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U.S.-JAPAN COORDINATED PROGRAM FOR MASONRY BUILDING RESEARCH

REPORT
9.1-1

DESIGN OF REINFORCED MASONRY RESEARCH BUILDING PHASE I

by

JOHN C. KARIOTIS
ALBIN W. JOHNSON

SEPTEMBER 1987

supported by:

NATIONAL SCIENCE FOUNDATION

GRANT NO. CES-8517031

Kariotis & Associates

This report presents the results of a research project which was part of the U.S. Coordinated Program for Masonry Building Research. The program constitutes the United States part of the United States - Japan, coordinated masonry research program conducted under the auspices of The Panel on Wind and Seismic Effects of the U.S. - Japan Natural Resources Development Program (UJNR).

This material is based on work supported by the National Science Foundation under the direction of Program Director, Dr. A.J. Eggenberger.

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KARIOTIS & ASSOCIATES
711 Mission Street, Suite D
South Pasadena, CA 91030
(213) 682-2871

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SECTION 1
INTRODUCTION

1.1 RESEARCH PLAN OF TCCMAR

The U.S.-Japan Coordinated Program for Masonry Building Research is coordinated on the U.S. side by the Technical Coordinating Committee for Masonry Research (TCCMAR). The goal of the TCCMAR research program is to formulate design recommendations for reinforced masonry. Progress to this goal begins with material model testing. It develops into masonry component testing and assemblage testing. Mathematical models are developed concurrently with experimental work and are validated by the data obtained in the testing. The final validation model is the full-scale research building.

The TCCMAR research was planned to culminate its experimental research with the testing of a full-scale building comparable to the full-scale reinforced masonry research building built and tested in Japan. The criteria for the TCCMAR research building was that it would incorporate the materials, construction techniques and design procedures that are indigenous to the United States.

1.2 CATEGORY 9 RESEARCH

Category 9 tasks of the TCCMAR research program include a preliminary and final design of a full-scale test structure, planning and construction of a test facility, preparation of a test plan, construction of the test structure, and conducting a full-scale test of a multistory reinforced masonry building.

Task 9.1, Design of Reinforced Masonry Research Building, is divided into two parts. The Phase 1 task includes the development of preliminary designs for candidate research buildings, estimation of the story shear capacity of each candidate building, estimation of the interstory force-displacement characteristics in both the elastic and inelastic range, and the reporting of the data to TCCMAR members.

Task 9.1 is coordinated with Tasks 2.1, 2.2 and 2.3 of Category 2 research. The preliminary mathematical models developed in these tasks are utilized to estimate interstory component capacity and force-displacement characteristics.

Task 9.1 provides data for Task 9.2, Specific Test Facility Requirements; Task 9.3, Facility Preparation; Task 9.4, Full-Scale Test Plan; and Task 9.1 - Phase II, Design of Reinforced Masonry Research Building.

The candidate research building should represent an assemblage of masonry components and common structural materials that equals the complexity of the component testing, both in geometric relationships and in randomness of openings in the reinforced masonry walls. The constraint on the selected geometry of masonry components was that the research building should reasonably match current reinforced masonry construction.

A range of load-capacities of the reinforced masonry components that interconnect the stories of the research building are estimated to provide data for Task 9.2, Specific Test Facility Requirements.

The preliminary design of the research building utilizes all facets of completed TCCMAR research to plan a combination of reinforced masonry components that will have, at maximum test displacements, shear and flexural strain magnitudes that are characteristic of inelastic displacements.

1.3 RESEARCH OBJECTIVES

The overall plan of the TCCMAR research includes the testing of a full-scale research building. The test of the full-scale building will validate the mathematical models that include the contribution to in-plane structural stiffness of horizontal and out-of-plane elements.

The specific objective of Task 9.1 is to design a research building that will incorporate all facets of prior component testing and mathematical modeling into a single test structure. The research building must provide a full-scale assembly for instrumentation and be equal to the complexity of predicting force-displacement and strain relationships for the reinforced masonry components that were previously tested.

1.4 RESEARCH PLAN

The research plan of Task 9.1-1 is composed of the following sub-tasks:

Identify candidate multistory masonry buildings that represent a significant portion of current masonry construction.

Develop a schematic research building for each candidate building. Incorporate into the research building, structural components that will significantly influence the structural response to seismic ground motions.

Develop preliminary structural designs for the proposed reinforced masonry research buildings with consideration of the seismic design requirements of the hazard zones of the United States (ATC 1978) (EHI 1984) (UBC 1982).

Report the results of the research to TCCMAR. Develop the general configuration of the research building.

Coordinate with Task 9.2, Facility Requirements. Report the results of the research.

SECTION 2
DEVELOPMENT OF THE RESEARCH BUILDING

2.1 CRITERIA FOR SELECTION OF THE CONFIGURATION

The selection of the research building as a multistory building in excess of five stories dictated that the building be of incombustible materials with a minimum fire rating of the structural components of one hour. The fire resistance criteria is common to building codes throughout the United States. To fit this materials' criteria, a concrete floor assembly was selected. Reinforced masonry buildings of this height are commonly bearing wall buildings with the reinforced masonry walls providing the vertical load carrying system and the lateral load resisting system.

Two typical configurations of bearing wall buildings were selected as candidates. Each had a single structural wall in the loading direction line B, Figure 1, and bearing masonry walls perpendicular to line B. Both of the proposed configurations had a single reinforced masonry wall coincident with the center of the laboratory strong wall. The load bearing walls, lines 1, 2, 3, and 4, Figure 1, stabilize the test wall and add flanges to the structural elements that comprise the lateral load resisting wall.

A single wall in the test loading direction was common to both preliminary designs, to minimize the total forces required to displace the wall to its peak strength, and to eliminate changes in the plan location of the center of resistance that would occur if multiple non-symmetric lines of walls were in loading direction. The common strain in the load bearing walls at their juncture with the lateral load resisting wall, line A, does cause the top of the walls to displace perpendicular to the loading direction. The stiffness of these walls in the load-normal direction greatly exceeds the stiffness of the wall on line A. The loading devices will be planned to accommodate these displacements normal to the loading direction.

Figures 1 and 2 illustrates the research building selected by the TCCMAR members. The differences of the two candidates were small, the configuration selected was based on the similarity to common configurations, stiffness in the load-normal direction, and the complexity of analytical solutions for strength and stiffness.

2.2 CRITERIA FOR THE STRENGTH AND STIFFNESS

The TCCMAR group has indicated that the full-scale testing will be by a Generated Sequenced Displacement (GSD) rather than a sequentially phased displacement series. GSD testing is also described as pseudo-dynamic testing. In this testing procedure, the displacements are computer controlled to simulate the structural response to a time-history of ground motions. The preliminary estimates of stiffness of the structural elements are revised at each load increment by data derived directly from the research building. GSD testing requires that the stiffness data points taken from the specimen have a minimum of discontinuities. Very stiff structures have a probability of discontinuity of stiffness data points. This is principally due to the difficulty of measuring small strains in the research building.

The initial stiffness of the research building should be a minimum and the cracking and yielding of the structural elements should cause a smooth progression of non-linear displacements.

These criteria indicated that the research building should have a peak strength that is appropriate for a seismic zone of 0.2 g intensity (ATC 1978). This design strength level will be appropriate for the described strength criteria.

The initial stiffness of the test building is not related to the probable peak strength. The reinforcing patterns have the principal influence on the peak strength. The rate-change in stiffness is related to the configuration of the structural elements and their relative stiffness at a common displacement. The opening pattern in line B was planned to simulate a reinforced masonry bearing wall building and to provide a smooth transition in stiffness degradation. The location of walls normal to the test wall, line B, cause the vertical reinforcement patterns to deviate from a desirable pattern and a desirable vertical reinforcement ratio. The flange effect, excess quantities of effective vertical reinforcement, can be partially offset by horizontal reinforcement patterns that allow tension field (shear) softening.

2.3 DESCRIPTION OF THE RESEARCH BUILDING

The research building is shown in plan in Figure 1. This plan is typical of levels 2 through 6. The floor construction is six inch thick hollow core pretensioned precast planks. The planks are covered with reinforced two-inch thick cast-in-place concrete. The planks bear on the masonry walls at lines 1 through 4. The floor extends from line B to C to simulate an entry balcony and to provide access for instrumentation and for the observation of the behavior of the research building. This portion of the floor is supported on a framework of steel tubes.

The elevation of the test wall is shown in Figure 2. The door and window openings are typical of each level, and each unit is typical or reversed plan. There is no wall at line A. This plan replicates a hotel suite or apartment building that is planned for a desirable view such as at the ocean front.

The bearing walls, lines 1 through 4, support the precast plank floor and stabilize the test wall, line B. These walls are reinforced vertically with uniformly spaced reinforcement. The reinforcement patterns are determined by estimates of the response to a seismic zone of 0.2 g intensity (ATC 1978). These walls are not penetrated by openings as they represent the division walls between occupants.

2.4 COMPONENTS OF THE RESEARCH BUILDING

The masonry units used for the research building are 6 x 8 x 16 inch nominal concrete blocks. Mortar is type S and the grout is coarse grout with an admixture to offset shrinkage. The anticipated prism strength is 3000 psi at 28 days. Open end blocks will be used to lay up the wall where splices at the floor levels are not detailed. Horizontal reinforcement is laid in bond beam blocks and terminated in a standard hook. All grout is consolidated by mechanical vibration. All masonry units are laid in running bond.

Reinforcement, #4 and larger, are specified as conforming to ASTM A 615, Grade 60. #3 bars are specified as Grade 40. The yield and ultimate stress of all the reinforcement will be determined by testing of the reinforcement. All the reinforcement as identified by a heat number will be tested. Non-identified reinforcement will not be used. The reinforcement will be centered in the masonry unit and wired in place at the bottom of the grout pour. The top of the reinforcement bar will be held in the center of the unit by a wire spacer. Construction joints will be at the top of each floor level, 1 through 6, and below each floor level as shown on Figure 7.

The sequence of construction is as follows:

> The masonry units are placed to the bottom of a floor level. The reinforcement is placed with extensions to the detailed splice position above the floor level.

Grout is placed in a single pour with maximum lifts of four feet. The grout level is held below the top of the top unit to form a key for the next lift.

The precast plank floors are placed on the bearing walls. The plank bears on the face shell of the masonry units.

A cut block is placed for an edge form at the exterior walls and the reinforcement for the topping is placed.

The planks are supported by temporary shoring at their center to minimize their relative deflections during the placement of the concrete topping.

The keyway between the planks is grouted during the concrete placement by dragging a small vibrator over the joint during the concrete placing operation.

The topping concrete is screeded to the floor level to complete a story.

The operation repeats with the mason placing the next story walls on a prepared construction joint at the floor level.

Construction joints at the bottom of a story height wall are made by exposing clean aggregate to a one-quarter inch amplitude. The first course above the construction joint will have cleanouts for the removal of mortar droppings and debris at every vertical bar. In addition to these cleanouts, a minimum of one cleanout per unit will be provided. The first unit laid above the construction joint will have a cutout of the crosswebs, similar to a bond beam block, top and bottom. The top cutout is to accommodate horizontal reinforcement; the bottom cutout is to allow an air or hose stream to sweep the construction joint clean from cleanout to cleanout.

SECTION 3
DESIGN OF THE RESEARCH BUILDING

3.1 NON-TECHNICAL DECISIONS

TCCMAR decided in its early deliberations that the five-level research building is a segment of a larger building. The location of the segment excised from the larger building was not fully defined, but it is considered to be a portion with additional stories above the five-levels of the research building. The horizontal extent of the larger building is greater than the research building but the repetitiveness of the wall element configurations will minimize any length effect.

The depth of the floor system perpendicular to the line B, was selected to provide adequate stability to the test wall. The floor system in itself is not tested in this experiment as a part of a complete structural system. The transfer of the applied loads to the test wall is a test of the connection of the floor to the wall, but the other joints of the floor system are not loaded.

This loading is not representative of inertial response to ground motions, but the presence of the floor system will have a significant effect on the strength and stiffness degradation of the test wall.

3.2 DESIGN DECISIONS

During the process of selecting a configuration for the research building, the following design decisions were made:

Six-inch concrete masonry units were selected for all reinforced masonry walls. This size was selected to match the structural element testing and to minimize the strength and stiffness of the research building.

The research building is considered as a segment of a larger multi-story reinforced masonry building. The design strength of the segment will assume that the full prototype structure is constructed in a seismic zone other than California, Nevada, or Alaska. The seismic zone is taken as EPA=0.2 g (ATC 1978).

The concrete masonry units will be solid grouted. Construction joints, connections of the floors to the walls, and similar construction details will be planned to minimize construction flaws and difficult grout placement.

Minimum vertical reinforcement to meet the strength requirements should be used. However, a decision was made to provide a minimum of four vertical bars in a pier. These bars will be evenly distributed over the pier width. The horizontal reinforcement patterns will be planned to have acceptable shearing distortion ductility. The quantity of horizontal reinforcement will be selected to provide a peak pier strength that is limited by the principal compressive strains at the edge of the pier.

Splices in the vertical reinforcement will not be made in critical compressive strain zones. The vertical bars that are within the compression block dimension will be spliced in stress zones that are seventy to eighty percent of the maximum stress. Lap length will be that now specified in national codes.

The test wall will include a wide variety of structural assemblies linked by the floors. These assemblies will have a wide variation in their displacement at peak strength.

Yield in the initially stiff assemblies will transfer load to the more flexible elements and provide a smooth transition in stiffness to the combined peak strength value. The subtypes of structural elements is as follows:

- o A wall with a central window opening, a wall intersection (flange) at one edge, and minimal coupling beams at the other edge.

- o A slender wall with a wall intersection at the center of the pier and minimal coupling beams at both wall edges.

- o A broad-based wall with two symmetrical window openings, a wall intersection at the center of the wall and minimal coupling beams at both wall edges.

- o A very slender wall with a wall intersection at one edge and minimal coupling beams at the other edge.

A reinforced masonry wall will extend to typical window sill height above level six. This wall will not be continuous over the typical door lintels. The wall extension maintains the stiffness relationship of the assemblage at lower levels of "beam", "column" and "joint".

Use of shear keys in addition to the detailed construction joints at level one was considered. The shear keys would be in the central half of the wall section. Shear keys are not recommended for the research building because the flexural yielding at the wall base is not expected to be the dominant mode of displacement.

3.3 DESIGN PROCEDURES

The Phase 1 design of the research building was completed prior to development of the analytical programs of TCCMAR Category 2 research. Analytical programs for the review of engineering designs are developed concurrently with the Phase 1 research. The current status of the analytical research is a validation phase. The experimental research utilized for the validation is Category 3, Task 3.1(a). This experimental research is the cyclic testing of six feet square reinforced masonry panels. These panels represent a fixed base single degree of freedom structural element. Analytical concepts developed during the validation research were used when applicable, but the design of the research building was based on available strength design theory that is similar to reinforced concrete design theory.

The effect of flanges, wall returns, or intersections, on the pier sections are known to have a very significant modification of the peak strength and the symmetry of the strength of reinforced masonry systems. To quantify this effect, the walls perpendicular to the test walls were designed in accordance with a concept of probable future seismic design requirements.

The dead and live load of the full-scale research buildings was calculated in accordance with the concepts developed by TCCMAR. The pretensioned floor planks were designed to meet the minimum requirements for the support of design dead and live loads. These planks are part of a structural assembly that couples the pier elements, Figure 2. It is desirable to minimize coupling moment capacity. The estimated end moments cause the associated shear to exceed the probable shear capacity of the combined floor - masonry beam sections. Addition of shear reinforcement in the grouted core of the masonry over the door, may not be fully effective in limiting damage to the masonry and/or floor, when the masonry section is loaded with combined principal compressive and normal tensile strains. Minimization of the preload in the planks and in the area of the prestress strand was the design goal.

The design of the reinforced masonry walls perpendicular to the test walls was based on estimates of the structural response to ground motions of an intensity of 0.2 g. A structural amplification of 1.5 and an element amplification of 2.0 was estimated. (Appendix A, Pg. W1) The spacing of the reinforcement required for the unfactored strength was less than that considered rational for uniform stress distribution. A vertical reinforcement of #4 bars at 48 inch spacing was chosen as a minimum. The axial load per story was computed, and the number of stories of the full-scale building was determined to be seven. As discussed previously in Section 3, the research building is a segment of a larger building. The basis for the number of stories was that the axial compressive stress at the wall base would be about 0.2 kips per square inch when loaded with the dead load of five levels and the superimposed load of the theoretical additional stories. The superposition at additional loading at the sixth level to stimulate the added stories is experimentally feasible. The total of the superimposed load is 270 kips. (Appendix A, Pg. W2).

The weight of the theoretical seven story building was estimated as 1,200 kips. The probable minimum moment capacity of the transverse walls is slightly less than the probable required strength but is considered adequate. (Appendix A, Pg. L1). The base shear associated with the required strength normal to the test wall was estimated as 480 kips (Appendix A, Pg. L2). The required horizontal reinforcement of #4 at 32 inch spacing was calculated by use of a formula developed by the U.S.-Japanese Cooperative Research (Matsumura, 1987). The vertical and horizontal reinforcement is shown in Figure 3.

The probable moment strength of piers 1, 2, 3 and 4 were calculated using different concepts of the strength of the coupling beams. Piers 1, 2, 3 and 4 are the shear wall structural elements as shown in Figure 2. The numbering of the elements corresponds to the piers, from left to right, that are linked by coupling beams. The probable flexural strength is based on a simplified representation of the flexural strength that does not calculate compressive strain at the fixed base. The design of the horizontal reinforcement is based on the estimated flexural strength. The total of the flexural strengths of the four piers was used to determine a probable base shear parallel to the test wall and a distribution of this base shear to the piers (Appendix A, Pg. L8). The probable base shear was estimated for two different stiffnesses and end rotations of the coupling beams.

3.4 RESULTS OF THE PRELIMINARY DESIGN PROCESS

The design process determined the quantities and distribution of the reinforcement within the reinforced masonry. The configuration of the masonry was determined by the process described in Section 2 and 3.

The quantities and distribution of the masonry reinforcement is as follows:

> The transverse walls on lines 1, 2, 3 and 4 are reinforced with #4 bars at 48 inches on center vertically. The horizontal reinforcement is #4 bars of 32 inches on center. One #4 bar is placed at each floor level. The vertical reinforcement is uniformly spaced as indicated on Figure 3.
>
> The test wall is vertically reinforced with #4 bars as shown on Figure 2. Figures 4, 5 and 6 indicate the specific details of the reinforcement. A minimum of four #4 bars are

in each pier, (wall section between doors and windows). When intersecting walls require an additional vertical reinforcing bar, Detail J, Figure 7, maintenance of symmetry requires that five #4 bars be used.

