Performance-Based Seismic Upgrade of a 14-Story Suspended Slab Building Using State-of-the-Art Analysis and Construction Techniques

David Mar, SE, Principal at Tipping Mar + associates, Berkeley, CA
Leo Panian, PE, Project Manager at Tipping Mar + associates, Berkeley, CA
Robert A. Dameron, PE, Principal at Anatech, San Diego, CA
Brian E. Hansen, PE, Engineer at Anatech, San Diego, CA
Shahriar Vahdani, PhD, PE, Director of Geotechnical Engineering, Golder Associates, Oakland, CA
Denis Mitchell, PhD, PEng, Professor of Civil Engineering, McGill University, Montreal, Canada
James Paterson, Graduate Student, McGill University, Montreal, Canada

Abstract

This paper describes several applications of state-of-the-art analysis and design strategies in the development of a finely tuned seismic retrofit solution for a unique structure. As the retrofit project was voluntary and had to comply with a limited fixed construction budget, its success relied on refined insight of structural response and input from a multi-disciplined team of consultants and builders to develop an effective, cost-sensitive, and minimally intrusive design. The investment in thorough analysis of the building systems and in extensive structural and material testing improved the confidence level of the design solutions and resulted in significant construction cost savings.

Sophisticated analyses were employed to estimate the seismic demands on the structure and to predict its response. These included cyclic pushover runs of non-linear finite-element models and non-linear dynamic time-history runs incorporating site-specific ground motion characteristics and soil-structure interaction. Earthquake risk probabilities were used to establish the performance objectives for the vertical and lateral systems to reflect their potential hazard.

The design features the application of carbon fiber composites for shear strengthening of concrete walls and the use of headed confining reinforcement to improve the seismic performance of lap splices and to enhance the shear behavior of the walls. The solution also incorporates low-overhead high-load capacity micro-piles to improve the seismic stability of the foundation.

Introduction

The building at 2150 Shattuck Avenue in Berkeley, California is characterized by an unusual and innovative structural system. Unlike traditional building structures, in which columns support the floors, this building utilizes twin reinforced concrete towers to support the weight of the floors, which are suspended on hangers. The tension hangers transfer their load by means of a large truss system atop the towers (See Figures 1 through 3). The cores, supported on drilled pier foundations, resist all vertical and lateral loading.

Due to the building's unusual construction and the potential for strong ground shaking anticipated at its near fault site, the seismic rehabilitation of this structure posed some unique engineering difficulties, and required careful and creative solutions.

Building Description

The patented existing structural system represented an innovative and relatively inexpensive method of construction when the building was built in 1969. This lift-slab construction method allowed all of the floors to be constructed on the ground and sequentially hoisted into place. This type of construction was employed in several similar buildings in the San Francisco Bay area during the same period of time.

The fourteen story building extends approximately 160' feet above grade at roof level. The structure has a single level basement extending approximately 10' below grade. The first three levels above grade, which form a rectangular podium measuring roughly 168' by 86', consist of traditional concrete and steel frame construction. The upper eleven stories, which comprise the suspended tower structure, are rectangular in plan measuring 164' by 64' (See Figure 3).

The tower floors consist of lightweight concrete over metal deck supported by wide flange beams with fully welded connections. The tower floors are supported by a set of sixteen hangers, which consist of 2" thick steel plates tapering from widths ranging from 11" to 17.5" at the roof to 5.5" at the lowest floor. At each level, the floor framing is connected to the hangers by means of single 2" diameter bolts. The hangers, in-turn, connect to the truss assemblies extending 18' above the roof level (See Figure 5). The steel trusses use heavy W14 compression members and 2" thick plate eye-bar tension straps. The truss members are interconnected with 6" to 8" diameter pins (See Figures 6 to 9).

The trusses are supported on twin reinforced concrete cores that support the entire weight of the structure and provide all of its lateral resistance. The rectangular cores measure 20' by 36' with 12" thick walls on the long sides and 20" walls on the short sides. In the long direction, the core walls are interrupted by openings at each floor. The core walls are typically reinforced with vertical #6 bars each face spaced at 14" and 18" in the long and short directions respectively, with some added corner reinforcement. The vertical reinforcement is made continuous by means of lap splices approximately 24 bar diameters in length occurring at each floor. In the horizontal direction, the core walls are reinforced with #4 bars each face spaced at 12" and #5 bars each face spaced at 12" in the long and short directions respectively. The coupling beams over the openings are reinforced with two #9 bars top and bottom and #4 stirrups spaced at approximately 8".

A 6' thick pile cap supports each core with 135 - 24" diameter concrete drilled piers. The piers are typically 25' in length and nominally connected to the cap with a straight extension of the vertical pile reinforcement of 21".

Seismic Risk and Hazard

Several factors made it essential to develop site-specific ground acceleration time history. First, the structure is located 1.5 km (0.9 mi.) from the active Hayward fault, where large intensity motions in both the horizontal and vertical directions are anticipated due to the activity of the fault and the close proximity to the project site (See Figure 4). Second, the non-redundant nature of the vertical system makes this structure particularly susceptible to vertical excitations. Third, the seismic retrofit is voluntary and the limited project budget was an integral design variable. As such, the goal was to develop and optimize a total solution based on the interrelated factors such as seismic performance, consequences of failure in different modes, costs of various retrofit schemes, and seismic risks. This required relating the risk to the intensity of seismic ground motions.

Initially three sets of recorded time history were selected[1] to represent events having a 10% probability of exceedance in 50 years (10/50) and a 10% probability of exceedance in 100 years (10/100). Each set consisted of two horizontal components and one vertical component of ground acceleration time history. These records were selected to best represent ground motions compatible with seismic setting, local geology, earthquake magnitude, and source-to-site distance. The selected time histories were modified in the frequency domain to match the target design response spectra developed for the project site using a site specific seismic risk analysis. The computed design response spectra probabilistically accounted for all significant faults within a 100 km radius of the site and incorporated near-field effects. (See Figures 12 & 13).

In addition, a plot of the spectral vertical acceleration, S_a, at a period of 0.5 second (the period of the fundamental vertical mode), versus return period were developed (See Figure 14). As expected, this plot shows that increasingly intense shaking

[1] Seismology studies were performed by Golder Associates

corresponds to rarer events. This plot was used to extrapolate beyond the two calculated design response spectra using ratios of S_a as a scaling factor.

Since the consequences of a failure in the hanger/truss system are catastrophic, it was necessary to reverse the traditional process of evaluating a structure for a given risk. As, such the design team determined the risk associated with the failure of the controlling critical component, and evaluated its acceptability. The lateral system, which is less critical in regards to consequences of failure, was evaluated at 10/100 event.

Vertical System

The vertical suspension system has a troubling lack of redundancy. Failure of a single critical member or pin connected joint in the truss could lead to progressive collapse. Details of the system are illustrated in Figures 6 to 11. The strategy was to model the entire suspended structure and supporting cores with SAP2000, and to perform a series of vertical elastic time-history analyses (See Figure 16) to estimate the demands on the system subjected to the vertical ground shaking records. Each beam of the floor system was modeled, and the masses of the floors and walls were discretized and assigned at beam intersections to appropriately capture the distribution. Partially composite properties were assumed to capture the out-of-plane stiffness of the floors. For the global analysis an overall damping ratio of 5% was chosen to reflect the interaction with partitions and the curtain wall. The time-history records resulting in the worst-case for critical members were scaled up until a failure in the system occurred, and the corresponding risk was determined. Failure was defined as, either exceeding the failure load of a critical member, or having a tension hanger or truss strap going slack. The reason for the latter restriction was the concern over impact loading on some potentially fracture-critical connections.

A systematic capacity evaluation of the vertical-load carrying components was performed to establish the system limit. Based on preliminary calculations, it was shown that all of the members and connections, with the exceptions of the large pinned truss connections, could withstand loading beyond the slack point. Due to the complex configuration of these connections and the simplifying assumptions used to calculate their capacities, it was felt that a more accurate evaluation was needed to establish their failure load.

Thus, representative joints were modeled using the program ABAQUS to capture their non-linear response to seismic demands[2] (See Figures 17 & 18). Based on these analyses, higher capacities were predicted than those established using simple design equations, indicating that the slack point could be reached prior to failure of critical components. Thus, the controlling time-history record and its probability of exceedance was considered to be that which just produced slack in either a hanger or a truss strap. Based on this criteria, the failure threshold corresponds to ground shaking having a return period of approximately 1600 years. This level is slightly above a maximum credible event (MCE) scenario earthquake, which has a return period of approximately 1400 years. It is a significantly larger ground motion than the 10/50, 475 year return period, minimum criteria provided by the building code. In essence, an implied reliability/redundancy factor of roughly 1.6 was achieved for the vertical system.

