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Structural Design of Dallas Main Center

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William LeMessurier graduated from Harvard University (A.B.c.l.) in 1947 and Massachusetts Institute of Technology (M.S.) in 1953. Although professionally an engineer, Mr. LeMessurier trained originally as an architect at Harvard. His design engineering practice is directed to structural innovation with a goal of constantly advancing the state of the art, combined with a special concern for structural aesthetics related to architecture. The Staggered Truss System and the use of Tuned Mass Dampers are two examples of this innovative spirit. In both teaching and practice, Mr. LeMessurier strives for spontaneous professional understanding between engineer and architect.

Mr. LeMessurier is a member of, among many professional societies, ASCE, ACI, NAE and the Boston Society of Structural Engineers. He has been the recipient of AISC's Award of Excellence five times, as well as the AISC's Special Award, in 1972.

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He has been involved in structures from Houston to New York, and Singapore to Saudi Arabia, including the Citycorp Center, New Boston City Hall, King Khalid Military City and Singapore Treasury Building.

SUMMARY

This paper will discuss a new design approach to very tall, slender buildings, used in the Dallas Main Center, which is a substantial advance in the state of the art. The building, a project of Bramalea Limited of Toronto, is a 73-storey office building rising 920 feet above grade, presently under construction in Dallas, Texas.

A three dimensional moment resisting steel frame spanning the entire building, carries all gravity loads to sixteen exterior concrete columns. Wind and gravity shears are resisted by the steel frame, while all axial gravity loads and overturning forces from wind are resisted by the concrete columns. The paper will discuss the design concept and its analysis, the special construction techniques required to assemble the steel frame, and the use of very high-strength concrete in the columns. Important aspects of the wind engineering, including a summary of wind tunnel testing and the consideration of an unusual vortex shedding phenomenon are discussed.

SOMMAIRE

Cette présentation traitera d'un nouveau concept de calcul appliqué à un édifice très élancé, au coeur du Dallas Main Center qui représente un grand progrès dans le domaine du calcul des charpentes. Ce bâtiment, dont le promoteur est Bramalea Limited de Toronto, est un édifice à bureau de 73 étages s'élevant environ 920 pieds au-dessus du sol, en voie de construction à Dallas au Texas. Un cadre rigide spatial qui enjambe un bâtiment au complet supporte les seize poteaux de béton extérieurs. Les efforts tranchants dûs au vent et au poids propre sont repris par la charpente d'acier, tandis que les charges de gravité axiales et les efforts de renversement dûs au vent sont contrecarrés par les poteaux en béton. La présentation portera sur la solution technique retenue et sur son analyse, les techniques de construction spéciales utilisées pour monter la charpente et l'utilisation d'un béton à très haute résistance pour les poteaux. Seront également abordés certains aspects importants des problèmes inhérents au vent dont, le résumé des essais en soufflerie et le phénomène inusité de délestage dû à l'effet de tourbillon.

STRUCTURAL DESIGN OF DALLAS MAIN CENTER

INTRODUCTION

Over the past twenty years, almost all very tall buildings have used some form of exterior tube structure, usually built with frames of steel such as New York's World Trade Center and Chicago's Sears Tower or frames of composite concrete and steel such as Houston's Texas Commerce Bank. Occasionally, braced exterior tubes have been used as in Chicago's John Hancock Tower and New York's Citicorp Center. These structures have greatly reduced the material previously required for strength and rigidity when buildings such as New York's Empire State Building were built with parallel rigid bents.

The structural design of the 70-story InterFirst Plaza at Dallas Main Center began with a challenge from the owners and the architect to look for a new system for the very tall building. The building's architectural form was established from the beginning by Bramalea's own staff and was chosen to maximize the number of corners on each floor and to generate a pleasing profile at the top. The basic external geometry was adopted by Dallas architects Jarvis, Putty, Jarvis and changed very little over the development of the design. (See Figures 1 and 2.)

PRELIMINARY CONSIDERATIONS

The use of a framed or braced external tube in Dallas Main Center would have effectively obstructed the tenants' views from the sixteen corners, therefore destroying the intent of the plan. A framed tube of steel with verticals at fifteen-foot centers was studied for comparative purposes but found to have no advantage over the final design.

