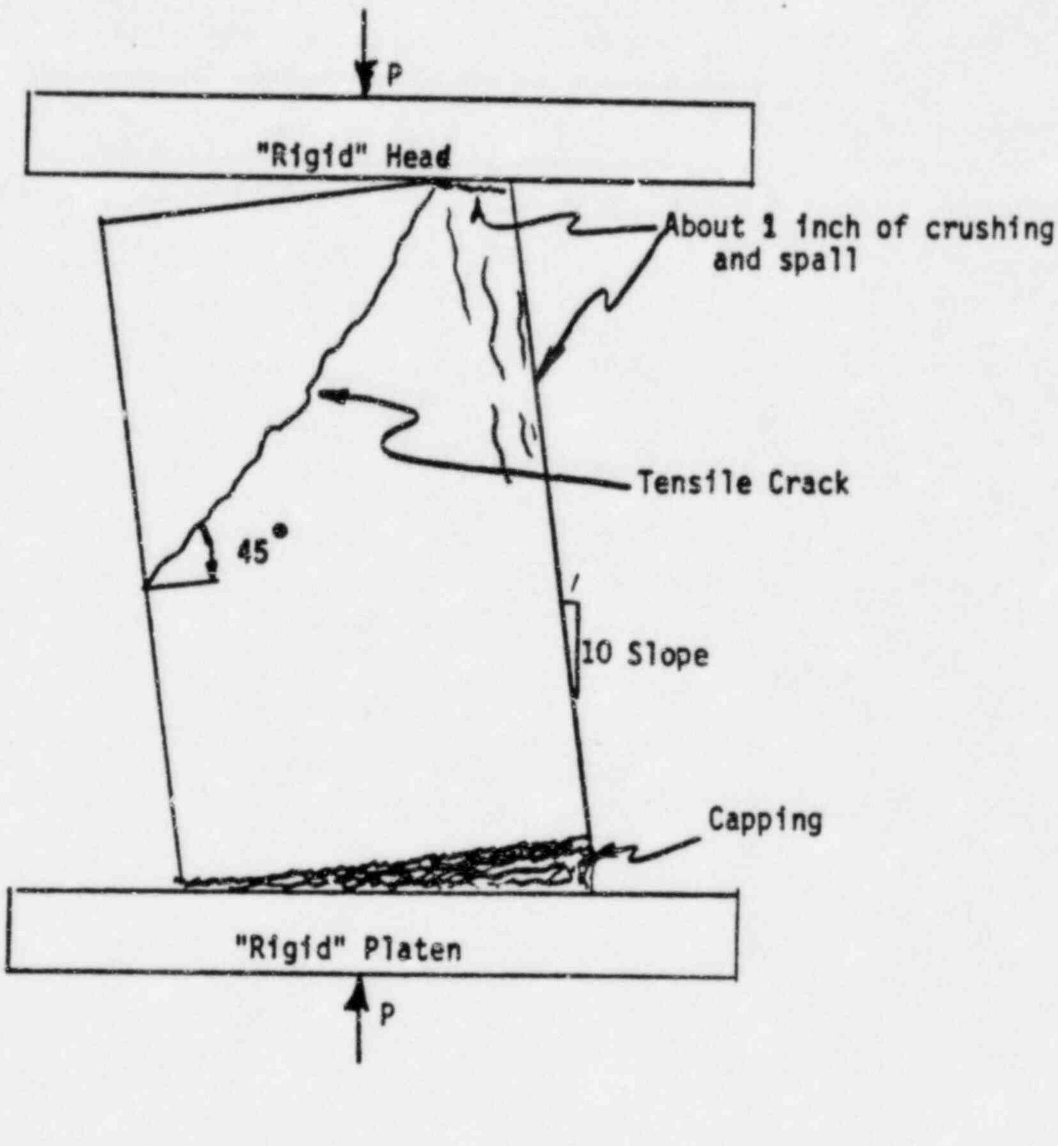
Two values of Δt were tested, ½ in. and ¼ in. For the latter, spalling was clearly occuring, and on four tests, the average P was 13,390 lb. and the average f_{ℓ} was 824 lb/in. As expected, these tensile line loading strengths are far below those for compressive line loading.

For more details on all these static tests described above, see Ref. 1.

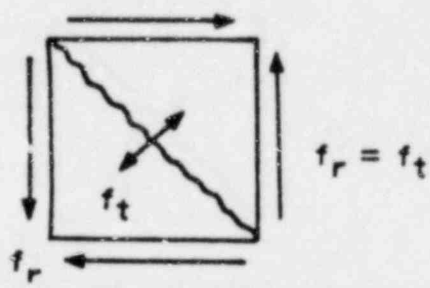
Development of Line Load Strength Prediction Techniques

During this reporting period consideration was given to the development of a prediction technique for line load resistance of walls based on compressive strength test data. Both brick and concrete block walls have been considered.

For brick test specimens consider the test set-up shown below.



Assume the tensile crack is caused by pure shear as shown below.



There are a number of sources relating tensile strength f_t to compressive strength $f_c{}'$:

From Ref. 2

for brick $f_t = 12\sqrt{f_c{}'}$

for mortar $f_t = 10\sqrt{f_c{}'}$

From Ref. 3

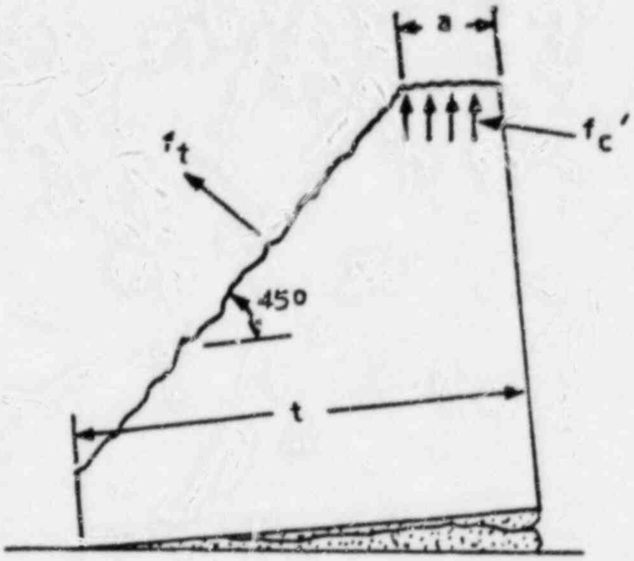
$f_t = 12\sqrt{f_c{}'}$

From Ref. 4

$f_t = 7.5\sqrt{f_c{}'}$

Therefore, as a rough average, let $f_t = 10\sqrt{f_c{}'}$

In the force diagram sketched below, if a unit width is assumed:



Total resistance, f_ℓ, is predicted to be

$$f_\ell = f_c'a + f_t \ (t-a) \cos 45^0$$

From Table 3, Appendix B, for the 15 tests with an ASTM, 3 brick and mortar composite, $f_c' = 2400$ psi.

