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OPTIMUM SEISMIC PROTECTION FOR NEW BUILDING CONSTRUCTION IN EASTERN METROPOLITAN AREAS

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DESIGN AND COSTING OF PROTOTYPE BUILDING

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16. Abstract (Limit: 200 words)

Results are presented for that part of the Optimum Seismic Protection Study which examines several prototype buildings for the purposes of obtaining initial building costs data and input data for dynamic analysis. An apartment building, 60 by 200 feet, was selected since the study emphasizes housing, and a large portion of urban construction consists of multi-unit dwellings. Each building is designed for five levels of seismic resistance. The city of Boston wind load of 20 lb/sq ft for structures greater than 800 feet distant from low mean water is used. Results are reported for six basic groups of concrete buildings selected for height, frame, and wall construction and another group of eleven story concrete buildings in which shear walls are replaced with moment frames. The design procedure for both concrete and steel buildings and strengths are described. A summary of the data for dynamic analysis and how it is computed is provided and results of cost comparisons given. A series of architectural drawings showing building types and charts depicting cost analyses, indices to computations, and structural design is included.

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OPTIMUM SEISMIC PROTECTION FOR NEW BUILDING CONSTRUCTION IN EASTERN METROPOLITAN AREAS

REPORT ON PROTOTYPE BUILDINGS

Preliminary Building Designs
Initial Building Cost Data
Description of Input Data for Dynamic Analysis

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Introduction

This report presents the results of that part of the Optimum Seismic Protection Study which examines several prototype buildings for the purposes of obtaining initial building cost data and input data for dynamic analysis.

One building configuration, an apartment building with plan dimensions 60 feet by 200 feet, was selected for all the prototype buildings. The choice of an apartment layout seems appropriate since the seismic protection study is emphasizing housing, and a large portion of urban construction is multi-unit dwellings. The simple rectangular layout and the framing systems used are intended to be representative of many buildings being built at the present time and in the near future.

One possible architectural plan, shown in Figure 1, consists of two rows of apartments separated by a common corridor. The exterior enclosure, the interior partitions separating apartments, and the corridor, elevator, and stair enclosures are masonry construction. These masonry partitions and exterior walls are detailed to be isolated from the structure and allow the expected lateral movements under seismic loads.

Each prototype building was designed for five levels of seismic resistance - Zone 0 (wind), Zones 1, 2 or 3 in accordance with the 1970 Uniform Building Code (UBC), and Zone 4, a "super zone" for which Z = 2.0 in UBC formula (14-1). The City of Boston wind load of 20 pounds per square foot for structures greater than 800 feet distant from low mean water was used. The Boston Building Code (1970) was used as a source for all live load requirements (typically 40 psf for apartments).

Typically, the designs of the prototype buildings have been completed and detailed to the extent that it serves the objectives of this portion of the study which are:

a) to provide architectural designs and structural systems which are indicative of current local practice,

b) to give the best possible data for dynamic analysis,

c) to provide an accurate cost comparison of all the prototype buildings.

Structural Framing

Three building heights were selected for the study - six, eleven and seventeen stories, and four types of structural systems - concrete frame, concrete shear wall, steel moment-resisting frame, and braced steel frame were evaluated. Typically, one prototype building utilizes two types of structural systems to resist lateral loads - moment frames on the exterior lines in the long direction, and concrete shear walls or steel braced frames in the short direction. This results in the six basic groups of buildings and their corresponding structural plan figures are shown below:

INDEX OF STRUCTURAL PLAN FIGURES				
BUILDING GROUP	ZONES 0&1	ZONE 2	ZONE 3	ZONE 4
17 STORY CONCRETE	FIG. 2	FIG. 3	FIG. 4	FIG. 5
11 STORY CONCRETE	FIG. 7	FIG. 7	FIG. 8	FIG. 9
6 STORY CONCRETE	FIG. 10	FIG. 10	FIG. 10	FIG. 11
17 STORY STEEL	FIG. 12	FIG. 12	FIG. 12	FIG. 13
11 STORY STEEL	FIG. 14	FIG. 15	FIG. 16	FIG. 15
6 STORY STEEL	FIG. 17	FIG. 17	FIG. 18	FIG. 18
11 STORY CONCRETE (MOMENT FRAMES IN TWO DIRECTIONS)	FIG. 6	FIG. 6	FIG. 6	FIG. 6

Figures 2-11 are preliminary structural plans for the basic group of concrete buildings. The notation used on the plans is (CMRF) concrete moment resisting frame, CSW (concrete shear wall) and each frame or shear wall is given a number (i.e. CMRF #14, CSW #6). Table 6A lists all the concrete frames and shear walls and what zone it is designed for. Figure 19 shows an elevation of a typical concrete moment resisting frame, and Figure 20 shows an elevation of a typical end shear wall with its connected beams and columns. Both figures are representative of larger scale drawings of all the frames and shear walls on file at LeMessurier Associates, Inc.

The basic groups of concrete buildings are framed with a 7" deep flat slab (no drop panels). Columns are spaced 20'-0" o.c. both ways. Spandrel beams and exterior columns form moment frames in the long direction. These frames have been designed to resist the total lateral forces on the building and any contribution of slab and interior columns has been ignored. Concrete shear walls resist lateral forces in the short direction. Typically, zones 0 and 1 were considered first and a shear wall 22 feet wide was placed at each end of the building. Interior shear walls were added as required. For simplicity and for architectural reasons, all shear walls run full height of the buildings.

In addition to the basic groups of concrete buildings, another group of eleven story concrete buildings was added. These buildings have the same moment frames in the long direction as the other group of eleven story concrete buildings. However, the concrete shear walls of the basic group have been replaced with moment frames on every column line (see Fig. 6). The floor system is a 7" deep one-way concrete slab spanning to beams which are members of the moment frames in the short direction. Columns on the exterior lines participate in both the long and short direction frames. All interior columns participate in the short direction frames.

Figures 12-18 are preliminary structural plans for the steel buildings. The notation used on the plans is SMRF (steel moment resisting frame) and SBB (steel braced bay). Table 6B lists all the steel frames and braced bays and what zone they are designed for. Figure 21 shows an elevation a typical steel moment frame, and Figures 22 and 23 show elevations of typical braced bays.

The floors of the steel buildings are a 5" overall depth concrete slab on metal deck spanning to intermediate steel beams at 6'-8" o.c. These beams are carried by girders spanning from column to column. As in the concrete buildings, the spandrel beams and exterior columns form moment frames in the long direction which resist the total lateral forces on the building. Steel K-bracing resists lateral forces in the short direction.

Design Procedure - Concrete Buildings

The general procedure used in designing the concrete moment resisting frames was to start with the wind and gravity loads case. Trial columns were selected on the basis of gravity loads in the bottom story using a column with a high ratio of reinforcing steel (8% maximum for zones 0 and 1, 6% maximum for ductile frame requirements in zones 2, 3 and 4), and a high concrete strength (f'c = 5000 psi). The column moments produced by wind and gravity loading were then compared to the bending capacity of the trial columns using interaction curves set forth by the ACI Code. Thus the column dimensions were selected by requirements of the lowest story and remained constant for the height of the building for architectural continuity and for re-use of formwork. Column reinforcing steel and concrete strengths were then varied at different floor levels to suit design requirements.

Concrete beams were selected by similar procedures to those used in selecting column sizes. First the typical bending moments due to dead and live loads were calculated using the appropriate load factors. Then a combined wind and gravity load analysis was made using its appropriate load factors, and the bending moments of the two cases were compared. Beam dimensions were selected on the basis of the larger moments. Reinforcing steel in the beams was then varied to suit design requirements while the cross section dimensions were held constant for a given building.

After designing a building for Zone 0 (wind and gravity loads), a combined Zone 1 earthquake and gravity load analysis was made and compared to the previous case. For most of the buildings, Zones 0 and 1 have very similar results for combined vertical and lateral loads. Combined earthquake and gravity load analyses were than made for Zones 2, 3 and 4, and member sizes increased as required. Table 7 lists beam and column sizes used in the concrete buildings for the various Zones.

Analysis of all moment resisting frames was carried out using a special purpose plane frame computer program (FRMST) developed at the University of California at Berkeley. The output of this program includes story drift, beam end moments, and column end moments, shears, and axial forces.

Although no drift limitations for wind and earthquake loading are specified in the Code, a limit of .0016 times the total building height for wind loads and a corresponding limit of .0033 for earthquake loads was used for this study. For the concrete buildings stress considerations governed all member design and drift was always within the limits described above. Drift was computed using rectangular gross concrete sections ignoring tee action of slabs. Table 4A shows the total building drift for all the concrete buildings.

After the design cycle, a representative beam and column joint in the lower stories of a Zone 3 concrete frame was checked for compliance with detailing requirements in the ACI and UBC Codes. Problems of overstress and congestion occurred and possible solutions were considered. The most probable solution would be to increase beam depths slightly to relieve joint overstress and reduce beam reinforcing and/or to make the columns slightly wider than the beams in order to allow both beam and column reinforcing to be continuous through the joint without conflict. However, the designs are preliminary in nature, and the scope of the project did not permit finalizing all member sizes to meet all the detail requirements. The costs would be affected little if any by increasing sizes of beams or columns slightly. The incremental cost of form and concrete would be offset by the reduction in reinforcing steel. For example, beam type B37 (12" x 24", 4-#10, Mu = 318 ft. - kips) is compared to beam type B42 (12" x 26", 4#9, Mu = 324 ft. - kips). Type B37 requires some compression steel, but the minimum amounts of bottom steel that extend through the beam column joint would satisfy this requirement and it would be present in both beams so it is not included in the cost comparison. Using the same unit costs for concrete, forms, and reinforcing as were used in the cost analysis, the added cost of form and concrete for B42 is $1.00 per linear foot, but the savings in reinforcing is $1.08 per linear foot.

It is known from experience that small increases in beam and column depths affect the static analysis very slightly. For example, the eleven story steel moment resisting frame in Zone 4 (SMRF #8) was first designed for stress requirements and checked by computer analysis for drift requirements (analysis #1). Drift was not acceptable, so beam stiffnesses were increased by about 38% over the lower seven stories and another analysis (#2) was done. Drift was still not acceptable, so column stiffnesses for the lower four stories were increased about 18% and another analysis (#3) was done. The maximum change of 6% in beam moments occurred between analysis #1 and #2 at the outboard columns, and most changes were on the order of 1% - 3%.

It is concluded from the above that finalizing concrete member sizes to meet all the detailing requirements affects the overall cost and the static analysis very little. Minor increases in stiffness do not produce significant changes to the static analysis, and it is assumed that the dynamic analysis would not be changed appreciably either.

For the design of shear walls, stability against overturning had to be considered. Overturning moments at foundation level were calculated using the factor "J" as computed by UBC formula (14-8), but for design of individual elements comprising the lateral force resisting systems, J = 1.0. In the higher earthquake Zones stability considerations required interior shear walls in addition to the two end walls. An interior wall was added with an 8 foot wide corridor opening at center. This wall effectively became two independent walls linked together by the slab, and they were designed as such. One computer analysis was made to determine the slab link moments and although moments are high, the slab can be adequately reinforced. For all concrete shear walls, the design forces of UBC formulas (32-1) and (32-2) were doubled in calculating shear and diagonal tension as required in Section 2632.

In the design of the end shear walls having connected spandrel beams and columns, the total frame and shear wall system was analyzed using the FRMST computer program, and the beams and columns were designed to resist the computed forces and moments. The shear wall, however, was designed to resist the total forces on the system.

The horizontal force factor "K" has been taken as 1.00 where the lateral force resisting system is provided by shear walls and

.67 where it is provided by a moment resisting space frame. It should be noted that the use of K = 1.00 requires that a total vertical load carrying system be furnished in addition to the shear wall or bracing system. In the case of the concrete shear wall buildings this means the incorporation of tied ductile columns into the shear wall system to carry both the superimposed loads and the load of the shear wall itself.

In computing Ft, the additional concentrated load at the top of the structure, in accordance with UBC formula 14-4, Ds equal to the total plan dimension out to out of the shear walls was used. Upon review, it was stated that for unlinked shear walls, a more correct dimension would be the average width of the individual elements. This would change our applied earthquake load distribution by the concentration of 15% of the total base shear. The major effect of this change appears to be in the design of upper story shear reinforcement. The cost effects of such changes have been included in the estimates.

Design Procedure - Steel Buildings

The general procedure of the design of the steel buildings is quite similar to that described for the concrete buildings. For the moment resisting frames in the long direction trial beams and columns were selected on the basis of gravity loads. An analysis was made for combined wind and gravity loading and final member sizes were selected for the more critical case. Steel columns were designed using the AISC Steel Column Design Computer Program. Combined earthquake and gravity load analyses were made for Zone 1 (where it was not obvious that wind governed) and Zones 2, 3 and 4. Member sizes were increased as required by analysis. Some columns participate in both the moment frames and the braced bays, and had to be designed for the more critical case. Steel moment frames which had to have member sizes increased to satisfy drift requirements were SMRF #4 (17 story - Zone 4), SMRF #7 (11 story - Zone 3) SMRF #8 (11 story - Zone 4) and SMRF #12 (6 story - Zone 4). When frames had to have member sizes increased to satisfy drift requirements, the girders were increased first and another analysis made. If a further analysis was required, column sizes and sometimes girder sizes were increased and another analysis made. When increasing girder or column sizes, an effort was made to keep the ultimate moment capacity of the columns greater than that of the girders in the lower two-thirds of the building.

For the short direction of the steel framed buildings, lateral forces are resisted by a K-braced system. As in the case of the concrete buildings, bracing was placed in the end walls first and interior bracing was added in the higher lateral force zones as required by stability considerations. Where the full end wall (60 foot width) was used, three 20 foot wide bays of K-braces comprise the lateral force resisting system (see Figure 22). This system was first analyzed as having four columns at a spacing of 20 feet.

However, it soon became apparent that for lateral loading the two inboard columns (relatively close to the neutral axis of the total system) contribute very little. It seemed that a more efficient system would result if the inboard columns (on column lines B and C) were removed, and some of their steel added to the outboard columns. This seemed reasonable since the gravity loads could be carried to the outside columns by the then formed Warren type truss with very little or no increase in diagonal and chord sizes. One of the greatest advantages of this system is to bring all the gravity loads to the outside columns to overcome the uplift force due to overturning. The uplift force on the outside column only reduces about 25%-30% when inboard columns are used. However, the gravity loads to resist the uplift force on the outside columns increase by 270%-280% when the inboard columns are removed. For these reasons the system shown in Figure 22 was used where overturning forces were relatively high.

Where stability considerations forced the use of additional lines of bracing in the interior (Zone 4, all heights), the same system as shown in Figure 22 would have been preferred from an engineering standpoint. However, architectural considerations of an 8 foot wide corridor at the center would not allow the continuous truss configuration. For this reason the system shown in Figure 23 was used. It is comprised of two simple K-braced bays separated by 8 foot long link beams forming the corridor space.

Once this system was chosen for the interior, the same system was used in the end walls for the Zone 4 buildings. Mixing of the two bracing systems (Figures 22 and 23) in one building did not seem feasible, for preliminary computations showed the stiffness of the system in Figure 22 to be 2-1/2 times greater than the system of Figure 23. The bracing system in Figure 23 could have been made stiffer by changing the 8 foot link beams to heavy girders, rigidly connecting the K-braced bays. However, the time required to model this system, find its relative stiffness, and determine its compatibility with a system of the type in Figure 22 was not justified in the scope of the project.

Table 4B lists total building drift for selected buildings in the short direction. Braced bays were modeled as single columns and analyzed by the STRESS computer program in order to determine drift. The Zone 4 case of all the building was analyzed first and the drift was acceptable for these three buildings. By inspection and comparison, drift was obviously acceptable for all but three buildings. These three buildings were then analysed, and drift was found to be acceptable without increasing member sizes above stress requirements.

Steel and Concrete Strengths

All structural steel designed for the prototype buildings was A36. This grade of steel is most widely used at present for this type of structure. To use a higher grade of steel would not be justified for these buildings, especially in the Zone 4 moment frames where stiffness requirements were greater than stress requirements.