After considering the large cost increase required to retrofit the vertical system, the owners accepted the level of risk implied by the hazard evaluation, and elected not to pursue higher performance levels. One exception was the decision to locally strengthen the truss support points at the top corners of the cores. In these areas, the concrete below the bearing plates of the truss would be overstressed under vertical ground shaking. Given the relative benefit-to-cost of this work, it was included in the project. Steel corner plates, secured with dowels, provide confinement to enhance the strength of these regions (See Figure 23).

Lateral System

The towers resist the inertial loads generated by the tower floors by mechanical locking, since the floors surround each core. Two 1” square steel bars welded to each pair of intersecting floor beams, positioned with a 1/32” gap at the corners of the core effectively lock the floors to the towers laterally (See

[2] This work was performed by Anatech Corp.

Figure. 10). The floors are vertically disconnected from the core, just as they were needed to be during the original hoisting operation. The lower three floors are cast-in-place concrete construction, which are not suspended for vertical support, and gain their lateral resistance from the core. The lightly reinforced core resists all of the lateral loads through shear and bending.

Under the anticipated seismic demands, the cores are expected to sustain significant inelastic deformations. An important component in the upgrade strategy was to understand the behavior of the cores, and to realistically assess their weaknesses as critical components for a cost effective upgrade.

This approach differs from judging deficiencies relative to current code requirements. Typical code based deficiencies of the structure include the lack of diagonal detailing at the coupling beams and the lack of boundary element detailing. Simply bringing the structure to code strength does not solve the problems and may, in fact, exacerbate them. For example, by yielding, the relatively weak coupling beams actually protect the overall stability of the cores by attracting damage that would otherwise occur in other more critical places. This effect reduces the shear demand on the core, as well as demands on the foundation, making it easier to achieve the desired failure mechanism. Our approach was to perform an analysis with sufficient degree of sophistication to rationally assess the limit-state of the structure subjected to the design ground motions. With this approach, a very different, rational, and coherent set of solutions is arrived at than would otherwise be indicated by a code based design deficiency approach.

The evaluation of the existing structure considered traditional flexural mechanisms based on the response of cantilever solid and coupled shear walls to represent the behavior in the east-west and north-south directions respectively. The expected response to diagonal loading, in particular, was not practicable based on simple hand calculations. Obvious weaknesses consisted of the core shear capacities, the lap splices in the vertical reinforcement, and the pier to cap connections, which are not detailed for tension loads. Less obvious are the coupling effects of the link beams and the compressive strain demands on the walls.

The presence of openings and coupling beams complicates the assessment of the cores for north-south loading and for diagonal loading. The diagonal load case is of particular interest because it is anticipated that the more intense fault-normal motions will attack the structure on a diagonal axis. Since plane sections of the coupled wall do not remain plane and the degree of coupling changes with inelastic action, the overall flexural capacity, and hence the expected shear demand, is difficult to quantify. The coupling beams for the upper third of the core were shown not to yield, while the lower coupling beams were subjected to large deformations which led to capacity degradation as the section wore out. This effect made conventional sectional calculations suspect. As such, it was difficult to determine, with accuracy, upper bound shear demands.

Retrofit Strategy

The strategy for seismic retrofit of the building sought to improve several key deficiencies to improve the seismic response of the structure, without necessarily increasing the lateral load capacity. To increase the ductility of the system, the shear capacity of the cores is enhanced with carbon fiber, the performance of the existing lap splices is improved with clamping pins, the compressive strain capacity of the core walls is increased with confining through-pins, and new micro-piles are used to augment the overturning resistance of the foundation. Determining how to proportion each improvement, and assessing the effectiveness of the retrofit is a more complex undertaking, which is elaborated upon in the analysis and testing sections below. However, this is in keeping with the project goal of spending enough on engineering to leverage much greater savings in the construction budget and obtain superior seismic response.

The addition of up to 3 layers of carbon fiber to the lower two levels in the north-south direction of the cores and 2 layers to the lowest level in the east-west direction of the cores is an effective of adding shear strength without increasing the flexural strength, and thereby, the shear demand (See Figures 20 & 21)[3].

[3] The Fyfe Co. advised the design team as to the relative costs of carbon and E-glass composites, as well as anchoring details.

This is one of the true uni-directional structural materials available, and it provided the added benefit of not reducing the usable office space.

Fixing the lap splices (See Figure 22) posed similar problems in that the fix needed to be architecturally discrete, while being effective enough to allow the vertical bars to yield. The retrofit scheme is to add three clamping through-pins, along the length of each lapped pair of vertical bars, for the lowest three levels of the core where vertical strains are expected to be highest.

The pins consist of special T-head reinforcing bars, which have a welded plate anchor at one end of a reinforcing bar and a threaded plate anchor at the other. The goal of the retrofit is to improve the capacity and cyclic durability of the splice by confining it and minimizing the dilational strains. The final design is subject to verification from the testing program at McGill University in Canada (described below) which was not complete at the time of this writing.

The same clamping pins are applied to the face of the lowest level core walls in an array at a spacing equal to the wall thickness. These pins will provide confinement for two regions of the walls in the plastic hinges. The first is the confinement of the in-plane diagonal compression struts which will form in the walls under the combined action of shear and flexure. The second is the confinement of the wall boundaries - openings, corners, and at the base of the wall - against vertical compressive stresses.

The foundation was improved by adding twelve micro-piles to each core (See Figure 25). The piles are connected to the core by new concrete buttress walls which reinforce the existing buttress and core walls (See Figure 24). The piles were proportioned to add enough strength so the foundation could resist the full flexural capacity of the cores.

The geotechnical engineer[4] provided load vs. deformation curves for the new piles and existing piers, which were incorporated in the dynamic analysis described below. This strengthening represents a lower-bound ultimate strength of the foundation system, since it does not account for some positive interaction with the existing basement walls and slab. With this un-quantified improvement to the foundation, the subsequent analysis considered an upper-bound fixed base condition for the cores along with a lower-bound flexible foundation case to bound the solution.

In summary, to characterize the inelastic behavior of the core towers and establish their capacity under seismic loads, the failure mechanism of the system had to be determined. Typically, the strengths of various components are calculated using simple design formulas. However, due to the complexity of this system, the traditional investigation methods could not reliably predict its inelastic response. This would lead to significant conservatism, which ultimately manifests itself in construction costs.

Code based estimates indicated that the cores would suffer shear failure, which would call for significant addition of composite fiber reinforcing, pushing the limits of cost and practicability. At this juncture, sensing that the solution relied on a better understanding of the seismic response, the design team turned to a detailed finite element analysis to develop a retrofit strategy. The following sections describe the study and the results obtained .

Detailed Finite Element Analyses

Based on preliminary analysis, the strengthening solution called for through-wall reinforcement to improve the performance of existing lap splices and to confine the concrete walls. In addition, up to five layers of carbon fiber/epoxy wrap was specified to provide adequate shear strength at the lower stories. However, the multiple layers of wrap are very expensive, and the placement of wrap above the 2nd story would pose significant construction difficulties in the occupied building. Thus, a verification and optimization study was undertaken to confirm satisfactory performance and maximize the effect of the construction budget.

To capture the seismic response of the structure ANATECH developed detailed finite element models of the complete building core. The first model included the retrofitting of all the vertical lap splices in the building (providing drilled and epoxy-bonded ties through the walls in the lap splice regions and the corners to provide confinement),

[4] Geotechnical work performed by Subsurface Consultants, Lafayette, CA

without any other retrofitting. Based on cyclic pushover analyses in the strong axis, weak axis, and diagonal directions, and on a strain-based damage criteria in the concrete, the areas in greatest need of carbon wrap were selected. After optimizing the thickness and locations for application of wrap, final cyclic pushovers were analyzed, and nonlinear time-history analyses (NTHA) were run to confirm the building's performance to design earthquake motions. Analysis of pile group overturning behavior was also performed for the proposed pile retrofit scheme. The time-history model was also used to compare the response predicted for flexible versus fixed-base foundation conditions.

