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Structural Assessment of the New U.S. Embassy Office Building in Moscow

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ABSTRACT

Public Law 99-591, The Continuing Appropriations Act for Fiscal Year 1987, directed the National Bureau of Standards (NBS) to conduct an independent analysis of the new United States Embassy Office Building being constructed in Moscow. The analysis was to include: "...an assessment of the current structure and recommendations and cost estimates for correcting any structural flaws and construction defects...." This report describes the investigation, which included field, laboratory and analytical studies, and its findings. The investigation did not address security and other nonstructural deficiencies. The investigation has identified important structural defects in the building and defined remedial measures to correct them. While important, these structural defects, in comparison to the total structural system for the building, are modest in scale and fully correctable.

KEYWORDS: Building; concrete; construction; Embassy; investigation; masonry; Moscow; progressive collapse; steel; structure.

EXECUTIVE SUMMARY

Public Law 99-591, The Continuing Appropriations Act for Fiscal Year 1987, directed the National Bureau of Standards (NBS) to conduct an independent analysis of the new United States Embassy Office Building being constructed in Moscow. The analysis was to include: "...an assessment of the current structure and recommendations and cost estimates for correcting any structural flaws and construction defects...." This report is submitted in response to the assignment.

NBS has analyzed the structural system of the Office Building and developed recommendations and cost estimates for correcting structural flaws and defects. The scope of the investigation was limited to the structural system, and was not concerned with defects that are neither structural nor threatening to the structural integrity of the building. Activities included review of the documentation for the design and construction of the site and building, formulation of criteria for the assessment, analysis of the structure as designed, field and laboratory investigations of the structure, analysis of the as-built structure, and development of required remedial measures. The structural integrity of the Office Building was assessed for compliance with good practice for important U.S. office buildings.

Structural materials and components used in the Office Building are generally of good quality. However, important deficiencies exist in the structure that must be corrected for adequate safety before the building is occupied. These include:

- Inspecting all of the joints between reinforced concrete columns and filling those found to be incomplete.

- Bracing four steel-core columns to provide adequate resistance to buckling.

- Inspecting and completing all joints between shear wall panels and adjacent panels or columns to provide adequate strength and stiffness for resistance to lateral forces.

- Attaching a system of steel straps to the top flanges of long-span beams on floors two through eight to protect against progressive collapse of the floor system.

- Filling gaps between masonry partitions in the core area and the surrounding beams and columns, and strengthening the partitions to provide an alternate load path in the event of a column failure.

- Installing shear connections between brick masonry and concrete partition walls, and strengthening a box beam connection.

- Removing and replacing cracked portions of parapet walls, and anchoring the parapet walls adequately to the structure below.

The total estimated cost based on Washington, D.C., prices for conducting these remedial structural measures is $1,490,000.

In addition, the following remedial measures are recommended for the serviceability and durability of the Office Building structure:

- Removing and replacing cracked portions of the penthouse walls.

- Providing vertical expansion joints in the corner piers of the exterior walls.

- Appropriately placing insulation in the corner piers and in cavities above windows.

- Carrying out a program to monitor the development of cracks present in the exterior walls, and to define remedial measures if needed.

The total estimated cost based on Washington, D.C., prices for conducting these additional remedial structural measures is $341,000.

Actual costs of the remedial structural measures will depend upon working conditions in Moscow and the means selected for performing the work. These costs do not include the costs of correcting nonstructural deficiencies in the Office Building. These costs do not include the costs for addressing security concerns for the Office Building.

The remedial structural measures do not involve major reconstruction and could be completed in less than a year if the Office Building were located in the United States.

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CHAPTER 1
INTRODUCTION

1.1 BACKGROUND

Public Law 99-591, The Continuing Appropriations Act for Fiscal Year 1987, directed the National Bureau of Standards (NBS) to conduct an independent analysis of the new United States Embassy Office Building being constructed in Moscow. The analysis was to include: "...an assessment of the current structure and recommendations and cost estimates for correcting any structural flaws and construction defects...." This report describes the investigation and presents its findings. It complements the summary report submitted to Congress [1.1].

Background information on the new United States Embassy Office Building in Moscow, hereafter called the Office Building, was provided by the Office of Foreign Buildings Operations (FBO) of the Department of State. The Office Building is located in the new Embassy complex on Konyushkovskaya Street one city block west of the present Embassy office building on Chaykovskogo Street. The new site was transferred to the United States under the terms of an 85-year lease as defined in the Embassy Sites Agreement between the United States and the Soviet Union signed on May 16, 1969. An agreement on December 4, 1972, established the conditions of construction for the Office Building.

The Office Building was designed from 1973 to 1976 by a combined partnership of two United States firms: Skidmore, Owings and Merrill of San Francisco and Gruzen and Partners of New York. This design established the form, appearance, loadings, structural system, and materials for the structure. The construction contract was signed on June 30, 1979, with the Soviet General Contractor, Sojuzvneshstrojimport. The Soviets were responsible for the detailed structural design and construction using a Soviet building system widely used in Moscow. Construction activities at the site began in 1979. The structural framing was in place in June 1982. The exterior walls were substantially complete in November 1983 when installation of the facing brick was finished. Construction work has been suspended since August 1985 except for placement of a temporary roof in November 1986. A heating system is operating in the Office Building.

1.2 SCOPE AND ORGANIZATION

The scope of the investigation was limited to the structural system of the Office Building. The investigation did not consider other systems such as heating or plumbing, nor other buildings at the new Embassy site. Also, the investigation did not consider construction defects that are neither structural nor threatening to structural integrity. For instance, incomplete and defective concrete vault walls on the eighth floor of the Office Building have concerned many official visitors. However, these vault walls do not have a structural function, nor do their deficiencies handicap the performance or durability of the structure. Therefore, they are not considered further in this report. In contrast, deficiencies in the facade masonry can be hazardous (if bricks should fall) or threaten durability (entrance of moisture can cause corrosion of structural members). These types of defects are considered in this report. In addition, nonstructural masonry partitions are assessed for their potential to provide alternate load paths in the event of failure of individual structural members.

Metric units are used in this report for consistency with the units used in the original plans, specifications and design documentation.

Chapter 2, SITE AND BUILDING DESCRIPTION, describes the Office Building site and the structural system.

Chapter 3, LOADING AND RESISTANCE CRITERIA, reviews the loading and resistance criteria for the Office Building and defines the criteria for the assessment.

Chapter 4, ASSESSMENT OF DESIGN, assesses the design of the Office Building in light of the criteria defined in Chapter 3.

Chapter 5, SITE AND LABORATORY INVESTIGATIONS, describes the site investigations and laboratory studies conducted to determine the as-built condition of the structural system. Field and laboratory data are summarized.

Chapter 6, ASSESSMENT OF THE EXISTING STRUCTURE AND RECOMMENDATIONS FOR REMEDIAL MEASURES, provides the assessment of the as-built structural system and recommendations for remedial measures needed to provide safety, serviceability and durability consistent with good U.S. practice for office buildings.

1.3 BASIS FOR ASSESSMENT OF STRUCTURAL INTEGRITY

The structural integrity of the Office Building was assessed in terms of good practice for U.S. office buildings. Loadings used for review were consistent with U.S. requirements for an important public building and the siting of the office building in Moscow. Resistances of structural materials and components to these loadings were evaluated in light of experiences with U.S. practices, knowledge of Soviet materials and components, and laboratory tests and field measurements. Remedial measures are recommended where members are overloaded in comparison to good practice for U.S. office buildings. The goal of the recommended remedial measures is for occupants of the Office Building to be as safe from structural hazards as they would be in a well-designed office building in the United States.

In addition to recommending remedial measures for instances in which the level of structural safety falls below the minimum for good practice in the United States, remedial measures also are recommended to improve the serviceability and durability of the structure.

1.4 ACKNOWLEDGEMENTS

Dr. Alexander Rosenbaum, of the firm Parsons, Brinckerhoff, Quade, and Douglas, Inc., made significant contributions to the investigation. He provided valuable insight into the Soviet design and construction practices, assisted in the interpretation of Soviet drawings and specifications, and performed a diligent review of various aspects of the Soviet design of the Office Building.

Dr. John W. Lyons, Director, and Mr. Samuel Kramer, Deputy Director, of the National Engineering Laboratory of the National Bureau of Standards made substantial individual contributions to the investigation in arranging for resources and clearances, in reviewing this report, and in coordinating this study with the many concerned agencies and Congressional committees. Dr. Charles G. Culver, Chief, Structures Division, and Dr. H.S. Lew, Leader, Structural Evaluation Group provided valued technical consultation and review.

Ms. Karen Perry provided administrative services vital to procurement and fiscal control for the project. Ms. Nancy Fleegle and Mrs. Carolyn Flood provided essential clerical support. Mr. Keith Mackley and Mr. Ray Mele provided skilled and timely support for the development of illustrations and assembly of the report. Mr. Lloyd N. Riddick III prepared drawings of the masonry details. Mr. James Little, Mr. Frank Rankin, Mr. Erik Anderson, Mr. Frankie Davis, and, Mr. Herbert Wechsler assisted in performing laboratory tests of steel, concrete, and masonry materials. Ms. Geraldine Cheok assisted in the analysis of concrete columns, and Dr. Leslie Struble performed analyses of field mortar samples. Dr. Richard Fields, Mr. T. Robert Shives, Mr. Charles H. Brady, and Dr. Tom Siewert of the NBS Institute for Materials Science and Engineering and Dr. Harry Rook and his staff of the NBS National Measurement Laboratory provided assessments of materials for the investigation. Dr. Roy Armstrong assisted in the foundation analysis.

1.5 REFERENCE

1.1 "Report to Congress on the Structural Assessment of the New U.S. Office Building in Moscow," National Bureau of Standards, NBSIR 87-3636, April 1987, 31 pp.

CHAPTER 2
DESCRIPTION OF SITE AND BUILDING

2.1 INTRODUCTION

A description of the Office Building and the site upon which it is located is presented in this chapter. First, the geology of the site and the subsurface conditions below the Office Building are discussed. Next, descriptions of the building's foundation, its structural system (columns, beams, floor slabs, shear walls, and connections), and its envelope (roof, exterior walls, and windows) are presented. Lastly a chronology of the Office Building from the time of the engagement of design services in June 1968 to the time when construction ceased in August 1985 is presented.

2.2 SITE DESCRIPTION

Descriptions of the site and the subsurface conditions are based on the Soviet site exploration report issued in February 1975.

2.2.1 Geology

The Embassy site is located in the Krasnopresnensk District of Moscow. Ground elevations at the site vary from 143 to 136 m above mean sea level, and before excavation the site generally sloped in the southwesterly direction toward the Moscow River. The site was explored by 15 drill holes carried to depths of 15 to 34 m and by eight vibratory probes, 11 to 17.5 m deep. Forty-five disturbed and 89 undisturbed soil samples, and 16 ground water samples were taken for FBO by the Soviets.

The site is located on the ancient alluvial terrace of the Moscow River which is intersected at the southwestern portion of the site by the alluvial terrace of the Presna River. Unconsolidated deposits (before excavation) consisted of a 2- to 6-m thick layer of fill consisting of sands, sandy silts, and in some instances sandy silts mixed with construction debris. Below this fill are Quaternary deposits consisting of irregular and sometimes interbedded layers of ancient alluvial and fluvioglacial sands with intervening silt, sandy silt, and clay layers and morainal silts. On part of the site these deposits are underlain by Upper Jurassic clays (with some silt lenses). The thickness of the alluvial deposits ranges from 1 to 4 m, that of the fluvioglacial deposits from 0.5 to 9 m, and that of the morainal deposits from 1 to 4 m. The Jurassic clay stratum ranges in thickness from 1 to 12 m and predominates in the northern part of the site where its upper surface lies directly below the fill.

The unconsolidated deposits are underlain by rock formations of Upper Carboniferous origin whose upper surface is at elevations ranging from 124 to 129 m above mean sea level. Three beds were identified in these deposits. The first was an "upper argillaceous marl" bed in which some residual deposits, up to 4 m thick, were encountered. These deposits were identified as stiff lilac-cinnamon clays with some layers of marl. The second bed is highly eroded limestone whose upper part is reduced to rubble. This bed is approximately 10 m thick. The third bed is a "middle argillaceous marl" layer which is similar to the upper argillaceous marl but is of older origin and underlies the limestone layer.

Two aquifer horizons were encountered in the borings: an upper aquifer horizon which is generally perched on the Jurassic clay layer, but in some instances is shallower and perched on morainal deposits; and a lower ground water horizon which is perched on the middle argillaceous marl bed and which saturates part of the limestone bed. The surface of the upper ground water horizon was encountered at depths ranging from 1.2 to 8.5 m below the original ground surface. Since the Jurassic clay layer is eroded toward the southwest corner of the site, the ground water surface slopes toward the southwesterly direction. This ground water layer is absent in areas which are not underlain by clays.

The Soviets reported that the upper aquifer horizon has a high sulfate content. At the location of the Office Building sulfate contents of approximately 300 mg/L were recorded.

2.2.2 Subsurface Conditions

The subsurface information for the Office Building is derived from four borings located near the corners of the building and carried to depths ranging from 24 to 34 m (C-1, C-2, C-3, and C-4). Boring locations are shown in figure 2.2.1 (each of these borings is actually a cluster of several in-situ exploration tests). Figures 2.2.2 and 2.2.3 show subsurface profiles as drawn by the Soviet site exploration team. The interpolation between the borings is not a straight-line interpolation. It is not known whether the geologists who drew these profiles had subsurface information from locations other than those of the four borings shown in figure 2.2.1.

The original ground elevation at the building site varied from 137 to 140 m. The top of the piles (butt) is at elevation 131.7, and pile tips penetrate the rock formations to an elevation of approximately 124 (figs. 2.2.2 and 2.2.3). Boring depths are from 10 to 20 m below the pile tips and thus provide information to an adequate depth. However, interpolation between the borings does not provide all the relevant subsurface information in the vicinity of the middle of the east column line. In this area, the surface of the limestone layer apparently drops below the level encountered in the borings for the rest of the Office Building (or possibly the limestone is more severely eroded than in other locations).

In addition to showing the soil strata at the site, figures 2.2.2 and 2.2.3 show bearing resistance and skin friction values assigned to the soil strata by the Soviet designers on the basis of in-situ and laboratory test data. The uppermost layers of fill and glacial clays are of no interest since they were excavated. Below the pile butts is a layer of fine to coarse, medium dense sands of fluvioglacial origin. These are underlain by medium to stiff clays of glacial, Jurassic, and, at boring C-4, Carboniferous origins.

The soil layers are underlain by a 9- to 12-m thick layer of severely weathered Carboniferous limestone. This limestone layer is in turn underlain by an approximately 7-m thick layer of Carboniferous stiff clay marls which in turn rests on marl.

The elevation of the top of the upper ground water horizon at the time the borings were taken (January 1975) was approximately 130 m, about 1.7 m below the pile butts. The ground water is perched on the clay layers which underlay the entire building site and should be assumed to be present at least during the wetter season (fall to spring).

2.3 STRUCTURAL SYSTEM DESCRIPTION

The design of the Office Building uses a structural system composed of steel and standard precast concrete members. The structural frame is square in plan, 39 m × 39 m between exterior column centerlines, and is eight stories tall plus a basement and a penthouse. The height of the building from the first floor to the penthouse roof is 37.50 m. Figure 2.3.1 shows an architectural rendering of the building. The penthouse is the cruciform-shaped structure inside the parapet walls at the top of the building.

Figure 2.3.2 is a plan view of the building showing the column layout for the basement through the eighth story. Figure 2.3.3 shows an elevation of the exterior column line on the east side of the building. This view shows the elevation at each floor level and the height of each of the stories. The typical story height is 4.2 m; basement and first story heights are 4.8 m, and the height of the penthouse is 3.25 m. Figure 2.3.4 shows the east elevation of the building. This photograph, taken in June 1982 before construction of the exterior brick walls, shows the structural system composed of columns, beams, and floor slabs. The precast concrete members were selected from Soviet catalogs of standard sections which list available dimensions and design strengths. Each of the components in the structural frame and the connections between components is discussed in the following paragraphs. The foundation system is also described.

The designations for columns, beams, walls, and connections used in this report are as follows. For columns, the north-south and east-west axes designations are given. North-south axes are designated by letters; east-west axes are designated by numbers (fig. 2.3.2). Columns in a specific story have designations preceded by the story number. For example, column 3-F/4B is the column at the intersection of axes F and 4B in the third story (fig. 2.3.3 shows story designations). Beams and walls are identified by giving the column axes at the ends of the member, for example, E/4B-F/4B.

2.3.1 <u>Foundation</u>

The building rests on a pile foundation. The piles are precast concrete piles ranging in length from 7 to 12 m. The piles have a 250-mm × 350-mm rectangular cross-section and are reinforced by four longitudinal, 12-mm diameter reinforcement bars. The concrete is Mark 300 concrete and the longitudinal reinforcement is A-II steel (yield strength of 3000 kg/cm^2). (See section 3.4 for an explanation of Soviet concrete and steel designations and properties.) Ties are 6-mm diameter reinforcing bars made of A-I steel (undeformed bars with a yield strength of 2400 kg/cm^2).

In figures 2.2.2 and 2.2.3, the position of a 7.7-m long pile in relation to the soil profile is shown. The 7.7-m length was chosen by the Soviet designers on the basis of hypothetical soil resistance. The actual lengths of the driven piles range from 2 to 11.2 m with most piles between 5 and 6 m. The building is supported by a total of 1092 piles each having a design strength of 60 t. Columns C/1A through F/1A and C/9A through F/9A (fig. 2.3.2) are supported by pile groups of 20 piles each (there is a pile cap for each pile group); columns J1/3 to J1/8 are supported by pile groups of 15 piles; columns A1/4 through A1/7 and the four corner columns are supported by groups of 16 piles; columns A1/3 and A1/8 are supported by groups of 12 piles; and the two halves of the core are supported by two large pile groups of 192 piles each, which support large pile mats. In addition, there is a separate pile foundation for the retaining wall parallel to column line A1 consisting of three rows of piles spaced 2.2 m apart. Most piles were driven to practical refusal into the decomposed limestone layer. However, the piles for column J1/4 and some of the piles for columns J1/5 through J1/7 could not be driven to refusal.

2.3.2 Columns

Columns are located around the perimeter of the building and in the central core which contains the elevator shafts and stairwells (fig. 2.3.2). Columns are precast concrete except for a few steel columns on the seventh and eighth stories and on the penthouse level (fig. 2.3.4). Cross-sections of the steel columns are shown in figure 2.3.5 and are designated as Type S-1, S-2, and S-3. These columns are made of C 38/23 steel (tensile strength of 3800 kg/cm^2 and yield strength of 2300 kg/cm^2). Two types of precast concrete columns are used in the building: composite steel core and concrete, and reinforced concrete. All precast columns are 400 mm square. Figure 2.3.6 shows cross-sections for the five types of composite columns which are composed of steel sections surrounded by Mark 400 concrete and reinforcing steel. These columns are used where steel beams frame into a column. Composite column Types SC-7, SC-9, and SC-12 have cores composed of C 46/33 steel (tensile strength of 4600 kg/cm^2 and yield strength of 3300 kg/cm^2), and composite column Types SC-2 and SC-5 have cores made of C 38/23 steel. Figure 2.3.7 shows cross-sections for the three types of precast reinforced concrete columns. Reinforced concrete column Types RC-2, RC-3, and RC-5, are made of Mark 300, Mark 500, and Mark 600 concrete, respectively. Column reinforcing steel is grade AIII (tensile strength of 6000 kg/cm^2 and yield strength of 4000 kg/cm^2).

Columns are manufactured in one-story segments and are joined approximately 600 mm above the floor level. The central portion of each column bears on the column underneath it. The four corner longitudinal reinforcing bars of each precast column are exposed so that when the columns are joined the longitudinal bars can be welded together. After welding, the space surrounding the bars is filled with concrete. Figures 2.3.8 and 2.3.9 show typical column to column connections. When a precast column is joined to a steel column, the longitudinal bars of the precast column are welded to reinforcing bars which are welded to the steel columns. The basement columns are fastened to concrete pile caps through concrete piers or pedestals.

2.3.3 Beams

Figure 2.3.10 is a view of the building showing the beam layout for floors 3 through 6. The beam layout for the other floors is the same except for additional steel beams in areas which support heavier floor loads. Steel beams are shown as single lines and concrete beams are shown by rectangular sections. (Solid rectangular sections indicate the location of shear walls. These walls are discussed in section 2.3.4.)

Steel beams are used for the 13.2-m spans and some of the 7.8-m spans between the exterior columns and the columns on the perimeter of the core, and where heavier floor loads occur. Steel beams are either rolled sections or built-up plate sections; these beams have "I," "box," or "channel" cross-sections. Tables 2.3.1(a) and (b) list the built-up plate sections and the rolled beam sections and their dimensions. The built-up steel beams are typically used for longer spans. The rolled steel beams are typically used for shorter spans, such as in the core. Figure 2.3.11 shows the 13.2-m long I-beams (Type 8 in table 3.2.1(a)) which span between the exterior columns and core columns.

Precast reinforced concrete beams have either an inverted "T" or "Z" cross-sectional shape as shown in figure 2.3.12. There are two sizes of inverted T-beams; these are 450 mm and 600 mm deep. The cross-sectional dimensions of the two inverted T-beams and the Z-beam are fixed; however, the design strength is varied by using different amounts of reinforcement and different concrete strengths. The concrete beams support precast floor planks or concrete slabs. The Z-beams are spandrel beams; one side of the beam supports a floor slab and the other side is built into the brick masonry wall.

Concrete beams framing into concrete columns are supported by reinforced concrete corbels or by steel brackets. Figures 2.3.13(a) and (b) show details of typical connections between concrete beams and columns. Steel plates built into the ends of the concrete beams are welded to the supporting steel bracket or to steel plates embedded in the corbel. The locations of these types of connections, referred to as Detail 3 and Detail 4, respectively, are shown in figure 2.3.10.

Many steel beams are supported by corbels or steel brackets. Large steel beams are supported by steel seats which are welded to plates embedded in the column. Figure 2.3.14 shows a typical beam to column connection between a concrete column, two concrete beams and a large steel beam. This is the type of connection that occurs at the exterior columns which support the 13.2-m steel beams that span between the exterior frame and the interior core (Detail 1 in fig. 2.3.10). Figure 2.3.15 is a view of this connection. The column has corbels which support the two concrete beams which frame into the column on opposite sides. The steel beam bears on a beam seat. Steel plates welded to the column and to the beam and erection bolts are used to keep the beam in place.

All the beam to column connections in the structure are designed mainly to transmit vertical loads. The plate welded to the top of a concrete or steel beam is designed to limit the bending moment that is transmitted to the column (figs. 2.3.13 and 2.3.14). This plate also provides lateral support.

There are many beam to beam connections in the building. A typical connection of this type is the connection between the 13.2-m and 7.8-m steel beams (Detail 2 in fig. 2.3.10) shown schematically in figure 2.3.16 and in the photograph in figure 2.3.17. This type of connection is made by field welding the beam to plates which are welded to the web and flanges of the supporting steel beam. Bolts are used to aid in erection. Connections between concrete beams and steel beams occur in the core. A typical connection of this type (Detail 5 in fig. 2.3.10) is shown in figure 2.3.18. In this connection, the steel beam bears on the flange of the concrete beam. The end-plate of the steel beam is welded to a steel saddle which straddles the web of the concrete beam.

2.3.4 Shear Walls

Resistance to lateral load is provided by 180-mm thick, precast, reinforced concrete shear walls located at the southeast and northwest corners of the core. Figures 2.3.2 and 2.3.10 show the location of these walls; a view of these walls is shown in figure 2.3.19. The basement shear walls are cast-in-place concrete; the shear walls on stories 1 through 7 are precast concrete. (The cast-in-place concrete walls in the eighth story are not part of the shear wall system.) Figure 2.3.20(a) shows an elevation view of a typical shear wall. The plan and elevation cross-sections in figures 2.3.20(b) and (c) show the location of the primary reinforcement in the wall. There is a vertical steel bar near each edge of the panel and mesh reinforcement on both faces of the panel.

The precast walls are manufactured one-story tall; vertical continuity between the walls is achieved by filling horizontal joints. The top and bottom of each wall is castellated (fig. 2.3.20(a)); once the walls are in place, these castellations are filled with concrete.

Shear walls are connected to columns or to adjacent shear walls along the vertical edges of the walls. These standard connection details are shown in figures 2.3.21 and 2.3.22, respectively. At three points along each vertical edge of a wall (fig. 2.3.20), steel plates embedded in the wall are welded to plates attached to the face of a column or to plates embedded in an adjacent wall. A 205-mm wide cast-in-place concrete joint exists adjacent to columns G/6B and D/4A, because standard precast panels were not available to match some of the column spacings. This joint is reinforced with vertical bars and lateral ties. At this joint the connection between the precast wall and the column requires 255-mm long plates, instead of the typical 65-mm plates.

2.3.5 Floor Planks and Slabs

Precast, prestressed, hollow core, concrete planks and precast, solid reinforced concrete planks, 220 mm thick, span between the beams. Figure 2.3.23 shows a cross-section of a hollow core plank (Type NV-52-18T) which is widely used in the Office Building. The concrete in this plank is Mark 200. The plank is reinforced with grade AIV wires (yield strength of 6000 kg/cm^2), 14 m in diameter, prestressed to a nominal value of 3600 kg/cm^2.

Floor planks are supported by the flanges of the concrete inverted T-beams and Z-beams. Where steel beams are used, the floor planks rest on the top flange of the beam. On floors 1 through 7 an 80-mm thick concrete topping containing wire mesh is cast on top of the final assemblage of beams and floor planks. On the more heavily loaded north side of the seventh floor and on the first floor, the floor capacity is increased by alternating the planks with cast-in-place reinforced concrete slabs. These slabs are 300 mm wide on the seventh floor and 400 mm wide on the first floor. On the heavily loaded eighth floor, a 200-mm thick reinforced concrete slab is cast on top of the precast planks. Figure 2.3.24(a) shows a concrete beam supporting precast planks which are covered by concrete topping; figure 2.3.24(b) shows a steel beam supporting precast planks and a reinforced concrete slab.

2.4 BUILDING ENVELOPE AND BRICK MASONRY PARTITIONS

The building envelope consists of the exterior walls, the roof, and the fenestration (doors, windows, and other openings). It does not include below-grade portions of the building. This section describes the building envelope and the brick partition walls in the core.

2.4.1 Exterior Walls

The building is 41.03 m on a side from outside edge to outside edge of exterior masonry walls. A moment resisting, reinforced concrete portal frame is used to support the exterior walls on the south, east, and west sides of the building as shown in figure 2.4.1. Figure 2.4.1(a) shows a plan of the large columns that make up this frame; figures 2.4.1(b) through (d) show elevations of the frame on the south, east, and west sides of the building. The frame is two stories high on the east and south sides. The masonry on the north side is supported by a 600-mm thick wall built of large precast concrete blocks.

The north and east sides of the building have ground elevations mostly at the second floor level. There are exterior stairs one story in height at the west corner of the north elevation and at the south corner of the east elevation. On the south, the main entrance and canopy are the only fenestration in the first story. On the west elevation there are windows in the first story which are narrower than those on floors above.

2.4.1.1 Wall Sections

From the second story through the seventh story there are 14 windows per side per story, evenly spaced, with a brick masonry pier between each window. A column is located at every other pier. The columns are spaced 5.4 m on center, and the windows are 1.9 m wide, spaced 2.7 m on center. The windows are recessed 885 mm from the building face. The piers between the windows are typically 790 mm × 930 mm in cross-section. Figure 2.4.2 shows plan views of piers at columns and between columns. Figure 2.4.3 is a vertical section showing masonry and beam details at the piers between windows. The piers at the corners of the building are all 2.01 m wide; these corner piers include a recess on each face that is one brick long and one wythe thick. A plan view of the corner detail is shown in figure 2.4.4. The eighth story has no windows; the walls at this story consist of 380 mm of brick and 200 mm of cast-in-place concrete.

The brick masonry in the Office Building has no expansion joints, and it is built partially within the structural frame. The exterior walls on the first through the seventh stories are tied to the columns and spandrel beams in the exterior frame (figs. 2.4.2 through 2.4.4). Figure 2.4.5 is a cross-section of the exterior masonry wall at the eighth floor; on this floor, the reinforced concrete slab extends into the masonry wall. The columns on the eighth story are not built integrally with the masonry walls as on the other stories (fig. 2.4.6).

2.4.1.2 Window Openings

Typical window heads and sills are shown in figure 2.4.7. Floating precast concrete lintels are used to support the brick over the window heads, except the brick facing, which is supported on galvanized steel angles. The sills and soffits for the windows are preformed metal sections. There are open voids above the soffits. The joint between the masonry and the metal soffits and sills is filled with a sealant for water tightness. Stainless steel flashing is provided at the edges of the sills where they abut the brick masonry.

2.4.2 <u>Parapet and Penthouse Walls</u>

Figure 2.4.8 is a plan of the penthouse and parapet walls. The cruciform-shaped penthouse is 3.25 m in height and is capped at an elevation of 176.00 m, which is the same height as the top of the parapet. A vertical section through the penthouse wall is shown in figure 2.4.9. The penthouse walls are 380 mm thick, and are built of facing brick, backed up with building brick.

A parapet wall surrounds the eighth story roof; this wall is 520 mm thick and 3.25 m in height. A vertical section through the parapet wall is shown in figure 2.4.10. This wall consists of facing brick on both sides of a two wythe thick building brick core. There is a reinforced concrete cast-in-place bond beam at the top of the parapet wall. A preformed metal coping protects the wall at the top.

2.4.3 <u>Brick Partitions in the Core</u>

Except for the precast shear walls and the cast-in-place concrete walls surrounding the "disintegrator" room on each floor, all of the core walls are constructed of brick masonry. Most of these masonry walls are two wythes thick. The masonry walls in the interior of the core are solid walls. The masonry walls around the perimeter of the core are hollow walls with the two wythes separated by 250 mm. These walls are shown in figure 2.4.11 which is a plan of the core.

2.4.4 Masonry Materials

2.4.4.1 Brick

Two types of brick were used in the building: facing brick and building brick. These brick have a specified size of 120 mm x 250 mm x 65 mm. The facing brick were manufactured in the United States; the backup building brick are of Soviet production. The facing brick were specified to comply with Standard Specification for Facing Brick, ASTM C 216 [2.1], Grade SW, Type FBS, except that a minimum compressive strength of 420 kg/cm^2 (6000 psi) was specified. This value is greater than the minimum specified in ASTM C 216. The building brick were specified to comply with Standard Specifications for Building Brick, ASTM C 62 [2.2], Grade SW, for exterior wall construction and Grade NW for interior work.

2.4.4.2 Mortar

The mortar specified for general use was to comply with Standard Specifications for Mortar for Unit Masonry, ASTM C 270 [2.3], Type N. This is consistent with the recommendations contained in ASTM C 270, except for exterior walls at or below grade, for which use Type S mortar is recommended.

The mortar specified for facing brick work was:

> "Mortar for face brick shall be premixed and prepackaged and to be of approved shade achieved by use of natural Portland cements. Compressive strength shall be a minimum of 126 kg/cm^2 at 28 days; complying with ASTM C 270, Type S mortar."

2.4.4.3 Anchors and Ties

Brick exterior walls are anchored to the concrete columns and concrete spandrel beams by 10-mm diameter deformed steel reinforcement which has been prefabricated into special shapes and welded to plates cast in the columns and spandrel beams. The details for these anchors are given in Soviet drawings 5KR·3·6. As shown in figure 2.4.12, anchors to beams occur near columns and anchors between piers and columns occur at three locations per story. Figure 2.4.13 shows the anchors at the corner columns. Prefabricated wall ties of U.S. manufacture are called for every third or fourth course.

2.4.5 Roofing

The roof is divided into three areas: penthouse, open areas, and snow melting rooms (figs. 2.3.1 and 2.4.8). The roof is sloped so that each quadrant will drain toward the snow melting room at the corner of that quadrant. At each extension of the penthouse there are large air louvers. The waterproofing system over the open areas, as designed by Skidmore, Owings, and Merrill, is a multi-layered system consisting of the following materials listed as detailed from top to bottom: 1) protection board (3 mm thick) over loose laid EPDM rubber sheet (1 mm thick); 2) two layers (each layer 51 mm) of rigid board roof insulation of polyisocyanurate foam core permanently bonded to rigid skins of aluminum foil; 3) protection board (3 mm thick) over loose laid EPDM rubber sheet (1 mm thick) over an aluminum-faced vapor barrier; 4) lightweight concrete sloped toward a drain in each of the snow melting rooms, and, 5) structural concrete deck. A concrete slab was placed over the waterproofing system.

The penthouse roofing system as designed consists of the following materials listed as detailed from top to bottom: 1) traffic pads loose laid over loose laid EPDM rubber sheet (1 mm thick); 2) lightweight concrete (sloped); 3) protection board (3 mm thick) over loose laid EPDM rubber sheet (1 mm thick); 4) two layers (each layer 76 mm) of rigid board roof insulation; 5) aluminum faced vapor barrier; and, 6) hollow core structural concrete deck.

The roofing system for the snow melting rooms consists of the following materials listed as detailed from top to bottom: 1) loose laid EPDM rubber sheet (1 mm thick) covered with ballast; 2) one layer (51 mm thick) and one tapered layer of rigid board roof insulation; and, 3) steel decking.

2.5 PROJECT CHRONOLOGY

This section documents the important dates in the design and construction of the Office Building. The activities and dates listed were obtained from a review of monthly inspection reports on file at the Office of Foreign Buildings Operations.

In May 1969, the United States and the Soviet Union signed an agreement on the reciprocal free use of land, and in December 1972, an agreement was signed designating the Embassy sites and the conditions of construction. In June 1968, the U.S. Department of State entered into an agreement with the San Francisco office of Skidmore, Owings, & Merrill and the New York office of Gruzen & Partners to provide architectural and engineering services. In September 1973, these firms were authorized to commence preliminary architectural and engineering design studies. Table 2.5.1 gives a chronology of the dates and activities important in the design of the Embassy complex.

Construction activities involving the foundation, structural frame, precast floor slabs and walls, exterior masonry walls, and interior masonry partitions are of primary importance in this report. Table 2.5.2 lists start and completion dates for these construction activities.

The construction sequence can be briefly summarized as follows. Work commenced on the foundation of the Office Building with the driving of piles in November 1979, and was completed in July 1982 when the mudslab was finished. Erection of the structural frame (including columns, beams, precast walls and floor planks) began in March 1981, and was essentially complete by May 1982, except for setting of floor planks on the penthouse and roof levels. Figure 2.3.4 is a view of the Office Building showing the nearly completed structural frame in June 1982. Work on cast-in-place walls, floor slabs, and floor plank toppings began in May 1982 and continued until construction ceased in August 1985. Laying of the exterior brick walls began in July 1982, and was completed in May 1984. Laying of interior brick walls began in February 1982 and continued until construction ceased in August 1985. A temporary roof was placed on the building in November 1986. Figure 2.5.1 shows the appearance of the Office Building in December 1986 as viewed from the southwest.

2.6 REFERENCES

2.1 Standard Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale). ASTM C 216-85a, Vol. 04.05, American Society for Testing and Materials, Philadelphia.

2.2 Standard Specification for Building Brick (Solid Masonry Units made from Clay or Shale). ASTM C 62-85a, Vol. 04.05, American Society for Testing and Materials, Philadelphia.

2.3 Standard Specification for Mortar for Unit Masonry. ASTM C 270-86, Vol. 04.05, American Society for Testing and Materials, Philadelphia.

Table 2.3.1(a). Dimensions and properties of built-up steel beams

Beam	Dimensions: Flanges (mm)	Dimensions: Web (mm)	Area (cm^2)	Moment of Inertia Strong Axis (cm^4)	Steel Strength Yield (kg/cm^2)	Steel Strength Ultimate (kg/cm^2)
5	600 × 32	736 × 16	501.8	619,715	3300	4600
6	300 × 32	736 × 16	437.8	525,290	3300	4600
7	400 × 32	736 × 16	373.8	430,865	3300	4600
8	400 × 25	750 × 10	275.0	335,575	3300	4600
8a	400 × 25	750 × 16	320.0	356,665	3300	4600
9	400 × 16	768 × 10	204.8	234,465	3300	4600
9a	400 × 16	768 × 16	250.9	257,115	3300	4600
10	300 × 16	768 × 10	172.8	185,285	3300	4600
10a	300 × 16	768 × 16	218.9	207,935	3300	4600
11	600 × 32	636 × 16	485.8	463,005	3300	4600
12	400 × 25	650 × 10	265.0	250,800	3300	4600
13	250 × 25	650 × 10	190.0	165,335	3300	4600
14	200 × 25	550 × 10	155.0	96,575	3300	4600
15	200 × 25	550 × 10	155.0	96,575	3300	4600
16	400 × 16	468 × 10	174.8	83,530	3300	4600
17	400 × 20	460 × 8	196.8	98,700	2300	3800
18	250 × 12	776 × 8	122.1	124,300	2300	3800
19	300 × 20	360 × 12	163.2	48,025	2300	3800
20	200 × 12	675 × 12	129.1	87,700	2300	3800
30	250 × 32	736 × 16	277.8	289,225	3300	4600

Table 2.3.1(b). Dimensions and properties of rolled steel beams

Beam	Section	Dimensions: Height (mm)	Dimensions: Flange Width (mm)	Dimensions: Web Thick. (mm)	Area (cm^2)	Mom. of Inertia Strong Axis (cm^4)	Steel Strength Yield (kg/cm^2)	Steel Strength Ultimate (kg/cm^2)
21	I	600	190	12.0	138.0	76,806	2300	3800
22	I	550	180	11.0	118.0	55,962	2300	3800
23	I	300	135	6.5	46.5	7,080	2300	3800
24	I	400	155	8.3	72.7	19,062	2300	3800
25	Channel	400	115	8.0	61.5	15,220	2300	3800
26	Channel	300	100	6.5	40.5	5,810	2300	3800
27	Box	300	200	13.0	81.0	11,620	2300	3800
28	Box	220	164	10.8	53.4	4,220	2300	3800
29	Box	400	230	16.0	123.0	30,440	2300	3800

Table 2.5.1 Design chronology for Embassy complex

Date	Activity
June 1968	Department of State entered into an agreement with SOM, San Francisco, and Gruzen & Partners, New York to provide architectural and engineering services.
Sept. 1973	U.S. architects and engineers authorized to commence preliminary architectural and engineering design studies.
Feb. 1975	Soviets approved preliminary design.
Sept. 1975	U.S. architects and engineers authorized to proceed into definitive design development and to prepare construction drawings and specifications.
Oct. 1976	Department of State delivered Russian language version of construction documents to the Soviets.
Mar. 1977	Final plans for U.S. embassy complex approved by Soviets.
Aug. 1977	Soviets began transposition of U.S. project documents into Soviet format.
Dec. 1977	Soviets delivered Technical Design documents in Russian language for review.
Mar. 1978	U.S. Foreign Building Office unconditionally approved Soviet Technical Design.
April 1978	Construction contract negotiations began.
April 1979	Contract for construction signed.
June 1979	Agreement reached with SVSI (Soviet contractor).
Sept. 1979	The embassy site was formally transferred to SVSI.
Oct. 1979	Construction began.

Table 2.5.2. Start and completion dates for construction activities on the Office Building

Activity	Start	Complete
Foundation		
Piles	Nov. 1979	Jan. 1980
Pile Caps	Nov. 1980	Mar. 1981
Mudslab	Sept. 1980	July 1982
Structural Frame		
Structural Steel	May 1981	May 1982
Columns, Beams, Precast Walls, and Planks	Mar. 1981	May 1982*
*95% Complete Overall		
Masonry		
Facing Brick	July 1982	Nov. 1983
Interior Brick Walls	May 1982	---**
**Dec., 1983: Brick 95% complete overall.		

NOTE: Construction ceased August 1985

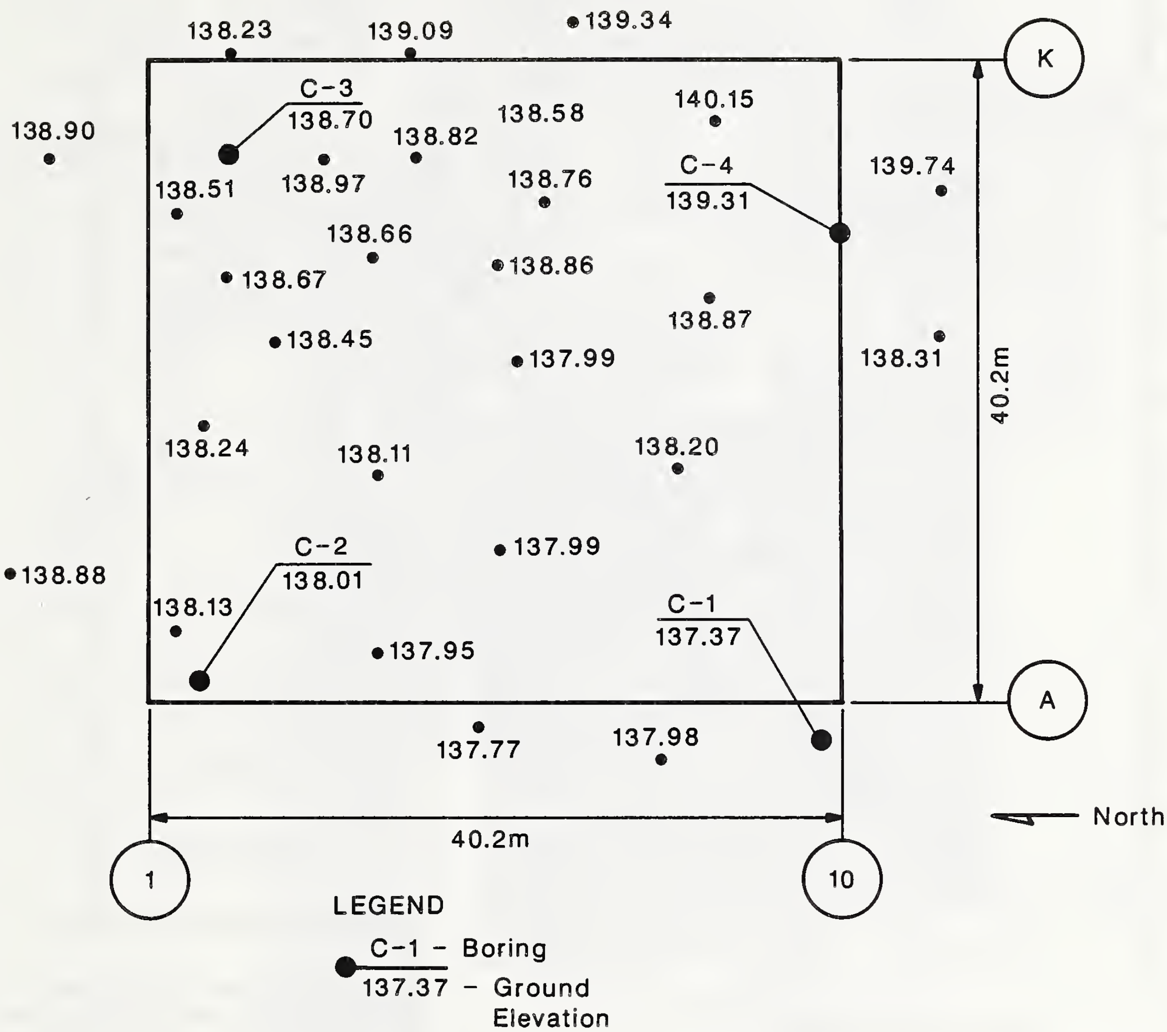


Figure 2.2.1. Bore hole locations from Soviet report

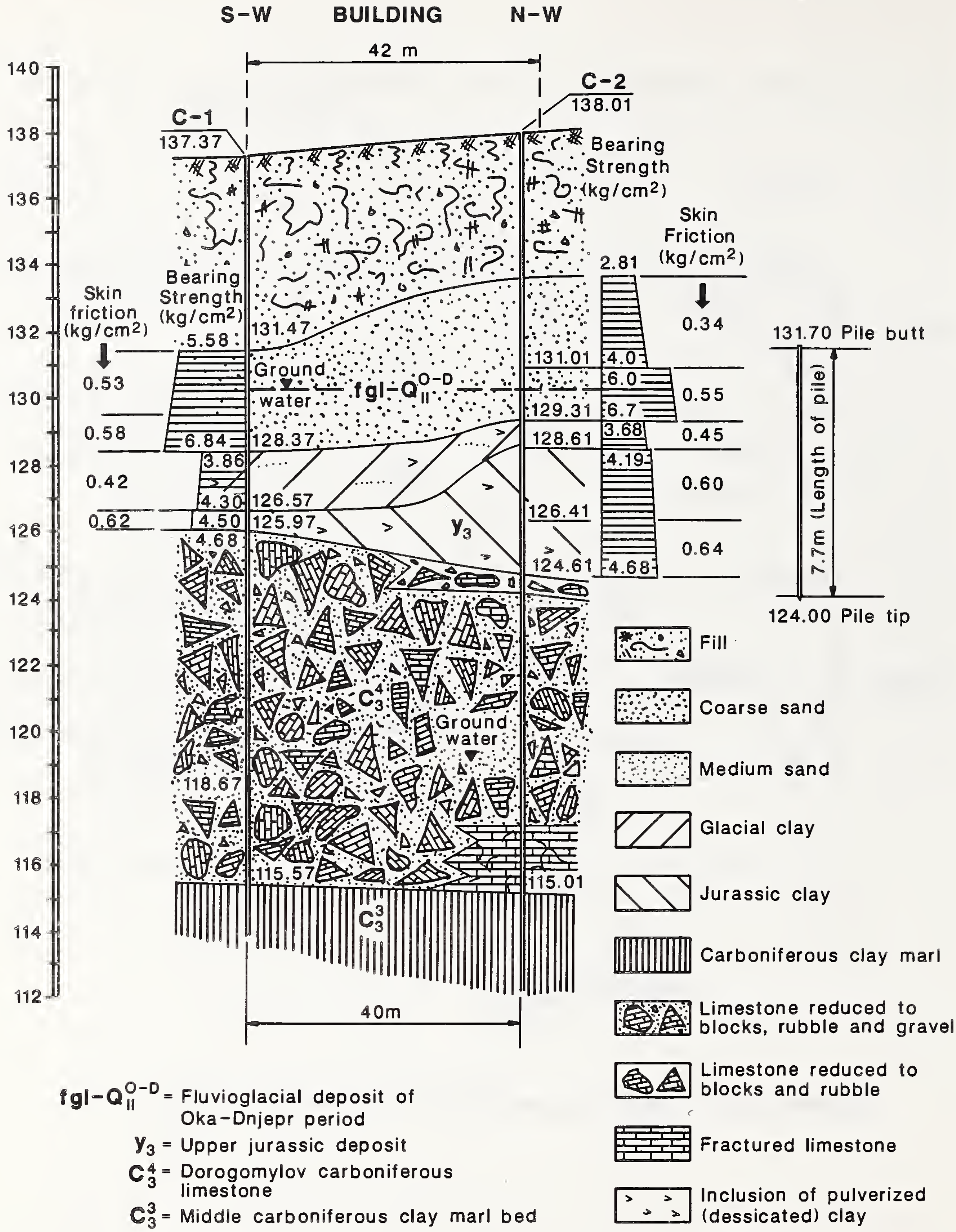


Figure 2.2.2. Geological profile along west side of building taken from Soviet exploration report

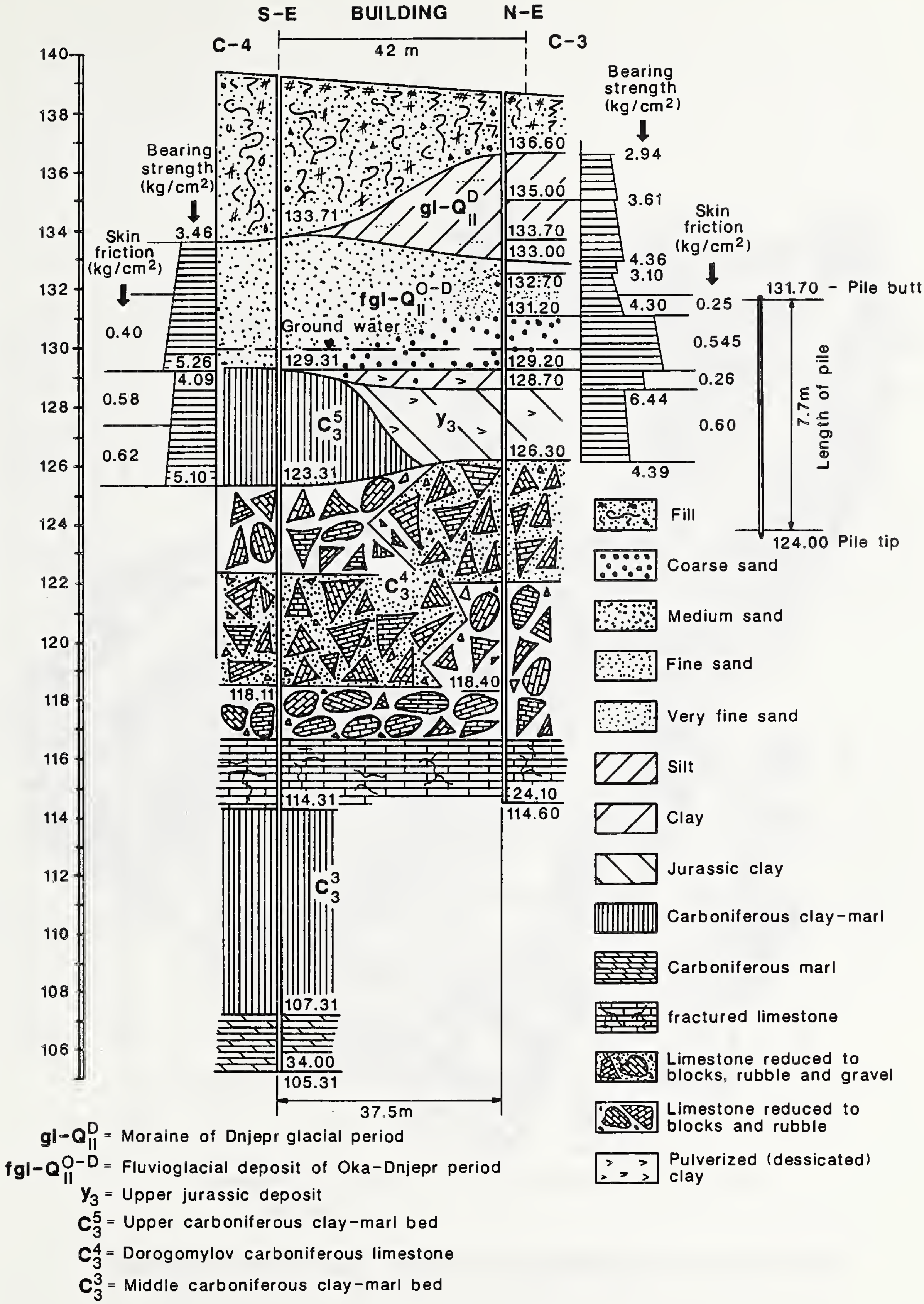


Figure 2.2.3. Geological profile along east side of building taken from Soviet exploration report

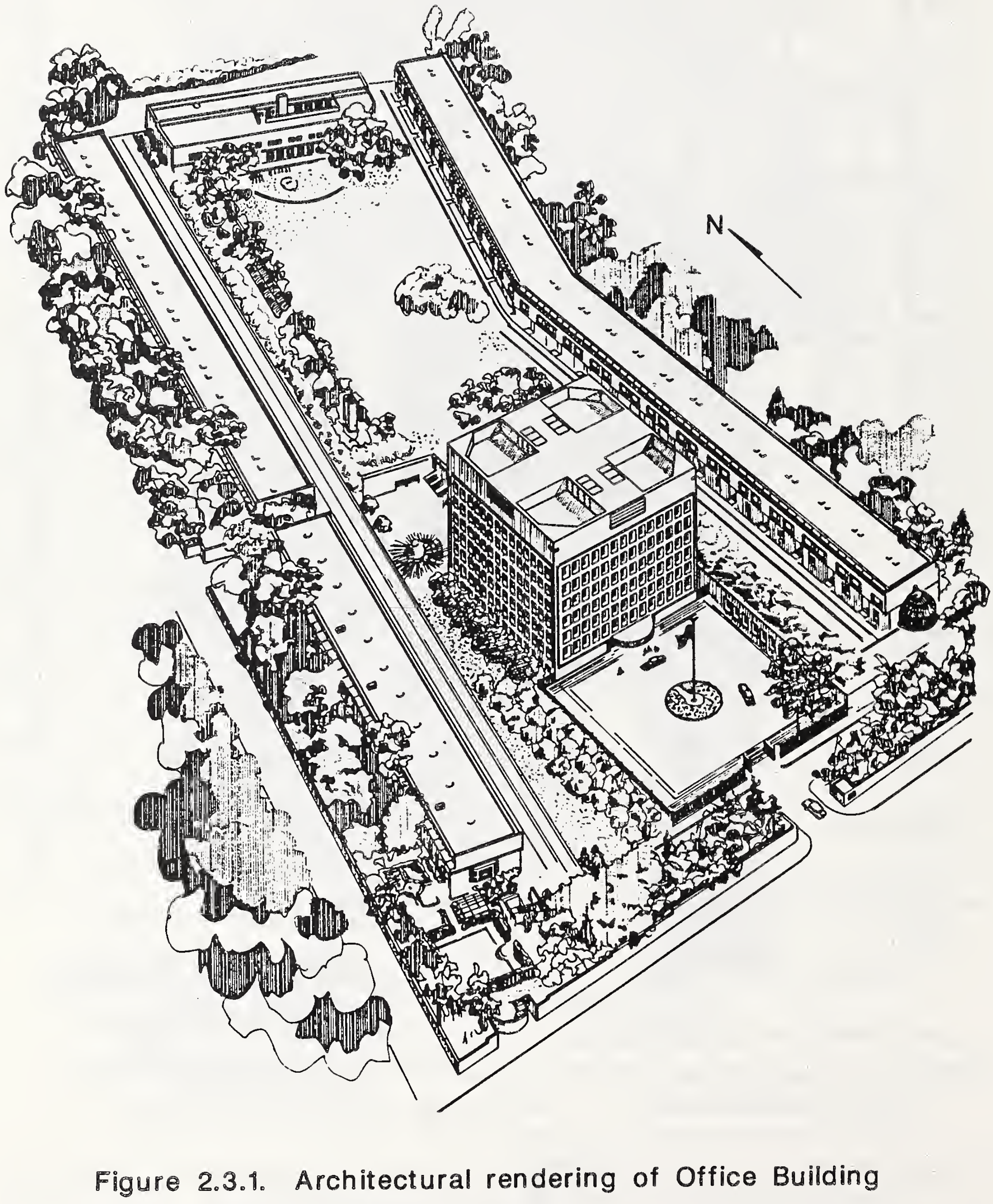


Figure 2.3.1. Architectural rendering of Office Building

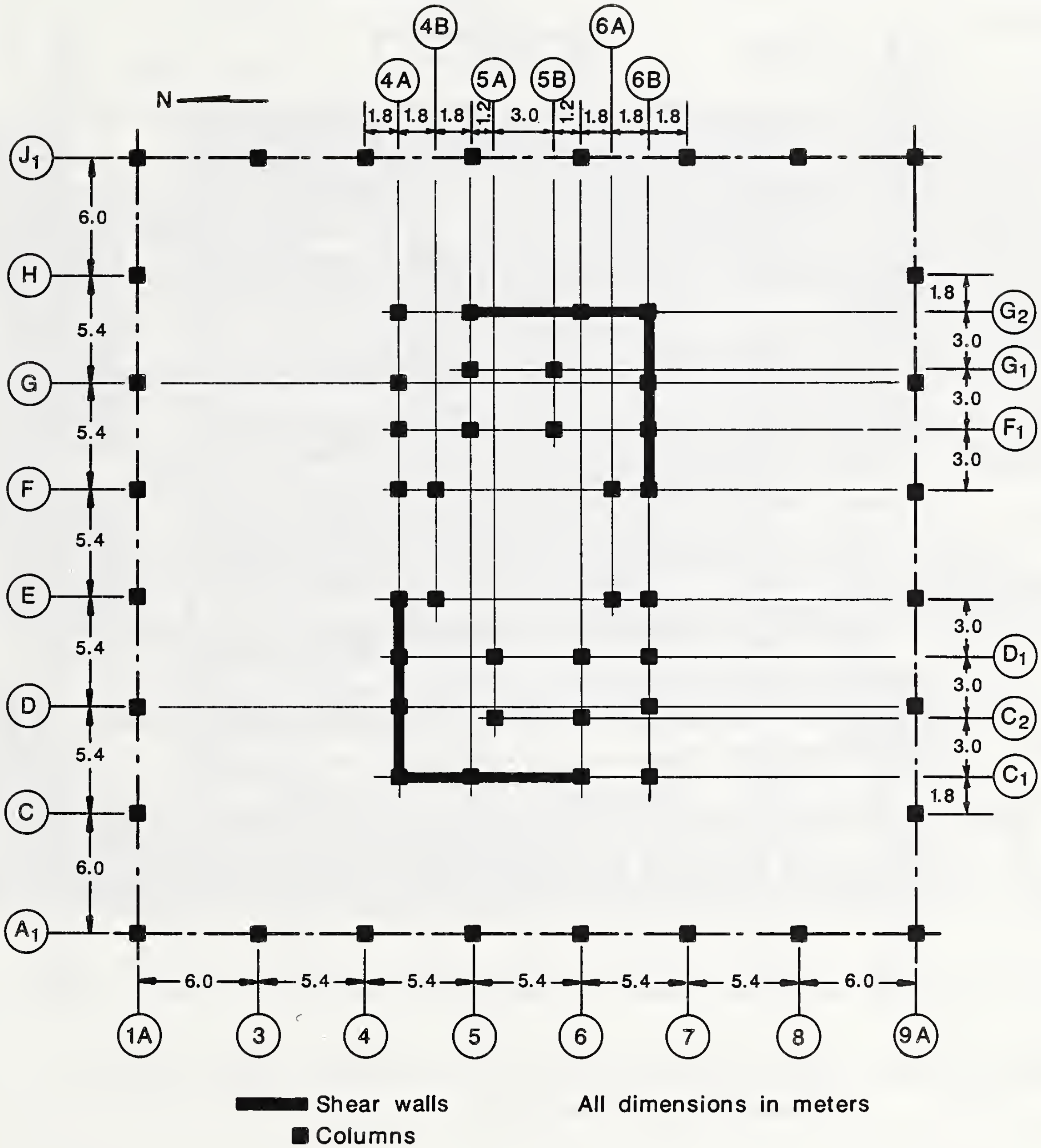


Figure 2.3.2. Plan view showing column layout

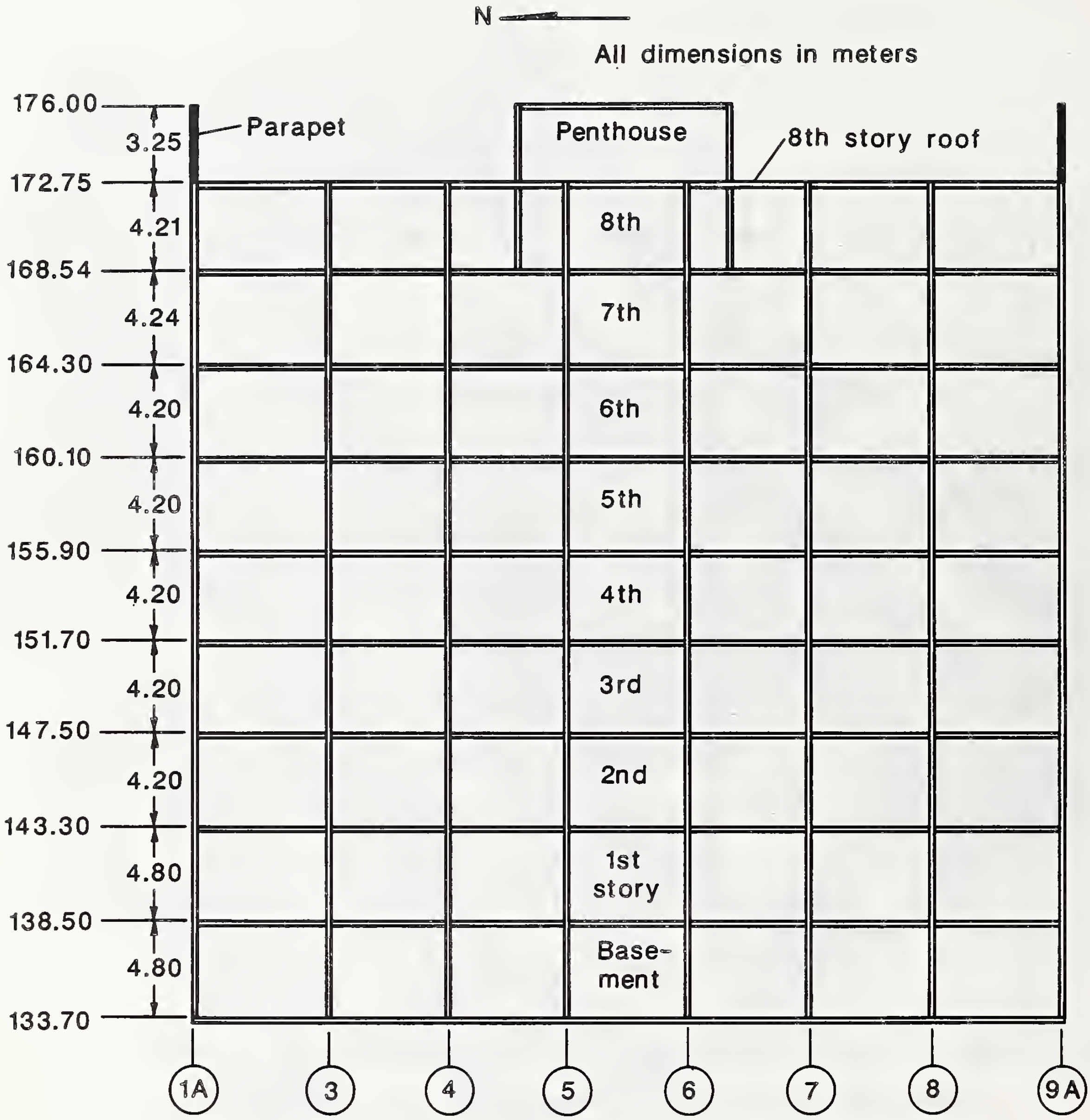


Figure 2.3.3. Schematic elevation view of exterior column line A1 on east side of building

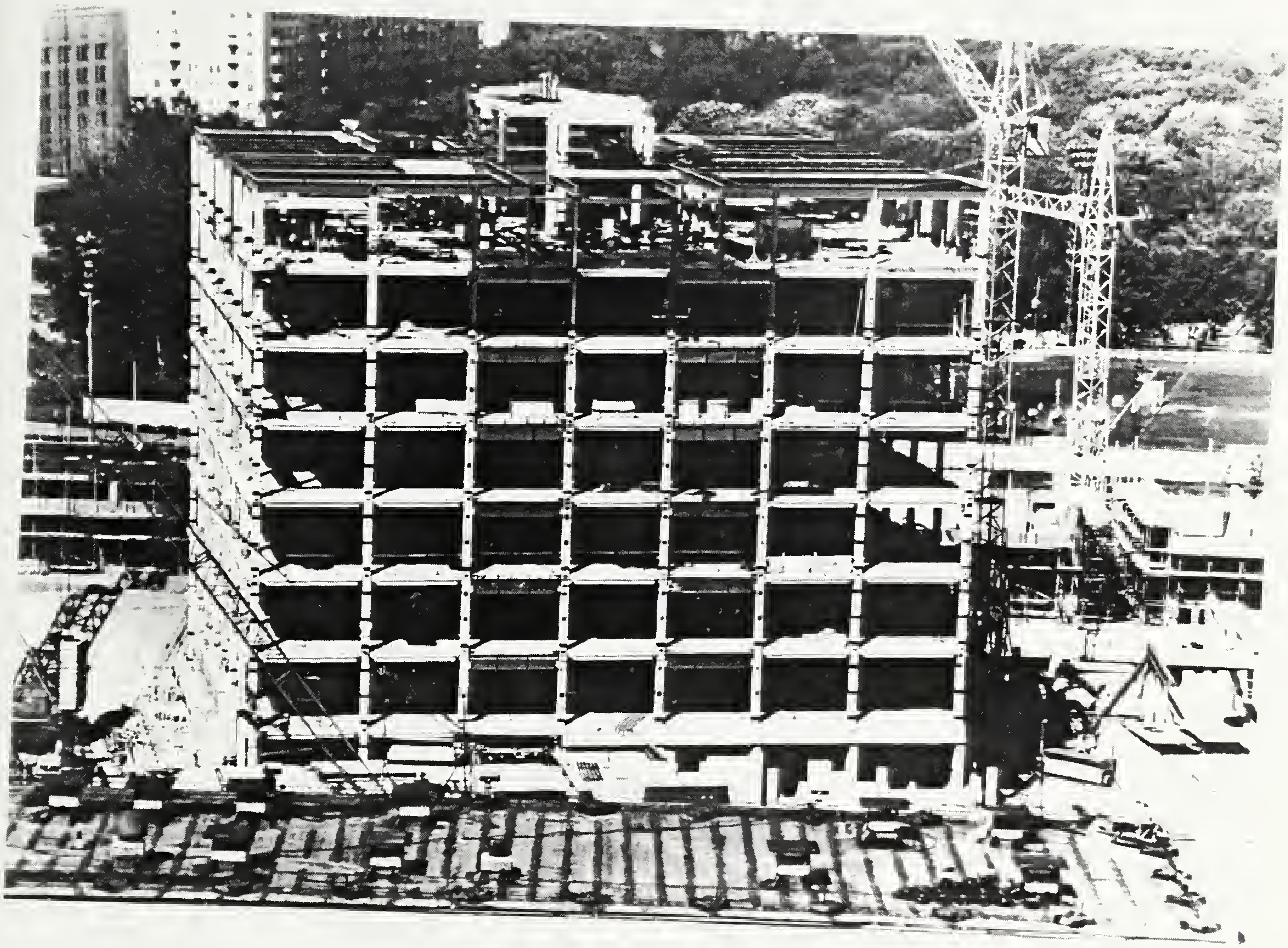
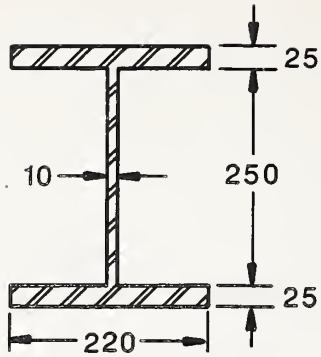
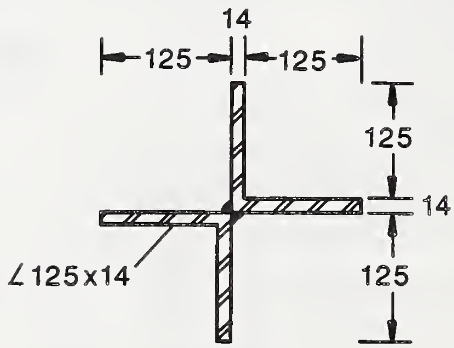


Figure 2.3.4. June 1982 photograph of east elevation of structure

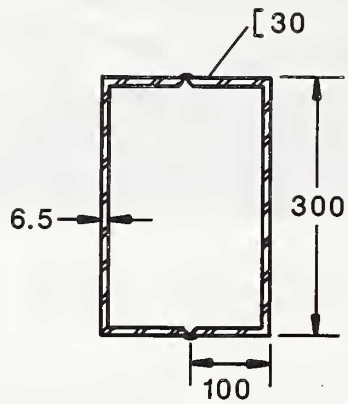


(a) S-1



(b) S-2

All dimensions in millimeters



(c) S-3

Figure 2.3.5. Cross-sections of steel columns

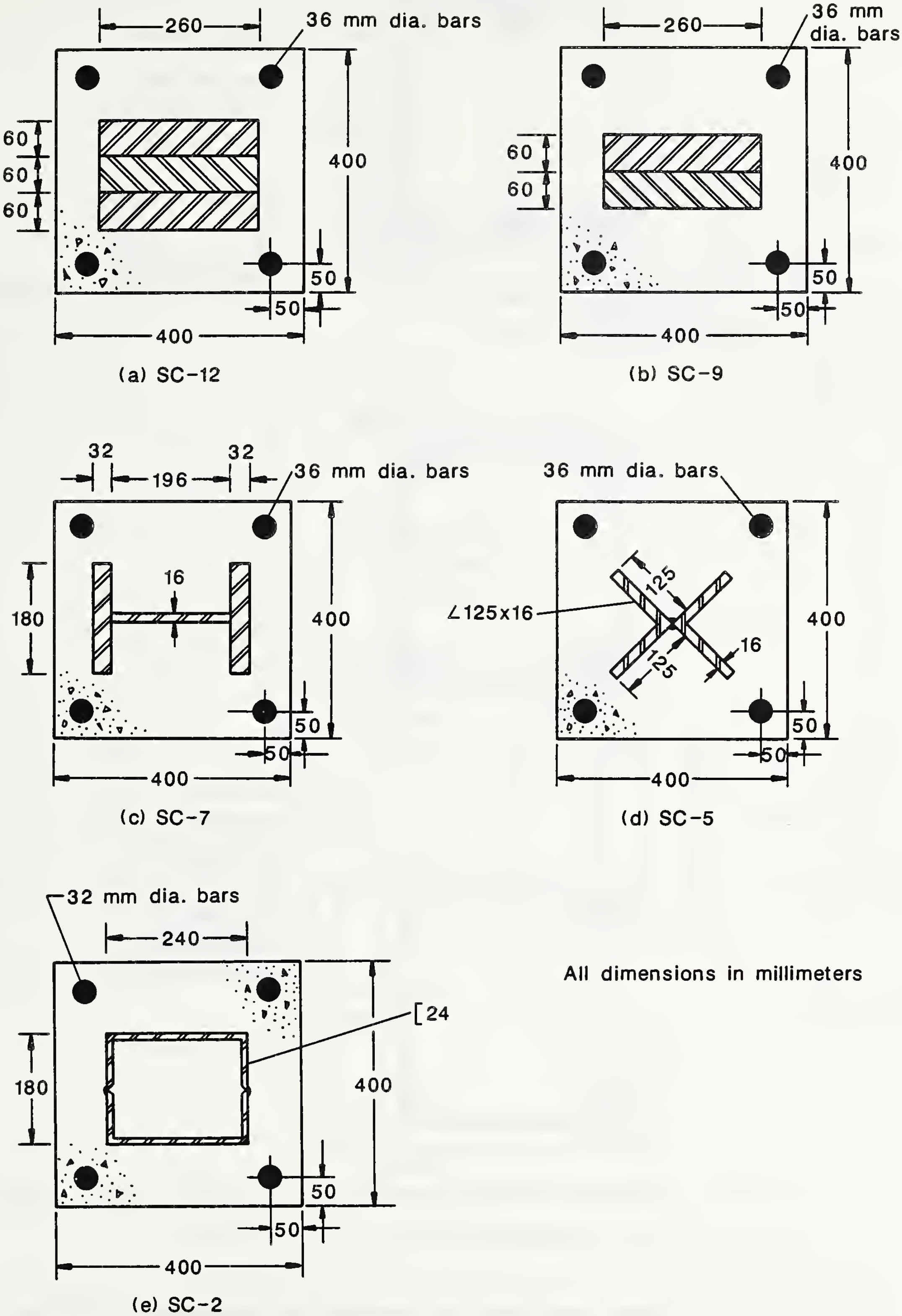


Figure 2.3.6. Cross-sections of composite columns

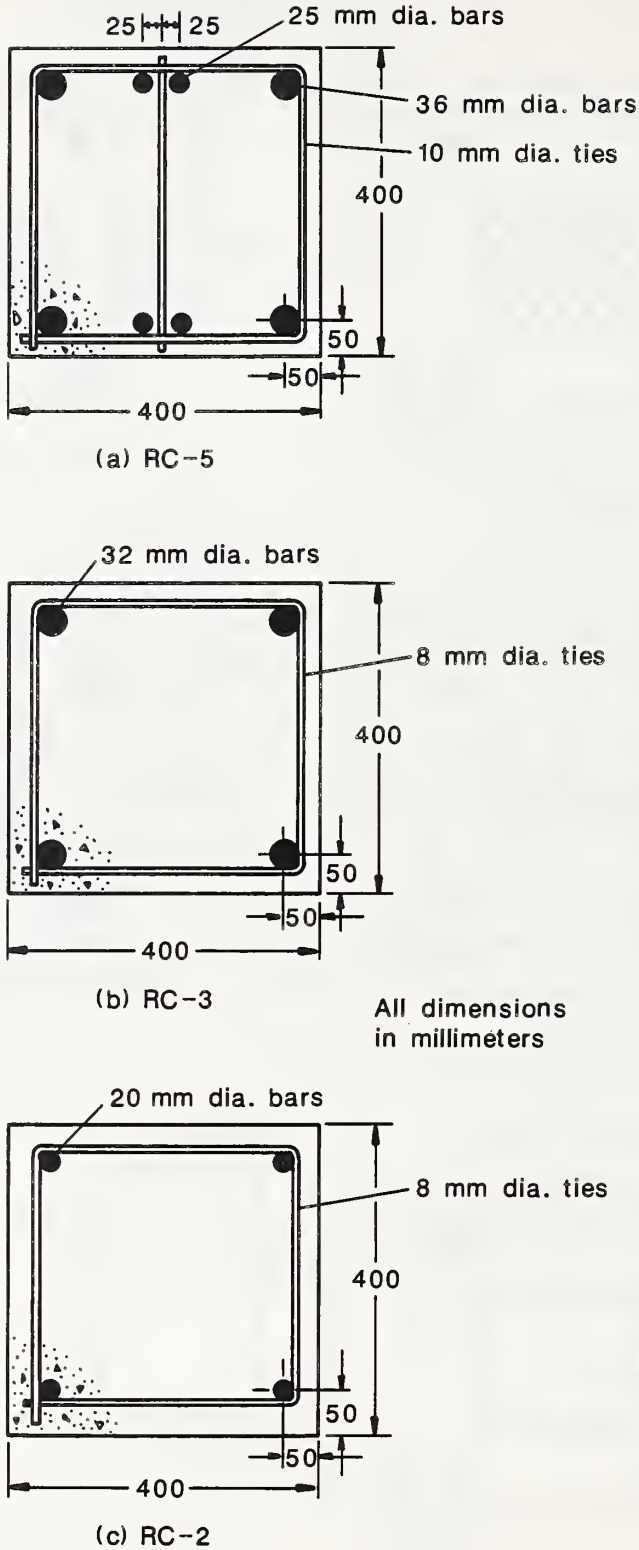


Figure 2.3.7. Cross-sections of reinforced concrete columns

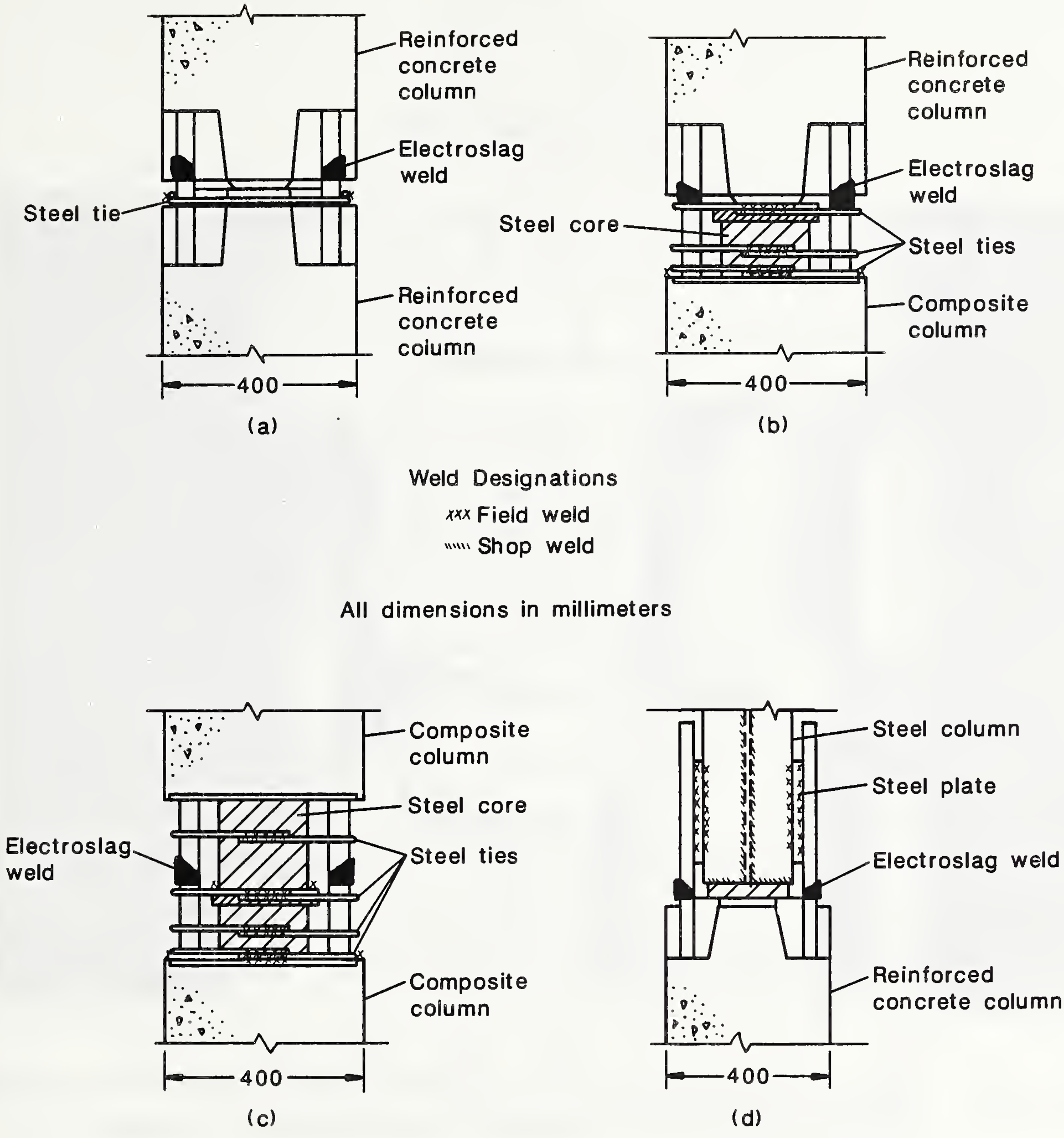


Figure 2.3.8. Column connections: a) reinforced concrete to reinforced concrete; b) reinforced concrete to composite; c) composite to composite; d) precast concrete to steel

Figure 2.3.9. Typical reinforced concrete column connection prior to filling with grout and concrete (The joint tie is not present)

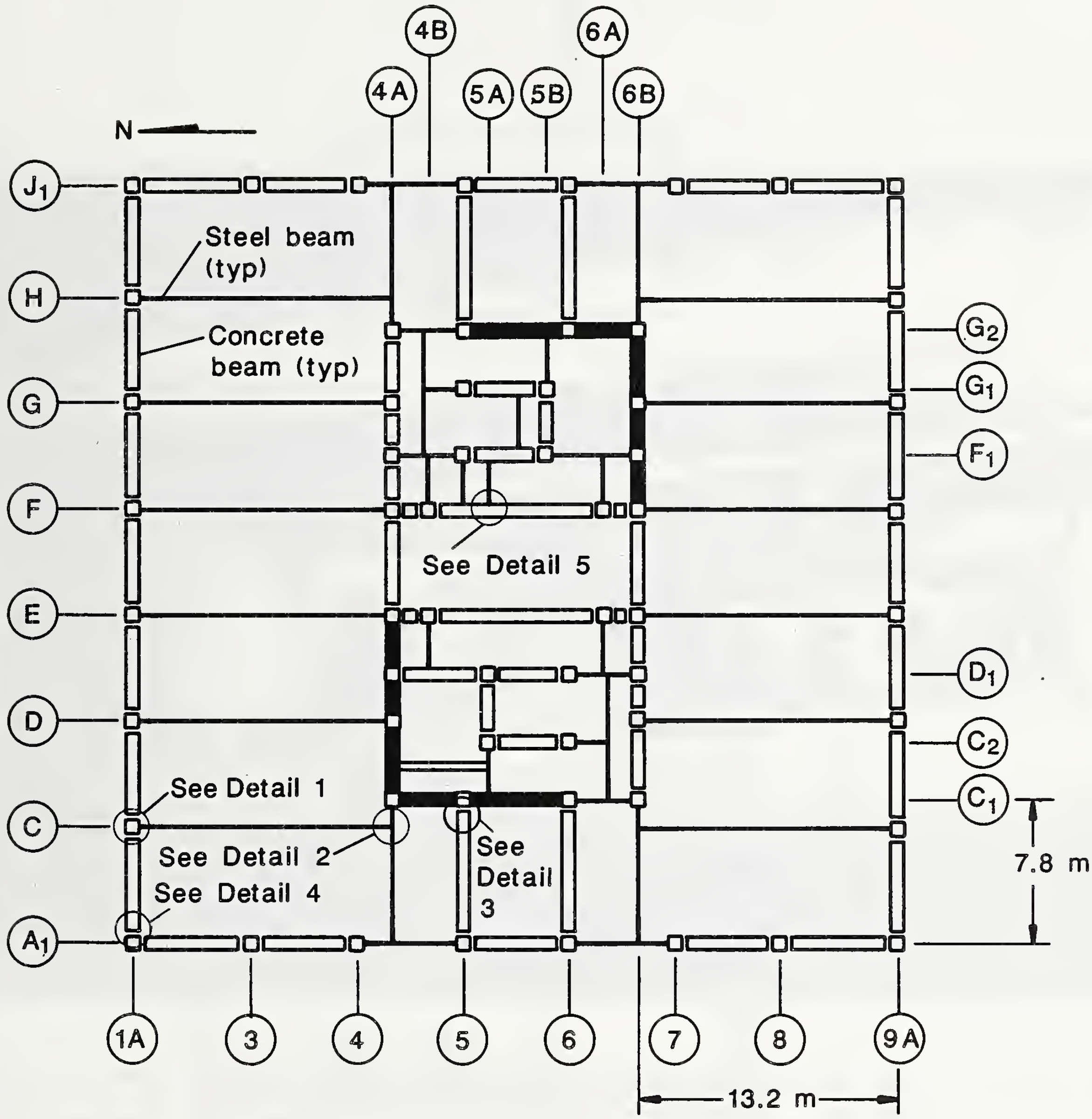
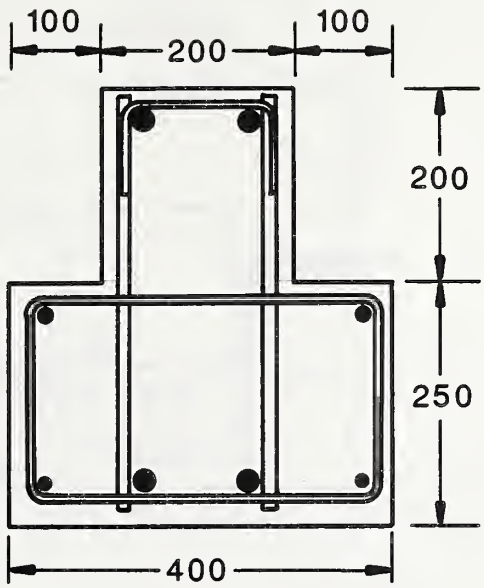


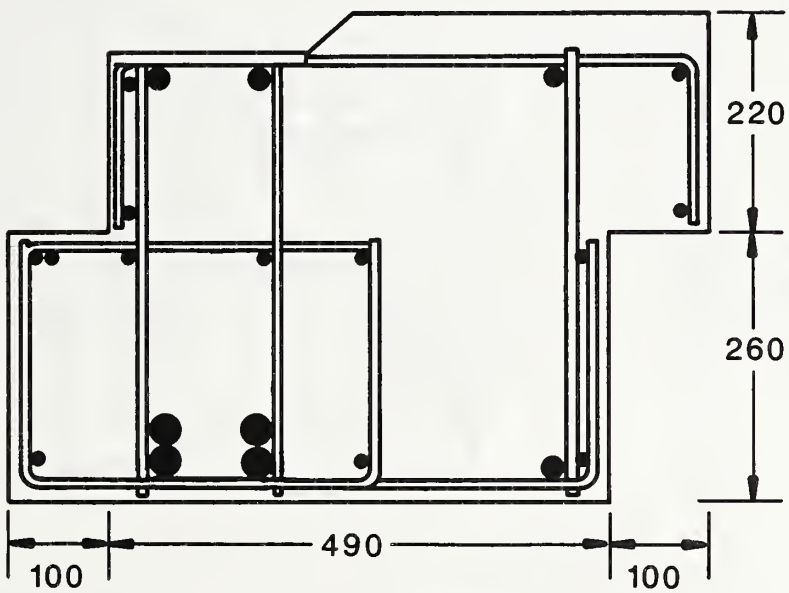
Figure 2.3.10. Plan view showing beam layout of typical floor

Figure 2.3.11. View along east-west direction showing 13.2 m long I-beams spanning between the exterior column line and interior core



(a)

All dimensions in millimeters



(b)

Figure 2.3.12. Precast concrete beam cross-sectional views

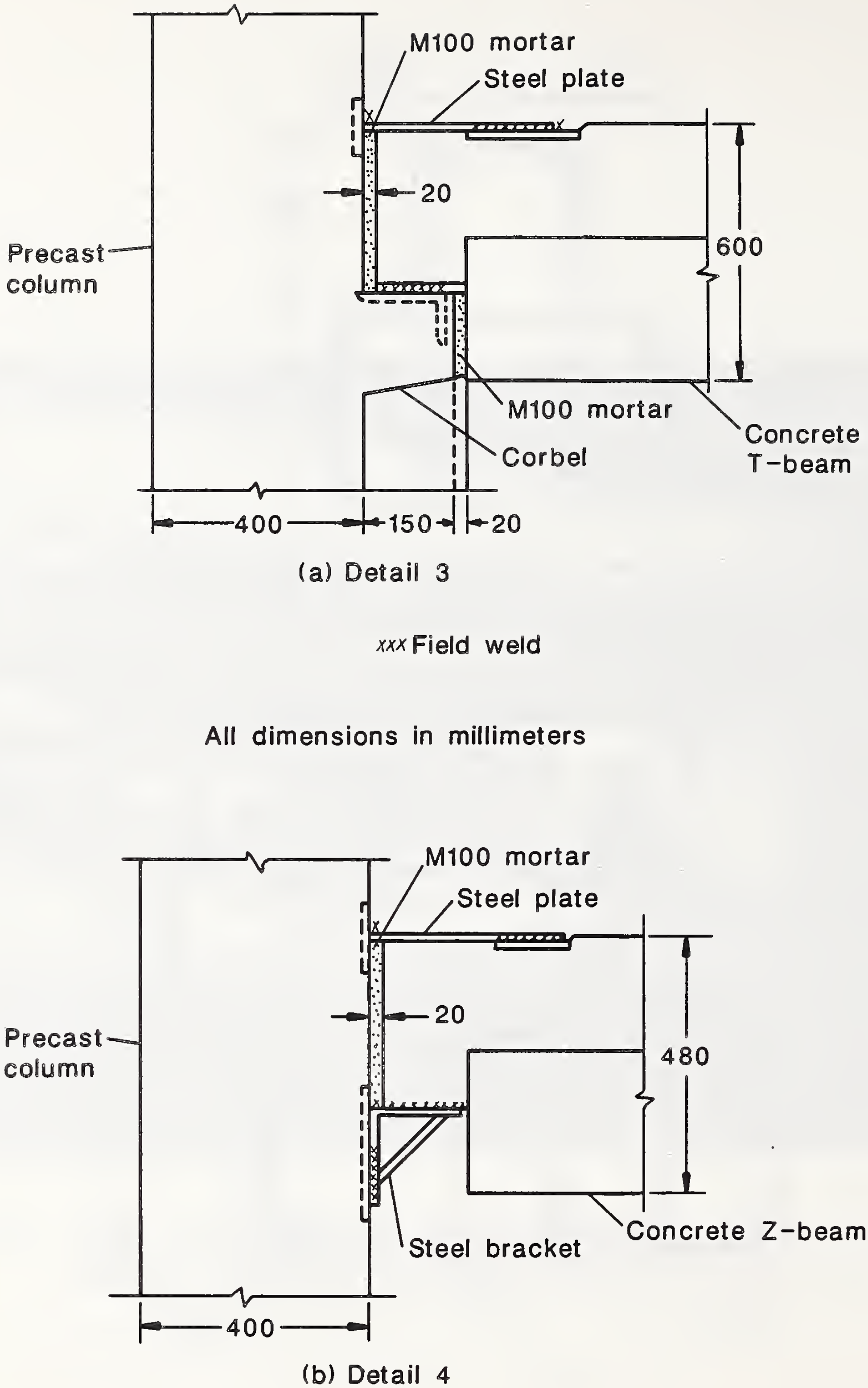


Figure 2.3.13. Reinforced concrete beam to column connections:
a) beam supported by corbel (Detail 3);
b) beam supported by steel bracket (Detail 4)

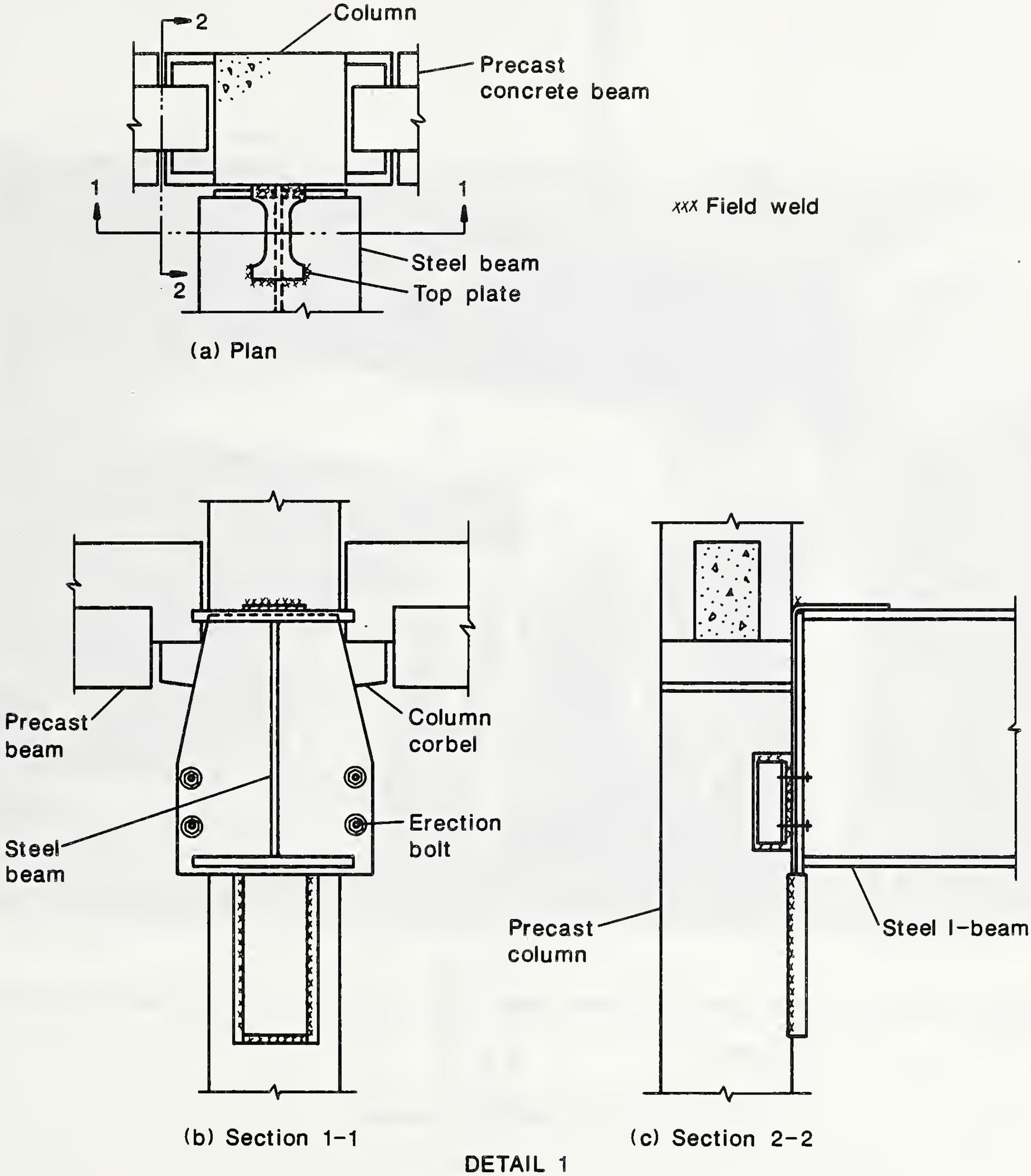


Figure 2.3.14. Steel beam to concrete column connection (Detail 1)

Figure 2.3.15. Example of steel beam to concrete column connection (Detail 1)

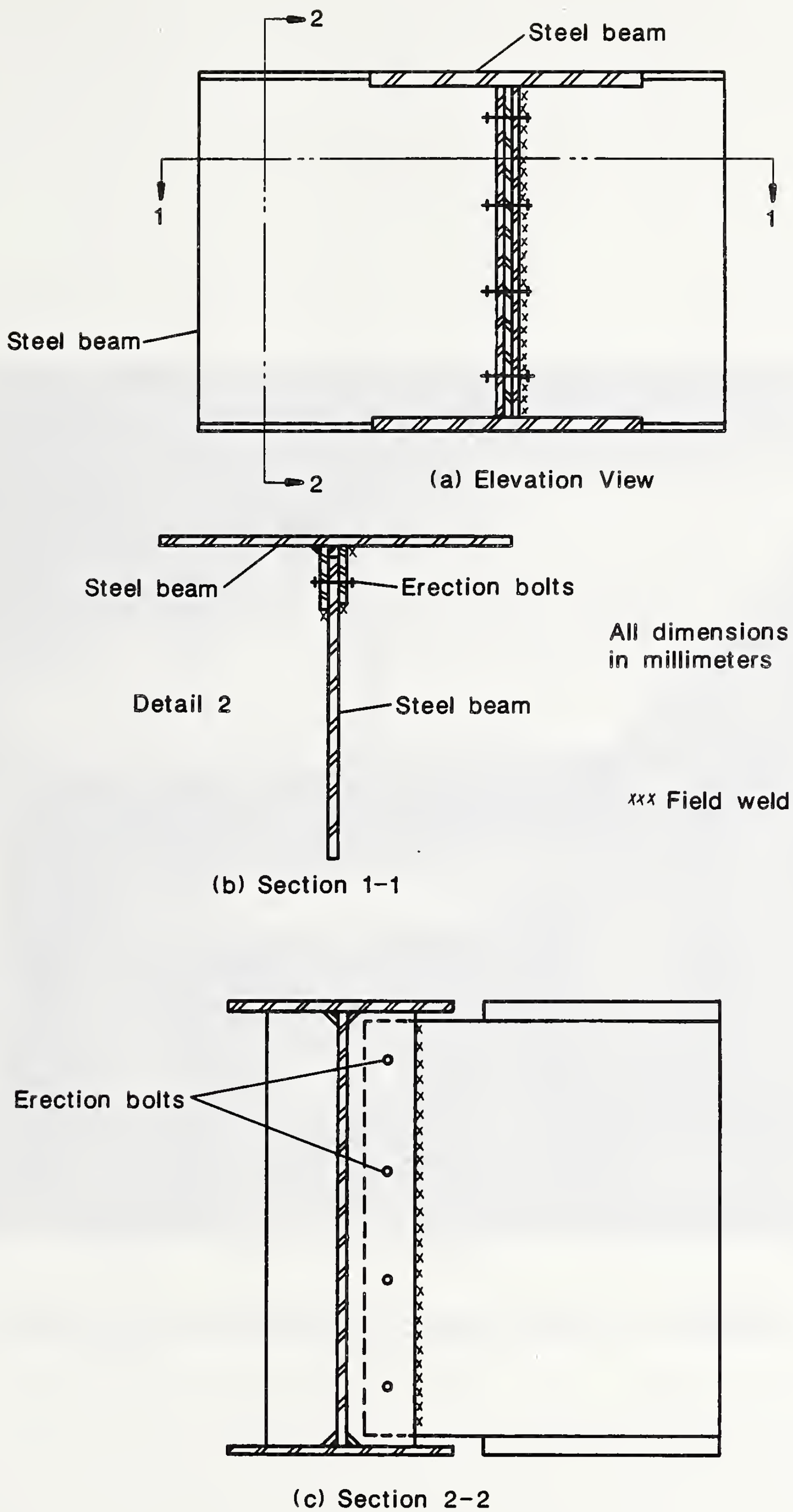
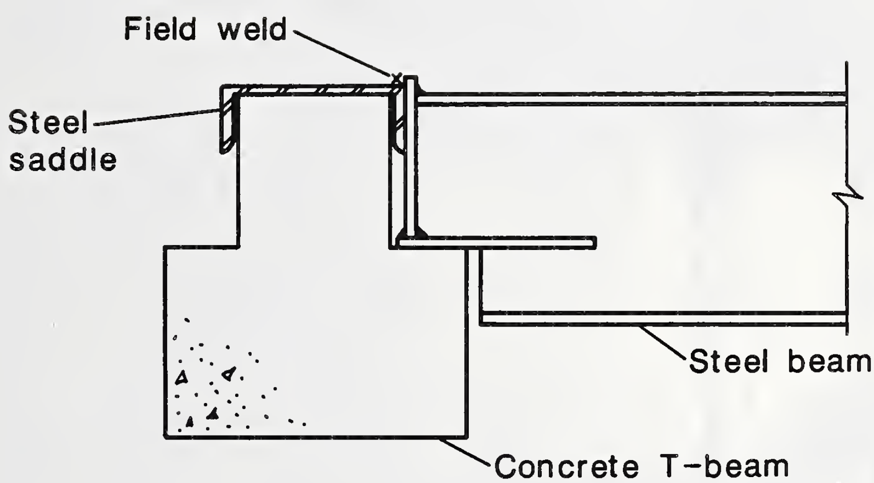


Figure 2.3.16. Steel beam to steel beam connection (Detail 2)

Figure 2.3.17. Example of steel beam to steel beam connection (Detail 2)



Detail 5

Figure 2.3.18. Steel beam to precast concrete beam connection

Figure 2.3.19. View showing shear walls

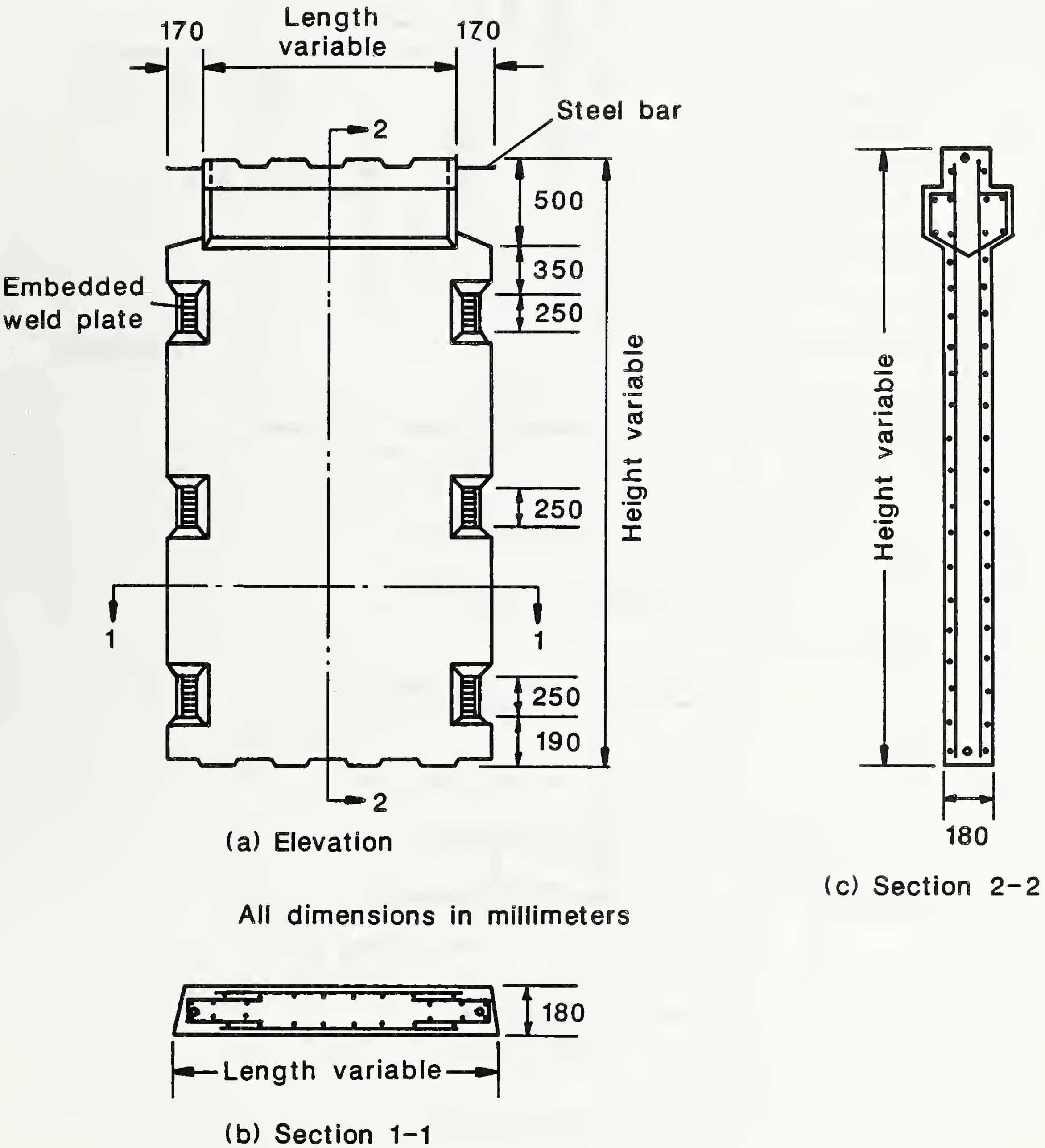


Figure 2.3.20. Shear wall and cross-sectional views showing reinforcement details

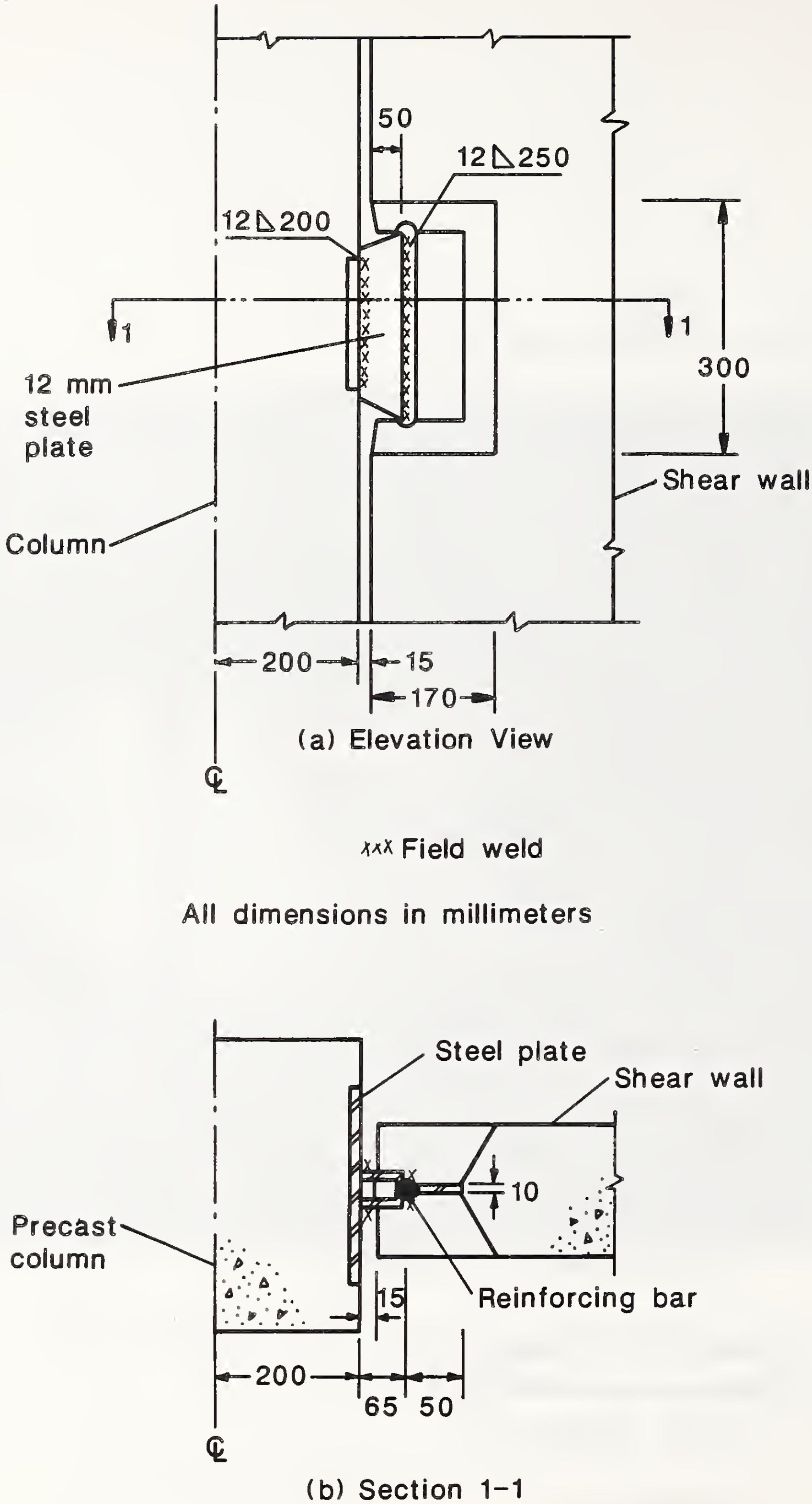
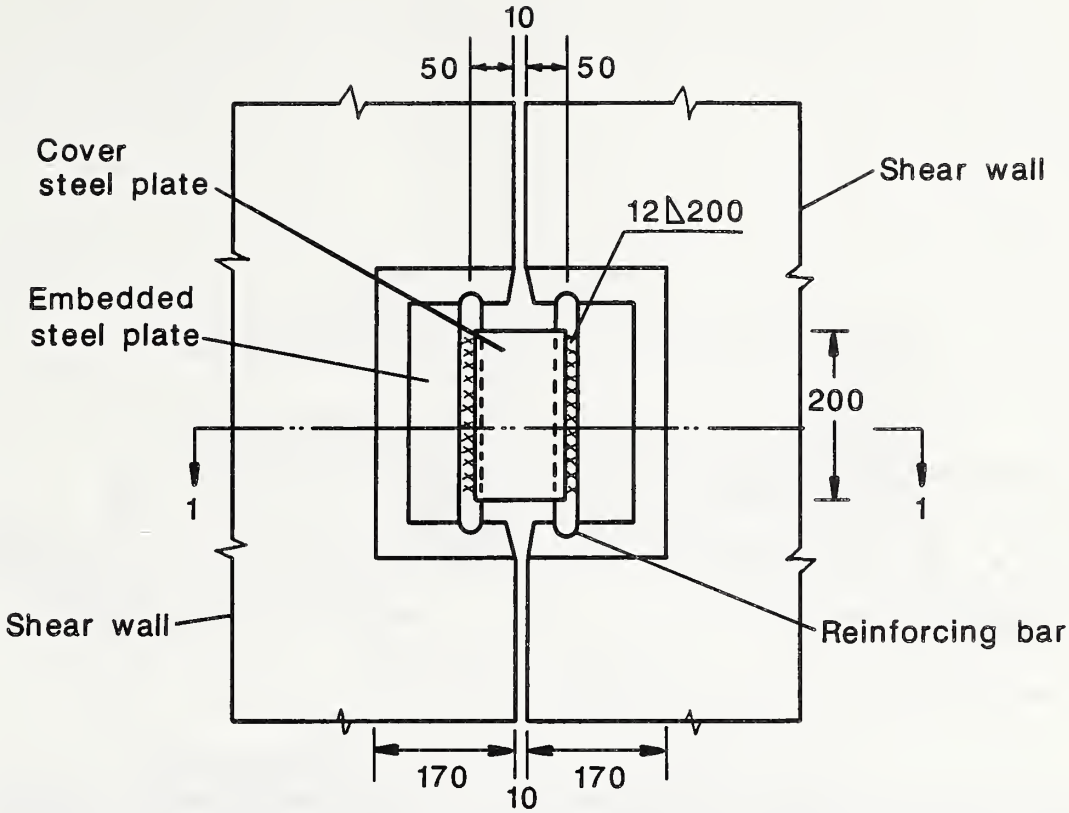