Reinforcing steel for the concrete buildings was designed to have a yield strength of 60,000 psi. This higher strength was used to reduce amounts of reinforcing - especially in the ductile moment frames where congestion is a problem. The unit premium cost of using 60,000 psi versus 40,000 psi yield strength reinforcing is very slight, but the amounts of steel saved using the higher strength can be substantial.

Concrete strength for all frame members exposed to weather was designed as 4000 psi - both to insure durability and to keep member depths to a reasonable size. In the eleven story and seventeen story buildings, some 5000 psi concrete was used in the lower story columns to keep their sizes minimum. The interior slab for the 17 story and 11 story concrete flat slab buildings have 3000 psi concrete. In the 6 story flat slab building where the interior column size reduces, 4000 psi concrete is used for the slab to satisfy peripheral shear requirements around the column. For the 11 story building with moment frames in both directions, 4000 psi concrete is used throughout to minimize concrete beam sizes. The slabs on metal deck for all the steel building have 3000 psi concrete.

Computation of Masses

Figure 25 shows sample computations for masses used in the concrete buildings. Computations were done on the basis of loads on a typical interior column and a typical exterior column, and these were summed up for the total masses per floor. Masonry block separating apartments and lining the corridor is the heavier concrete block weighing 55 psf of wall area. Partitions within each apartment are drywall partitions, and their weight was averaged at 14 psf of floor area. The amount of masonry block walls used in the computations was for a column separating apartments rather than one which falls inside an apartment, so that the total masses used are on the conservative side. Table 5 provides a reference list of the design computations on file at LeMessurier Associates, Inc.

Foundations

Foundations for all designs have been designated as spread footings on firm ground. Except for increases where required for vertical loads due to lateral loads, no foundation changes have been included.

Dynamic Analysis Input

A summary of the data for the individual building elements required for dynamic analysis and how it was computed is given as follows:

The data required for concrete moment-resisting frames were ultimate moment capacities and moment of inertia for both beams and columns. Beam ultimate moment capacities were calculated by ACI Code formulas with no capacity reduction factor (Ø) included. The beam ultimate moment capacity given was the capacity at the negative region (the values were averaged if the two end capacities varied). This was done assuming that for high lateral loads, hinges would form at the beam ends first, and that there would always be enough reinforcing in the middle of the beam to satisfy positive bending requirements. Beams were assumed to have cracked sections over a significant portion of their length under the earthquake loadings, so that beam moments of inertia were typically taken as 0.4 times the moment of inertia of the uncracked section.(1) Ultimate moment capacities for columns were selected from interaction diagrams based on ACI Code formulas. Realistic values of dead loads (no load factor or live loads included) were used with interaction diagrams to determine ultimate moment capacity. Again, no capacity reduction factor (Ø) was included. Concrete column moments of inertia were given as the moment of inertia of the uncracked section assuming that vertical loads would keep the entire cross section of the column in compression.

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(1) "Non-Linear Dynamic Response and Ductility Requirements of Building Structures Subjected to Earthquakes" by S. A. Anagnostopoulos, Structures Publication #349, MIT Department of Civil Engineering.

The data given for concrete shear walls were modulus of elasticity, moment of inertia, shear area, ultimate moment capacity, and ultimate shear capacity. Shear wall moment of inertia was taken as 0.5 times the moment of inertia of the uncracked section,[(1)] and the procedure used to determine ultimate moment capacity was the same as for columns. Ultimate shear capacity was computed by a summation of values given by ACI formulas 11-13 and 11-33.

The data required for steel moment resisting frames were ultimate moment capacity and moment of inertia for beams and columns. Steel beam ultimate moment capacities were given as the plastic moment capacity (Mp) as listed in the AISC Manual of Steel Construction. Ultimate moment capacities for steel columns were given as the lesser value computed from AISC interaction formulas 2.4-2 and 2.4-3. Realistic values of dead loads (no load factor or live loads included) were used for values of (P) in these formulas.

The data required for the steel braced bays was given in the same format as that of the concrete shear walls - modulus of elasticity, moment of inertia, shear area, ultimate moment capacity, and ultimate shear capacity. The moment of inertia of the braced bay was computed as the moment of inertia of the columns about an axis centered on the full bay. The ultimate moment capacity was computed as the critical buckling load of the column times the width of the braced bay. Formulas for equivalent shear area were developed equating axial elongations of diagonals and chords of the articulated system to the shear distortion of a single member with a solid web. Ultimate shear capacity of the braced bays was computed as the horizontal force required to produce critical buckling loads in all the diagonals of a given bay.

Detailing and Cost Considerations

Members of moment resisting frames used in lateral force resisting systems for Zones 2, 3 and 4 must be detailed for ductile behavior by code. The steel buildings present little additional complexity over a normally connected building in which moment connections develop the plastic capacity of the beams. Cost estimates have been based on shop welds, field bolted connections. Shop and field inspection of such construction is normal and does not add to the cost premium.

The concrete buildings, however, are affected more drastically by the ductile frame requirements. First, the relative amount of small bent bars for stirrups and ties increases markedly, then the difficulties in placement of reinforcement, particularly at column/beam intersections, are multiplied, and, to see that the job is done correctly, more field inspection is necessary. All of these items contribute to the larger unit cost estimate for the reinforcement of Zone 2, 3 and 4 structures.

Of the non-structural elements which might contribute to seismic design cost premiums only the isolation and reinforcement of masonry partitions and walls are considered of sufficient magnitude to affect the results of the estimates presented herein. Typical isolation details were developed and changes in wall reinforcement were assessed to determine the incremental cost increases for these items.

Results of Cost Comparisons

Table 1 shows the total costs and square foot costs of the superstructures for each of the buildings. It should be noted that the relative structural costs of steel and concrete buildings must not be used for direct comparison in choosing a structural material. The steel structure would require additional items of cost such as hung ceilings and fireproofing to produce a comparable end product.

Table 2 shows the incremental square foot costs to be added to Table 1 for non-structural seismic costs. The square foot superstructure and masonry costs are combined into Table 3 and the percentages that these items represent of the total construction cost are also shown. The basic building cost has been held constant for all Zone 0 buildings at $28 per square foot. This is consistent with published figures of building costs and our own experience.

Figure 24 plots the percentage increases over base Zone 0 costs for the Zones 1, 2, 3 and 4 for each of the building considerations. For comparison we have also plotted the results of the Pilot Building Study published in MIT Report No. 2 of this project. It is noted that the 13 story Pilot Building has greater percentage cost increases than the 17 story steel framed building of this report. The reasons for this are:

1) the story height of the Pilot Building is 12 feet making the total building height just about the same height as the 17 story steel building,
2) the bay sizes of the Pilot Building are 28'-4" versus 20'-0" in the prototype buildings,
3) the Pilot Building has moment frames in both directions and all the steel on column lines participates in resisting lateral loads whereas the prototype steel building have both frames and braced bays, and not all the steel participates in resisting lateral loads.

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Subject MIT	Made by VRS	Job No. 7941
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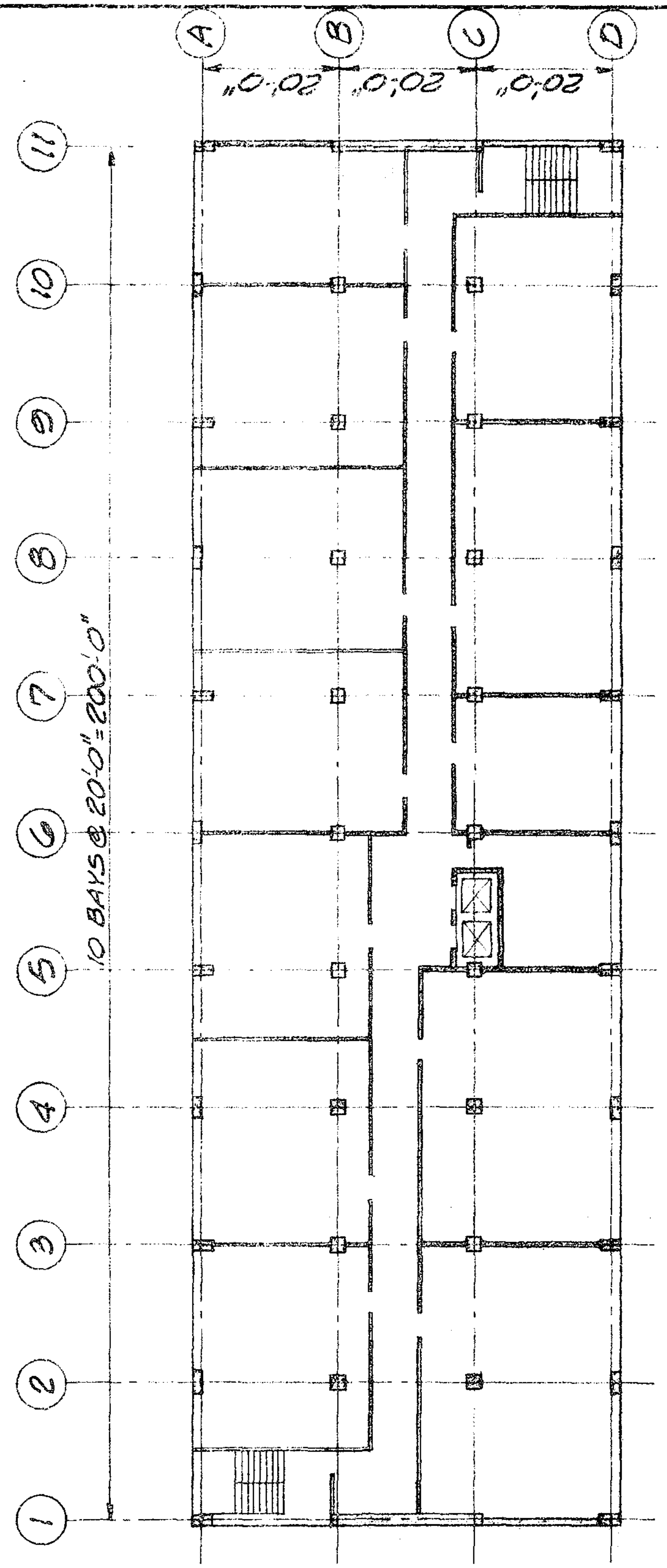

TYPICAL FLOOR LAYOUT

FIGURE 1

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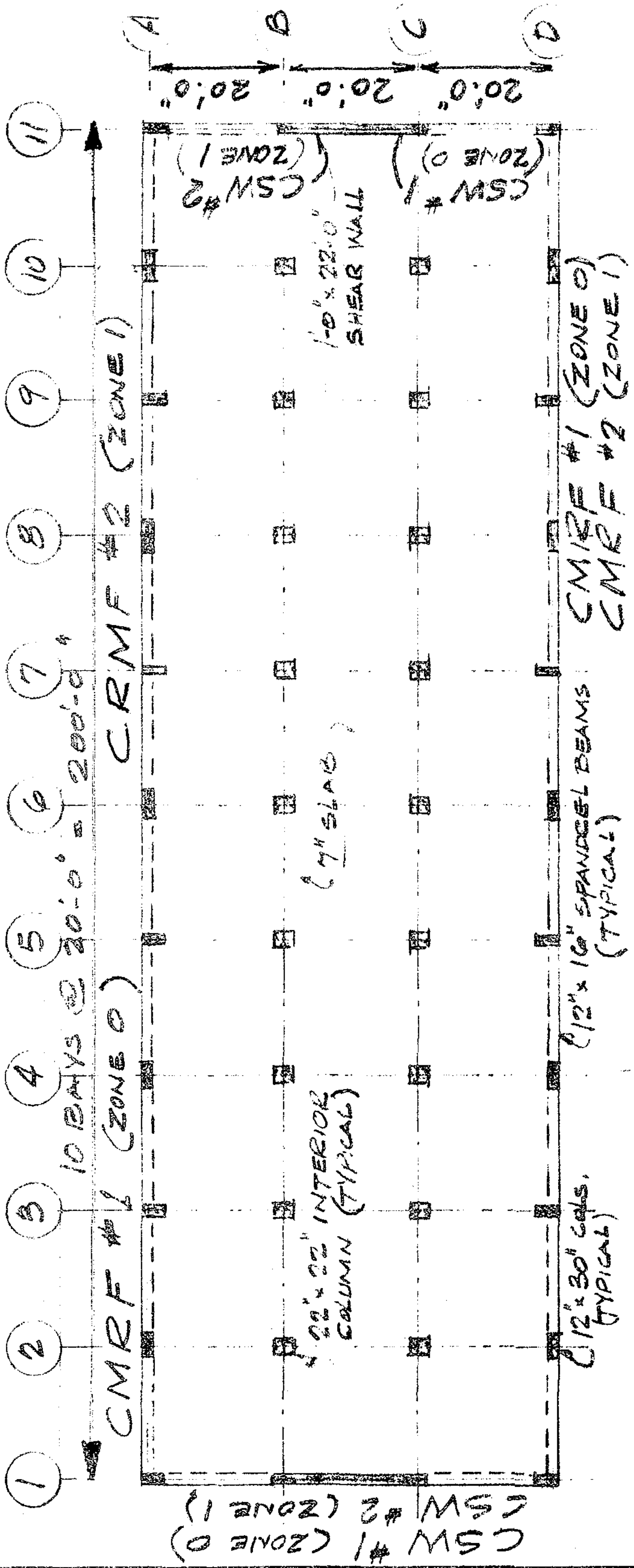

TYPICAL FLOOR FRAMING FOR 17 STORY BUILDING

PLAN APPLICABLE FOR ZONE 0 AND ZONE 1

FIGURE 2

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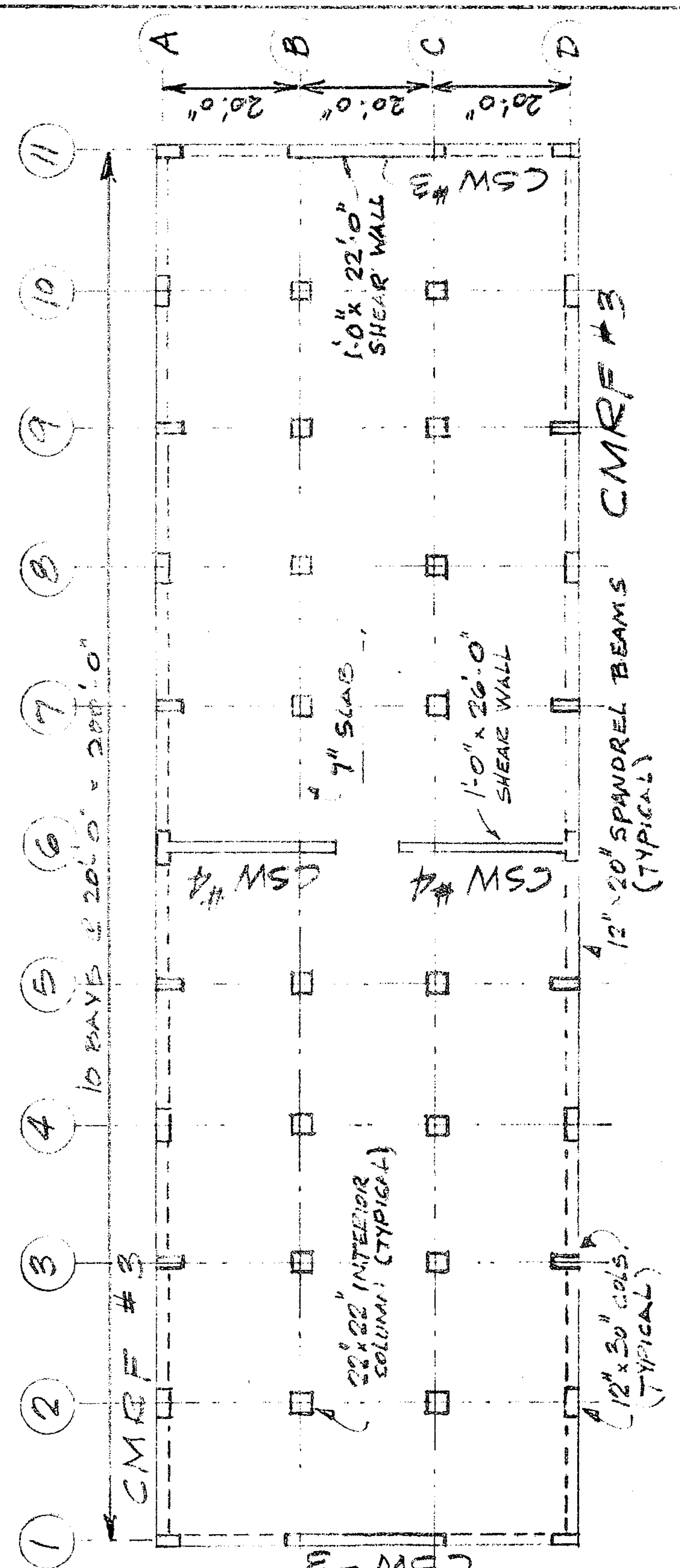

