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SEISMIC DESIGN OF THE VETERAN'S ADMINISTRATION HOSPITAL
AT LOMA LINDA, CALIFORNIA

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SUMMARY

The 500 bed Veteran's Administration Hospital in Loma Linda, California, now under construction, lies in a region of high seismicity and was designed for the new VA criteria of remaining operational after a major earthquake. Extensive studies were performed to determine site characteristics and, as a result, design lateral forces on the structure were significantly larger than required by conventional codes.

This paper summarizes the design procedure used and describes the resulting structure.

INTRODUCTION

Early in 1973, design began on a 500 bed Veteran's Administration Hospital in Loma Linda, California. The new facility was to replace beds lost due to the San Fernando Earthquake of 1971, to add needed beds in the San Bernardino area, and to serve as a prototype for a new building system developed for the VA. The timing of the project was such that the severe damage to hospitals in the San Fernando Earthquake was still fresh in mind but sufficient time had not elapsed to allow full development of data and conclusions. As a result, the VA required that their new hospitals be designed to "remain operational after a major earthquake", although no code had been developed for their new design philosophy. Since that time both the VA and the State of California have codified design criteria intended to meet that requirement for hospitals in their jurisdiction. (4,6)

The building system to be used in this facility was an open system developed for the VA by the joint venture of Stone, Marraccini & Patterson and Building Systems Development, that primarily organizes and integrates the many complex subsystems required in modern hospitals. (1,2,5,10)
Structural requirements of the system were an intermediate span shallow floor system, large story to story heights, with lateral force resisting elements concentrated in "permanent" mechanical and vertical transportation towers to keep the functional hospital space as open as possible. In terms of planning, the hospital is considered as an assembly of large scale service modules of approximately 10,000 sq. ft. (930 m²), having variable content and organization. These modules have certain common characteristics which permit their assembly into hospitals of widely different size, program, siting and aesthetic treatment. The common characteristics of these service modules include their essential mechanical and electrical independence, interstitial space which separates functional and service activities, and common subsystem characteristics and disciplines.

Varying structural demands had been a requirement of the system and different site and environmental conditions had been considered in its development. However, conditions such as extreme seismicity coupled with changes in seismic design philosophy had not been contemplated. The San Bernardino Valley is seismically very active and the final site selected had 11 known active faults within a 65 mile (105 km) radius including the San Jacinto and two segments of the San Andreas. Fortunately the flexibility of the open system allowed satisfaction of these severe structural requirements within the system framework.

Considering the size, importance, and nature of the project, and the new design philosophy for hospitals as a result of San Fernando, extensive efforts were necessary to assure an appropriate seismic design.

SITE STUDIES

In January 1972 a Site Evaluation Report was done to determine the feasibility of building the proposed facility on two sites in Loma Linda considering the high seismicity in the area. The conclusions of that report, in part, were "the proposed hospital can be so designed and constructed at either of the two potential sites as to remain operational during the occurrence of a major earthquake". However, the trace of the Loma Linda fault was believed to be located near the Southeast corner of the site most desirable to the VA and a considerable setback from the inferred trace was recommended. Further recommendations included use of a structural system consisting of a ductile moment resisting space frame and properly designed shear walls, a design force level approximately twice that of the Uniform Building Code, and a comprehensive investigation of the site finally selected to develop further data for design.

The final Site Seismic Hazard Report on the selected site was completed in January 1973. Significant conclusions of that report included the following:

1 - Based on extensive trenching, the surface trace of the Loma Linda fault does not pass through the site. The report said, "Conclusive evidence indicated that no surface faults exist on the Loma Linda site; therefore, the probability of surface faulting through the site is essentially zero". The most likely location of the Loma Linda fault was said to be 200 to 400 feet (61 m to 122 m) Southwest of the site and no specific setback dimensions were recommended.

2 - Although some ten fault zones within 65 miles of the site were studied, the San Andreas and the San Jacinto were identified as being most significant in terms of site ground shaking.

3 - Because of a deep groundwater table, there was no liquefaction potential at the site.

4 - Preliminary ground response analysis indicated acceleration amplification factors of 0.9 to 1.1 from rock-like material to the ground surface. Surface accelerations were estimated to be 0.6 to 0.7g (g = gravitational acceleration) from large events on the San Jacinto and 0.5 to 0.55g from large events on the San Andreas.