The vertical reinforcement in the wall beams above and below the window is #4 bars spaced at 24 inches. These bars are terminated by a standard hook at the window head and sill. The vertical bars are spliced at each floor level.

The horizontal reinforcement in the piers at the window level is #4 at 8 inch spacing in levels 1, 2 and 3. The horizontal reinforcement is #3 at 8 inch spacing in levels 4 and 5. A #4 horizontal is continuous above the window and door. Two #4 bars spaced at 8 inches are continuous between doorways. Additional horizontal reinforcement below the window is spaced at 16 inches on center. All horizontal reinforcement is terminated with a standard hook.

Horizontal reinforcement in the piers between the doors, or door and wall corner is #4 bars at 16 inches on center. The first bar is placed in the first masonry unit that is laid at the floor level. Horizontal reinforcement is terminated at a door edge in a standard hook or by a 90 degree bend at the wall corner.

SECTION 4
CONSTRUCTION DETAILS OF THE RESEARCH BUILDING

4.1 FLOOR SYSTEM

The floor system is an assembly of six inch thick hollow core planks. The preliminary design is based on the use of a plank that is manufactured on the West Coast. This plank is one meter in width and the module is 3 feet, 4 inches. The planks are factory cast in a prestressing bed. The planks are pretensioned with four prestressing cables. The prestressing stands are 3/8 inch round with a minimum ultimate stress of 270 kips per square inch.

The plank is topped with two inch thick concrete. The topping concrete is reinforced with #4 bars at 18 inches perpendicular to the planks. Continuity of the longitudal tensile capacity of the plank is provided by #4 bars at 12 inch spacing at the ends of the planks.

The planks are supported on mid-center shoring during the field concrete placing operation. The edge keyway is grouted during the concrete placing operation.

4.2 INTERCONNECTION DETAILS

The research building is constructed of three structural materials. Plant produced pretensioned concrete floor panels, site manufactured reinforced masonry and on site-cast reinforced concrete topping on the floor planks. These materials are integrated on site by reinforcement. The manufacture of the floor planks essentially prohibits extension of reinforcement from the off-site manufactured planks. The interconnection of these units to the structure must be accomplished by bonding the topping to the floor plank and embedding interconnection reinforcement in the topping.

The pretensioned plank provides a very substantial quantity of reinforcement at the floor level when the plank is parallel to the masonry wall. In addition to this reinforcement, the plank is precompressed. Diagonal tension cracks will be intercepted and constrained at each floor level. The anomaly in strain at the upper and lower boundaries of the floor will increase transfer stress at the juncture. These interconnections are made in the research building by the cast-in-place concrete. The topping is bonded to the top and edge of the plank and is cast into the cells of the masonry unit below the floor level. Detail A/7, B/7, C/7, and D/7, Figure 7, illustrates the interconnection of the floor system to the masonry wall system. Detail D/7, Figure 7, illustrates the special case where the floor system and a portion of the masonry wall forms a coupling beam. The strain transfer at the bottom of the floor system will probably exceed the materials capacity. Degradation of this joint is anticipated. The closely spaced vertical reinforcement in this beam, #3 bars at 8 inches on center, is specified to minimize the probability of detachment of the masonry units.

Interconnection of the masonry walls at corners and intersections are made by grouted cores that join the vertical grout cores, Detail I/7, J/7, Figure 7. These details are not common to current construction practice but are required for the research building. The flange effect has a very substantial modification of the flexural strength of a pier; the shearing strains that are associated with these flexural strengths require that the interconnection strength be equal to the shearing strength of masonry laid in running bond.

4.3 ANCHORAGE AND SPLICING OF REINFORCEMENT

Horizontal reinforcement is terminated at a door or window edge with a standard hook around the vertical reinforcement. At a corner, the horizontal reinforcement terminates with a 90 degree bend. It is lapped a splice length with the horizontal reinforcement in the intersecting wall when the horizontal levels of the reinforcement coincide. Horizontal reinforcement is spliced with a 40 diameter lap in the center of the pier, when the pier length exceeds three times the splice length. When the pier dimension is less than this multiple of splice length, the horizontal reinforcement is continuous.

Horizontal reinforcement is not spliced in beam sections. The wall assembly is considered as an assembly of a beam, column and joint. Beam reinforcement (horizontal bars) is spliced in the center of a joint.

Vertical reinforcement, with some restrictions, is generally spliced with a 40 diameter lap immediately above each floor level (including level 1). Splicing of the vertical reinforcement is limited to reinforcement that is not within the estimated in-plane length of the flexural compression block. Vertical bars in this zone may not be spliced. These edge bars may be spliced at the floor level above the critical level. This restriction is applicable to the edge bars at the doorway.

4.4 LOADING SYSTEM

Design of a loading was not a part of this research. However, schematic planning for loading the research building was coordinated with Task 9.3 of Category 9 research. It is anticipated that the load application will be at each floor level by four steel struts that are symmetrical with the intersection of the floor level and the test wall at line B.

Openings are required through the traverse walls, lines 1, 2, and 3, to allow the load strut to load each bay, Figure 3. The maximum probable load to be applied to the research building is 780 kips. The loading application probably will require that 30 percent of the total base shear be applied at a single level. The uniform application in each bay of this loading would require a connection capacity of about 40 kips per load point. The load must be applied uniformly to each bay; this requirement implies that the load strut is infinitely stiff, a condition that is impossible to attain. Loading of the floor slabs through elastoplastic shear elements of variable thickness in conjunction with a variable section load beam can approach the ideal condition.

The displacement at the top of the research building at the peak strength can not be determined at this time. However, an estimation that the peak strength will coincide with a drift angle of 1/100 is supported by the Japanese experimental program. This estimation of peak strength is at a greater drift angle than that determined by the three-story high Japanese specimens, (Yamazaki, 1987). The estimated was modified to account for the larger variability of initial stiffness of the structural elements that comprise the U.S. TCCMAR research building.

SECTION 5
PROPOSED PHASE 2 RESEARCH

5.1 CRITERIA FOR THE FINAL DESIGN

The final design of the research building will be based on an analysis of the preliminary design. The analytical tools will be those developed by Category 2 research. The force-displacement relationships of the piers and coupling beams will be fully developed by use of structural component models (SCM) developed in Task 2.1 of Category 2 research. Envelope and cyclic hysteretic behavior will be plotted for the range of displacement of interest. These loading, unloading, reloading, and degrading hysteretic plots will be replicated by interstory springs in the Lumped Parameter Model (LPM). The LPM will be excited by time-histories of relevant ground motions. The preliminary design will be modified as necessary to provide a minimum strength and a stable cyclic behavior.

5.2 PROBABLE VARIATIONS OF FINAL DESIGN FROM PRELIMINARY DESIGN

The preliminary design did not have a method for the determination of the end rotation-shear relationship of the coupling beams or a method for determination of the rotation of the interface of the coupling beam and the piers. The experimental testing of Category 3 research and development of analytical models to replicate and extrapolate the testing will probably have a significant impact on the assumptions used to estimate the effects of the coupling beams on the peak strength and stiffness degradation of the piers.

SECTION 6

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SECTION 7

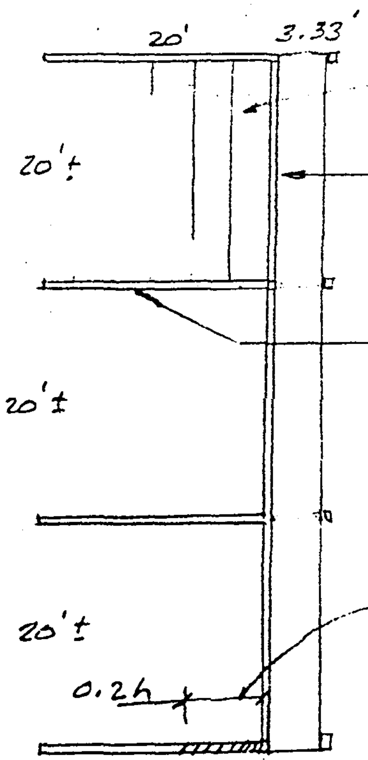
APPENDIX A

CLIENT NSF

KARIOTIS & ASSOCIATES
STRUCTURAL ENGINEERS

SHT. A-1 OF
BY MK
DATE 16 SEPT
JOB NO. 85 100.

SUBJECT: CATEGORY 9 - TEST BUILDING
ITEM DESIGN CRITERIA



TYP FLOOR FRAMING PLAN

EPA ~ 0.2 G

STRENGTH DESIGN CONCEPTS

$f'_m = 2000$ psi, ($f_y = 67$ KSI FROM TEST PROGRAM

ALL WALLS 6" CONC BLOCK, GROUTED SOLID

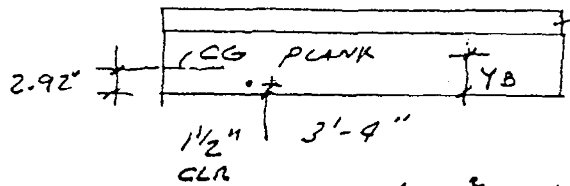
DL, 2" CONC TOPPING	25	PSF
6" CONC PLANK	47	PSF
CEILING	3	PSF
PARTITIONS	5	PSF
DL =	80	PSF

LL (APT OR HOTEL), LL = 40 PSF

W = 80 x 20' = 1600 #/' FLR
56 x 8.67 = 486 #/' WALL
2086 #/ / FLOOR DL

CLIENT NSF	KARIOTIS & ASSOCIATES STRUCTURAL ENGINEERS	SHT. B-8 OF	BY MK
SUBJECT: CATEGORY 9 - TEST BUILDING		DATE 16 SEPT 87	
ITEM PRESTRESSED PLANK		JOB NO. 85-1007	

SPAN ~ 20'



2" TOPPING $f'_c = 3$ TYPE II, 1 HR

6" PLANK $f'_c = 4$

	A in²	Y	$A \cdot Y$ in³	d	Ad^2	I_o
TOPPING	80	7"	560	2.72	591.87	26.67
PLANK	160 in²	2.92"	467.2	1.36	295.94	635.00
	240 in²		1027.2 in²		887.81	661.67

COMPOSITE:

$Y_B = 1027.2 / 240 = 4.28"$

$Y_T = 3.72"$

$I_T = 1549.48 \text{ in}^4$

$S_B = 362.03 \text{ in}^3$

$S_T = 416.53 \text{ in}^3$

PLANK

$Y_B = 2.92$

$I = 635 \text{ in}^4$

$S_T = 206 \text{ in}^3$

$S_B = 217 \text{ in}^3$

			M
PLANK @ ERECT.	$w = 47 \text{ PSF} \times 3.33' =$	156.51 #/'	7.80 'k
W/ WET CONC	$= (47+25)\ 3.33' =$	239.76 #/'	11.99 'k
W/ LL & PART & CEIL'G	$(40+8)\ 3.33 =$	159.89 #/'	7.99 'k

4 - 3/8" Φ STRANDS, $f_{pu} = 270$ KSI (MIN. STRAND PATTERN AT LOCAL PLANTS)

$e_{plank} = 2.92" - 1\tfrac{1}{2}" - 3/16" = 1.23"$ + TENSION

$e_{composite} = 4.28 - 1\tfrac{1}{2}" - 3/16" = 2.59"$ - COMPRESSION

CAPACITY OF STRAND = $0.7 \times 270_{KSI} \times 0.085 \text{ in}^2$ = 16.07 k @ transfer

AFTER LOSSES, SAY $0.78 \times$ transfer = 12.53 k @ service

AT TRANSFER @ ENDS:

		BOT	TOP
P/A	$= 4 \times 16.07$ STRANDS / 160 in² =	-0.40 KSI	-0.40 KSI
Pe/S_T	$= \dfrac{16.07^k \times 4 \times 1.23"}{206 \text{ in}^3}$		0.38 KSI
Pe/S_B	$= 16.07 \times 4 \times 1.23 / 217$	-0.36 KSI	
		-0.76 KSI	-0.02 KSI

SAY $f'_{c\,initial} \sim 0.7 \times 4 \text{ KSI} = 2.8 \text{ KSI}$

ALLOWED $0.6 \times 2.8 = -1.68$ KSI OK

SERVICE LOADS:
AT MIDSPAN

		BOT	TOP
P/A		-0.40 KSI	-0.40 KSI
$P.e/S_B, S_T$	$= \frac{12.53^k \times 1.23'' \times 4}{217,206}$	-0.28	0.30 KSI
$M_{DL}/S_B, S_T$	$= \frac{11.99 \times 12}{217,206}$	0.66 KSI	-0.70 KSI
		-0.02 KSI	-0.80 KSI
$\frac{M_{LL}}{S_B, S_T}$	$= \frac{7.99'k \times 12}{362,416.5}$	0.26 KSI	-0.23
	D+L	0.24 KSI	-1.03 KSI

ALLOWED TENSION $6\sqrt{4000}/10^3 = 0.379$ KSI OK
" COMP 0.45×4 -1.80 KSI OK

DEFLECTION @ MIDSPAN: $ML^2/8E_cI$ FROM PRESTRESS
$5wL^4/384E_cI$ VERTICAL

AT RELEASE (W/ LOSSES): AT ERECTION:

$$\frac{12.53^k \times 4 \times 1.23'' (20)^2\, 144}{(8)(3.8\times10^3)\, 635} = 0.18'' \uparrow \times 1.65 \qquad 0.30 \uparrow$$

$$\frac{(5)(.240^{k/\prime})(20)^4\, 1728}{(384)(3.8\times10^3)\, 635} = 0.36'' \downarrow \times 1.75 \qquad 0.63 \downarrow$$

$0.33 \downarrow$

MOMENT $d = 8'' - 1\tfrac{1}{2}'' - 3/16 = 6.31''$

$M_U = \phi f_{ps} A_{ps} (d - a/2)$

$f_{ps} = f_{pu}(1 - (0.5)\rho_p \times f_{pu}/f'_c) = 270\left(1 - 0.5 \times \frac{0.34\text{ in}^2}{40 \times 6.31} \times 270/3\right) = 253.6$

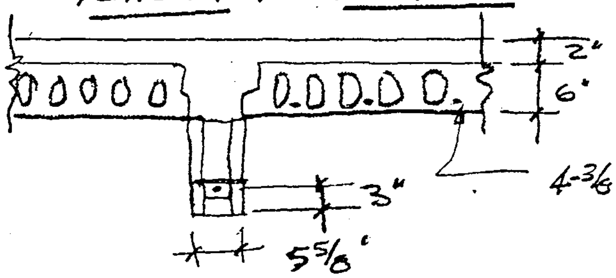
$a = 0.34\text{ in}^2 \times 253.6\text{ ksi} / (0.85)(3)\, 40'' = 0.85''$

$M_U = (0.9)(253.6)0.34\,(6.31' - .85/2) = 456.9''k /12''/' = 38.07'k$

$M_{U\,DEMAND} = 1.4 \times 12'k + 1.7 \times 8'k = 30.00'k$ OK

Flexural Capacity Estimate of coupling bms.

TENSION IN BOTTOM



M_u 2 planks pg B-2 = $38.1^{1K} \times 2$

$$M_n = \frac{38.1 \times 2}{0.9} + 0.2 \times 67. \times 1.4' = 103.4^{1K}$$

TENSION IN TOP

Preload in 2 panels = $12.53^K \times 8$ strand = 100.24^K

Required compressive block @ 3000psi ~ $33.41^{□"}$

Area masonry = $5.62" \times 12" = 67.44^{□"}$

Tu strand = $0.34^{□"} \times 253.6^{ksi} \times 2 = 172.45^K$

member critically over reinforced

$$M_n \sim 172.45 \times \left(\frac{6"+1"}{12}\right) = 100.6^{1K}$$

$$V_n \sim \frac{103.4 + 100.6}{3.33'} \sim 61^K$$

Materials Properties

$f'_m \sim 3000$ psi $F_y = 60^{ksi}$ for #4 & larger

Design seismic zone EPA = 0.2 g

Estimate response in upper stories @ 1.5× base EPA = 0.3

Wall amplification 2.0

6" conc blk grouted solid $w = 57\ \#/\square'$

$M \sim (0.3 \times 2.0) \times 57 \times \frac{8.0^2}{10} = 0.22\ ^{'k}/_{'}$

Near fixed @ floor level.

$d = 2.8"$ Use strength design as for concrete

$A_s = \frac{0.22}{4.45 \times 2.8"} = 0.02\ ^{\square"}/_{ft}$ Use #4 @ 48"

$A_s = 0.05\ ^{\square"}/_{ft.}$

$\rho_v = 0.00741$

Axial loads on walls - design per ft. of wall

Design load $80\ \#/\square' \times 20' = 1.6\ ^k/_{'}$

Wall wt $57\ \#/\square' \times 8.0' = 0.46\ ^k/_{'}$

LL (60% reduction) $40 \times .4 \times 20 = 0.32\ ^k/_{'}$

$2.38\ ^k/$ story

$\frac{2.38}{5.62 \times 12"} = 35.3$ psi story

Actual load in test per ft. of wall

Floor @ $72\ \#/\square' \times 20 = 1.44\ ^k/_{'}$

Wall $0.46\ ^k/_{'}$

$1.9\ ^k/$ story

Axial @ Research bldg = $5 \times 1.9^{k}/_{1}$ /story + $3.3 \times .057 = 9.69^{k}/_{1}$

axial stress = $0.14\ ^{k}/_{\square''}$

If seven story bldg axial stress @ base = $7 \times 1.9^{k}/_{1} = 13.3^{k}/_{1}$

Axial stress = $0.20\ ^{k}/_{\square''}$ Lines 2 & 3

@ lines 1 & 4

Load = $72 \times 10 = 0.72^{k}/_{1}$

Wall $57 \times 8.0 = \underline{0.46^{k}/_{1}}$

$1.18^{k}/_{1}$ /story $\times 7 = 8.26^{k}/_{ft}$

Axial stress = $0.12\ ^{k}/_{\square''}$

Wall Line B precast plank parallel to wall
camber growth upward

$57^{\#}/_{\square'} \times 0.74$ solid $\times 8.67' \times 7$ stories = $2.6^{k}/_{1}$

net $\dfrac{2.6^{k}/_{1}}{5.62 \times 12} \times \dfrac{20.33}{14.34} = 55$ psi

Added load Lines 2 & 3 = $3.61^{k}/_{1}$

Lines 1 & 4 = $2.17^{k}/_{1}$

Line B = $0.61^{k}/_{1}$

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ITEM Lateral Load design - normal to test wall		JOB NO. 85-1007

Seismic response 7 story bearing wall bldg.

