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DUCTILE LARGE-SPAN FRAMING SYSTEM FOR TALL AND SLENDER STEEL BUILDINGS IN SEISMIC REGIONS

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Introduction

Building Codes establish the level of lateral loads originated by either earthquake or wind, to be applied in the structures. Likewise they limit the amount of lateral deformation and story drift to values which do not cause non-structural damage.

Ductility, or inelastic deformation of the structural system is commonly recognized as the most important characteristics of the lateral load resisting system, to absorb the peak loading produced by extreme earthquakes, on highly seismic zones; thus, it is always desirable that in such circumstances, structures may count with such a reserve of strength.

This mere fact has been originating the increasing use of structural steel lately in seismic zones, as compared to the more traditionally common material, the reinforced concrete, whose ductile design has been penalized by the recent Code's revisions and upgrading, at the light of many unsatisfactory behaviours in recent major earthquakes. [Ref. 1].

In high rise construction, bare-steel frames alone, show up high flexibility to lateral deformations, in particular, when moment-resisting-frames (MRF) are the only source of lateral resistance. A combination of MRF with concrete shear walls (CSW) or with steel eccentric bracing (SEB), renders the desirable stiffness to lateral deflections without sacrificing ductility. However diagonal braces are sometimes architecturally unaccepted.

Another possibility of increasing the frame stiffness to lateral deformations is the use of composite steel-concrete columns. This combined system has been found to be highly economical to construct at the time that its facilitates a high-speed of construction [Ref. 2], since the bare-steel skeleton may be designed with sufficient strength as to allow the contractor optimum construction sequence and spread in the various construction activities.

The present paper deals with a 28-story above grade, two-underground slender structure, designed for a high-seismicity zone, where lateral deflections and high overturning moments, presented a problem difficult to solve by traditional structural systems. The solution adopted is exposed hereafter.

Description of the building:

Figures 1 and 2 show a plan and front elevation of the building. Urban Code restrictions required the tower zone of the building to separate from the property lines A, F, 1 and 9, to column lines C, E, 2 and 8, respectively, as shown by the bold-dotted line, rendering a slenderness ratio, height-to-width of 91.92 m/18.80 m $\approx$ 5.

In a three-column two-bay frame (C, D and E), the intermediate column D carries more than three times gravity loads, than the outer columns C and E. Thus, when earthquake loads are present, the overturning moment is high enough, the vertical uplift on one of the exterior columns may not be counteracted by its reduced gravity load, rendering difficult to handle uplift resultant column loads.

By removing columns on line D on the tower levels, the entire gravity load goes to both columns C and E. In the case of the studied building, the increment in gravity load, was big enough as to counteract the overturning uplift. (Fig. 3).

However both gravity bending moments on the now large "single-span" floor beams (L = 18.80), and lateral-loading moments on the exterior columns are largely increased by the suppression of the center column C, complicating the structural design of members and its connections. Figures 4 and 5 illustrate schematically all of the above.

It was therefore required to add some shear-links on the beams at the thirds of the span, at every other story, in order to reduce the magnitude of the bending moments produced both by gravity loading and lateral loading, as well as to stiffen vertically and laterally the structural system. The interior shear links carry no vertical load, but pure shear, as indicated on Figure 6.

Figure 6a shows schematically the way in which the shear-links AB_1, CB_2, DE_1, and FE_2 may be placed on the long-span beam at consecutive levels. It is evident that if they are not interconnected, when the beam deflects by gravity loads, its curvature produces a separation of the shear-links ends B_1 and B_2 or E_1 and E_2 in the amount of $\mathbf{\Phi_1 h}$ (fig. 6b). In order to restrain this separation movement, the shear-link's ends have to be connected by a pure shear bolted connection at the webs, with slotted vertical holes to avoid the transfer of vertical axial loads, as shown in detail No. 3, on Figure 19.

The shear force F1 necessary to apply to each of the shear-link's ends to bring them together will be the design force for the shear connection. Such a force imposes on the long-span beam, concentrated moments of magnitude $\mathbf{F1 \cdot h/2}$, which tend to "straighten" the beam and to reduce its vertical deflection (Fig. 7). The original bending moment diagrams of figure 4b are therefore reduced in magnitude as indicated in Figure 7a.

This convenient implementations work also fine for lateral loading, as indicated on Figure 8. It is clear that both the bending moment diagrams and the shear diagrams are greatly reduced by the connection of the shear-links, as shown in Figure 8a in comparison with Figure 5b.

Once this arrangement was discussed with the Architect of the Project, an alternate possible arrangement was contemplated, consisting in using both shear links in one long-span beam placed to the same side of the beam (say, its upward side), to be interconnected with the shear links placed on the opposite side of the next beam above it (say, its downward side) as indicated in Figure 9. This arrangement converts two consecutive long-span floor beams into a one-story height Vierendeel truss, where the chords are the beams, and the vertical members are both the shear links and the outside columns. Thus, we can integrate each tower frame by a series of a story-high Vierendeels spanning the total width of the tower and locating them at every other floor. However, in order to balance the building stiffness, we considered convenient to alternate the levels of the Vierendeels, from one frame to the next, in a staggered manner.

The Project Architect approved this scheme, which rendered column-free areas of 18.80 m by 18.40 x 19.30 m. at every floor, which is suppose to greatly increase the rentability of this building.

Another interesting feature of this building is that the Architect wanted to achieve a dramatic cut of two floors, at the front part of the buildings, triangular in plan shape (dotted line zone on Figure 10), as indicated in Figure 2. This challenging problem was solved out by suspending 15 floors and supporting 3 more, from some steel belt trusses placed on the 25th floor, along the C, E and 8 column lines, as shown on Figure 10.

The top trusses had a dual effect in the building framing: on one side they allowed us to create the architectural effect the "notch", by suspending the floors from the cantilevered portions of the trusses along C and 8 lines; on the other side, the in-plane stiffness of those top trusses served to "straighten" the curvature of the laterally deflected building shape, reducing the absolute and relative lateral deflections of the building in both directions, as well as its period, as shown in Figures 12 and 13. Figure 11 corresponds to the wire-undeflected tridimensional model of the building as analyzed by the ETABS program [Ref. 3].

This building was designed in accordance with the Mexico City Building Code [Ref. 4], for a seismic coefficient of 0.40 g., on the soft ground (lake/bed) zone.

We may comment that the results of the dynamic analysis showed that the first mode of the building, whose natural period $T_1 = 3.02$ sec., occurred on the longitudinal direction of the building, contrary to what it could be otherwise expected. This was caused by the stiffening effects of the alternated-Vierendeel trusses on the short direction of the tower. The second and third modes occurred in the short direction ($T_2 = 2.69$ sec.) and in the torsional direction ($T_3 = 2.07$ sec), respectively. The deflected shapes of the first two modes are depicted in figures 12, 13, 14a and 14b. Figure 14c shows the third mode deflected shape (torsional) in plan.

Figure 15 presents the bending moment diagram on one transversal frame for vertical loading, while Figure 16 shows up the shear force diagram for lateral loading.

In order to show these diagrams more clearly, Figures 17 and 18 are "windows" of Figs. 15 and 16 showing only a portion of the frame from levels OF-6 and OF-10.

It's simple to observe that in the case of gravity load (fig. 17a and 17b) the magnitude of the bending moments on both the beams and the shear links are very uniform through the entire height of the building, which originates a large-degree of normalization on the structural sections. Likewise, in the case of the lateral loading (Figs. 18a and 18b), at the Vierendeel locations the shear-links, take all the horizontal shear at its respective levels, liberating the exterior columns from shear. However, at alternate levels, where the Vierendeel trusses does not exists, all the shear is being carried by the outside columns. This was the primary cause which originated the staggering of the Vierendeel Trusses from frame-to-frame, at every alternate level, in order to avoid sudden changes in stiffness and strength from one floor to the next.

In the longitudinal direction the lateral loads are resisted by frame action through the moment resistant frame integrated by the exterior columns and the longitudinal spandrels on lines C and E. It was found convenient to simplify the construction procedure by using steel columns with its minor resisting direction to rigidly connect the spandrels, and the flanges (strong direction) to connect the Vierendeel trusses. The steel column was designed to support any contingency load during the erection work, as they were going to be embedded in reinforced concrete at a later stage, to work in composite action for the design loads. Thus, the reinforced concrete provides the most part of the column strength and stiffness on the longitudinal direction. The finished column dimensions resulted 100 by 100 cm.

The average steel weight for parking levels was 90.7 kg/m^2 and 82.8 kg/m^2 in the office tower levels. The total weight of the building, including columns, floor beams, Vierendeels, top trusses, tension members and connections was 2088 ton, rendering a gross average weight of 87.4 kg/m^2 on the entire building area. Such a steel weight resulted above the average weight obtained by the author in references 6 and 7 with the regular Staggered Truss System, as a result of the comparatively larger weight of a Vierendeel truss versus a regular truss with diagonals (Pratt, Warren, etc.). Another important reason for the higher average weight in regard the two referenced jobs, was the fact that the first two buildings were designed in accordance with the 1986 Mexico City Building Code, whereas the building presented in this paper was designed in accordance with the 1987 Mexico City Building Code, which imposes much larger seismic forces as well as serious design restrictions.

The steel used was of ASTM A-36 quality in its great majority, with some sections in ASTM 572 grade 50 steel. All sections were rolled sections in accordance to ASTM A-6.

Construction Procedure

Given the sizes of the structural elements for the tower portion of the building, and importantly, the way in which they will be working both during the construction phase and in the final completed phase, it was found necessary to implement the following fabrication and erection procedure:

The main floor beams (Vierendeel's chords) will be fabricated in half its length (9.40 m.) with an splice at the center of the total span. They will include the corresponding portions of the shear-links shop welded to the beam (chord) to be field spliced with the next fabricated members so as to integrate a full Vierendeel. Thus, the chords will be field spliced in one single element and erected in one hook operation; the chords will be field welded to the exterior columns.

The secondary floor beams will then be erected spanning from a certain Vierendeel's top chord to the next Vierendeel's bottom chord. Spandrel beams will be field welded to the columns.

Temporary bracing will be required during erection until the concreting of the columns provides sufficient lateral stability. This may enable the steel erector to be working on about seven levels above the concreting of the columns, if required.

Finally the steel metal deck is laid down on the floor beams; the stud shear connectors are welded to the top flange of the floor members through the metal deck; the wire mesh and reinforcing steel is placed and then the concrete for the slab is poured.

The sequence of erection will be repeated floor to floor as indicated in figure 19.

Figure 20 shows up the connection between spandrels and Vierendeel's chords to the columns. Once the connection has been totally finished and the bolted and welded connections had been thoroughly inspected, the reinforcement of the concrete column is placed and the stirrups welded to the appropriate beam's webs to receive the concrete.