To determine response spectra for design, it was necessary to determine the appropriate size and source of specific events to be studied, and the characteristics of those events. The following considerations were discussed for this determination:

1 - The VA criteria that the hospital should be operational after a major earthquake indicated that the events studied in detail should be large.

2 - Maximum credible events postulated in the Site Seismic Hazard Report had recurrence intervals of 100 to 300 years; therefore, the events to be studied should be slightly smaller than these maximums.

3 - Earthquakes emanating from the San Andreas and San Jacinto faults undoubtedly would produce the most intense ground motions. Because of the different distances from the sites, the predominant frequencies produced by these sources would be slightly different and, in fact, would probably bracket the range of periods of structures proposed for the site.

It was concluded that the effect on the proposed structures of events with the characteristics as shown in TABLE 1 should be studied in detail to determine appropriate design parameters. Also listed in TABLE 1 are time histories used to represent the chosen events in the subsequent Site Response Studies and response spectra development.

<table>
<thead>
<tr>
<th>SOURCE</th>
<th>DISTANCE FROM SITE</th>
<th>MAGNITUDE</th>
<th>MAXIMUM ACCELERATION</th>
<th>AVERAGE ACCELERATION</th>
<th>TIME HISTORIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Andreas (SAL)</td>
<td>7 mi.</td>
<td>8+</td>
<td>0.53 g</td>
<td>0.59 g</td>
<td>Synth 8+(CAL) A-1 (Caltech)</td>
</tr>
<tr>
<td>San Jacinto (621)</td>
<td>1½ mi.</td>
<td>7-7½</td>
<td>0.65 g</td>
<td>0.56 g</td>
<td>Lake Hughes Taft</td>
</tr>
</tbody>
</table>

TABLE 1 - DESIGN EARTHQUAKE CHARACTERISTICS

The time histories were projected through the site soils using varying depths to rock and varying soil properties by procedures developed by Seed and Idriess at the University of California. The average of the
maximum calculated surface accelerations are also listed above. Response Spectra were calculated at the surface for structural damping of 5% and 10% of critical. The smooth spectra shown in FIGURE 1 represent the upper average of the computed spectra for events SAI and S71. These smoothed spectra were used in the structural pseudo-dynamic analysis to determine their effects on the proposed structures. Also shown in these figures are the 1959 Houssner generalized spectra and the 1969 Newark Hall suggested generalized spectra.

**DESIGN PHILOSOPHY**

The predominant feature of the Loma Linda lateral design philosophy is the phrase, "operational after a major earthquake." Early discussions revealed difficulty in specifically identifying a design level consistent with this criteria. However, in general terms, it was agreed that structural drift should be kept small to prevent damage to contents and that ductility requirements should be kept low to minimize structural damage.

A dual lateral force resisting system of concrete shear walls and a ductile moment resisting "back up" frame was chosen for the following reasons:

1 - Concrete shear walls as primary elements provide lateral deflection control essential to minimizing non-structural damage and internal disruption.

2 - At high lateral force levels it would be difficult, if not impossible, to prevent the vertical-load-carrying frame from taking some part in lateral resistance. This participation suggests a need for ductility considering the uncertainties that must be dealt with in earthquake resistant design. The ductility of steel makes its use logical.

3 - In the event of the unpredictable truly cataclysmic earthquake the participation of the frame becomes more significant and its use is mandatory to insure structural stability.

4 - The cost of this system is competitive with alternatives and in most cases proves most economical.

Therefore, the stiff primary shear wall system would be designed for a high force level that is capable of withstanding at low lateral deflections the design events previously described. The back up frame will be designed to carry all vertical loads, including the shear walls, and to maintain stability laterally for some smaller force level. The reduced force level on the frame has been embodied in the Structural Engineers Association of California Blue Book(6) for some time and is based upon the following rationale:

1 - The shear walls will provide large damping and energy absorption for the frame.

2 - The frame will have a much larger period than the primary system.

3 - The most intense shaking will probably be over by the time the frame is forced to act.

The chances of the back up frame being forced to work to its full capacity were small, but considering the extent and importance of the facility, they could not be ignored. The uncertainties in earthquake characteristic prediction and construction quality makes mandatory the use of a frame capable of redistributing forces and preventing collapse.

**CONFIGURATION STUDIES**

It has long been acknowledged that the configuration, and the simplicity and directness of the seismic resistant system of a structure is just as important, if not more important, than the actual lateral design forces. For this reason, it is necessary for the Structural Engineer to work closely with the Project Architect in early configuration studies. The architect, therefore, balanced VA program requirements, project cost, and inherent seismic resistance in his configuration determination.

