2010 E-DEFENSE FOUR-STORY REINFORCED CONCRETE AND POST-TENSIONED BUILDINGS – COMPARATIVE STUDY OF EXPERIMENTAL AND ANALYTICAL RESULTS

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ABSTRACT

A series of shaking table tests were conducted on two, full-scale, four-story buildings on the NIED E-Defense shake table in December 2010. The buildings were almost identical in geometry and configuration; one building utilized a conventional reinforced concrete (RC) structural system with shear walls in one direction and moment frames in the other direction, whereas the other building utilized the same systems constructed with post-tensioned (PT) members. The buildings were simultaneously subjected to increasing intensity shaking until large deformations were reached to assess performance in service, design, and maximum considered earthquake shaking. Nonlinear response history analyses were conducted for the shear wall direction of the two buildings using CSI Perform3D in order to compare analytical and experimental results. Although the analytical models captured global response parameters reasonably well for the service- and design-level events, some inconsistencies between the simulated and measured responses were noted in the collapse-level event.

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A series of shaking table tests were conducted on two, full-scale, four-story buildings on the NIED E-Defense shake table in December 2010. The buildings were almost identical in geometry and configuration; one building utilized a conventional reinforced concrete (RC) structural system with shear walls in one direction and moment frames in the other direction, whereas the other building utilized the same systems constructed with post-tensioned (PT) members. The buildings were simultaneously subjected to increasing intensity shaking until large deformations were reached. Nonlinear response history analyses were conducted for the shear wall direction of the two buildings using CSI Perform3D in order to compare analytical and experimental results. Although the analytical models captured global response parameters reasonably well for the service- and design-level events, some inconsistencies between the simulated and measured responses were noted in the collapse-level event.

Introduction

The 2010 NIED E-Defense tests included testing of two buildings, a conventional reinforced concrete (RC) building, and a high-performance post-tensioned (PT) building. The two buildings were similar in geometry and configuration, with shear walls in one principle direction, and moment frames in the other direction. The buildings were subjected to increasing intensity shaking using the Kobe and Takatori records until large deformations were reached. The conventional RC building was designed according to the Japanese Standard Law (2007) and Architectural Institute of Japan requirements (AIJ, 1999), and also satisfied a majority of ASCE/SEI 7-05 and ACI 318-08 requirements for Special RC Structural Walls and Special RC moment frames (with an exception of strong column-weak beam requirements). The PT building was designed using a performance-based seismic design methodology and included high performance, post-tensioned lateral force-resisting systems. Moment frames consisted of precast prestressed beam and column elements, whereas structural walls utilized unbonded post-tensioned and mild steel to provide re-centering and energy dissipation characteristics. In addition, the PT building incorporated high

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performance materials such as high-strength concrete with steel fibers and high-strength transverse reinforcement. To meet the various design objectives, the base shear design strength of the PT building ended up being about 1.5 times that of the RC building in both directions. This study focuses on providing comparisons between measured and predicted (analytical) responses for the shear wall direction of the two buildings. Although use of relatively complex nonlinear modeling approaches have become common for design of shear wall buildings [1], field and laboratory data for full-scale buildings subjected to multi-axis shaking are lacking to assess the reliability of these models. Experimental data are mostly available for two-dimensional, moderate-scale structures tested under quasi-static loading (e.g. [2], [3]), and relatively limited buildings systems tested under uniaxial motions on shaking tables [4]. This is especially true for unbonded post-tensioned wall systems. Therefore, the full-scale, three dimensional, dynamic tests on the NIED E-Defense shaking table provide information to fill an important knowledge gap as well as a wealth of data to assess the ability of both simple and complex nonlinear modeling approaches to reliably predict important global and local responses, including system interactions. This paper presents results obtained from nonlinear response history analyses of the RC and PT buildings along with comparisons with experimentally measured data. The models were developed using Perform 3D [5], because this software is commonly used in engineering practice in the United States, and similar programs are used worldwide. Analysis results for a range of responses are compared including roof drifts, inter-story drifts, base overturning moments, floor accelerations, base wall rotations, and wall shear deformations. The test program, analytical models, and the ability of the analytical models to capture the measured responses are discussed in the following sections. Detailed information about the test program, including information about instrumentation and ground (table) motions is provided in PEER Report 2011/104 [6].

