SEISMIC UPGRADE OF BUILDING 311

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SUMMARY

Building 311, a two-story, reinforced concrete structure, was originally designed and constructed per the earthquake requirements contained in the 1961 Uniform Building Code. A significant difference exists between today's aseismic building codes for new construction and the 1961 Uniform Building Code. Presented is a unique structural/architectural building upgrade involving external building buttresses, drilled piers, and sophisticated connection details for the earthquake protection of building occupants. Discussions will center on concept evaluations, structural member configurations, static analysis - checked and evaluated by the use of dynamic analysis, and connection details developed through state of the art design and analysis information.

INTRODUCTION

Originally designed under the requirements of the 1961 Uniform Building Code (UBC) and constructed during 1964, Building 311 suffered considerable structural damage -- vertical column splitting and/or concrete cracking at the beam/column joints -- during the January 1980 Livermore earthquake (Richter Magnitude 5.5 - 5.8). Peak ground acceleration at the building site was estimated to be in the order of 0.25g. Following the earthquake, as part of a remedial repair effort, most of the concrete cracks were repaired by injecting high-strength epoxy grout into the cracks.

BUILDING DESCRIPTION

Building 311 was originally constructed as a two-story, cast-in-place reinforced concrete (3000 psi), non-ductile moment frame structure with 37,000 ft.^{2} of gross floor space. It is approximately 95'(N-S, Transverse Direction) x 190'(E-W, Longitudinal Direction) in plan and consists of two levels plus a penthouse (Plate 1). The typical bay size is 23'-9" square. Its primary lateral force resisting system is a "moment resisting frame" founded on drilled piers. The first floor is a slab-on-grade, and the second floor and roof are framed with concrete beams, girders, and infilled with concrete pan joists. Building columns are of reinforced concrete. There are no column ties within the building columns other than in the immediate vicinity of the ground, second floor, and roof slabs. The non-ductile frames were designed using a uniform lateral force distribution (1961 UBC) and a base shear coefficient of 0.067. This type of framing system has a history of

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partial or total collapse in moderate to strong earthquake shocks, i.e. Helena, Montana - 1930, Caracas, Venezuela -1967, San Fernando, California - 1971, Managua, Nicaragua - 1972, etc. The 1961 UBC lateral force requirements were one of the first sets of aseismic design provisions which recognized the essential dynamic effects of structural response during earthquakes. One should note that a significant difference exists between today's current aseismic building codes for new construction and the 1961 Uniform Building Code. The 1979 and 1982 editions of the UBC would require Building 311 to resist a non-uniformly applied static earthquake force with a base shear coefficient of 0.094.

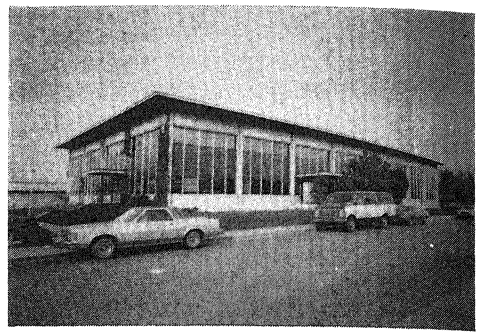

Plate 1. Building 311 - as originally constructed.

STRUCTURAL UPGRADE CRITERIA

Structural Design Criteria was prepared for upgrading the seismic performance of Building 311. Consideration was given with respect to present-day analysis and design procedures, along with the seismic potential of the Livermore Valley and the surrounding areas.

- The basis of design and allowable unit stresses for the various building structural elements shall be in accordance with the latest edition of the Uniform Building Code and ERDA Appendix 6301, Facilities General Design Criteria, Part 1, Section C.

- Present 1979 UBC Requirements upgraded for a total building lateral seismic force of 0.25W (static). Lateral forces on elements of structures and nonstructural components shall be upgraded proportionally.

- Connection evaluation and design, due to seismic forces, shall account for an additional load factor of 1.5 (connection design/evaluation loads = 1.5 x 0.25W loads). This will help to assure that the building's structural elements can reach their maximum potential for ductility.

The upgraded building was then checked using site specific ground response spectra for its ability to survive without sustaining collapse during an earthquake with a 0.50g peak ground acceleration.

