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EXISTING STRUCTURES EVALUATION

Part V: Applications

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13. ABSTRACT

The objective of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing (NFSS)-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures

Past efforts in this program have been concerned with examining exterior walls, window glass, and steel frame connections. In this phase, the previously developed mathematical models for exterior walls were used to predict the collapse overpressure for selected structures. The report presents the results of the dynamic analysis of the exterior walls of one structure located on the Stanford University campus, and five structures located in Detroit, Michigan.

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Final Report *July 1971*

Detachable Summary

EXISTING STRUCTURES EVALUATION

Part V: Applications

By: C. K. WIEHLE
J. L. BOCKHOLT
Facilities and Housing Research

Prepared for:

OFFICE OF CIVIL DEFENSE
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OCD REVIEW NOTICE

This report has been reviewed in the Office of Civil Defense and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Office of Civil Defense.

SUMMARY

Objective

The objective of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing NFSS-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures.

Background

Past efforts in this program have been concerned with examining exterior walls, window glass, and steel frame connections. This report presents the results of the dynamic analysis of the exterior walls of one structure located in Stanford, California, and five structures located in Detroit, Michigan.

Initially, in the application phase of the study, it was intended to analyze a number of structures in the San Jose area. Although detailed information was obtained for six structures in this area, it was possible during this effort to analyze only one structure located on the Stanford University Campus.

In addition, as part of an integrated program to develop an all-effects survey procedure, five NFSS buildings located in Detroit were analyzed using information provided by an on-site field survey conducted by Research Triangle Institute (RTI). Each building was first analyzed using the field survey data, and then a second, independent analysis was made using building plan data.

The predictions of the collapse overpressure of the buildings were based on a dynamic analysis of the exterior walls using the previously developed procedures. Although the evaluation procedure is in a developmental stage, the prediction of damage to actual NFSS buildings was of value in providing guidance in planning the research effort and in providing interim predictions for the collapse overpressure of actual structures for use by OCD.

At the present time, the evaluation procedure has not been extended to include the collapse of the structural frame under dynamic loading. Therefore, to use the interim techniques for predicting the collapse of the exterior walls, it was necessary to assume that the frame did not fail at a lower overpressure level than the exterior walls. For the building located at Stanford, this assumption did not influence the collapse prediction, since it is a reinforced concrete, load-bearing wall structure. Also, for three of the five Detroit buildings analyzed, the frame assumption probably did not affect the predictions. However, as discussed in the main body of the report, for two of the Detroit buildings, it is probable that an overall collapse of the structure would occur at a lower overpressure than that predicted for the exterior walls.

Analysis

Since the Stanford University building was not part of the field survey exercise, the analysis of this structure is presented separately. Wilbur Hall #6 is a load-bearing wall structure with 8-in. thick reinforced concrete exterior walls. The wall was analyzed as a two-way wall, fixed on four sides, and since the collapse of the first story controlled the collapse of the building, this was the only wall panel for which a probabilistic analysis was made. The results of the analysis are as follows:

Table S-1 (Concluded)

Case*	Location Side	Location Floor	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi: Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
General Electric Service Building								
F1	B	1	RC-6	24	19.9	0.4	19.4	20.4
F2	B	4	RC-6	24	24.0	1.1	22.7	25.4
P1	B	1	RC-6	24	46.9	2.9	42.2	50.6
P2	B	4	RC-6	24	92.5	8.7	81.4	103.6
P3	B	4	A-1 way	8.5	0.7	0.5	0.1	1.3
P3A	B	4	U-8	8.5	0.3	0.2	0.1	0.5
Clara Barton Elementary School								
F1	A	1	A-1 way	16	18.9	2.0	16.4	21.4
F2	B	1	A-2 way	16	15.2	3.0	11.3	19.1
F3	D	1	A-1 way	16	11.7	2.0	9.2	14.3
P1	A	1	A-1 way	8	5.6	1.0	4.3	6.8
P2	B	1	A-2 way	12	9.9	2.2	7.2	12.7
P3	D	1	A-1 way	8	4.3	0.9	3.2	5.4

* The prefix F identifies walls analyzed using field survey data, and P those analyzed using building plan data.

† Each wall is designated with a letter to identify the wall type and a number to identify the wall support condition. The key to the wall types and support cases are given in Table S-2.

Table S-2

WALL TYPE AND SUPPORT KEY

Letter	Wall Type
U	Unreinforced masonry unit wall
A	Arching wall
RC	Reinforced concrete wall

Number	Support Case
1	Two-way, simply supported on four edges.
2	Two-way, fixed on four edges.
3	Two-way, fixed on vertical edges; simply supported on horizontal edges.
4	Two-way, simply supported on vertical edges; fixed on horizontal edges.
5	One-way, simply supported on opposite edges.
6	One-way, fixed on opposite edges.
7	One-way, propped cantilever.
8	One-way, cantilever.

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P3A	B	4	U-8	8.5	0.3	0.2	0.1	0.5
Clara Barton Elementary School								
F1	A	1	A-1 way	16	18.9	2.0	16.4	21.4
F2	B	1	A-2 way	16	15.2	3.0	11.3	19.1
F3	D	1	A-1 way	16	11.7	2.0	9.2	14.3
P1	A	1	A-1 way	8	5.6	1.0	4.3	6.8
P2	B	1	A-2 way	12	9.9	2.2	7.2	12.7
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The objective of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing NFSS-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures.

Past efforts in this program have been concerned with examining exterior walls, window glass, and steel frame connections. In this phase, the previously developed mathematical models for exterior walls were used to predict the collapse overpressure for selected structures. The report presents the results of the dynamic analysis of the exterior walls of one structure located on the Stanford University campus, and five structures located in Detroit, Michigan.

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I INTRODUCTION

Under contract to the Office of Civil Defense, Stanford Research Institute is developing a procedure for the evaluation of existing structures subjected to nuclear air blast. The objective of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing NFSS-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures.

Background

Past efforts in this program have been concerned with examining exterior walls (Refs. 1 and 2), window glass (Ref. 3), and steel frame connections (Ref. 4). This report presents the results of the dynamic analysis of the exterior walls of one structure located in Stanford, California, and five structures located in Detroit, Michigan.

Initially, in the application phase of the study, it was intended to analyze a number of structures located in the San Jose area. Therefore, a procedure was established for selecting candidate structures that would be appropriate for analysis with the available techniques. Although detailed information was obtained for six structures in the San Jose area, it was possible during this effort to analyze only one structure located on the Stanford University campus.

As part of an integrated program to develop a survey procedure for all nuclear weapon effects, Research Triangle Institute (RTI) made an on-site field survey during November 1970 of five preselected NFSS buildings in Detroit. The survey was conducted primarily to obtain a complete structural description of the buildings that would be adequate for building damage and casualty prediction purposes. The results of the field survey were recorded on predesigned forms and included sketches and photographs. A complete copy of this information, together with the building plans, was then provided to SRI for analysis of the buildings.

To predict the collapse overpressure of the exterior walls of the five Detroit buildings, two analyses of each building were performed. First, an analysis was made using the data obtained during the RTI on-site survey. A second analysis of the same building was then made independently using data obtained from the actual building plans, which were furnished to RTI by the Detroit Bureau of Buildings. This procedure provided a check on the adequacy of the proposed field survey data form, and emphasized areas of possible improvement.

Analysis Limitations and Discussion

The predictions of the collapse overpressure of the buildings were based on a dynamic analysis of the exterior walls using the procedures presented in Refs. 1 and 2. That is, the intent in this study was to predict the blast damage to actual NFSS structures, even though only interim techniques were available for analyzing wall elements. This procedure was of value in providing guidance in planning the research effort and in providing interim predictions for the collapse overpressure of actual structures for use by OCD.

At the present time, the evaluation procedure has not been extended to include the collapse of the structural frame under dynamic loading. Therefore, to use the interim techniques for predicting the collapse of the exterior walls, it was necessary to assume that the frame did not fail at a lower overpressure level than the exterior walls. For the building located at Stanford, this assumption did not influence the collapse prediction since it is a reinforced concrete, load-bearing wall structure. Also, for three of the five Detroit buildings analyzed, the frame assumption probably did not affect the predictions. However, as discussed later in the report, for two of the Detroit buildings, it is most probable that an overall collapse of the structure would actually occur at a lower overpressure than that predicted for the exterior wall.

In addition, the method of construction of an arching type wall is extremely important in the determination of its resistance function. For example, if a wall is constructed such that the closing joint at the top of the wall (between the wall and the floor beam or slab) is well mortared, it is reasonable to assume that the wall can develop its maximum arching force. On the other hand, if the top mortar joint is improperly made, or if a gap exists between the wall and beam, the arching resistance is reduced in proportion to the size of the gap.* Also, a gap, or improperly mortared top joint, may result in a collapse mechanism that prevents the development of arching resistance. Since there is no information available on the actual construction techniques used for any of the structures analyzed in this study, it was assumed that if the wall was of the arching type, the maximum arching resistance was developed.

For the evaluation of the exterior wall elements in this study, failure implies collapse or disintegration of the wall. Furthermore, the predicted collapse overpressures given are for the incipient collapse of the wall, which is defined as that point in the response where the wall can be considered as on the threshold of collapse. The pressure at incipient collapse is therefore the load that is just sufficient in

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* The resistance function for arching walls with a gap, or elastic supports, is presented in Ref. 1.

magnitude to cause a collapse of the wall--a load of slightly lesser magnitude would not result in collapse.

It should be noted that the load-time function on a wall in an actual structure subjected to nuclear blast is a complex phenomenon, and a precise description of the loading function is not too meaningful in comparing collapse predictions. Therefore, the predicted collapse overpressures given in this report are the peak incident overpressures of the free-field blast wave that results in collapse of the wall.

Acknowledgment

The authors gratefully acknowledge the assistance and guidance of G. N. Sisson and N. A. Meador of the Office of Civil Defense during the conduct of this program. The authors are also grateful to M. D. Wright of Research Triangle Institute for providing the photographs and field survey information for the Detroit buildings.

II BUILDING ANALYSIS--STANFORD

Introduction

Early in this program, it was intended to predict the collapse overpressure for five or more NFSS structures located in the San Jose area. To select candidate structures, a procedure was established for selecting those structures that would be amenable to analysis with the interim prediction techniques available.

Because of the large number of identified NFSS structures in the San Jose area, the first step in the procedure was to select only those buildings that make up the 55 NFSS structure sample for San Jose from Ref. 5. The 55 structures were then categorized as follows.

A. Masonry wall
   1. Unreinforced brick
      a. Load bearing
      b. Curtain
   2. Concrete block
      a. Load bearing, reinforced, without masonry veneer
      b. Curtain, unreinforced, without masonry veneer
      c. Curtain, reinforced, with masonry veneer

B. Concrete wall
   1. Precast, without masonry veneer
      a. Load bearing
      b. Curtain
   2. Cast-in-place, without masonry veneer
      a. Load bearing
      b. Curtain
   3. Cast-in-place, with masonry veneer
      a. Load bearing
      b. Curtain

Next, an on-site inspection of each building was made and all buildings for which plans were not available were eliminated from further consideration. From the remaining buildings, an attempt was made to identify a suitable candidate for each of the above listed categories. The final building selection was based on a detailed examination of the building plans and an on-site inspection to determine those structures best suited for analysis. The following structures make up the final selection:

A. Masonry wall

Sears & Roebuck, reinforced brick, curtain wall

B. Concrete wall

Wilbur Hall #6, reinforced concrete, load-bearing wall
Barnes House, reinforced concrete, load-bearing wall
VA Hospital Building 5A, reinforced concrete, curtain wall
VA Hospital Building 24B, reinforced concrete, curtain wall

As mentioned previously, during this period of the effort, it was only possible to analyze one of the above structures: Wilbur Hall #6, Stanford University, California.

Wilbur Hall #6

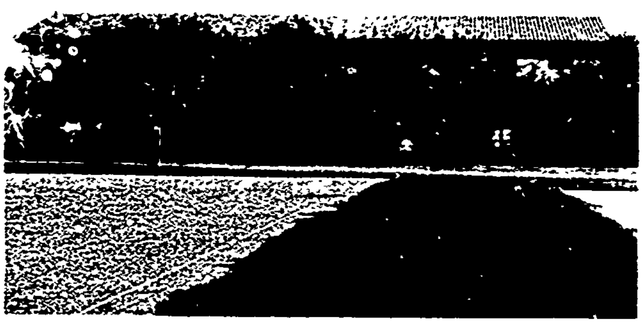
Description

Wilbur Hall #6 is a student dormitory located on the campus of Stanford University; it was constructed in 1954. The building consists of three stories with an overall height of about 35 ft and plan dimensions of 41 by 136 ft, which provide an area of about 5,560 sq ft per floor. There is no basement. A photograph of the building is shown in Figure 1.

SIDE A

SIDES B AND C

NOT REPRODUCIBLE



SIDE D

FIGURE 1 PHOTOGRAPHS OF WILBUR HALL #6

The building is a reinforced concrete, load-bearing wall structure. The 8-in. thick concrete exterior walls are reinforced with #4 bars at 16 in. on center in both the vertical and horizontal directions. Reinforcement is in both faces, with a clear distance of 1-1/2 in. at the outer face and 1 in. at the inner face. The 7-in. thick concrete interior load-bearing partitions, which form the hallway, extend the length of the building and are reinforced with #4 bars at 12 in. on center placed in the center of the wall in both vertical and horizontal directions. The steel reinforcement was intermediate grade A-15-52T, and the concrete was specified to have an ultimate compressive strength of 2500 psi.

As noted in the plan view in Figure 2, the layout of the building consists of a central hallway, with student rooms or lounge areas on both sides. The 5 ft 5 in. wide hallway was the only area designated as shelter space in the NFSS. The first and second floor hallways each contained 60 shelter spaces. The dimensions of a typical student room are 12 ft wide by 17 ft long and contain a 5 ft 4 in. by 4 ft 8 in. window, glazed with double strength glass.

Analysis

To predict the collapse overpressure for a load-bearing wall structure using the methods presented in Refs. 1 and 2, it is necessary to reduce the structure to a series of wall elements. The approach used was first to analyze various exterior wall segments in a deterministic manner to find the weakest segment and then to analyze only this segment statistically with the Monte Carlo procedure discussed in Ref. 2.