Roof $61' \times 24'$ @ $86^{\#}/_{\square'}$ = 126^{K}

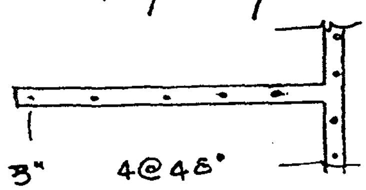
Walls @ $57^{\#}/_{\square}$ $\times (4 \times 20 + 61 \times 0.74) \times \frac{8'}{2} = 28.5^{K}$ 154.5^{K}

Typical Floor = 117^{K}

57^{K} 174^{K}

Total 6 floors + roof = $1,199^{K}$

Probable response of yielding structure $0.3g \times 1,199^{K} = 360^{K}$



3" 4@48"

Assumed yield 67^{ksi}

Lines 1 & 4 $M_n \sim (5 \times 0.20 \times 67^{ksi}) \times \frac{21}{2} + (8.26 \times 21) \times \frac{21}{2}$

$703.5^{'K} + 1821^{'K} = 2,525^{'K}$

Lines 2 & 3 $M_n \sim 703.5^{'K} + (13.3 \times 21) \times \frac{21}{2} = 3,636^{'K}$ (2,933)

Total M_u = $2 \times 2,525 + 2 \times 3,636 = 12,321^{'K}$

M_E @ base $\sim 360^{K} \times 60.7 \times 0.6$ estimated $= 13,100^{'K}$

Response underestimated but moment capacity adequate

Wall 1 & 4 $\curvearrowright$ $M_n \approx 2525^{1K} + (4 \times 0.2 \times 67^{ksi}) \times 21' = 3{,}650^{1K}$

Wall 2+3 $\curvearrowright$ $M_n \approx 3636 + (1{,}126) = 4{,}762^{1K}$

1,4 $V_{MC} = \dfrac{3650}{58.3' \times 0.6} = 104.5^{K}$ Design shear, gross section = 74 psi

2,3 $V_{MC} = \dfrac{4{,}762}{58.3 \times 0.6} = 136.13^{K}$ Design shear, gross section 96 psi

Total base shear = 481.26^{K} 40% bldg wt.

Use shear formula (Matsumura, 1987)

$$V_u = \left\{ k_u k_p \left[\frac{0.76}{(h/d) + 0.7} + 0.012 \right] \sqrt{f'_m} + 0.18\, \gamma \cdot \delta \sqrt{\rho_h f_y \cdot f'_m} + 0.2\, \sigma_o \right\}$$

$\times 10^3 t \cdot j$ = Load KN

$k_u = 1.0$ fully grouted

length meters

All other units are MPa

f_m - gross area

$k_p = 1.16\, \rho_v^{0.3}$

$j = 7/8\, d$

$\delta = 0.6$ cantilever walls

$\gamma = 1.0$ fully grouted

$f'_m = 3000\,psi = 20.69\,MPa \quad \sqrt{f_m} = 4.55$

$h/d = \dfrac{60.7}{21.0} = 2.89$

$\rho_v = \dfrac{9 \times 0.20^{\square''}}{21 \times 5.62 \times 12^{"}} = .00127 \qquad 1.16\,\rho_v^{.3} = 0.16$

$k_u k_p \left[\dfrac{0.76}{2.89 + 0.7} + 0.012 \right] 4.55 = 0.16\,MPa$ 23.8 psi

@1&4 $0.2\,\sigma_o = \dfrac{82.6^{K} \times 21'}{5.62'' \times 12'' \times 21'} \times 0.2 = .2 \times 0.12^{ksi}$ 24.5 psi

@3&4 $0.2\,\sigma_o = \dfrac{13.34}{5.62 \times 12} \times 0.2 =$ 39.4 psi

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Walls 1 & 4 Design shear, gross = 74 psi

$\frac{gross}{j}$ = 84.57 psi

contribution of horizontal reinforcement required

= 84.57 − 23.8 − 24.5 = 36.27 psi or 0.25 MPa

Walls 2 & 3

$\frac{96\,psi}{0.875}$ − 23.8 − 39.4 = 46.51 psi or 0.32 MPa

$0.18 \times 1.0 \times 0.6 \sqrt{\rho_h f_y \cdot f'_m} = 0.25$ or 0.32 MPa

Line 1 & 4 $\rho_h = \left(\frac{0.25}{0.18 \times 1.0 \times 0.6}\right)^2 / f'_m \times f_y$ = 6.71×10^{-4} = 0.05 $^{\square''}/ft$

5.60×10^{-4} #4's

$f_y = 56{,}000/145 = 386$ MPa $f'_m = 20.69$ MPa Use #3 @ 24"

#3 bar

$A_s = 0.055$ $^{\square''}/ft.$

#4 @ 5', excessive

Line 2 & 3 $\rho_h = \left(\frac{0.32}{0.18 \times 1.0 \times 0.6}\right)^2 / f'_m \times f_y$ = 1.10×10^{-3} #3's = 0.07

9.2×10^{-4} #4's = 0.06

#3 @ 16" 0.08 $^{\square''}/ft$

#4 @ 2'-8" 0.07 $^{\square''}/ft$

Reinforcement could be reduced in each story level except minimum reinf used of 0.07 used #4 @ 2'-8"

2 bars in each story level

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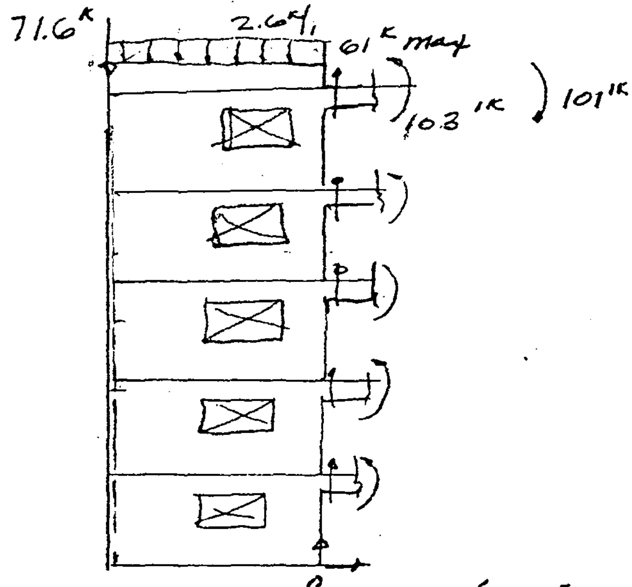
ITEM Lateral Load Design - Test wall Pier 1		JOB NO. 85-1007



effective flange width = 0.2h

$0.2 \times 43.35' = 8.67'$

2 additional #4's

+ Load of wall & applied load.

Load Line 1 $8.26^k/_1 \times 8.67 = 71.6^k$

Load Line B $2.6^k/_1$

$$M_n = (71.6^k + 2 \times 0.2 \times 67^{ksi}) \times 13.8 = 1358^{1k}$$

$$2.6 \times 13.83^2/2 = 248.7^{1k}$$

$$4 \times 0.2 \times 67^{ksi} \times 11.2' = 600.3^{1k}$$

$$103^{1k} \times 5 \text{ levels} \times 0.75 = 386^{1k}$$

$$2{,}593^{1k}$$

$$M_n = 61^k \times 5 \times 0.75 \times 13.83' = 3{,}164^{1k}$$

$$101^{1k} \times 5 \times 0.75 = 379^{1k}$$

$$2.6 \times 13.83^2/2 = 249$$

$$4 \times 0.2 \times 67^{ksi} \times 11.8' = 632^{1k}$$

$$4{,}424^{1k}$$

Assume min degradation in coupling bms.

Flexural yield in right pier, min yield in left pier

Axial load left pier ~ $229^k + 36^k + 27^k + 56^k = 348^k$

coupling shear, axial load, coupling minimum, 4 #4

Assume entire shear (170^k) is carried by left pier. Axial force is high (384^k) but flange is in compression face. Critical compressive strain unlikely.

$V \sim 170^k$ $\quad v_{gross} = 420 \text{ psi} = 2.9 \text{ Mpa}$

$\frac{V}{t j d} = 480 \text{ psi} = 3.31 \text{ Mpa}$

Try #4@8" $\rho_h = .00443 \quad 4.43 \times 10^{-3}$

$K_u = 1.0 \quad h/d = \frac{3.67}{6.0} = 0.61 \quad \sqrt{f'_m} = 4.55$

$\rho_v = \frac{0.8}{5.62 \times 72} = 1.98 \times 10^{-3} \quad K_p = 0.18$

$\delta = 0.9$, nearly restrained @ top

$K_u K_p \left[\frac{0.76}{0.61 + 0.7} + 0.012\right] 4.55 = 0.48$

$0.18 \times 1.0 \times 0.9 \sqrt{.00443 \times 462.0 \times 20.7} = 1.05$

$0.2 \times \frac{348 \times 10^3}{72 \times 5.62 \times 145} = \underline{1.19}$

2.72 Mpa < 3.31 req

#5@8" $\rho_h = 0.01$

$0.18 \times 1.0 \times 0.9 \sqrt{0.01 \times 462 \times 20.7} = 1.31 \text{ Mpa}$

$\Sigma v = 2.98 \text{ Mpa}$

Formula underestimates peak strength & assumptions of mobilization of coupling bm restraint when shear distortion is predominate is improbable. Use #4@8" for planning at this time.

Most probable $V \sim 100^k$ $\frac{V}{t j d} = 282 \text{ psi} \quad 1.95 \text{ MPa}$

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DATE 16 Sept 1987

SUBJECT: Catagory 9.1 - Research Building

ITEM Lateral Load design - Test Wall-Pier 1

JOB NO. ES-1007

$P \sim 98^{k} + 360'' - 229^{k} + 56^{k} = -39^{k}$

Not rational

flange effect

more probable is 114^{k} axial

$V = 90^{k}$ $\quad \dfrac{V}{tjd} = 457\ psi$ $\quad 3.16\ MPa$

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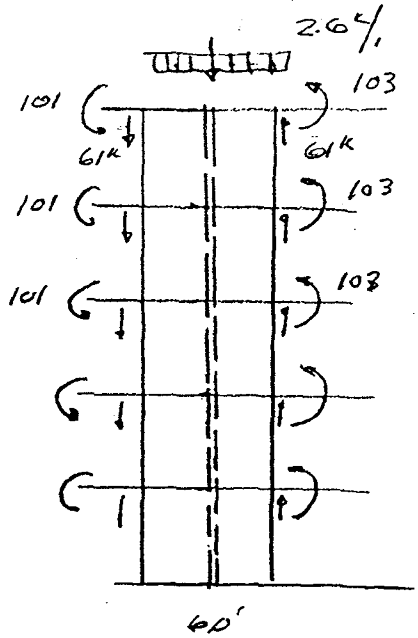
DATE 16 Sept 1987

JOB NO. 85-1007

SUBJECT: Category 9.1 – Research Building

ITEM Lateral Load Design – Pier 2 ½



Load line 2 = 13.3 k/' × 6.67 = 115.3 k

$M_n \sim$ 3 × 0.2 × 6.67 × 4.5' = 181 'k

2 × 0.2 × 6.7 × 3.0' = 80 'k

115.3 k × 3.0' = 346 'k

101 × 5 × 0.75 = 379 'k

103 × 5 × 0.75 = 386 'k

61 k × 5 × 0.75 × 6' = 1,373 'k

2,745 'k

M_n = 2,745 'k

P_{axial} = 115 k + 2.6 k/' × 6.0 (15.6 k) + 2 × 0.2 × 67 (27 k) = 157 k
reinf in crosswall

w/ minimum effects of coupling bms V = 53 k

$\frac{V}{b_j d}$ = 0.15 ksi = 1.03 MPa

K_4 = 1.0 h/d = $\frac{7.33}{6.0}$ = 1.22 $\sqrt{f'_m}$ = 4.55

minimal restraint K_u = 0.6 $K_p = \left(\frac{5 \times 0.2}{5.62 \times 72}\right)^{0.3} \times 116 = 0.19$

$1.0 \times 0.19 \left(\frac{0.76}{1.22 + 0.7} + 0.012\right) 4.55$ = 0.35 MPa

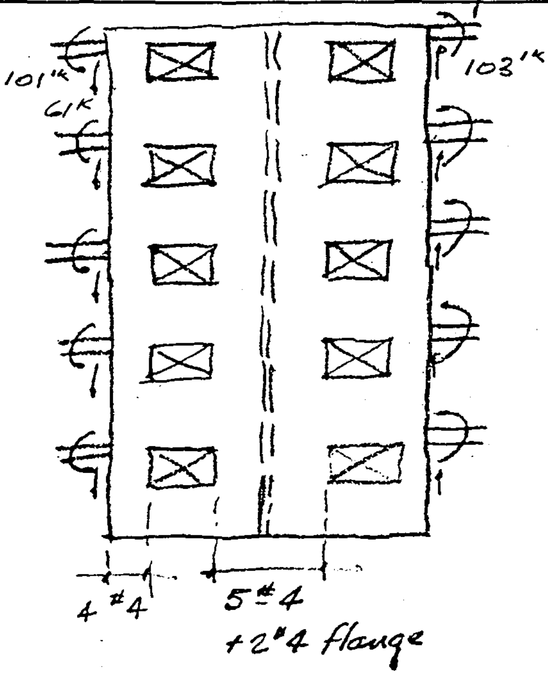
$0.2 \left(\frac{157^k \times 10^3}{562 \times 72 \times 145}\right)$ = 0.54 MPa

$0.18 \times 1.0 \times 0.6 \sqrt{.00443 \times 462 \times 20.7}$ = 0.70 MPa (#4 @ 8")

1.59 MPa

Use #4 @ 16" 0.5 MPa

Σ = 1.39 MPa adequate



Load Line 3 = 115.3k

$M_n \sim$

$2.6^{k/'} \frac{27.3^2}{2}$	= 969'k
$61^k \times 5 \times 0.75 \times 27.3'$	6,245'k
$101'^k \times 5 \times 0.75$	379'k
$103'^k \times 5 \times 0.75$	386'k
$115.3^k \times 13.8$	1,591'k
$4 \times 0.2 \times 67 \times 25.6$	1,372'k
$7 \times 0.2 \times 67 \times 13.8$	1,294
	12,236'k

$M_n = 12,236'^k$

Axial load on wall $\sim 115.3^k + 71^k + \overset{147^k}{11 \times 0.2 \times 67} + \overset{9^k}{\frac{1,020'^k \times .75}{3 \times 27.3'}} = 334^k$

coupling effects cancel
except M. M effect is equiv. to added reinf.

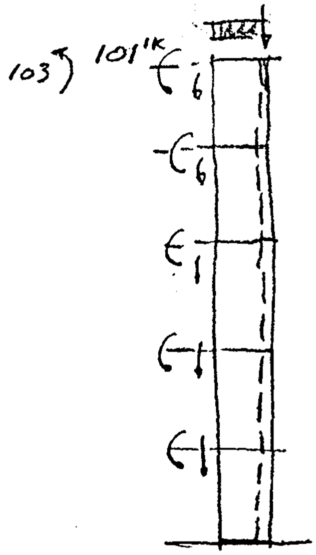
Assume Left pier is in tension
middle & right pier share axial & shear

$P_2 = P_3 = 334^k/2$ $\quad V_2 = \frac{10}{13.3} \times 300^k = 226^k$

$V_3 = \frac{3.3}{13.3} \times 300^k = 75^k$

$\frac{V}{bjd} = 383\ psi = 2.64\ MPa$

By reference to sht. L4a Use #4@8"
in piers @ window section $V_a \sim 2.64\ MPa$
but yielding will transfer shear distortion
to Pier 1, 2 & 4



Load Line 4 71.6^K

Load Line B 2.6^K/'

Shear coupling bm 61^K

M tension Top 101^1K

M tension bottom 103^1K

M_n = 101 x 5 x 0.75 — 379^1K

61^K x 5 x 0.75 x 3.0 — 686^1K

$2.6 \times \frac{3.0^2}{2}$ — 12^1K

3 x 0.2 x 67 x 1.8' — 72^1K

1,149^1K

M_n = 103 x 5 x .75 = 386^1K

71.6 x 3.0 = 215^1K

3 x 0.2 x 67 x 3' = 121^1K

$2.6 \times 3.0^2/2$ = 12^1K

733^1K

V probable = 30^K

v = 0.15^ksi = 1.05 MPa

Use #4 @ 16" horz.

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Base pier rigidities on flexural capacity

Pier	M_n	Stiffness	V	M_n	Stiffness	V
1	2593^{1K}	0.14	100^K	4424^{1K}	0.22	170^K
2	2745^{1K}	0.15	108^K	2745^{1K}	0.14	106^K
3	$12,236^{1K}$	0.65	467^K	$12,236^{1K}$	0.61	471^K
4	$1,149^{1K}$	0.06	43^K	733^{1K}	0.04	31^K
	$18,726^{1K}$			$20,138^{1K}$		

$$V = \frac{18,726^{1K}}{43.4 \times 0.6} = 719^K \qquad \frac{20,138^{1K}}{43.4 \times 0.6} = 773.35^K$$

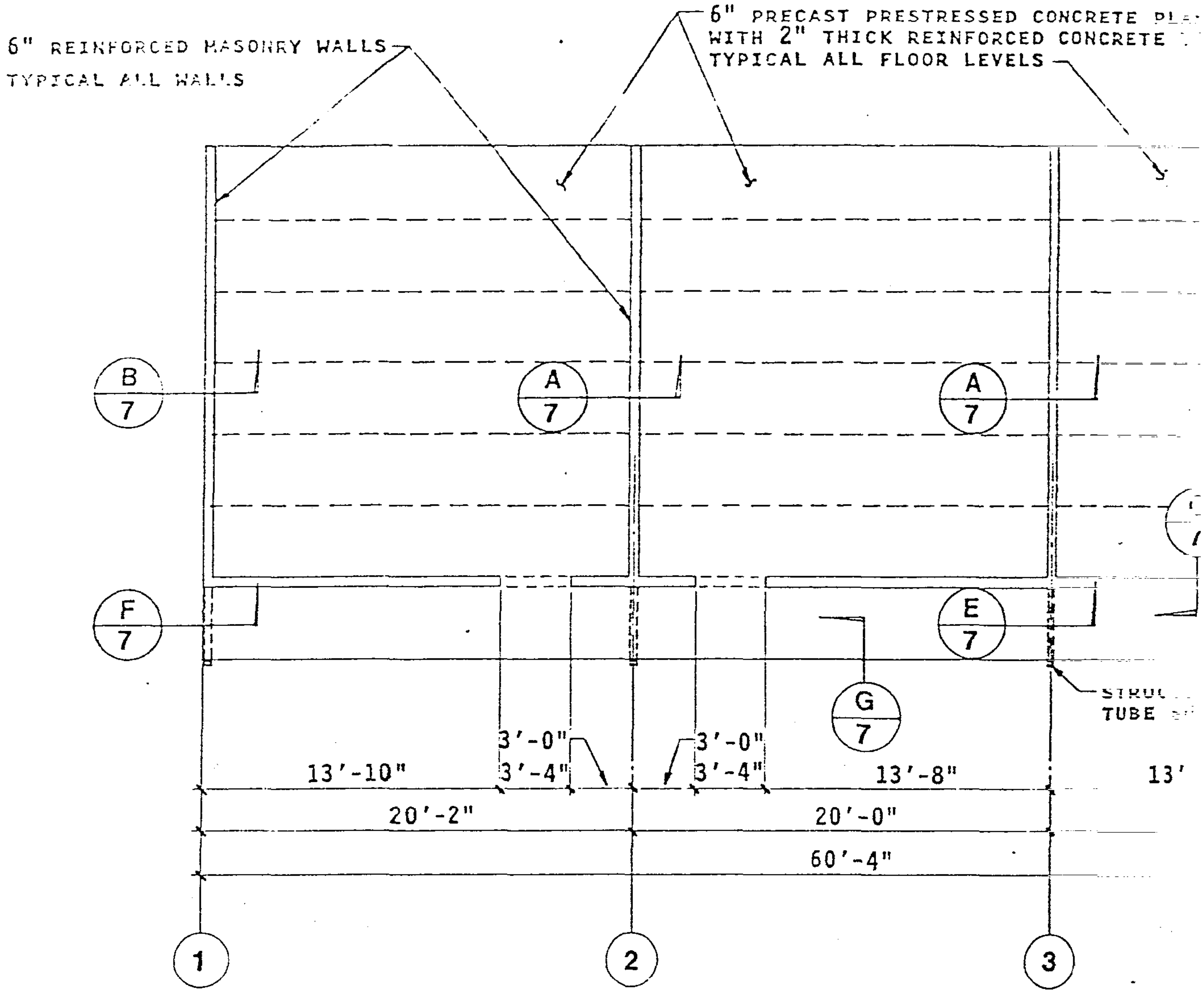
If coupling bms deteriorate & shear reduces to 1/3 of $M_{uA} + M_{uB}/\ell$

