ENGINEERING IMPLICATIONS OF THE BUCHAREST COMPUTING CENTER COLLAPSE

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SUMMARY

A dramatic building failure in the Romanian earthquake of March 4, 1977, was the collapse of a modern computing center in Bucharest. This paper discusses the collapse, identifies three contributing factors, and considers their implications for earthquake resistant design.

INTRODUCTION

In the Romanian earthquake of March 4, 1977, the computing center of the Ministry of Transportation in Bucharest collapsed. In so doing, it shared a distinction achieved by such predecessors as the Four Seasons apartment building in Anchorage and, perhaps to a lesser degree, the Olive View Hospital in San Fernando. All were intended to comply with modern seismic building codes in their design and construction. What went wrong at Bucharest? What ought we do to reduce the risk of repetition?

THE BUILDING

The computing center was a three-story reinforced concrete structure of striking architectural design. Fig. 1 shows the building before the earthquake. It was a flat plate building with service towers at either end housing the stairwells, elevators, etc., and a main building in between. The central building was structurally separated from the service towers. The service towers remained intact, but the central building collapsed, as shown in Fig. 2.

The main building was a three-story structure 30 x 30 m. in plan, consisting of cast in place reinforced concrete columns and floors and precast concrete walls. The floors were supported on just nine columns, three rows of three, spaced on 12 m. centers, as indicated in Fig. 3. Square tied columns were used in the upper two stories. The ground story columns were also tied, but they were tapered from either 50 or 60 cm. square at the top to a fluted shape 1 m. across at the bottom, as shown in Fig. 4. They were supported on spread footings. The floors and roof were cellular flat plates either 50 or 55 cm. thick, comprising 6 cm. thick slabs top and bottom separated by 20 cm. wide ribs at 1.2 m. centers both directions. The mass of the slabs was 680 kg/m^2 (140 psf) for the floors and 635 kg/m^2 (135 psf) for the roof. The exterior walls were precast panels supported on the floors. Their mass was 1200 kg/m (820 lb/ft) of

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wall length. The exact numbers are unimportant for present considerations. The point is that it was a massive building.

The exterior walls had continuous bands of windows in each story, and thus did not act as shear walls to limit intrastory lateral displacement. Moreover, the ground story walls were offset so as to bypass the exterior columns. Thus virtually the entire lateral strength and stiffness of the main building was provided by the nine columns, unaided by any structural walls.

The ground story columns were reinforced by 20 to 36 longitudinal bars 20 to 25 mm. Ø, contained in 8 mm. Ø ties. In all but the center column, some of the longitudinal bars stopped short of the full height of the column. Because of the tapered and fluted shape of the columns, ties were made up in sets, each set consisting of a square closed hoop and four hairpin bars, as shown in Fig. 4. Tie sets were spaced at 15 cm. in the bottom 1.25 m. of each story and at 20 cm. for the rest of the free height. Four of the longitudinal bars were contained within the closed square hoops; the rest lay within the hairpin bars but outside the closed hoops. The concrete was of good quality and the building showed evidence of good workmanship and care in its construction.

MODES AND PERIODS

Dynamically, this was a nice clean building, uncomplicated by any added stiffness due to shear walls or nonstructural components. The main building was essentially a three degree of freedom system having only the columns to provide lateral stiffness. The three modes and their associated periods are shown in Fig. 5. These are the modes computed on the idealized condition of fully fixed column bases and completely rigid floors. Any flexibility in either the footings or the floors would lengthen the computed periods.

GROUND MOTION

The ground motion was not recorded at the building site, but it was recorded at INCERC, the Building Research Institute, just a few kilometers away. A spectrum of the N-S component of that motion is shown in Fig. 6. It is not what we think of as the usual spectrum, for the pronounced peak out in the 1.5 to 2 second period range is not what we are accustomed to seeing. However, there is no question that the recorded motion did occur in Bucharest, and the regional geology gives cause to believe that this may be a prevalent characteristic of Bucharest ground motion. The spectral values are large. For comparison, the Housner spectrum intensity for 5% of critical damping is 247 cm. for the Bucharest ground motion, and 135 cm. for El Centro 1940.

if the following condition is fulfilled:

$$0.33 \cdot K_{c.н.} < T_{т.б.} < 1.25 \cdot K_{c.н.} \qquad \text{(Eq.2)}$$

where $T_{т.б.}$ - the period of free oscillations of the fundamental tone of a heavy concrete frame building.

As a rule, rigid frame buildings with height up to 7 storeys meet the condition (2).

While designing, dynamic calculations of such systems are made under elastic state assuming that frame units and the connections are absolutely rigid and the behaviour of frame elements is non-linear. A number of investigators [1, 2] prove that when the distortion of framed units and non-linear behaviour of frame elements are not taken into consideration under considerable horizontal loads it can lead to the reduction of the reserve coefficient and, as consequence, to insufficient reinforcement.

In this connection, in order to introduce keramzite concrete frame systems into seismoresistant construction the laboratory of TSNIIEP zhilischa dealing with strength tests has carried out a complex of tests on a full-scale rigid frame skeleton and its keramzite concrete elements. Below, there are given the results of investigations of two frame units which are the main elements determining the reliability and rigidity of the whole skeleton.

