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Analysis of Non-Reinforced Masonry Building Response to Abnormal Loading and Resistance to Progressive Collapse

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W. McGuire and E. V. Leyendecker

Center for Building Technology
Institute for Applied Technology
National Bureau of Standards
Washington, D. C. 20234

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Final Report

Prepared for
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U. S. DEPARTMENT OF COMMERCE, Frederick B. Dent, Secretary
NATIONAL BUREAU OF STANDARDS, Richard W. Roberts, Director

SI Conversion Units

In view of present accepted practice in this technological area, U.S. customary units of measurement have been used throughout this report. It should be noted that the U.S. is a signatory to the General Conference on Weights and Measures which gave official status to the metric SI system of units in 1960. Readers interested in making use of the coherent system of SI units will find conversion factors in ASTM Standard Metric Practice Guide, ASTM Designation E 380-72 (available from American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania 19103). Conversion factors for units used in this paper are:

Length

1 in = 0.0254* meter

1 ft = 0.3048* meter

Area

1 in^2 = 6.4516* x 10^{-4} meter^2

1 ft^2 = 9.2903 x 10^{-2} meter^2

Force

1 lb (lbf) = 4.448 newton

1 kip = 4448 newton

Pressure, Stress

1 psi = 6895 pascal (N/m^2)

1 psf = 47.88 pascal (N/m^2)

Moment

1 lbf-ft = 1.3558 newton-meter

*Exact value

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ANALYSIS OF NON-REINFORCED MASONRY BUILDING RESPONSE TO ABNORMAL LOADING AND RESISTANCE TO PROGRESSIVE COLLAPSE

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ABSTRACT

Five case studies of susceptibility to progressive collapse were made of non-reinforced masonry bearing wall buildings. All were assumed to comply with governing building codes. Based on the assumed failure mechanisms, analysis indicated that two of the structures had excellent resistance to progressive collapse, one was marginal, and two had little resistance to progressive collapse. Analytical approaches used are illustrated and areas of needed research are identified.

Key Words: Abnormal loading; Building; Gas explosion; Load-bearing masonry; Load-bearing walls; Masonry; Masonry research; Progressive collapse.

1. Introduction

Some multistory buildings may collapse progressively following local failure caused by abnormal loading. A history of this phenomenon and examples of its occurrence have been described by Somes [9].1/ The National Bureau of Standards, sponsored by the Department of Housing and Urban Development, is conducting research to provide information needed in the development of rational criteria for the prevention of progressive collapse. As part of this research, analytical studies are being made of the response of actual buildings to abnormal loads.

This paper is a report on the findings of case studies of non-reinforced masonry structures. It also contains recommendations for further research.

The case studies have a three-fold purpose:

1. The assessment of susceptibility to progressive collapse of some common types of contemporary building construction.

2. The illustration of some analytical approaches that can be used to determine susceptibility to progressive collapse.

3. The identification of specific subjects of research, the results of which are needed before comprehensive and rigorous design criteria can be developed for specific materials.

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1/ Figures in brackets designate references listed at the end of the report.

It is patently impossible to investigate all the possibilities for abnormal loading, all the ways in which structures are being built, and all the ways in which they resist, or fail to resist, abnormal loading. But, even within the limited scope of the case studies, it is believed the above three objectives can be achieved.

2. Case Study Approach

2.1 Building Information

In order that the case studies would be realistic, drawings and specifications for several masonry bearing-wall buildings were reviewed. It is assumed that all designs provided the required margins of safety with respect to failure under the required building code design loads. Five case studies typical of those reviewed were chosen. These ranged from six to ten stories tall and were of non-reinforced masonry construction. Portions judged potentially sensitive to progressive collapse were then studied analytically.

Based on the assumed failure mechanisms, it can be shown that the progressive collapse resistance of one of the cases studied is relatively low. The probability of progressive collapse in <u>a particular building</u> is very small even though the probability of encountering at least one progressive collapse in <u>a class of buildings</u> (such as non-reinforced masonry) susceptible to such failure may be high. The reasonable and proper conclusion to draw from a finding of susceptibility to progressive collapse in one or more of the five cases studied is that the problem could be prevalent in this type of construction and should be investigated at greater depth.

2.2 Problem Statement

The first question posed in each study is whether, under any feasible abnormal load, a primary structural element would fail. If the building proves invulnerable to such local collapse then of course the investigation stops right there; there would be no problem. If, however, local failure is indicated, the next question studied is whether the failure could spread progressively to a substantial portion of the building or whether there exist alternate paths which would enable the structure to bridge the failure with only local distortion and damage.

Studies of this sort obviously involve judgement and selectivity. One cannot study every member of a large building without spending an unreasonable amount of time. In each case the building plans were inspected and a primary structural element was selected for analysis. This was a member which seemed, on the basis of engineering judgement, to be vulnerable to local failure under abnormal loading, and to present possibilities for the initiation of a progressive failure. In all cases exterior elements were analyzed; however, in each building there appear to be interior elements which have about the same degree of resistance (or lack of resistance) as the one studied. It was not considered necessary to analyze all possibilities of progressive collapse since the objectives of this report could be achieved by the analysis of a few selected elements.

To analyze a structure, an abnormal loading function must be postulated. Experience has shown that there is a relatively high probability of a natural gas explosion in one apartment. Although further research on the pressure-time histories of gas explosions is desirable, sufficient information exists to develop plausible loading functions [3,6,8]. Since the analysis of resistance to such loading functions yields the type of information desired in these studies, they were employed as a basis for analysis. The details of their use will be illustrated in the case studies.

Definitions of effective alternate structural paths must also be left to the case studies. The alternate path method is a general approach. Its application involves an examination of the structure that remains after loss of a primary element to determine whether stable equilibrium--of at least a temporary nature (such as for evacuation of occupants)--is possible through arching, cantilever, catenary, or similar action. It may require consideration of the resistance of elements whose load bearing capabilities were disregarded in the original design.

The case studies are summarized in the following sections. The first two of the five buildings were studied extensively and the other three superficially. Recommendations for further studies and research conclude the report.

3. The "Adams" Building

3.1 Description

The "Adams" Building is an eight-story brick-masonry, bearing-wall apartment house. Figure 3.1 is an elevation view and half section of the building portion studied. The walls and piers are lettered for identification. A plan of a typical floor in the portion studied, is shown in figure 3.2 and the layout of a single-bedroom apartment is shown on figure 3.3. In the part investigated, the brick cross wall spacing is 92 ft.

Floor construction is of 8-in deep, one-way, prestressed, precast concrete planks. They bear on the corridor walls and on the exterior walls, piers, and lintels (figure 3.1). There are no longitudinal ties (floor reinforcement parallel to the planes of the walls) between the ends of the planks. There are ties at the one-third points of the floor span which will be discussed later (figure 3.14).

The brick walls and piers are reinforced only at the floor-to-wall connections as shown in figure 3.4 (the plank reinforcement is not shown on the drawing). The double-angle lintels have a 6-in minimum width of bearing on the exterior walls and piers.

3.2 Structural Response

3.2.1 Critical Element

The structural element selected for analysis of vulnerability to local failure is a one-story section of Pier B (see figure 3.1). Pier B is 3-ft wide and 8-in thick of solid brick, single-wythe construction. Loss of one story of Pier B would result in loss of support for a 15-ft length of floor (3 ft of pier, one 4-ft spandrel, and one 8-ft spandrel). There is no apparent potential for post-failure arching over such a damaged section. Pier B is also located wholly within one apartment (see figure 3.3) and thus subject to the full initial effect of an explosion within a single living unit.

3.2.2 Loading Function

The most useful information on natural-gas explosions currently available is the work of Rasbash [6], Stretch [8] and Mainstone [3,4]. Rasbash has presented the following empirical equation for estimating the peak pressure:

$$p_m = 1.5\ p_v + 0.4K \qquad \text{(Eq. 1)}$$

where

p_m = peak pressure, psi

p_v = pressure at which vents open, psi

K = venting ratio

= (minimum cross-section area of room)/(vent area)

In cases such as the "Adams" Building in which venting is supplied by the breaking of window glass, Rasbash and Stretch suggest using p_v = 1 psi in Equation 1. However, Mainstone has prepared empirical charts and suggestions for estimating a range of breaking pressures (p_v) for windows of different sizes and types [3].

A natural gas explosion is a relatively slow deflagration. Mainstone suggests that the pressure rise time is of the order of 0.1 second and the decay period lasts about 0.2 seconds [4].

These recommendations of Rasbash, Stretch, and Mainstone are applied in figure 3.5 with the resulting range of maximum pressures as shown. Recognizing the approximate nature of pressure and time calculations, and allowing for the scatter to be expected, it is reasonable to conclude that a 3-psi peak pressure is highly feasible and that resistance to anything less than 3.5 psi would indicate questionable structural integrity in the event of a natural gas explosion. Variations in rise time and duration are not included because, as shall be seen, structural response is insensitive to changes in these parameters for such a relatively slow pulse.

This estimation of the loading function assumes that a natural gas-air mixture covers the living/dining room and the bedroom prior to detonation and also that partitions separating rooms remain essentially in place until after the windows break and the pressure reaches its peak intensity.

3.2.3 Resistance Interaction Relationship

Pier B will be analyzed as an unreinforced, one-story element subjected to axial force and bending. The most reliable current information on the resistance of masonry members subjected to this loading combination is contained in the papers of Yokel, Mathey, and Dikkers [12,13,14,15]. They propose the following interaction equations for the combined axial force and bending resistance of sections which crack at or before the ultimate load [15]:

$$M_c = \frac{t}{6}\,(s\ P_o + P) \qquad \text{(Eq. 2)}$$

or

$$M_e = \frac{Pt}{2} \left(1 - 1.33 \frac{P}{P_o} \left[\frac{a - 2s}{(a-s)^2}\right]\right) \qquad \text{(Eq. 3)}$$

whichever is greater. In the equations,

- A = Area of net section
- a = Flexural compressive strength coefficient
- f'_m = Compressive strength of masonry
- f'_t = Tensile strength of masonry
- M_c = Cracking moment
- M_e = Maximum moment capacity, computed using linear stress gradients
- P = Applied vertical compressive load
- P_o = Short wall axial load capacity (f'_m A)
- s = Ratio of tensile strength to axial compressive strength of masonry (f'_t/f'_m)
- t = Thickness of wall

Equation 2 controls for cases in which failure occurs immediately upon first cracking of the section. Equation 3 controls for cases in which tests have demonstrated that there is post-cracking resistance. Equations 2 and 3 are plotted on figure 3.6 for values of the Pier B parameters. The parameters f'_m and f'_t are based on data in "Recommended Practice for Engineered Brick Masonry [7]". The value of f'_m was obtained from table 2 based on an assumed brick compressive strength of 10,000 psi and Type S mortar. The value of f'_t = 175 psi was obtained from table 5.7 for a wall thickness of 7.5 in using a single-wythe brick. The compressive strength coefficient "a" of 1.6 was based on the work of Yokel and Dikkers [13].

Separate calculations, which are not included herein, have shown that neither shearing resistance nor flexural instability control the resistance of Pier B.

3.2.4 Analysis

Under loads of short duration, a dynamic analysis may be required in order to obtain accurate results. In the present case, the natural period of the pier is approximately one-fourth of the rise time of the explosion (figure 3.7). Because of this difference, a static analysis of the pier's response would appear to give a satisfactory indication of true behavior. Further investigation based upon the triangular pulse shape, shown in figure 3.5, following procedures in Biggs [1] and Jacobsen and Ayre [2], indicate that use of the peak pressure in a static analysis is justified. The problem will therefore be treated as a static one.

To study differences in behavior following an explosion in the upper, middle, and lower regions of the building, the 8th-story, 5th-story, and 1st-story elements of Pier B are analyzed. Figure 3.8 describes the vertical loads used in each analysis. It is assumed that internal pressure in the outer quarter of the apartment where the explosion occurs, causes tension in Pier B. The vertical forces in the analyzed elements are tabulated on Figure 3.9 and the bending moments due to floor load eccentricity on figure 3.10.

Figure 3.11 indicates that if a blast having a peak intensity of 1.4 psi occurred in an eighth-story apartment, the section of Pier B contiguous to that apartment would collapse. A trial and error solution of the interaction equation was used, with only the successful trial being shown. Although the interaction equation applies to the resistance of a cross section of the element, failure of the midheight section will precipitate collapse of the entire member.

In lower stories, Pier B may enjoy some joint continuity because of dead and live load compression which is not counterbalanced by blast uplift. Assuming continuity, a frame analysis (not reproduced) indicated the end and midheight bending moments due to blast to be approximately equal. They have, therefore, been assumed equal in the analysis of the first story and fifth-story.

Figure 3.12 shows that a blast pressure intensity of about 3.4 psi in a fifth-story apartment would cause failure of the top, middle, and bottom sections of the contiguous Pier B, that is, collapse of the element. A pressure of 5.4 psi would be required for a similar occurrence in the first-story, as indicated on figure 3.13. The increase in resistance as one goes down the building is a result of the increasingly beneficial effect of the dead and live load compression. Note that the live load used herein was a reduced live load of 5 psf.

Comparing these results with the postulated loading function (figure 3.5), the conclusion is that a natural gas explosion in the upper three stories would almost certainly cause failure of a Pier B in that story. An explosion in the next two stories would probably cause such local failure of Pier B and an explosion in the lower three stories would probably not. Since there is a good chance of local failure, the question of whether the failure will progress or be contained must be answered.

If a one-story section of Pier B collapses, the two adjacent spandrels deriving support from that section will fall with it (see figures 3.1 and 3.4). The presence of windows on each side of the pier in each story precludes the possibility of effective arching in the plane of the exterior face. An explosion which removed a story-height section of Pier B will also render the partitions of at least the focal apartment incapable of resisting any direct vertical load.

The shear keys between adjacent planks cannot provide effective flexural resistance in the direction transverse to the plank span. There are, however, two lines of bridging buried in the floor as shown on figure 3.14. The intended function of this reinforcement was to improve the diaphragm action of the floors in transmitting wind loads to the cross walls. It is conceivable that in the event of loss of a section of Pier B below a particular floor, the bridging in that floor would serve as a catenary to span the resulting 15-ft gap. Making the generous assumption that the bars would not pull out of their concrete encasement, the slab is analyzed approximately on figure 3.15. This analysis shows that the bars would yield and permit the exterior ends of the floor planks to deflect approximately 15 in. This is more than sufficient to cause loss of support for the section of Pier B in the story above, resulting in its collapse. This chain of events can progress to the top of the building.

3.3 Conclusions for "Adams" Building

Exactly what would happen during and following the progressive loss of a full Pier B is impossible to analyze quantitatively. It seems clear that the ends of the planks which derive their support from Pier B would deflect a foot or more and that a number of them would pull away from the sagging bridging. The restraint at the inner, corridor wall, ends would not prevent this. An optimistic appraisal of the possibilities is that floor failure would progress upward several stories from the focal point until the accumulative effect of the interior partitions, acting in the fashion of a crumpled egg crate structure, would be sufficient to shore up the remaining floors. A more pessimistic appraisal is that the partitions in the story immediately above the initial failure story would be incapable of resisting collapse of their ceiling planks and that these partitions would be completely ineffective under the combination of load from above and loss of support from below (the failed ceiling of the apartment with the initial failure). The complete story-by-story failure of this tier of apartments would then progress to the roof. In addition, the falling debris could cause failure to the ground. Inward and lateral spread is a further possibility because of overloading (in both axial force and bending) of the corridor wall and the exterior piers and walls adjacent to the collapsed Pier B.

Regardless of whether one is optimistic or pessimistic in speculating on the consequences of blowing out a one-story section of a Pier B, one conclusion is clear: failure would be extensive and fall well within any definition of progressive collapse.

4. The "Baker" Building

4.1 Description

The "Baker" Building is an eight-story brick-masonry, bearing-wall apartment house. Figure 4.1 is an elevation view and half section of the building portion studied. A plan of a typical three-bedroom corner apartment is shown on figure 4.2. There are no interior brick cross walls normal to the wall shown in elevation "E".

Floor construction is of 8-in deep, one-way prestressed, precast hollow-core concrete planks which bear on the interior concrete masonry walls and on the exterior brick walls, piers, and lintels (figure 4.1).

The brick walls and piers are reinforced only at the floor-to-wall connections (figure 4.3). The plank reinforcement is not shown on the sketch. There are no longitudinal ties (floor reinforcement parrallel to the wall portion analyzed) between the planks. The triple angle lintels have 8-in bearing on the exterior walls and piers.

4.2 Structural Response

4.2.1 Critical Element

The structural element selected for analysis is the 14 ft-6 in long segment of the wall which extends from the side of the window in bedroom 1 to the face of the 12-in by 16-in pier in bedroom 2 (see figure 4.2). It is assumed that the 13 ft-0 in wall in bedroom 1, the spandrels above and below the window in bedroom 1, and the 12 in by 16 in pier, will all survive the initial blast even if the wall segment under consideration is blown out. The wall segment is located wholly within one apartment and thus subject to the full initial effect of an explosion within the living unit.

The above assumptions as to remaining elements are considered generous and there may be other, more vulnerable elements in this building. But one of the purposes of this study is to investigate the behavior of different types of structural members. The wall segment selected for analysis provides a good basis for study of the blast and progressive collapse resistance of planar elements of brick masonry such as post-failure in-plane arching over a damaged wall segment.

4.2.2 Loading Function

Peak pressures are calculated from Equation 1, the Rasbash fromula (figure 4.4). The pressures are somewhat higher than in the previous study because of the smaller amount of venting. In the present case, a 3 psi peak pressure is highly feasible and resistance to anything less than 4 psi would indicate questionable structural integrity.

4.2.3 Resistance Interaction Relationship

A one-foot wide vertical strip of the wall is analyzed for blast resistance. The Yokel, Mathey, and Dikkers formulas, Equations 2 and 3, for this strip are plotted on figure 4.5. The parameters f'_m and f'_t are based on data in "Recommended Practice for Engineered Brick Masonry [7]". The value of f'_m was obtained from table 2 based on an assumed brick compressive strength of 10,000 psi and Type S mortar. The value of f'_t = 140 psi was obtained from table 5.7 for a multi-wythe brick wall of 7 ft-8 in thickness. The compressive strength coefficient "a" of 1.6 was based on the work of Yokel and Dikkers [13].

4.2.4 Analysis

a). General

The possibility of wall failure in the eighth, fourth, or first story is investigated in figures 4.6 through 4.8. The analysis of the one-foot wide strip of wall proceeds just as in the previous study. The intensity of direct stress on the wall is less than that on Pier B of the "Adams" Building. Since there is less beneficial compression, the calculated failure pressures are somewhat smaller in this case than in the previous one. This, combined with the higher possible blast pressures, results in a more critical situation regarding the probability of failure of a one-story element in the lower portions of the building. In this structure it may be concluded that a natural gas explosion in the upper three stories would almost certainly cause failure of a wall segment and that an explosion in the lower five stories would probably cause failure.

The question of whether failure will progress is more difficult to answer in this case than in the "Adams" Building. There are no internal ties between floor planks which might develop catenary action and permit the floor to bridge over a damaged area. The possibility that the planks may hang from the exterior wall above and that this wall may successfully arch over the blasted apartment must be investigated. It will be assumed that the bent bars which connect the floor to the wall (figure 4.3) function as proper hangers, although they appear rather short.

An attempt was first made to assess the ultimate arching capability of the exterior wall. Various failure mechanisms in the wall panel above a removed segment were assumed and analyzed. However, the problem of in-plane failure of a brick wall is not as well documented as the ultimate resistance of steel and concrete frames, therefore firm conclusions could not be reached from simple mechanism analysis. Also, even though failure in certain modes appeared possible, displacement following such theoretical failure would bring segments of the wall into contact with other elements which would probably offer sufficient resistance to prevent complete collapse, at least for a time sufficient to permit evacuation of the building. So many assumptions were necessary to permit analysis of these post-failure conditions as to make the results meaningless hence, they are not reported herein.

The approach finally adopted was to analyze the wall above a failed segment as an elastic body using the SAP plane stress, finite element program [11]. If analysis shows that all of the principal tensile stresses in the wall are tolerably small and that the required reactions can be developed by the portions of the structure which remain after an explosion, it is a convincing demonstration that collapse will not be progressive. The converse does not necessarily hold. Large tensile stresses may simply signify failure of another small section of wall with arching over this region. Repeated analysis with succeeding sections of wall removed may enable one to conclude whether the process stops with a sufficient portion of structure left to ensure overall stability, or whether collapse will progress. In the present case successive iterations were not necessary since, as will be shown, the stresses and reactions obtained in the first set of analyses were small enough to justify the conclusion that the wall will arch above the failed story provided, of course, that the basic and rather generous assumptions regarding the integrity of the pier and remaining walls in the apartment with the initial failure are realized (section 4.2.1).

This analysis should be viewed more as a demonstration of an approach than as a refined tool. Further research is needed before designers can interpret with certainty the results of analyses such as these. For example, the magnitude of tensile stress which indicates a given probability of cracking has not been defined. The significance of local stress concentration needs codification. The limits of treating a combination of bricks and mortar as an homogenous, isotropic medium are not clear nor is it certain whether, if these limits are serious, they can be overcome in a practical way. Suggestions for some of this research will be presented later. The approach looks promising. If it can be refined, it should be of general use to designers of masonry structures.

b). Plane Stress Analysis and Results

Two stories of the wall above a failed segment were idealized as shown on figure 4.9. The wall was treated as a plane stress member with the thicknesses indicated. The 12-in by 16-in pier at the left side of the wall in figure 4.9 was considered to be an edge beam as was an 8-in by 16-in strip of the wall perpendicular to the right side of the wall shown in figure 4.9. To study the effect of variations in the resistance offered by the three floors, three sets of boundary conditions were assumed as indicated in figure 4.9. The full weight of the wall and edge beams was included in all analyses. For each set of boundary conditions, three floor loading conditions were used, making a total of nine analyses. At each loaded floor, the dead and reduced live load on the outer half of the floor span were applied to the wall. Changing the number of floors loaded gave some indication of whether loads on upper floors are transmitted to the side supports through arching rather than flexural action.

The reactions for all analyses are shown on figure 4.10. Figures 4.11, 4.12, and 4.13 are plots of principal stresses in the planar elements for the second set of boundary conditions (horizontal restraint at each floor and vertical support at the bottom only). It is believed that these are the most realistic support conditions. Figure 4.14 contains a principal stress plot for an additional analysis--one made using one story only and a finer mesh. Distributions of horizontal stress are shown on figure 4.15, for one loading and two boundary conditions.

Observation of the reactions (figure 4.10) indicates that in all cases they could probably be supplied by adjacent portions of the building without overstressing. There is a question of whether the 2.12 kip horizontal reaction at the base of the wall in the case of all floors loaded (figure 4.10-Boundary Set B) could be realized. Since there are no ties in the floor, this force would have to be developed through a combination of bending resistance of the end wall and shearing resistance of the small segment of wall remaining in the story below. They seem to be capable of providing such resistance. As seen from figure 4.10, some other calculated reactions are larger than this one. However, they occur either in regions removed from the damaged area, e.g., the 3.32 kip reaction at the floor above, or they require only the readily mobilized compressive resistance of the building, e.g., the 19.07 kip reaction at the lower, left corner.

The maximum calculated tensile stress in the two-story analyses which are reproduced is 82.6 psi. It occurs below a corner of the lower window (figure 4.13). Since the mesh used is rather coarse and there is a stress gradient in this area, the peak stress at the corner will actually be higher than this (compare with figure 4.14). There would probably be localized cracking in this region, but it is doubtful that it would be serious. Over most of the structure analyzed, both tensile and compressive stresses are tolerably small.

The principal stress directions in figures 4.11 through 4.13 illustrate the way forces arch around window openings and in the direction of supports. However, the study is inconclusive as to the full extent and character of the cracking that might occur in a multistory wall of a large number of tiers. For example, it can be seen that the increase in maximum principal stress along the bottom of the wall is approximately proportional to the magnitudes of the distributed floor loads added successively to the upper stories (compare figures 4.11, 4.12, and 4.13). Additional studies would be needed to determine the level above which most of a superimposed load is transmitted to the side supports rather than affecting the midspan horizontal stress at the bottom of the wall.

4.3 Conclusions for "Baker" Building

In spite of the above indication that the analysis is not complete, it may still be concluded that this wall would not collapse progressively if the assumptions as to the elements remaining in the apartment containing the explosion are valid (section 4.2.1). As stated earlier, however, there may be other more vulnerable elements that were not investigated because of a desire to focus on wall behavior.

Looking at the results as a semi-quantitative demonstration of an approach, there are several things to be said. The comparative stress distributions on figure 4.15 illustrate the beneficial effects of horizontal constraint at each floor. The refined mesh (see figure 4.14) gives a good idea of the magnitude of stress concentration around windows and points of support or constraint. Treatment of the wall as an elastic medium appears reasonable in view of the brittle nature of bricks and mortar. With the computer programs currently available, a finite element analysis is not difficult or expensive. Input is straightforward, running times of even moderately large all-purpose programs seem reasonable, and output presentation is clear. If it is possible to conclude successfully the research needed to quantify the effects of non-homogeneity, to establish permissible (or failure) stresses on a realistic (probably probabilistic) basis, and to guide engineers in ways of reducing real walls to realistic analytical models, the use of modern methods of plate and shell analysis can become practical design office tools.

