PREPARED DISCUSSION BY H. AOYAMA, JAPAN

on paper No.5 "Evaluation of Olive View Hospital Behavior on Earthquake Resistant Design", by L.G. Selna, M.D. Cho, and R.K. Ramanathan, California, U.S.A.

1- Based on design material properties and ACI capacities the story shear capacity coefficient in the first story in this paper is 0.44.

A similar analysis at the University of Illinois gave story shear capacity coefficient of 0.39. Was the capacity of tied columns that would have failed prematurely excluded in your analysis? Was the so-called P-delta effect included?

2- The story shear capacity coefficient of the ground story of 0.45 obtained at the University of Illinois could be too high because the capacity limit of floor system was not considered. However, the estimation of the capacity of floor system depends on many assumptions such as effective width or axial force. What were the assumptions in your analysis?

Discussion Closure: Session 1A, Paper 5

EVALUATION OF OLIVE VIEW HOSPITAL BEHAVIOR ON EARTHQUAKE RESISTANT DESIGN(1)

L.G. Selna(2) - The query by Prof. Aoyama regarding the assumptions used in the computation of the story shear capacity coefficients for the main building is most welcome. His initial suggestions and guidance(3) provided the basis for the story shear capacity study(4) conducted at UCLA.

Based on design material properties and ACI capacities a story shear capacity coefficient for the first floor was computed to be 0.44. Prof. Aoyama computed 0.39 in a similar study at the University of Illinois(UI). With reference to first floor capacities he asks two questions: 1) Was the capacity of tied columns that failed prematurely excluded in your analysis? 2) Was the so-called P-delta effect included?

Corresponding results for the ground floor are also in question. At UCLA 0.32 was found, and at UI 0.45 was obtained. With reference to the ground floor he states, "The story shear capacity coefficient of the ground story of 0.45 obtained at UI could be too high because the capacity limit of the floor system was not considered. However, the estimation of the capacity of the floor system depends on many assumptions such as effective width or axial force." He then asks, "What were the assumptions in your analysis?"

The assumptions used in the story shear capacity calculations conducted at UCLA were as follows:

1) Design materials strengths were used.
2) ACI(5) procedures were followed for computing moment and shear capacities of reinforced concrete beams and columns; spalling of the column shells was considered in the computation.
3) Axial dead loads were used in finding moment and shear capacities of columns.
4) An incremental static structural analysis procedure was followed. This analysis treated: a) P-delta effect;

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1 June 1973 by L.G. Selna, M.D. Cho, and R.K. Ramanathan

2 Discussion of question by Prof. Aoyama regarding story shear capacity coefficient prepared by L.G. Selna.

3 Private communication from Prof. H. Aoyama, Dec. 1971.

4 Selna, L.G. (Editor), "Olive View Hospital Damage Report," UCLA-ENG Report (in preparation), Univ. of Calif., Los Angeles.

5 "Building Code Requirements for Reinforced Concrete," ACI 318-71, American Concrete Institute, Detroit, 1971.

b) flexural hinging and shear failure in the columns; c) hinging in the beams; d) hinging and punching in the slab due to moment transfer between the columns and the drop panels; for computing the moment capacity of the slab a full panel width was used; e) structural coupling between ground and first floor framing system.

5) Also a virtual displacement equilibrium calculation based on field observations and computed element capacities was performed.

The story shear coefficients found from the capacity assumptions, field observations of damage, and the structural analysis procedure are depicted in Table 1.

The clamped column capacities (Row 1) are based on full hinging in the tied and spiral columns and no rotation in the beams or drop panels. The high C values, in excess of 0.4, show that the building had a potential for high story shear development in the columns - about 5 times the design story shear.

Rows 2 through 4 in the table are based on the virtual displacement computation using the hinging patterns observed in the field. The second row shows that when the beam and drop panel hinging is considered the story shear capacity drops in the ground floor while remaining the same in the first floor.