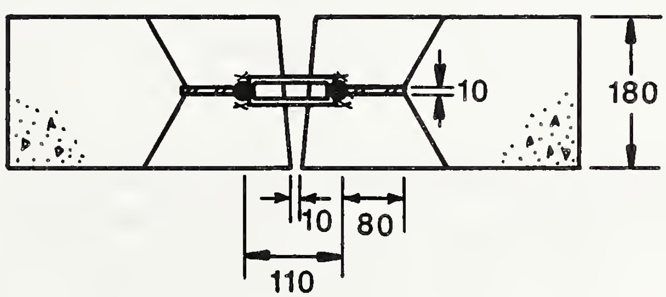
Figure 2.3.21. Shear wall-to-column connection



(a) Elevation

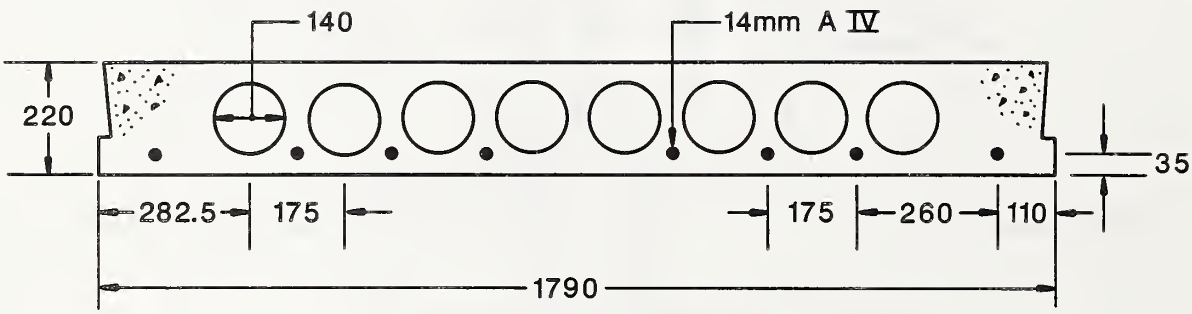
xxx Field weld

All dimensions in millimeters



(b) Section 1-1

Figure 2.3.22. Shear wall-to-shear wall connection



Plank NV-52-18T:

Mark 200 Concrete

Length = 5160

Span length = 5080

Prestress = 3600±950 kg/cm^2

Yield strength = 6000 kg/cm^2

All dimensions in millimeters

Figure 2.3.23. Cross-section of prestressed, hollow core plank

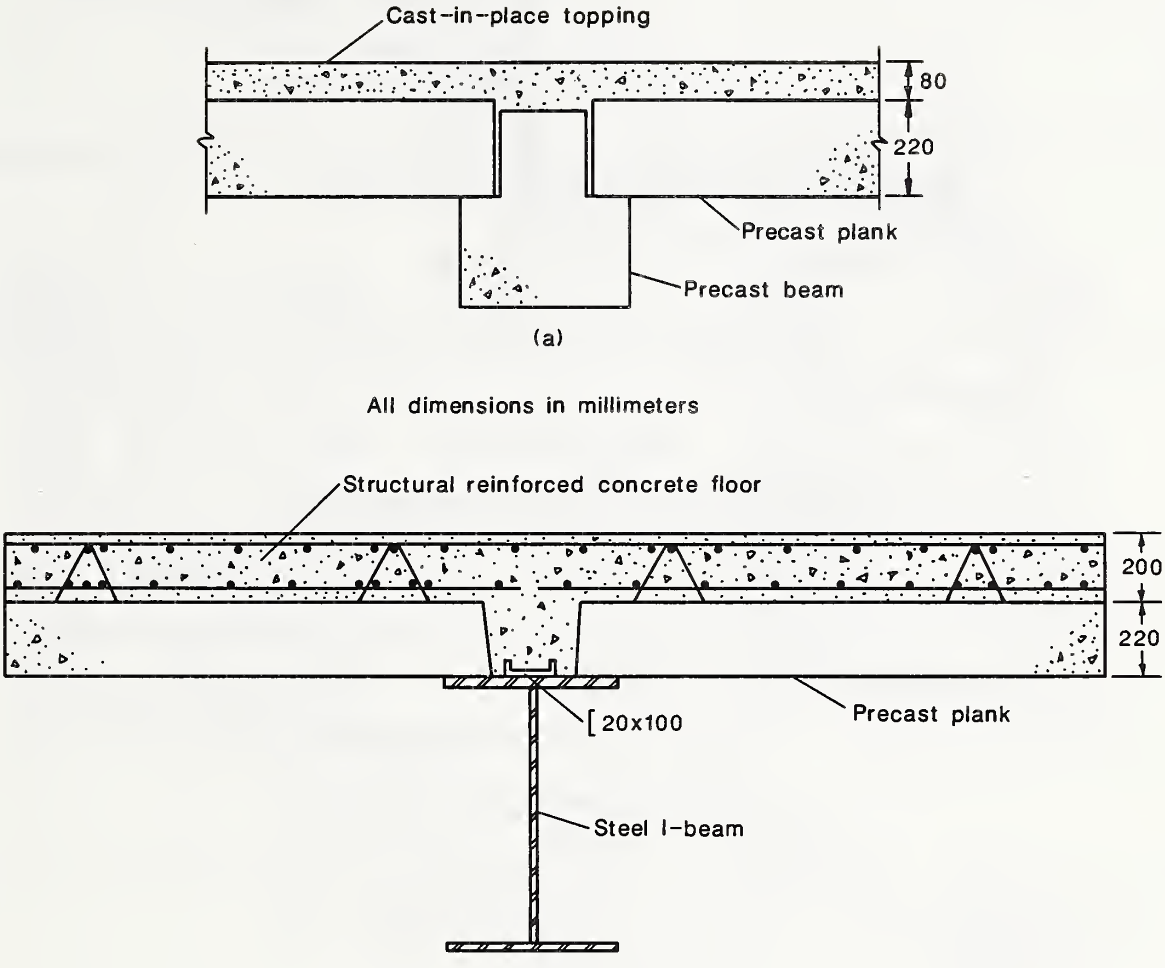


Figure 2.3.24. a) Precast beam supporting planks and topping; and
b) Steel beam supporting planks and reinforced concrete slab on eighth floor

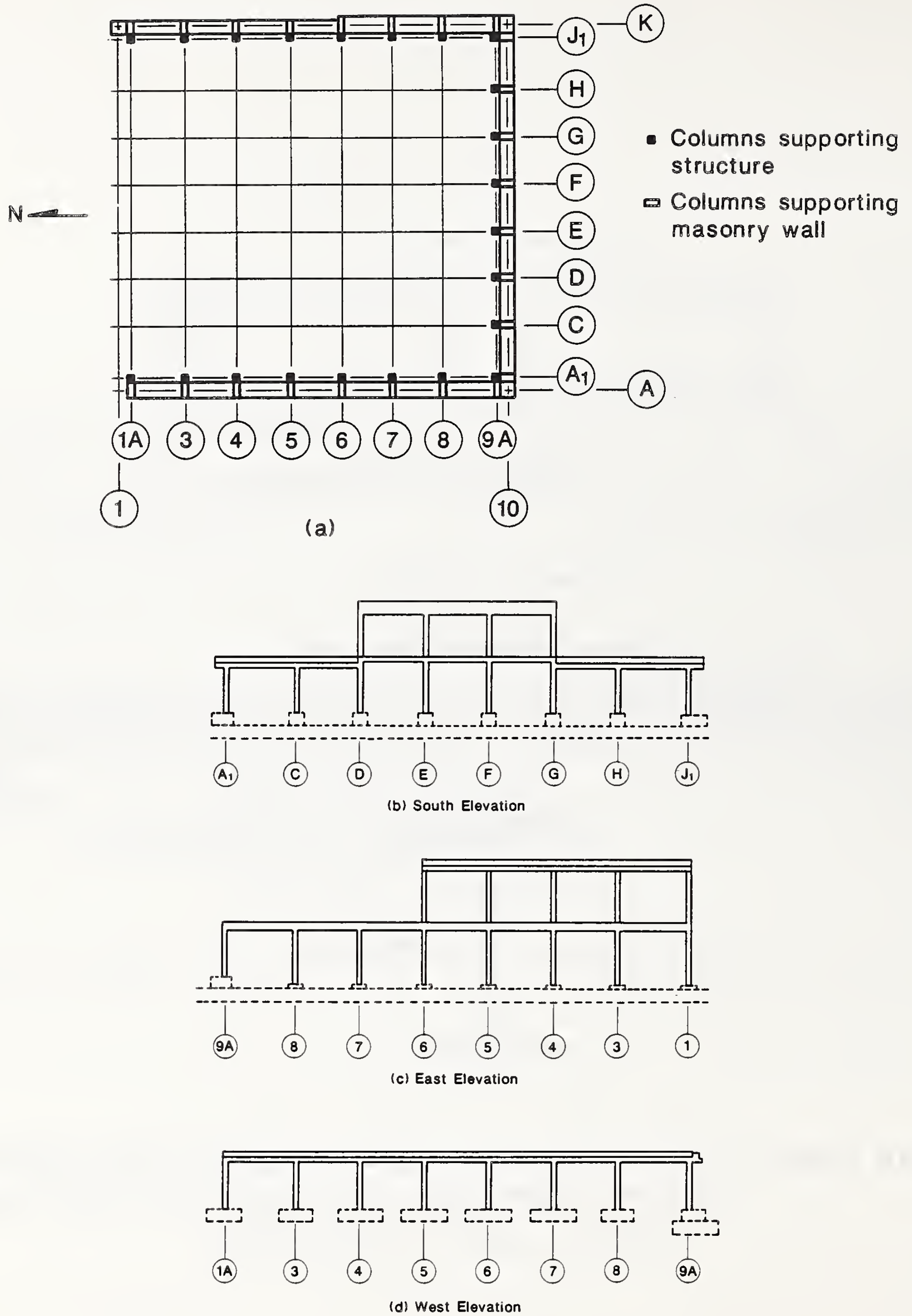


Figure 2.4.1. Structural frame supporting masonry walls; a) plan of supporting columns; b) south elevation; c) east elevation; and, d) west elevation

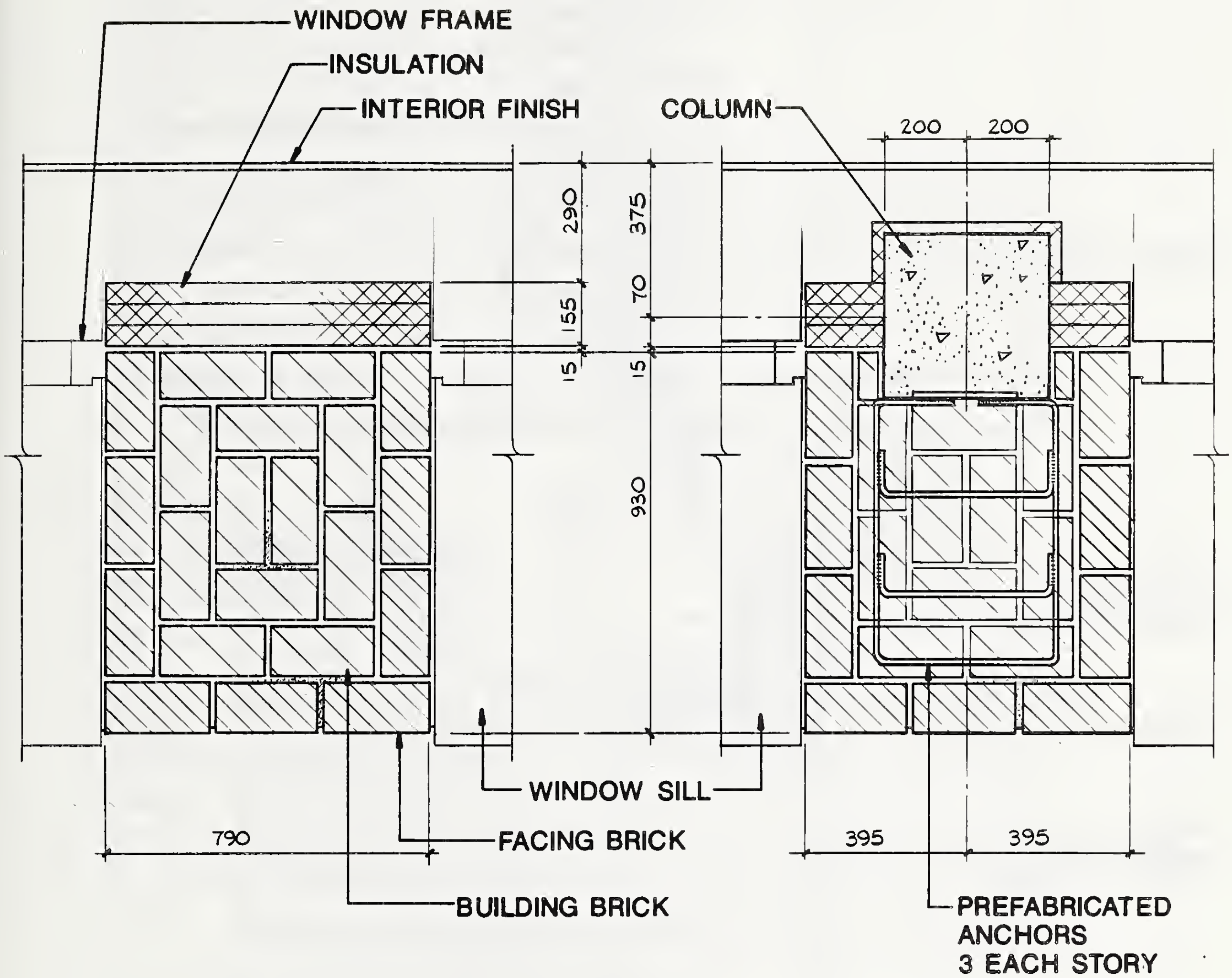


Figure 2.4.2. Plan of typical piers at and between columns

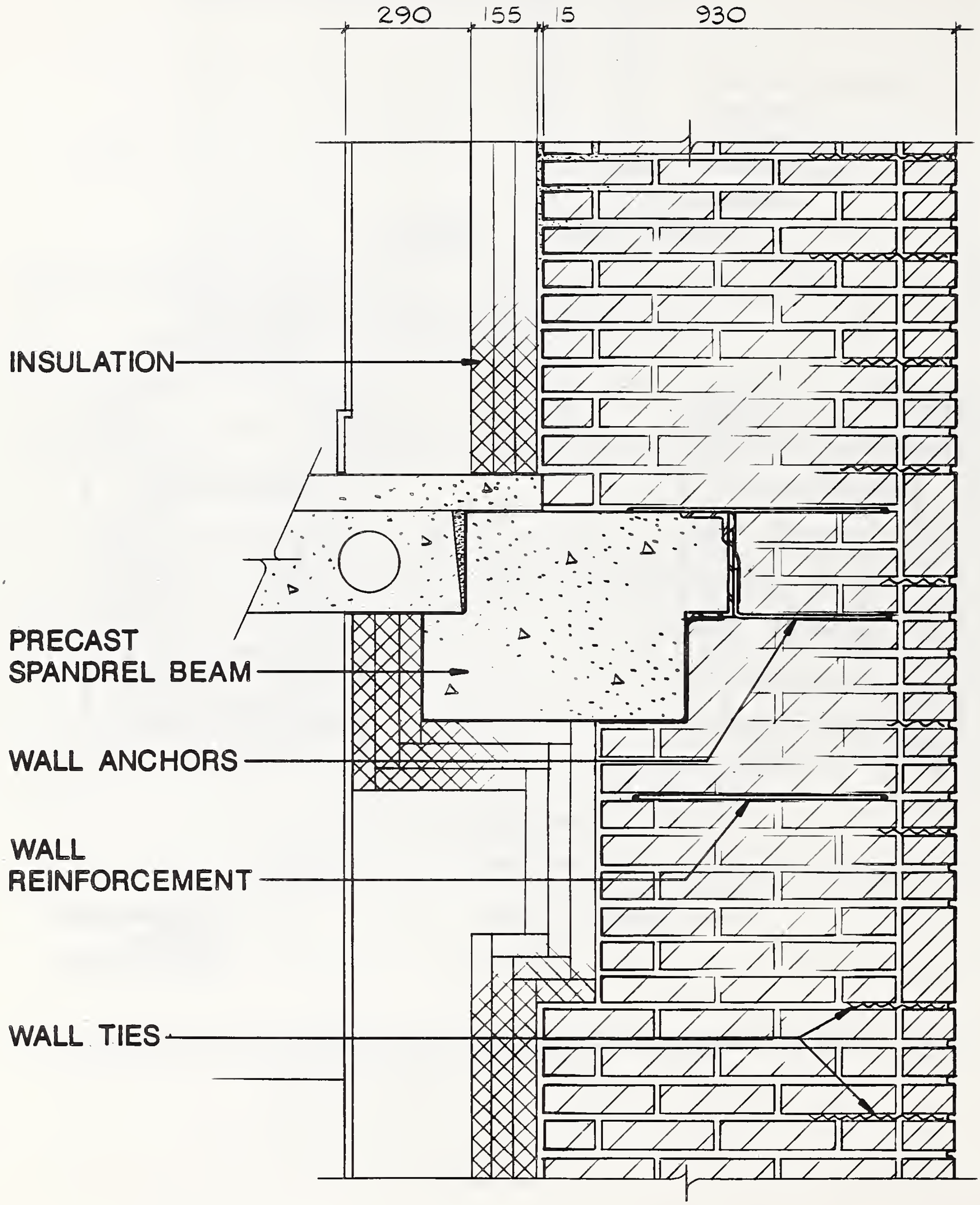


Figure 2.4.3. Typical pier and spandrel section

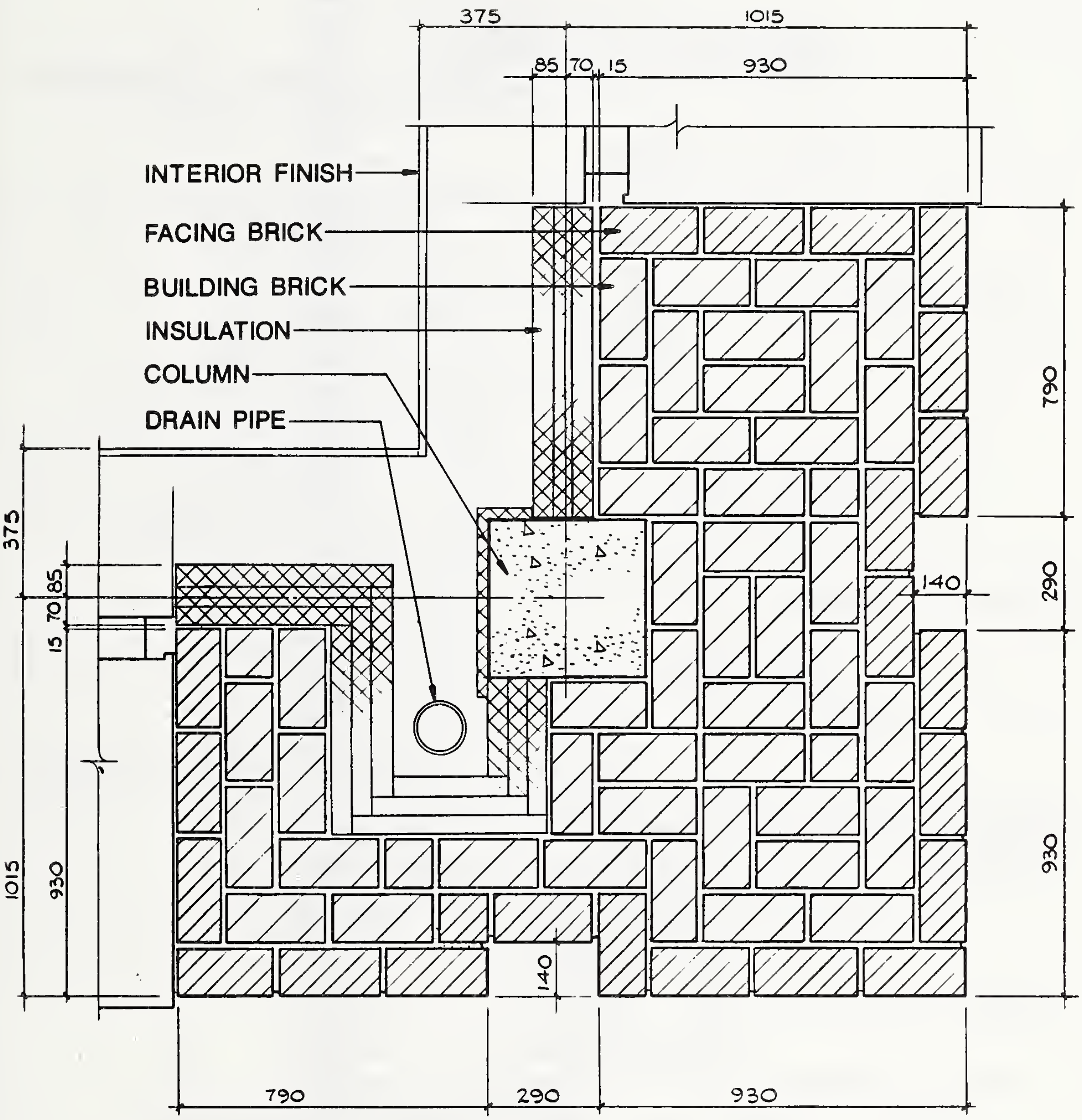


Figure 2.4.4. Corner pier plan

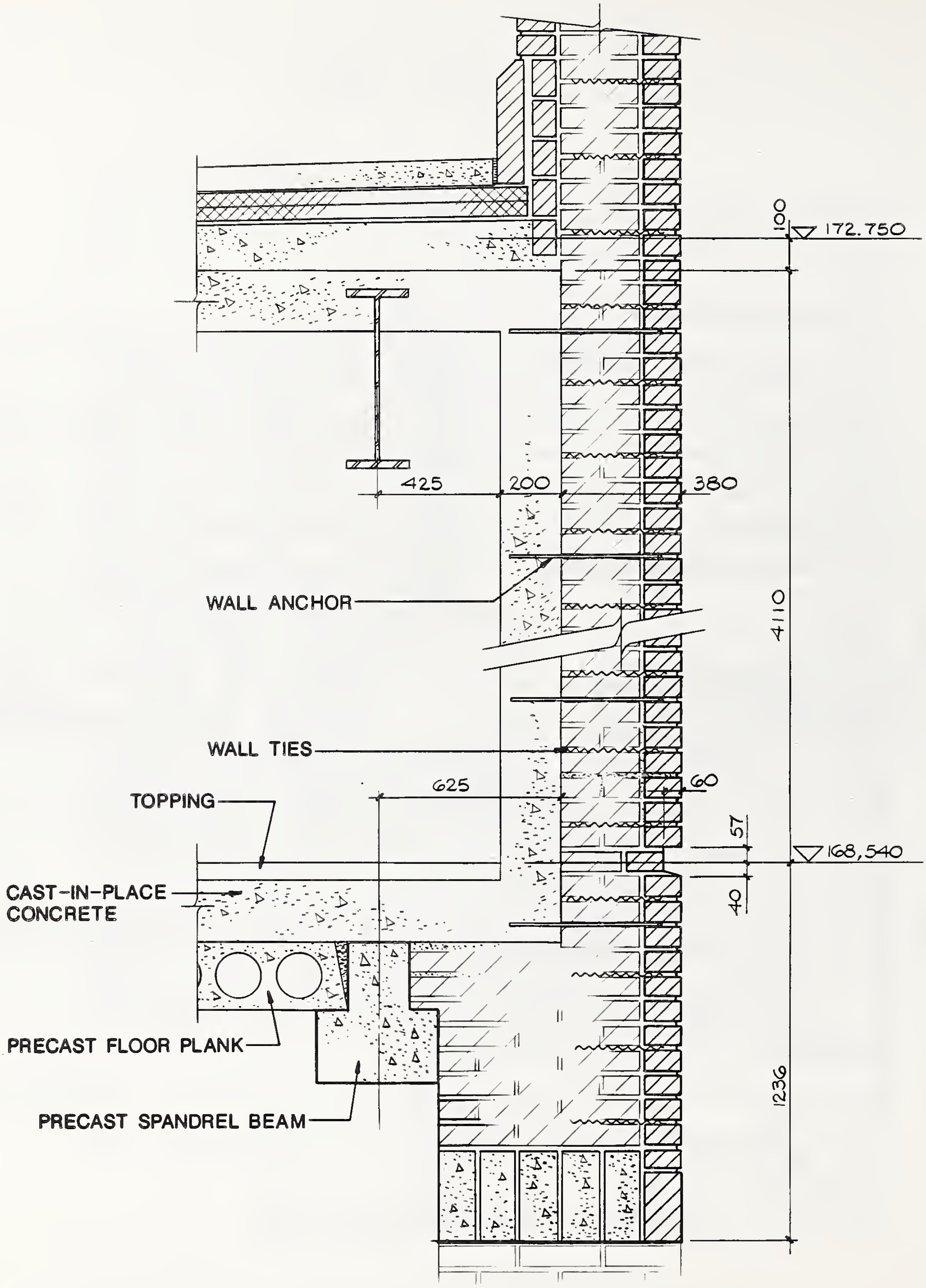


Figure 2.4.5. Eighth story wall section

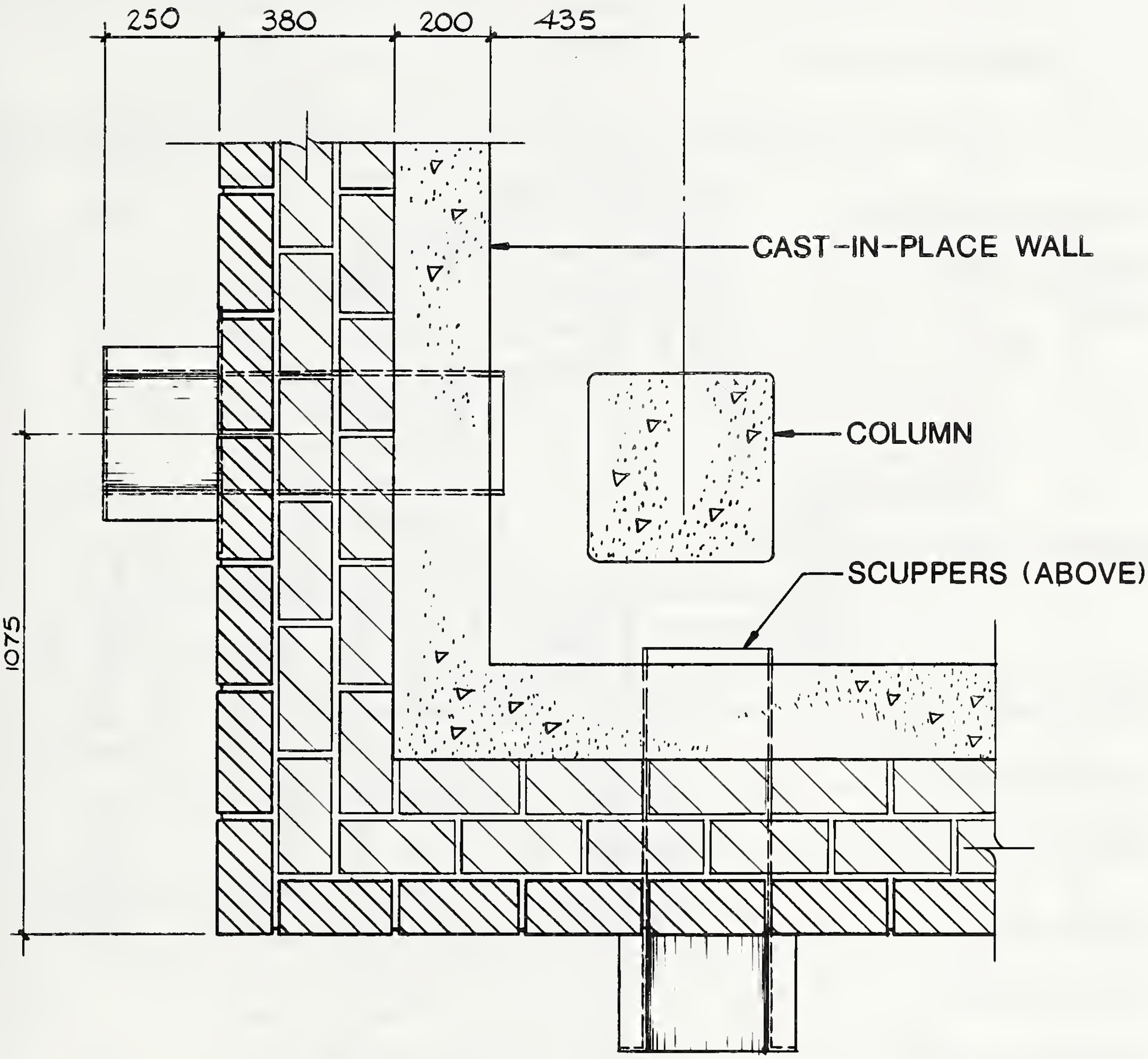


Figure 2.4.6. Plan of eighth story wall at corner

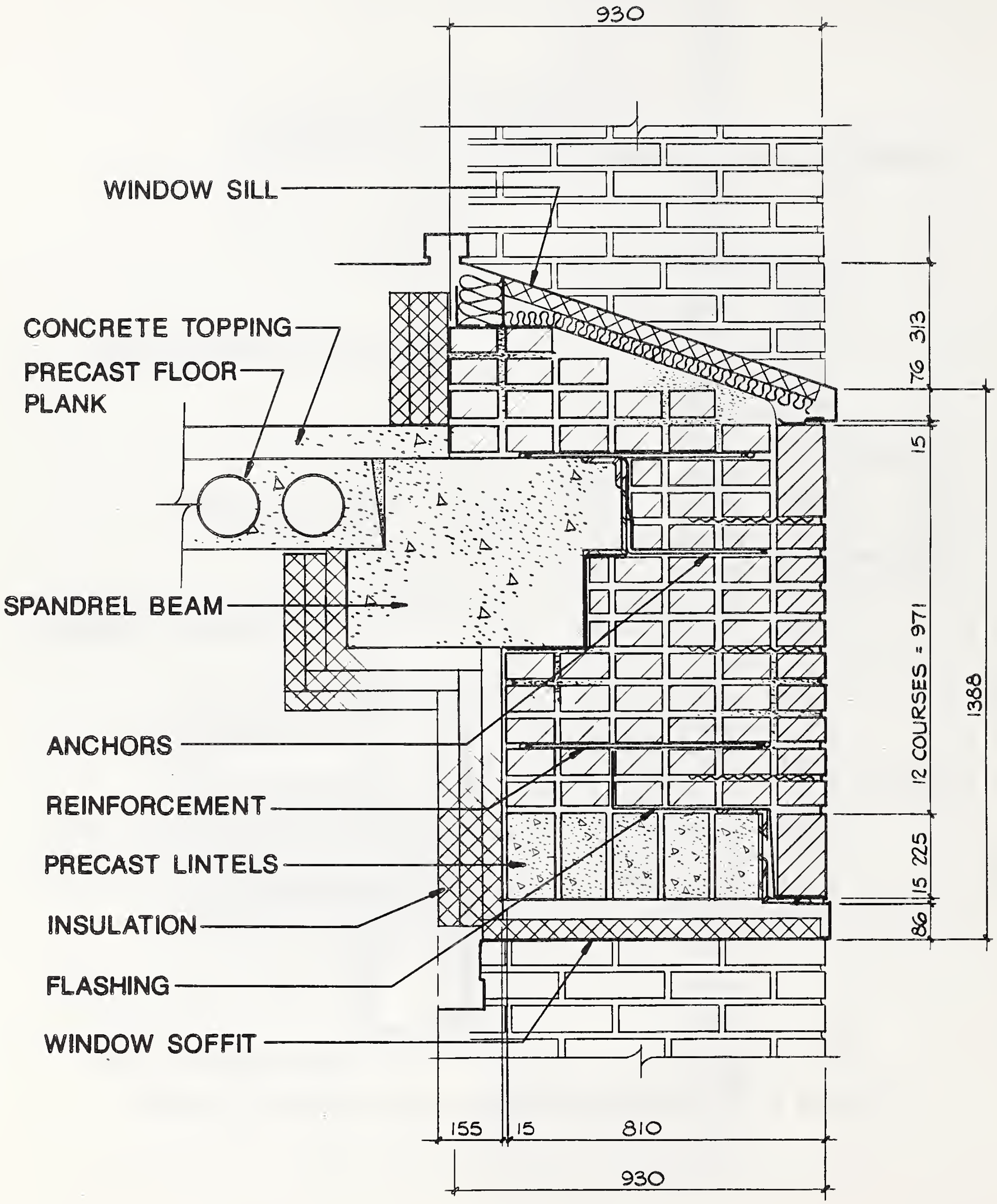


Figure 2.4.7. Typical window head and sill section

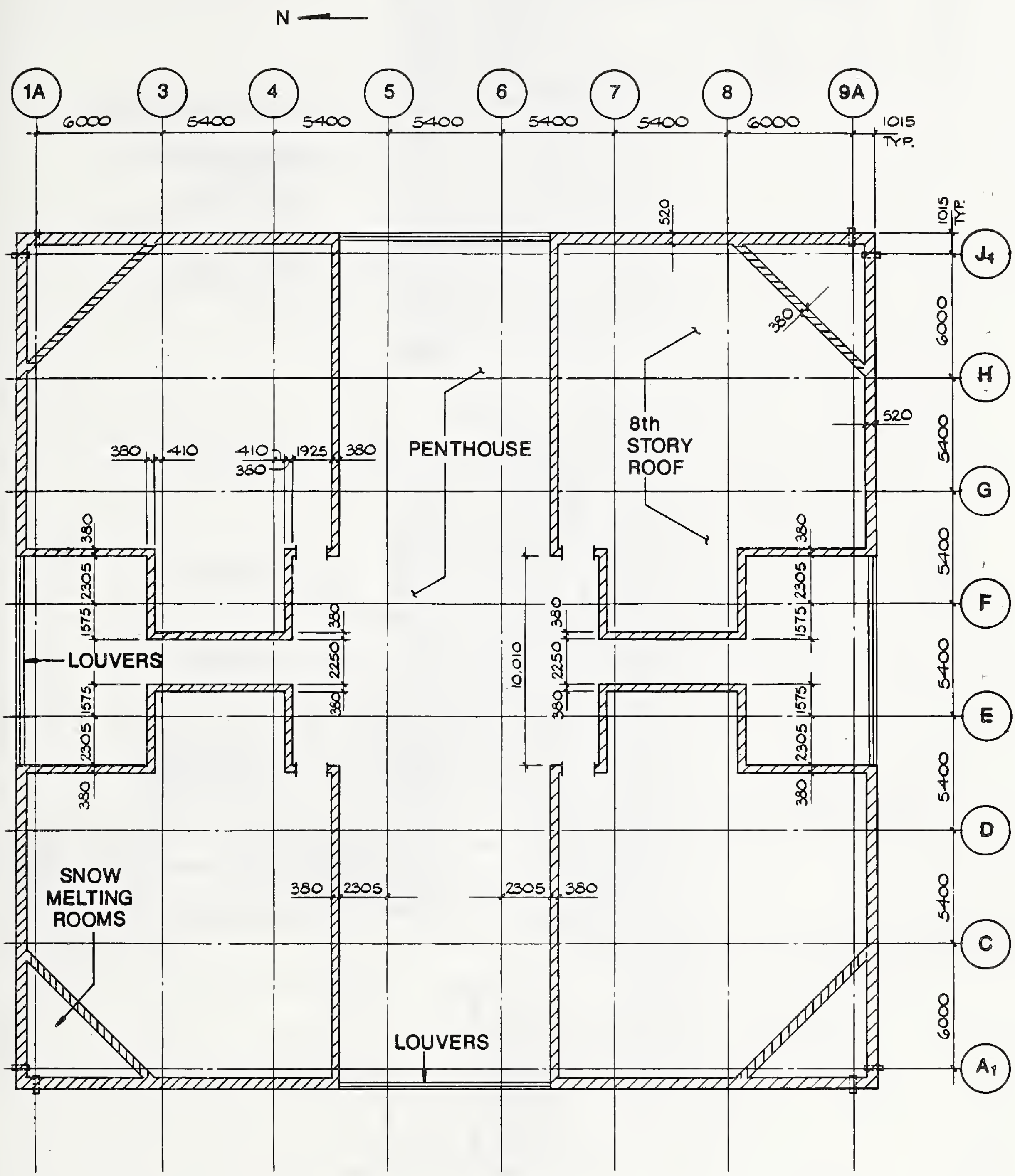


Figure 2.4.8. Plan showing penthouse and parapet wall

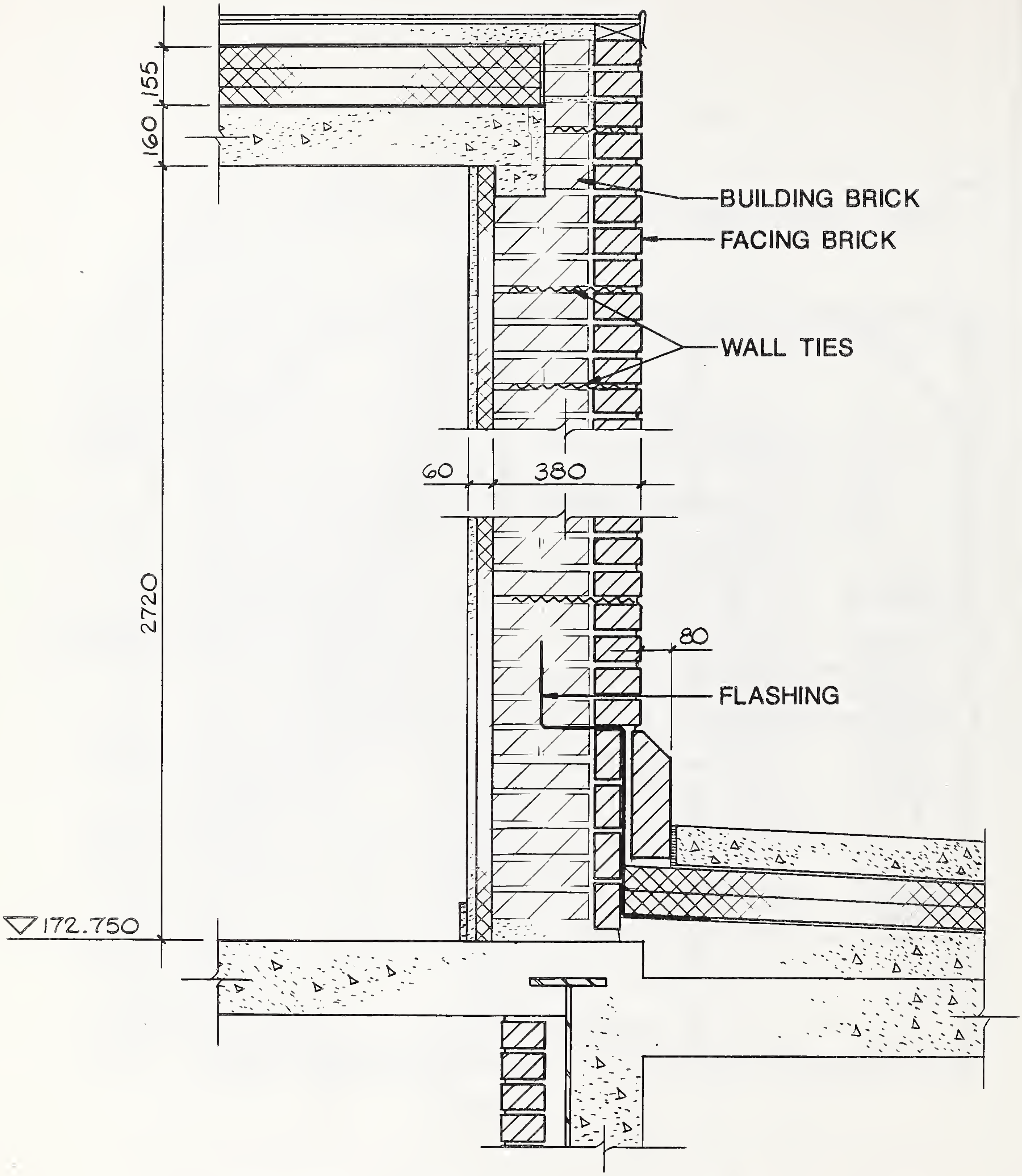


Figure 2.4.9. Penthouse wall section

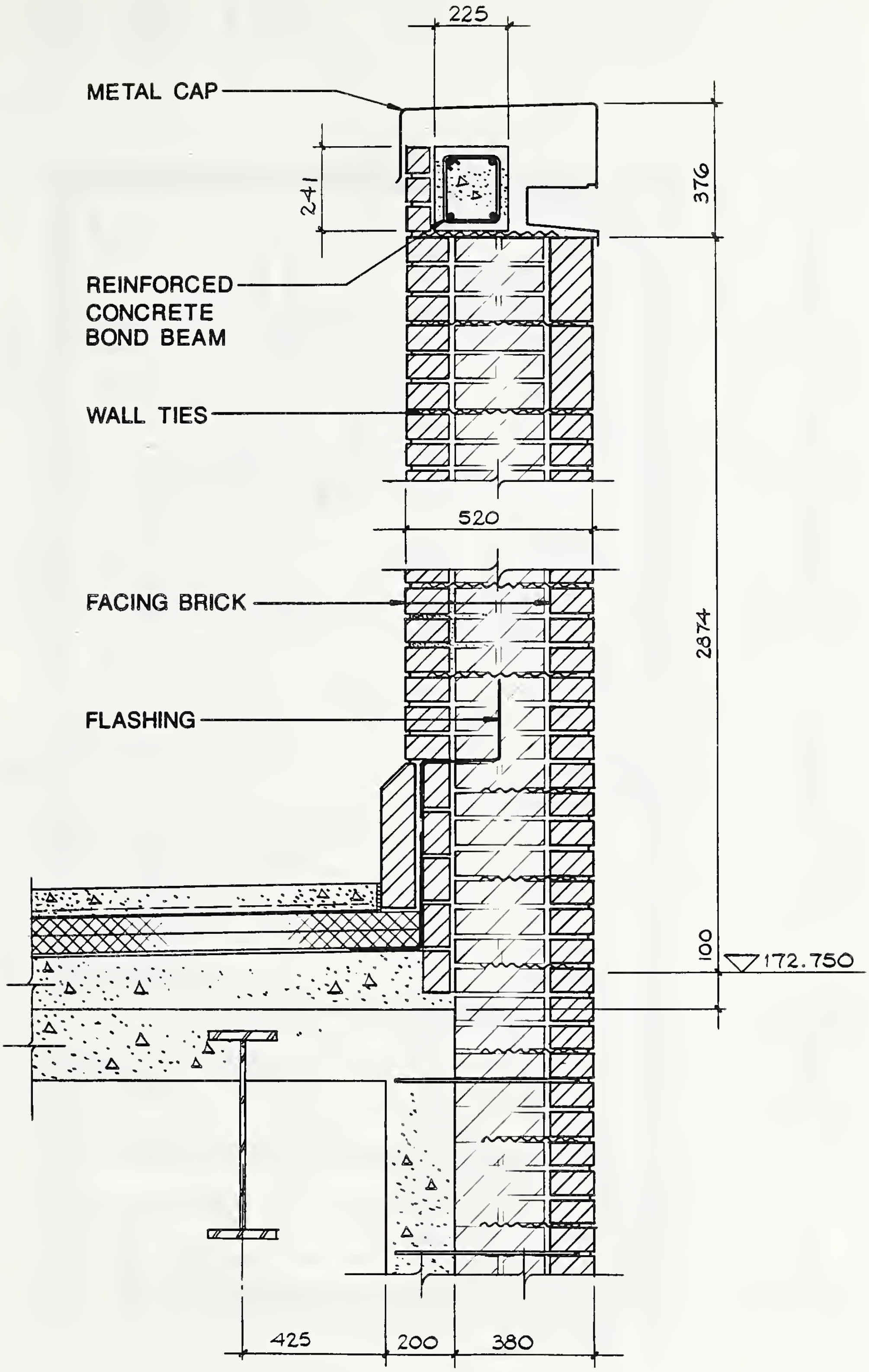


Figure 2.4.10. Parapet wall section

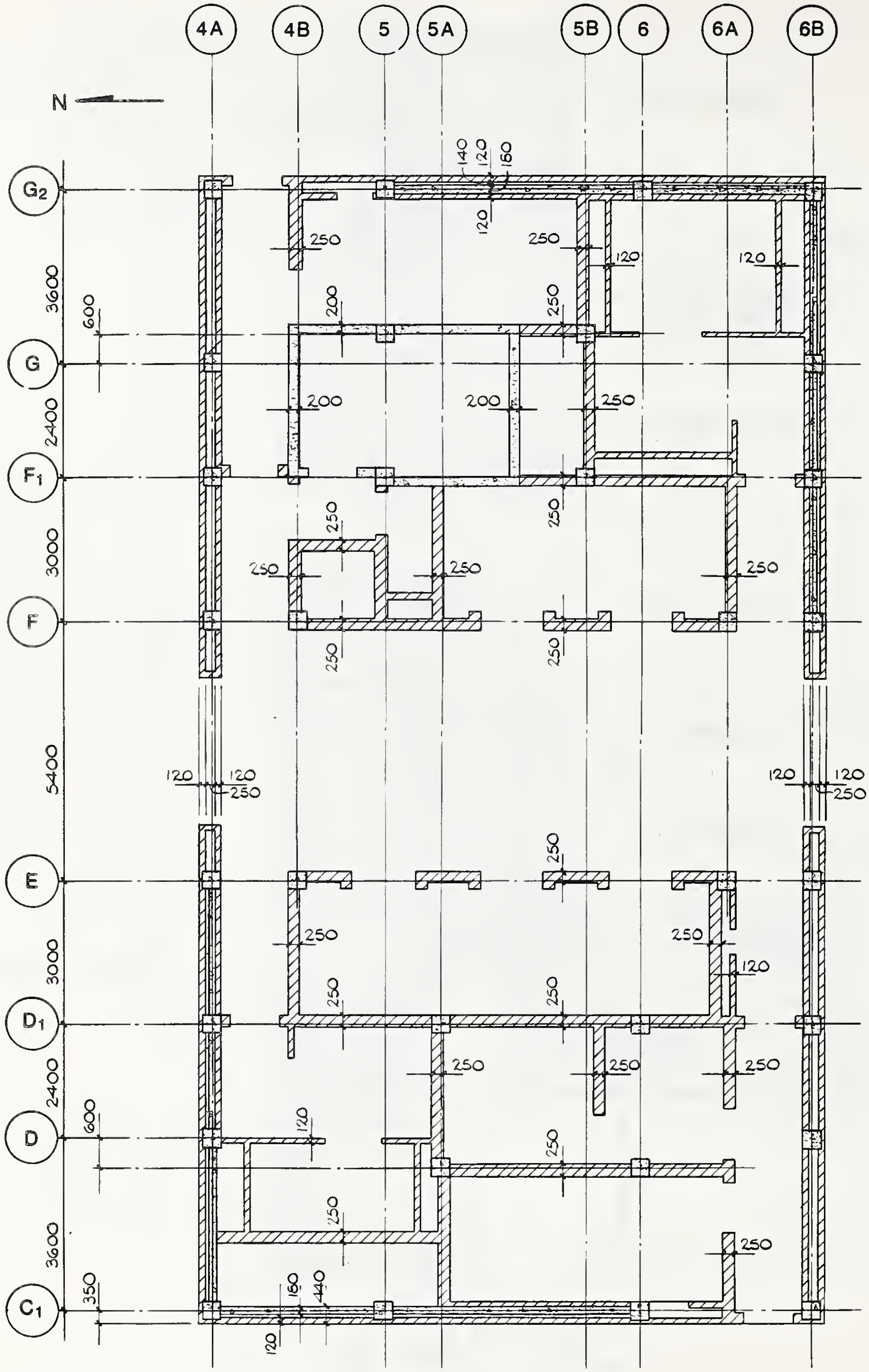


Figure 2.4.11. Plan of the core showing masonry walls

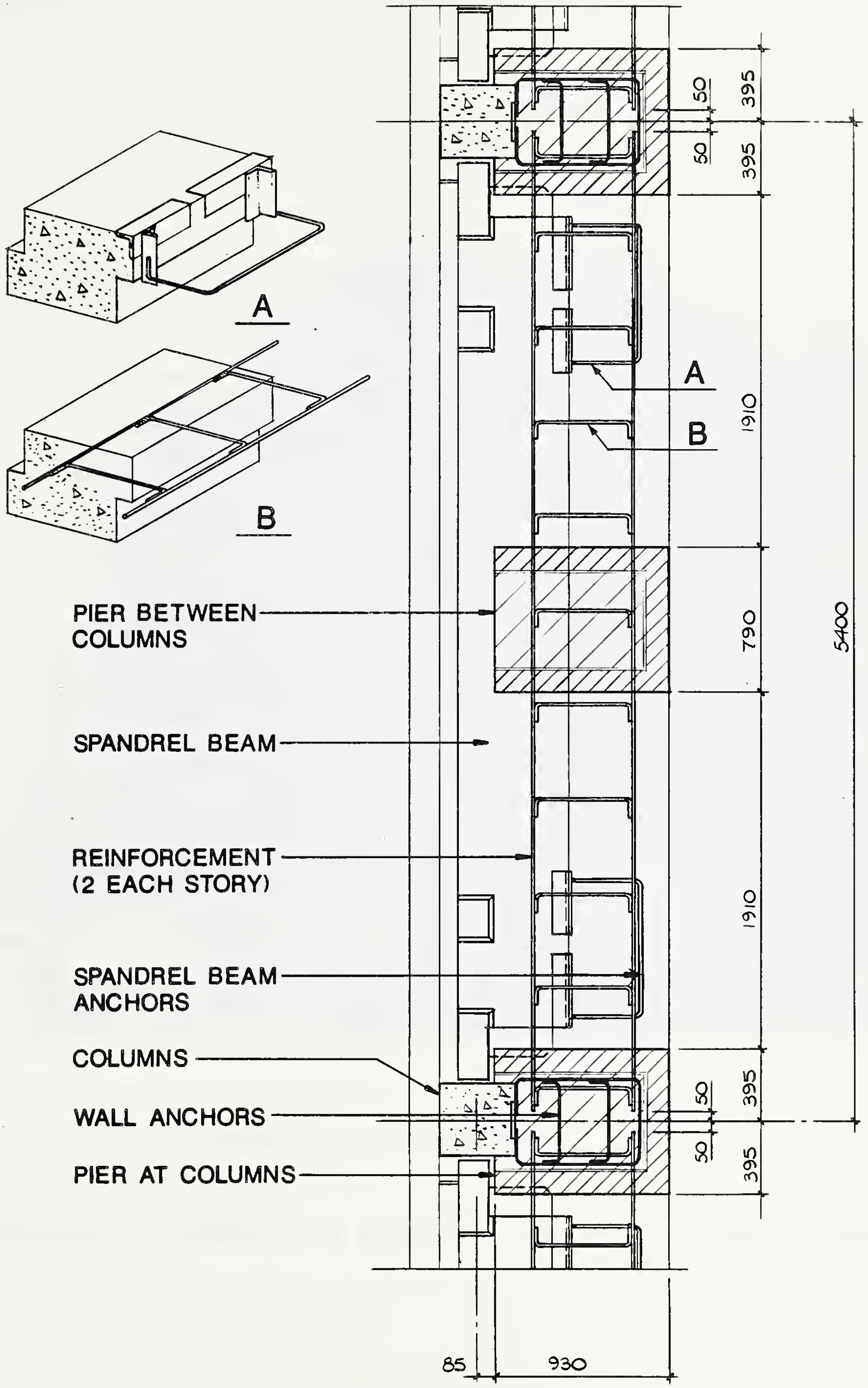


Figure 2.4.12. Anchorage of walls to concrete spandrel beams

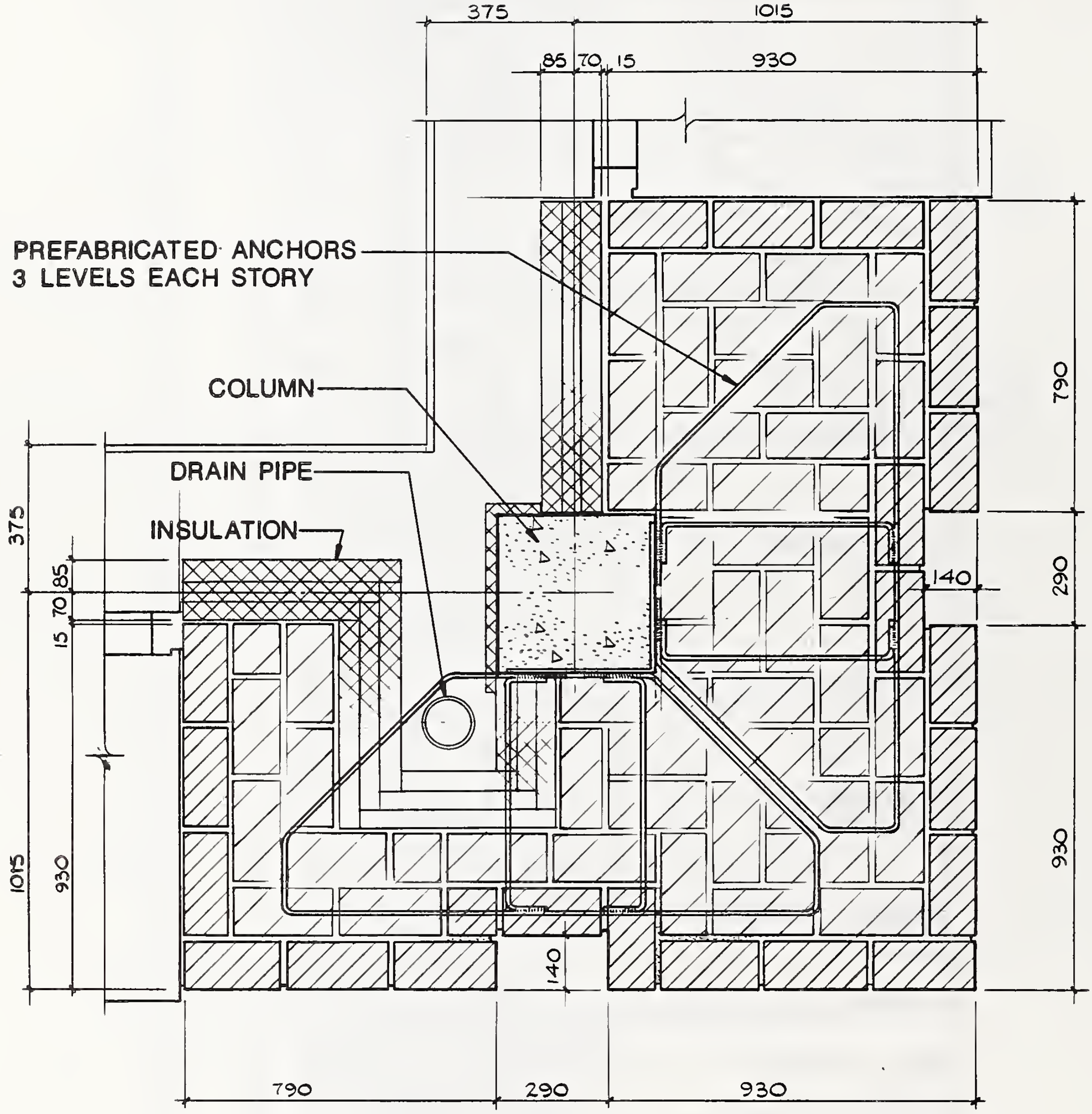


Figure 2.4.13. Anchorage of walls to columns at corners

Figure 2.5.1. Photograph showing a view of the Office Building from the southwest in December 1986

CHAPTER 3
LOADING AND RESISTANCE CRITERIA

3.1 INTRODUCTION

This chapter presents the loading and resistance criteria used to assess the structural integrity of the Office Building and to specify recommended remedial measures. The criteria were selected:

(1) To provide a level of safety consistent with good practice for U.S. office buildings.

(2) To meet criteria specific to the use of this structure as a U.S. Embassy office building.

(3) To be consistent with local environmental and site conditions.

(4) To account for the characteristics of the Soviet structural system (which differ from those encountered in the United States) and the strength and other material properties used in the construction of the Office Building.

The Office Building is not assessed in light of Soviet design criteria and accepted Soviet construction practices. The former are not necessarily relevant to U.S. requirements, the later are neither necessarily relevant nor were they available to the investigators.

Good practice for U.S. office buildings is defined as the practice incorporated in the provisions of the following documents:

For loads: American National Standard ANSI A58.1-1982, "Minimum Design Loads for Buildings and Other Structures" (ANSI Standard A58.1) [3.1]. The provisions of ANSI Standard A58.1-1982 were developed on the basis of professional consensus on good practice, and are widely referenced in U.S. building codes.

For resistance of reinforced concrete members: ACI Standard 318-83, "Building Code Requirements for Reinforced Concrete" [3.2].

For resistance of structural steel members: "Load and Resistance Factor Design," American Institute of Steel Construction, First Edition, 1986 [3.3].

For resistance of brick masonry: "Building Code Requirements for Engineered Brick Masonry," Brick Institute of America, 1969 [3.4].

For allowable loads on piles: "Uniform Building Code" [3.5], 1985 Edition, a model building code widely used in the United States for determining such loads.

Special requirements related to the use of the structure as a U.S. Embassy office building are taken from "Engineering Design Criteria" [3.6] issued by the Department of State, Office of Foreign Buildings Operations (FBO), in April 1968. This was the standard set of criteria in force at the time the Office Building was designed. Subsequent documents issued by the State Department include "Planning Procedures and Engineering Criteria" [3.7], October 1983 and "Architectural and Engineering Design Guidelines and Criteria for New Embassy Buildings" [3.8], November 1986. The criteria developed in this Chapter are in substantial agreement with requirements in the 1968 and 1983 documents. The 1986 criteria are considerably more stringent than those of the 1968 and 1983 documents. Since the application of the 1986 criteria to the Office Building and its site has not been' required by the State Department, they were not used in the assessment presented in this Report.

For consistency with local environmental and site conditions, information was obtained from relevant Soviet documents, including Soviet standard SNiP II-6-74 "Construction Standards and Regulations, Part II Design Standards, Chapter 6, Loads and Actions" [3.9]; and from Soviet standard SNiP II-A.12-69*, "Construction Standards and Regulations, Part II Design Standards, Chapter 12, Construction in Seismic Regions" [3.10]. Information on the seismicity of the Moscow region was also obtained from the 1983 State Department Planning Procedures and Engineering Criteria and from the U.S. Geological Survey.

To account for the characteristics of the Soviet structural system (including dead loads of various elements and components), and for the strength and other material properties used in the construction of the Office Building, information was obtained from relevant Soviet standards, from the Soviet working structural drawings, and from the Soviet calculation notes on vertical loads dated September 25, 1979.

This chapter reviews loading and resistance criteria set forth in the various documents listed above; selects, with supporting rationales, the loading and resistance criteria adopted as a basis for assessing the structural integrity of the Office Building and for specifying recommended remedial measures; and examines and selects criteria for reducing the risk of progressive collapse, in accordance with the requirements of ANSI Standard A58.1. The discussion of loading criteria is divided into (1) floor loads (gravity loads), including dead and live loads and (2) environmental loads, including wind, snow, and earthquake loads. The discussion of resistance criteria is divided into (1) resistance of reinforced concrete members and (2) resistance of steel members and connections.

3.2 FLOOR LOADING CRITERIA

This section includes a review of floor loading criteria as excerpted from (1) ANSI Standard A58.1-1982, (2) the 1968 State Department Engineering Design Criteria, and (3) the Soviet calculation notes on vertical loads dated September 25, 1979.

3.2.1 ANSI Standard A58.1-1982

With respect to dead loads, ANSI Standard A58.1 requires that "in estimating dead loads for purposes of design, the actual weights of materials and constructions shall be used" [3.1, section 3.1, p. 10]. Dead loads conforming to this requirement are listed in the Soviet calculation notes reviewed in section 3.2.3 of this report.

With respect to live loads, the ANSI Standard A58.1 requirement for offices is 50 psf (244 kg/m^2); for lobbies it is 100 psf (487 kg/m^2); and for library stack rooms it is 150 psf (731 kg/m^2) [3.1, table 2, p.25]. ANSI Standard A58.1 requires live loads in excess of 150 psf (731 kg/m^2) only for: armories and drill rooms, heavy manufacturing areas, areas subjected to trucking, and heavy storage warehouses.

Section 4.7 of ANSI Standard A58.1 permits live load reductions for members having an influence area of 400 ft^2 or more, in accordance with the formula:

$$L = L_o[0.25 + 15/\sqrt{A_I}] \tag{3.2.1}$$

where L = reduced live load, L_o = unreduced live load, and A_I = influence area, in square feet.

The influence area is four times the tributary area for a column, two times the tributary area for a beam, and is equal to the panel area for a two-way slab. The reduced design live load shall be no less than 50 percent of the unreduced live load for members supporting one floor nor less than 40 percent of that load otherwise. For live loads of 100 psf or less, no reduction is allowed for areas to be occupied as places of public assembly, for one-way slabs, or for roofs. For live loads that exceed 100 psf, design live loads on members supporting more than one floor shall be reduced 20 percent, but live loads in other cases shall not be reduced except as permitted by the authority having jurisdiction.

3.2.2 1968 Engineering Design Criteria

According to the Engineering Design Criteria issued by the State Department [3.6], specified live loads in office buildings are as follows:

Office space	80 psf (390 kg/m^2)
Stairs, balconies, corridors, storage, lobby and assembly areas	100 psf (487 kg/m^2)
Libraries (stacks), including area for film and tape library	150 psf (731 kg/m^2)
Flat roofs	80 psf (390 kg/m^2)

It is also required that, unless otherwise directed, office buildings having flat roofs shall be designed to safely support the dead and live loads of an additional floor.

It is noted that the live load specified for offices (80 psf) is larger than the minimum live load required by ANSI Standard A58.1 (50 psf). According to the Office of Foreign Buildings Operations, the larger loads in the State Department criteria are specified because of the use of file cabinets weighing as much as 1000 lb, and because of the installation of semi-permanent booths.

The Planning Procedures and Engineering Criteria issued in 1983 [3.7] require the same loads as the 1968 Engineering Design Criteria, except that the loads are expressed in kilograms force per square meter, and they are rounded up as follows: 400 kg/m^2 in lieu of 390 kg/m^2, 500 kg/m^2 in lieu of 487 kg/m^2, and 750 kg/m^2 in lieu of 731 kg/m^2. The 1986 Architectural and Engineering Design Guidelines and Criteria for New Embassy Buildings [3.8] requires that offices, stairs, balconies, office building corridors, lobby areas, fixed seating assembly, library reading rooms, Ambassador's residential and representational areas, safehaven areas, and all habitable and accessible attics be designed for a live load of 100 psf (500 kg/m^2), and specifically prohibits live load reductions.

3.2.3 Soviet Calculation Notes Dated September 25, 1979

Unit loads in kg/m^2 used in the Soviet calculations are listed in table 3.2.1. Dead loads correspond to the actual weights of materials and constructions, as estimated on the basis of the final design. For this reason, they are higher than the preliminary dead loads noted on the Skidmore Owings and Merrill/Gruzen & Partners (SOM) structural drawings dated April 16, 1976. Live loads are the same as those specified by the 1968 Engineering Design Criteria and in the SOM structural drawings.

3.2.4 Floor Loading Criteria Adopted for Assessment of Structural Integrity

In view of their conformity to the requirements of ANSI Standard A58.1 and the requirements of the 1968 Engineering Design Criteria, the loads listed in table 3.2.1 are adopted as the floor loading criteria used to assess the structural integrity of the Office Building and to specify recommended remedial measures. Live loads are reduced in accordance with the provisions of ANSI Standard A58.1. Note that such reduction is not permitted under the requirements of the 1986 Architectural and Engineering Design Guidelines which are more stringent than the requirements of the 1968 Engineering Design Criteria and the 1983 Planning Procedures and Engineering Criteria.

3.3 ENVIRONMENTAL LOADING CRITERIA

3.3.1 Wind Loads

This section reviews wind load requirements in ANSI Standard A58.1, as well as relevant information on the Moscow wind climate taken from the Soviet standard SNiP II-6-74. The 1968 Engineering Design Criteria issued by the Department of State do not contain any specific information on wind loads.

3.3.1.1 ANSI Standard A58.1-1982

In accordance with section 6.5.1 of ANSI Standard A58.1, the velocity pressure in psf at height z is calculated as follows:

$$q_z = 0.00256\ K_z(IV)^2 \qquad (3.3.1)$$

where V is the basic wind speed in mph, I is an importance factor and K_z is a velocity pressure exposure coefficient. The basic wind speed, V, is the greater of the 50-year fastest mile speed at 10 m above ground in open terrain, or 70 mph. I is equal to 1.0 for typical structures and to 1.07 for exceptionally important structures not exposed to hurricane winds. The value of K_z depends upon height above ground and the nature of surrounding terrain. For large cities, at 37.1 m above ground (the height of the Office Building), $K_z = 0.484$. Since the minimum basic wind speed used in ANSI Standard A58.1 is 70 mph -- a value higher than the 50-year fastest mile speed estimated for Moscow which, as shown in section 3.3.1.2, is 58 mph -- the velocity pressure at the building top, q_h, can be calculated as:

$$q_h = (0.00256)(0.484)[(1.07)(70\ \text{mph})]^2 = 6.95\ \text{psf} \qquad (3.3.2)$$

The design pressure, p_z, at height z is obtained by multiplying the velocity pressure by a gust response factor, G, [3.1, table 8] and by an aerodynamic coefficient, C_p, [3.1, figure 2]. For a 37.1 m high building, the gust response factor is G = 1.50. The aerodynamic coefficient is $C_p = 1.3$. Thus, the total design wind pressure at the top of the building, p_h, is:

$$p_h = (1.50)(1.3)(6.95\ \text{psf}) = 13.55\ \text{psf} \qquad (3.3.3)$$

At lower elevations the wind pressures have the values $p_z = (K_z/0.484)p_h$. Values of K_z and the corresponding p_z obtained for elevations at the Office Building floor levels are presented in table 3.3.1.

The total wind load acting on the building, based on the value $p_h = 13.55$ psf, was determined to be approximately 135,600 lbf. It is necessary to check whether this total wind load meets the requirement for minimum wind loading in section 6.4.2.1 of the ANSI Standard, which states that, "The wind load used in the design of the main wind-force resisting system for buildings and other structures shall be not less than 10 lbf/ft^2 multiplied by the area of the building or structure projected on a vertical plane that is normal to the wind direction." For the Office Building this minimum load is approximately:

$$p_{min} = (10\ \text{psf})(41\ \text{m})(37.1\ \text{m})(10.76\ \text{ft}^2/\text{m}^2) = 163{,}700\ \text{lbf} \qquad (3.3.4)$$

which is more than the load determined above.

In order to meet the requirement of section 6.4.2.1 of the ANSI Standard, it is necessary to use a wind load with a pressure at the top of the building $p_h = (163{,}700/135{,}600)(13.55\ \text{psf}) = 16.4$ psf. Wind pressures at lower elevations then have the values $p_z = (K_z/0.484)(16.4\ \text{psf})$. For comparison to Soviet design wind pressures below, the ANSI Standard A58.1 wind pressure at the top of the building, in metric units, is 79.7 kg/m^2.

3.3.1.2 Soviet Standard SNiP II-6-74

SNiP II-6-74 [3.9] contains information on the Moscow wind climate and on the magnitude of wind loads for which structures similar to the Office Building are routinely designed in the Soviet Union.

According to section 6 of SNiP-6-74, static wind pressures, q_H^C, are determined by the formula:

$$q_H^C = q_o kc \qquad (3.3.5)$$

where q_o = velocity pressure, k = coefficient that takes into account the variation of the velocity pressure with height, and c = aerodynamic coefficient.

For the Moscow region, section 6.4 of SNiP II-6-74 specifies a velocity pressure at 10 m above ground of q_o = 27 kg/m^2, corresponding to a wind speed in open terrain averaged over a period of two minutes and having a 5 year mean recurrence interval. Since the relationship between wind pressure, p, and velocity, v, is $p = 1/2\ \rho v^2$ where ρ is the air density, the Soviet wind speed can be estimated to be approximately equal to:

$$v = [(2)(27\ \text{kg/m}^2)/(0.125\ \text{kg s}^2/\text{m}^4)]^{1/2} = 20.8\ \text{m/s} \qquad (3.3.6)$$

where 0.125 kg s^2/m^4 is the air mass per unit volume. For comparison to ANSI Standard A58.1, this 5-year wind speed at 10 m above ground in open terrain averaged over two minutes is converted to a 50-year, fastest mile wind speed (in mph) according to:

$$V = (20.8\ \text{mph})(1.21)(1.03)(2.237) = 58\ \text{mph} \qquad (3.3.7)$$

In eq. (3.3.7), the factor 1.21 is used to convert the 5-year wind to a 50-year wind [3.11], the factor 1.03 is used to convert the 2-minute speed to a fastest mile speed [3.12], and the factor 2.237 is used to convert speeds in m/s to speeds in mph.

The factor k in eq. (3.3.5) depends upon the type of surrounding terrain. For urban terrain, k is given in table 7, section 6.5 of SNiP II-6-74 as follows:

Height above ground, m	10	20	40	60
urban terrain, k	0.65	0.9	1.2	1.45

The factor c in eq. (3.3.5) depends upon the building geometry. For a rectangular building with a flat roof, a square shape in plan, and a height to width ratio of unity, c = 1.3 [3.9, case 2, p. 16].

Section 6.1 of SNiP II-6-74 requires consideration of dynamic wind loading only for buildings with heights in excess of 40 m. Since the Office Building is less than 40 m high, the unfactored wind pressure at the top of the building consistent with the provisions of the Soviet Standard SNiP II-6-74 would be:

$$q_H^{\,C} = (27\ kg/m^2)(1.2)(1.3) = 42.1\ kg/m^2 \qquad (3.3.8)$$

This value is lower than the value estimated in accordance with ANSI Standard A58.1 (79.7 kg/m^2).

Note that the Soviet calculation notes dated December 5, 1980 conservatively include the dynamic component of the wind loads, in spite of the SNiP II-6-74 provision limiting consideration of dynamic loads to structures with height above 40 m. The total unfactored wind load (i.e., the sum of the unfactored static and dynamic loads) calculated in the Soviet calculation notes is 72.1 kg/m^2 at the top of the building. It has been verified that the Soviet calculations of the dynamic wind loads contain an error on the conservative side in the selection of the dynamic response factor specified in SNiP II-6-74 as a function of the fundamental natural frequency of the structure.

3.3.2 Snow Loads

The Office Building roof snow load, calculated in accordance with SNiP II-6-74 provisions for snow loads on buildings in Moscow, is 100 kg/m^2. This is considerably lower than the specified roof live load of 390 kg/m^2. It is therefore reasonable to conclude that snow loads will cause no overstressing of the structural members supporting the roof.

3.3.3 Earthquake Loads

The purpose of this section is to present information on the seismicity of the Moscow area and on the treatment of seismic loads in that area. The applicable provisions of ANSI Standard A58.1 used in the assessment of the integrity of the Office Building are then briefly reviewed in light of this information.

3.3.3.1 Seismicity of the Moscow Region

Soviet Standard SNiP II-A.12-69* (Construction in Seismic Regions [3.10]), does not include Moscow among the seismic regions of the USSR and includes no requirements for seismic loads in the Moscow region. The 1983 Planning Procedures and Engineering Criteria issued by the Department of State [3.7] evaluates the Moscow region as belonging to seismic zone 0, as defined in the Uniform Building Code [3.5], and rates the degree of confidence attached to this evaluation as low. (In the Uniform Building Code, zone 0 is described as one with no damage due to earthquakes.)

According to information provided by the Geological Survey, United States Department of the Interior, while earthquakes have occurred in the Moscow area, none have occurred in Moscow itself, and only a very few earthquakes have occurred within 400 km of the city. Of the earthquakes that have occurred within 400 km of Moscow, the closest earthquake of any significant size occurred more than five centuries ago (1467) near Rostov, 100 km northeast of Moscow. This earthquake was estimated to have a magnitude of about 3.5 and to have produced shaking of about intensity IV on the Modified Mercalli scale. The others occurred in 1596 near Gorkii (magnitude about

3.7), in 1896 near Lipetsk (magnitude about 3.6), in 1903 near Orel (magnitude about 3.0), and in 1954 near Tambov (magnitude about 4.8).

According to the Uniform Building Code, seismic zone 1 corresponds to intensities V and VI on the Modified Mercalli scale and zone 0 corresponds to lower intensities. Given the fact that the historical record, going back for at least five hundred years, does not include the occurrence of any earthquake in Moscow and does not include the occurrence of any earthquake greater than intensity IV in the Moscow region, the State Department rating of Moscow as belonging to seismic zone 0 appears to be justified.

Figure 18 in "Engineering Geology of the USSR, Part I, Moscow" [3.13] includes the Moscow region in a seismic zone with intensity IV or less. Reference 3.14 ("Seismic Zoning of the USSR") states that there are no seismic effects on buildings in zones with intensity IV. In fact, seismic effects in zones with intensity V (more intense than Moscow) include only "slight damage - thin cracks in plaster" and these effects are limited to "buildings of broken stones, rural structures, houses made of sun-dried brick and adobe houses."

The Uniform Building Code specifies no seismic loading for structures in zone 0. Since the 1968 Engineering Design Criteria and the 1983 Planning Procedures and Engineering Criteria, issued by the State Department state that the Uniform Building Code is an acceptable reference for earthquake design, the same is true of these two documents.

3.3.3.2 Seismic Provisions in ANSI Standard A58.1-1982

ANSI Standard A58.1 classifies part of the U.S. territory as seismic Zone 0. ANSI Zone 0 includes, but is more extended geographically than, the seismic zone 0 defined in the Uniform Building Code.

Unlike the Uniform Building Code, which has no requirements for seismic design in zone 0, ANSI Standard A58.1 sets forth the following minimum requirements concerning structures in Zone 0:

> (from section 9.11.1)
> Concrete or masonry walls shall be anchored to all floors and roofs that provide lateral support for the wall.
>
> (from section 9.11.2)
> All parts of the building that transmit seismic forces shall be connected through a continuous path to the resisting element. At a minimum, the connection and the elements along the path to the resisting element shall be capable of resisting a force equal to ... 0.05 ... times the weight of the portion being connected.

Section 9.1 of ANSI Standard A58.1 states that, for buildings in Zone 0, compliance with sections 9.11.1 and 9.11.2 will satisfy the requirement that a building and every portion thereof shall be designed and constructed to resist the earthquake effects to which it may be subjected during its life. Note that ANSI Standard A58.1 – like the Uniform Building Code – has no requirement concerning the lateral seismic force acting on the structure as a whole.

However, since the probability of simultaneous occurrence of earthquakes and windstorms is small, it is implicit in the provisions of both the Uniform Building Code and ANSI Standard A58.1 that the structure as a whole is designed to resist seismic loads approximately equivalent to the design wind loads.

3.3.4 Environmental Loading Criteria Adopted for the Assessment of Structural Integrity

The following environmental loads are used for the assessment of the structural integrity of the Office Building and for recommended remedial measures:

Wind Loads: At the top of the building, design wind pressure = 79.7 kg/m^2. At lower heights, z, above the ground, p_z = $(K_z/0.484)79.7$ kg/m^2, where K_z is for urban terrain (exposure A), taken from table 6 of ANSI Standard A58.1.

Snow Loads: The requirements of the 1968 State Department Engineering Design Criteria concerning live loads on roofs are considerably more severe than roof snow load requirements in Moscow. For this reason the live loads listed in table 3.2.1 for the eighth floor roof and the penthouse floor are used in lieu of snow loads.

Earthquake Loads: It is required that the structure comply with the requirements of sections 9.11.1 and 9.11.2 of ANSI Standard A58.1.

3.4 MEMBER AND CONNECTION RESISTANCES

The structural integrity of the Office Building will be assessed in terms of current U.S. design practice. However, the construction materials are specified in terms of Soviet practice. Thus it is necessary to explain how material properties which are specified according to Soviet practice will be used to determine structural resistance according to U.S. practice. This section reviews, in general terms, the procedures used for computing the structural resistance of structural members in the Office Building. Some basic differences between U.S. and Soviet practice are mentioned. Since the Office Building is composed of reinforced concrete and structural steel members, the discussion addresses both member types.

3.4.1 Resistance of Reinforced Concrete Members

3.4.1.1 Nominal Strength of Reinforced Concrete Members

In discussing the resistance or strength of a structural element, such as a beam or a column, a distinction is made between the nominal strength and the design strength. The nominal strength is based on the dimensions of the member and the strength of the materials. The nominal strength may be considered as the "ideal" strength [3.15]. For some types of member resistances, such as the bending strength of beams, the nominal strength is computed by using principles of mechanics. In other cases, nominal strengths are based on empirical formulas derived from extensive test results.

Because of inherent variability, the strength of the material in a structure will vary from point to point. To add to the safety margin, the nominal strength of a member is based on the "minimum strength" of the material. The minimum strength is obtained from the distribution of strength of the material so that only a small proportion of the material in the structure is expected to be weaker than the minimum strength.

There is a difference between U.S. and Soviet practices in defining the minimum strength of concrete. In the United States, the minimum strength of concrete represents the strength that is expected to be exceeded in the structure with 90 percent probability [3.2]. This minimum strength is referred to as the "specified strength". In Soviet practice [3.16], the minimum strength is taken as the value to be exceeded with 95 percent probability, and this is referred to as the "nominal concrete strength."

3.4.1.2 Design Strength of Reinforced Concrete Members

In assessing the structural safety of a member according to either U.S. or Soviet practice, the actions due to the loads must not exceed the design strength. Because of approximations in the calculation methods, variations in workmanship, dimensions and material strengths, the design strength of a member is taken to be less than its nominal strength. The design strength can be thought of as the "dependable strength" [3.15].

There is a significant difference between the U.S. and Soviet practices for computing design strength. In U.S. practice [3.2], the member design strength is obtained by multiplying the nominal strength of the member by a "capacity reduction factor." The nominal member strength is calculated from the specified strength of concrete and the minimum yield strength of the reinforcing steel.

In Soviet practice [3.16], the member design strength is calculated from the design strengths of the concrete and the steel. The design strength of concrete is obtained by dividing its nominal strength (minimum strength) by a factor, such as 1.3. Likewise, the design strength of steel reinforcement is obtained by dividing the minimum yield strength by another factor, such as 1.15. These design strengths are used in formulas for computing the design strength of a structural element.

To summarize, in U.S. practice the design strength of a reinforced concrete structural element is obtained by multiplying its nominal member strength by a single capacity reduction factor. The value of the capacity reduction factors is dependent on the type of member resistance that is being evaluated. In Soviet practice, separate reduction factors (partial factors) are applied to the nominal strengths of the concrete and the steel, and these reduced strengths are used to compute the design strength of the member.

3.4.1.3 Specification of Material Grades

In U.S. practice, the grade of concrete is specified in terms of the compressive strength of a standard cylinder specimen (6 in. diameter, 12 in. height). The specified compressive strength, f_c', is used in computing member resistances and it represents the minimum strength as described in section 3.4.1.1. The average strength of the concrete in the structure is expected to exceed the specified strength by an amount which is dependent on the standard deviation of the concrete that is supplied. The quality of the supplied concrete is monitored by testing standard cylinder specimens which have been molded from samples of the concrete as delivered.

In Soviet practice, the grade of concrete is specified in terms of the average compressive strength of standard cube specimens (200 mm edges). For example, a "Mark 400" concrete represents concrete with an average cube strength of 400 kg/cm^2. On-site quality control is based on the compressive strength of cube specimens. The value of the minimum cylinder strength of concrete must be derived from the average cube strength so that member resistances can be determined according to U.S. practice.

In Soviet practice [3.16], the nominal compressive strength is derived from the average cube strength by the following steps:

(1) the nominal (minimum) cube strength is calculated from the average cube strength and the variability of strength, and

(2) the nominal cube strength is converted to a nominal prism strength.

The nominal cube strength is obtained from the average cube strength by assuming a normal probability distribution for the strength of concrete and using a coefficient of variation of 0.135 for normal weight concrete. The nominal prism strength is then found from the following expression, which converts cube strength to prism strength [3.16]:

$$Rp = Rc\ (0.77 - 0.0001\ R) \qquad (3.4.1)$$

where Rp = nominal prism strength, Rc = nominal cube strength, and R = average cube strength. For R greater than 500 kg/cm^2, Rp = 0.72 Rc.

Based on the above discussion, the following procedure was used to calculate the minimum cylinder compressive strength corresponding to a given "Mark" concrete specified for various components of the Office Building:

(1) Using the average cube strength (equal to the "Mark" value) and a coefficient of variation of 0.135, the cube strength expected to be exceeded with 90 percent probability was computed.

(2) Assuming cylinder strength is equal to prism strength, eq. (3.4.1) was used to determine the minimum cylinder strength.

The following gives the minimum cylinder strengths corresponding to concrete having different "Mark values":

Concrete Mark (kg/cm^2)	100	200	300	400	500	600
Minimum Cylinder Strength (kg/cm^2)	63	124	184	241	298	357

The computed values of the minimum cylinder strength are strongly dependent on the factor used to convert cube strength to cylinder strength. For the above calculations, the factor varied from 0.76 to 0.72 as the average cube strength varied from 100 to 500 kg/cm^2. These factors tend to be lower than those quoted by other researchers. For example, it has been reported [3.17] that the conversion factor is an increasing function of cube strength, and for cube strengths in excess of 300 kg/cm^2 the factor is greater than 0.9. Thus it is believed that the cube strength-cylinder strength conversion factors given in the Soviet practice [3.16] are low, and the above computed values of minimum cylinder strength are probably conservative estimates.

In U.S. practice, the grade of reinforcing steel is defined by its specified yield strength, f_y. For example, Grade 60 corresponds to a specified yield strength of 4225 kg/cm^2 (60,000 psi). Just as in the case of concrete strength, the specified yield strength is a minimum value and the average yield strength is greater. A study of the mechanical properties of reinforcing steel used in U.S. practice gave the following statistics for Grade 40 and Grade 60 reinforcement [3.18].

Grade	Yield Strength kg/cm^2 Specified	Average	Coefficient of Variation
40	2817	3437	0.107
60	4225	5000	0.093

Assuming a normal probability distribution, the specified yield strengths correspond to a strength that is exceeded with 95 percent probability.

In Soviet practice [3.16], the grade of reinforcing steel is specified by classes having different nominal yield strengths. The classes of reinforcing steel used in the components of the Office Building are primarily AI, AII and AIII, for which the nominal yield strengths are 2400, 3000 and 4000 kg/cm^2, respectively.

As part of the field investigation discussed in Chapter 5, five samples of 32-mm diameter reinforcing bars were removed from the column joints in the seventh story along the east exterior column line. At these locations steel columns join with Type 3 reinforced concrete columns. The bars are Type AIII. Tensile test specimens were prepared from the bars and the average yield strength was 4200 kg/cm^2 with a coefficient of variation of 0.02. Using the properties of the t-distribution, the specified nominal strength of 4000 kg/cm^2 for class AIII would be exceeded with slightly more than 95 percent probability. Thus, in terms of U.S. practice, these bars could be classified as having a specified yield strength of 4000 kg/cm^2.

Based on the above data, it is reasonable to assume that the nominal yield strengths given for the different Soviet grades of reinforcement can be used as the specified yield strengths for the purpose of assessing safety of reinforced concrete members according to U.S. practice.

3.4.2 Resistance of Steel Members and Connections

The structural safety of structural steel members and their connections will be assessed by using the current U.S. practice known as Load and Resistance Factor Design (LRFD) [3.3]. The procedure is analogous to that used for reinforced concrete members:

(1) the nominal member strength is computed based on the member dimensions and the specified yield strength, F_y, of the steel,

(2) capacity reduction factors are used to compute the design strength, and

(3) the design strength is compared with the actions due to the loads.

In U.S. practice, the specified yield strength will be lower than the average yield strength. In a background study leading to the development of the LRFD approach [3.19], the following statistics were adopted for yield strength of structural steels in rolled shapes:

	Average Yield Strength	Coefficient of Variation
Flanges	$1.05\ F_y$	0.10
Plates and webs	$1.10\ F_y$	0.11

In Soviet practice [3.20], the class of steel is specified in terms of nominal values of yield and ultimate tensile strengths. For example, rolled sections are made of steel class C 38/23, which signifies an ultimate tensile strength of 3800 kg/cm^2 and a yield strength of 2300 kg/cm^2. This same class is used for some built-up members in the Office Building. Other built-up members are made of higher strength, class C 46/33 steel, which has a nominal yield strength of 3300 kg/cm^2.

During this investigation, information on the relationship between the nominal strengths and average yield strengths of the Soviet steel grades was not available. However, samples of steel were obtained from three types of structural members: 1) a rolled section, 2) a low strength built-up section, and 3) a high strength built-up section. Tensile test specimens were made from the samples and the resulting yield strengths were as follows:

	Yield strength kg/cm^2	Actual/Nominal Yield Strength
Rolled section	3280	1.42
Flange plate of low strength built-up section	3580	1.57
Flange plate of higher strength built-up section	5330	1.62

The measured yield strengths of the Soviet steel samples exceed their nominal values by substantial margins. Thus, for the structural assessment of structural steel members, the Soviet nominal yield strengths will be used as the specified yield strength in the LRFD analysis.

In the Office Building, welded connections are used to join steel members. In U.S. practice, the strength of the weld metal from electrodes is specified in terms of minimum tensile strength. For example, E70 electrodes correspond to weld metal with a minimum specified tensile strength of 72,000 psi. The specified minimum tensile strengths are incorporated into the LRFD formulas for computing the design strength of welded connections.

In Soviet practice [3.21], strength of weld metal is also specified in terms of tensile strength. In the general notes for structural working drawings KM-3-5, which deal with steel members, electrodes with ultimate strengths of 4600 and 5000 kg/cm^2 are specified. However, the details of the welded connections do not indicate which type of electrode should be used. Therefore, a tensile strength of 4600 kg/cm^2 was used to asses the safety of welded connections by the LRFD approach.

3.4.3 Summary

This section has explained how material strengths specified according to Soviet practice will be used to assess structural adequacy according to current U.S. practice. The greatest difference between U.S. and Soviet practice is the manner for specifying the grade of concrete. The procedure has been described for converting average cube strength specified in Soviet practice to a minimum cylinder strength for use in assessing structural adequacy according to U.S. practice. For reinforcing steel, the specified yield strength is taken to be the same as the nominal yield strength specified by Soviet practice. In assessing the adequacy of structural steel members, the specified yield strength is taken to be equal to the nominal value specified in Soviet practice. Likewise, the Soviet values for tensile strength of welding electrodes are used to assess the adequacy of welded connections.

3.5 CRITERIA FOR DESIGN AGAINST PROGRESSIVE COLLAPSE

3.5.1 Background

Progressive collapse has been a concern to designers since the 1968 chain reaction collapse of the Ronan Point apartment building in England, which was triggered by a gas explosion in an 18th story apartment and propagated all the way to the ground floor. Since that time, various committees have looked at the problem and developed recommendations, beginning with the U.K. Royal Commission report and recommendations issued as a result of the Ronan Point collapse [3.22].

In this country, the first recommendations for design against progressive collapse were developed by the National Bureau of Standards in conjunction with "Operation Breakthrough" [3.23] and the first U.S. standard with provisions for avoidance of progressive collapse was ANSI A58.1-1972 [3.24]. Subsequently, further studies were conducted in an effort to define the scope of the problem and to develop design recommendations [3.25-3.29]. The best information currently available for U.S. practice is contained in (1) ANSI Standard A58.1-1982 [3.1]; (2) a set of recommendations for large-panel structures developed by the Prestressed Concrete Institute (PCI) [3.30], and (3) recommendations resulting from a workshop sponsored jointly by the National Science Foundation, NBS, and the Department of Housing and Urban Development (HUD) [3.31]. The 1983 and 1986 FBO design guidelines for U.S. embassies call for design against progressive collapse [3.7,3.8]. A summary of regulatory approaches is provided in reference 3.32.

It should be noted that while ANSI Standard A58.1 addresses buildings in general, the PCI document [3.30], as well as most studies on the subject of progressive collapse, deals with concrete panel structures, rather than frame structures similar to the Office Building.

ANSI Standard A58.1 guidelines for design against progressive collapse have been incorporated in the Basic National Building Code of the Building Officials Conference of America (BOCA) [3.33] and are therefore included in the building code provisions in many parts of the United States.

3.5.2 Existing U.S. Standards

Guidelines for design against progressive collapse are provided in ANSI A58.1-1982, section 3.1 General Structural Integrity. "General structural integrity" is defined as "the quality of being able to sustain local damage with the structure as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage." It is further stated in section 3.1 that the most common method of achieving structural integrity is through an arrangement of structural elements that gives stability to the entire structural system, combined with the provision of sufficient continuity and energy absorbing capacity (ductility) in the components and connections of the structure to transfer loads from any locally damaged region to adjacent regions capable of resisting those loads without collapse.

More explicit information is provided in Appendix A1.3 of ANSI Standard A58.1, which is not part of the standard itself but is included for information only. It is stated in Appendix A1.3 that it is impractical to design a structure to resist severe abnormal loads acting on a large portion of it, however cautions can be taken to limit the spread of an initial local failure from element to element.

The design alternatives discussed below fall into two categories: Direct design, where resistance to progressive collapse is considered explicitly; and indirect design, where general structural integrity is implicitly provided by minimum levels of strength, continuity and ductility.

Two methods of direct design are recognized:

(1) The alternate path method, where local failure is allowed to occur but an alternate load path is provided around the failed structural element.

(2) The <u>specific local resistance</u> method, where sufficient strength is provided to resist failure from anticipated accidental loads.

Several guidelines for direct design solution are listed in Appendix A1.3 of ANSI Standard A58.1. Those of potential interest for the Office Building are: internal load-bearing partitions; catenary action of floor slabs; and beam action of walls.

Another U.S document is the report of the Prestressed Concrete Institute Committee on Precast Bearing Wall Panels [3.30]. The relevant provisions of reference 3.30 are briefly summarized below:

(a) For abnormal loads, a capacity reduction factor of 1 should be applied in conjunction with ACI 318 [3.2].

(b) For the evaluation of progressive collapse the design loads consist of the unfactored service dead load plus 2.5 percent of the service dead load applied laterally at each floor level [3.30, section 2.2]. (Note that reference 3.34 requires design for UBC Zone 3 earthquake load which results in higher lateral loads.)

(c) Continuous peripheral (circumferential) ties at each floor or roof level should be provided as required to develop diaphragm action or a force of 16,000 lb, whichever is more (at yield load). If there is an expansion joint, each part of the building should be tied separately.

(d) Longitudinal ties (in the direction of the floor span) connecting floor or roof elements that abut over internal walls or connecting external bearing walls with floors or roofs should be capable of resisting a force of 2.5 percent of the service load but not less than 1,500 lb per lineal foot.

(e) Transverse ties have requirements identical to those for longitudinal ties.

Note that the horizontal tie requirement of 2.5 percent of the service load does not insure effective membrane action if the supporting member of the floor span is removed.

3.5.3 <u>Criteria for the Office Building</u>

As previously noted, consideration of progressive collapse is incorporated in U.S. standards in the form of provisions for "general structural integrity." However, the standards do not stipulate explicit design criteria to implement these provisions. The criteria presented in this section were developed in order to apply the provision for general structural integrity to the Office Building.

In the Office Building, where primary load bearing members are for the most part prefabricated and the connections between these members have, in most cases, limited moment resistance, general structural integrity is not provided implicitly, and must therefore be ascertained by direct design. The following criteria are used to determine vulnerability to progressive collapse.

3.5.3.1 Specified Loads

Vertical loads: 1.0 times dead load plus 70 kg/m^2 live load for office floors (allowance for snow loads is not necessary as there are provisions for snow removal from roof).

Horizontal loads: 0.25 times design wind load.

All loads will be considered ultimate loads.

3.5.3.2 Resistance of Structural Members

The load capacity of members resisting the loads stipulated in section 3.5.3.1 shall be determined using a capacity reduction factor (ϕ factor as defined in reference 3.2) of unity and material strengths as defined by applicable standards. In absence of specific guidance provided by standards, the material strength will be taken as the 10% exclusion strength (minimum strength as discussed in section 3.4.1.1) calculated on the basis of the available test data.

3.5.3.3 Performance Criteria

The following criteria were adopted to assess the resistance to progressive collapse of the Office Building:

(1) The failure of any primary structural member shall not cause progressive collapse propagating beyond one story level above or below the affected structural member vertically, or to the next primary structural member horizontally under the loading stipulated in section 3.5.3.1.

The following members are considered: one column; one girder (failure at one cross sectional location); one shear wall panel.

(2) The failure of a floor panel shall not precipitate the failure of the floor panel below it.

3.5.3.4 Commentary

Design against progressive collapse generally considers two methods: (1) the alternate path method which requires that there be an alternative path of carrying the gravity loads to the foundation if a primary structural member is removed; (2) the specific local resistance method whereby a vulnerable primary structural member is made strong enough to resist abnormal loads, and thus an alternative load path does not have to be provided.

The alternate path method protects the structural system by limiting the area of distress associated with a structural failure and thus the consequences of the failure. This method is independent of the specific hazards considered, which may range from failures of members or connections as a result of structural deficiencies to explosions.

The specific local resistance method can only be applied when the hazards that could cause a primary member to fail are defined. The method would not be effective in the case of structural deficiencies, and has generally not been advocated by the profession because of the difficulties in defining abnormal loads.

The 70 kg/m^2 floor live load is stipulated on the basis of the findings of an NBS study of floor loads in office buildings [3.35] and takes account of permanent live loads such as office furniture. The lateral load of 25% of the design wind load is stipulated because it is unreasonable to assume that a structural failure will occur during a major windstorm with a long mean recurrence interval. The value of 25% of the design wind load approximately corresponds to an extreme wind load with a one month mean recurrence interval. The value was derived from U.S. statistical data [3.11].

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Table 3.2.1. Dead and live loads used in the Soviet calculation notes

Location	Item	Unit Load kg/m^2
Typical floor 2nd through 6th	Reinforced concrete planks, 220 mm thick	400
	Concrete topping, 80 mm thick	192
	Partitions	96
	Suspended ceiling and mechanical equipment	48
	Steel beams with encasing concrete	140
	Dead Load (Total)	876
	Live Load	380
1st floor typical section (see figure 3.2.1)	Dead Load	876
	Live Load	380
section 1	Live Load	730
section 2	Live Load	490
7th floor typical section (see figure 3.2.2)	Dead Load	876
	Live Load	380
section 1	Live Load	790
8th floor	Reinforced concrete planks	400
	Reinforced concrete slab, 200 mm thick	500
	Floor Tile, 60 mm thick	144
	Partitions	50
	Suspended ceiling and mechanical equipment	48
	Steel beams and encasing concrete	140
	Dead Load (Total)	1282
	Live Load	730
Roof over 8th story section 1 (see figure 3.2.3)	Monolithic R/C slab, 220 mm thick	550
	Light concrete, avg. 130 mm thick	169
	Mortar, 20 mm thick	36
	Foam, 120 mm thick	36
	Reinforced mortar, 40 mm thick	80
	Waterproofing	10
	Slab, 100 mm thick	240
	Suspended ceiling and mechanical equipment	48
	Steel beams and encasing concrete	140
	Dead Load (Total)	1310
	Live Load	390
section 2	Live Load	730

(continued on next page)

Penthouse floor	Monolithic R/C slab, 220 mm thick	550
	Lining, 100 mm thick	240
	Suspended ceiling and mechanical equipment	48
	Steel beams with encasing concrete	140
	Dead Load (Total)	978
	Live Load	730
Penthouse roof	Reinforced concrete planks, 220 mm thick	400
	Foam, 120 mm thick	36
	Light concrete, avg. 70 mm thick	91
	Mortar slab, 40 mm thick	80
	Waterproofing	10
	Gravel, 20 mm	50
	Steel beams and encasing concrete	80
	Dead Load (Total)	747
	Live Load	390
All floors dead loads from walls and partitions kg/m^2 of vertical surface	Brick partition wall with plaster on both sides, 160 mm thick	290
	Brick partition wall with plaster on both sides, 290 mm thick	520
	Brick partition wall with plaster on both sides, 550 mm thick	990
	Reinforced concrete wall, 200 mm thick	500
All floors dead loads from girders, beams and columns kg per lineal m	Steel beams with plaster	400
	Large reinforced concrete beams	700
	Columns with flexible reinforcement	720
	Columns with steel core	750

Table 3.3.1. ANSI A58.1 wind loads at Office Building floor levels

floor	height, z (m)	K_z	p_z (psf)
1	0.0	0.12	3.33
2	4.8	0.12	3.33
3	9.0	0.19	5.28
4	13.2	0.24	6.68
5	17.4	0.29	8.06
6	21.6	0.33	9.18
7	25.8	0.38	10.58
8	30.0	0.42	11.68
9	34.2	0.46	12.81
10	37.1	0.48	13.55

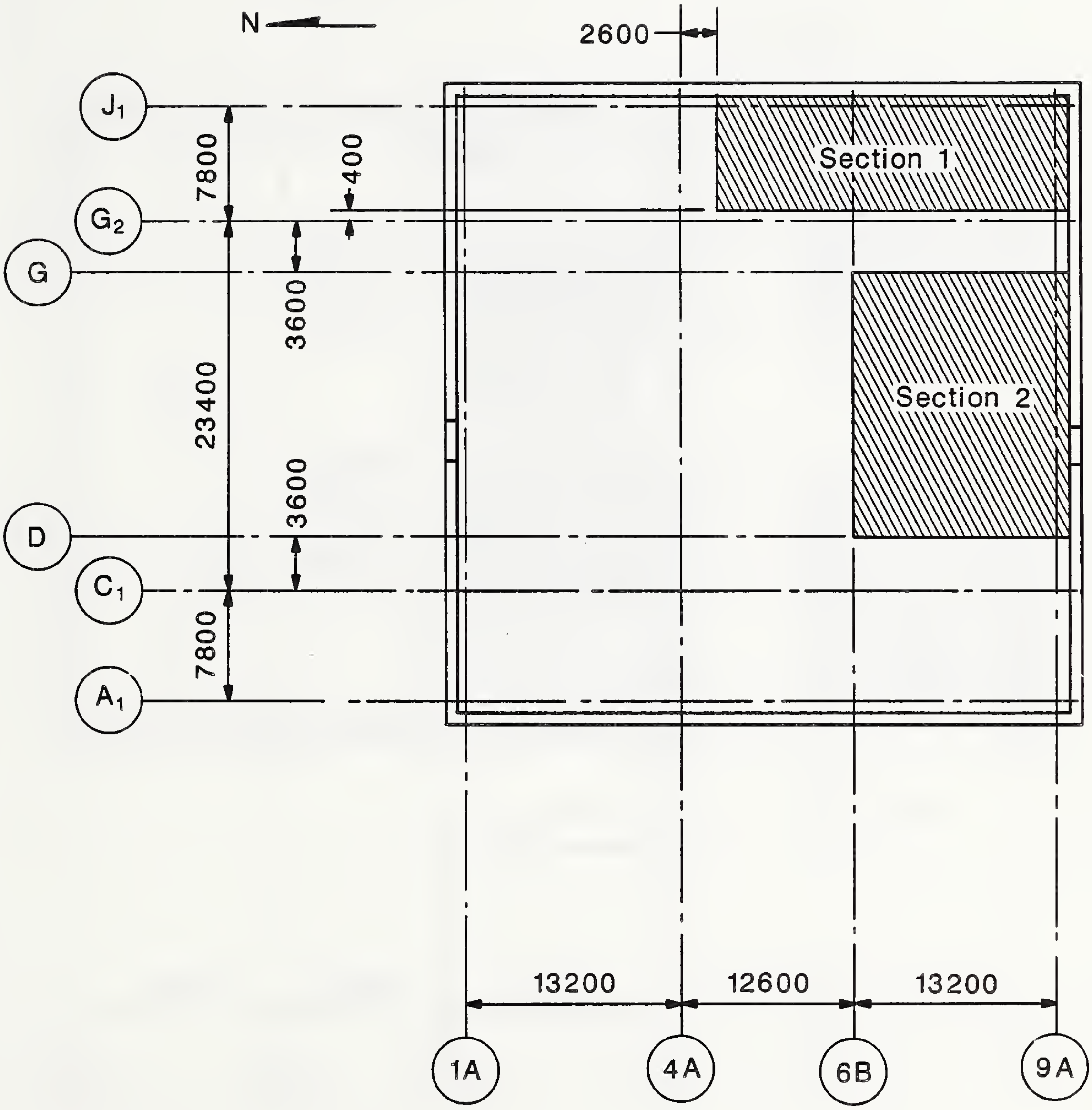

Figure 3.2.1. First floor plan, including special loading sections

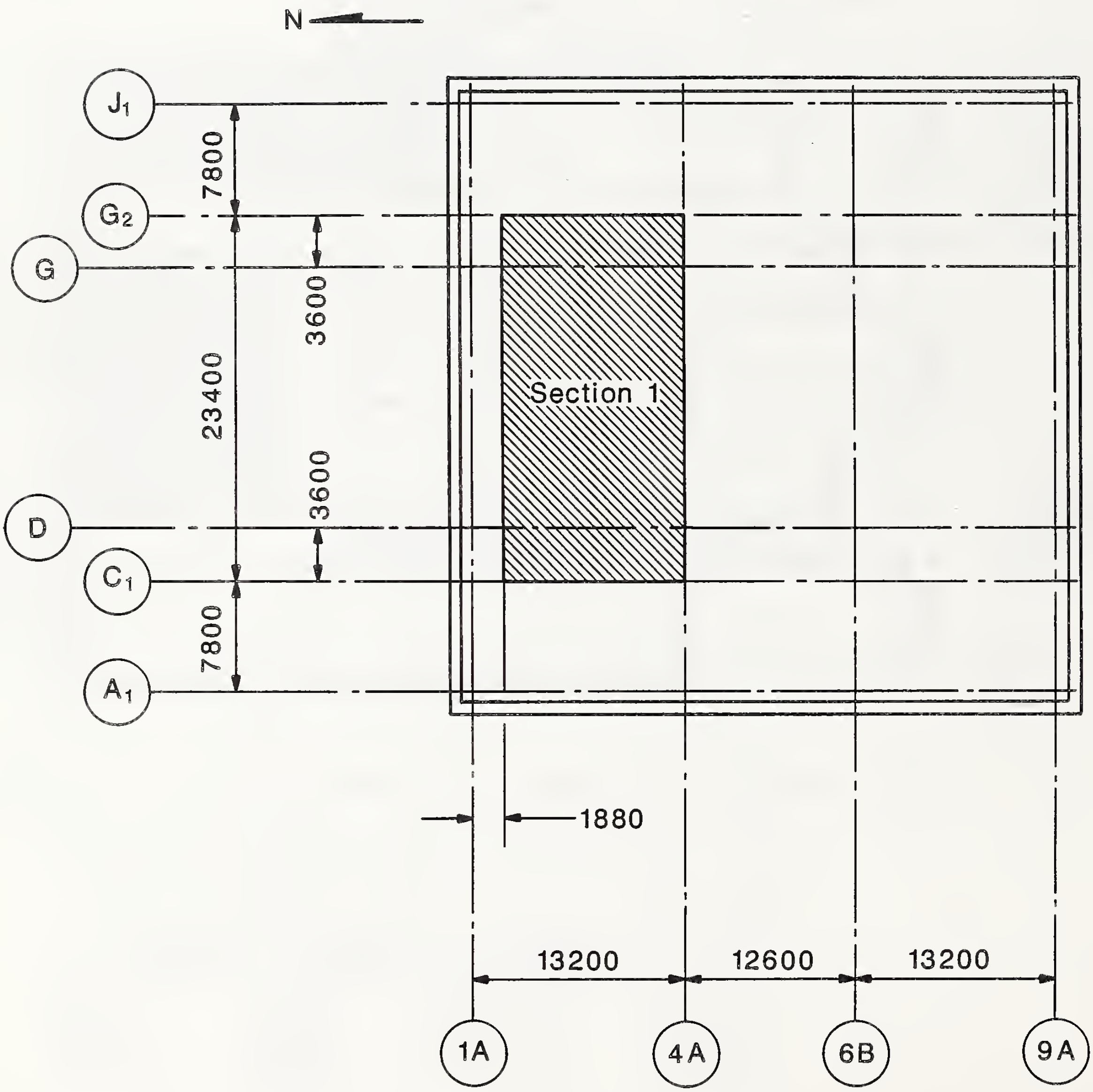

Figure 3.2.2. Seventh floor plan

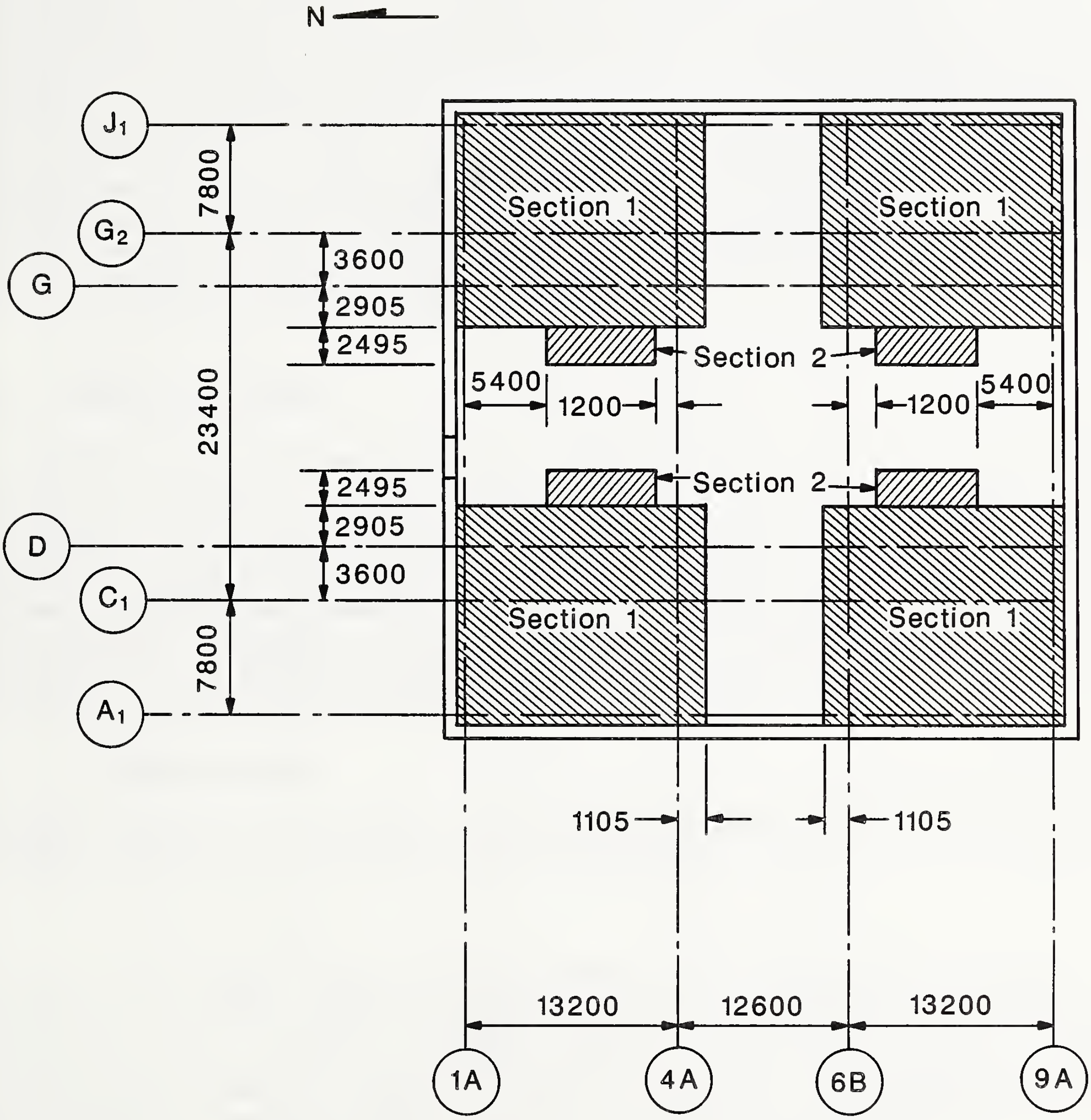

Figure 3.2.3. Roof over eighth story

CHAPTER 4
ASSESSMENT OF DESIGN

In this chapter the ability of the as-designed structural system to resist the loads defined in Chapter 3 will be determined. There are three basic load resisting sub-systems in the Office Building: the structural frame which resists vertical loads; the reinforced concrete shear walls which resist lateral loads; and the foundation system which transmits loads from the structural frame and shear walls to the underlying bedrock. The assessment presented in this chapter is based on the following procedure: the loading criteria of Chapter 3 were used as the basis for determining member loads and reactions; the design strength of each of the structural elements described in Chapter 2 was determined using material properties as specified in the design drawings; and, members with a design strength less than the required strength were identified so that during the field investigation in-place material properties, section dimensions, and loading conditions could be determined. (Results of the field investigation are presented in Chapter 5.) In addition, assessments of the pile foundation and the vulnerability of the structural system to progressive collapse are presented. The chapter concludes with an assessment of the brick masonry enclosure.