5. The "Carter" Building

5.1 Description

The "Carter" Building is also an eight-story brick masonry, bearing-wall apartment house. Figure 5.1 is an elevation view and section of the building portion studied. A plan of a typical apartment in this region is shown on figure 5.2. In the part investigated, the brick cross wall spacing is 65 feet.

Floor construction is of 14-in deep steel bar joists with a 2 1/2-in cast-in-place concrete deck. The concrete is placed on corrugated slab form reinforced with 6 x 6-10/10 welded wire fabric. The joists bear on the exterior walls (figure 5.3).

The brick walls are not reinforced except that over each window there are three horizontal layers of prefabricated joint reinforcement spaced 8 in o.c. vertically and extending 2 ft beyond the window opening on each side. The double-angle lintels shown in figure 3.5 have 5-inch bearing.

5.2 Structural Response

5.2.1 Critical Element

Detailed calculations were not made for this building. There are sufficient similarities between it and the previous two buildings to permit reasonable qualitative analysis of its resistance. The behavior of the 5ft-7in wide section of wall between bedrooms 1 and 2 will be discussed (figures 5.1 and 5.2) in the following sections.

5.2.2 Loading Function

Application of the Rasbash formula yields peak gas explosion pressures of 2.7 and 4.2 psi in bedrooms 1 and 2 respectively. The difference is a result of the different venting in the two rooms. Just as in the "Baker" Building, an average peak pressure of 3 psi is highly feasible and resistance to anything less than 4 psi would indicate questionable structural integrity.

5.2.3 Resistance Interaction Relationship

The resistance of a one-foot wide vertical strip of the wall is essentially the same as that of the "Baker" Building (figure 4.5).

5.2.4 Analysis

The steel bar joist floors are somewhat lighter than the concrete plank floors in the "Baker" Building. On the other hand, the wall in the "Carter" Building is subjected to the vertical reactions from the spandrels on each side at each floor. In the "Baker" Building, the spandrel reactions affect only a small region at one end of the wall segment. For practical purposes, the net effects of the vertical load in the two buildings are about the same. Thus, the possibility of failure of a one-story section of the exterior wall considered is about the same as in the "Baker" Building. Therefore, it may be concluded that a natural gas explosion in the upper three stories of the "Carter" Building would almost certainly cause failure of a one-story section of the wall and that there would be a high probability of failure of such a section should an explosion occur in one of the lower stories.

In considering what might happen after a one-story section of wall is removed, it is useful to observe that the general wall layout is, in terms of structural behavior, intermediate between that of the "Adams" Building and the "Baker" Building (compare figures 5.1, 3.1, and 4.1). In the "Adams" Building resistance is in discrete piers without any arching potential. In the "Baker" Building, the in-plane arching capability of the solid wall is considerable. In the present case, the wall above an apartment containing an explosion is punctured by window openings, but there are still continuous expanses of masonry which may have some potential for bridging over a gap through in-plane redistribution of forces. A finite element analysis would be of use in evaluating this possibility. One was not made because, for the purpose of this report, that approach was adequately demonstrated in the study of the "Baker" Building. In lieu of an analysis, the results of the earlier studies will be used as a guide to a discussion of what would probably happen in the stories immediately above the apartment containing an explosion.

If a 5 ft-7 in wide section of bedroom wall is blown out in one apartment, the spandrels which it supports will go with it. There is no vertical reinforcement in the brick above the adjacent windows and flashing at the floor level interrupts the masonry bond. The 2-1/2 in concrete floor above will attempt to span the 14 ft-4 in gap created by the loss of the wall segment and the lintels supported on the wall. Assuming that the floor mesh acts as a catenary and making an analysis similiar to that on figures 3.14 and 3.15, it can be shown that the maximum deflection of the floor deck will be about 6 in. The wall above will lose support and, since it has no reinforcement, it will crack under its own weight. Whether it will fall out and permit sequential failure up to the roof is impossible to determine without a more thorough analysis. Adjacent spandrels would resist the large order displacements and rotations required to permit a clean drop of any substantial height of the wall strip which has been considered to be the critical element. It is probable that failure will be arrested after several stories of sagging and realignment of walls, spandrels, floors, and interior partitions.

5.3 Conclusions for "Carter" Building

It is quite clear that it is possible to lose a one-story section of the critical wall through a gas explosion. The analysis has not been carried far enough to support definite conclusions whether the collapse would be progressive. Based largely on an interpretation of the results of the two earlier studies, it is believed that there would be significant damage and at least partial collapse extending several stories above the source of the explosion.

6. The "Drake" Building

6.1 Description

The "Drake" Building is a six-story, brick and concrete block, bearing-wall apartment house. Figure 6.1 is a partial elevation and section of the building portions studied. A plan of a typical apartment in this region is shown on figure 6.2. The masonry cross-wall spacing is approximately 150 feet.

Floor construction is of 16-in deep steel bar joists with a 2 1/2-in cast-in-place concrete deck. The concrete is placed on corrugated slab form and reinforced with 6 x 6- 8/8 welded wire fabric. The joists bear on reinforced masonry tie beams at the exterior walls. These 12-in thick walls are constructed with 4-in thick brick and 8-in thick concrete block (figure 6.3). The brick and block are bonded with prefabricated joint reinforcement in each block bed joint (8-in vertical spacing).

6.2 Structural Response

6.2.1 Critical Element

Detailed calculations were not made for this building. The behavior of the 4 ft-8 in wide section of wall between bedrooms 1 and 2 (figures 6.1 and 6.2) will be discussed. The similarities and the differences between this and the previous building will be featured.

6.2.2 Loading Function

The peak gas explosion pressures (Equation 1) are 2.8 psi in bedrooms 1 and 2. Resistance to anything less then 3 psi would indicate questionable structural integrity.

6.2.3 Resistance Interaction Relationship

If the assumption is made that the exterior wall acts as a solid, monolithic wall of 12-inch thickness, the resisting function (Equation 2 and 3) are as shown on figure 6.4. If, at the other extreme, composite action is completely disregarded and the wall is assumed to act as an 8 in thick, unreinforced wall, the resisting function is essentially the same as that of the "Baker" Building (figure 4.5). True behavior is somewhere between these extremes. No attempt was made to develoe a more accurate measure of resistance in this exploratory study. The curves on figures 4.5 and 6.4 will be considered as lower and upper bounds in discussing performance.2/

6.2.4 Analysis

Loading conditions on the actual element are about the same as those on the element studied in the "Carter" Building. The internal pressure needed to blow out a one-story strip on the top floor will be between 1.6 psi (figure 4.8) and 3.3 psi (figure 6.4). The indication is that if the explosion occurred in the upper two stories of the building there would be a good possibility that the wall would blow out, but if it occurred in the lower half, it is doubtful that failure would occur.

2/ Reference 15 contains information on the behavior of cavity and composite walls. None of the data presented corresponds directly to the walls of the "Carter" Building, but the findings generally confirm the lower and upper bound approach used here.

If failure of a one story section did occur, the presence of the tie beam at each floor (figure 6.3) would cause subsequent behavior to be quite different from that of the otherwise similar "Carter" Building. The tie beam is capable of spanning the 12 ft-8 in gap caused by the loss of a wall and two adjacent spandrels, without excessive deflection. It will provide some vertical support for the wall above and, probably more important, it should serve as a tie in enabling the wall to develop its in-plane arching capability.

6.3 Conclusions for "Drake" Building

Although this study has been cursory, there is good reason to conclude --based on the previous, more detailed analyses--that in this case, even if a one-story section of wall were blown out, collapse would not be progressive.

7. The "Edwards" Building

7.1 Description

The Edwards Building is a ten-story, brick and concrete block, bearing-wall apartment house. Figure 7.1 is a partial elevation and section of the building portion studied. A plan of a typical corner apartment is shown on figure 7.2.

Floor construction is of 8-in deep, one-way, prestressed precast concrete planks. They bear on the cross walls (figure 7.3). The interior concrete masonry cross walls are reinforced by vertical bars placed in grout filled cells. The amount of reinforcement varies with height, being greatest in the bottom story. Brick and block in the exterior wall are bonded with prefabricated joint reinforcement in every second bed joint, i.e., 16 in on center.

7.2 Structural Response

7.2.1 Critical Element

The behavior of the north wall in the region of the kitchen will be discussed, without the aid of calculations.

7.2.2 Loading Function

If a natural gas explosion occurs in the living/dining room area, the peak gas explosion pressure (Equation 1) is approximately 2.7 psi. Resistance to anything less than 3 psi would indicate questionable structural integrity.

7.2.3 Resistance Interaction Relationship

The resistance of a vertical strip of the north wall will be approximately the same as that of a wall strip of the "Drake" Building if the wall is unreinforced. Everything that was said in the previous study regarding upper and lower bounds of resistance applies here as well. If the wall is reinforced, the resistance is of course considerably greater.

7.2.4 Analysis

Even without reinforcement, the resistance of this wall is greater than that of the "Baker" Building. A gas explosion in one of the upper three stories might blow out a section of wall but, below this, it is probable that the wall would not be breached. With reinforcement, effective resistance to local damage is probable in all but the top story.

Since the wall is not pierced by windows, its arching and corbeling capabilities appear excellent. A finite element analysis would be useful to confirm or disprove this supposition. Even without such an analysis, previous studies indicate that failure of this wall would be localized and not affect more than two or three stories at most.

7.3 Conclusions for "Edwards" Building

Of the five buildings studied, the progressive collapse resistance of the Edwards Building appears to be the best. Floor ties at the ends of the precast planks would improve the integrity without adding substantially to the cost but, even without these, there does not appear to be much chance of progressive collapse.

8. Conclusions - Progressive Collapse

Five typical non-reinforced masonry buildings were studied, two in detail and three superficially. Progressive collapse following a natural gas explosion is a serious possibility in one (the Adams Building) and also very probable in a second (the Carter Building). One (the Baker Building) appears marginal if it is recognized that the assumption made regarding the integrity of the 12-in x-16 in pier (section 4.2.1 and figure 4.2) was very liberal. In the remaining two, reasoning indicates that failure would be arrested before there were serious consequences in regions removed from the source of the explosion.

The probability of progressive collapse in <u>a particular building</u> is very small even though the probability of encountering at least one progressive collapse in <u>a class of buildings</u> (such as non-reinforced masonry) susceptible to such failure may be high. However, the substantial danger of progressive collapse in almost half of the buildings of the small sample studied in this report is considered sufficiently great to warrant further research in the resistance of non-reinforced masonry structures. The research needs that have become apparent in the course of these studies are outlined in the following section.

9. Recommendations for Study and Research

9.1 Design Against Progressive Collapse

Three measures used in dealing with progressive collapse were identified in early British response to the Ronan Point collapse [see reference 9 for a review]:

a). General Structural Integrity--provision of sufficient redundancy to give the structure a good chance of suffering only local failure under unpredictable abnormal loading. This is accomplished through joint continuity and inter-member ties. Experience has shown that general structural integrity is inherent in most conventional steel and concrete structures. It has not, as yet, been demonstrated that the same is true for the newer forms of masonry construction and industrialized construction.

b). Specific Local Resistance--design of members to resist failure under a presumably realistic, predictable, abnormal loading. Provision of "5 psi resistance" is an example of this approach.

c). Alternate Paths--provision of ability to span temporarily the gap left by the destruction of a member not capable of resisting the predictable abnormal loading. Conceptual removal of a member incapable of resisting 5 psi and verifying alternate paths in the remainder of the structure is an example of the use of this measure.

It appears that all three measures will continue to be used in combating progressive collapse. It is reasonable to try to provide in all buildings structural integrity comparable to that found in conventional steel and concrete construction. In many structures there will be members whose loss under a reasonable abnormal loading cannot be tolerated. Such members should be designed for specific local resistance. In general, however, local failure under abnormal loading should be acceptable provided alternate paths exist to prevent immediate progressive collapse.

Objections to the earlier British progressive collapse criteria centered not on these three measures, as such, but rather on details of the ways in which their implementation was prescribed. American criteria should retain the measures but try to avoid the pitfalls in the early U.K. standards. The research suggested in section 9.3 should help to do this, at least in the formulation of specifications for the prevention of progressive collapse in masonry structures.

9.2 Engineered Masonry Construction - State of the Art

There are paradoxes in masonry construction. It is often thought of as being conventional, yet some of the newest masonry structures are, in fact, very unconventional. It is the oldest way of building large structures yet, in a number of respects, its technology lags behind that of newer forms. In spite of its age, much of masonry's great versatility and amenability to innovation are just beginning to be exploited.

To put masonry technology on a level of sophistication comparable to that in steel and concrete, construction control has to be improved, structural analyses appropriate to the medium have to be introduced, codes must be rationalized and modernized, and the research needed to support these developments must be conducted. It is inevitable, therefore, that in suggesting research on progressive collapse in masonry one must extend the scope to include subjects relating not only to the progressive collapse problem, but to masonry design and construction in general.

9.3 Suggested Studies and Research

The following ideas stem from the work done in the case studies reported above:

a). Joint Behavior--General structural integrity and ability to provide alternate paths depend upon the behavior of connections between the floor elements (concrete or steel) and the masonry walls. There seems to be little research on this subject. There is need for large, comprehensive testing programs of typical existing connections as well as of designs that might emerge from the research. The programs should be designed so as to have statistical significance. Some of the specific topics to be investigated are:

1). The effective rigidity of joints. Moment-rotation diagrams. The development of resisting moments in masonry walls supporting loaded floors.

2). Distribution of contact pressure under slabs supported by walls. (This has been the subject of several investigations)

3). Local contact stresses and crushing under bends or hooks in reinforcing bars embedded in masonry. Tear-out of such bends or hooks.

4). Effectiveness of bond and anchorage of reinforcing bars embedded in shear keys in floors, floor topping, and several courses of masonry (see figure 3.4 and 4.3).

b). Bond Between Reinforcement and Masonry--This is related to the connection problem. Allowable bond stresses have been established for masonry but the evidence on which they are based is rather sparse and rather old. Further, it is not clear how well these data apply to the common case in which reinforcing bars run vertically through a number of courses of masonry (see figures 3.4 and 4.3). Statistically significant series of pull-out tests should be run to study the roles of the various parameters: bars, mortar, masonry unit, and workmanship.

c). In-Plane Behavior of Walls--Masonry is often used as a planar structural element. Experience has shown that it has ability to arch, corbel, or otherwise distribute forces around openings, re-entrant corners, and zones of tensile stress. But there seems to have been no effort to develop rational methods for analyzing this capability. Research applying finite element techniques to typical situations should be undertaken. The effects of cracking, orthotropy, and non-homogeneity should be included. Tests of walls would also be necessary to confirm the analytical methods or to help in modifying them (See section 4.2.4).

d). Dynamic Response--The above suggestions deal with static behavior. Numerous studies have been made of the response of walls to conventional explosions. There seem to have been few quantitative studies of the resistance of walls and connections to the slower dynamic loading encountered in gaseous deflagrations. In the area of response to slow explosions specific topics to consider are:

1). The comparison of theoretical and measured vibration characteristics of walls and piers. Refinement of methods for calculating frequencies of masonry elements.

2). Non-destructive experimental studies of response of masonry walls to pulses of various durations and peak pressures. Verification of conditions under which a static analysis is valid.

3). Destructive experimental studies to check the validity of analytical failure mechanisms and load-deformation relationships (moment-rotation and stress-strain diagrams) used in applying these mechanisms. One of the difficult problems in applying past test results to wall analysis is the assessment of whether the boundary conditions present in the tests match those in the actual structure. Constraint against in-plane movement can substantially alter resistance to lateral pressure. Wall tests should attempt to simulate the translational and rotational restraint conditions present in actual structures.

4). Following investigation of the static behavior of joints (Item (a) above), comparative tests of response to slow explosions would be important in verifying the applicability of the static tests to the abnormal loading problem.

e). Stress-Strain Properties of Masonry--This is the first item recommended for research in reference 15. In that reference it is advised that, "this study should include a thorough investigation of the relationship between compression strength in one-dimensional compression and in flexure and investigation of the stress distribution corresponding to linear strain gradients." This recommendation is strongly seconded as probably the most important piece of research needed to improve basic understanding of masonry behavior. Until the apparent increase in material strength with strain gradient is more thoroughly understood, any analytical method for calculating resistance to flexure or combined flexure and direct force is open to the criticism of being weakly founded and of uncertain generality.

Reference 15 contains other suggestions for important research. These should be implemented, but it is believed that the investigation of stress-strain properties deserves priority.

f). Bond Beams--Intuitively, tie or bond beams such as illustrated on figure 6.3 appear to be effective in increasing general structural integrity and resistance to progressive collapse. However, this intuitive impression has not been the subject of intensive study. One might, for example, analyze and then test the relative resistance of two masonry walls (one with and one without a tie beam such as shown on figure 6.3) to in-plane forces following loss of a section of wall below the one considered.

g). Composite Walls--Considerable work had been done on the interaction of two types of masonry in one wall. But the resistance to gaseous deflagration of a composite wall such as that shown in figure 6.3 is not at all clear. Tests of walls of this type, perhaps in conjunction with some of the studies described in Item (d) above, would be informative.

h). Design Examples--Detailed methods for the verification of alternate paths will probably remain beyond codification because of the many different situations encountered in practice. Nevertheless, a publication containing illustrative examples drawn from actual, representative structures, would be a useful document.

10. Acknowledgements

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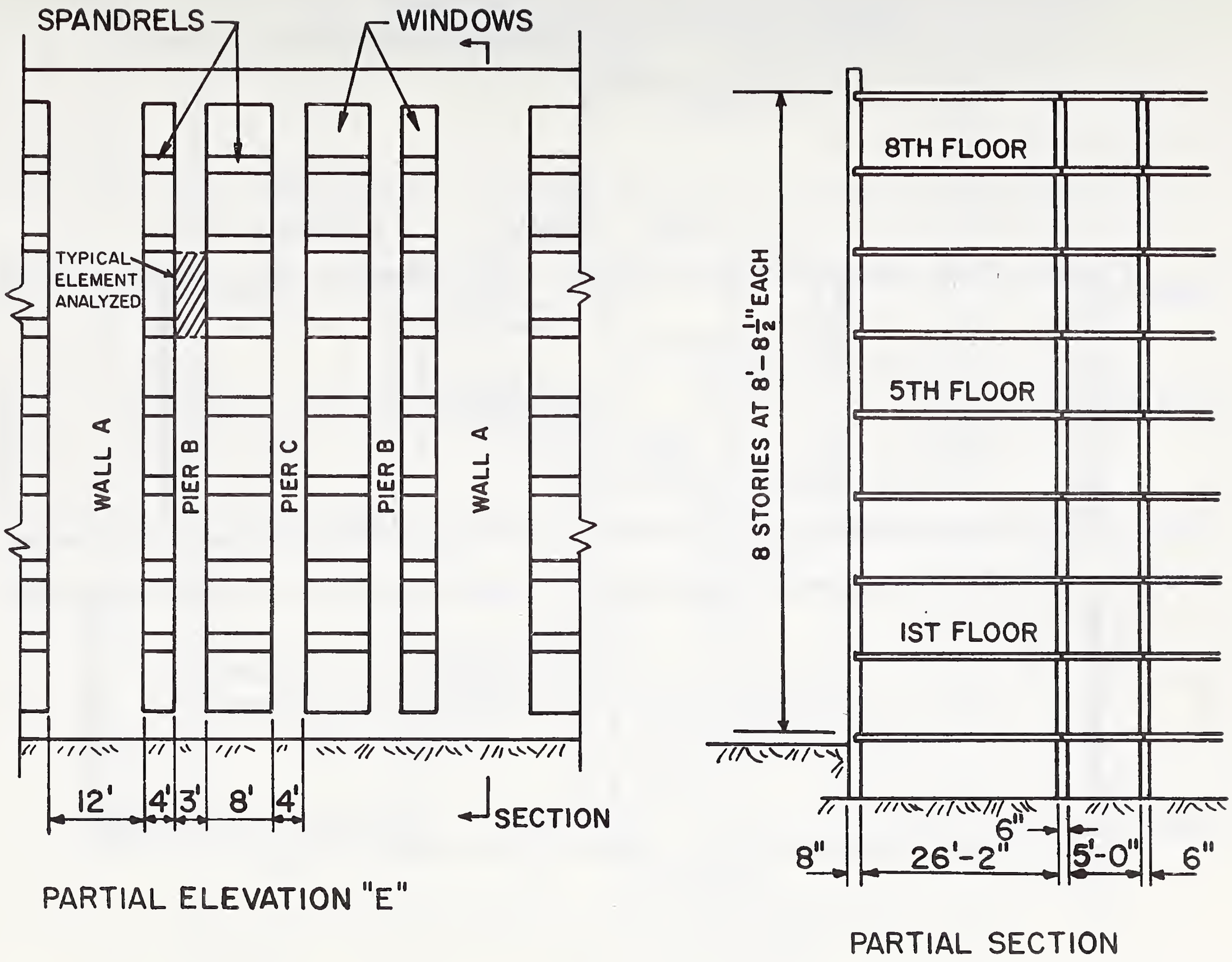


Figure 3.1 Partial Elevation and Section, Adams Building.

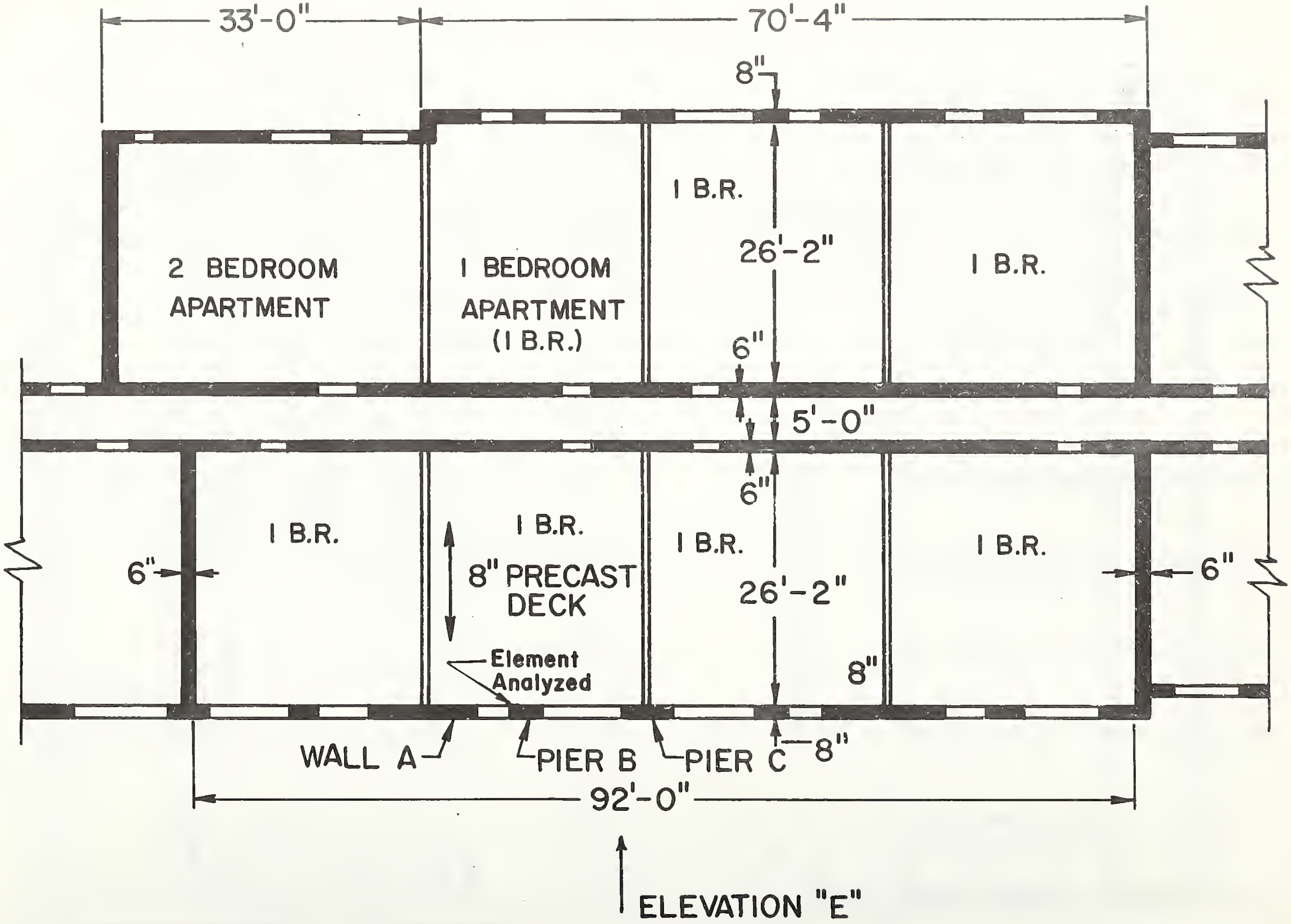


Figure 3.2 Partial Floor Plan, Adams Building.