For planning and aesthetic reasons a low building was required, so it was decided to point toward as low and stiff a building as possible to minimize drifts and possibly lower response from the projected response spectra peaks (Period = 0.3 sec. for S71 and Period = 0.8 sec. for SAI).

Solutions using multiple and single buildings of four and five stories, with full basement, half basement and no basement combinations were studied for optimization of the above three parameters. Symmetry, shear wall availability, separation joint requirements, and continuity of vertical stiffnesses were considered in evaluating seismic resistance.

All solutions studied utilizing basements produced vertical stiffness discontinuity at the first level. Multi-building solutions were difficult to make symmetrical and all required several separation joints. It was finally decided that a single block configuration, almost square in plan, with no basement, provided the best characteristics considering the generalized three parameters studied.

After the shape was determined, various combinations of shear wall locations were tested for sufficiency in extent, symmetry and distribution. Because of the large plan area, it was necessary to balance shear wall rigidities throughout the plan to maintain low diaphragm stresses.

The final configuration is shown in FIGURES 2 and 3. The structure is four floors and is built up of service modules as defined by the building system previously discussed. The plan is essentially symmetrical, has an even distribution of shear walls throughout and, because of the regular framing layout, has direct distribution members to all walls.

**DESIGN FORCE DETERMINATION**

Given a response spectrum shape (normalized to surface acceleration equal to 1), in our case tailored to the site by a Site Response Study, there are still many variable parameters that must be studied and estimated before actual design lateral forces can be obtained. A compilation of these parameters is listed below as they are used. In parentheses are shown the values studied for this project.
1 - Approximate the structural damping of the proposed structure and adjust the spectrum accordingly.

(5%, 10%)

2 - Proportion the response spectrum shape to the estimated maximum ground surface acceleration.

(SI - 0.55g to 0.65g; SA1 - 0.6g to 0.65g)

3 - Linearly reduce the spectrum to take into account the following effects that are difficult to account for numerically in dynamic analysis of the structure using modal analysis:

a. Reduction of high singular acceleration spikes.

b. High repetitive accelerations producing small displacement responses not causing damage proportional to the computed forces.

c. Duration effects.

d. Out of phase input motions due to large building size.

It has been suggested that this reduction could be as high as 50 percent if all effects were present.

(0.7 to 1.0)

4 - Assign a ductility to the structure for determination of yield level forces from the adjusted spectrum. The ductility can also be calculated as a demand, after a force level has been set and the structure designed.

(2 to 3)

5 - The analysis and design of the structure is then dependent upon the determination of the periods of the building; or the modeling assumptions made for both the structure and its base if a computer is used for calculation of all mode shapes and periods.

(T = 0.2 to 0.3 sec.)

6 - Combine the modal responses for a total response.

(Root mean square, Sum of Modes 1 and 2)

All of the above steps were performed and the effects of varying all of the listed parameters were studied. For the configuration proposed for Loma Linda, the SI spectrum was used because its peak (0.25 to 0.35 sec.) approximately coincides with the building period. It was not considered reasonable to assume that the building period could lengthen to the peak of SA1 spectrum (0.8 sec.) especially considering the spectral value of SA1 in the critical period range is approximately 60% of SI1.

Based upon the above described studies, particularly the uncertainties involved, it was decided to set the base shear for the structure at 0.5 W (W = Building Mass), design elements at yield level, and calculate an approximate ductility demand under various conditions.

The peak of the SI1 response spectrum was set at the first mode period in the following calculations, thereby eliminating the period as a variable. An infinite array of ductility demands still can be generated by varying other parameters, but the most reasonable combinations are listed in Table 2.

