

High-Rise System Developments in Concrete

By Hal Iyengar*

1. Introduction

Developments in concrete high-rise buildings have undergone a dramatic evolutionary change in the last twenty-five years. Simple systems, such as shear wall buildings that were exclusively used prior to the 1960's, have been transformed to a considerable number of systems suitable for commercial, residential and mixed uses up to 80 stories or more. In many cases, the reinforced concrete elements have been combined with structural steel to produce a more effective mixed system. The progressive development of higher-strength concrete together with advancement in mechanization of formwork and placement methods have fueled this systems development. The advantages offered by concrete, with respect to moldability for shaping as well as massivity and rigidity for structural purposes, have been exploited to the fullest. This paper is developed somewhat as a review of the systems developments that have occurred in the past twenty-five years together with discussion of some current applications.

2. Special Merits of Concrete

a. Lateral Load Resistance:

Concrete has been said to be the cheapest material that one could buy on a global basis which can readily be molded and so placed in the building to accept lateral forces. The properties of concrete that are most attractive are its rigidity and its ability to be cast into different types of structural elements. Shear wall elements and/or punched wall or framed-tube elements with monolithically-cast beam-column joints are primary elements used for lateral load resistance. It is well known that equivalent cantilever-type structures are most efficient at resisting wind forces and developing the needed lateral stiffness. Such cantilevers can be produced by placing walls around building cores so that the core wall system can be considered a cantilever tube. Similarly, if the punched wall is placed on the exterior, the entire exterior form can be utilized as a cantilever tube system. The proportionality of the members of the punched tube, the wall thickness and the shape of the wall can all be controlled readily to fit any particular structural requirements and this aspect of customizing to meet different requirements makes concrete particularly suitable for the lateral load resistive system. In mixed steel-concrete systems, concrete is used in vertical elements only while horizontal framing is

* Partner & Director of Structural Engineering, Skidmore, Owings & Merrill, Chicago, Illinois

obtained from structural steel. The mechanization of formwork for the construction of vertical elements has significantly contributed to economy and speed of these systems. Panelized shear wall formwork, slip forms, gang forms and automated form lifters all contribute to this construction efficiency.

The reliable production of higher-strength concrete has extended the application to taller structures in the range of 50 to 80 stories. Concrete strengths in the range of 6,000 to 8,000 PSI are commonly used, whereas in some cases, strengths up to 12,000 PSI have been utilized. The aspect of higher elastic modulus with higher strength again contributes to improved rigidity.

In tall structures beyond 50 stories, the aero-dynamic behavior and the resultant motion perception are important structural considerations. Pure steel buildings tend to become lighter which, for the same fundamental period, will cause higher motion perception levels. Significant perception improvements can be obtained with a larger mass in the system for the same fundamental period of vibration. In addition, the higher mechanical damping of concrete assists in the reduction of perception levels. Because of these properties, concrete is being more readily adopted in taller structures. However, massivity of members may cause space and planning inefficiency in structures beyond about 80 stories, unless even higher-strength concrete is available in the future.

b. Floor Framing

Earlier floor framings were beam and slab arrangements cast in place with board forms. These were labor intensive and slow to construct. These were soon replaced by pan joist and waffle joist systems which were standardized into certain sizes and depths. Spans up to about 30 ft. were possible, after which the framing was very massive. Initially, the pans were of metal and were either 20 or 30-inches wide which limited their range of applicability. The same type of system is now made more efficient and lighter by use of molded fiberglass form pans which can readily be adapted to spacings up to 7'-6". The pans can be fabricated into the entire spans and can either be lifted into place or mounted on table forms. This development alone has made high-rise floor systems more economical and adaptable. Post-tensioning of ribs in this system is being used more frequently which further reduces the depth and materials involved. With these developments, spans up to 40 ft. are possible with lighter floor framing.

Flat plates and flat slabs are obviously unique to concrete construction and they are advantageous where minimum floor-to-floor height is required. Flat plates are typically used for apartment buildings where

the flat surface of the flat plate is merely painted to form the finished ceiling. Flat slabs with drop panels with or without capitals are used in garages, warehouses and various such applications.

c. Architectural Concrete

The integral fire protection capability and the ability to be finished into a tough weatherproof surface makes exposed concrete systems possible. The ability to be molded into different shapes offers concrete special advantages in various articulated architectural expressions. In high-rise buildings, a significant cost is that of cladding and the window wall. Architectural concrete offers a more economical alternative. In many instances, special exposed aggregates or color-controlled concrete has been utilized. The recent development of durable epoxy paint systems further enhances this type of application. Architectural concrete also truly integrates the structure with the enclosure as in the case of the punched wall tubular system or other similar systems. However, the effects of temperature foreshortening of exterior columns need to be considered when used in structures beyond 20-25 stories in extremely cold climates.

3. Types of Structural Systems

Several structural systems that have been utilized in the past twenty-five years are briefly outlined. Fig. 1 shows some of the system possibilities in a chart form with an indication of the range of applicability in height or number of stories.

Frame buildings, which rely on predominant Vierendeel frame action, are suitable only up to about 10-20 stories. This is the forerunner to many newer systems after the 1960's where frame beams were made part of the beam-slab floor framing. In some applications, standard pan joists or waffle slabs were used, with the pans at the frame lines filled in and reinforced for frame beam action. Currently, this frame system is generally used for low-rise buildings in the 4 to 6-story range.

Structural systems which derive all their lateral stiffness and strength from only shear walls are feasible up to about 30 to 40 stories. For higher structures, the wind stresses tend to control the design and, therefore, the increased thickness of the shear walls would reduce the core and structural efficiencies. A primary advantage of this system is the absence of any wind framing requirements outside the core area and the floor may be designed for gravity loads only. If span dimensions on the order of 25 ft. do not reduce the floor rentability, an ideal solution would be a flat plate. The result is a lower floor-framing labor cost and lower floor-to-floor height. The reduced height and cubage of the building also result in window wall and mechanical savings.

Initial versions of shear wall buildings were plane elements connecting columns in two directions. Later versions organized the walls around the central building core so that the core wall system can be considered as an internal cantilever tube.

Frame-shear wall interacting systems were a logical extension of the shear wall system which was generally created by the addition of shear frames on the fascia. The shear wall-frame interaction brought about substantial structural benefits in terms of lateral load resistance and strength.

In this system, the fascia plane frame parallel to the direction of the lateral load was considered in the interaction with the core walls. Many outstanding examples of this system up to 40 stories (Brunswick Building, Chicago) currently exist. A study of some recent efficient buildings indicates that the frame stiffness is large enough to reduce the free cantilever deflection of the wall to about one-third its value after interaction. Since the interaction involves two subsystems, studies for optimum combination should be performed. In general, shear walls should be positioned for large gravity load tributary areas which not only reduce the possibility of uplift, but also increase the capacity to resist wind overturning moments because of the increase permitted in the stresses when wind forces are considered.

Framed-tube structures when properly proportioned will develop high efficiencies. Wide columns and deep beams at spacings on the order of 6 ft. to 10 ft. will develop an effective cantilever mode of behavior. The absence of the core walls is an advantage in core planning and mechanical distributions. The development of the equivalent cantilever tubular system represented a significant milestone in the evolution of tall building systems. While all previous system improvements contributed to extend the range of application of frame-type behavior, the radical departure occurred only when the structure was placed on the perimeter and was so interconnected to act like a three-dimensional cantilever utilizing the entire exterior form. The characteristics of this exterior structure were that of a wall, giving rise to the terminology "tube structure" to designate the silo-like cantilever behavior of this structure. With this innovation, the structure had emerged from the interior to the exterior, thereby significantly impacting the architectural expression of the facade and the shaping of the overall form. The material most readily adaptable to create the wall-like structure was concrete where wide columns and deep beams were cast monolithically in a closely-spaced formation. The structural proposition was one of examining different spacings and member proportions, while that of architecture was one of articulation of the tube character. Many notable structures, such as the One Shell Plaza Building in Houston and others, were built with this principle.

The development of the interior core system as a wall tube and that of the exterior as a framed tube offered opportunities for larger stiffness and,

therefore, higher heights when the two were combined to form a "tube-in-tube" system. In some mixed steel-concrete system forms, this system is used with the floor element replaced with steel framing.

While punched-tube systems offered considerable latitude for application into high-rise concrete buildings, it is well to remember that system developments in structural steel were also occurring. It was then possible to borrow from steel to concrete and vice versa and tailor them for the nature and technology of either concrete or steel. The bundled-tube system of the Sears Tower in Chicago was adaptable to bundling or clustering of concrete tube systems. Such modulations allowed not only clustering, but also vertical profile modulations. They were then suitable for multiple-use structures or structures demanding variable floor sizes. The integration of space and architecture with structure further promoted advances in systems technology.

The exterior-braced, trussed-tube system, which was a unique development in steel tubular buildings such as the John Hancock Center in Chicago, was also adaptable to concrete. The concrete tube character was that of a punched wall. Diagonalization was then achieved by filling in window fenestrations in the path of the diagonal. This diagonalized tube was more effective in stiffness development and when combined with ultra-high strength concrete, it can produce considerable economic benefits. Architecturally, a new vocabulary is created with a true structural expression.

Systems for apartment buildings use flat plates without drop panels almost exclusively. The advantages are lower floor-to-floor height, a flat surface which is merely painted to produce the finished ceiling, better sound isolation between floors, adequate fire resistance between floors and a more economical type of construction. Columns or walls can be introduced at closer intervals than in an office building. In most cases, shear walls are sufficient to carry all the lateral loads. Exterior framed tubes, with or without interior walls, are also possible in conjunction with flat plates. Bundled-tube systems with full cross walls, which are only penetrated by the corridors, may be more effectively used in apartment buildings.

4. Recent Concrete High-Rise Systems

a. Olympia Center, Chicago

A recent example of a multiple-use concrete tower in Chicago is that of the 64-story Olympia Center. The program involved 3 floors of below-grade parking, 6 floors of department store, 17 floors of office and 40 stories of apartments. The floor size is 118 ft. x 172 ft. at the bottom which reduces to 60 ft. x 172 ft. at the apartment levels. The building is transitioned from the 5th to the 30th floors by a continuous curve, as shown in Fig. 2. This transition is consistent with the requirements for an exterior framed-tube system in concrete, in that

the moldability of the concrete makes such a curvature possible. Another aspect of a commercial-office-apartment combination is the integration of the facade fenestration requirements in the same system. Commercial space requires very little window fenestration, office space requires regular fenestrations reflecting the modular nature of the office space, and the apartments require combinations of opaque and window spaces reflecting requirements of different rooms. In addition in the Olympia Center, two-story duplex apartments were desired which required removal of the spandrel beams over the living room width. The bearing wall nature of the framed tube permits different degrees of openness desired for different occupancies. A study of the building's structural elevation reveals more solid portions in the commercial area, a consistent framed-tube grid in the office areas and a more flexible grid which begins to open up at the apartment levels with duplexes in the middle of the faces, eventually growing out to the corner areas and finally turning into a bay frame structure at the top few floors. All these grid modulations are within the vocabulary of a concrete framed tube which can readily be molded to different requirements as a bearing wall. The floor framing shows a wide pan joist type of framing for the longer-span office framing to flat plate framing for the apartments.

b. One Magnificent Mile, Chicago

The 57-story One Magnificent Mile project typifies another approach to achieving vertical modulation of spaces--that of bundling or clustering of different tubes. The system once used for the Sears Tower is now used for a multiple-use project in reinforced concrete. The free-form structure is composed of three near-hexagonal, reinforced concrete framed tubes with the highest tube at 57 stories and the others at 49 and 22 stories each (Fig. 3). The arrangement of tubes and their orientation was determined from the site configuration and optimization of vistas to Lake Michigan. The clustering principle was highly useful in molding the overall form around this L-shaped site with a diagonal frontage. The hexagonal shape for each tube created a highly-faceted format for the overall architectural form. The lower 20 stories, which have all three tubes, are occupied by commercial and office space with the rest devoted to apartments. It should be noted that the double and single modules are especially suited for apartment layouts. The floor system is a reinforced concrete, flat slab system in both office and apartment floors. The structure for the tubular lines involves columns at close centers and deep spandrel beams. The wall character of the concrete framed tube again provides the context for a variable facade grid between office and apartment occupancies.

c. Onterie Center, Chicago

Onterie Center is a multi-use, high-rise complex located on the Lake Michigan shore line near Downtown Chicago. It is comprised of two towers, a 58-story main tower with a tapering auxiliary low-rise

building (Fig. 4). The total building with an area of 92,000m² is divided into five distinct areas by function. On the ground floor of the main tower and the connecting low-rise building are the main public lobby and 1,860m² of commercial space. The parking area occupies the basement and four floors above the lobby. Floors 6 through 10 of the tapering base, as well as floors 2 through 11 of the auxiliary tower, provide office space and are organized around two interior atriums. The sky lobby at level one includes a health club, swimming pool, hospitality room and mechanical equipment space. Levels 12 through 58 consist of one, two and three-bedroom apartments, for a total of 593 units.

Mixed-use, high-rise structures demand flexibility in column spacing and core layout. Therefore, maximum structural efficiency and functional flexibility can be achieved by utilizing the exterior frame of the building only for the lateral force-resisting system. In the Onterie Center's main tower, the entire lateral force-resisting system is achieved by closely-spaced exterior columns and spandrels of reinforced concrete construction. Additional lateral stiffness and structural efficiency were achieved by in-filling the window space with concrete in a diagonal pattern. These reinforced concrete in-fill panels act not only as diagonal braces as in steel high-rise structures, but also act as shear panels as well (Fig. 5).

By going diagonally across the building, the in-fill panels tie columns and spandrels together and help to evenly distribute the gravity loads on adjacent columns. They also help to reduce the shear lag in the tube frame and thus contribute to the structural efficiency of the system. As a result, the entire lateral stability of the building is achieved by two exterior diagonalized tubular channels located at each end of the tower structure. Interior columns carry gravity loads only, thus allowing more flexibility in planning the interior space and eliminating the differential creep between the core walls and adjacent columns, had a core wall been utilized in resisting lateral loads.

5. Mixed Steel/Concrete Systems

Mixed steel/concrete systems have emerged into a well established new system that can be used as readily as either steel or concrete systems for high-rise buildings. Such mixed systems involve reinforced concrete and structural steel components which together resist all the forces of gravity and wind. A wide variety of mixed forms that is generally applicable, such as the composite tubular system and concrete core braced systems, have been widely used. The properties of concrete that are most attractive are its rigidity and its ability to be cast into different types of structural elements. Therefore, most mixed system compositions rely on concrete for lateral load resistance. Shear wall elements and/or punched wall or framed-tube elements with monolithically cast beam-column joints are primary elements used for lateral load resistance. Steel floor framings are used in mixed systems, which are advantageous because of their ability to span

longer distances with lighter members and make possible larger column-free space. Steel is also more effective as gravity columns because of its slenderness and size.

a. First Canadian Centre, Calgary

The First Canadian Centre in Calgary, Canada consists of two towers and a 10-story banking pavilion, located in an L-shaped site in downtown Calgary. The two towers are 64 and 43 stories tall. A sculpted form and a form which provides diagonal vistas to mountains and the city was highly desirable for this prominent corner site. The result was a two-tower concept, each one facing a street. Each tower is similarly shaped, basically involving a parallelogram with truncated and re-entrant corners. Each tower also involves a variation in vertical profile, which can be observed from the model photograph. The structural concept is based on a tube-in-tube concept involving an exterior reinforced concrete framed tube and an interior shear wall core tube. Structural steel floor framing and other interior steel columns complete the system, as shown in Fig. 6. The tube system on the exterior is a combination of framed tube with beams and columns and solid walls at the corners. The structural essence of the concrete framed tube wall is that of a bearing wall and as such, the punched wall and solid wall are both compatible parts of the system. The columns are provided at 9'-8" centers along the flat faces and along the diagonal lines.

b. Southeast Financial Center, Miami

The Southeast Financial Center is a high-rise office development in the downtown area. It consists of a 53-story office tower reaching 730 ft. above grade and is the tallest building in Miami. The site has views to the east over Bay Front Park and as such, the orientation of the office toward the east and a dramatic profile modification at the top floors to optimize vistas was a design requirement.

An extremely rigid and efficient wind-resistive structural system was required to resist high wind forces. In addition, a more massive structure would alleviate potential difficulties with foundation uplift. A composite bundled-tube system was selected over others as being the most appropriate to meet the criteria. The bundled tube is composed of an approximate rectangular tube and a stepped triangular tube (Fig. 7). The common wall between the two tubes is made up of solid walls and a beam-column arrangement. The provision of this interior frame line was essential to improve the tubular efficiency. The interior 40 ft. spans and core framing are framed with non-rigid, simple steel members. The use of the concrete composite tube had the following advantages: (1) development of extreme rigidity and strength for wind resistance; and (2) massiveness of concrete eliminated potential uplift difficulties and produced better dynamic behavior.

c. Tower 49, New York

A recent trend in exterior architecture has been to express facade steps, protruding triangular bays, and other facade profile modulations. These demand a lighter structure on the exterior as can be provided in structural steel. The attitude here is one of a real curtain wall which can be easily manipulated and geometrically applied to a light structure which can support the exterior bays of the floor. In these instances, a structure that is concentrated in the core of a building for wind resistance will offer flexibility of framing on the exterior. A logical mixed combination here is a concrete shear wall core which resists all wind forces surrounded by simple steel framing for floors and exterior columns.

Initial systems were constructed utilizing step-form techniques, whereby the core would be advanced 3 or 4 floors ahead of the exterior steel by forming one floor at a time. Recent use of individually adjustable slip-form jacks makes slip-form techniques practical for building applications. In this process, the core would be slip-formed in its entirety in a short period of time prior to erection of steel. Floor beam connection details and attachments will have to be suitably planned to accommodate slip-forming.

