The Aseismic Upgrading of the Structures in the “Grande Albergo” Hotel – Potenza (Italy)

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The building runs up fifteen storeys, with a maximum total height of 53 m. It was nearly 30 years old when it was subjected to a strong earthquake in November 1980, which produced substantial cracking and collapse of masonry walls, but no serious damage of reinforced concrete frames. After having restored the limited structural damages, new structures were added, designed to take up all seismic horizontal forces, leaving the old ones the task of carrying normal loads. They consist of three large prestressed trestles, 30 m high, independent from the existing frame and rising from foundations to 10th floor level.

THE ORIGINAL BUILDING

The Building was erected during the years 1952-53; it runs up fifteen storeys, six of which lie below the level of the main entrance, on the upper road. Its maximum total height is 53 m. The bearing structures are reinforced concrete, on pile foundations that follow the sloping ground. The ground itself is retained bye a stone wall and several reinforced concrete walls inserted in the main frame (see fig 1, where is represented also a photo of the structural model, with first hypothesis of two floors less for the tower).

When the hotel was built, Potenza was not officially included in the seismic areas, so that design (Architect: Antonio Costabile, Structures Aldo Arcangeli)[1] did not explicity consider the additional forces called for by the aseismic construction code. But considering the building’s special features –its height in particular-, in laying it out and in making the frame static calculations a number of measures were taken, aiming at ensuring sufficient stiffness overall and strength under any abnormal horizontal external forces. These measures were chiefly :

- L or T shaped, or elongated rectangular, columns, oriented in each orthogonal plan direction;
- Hollow-clay brick floor structures with an upper concrete slab,reinforced with a grid of distribution bars running at 45° with the ribbing;
- The central frame of the outside stairwell, having a total horizontal cross-section 3.00 * 0.35 m, comprises two vertical uprights and diagonal rods following the pattern of the stairflights, thus forming a stiff vertical trestle. (See fig. 2).

[1] On that time M.Leggeri was a young student for a Doctorate in Civil Engineering (1957). He could join to the team for the upgrade of the structures, after EQ of november 1980. Then, since 1965 Arch. Costabile ed Ing.Leggeri worked together in the same office in Potenza, named Archstudio.
Lather, Arch Costabile, in 1992 (80 years old), died.