4.1 ANALYTICAL MODELS

A decision was made early in the investigation to pursue two independent structural analyses: a manual analysis based on simplified assumptions similar to those used in the design of the building; and, a linear-elastic computer analysis. The computer models, while requiring greater preparation time, included accurate representations of all load resisting components (slabs, beams, columns, and shear walls) and their support conditions. The models also had the advantage of being a database for determining member forces and stresses for different loading combinations. Two loading conditions were analyzed: vertical (dead and live) loads and lateral (wind) load.

4.1.1 Manual Analyses

Manual analyses were used to identify potentially overstressed structural elements prior to the second site visit in February 1987.

4.1.1.1 Vertical Loads

In the manual analysis of vertical loads it was assumed that beams and floor planks are simply supported and that beam reactions act at the column centerlines. In general, member loads were determined by considering tributary areas and the resulting column loads were tabulated using a "spreadsheet" computer program. A table of column live and dead loads (both factored and unfactored, with allowances for live load reduction as specified in section 4.7 of ANSI Standard A58.1) is presented in Appendix 4.1.

4.1.1.2 Lateral Load

In considering the response of the Office Building to wind loading, it is recognized that the exterior masonry walls would resist a portion of the load. Thus, the shear wall system would not be required to resist all wind loads. However, as explained in Chapter 2, the exterior masonry walls are supported on reinforced concrete frames on three sides of the building. They would not be able to transfer the loads directly to the foundation; instead, loads in the exterior masonry would be transferred to the shear wall through the floor system. Therefore, at the basement level, the shear wall system would be required to resist the full wind load irrespective of the upper story participation by the masonry wall. For the purpose of this design review, it is assumed that the shear wall system provides the only resistance to lateral wind loads.

Vertical gaps exist between the columns and shear wall panels and between adjacent shear wall panels, and horizontal gaps exist between shear wall panels (fig. 4.1.1). The upper and lower edges of the wall panels are castellated to enhance transfer of shear forces at the horizontal joint once the joint is filled. Standard Soviet details call for the vertical joints to be filled with grout and the horizontal joints to be filled with concrete. Thus, in the manual analysis, the shear walls were assumed to act as a pair of monolithic, cantilever beams, with each wall receiving half of the lateral load.

4.1.2 Computer Analysis

Computer models were used to investigate more complex three-dimensional behavior, such as the effects of bending moments and shear forces induced in column lines by eccentric beam loads. The models were also used to determine the effects of local detailing such as the presence or absence of concrete or grout in shear wall joints and the determination of lateral load induced stresses in the welded shear wall connections.

Several finite element computer models were developed during the investigation. Beams and columns were modeled using two-node, 12 degree-of-freedom space frame elements which had six degrees-of-freedom (three translations and three rotations) at each end node. Where intermediate shear and moment results were desired for constructing column shear and bending moment diagrams, several elements were used to model each one-story column segment. Slab sections (planks and topping) and shear wall panels were modeled using four-node plate bending elements which also had six degrees-of-freedom per node (three translations and three rotations). Each shear wall panel (fig. 4.1.1) was modeled with a minimum of 15 plate bending elements to ensure that local stress concentrations could be accurately obtained. Plate bending elements were used to model the grout and concrete in the vertical and horizontal gaps in the shear wall system. Frame elements were used to model the three welded steel plate connections on each side of the shear wall panels.

Data required for the space frame and plate bending elements consisted of:

- Three dimensional cartesian coordinates of the end nodes (x-axis = east; z-axis = south; y-axis = elevation). The origin was at the northwest corner of the Office Building, at the basement floor level.

- Material properties including modulus of elasticity and Poisson's ratio.

- Transformed, uncracked section properties, including cross-sectional area; moments of inertia; dimensions, and the orientation of the major and minor axes.

The models were generated using the interactive graphics computer facility at the NBS Center for Building Technology. The models were solved on a remote CONVEX C-1 computer using the general engineering analysis program ANSYS. Post-processing was carried out at the interactive graphics facility.

4.1.2.1 Vertical Loads

The purpose of this model was to determine the axial loads, shear forces, and bending moments in the columns. The results of this analysis provided a check on the manual calculations which considered only the effects of concentric axial loads.

Figures 4.1.2 and 4.1.3 show computer images of the vertical loads model. This model included 7722 finite elements comprising all beams, columns, and shear walls in the Office Building. Because planks were simply supported, uniformly distributed dead and live loads could be directly translated to beam line loadings, thus eliminating the need to model floor slabs in the vertical loads model. The model also included shear wall joints and accounted for beam reaction eccentricities. The basement columns were modeled as rigidly fixed at their base.

Two load cases were considered: unfactored dead load and unfactored live load. During post-processing appropriate load factors were applied and the total factored loads were obtained; graphical displays of the resulting column axial loads, shear forces, and bending moments were generated.

4.1.2.2 Lateral Load

The primary purpose of the lateral (wind) load computer model was to determine the effect of the condition of the shear wall joints on the structure's ability to resist lateral load. A secondary consideration was to determine the effect of lateral load on column stresses.

This model was the same as the vertical loads model except for the removal of all beam elements and the addition of plate bending elements at each story level to model the floor slabs. The beams play a minor role in transmitting lateral load to the shear walls, due to the much greater in-plane stiffness of the floor slabs. Thus the beams were eliminated in the lateral load model to save computational time. Figures 4.1.4 and 4.1.5 show computer generated images of the lateral load models.

Wind loading as specified in ANSI A58.1-1982 was applied as nodal forces to the upwind side of the building at the intersection of the floor slabs and edge columns. Nodal forces were determined on the basis of tributary area and the surface pressure at a given elevation (see section 3.3.1.1).

Two variations of the lateral load model were developed: one in which the shear wall joints (both horizontal and vertical) were assumed to be properly filled with concrete or grout; and a second in which the joints were not filled. The second model was used because it was known from the December 1986 site visit that many joints were empty or incompletely filled. The model for the unfilled joints was obtained by eliminating plate elements forming the joints. However, the frame elements representing the welded connections were retained.

4.2 VERTICAL LOADS AND RESISTANCES

The design strengths of the floor planks, beams, and columns used in the Office Building were determined. Steel and concrete material strengths were the nominal values specified in the Soviet plans and catalogs. The calculated design strengths were compared with the largest required strength for each type of member or connection. The required strengths were computed using the appropriate load factors for the members.

4.2.1 <u>Calculation of Load Effects</u>

4.2.1.1 Vertical Loads on Columns, Beams, and Planks

Figure 4.2.1 is a schematic view of a portion of the structure which illustrates how vertical loads are transferred from floor slabs to beams, and to columns. The load acting on a column is computed as the load from the column above plus the loads transmitted to the column by the beams framing into that column. Beam loads are computed as the sum of three components: 1) member self-weight, 2) loads from floor planks which rest on the beams, 3) loads due to interior masonry partition walls.

Loads on the floor planks are computed as the area of the floor plank (in square meters) times the uniformly distributed dead and live loads (t/m^2) given in Chapter 3. The floor plank area used in computing loads at each floor is the tributary area. The influence area for a column is four times the tributary area; the influence area is used to calculate a live load reduction factor.

As an example of how column loads are calculated, consider the third story core column G/4A shown in figure 4.2.2. This column supports a 13.2-m steel beam which spans between the exterior frame and the core, and 2.4- and 3.6-m concrete beams which span between core columns. The beams and column layout is similar to that shown in figure 4.2.1.

The steel beam supports floor planks with a tributary area of (13.2 m)(5.4 m) = 71.28 m^2; the uniformly distributed dead load on the planks is 0.736 t/m^2. The steel beam self weight is 0.36 t/m and the line load due to the distributed dead load is (0.736 t/m^2)(71.28 m^2)/(13.2 m) = 3.974 t/m. The total line load is therefore 0.36 + 3.974 = 4.334 t/m (see fig. 4.2.2). The end reaction for this uniformly loaded beam is one half the total load acting on the beam, or (4.334 t/m)(13.2 m)/2 = 28.60 t.

The reinforced concrete beams have a self weight of 0.2 t/m and they support masonry walls which contribute a line load of 1.131 t/m. The end reactions are 4.73 t for the 3.6-m beam and 3.16 t for the 2.4-m beam.

The sum of the three beam end-reactions acting on the column is 28.60 + 4.73 + 3.16 = 36.49 t. Column loads at all floors are computed similarly, and the forces are accumulated from the top down. Column loads due to live loads are also computed similarly, except that the load reduction factor is used to modify the load as described in Chapter 3. (See Appendix 4.1 for a summary of all column dead and live loads.)

4.2.1.2 Bending Moments on Columns

Beams framing into columns are connected either at the face of the column, or they bear on corbels or steel brackets a small distance out from the column face. The beam end-reactions therefore act eccentrically to the column center line (fig. 4.2.2) and cause bending moments to be introduced into the column. The interaction of axial load and bending moments must be considered in calculating the column design strengths.

Moments applied to the columns are computed as the sum of the products of the eccentric offsets and the beam end-reactions about each perpendicular axis. At column G/4A the 13.2-m steel beam reaction acts at the face of the column (eccentricity equals 200 mm) and the concrete beam reactions act at the center of the corbels (eccentricity is 270 mm). Thus the moments acting on the column are 5.72 t·m about the east-west or z-axis and 1.28 t·m - 0.85 t·m = 0.43 t·m about the north-south or x-axis.

Eccentric beam loads produce shear forces in the column which cause the bending moments to vary linearly along the column. Figure 4.2.3 shows typical bending moment and shear force results obtained from the vertical loads model for column line E/4B. Factored axial loads and factored shear forces and bending moments about the x- and z-axes are plotted for each story level. These plots and similar plots for each of the critical column lines were used to obtain the maximum required strengths for each of the different types of columns. These required strengths are compared to the column design strengths in section 4.2.4.

4.2.2 Design Strength of Precast Planks

Two types of precast concrete floor planks are used in the building: prestressed, hollow core planks and reinforced, solid planks (see section 2.3.4). Since these planks span in one direction between beams, they were analyzed as wide beams.

The design strength for a typical solid plank (type TP-35-12) was calculated as specified in Chapter 10 of ACI 318-83 [4.1]. The calculations show that this plank has a design strength of 5410 kg/m^2 where the design strength is expressed in terms of the uniformly distributed pressure loading causing a moment equal to the design loading resistance. Floor loads are given in table 3.2.1 of Chapter 3. The heaviest load on this type of plank occurs in the southeast corner of the first floor (Floor Section 1) where the live load is 730 kg/m^2. This live load results in a factored load of 1.4(700 $kg.m^2$) + 1.7(730 kg/m^2) = 2470 kg/m^2, which is much less than the design strength.

The design strength of a typical hollow core plank (type NV-52-18T) was calculated as specified in Chapter 18 of ACI 318-83 which gives design procedures for prestressed concrete. The calculations show that the planks have a design strength of 2000 kg/m^2. The heaviest load on this type of floor plank occurs along the south side of the first floor (table 3.2.1, Floor Section 2) where the live load is 490 kg/m^2. This live load results in a factored load of 2069 kg/m^2 which is sufficiently close to the design strength of the plank to be acceptable.

For both solid and hollow core slabs, mid-span deflections were found to satisfy the deflection criteria specified in section 9.5 of ACI 318-83.

On the heavily loaded eighth floor, a 200-mm thick reinforced concrete slab is cast on top of the precast floor planks. The required strength for the eighth floor is 1.4(1142 kg/cm^2) + 1.7(730 kg/cm^2) = 2840 kg/cm^2 (table 3.2.1). A conservative estimate of the design strength of this floor system was determined assuming the bond between the precast plank and the cast-in-place slab is insufficient to assure composite action. The design load produces a negative moment which exceeds the negative moment capacity of the cast-in-place slab. Therefore, assuming the slab cracks over the supporting beams where the negative moment is largest, the floor system can be analyzed as being made up of simply-supported slab and planks. The planks (for example, type NV-52-18T discussed above) have a design strength of 2000 kg/m^2. The cast-in-place slab has a design strength of 1930 kg/m^2. Their combined strength of 3930 kg/m^2 is greater than the required strength of 2840 kg/m^2.

4.2.3 Design Strength of Beams

4.2.3.1 Steel Beams

The types of steel beams in the Office Building were described in section 2.3.2. Table 2.3.1 lists the major built-up and rolled beam sections, their dimensions, and their section and material properties. All sections are singly or doubly symmetric.

The flexural design strength of each beam section was determined according to section F1 of the LRFD Specification [4.2]. The flexural design strength is equal to $\phi_b(M_n)$:

$$\phi_b M_n = \phi_b (F_y)(Z) \qquad (4.2.1)$$

where ϕ_b = 0.90; M_n = nominal flexural strength; F_y = yield strength of beam; and, Z = plastic section modulus. Equation 4.2.1 applies when the unbraced length is less than a limiting value, which was true for all cases. For the 13.2-m beams, shear connectors (fig. 4.2.1) were placed at 1-m spacings. These shear connectors attached the compression flange of the beam to the floor slab, and the beam was therefore considered to be braced against lateral-torsional buckling. Therefore, no reductions in the flexural design strength of any of the beams was required.

The design shear strength of each beam section was determined according to section F2 of the LRFD Specifications. Prior to calculating design shear strengths, each beam section was checked for compactness. The 700- and 800-mm deep beams with 10-mm thick webs (types 8, 9, 10, 12, 13, and 18) were found to be non-compact. For compact sections, the design shear strength is equal to $\phi_v(V_n)$:

$$\phi_v V_n = \phi_v (0.6)(F_{yw})(A_w) \qquad (4.2.2)$$

where: ϕ_v = 0.90;
V_n = nominal shear strength;
F_{yw} = yield strength of web; and,
A_w = area of web.

The shear strengths of the non-compact sections were reduced as specified in section F2.2 of the LRFD Specifications.

The largest factored moments and shear forces on each type of beam section were determined. These forces were then compared to the flexural and shear design strengths of each beam section. Table 4.2.1 shows the results of this comparison. In all cases, the design shear strengths exceed the maximum required strengths. The flexural design strengths also exceed the required strengths for all beams, except for built-up section No. 12 and rolled section No. 28. However, for both of these beams the design strength is less than 3 percent under the required strengths, and is not considered significant.

4.2.3.2 Concrete Beams

As described in section 2.3.2 and shown in figure 2.3.12, reinforced concrete beams with inverted T or Z cross-sectional shapes are used in the Office Building. These beams are supported at their ends by concrete corbels or steel brackets, and thus can be analyzed as simply-supported. The steel used in all the beams for both flexural and shear reinforcement was AIII steel which has a nominal yield strength of 4000 kg/cm^2.

All of the concrete beams used in the interior of the building and many of those in the exterior frame are inverted T-beams. Table 4.2.2 lists the various types of inverted T-beams and their specified concrete strengths. Beam designations indicate the length of the beam and the Soviet nominal design strength of the beam. For example, the designation R-86-12 indicates that the beam is 8.6 m in length and is designed to carry a load of 12 t/m.

The flexural design strength of each inverted T-beam was determined according to Chapter 10 of ACI 318-83 and the procedure presented in reference 4.3. The T-beams are doubly reinforced (see fig. 2.3.12(a)), and therefore in the flexural design calculations, consideration was given to whether the compression steel had yielded when the concrete reached its limiting tensile strain of 0.003. Table 4.2.2 lists, for each beam, the areas, A_s and A'_s, of tensile and compressive reinforcing steel, the effective depths, d and d', to the centroids of the tensile and compressive steel, and the width, b, of the web.

The maximum factored moment (required strength) on each of the various types of inverted T-beams was determined assuming beam spans from column centerline to column centerline. The design strengths, required strengths, and the results of a comparison between the two are given in table 4.2.2. For most of the beams the design strength approximately equals or exceeds the required strength. However, there are four short beams for which the design strength is inadequate. Beams R-26-8, R-32-8, R-14-12, and R-38-8 are understrength by 17, 22, 28, and 45 percent, respectively.

The design strengths of the understrength beams were re-evaluated by considering their actual span lengths in computing the factored bending moment. It was assumed that the spans were equal to the distances between the centers of the supporting corbels. For the standard corbel detail, the new span lengths are 550 mm shorter than the distances between column centerlines. This change in span will result in significant reductions in calculated bending moment for the beams with short spans. Based on the shorter span lengths, design strengths and required strengths are as follows:

BEAM	DESIGN STRENGTH (t·m)	REQUIRED STRENGTH (t·m)
R-38-8	13.7	19.6
R-32-8	8.9	7.2
R-26-8	5.8	4.6
R-14-12	2.3	1.6

Thus, only the Type R-38-8 beam is understrength. This type of beam is heavily loaded at two locations in the first floor; these two beams were examined during the site investigation.

The design shear strength of each inverted T-beam was determined according to Chapter 11 of ACI 318-83. Each of the beams contained shear reinforcement along the length of the beam in the form of stirrups (see figure 2.3.12(a)). In some cases, the spacing of the stirrups varied along the length of the beam. Table 4.2.3(a) lists, for each beam, the area of shear reinforcement, A_v, the various stirrup spacings, S1, S2, and S3; the distances X1, X2, and X3 indicate the beginning of the specified stirrup spacing.

The maximum factored shear forces on each of the various types of T-beams was determined. The design strengths, required strength at the start of each stirrup spacing, and the results of the comparison between the two are given in table 4.2.3 (b). In all cases, the stirrup spacings were less than the maximum allowed, and the design shear strength exceeded the shear strengths except for beam R-38-8.

The Z-beams span between columns in the exterior frames on the second through seventh floors. There are two designations for Z-beams: RF-56-8 and RF-50-8. The lower flange of the Z-beam is used to support floor planks on the interior of the building. A portion of the Z-beam is encased by the exterior masonry wall (fig. 2.4.3). The flexural and shear design strengths and the required strengths for the Z-beams are given in tables 4.2.2 and 4.2.3. In all cases, the design strengths of the Z-beams are adequate. Because the beam is an integral part of the wall, torsional rotations are restrained.

The shear strength of the inverted T-beam and Z-beam flanges was also calculated to determine if the shear strength was sufficient to resist the loads imposed by the floor planks which rest on the beam flanges. Using the equation given in section 6.2 of ACI 318.1-83 [4.4] for shear resistance of structural plain concrete, the strength of the concrete flange alone (neglecting shear reinforcement) was found to exceed the required strength in all cases.

4.2.4 Design Strength of Columns

4.2.4.1 Steel Columns

The three types of steel columns used in the upper stories of the Office Building are shown in figure 2.3.5. The nominal axial compressive strength of each of these columns was determined according to section E of the LRFD Specification. Table 4.2.4 lists the three steel columns with their area, moment of inertia, unbraced length, and yield strength. All the flanges, webs, etc. of the steel sections have width to thickness ratios less than the limiting values; therefore, local buckling is not a problem. The critical buckling stress was calculated for each column. The flexural torsional buckling stress for section S-2 was also calculated, but did not control as it exceeded the critical buckling stress. The axial compressive design strength was calculated by multiplying the critical buckling stress and the area of the cross-section by the phi factor 0.85. Although the design specifies that section S-3 (box section) should be filled with concrete, the design strength was calculated based on the area of the steel only.

The moments induced in the steel columns due to eccentrically applied beam reactions are small. A check of the most heavily loaded Type S-1 column showed that a more rigorous analysis involving interaction equations for a combined state of flexure and axial compression (section H1 of the LRFD Specification) was unnecessary.

The largest loads on each column type were identified and the factored loads (required strengths) were compared with the design strengths as shown in table 4.2.4. In all cases, the design strengths exceed the required strengths.

4.2.4.2 Concrete Columns

As discussed in section 2.3.1, there are two types of precast concrete columns: composite columns with a steel core (fig. 2.3.6(a)-(e)) and reinforced concrete columns (fig. 2.3.7(a)-(c)). The design strengths of both types of columns were determined according to the procedures given in Chapter 10 of ACI 318-83. As columns were subjected to both axial loads and moments induced by eccentric beam loads, axial force and bending moment (P-M) interaction curves were developed for each column cross-section. A California Department of Transportation computer program called YIELD, which was developed to calculate P-M diagrams according to procedures specified in the ACI code, was used to obtain the interaction curves. Since the program is intended for reinforced concrete columns, the structural steel cores in the composite columns were represented by equivalent areas of reinforcing bars distributed so as to approximate the core shapes. The concrete and steel material properties used in the calculations were the nominal values specified on the Soviet plans. The specified cylinder strengths of concrete were obtained from the procedure discussed in section 3.4.13.

Slenderness effects were also considered as specified in section 10.10 of ACI 318-83. The unbraced lengths of the columns were taken as the clear heights of columns between floors, and the columns were assumed to be pinned at their ends. The radius of gyration was taken as 0.3 times the column width for the reinforced concrete columns, and for the composite columns equation 10.13 of ACI 318-83 was used. For each column cross-section, the end moments were obtained for the most critically loaded column; these end moments were used to determine whether the column satisfied the slenderness criteria (section 10.11.4.1, ACI code). The results of the slenderness evaluations are given in table 4.2.5.

The reinforced concrete columns are not slender. For these columns, the moments are small enough so that the concentric axial design strength controls the column capacity (section 10.3.5.2 of ACI 318-83). Table 4.2.6 lists the concentric axial design strengths. The comparison of factored loads and column capacities showed that a large number of Type RC-5 reinforced concrete columns in the basement through the fourth stories do not have the required axial design strength. Table 4.2.7 lists the Type RC-5 columns that are understrength and gives the ratio of design strength to required strength.

All of the composite columns must be analyzed as slender columns about both their strong and weak axes, except for types SC-7 and SC-2 which are not slender about their strong axes. Moment magnification factors were calculated and used to determine the required moment capacity for the columns. In this calculation, the product of the elastic modulus (E) and the moment of inertia (I) of the composite section is required. The modulus for concrete was obtained using the equation in section 8.5.1 of ACI 318-83 and equation 10.14 of that Code was used to obtain the composite EI. Because the actual moments on the columns are small, the required moment capacity is based on the moment obtained by multiplying the minimum eccentricity specified in section 10.11.5.4 of ACI 318-83 by the factored axial load on the column. Even after the moments were amplified for composite column Types SC-12, SC-7, SC-5, and SC-2, the moments were small enough so that the column capacity was controlled by the concentric axial design strength. For Type SC-9 columns, the computed weak axis buckling strength is less than the value required to apply the ACI code equations, and thus these columns are inadequate. This is critical were the Type SC-9 columns are not braced by walls in their weak direction. Critical columns are located on the second and third stories at the corners of the core not braced by shear walls (column lines C1/6B and G2/4A). Other heavily loaded Type SC-9 columns in the core are braced by shear walls in their weak direction; therefore the capacity of these columns is controlled by the concentric axial design strength.

The strengths of concrete used in the assessment of the precast concrete column were based on the average cube strengths specified in the plans. For the reinforced concrete columns, the concentric axial design strength is a direct function of the concrete strength. Likewise, the buckling strength of the composite columns is influenced by the modulus of elasticity of the concrete. Thus, for the assessment of the as-built structure, it is necessary to use actual material strengths. For this reason, cores were taken from the columns identified as being understrength.

4.2.5 Design Strength of Beam-to-Column and Beam-to-Beam Connections

The steel beam-to-column and beam-to-beam connections in the Office Building (see section 2.3.3) are primarily welded connections which can be analyzed as simple connections since they are designed not to transmit significant bending moments. The design strength of each type of connection was determined using section J2 of the LRFD Specification. For welded connections with fillet welds loaded in shear, the design strength, R, is:

$$R = \phi[(0.6)(F_{exx})(t)(\ell)] \qquad (4.2.3)$$

where: ϕ = 0.75;
F_{exx} = the tensile strength of weld metal;
t = the weld throat; and,
ℓ = the weld length.

A weld tensile strength of 4600 kg/cm2 was used in all calculations (see section 3.4.2). Connection dimensions and weld sizes were obtained from the Soviet design plans KM.3.5 which show steel connection details.

The shear strength of the web of each beam framing into a connection was also checked. Web shear strength was determined as:

$$R_{web} = \phi[(0.6)(F_y)(A_w)] \qquad (4.2.4)$$

where ϕ = 0.75;
F_y = the yield strength of web metal; and,
A_w = the area of the web.

Beam reactions were calculated using LRFD load factors of 1.2 for dead loads and 1.6 for live loads. The design strength of each type of connection was compared to the heaviest beam end reactions imposed on that type of connection. Table 4.2.8 lists each type of connection, weld dimensions and design shear strengths, web design shear strengths, and the results of the comparison between design strengths and loads. In all cases, the design strength of the connection exceeded the imposed loads.

4.2.6 Summary

The comparisons of the calculated design strengths for planks, beams, and columns with the largest factored loads imposed upon each type of member resulted in the following conclusions:

- The design strengths of precast floor planks are adequate.

- The design flexural and shear strengths of steel beams are adequate.

- The design strengths of steel columns are adequate.

- The flexural and shear design strengths of the concrete beams are adequate except for two R-38-8 inverted T-beams on the first floor. The design shear strengths of the beam flanges which support floor planks are adequate.

- The strengths of the reinforced concrete columns are controlled by the concentric axial design strength for short columns. Thirty-five Type RC-5 columns on the lower stories are understrength. Cores were obtained from a representative sample of these understrength columns to determine the in-place concrete strength.

- The design strengths of composite steel core columns are adequate except for Type SC-9 columns located at the corners of the core in the second and third stories. Cores were taken from these columns.

- The design strength of all beam-to-column and beam-to-beam connections are adequate.

4.3 LATERAL LOADS AND RESISTANCES

As discussed in Chapter 3, the only significant lateral load that must be resisted by the Office Building is wind. This section describes manual and computer analyses to assess the ability of the structure to resist wind loadings. In addition, seismic criteria for the Office Building require that the connections be able to resist a specified amount of lateral load. The ability of the connections to resist this load is assessed.

4.3.1 Loads Acting on Shear Wall System

Lateral design loads for the Office Building were determined on the basis of ANSI A58.1-1982 wind loading criteria (see section 3.3.1.1). The design pressure profile for the Office Building is given in table 3.3.1 and is shown in figure 4.3.1. This pressure distribution accounts for both windward and leeward building pressures and was assumed to act on the 41-m width of the building. Pressures were calculated at the level of each floor and at the top of the parapet wall. Individual story shears were calculated by multiplying the p_z values by the width of the building and the height of a particular floor level. Story shear, total shear, and overturning moments are summarized in table 4.3.1. Shear and moment distributions due to wind load are shown in figure 4.3.2.

4.3.2 Manual Analysis

The following assumptions were made in the manual analysis of the shear wall system: 1) One-half of the moments and shear force presented in table 4.3.2 are applied to each of the two "L"-shaped shear walls; 2) The columns and shear wall panels act as a composite section; 3) The contribution to bending resistance from shear walls perpendicular to wind load is neglected; and, 4) The shear walls perpendicular to the face of the building subjected to wind load are assumed to act as cantilever beams.

The following failure mechanisms were investigated: 1) shear failure of horizontal joints; 2) failure of the shear wall to column connections; and, 3) failure of the shear wall due to overturning.

4.3.2.1 Analysis of Horizontal Shear Wall Joint

Figure 4.3.3(a) shows a plan view of the west shear wall. The direction of the applied wind load was from east to west, and the portion of the shear wall between columns E/4A and Cl/4A was assumed to resist one-half of the total wind load. Figure 4.3.3(b) shows the dimensions used to locate the neutral axis and compute the moment of inertia of the composite cross-section. For this analysis, it was assumed that the entire cross-section was made of concrete. The centroid of the section is 4.72 m east of column Cl/4A. The maximum beam shear stress occurs at the centroid, F_v, and was calculated as:

$$F_v = VQ/Ib = 3.2 \text{ kg/cm}^2 \qquad (4.3.1)$$

where:
- V = total shear force = 36.97 t;
- Q = section moment of area with respect to the neutral axis = 2.5×10^6 cm^3;
- I = principal moment of inertia = 1.6×10^9 cm^4; and,
- b = thickness of shear wall = 18 cm.

To obtain the required shear strength, the result of eq. (4.3.1) must be multiplied by the appropriate factors for dead (D), live (L), and wind (W) loads:

$$0.75(1.4D + 1.7L + 1.7W) \qquad (4.3.2)$$

Dead and live loads do not induce horizontal shear stress in the shear wall joints, so eq. (4.3.2) reduces to 1.3W. Thus, the required shear strength is 1.3(3.2 kg/cm^2) or 4.2 kg/cm^2.

Compressive stresses caused by dead and live loads increase the shear strength of the joint. This effect is ignored in the assessment because the weight of the wall panels is primarily carried by the connections between the panels and the columns, not by the horizontal joint.

By using the relationship for design shear strength of plain structural concrete (section 6.2 of ACI 318.1-83), the required cylinder strength would be 148 kg/cm^2. This assumes that the entire joint (fig. 4.1.1) is effective in resisting shear; this assumption may be unconservative, since it is questionable to rely on bond across a construction joint. If it is assumed that only the columns and castellated pockets resist shear stress (by means of mechanical interlock) the shear area is reduced to two-thirds of the gross area; the resulting required shear strength would be increased by a factor of 1.5. Since shear strength is assumed to be a function of the square root of the compressive strength, the required concrete strength is increased by a factor of 2.25. That is, concrete with a cylinder strength of 330 kg/cm^2 would be required. Soviet design details specify a Mark 300 concrete, for the horizontal shear wall joints. Based on the manual analysis, the concrete should be stronger than Mark 300.

4.3.2.2 Loads in Shear Wall-to-Column Welded Connections

In a prismatic beam, maximum shear stress occurs at the neutral axis. When considering loads on shear wall-to-column connections, the maximum loads will, therefore, occur in the vertical joints closest to the neutral axis of the composite shear wall-column section. The load acting on the connections on the east face of column D/4A in the first story will be used to represent the maximum load condition for the welded connections. For this case, the moment of the area (Q) is 1790000 cm^3; the resulting horizontal beam shear stress is 2.2 kg/cm^2.

Each shear wall panel is connected to a column at three points per story. The height of the first story is 4.8 m. Thus, each of the first story connections must be able to resist the shear force developed in 1.6 m of wall height, or (160 cm)(18 cm)(2.2 kg/cm^2) = 6300 kg; multiplying this by the wind load factor of 1.3 gives a required shear resistance of 8.2 t. Typical details for these shear wall-to-column connections are shown in figure 2.3.21. The strength of the connection is controlled by the shear strength of the two welds connecting the 12-mm plates to the columns. Using eq. (4.2.3) the design strength of the weld is 70.3 t. The strength of connections between shear wall panels (fig. 2.3.22) is also controlled by the shear strength of two welds of similar dimension. Thus the welded connections are adequate.

4.3.2.3 Overturning Moments at the First Floor

According to ACI 318.1-83 for structural plain concrete no tension is permitted to be transmitted across construction joints. The horizontal joint between the shear wall panels can be considered to be a construction joint. Under overturning moment due to wind load, the shear wall section will crack at the joint up to the equilibrium point established by the cracked section neutral axis. Columns within the cracked zone will thus be loaded in tension (or, more precisely, unloaded, since dead load axial forces are already present). As an example, see column E/4A in figure 4.3.4. The total unfactored overturning moment at the first floor level is 1680 t·m, half of which is resisted by each shear wall. The location of the neutral axis was determined by an iterative procedure, and is as shown in figure 4.3.4; the transformed moment of inertia was 25.5 m^4.

The maximum tensile stress for this section occurs at column E/4A and was calculated to be 17.1 kg/cm^2. Using the transformed section for E/4A gives a total tensile force of 88 t. However, the unfactored dead load in this column is 459 t. Using the appropriate load factors on dead (0.9) and wind (1.3) loads, the resulting load is 299 t (compression), and column E/4A remains in compression.

The maximum compressive stress occurs at column C1/4A and was calculated to be 12.5 kg/cm^2. The resulting compressive force is 65 t. At the first floor, the maximum factored compressive load considering dead, live, and wind loads is 680 t, which is less than the combined dead and live load condition (796 t) and therefore does not control.

4.3.2.4 Overturning Moments at the Pile Cap Level

At the basement level, the shear walls are made of cast-in-place concrete; the walls are 300 mm thick. Figure 4.3.5(a) is an elevation view of the west shear wall between columns C1/4A and E/4A. According to Soviet drawing 5KR·1·1·03e, the walls are anchored into the pile cap with pairs of 20-mm reinforcing bars at spacings of 380, 290, and 340 mm as shown in figure 4.3.5(a). The pile cap is 1.2 m below the basement floor and the overturning moment at this elevation is 1080 t·m. For this case, it was assumed that there was no bond between the shear wall and pile cap. Thus the resisting cross-section is composed of the columns and the anchor bars. The transformed areas (in terms of concrete) of the columns and bars were used to determine the location of the neutral axis, as shown in figure 4.3.5(b). The transformed moment of inertia is 22.4 m^4. The wind induced loads in columns E/4A and C1/4A are 114 t (tension) and 110 t (compression), respectively. As was the case at the first floor, these loads do not result in a net tensile force in column E/4A nor do they produce the controlling compression loading in C1/4A. The factored tensile stress in the anchor bars adjacent to column E/4A is about 200 kg/cm^2, which is well below the yield strength of 4000 kg/cm^2. Thus the connection of the shear walls to the pile cap is adequate.

4.3.2.5 Summary

On the basis of the above calculations, the shear wall design is sufficient to safely resist the design wind loading provided that the horizontal shear wall joints are properly filled with concrete having a cylinder strength of at least 440 kg/cm^2. The Soviet plans call for Mark 300 concrete, which is inadequate. Thus, for the assessment of the shear wall system, the in-place strength of the concrete in the horizontal joints must be determined.

4.3.3 <u>Computer Analysis of the Shear Wall</u>

In the manual analysis, shear wall joints were assumed to be filled and the walls were assumed to act as monolithic sections. When concrete is not present in the shear wall joints, the resistance to lateral loads occurs by an entirely different mechanism. This complex behavior is best illustrated by means of computer-generated results from the lateral load finite element model of the structure. The contour lines on the various plots presented in this section delineate zones of equal stress in the shear walls. The computer analysis was performed for a maximum wind pressure of 66 kg/m^2 at the top of the building, whereas a value of 79.7 kg/m^2 must be used to satisfy the ANSI A58.1 minimum load requirement (see section 3.3.1.1). Hence the stress shown in the display must be increased by a factor of 1.21.

The behavior of the lateral load model with filled joints is presented first. Figure 4.3.6 shows an isometric view of the overall structure with contours of equal maximum principal stress on the shear walls. This view is from the southeast corner and the wind loading is from the east. Column elements have been erased for clarity of the stress display. This figure shows that the north-south shear wall panels (those perpendicular to the lateral load) play an important role in resisting the applied load. This contribution was neglected in the manual analysis.

Figure 4.3.7 shows that when the joints are filled, the shear wall panels parallel to the applied lateral load behave, as expected, like monolithic cantilever beams.

Figure 4.3.8 shows a close-up of the basement and first story shear wall panels between columns E/4A and C1/4A. The maximum unfactored shear stress in the horizontal joints is 2.6 kg/cm^2. The factored shear stress (ignoring the increase in shear stress caused by compressive gravity loads) is equal to 3.3 kg/cm^2, which is close to the value of 4.2 kg/cm^2 obtained by the manual analysis. The lower value obtained from the finite element analysis can be attributed to participation of the perpendicular shear wall section in the load resistance, as well as to the portion of the shear resisted by the columns.

The behavior discussed above is for filled joints and it is in sharp contrast to that obtained when vertical and horizontal shear wall joints are empty. To investigate the extreme effects of joint conditions, the lateral analysis was performed with empty vertical and horizontal joints in all stories. This is not truly representative of the real structure because the basement shear walls are cast-in-place and there are no vertical gaps between the columns and walls. Figure 4.3.9 is a plot of maximum principal stress for shear walls with empty joints; compare this figure with figure 4.3.7. There are several important differences: 1) The individual shear wall panels act independently. Figure 4.3.10 shows that the flexural stress fields in each basement shear wall panel are independent of the stresses existing in adjacent panels (compare with fig. 4.3.7). When each wall panel acts independently, the shear wall system is more flexible; the analysis showed deflections that were 225 percent larger than when the joints were filled. 2) The maximum tensile stress in the lower story shear wall panels increases by a factor of approximately 1.5. 3) A tension stress field is set up within each shear wall panel.

Figure 4.3.11 shows a close-up of the critical basement and first story shear wall panels between columns E/4A and C1/4A. The high principal tensile stresses in the lower right corners of the basement panels are attributable to flexural effects, since these panels are fixed at their base to the pile cap by means of dowel bars (fig. 4.3.5(a)). Diagonal tension stresses as high as 4.8 kg/cm^2 (unfactored) were calculated near the upper left (west) connection bracket at C1/4A. However, the actual stresses in the basement wall panels are much less than indicated by the analysis for the following reason. In the lateral loads computer model a uniform thickness (transformed thickness) of 185 mm was used for all the shear walls; however, subsequent study of the cast-in-place basement portion of the shear walls showed this particular section to be 300 mm thick. Therefore, the largest unfactored diagonal tension stress is 3.0 kg/cm^2, or applying appropriate load factors, 3.8 kg/cm^2.

The allowable shear stress in plain concrete (section 6.2, ACI 318.1-83), for Mark 300 concrete called for in the Soviet drawings is 4.7 kg/cm^2. In a section loaded predominantly in shear, as these panels are, the allowable diagonal tensile stress is equal to the allowable shear stress. Thus the basement shear wall panels have adequate strength.

The maximum tensile stresses in the first story shear wall occur at the top connector to column C1/4A and the bottom connector to column D/4A (fig. 4.3.11); the factored stress has a value of 5.3 kg/cm^2. Thus the required diagonal tensile strength would exceed the design shear strengths if the joints are unfilled and Mark 300 concrete were present in the shear walls.

When the shear wall joints are empty, the shear wall-to-column connections experience greater loads than when the joints are filled. In addition, the columns must transmit all the shear force between floors (see section 4.3.4). In figure 4.3.12, the horizontal and vertical forces that exist at the shear wall connections when the joints are full are compared with the forces which exist when the joints are empty. This comparison is made for the connections between the shear wall panel and the east face of column line C1/4A. As an example, the maximum vertical force in the connections increases from 0.06 t (negligible) to nearly 3.8 t when the joints are empty; the horizontal force increases from 0.34 t to 7.2 t. The resultant factored force vector for these maximum values is 10.6 t, which is still well below the 71 t design strength calculated in section 4.3.2.

4.3.4 Lateral Load Effects on Columns

Lateral load induced axial loads, shears, and moments at each story for column lines C1/4A and E/4A were plotted from the results of the lateral load computer analysis. The application of wind load against the east face of the building induces axial compression forces into column line C1/4A and axial tensile forces into column line E/4A. Figures 4.3.13(a) and (b) show the axial load and shear force for column line C1/4A for the cases of empty shear wall joints and for fully filled joints, respectively. The largest axial compressive loads induced by wind occur in the lower story columns; maximum values of 28.3 t for the wall with empty joints and 23.3 t for the wall with fully filled joints were obtained. Shear forces induced by wind were small, being about 2 t for the wall with full joints and 5 t for the wall with empty joints. Maximum column moments were also small, being less than 0.8 t·m for the wall with full joints and about 3 t·m for the wall with empty joints. The plots of axial load and shear for column line E/4A are similar, except that the axial loads are tensile, with a maximum value of 33.5 t.

The shear forces are less than one-tenth of the shear strength of the concrete columns. Moments are negligible. The highest axial compressive loads occur in column C1/4A. For column C1/4A, the factored load is given by eq (4.3.2) and is equal to 776 t. This load is less than the 897 t obtained for gravity loads alone for a type SC-12 column (the basement story column). Thus, in support of the manual analysis, it is concluded that the wind load effects on columns are insignificant.

4.3.5 Seismic Requirements

ANSI A58.1-1982 requires that connections be able to transmit a seismic (lateral) force equal to five percent of the dead weight of the members that it supports. This is the only seismic requirement for the Office Building (see section 3.3.3).

For seated connections between the major steel beams and columns (fig. 4.2.1), lateral load is resisted by a plate which is welded to the column and to the top flange of the beam and by four erection bolts. The design strength of the welded top plate is controlled by the shear strength of the plate which is 13.8 t. The largest unfactored dead load on this type of connection is 66.1 t. Five percent of this value is 3.3 t. Thus the top plate provides the required lateral resistance.

For connections where beams are supported by column corbels or steel brackets (figs. 2.3.13 and 4.2.1), the lateral load is resisted by the welded top plate and the welds made between the supporting corbel or bracket and the beam. The largest unfactored dead load imposed upon this type of connection is 25.3 t. Five percent of this value is 1.3 t. The design shear strength of the top plate alone is 7 t, which provides the required lateral resistance.

4.3.6 Summary

The following conclusions were drawn from the wind load analyses presented in this section:

- The as-designed shear wall system is sufficient to resist the design wind loading. The shear wall system can be assumed to act as a monolithic cantilever beam provided that the horizontal joints are properly filled with concrete.

- Lack of concrete and grout in the horizontal and vertical shear wall joints will lead to substantially greater lateral deflections if the shear walls are considered to provide the sole mechanism for resistance to lateral loads.

- Lack of concrete in the horizontal shear wall joints will lead to the formation of diagonal tensile stresses in the shear wall panels.

- The capacity of the shear wall-to-column welded connections are adequate even for the condition of the empty joints.

- Wind loading does not introduce significant axial and shear forces into the columns.

- The lateral load resistance of the beam-to-column connections satisfies the seismic requirements.

4.4 ASSESSMENT OF FOUNDATION

4.4.1 Structural Strength of Piles

Precast concrete piles of rectangular cross-section, 250 × 350 mm, were used; the specified concrete strength is Mark 300, corresponding to a cylinder strength of 184 kg/cm^2 (see section 3.4.1). The specified strength of 184 kg/cm^2 corresponds to a modulus of elasticity of 202,000 kg/cm^2 based on ACI 318-83 [4.1]. Section 2909(d) 1 of the 1985 Uniform Building Code (UBC) [4.5] specifies the minimum concrete strength to be 3000 psi (211 kg/cm^2) which is greater than the computed cylinder strength. However, tests performed at NBS indicate that the ratio of cylinder strength to cube strength is approximately 0.9, instead of the 0.75 used in Soviet practice. Based on a 0.9 ratio, the equivalent cylinder strength for Mark 300 concrete is 224 kg/cm^2. Compression test on cores (to be discussed in Chapter 5) taken from other precast elements made of Mark 300 concrete (the shear walls) indicate that the average strength was 430 kg/cm^2 with a coefficient of variation for the sample of 0.13, which would produce a 10 percent exclusion limit concrete strength of 358 kg/cm^2; this strength corresponds to a modulus of 286,000 kg/cm^2. Rebound (unloading) slopes from three static pile load tests which will be discussed later in this section also indicate that the concrete modulus of the piles was much higher than 202,000 kg/cm^2. Therefore, there is strong evidence that the concrete strength of the as-built piles exceeded the specified strength by a substantial margin.

Section 2909(d) 2 of the UBC specifies minimum requirements for reinforcing ties. A review of the pile reinforcement details (Soviet document REG. 731) indicated that the UBC requirements are satisfied.

Allowable stresses are stipulated in section 2909(b) 2 of the UBC to be 0.33 f'_c for concrete and 0.34 f_y or 25,500 psi (1795 kg/cm^2), whichever is less, for steel.

The piles were reinforced by four 12-mm diameter longitudinal bars of A-II steel (yield strength of 3000 kg/cm^2). Thus the design capacity is equal to $0.33(184\ kg/cm^2)(25\ cm)(35\ cm) + 0.34(3000\ kg/cm^2)[4(1.13\ cm^2)] = 57{,}700$ kg or 57.7 t. This allowable stress design capacity is used for comparison with service (unfactored) loads.

4.4.2 Geotechnical Load Capacity

The geotechnical load capacity of piles can be estimated on the basis of assumed strength properties of the soil profile. Such estimates should be verified by specifying load tests, monitoring of driving resistance, or more preferably both. The Soviets presented calculation sheets in which the geotechnical load capacity was estimated for piles driven into the weathered limestone layer, which is approximately 7 m below the pile butt elevation. Available information indicates that specifications called for piles to be driven to practical refusal into the limestone layer. There are also records from three static load tests, and additional information indicates that about three percent of the piles were re-driven some period of time after they were initially installed to either verify their original driving resistance or to check for heave and relaxation.

4.4.2.1 Load Capacity Estimates on the Basis of Soil Profiles

Subsurface data on the U.S. Embassy site were obtained from four subsurface exploration sites near the four corners of the Office Building (fig. 2.2.1). Soil profiles were developed on the basis of the exploration information and in-situ and laboratory tests on the soil and rock deposits. The soil profiles that were developed and the designers' estimated values for frictional and bearing resistance are shown in figures 2.2.2 and 2.2.3. The designers' estimated pile load capacity is based on the skin friction values shown, and a point bearing resistance of 54.5 kg/cm^2 in the limestone stratum. The resulting estimated ultimate pile capacities range from 81 t at boring C-3 to 95 t at boring C-2. These static calculations would indicate a factor of safety for the 57.7 t structural design capacity on the order of 1.5 to 2.0.

4.4.2.2 Load Capacity Estimates on the Basis of Pile Tests

Data from three static pile tests are available. Two of the piles tested, piles No. 371 and 367 (refer to fig. 4.4.1), are in an area where driving stopped short of practical refusal, and one test (pile No. 353) is for a pile which penetrated to only a shallow depth. The results of these load tests are shown in figures 4.4.2 through 4.4.4.

The data in table 4.4.1, figures 4.4.2 through 4.4.4, and the discussion that follows are based on brief summaries that were made available to NBS. Detailed test and installation records were not available for review. A pile load test only produces data on the relationship between the load and displacement of the pile under the particular test conditions. Unless the pile suddenly plunges when the load reaches a critical level, the determination of ultimate bearing capacity based on load settlement data is to some degree a matter of interpretation.

Section 2908(c) of the UBC states three methods for determining the allowable axial load of a single pile by a load test: 1) The allowable load shall not exceed 50 percent of the "yield point" under the test load; 2) The allowable load shall not exceed 50 percent of the load which causes a net settlement of 0.25 mm per ton of test load which has been applied for at least 24 hours; 3) The allowable load shall not exceed 50 percent of the load under which no settlement takes place in a 40 hour period. Only one of the listed criteria must be satisfied. Because the test load was not held long enough to determine compliance with criteria 2 or 3, only compliance with criterion 1 is considered.

The implication of criterion 1 is that the load test results should indicate a minimum factor of safety of 2 against bearing capacity failure under service load. The yield point (or ultimate bearing capacity) is defined as the load which produces a disproportionate increase in settlement.

In the following evaluation of the test results, a procedure first suggested by Davisson [4.6] is used to determine the yield load. This procedure makes use of an index displacement that is defined as elastic compression of the pile shaft under point bearing conditions. The ultimate capacity is defined as the load which results in a butt displacement that is approximately 6.35 mm greater than the index displacement. The index lines in figures 4.4.2 through 4.4.4 represent the index displacements calculated for assumed pile lengths and concrete moduli at the time of testing. Two index lines are shown. One is based on the concrete modulus of 202,000 kg/cm^2 which corresponds to the specified concrete strength of 184 kg/cm^2, and the other is based on a concrete modulus of 286,000 kg/cm^2 which corresponds to a cylinder strength derived from tests of cores taken from precast Mark 300 concrete elements. The Davisson failure criterion is defined by a line on the load-settlement plot that is parallel to and offset 6.35 mm below the index line. Failure lines corresponding to a concrete modulus of 286,000 kg/cm^2 are shown in figures 4.4.2 through 4.4.4.

Because none of the load tests cross the failure line, the ultimate bearing capacities of the piles were not directly determined. Hence, based on a strict interpretation of UBC criterion 1, the piles would have allowable loads of 1/2 the maximum test load, and the safety factor under this load would be in excess of 2. Following is a more detailed discussion and evaluation of each test.

<u>Pile No. 371</u> was subjected to a load test approximately 7 months after installation to a final penetration rate of 2.9 cm per ten blows and a final penetration of 11.2 m. The pile was loaded in increments, with each increment maintained for 2 to 3 hours up to a load of 90 t. The load was then raised to a maximum level of 100 t for a short duration.

The gross butt displacement was 3.4 mm at the maximum test load of 100 t. After unloading, the net butt displacement was 1 mm. The small gross and net displacements indicate that the pile had an ultimate bearing capacity considerably in excess of the 100 t test load. Because of the small displacements, it is not possible to evaluate the ultimate capacity by extrapolation. It is estimated that the ultimate geotechnical load capacity of this pile exceeds 120 t by a considerable margin.

The butt displacements of pile 371 plot above the elastic index lines shown in figure 4.4.2 throughout the loading cycle, indicating that either: 1) the resistance mobilized was primarily friction; or 2) the elastic index lines do not apply to this pile. The pile length at the time of testing was not reported and the indicated index line is based on an assumed pile length of 11.7 m (reported penetration depth is + 0.5 m).

Pile No. 367 was subjected to a load test approximately 6.5 months after installation to a final penetration rate of 6.5 cm per 10 blows and a final penetration of 10.2 m. The pile was loaded in increments, with each increment maintained for approximately 3 hours up to a load of 90 t. The load was then raised to a maximum level of 100 t for approximately 0.5 hours. At the maximum test load of 100 t, the butt displacement was 5.1 mm. After unloading the net butt displacement was 2.5 mm. These small gross and net displacements indicate that the pile has an ultimate capacity considerably in excess of the 100 t test load. It is estimated that the ultimate capacity exceeds 120 t.

The butt displacements for pile 367 plot above the upper elastic index line in figure 4.4.3 during the loading cycles for loads less than approximately 70 t, indicating that either: 1) the resistance mobilized approximately 50 t in friction; or 2) the pile modulus was higher than that assumed for the elastic index line.

Pile No. 353 was subjected to a load test approximately 7 months after installation to a final penetration rate of 0.8 cm per 10 blows and a final penetration depth of 3.5 m. The pile was loaded in increments, with each increment maintained for approximately 3 hours up to a load of 90 t. At the maximum test load of 90 t the gross butt displacement was 6.5 mm. After unloading the net butt displacement was 3.7 mm. Based on a net displacement of 3.7 mm, Pile No. 353 is judged to have a bearing capacity in excess of the 90 t test load. Projection of the load-displacement curve and use of the Davisson criteria indicate that the ultimate capacity of pile 353 is approximately 100 t.

The butt displacements of Pile 353 plot below the elastic index lines in figure 4.4.2 throughout the test indicating that there was negligible friction resistance during the test. The indicated index line is based on an assumed pile length of 4 m (reported penetration depth + 0.5 m). The estimated elastic compression of the pile shaft (approximated by the elastic index line) represents only a small portion (± 1.5 mm) of the 6.5-mm gross displacement at 90 t. The remaining displacement must represent either: 1) tip movement; or 2) inelastic compression of the pile shaft caused by damage during installation. Generally structural damage would be reflected in high creep displacements or unusual rebound curves, neither of which are evident from the data. Hence it appears that the difference between pile 353 and the other two piles is attributable to differences in the behavior of the supporting soil.

A strict interpretation of the load tests on the basis of UBC criteria would indicate allowable loads of 45 t for pile 353 and 50 t for piles 367 and 371, because the load tests failed to verify higher loads. However the rational evaluation of the test results presented above indicates that allowable loads of 50 t for pile 353, and 60 t or higher for the other two piles would have been obtained if the tests had been carried to higher loads. Thus two of the three piles tested have load capacities in excess of the 57.7 t structural load capacity of the piles. The third pile probably has a geotechnical load capacity of 50 t.

4.4.2.3 Load Capacity Estimates from Wave Equation Analysis

Computations based on the one-dimensional wave equation analysis of the pile-soil-hammer-cushion system result in a predicted relationship between the pile capacity at the time of driving and the driving resistance [4.7]. In order to perform such an analysis, accurate information on the hammer and cushion characteristics is required. The following information was received:

Type of pile driver:	overhanging C 330 drop hammer
Weight of Hammer:	2.5 t
Drop of hammer:	1.6 meters
Pile cap:	welded steel
Drive shoe:	wooden insert

Details such as weight of the drive cap, dimensions and properties of the wooden insert, and operating characteristics of the hammer were not known. To compensate for these deficiencies in information, assumptions were made and parametric studies were conducted to determine lower and upper bound effects that could result from errors in these assumptions. The following variables were used:

Soil properties: Standard values of soil quakes (0.1 in.) and damping (0.05 s/ft at side and 0.15 s/ft at tip) were used, and the percentage of point resistance was assumed to vary from 60 to 90 percent.

Pile properties: The concrete modulus was taken as 202,000 kg/cm^2, except in one analysis a modulus of 282,000 kg/cm^2, and the effect of the four 12-mm bars was considered in estimating pile stiffness. Pile lengths of 7.3 and 12.2 m were considered.

Drive head weight: A drive head weight of 182 kg was assumed.

Cushion block: The cushion block was assumed to have stiffnesses ranging from 62,500 kg/cm to 250,000 kg/cm, and a coefficient of restitution of 0.5. These assumptions were based on an assumed wood modulus of approximately 2,800 kg/cm^2 which is the typical value for oak cushions in the U.S. For cushion areas equal to the pile areas, the stiffness range corresponds to cushion thicknesses ranging from 10 to 40 cm.

Hammer characteristics: In the absence of detailed information on the characteristics of the diesel hammer used to drive the piles, a drop hammer model was used in the analysis. Operation efficiency of the hammer was varied from 50 to 90 percent.

The effects of pile length and percentage of point resistance on the predicted ultimate pile capacity are shown in figure 4.4.5 for a hammer efficiency of 70 percent and a cushion stiffness of 125,000 kg/cm^2. The variables of pile length and percent of point resistance do not have a significant effect on penetration rates. Therefore subsequent analyses were performed for pile lengths of 7.3 m and a 90 percent point resistance, which represent the lower bound condition in figure 4.4.5. Increasing the concrete modulus (pile stiffness) was also found to have little effect on penetration rates (the predicted capacity slightly increases while the driving stresses tend to be somewhat lower).

Figures 4.4.6 and 4.4.7 show the effects of cushion stiffness and hammer efficiency, respectively, on penetration rates. The predicted pile capacity is significantly affected by both of these variables.

For the purpose of predicting the pile capacity where driving was stopped at penetration rates from 2 to 8 cm per 10 blows, the 50 percent and 70 percent efficiency curves in figure 4.4.7 were selected to provide conservative estimates of the ultimate capacity range. The curves are based on 7.3-m piles, a 90 percent point resistance, and a cushion stiffness of 62,500 kg/cm^2. The 70 percent hammer efficiency curve is taken as an upper bound and the 50 percent efficiency curve is taken as a lower bound. The curves approximately apply to all pile lengths used, as well as to the range of point resistances that can be reasonably expected on this project.

It can be deduced from figure 4.4.8 that the capacity of piles driven to penetration rates from 5.5 to 6.5 cm for 10 blows would have ultimate capacities ranging from 95 to 120 t, and that piles driven to penetration rates from 3 to 3.5 cm for 10 blows would have ultimate capacities ranging from 115 to 140 t.

4.4.2.4 Conclusions on Geotechnical Load Capacity

The subsurface data shown in figures 2.2.2 and 2.2.3, as well as the data from the remainder of the site show a continuous stratum of weathered Carboniferous limestone whose upper surface under the Office Building is at elevations ranging from 123.3 to 126.3 m. It was reasonable for the geologists to assume that piles supporting the Office building could be driven to bearing on this layer.

Of the 1,092 piles driven for the office building, 1,068 were driven to penetration rates of 1 cm or less per 10 blows. The furnished length of the piles was typically 7 m and the final penetrations were generally on the order from 4 to 6 m. The 24 remaining piles were stopped at final penetration rates on the order of 2.5 to 6.5 cm per 10 blows and penetrations of approximately 6.2 to 11.2 m. One of these piles (no. 770) is in the area of the core and is of little concern, since it is a single pile near the center of a group of 192 piles. The other 23 piles are concentrated in an area which includes the entire foundation for column J1/4, and part of the foundations for columns J1/5 and J1/6.

The reason for this localized change in pile resistance must be the nature of the subsurface conditions in the vicinity of column J1/4, and could be attributed either to a depression in the surface of the limestone layer which would have been filled by less consolidated material or alternately more severe weathering of the limestone at that particular location. In accordance with requests from the U.S. project engineer at the site, borings were taken to explore this condition. Logs of these borings have not been made available.

Most of the piles driven in this area were 12 m long (the longest length available from the precasting plant) and were driven on December 27, 1979. However, some 7-, 8-, 9-, and 11-m piles were driven between December 10 and December 27, 1979. It is reasoned that the contractor decided to switch to the longer piles after some shorter piles could not be driven to refusal. Two of the previously discussed pile tests were performed in that area.

Table 4.4.2 lists all the piles that could not be driven to refusal, together with the data obtained from the driving logs, conservative lower bounds and best estimates of the geotechnical load capacity obtained by a one-dimensional wave equation analysis, and results of static pile tests. (Pile locations are identified in figure 4.4.1.)

In summary, it is concluded on the basis of available information on subsurface conditions, static pile tests, and pile driving records that the geotechnical load capacity of the piles is 100 t or more and that the safety factor with respect of the allowable pile load of 57.7 t generally exceeds 2.0.

4.4.3 Installation Stresses

The fact that piles have adequate structural strength and geotechnical load resistance does not by itself ensure that their integrity was not impaired during their installation. Two potential causes of damage are handling stresses and driving stresses.

4.4.3.1 Handling Stresses

Lifting hooks are installed so that piles are picked up with the longer (35 cm) side in a vertical position and positioned to minimize handling stresses. The following dimensions are specified:

Pile Length (m)	Distance from ends to Lifting Hooks (m)	Distance Between Lifting Hooks (m)
7	1.45	4.1
8	1.7	4.6
9	1.9	5.2
10	2.1	5.8
11	2.3	6.4
12	2.5	7.0

The largest stress occurs in the 12-m pile. Using the weights listed in the Soviet pile catalog (REG. 731), the maximum negative static moment of a lifted pile is 717 kg·m at the lifting hooks, and the maximum positive moment is 699 kg·m in the center of the pile. The ultimate cross-sectional resisting moment is 1649 kg·m. This leaves an adequate margin for impact loads. Thus the piles have adequate strength to resist handling stresses.

4.4.3.2 Driving Stresses

Driving stresses obtained by the wave equation analysis are shown in figure 4.4.8. The driving stresses shown are for a cushion stiffness of 62,500 kg/cm. If a stiffer cushion block were used, calculated driving stresses would be higher. For instance, for a 125,000 kg/cm cushion stiffness and a 70 percent efficiency, driving stresses as high as 265 kg/cm^2 were calculated. A comparison of specified concrete compressive strength and tensile capacity of the pile (based on 12-mm vertical bars) with the calculated driving stresses in compression as well as in tension indicates that the driving stresses were uncomfortably high. Even though concrete strength under short loading pulses is higher than the standard cylinder strength, this strength increase is offset by the effects of low cycle fatigue. Therefore, in U.S practice, driving stresses are kept below the cylinder strength [4.6].

Driving the piles to a resistance of less than 1 cm for the last 10 blows was not prudent. The preceding analyses indicate that adequate geotechnical resistance could have been developed by driving the piles to a penetration of 5 cm for the last 10 blows. Non-prestressed, precast concrete piles are not commonly used in U.S. practice. However, prestressed piles, which have higher strength concrete than that specified in the Soviet catalog, would not normally be driven to a resistance greater than 6 to 7 blows to the inch, which would correspond to a penetration of about 3.5 to 4 cm for the last 10 blows.

The observation that calculated driving stresses were high and driving was hard is corroborated by the following quotation from a letter dated February 22, 1980: "...when a pile reached 'refusal', i.e. the tip got embedded in rock, stopping the downward movement of the pile, more often than not the pile top would start disintegrating until the hammer stopped. This did not cause any problem, because the upper 3 to 4 feet of all piles were chopped down before the pile caps were poured."

In general it is reasonable to assume that the high driving stresses were confined to the upper part of the pile which was removed after driving. However it cannot be completely ruled out that some piles may have sustained damage. Such damage is not likely to affect their load capacity (a decrease in pile strength would have been observed in the field as a drop in driving resistance), but it could affect their durability. As noted in the site description, the groundwater has a high sulfate content; it is therefore aggressive to regular concrete. The records indicate that a sulfate resisting cement was used in the concrete for the piles.

4.4.4 <u>Foundation Design</u>

4.4.4.1 Vertical Loads and Overturning Moments

Columns around the building perimeter rest on pile caps which are supported by pile groups. The core is supported by two pile mats. All piles are spaced

1.1 m apart. In analyzing the forces transmitted to individual piles, the pile groups were assumed to act compositely as a single member. Thus ordinary beam theory was used to analyze the effects of eccentric loads on the pile groups.

Columns 1A/C through 1A/H and 9A/C through 9A/H are supported by pile groups of 20 piles each. The pile groups are 4 piles wide and 5 piles long with the long axis perpendicular to the perimeter wall. The pile groups support a maximum unfactored column load of 560 t at their center, a 232 t load resulting from the supported portion of the exterior masonry wall and acting at an eccentricity of 0.6 m, and a 5.0 × 3.0 × 1.1-m thick pile cap weighing 59 t. The maximum unfactored pile load resulting from these loads is 49 t and occurs at foundation 1A/E.

Columns J1/3 to J1/8 are supported by pile groups of 15 piles each. The pile groups are 3 piles wide and 5 piles long with the long axis perpendicular to the perimeter wall. The pile groups support a maximum unfactored column load of 354 t at their center which occurs at column J1/5, a 232 t load from the exterior masonry wall acting at an eccentricity of 0.6 m and a 2.8 × 5.0 × 1.1-m thick pile cap weighing 42 t. The maximum unfactored pile load resulting from these loads is 50 t.

Columns A1/4 through A1/7 and the four corner columns are supported by groups of 16 piles; pile groups are square. Pile groups support a maximum unfactored column load of 337 t at column A1/7 acting at an eccentricity of 0.35 m, a 232-t load from the exterior masonry wall acting at an eccentricity of 0.25 m, and a 3.9 × 3.9 × 1.1-m thick pile cap weighing 46 t. The maximum resulting unfactored pile load is 43 t.

Columns A1/3 and A1/8 are supported by groups of 12 piles each. The pile groups are 3 piles by 4 piles with the long axis perpendicular to the perimeter wall. The pile groups support a maximum unfactored column load of 248 t acting at an eccentricity of 0.35 m, a 232-t masonry wall load acting at an eccentricity of 0.25 m, and a 2.8 × 3.9 × 1.1-m thick pile cap weighing 33 t. The maximum resulting unfactored pile load is 46 t.

The two halves of the core are supported by two large pile groups of 192 piles each, which support large pile mats. The pile groups are 16 piles by 12 piles, with the long axis in the north-south direction. On the slightly more heavily loaded eastern mat the pile group resists a resulting unfactored vertical load of 6842 t transmitted by the columns, and a load of 597 t from a 17.1 × 12.7 × 1-m thick pile cap. The resultant column load acts at an eccentricity of 0.17 m in the north-south direction and an eccentricity of 0.11 m in the east-west direction. The maximum resulting unfactored pile load is 40 t. In addition, each pile group resists an overturning moment of approximately 1,000 t·m caused by the unfactored wind load (fig. 4.3.7) which generates maximum pile loads of an additional 1.7 t for wind acting in the north-south direction, and 2.2 t for wind acting in the east-west direction.

4.4.4.2 Pile Caps

The pile caps and mats are 1.1 and 1.0 m thick, respectively, and they are made of Mark 300 concrete reinforced with 36-mm grade AIII bars at a spacing of 200 mm in both directions. The punching shear resistance was analyzed according to Chapter 15 of ACI 318-83. Two heavily loaded locations were examined: at column G2/4A in the core where the factored column load is the largest (937 t); and, at exterior column C/9A where the combined factored load from the structure and the exterior masonry wall is 1175 t. The punching shear resistance was found to be sufficient at both locations.

4.4.4.3 Lateral Loads

The total lateral load acting on each of the pile groups supporting the core of the office building is approximately 35 t (fig. 4.3.2). The pile cap acts like a stiff membrane, and the shear wall imparts a lateral load to one edge of the pile group; the piles must resist the direct shear force plus a shear force produced by torsion. The total resulting maximum shear force imparted to the most critically loaded pile (in the corner of the group) is 411 kg for wind acting in the north-south direction and 536 kg for wind acting in the east-west direction. The average shear force per pile is smaller. These loads can be safely resisted by the pile foundation.

4.4.4.4 Negative Friction

Negative friction could be caused if compressive load is imposed on compressible soil strata surrounding the piles after the piles are driven. Clay layers would be of concern. In accordance with figures 2.2.2 and 2.2.3 the piles are embedded for approximately 50 percent of their height in fluvioglacial sandy deposits and for the other half in clays. These clay deposits are either very ancient or of more recent glacial origin. Even though no records of odometer tests are available, it is reasonable to assume that the ancient, as well as the glacial clays were subjected to heavy pre-consolidation loads, caused by overburden layers which subsequently eroded and by the weight of the glacial ice sheets which covered the area. The weight of the 7-m overburden removed during construction is approximately 12 t/m^2. The total weight of the building is approximately 20 t/m^2. Most of the building weight is transmitted by the piles to the underlying limestone stratum. Only a very small portion of that weight, such as basement slabs, would not be directly supported by the piles. Thus the weight imposed on the soil layers surrounding the piles is much less than the weight removed by the building excavation, and negligibly small when compared to the pre-consolidation pressures to which the clay layers were subjected in their geological past. Thus negative friction effects are considered negligible.

4.4.4.5 Settlements

Most of the weight of the structure is transmitted by the piles to the carboniferous limestone layer in which the piles are embedded. The limestone layer is underlain by a 7-m thick layer of middle carboniferous clay marl. The compression of the limestone layer would be negligible, however the clay marl is much more compressible and could be subject to consolidation settlement. The net load imposed by the building (the difference between the load removed by excavation and the weight of the building) is approximately 8 t/m^2. This is an extremely small pressure, and the resulting total settlement would not be significant. Potential differential settlements effects between foundation J1/4 where the piles were not driven to refusal and adjacent foundations were considered. In accordance with the results of the static load test data these differential settlements (if any) should be very small and would not cause any visible structural distress.

4.4.5 Summary

The entire building is supported by a pile foundation. Most of the piles were driven to refusal into a layer of decomposed limestone; however, several piles could not be driven to refusal. The piles are of precast reinforced concrete with a 250 × 350-mm cross-section. They were delivered to the job site in lengths ranging from 7 to 12 m, and driven to depths ranging from 2 to 11.2 m. The remaining length was cut off in the field, however a 250-mm length of the vertical bars was left protruding to be embedded in the pile cap. The peripheral columns are supported by pile groups of various sizes, capped by 1.1-m thick pile caps. The core columns and shear walls are supported by two large pile groups, capped by 1-m thick pile mats.

The allowable unfactored load for the piles is 57.7 t and the unfactored dead and live loads acting on the piles are 50 t or less. There are no batter piles to resist horizontal load, but the maximum horizontal pile load is less than 0.5 t. It is therefore concluded that the foundation can safely support the dead and live loads established for this assessment. Anticipated effects of settlements and differential settlements are negligible.

Stresses incurred during handling and transportation of the piles are judged to be in a safe range. However, calculated compressive as well as tensile stresses caused by driving are high relative to the specified strength of the piles. Driving of the piles was also unnecessarily hard. The effect of the resulting damage on the integrity of the piles was mitigated by cutting off a long section from the top of the piles (where the cracking occurs). It cannot be ruled out that some of the installed piles may have residual damage from the hard driving; however, it is unlikely that this damage could affect their load capacity.

4.5 PROGRESSIVE COLLAPSE ANALYSIS

4.5.1 <u>Methodology</u>

Compliance with the progressive collapse criteria of section 3.5.3.2 is determined considering effects of the failure of:

- A floor panel or an individual floor plank in section 4.5.2
- An individual major steel beam in section 4.5.3
- An individual column in section 4.5.4

In each instance, the failure of one single member at one point is considered.

The failure of a shear wall panel would not cause progressive collapse. Since each shear wall panel is fastened to the adjacent columns, there would be no critical loss of support for higher shear wall panels if a lower story shear wall panel fails, and the stipulated horizontal load can be resisted even if one of the shear walls is lost.

Failures of individual concrete beams are not considered likely to induce progressive collapse. Concrete beams in the core and in the exterior walls generally would be supported by masonry walls below. Concrete beams on column lines 5 and 6 will be treated in the discussion in section 4.5.3, as will steel beams along the exterior walls and on column lines 4A and 6B.

4.5.2 Failure of Floor Panels and Planks

4.5.2.1 Introduction

The floor system, except for the eighth floor and roof, typically consists of precast prestressed planks covered by cast-in-place, wire-mesh-reinforced concrete topping (section 2.3.5). In this analysis a "floor panel" is defined as that portion of the floor affected by the failure of a supporting beam. The largest floor panels are outlined in figure 4.5.1. These panels are considered the most critical because they are the most vulnerable to the loss of a supporting girder and their failure would subject the floors and beams below to the greatest dynamic loads. The large panels outlined in figure 4.5.1 are 13.2 m long and 10.8 or 11.4 m wide.

The floor system, shown in figure 2.3.24(a) consists of 220-mm thick prestressed concrete planks weighing 400 kg/m^2, covered by an 80-mm thick concrete topping slab which is assumed to weigh 192 kg/m^2. The topping slab is reinforced by one layer of 110 × 110-mm welded wire mesh made of 5-mm diameter grade B-I (tensile strength of 5000 kg/cm^2) cold drawn steel. The Soviet vertical load calculations also allow for a suspended ceiling weighing 48 kg/m^2. The eighth floor and roof panels are heavier (fig. 2.3.24(b)). The eighth floor has a 200-mm thick cast-in-place concrete slab over the precast planks which is assumed to weigh 500 kg/m^2 and a tile topping covering the concrete slab which weighs 144 kg/m^2. The concrete slab is reinforced by one layer of 18-mm diameter reinforcing bars spaced 200 mm on center near the bottom, and 12-mm diameter bars spaced 200 mm on center acting as negative reinforcement near the top. The negative reinforcement is centered over the beams and extends a distance of 1.3 m to either side from the centerline of the beams. The bottom reinforcement is not continuous over the supporting beam. There also are heavily loaded areas on the second and seventh floor. These areas have cast in place concrete beams between the prestressed planks and the same 80-mm thick topping as the regular floors. The eighth story roof has a 220-mm thick cast-in-place concrete slab and a 130-mm thick concrete topping.

The regular floors are supported by welded steel plate beams (figs. 2.3.10, 2.3.11 and 2.3.17) where the span is long, and in some instances by concrete beams where the span is short. In this analysis, the steel beams are of interest. Several sizes of beams are used in different parts of the building. Under the regular floors the long span beams (which are the most critical) are "Type B-8". These are 800 mm high with a 10-mm thick web and 400 mm wide and 25-mm thick flanges. No web stiffeners are used. The specified yield strength of the steel is 3300 kg/cm^2. The beam itself weighs 204 kg/m. The beam plus the sprayed-on fireproofing are estimated to weigh 240 kg/m. The heavily loaded floor areas on the second and seventh floors are supported by Type B-7 beams, which have the same overall size as Type B-8, except that the web is 16 mm thick and the flanges are 32 mm thick. These beams weigh 278 kg/m. The entire eighth floor is supported by larger beams, which have 500-mm wide flanges. The eighth floor roof system is similar to that of the eighth floor, except for the reinforcement details; the roof is supported by concrete encased steel beams.

Several failure modes and their potential consequences are considered. These include: the failure of an individual floor plank; the effects of the failure of a supporting beam; and the effects of the impact of falling floor planks on the floor system below.

4.5.2.2 Failure of a Floor Plank

A floor plank failure from overload or extraordinary weakness is unlikely to result in a plank falling. Planks restrained at the ends by topping and poured concrete extending into the holes (fig. 2.3.2.3). Even a shear failure also is unlikely to lead to a floor plank falling. Most of the load would be transferred to the topping and the broken plank would hang by its flexural reinforcement.

However, floor planks would fall if a beam failed and fell. The topping is not well bonded to the planks (section 5.3.1). There is no continuity of reinforcing to adjacent floor planks, and as the beam falls out below, the planks it supports on either side would fold down, eventually slip off the top flange, rotate about their support on the adjacent beams, and fall to the floor. This situation is analyzed in this section. Would a falling floor plank on the typical floor cause failure of the floor plank that receives it below?

Attention is given to the ability of the topping to remain suspended between adjacent beams. If the topping can remain suspended it would reduce hazards to persons on the falling floor and reduce the loads induced on the floor below. In addition, attention is given to the ability of the receiving floor to resist the impact of a falling floor plank.

4.5.2.3 Ability of the Topping to Remain Suspended

The topping and its reinforcement were described in sections 2.3.5 and 4.5.2.1. The spans for suspension, if a beam support were lost, are 10.8 m and 11.4 m (fig. 4.5.1).

The loading to be supported (sections 3.5.3.1 and 3.2.3) is 70 kg/m^2 for partial live load, 96 kg/m^2 for partitions, and 192 kg/m^2 for the topping itself. Thus the total loading is 358 kg/m^2.

Properties of the wire mesh are given in table 5.3.2 based on samples acquired at the site. The resulting relationship between in-plane force per meter of mesh and axial strain is shown in figure 4.5.2. The peak force is 11 t/m at the onset of plastic instability under constant load at 2 percent strain.

During the February 1987 field investigation two cores were taken from each floor slab in stories 1 through 6. These cores indicated lack of good bond between the concrete topping and the precast planks. It is therefore assumed that the dead weight of the planks is sufficient to cause separation between the planks and the topping once a supporting beam has fallen.

When a beam in the center of panel A or B in figure 4.5.1 fails, it leaves an unsupported floor panel with a width of 10.8 m for interior panels (A), and 11.4 m for the exterior panels (B). The thickness of the concrete topping is small with respect to these spans and as the beam drops, the concrete topping will deform into a catenary shape supported by the wire mesh. The concrete planks supported by the falling beam will peel away from the concrete topping and initially drop together with the beam. Eventually as their tilt increases they will slip off the falling beam and drop sequentially (they will not hit the floor below simultaneously). Whether the topping will hang in a catenary shape or fail depends on the strength and ductility of the wire mesh and on the reaction forces that can be developed along the edges of the failed panel.

The concrete topping has to support 358 kg/m^2 by catenary action. Actual stresses and deflections at catenary equilibrium must be determined on the basis of the load-deformation characteristics of the wire mesh shown in figure 4.5.2. Loads are given in tons per 1-m width of mesh and deformations are in percent of length.

The following general catenary equations were used to determine the geometry of the suspended topping slab and the tensile forces acting on the wire mesh:

$$S = (L/2)[1+16(t)^2]^{1/2} + [L/(8t)]\ell n\{4t+[1+16(t)^2]^{1/2}\} \quad (4.5.1)$$

$$T_{max} = (wL/8t)\{[1+16(t)^2]^{1/2}\} \quad (4.5.2)$$

$$T_{av} = (wL/8t)\{1+[16(t)^2]/3\}/\{1+[8(t)^2/3]\} \quad (4.5.3)$$

$$H = wL/8t \quad (4.5.4)$$

where:
- S = the length of the catenary curve
- L = the length of the span
- t = the sag to span ratio
- T_{max} = the maximum tensile force (at the reaction)
- T_{av} = the average tensile force
- H = the horizontal component of the tensile force (also the minimum tensile force in center of span)
- w = the load per unit length of span
- s = the sag = tL

A solution is found by balancing the length increase associated with the catenary curve against the elongation resulting from the forces acting on the supporting wire strands.

The following solutions were obtained:

10.8-m span: $t = 0.049$ ($s = 0.520$ m)
$T_{max} = 10{,}300$ kg/m
elongation = 0.64 percent
maximum stress = 5,760 kg/cm^2

In this instance there is a considerable safety margin. Even though the stresses are high, they would decrease with any further elongation. Note that the ductility demand is only 0.6 percent, while the elongation of the wires at plastic instability is approximately 2 percent. Thus, the system is in stable equilibrium.

11.4-m span: $t = 0.051$ ($s = 0.58$ m)
$T_{max} = 10{,}400$ kg/m
elongation = 0.69 percent
maximum stress = 5,850 kg/cm^2

As in the previous case, the topping slab can support its own weight, the weight of partitions, and the assumed live load by catenary action with a considerable margin of safety.

The results of the catenary analysis show that the topping will remain suspended in the event of a beam failure in a typical floor if the topping is adequately anchored. If the mesh was rolled out normal to the beam axes the mesh would be continuous between the east and west exterior walls. If mesh was rolled out parallel to the beam axes, it would be anchored by lapping of the sheets of wire mesh in the interior of the floor system. The development length required to develop the full tensile strength of the bars in the welded wire mesh can be determined in accordance with ACI 318-83 by the following equation:

$$l_d = 0.27\,(A_w\ f_y)/[s_w\ (f'_c)^{1/2}] \tag{4.5.5}$$

where: l_d = the development length
A_w = the cross sectional area of the mesh parallel to l_d
s_w = the center to center spacing of the wire mesh normal to l_d
f_y = the yield strength of the steel
f'_c = the 28 day cylinder strength

Equation 4.5.5 is not dimensionally consistent with the result, which has the dimension of length. Customary U.S. units must therefore be used to calculate the results. For the welded wire mesh used, the development length based on a cylinder strength of 149 kg/cm^2 (corresponding to Mark 200 concrete) and an average tensile strength of 6060 kg/cm^2 is 81 mm. Based on construction photographs, the sheets of wire mesh appear to have been generally placed with an adequate overlap to develop the tensile strength of the wires if the concrete maintains sufficient integrity to transfer the tensile forces between the lapped meshes.

No positive anchorage presently exists along the east and west exterior walls from column lines 1A to 4A and 6B to 9A. To ensure catenary action of the concrete topping in B-type panels (fig. 4.5.1), ties or anchors capable of resisting a horizontal reaction force on the order of 12 t/m and transmitting it to the welded wire mesh would have to be installed as a retrofit.

4.5.2.4 Fall of a Floor Plank

In the event of a beam failure, the planks it supports on each side will come free at the falling beam, hinge about the supports on the adjacent beams, and rotate to impact the floor planks below. This analysis considers whether the receiving plank on the floor below can resist the falling plank. The analysis considers the prestressed, precast, hollow core planks used for the longer spans (fig. 2.3.23).

A dynamic, inelastic modeling technique [4.8] was used to study the response of the coupled falling and receiving planks with the numerical integration parameter beta = 0.25 and no damping. The falling plank was considered to have a mass of 0.38 t/m^2 (based on information in plank catalogs), and the free end was assumed to fall as a rigid body prior to impact. The receiving plank was assigned a mass of 0.572 t/m^2 corresponding to the plank plus topping. Five mass points were used for each plank: at ends, quarter and mid point for the falling plank; at ends, 0.188 m from ends and mid point for the receiving plank.

Impact was modeled as inelastic with initial velocity of the contacting mass points on the two planks taken equal to the value required for conservation of momentum. Only the velocity component normal to the receiving plank was considered to drive the out-of-plane displacements of the receiving plank.

Typical 1.8-m wide planks were considered. The stiffness parameter EI = 2430 $t \cdot m^2$ was calculated using the moment of inertia of the uncracked section and an elastic modulus of concrete based on the equation given in section 8.5 of ACI 318-83 for f'_c corresponding to the specified strength of Mark 200 concrete. An unfactored ultimate moment of 13 t·m, corresponding to the values of section 4.2.2, was used to define the resistances of both planks. An elastic-plastic resistance function was considered.

Figures 4.5.3 and 4.5.4 show the displacement-time histories, after impact, for the midspan of the falling plank and the impact point of the receiving plank. The falling plank did not rebound, but continued to press against the receiving plank to the time of its maximum displacement (0.12 sec after impact). The receiving plank's ductility demand was 9 times the yield deflection. The ductility demand for the falling plank was 27 times the yield deflection.

The receiving plank is likely to resist for a ductility demand less than 10 [4.9]. This assessment is tempered by Soviet test results reporting ultimate deflections of about 30 mm in plank tests. However, this ultimate displacement is likely to correspond to yielding of the reinforcement, rather than crushing of the concrete and marked loss of resistance. Although U.S. test results [4.10] suggest that the ductility demand of the receiving plank can be realized, it would be desirable to have direct evidence of the ductility available from Soviet planks. The falling plank is likely to break up at its much higher ductility demand. This would reduce the loading on and the ductility demand in the receiving plank.

Thus, progressive collapse is not indicated to occur as a result of the fall of a typical floor plank on to another typical floor plank. This finding is conservative in that the actual concrete strength is expected to exceed that specified (section 5.3), and no structural function has been considered for the topping on the receiving plank. In the event of loss of resistance of the receiving plank, a considerable reserve capacity remains in its topping if the topping is anchored. This finding also is dependent on the topping remaining in place above the falling plank. If the topping fell too, the impact energy would be greatly increased and the receiving plank might fail.

4.5.2.5 Ability of the Eighth Floor and Roof Slabs to Remain Suspended

As shown in section 2.3.5, the eighth floor system is very different from the typical floor. As shown in figure 2.3.24(b), a 200-mm slab is poured on top of the 220-mm plank. The transverse, lower steel in the slab does not continue over the supporting beams. Should a beam fail, the plank and slab would break at the beam line and hinge about adjacent beams. The collapsed panel will follow the falling beam for some distance and will eventually partially drop off the beam. The impact of the falling eighth floor will hit the seventh floor approximately 1.5 m from the centerline of the beam below the falling beam.

Considering that the preceding dynamic analysis shows the receiving typical floor plank just able to sustain a blow from a 0.38 t/m^2 typical plank, it could not sustain the impact of an element of the eighth floor system that weighs 1.094 t/m^2 (table 3.2.1). The ductility demand would be increased far more than the 3:1 ratio of the masses; little inelastic energy absorption would occur in the heavy, falling slab. The resulting ductility demand for the receiving plank would be in excess of 100.

Thus, progressive collapse will occur in the event of the failure of an eighth floor beam unless the beam is restrained from releasing its supported planks and slab to fall on the floor below. A restraining system is recommended in section 6.3 which will achieve this purpose.

The roof panels consist of a 220-mm thick cast-in-place concrete slab, overlain by a 130-mm thick lightweight concrete slab and roofing material. The system weighs 1310 kg/m^2. The supporting beams are encased in the cast-in-place slab and tied to it by adequately anchored reinforcing bars. If a beam failed, it would remain embedded in the cast in place slab and the anchored reinforcing bars could transmit tensile forces across the failed beam. Thus a collapse of this slab in the case of the failure of a major beam is unlikely. Should a collapse occur, much of the energy generated by the falling debris would be dissipated yielding of the reinforcement. The heavy beams and floor system of the eighth floor would prevent a downward propagation of such collapse.

4.5.3 Failure of a Beam

4.5.3.1 Typical Long Span Beam

The typical long span steel beam is a B-8 on floors 2 through 6 such as those on column line D. The scenario considered for analysis is an abrupt failure of a beam, such as would occur from brittle fracture originating at a flawed weld. The falling beam would separate from its supported planks, as discussed in section 4.5.2, and fall freely to the floor below. Two cases are analyzed: failure at one end with the falling beam in one piece, and failure at the middle with the falling beam in two equal pieces. Case 1 would threaten shear failure of the receiving beam since impact occurs just 400 mm from the end. Case 2 would generate essentially flexural response in the receiving beam.

Consideration of the times to fall as rigid bodies show that the floor planks impact 0.79 sec after breaking loose (they must fall 3.9 m) while the time for the beam to fall is 0.65 sec in case 1 and 0.67 sec in case 2 (they fall only 3.1 m before contact and through smaller arcs). Therefore, response analysis for a falling beam considered the impact of a beam alone on the receiving beam. The results for a falling plank striking a receiving plank on unyielding supports, discussed in section 4.5.2.4, and the falling beam striking the receiving beam alone, discussed here, are subsequently analyzed for possible interactive effects.

For the representative B-8 beam of floors 3 to 6 the falling beam was considered to have the following properties: mass 240 kg/m considering the weight of the steel cross section plus a 10 percent allowance for sprayed-on fireproofing; mass points taken at end, quarter and center points; yield moment 302 t·m; yield shear 148 t.

The receiving beam was modeled as another B-8 section with the same yield moment and shear. It is important to note that the connection to the column, figure 2.3.14, is strong enough to develop the strength of the beam web in shear. The vibrating mass was taken at 5.108 t/m to correspond to a typical 5.4-m beam spacing and a partial live load of 70 kg/m2. For case 1, receiving beam mass points were taken at end, quarter and center points plus the impact point 400 mm from one end. For case 2, receiving beam mass points were taken at the impact points 800 mm each side of the center line, ends and points halfway between ends and impact points.

The impact was modeled as inelastic with momentum conserved at the points of impact and the mass points of the falling and receiving beam constrained to stay together in subsequent vibration. This does not model rebound and subsequent impact, which would have occurred in case 2, but does define the distribution of energy absorption between the falling and receiving beams prior to rebound, in which interval most occurred, and gives an indication of the distribution of subsequent energy absorption.

Case 1 is illustrated in figure 4.5.5, which shows the vertical displacement-time history at the point of impact of the receiving beam. The receiving beam yielded in shear between the support and the point of impact. The inelastic deformation of the receiving beam was 0.2 percent — a small amount about equal to the elastic strain at yield.

Case 2 is illustrated by figures 4.5.6 and 4.5.7, which show displacement-time histories in a transverse direction for the midpoint of one of the segments of the falling beam, and one of the impact points on the receiving beam, respectively. The receiving beam did not yield. In both cases the energy of the impact was absorbed in inelastic deformations and the elastic, vibrating energy of the falling beam, and the elastic deflection of the receiving beam.

Maximum deformations occurred at 0.135 sec after impact in case 1 and at 0.12 sec after impact in case 2. Thus, the effect of impact of the falling planks is not additive to that of the falling beam. The accelerations resulting from the dynamic reactions of the receiving planks bearing on the receiving beam would be opposite to, and approximately equal to, the accelerations of the receiving beam rebounding from the earlier impact of the falling beam. Thus failure of a typical long span beam does not threaten progressive collapse.

4.5.3.2 Failure of Other Beams

Steel beams on column lines 4A and 6B support a long span steel beam as well as floor planks on one side. Were a failure to occur near the column connection on the core end, or near midspan, the similar properties of the beam section to those considered in section 4.5.3.1 indicate that the impact would be resisted by the receiving beam. A failure near the exterior wall deserves further consideration. The connection to the supporting steel beam at the exterior wall connects only one side of the beam web and will not develop the strength of the beam web in shear. Fortunately, that beam, for example, A1/4-A1/5, is weaker in flexure than the strength of the connection of the beam, for example, C1/4A-A1/4A. Thus, the flexural response of the supporting beam should prevent a brittle failure of the connection of the receiving beam and progressive collapse would not occur.

The concrete beams along column lines 5 and 6 were not analyzed for progressive collapse. The mass ratios for falling and receiving beams would be similarly favorable to avoidance of progressive collapse. Beams in the core area generally have masonry walls below them that would prevent any progressive collapse in the event of beam failure.

4.5.3.3 Summary

Failure of an individual floor plank does not threaten progressive collapse as is described in section 4.5.2.2.

Failure of a major beam on the eighth floor would threaten progressive collapse by virtue of the heavy floor system that would penetrate the floor below, section 4.4.2.5. This can be prevented by the remedial measure of installing a restraining system as described in section 6.3.

Failure of a major beam on floors 2 through 7 presents a more equivocal situation. The falling beam itself would not seriously damage the beam below, and progressive collapse would not occur as a result of a beam failure if: (1) the topping were anchored at the east and west exterior walls so as not to add to the energy of the fall; (2) either no laps normal to the beam axes occurred in the reinforcement of the topping or bond were adequate to hold together lapped reinforcement; and (3) the floor planks have a ductility factor in excess of 9. Data are not available on the ductility of actual planks and the presence or performance of laps in the reinforcement of the topping, so the resistance to progressive collapse is considered marginal. A restraining system preventing falling of a failed beam and its supported planks prevents both progressive collapse and hazards from falling debris. It is recommended and described in section 6.3.

Failure of a major beam supporting the eighth story roof does not threaten progressive collapse.

4.5.4 Columns

4.5.4.1 Introduction

The failure of a column propagates upward along the column line. Its propagation can be avoided only if an alternate path is provided by which the supported loads can be transmitted to the foundation.

An alternate load path can be provided by transferring the load supported by the failed column to adjacent columns or to structural or non-structural walls which can support vertical load. Load transfer to adjacent columns may be accomplished by existing elements such as shear walls and non-structural partitions, or by new structural members which would be added as a retrofit.

All concrete column segments are connected by four continuous reinforcing bars (figs. 2.3.6 and 2.3.7). These bars can support several story levels in tension. Thus load transfer does not necessarily have to occur on the level immediately above the failed column, or in each story level.

In accordance with the criteria in section 3.5, the failure of a single column is considered. However, a column could fail together with adjacent wall elements. This possibility also is discussed in the analysis.

4.5.4.2 Failure of Exterior Perimeter Column

All the exterior perimeter columns and their connecting concrete spandrel beams are embedded in, and tied to, the exterior masonry walls. The bond between the perimeter wall and the structural frame is further enhanced by the expansion of the masonry wall and shrinkage of the concrete elements, which cause the frame to exert a downward pressure on the wall (section 4.7). Adjacent to each column, and between any two columns, there are masonry piers which at their smallest cross section are 810 mm deep and 790 mm wide (fig. 2.4.2). The spandrel beams connecting the exterior perimeter columns (fig. 2.3.12) are embedded in masonry beams which span between the masonry piers (fig. 2.4.7). The spandrel beams are anchored to the masonry (fig. 2.4.3) and welded to the columns (fig. 2.3.13).

The most critical columns are Columns C through H in lines 1A and 9A, which all support major beams. Of these Column E/1A is the most heavily loaded (Appendix 4.1). The following data apply to this column: the maximum load is 464 t; the maximum incremental (story) load is 85 t, occurring at the eighth floor; and, the average incremental (story) load is 51 t.

If one of these columns were to fail in the first through sixth story, the column above it would be restrained in several ways: (1) It is connected to the adjoining masonry piers by six 10-mm ties which have a combined ultimate shear strength of 11 t; (2) it is connected to spandrel beams on both sides with welds which can resist ultimate shear loads on the order of 47 to 78 t each, depending whether the space between the spandrel beam and the column is effectively grouted (see analysis in 4.5.4.3 (2)). The spandrel beams in turn are embedded in the masonry beam which is connected to both the masonry piers next to the column and the intermediate masonry piers. The ultimate shear strength of the spandrel beam at the column connection is calculated to be 40 t, and that of the masonry beam also 40 t (assuming that the shear strength of the masonry is 7 kg/cm^2 — see discussion in 4.5.4.3 (2)).

To determine the load capacity of the pier, data are taken from table 5.10.10, which indicates an average masonry prism compressive strength of 254 kg/cm^2 and a minimum strength of 190 kg/cm^2. The 10 percent exclusion limit strength calculated from the data in table 5.10.10 is 180 kg/cm^2. The load is considered to be transferred at an eccentricity of 1/3 the thickness of the pier (the actual load eccentricity is difficult to determine because of the complex nature of the load transfer mechanism, which includes the restraining effects of floors at each level). On this basis it is estimated that the vertical load capacity of these piers is 540 t. Thus in the case of the failure of a single column, the load could be transferred to the adjacent pier. As a redundant mechanism, the spandrel and masonry beams could transfer load to the intermediate pier, which is discussed subsequently.

Thus, it can be seen from the preceding discussion that if a column in the first through sixth story were to fail, the load would be transmitted to the pier at the column one level above. The load transfer could occur at each of stories 1 through 7. Therefore it is concluded that a progressive collapse will not occur as a result of the failure of a single perimeter column in the first through the sixth stories.

In the case of a column failure in the seventh story, this load transfer mechanism would not work, because masonry piers do not exist in the eighth story. Neither is there a path by which to transfer an eighth story column load to a seventh floor pier. Thus, unless remedial measures are implemented to provide a load transfer mechanism, failure of a perimeter column in the seventh story would propagate upward and affect the eight story column and the roof beam it supports.

Since remedial actions are required to ensure support for the eighth floor beams (section 4.5.2.5), no additional remedial action is required to prevent the failure of an eighth story perimeter column if the supporting seventh story column were to fail. Failure of a seventh story column failure would not precipitate a failure of the affected eighth floor panel. The roof panel affected by the loss of support of an eighth story column also is expected to remain suspended by slab action (4.5.2.5).

4.5.4.3 Load Transfer Between Columns in the Core

(1) Load Transfer Mechanisms

Figures 4.5.8, 4.5.9, and 4.5.10 show three adjacent columns lines with walls between them, and illustrate the load transfer mechanisms for interior core columns. The walls are bounded on all four sides by columns and beams and can function as structural members. When the column failure illustrated in figure 4.5.8 occurs, there is a wall panel only on one side of the column. The wall panels above the failed column act as a deep cantilever beam and transfer the load to the adjacent column line. In the case of the failure illustrated in figure 4.5.9, the wall panels on both sides of the column line above the failed column act like a deep beam and transfer the load to the adjacent columns. The shorter wall panel to the right of the column line of the failed column will pick up a larger portion of the column load because of its greater stiffness. Figure 4.5.10 illustrates the important role of the beam-to-column connection in the load transfer mechanism. Should this connection fail in shear, the column line above the failed column would move downward, because, except for the case of the structural shear walls, there is no structural connection between walls and the adjacent columns. If a column near the top of the building fails (for instance in the seventh story), the cantilever or other beam formed by the wall panels and surrounding beams would not be as deep, and the moment resistance provided by the beams at the boundary of the wall panel may become critical. For example, in the case of the one sided cantilever action illustrated in figure 4.5.8, the top beam would be in tension and the bottom beam in compression.

Figure 4.5.11 illustrates another load transfer mechanism, whereby the column load is transmitted to the adjacent wall panel which supports it as a bearing wall. In this case the beam also plays a critical role in the load transfer.

(2) Columns Connected by Shear walls

In the case of a column between shear walls, there are no beams. However, load and moment transfer between the column and the wall can be accomplished by the shear wall-to-column connections (figs. 2.3.21). When a shear wall panel transmits load between two adjacent columns, it is subjected to a shear load equal to the incremental column load (the additional column load generated at its story level) and to a moment equal to the incremental column load times the distance from the center of the transmitting column to the face of the receiving column. Shear wall panels can also support gravity loads transmitted from columns in bearing; the loads can be transmitted via the wall-to-column connection. The bearing mechanism is not a preferred because the shear wall panels adjacent to the failed column could also fail. However, it is important to know that the transfer of column loads to the walls, which can support these loads in bearing, provides a redundant alternate load path.

Thus the shear, moment and bearing capacity of the shear walls must be checked.

The shear walls (fig. 2.3.20) are 180 mm thick. They are connected to the columns in each story level by three welded connections. The distance between these connections varies with story height. For the typical story height of 4.2 m these connections are spaced 1.2 m apart. In addition, there is a 32-mm diameter reinforcing bar at each floor level, which is welded to a bracket embedded in the column.

The concrete strength of the as-built walls was determined to have a lower limit of 350 kg/cm^2 (table 5.3.10), the reinforcement has a yield strength of 4000 kg/cm^2, and the welds have a tensile strength of 4600 kg/cm^2.

The welded connections include two 12-mm welds which are 200 mm long. The ultimate shear strength the connections (V_{uw}) is 281 t per story.

The shear strength of the concrete in the wall is 9.93 kg/cm^2 (section 11.3 of ACI 318-83). The ultimate shear strength (V_{uc}) of a 4.2-m high wall panel is (9.93 kg/cm^2)(18 cm)(420 cm) = 75 t; for the 4.8-m panel, the ultimate shear strength is 85.8 t.

For the 184 kg/cm^2 concrete strength specified in the plans (Mark 300 concrete), the shear strength would be 54 t for the 4.2-m height and 62 t for the 4.8-m height.

The moment resistance of a panel (4.2-m story height) is obtained from the contribution of the connection plus the contribution of the horizontal 32-mm bars:

$$M \text{ (connections)} = 93.7(2.41) = 225.7 \text{ t·m}$$
$$M \text{ (bars)} = 32(4.1) = 131.8 \text{ t·m}$$

The total resisting moment for the 4.2-m high story is 357 t·m.

The bearing resistance of the shear wall is calculated by the empirical equation (eq. 14.1) given in ACI 318-83, using a strength reduction factor of unity:

$$P_u = 0.55 f'_c A_g [1 - (kH/32t)^2] \qquad (4.5.6)$$

where:

f'_c = compressive strength of concrete
P_u = ultimate load
A_g = gross cross section area
k = column end fixity coefficient
H = height of wall between lateral supports
t = thickness of wall

Conservatively assuming that k = 1, the following values of axial load capacity (P_u) were computed for different values of concrete strength and story height:

	Concrete Strength	
	Specified ($184\ kg/cm^2$)	In-place ($350\ kg/cm^2$)
4.8-m Story	35 t/m	58 t/m
4.2-m Story	56 t/m	94 t/m

Column 1 of table 4.5.1 lists the columns which are connected to shear walls (refer to figure 2.3.10), together with the magnitude of the load transferred to adjacent columns in the case of a failure. The load transfer mechanisms assumed in the table are either cantilever beam action by the shear wall if the shear wall is only on one side of the column, or deep beam action if the shear wall is on both sides of the column. In either case, the shear capacity is taken as 75 t.

Columns 2 and 3 of table 4.5.1 list the total column load and the incremental loads at each story level. Ordinarily the maximum load increment is contributed at the first floor level, but in some instances it occurs at the highest floor (Appendix 4.1). Column 4 lists the column function; most of these columns support beams which in turn support major floor areas. The beams running north-south are designated as major beams, since they support large floor areas. All others are designated as minor beams. In the case of column C1/4A and G2/6B, major beams are indirectly supported by framing into other beams. Columns 5 and 6 of table 4.5.1 list the columns to which loads are transferred, together with the maximum load that these columns would have to support (if failure occurred at the first story level, so that the column would pick up the cumulative loads from all the stories). The portion of the load transferred to each adjacent column was determined in accordance with the stiffnesses of the connecting shear walls. The preferred load transfer mechanism is through shear at each story level, because it can be relied on even if a column fails together with the adjacent shear wall panels. An alternate, redundant mechanism is load transfer to the shear wall itself in bearing. The required column capacity for the worst case of load transfer is listed in column 7. Column 8 lists the ultimate column capacity, calculated for a capacity reduction factor of 1 and for a concrete strength of 510 kg/cm^2, which is the concrete strength is estimated to be equaled or exceeded in 90 percent of all cases, based on the strength of cores taken in the field (section 5.3.3.2).

The following conclusions can be drawn from table 4.5.1:

1. The shear walls have enough load capacity to transmit the incremental column loads. The largest incremental column load (82.5 t) occurs at column D/4A where it is shared by two shear walls.

2. Moment capacity of the shear wall controls the load transfer capacity of the shear walls only in the case where failure occurs in the seventh story. In this case, the moment capacity would limit the shear that can be transmitted by a 5.4-m wide shear wall panel to 67 t, which is also adequate to transmit the load in all instances.

3. Two columns, D1/4A and F1/6B, appear to be slightly overloaded if the 10 percent exclusion limit concrete strength is used to calculate their capacity. However, if the average strength of concrete is used, their capacity is 837 t, and the bearing capacity of the two adjacent shear walls, which would have to fail simultaneously, is 348 t. Thus a failure of these columns would be unlikely even if the maximum load is transferred to them.

It is therefore concluded that the columns connected to the structural shear walls cannot collapse progressively.

(3) Columns Adjacent to Non-Structural Partitions

Brick Masonry Properties:

Non-structural partitions can resist compressive and shear loads, even though they were not built for this purpose. The partitions in the core are mostly of brick masonry and some are of reinforced concrete (fig. 2.4.11). Where partitions between columns can be used to transfer load between columns or to transmit column loads to the foundation, they are bounded on all four sides by structural members. The vertical sides are bounded by columns and the horizontal sides (top and bottom) are bounded by beams which span between columns at each story level. Only partitions at the basement level rest directly on the structural foundation.