STRUCTURAL UPGRADE SOLUTION

Many different alternate schemes for upgrading the building structure were considered: various arrangements of additional concrete shear walls within the confines of the existing building framing system, addition of diagonally-braced framing elements, and the use of exterior bracing

elements, both steel and concrete. Selection of the best candidate scheme was based on many factors: cost, structural independence from the existing structure, disruption to existing occupants, disruption to existing mechanical and electrical systems, aesthetics, impact on living space, and anticipated construction time. The best candidate scheme for the seismic upgrade of any building is that scheme chosen for the uniqueness of the particular project and the particular building.

The structural upgrade solution selected constructs and attaches to the existing building a series of external two-story-high concrete shear walls (buttresses) on three of the four sides of Building 311, two on the west end, two on the east end, four on the south side, and none on the north side (Plate 2). The concrete buttresses are anchored to the ground using 3'-diameter x 40'-long drilled concrete piers, four piers per buttress. The concrete buttresses were designed to be attached to the building's second floor level by the use of rebar embedded in the buttress and then welded to A-36 steel C10 x 20 channel drag struts, which are, in turn, attached to both sides of a second floor concrete beam by a series of specially-designed steel bolts. There are two drag struts per buttress at the second floor level. Each concrete buttress was also designed to be attached to the concrete roof slab by the use of rebar embedded in the buttress and then welded to an A-572 12" x 1" steel plate drag strut, which is, in turn, attached to the top of the concrete roof beams by shear connectors.

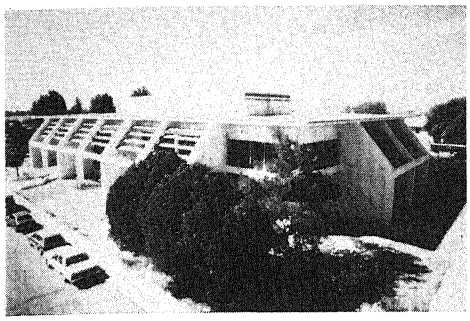

Plate 2. Building 311 - structural upgrade.

The upgrade design for the drag strut connections at the second floor use a straight pattern of 10 (Custom-made) A-36 steel bolts with an approximate ultimate tensile strength of 73,000 psi (Plate 3) spaced approximately nine inches apart on center at each buttress. These bolts have a length of 16" with an overall diameter of 3-3/4" for that portion of the bolt passing through the 16"-wide concrete beam. The bolts are necked down at each end to a 1-1/4" diameter threaded portion for bolting to the steel drag struts. In order to insure a tight fit after placement, the 3-3/4" diameter bolts were epoxy-grouted into their respective concrete beam bolt holes. Drag strut connections at the roof use a straight pattern of 13 specially-made shear connectors (Plate 4) spaced approximately seven inches on center at each buttress. The shear connectors are basically a 10"-long by 5/16" steel angle with 5" and 1-3/4" legs. Welded to the back of

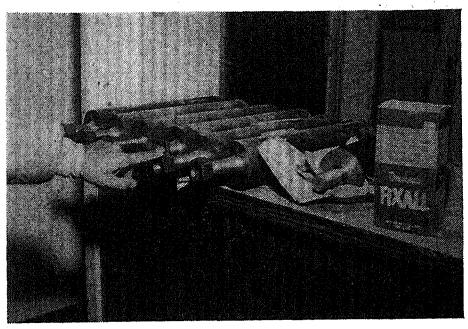

Plate 3. Second floor drag anchors.

the 5"-long leg are two 7-1/2" x 1-1/2" diameter A-307 Type B bolts used for attaching the shear connectors to the steel plate drag struts. The shear connectors are completely embedded in a trench located in the top of the roof beams and the trench is then filled with high-strength epoxy grout (Plate 5).

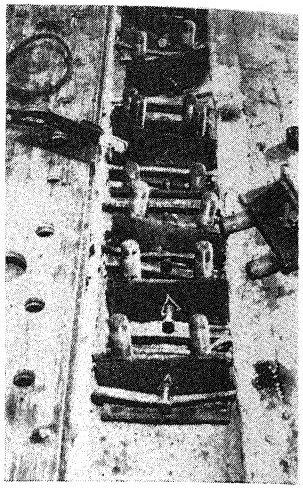

Plate 4. Roof drag strut connections.