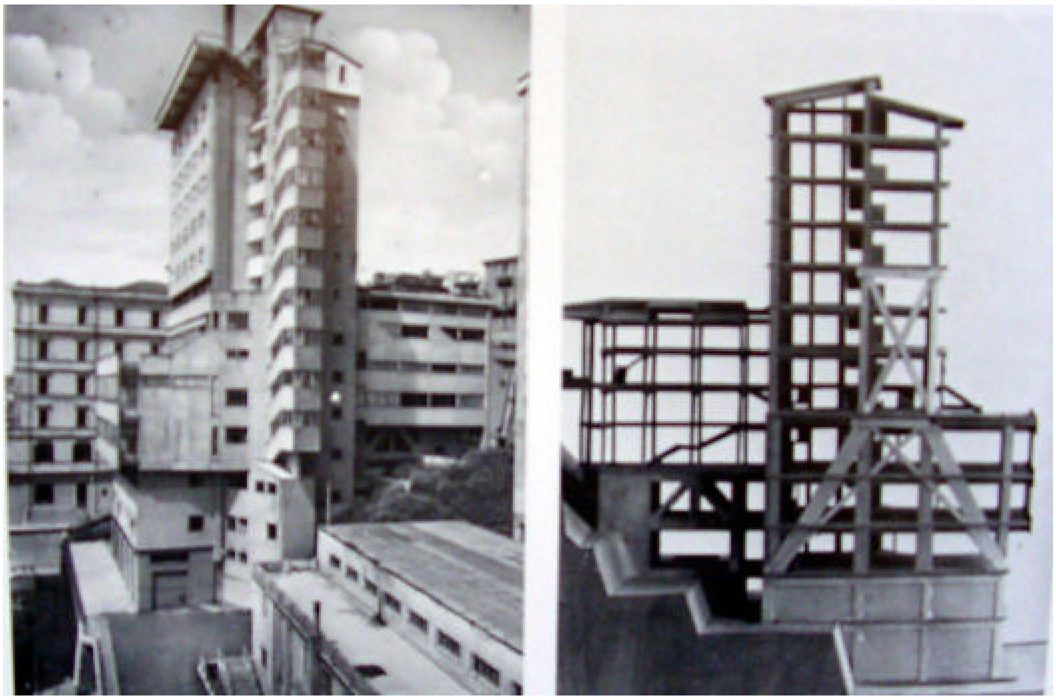

Fig. 1 : the original building (from S-E) and structural model

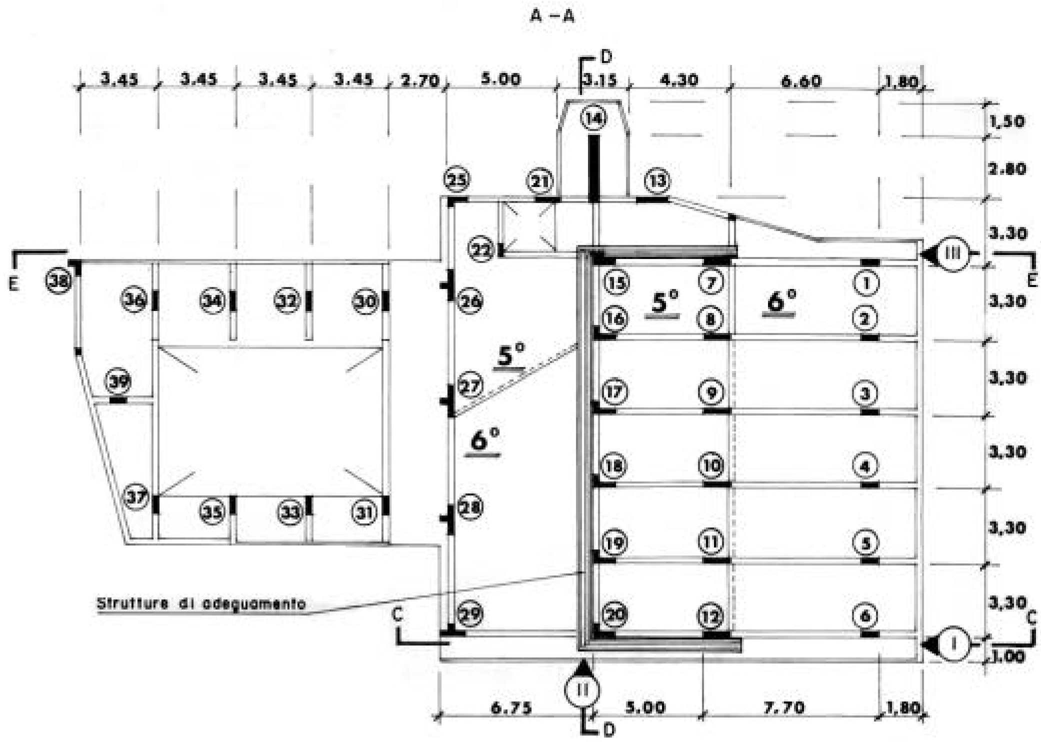

Fig. 2 : Section A – A (Plan of 5th and 6th floor Structures)

EARTHQUAKE-CAUSED DAMAGE

During the November 23, 1980 earthquake (6.8 degrees on the Richter Scale), the building took a strong shaking, the waves moving prevalently East-West. There was substantial cracking and collapse of the masonry partition walls and of the inside lining of the perimetral walls, especially in the lowers storeys of the tower and those lower lying yet. But damage to structures was much lighter, having characteristics that often made it easy to discern how they had behaved statically.

The vertical frame of the outside stairwell (described above) is the stiffest element in the whole frame, in the direction of the main shock. Approximate calculation led to estimate that about 85% of all horizontal forces were absorbed by it, and the study of the damages it underwent conform this :

- No cracks in the vertical uprights, were the traditional forces were counterbalanced by the loads;
- Traction cracks in the diagonal rods, they running perpendicularly to the rod's axes;
- Small cracks in the connections between uprights and rods, where substantial secondary bending stresses developed.

The minor nature of all these cracks suffices to show that the steel was not stressed beyond its elastic limit.

The connection between the stairwell trestle and the building is basically a beam coplanar with the trestle, which supported heavy alternating forces : about 22 tons per storey. The concrete easily absorbed the compression forces, while the traction forces, which generated cracking in the beam and in the adjacent floor slabs, were absorbed by the longitudinal bars. Here too, the steel was not stressed beyond its elastic limit.

No damage was visible in the upper tower storeys.

In the lower storeys, and in other parts of the structure, the small cracks found could all be repaired with simple local operations. The absence of cracking in the lower storeys is to be attributed to the stiff reinforced concrete walls, these exhibiting no trace of damage. The foundation structures could not to be inspected, but since any slightest movement of the foundations would have produced obvious consequences in the overlaying frame, it may be deduced that the foundations too were unharmed.

THE MEASURES ADOPTED

GENERAL REMARKS.