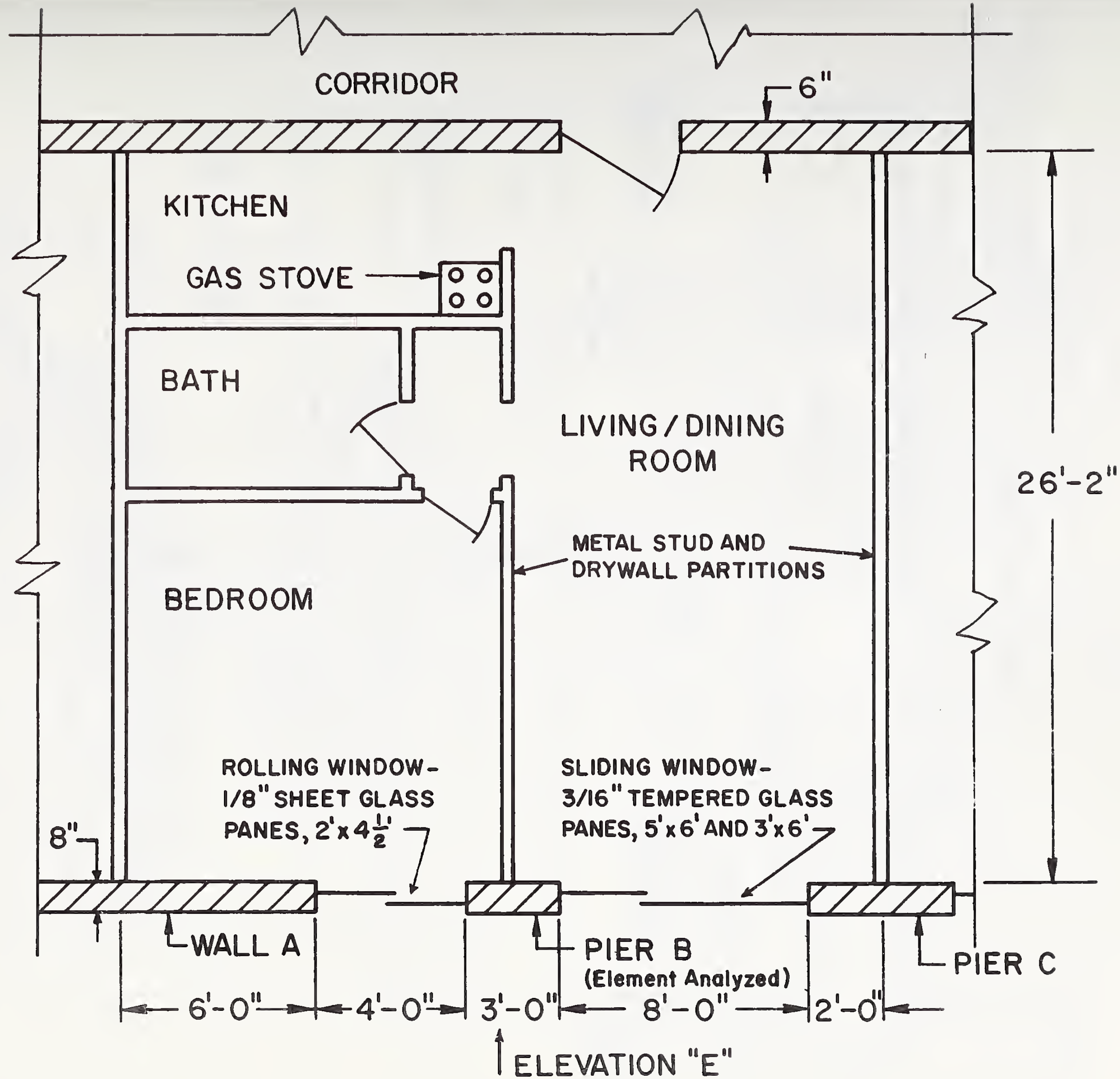


Figure 3.3 Typical One-Bedroom Apartment, Adams Building.

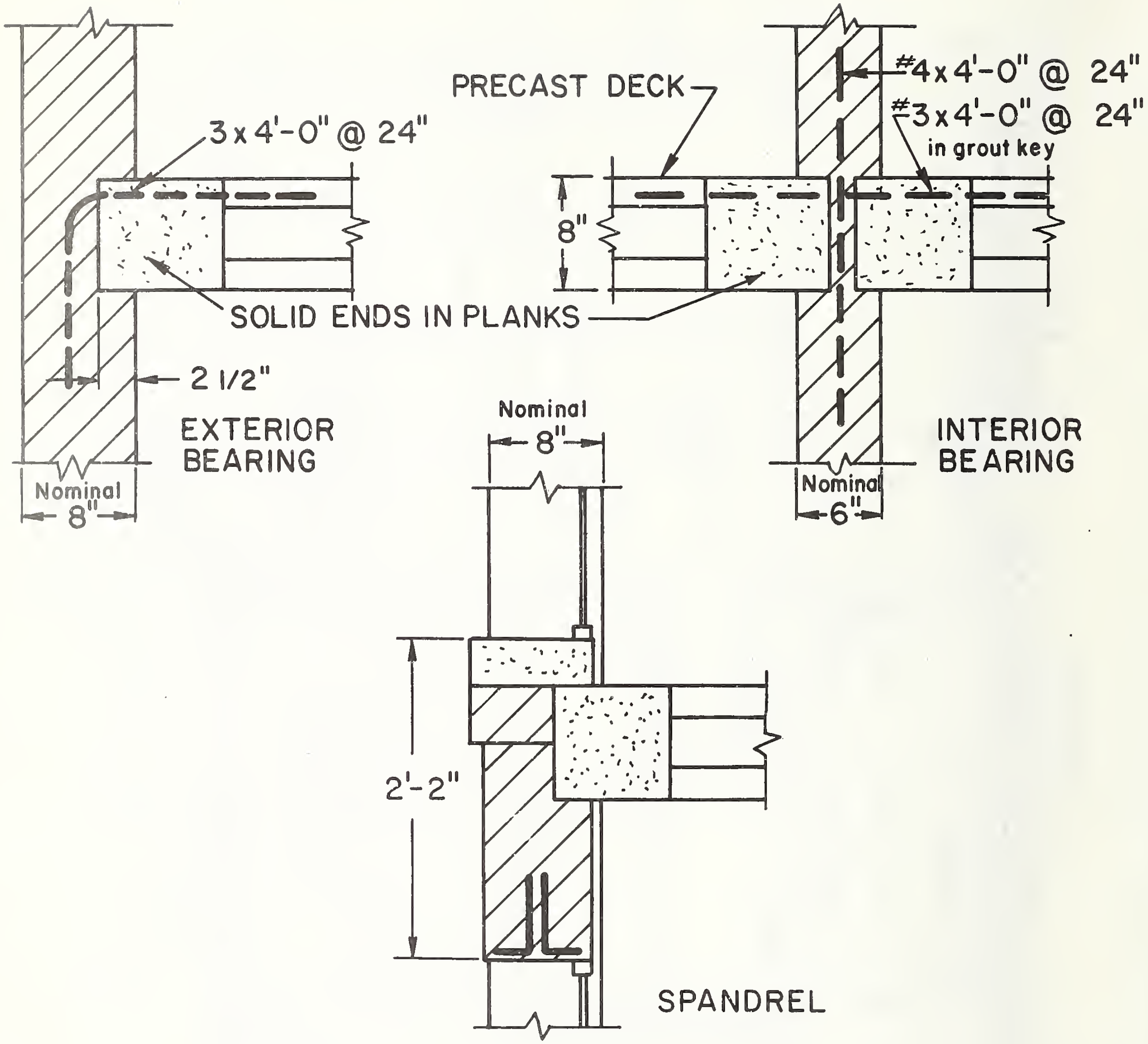


Figure 3.4 Floor-Wall Connection Details, Adams Building.

PRESSURE, $p_m = 1.5\ p_v + 0.4K$

VENT OPENING PRESSURE, p_v

(a) Rasbash-Stretch, Reference 6 and 8
$p_v = 1$ psi for all cases

(b) Mainstone, Reference 3 (charts)

Bedroom: Window area = 0.84 m^2 (single pane)
Window aspect ratio = 2.25
Limiting p_v* from figure 2, reference 3
$p_v = 1.5 \times 0.45$ psi
$p_v = 0.68$ psi

Living/Dining Room: Window area = 2.8 m^2 (single pane)
Window aspect ratio = 1.2
Limiting p_v* from figure 2, reference 3 (use
3 times 3/16 in sheet glass values
$p_v = 1.5 \times 3 \times 0.29$ psi
$p_v = 1.35$ psi

*Mainstone's "limiting p_v" is 1.5 times the "most likely" value read from the charts in reference 3.

VENTING RATIO, K

(a) Bedroom - Use full rolling window opening plus 1'-0" depth of fixed glass below
$K = 11.5' \times 8.0'/4.0' \times 5.5' = 4.2$

(b) Living/Dining Room - Use full sliding window opening
$K = 11.5' \times 8.0'/8.0' \times 6.0' = 1.9$

LOADING FUNCTION

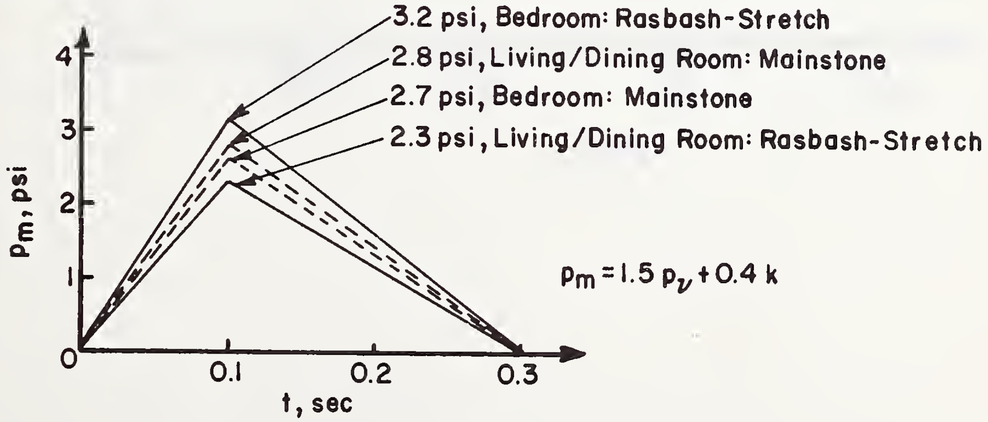


Figure 3.5 Loading Function for Pier B, Adams Building.

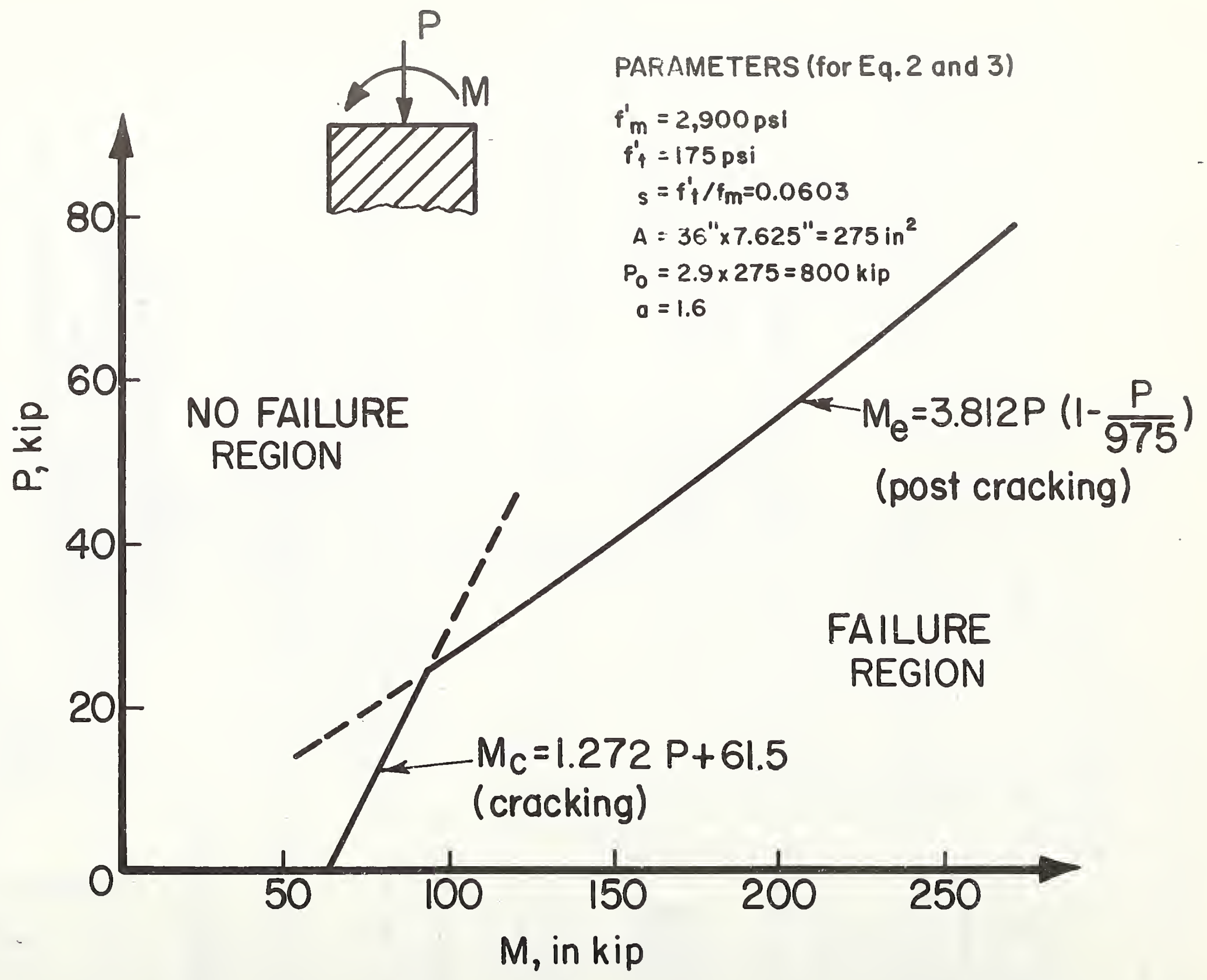


Figure 3.6 Interaction Diagram for Pier B, Adams Building.

$$T = \frac{2\ell^2}{\pi}\sqrt{\frac{m}{(EI)_{eff}}}$$

T = natural period

ℓ = pier height

$(EI)_{eff}$ = effective pier stiffness = 0.7 EI [13]

m = mass of pier

use:

$$\ell = 96 \text{ in}$$

$$(EI)_{eff} = 0.7EI = 2.69 \times 10^9 \text{ \# in}^2$$

$$m = 0.0519 \text{ \# sec}^2/\text{in}^2$$

$$T = 0.025 \text{ sec}$$

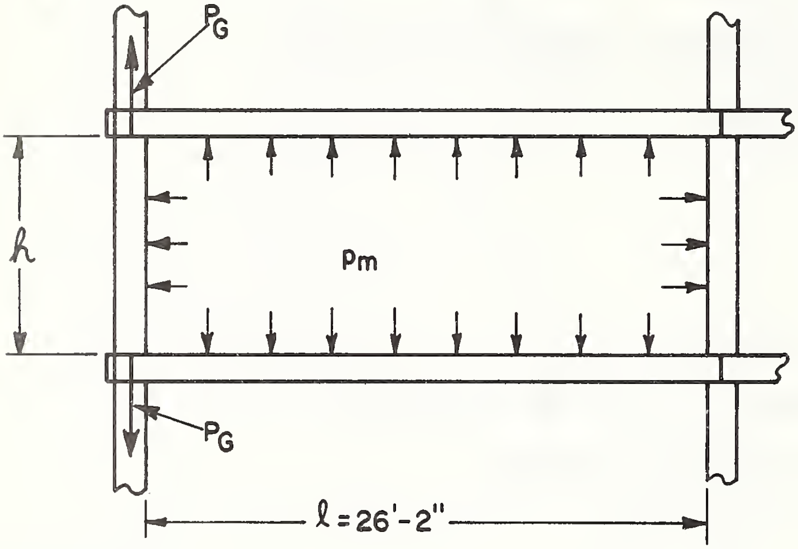
$$\frac{t_m}{T} = \frac{0.10}{0.025} = 4$$

where t_m = rise time to maximum pressure [4]

Figure 3.7 Natural Period for Pier B, Adams Building.

Assumed Loading for pier B:

- Zero live load on roof
- 5psf live load on all floors
- 9 feet of roof and floor width tributary to Pier B
- 6 feet of spandrel length tributary to Pier B
- Uplift due to explosion on outer 1/4 of floor and ceiling



$$P_G = p_m \times 9' \times \frac{26.17'}{4} \times 12^2$$

$$P_G = 8500\ p_m$$

(P_G in lb and p_m in psi)

Unit Loads:

8" Roof Deck	.060 ksf
Roofing	.002
Roof Dead Load	.062 ksf

Roof D.L. on Pier = .062x13.1x9
= 7.30 kip

8" Floor Deck	.060 ksf
Partitions	.005
Floor Dead Load	.065 ksf

Floor D.L. on Pier = .065x13.1x9
= 7.65 kip/floor

Floor L.L. on Pier = .005x13.1x9
= 0.59 kip/floor

Parapet Load = 3'x 9'x 0.080 = 2.16 kip/pier

Spandrel: Lintel 2 x 0.080 = 0.160

Windows 6.5 x 0.025 = 0.016

0.176 kip/ft/spandrel

0.176 x 6 = 1.05 kip/pier/story

Pier weight = 3' x 8.71' x 0.080 = 2.10 kip/story

Figure 3.8 Vertical Loads, Adams Building.

ITEM	LOAD
Parapet	2.16 kip
Roof Dead Load	7.30
	9.46 kip 8th Story - Top
1/2 Pier	1.05
	10.51 kip 8th Story - Midheight
1/2 Pier	1.05
	11.56 kip 8th Story - Bottom
Dead Load, 3 Floors (3x9.65)	22.90
Live Load, 3 Floors (3x0.59)	1.77
Piers, 2 stories (2x2.10)	4.20
Spandrels, 3 Floors (3x1.05)	3.15
	43.58 kip 5th Story - Top
1/2 Pier	1.05
	44.63 kip 5th Story - Midheight
1/2 Pier	1.05
	45.68 kip 5th Story - Bottom
Dead Load, 3 Floors	22.90
Live Load, 3 Floors	1.77
Piers, 2 stories (2x2.10)	4.20
Spandrels, 3 Floors (3x1.05)	3.15
	77.70 kip 1st Story - Top
1/2 Pier	1.05
	78.75 kip 1st Story - Midheight
1/2 Pier	1.05
	79.80 kip 1st Story - Bottom

(unit loads from figure 3.8)

Figure 3.9 Pier Forces, Adams Building.

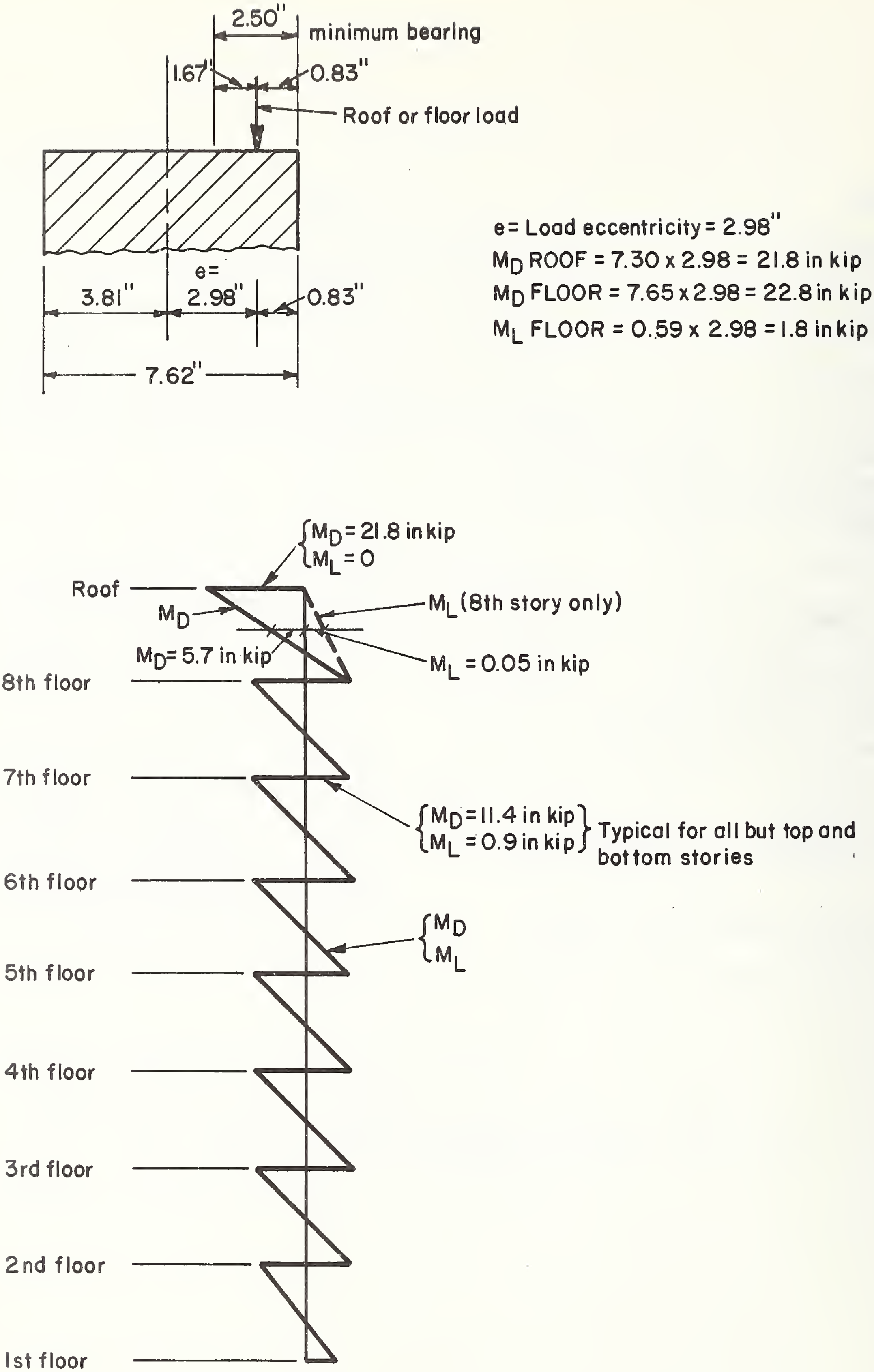
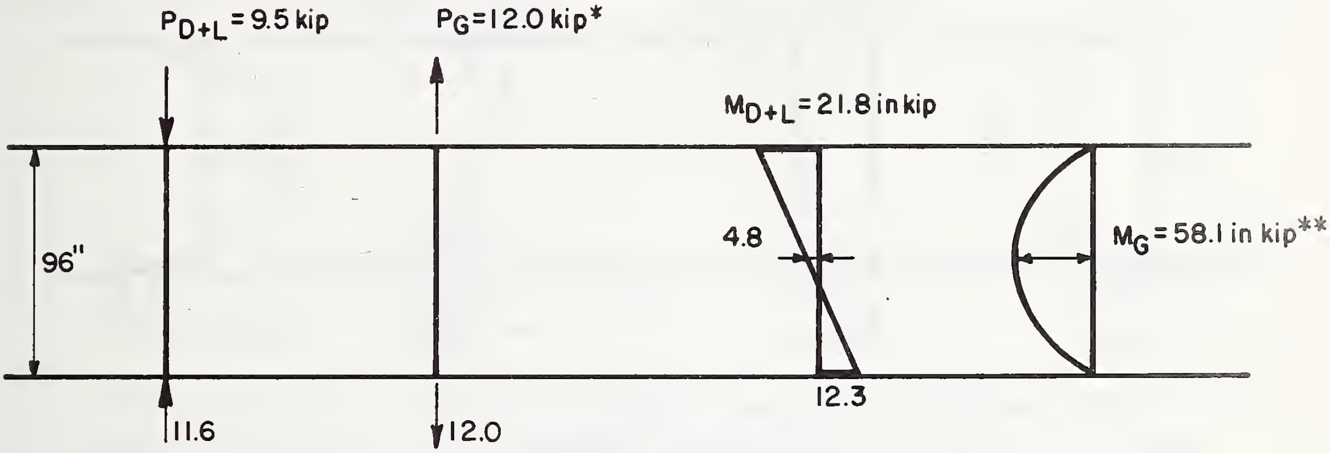


Figure 3.10 Pier Moments due to Eccentricity, Adams Building.