Pier	M_n	Stiffness		M_n	Stiffness	
1	2336^{1K}	0.20	90^K	2062^{1K}	0.18	79^K
2	$1,320^{1K}$	0.11	50^K	1320^{1K}	0.12	53^K
3	$7,563^{1K}$	0.65	291^K	$7,563^{1K}$	0.66	290^K
4	439^{1K}	0.04	18^K	477^{1K}	0.04	18^K
	$11,658^{1K}$			$11,422^{1K}$		

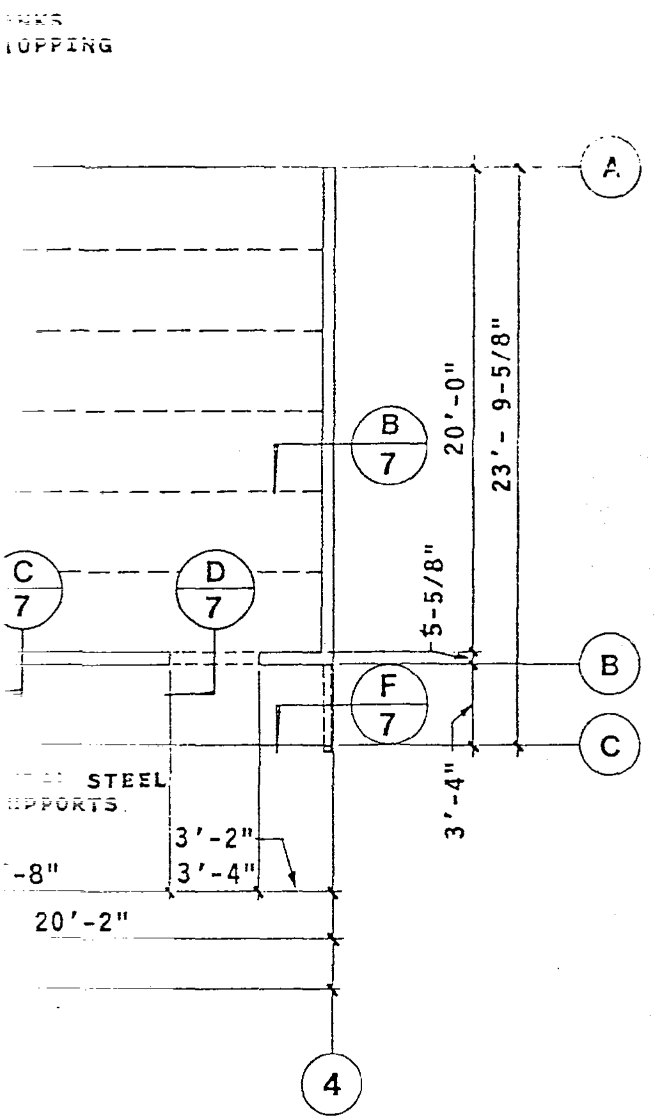
$V = 448^K$ $\qquad$ $V = 439^K$

SECTION 8

APPENDIX B



PLAN - LEVELS 2 THROUGH 6



CONSTRUCTION NOTES

ALL MASONRY UNITS SHALL BE 6 x 8 x 16 NOMINAL CONCRETE BLOCK CONFORMING TO ASTM C90, GRADE N. ALL UNITS SHALL BE BOND BEAM TYPE UNITS EXCEPT WHERE NOTED AS OPEN END BOND BEAM UNITS.

MORTAR SHALL BE TYPE S AND PROPORTIONED BY VOLUME.

GROUT SHALL BE COARSE GROUT. AN ADMIXTURE TO COMPENSATE FOR SHRINKAGE SHALL BE USED.

ALL GROUT SHALL BE CONSOLIDATED BY MECHANICAL VIBRATION.

CLEANOUTS SHALL BE PROVIDED AT EACH VERTICAL BAR FOR INSPECTION AND CLEANING OF THE CONSTRUCTION JOINT. THE VERTICAL BAR SHALL BE WIRED TO THE DOWEL BEFORE CLOSING THE CLEANOUT BY A FORM.

CONSTRUCTION JOINTS AT FLOOR LEVELS SHALL BE MADE BY EXPOSING THE CONCRETE AGGREGATE TO 1/4 INCH AMPLITUDE. CUTOUTS IDENTICAL TO BOND BEAM BLOCKS SHALL BE MADE IN THE BOTTOM OF THE CROSSWEBS OF THE STARTER BLOCK. A WATER OR AIR STREAM SHALL BE USED TO CLEAN THE CONSTRUCTION JOINT OF MORTAR AND DEBRIS BEFORE CLOSING THE CLEANOUTS.

CONSTRUCTION JOINTS IN THE MASONRY WALLS SHALL BE MADE BY STOPPING THE GROUT POUR 1-1/2 INCH BELOW THE CROSSWEB IN THE TOP UNIT.

ALL REINFORCEMENT #4 AND LARGER SHALL CONFORM TO ASTM A615, GRADE 60. NUMBER 3 BARS SHALL CONFORM TO ASTM A615, GRADE 40. ALL REINFORCEMENT SHALL BE IDENTIFIED BY A HEAT NUMBER AND TESTED FOR YIELD AND ULTIMATE STRESS.

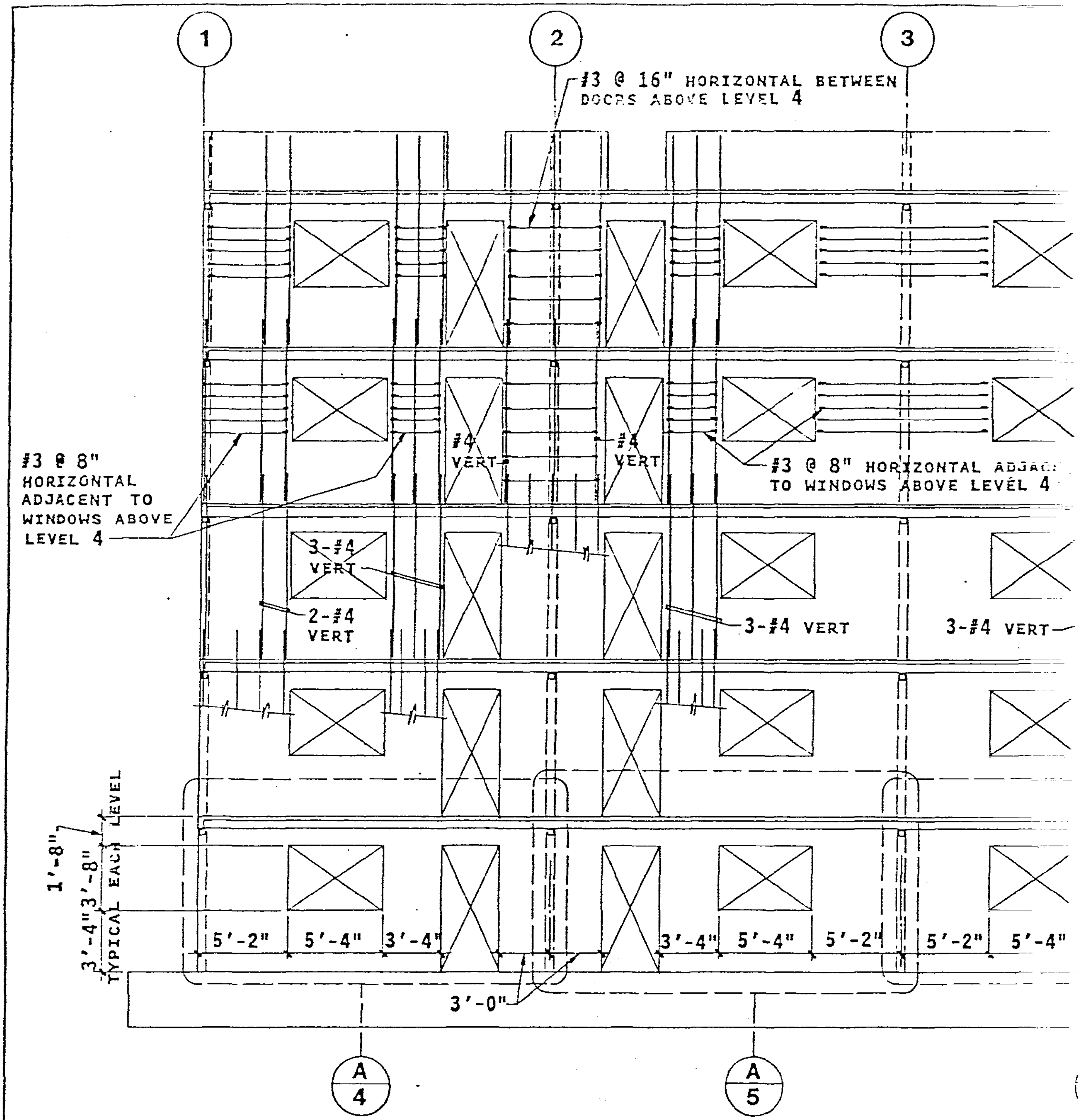
PRECAST CONCRETE PLANKS SHALL BE POST-TENSIONED WITH AN EQUIVALENT OF 4, 3/8 INCH, 270 KIP, STRANDS PER 3'-4" WIDE UNIT. SEE SECTION 2.4 FOR THE SEQUENCE OF CONSTRUCTION.

PRECAST PLANKS SHALL BE TOPPED WITH A 2 INCH THICK CAST-IN-PLACE CONCRETE SLAB. THE CONCRETE SLAB SHALL BE REINFORCED WITH #4 BARS AT 18 INCHES O.C. PERPENDICULAR TO THE PLANKS. REINFORCEMENT PARALLEL TO THE PLANKS SHALL BE PROVIDED AT THE BEARING WALLS.

REINFORCEMENT SHALL BE SPLICED BY 40 DIAMETER LAP SPLICES AT THE LOCATIONS NOTED ON FIGURES 2 THROUGH 7. VERTICAL REINFORCEMENT AT THE ENDS OF WALLS AND EDGES OF DOORS SHALL NOT BE SPLICED AT LEVEL 1.

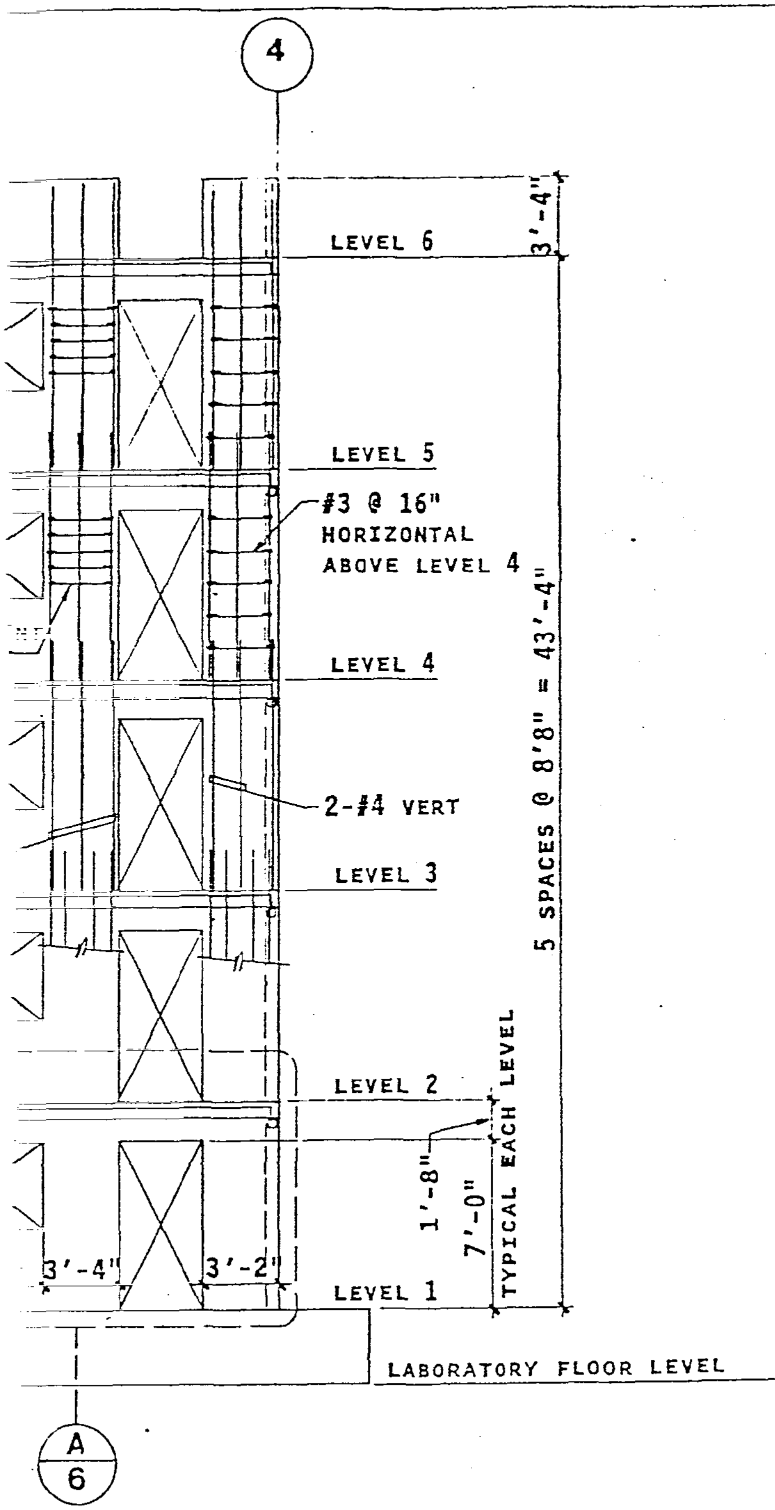
TYPICAL PLAN - LEVELS 2 THROUGH 6
AND CONSTRUCTION NOTES

FIGURE 1



ELEVATION - TEST WALL - LINE (B)

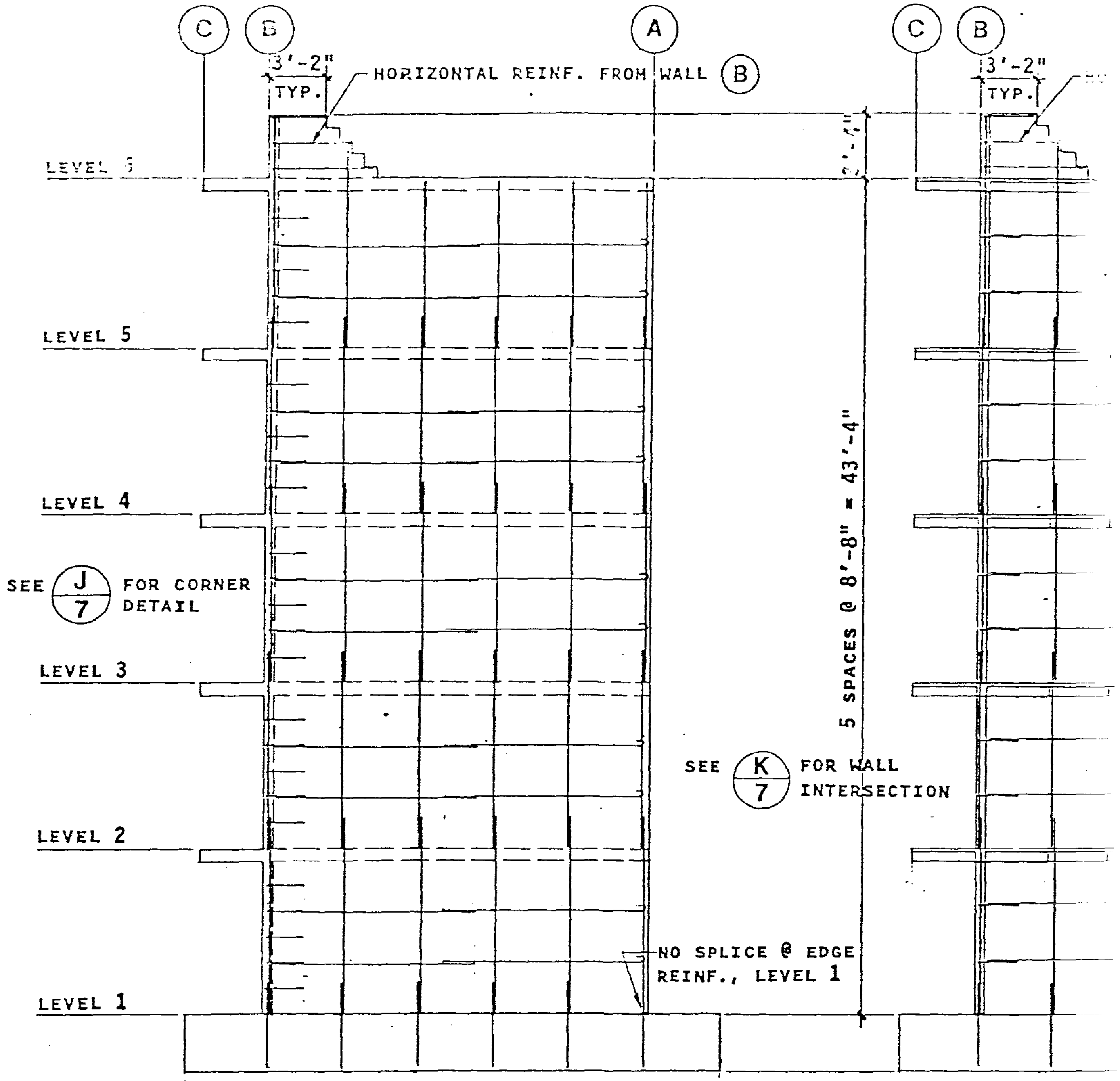
SEE FIGURES 4 THROUGH 6 FOR TYPICAL WALL REINFORCEMENT SIZE AND PLACEMENT. REINFORCEMENT SHOWN IN FIRST LEVEL IS TYPICAL EXCEPT AS NOTED ON THIS ELEVATION FOR LEVELS (3) AND ABOVE. IF REINFORCEMENT SIZE AND PLACEMENT ARE NOT INDICATED HEREIN, USE REINFORCEMENT SIZE AND PLACEMENT AS SHOWN AT FIRST LEVEL BY FIGURES 4, 5, AND 6.