Fig. 8 shows an example of a 44-story core braced system which was configured to fit an unusual site. The core is augmented by fascia moment frames.

Conclusions

Considerable advancements have been made in furthering the systems methodology for concrete high-rise buildings. Improvements in construction methods and the strength of concrete have freed the constraints on ultra-tall, concrete structures. Innovative systems combinations involving steel and concrete are being used to best utilize the merits of each.

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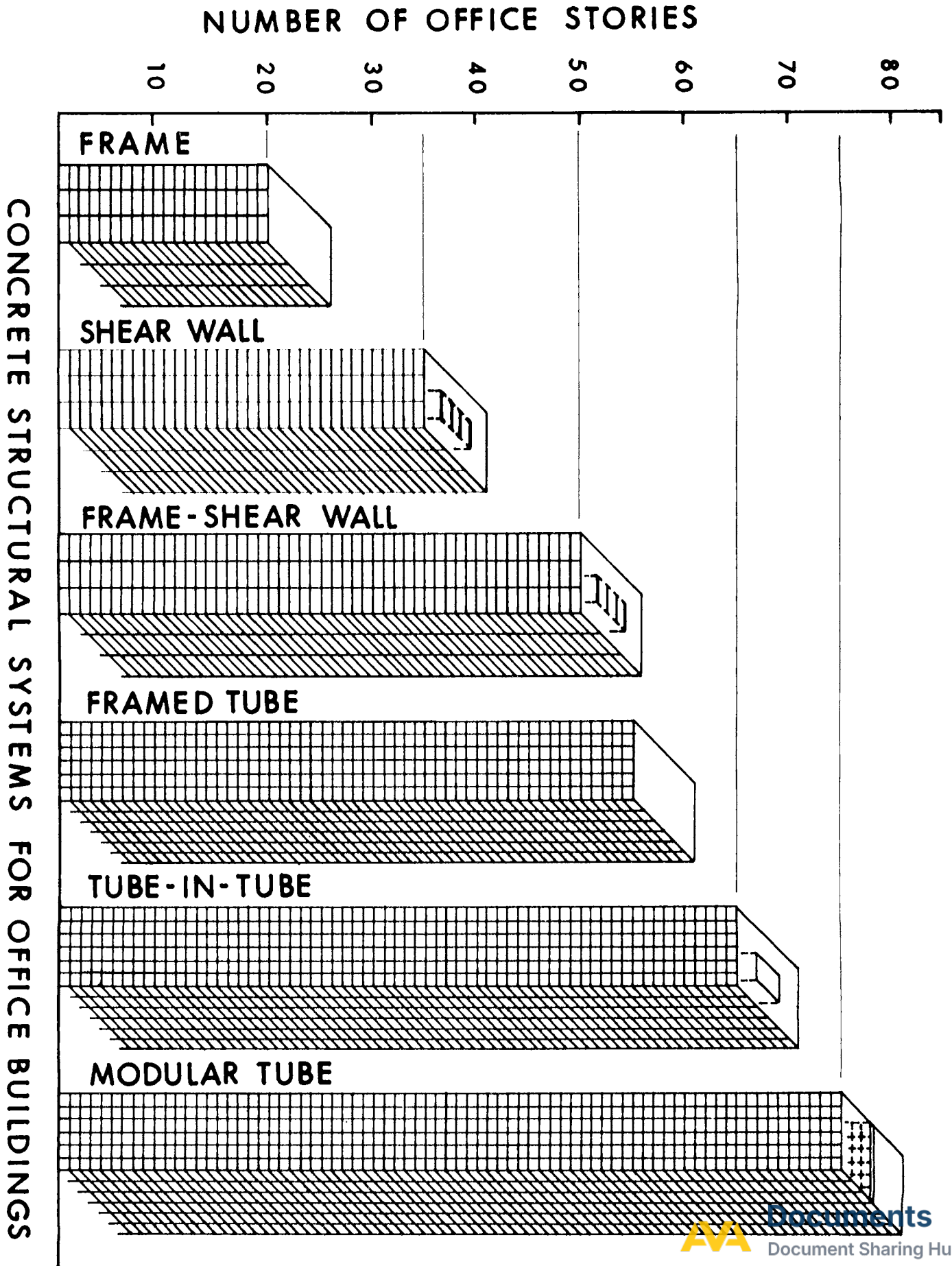
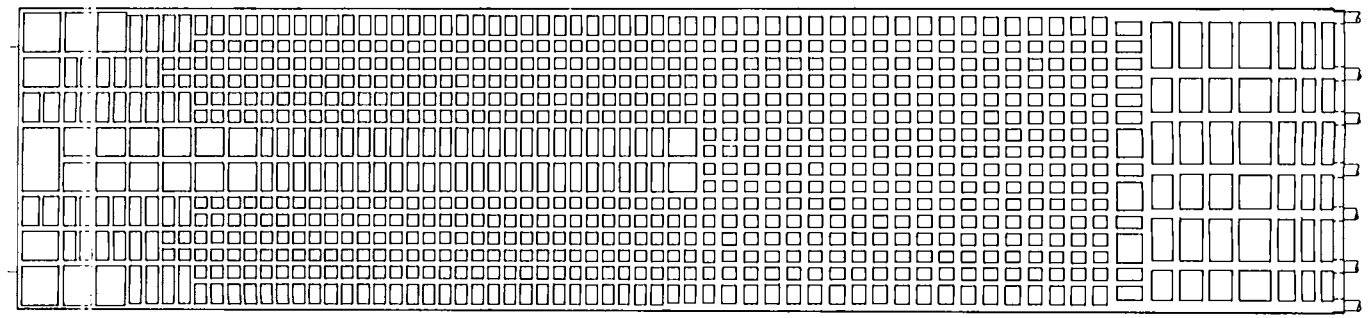
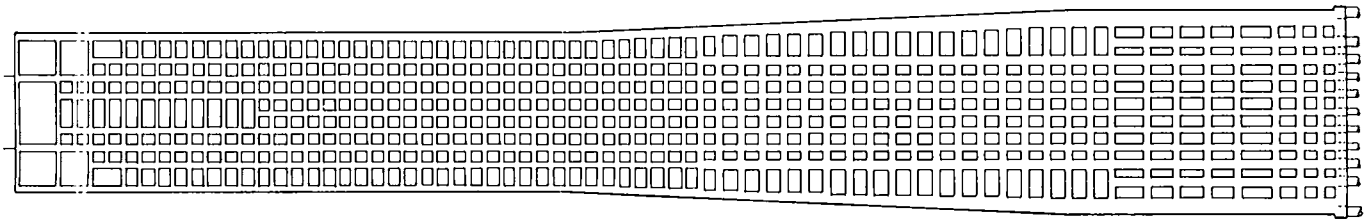


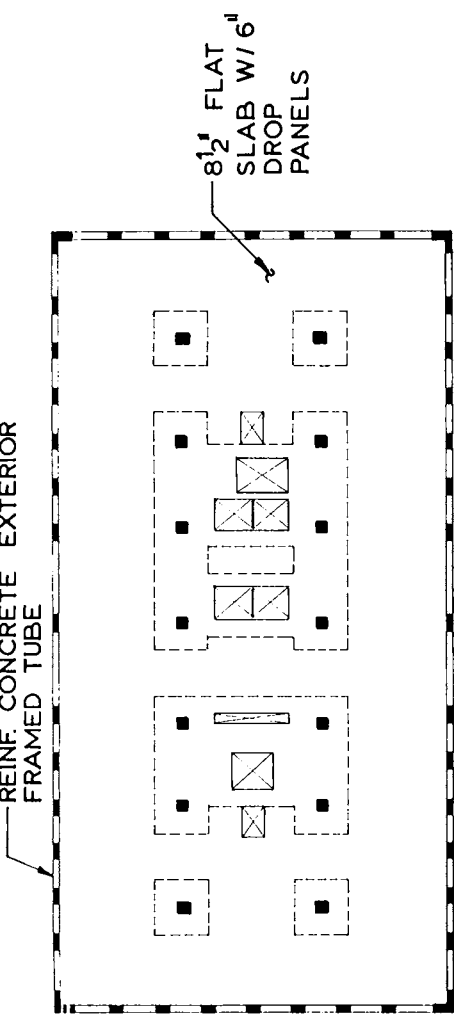
Fig. 1



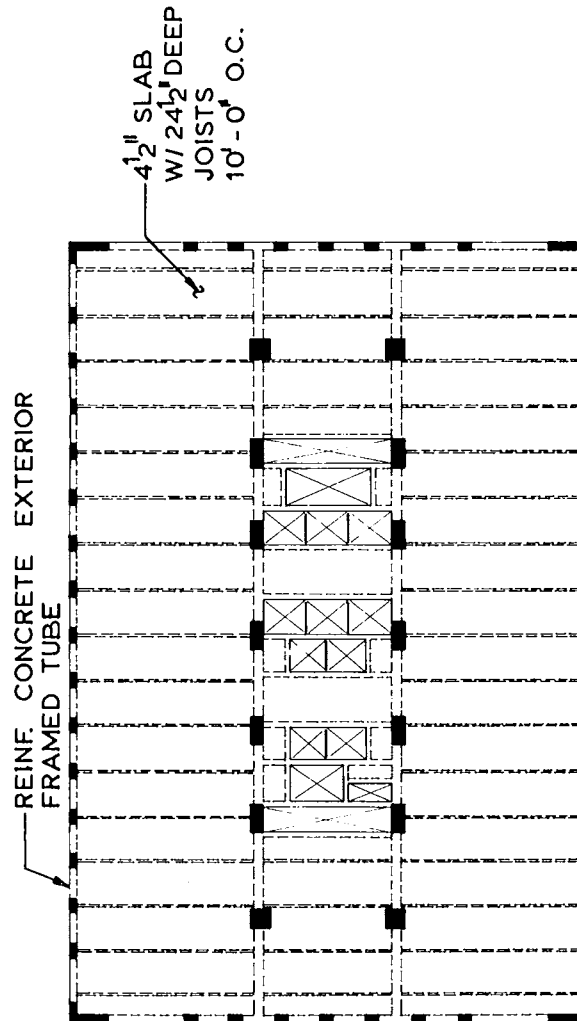
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ELEVATION



WEST
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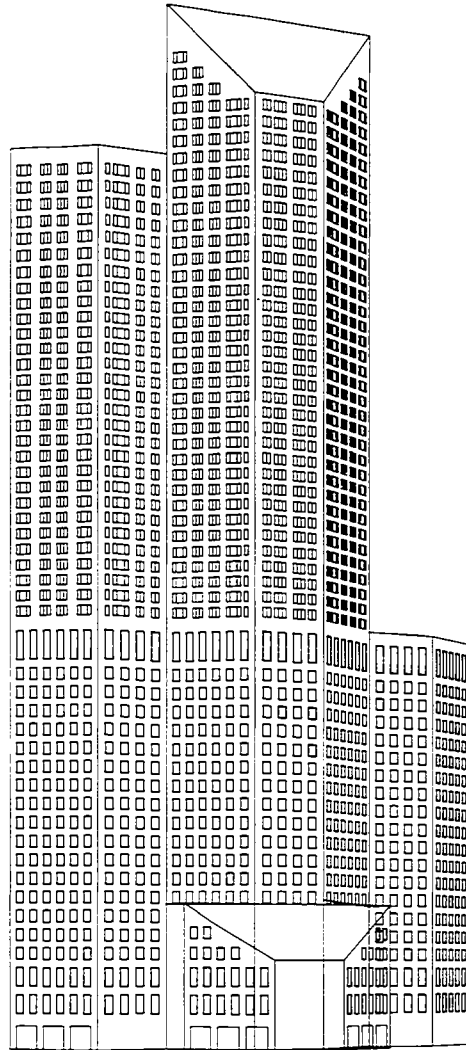
TYPICAL CONDOMINIUM
FRAMING PLAN



TYPICAL OFFICE
FRAMING PLAN

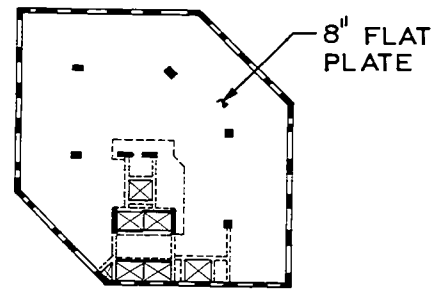
STRUCTURAL SYSTEM OLYMPIA CENTRE

Fig. 2

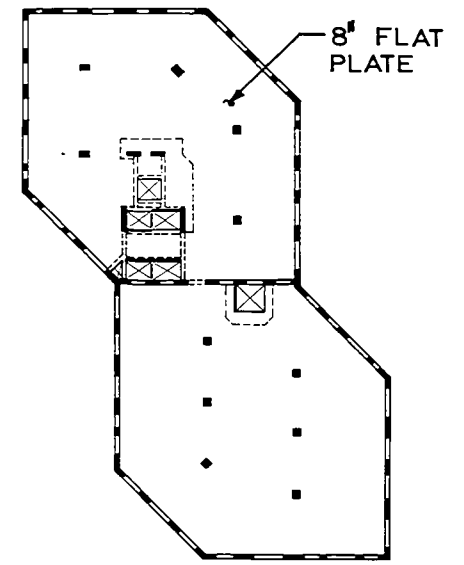


BUNDLED TUBE
CONCEPT

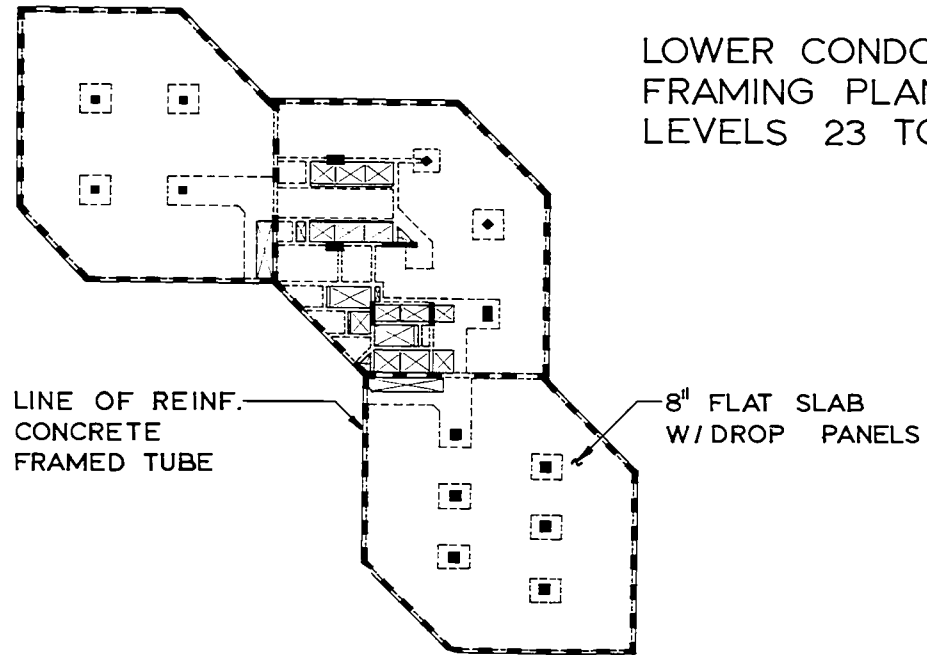
STRUCTURAL SYSTEM
ONE MAGNIFICENT MILE



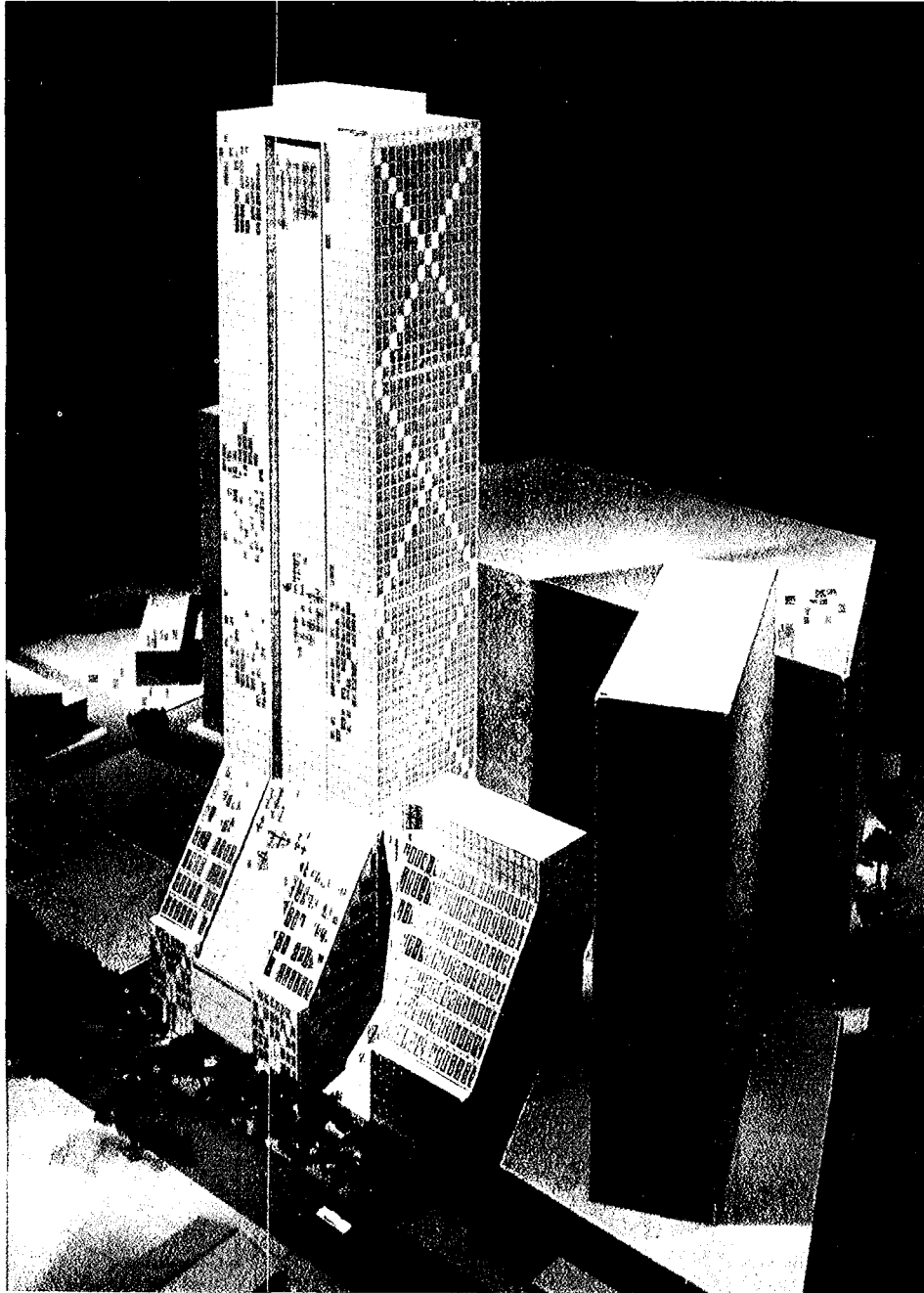
UPPER CONDOMINIUM
FRAMING PLAN
LEVELS 50 TO ROOF



