MODIFICATION OF STRUCTURES TO SATISFY NEW SEISMIC CRITERIA

by

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SYNOPSIS

An existing hospital facility was evaluated for earthquake resistance in accordance with new criteria, more severe than the original design criteria, established by the Veterans Administration (VA), Office of Construction. The conclusions of the evaluation are that the hospital complex does not conform to the new seismic criteria, and the structure will require major corrective action. Several strengthening modification schemes were explored for their structural feasibility, dynamic characteristics, and functional application. The decision process for selecting the proposed scheme is outlined here, and some of the structural details are presented.

INTRODUCTION

Seismic design criteria have changed substantially in recent years, influenced by results of current research, development of state-of-the-art techniques, and experience from recent earthquakes. The seismic resistance capacities of many existing public and government buildings, especially hospitals, are being evaluated to determine their degree of conformance to current seismic design requirements. For example, the VA has been conducting a program to evaluate the earthquake resistance of their hospital facilities.

After the 1971 San Fernando earthquake, the VA authorized an extensive program to reduce the earthquake risk at VA hospitals, and an appointed committee of consultants developed requirements for earthquake-resistant design of these facilities.[1] These requirements have been issued for designing new hospitals and for determining needed corrective measures for existing hospitals in geographic areas subject to earthquake activity.[2] Consultants were also retained to study the seismic and geologic hazards of geographic areas where VA hospitals are located and where moderate or major earthquake damage has occurred. In addition, instruments to record future earthquake motion have been or are being installed at these sites. The most critical sites have been studied by architect-engineer firms to determine whether the structures can withstand earthquake forces. Those structures that cannot withstand the prescribed earthquake force were studied further to propose strengthening schemes.

The VA General Medical Hospital in Memphis, Tennessee, is one such site that was studied by the joint venture of Walk Jones + Francis Mah, Inc., of Memphis, Tennessee, and URS/John A. Blume & Associates, Engineers (URS/Blume), of San Francisco, California. This study evaluated the overall hospital complex, constructed in 1967, which is approximately 390 ft by 450 ft in plan and is structurally divided into five buildings separated

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by 1-in. expansion joints. One of these building units includes a 15-story tower structure about 155 ft by 150 ft in plan. The remainder of this building unit and the other building units are 2 or 3 stories in height. This paper is limited to the 15-story tower structure.

The tower structure rises from the northeast corner of a 175-ft by 240-ft low-rise building and extends an additional 12 stories. Typical floor framing consists of 14-1/2 in. deep reinforced concrete waffle slabs supported by reinforced concrete columns typically spaced at 21 ft 8 in. In the center of the tower, reinforced concrete shear walls form part of the elevator cores (Figure 1). These walls are continuous down to the mat foundation. From the standpoint of earthquake resistance, the lateral forces are resisted by a combination of a shear-wall system and a waffle slab-column framing system. The tower structure is relatively symmetrical above the third floor but loses some symmetry in the enlarged floor area from the third floor to the first floor. Below the first floor, the lateral force-resisting system becomes eccentric due to the location of additional shear walls.

STRUCTURAL EVALUATION

Lateral forces were applied by using a response spectrum modal analysis, Method 2, of VA Handbook H-08-8.[2] The design spectrum is defined by:

$$S_a = \alpha \, (DAF) \, A_{max} \tag{1}$$

where:

S_a = spectral acceleration

α = a factor listed in Reference 2, Table 1 (in this case equal to 2/3)

DAF = dynamic amplification factor from Reference 2 (reproduced in Figure 2)

A_{max} = peak horizontal ground acceleration from the site evaluation study (Amendment 1 to Appendix A, Reference 2, in this case, equal to 0.25g)

The tower structure was analyzed with the aid of a structural analysis digital computer program, FRMSTC-4,[3] developed by the University of California and modified by URS/Blume. Mathematical models were created for the two primary directions of the building, including the low-rise portion. Calculated story masses, geometric characteristics, and structural member properties were used as input data. Output data included structural response characteristics such as periods, mode shapes, and participation factors for eight modes of vibration and force distribution to structural members for three modes of vibration.

The procedure of the modal analyses of the structure is summarized as follows. (Results for the north-south direction are shown in Table 1; east-west results are similar.) Period was obtained from the computer analysis, *DAF* from Figure 2, and S_a from Equation (1). The effective modal weight (*EMW*) was defined as the ratio of the equivalent base shear coefficient (*V/W*) to the spectral acceleration (S_a). The roof participation factor

(*RPF*) was defined as the ratio of roof acceleration, a_t, (or displacement, Δ_t), to the spectral acceleration (or displacement). A more detailed example of the procedure can be found in Reference 4.