This type of join assures full ductile behaviour of the building's frame and guarantees the maximum value of the ductility factor given by the Mexico City Building Code, (Q = 4), affected by a certain irregularity coefficient of 0.8 recommended by the Code for the tallness, slenderness and particular shape of the building, rendering and effective Q value of 4 x 0.8 = 3.2.

The connection was designed in accordance with the criteria stated in the Mexico City Building Code and revised to conform the design recommendations of references 8 and 9.

Conclusions

The structural system used in this building permits the combined use of reinforced concrete and steel, in a convenient and practical way.

The so-called "staggered truss framing system" [Ref. 5] has been successfully used in Mexico [Ref. 6], demonstrating excellent seismic behaviour. The present system, which may also be called "Staggered Vierendeel Truss System" is a result of a good coordination between the Architect, the Structural Engineer and the Developer.

It is the authors opinion that this case may be cited as an example of a good interdisciplinary professional relationship, since the planning stage of a project.

This novel structural system shows up an adequate solution to solve the problem of tall-slender-structures, under in high seismic zones. Lateral deflections of the building are minimized with this particular solution, and adequate levels of economy can be achieved.

The fabrication and construction of the building is actually proceeding at a fast rate. The benefits of incorporating large column free areas at every single office level, are advantageously considered by the Architects and Developers.

Credits

The building discussed in this paper is currently being erected at the corner of Avenues Insurgentes and Colonia del Valle, in Mexico City.

The Architect/Developer, and Construction Manager of the building is **Arch. José Picciotto Cherem.** The structural steel is manufactured and erected by **COREY, S.A. de C.V.**; the geothecnical investigation was carried out by **TGC** Mexico City.

The design of the buildings was directed by the author in his Consulting Engineering Firm **Enrique Martinez Romero, S.A.** with the valuable assistance of Msrs. Javier Horvilleur and Larry Griffis from **Walter P. Moore and Associates** from Houston, Texas.

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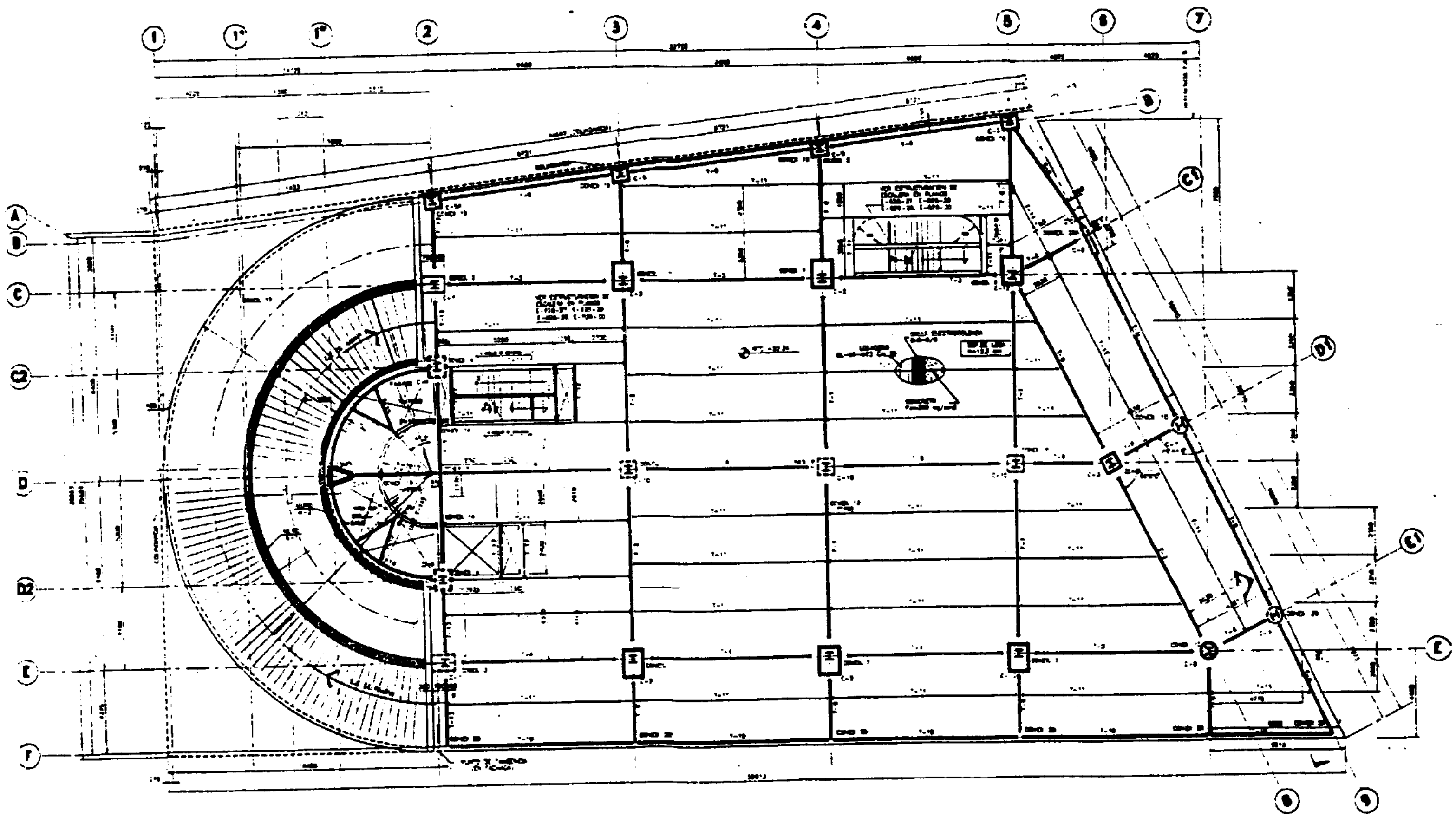
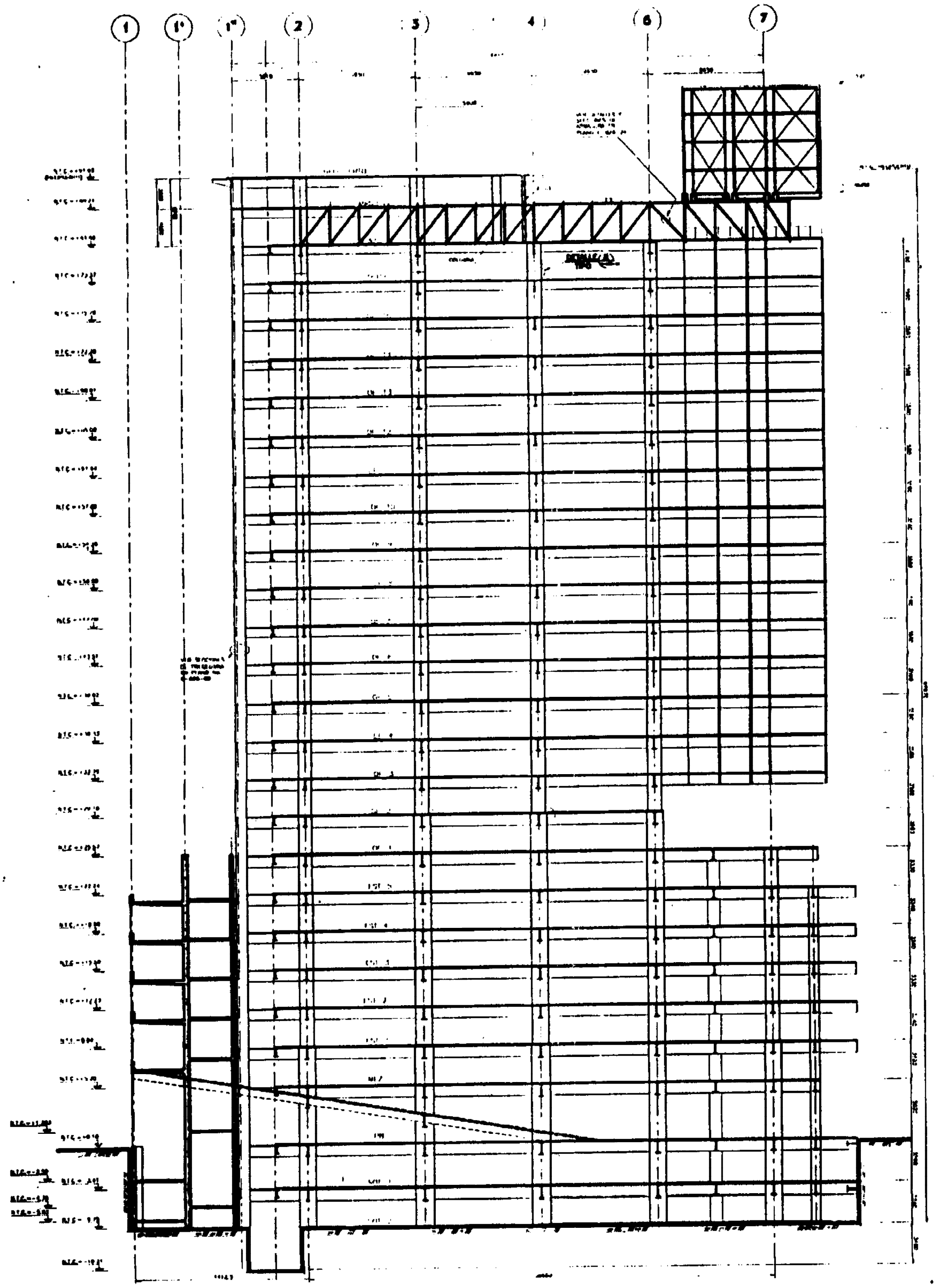