When subjected to the design seismic motions, the following structural components were shown to be highly stressed: the segments between the door openings in the core (which behave as shear links or coupling beams), the walls in the lower stories due to bending and shear, and the corners in the lower stories due to shear and flexural compression. Of particular concern is the response and damage due to shaking in a diagonal direction because the shear load path associated with diagonal motion is not easily evaluated by conventional design methods.

Finite Element and Material Modeling

Illustration of the computational grid with the solid modeling of the lower floors is shown in Figure 26. The figure shows the footprint of the existing pile locations and of the partial floor slab in the interior of the core. This partial slab and its connecting beams were found to be important to the representation of the lateral stiffness of the core walls. The modeling approach used concrete modeled with 3D solid elements (in the lower three floors for the pushover models) and shell elements in the upper floors (all the floors for the NTHA models). Rebar was modeled explicitly with sub-elements, piles were modeled explicitly with nonlinear pile-soil interaction springs, and passive soil resistance was modeled with nonlinear springs. The various foundation elements consisted of spring-truss element pairs placed in series to represent realistic hysteretic behavior of piles and of passive and active soil-foundation interaction. The concrete analysis program ANACAP-U (which also utilizes the general purpose program ABAQUS) was used. ANACAP-U has extensive experimental verification and has been used in many similar applications. The constitutive (material) model (ANACAP-U) was developed by ANATECH over the past two decades for the nonlinear analysis of reinforced concrete[5].

The approach is based on the smeared-cracking method for tension and J_2-plasticity for compression that permit the incorporation of cracking, compressive plasticity (softening), cyclic hysteresis, stiffness degradation, rebar fracture, rebar bond-slip and ultimate structural failure. Within the model, cracking and all other forms of material non-linearity are treated at the finite element integration points. Thus, the cracking and stress/strain state can vary within an element. Cracks are assumed to form perpendicular to the principal stress directions in which the cracking criterion is exceeded. Multiple cracks are allowed to form, but they must be mutually orthogonal. When cracking occurs, the stress normal to the crack direction is reduced to zero, resulting in redistribution of stresses around the crack. Once a crack forms, the direction of the crack remains fixed, and it can never "heal." However, cracks may close and re-open under load reversals. The shear stiffness is also reduced upon cracking and further decays as the crack opens. This effect, known as "shear retention", is attributed to crack roughness and aggregate interlock. Once crack strains begin to increase significantly, the model represents "shear shedding" (the decrease of concrete shear stress in heavily cracked zones) that is observed in laboratory tests. Reinforcement is modeled as individual sub-elements within the concrete elements. Rebar sub-element stiffness is superimposed onto the concrete element stiffness in which the rebar resides. The rebar material behavior is handled with a separate constitutive model that treats the steel plasticity, strain hardening, and bond-slip behavior. The concrete and rebar formulations can handle arbitrary strain reversals at any point in the response, whether in tension or compression.

Boundary Conditions & Loading

[5] This software is based on the work by Rashid, the original developer of the smeared-cracking finite element in 1968. During the 1980's it was used successfully to predict large-scale nuclear containment structural behavior where predicting continuum response is critical to the solution. The methodology has also been successfully applied to a wide variety of beams, slabs, and columns, many of which are calibrated to and validated by laboratory tests.

The floors external to the core have only a small gap separating them from the core walls. Thus, during a seismic event the floors will transmit lateral inertial forces to the core along the lines of contact and influence the deformation of the core cross-section, i.e. the diaphragm stiffness of the floors will force the core-to-floor contact lines to remain straight, rather than bulge outward.

After a few trial analyses that produced bulging deformation in the core walls, equation constraints were applied to force the walls of the cores between corners at each floor line to deform in a line. With these constraints, the core cross-section could still expand, contract or distort torsionally, while being restrained from "ovalizing" at each floor level. By using these constraints in the dynamic analysis, it was also convenient to distribute the floor mass evenly to the four corners of the core at each floor level.

For the pushover analysis, a modal shape assumption was needed which captures the predominant behavior of the building. Since predicting extensive damage and non-linearity requires displacement application rather than force (for numerical stability), the modal assumption was a two-step process. First, a force distribution was assumed based on preliminary dynamic analyses. Second, the displaced shape resulting from these forces was calculated. Cyclic pushover analysis was then based on increasing peak amplitudes that were a multiple of this initial displacement pattern. This had the effect of including some higher mode participation in the pushover analysis, rather than only the first mode that would be achieved with a simple inertial push.

For the pushover analysis, the foundation was assumed to be rigid. This assumption tends to provide an upper-bound representation of damage that can occur in the lower floors of the building, rather than the more distributed deformations that result from a flexible foundation. The fixed base is a conservative evaluation assumption in the pushover analysis.

For the time-history analysis, both foundation assumptions were modeled and evaluated. The NTHA also employed Rayleigh damping with target anchor points at 5% of critical damping. No Rayleigh Damping was assigned to the foundation, however, because of its significant hysteretic damping, and the desire for damping to remain conservative.

Finite Element Analysis Results and Conclusions

The pushover hysteresis plots for the structure without carbon wrap (but with confinement retrofit) showed the development of large reductions in stiffness, indicating the accumulation of damage associated with concrete cracking, rebar yielding, and bond slip (See Figures 27 & 28). The greatest degradation was in the strong axis push case as shown in Figure 27. Using widely accepted structural component performance standards, seismic "failure" of a component can be defined by 15-20% drop in strength from the peak. This first occurs after the third cycle at ductility 2.5, or at 10.5" of north-south roof displacement. Concrete strain limits were established representing damage milestones that, if exceeded, were judged to require carbon wrap. These were tension strains in rebar larger than 0.02, dilational strains perpendicular to lap-spliced rebar in excess of 0.001, or strains in shear reinforcement in excess of approximately 0.004 to 0.006. The strain contour information supports a conclusion of using carbon wrap of the core only below the 3rd floor. Comparisons of performance with and without the carbon wrap showed noticeable differences in the north-south direction, but insignificant differences in the east-west. The differences are a 5% strength enhancement, increased ductility, and reduced concrete strains in the affected walls. Horizontal strain contours with and without carbon wrap are shown in Figures 29 & 30. Direct comparisons of concrete strains show that the carbon wrap effectively reduces strains and damage in the first two floors down to acceptable levels, especially for the North-South motion. It should be noted that the same ductility level corresponds to a larger displacement for the retrofitted model, making a direct comparison of strains somewhat misleading.

The NTHA was run first with a fixed base model. The analysis used 10% probability/100 year records. Peak demands (17 inch roof displacement), approximately equal to the displacement ductility demands of 4, were obtained, for which the pushover capacity limits and damage estimates were documented. Some significant "permanent set" displacement of the towers due to plastic action was found for this case, but this damage prediction was judged to be very conservative because of the fixed base assumption. The flexible foundation analysis model provided the most realistic simulation of the time-history response of the building. Time-history response at selected floors is shown in Figures 31-40. The analysis used the 10% probability/100 year records. The maximum drift was at the roof level, equal to 13-1/2 inches east-west and only 8-1/2 inches north-south (a maximum diagonal displacement of about 14 inches). This peak demand was lower than the response with the fixed base model and is well-bounded by the displacement ductility of 4 that was documented in the pushover analyses. The trend toward permanent set displacement of the towers due to plastic action that was observed in the fixed base analysis was completely absent from the flexible foundation analysis. Thus, the flexible foundation shows significantly better seismic performance. This appears to be due to the added hysteretic energy absorbed in the foundation elements and due to the flexibility below the basement which tends to "fuse-protect" the building from larger deformations occurring in the lower stories of the building.

Comparisons of rotation versus height showed that substantial rotations occur in the fixed-base model, concentrated in the 1st story of the building (thus concentrating large curvature, strain, and damage), while the rotations in the flexible foundation case occur in the foundation itself, and to a lesser degree, in distributed fashion across the lowest 5 stories (See Figures 41 & 42). The flexible foundation analysis confirms the desirable seismic performance that was sought in the retrofit design.

Experimental Verification

The innovative use of headed reinforcement in the retrofit of the shear wall core also sparked an experimental research program at McGill University. Four wall specimens representing the details used in the existing structure and the proposed retrofit will be tested under reversed cyclic loading. Two of the walls will have the “as-built” details and two companion walls will be retrofitted to demonstrate the effectiveness of the retrofit technique.