TYPICAL FLOOR FRAMING FOR 17 STORY BUILDING

PLAN APPLICABLE FOR ZONE 2

FIGURE 3

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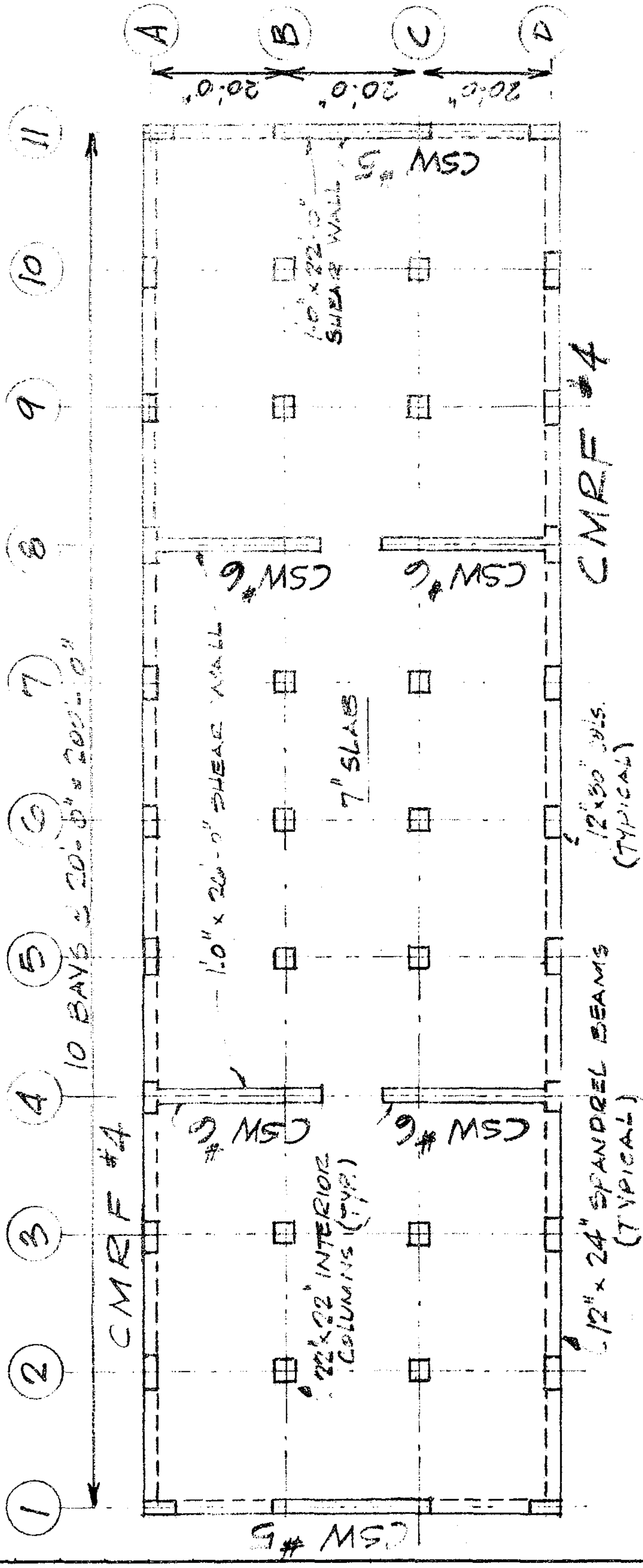

TYPICAL FLOOR FRAMING FOR 17 STORY BUILDING

PLAN APPLICABLE FOR ZONE 3

FIGURE 4

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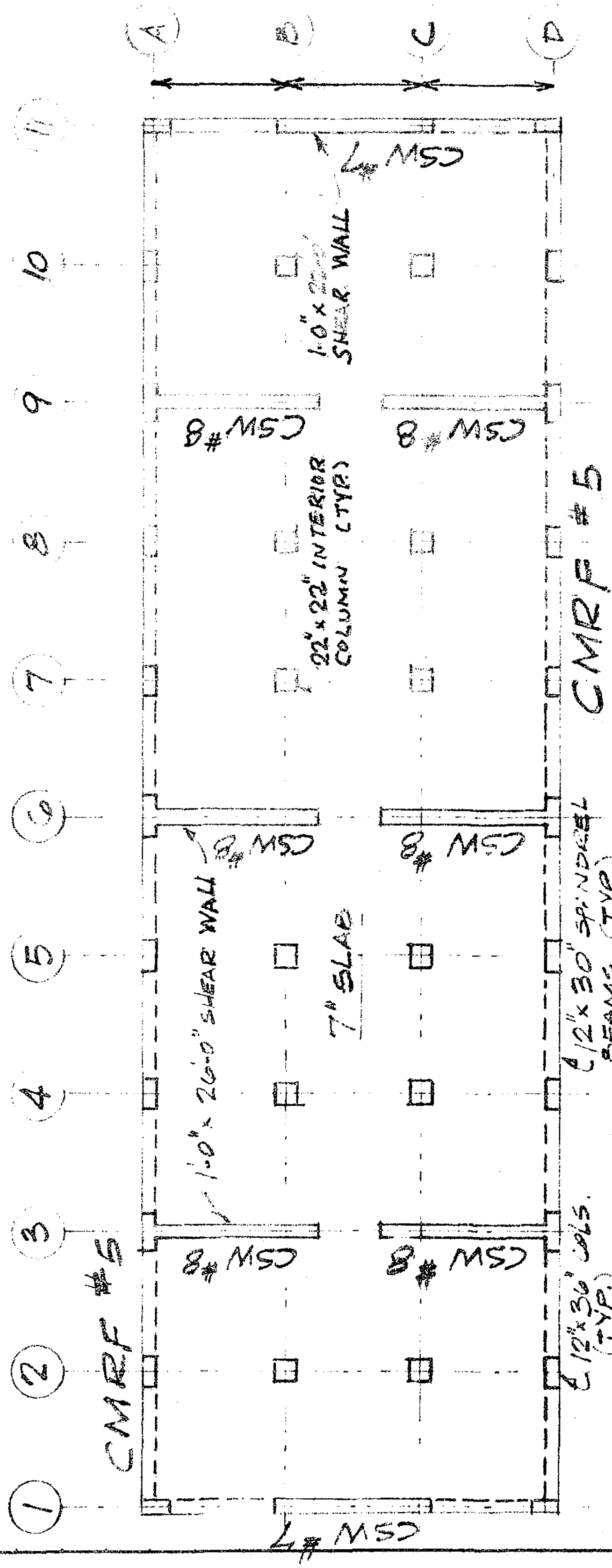
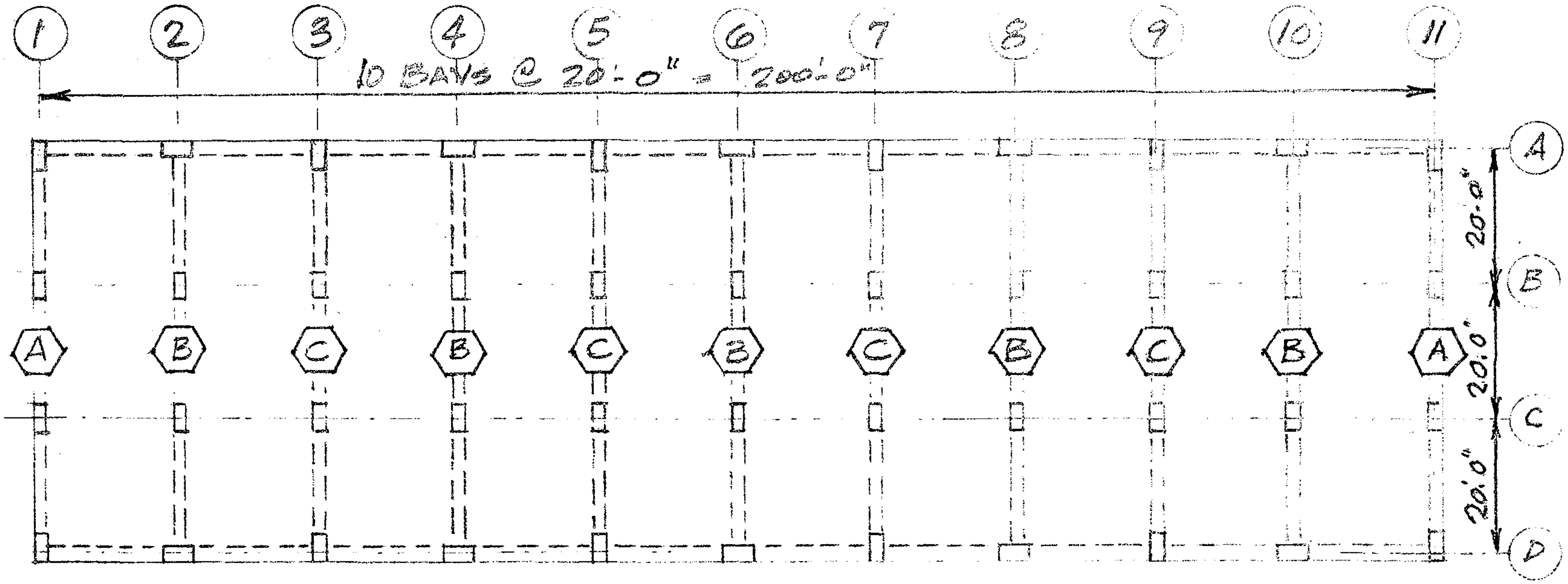


TYPICAL FLOOR FRAMING FOR 17 STORY BUILDING

PLAN APPLICABLE FOR ZONE 4

FIGURE 5



TYPICAL FLOOR FRAMING PLAN FOR 11 STORY BUILDING
WITH MOMENT RESISTING FRAMES IN 2 DIRECTIONS

FIGURE 6

LOADING CASE	FRAME TYPES IN SHORT DIRECTION			GIRDER SIZE	COLUMN SIZE	FRAME TYPES IN LONG DIRECTION	GIRDER SIZE	COLUMN SIZE
	A	B	C					
ZONE 0	2-CMRF #6	5-CMRF #7	4-CMRF #8			2-CMRF #9		
ZONE 1	2-CMRF #6	5-CMRF #7	4-CMRF #8			2-CMRF #10		
ZONE 2	2-CMRF #11	5-CMRF #12	4-CMRF #13			2-CMRF #14		
ZONE 3	2-CMRF #15	5-CMRF #16	4-CMRF #17			2-CMRF #18		
ZONE 4	2-CMRF #19	5-CMRF #20	4-CMRF [illegible]			2-CMRF #22		

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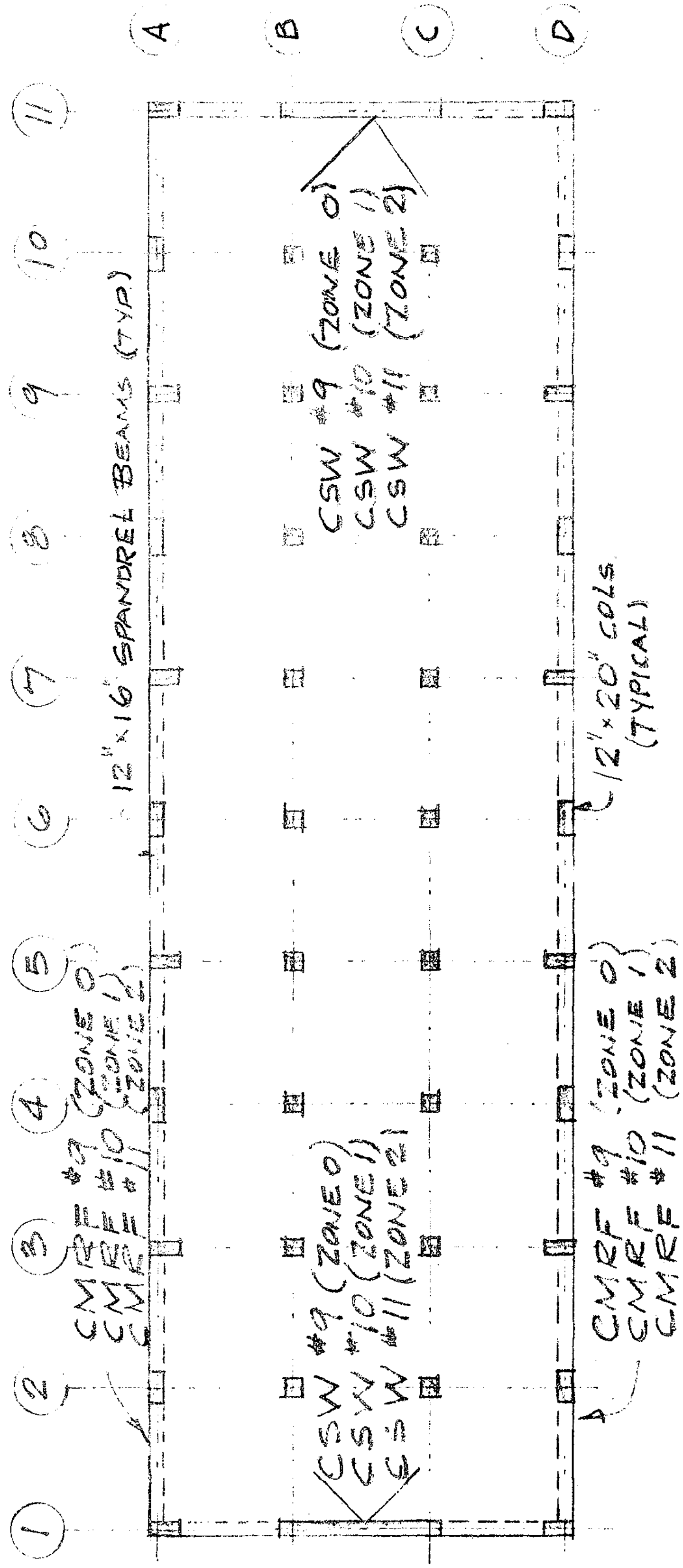
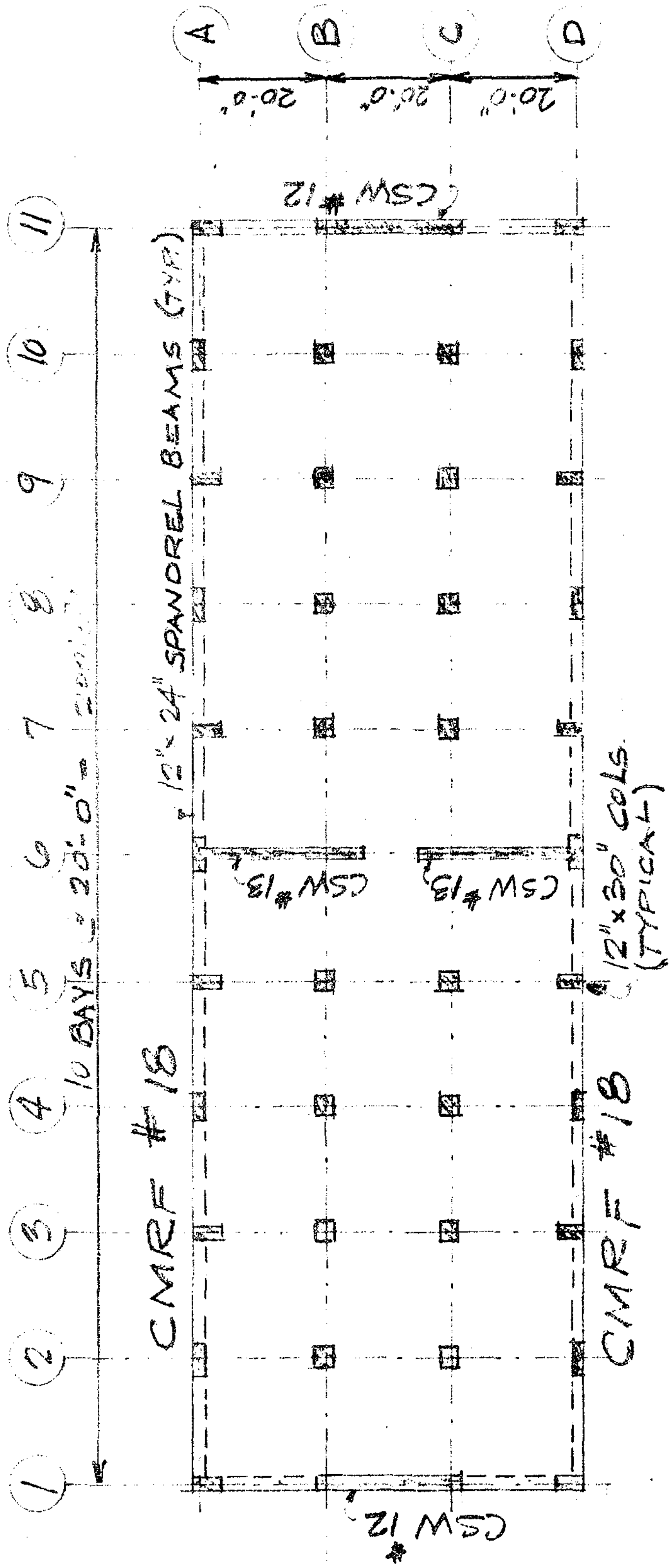


TYPICAL FLOOR FRAMING FOR 11 STORY BUILDING

(PLAN APPLICABLE FOR ZONES 0, 1, & 2)

FIGURE 7

LeMessurier Associates, Inc	Subject MIT EARTHQUAKE STUDY	Made by AW	Job No. 7941
		Checked by	Date
		Approved by	Sheet No.