Line load tests were also made on test specimens that were three bricks high, in a geometry like sketches (a) and (b) shown at the beginning of the static test portion of this section. The following results were observed.

f_ℓ = 3100 lb/in. (4 in. thick line load specimen, 14 tests)

f_ℓ - 4500 lb/in. (8½ in. thick line load specimen, 14 tests)

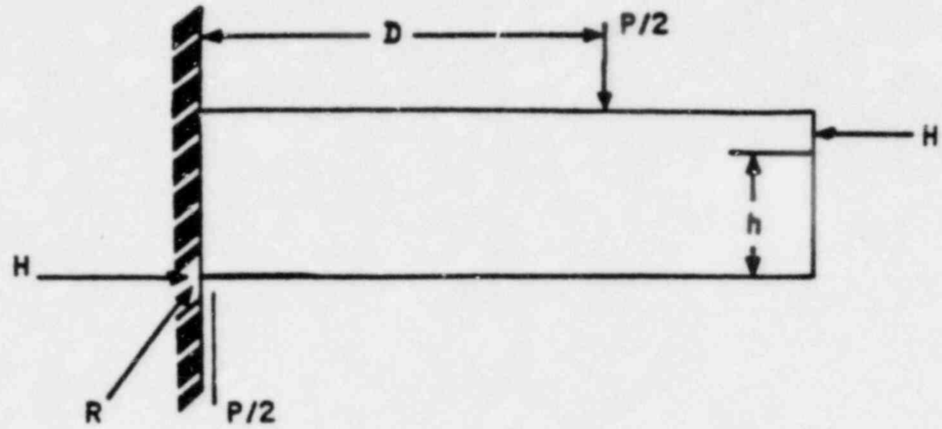
For this test series, using the observed f_c' of 2400 psi, an "a" of 1 in., and the 4 and 8½ in. thicknesses for "t", the total resistance equation gives f_ℓ values of 3439-lb/in. and 4998-lb/in. respectively.

Thus, we have the following:

Wall Thickness	f_ℓ (predicted) lb/in	f_ℓ (test) lb/in
4	3439	3300
8½	4998	5100
12	6557	-

A second type of static test series, in which conditions more nearly simulate those of an arching wall, was conducted at the shock tunnel site and are reported in Appendix B. In this series, beams were built horizontally into a 4 ft. wide, heavy-walled passageway and mortared in so that they performed as a rigid arch. They were loaded in a direction normal to their faces at the 1/3 points. (See Fig. B-3-B and the upper part of

Fig. B-4). A free-body diagram of one-half the beam is sketched below.



Taking moments about the left lower corner we have

$$H_h = (P/2)d, \text{ or } H = Pd/2h$$

in which h, the uncrushed portion of the beam is about 3 in., d = 16 in., and P = 10,479 lb. Therefore, H = 27,944 lb., the resultant force $R = \sqrt{H^2 + (P/2)^2} = 28,431$ lb., and the line load at failure, calculated by dividing R by the brick width of 8.625 in., is 3296 lb/in. This compares quite favorably with either the calculated value of 3439 lb/in. or the static test value of 3300 lb/in.

From the work on solid brick just described and from test observations on hollow clay tile and concrete block composites, several statements can be made with some confidence.

1) Hollow units, like concrete block and clay tile, have lower line load capacity than solid units.
2) From static test observations, the width of crushed material with hollow units is less than with solid units.
3) The shear/tension type of failure that goes along with the line-load phenomenon is apparently a reduced value with hollow walls because of the lack of shear/tension material.

From these observations we can make the following hypothesis: The ultimate line load capacity is proportional to the shear/tension capability of the unit. Further, the width of the crushed zone is reduced or controlled by the shear/tension capacity of the unit. Therefore, referring to the force diagram given earlier for the brick, the total line load resistance should be

$$f_{\ell} = k\left[f_c a + f_t (t-ak) \cos 45^{\circ}\right]$$

Where $k = b_s/b$

and b_s is the width of sheared material (Webs)

and b is the width of the block

This expression is identical with that for brick, except for the addition of the factor "k".

As with brick, two types of static tests have been made with which this expression can be evaluated. In the first, a beam made of concrete blocks was constructed between walls of a passageway and loaded in a direction normal to its face at the 1/3 points. (See Fig. B-3-A and lower part of Fig. B-4). This is identical with the brick beam test geometry which led to the following equation for H, the horizontal force at the beam edge.

$$H = Pd/2h$$

Using P from the static tests (11,600 lb.), d = 16 in., and h $\simeq$ 6.625* we have: H = 14,010 lb. The resultant force R = 15,160, and the line load at failure (R divided by the block width of 15.625 in.) is f_{ℓ} = 970 lb/in.

Let us compare this result with that from the line load predictor equation for a 16.in. block. For a nominal 16 in. block, the total block width is about 15-5/8 in. and the material through the center of the block where shearing takes place consists of three webs each about 1.27 in. thick. Therefore:

$$k = 3(1.27)/15.625 = 0.244$$

Compressive strength tests reported in Appendix B give f_c' = 2560 psi, $f_t = 10\sqrt{2560}$ (see derivation for brick mortar composite; this is a value for mortar), and a = 1 in. Thus:

$$f_{\ell} = 0.244\ (2560 + 10\ \sqrt{2560}\ \ (7.625-0.244)\ 0.707) = 1268\ \text{lb/in.}$$

which is reasonably close to the value from the beam tests (970 lb/in).

In a second static test series, a single nominal 8 x 8 x 8 in. block was loaded as shown at the beginning of this portion of the section. Corrected measurements of f_{ℓ} reported in Appendix B are f_{ℓ} = 4000 lb/in. The corrected value was derived by using a value of 7.625 which is the width of a standard concrete block in place of 8.8 in., the nominal width of a standard brick, which had inadvertently been used.

*The blocks were nominally 7.625 in. thick; an h of 6.625 in. allows for a crushing zone with a total width of one inch.

In the prediction equation used earlier k = 2 (1.27)/7.625 = 0.3331, so that f_ℓ = 1722 lb/in. The measured value does not compare favorably with the predicted value (3961 lb/in. vs. 1722 lb/in.), for reasons unknown. Results of other static and dynamic shock tunnel tests suggest that the experimental value may be questionable.

COMPARISON OF TEST RESULTS WITH THEORY

Test information given in Table 2-2 tends to confirm the results of the analytical work described earlier. Among walls with no window or door openings that underwent <u>rigid arching</u>, 8-in. thick brick, one-way arched walls failed on <u>initial</u> reflected pressure loadings of between about 13 and 19 psi (5.5 to 8 psi incident). As shown in the Table, some other walls were first loaded at lower levels and cracked, but they did not fail until loaded again, sometimes even a third time, at either the same low overpressure level or at higher levels. A single 8-in. thick, concrete block, one-way arched wall was tested to failure on initial loading, of about 10 psi loading pressure, (4.5 psi incident). Two similar walls subjected to <u>two way</u> arching failed at 9 and 11 psi loading pressures (4-5 psi incident), i.e. at about the same overpressure loading as for one-way arching, instead of at expected higher values. However, expected higher strength in two way arching was found for 4-in. thick brick walls, which were about 30 percent stronger than similar one-way arched walls.

Brick concrete block composite, one-way arched walls appeared to have strengths like those of similarly mounted brick walls, failing at over 11 psi loading pressure (about 5 psi incident).