TRY $p_m = 1.4$ psi   Assume pinned-pinned piers



$^*P_G = 8.5\ p_m = 12.0$ kip (from figure 3.8)

$^{**}M_G = 1/8\ w h^2 = 1/8 \times (1.4 \times 36) \times (96)^2 = 58.1$ in kip

$P_{D+L+G} \approx 0$

$M_{D+L+G} = 62.9$ in kip

From Eq. 2, $M_c = 61.5$ in kip $<$ 62.9 in kip

use $\underline{p_m = 1.4 \text{ psi}}$

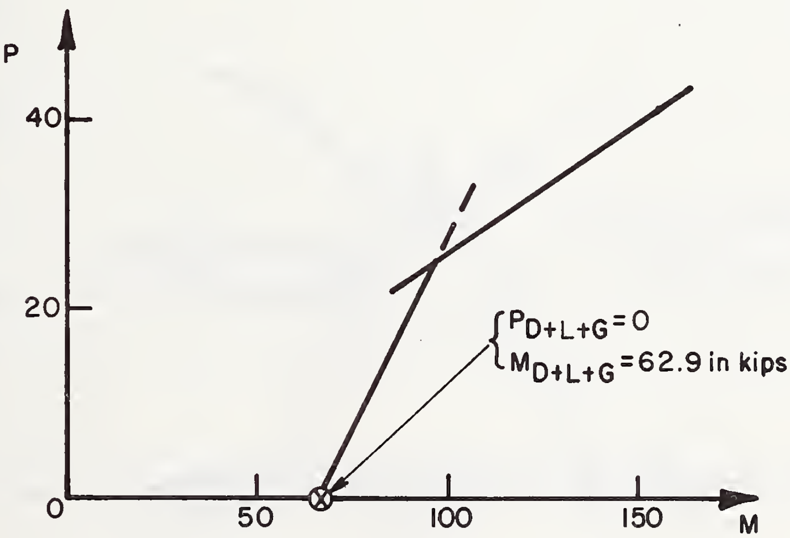
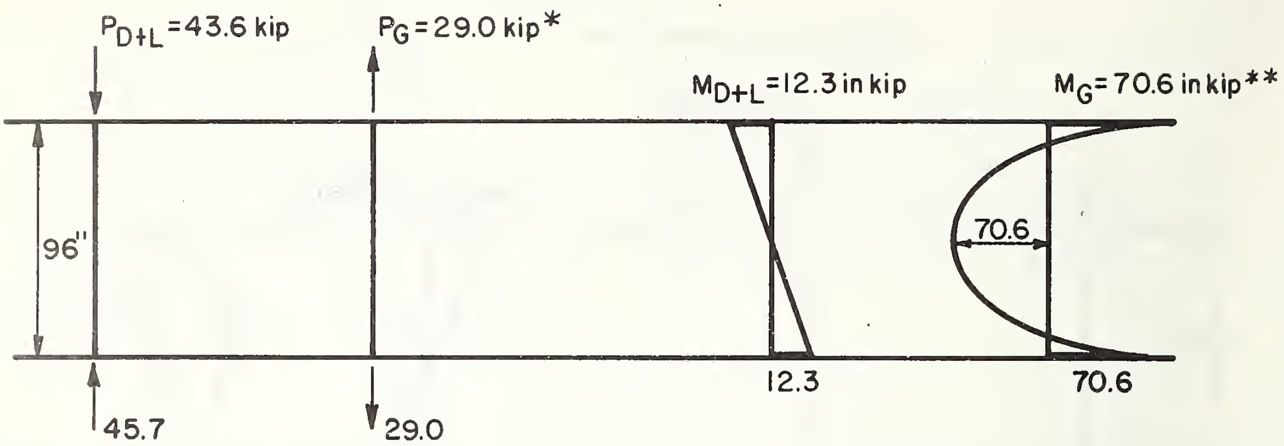


Figure 3.11 Eighth-Story Analysis for Pier B, Adams Building.

TRY $\underline{p_m = 3.4 \text{ psi}}$ Assume equal end and midspan moments



$^*P_G = 8.5\, p_m = 29.0$ kip (from figure 3.8)

$^{**}M_G = 1/16\, w h^2 = 1/16 \times (3.4 \times 36) \times (96)^2 = 70.6$ in kip

$P_{D+L+G} = 16.7$ kip

$M_{D+L+G} = 82.9$ in kip

From Eq. 2, $M_c = 1.272 \times 16.7 + 61.5 = 82.7$ in kip ≈ 82.9 in kip

use $\underline{p_m = 3.4 \text{ psi}}$

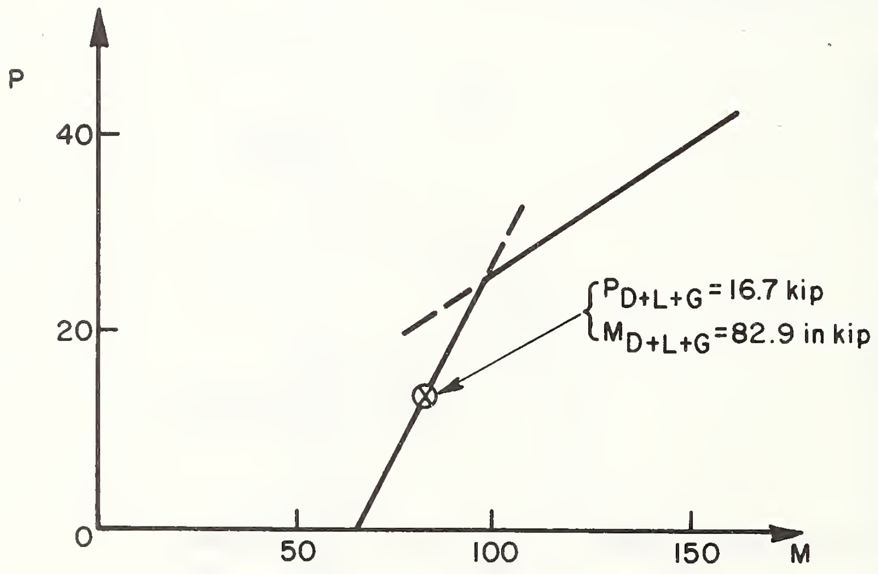
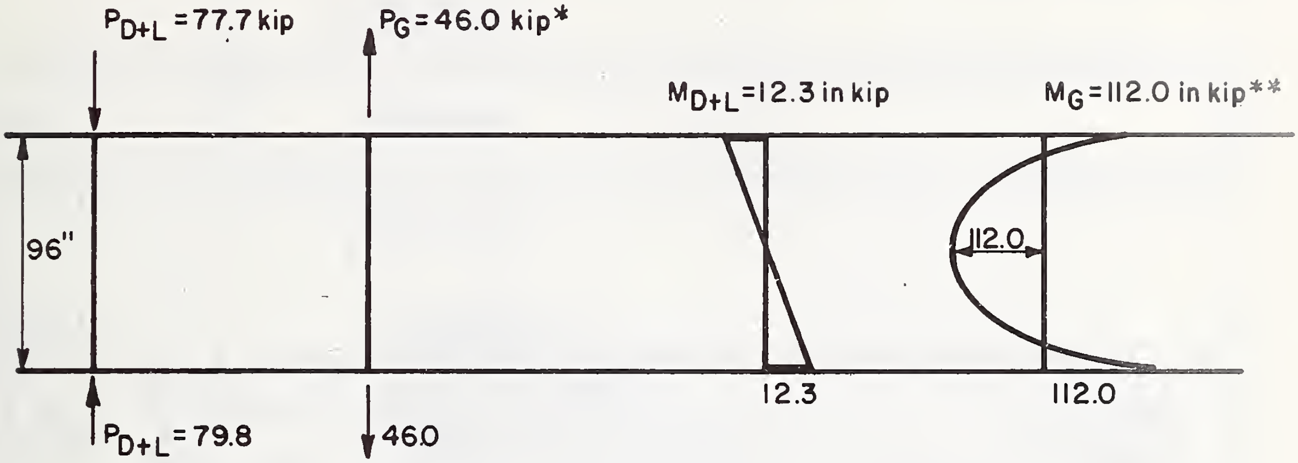


Figure 3.12 Fifth-Story Analysis for Pier B, Adams Building.

TRY $p_m = 5.4$ psi Assume end and midspan moments



$*P_G = 8.5\, p_m = 46.0$ kip (from figure 3.8)

$**M_G = 1/16\ w\,\ell^2 = 1/16 \times (5.4 \times 36) \times (96)^2 = 112.0$ in kip

$P_{D+L+G} = 33.8$ kip

$M_{D+L+G} = 124.3$ in kip

From Eq. 3, $M_e = 3.812 \times 33.8 \left[1 - \frac{33.8}{975}\right] = 124.5$ in kip ≈ 124.3 in kip

use $\underline{p_m = 5.4 \text{ psi}}$

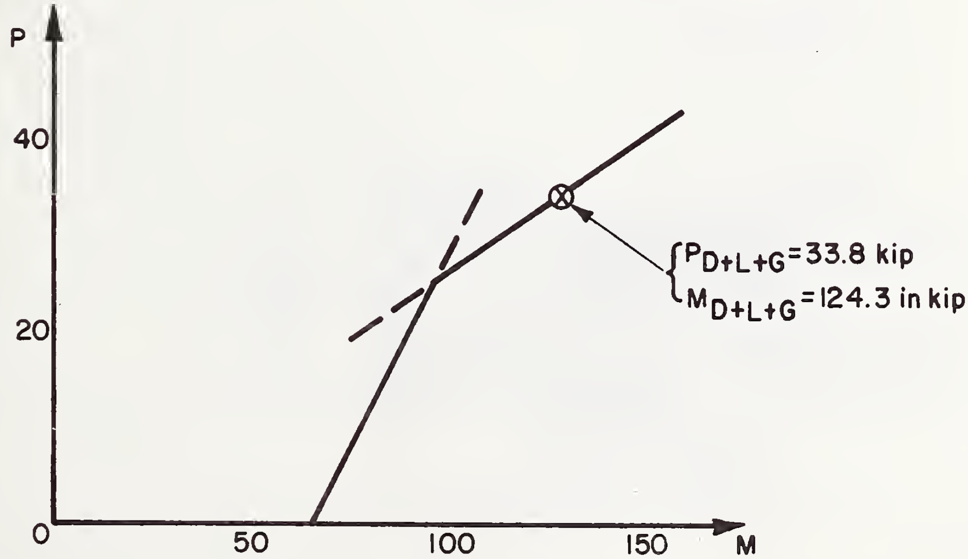


Figure 3.13 First-Story Analysis for Pier B, Adams Building.

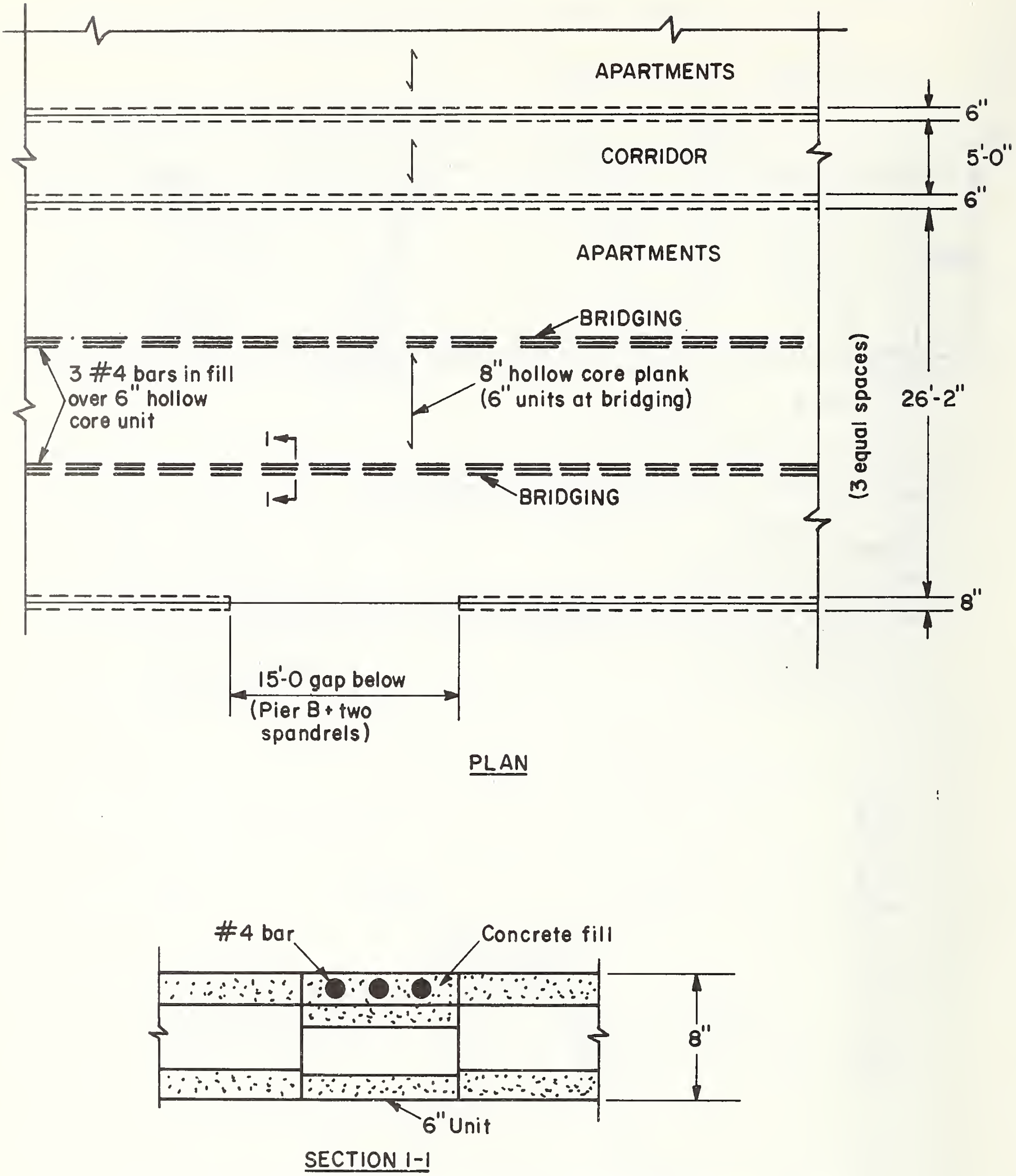


Figure 3.14a Bridging, Adams Building.

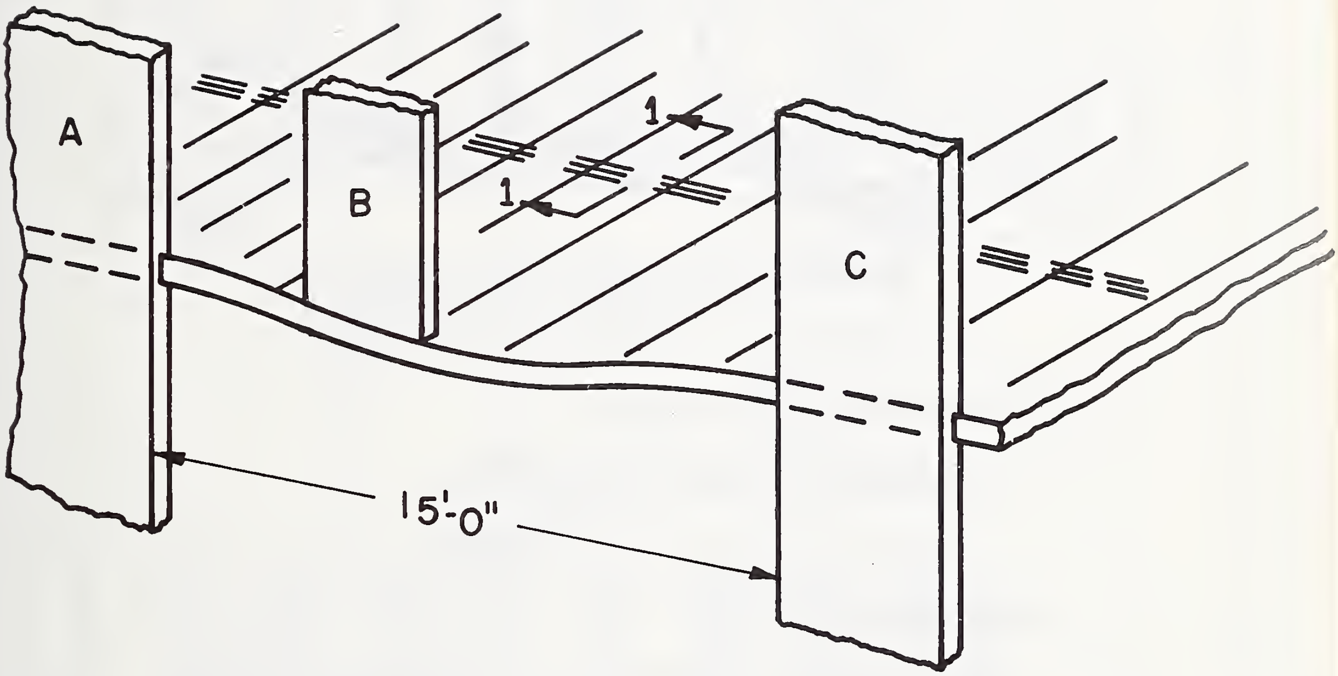
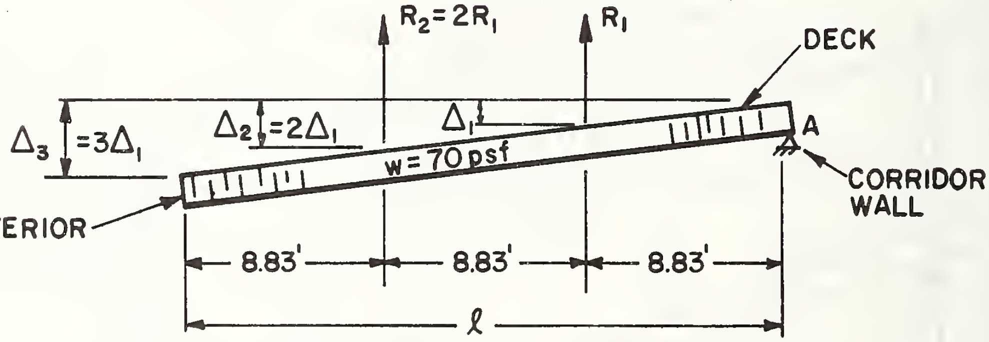


Figure 3.14b Bridging, Adams Building.

ASSUMPTIONS

- Assume 60 psf planks + 5 psf partitions + 5 psf live = 70 psf
- Assume plank rotates about corridor wall and derives other support from two catenarys of equal stiffness

FLOOR DECK

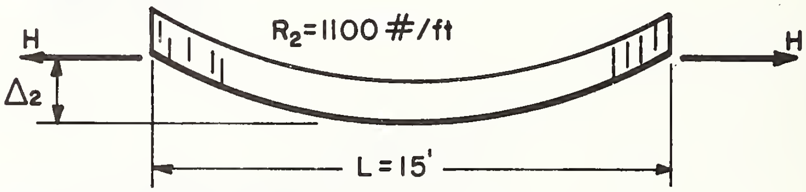


$$\sum M_A = R_1 \times \frac{\ell}{3} + R_2 \times \frac{2\ell}{3} - w\ell \times \frac{\ell}{2} = 0$$

$$R_1(8.83 + 2 \times 2 \times 8.83) - \frac{70(3 \times 8.83)^2}{2} = 0$$

$$R_1 = 550 \text{ lb}$$

BRIDGING CATENARY:



Assume 3-#4 bars yield (steel area = 3×0.20 in^2, yield stress = 60,000 psi)

$$H = 3 \times 0.20 \times 60{,}000 = 36{,}000^{\#}$$

$$\Delta_2 = \frac{R_2 L^2}{8H} = \frac{1100(15)^2(12)}{8 \times 36{,}000} = 10.3''$$

END DEFLECTION: $\underline{\Delta_3} = \frac{3}{2} \times 10.3 = \underline{15.5''}$

Figure 3.15 Alternate Path Analysis, Adams Building.

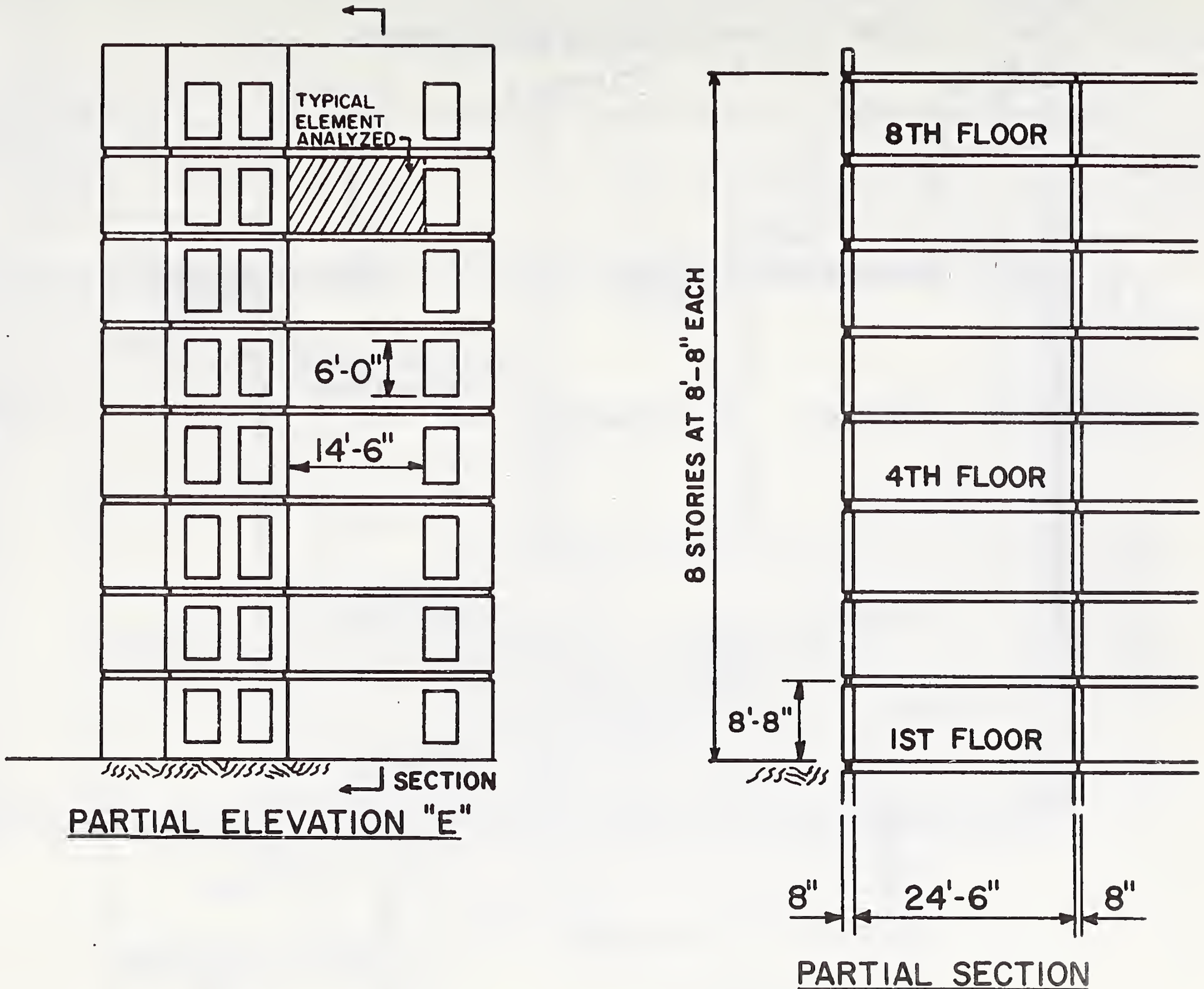


Figure 4.1 Partial Elevation and Section, Baker Building.