LOWER CONDOMINIUM
FRAMING PLAN
LEVELS 23 TO 49

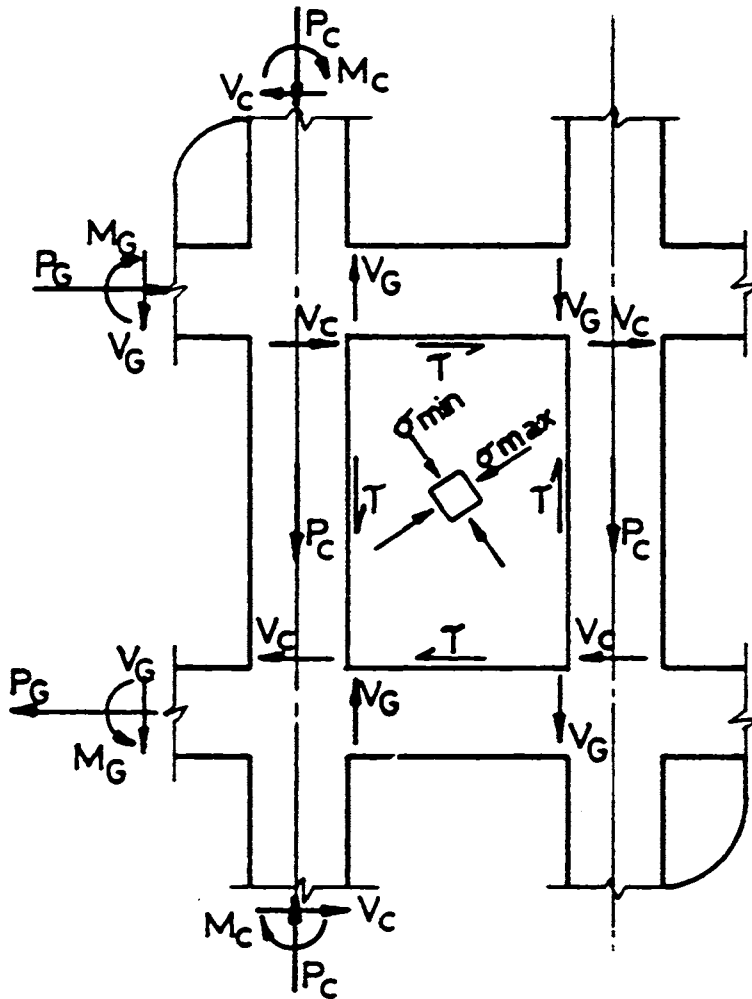


OFFICE FRAMING PLAN
GROUND TO LEVEL 22



Onterie Center

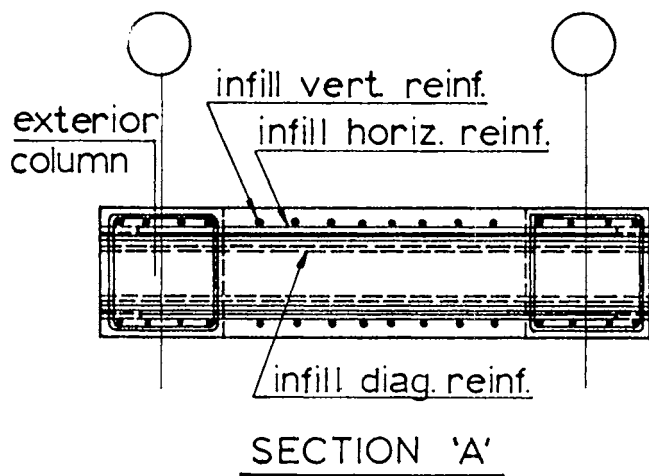
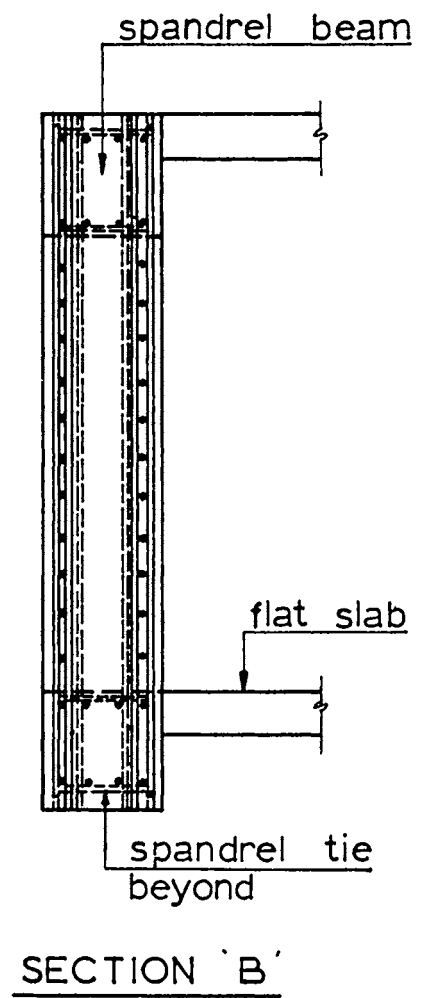
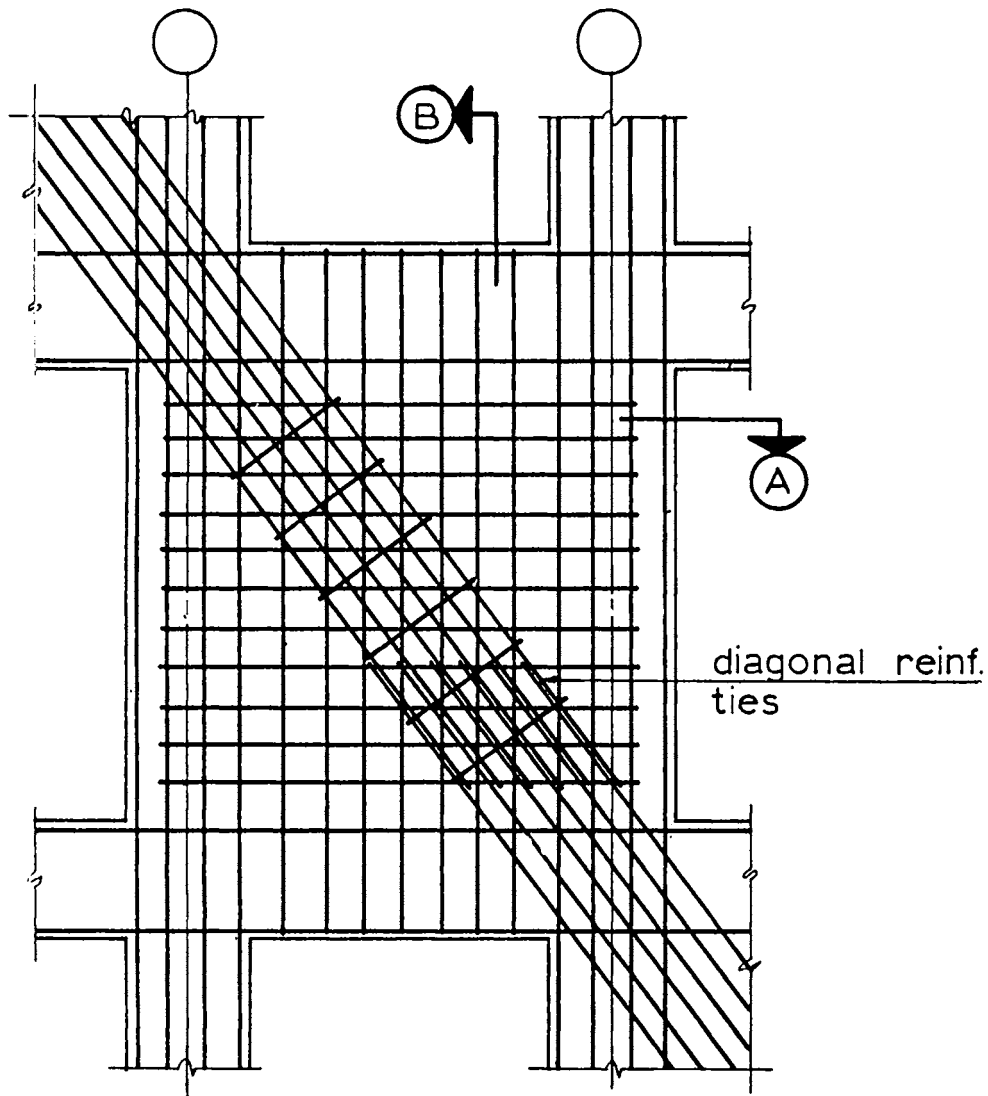
Fig. 4



P_c, V_c, M_c axial shear and moment in columns
 P_g, V_g, M_g axial shear and moment in spandrel beams
 $\sigma_{min}, \sigma_{max}$ principal stresses in infill panel
 T shear stress in infill panel