Figure No. 1



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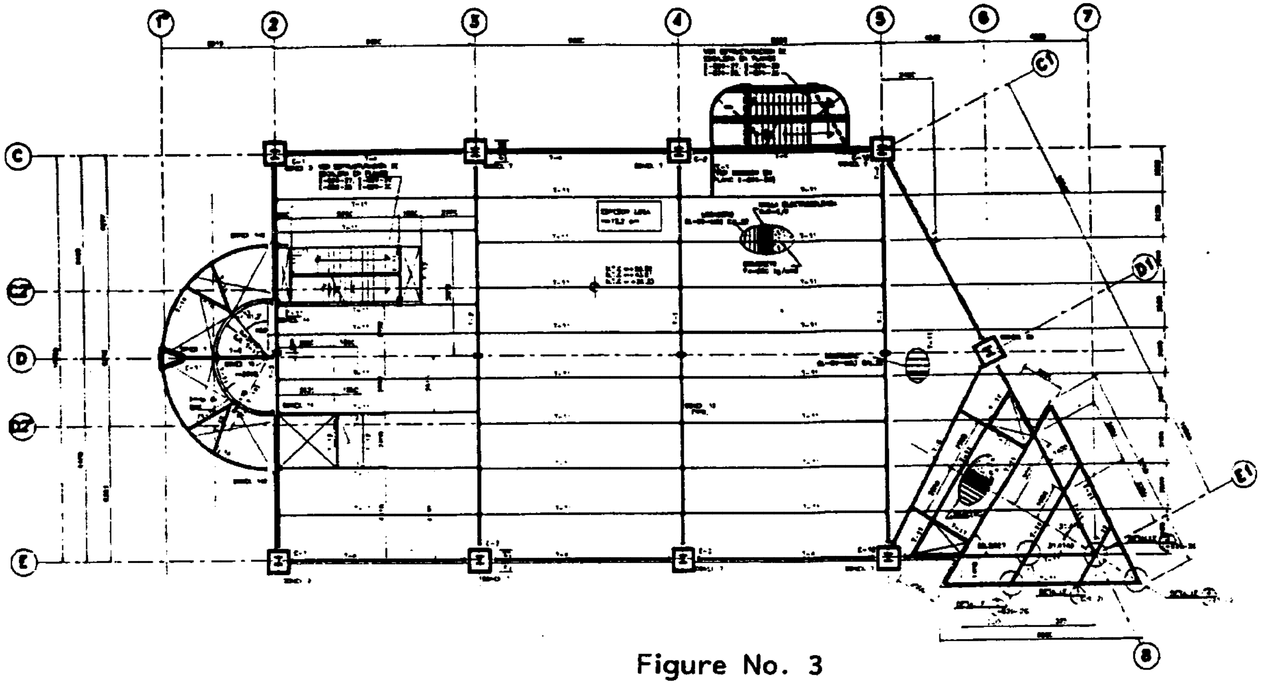

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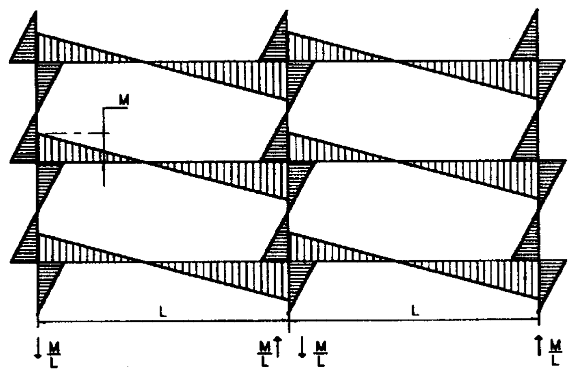


FIGURE 5a

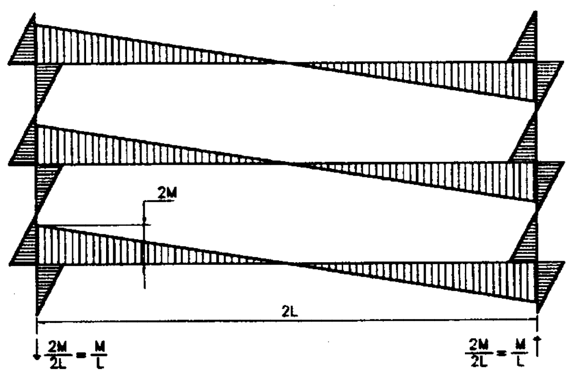


FIGURE 5b

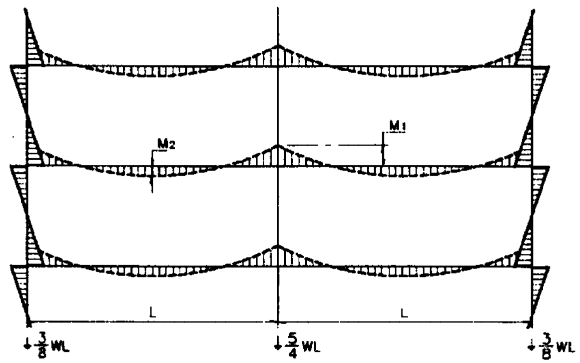


FIGURE 4a

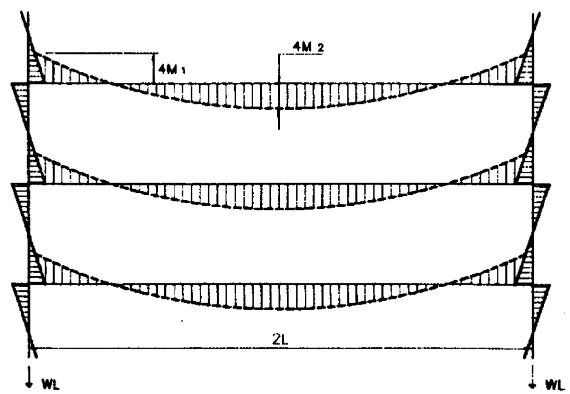
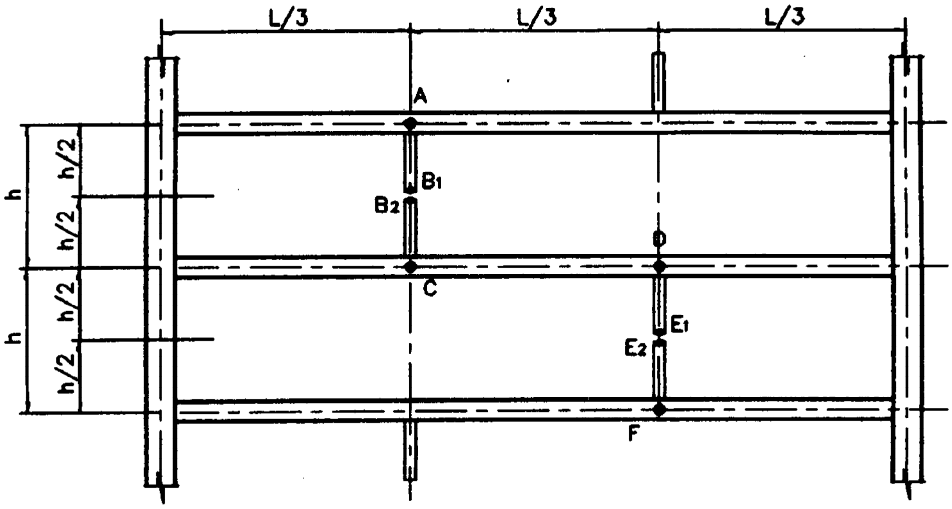
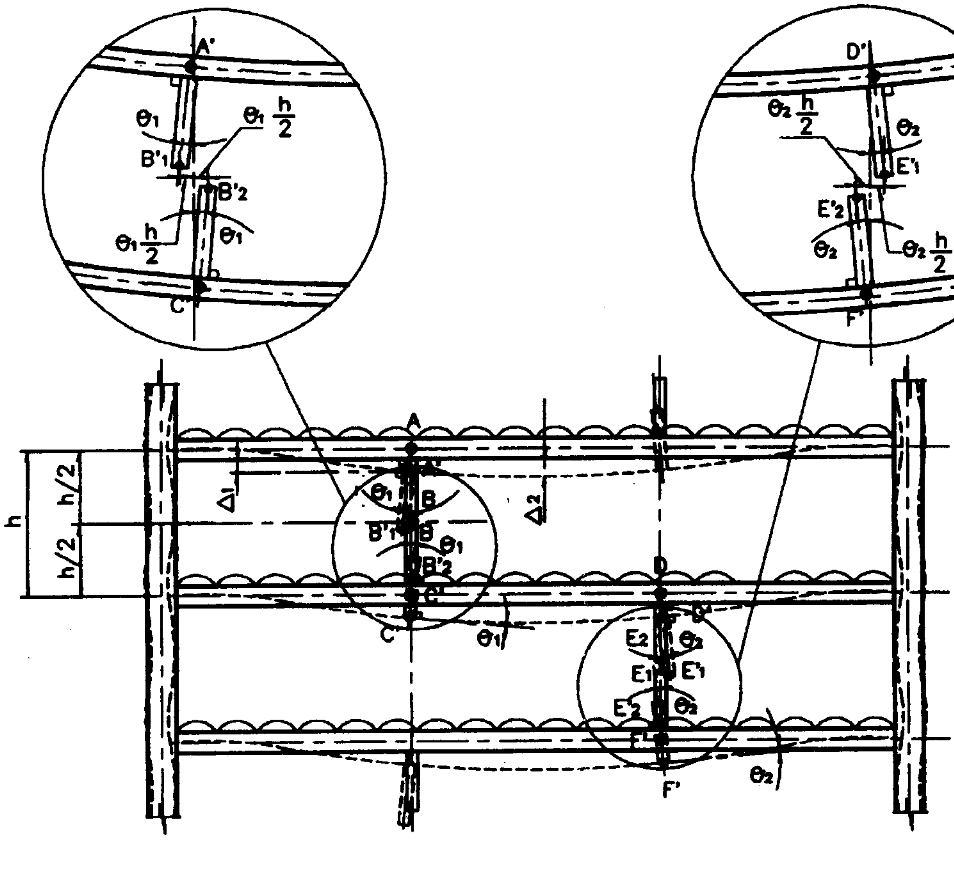


FIGURE 4b



F I G U R E 6a



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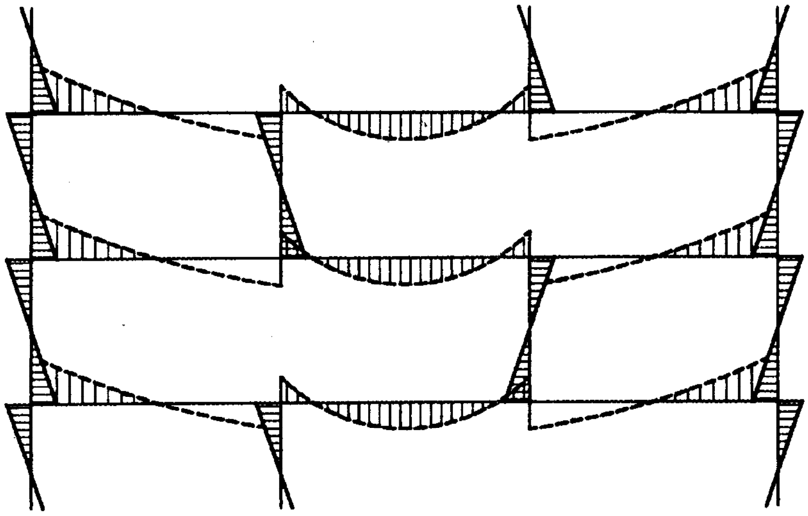