In order to preserve the openness provided by the floor to ceiling reflective glass walls, it was decided to cantilever the entire perimeter up to twenty feet from column centerlines giving complete flexibility for exterior office arrangements. This decision generated sixteen columns set back from the building perimeter.

The service core of the building had a long evolution resulting in a novel design having an open central bay with radiating elevator lobbies. The geometry of the core relates logically to the unusual perimeter configuration and has proved to be attractive to tenants.

The building diagram thus became a core of seven thirty-foot square bays plus sixteen columns in the arrangement shown in Figure 2. The usual approach to design with this configuration would be to consider carrying all lateral loads on concrete walls or braced steel frames in the fixed elements of the core. The core is 85 feet across its shortest dimension which is generous enough to brace fifty stories plus two basements at a height to width ratio of eight to one. Since Dallas Main Center has a height of 950 feet above footing elevation, the core alone would have a height to width ratio of more than eleven to one. At such a high aspect ratio the requirements of lateral stiffness and overturning resistance made reliance on the core alone unfeasible. By connecting the core to the columns with stiff wind girders in the floors, this difficulty could be overcome.

Designs were developed in both steel and concrete using the core walls connected to the columns with haunched girders. An all steel design at forty pounds per square foot was much too costly, but an all concrete design with 24-inch core walls, haunched girders and ribbed floors was seriously considered. After detailed analysis by Austin Commercial, the General Contractor, it was found that the economy of concrete was offset by a substantially greater construction time than needed for a steel building. In 1981, when the choices were made, the cost of construction financing was at a historical high which severely penalized concrete. So the owners asked us to search for a solution using a maximum of thirty pounds of steel per square foot.

Since the external framed tube was architecturally undesirable and the conventional braced core with rigid girders too costly, radical innovation was demanded. At times like this, the writer has found that the best strategy is to return to basic principles of structural design.

DESIGN PHILOSOPHY

In a very tall building, the central problem is to resist the lateral force of wind. Efficient resistance to overturning and excessive lateral movement must be provided by the structural system. The force of the wind on a tall building almost always has a very large dynamic component which is in turn a function of the building's mass and stiffness as well as its natural damping.

If one discards the conventional wisdom of designing for gravity first and designs directly for wind, different structures result. If one thinks of a tall building as a cantilever into space from the face of the earth, one naturally would concentrate the bending strength in chords at the edges of the structure and interconnect these chords with an efficient shear system.

The deformation of such a building consists of two parts. Overall bending of the building results from the stretching and compressing of the chords and overall shear distortion comes from the local deformation of the "web" elements which interconnect the chords. The boundary that separates very tall buildings from ordinary high rises could be defined as the point where bending deformation is more than half of the total for a given structural configuration.

STRUCTURAL DESIGN

Applying this philosophy to Dallas Main Center suggested using only the sixteen perimeter columns as "chords" and interconnecting them with a suitable shear system. An ideal shear system might be an egg crate of thin walls on all grid lines or a system of diagonal braces passing through interior space. Neither of these would be suitable for the building's ultimate purpose as open office space. The remaining possibility was a three-dimensional rigid frame with horizonal members at thirty-foot centers connected to vertical members in the core walls. Making an initial assumption that half of the building deformation would be in the chords and half in the shear system, the member sizes were found for chords and frames for an assumed wind load of thirty pounds per square foot and lateral deflection of one part in four hundred. Thirty-six inch wide flange rolled steel shapes were assumed for the rigid frame elements and sized for drift. The resulting frame was then checked for its capacity to carry all gravity load and span between the sixteen columns, an average distance of 135 feet. Surprisingly, little change in member sizes resulted from the consideration of gravity loads using steel with a 50 ksi yield point.

To be economical, a steel structure must have a minimum of complex field connections. The three-dimensional rigid frame was designed to be built with elements spliced at points of zero moment wherever possible. This resulted in subassemblies called horizontal "trees" made of a thirty-foot long girder with stubs projecting one-half story above and below at its quarter points. (See Figure 2.)

