TECHNICAL REPORT C-78-3

STRUCTURAL INTEGRITY OF BRICK-VENEER BUILDINGS

by

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Vicksburg, Miss. 39180
Floodproofing individual homes is an important aspect of the total solution of flood damage reduction. This report gives insight into the structural resistance of brick-veneer walls subjected to hydrostatic water loading. There are many variables affecting the response of a brick-veneer wall; therefore, the approach of this study was to obtain limited experimental data by testing three walls, analyze these data, and compare them to analytical solutions.

(Continued)
20. ABSTRACT (continued).

Wall 1 was typical of the end wall of a house (no roof rafter or ceiling joist restraints). After about 2 ft of water, the wall deflections (order of magnitude of $10^{-3}$ in.) increased drastically for small increases in water depth and failed at about 2.4 ft of water.

The analytical results for Wall 1 compare favorably with the experimental results.

Wall 2 was constructed just as Wall 1 except it had a 3-ft x 6-ft 8-in. door in its center. The significant factors as indicated by the experimental results of Wall 2 are:

a. In general, the wall deflected forward toward the water loading for low water loads; then, backward as the water depth became greater than 0.8 to 1.6 ft.

b. The wall deflections were very small ($10^{-3}$ in.) until 2 to 2.4 ft of water at which time the wall began to deflect drastically backward for small increases in water depth.

c. Wall 2 (with door opening) deflected more forward but about the same backward as Wall 1.

Wall 3 was constructed just as Wall 1 except it included roof rafter and ceiling joist restraints.

The significant findings from the experimental results of Wall 3 are:

a. In general, the roof rafter and ceiling joist restraints decrease the movement of the wall toward the water loading.

b. The roof rafter and ceiling joist restraints are sufficient to cause a change in the failure mechanism from that which was experienced in Walls 1 and 2. The failure mechanism for Walls 1 and 2 was deflection and failure of the brick wall. The failure mechanism for Wall 3 was beam failure of the studs and a resulting collapse of the brick wall.

c. The deflection of the brick wall begins to increase rapidly with water depth after about 1-1/2 ft but the increase is not as great as was experienced by Walls 1 and 2. This is indicated by the fact that the wall did not collapse until about 57 in. of water loading.

d. Even though the wall can withstand greater water depths, it fails suddenly and totally when the stud wall fails.

Based on the test results the upper bound for the failure of Walls 1, 2, and 3 was arbitrarily established at approximately 2 ft of water depth or at a deflection of 0.01 in.

The structural integrity of brick-veneer Walls 1 and 2 was completely lost at about 2.4 ft of water loading. The type restraint did cause a change in the total capacity of the wall to resist hydrostatic loading because Wall 3 did not collapse until 57 in. of water loading had been attained.

It is true that the finishing material on the inside of the studs will help strengthen the walls but this advantage is offset somewhat because no wave or debris loading was imposed on the walls in these tests.

It is felt that without modifications a brick-veneer wall cannot be expected to withstand more than about 2 ft of hydrostatic pressure and if some safety factor is desired a limitation of 1-1/2-ft of water should be imposed.
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The experimental and analytical investigation of the structural integrity of existing brick-veneer walls was performed for the Corps of Engineers, Lower Mississippi Valley Division (LMVD) by the Concrete Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES).

The contract was monitored by Mr. Lawrence Flanagan of the LMVD. Mr. Flanagan worked very closely with the project and was very helpful in planning and conducting the study. His help and knowledge in the field were greatly appreciated.

The study was performed under the direction of Messrs. B. Mather, J. M. Scanlon, G. C. Hoff, and J. E. McDonald, Concrete Laboratory. The instrumentation work was performed by Mr. Dale Glass. The tests were conducted by Dr. Carl E. Pace. The report was prepared by Dr. Carl E. Pace and Mr. Roy L. Campbell.

The Commander and Director of WES during the conduct of this test program and the preparation and publication of this report was COL John L. Cannon, CE. Mr. F. R. Brown was Technical Director.
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U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

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<td>inches</td>
<td>2.540000 E-02</td>
<td>metres</td>
</tr>
<tr>
<td>feet</td>
<td>3.048000 E-01</td>
<td>metres</td>
</tr>
<tr>
<td>pounds (mass)</td>
<td>4.535924 E-01</td>
<td>kilograms</td>
</tr>
<tr>
<td>pounds (force)</td>
<td>4.448222 E+00</td>
<td>newtons</td>
</tr>
<tr>
<td>pounds (mass) per cubic foot</td>
<td>1.601846 E+01</td>
<td>kilograms per cubic metre</td>
</tr>
<tr>
<td>pounds (force) per square inch</td>
<td>6.894757 E-03</td>
<td>megapascals</td>
</tr>
<tr>
<td>tons (force) per square foot</td>
<td>9.576052 E-03</td>
<td>megapascals</td>
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<tr>
<td>feet per second</td>
<td>3.048000 E-01</td>
<td>metres per second</td>
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PART I: INTRODUCTION

Necessity for Study

1. In the United States, many buildings and other properties are located within flood plains and are, therefore, a potential source of flooding, expense, and disruption of life. For this reason, the Federal Government has invested over eight billion dollars in flood control projects since 1930. This represents only part of the total Government cost when one considers the subsidies for personal loss and insurance coverage.

2. The flood control projects have not provided the total solution and now a more comprehensive approach to flood damage reduction is being implemented. The evaluation of structural integrity and necessary structure modifications to reduce flood damage are important parts of the total program.

3. Floodproofing individual buildings is one of several alternatives (channel improvements, levees, etc.) for reducing flood damage. Estimates and comparisons of cost for the alternatives can only be obtained if adequate knowledge of the necessary engineering and construction associated with them are known. At present, the phenomenon of floodproofing individual buildings is not understood well enough to evaluate adequacy or to determine floodproofing costs. In many cases unless the structure is made adequate more damage can occur to the floodproofed building than would have occurred to it without floodproofing.

4. The knowledge of the phenomenon and subsequent protection will also allow the correct emphasis on floodproofed construction and building modifications in the building codes. In some populated areas it may be more feasible to build and floodproof certain structures to withstand specific flooding than to restrict or eliminate construction in the flood plain.

5. The ability of structures to withstand flooding should be known well enough that proven engineering concepts can be used to determine remedial measures for inadequate construction before floodproofing begins.
6. This study is necessary for two reasons:

   a. To acquire the information needed for evaluating structure adequacy to withstand specific flood loads and to define needed modifications for structures that are inadequate.

   b. To learn the phenomena associated with buildings subjected to flood loadings in order to evaluate the economy of various alternatives for the protection of buildings from flood damage.