An overall study of the situation led to the conclusion that the structure's bearing capacity was substantially sound, that it could go on taking its normal its normal operating loads ad as well, with every probability, could further take anomalous loads. The months succeeding the main shock verified this conclusion : repeated aftershocks, even if of lower magnitude, did not appreciably worsen the damage done by the main shock. It was also concluded that the building's 28 years of operating life excluded any intrinsic structural defects, this being confirmed by its having passed a real-life test far more severe than any proof-test that might be administered. And strength test made on concrete samples taken from various parts of the structure confirmed this conclusion too.

LOCAL REPAIRS

Local, limited cracking was repaired by extensive application of steel plates epoxy-resin glued to the outer face of beams and columns, wherever these displayed the minimum indication of cracking. In the vertical stairwell trestle, the empty spaces between uprights and diagonal rods were filled with pours of reinforced concrete, well tied to the existing structures, so as to further increase their strength and eliminate any crack-caused weakening.

ASEISMIC UPGRADING

Having restored the frame bearing capacities to normal strength, the structure had now to be completed so as to inable it to take seismic forces without it undergoing excessive stress. This was prescribed by the code that hade come out in the meanwhile, and was advised anyway by the building's purpose.

It was thus first of all decided to lighten the non-bearing members as far as possible: mainly, flooring and subflooring, partition walls, inside perimetral walls claddings, etc.

This way, a considerable reduction of dead loads was obtained. More difficult was the operation in the frame, to be made by inserting structural members capable of taking the horizontal forces. In-depth studies of the several possibilities led to the solution set forth below.

THE UPGRADING STRUCTURES

GENERAL CRITERIA

The most obvious solution –the insertion of full walls into the vertical grid of the existing frame- was opted against, for it wood involve substantial collaboration statistics wise of the old structures, giving rise in them and in their foundations to relevant additional stresses, that might concentrate in some areas and thus reach too strong values. Therefore, the task of taking up the horizontal forces was assigned wholly to new, completely independent, structures, which would take up all of them, while the existing structures would bear all of the vertical loads. Such a task breakdown could be obtained only by new structures much stiffer than the old, so that, for equal strains, they would bear much more of the stresses. This was achieved first of all by choosing a triangular grid for them, rather than the rectangular grid of the existing frame, and then by building them by a material having the least possible strain ability: prestressed concrete, rather then steel or ordinary reinforced concrete.

The detail definition of these structures' shapes posed special problems, if they were to :

- Blend with the existing without interfering with their main members, whether in the standing structure, whether in the foundations;
- Leave the free spaces necessary for the cable tensioning operations;
- Not jeopardize the hotel normal functions, nor harm its aesthetics.

The arrangement chosen met these requirements, although demanding some small adaptation of the spatial distribution. This did not, however, change the building's operating characteristics.

DESCRIPTION

The added structures comprise three large frames (see fig. 3), solidly joined together , that essentially buttress the tower body : two crosswise, one at each end of the tower, running North-South in plan, and one central, running longitudinally East-West. They rise up better than 30 m from the foundations to the 10th storey floor structure, and are tied to the existing structure ad the 2nd , 6th and 10th floors.