Forces on Typical In-fill Panel
Caused by Lateral Loads

Fig. 5a



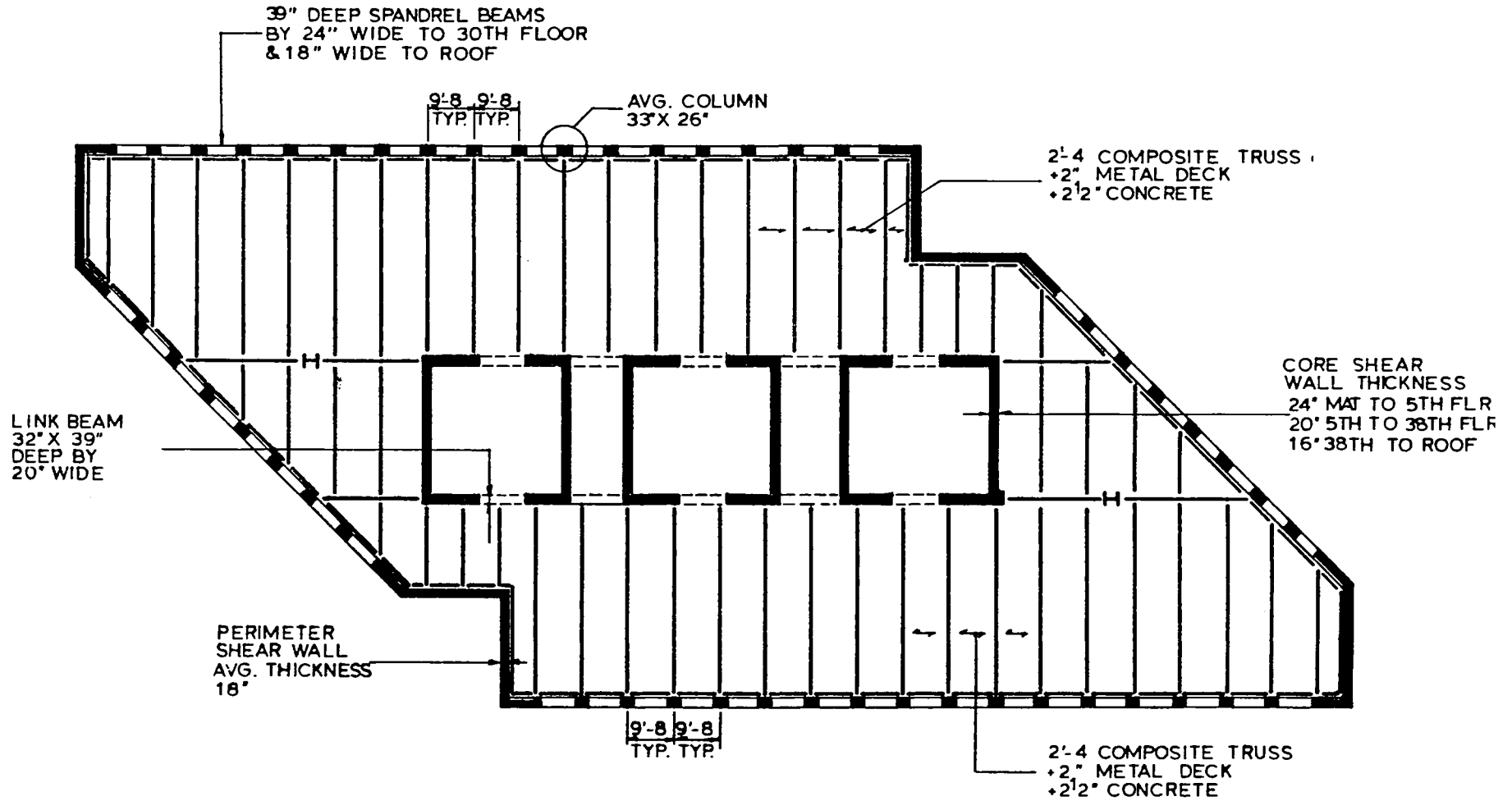
Typical In-fill Panel Detail

Fig. 5b



First Canadian Centre

Fig. 6a



First Canadian Centre
Lower Floors

Fig. 6b

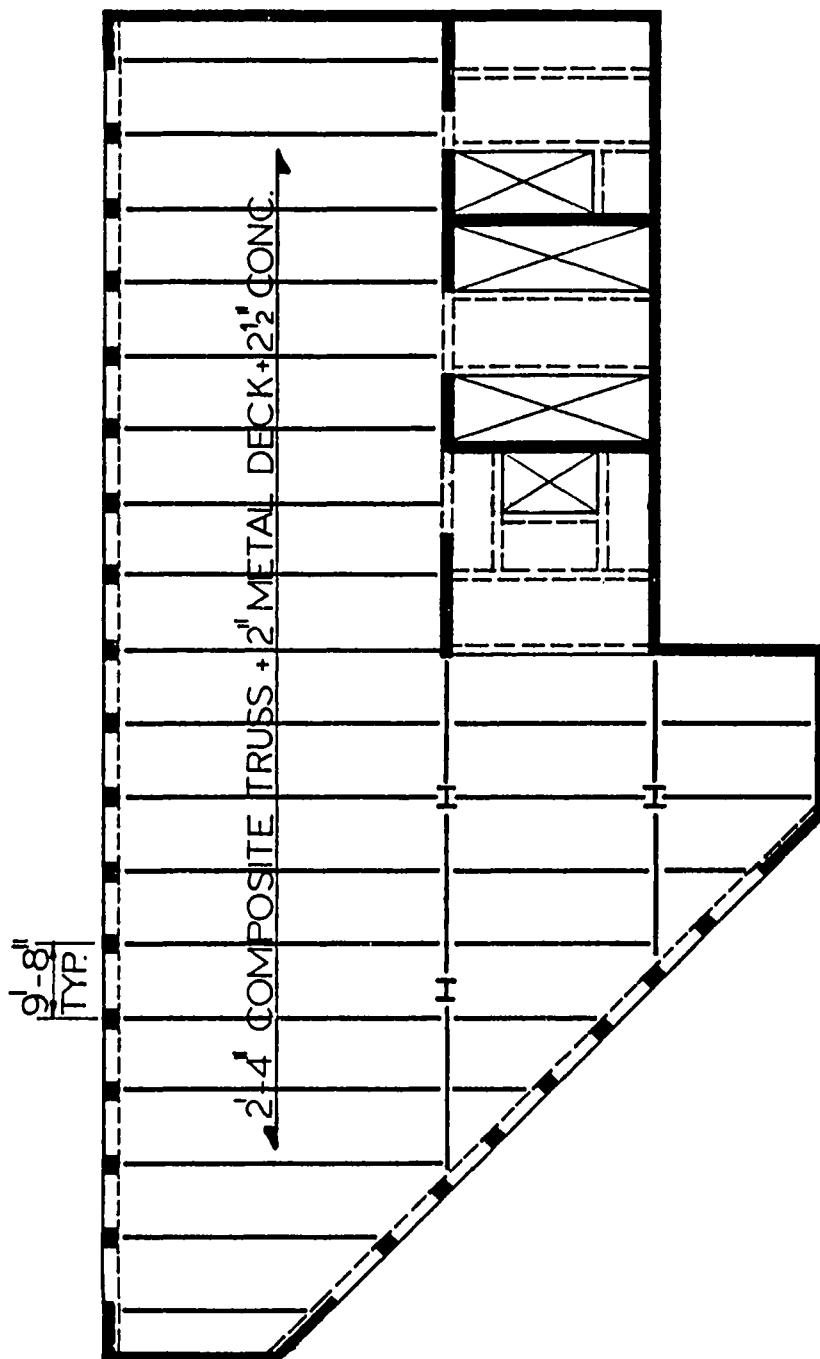


Fig. 6c

First Canadian Centre
Upper Floors



Southeast Bank

Fig. 7a

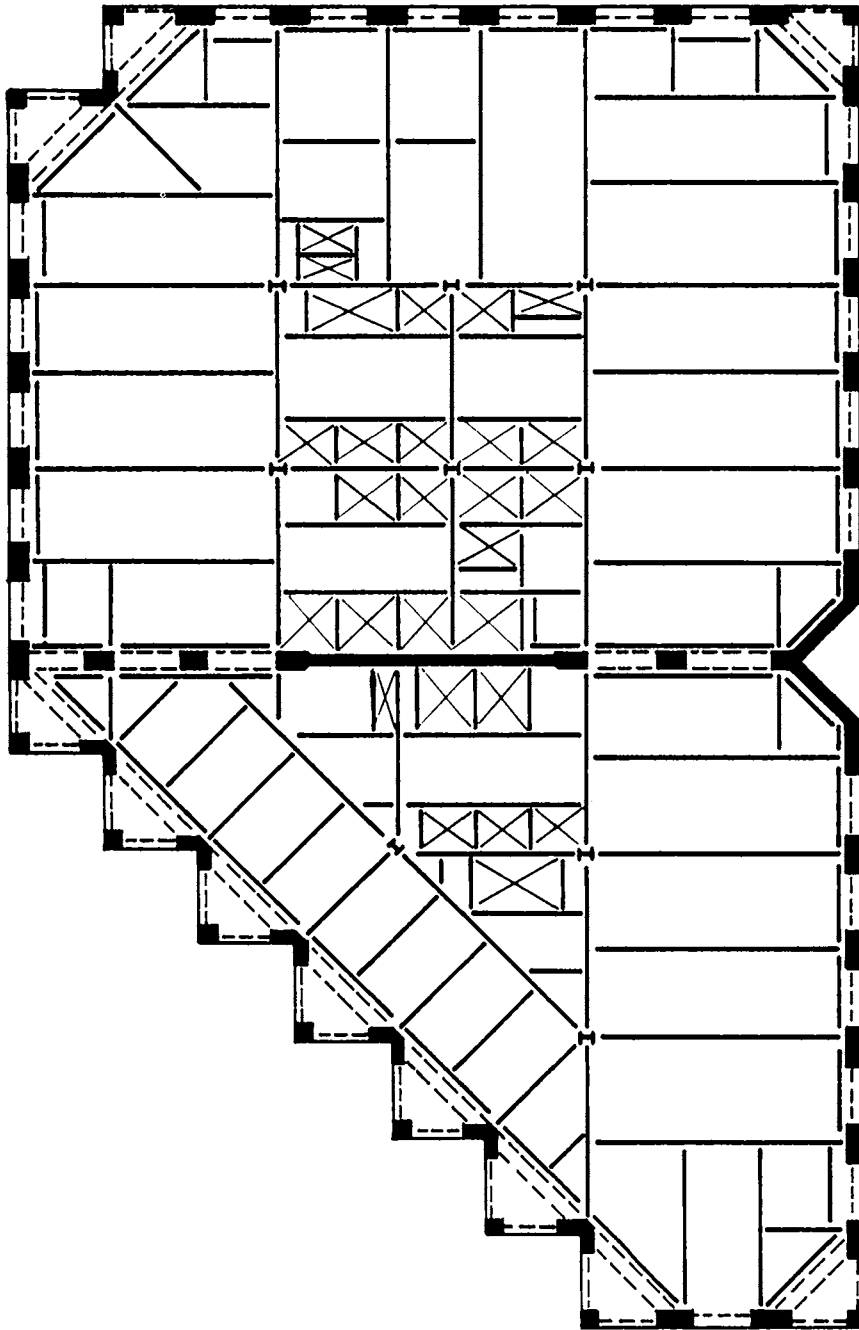
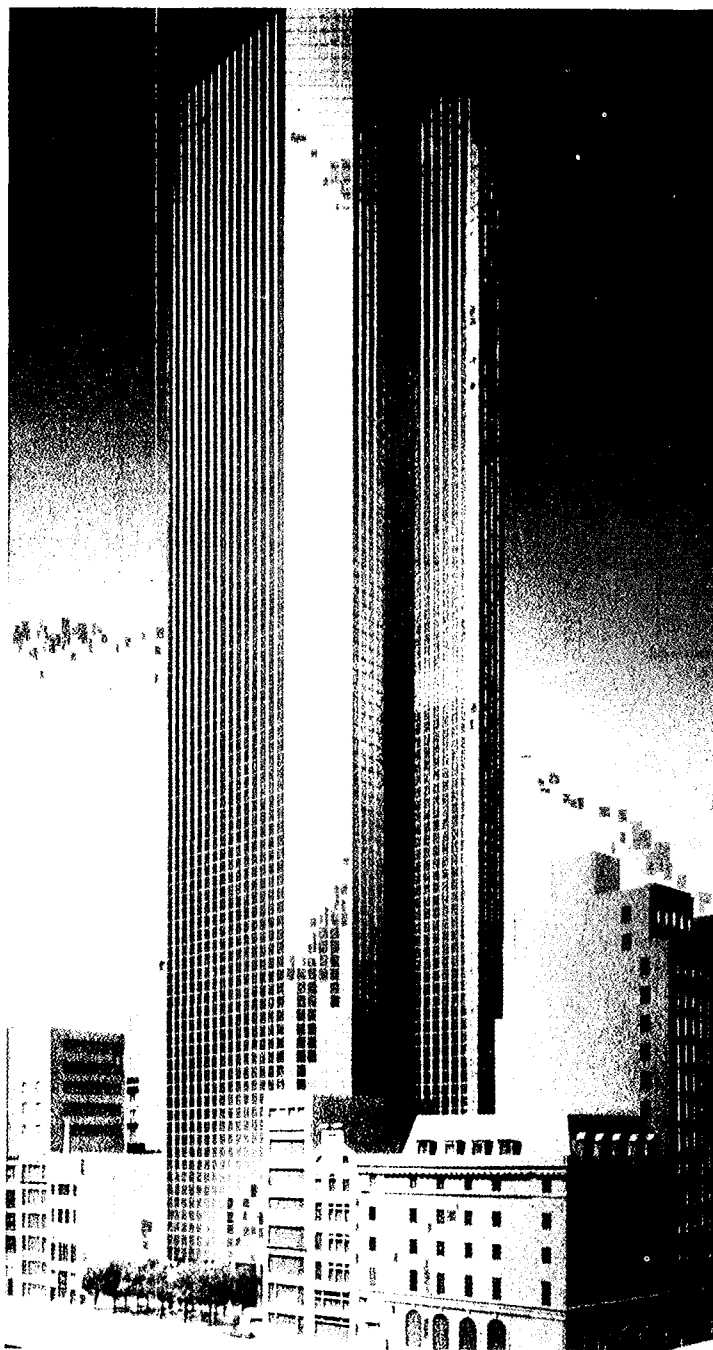


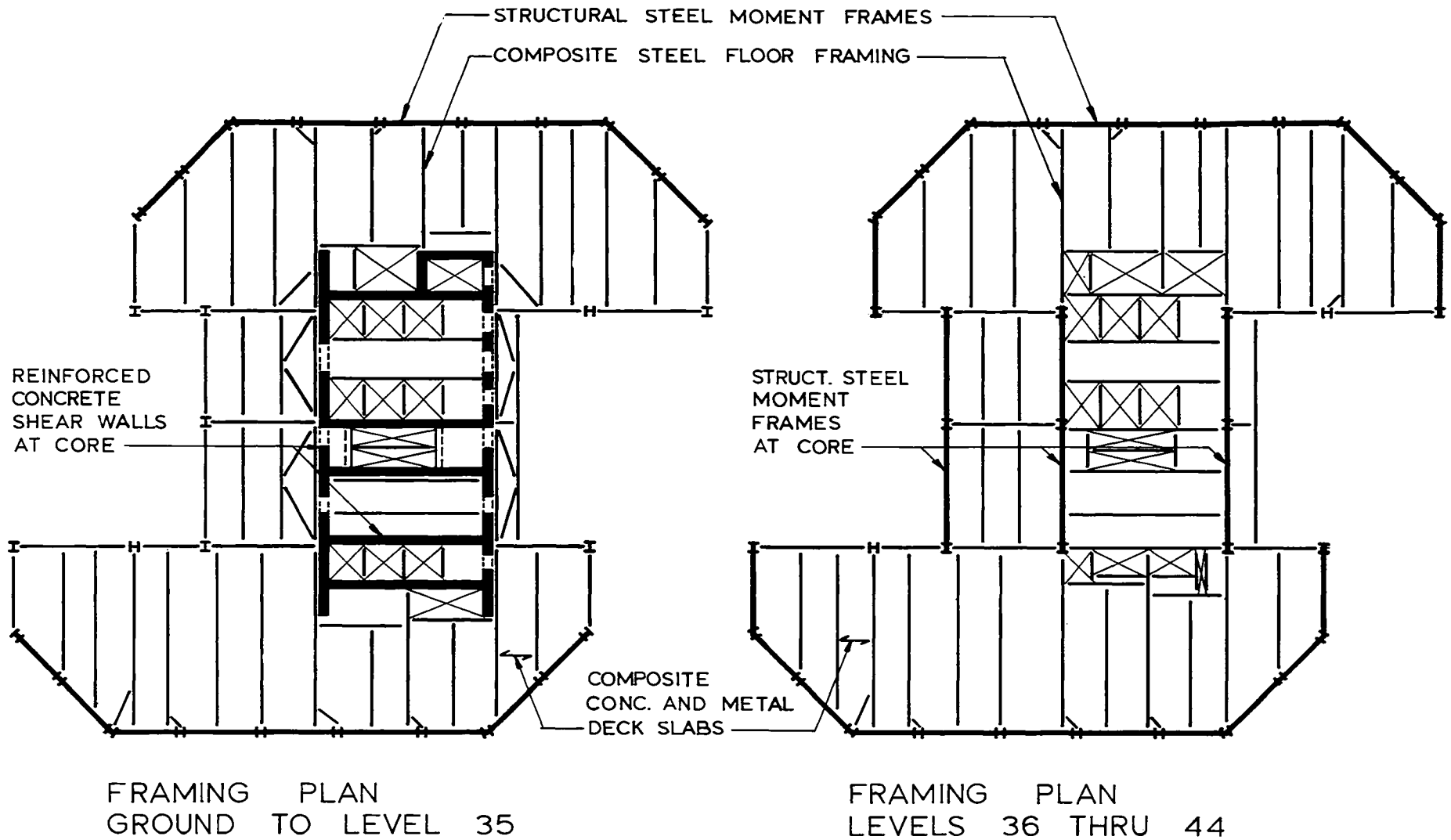
Fig. 7b

Southeast Bank



Tower 49

Fig. 8a



STRUCTURAL SYSTEM
TOWER 49

Optimization of High-Rise Concrete Buildings

By Jaime Moreno* and John Zils**

The optimum design of high-rise buildings should satisfy architectural and engineering performance criteria according to codes and local regulations at the most economical cost. The large variety of construction materials and structural systems makes the task of obtaining the optimum solution difficult for the designers. This study is related to the structural variables influencing the economical choice of materials and systems in cast-in-place multi-story construction.

An efficient and economical structural system can evolve only through an understanding of the significant factors affecting the design of a tall building. This optimization study has considered a number of these factors in order to evaluate their influence on the optimization process. These factors can be summarized as follows:

1. Design lateral force (wind)
2. Height-to-width ratio of building, commonly known as Aspect Ratio
3. Criteria for lateral stiffness (Drift Ratio)
4. Type of occupancy (office vs. apartment)
5. Influence on foundation system
6. Fire rating considerations
7. Local availability and cost of primary construction materials

The final choice of a structural system depends upon the factors mentioned above. It should be obvious that there cannot be any single structural system that is valid for all cases. It is this philosophical attitude that is essential for the architect and engineer in evaluating the best possible structural system for a particular project, for a particular location, at a particular time.

The most common types of multi-story construction are residential and office buildings. Architectural layouts for residential buildings have their maximum performance from spans of 15 to 24 feet, and for office buildings from spans of 24 to 30 feet. Besides the column, caisson, and floor system considerations for these two types of buildings, the lateral load consideration is an important concept in the design of high-rise structures.

A maximum recommended drift of $1/500$ of the height of the building allows the buildings without shear walls (frame buildings) to be built to a certain number of stories depending on the slab thickness and column sizes. When shear walls are added to the frame buildings, they can be built still taller, satisfying the maximum drift limitations. However, depending on the height,

* Manager Technical Marketing, Material Service Corporation, Chicago

** Associate Partner, Skidmore, Owings & Merrill

frame/shear wall buildings have a maximum economy and performance limitation. The tubular concept provides a means to increase the feasible height of buildings in an economical manner. The variables influencing the economic solution of the buildings to be considered in this study are: floor systems, columns, caissons, and building stiffness to support lateral loads. They will be studied and evaluated according to Chicago unit prices for materials.

I. FRAME BUILDINGS

Frame buildings are those with exterior column spacing equal to or longer than 15 feet and made of monolithic concrete construction. In this type of building, the above-mentioned variables are considered.

FLOOR SYSTEMS. The most typical cast-in-place floor systems are studied for residential and office buildings. A typical interior bay is considered in each of the slab systems: flat plate for residential buildings with spans from 15 to 24 feet, flat slabs for office buildings with spans from 24 to 27 feet, and beam slab floor systems for 30-foot bays.

The most typical superimposed loads were considered for the slabs. The flat plate is used with residential loadings of 40 psf for live load and 20 psf for superimposed dead load which give an ultimate load of about 100 psf. The flat slab is used with ultimate superimposed load of 150 psf which includes live loads, partitions, and suspended ceiling.

The slab thicknesses are the minimum thicknesses recommended by the ACI 318-83 Code for the above-mentioned structural floor systems.

The amount of reinforcing in psf for the various bay sizes was obtained from the CRSI Handbook for normal weight concrete, and from the CRSI Structural Lightweight Concrete Design Supplement. The values obtained from the Handbooks are shown in Figure 1 which is presented for the structural floor systems under consideration.

COLUMNS. Columns supporting interior panels corresponding to the aforementioned floor systems with the same loads and characteristics were designed for the two types of buildings in the following manner:

As seen in Figure 2, high-strength concrete will carry a compression load at a lower cost than any lower-strength concrete or reinforcing bars. Consequently, high-strength concrete is recommended for columns that carry large compression loads. The prices used in the graphs are 1985 average unit prices for materials in the Chicago area. The computations for the development of the Cost-Load Evaluation graph are included in Addendum 1.

The concrete strength in the columns was specified to obtain the maximum economy. The proper use of the maximum available concrete strength is specified to obtain the minimum cross-sectional

area in the column and one percent reinforcing. See Figures 3 and 4. Computer print-outs for the columns shown in these figures are included in Addendum 2. However, given a minimum column section, it is not economical to use a concrete strength of 11 ksi to obtain one percent reinforcing when with the same column section, one percent of reinforcing can be obtained with the use of 10 ksi or a lower concrete strength.

COLUMNS SUPPORTING PLATES. The columns support interior square panels 15, 18, 21, and 24 feet square with plate thicknesses of 5, 6, 7, and 8.5 inches respectively. These spans are mainly used in residential structures with 40 psf as live load and 20 psf as superimposed dead load. The columns were analyzed and designed according to ACI 318-83 and the Chicago Building Code for live load reduction.

Two series of computer runs were carried as follows:

1. Normal weight concrete columns supporting 110 psf lightweight slabs plus the live and superimposed load described above: one percent reinforcing for maximum economy was maintained in the columns by changing concrete strengths from 3 to 11 ksi up to 35 stories and changing concrete strengths and column sizes for buildings over 35 stories.
2. Normal weight concrete columns supporting 150 psf normal weight slabs plus the aforementioned live and superimposed loads: only the dead load of the slab was changed from 110 to 150 psf for this run as compared to the lightweight runs. The same column sizes were maintained at the same story level, and the concrete strengths were changed at the same story level as in the lightweight runs.

The total amount of reinforcing shown in the results includes 20 percent for overlapping of vertical bars and #3 ties.

The unit prices used to obtain total prices per square foot of slab are:

Reinforcing in place			\$ 0.38/lb.
Column forms			2.80/sq. ft.
Concrete placing			16.00/cu. yd.
Concrete:			
3 ksi	\$ 46.00/cu. yd.	4 ksi	\$ 50.00/cu. yd.
5	54.00	6	58.93
7	62.59	8	67.21
9	68.62	10	78.45
11	88.27	12	96.60
13	104.94	14	113.27

The above prices are Material Service book prices for 1985. This study included strengths to 14 ksi currently available in the Chicago area.

COLUMNS SUPPORTING SLABS. Columns supporting slabs were analyzed and designed in the same manner as columns supporting plates. But in this case, the columns were supporting interior panels from 24- to 30-foot spans and the same superimposed dead and live loads used to obtain the slab reinforcing. The computer runs were carried in the same manner previously described.

The extent of this column investigation is summarized in Table I which shows sizes and maximum concrete strengths used in each column. The results are shown in Tables II and III for residential and office buildings. From these tables, the amount of reinforcing per square foot for residential and office buildings is shown in Figures 5 and 6. Some of the results were used to develop Figures 7 and 8 for 40-story residential and office buildings.

CAISSONS. The caissons are those which support interior panels and columns corresponding to the floor systems described above with the same loads and the same characteristics, plus the dead load of the columns used for the column curves. Unfactored column loads for the design of the caissons were determined according to ACI 318-83 and the current Chicago Building Code, as indicated by the previously-mentioned computer programs.