The roof displacements, relative to the ground, are shown in Table 1. They are based on the computer model, which assumed gross concrete sections. The VA design deflections are based on cracked sections, which are approximately twice the values of those shown in Table 1. For a comparison with the base shear coefficients (V/W), the *Uniform Building Code*[5] requirements for 25 psf wind pressure is equivalent to about a 0.014 base shear coefficient.

The greater of the square root of the sum of the squares (RSS) or the absolute sum of the two modes (TMS) of the base shear coefficients (V/W) represents the lateral force design criterion of the VA requirements.[2,4] When these lateral forces were applied to the structure, the calculated results indicated that the lateral force-resisting system of the structure would be substantially overstressed. Shear and bending stresses in the reinforced concrete shear walls were as high as three times their rated ultimate capacities. As these elements yielded, additional load would be distributed to the waffle slab-column framing system, which would also be stressed beyond its ultimate resistance capacity. It was concluded that the existing structure did not conform to the new VA seismic requirements and should be considered a nonconforming structure that would require major corrective action to meet these requirements.

DEVELOPMENT OF MODIFICATION SCHEMES

Strengthening an existing structure to resist lateral forces can be approached in three general ways: strengthen existing lateral force-resisting elements, add new lateral force-resisting elements, or both. An initial study concluded that the first and last approaches were not feasible due to the nature of the existing structure and the large amount of additional lateral force-resistance capacity required. Therefore, a new lateral force-resisting system would have to be added to the structure. The addition of new structural elements will cause the structure to be more rigid; therefore, the natural periods of vibration will be shorter. Because of the period dependency of the design forces, shortening the period of this structure will increase the required applied lateral forces. Three basic modification schemes were considered, each stiffening the structure to a different degree. The most efficient modification scheme will supply the required additional strength with minimal additional stiffness; however, other considerations are costs and functional application of the modification scheme to minimize disruption of the operation of the hospital.

The first basic modification scheme consisted of providing shear walls on four sides of the building. The height of the reinforced concrete shear walls would vary from bay to bay, ranging from 7 to 15 stories, depending on the demands of the applied lateral forces. On the east and west sides, the walls would be located on the exterior column lines. On the north and south sides of the tower, the shear-wall scheme was complicated by the cantilevered floor slab that extends 9 ft beyond the exterior column line. Three locations for the north and south walls were investigated. One location was on the exterior column lines; this was structurally sound, but it was a poor functional solution because it intersected many hospital rooms, thereby reducing the bed capacity. Another solution to the shear wall scheme was to place the walls at the end of the cantilevered slab. This was more acceptable functionally but less acceptable structurally. The third solution was to place the walls at or near the interior corridor walls. This had some good structural and functional aspects, but it would disrupt the functioning of the hospital during construction. A preliminary analysis of the shear wall schemes indicated a relatively rigid structure (first mode period of 0.65 sec) that would result in the application of relatively large lateral forces.

The second basic modification scheme consisted of providing a lateral force-resistant reinforced concrete framing system at the exterior column lines of the tower structure. Each of the existing reinforced concrete exterior columns would be enlarged from 2 ft square to 4 ft square. Large reinforced concrete spandrel beams, roughly 2 ft by 5 ft, would be constructed to join to the columns. This system had some advantages over the shear-wall scheme. It provided a more flexible lateral force-resisting system, thereby lengthening the period and reducing the applied lateral forces; and it provided a more ductile system, thereby reducing the α-value.[2] The frame system did not have the functional problems of the shear wall scheme, but it had other functional problems. For example, the increased column dimensions interfered with some of the minimum floor-space requirements for each hospital bed. A preliminary analysis of the framing scheme gave a calculated fundamental period of about 0.9 sec and indicated that there would be some problems due to excessive uplift (tension) forces in the corner columns. To reduce the uplift problem of the corner columns, the stiffness of the spandrel beams at the end bays of the frames was reduced to redistribute some forces away from critical corner columns to the less critical center bay columns. In other words, by changing selected beam and column sizes, a more efficient distribution of shear, bending, and axial forces was obtained. The fundamental period lengthened to about 1.1 sec. Although this scheme is good structurally, it still had some poor functional aspects.