4
3'-4"
LEVEL 6
LEVEL 5
#3 @ 16"
HORIZONTAL
ABOVE LEVEL 4
LEVEL 4
2-#4 VERT
LEVEL 3
5 SPACES @ 8'8" = 43'-4"
LEVEL 2
1'-8"
7'-0"
TYPICAL EACH LEVEL
3'-4"
3'-2"
LEVEL 1
LABORATORY FLOOR LEVEL
A
6

47

FIGURE 2



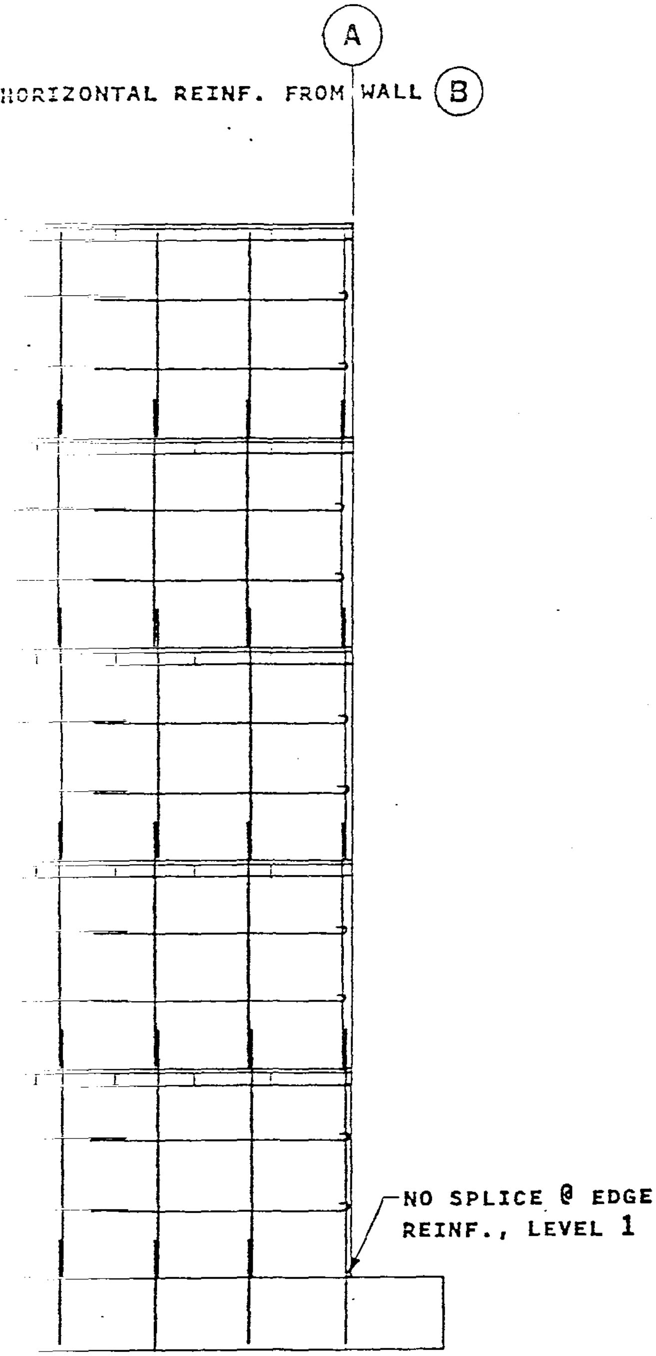
ELEVATION - WALL LINE - 4

WALL LINE 1 OPPOSITE HAND

ELEVATION -

NOTES: ALL VERTICAL REINFORCEMENT IS #4 @ 48" o

ALL HORIZONTAL REINFORCEMENT IS #4 @ 32"

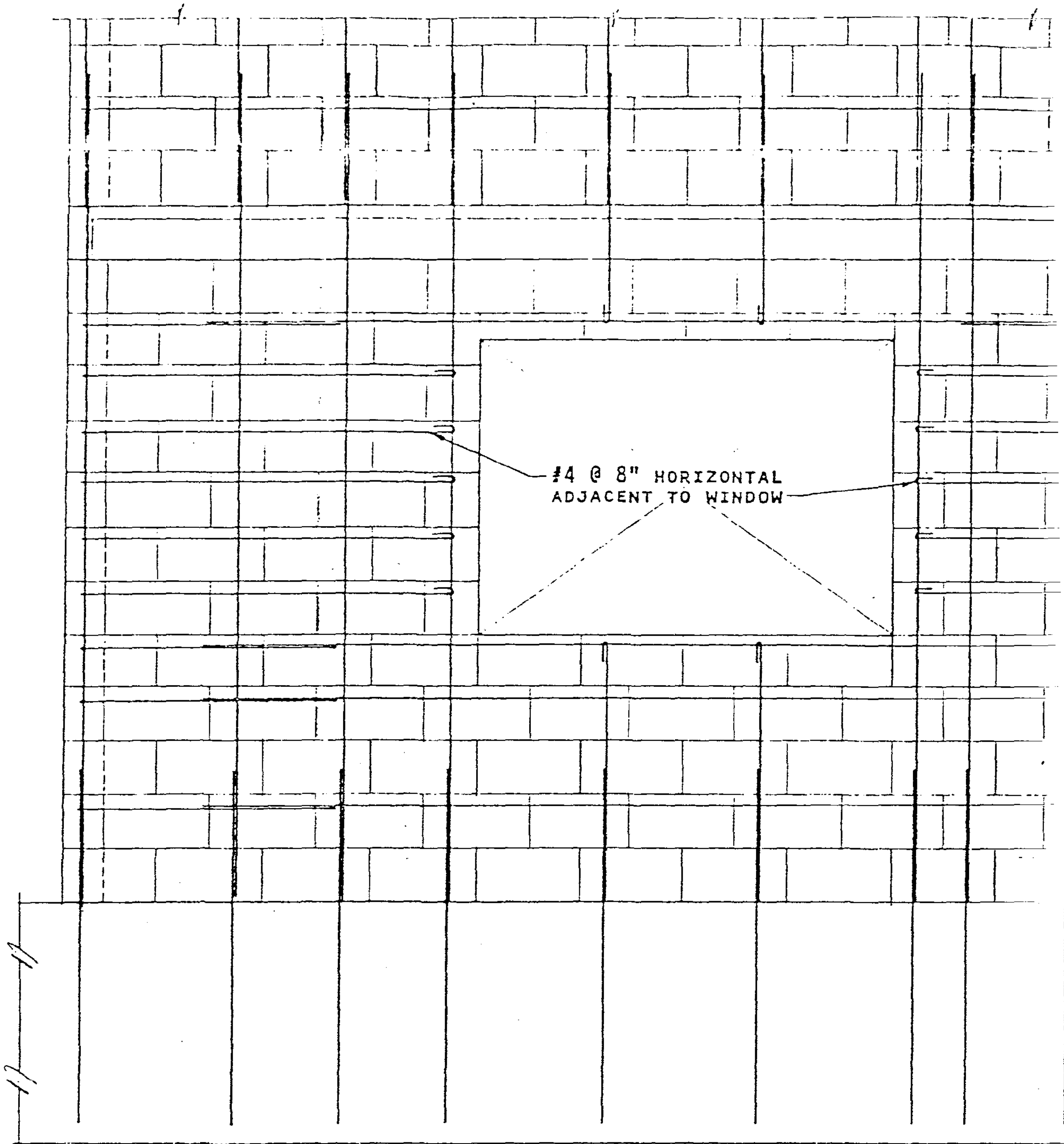


- WALL LINES - 2 3

O.C.

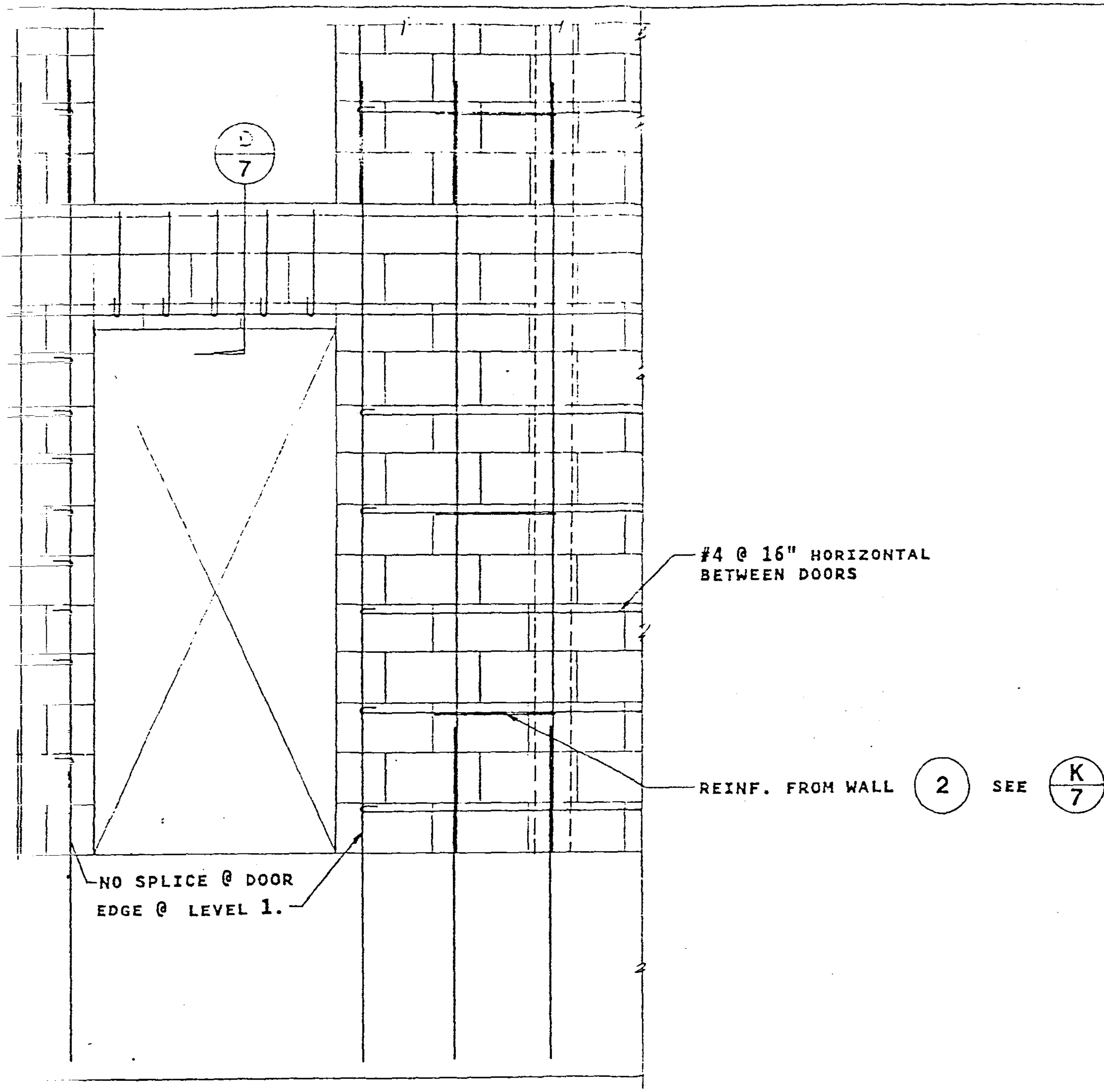
O.C.

FIGURE 3



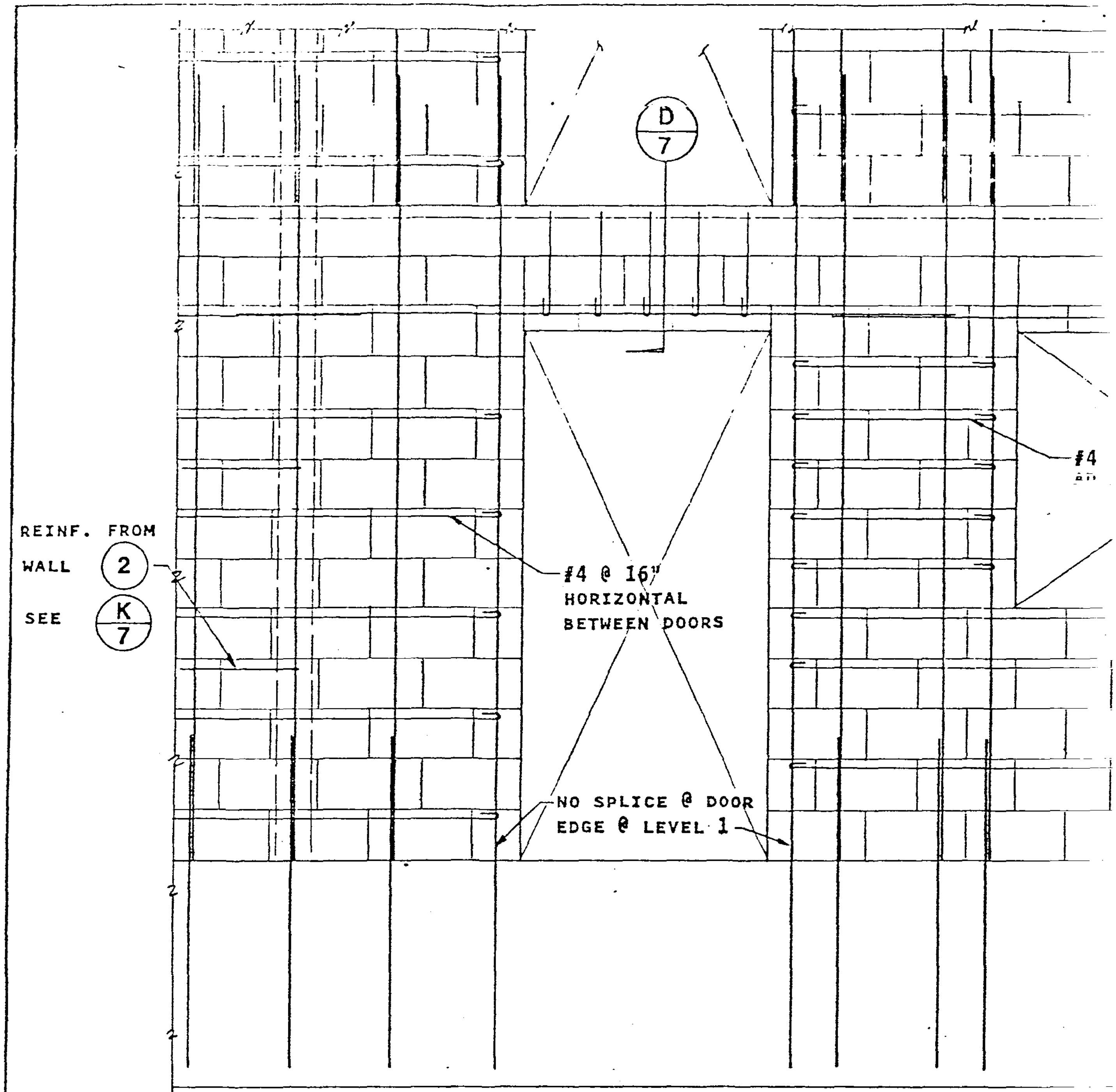
PARTIAL ELEVATION A/4

NOTES: ALL VERTICAL AND HORIZONTAL REINFORCEMENT IS #4 BARS. REINFORCEMENT NOTED IS TYPICAL LEVELS UNLESS SPECIFICALLY NOTED OTHERWISE IN FIGURE 2.


D
7
#4 @ 16" HORIZONTAL
BETWEEN DOORS
REINF. FROM WALL
2
SEE
K
7
NO SPLICE @ DOOR
EDGE @ LEVEL 1.