The first set of prototype test specimens were designed to have similar properties to the walls in the actual as-built structure. The specimens have cross section dimensions of 300 mm (12 in.) by 1200 mm (47 in.). The test specimens were chosen to simulate the most critical portions of the core near the door openings. The amount of vertical reinforcement was chosen to provide a similar yield force to the reinforcement in this portion of the core. There are lap splices in the vertical reinforcement at the floor levels in the as-built structure. The ratio of bar diameter to lap length is similar to that in the as-built structure. The horizontal reinforcement in the as-built structure is poorly detailed at the boundary element regions of the walls. The horizontal bars are anchored in the boundary regions using only 90-degree hooks. These poor anchorage and confinement details were also simulated in the test specimens. The details for Specimen 1 are given in Figure 43. While Specimen 1 has lap splices at its base, Specimen 2 represents the situation where lap splices occur above the base of the wall.

Companion specimens 1R and 2R were constructed with identical details to those of the unretrofitted specimens. These specimens will be retrofitted with headed reinforcement to increase confinement, as well as with carbon fiber wrapping to increase the shear strength.

For Specimen 1R the retrofit includes added reinforced concrete to thicken the base of the wall over the lap length. Headed dowels will anchor this concrete to the foundation block and headed reinforcement through the wall will provide additional confinement. The specimen was strengthened with bands of carbon fiber wrapping epoxied to the surface of the concrete to increase the shear resistance of the wall such that flexural hinging can occur. Although the actual structure will be retrofitted using continuous sheets of carbon fiber wrap, a reduced amount of carbon fiber wrap is being used on the test specimens to bring the shear capacity to the desired level. Special headed bars, with a threaded plate on one end, provide pins through the wall to add confinement in the plastic hinging region. Additional headed bars were epoxied into the ends of the wall to provide confinement in the boundary regions (See Figure. 44). Specimen 2R will examine retrofit details for the lap splices located above the base of the wall.

The walls will be tested in their horizontal position and are subjected to reversed cyclic loading.

Conclusion

This design approach focused not on increasing the strength of the structure overall, but on modifying the structural behavior such that acceptable ductile failure modes result. The structure's weaknesses consist of the cores' insufficient shear capacity, relative to their flexural mechanism demand, the inadequate lap splices of the vertical core reinforcement, and a lack of sufficient foundation strength.

The retrofit design was optimized with the aid of detailed analyses of the structure subjected to seismic loading. The shear strength deficit is overcome without increasing the flexural demand through the addition of unidirectional carbon fiber fabric in the plastic hinge region. The lap splices are fixed by an innovative technique, which will undergo testing at McGill University, of adding headed reinforcing pins to confine the concrete and improve bond. Finally, the foundation strength is increased with the addition of micro-piles and buttress walls.

To efficiently address the seismic hazards with a carefully crafted structural solution, the project relied on collaboration among various professionals to integrate insight and expertise from different disciplines, and to develop a cost-effective, broad-based, and state-of-the-art upgrade.

Acknowledgments

The authors are grateful to Headed Reinforcement Corporation (HRC) for funding the laptesting at McGill University, as well as the Fyfe Co. LLC for providing the carbon fiber wrap for the tests.

Figure 1. Original construction photo.

Figure 2. Original construction photo

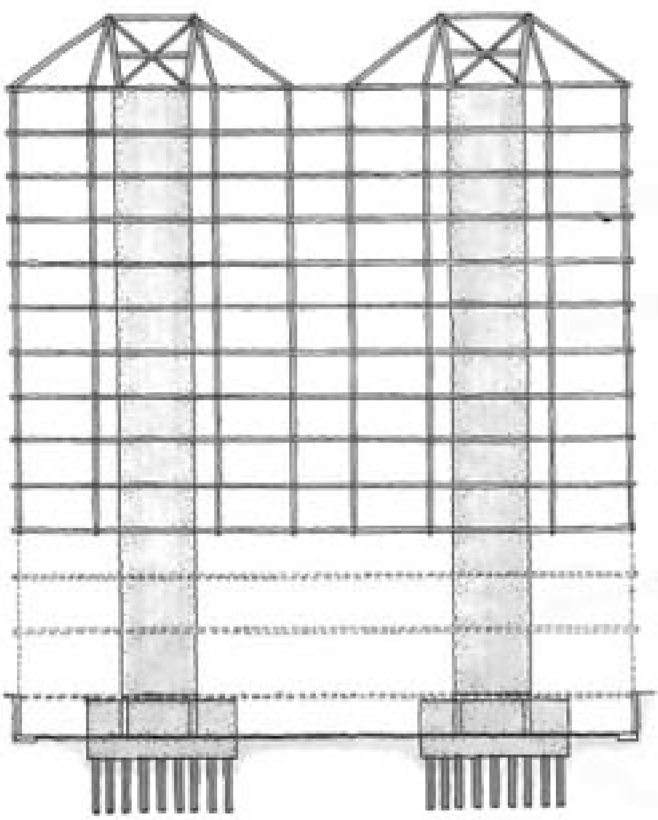


Figure 3. Structural system diagram.

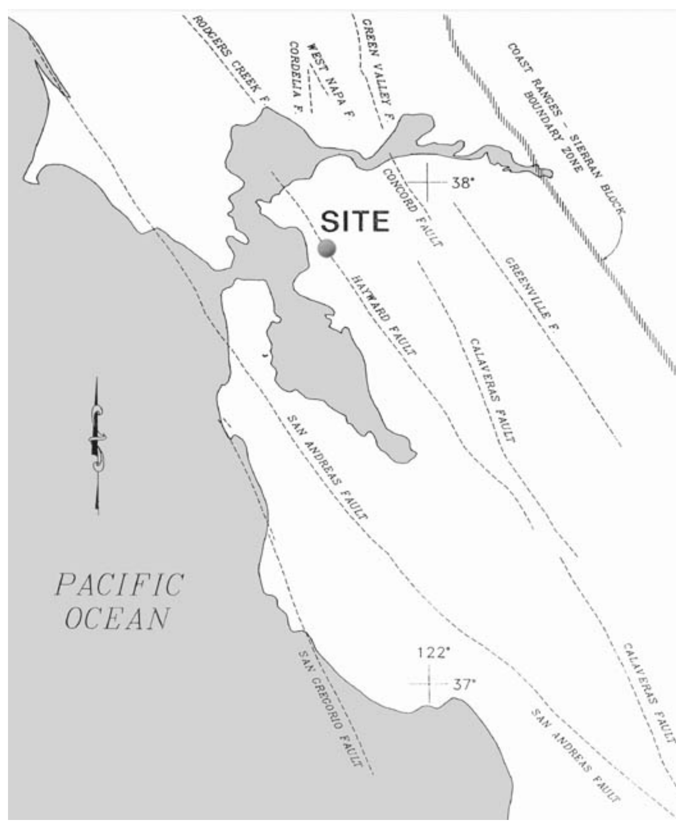


Figure 4. Site and fault map.