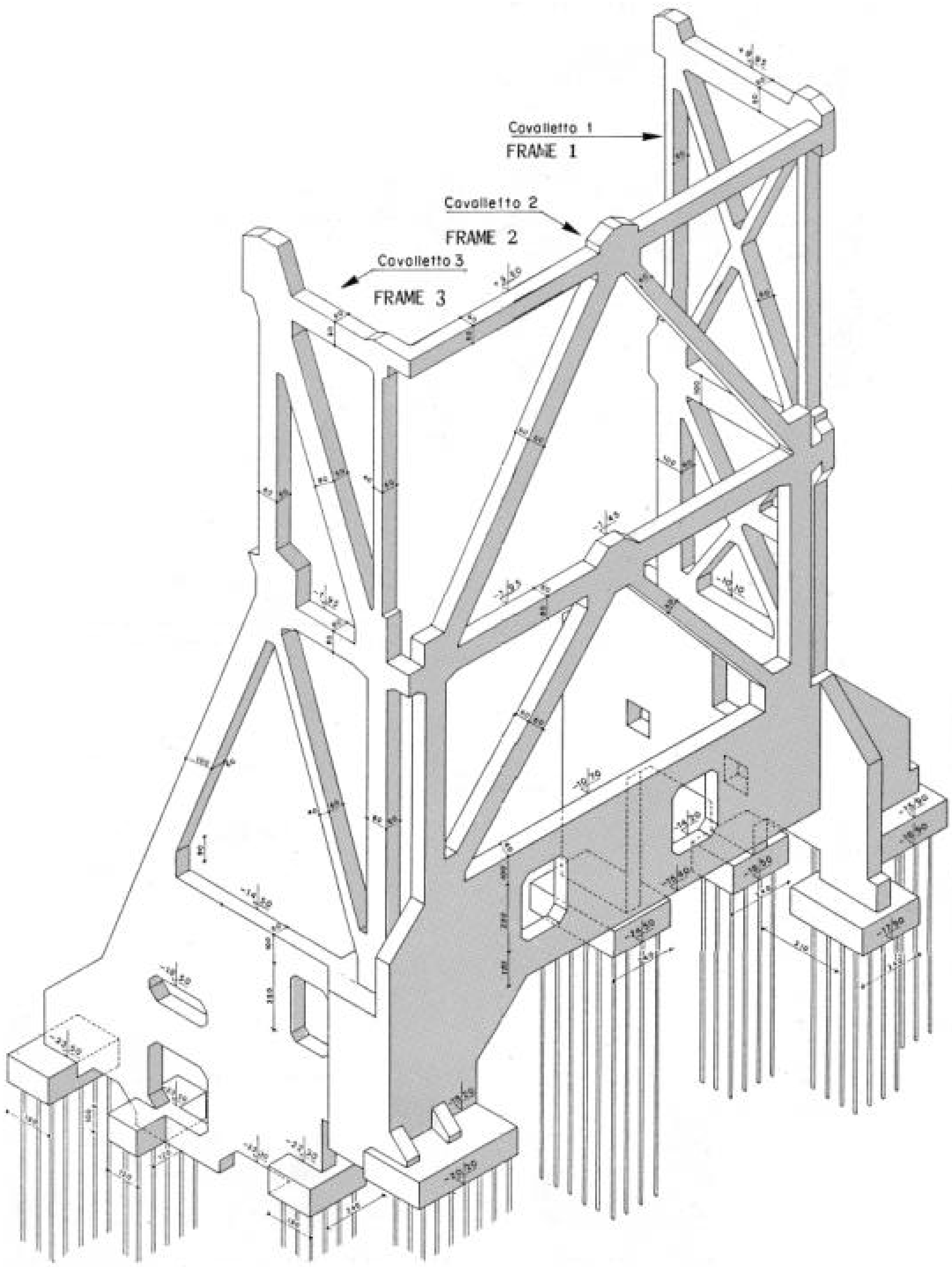

Fig. 3 : Isometric View of the new Structures

It was considered that the existing structures were easily capable of transmitting at these elevations the horizontal forces generated at the intermediate and higher levels.

The lower part of the frames are made up of stiff full walls, and stand upon micro-piles foundations, which are independent of the old. Their upper areas are shaped according to a triangular grid, with constant depth of 60 cm, set 5 cm away from the adjacent old structures (see fig. 4, 5, 6). Their plane arrangement is such that the Center of Stiffnesses, in both normal directions, is very near the Center of Masses, so that horizontal torsional effects are cut down to the minimum.

