Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 9 (1972)

Artikel: Westcoast Building cable suspended structures Vancouver, B.C.

Autor: Babicki, B.B.

DOI: https://doi.org/10.5169/seals-9678

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DISCUSSION PRÉPARÉE • VORBEREITETE DISKUSSION • PREPARED DISCUSSION

Westcoast Building Cable Suspended Structure Vancouver, B.C.

Westcoast Building — Structure à câbles suspendus à Vancouver, B.C.

Westcoast Building — Seilverspannter Bau in Vancouver, B.C.

B.B. BABICKI
M. Sc., P. Eng., Principal of Bogue Babicki & Associates
Consulting Engineers
Vancouver, B.C. Canada

The Westcoast Office Building is located in Vancouver Canada in a very picturesque setting and on one of the major arteries of the city connecting the down town business core to the residential areas. This building has a total of 152,000 square feet of office area and covered parking accommodations for 200 cars. (fig. #1) It was designed for Westcoast Transmission Co. by Bogue Babicki & Associates, Consulting Engineers of Vancouver and construction completed in 1970.

Governing factors in the concept of the building were, the least interferance with the natural setting and earthquake resistance since Vancouver is located on one of the severest earthquake zones extending from California to Alaska.

The building in its final form has a 277 foot high concrete centre core 36 x 36 feet in plan area and accommodating 21 levels from foundation to top as follows: (fig. #2)

Three underground parking levels, equivalent of three levels of open plaza space, twelve levels of typical office floors, 110 x 110 feet in plan area suspended from the centre core above the plaza space plus, three levels within the core above the roof for mechanical and elevator equipment.

This solution has the following advantages:
The open plaza space under the building eliminates obstruction of a fine view of sea & mountains from the street level and isolates the first office floor from the direct effect of the street noise. Elimination of columns improves the efficiency of the underground parking. But, from a purely structural point of view as our main

concern, this system indicates a more favourable behaviour under seismic forces.

The reasoning behind this statement can be outlined as follows:

In most of the conventional high-rise buildings the vertical cores are used as main elements to transfer lateral forces but their contribution in supporting the vertical loads are very limited since the columns are made to carry most of the vertical loads. This arrangement results in an unfavourable stability and stress distribution in the core. By using the centre core to support all the vertical loads, the stability of the core is improved in transferring the lateral forces and the prestressing effect achieved due to the vertical loading, results in a more favourable stress distribution to the extent of eliminating tensile stresses in the core. In the case of this particular building these advantages were achieved within the dimensions of the core established for a conventional building.

Furthermore, a building supported on a single centre core element during an earthquake will behave in a more predictable manner for which it can be more realistically designed than the dynamically complex conventional buildings.

Due to the nature of this building and the prestressing effect on the core mentioned earlier it becomes obvious that the core is less susceptible to the damaging effect of the earthquake forces than the core of a conventional building and since the core carries all of the seismic loads, then the whole building is less susceptible to damage.

Once the rationality of the approach of carrying the full load on a centre core is established, a system for hanging the floors from the centre core was devised by the use of bridge strand continuous cables supported directly on the rounded top of the centre core and splayed to the outside perimeter of the building at the roof level. (fig. #3)

The use of cables continuous over the core eliminated the problem of splicing and anchoring of these hangers to the centre core

Six sets of 2 3/4" ø continuous cables are supported on a centre core crown structure consisting of exterior core walls rounded at the top and two diagonal concrete arches. (fig. #4) The main roof beams which are provided with bearing saddles accommodate the change of direction of the continuous cables and transfer compression forces to a ring on the centre core. These beams are suspended by additional sets of cables which are draped over the core in a similar manner to the continuous cables.

The floor beams are simply supported at the core in pockets cast into the core walls, and are attached to the cables by means of friction clamps. (fig. #5) The floor system consists of a steel deck filled with concrete topping and attached to the beams with shear lugs to achieve composite action.

The centre core is treated as a free standing cantilever and supported on a 55 x 55 feet concrete raft foundation six feet thick. A 14 x 14 feet cut-out is provided in the centre of the foundation raft to distribute more evenly the stresses imposed on soil, thus minimizing the rocking of the building due to reversal of the lateral earthquake and wind loads.