The effects of spiral column spalling damage can be evaluated in Row 3. The capacity decrease is greater in the first floor because the moment that can be developed at the top of the ground floor column remains unaffected by the spalling. The column moment capacity reductions, which occur at the base of the ground floor, at the base of the first floor column, and at the top of the first floor columns, lower the C in the ground floor from 0.32 to 0.25 while in the first floor C reduces from 0.44 to 0.28.

In Row 4 the remaining hinging results which consider tied column failure and P-delta effects are presented. Considering the various virtual displacement hinging studies these are the most realistic indicators of the present condition of the building. The ground floor result is 18,000 kips and the first floor result is 12,000 kips. These results were found considering full column capacities in ground and first floor for the ground floor result, and reduced column capacities in the ground and first floors for the first floor results. This tended to give a slightly low value in the ground floor result. In any case the values are quite accurate. The C values are 0.14 for both floors which indicates that even in its damaged condition the building had a capacity which was twice its original design story shear.

Rows 5, 6, and 7 of Table 1 refer to the force-deflection studies mentioned in assumption 4. The full capacity results presented in Row 5 agree well with similar results based on assumed hinging which are presented in Row 2. The difference is due mostly to representation of tied column failures in the force-deflection results. Again the C profile is undesirable: C=0.21 in the ground floor and C=0.44 in the first floor. Still more unique is that the first floor shear capacity was greater than the ground floor shear capacity. Of course equilibrium constraints would have prevented the development of the capacity forces in the first floor except when the north retaining wall was acting as it did in the earthquake. This gave the ground floor sufficient shear capacity to allow full development of shear capacity in the first floor.

The next row (6) indicates the loss due to spalling of shells in the spiral columns. In this case C drops to 0.14 in the ground floor and 0.27 in the first floor.

Of all the studies the results of Row 7 give the best indication of the damaged condition of the building. To best match the damaged condition of the building the full column capacity was idealized in the ground floor while reduced column capacity was used in the first floor. P-delta and tied column failure effects were included. The story shear capacity results are 22,400 kips in the ground floor and 13,600 kips in the first floor. These results are slightly higher than in Row 4 because of the column cross section assumptions. The ground and first story shear capacities in this damaged state were approximately twice the original design story shear.

These results show that a range of capacity coefficients was obtained. The values presented in the paper correspond to Row 2 of Table 1 because they represented kind of an ideal condition with the tied columns acting to resist the earthquake. The authors thank Prof. Aoyama for his interesting questions.

Table 1

LOWER FLOOR STORY SHEAR SUMMARY

Row	Idealization	GROUND FLOOR Resulting Shear Capacity (kips)	GROUND FLOOR C	FIRST FLOOR Resulting Shear Capacity (kips)	FIRST FLOOR C
1	Columns Clamped Against Rotation Top and Bottom	53,000	0.41	36,660	0.44
2	Assumed Hinging, Full Spiral Column Capacity, Tied Columns Included	39,300	0.32	36,660	0.44
3	Assumed Hinging, Reduced Spiral Column Capacity, Tied Columns Included	31,180	0.25	23,170	0.28
4	Assumed Hinging, Tied Columns Failed Variable* Spiral Column Capacity, P-Δ Effect	18,000	0.14	12,000	0.14
5	Force-Deflection, Full Spiral Column Capacity, Tied Columns Failed (R = 0.25)	27,500	0.21	36,200	0.44
6	Force-Deflection, Reduced Spiral Column Capacity, Tied Columns Failed (R = 0.25)	18,700	0.14	22,400	0.27
7	Force-Deflection, Field Condition** Spiral Column Capacity, Tied Columns Failed P-Δ Effect	22,400	0.17	13,600	0.16

*Full Capacity in Ground and First Floor for Ground Floor Result
Reduced Capacity in Ground and First Floor for First Floor Result

**Full Capacity in Ground Floor and Reduced Capacity in First Floor