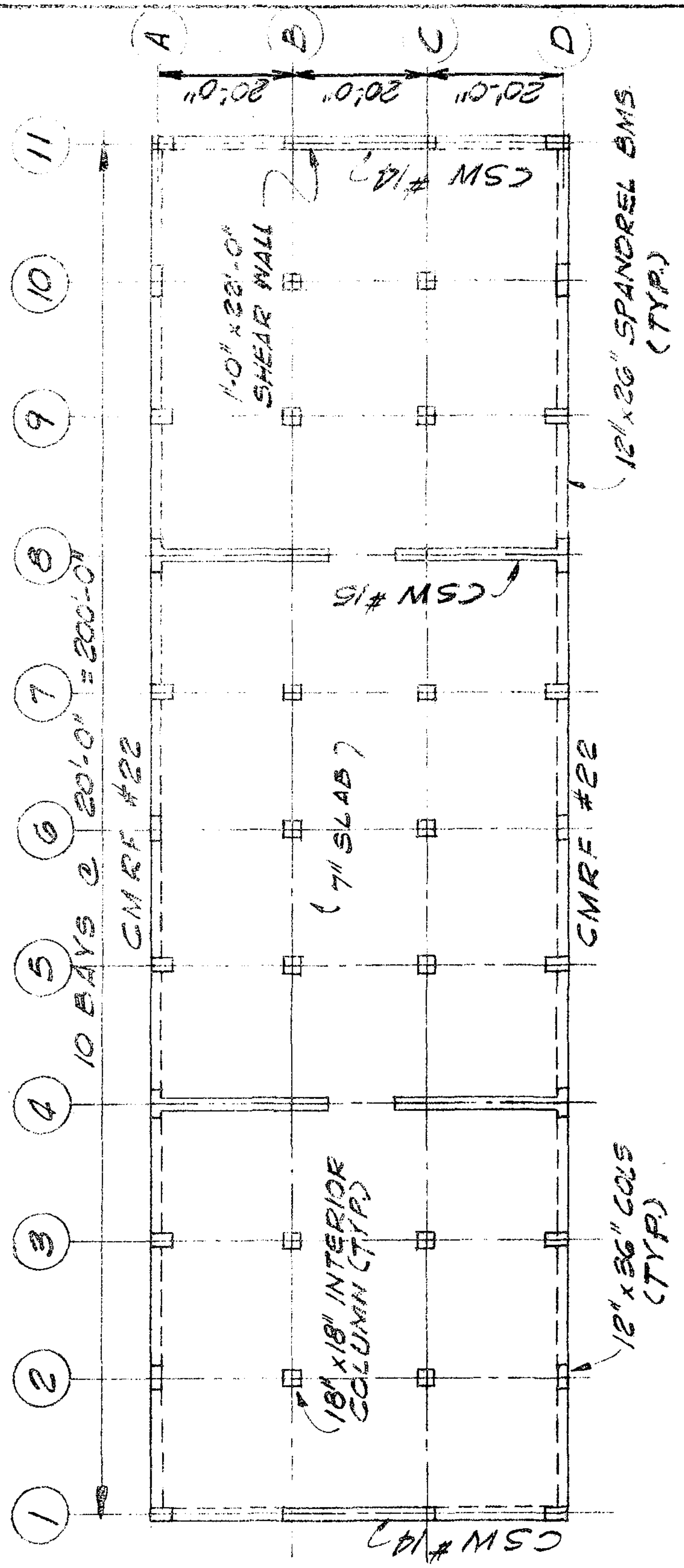


TYPICAL FLOOR FRAMING FOR 11 STORY BUILDING

ZONE 3

FIGURE 8

LeMessurier Associates, Inc	Subject MIT EARTHQUAKE STUDY	Made by ALW	Job No. 7941
		Checked by	Date 20 OCT. 1972
		Approved by	Sheet No.

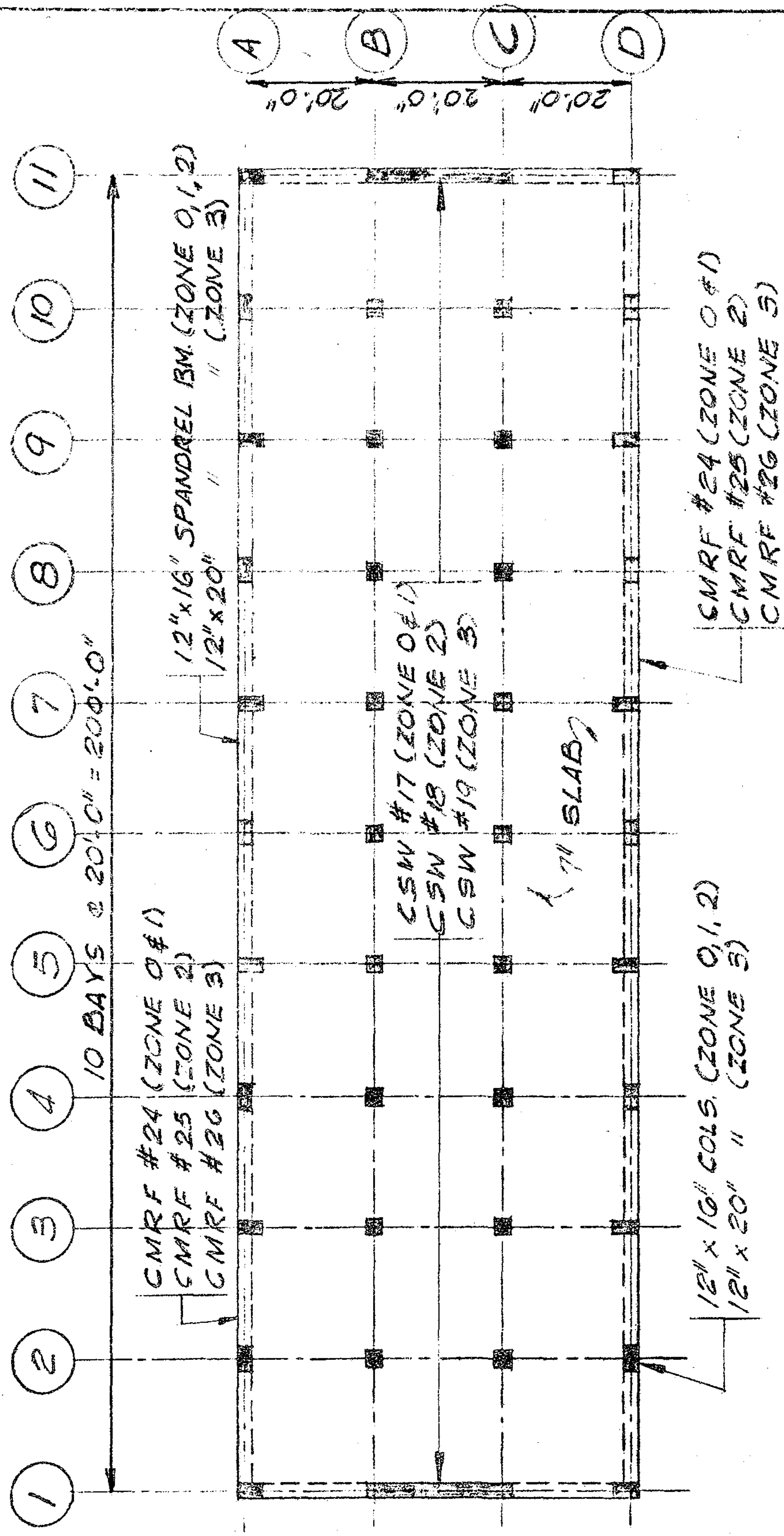


TYPICAL FLOOR FRAMING FOR 11 STORY BUILDING

ZONE 4

FIGURE 9

LeMessurier Associates, Inc	Subject MIT	Made by AW	Job No. 7941
	EARTHQUAKE	Checked by	Date
	STUDY	Approved by	Sheet No.

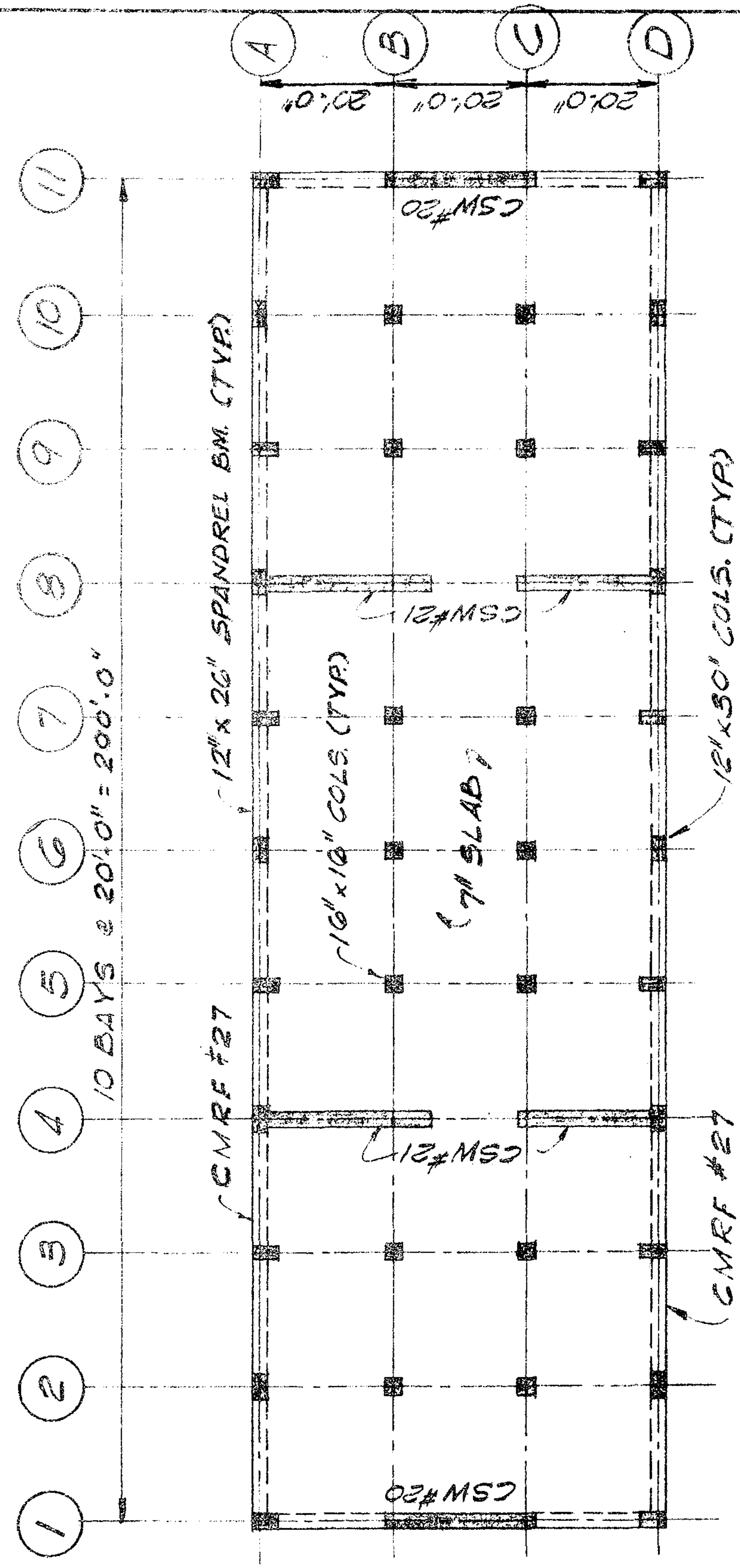


TYPICAL FLOOR FRAMING FOR 6 STORY BUILDING

ZONES 0,1,2,3

FIGURE 10

LeMessurier Associates, Inc	Subject MIT EARTHQUAKE STUDY	Made by AW	Job No. 7941
		Checked by	Date
		Approved by	Sheet No.