<table>
<thead>
<tr>
<th>STRUCTURAL DAMPING</th>
<th>INPUT ACCELERATION</th>
<th>RESPONSE SPECTRUM REDUCTION</th>
<th>CALCULATED DUCTILITY DEMAND</th>
</tr>
</thead>
<tbody>
<tr>
<td>10%</td>
<td>0.55</td>
<td>0.7</td>
<td>1.3</td>
</tr>
<tr>
<td>5%</td>
<td>0.55</td>
<td>0.7</td>
<td>1.7</td>
</tr>
<tr>
<td>5%</td>
<td>0.65</td>
<td>0.7</td>
<td>2.0</td>
</tr>
<tr>
<td>5%</td>
<td>0.60</td>
<td>0.8</td>
<td>2.1</td>
</tr>
<tr>
<td>10%</td>
<td>0.65</td>
<td>1.0</td>
<td>2.2</td>
</tr>
<tr>
<td>5%</td>
<td>0.55</td>
<td>1.0</td>
<td>2.4</td>
</tr>
<tr>
<td>5%</td>
<td>0.65</td>
<td>1.0</td>
<td>2.9</td>
</tr>
</tbody>
</table>

*Considered most probable combination

Since the ductilities calculated are well within the accepted range for properly reinforced shear walls with steel column edge members, especially with the stress and reinforcing criteria that were used, and since shear wall design is practical both economically and architecturally at this level, 0.5W was set as the design lateral force. Actual member forces were determined by normalizing the full spectrum RMS computer solution to 0.5W base shear.

Using this criteria for shear wall design, the calculated maximum story drift-to-story height ratios under full spectral input loadings was approximately 0.004, well within presently accepted standards for hospitals.

**DESIGN**

Basic framing for the structure can be seen in Figure 4. Three steel girders, placed at 40'-6" o.c. (12.35 m) run longitudinally spanning the 22'-6" (6.86 m) column spacing. Eight such bays make up the service module which is the structure's basic building block. Steel beams, composite with the steel deck and concrete floor, span transversely at 11'-3" o.c. (3.43 m). The 22'-6" x 40'-6" column spacing is consistent throughout except for 54' (16.47 m) inside bays at the courts. Shear
walls are always placed at the perimeter of service modules to minimize planning interferences. Interior girders are dropped below the beams to minimize interference with services running between the beams; this also allows beam continuity across the module. All of the above framing characteristics are features of the systems integration for which Loma Linda is the prototype.

The design of the structure can be split into three basic areas: the steel moment resisting space frame, the concrete shear walls, and the foundations. Each will be discussed separately.

**STEEL FRAME**

The steel frame was designed for a lateral force of 0.05% or 10% of the total design lateral force. This is less than the usual 25% used for back up frames, but it was felt that this large lateral design force was necessary for the back up frame to perform its function. A force of 0.05% is still more than three times that required by the Uniform Building Code for a back up frame and about equal to the total lateral force required if the frame was the structure's primary lateral system. Final design of elements was based upon the greater values obtained considering the frame acting in conjunction with the shear walls for 0.5% or the frame acting alone for 0.05%. Lateral drift was not considered critical in the design of the frame and yield level stresses were allowed in conjunction with lateral forces.

Since the force level in the frame was of a conventional order of magnitude, few special problems were encountered. The large collector forces created by the overall 0.5% level increased the steel weight of the members affected 10 to 20 percent. The criteria that the frame vertically support the weight of the shear walls required addition of intermediate columns between shear wall columns greater than 22'-6" apart. The final frame steel weight was 16.5 lb/sq. ft. (80 kg/m²).

**SHEAR WALLS**

Although shear walls in the project were used in lengths of 40'-6", 45', 54', and 91' (12.35, 13.72, 16.47, 24.7 m), with a variety of penetration configurations for architectural use (functional zone) and mechanical use (service zone), a typical wall is represented in FIGURE 5. The exclusive use of "infill" walls that simply enclose portions of the overall steel framing pattern has several advantages:

1. There are always beams or girders parallel and on line with the walls to serve as lateral force collectors.
2. The continuation of these members through the wall allows direct transfer of forces from the diaphragm to the wall.
3. The columns at the end of walls form the required ductile flange members for wall bending.
4. Frame members are in correct position to provide vertical support for shear wall dead load.

Thickness of walls varied from a minimum of 12" (30.5 cm) to a maximum of 24" (71 cm). Minimum reinforcing to concrete ratio was 0.25% in each direction. Heaviest wall steel used was 1.2 in/ft (2540 lb/ft²) each way on each face of a 24" wall.

The predominantly first period response of the stiff shear walls coupled with the high force level created large bending moments at the base of the shear walls. Net tension forces in the wall flanges were as high as 3,900,000 lb. (16,910 KN). This required use of a concrete "tension block" to transfer these forces to the foundation system (see FIGURE 6). The tension block is cast over the steel column base plate and the tensile forces are transferred to the concrete through upward bearing. The block is anchored to a heavily reinforced pier cap which spans between the cast-in-place reinforced concrete pier foundations.