This structural system with a definite delineation between the reinforced concrete and structural-steel parts, lends itself to a slip-form method of pouring the centre core. The concrete pour was a continuous twenty day and night operation, with an average speed of forms of 6-9 inches per hour.

The second stage of the construction was the erection of the roof girders and the draping of the floor-supporting cables. (fig. #6)

After completion of the draping of cables, the erection of the typical floors progressed at a rate of about one floor per day. (fig. #7) The 550 tons of G40.12 structural steel used averaged to a relatively light weight of only 7.5 lbs. of steel per square foot, and compares favourably with conventional construction.

Once the structural-steel erection was completed, the exterior curtain walls consisting of small aluminum mullions attached to independent light frames were connected to the floors only at the cable points. Solar bronze windowns give a mirror-like appearance to the exterior skin. (fig. #8)

Design Analysis:

We will concern ourselves with only two major aspects of the design analysis in the scope of this paper:

1. The design considerations of the concrete centre core.
2. Dynamic behaviour of the suspended system.

In designing the concrete core it was assumed to be a free standing cantilever fully fixed to the raft foundation. Horizontal roller type joints were provided on the parking decks where they met the core, to prevent the lateral support of the core by these decks.

In one direction the core walls contain no openings and for loads in that direction the core can be considered as a single cantilever thirty six feet deep with some flanges. In the other direction the multitude of openings tends to create two vertical elements instead of one. For example if the lintels from floors one to twelve have no stiffness, then the core would consist of two vertical elements in this region constrained to have the same horizontal deflection at each floor. These vertical elements would have the axial and bending of frame action. If on the other hand, the lintels had infinite stiffness one could assume a linear distribution of bending stress across the full thirty six foot depth. The real structure must be somewhere in between. The stress distribution is further complicated by the large opening at plaza level, numerous openings for ventilation ducts, and the solid upper part which adds stiffness to the system.

In order to determine this stress distribution, a finite element analysis of the core was run on a Univac 1108 computer by Hooley Engineering of Vancouver. Figure 9a shows how the core was modeled with 753 nodes, 656 rectangular finite elements in plane stress,and 389 pin ended members under axial load. At two degrees of freedom per node, this mesh generates a structure stiffness matrix with 1488 unknowns and a half band width of 22. Computer time comprised of a 2 1/2 min run. Data was generated within the computer with the openings represented by finite elements with zero modulus.

A more complex three dimensional analysis was not deemed necessary for this structure as the effect of walls at right angles could be easily included by the addition of vertical pin ended members. Five vertical lines, each consisting of an average of 80 of these two noded members, ran up the building to interconnect all vertical degrees of freedom on the line. Their areas duplicated the areas of the walls at right angles which they replaced except near the top where shear lag dictated a reduced area.

As mentioned previously, the stiffness of the lintel is all important in determining the stress distribution. For this reason a fine mesh of 54 finite elements was used to model a single lintel, as shown in Figure 9b. Horizontal pin ended bars represented the effect of the floor slab as top flange of the lintel. Such a mesh gave an excellent evaluation of stiffness for the lintel including shear deflection and the effect of the pocket for floor beams. The equivalent thickness of the two elements used on the full core to replace the lintel was chosen to duplicate the results of the fine mesh.

Stress at the element centre line was found from the stress matrix and eight nodal deflections. Summations of these shear and normal stresses over the thirty six foot width at several cross section together with the force in the bars, confirmed overall equilibrium to within one percent as a final check.

The very low slenderness ratio of the core shows no need to consider the moment induced by the action of vertical loads on horizontal deflections. A linear programme was then used and output confirmed this fact. As well, tilting of the foundation of this type of structure will not induce extra stress in the superstructure as would be the case if a frame surrounded the core.

The above analysis was carried out for the three load vectors corresponding to dead and live load, wind, and earthquake. Since the magnitude and distribution of seismic loads depends upon the natural frequency and shape of the first few normal translational modes, it was necessary to calculate these first. Rather than find frequecies from the full structure matrix of order 1488 a reduced structure matrix was generated of order ten. This is best done by the application of ten loads, one at a time, to ten points equally spaced (approximately) up the core. The ten deflections at these ten levels then give one column of the reduced flexibility matrix. A standard eigenvalue subroutine together with ten masses gave the first three mode shapes and frequencies with sufficient accuracy.