The following parameters were used for the design of the caissons: soil capacity, 30 ksf at 75 feet, and $f'_c = 5000$ psi; minimum shaft size 2' 6"; and bell size equal to three times shaft size.

Two series of computer runs were made to obtain the loads for the design of the caissons: the first for the 110 pcf lightweight slab and normal weight columns, and the second computer run for the normal weight slabs and columns. In this second run, only the dead load of the slabs was changed from 110 to 150 pcf. With the loads established, the caissons were designed for static loads only.

Caisson volumes were obtained for caissons which support interior panels of multi-story buildings of up to 40 stories for the two structural floor systems under consideration, first using normal weight in the floor systems, and second using lightweight in the floor systems. See Figures 9 and 10. The difference between the caisson volumes divided by the tributary area of the caisson corresponds to the savings per square foot of structure supported. These savings are shown in Figure 11.

The curves do not show any savings in caissons supporting fewer stories because the minimum caisson size is used for both normal weight and lightweight slabs. The savings become evident when at a particular story a normal weight slab requires a larger caisson than a lightweight slab requires.

The curves stop when the number of stories which are to be supported requires a caisson too large to be practical or economically feasible under the established soil conditions.

LATERAL DRIFT. The lateral deflection of multi-story buildings

due to wind load was computed using the Portland Cement Association program STMFR and the Chicago Building Code for wind loads.

Two types of buildings were analyzed. The first type was buildings using flat plate construction with typical square panels of 15, 18, 21, and 24 feet and with plate thickness of 5, 6, 7, and 8.5 respectively. The columns supporting these plates and residential loads of 40 psf used for the analysis have the same dimensions used in the optimization of columns supporting the aforementioned spans. The number of stories used in the study was from 10 to a number which will produce a drift at the top of not more than 0.0025H.

The second type was buildings using flat slab construction with typical square panels of 24, 27, and 30 feet and with drops of 5, 5.5, and 7.5 inches for normal weight slabs and 3.5, 4, and 5.5 inches for lightweight slabs. Also, a two-way beam slab was analyzed for the 30-foot bay with 5- and 6-inch slabs for lightweight and normal weight respectively and 10-foot by 10-inch beams at the column strip.

For this analysis, a typical four-span frame was analyzed with a wind surface of 15, 18, and 21 feet for plates and 24, 27, and 30 feet for slabs. The stiffness was in accordance with column and slab dimensions used in the column and slab optimization.

When the drift at the top of these frames exceeded the above-mentioned 0.0025 of the height, 10-inch by 10-foot shear wall was added to the four-frame original structure. New computer runs were made with this condition until the drift at the top exceeded 0.0025 of the height.

In order to have a larger number of stories after the shear wall concept was not feasible anymore, the tubular concept was used. The results are summarized in Figure 12.

FINAL CURVES. Using the previously-mentioned results for slabs, columns, and caissons, unit prices for materials and labor in the Chicago area were used to economically evaluate the results. Table IV shows slab prices for residential and office buildings. Column and caisson prices were obtained from Tables II and III. The following unit prices were used to obtain comparative differential costs:

Additional cost of lightweight	\$12.00/cu. yd. (\$0.44/cu. ft.)
Reinforcing in place	0.38/lb.
Caisson work	6.50/cu. ft.

The purpose of these comparative differential costs is to determine at what number of stories in multi-story buildings the use of lightweight is more economical than the use of normal weight concrete for the floor systems under consideration. The results are shown in Figure 13.

For the final curves, the savings in slab and column reinforcing curves were multiplied by the unit cost listed above. Also, the caisson savings were multiplied by the unit cost and added to the

savings in the columns and slabs. The additional cost of lightweight concrete was subtracted from the previous figures and the results are shown in Figure 13.

It can be seen in the flat plate curves used in residential buildings that it is more economical to use lightweight concrete after 20 stories in buildings having 21-foot square bays. However, when the flat plate has 24-foot square bays, it is more economical to use lightweight concrete in the slabs after 27 stories.

For flat slabs, it is more economical to use lightweight concrete after 16 stories in buildings having 24-foot square bays. But, when the flat slab has bays larger than 27-feet square, it is more economical to use lightweight.

The costs of placing and finishing of concrete and formwork and shorings are not included in these figures because they are considered by contractors to be the same for normal weight or lightweight concrete. Only differential costs are included in Figure 13.

We have evaluated the optimization process considering the aforementioned factors for frame structures and shear wall structures and will now examine the exterior tubular-type structures.

II. TUBULAR BUILDINGS

The exterior tubular system is formulated by arranging a system of very closely-spaced exterior columns and connecting them together at each floor with deep perimeter spandrel beams (Figure 14). In this way, the structure behaves like a true vertical cantilever tube with small punched openings to create windows in lieu of a column and beam system with bending for each individual member. The general concept was probably first used by Skidmore, Owings & Merrill (SOM) in 1961 in the 43-story DeWitt/Chestnut Apartment Building in Chicago (Figure 15). The exterior columns were spaced at 5' 6" centers around the perimeter of the building and were designed to resist the total wind load. Subsequently, the office building at 500 North Michigan Avenue was designed and built (Figure 16). Following these two projects, numerous examples of residential and office buildings using the tubular concept have been designed and built.

This system of closely-spaced exterior columns and rigid spandrels may be called the "framed tube" system of construction. While the first impression of this system is its tube-like configuration, further investigation will indicate that the overall behavior of such systems is a combination of frame (shear wracking) behavior and true cantilever behavior. Under lateral loads acting on such a structure, two distinctly separate behaviors take place. First, the entire structure acts like a tube causing compression and tension forces in all the exterior columns and deflection similar to a true cantilever. Second, the two exterior frames of the structure parallel to the direction of the wind act as rigid frames subjected to the full wind load. They undergo wracking movements at each story as would be expected in any frame structure.

This tubular behavior which results in axial stress in the columns does not develop full 100 percent efficiency due to a characteristic known as shear lag. This shear lag effect is caused by the flexibility of the spandrel beam (Figure 17). The flexibility increases the axial stresses in the corner columns and reduces the axial stresses in the other exterior columns. A measure of the tubular efficiency of a particular project can be shown by calculating the shear wracking/frame action portion of the lateral deflection and the cantilever action portion of the deflection. If the tubular deflection accounts for at least 75 percent of the total deflection, this then indicates relatively good cantilever behavior. Based on this combined behavior, it is evident that in order to increase the efficiency of this system, the spandrel beam stiffness must be increased to a very high level (to reduce shear lag), and the columns should be oriented with their long direction along the face of the building (to reduce shear wracking at each floor). In general, a basic criteria of keeping the column stiffness (I_c/L_c) approximately equal to the spandrel beam stiffness (I_b/L_b) results in a reasonably efficient tubular structure. Therefore, in practice, the framed tube system will result in larger lateral deflections than that computed for an ideal equivalent solid tube.

The greatest advantages of the tubular system are that it not only provides an economical structure over a wide number of stories in the building, but also provides for the traditional architectural articulation of exterior window treatment on the facade of the building (Figure 18).

With these parameters in hand, the exterior tubular concept was investigated from a 30-story to a 60-story structure in 5-story increments (Figures 19, 20). This range of application was based on previous experience with these types of structures. Overall plan dimensions, interior column arrangements and core size were determined for both residential and office buildings and for various heights of structures. The office building utilized a 30-foot by 30-foot interior bay which resulted in a two-way banded-slab floor system while the residential building utilized a 20-foot by 20-foot interior bay which resulted in a 6 3/4-inch-thick, two-way flat plate. These two floor systems were previously discussed in this paper regarding their optimization characteristics. Typical gravity loads were used to include live loads for office and residential structures in accordance with the Chicago Building Code. Lateral wind forces were in conformity to the Chicago Building Code.

The exterior tubular configuration used for this study consists of columns at 10-foot centers connected by an exterior perimeter spandrel beam (Table V). The spandrel beam was sized based on stiffness criteria developed by Khan and Amin¹ in 1973 and somewhat modified by formwork considerations. The exterior tubular columns were sized for gravity loads to include the lightweight concrete slab for a maximum of approximately one percent reinforcement and concrete strength as required. A maximum concrete strength of 8000 psi was used for this study. For the normal weight concrete slabs, the column size and concrete strength were kept the same as the lightweight slab and the reinforcement per-

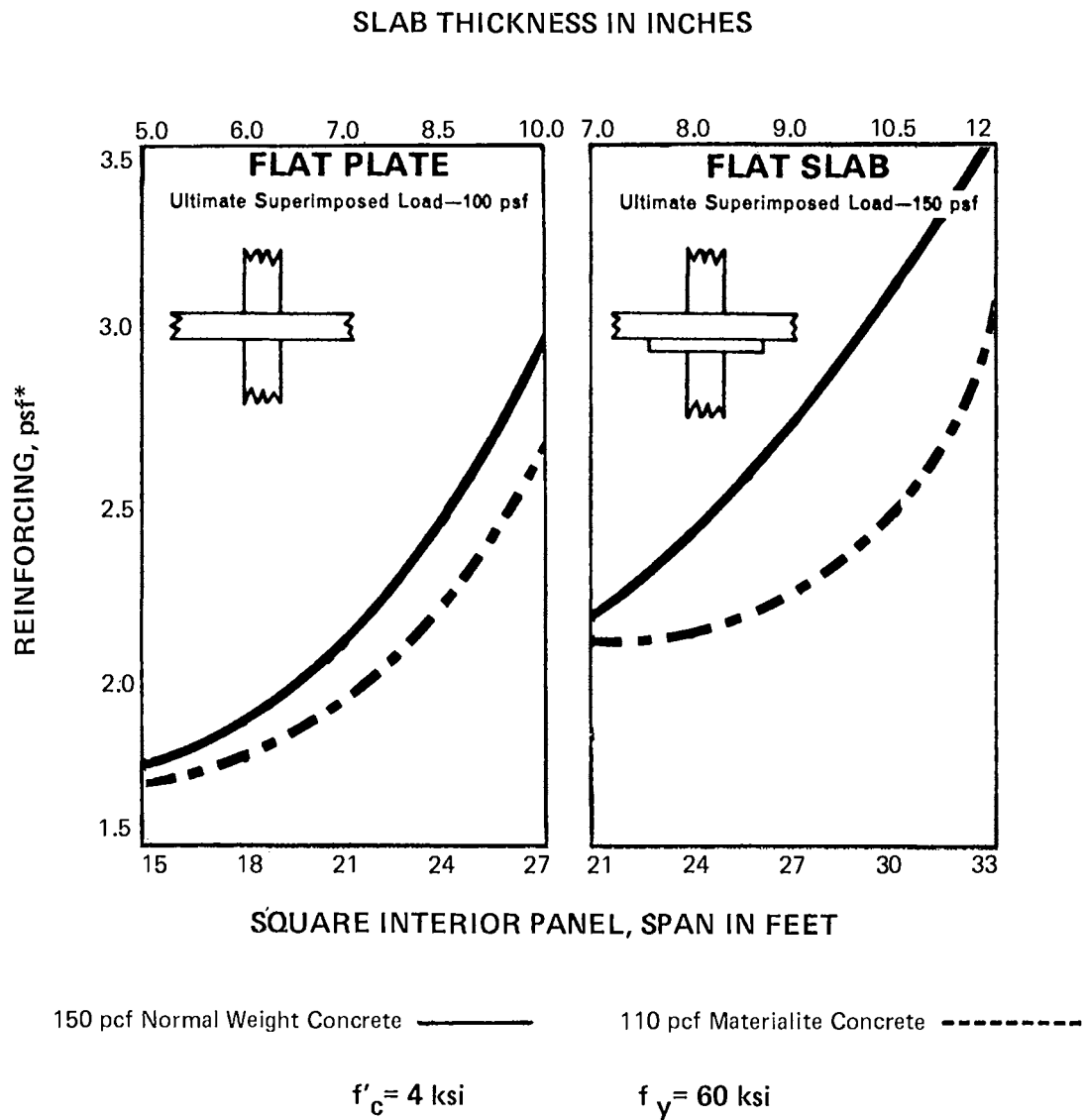
cent was increased as required. This concept for initial column sizing was based on the results of column optimization procedures developed earlier in this paper. Once these initial member sizes were determined for both office and residential use structures, a three-dimensional analysis was run for both lateral and gravity loads. In this way, the structure was initially sized and designed for gravity forces and the wind premium, if any, would be determined from the lateral load analysis. The idea here was that previously the design had been optimized for each individual element of the structure and now these pieces were being assembled into a complete building to see if these optimized pieces resulted in an optimized building.

Results of the lateral load analysis indicate that the structures' lateral stiffness was adequate up to about 50 stories for the office structures and about 55 stories for the residential structures (Figure 21). The stiffness of both 30-story structures was in excess of $H/1000$ which would indicate too much material in the structure and therefore not an optimum structure. These results correspond quite closely with the information shown in Table 4.2 of the CB Monograph prepared by the Tall Buildings Council. Design of the columns and spandrels using results of this analysis shows that for structures up to 40 stories in height, column designs are controlled by gravity forces while spandrel beam reinforcement is one percent or less and controlled by wind forces. In a tubular structure, spandrel beam designs are generally controlled by wind since very little gravity force is transmitted through them. Above 40 stories, the column designs are controlled by wind forces as can be seen in the attached figures (Figures 24, 25). Column reinforcement percent for the normal weight concrete slabs exceeds three percent for the 60-story structure. Also, the corner column for all the structures in excess of 50 stories resulted in an uplift condition (tension) for lateral wind forces. The spandrel beam reinforcement was again controlled by wind and exceeded 1.5 percent for structures over 55 stories.

The resultant designs were then combined with the column cost data and slab cost data previously developed in this paper in order to arrive at a final total cost for the structure. Cost curves relating to columns and slabs used in this study are shown in the attached tables (Figures 22, 23, 24, 25). Final total costs for the tubular structures are shown on the attached tables and figures (Tables VI, VII; Figures 26, 27). This cost data corresponds closely with data prepared by Khan and Iyengar and shown in the attached figure (Figure 28).

This study indicates that tubular structural systems in concrete can be structurally optimized within the parameters discussed up to the 50- to 60-story range. The study presented made various assumptions regarding the building configuration. However, the general tendencies and conclusions regarding behavior and cost remain relatively valid for other configurations assuming proper tubular behavior.

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- 1 Khan, F. R. and Amin, N. R., "Analysis and Design of Framed Tube Structures for Tall Concrete Buildings," ACI Journal Special Publication 36, 1973.

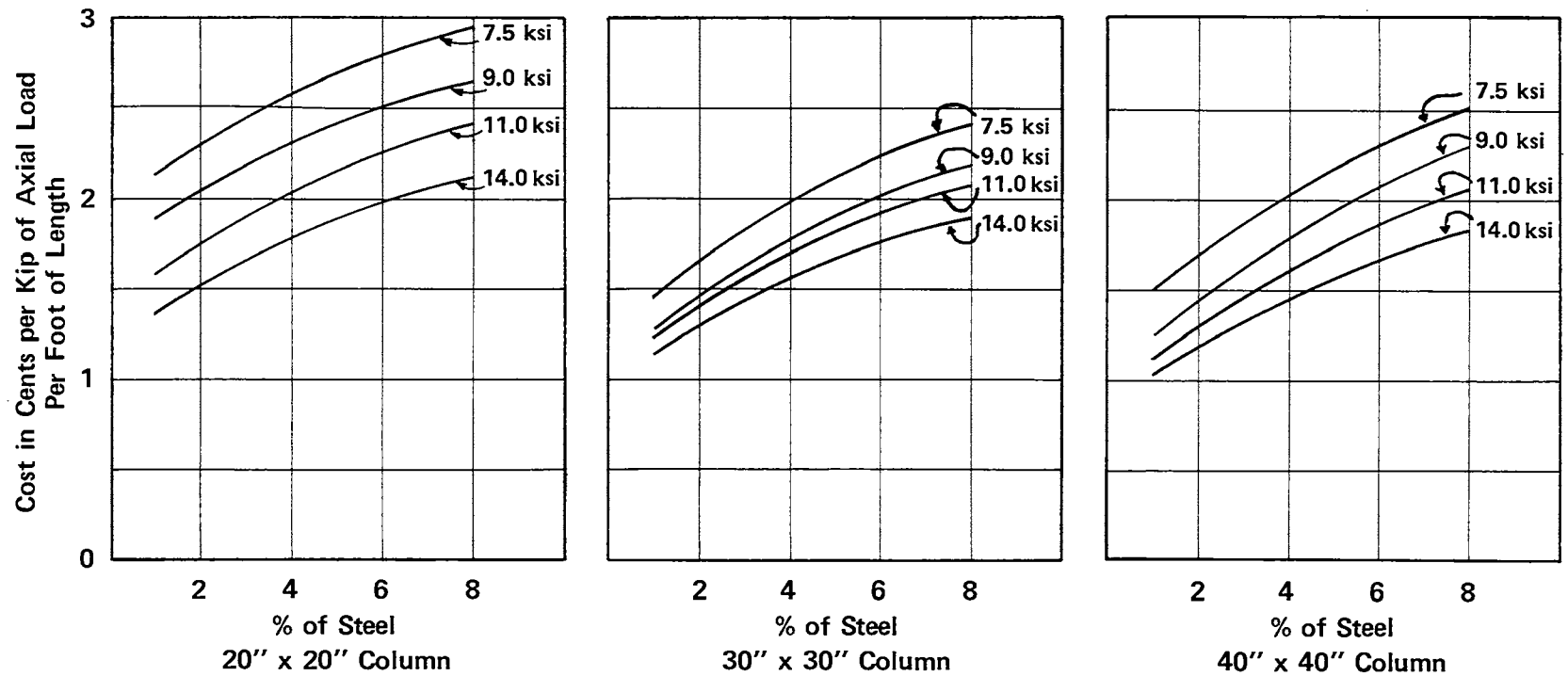


*Based upon "CRSI HANDBOOK" and "CRSI HANDBOOK, STRUCTURAL LIGHTWEIGHT CONCRETE DESIGN SUPPLEMENT."