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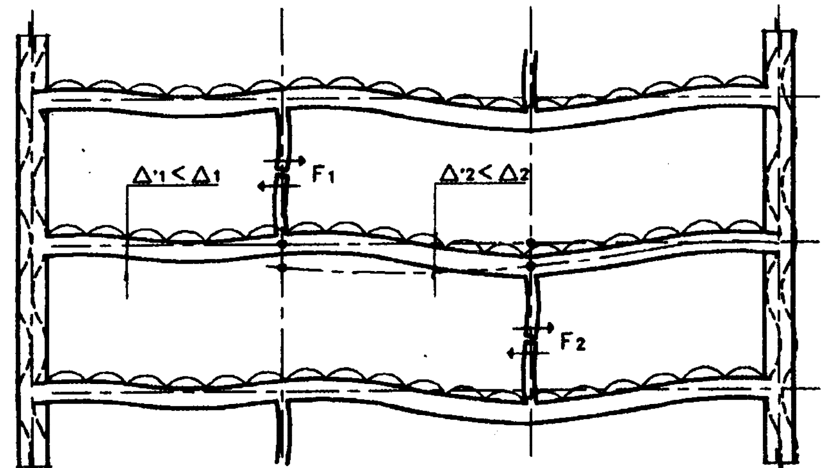

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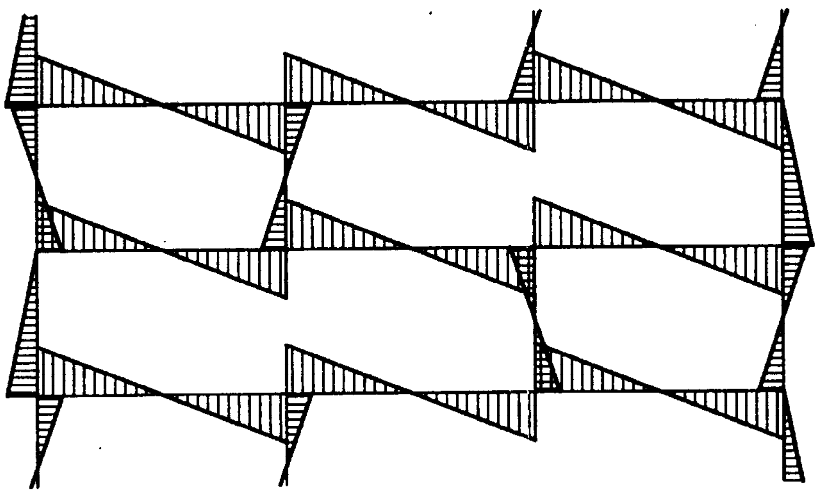

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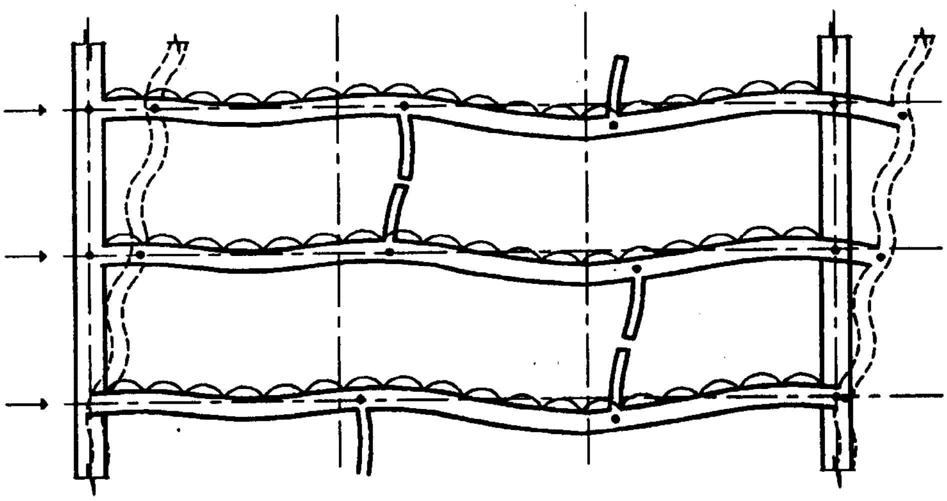

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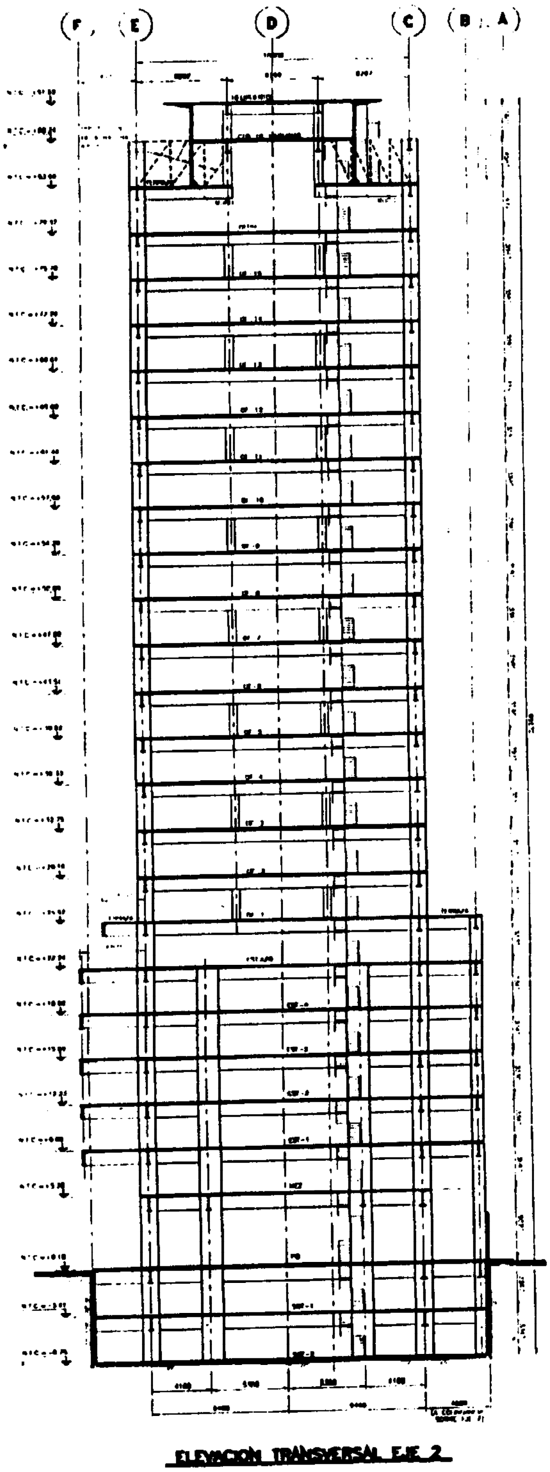
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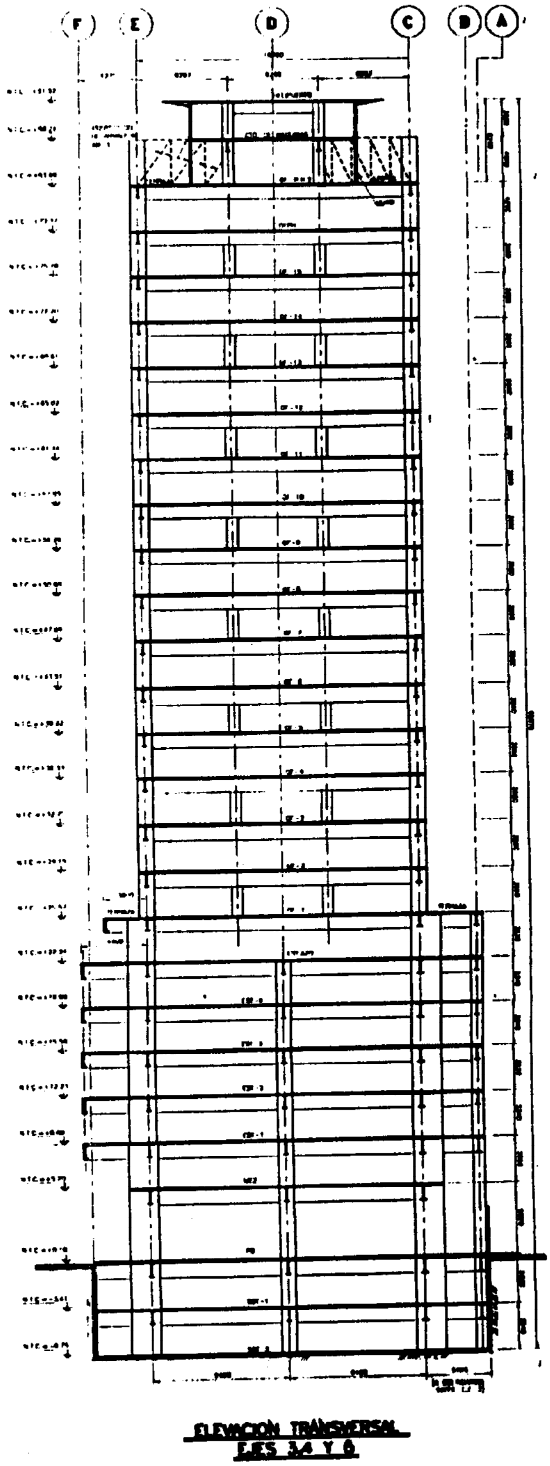
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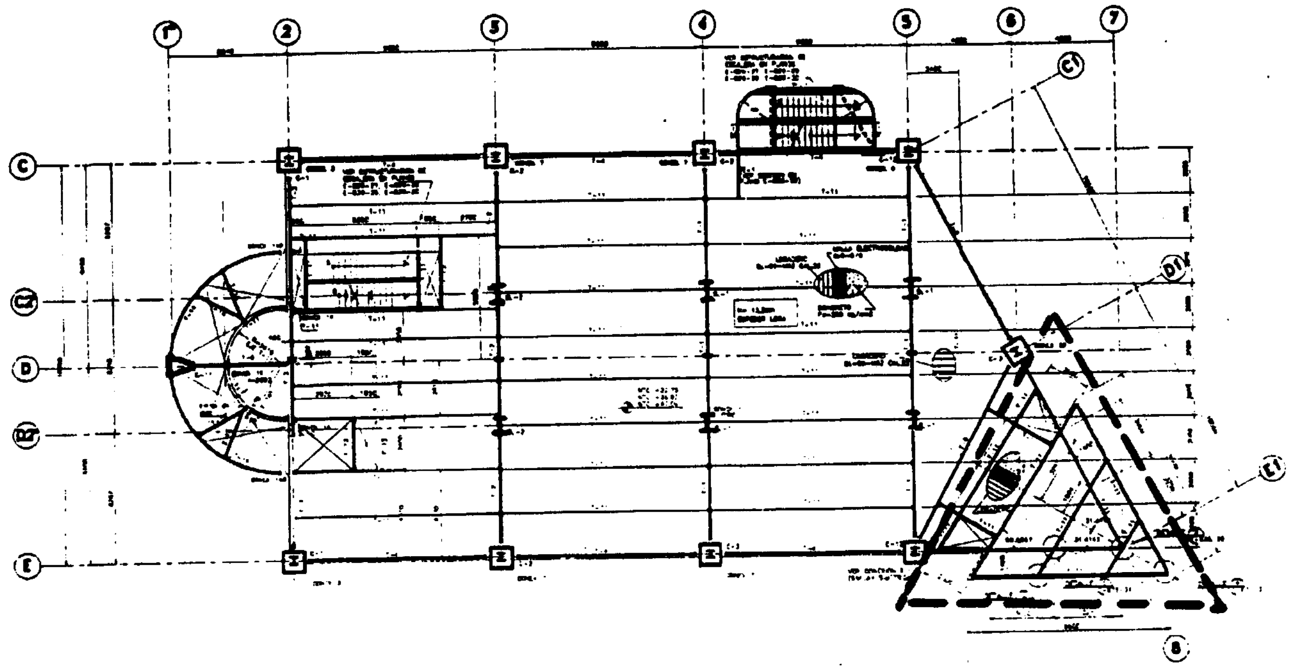
Figure No. 9