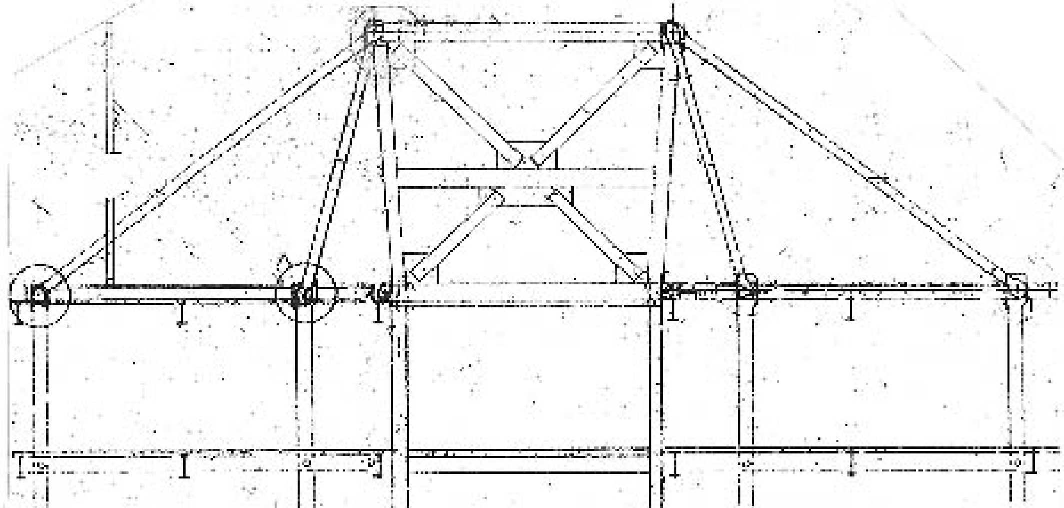


Figure 5. Overall truss diagram

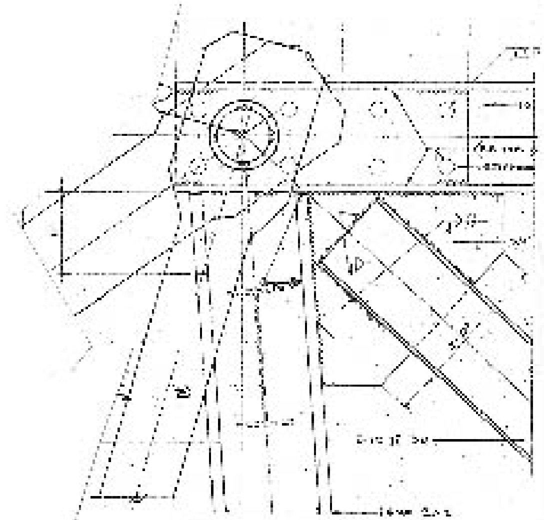


Figure 6. Upper strap detail

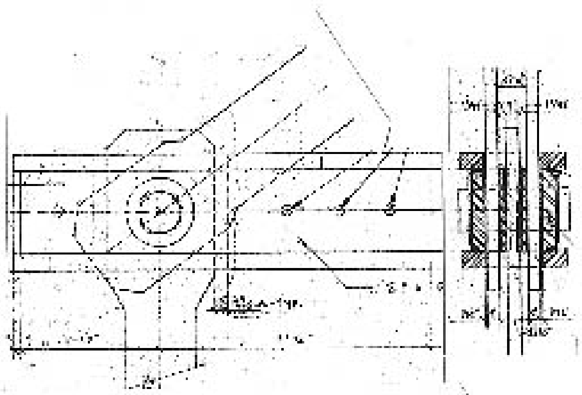


Figure 7. Lower outside strap/hanger detail

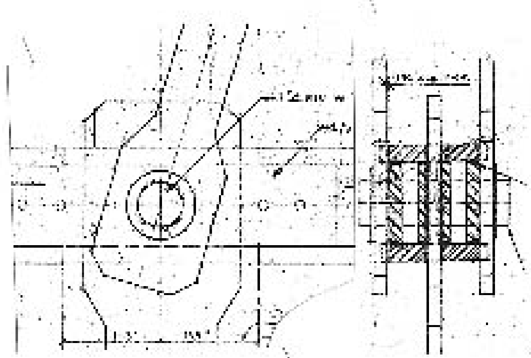


Figure 8. Lower inside strap/hanger detail

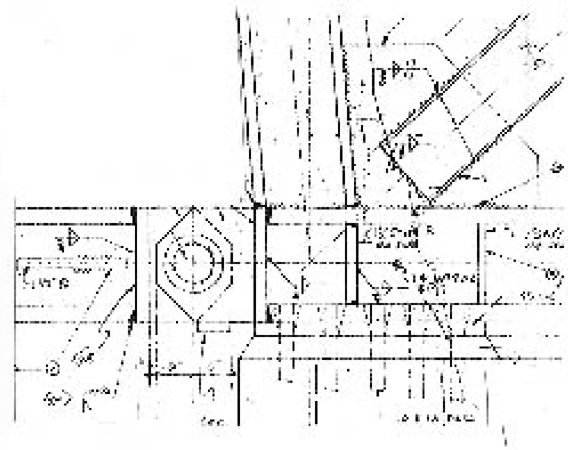


Figure 9. Truss to core detail

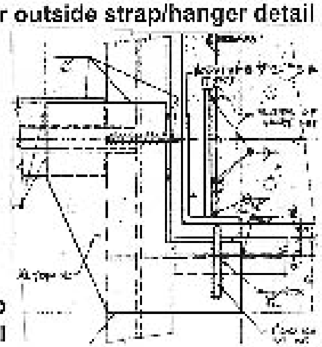


Figure 10. strap to floor detail

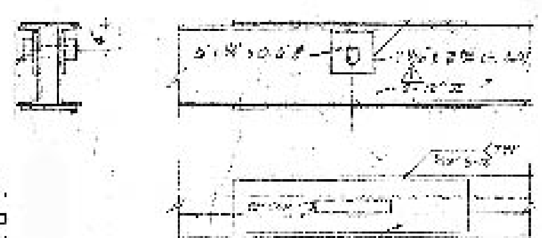


Figure 11. Diaphragm to core detail

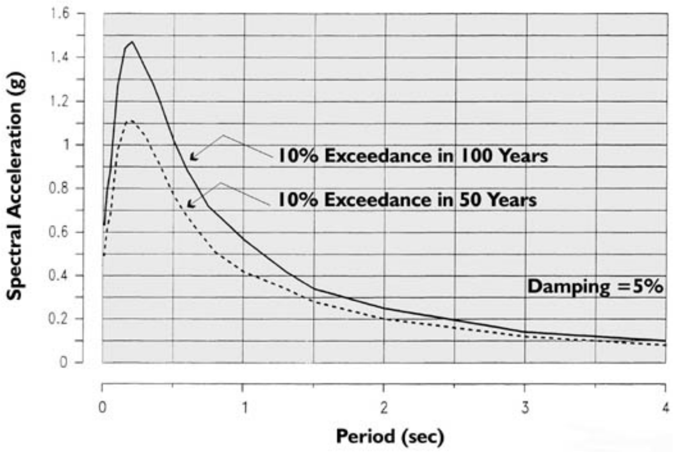


Figure 12. Site specific responses spectra – horizontal.

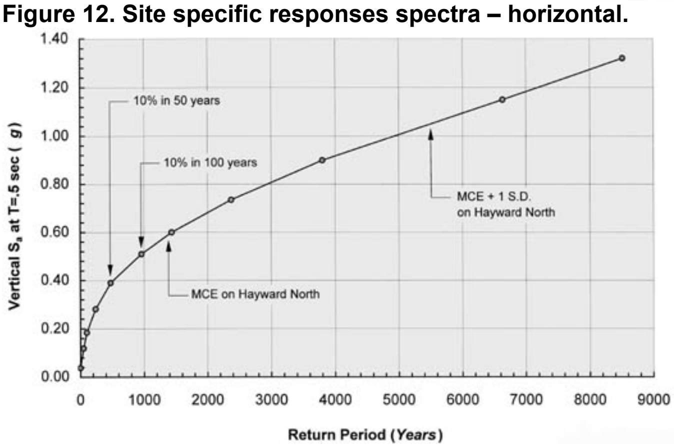


Figure 14. Vertical spectral acceleration (Sa) at T=.5sec vs. EQ return period.

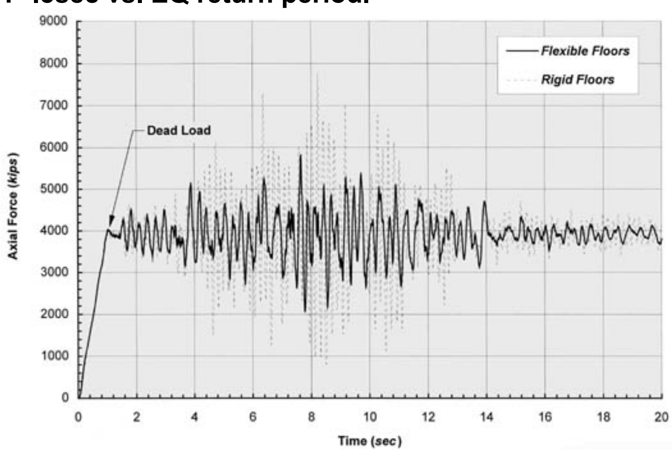


Figure 15. Strap force under effects of vertical EQ excitation.