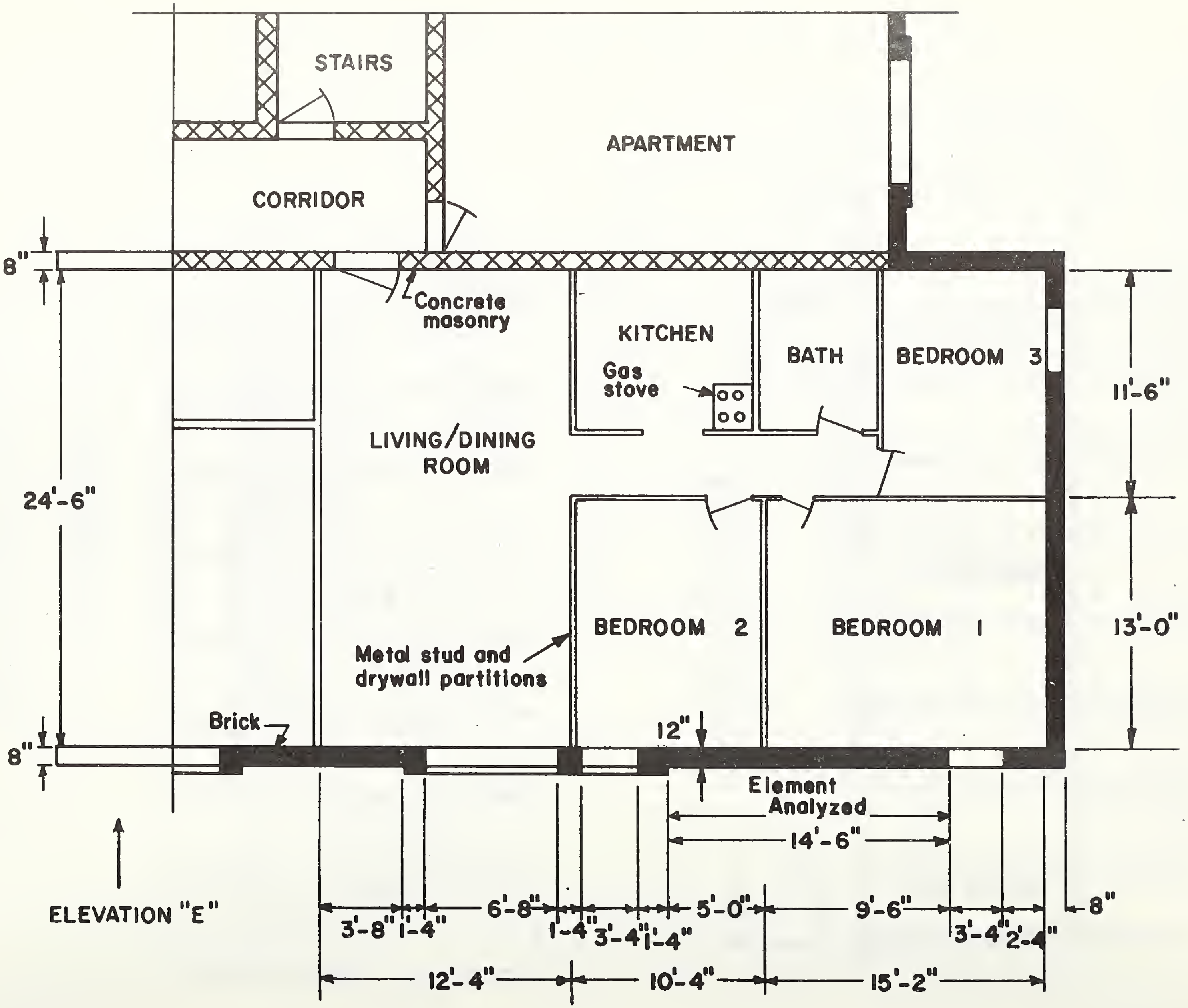


Figure 4.2 Apartment Plan, Baker Building.

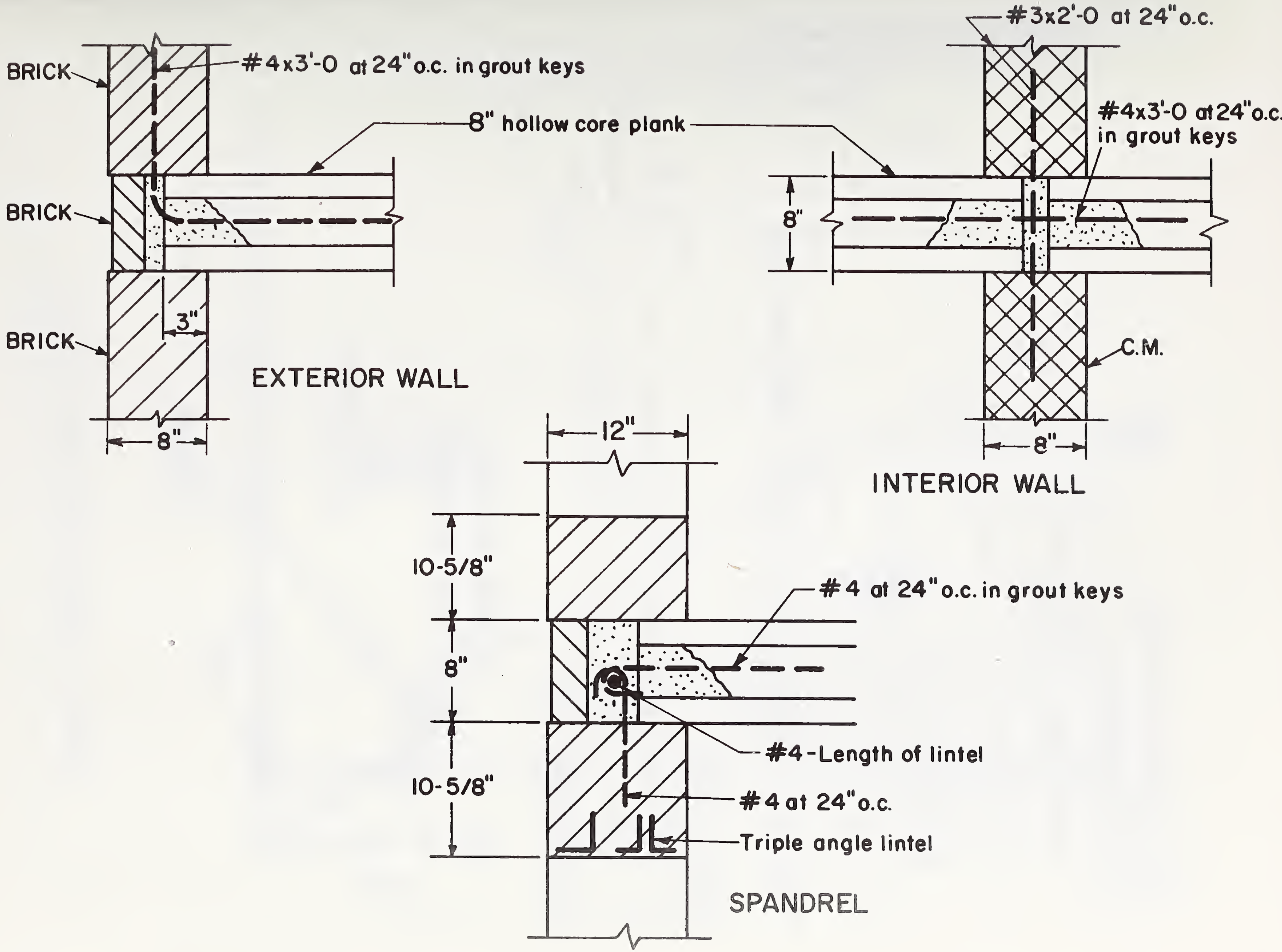


Figure 4.3 Floor-Wall Connection Details, Baker Building.

Pressure, $p_m = 1.5\ p_v + 0.4K$

VENT OPENING PRESSURE, p_v

(a) Rasbash-Stretch, Reference 6 and 8
P_v = 1 psi for all cases

(b) Mainstone, Reference 3, (charts)

Window area = 0.84 m^2
Aspect ratio = 1.00
Limiting p_v* from figure 2, reference 3

p_v = 1.5 x 0.7 psi

p_v = 1.05 psi, use 1.0 psi as in (a)

*Mainstone's "limiting p_v" is 1.5 times the "most likely" value read from the charts in reference 3.

VENTING RATIO, K

(a) Bedroom No. 1 - K = 13.0' x 8.0'/3.33' x 6.0' = 5.2

(b) Bedroom No. 2 - K = 10.33' x 8.0'/3.33' x 6.0' = 4.2

LOADING FUNCTION

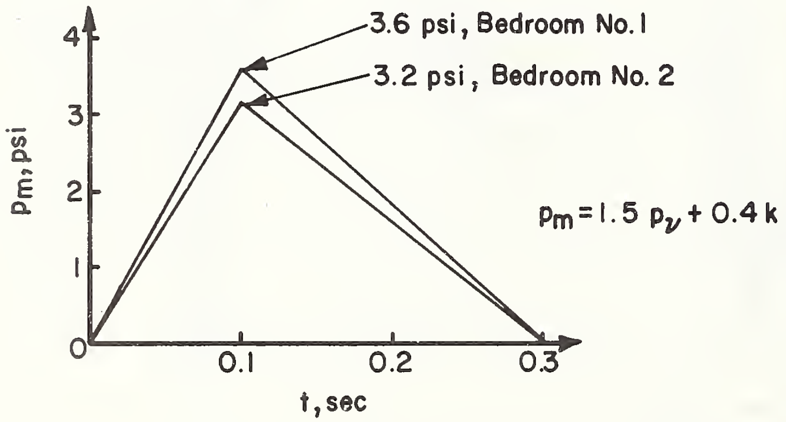


Figure 4.4 Loading Function, Baker Building.

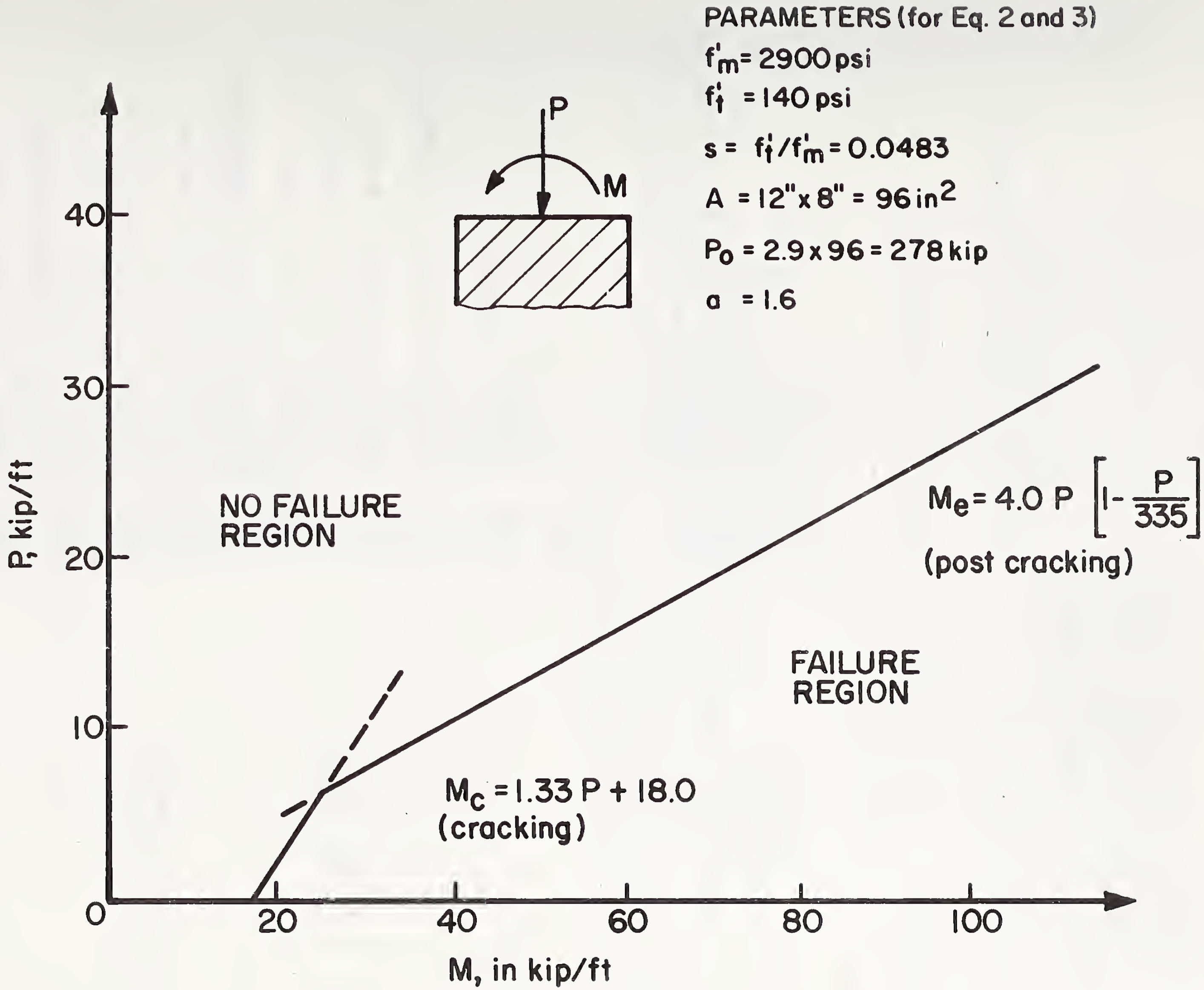


Figure 4.5 Interaction Diagram, Baker Building.

Assumed Loading:

- Zero Live on roof
- 5 psf Live Load on floor
- Uplift due to explosion on outer 1/4 of floor and ceiling

$P_G = p_m \times 12'' \times \frac{24.5'}{4} \times 12''$ (refer to figure 3.8 for diagram)

$P_G = 0.88\ p_m$ (P_G in lb/ft and p_m in psi)

Unit Loads:

8" Roof Deck	.060 ksf	8" Floor Deck	.060 ksf
Roofing	.002	Partitions	.005
	.062 ksf	Floor Dead Load	.065 ksf

Roof D.L. on Wall=.062x12.25=.76 k/ft

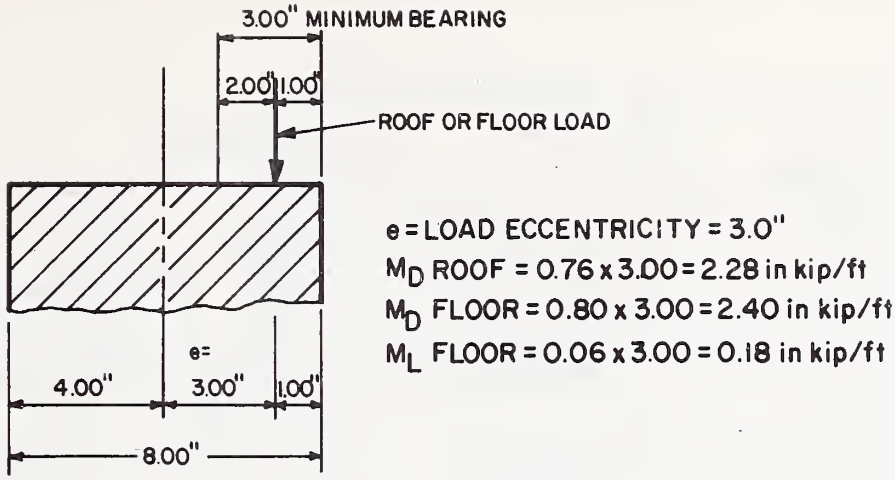
Floor D.L. on Wall=.065x12.25=.80 kip/ft

Floor D.L. on Wall=.005x12.25=.06 kip/ft

Wall weight = 8.67'x.080=0.70 kip/story

ITEM	LOAD		
Roof Dead Load	0.76	kip/ft	
	0.76	kip/ft	8th Story - Top
1/2 Wall	.35		
	1.11	kip/ft	8th Story - Midheight
1/2 Wall	0.35		
	1.46	kip/ft	8th Story - Bottom
Dead Load - 4 Floors 4x0.80	3.20		
Live Load - 4 Floors 4x0.06	.24		
	4.90	kip/ft	4th Story - Top
1/2 Wall	.35		
	5.25	kip/ft	4th Story - Midheight
1/2 Wall	.35		
	5.60	kip/ft	4th Story - Bottom
Dead Load - 3 Floors 3x0.80	2.40		
Live Load - 3 Floors 3x0.06	0.18		
	8.18	kip/ft	1st Story - Top
1/2 Wall	.35		
	8.53	kip/ft	1st Story - Midheight
1/2 Wall	.35		
	8.88	kip/ft	1st Story - Bottom

Figure 4.6 Vertical Loads and Forces, Baker Building.



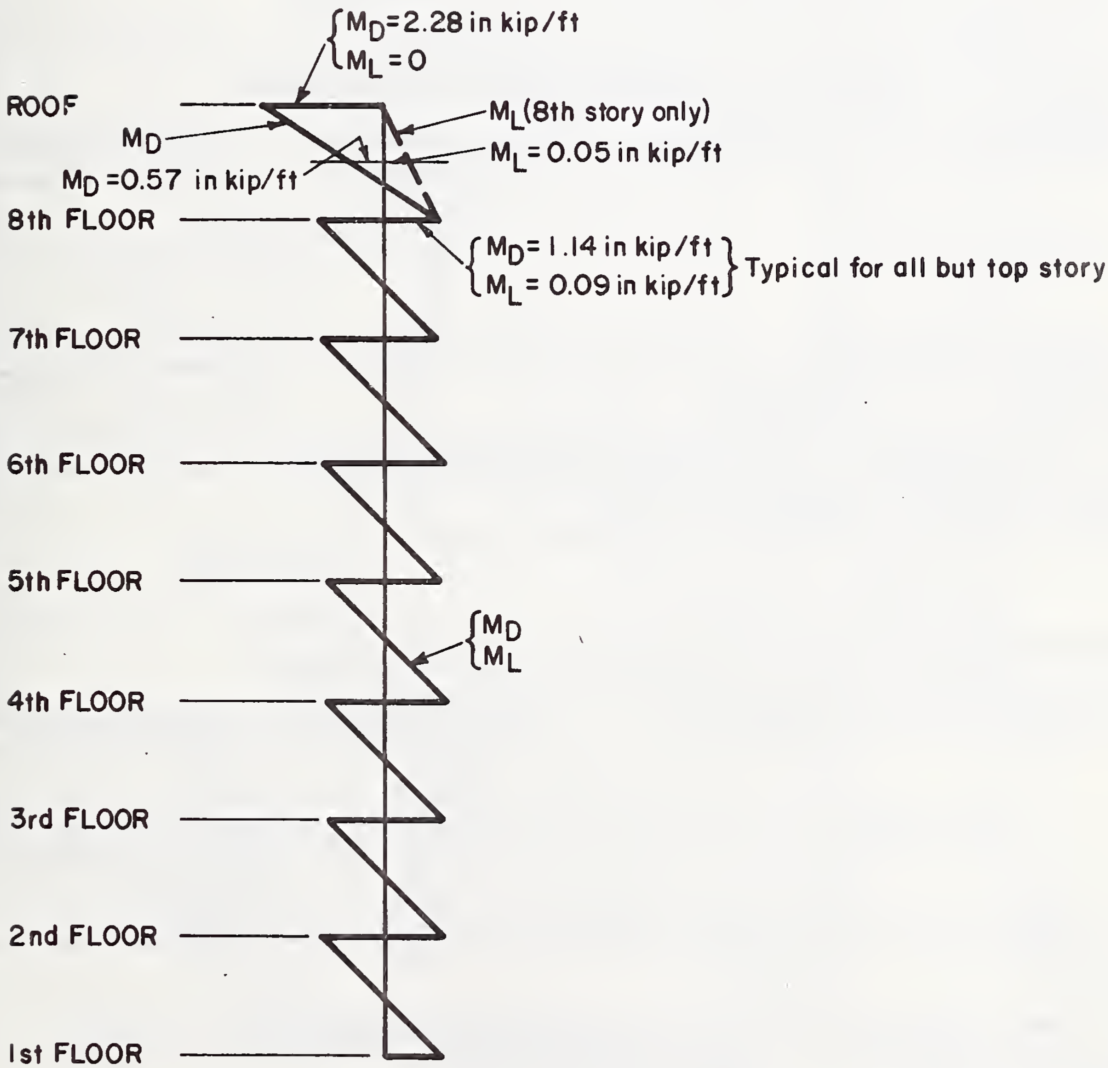
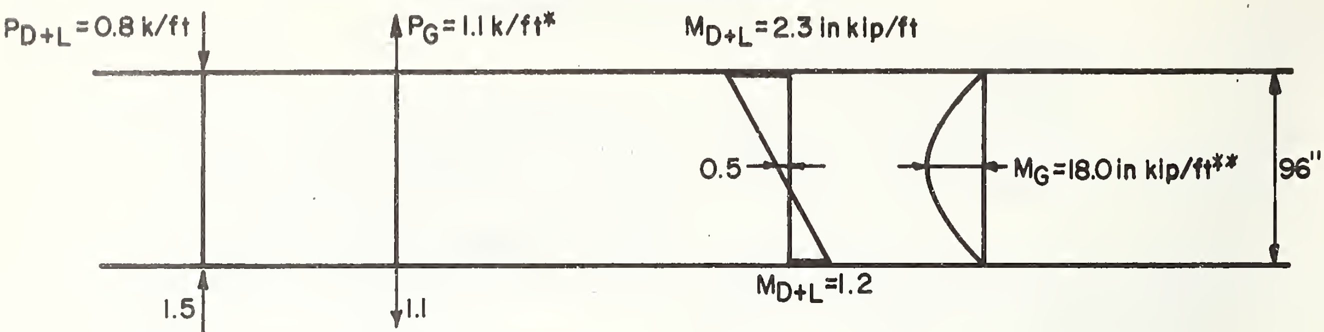


Figure 4.7 Wall Moments due to Eccentricity, Baker Building.

8th STORY - Try $p_m = 1.3$ psi   Assume pinned-pinned piers



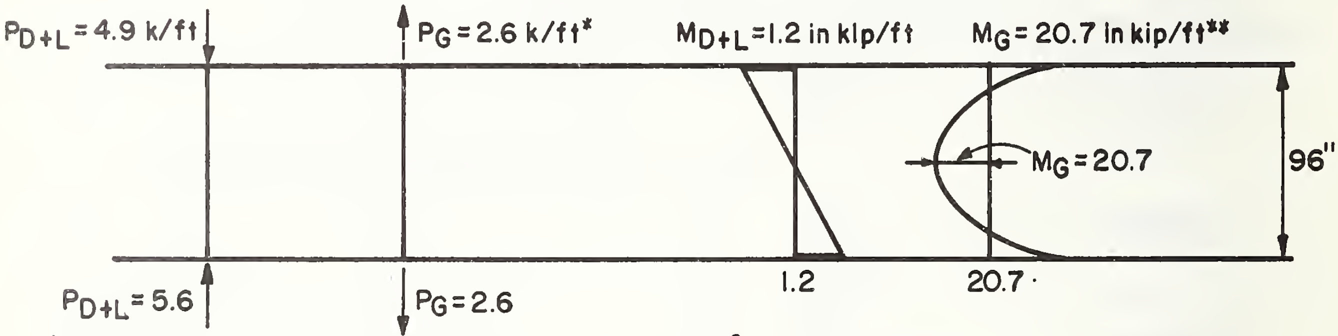
*$P_G = 0.882\ p_m = 1.1$ k/ft (from figure 4.6)

**$M_G = 1/8\ wh^2 = 1/8 \times 12 \times 1.3 \times (96)^2 = 18.0$ in kip/ft

$P_{D+L+G} = 0.4$ k/ft , $M_{D+L+G} = 18.5$ in kip/ft

From Eq. 2, $M_C = 0.5 + 18.0 = 18.5$ in kip/ft   use $\underline{p_m = 1.3 \text{ psi}}$

4th STORY - Try $p_m = 3.0$ psi   Assume equal end and midspan moments

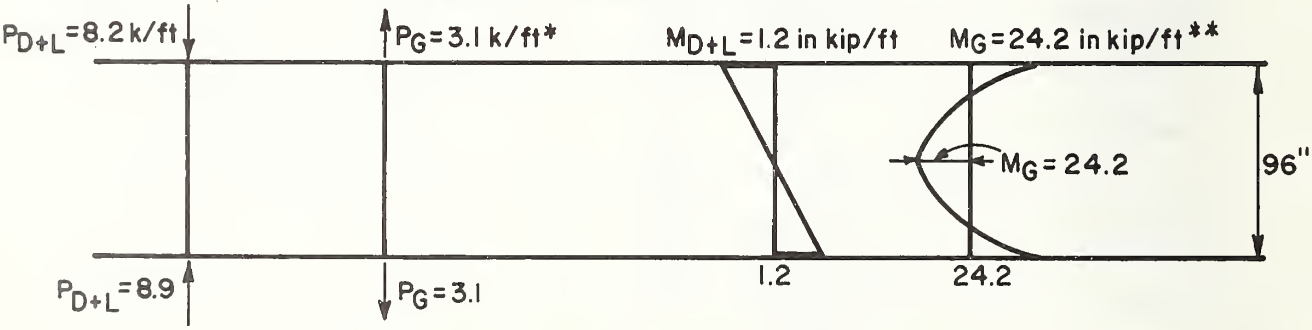


*$P_G = 0.882 \times 3.0 = 2.6$ k/ft   **$M_G = 1/16 \times 12 \times 3.0 \times (96)^2 = 20.7$ in kip/ft

$P_{D+L+G} = 3.0$ kip/ft $M_{D+L+G} = 21.9$ in kip/ft

From Eq. 2, $M_C = 18.0 + 4.0 = 22.0$ in kip/ft $\approx$ 21.9 in kip/ft   use $\underline{p_m = 3.0 \text{ psi}}$

1st STORY - Try $p_m = 3.5$ psi   Assume equal end and midspan moments

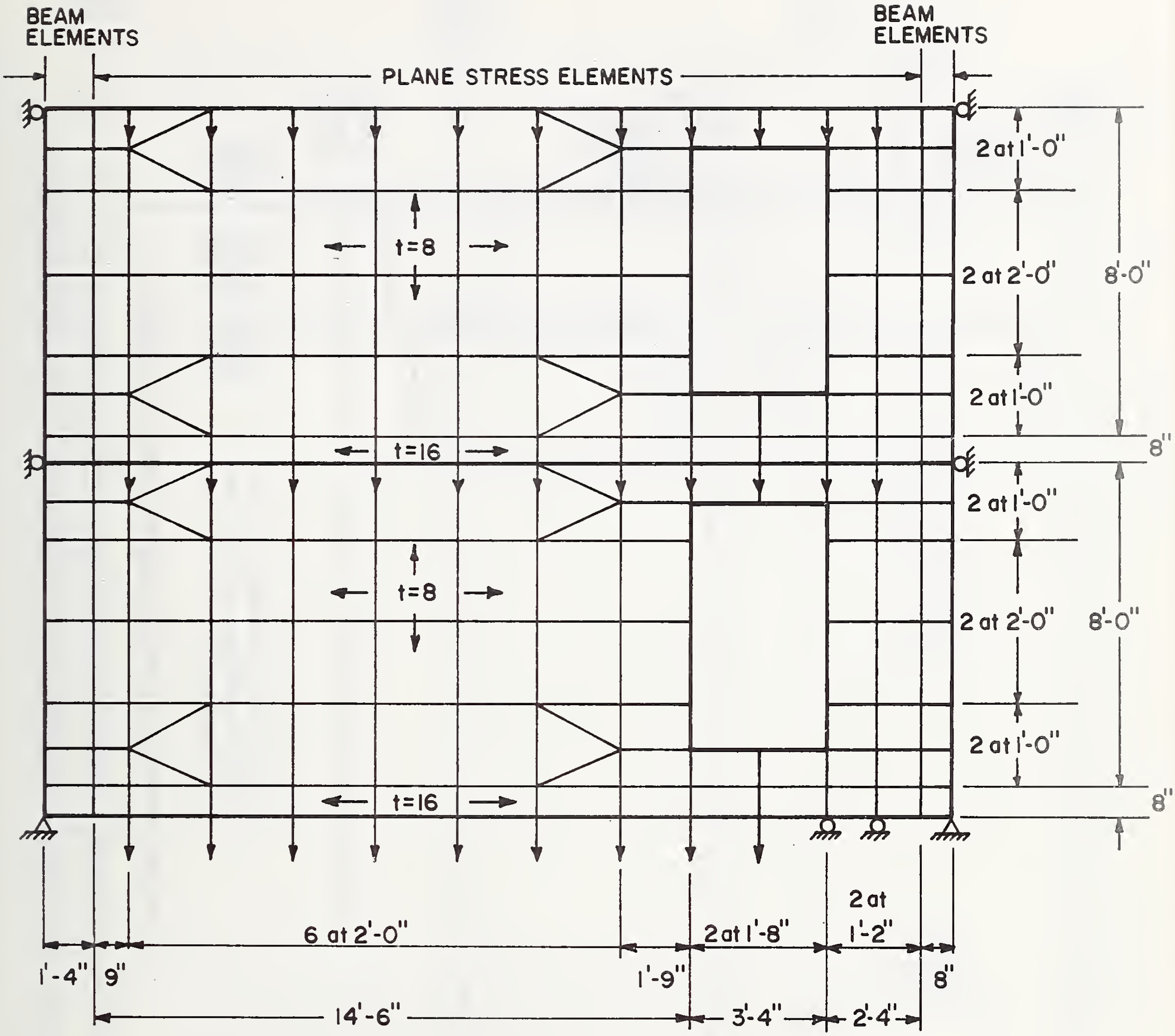


*$P_G = 0.882 \times 3.5 = 3.1$ k/ft   **$M_G = 1/16 \times 12 \times 3.5 \times (96)^2 = 24.2$ in kip/ft

$P_{D+L+G} = 5.8$ kip/ft $M_{D+L+G} = 25.4$ in kip/ft

From Eq. 2, $M_C = 18.0 + 7.7 = 25.9$ in kip/ft $\approx$ 25.4 in kip/ft   use $\underline{p_m = 3.5 \text{ psi}}$

Figure 4.8 Story Analysis, Baker Building.