FIGURE 1

1985 Unit Prices in the Chicago Area:
Concrete (includes \$16/cu. yd. for placing)

7.5 ksi = \$ 80.40 Formwork: \$2.80/sq. ft.
9.0 ksi = 84.62 Reinforcement: \$0.38/lb.
11.0 ksi = 104.27
14.0 ksi = 129.27



COST-LOAD EVALUATION

FIGURE 2

**COLUMN REINFORCING ($f_y = 60$ ksi)
FOR A 21 FT. SQ. INTERIOR PLATE**

24" x 24" Interior Column
Area = 576 Sq. In.

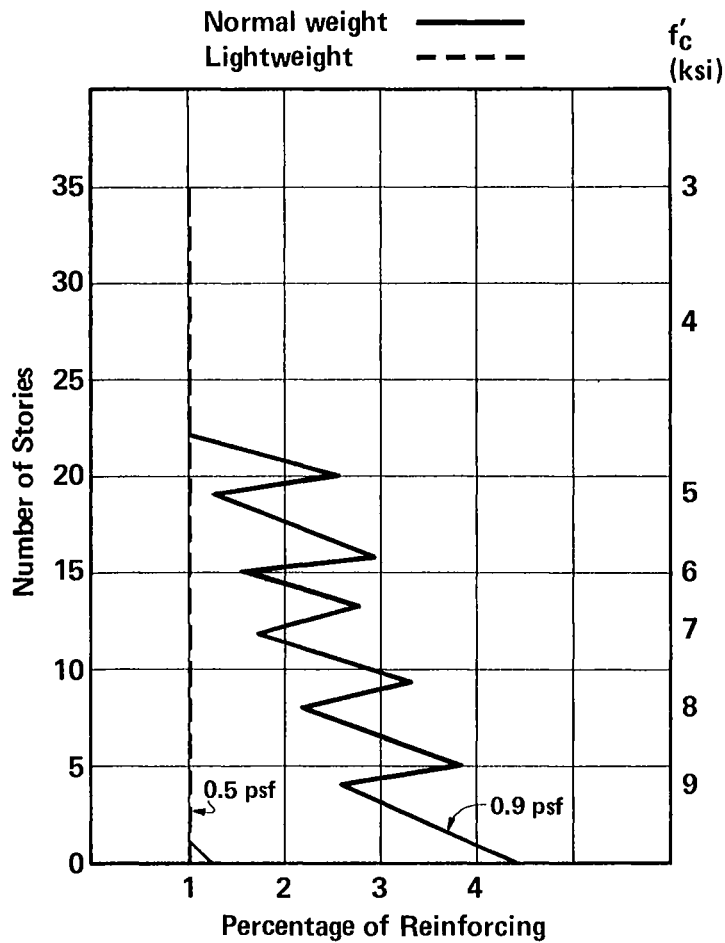


FIGURE 3

COLUMN REINFORCING **FOR A 30' x 30' TWO-WAY BEAM SLAB INTERIOR BAY**

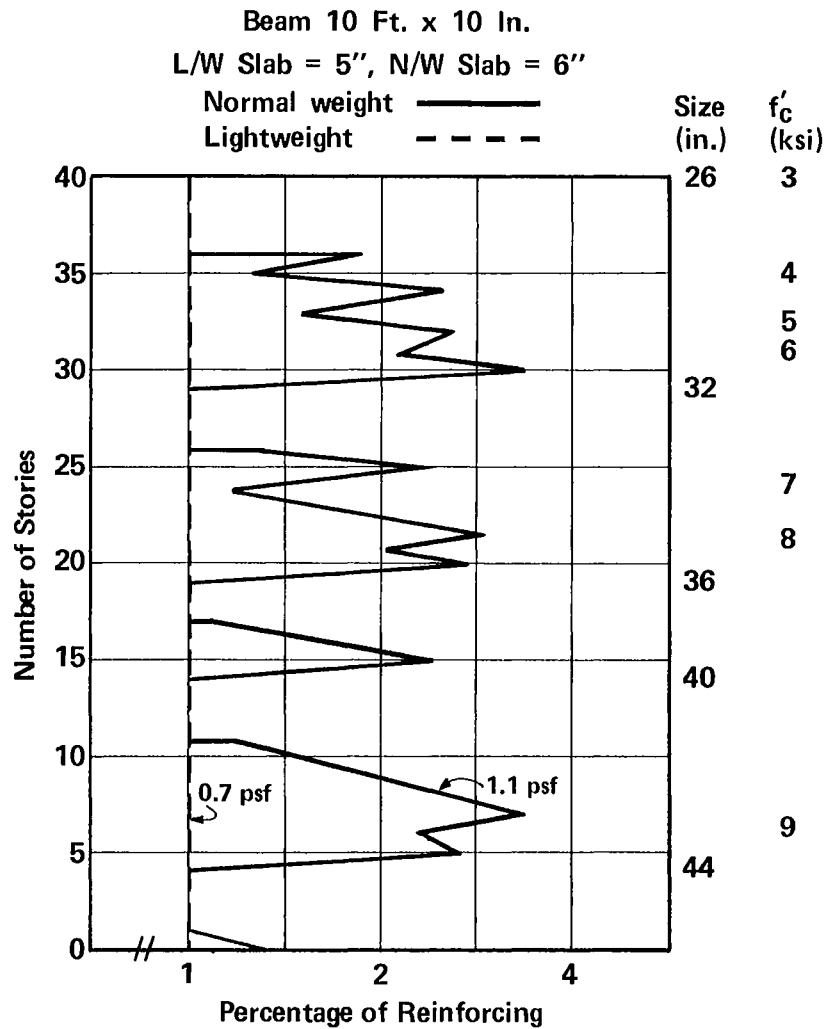


FIGURE 4

TABLE I
COLUMN SIZES AND MAXIMUM CONCRETE STRENGTHS

PLATES			SLABS		
<u>15-ft. Bay</u>	Size (in)	f'_c (ksi)	<u>24-ft. Bay</u>	Size (in)	f'_c (ksi)
10 stories	12	5		17	8
15	12	6		20	9
20	12	9		24	9
25	14	8		26	9
30	16	8		29	9
35	17	8		26-29-32	9
40	14-16-18	8		26-29-32-35	9
<u>18-ft. Bay</u>			<u>27-ft. Bay</u>		
10 stories	12	7		20	8
15	15	7		24	8
20	16	9		24-28	8
25	17	9		26-30-32	9
30	18	9		26-30-34	9
35	20	9		26-30-34-38	8
40	16-19-22	8		26-30-34-38-42	9
<u>21-ft. Bay</u>			<u>30-ft. Bay</u>		
10 stories	14	7		21	8
15	18	7		20-24-26	8
20	18	9		24-28-30	8
25	20	9		24-28-30-34	8
30	22	9		24-28-30-36	9
35	24	9		26-32-36-40-44	8
40	18-22-26	9		26-32-36-40-44	9
<u>24-ft. Bay</u>					
10 stories	16	8			
15	20	8			
20	22	9			
25	24	9			
30	26	9			
35	22-26-30	9			
40	20-24-28-32	9			

TABLE II

RESIDENTIAL BUILDINGS

COLUMN & CAISSON QUANTITIES

L.L: 40 psf

Sup. D.L. = 20 psf

PLATES	No.st	L/W Cost sq.ft		N/W Cost sq.ft		Reinf. #/sq.ft		Caisson Vol. sq.ft	
		Col.	Caisson	Col	Caisson	L/W	N/W	L/W	N/W
<u>15 ft sq</u>	10	0.30	0.42	0.32	0.42	0.30	0.3	0.065	0.065
	15	0.30	0.30	0.34	0.50	0.3	0.4	0.065	0.077
	20	0.30	0.38	0.37	0.39	0.3	0.4	0. 58	0.060
	25	0.40	0.31	0.48	0.33	0.3	0.6	0. 48	0.050
	30	0.51	0.27	0.59	0.41	0.4	0.6	0. 42	0.063
	35	0.58	0.35	0.66	0.36	0.58	0.7	0.054	0.056
	40	0.53	0.32	0.63	0.33	0.40	0.7	0.049	0.051
	45								
<u>18 ft sq</u>	10	0.22	0.31	0.26	0.52	0.20	0.3	0.047	0.08
	15	0.32	0.36	0.37	0.38	0.30	0.4	0.056	0.058
	20	0.37	0.41	0.40	0.42	0.30	0.5	0.063	0.065
	25	0.41	0.35	0.50	0.37	0.30	0.6	0.054	0.056
	30	0.46	0.31	0.57	0.42	0.40	0.7	0.047	0.065
	35	0.56	0.36	0.69	0.37	0.50	0.8	0.055	0.058
	40	0.53	0.33	0.69	0.43	0.40	0.9	0.050	0.066
	45								
<u>21 ft sq</u>	10	0.22	0.40	0.27	0.42	0.20	0.3	0.061	0.064
	15	0.34	0.41	0.40	0.43	0.30	0.4	0.064	0.066
	20	0.34	0.34	0.44	0.46	0.30	0.6	0.052	0.071
	25	0.42	0.37	0.49	0.50	0.30	0.5	0.057	0.078
	30	0.50	0.42	0.65	0.43	0.40	0.8	0.065	0.067
	35	0.59	0.37	0.75	0.49	0.50	0.9	0.057	0.075
	40	0.55	0.34	0.72	0.4	0.40	0.9	0.052	0.068
	45								
<u>24 ft sq</u>	10	0.22	0.48	0.29	0.49	0.20	0.4	0.073	0.078
	15	0.32	0.46	0.43	0.49	0.30	0.5	0.070	0.076
	20	0.39	0.37	0.54	0.50	0.30	0.7	0.057	0.077
	25	0.46	0.40	0.62	0.52	0.40	0.8	0.061	0.080
	30	0.54	0.43	0.69	0.55	0.50	0.8	0.067	0.085
	35	0.56	0.40	0.73	0.50	0.40	0.9	0.061	0.077
	40	0.58	0.43			0.50		0.065	
	45								

TABLE III

OFFICE BUILDINGS

COLUMN & CAISSON QUANTITIES

L.L. 50 psf

Sup. D.L. = 30 psf

<u>SLABS</u>	<u>No.st.</u>	<u>L/W Cost sq.ft</u>		<u>N/W Cost sq.ft</u>		<u>Reinf. #/sq.ft</u>		<u>Caisson Vol./sq.ft</u>	
		<u>Col.</u>	<u>Caisson</u>	<u>Col.</u>	<u>Caisson</u>	<u>L/W</u>	<u>N/W</u>	<u>L/W</u>	<u>N/W</u>
<u>24 ft. sq.</u>	10	\$0.35	0.49	0.47	0.52	0.3	0.6	0.076	0.079
(A: 576°)	15	0.47	0.47	0.66	0.64	0.4	0.5	0.073	0.099
<u>D.L.</u>	20	0.65	0.50	0.81	0.52	0.5	0.9	0.077	0.080
<u>N/W:136</u>	25	0.76	0.52	1.00	0.56	0.6	1.2	0.080	0.086
<u>L/W:107</u>	30	0.94	0.46	1.21	0.59	0.7	1.5	0.072	0.090
	35	0.94	0.49	1.22	0.60	0.7	1.5	0.075	0.092
	40	1.01	0.53	1.26	0.56	0.8	1.5	0.081	0.086
	45	1.06	0.48	1.42	0.60	0.8	1.8	0.074	0.092
<u>27 ft. sq.</u>	10	0.38	0.56	0.51	0.58	0.3	0.7	0.086	0.090
(A:727°)	15	0.53	0.53	0.77	0.69	0.4	1.1	0.081	0.106
<u>D.L.</u>	20	0.65	0.52	0.87	0.65	0.5	1.1	0.079	0.100
<u>N/W:150</u>	25	0.73	0.54	0.97	0.66	0.6	1.2	0.083	0.102
<u>L/W:117</u>	30	0.81	0.55	1.06	0.67	0.6	1.3	0.085	0.103
	35	0.89	0.50	1.16	0.63	0.7	1.4	0.078	0.097
	40	0.97	0.53	1.29	0.65	0.7	1.6	0.082	0.100
	45	1.02	0.56	1.41		0.8	1.8	0.087	
<u>30 ft.sq.</u>	10	0.34	0.47	0.42	0.62	0.3	0.5	0.073	0.095
(A:900°)	15	0.43	0.44	0.53	0.56	0.3	0.6	0.068	0.086
<u>D.L.</u>	20	0.61	0.45	0.72	0.54	0.5	0.8	0.069	0.084
<u>N/W:130</u>	25	0.63	0.45	0.76	0.55	0.5	0.8	0.069	0.085
<u>L/W:106</u>	30	0.71	0.46	0.86	0.58	0.5	0.9	0.071	0.089
(Beam-	35	0.84	0.49	0.93	0.59	0.6	0.9	0.076	0.090
Slab)	40	0.87	0.51	1.04		0.7	1.1	0.079	
	45	0.96		1.11		0.8	1.2		