Objective

7. The objective of this study is to determine the allowable depth of flood water that will not adversely affect the structural integrity of a brick-veneer wall and to determine necessary modifications for inadequate brick-veneer construction in order that it can be effectively floodproofed. This report defines the structural problems as related to floodproofing existing brick-veneer dwellings and later reports will consider other aspects such as waterproofing the building.

Approach

8. In floodproofing individual homes, there are a number of variables such as wall length, door and window openings, etc., which affect the response of the brick-veneer wall. It would be too expensive and time-consuming to test walls varying each parameter separately and define the general response of the wall by experimentation only. A more feasible alternative is to study the walls both experimentally and analytically. Analytical solutions for the experimental situations can be compared to the test results and decisions made as to the accuracy of the analytical results in relation to the testing and practical problem application. If the analytical results are sufficient, they can be used efficiently and economically to define the wall response as affected by specific variables.

9. The approach will then be to obtain limited experimental data, analyze these data, and compare them to analytical solutions and thereby
validate the analytical method. The analytical method can then be used to define the response of any brick-veneer wall which is subjected to water loads.

**Scope**

10. The scope of this work is limited to the structural response of the common brick-veneer home construction. Even though the study is for brick-veneer construction, it will give insight into the floodproofing of individual homes or buildings in general.
11. Experimental testing is required to give results which will help define the integrity of brick-veneer walls subjected to water loads. The construction of the common brick-veneer wall is such that its response to load is dependent on many variables and can best be studied experimentally and analytically. For the experimental study, three brick-veneer walls were built and tested.

12. The first wall was typical of an end wall of a house. In supporting loads it was the most critical because the top plate on the stud framing did not have roof rafter and ceiling joist restraints which could transfer their resistance through the wall ties to the brick-veneer wall. The general plans for the wall construction are given in Figure 2.1. Wall 2 and Wall 3 differ from Wall 1 as follows:

   a. Wall 2 - Had a 3-ft door in its center.*
   b. Wall 3 - Had roof rafter and ceiling joist restraints.

The addition of the door is indicated in Figure 2.1 and is shown in Figure 2.4. The roof rafter and ceiling joist restraints are presented by photograph (Figure 2.5).

13. The walls were built to realistically represent those which actually exist in typical home construction. The construction was made as simple as possible with few variations between each of the three walls. This was done so that the test results could be compared and the differences in wall response clearly delineated as caused by specific variations in construction.

14. In designing the prototype walls, decisions were made concerning geometry, boundary conditions, wall restraints, and loadings. The overall dimensions of the wall were selected as 8 ft high by 26 ft long. In reality, flood waters would be on all sides of the house. The walls to be studied experimentally were subjected to restraints and loading conditions similar to those which would exist on a wall which is an integral part of a house. If the house is sealed from water penetration,

* A table of factors for converting U. S. customary units of measurements to metric (SI) units is presented on page 7.
the forces against opposite walls are the same (at present, forces caused by debris and the flow of water are neglected) causing no lateral deflection of roof rafters at the intersection of the gable. In a like manner, there would be only small deflections of the ends of the walls; their movements would be due only to structure deformations. To simulate the real situation stub walls were constructed at $90^\circ$ to the wall which was to be loaded. To represent the effect of the perpendicular wall framing, the end studs were braced so they would be restrained perpendicularly to the wall being tested.

15. Wall 1 was constructed with wallboard, stud framing, brick-veneer, and clips between brick and studs utilizing common homebuilding techniques. The wallboard was left off of Wall 2 and Wall 3 in order to observe cracking or other response characteristics that could be seen from the back of the brick wall.

16. The stud framing, wallboard, and wall clips as constructed for Wall 1 are presented in Figure 2.2. The wall clips were spaced 32 in. on centers with horizontal direction (on every other stud) and between every fifth layer of brick in the vertical direction for all three walls. The overall view of Wall 1 after it was constructed is given in Figure 2.3.

17. Wall 2 and Wall 3 were constructed as has been discussed and are presented in Figures 2.4 and 2.5, respectively.

18. The walls were tested by a horizontal water load which was contained by a trough and plastic liner (Figure 2.6). The test loading was somewhat less than that which would actually be encountered because no impact forces due to wave action, flowing water, vibration, or debris loads were imposed; the walls were merely statically loaded. The water depth in front of the wall was increased at about 1 to 2 ft per hour.

19. As the water depth increased, deflections of the wall were monitored defining the deflected response of the walls. The gages used were Linear Variable Differential Transformers (LVDT's), (Figure 2.7) which were held stationary (Figure 2.8) by a 1/2-in.-diameter rod which was clamped to an independent bracing system. The gage bracing had to be very rigid to eliminate motion caused by factors such as wind. This was true because deflections of very small amplitude (on the order of $10^{-3}$ in.

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or less) are very significant in brick-veneer wall response. Plastic wind-screens are shown in Figures 2.4 and 2.5. The gage bracing system for Walls 1 and 2 is shown in Figure 2.8. That for Wall 3 was very similar.

20. The LVDT gages were placed so as to define vertical and horizontal lines of deflection which in turn defined the overall deflection of the walls. The gage locations are given in Figures 2.9, 2.10, and 2.11 for Wall 1, Wall 2, and Wall 3, respectively. The stud and wall tie locations are given in Figures 2.12 and 2.13 for Walls 1 and 2, respectively. The stud, wall tie and roof rafter and ceiling joist locations for Wall 3 are presented in Figure 2.14. The stud framing deflection was measured for Wall 2 and Wall 3 and the deflection of the top plate, roof rafters, and ceiling joists was monitored on Wall 3.

21. Since the wall clips were shifted somewhat to one end of the walls, that end would experience a little different deflection than the other. This did not drastically affect the deflection on the side of the wall to which the clips were shifted; therefore, only two gages were positioned on that side to monitor the effect. It would have been desirable to monitor both sides of the wall with a dense population of gages but the number of gages available for use was limited.

22. Wall 1 was tested without roof rafter and ceiling joist restraints to represent the worst condition of response. This representation of the end wall in a house neglected the restraints of ceiling and the stiffening of finishing materials (paneling, sheet rock, etc.) on the inside of the studs since this restraint and stiffening is questionable.