Boundary Set A - Horizontal and vertical restraint at each floor

Boundary Set B — Horizontal restraint at each floor, vertical restraint at bottom only (as shown)

Boundary Set C - No horizontal restraint, vertical restraint at bottom only

Figure 4.9 Finite Element Idealization, Baker Building.

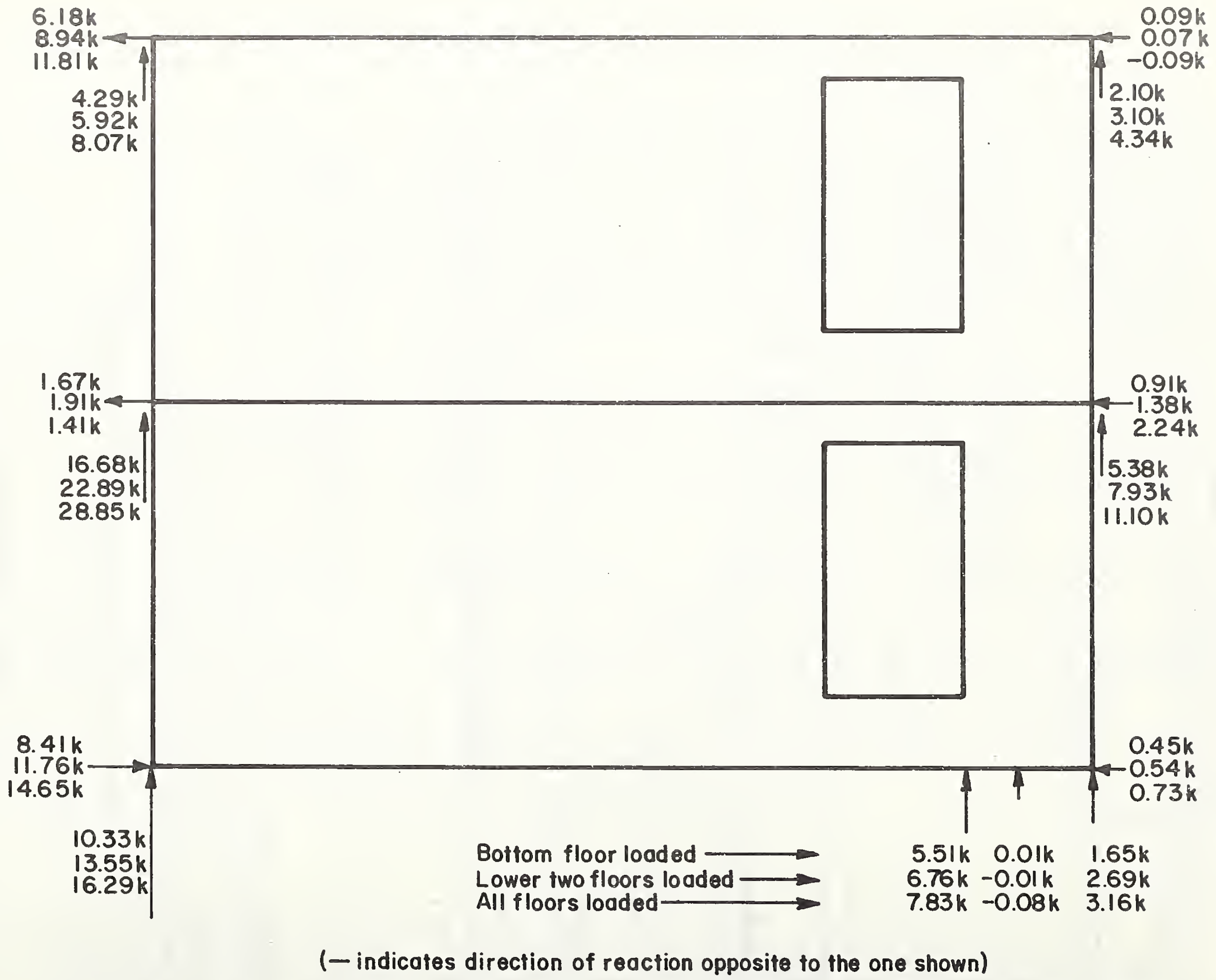


Figure 4.10a Wall Support Reactions for Boundary Set A, Baker Building.

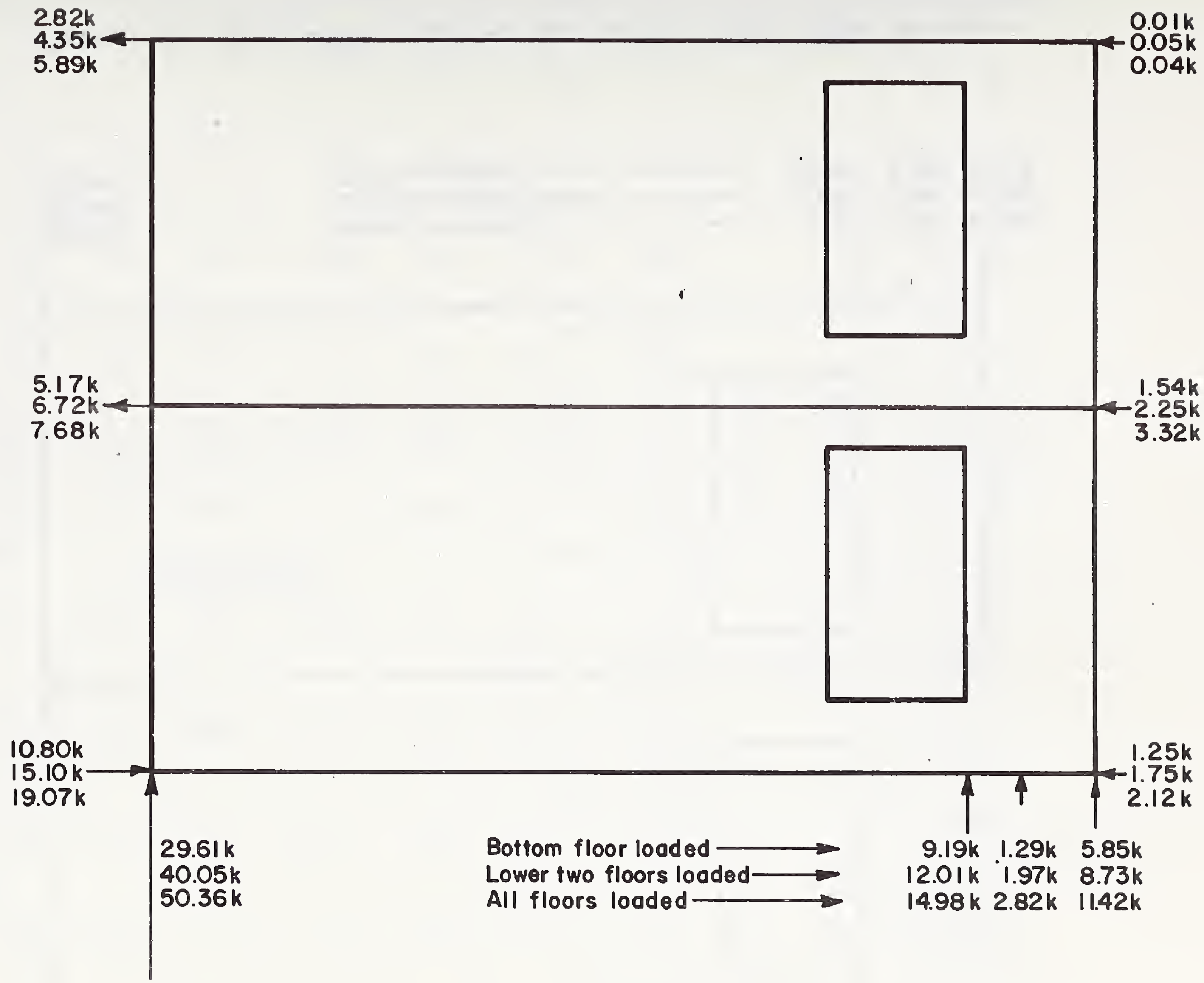


Figure 4.10b Wall Support Reactions for Boundary Set B, Baker Building.

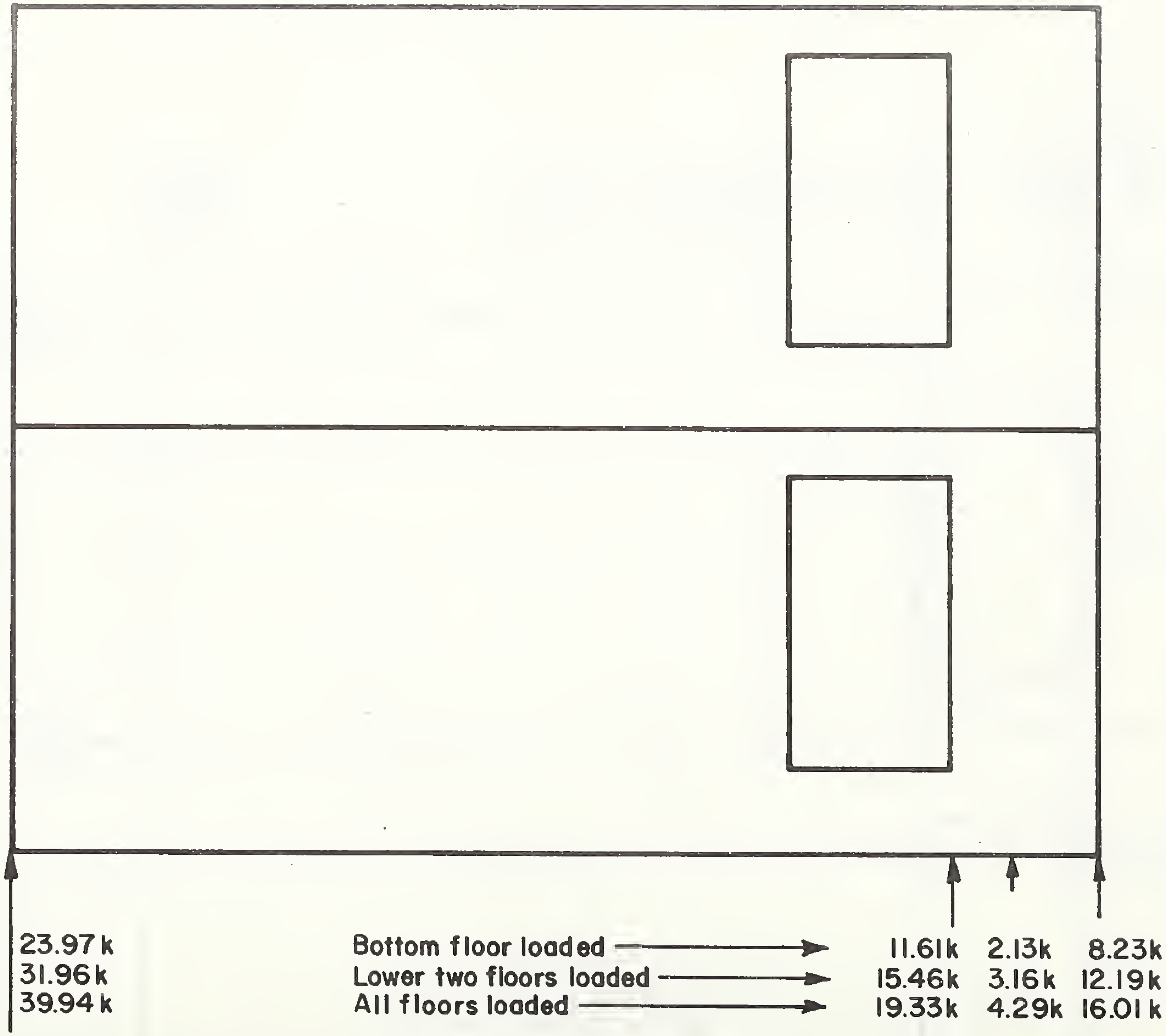


Figure 4.10c Wall Support Reactions for Boundary Set C, Baker Building.

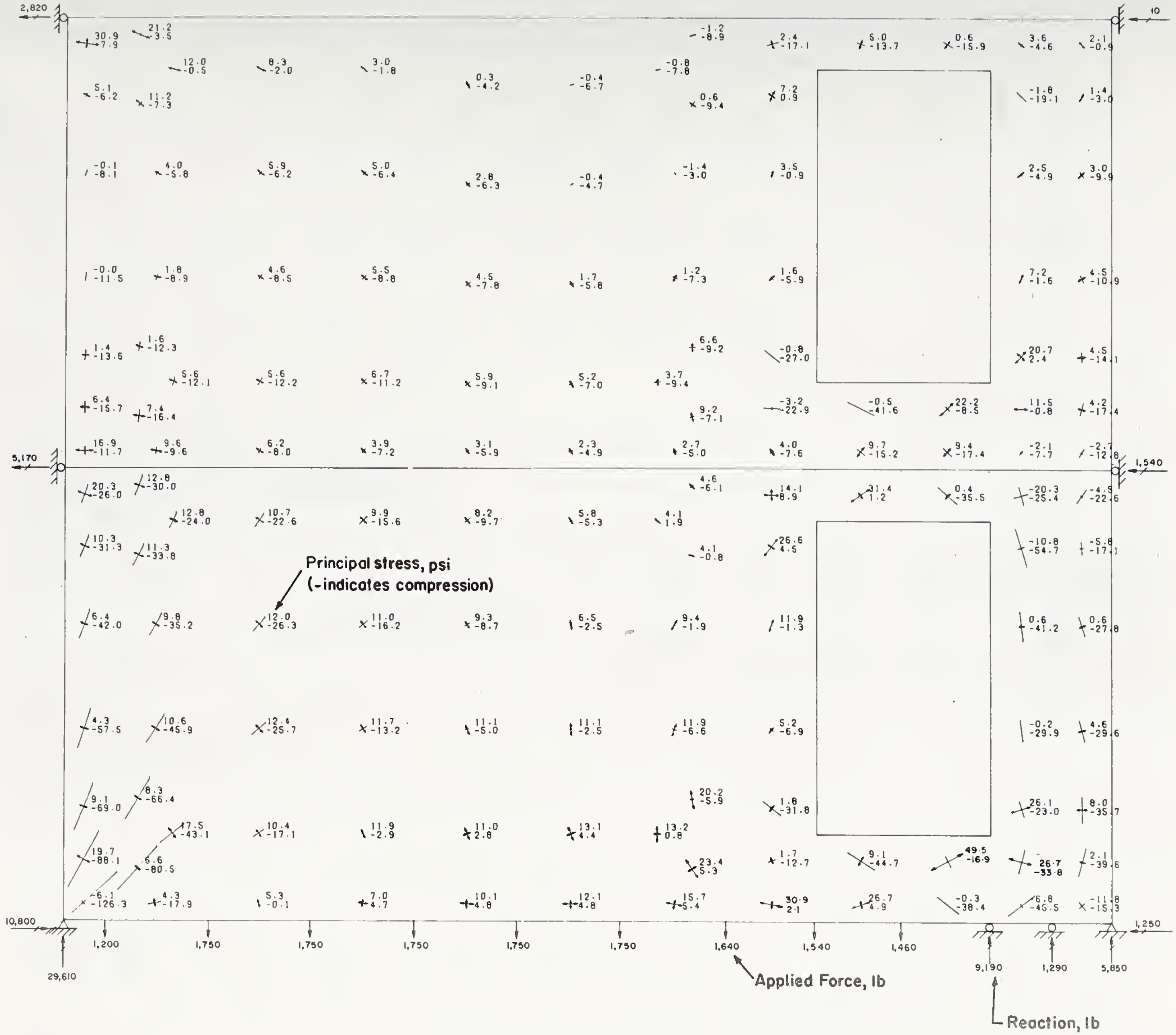


Figure 4.11 Wall Stress Distribution for Boundary Set B with Bottom Floor Loaded, Baker Building.

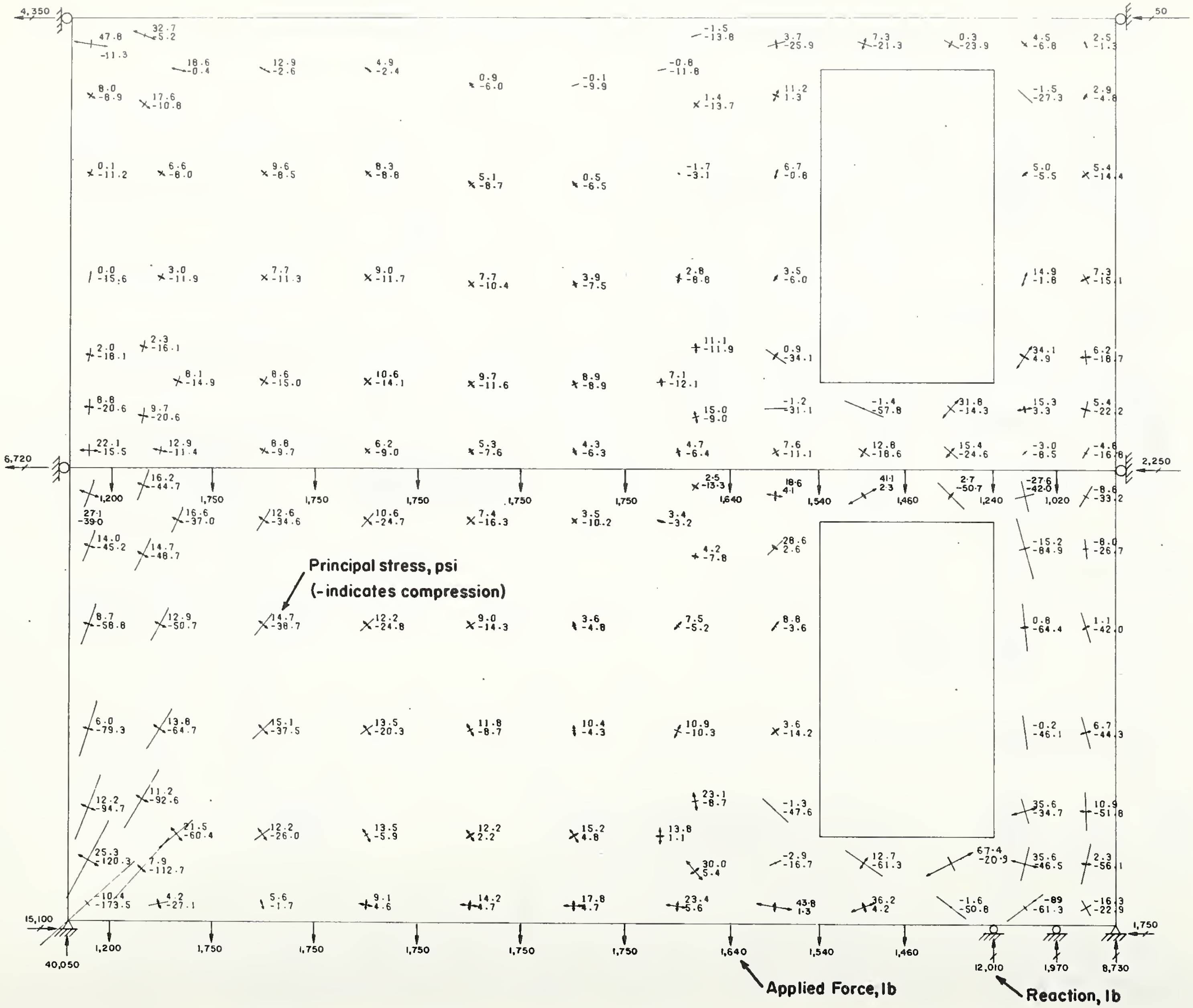


Figure 4.12 Wall Stress Distribution for Boundary Set B with Lower Two Floors Loaded, Baker Building.

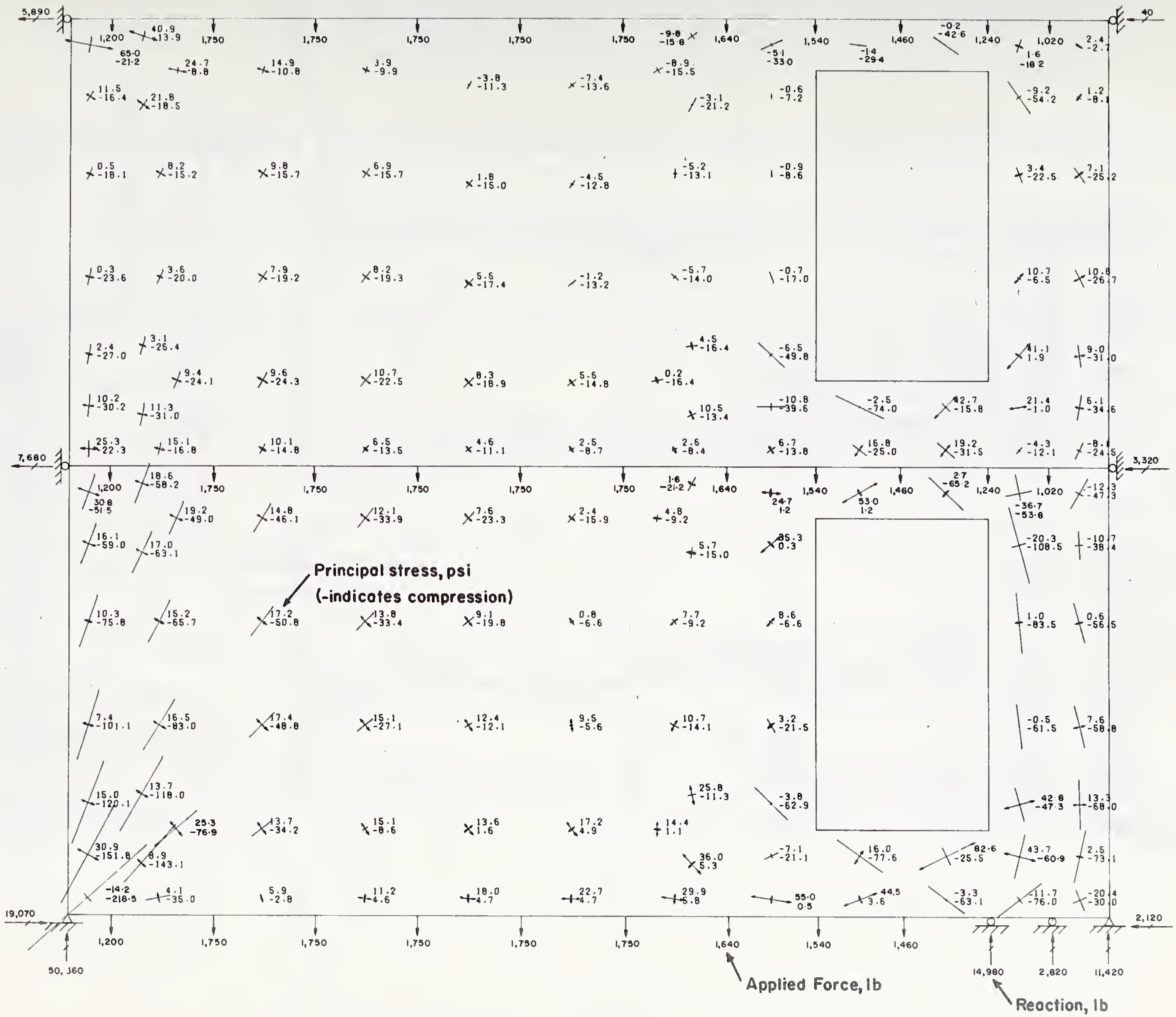


Figure 4.13 Wall Stress Distribution for Boundary Set B with All Floors Loaded, Baker Building.