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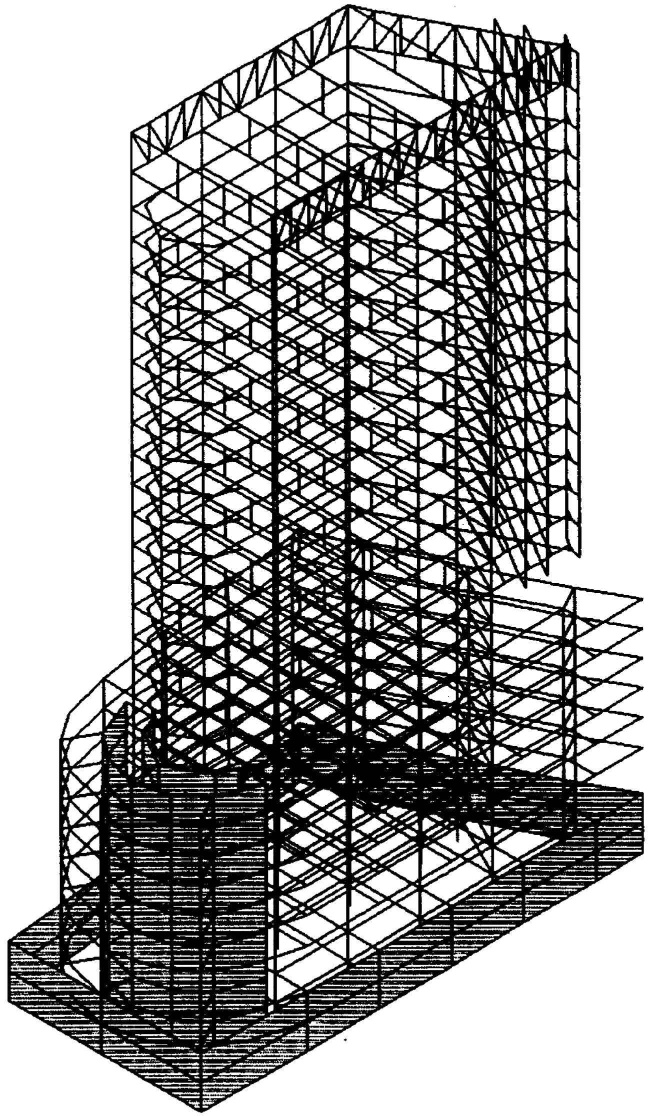
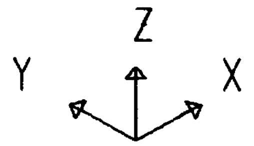