In the dynamic behaviour of the suspended system a major concern was the close proximity of the natural frequency of the centre core with the natural frequency of the suspended system.

To improve on the dynamic behaviour of the structure as a whole, it had to be considered that the translational mode of vibration of the core will have minimal effect on the vertical vibration of the suspended system. After the analysis and design of the suspended system for the vertical loads a dynamic analysis for the natural frequencies of this system was carried out and found to be well separated from the first and second modes of vibration of the centre core. (fig. #10)

The behaviour of this building during an earthquake was one of the considerations in the concept of this building. Our analysis of this building has revealed no adverse effect due to this system. In fact this building has a longer period than if it had a surrounding rigid frame and falls well below the peak acceleration of the classical earthquake spectrum of El Centro, consequently generates smaller seismic forces. (fig. #11)

For the very same earthquake consideration a research programme was devised and in two construction stages of this building various dynamic test using a vibration generator, were carried out.

Actual dynamic properties of this building were learned and the structure reanalysed under the light of this information. This research and its findings will be the subject of another paper in the near future.

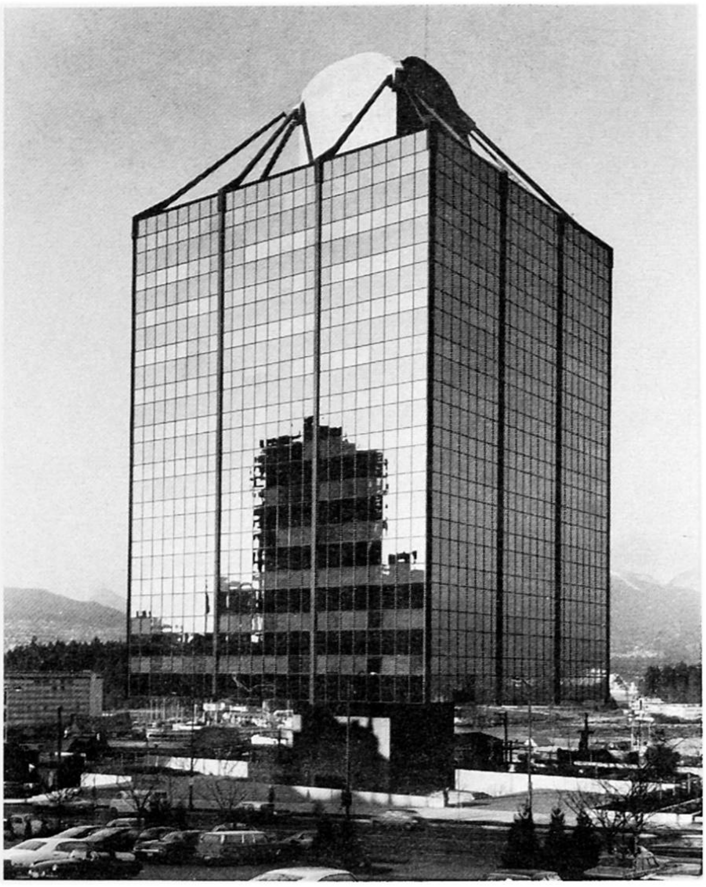


Fig. 1
The Weastcoast Office Building

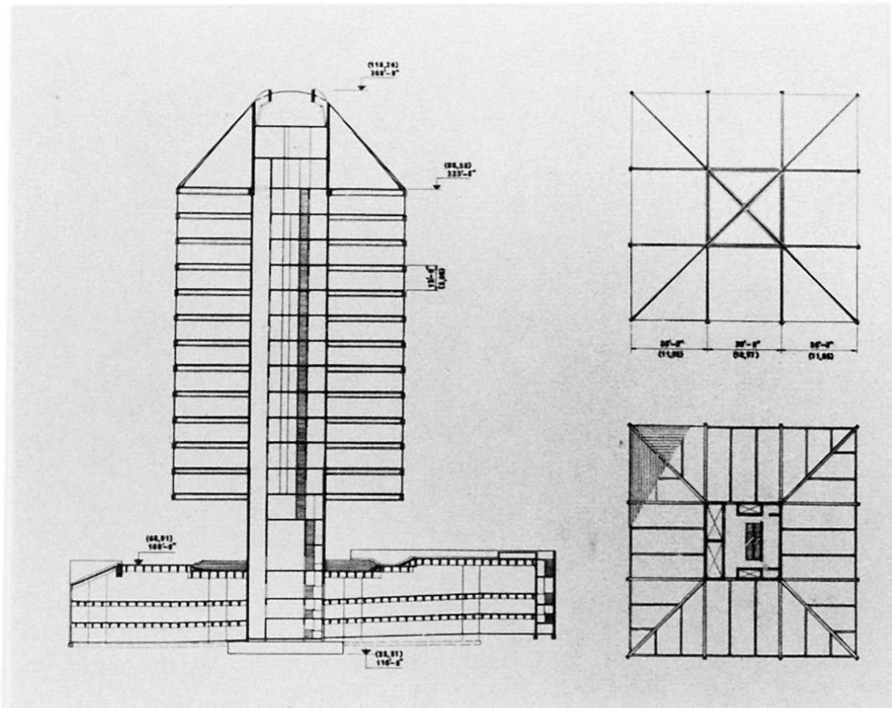


Fig. 2
Building in its Final Form

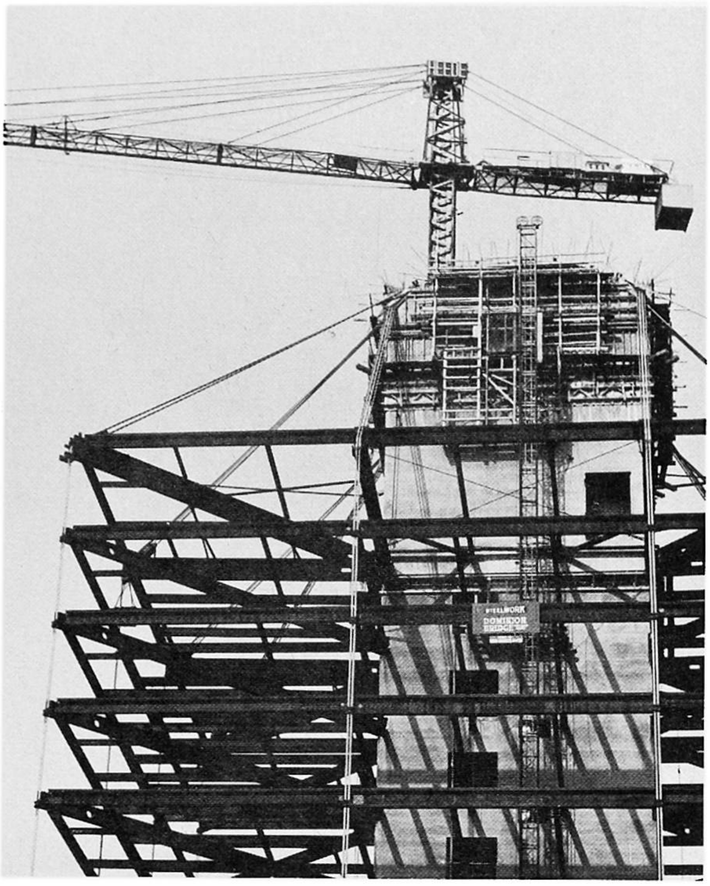