TABLE 2-2

Summary of Arched Wall Tests

Solid Walls

Test Number	Incident (and Reflected) Overpressure (psi)		Remarks
			4-in. Brick (one-way)*
68a	.75	(1.5)	Wall cracked
68b	1.7	(3.5)	Wall failed
			4-in. Brick Arched Wall (two-way)**
83a	2.2	(4.7)	Wall cracked
83b	2.1	(4.4)	Wall failed
			8-in. Brick (one-way)*
71a	1.9	(4.1)	Test for natural period
71b	2.9	(6.4)	Wall cracked
71c	4.3	(9.2)	Cracks enlarged
74	5.5	(12.9)	Wall failed
75	5.9	(13.8)	Wall failed
76	5.6	(13.1)	Wall failed
87a	5.7	(13.4)	Wall cracked
87b	6.3	(14.2)	Cracks enlarged
88a	7.8	(19.0)	Wall cracked
88b	3.6	(8.0)	Cracks enlarged
94	7.8	(19.0)	Wall failed
96	6.7	(15.5)	Wall failed (pre-split)

*Geometrically restrained on top and bottom.
**Geometrically restrained on all four sides.

TABLE 2-2 (cont.)

Summary of Arched Wall Tests

Solid Walls

Test Number	Incident (and Reflected) Overpressure (psi)	Remarks
		8-in. Brick (one-way) with a gap
97	2.3 (4.9)	Wall failed
98	1.9 (4.9)	Wall failed
		8-in. Concrete block (one-way)*
77	3.3 (8.2)	Wall cracked
77	2.0 (4.3)	No additional damage
77	3.4 (8.5)	Wall failed
78	4.5 (10.2)	Wall failed
		8-in. Concrete block (one-way)* with a gap
115	4.1 (9.1)	Wall failed
116	1.7 (3.5)	Wall failed
		8-in. Concrete block arch wall (two-way)**
89	5.0 (11.4)	Wall failed
90	4.0 (4.3)	Wall failed
(Arched)		10-in. Composite brick and Concrete block Arched wall (one-way)*
79	5.6 (13.1)	Wall failed
92a	3.5 (7.8)	Wall cracked

*Geometrically restrained on top and bottom.
**Geometrically restrained on all four sides.

TABLE 2-2 (cont.)

Summary of Arched Wall Tests

Solid Walls

Test Number	Incident (and Reflected) Overpressure (psi)	Remarks
		10-in. Composite brick and concrete block Arched Wall (one-way) (cont)
92b	3.5 (7.8)	No additional damage
92c	5.0 (11.4)	Cracks enlarged
Walls with an opening		8-in. Brick wall with window (38" x 62") (one-way)*
80a	5.7 (13.4)	Wall cracked
80b	6.3 (14.2)	Wall failed
84a	6.4 (14.5)	Wall cracked
84b	7.8 (19.0)	Wall failed
85a	6.2 (14.0)	Wall cracked
85b	5.8 (13.5)	Cracks enlarged
85c	7.5 (18.0)	Slight additional cracking
85d	9.5 (23.8)	Wall failed
		8-in. brick with doorway (one-way)
86a	6.1 (14.3)	Wall cracked
86b	8.4 (20.5)	Cracks enlarged
		8-in. Brick with doorway (with gap)
95	8.6 (21.2)	Wall failed

*Geometrically restrained on top and bottom.

A few tests were made of walls with window and doorway openings, mounted so as to undergo rigid arching. As expected, these were stronger than solid walls (or rather they required higher incident overpressures to cause failure). Indeed, the single 8-in. thick, one-way arching wall with a doorway only cracked when subjected to a 6.1 psi incident pressure, and the cracks only enlarged when struck by a second shock with an incident overpressure of 8.4 psi.

(All similar walls without any openings failed at or below 8 psi incident overpressure.) Similarly, one of the two 8-in. thick brick walls with a window opening only failed when subjected to an incident overpressure of 7.8 psi (after three earlier loadings from 5.7, 6.3, and 6.4 psi incident overpressures). The second such wall only failed at 9.5 psi incident, after being struck and cracked by 6.2, 5.8, and 7.5 incident overpressure shocks.

Most of the tests involving gapped arching were undertaken during this reporting period. The single previous test was made before the decrease in strength due to the presence of a small gap was appreciated. It was an 8-in. thick wall with a doorway and was subjected to an incident overpressure of 8.6 psi. It failed catastrophically. The more recent tests used shock waves with overpressures much closer to expected failure overpressures. Two 8-in. thick brick walls failed at 1.9 and 2.3 psi incident overpressure, and one concrete block interior wall failed at 2.0 psi incident overpressure. (One additional test was conducted for debris data on a concrete block interior wall at an incident overpressure of 4.1 psi with expected catastrophic results.)

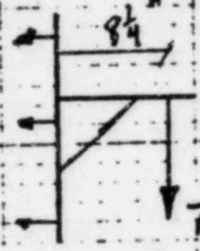
ENCLOSURE B

Sample Calculation

Determine wall loads due to Pipe Support

(Ref to as-builts on pgs 89-91)

H_2 PARGE LINE



750# (SWEC DECAL LOAD FOR 2"ϕ PIPE)
Ref. WI# 2 EES # EB0001

SHEAR/BOLT = 750#/6 = 125# ✓

ASSUME NEUTRAL AXIS

$$\frac{11.5}{2}\bar{y}^2 - 2(0.196)(\bar{y}-1.5) = 2(.196)(7.5-\bar{y}) + 2(.196)(13.5-\bar{y})$$

$$5.75\bar{y}^2 + .392\bar{y} - 7.644 = 0$$

$$\bar{y} = \frac{-b \pm \sqrt{b^2-4ac}}{2a} = 1.12$$ ✓

$$\therefore 5.75\bar{y}^2 = .392\left((1.5-\bar{y}) + (7.5-\bar{y}) + (13.5-\bar{y})\right)$$

$$5.75\bar{y}^2 + 1.176\bar{y} - 8.82 = 0$$

$$\bar{y} = 1.14"$$ ✓

$$\therefore I = \tfrac{1}{12}(11.5)(1.14)^3 + (11.5\times1.14)(.67)^2 + .392(.36^2+6.36^2+12.36^2)$$

$$= 82.88\ in^4 \quad 83.1$$

$$f_t = Mc/I = \frac{(750^{\#})(8.25")(12.36")}{82.88} = 922.7\ psi$$

$$F_{T_{top}} = 361^{\#}$$

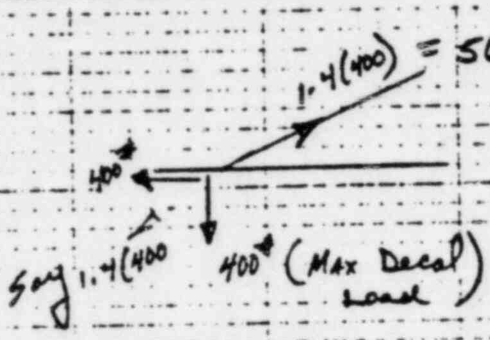
Sample Calculation

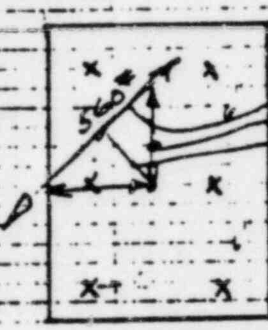
$Tension/_{Bolt_{TOP}} = 361^{\#}/2 = 180.5^{\#}$ ✓