An examination of Wilbur Hall #6 indicated that the most severe case would be for a blast wave striking the long side of the building at normal incidence, thus only the exterior wall on the long side was considered. The wall was treated as a series of continuous panels, supported at the

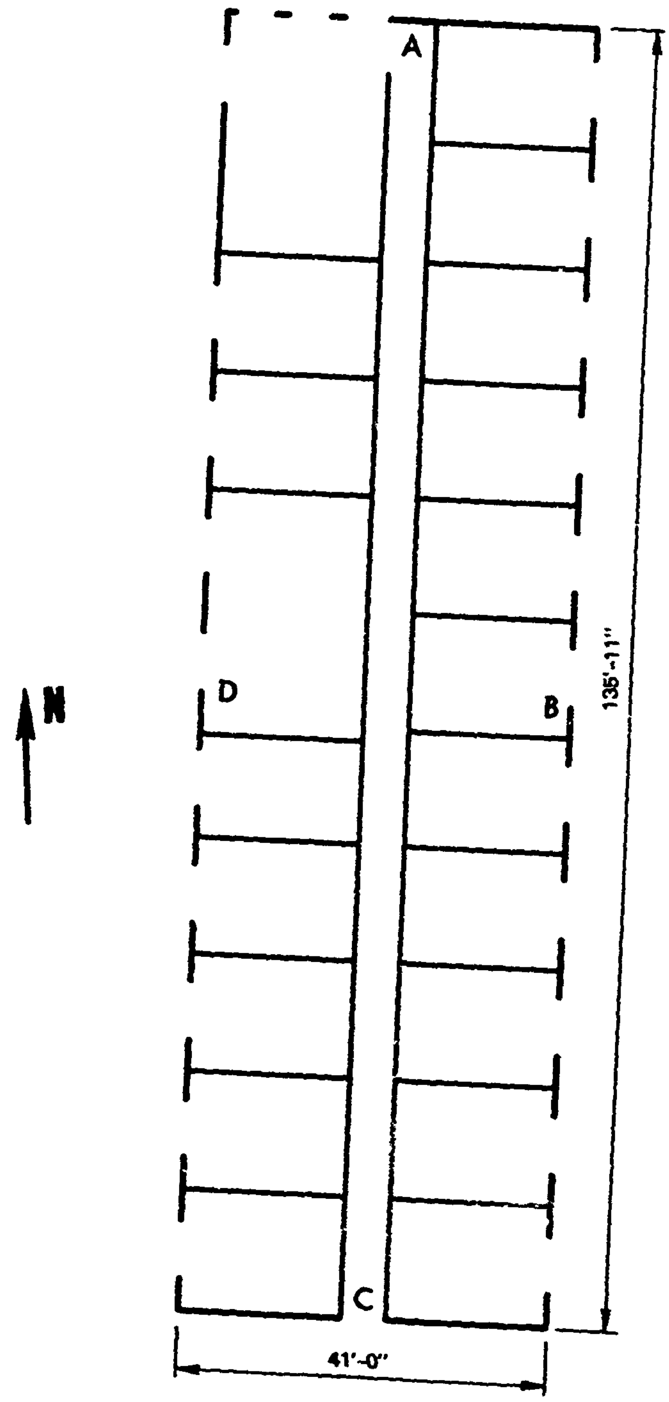


FIGURE 2 FIRST FLOOR PLAN VIEW OF WILBUR HALL #6

floor levels and at the interior partitions dividing the rooms. Each of these supports was assumed to be nonyielding, and the panels were therefore analyzed as two-way reinforced concrete walls, fixed on four edges. One panel on each of the three stories was analyzed. Since the geometric and physical properties were the same for each story, only the axial load on the wall and the clearing distance were considered to change from story to story. Details of a typical wall panel are shown on Figure 3.

The effect of the window opening on the resistance curve for a two-way action wall was calculated using the method discussed in Ref. 2. It is assumed in the method that the window is centrally located in the wall. Even though this is not the case for the above wall panels, the error is believed to be of minor importance and is somewhat compensated for by the marginal reinforcement steel located around the four edges of the opening.

The methods outlined in Refs. 1 and 2 were used to calculate the exterior and interior loadings on the exterior wall. In calculating the net load acting on the walls, the window glass was assumed to fracture in 3 msec (Ref. 3). To determine the volume into which the room filling takes place, it was assumed that all interior partitions remained intact. This assumption appears reasonable since the symmetrical arrangement of the rooms would result in a zero net load on the side partitions, which separate adjacent rooms. Although the partition at the rear of the room would be subjected to a significant load, it was assumed to be of sufficient strength to remain standing.* However, it is unlikely that the doors would survive at an overpressure of sufficient strength to collapse

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* An analysis of the interior load-bearing partition at a pressure level sufficient to cause collapse of the exterior wall showed that the interior partition did not fail.

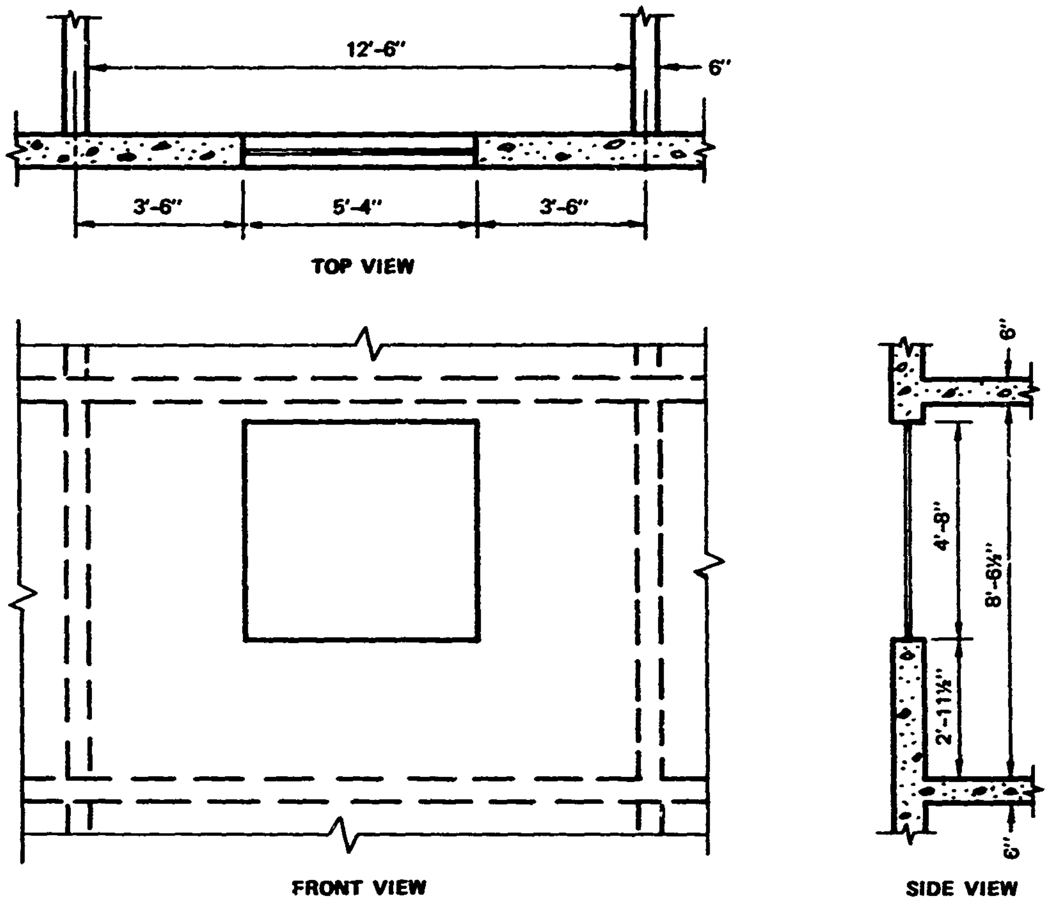

FIGURE 3 TYPICAL EXTERIOR WALL PANEL DETAIL FOR WILBUR HALL #6

the exterior wall. Thus, the room-filling process will eventually expand into the hallway and subsequently into the room on the opposite side of the building. Since the computer routine used for the room-filling analysis was developed for a single room only, rather than a series of rooms, a compromise solution was obtained by assuming that the volume for the room-filling calculation was equal to the room volume plus the hallway volume adjacent to the room. In view of the small error induced by this assumption,* a more sophisticated procedure was unwarranted.

As stated previously, the clearing distance for the wall panels varied from story to story. Using the method outlined in Ref. 2, the following values were obtained for the clearing distance for the exterior wall panels on the long side of the building:

	Clearing Distance, S(ft)		
Story	Minimum	Mean	Maximum
1	2.90	14.26	25.62
2	2.93	10.00	17.08
3	2.93	5.74	8.54

The vertical axial load acting on the exterior wall also varies in each story. Considering building dead load only, values of 310, 190, and 70 lb/in. of wall width were obtained for the axial load on the first, second, and third story wall panels, respectively.

To account for the dynamic increase in the steel and concrete strengths, the recommendations given in Ref. 6 were used. This resulted

* Computer analyses made varying the room volume from 1650 cu ft (volume of a single room) to 3950 cu ft (volume of two rooms plus adjacent hallway) show a maximum difference of 15 percent in the incipient collapse overpressure of the exterior wall for the two extremes.

in a dynamic yield strength of 52,000 psi for the intermediate grade reinforcing steel, and a dynamic ultimate concrete compressive strength of 3125 psi (a 25 percent increase over the specified static strengths).

Probability distributions for the clearing distance and the dynamic yield strength of the steel were required for use in the statistical analysis. A variation of the concrete strength was not included in the statistical analysis since parametric studies presented in Ref. 1 revealed that for the range of expected values the ultimate concrete compressive strength had little effect on the incipient collapse overpressure for a reinforced concrete wall. Since little information was available on the most likely clearing distance, it was assumed to be normally distributed with a mean value equal to that given in the preceding tabulation. The standard deviation was obtained by assuming that the minimum and maximum clearing distances occurred 2.5 and 97.5 percent of the time, respectively. This results in standard deviations of 5.80, 3.61, and 1.43 psi for the first, second, and third stories, respectively.

Statistical data on the static yield strength of intermediate grade reinforcing steel is given in Ref. 7. Although these results were for static tests, and since no corresponding information was found for dynamic tests, it was assumed that the coefficient of variation remained the same. Applying the coefficient of variation of 0.124 found in the static tests to the present case, a standard deviation of 6500 psi was obtained. Although Ref. 7 uses a log-normal probability distribution to fit the data,* a normal distribution was also found to give a good fit, although having larger variations near the extreme values. Since for this analysis, the extremes are of minor importance and since random number

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* Reference 8 uses a Beta-distribution to fit the same data for the reinforcing steel.

generators for a normal distribution were readily available, the dynamic yield strength was assumed to be normally distributed. For a mean dynamic yield strength of 52,000 psi and a standard deviation of 6500 psi, the resulting 2.5 percent and 97.5 percent cumulative probability values are 39,260 and 64,740 psi. These values appear reasonable, since very few of the test values will fall below the specified minimum yield strength of 40,000 psi. Similarily, it is doubtful that more than a few yield strengths will exceed 65,000 psi, since the ultimate strength is usually around 70,000 psi.

The wall and load properties used in the analysis are summarized as follows:

Wall Data

L_v = 8 ft 1/2 in.

L_H = 12 ft 0 in.

t_w = 8 in.

f'_{dc} = 3125 psi

E_c = 3.0×10^6 psi

f_{dy} = 52,000 psi (mean); 6500 psi (standard deviation)

E_s = 30.0×10^6 psi

P_v = 310 lb/in. - first story
190 lb/in. - second story
70 lb/in. - third story

L_{VW} = 4 ft 8 in.

L_{HW} = 5 ft 4 in.

Reinforcement Data

Section*	p	d (in.)	p'	d'(in.)
1	0.00185	6.75	0.00185	1.75
2	0.00185	6.75	0.00185	1.75
3	0.00185	6.25	0.00185	1.25
4	0.00185	6.25	0.00185	1.25

Load Data

W = 1 Mt, surface burst

P_o = 14.7 psi

c_o = 1120 fps

S = 14.26 ft (mean); 5.80 ft (standard deviation) - first story
10.00 ft (mean); 3.61 ft (standard deviation) - second story
5.74 ft (mean); 1.43 ft (standard deviation) - third story

Room Filling Data

Room volume = 2300 cu ft

Number of openings = 1 (front wall)

Area of opening = 24.9 sq ft

Delay at opening = 3 msec

Ambient air density = 0.076 pcf

Results

The following incipient collapse overpressures were obtained for the deterministic portion of the analysis:

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* The locations of the sections are given in Ref. 2, page 55.

First story	-	18.3 psi
Second story	-	19.0 psi
Third story	-	19.8 psi

These results indicate collapse of the exterior wall to be initiated at the first story level. Since the walls are load-bearing, collapse at this level essentially means collapse of the building. Therefore, the statistical portion of the analysis was restricted to the first story. The results of this analysis are summarized as follows:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
P1	18.5	2.0	15.9	21.1

These results are shown graphically on Figure 4.

Because of the uncertainty of the actual distribution of the clearing distance, a series of analyses was made varying S between the minimum and maximum values that would be expected. These results are shown in Figure 5. As can be seen, the value chosen for S has a large effect on the incipient collapse overpressure obtained.