The third modification scheme (Figures 3 and 4), the one that was proposed, is a combination of the best features of the shear wall and the framing schemes. It places a rigid reinforced concrete frame around the perimeter of the tower by combining shear walls at the corners, pier-like columns (2 ft by 7 ft) at the present exterior column line locations, and spandrel beams connecting the shear walls and piers. The corner shear walls extend the full height of the tower, and the piers extend to varying heights ranging from the seventh to twelfth floors. This latter scheme appears to be the least disruptive to the operation of the hospital and the most satisfactory in a structural engineering sense.

In the preliminary analysis of the proposed modification scheme, sizes and locations of the framing members were adjusted to develop an efficient lateral force-resisting system. The shear walls take a substantial portion of the shear forces; however, the framing system provides support to reduce the buildup of overturning moments at the base of the shear walls. The calculated fundamental period of the structure is about 0.84 sec. Table 2 summarizes the modal analysis for comparison with the existing structure in Table 1. The modification scheme increases the weight of the structure by about 25%, and this additional weight must be considered with the base shear coefficients when comparing the design base shears. The recommended modification scheme has been presented in its preliminary stages. When final sizes and details are selected and additional architectural considerations are given, the modified structure can supply a ductile lateral force-resisting system with some built-in redundancy and a pleasing architectural appearance.

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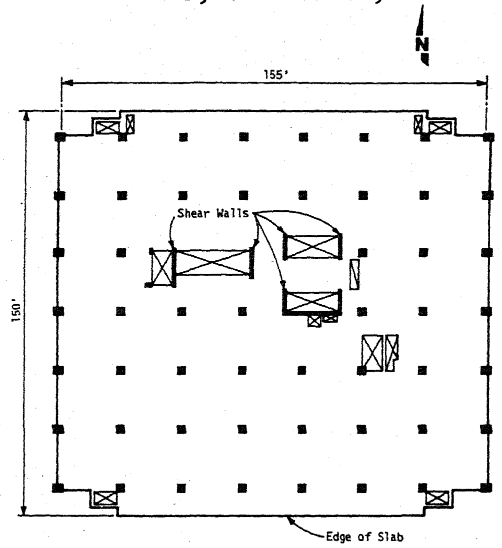

FIGURE 1 TOWER: TYPICAL FLOOR PLAN

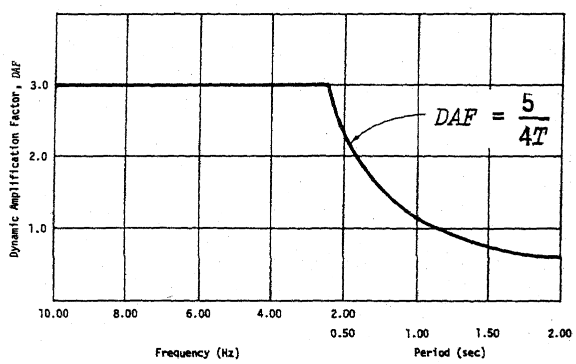

FIGURE 2 VARIATION OF *DAF* WITH FREQUENCY AND PERIOD (from Reference 2)