MID BOLTS : Force $= 6.36/_{12.36}(180.5) = 92.9^{\#}/_{BOLT}$ ✓

Bottom Bolts $= .36/_{12.36}(180.5) = 5.25^{\#}/_{BOLT}$ ✓

DETERMINE BOLT LOADS DUE TO 1" line





HORIZ SHEAR = VERT SHEAR $= 396^{\#}/6\,BOLTS = 66^{\#}/BOLT$

MOMENT $= 7''(560^{\#}) = 3920''\text{-}^{\#}$ ✓

let f = bolt stress due to unit torque

$\therefore \Sigma M = 0$

$6(4.5f) + 4(7f) = 3920''\text{-}^{\#}$

$f = 71.3^{\#}/_{in}$ ✓

Max Shear due to moment $= (4.25^2 + 7^2)^{\frac{1}{2}}(71.3)$

$= 583.9^{\#} + 1.4(66)$

$= 677^{\#}$

Total Shear /BOLT = $677^{\#} + 125^{\#} = 803^{\#}$

1350

Bearing on ½ ϕ BOLT = $803^{\#} / .5"(5") = 321$ psi $< .375 \, f_m$

$= 506$ psi

but the decal loads are very conservative for example the $400^{\#}$ load assumed in both directions (1"ϕ line) is more realistically equal to:

$$DL = \frac{4.25' + 10'}{2}\left(2.17^{\#}/FT + \left[\frac{\pi(1)^{2"}}{4} \times 12" \times 64^{\#}/FT^3 \times \frac{1 FT^3}{1728 in^3}\right]\right) = 17.9^{\#}$$

SAY SSE $= 1.5(.46g)(17.9^{\#}) = 12.35^{\#}$

$$DL + SSE = 12.3 + 17.9 = 30.25^{\#} << 400^{\#}$$

[illegible] LOADS CAN [illegible] [illegible]

WHICH [illegible] BE OK.

TENSION CAPACITY OF ½"ϕ HILTI ($f'_c = 2000$ psi) (5½" LE

$= 8250^{\#} / 4_{F.S.} = 2062^{\#} >> 922.7$ psi $(.196 \, in^2) = 180$

OK

Sample Calculation

Check Effect on Wall

Check Local Block Pull-out due to tension

$T = 361^{\#}$

$\sigma_T = 361^{\#} / 8" \times 16" = 2.8$ psi OK

Check Block pullout due to shear

$V = 2(803^{\#}) / (8 \times 16) = 12.5$ psi OK

For these very conservative values there does not appear to be any local problems.

Gross effects

Inplane moment due to tension $= 361^{\#}(41.2') + 186^{\#}(10.7')$
$= 6034'\text{-}\#$
6033.4

$I \approx \frac{1}{12}(18")(11 \times 12")^3 = 3449952 \text{ in}^4$

$$\sigma_T = Mc/I = \frac{(6034 \times 12)(11/2 \times 12)}{3449952} = 1.4 \text{ psi}$$

OK

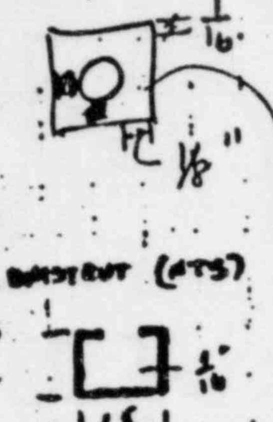
Sample Calculation

Total load est. on pipe support

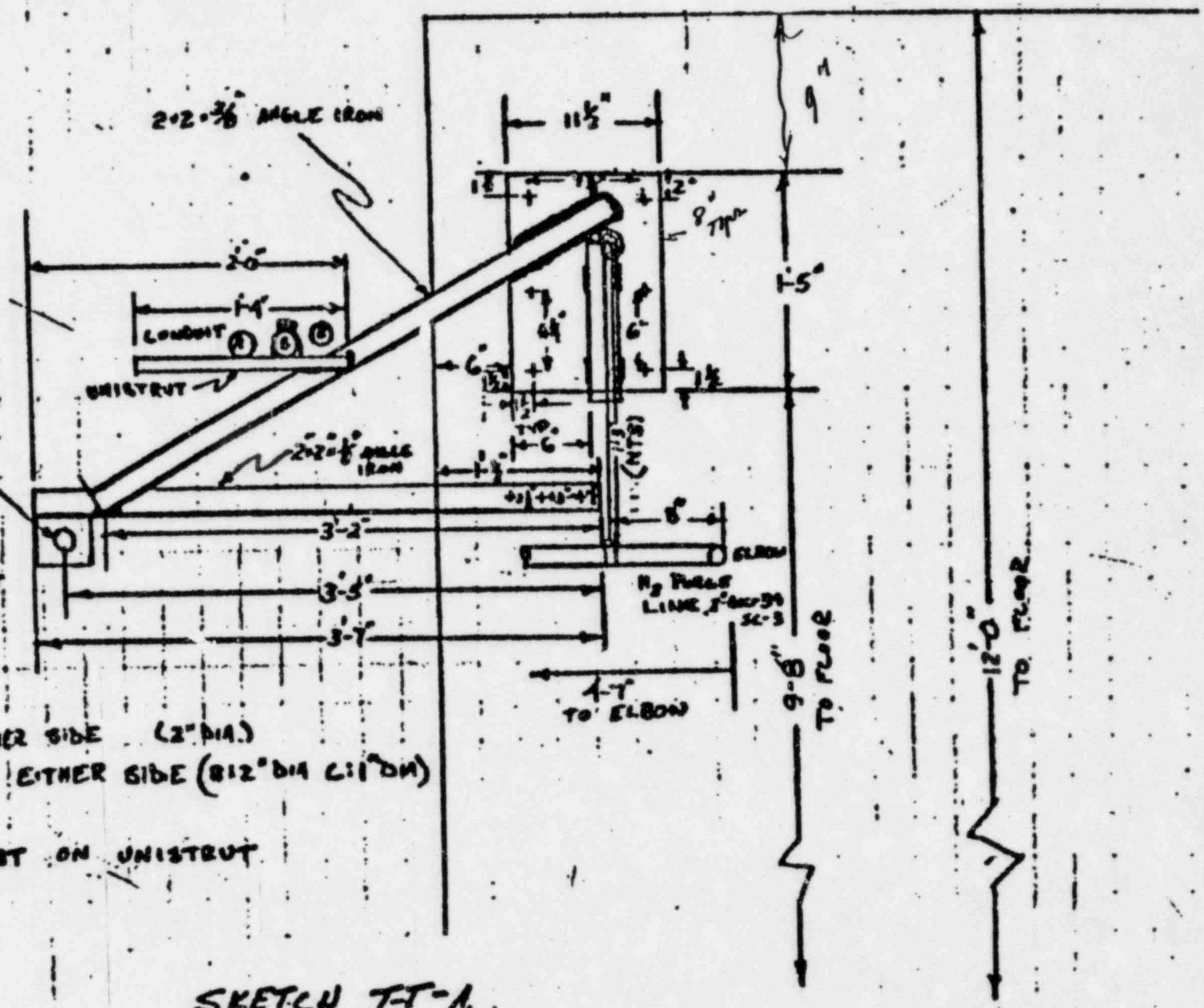
	Length	#/ft	SSE (2×peak)=1g	DL+SSE =2DL
2" purge line :	$\frac{7+9.67}{2}$	5#/ft		83.3#
1" vent line :	$\frac{4.25+10}{2}$	2.2#/ft		31.35#
Conduit :	$\frac{5+3+7+5}{2}$	≈ 3#/ft		60#
				175# ✓