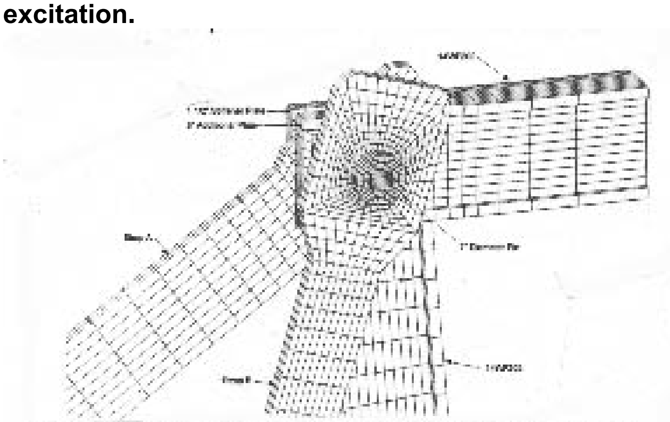


Figure 17. Non-linear solid F.E. model of truss joint

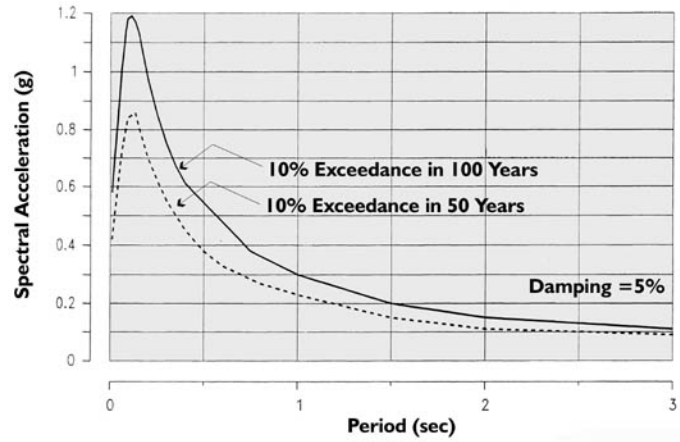


Figure 13. Site specific response spectra – vertical

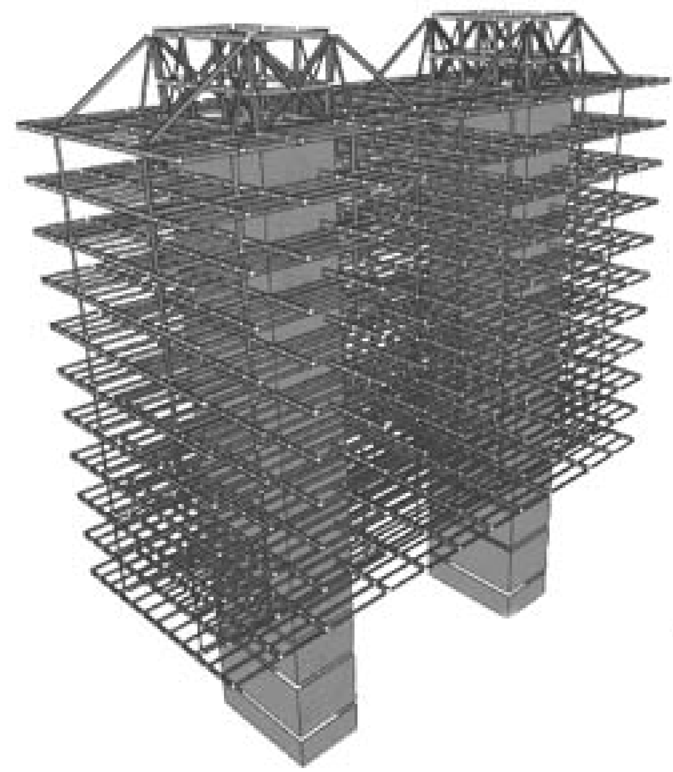


Figure 16. Model of core, floor, hanger and truss system to capture vertical EQ response.

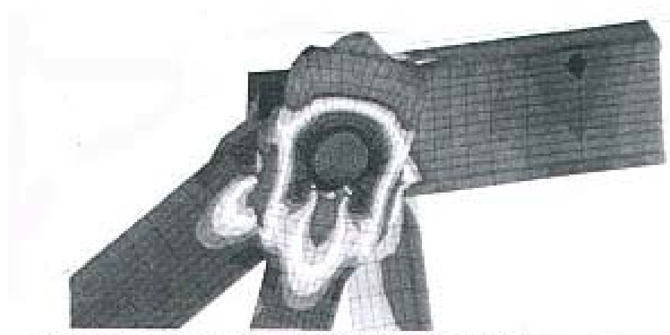


Figure 18. Truss joint (Figure17) with stress contours.

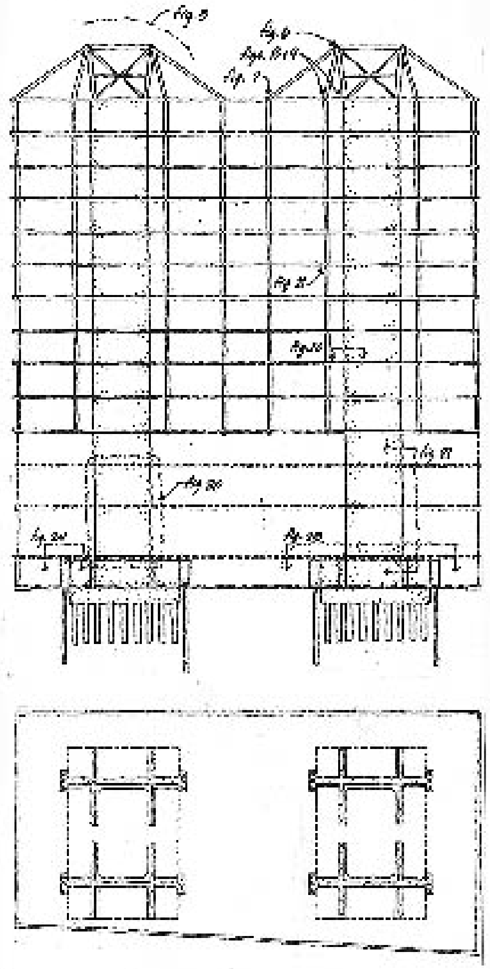


Figure 19. Overall building diagram

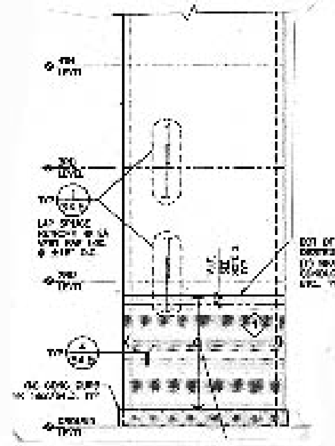


Figure 20. Carbon strengthening and confinement of core, E-W dir.

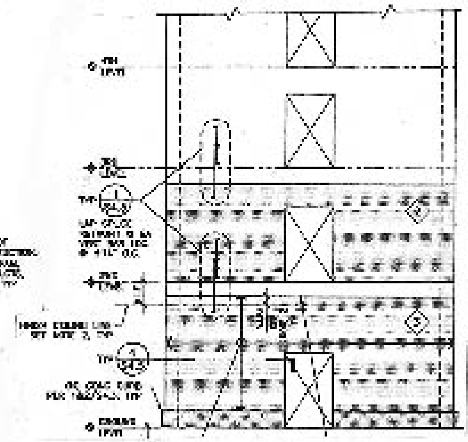


Figure 21. Carbon strengthening and confinement of core, N-S dir.

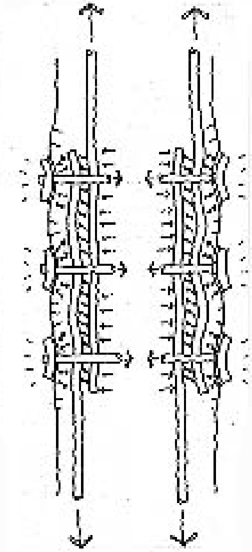


Figure 22. Detail to improve lap splices with confining pins

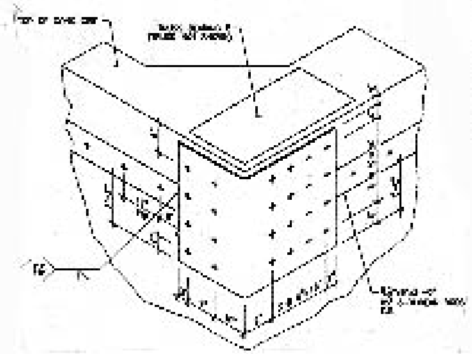


Figure 23. Strengthening detail of core corners below truss

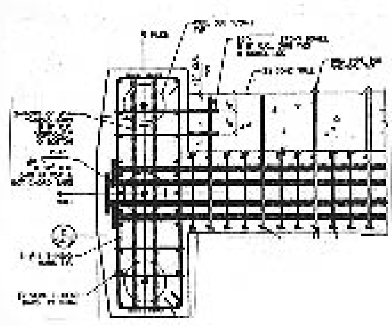


Figure 24. Pile and anchorage detail.