Fig. 3
A System for Hanging the
Floors from the Centre Core

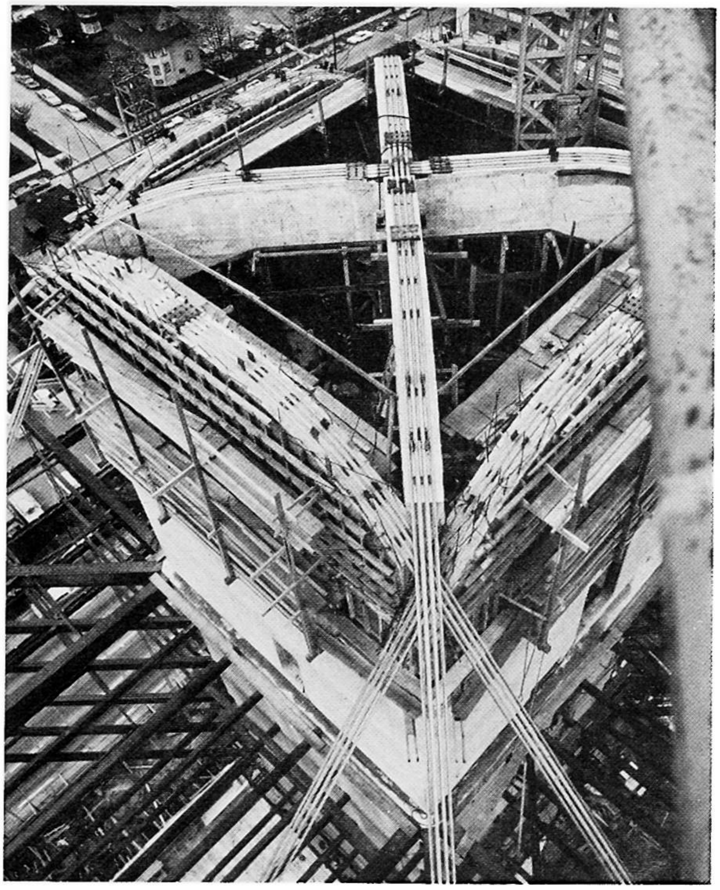


Fig. 4
Six Sets of Continuous
Cables Supported on a
Centre Core

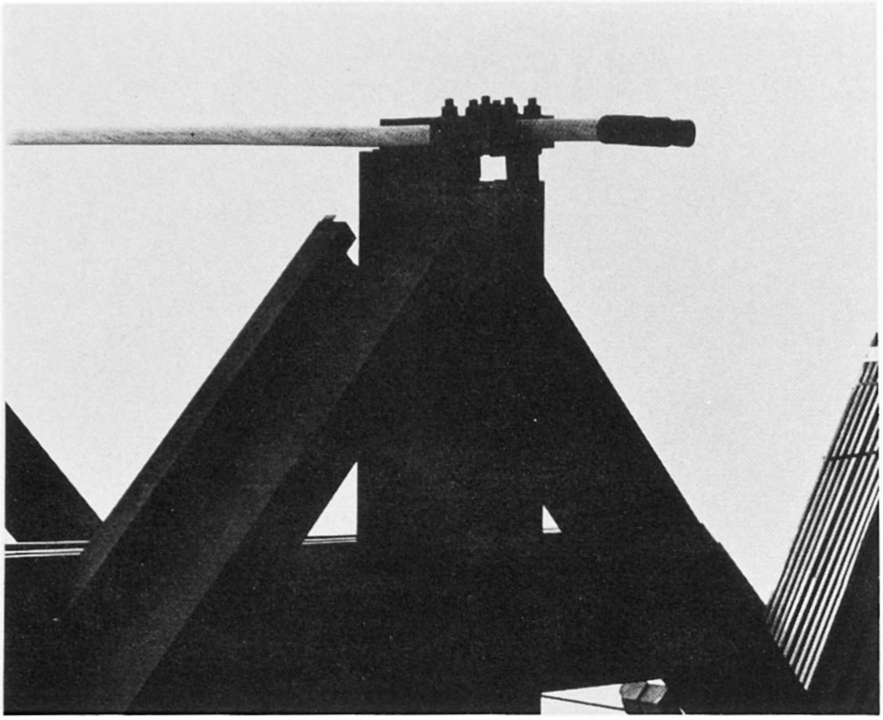


Fig. 5
The Floor Beams Attached to the Cables by Means of Friction Clamps

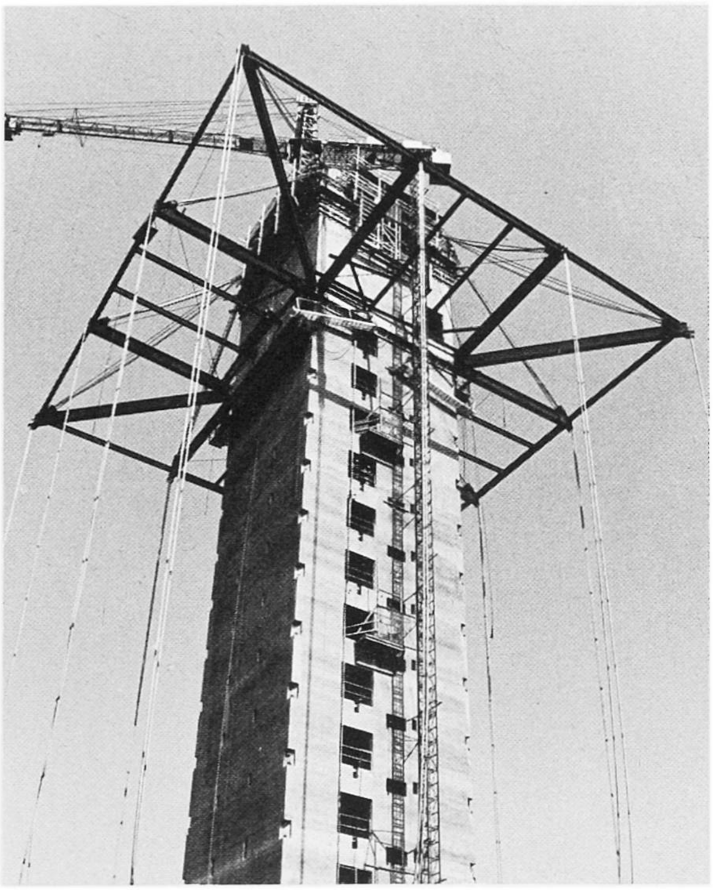


Fig. 6
Second Stage of Construction
Erection of Roof Girders and Draping of the Cables

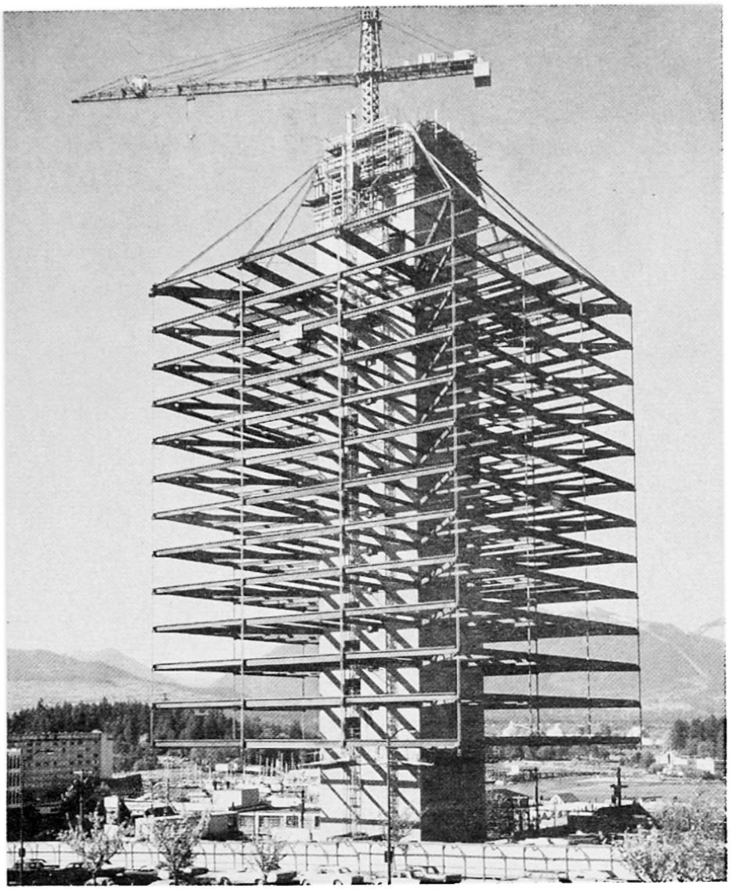


Fig. 7
Erection of the Typical Floors Progressed at a Rate of about one Floor per Day