23. Wall 2 was tested to see how a 3-ft door opening affected the wall response.

24. Wall 3 gave some indication of how roof rafter and ceiling joist restraints affect the wall response.

25. The test results from these three walls should be sufficient to compare and evaluate analytical results.
Figure 2.1 Wall plans for testing and studying the feasibility of floodproofing existing brick-veneer dwellings.
Figure 2.2 Stud wall, wallboard, and wall tie construction.
Figure 2.3 Overall view of Wall 1.
Figure 2.4  Overall view of Wall 2.
Figure 2.5 Overall view of Wall 3.
Figure 2.6 Trough and plastic liner containing water which causes hydrostatic pressure on the brick veneer wall.
Figure 2.7 LVDT gage.
Figure 2.8 Independent gage bracing system.
Figure 2.9 Wall 1 - elevation view of gage locations.
Figure 2.10 Wall 2 - elevation view of gage locations.
Figure 2.11 Wall 3 - elevation view of gage locations.
Figure 2.12  Stud and wall tie locations, Wall 1.
NOTE: MIDDLE STUDS 16" ON CENTER

Figure 2.13 Stud and wall tie locations, Wall 2.
Figure 2.14  Stud wall tie and roof rafter and ceiling joist locations, Wall 3.
PART III: PLANNING OF ANALYTICAL COMPUTATIONS

26. In the analytical study, the solutions for the deformations of the experimental wall as tested can be obtained by using the finite element method. The material properties, geometry, boundary conditions, and loading had to be known in order to model and get the solution for the brick walls. Plate elements can be used to model the brick wall if their material properties (modulus of elasticity (E), shear modulus (G), and Poisson's ratio (μ)) are known. After the analytical solution was obtained, the experimental results were used to compare and evaluate the analytical method.

27. In the determination of material properties it was concluded that tests on brick or mortar individually would not give the needed properties because the wall was made of a composite of the two materials. Tests were conducted on sections of brick and mortar laid as in the walls to determine the composite properties. The following is a discussion of how the composite properties of the brick wall were obtained.

28. The modulus of elasticity was obtained by testing sections of a brick wall in third-point loading (Figure 3.2) and monitoring the maximum deflection versus load. In the equation

\[ \Delta_{\text{max}} = \frac{Pa}{24EI} (3a^2 - 4a^2), \]

the only unknown is "E." "I" is the moment of inertia of the section being tested. The remaining terminology in the above equation is given in Figure 3.1.

![Figure 3.1. Terminology for third-point loading.](image-url)
A number of tests were performed on different specimens of various length and width. The "E" values were calculated and averaged; the average value obtained and used in the analytical computations was $5.7 \times 10^6$ psi.

29. Some testing was done to determine the feasibility of obtaining "G" from test sections of wall (see Figure 3.3). Without elaborate test setups, it was not practicable to obtain accurate "G" values for the brick wall. The load-deflection curve could be measured accurately but complications arose in the following:

a. Rotation must be eliminated by restraining the boundaries of the sheared section (pure shear must be approximated in the mortar joint).

b. In the experimental setup (Figure 3.4) the thickness of the material ($X_1$) is too small for accurate determination. The shear tests were not used to obtain "G" but they did give some indication of the shearing strength of the brick wall at the mortar joints. The test wall was exposed to moment as were those sections tested in the laboratory. The shearing strength is approximately 10 psi.

30. From past experience, it was concluded that the finite element solution for the brick wall would not be very sensitive to Poisson's ratio and shear modulus; therefore, Poisson's ratio was estimated as 0.3 and "G" then calculated from "E" and "μ" by the equation $G = \frac{E}{2(1+\mu)}$. "G" was calculated to be $2.2 \times 10^6$ psi.

31. As the brick walls were constructed, samples of mortar were taken at the third, half, and two-third's positions of construction. Specimens taken were eighteen 2-in. by 4-in. and six 6-in. by 12-in. cylinders. The 6- by 12-in. specimens were tested at 28 days and the average material properties for the mortar are as follows:

$f_c = 1100$ psi
$E_v = 0.8 \times 10^6$ psi
$\mu = 0.11$

These values were not obtained to be used in the analysis but were obtained to document the characteristics of the mortar used in constructing the walls.
32. The analytical solution can be solved in two ways as follows:
   a. By a two-dimensional finite-element model using material properties, boundary conditions, applied loads, and wall clip restraints in calculating the deflection of the brick wall. The wall-clip restraints can be determined from load-deflection tests of clips and the measured deflection of clips during the experimental test.
   b. Model the total three-dimensional structure in the finite-element solution.
      (1) Use plate elements, material properties, and boundary conditions for the brick wall.
      (2) Use truss elements with appropriate material properties to represent the wall clips.
      (3) Use beam elements and appropriate material properties to represent the studs. Boundary conditions as obtained from experimental tests would be used for the restraints of studs, roof rafter and ceiling joists.

33. The first procedure was tried and reasonable results were obtained for specific wall responses, but after careful study it was found that the loading from clip to clip oscillated dramatically and this two-dimensional solution was not accurate enough for obtaining general results. This left the second method for analytical computations as the only possible alternative for accurate computations. These problems and results will be discussed in detail in Part IV. The two methods are mentioned here to make clear the restraints which are needed for analytical solutions. The remainder of this part of the report will present the results of restraint tests and explain how they were obtained.

34. Besides the material properties of the brick walls, there are three types of restraints which must be considered. These restraints are:
   a. Wall clips
   b. Roof rafter and ceiling joists
   c. Connection of studs to base plate

Walls 1 and 2 had only the wall clips and stud connections to base plate restraints. Wall 3 had the additional restraints of roof rafters and ceiling joists.

35. First, consider the restraint of roof rafter and ceiling joist at its connection to the top plate of the stud wall. The reason the
load-deflection curve for this connection needs to be known is:

a. In a three-dimensional model of the brick-veneer wall this boundary condition or restraint must be known and used as input in the analytical solution.

b. To show the strength of this connection which can be related to imposed water loading on the wall.

c. From the load-deflection relation and the measured deflection of the top plate, the load actually transferred by the wall clips to the top plate can be determined.

The variables which can affect the strength of this restraint are:

a. Kind of lumber used.

b. Way the connection is nailed.

c. Slope of roof rafter.

d. Amount of dead load on top of the roof rafter producing friction at the connection.

A test setup for roof rafter and ceiling joist restraints is presented in Figure 3.5.