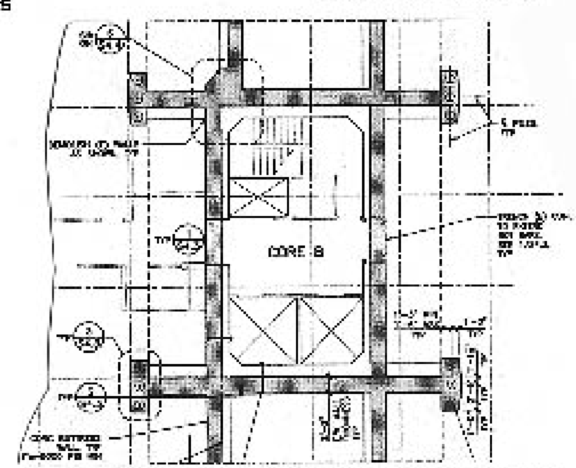
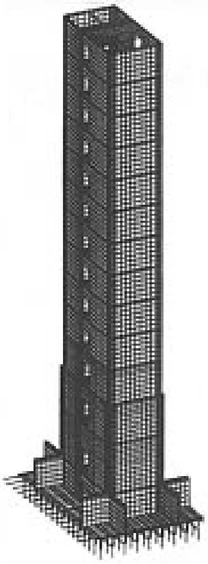


Figure 25. Foundation plan with added piles and buttress walls at core



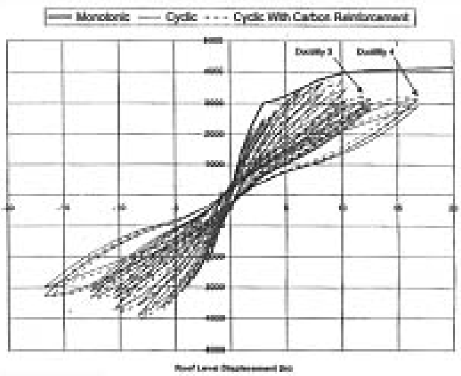


Figure 27. Cyclic pushover response curve for fixed-base core - N-S dir.

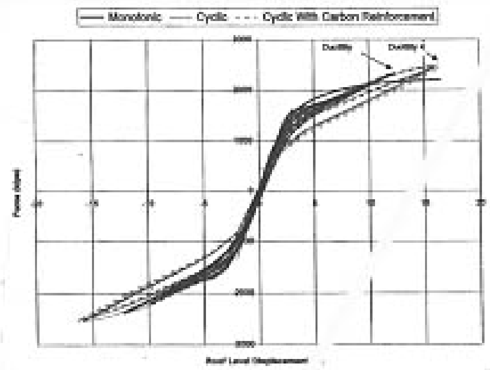
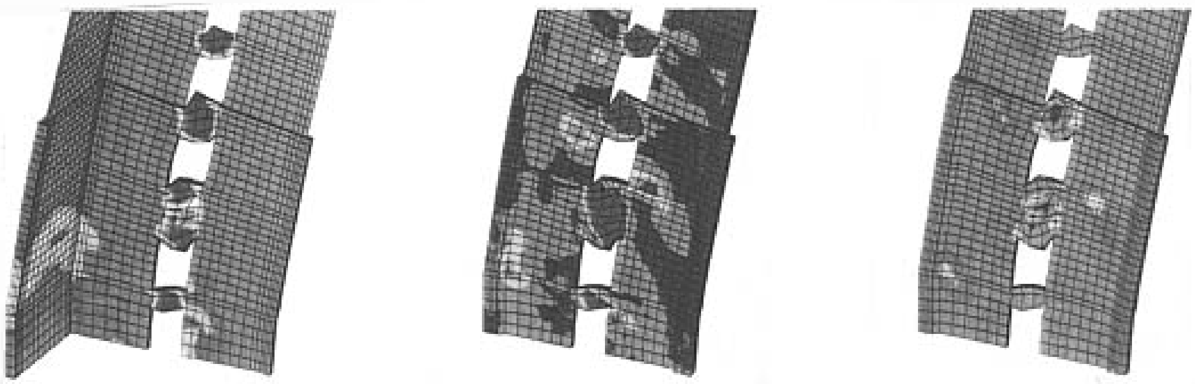


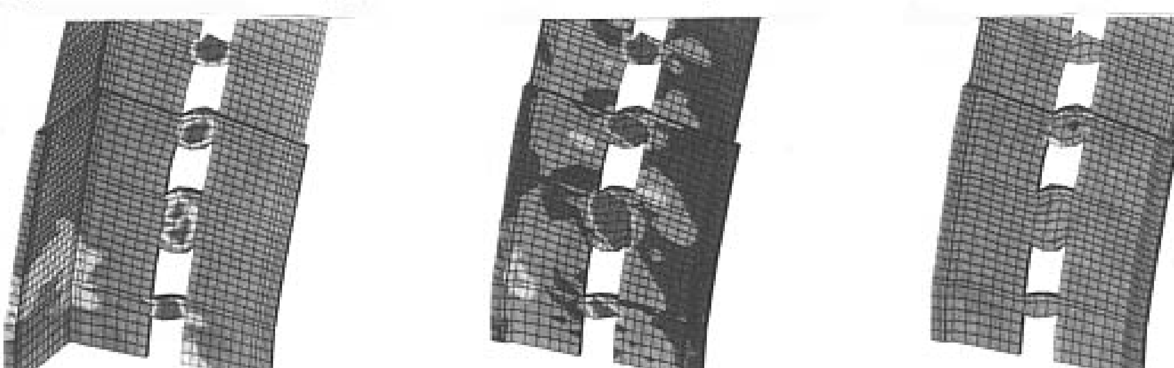
Figure 28. Cyclic pushc er response curve for fixed-base c re - E-W dir.

Figure 26. Shell and solid non-linear F.E. model of core and foundation



A. Vertical strains B. Shear strains C. N-S dir. strains

Figure 29. Strain contour diagrams for fixed base core, ductility = 4, N-S dir. - without carbon strengthening



A. Vertical strains B. Shear strains C. N-S dir. strains

Figure 30. Strain contour diagrams for fixed base core, ductility = 4, N-S dir. - with carbon strengthening