SCALE 1" = 1'-0"

Sample Drawing



1 1/2 OR-38-153
SEAMLESS CARBON
STEEL EXTRA STRONG
2.17 lbs/running ft
4'-3" AND 10'-0" TO SUPPORT
EITHER SIDE



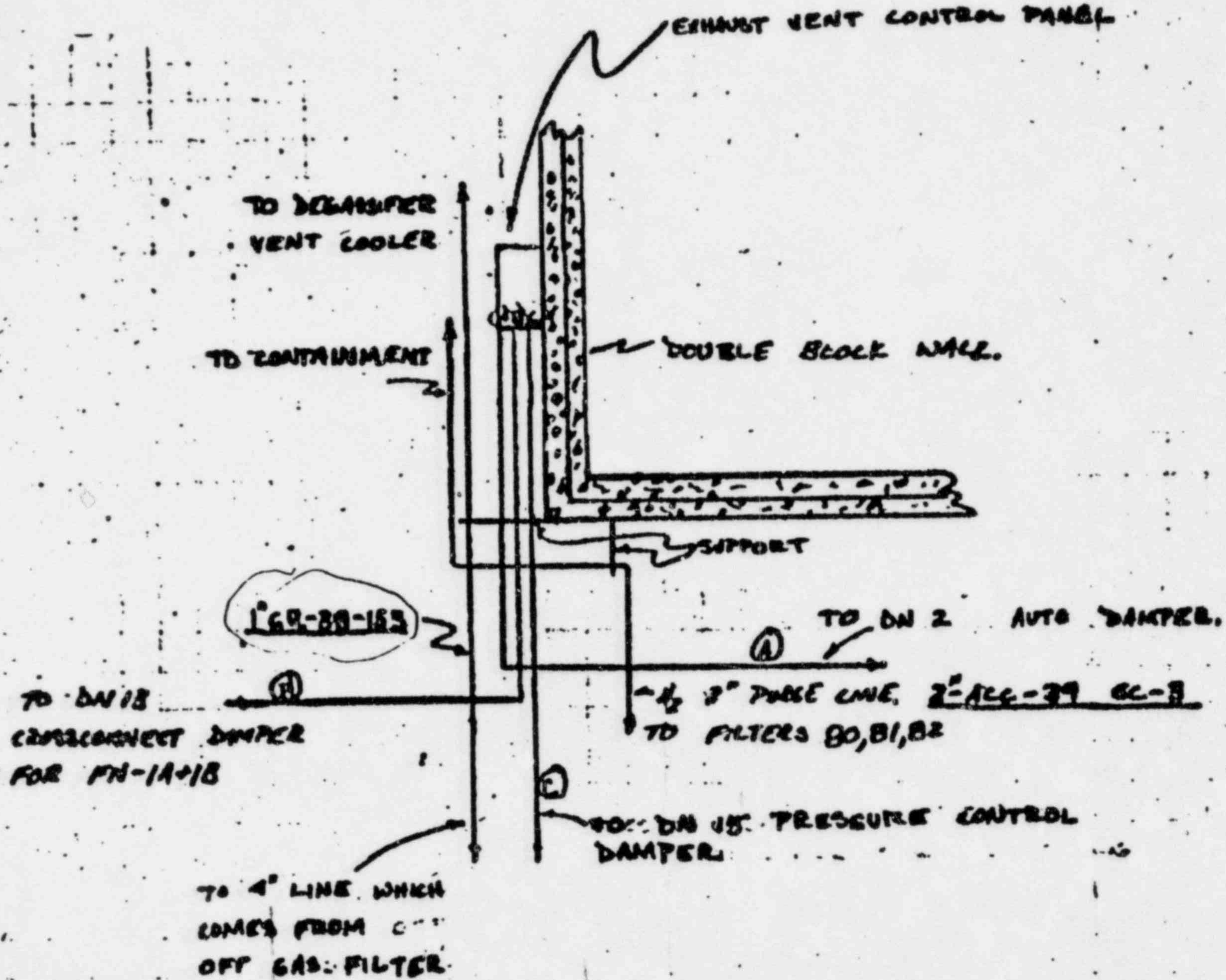
SUPPORTED
CONDUIT A 5'-0" AND 3'-0" EITHER SIDE (2" DIA.)
SUPPORTED
" B&C 5'-0" AND 7'-0" EITHER SIDE (B 1/2" DIA C: 1" DIA)

NOTE "C" DOES NOT REST ON UNISTRUT

SKETCH T-T-1.