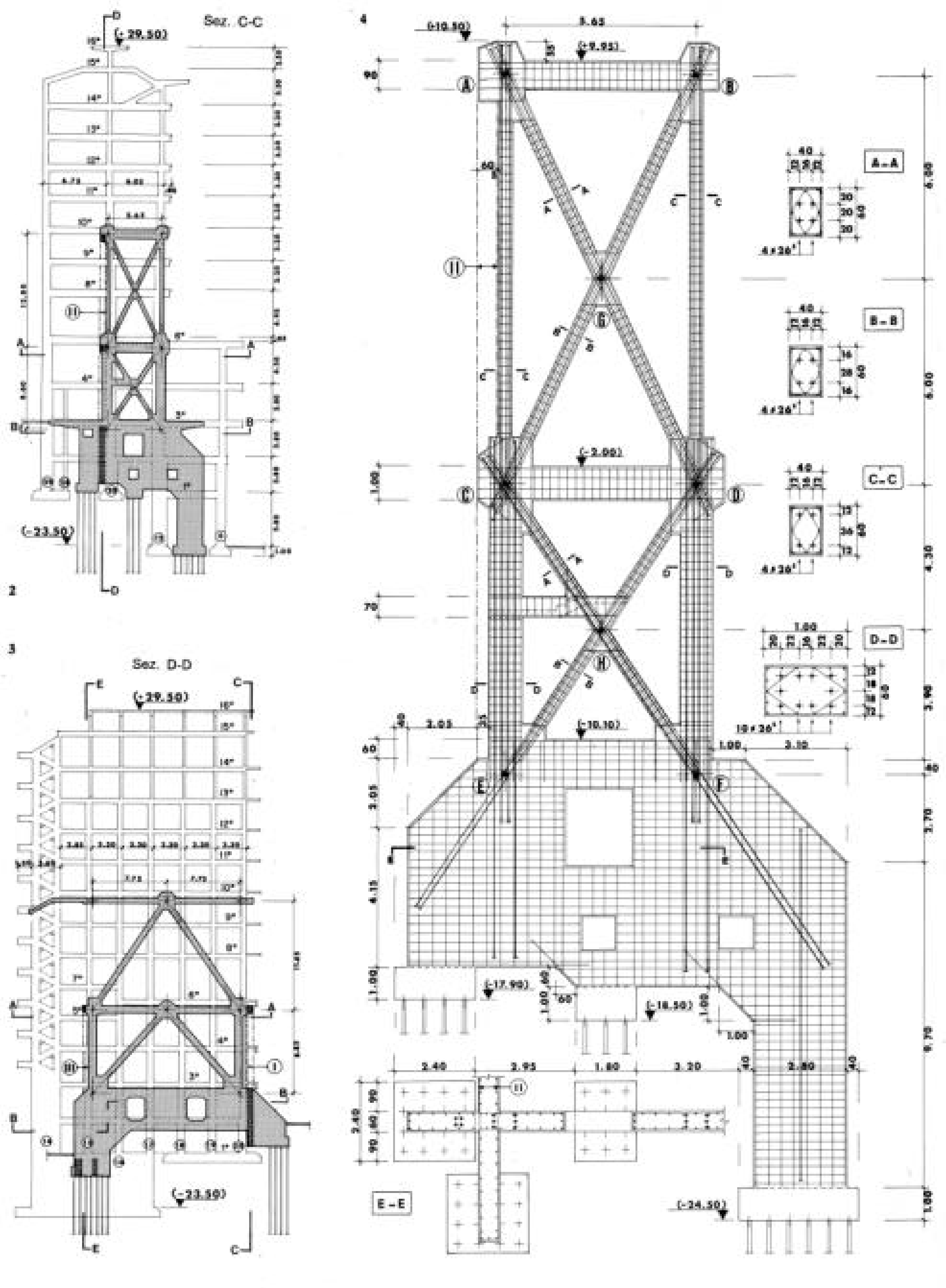

Fig. 4 : Section C-C (Frame 1)