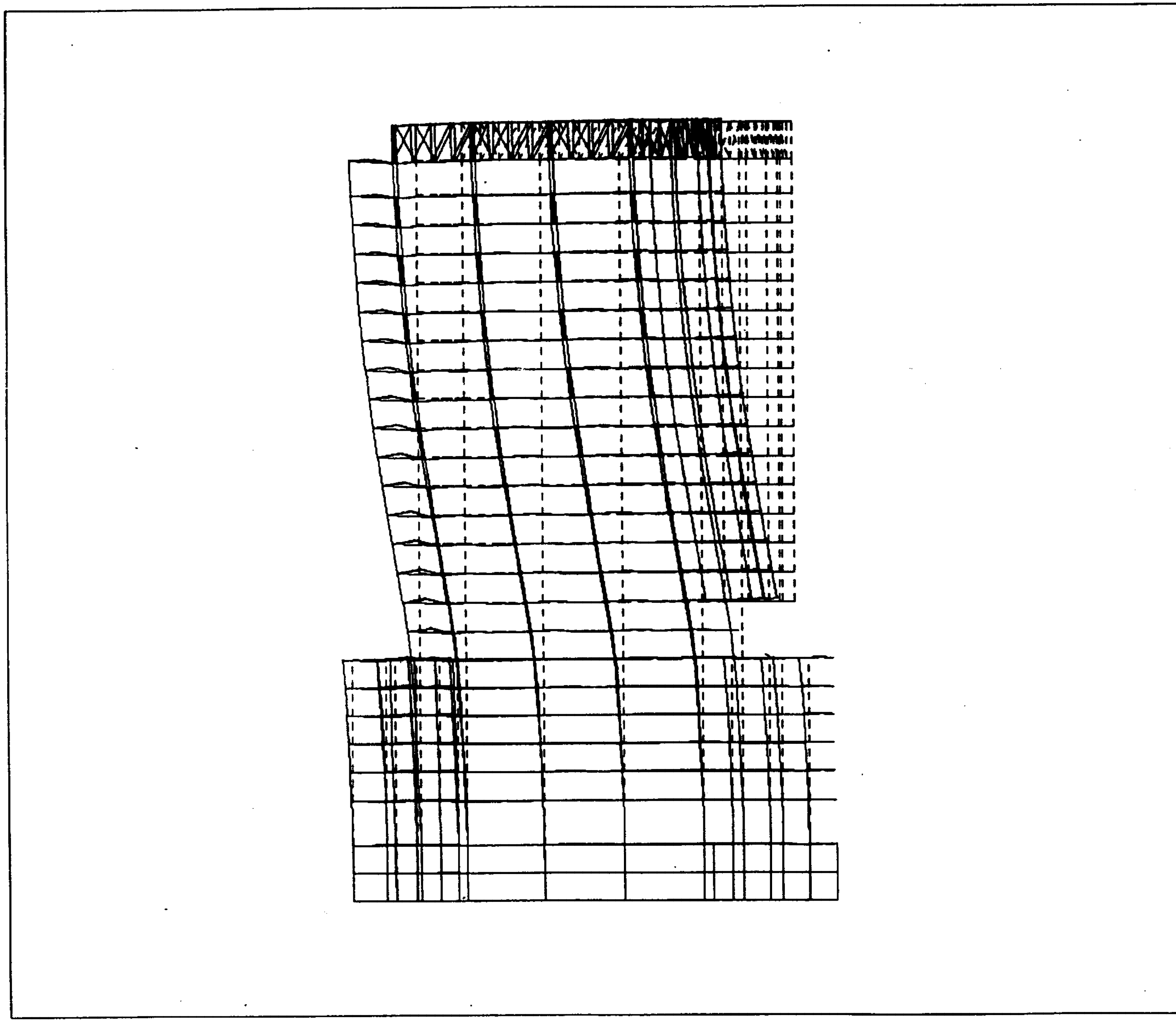
INSURC7P.PST

UNDEFORMED
SHAPE

TOP ESTR
BOT SOTI

OPTIONS
HIDDEN LINES

ETABS



INSURC7P.PST

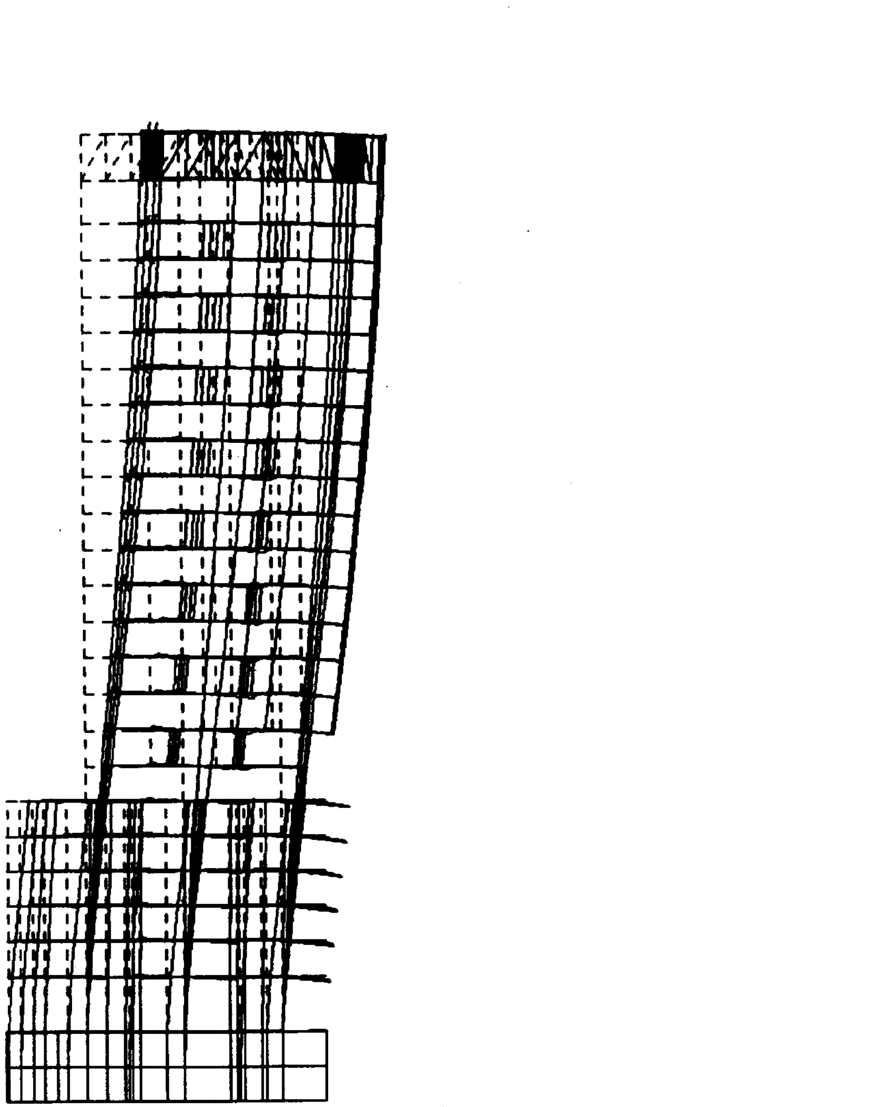
MODE
SHAPE

MODE 1

TOP ESTR
BOT SOT1

MINIMA
X-0.1912E+00
Y-0.2438E-01
Z-0.7137E-02
MAXIMA
X-0.2964E-04
Y 0.1189E-01
Z 0.1129E-01

ETABS



INSURC7P.PST

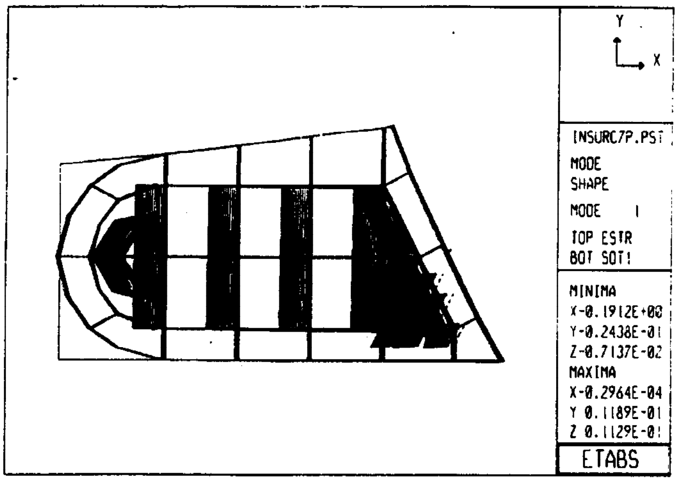
MODE
SHAPE

MODE 2

TOP ESTR
BOT SOTI

MINIMA
X-0.1482E-01
Y-0.2235E+00
Z-0.1831E-01
MAXIMA
X 0.2905E-01
Y 0.3926E-04
Z 0.9718E-02

ETABS



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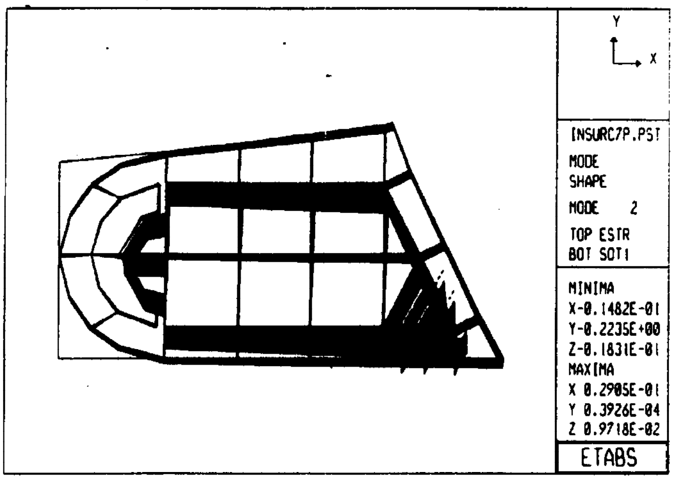

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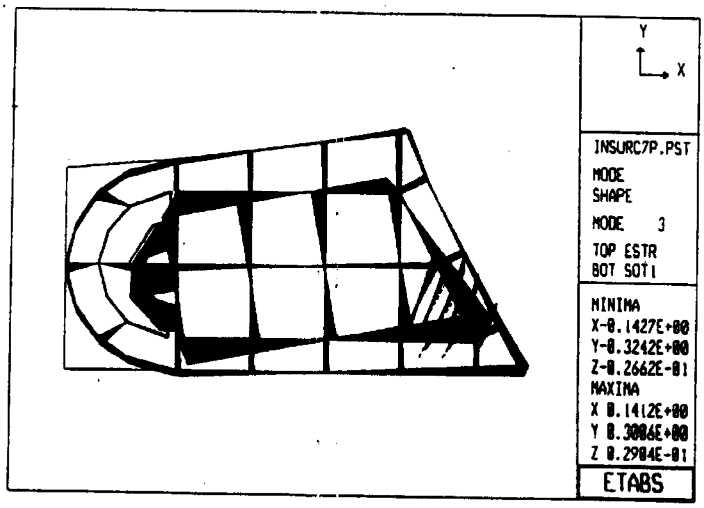

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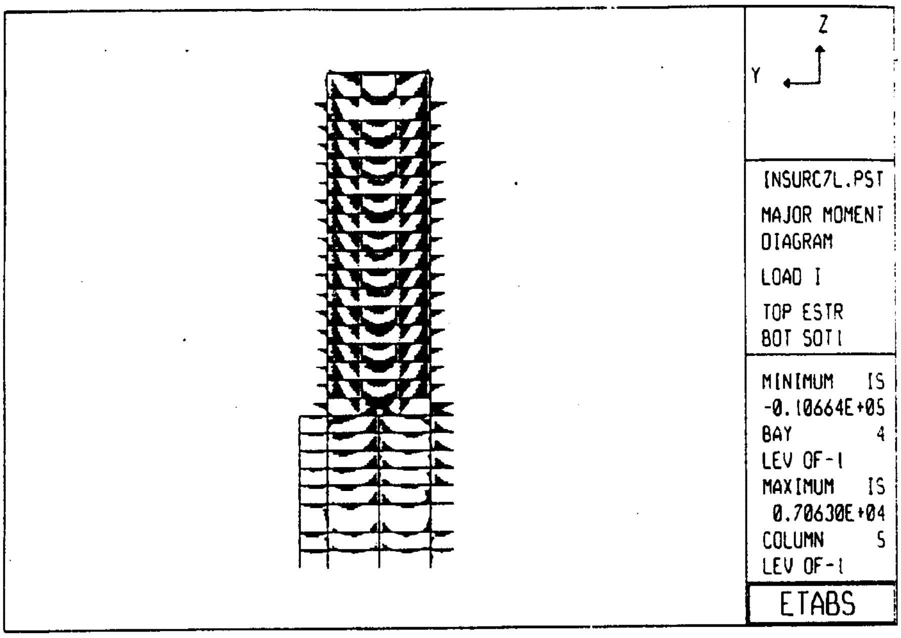

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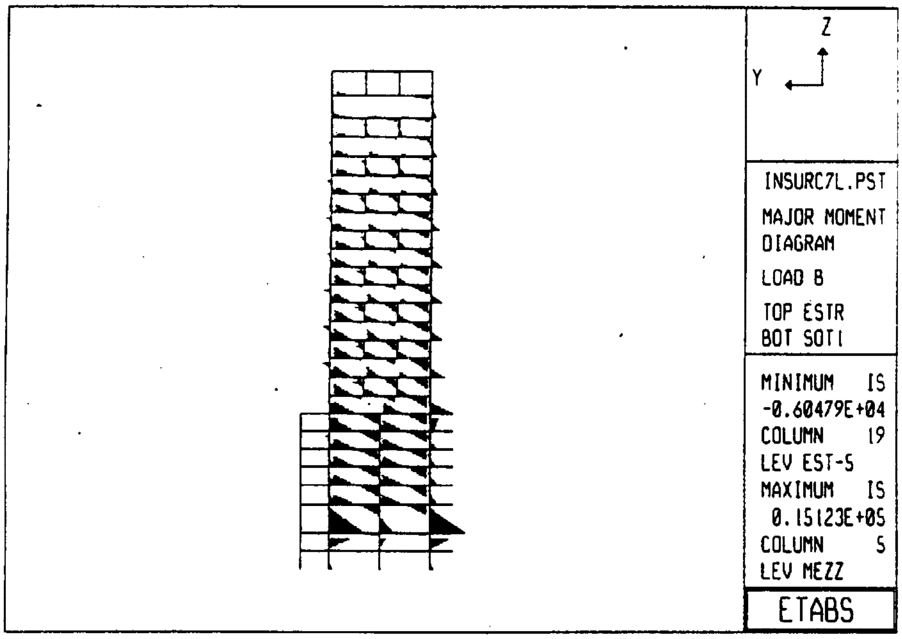

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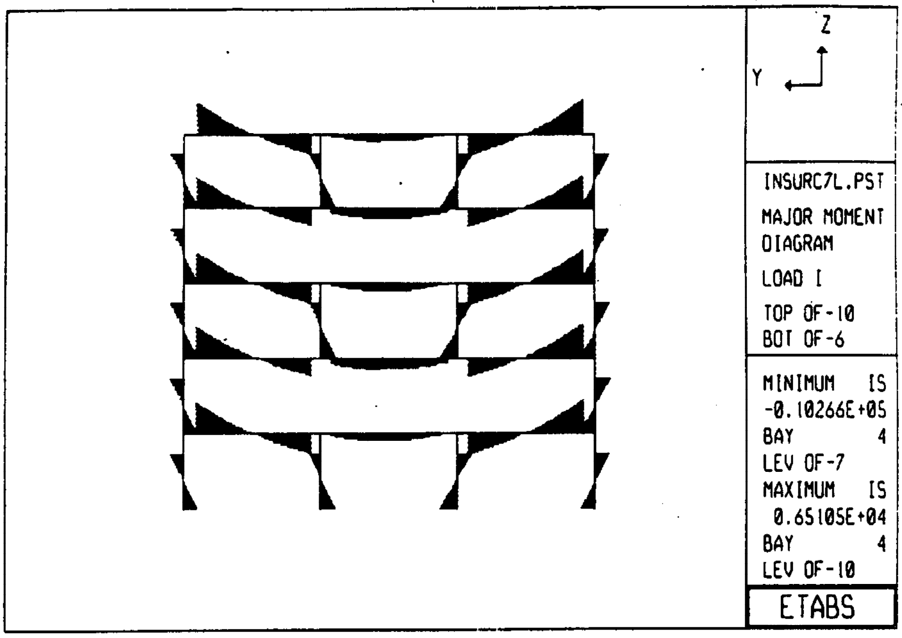

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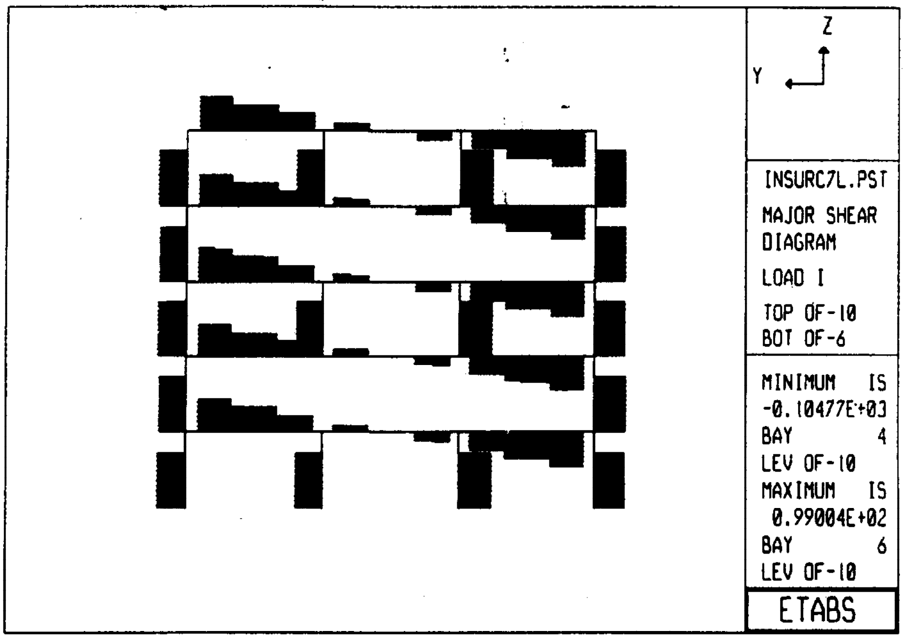