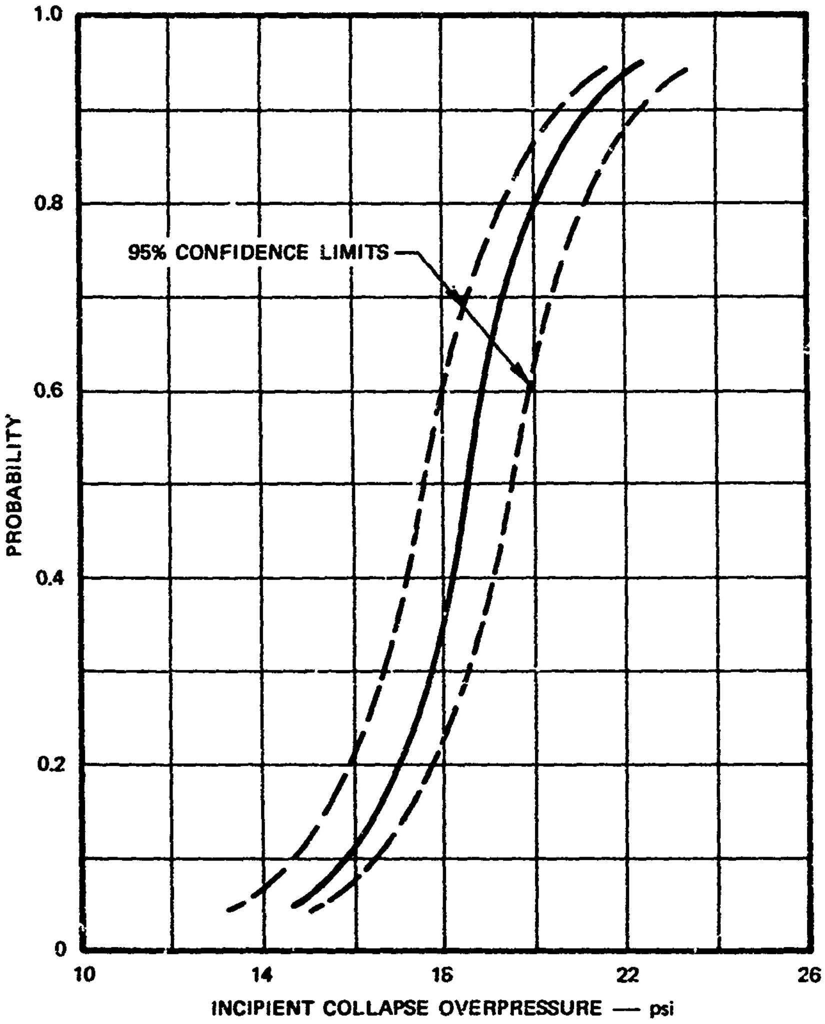


FIGURE 4 PROBABILITY OF OCCURRENCE OF PEAK INCIDENT OVERPRESSURE AT INCIPIENT COLLAPSE FOR WILBUR HALL #6

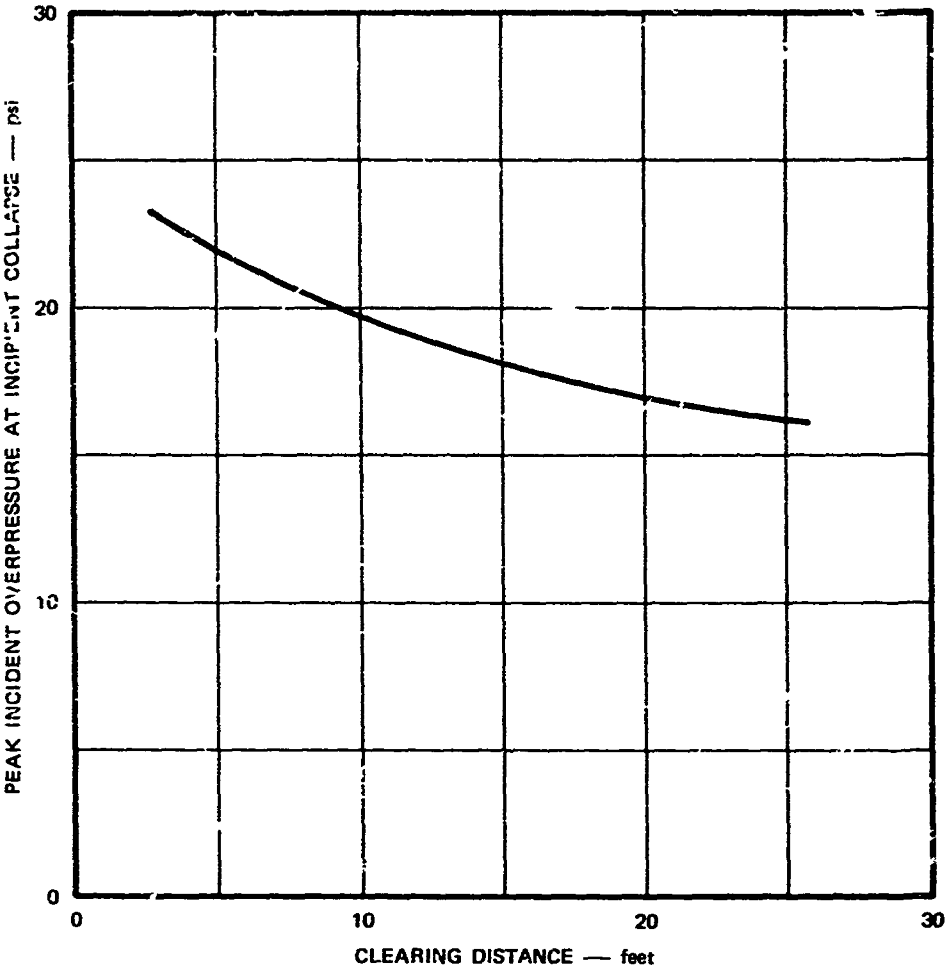


FIGURE 5 EFFECT OF CLEARING DISTANCE ON INCIPIENT COLLAPSE OVERPRESSURE FOR WILBUR HALL #6

III BUILDING ANALYSIS--DETROIT

Introduction

The analysis of each of the five Detroit NFSS buildings is presented in this section. In each subsection, a description of the building is given, together with a copy of the photographs provided by RTI. The building is described as it was designed, and therefore there may be some discrepancies between the building descriptions and the field survey data presented in the Appendix. Following the description, the analysis of the building is presented in two subsections; the first using the field survey data and the second using the building plan data.

The exterior walls for which collapse predictions were made were analyzed using the probability technique presented in Ref. 2. Therefore, the collapse values are given as having a 10, 50, or 90 percent probability of occurrence.

In general, the procedure used to make the collapse predictions was first to make a detailed examination of the field survey data, sketches, and photographs. From this information, the walls that were believed to be important to the failure of the structure or to the production of significant casualties were selected for analysis. Although it was not feasible to analyze every wall in all five buildings for this phase of the effort, the walls selected were representative for each building. The input data required in the computer programs consist of the wall and load properties, including probability distributions where needed. Although the geometric wall properties were usually available from the field survey data, the properties of the masonry materials were not available. Since this is generally the case for existing structures, it was necessary to assume values for the material properties required in the analysis. The material properties used in this study are summarized in Table 1; they were based on previous data.

After the walls were analyzed using the field survey data, the building plans were examined in detail and a new set of input data was prepared for each building. The properties of the masonry materials were usually not specified on the plans, and therefore, the values in Table 1 were also used for the building plan data analysis. However, data on the reinforcing steel used in the concrete walls were generally given on the plans, and the values used in the analysis are presented in the appropriate building analysis subsections.

An important factor in the prediction of the collapse of a structure is the method used to determine the transient blast loading. For this study, the front face, interior, and net loading on each wall was calculated by the procedure discussed in Ref. 2. It was assumed that each wall being analyzed was struck at normal incidence by a plane Mach waveform created by a 1 Mt surface burst: that is, each wall was analyzed as though it were the "front face" of the building with an ideal blast wave advancing at normal incidence to it. For this limited study, it was not possible to analyze the side and rear walls for the effect of a blast wave engulfing the structure. As noted in Ref. 2, because of the time relationship between the interior and exterior blast pressures and the design of some wall elements, it is possible that a side or rear wall of a structure may be expected to collapse at a lower incident overpressure than that predicted for the front wall.

Table 1

STRUCTURAL PROPERTIES OF MASONRY MATERIALS

Material	γ (pcf)	f_r (psi) Mean	f_r (psi) Standard Deviation	f'_m (psi) Mean	f'_m (psi) Standard Deviation	E_m* (psi)	t_f (in.)
Brick	120	100	42	2000	600	1.0×10^6	--
Concrete	145	$8\sqrt{f'_c}$	0	3750	0	3.0×10^6	--
Concrete block, 4 in.	90	60	25	1200	350	1.0×10^6	1.375
Concrete block, 8 in.	83	60	25	1200	350	1.0×10^6	1.500
Concrete block, 12 in.	80	60	25	1200	350	1.0×10^6	1.750
Structural clay tile, 4 in.	75	50	20	1750	450	$.75 \times 10^6$	0.75
Structural clay tile, 6 in.	60	50	20	1750	450	$.75 \times 10^6$	0.75
Structural clay tile, 8 in.	50	50	20	1750	450	$.75 \times 10^6$	0.75

* Values given are for analyzing an unreinforced masonry wall without arching. For walls in which arching occurs, a mean value of $E_m = 1000\, f'_m$ and a standard deviation of $300\, f'_m$ are to be used.

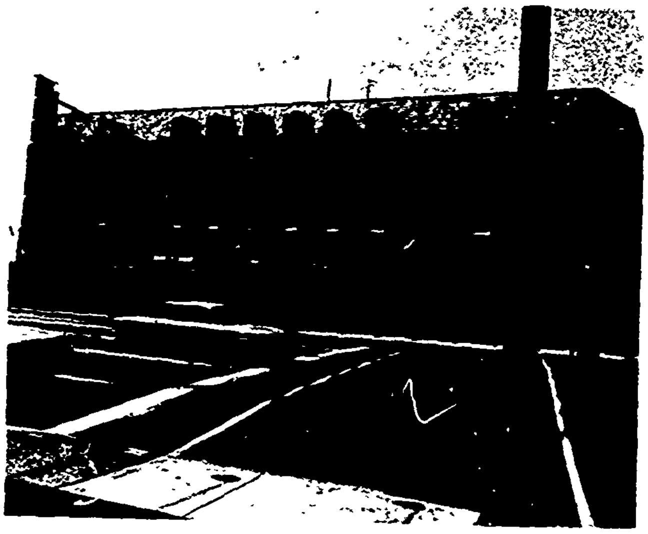
Montgomery Ward Store

Description

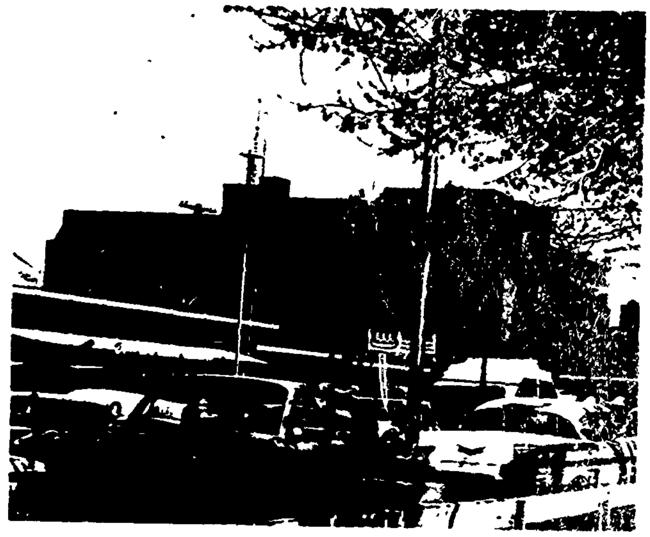
The Montgomery Ward store is located at 14455 Gratiot Avenue, Detroit, Michigan, and was constructed in 1939. The building consists of three stories and an unexposed basement, with a mezzanine between the first and second floors. The overall height of the building is about 49 ft with plan dimensions of 151 ft by 175 ft (avg), which provide an area of about 26,100 sq ft per floor. As noted on Figure 6, the ground floor exterior wall has large show windows on the two sides facing the street, whereas on side B, which is windowless, the first story is shielded by an adjacent structure. The second and third stories have windows in all walls except side B, but the percent openings on side C is much less than on sides A and D. The mezzanine is windowless and extends about 10 ft into the first floor area.

The structural steel frame is of riveted and bolted construction, with tile fireproofing for the columns and concrete for the beams. The floor consists of a 3- or 4-in. thick concrete slab with a 3-in. thick terrazzo decking and is supported primarily on steel joists spanning between the frame beams. However, a small portion of the floor is supported by a reinforced concrete ribbed slab.

The exterior walls are constructed of a 4-in. brick facing backed with an 8-in. structural clay tile, with a decorative stone treatment on sides A and D. The walls are unreinforced, and the 8-in. clay tile is inset in the structural frame, while the brick facing is continuous over the columns and beams. The first floor exterior wall below the mezzanine floor on sides A and D consists of 3-ft wide wall elements backed by columns with the 15-ft wide windows between. The interior partitions in the basement are 8-in. concrete or unreinforced concrete block and on all other floors are 4-in. timber studwall.

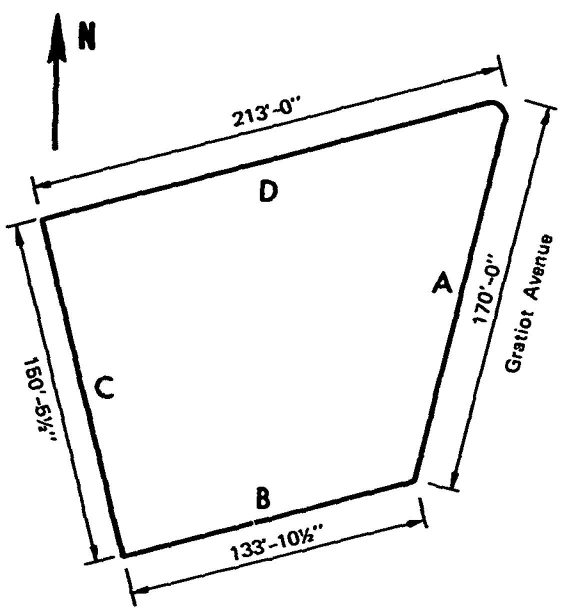


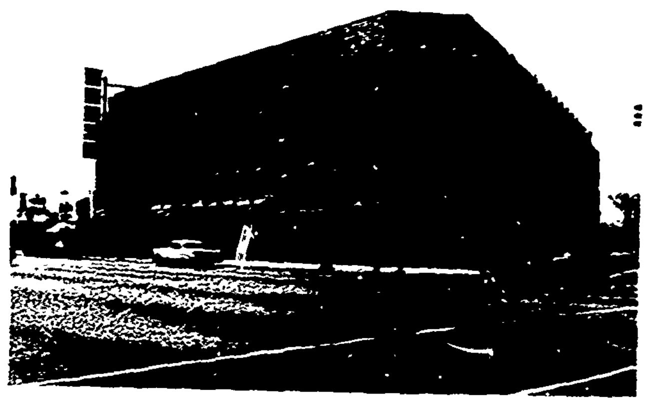
SIDE A



SIDES B and C

NOT REPRODUCIBLE





SIDES A and D

SOURCE: RTI.

FIGURE 6 PHOTOGRAPHS AND PLOT PLAN OF MONTGOMERY WARD STORE

Analysis

Field Survey Data. During the on-site survey, the exterior walls were classified as tile panel walls with masonry veneer. Since the support conditions for the walls were not given and since panel walls are defined as "nonload bearing walls that · :upported by the structural framework of the building at each floo: .1," it could not be determined from the survey data whether the walls could arch between columns and beams. Therefore, the wall support was treated as an unknown parameter, and it was necessary to perform an analysis for both an arching and non-arching wall.

If each wall were considered to be oriented normal to the direction of propagation of the blast wave, an examination of the survey data and photographs would indicate that the weakest wall is probably the windowless exterior wall on side B above the first floor level.

For sides A and D, the relatively large window area at the ground floor level, the rapid diffraction of the blast wave around the 3-ft wide wall elements between windows and the lateral support provided by the mezzanine floor would result in a high predicted collapse overpressure for the first floor wall. Therefore, the collapse overpressure for these walls was not calculated since the protection provided by the exterior walls on the first floor would be no greater than the strength of the window glass. Although both the second and third floor walls of sides A and D have window openings, the third floor wall is constructed as a sloping roof and only the second floor wall could be analyzed by the methods developed in this program. Since side C is of similar construction to side B, except that it does have some windows, it was not analyzed since its collapse overpressure was estimated to be between that for sides A and B.

Using the information from the on-site survey, it was found necessary to analyze the following five cases to estimate the collapse overpressure of the Montgomery Ward building:

F1. Side A, first floor. Controlled by strength of window glass.

F2. Side A, second floor. One-way unreinforced masonry wall with fixed-edge supports and without arching.

F3. Side A, second floor. One-way arching wall.

F4. Side B, second floor. Two-way unreinforced masonry wall with simple supports and without arching.

F5. Side B, second floor. Two-way arching wall.

For each of the four wall cases, the 4-in. brick and 8-in. tile were assumed to be sufficiently bonded to develop the bending or arching strength of a 12-in. thick wall. The dimensions and wall properties used in the analysis are given in Table 2.

The results of the analysis using the field survey data are given in the following tabulation:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
F1	Controlled by strength of window glass			
F2	0.6	0.1	0.5	0.7
F3	3.6	0.8	2.6	4.7
F4	0.6	0.0	0.5	0.6
F5	4.3	1.0	3.0	5.6

Table 2

MONTGOMERY WARD STORE
WALL PROPERTY DATA

Case	Location Side	Location Floor	Wall Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	S (ft) Mean	S (ft) Standard Deviation	Number of Openings	Area of Openings (sq ft)	Location	Delay (msec)	Room Volume (cu ft)
Field survey data														
F1	A	1	U-5	Brick (4) Tile (8)	12	288	-							
F2	A	2	U-6	Brick (4) Tile (8)	12	144	-	15.8	7.3	1	400	Front	3	317,100
F3	A	2	A One-way	Brick (4) Tile (8)	12	144	-	15.8	7.3	1	400	Front	3	317,100
F4	B	2	U-1	Brick (4) Tile (8)	12	144	264	27.0	0	2	50/100	Side/ Side	18/38	317,100
F5	B	2	A Two-way	Brick (4) Tile (8)	12	144	264	27.0	0	2	50/100	Side/ Side	18/38	317,100
Building plan data														
P1	A	2	A One-way	Brick (4) Tile (8)	12.5	138	-	15.8	7.3	1	440	Front	3	341,500
P2	B	2	A Two-way	Brick (4) Tile (8)	12.5	138	252	27.0	0	2	55/110	Side/ Side	20/40	341,500

* See Table 10 for a key to wall types.