36. The roof-rafter slope and dead load made no noticeable difference in the strength of the roof rafter and ceiling joist restraint (Figures 3.6 and 3.7). The connection was nailed in a standard manner (Figure 3.8) with reasonable positioning of nails into the member. Later, tests were conducted with nails driven for maximum penetration into the top plate as well as going through enough of the roof rafter and ceiling joist to hold. Connections with nails placed for maximum penetration were slightly stronger but as long as the nails were placed in a reasonable manner the difference in the strength of the restraint was slight. A comparison of Figure 3.6 and Figure 3.7 shows that there is some difference in the strength between using hard pine or spruce lumber but this difference is small. The pine caused an increase in slope of the load-deflection curves. The maximum restraint of this connection is about 1200 to 1500 lbs.

37. The restraint due to wall ties was determined. There are several types and thicknesses of wall ties (Figure 3.9) but the two most commonly used (22 and 28 gauge) were tested. The test setup is presented in Figure 3.10. The clips have a wide variation in load-deflection
In general, the 22-gauge clips have a maximum strength of 100 to 200 lb (Figure 3.11). The 28-gauge clips have a maximum strength of about 40 to 60 lb (Figure 3.11). The variation of the strength of the 28-gauge clips is greater than that of the 22-gauge clips. Mortar that collects on the wall tie is the main factor that affects their load-transfer capacity. Figure 3.11 shows that the load transfer increases drastically if mortar has caught on the wall clip between the stud and brick wall. The load transfer to the stud, because of mortar, may be at least as high as 750 lb. The variation of the amount of mortar on a clip may be from zero to a considerable amount. At first, it seems that the individual load-deflection curves must be known for a wide variation in wall clip and mortar restraint. Luckily, this is not the case because, for reasonable water loads, the required load transfer is much lower than the maximum. In fact, it is within a range where the slope of all curves is very similar and one relationship will reasonably represent the load-transfer relation.

38. The restraint of the stud connection to the base plate is given in Figure 3.12. The setup used in testing this connection is given in Figure 3.13.

39. The above knowledge of material properties and restraint conditions allowed a solution by the finite-element method.
Figure 3.2 Third-point loading and failure of brick wall section.
Figure 3.3 Testing brick and mortar specimens to determine shear strength.
Figure 3.4 Shear modulus determination.

SHEAR STRESS ($\gamma_s$) $\frac{V}{Ld} = \frac{V}{Ld} \frac{Vx_1}{\Delta/x_1 \Delta Ld}$

V = 1/2 P (WHERE P IS TOTAL APPLIED FORCE)

$dY/dX$ = SHEAR STRAIN

$\Delta$ = MEASURED DEFLECTION

$x_1$ = THICKNESS OF MORTAR JOINT

$d$ = WIDTH OF MORTAR JOINT

$L$ = LENGTH OF MORTAR JOINT
Figure 3.5 Experimental setup for testing roof rafter and ceiling joist restraints.
Figure 3.6 Roof rafter and ceiling joist load deflection curves - spruce.
Figure 3.7 Roof rafter and ceiling joist load deflection curves - #2 pine.
Figure 3.8 Nailed roof rafter and ceiling joist connection, connection has been failed.
Figure 3.9 Three types of wall ties.
Figure 3.10 Test setup for wall tie tests.
Figure 3.11 Load deflection curves for wall ties.
Figure 3.12 Stud to base plate load deflection relations.
Figure 3.15 Test setup for determining stud to base plate restraint.
Experimental Results

40. The experimental test design was discussed in Part III. The location of gages for measuring deflection is again given in Figure 4.1 for immediate reference while analyzing the experimental results.

41. Water depth was increased at the front of the wall at about 1 to 2 ft per hour; thereby, loading the brick-veneer wall with horizontal pressure. As the horizontal load increased, the gages were monitored and the deflected slope of the wall measured.

42. After the test, the deflection measurements were converted from mili-volt values to inches of deflection and the variation of water depth versus wall and stud deflection was plotted. These plots are given in Figures 4.2 through 4.11. Gage 5 was the only gage which did not give reliable data and was therefore omitted from these plots.

43. The deflection at any specific point on the wall, as indicated by individual gages, followed a smooth variation. After about 2 ft of water, the wall deflection increased drastically for small increases in water depth. This means that the wall was beginning to act plastically and deflect large amounts for small increases in water load. The wall had failed for sustained loading when the water depth was about 2.4 ft. There were no roof rafter or ceiling joist restraints on this wall. If roof rafter and ceiling joist restraints were provided, there should be an increase in wall strength and a corresponding decrease in wall deflection until the wall studs began to fail. Without the roof rafter and ceiling joist restraints, the stud wall provided insignificant restraint and the wall could continue to deflect and fail.

44. The gages were arranged in horizontal and vertical lines in order to give the variation of wall deflection in cross sections. As will be seen later, the number of gages available was not enough to define the horizontal and vertical wall deflections.

45. In general, much of the upper part of the wall deflected forward or toward the load for water depths up to approximately 1 ft. This
was surprising and at first seemed illogical. As the water increased to depths greater than 1 ft, the total wall began to move backward. There are two reasons the gages show forward wall movement during low water loads:

a. The wall at the locations of water pressure deflected away from the loading. This caused differential lengths in those areas to be lengthened. The lengthening tended to pull the higher portions of the wall causing the wall to cup forward toward the water loading.

b. In effect the wall was pivoting about the lower line of horizontal wall tie restraints. The lower wall ties had greater restraint than those higher because they were closest to the base of the studs; thereby causing the top part of the wall to pivot forward.

46. The cupping and pivoting action is indicated by the deflection of the lower parts of the wall (see gages 1, 2, 3, 4, and 6, Figures 4.6 and 4.7). These gages do not deflect toward the loading. Gages 1, 2, and 3 are just below the first line of wall ties and show deflections generally away from the water load confirming the above reasoning. The gages located higher on the wall deflected forward until the water depth was about 1 ft and then they began to deflect backward as does the total wall.

47. This is as one would expect because finally the water loading will dominate and the wall will be pushed in the direction of the load and even the oscillation in the wall will be only superimposed on the wall's backward movement.

48. The gages are not solely affected by the lowest line of wall ties because the higher lines also have restraining effects. This is seen by how the wall deflects at gages 15 and 17 (Figure 4.10). They do not deflect forward as the lower lines of gages (11, 12, 13, and 14) in Figure 4.9. While considering the wall tie restraints, we are lead to an important response of the wall; the wall is caused to oscillate between the wall ties.

49. These oscillations depend upon the amount of mortar caught on the ties and upon the tie locations. These deflections are similar in phenomena to that of deflections of a continuous beam which is loaded
only in certain spans. Consider the deflection of the beam shown below which is loaded with one point load.