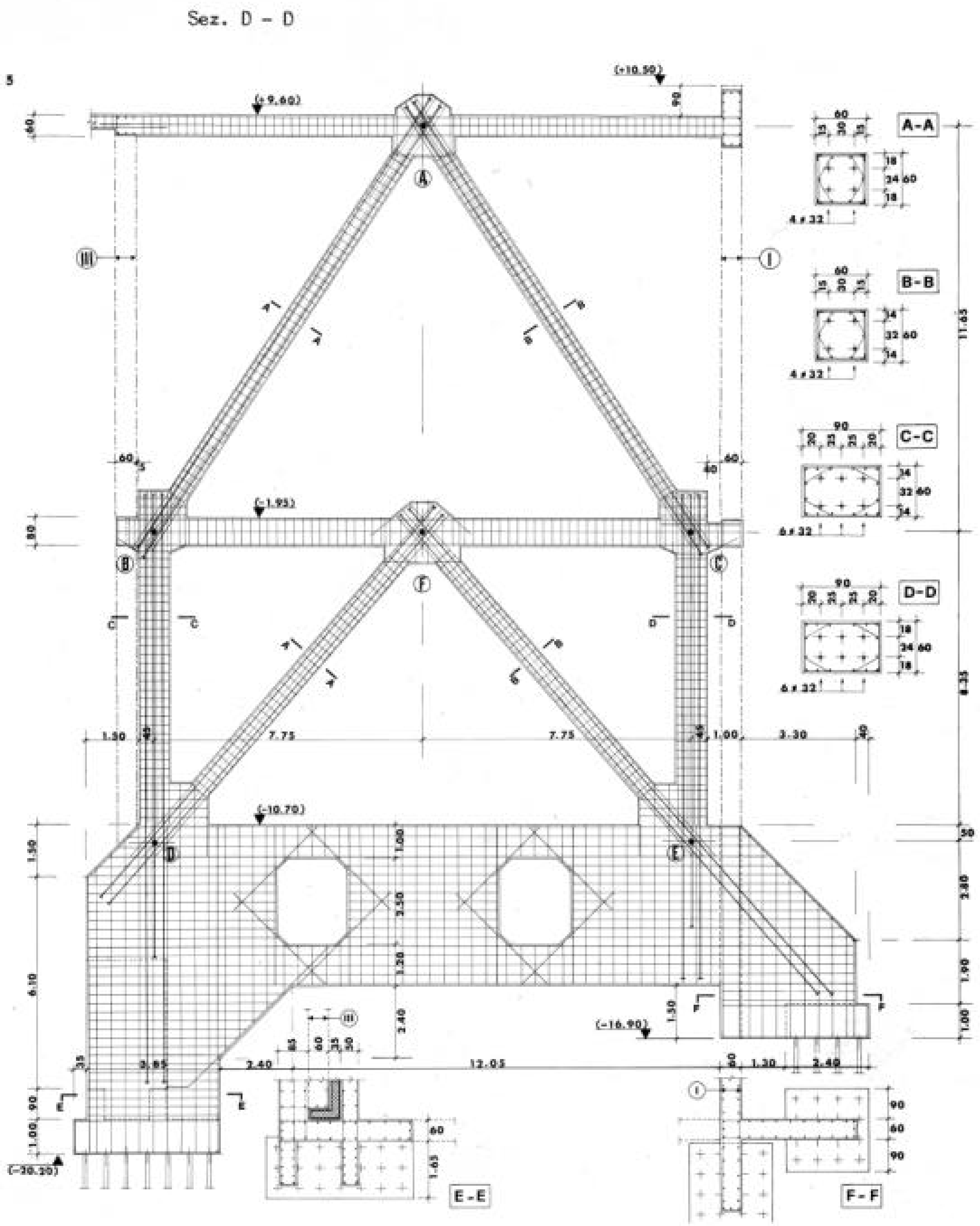

Fig. 5 : Section D-D (Frame 2)