As noted in the tabulation, for the second story wall of side A the mean predicted collapse overpressure for case F2 is 0.6 psi and for case F3 is 3.6 psi; the values are different by a factor of about six. For the second story wall for side B, the mean predicted collapse overpressure for case F4 is 0.6 psi and for case F5 is 4.3 psi, which differ by a factor of about seven. This large discrepancy, of course, results from the difference in the assumed support conditions and indicates the importance of obtaining definitive wall support information in any proposed survey. Unfortunately, for this study, the survey data did not provide sufficient information to determine which predicted collapse overpressure is the more realistic. However, it should be noted that the results of a more recent survey and analysis exercise indicate that, for most buildings, the wall support condition can be determined by the on-site survey team.

Building Plan Data. An examination of the building plans showed that the 8-in. tile backing for the exterior walls is inset between the structural steel columns and beams and that the 4-in. brick facing is continuous over the frame members. Therefore, under a lateral load, the wall would develop its resistance in arching. The overall wall thickness was 12-1/2 in. and, since the facing brick is bonded to the tile with a mortar joint, the total thickness of the wall is assumed to be effective in providing the resistance. The decorative stone facing on portions of the exterior walls did not affect the wall panels used in the analysis.

The specific walls analyzed for this phase are the same as those discussed under the survey data; however, with the information available from the building plans, it was only necessary to analyze the following two cases to estimate the collapse overpressure of the Montgomery Ward store:

P1. Side A, second floor. One-way arching wall.

P2. Side B, second floor. Two-way arching wall.

The dimensions and wall properties used in the analysis are given in Table 2. Note the minor differences with those used for the analysis using survey data.

The results of the analysis using the building plan data are given in the following tabulation:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
P1	4.1	1.0	2.8	5.4
P2	4.7	1.3	3.1	6.4

As can be seen in the tabulation, the analysis using the building plan data resulted in a 50 percent probability of collapse for the second story wall on side A at an overpressure of 4.1 psi and on side B at 4.7 psi. These values are approximately 20 percent higher than those obtained from the field survey data for these walls when analyzed with the same support conditions. This difference results from the variation in the dimensions and properties of the walls used in the two analyses and is minor when compared with the large differences in the predicted collapse overpressure resulting from the variation in the support conditions discussed in the previous subsection. This again points out that if valid predictions of the collapse of blast loaded walls in actual buildings are to be made using only field survey data, the important building parameters, i.e. the wall support conditions, must be obtained.

St. Stephen A.M.E. Church

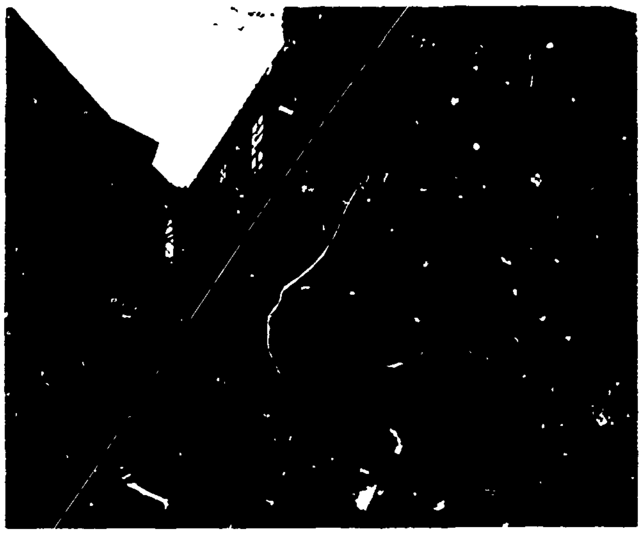
Description

The St. Stephen A.M.E. Church is located at 6000 Stanford Avenue, Detroit, Michigan, and was constructed in 1922. The portion of the church of primary interest for this study was the Community Building constructed adjacent to the north wall of the church in 1949. This building consists of two upper stories and a basement with a 5-ft exposure above grade. A gymnasium, which extends for two floors, occupies the major portion of the basement and first floor levels, with a few offices on the west side. The second floor is used as office and classroom space. The height of the Community Building is about 36 ft, and its plan dimensions are 111 ft by 91 ft, which provides an area of about 10,100 sq ft on both the basement and first floors and 8,600 sq ft on the second floor. Figure 7 shows the exterior walls and window location for sides A and B of the Community Building. The windows on side C, although not shown on the figure, are similar in size and location to those on side B.

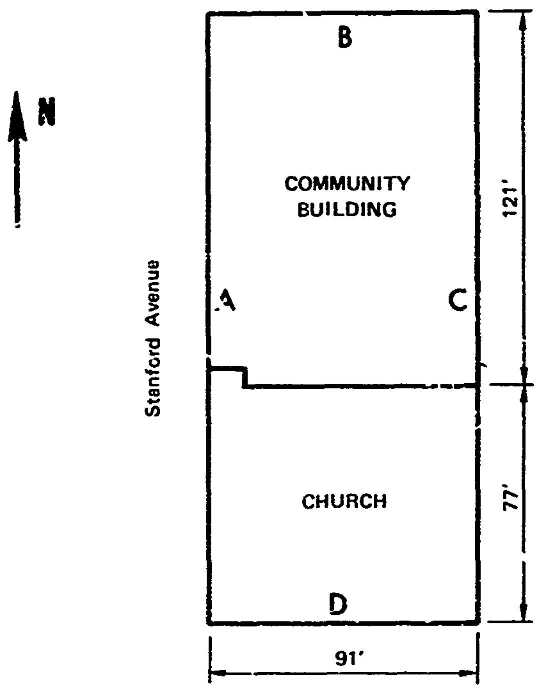
The Community Building has a structural steel frame, with plaster fire protection for all interior columns. The floors are 2-1/2-in. thick reinforced concrete slabs supported on steel joists that frame into the main beams and girders.

The exterior walls are constructed of a 4-in. brick facing backed with an 8-in. cinder block. The walls are unreinforced, and the 8-in. cinder block is inset in the structural frame, while the brick facing is continuous over the columns and beams. The interior partitions were constructed of 8-in. unreinforced concrete block.

SIDE A



SIDE B



SOURCE: RTI.

FIGURE 7 PHOTOGRAPHS AND PLOT PLAN OF ST. STEPHEN A.M.E. CHURCH AND COMMUNITY BUILDING

Analysis

Field Survey Data. During the on-site survey, the exterior walls of the Community Building were classified as nonreinforced concrete block curtain walls, both with and without masonry veneer, and were described as continuous over the supports. Since curtain walls are defined as "self-supporting exterior walls which are independent of the frame, although they are usually laterally anchored to the frame at each floor level," the walls were analyzed as unreinforced masonry unit walls, which were continuous over the supports and were nonarching.

An examination of the survey data and photographs indicated that the weakest walls in the Community Building were the 8-in. thick concrete block walls on sides B and C. Since these were the exterior walls enclosing the gymnasium area, they spanned two floor levels from the basement to the second floor lines. The collapse overpressure of the wall on side A would be expected to be greater than that of the walls on sides B and C since the wall was supported at each floor level and also it was 12 in. thick. The exterior walls enclosing the church area were not analyzed in this study.

To estimate the collapse of the Community Building, either the wall on side B or C could be analyzed since both walls are the same thickness and span and the window openings into the gymnasium are the same. Therefore, only the following case was considered:

F1. Side C, basement and first floor. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

Even though 5 ft of the gymnasium wall are below grade, for analysis purposes, it was assumed that the wall was loaded uniformly over its entire height by the air blast. This assumption would tend to underestimate the collapse overpressure, since the blast-induced soil loading on the portion of the wall below grade would not be expected to be as great as the loading produced by the air blast on the portion of the wall above ground. However, the error in the predicted collapse overpressure is believed to be small since as the wall deflects inward it would lose contact with the soil and the blast loading would become more uniformly distributed. The dimensions and wall properties used in the analysis are given in Table 3.

The results of the analysis using the field survey data are given in the following tabulation:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
F1	1.0	0.1	0.8	1.1

Building Plan Data. An examination of the building plans indicated two major differences between the design of the walls of the Community Building and the wall data obtained in the survey. First, the exterior walls on sides B and C have an overall thickness of 12 in. instead of the 8 in. as noted during the survey. The unreinforced masonry walls were constructed with a 4-in. brick facing bonded to an 8-in. concrete or cinder block backing. Second, the 8-in. masonry block backing is inset between the structural steel columns and beams, and the 4-in. brick facing is continuous over the frame members.

The specific wall analyzed was the same as for the analysis with the survey data, although the wall type was considered as arching, as follows:

P1. Side C, basement and first floor. Two-way arching wall.

The dimensions and wall properties used in the analysis are given in

Table 3

ST. STEPHEN A.M.E. CHURCH AND COMMUNITY BUILDING
WALL PROPERTY DATA

Case	Location Side	Location Floor	Wall Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	L_{VW} (in.)	L_{HW} (in.)	P_v (lb/in.)	S (ft) Mean	S (ft) Standard Deviation	Number of Openings	Area of Openings (sq ft)	Location	Delay (msec)	Room Volume (cu ft)
Field survey data																	
F1	C	1	U-2	Concrete block(8)	8	240	240	60	168	46	18.9	7.4	2	350/50	Front/side	3/20	158,200
Building plan data																	
P1	C	1	A two-way	Brick(4) Concrete block(8)	12	174	336	49	177	-	18.9	7.4	2	300/180	Front/side	3/45	187,440

* See Table 10 for a key to wall types.

Table 3. The wall was assumed to be uniformly loaded by the air blast, as discussed in the previous subsection.

The results of the analysis using the building plan data are given in the following tabulation:

Case	Predicted Collapse Overpressure, psi			
	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
P1	4.6	0.8	3.5	5.7

The mean predicted collapse overpressure for the wall that was analyzed using the building plan data is seen to be about five times greater than that obtained using the field survey data. As was noted for the Montgomery Ward store, the difference in the values for the Community Building was also primarily because of the difference in the assumed support conditions for the two analyses, although some of the difference can be attributed to the difference in the wall properties used, especially the wall thickness.

Detroit Housing Commission Apartments

Description

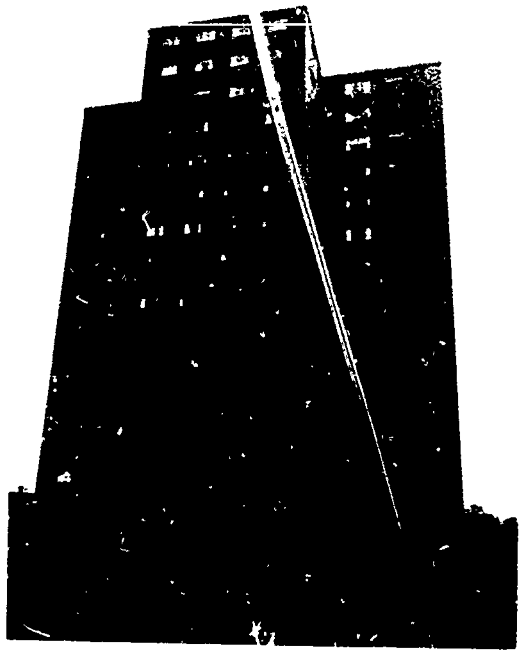
The Edward J. Jeffries Homes of the Detroit Housing Commission are located at 1121 W. Canfield Avenue, Detroit, Michigan, and were constructed in 1953 and 1954. Since all buildings in the complex are similar, the discussion and analysis of one building applies to the other buildings. The structure consists of 14 stories and a basement, or ground floor, with a 5-ft exposure above grade. The ground floor houses the utilities and other building facilities, and the upper floors consist of eight apartments each. The overall height of the building is 128 ft and the overall plan dimensions are 102 by 90 ft, which provide an area of about 6000 sq ft per floor. Figure 8 shows the exterior walls and window openings for the structure.

The building was constructed with a typical reinforced concrete frame and 6-in. thick solid reinforced concrete slab supported on reinforced concrete beams spanning between columns.

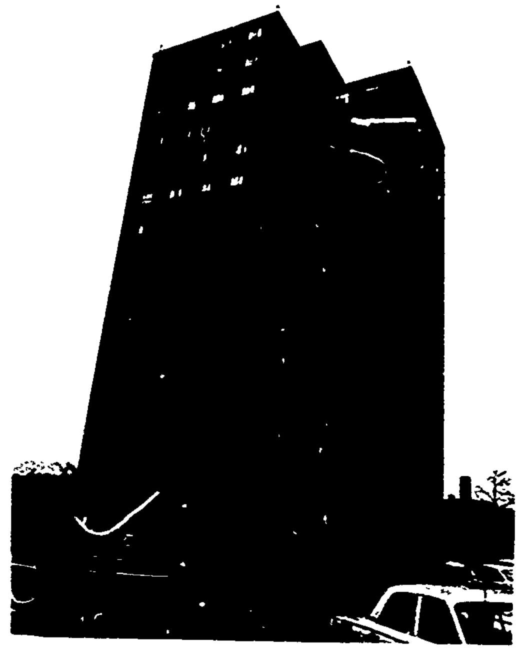
The exterior walls on the upper floors are constructed of a 4-in. brick facing backed with a 6-in. structural clay tile. The walls are unreinforced and the 6-in. clay tile is inset in the structural frame, while the brick facing is continuous over the columns and beams. The exterior walls on the ground, or basement story, are of reinforced concrete construction. Aboveground, the 8-in. thick concrete section is faced with 4-in. thick brick, whereas below ground, the brick is discontinued and the concrete thickness is increased to 13 in. The interior partitions on the ground floor are constructed of 4-in. clay tile or unreinforced concrete block, and on all other floors are 4-in. clay tile, 4-in. unreinforced concrete block, or 2-in. timber studwall.

Analysis

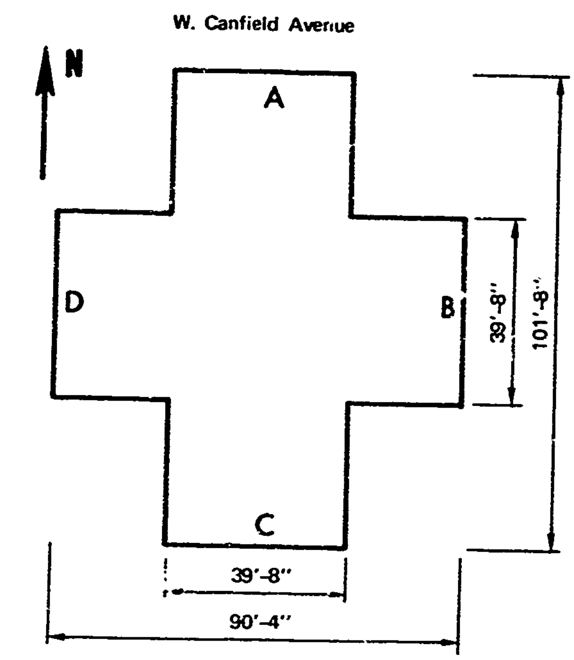
Field Survey Data. During the on-site survey, the exterior walls were classified as concrete panel walls with masonry veneer for the ground floor, and tile panel walls with masonry veneer for all upper floors. Even though the walls were described as panel walls, it was stated in the field survey that the observed and estimated support conditions were that the tile was inset in the frame and the brick facing was continuous over the supports. Because of this apparent anomaly, it was decided to analyze the unreinforced masonry walls on the upper floors for both arching and nonarching supports.