The steel column flanges of the walls are made composite with the concrete by shear friction, utilizing welded transfer dowels (FIGURE 7). The floor to wall and beam to wall connection is shown in FIGURE 8. Shear is transferred through the beam from the wall above to the wall below (or from the beam to the wall in the case of beam collectors) also by shear friction but using dowels placed through holes pre-punched in the beam flanges. The lightweight floor concrete, weaker in shear than the hardrock wall concrete, is not cast through the wall. The steel decking was temporarily supported independently of the main beams at the wall to further prevent a shear weakness at the floors and to allow the contractor freedom to cast slabs and walls independently. The use of "infill" walls in fact, used this advantage by casting slabs continuously and letting the more complicated wall construction progress at its own rate.

**FOUNDATIONS**

The use of drilled cast in place concrete friction piers was determined by an early economic analysis. Technically, their characteristics of small settlement and high uplift resistance were mandatory.

As with the shear wall design, axial overturning forces at walls required careful consideration of the foundation response to high force levels. The piers were designed to transfer 90% of the calculated wall overturning moment into the soil. The 10% reduction in calculated forces was arbitrary and probably should have been larger. It stems from the fact that although the shear wall flexure exists, the actual tendency to overturn does not, especially in a short period building. The rapid reversals in direction do not allow time for significant overturning action to take place. Field observation of earthquake damage indicates that designing foundations for full overturning is overly conservative. It was decided that small vertical slips between pier and soil at responses over 0.5% was acceptable as long as vertical load carrying capacity was maintained by insureing pier ductility.

The treatment of lateral forces at the foundations was a much more difficult problem. Coupling the structure to the soil for the structural
design level of 0.5W was a consistent design approach, but the interaction between soil and foundations at higher force levels had to be investigated. Plastic hinging of the piers at the top and at the point of centreoflexure, as well as soil yielding was involved and the order in which these occurred was critical.

Initially it was determined that the piers under walls placed for vertical load and overturning were totally insufficient to transfer the distributed lateral forces, wall by wall, into the ground. In some walls, it would have been impossible from a physical standpoint to provide the required capacity without reverting to battered piers. Battered piers were undesirable unless they were used throughout because of their incompatibility with the rest of the building, and battered piers throughout were uneconomical and would provide no allowance for desirable minor movements and energy absorption. The greatest concern was to prevent different responses of individual walls due to different soil coupling stiffnesses. Such action would negate any dynamic analysis which assumes a single response to a single input and would have severely damaging effects on the structure.

For these reasons, it was decided to make all piers, including the piers away from shear walls, capable of resisting lateral forces, and tie the building together at the ground level sufficiently to force unified action of the building base. Although foundation tie beams between individual pier locations were already contemplated, it was necessary to have diaphragm action in the slab on grade to distribute the concentrated loads at walls throughout the building. In order to then properly design this system for overall ductility, it was necessary to determine pier stiffness under a wide variety of conditions. The most difficult portion of overall pier stiffness to determine was the basic pier-soil interaction, but varying top fixity conditions as well as group action also had to be considered. Exhaustive computer studies were done to determine the effects of these parameters and pier rigidities were calculated. A lateral load of 0.5W was distributed to the piers according to these rigidities and bending moments calculated at that load level. It was discovered that the rotation of the top of piers at free-standing columns was causing certain columns to hinge between the first and second floor. Tie beams, originally designed for axial loads only, were increased in size and properly reinforced so that the combined stiffness of column and tie beams always exceeded the pier stiffness, thus forcing the hinge into the pier. The piers were designed at the 0.5W moment at the point of first yield, which, as discussed above, always occurred at the pier-cap intersection. However, the pier could continue to take additional load until a second hinge formed at the point of centeroflexure or until the soil itself yielded. To insure ductility, shear reinforcing was provided on a pier by pier basis for this limiting condition. This design procedure will insure consistent elastic response up to 0.5W load and will prevent loss of vertical load carrying capacity at greater loads caused by sudden shear failure in the piers or column hinging.

**COSTS**

The additional cost of the design at Loma Linda is difficult to estimate without a total parallel code design because such a large increase in design parameters changes the scope of the lateral systems. However, costs of designs at other force levels were estimated in connection with final establishment of the design forces used. By extension of this information, the difference in construction cost between Loma Linda and a conventional code (1973 UBC) design can be estimated. These additional costs are summarized in Table 3.