There is a tendency for the beam to deflect as shown. The plots showing cross sections of vertical wall deflections at water depths of 1, 1.5, and 2 ft are given in Figures 4.12 through 4.14. Cross sections showing the horizontal deflected shape are given in Figures 4.15 and 4.16. The deflections of wall studs which were monitored are given in Figures 4.5 and 4.11.

50. The vertical section was oscillatory because of the restraint of the wall ties. The horizontal and vertical oscillation of the wall was not well defined because there were not enough gages measuring wall deflections. The gage points are not dense enough in relation to the tie positions to say that the oscillation shown in Figures 4.12 through 4.16 is accurate. In fact, the only thing that these data points show is that there is an oscillation in the deflected shape of the wall. This oscillation will cause the wall ties to vary in the amount of restraint that they produce. In fact, some of the wall ties may act in tension while the wall is being loaded. Gage 21 on the stud shows that the stud deflects less forward but more backward than the associated position on the wall as measured by gage 12. There are no wall ties on this line of studs but if a clip were here it would be in tension. This is an isolated example; therefore, many variations can exist. This has significant impact on obtaining analytical results for the wall as will be discussed later.

51. An important fact is that the wall deflection was very small until approximately 2 ft of water loading; then, there began to be large increases in deflection for very small increases in load.
52. It is important to realize that the deflection of the wall even for the maximum presented in the water depth versus deflection curves was very small, and was not close to the magnitude of that shown in the failure pictures of the wall. The deflections were in the range where if one was not very observant he would not be able to see any deflection of the wall at all. This means that the wall itself was seriously damaged at relatively small deflections. Even if the wall had been restrained, it would have lost its integrity after it had been strained into the plastic range due to failure in bond. To be on the safe side the wall deflection should be kept below approximately 0.01 in. in the direction of the water loading. This suggests that a logical criteria for failure is to limit the maximum deflection of the wall to 0.01 in., or an alternate, more practical but less general limitation would be on the maximum height of water loading as given later.

53. The deflection of the wall is mentioned because it is probably a more reliable guide for general wall configurations than the water depth. For example, a water depth on a very short wall may not produce much deflection or damage but the same depth on a longer wall would. Similar damage would more than likely occur around the same wall deflection. The deflection criteria will only be practicable after further study and failure charts are developed by computer solutions.

54. This is the reason the computer study is so important because parameter studies can be made and the various effects of variables delineated. Finally curves could be drawn depicting the effect of the variables. If the wall deflects more than 0.01 in. it is damaged and there is a chance that it will not even support service loads or vibrations to which it will be subjected during normal operation without degrading its appearance or losing its ability to serve its purpose. For example, if the wall is loaded until it is damaged it may crack under service loads damaging the waterproofing and not only look bad but allow leaks when subjected to another flood.

55. For Wall 1, a severe loss of integrity began at 2 ft of water with complete loss occurring at 2.4 ft. A picture presenting wall load deflection is shown in Figure 4.17. The failure at 2.4 ft of
water is presented in Figures 4.18 and 4.19. This suggests that common brick-veneer houses which are floodproofed should not be subjected to more than 1-1/2-ft of water for conservative loading and not over 2 ft of water loading without structural modification.

Analytical Results

56. Restraint and material property determinations for the brick-veneer wall system as well as the planning of the analytical solution was presented in Part III. Originally, the plan was to use a two-dimensional model and the finite element method to obtain the deflected shape of the brick-veneer wall. The finite element solution would be obtained by:

a. Using plate elements to represent the wall
b. Determining wall tie restraints as outlined below for specific water depths and using these forces as input values.
   (1) Use experimental data to get relative deflections of studs and wall.
   (2) Extrapolate restraints horizontally and vertically to ties where stud and the corresponding wall deflections are not known.
   (3) From load deflection properties of wall ties obtain the forces which the ties exert on the wall.
c. Inputting water loads, geometry, material properties, and boundary conditions.
d. Solving the problem by computer for deflected shape.

It was known that the two-dimensional model would be problem dependent; but the solution would be economical and would indicate the accuracy of the analytical solutions when they were compared with the test data. A solution of the two-dimensional model was obtained; however, it was found that it could not be implemented with sufficient accuracy for the walls tested because of the oscillating variation of the wall tie loads. The wall oscillated between the ties causing a nonlinear variation in the wall tie loads such that the number of gages used in the experimental testing were not adequate to determine the loads in the wall ties; therefore, the three-dimensional model discussed in Part III would have to be used for an accurate solution of the brick-veneer wall problem.
57. Initially, a three-dimensional finite-element grid was constructed to model Wall 1. As such, it contained no opening within the wall and no ceiling or roof restraints. The various components of this wall were modeled as follows:

<table>
<thead>
<tr>
<th>Components</th>
<th>Element Type Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick wall</td>
<td>Plate</td>
</tr>
<tr>
<td>Ties</td>
<td>Truss</td>
</tr>
<tr>
<td>Studs</td>
<td>Beam</td>
</tr>
<tr>
<td>Known Deflections</td>
<td>Boundary Elements</td>
</tr>
</tbody>
</table>

A data check using the finite-element program (SAP IV) showed the bandwidth to be 875. As SAP IV did not have a three-dimensional bandwidth minimizer, the bandwidth could not be reduced, thereby, making the solution by SAP IV too expensive.

58. A finite-element program (SAP V) recently released from the University of California at Berkeley has a three-dimensional minimizer and is now available for use at the Waterways Experiment Station (WES). This program was used to reduce the bandwidth for Wall 1 from its present 875 to 115, thereby, making the solution for the wall both economical and feasible.

59. The three-dimensional solution could not be completed within the available funding; therefore, a discussion of some findings from approximate two-dimensional solutions is given below.