SIDE A



SIDES B and C



SIDE D

SOURCE. RTI.

FIGURE 8 PHOTOGRAPHS AND PLOT PLAN OF DETROIT HOUSING COMMISSION EDWARD J. JEFFERIES HOMES

The walls and window arrangements were similar for all sides of the building, and therefore the wall on one side only was analyzed. However, since the window arrangement and horizontal span of the wall on any floor level varied between the central portion of the building and the outer portions, the two walls were analyzed separately. Therefore, using the field survey data, it was necessary to analyze the following five cases to estimate the collapse overpressure of the Detroit Housing Commission Apartments:

F1. All sides, upper floors, central portion of the building.* Two-way arching wall.

F2. All sides, upper floors, central portion of the building. Two-way unreinforced masonry unit wall with simple supports.

F3. All sides, upper floors, outer portion of building.* Two-way arching wall.

F4. All sides, upper floors, outer portion of building. Two-way unreinforced masonry unit wall with simple supports.

F5. All sides, ground floor. Two-way reinforced concrete wall with fixed-edge supports.

Although it was stated in the survey data that the walls on the upper floors were constructed with a 4-in. thick brick facing backed with an 8-in. thick tile, it was also noted that the overall wall thickness was measured as 14 in. For the analysis, it was assumed that this difference in thickness was because of the thickness of the interior wall finish and that the brick and tile were well bonded with a mortar joint.

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* The central portion of the building is that portion which appears as the center wing of the building when it is viewed in elevation (see Figure 8). The outer portion of the building is that portion which appears as the outer wings of the building when it is viewed in elevation.

Therefore, it was assumed that the 12-in. thickness was effective in developing either the arching or bending resistance of the wall. The dimensions and wall properties used in the analysis are given in Table 4, and the steel reinforcement data are shown in Table 5.

The results of the analysis using the field survey data are given in the following tabulation:

Case	Predicted Collapse Overpressure, psi			
	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
F1	15.2	2.7	11.8	18.6
F2	2.4	0.3	2.1	2.7
F3	12.7	2.2	10.0	15.5
F4	2.2	0.3	1.8	2.5
F5	9.0	1.1	7.7	10.4

Building Plan Data. An examination of the building plans indicated several differences between the design of the apartment buildings and the data obtained in the field survey. The exterior walls on the upper floors were found to be constructed of a 4-in. brick facing backed with a 6-in., rather than an 8-in., structural clay time. The walls are unreinforced and the 6-in. tile is inset in the frame, while the brick was continuous over the columns and beams. The overall thickness of the upper story walls was found to be 11 in., rather than the 14 in. noted in the survey data, and the brick facing is bonded to the tile with a mortar joint.

The exterior wall on the ground floor was found to have been designed with an overall thickness of 13 in. Aboveground the wall is constructed of 8-in thick reinforced concrete faced with 4-in. thick brick with a 1-in. wide cavity. Just below grade, the concrete section is increased

Table 4

DETROIT HOUSING COMMISSION APARTMENTS
WALL PROPERTY DATA

Case	Location Side	Location Floor	Wall Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	L_{VH} (in.)	L_{HH} (in.)	S (ft) Mean	S (ft) Standard Deviation	Number of Openings	Area of Openings (sq ft)	Location	Delay (msec)	Room Volume (cu ft)
Field survey data																
F1	ABCD	All	A Two-way	Brick (4) Tile (8)	12	92	216	48	108	11.3	5.2	2	36/37	Front/side	3/17	5,440
F2	ABCD	All	U-1	Brick (4) Tile (8)	12	92	216	48	108	11.3	5.2	2	36/37	Front/side	3/17	5,440
F3	ABCD	All	A Two-way	Brick (4) Tile (8)	12	92	157	24	54	17.7	8.1	2	37/36	Front/side	3/20	5,440
F4	ABCD	All	U-1	Brick (4) Tile (8)	12	92	157	24	54	17.7	8.1	2	37/36	Front/side	3/20	5,440
F5	ABCD	Basement	RC-.	Brick (4) Concrete (8)	12	110†	216	36	108	11.3	5.2	2	27/27	Front/side	3/20	5,440
Building plan data																
P1	ABCD	All	A Two-way	Brick (4) Tile (6)	10.375	85	200	49	112	11.3	5.2	2	38/40	Front/side	3/15	4,800
P2	ABCD	All	A Two-way	Brick (4) Tile (6)	10.375	85	157	26	56	17.7	8.1	2	40/38	Front/side	3/12	4,800
P3	ABCD	Basement	RC-2	Concrete	8	83	200	34	56	11.3	5.2	2	26/26	Front/side	3/15	5,280
P1′	Interior		A One-way	Concrete block	4	95	--	--	--	11.3	5.2	2	38/40	Front/side	3/15	4,800
P2′	Interior		A One-way	Concrete block	4	95	--	--	--	17.7	8.1	2	40/38	Front/side	3/15	4,800

* See Table 10 for a key to wall types.
† 5 ft aboveground.

Table 5

DETROIT HOUSING COMMISSION APARTMENTS
STEEL REINFORCEMENT DATA

Case	Support Case*	Section†	A_s (sq in./ft)	d (in.)	A_s' (sq in./ft)	d′ (in.)	Rebar Number	f_{dy} (psi) Mean	f_{dy} (psi) Standard Deviation
Field survey data									
F5	2	1	0.144‡	8.0	0	0	3	42,000	5,500
		2	0.240‡	8.0	0	0			
		3	0.144‡	4.0	0	0			
		4	0.240‡	4.0	0	0			
Building plan data									
P3	2	1	0.312‡	6.2	0	0	5	52,000	6,500
		2	0.312‡	5.5	0	0			
		3	0.312‡	1.8	0	0			
		4	0.312‡	2.5	0	0			

* See Table 10 for a key to support cases.
† Ref. 2
‡ Estimated

to 13 in. thick to provide a 5-in. wide shelf for supporting the brick facing. Because of the cavity between the brick and concrete, the effective thickness of the wall was assumed to be 8 in. for analysis purposes and the effective vertical span was assumed as the distance between the bottom of the first floor beam and the shelf in the concrete wall.

The specific exterior walls analyzed for this phase were the same as those discussed under the field survey data; however, with the information available from the building plans it was only necessary to analyze the following three cases to estimate the collapse overpressure:

P1. All sides, upper floors, central portion of building. Two-way arching wall.

P2. All sides, upper floors, outer portion of building. Two-way arching wall.

P3. All sides, ground floor. Two-way reinforced concrete wall with fixed-edge supports.

The dimensions and wall properties used in the analysis are given in Tables 4 and 5.

Since mortality predictions in a large multistory building were of special interest to OCD, the interior walls of these apartments were also analyzed. The interior walls analyzed were the 4-in. concrete block walls separating adjacent apartments and between each apartment and the hall or stairway. Figure 9 is a plan showing the location of these interior walls on the upper stories; the interior studwalls within each apartment are not shown. Because of symmetry, it was only necessary to analyze the walls for a blast wave approaching from one direction. However, since the interior walls are loaded by the room pressure and since the room pressure is influenced by the orientation of the room within the

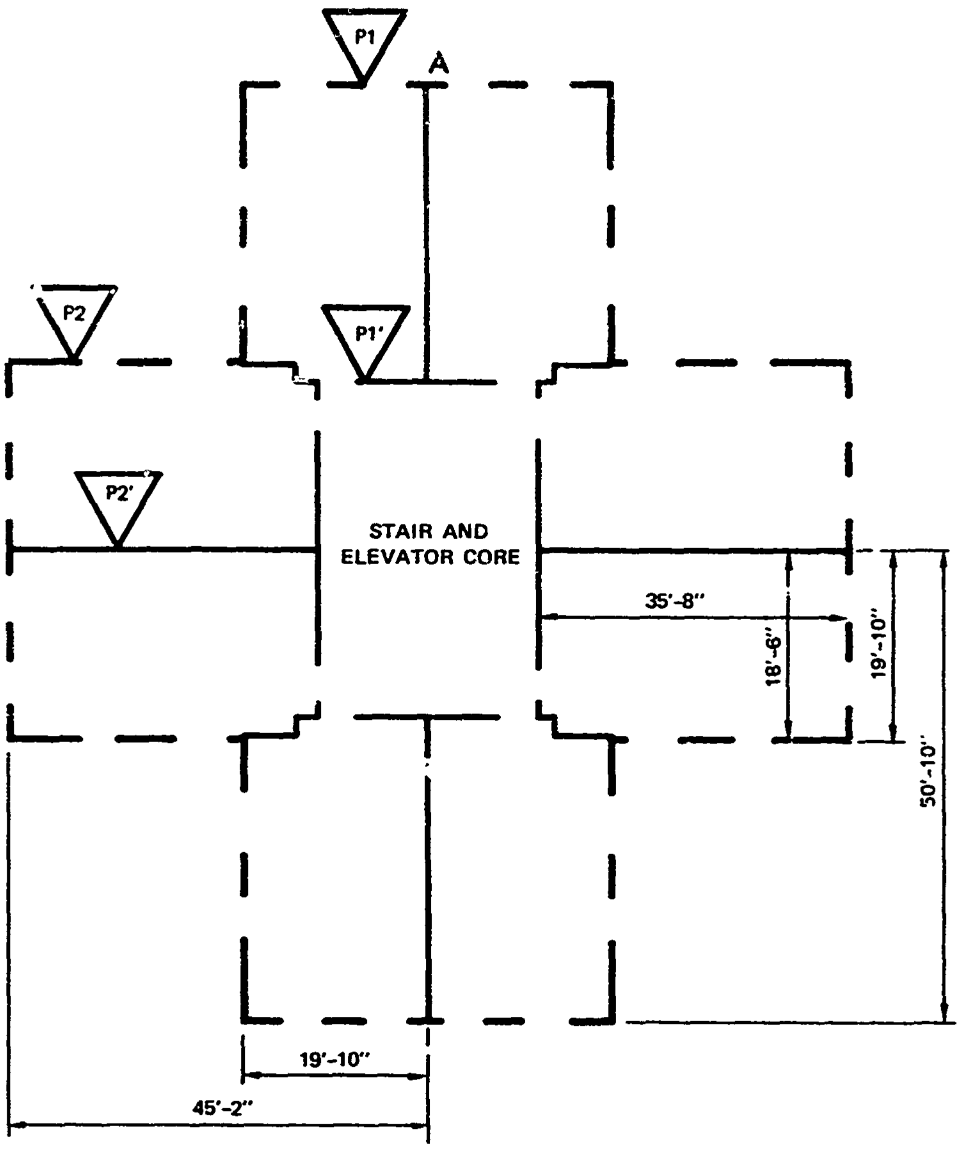


FIGURE 9 PLAN VIEW OF INTERIOR WALLS ON UPPER STORIES
DETROIT HOUSING COMMISSION APARTMENTS

building, it was necessary to determine the collapse overpressure for the following two cases:

P1′. All sides, upper floors, between adjacent apartments. Interior one-way arching wall.

P2′. All sides, upper floors, between apartment and hall or stairway. Interior one-way arching wall.

These walls are indicated on Figure 9 for a blast wave striking side A. The dimensions and wall properties used in the analysis are given in Table 4.

The results of the analysis using the building plan data are given in the following tabulation:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
P1	15.5	3.0	11.6	19.4
P2	12.1	2.3	9.2	15.0
P3	16.2	2.5	13.0	19.4
P1′	1.3*			
P2′	1.1*			

As can be seen in the tabulation, for the exterior arching walls on the upper floors, the mean collapse overpressure using the survey data are within a few percent of those obtained using the building plan data. This

* Only the mean value was obtained for the collapse overpressure for the interior walls.

close agreement is somewhat misleading, since it is partially because of the effect of the different wall thickness and span lengths used in the two analyses, which tended to compensate for each other. For the reinforced concrete wall on the ground floor level, the predicted collapse overpressure using plan data is approximately 80 percent higher than that found when using the survey data. This large difference results from the difference in the wall thickness and span assumed for the two analyses.

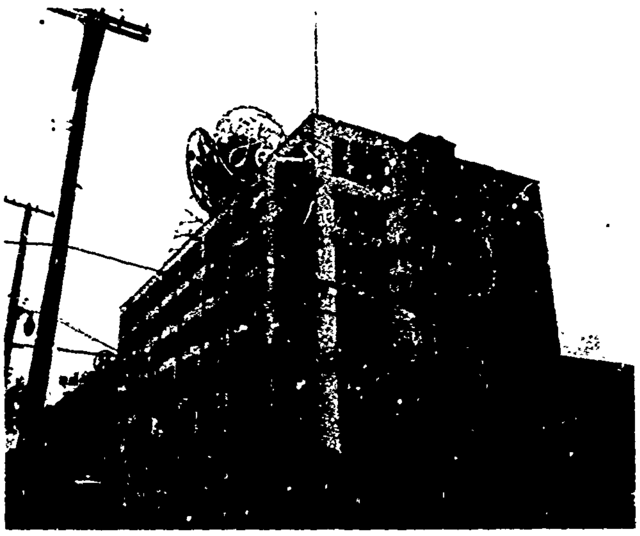
As discussed in Section 1, to be able to use the exterior wall models for predicting building collapse, it was necessary to assume for the analysis that the structural frame did not collapse. Since the incident overpressure required to collapse the exterior wall is about 16 psi for the mean value, the structure will be subjected to large lateral forces during both the diffraction and drag phases, for which it was not designed. Since the overall height of the building is 128 ft, it is possible that the frame may experience a failure at a lower overpressure than that predicted for the collapse of the exterior walls.

General Electric Service Building

Description

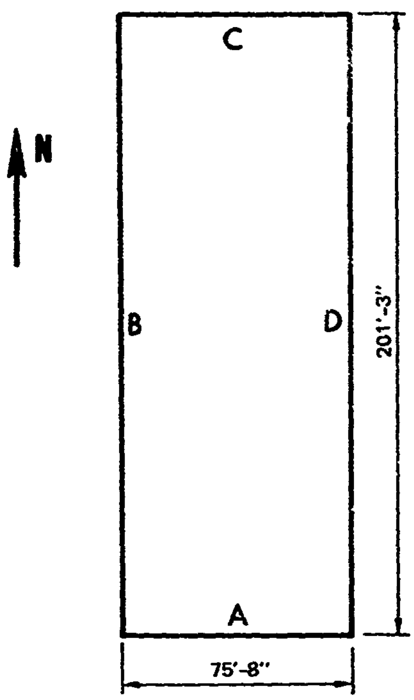
The GE Service Building is located at 700 Antoinette, Detroit, Michigan, and was constructed in 1924 for use as a warehouse. The building consists of five stories, with an overall height of about 81 ft and plan dimensions of 76 by 201 ft, which provides an area of about 15,200 sq ft per floor. There is no basement. As noted on Figure 10, the exterior walls on the first four floors have large window areas, e.g., the windows on the second floor of side B comprise 51 percent of the wall area. The fourth and fifth floors have been converted to office use.

The structural framing is of conventional reinforced concrete flat slab construction with column capitals and drop panels. The reinforced



SIDES A and B

SIDES A and D



Antoinette Avenue

SOURCE: RTI.

FIGURE 10 PHOTOGRAPHS AND PLOT PLAN OF GENERAL ELECTRIC SERVICE BUILDING concrete floor slabs were designed for warehouse type floor loads, which resulted in the slab thickness varying between 10-1/2 in. on the fifth floor to 13-1/4 in. on the second floor.