<table>
<thead>
<tr>
<th>ELEMENT</th>
<th>ADDITIONAL STRUCTURAL COST OVER CONVENTIONAL DESIGN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Walls</td>
<td>$800,000 - $1,200,000</td>
</tr>
<tr>
<td>Steel Frame</td>
<td>300,000 - 400,000</td>
</tr>
<tr>
<td>Foundations</td>
<td>350,000 - 400,000</td>
</tr>
<tr>
<td>Slab on Grade</td>
<td>100,000 - 200,000</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>$1,550,000 - $2,350,000</td>
</tr>
</tbody>
</table>

Percentage increase of structural cost ($15,850,000) - 10-15%
Percentage increase of total cost ($55,100,000) - 2.5-4.3%

**Table 3 - Increase in Construction Costs**

The costs are consistent with those reported by the Applied Technology Council in a recent report (3) done to investigate the problems of general use of the response spectrum approach in design.

Additional cost and general information, along with project acknowledgements, are contained in Table 4.
OWNER: Veteran’s Administration, Washington, D.C.
LOCATION: Loma Linda, CA, approximately 60 miles east of Los Angeles
AREA: 724,063 square feet

BASIS OF DESIGN: Development Study, VA Hospital System by SMB/BSD, a Joint Venture, for the VA Office of Construction Research Staff

DESCRIPTION: Four floors made up of 15,000 S.F., planning modules each mechanically independent. All services distributed through interstitial spaces between floors. Exterior of concrete and stucco. Structure consists of a steel frame with 40"-6" span steel composite beams. Basic lateral system is concrete shear walls. Interstitial spaces formed with hung steel purlins and poured gypsum concrete.


CONTRACTORS: PHASE 1 - Structural and Site Grading:
Robert McKee, Inc.

PHASE 2 - Architectural, Mechanical, Landscape:
J.W. Bateson Co., Inc.

APPROXIMATE COSTS:

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost</th>
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</thead>
<tbody>
<tr>
<td>Site and Landscape</td>
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</tr>
<tr>
<td>Structural</td>
<td>15,850,000 ($21.90 psf)</td>
</tr>
<tr>
<td>Architectural, Transport.</td>
<td></td>
</tr>
<tr>
<td>Equipment</td>
<td>25,900,000 ($25.90 psf)</td>
</tr>
<tr>
<td>HVAC</td>
<td>6,500,000 ($9.00 psf)</td>
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<tr>
<td>Plumbing</td>
<td>6,500,000 ($9.00 psf)</td>
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<tr>
<td>Electrical</td>
<td>4,500,000 ($6.20 psf)</td>
</tr>
<tr>
<td></td>
<td>$55,100,000</td>
</tr>
</tbody>
</table>

DESIGN TEAM:

Architect: Stone, Harragorni & Patterson and Building Systems Development, A Joint Venture
Site Selection Analysis: LeRoy Crandall and Associates

Site Seismic Hazard, Site Response and Soil Reports: Woodward Lundgren and Associates

Structural Engineers: Rutherford & Chekene
(Consultants: Henry J. Degenkolb, H. Bolton Seed, Ray Clough, Ed Wilson, and Ken Lennert)

Civil Engineers: Rutherford & Chekene

Mechanical Engineers: Ayres & Hayakawa

Electrical Engineers: Cohen, Lebovich and Pascoe

Landscape Architect: Arutunian/Kinney Associates

TABLE 4 - LOMA LINDA HOSPITAL
PROJECT SUMMARY AND ACKNOWLEDGMENTS

NOTE: SPECTRA FOR SA-1 AND SJ-1 REPRESENT UPPER AVERAGES OF THE COMPUTED SPECTRA
N = NEWMARK HALL (1969), H = HOUSNER (1959)

FIGURE 1 - SITE RESPONSE SPECTRA
FIGURE 4 - TYPICAL SERVICE MODULE FRAMING

FIGURE 5 - TYPICAL SHEAR WALL

FIGURE 6 - TYPICAL TENSION BLOCK
FIGURE 7 - TYPICAL COLUMNS AT SHEAR WALL

FIGURE 8 - TYPICAL FLOOR/SHEAR WALL DETAIL

BIBLIOGRAPHY


4 - California, State of, California Administrative Code, Title 17 Public Health, available from State Office of Procurement, Publications Section, P.O. Box 20191, Sacramento, CA 95820.