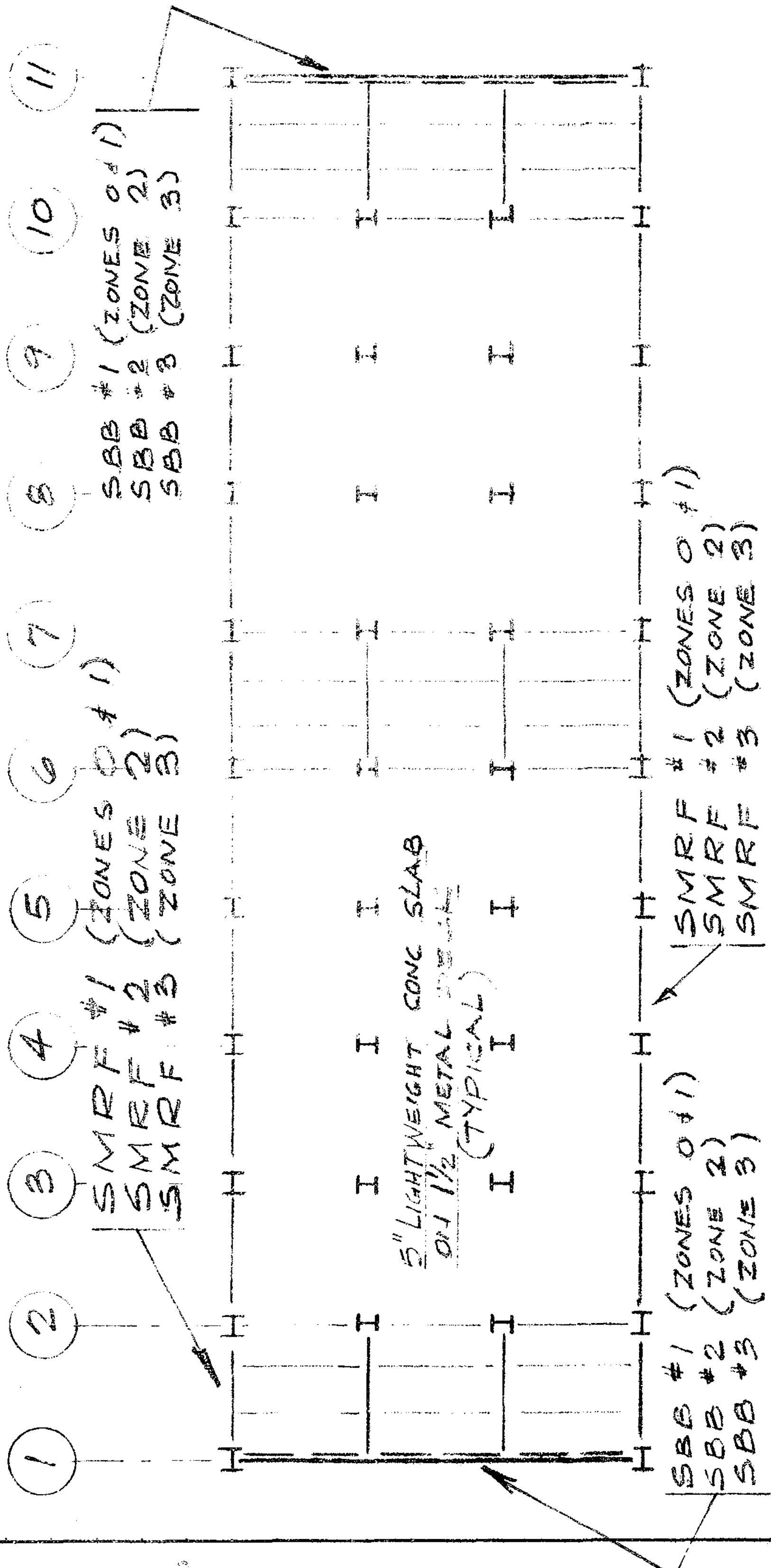


TYPICAL FLOOR FRAMING FOR 6 STORY BUILDING

ZONE 4

FIGURE 11

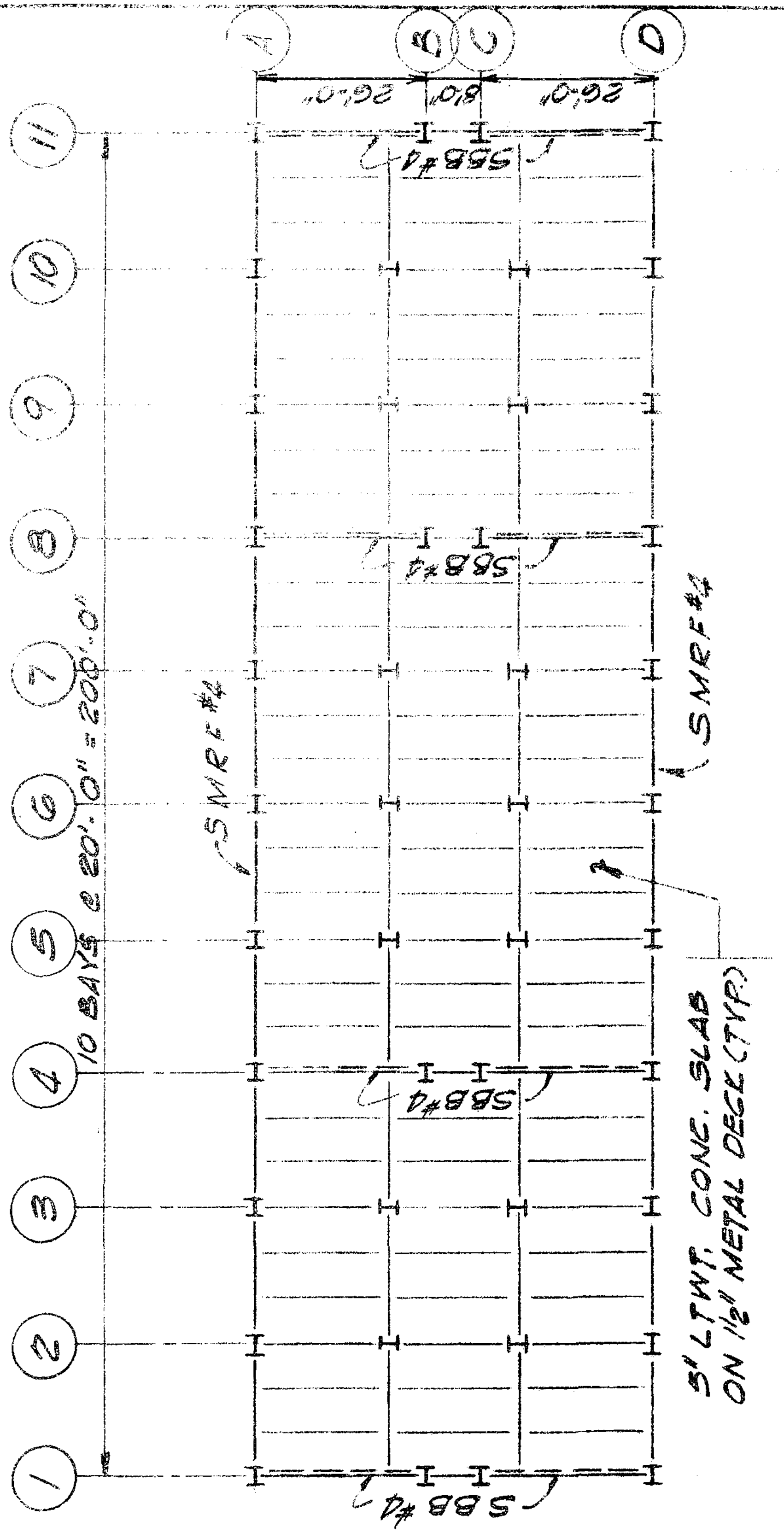
LeMessurier Associates, inc	Subject MIT EARTHQUAKE STUDY	Made by AW	Job No. 7941
		Checked by	Date
		Approved by	Sheet No.



TYPICAL FLOOR FRAMING FOR 17 STORY STEEL BUILDING

FIGURE 12

LeMessurier Associates, Inc	Subject MIT EARTHQUAKE STUDY	Made by AW	Job No. 7941
		Checked by	Date
		Approved by	Sheet No.



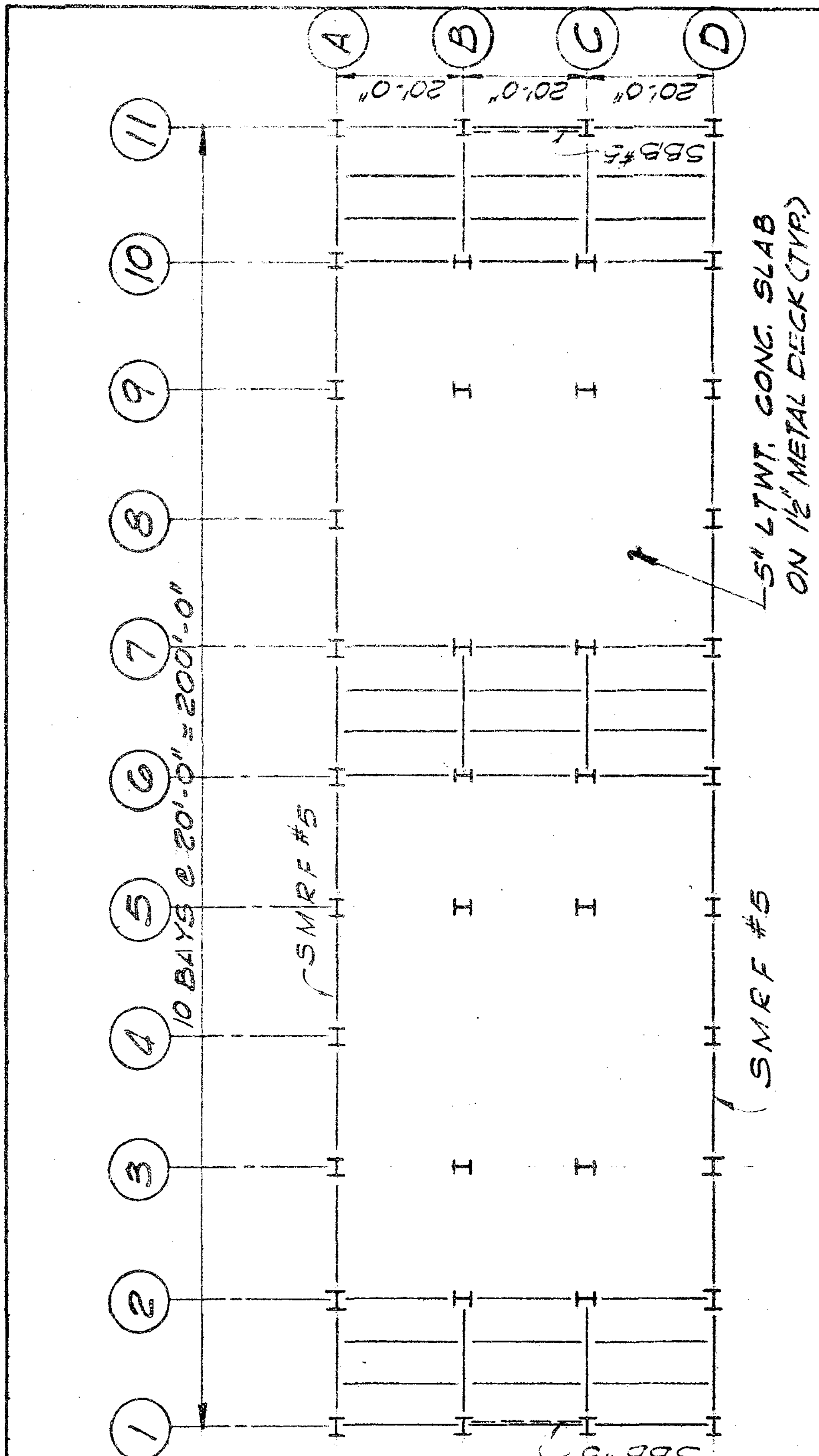
TYPICAL FLOOR FRAMING FOR 17 STORY STEEL BUILDING

ZONE 4

FIGURE 13

LeMessurier Associates, Inc

Subject	MIT EARTHQUAKE STUDY
Made by	AW
Checked by	
Approved by	
Job No.	7941
Date	
Sheet No.	

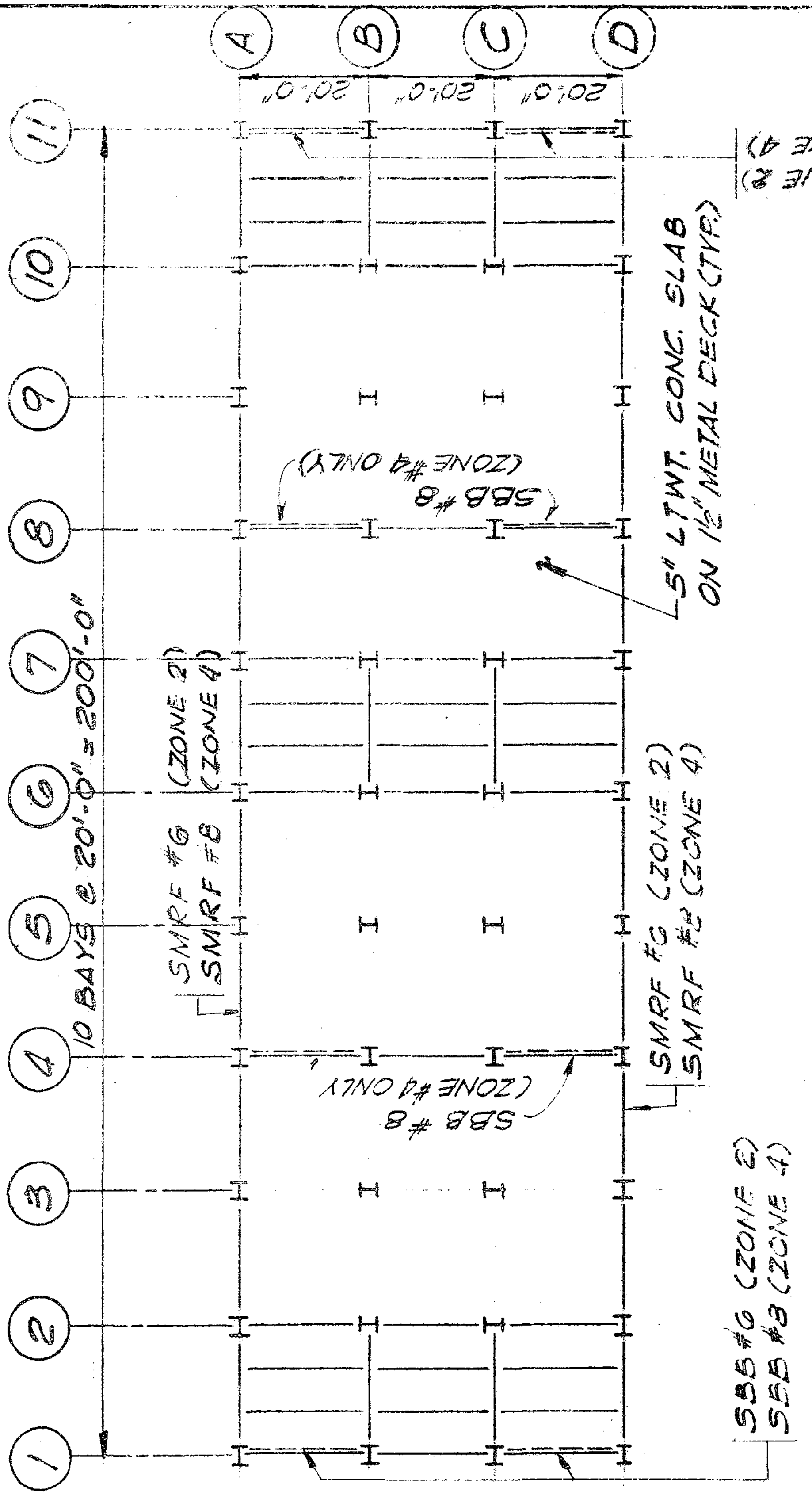


TYPICAL FLOOR FRAMING FOR 11 STORY BUILDING

ZONES 0 & 1

FIGURE 14

LeMessurier Associates, Inc	Subject MIT EARTHQUAKE STUDY	Made by AW	Job No. 7941
		Checked by	Date
		Approved by	Sheet No.



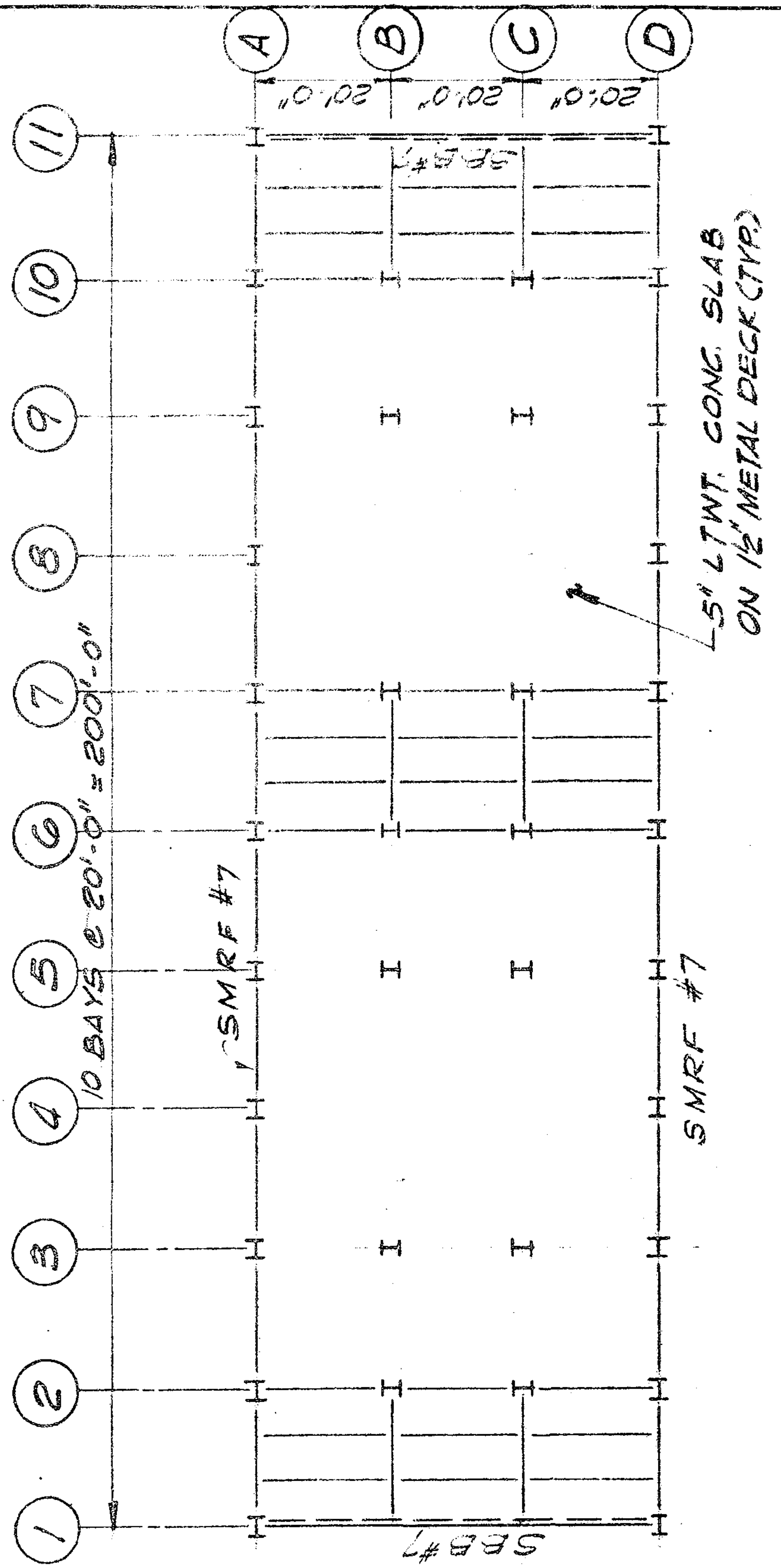
TYPICAL FLOOR FRAMING FOR 11 STORY BUILDING

ZONES 2 & 4

FIGURE 15

LeMessurier Associates, Inc

Subject MIT	Made by AW	Job No. 7941
EARTHQUAKE	Checked by	Date
STUDY	Approved by	Sheet No.