The structure of the experimental unit taken from a real 3-storey building is presented on fig. 1. The installation of the sample on the stand was performed in accordance with the scheme given on fig. 2. The average concrete strength of the units made up 23.55 mega-pascal, the initial elastic modulus - 13970 mega-pascal. Under constant load N = 176.6 kN, the load P_K was applied to the end of a cantilever by stages increasing up to the given value at every cycle of tests and then decreasing up to zero. Concrete deformation was registered by electric tensometers and clock-type indicators located in the central zone of the unit and in the adjacent areas of posts and cross-bars.

At all stages of tests there was observed a proper grip of cross-bar longitudinal working reinforcement with keramzite concrete. When a transverse force at the supporting section (at the base) of the cross-bar made up Q = 15.7 kN (M = 0.19 Multimate) there appeared cracks in the tensiled zones of concrete with stresses being 1.12 mega-pascal; under further loading the cracks opened, however, the destruction did not occur as the ultimate state of the units was determined by the strength and deformation of the central zones. Their plastic deformation began when Q = 17.6 kN = $0.3 Q_{ult}$ (1st unit) and Q = 24.4 kN = $0.31 Q_{ult}$ (2nd unit).

Under load being Q = 53.0 kN = $0.93 Q_{ult}$, in the central zone of the first unit a visible crack developed perpendicularly to the extended diagonal which quickly opened and under force being Q = 56.9 kN the unit was destructed. Q ult. and Mult. are ultimate values of uses T^{-1}. Although soundly based in scientific principles, these provisions nevertheless reward the designer for using a flexible structure with a long period. American codes provide a further bonus in the form of a K factor related to the type of structure. K has its lowest value, 2/3, for an unbraced ductile moment-resisting frame. Along the same lines, the ATC-3 recommendations provide the greatest reduction coefficient R for the unbraced ductile moment-resisting frame. The use of shear walls or bracing incurs the penalty of a greater design force; yet a review of the record of structural damage in earthquakes would, I believe, confirm the correctness of Dr. Naito's principles.

The third factor, and the most crucial, was the lack of a second line of defense. Once a column failed, there was no other structural component to come to the rescue. A second line of defense is not mandated by any building code of which I am aware. The ATC-3 recommendations address the issue in a provision for non-redundant systems which provides "The design of a building shall consider the potentially adverse effect that the failure of a single member, connection, or component would have on the stability of the building and appropriate design modifications shall be made to mitigate this effect."

In a sense, the Bucharest computing center was an extreme example of the soft ground story--a source of much grief in one earthquake after another. We ought now seriously to consider our building codes in the light of this disaster and decide whether earthquake engineering is well served by continuing the double bonus our codes now offer as an incentive for selecting flexible, unbraced structural systems.

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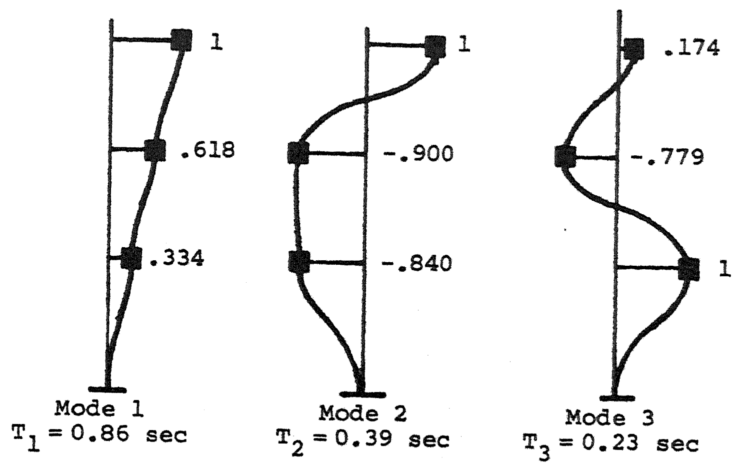

Fig. 5. Modes of vibration.

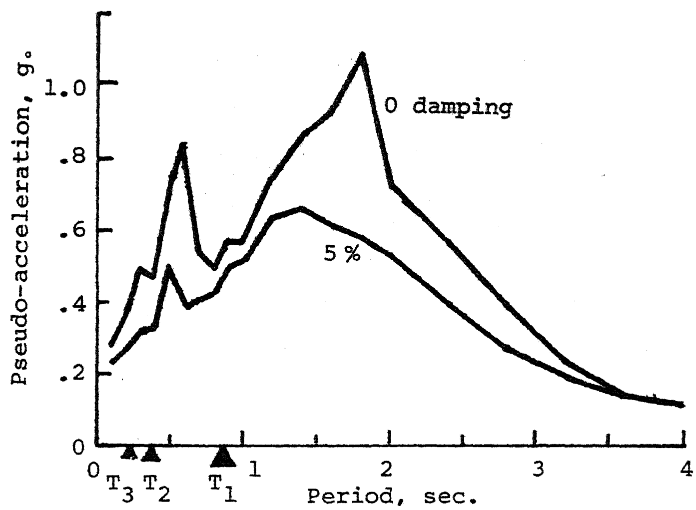

Fig. 6. Acceleration response spectrum.