60. Since the wall tie restraints cannot be estimated, a comparison of experimental results (restrained by wall ties) and analytical results (without wall tie restraint) are presented in Figure 4.20. This comparison shows that analytical solutions are very promising. The deflections given by the analytical solution are somewhat greater than those from the experimental results as would be expected. Since Wall 1 did not have roof rafters and ceiling joist restraints, the wall tie loads are not large relative to the water loading; therefore, the above comparison is probably close.
Figure 4.1  Wall 1 - elevation view of gage locations.
Figure 4.2 Wall 1 - brick-veneer wall deflections, vertical gage line.
Figure 4.3 Wall 1 - brick-veneer wall and stud deflections, vertical gage line.
Figure 4.4 Wall 1 - brick-veneer wall and stud deflections, vertical gage line.
Figure 4.5 Wall 1 - stud deflections, vertical gage line.
Figure 4.6 Wall 1 - brick-veneer wall deflections, horizontal gage line.
Figure 4.7 Wall 1 - brick-veneer wall deflections, horizontal gage line.
Figure 4.8 Wall 1 - brick-veneer wall and stud deflections, horizontal gage line.
Figure 4.9 Wall 1 - brick-veneer wall and stud deflections, horizontal gage line.
4.10 Wall 1 - brick-veneer wall and stud deflections, horizontal gage line.
Figure 4.11 Wall 1 - stud deflections, horizontal gage line.
Figure 4.12 Vertical wall deflection, gages 1, 4, 7, 11, and 15.
Figure 4.13 Vertical wall deflection, gages 2, 8, 12, and 16.
Figure 4.14 Vertical wall deflection, gages 3, 6, 9, 13, and 17.
Figure 4.15  Horizontal wall deflection, gages 7, 8, 9, and 10.
Figure 4.16 Horizontal wall deflection, gages 11, 12, 13, and 14.
Figure 4.17 Wall 1 - wall deflection.
Figure 4.18 Wall 1 - failure pattern of wall.
Figure 4.19 Wall 1 - wall failure.
Figure 4.20 Comparison of analytical and experimental results (deflection is times $10^{-3}$ in inches).
PART V: WALL 2 - EXPERIMENTAL RESULTS

61. The location of the gages for measuring deflection is again given in Figure 5.1 for reference while analyzing the experimental results.

62. The water depth was increased at the front of the wall at about 18 in. per hour; thereby loading the brick-veneer wall with horizontal water pressure. As the water depth increased, the gages were monitored and the deflected shape of the wall measured at the positions indicated in Figure 5.1. The plots of water depth vs wall deflection are in Figures 5.2 through 5.17.

63. In general, the vertical sections of gage measurements showed progressively more deflection with an increase in wall height as the water loading increased. This was true for both forward (toward water loading) and backward (from water loading) wall deflections. The only exception was that the wall deflection at gage 24 did not deflect forward as much as lower positioned gages.

64. The bottom and side restraints had less effect on the forward deflection of Wall 2 than for Wall 1 because of the door in the middle of Wall 2.

65. The gages (23, 24, and 26) near the top of Wall 2 deflected forward. This was not the case for gages 15 and 16 of Wall 1. The deflection vs water depth plots as given in Figures 5.2 through 5.10 are for the initial loading of the wall. The wall failed during the initial loading but it was unloaded and much of the initial deflection was recovered after the water load was removed from the wall. The next morning some of the gages were placed back against the wall and it was reloaded. The deflection vs water depth for the reloading is given in Figures 5.11 through 5.17. The forward deflection during reloading was less than during the initial loading, but the failure of the wall was at a lower water depth (approximately 2 ft). The significant factors as indicated by the experimental results of Wall 2 are:

a. In general the wall deflected forward toward the water loading for low water loads, then backward, as the water depth became greater than 0.8 to 1.6 ft.
b. The wall deflections were very small ($10^{-3}$ in.) until 2 to 2.4 ft of water at which time the wall began to deflect drastically backward for small increases in water depth.

c. Wall 2 (with door opening) deflected more forward but about the same backward as Wall 1. The result was that the backward deflection causing failure of the wall at about the same load as for Wall 1. The lintel strengthened the wall at the door opening causing the opening to have little effect on the final response of the wall. The strength it imparted to the wall was sufficient to shear the steel from the motor joint.

66. The wall deflected forward for low water loads because cord lengthening in the vicinity of the loading caused the upper part of the wall to cup forward. In an actual house the finishing materials on the inside of the studs will give support to the wall and allow it to experience a deeper water loading than indicated by these tests. Computer solutions could be used to determine the effect of such restraints. The significant point is that the deflections recorded for Wall 2 at a given water depth should be an upper bound of deflections experienced in an actual brick-veneer house under the same loading.

67. The failure of wall is shown in Figures 5.18 and 5.19. The deflections shown in Figures 5.2 through 5.17 are much less than those experienced at complete failure of the wall.
Figure 5.1 Wall 2 - elevation view of gage locations.
Figure 5.2 Wall 2, brick veneer wall deflections, vertical gage line, initial loading.
Figure 5.3 Wall 2, brick-veneer wall deflections, vertical gage line, initial loading.
Figure 5.4 Wall 2, brick-veneer wall deflections, vertical gage line, initial loading.
Figure 5.5 Wall 2, brick-veneer wall deflections, vertical gage line, initial loading.
Figure 5.6 Wall 2, brick-veneer wall deflections, horizontal gage line, initial loading.
Figure 5.7 Wall 2, brick-veneer wall deflections, horizontal gage line, initial loading.
Figure 5.8 Wall 2, brick-veneer wall deflections, horizontal gage line, initial loading.
Figure 5.9 Wall 2, brick-veneer wall deflections, horizontal gage line, initial loading.
Figure 5.10 Wall 2, brick-veneer wall deflections, horizontal gage line, initial loading.
Figure 5.11 Wall 2, brick-veneer wall deflections, vertical gage line, reloading.
Figure 5.12 Wall 2, brick-veneer wall deflections, vertical gage line, reloading.
Figure 5.13 Wall 2, brick-veneer wall deflections, horizontal gage line, reloading.
Figure 5.14 Wall 2, brick-veneer wall deflections, horizontal gage line, reloading.
Figure 5.15 Wall 2, brick-veneer wall deflections, horizontal gage line, reloading.
Figure 5.16 Wall 2, brick-veneer wall deflections, horizontal gage line, reloading.
Figure 5.17 Wall 2, brick-veneer wall deflections, horizontal gage line, reloading.
Figure 5.18 Wall 2, wall failure, front view.
Figure 5.19 Wall 2, wall failure, side view.
68. Wall 3 was constructed to evaluate the response of a brick-veneer wall whose supports included roof rafter and ceiling joist restraints. These restraints were located along the top plate of the stud framing. The first restraint was at the end of the stud framing, the second 1.0 ft from the end, and the remaining at 2.0 ft on center. The roof rafters were constructed using a 4/12 roof pitch. Roof rafter and ceiling joist locations are again given in Figure 6.1 for reference while analyzing the experimental results.

69. The stud framing for Wall 3 was constructed the same as for Wall 1. The first stud was located 0.42 ft from the end of the brick-veneer wall. The second stud was located 1.25 ft from the first and the remaining studs were on 1.5-ft centers. The stud locations are also presented in Figure 6.1.