The exterior walls were primarily made up of the exterior columns, which were generally 4 ft 1 in. wide by 2 ft thick. The spandrels below the windows were constructed of reinforced concrete on the first and fifth floors, and 8-1/2-in. thick masonry units inset between columns on all other floor levels. The interior partitions on the fourth floor are of the light metal movable type, and on the fifth floor are of permanent 4-in. studwall construction.

Analysis

Field Survey Data. During the on-site survey, the exterior walls were classified as 12-in. thick concrete curtain walls without masonry veneer. However, under the type of frame category, the survey data also described wall columns that were 24-in. thick by 49-in. wide reinforced concrete members. Because of this apparent anomaly, and since the support conditions were described as curtain wall, it was assumed for the analysis that the exterior walls were 24-in. thick nonload bearing reinforced concrete curtain walls that were continuous over the floor supports. It was also assumed that the 12-in. thick reinforced concrete curtain walls described in the survey data were the spandrel walls under the large window openings.

For a blast wave at normal incidence to the exterior wall, the weakest wall would be the first story wall since the span between the first and second floors is greater than that between any other two floor lines. Because of the wide window openings and large open area in the interior of the building, room filling would probably not occur to any significant degree, and the blast loading on the wall members was therefore calculated by assuming that the 49-in. wide wall elements were isolated from each other. The collapse overpressure for the 12-in. thich reinforced concrete spandrels, which were inset between the exterior wall elements, was not calculated. Since the walls on the four sides of the building were similar, only the following two walls were analyzed:

F1. Side B, first floor. One-way reinforced concrete wall, with fixed-edge supports.

F2. Side B, fourth floor. One-way reinforced concrete wall, with fixed-edge supports.

The dimensions and wall properties used in the analysis are given in Table 6, and the steel reinforcement data in Table 7.

The results of the analysis using the field survey data are given in the following tabulation:

Case	Predicted Collapse Overpressure, psi: Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
F1	19.9	0.4	19.4	20.4
F2	24.0	1.1	22.7	25.4

Building Plan Data. An examination of the building plans indicated two major differences between the design of the exterior walls of the building and the interpretation of the wall data obtained from the field survey. First, the vertical exterior wall located on the sides of the window openings and extending the full height of the building are the exterior columns of the building and are reinforced accordingly. Second, the spandrel walls on the first and fifth floors are constructed of

Table 6

GENERAL ELECTRIC SERVICE BUILDING
WALL PROPERTY DATA

	Location		Wall						S (ft)	
Case	Side	Floor	Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	P_V (lb/in.)	Mean	Standard Deviation
Field survey data										
F1	B	1	RC-6	Concrete	24	255	-	5,060	2.0	0
F2	B	4	RC-6	Concrete	24	171	-	1,840	2.0	0
Building plan data										
P1	BD	1	RC-6	Concrete	24	240	-	6,490	2.0	0
P2	BD	4	RC-6	Concrete	24	129	-	2,250	2.0	0
P3	BD	4	A One-way	Brick	8.5	-	252	-	2.5	0
P3A	BD	4	U-8	Brick	8.5	42	-	0	2.5	0

* See Table 10 for a key to wall types.

Table 7

GENERAL ELECTRIC SERVICE BUILDING
STEEL REINFORCEMENT DATA

Case	Support Case*	Section†	A_s (sq in./ft)	d (in.)	A_s' (sq in./ft)	d′ (in.)	Rebar Number	f_{dy} (psi) Mean	f_{dy} (psi) Standard Deviation
Field survey data									
F1, F2	6	1	0.288	22.0	0.288	2.0	3	42,000	5,500
		3	0.288	22.0	0.288	2.0			
Building plan data									
P1, P2	6	1	2.448	21.5	2.448	2.5	10	42,000	5,500
		3	2.448	21.5	2.448	2.5			

* See Table 10 for a key to support cases.
† Ref. 2

reinforced concrete, and are 17- and 14-1/2 in. thick, respectively. The first floor spandrel extends to the foundation, and the fifth floor spandrel is constructed monolithically with the floor slab. The spandrels on the second through the fourth floors are 8-1/2-in. thick unreinforced masonry units placed directly on top of the floor slab. All masonry unit spandrel walls are inset between columns.

The walls analyzed were the wall columns analyzed previously, and the spandrel wall on the fourth floor. Since the sill height of all masonry unit spandrels is either 3 ft 8-1/2 in. or 3 ft 10 in., the collapse overpressure would be the same for this type of wall on all floors. However, since the masonry spandrels could develop either a horizontal arching action between columns or a vertical cantilever action from the slab, the spandrels were analyzed for both modes. The specific walls analyzed were as follows:

P1. Side B, first floor. One-way reinforced concrete wall, with fixed-edge supports.

P2. Side B, fourth floor. One-way reinforced concrete wall, with fixed-edge supports.

P3. Side B, fourth floor. One-way horizontal arching masonry unit spandrel wall.

P3A. Side B, fourth floor. Cantilevered unreinforced masonry unit spandrel wall.

The dimensions and wall properties used in the analysis are given in Tables 6 and 7.

The results of the analysis using the building plan data are given in the following tabulation:

Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
	Predicted Collapse Overpressure, psi			
P1	46.9	2.9	42.2	50.6
P2	92.5	8.7	81.4	103.6
P3	0.7	0.5	0.1	1.3
P3A	0.3	0.2	0.1	0.5

The mean predicated collapse overpressure values for side B, using the plan data, were found to be 46.9 psi for the first floor wall and 92.5 psi for the fourth floor wall. These values can be compared directly with the values of 19.9 and 24.0 psi obtained for the same walls, respectively, using the field survey data. The differences in the predictions are primarily the result of the difference in the assumed steel ratios and wall spans for the two cases as can be seen in Tables 6 and 7. In the survey data analysis, the exterior 49-in. wide walls were assumed to be curtain walls with a nominal 0.20 percent steel reinforcement, whereas the building plans indicated that the walls were the exterior columns with a steel ratio of 1.70 percent.

Because of the high values for the predicted collapse overpressure for the exterior walls, it is well to remember that to make collapse predictions using the interim wall models, it was necessary to assume that the building would not collapse at a lower overpressure than that predicted for the exterior walls. This may not be a reasonably valid assumption for this case, where it is conceivable that the building could collapse in some other mode, e.g., overturning or foundation failure, before the predicted collapse of the heavily reinforced exterior walls. Therefore, for estimating the mortality for this building, the collapse of the cantilevered masonry unit spandrel walls on the second, third, and fourth floors at 0.7 psi would be more meaningful.

Clara Barton Elementary School

Description

The Clara Barton School is located at 8535 Otto Avenue, Detroit, Michigan, and was constructed in 1946. The building consists primarily of one-story classrooms, although there is a mezzanine floor and an unexposed basement over part of the building area. The overall height of the school varies from 16 ft to 22 ft, with overall plan dimensions of 134 ft by 224 ft, which provide an area of about 19,400 sq ft for the first floor, 2,300 sq ft for the basement, and 1,200 sq ft for the mezzanine floor. Figure 11 shows a location plan for the school, and Figure 12 shows the window and wall area on the four sides of the building.

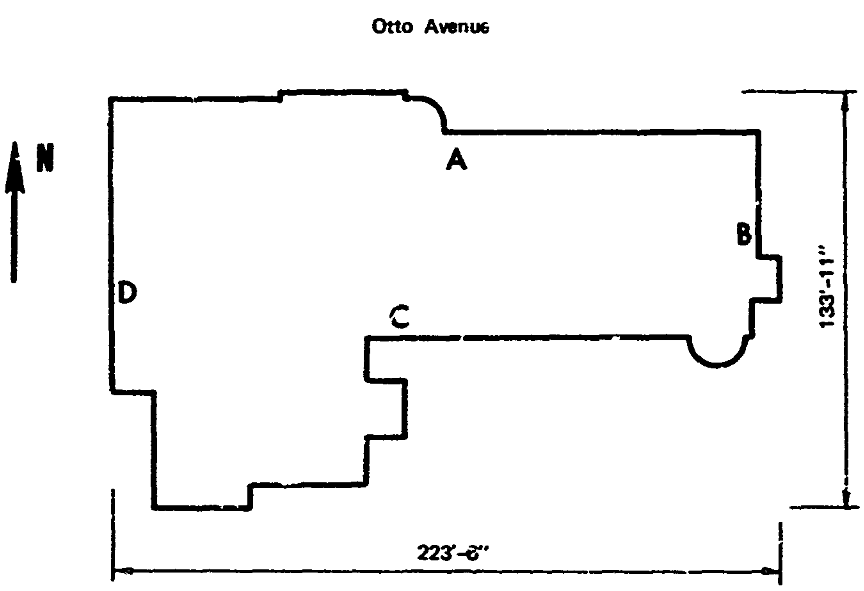


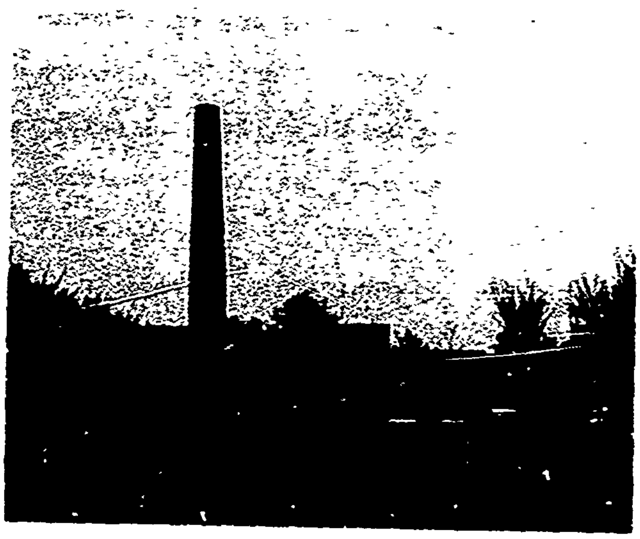
FIGURE 11 PLOT PLAN OF CLARA BARTON ELEMENTARY SCHOOL

SIDE A

SIDE B

NOT REPRODUCIBLE

SIDE C



SIDE D

SOURCE: RTI.

FIGURE 12 PHOTOGRAPHS OF CLARA BARTON ELEMENTARY SCHOOL

The school was constructed with a reinforced concrete frame. The first floor over the basement area is a 6-in. thick reinforced concrete slab supported on reinforced concrete beams.

The exterior walls are constructed of a 4-in. brick, or stone facing, which is backed with 4- or 8-in. thick cinder block. The walls are unreinforced and the cinder block is inset in the structural frame, while the brick or stone facing is continuous over the columns and beams. The interior partitions are generally either 6- or 8-in. thick, and are constructed of cinder block or structural glazed tile.

Analysis

Field Survey Data. During the on-site survey, the exterior walls were classified as unreinforced concrete block curtain walls with masonry veneer. Even though the walls were classified as curtain walls, it was also stated in the survey data that the 12-in. thick concrete block was inset in the frame and that the 4-in. thick brick was continuous over the frame members. Since the photographs indicated that the concrete block was inset in the frame, all exterior walls were analyzed as arching walls. It was assumed that the 4-in. thick brick facing was well bonded to the concrete block with a mortar joint, and therefore the total wall thickness of 16 in. would be effective in providing the wall resistance.

For this structure, the collapse overpressure of the exterior walls on all four sides was determined for a blast wave at normal incidence to each side. The large windows in the classroom area on side A in Figure 12 are similar to those on side C, and therefore, an analysis of the wall on side A would apply to both sides. The narrow vertical wall between the windows is located at the column line, and the wall below the window opening would therefore be expected to collapse at a lower overpressure.

Since the wall was inset between columns, it was analyzed as a horizontal arching wall. The analysis of the windowless wall adjacent to the entrance on side B would also apply to a similar entrance area on side C. To determine the collapse overpressure for the gymnasium wall on side D, the wall below the windows was analyzed for horizontal arch action between columns.

Using the information from the field survey, the following three walls were analyzed to estimate the collapse overpressure of the school:

F1. Side A, first floor. One-way horizontal arching wall.

F2. Side B, first floor. Two-way arching wall.

F3. Side D, first floor. One-way horizontal arching wall.

The dimensions and wall properties are given in Table 8.

The results of the analysis using the field survey data are given in the following tabulation:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
F1	18.9	2.0	16.4	21.4
F2	15.2	3.0	11.3	19.1
F3	11.7	2.0	9.2	14.3

Building Plan Data. An examination of the building plans indicated one primary difference between the design of the exterior walls of the school and the data obtained from the field survey. The plans show that the effective thickness of the exterior walls on sides, A, C, and D was 8 in. rather than the 16 in. noted in the survey data. The walls on these sides are constructed with a 4-in. thick brick or stone facing, which is

Table 8

CLARA BARTON ELEMENTARY SCHOOL
WALL PROPERTY DATA

Case	Location Side	Location Floor	Wall Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	3 (ft) Mean	3 (ft) Standard Deviation	Number of Openings	Area of Openings (sq ft)	Location	Delay (msec)	Room Volume (cu ft)
Field survey data														
F1	A	1	A One-way	Brick (4) Concrete block(12)	16	-	144	4.0	0	1	192	Front	3	8,740
F2	B	1	A Two-way	Brick (4) Concrete block(12)	16	144	276	12.7	0	2	96/96	Side/ Side	9/19	8,740
F3	D	1	A One-way	Brick (4) Concrete block(12)	16	-	138	14.0	5.5	1	400	Front	3	45,600
Building plan data														
P1	A	1	A One-way	Brick (4) Concrete block (4)	8	-	160	9.4	4.2	1	223	Front	3	8,065
P2	B	1	A Two-way	Brick (4) Concrete block (8)	12	136	282	12.9	2.1	2	125/125	Side/ Side	10/17	8,085
P3	D	1	A One-way	Brick (4) Concrete block (4)	8	-	124	14.0	5.5	1	291	Front	3	43,200

* See Table 10 for key to wall types.

backed with a 4-in. thick cinder block. The cinder block is inset between column and beams, and the brick is continuous over the frame members. The apparent thickness of the wall is greater as a result of radiator heating ducts and structural glazed tile on the interior of portions of the building. However, there is a large cavity between the tile and the cinder block, making the tile ineffective in increasing the arching resistance of the wall. The wall adjacent to the entrance on side B is constructed with a 4-in. thick brick facing, which is backed with an 8-in. thick cinder block. The brick facing in all walls is bonded to the cinder block with mortar.

The specific walls analyzed were the same as those analyzed previously with the survey data and were as follows:

P1. Side A, first floor. One-way horizontal arching wall.

P2. Side B, first floor. Two-way arching wall.

P3. Side D, first floor. One-way horizontal arching wall.

The dimensions and wall properties used in the analysis are given in Table 8.

The results of the analysis using the building plan data are given in the following tabulation.

Case	Predicted Collapse Overpressure, psi			
	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
P1	5.6	1.0	4.3	6.8
P2	9.9	2.2	7.2	12.7
P3	4.3	0.9	3.2	5.4

The mean predicted collapse overpressure for the exterior walls analyzed with the field survey data range from 1.5 to 3.4 times greater than those obtained using the building plan data. This difference can be attributed to the differences in the wall properties assumed for the two analyses, especially the wall thickness.

IV SUMMARY AND DISCUSSION

Since the discussion of the Stanford building, Wilbur Hall #6, presented in Section II is complete in itself and since the building was not part of the field survey exercise, the remainder of this section is concerned only with a discussion of the five buildings located in Detroit.