The conditions of partition walls and the strength of the masonry used in their construction are discussed in sections 5.8 and 5.10.7. Test data are given in tables 5.10.10 to 5.10.12. The average prism compressive strength from seven tests was 254 kg/cm^2 and the strength ranged from 191 kg/cm^2 to 353 kg/cm^2. On the basis of the sample tested the 10 percent exclusion limit strength is approximately 184 kg/cm^2. Shear strengths of two specimens taken from the site, based on diagonal compression tests were 9.16 and 8.03 kg/cm^2, respectively. The ratio of these strengths can be compared with U.S. data which are reported in reference 4.11, which indicate that the shear strength is roughly proportional to the square root of the compressive strength (in psi) and that the shear strength for 90 percent of the test results falls between the limits of 2.5 and 4.5 times the square root of the compressive strength (in psi). The shear strength corresponding to 2.5 times the square root of the 10 percent exclusion limit compressive strength is 9 kg/cm^2. In this analysis the shear strength is conservatively assumed to be 7 kg/cm^2, which is the maximum strength allowed in reference 4.11. This strength will be applied to the gross section without a reduction for stress distribution. The compressive strength of the brick masonry is conservatively assumed to be the 10 percent exclusion limit strength of 180 kg/cm^2. In accordance with recommendations in reference 4.12, the ultimate compressive strength of masonry should be assumed as 2.5 times the design strength permitted in reference 4.11, which in this case would be 90 kg/cm^2 times wall cross-section times appropriate slenderness and eccentricity reductions. This latter strength is considered a conservative lower limit for the masonry strength.

The condition of each partition was cataloged and is also taken into consideration in this analysis (table 5.8.1).

Load Capacity of Masonry Walls

The only masonry walls considered effective in transmitting column forces are 2-wythe masonry walls which are 250 mm thick. The following capacities are calculated:

V_m = 375(25)(7)(0.001) = 65 t for the 4.2-m story

V_m = 4.35/3.75(65) = 75 t for the 4.8-m story

P_m = (100)(25)(90)(0.78)(0.001) = 175.7 t/m for the 4.2-m story

P_m = (100)(25)(90)(0.72)(0.001) = 162 t/m for the 4.8-m story

Where V_m is the shear capacity and P_m is the bearing capacity. Slenderness reductions for bearing capacity calculations are in accordance with table 6 in Reference 4.11.

Capacities of masonry walls are multiplied by 1.1 for "good" conditions and by 0.9 for "poor" conditions (section 5.8).

Strength Concrete Beam Connections

There is a positive connection between a concrete beam and the column via three welds (fig. 2.3.13): two 10-mm welds, 120 mm in length (parallel to the beam), are specified between the beam and column corbel; one 10-mm weld, 160 mm in length (perpendicular to the beam), is specified for joining the top plate to the column (fig. 4.2.1). Since there is a 20-mm space between the beam and the column, the top weld will only be effective if this space is filled with grout as indicated in the plans. Field observation reports indicate that this is not always the case. The specified tensile strength of the welds is 4600 kg/cm^2.

Thus, using eq. (4.2.3), the ultimate shear capacity (V_s) of the beam-to-column welds is:

V_s = (0.6)(46)(0.707)(10)(240+160)(0.001) = 78 t

and,

V_s = 47 t if the 20-mm gap is not grouted

The tensile capacity of the beam connection depends on the same two welds at the base of the beam. At the top of the beam, it is controlled either by the top weld, or by the minimum cross section of the top plate. The minimum cross section of the plate is 800 mm^2 for the larger type connections or 640 mm^2 for the smaller type connections. The tensile strength of the steel is 3300 kg/cm^2. Thus:

T_{sl} = (0.6)(46)(0.707)(10)(240)(0.001) + (800)(33)(0.001) = 73 t

T_{ss} = T_{sl} - (160)(33)(0.001) = 68 t

Where T_{sl} and T_{ss} are the tensile capacities of the large and small connections, respectively. These capacities are only considered critical if no beam frames into the column from the direction opposite to that of the applied tensile force.

Column Lines 4A and 6B

Some of the columns in lines 4A and 6B are connected by structural shear walls and are not discussed in this section. The other eight columns support major beams. The most critical columns are C1/6B and G2/4A. In the second through the sixth stories, the partitions on the sides of these columns are composed of 2-wythe walls with hollow cores (refer to fig. 2.4.11 for partition plan). Of the two wythes, only one lies directly below the beams along 4A and 6B. In the first story, the space between columns C1/6B and D/6B is partially open. The columns bounded by the hollow partition walls are listed in table 4.5.2. The average incremental loads vary from 27 t to 60 t and the maximum incremental loads vary from 32 to 83 t. The 1-wythe masonry wall below the beams cannot be relied upon to transmit the incremental column loads in shear, or to resist large compressive loads. A retrofit is therefore required for these walls. Provided the 20-mm gap between beam and column is properly grouted, the shear capacity of the concrete beam-to-column connection is 78 t and is adequate. Between columns C1/6B and C2/6B, the second floor is supported by a steel box beam. The column connection of the box beam has the same shear capacity as the concrete beam connections, namely 78 t. There is no need to secure the first floor space between the columns, if the other floors can "hang" on adjacent columns. However, the shear capacity of the beam connection supporting the second floor is important, because its failure would cause the supported second floor partition to drop.

The capacity of the hollow partition walls between the columns along 4A and 6B can be increased by filling the cavities with concrete having a shear strength equal to that of the masonry. This would create a 400-mm thick wall (the wall which is not framed by the beam is not counted) with a shear capacity of 80 to 100 t. This would be adequate to secure all the columns (the weight of the wall would add about 4 to 5 t to the average story shear force). Mark 300 concrete (184 kg/cm^2) would be adequate. The two wythes of masonry should be tied prior to concrete placement to prevent damage, and care should be taken to completely fill the void under the beam.

The connection between the steel box beam and columns C1/6B and D/6B is similar to the concrete beam connections (fig. 2.3.13(b)) and can resist a shear force of 78 t. At column C1/6B, the addition of the concrete core will increase the total shear force to approximately 90 t. Thus there is a 12 t deficiency. The connection should be retrofitted to increase the shear capacity by 12 t. This can be accomplished by adding a 125-mm length of AWS E70XX 10-mm fillet weld (or 4600 kg/cm^2 10-mm Soviet weld). If there is no room for a weld connecting the box beam directly to the column, the force will have to be transmitted via a welded plate. For this case the total weld length required would be 250 mm (125 mm at the base plate, and 125 mm on the beam, with two connecting A441 steel plates).

Columns in the Interior of the Core

The columns located in interior of the core are listed in table 4.5.3, which gives the same type of information as the previous tables. Each column must be considered separately (refer to figure 2.4.11 for core plan).

Column C2/6:
Load is transferred to C2/5A by a 2-wythe masonry partition and a beam. The weight is 11 t for 4.2-m story and 13 t for 4.8-m story. The maximum story shear load is 40 t, and the average story shear load is 27 t. The shear capacity is 49 t for average and 56 t for maximum. Column secured.

Column C2/5A:
Load is transferred by 2-wythe masonry partitions and beams to columns C2/6 and D1/5A. Column secured.

Column D1/5A:
Load is transferred by 2-wythe masonry partitions and beams to columns C2/5A and D1/6. Column secured.

Column D1/6:
Load is transferred by 2-wythe masonry partition and beam to column D1/5A. Refer to column C2/6. Column secured.

Column F1/5B:
Load is transferred by a 2-wythe masonry wall with openings to G1/5B; in the other direction, there is a 2-wythe masonry wall and then a 200-mm reinforced concrete wall to F1/5. There are also beams in both directions. The average incremental load is 30 t and the maximum load is 35 t. Concrete weight can be discounted because of a cross wall which also contributes to shear resistance. The shear capacity of the concrete wall is estimated to be 60 t. The masonry wall with openings is not considered effective in transmitting shear. Thus all the load is transferred to column F1/5. The joint between the masonry and concrete walls may be a problem, causing a shear failure through the beams along the joint. A retrofit providing positive connections with 40-t shear capacity per story is recommended (section 6.3.3).

Columns F1/5, G1/5 and G1/5B:
See column F1/5B.

Columns E/6A, E/4B, F/4B and F/6A:
These columns support the floor in the elevator lobby. Along column lines E and F, they are connected by concrete beams and a 2-wythe masonry walls with openings for elevator doors. These walls are not considered very effective in shear because of the long span (9 m) and the openings. The elevator doors are 1.3 m wide and 2 m high. This leaves a 1.75-m high net section to resist shear, which at best would have a capacity of 30 t. An alternate load transfer mechanism is bearing. The bearing area available would be as follows: On the west side there are two 1.4-m piers and one 0.95-m pier next to the unaffected column (the pier next to the affected column is not counted). This would result in a bearing capacity of 656 t for the 4.2-m story and 608 t for the 4.8-m story. On the east side, where there are only

three elevator doors, the capacity would be 887 t for the 4.2-m story and 875 t for the 4.8-m story. These capacities are considered adequate for the support of the concrete beam which supports the elevator lobby. If a column in the first story were to fail, the beam connection might also fail. However, the collective capacity of the beam connections up the column line is sufficient to prevent a progressive collapse. Thus <u>columns E/6A, E/4B, F/4B and F/6A are secured</u> against progressive collapse.

4.5.4.4 Summary

(1) The exterior perimeter columns in stories 1 through 6 of the are secured against progressive collapse by load transfer to the adjacent masonry piers. Failure of a seventh story exterior column would cause the eighth story column above it to fail. The consequences of such a failure would be limited, because proposed retrofit measures would prevent the propagation of a failure of the supported eighth floor beam, and the loss of support of the supported roof beam is not expected to cause a failure which would propagate to the eighth floor.

(2) The core columns connected to the precast shear walls are secured against progressive collapse by the load transfer mechanism provided by the shear walls.

(3) Column C1, D, D1, and E in line 6B, and columns F, F1, G, and G2 in line 4A can be secured against progressive collapse by the masonry partition walls and the connecting beams provided the retrofit measures recommended in section 6.3.3 are implemented.

(4) Columns C2 and D1 in lines 6 and 5A are secured against progressive collapse by the connecting masonry partitions and connecting beams.

(5) Columns F1 and G1 in lines 5 and 5B can be secured against progressive collapse by the connecting masonry and connecting beams provided the retrofit measures recommended in section 6.3.3 are implemented.

(6) Columns 4B and 6A in lines E and F are secured against progressive collapse by the connecting masonry walls in lines E and F which have adequate bearing capacity to transmit the column loads to the foundation.

4.5.5 <u>Conclusions</u>

A progressive collapse analysis has been performed considering the failure of one of the following:

- o A floor panel or an individual floor plank
- o An individual long-span steel beam
- o An individual column

It has been shown that the Office Building as currently designed does not in all instances have alternate load paths to prevent progressive collapse. Thus remedial measures are required; they are discussed in Chapter 6.

4.6 ASSESSMENT OF BRICK MASONRY EXTERIOR WALLS

The exterior brick masonry walls of the Office Building are designed to be self-supporting. These walls are intended to resist their self-weight and to transfer lateral wind loads to the structural frame. Typically, a non-load bearing wall is detailed with expansion joints and flexible anchorages. However, as described in Section 2.4, the exterior wall has no expansion joints and is anchored tightly to and built partially within the structural frame. Therefore, unintended structural loads may arise due to interaction between the wall and frame.

This section reviews the anchorage details to determine whether the exterior walls can transfer lateral loads to the frame and discusses factors which may give rise to unintended stresses in the walls because of differential movements between the exterior walls and frame. The design of the parapet wall and the adequacy of the details provided to prevent ingress of water are assessed.

4.6.1 Anchorage

The exterior walls are anchored to the structural frame which provides lateral support. Wind forces act both as pressure and suction; wind pressure is resisted by the bearing of the wall on the concrete columns and spandrel beams, and wind suction is resisted by tension in the anchors. The walls have the required strength to safely resist the design wind loads.

The anchors are described in Section 2.4.4.3; they consist of four 10-mm diameter bars along each spandrel beam and six 10-mm diameter bars along each column. The steel used for anchors has a yield strength of 4000 kg/cm^2, and each anchor would have a yield load of 3.1 t. A typical wall section between columns and one story in height has an area of $5.4 \times 4.2 = 22.7\ m^2$. The design wind pressure on the exterior walls is 68.7 kg/m^2 at the eighth floor level. The maximum wind load acting on a single panel would be 1.6 t. Assuming the load were caused totally by suction and that it were equally distributed among the anchors, each anchor would be required to resist 0.16 t. Therefore, the anchors have adequate tensile capacity to resist the design loads.

The anchorage of the exterior masonry wall to the interior reinforced concrete wall at the eighth story (Soviet drawings 5KR·3·6·9) consists of 10-mm diameter bars bent in a U-shape with 400-mm long legs and a 300-mm long base. These anchors are detailed at spacings of 1 m horizontally and 730 mm vertically. The base of the U-shape is embedded in the masonry and the legs extend into the concrete. This detail provides adequate anchorage of the exterior masonry wall to the interior concrete wall.

4.6.2 Stresses Due to Differential Movement

Differential movements of the exterior wall and the structural frame are largely constrained because the walls are 1) joined with cast-in-place concrete at the eighth floor level, 2) built tightly around the spandrel beams which span between exterior columns at each floor level, and 3) anchored to the exterior columns.

Differential movements can result from unequal thermal expansion and contraction of the structural frame and the masonry walls, moisture expansion of the masonry walls, and elastic shortening, creep, and shrinkage of the concrete columns of the structural frame. When constrained, differential movements can produce stresses in the masonry and the frame which were not considered in the building design. Frequently, the induced loads cause distress at the corners of masonry walls or areas of minimum cross section, such as piers. Because of the complex interaction between the exterior walls and structural frame, it is difficult to calculate the actual stresses in the exterior walls of the Office Building. For the purpose of this assessment, a simple analysis is performed to give an upper bound estimate of the stresses that might be induced by constrained differential movement.

The differential movements listed above cause the exterior masonry walls to expand and the structural frame to contract. Because of the structural interaction between the two, the walls would be subjected to compressive stresses in addition to self-weight stresses, and the columns in the frame would be relieved of their compressive stresses. Theoretically, it is possible for the columns to be placed in a state of tension.

The following assumptions are used to calculate the upper bound values of compressive stresses that could be developed in the masonry walls:

- Masonry walls are restrained from vertical expansion at the eighth floor
- Columns loads are transferred entirely to the brick masonry walls, and there is no redistribution of loads between columns
- Column loads include unfactored dead loads plus 20 percent of unfactored design live loads
- The walls carry their self-weight
- Additional stresses due to expansion of the masonry are limited by the tensile yield strength of the columns
- There is a 15 °C differential between the exterior masonry and the frame

In addition, the brick masonry was assumed to be uniform with the following properties:

- Compressive strength of brick units is 563 kg/cm^2 and Type M mortar is used
- The modulus of elasticity of the masonry is 176,000 kg/cm^2
- The coefficient of moisture expansion for brick masonry is 0.0002
- The coefficient of thermal expansion of brick masonry is $6.5 \times 10^{-6}/^{\circ}C$

The compressive stresses in the masonry walls were calculated at the first story of a corner pier and at the second story of the pier along the heavily-loaded, column line E/1A. In both cases, the above assumptions result in tensile yielding of the columns. Thus, for this limiting condition, the compressive stresses in the masonry are the sums of self-weight, loads transferred from the columns, and the tensile capacity of the seventh-story columns. For the Type RC-2 columns at the corners, the longitudinal reinforcement consists of four, 20-mm AIII bars, and the tensile yield strength of the column is 50 t. At column line E/1A, there is a Type SC-9 at the seventh story with four 36-mm bars, and two 120 x 260 mm welds connecting the steel cores. The tensile capacity of the Type SC-9 column is 283 t. The following summarizes the compressive loads in the masonry based on this upper bound analysis:

	Corner Pier	Pier at E/1A
Masonry self-weight	177 t	83 t
Column load (DL+0.2LL)	124 t	334 t
Column tensile capacity	50 t	283 t
Total	351 t	700 t
Area (cm^2)	24,600	7,350
Self-weight stress	7 kg/cm^2	11 kg/cm^2
Maximum compressive stress	14 kg/cm^2	95 kg/cm^2

For masonry made with Type M mortar and brick units of compressive strength 563 kg/cm^2, the assumed compressive strength is 132 kg/cm^2 for uninspected construction and 197 kg/cm^2 for inspected construction [4.11]. The allowable stresses are 0.20 of these values, or 26 kg/cm^2 and 39 kg/cm^2.

This maximum stress analysis demonstrates that there is a potential for the development of significant compressive stresses in the exterior masonry because of the built-in restraints. However, the expected values of the stresses are difficult to calculate, as it requires a quantitative description of the structural interaction between the masonry and the frame. The properties of the facing brick masonry and backup building brick masonry must be known, and the interaction between these wythes, which are tied with wire ties not masonry headers, must also be understood. The scope of the investigation did not permit a detailed analysis, but masonry properties have been determined and are discussed in Chapter 5.

4.6.3 Parapet Walls

The parapet walls in the Office Building are 3.25 m high and 520-mm thick, for a height to thickness (h/t) ratio of 6.25. This is in excess of the height/thickness (h/t) value permitted by U.S. building codes [4.13], which typically require an h/t of three or four for solid unreinforced masonry parapet walls. The parapet wall is also weakened by flashing (fig. 2.4.10) which reduces its effective cross section. The parapet walls are laterally supported by the penthouse walls and the diagonal cross-walls of the snow melting rooms. The maximum distance between lateral supports is 8.525 m. In addition, a reinforced concrete bond beam at the top of the wall (Soviet drawings 5KR·3·6 and fig. 2.4.10) is 241 mm in height and 225 mm thick. This beam is reinforced with four 12-mm diameter bars spaced 170 mm apart vertically and 145 mm apart horizontally.

Section 9.10 of ANSI A58.1-1982 requires that parapets in earthquake Zone 0 be designed to resist a lateral force (F_p) given by $F_p = ZIC_pW_p$. For the Office Building: seismic zone coefficient, Z = 0.125; importance factor, I = 1.5; horizontal force factor, C_p = 0.8; and weight of component (parapet), W_p = 3210 kg/m. For these values F_p is 482 kg/m. Assuming the lateral load is uniformly distributed on the parapet wall, the uniform lateral pressure is 148 kg/m^2. The flashing is located 2.65 m from the top of the wall, and the bending moment at this cross section $148 \times (2.65)^2/2 = 520$ kg·m/m. For a reduced wall thickness of 260 mm, the flexural tensile stress 4.6 kg/cm^2 and the dead load stress 0.5 kg/cm^2. The net tensile stress is 4.1 kg/cm^2, and the allowable flexural stress for unreinforced brick masonry built with Type M mortar and with inspection is 2.5 kg/cm^2 [4.11].

The ability of the bond beam to help resist the lateral load was also considered. It was assumed that Mark 300 concrete and grade AIII steel were used. For a span length of 8.525 m, the bond beam would be able to resist a distributed load of 170 kg/m. This is only 35 percent of the total lateral load of 482 kg/m.

Thus it is concluded that the parapet walls do not meet the requirements of U.S. building codes for unreinforced masonry and do not meet the ANSI criteria for lateral load resistance. Remedial measures are required and these are discussed in Chapter 6.

4.6.4 Resistance to Water Penetration

The penetration of water into the exterior masonry can have an adverse effect over time on the masonry and surrounding elements. Freezing and thawing of water within masonry walls can cause disruption and failure of masonry units and mortar, and the entrance of water can lead to corrosion of anchors, ties and other embedded metal accessories. The design details, with the exception of the window soffits (fig. 2.4.7), appear to be adequate for protection against excessive water entrance and accumulation; the flashings shown in the details appear to be placed appropriately. The conditions of the window soffits were examined during the site investigations and are discussed in Chapter 5.

4.7 CONCLUSIONS

The conclusions presented below are based primarily on knowledge of the structural system and material properties as specified in the Soviet plans. Where as-built data was required prior to making a final assessment, specific needs are listed. The assessment of the as-built data is given in Chapter 5.

4.7.1 Beams and Floor Planks

- The design strengths of the precast floor planks are adequate.

- The design flexural and shear strengths of all steel beams are adequate.

- The flexural and shear design strengths of all concrete beams are adequate except for two inverted T-beams (Type R-38-8). The as-built condition of these critically loaded beams was investigated.

- The shear strength of the concrete beam flanges is adequate.

- Welded connections are adequately designed.

4.7.2 Columns

- The design strengths of steel columns are adequate.

- The reinforced concrete columns are not slender and their capacity is controlled by the concentric axial design strength for short columns. Thirty-five Type RC-5 columns in the lower stories are understrength. The understrength columns are listed in table 4.2.7. Cores were taken from these columns to determine the in-place concrete strength.

- All of the composite steel core columns are slender and the procedures of ACI 318-83 were used to evaluate slenderness effects. The design strengths of these columns are adequate except for Type SC-9 columns located at the corners of the core in the second and third stories. Cores were taken from Type SC-9 columns before making a final assessment.

4.7.3 Shear Walls

- The as-designed shear wall system is sufficient to resist the design wind load provided that the horizontal shear wall joints are properly filled.

- Empty joints would cause the shear wall panels to act independently. Diagonal tensile stress could occur under the design wind load that exceed the design shear strength of the specified concrete. In addition, the rigidity of the shear wall system would be reduced significantly.

- The shear wall-to-column welded connections are adequately designed.

4.7.4 Foundation

- The piles have adequate structural and geotechnical load capacity and the foundation can safely support the design loads. Anticipated effects of settlements are negligible.

- Driving of the piles was hard, and the calculated stresses during driving of piles are high relative to the specified concrete strength. However, it is unlikely that the load capacity of the piles was affected.

4.7.5 Progressive Collapse

- The failure of a major load supporting beam from the first through eighth floors will not precipitate a progressive collapse provided remedial measures are taken to provide a restraining system (section 6.3).

- The failure of a major load supporting steel beam on the roof is not likely to precipitate a progressive collapse.

- The failure of any column will not precipitate a progressive collapse provided retrofit measures are implemented to enhance the structural performance of partition walls (section 6.3).

4.7.6 Brick Masonry Exterior Walls

- Wall anchors are adequately designed.

- Parapet walls do not meet the requirements of U.S. building codes for unreinforced masonry and do not meet ANSI criteria for lateral force resistance.

- There is a potential for developing significant compressive stresses in the exterior masonry walls because of the interaction between the structural frame and the exterior walls.

4.8 REFERENCES

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Table 4.2.1. Flexural and shear strengths of steel beams

BEAM	COMPACT SECTION	Z (cm^3)	DESIGN STRENGTHS M ($t\cdot m$)	DESIGN STRENGTHS V (t)	REQ'D STRENGTHS M ($t\cdot m$)	REQ'D STRENGTHS V (t)	DESIGN/REQ'D M	DESIGN/REQ'D V
BUILT-UP PLATE SECTIONS								
5	YES	16912	502.3	228.1	287.4	85.9	1.75	2.66
6	YES	14455	429.3	228.1	329.7	85.4	1.30	2.67
7	YES	11997	356.3	228.1	262.8	79.3	1.36	2.88
8	NO	9156	271.9	116.0	194.8	59.0	1.40	1.97
8a	YES	10000	297.0	228.1	126.5	95.1	2.35	2.40
9	NO	6492	192.8	113.3	111.6	49.2	1.73	2.30
9a	YES	7377	219.1	228.1	81.0	38.3	2.71	5.95
10	NO	5238	155.6	113.3	110.8	35.5	1.40	3.19
10a	YES	6122	181.8	228.1	65.9	48.2	2.76	4.73
11	YES	14444	429.0	199.6	326.7	89.3	1.31	2.23
12	NO	7806	231.8	117.1	238.7	86.4	0.97	1.36
13	NO	5275	156.7	117.1	140.8	62.6	1.11	1.87
14	YES	3631	107.8	106.9	24.2	21.6	4.45	4.94
15	YES	3631	107.8	106.9	90.1	46.2	1.20	2.31
16	YES	3645	108.3	89.1	67.3	43.9	1.61	2.03
17	YES	4263	88.2	49.7	64.6	25.6	1.37	1.94
18	NO	3568	73.9	56.8	33.8	23.2	2.19	2.45
19	YES	2669	55.2	59.6	46.8	24.0	1.18	2.48
20	YES	3022	62.6	104.3	30.5	72.5	2.05	1.44
30	YES	8311	246.8	228.1	80.6	52.0	3.06	4.39
ROLLED SECTIONS								
21	YES	2780	57.5	89.4	32.0	20.1	1.80	4.46
22	YES	2238	46.3	75.1	27.0	21.6	1.71	3.48
23	YES	519	10.7	26.1	9.7	12.9	1.11	2.02
24	YES	1048	21.7	41.2	16.6	13.9	1.30	2.97
25	YES	837	17.3	39.7	11.2	9.3	1.55	4.28
26	YES	426	8.8	24.2	3.2	7.2	2.72	3.36
27	YES	852	17.6	48.4	14.5	24.3	1.21	1.99
28	YES	422	8.7	29.5	9.0	10.9	0.97	2.72

Table 4.2.2. Flexural strength of reinforced concrete beams

TYPE	f'_c (kg/cm²)	b (cm)	A_s (cm²)	d (cm)	A'_s (cm²)	d' (cm)	DESIGN STRENGTH (t·m)	REQUIRED STRENGTH (t·m)	DES STR/ REQ'D STR
R-86-12	241	20	65.44	50.91	48.25	6.9	102.70	98.20	1.05
R-74-12	241	20	56.29	54.30	32.17	5.4	93.44	81.61	1.14
R-56-12	298	20	36.95	38.30	24.63	4.3	44.89	42.33	1.06
R-56-8	241	20	24.63	39.70	6.28	2.5	28.65	12.41	2.31
R-50-8	241	20	19.63	40.00	6.28	2.5	24.53	18.09	1.36
R-50-12	298	20	28.40	39.60	12.32	2.9	35.52	34.29	1.04
R-44-12	241	20	22.13	39.90	6.28	2.5	26.75	27.20	0.98
R-38-8	184	20	9.82	41.20	6.28	2.5	13.66	24.80	0.55
R-32-12	241	20	9.82	41.20	6.28	2.5	13.72	8.40	1.63
R-32-8	184	20	6.28	41.50	6.28	2.5	8.89	11.24	0.78
R-26-12	241	20	6.28	41.50	6.28	2.5	8.92	6.57	1.36
R-26-8	184	20	4.02	41.70	6.28	2.5	5.76	6.93	0.83
R-20-12	184	20	4.02	41.70	6.28	2.5	5.76	4.44	1.30
R-14-12	184	20	1.57	42.00	6.28	2.5	2.32	3.22	0.72
RF-56-8	184	49	24.54	38.50	7.85	2.0	30.82	26.60	1.16
RF-50-8	184	49	22.80	38.50	7.85	2.0	28.93	21.54	1.34

Table 4.2.3(a). Shear strength of reinforced concrete beams: stirrup spacings

TYPE	A_v (cm^2)	S1 (mm)	X1 (mm)	S2 (mm)	X2 (mm)	S3 (mm)	X3 (mm)
R-56-12	1.57	75	230	150	830		
R-50-12	1.57	150	230				
R-86-12	2.26	75	230	150	1130	300	2780
R-74-12	2.26	75	230	150	980	300	2330
R-56-8	1.01	150	230	300	1280		
R-50-8	1.01	150	230	300	1130		
R-44-12	1.57	150	230				
R-32-12	1.01	150	230				
R-26-12	1.01	150	230				
R-38-8	1.01	150	230				
R-32-8	0.57	150	230				
R-26-8	0.57	150	230				
R-20-12	0.57	150	230				
R-14-12	0.57	150	230				
RF-56-8	2.26	100	230	150	530	300	1280
RF-50-8	1.57	75	230	100	380	150	680

Table 4.2.3(b). Shear strength of reinforced concrete beams

	STRENGTHS FOR S1			STRENGHTS FOR S2			STRENGTHS FOR S3		
TYPE	DES (t)	REQ (t)	DES/REQ	DES (t)	REQ (t)	DES/REQ	DES (t)	REQ (t)	DES/REQ
R-56-12	33.21	22.73	1.46	19.58	17.08	1.15			
R-50-12	20.25	19.95	1.01						
R-86-12	59.29	38.60	1.54	33.21	31.30	1.06	20.17	7.74	2.61
R-74-12	63.23	35.63	1.77	35.42	20.82	1.70	21.51	13.09	1.64
R-56-8	14.65	6.68	2.19	10.10	3.78	2.67			
R-50-8	14.76	10.52	1.40	10.18	6.05	1.68			
R-44-12	19.79	17.32	1.14						
R-32-12	15.20	10.29	1.48						
R-26-12	15.31	5.37	2.85						
R-38-8	14.46	16.81	0.86						
R-32-8	10.43	7.51	1.39						
R-26-8	10.48	5.67	1.85						
R-20-12	10.48	3.82	2.74						
R-14-12	10.55	2.54	4.15						
RF-56-8	34.28	14.18	2.42	24.42	12.41	1.97	14.56	8.00	1.82
RF-50-8	32.10	12.40	2.59	25.25	11.50	2.20	18.40	9.80	1.88

Table 4.2.4. Design strengths of steel columns

TYPE	AREA (cm^2)	WEAK AXIS I (cm^4)	UNBRACED LENGTH (cm)	YIELD STRENGTH (kg/cm^2)	DESIGN STRENGTH (t)	REQUIRED STRENGTH (t)	DESIGN STR/ REQ'D STR
S-1	135.0	4439	273	3300	330	165	2.0
S-2	66.8	1528	369	2300	102	86	1.2
S-3	81.0	5186	340	2300	147	33	4.4

Table 4.2.5. Slenderness evaluation

Column Type	Unbraced Length (m)	Strong Axis	Weak Axis
RC-5	3.9 and 4.5		
RC-3	3.9 and 4.5		
RC-2	3.9		
SC-12	3.9 and 4.5	slender	slender
SC-9	3.9 and 4.5	slender	slender
SC-7	3.9		slender
SC-5	3.9	slender	slender
SC-2	3.9		slender

Table 4.2.6. Concentric axial design strength of concrete columns

COLUMN TYPE	CORE		AIII REINFORCING STEEL*	CONCRETE		DESIGN STRENGTH
	F_y (kg/cm^2)	Area (cm^2)	Area (cm^2)	f'_c (kg/cm^2)	Area (cm^2)	(t)
RC-5	--	--	69.8	357	1530	417
RC-3	--	--	32.3	298	1568	294
RC-2	--	--	12.6	78.4	1580	167
SC-12	3300	468.0	40.7	241	1091	1081
SC-9	3300	312.0	40.7	241	1247	811
SC-7	3300	146.6	40.7	241	1413	524
SC-5	2300	74.9	40.7	241	1484	358
SC-2	2300	61.2	32.3	241	1136	281

*AIII Reinforcing steel has a yield strength of 4000 kg/cm^2

Table 4.2.7. Type RC-5 columns with design strengths less than required strengths

STORY	LOCATION	DESIGN/REQUIRED STRENGTH
B	A1-4	0.89
	A1-5	0.86
	A1-6	0.85
	A1-7	0.86
	E -4B	0.82
	E -6A	0.82
	F -4B	0.70
	F -6A	0.79
	F1-5	0.89
	F1-5B	0.96
	F1-6B	0.94
	J1-4	0.89
	J1-5	0.81
	J1-6	0.81
	J1-7	0.85
1	A1-4	0.98
	A1-5	0.95
	A1-6	0.94
	A1-7	0.98
	E -4B	0.91
	E -6A	0.91
	F -4B	0.78
	F -6A	0.89
	J1-5	0.95
	J1-6	0.94
2	C1-5	0.82
	C1-6	0.82
	F -4B	0.88
	G2-5	0.82
	G2-6	0.75
3	C1-5	0.93
	C1-6	0.93
	G2-5	0.93
	G2-6	0.85
4	G2-6	0.98

Table 4.2.8. Steel beam connections

	Connection Type	Shear Capacty of Weld Length mm	Leg mm	Capacity ton	Shear Capactiy of Web Width mm	Depth mm	F_u kg/mm^2	Capacity ton	Most Heavily Loaded Connection Location ton	Dead ton	Live ton	1.2D + 1.6L	Capacity/Load Weld	Beam
1		1472	8	172.3	16	736	46	243.8	8-D/1A	66.13	27.46	123.29	1.40	1.98
2 a		1472	8	172.3	16	736	46	243.8	1-G2/6B	49.4	27.55	103.36	1.67	2.36
b		360	8	42.2	12	360	38	73.9		12.42	3.69	20.81	2.03	3.55
3		260	10	38.1	11	500	38	94.1	1-E/1A	15.44	2.43	22.42	1.70	4.20
4 a	corbel								1-C1/6B	7.87	0	9.44		
b		1472	8	172.3	10	750	38	128.3		64.24	12.65	97.33	1.77	1.32
5	corbel								1-H1/9A	13.64	2.73	20.74		
6 a	I55 on seat			74.9					1-C1/6	7.87	0	9.44	7.93	
b	corbel									26.28	6.67	42.21		
7	beam seat			74.9					1-G2/5	24.25	3.58	34.83	2.15	
8 a	steel bracket			43.9					1-A1/5	8.82	2.34	14.33	3.06	
b	girder on corbel									19.62	6.67	34.22		
9 a	corbel								1-I1/4	11.62	3.08	18.87		
b	corbel									29	5.68	43.89		
10		700	8	82.0	10	700	46	144.9	1-I1/6B	24.4	15.45	54.00	1.52	2.68
11 a	corbel								1-A1/8	8.28	0	9.94		
b	beam seat			74.9						9	3.08	15.73	4.76	
12 a		1460	10	213.7	16	736	46	243.8	1-C/6B	64.86	13	98.63	2.17	2.47
b	I40 on girder									8.61	0	10.33		
13 a		920	10	134.6	10	468	46	96.9	C/8	25.9	5.54	39.94	3.37	2.43
b	I40 on girder									8.28	0	9.94		
14		1000	8	117.1	12	500	38	102.6	n 1-H/8	23	4.67	35.07	3.34	2.93
15 a		660	10	96.6	12	330	38	67.7	n 1-G2/8	12.42	3.69	20.81	4.64	3.25
b		660	10	96.6	12	330	38	67.7		8.79	2.61	14.72	6.56	4.60
16		660	10	96.6	12	330	38	67.7	n 1-G2/9A	12.42	3.69	20.81	4.64	3.25
17		340	8	39.8	8.3	340	38	48.3	n 1-J/3	8.26	0	9.91	4.02	4.87
18 a		500	8	58.5	12	500	38	102.6	1-E/3	16.45	2.43	23.63	2.48	4.34
b	I40 on girder									6.46	0	7.75		
19	stairs													
20	stairs								n 2-F/4			0.00		
21 a	I40 on girder								n 1-G/4	8.2	0	9.84		
b	I40 on girder									8.2	0	9.84		
22 a		320	8	37.4	8.3	320	38	45.4	n 1-C/5	4.41	0	5.29	7.08	8.58
b		320	8	37.4	8.3	320	38	45.4		7.95	0	9.54	3.93	4.76
23 a	est	200	8	23.4	8	200	38	27.4	2-D/4A	1.86	0.75	3.43	6.82	7.97
b	est	200	8	23.4	8	200	38	27.4		1.86	0.75	3.43	6.82	7.97
24 a	girder on corbel								1-A1/7	20.71	5.48	33.62		
b	steel bracket			43.9						8.27	0	9.92	4.42	
25 a	C30 on corbel						0		7-G2/4A	4.98	1.41	8.23		
b		1000	12	175.6	16	736	46	243.8		43.96	14.45	75.87	2.31	3.21
c	I55 on corbel									14.78	2.42	21.61		
26 a		250	6.5	23.8	6.5	250	38	27.8	n 7-H/3	3.9	1.82	7.59	3.13	3.66
b		250	6.5	23.8	6.5	250	38	27.8		3.9	1.82	7.59	3.13	3.66
27 a	C30 on girder								n 7-G2/4	4.98	1.41	8.23		
b		700	8	82.0	10	700	46	144.9		25.79	6.56	41.44	1.98	3.50
28		680	8	79.6	12	680	46	168.9	n 7-H/4	25.04	6.86	41.02	1.94	4.12
29 a		1000	12	175.6	16	736	46	243.8	8-A1/5	23.87	12.83	49.17	3.57	4.96
b		900	10	131.7	16	736	46	243.8		37.38	12.83	65.38	2.01	3.73
c	beam seat			74.9						3.18	0	3.82	19.63	0.00
30 a		1472	8	172.3	16	736	46	243.8	8-F/1A	68.73	27.46	126.41	1.36	1.93
b		1000	12	175.6	10	768	38	131.3		32.63	8.58	52.88	3.32	2.48
31 a	steel bracket			25.8					8-J/4	4	0	4.80	5.38	
b		1000	12	175.6	16	736	46	243.8		67.34	27.46	124.74	1.41	1.95
32 a	I55 on corbel								8-F/4A	4	0	4.80		
b	beam seat			74.9						4.5	0.93	6.89	10.87	
c		1472	8	172.3	16	736	46	243.8		67.34	27.46	124.74	1.38	1.95
33 a	I55 on seat			74.9					8-F1/4B	15.65	2.16	22.24	3.37	
b	I55 on corbel									4	0	4.80		
c		340	10	49.8	8.3	320	38	45.4		4.16	0	4.99	9.97	9.10
34 a	girder on corbel								8-C1/6	23.87	12.83	49.17		
b	I55 on bracket						43.9							

Table 4.2.8. Steel beam connections (continued)

		Connection Type	Shear Capacty of Weld Length mm	Leg mm	Capacity ton	Shear Capactiy of Web Width mm	Depth mm	F_u kg/mm^2	Capacity ton	Most Heavily Loaded Connection Location ton	Dead ton	Live ton	1.2D + 1.6L	Capacity/Load Weld	Beam
35	a	I55 on corbel								8-G2/4A	14.78	2.42	21.61		
	b	beam seat			74.9						4.73	0	5.68	13.20	
	c		1472	8	172.3	16	736	46	243.8		49.31	26.23	101.14	1.70	2.41
36		I40 on I55								n 8-4/F1	4.16	0	4.99	0.00	0.00
37		steel column on girder								8-D1/9A					
38	a	girder on column								PH-C/1A	51.68	14.65	85.46		
	b		500	8	58.5	12	500	38	102.6		26.84	4.6	39.57	1.48	2.59
	c		300	6.5	28.5	6.5	300	38	33.4		2.45	0	2.94	9.71	11.34
39			1060	8	124.1	16	500	46	165.6	PN-C/4A	51.68	14.65	85.46	1.45	1.94
40	a	girder on column								PH-E/1A	52.67	21.14	97.03		
	b		200	8	23.4						4.91	0	5.89	3.97	
	c		200	8	23.4						1.36	0	1.63	14.35	
41	a	girder on column								PH-E/4A	71.17	19.84	117.15		
	b		600	8	70.3	12	600	46	149.0		2.85	1.31	5.52	12.74	27.02
	c		300	8	35.1	6.5	300	38	33.4		4.74	1.65	8.33	4.22	4.00
	d		300	8	35.1	6.5	300	38	33.4		5.93	1.97	10.27	3.42	3.25
42	a	girder on column								nPH-D1/1A	53.47	15.97	89.72		
	b		300	8	35.1	6.5	300	38	33.4		1.36	0	1.63	21.52	20.43
	c		200	8	23.4						1.36	0	1.63	14.35	
43	a	girder on column								nPH-D1/4A	29.68	7.19	47.12		
	b		420	12	73.8	12	420	46	104.3		35.35	11.49	60.80	1.21	1.72
	c		1000	8	117.1	10	468	46	96.9		39.12	11.81	65.84	1.78	1.47
	d		300	8	35.1	6.5	300	38	33.4		2.45	0	2.94	11.95	11.34
44	a	girder on column								PH-D/4A	38.74	10.82	63.80		
	b		300	8	35.1	6.5	300	38	33.4		1.64	0	1.97	17.85	16.94
	c		300	8	35.1	6.5	300	38	33.4		2.45	0	2.94	11.95	
45	a	girder on column								PH-C1/4A	44.13	12.57	73.07		
	b		600	10	87.8	10	650	46	134.6		43.97	12.09	72.11	1.22	1.87
	c		300	8	35.1	6.5	300	38	33.4		2.45	0	2.94	11.95	11.34
46	a		550	12	96.6	10	550	46	113.9	PH-E/3	33.01	6.5	50.01	1.93	2.28
	b		300	8	35.1	6.5	300	38	33.4		7.09	1.29	10.57	3.32	3.15
47			500	12	87.8	12	500	38	102.6	nPH-F/3	22.74	6.5	37.69	2.33	2.72
48	a		500	8	58.5	12	550	38	112.9	PH-A1/4	11.58	3.16	18.95	3.09	5.96
	b		500	8	58.5	12	636	46	158.0		12.92	3.46	21.04	2.78	7.51
49			600	8	70.3	12	676	46	167.9	PH-A1/4A	34.93	11.08	59.64	1.18	2.82
50	a		500	12	87.8	12	500	38	102.6	PH-C1/4A	23.19	7.02	39.06	2.25	2.63
	b		936	8	109.6	10	500	46	103.5		34.94	11.08	59.66	1.84	1.73
51	a	girder on column								PH-A1/6	16.69	11.39	38.25		
	b		300	8	35.1	6.5	300	38	33.4		2.45	0	2.94	11.95	11.34
	c		300	8	35.1	6.5	300	38	33.4		1.32	0	1.58	22.17	21.05
52	a	bracket (est 12)			43.9					PH-C1/6	16.69	11.39	38.25	1.15	
	b	bracket (est 12)			43.9						21.52	5.53	34.67	1.27	
	c	corbel									7.51	0.46	9.75		
53	a	I60 on column								PH-A1/1A	10.15	1.95	15.30		
	b		500	8	58.5	12	500	38	102.6		10.15	1.95	15.30	3.83	6.71
54	a	I60 on column								PH-A1/3	26.84	4.6	15.30		
	b	I60 on column									11.58	3.16	18.95		
55			600	8	70.3	10	650	46	134.6	n PH-A1/4A	16.13	4.2	26.08	2.69	5.16
56			500	10	73.2	10	500	38	85.5	n PH-C/1A	25.16	3.68	36.08	2.03	2.37
57	a		330	8	38.6	8.3	330	38	46.8	n R-C/6	7.78	3.62	15.13	2.55	3.10
	b		330	8	38.6	8.3	330	38	46.8		3.97	1.85	7.72	5.00	6.06
58	a	C30 on corbel								R-C1/6	3.97	1.85	7.72		
	b	est	400	8	46.8	25	200	38	85.5		1.85	0.73	3.39	13.82	25.24
			330	8	38.6	8.3	300	38	42.6		1.85	0.73	3.39	11.40	12.57
			200	8	23.4				0.0		1.85	0.73	3.39	6.91	
59	a	bracket (14-250)			51.2				0.0	R-E/6A	11.31	4.92	21.44	2.39	
	b	bracket (14-250)			51.2				0.0		4.75	1.58	8.23	6.22	
	c		500	6	43.9	6.5	300	38	33.4		2.22	0.89	4.09	10.74	8.16
60	a	bracket (12-200)			35.1					n R-D1/6A	2.59	3.51	8.72	4.02	
	b		250	8	29.3	5.4	220	38	20.3		2.22	0.89	4.09	7.16	4.97

Table 4.2.8. Steel beam connections (continued)

	Connection Type	Shear Capacty of Weld Length mm	Leg mm	Capacity ton	Shear Capactiy of Web Width mm	Depth mm	F_u kg/mm^2	Capacity ton	Most Heavily Loaded Connection Location ton	Dead ton	Live ton	1.2D + 1.6L	Capacity/Load Weld	Beam
61 a	bracket (12-200)			35.1					R-F1/5	6.05	2.46	11.20	3.14	
b		700	8	82.0	8.3	300	38	42.6		3.78	1.67	7.21	11.37	5.91
c		600	8	70.3						3.78	1.67	7.21	9.75	
62		250	8	29.3	6.5	300	38	33.4	R-F1/4B	6.05	2.46	11.20	2.61	2.98
63	I40 on corbel								R-G1/5	3.24	1.46	6.22		
64	column connection													
65 a	girder on column								PH-E/6A	28.69	12.32	54.14		
b		270	8	31.6	6.5	300	38	33.4		2.97	1.97	6.72	4.71	4.97
c		250	6	22.0	6.5	300	38	33.4		1.86	0.99	3.82	5.75	8.74
66	column connection													
67 a		520	10	76.1	12	520	38	106.7	n PH-D1/4A	23.19	7.02	39.06	1.95	2.73
b		250	6	22.0	6.5	300	38	33.4		2.64	0	3.17	6.93	10.53
68	column connection													
69 a	C30 on corbel								1-D/6B	12.59	2.05	18.39		
b	corbel									18.23	1.71	24.61		
c		1472	8	172.3	16	736	46	243.8		58.26	14.42	92.98	1.85	2.62
70	steel beam on T-beam ledge								n 8-F1/5A	2.73	0	92.98		
71 a	beam seat			74.9					2,8-F1/5	5.68	0.86	3.28	22.86	
72		500	6	43.9	5.4	220	38	20.3	2,8-F/5	5.68	0.86	8.19	5.36	2.48
73	beam seat			74.9				0.0	2,8-C2/5A	11.97	0.58	15.29	4.90	
74 a		340	8	39.8	8.3	340	38	48.3	2,8-C1/5A	6.53	0.91	9.29	4.28	5.19
b		340	8	39.8	8.3	340	38	48.3		7.56	0	9.07	4.39	5.32
75		680	8	79.6	4.3	340	38	48.3	1-C1/5A	7.57	0.46	9.82	8.11	4.91
76 a		340	8	39.8	8.3	340	38	48.3	2,8-C1/5A	6.53	0.91	9.29	4.28	5.19
b		340	8	39.8	8.3	340	38	48.3		7.56	0	9.07	4.39	5.32
77		340	8	39.8	8.3	340	38	48.3	2,8-C1/5A	10.01	0.33	12.54	3.17	3.85
78 a	beam on shear wall ledge								2,8-C1/4A	7.35	0.91	10.28		
b	beam on shear wall ledge									7.55	0	9.06		
79	I30 on column								1-E/6A	5.9	0	7.08		
80	beam on T-beam ledge								1,8-E/4B	4.09	0	4.91		
81	beam seat			74.9					1,8-E/4B	4.09	0	4.91	15.26	
82		470	8	55.0	11	470	38	88.4	1,8-G2/4B	10.13	0.26	12.57	4.38	7.03
83 a		470	8	55.0	11	470	38	88.4	1,8-F1/4B	10.13	0.26	12.57	4.38	7.03
b		500	6	43.9	5.4	220	38	20.3		4.88	0.61	6.83	6.43	2.97
84	beam on T-beam ledge								1,8-F1/5A	12.15	0.51	15.40		
85	beam on small column at entrance													
86	beam on Z-beam ledge at entrance													
87	beam sits in wall								n R-C2/6A	13.62	5.47	25.10		
88	beam sits in wall								n R-D1/6A	4.93	2.14	9.34		
89	beam sits on wall								n R-C2/6A	3.24	1.46	6.22		
90	beam sits on T-beam ledge								1-E/5A	7.27	0	8.72		
101 a	bracket (10-200)			29.3		concrete beam			R-F/5A	5.65	2.46	10.72	2.73	
b	bracket (10-200)			29.3		concrete beam					11.31	4.92	21.44	1.37

Notes: Weld metal ultimate tensile strength, F_u = 46 kg/mm^2
Weld capacity reduction factor, ϕ = 0.75.
Locations of most heavily loaded connections not at a column are indicated with an "n" and the nearest column.
Column type numbers correspond to detail numbers in Soviet working drawings.

Table 4.3.1. Story shear, total shear, and overturning moment summary

Floor Level	Height (m)	K_z	p_z (kg/m^2)	Story Shear (t)	Centroid Height (m)	Total Shear (t)	Overturning Moment (t·m)
BSMT	-5.2					73.94	2064.68
1	0	0.12	19.74	3.88	2.40	73.94	1680.18
2	4.8	0.12	19.74	4.40	6.90	70.05	1334.58
3	9.0	0.19	31.26	6.14	11.10	65.66	1049.56
4	13.2	0.243	39.99	7.56	15.30	59.53	786.64
5	17.4	0.291	47.84	8.84	19.50	51.96	552.50
6	21.6	0.333	54.82	10.16	23.70	43.12	352.81
7	25.8	0.384	63.13	11.44	27.92	32.96	193.03
8	30.04	0.417	68.59	12.40	32.15	21.51	77.54
Roof	34.25	0.457	75.16	9.10	35.69		
PH	37.12	0.484	79.66				

Notes: maximum wind velocity = 113 km/h; C_p = 1.3; G = 1.5; I = 1.07

Table 4.4.1. Summary of load test results

	Pile No. 353	Pile No. 367	Pile No. 371
Date Driven	Nov. 25, 1979	Dec. 14, 1979	Dec. 27, 1979
Date Tested	July 3-4, 1979	July 1-2, 1979	July 11-12, 1980
Furnished Length, m	7	11	12
Penetration Length, m	3.5	10.2	11.2
Final Resistance cm/10 blows	0.8	6.5	2.9
Maximum Test Load, ton	90	100	100
Gross Displ. at Max. Load, mm	6.5	5.1	3.4
Net Displ. after Unloading, mm	3.7	2.5	1.0
Extrapolated Capacity, ton	100	120 +	120 ++

Table 4.4.2. List of marginal piles

Date	Pile No.	Approx. Location	Length m	No. of Strokes	Driving Depth m	Penetration Last 10 Strokes, cm	Est. Capacity t	Test Load t
12/10	770	D1/6A*	8	198	7.5	2.8	118 - 143	
12/11	377	J/4	9	258	8.3	5.8	99 - 120	
12/11	372	J/4	9	257	8.3	5.9	98 - 119	
12/12	340	J5/J6	8	173	7.5	6.2	97 - 117	
12/12	325	J/6	7	151	6.2	5.6	100 - 121	
12/14	316	J/7	7	178	6.0	5.8	99 - 118	
12/14	367	J/4	11	327	10.2	6.5	96 - 116	120+
12/15	376	J/4	11	331	10.2	6.3	97 - 117	
12/27	362	J/4	11	no data	5.0	2.5	120 - 144	
12/27	371	J/4	12	no data	11.2	2.9	117 - 142	120+
12/27	366	J/4	12	no data	11.2	2.9	117 - 142	
12/27	375	J/4	12	no data	11.2	2.9	117 - 142	
12/27	370	J/4	12	no data	11.2	2.9	117 - 142	
12/27	365	J/4	12	no data	11.2	3.5	114 - 137	
12/27	361	J/4	12	no data	11.2	3.5	114 - 137	
12/27	378	J/4	12	no data	11.2	3.5	114 - 137	
12/27	374	J/4	12	no data	11.2	3.5	114 - 137	
12/27	369	J/4	12	no data	11.2	3.6	112 - 135	
12/27	364	J/4	12	209	7.5	3.5	114 - 137	
12/27	373	J4/H4	12	215	10.4	3.5	114 - 137	
12/27	368	J4/H4	12	225	10.9	3.5	114 - 137	
12/27	363	J4/H4	12	249	11.2	3.5	114 - 137	
12/28	354	J5/H5	12	251	10.6	3.0	116 - 141	
12/28	349	J5/H5	12	262	11.3	3.1	115 - 140	

*The boring nearest each of these location is C-3 (fig. 2.2.1) except for the first entry (12/10) which is nearest C-1/C-4.

Table 4.5.1. Load transfer between core columns connected to shear walls

Col No. (1)	Max Load (t) (2)	Incr. Load* (t) (3)	Function (4)	Transfer to Col.** (5)	Transfer to Col.** (6)	Max Req. Capacity (t) (7)	Ultimate Capacity (t) (8)
C1/4A	542.3	69.2 60.0	Major Beam	D/4A 271 t	C1/5 271 t	715	1744
D/4A	433.0	82.53 50.0	Major Beam	C1/4A 173 t	D1/4A 259 t	704	1744
D1/4A	254.7	28.5 24.8	Mech.Eq., Parts.	E/4A 113 t	D/4A 142 t	768 (crit.)	754
E/4A	512.3	73.2 53.3	Major Beam	D1/4A 513 t	-	625	1744
C1/5	394.8	49.7 47.6	Minor Beam	C1/4A 237 t	C1/6 158 t	784	1386
C1/6	389.0	46.3 41.7	Minor Beam	C1/5 389 t	-	547	1386
F/6B	485.1	62.8 49.9	Major Beam	F1/6B 485 t	-	611	1744
F1/6B	280.0	33.8 26.5	Mech.Eq. Parts.	F/6B 126 t	G/6B 154 t	765 (crit.)	754
G/6B	408.3	58.9 45.7	Major Beam	F1/6B 204 t	G2/6B 204 t	675	1744
G2/6B	533.0	73.3 55.7	Major Beam	G/6b 267 t	G2/6 267 t	737	1744
G2/6	422.2	47.4 44.4	Minor Beam	G2/6B 170 t	G2/5 252	818	1386
G2/5	395.6	52.4 41.6	Minor Beam	G2/6 396 t	-	648	1386

* Top number is maximum incremental load, bottom number is average incremental load.

** Maximum load on column is shown below column identification.

Table 4.5.2. Load transfer between core columns adjacent to hollow core masonry walls

Col No.	Max Ld. (t)	Incr. Load* (t)	Function	Transfer to Col.**	Transfer to Col.**	Required Capacity (t)	Ultimate Capacity (t)
C1/6B	562.7	78.5 59.4	Major Beam	D/6B 563 t	-	736	1744
D/6B	432.2	82.9 48.7	Major Beam	C1/6B 173 t	D1/6B 259 t	995	1744
D1/6B	262.8	28.0 25.3	Mech.Eq., Part.	D/6B 146 t	E/6B 117 t	748 (crit.)	754
E/6B	485.0	62.8 49.9	Major Beam	D1/6B 485 t	-	602	1744
F/4A	514.9	82.0 53.8	Major Beam	F1/4A 515 t	-	633	1744
F1/4A	265.4	31.4 26.6	Mech.Eq., Part.	F1/4A 118 t	G/4A 147 t	780 (crit.)	754
G/4A	442.2	77.3 49.9	Major Beam	F1/4A 265 t	G2/4A 177 t	1006	1744
G2/4A	563.8	76.4 59.5	Major Beam	G/4A 564 t	-	741	1744

* Top number is maximum incremental load, bottom number is average incremental load.

** Maximum load on column is shown below column identification.

Table 4.5.3. Load transfer between interior core columns

Col No.	Max Load (t)	Incr. Load (t)	Function	Transfer to Col.	Transfer to Col.	Type of Transfer	Required Capacity (t)	Ultimate Capacity (t)
C2/6	162.1	28.4 16.8	Stairs, Mech. Eq.	C2/5A 162 t	-	Mas. Part.	267	754
C2/5A	253.2	29.3 26.8	Stairs, Mech. Eq.	C2/6 105 t	D1/5A 149 t	Mas. Part.	415	754
D1/5A	258.2	33.5 27.4	Elev. Shaft	C2/5A 151 t	D1/6 107 t	Mas. Part.	495	754
D1/6	236.6	27.4 25.2	Elev. Shaft	D1/5A 237 t	-	Mas. Part.	344	754
F1/5B	279.6	34.9 29.8	Elev. Shaft	F1/5 280 t	-	Mas. Part. Conc. Wall	580	754
F1/5	301.2	37.2 32.2	Elev. Shaft	F1/5B 301 t	-	Conc. Wall Mas. Part.	418	754
G1/5	150.7	17.1 15.6	Stairs, Mech.Eq.	G1/5B 151 t	-	Conc. Wall Mas. Part.	228	754
G1/5B	188.0	23.2 19.2	Stairs, Mech. Eq.	G1/5 188 t	-	Conc. Wall Mas. Part.	339	754
E/6A	314.2	35.6 32.8	Elev. Lobby	E/6B 314 t	-	Ret. Cant. Beam	314	754
E/4B	316.7	37.0 33.7	Elev. Lobby	E/4A 317 t	-	"	317	754
F/4B	365.4	36.7 39.1	"	F/4A 365 t	-	"	365	754
F/6A	324.6	36.5 34.0	"	F/6B 325 t	-	"	324	754

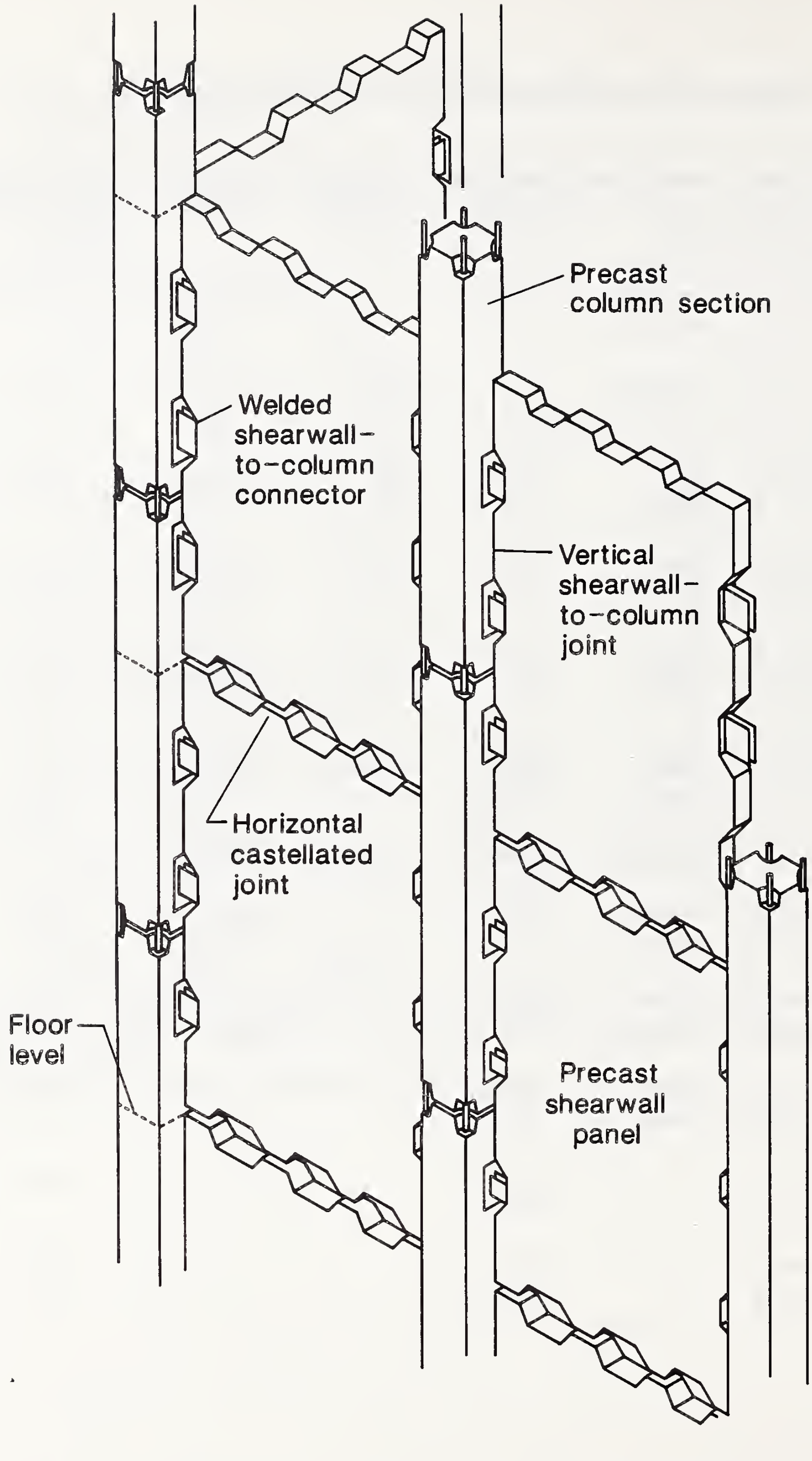


Figure 4.1.1. Schematic view of precast shear wall assembly

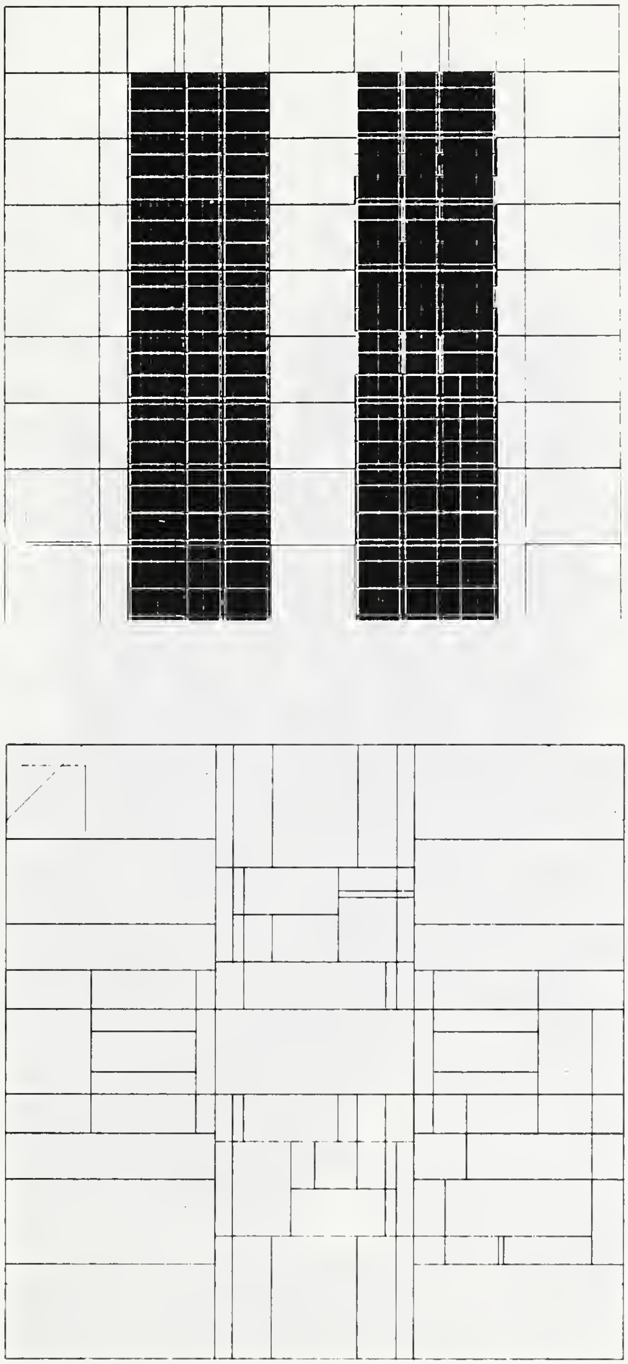


Figure 4.1.2. Profile and eighth floor plan of the finite element model for the vertical loads analysis

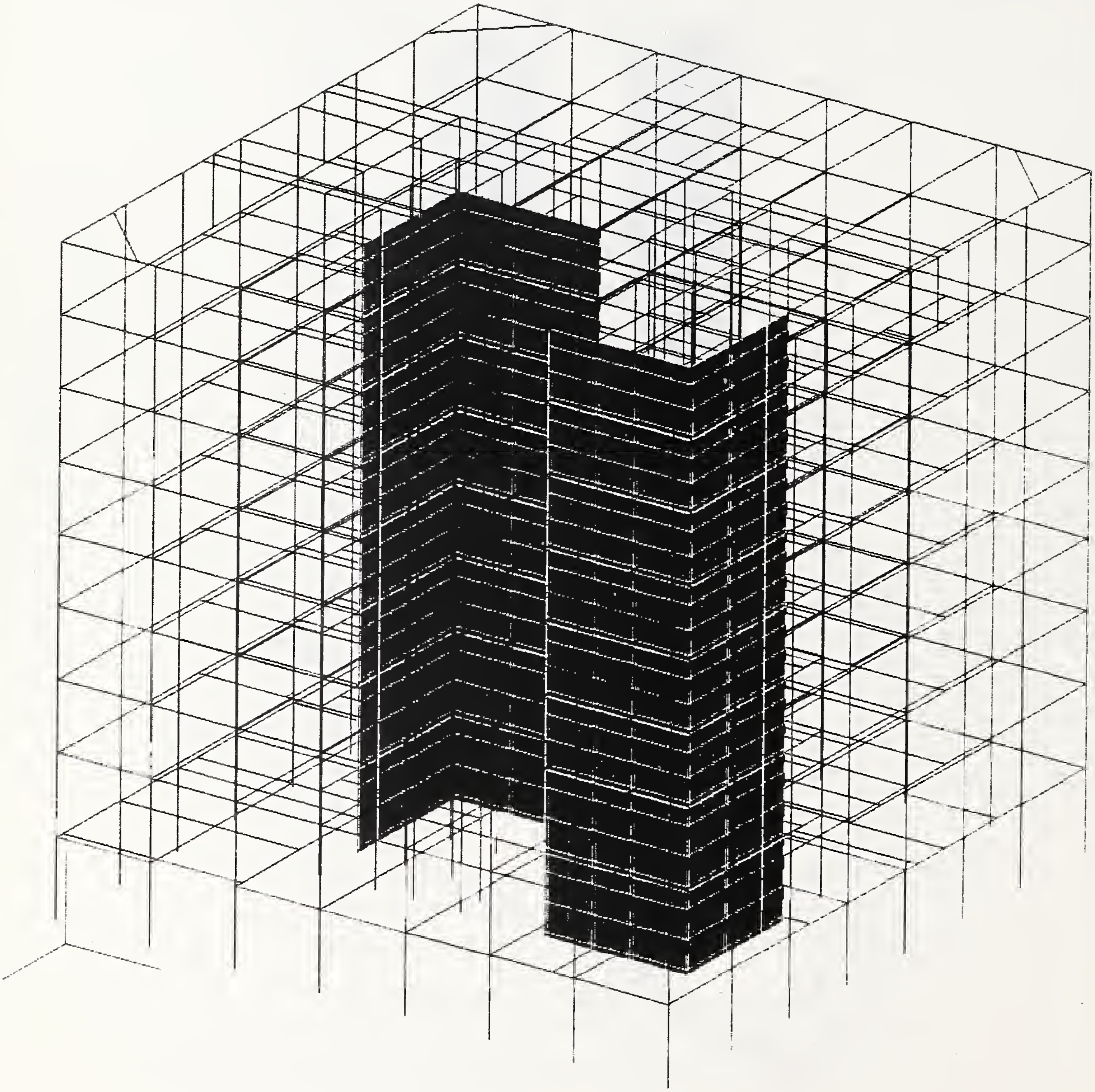


Figure 4.1.3. Isometric view of complete finite element model for the vertical loads analysis

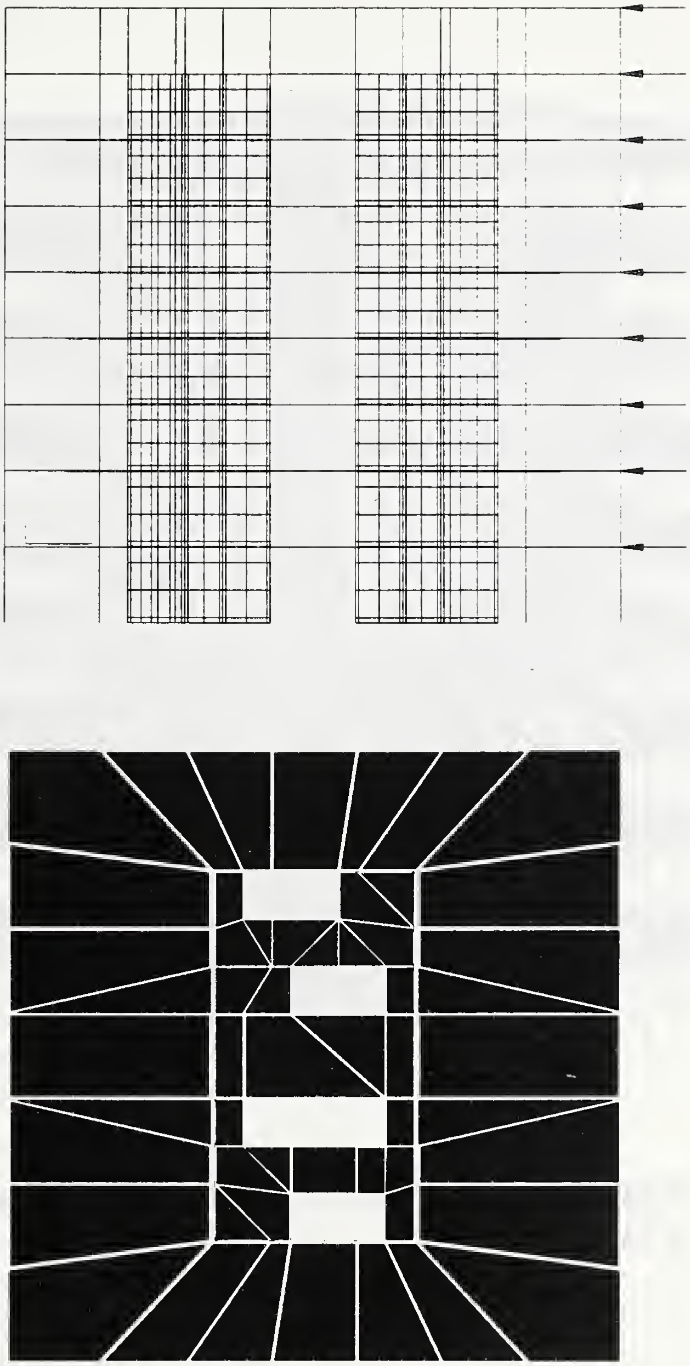


Figure 4.1.4. Profile and plan of the finite element model for the lateral load analysis

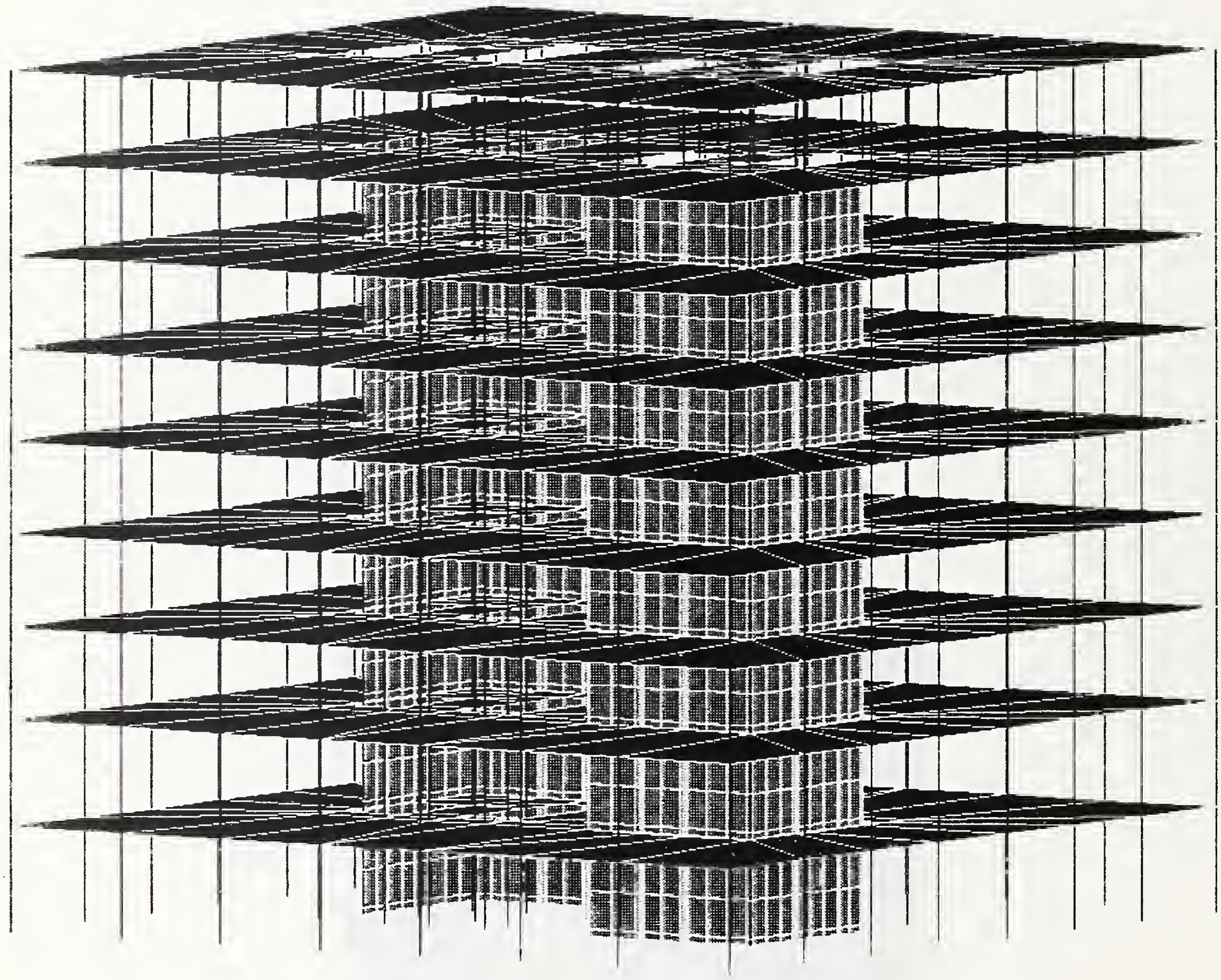


Figure 4.1.5. Isometric of finite element model for the lateral load analysis

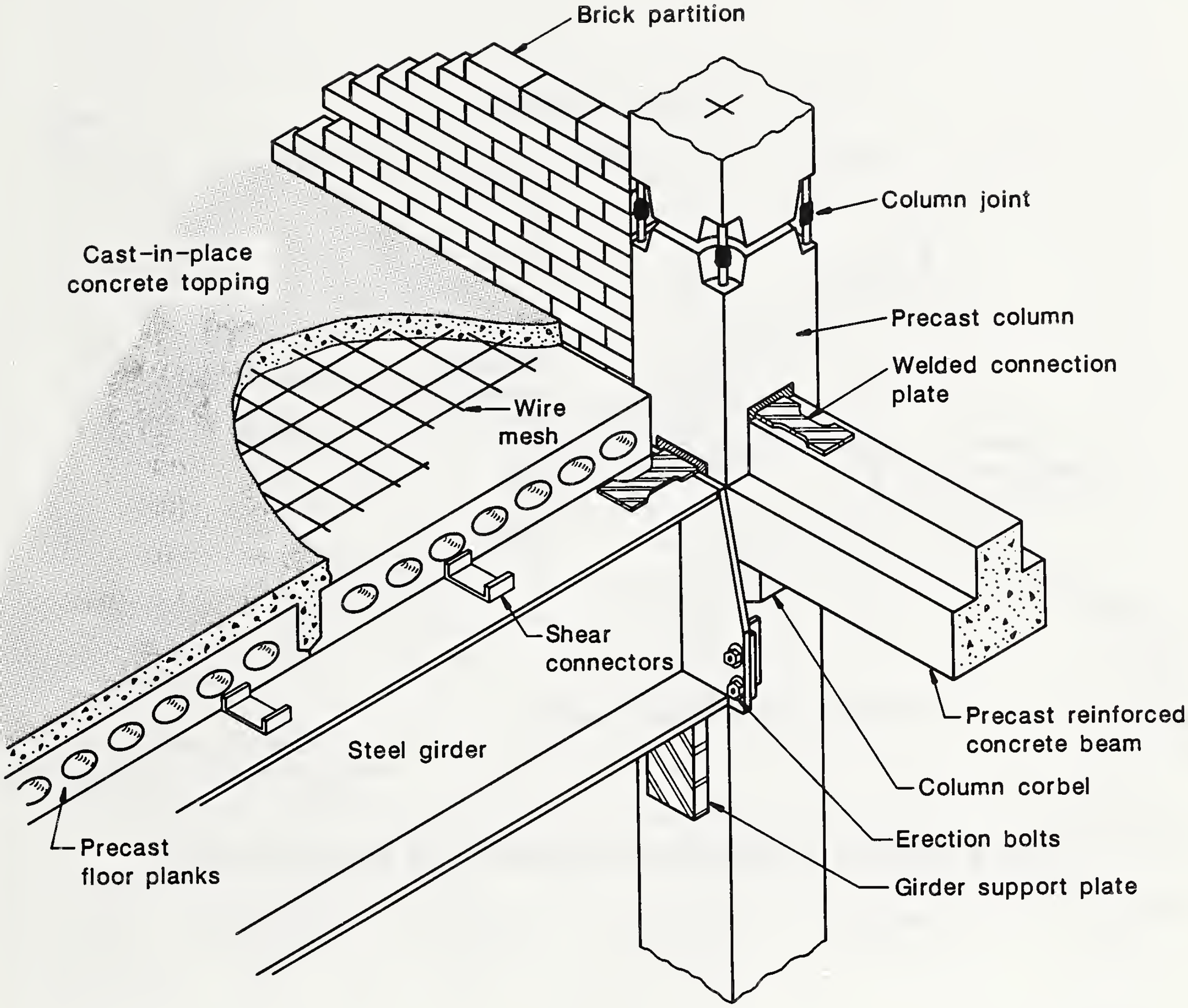


Figure 4.2.1. Schematic of structural system

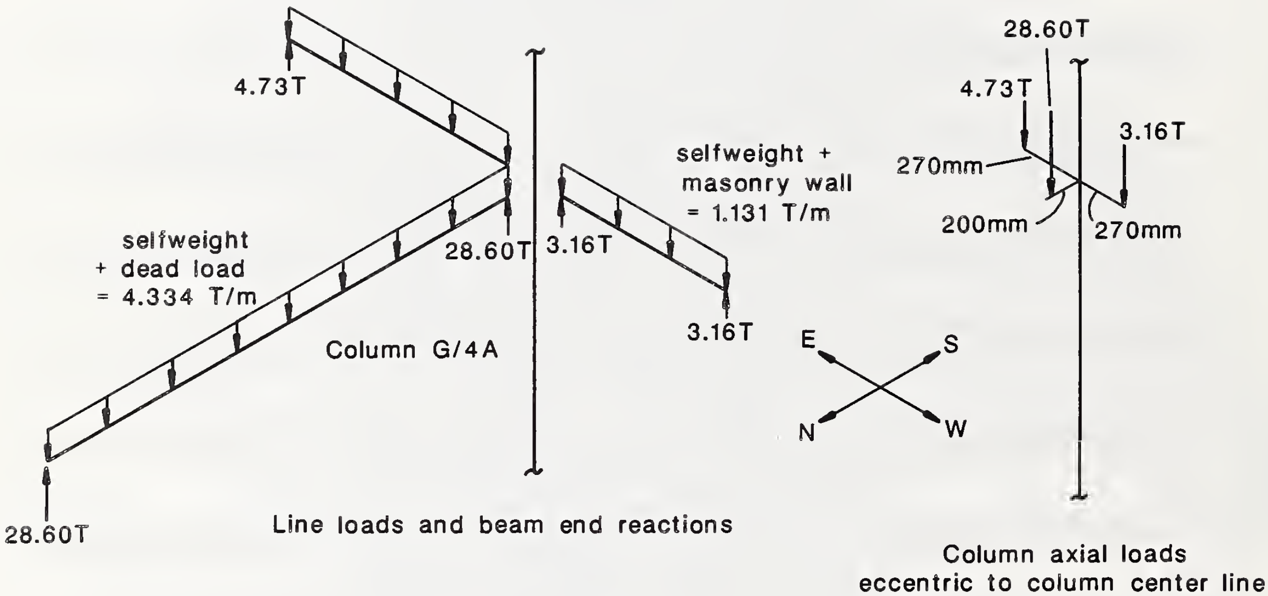


Figure 4.2.2. Schematic of beam and column loads

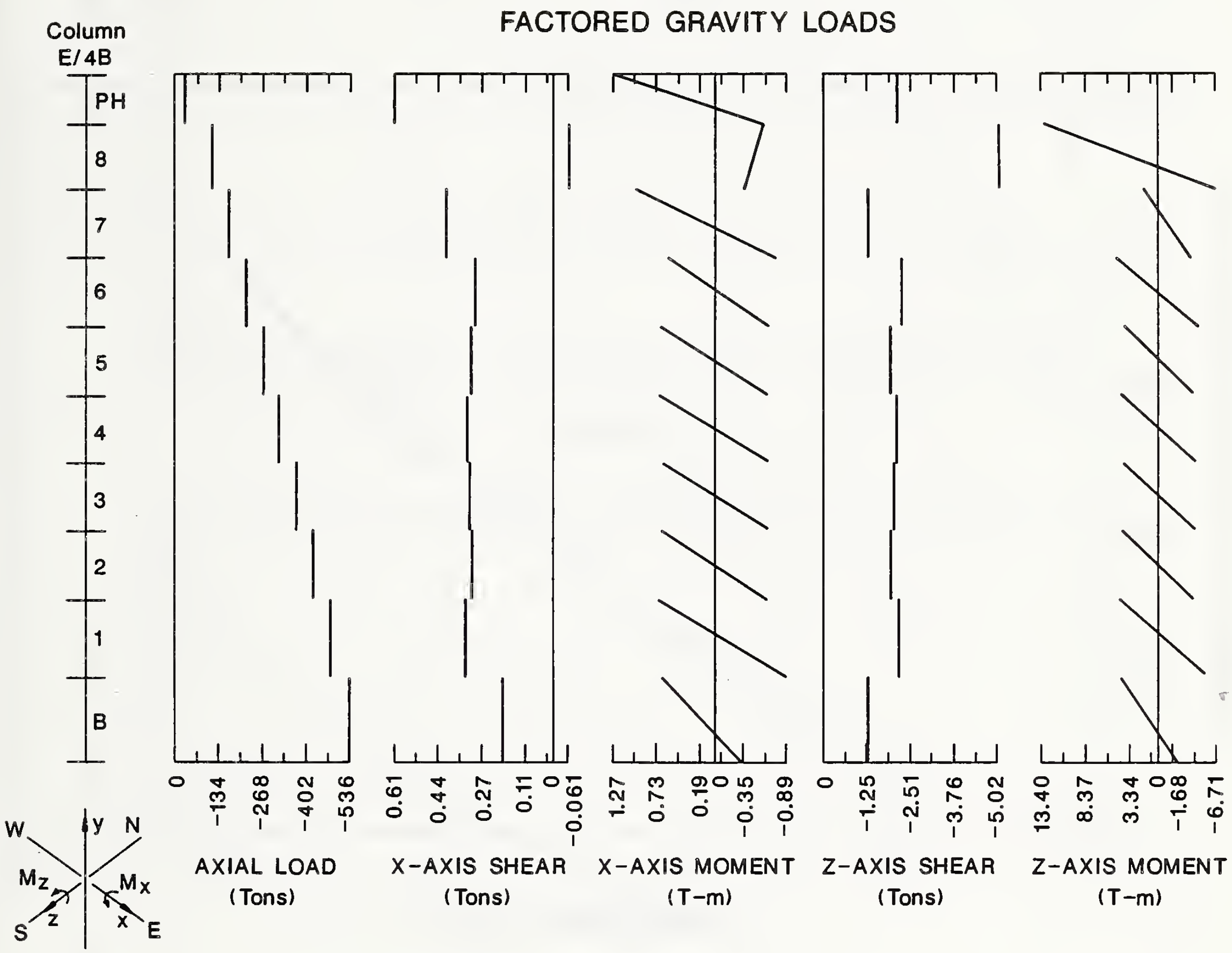


Figure 4.2.3. Axial loads, shear forces, and moments on column line E/4B

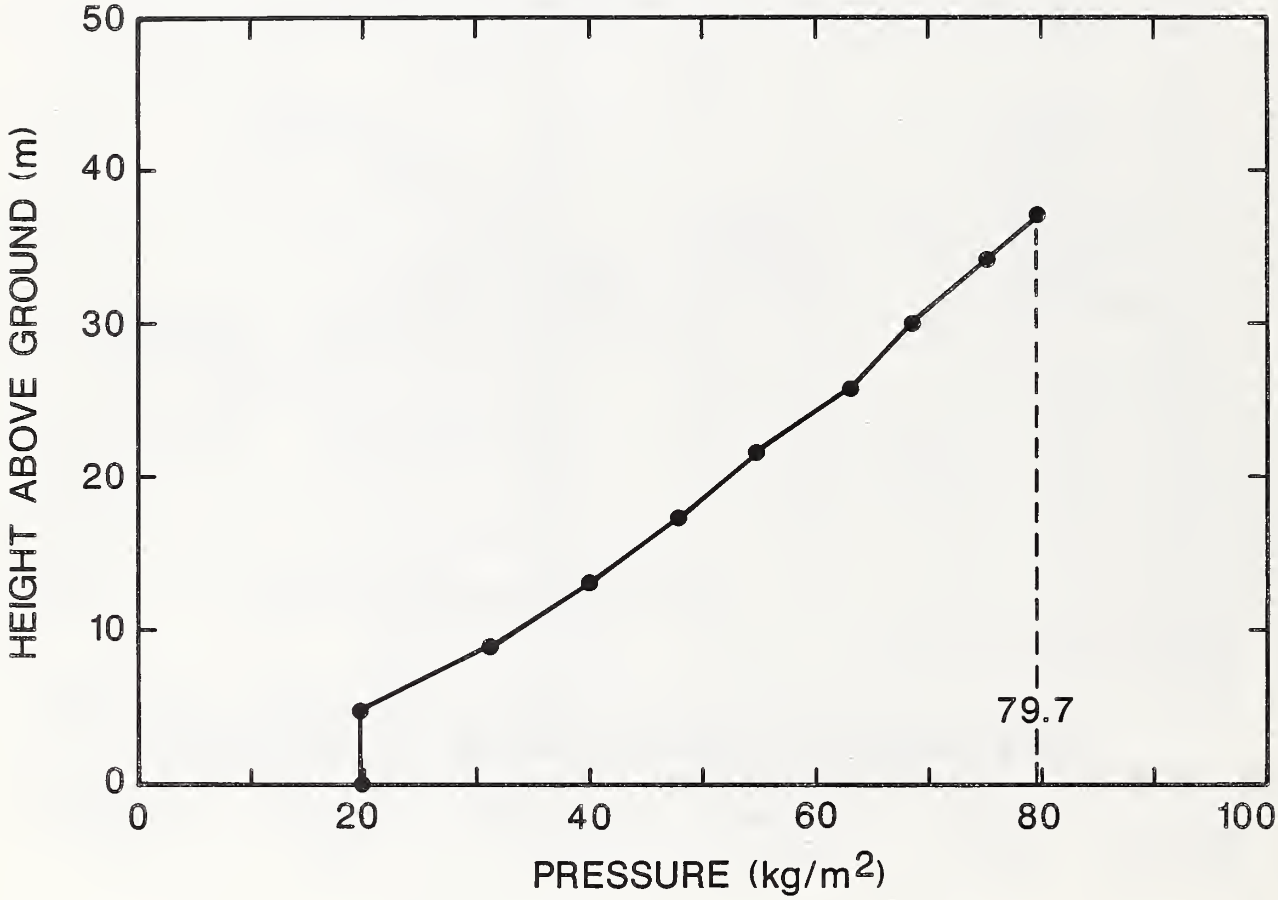


Figure 4.3.1. ANSI A.58.1-1982 wind pressure distribution for the Office Building

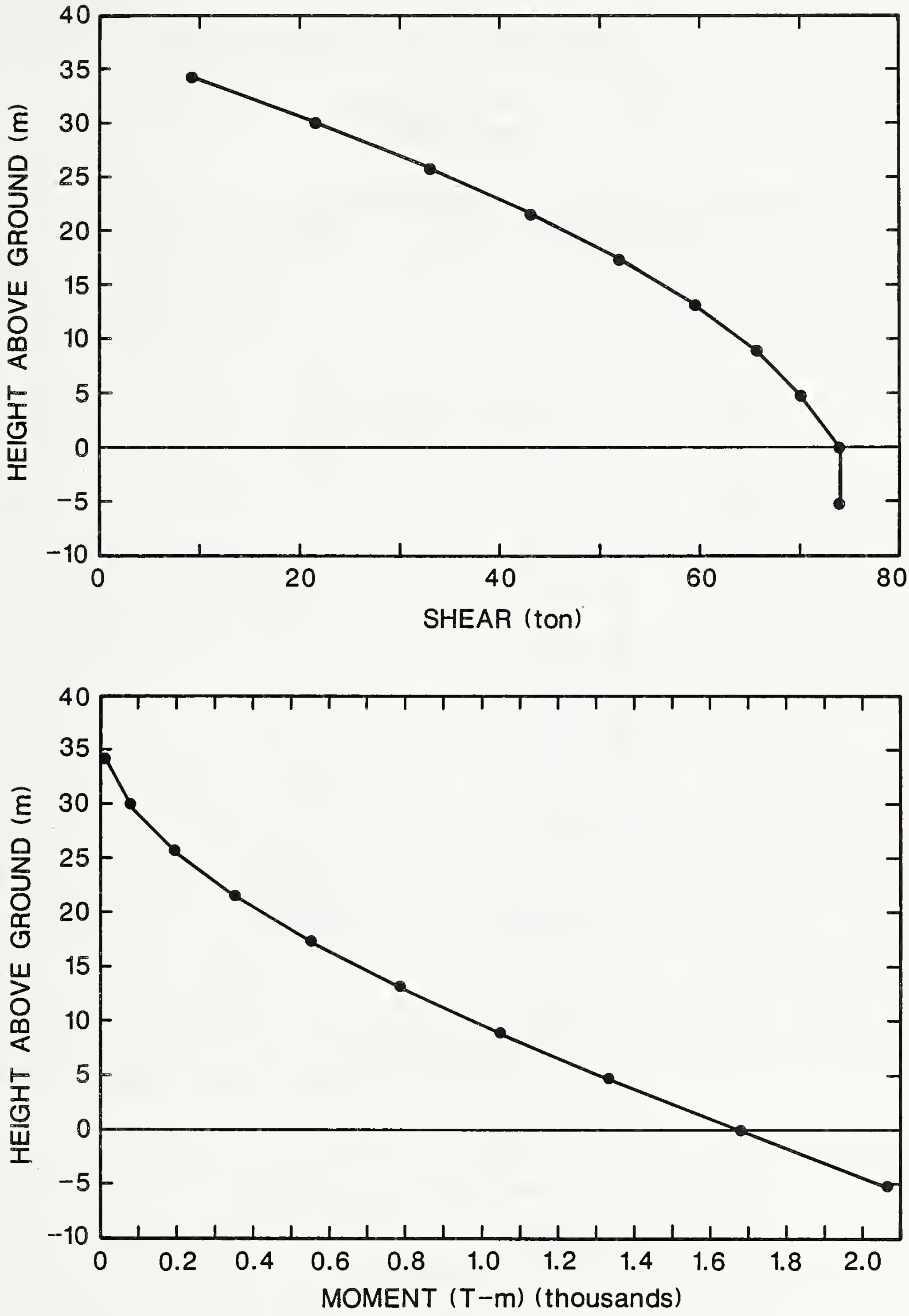
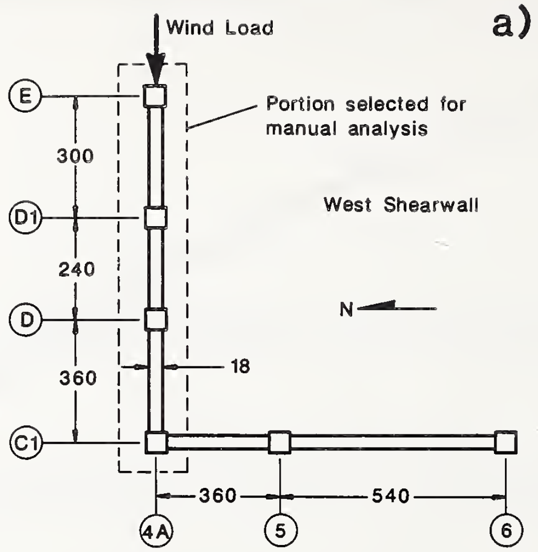


Figure 4.3.2. Shear force and overturning moment versus height above ground



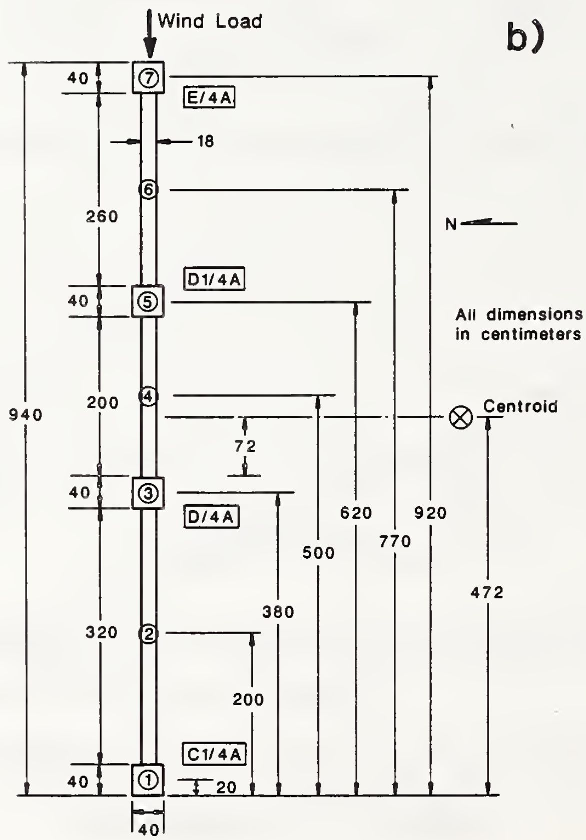
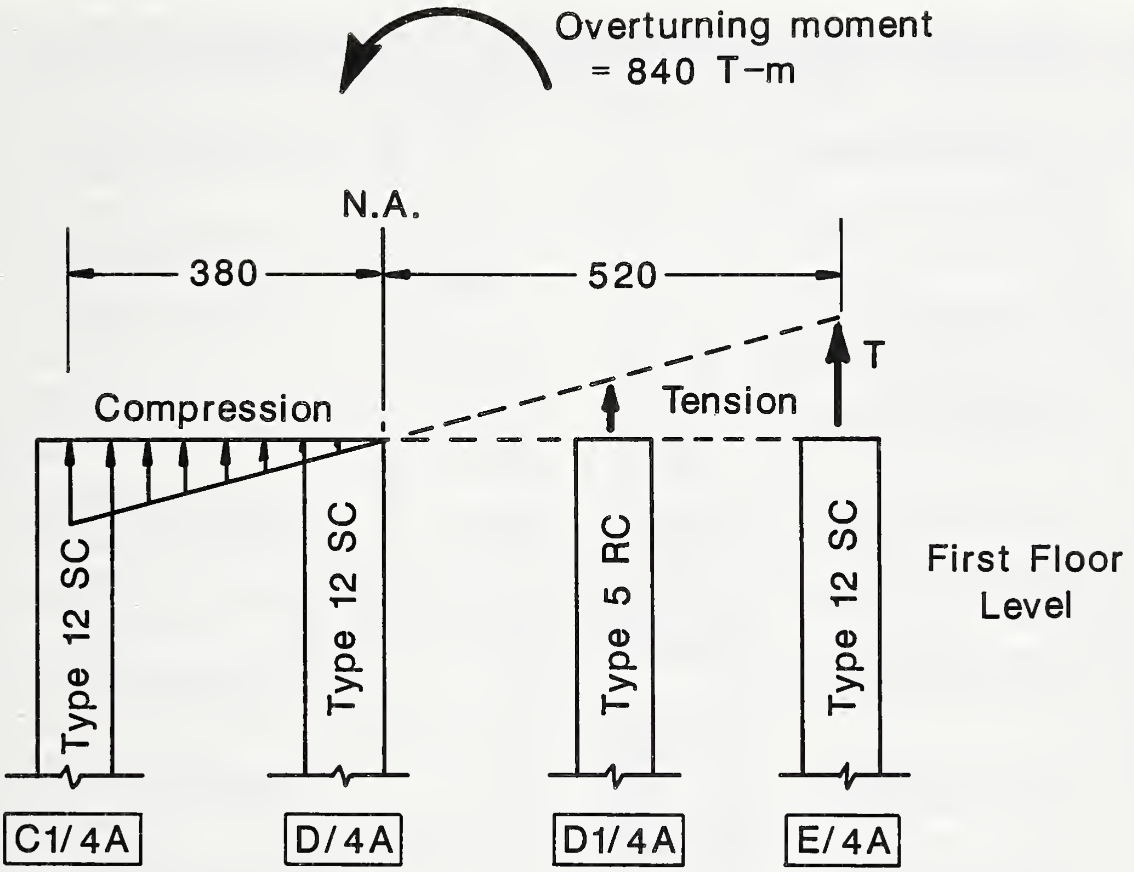


Figure 4.3.3 a) Plan of west shear wall; and, b) cross-section of analyzed portion



RC = Reinforced concrete column
SC = Steel core concrete column

Figure 4.3.4. Overturning moment on first floor horizontal shear wall joint

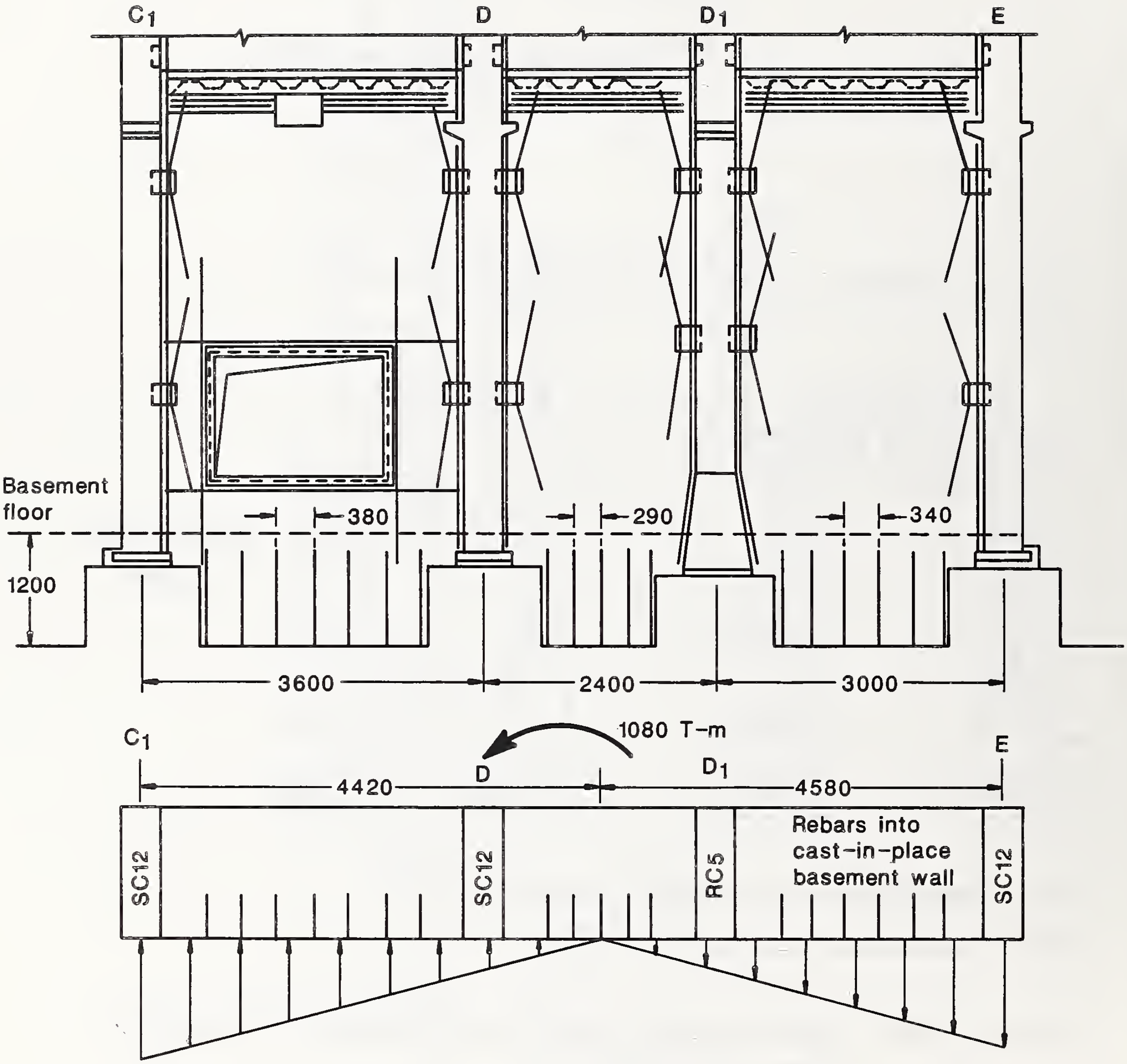


Figure 4.3.5. a) Cast-in-place shear wall in basement story; and b) overturning moment at pile cap elevation

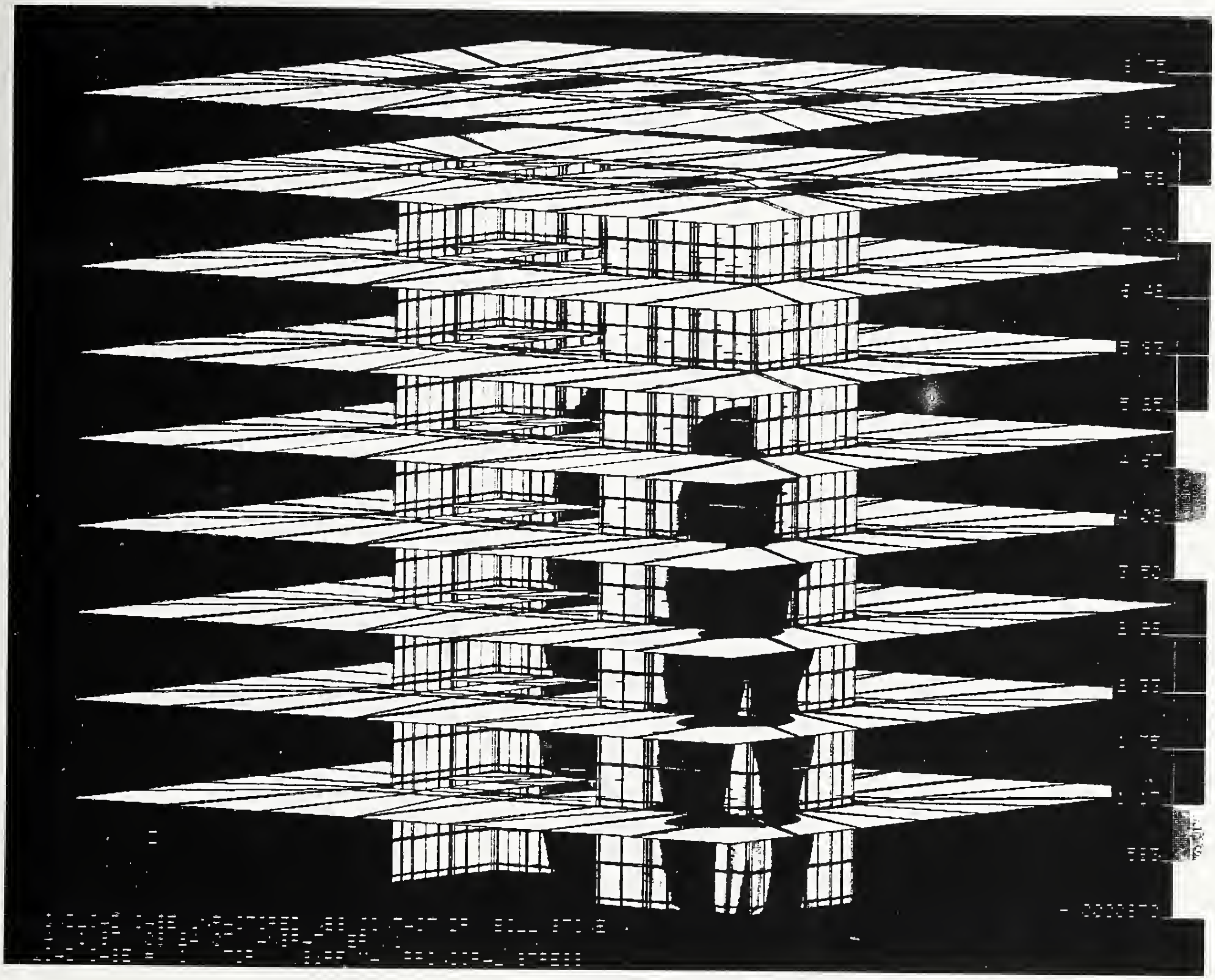


Figure 4.3.6. Principal tensile stress contours on shear walls (view from southeast corner, load from east)