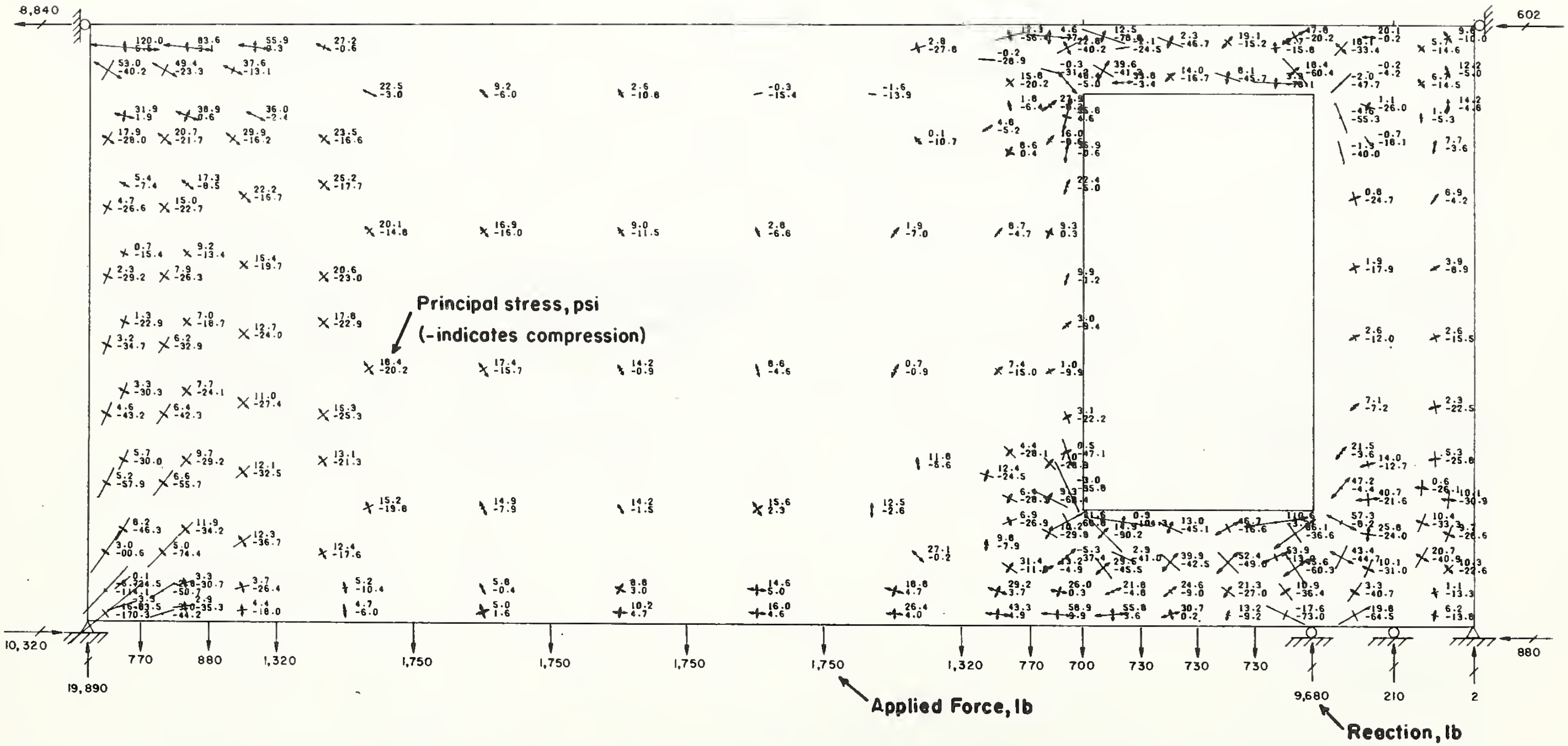


Figure 4.14 Wall Stress Distribution for Boundary Set B with Fine Mesh, Baker Building.

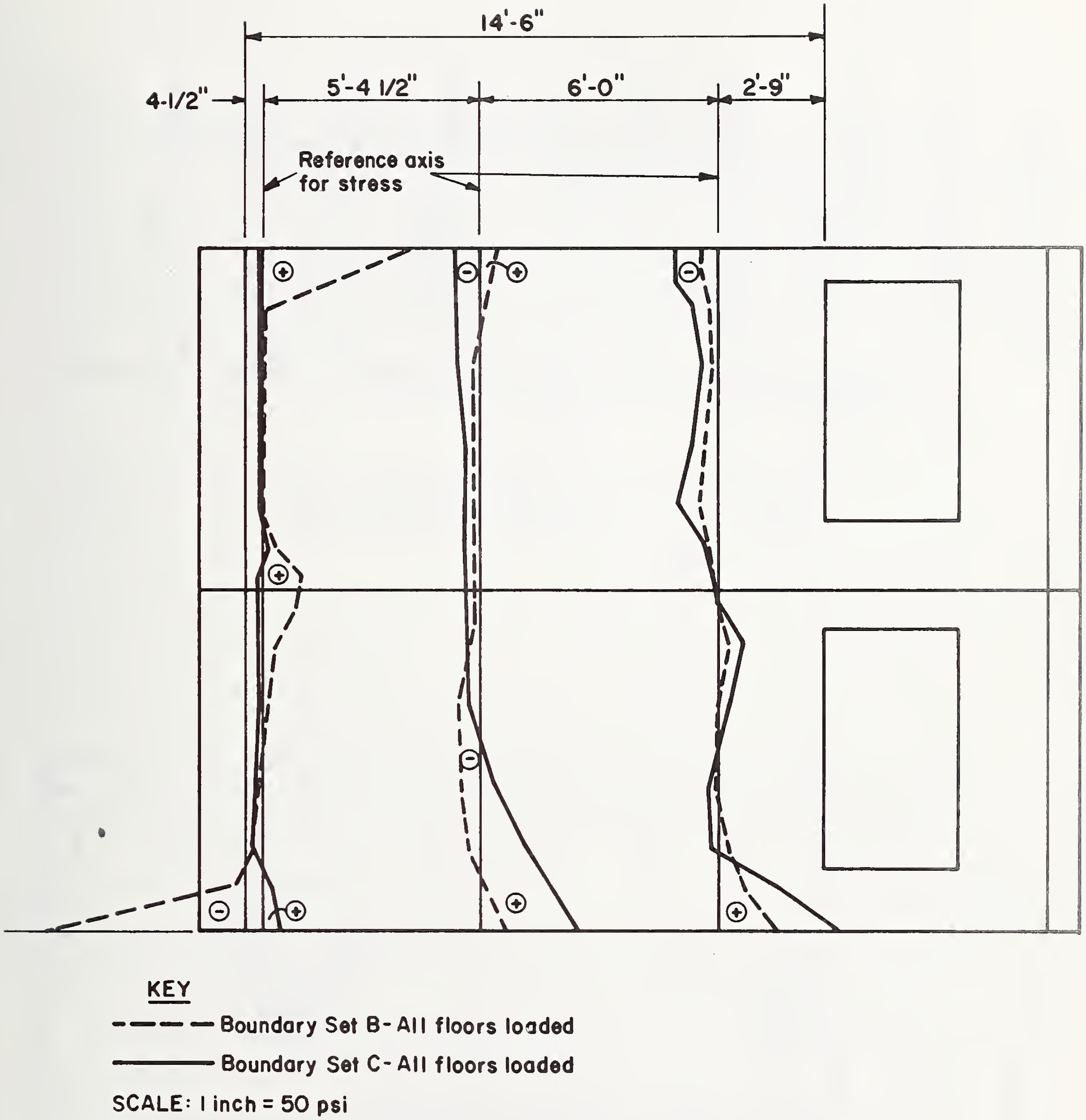


Figure 4.15 Horizontal Stress Distribution, Baker Building.

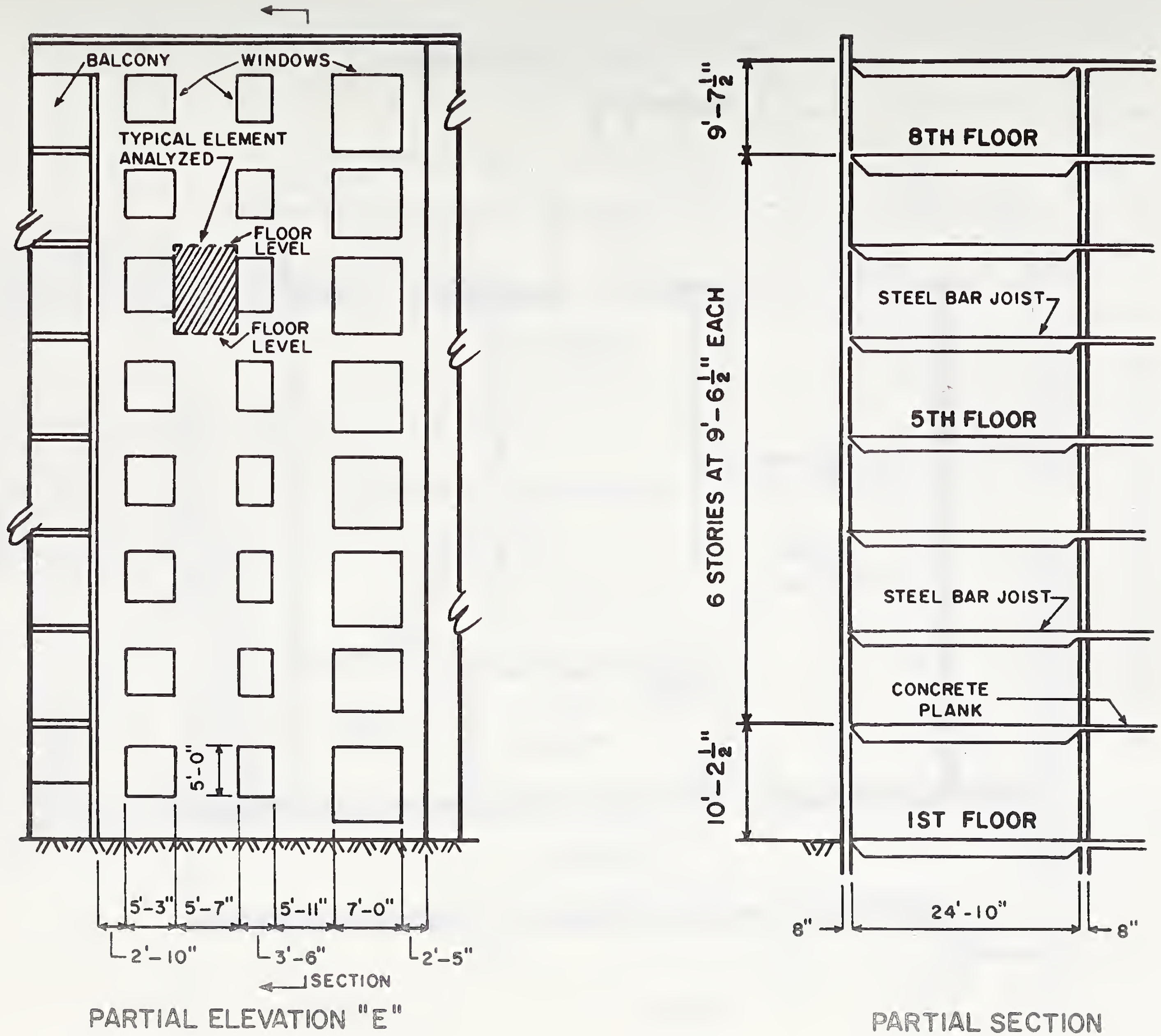


Figure 5.1 Partial Elevation and Section, Carter Building.

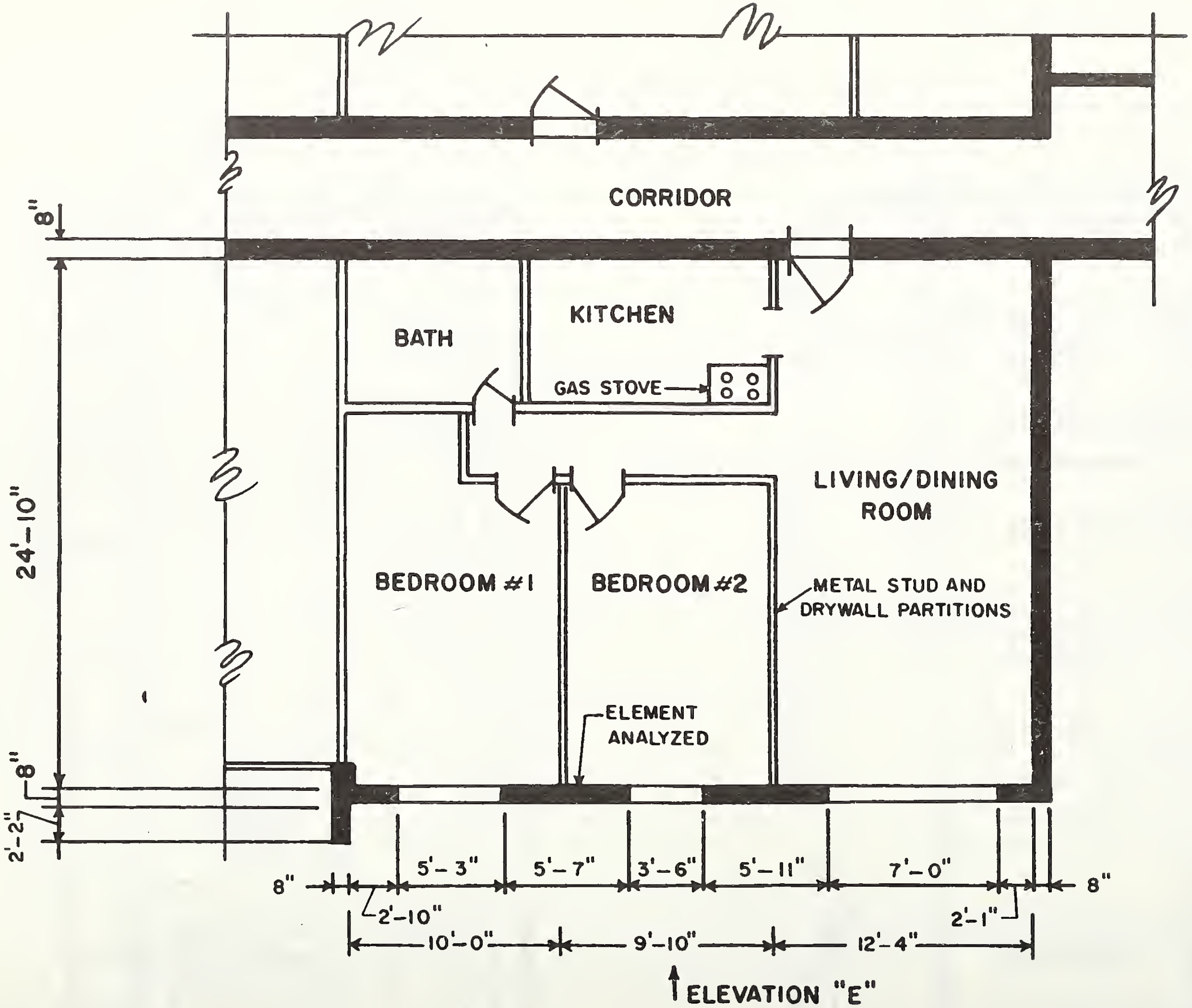
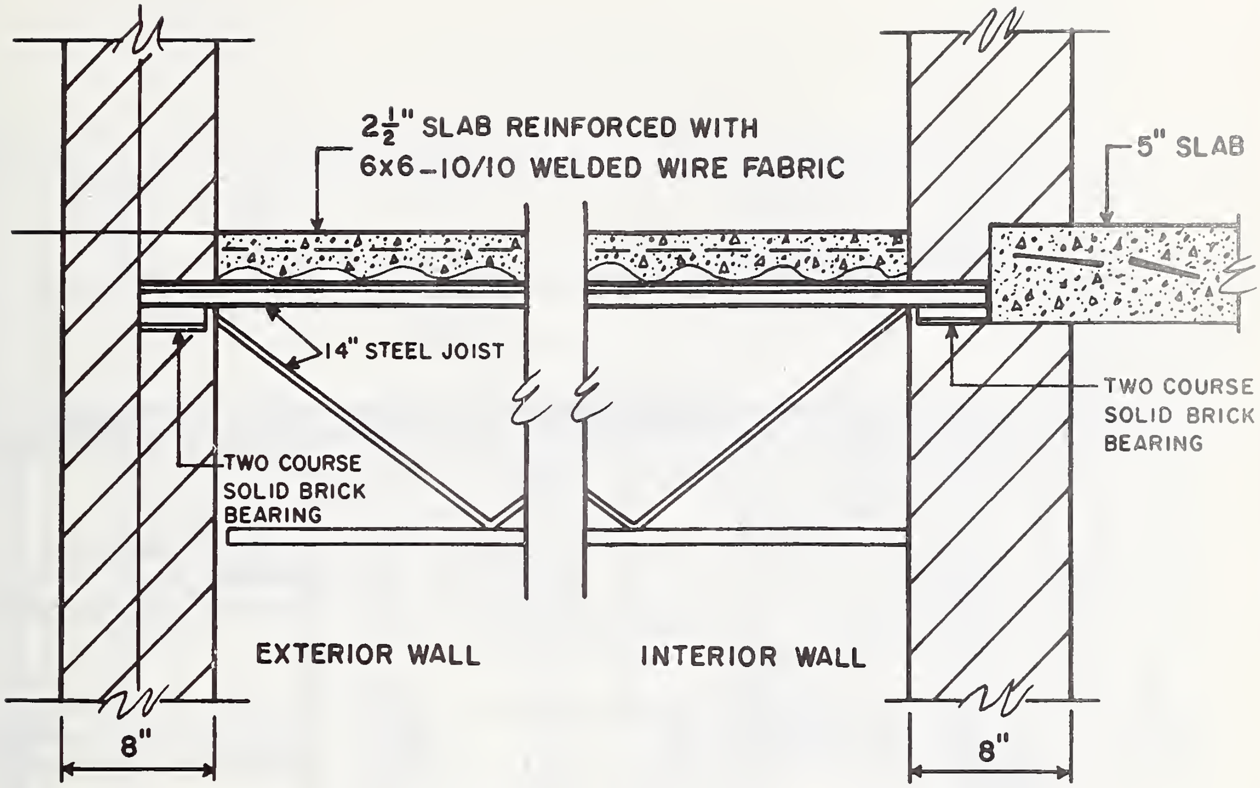


Figure 5.2 Apartment Plan, Carter Building.



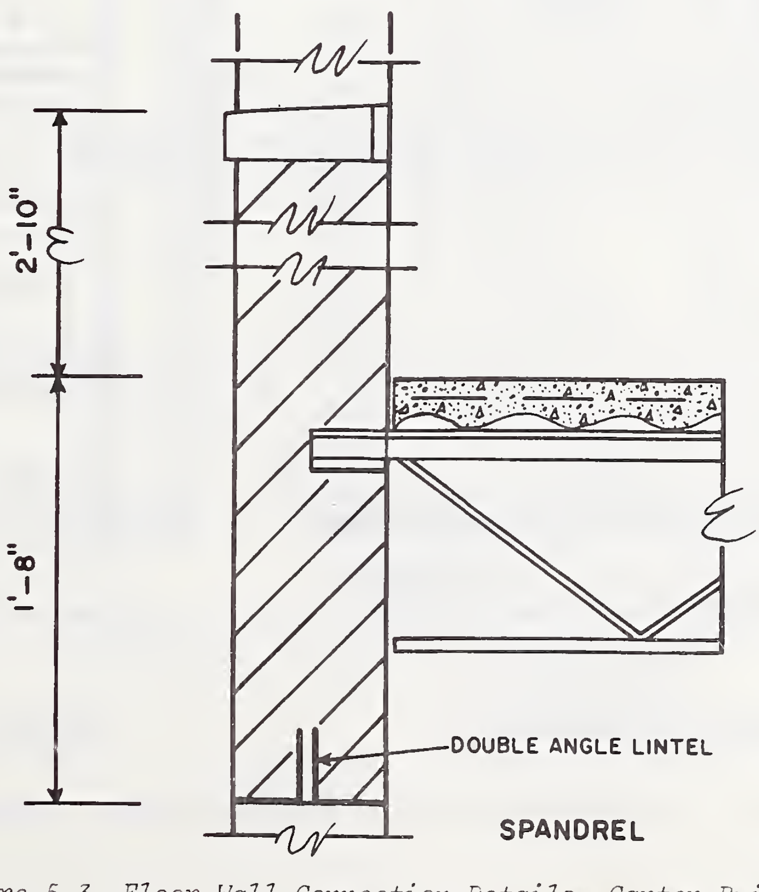


Figure 5.3 Floor-Wall Connection Details, Carter Building.

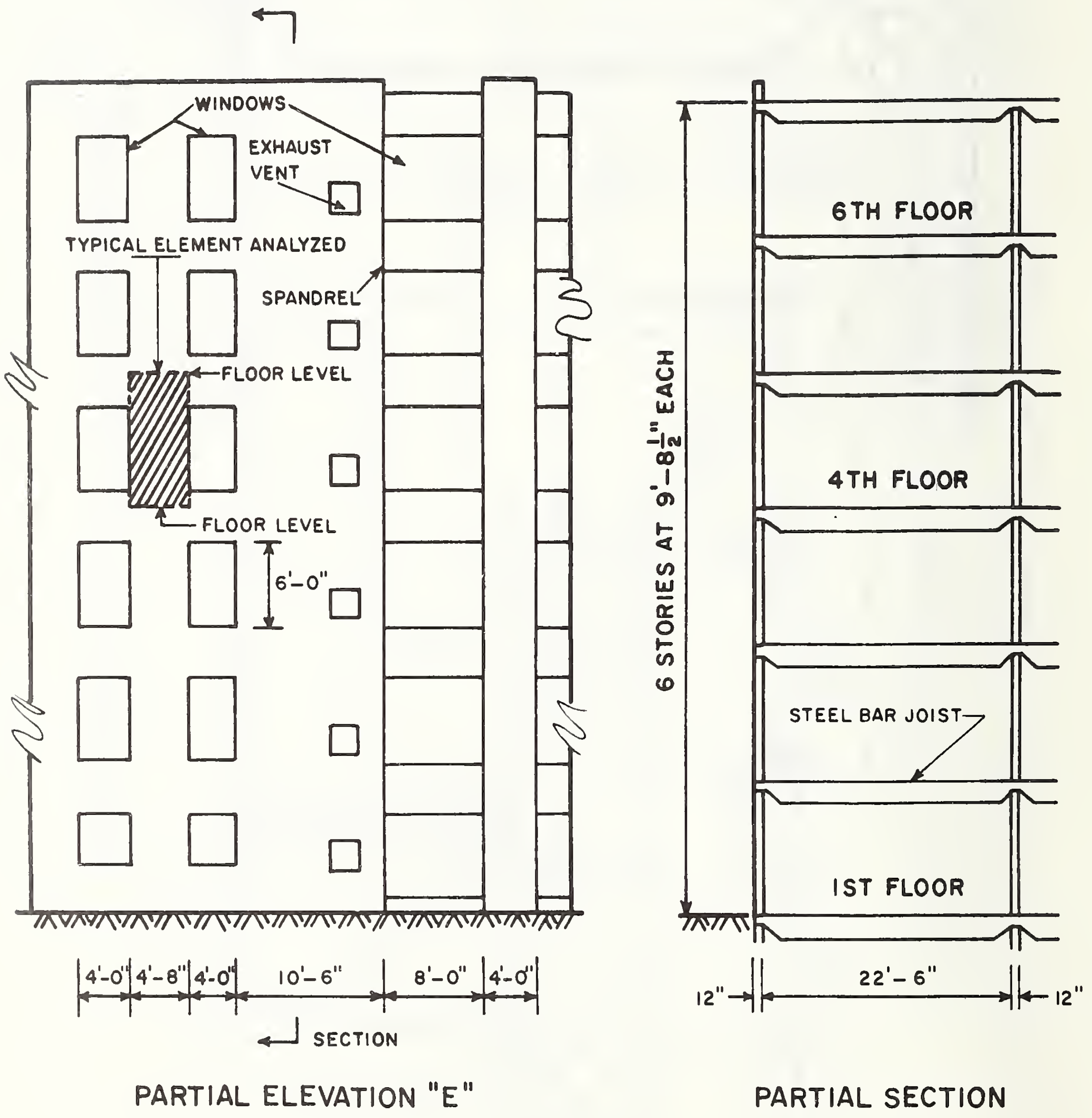


Figure 6.1 Partial Elevation and Section, Drake Building.