In early consultation with the steel fabricators, Mosher Steel Co., and the steel erectors, John F. Beasley Co., all of Dallas, the general approach to fabrication and erection was worked out. Analysis showed that four horizontal levels of the three-dimensional Vierendeel frame were sufficient to span the building when connected. By using temporary jacks, four levels could be erected with one inch camber and then released to span the entire building. After release, the four new levels can be connected to deliver shear only to levels below.

With a suitable steel web system to span the building, the sixteen columns all participate in resisting axial forces from wind as well as gravity. But because of the extreme ratio of the building's height to structural width, the axial wind forces are very large, and pure steel columns would be designed for axial stiffness rather than strength. In this case, concrete has a great cost advantage. Compared to concrete having a strength of 10,000 psi, the cost for an equal amount of axial stiffness in steel is five times higher.

The scheme which was finally adopted uses concrete for the sixteen columns and steel for the three-dimensional Vierendeel frame and floor construction. A 36-inch steel column was used in the center of the concrete to facilitate steel erection and was designed to carry gravity and wind forces for nine stories above the concrete. In the final form, the columns are actually reinforced concrete with 1% of 75 ksi reinforcing steel and 1% of 50 ksi structural steel. The largest columns are seven feet square and carry 30,000 kips of gravity and 10,000 kips of wind.

The design of the lowest stories at the lobby level is different from typical floors because of the architect's wish to have a dramatically high lobby. Only the service core exists below the third level above grade. To replace the rigidity lost by eliminating the lower floor girders, the walls of the service cores have diagonal bracing to carry the total wind shear which is finally transmitted to the concrete columns through the diaphragm action of the floor just above the footing level. All vertical and horizontal forces are carried to the earth by the sixteen concrete columns supported by individual concrete footings bearing on limestone.

CONSTRUCTION

In order to start the construction of the building, temporary supports were built under the vertical members in the service core area. When twelve stories of steel were in place and permanently connected, the temporary supports were removed. The entire core area was built one inch higher than its final elevation. When the temporary supports were removed in October 1983, the core area came down three/quarters of an inch. The remaining camber was provided for the weight of finish materials to be added later.

After the first twelve levels were completed and lowered, the typical floors have been erected in four floor increments at the rate of three working days per floor. On March 1, 1984 fifty floors should be in place with topping out expected by July 1984.

WIND DESIGN

The preliminary design of the structure prior to wind tunnel tests was based on the combination of a uniform wind load of 30 psf and a drift limit of .0025. From the point of view of stiffness, the writer refers to this criterion as the "rule of 12,000 psf." In other words, the uniform wind force divided by the drift limit must be at least 12,000 psf. Based on previous experience with the Citicorp Tower in New York, it was likely that these criteria would not limit the acceleration in a ten year storm to acceptable levels. To improve human comfort levels if necessary, a mechanical damping system was included in the building budget at 4.5 million dollars.

Based on these criteria, a steel frame resulted weighing twenty pounds per square foot. To this the cost equivalent of the sixteen concrete columns was added at four pounds per square foot assuming erected steel at $1200 per ton and reinforced concrete in place at $270 per yard.

The calculated period of vibration in the short direction was nine seconds including P-Delta effects at realistic live loads. An aeroelastic model was tested by Morrison, Harshfield, Theakston & Rowan, Limited, of Guelph, Ontario, under the direction of Dr. Peter A. Irwin. The tests showed that in winds approximately parallel to the long axis of the building vortex shedding would occur at intervals matching the building's period and leading to a large crosswind dynamic response. Although the wind necessary to produce this response would be a rare event from the critical direction, it was decided to attenuate the return period of the critical wind by shortening the building's period to eight seconds. This was done by adding steel to the shear system increasing the steel weight from twenty to twenty-four pounds per square foot. Fortunately, the decrease in period was enough to reduce accelerations to acceptable levels and a damping mechanism was unnecessary. The construction budget was therefore unaffected by the steel increase. This was a happy result for this project, but it is not typical. In many cases, excessive acceleration cannot be affordably reduced by increased stiffness.