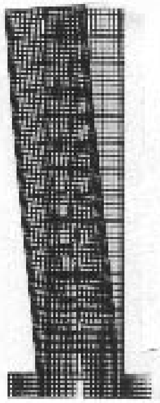


Figure 31. Core deflected shape, N-S dir (T=10.96 s) - fixed base model

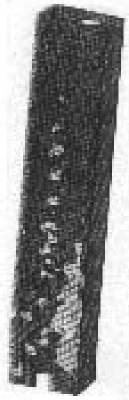


Figure 32. Core shear strain contours (T=10.96 s) - fixed base model

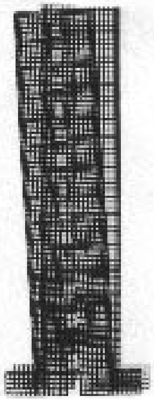


Figure 33. Core deflected shape, N-S dir (T=10.96 s) - flexible foundation model

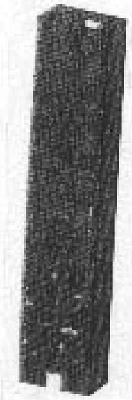


Figure 34. Core shear strain contours (T=10.96 s) - flexible foundation model

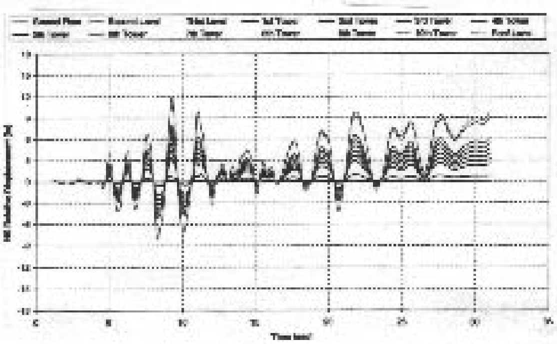


Figure 35. NTHA drift plot - N-S dir., fixed base model

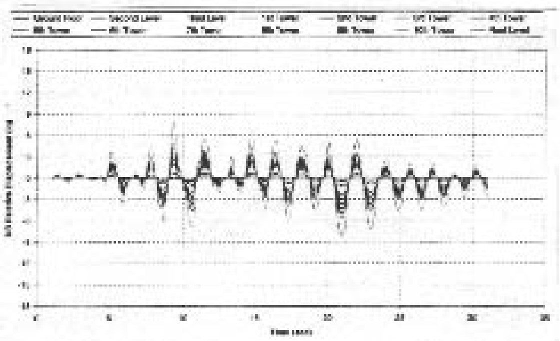


Figure 36. NTHA drift plot - N-S dir., flex. fdn. model

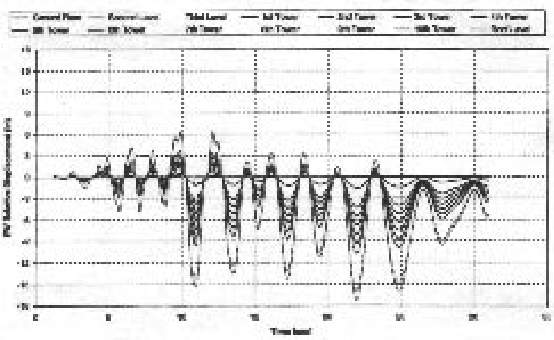


Figure 37. NTHA drift plot - E-W dir., fixed base model

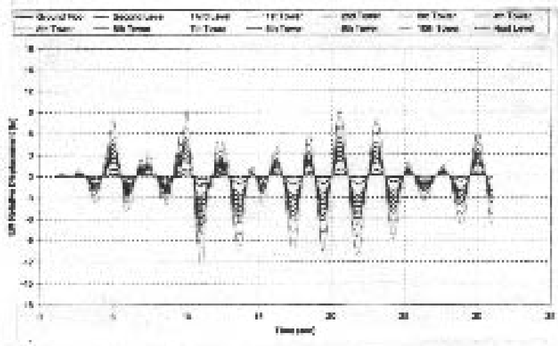


Figure 38. NTHA drift plot - E-W dir., flex. fdn. model

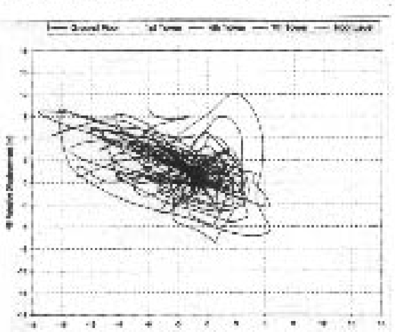


Figure 39. NTHA drift plot - plan view, fxd. base model

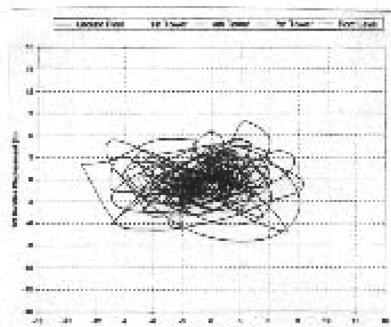


Figure 40. NTHA drift plot - plan view, flex. fdn. model

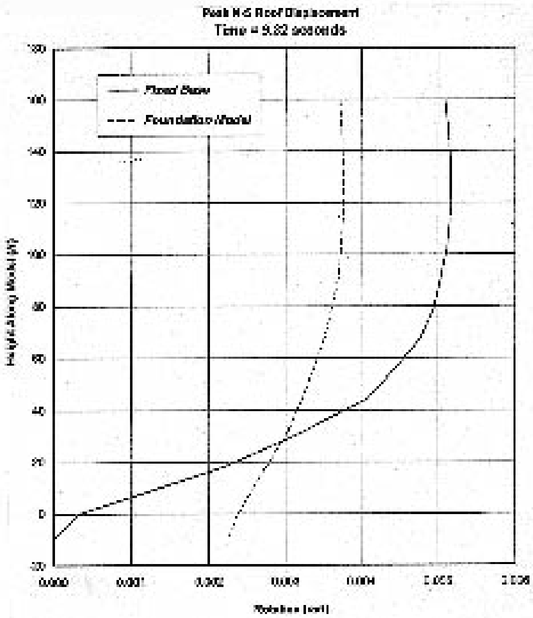


Figure 41. NTHA height vs. rotation, N-S dir.

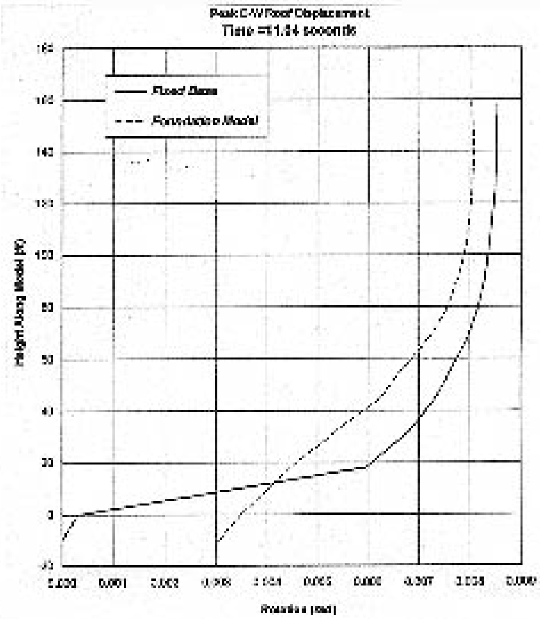


Figure 42. NTHA height vs. rotation, E-W dir.

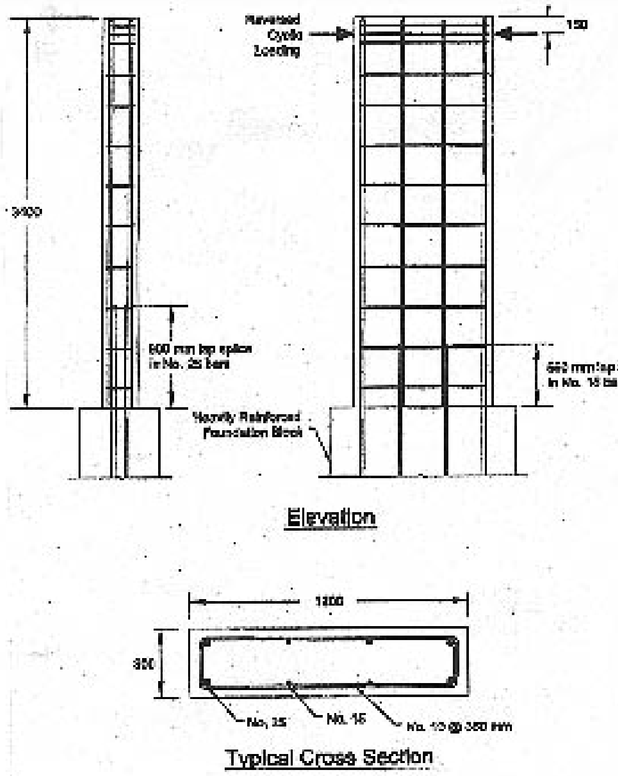


Figure 43. Test specimen replicating the as-built lap splice condition

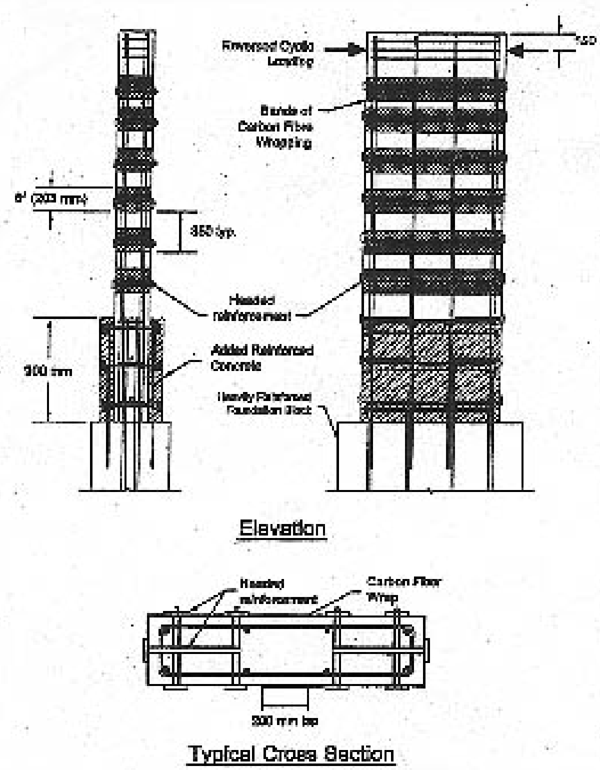


Figure 44. Test specimen replicating the retrofitted lap splice condition