RESIDENTIAL BUILDINGS
FROM 10 TO 40 STORIES
REINFORCING COLUMN QUANTITIES
INTERIOR FLAT PLATES, SQUARE BAYS

Normal weight Concrete ———
Lightweight Concrete - - - -

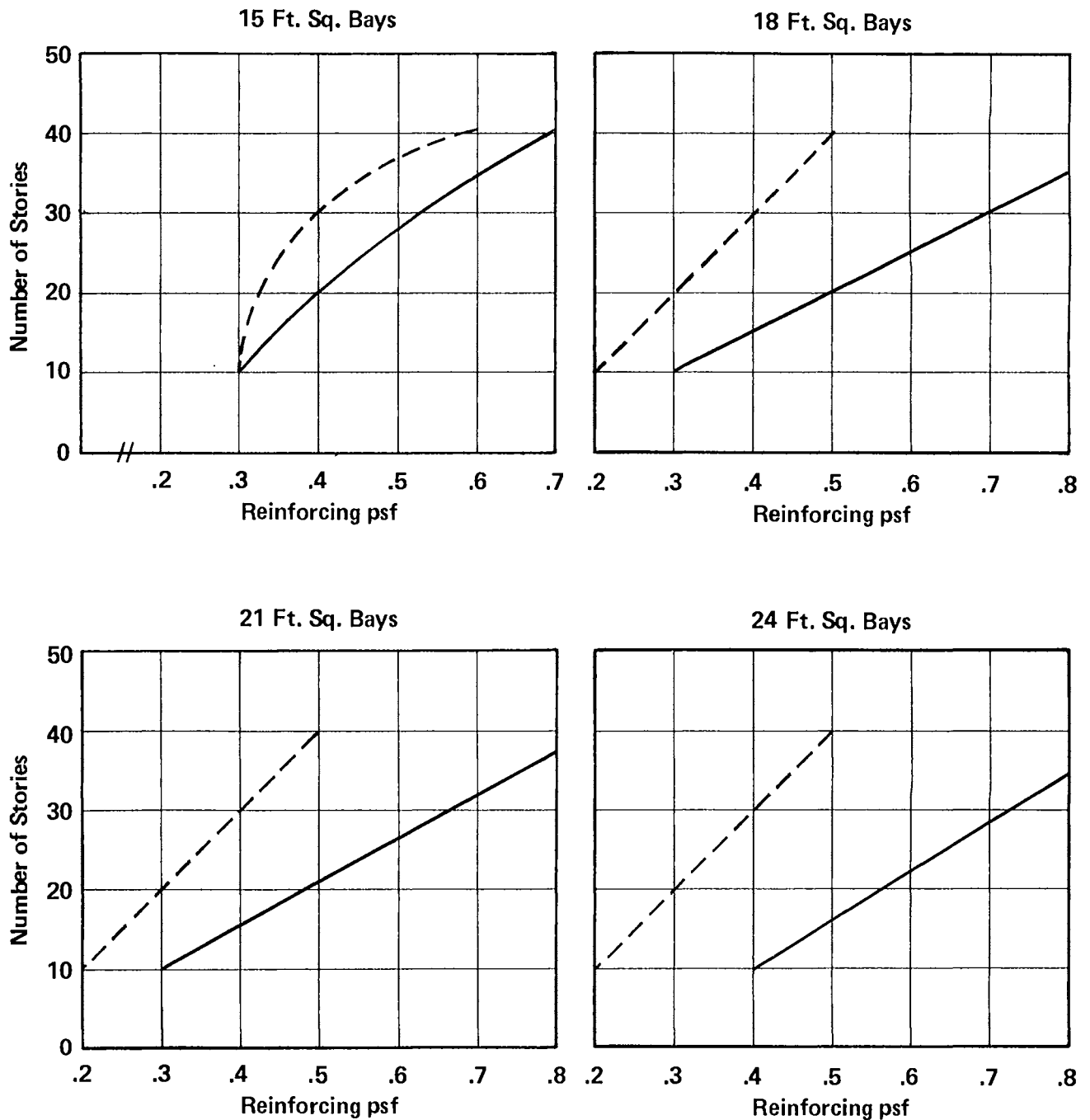


FIGURE 5

**OFFICE BUILDINGS
FROM 10 TO 40 STORIES
REINFORCING COLUMN QUANTITIES
INTERIOR FLAT SLABS, SQUARE BAYS**

Normal weight Concrete —————

Lightweight Concrete - - - - -

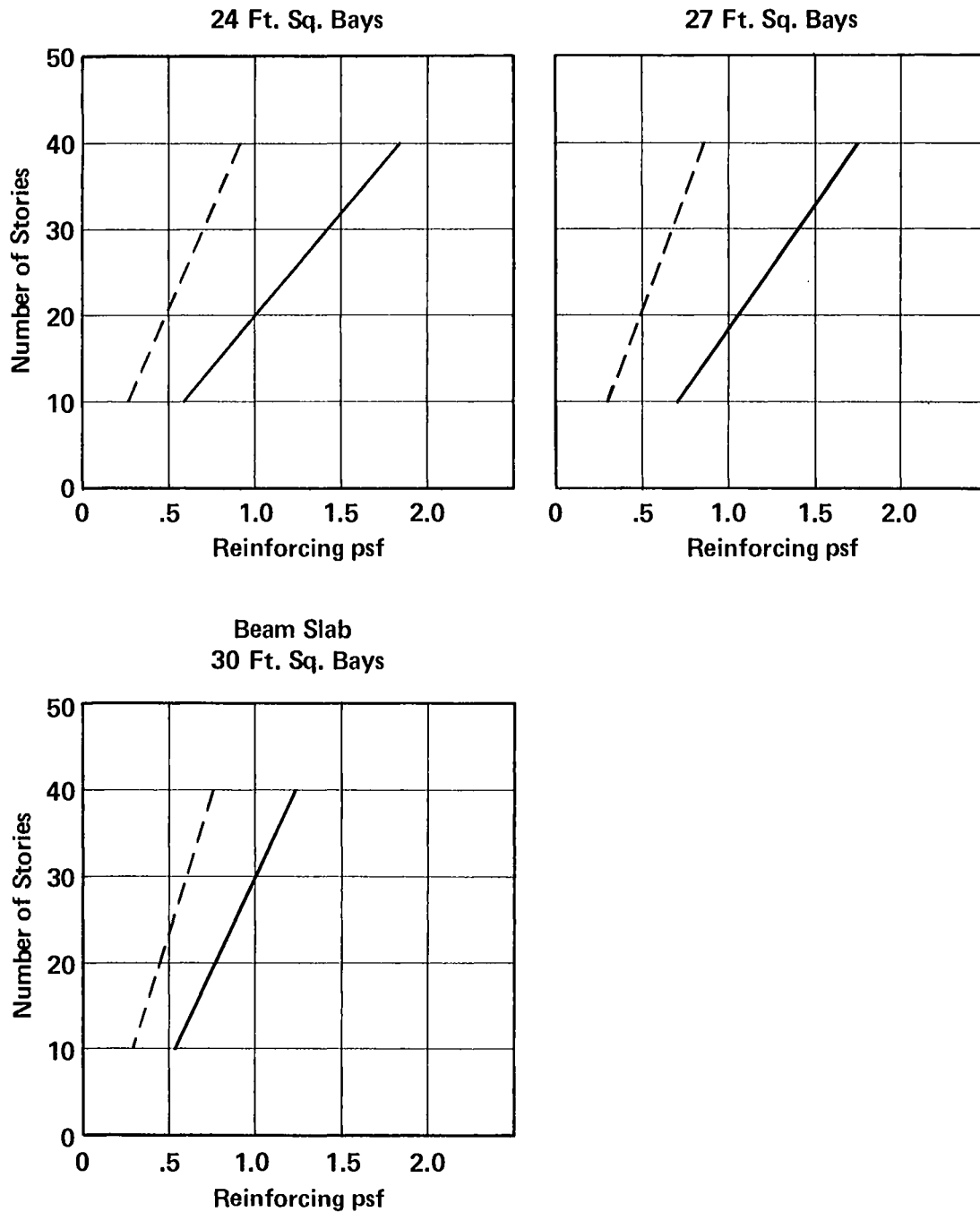


FIGURE 6

REINFORCING (60 ksi) FOR NORMAL WEIGHT COLUMNS SUPPORTING PLATES

(D. L. + 20 psf, L. L. = 40 psf, Chicago Code L. L. Reduction)

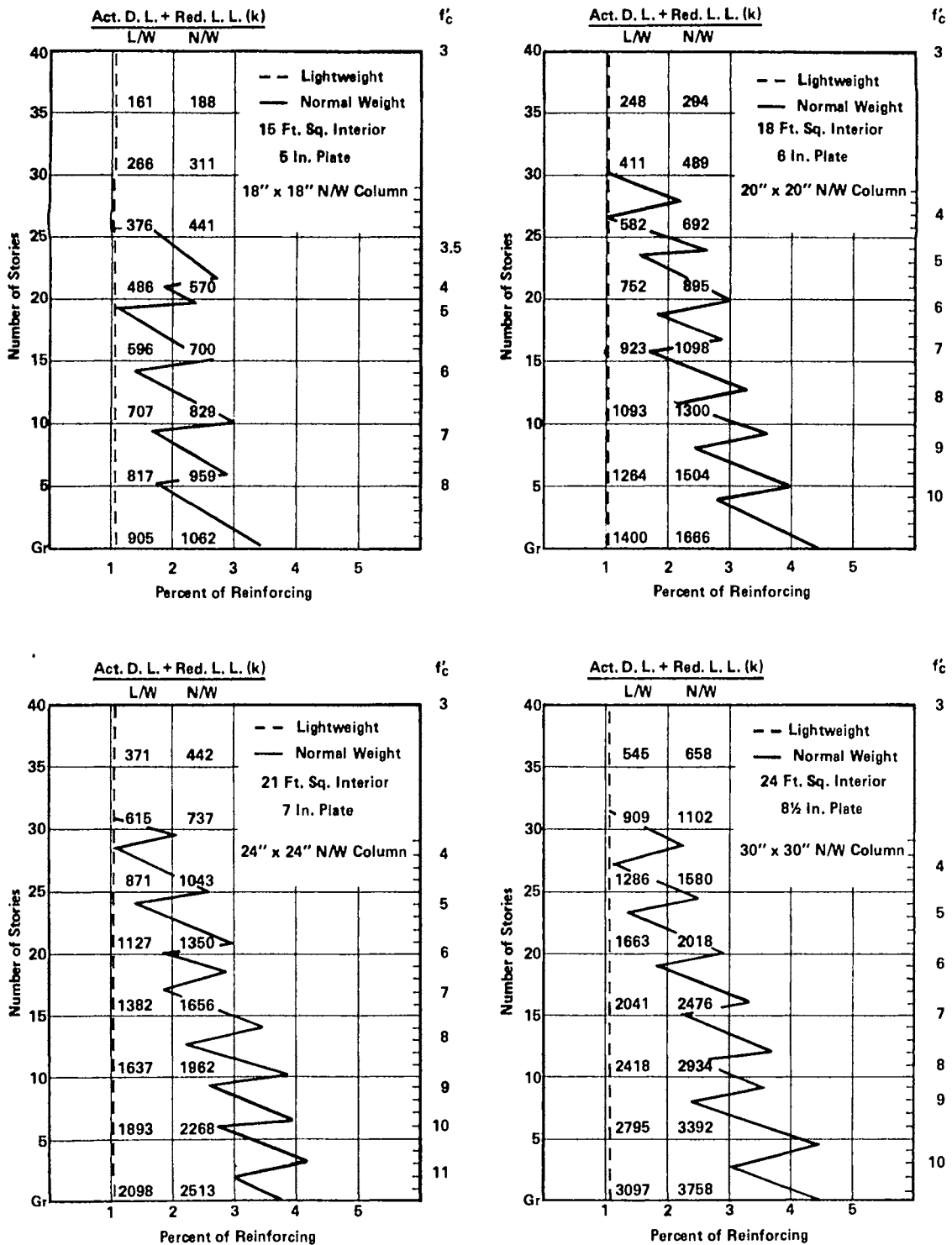


FIGURE 7

REINFORCING (60 ksi) FOR NORMAL WEIGHT COLUMNS SUPPORTING SLABS
(D. L. + 20 psf, L. L. = 80 psf, Chicago Code L. L. Reduction)

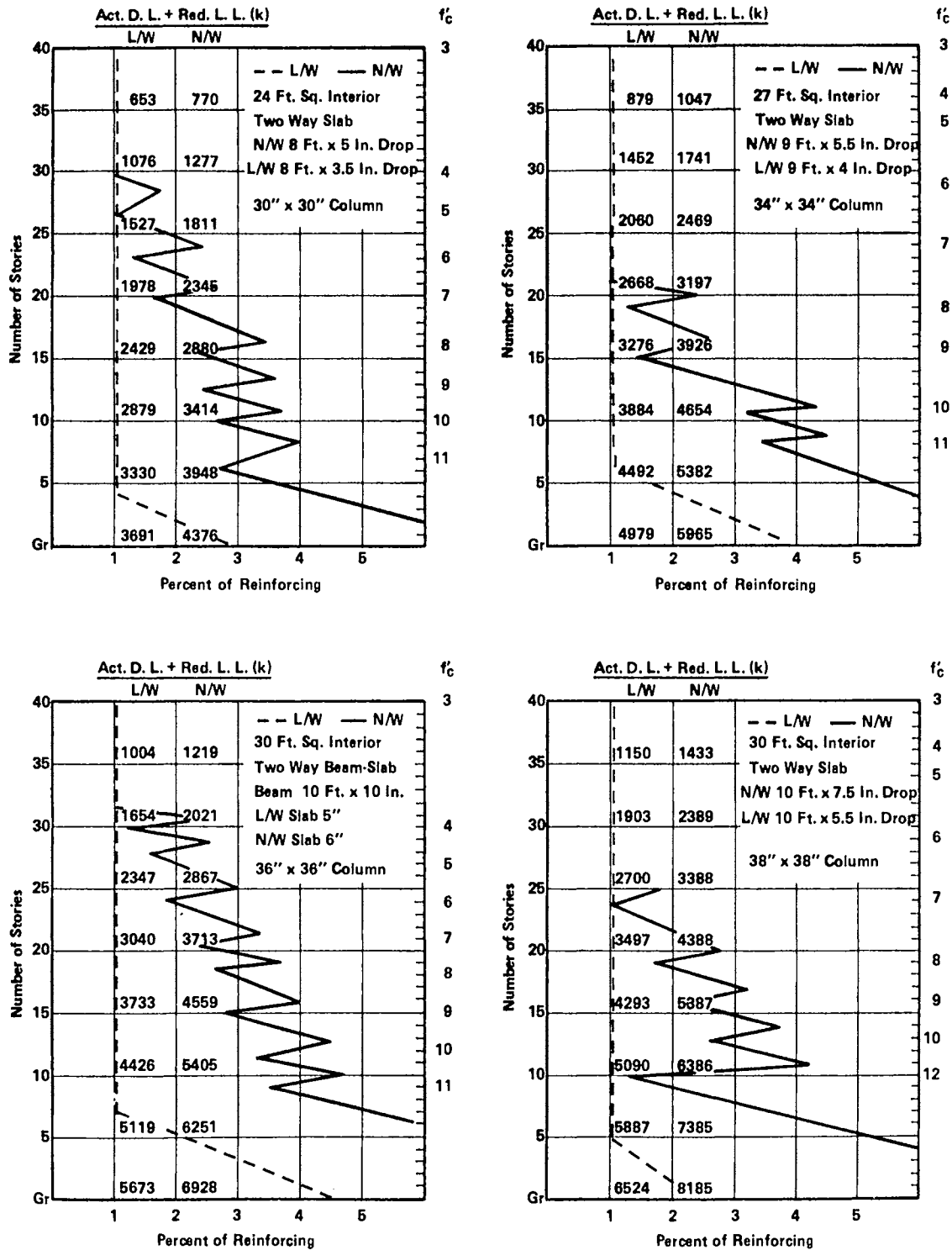


FIGURE 8

**OFFICE BUILDINGS
CAISSON VOLUME QUANTITIES
INTERIOR FLAT SLABS, SQUARE BAYS**

Normal weight Concrete ———
Lightweight Concrete - - - - -
Concrete: 5 ksi, Soil Capacity: 30 ksf at 75 ft.

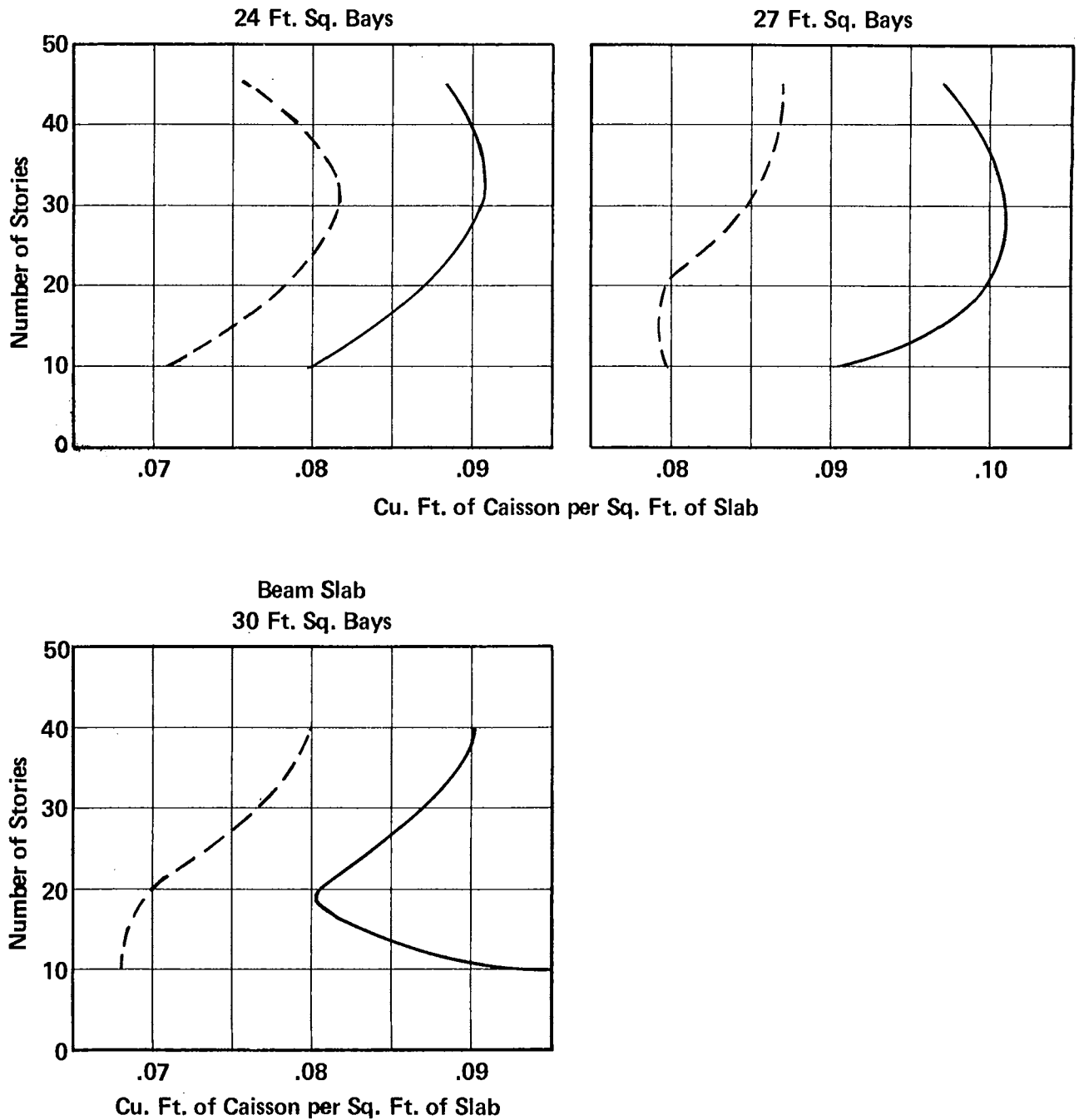


FIGURE 9

**RESIDENTIAL BUILDINGS
CAISSON VOLUME QUANTITIES
INTERIOR FLAT PLATES, SQUARE BAYS**

Normal weight Concrete ———
Lightweight Concrete - - - -
Concrete: 5 ksi, Soil Capacity: 30 ksf at 75 ft.