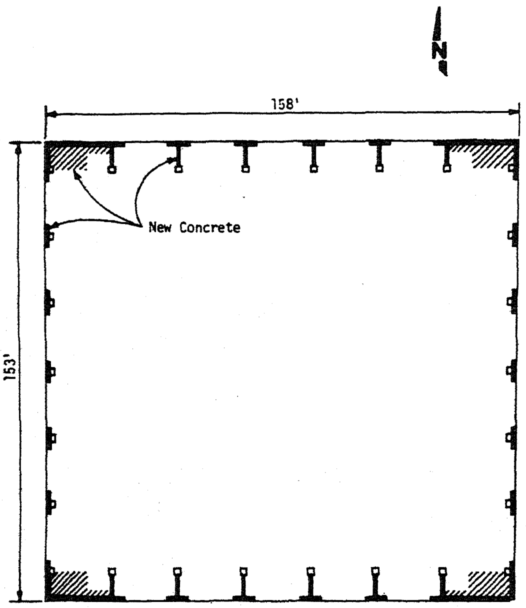

FIGURE 3 PROPOSED MODIFICATION SCHEME, PLAN BELOW SEVENTH FLOOR

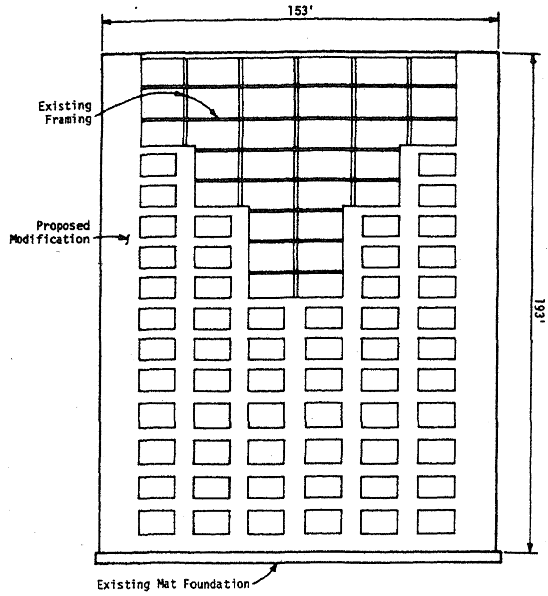

FIGURE 4 PROPOSED MODIFICATION, WEST ELEVATION

Mode	1	2	3	RSS	TMS
Period (sec)	1.91	0.58	0.31		
DAF	0.65	2.16	3.0		
S_a (g)	0.108	0.360	0.500		
EMW	0.59	0.11	0.06		
RPF	1.34	0.55	0.38		
V/W	0.064	0.040	0.030	0.081	0.104
a_t (g)	0.145	0.198	0.190	0.310	0.388
Δ_t	5.17	0.65	0.18	5.21	5.82

TABLE 1 EXISTING STRUCTURE, NORTH-SOUTH DIRECTION (α = 2/3)

Mode	1	2	3	RSS	TMS
Period (sec)	0.84	0.36	0.17		
DAF	1.49	3.00	3.00		
S_a (g)	0.124	0.25	0.25		
EMW	0.60	0.22	0.08		
RPF	1.66	0.89	0.37		
V/W	0.074	0.054	0.020	0.094	0.128
a_t (g)	0.206	0.223	0.093	0.318	0.429
Δ_t	1.42	0.28	0.03	1.45	1.70

TABLE 2 PROPOSED MODIFIED STRUCTURE (α = 1/3)

DISCUSSION

D.B. Naik (India)

The discussor would like to know how the connection between existing columns and walls and new additions of columns and walls was carried out ?

J.D.M. Lloyd (U.K.)

1. Will the author kindly give some indication of the increase in capital cost (in terms of percentage preferably), of strengthening the buildings to meet zone 3 requirements from those for which it was designed assuming these to be zone 1.

2. How does the wind shear requirements compare with those of the earthquake requirements for this building and at what velocity do the shear values cross.

M.K. Aggarwal (India)

What is going to be the effect of modification of existing foundations and what measures are proposed to strengthen the present foundations to take this extra load because of modifications.

Author's Closure

In answer to B.D. Naik's question, the new concrete frame will be keyed into and/or anchored to the existing structure. The new spandrel beam will overlap the existing concrete slab by about 1 inch. Where new concrete is placed against existing concrete, the existing concrete will be roughened by sand-blasting or chipping. Drilled anchors will be used to dowel the new to the existing concrete.

In answer to J.D.M. Lloyd's first question, the estimated cost of the proposed modification is roughly $13 per square foot. Approximately one-half of this cost is structural. The balance is for architectural, mechanical and electrical costs that are required because of the structural modifications. The proposed modification is to satisfy the new VA requirements, not the UBC (Uniform Building Code) Zone 3; however, a parallel example of updating a similar seismic Zone 1 building to seismic Zone 3 (i.e., an increase of 4 times the lateral force requirements) would give about the same cost for strengthening an existing building. It must be noted that if the new seismic criteria had been in force at the time of the original design, the additional costs would have been insignificant or, at most, a small fraction of the above. A little additional reinforcing steel at some critical locations would have increased the seismic capacity of the building substantially. For example, if some of the bottom reinforcing of the beams and waffle slabs had extended beyond the column supports there would have been some reserve strength for reversal of bending moments that would occur when the seismic reaction is greater than gravity load reaction.

In answer to Lloyd's second question, the wind requirements for a UBC wind-pressure-map area of "25", which varies from 20 psf at the base to 40 psf at the top, is equivalent to a base shear coefficient of about 0.014. Adjusting for differences in load factors, this value should be increased to 0.018. The distribution of forces differs between wind and seismic forces so that an exact equivalent can not be made; however, the wind shear requirements are roughly one-quarter of the new seismic force requirement. In other words, the seismic force criteria would be roughly equivalent to a wind-pressure-map area of "100" which is two times the largest pressure zone in UBC.

In answer to M.K. Aggarwal's question, some modification has been anticipated for the existing mat foundation. The mat may have to be extended, possibly by as much as five feet on all sides. A more detailed analysis to determine the actual requirements for the foundations will be conducted during the final design.