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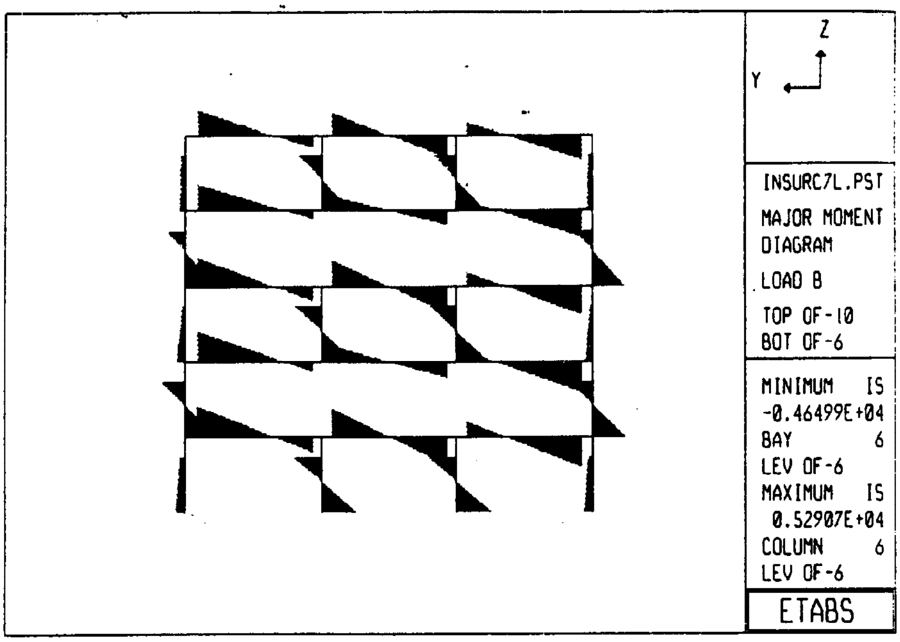

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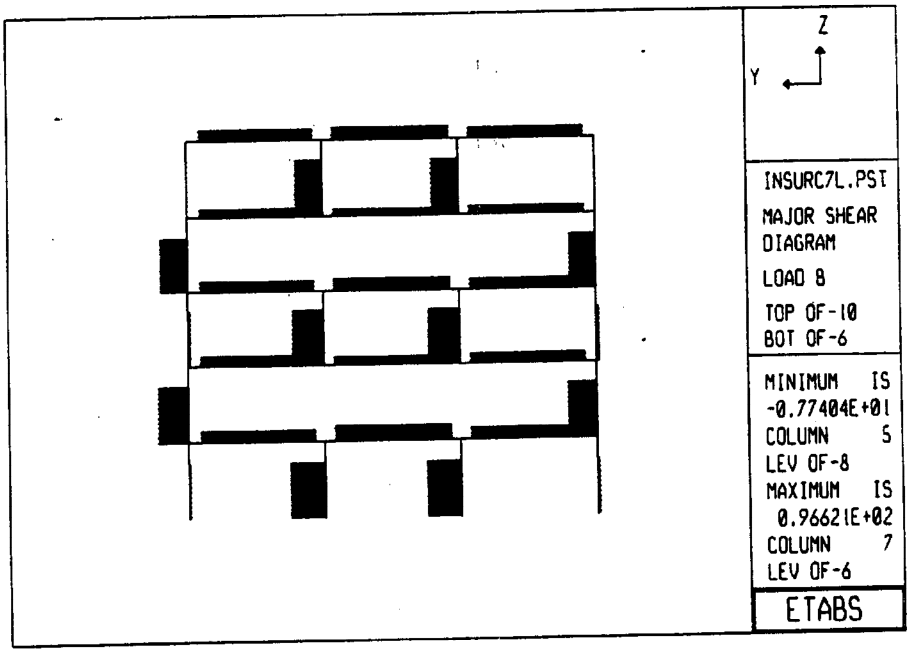

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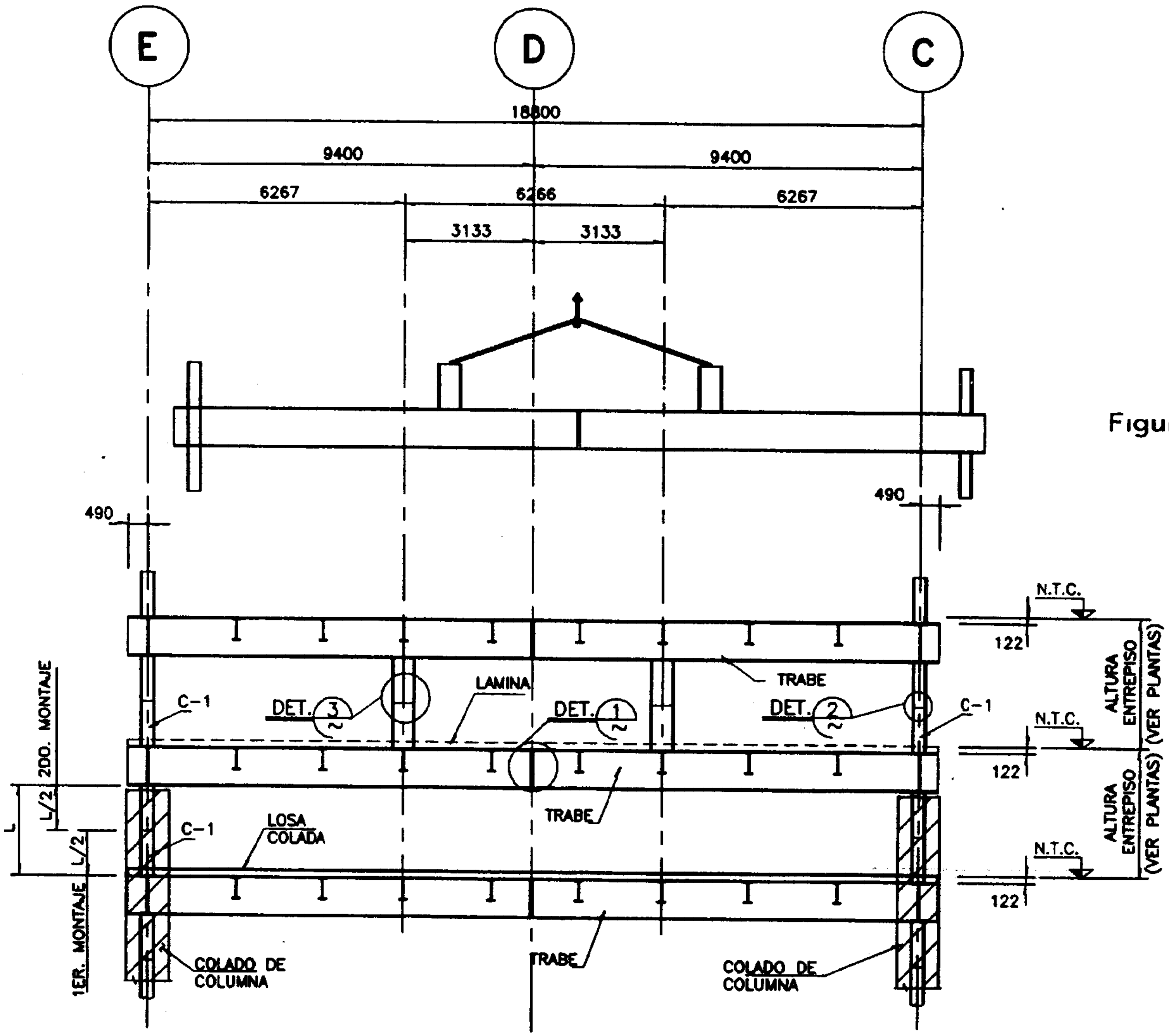
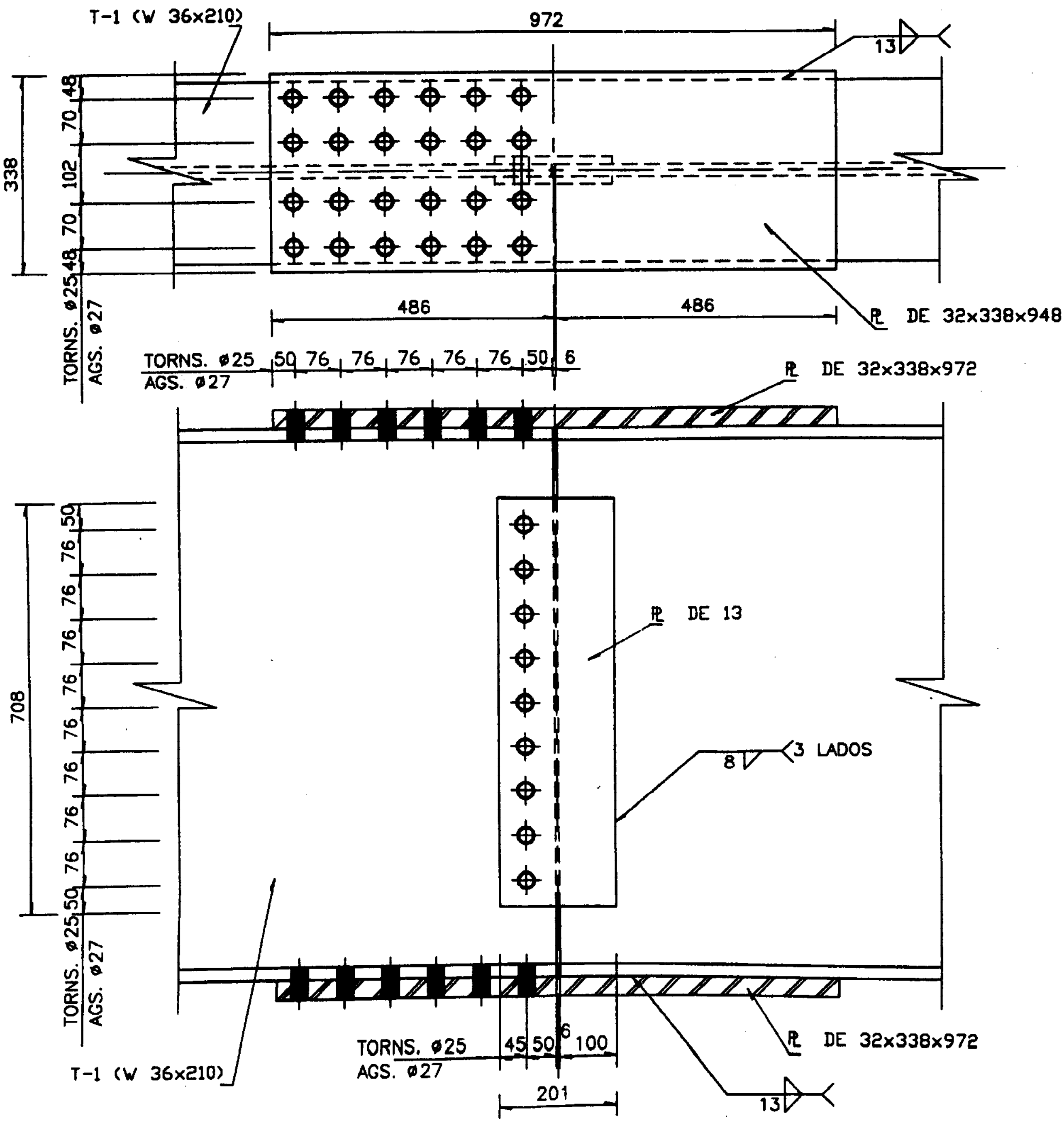