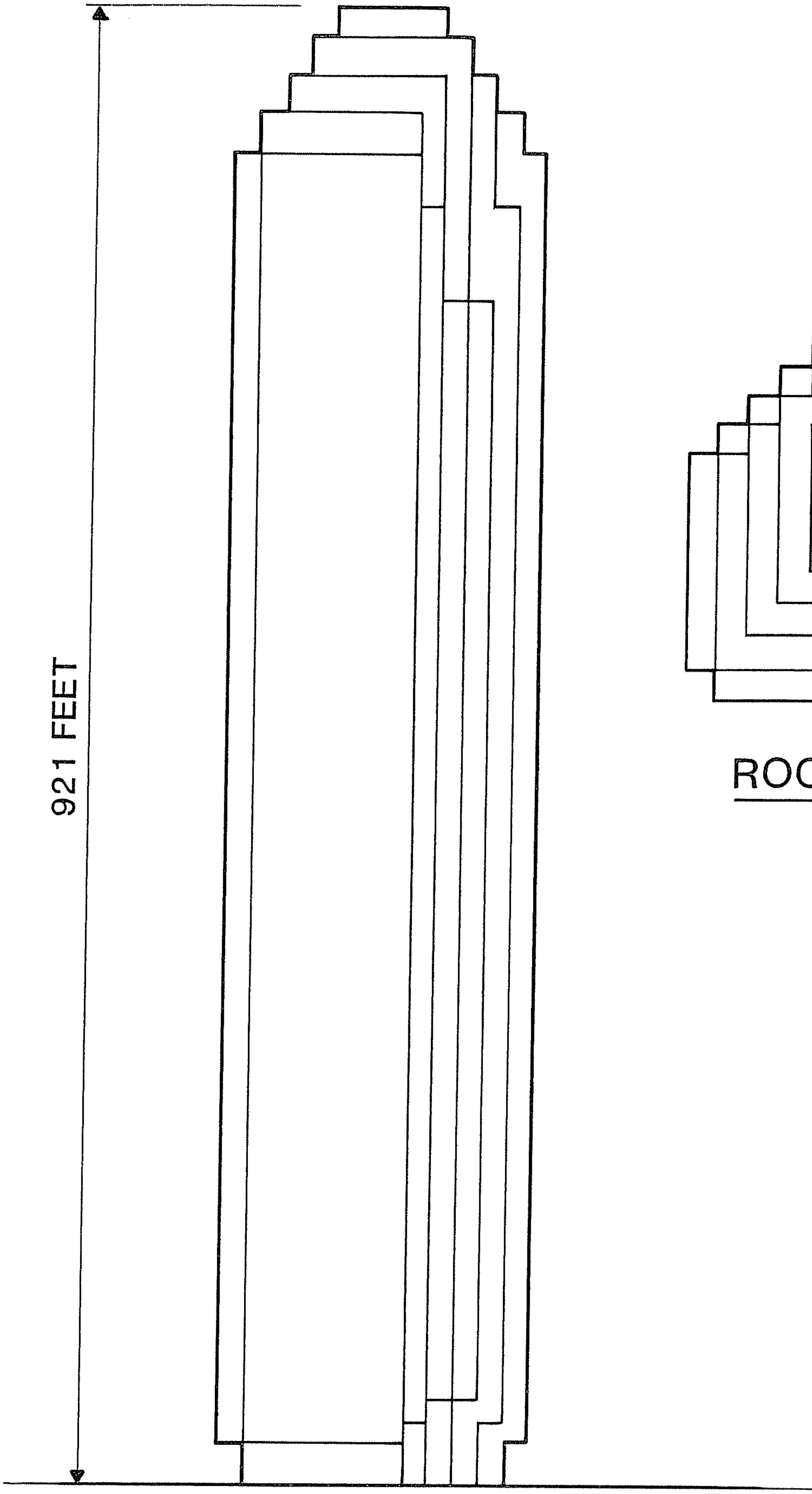
To achieve the increased stiffness in the shear system, it was necessary to provide greater moment of inertia in the floor members of the three-dimensional Vierendeel frame than available in American rolled shapes. Fabricators were given the choice of making built-up sections or using European rolled shapes which are available in heavier sections. The choice of the fabricator was to use special rolled sections from Luxemborg.