**Description of the Test**

The E-Defense shake table, the largest in the world, has plan dimensions of 20 m × 15 m allowing the two buildings to be tested simultaneously as shown in Figure 1(a). Each building weighed approximately 5900 kN and the combined weight of the two buildings was 98% of E-Defense table capacity. Descriptions of the RC and PT buildings are summarized in the following subsections.

![Figure 1. (a) Overview of the test buildings; (b), (c) instrumentation of the RC shear wall.](image)

**Test Buildings**

The lateral-force-resisting systems for the test buildings consisted of two-bay moment frames in the longitudinal-direction (-x) and two structural walls, one at each end of the building plan, in the transverse-direction (-y) (Figure 2). Story heights at all levels for both buildings were 3 m,
producing a building with an overall height of 12 m. Floor plan dimensions were 14.4 m (x) and 7.2 m (y).

**Figure 2.** Plan and elevation views of the test specimens.

**RC Building – Wall Direction**

Member cross-section dimensions were 500 mm × 500 mm for columns, 250 mm × 2500 mm for walls, 300 mm × 400 mm for interior beams at Axis B, and 300 mm × 300 mm for beams at axes A and C. Additional beams with cross sections of 300 × 400 mm supported the floor slab at intervals of 1.5 m. A 130 mm-thick floor slab was used at floor levels 2 through 4 and at the roof level. The design concrete compressive strength was 27 MPa. Primary longitudinal reinforcement consisted of 19 mm and 22 mm diameter bars. The actual material properties of concrete and steel used in the test buildings are presented in Table 1. Reinforcement details of shear walls are presented in Figure 3. It is noted that transverse reinforcement was different in the North (Axis A) and South (Axis C) walls.

**Figure 3.** Cross-sections of the shear walls.

**PT Building – Wall Direction**

In the PT building, member cross sections consisted of 450 mm × 450 mm columns, 250 mm × 2500 mm walls, and 300 mm × 300 mm beams. Column PT tendons were grouted while the tendons located in walls and beams were unbonded (sheathed and greased) from anchor to anchor. PT tendons were stressed to 60% of the yield stress for the walls and exterior beams in the y-direction, and 80% of the yield stress for all other prestressed members. Walls were constructed
of four precast concrete panels with eight D22 mild steel bars, unbonded over a length of 1.5m across the foundation-wall interface, to provide energy dissipation for the Unbonded, Post-Tensioned (UPT) wall. The design concrete compressive strength for the PT building was 60 N/mm$^2$. High-performance fiber reinforced cement composite was used at the first and second story wall panels of the North wall, while conventional concrete mix was used for the remaining wall panels. The actual concrete and steel properties are presented in Table 1 whereas reinforcement details of the UPT walls are shown in Figure 3.

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**Test plan, Ground (Table) Motions, and Instrumentation**

The test buildings were subjected to the JMA-Kobe motions recorded in 1995, scaled by 25%, 50%, and 100%, to produce a range of shaking intensities. At the completion of these tests, two additional tests were conducted using the JR-Takatori record scaled by 40% and 60%. Pseudo acceleration and displacement spectra of the Kobe ground motions are presented in Figure 4(a) and 4(b), respectively, along with spectra for a service level (SLE; 50% in 30 years), design level (DBE; 10% in 50 years), and maximum considered earthquake level (MCE; 2% in 50 years) based on ASCE 7-10 requirements [7] assuming that the buildings were located in downtown Los Angeles for Site Class B. Peak spectral accelerations observed on the shaking table were 0.89g, 1.58g and 3.42g at 25%, 50% and 100% Kobe records, respectively. It is noted that spectral acceleration demands for the 25% Kobe record are close to the SLE spectrum. For the 50% Kobe record, the demands are bounded by the DBE and MCE spectra near building fundamental periods (approximately 0.3 sec for both buildings), whereas the demands for the 100% Kobe record were much higher than the MCE spectrum.