70. Most ties in Wall 3 were placed at five-brick intervals vertically. Ties were placed horizontally on every other stud starting with the third stud. Due to this alternate placing and the geometry of the stud framing the ties were unsymmetrically located about the center line of the brick-veneer wall. Tie locations are presented in Figure 6.1.

71. Tie deflections and therefore wall deflections were comparatively less in Walls 2 and 3 than in Wall 1 under compressive loading because of the omission of wallboard in Walls 2 and 3. The wallboard is normally attached to the outside face of the stud framing and the ties are attached to the outside face of the wallboard. This wallboard is more compressive than its supporting studs and its omission should result in less deflection, but a negligible difference as far as comparative results are concerned. The purpose for omitting the wallboard from Wall 3 was to observe wall response and crack development along the backside of the brick-veneer wall.

72. Tie restraints should also vary between walls for the same loading because the amount of mortar at each tie location was randomly varied. The data generated by this investigation are insufficient for determining the exact effects of such variables in relation to wall
response. Such determinations can be best resolved through the use of computer code programs such as SAP V. Computer program solutions can also be used to delineate the significance of other variables such as wall length, boundary restraints, material properties, etc.

73. Gage locations for Wall 3 were the same as those for Wall 1, except for the additional gages that were located on the stub wall and ceiling joists. All gages on the stud and brick-veneer wall were located between vertical lines of wall ties except for gages 10 and 17. A gage was also located at the top of each stub wall. The remaining gages were located on the ceiling joist, one at each end and one at the middle. Gage locations are presented in Figure 6.2.

74. Wall 3 was loaded at a rate of approximately 17 in. per hour. Deflections were recorded at 3-in. increments of water depth starting at an initial depth of 6 in. The wall was loaded to failure under hydrostatic water pressure.

75. Wall 3 with its ceiling joist and roof restraints represents the side or similarly braced wall in a brick-veneer dwelling. Plots of water depth vs brick wall deflection for Wall 3 are presented in Figures 6.3 through 6.12. The curves in these plots can be segmented into two parts:

a. Small deflections (water depth of 1.5 ft or less)

b. Large deflections (water depth greater than 1.5 ft)

76. For segment "a" the movement was very small and somewhat erratic. In general the movement was away from the applied hydrostatic loading except for gage 10 and the gages along the top of wall (18, 19, and 20). The negative deflections recorded for the top of the wall were due to the cord lengthening. This lengthening was due to the applied hydrostatic loading being located low on the wall causing the top of the stud framing to try to move toward the applied loading. Due to the ceiling joist and roof rafter restraints the negative movement along the top of the wall was less pronounced than that of Wall 2.

77. For deflections of water depths greater than 1.5 ft the wall began to deflect drastically away from the applied loading for small increases in water depth. At a 2-ft water depth all deflections of brick-veneer wall were positive or away from the water loading.
78. Gages on the stud framing generally showed very small deflections for water depths of 1.75 ft or less. For water depths between 1.75 and 2.00 ft, the stud wall began to deflect away from the applied loading. The deflections were all positive at a water depth of 2.25 ft and generally showed large deflections for small increases in water depths for depths greater than 2.25 ft. These deflections were smaller than those recorded for the same water depth on the brick wall. The stud framing deflections are presented in Figure 6.11.

79. Gages located on the top of the stub wall showed that the stub wall generally deflected toward the applied loading for water depths of 1.75 ft and below and away from the applied loading for water depth greater than 1.5 ft. The deflections for the top of the stub wall were all positive for water depths of 3.0 ft and above.

80. Gages located on ceiling joints generally showed the end joist moving toward the applied loading for water depths of 1.5 ft and below and away from the applied loading for water depths greater than 1.75 ft. The deflections recorded for a joist near the midspan of the wall showed a continuous movement away from the applied hydrostatic loading. This midspan movement became more pronounced after water depth exceeded 1.5 ft. Stub wall and ceiling joist deflections are presented in Figure 6.12.

81. The movement of portions of the support components of the brick-veneer wall toward the applied loading was the result of the cord lengthening in the brick wall as previously discussed. This action was generally dominate over relative movement of the total wall away from the applied hydrostatic loading for water depths of 1.75 ft and below. For water depths greater than 1.75 ft the relative movement of the total wall dominated.

82. Total collapse of the brick-veneer wall occurred at a depth of 57 in. and at a total applied force of 18,300 lb. This failure was sudden and appeared to result from the failure of the supporting studs. The remains of the brick-veneer wall after failure is presented in Figures 6.13 and 6.14.
The significant findings from the experimental results of Wall 3 are:

a. In general, the roof rafter and ceiling joist restraints decrease the movement of the wall toward the water loading.

b. The roof rafter and ceiling joist restraints are sufficient to cause a change in the failure mechanism from that which was experienced in Walls 1 and 2. The failure mechanism for Walls 1 and 2 was deflection and failure of the brick wall. The failure mechanism for Wall 3 was beam failure of the studs and a resulting collapse of the brick wall.

c. The deflection of the brick wall began to increase rapidly with water depth after about 1-1/2-ft but the increase is not as great as was experienced for Walls 1 and 2. This is indicated by the fact that the wall did not collapse until about 57 in. of water loading.

d. Even though the wall can withstand greater water depths, it fails suddenly and totally when the stud wall fails.
Figure 6.1  Stud, wall tie and roof rafter and ceiling joist locations, Wall 3.
Figure 6.2 Wall 3 - elevation view of gage locations.
Figure 6.3 Wall 3, brick-veneer wall deflection, vertical gage line.
Figure 6.4 Wall 3, brick-veneer wall deflections, vertical gage line.
Figure 6.5 Wall 3, brick-veneer wall deflections, vertical gage line.
Figure 6.6 Wall 3, brick-veneer wall deflections, horizontal gage line.
Figure 6.7 Wall 3, brick-veneer wall deflections, horizontal gage line.
Figure 6.8 Wall 3, brick-veneer wall deflections, horizontal gage line.
Figure 6.9 Wall 3, brick-veneer wall deflections, horizontal gage line.
Figure 6.10 Wall 3, brick-veneer wall deflections, horizontal gage line.
Figure 6.11 Wall 3, stud wall deflections.
Figure 6.12 Wall 3, stub wall and ceiling joists deflections.
Figure 6.13 Wall 3, wall failure, front view.
Figure 6.14 Wall 3, wall failure, side view.
84. Floodproofing individual homes is an important aspect of the total solution of flood damage reduction. This report gives insight into the structural resistance of brick-veneer walls subjected to hydrostatic water loading. The knowledge of the ability of a brick-veneer wall to resist water loads allows one to

a. Evaluate the structural feasibility of floodproofing a brick-veneer home.

b. Determine floodproofing costs.

c. Determine remedial measures for inadequate construction.