CONCLUSION

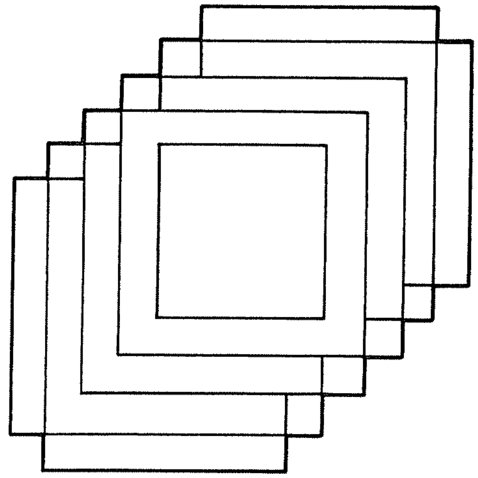
The new tower known as InterFirst Plaza at Dallas Main Center will be Dallas' tallest building at 921 feet above grade and 950 feet above the lowest basement. The building will contain approximately two million square feet with typical floors of 28,900 square feet. The project is a development of Bramalea Limited, InterFirst Bank Dallas and PIC Realty Corporation, a subsidiary of The Prudential Insurance Company of America.

The building's structure is a fundamentally new system for very tall buildings. Using a steel space frame to span the entire building leads to the concentration of all gravity and wind overturning forces on sixteen concrete columns. The building's reflective glass curtain wall is cantilevered up to twenty feet from the columns providing flexible offices with unencumbered views. In all successful designs, the structural system must provide strength, rigidity and functional utility at an attractive price. By combining the virtues of both steel and concrete, the goals of the owner and architect have been achieved.

The structural engineering was carried out by the Brockette/LeMessurier Joint Venture, a partnership for the project of Brockette, Davis, Drake of Dallas and LeMessurier Associates/SCI of Cambridge, Massachusetts.