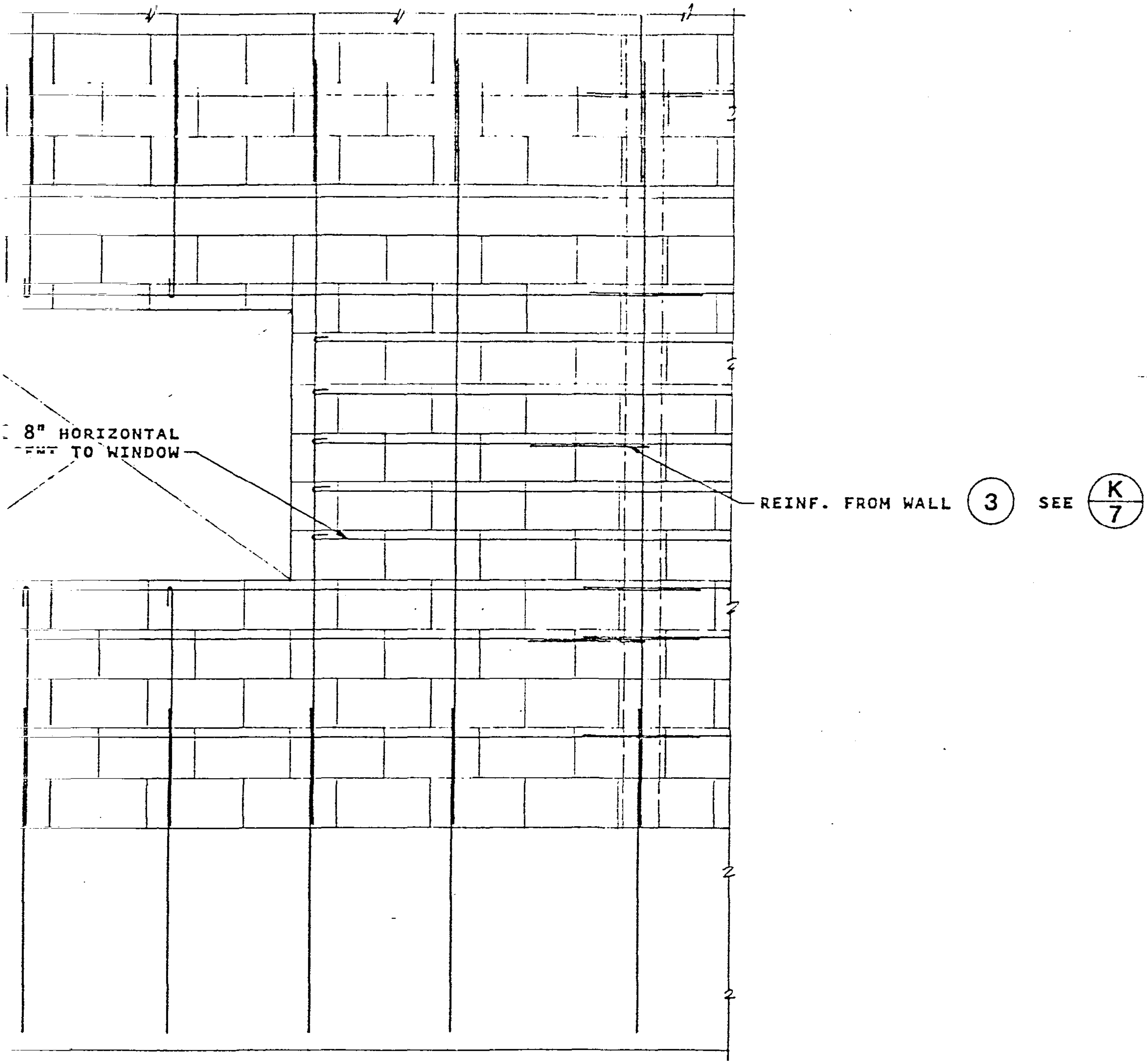
ALL

FIGURE 4



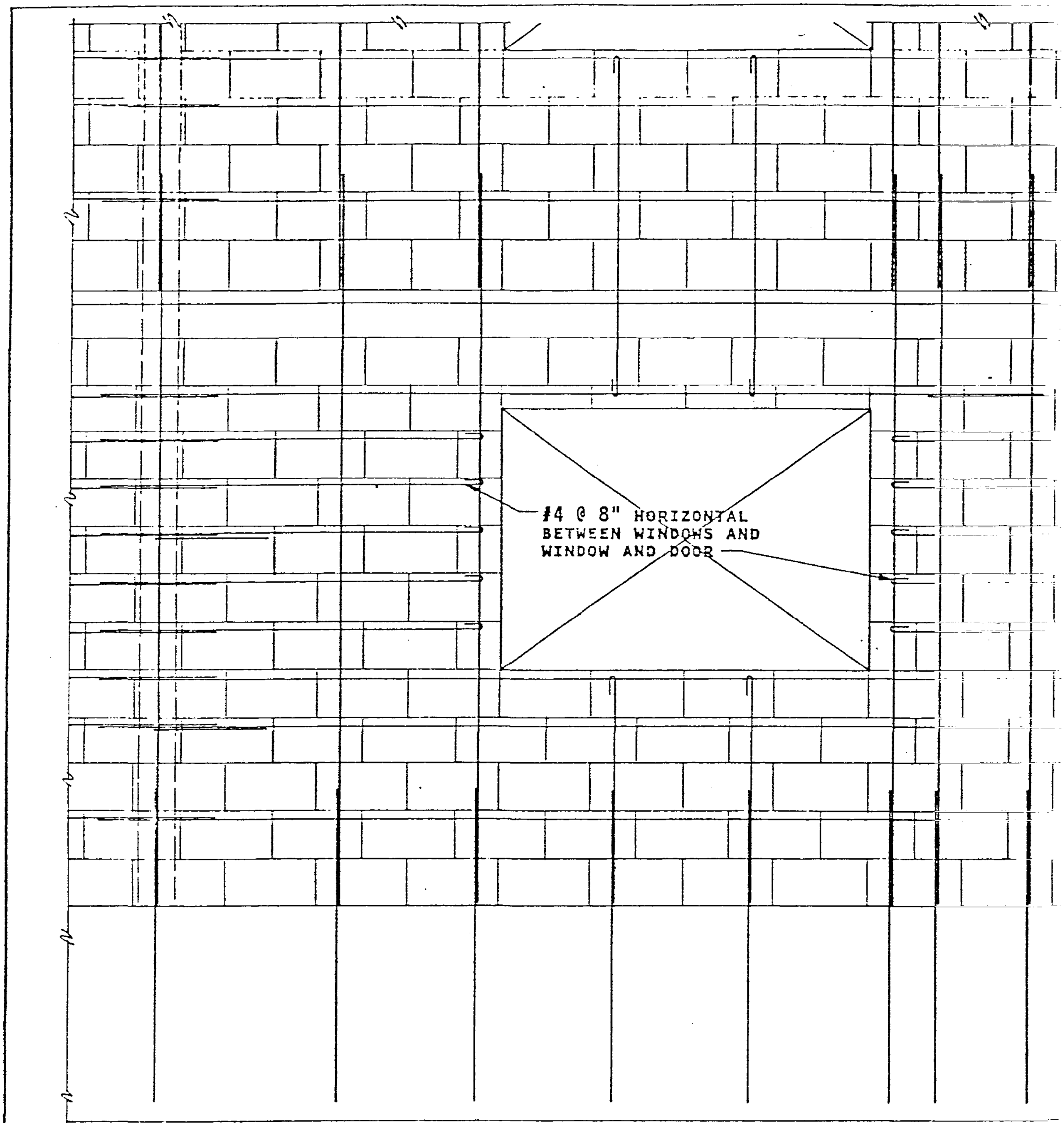
PARTIAL ELEVATION A/5

NOTES: ALL VERTICAL AND HORIZONTAL REINFORCEMENT IS #4 BARS. REINFORCEMENT NOTED IS TYP... LEVELS UNLESS SPECIFICALLY OTHERWISE IN FIGURE 2.



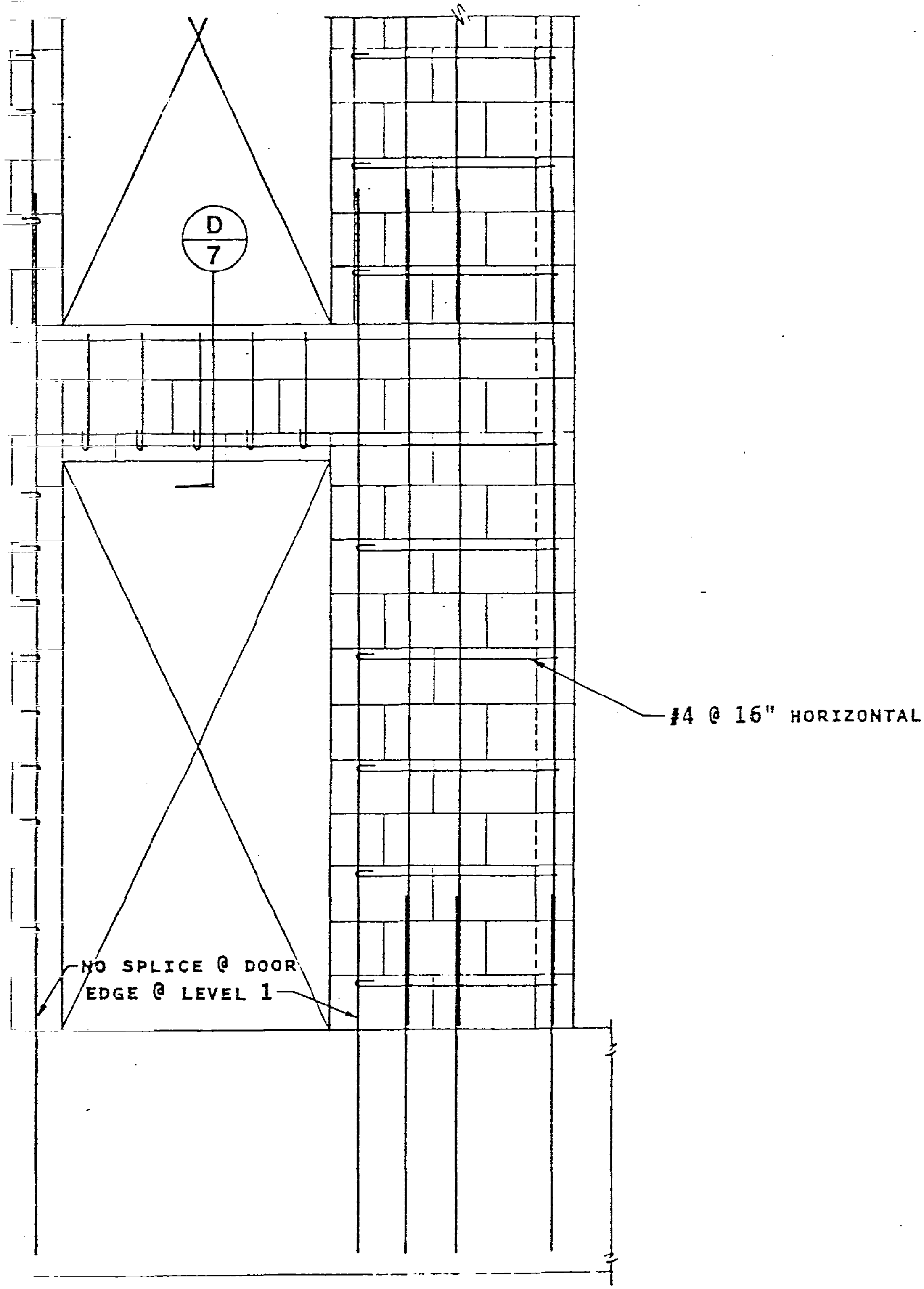
AT ALL

FIGURE 5



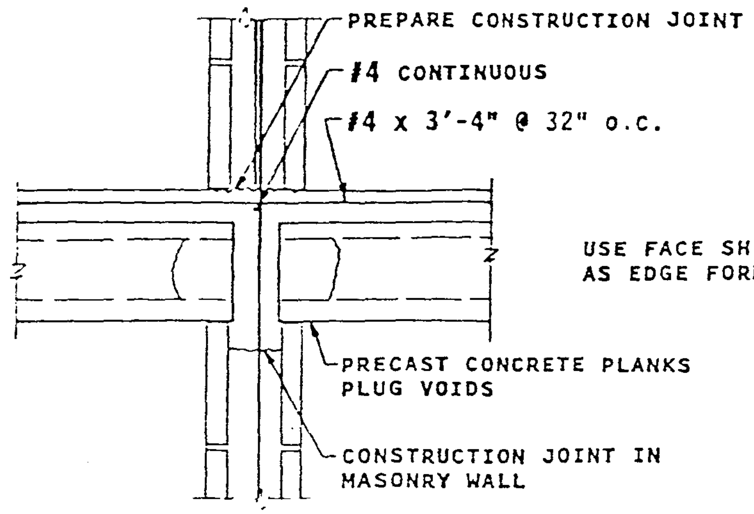
PARTIAL ELEVATION A/6

NOTES: ALL VERTICAL AND HORIZONTAL REINFORCEMENT IS #4 BARS. REINFORCEMENT NOTED IS TYPICAL AT LEVELS UNLESS SPECIFICALLY NOTED OTHERWISE IN FIGURE 2.