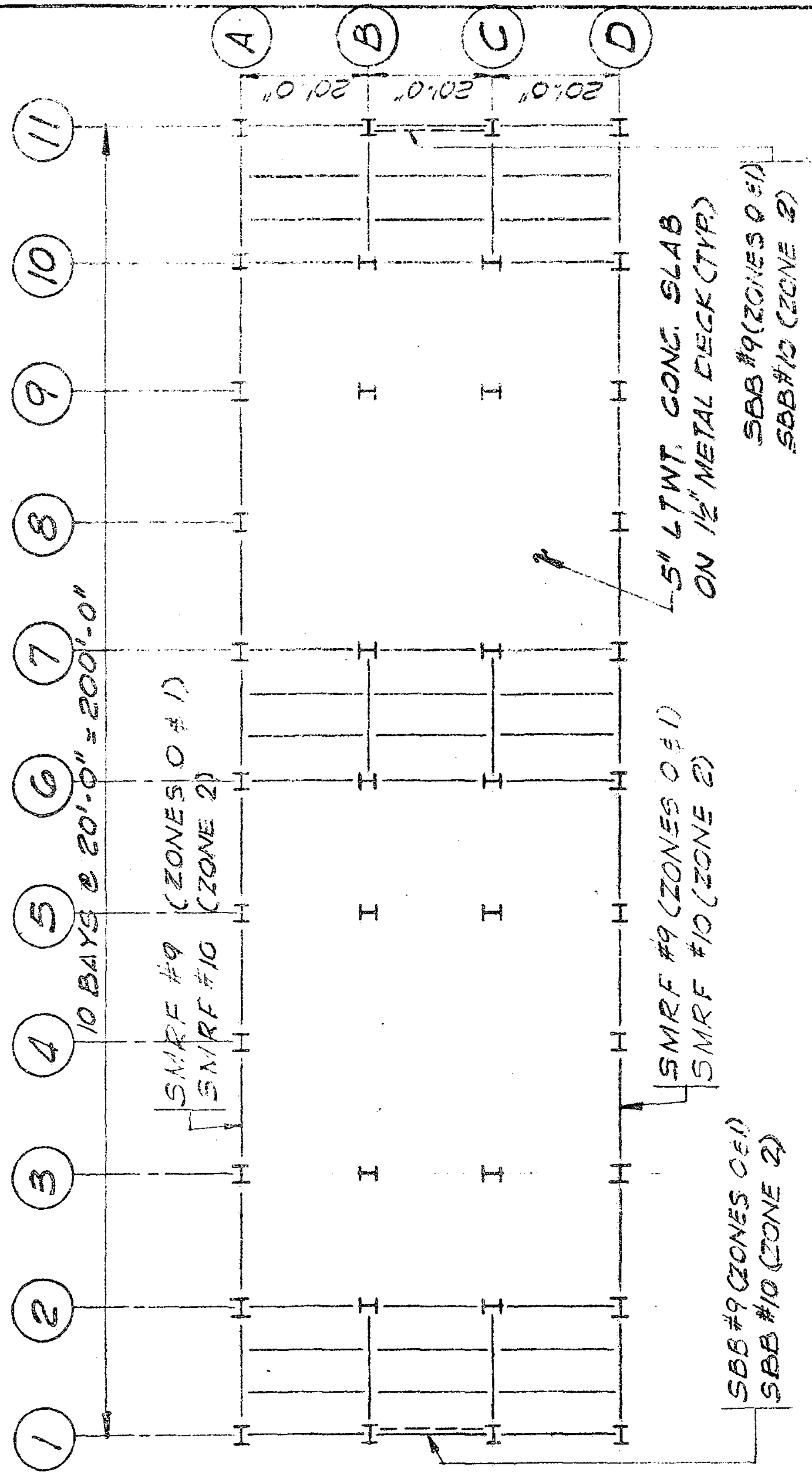


TYPICAL FLOOR FRAMING FOR 11 STORY BUILDING

ZONE 3

FIGURE 1G

LeMessurier Associates, Inc	Subject MIT EARTHQUAKE STUDY	Made by AW Checked by Approved by	Job No. 7941 Date Sheet No.

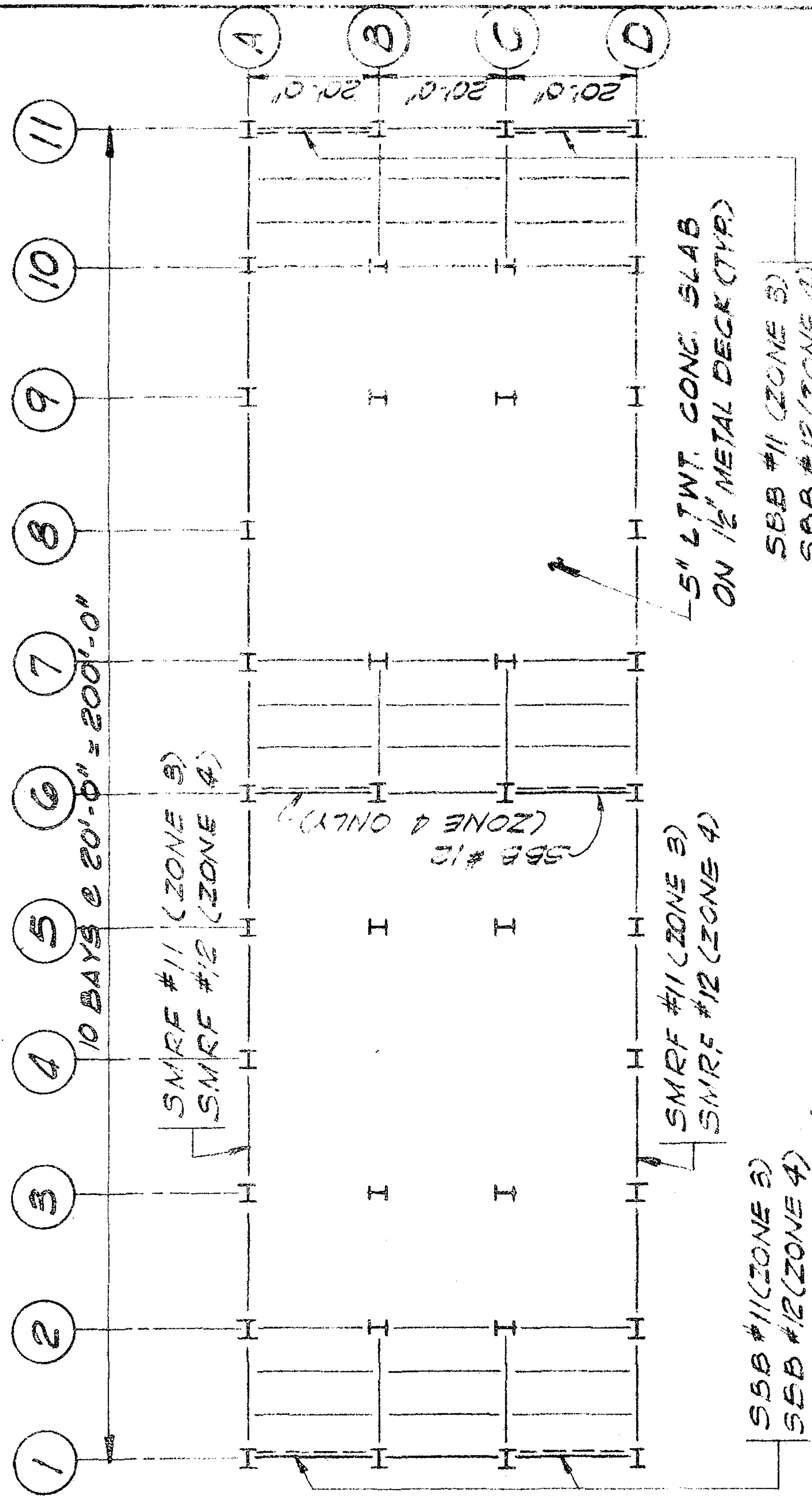


TYPICAL FLOOR FRAMING FOR 6 STORY BUILDING

ZONES 0, 1, 2

FIGURE 17

LeMessurier Associates, Inc	Subject MIT EARTHQUAKE STUDY	Made by AW	Job No. 7941
		Checked by	Date
		Approved by	Sheet No.

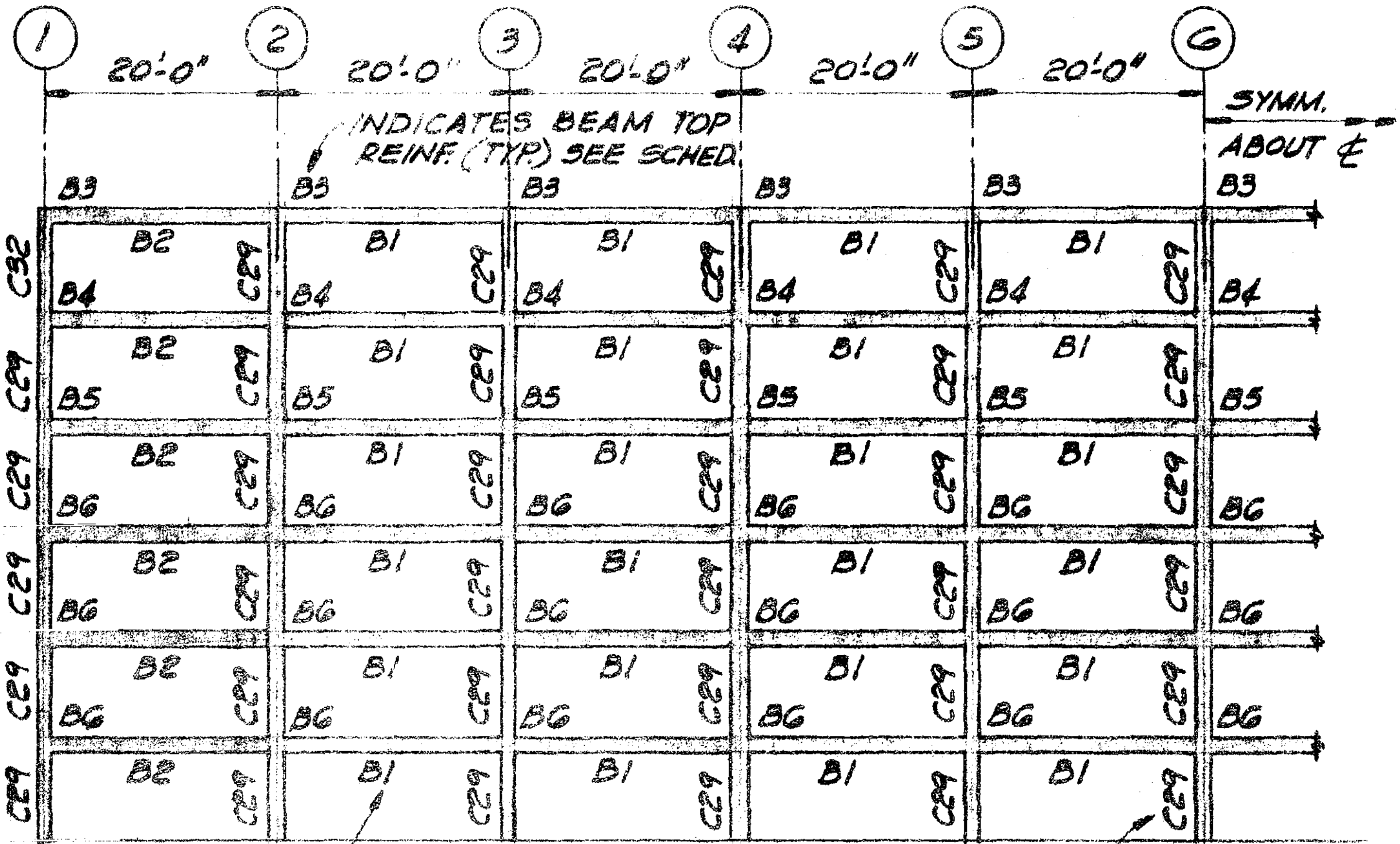


TYPICAL FLOOR FRAMING FOR 6 STORY BUILDING

ZONES 3 & 4

FIGURE 18

FIGURE 19



ELEVATION OF CONCRETE MOMENT RESISTING FRAME #25 (CMRF #25)

(REFER TO PLAN FIGURE 10 FOR LOCATION)

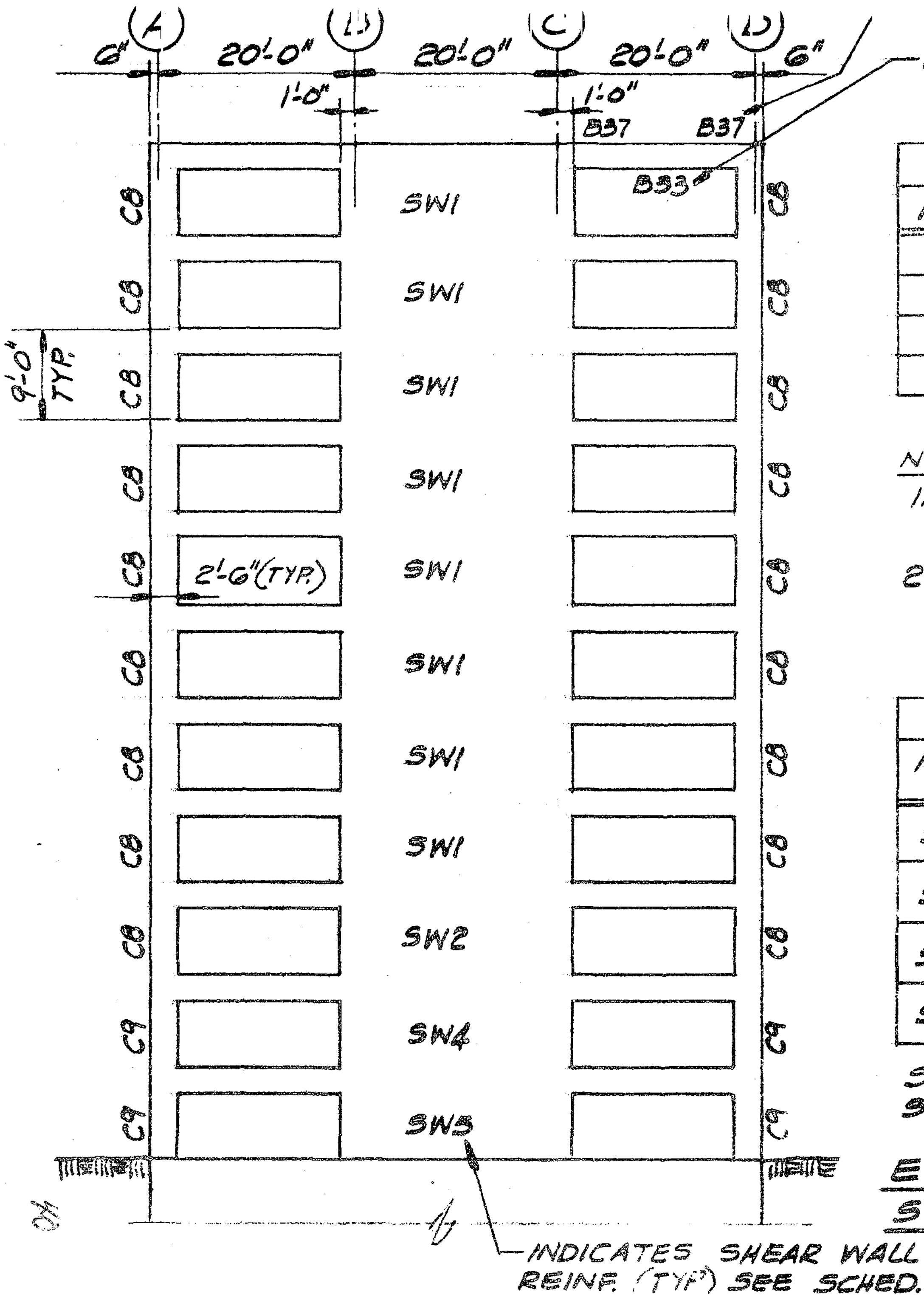
(DESIGNED FOR ZONE 2 EARTHQUAKE & GRAVITY LOADS)

BEAM & COLUMN SCHEDULE			
MARK	REINF.	M_u (FT.K)	M_y (FT.K)
B1	2-#7	61.2	68.0
B2	2-#8	78.6	87.5
B3	2-#9	97.2	108.0
B4	4-#7	114.0	126.5
B5	2-#10	119.3	132.8
B6	3-#9	136.8	152.0
C29	4-#7		
C32	4-#10		

NOTE:

1. ALL BEAMS IN THIS FRAME ARE 12" WIDE x 16" DEEP W/CONC. STRENGTH f'_c = 4000 PSI.
2. ALL COLUMNS IN THIS FRAME ARE 12" x 16" W/CONC. STRENGTH f'_c = 4000 PSI.