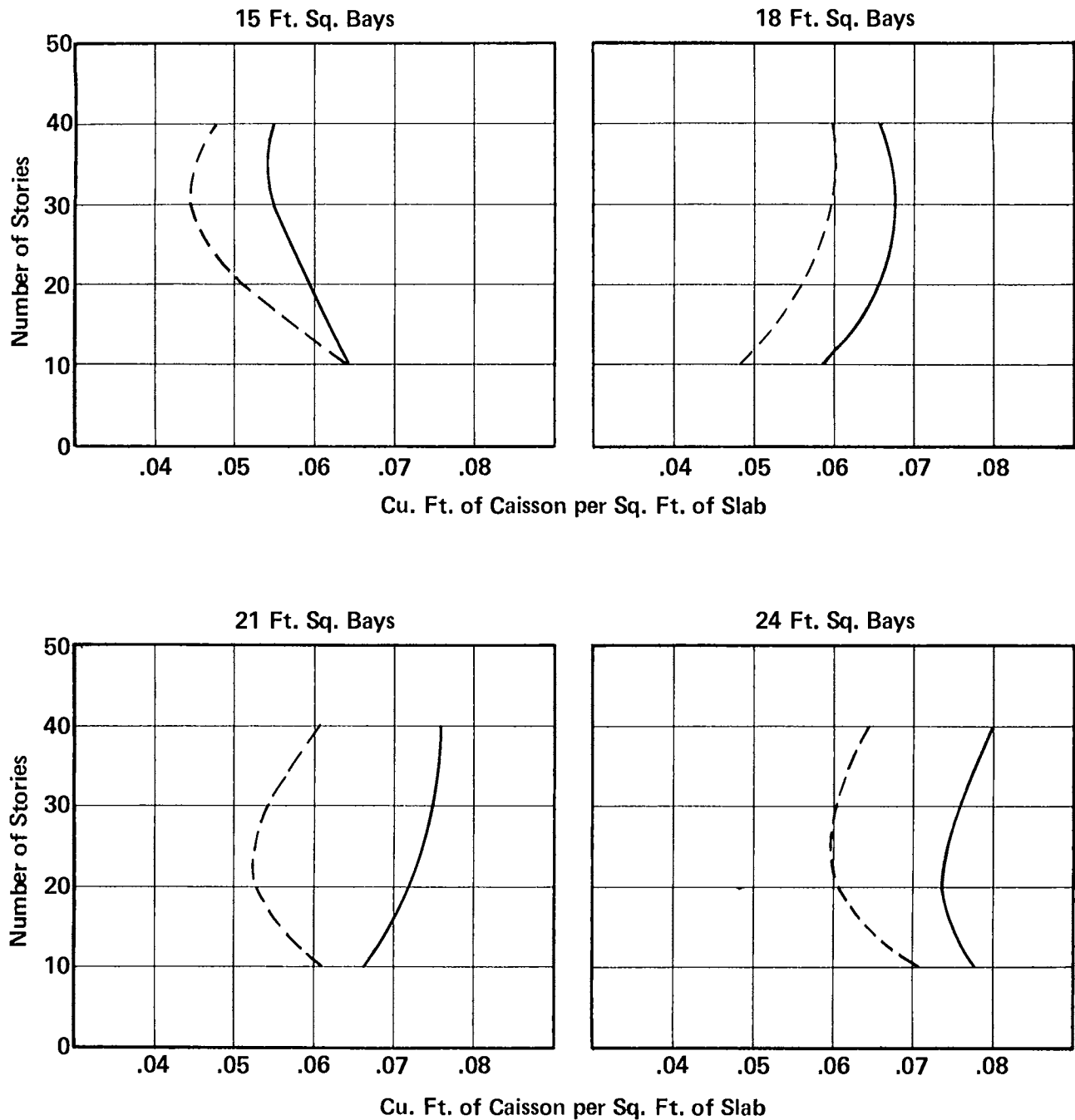


FIGURE 10

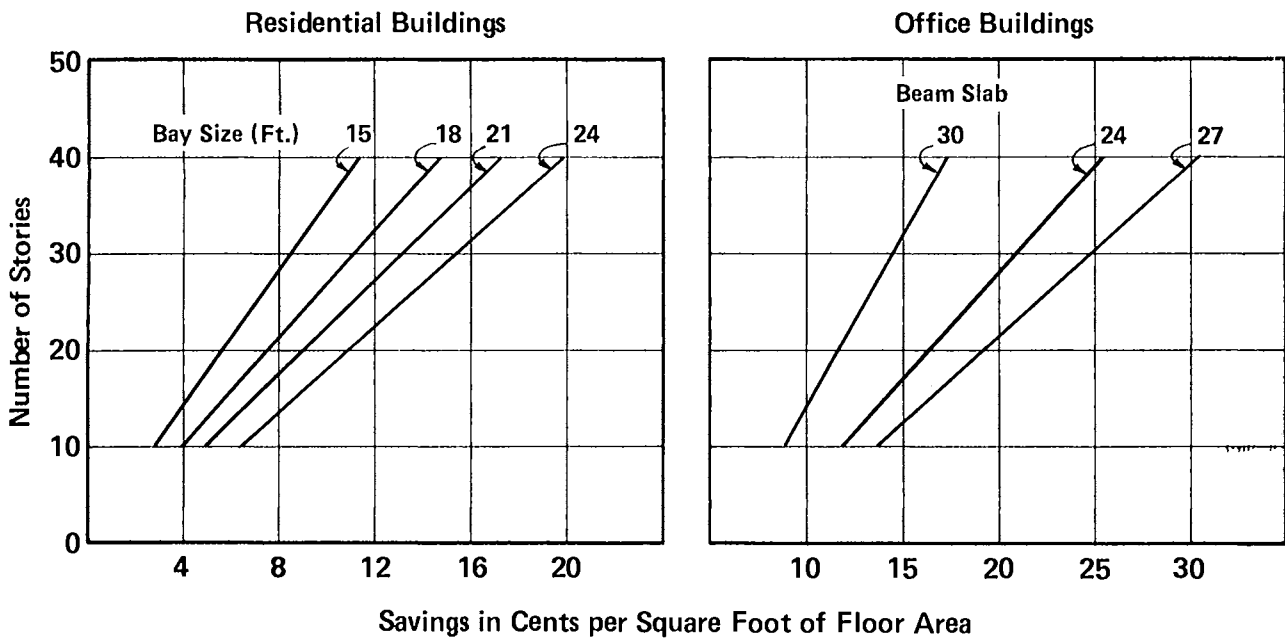
TABLE IV
SLAB PRICES

<u>Bay Size</u>	<u>Concrete</u>		<u>Reinforcing</u>		<u>Forms</u>	<u>Finishing</u>	<u>Total \$/sq. ft.</u>	<u>Dif.</u>
	Quant. (cu.ft./ sq.ft.)	Price (\$)	Quant. (lb)	Price (\$)	Price/ sq.ft.	Price/ sq.ft.		
15 ft.								
N/W	.42	1.03	1.75	.67	1.30	.30	3.30	
L/W	.42	1.21	1.68	.64	1.30	.30	3.45	+.15
18 ft.								
N/W	.50	1.22	1.80	.68	1.30	.30	3.50	
L/W	.50	1.44	1.75	.66	1.30	.30	3.70	+.20
21 ft.								
N/W	.58	1.42	2.10	.80	1.30	.30	3.82	
L/W	.58	1.68	1.90	.72	1.30	.30	4.00	+.18
24 ft.								
N/W	.71	1.74	2.40	.91	1.30	.30	4.25	
L/W	.71	2.05	2.20	.84	1.30	.30	4.49	+.24
24 ft.								
N/W	.71	1.74	2.35	.89	1.30	.30	4.23	
L/W	.71	2.05	2.10	.80	1.30	.30	4.45	+.22
27 ft.								
N/W	.80	1.96	2.70	1.02	1.30	.30	4.58	
L/W	.80	2.31	2.25	.85	1.30	.30	4.76	+.18
30 ft.								
N/W	.73	1.78	4.20	1.60	1.30	.30	4.98	
L/W	.70	2.02	3.80	1.44	1.30	.30	5.06	+.12

Normal weight and lightweight concrete is 4 ksi (\$66/cu. yd. for normal weight in place)
Additional cost for lightweight concrete is \$12/cu. yd. (\$78/cu. yd.)

SAVINGS IN INTERIOR COLUMNS AND CAISSONS WHEN LIGHTWEIGHT CONCRETE IS USED IN THE FLOORS

COLUMN SAVINGS



CAISSON SAVINGS

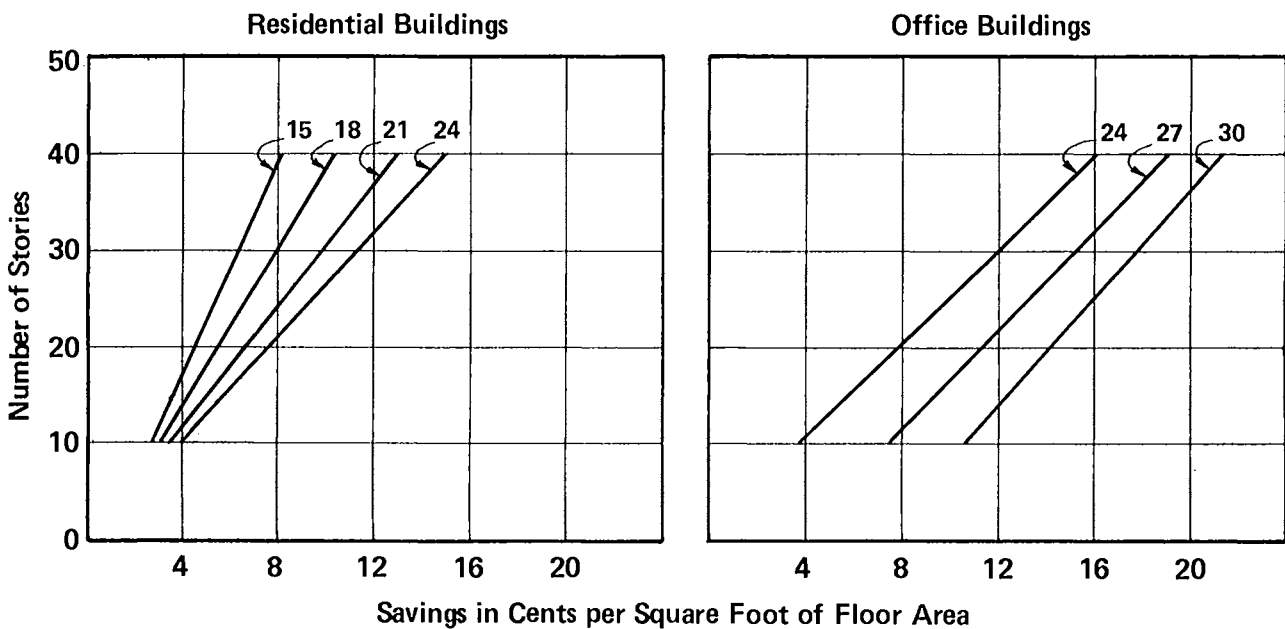


FIGURE 11

**LATERAL DISPLACEMENT
CHICAGO CODE FOR WIND LOADS
RESIDENTIAL BUILDINGS**

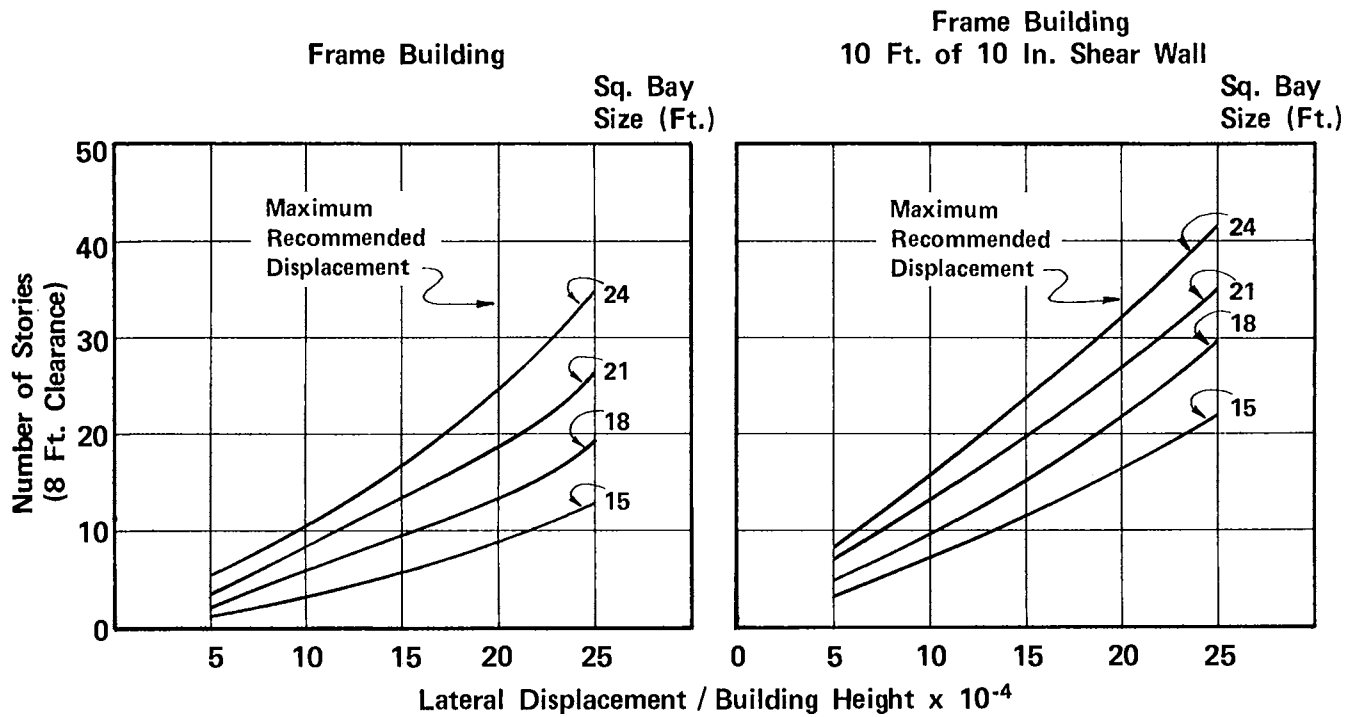


FIGURE 12

TOTAL SAVINGS USING LIGHTWEIGHT CONCRETE IN THE FLOORS

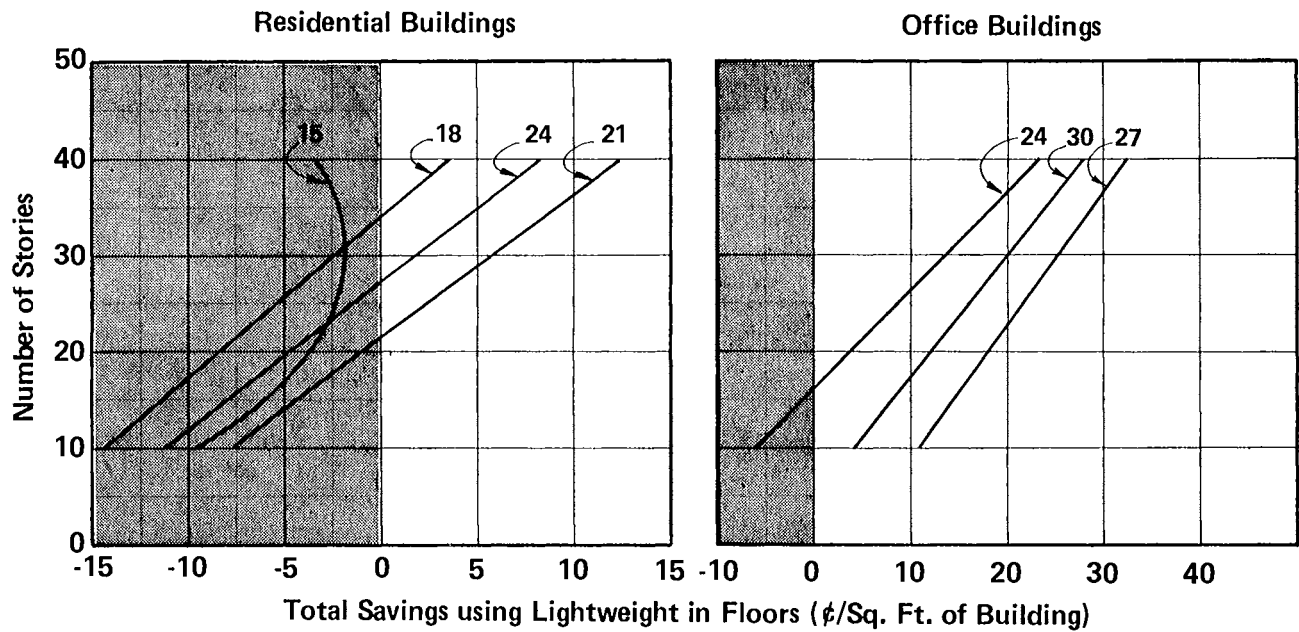


FIGURE 13

ADDENDUM I

COST IN CENTS PER KIP OF AXIAL LOAD

20 x 20 COLUMN

PER FOOT OF LENGTH

REINFORCING

FORMS

CONCRETE/CU. YD.

60 k Grade = \$760/ton

	f'_c	Cost + Placing = Total
\$2.80/sq.ft.	7.5 ksi	\$ 64.40 + 16.00 = \$ 80.40
	9.0 "	68.62 " = 84.62
	11.0 "	88.27 " = 104.27
	14.0 "	117.74 " = 129.27

$\frac{f'_c}{f_y}$	$\frac{7500}{60 \quad 75}$		$\frac{9000}{60 \quad 75}$		$\frac{11,000}{60 \quad 75}$		$\frac{14,000}{60 \quad 75}$	
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Forms, 6.67 Sq.Ft @ \$2.80/Sq.Ft	18.67	18.67	18.67	18.67	18.67	18.67	18.67	18.67
Concrete, (\$ /27.0) Area x 1 Ft.	8.31	8.31	8.74	8.74	10.77	10.77	13.36	13.36
Reinf.								
Ties 3.04 lb. @ \$0.38/lb	1.16	1.16	1.16	1.16	1.16	1.16	1.16	1.16
Vertical								
4" or 1%, 13.6 lb. @ 0.38	5.17		5.17		5.17		5.17	
13.6 lb. 0.405		5.51		5.51		5.51		5.51
16" or 4% 54.4 lb. @ 0.38	20.67		20.67		20.67		20.67	
0.405		22.03		22.03		22.03		22.03
32" or 8% 108.8 lb. @ 0.38	41.34		41.34		41.34		41.34	
0.405		44.06		44.06		44.06		44.06

$P = 0.8 \times 0.7 [0.85 f'_c (A_g - A_s) + f_y A_s]$ in kips

4" or 1%	15.47	15.81	18.31	18.64	22.08	22.41	27.73	28.07
16" or 4%	19.08	20.43	21.82	23.17	25.48	26.82	30.96	32.31
32" or 8%	23.88	26.56	26.52	29.20	30.02	32.70	35.27	37.96

Cost, \$ /ft. of column

1%	33.31	33.65	33.74	34.08	35.77	36.11	38.36	38.70
4%	48.81	50.17	49.24	50.60	51.27	52.63	53.86	55.22
8%	69.48	72.20	69.91	72.63	71.94	74.66	74.53	77.25

Cost, ¢ /kip

1%	2.15	2.13	1.84	1.82	1.62	1.61	1.38	1.38
4%	2.55	2.46	2.26	2.18	2.05	1.96	1.74	1.71
8%	2.90	2.72	2.64	2.49	2.40	2.28	2.11	2.04