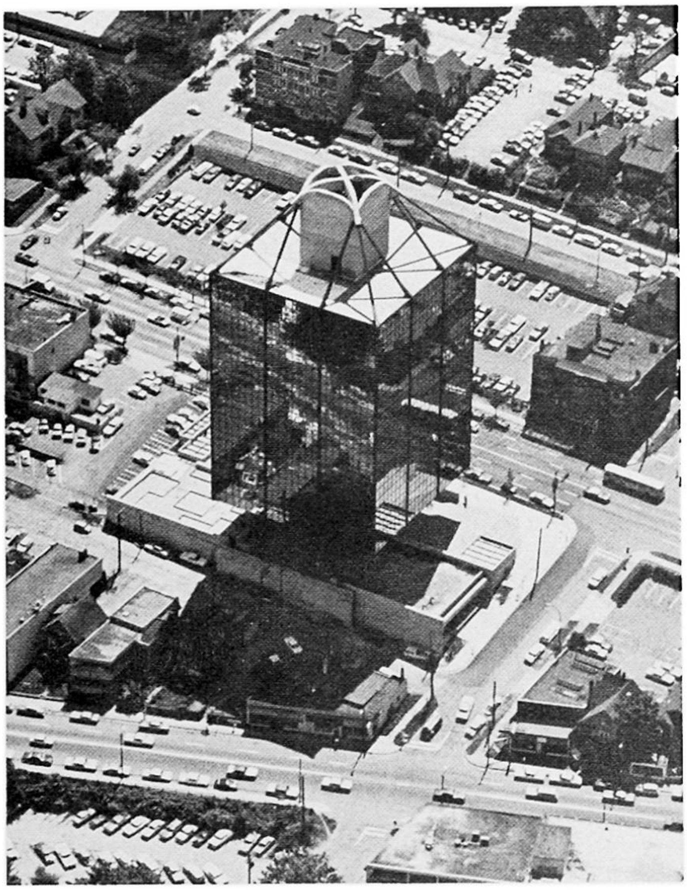
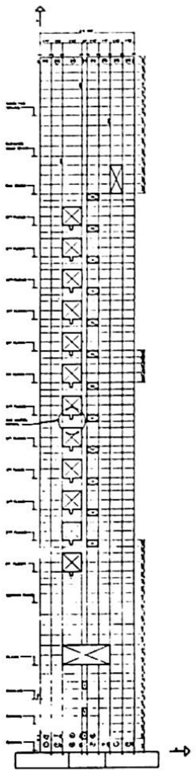
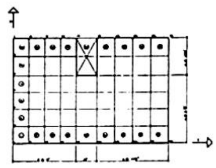


Fig. 8
Mirror-like Appearance to the Exterior Skin

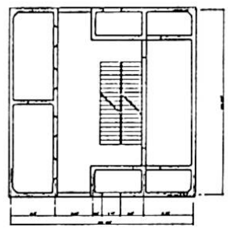
Fig. 9
Network of Finite Elements



a) Core Elevation



b) Lintel Detail



c) Core Section

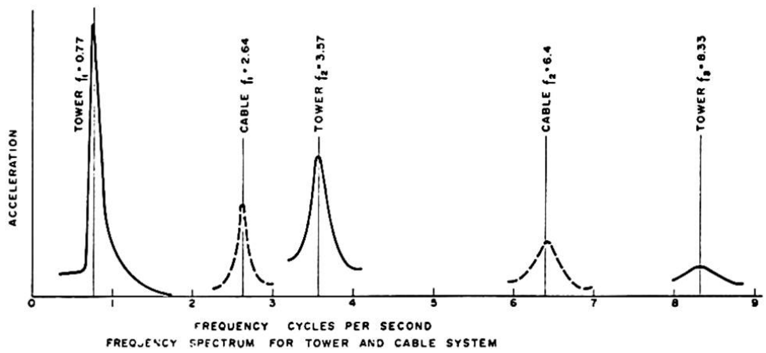


Fig. 10

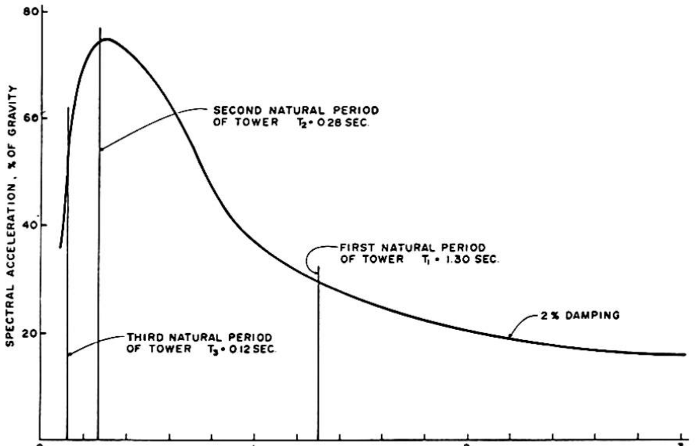


Fig. 11

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