The two test buildings were heavily instrumented to enable performance assessment and post-test analytical studies. A total of 609 channels of data were collected during the tests for RC and PT specimens, including accelerometers, displacement transducers (wire potentiometers, laser-type displacement transducers, and linear variable differential transducers (LVDTs)), and strain gauges. Typical instrumentation of the shear walls are shown in Figure 1(b) and (c).
Test Results

Figure 5(a) shows the roof drift histories of the RC building. Peak roof drifts are 0.2% (δ=23.5 mm), 0.84% (δ=100.7 mm), and 2.54% (δ=304.2 mm) for 25%, 50% and 100% Kobe records, respectively. Residual roof level displacement of 21 mm (0.2% drift) is noted for the 100% Kobe record. Figure 6(a) presents the building overturning moment versus roof drift relations, with base moment calculated as floor masses times absolute floor accelerations, multiplied by the associated floor heights from the base. Results presented in Figure 6(a) indicate essentially elastic response for the 25% Kobe record and some inelastic response (yielding, along with modest stiffness and strength degradation) for the 50% Kobe record. Significant yielding and stiffness degradation, along with modest strength degradation, are noted for the 100% Kobe record. Based on test observations, strength loss was likely due to concrete crushing and reinforcement buckling at wall boundaries (Figure 7(a)). Following crushing of concrete at the wall boundaries, substantial sliding was observed at the wall base for the 50% and 100% Kobe records.

Fig. 5(b) presents the roof drift time histories for the PT building. Reported roof drift values correspond to the center of plan. It is noted that some torsional response was observed for the PT building (particularly for the 100% Kobe record) due to different degrading responses of the two UPT walls. Peak roof drifts at the center of plan are 0.16% (δ=19.2 mm), 0.51% (δ=61.2 mm) and 1.56% (δ=187.2 mm) for 25%, 50% and 100% Kobe records, respectively. Complete self-centering response was achieved for all three tests. Peak drift values were generally less than those measured for the RC building. Fig. 6(b) presents the building overturning moment versus roof drift relations for the PT building. For the 25% Kobe record, responses are nearly elastic without significant energy dissipation or softening. For the 50% Kobe record, minor energy dissipation (associated with yielding of the mild steel reinforcement) and softening behavior (associated with gap opening at the base joint) are observed. For the 100% Kobe record, significant hysteretic energy dissipation and stiffness degradation are observed. Minor damage occurred only at the wall-foundation interface and was limited to concrete spalling at wall toes of axis C while axis A wall remained essentially intact (Fig. 7(b)).
Figure 6. Base moment vs. roof drift outputs of (a) RC building, (b) PT building.

Figure 7. Damage on the (a) RC (Axis A) and (b) PT (Axis A) shear walls under 100% Kobe record.

Modeling and Results

Analytical models for the shear wall directions of RC and PT buildings (axes A, B and C in Figure 2) were developed using Perform 3D. For the RC building, the model was based on current modeling techniques [8] and recommendations provided by [1]. For the PT building, the Unbonded Post-Tensioned (UPT) walls and beams were modeled based on recommendations by [9], [10], [11]. Three-dimensional and elevation views of the RC and PT models are shown in Figure 8. The RC model consists of shear walls with fiber cross sections and frame elements with plastic hinges for beams and columns. In the PT building fiber sections were used for the walls, beams and columns. Additional information on the modeling of each building is provided below.