FIGURE 20



BEAM & COLUMN SCHEDULE			
MARK	REINF.	M_U (FT.^{K})	M_Y (FT.^{K})
B33	2-#10	225.0	250.0
B37	4-#10	318.0	353.0
C8	8-#7		
C9	12-#8		

NOTES:

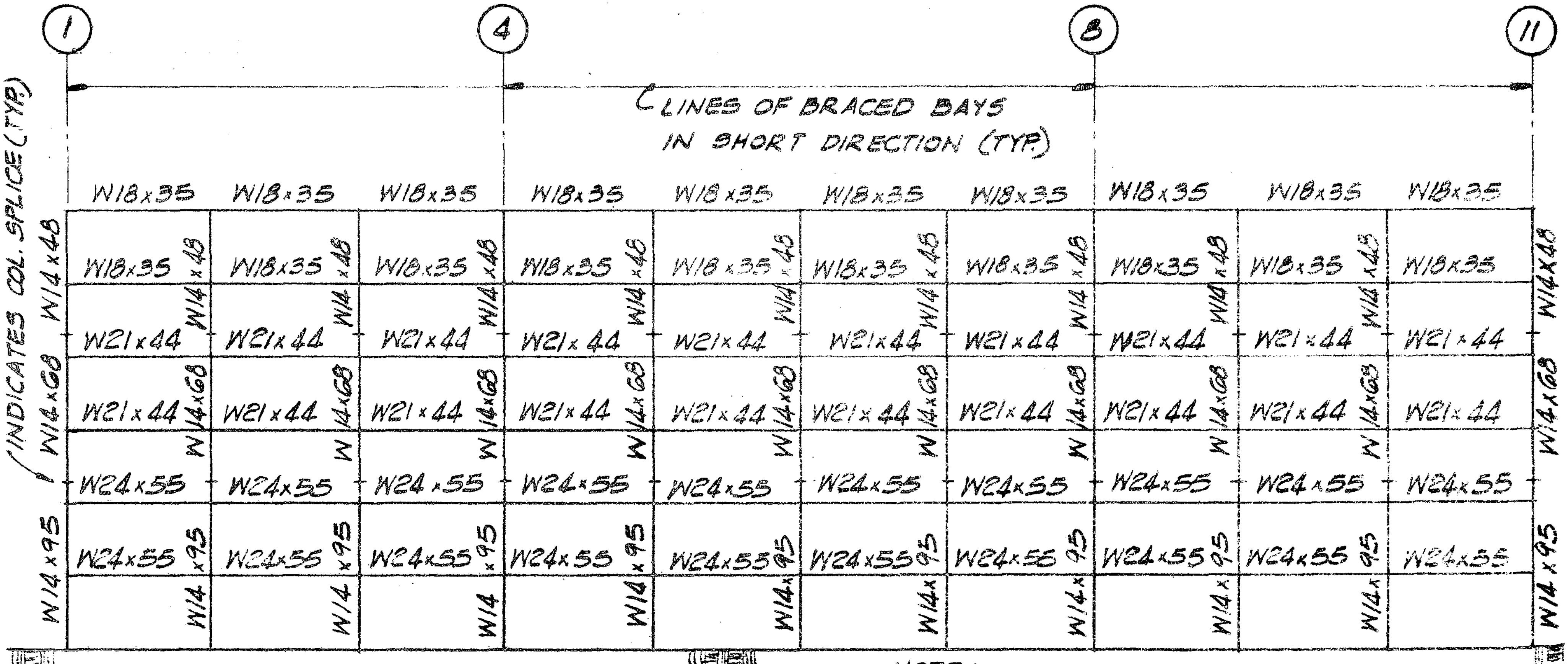
1. ALL BEAMS SHOWN ARE 12" WIDE x 24" DEEP W/CONC. STRENGTH f'_c = 4000 PSI.
2. ALL COL'S. SHOWN ARE 12"x30" W/CONC. STRENGTH f'_c = 4000 PSI.

SHEAR WALL SCHEDULE		
MARK	VERT. REINF. IN EACH END	REMAINDER OF REINF.
SW1	2-#8	#4@12" H.E.F. #4@12" V.E.F.
SW2	4-#8	
SW4	3-#14S	
SW5	4-#14S	

SHEAR WALL IS 1'-0" x 22'-0" W/CONC. STRENGTH f'_c = 4000 PSI.

ELEVATION OF CONCRETE SHEAR WALL #12 (SW#12)

(DESIGNED FOR ZONE 3 EARTHQUAKE & GRAVITY LOADS SEE FIG. 8 FOR PLAN)



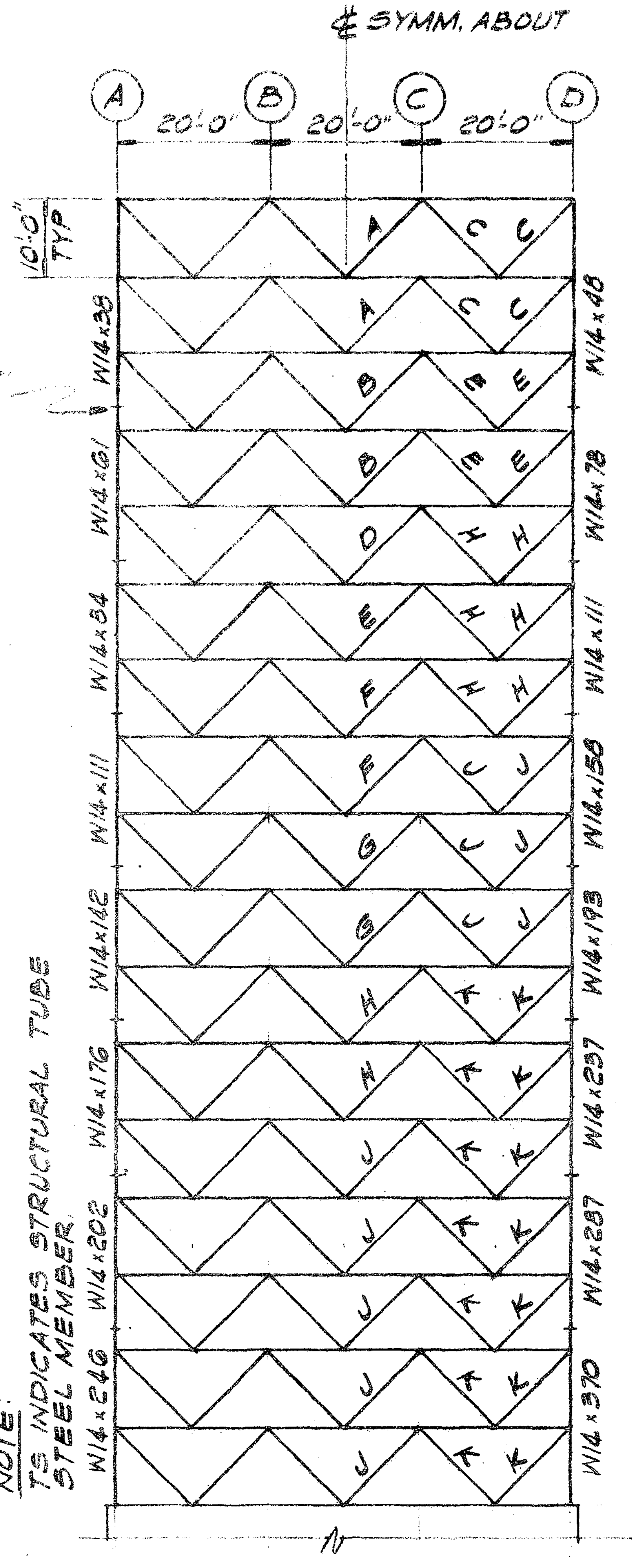
NOTE:
ALL STRUCT. STEEL TO BE ASTM A36.

ELEVATION OF STEEL MOMENT RESISTING FRAME #12 (SMRF #12)

(DESIGNED FOR EARTHQUAKE ZONE 4 & GRAVITY LOADS)
REFER TO FIG. 18 FOR PLAN LOCATION.

FIGURE 21

FIGURE 22



LEGEND

A = TS 4x4x1/4
B = TS 4x4x3/8
C = TS 4x4x1/2
D = TS 6x4x3/8
E = TS 6x4x1/2
F = TS 8x4x3/8
G = TS 8x4x1/2
H = TS 8x6x5/16
J = TS 8x6x3/8
K = TS 8x6x1/2

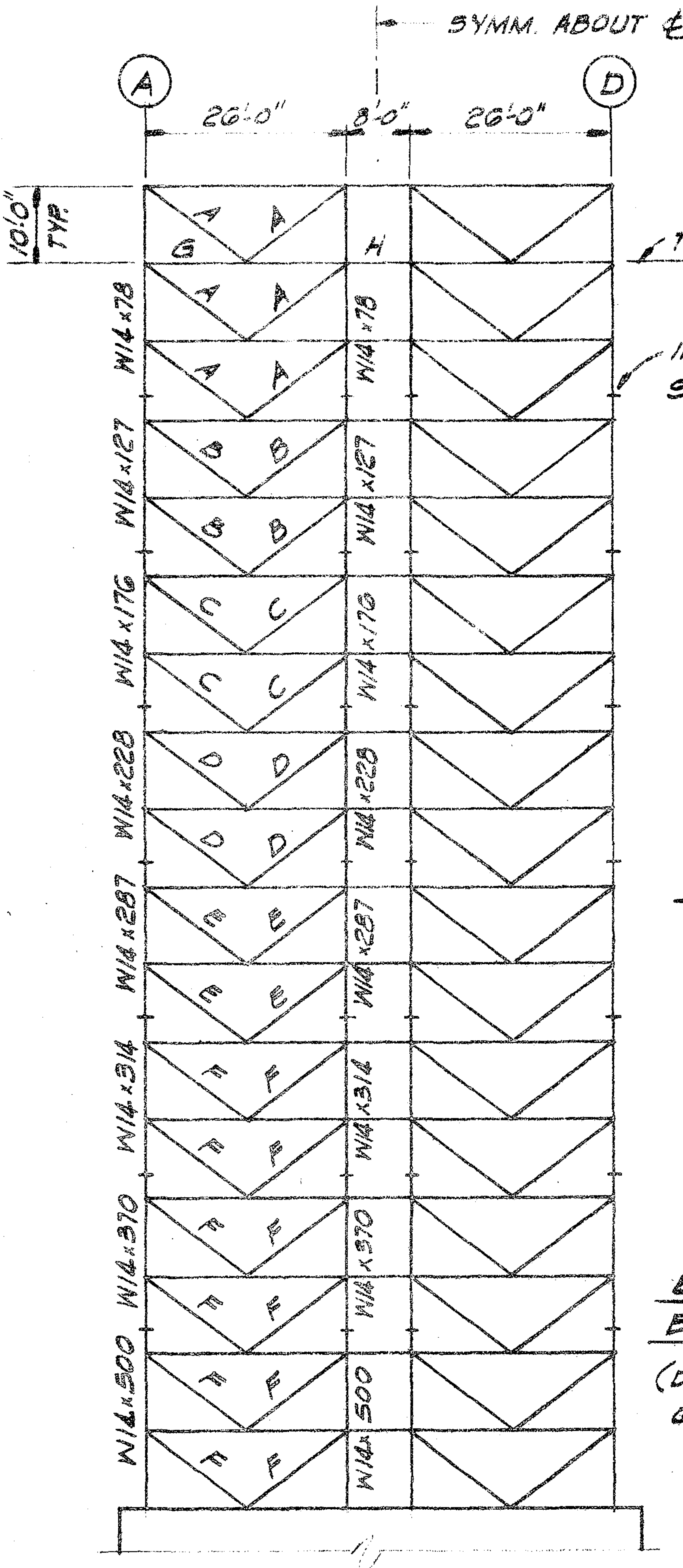
NOTE:
TS INDICATES STRUCTURAL TUBE STEEL MEMBER.

ELEVATION OF STEEL BRACED BAY #3 (SBB #3)

(DESIGNED FOR ZONE 3 EARTHQUAKE & GRAVITY LOADS)

SEE FIGURE 12 FOR PLAN LOCATION

FIGURE 23



LEGEND

A = TS 8x4x1/4
B = TS 8x4x5/16
C = TS 8x4x3/8
D = TS 8x4x1/2
E = TS 8x6x1/4
F = TS 8x6x5/16
G = W18x50
H = W14x30

NOTE:

TS INDICATES STRUCTURAL STEEL TUBE MEMBER.

ELEVATION OF STEEL BRACED BAY #4 (SBB #4)

(DESIGNED FOR ZONE 4 EARTHQUAKE & GRAVITY LOADS)

SEE FIG. 13 FOR PLAN LOCATION.