71B WALL SECTION

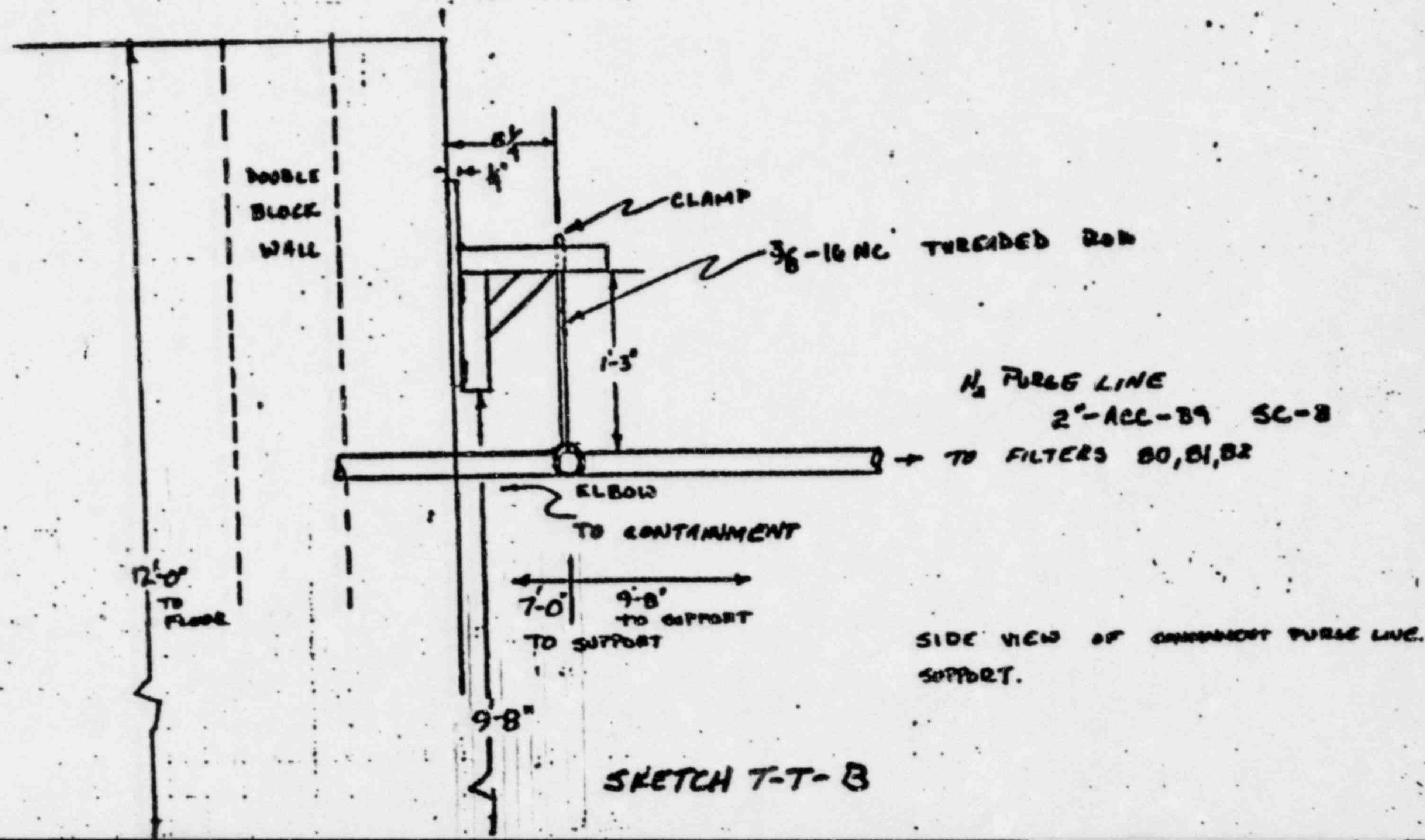
Sample Drawing



SKETCH T-T-C

SUPPORT ON THE WALL SECTION
NOT TO SCALE.

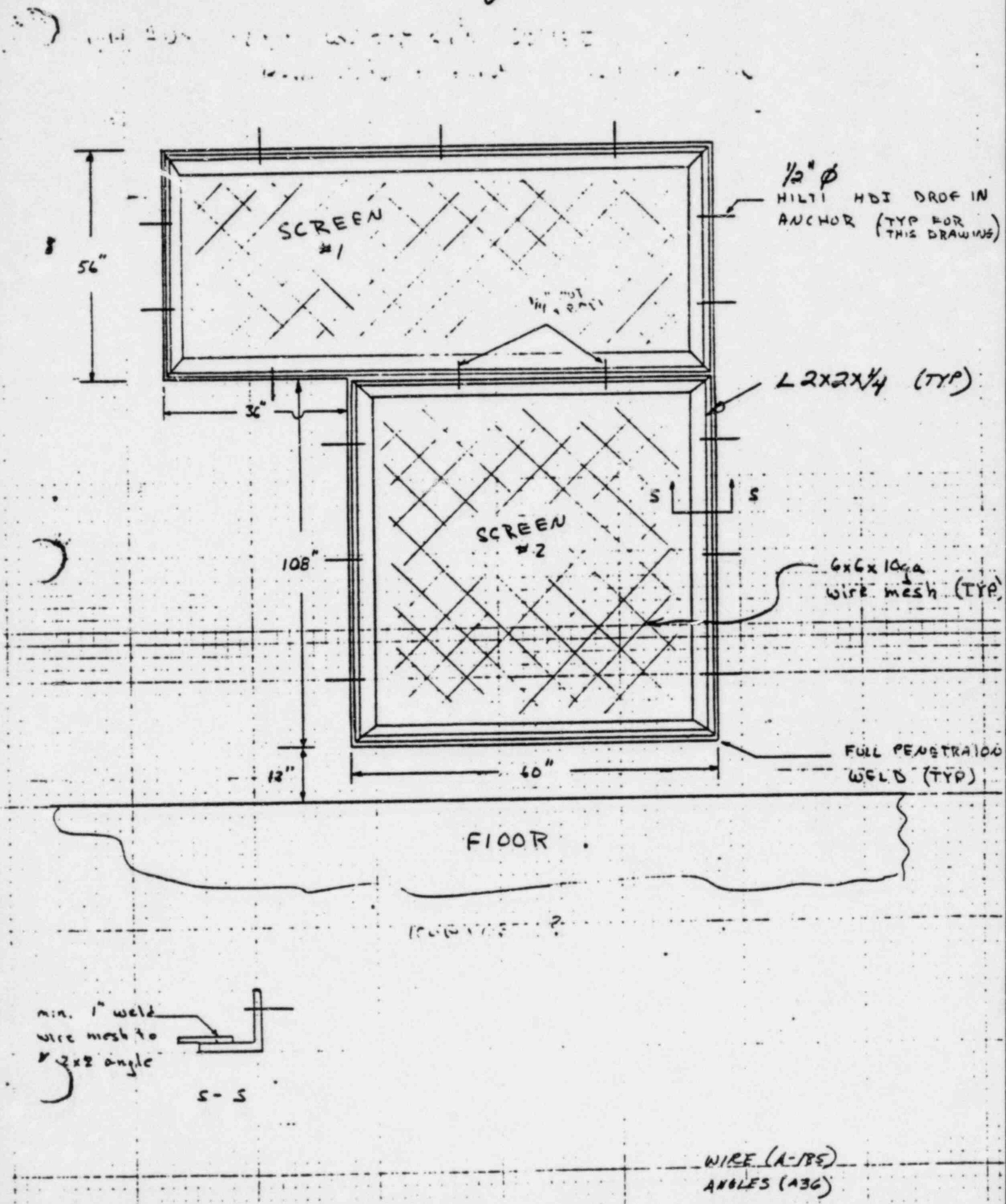
Sample Drawing



SKETCH T-T-B

ENCLOSURE C

Sample Drawing



ENCLOSURE D

FJG

Control Room East Wall col 9-10 (EL 21'-0")

1) **Determine I_H + I_V of 8" as-built block** (see p80)

$$I_H = \frac{1}{12}\left((16")(7.5)^3 - (4.5+4.5+2(.5))(4.25)^3\right) = 498 \text{ in}^4/\text{block}$$

$$= 374 \text{ in}^4/\text{ft}$$

$$I = \Sigma I + Ad^2 = \tfrac{1}{12}(12)(1.75)^3 + (12)(1.75)(3)^2 + \tfrac{1}{12}(12)(1.5)^3 + (12)(1.5)(2.9)^2$$

$$= 350 \text{ in}^4/\text{ft}$$

all assumption based on 12" "as built" block pertain to 8" "as built" as well

$A = 16(7.5) - 10(4.25) = 77.5 \text{ in}^2/\text{block} = 58.1 \text{ in}^2/\text{ft}$

2) **Determine wall's frequency** (HORIZ. SPAN)

$$\omega = \frac{(1)^2 \pi^2}{(7.33 \times 12)^2} \sqrt{\frac{(1000)(1350)(350)(32.2)(144)}{55}}$$ Ref #10 pg 154

55 #/ft

$$\omega = \frac{\pi^2}{(7.33 \times 12)^2}(799585) = 254.6$$

$$f = \frac{\omega}{2\pi} = \frac{254.6}{2\pi} = 40.5 \; ; \; T = 0.0247$$

$$f_{1/2} = \frac{\omega_{1/2}}{2\pi} = \frac{127.3}{2\pi} = 20.3 \; ; \; T = 0.05$$