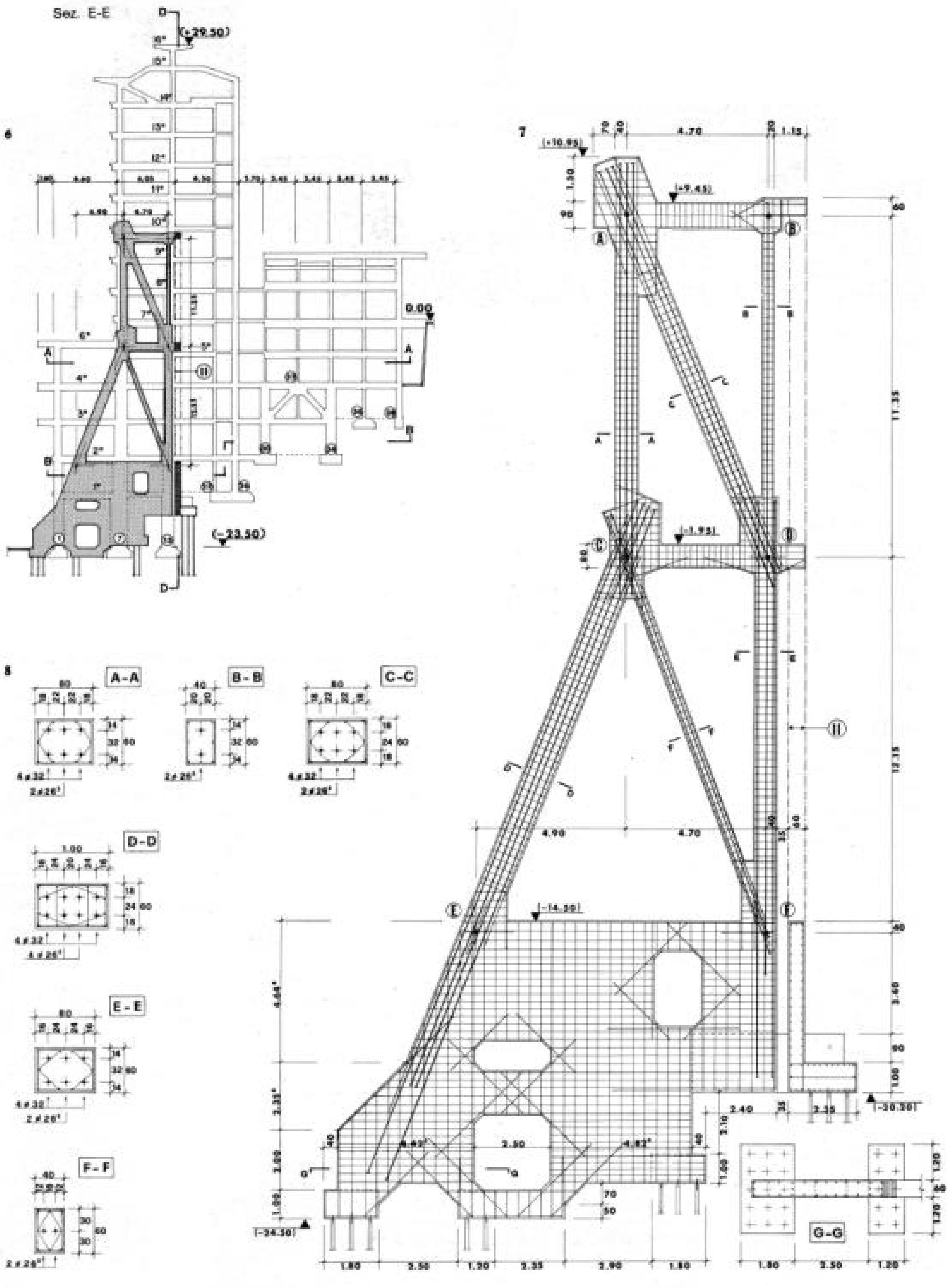

Fig. 6 : Section E-E (Frame 3)

After some time had passed from the completion of the aseismic frames, they were tied to the existing structures by means of through pins and grouting. The delay was necessary for the new foundations to settle and so that the shortening of the rods during tensioning would not induce harmful additional stresses in the existing structures.

Special care was devoted to the study of the procedures affected by the passage of the new, so as not to harm them statically.

During construction, surveying the exact routes for the rod centerlines, many of the rods being inclined and all of them passing through more than one floor structure, was as practically difficult as it was important. In fact, any departure from a rectilinear route would be a source of transverse forces in the rods at the time of tensioning.

This difficulty was got over by first arranging cages made up of steel angle-irons that precisely defined the required profiles and permitted the exact placement of the pretensioning bars.

They were then incorporated in the pour, together with the other stirrups and longitudinal reinforcing bars (see fig. 7).

The prestressing used the materials and procedures of the DYWIDAG System, ϕ 26.5, and ϕ 32 mm bars being used, of 85/105 steel.

They were laid in segments and then joined together with sleeves. The tensioning was performed from the bar upper ends, their lower ends having been first fixed-anchored.

The bars were all tensioned in pairs, symmetric relative to the cross-section center, so as to avert any stress excentricity.

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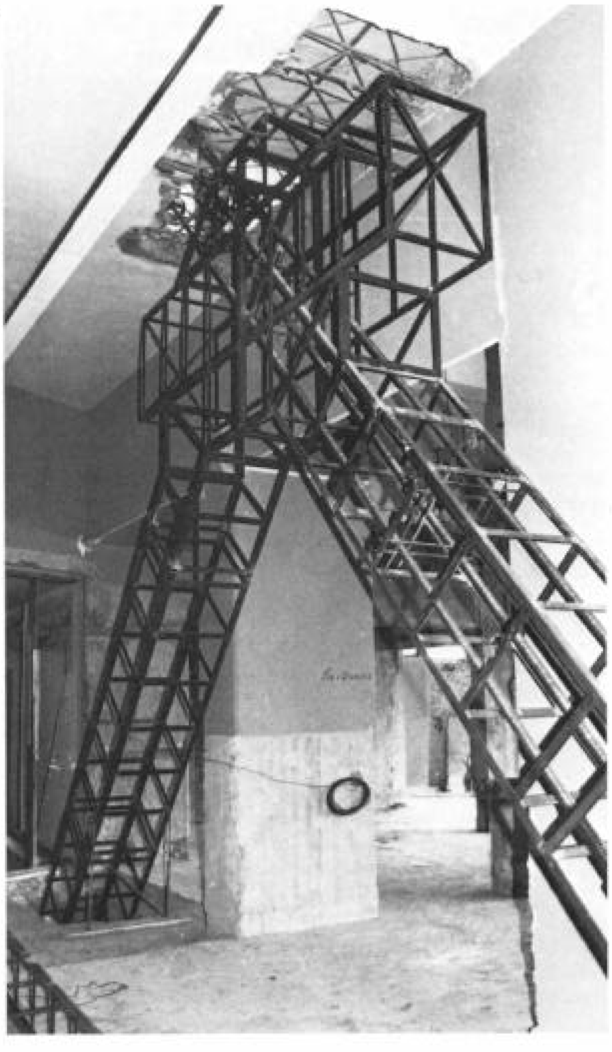

15 - Ingabbiatura di tracciamento ed armatura delle aste del cavalletto n. 1; 16 - In primo piano un'asta del cavalletto n. 2, sullo sfondo il cavalletto n. 1 e a destra, vista delle barre di precompressione con le relative giunzioni intermedie; 17 - Il cavalletto n. 2: incrocio delle ingabbiature di tracciamento in corrispondenza del nodo al quinto solaio; 18-19 - Barre di precompressione: giunzione e campane di ancoraggio; 20 - Le operazioni di tesatura delle barre di precompressione; 21 - Il piede del cavalletto n. 3 in corrispondenza del piano terra; 22 - L'inserimento della struttura di adeguamento sismico nella preesistente costruzione.