The stiffness characteristics of the concrete buttresses are such that they are many times stiffer than the existing concrete moment frame of the building and, thus, the concrete buttresses become an extension of the ground. The earthquake ground motion is, therefore, transmitted to Building 311 by the external concrete buttresses "pushing" or "pulling" on the building, and not by the originally intended bending action of the building columns. The external concrete buttresses push directly against the concrete building frame and pull against the steel drag strut members attached to the building. The "pushing" and "pulling" of the buttresses produce earthquake loads which are distributed to and reacted by the inertia mass of the building. Introducing the concrete buttresses changed the original, predominate building frequencies from 1.2 HTZ and 4.5 HTZ to 5.3 HTZ and 9.2 HTZ. The building columns will see very little of the horizontal seismic forces. The stiffness (load/deflection characteristics) of the buttress/building attachment mechanisms (pulling or pushing) will control both the magnitude of seismic forces acting upon and distributed among the buttresses by the building. The corrective design also provides high load capacity drag strut connections in close proximity to the buttress locations with strength capacities commensurate with the non-uniform load distribution across the second floor drag strut bolts and the roof shear connectors.

Plate 5. Roof shear connector trench

DESIGN ANALYSIS -- BUTTRESS/BUILDING CONNECTIONS

Since the new concrete shear walls (buttresses) were placed externally to the building and not within the confines of the building, load transfer between the drag struts (collectors) and their respective concrete anchors imbedded in the structure is not uniform along the pattern of anchors. The load transfer distribution for the anchors is a higher order mathematical function, which reduces quite quickly in magnitude as one moves in succession away from the first anchor nearest to the buttress. The first drag strut connecting anchor adjacent to a buttress must resist seismic forces several times larger than the last anchor furthest from a buttress. An assumed

uniform reaction of seismic forces by a series of anchors is only accurate when the various connecting parts of the drag strut or collector are rigid when compared to the anchor stiffness. Seismic forces will distribute among the buttresses in proportion to their buttress/building connection rigidities, pulling on the steel drag strut members vs. pushing directly against the concrete building frame. Figure 1 displays a series of "Load-Deformation Curves" for lateral loads only applied to single 5/8", 3/4", 7/8", 1", and 1-1/4" diameter Ramset trubolt wedge anchors installed with an embedment depth of 4-1/2 diameters in stone aggregate concrete having a 28-day compressive strength of 3155 psi, 3448 psi, 3699 psi, and 3698 psi, respectively (Ref. 1).

A series of computer models, comprised of possible typical buttress/drag strut connection details were assembled: buttress connecting rebar attached to drag strut channels, which are, in turn, attached to a typical concrete floor beam by 1-1/4" (24 ft. drag strut only) and 5/8" diameter Ramset trubolt wedge anchors on a center spacing of 18 inches for a drag strut length of 24 ft., 48 ft., and 72 ft. -- one, two,and three building bays, respectively. Wedge anchor stiffness was determined from Figure 1. Figure 2 displays the resulting load transfer between the steel drag strut and the concrete anchors spaced along the concrete beam as a percentage of the total applied drag strut force P. It can be seen that the length of the drag strut or the number of concrete anchors has little effect on the magnitude of load required to be resisted by the first few anchors in the series. Based on the 0.25W modified UBC elastic design criteria, the static design force applied to a typical east-west second floor drag strut, if one assumes a 43% Tension and a 57% Compression force distribution between buttresses, is equal to 56 kips. From Figure 2, the seismic design force with the 1.5 connection factor acting on the first concrete wedge anchor is approximately 14.3 kips (17% x P x 1.5) and 12.2 kips (14.5% x P x 1.5) for a 1-bay series of 1-1/4" and 5/8" anchors, respectively. The design allowable for the 1-1/4" anchor with a 10" embedment depth is 13.3 kips (embedment depth of 5-1/2" is 9.8 kips), and for the 5/8" anchor it is 2.8 kips, based on 4000 psi concrete (Ref. 2). In the north-south direction, seismic forces are distributed <u>equally</u> among the buttresses. It was therefore determined that normal concrete wedge anchors could not be used as part of the connection details. Concrete wedge anchors are required to have a minimum safety factor of 4 against failure when used with UBC or manufacture guidelines.