3) allowing T to vary from 0.02 to 0.05 peak accel. will be 0.47g Ref. #4 p 44

FJK

3) cont. — to allow for higher modes let $\ddot{u} = 1.3$(peak accel.)

$$\ddot{u} = 1.3(.47) = 0.61g$$ ✓

4) $\bar{M} = 1/8\,(0.61)(55)(7.33)^2 = 225.7'^{\#}$ ✓ ✓

5) $\sigma_T = Mc/I = \dfrac{(225'^{-\#})(12''/ft)(3.75'')}{350\ in^4} = 29\ ^{\#}/in^2 < 32\ ^{\#}/in^2$ ✓ ✓

5a) $\sigma_V = .61(55)\left(\frac{7.33}{2}\right) / (1)(1.5)(12) = 6.83$ psi ✓ ($V = 123^{\#}/ft$)

6) CHECK STRESS DUE TO E-W DEFLECTION (out of plane)

$\Delta_{E-W} = .003\left(\frac{10'}{15'}\right) = .002''$ Ref # ✓

$$M = \frac{3EI\Delta}{l^2} = \frac{3(1000)(1350)(6374)(.002)}{(10\times12)^2}$$ Ref #9 p. II-A-4 (Hinged, fixed & ignoring Shear deformation)

$M = 210\ ''^{-\#}/ft$ ✓

$$Mc/I = \frac{(210)(3.75)}{374} = 2.10\ ^{\#}/in^2 < 16\ ^{\#}/in^2 - ok$$

7) CHECK TENSILE STRESS DUE TO N-S DEFLECTION ✓

$\Delta_{N-S} = .00133$ in Ref # 7 pg 2

$$M = \frac{3EI\Delta}{l^2(1+\phi_y)}$$ Ref #9 p II-A-4 (Hinged/fixed!)

$$= \frac{3(1000)(1350)(205580)(.00133)}{(10\times12)^2\left(1 + \frac{3(1000)(1350)(205580)}{(540000)(425)(10\times12)^2}\right)}$$

$= 61,424.7\ ''^{-\#}$ ✓

$I = \Sigma A d^2 = 2(58)\left(\frac{7.33}{2}\right)\left(\frac{7.33\times12}{4}\right)$

$= 205,580.7\ in^4$ ✓

$A_y = (58\ in^2/ft)(7.33\ ft)$

$= 425.7\ in^2$ ✓

) cont.

$$\sigma_T = Mc/I = \frac{(61424)(\frac{7.53}{2} \times 12)}{205580 \text{ in}^4} = 13.1 \ \#/in^2 < 16 \text{ psi}$$

8) CHECK IN PLANE SHEAR

using ZPA = 0.30 g ✓ Ref # 4, pg 45

$V/ft = (55 \#/ft)(10 ft)(0.30g) = 165 \#/ft$

$V/A = 165 \#/ft / 58 \ in^2/ft = 2.84 \ \#/in^2$ — ok

$T \text{ or } C = \frac{(55\#/ft)(7.33ft)(.3g)}{\sim 3.5''(12'')} = 2.9 \text{ psi}$

9) CHECK SHEAR STRESS DUE TO N-S DEFLECTION (IN PLANE)

$\Delta_{N\text{-}S} = .00133$ in ✓

$$P = \frac{3EI\Delta}{l^3(1+a_y)} = \frac{3(1000)(1350)(205580)(.00133)}{(10\times12)^3 \left(1 + \frac{3(1000)(1350)(205580)}{(540000)(425)(10\times12)^2}\right)}$$

$= 512.8 \#$ ✓

$$\sigma_v = P/A = \frac{512.8\#}{425 \text{ in}^2} = 1.2 \ \#/in^2$$

10) Combined SHEAR Stress due to Inertial and displacement loading

✓

IN-PLANE SHEAR $= 2.84 \#/in + 1.2 \#/in = 4.04 \ \#/in^2$ — OK

TENSION DUE TO INPLANE = 13.1 psi + 2.9 psi = 16 psi ~ = ACI

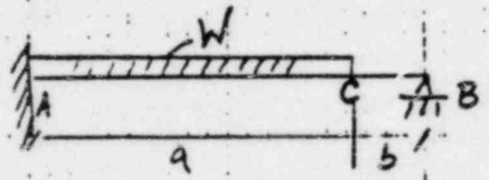
Out of Plane = 29 psi (T ⊥ BED) ; $T_{\parallel \text{BED}}$ = 21 psi

11) Conclusion: WALL APPEARS BORDERLINE FOR OUT OF PLANE LOADS AND CAPACITY OF DOOR FRAME TO ACT AS A SUPPORT IS QUESTIONABLE ∴ DESIGN SOUTH END SUPPORT TO INSURE AGAINST COLLAPSE IN S-E DIRECTION (TOWARD MCB)

DESIGN SUPPORT FOR SOUTH END OF EAST BATHROOM WALL

load = 123#/ft

Design as propped cantilever



$M_A = -\frac{Wa}{8}(2-n)^2$ where $n = a/L$

$= \frac{123^{\#/'}(10')(10')}{8}\left(2-{}^{10}/_{12}\right)^2 = 2093^{1-\#}$

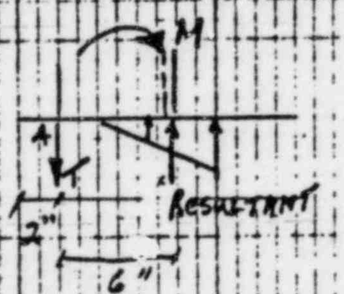
$R_A = \frac{W}{8}\left[8-n^2(4-n)\right] =$ 892# ← check