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Fig. 7 : Cages made up of steel angle-irons

CALCULATION CRITERIA

The calculation were carried out in accordance with the Italian code, the seismic forces being considered by a static analysis. No account was taken of such favourable factors as the presence of the reinforced concrete walls in the lower parts and the considerable lightening of the dead load owing to the non-bearing construction member (mentioned above). Thus, a greater safety factor was achieved, one that would amply compensate for the inevitable approximation in assessing the amount and distribution of the forces in play and in the correspondence between the static scheme and its actual dynamic counterpart in the phenomenal world.

With the unit weights and the total weights per each floor structure determined, together with the corresponding horizontal forces, corrected bye the prescribed storey coefficients, the stresses in the individual rods in each aseismic frame, induced by external and prestressing forces, were computed. These were calculated by solving the force-balance and strain-congruence equations, without any very laborious development : at most a system of three linear equations had to be resolved.

The stress distribution in the aseismic frame was also calculated using the computer (mainframe), the result obtained being in excellent agreement with those got by traditional means.
It was found though that the computer calculations, formulated without taking account of the static scheme symmetry, provided somewhat less approximate results, considering the greater complexity of the processing demanded.

The bar pretensioning was so proportioned as to keep a residual compression in concrete even when external forces act so as to generate maximum traction in each individual rod.

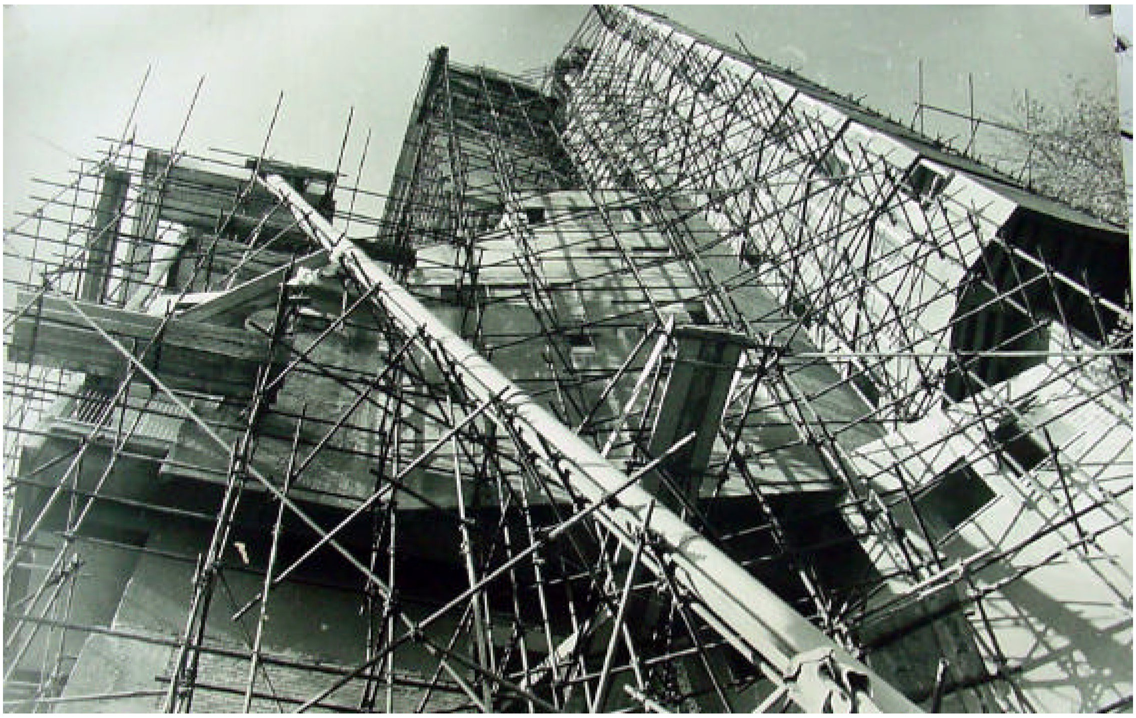

Fig. 8 - Works nearing completion

NOTES

[1] All the figures are reprinted with approval from an article by the same Authors, in “L’Industria Italiana del Cemento” – Rome, N. 610, April 1987, pages 276-284.

[2] After many year of completion, it is now the right moment to go back, in the light of the latest developments of calculations criteria and the researches extended for the territory, such local seismic sources, laws of amplification and attenuation and effects produced on the new structures of the Hotel by earthquake of May 5, 1990 with epicenter far only 4.9 Km. (See References below).

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