The predicted collapse overpressure for all five Detroit buildings and for both the field survey and building plan analyses are summarized in Table 9. As noted in the discussion for each building in Section III, the difference in the collapse overpressure of a specific wall, when using the field survey or building plan data, can be attributed primarily to the difference in the assumed support conditions. Because the predicted pressures differed by factors as large as seven, it is important that the problem be resolved before the survey of additional sample buildings. For the first three buildings noted in Table 9, the large differences in overpressure values for the two analyses resulted from a lack of explicit information in the field survey data as to whether the wall backing wythe was inset in the structural frame. For the analysis of unreinforced masonry unit walls this information is critical, since it is assumed that an inset wall will develop its resistance through arching forces as a result of the edge restraint provided by the structural frame and adjacent wall panels. If the wall is not inset in the frame, then the development of the wall resistance is provided by the bending resistance and vertical in-plane forces, which is usually much less than for the arching case.

To resolve the problem of the type of wall support for this study, it was necessary either to interpret the field survey data or to perform analyses that included several of the most likely support conditions.

Table 9

SUMMARY OF WALL ANALYSES

Case*	Location Side	Location Floor	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
Montgomery Ward store								
F1	A	1	U-5	12	Controlled by glass failure.			
F2	A	2	U-6	12	0.6	0.1	0.5	0.7
F3	A	2	A-1 way	12	3.6	0.8	2.6	4.7
F4	B	2	U-1	12	0.6	0.0	0.5	0.6
F5	B	2	A-2 way	12	4.3	1.0	3.0	5.6
P1	A	2	A-1 way	12.5	4.1	1.0	2.8	5.4
P2	B	2	A-2 way	12.5	4.7	1.3	3.1	6.4
St. Stephen A.M.E. Church Community Building								
F1	C	B,1	U-2	8	1.0	0.1	0.8	1.1
P1	C	B,1	A-2 way	12	4.6	0.8	3.5	5.7
Detroit Housing Commission apartments								
F1	All	Upper Central	A-2 way	12	15.2	2.7	11.8	18.6
F2	All	Upper Central	U-1	12	2.4	0.3	2.1	2.7
F3	All	Upper Outer	A-2 way	12	12.7	2.2	10.0	15.5
F4	All	Upper Outer	U-1	12	2.2	0.3	1.8	2.5
F5	All	Ground	RC-2	12	9.0	1.1	7.7	10.4
P1	All	Upper Central	A-2 way	10.4	15.5	3.0	11.6	19.4
P2	All	Upper Outer	A-2 way	10.4	12.1	2.3	9.2	15.0
P3	All	Ground	RC-2	8	16.2	2.5	13.0	19.4
P1′	Interior		A-1 way	4	1.3	--	--	--
P2′	Interior		A-1 way	4	1.1	--	--	--

Table 9 (Concluded)

Case*	Location Side	Location Floor	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi: Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
General Electric Service Building								
F1	B	1	RC-6	24	19.9	0.4	19.4	20.4
F2	B	4	RC-6	24	24.0	1.1	22.7	25.4
P1	B	1	RC-6	24	46.9	2.9	42.2	50.6
P2	B	4	RC-6	24	92.5	8.7	81.4	103.6
P3	B	4	A-1 way	8.5	0.7	0.5	0.1	1.3
P3A	B	4	U-8	8.5	0.3	0.2	0.1	0.5
Clara Barton Elementary School								
F1	A	1	A-1 way	16	18.9	2.0	16.4	21.4
F2	B	1	A-2 way	16	15.2	3.0	11.3	19.1
F3	D	1	A-1 way	16	11.7	2.0	9.2	14.3
P1	A	1	A-1 way	8	5.6	1.0	4.3	6.8
P2	B	1	A-2 way	12	9.9	2.2	7.2	12.7
P3	D	1	A-1 way	8	4.3	0.9	3.2	5.4

* The prefix F identifies walls analyzed using field survey data, and P those analyzed using building plan data.

† Each wall is designated with a letter to identify the wall type and a number to identify the wall support condition. The key to the wall types and support cases are given in Table 10.

Table 10

WALL TYPE AND SUPPORT KEY

Letter	Wall Type
U	Unreinforced masonry unit wall
A	Arching wall
RC	Reinforced concrete wall

Number	Support Case
1	Two-way, simply supported on four edges.
2	Two-way, fixed on four edges.
3	Two-way, fixed on vertical edges; simply supported on horizontal edges.
4	Two-way, simply supported on vertical edges; fixed on horizontal edges.
5	One-way, simply supported on opposite edges.
6	One-way, fixed on opposite edges.
7	One-way, propped cantilever.
8	One-way, cantilever.

Neither of these alternatives proved entirely satisfactory. First, any interpretation of the survey data for an important parameter, i.e., the type of wall support, places the responsibility for a decision on the analyst that he cannot possibly resolve without the benefit of an on-site inspection of the building.

Second, if an analysis is performed for two or more possible wall support conditions, the additional analyses only provide quantitative results for more cases; they do not necessarily provide better results, since there is no rational way at the present time for the analyst to select the more meaningful collapse prediction. It is conceivable, however, that if sufficient statistical information were available to relate the support condition to various building parameters, e.g., the type and year of construction, then the type of support could be developed into a probability distribution for use in the analysis. For the present, it is recommended that the decision on the type of support for the exterior walls of a building be made at the time of the on-site survey.

Another discrepancy between field and plan data, which influences the analytical results, is the difference in the thickness of the exterior walls. Since some discrepancies were noted between the surveyed and designed thickness for the walls of four of the five buildings, it is apparent that the wall thickness of a completed building is a difficult parameter to measure. From an analysis standpoint, there are two distinct problems concerning the wall thickness. First, as noted for the Church Community Building, the survey indicated that the wall was 8 in. thick, whereas the drawings indicated a 12-in. thick wall. Since the wall was constructed with a 4-in. thick brick facing that was bonded with mortar to an 8-in. thick cinder block, the wall thickness for the analysis using the building plan data was 12 in. For this type of discrepancy to occur, there was an error in either the survey or the building plan data that could be resolved by a careful inspection of the building.

Second, for the Elementary School, the survey data showed an overall wall thickness of 16 in. and that the wall was constructed of a 4-in. thick brick facing and a 12-in. thick concrete block backing. Although the drawings indicated an overall thickness of 16 in., they also showed that the wall had a large cavity housing the radiator and heating ducts. For the building plan data analysis, therefore, the effective thickness of the wall was taken as only 8 in.* For this case, the discrepancy could probably not be resolved during the on-site survey, unless there was prior knowledge relative to the construction of a specific type of wall.

From the analysis of the Detroit buildings, it is apparent that if the proper building information is obtained in an on-site field survey, then there is generally good correlation between the collapse predictions for both the field survey and building plan data. Furthermore, based on a recent field survey and analysis exercise, it appears that the necessary building data can be obtained by the field survey team.

* For side B, the overall wall thickness was 12 in., since there was an 8-in. thick concrete block backing.

Appendix

FIELD SURVEY DATA

By

M. D. Wright
Research Triangle Institute

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address MONTEGomery WARD & CO.
14455 GRATIOT, DETROIT

2. Standard Location 4333 0382 3. Facility Number 00028

4. Number of Stories 3 5. Height of Building 48

6. Story Height: Bas. 12 1st 24 Upper 12
Upper (If Change) — Story of Change —

7. Dimensions: Side A 151 Side B 175 (avg)

8. Plan Area: a. Basement 11,325 b. First Story 11,325
c. Upper Stories 11,325 d. Upper Stories if Change —

9. Fallout Shelter Stories and Areas:

Story	No. of Rooms with Shelter	Shelter Area
0	—	—
—	—	—
—	—	—
—	—	—
—	—	—
—	—	—

10. a. Plans Available yes b. Specs. Available yes
c. Location Detroit Bureau of Bldgs. d. Contact Mr. Gerlock

11. Building Use 53 12. Year Constructed 1939

13. Building Code Reference

14. General Condition Good

15. Hazards: Gas fired boiler in basement. Air Conditioning equipment on roof.

1. Building code suggests minimum live load of 100 psf on grade floor and 75 psf on upper floors.

2. Snow load - 30 psf
Wind load - 20 psf

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B. Structural Details

		Side A	Side B	Side C	Side D	Source
		16	16"	16"	16"	[illegible]
1.	Type of Substructure	2	2	2	2	est. obs.
2.	Basement Exposure	0	0	0	0	obs.
3.	Type of Exterior Walls:					
Wall thickness	Basement	—	—	—	—	—
measured 13 1/2"	First	8"-4" 59	8"-4" 59	8"-4" 59	8"-4" 59	Sanborn maps & Meas.
	Upper	"	"	"	"	"
	Upper (If Change)	—	—	—	—	—
	Story of Change	—	—	—	—	—

Height and Width of Panels: (1st) 21X24 & (upper) 21X12 — Source: Meas.

Support Conditions: ______ —

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner — outer — —

Horizontal: inner — Outer — —

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel — —

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel — —

Compressive Strength of Concrete — —

b. Masonry Walls:

Compressive Strength of Mortar 3000 psi — Source: Bldg. Code est

4.	Percent Apertures:	Basement	0	0	0	0	obs.
	1' Sill heights	First	35	0	15	18	Meas.
		Upper	25	0	20	20	"
		Upper (If Change)	10			10%	"
		Story of Change	3			3	"
	Height and Width of Apertures:		5X10 (2nd) 18X10 (1st)	—	3X5 (upper) 10X18 (1st)	5X10 (2nd) 10X18 (1st)	"
5.	Type of Foundation		110 & 120				est.

Source

6. Type of Frame 220 — San. map & data

Dimensions of Columns 10" WF — meas. scale

Dimensions of Beams 14" I — est.

a. Reinforced Concrete Frame

Bar Size and Spacing — —

Type Reinforcement — —

Concrete Compressive Strength — —

b. Steel Frames

Type of Steel 60,000 psi (ultimate strength) — Est

Fireproofing for Steel Frames 3 + 6 2" tile — obs.

7. Roof: Slope 12 Frame 23 + 25 Deck 4" 23 Covering 42 — obs.

Height of Parapet Walls: Side: A. 38" B. 38" C. 38" D. 38" — obs.

8. Floors: First Frame 12 + 13 Deck 6" 23 — est. & meas.

Upper Frame 12 + 13 Deck 6" 23 — "

Upper (If Change) Frame — Deck — —

Story of Change — —

Framing into Bearing Walls: — —

Spans: Parallel to Side "A" 21' Parallel to Side "B" 22' — meas.

a. Reinforced Concrete Floors

Bar Size and Spacing — —

Type Reinforcement — —

Concrete Compressive Strength — —

b. Structural Steel Floors

Beam Size 14" I + open web joist — maint. Eng. est.

Type of Steel 60,000 psi (ultimate strength) — Est

9. Type of Interior Partitions: Basement 60% - 8" 31 + 40% 8" 26 — obs. & est.

First 27 - 4" — obs. est.

Upper 27 - 4" — obs. est.

Upper (If Change) — —

Story of Change — —

C. Geological Data — Source

1. Depth of Water Table ________ 2. Rock Below Grade ________ ________

3. Soil Type ________________________ ________

4. Design Bearing Capacity of Soil ________________ ________

D. Fire Vulnerability

	Side A	Side B	Side C	Side D	Source
1. Adjacent Buildings - Stories	—	1	1	—	obs
Distance	—	0	0	—	obs.

2. Velocity and Direction of Prevailing Winds ________________ ________

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods.

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address ST. STEPHEN AME CHURCH 6000 STANFORD, DETROIT
2. Standard Location 4333 0109   3. Facility Number 06086
4. Number of Stories 2 + 3   5. Height of Building 30
6. Story Height: Bas. 10   1st 10+20   Upper 10
   Upper (If Change) —   Story of Change —
7. Dimensions: Side A 198'   Side B 91'
8. Plan Area: a. Basement 18018   b. First Story 18018
   c. Upper Stories 10250   d. Upper Stories if Change —
9. Fallout Shelter Stories and Areas:

Story	No. of Rooms with Shelter	Shelter Area
0	—	—
—	—	—
—	—	—
—	—	—
—	—	—
—	—	—

10. a. Plans Available yes   b. Specs. Available yes
    c. Location Detroit Bureau of Buildings   d. Contact M. Gerlach
11. Building Use 31 + 72   12. Year Constructed 1922 + 1949
13. Building Code Reference
14. General Condition Good
15. Hazards: natural gas furnace in basement

1- Building code suggests minimum live load of 80 psf.

2 - Snow load - 30 psf
wind load - 20 psf

B. Structural Details

	Side A	Side B	Side C	Side D	Source
1. Type of Substructure	14" 12" 2 + 4	12" 4	14" 12" 2 + 4	14" 2	dwg. + est.
2. Basement Exposure	5'	5'	5'	5'	meas.
3. Type of Exterior Walls:					
Basement	8"-4" 12"-4" 34	8" 44	8" 12"-4" 44 + 34	12"-4" 34	meas. + est.
First	8"-4" 34	8" 44	8" 8"-4" 44 + 34	8"-4" 34	meas. + est.
Upper	8"-4" 34	8" 44	8" 8"-4" 44 + 34	8"-4" 34	"
Upper (If Change)	—	—	—	—	—
Story of Change	—	—	—	—	—

Height and Width of Panels: 10'x20' + 20'x20' — meas.

Support Conditions: Continuous over supports. — dwg.

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner — outer — —

Horizontal: inner — Outer — —

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel — —

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel — —

Compressive Strength of Concrete — —

b. Masonry Walls:

Compressive Strength of Mortar 3000 Psi. — bldg. code.

4. Percent Apertures:

	Side A	Side B	Side C	Side D	Source
Basement	30	2	5	30	meas.
First	15	35	25	25	"
Upper	40	25	27	—	"
Upper (If Change)	—	—	—	—	—
Story of Change	—	—	—	—	—
Height and Width of Apertures:	18'x8'	14'x5'	14'x5'	6'x12'	meas.

sill heights variable, mostly 2' + 4' except in gym.

5. Type of Foundation 120 — est.

rafters in church roof are 10" x 2" steel channel

Source

6. Type of Frame 220 — obs. est.
   Dimensions of Columns 7" x 7" & 11" x 13" (interior columns in plaster in church) — meas.
   Dimensions of Beams 10" I — est.
   a. Reinforced Concrete Frame
      Bar Size and Spacing — —
      Type Reinforcement — —
      Concrete Compressive Strength — —
   b. Steel Frames
      Type of Steel 60,000 psi (ultimate strength) — assoc. calc. est.
      Fireproofing for Steel Frames 6 — obs.
7. Roof: Slope 11+12 Frame 23 Deck 3 1/4 33 Covering 44 42 — obs. & meas.
   Height of Parapet Walls: Side: A. — B. — C. — D. — —
8. Floors: First Frame 12 Deck 4" 23 — obs. & est.
   Upper Frame 12 Deck 4" 23 — "
   Upper (If Change) Frame — Deck — —
   Story of Change — —
   Framing into Bearing Walls: — —
   Spans: Parallel to Side "A" 15' Parallel to Side "B" 15' — meas.
   a. Reinforced Concrete Floors
      Bar Size and Spacing .25% — est.
      Type Reinforcement
      Concrete Compressive Strength 3000 psi — assoc. calc.
   b. Structural Steel Floors
      Beam Size 10" I with 6" flange — est.
      Type of Steel 60,000 psi (ultimate strength)
9. Type of Interior Partitions: Basement 12" 11 & 8" 24 — obs.
   First 8" 24 — "
   Upper 8" 24 — "
   Upper (If Change) — —
   Story of Change — —

C.	<u>Geological Data</u>					**Source**
	1. Depth of Water Table ________ 2. Rock Below Grade ________					________
	3. Soil Type ________					________
	4. Design Bearing Capacity of Soil ________					________
D.	<u>Fire Vulnerability</u>					
		Side A	Side B	Side C	Side D	
	1. Adjacent Buildings - Stories	—	2	2	–	obs.
	Distance	–	15	22	–	Meas.
	2. Velocity and Direction of Prevailing Winds ________					________

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods.