As a result of an exhaustive, iterative design/analysis approach, the shear anchor designs previously described under "STRUCTURAL UPGRADE SOLUTION" were selected for the second floor and roof. Design variables considered during the selection process included connector strength at elastic and inelastic levels, connector stiffness, concrete bearing/tensile/shear/compressive stress levels, overall total connector system stiffness (tension stiffness vs. compression stiffness) for distributing equal seismic forces among buttresses, etc. Finite element models of a single roof connector (Figure 3), a single second floor connector, and a point load acting in compression on the edge of the second floor and roof concrete slabs were used to determine the elastic stiffness rates of these individual structural elements when imbedded in concrete. Computer models were then made for determining the overall stiffness of the specially made assembled connector system, 10 bolts at the second floor level and 13 shear connectors at the roof level (Figure 4) for each buttress, along with their respective drag strut

elements. The resulting load distribution along the individual second floor and roof connectors in the east-west direction of the building is plotted as a percentage of the total applied drag strut force P as shown in Figures 6 and 7.

The overall elastic stiffness rates of these assembled drag strut/connectors were determined. The computed spring rates were then introduced as structural spring elements used to attach the concrete buttresses to the building frame as part of a totally integrated finite element spring/mass model for the entire building and new buttresses (Figure 5). The dynamic properties of the upgraded building, the earthquake forces at 0.5g PGA with 10% damping, and the respective distribution of seismic forces -- static as well as dynamic -- between the concrete buttresses and the various building elements were then determined. It was found that the seismic forces distributed almost equally between the buttresses (43% tension and 57% comprsession) in the E-W direction. The almost equal seismic force distribution between the E-W buttresses was due, in fact, to the care exercised in trying to achieve a drag strut/connector design with tension spring rates approximately equal to the compression spring rates of the buttresses pushing directly against the edge of the building's second floor and roof concrete slabs. The drag strut forces in the east-west direction are equal to 56 kips/171 kips (0.25W/0.50g) and 241 kips/855 kips (0.25W/0.50g) for the second floor and roof, respectively.

Once the earthquake force distributions were determined within the over-all building model, one has only to return to the local drag strut/connector models (Figures 6 and 7) for determining the approximate seismic forces acting on the individual concrete connectors. Seismic design forces in the east-west direction acting on the first second floor bolt (single shear) and the first roof shear connector (2 bolts) are 26 kips/52 kips (0.25W/0.50g) and 65 kips/155 kips (0.25W/0.50g), respectively. The seismic forces for the first second floor bolt and the first roof shear connector for the 0.25W design case includes the 1.5 connection factor. The seismic load capacities (no threads in the shear plane) for the 1-1/4" diameter specially-fabricated second floor bolts and for each of the 1-1/2" diameter roof bolts attached to the 10"-long angle shear connectors are approximately 26 kips/52 kips (0.25W/0.50g) and 46 kips/92 kips (0.25W/0.50g), respectively. The 10"-long steel angles embedded in the concrete roof beams each have a design capacity of 65 kips/122 kips for the 0.25W/0.50g seismic design criteria. The anchor strength capacities are based on discussions with AISC personnel, Material Experts, ASTM Material Standards, Mill Reports, and References 3, 4, and 5.

CONSTRUCTION COSTS

The structural upgrade costs for Building 311 were approximately $1.5M (1982 dollars). Building replacement costs in 1982 dollars were approximately $4.6M. Construction time associated with the structural portion of the upgrade was about 12 months.

AUTHORS' ACKNOWLEDGEMENT

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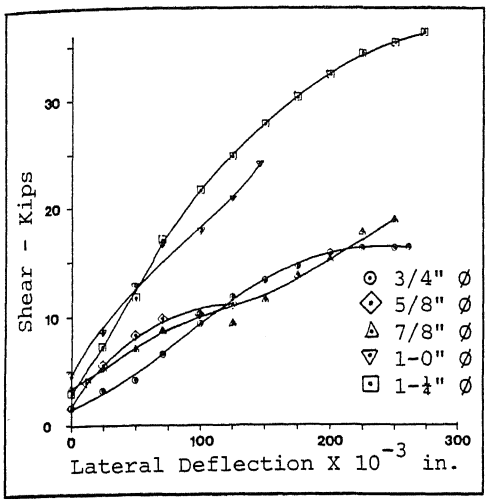