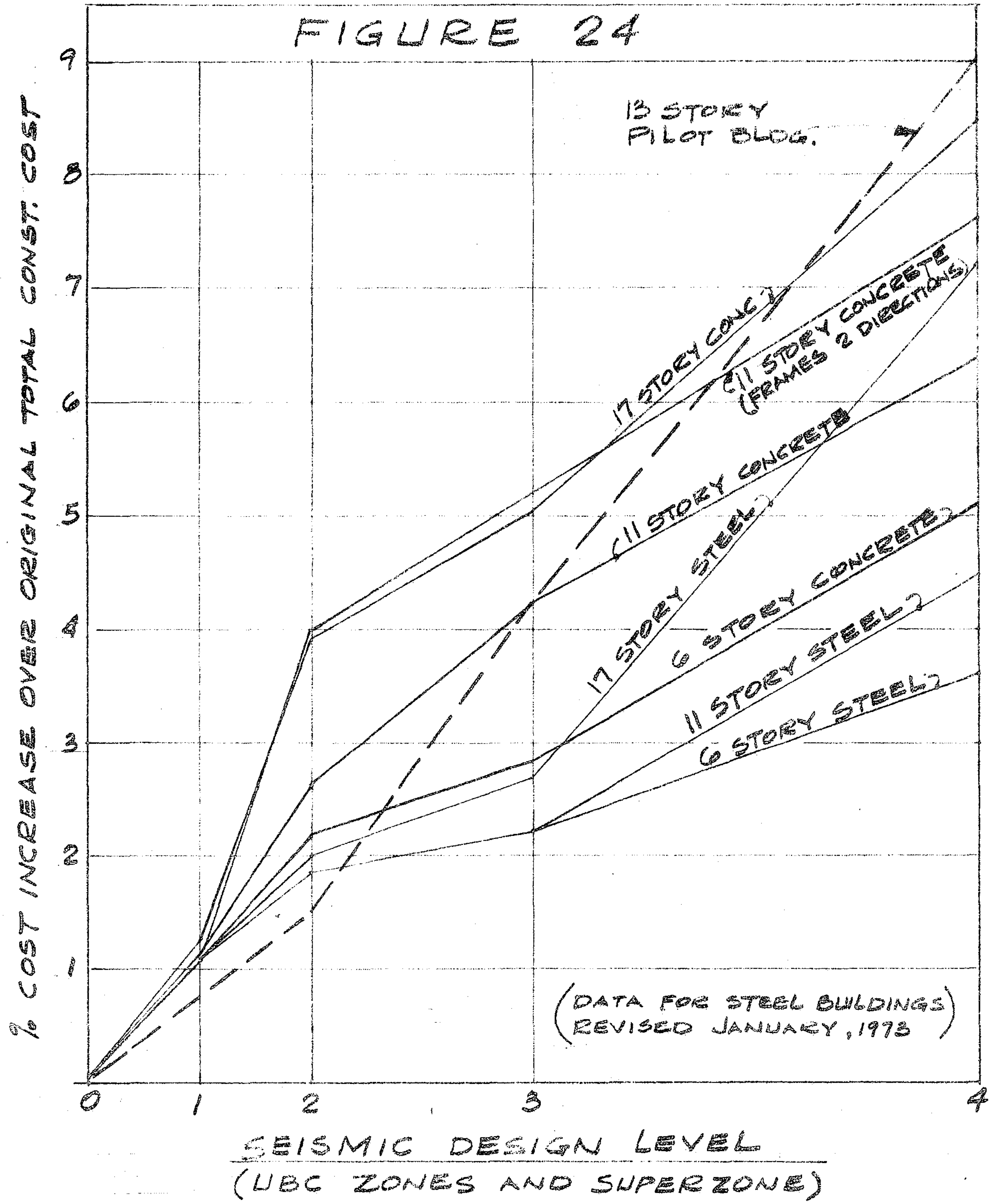


FIGURE 24
13 STORY PILOT BLDG.
17 STORY CONC.
11 STORY CONCRETE (FRAMES 2 DIRECTIONS)
11 STORY CONCRETE
17 STORY STEEL
6 STORY CONCRETE
11 STORY STEEL
6 STORY STEEL
(DATA FOR STEEL BUILDINGS REVISED JANUARY, 1973)
% COST INCREASE OVER ORIGINAL TOTAL CONST. COST
SEISMIC DESIGN LEVEL
(UBC ZONES AND SUPERZONE)
0
1
2
3
4
5
6
7
8
9

FIGURE 25

SAMPLE COMPUTATIONS SHOWING MASSES USED IN DETERMINING EARTHQUAKE FORCES IN THE CONCRETE BUILDINGS

LOADS TO TYPICAL INTERIOR COLUMN

TRIBUTARY AREA = 20 × 20 = 400 SF

DEAD LOADS

DRYWALL PARTITIONS =	14# × 400 =	5600#
MASONRY PARTITIONS =	55# × 8.5' × 35' =	16400#
7" FLAT SLAB =	88# × 400 =	35200
COLUMN WT =	300# × 8.5' =	2600
		59800#

LOADS TO TYPICAL EXTERIOR COLUMN

TRIBUTARY AREA (TO FACE OF SPANDREL BEAM) = 9.5 × 20 = 190 SF

DEAD LOADS

DRYWALL PARTITIONS =	14# × 190 =	2600#
INTERIOR BLOCK =	55# × 8.5 × 8.5' =	4000#
EXTERIOR MASONRY =	83# × 6.5 × 3/4 × 18 =	7300
7" SLAB =	88# × 190 =	16700#
SPANDREL BEAM =	18' × 2.5' × 150# =	6800#
COL WT =		3000
		40400

LOADS FOR TYPICAL FLOOR

18 INTERIOR COLUMNS	@ 59.8K =	1078K
22 EXTERIOR COLUMNS	@ 40.4K =	887
4 CORNER COLUMNS	@ 27.2K =	109
	TOTAL FLOOR LOAD =	2074K

EARTHQUAKE FORCES FOR 17 STORY CONCRETE BUILDING LONG DIRECTION MOMENT FRAMES · ZONE 3

$T = 0.1N = .1(17) = 1.7$ SEC

$C = \frac{0.05}{\sqrt[3]{T}} = \frac{0.05}{\sqrt[3]{1.7}} = \frac{0.05}{1.19} = 0.042$

$Z = 1.0$ (ZONE 3) $K = 0.67$ (DUCTILE FRAME)

$V = ZKCW$

$V = (1.0)(0.67)(0.042)(17)(2074)$

$= 993^K$ (BASE SHEAR)

TABLE - 1

SUPERSTRUCTURE COST ESTIMATES
(COST/SQUARE FOOT AND TOTAL STRUCTURAL COST)

	ZONE 0	ZONE 1	ZONE 2	ZONE 3	ZONE 4
CONCRETE BUILDINGS					
17 STORY (S.W.)	5.25	5.29	6.05	6.37	7.27
	$1,070,000	$1,079,000	$1,235,000	$1,300,000	$1,483,000
11 STORY (S.W.)	4.99	5.01	5.42	5.88	6.43
	$658,700	$661,300	$715,400	$775,600	$848,400
11 STORY (FR.)	5.19	5.19	6.01	6.35	6.93
	$685,600	$685,600	$793,300	$838,400	$921,500
6 STORY (S.W.)	5.05	5.05	5.35	5.55	6.14
	$363,400	$363,400	$385,800	$400,000	$442,400
STEEL BUILDINGS					
17 STORY	3.69	3.69	3.95	4.15	5.35
	$752,000	$752,000	$806,400	$847,000	$1,092,000
11 STORY	3.51	3.51	3.73	3.83	4.42
	$463,000	$463,000	$492,500	$505,000	$584,000
6 STORY	3.31	3.31	3.54	3.63	3.97
	$238,000	$238,000	$254,600	$261,500	$286,100

TABLE - 2

NON-STRUCTURAL MASONRY COST
DOLLARS PER SQUARE FOOT

	ZONE 0	ZONE 1	ZONE 2	ZONE 3	ZONE 4
MASONRY	$2.52	$2.82	$2.82	$2.82	$2.87

TABLE - 3

SQUARE FOOT ESTIMATES AND % OF TOTAL CONSTRUCTION COST OF ITEMS AFFECTED BY SEISMIC DESIGN

	ZONE 0	ZONE 1	ZONE 2	ZONE 3	ZONE 4
CONCRETE BUILDINGS					
17 STORY (S.W.)	7.77	8.11	8.87	9.19	10.14
	% 27.75	% 28.96	% 31.68	% 32.82	% 36.21
11 STORY (S.W.)	7.51	7.83	8.24	8.70	9.30
	% 26.82	% 27.96	% 29.43	% 31.07	% 33.21
11 STORY (FR.)	7.71	8.01	8.83	9.17	9.85
	% 27.54	% 28.61	% 31.54	% 32.75	% 35.18
6 STORY (S.W.)	7.57	7.87	8.17	8.37	9.01
	% 27.04	% 28.11	% 29.17	% 29.89	% 32.18
STEEL BUILDINGS					
17 STORY	6.21	6.51	6.77	6.97	8.22
	% 22.18	% 23.25	% 24.18	% 24.89	% 29.36
11 STORY	6.03	6.33	6.55	6.65	7.29
	% 21.54	% 22.61	% 23.39	% 23.75	% 26.04
6 STORY	5.83	6.13	6.36	16.45	6.84
	% 20.82	% 21.89	% 22.71	% 23.04	% 24.43

(DATA FOR STEEL BUILDINGS REVISED JANUARY, 1973)

TABLE - 4A			
17 STORY CONCRETE BUILDING 153'-0" HT.			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	1.21" l/1510 (*)	2.23" l/820 (*)	
1	2.30" l/800	2.36" l/780	
2	2.76" l/665	1.87" l/980	
3	3.13" l/585	2.17" l/850	
4	3.30" l/555	2.97" l/620	
11 STORY CONCRETE BUILDING 110'-0" HT.			
ZONE	LONG DIRECTION	SHORT DIRECTION (S.W.)	SHORT DIRECTION (M.F.)
0	0.65" l/2030 (*)	0.58" l/2270 (*)	
1	1.28" l/1030	0.65" l/2030	
2	2.62" l/504	1.29" l/1020	1.97" l/670
3	2.36" l/560	0.90" l/1470	1.33" l/990
4	3.52" l/375	1.15" l/1150	1.85" l/710
6 STORY CONCRETE BUILDING 54'-0" HT.			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	0.28" l/2300 (*)	0.073" l/8900 (*)	
1	0.55" l/1180	0.088" l/7350	
2	1.10" l/590	0.197" l/3300	
3	1.23" l/530	0.365" l/1780	
4	1.13" l/575	0.175" l/3700	

l INDICATES BUILDING HEIGHT

(*) DENOTES BUILDING DRIFT CAUSED BY WIND

NOTE: ALTHOUGH BUILDING DRIFT FOR EARTHQUAKE ZONE 1 IS ALWAYS GREATER THAN THAT FOR WIND (ZONE 0), STRESS REQUIREMENTS ARE USUALLY GREATER FOR WIND (ZONE 0).

TABLE - 4B

17 STORY STEEL BUILDING 170'-0" HT.			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	1.80" ℓ/1133 (✱)		
1	3.17" ℓ/643		
2	6.3" ℓ/324	2.04" ℓ/1000	
3	6.77" ℓ/300	3.25" ℓ/630	
4	6.9" ℓ/295	5.83" ℓ/350	
11 STORY STEEL BUILDING 110'-0" HT.			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	1.4" ℓ/940 (✱)		
1	1.58" ℓ/835		
2	2.83" ℓ/465	3.93" ℓ/336	
3	4.17" ℓ/316		
4	4.45" ℓ/297	4.02 ℓ/328	
6 STORY STEEL BUILDING 60'-0" HT.			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	0.40" ℓ/1800 (✱)		
1	1.08" ℓ/670		
2	1.74" ℓ/415		
3	2.40" ℓ/300		
4	2.09 ℓ/344	1.60" ℓ/450	

ℓ INDICATES BUILDING HEIGHT

(✱) DENOTES BUILDING DRIFT CAUSED BY WIND

NOTE: ALTHOUGH TOTAL BUILDING DRIFT FOR EARTHQUAKE ZONE 1 IS ALWAYS GREATER THAN THAT FOR WIND (ZONE 0), STRESS REQUIREMENTS ARE USUALLY GREATER FOR WIND (ZONE 0).

TABLE - 5

INDEX TO COMPUTATIONS	
VOLUME NO.	CONTENTS
1	GENERAL COMPUTATIONS. TYPICAL BUILDING LOADS, STEEL COLUMN DATA COMPUTATIONS.
2	CONCRETE BEAM AND COLUMN DATA.
3	17 STORY CONCRETE BUILDINGS. ANALYSIS OF LONG DIRECTION MOMENT FRAMES ZONE 0-4.
4	17 STORY CONCRETE BUILDINGS. BEAM AND COLUMN DESIGN FOR LONG MOMENT FRAMES.
5	17 STORY CONCRETE BUILDINGS. SHEAR WALL DESIGN.
6	11 STORY CONCRETE BUILDINGS. COMPUTER ANALYSIS OF LONG FRAMES, SHEAR WALLS ZONES 0-4.
7	11 STORY CONCRETE BUILDINGS. MOMENT FRAMES IN SHORT DIRECTION.
8	6 STORY CONCRETE BUILDINGS. COMPUTER ANALYSIS FOR LONG FRAMES, SHEAR WALLS.
9	17 STORY STEEL BUILDINGS. COMPUTER ANALYSIS FOR LONG FRAMES AND DESIGN - ZONES 1 AND 2.
10	17 STORY STEEL BUILDINGS. COMPUTER ANALYSIS FOR LONG FRAMES AND DESIGN - ZONES 3 AND 4.
11	17 STORY STEEL BUILDINGS. SHORT DIRECTION BRACED BAYS ZONES 0-3.
12	17 STORY STEEL BUILDINGS. SHORT DIRECTION BRACED BAYS ZONE 4.
13	11 STORY STEEL BUILDINGS. COMPUTER ANALYSIS OF LONG DIRECTION MOMENT FRAMES.
14	11 STORY STEEL BUILDINGS. SHORT DIRECTION BRACED BAYS.
15	6 STORY STEEL BUILDINGS. LONG DIRECTION MOMENT FRAMES AND BRACED BAYS.
16	COST DATA.

TABLE - 6A			
INDEX OF DRAWING ELEVATIONS SHOWING STRUCTURAL SYSTEMS DESIGNED FOR WIND OR EARTHQUAKE LOADS			
17 STORY CONCRETE BUILDINGS			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	CMRF #1	CSW #1	
1	CMRF #2	CSW #2	
2	CMRF #3	CSW #3 & #4	
3	CMRF #4	CSW #5 & #6	
4	CMRF #5	CSW #7 & #8	
11 STORY CONCRETE BUILDINGS			
ZONE	LONG DIRECTION	SHORT DIRECTION (S.W.)	SHORT DIRECTION (M.F.)
0	CMRF #9	CSW #9	CMRF #6, #7 & #8
1	CMRF #10	CSW #10	CMRF #6, #7 & #8
2	CMRF #14	CSW #11	CMRF #11, #12 & #13
3	CMRF #18	CSW #12 & #13	CMRF #15, #16 & #17
4	CMRF #22	CSW #14 & #15	CMRF #19, #20 & #21
6 STORY CONCRETE BUILDINGS			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	CMRF #23	CSW #16	
1	CMRF #24	CSW #17	
2	CMRF #25	CSW #18	
3	CMRF #26	CSW #19	
4	CMRF #27	CSW #20 & #21	

NOTATION:
CMRF - CONCRETE MOMENT RESISTING FRAME
CSW - CONCRETE SHEAR WALL

TABLE - 6B

INDEX OF DRAWING ELEVATIONS SHOWING STRUCTURAL SYSTEMS DESIGNED FOR WIND OR EARTHQUAKE LOADS

17 STORY STEEL BUILDING			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	SMRF #1	SBB #1	
1	SMRF #1	SBB #1	
2	SMRF #2	SBB #2	
3	SMRF #3	SBB #3	
4	SMRF #4	SBB #4	
11 STORY STEEL BUILDING			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	SMRF #5	SBB #5	
1	SMRF #5	SBB #5	
2	SMRF #6	SBB #6	
3	SMRF #7	SBB #7	
4	SMRF #8	SBB #8	
6 STORY STEEL BUILDING			
ZONE	LONG DIRECTION	SHORT DIRECTION	
0	SMRF #9	SBB #9	
1	SMRF #9	SBB #9	
2	SMRF #10	SBB #10	
3	SMRF #11	SBB #11	
4	SMRF #12	SBB #12	

NOTATION:
SMRF - STEEL MOMENT RESISTING FRAME
SBB - STEEL BRACED BAY

TABLE - 7			
BEAM & COLUMN SIZES FOR CONCRETE BLDGS.			
17 STORY	ZONE	BEAM SIZE	COL. SIZE
	0 & 1	12 × 16	12 × 30
	2	12 × 20	12 × 30
	3	12 × 24	12 × 30
	4	12 × 30	12 × 36
11 STORY	0 & 1	12 × 16	12 × 20
	2	12 × 16	12 × 20
	3	12 × 24	12 × 30
	4	12 × 26	12 × 36
6 STORY	0 & 1	12 × 14	12 × 16
	2	12 × 16	12 × 16
	3	12 × 20	12 × 20
	4	12 × 26	12 × 30