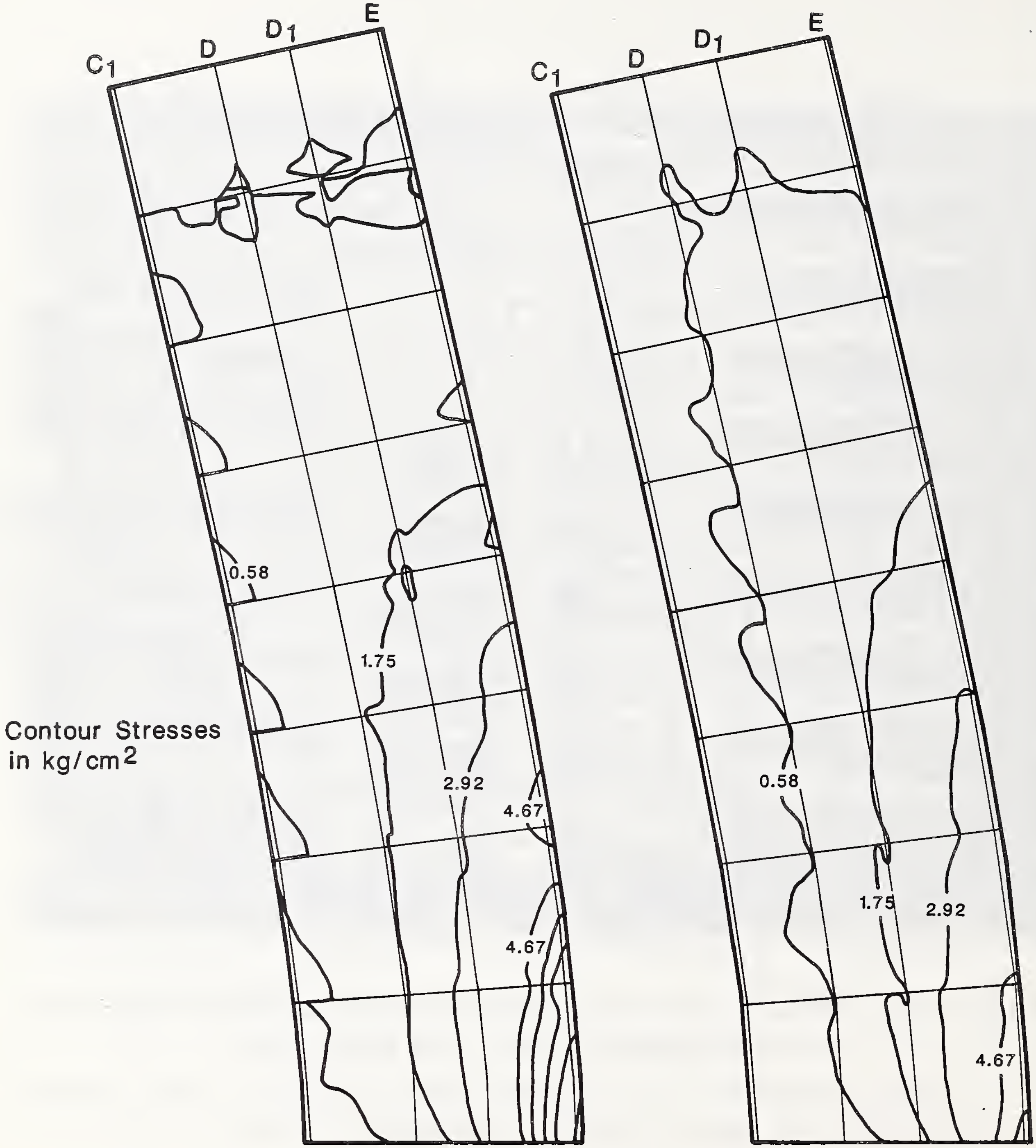


Figure 4.3.7. Principal tensile stress contours and deflected shape of shear wall system when joints are filled

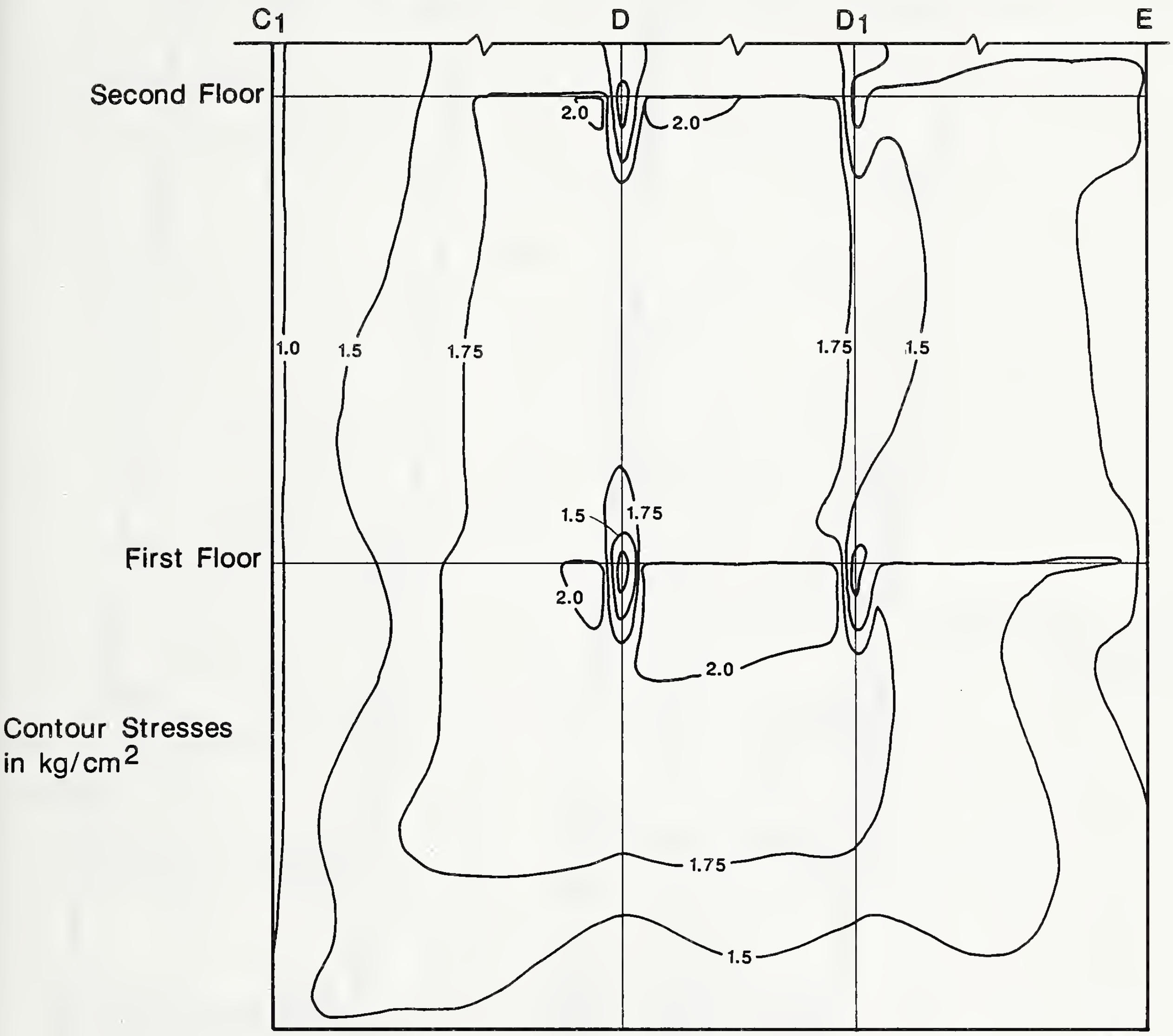


Figure 4.3.8. Shear stress contours in basement and first story shear walls between column lines E/4A and C1/4A

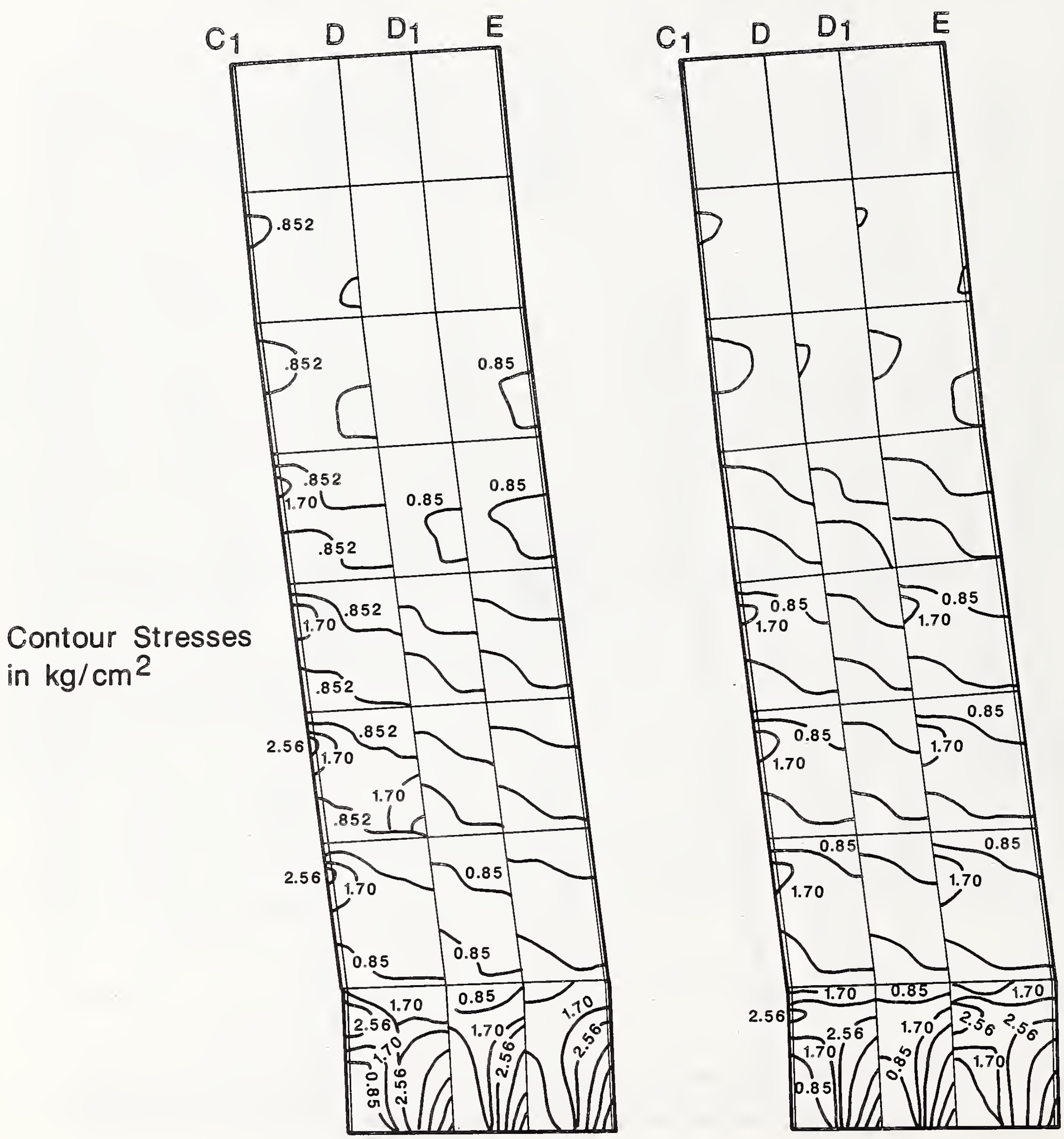


Figure 4.3.9. Principal tensile stress contours and deflected shape of shear wall system when joints are unfilled

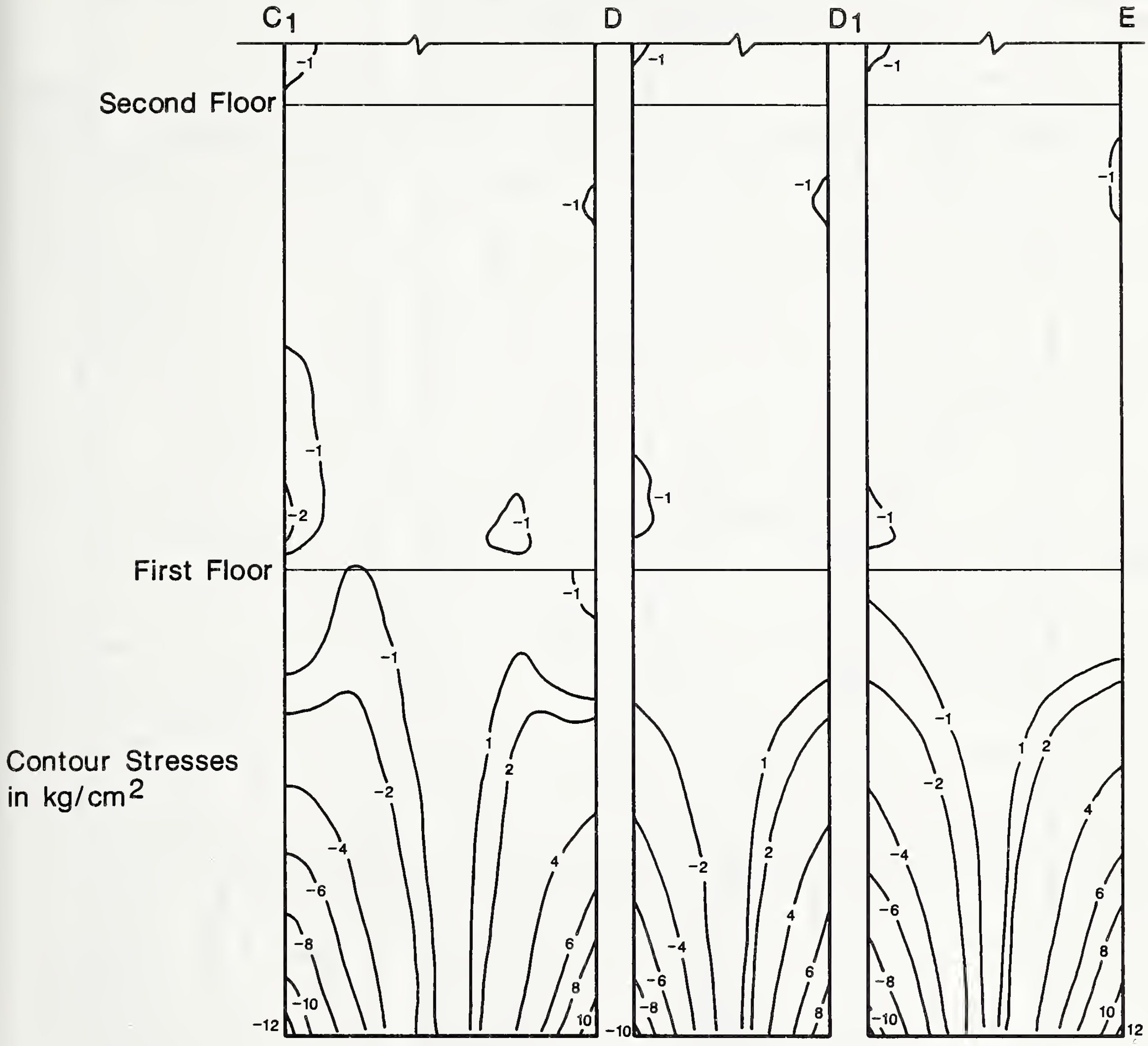


Figure 4.3.10. Vertical stress contours in basement and first story shear walls between column lines E/4A and C1/4A; vertical and horizontal joints are unfilled

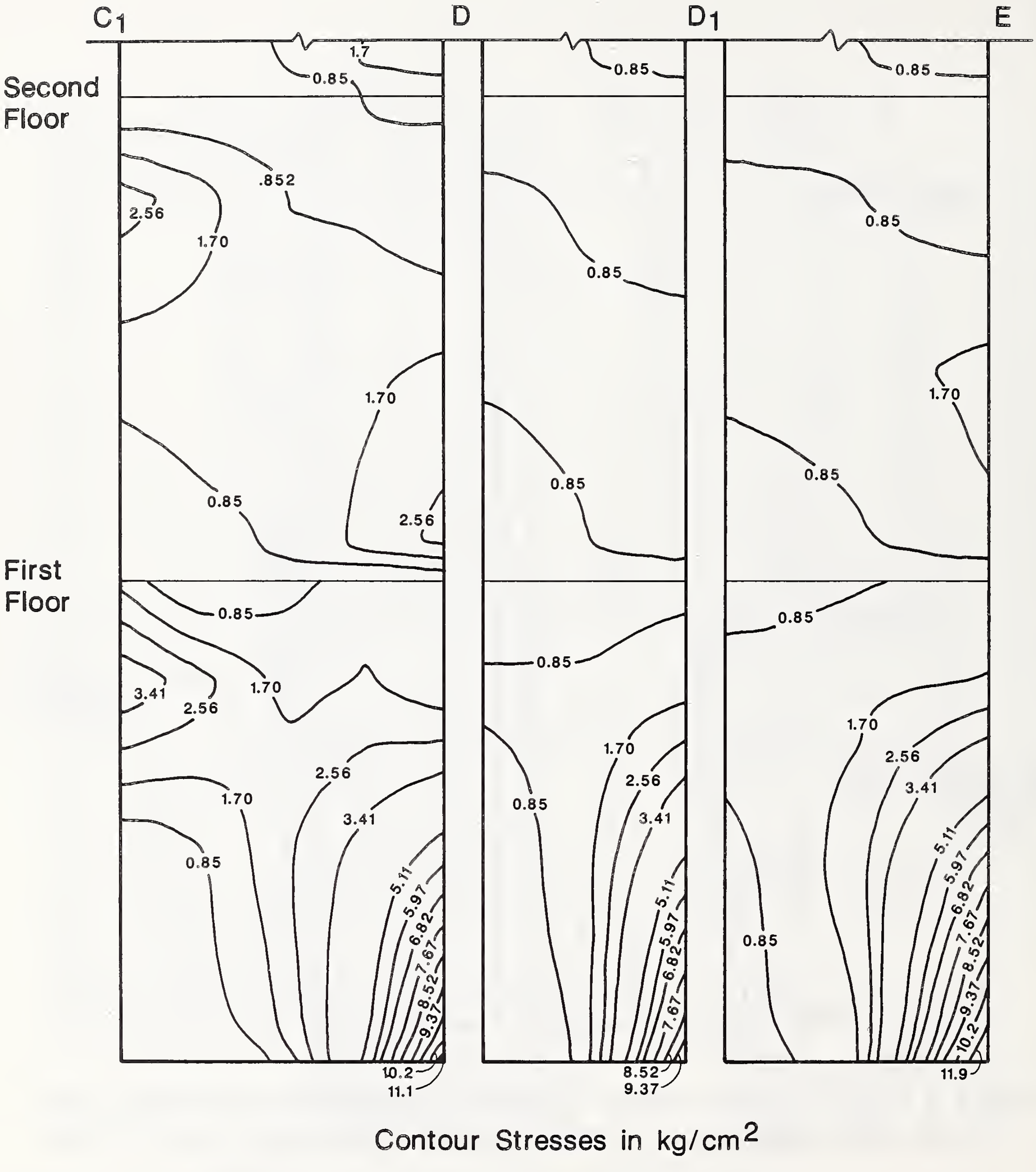


Figure 4.3.11. Principal tensile stress contours in basement and first story shear walls between column lines E/4A and C1/4A; unfilled joints

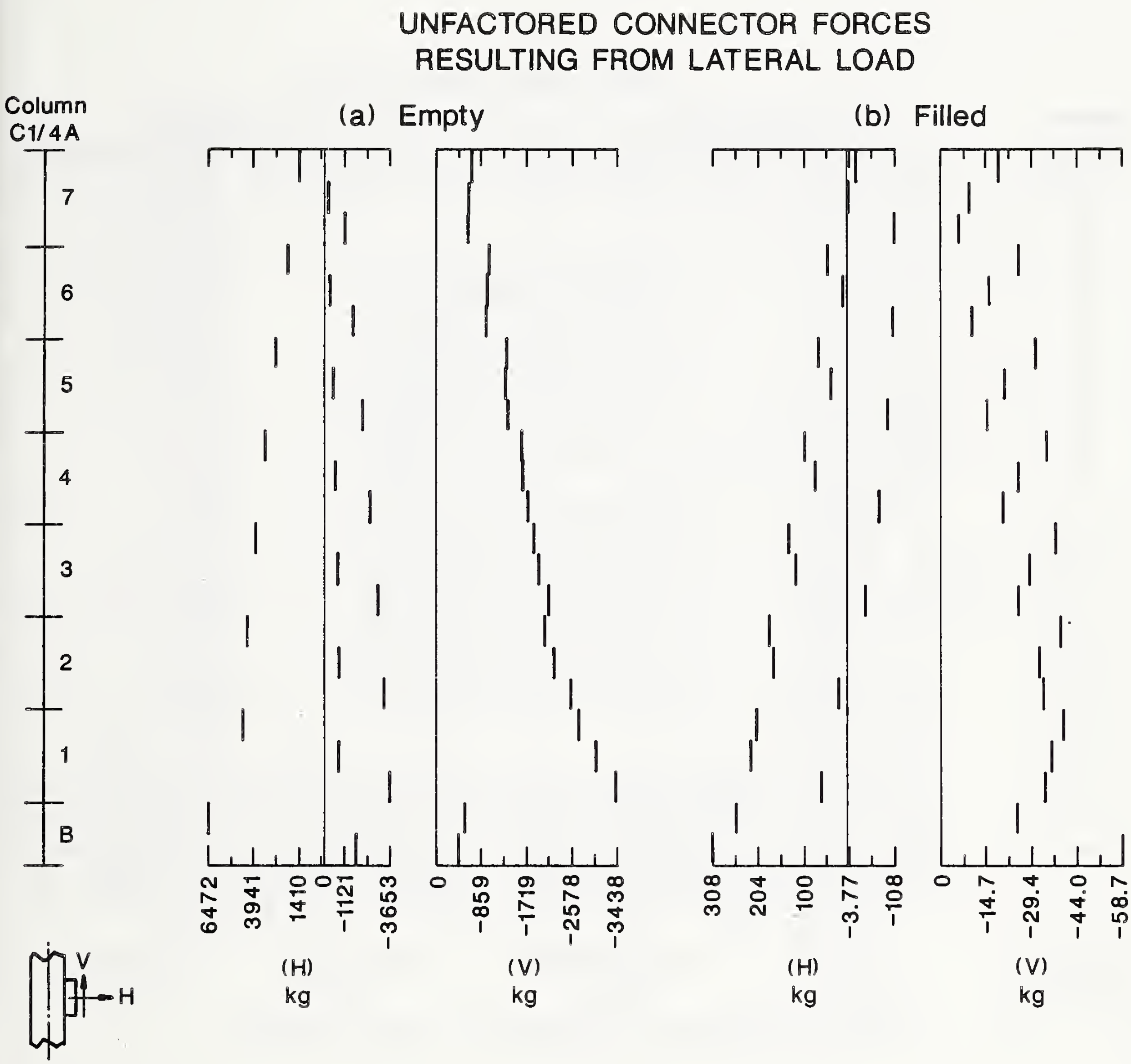


Figure 4.3.12. Horizontal and vertical forces in shear wall-to-column connectors at column line C1/4A for the cases of; a) empty joints: and b) filled joints

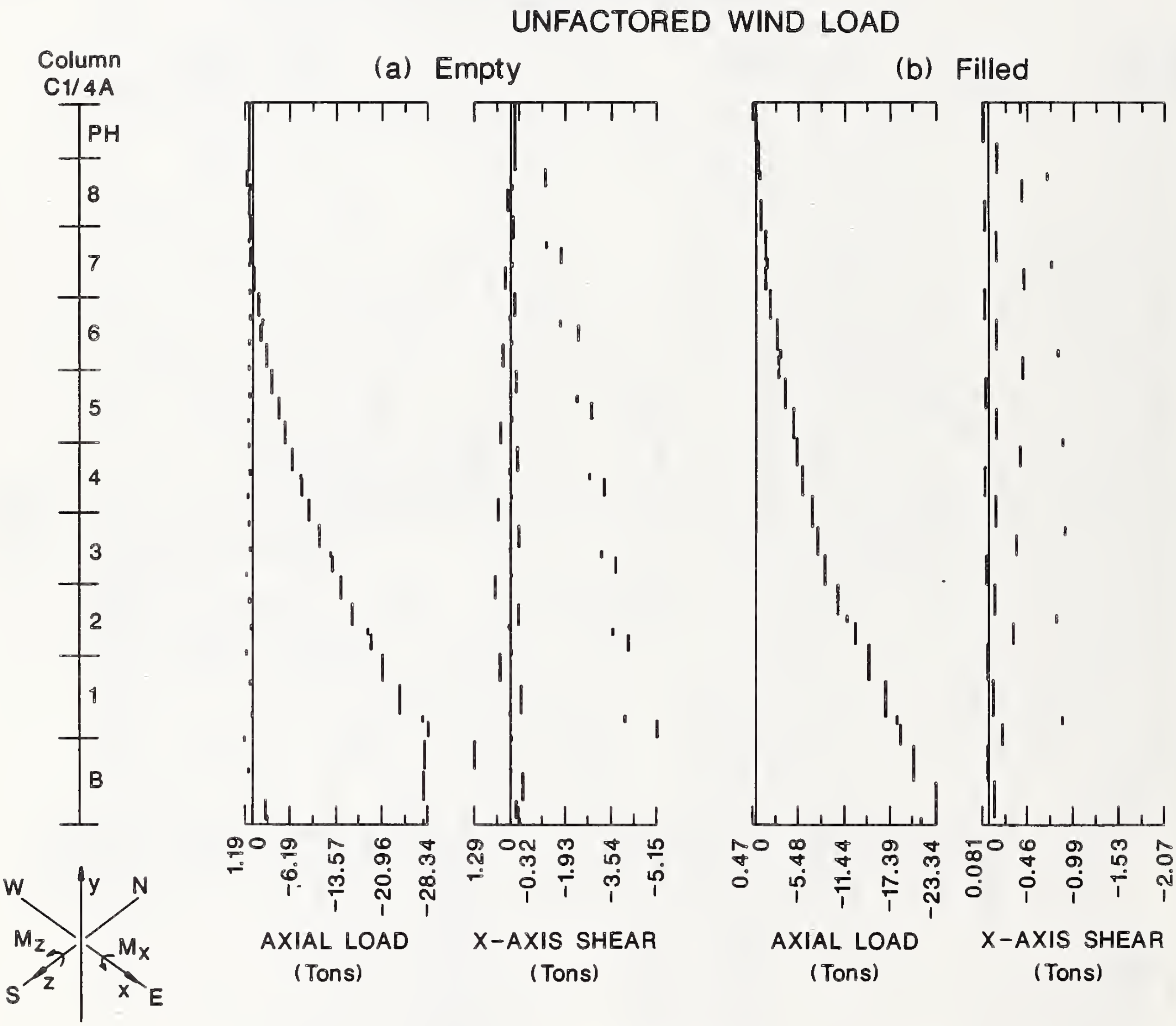


Figure 4.3.13. Axial and shear forces for column lines C1/4A: a) empty joints; and b) filled joints

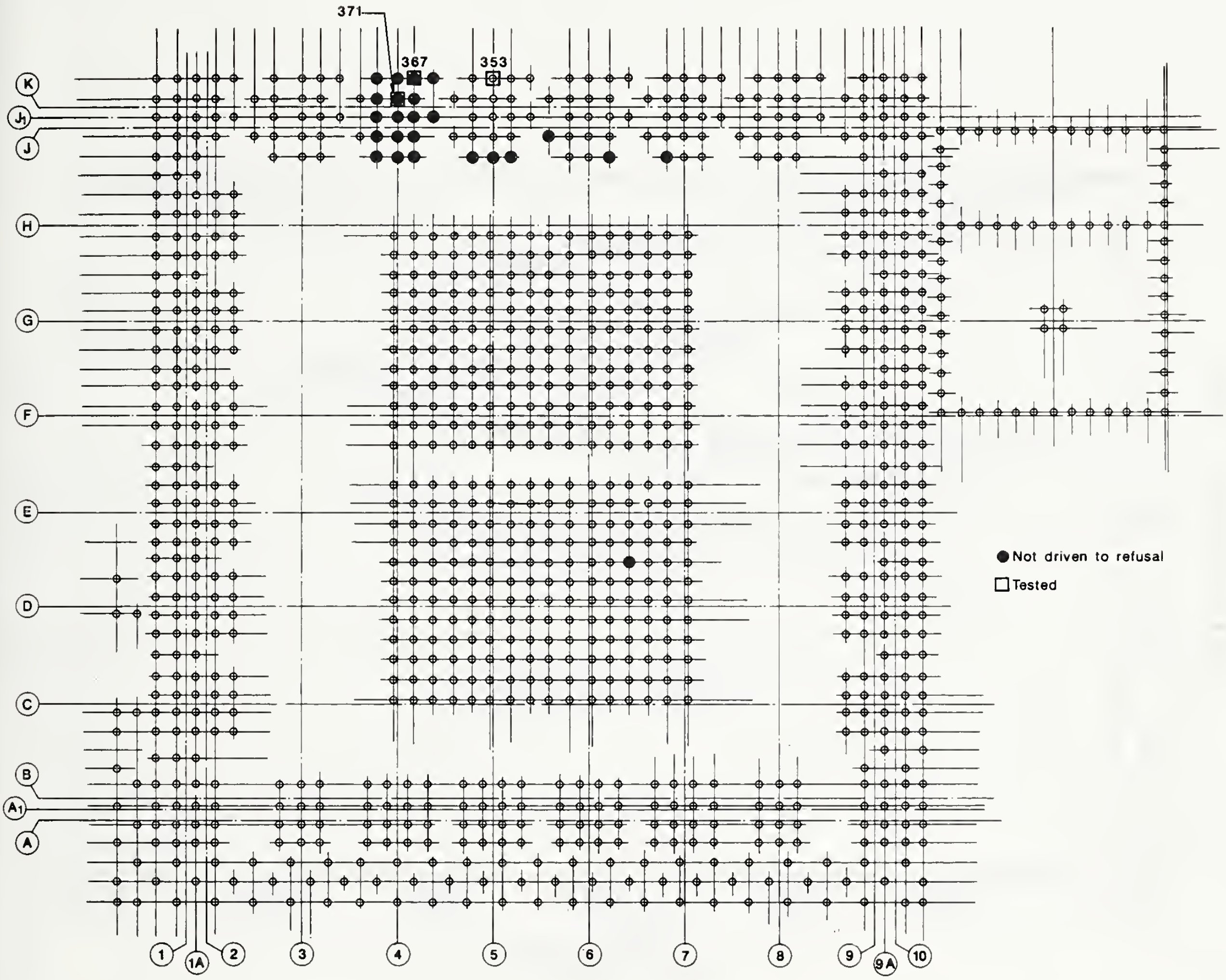


Figure 4.4.1. Layout of foundation piles

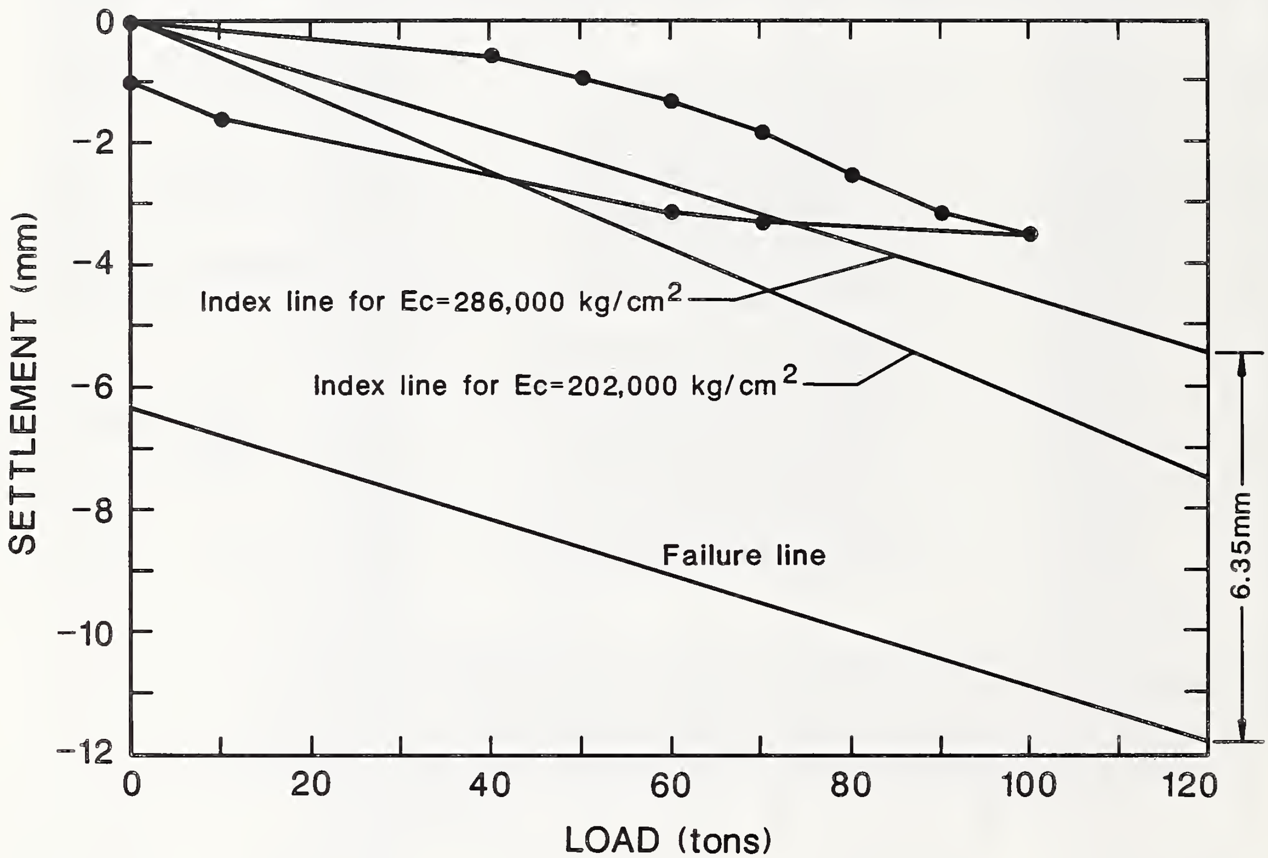


Figure 4.4.2. Static load test on pile No. 371

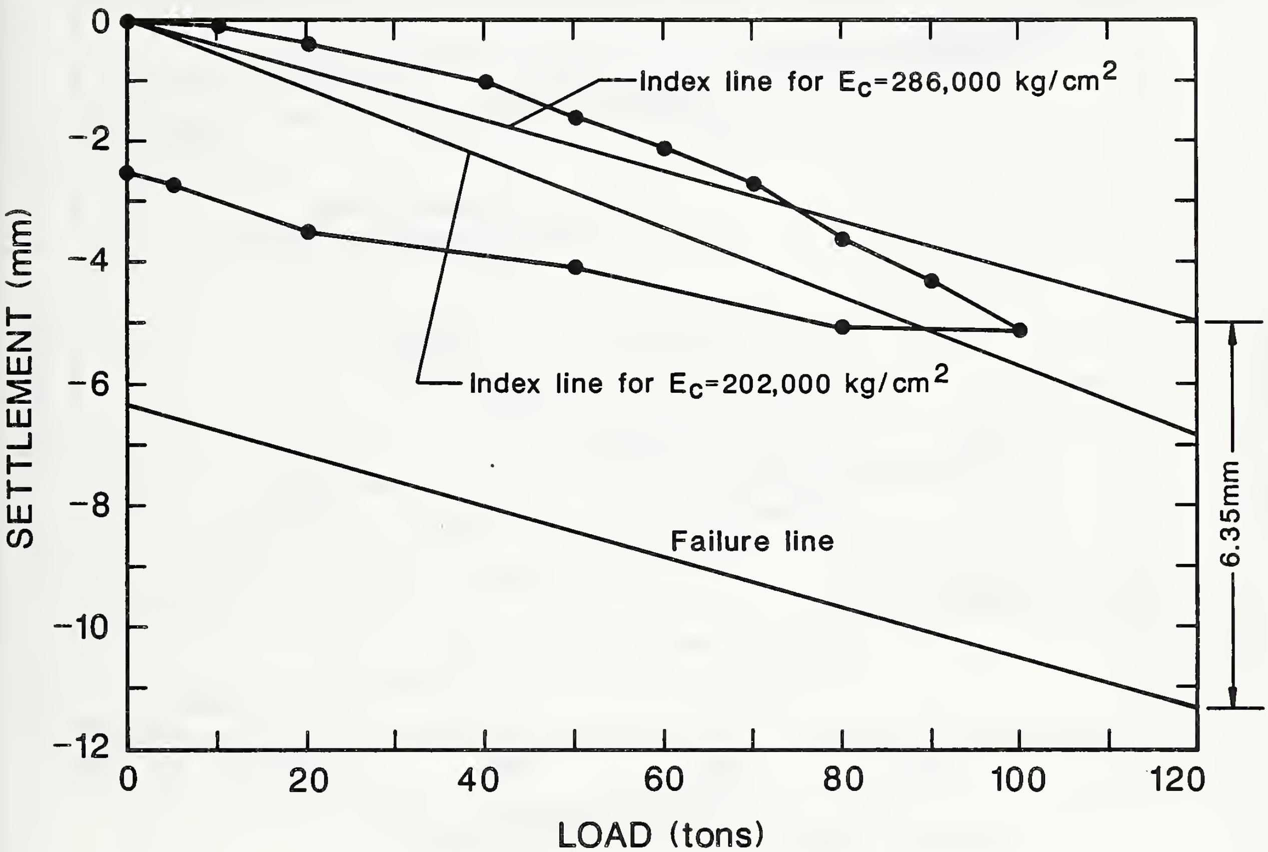


Figure 4.4.3. Static load test on pile No. 367

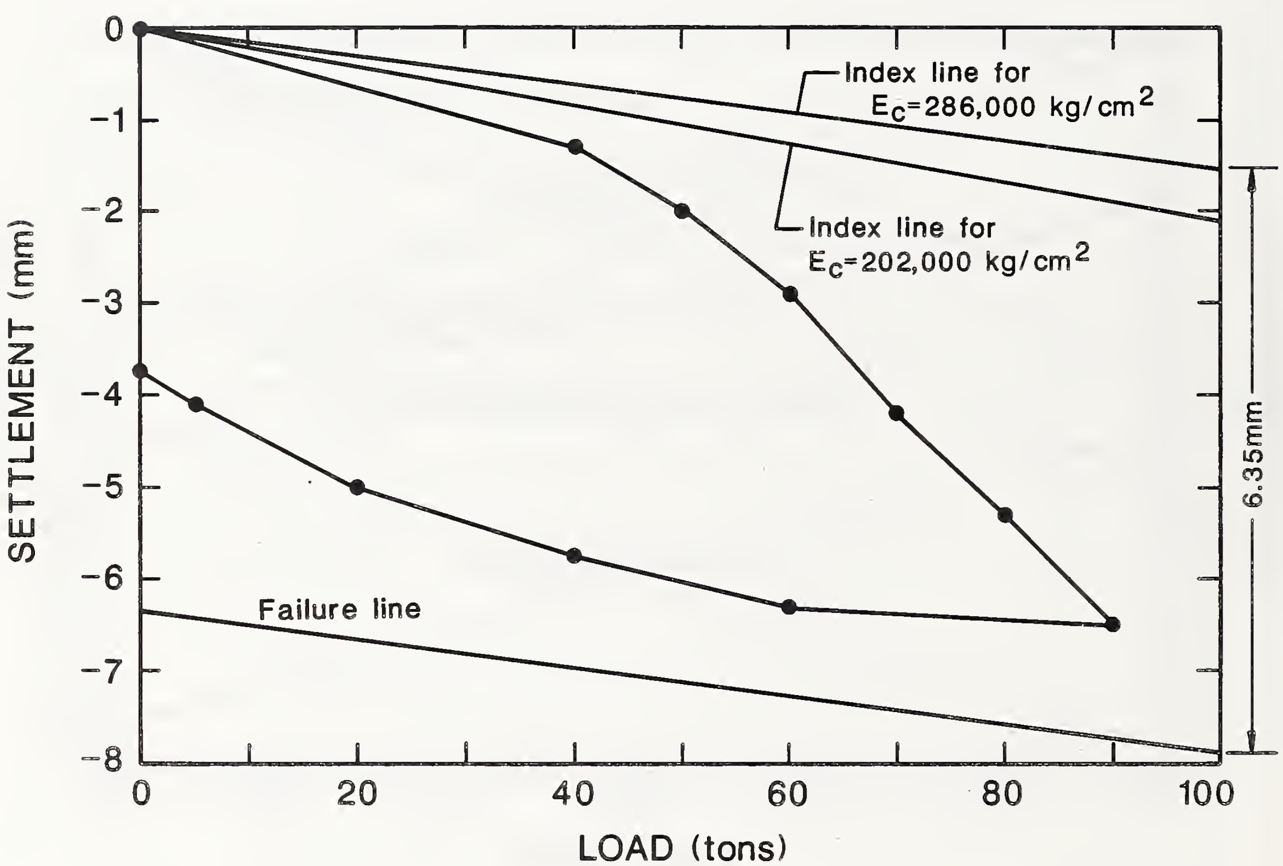


Figure 4.4.4. Static load test on pile No. 353

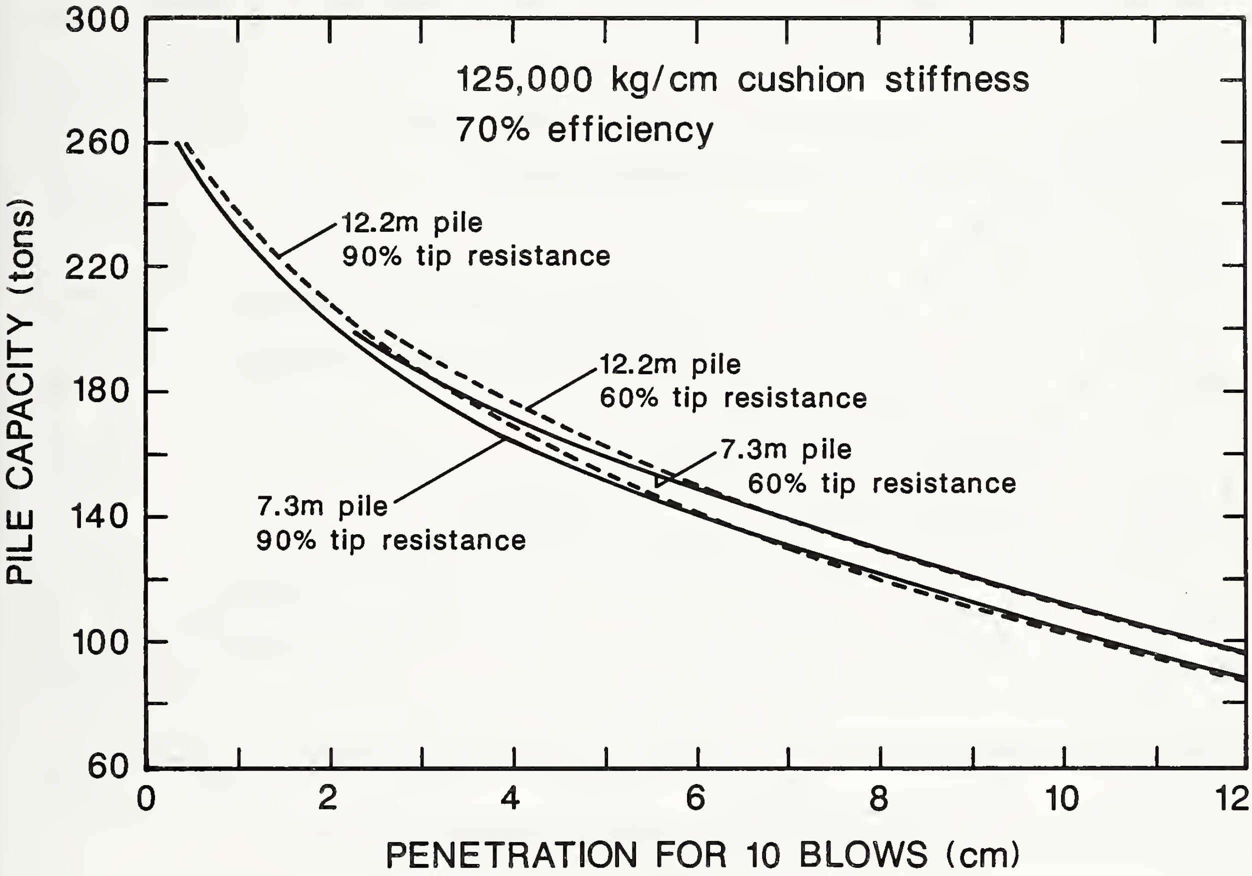


Figure 4.4.5. Effect of pile length and percent of tip resistance on load capacity

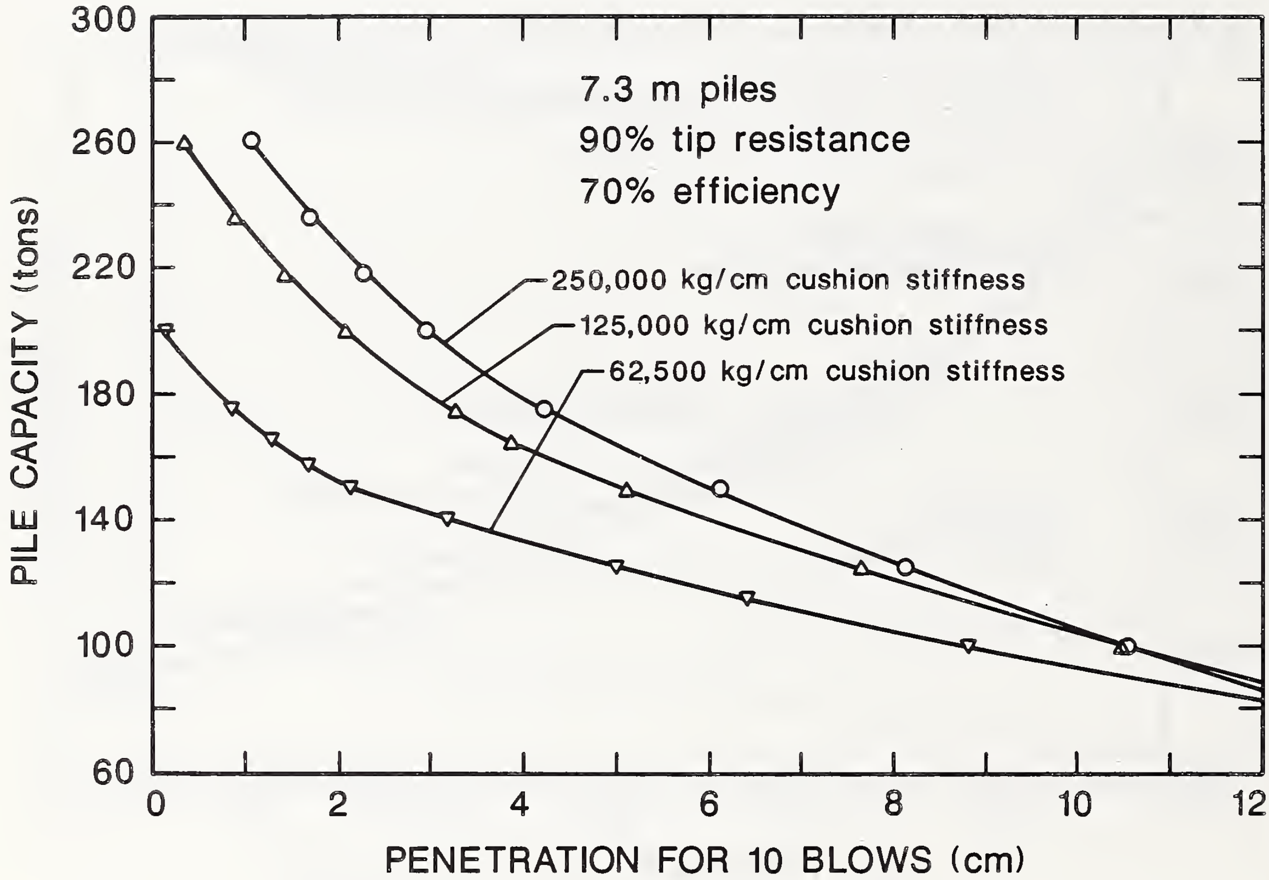


Figure 4.4.6. Effect of cushion thickness on pile capacity

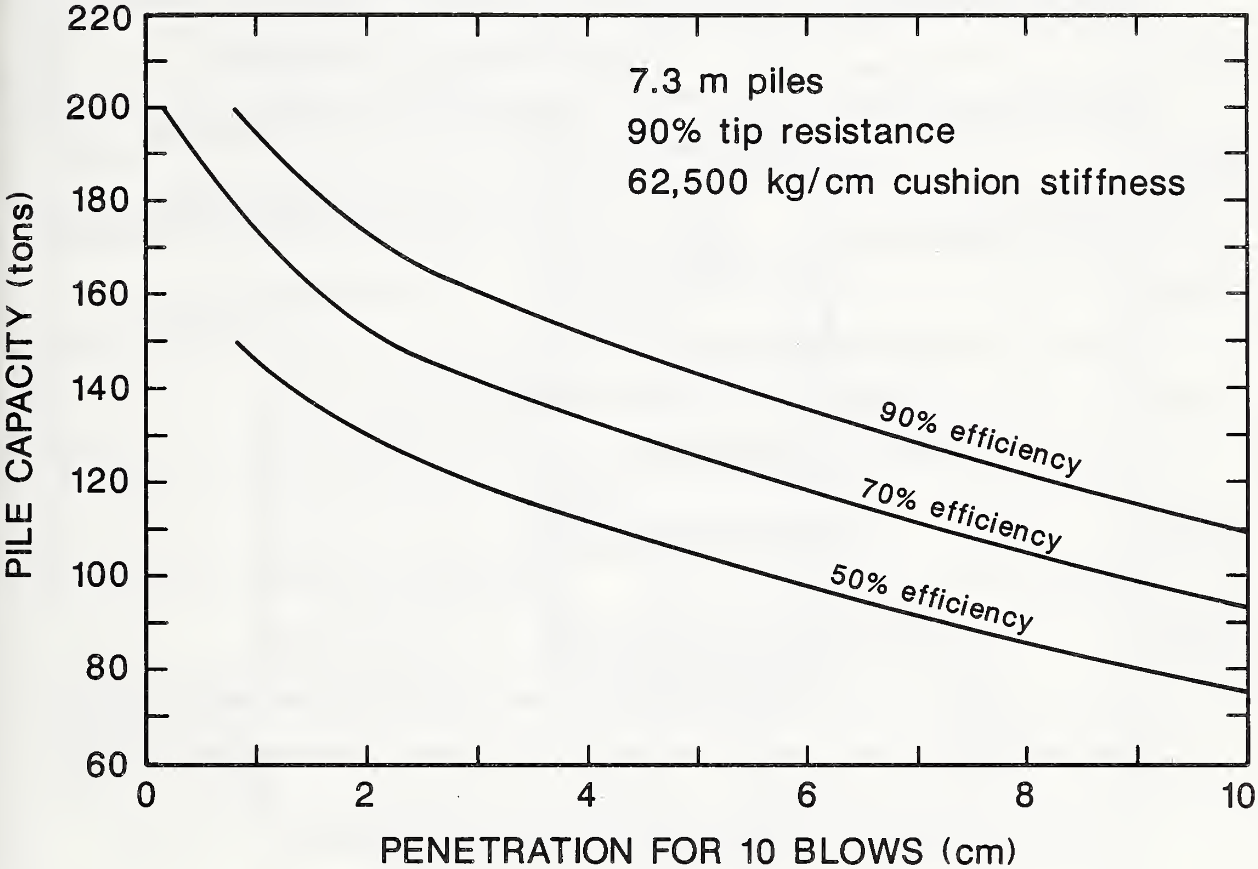


Figure 4.4.7. Effect of energy transmission efficiency on pile capacity

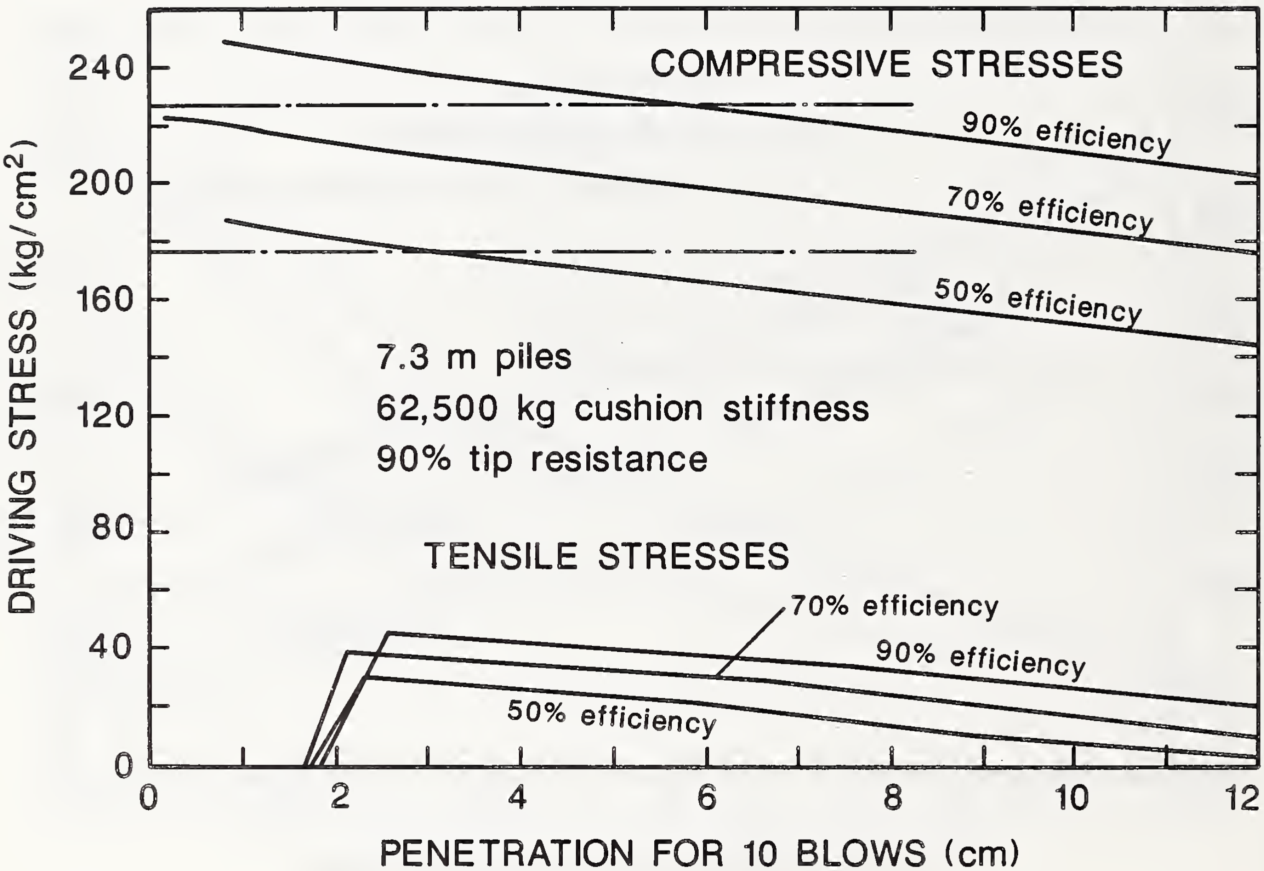


Figure 4.4.8. Estimated driving stresses

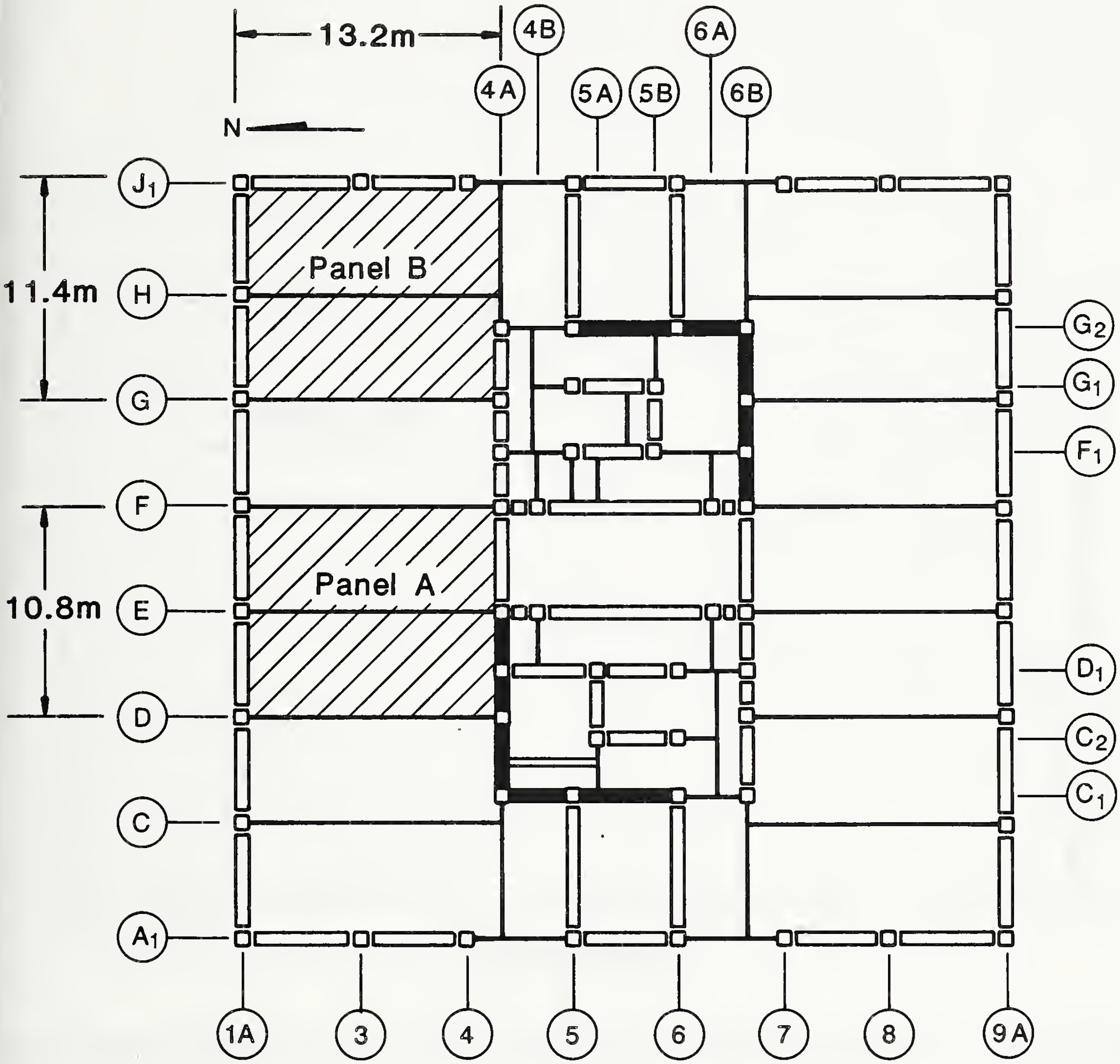


Figure 4.5.1. Plan of typical floor showing Panels A and B

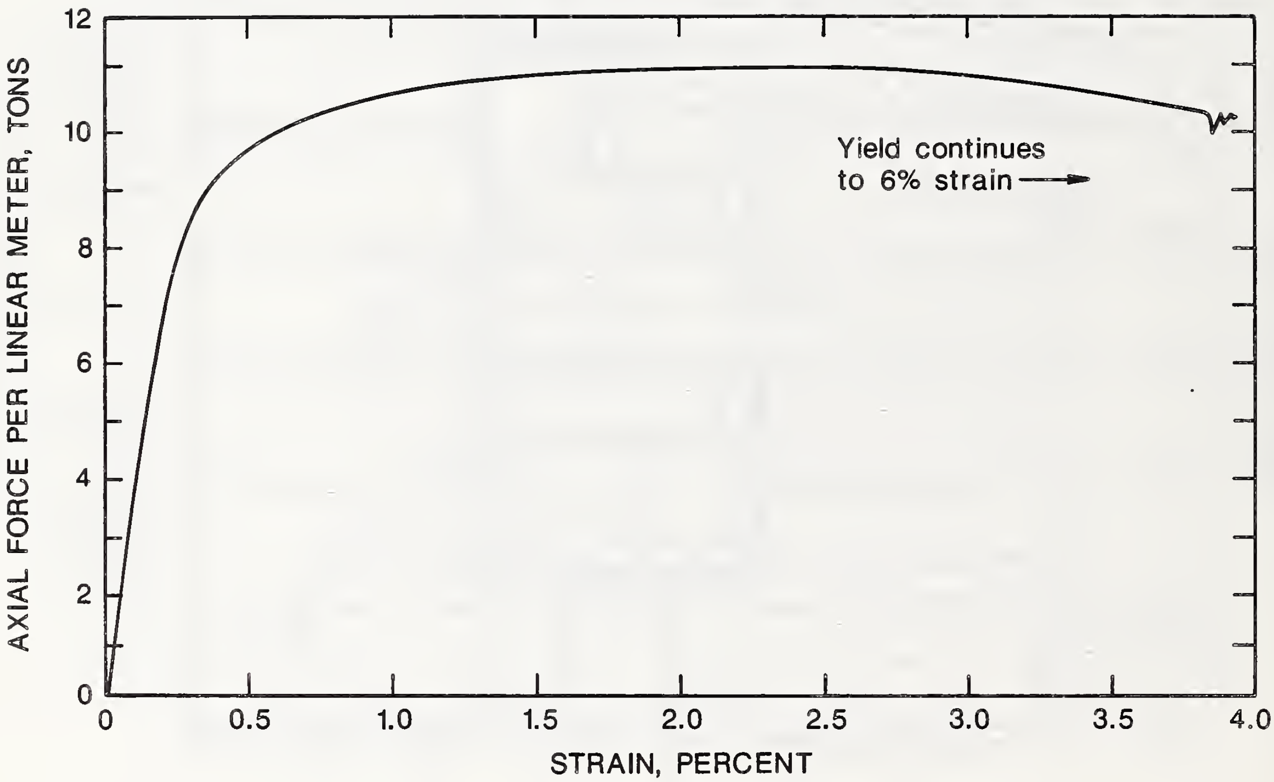


Figure 4.5.2. Load-deformation characteristics of the welded wire mesh

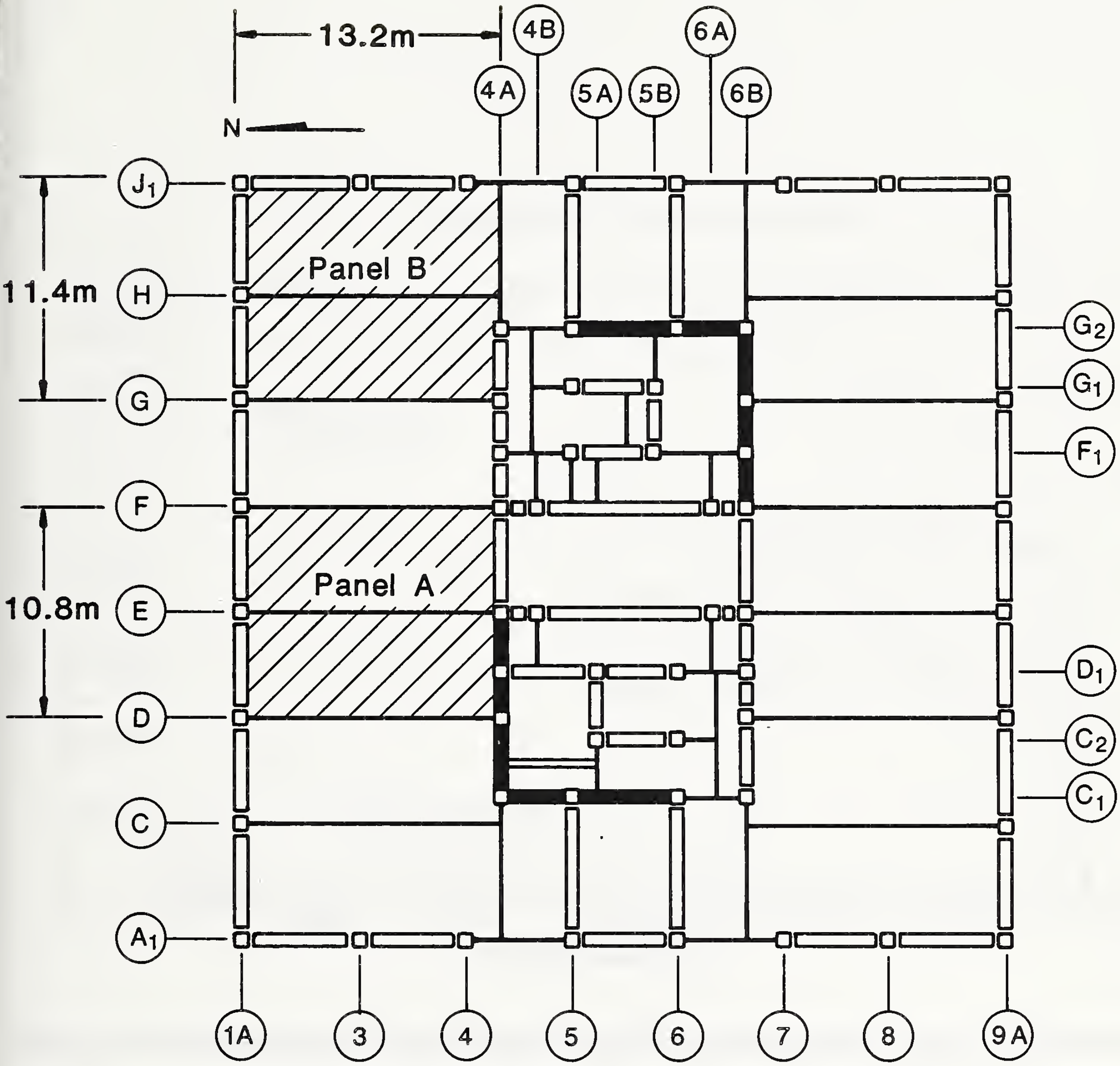


Figure 4.5.1. Plan of typical floor showing Panels A and B

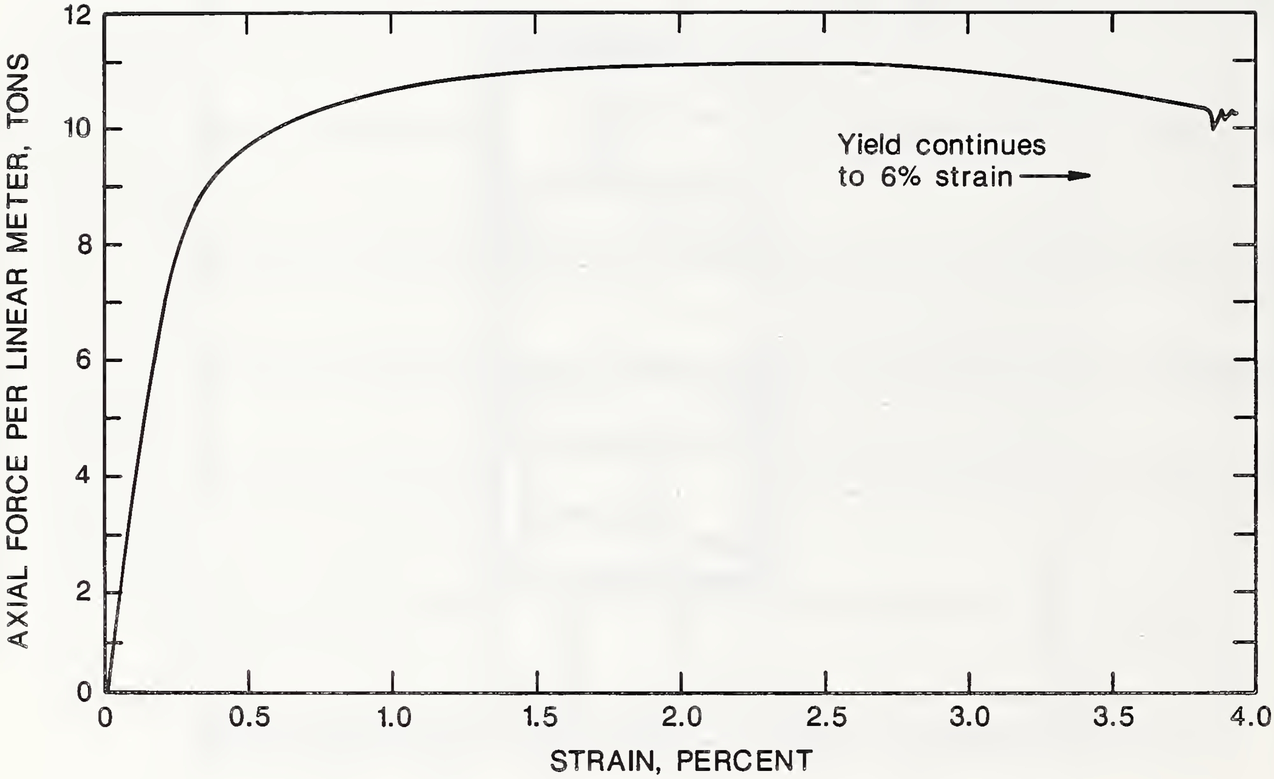


Figure 4:5.2. Load-deformation characteristics of the welded wire mesh

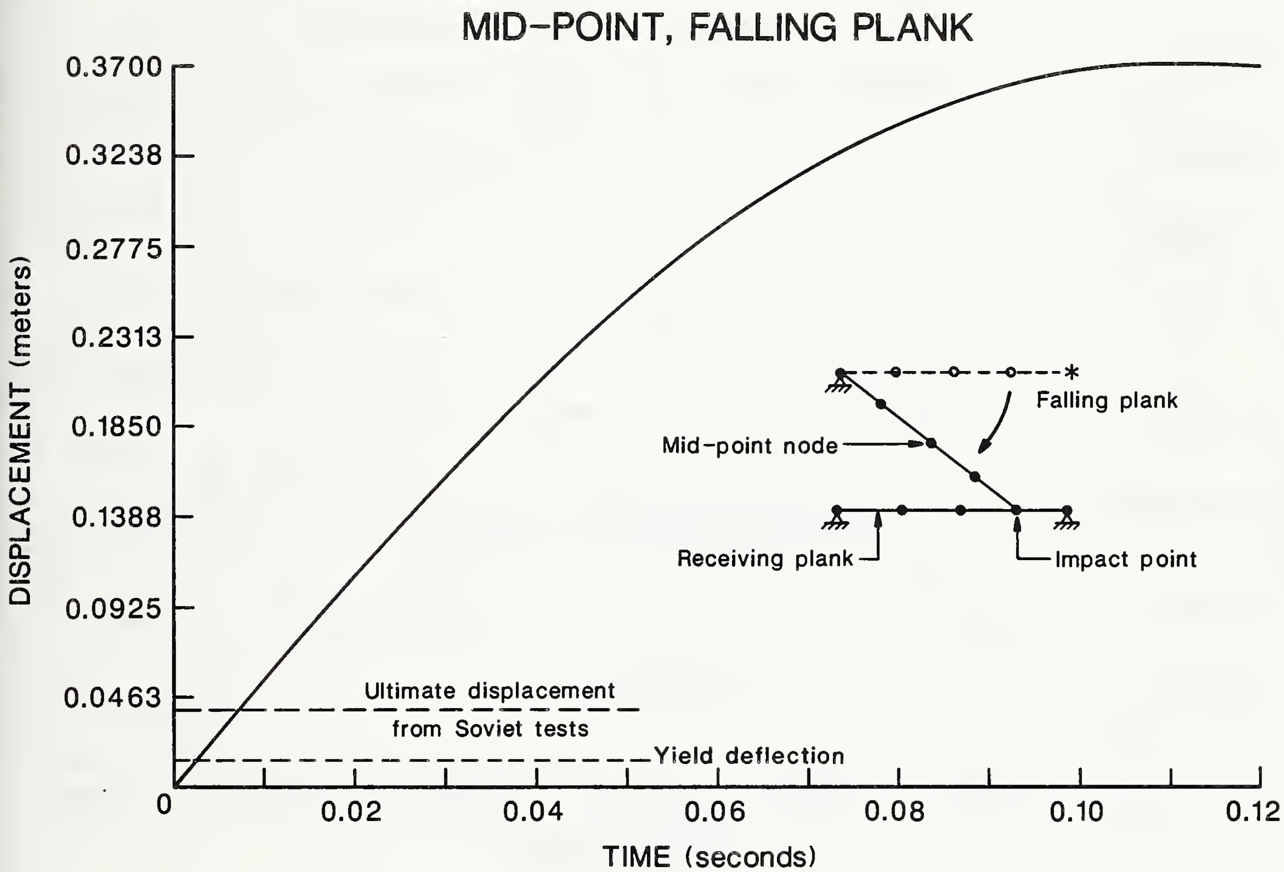


Figure 4.5.3. Displacement-time history after impact at the midspan of the falling plank

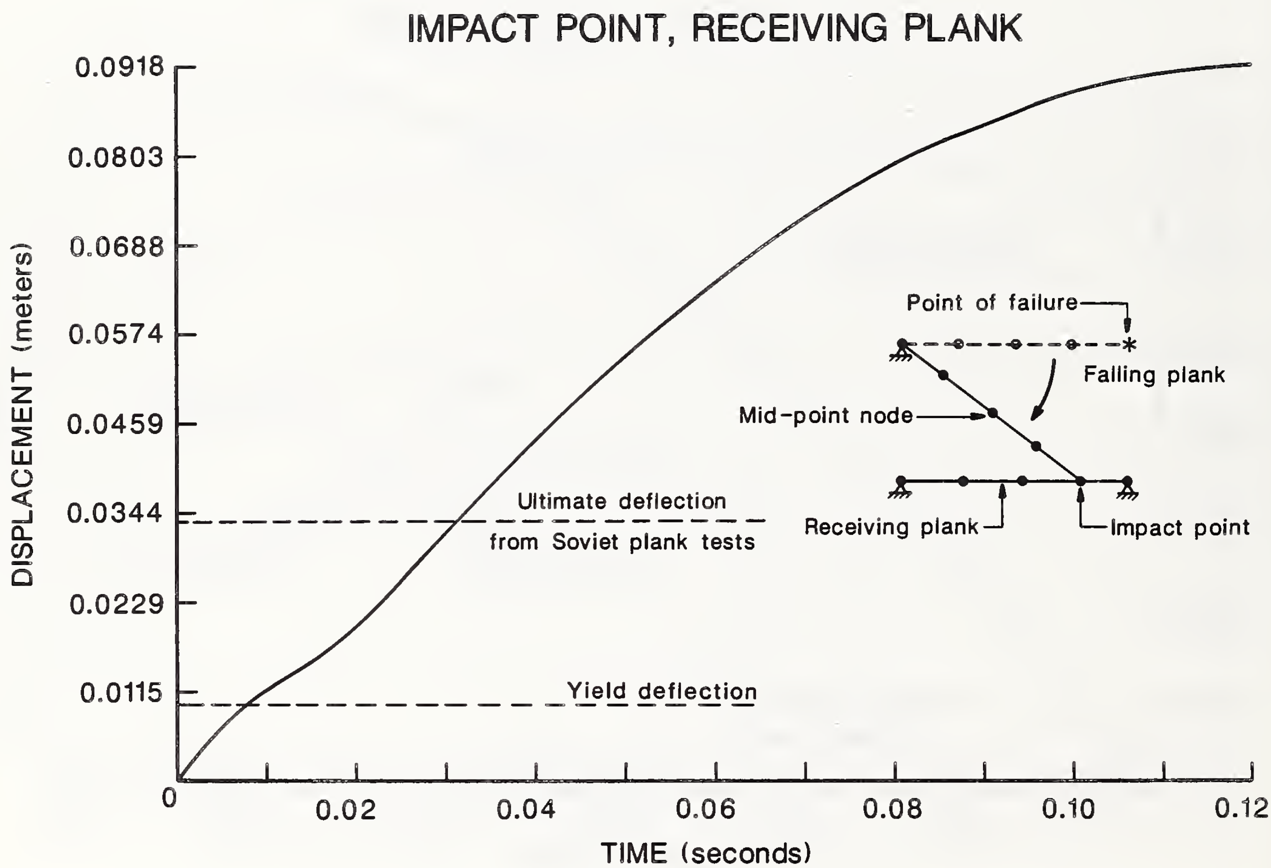


Figure 4.5.4. Displacement-time history after impact at the impact point of the receiving plank

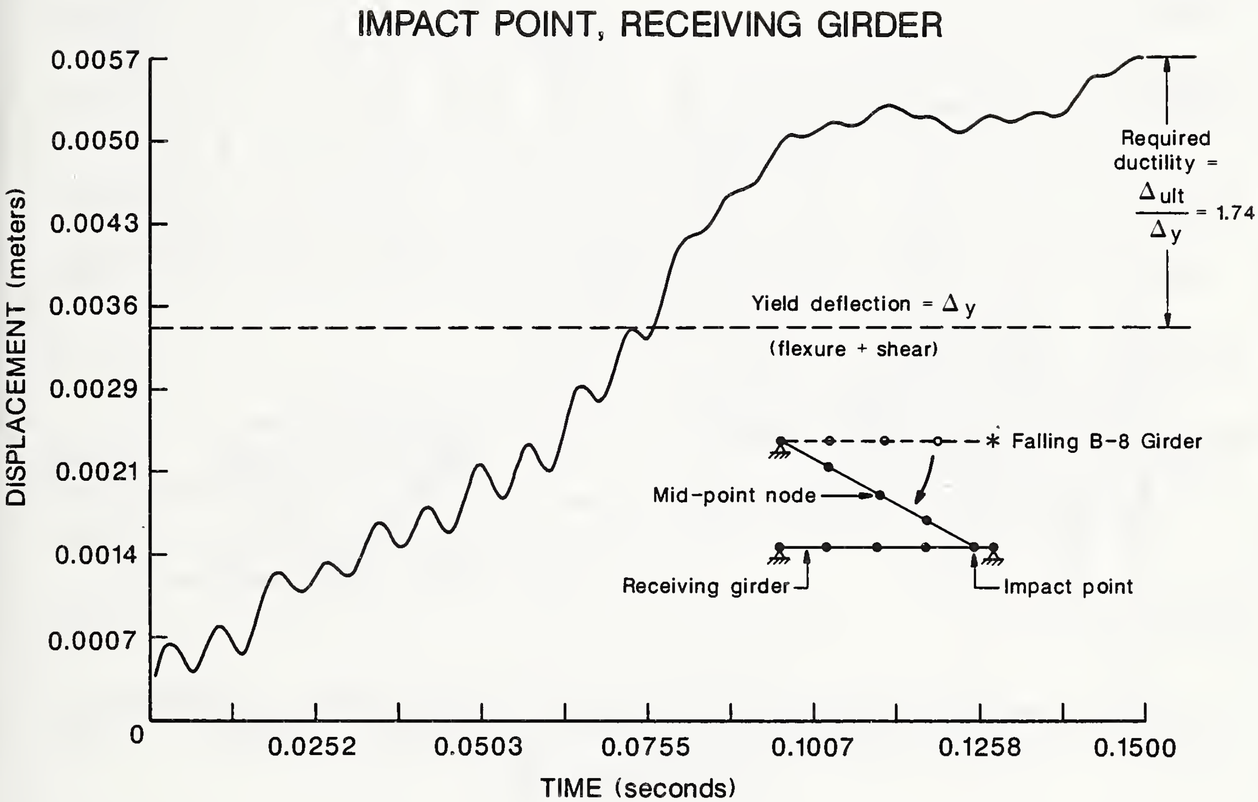


Figure 4.5.5. Vertical displacement-time history at the point of impact of the receiving girder for an end-point girder failure

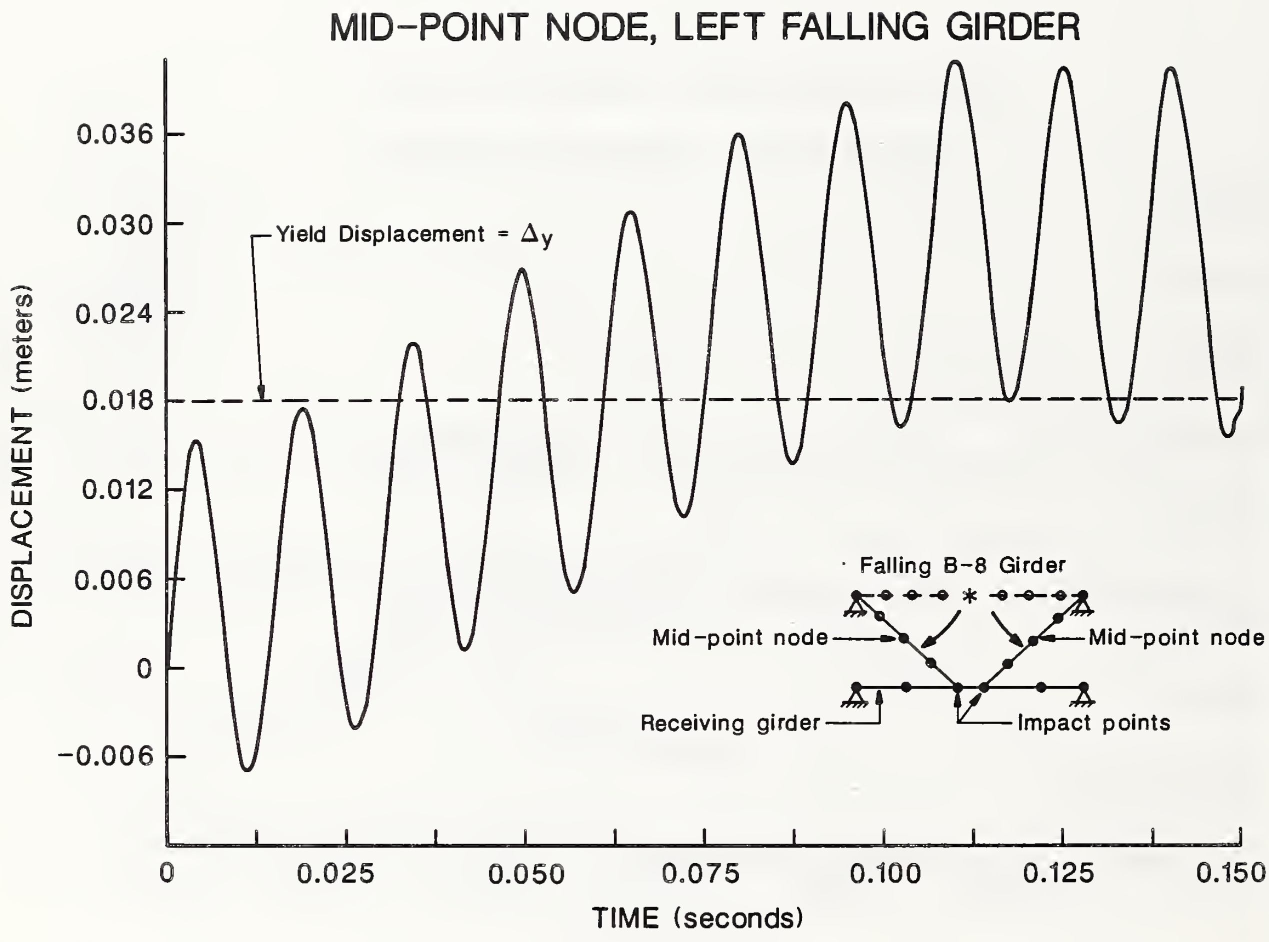


Figure 4.5.6. Displacement-time history in a transverse direction for the midpoint of one of the segments of the falling girder for a mid-point girder failure

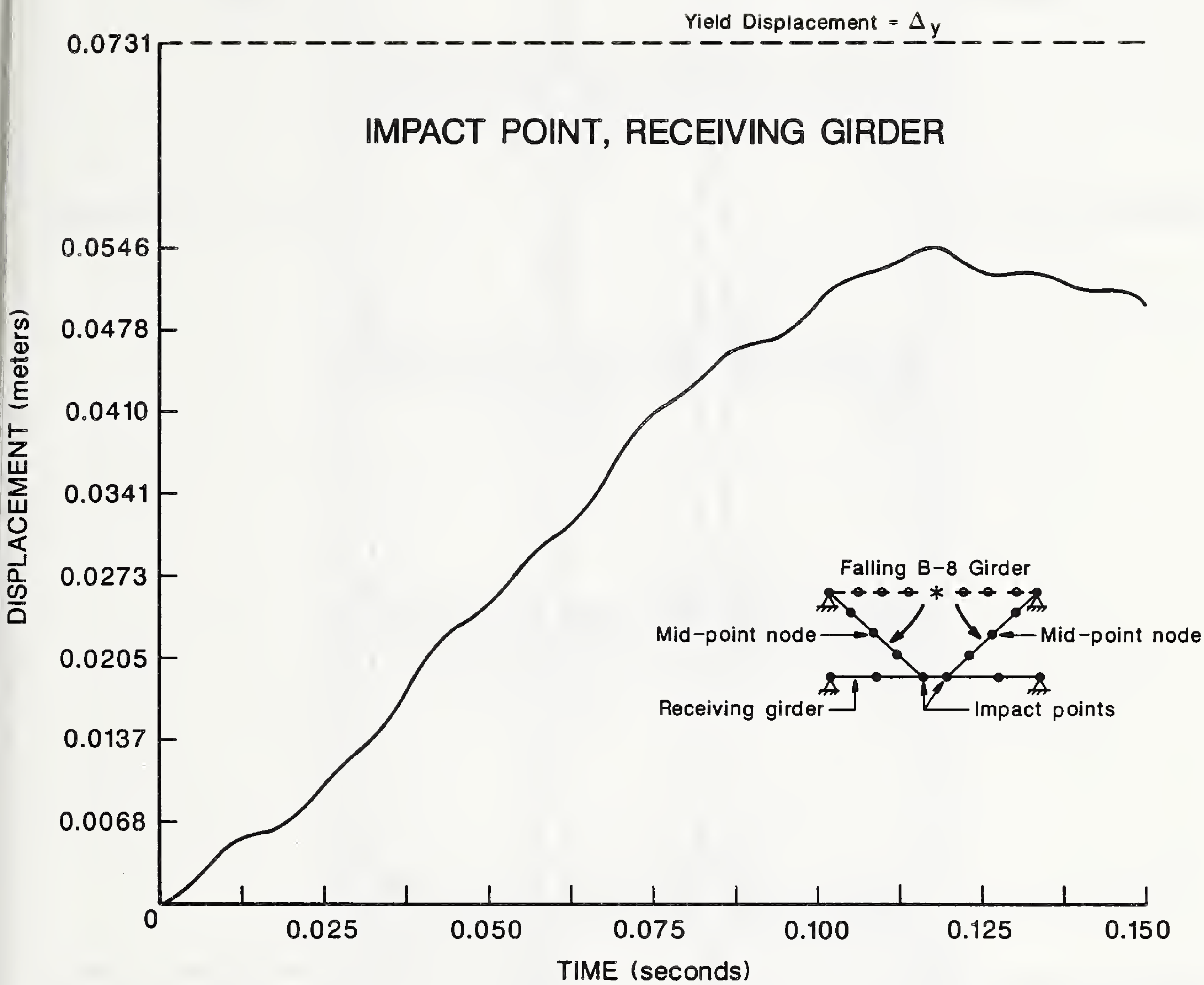


Figure 4.5.7. Displacement-time history in a transverse direction for one of the impact points on the receiving girder for a midpoint girder failure

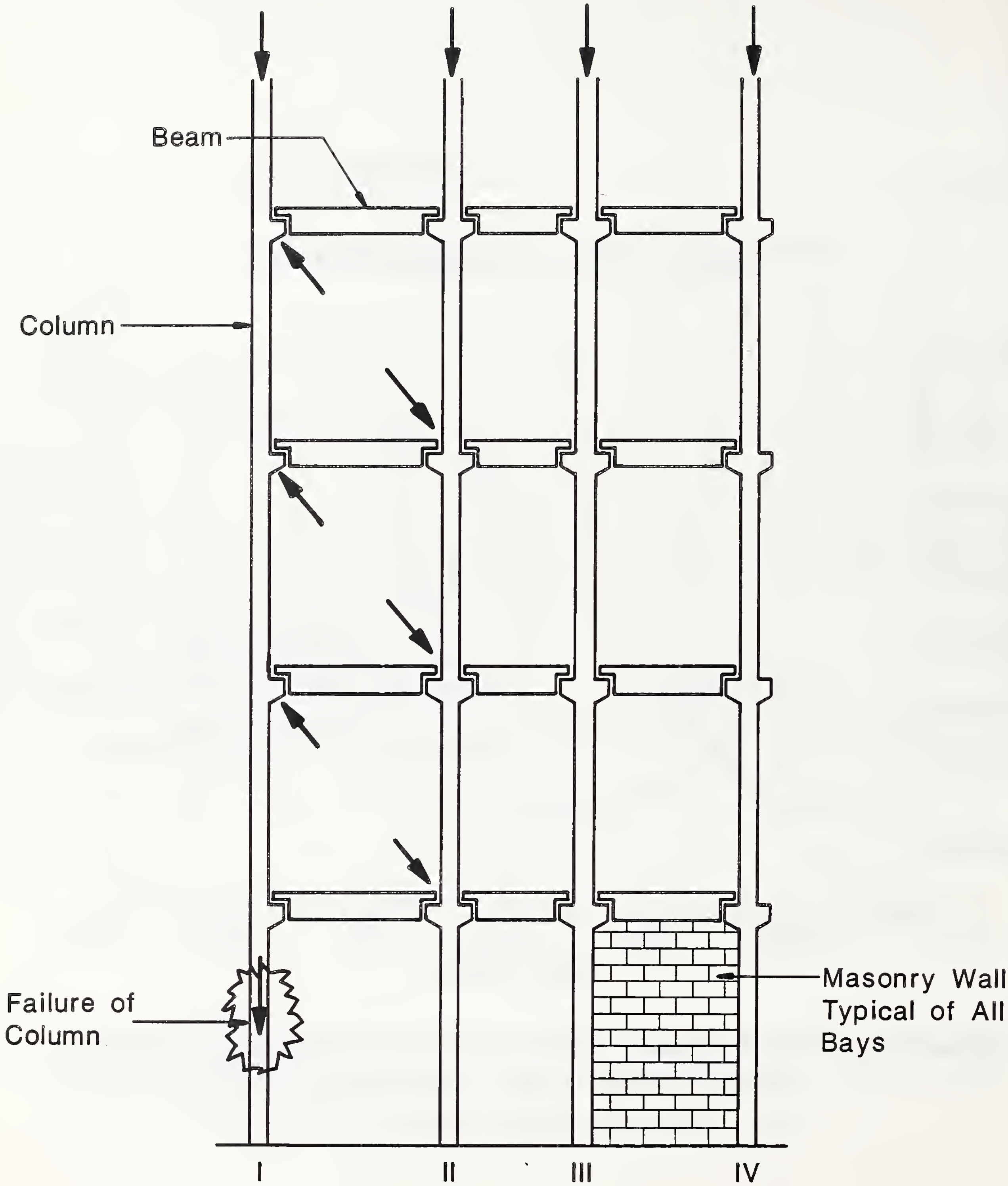


Figure 4.5.8. Column-load transfer by cantilever action of walls

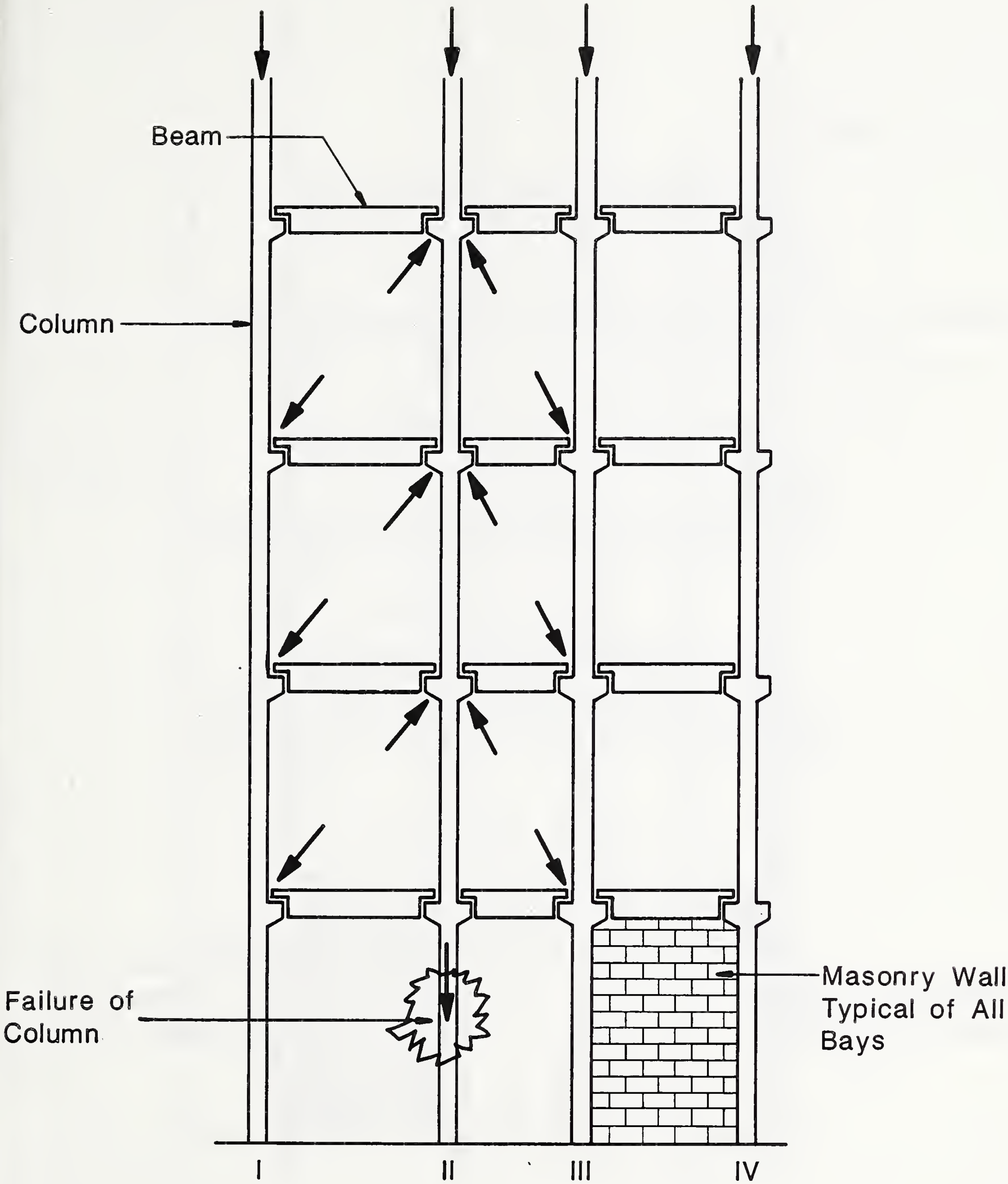


Figure 4.5.9. Column–load transfer by deep beam action of walls

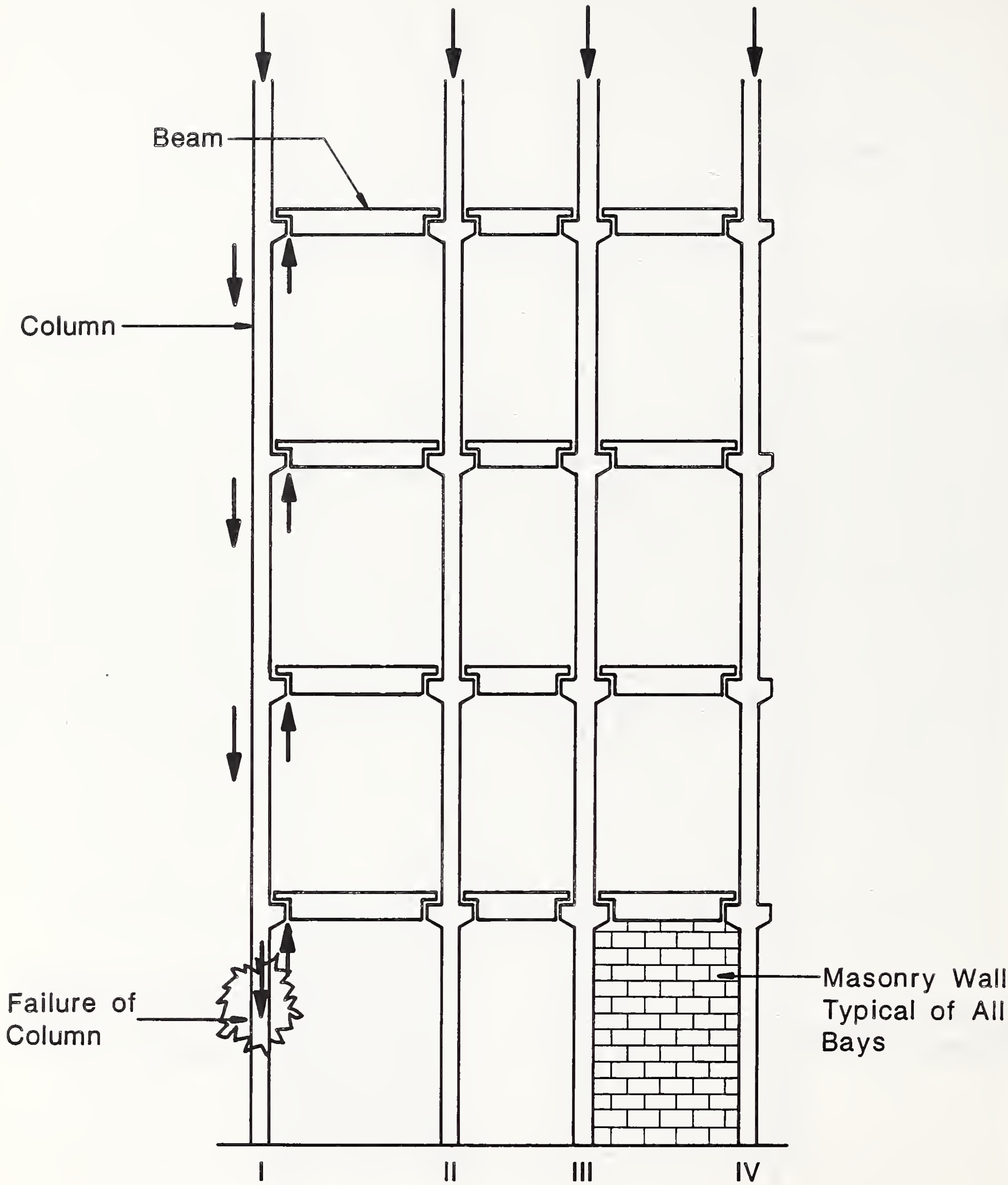


Figure 4.5.10. Shear force transfer by beam connection

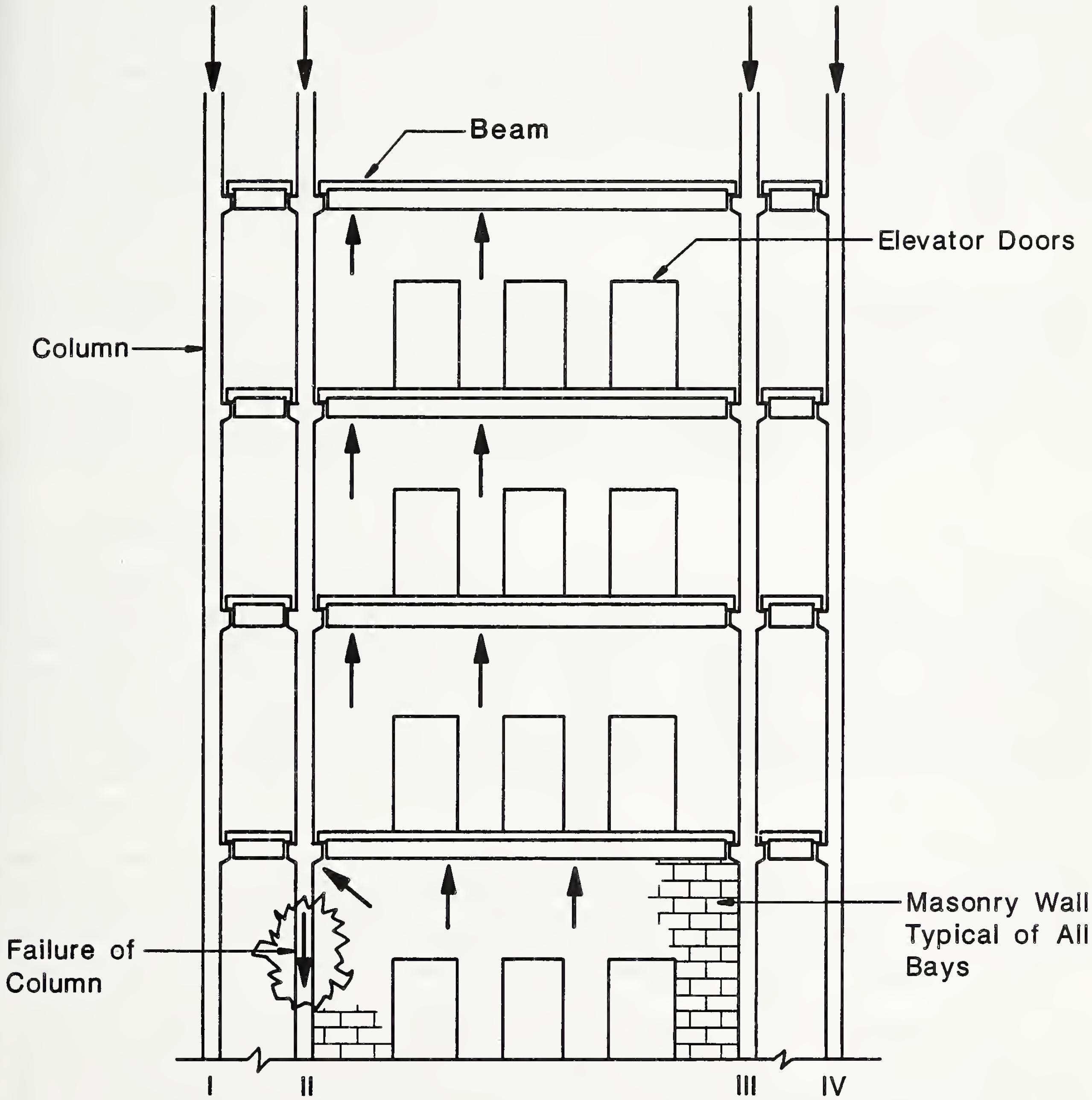


Figure 4.5.11. Transfer of column load to bearing wall

CHAPTER 5
SITE AND LABORATORY INVESTIGATIONS

5.1 INTRODUCTION

As part of this investigation, two site visits were made to the Office Building. The first visit took place during December 17 to December 19, 1986. The purpose of this trip was to study the structure and its condition to assist in planning for a detailed site investigation. The second visit took place from February 17 to March 6, 1987, and involved a detailed investigation of the structural system and the building envelope. The objective was to document aspects of the as-built structure which would have significant impacts on the structural and envelope assessment. During the second site visit, various material samples were obtained and shipped to NBS, where they were tested.

This chapter presents the findings of the field investigation and associated laboratory studies. The chapter begins by discussing the documents used in this study. Next, the results of the field investigation of the structural system are presented. The presentation is divided into two parts: a review of the structural members, and a review of the connections between members. The remaining sections review the results of the field investigation of the building envelope and the results of tests of masonry materials.

5.2 DATA SOURCES

The Office of Foreign Buildings Operations (FBO), Department of State, cooperated by providing NBS with various sources of data related to the Office Building. Copies of key structural and architectural drawings were provided. Copies of Soviet catalogs of the building components used in the Office Building were also obtained; they contained valuable information on the details of the structural elements. Although a complete set of catalogs for every type of element was not available, there was sufficient information to gain an understanding of main structural elements and their connection details.

Access was provided to copies of daily inspection reports and monthly progress reports. A review of this information gave no indications of major structural problems during construction. The only significant observation is that the inspection reports indicate that some damaged precast planks were placed in the structure and the Soviet contractor was requested to have them replaced.

Copies of videotapes taken during various stages of construction were also provided by FBO. These tapes were intended to show the state of construction of the entire Embassy compound, and they provided only limited information about the Office Building. However, they were useful in providing an overview of the project and details of how the various building components were installed.

5.3 EXAMINATION OF STRUCTURAL MEMBERS

5.3.1 Floor Slabs

The analysis of typical precast floor planks reported in Chapter 4 indicated that they would be capable of resisting the design loads. Therefore, the field investigation did not include a detailed examination of these elements. Aside from several planks that were obviously damaged during construction, a visual inspection revealed no signs of distress from applied loads.

The floor planks in the Office Building were found to be covered with cast-in-place topping except for areas within the core. An examination was performed to determine the degree of bonding between the topping and the planks.

A general survey of the topping on floors 2 through 7 was performed by "sounding" with a steel bar. The bar was dropped onto the topping from a height of about 30 cm and the resulting sound was noted. A "hollow" or "drummy" sound was taken to be indicative of topping not bonded to the planks. Regions which sounded "drummy" were found on all floors, and it was noted that the drummy regions contained extensive cracks in the topping. The second and third floors appeared to have more drummy sounding areas than the other floors. On each of the floors surveyed, two core samples were drilled to determine the bond strength between the topping and the planks. One core was drilled in a region which sounded drummy and the other in a region which sounded solid. The drilling locations were determined using a covermeter (magnetic device for locating reinforcement) so that the wire mesh would be cut allowing its depth to be measured. On the second and third floor, wire mesh was exposed at several unfinished portions of the floors and the wire spacing was measured to be 100 mm. However, on the fourth floor (west), the covermeter indicated a 150-mm spacing. Table 5.3.1 summarizes the data obtained from coring the floor topping.

None of the cores that were drilled remained intact. Even the regions of the topping which sounded solid were not bonded to the floor planks. After the cores were removed, a borescope was used to look at the interface between topping and planks. In drummy sounding areas, the topping had delaminated from the planks or there were voids at the interface. Figure 5.3.1 is a photograph showing the condition of the interface in core hole 3-2. The photograph shows the wire mesh, a void directly below the wire, and separation between the topping and plank. In the solid regions, examination of the joints did not indicate delaminations. However, as shown by the separation which occurred during core drilling, those regions which had good contact at the interface still did not have much bond strength.

Wire mesh was found exposed adjacent to the elevators, and seven wire samples were taken from different floors. The samples were about 40 cm long and included short segments of the transverse wires. The samples were shipped to NBS where they were tested in tension. The measured properties were yield strength (0.2 percent offset), ultimate strength, and percent elongation over a 50-mm gage length. The wire diameter was 5 mm. The results are summarized in table 5.3.2.

5.3.2 <u>Beams</u>

5.3.2.1 Steel Beams

Dimensions and in-place hardnesses were measured on about 40 steel beams. Beams were selected from all floors. Dimensions were found to be in accord with the plans.

Hardness tests were performed with an impact device that monitors the velocity of a spring driven mass before and after impact with the test object. A digital readout device gives the hardness number, Ld, which is the relative velocity before and after impact multiplied by 1000. Correction factors are applied to account for the orientation of the impact direction with respect to gravity. As hardness of the metal increases, the Ld number increases. The manufacturer provides conversion tables to convert the Ld values to other standard hardness values, such as Brinell. Empirical correlations may then be used to estimate the steel strength. Prior to testing, paint and mill scale were removed from the test region.