Figure No. 19

ESTRUCTURACION TIPO

(ELEVACION)



DETALLE 1
(TRABE T-1)

Figure No. 19

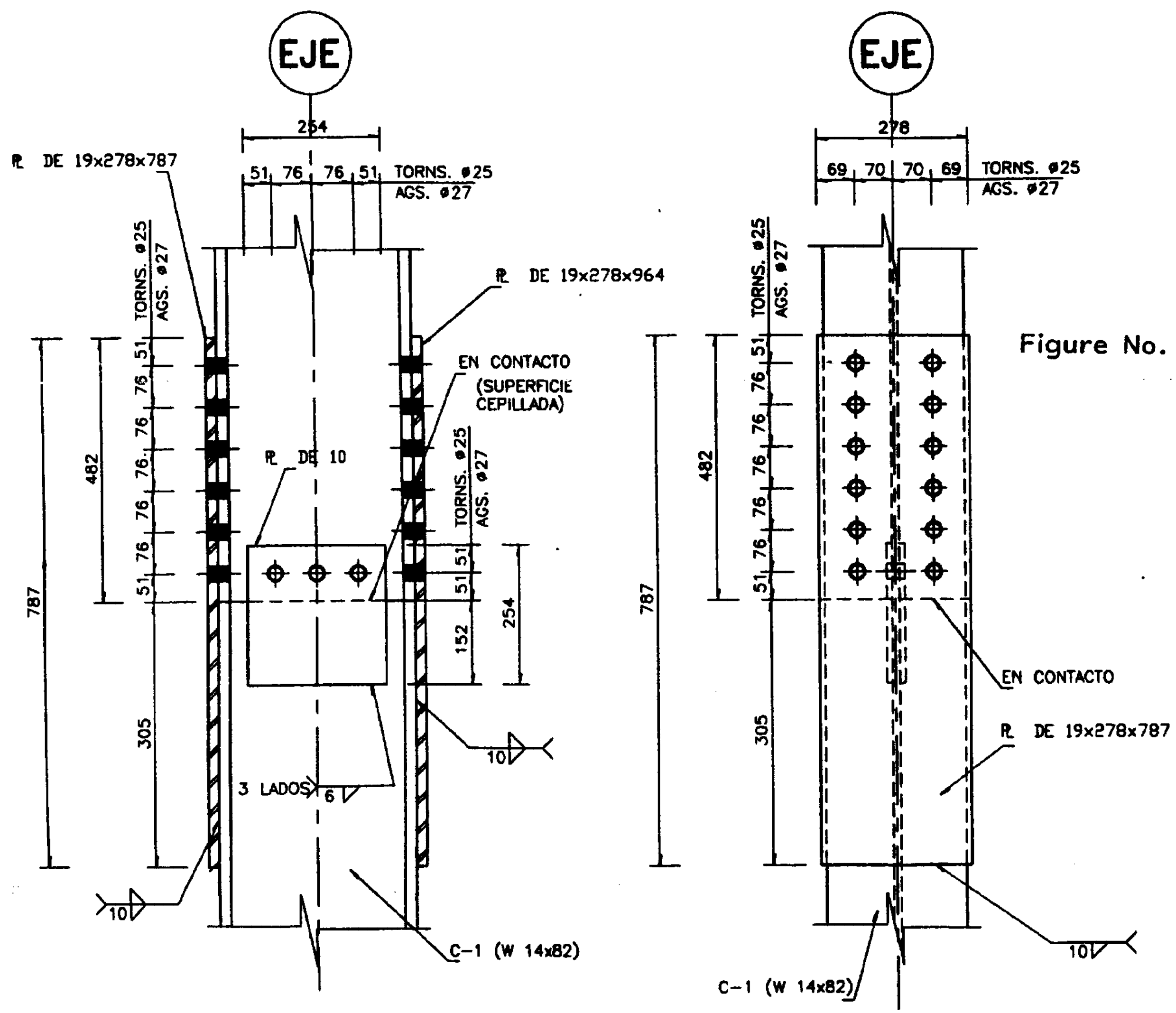


Figure No. 19

DETALLE 2

(COLUMNA C-1)

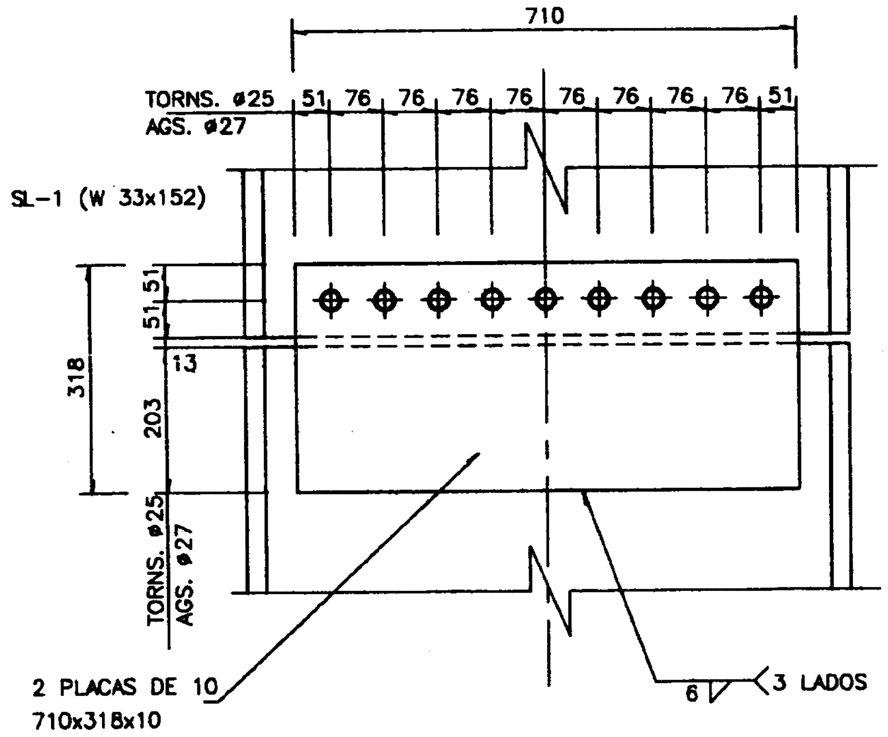
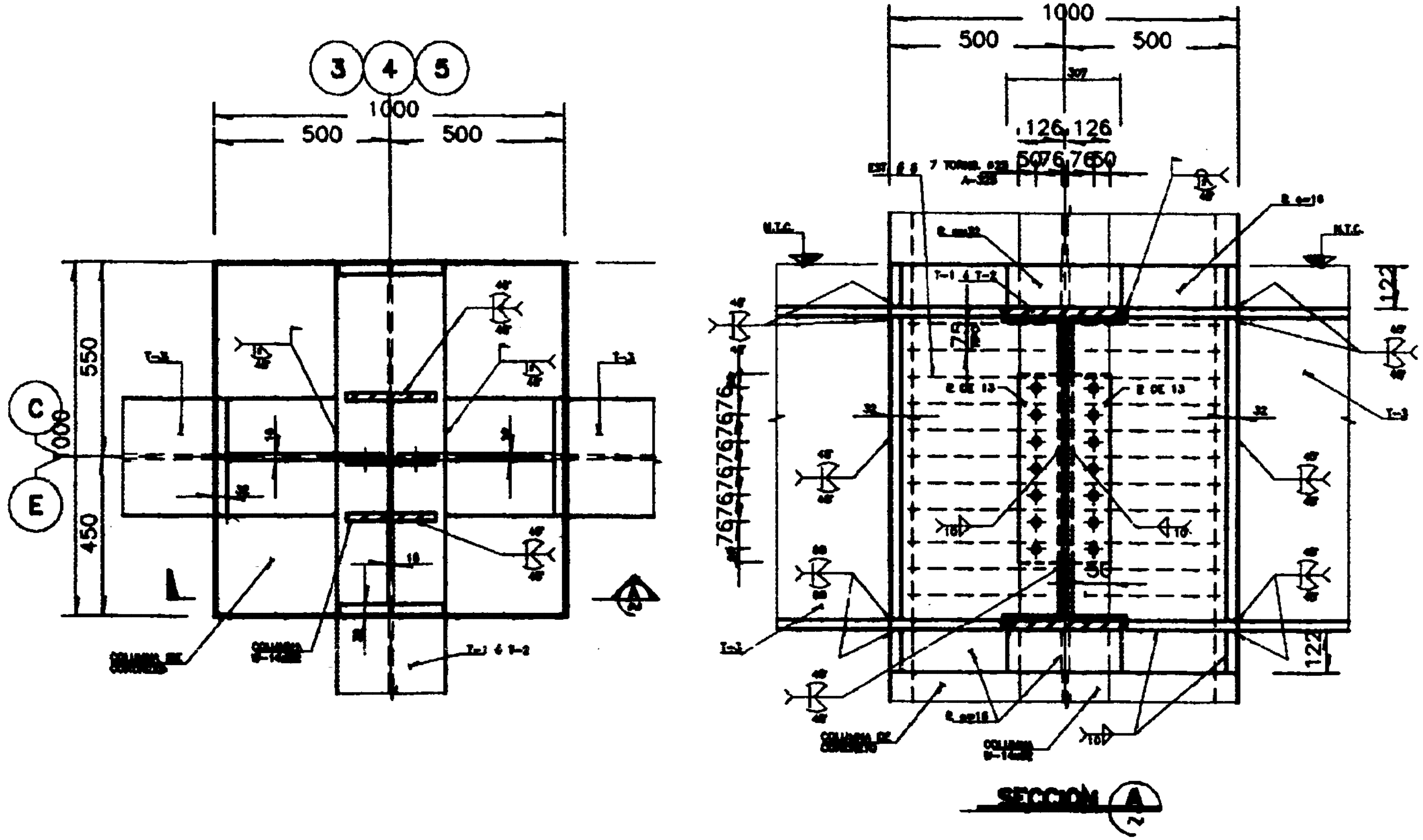


Figure No. 19

DETALLE 3
(SHEAR LINK SL-1)



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