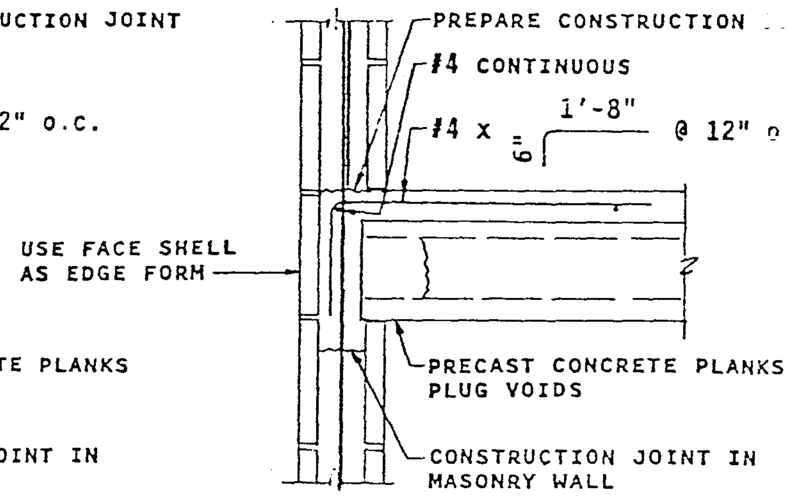


ALL

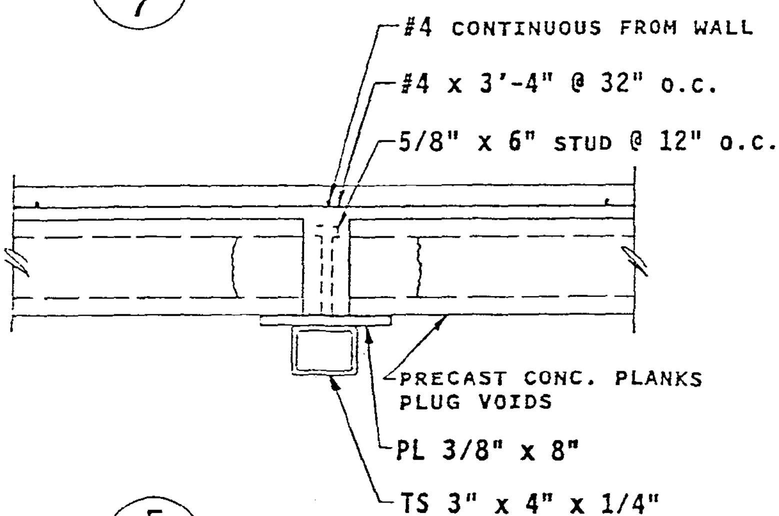
FIGURE 6



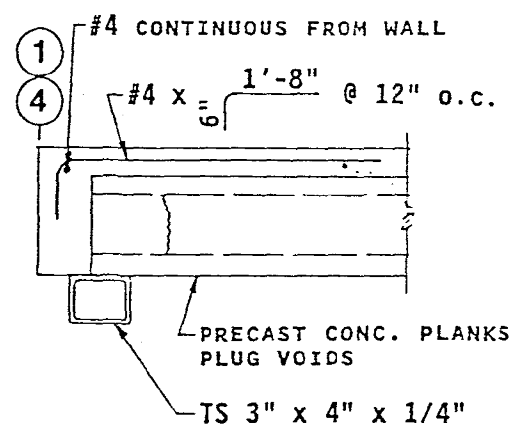
A
7



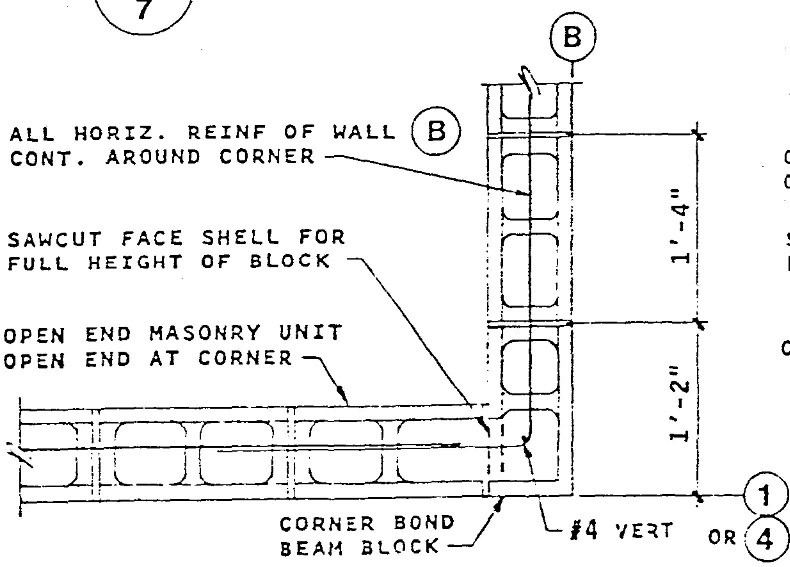
B
7

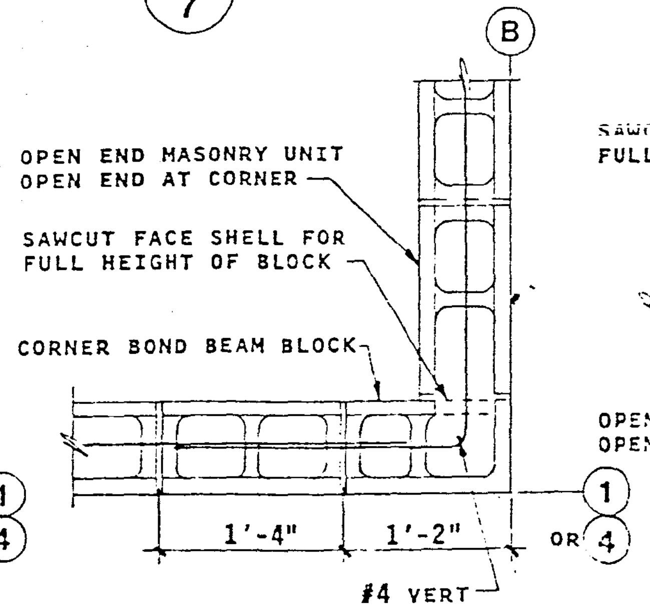


E
7



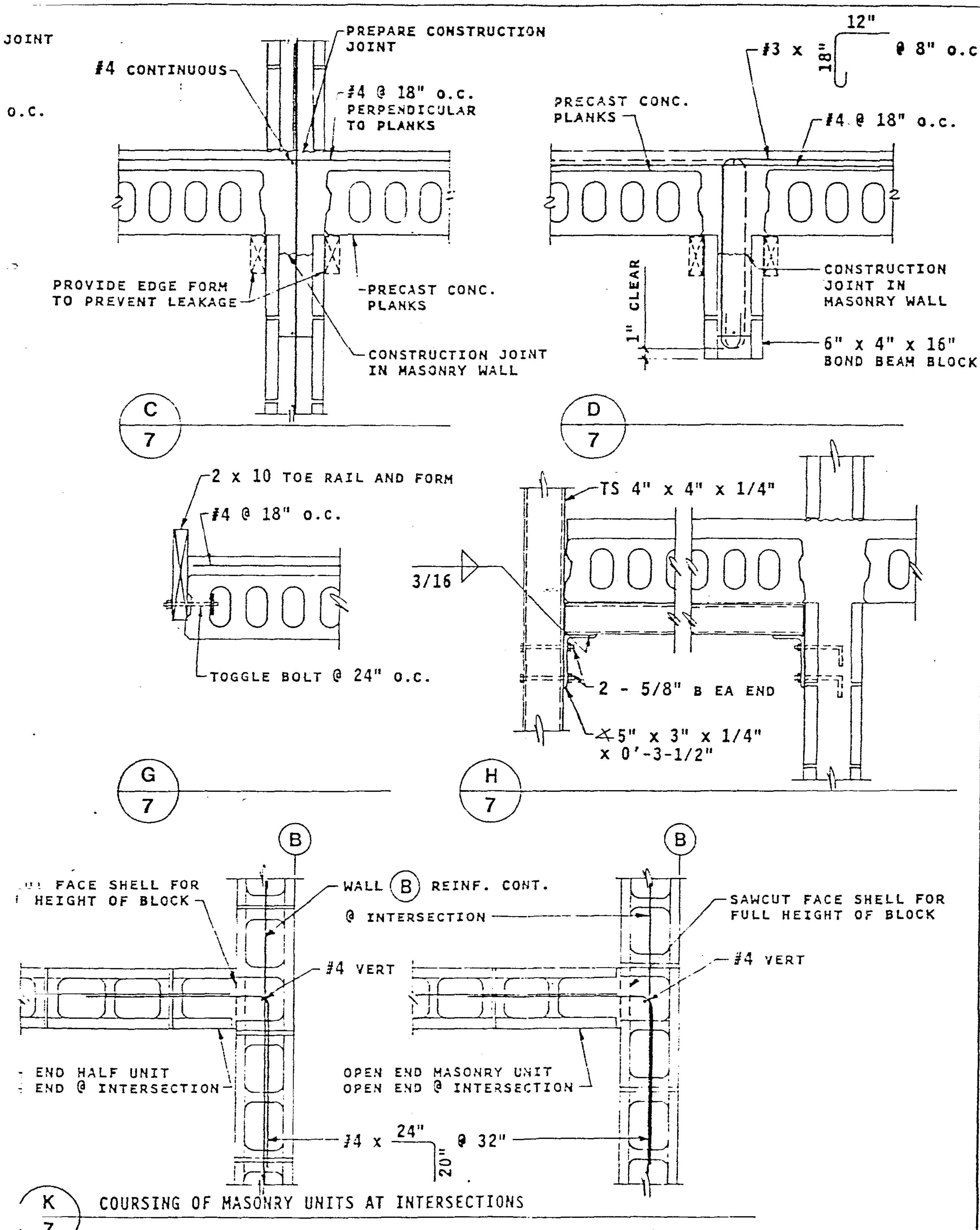
F
7





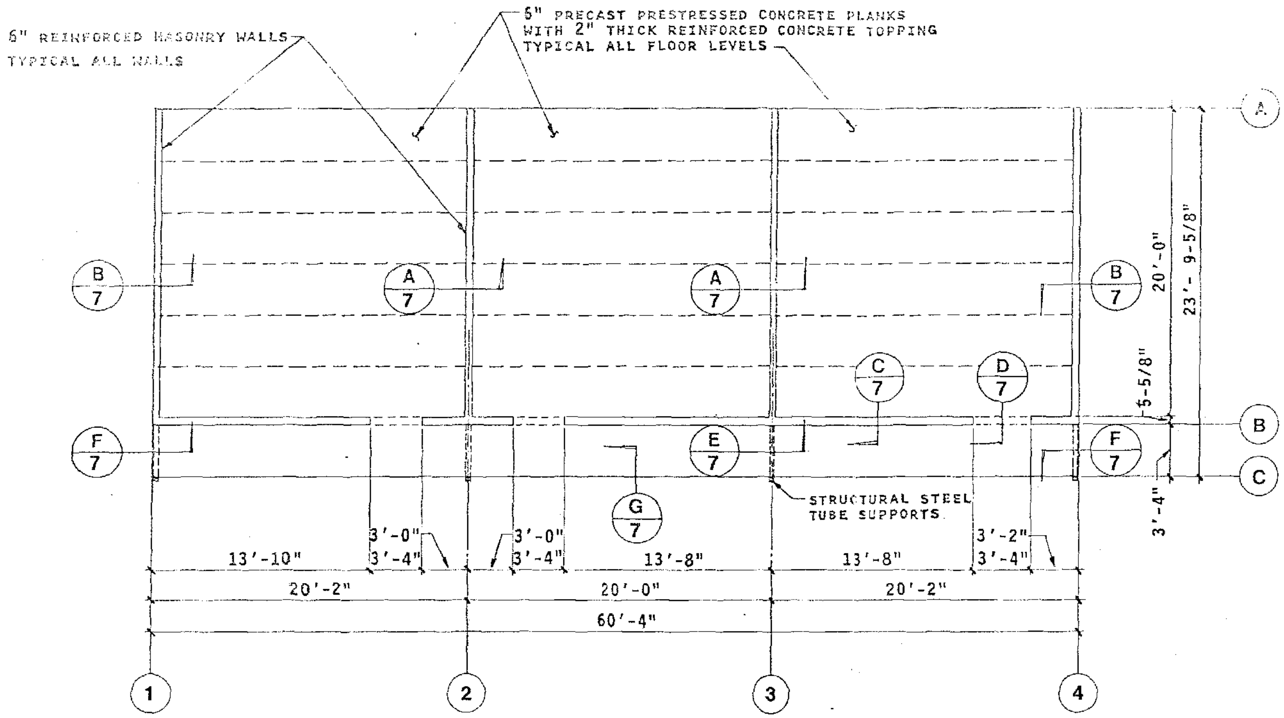
J
7

COURSING OF MASONRY UNITS AT CORNER



JOINT
O.C.
#4 CONTINUOUS
PREPARE CONSTRUCTION JOINT
#4 @ 18" o.c. PERPENDICULAR TO PLANKS
PROVIDE EDGE FORM TO PREVENT LEAKAGE
PRECAST CONC. PLANKS
CONSTRUCTION JOINT IN MASONRY WALL
C
7
12"
#3 x 18" @ 8" o.c.
PRECAST CONC. PLANKS
#4 @ 18" o.c.
CONSTRUCTION JOINT IN MASONRY WALL
1" CLEAR
6" x 4" x 16" BOND BEAM BLOCK
D
7
2 x 10 TOE RAIL AND FORM
#4 @ 18" o.c.
TOGGLE BOLT @ 24" o.c.
G
7
TS 4" x 4" x 1/4"
3/16
2 - 5/8" B EA END
5" x 3" x 1/4" x 0'-3-1/2"
H
7
B
FACE SHELL FOR HEIGHT OF BLOCK
WALL B REINF. CONT. @ INTERSECTION
#4 VERT
END HALF UNIT END @ INTERSECTION
OPEN END MASONRY UNIT OPEN END @ INTERSECTION
#4 x 24" 20" @ 32"
SAWCUT FACE SHELL FOR FULL HEIGHT OF BLOCK
#4 VERT
K
7
COURSING OF MASONRY UNITS AT INTERSECTIONS

FIGURE 7



PLAN - LEVELS 2 THROUGH 6

CONSTRUCTION NOTES

ALL MASONRY UNITS SHALL BE 6 x 8 x 16 NOMINAL CONCRETE BLOCK CONFORMING TO ASTM C90, GRADE N. ALL UNITS SHALL BE BOND BEAM TYPE UNITS EXCEPT WHERE NOTED AS OPEN END BOND BEAM UNITS.

MORTAR SHALL BE TYPE S AND PROPORTIONED BY VOLUME.

GROUT SHALL BE COARSE GROUT. AN ADMIXTURE TO COMPENSATE FOR SHRINKAGE SHALL BE USED.

ALL GROUT SHALL BE CONSOLIDATED BY MECHANICAL VIBRATION.

CLEANOUTS SHALL BE PROVIDED AT EACH VERTICAL BAR FOR INSPECTION AND CLEANING OF THE CONSTRUCTION JOINT. THE VERTICAL BAR SHALL BE WIRED TO THE DOWEL BEFORE CLOSING THE CLEANOUT BY A FORM.

CONSTRUCTION JOINTS AT FLOOR LEVELS SHALL BE MADE BY EXPOSING THE CONCRETE AGGREGATE TO 1/4 INCH AMPLITUDE. CUTOUTS IDENTICAL TO BOND BEAM BLOCKS SHALL BE MADE IN THE BOTTOM OF THE CROSSWEBS OF THE STARTER BLOCK. A WATER OR AIR STREAM SHALL BE USED TO CLEAN THE CONSTRUCTION JOINT OF MORTAR AND DEBRIS BEFORE CLOSING THE CLEANOUTS.

CONSTRUCTION JOINTS IN THE MASONRY WALLS SHALL BE MADE BY STOPPING THE GROUT POUR 1-1/2 INCH BELOW THE CROSSWEB IN THE TOP UNIT.

ALL REINFORCEMENT #4 AND LARGER SHALL CONFORM TO ASTM A615, GRADE 60. NUMBER 3 BARS SHALL CONFORM TO ASTM A615, GRADE 40. ALL REINFORCEMENT SHALL BE IDENTIFIED BY A HEAT NUMBER AND TESTED FOR YIELD AND ULTIMATE STRESS.

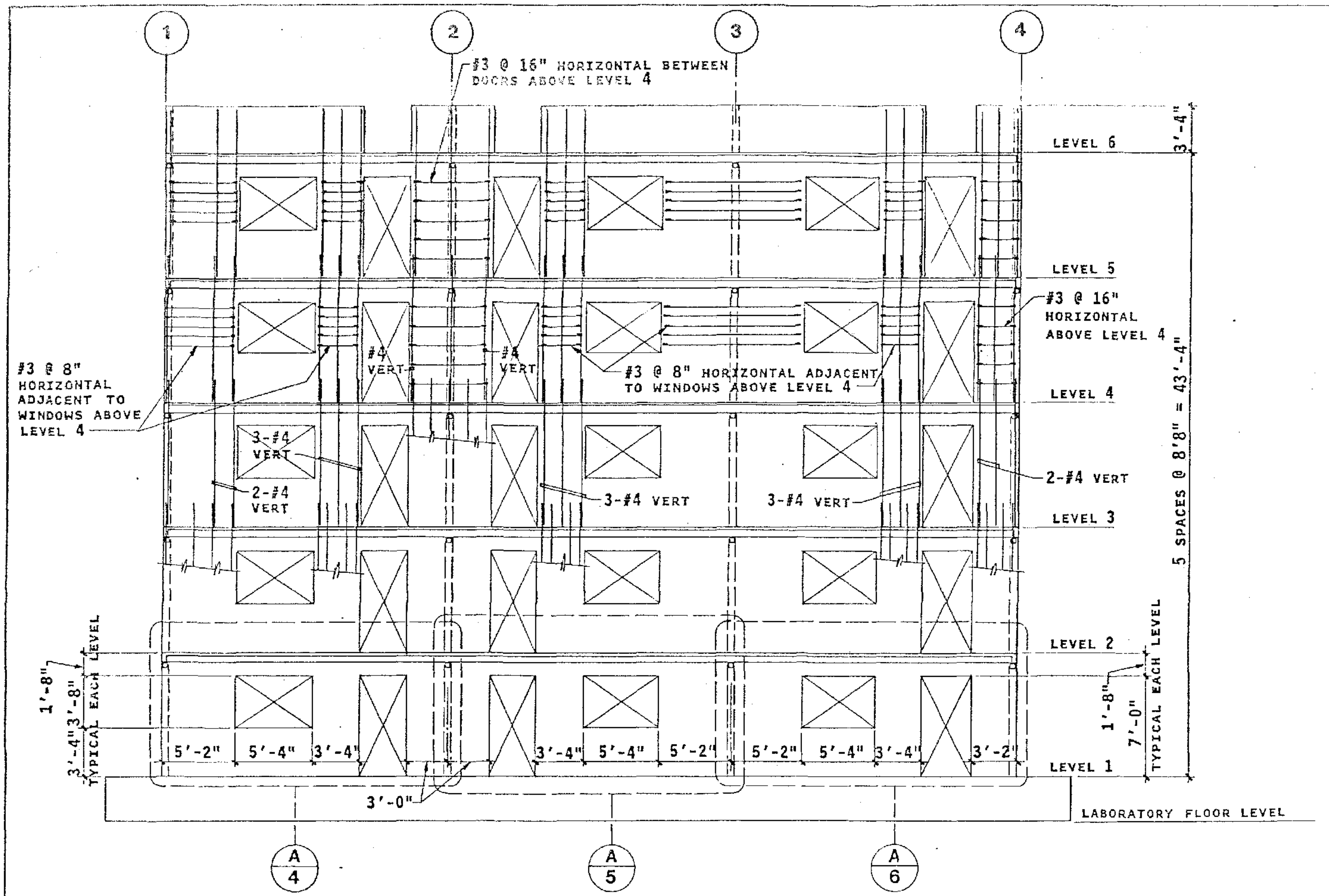
PRECAST CONCRETE PLANKS SHALL BE POST-TENSIONED WITH AN EQUIVALENT OF 4, 3/8 INCH, 270 KIP, STRANDS PER 3'-4" WIDE UNIT. SEE SECTION 2.4 FOR THE SEQUENCE OF CONSTRUCTION.

PRECAST PLANKS SHALL BE TOPPED WITH A 2 INCH THICK CAST-IN-PLACE CONCRETE SLAB. THE CONCRETE SLAB SHALL BE REINFORCED WITH #4 BARS AT 18 INCHES O.C. PERPENDICULAR TO THE PLANKS. REINFORCEMENT PARALLEL TO THE PLANKS SHALL BE PROVIDED AT THE BEARING WALLS.

REINFORCEMENT SHALL BE SPLICED BY 40 DIAMETER LAP SPLICES AT THE LOCATIONS NOTED ON FIGURES 2 THROUGH 7. VERTICAL REINFORCEMENT AT THE ENDS OF WALLS AND EDGES OF DOORS SHALL NOT BE SPLICED AT LEVEL 1.

TYPICAL PLAN - LEVELS 2 THROUGH 6
AND CONSTRUCTION NOTES

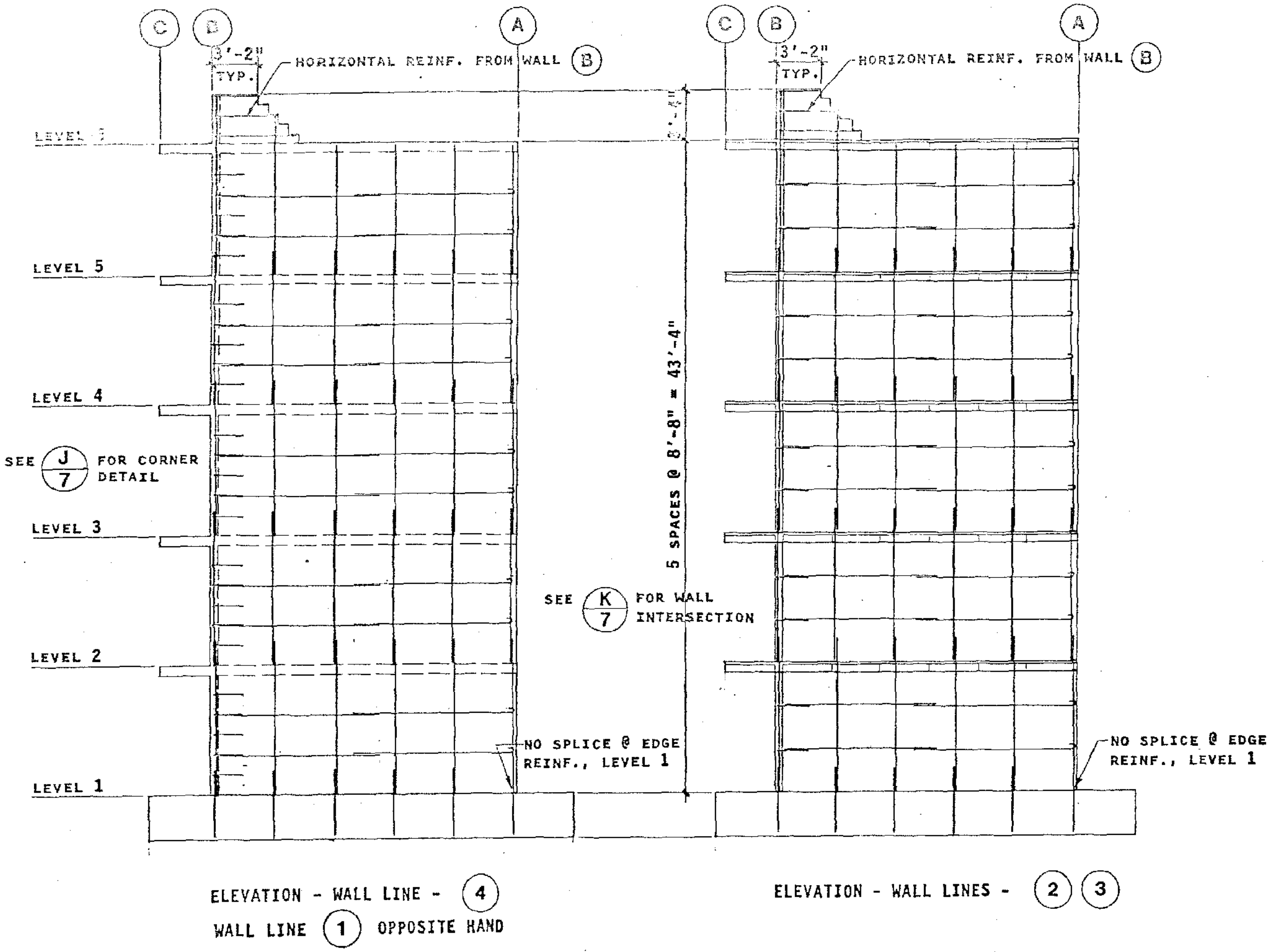
FIGURE 1



ELEVATION - TEST WALL - LINE (B)