Hardness data are shown in table 5.3.3 for beams specified to be made of high strength steel (Mark 14G2-6) with a nominal yield strength of 3300 kg/cm^2. Table 5.3.4 shows hardness data for beams designed to have ordinary strength steel (Mark VSt3ps6) with a nominal yield strength of 2300 kg/cm^2. The letters "u", "h" and "d" indicate the travel direction (up, horizontal, and down) of the mass before impact; Ld values are corrected to a "down" direction. Note that the webs of built-up sections tend to have hardnesses that are less than the flanges.

Samples were cut from a built-up section of high strength steel (beam 8-G2/4A-J/4A, section b8a), a built-up section of ordinary steel (beam 7-J/6-J/7, section b18), and a rolled section of ordinary steel (beam R-D/1A-E/1A, section b26a). Figure 5.3.2 shows the portion removed from the type b8a beam on the eight floor (see table 2.3.1 for beam dimensions). Samples removed from the built-up sections included the shop welds used to splice plates (many web and flange plates have been spliced together from smaller plates) and to connect the web and flange. Tensile specimens were made from the removed portions according to ASTM A 370-77 [5.1]. Prior to tensile testing, 10 hardness tests were performed on each specimen. Table 5.3.5 gives the tensile and hardness properties of the specimens. An error was made in making the tensile specimens from the web of section b8a; both specimens included the web plate splice weld. At this location, the weld was of poor quality so the tensile strengths were uncharacteristically low and are not reported. However, based on the hardness value, it is likely that the strength of the web of beam b8a is similar to that of the flange of beam b18. The steel in the flange of the b8a-beam on the eighth floor is made of much higher strength steel than indicated in the plans.

Specimens from the beam samples were chemically analyzed using multi-element, atomic emission spectroscopy. Table 5.3.6 summarizes the results of the chemical analysis. Specimens were also prepared for metallographic analysis to examine grain structure. The specimen from the flange of the beam b8a has a significantly higher amount of vanadium (V) and has a finer grain structure than the other specimens. These factors help to explain why the flange of beam b8a had a higher yield strength than the other samples. Metallographic analyses were also performed on samples of the welds. There were no unusual results.

The drawings indicate that lateral restraint of beams spanning from the exterior framing to the core framing is provided by 100-mm lengths of 200-mm channel welded to the top flange at 1000-mm intervals and embedded in the concrete placed between floor planks (fig. 4.1). The presence of these connectors was confirmed by radar inspection and by removing concrete on the second floor to observe two of the connectors.

Fireproofing is not yet in place on steel members above the fourth story. Fireproofing was measured on six beams as shown in table 5.3.7. In all but one instance the minimum specified thickness of 32 mm was equalled or exceeded. Fireproofing is missing occasionally below the fourth story, particularly on the steel connections between concrete beams and columns. Samples of fireproofing were removed from stories 2, 3, and 4 and subjected to asbestos analysis at NBS. No asbestos was observed in any of the samples.

5.3.2.2 Concrete Beams

The review of the design discussed in Chapter 4 indicated that two of the type R-38-8 inverted T-beams on the first floor did not have the margin of safety required by current U.S. practice. Therefore, during the site investigation these potentially understrength beams were examined.

One of the critically loaded R-38-8 beams is located between columns 5A/D1 and 6/D1 and the other is located between columns 5A/C2 and 6/C2. These beams support masonry walls along their lengths and support two other beams which also carry masonry walls. From within the elevator shaft it was possible to carry out a close visual inspection of the beam between columns 5A/D1 and 6/D1. There were no signs of flexural- or shear-type cracking. The Soviet catalog indicated that this beam should be made with Mark-300 concrete. Two cores were removed from the beam were tested for compressive strength at NBS. The results were 559 and 548 kg/cm^2, which exceed the design strength by a factor of three.

5.3.3 Columns

5.3.3.1 Steel Columns

Steel column dimensions and hardnesses were reviewed where such columns occur in the seventh, eighth, and penthouse stories. Dimensions were in accord with plans for the S1 and S3 columns. S2 columns were found to be made up of two equal leg angles with legs 160 mm wide and 11 mm thick. They are interconnected by 12-mm thick plates at 740-mm spacing, rather than welded heel to heel as shown in figure 2.3.5.

Hardness data for one of each type of column section are as follows:

Column	Section	Element	Hardness Value - Ld Median	Corrected
9-F/4A	S3	flange	490 h	480
8-E+/1A	S2	flange	400 h	388
8-A1/?	S1	flange	450 h	439

According to drawing KM.3.5.7, column type S1 should be made of steel with a nominal yield strength of 3300 kg/cm^2, while column types S1 and S2 are rolled shapes of ordinary strength steel (yield strength 2300 kg/cm^2). The above hardness values suggest that the S3 column is made of stronger steel than specified.

5.3.3.2 Concrete Columns

The review of the column designs in Chapter 4 showed that many of the Type RC-5 reinforced concrete columns were not structurally adequate. These columns were specified to be made of Mark 600 concrete (corresponding to a cylinder strength of 357 kg/cm^2) which was used to calculate the strength of Type RC-5 columns (refer to section 3.4.1.3). The strength of these columns depends on the actual compressive strength of the concrete; thus core samples were taken from potentially overloaded Type RC-5 reinforced concrete columns. Core samples were also taken from representative samples of other column types in the Office Building. Nominal 75-mm core drills were used and cores were drilled 200 mm deep. For composite columns, the cores were drilled up to the embedded steel section. The cores were shipped to NBS, where they were trimmed and tested for uniaxial compressive strength according to ASTM C 42-85 [5.2].

The results of the tests on the concrete cores are summarized in table 5.3.8. For cores removed from Type RC-5 reinforced concrete columns, the average compressive strength is 596 kg/cm^2 and the coefficient of variation is 0.10. Thus the average in-place compressive strength exceeds the value used in the design check by a factor of 1.66. The significance of this is discussed in Chapter 6. The average strengths of cores from other column types are also greater than the values assumed in the design review.

As described in subsequent section 5.5, the reinforced concrete columns are designed with five layers of transverse reinforcement at their ends. This reinforcement is critical to prevent splitting of the concrete due to concentrated loads in the joint region. According to details shown in the Soviet catalog "Reinforcing Units for Albums RS 2275-79, RS 2276-79, RS 2269-79," the transverse reinforcement is composed of 5 grids of 12-mm reinforcing bars, evenly spaced within a 300-mm length at each end.

Two Type RC-5 reinforced concrete columns in the second story of the Office Building were checked with a radar system to verify the presence of the transverse reinforcement. Figure 5.3.3 shows the equipment for radar inspection. A hand-held antenna is moved along the column surface and the reflected signals are displayed on a plotter. Figure 5.3.4 shows the results of the inspection on column E/6A. The sketch on the right side indicates the locations of reference lines drawn on the column at 200-mm spacing. The radar scan shown on the left clearly indicates that transverse reinforcement is present in the correct location. In addition, the presence of column ties can be seen above the column joint. The tie spacing is about 200 mm. Similar results were recorded for column F/6A.

At the seventh story, reinforced concrete columns are joined to Type S1 steel columns using the connection detail shown in figure 2.3.8(d). At these locations, unused lengths of reinforcing bars are exposed. Samples of 32-mm bars approximately 160 mm long were taken from columns J1/3 (2 bars), J1/4, J1/5, and J1/6. The bar deformations were in a herringbone pattern, which signify grade AIII bars having a nominal yield strength of 4000 kg/cm^2. Flat tensile test specimens were prepared according to ASTM A 370-77 [5.1], and the test results are given in table 5.3.9. The average yield strength is 4200 kg/cm^2, and the average ultimate strength is 6900 kg/cm^2.

5.3.4 Shear Walls

According to Soviet catalogs, precast shear walls were to be made of Mark 300 concrete (184 kg/cm^2 design strength). To verify that concrete strength was as specified, core samples were taken from representative wall panels. Since the precast shear walls below the fourth story were covered with brick masonry, core samples were taken from stories 4, 5, 6, and 7. In each story, two wall panels were randomly selected for coring, and cores were drilled through the thickness of the walls. The cores were shipped to NBS and tested for compressive strength. The test results are summarized in table 5.3.10. The core from the fourth story shear wall between columns C1/4A and D/4A contained a large crack-like defect and was not tested. The average core strength is 430 kg/cm^2 and the coefficient of variation is 0.13. The lowest strength is 321 kg/cm^2. These are considerably stronger than the concrete strength required.

During the visual examination of shear wall panels, a long crack was found in the panel between columns C1/4A and D/4A in the fourth story. Figure 5.3.5 shows a photograph of the panel with the crack highlighted with a marking pen. The crack was adjacent to the core described above. The crack in the core did not appear to be the same crack on the face of the shear wall panel. The cause of these cracks is uncertain. However, the nature of the crack in the core suggests that it probably occurred at a very early age, and might have been caused by premature lifting of the panel.

5.4 EXAMINATION OF CONNECTIONS

5.4.1 Beam-to-Column

Connections of steel beams to columns were observed to conform to plans. The large beam end reactions coming from the 13.2-m steel beams are resisted by the seated connections shown in figure 2.3.14. These connections also include four erection bolts and a moment limiting top plate. The erection bolts are

20 mm in diameter and have a measured hardness Ld-value of 460. The bolts and the top plate provide lateral resistance at these connections.

Connections of reinforced concrete beams to columns were examined on floors 3, 4, 5, 6, and 7. The connections of the facade Z-beams (spandrel beams) and the 7.4-m floor beams spanning from exterior to core columns were examined. Not all connections on these floors were visible because some had been covered over by fireproofing or mortar. The following observations were made:

- Details for these connections call for grout in the space between the columns and the beams. This grout did not exist in all cases.
- The beams were connected to the column brackets or corbels with 100-mm long fillet welds on both sides of the beams.
- Many of the steel brackets supporting the 7.4-m beams at the exterior columns had not been painted and were rusted (fig. 5.4.1).

5.4.2 Column-to-Column

Precast concrete column sections in the Office Building are composite columns and reinforced concrete columns. The one-story tall column segments are joined using the connection details shown in figure 2.3.8. The steel cores of the composite columns bear directly on each other at the joints, and any defects (such as voids) in the concrete used to fill the joint region are not expected to have detrimental effects on column capacity. However, for the reinforced concrete columns, the joint detail provides for direct bearing on a 120-mm diameter "button" cast into the column base. Figure 5.4.2 shows the Soviet joint details for reinforced concrete columns. The 20-mm gap between column segments should be filled with Mark 200 grout and the pockets around the four corner bars should be filled with Mark 300 concrete. Additional column joint details are described in section 5.5.

During the first site visit, many defective column joints were observed. An example is shown in figure 5.4.3, which shows the connection of a Type RC-5 column to its supporting pedestal. It is seen that the pockets around the corner bars are not properly filled with concrete. (As a result of these observations, an experimental program was designed and performed at NBS to quantify the effects of defects on joint behavior. The results are presented in the next section.) The design review performed in chapter 4 showed a number of Type RC-5 reinforced concrete columns to be overloaded. For these reasons, field inspection of the joints for those columns identified as being potentially overloaded was carried out during the second site visit.

The field inspection was performed by drilling 12-mm diameter holes into the grout-filled 20-mm gap and looking inside with a side-viewing borescope. In most cases this procedure was complicated by brick masonry walls or plaster and wire lath covering the inspection location. In several such cases, removal of the covering revealed inadequately grouted or concreted joints.

Table 5.4.1 summarizes the inspection results for 24 column joints. Part (a) of the table includes 10 joints from Type RC-5 reinforced concrete columns identified as being overloaded (see table 4.2.7); two joints were from the seventh and eighth stories where voids were visible without drilling. Part (b) of the table includes seven joints along vertical column line G2/5; the second and third story columns on this line are also overloaded. In the table, the figures on the right are sketches of the bore hole locations and the shapes of observed voids. Of the 12 Type RC-5 concrete columns identified as being overloaded, three had significant voids at the column joint.

In some cases, voids were located in joints which from the exterior appeared to be properly grouted. For example, figure 5.4.4 shows the exterior appearance of the joint at column 6-G2/6, and figure 5.4.5 is the internal view; the bearing button is seen on the right of the photograph and the 20-mm gap is empty.

Along vertical column line G2/5, five of the seven joints inspected had significant voids (table 5.4.1 (b)). Some of these joints had no grout in the 20-mm gap and signs of distress were present on the 120-mm button (fig. 5.4.6). Furthermore, the joints of the fourth and sixth story columns along G2/5 had the outward appearance of properly completed joints, or were covered by wire lath and plaster. In some cases, such as shown in figure 5.4.7, the joints appeared to have been covered with mortar when the brick partition walls were built.

Important conclusions drawn from the data in table 5.4.1 include:

- The inspected basement columns, which represent the most heavily loaded Type RC-5 columns, all appear to have been correctly grouted and concreted.

- The number of voids discovered in column line G2/5 indicates poor quality control of the grouting and concreting of the column joints.

Poor quality control in filling column joints was not limited to reinforced concrete columns. In the design review, composite columns were not identified as being critical, and an in-depth examination of their joints was not conducted. However, in the course of a general visual inspection of those columns which were accessible, many examples of defective joints were noted.

In the penthouse, the joints of reinforced concrete columns (Type 2) were found to have no grout or concrete. Since these columns were under light loads, corner reinforcing bars were removed from three columns in order to examine the quality of the electroslag welds used to join the corner bars. Prior to cutting the bars, grout was placed in the 20-mm gap. Figure 5.4.8 shows the joint of column G2/5 prior to removal of a left bar, and figure 5.4.9 shows the removed bars. For two specimens, the weld joins 20-mm and 32-mm diameter grade AIII bars. For the third specimen, the weld joins a 20-mm grade AIII bar to a 25-mm grade AII bar. The G2/5 specimen was prepared for a tensile test at the NBS laboratory in Boulder. After testing, the specimen was sliced and subjected to metallographic analysis. The other specimens were subject to metallographic and chemical analyses. The specimen from column G2/5 failed in the 20-mm bar at a stress of 7350 kg/cm^2. The average ultimate strength of the reinforcing bars taken from the seventh story (refer to section 5.3.3.2) is 6900 kg/cm^2. Thus it appears that the electroslag welds used to join the corner reinforcing bars are capable of developing the ultimate strength of the bars. The metallographic analysis revealed no unusual characteristics in the welds.

5.4.3 Shear Walls

The effectiveness of the interior shear walls in resisting lateral load depends upon the ability of the wall to behave as a monolithic cantilever beam. This requires proper joining of the shear wall panels to the columns and to each other. According to Soviet catalog DS 27-1-79, the joining of shear wall panels requires the following steps:

- The horizontal, castellated gap between panels must be filled with Mark 300 concrete (fig. 4.1.1). This is to transfer horizontal shear stresses between wall panels.

- The three steel shear wall brackets on each side of a panel must be welded to the adjacent column or shear wall (figs. 2.3.21 and 2.3.22). This ensures transfer of vertical shear stresses between panel and column.

- The vertical gaps between the shear wall panels and columns must be filled with Mark 100 grout. This assists in achieving monolithic behavior of the walls and columns and helps reduce shearing loads on the welded connections.

A visual inspection of the shear walls was conducted to determine the adequacy of the concreting and grouting operations listed above. In addition, tests were performed on selected shear wall-to-column connections to verify conformance with the intended design.

Inspection of all shear wall panels was not possible because many were covered with brick masonry walls. However, the exterior faces (away from the core) of the shear walls were exposed in stories 4 through 7. The interior face of the east shear wall was visible from the east stairwell and horizontal joints at the sixth and seventh floors were visible. Two open mechanical chases exist which run the full height of the building adjacent to the interior faces of the shear walls and are accessible from the eighth floor. One is at the southeast corner of the core between columns G/6B and G2/6b; the other is at the northwest corner of the core between columns C1/4A and C1/5. The shear walls were inspected from inside the chases using technical mountaineering equipment. Figure 5.4.10 shows an NBS team member descending the northwest chase.

5.4.3.1 Horizontal Shear Wall Joints

Figure 5.4.11 is a view of the inside face of the west shear wall between column lines C1/4A and C1/5 at the seventh floor level. The photograph shows the smooth textured precast concrete panel and the rough textured concrete placed to fill the castellated shear wall joint. During construction, formwork was built on the interior faces of the shear walls to hold the concrete to be placed in the joint. The hardened concrete reflects the shape of the formwork, and, from the inside face, the concrete appears to fill completely the horizontal joint. However, the rough texture of the joint suggests that vibrators were not used to consolidate the concrete. It was decided to inspect the horizontal joint shown in figure 5.4.11 from the exterior face of the wall to see if the concrete had filled the joint.

Figure 5.4.12 shows the appearance of the horizontal joint at the seventh floor along column line C1. As shown in the top photograph, the joint is covered with a course of brick masonry; the masonry was bonded with strong mortar and was difficult to remove. The bottom photograph shows the exposed joint on the opposite face shown in figure 5.4.11. It is clear that the concrete placed in the formwork on the interior face (which appears to have produced a good joint) was not vibrated so as to fill the castellated pockets to the full width of the panels.

The horizontal shear wall joints on the fourth floor were also examined after removing masonry at the floor level on the exterior face. Between columns G2/5 and G2/6, the joint was observed to be incompletely filled, even though its appearance was good on the interior face. Between columns G2/6B and G/6B, however, the joint was properly filled. The space between the wall panel and the floor topping was completely filled with concrete (fig. 5.4.13). A core sample was taken from this joint and tested at NBS. The compressive strength of the core was 550 kg/cm^2, which is greater than the strength of Mark 300 concrete which was specified for the joint.

5.4.3.2 Vertical Shear Wall Joints

The design calls for grout to be placed in the 15-mm vertical gap between columns and shear wall panels. Where it was possible to inspect these vertical joints (above the fourth floor), grout was observed to be incompletely placed, and in some places it was totally absent (fig. 5.4.14). Where grout was present, it appeared to have been trowelled into place from the interior face of the shear wall and often the gap was not completely filled to the exterior face.

In addition to the standard shear wall-to-column joints described above, a 205-mm wide cast-in-place joint exists adjacent to columns G/6B and D/4A, because standard precast panels were not available to match the column spacing. The concrete to be placed in this joint is reinforced with four 25-mm vertical bars arranged in a rectangular pattern and 8-mm lateral ties at 200-mm intervals. At these joints, the connections to the columns require longer weld plates than the standard connection (255 mm versus 65 mm). Because of the increased flexibility of the longer plate, this joint will not be effective in transferring shear forces unless the joint is properly concreted. Figure 5.4.15 shows the typical appearance of these joints on the sixth and seventh floors where concrete placements were incomplete.

5.4.3.3 Shear Wall-to-Column Connections

In-place tests were performed to determine whether the as-built welded shear wall connections conformed with the design. Ten joints were randomly selected for examination. In addition to standard joints between shear walls and columns, the wide joint at column G/6B in the seventh story and a shear wall-to-shear wall joint between columns G2/5 and G2/6 in the fourth story were inspected.

The inspection procedure was as follows:

- Test sections on the welds and the connecting plates were prepared with a surface grinder. An attempt was made to achieve a smooth surface for the hardness testing.

- A total of 10 hardness measurements were taken for each component of the joint. Using the correlation data in table 5.3.5, the yield strength of the steel plates can be estimated.

- An ultrasonic thickness gage was used to measure the thicknesses of the plates embedded in the shear walls and the plate welded between the shear walls and columns.

Table 5.4.2 summarizes the results of the in-place tests. The "Item" column indicates the test components, which are as follows: SWP is the plate embedded in the shear wall; CP is the plate welded between the shear wall and column; and W is the weld joining the column plate to the shear wall. For the shear wall-to-shear wall joint (fig. 2.3.22), the cover plate thickness was measured. The "Height" column indicates the top (T), middle (M) or bottom (B) connection of the wall panel.

According to the Soviet catalogs for the shear walls (RS 3170-77 and 5 KX-3-5), the weld plates embedded into the shear walls are 10 mm thick and of ordinary strength steel (nominal yield strength 2300 kg/cm^2). The connection details in Soviet catalog DS 27-1-79 specify 12-mm thick plates for the connections to the columns. The average hardness value of the plates embedded in the shear wall is 363, and the estimated yield strength (based on the data in table 5.3.5) is 3000 kg/cm^2. The plates welded to the columns have an average hardness of 381, and so their yield strength is similar to the shear wall plates. The average hardness of the weld metal is 433. Thus the weld metal is stronger than the base metal, and its estimated ultimate strength is 6000 kg/cm^2. It is seen that the measured thicknesses of the plates are slightly smaller than specified, but this may have resulted from the grinding. In general, the weld lengths were found to be about 10 mm shorter than specified (figs. 2.3.21 and 2.3.22).

5.5 LABORATORY STUDY OF COLUMN JOINTS

5.5.1 Introduction

The connection between reinforced concrete columns is made by welding the four corner reinforcing bars, grouting the gap surrounding the central bearing "button," and filling the pockets around the corner bars with concrete. Observations of these joints showed that many were defective. In some cases, the concrete around the corner bars contained voids, and in some cases the grout between column segments was partially or totally absent. This raises the question of the effects of these defects on column capacity.

To determine the effects of joint defects on column capacity, a series of laboratory tests was carried out on eight reinforced concrete specimens with simulated column joints. Four joint conditions were tested: with concrete and grout, with concrete only, with grout only, with neither concrete nor grout. Two specimens were made for each condition. The specimens were axially loaded to determine the compressive strength of the different joints.

The following sections describe the details of the column connections and the construction of the laboratory test specimens. The test procedure is outlined and the results of each test are presented. Ultimate capacities are compared and modes of failure are discussed. Conclusions are drawn as to whether the grout and concrete are essential to achieve sufficient capacity of the column and to ensure correct behavior of the joint.

5.5.2 Description of Connections

The column chosen for study was a Type RC-5, reinforced concrete column. Figure 5.5.1 shows sectional views of a Type RC-5 column. The column has 40-mm longitudinal reinforcing bars in each corner, four 25-mm longitudinal bars on the sides, and 10-mm lateral ties spaced 250 mm on center along the length of the column. AIII steel is used for the longitudinal reinforcing bars (nominal yield strength is 4000 kg/cm^2), and AI steel is used for the lateral ties (nominal yield strength is 2400 kg/cm^2). The specified concrete is Mark 600 (design compressive strength is 337 kg/cm^2). This column is widely used in the Office Building around the building perimeter and within the core.

The geometries of the top and bottom ends of Type RC-5 column are shown in figures 5.5.2(a) and (b), respectively. Both ends have pockets around the corner reinforcing bars to allow for welding after the columns are positioned. Note that only the 40-mm corner bars are made continuous by welding. The 25-mm bars terminate about 10 mm before the ends of the column. The bottom end of the column has a 120-mm diameter, 20-mm thick bearing button its center. This button is provided to facilitate vertical alignment of the columns during construction.

The column joint detail, as given in Catalog PS-27-1-79, is shown in figure 5.4.2, and figure 2.3.9 shows a joint without grout or concrete. Once the columns are aligned, the corner bars are welded using an electroslag welding process. Two 10-mm diameter U-shaped stirrups are placed around the center of the joint, and the overlapping legs are welded together. The 20-mm gap surrounding the central bearing button is filled with a Mark 200 grout; this assures whole-area bearing between the column segments. The pockets around the corner bars are filled with Mark 300 concrete; this protects the bars and prevents the bars from buckling under load.

Transverse reinforcement grids made of 12-mm bars are placed at each end of the column. The grid pattern is the same at both ends and is shown in figure 5.5.3. The transverse reinforcement confines the concrete above and below the joint and prevents vertical splitting.

5.5.3 Test Specimens

The column specimens built in the laboratory were similar to Type RC-5 columns. Sixteen column stubs, 800 mm in height, were constructed. Top and bottom ends were joined together to make eight column-to-column joint specimens, 1.6 m tall. Figure 5.5.4 shows the formwork and reinforcing cages for the column stubs. Cross-sectional dimensions for the specimens were the same as for a Type RC-5 column, and reinforcing bar sizes were matched as closely as possible using standard U.S. Grade 60 reinforcing bars (4225 kg/cm^2 nominal yield strength). Number 11 bars (35 mm nominal diameter) were used for the corner bars, and #8 bars (25 mm nominal diameter) were used for the other longitudinal bars. The transverse grids were made with #4 bars (12.7 mm nominal diameter). Concrete with a nominal cylinder strength of 420 kg/cm^2 was used. The maximum aggregate size was 10 mm.

Four joint conditions were considered. These are shown in Figure 5.5.5, and are as follows:

1) Full joint (grout and concrete completely fill the joint);
2) Concrete only;
3) Grout only; and
4) Neither concrete nor grout in the joint.

Two specimens were constructed for each condition. Table 5.5.1 lists the eight test specimens, the joint condition being tested, and the compressive strength at the time of testing of the column concrete, joint concrete, and grout. Figure 5.5.6 shows photographs of specimens with the four different joint conditions.

Various test specimens were made for measuring the strength of the materials used in the column specimens. For the column concrete, 150 × 300- mm cylinders and 150-mm cubes were cast and cured under water. These specimens were used to examine the relationship between cylinder and cube strengths. To obtain a measure of the concrete strength in the columns, a form the same size as the column forms was built, sixteen 150 × 300-mm cylinders were placed in the form, and the form and cylinders were filled with concrete. Thus the cylinders in the form were exposed to the same temperature history as the concrete in the columns. The compressive strengths of these cylinders are reported in table 5.5.1. For the joint concrete placed around the bars, 150 × 300-mm cylinders were used, and for the joint grout, 100 × 200-mm cylinders were used.

The column specimens were tested under axial compression in a hydraulic testing machine with a 5450-t axial load capacity. Load was applied at a constant rate until the ultimate capacity of each specimen was attained. Linear variable differential transformers (LVDT) were used to measure vertical displacements over a 420-mm gage length (220 mm above the joint and 200 mm below the joint) on three sides of each specimen. Two LVDT's were used to measure lateral displacements across the joint. Load and displacement measurements were automatically recorded and plotted during testing. The eight tests were also recorded on videotape.

5.5.4 Test Results

Table 5.5.1 lists the failure load of each column specimen and the ratio of the average failure load for each joint condition to the average failure load of the fully filled joints. Figure 5.5.7 shows vertical load-displacement curves for each test. Figure 5.5.8 shows each type of specimen after testing.

The two specimens containing full joints showed no distress in the joint during testing, and the failure loads were 771 and 824 t. Load-displacement behavior was nearly linear up to the ultimate load. Failure of the specimens occurred by vertical splitting of the concrete above the transverse reinforcement grids.

The specimens containing only grout in the joints exhibited behavior surprisingly similar to that observed in the specimens with full joints. The failure loads were 660 and 735 t; the average of these is 87 percent of the average capacity of the specimens with full joints. No distress occurred in the joints during testing except for some spalling of the grout at the edges of the columns. The load-displacement behavior was nearly linear and identical to the load-displacement behavior for the full joint specimens up to a load of about 500 t (fig. 5.5.8(b)). The specimens failed by vertical splitting of the concrete along the longitudinal reinforcing bars above and below the joint.

The specimens containing only concrete around the corner bars displayed less stiff load-displacement behavior compared with the previous cases. At a load of about 470 to 480 t, the load-deflection curve becomes nearly horizontal. Extensive deformation occurs without significant increase in load as the bearing button crushes and the concrete around the bars undergoes compressive failure. When the button has completely crushed, as evidenced by a measured vertical displacement of about 20 mm across the joint, the full surface of top column bears on the bottom column and the load increases rapidly until the ultimate capacity is reached. Final failure occurs by vertical splitting and spalling of the concrete outside the joint area. The "failure" load for this case is taken as the load at the start of the plateau in the load-displacement curve. This is analogous to a yield strength for a ductile material. This is a better measure of capacity than ultimate load because to reach ultimate load would require unacceptable large axial deformation. The start of the plateau is defined as the load producing an axial shortening of 2.5 mm as measured by the LVDT. The failure loads are marked with an "x" on figure 5.5.8, and the values are 469 t for Specimen No. 1 and 478 t for Specimen No. 2. The average of these failure loads is 59 percent of the average failure load reached by the specimens with full joints.

Column specimens containing neither grout nor concrete in the joint had the lowest capacities and showed the least stiff load-displacement behavior. The behavior of these specimens was similar to the specimens with concrete only in the joints. In both cases, the joints experienced severe distress. Large vertical displacements began to occur as the bearing button began to crush. The corner bars buckled, and in each test one of the corner bar welds cracked. For Specimen No. 3, the load-displacement curve exhibits the following behavior: an initial maximum occurred at a load of about 454 t prior to buckling of the bars; a drop in load occurred as the bars buckled; finally, when the button had completely crushed there was a significant increase in load until final failure occurred. The final failure occurred by vertical splitting and spalling of the concrete above the joint. For Specimen No. 5, the test was stopped before the ultimate load was reached because of the significant damage that had occurred in the column. The failure load for these specimens is also taken as the load corresponding to an axial shortening of 2.5 mm. For Specimen No. 3, the failure load was 350 t, and for Specimen No. 5 the failure load was 346 t. The average of these failure loads is 44 percent of the average failure load attained by the specimens with full joints.

5.5.5 Summary

Test results on reinforced concrete column-to-column connections show that the connection performs well when the joint is completely filled with grout and concrete and when the joint contains only grout in the 20-mm thick space surrounding the central bearing button. However, when the grout is lacking in the joint, the performance of the connection is very poor. It appears that the grout is essential to good performance of the joint, while the concrete around the corner bars in the joint does not significantly affect the joint behavior.

5.6 EXAMINATION OF BRICK MASONRY ENCLOSURE WALLS

5.6.1 Introduction

The as-built condition of the brick masonry in the Office Building was examined. Visual observations, notes, and photographic documentation of the exterior were made from ground level, and rope climbing gear was used to descend the building to observe the corners, parapet walls, and eighth story walls (fig. 5.6.1). The interior of the parapet walls and the exterior of the penthouse walls were observed from the roof. The interior masonry partitions within the building core also were examined.

Field documentation included crack length and width, slope of walls, and conditions inside of walls. In addition, conditions around windows and the interior corners were observed. Samples of brick, mortar and masonry wall sections were taken and shipped to NBS for laboratory studies.

This section describes cracking observed in the exterior walls, including the parapet walls. Although additional cracking may exist which was not seen during this investigation, most of the major cracks were observed and are discussed below.

5.6.2 Crack Patterns

5.6.2.1 South Elevation

Figure 5.6.2 shows the major cracks observed on the south elevation. Extensive vertical cracking exists at the southwest corner. (Refer to figure 2.4.4 for a review of the details at a typical corner.) A crack runs the full height of the recess in the south face of the corner. Other intermittent, vertical cracks exist between the recess and the edge of the corner.

Horizontal cracks exist at the top of the parapet wall (fig. 5.6.3), extending the full width of the wall. Vertical cracks exist adjacent to the louver opening (fig. 5.6.4), and there is a vertical crack at the east corner. The cracks in the parapet wall vary in width, the widest are 3 mm.

5.6.2.2 East Elevation

Figure 5.6.5 shows the major cracks observed on the east elevation. At the northeast corner (fig. 5.6.6), a vertical crack runs the full height of the recess; near the base, this crack is 3 mm wide (fig. 5.6.7). Other intermittent vertical cracks exist on either side of the recess.

In the parapet wall, a horizontal crack extends from the south corner to the louver opening. Vertical cracks exist on either side of the louver opening.

5.6.2.3 North Elevation

At the north elevation, only the west corner was viewed by descending with climbing gear. There may be cracks in the parapet wall which are not recorded. Figure 5.6.8 shows cracks observed from ground level with the aid of binoculars. Two cracks exist at the northeast corner. One crack is west of the recess in the second story (fig. 5.6.9). The presence of this crack was mentioned in the FBO monthly construction report in August 1984. It is reported to have been one of the first cracks noted in the exterior walls. The other vertical crack is in the recess. It may extend beyond the third story, which is far as it could be discerned with binoculars.

Narrow vertical cracks were noted in four additional piers of the second story at columns H/1A, G/1A, F/1A, and E/1A.

At the northwest corner, two vertical cracks exist below the recess at the bottom of the steps. Another vertical crack exists near the west corner between the window head at the second story and the window sill at the third floor level.

5.6.2.4 West Elevation

Figure 5.6.10 shows the cracks noted on the west elevation. There are horizontal cracks at the top of the parapet wall which extend across the length of the wall. Vertical cracks exist on either side of the louver opening, and there is a horizontal crack below the opening.

At the southwest corner, a vertical crack occurs starting at the top of the recess and extending to the base of the wall at ground level. Other intermittent vertical cracks occur between the recess and the corner of the building.

There is a vertical crack in the pier at column A1/5, at the second story level, which runs from the level of the window sill to the level of the window head (fig. 5.6.11).

5.6.2.5 Interior Examination at Corners

All corners, except the southeast corner, exhibit vertical cracks. The most severe cracking occurs at the northeast and southwest corners, where cracking exists on both sides of the corners. As shown in the elevation views, some cracks within the recesses extend from the base to the seventh story. These corners were carefully observed, both inside and outside, to determine whether the cracking had extended through the backup brickwork. A hacksaw blade, inserted into cracks, served as a probe to determine the approximate crack depth. Cracks were probed from the outside and were found to extend through the exterior wythe but not through the backup brickwork.

The northeast corner, at the second story level (ground level outside), was inspected from the inside. Prior to inspection, the drywall finish and insulation (fig. 5.6.11) had to be removed. The vertical cracks observed on the outside were not seen to extend to the inside. It was noted that head and bed joints of the backup brickwork were not consistently well-filled with mortar. The interior southwest corner was observed at the first story, and no cracks were seen to extend through the backup brickwork.

5.6.2.6 Cracking of Piers

As described above, four piers (fig. 2.4.2 for details) on the north elevation and one on the west elevation showed distress by vertical cracking. In all cases the cracks were in piers anchored to columns; cracks were not seen in any of the intermediate piers. The largest crack was in the pier on the west elevation at column A1/5. This pier also had a vertical crack in the north side approximately 500 mm in length. On the inside, cracks a millimeter in width were seen on both sides of column A1/5 (fig. 5.6.13). These cracks diminish to a hairline at the top and at the bottom of the story. To investigate further the masonry at this column, a 300-mm deep hole was drilled toward the center of the column and examined with a borescope. Cracks were not seen, but voids were noted in the head and collar joints of the backup brickwork.

5.6.3 Eighth Story Walls

At the eighth story, the exterior, windowless masonry walls are designed to be anchored to interior cast-in-place concrete walls (fig. 2.4.5). There was no distress noted in the exterior walls except for one horizontal crack below the louver opening on the west elevation. There was no cracking in the recess at the floorline even though stress concentrations might be expected there.

On the inside, the eighth story walls had 10-mm bars burned off at the face of the concrete. Apparently, the concrete walls were cast using the brick exterior as the form on one side and the bars as ties for the interior forms. Three locations were randomly selected to take 75-mm cores to verify whether the ties extended from the brickwork into the concrete and to determine the nature of the interface between the concrete and the brick masonry. Two of the cores could not be extracted, indicating that the ties extended into the masonry. For the third core, the tie was at an angle so that it was cut during coring. This core was extracted intact; the core broke off through the brick, indicating good bond between the cast-in-place concrete and the exterior brick masonry wall (fig.5.6.14).

5.6.4 Anchors and Ties

The corners are designed to be anchored to columns at three elevations per story (fig. 2.4.13). All corners in stories 4, 5, 6, and 7, plus one corner at the second story and one at the first story were examined from the inside. In 16 corners, anchors were seen at two of three elevations (bottom, middle, or top). At the southeast corner column, fourth story, three anchors were seen; and at the northeast corner column, second story, only one anchor was seen. In a single case, where the floor had been exposed at the column base, an anchor was seen at the floorline.

At the second story level on the east and north elevations, a cover meter was used to verify the location of corner anchors. The cover meter was sensitive to the location of anchors at the recess where the anchors are close to the surface, but was not helpful in locating anchors which were not close to the surface. In one location the presence of an anchor was verified by drilling around the anchor and removing the mortar so that the anchor could be seen from the exterior.

Figure 2.4.2 shows the intended anchorage details at non-corner columns. At three columns, the piers next to columns were cut open form the inside. In each case the anchors were found as specified (fig. 5.6.15).

The existence of masonry wall ties was not systematically verified. However, FBO inspectors who were at the job site indicated that wire ties were placed in every third, fourth, or fifth course.

5.6.5 Other Considerations

The masonry workmanship was generally good for the facing brick, with both head and bed joints well-filled, and the mortar joint face well-tooled to a concave finish. However, the backup masonry does not appear to have well-filled head and collar joints, and many bed joints are not completely filled. This is based upon viewing the walls from the inside, cutting out wall sections, and using a borescope.

Water penetration through the masonry to the interior does not appear to be a problem. However, a great deal of efflorescence has occurred, indicating that water has entered the walls over time and has carried soluble salts in the masonry to the surface. Most of the water penetration probably occurred during the time the building lacked the temporary roof currently in place.

As shown in figure 2.4.4, the drawings required that insulation for the drain pipes for the snow melting rooms be placed between the pipes and the exterior walls. Inspection of a number of corners revealed that this insulation was incorrectly placed on the inside of the pipes (fig. 5.6.12). It was reported that during winter some drains had frozen and caused problems of water backup at the eighth story roof.

5.7 EXAMINATION OF PARAPET AND PENTHOUSE WALLS

Parapet and penthouse walls are discussed together because they intersect each other, and there are similarities in the exposure they experience and their overall performance. However, parapet walls experience more severe exposures and have less support than the penthouse walls.

5.7.1 Observed Cracking

Access to each section of the roof is through the penthouse (refer to figure 2.4.8 for the plan of the penthouse and parapet wall). Figure 5.7.1 shows the appearance of a typical section as viewed from the penthouse roof. The cracking observed in each section is described. In addition to visual inspection, brick were removed from the penthouse and parapet walls, and three openings were made in the temporary waterproof membrane at the roof and the top of the parapet walls.

Southwest Section

In the east penthouse wall there are three horizontal cracks and a small vertical crack. Also there are some horizontal cracks in the north penthouse wall. Vertical cracks exist on each side of the diagonal wall of the snow melting room. There is a diagonal crack and open joints at the intersection of the penthouse wall and the west parapet wall (fig. 5.7.2).

Southeast Section

Small vertical and horizontal cracks exist in the west wall of the penthouse. Narrow vertical cracks occur in the mortar joints on both sides of the diagonal wall of the snow melting room.

Northeast Section

Numerous cracks were seen in the northeast section. There is a horizontal crack in the penthouse wall on the northeast. Considerable cracking occurs on both sides of the intersection of the west penthouse wall and the penthouse recess. In the recess, a vertical crack extending the full height of the wall exists below the beam support. About a meter down from the top of the wall a horizontal crack also occurs in this vicinity. In the north parapet wall a horizontal crack extends from the penthouse to the diagonal wall of the snow melting room. On the east parapet wall there is a vertical crack at the intersection of the snow melting room with the parapet wall. There is a small crack at the base of the opening to the snow melting room.

Northwest Section

There is a horizontal crack which extends the full length of the east penthouse wall; it continues into the north parapet wall, and ends at the opening of the snow melting room. There is a vertical crack at the intersection of the diagonal wall with the west parapet wall. A horizontal and a vertical crack exist in the west parapet wall, and there is a horizontal crack extending less than halfway across the south penthouse wall.

5.7.2 Removal of Brick

In order to better understand the conditions of the penthouse and parapet masonry walls, brick were removed in the vicinity of the most severe cracking.

A brick was removed at the intersection of the vertical and horizontal cracks in the penthouse wall in the northeast section (fig. 5.7.3). This was very difficult to do by hand, as the mortar was extremely hard and well-bonded to the brick in spite of the adjacent crack. Head and bed joints were well-filled, but the collar joint was mostly void of mortar. Ice was seen in the collar joint and open bed joint of the backup brick. Other adjacent bed and head joints were open to a considerable extent. The collar joint immediately above the removed brick was probed and found to be open. A steel wall tie was found in the bed joint.

A second brick was removed from the northwest section, where a 3-mm wide horizontal crack exists at the intersection of the snow melting room and the north parapet wall. At this location, the crack was one wythe in depth. Using a plumb line, it was found that the wall was out of plumb about 3 mm for every 400 mm. The brick taken from this location was easily removed because the horizontal crack beneath the brick was quite wide. The head and bed joints were well filled, and the collar joint was better filled than the penthouse collar joint discussed above.

5.7.3 Opening of Temporary Roof and Cap

Permanent roofing, coping, and flashing have not been placed. There is temporary, singly-ply membrane over the top of the parapet walls and the penthouse roof. The membrane was opened at three locations to observe the condition of the masonry. The locations were immediately above where brick were removed, as described above, and at the intersection of the penthouse walls defining the northeast corner of the cruciform.

Where the roof was opened above the brick removed from the penthouse wall, little could be seen as the concrete or mortar had been placed flush with the top of the wall. Cracks were not observed.

At the intersection of the two penthouse walls, it was noted that the concrete deck had been placed directly against the outside facing brick wythe. No cracks were seen.

Temporary rigid insulation was found in the cavity below the covering over the parapet wall. Removal of the insulation revealed the reinforcement for a bond beam. The cage included four bars and appeared to be consistent with the design (fig. 2.4.10). To the east, the cavity had been filled with concrete to complete the bond beam. There was a longitudinal crack at the base of the cavity and the exterior facing brick.

5.8 EXAMINATION OF CORE WALLS

Except for one room per story, the partition walls in the core of the building are of brick masonry. These walls were not intended to serve a structural function. However, they can play a critical role in providing alternate load paths in the event of a column failure (see section 4.5). Wall specimens were removed and brought back to the NBS laboratory to obtain estimates of the compressive and shear resistance of these walls. The results of these tests are presented in section 5.10.7.

Brick masonry core walls were visually inspected from one side in stories one through eight. The walls selected for observation are noted in figures 5.8.1 and 5.8.2. The numbers are used to identify the walls and the arrows indicate the direction from which they were viewed.

The walls were examined to gain a qualitative impression of their potential performance as load bearing elements. The walls were classified into one of three categories: G = good; F = fair; P = poor. Table 5.8.1 summarizes the visual observations. In addition to the overall quality, there were a number of reoccurring conditions which were considered in the evaluation. These are given a number 1 through 7 as shown at the bottom of table 5.8.1. These walls were specified to be plastered with portland cement sand plaster which would contribute to their structural integrity. Figures 5.8.3 shows an example of good quality wall in the stairwell. Figure 5.8.4 shows the walls at the elevators. Figure 5.8.5 shows a wall with duct penetrations.

5.9 EXAMINATION OF WINDOWS, SOFFITS, AND SILLS

The condition of the windows and their surroundings is important to the overall performance of the building envelope. The entrance and accumulation of water can lead to corrosion of metal and disruption of masonry due to freezing and thawing.

5.9.1 Condensation and Frost

During the first site visit, windows and their surroundings were examined to ascertain water tightness and structural stability. Of the 334 typical windows, 34 from the second through seventh story were randomly chosen for detailed examination. Prior to this visit cold weather was experienced with temperatures as low as -20°C.

The cavities above the window soffits (fig. 2.4.7) were examined. In the second and third stories, the areas around the windows were partially or completely finished. Thus, the cavity was examined in only 20 of the windows. In all cases, frost was found in the space between the insulation and lintels;

in some cases frost buildup was extensive. In the upper stories there was corrosion of bolts used to anchor the soffit to the lintel. In the lower stories these bright metal bolts were not corroded.

A second inspection was made of the windows and the surrounding construction on February 25, 1987. Every third window of those previously inspected was selected for inspection. Weather prior to this inspection had been mild with daytime temperatures above freezing as was the temperature during the inspection. Frost was seen above the north windows and one of the two west windows. There was no frost in cavities above the other windows. However, there was considerable water and wet insulation and in some cases ice buildup at the window heads. An example of the water seen in this cavity can be seen in figure 5.9.1. The lack of frost on the south and east elevations is attributable to the warmer temperatures due to the sun warming up the fascia of the soffit. On the north elevation, direct rays of the sun had not reached these windows; thus frost continued to occur.

5.9.2 Lintels and Spandrel Beams

A review of the bearing of the lintels indicated that, generally, good bearing was provided with well-filled mortar joints. In a few exceptions, the joints were not completely filled, but the bearing provided appeared to be sufficient to carry loads from above.

Spandrel beams above the windows were covered with plaster, and it was difficult to see the joints between the beams and the masonry below. However, no cracks were noted, indicating that the beams are well built into the masonry.

5.10 LABORATORY INVESTIGATION OF MASONRY MATERIALS

5.10.1 Test Specimens

Samples of mortar were taken from partition walls in the fourth story, and unused Soviet building brick were randomly selected from each story. These specimens were shipped to NBS for laboratory investigations.

In order to obtain a quantitative evaluation of the structural potential of the masonry core walls, four wall samples were cut out (fig. 5.10.1) and shipped to NBS. These specimens were one wythe in thickness, nominally 120 mm, and not less that 500 × 500 mm in face area.

Facing brick from the original production run in early 1982 were still available from the U.S. producer and were shipped to NBS to determine the physical properties of the individual units and of masonry specimens fabricated with the units.

5.10.2 Standard Tests of Brick

Laboratory tests were conducted on units of facing brick and building brick to determine the following properties: size, weight, void area, initial rate of absorption, absorption, saturation coefficient, compressive strength, and efflorescence. The tests were conducted following, in general, the procedures given in ASTM C 67-85 [5.3]. Test results for facing brick were compared with the requirements of ASTM C 216-85a [5.4] and results for building brick were compared with the requirements of ASTM C 62-85a [5.5].

Figure 5.10.2 shows the appearance of the facing brick and the building brick. Table 5.10.1 summarizes the dimensions and weights of individual units. Facing brick and building brick units are nominally 250 mm long, 120 mm wide and 65 mm thick. The building brick have less void volume and more mass than the facing brick.

Table 5.10.2 gives the results of the initial rate of absorption tests. The results for the facing brick are less consistent than for the building brick. However, neither type of brick had an initial rate of absorption greater than 30 g/min/194 cm^2. Thus, according to recommendations in ASTM C 216-85a and C 62-85a, neither type of brick require prewetting prior to laying.

Table 5.10.3 gives the results of absorption tests and gives the saturation coefficient. The saturation coefficient was calculated based on the results of these two absorption tests. For all test specimens, the absorption after the 5-h boiling test was less than the 17.0 percent maximum given in ASTM C 216-85a and in ASTM C 62-85a for brick exposed to severe weathering. The saturation coefficient for the building brick specimens met the requirements for exposure to moderate weathering but did not meet the requirements for exposure to severe weathering. The saturation coefficient for the facing brick specimens met the requirements for exposure to severe weathering.

Table 5.10.4 gives the results of flatwise compression tests on saw-cut, half brick specimens; strength is based on the gross area. For exposure to severe weathering, the ASTM specifications require a minimum compressive strength of 211 kg/cm^2. The average compressive strength for both the facing and building brick exceeded 570 kg/cm^2, which is far above the required minimum value.

Five facing brick were tested for efflorescence. After seven days of testing, there was slight efflorescence on one brick and even less than this amount on another specimen. No efflorescence was seen on the other specimens. No attempt was made to identify the type of salt deposited on the two facing brick. The facing brick specimens satisfy the requirements for efflorescence given in ASTM C 216-85a. Only one full and two half building brick specimens were available for testing. After three days, a considerable amount of efflorescence was visible on the whole brick. After seven days, the efflorescence was extensive. No efflorescence was observed on the two half building brick. There are no efflorescence requirements for building brick in ASTM C 62-85a.

5.10.3 Modulus of Elasticity of Brick

Half-brick specimens of facing brick and of building brick were instrumented with electrical resistance strain gages and tested flatwise in compression to determine the secant modulus of elasticity at 50 percent of ultimate load. The results are given in table 5.10.5.

5.10.4 Moisture Expansion Reversibility of Brick

At present, there is no standard test method to measure moisture expansion of brick. It has been suggested [5.6] that it can be determined by heating the brick to 700°C so as to reverse the expansion that might have occurred between the time of manufacture and the time of sampling. Robinson [5.7] suggests that a desorption temperature between 400 and 500°C satisfies the requirements to remove moisture expansion without crystalline inversions.

The procedure used in this study was to place brick in a furnace and raise the furnace temperature to 550°C. It took about four hours for the furnace to reach 550°C. The brick were heated at 550°C for 24 hours and then the furnace was turned off. The brick were removed from the furnace after 16 to 18 hours while the furnace was at about 60°C. The brick were placed in a desiccator for 4 to 5 hours before reaching room temperature (21°C). Prior to heating, stainless steel gage points were attached to the units using castable alumina fine and water. The change in length of the test brick was measured using a dial gage comparator. Multiple readings were taken per brick.

Table 5.10.6 lists the values of contraction due to heating. The measured average coefficient for facing brick is 0.00036 and for building brick it is 0.00029. It is reported that predicted moisture expansion coefficients after 5 years range from 0.00006 to 0.0015 with nearly 80 percent of the sets having expansion less than 0.0006 [5.6]. The measured values are consistent with the expectations.

Provided no other mechanism is present, it has been suggested that the ratio of masonry expansion to brick expansion is approximately 0.6 [5.6]. This gives predicted 5-year masonry expansion coefficients ranging from less than 0.00004 to 0.0009, while about 80 percent of masonry would have coefficients less than 0.0004. Plummer, on the other hand, has recommended a coefficient of moisture expansion of 0.0002 for masonry [5.8]. If the ratio of 0.6 is applied to the measured coefficients, the results are 0.00021 for facing brick masonry and 0.00017 for building brick masonry, these are in agreement with Plummer.

5.10.5 Coefficient of Linear Thermal Expansion of Brick Masonry

The coefficient of linear thermal expansion was measured for five wall panels constructed in the laboratory and one section of a wall panel from the Office Building. The laboratory panels were two brick wide, seven courses high, and one brick in thickness. Two wall panels were made with facing brick and Type M mortar (ASTM C 270-86) [5.9], two were made with facing brick and Type S mortar, and the fifth was made with building brick and Type M mortar. The building brick wall section from the site was two brick in length and three courses high.

Horizontal and vertical length measurements were made at different temperatures using 10-inch and 5-inch Whittemore Gages. Brass gage points were set in drilled holes using an epoxy resin adhesive. The 10-inch gage lengths included one mortar joint in the horizontal direction and three mortar joints in the vertical direction. The 5-inch gage lengths in the vertical direction included two mortar joints. Measurements were made at temperatures of about -18°, 2°, 21°, and 35°C.

Table 5.10.7 gives the coefficient of linear thermal expansion as determined from measurements at -18° and 35°C. The measured values of 6.4×10^{-6}/°C for facing brick and 7.0×10^{-6}/°C for building brick are consistent with recommended values for design of 6.5×10^{-6}/°C [5.10].

5.10.6 <u>Tests of Mortar Samples</u>

Specimens of hardened mortar taken from masonry in the field were analyzed to determine their composition and physical properties. The information assisted the analysis of the masonry walls and the preparation of masonry specimens for laboratory testing.

5.10.6.1 Composition

Two mortar specimens were examined by scanning electron microscope (SEM) to determine size distribution of sand, and proportions of sand, paste and voids. Preliminary examination indicated that the particle size distribution of the sand is within the range of mortar sand according to ASTM C 144-84 [5.11]. The SEM examination indicated volume fractions of 42 percent sand, 40 percent cement paste (unhydrated cement, hydrated cement, and fine voids and pores), and 18 percent large voids.

Cement content of the mortar was determined chemically according to ASTM C 85-66(1973) [5.12]. The mortar was ground to pass a 200-mesh sieve, then partially dissolved in HCl, followed by NaOH. Cement content may be calculated either using the amount of calcium or the amount of silica soluble in the HCl and NaOH solutions. The test method provides dissolution of most or all cement and hydrated cement constituents, and minimizes dissolution of sand. Ideally, samples of the aggregate are treated according to the same method to correct for calcium and silica derived from aggregate constituents, but the sand used in the present mortar was not available. In addition, if the mortar contains added lime, the cement content based on the level of soluble calcium will be too high.

To calculate cement content, the levels of calcium or silica in the cement must be known or assumed. In the present study, it was assumed that portland cement was used, containing 60 percent by weight CaO and 20 percent SiO_2, and that the cement contained no pozzolanic material. With these assumptions, the cement content is estimated as 40 percent by weight based on the soluble calcium and as 55 percent based on the soluble silica.

5.10.6.2 Compressive Strength

To determine compressive strength, seven specimens were cut from the pieces of mortar taken from the site. The thickness of the samples varied as noted in table 5.10.8. The specimens were capped with high strength gypsum plaster and tested in compression. The strength of the samples was converted to that of a 50-mm mortar cube based on data of relative compressive strength as a function of the height/width ratio of the mortar samples. These unpublished data were provided by the National Concrete Masonry Association, and they are in agreement with conversion factors given by the Bureau of Reclamation [5.13] for relative cylinder strength as a function of height/diameter values. The measured compressive strengths are divided by the factors given in table 5.10.8 to obtain the equivalent strength of a 50-mm cube. The average estimated compressive strength of 50-mm cubes is 377 kg/cm^2.

5.10.6.3 Efflorescence

One sample of hardened mortar taken from the field was tested for efflorescence using the same procedure as used for brick. Considerable efflorescence was observed.

5.10.6.4 Mortar Type

Based on laboratory testing and field observations, it is concluded that the Soviet mortar is similar to Type M mortar as specified in ASTM C 270-86 [5.9]. Type M mortar has a high compressive strength and a high portland cement content (1 part cement:2-1/4 to 3 parts sand)

5.10.7 Structural Testing of Assemblies

Compressive strength, modulus of elasticity, flexural strength, and diagonal tensile (shear) strength were determined for masonry specimens fabricated in the laboratory and for specimens cut from wall panels obtained from the Office Building. Specimens were made with facing brick and building brick and with Type S and Type M portland cement lime mortars [5.9]. Laboratory prepared specimens include prisms that were seven brick high and one brick wide and panels that were seven brick high and two brick wide (fig. 5.10.3).

5.10.7.1 Compressive Strength and Elastic Modulus

Compressive strength was determined according to ASTM E 447-84 [5.14]. Vertical deformations during loading were measured with linear variable differential transformers (LVDT) using a 330-mm gage length. The secant modulus of elasticity was determined at 50 percent of the ultimate load. Table 5.10.9 gives the results for prisms built with facing brick and Type M and Type S mortar.

Compressive strength of masonry constructed from Soviet building brick was determined for prisms built with Type M mortar and for specimens from the wall panels taken from the site. Dimensions of specimens cut from the wall panels varied because of variations in the available intact uniform sections of wall. Table 5.10.10 gives the results of the tests on building brick specimens.

5.10.7.2 Flexural Bond Strength

Flexural bond strength was determined according to ASTM E 518-80 [5.15] using third-point loading and a clear span of 483 mm. All test prisms were made in the laboratory: for facing brick Type S and Type M mortars were used; for building brick Type M mortar was used. No flexural tests were made of specimens cut from samples of in-place brickwork. The flexural strength (modulus of rupture) was computed on a gross area basis neglecting the weight of the prisms. Table 5.10.11 gives the results of these tests.

5.10.7.3 Diagonal Tension (Shear) Strength

Diagonal tension tests were performed on two panels cut from the interior walls of the Office Building and on five panels constructed in the laboratory. The procedure for testing described in ASTM E 519-81 [5.16] was modified to accommodate the size of the panels, which were approximately 460 mm square (fig. 5.10.4) as compared with the standard size of 1200 mm square. Diagonal tensile strength, or shear strength, was calculated on a gross area basis and the results are shown in table 5.10.12.

5.11 SUMMARY

The objective of the field investigation was to obtain information on those aspects of the as-built structure and its enclosure which would impact on the performance of the Office Building. A review was made of the main structural components and the building envelope. The following conclusions are drawn from the results of the site investigation and laboratory studies.

The load-bearing system is, on the whole, constructed as expected from the review of the construction documents. No significant deficiencies were noted in the structural steel elements. In-place hardness tests and laboratory tests of steel samples gave no indications that materials of less strength than specified were used. Examination of welded connections revealed general conformance with the design. Tests of core samples taken from various reinforced concrete structural elements indicate compressive strengths in excess of the specified values.

The as-built conditions of some joints between precast elements were found to have significant deviations from the design. Notable among these were the incompletely grouted joints between segments of reinforced concrete columns. Laboratory tests revealed that lack of grout between column segments causes a drastic reduction in the column capacity. Deficiencies were also found in the horizontal and vertical joints of the shear wall system.

A detailed visual examination of the exterior brick masonry walls was performed from ground level and by descending the building. The exterior walls showed vertical cracking on all sides of the building. Vertical cracks on two corners extend from ground elevation to the top of the seventh story. Vertical cracks were seen in five piers between windows at second story columns. The most severe horizontal cracking occurs in the parapet walls. Vertical cracks were also seen in the parapet walls. The penthouse walls show both horizontal and vertical cracks, but they are fewer in number and smaller in size than the cracks in the parapet walls.

Inspection of brick partitions in the core area revealed that many of these walls are either unfinished or otherwise incomplete at the top so that they do not fit tightly in the building structure. Wall specimens and Soviet-produced building brick were brought back to the NBS laboratories for testing to provide physical property information needed for analyzing the as-built construction. Tests were also performed on specimens facing brick.

Site investigation also indicated that fireproofing is missing from a number of beam-column connections. Insulation is placed improperly in some corner piers and threatens to allow freezing of the drains for the snow melting areas of the roof. The cavities above the windows soffits are not insulated. This allows ice build-up and possible damage to the windows and walls.

5.12 REFERENCES

5.1 Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A 370-77, Vol. 03.01, American Society for Testing and Materials, Philadelphia, Pa.

5.2 Standard Method of Obtaining and Testing Drilled Cores and Sawn Beams of Concrete, ASTM C 42-85, Vol. 04.02, American Society for Testing and Materials, Philadelphia, Pa.

5.3 Standard Methods of Sampling and Testing Brick and Structural Clay Tile, ASTM A 67-85a, Vol. 04.05, American Society for Testing and Materials, Philadelphia, Pa.

5.4 Standard Specification for Facing Brick (Solid Masonry Units Made From Clay or Shale), ASTM C 216-85a, Vol. 04.05, American Society for Testing and Materials, Philadelphia, Pa.

5.5 Standard Specification for Building Brick (Solid Masonry Units Made From Clay or Shale), ASTM C 62-85a, Vol. 04.05, American Society for Testing and Materials, Philadelphia, Pa.

5.6 Lomax, J. and Ford, R.W., Investigations into a Method for Assessing the Long Term Moisture Expansion of Clay Brick, Transactions and Journal of the British Ceramics Society, Vol. 82, No. 3, pp. 79-82, 1983.

5.7 Robinson, G.C., The Reversibility of Moisture Expansion, American Ceramics Society Bulletin, Vol. 64, No. 5, pp. 712-15, 1985.

5.8 Plummer, H.C., <u>Brick and Tile Engineering</u>, Structural Clay Products Institute, Nov. 1962.

5.9 Standard Specification for Mortar for Unit Masonry, ASTM C 270-86, Vol. 04.05, American Society for Testing and Materials, Philadelphia, Pa.

5.10 Technical Notes on Brick Construction, 18 - Differential Movement, Causes and Effect, Brick Institute of America, Reston, Va., 1963.

5.11 Standard Specification for Aggregate for Unit Masonry, ASTM C 144-84, Vol. 04.05, American Society for Testing and Materials, Philadelphia, Pa.

5.12 Standard Test Method for Cement Content of Hardened Portland Cement Concrete, ASTM C 85-66(1973), Vol. 04.02, American Society for Testing and Materials, Philadelphia, Pa.

5.13 Concrete Manual, U.S. Department of the Interior, Bureau of Reclamation, 7th Edition, Denver, Colo., 1963.

5.14 Standard Test Methods for Compressive Strength of Masonry Prisms, ASTM E 447-84, Vol. 04.07, American Society for Testing and Materials, Philadelphia, Pa.

5.15 Standard Test Methods for Flexural Bond Strength of Masonry, ASTM E 518-80, Vol. 04.07, American Society for Testing and Materials, Philadelphia, Pa.

5.16 Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages, ASTM E 519-81, Vol. 04.07, American Society for Testing and Materials, Philadelphia, Pa.

Table 5.3.1. Summary data from floor topping cores

Floor/Location(ID)	Condition	Topping Depth (mm)	Mesh Spacing (mm)	Mesh Depth (mm)
2/South(2-1)	Drummy	50	100 x 100	45
2/South-West(2-2)	Solid	53	100 x 100	45
3/East(3-2)	Drummy	70	100 x 100	60
3/South(3-1)	Solid	42	100 x 100	35
4/South-East(4-2)	Drummy	100	100 x 100	85
4/West(4-1)	Solid	92	150 x 150	75
5/North-East(5-1)	Drummy	73	150 x 150	65
5/East(5-2)	Solid	78	150 x 150	68
6/East(6-2)	Drummy	95	100 x 100	80
6/North(6-1)	Solid	92	150 x 150	75
7/South(7-1)	Drummy	60	100 x 100	50
7/South-East(7-2)	Solid	60	150 x 150	45

Table 5.3.2. Results of tensile tests on wire mesh samples

Specimen Location Floor	Yield Strength (kg/cm^2)	Ultimate Strength (kg/cm^2)	Percent Elongation
2	5790	6390	7.8
2	5170	5970	6.2
3	5200	5990	6.2
5	4960	6100	4.7
6	4420	5530	6.2
7	5760	6890	6.2
8	5150	5970	---

Table 5.3.3. In-place hardness data for higher strength steel members

Floor	Member	Section	Element	Hardness Value - Ld Median	Corrected
PH	E/1A-E/4A	b11	flange	480 u	456
PH	D/1A-D/4A	b12	flange	500 u	476
PH	F1/1A-F1/4A	b12	flange	520 u	498
8	J/4-J/5	b30	flange	428 d	428
			web	416 h	405
8	F/1A-F/4A	b5	flange	440 u	415
8	A1/4A-C1/4A	b8a	flange	473 d	473
8	C/6B-C/9A	b6	flange	421 d	421
6	C/1A-C/4A	b8	flange	416 d	416
6	A1/4A-C1/4A	b10	flange	417 d	417
5	H/1A-H/4A	b8	flange	477 d	477
5	G2/4A-J/4A	b10	flange	411 d	411
4	H/1A-H/4A	b8	flange	460 d	460
4	G2/4A-J/4A	b10	flange	404 d	404
3	C/1A-C/4A	b8	flange	396 d	396
			web	390 h	378
3	A1/4A-A1/C1	b10	flange	392 d	392
			web	407 h	396
2	C/1A-C/4A	b6	flange	444 d	444
			web	394 h	372
2	A1/4A-C1/4A	b9a	flange	392 d	392
			web	392 h	380
1	H/6B-H/9A	b6	flange	422 d	422
			web	392 h	380
1	G2/6B-J/6B	b8	flange	403 d	403
			web	416 h	405

Table 5.3.4. In-place hardness data for ordinary strength steel members

Floor	Member	Section	Element	Hardness Value - Ld Median	Corrected
Roof	Along H	b17	flange	455 u	431
Roof	G2/5-J/5	b24	flange	390 u	363
PH	D1/6B-E/6B	b20	web	376 h	364
PH	F/4A-F1/4A	b20	web	370 h	358
8	F/3A-G/3A	b22	web	370 h	358
6	A1/4-A1/5	b18	flange	390 d	390
5	J/4-J/5	b18	flange	387 d	387
1	E/1A-F/1A	b22	web	389 h	377

Table 5.3.5. Tensile test data for steel beam samples

Specimen Number/Beam Section	Type of Steel	Element	Yield Strength (kg/cm^2)	Ultimate Strength (kg/cm^2)	Percent Elongation	Hardness Ld-value
1/b8a	Higher	Flange	5300	6170	17.1	462
2/b8a	"	"	5350	6180	16.7	468
3/b8a	"	"	5330	6210	17.6	469
4a/b8a	"	Web	****	****	****	401
4b/b8a	"	Web	****	****	****	409
5/b26a	Rolled	Flange	3330	4810	26.8	398
6/b26a	"	"	3220	4783	28.7	386
7/b18	Ordinary	Flange	3590	5180	21.7	402
8/b18	"	"	3560	5150	23.2	399
9/b18	"	"	3590	5180	22.2	396
10/b18	"	Web	3670	6120	17.6	436

Table 5.3.6. Chemical analysis of steel samples (values in percent)

Element	Higher strength Flange	Higher strength Web	Rolled shape Flange	Ordinary Strength Flange	Ordinary Strength Web
C	0.13	0.087	0.17	0.079	0.11
P	0.021	0.021	0.037	0.026	0.025
S	0.025	0.021	0.028	0.019	0.022
Mn	1.44	1.44	0.66	1.39	1.73
Si	0.40	0.90	0.061	0.63	0.74
Ni	0.030	0.051	0.052	0.046	0.096
Cr	0.030	0.025	0.025	0.025	0.072
V	0.087	0.003	0.001	0.004	0.007
Ti	0.003	0.019	0.001	0.007	0.028
Mo	0.003	0.004	0.003	0.004	0.013
Cu	0.096	0.19	0.027	0.084	0.089
Co	0.004	0.010	0.013	0.010	0.010
Al	0.025	0.005	0.002	0.004	0.006
Nb	0.002	0.003	0.001	0.002	0.003
Zr	0.002	0.002	0.001	0.002	0.002
As	0.009	0.001	0.11	0.001	0.001

Table 5.3.7. Measurements of fireproofing thickness

Floor	Member	Element	Thickness (mm)
5	G2/4A-J/4A	flange	38
3	C/1A-C/4A	flange	64
3	A1/4A-C1/4A	flange	50
2	C/1A-C/4A	flange	50
2	A1/4A-C1/4A	flange	32
		web	50
1	G2/6B-J/6B	flange	25
		web	32

Table 5.3.8. Results of tests of concrete cores from columns

Type	Location	Dia. (cm)	Length (cm)	Density (kg/m^3)	Ultimate Load (kg)	L/D	Cor. Factor	Corrected Strength (kg/cm^2)
5C	B-A1/5	7.16	12.52	2316	23154	1.75	0.98	564
5C	B-E/4B	7.17	14.22	2367	27830	1.98	1.00	690
5C	B-E/6A	7.16	12.24	2314	26241	1.71	0.98	640
5C	B-F/4B	7.02	13.79	2289	19113	1.96	1.00	494
5C	B-J/6	6.95	10.85	2384	22428	1.56	0.96	567
5C	1-F/4B	7.00	13.79	2367	25061	1.97	1.00	652
5C	1-F/6A	7.00	13.72	2339	22291	1.96	1.00	580
5C	2-A1/5	7.16	13.97	2335	26673	1.95	1.00	664
5C	2-C1/6	6.96	13.59	2341	20929	1.95	1.00	551
5C	2-G2/6	7.15	14.15	2328	22473	1.98	1.00	561
3C	5-F/4B	7.12	14.17	2273	25242	1.99	1.00	635
3C	6-C1/5	7.08	11.48	2272	17842	1.62	0.97	439
3C	6-G2/6	7.11	13.97	2293	20793	1.97	1.00	525
2C	5-A1/9A	7.19	12.95	2271	18069	1.80	0.98	436
2C	5-J/1A	7.16	14.10	2328	25969	1.97	1.00	646
2C	6-A1/3	7.09	14.07	2240	14710	1.98	1.00	373
2C	6-A1/9A	7.19	14.02	2308	24607	1.95	1.00	606
12S	B-C1/6B	7.16	12.52	2316	22655	1.75	0.98	552
12S	B-G2/6B	6.98	11.63	2235	19113	1.67	0.97	485
9S	2-G2/4A	7.17	14.02	2260	13438	1.96	1.00	333
9S	3-D/9A	7.00	6.22	2428	25260	0.89	0.84	552
9S	4-D/9A	7.00	7.12	2357	23740	1.02	0.87	538
9S	4-H/1A	6.99	6.47	2343	25420	0.93	0.85	564
9S	4-G/9A	7.01	6.21	2395	24020	0.89	0.84	523
9S	5-D/9A	7.02	6.50	2375	24452	0.93	0.85	537
9S	5-C/9A	6.99	6.37	2311	23430	0.91	0.85	519
7S	7-F/4A	6.97	14.07	2322	18251	2.02	1.00	479
7S	7-G2/6B	6.97	14.05	2303	21701	2.02	1.00	569
5S	8-G2/6B	6.97	12.07	2278	19386	1.73	0.98	498

Table 5.3.9. Tensile Test Data for 32-mm Reinforcing Bar Samples

Column	Yield Strength (kg/cm^2)	Ultimate Strength (kg/cm^2)	Percent Elongation	Hardness Ld-value
J/3(1)	4310	6840	27	453
J/3(2)	4190	6630	33	442
J/4	4090	6700	45	449
J/5	4200	7170	28	478
J/6	4190	7150	25	476

Table 5.3.10. Results of tests of concrete cores from shear walls

Story	Location	Dia. (cm)	Lengths (cm)	Density (kg/m^3)	Ultimate Load (kg)	L/D	Cor. Factor	Corrected Strength (kg/cm^2)
4	G2/6B-G2/6	7.18	14.50	2236	19704	2.02	1.00	487
5	F1/6B-G/6B	7.18	13.84	2268	17570	1.93	0.99	430
5	D1/4A-E/4A	7.18	13.21	2277	19477	1.84	0.99	476
6	G2/5-G2/6	7.20	13.89	2255	18296	1.93	0.99	446
6	C1/4A-D/4A	7.18	13.79	2278	18841	1.92	0.99	461
7	G/6B-G2/6B	7.17	12.80	2298	13211	1.78	0.98	321
7	F/6B-F1/6B	7.17	14.00	2256	16072	1.95	1.00	398

Table 5.4.1. Summary of column joint investigation

a) Joints of heavily loaded and other columns

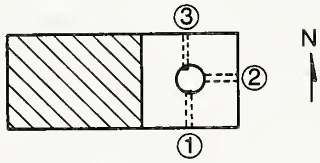
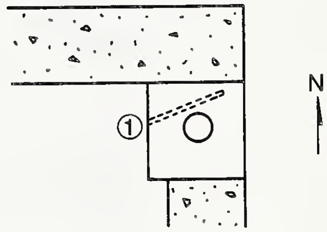
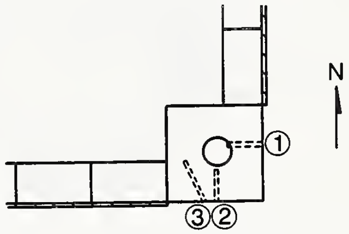
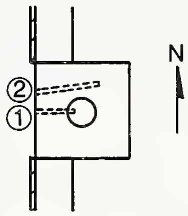
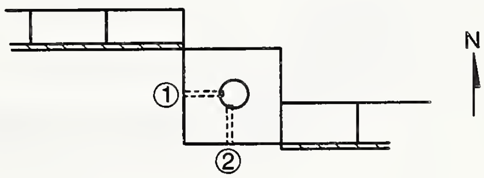
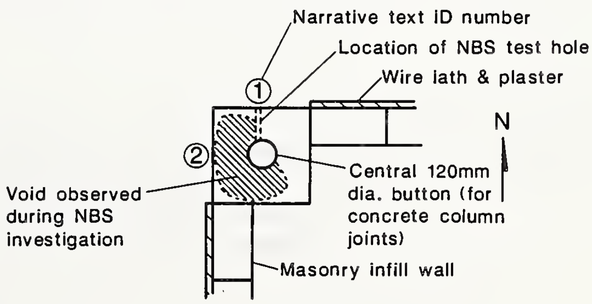
Column Designation	Notes	Joint Detail
B-A1/5	1. 100mm hole; no voids. 2. 110mm hole; no voids. 3. 110mm hole; no voids.	
B-E/4B	1. 140mm hole; no voids. Only accessible from one side.	
B-E/6A	1. 95mm hole; no voids. 2. 95mm hole; no voids. 3. 95mm hole; no voids.	
B-F/4B	1. 140mm hole; no voids. 2. 170mm hole; no voids. back and side not accessible.	
B-J1/6	1. 120mm hole; no voids. 2. 120mm hole; no voids.	
1-F/4B	1. 95mm hole; no voids. 2. 10mm hole breaks open into large void. Center button plainly visible.	

Table 5.4.1. Summary of column joint investigation

a) Joints of heavily loaded and other columns (continued)

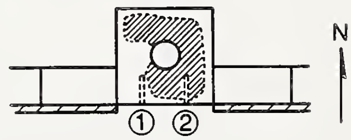
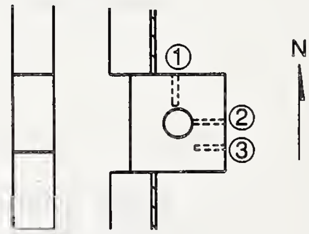
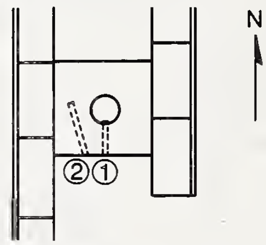
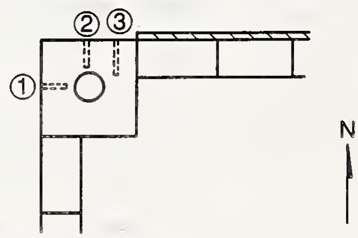
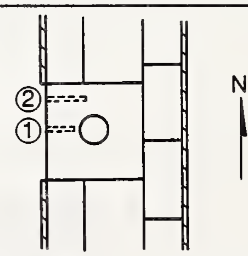
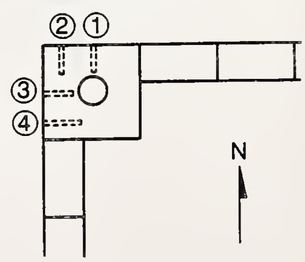
Column Designation	Notes	Joint Detail
1-F/6A	1. 95mm hole; no voids 2. 15mm wall; breaks open into large void. No grout in joint. Also, 1mm gap under half of button.	
2-A1/5	1. 65mm hole; no voids. 2. 65mm hole; small horizontal void on east face but check with wire show it to be local. 3. 65mm hole; no voids.	
2-C1/6	1. 140mm hole; no voids. 2. 130mm hole; no voids.	
2-F/4B	1. 95mm hole; no voids. 2. 180mm hole; no voids. 3. 95mm hole; no voids.	
2-G2/6	1. 100mm hole; no voids. 2. 100mm hole; no voids.	
5-F/4B	1. 100mm hole; no voids. 2. 100mm hole; no voids. 3. 100mm hole; no voids. 4. 100mm hole; no voids.	

Table 5.4.1. Summary of column joint investigation

a) Joints of heavily loaded and other columns (continued)

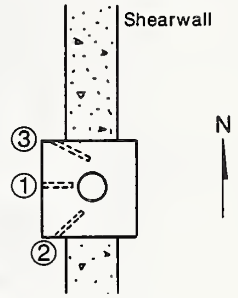
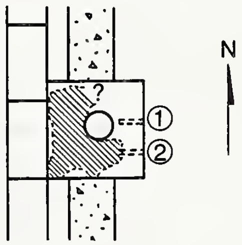
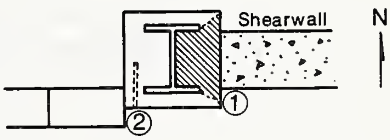
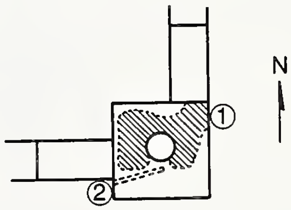
Column Designation	Notes	Joint Detail
6-C1/5	1. 160mm hole; no voids 2. 160mm hole; no voids 3. 160mm hole; no voids	
6-G2/6	1. 145mm hole; breaks open into void. 2. 100mm hole; breaks into large void extending to south and west faces.	
7-F/6B	1. Large gap on east side of joint; no drilling necessary. Central steel core plainly visible. 2. 160mm drill; no voids	
8-E/6A	1. 10mm hole; breaks into large to void; rebar pocket is also exposed (no concrete). 2. 190mm hole; no voids.	

Table 5.4.1. Summary of column joint investigation

b) Joints along column line G2/5

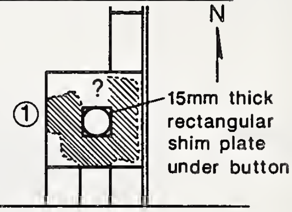
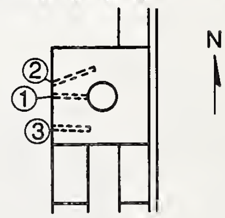
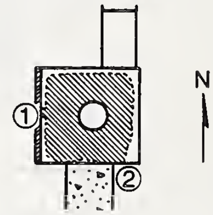
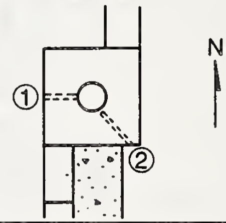
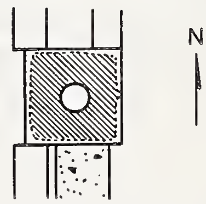
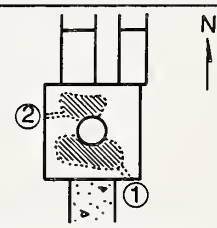
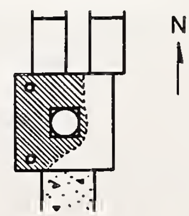
Column Designation	Notes	Joint Detail
2-G2/5	1. Access with chipping hammer (20mm gap void hidden behind plaster covered wire lath, as were all G2/5 columns except for 8th floor).	
3-G2/5	1. 180mm hole; no voids. 2. 190mm hole; no voids. 3. 180mm hole; no voids.	
4-G2/5	1. 5mm hole; breaks into void. No grout at all in 20mm gap. 2. Internal void visible from small surface hole without drilling.	
5-G2/5	1. 170mm hole; no voids. 2. 150mm hole; no voids.	
6-G2/5	1. 10mm hole; breaks into complete void around central button. No grout at all.	
7-G2/5	1. 140mm hole; breaks into void on west side: 3cm x 10cm. 2. 170mm hole; breaks into void on north side: 3cm x 6cm.	
8-G2/5	1. 20mm gap exposed to surface on west side. Estimate 75% of gap contains no grout.	

Table 5.4.2. Results of tests on shear wall-to-column joints

Story	Shearwall	Column	Item	Height	Hardness Value -Ld Average	C.O.V.	Thickness (mm)
6	D1/4A-E/4A	D1/4A	SWP	M	361	0.02	9.4
6	C1/5-C1/6	C1/6	SWP	T	381	0.04	9.5
5	C1/4A-D/4A	C1/4A	SWP	T	355	0.05	9.6
7	D/4A-D1/4A	D/4A	SWP	B	355	0.03	11.6
7	F1/6B-G/6B	G/6B	SWP	M	365	0.04	11.6
4	G2/5-G2/6	Joint	SWP	M	333	0.02	9.6
4	D1/4A-E/4A	D1/4A	SWP	M	394	0.03	9.2
6	G2/5-G2/6	G2/6	SWP	M	377	0.05	9.4
5	G2/5-G2/6	G2/6	SWP	M	370	0.05	9.5
6	C1/4A-C1/5	C1/5	SWP	T	341	0.06	9.4
6	D1/4A-E/4A	D1/4A	CP	M	397	0.06	11.1
6	C1/5-C1/6	C1/6	CP	T	387	0.04	11.1
5	C1/4A-D/4A	C1/4A	CP	T	375	0.04	11.2
7	D/4A-D1/4A	D/4A	CP	B	397	0.03	11.1
7	F1/6B-G/6B	G/6B	CP	M	369	0.03	11.8
4	G2/5-G2/6	Joint	CovP	M	358	0.03	11.5
4	D1/4A-E/4A	D1/4A	CP	M	382	0.05	11.2
6	G2/5-G2/6	G2/6	CP	M	380	0.02	11.4
5	G2/5-G2/6	G2/6	CP	M	376	0.03	11.1
6	C1/4A-C1/5	C1/5	CP	T	388	0.02	11.2
6	D1/4A-E/4A	D1/4A	W	M	439	0.08	
6	C1/5-C1/6	C1/6	W	T	426	0.03	
5	C1/4A-D/4A	C1/4A	W	T	454	0.13	
7	D/4A-D1/4A	D/4A	W	B	428	0.08	
7	F1/6B-G/6B	G/6B	WCov	M	440	0.02	
7	F1/6B-G/6B	G/6B	WBar	M	418	0.01	
4	G2/5-G2/6	Joint	W	M	449	0.03	
4	D1/4A-E/4A	D1/4A	W	M	437	0.06	
6	G2/5-G2/6	G2/6	W	M	446	0.01	
5	G2/5-G2/6	G2/6	W	M	395	0.04	
6	C1/4A-C1/5	C1/5	W	T	433	0.03	

Notes: CovP = cover plate
WCov = weld to cover plate
WBar = weld to reinforcing bar

Table 5.5.1. Concrete and grout strengths and failure loads in column joint tests

Specimen	Joint Condition	Column Concrete (kg/cm^2)	Joint Concrete (kg/cm^2)	Joint Grout (kg/cm^2)	Failure Load (ton)	Failure Load/ Failure Load Full Joint
7	Full	410	370	260	771	1.00
6	Full	430	395	280	824	
8	Grout	410	---	260	660	0.87
4	Grout	425	---	270	735	
2	Concrete	420	370	---	478	0.59
1	Concrete	430	395	---	469	
3	Nothing	420	---	---	350	0.44
5	Nothing	424	---	---	346	

Table 5.8.1. Qualitative assessment of core walls based on visual inspection

BRICK PARTITION OBSERVED

STORY & LOCATION	1	2	3	4	5	6	7	8	9	10	11	12	13	14
1 W	- (4)	- (1)	P	F-P	G	G (1)	F-P	F	F (3)	F (3)	F-P (3)	F	F (3)	P (1)
2 W	- (4)	F-P (3)	F-P	- (5)	- (4)	- (4)	- (4)	P (4)(3)	- (4)	P (3)	F-P (3)	P (3)	- (5)	- (4)
3 W	- (4)	P (3)	P (5)	P (5)	- (4)	- (4)	- (4)	- (4)(1)	- (4)	F-P (3)	F (3)	F-P (3)	P (3)	P (1)
4 W	G	F-P (3)	F (2)	F	- (4)	- (4)	- (4)	- (4)	- (4)	P	F (3)	F-P	F-P	F
5 W	G	F-P	F-P	F-P	- (4)	- (4)	- (4)	- (4)(1)	- (4)	F	F-P	F-G	F (7)	F (2)
6 W	G	F	F	F	- (4)	- (4)	- (4)	- (4)(1)	- (4)	F-G	F (3)	F-G	F (7)	G (2)
7 W	G	F-P	F-P	F-P	- (4)	- (4)	- (4)	- (4)	- (4)	F	F (1)(3)	F	F	F-G (7)
8 W	G	F	F	F	- (4)	- (4)	- (4)	- (4)	- (4)	F-G (1)	G	F-G	F	F (3)
1 E	F	F-P	F (1)	F	F-P	F	F	F-P	F-P (3)	F-P	F-P	G (2)		
2 E	F-P	F-P	F-P	- (5)	F (3)	- (5)	F-G	- (6)	G	F-G	F-P (3)	P (1)		
3 E	F-G	F	F-P (3)	F-P (5)	F-G	- (5)	G	- (6)	F-G	F-G	- (5)	F-G (2)		
4 E	F	F	F	F	F (3)	F (2)	F	- (6)	F	F	F	F (2)		
5 E	F	F	F	F	G	F (2)	F-G	- (6)	G	G	F (3)	F (3)		
6 E	P	F	F	F	F (3)	F (3)	G	- (6)	G	G	F (3)	F (3)		
7 E	F-G	F	F	F	P	- (6)	F-G	- (6)	G	F-G	F-P	F (3)		
8 E	G	F	F	F (3)	- (6)	G (1)	G (1)	- (6)	F-P (3)	G (1)	F-G	F (3)		

Notes:

G = good
F = fair
P = poor

1 = wall penetrations or openings
2 = duct penetrations near top of wall
3 = open or incomplete filling at top of wall
4 = walls covered with cement plaster
5 = walls covered in whole or in part with drywall
6 = walls not in place, all or in major part
7 = top of wall hidden by duct work

Table 5.10.1. Dimensions, weight and void area of brick

Type	Specimen	Length (mm)	Width (mm)	Height (mm)	Room Dry Weight (g)	Percent Void Area
Facing	D2	252	120	64	2666.3	24.5*
Facing	B2	250	120	64	2695.3	24.1*
Facing	D1	247	116	64	2681.8	26.4*
Facing	A3	251	117	64	2693.8	25.8*
Facing	C3	249	116	64	2684.4	25.9*
Average		250	118	64	2684.38	25.3*
Building	3-6A	251	120	65	3616.8	6.1
Building	6-6A	249	120	64	3438.4	7.9
Building	5-6A	249	120	64	3377.9	8.4
Building	7-6A	250	120	65	3497.1	7.6
Building	8-6A	250	120	64	3442.5	7.3
Building	1-6B	249	122	63	3553.8	6.2
Average		250	120	64	3487.6	7.3

*Measured on brick other than listed in this table.

Table 5.10.2. Initial rate of absorption of brick

Type	Specimen	Room Dry Weight (g)	Area* (cm^2)	Absorption After 1 Minute (g)	Weight Gain Corrected to 194 cm^2 (g/min)
Facing	D2	2688.3	225.0	34.6	29.75
Facing	B2	2695.3	223.5	31.5	27.27
Facing	D1	2681.8	213.9	10.9	9.86
Facing	A3	2693.8	220.1	26.0	22.86
Facing	C3	2662.7	216.9	16.3	14.54
Building	3-6A	3616.8	282.4	33.5	22.96
Building	6-6A	3438.4	275.2	35.8	25.17
Building	5-6A	3377.9	273.2	27.2	19.27
Building	7-6A	3497.1	277.2	36.0	25.13
Building	8-6A	3442.5	277.7	32.6	22.72

Table 5.10.3. Absorption and saturation coefficient of brick

Type	Specimen	% Absorption After 24-h Submersion	% Absorption After 5-h Boiling	Saturation Coefficient
Facing	A2	9.27	14.01	0.66
Facing	A1	9.48	14.17	0.67
Facing	A4	8.65	13.05	0.66
Facing	A5	7.16	10.75	0.67
Facing	A3	9.26	13.39	0.69
Facing	A6	8.11	13.43	0.60
Building	1-6A	13.23	15.23	0.87
Building	3-6A	13.07	14.93	0.88
Building	5-6A	12.38	14.12	0.88
Building	6-6A	12.65	14.47	0.87
Building	7-6A	13.17	14.84	0.89
Building	8-6A	23.03	14.94	0.87

Table 5.10.4. Compressive strength of brick

Type	Specimen	Average Length (mm)	Average Width (mm)	Maximum Load (kg)	Compressive Strength (kg/cm^2)
Facing	1RM	125.0	116.3	90,120	620
Facing	1RC	121.9	117.1	83,080	582
Facing	2DC	119.6	117.6	92,840	660
Facing	2LM	120.4	115.1	88,300	638
Facing	3LC	121.1	117.1	73,780	520
Facing	3DM	125.0	118.9	72,190	486
				Average	585
Building	1-6	136.6	122.4	86,710	519
Building	3-6	125.0	119.4	91,030	611
Building	5-6	122.4	117.9	85,125	591
Building	6-6	122.1	120.7	84,670	575
Building	7-6	124.2	121.7	87,620	580
Building	8-6	125.7	119.4	89,665	598
				Average	579

Table 5.10.5. Modulus of elasticity of brick

Type	Specimen	Gross Area (cm^2)	Ultimate Stress (kg/cm^2)	Modulus of Elasticity (kg/cm^2)
Facing	A1	147.6	471	167,042
Facing	A3	144.3	516	195,634
Facing	A4	147.9	508	158,873
Facing	A5	140.8	577	180,493
Facing	A6	148.3	454	150,141
		Average	505	170,437
Building	3-6A	146.5	624	123,732
Building	5-6A	146.5	633	125,563
Building	6-6A	148.9	581	124,718
Building	7-6A	145.5	535	102,817
Building	8-6A	145.6	547	105,211
		Average	584	116,408

Table 5.10.6. Contraction of bricks due to heating

Type	Specimen	Length (mm)	Contraction (mm)	Contraction (mm/mm)
Facing	H-1	247.9	0.122	0.00049
Facing	H-2	247.6	0.069	0.00028
Facing	H-3	247.9	0.074	0.00030
			Average	0.00036
Building	4-6	252.5	0.066	0.00026
Building	2-6	248.9	0.079	0.00032
Building	8-6	247.9	0.071	0.00029
			Average	0.00029

Table 5.10.7. Coefficient of linear thermal expansion of brick masonry

Panel No.	Type	Mortar	Coefficient of Linear Thermal Expansion ($10^{-6}/^{\circ}C$) Horizontal	Vertical
M-1	Facing	M	6.7	5.4
M-2	Facing	M	7.4	5.2
S-1	Facing	S	6.8	6.1
S-2	Facing	S	7.2	6.3
B-1	Building	M	7.7	7.0
B-2	Building	(M)*	6.7	6.8

*Soviet field mortar.

Table 5.10.8. Compressive strength of field mortar

Sample No.	Width (mm)	Length (mm)	Thickness (mm)	Maximum Load (kg)	Conversion Ratio	Compressive Strength Converted to 50-mm cube (kg/cm^2)
1A-1	52.2	52.2	20.3	15,708	1.6	360
1A-2	52.0	51.5	19.5	18,932	1.7	416
3-1	50.7	51.3	18.8	17,570	1.7	398
3-2	50.3	52.0	17.2	20,090	2.1	366
4A-1	51.8	52.1	26.2	13,756	1.4	364
4A-2	50.6	50.5	25.1	12,667	1.4	354
5	50.2	52.8	17.1	19,068	1.9	379
					Average	377

Table 5.10.9. Compressive strength and modulus of elasticity of facing brick prisms

Specimen No.	Ultimate Load (tons)	Compressive Strength (kg/cm^2)	Modulus of Elasticity (kg/cm^2)
S-1	70.82	241	145,352
S-2	72.64	247	180,423
S-3	66.28	225	136,338
S-4	65.60	223	136,338
S-5	68.78	234	134,366
	Average	234	146,563
M-1	96.25	327	186,479
M-2	110.55	376	197,606
M-3	105.78	359	180,423
M-4	98.97	336	172,394
M-5	113.05	384	172,394
	Average	356	181,859

Note: 7-brick high prisms, 556 mm
Gross area = 295 cm^3
Gage length = 330 mm

Table 5.10.10. Compressive strength and modulus of elasticity of building brick prisms

Specimen	Length (mm)	Width (mm)	Height (mm)	Gage Length (mm)	Ultimate Load (ton)	Compressive Strength (kg/cm^2)	Modulus of Elasticity (kg/cm^2)
B-1	241	121	241	152	55.62	191	97,183
B-2	171	121	171	152	56.07	271	84,507
B-3	243	121	243	241	60.38	206	73,944
B-4	257	121	257	241	68.67	222	88,732
B-5	248	121	235	241	105.33	353	110,704
B-6	197	121	302	241	65.04	274	114,507
B-7	178	121	308	241	55.27	258	90,563
					Average	254	94,306
B-8	249	118	229	165	116.45	396	116,479
B-9	249	118	229	165	113.27	386	89,366
B-10	249	118	311	241	108.28	369	92,465
					Average	384	99,437
B-11	249	118	556	483	78.77	268	73,380
B-12	249	118	556	483	90.91	309	102,676
					Average	289	88,028

Note: B-1 through B-7, field specimens cut from walls
B-8 through B-10, laboratory specimens from flexural tests
B-11 and B-12, 7-brick high laboratory specimens

Table 5.10.11. Flexural bond strength tests of laboratory prisms

Specimen	Width (mm)	Depth (mm)	Ultimate Load (kg)	Modulus of Rupture (kg/cm^2)
Facing Brick				
S-6	248	114	213	3.2
S-7	248	114	403	6.0
S-8	248	114	342	5.1
S-9	248	114	263	3.9
S-10	248	114	229	3.4
			Average	4.3
M-6	248	114	349	5.2
M-7	248	114	283	4.2
M-8	248	114	503	7.5
M-9	248	114	292	4.4
M-10	248	114	311	4.6
			Average	5.2
Building Brick				
M-1	249	118	213	3.0
M-2	249	118	176	2.4
			Average	2.7

Note: Clear span = 483 mm

Table 5.10.12. Diagonal tension (shear) tests of brick panels

Panel	Length (mm)	Width (mm)	Thickness (mm)	Ultimate Load (kg)	Diagonal Tension (Shear) Strength (kg/cm^2)
Field 5-N	457	457	121	6320	8.1
Field 6-W	464	460	121	7210	9.2
Lab B-M	457	457	118	10374	13.6
Lab F-S1	457	457	118	10397	13.7
Lab F-S2	457	457	118	10397	13.7
Lab F-M1	457	457	118	16571	21.8
Lab F-M2	457	457	118	16616	21.8

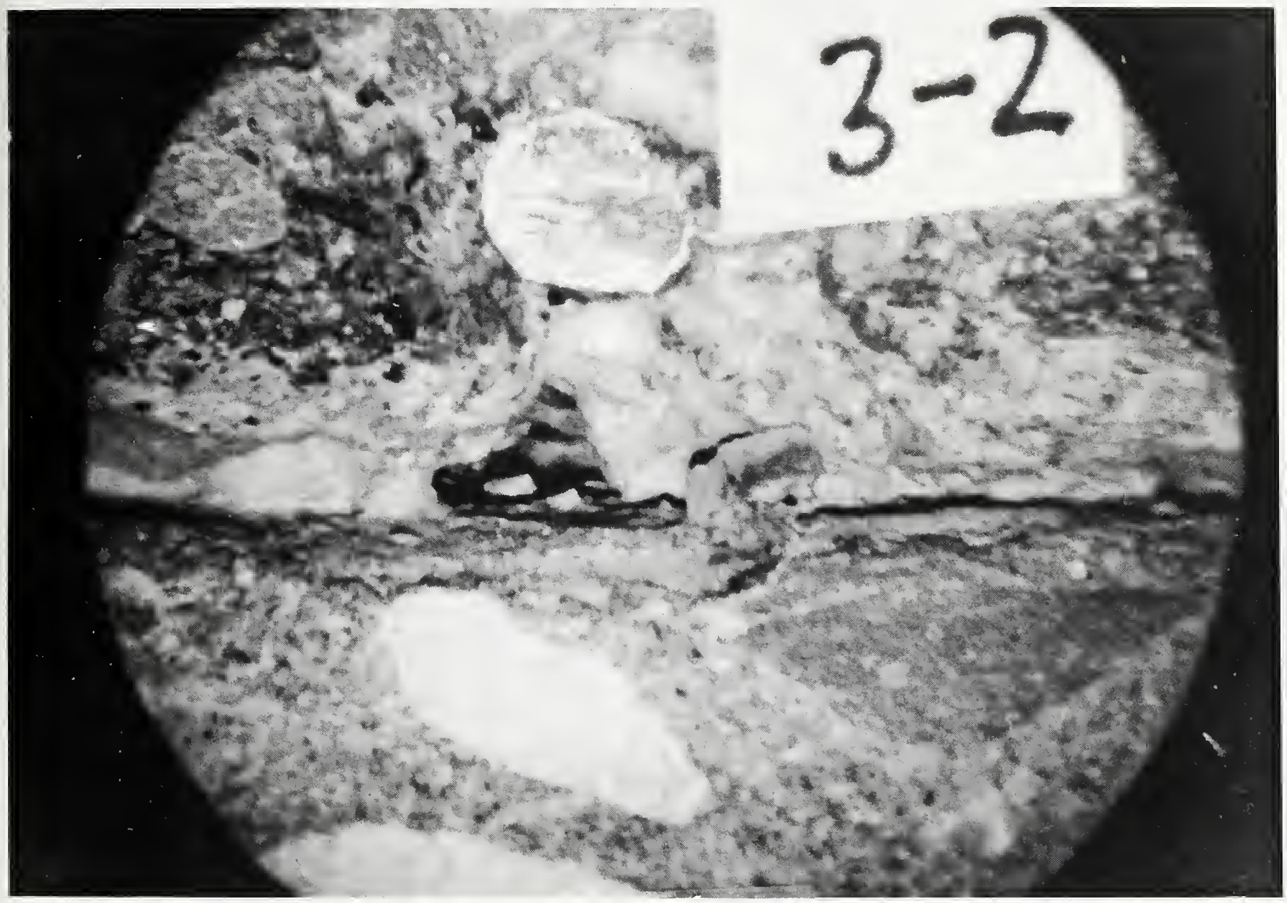


Figure 5.3.1. Borescope view of core wall at hole 3-2 showing joint between topping and plank at a location which sounded drummy

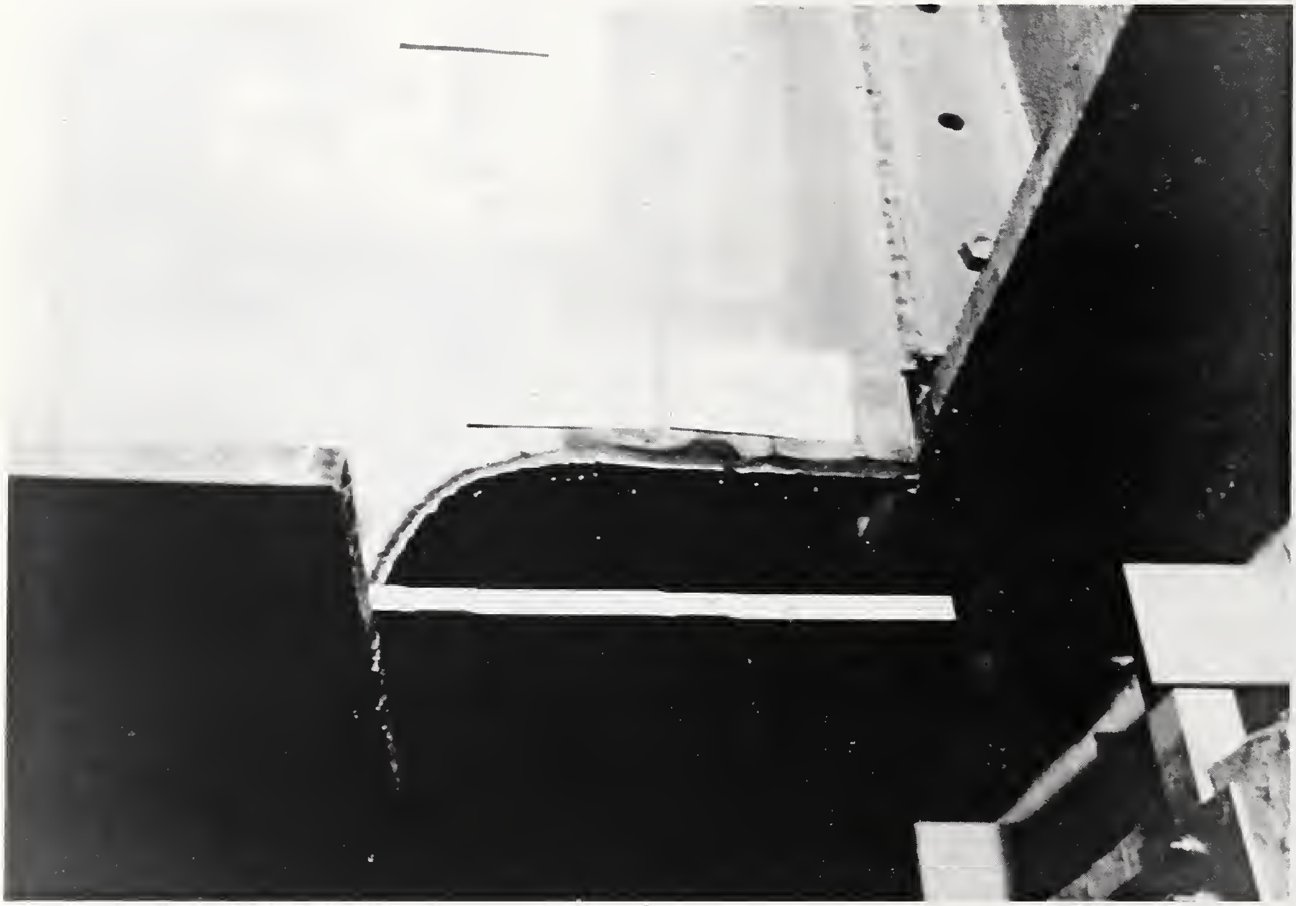
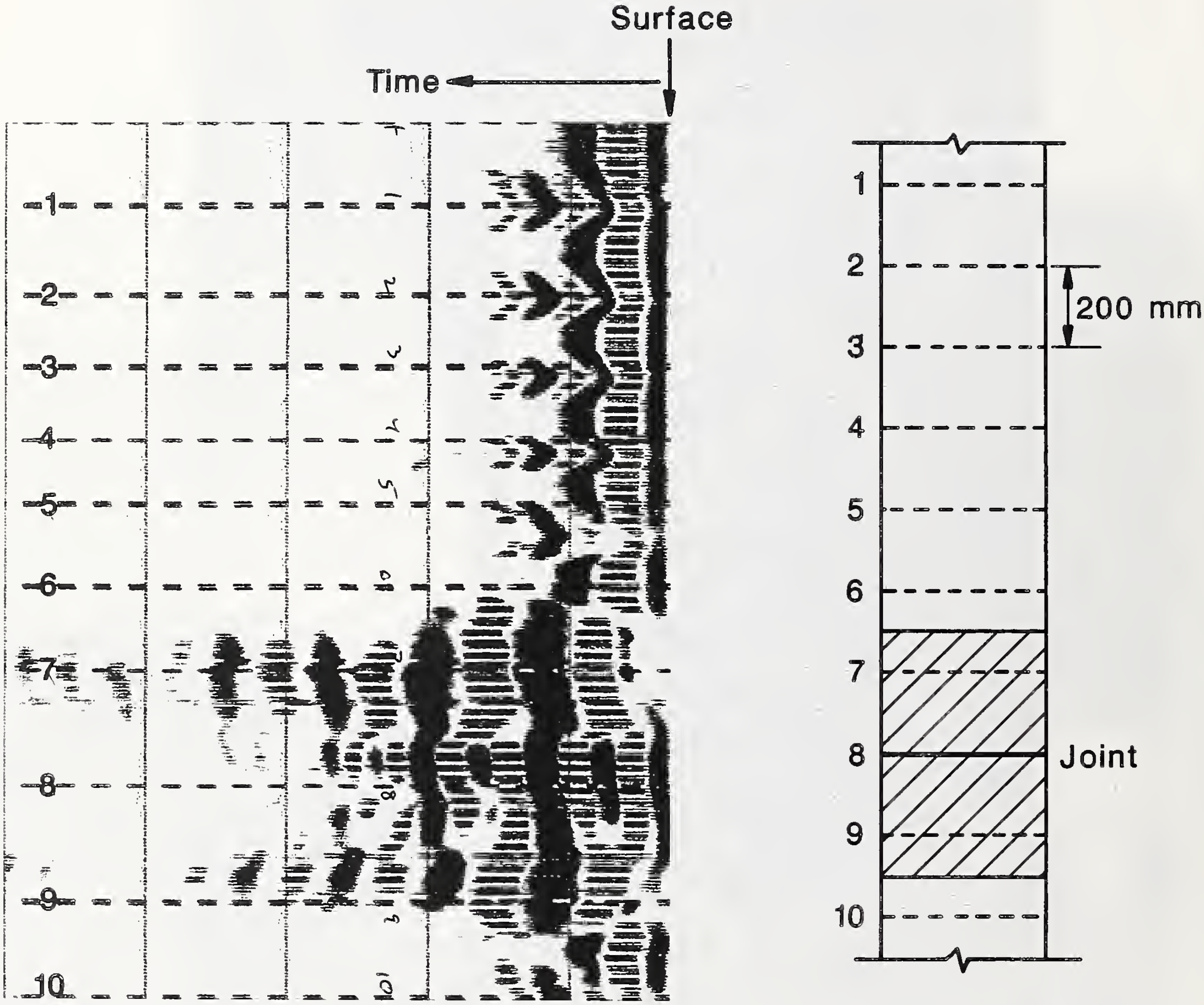


Figure 5.3.2. Section removed from built up beam of higher strength steel

Figure 5.3.3. Radar inspection of column



Radar Scan

Column 2-E/6A

Figure 5.3.4. Radar scan for column E/6A in second story

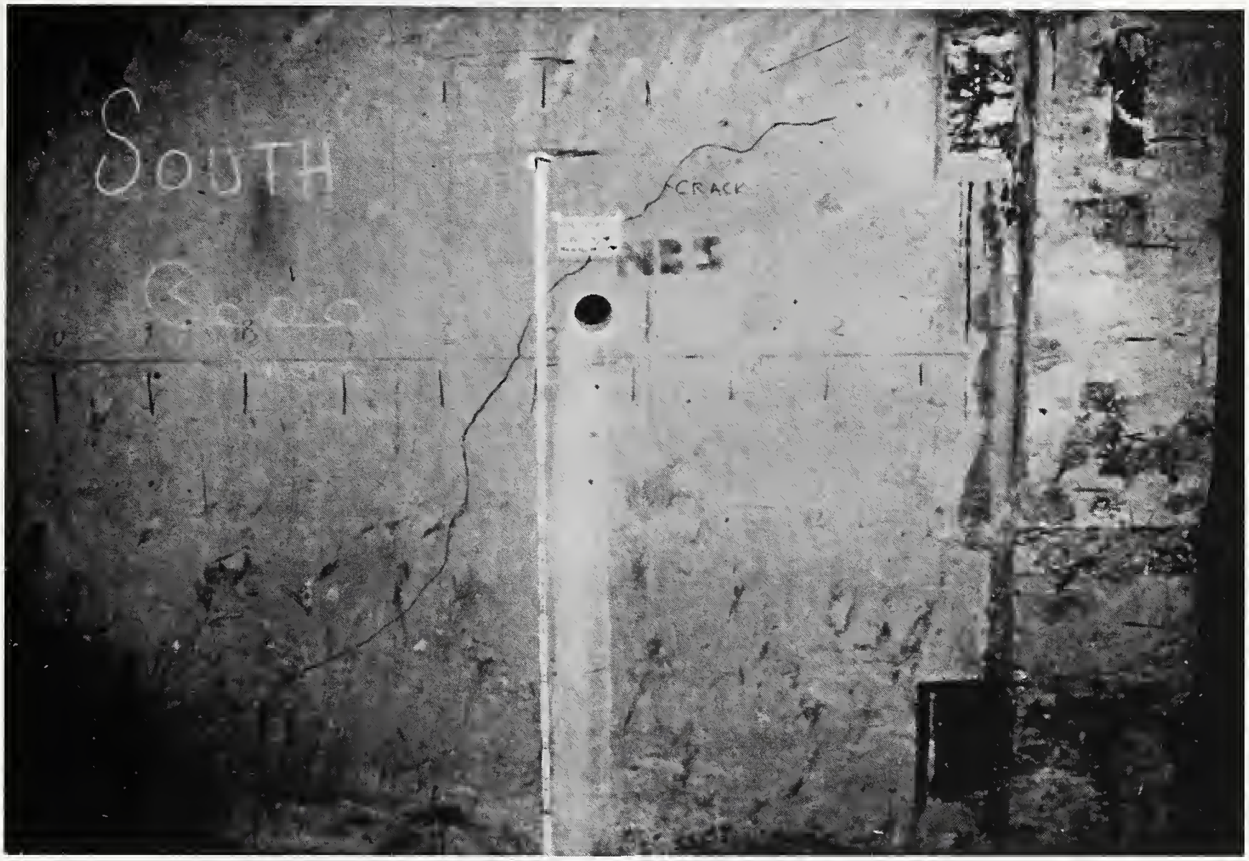


Figure 5.3.5. Outline of crack in fourth story shear wall between columns C1/4A and D/4A