$R_B = \frac{Wn^2}{8}(4-n) =$ 338#

Check Capacity of ∠6×6×½ ; S = 4.61

$f_b =$ 2093×12/4.61 = 5.45 ksi OK

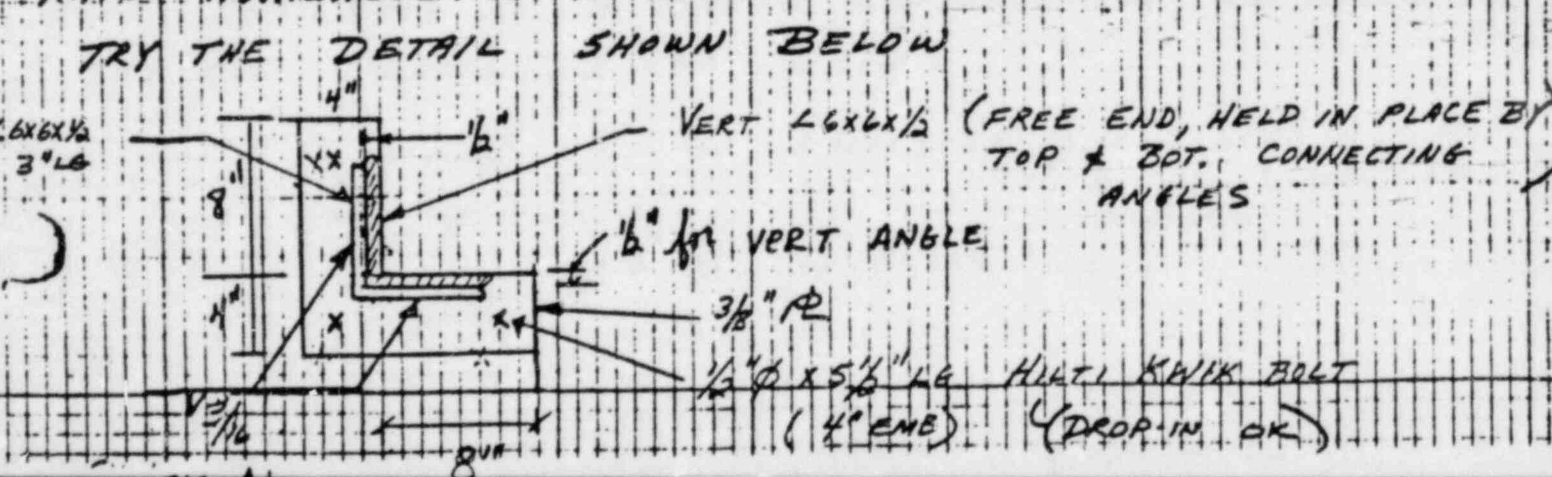
Check Base ℞



$\Sigma M_A = 0$ $R(6") = M = 2.093^{1-K}$

$R = 1.186^K = T$

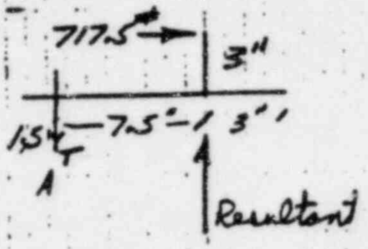
THIS VALUE IS TOO LARGE FOR THE ANCHORS WE HAVE AVAILABLE ∴ A PINNED SUPPORT MUST BE DESIGNED TRY THE DETAIL SHOWN BELOW



DESIGN REVISED BASE CONNECTION

$R_A = 10'(123^{\#}/ft)\left(1 - \frac{10}{24}\right) = 717.5^{\#}$ ∴ $R_B = 1230 - 717.5 = 512.5^{\#}$

ASSUME WELDED L6X6X½ IS 3" HIGH



$\Sigma M_A = 0 \quad 717.5^{\#}(3") = R(7.5")$ ✓

$\therefore R = T = \frac{717.5(3)}{7.5} = 287^{\#}$

ASSUME FACTOR OF 3 to ACCOUNT FOR Prying ACTION

∴ TENSION = 861# SHEAR = 717.5/3 = 239# ✓

TRY ½"φ X 5½" LG HILTI w/ 4" EMB ✓

Tension Cap (f'_c = 3000psi) = (7000 + 7275 + 9450 + 11225)/4 = 8737.5#

Shear Cap = (7441 + 8897 + 10232 + 8316)/4 = 8722# ✓

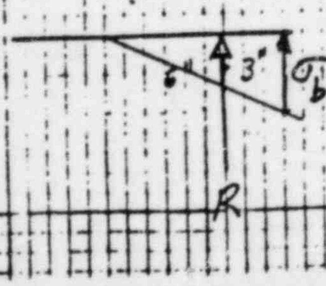
Bolt Stress: 861/8737.5 + 239/8722 = .099 + .027 = .126 < .2 OK ✓

Size PL

Tension side $M = 861^{\#}(2") = 1722^{"-\#}$ ✓ $S_{req} = 1722/32.4ksi = .05$

$S_{\frac{1}{2}"\times 4"} = \frac{1}{6}(4)(.5)^2 = .167$ ✓ OK TRY 3/8" PL S = .093 in³ OK

Comp. Side

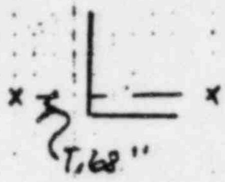


R = 861# assume 6" PL WIDTH

$R = \frac{1}{2}(6)(\sigma_b)(9) \Rightarrow \sigma_b = 861^{\#}(2)/54in^2$
$= 31.9 psi$ ✓

$M_{@ angle} = (3"\times 6")(31.9psi)(1.5")(2/3) + \frac{1}{2}(3\times 6)(\frac{1}{3}\times 31.9)(2$
$= 765.6"-\#$ ✓ 3½"D mL

Check Req'd weld (ASSUME 3/16" FILET)



$A_{weld\ THROAT} = (3/16")(.707)(\underbrace{5"+5"}_{conservative}) = 1.33\ in^2$ ✓

$I_x = \frac{1}{12}(5)(3/16 \times .707)^3 + .66(1.68)^2 + \frac{1}{12}(3/16 \times .707)(5)^3 + .66(5-1.6$

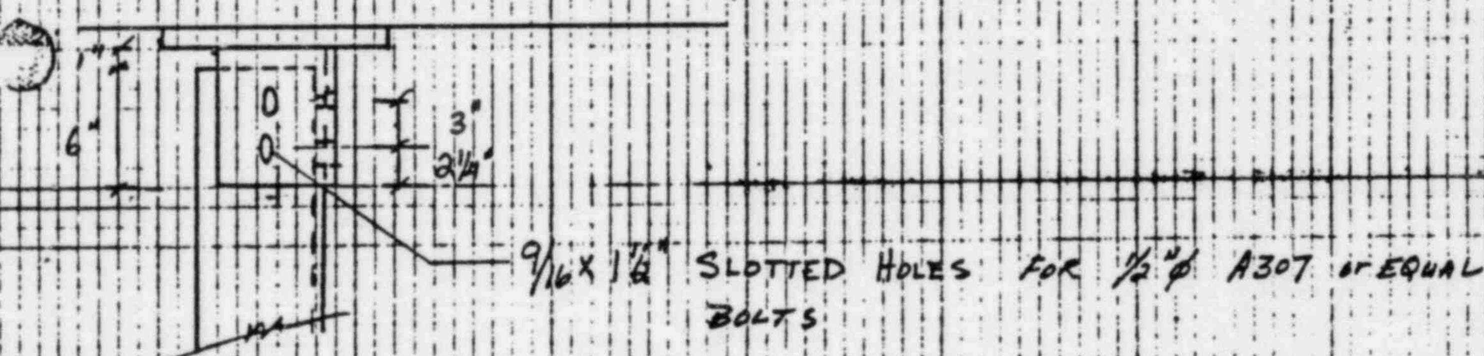
$= 10.5\ in^4$ ✓

Shear: $717.5^{\#} / 1.32\ in^2 = .54\ ksi$ ✓

Tension: $3(717.5)(5") / 10.5 = 1.02\ ksi$ ✓ ∴ 3/16" filet OK

conservative

Design Top Connection



N-N

Reaction = $512.5^{\#}$ $M_{max} = 512.5(7") = 3587.5^{"\#}$ ✓

Assume 3/16" FILET

SHEAR: $512.5^{\#} / 1.32\ in^2 = .39\ ksi$ ✓ O.K

TENSION: $3587.5^{"\#}(5") / 10.5\ in^4 = 1.71\ ksi$ ✓

Sample Drawing