There are many variables affecting the response of a brick-veneer wall; therefore, the approach of this study was to obtain limited experimental data by testing three walls, analyze these data, and compare them to analytical solutions. Useful information was obtained from the experimental data and the experimental results were used to validate the analytical method. In the future a good analytical method is essential to:

a. Perform parameter studies to completely define the affect of various variables on the structural integrity of the brick-veneer wall.

b. Perform detailed analysis for critical structures to be sure that they are structurally sound when they are subjected to specific floodloadings.

c. Broaden the knowledge of the response of various buildings such that simple and expedient media such as graphs, tables, etc. can be developed and presented in manuals for the home owner to use in evaluating his need for floodproofing.

d. Determine the effect of specific remedial measures on the structural integrity of the brick-veneer home.

85. Wall 1 was typical of the end wall of a house (no roof rafter or ceiling joist restraints). The deflections at specific points on the wall followed a smooth variation. After about 2 ft of water, the wall deflections increased drastically for small increases in water depth. The wall had failed for sustained loading when the water depth was about 2.4 ft. The deflection of the wall is very small (on an order of magnitude of $10^{-3}$ in.) until the wall begins to fail then the deflection increases rapidly with water depth.
86. The analytical results for Wall 1 compare favorably with the experimental results. It is seen that a three-dimensional solution is necessary to adequately model the general response of a brick-veneer wall.

87. Wall 2 is constructed just as Wall 1 except it has a 3-ft door opening in its center. The significant factors as indicated by the experimental results of Wall 2 are:

a. In general the wall deflected forward toward the water loading for low water loads then backward as the water depth became greater than 0.8 to 1.6 ft.

b. The wall deflections were very small \((10^{-3} \text{ in.})\) for depths up to 2 to 2.4 ft of water at which time the wall began to deflect drastically backward for small increases in water depth.

c. Wall 2 (with door opening) deflected more forward but about the same backward as Wall 1. The backward deflection causing failure of the wall was about the same as for Wall 1. The lintel strengthened the wall at the door opening; thereby, causing the opening to have little effect on the final response of the wall.

88. Wall 3 was constructed just as Wall 1 except it included roof rafter and ceiling joist restraints.

89. The significant findings from the experimental results of Wall 3 are:

a. In general, the roof rafter and ceiling joist restraints decrease the movement of the wall toward the water loading.

b. The roof rafter and ceiling joist restraints are sufficient to cause a change in the failure mechanism from that which was experienced in walls 1 and 2. The failure mechanism for Walls 1 and 2 was deflection and failure of the brick wall. The failure mechanism for Wall 3 was beam failure of the studs and a resulting collapse of the brick wall.

c. The deflection of the brick wall begins to increase rapidly with water depth after about 1-1/2 ft but the increase is not as great as was experienced for Walls 1 and 2. This is indicated by the fact that the wall did not collapse until about 57 in. of water loading had been attained.

d. Even though the wall can withstand greater water depths, it fails suddenly and totally when the stud wall fails.

90. Based on the test results, the upper bound for the failure of Walls 1, 2, and 3 was arbitrarily established at approximately 2 ft of
water depth or at a deflection of 0.01 in. The critical water depths recorded at the 0.01-in. deflections are as follows:

<table>
<thead>
<tr>
<th>Wall</th>
<th>Minimum Water Depth recorded for a 0.01 in. deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.1 ft</td>
</tr>
<tr>
<td>2</td>
<td>2.1 ft</td>
</tr>
<tr>
<td>2</td>
<td>1.8 ft (RELOAD)</td>
</tr>
<tr>
<td>3</td>
<td>2.1 ft</td>
</tr>
</tbody>
</table>

From this one might conclude that all three walls were structurally damaged at about the same hydrostatic loading. Upon reloading a wall that has been loaded beyond its elastic limit (for example, Wall 2) it will support less water loading than initially.

91. The structural integrity of the brick-veneer Walls 1 and 2 was completely lost at about 2-1/2 ft of water loading. The type restraint did cause a change in the total capacity of the wall to resist hydrostatic loading because Wall 3 did not collapse until 57 in. of water loading was attained.

92. It is true that the finishing on the inside of the studs will help strengthen the walls; however, no wave or debris loading was imposed on the walls in these tests.

93. Without modifications a brick-veneer wall cannot be expected to withstand more than about 2 ft of hydrostatic pressure, and if some safety factor is desired a limitation of 1-1/2 ft of water should be imposed.

94. The modifications to support water depths greater than 2 ft should mainly be in two areas:

   a. Support to the top plate of walls without roof rafter and ceiling joist restraints.

   b. Added thicknesses to the walls (extra layer of brick planters, retaining walls, etc.) to an elevation somewhat above the expected height of flood waters.

Modifications can be designed to withstand water loads much higher than 2 ft.
95. At the present time existing brick-veneer homes should not be expected to withstand more than 2 ft of water without structural damage.
PART VIII: RECOMMENDATIONS

96. This report gives preliminary insight into the structural integrity of existing brick-veneer walls subjected to water loadings. The next step is to determine several expedient, feasible, and effective ways of waterproofing the brick-veneer walls, eliminating underseepage, and waterproofing closures such as doors and windows. From the alternatives a home owner can pick the option that best fits his situation and realistically prepare his home to resist flood waters. The above measures will have bridged the crisis stage of floodproofing individual homes. Much more must then be done to make floodproofing realistic and safe.

97. Much was learned from testing the three specific brick walls in this study but there are many variables which will affect the response of the wall. Some of these are:

a. wall length
b. wall height
c. number and type of openings
d. impact load
e. wave loads
f. vibrations
g. roof rafter and ceiling joist restraints
h. inside finishing materials
i. stud to base plate restraints
j. wall tie restraints
k. oscillation between wall ties
l. boundary effects
m. material properties
n. structural modifications

It is too expensive and time consuming to vary these parameters experimentally and determine their effect and the general response of the wall; therefore, it is necessary that analytical solutions and plans for analytical studies be developed. They can be used to make parameter studies, determine the effect of structural modifications, and develop aids for helping the home owner effectively and economically prepare his home to resist a specific flood load.
98. It is also recommended that modifications be studied which will allow a brick-veneer home to resist flood loads greater than 2 ft without structural damage.
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