Figure 5.4.1. Steel bracket at exterior column supporting 7.4-m reinforced concrete beam

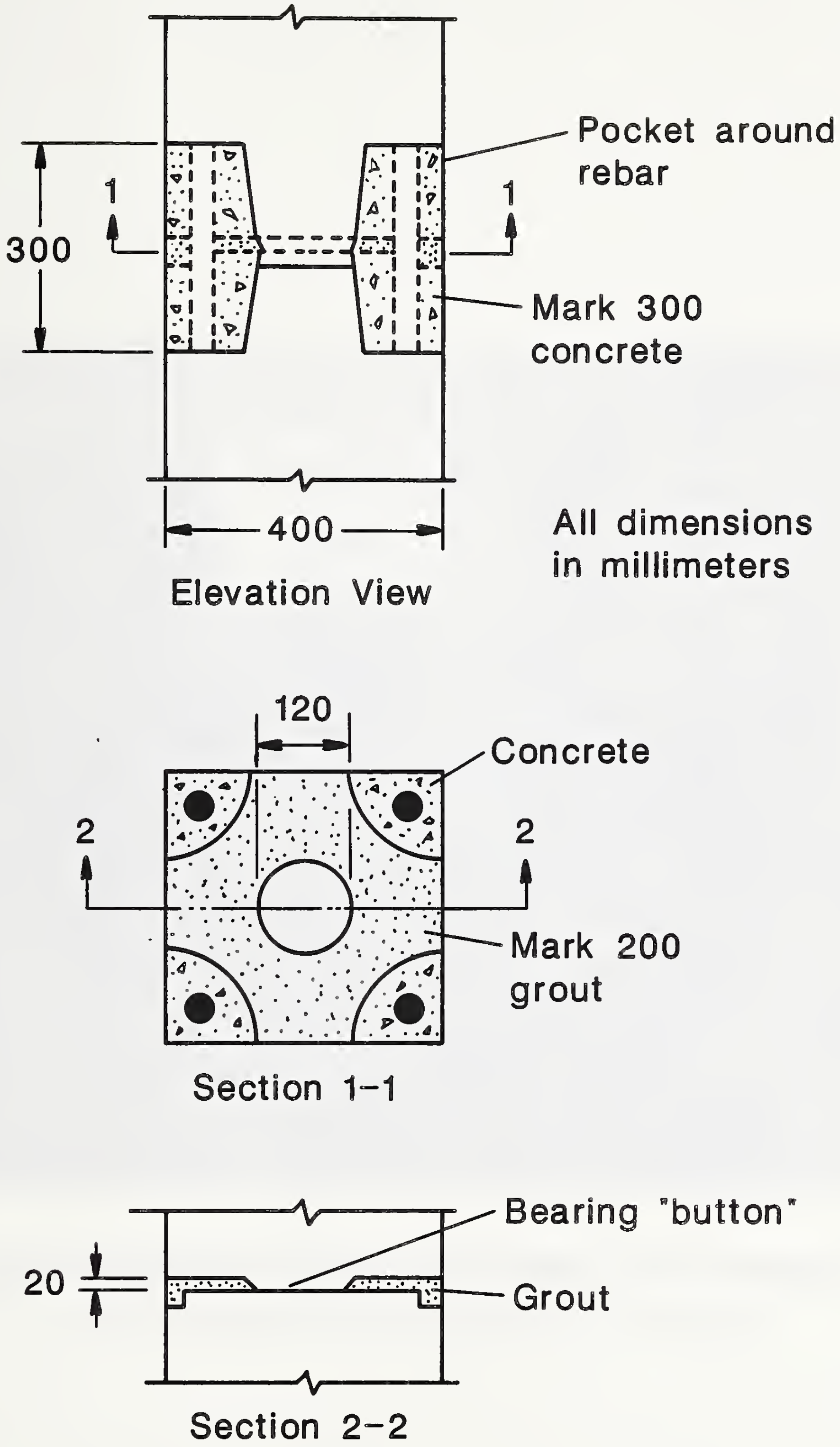


Figure 5.4.2. Details of joints between reinforced concrete column segments

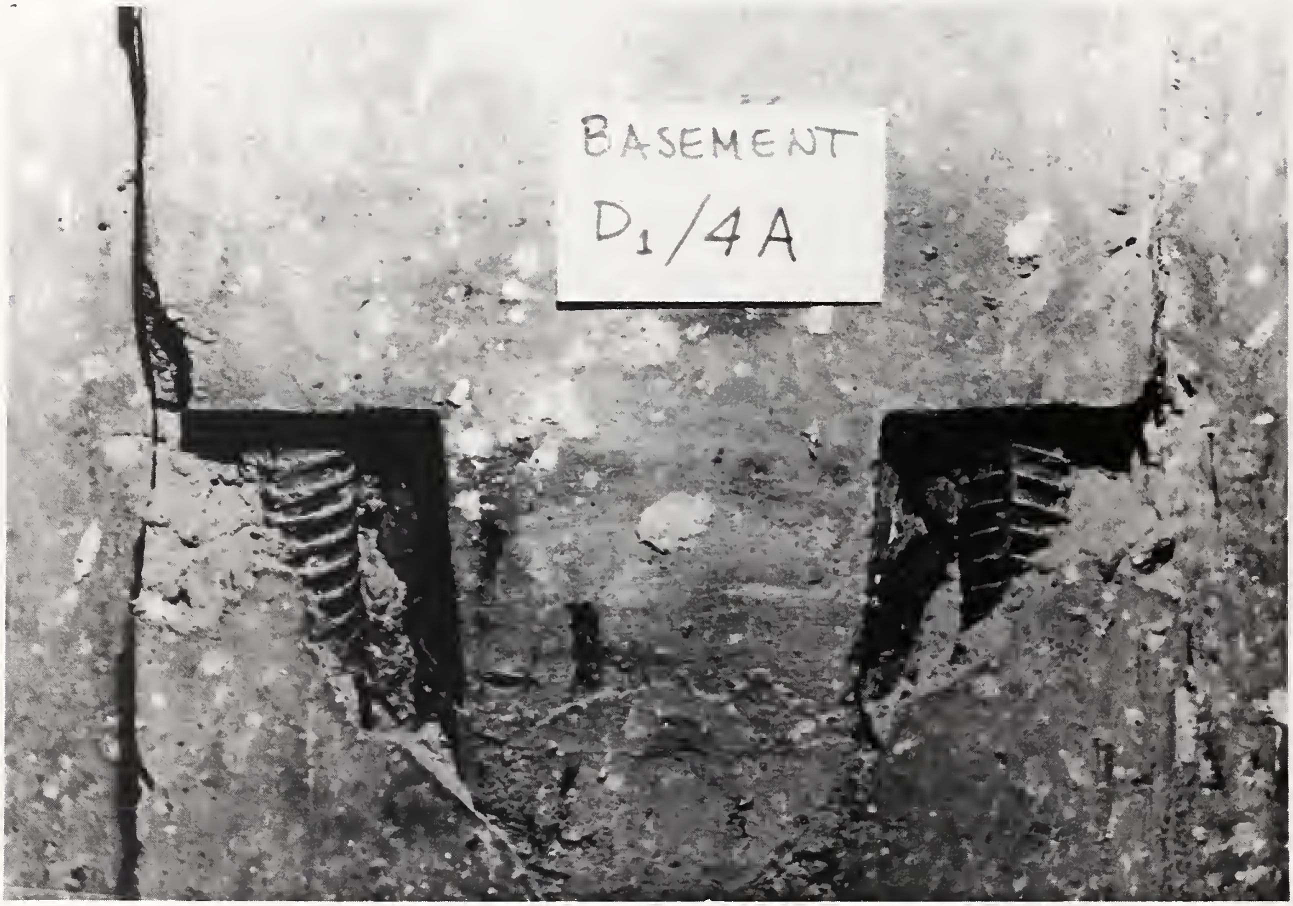


Figure 5.4.3. Defective joint for column D1/4A

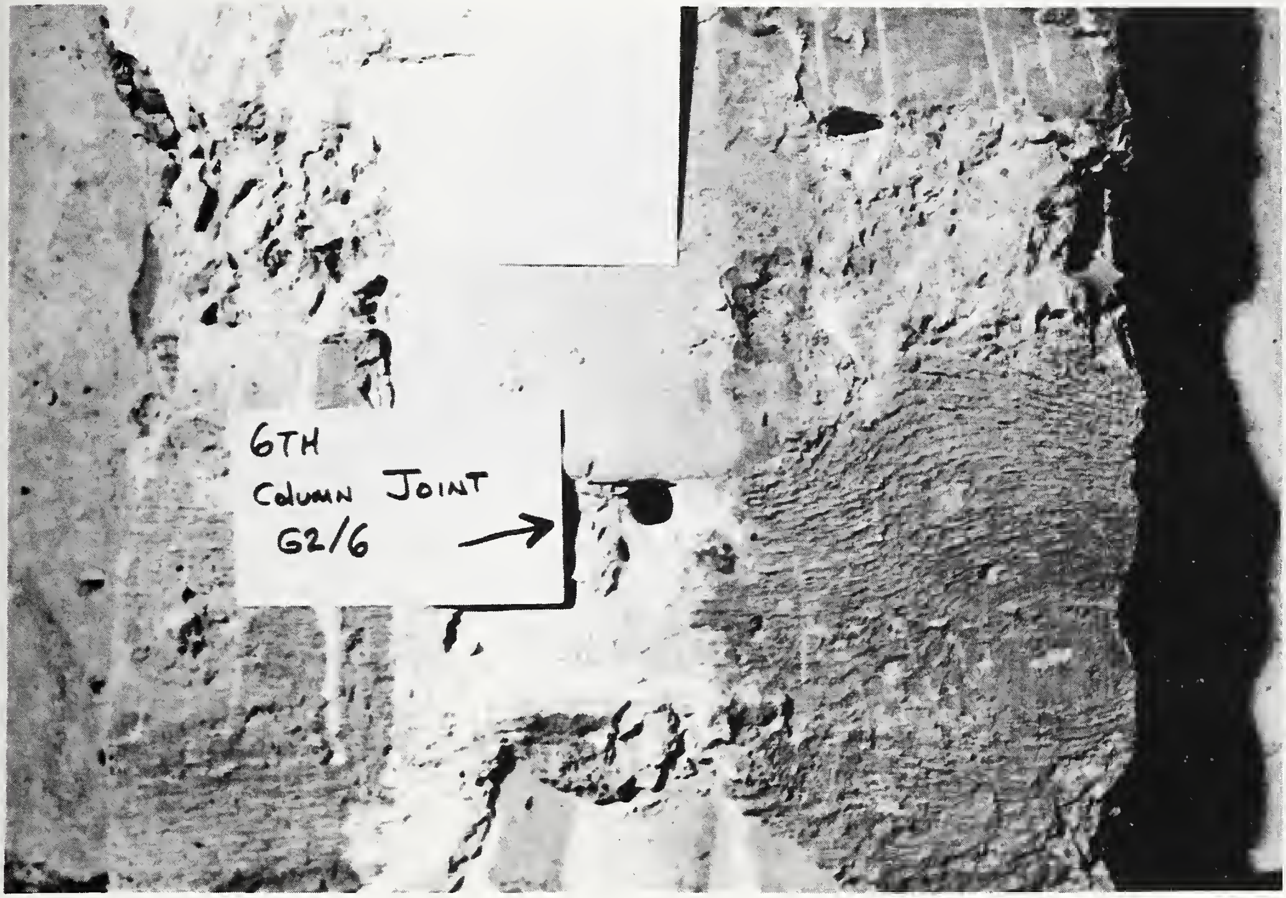


Figure 5.4.4. External appearance of apparently properly grouted and concreted column joint at 6-G2/6

Figure 5.4.5. Borescope photograph inside joint of 6-G2/6 showing that no grout has been placed around the bearing button (at right)

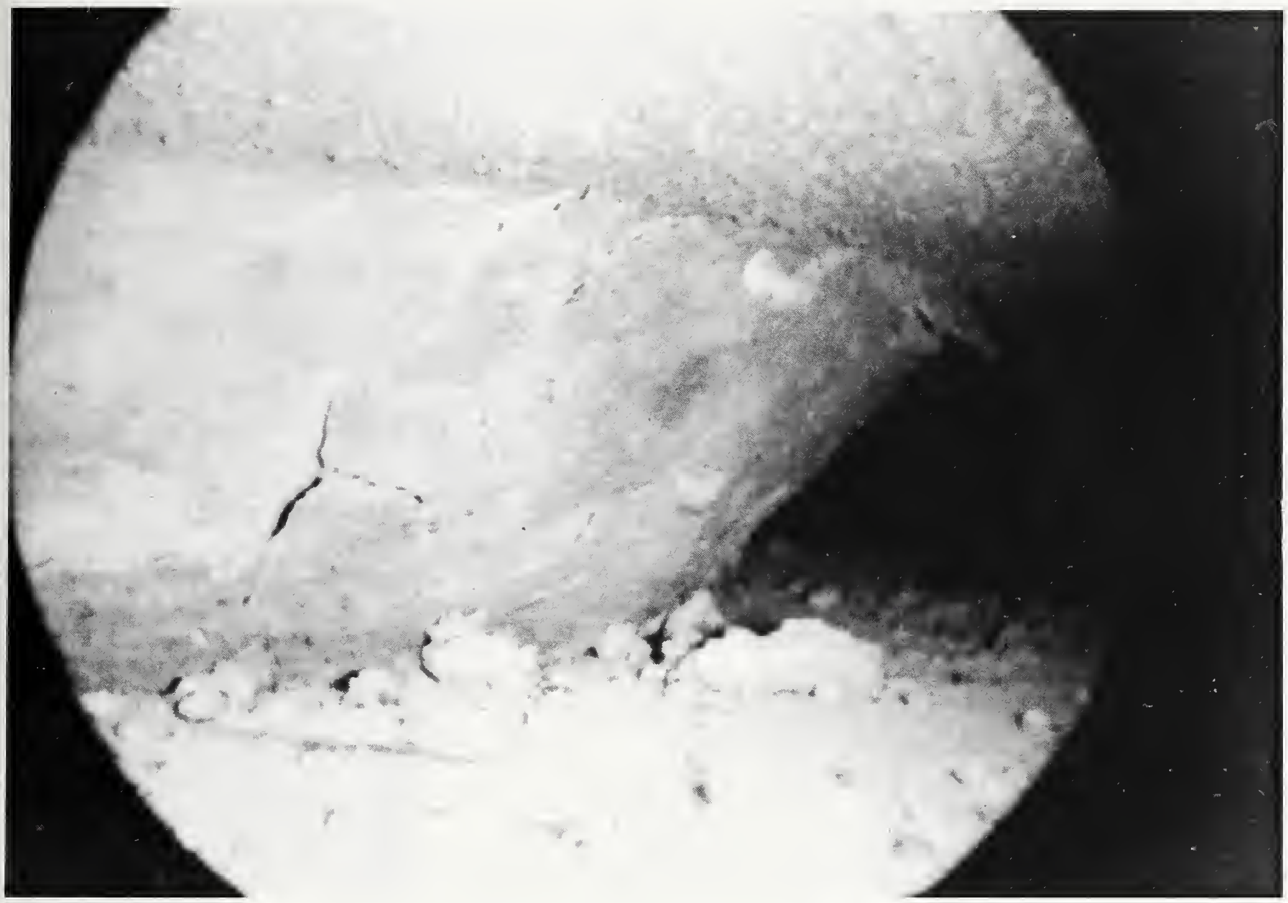


Figure 5.4.6. Borescope photograph inside joint of 6-G2/5 showing evidence of distress in the bearing button

Figure 5.4.7. Internal view of joint 4-G2/5 showing limited penetration of mortar placed over east column face

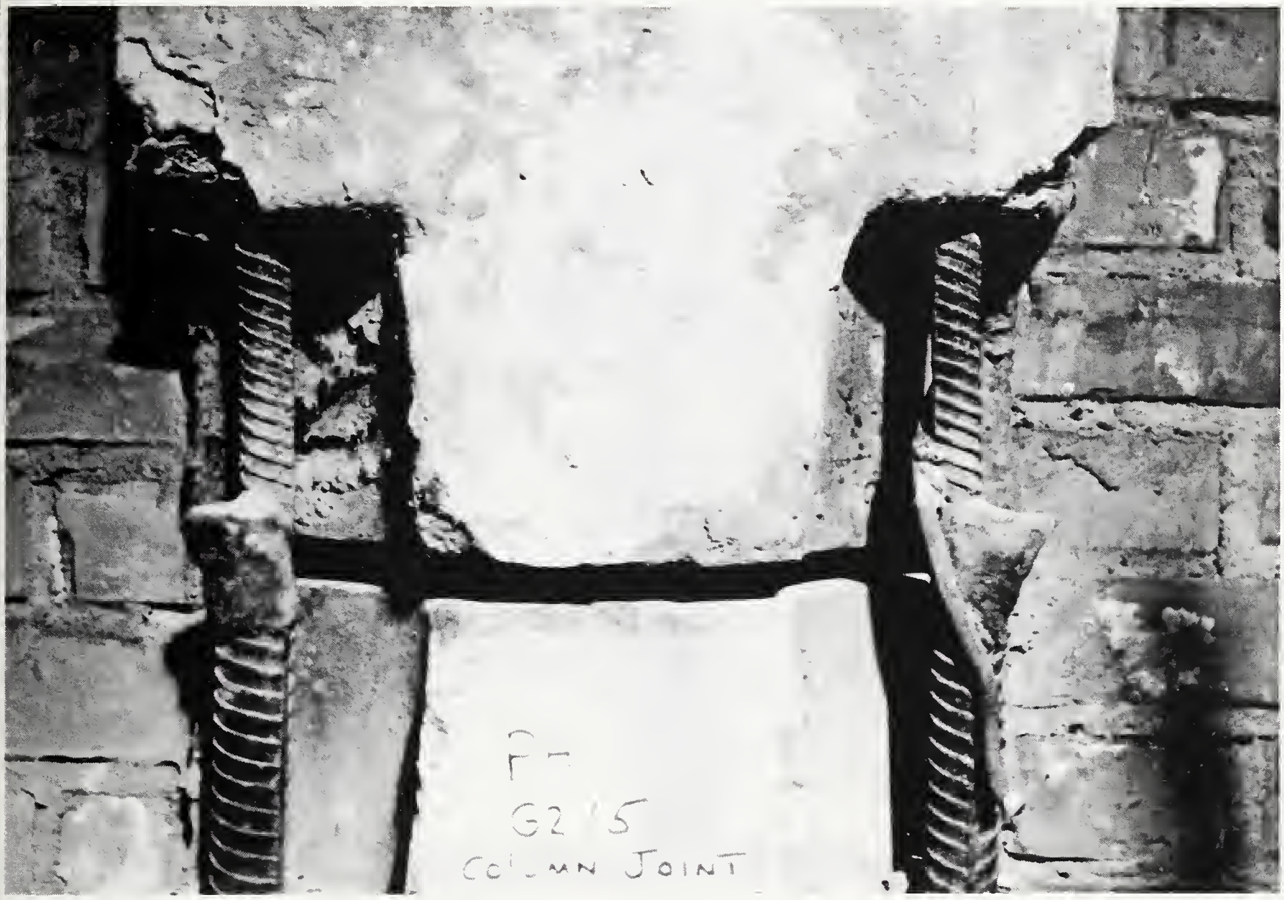


Figure 5.4.8. Joint of column G2/5 in penthouse prior to removal of bar from left corner

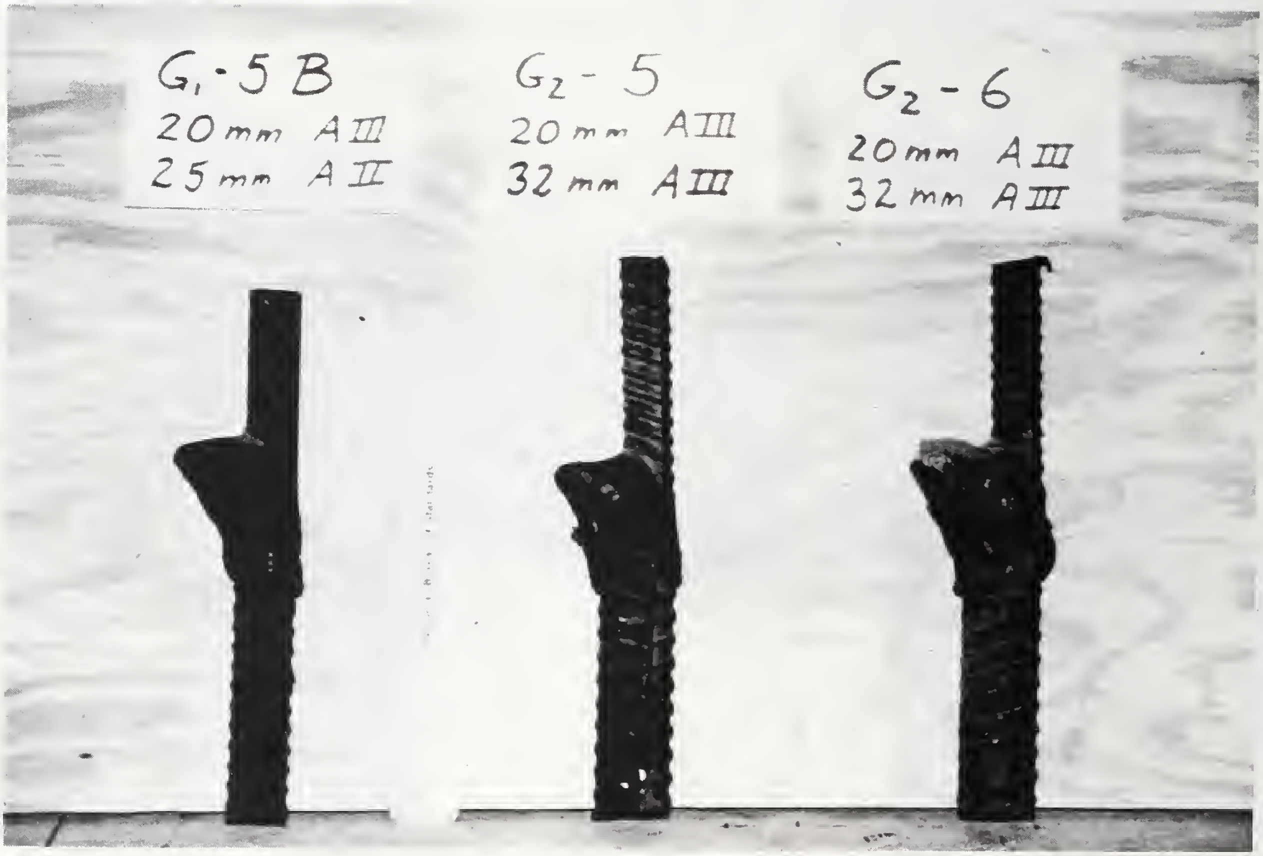


Figure 5.4.9. Reinforcing bar samples from penthouse columns

Figure 5.4.10. NBS investigator descends mechanical chase at northwest core for inspection of interior shear wall joints

Figure 5.4.11. Horizontal joint in west shear wall at seventh floor: note rough texture of formed vertical surface

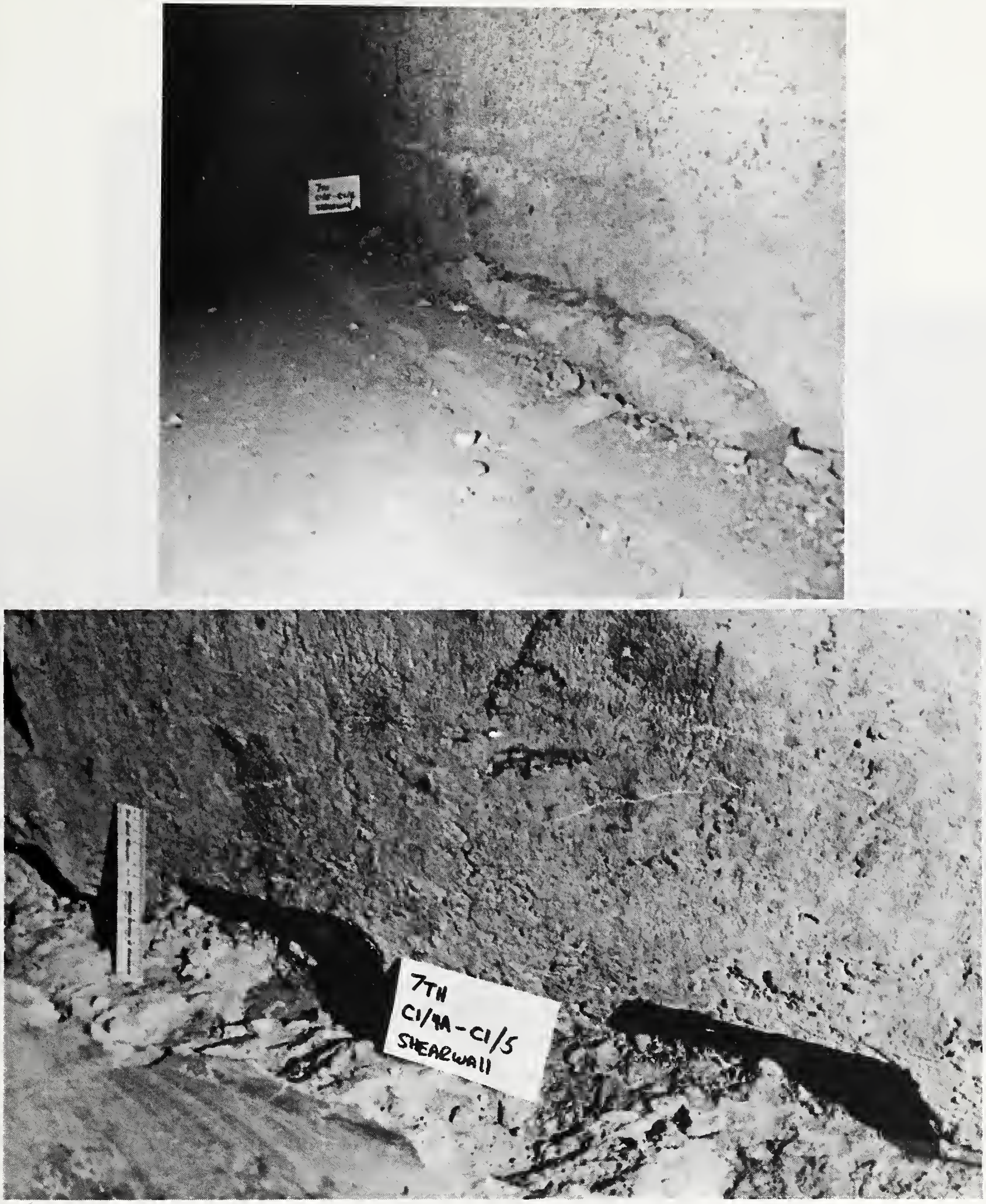


Figure 5.4.12. Exterior view of seventh floor horizontal shear wall joint along column line C1. Top shows course of masonry between the wall panel and the floor topping. Bottom shows incomplete filling of horizontal joint revealed after removing masonry.

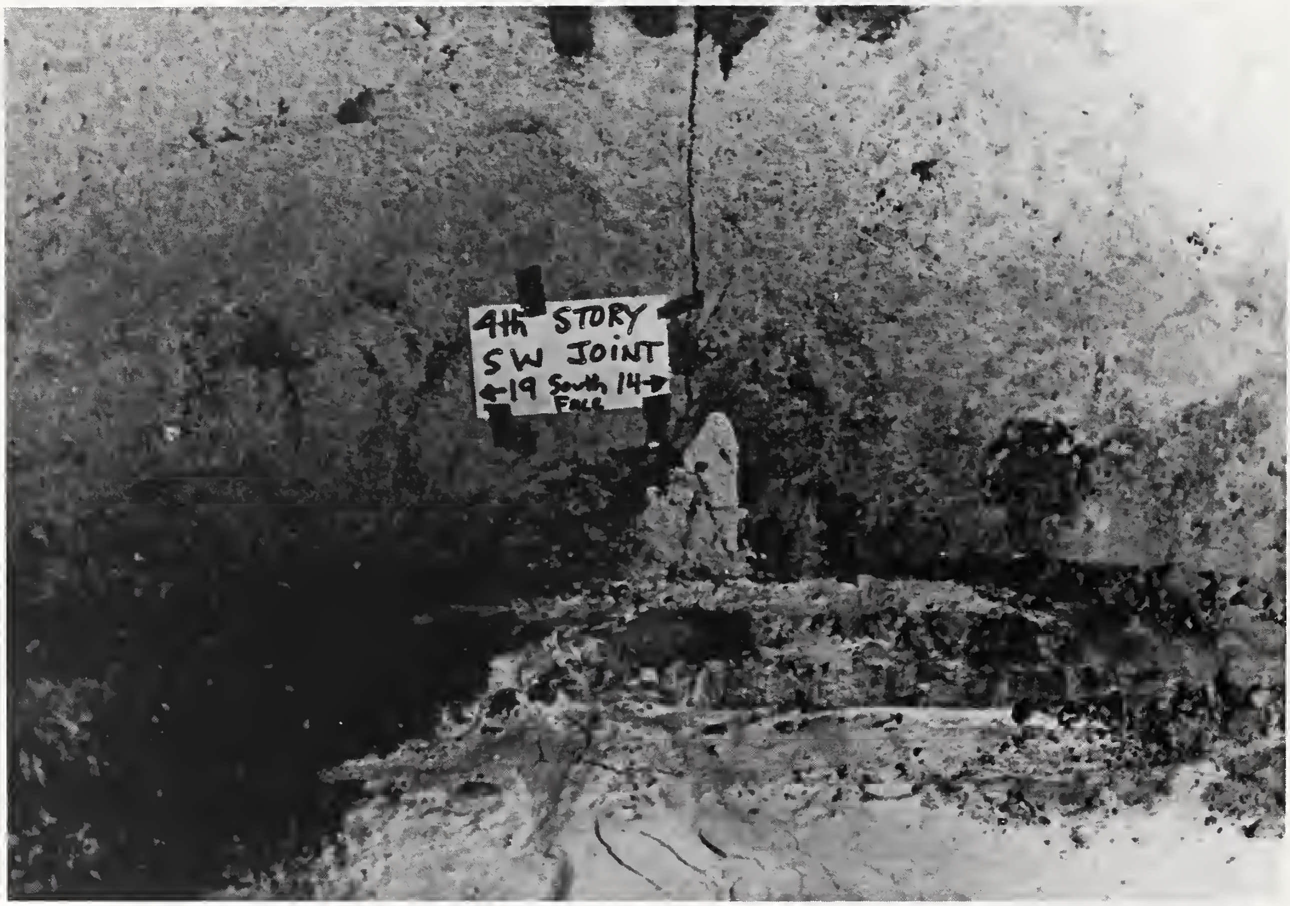


Figure 5.4.13. Location of core drilled into concrete of shear wall joint on the fourth floor between columns G2/6B and G/6B. Concrete was placed between wall panel and floor topping.

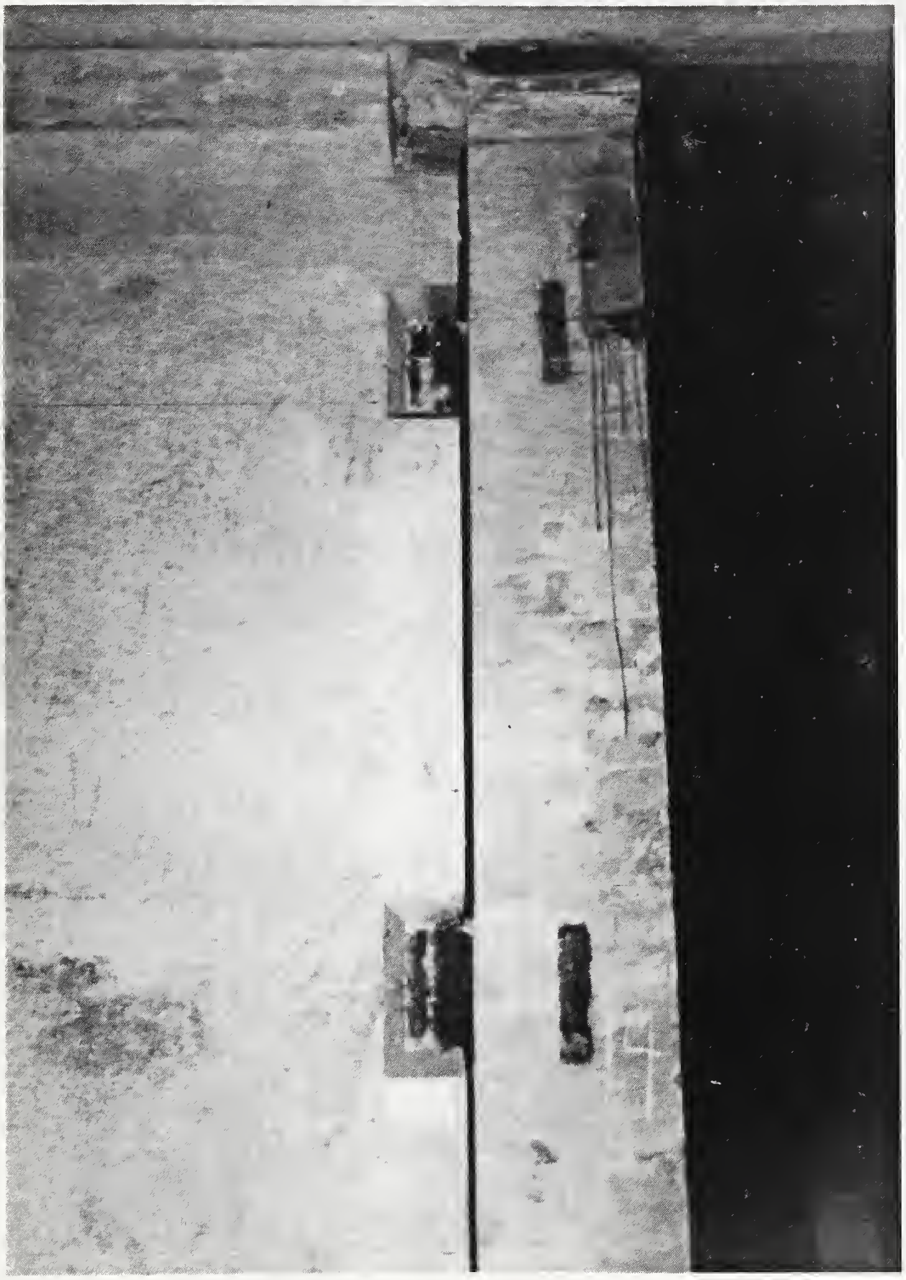


Figure 5.4.14. Ungrouted vertical shear wall-to -column joint at 5-G2/6B

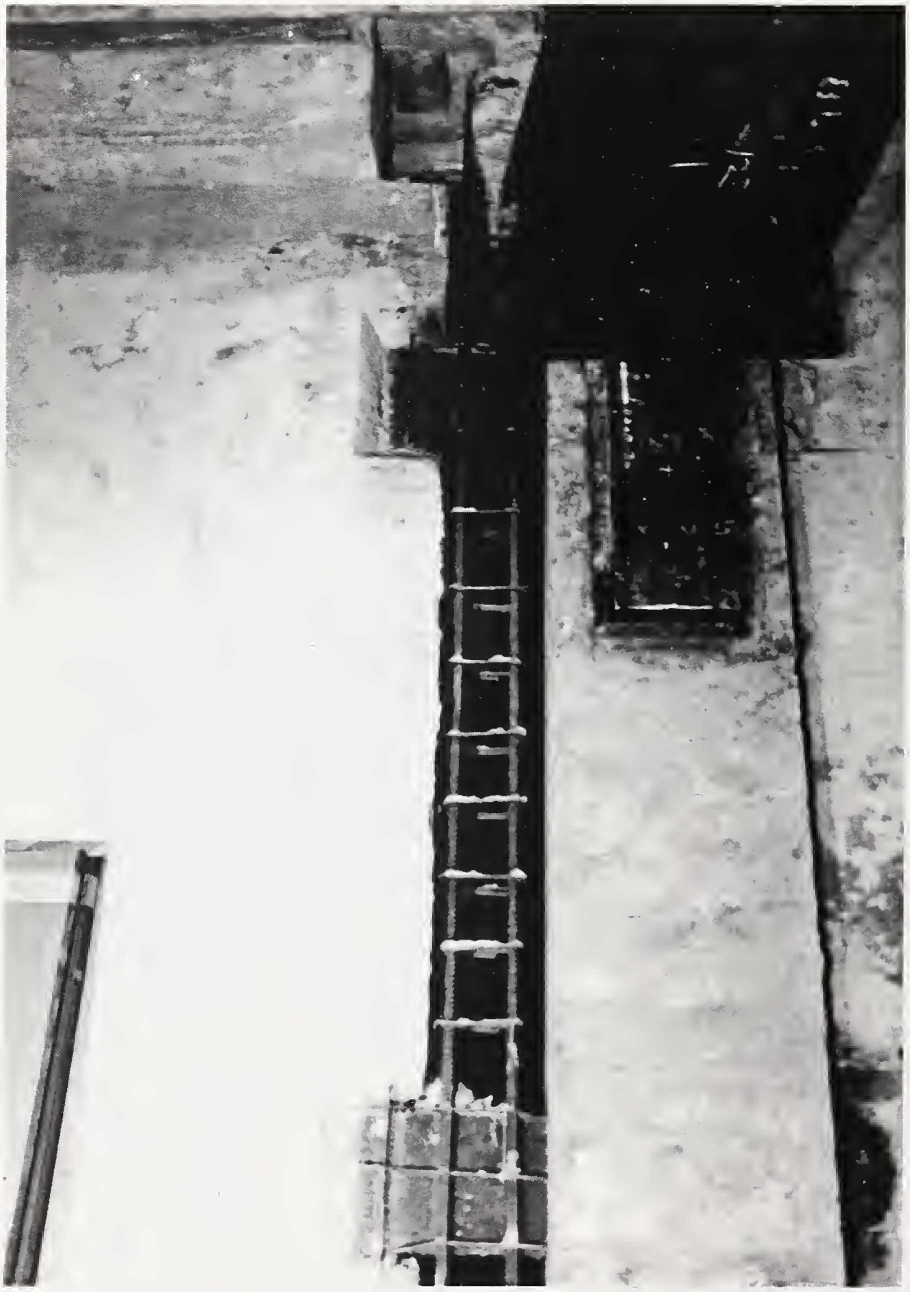


Figure 5.4.15. Wide shear wall-to-column joint at column 7-G/6B

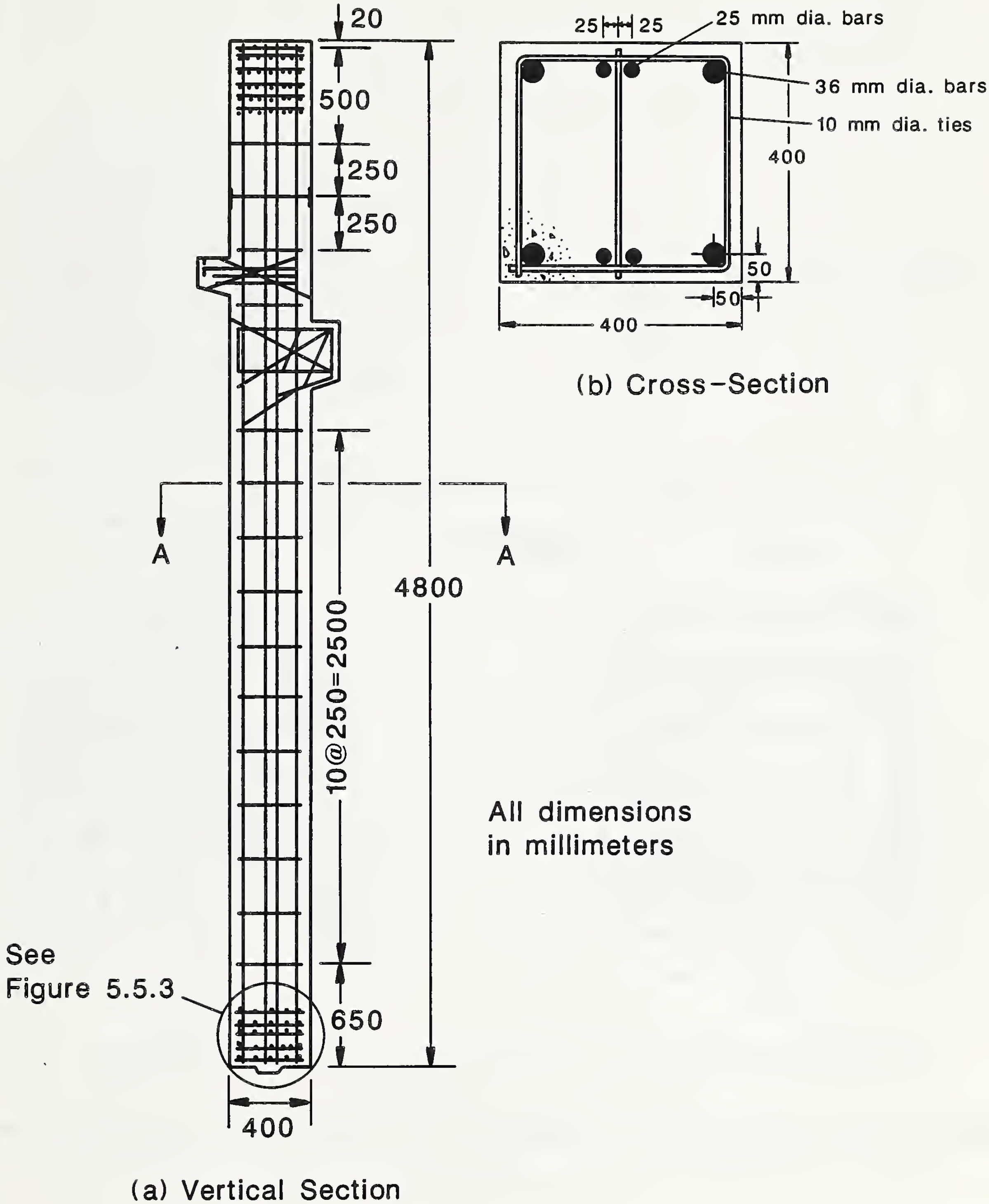


Figure 5.5.1. Sections through Type RC-5 reinforced concrete column

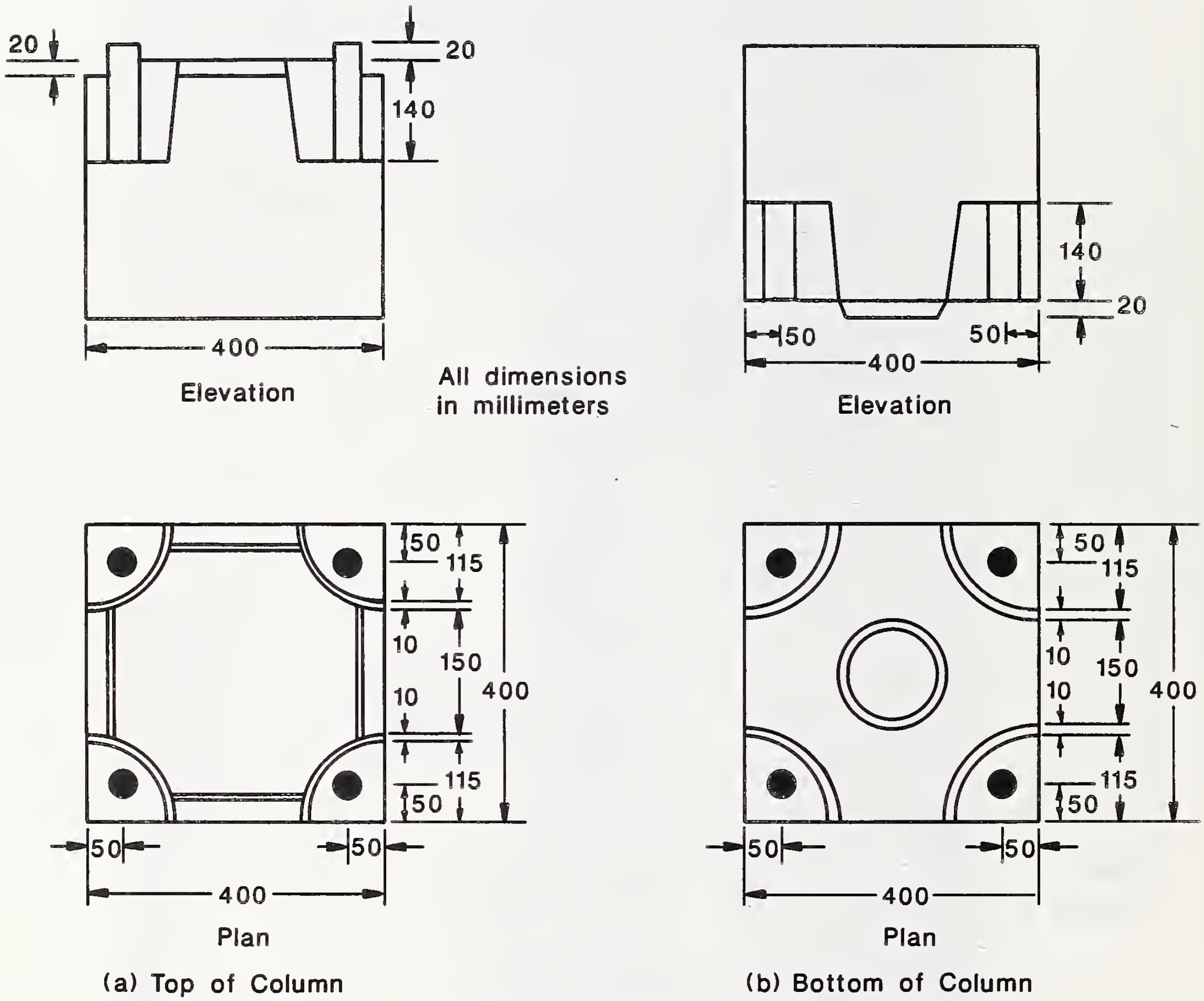


Figure 5.5.2. Geometries of column ends: a) top end and b) bottom end

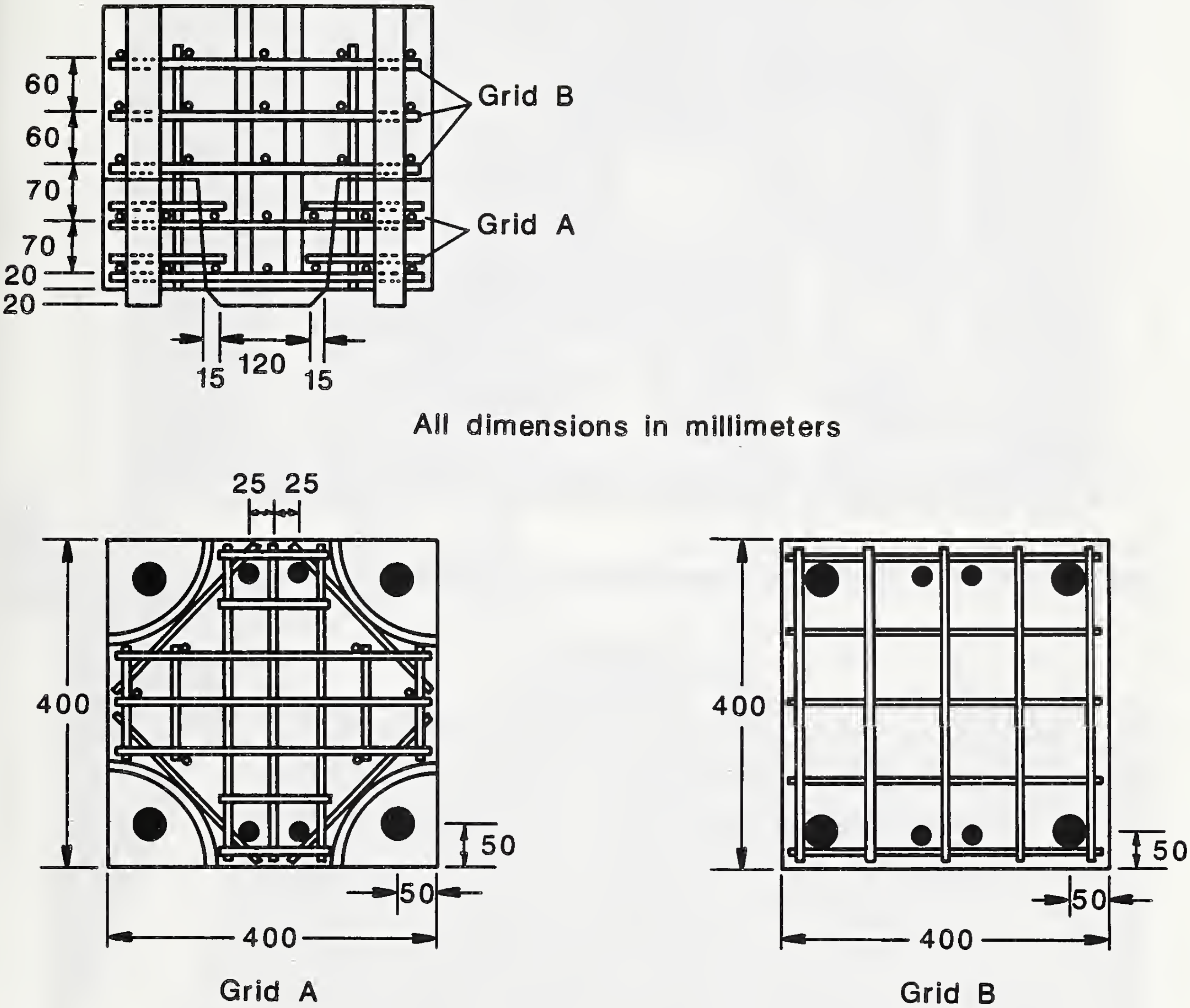


Figure 5.5.3. Transverse reinforcement at ends of reinforced concrete columns

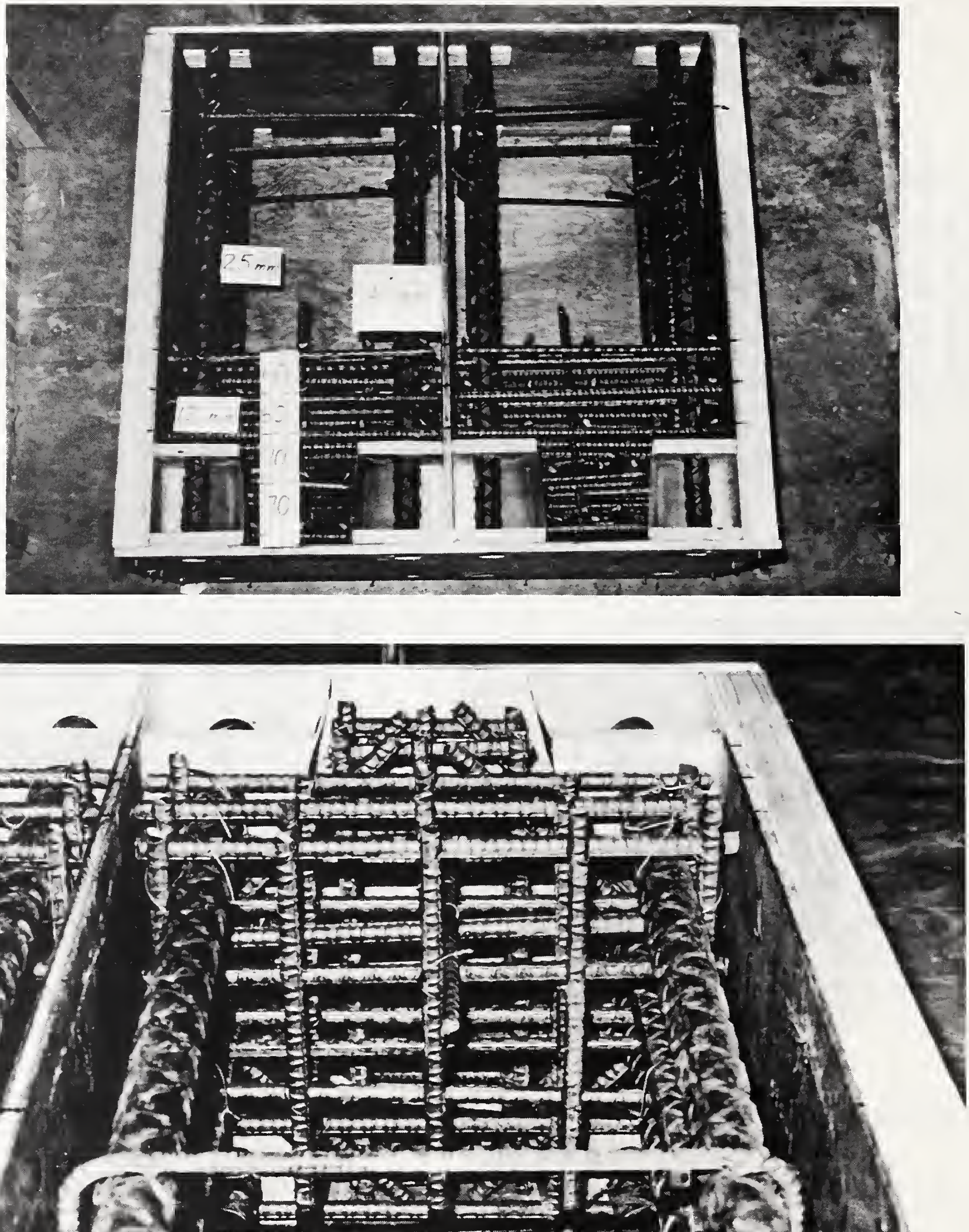
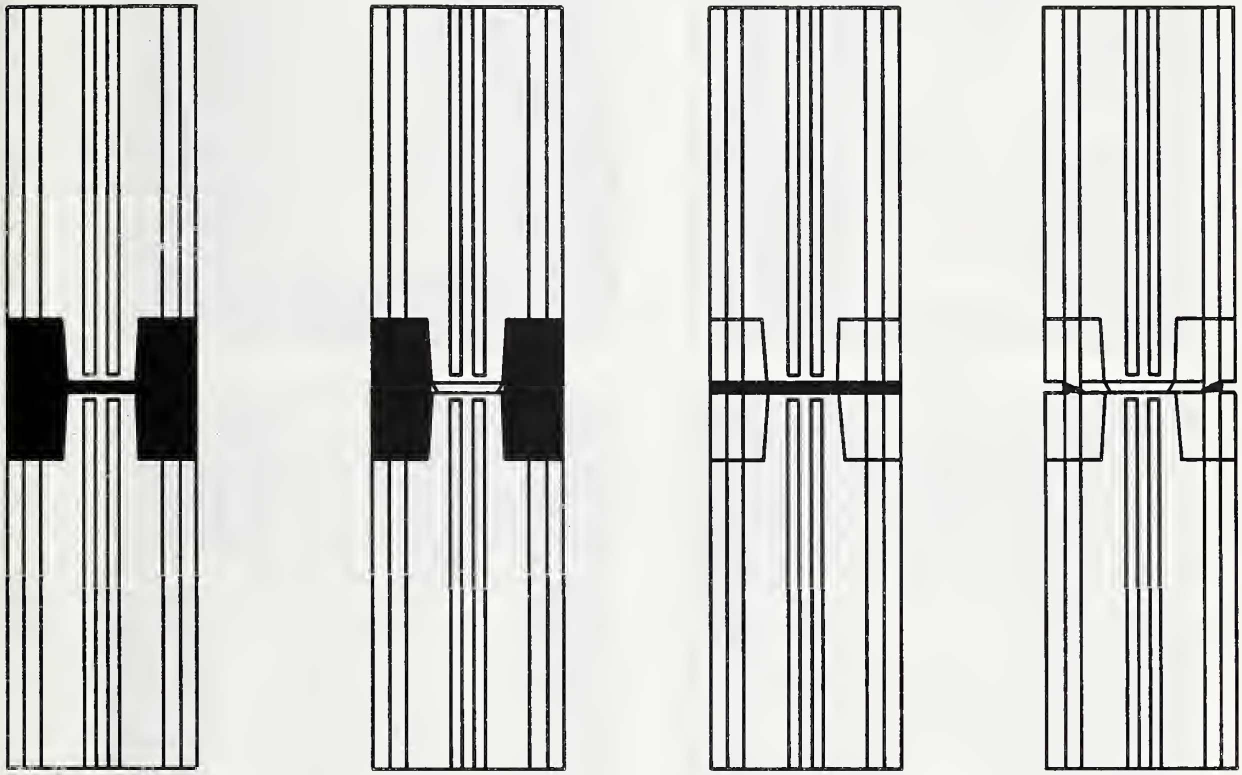


Figure 5.5.4. Formwork for column stub with reinforcing cage in place (top): Close-up view of the reinforcing cage (bottom)



(a) Full joints (b) Concrete only (c) Grout only (d) Empty joints

Figure 5.5.5. Types of joints in the test program

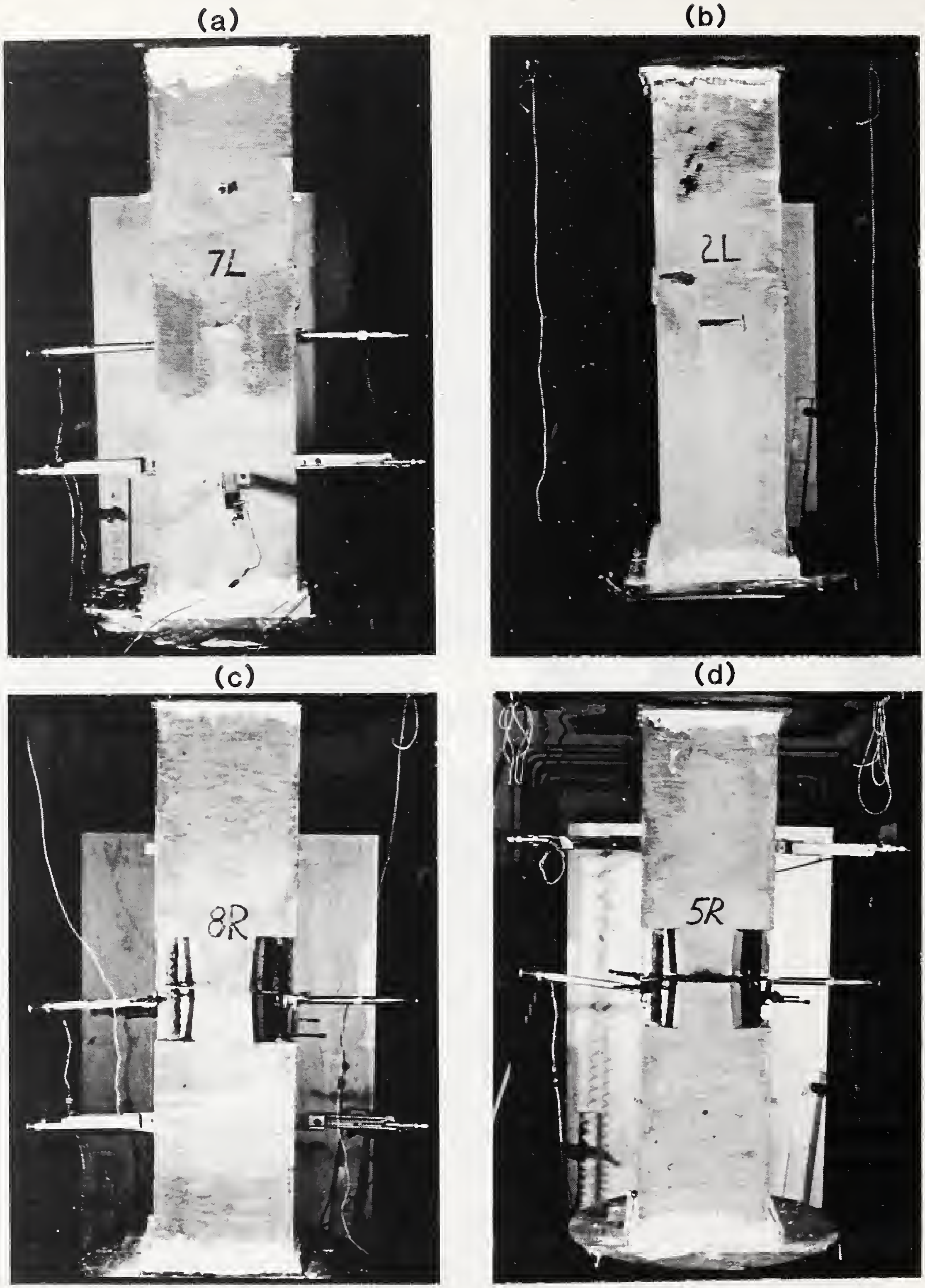


Figure 5.5.6. Specimens prior to testing showing the four joint conditions: a) full: b) concrete only: c) grout only: and d) empty

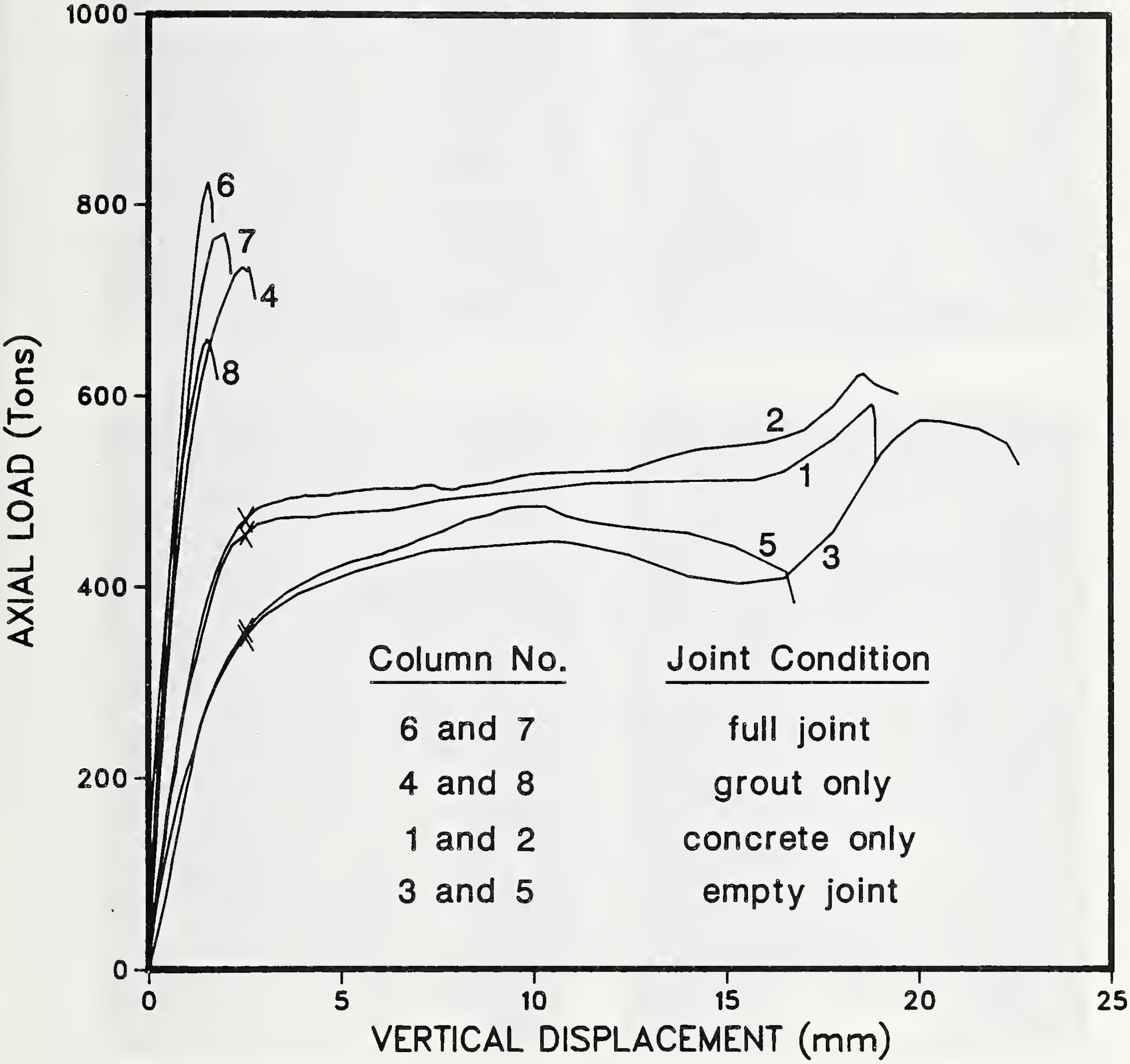


Figure 5.5.7. Load-displacement results for the eight specimens

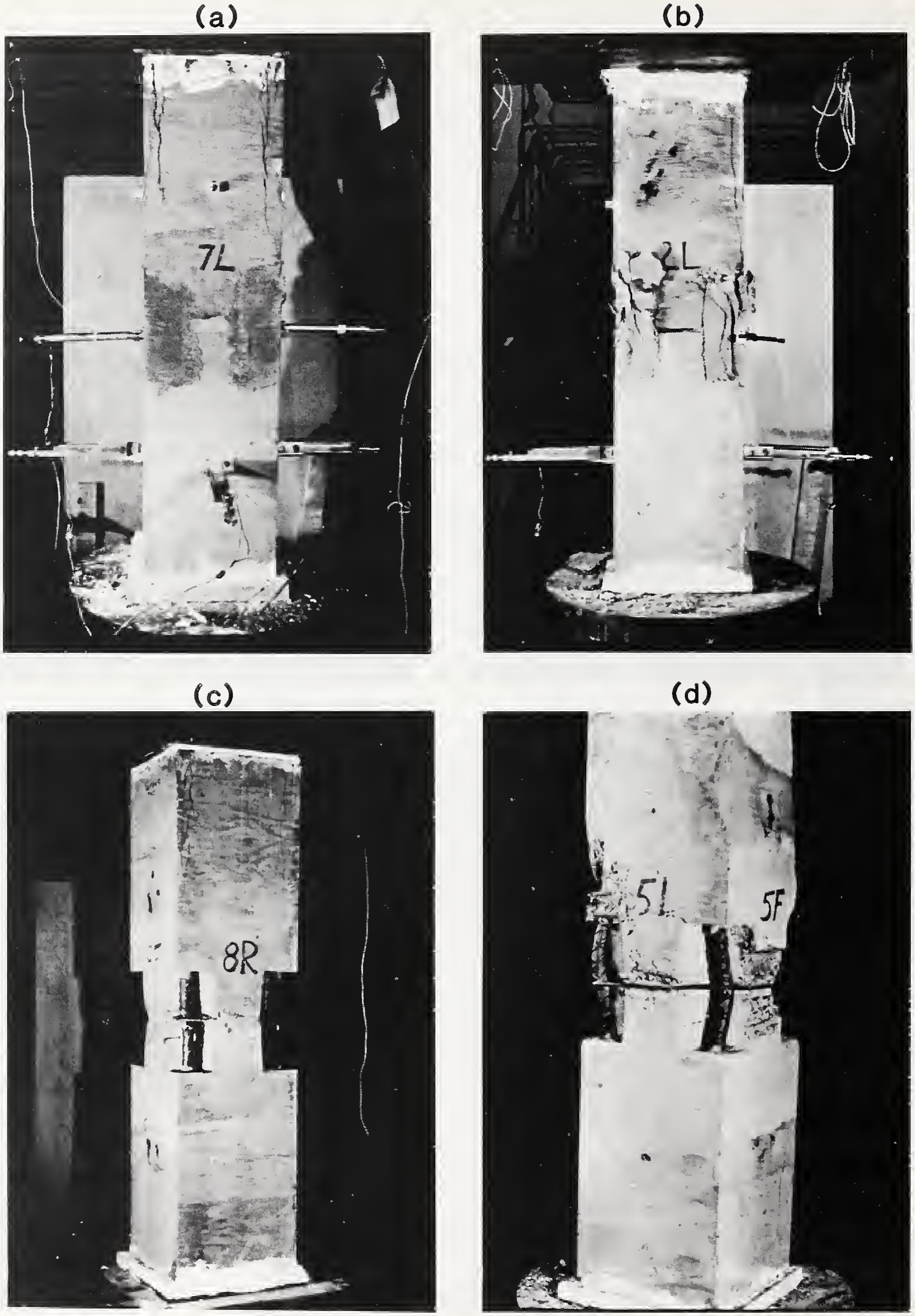


Figure 5.5.8. Failed test specimens: a) full; b) concrete only; c) grout only; and d) empty

Figure 5.6.1. Examination of exterior walls using technical mountaineering equipment

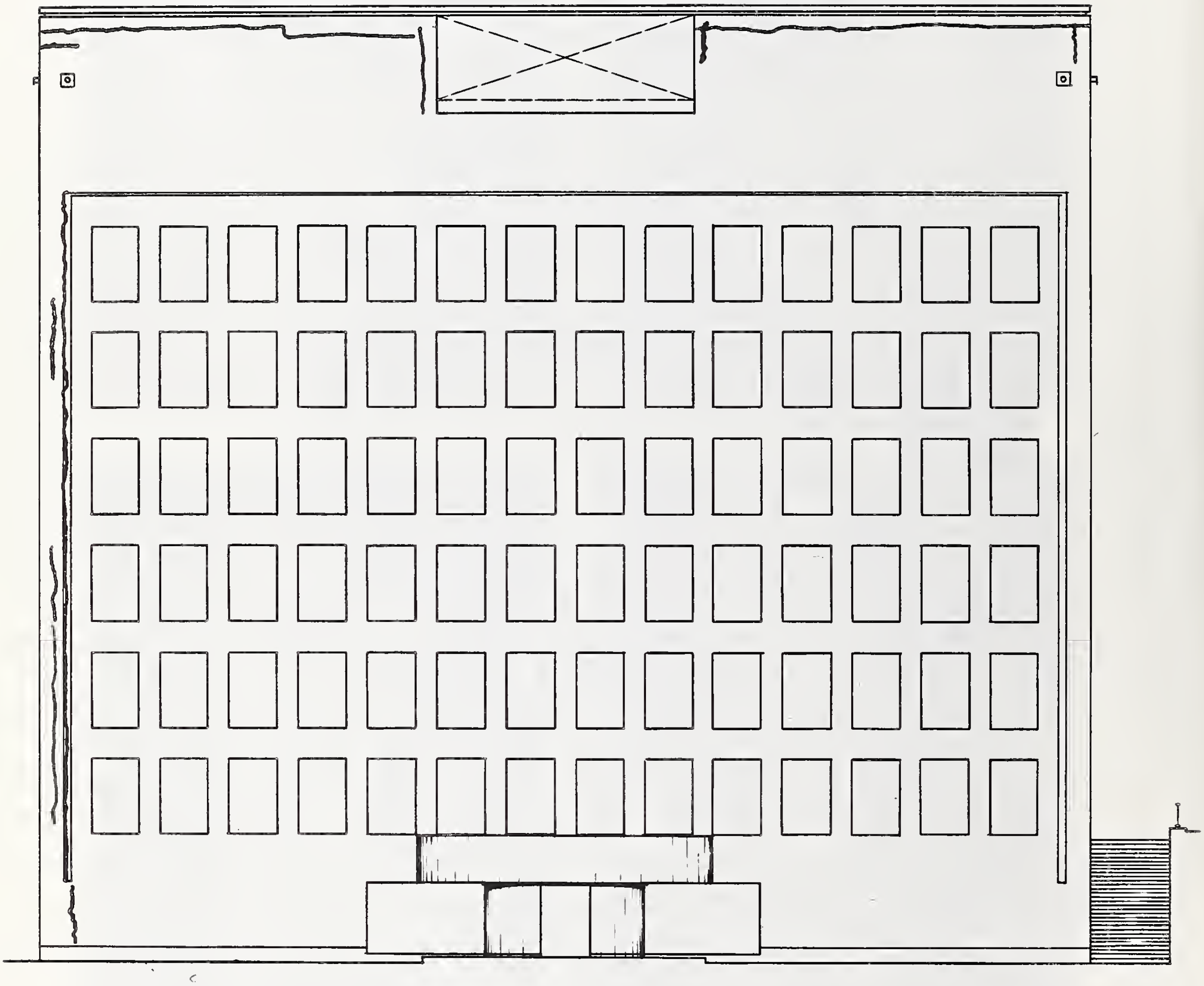


Figure 5.6.2. Crack patterns on south elevation

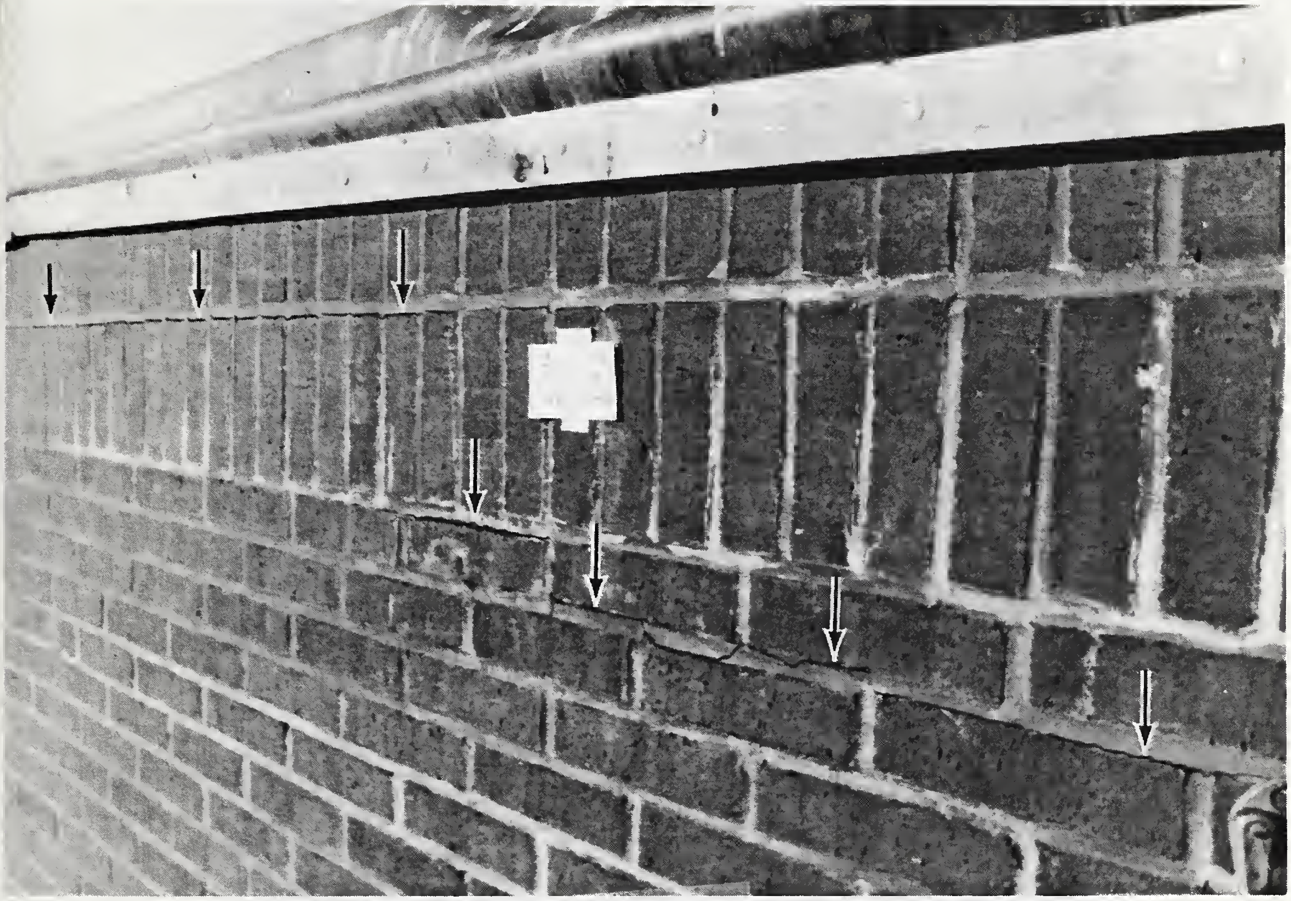


Figure 5.6.3. Horizontal cracks on west side of south elevation of parapet wall

Figure 5.6.4. Vertical crack west of louver opening on south elevation

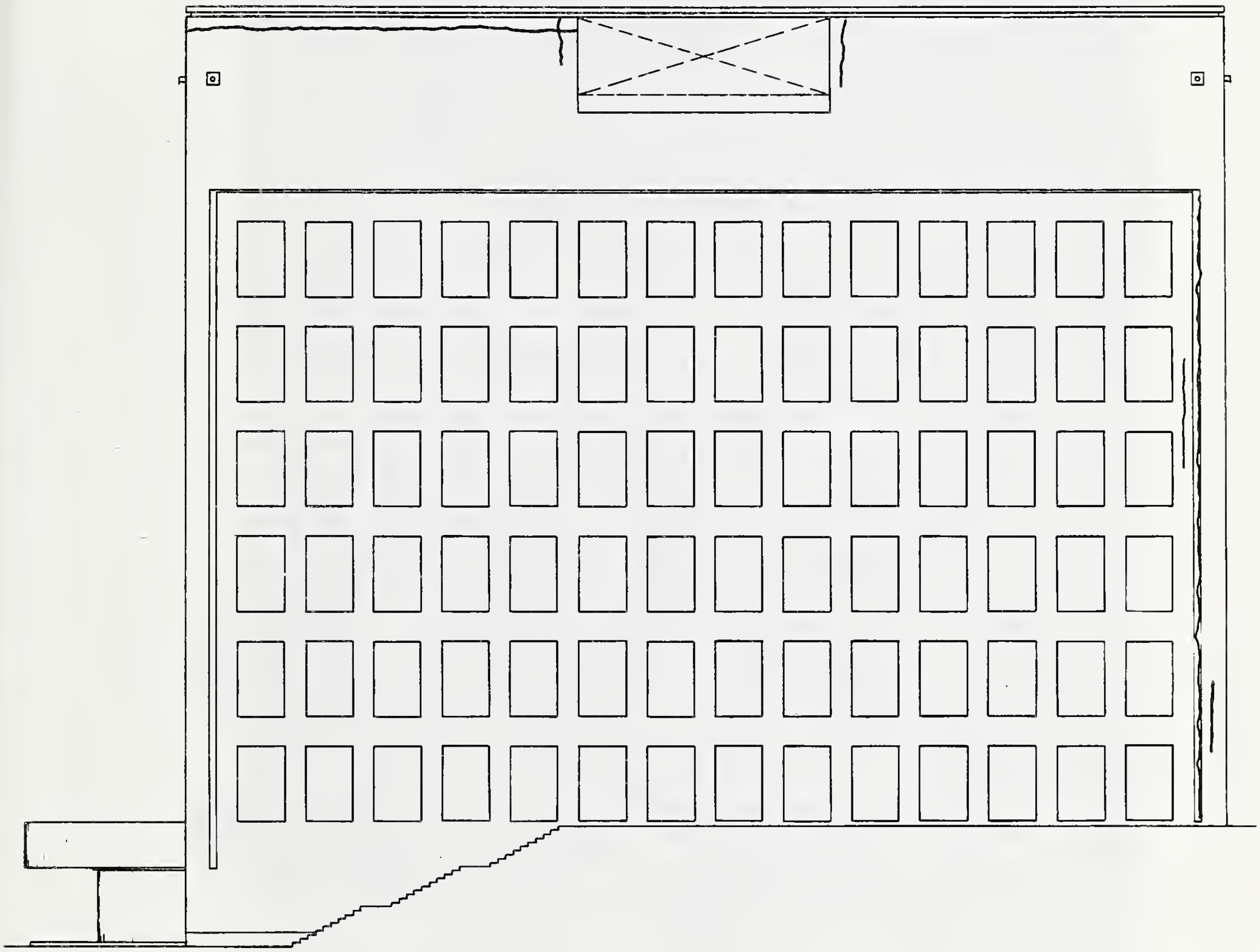


Figure 5.6.5. Crack patterns on east elevation

Figure 5.6.6. Ground level view of northeast corner, east elevation

Figure 5.6.7. Crack in recess of north corner, east elevation

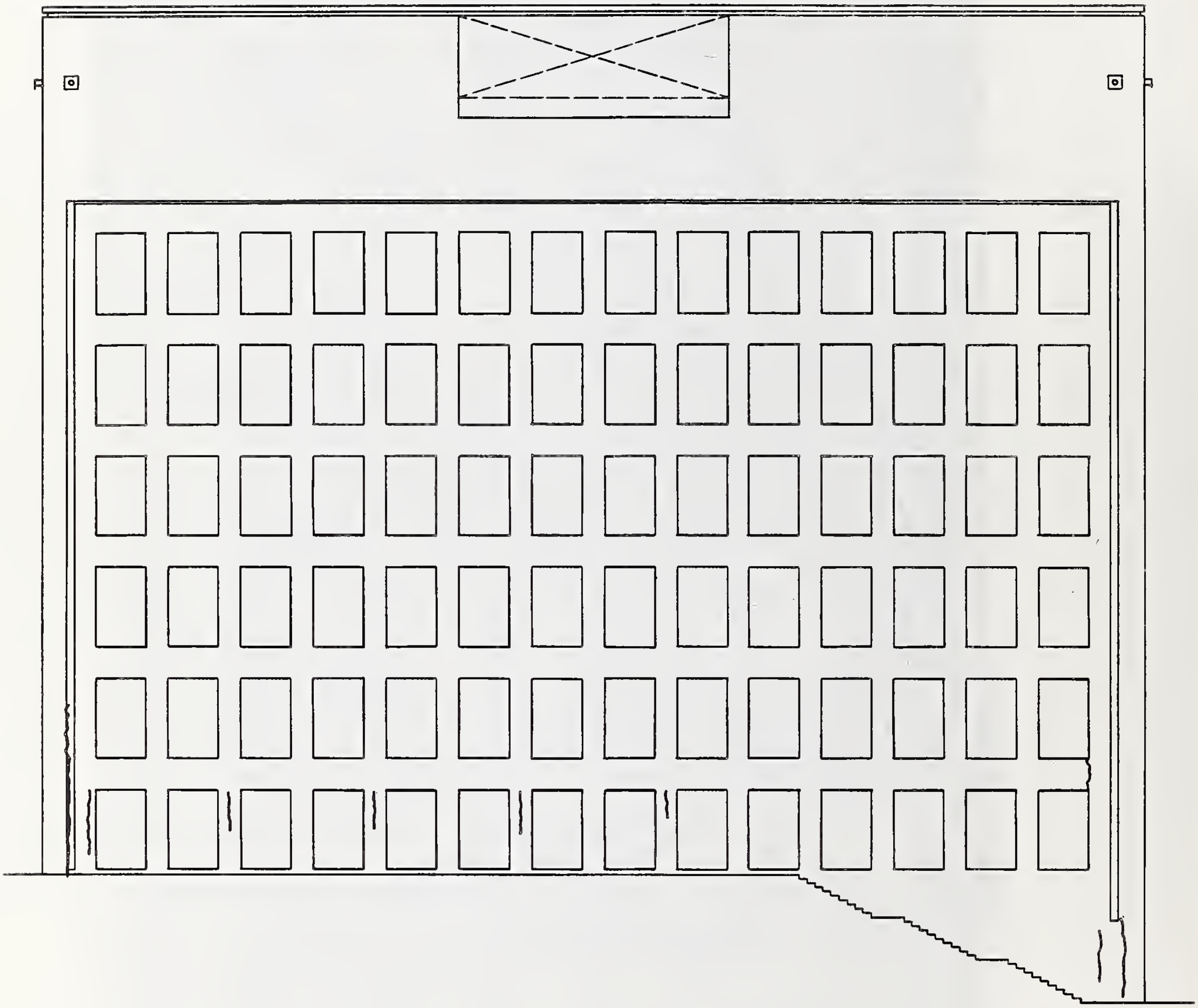


Figure 5.6.8. Crack patterns on north elevation as observed from ground level

Figure 5.6.9. Crack near window at east corner of north elevation

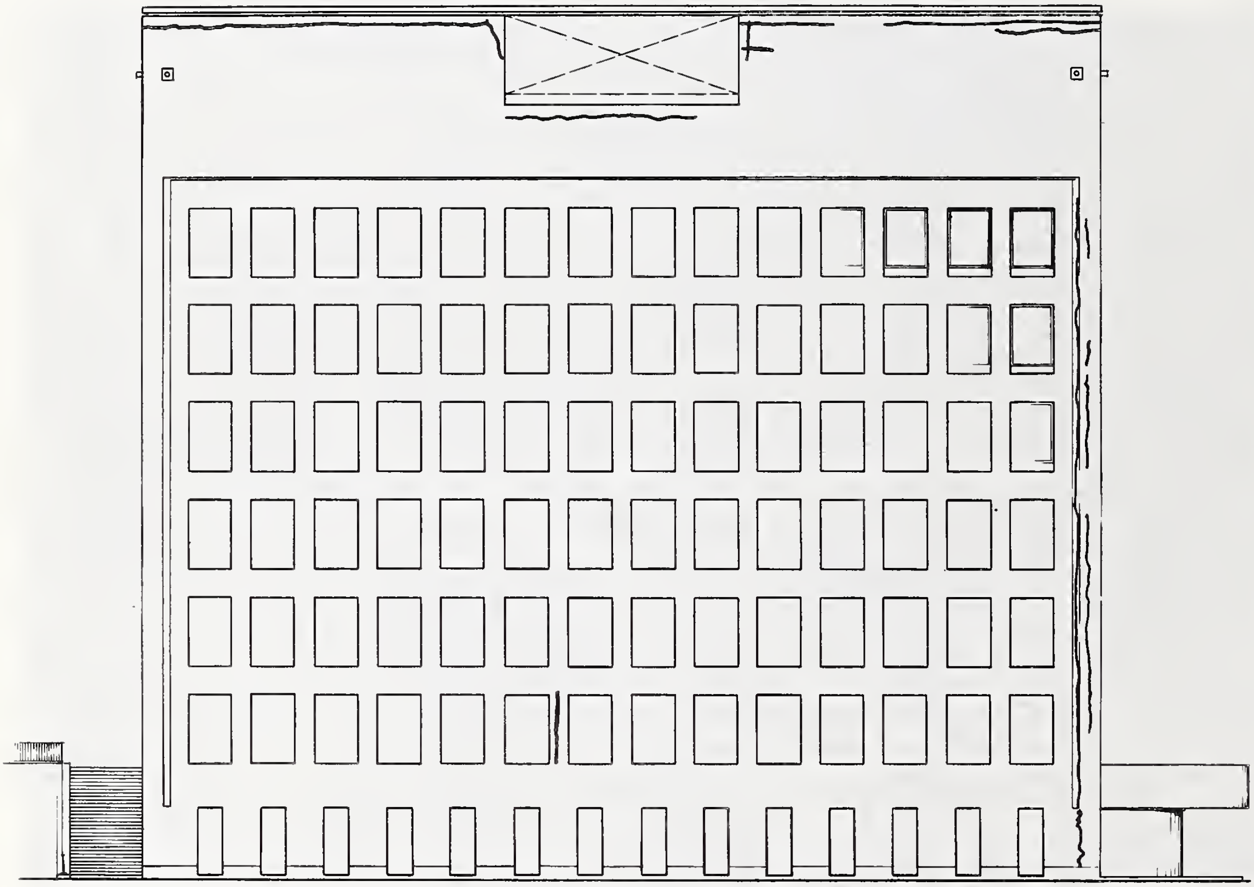


Figure 5.6.10. Crack paterns on west elevation

Figure 5.6.11. Crack in second story pier at column A1/5, west elevation

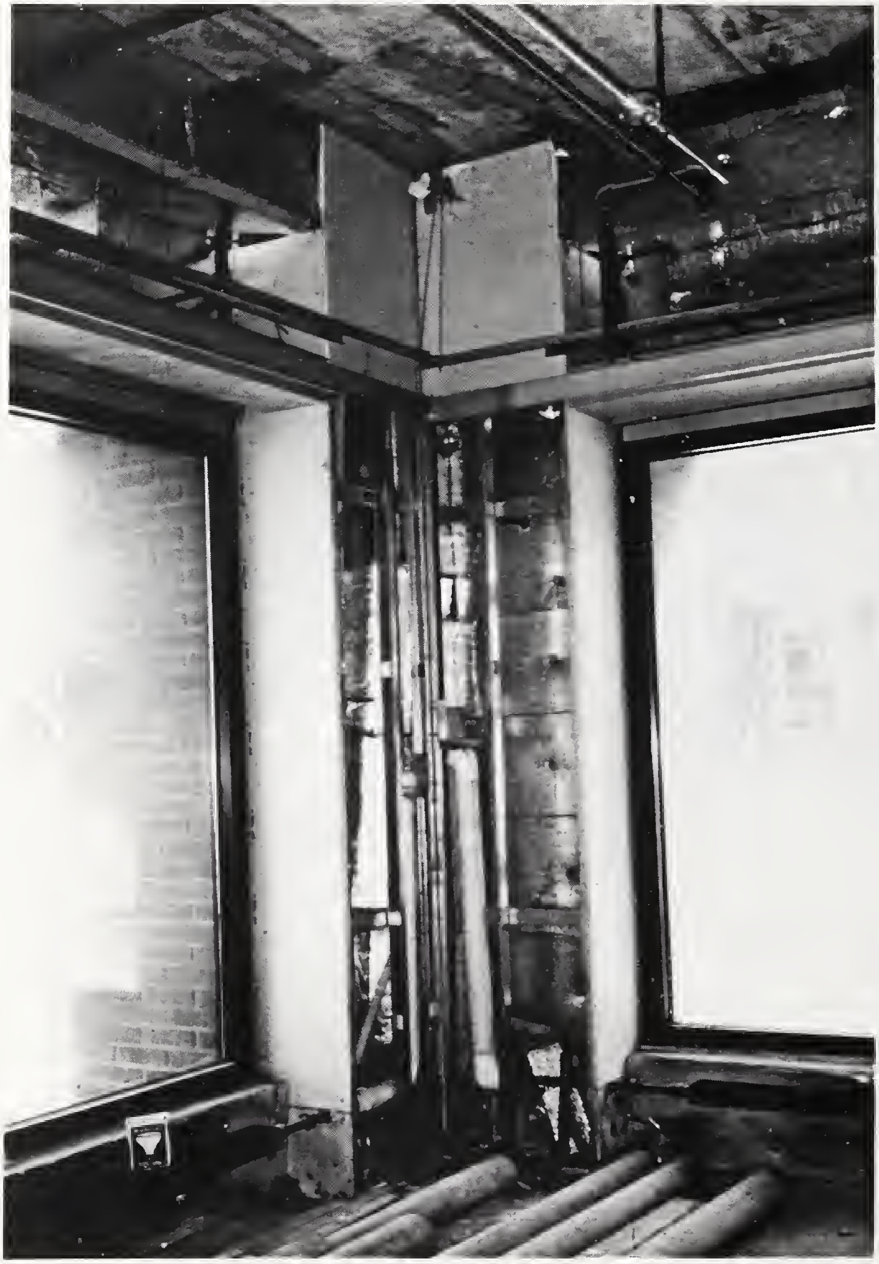


Figure 5.6.12. Insulation in northeast corner in second story

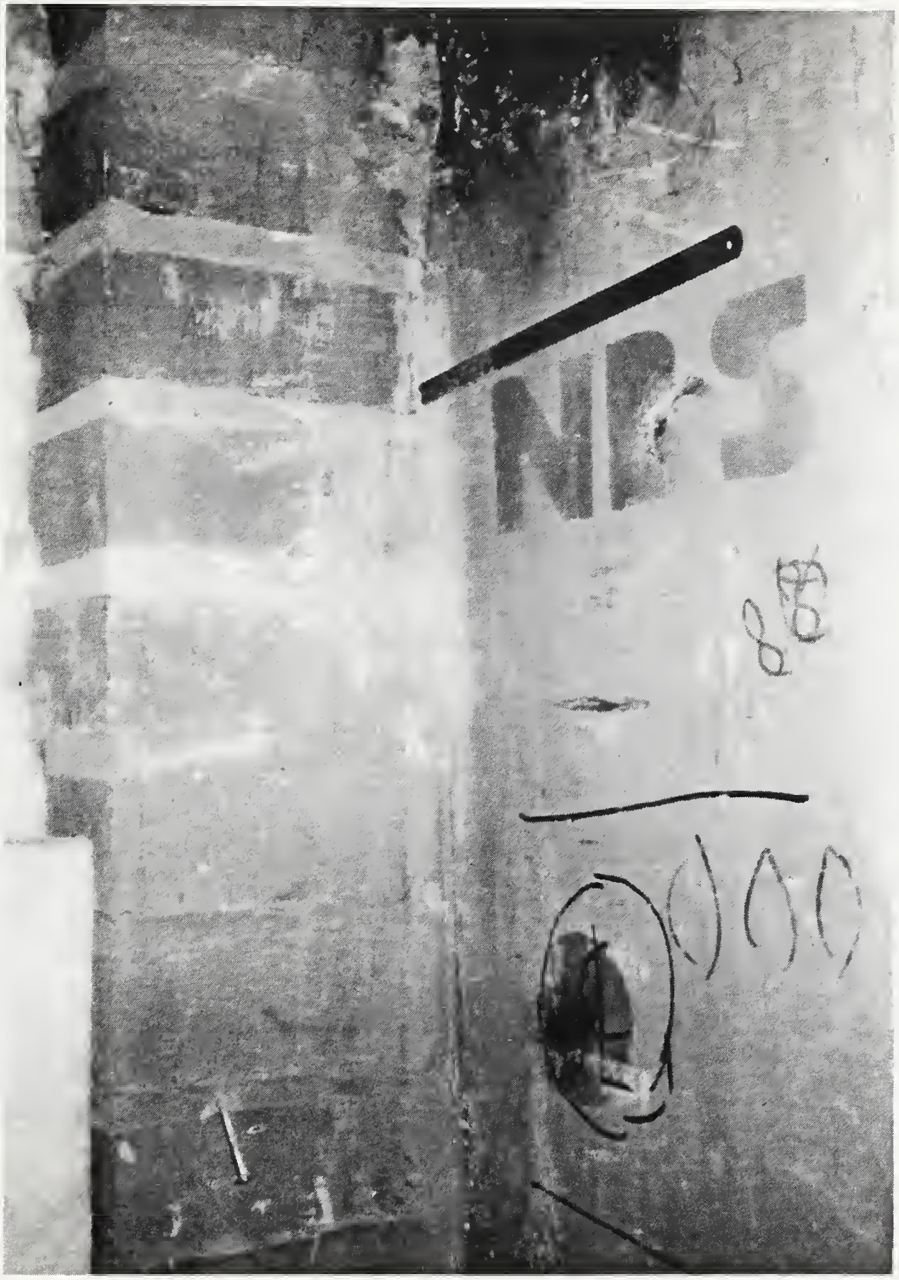


Figure 5.6.13. Crack between column A1/5 and brickwork in second story

Figure 5.6.14. Core from eighth story wall showing bond of concrete to brick

Figure 5.6.15. Anchor between masonry and column

Figure 5.7.1. Northeast section of roof showing parapet walls, snow melting room and recess in penthouse wall

Figure 5.7.2. Cracks at intersection of parapet and penthouse walls

Figure 5.7.3. Brick removed at intersection of vertical and horizontal cracks in penthouse wall, northeast section

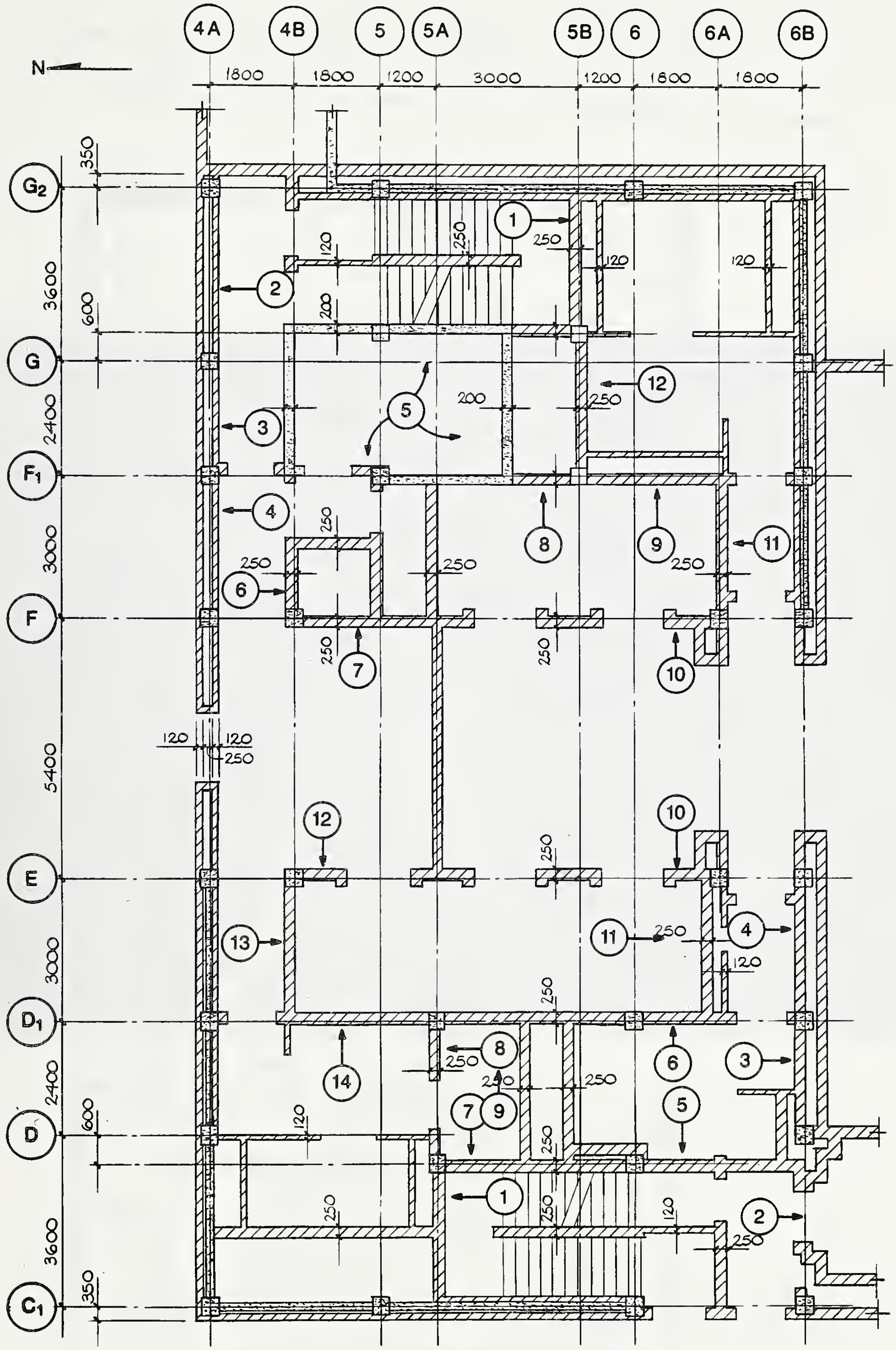


Figure 5.8.1. Partition walls in core of first story

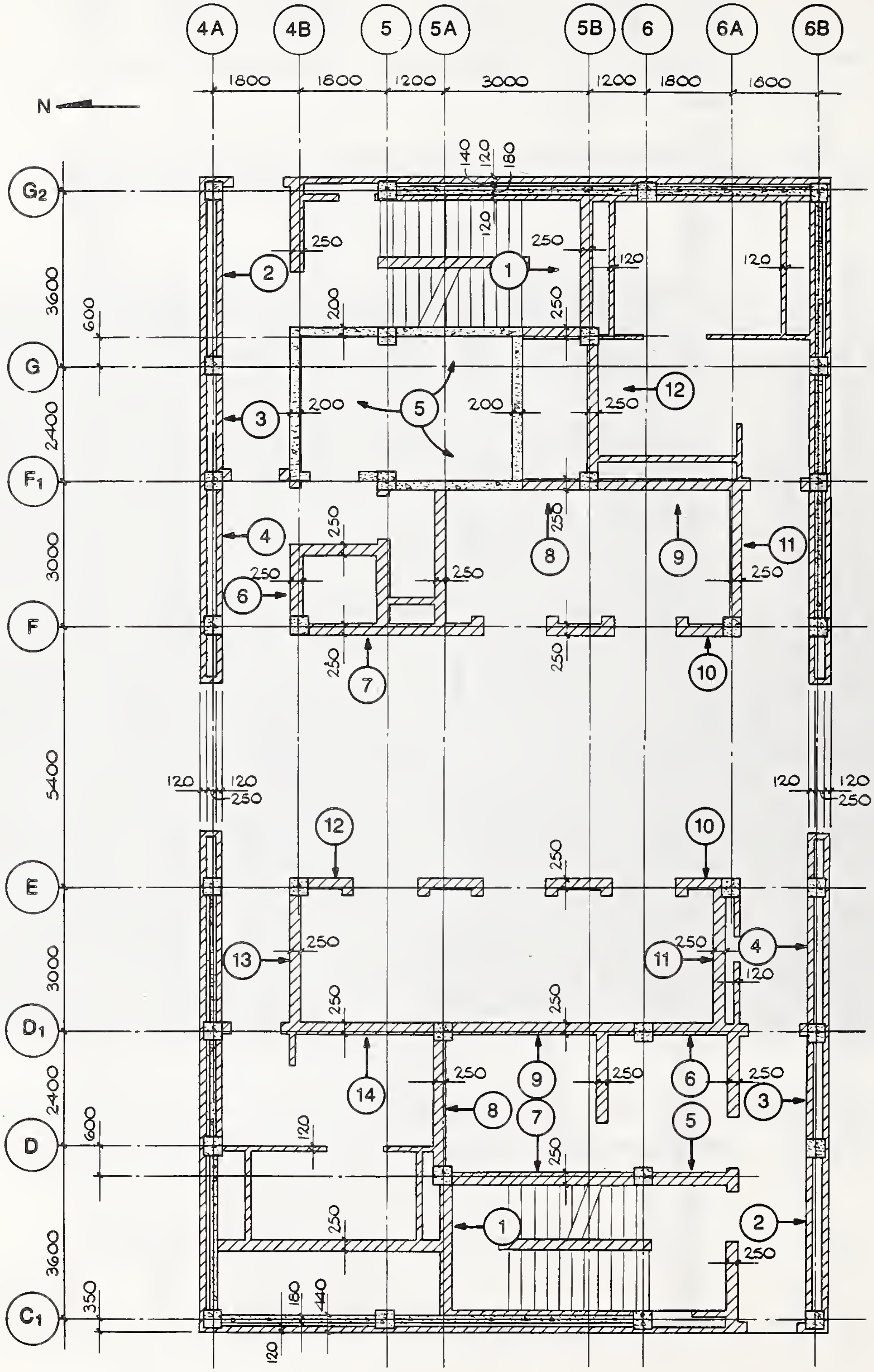


Figure 5.8.2. Partition walls in core of second through eighth stories

Figure 5.8.3. Good quality partition wall in stairwell

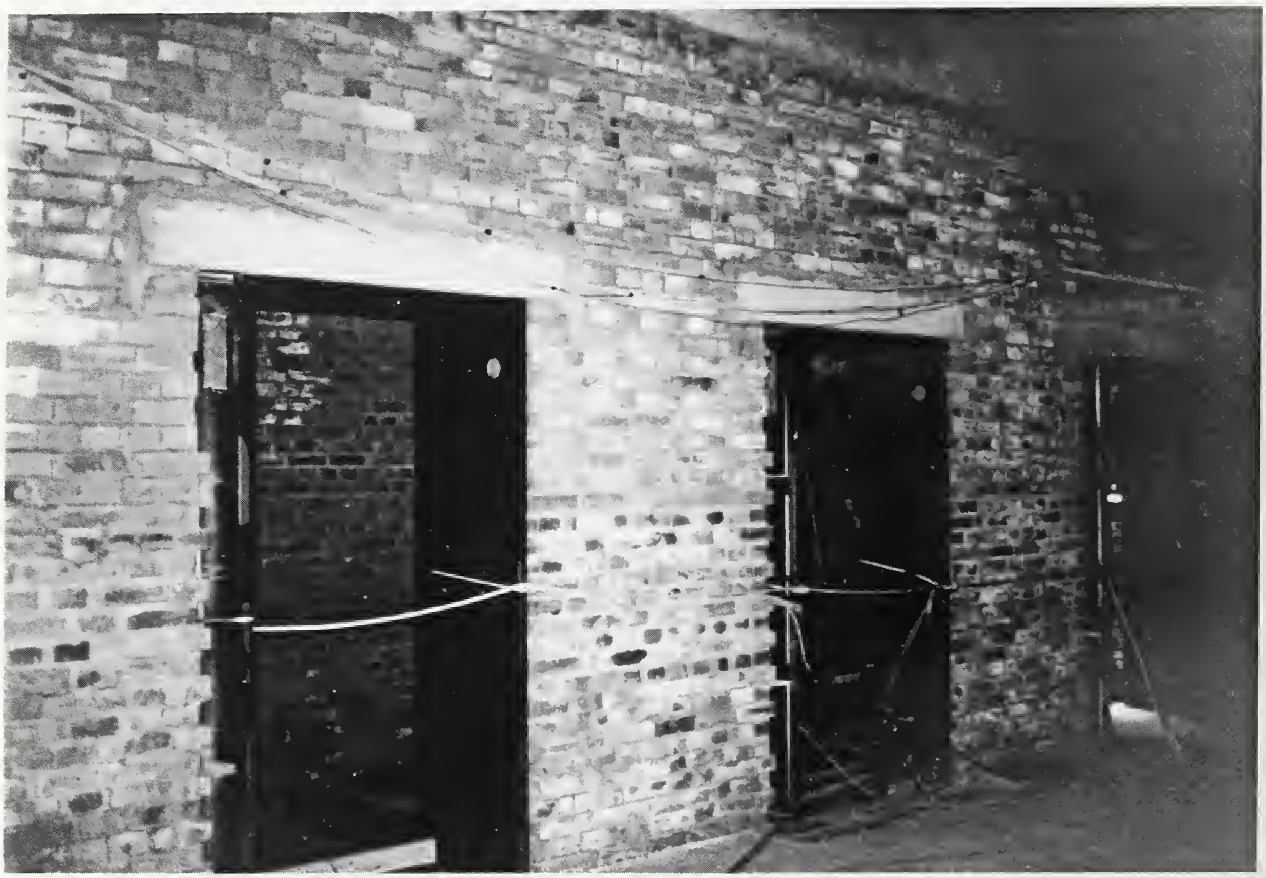


Figure 5.8.4. Brick walls at elevators

Figure 5.8.5. Duct penetration in partition wall

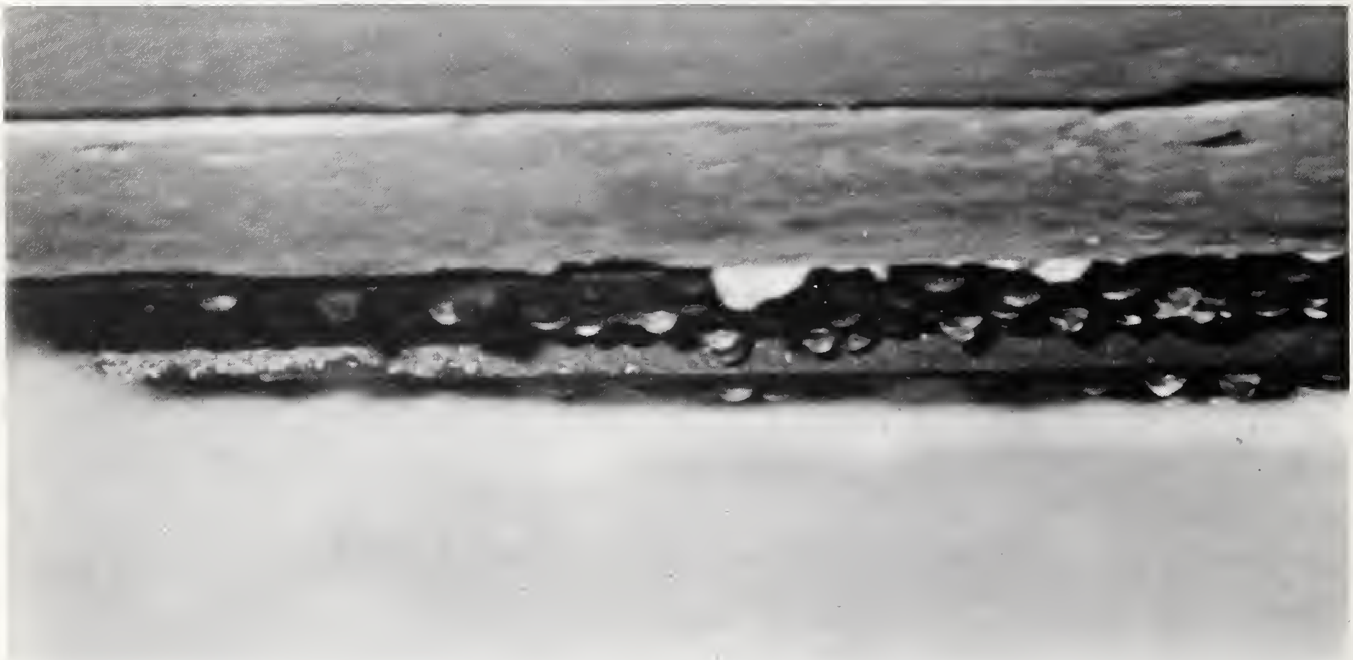


Figure 5.9.1. Water in soffit cavity above windows

Figure 5.10.1. Wall specimen marked for cutting

(a) (b)

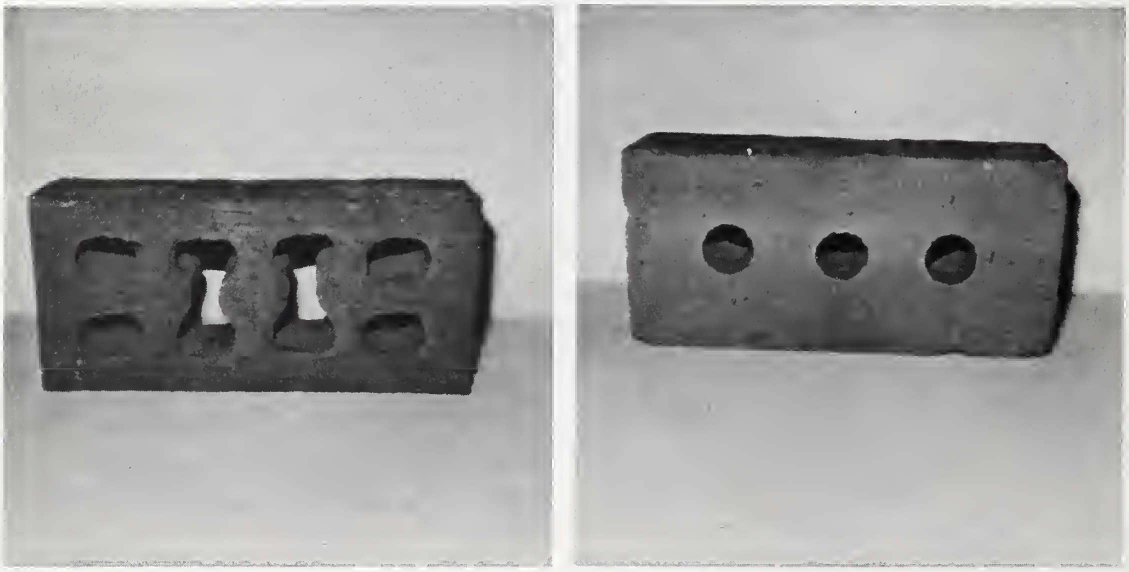


Figure 5.10.2. Appearance of a) facing brick, and b) building brick

Figure 5.10.3. Laboratory-built prism and panel specimens

Figure 5.10.4. Diagonal tension specimen taken from Office Building

CHAPTER 6
ASSESSMENT OF EXISTING STRUCTURE AND RECOMMENDATIONS FOR REMEDIAL MEASURES

6.1 INTRODUCTION

Chapter 4 reviewed the design of the structural system of the Office Building based on the loading and resistance criteria adopted in Chapter 3. Some structural elements were found to have less than the required strength. Chapter 5 reviewed information gathered during the site investigation and from laboratory tests. Important observations affecting the structural performance of the Office Building include:

- Column joints in the Office Building are not properly filled.
- Shear wall joints are not properly filled.
- Proper grouting of column joints is essential for adequate joint behavior.
- Proper filling of shear wall joints is required to achieve monolithic shear wall behavior.
- The compressive strength of concrete cores taken from various elements in the structure were in excess of specified values.
- The exterior brick masonry walls have severe cracking.
- Progressive collapse apparently was not considered in the design; alternate load paths do not presently exist for some structural members.