Fig. 1 Ramset Load-Deformation Curves

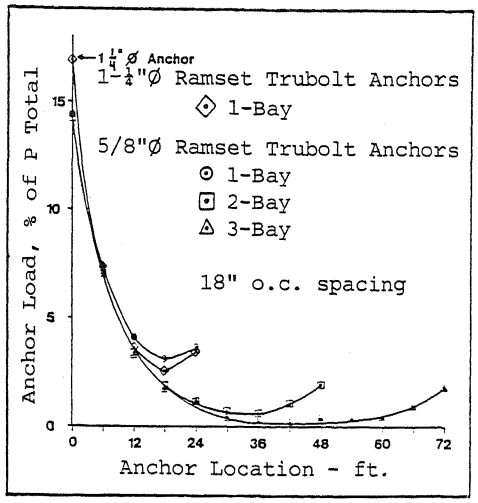

Fig. 2 Ramset Anchor Load Distribution

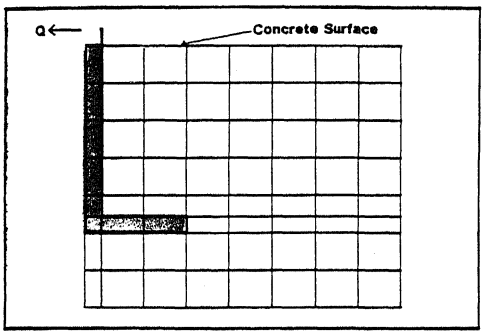

Fig. 3 Roof Connector Element Model

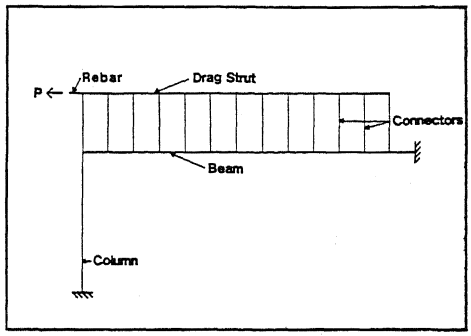

Fig. 4 Roof Connector System Model

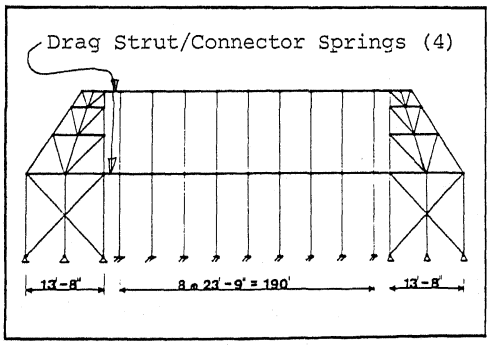

Fig. 5 Building Model, E-W

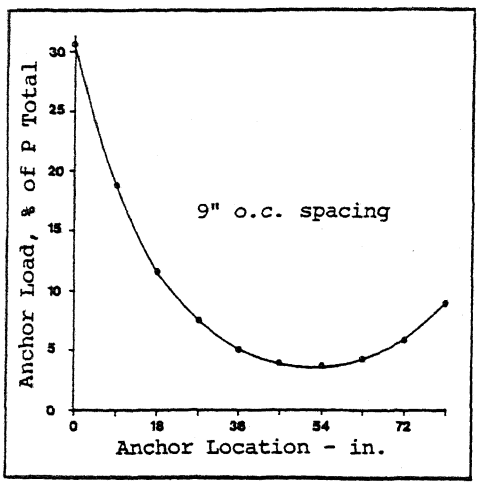

Fig. 6 Second Floor Anchor Load Distribution, E-W

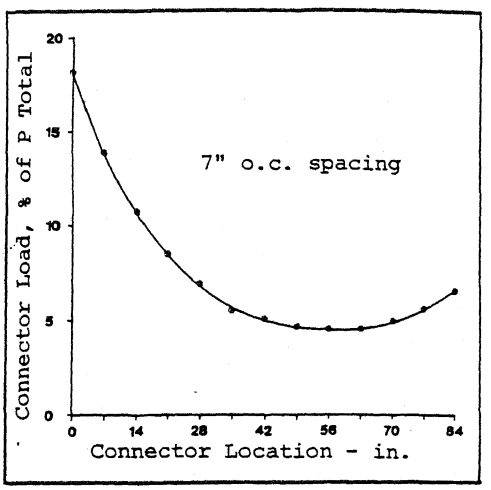

Fig. 7 Roof Connector Load Distribution, E-W

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