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address DETROIT HOUSING COMMISSION 1121 W. CANFIELD

2. Standard Location 4333 0046 3. Facility Number 02292

4. Number of Stories 14 5. Height of Building 115

6. Story Height: Bas. 10' 1st 8'5" Upper 8'5"

Upper (If Change) — Story of Change —

7. Dimensions: Side A See Plan Side B See Plan

8. Plan Area: a. Basement 6000 b. First Story 6000

c. Upper Stories 6000 d. Upper Stories if Change —

9. Fallout Shelter Stories and Areas:

Story	No. of Rooms with Shelter	Shelter Area
0		
1 thru 12		

10. a. Plans Available yes b. Specs. Available yes

c. Location Detroit Bureau of Buildings d. Contact Mr. Harlach

11. Building Use 11 12. Year Constructed 1953-54

13. Building Code Reference

14. General Condition Good

15. Hazards:

1- Building code suggests minimum live load of 40 psf.

2- Snow load – 30 psf
Wind load – 20 psf

B. Structural Details

		Side A	Side B	Side C	Side D	Source
1.	Type of Substructure	14" 2	14" 2	14" 2	14" 2	meas. [illegible]
2.	Basement Exposure	5'	5'	5'	5'	meas.
3.	Type of Exterior Walls:					
Wall thickness Measured at 14"	Basement	8"-4" 51	8"-4" 51	8"-4" 51	8"-4" 51	meas.
	First	8"-4" 59	8"-4" 59	8"-4" 59	8"-4" 59	meas + See maps.
	Upper	8"-4" 59	8"-4" 59	8"-4" 59	8"-4" 59	meas. + [illegible]
	Upper (If Change)	–	–	–	–	–
	Story of Change	–	–	–	–	–

Height and Width of Panels: 20' x 8'5" See Floor Plan Attached — meas

Support Conditions: Tile is set into frame, Brick facing is Continuous. — obs. + est.

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner — outer — —

Horizontal: inner — Outer — —

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel — —

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel — —

Compressive Strength of Concrete — —

b. Masonry Walls:

Compressive Strength of Mortar 3000 PSI — est.

4.	Percent Apertures:	Basement	25	25	25	25	meas.
3' sill ht except		First	20	20	20	20	meas.
Small windows have		Upper	20	20	20	20	meas.
58" Sill height		Upper (If Change)	–	–	–	–	–
		Story of Change	–			–	
	Height and Width of Apertures:		4½x4	6½x4 (4½x2)	4½x4	4½x2 5½x4	meas.
5.	Type of Foundation		120				est.

		Source
6. Type of Frame	111	obs.
Dimensions of Columns	12"x24", 8"x22", 18"x36" (see sketch)	meas
Dimensions of Beams	12" x 10" (Beams have widened sect. as they pass over larg. columns)	Meas in Bas.
a. Reinforced Concrete Frame		
Bar Size and Spacing	2.0% for Columns 1.0% for Beams	Est *
Type Reinforcement	60,000 PSI (ultimate Strength)	Est
Concrete Compressive Strength	3000 psi.	est.
b. Steel Frames		
Type of Steel	—	—
Fireproofing for Steel Frames	—	—
7. Roof: Slope 12 Frame 26 Deck 33 4" Covering 42		maint. eng.
Height of Parapet Walls: Side: A. 0 B. 0 C. 0 D. 0		obs.
8. Floors: First	Frame 14 Deck 23'-6"	maint. eng.
Upper	Frame 14 Deck 23'-6"	"
Upper (If Change)	Frame — Deck —	—
Story of Change	—	—
Framing into Bearing Walls:	—	
Spans: Parallel to Side "A" 20' Parallel to Side "B" 15'		meas.
a. Reinforced Concrete Floors		
Bar Size and Spacing	.25% reinforcement	est.
Type Reinforcement	60,000 psi Bars (ultimate strength)	est.
Concrete Compressive Strength	3000 psi	est.
b. Structural Steel Floors		
Beam Size	—	—
Type of Steel	—	—
9. Type of Interior Partitions: Basement	20% 4" 30 (tile) 4- 80% 14-4"	obs.
First	20% 30 (tile)-4", 50% 24-4", 30% 27-2½"	obs.
Upper	"	"
Upper (If Change)	—	-
Story of Change	-	—

* Might be higher in lower stories of this 14 story high rise Bldg.

C. Geological Data — Source

1. Depth of Water Table ________ 2. Rock Below Grade ________ ________

3. Soil Type ________________________________ ________

4. Design Bearing Capacity of Soil ________________ ________

D. Fire Vulnerability

	Side A	Side B	Side C	Side D	
1. Adjacent Buildings - Stories	___	___	___	___	___
Distance					

2. Velocity and Direction of Prevailing Winds ________________ ________

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods.

ACI manual

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address GENERAL ELECTRIC
700 ANTOINETTE, DETROIT

2. Standard Location 4333 0041 3. Facility Number 04079

4. Number of Stories 5 5. Height of Building 78'

6. Story Height: Bas. - 1st 22' Upper 15'
Upper (If Change) 12 Story of Change 3d Story

7. Dimensions: Side A 75' Side B 201'

8. Plan Area: a. Basement — b. First Story 15,075 sq.ft.
c. Upper Stories 15,075 d. Upper Stories if Change —

9. Fallout Shelter Stories and Areas:

Story	No. of Rooms with Shelter	Shelter Area
3	—	—
4	—	—
—	—	—
—	—	—
—	—	—
—	—	—

10. a. Plans Available yes b. Specs. Available yes
c. Location Detroit Bureau of Buildings d. Contact Mr. Ukalack

11. Building Use 54 12. Year Constructed 1924

13. Building Code Reference

14. General Condition Good

15. Hazards: Gantry crane on 1st story.

1- Building Code suggests minimum live load of 125 psf for light storage and 250 psf for heavy storage.

2- Snow load — 30 psf
Wind load — 20 psf

B. Structural Details

	Side A	Side B	Side C	Side D	Source
1. Type of Substructure	12" 2	12" 2	12" 2	12" 2	est.
2. Basement Exposure	—	—	—	—	—
3. Type of Exterior Walls:					
Basement	—	—	—	—	—
First	12" 41	12" 41	12" 41	12" 41	meas.
Upper	12" 41	12" 41	12" 41	12" 41	"
Upper (If Change)	—	—	—	—	—
Story of Change	—	—	—	—	—

Height and Width of Panels: (1st) 25'x20' + (2nd) 25'X15' + (upper) 25'x12' — meas.

Support Conditions: Curtain wall. — observ. est.

a. Reinforced Concrete Walls:

Bar Size and Spacing: Total -20% (50% inner / 50% outer) — Est.

Vertical: inner ______ outer ______

Horizontal: inner ______ Outer ______

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel: 1" — Est

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel: 11" — Est

Compressive Strength of Concrete: 3000 psi — est.

b. Masonry Walls:

Compressive Strength of Mortar: — — —

4. Percent Apertures:		Side A	Side B	Side C	Side D	Source
	Basement	—	—	—	—	—
40" sill heights.	First	40	55	25	—	
	Upper	40	45	40	45	meas.
	Upper (If Change)	35	40	35	40	obs.
	Story of Change	3	3	3	3	obs.
	Height and Width of Apertures:	14X10 14X15 14X8	21X10 21X15 21X8	20X12	21'X10'	meas

5. Type of Foundation: 110+120 — est.

Source

6. Type of Frame 121 * — obs.

Dimensions of Columns Wall 49"x24", Interior 28" dia. — meas.

Dimensions of Beams — —

a. Reinforced Concrete Frame (1% in Beams

Bar Size and Spacing 3% in columns) — est.

Type Reinforcement 60,000 psi Bars (without stirrups) — est. } Sadye

Concrete Compressive Strength 3000 psi. — est.

b. Steel Frames

Type of Steel — —

Fireproofing for Steel Frames — —

7. Roof: Slope 12 Frame 29 Deck 33 6" Covering 42 obs. & meas.

Height of Parapet Walls: Side: A. — B. — C. — D. — —

8. Floors: First Frame 17 Deck 23 9" meas.

Upper Frame 17 Deck 23 9" meas.

Upper (If Change) Frame — Deck — —

Story of Change — — —

Framing into Bearing Walls: — —

Spans: Parallel to Side "A" 25' Parallel to Side "B" 25' meas.

a. Reinforced Concrete Floors

Bar Size and Spacing .25% — est.

Type Reinforcement 60,000 psi. bars (without stirrups) — est. } Sadye

Concrete Compressive Strength 3000 psi. — est.

b. Structural Steel Floors

Beam Size — —

Type of Steel — —

9. Type of Interior Partitions: Basement — —

First none —

Upper none —

Upper (If Change) 28 44" 42" obs.

Story of Change 4 —

Stories 4 and 5 are Partitioned into office space. Light metal movable partitions are used on story 4 and Permanent stud wall w/plaster on story 5.

* Drop panels are 5" thick and 114" square.

C. Geological Data — Source

1. Depth of Water Table ________ 2. Rock Below Grade ________ ________

3. Soil Type ____ ________

4. Design Bearing Capacity of Soil ________________________ ________

D. Fire Vulnerability

	Side A	Side B	Side C	Side D	
1. Adjacent Buildings - Stories	—	—	2	2	obs.
Distance					

2. Velocity and Direction of Prevailing Winds ________________ ________

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods.

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address CLARA BARTON ELEM. School 8535 OTTO DETROIT, MICH

2. Standard Location 4333 0158 3. Facility Number 04511

4. Number of Stories 1 5. Height of Building

6. Story Height: Bas. 10½' 1st 12'2" gym 19½' Upper 7' (two small rooms on 2nd floor)

Upper (If Change) — Story of Change —

7. Dimensions: Side A 202' Side B 60'

8. Plan Area: a. Basement 2268 b. First Story 17163

c. Upper Stories 1197 d. Upper Stories if Change —

9. Fallout Shelter Stories and Areas:

Story	No. of Rooms with Shelter	Shelter Area
0	1	2268
1	—	—

10. a. Plans Available yes b. Specs. Available yes

c. Location Bureau of Bldgs. d. Contact Mr. Gerlach

11. Building Use 21 12. Year Constructed 1946

13. Building Code Reference

14. General Condition GOOD

15. Hazards: Coal fired boiler off basement

1- Building Code suggests minimum live load of 80 psf.

2- Snow load - 30 psf

3- Wind load - 20 psf

B. Structural Details

		Side A	Side B	Side C	Side D	Source
1.	Type of Substructure	12" 2	12" 2	12" 2	12" 2	San. maps. meas.
2.	Basement Exposure	0	0	0	0	Visual
3.	Type of Exterior Walls:					
	Basement	12" 31	12" 31	12" 31	12" 31	meas.
	First	12'-4" 34	12'-4" 34	12'-4" 34	12'-4" 34	Visual meas.
	Upper	—	—	—	—	—
	Upper (If Change)	—	—	—	—	—
	Story of Change	—	—	—	—	—

Height and Width of Panels: 12'8" X 17'6" And 12'8" X 10'6" meas.

Support Conditions: Conc. Bl. are set into Frame, Brick veneer is continuous elev.

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner — outer — —

Horizontal: inner — Outer — —

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel — —

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel — —

Compressive Strength of Concrete — —

b. Masonry Walls:

Compressive Strength of Mortar 3000 psi est.

			Side A	Side B	Side C	Side D	Source
4.	Percent Apertures:	Basement	0	0	0	0	elev.
	4 3" sill ht:	First	30	15	30	30	meas.
		Upper	—	—	—	—	—
		Upper (If Change)	—	—	—	—	—
		Story of Change	—	—	—	—	—
	Height and Width of Apertures:		4'X8', 12'X8'	12'X8'	12'X8'	4'X8', 5'X10'	meas.

5. Type of Foundation 110 + 120 est.

Source

6. Type of Frame ~~~///~~~ ... obs. (SAR maps)

6. Type of Frame: /// — Source: SAR maps, obs.
Dimensions of Columns: 16" X 14" — meas.
Dimensions of Beams: 14" X 14" — meas.

a. Reinforced Concrete Frame
Bar Size and Spacing: 1.0% — Est.
Type Reinforcement: 60,000 psi (ultimate Strength) — est.
Concrete Compressive Strength: 3000 PSI — Est.

b. Steel Frames
Type of Steel: — —
Fireproofing for Steel Frames: — —

7. Roof: Slope 12 Frame 26 Deck 33-6" Covering 41 — NFSS & est.
Height of Parapet Walls: Side: A. 0 B. 0 C. 0 D. 0 — obs.

8. Floors: First Frame 14 Deck 23'-6" — NFSS & obs.
Upper Frame — Deck — —
Upper (If Change) Frame — Deck — —
Story of Change —
Framing into Bearing Walls: — —
Spans: Parallel to Side "A" 21' Parallel to Side "B" 12'8" — meas.

a. Reinforced Concrete Floors
Bar Size and Spacing: 0.25% — Est.
Type Reinforcement: 60,000 psi Bars (ultimate strength) — est.
Concrete Compressive Strength: 3000 psi — est.

b. Structural Steel Floors
Beam Size: — —
Type of Steel: — —

9. Type of Interior Partitions: Basement: none — obs.
First: 30+21.* (8") — obs.
Upper: — —
Upper (If Change): — —
Story of Change: — —

Partitions are tile from the floor up to a height of 5' and are cinder block from that point to the ceiling

C. Geological Data — Source

1. Depth of Water Table ________ 2. Rock Below Grade ________ ________

3. Soil Type ________________________________ ________

4. Design Bearing Capacity of Soil ________________ ________

D. Fire Vulnerability

	Side A	Side B	Side C	Side D	
1. Adjacent Buildings - Stories	—	—	—	—	—
Distance					

2. Velocity and Direction of Prevailing Winds ________________ ________

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods.

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NOMENCLATURE

A_s	Area of reinforcing steel in tension zone, sq in.
A_s'	Area of reinforcing steel in compression zone, sq in.
c_o	Ambient sound velocity ahead of shock, fps
d	Distance from the compression face to the centroid of the tension steel, in.
d'	Distance from the compression face to the centroid of the compression steel, in.
E_c	Modulus of elasticity of concrete, psi
E_m	Modulus of elasticity of masonry, psi
E_s	Modulus of elasticity of steel, psi
f_c'	Compressive strength in concrete, psi
f_{dc}'	Dynamic compressive strength in concrete, psi
f_{dy}	Dynamic yield strength of reinforcing steel, psi
f_m'	Ultimate compressive strength of masonry unit wall, psi
f_r	Modulus of rupture of concrete, psi
L_H	Horizontal length (width) of wall, in.
L_{HW}	Horizontal length (width) of window, in.
L_V	Vertical length (height) of wall, in.
L_{VW}	Vertical length (height) of window, in.
p	Steel ratio, tension steel
p'	Steel ratio, compression steel
P_o	Ambient atmospheric pressure, psi
P_v	Total vertical force per unit width, lb/in.
S	Clearing distance, ft
t_f	Thickness of flange of hollow masonry block unit, in.
t_w	Thickness of wall, in.
W	Weapon yield, kt
γ	Unit weight, pcf