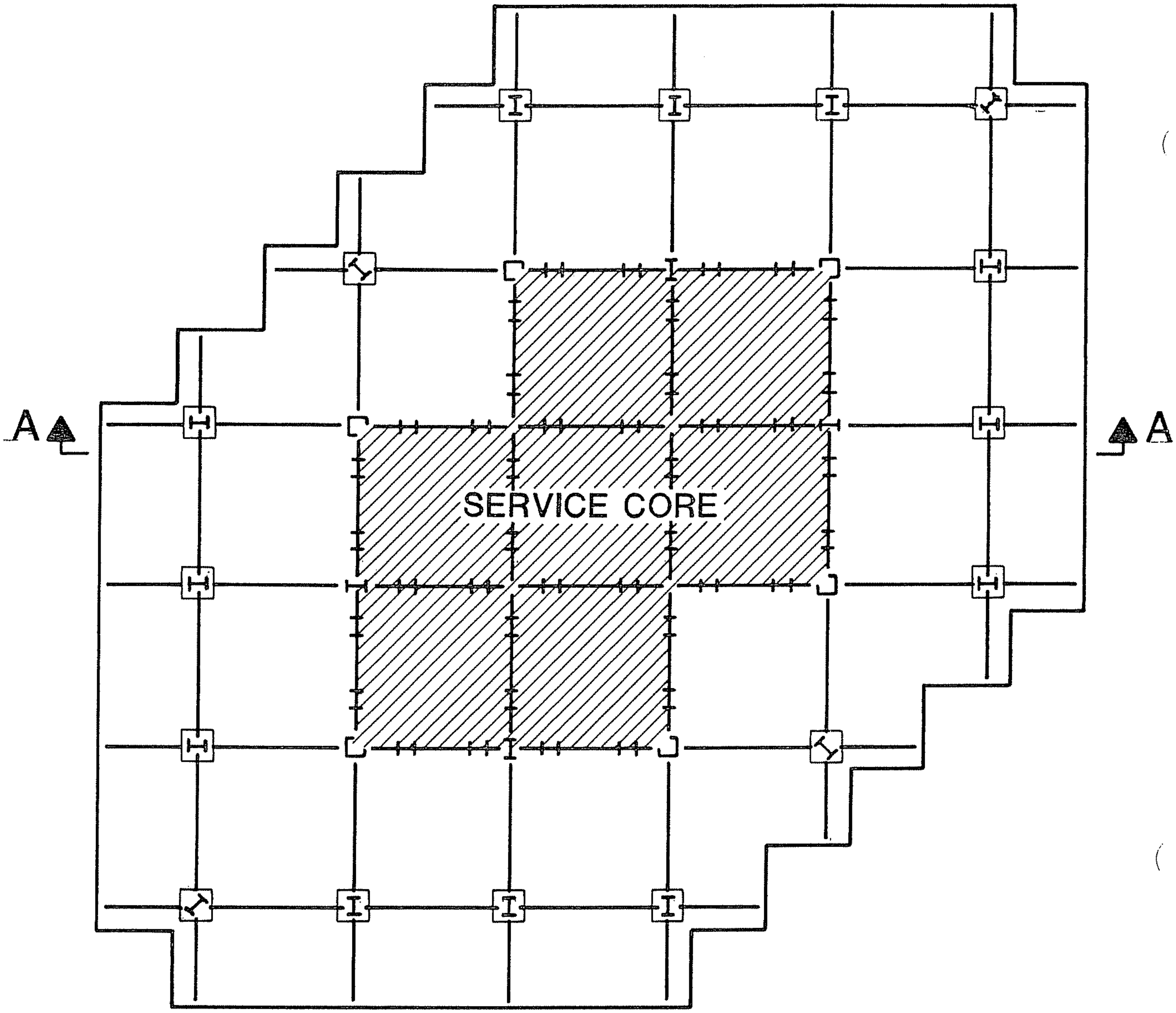


ELEVATION

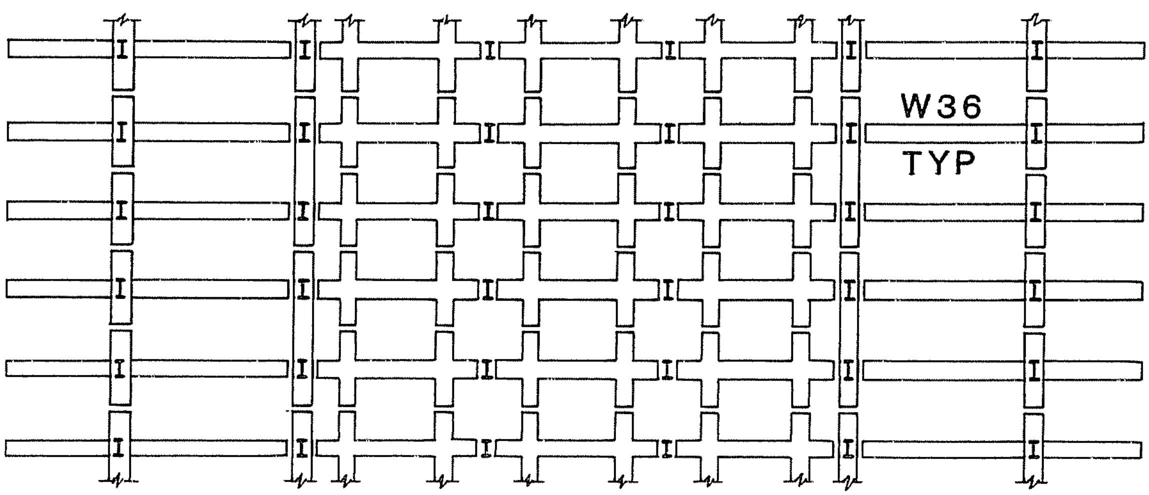


ROOF PLAN

FIGURE 1



FLOOR FRAMING PLAN



SECTION A - A

FIGURE 2