This chapter assesses the integrity of the as-built structure in light of current U.S. design practice. Where deficiencies exist, remedial measures are recommended to bring the structure to a level of safety that is consistent with good U.S. practice. The resistance of the structure to vertical and lateral loads is assessed first. Next, recommendations are given for making the structure resistant to progressive collapse. Finally, remedial measures are suggested for repairing the exterior brick masonry and for improving future performance of the enclosure. Cost estimates for the remedial measures are based on April 1987 prices for the Washington, D.C., area.

6.2 RESISTANCE TO VERTICAL AND LATERAL LOADS

6.2.1 Floor slabs

The design review of precast planks indicated no deficiencies. Other than obvious damage to some planks which occurred during construction, no evidence of structurally induced distress was noted during the site investigation. The damaged planks have been noted by FBO and are being considered for replacement. The cost of these repairs has not been included in this assessment. Thus no remedial measures are recommended for the floor planks as part of this study. However, modifications to the floor system to make the structure resistant to progressive collapse are discussed in the next section.

6.2.2 Beams

The design review of steel beams and their connections indicated no significant deficiencies. The field investigation gave no indications that steel with lower than the specified yield strength was used. Thus no remedial measures are required for the structural steel beams.

The design review of concrete beams revealed that two of the R-38-8 beams in the first floor were underdesigned. The required flexural strength was 19.6 t·m, whereas the design strength was 13.7 t·m. As reported in Chapter 5, cores taken from one of the heavily loaded beams had a compressive strength of about 550 kg/cm^2. While this is considerably greater than the compressive strength used in the design review, the flexural capacity is only slightly affected by the increased concrete strength. This is because the beam is designed so that its strength is controlled by the amount of tension steel rather than the strength of concrete. The flexural strength based on core strength is 13.9 t·m.

During the site investigation, the sources of loading on the critical R-38-8 beams were examined. In conformance with the Soviet architectural drawings, these beams support masonry walls as shown in figure 6.2.1. The tributary floor area contains sizeable cutouts where live loads cannot act. A revised loads calculation was performed for beam D1/5A-D1/6 using a unit weight of 1.9 t/m^3 for the 250-mm thick masonry walls. The resulting unfactored loads on the beam are shown at the bottom of figure 6.2.1. The analysis assumes that the wall directly above the beam imparts a uniformly distributed dead load. However, the real moment produced by the dead load of the wall is smaller than computed by assuming a uniformly distributed load because of arching action of the masonry wall [6.1].

To assess the adequacy of the R-38-8 beams, load factors are applied to the loads shown in figure 6.2.1 to obtain the required strength. Load factors are intended to allow for uncertainties in the loading. For these beams, dead load is predominant. Since the dead load in an existing structure is well-defined, it would be reasonable to use a lower load factor in the assessment than the value of 1.4 used in the design situation. This approach is permitted by section 20.2.2 of the ACI Code [6.2] when performing an evaluation of an existing structure by analysis, provided that the Building Official is satisfied that the reduced load factor does not violate the intent of the Code. Thus a dead load factor of 1.2 and a live load factor of 1.7 were used to compute the required flexural strength. The resulting required strength is 14.5 t·m. The design strength of 13.9 t·m is judged to be sufficiently close to the required strength. In addition, for a concrete strength of 550 kg/cm^2, the design shear strength of the beam is 18.1 t, which exceeds the required strength of about 13 t. It is concluded that the R-38-8 beams are not critically overloaded.

Based on the above analysis, it is concluded than no remedial measures are required for the concrete beams. The connections of concrete beams to columns were in general conformance with the plans, and no remedial actions are required.

6.2.3 Columns

6.2.3.1 Reinforced concrete columns

The design review indicated that 35, Type 5-RC reinforced concrete columns were overloaded. The column design strength of 417 t is based on Mark 600 concrete with a design cylinder strength of 357 kg/cm^2. The results of tests on cores taken from ten Type 5-RC columns indicate an average compressive strength of 596 kg/cm^2 and a coefficient of variation of 0.10. Assuming concrete strengths are normally distributed, the 10th percentile strength is 511 kg/cm^2. Thus the column strength should be re-evaluated considering greater in-place concrete strength.

Using a cylinder strength of 511 kg/cm^2, the design strength of the Type RC-5 columns is 529 t (see section 4.2.4.2 for procedure), and 34 of the previously overloaded columns have sufficient design strength. Column F/4B in the basement, for which the required strength from the vertical load analysis is 592 t, still is overloaded. However, other factors should be considered.

During the design stage, the design strength of a concrete member is computed by multiplying its nominal strength by a strength reduction factor. For columns with lateral ties, the reduction factor specified in the ACI Code is 0.7. The purpose of the reduction factor is to account for uncertainties in such factors as material strength, workmanship, member geometry, and complexities involved in the analysis. However, in assessing the capacity of an existing structure, some of these uncertainties are absent: the member geometry and material strengths can be measured. Thus it is reasonable to use larger values of the strength reduction factor. Unfortunately, Chapter 20 of the ACI Code does not provide specific guidance on how much of an increase is permitted. The matter is left up to the judgment of the engineer pending concurrence by the Building Official that the intent of the Code is being satisfied.

In assessing the in-place design strength, it is reasonable to increase the strength reduction factor from 0.7 to 0.8. By doing so, the design strength of the Type 5-RC column, with a compressive strength of 511 kg/cm^2, is increased to 604 t. This exceeds the required strength of 592 t for the most heavily loaded Type 5-RC column. Thus no remedial measures are required.

6.2.3.2 Steel core composite columns

The design review reported in section 4.2.4.2 indicated that Type SC-9 composite columns in the second and third stories at column lines C1/6B and G2/4A are inadequate because of the slenderness effect about the weak axis (fig. 2.3.6). The review assumed a concrete cylinder strength of 241 kg/cm^2 and an associated elastic modulus computed according to the ACI Code. The average compressive strength of cores taken from Type SC-9 columns is 509 kg/cm^2 (table 5.3.8). Individual strengths were in excess of 500 kg/cm^2 except for a core taken from column 2-G2/4A which had a strength of 330 kg/cm^2. It is not known whether the low strength is indicative of this entire column or only of the location where the core was taken. To obtain an upper bound on the probable capacity of the Type SC-9 columns, the axial load-moment interaction diagram for the cross section was determined for a concrete strength equal to 500 kg/cm^2. In this case a strength reduction factor of 0.7 was retained because of the added complexity associated with slender column analysis. Figure 6.2.2 shows the computed design strength interaction diagram. Using an unbraced length of 3.9 m and a factored axial load of 735 t, which is the heaviest load for the Type SC-9 column (2-G2/4A), the magnified bending moment, due to the slenderness effect, is 226 t·m. The combination of axial load and moment is plotted on the figure and it exceeds the design strength envelope. Thus these Type SC-9 columns require bracing to reduce their slenderness.

To obtain an estimate of the maximum allowable unbraced length, the strength of column 2-G2/4A was determined assuming a concrete strength equal to the core strength, that is, 330 kg/cm^2. It was found that for an unbraced length less than 3.6 m, slenderness is reduced sufficiently so that the Type SC-9 column can safely support the factored axial load of 735 t.

Figure 6.2.3 is a plan view of the brick masonry around column 2-G2/4A. There is a partition on the west side, and there is a partition with a doorway, approximately 2.2 m high, on the south side of the column. It could be argued that the surrounding single wythe of masonry would provide bracing for the column. However, this relies on the tensile strength of the single-wythe wall. It would be more prudent to provide anchorage to the thicker wall above the doorway. A possible scheme is to anchor an L-shaped strap (100 × 10-mm in cross section) to the masonry wall (fig. 6.2.3). The strap should be anchored at a height of 3 m. For an unbraced length of 3 m, the combination of axial load and magnified moment falls well within the design strength envelope, as shown in figure 6.2.4.

6.2.3.3 Column joints

The site investigation uncovered instances where the 20-mm gap between segments of reinforced concrete columns was not properly filled with grout as required in the design. The NBS tests of column joint specimens indicated that the absence of grout can result in more than a 50 percent reduction in the axial load capacity. In the course of the site investigation, it was not possible to inspect all joints of reinforced concrete columns. Hence it is recommended that all joints between reinforced concrete columns be exposed and inspected by drilling two holes into the 20-mm gap. The holes may be drilled from the same face of the column but should be located on either side of the seating button. The interior of the joints should be inspected with a borescope. In cases where voids are found within the 20-mm gap, high strength grout (550 kg/cm^2 compressive strength) should be pressure injected into the holes to fill the voids. In cases where the exposed joints also contain voids in the pockets around the corner reinforcing bars, stiff grout or concrete should be applied to fill the voids.

6.2.4 Shear wall joints

The conditions of the inspected horizontal and vertical joints in the shear wall system were highly variable, ranging from complete filling, as required by the plans, to open joints.

The analysis presented in Chapter 4 indicates that performance of the shear wall system in terms of strength and stiffness is improved with filled joints. Thus it is recommended that all vertical and horizontal shear wall joints be exposed and inspected. Unfilled vertical joints should be filled with grout having a compressive strength of at least 200 kg/cm^2. Unfilled horizontal joints should be filled with concrete having an average cylinder strength of 250 kg/cm^2.

6.3 RESISTANCE AGAINST PROGRESSIVE COLLAPSE

Chapter 4 analyzed the susceptibility of the structure to progressive collapse. The following remedial measures are recommended to provide for alternate load paths in the event of a member failure.

6.3.1 Restraints for Long-Span Beams

The cast-in-place slab over the eighth floor planks lacks continuous steel reinforcement, and neither membrane action nor slab suspension over two spans can be achieved. If a supporting beam were to fail, progressive collapse would occur as a panel of the heavy eighth floor slab would fall through the floor below. Thus remedial measures are needed to suspend the floor panel in the event that one of the beams should fail or loose support.

Progressive collapse would also occur in the case of a failure of long-span beams supporting floors 2 through 7 along column lines C and H. If other long-span beams supporting floors 2 to 7 were to fail, the floor planks and topping have marginal ability to resist falling planks. However data to assure the resistance are not available. Therefore remedial measures are also recommended to suspend the floor panels in floors 2 through 7 in the event of long-span beam failure.

Beam collapse can be restrained by welding ductile steel straps to the underside of the flanges of the beams to suspend the entire floor system in tension.

The long-span beams supporting the eighth floor have 500-mm wide flanges which provide a 130-mm wide support shelf for the 5160-mm long floor planks. Permitting the floor to sag a maximum of 1 m leaves a 30-mm support shelf for the planks. Thus at a 1-m sag the planks will not slip off their support. If the straps are spaced 1.2 m on center and the load on the floor system is 1.254 t/m^2, the force that must be resisted by each strap for the 10.8-m span is:

$$T = 1.2[1.254(5.4)(2.7) + 0.122(5.4)](5/4.9) = 23.2 \text{ t}.$$

For the 11.4-m span the force is 24.5 t. The required elongation is 0.22 m, and the strain in the straps is (0.11/4.9)(100) = 2%, a strain which is acceptable for mild steel. Assuming that A36 steel straps are used, the required cross sectional net area is 987 mm^2.

Details of the proposed eighth floor strapping are shown in figure 6.3.1. Straps with a cross section of 76 mm × 13 mm are suggested. Connections of the straps should develop their tensile strength; fillet welds 10 × 260 mm (130 mm on each side of the strap) using E70XX electrodes would be adequate (the welded connection must resist about twice the yield capacity of the strap to account for strain hardening). The straps in the exterior spans can be fastened into the plank/slab system using 25-mm anchor bolts (four per strap) as shown in figure 6.3.1. The exterior straps should be 90-mm wide at the anchored ends to provide a net width of 101 mm. Alternatively, the exterior straps can be fastened to the spandrel beams using a detail similar to that shown in figure 6.3.2. If this alternative is selected, it is necessary to anchor the spandrel beam to the cast-in-place concrete slab to increase the lateral-load resistance of the spandrel beam.

The long-span beams supporting floors 2 through 7 have 400-mm wide flanges which provide an 80-mm wide support shelf for the 5160-mm long floor planks. Permitting the floor to sag a maximum of 0.8 m leaves a 16-mm support shelf for the planks. Thus at a 0.8 m sag the planks will not slip off their support. If the straps are spaced at 1.2 m on center and the load on the slab is 0.81 t/m^2, the force that must be resisted by each strap is calculated as:

$$T = 1.2[0.81(5.4)(2.7) + 0.102(5.4)](5.06/5)(1/0.8) = 18.6 \text{ t}$$

For the 11.4-m span the force is 19.6 t. The required elongation is 0.22 m, and the strain in the straps is (0.11/4.9)(100) = 1.2%. The required cross-sectional net area for the steel straps is 773 mm^2. Straps with a cross section of 13 × 60 mm are adequate, and 10 × 200-mm fillet welds are recommended. At column lines A1 and J1 the straps should be welded to steel angles which are anchored to the Z-spandrels. The spandrels, in turn must be anchored to the masonry beam to insure shear transfer. Details of the strapping are given in figure 6.3.2.

6.3.2 Core Partitions

The masonry walls will provide alternate load paths in the event of a column failure. The exterior masonry is well connected to the structural frame and has sufficient cross-section to provide the necessary load transfer in case an exterior column were to fail. The partitions in the core, however, require remedial work to provide the alternate load paths.

The 2-wythe brick masonry walls along lines 4A and 6B should be strengthened by filling the cavities with concrete of at least 300 kg/cm^2 compressive strength. The concrete can be pumped into place through openings made into the masonry; the two wythes should be tied prior to placement of concrete to prevent displacement by the pressure of the fresh concrete. Care should be taken to ensure that concrete completely fills the voids under the beams.

Shear connections should be installed between the brick masonry and cast-in-place concrete walls along column lines F1 and G1 in stories 1 through 8. The connection should have at least 40 t shear capacity per story. A recommended approach is to join the two types of walls by placing steel angles and plates across the joint and anchoring them into the walls. In each story, two 75 × 75 × 12-mm, 300-mm long angles should be anchored at the interior corners using four 25-mm bolts; on the flush side, 300 × 300 × 12-mm plates, should be used.

The shear capacity of the connection of the second floor steel box beam at column C1/6B should be increased by 12 t. This can be accomplished by welding a connecting plate between the beam and column corbel using a 10 × 120-mm fillet weld (E70XX electrode).

In addition to these measures, the joints between all brick walls and the surrounding structural members should be tightly filled. This is further discussed in the next section. Also, the gaps between reinforced concrete beams and adjacent columns should be inspected, and all unfilled gaps should be grouted.

6.4 ASSESSMENT OF BRICK MASONRY

6.4.1 Bases for Assessment

The bases for the assessment of brick masonry walls and the recommendations for remedial measures are:

- A review of U.S. and Soviet plans and specifications;
- An analysis of the masonry based upon U.S. engineering practice as applied to the Moscow site and climate;
- The observations from on-site inspections; and
- The results of laboratory tests of materials and assemblies taken from the site and on specimens constructed in the laboratory.

In assessing the performance of building systems, components, and materials, it is essential to (1) review the design, (2) examine the appropriateness of the materials, and (3) consider the workmanship in executing the construction.

6.4.1.1 Design of Masonry

With the exception of the parapet walls, the design of the masonry is sufficient to resist the design loads. The anchorage of the masonry to the structural frame is more than adequate to transmit wind loads from the walls to the frame and to provide lateral support for the walls. The masonry ties between facing and building brick are consistent with good U.S. practice. The materials were manufactured to meet U.S. specifications and the ties are spaced more closely than required by U.S. codes and practice.

A structure such as the Office Building has tendencies for differential movement between the masonry walls and the structural elements, which may lead to distress and cracking [6.3]. The principal means for accommodating differential movement are to provide for horizontal and vertical expansion joints [6.4] and methods of flexible anchorage [6.5]. The Office Building has no built-in expansion joints and the anchorage to the frame is not detailed in a flexible manner.

The design details, with one general exception (window soffits), appear to be adequate for providing protection from water entrance and condensation. The flashing shown in the details is appropriately placed.

6.4.1.2 Materials and Workmanship

The facing brick and building brick are of suitable quality for the use intended. The mortar used is a portland cement mortar with high compressive strength and with good to moderate bond strength to some brick.

The workmanship exercised in building the brick masonry varies considerably. For the exterior facing, the joints appeared to be well-filled and well-tooled with a concave joiner. The backup brick and the core walls do not exhibit equivalent care in filling the joints. Head and collar joints in particular were found to be partially filled.

6.4.2 Enclosure Walls

6.4.2.1 Assessment

The enclosure walls have vertical cracks at corners and at second story piers between windows. However, the scope of this investigation did not permit a definitive determination of their cause. These vertical cracks may result from unintended vertical compressive stresses in the masonry due to differential movements between the masonry and the concrete frame. Distress near the corners is further aggravated by horizontal forces induced by moisture expansion and thermal expansion of the brick masonry. Horizontal cracking below the louver opening on the west elevation was probably caused by freezing and thawing of water which had entered into the wall.

6.4.2.2 Remedial measures

Vertical expansion joints should be cut in the recesses of the corner piers for the full height of the recesses. The joint should be placed in the corner of the recess nearest to the building corner. The cut should extend through the facing wythe of brick but not through the backup masonry; i.e., it should be about 120 mm deep. In making the cut, the anchors should not be cut and the wire ties should be left intact if possible. The width of the cut should be between 15 and 20 mm. Prior to sealing the cut, the exposed portions of the wall anchors should be coated with an appropriate material to prevent corrosion. A compressible material (back up rod) should be placed in the cut to serve as backup for a permanent elastic sealant.

In order to formulate additional remedial measures, a more complete understanding of the cause of the vertical cracking should be acquired. A monitoring program should be established to determine whether cracks are growing in number, length, and width. The monitoring program should include a quantitative documentation of the cracking at 6-month intervals for two years. Strain relief tests should be performed on the facing brick to measure the in-place vertical strains. Modelling of the interaction between the masonry and the frame should be performed to obtain reliable estimates of the induced masonry stresses.

The horizontal crack below the louver opening on the west elevation should be repaired. The facing should be removed down to the crack and below if the facing appears distressed further down. The backup brick should also be removed if it is distressed. The facing, along with any removed backup brick, should be replaced, ensuring that all joints are well-filled. The facing brick should be tied to the backup brick with prefabricated metal ties as were specified for the original construction.

6.4.3 Parapet and Penthouse Walls

6.4.3.1 Assessment

The parapet walls and the exterior of the penthouse walls are extensively cracked horizontally and vertically. Parapet walls, in general, have a history of being problem areas because of their severe exposure and lack of lateral support at the top [6.5]. Hence, the cracking observed at the top of the Office Building is not unexpected.

It appears that considerable water had entered the parapet and penthouse walls, as indicated by masonry staining where water flowed out of openings and by observations of ice upon opening some walls. The entrance of water and freezing and thawing action contributed to the cracking.

The parapet walls lack adequate strength and sufficient anchorage to the structure below.

6.4.3.2 Remedial measures for parapet walls

The cracked masonry should be removed down to the level of sound, uncracked masonry (fig. 6.4.1). In some areas this will be as much as 2 m from the top of the wall. In no case should less than 500 mm of wall be removed, which will include existing cast-in-place bond beams where they occur.

Vertical holes should be drilled through the walls into the eighth story masonry. These holes should be between 75 mm and 100 mm in diameter. They should be drilled through the center of the wall and extend 1 m below the structural roof slab of the eight story. Dowels 20 mm in diameter should be placed not less than 1 m apart and not more than 500 mm from each corner. The dowels should extend to the top of the parapet wall. The dowels are to be grouted in place and the grout should be well consolidated. The holes should be pre-wetted with water prior to grout placement.

Reconstruct the masonry at the top of the wall, building in a reinforced brick masonry bond beam as shown in figure 6.4.1. The reinforced section should be grouted with grout meeting the requirements of ASTM C 476-83 [6.6]. The tops of parapet and penthouse walls should be capped to prevent entrance of water. The cap should provide for a drip to carry water away from the face of the walls.

6.4.3.3 Remedial measures for penthouse walls

The two wythes of masonry (250 mm thickness) at the top of the walls should be removed to a depth of 640 mm (8 courses). Where the facing brick has horizontal cracking below this level, it should be removed to the level of sound, uncracked masonry.

Replace the removed brick, ensuring that the joints are well filled with mortar. Construct a reinforced brick masonry bond beam at the top of the wall as shown in figure 6.4.2. The bond beam should contain three 12-mm bars and 10-mm ties spaced every 1 m.

6.4.3.4 Materials and Workmanship

The brick used in the original construction have appropriate properties and similar brick would be acceptable for reconstruction. To achieve the best match in appearance, facing brick should be obtained from the manufacturer of the original production. The appearance of building brick is not important, hence building brick which comply with ASTM C 67-85a, grade MW [6.7], would be acceptable.

The mortar should be Type S portland cement and lime meeting the requirements of ASTM C 270-86 [6.8]. For facing brick, the mortar should be tinted to match the surrounding masonry.

It is important that all joints in the reconstructed masonry be completely filled. Slushing of joints normally does not meet this requirement. Acceptable techniques include shoving and grouting of joints to assure that all head, bed, and collar joints are filled.

Reconstruction should be done in accordance with Chapter 4 of "Recommended Practice for Engineered Brick Masonry" [6.9].

6.4.4 <u>Masonry core walls</u>

6.4.4.1 Assessment

The brick masonry walls in the core of the building were not intended to be structural. However, the progressive collapse analysis (section 4.6) indicates that they are important for providing alternate load paths in the event of a column failure. As built, the mortar joints in the walls are not well-filled. In many cases, the joints between the top of the walls and beams have not been completed. If the walls are required to serve a structural function, their performance can be substantially improved by ensuring that the surrounding joints between columns, floors, and beams are well-filled. The walls will be further strengthened when they are covered with portland cement sand plaster as now completed in some areas.

6.4.4.2 Remedial measures

The top of all core walls should be inspected to determine the condition of the top joints. In cases where the top is not completed or tightly filled with brick and mortar, it should be completed and well pointed using a type S portland cement lime mortar. In addition, walls containing openings, such as door jams or duct penetrations, should be completed and the joints between penetrating components should be well-filled.

6.4.5 Insulation in Window Soffits and Corners

Window soffits have cavities between the insulation and the underside of the masonry lintels. These cavities permit the condensation of water and the formation of frost within the space. This can be deleterious to the insulation and to the anchorage of the soffits to the underside of the masonry, and under some conditions may contribute to the deterioration of the masonry itself.

The cavities above the soffits should be filled with a suitable insulation, such as polyurethane or other materials which can be frothed into place. Fibrous insulation is also acceptable provided it is carefully placed so as to fill the cavity. If the insulation does not serve as a vapor barrier, one should be applied on the inside to prevent the condensation of water vapor and its migration into this space. The insulation should have volumetric stability; it should not be hygroscopic nor contribute to corrosion.

The four corners containing the drain pipes for the snow melting rooms are not properly insulated. Insulation has been placed on the inside (warm side) of the pipes rather than the outside as detailed in the plans. This provides no protection against freezing, which could cause rupture of the drains or blockage, resulting in water damage to the building.

Drywall and insulation should be removed from the completed corners in stories 1, 2, and 3. The corners should be cleaned of debris, insulation should be correctly placed, and the drywall replaced.

6.5 COST OF REMEDIAL MEASURES

A professional engineering consultant was retained to provide cost estimates for the recommended remedial measures [6.10]. The estimates are based on the following assumptions:

- o The cost for remedial work is based on April 1987 costs for labor and materials in the Washington, D.C., area.
- o The contractor is not mobilized at the site and will have to provide a temporary site office and the associated facilities and equipment to perform the work.

- The contractor is established in the metropolitan Washington, D.C. area and the contractor's home office will provide support services for purchasing, estimating, scheduling, etc.

- Estimates are based on quantities supplied by NBS.

The recommended remedial measures are in two categories. The first includes those which correct structural deficiencies, and these must be performed before the Office Building is occupied. The second includes measures for improving the durability of the Office Building, and these do not need to be performed prior to occupancy.

The following summarizes the estimated costs for remedial measures to correct structural deficiencies and provide adequate safety:

Remedial Measure	Cost
Provide lateral bracing to steel-core composite columns; install shear connections between masonry and concrete partitions; and strengthen box beam connection	$9,000
Expose, inspect, and fill reinforced concrete column joints with grout and/or concrete	$132,000
Expose, inspect, and fill vertical and horizontal shear wall joints with grout and concrete, respectively	$155,000
Provide steel straps between flanges of long-span beams in the second through the eighth floors	$546,000
Fill gaps between masonry core walls and the surrounding beams and columns; fill hollow portions of core walls with concrete	$263,000
Remove parapet walls down to a level below substantial cracking, provide anchors to structural system below, and rebuild walls	$385,000
TOTAL	$1,490,000

The following summarizes the estimated costs for remedial measures to improve the durability of the Office Building:

Remedial measure	Cost
Cut and seal vertical expansion joints in corners of exterior masonry wall	$22,000
Conduct a program to monitor the growth of vertical cracks in the exterior masonry walls	$200,000

Remove and rebuild cracked penthouse walls....................$94,000

Place insulation in window soffits and in corner piers..........$25,000

TOTAL $341,000

These costs are based on the assumptions listed above. The actual cost to perform the remedial measures will depend on the working conditions in Moscow and the means selected for performing the work. Nevertheless, it is seen that a relatively modest expense is required to correct structural deficiencies and bring the Office Building to a level of safety consistent with U.S. practice.

6.6 SUMMARY

Deficiencies identified in review of the design and site investigations have been assessed and remedial measures defined as needed to achieve required performance. Cost estimates to accomplish the remedial measures have been provided on the basis of costs for similar work done in Washington, D.C.

Two concrete beams were found to be underdesigned. Neither is expected to be subjected to large live loads from occupancy and neither shows signs of distress. The loads come mostly from masonry partitions that arch over the midspan regions of the beams; this reduces the bending moments below the values calculated on the basis of uniformly distributed loads. Therefore, no remedial measures are recommended.

The design review showed a number of reinforced concrete columns to be overloaded. However, core tests showed higher-than-specified concrete strengths giving adequate levels of resistance. Four steel-core columns are inadequately braced against buckling. They can be firmly attached to adjacent masonry walls to provide the required bracing. The estimated cost for this remedial measure is $1,000.

Site investigation showed that the integrity of all joints of reinforced concrete columns is questionable. Laboratory studies showed that failure to place grout and concrete in the joint severely reduces the column strength. Each joint should be exposed and drilled to determine whether the joint is filled with grout and concrete, and each deficient joint should be filled. The estimated cost for this remedial measure is $132,000.

Site investigation showed that few observed shear wall joints were grouted and concreted in accord with the plans and with the requirements for appropriate strength and stiffness. Each vertical and horizontal joint of each shear wall panel should be exposed, inspected for complete grouting and concreting of the joint, and filled if needed. The estimated cost for this remedial measure is $155,000.

Floor planks were determined to have adequate strength as designed. All steel beams and connections were designed to have adequate resistance and no deficiencies in materials or fabrication were identified at the site. However, were a long-span beam on floors two through eight to fail, a progressive collapse might occur. This can be avoided by fastening steel straps perpendicular to the top flanges of long-span beams so that the beams would remain suspended if one were to fail. The estimated cost for this remedial measure is $546,000.

Consideration of the potential for progressive collapse showed that alternate load paths are not presently available in the event of the failure of some columns in the core area of the building. Gaps between masonry walls and adjacent beams and columns should be filled and the walls strengthened to provide the needed alternate load paths. The estimated cost is $263,000. In addition, shear connections should be installed between the brick masonry and concrete partition walls along column lines F1 and G1, and the connection of the second floor box beam at column C1/6B should be strengthened. The estimated cost is $8,000.

Parapet walls should be taken down to a level below substantial horizontal cracking, anchored to the structural system below, and restored with properly designed and constructed masonry. The estimated cost for this remedial action is $385,000.

The following remedial measures should be made to improve the serviceability and durability of the Office Building.

Site investigation showed substantial cracking in the exterior masonry walls. Vertical expansion joints should be provided in the corner piers to allow for differential movement and sealed to prevent entrance of moisture. The estimated cost for this remedial measure is $22,000.

The site investigation showed vertical cracks in the masonry walls. The scope of this investigation did not permit a definitive determination of their cause. They may result from excessive vertical compressive stress leading to vertical splitting. A program should be conducted for long-term monitoring of these cracks to determine whether they are growing in extent or number. From these observations and further analyses, appropriate remedial measures can be formulated, if required. For a two-year program, the estimated cost is $200,000.

Cracked penthouse walls should be taken down to a level below substantial cracking and properly rebuilt. The estimated cost for this remedial action is $94,000.

Insulation should be placed appropriately in corner piers and in cavities above windows. The estimated cost for this remedial action is $25,000.

6.7 REFERENCES

6.1 Wood, R.H., "Studies in Composite Construction: Part I, The Composite Action of Brick Panel Walls Supported on Reinforced Concrete Beams, National Building Studies," Research Paper #13, Dept. of Scientific and Industrial Research, London, 1953, 25 pp.

6.2 Building Code Requirements for Reinforced Concrete, ACI 318-83, American Concrete Institute, Detroit, MI, 1983.

6.3 Differential Movement, Causes and Effect, Part I of III, Technical Notes on Brick Construction, No. 18, Brick Institute of America.

6.4 Differential Movement, Causes and Effect, Part II of III, Technical Notes on Brick Construction, No. 18A, Brick Institute of America.

6.5 Differential Movement, Causes and Effect, Part III of III, Technical Notes on Brick Construction, No. 18B, Brick Institute of America.

6.6 Standard Specification for Grout for Reinforced and Nonreinforced Masonry. ASTM C 476-83, Vol. 04.05, American Society for Testing and Materials, Philadelphia.

6.7 Standard Specification for Facing Brick (Solid Masonry Units Made From Clay or Shale. ASTM C 216-85a, Vol. 04.05, American Society for Testing and Materials, Philadelphia.

6.8 Standard Specification for Mortar for Unit Masonry. ASTM C 270-86, Vol. 04.05, American Society for Testing and Materials, Philadelphia.

6.9 Gross, James G., Dikkers, Robert D., and Grogan, John G., "Recommended Practice for Engineered Brick Masonry," Structural Clay Products Institute. 1969

6.10 Cost Estimate for Correction of Structural and Construction Defects at U.S. Embassy, Moscow, U.S.S.R. Turner International. April 1987.

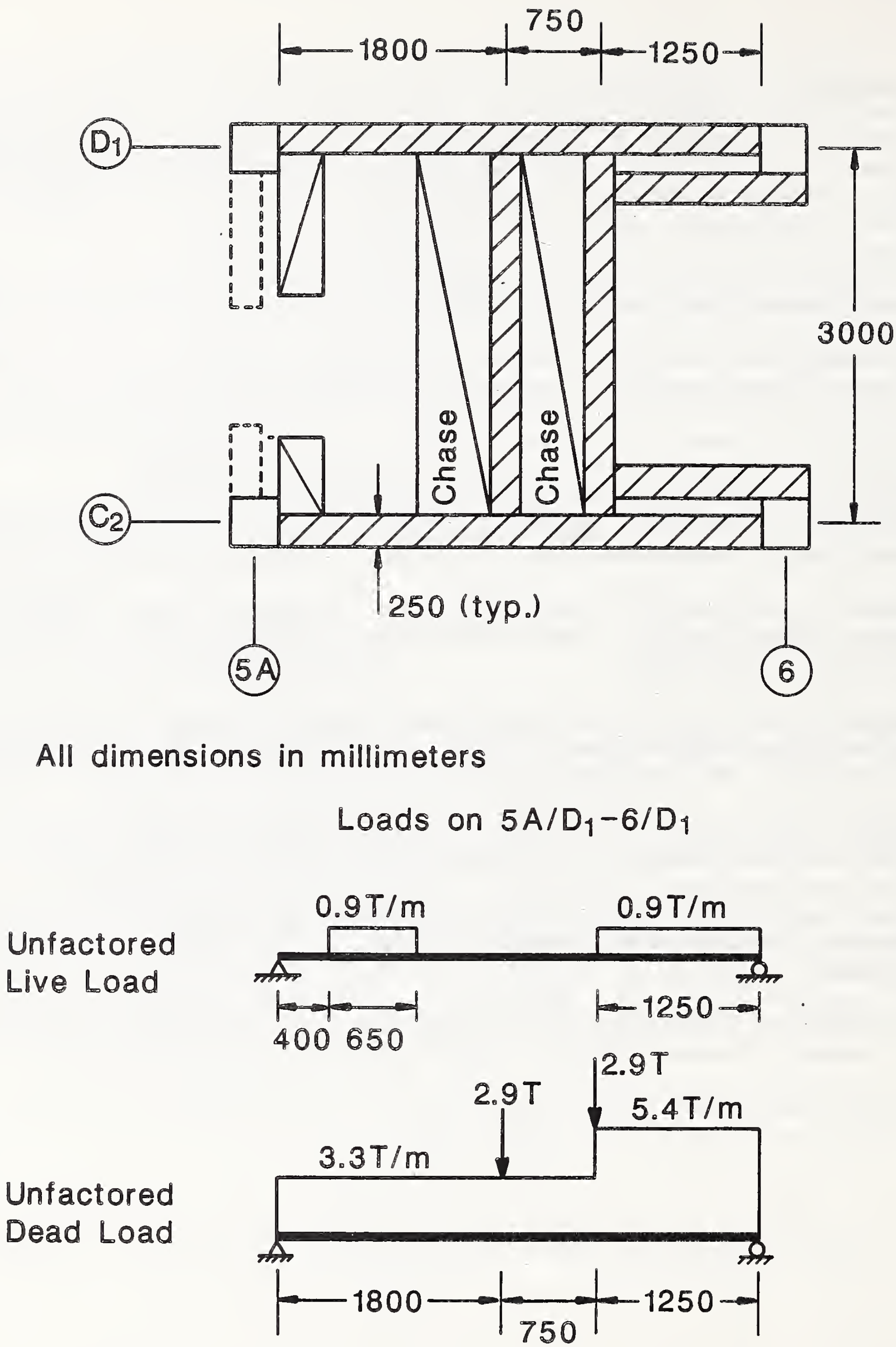


Figure 6.2.1. Plan of masonry partitions supported by R-38-8 beams (top) and resulting beam loading (bottom)

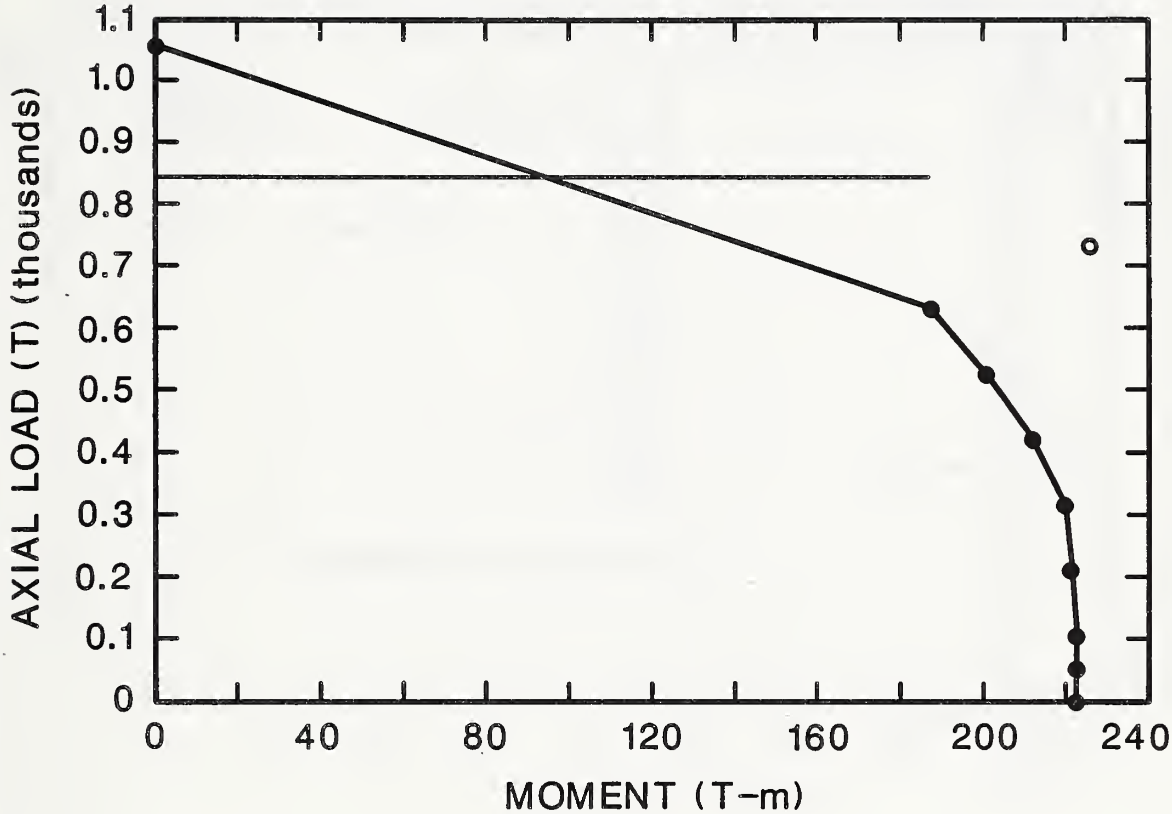


Figure 6.2.2. Design strength axial load-moment diagram for Type SC-9 column with 500 kg/cm^2 concrete strength

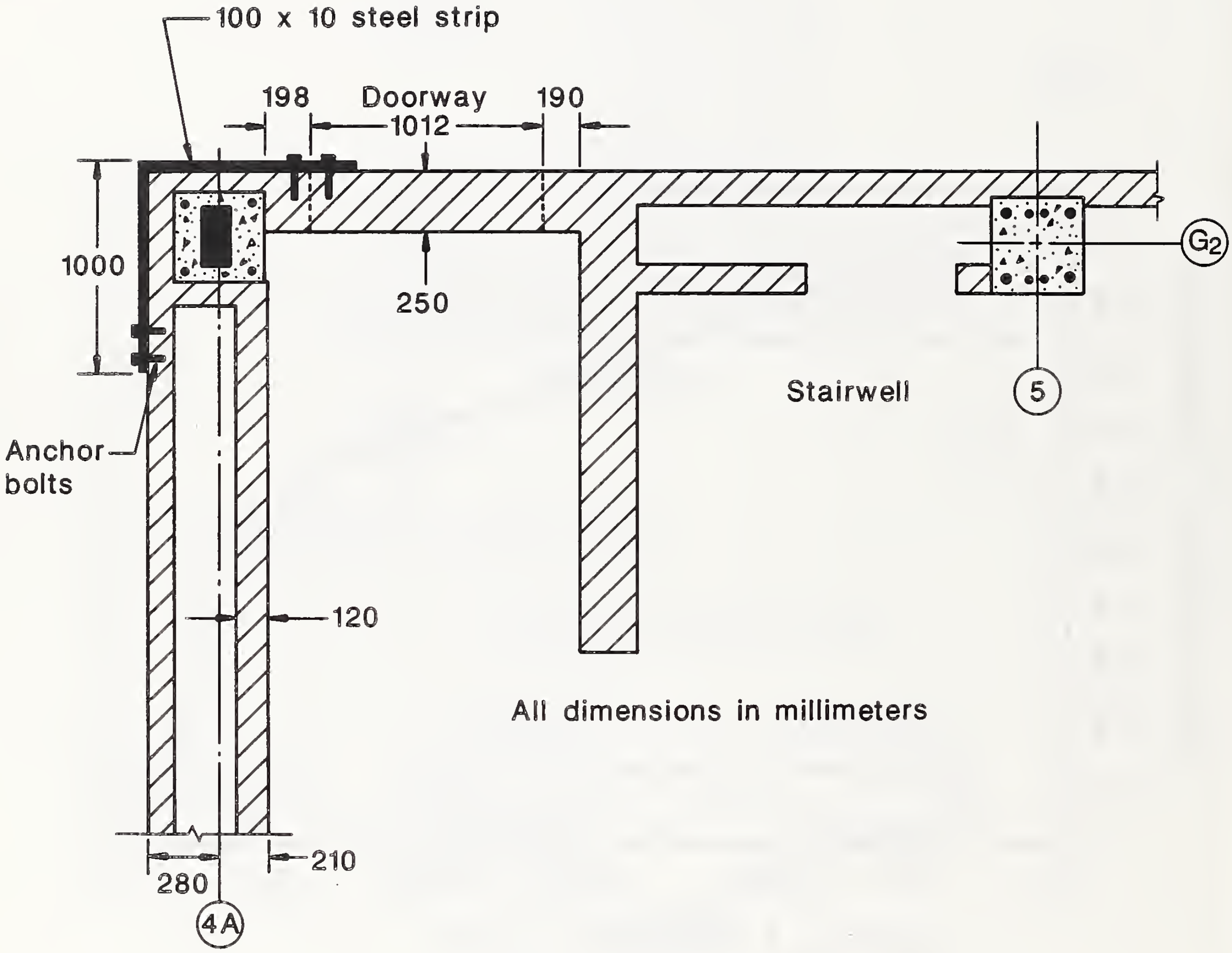


Figure 6.2.3. Masonry wall details around column 2-G2/4A and recommended method to reduce unbraced length

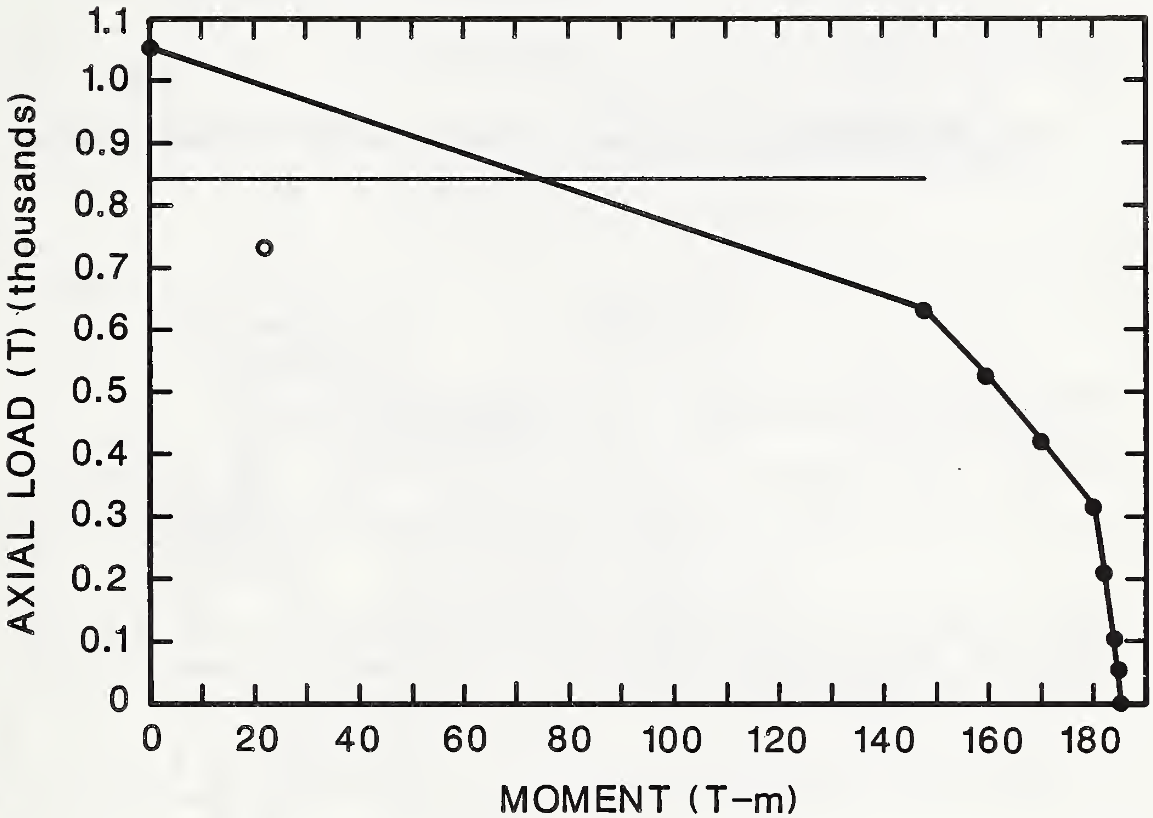


Figure 6.2.4. Design axial load-moment interaction diagram for Type SC-9 column with 330 kg/cm^2 concrete strength

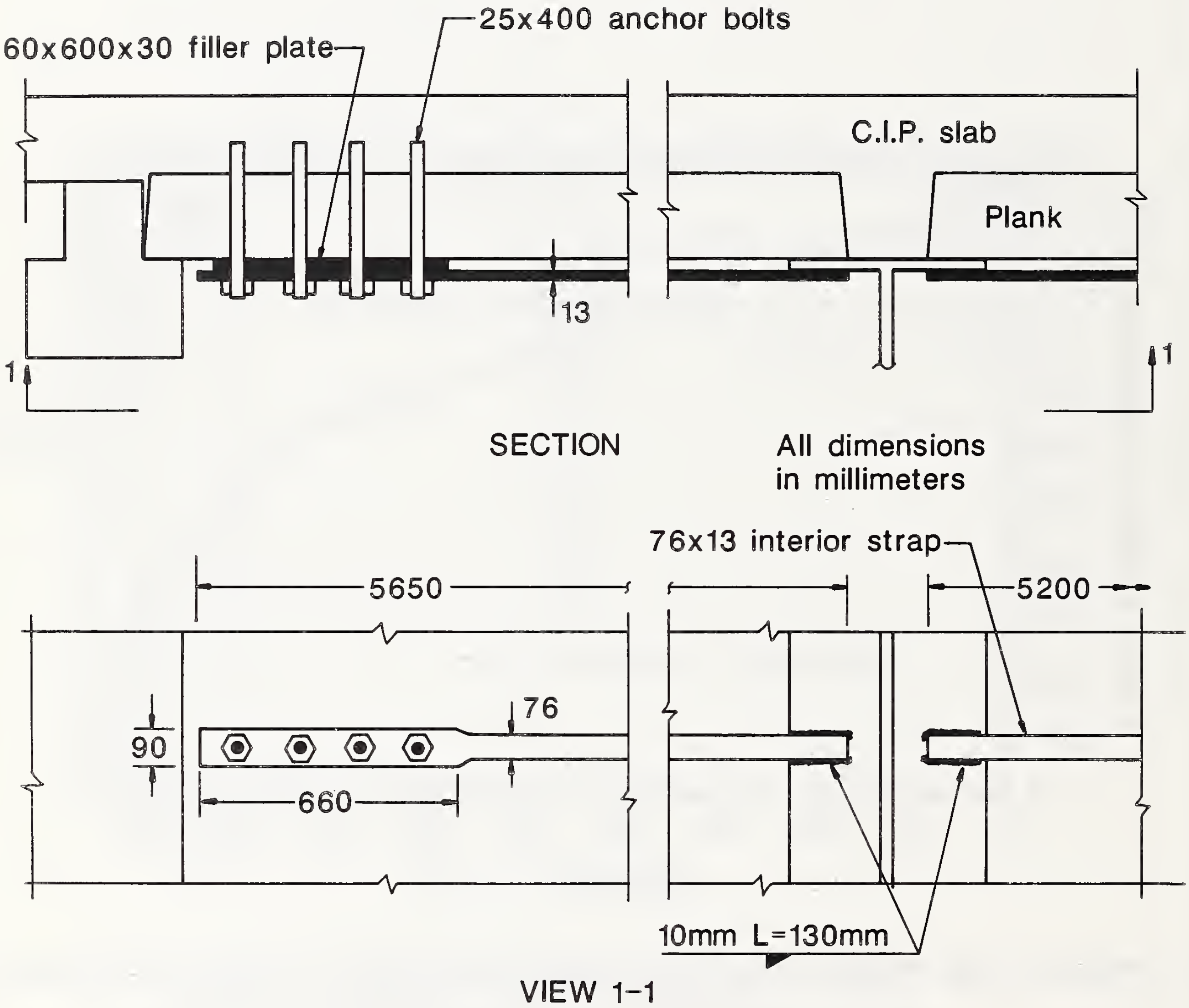


Figure 6.3.1. Steel strap system to support eighth floor

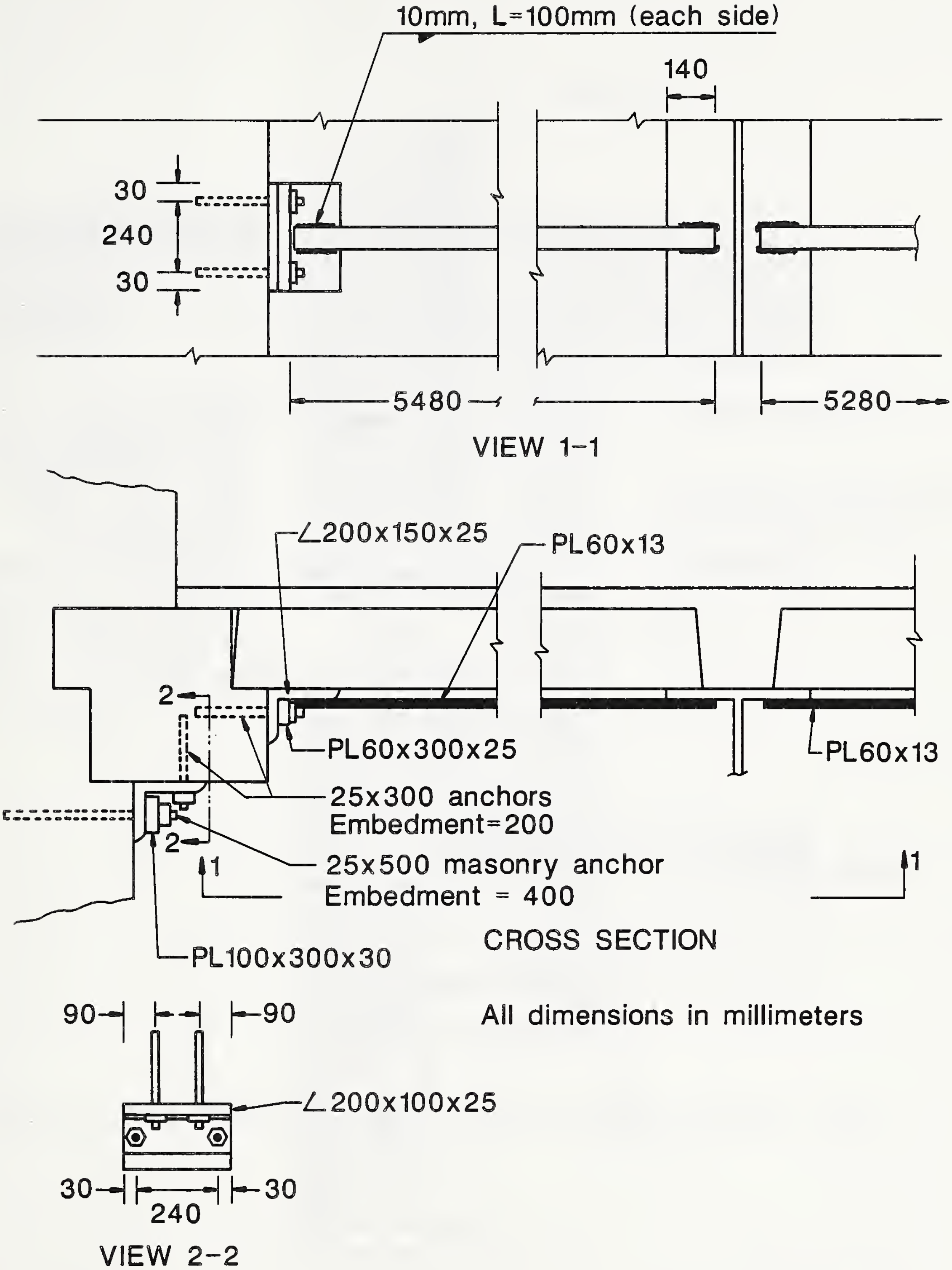


Figure 6.3.2. Steel strap system to support second through seventh floors

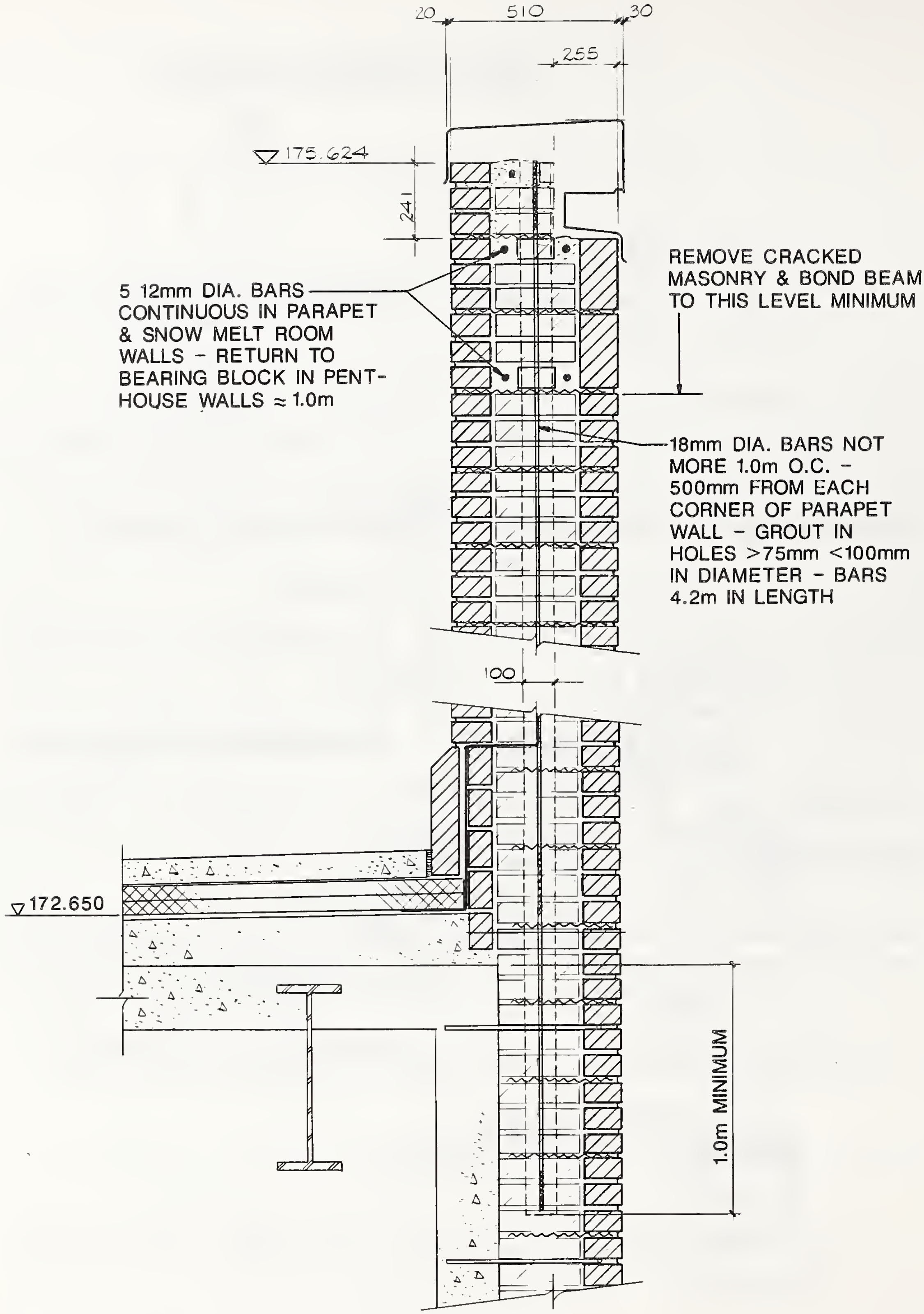


Figure 6.4.1. Detail of remedial measure for parapet walls

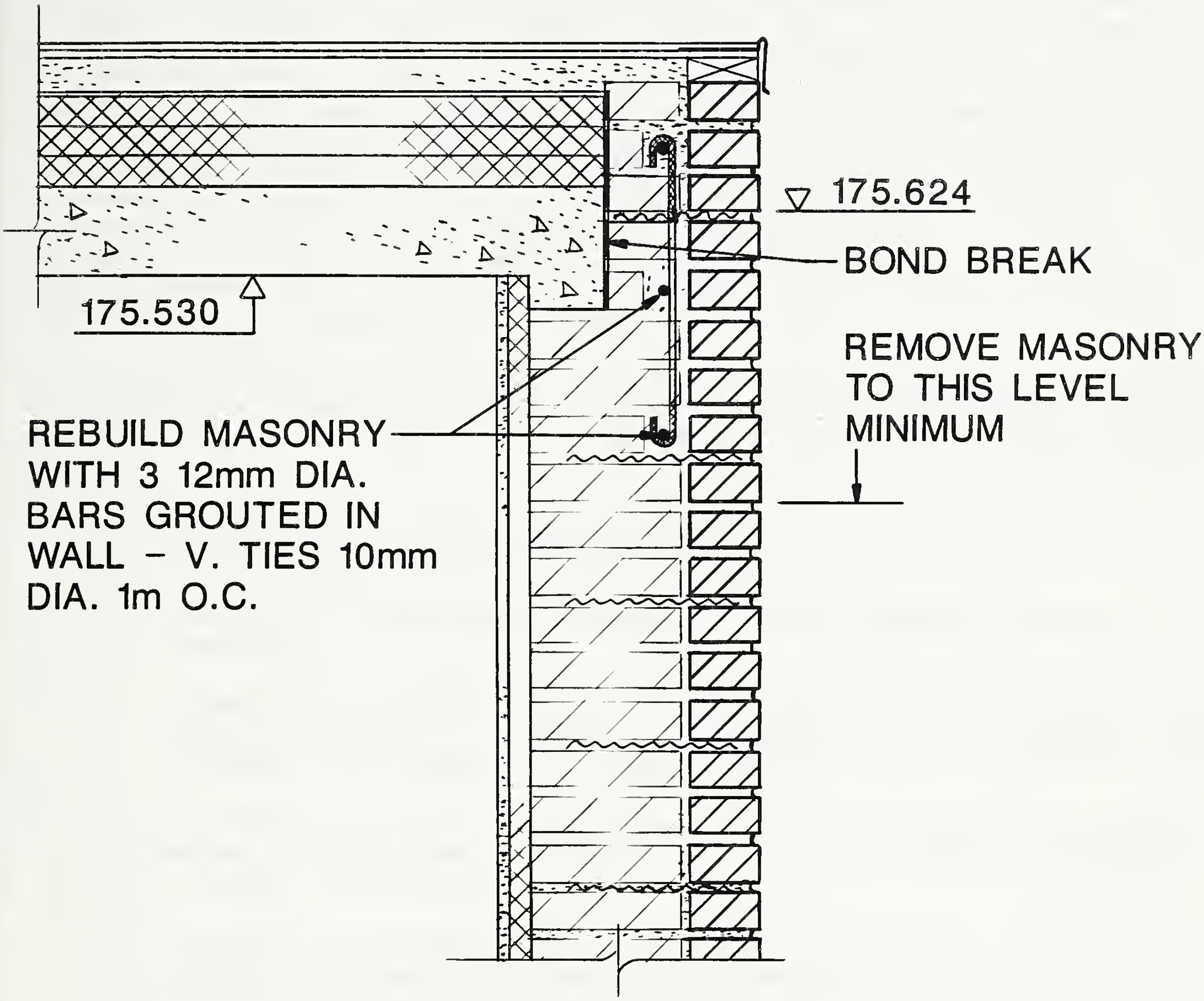


Figure 6.4.2. Detail of remedial measure for penthouse walls

CHAPTER 7
CONCLUSIONS

The National Bureau of Standards has analyzed the structural system and exterior brick masonry of the U.S. Embassy Office Building in Moscow and has developed recommendations and cost estimates for correcting structural deficiencies. The scope of the investigation was limited to the structural system and building envelope, and was not concerned with defects that do not threaten the structural integrity of the building, nor with security aspects of the Office Building. Activities included review of the documentation for the design and construction of the building, formulation of criteria for the assessment to provide a level of safety consistent with good U.S. practice for important office buildings, analysis of the structure as designed for compliance with the criteria, field and laboratory investigations of the as-built characteristics of the structure, analysis of the as-built structure, and development of required remedial measures.

Structural materials and components used in the Office Building are generally of good quality. However, important deficiencies exist in the structure that must be corrected for adequate safety before the building is occupied. These include:

- Inspecting all of the joints between reinforced concrete columns and filling those found to be incomplete.

- Bracing four steel-core columns to provide adequate resistance to buckling.

- Inspecting and completing all joints between shear wall panels and adjacent panels or columns to provide adequate strength and stiffness for resistance to lateral forces.

- Attaching a system of steel straps to the top flanges of long-span beams on floors two through eight to protect against progressive collapse of the floor system.

- Filling gaps between masonry partitions in the core area and the surrounding beams and columns, and strengthening the partitions to provide an alternate load path in the event of a column failure.

- Installing shear connections between brick masonry and concrete partition walls, and strengthening a box beam connection.

- Removing and replacing cracked portions of parapet walls, and anchoring the parapet walls adequately to the structure below.

The total estimated cost based on Washington, D.C., prices for conducting these remedial structural measures is $1,490,000.

In addition, the following remedial measures are recommended for the serviceability and durability of the Office Building structure:

- Removing and replacing cracked portions of the penthouse walls.
- Providing vertical expansion joints in the corner piers of the exterior walls.
- Appropriately placing insulation in the corner piers and in cavities above windows.
- Carrying out a program to monitor the development of cracks present in the exterior walls, and to define remedial measures if needed.

The total estimated cost based on Washington, D.C., prices for conducting these additional remedial structural measures is $341,000.

Actual costs of the remedial structural measures will depend upon working conditions in Moscow and the means selected for performing the work. These costs do not include the costs of correcting nonstructural deficiencies in the Office Building.

The remedial structural measures do not involve major reconstruction and could be completed in less than a year if the Office Building were located in the United States.

APPENDIX 4.1
COLUMN LOADS

This Appendix presents the results of the manual analysis of the vertical loads. The tables give the column loads in tons, the associated tributary areas and the live load reduction factors as specified in ANSI A58.1-1982. The following tables are presented:

- o Table AD: Accumulated unfactored dead load;
- o Table AL: Accumulated unfactored live load;
- o Table AU: Accumulated factored loads (1.4 dead + 1.7 live);
- o Table AT: Accumulated column tributary areas for floors 1 through 7 (live load = 380 kg/cm^2); and
- o Table AR: Live load reduction factors.

Columns are designated by their orthogonal building axes (a letter axis and a digit axis). Loads are accumulated from the roof down to the first floor. The loads given for each floor level are the loads on the columns supporting that floor. For example, the row labelled "2nd Floor" gives the loads in the columns supporting the second floor, that is, they are the loads on the first-story columns.

TABLE AD-1. Column Accumulated Unfactored DL Rev. 03.08.87

LETTER AXES :	AXIS A1								AXIS C		AXIS C1			
DIGIT AXES :	1A	3	4	5	6	7	8	9A	1A	9A	4A	5	6	6B
ROOF												13.03	13.03	
PENTHOUSE	20.90	37.63	34.60	20.28	20.28	34.60	37.63	20.90	77.79	77.79	83.82	63.68	59.17	83.82
8th FLOOR	39.36	63.51	91.61	82.74	85.46	97.04	63.51	39.36	132.73	132.73	146.16	107.36	100.76	157.89
7th FLOOR	52.51	82.42	117.23	109.71	111.47	120.73	82.42	52.51	183.77	169.39	208.35	145.03	139.90	209.27
6th FLOOR	65.67	101.33	140.93	135.72	137.48	144.42	101.33	65.67	220.43	206.05	256.17	182.71	179.04	260.66
5th FLOOR	78.83	120.25	164.62	161.73	163.49	168.12	120.25	78.83	257.09	242.71	303.99	220.38	218.19	312.04
4th FLOOR	91.99	139.16	188.31	187.74	189.50	191.81	139.16	91.99	293.75	279.38	351.81	258.05	257.33	363.43
3rd FLOOR	105.14	158.07	212.01	213.75	215.51	215.50	158.07	105.14	330.42	316.04	399.63	295.73	296.47	414.81
2nd FLOOR	118.79	177.50	238.14	240.21	241.97	239.68	177.50	118.79	375.16	353.05	459.02	336.03	333.35	469.67
1st FLOOR	136.34	196.93	265.95	270.43	273.07	277.58	204.58	138.20	439.83	425.63	527.34	385.31	379.25	547.20

TABLE AL-1. Column Accumulated Unfactored LL

LETTER AXES :	AXIS A1								AXIS C		AXIS C1			
DIGIT AXES :	1A	3	4	5	6	7	8	9A	1A	9A	4A	5	6	6B
ROOF												5.42	5.42	
PENTHOUSE	3.90	7.76	6.62	11.39	11.39	6.62	7.76	3.90	19.25	19.25	24.66	23.97	22.80	24.66
8th FLOOR	10.47	20.24	30.65	37.05	37.05	30.65	20.24	10.47	46.71	46.71	51.32	37.06	36.80	53.01
7th FLOOR	13.89	25.46	37.50	43.76	43.52	37.00	25.46	13.89	58.97	55.63	64.09	42.50	42.71	62.55
6th FLOOR	15.87	28.57	41.24	47.68	47.53	40.92	28.57	15.87	63.30	61.42	68.11	45.77	46.31	68.82
5th FLOOR	17.55	31.34	44.70	51.25	51.12	44.43	31.34	17.55	68.30	66.70	72.96	48.68	49.52	74.54
4th FLOOR	19.10	33.93	47.98	54.61	54.50	47.74	33.93	19.10	73.15	71.71	77.70	51.41	52.54	79.98
3rd FLOOR	20.57	36.40	51.13	57.85	57.75	50.91	36.40	20.57	77.88	76.55	82.33	54.02	55.44	85.25
2nd FLOOR	21.98	38.79	54.30	61.00	60.90	53.99	38.79	21.98	82.98	81.27	87.75	56.58	58.24	90.40
1st FLOOR	23.35	41.12	57.32	64.07	63.95	56.94	41.00	23.22	88.36	86.71	93.46	59.06	60.67	96.58

TABLE AU-1. Column Accumulated Ultimate Loads. 1.4DL+1.7LL

LETTER AXES :	AXIS A1								AXIS C		AXIS C1			
DIGIT AXES :	1A	3	4	5	6	7	8	9A	1A	9A	4A	5	6	6B
ROOF												27.46	27.46	
PENTHOUSE	35.89	65.88	59.70	47.75	47.75	59.70	65.88	35.89	141.64	141.64	159.27	129.90	121.60	159.27
8th FLOOR	72.90	123.32	180.35	178.83	182.63	187.95	123.32	72.90	265.22	265.22	291.87	213.30	203.62	311.16
7th FLOOR	97.13	158.67	227.87	227.99	230.05	231.93	158.67	97.13	357.52	331.72	400.64	275.30	268.47	399.32
6th FLOOR	118.91	190.44	267.41	271.06	273.27	271.75	190.44	118.91	416.21	392.89	474.42	333.60	329.39	481.91
5th FLOOR	140.19	221.62	306.46	313.54	315.79	310.89	221.62	140.19	476.03	453.19	549.61	391.29	389.65	563.58
4th FLOOR	161.25	252.50	345.20	355.67	357.95	349.68	252.50	161.25	535.60	513.04	624.61	448.67	449.59	644.77
3rd FLOOR	182.17	283.18	383.73	397.59	399.88	388.25	283.18	182.17	594.98	572.59	699.44	505.84	509.30	725.67
2nd FLOOR	203.67	314.45	425.71	439.99	442.29	427.34	314.45	203.67	666.30	632.43	791.80	566.62	565.70	811.22
1st FLOOR	230.57	345.61	469.77	487.52	491.01	485.41	356.11	232.95	765.98	743.30	897.15	639.84	634.09	930.28

TABLE AD-1. Column Accumulated Unfactored DL (cont.) Rev. 03.08.87

LETTER AXES :	AXIS C2		AXIS D				AXIS D1			
DIGIT AXES :	5A	6	1A	4A	6B	9A	4A	5A	6	6B
ROOF	11.19	9.88						10.74	9.38	
PENTHOUSE	32.84	26.19	40.72	41.04	41.04	40.72	65.18	37.26	35.56	57.67
8th FLOOR	59.18	40.88	133.95	119.10	94.92	116.15	87.15	63.03	60.00	82.41
7th FLOOR	85.53	55.57	184.63	171.16	134.32	151.12	109.11	88.81	84.44	107.15
6th FLOOR	111.87	70.26	219.60	215.38	173.72	186.09	131.08	114.58	108.88	131.89
5th FLOOR	138.21	84.96	254.57	259.60	213.12	221.06	153.04	140.35	133.31	156.63
4th FLOOR	164.55	99.65	289.54	303.82	252.52	256.03	175.01	166.13	157.75	181.37
3rd FLOOR	190.89	114.34	324.51	348.05	291.92	291.00	196.97	191.90	182.19	206.11
2nd FLOOR	220.72	130.64	348.37	367.53	338.20	326.38	221.76	220.72	209.52	233.98
1st FLOOR	249.82	158.85	398.54	419.99	419.60	389.35	250.04	254.02	232.61	258.17

TABLE AL-1. Column Accumulated Unfactored LL (cont.)

LETTER AXES :	AXIS C2		AXIS D				AXIS D1			
DIGIT AXES :	5A	6	1A	4A	6B	9A	4A	5A	6	6B
ROOF	4.62	3.92						4.22	3.51	
PENTHOUSE	11.11	8.32	10.82	10.82	10.82	10.82	18.68	11.30	11.13	17.43
8th FLOOR	12.89	10.55	45.67	38.76	36.82	44.21	20.50	14.32	13.98	19.59
7th FLOOR	14.67	12.78	67.80	64.92	45.38	52.77	22.32	17.34	16.83	21.75
6th FLOOR	16.45	14.63	68.77	65.11	50.91	58.30	24.10	19.29	18.77	23.59
5th FLOOR	17.48	15.85	72.27	68.12	55.94	63.33	25.15	20.83	20.24	24.79
4th FLOOR	18.43	16.97	76.24	71.94	60.71	68.10	26.11	22.24	21.59	25.88
3rd FLOOR	19.32	18.02	80.34	76.01	65.31	72.70	27.02	23.57	22.86	26.91
2nd FLOOR	20.17	19.02	82.80	76.71	69.79	77.18	27.88	24.85	24.08	27.89
1st FLOOR	20.92	20.44	86.93	81.22	78.98	86.44	29.07	26.15	25.11	28.86

TABLE AU-1. Column Accumulated Ultimate Loads. 1.4DL+1.7LL (cont.)

LETTER AXES :	AXIS C2		AXIS D				AXIS D1			
DIGIT AXES :	5A	6	1A	4A	6B	9A	4A	5A	6	6B
ROOF	23.52	20.50						22.20	19.10	
PENTHOUSE	64.87	50.81	75.41	75.85	75.85	75.41	123.01	71.37	68.71	110.37
8th FLOOR	104.77	75.17	265.17	232.63	195.49	237.77	156.86	112.59	107.77	148.68
7th FLOOR	144.68	99.53	373.75	349.99	265.20	301.28	190.70	153.81	146.82	186.98
6th FLOOR	184.58	123.24	424.35	412.22	329.76	359.63	224.47	193.21	184.33	224.75
5th FLOOR	223.22	145.88	479.26	479.24	393.46	417.14	257.02	231.90	221.04	261.42
4th FLOOR	261.71	168.35	534.96	547.66	456.73	474.20	289.41	270.38	257.55	297.91
3rd FLOOR	300.10	190.71	590.88	616.48	519.72	530.99	321.69	308.73	293.93	334.29
2nd FLOOR	343.30	215.23	628.47	644.95	592.12	588.13	357.85	351.25	334.26	374.99
1st FLOOR	385.32	257.13	705.73	726.06	721.70	692.04	399.47	400.08	368.33	410.49

TABLE AD-1. Column Accumulated Unfactored DL (cont.)

LETTER AXES :	AXIS E						AXIS F					
DIGIT AXES :	1A	4A	4B	6A	6B	9A	1A	4A	4B	6A	6B	9A
ROOF			12.85	17.63					12.85	17.63		
PENTHOUSE	54.51	81.87	51.10	49.80	81.87	54.51	54.51	80.03	49.38	51.29	81.87	54.51
8th FLOOR	133.75	140.36	82.31	81.01	140.36	133.75	154.15	157.54	87.51	83.63	140.36	133.75
7th FLOOR	174.14	185.33	113.52	112.22	184.39	168.72	194.54	202.51	125.64	115.97	184.39	168.72
6th FLOOR	209.11	229.37	144.73	143.43	228.42	203.69	229.51	246.54	163.77	148.31	228.42	203.69
5th FLOOR	244.08	273.40	175.94	174.64	272.45	238.66	264.48	290.58	201.91	180.65	272.45	238.66
4th FLOOR	279.05	317.43	207.14	205.85	316.49	273.63	299.45	334.61	240.04	212.99	316.49	273.63
3rd FLOOR	314.02	361.46	238.35	237.06	360.52	308.60	334.42	378.64	278.17	245.33	360.52	308.60
2nd FLOOR	389.48	433.71	272.69	271.39	414.18	343.98	374.80	432.86	320.06	280.89	414.18	343.98
1st FLOOR	446.69	493.79	309.35	306.69	468.36	382.40	430.37	496.13	356.66	316.98	468.36	382.40

TABLE AL-1. Column Accumulated Unfactored LL (cont.)

LETTER AXES :	AXIS E						AXIS F					
DIGIT AXES :	1A	4A	4B	6A	6B	9A	1A	4A	4B	6A	6B	9A
ROOF			4.93	7.39					4.93	7.39		0.00
PENTHOUSE	21.14	24.77	21.68	22.67	24.77	21.14	21.14	25.76	25.42	22.23	24.77	21.14
8th FLOOR	55.72	51.70	27.25	28.24	51.70	55.72	57.18	54.15	32.41	28.08	51.70	55.72
7th FLOOR	79.76	80.48	31.96	32.95	60.67	64.28	81.22	82.93	37.88	32.95	60.67	64.28
6th FLOOR	86.27	88.73	34.73	35.72	66.51	69.81	87.73	91.18	41.17	35.82	66.51	69.81
5th FLOOR	91.62	94.49	37.17	38.16	71.83	74.84	93.08	96.94	44.10	38.37	71.83	74.84
4th FLOOR	96.56	99.78	39.45	40.44	76.88	79.61	98.02	102.23	46.85	40.74	76.88	79.61
3rd FLOOR	101.28	104.81	41.63	42.62	81.76	84.21	102.74	107.26	49.48	43.01	81.76	84.21
2nd FLOOR	106.53	110.94	43.73	44.72	86.67	88.69	107.29	112.29	52.02	45.20	86.67	88.69
1st FLOOR	110.96	115.80	45.97	46.96	104.62	106.15	111.55	117.37	54.46	47.51	104.62	106.15

TABLE AU-1. Column Accumulated Ultimate Loads. 1.4DL+1.7LL (cont.)

LETTER AXES :	AXIS E						AXIS F					
DIGIT AXES :	1A	4A	4B	6A	6B	9A	1A	4A	4B	6A	6B	9A
ROOF			26.37	37.24					26.37	37.24		
PENTHOUSE	112.25	156.72	108.39	108.26	156.72	112.25	112.25	155.84	112.35	109.59	156.72	112.25
8th FLOOR	281.97	284.39	161.56	161.42	284.39	281.97	313.01	312.60	177.61	164.81	284.39	281.97
7th FLOOR	379.38	396.28	213.26	213.12	361.29	345.48	410.43	424.50	240.30	218.37	361.29	345.48
6th FLOOR	439.41	471.95	261.65	261.52	432.86	403.84	470.45	500.17	299.28	268.53	432.86	403.84
5th FLOOR	497.45	543.40	309.50	309.37	503.54	461.34	528.49	571.61	357.64	318.14	503.54	461.34
4th FLOOR	554.81	614.03	357.07	356.94	573.77	518.41	585.86	642.25	415.70	367.45	573.77	518.41
3rd FLOOR	611.79	684.23	404.47	404.33	643.71	575.20	642.83	712.45	473.54	416.59	643.71	575.20
2nd FLOOR	726.37	795.78	456.10	455.97	727.20	632.34	707.12	796.90	536.51	470.09	727.20	632.34
1st FLOOR	813.99	888.18	511.23	509.20	833.56	715.81	792.15	894.11	591.90	524.54	833.56	715.81

TABLE AD-1. Column Accumulated Unfactored DL (cont.) Rev. 03.08.87

LETTER AXES	AXIS F1				AXIS G				AXIS G1	
DIGIT AXES	4A	5	5B	6B	1A	4A	6B	9A	5	5B
ROOF		10.35	10.74						9.88	11.19
PENTHOUSE	53.22	35.81	37.67	65.18	40.72	41.04	41.04	40.72	26.19	32.84
8th FLOOR	77.96	67.20	66.15	89.87	136.55	113.94	94.92	116.15	40.88	50.97
7th FLOOR	102.70	98.60	94.64	114.56	187.23	165.99	134.32	151.12	55.57	69.09
6th FLOOR	127.44	129.99	123.12	139.26	222.20	205.39	173.72	186.09	70.26	87.22
5th FLOOR	152.18	161.39	151.60	163.95	257.17	244.79	213.12	221.06	84.96	105.34
4th FLOOR	176.92	192.78	180.08	188.64	292.14	284.19	252.52	256.03	99.65	123.47
3rd FLOOR	201.66	224.18	208.56	213.33	327.11	323.59	291.92	291.00	114.34	141.59
2nd FLOOR	229.53	259.26	240.17	240.75	362.49	369.87	338.20	326.38	130.64	162.14
1st FLOOR	260.80	296.20	274.76	274.24	408.76	428.94	394.86	384.11	147.58	185.20

TABLE AL-1. Column Accumulated Unfactored LL (cont.)

LETTER AXES	AXIS F1				AXIS G				AXIS G1	
DIGIT AXES	4A	5	5B	6B	1A	4A	6B	9A	5	5B
ROOF		4.13	4.22						3.92	4.62
PENTHOUSE	17.36	13.78	11.49	18.68	10.82	10.82	10.82	10.82	8.32	11.11
8th FLOOR	19.52	17.49	15.62	22.25	45.67	38.28	36.82	44.21	10.55	12.31
7th FLOOR	21.68	21.13	19.51	25.80	67.80	64.44	45.38	52.77	12.78	13.51
6th FLOOR	23.52	23.17	21.72	27.80	68.77	64.56	50.91	58.30	14.63	14.71
5th FLOOR	24.72	24.96	23.66	29.54	72.27	67.39	55.94	63.33	15.85	15.88
4th FLOOR	25.81	26.61	25.46	31.14	76.24	71.05	60.71	68.10	16.97	16.60
3rd FLOOR	26.84	28.18	27.17	32.66	80.34	74.96	65.31	72.70	18.02	17.26
2nd FLOOR	27.82	29.68	28.81	34.12	84.45	78.95	69.79	77.18	19.02	17.89
1st FLOOR	28.89	31.33	30.56	35.74	88.57	82.98	83.90	95.91	19.77	18.49

TABLE AU-1. Column Accumulated Ultimate Load. 1.4DL+1.7LL (cont.)

LETTER AXES	AXIS F1				AXIS G				AXIS G1	
DIGIT AXES	4A	5	5B	6B	1A	4A	6B	9A	5	5B
ROOF		21.51	22.20						20.50	23.52
PENTHOUSE	104.02	73.55	72.27	123.01	75.41	75.85	75.85	75.41	50.81	64.87
8th FLOOR	142.33	123.81	119.17	163.65	268.81	224.59	195.49	237.77	75.17	92.28
7th FLOOR	180.63	173.95	165.66	204.26	377.39	341.94	265.20	301.28	99.53	119.70
6th FLOOR	218.40	221.38	209.29	242.22	427.99	397.31	329.76	359.63	123.24	147.11
5th FLOOR	255.07	268.37	252.47	279.74	482.90	457.27	393.46	417.14	145.88	174.48
4th FLOOR	291.56	315.13	295.40	317.04	538.61	518.66	456.73	474.21	168.35	201.07
3rd FLOOR	327.94	361.75	338.17	354.19	594.53	580.46	519.72	530.99	190.71	227.57
2nd FLOOR	368.64	413.42	385.22	395.05	651.05	652.04	592.12	588.13	215.23	257.40
1st FLOOR	414.23	467.94	436.62	444.70	722.84	741.59	695.44	700.81	240.22	290.71

TABLE AD-1. Column Accumulated Unfactored DL (cont.) Rev. 03.08.87

LETTER AXES :	AXIS G2				AXIS H		AXIS J1							
DIGIT AXES :	4A	5	6	6B	1A	9A	1A	3	4	5	6	7	8	9A
ROOF		13.03	13.03											
PENTHOUSE	83.82	59.17	63.68	83.82	77.79	77.79	20.90	37.63	24.60	20.28	20.28	24.60	37.63	20.90
8th FLOOR	149.74	100.76	111.32	152.58	132.73	132.73	39.36	63.51	81.61	82.74	85.46	87.04	63.51	39.36
7th FLOOR	211.93	139.90	152.97	198.66	183.77	169.39	52.51	82.42	107.23	109.71	111.47	110.73	82.42	52.51
6th FLOOR	263.31	179.04	194.61	244.74	220.43	206.05	65.67	101.33	130.93	135.72	137.48	134.42	101.33	65.67
5th FLOOR	314.70	218.19	236.25	290.82	257.09	242.71	78.83	120.25	154.62	161.73	163.49	158.12	120.25	78.83
4th FLOOR	366.08	257.33	277.90	336.90	293.75	279.38	91.99	139.16	178.31	187.74	189.50	181.81	139.16	91.99
3rd FLOOR	417.47	296.47	319.54	382.98	330.42	316.04	105.14	158.07	202.01	213.75	215.51	205.50	158.07	105.14
2nd FLOOR	472.33	333.90	364.38	435.76	367.42	353.05	118.79	177.50	226.19	240.21	241.97	229.68	177.50	118.79
1st FLOOR	547.76	384.28	409.65	513.98	416.61	414.27	136.34	200.94	266.76	279.20	270.55	259.63	196.93	131.46

TABLE AL-1. Column Accumulated Unfactored LL (cont.)

LETTER AXES :	AXIS G2				AXIS H		AXIS J1							
DIGIT AXES :	4A	5	6	6B	1A	9A	1A	3	4	5	6	7	8	9A
ROOF		5.42	5.42											
PENTHOUSE	24.66	22.80	23.97	24.66	19.25	19.25	3.90	7.76	6.62	11.39	11.39	6.62	7.76	3.90
8th FLOOR	53.01	36.80	39.11	52.94	46.71	46.71	10.47	20.24	30.65	37.05	37.05	30.65	20.24	10.47
7th FLOOR	65.78	42.71	45.96	62.65	58.22	55.63	13.89	25.46	37.45	43.65	43.53	37.00	25.46	13.89
6th FLOOR	72.36	46.31	50.85	69.01	63.30	62.42	15.87	28.58	41.24	47.62	47.52	40.91	28.58	15.87
5th FLOOR	78.20	49.52	54.82	74.75	68.30	66.70	17.55	31.34	44.70	51.20	51.12	44.43	31.34	17.55
4th FLOOR	83.72	52.54	58.43	80.21	73.15	71.72	19.10	33.92	47.97	54.57	54.50	47.73	33.92	19.10
3rd FLOOR	89.03	55.44	61.84	85.48	77.88	76.56	20.57	36.40	51.13	57.82	57.74	50.91	36.40	20.57
2nd FLOOR	94.22	58.24	64.58	90.62	82.53	81.27	21.99	38.80	54.20	60.96	60.90	53.99	38.80	21.99
1st FLOOR	100.10	70.95	78.19	119.05	87.49	105.07	23.35	41.12	57.19	72.91	79.54	73.49	53.32	27.90

TABLE AU-1. Column Accumulated Ultimate Loads, 1.4DL+1.7LL (cont.)

LETTER AXES :	AXIS G2				AXIS H		AXIS J1							
DIGIT AXES :	4A	5	6	6B	1A	9A	1A	3	4	5	6	7	8	9A
ROOF		27.46	27.46											
PENTHOUSE	159.27	121.60	129.90	159.270.0	141.63	141.630.0	35.89	65.87	45.69	47.76	47.76	45.69	65.87	35.89
8th FLOOR	299.75	203.62	222.34	303.610.0	265.23	265.230.0	72.90	123.32	166.36	178.82	182.63	173.96	123.32	72.90
7th FLOOR	408.53	268.46	292.29	384.620.0	356.25	331.720.0	97.13	158.67	213.79	227.80	230.06	217.92	158.67	97.13
6th FLOOR	491.64	329.38	358.90	459.950.0	416.21	394.580.0	118.91	190.44	253.41	270.96	273.26	257.74	190.44	118.91
5th FLOOR	573.51	389.65	423.95	534.230.0	476.03	453.180.0	140.19	221.63	292.46	313.47	315.78	296.91	221.63	140.19
4th FLOOR	654.83	449.59	488.40	608.020.0	535.60	513.050.0	161.26	252.50	331.19	355.61	357.94	335.68	252.50	161.26
3rd FLOOR	735.81	509.30	552.49	681.480.0	594.98	572.600.0	182.17	283.18	369.74	397.54	399.88	374.25	283.18	182.17
2nd FLOOR	821.43	566.47	619.92	764.120.0	654.69	632.440.0	203.68	314.45	408.81	439.93	442.29	413.34	314.45	203.68
1st FLOOR	937.03	658.61	706.43	921.960.0	731.98	758.600.0	230.56	351.23	470.68	514.82	513.99	488.42	366.34	231.47

TABLE AT-1. Accumulated Column Tributary Areas for Floors 1 to 7 and 11 = 380 kg/sq.m Rev. 03.08.87

LETTER AXES :	AXIS A1								AXIS C		AXIS C1			
DIGIT AXES :	1A	3	4	5	6	7	8	9A	1A	9A	4A	5	6	6B
ROOF														
PENTHOUSE														
8th FLOOR														
7th FLOOR	9.00	17.10	21.42	22.88	23.70	23.10	17.10	9.00	30.69	37.62	28.67	18.24	20.60	41.53
6th FLOOR	18.00	34.20	44.52	46.58	47.40	46.20	34.20	18.00	68.31	75.24	65.67	36.48	41.20	83.06
5th FLOOR	27.00	51.30	67.62	70.28	71.10	69.30	51.30	27.00	105.93	112.86	102.67	54.72	61.80	124.59
4th FLOOR	36.00	68.40	90.72	93.98	94.80	92.40	68.40	36.00	143.55	150.48	139.67	72.96	82.40	166.12
3rd FLOOR	45.00	85.50	113.82	117.68	118.50	115.50	85.50	45.00	181.17	188.10	176.67	91.20	103.00	207.65
2nd FLOOR	54.00	102.60	137.72	141.38	142.20	138.60	102.60	54.00	222.57	225.70	220.87	109.70	123.60	249.25
1st FLOOR	63.00	119.70	160.96	165.08	165.60	161.10	118.80	62.10	260.19	262.70	260.97	128.10	141.70	286.65

TABLE AR-1. LL Reduction Factors in Accordance with A58.1

LETTER AXES :	AXIS A1								AXIS C		AXIS C1			
DIGIT AXES :	1A	3	4	5	6	7	8	9A	1A	9A	4A	5	6	6B
ROOF												1.00	1.00	
PENTHOUSE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8th FLOOR	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
7th FLOOR	1.00	.80	.74	.73	.72	.73	.80	1.00	.66	.62	.68	.79	.75	.60
6th FLOOR	.79	.64	.59	.58	.58	.59	.64	.79	.53	.51	.53	.63	.61	.50
5th FLOOR	.69	.57	.53	.52	.52	.52	.57	.69	.47	.47	.48	.56	.54	.45
4th FLOOR	.63	.53	.49	.49	.48	.49	.53	.63	.44	.44	.44	.52	.50	.43
3rd FLOOR	.59	.50	.46	.46	.46	.46	.50	.59	.42	.42	.42	.49	.48	.41
2nd FLOOR	.56	.48	.44	.44	.44	.44	.48	.56	.40	.40	.40	.47	.46	.40
1st FLOOR	.54	.46	.43	.43	.43	.43	.46	.54	.40	.40	.40	.45	.44	.40

TABLE AT-1. Accumulated Column Tributary Areas for Floors 1 to 7 and LL = 380 kg/sq.m (cont.) Rev. 03.08.87

LETTER AXES :	AXIS C2		AXIS D				AXIS D1			
DIGIT AXES :	5A	6	1A	4A	6B	9A	4A	5A	6	6B
ROOF										
PENTHOUSE										
8th FLOOR										
7th FLOOR	4.68	5.90	16.52	12.41	35.64	35.64	4.80	7.95	7.50	5.70
6th FLOOR	9.36	11.80	52.16	49.31	71.28	71.29	9.60	15.90	15.00	11.40
5th FLOOR	14.04	17.70	87.81	86.21	106.92	106.93	14.40	23.85	22.50	17.10
4th FLOOR	18.72	23.60	123.45	123.11	142.56	142.57	19.20	31.80	30.00	22.80
3rd FLOOR	23.40	29.50	159.09	160.01	178.20	178.22	24.00	39.75	37.50	28.50
2nd FLOOR	28.08	35.40	180.39	166.31	213.84	213.85	28.80	47.70	45.00	34.20
1st FLOOR	32.48	43.90	216.03	206.21	240.93	236.25	35.80	56.16	51.53	37.58

TABLE AR-1. LL Reduction Factors in Accordance with A58.1 (cont.)

LETTER AXES :	AXIS C2		AXIS D				AXIS D1			
DIGIT AXES :	5A	6	1A	4A	6B	9A	4A	5A	6	6B
ROOF	1.00	1.00						1.00	1.00	
PENTHOUSE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8th FLOOR	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
7th FLOOR	1.00	1.00	.81	.90	.63	.63	1.00	1.00	1.00	1.00
6th FLOOR	1.00	.92	.57	.58	.52	.52	.99	.82	.84	.93
5th FLOOR	.86	.79	.49	.50	.47	.47	.85	.72	.73	.80
4th FLOOR	.78	.72	.46	.46	.44	.44	.77	.66	.67	.73
3rd FLOOR	.72	.67	.43	.43	.42	.42	.72	.61	.62	.68
2nd FLOOR	.68	.63	.42	.43	.41	.41	.68	.58	.59	.64
1st FLOOR	.65	.60	.41	.41	.40	.40	.63	.56	.57	.62

TABLE AT-1. Accumulated Column Tributary Areas for Floors 1 to 7 and LL = 380 kg/sq.m (cont.) Rev. 03.08.87

LETTER AXES :	AXIS E						:	AXIS F					
DIGIT AXES :	1A	4A	4B	6A	6B	9A	:	1A	4A	4B	6A	6B	9A
ROOF :													
PENTHOUSE :													
8th FLOOR :													
7th FLOOR :	9.84	3.33	14.70	14.70	38.05	35.64		9.84	3.33	18.40	15.40	38.05	35.64
6th FLOOR :	45.48	41.38	29.40	29.40	76.10	71.28		45.48	41.38	36.80	30.80	76.10	71.28
5th FLOOR :	81.12	79.43	44.10	44.10	114.15	106.92		81.12	79.43	55.20	46.20	114.15	106.92
4th FLOOR :	116.76	117.48	58.80	58.80	152.20	142.56		116.76	117.48	73.60	61.60	152.20	142.56
3rd FLOOR :	152.40	155.53	73.50	73.50	190.25	178.20		152.40	155.53	92.00	77.00	190.25	178.20
2nd FLOOR :	193.30	203.53	88.20	88.20	229.55	213.84		188.00	194.83	110.40	92.40	229.55	213.84
1st FLOOR :	228.80	242.95	104.13	104.13	232.97	213.84		221.95	235.53	128.40	109.10	232.97	213.84

TABLE AR-1. LL Reduction Factors in Accordance with A58.1 (cont.)

LETTER AXES :	AXIS E						:	AXIS F					
DIGIT AXES :	1A	4A	4B	6A	6B	9A	:	1A	4A	4B	6A	6B	9A
ROOF :			1.00	1.00						1.00	1.00		
PENTHOUSE :	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
8th FLOOR :	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00
7th FLOOR :	.98	1.00	.85	.85	.62	.63		.98	1.00	.78	.83	.62	.63
6th FLOOR :	.59	.61	.67	.67	.51	.52		.59	.61	.63	.66	.51	.52
5th FLOOR :	.50	.51	.59	.59	.46	.47		.50	.51	.56	.59	.46	.47
4th FLOOR :	.46	.46	.55	.55	.44	.44		.46	.46	.52	.54	.44	.44
3rd FLOOR :	.44	.43	.52	.52	.42	.42		.44	.43	.49	.51	.42	.42
2nd FLOOR :	.41	.41	.49	.49	.40	.41		.42	.41	.47	.49	.40	.41
1st FLOOR :	.40	.40	.47	.47	.40	.41		.40	.40	.45	.47	.40	.41

TABLE AT-1. Accumulated Column Tributary Areas for Floors 1 to 7 and LL = 380 kg/sq.m (cont.) Rev. 03.08.87

LETTER AXES :	AXIS F1				AXIS G				AXIS G1	
DIGIT AXES :	4A	5	5B	6B	1A	4A	6B	9A	5	5B
ROOF										
PENTHOUSE										
8th FLOOR										
7th FLOOR	5.70	9.80	10.90	9.40	16.52	12.41	35.64	35.64	5.90	3.16
6th FLOOR	11.40	19.60	21.80	18.80	52.16	48.05	71.28	71.28	11.80	6.32
5th FLOOR	17.10	29.40	32.70	28.20	87.80	83.69	106.92	106.92	17.70	9.48
4th FLOOR	22.80	39.20	43.60	37.60	123.44	119.33	142.56	142.56	23.60	12.64
3rd FLOOR	28.50	49.00	54.50	47.00	159.08	154.97	178.20	178.20	29.50	15.80
2nd FLOOR	34.20	58.80	65.40	56.40	194.72	190.61	213.84	213.84	35.40	18.96
1st FLOOR	40.60	69.90	77.35	67.20	230.36	226.25	233.52	213.84	39.90	22.11

TABLE AR-1. LL Reduction Factors in Accordance with A58.1 (cont.)

LETTER AXES :	AXIS F1				AXIS G				AXIS G1	
DIGIT AXES :	4A	5	5B	6B	1A	4A	6B	9A	5	5B
ROOF									1.00	1.00
PENTHOUSE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8th FLOOR	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
7th FLOOR	1.00	.98	.94	1.00	.81	.90	.63	.63	1.00	1.00
6th FLOOR	.93	.77	.74	.78	.57	.58	.52	.52	.92	1.00
5th FLOOR	.80	.67	.65	.68	.49	.50	.47	.47	.79	.99
4th FLOOR	.73	.62	.60	.62	.46	.46	.44	.44	.72	.89
3rd FLOOR	.68	.58	.56	.58	.43	.43	.42	.42	.67	.83
2nd FLOOR	.64	.55	.53	.55	.41	.42	.41	.41	.63	.77
1st FLOOR	.61	.52	.51	.53	.40	.40	.40	.41	.61	.74

TABLE AT-1. Accumulated Column Tributary Areas for Floors 1 to 7 and LL = 380 kg/sq.m (cont.) Rev. 03.08.87

LETTER AXES :	AXIS G2				AXIS H		AXIS J1							
DIGIT AXES :	4A	5	6	6B	1A	9A	1A	3	4	5	6	7	8	9A
ROOF														
PENTHOUSE														
8th FLOOR														
7th FLOOR	28.67	20.60	23.60	41.36	30.69	37.62	9.00	17.10	21.42	23.37	23.70	23.10	17.10	9.00
6th FLOOR	70.20	41.20	47.20	82.72	68.31	75.24	18.00	34.20	44.52	47.07	47.40	46.20	34.20	18.00
5th FLOOR	111.73	61.80	70.80	124.08	105.93	112.86	27.00	51.30	67.62	70.77	71.10	69.30	51.30	27.00
4th FLOOR	153.26	82.40	94.40	165.44	143.55	150.48	36.00	68.40	90.72	94.47	94.80	92.40	68.40	36.00
3rd FLOOR	194.79	103.00	118.00	206.80	181.17	188.10	45.00	85.50	113.82	118.17	118.50	115.50	85.50	45.00
2nd FLOOR	236.39	123.60	147.90	248.20	218.77	225.70	54.00	102.60	136.92	141.87	142.20	138.60	102.60	54.00
1st FLOOR	274.29	123.60	153.15	271.55	254.41	231.74	63.00	119.70	159.99	149.71	142.20	138.60	102.61	54.00

TABLE AR-1. LL Reduction Factors in Accordance with A58.1 (cont.)

LETTER AXES :	AXIS G2				AXIS H		AXIS J1							
DIGIT AXES :	4A	5	6	6B	1A	9A	1A	3	4	5	6	7	8	9A
ROOF		1.00	1.00											
PENTHOUSE	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
8th FLOOR	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
7th FLOOR	.68	.75	.72	.61	.66	.62	1.00	.80	.74	.72	.72	.73	.80	1.00
6th FLOOR	.52	.61	.58	.50	.53	.51	.79	.64	.59	.58	.58	.59	.64	.79
5th FLOOR	.47	.54	.52	.46	.47	.47	.69	.57	.53	.52	.52	.52	.57	.69
4th FLOOR	.43	.50	.49	.43	.44	.44	.63	.53	.49	.49	.48	.49	.53	.63
3rd FLOOR	.41	.48	.46	.41	.42	.42	.59	.50	.46	.46	.46	.46	.50	.59
2nd FLOOR	.40	.46	.44	.40	.40	.40	.56	.48	.45	.44	.44	.44	.48	.56
1st FLOOR	.40	.46	.43	.40	.40	.40	.54	.46	.43	.44	.44	.44	.48	.56

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Public Law 99-591, The Continuing Appropriations Act for Fiscal Year 1987, directed the National Bureau of Standards (NBS) to conduct an independent analysis of the new United States Embassy Office Building being constructed in Moscow. The analysis was to include: "... an assessment of the current structure and recommendations and cost estimates for correcting any structural flaws and construction defects...." This report describes the investigation which included field, laboratory and analytical studies, and its findings. The investigation did not address security and other nonstructural deficiencies. The investigation has identified important structural defects in the building and defined remedial measures to correct them. While important, these structural defects, in comparison to the total structural system for the building, are modest in scale and fully correctable.

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