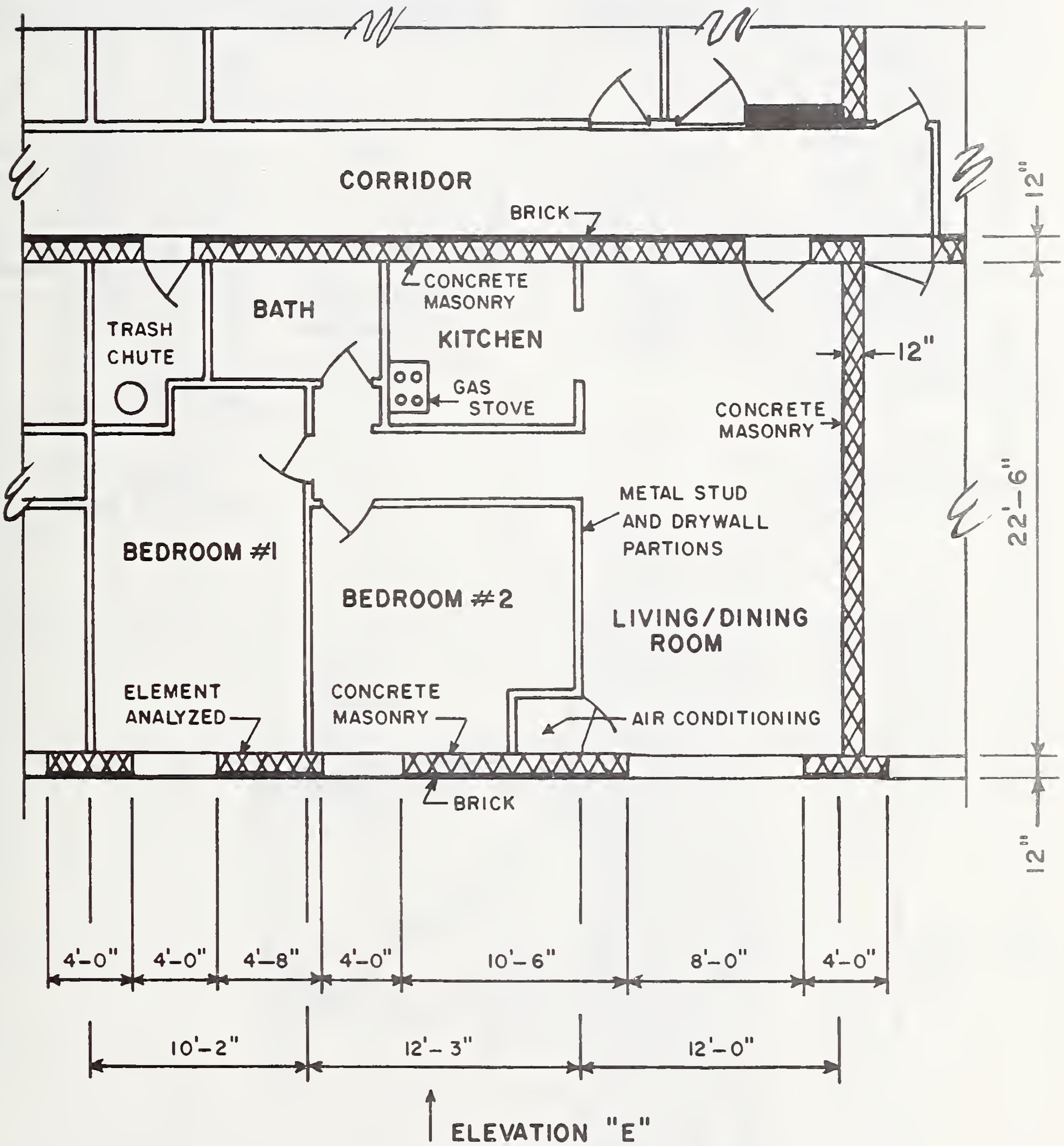


Figure 6.2 Apartment Plan, Drake Building.

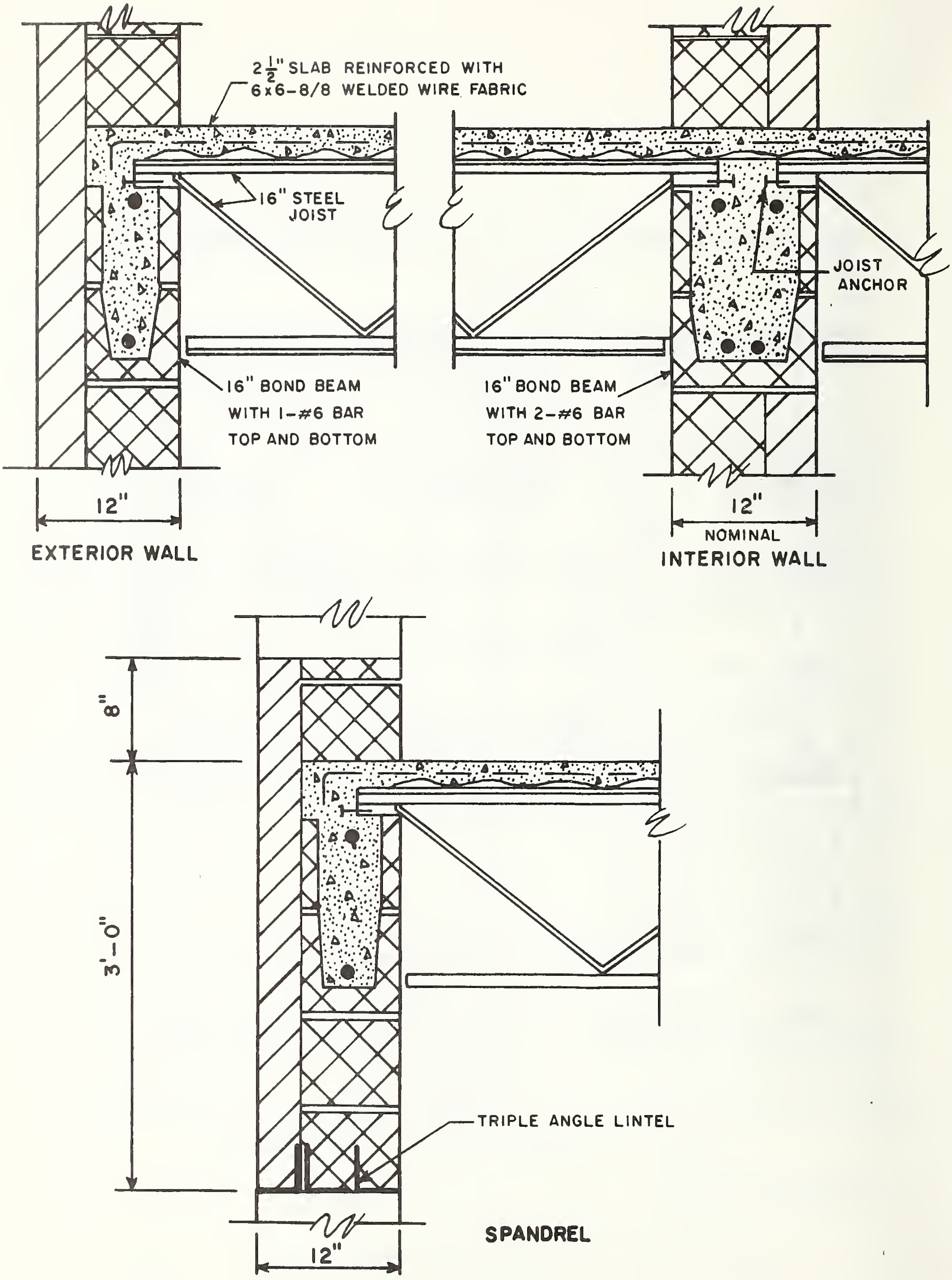


Figure 6.3 Floor-Wall Connection Details, Drake Building.

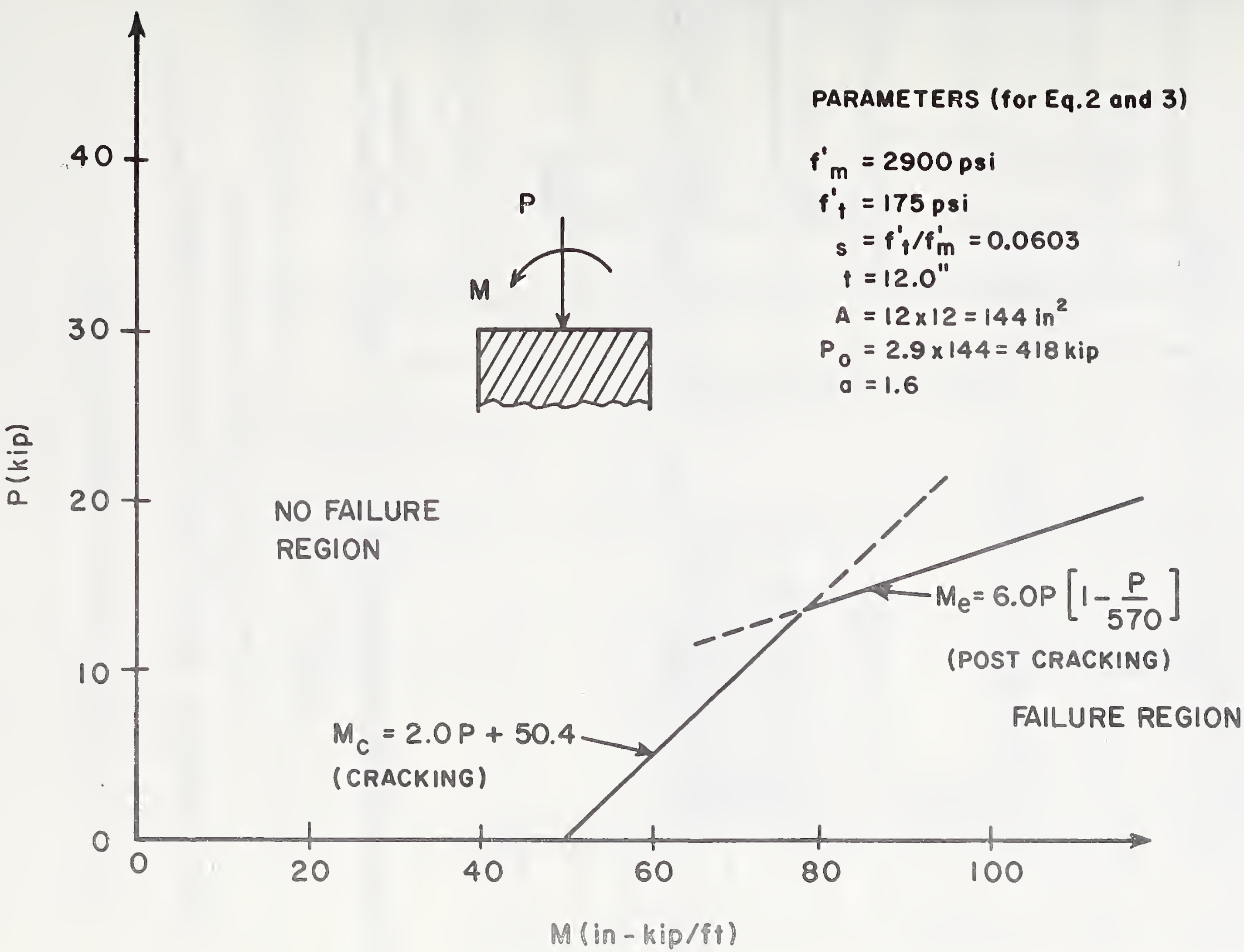


Figure 6.4 Interaction Diagram, Drake Building.

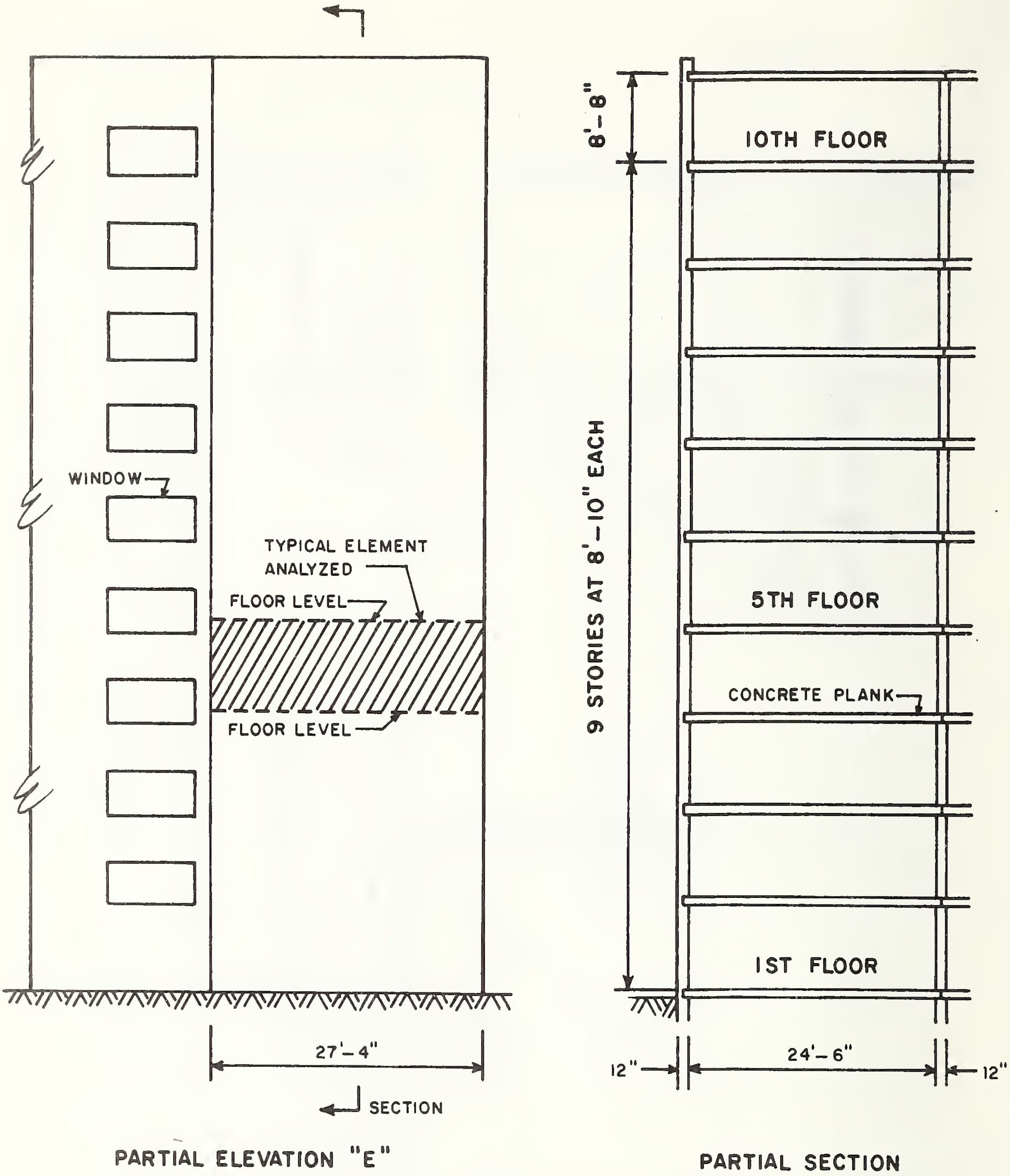
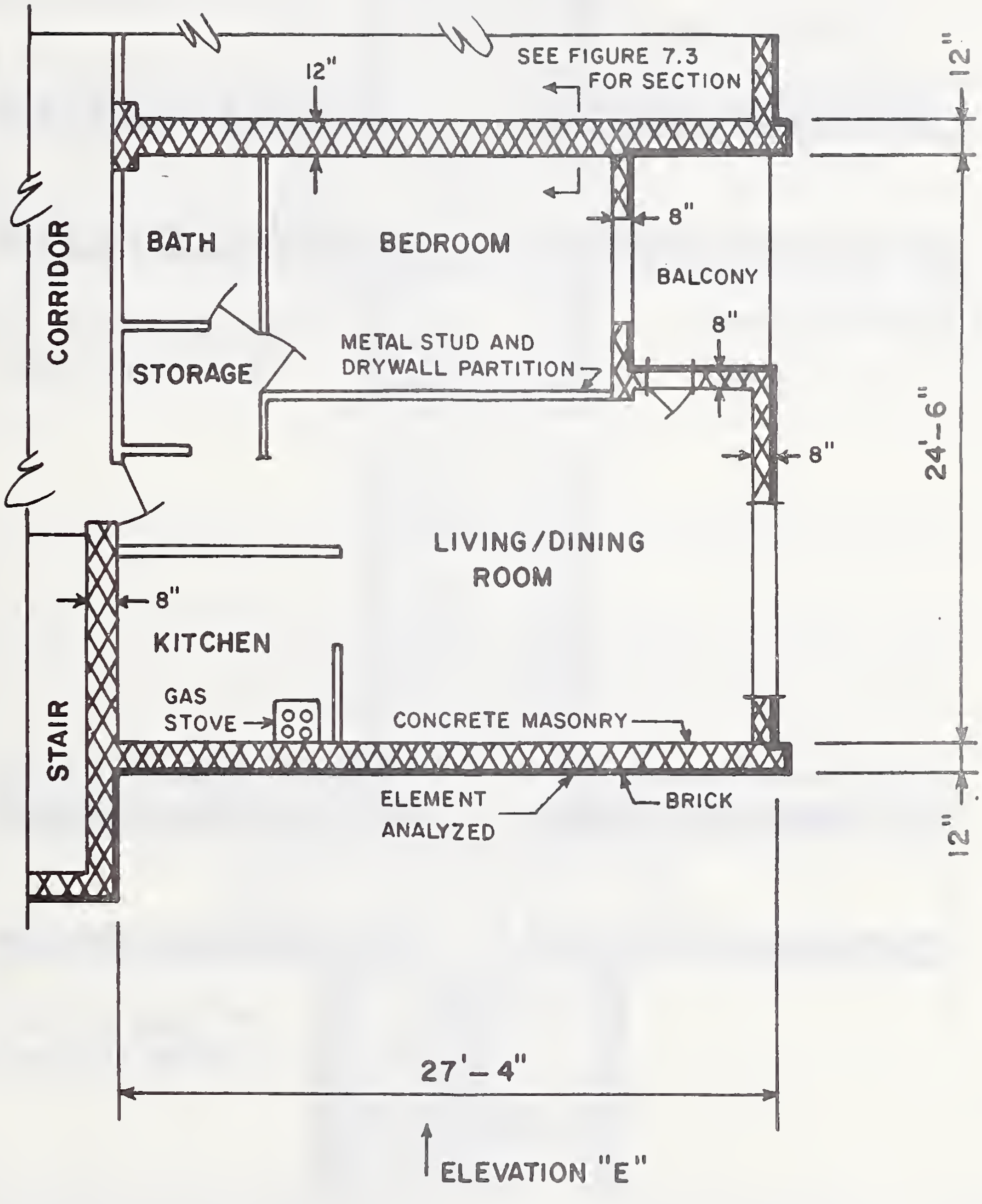


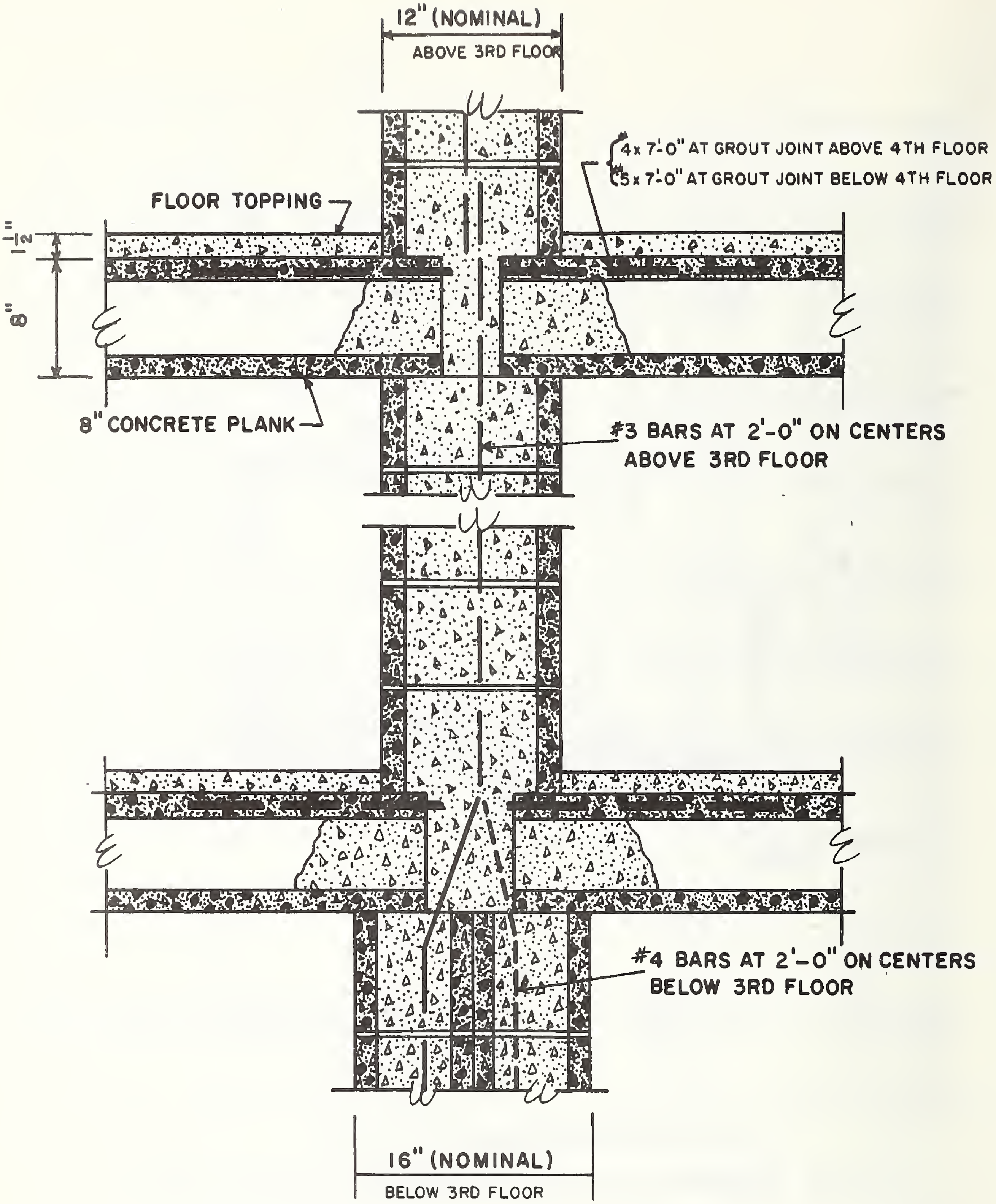
Figure 7.1 Partial Elevation and Section, Edwards Building.



12" WALLS ARE BEARING WALLS

8" WALLS ARE NON-BEARING WALLS

Figure 7.2 Apartment Plan, Edwards Building.



(SEE FIGURE 7.2 FOR LOCATION)

Figure 7.3 Typical Bearing Wall Detail, Edwards Building.

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15. SUPPLEMENTARY NOTES

16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

Five case studies of susceptibility to progressive collapse were made of non-reinforced masonry bearing wall buildings. All were assumed to comply with governing building codes. Based on the assumed failure mechanisms, analysis indicated that two of the structures had excellent resistance to progressive collapse, one was marginal, and two had little resistance to progressive collapse. Analytical approaches used are illustrated and areas of needed research are identified.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)* Abnormal loading; Building; Gas explosion; Load-bearing masonry; Load-bearing walls; Masonry; Masonry research; Progressive collapse.

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