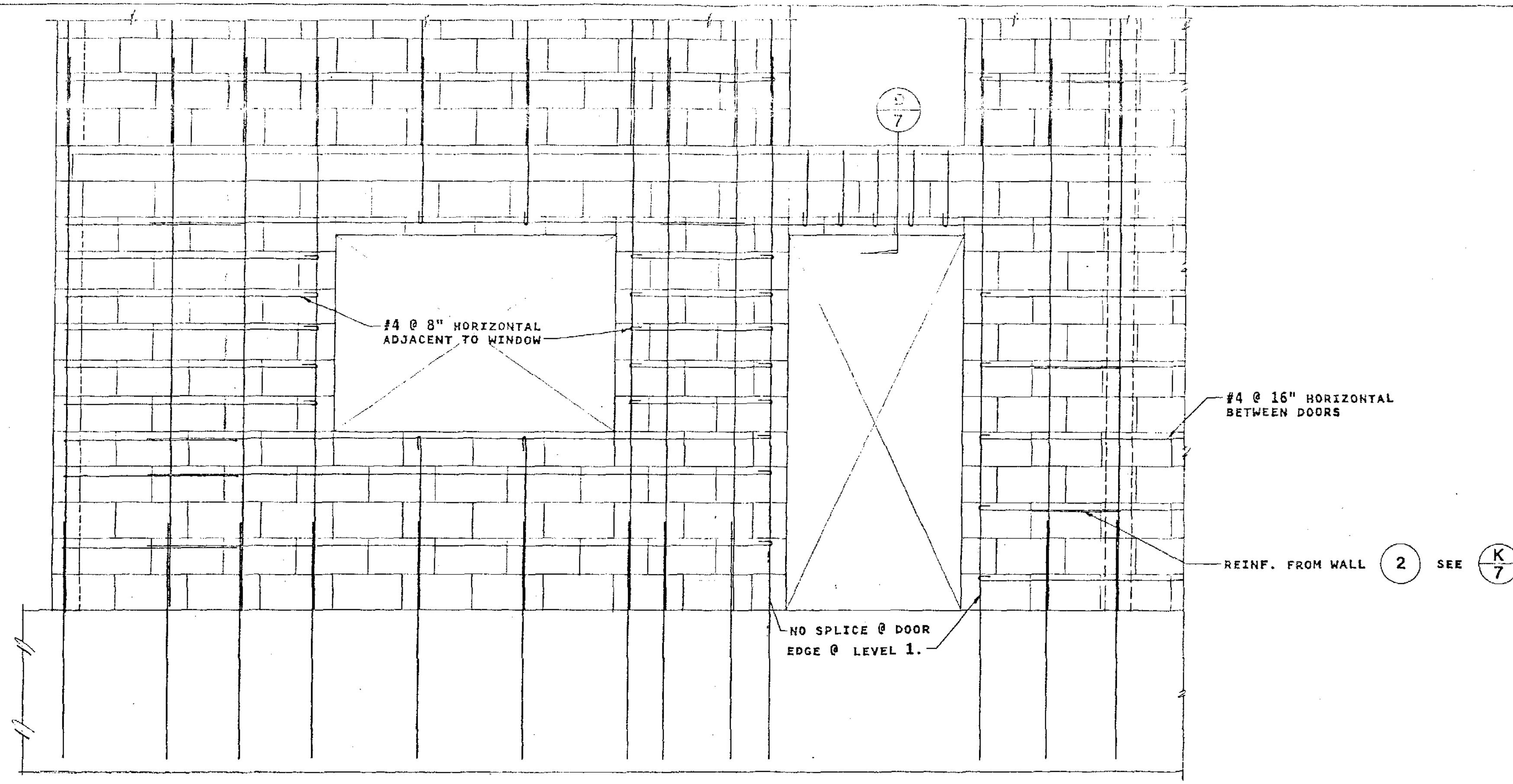
SEE FIGURES 4 THROUGH 6 FOR TYPICAL WALL REINFORCEMENT SIZE AND PLACEMENT.
REINFORCEMENT SHOWN IN FIRST LEVEL IS TYPICAL EXCEPT AS NOTED ON THIS ELEVATION FOR LEVELS (3) AND ABOVE. IF REINFORCEMENT SIZE AND PLACEMENT ARE NOT INDICATED HEREIN, USE REINFORCEMENT SIZE AND PLACEMENT AS SHOWN AT FIRST LEVEL BY FIGURES 4, 5, AND 6.

FIGURE 2



NOTES: ALL VERTICAL REINFORCEMENT IS #4 @ 48" o.c.
ALL HORIZONTAL REINFORCEMENT IS #4 @ 32" o.c.

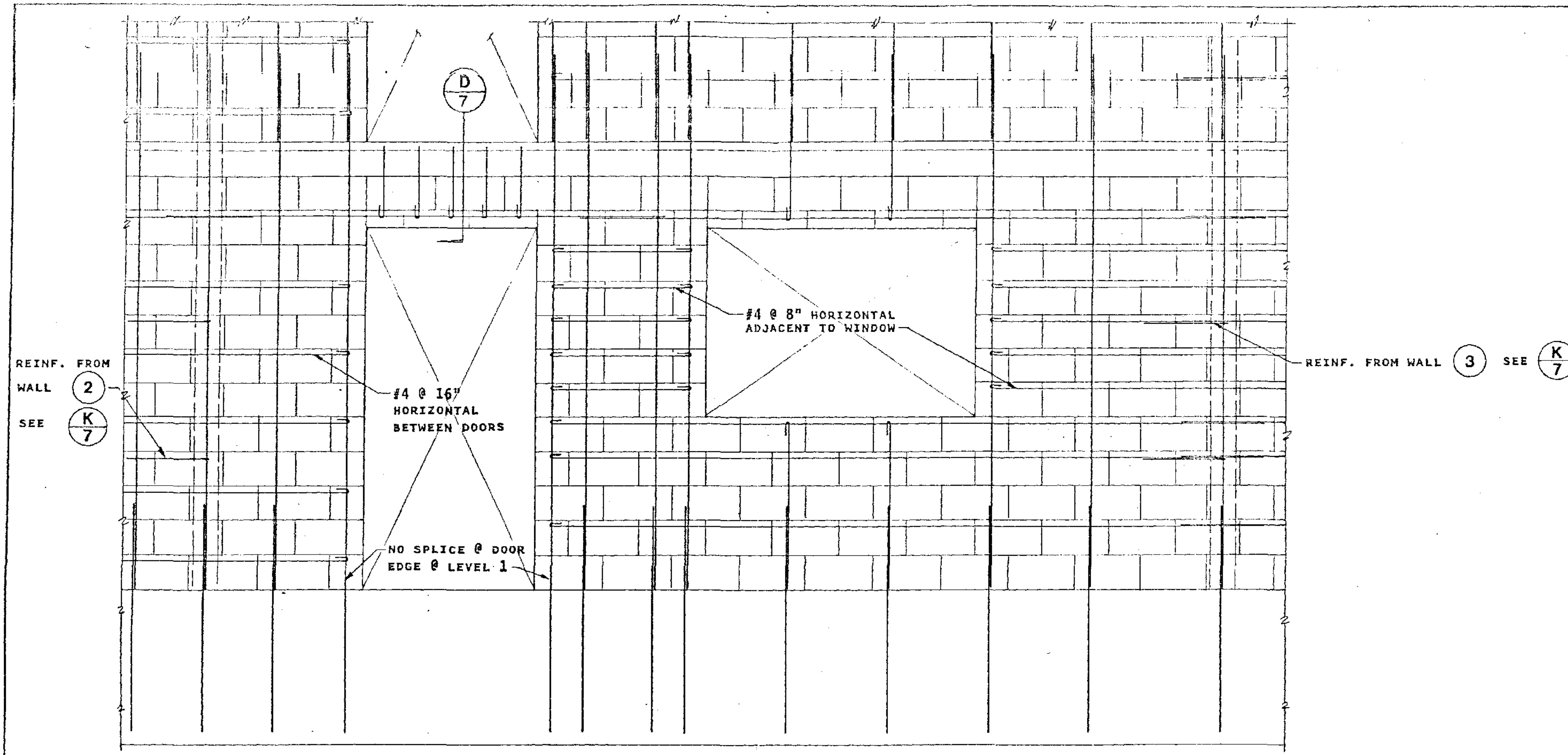
FIGURE 3



PARTIAL ELEVATION A/4

NOTES: ALL VERTICAL AND HORIZONTAL
REINFORCEMENT IS #4 BARS.
REINFORCEMENT NOTED IS TYPICAL AT ALL
LEVELS UNLESS SPECIFICALLY NOTED
OTHERWISE IN FIGURE 2.

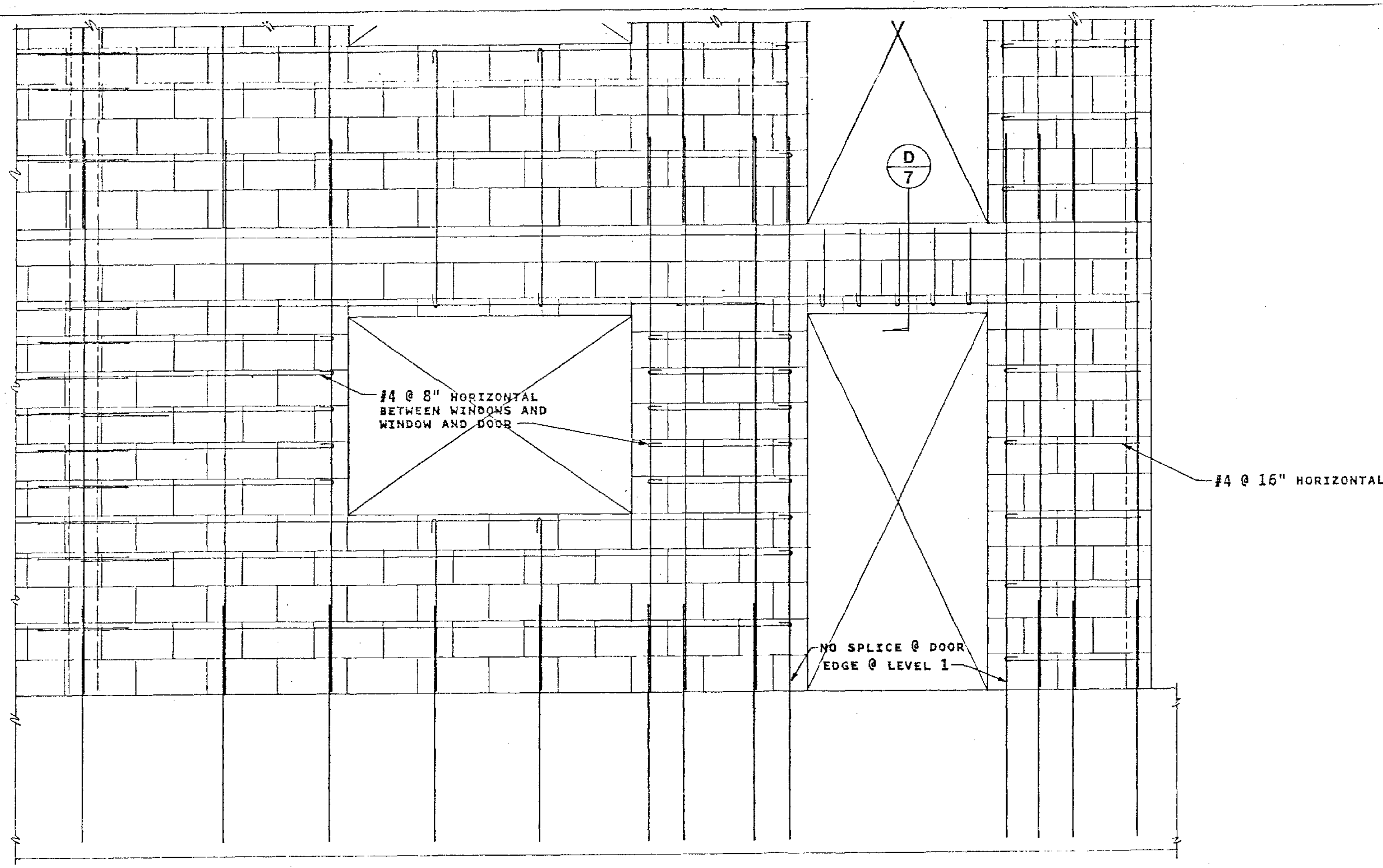
FIGURE 4



PARTIAL ELEVATION A/5

NOTES: ALL VERTICAL AND HORIZONTAL REINFORCEMENT IS #4 BARS. REINFORCEMENT NOTED IS TYPICAL AT ALL LEVELS UNLESS SPECIFICALLY NOTED OTHERWISE IN FIGURE 2.

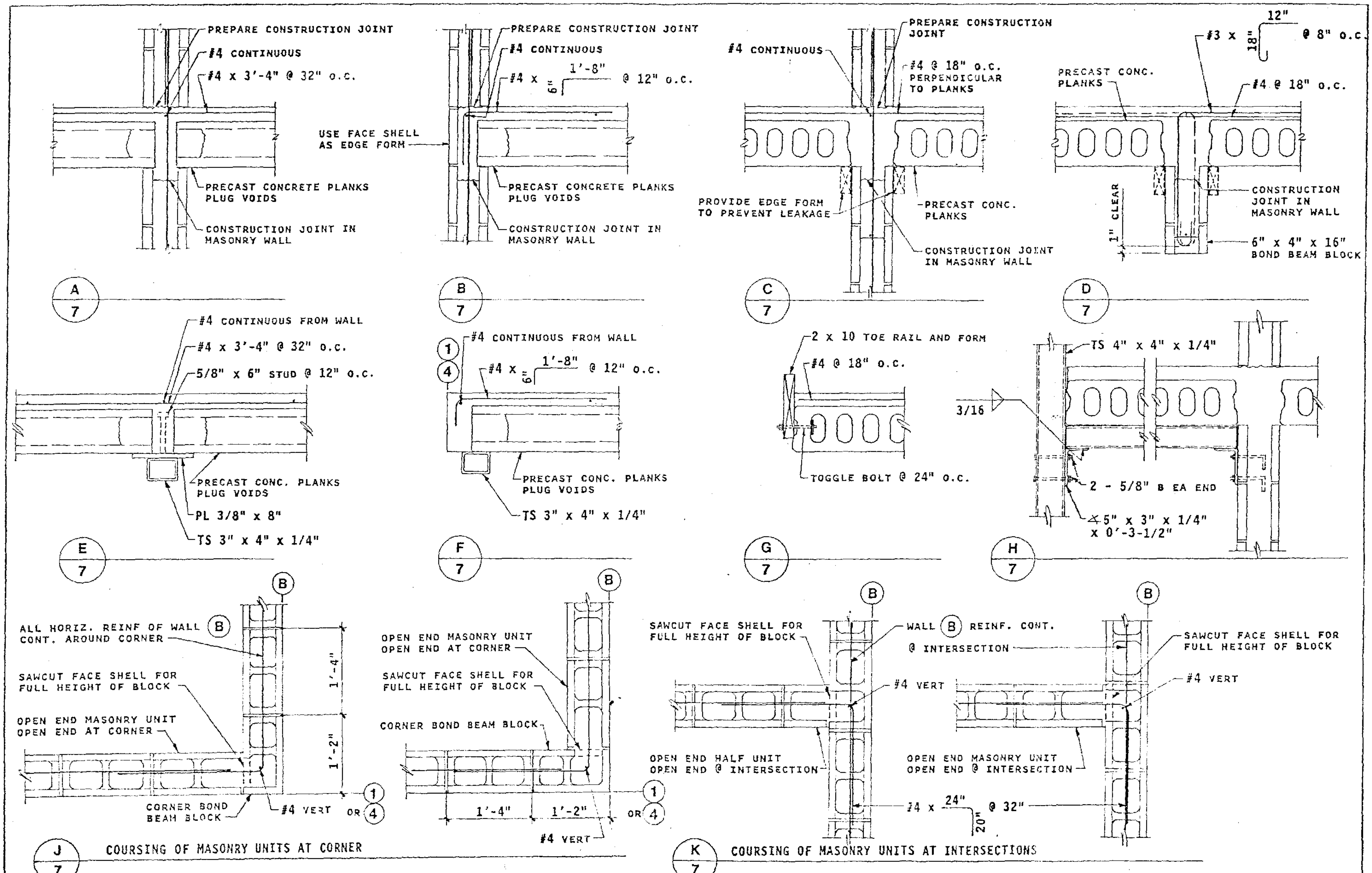
FIGURE 5



PARTIAL ELEVATION A/6

NOTES: ALL VERTICAL AND HORIZONTAL REINFORCEMENT IS #4 BARS.
REINFORCEMENT NOTED IS TYPICAL AT ALL LEVELS UNLESS SPECIFICALLY NOTED OTHERWISE IN FIGURE 2.

FIGURE 6



PREPARE CONSTRUCTION JOINT
#4 CONTINUOUS
#4 x 3'-4" @ 32" o.c.
PRECAST CONCRETE PLANKS PLUG VOIDS
CONSTRUCTION JOINT IN MASONRY WALL
A/7
PREPARE CONSTRUCTION JOINT
#4 CONTINUOUS
#4 x 6" 1'-8" @ 12" o.c.
USE FACE SHELL AS EDGE FORM
PRECAST CONCRETE PLANKS PLUG VOIDS
CONSTRUCTION JOINT IN MASONRY WALL
B/7
#4 CONTINUOUS
PREPARE CONSTRUCTION JOINT
#4 @ 18" o.c. PERPENDICULAR TO PLANKS
PROVIDE EDGE FORM TO PREVENT LEAKAGE
PRECAST CONC. PLANKS
CONSTRUCTION JOINT IN MASONRY WALL
C/7
#3 x 18" 12" @ 8" o.c.
PRECAST CONC. PLANKS
#4 @ 18" o.c.
1" CLEAR
CONSTRUCTION JOINT IN MASONRY WALL
6" x 4" x 16" BOND BEAM BLOCK
D/7
#4 CONTINUOUS FROM WALL
#4 x 3'-4" @ 32" o.c.
5/8" x 6" STUD @ 12" o.c.
PRECAST CONC. PLANKS PLUG VOIDS
PL 3/8" x 8"
TS 3" x 4" x 1/4"
E/7
1
4
#4 CONTINUOUS FROM WALL
#4 x 6" 1'-8" @ 12" o.c.
PRECAST CONC. PLANKS PLUG VOIDS
TS 3" x 4" x 1/4"
F/7
2 x 10 TOE RAIL AND FORM
#4 @ 18" o.c.
TOGGLE BOLT @ 24" o.c.
G/7
TS 4" x 4" x 1/4"
3/16
2 - 5/8" B EA END
∠5" x 3" x 1/4" x 0'-3-1/2"
H/7
B
ALL HORIZ. REINF OF WALL B CONT. AROUND CORNER
SAWCUT FACE SHELL FOR FULL HEIGHT OF BLOCK
OPEN END MASONRY UNIT OPEN END AT CORNER
1'-4"
1'-2"
CORNER BOND BEAM BLOCK
#4 VERT
1 OR 4
J/7
COURSING OF MASONRY UNITS AT CORNER
B
OPEN END MASONRY UNIT OPEN END AT CORNER
SAWCUT FACE SHELL FOR FULL HEIGHT OF BLOCK
CORNER BOND BEAM BLOCK
1'-4"
1'-2"
1 OR 4
#4 VERT
B
SAWCUT FACE SHELL FOR FULL HEIGHT OF BLOCK
WALL B REINF. CONT. @ INTERSECTION
#4 VERT
OPEN END HALF UNIT OPEN END @ INTERSECTION
B
SAWCUT FACE SHELL FOR FULL HEIGHT OF BLOCK
#4 VERT
OPEN END MASONRY UNIT OPEN END @ INTERSECTION
#4 x 20" 24" @ 32"
K/7
COURSING OF MASONRY UNITS AT INTERSECTIONS

FIGURE 7