Figure 8. Three-dimensional and elevation views of the RC model and the PT model.
The RC shear walls were modeled using 4-noded, uniaxial, fiber “Shear Wall Elements”. Plane sections are assumed to remain plane after loading and uniaxial material models for concrete and reinforcement are used to determine section and element responses. Unconfined concrete was modeled using a stress-strain relation based on the results of material characterization tests that were performed prior to the shake table testing [6]. Tension behavior of concrete was included with peak tensile capacity of \( f'_c = 7.5 \sqrt{f'_t} \) and post-peak stiffness of \( E_c = 0.05E_c \) [12], where \( E_c \) is modulus of elasticity of concrete. The stress-strain relations of the reinforcement were defined using trilinear relationships based on the test results presented in Table 1, whereas the shear behavior was modeled using a trilinear relation similar to that recommended by ASCE 41-06 Supplement #1 [14]. The uncracked shear modulus was taken as \( G_c = E_c/2(1 + \nu) = 0.4E_c \) and shear cracking was assumed to occur at \( 0.25 \sqrt{f'_c} \text{MPa} \left(3\sqrt{f'_t}, \text{psi}\right) \), but not greater than \( 0.5V_n \), where \( V_n \) is the ACI 318-08 nominal wall shear strength. The post-cracking slope was taken as 0.01\( E_c \) to account for nonlinear shear deformations due to shear-flexure interaction ([13], [1]). Beams and columns were defined as elastic beam-column elements with rigid end zones and plastic hinges at member ends. Elastic effective stiffness of 0.3\( E_c \) was used for both beams and columns [14]. Beam moment-rotation hinges were modeled using tri-linear backbone curves, whereas for the column plastic hinges, moment-axial capacity interaction curves were calculated using actual material properties. Potential impact of reinforcing bar slip/extension was modeled explicitly by adding nonlinear moment-rotation springs at the base and top of the columns, as well as at the beam-column interfaces, with stiffness values of \( (M_i/\theta_j) \). The contribution of slip/extension was estimated using the approach recommended by [15], and described by [16]. Slip/extension deformations in the walls were neglected because they generally do not contribute significantly and are typically more important for low-rise walls than for slender walls [14]. To include the effects of cyclic loading in stiffness reduction, cyclic degradation was modeled in the reinforcing steel behavior, as well as in the beam moment-rotation hinges as described by [16]. Strength degradation in beams and columns were modeled based on the backbone parameters recommended by ASCE 41-06. Strength loss interaction was integrated to the model such that strength loss in positive direction also resulted in strength loss in negative direction, and vice versa.

The UPT walls were modeled using fiber shear wall elements, similar to the RC walls, but with modifications to capture the gap-opening behavior at the wall-foundation interface and to account for the different material properties. The confined concrete stress-strain relationship was defined based on the Razvi and Saatcioglu model [17]. Elastic uncracked shear behavior was defined \( (G_c=0.4E_c) \) since the majority of lateral displacements in UPT walls is attributed to rocking at the wall-foundation interface and contributions of wall shear deformations are expected to be small. The unbonded PT steel and the unbonded length of the energy dissipating (D22) bars were implemented as inelastic bar (truss) elements, placed outside of the fiber section as strain compatibility is not enforced between concrete and steel over the unbonded lengths. A tri-linear force-deformation relationship that approximates the actual stress-strain relation of the PT and mild steel was assigned to the truss elements. The prestressing force was simulated as an element load (initial strain) in the PT bar element. The gap-opening behavior at the base of the wall was modeled using elastic gap-hook bar elements with no tension strength; therefore, they act like compression-only springs and allow uplift in tension. In the PT building, beam mild reinforcement does not cross the beam-joint interface and moment capacity is only provided by unbonded post-tensioning steel. This connection allows a gap to open and close at the beam-column and beam-
wall interface, producing a nonlinear elastic moment-rotation behavior (provided that the PT steel does not yield). Similar to the UPT walls, UPT beams were modeled using inelastic fiber beam sections and horizontal inelastic truss elements with initial strain to model the unbonded PT steel. The beam fiber segment closest to the joint consists only of concrete fibers with zero tensile strength to implement the gap-opening behavior. As a rigid diaphragm assumption would not allow the horizontal truss elements to precompress the beam or elongate, the slab was explicitly modeled using elastic shell elements with a small value for effective bending thickness and the actual slab thickness (130mm) for effective membrane thickness. The columns were modeled using inelastic fiber column sections with the bonded PT steel included in the fiber section. A limitation of the approach is that the initial strain in the PT steel cannot be explicitly accounted for. The effects of the initial PT strain, namely precompressing the concrete section, delaying cracking and causing the PT to yield earlier, can only be accounted for indirectly: either by applying an axial compressive force equal to the initial PT force, or by artificially increasing the elastic modulus of the PT steel. The first approach closely matches the initial stiffness of the column section (from moment-curvature analysis) but overestimates yield curvature and ultimate moment capacity. The second approach matches the strength and yield curvature but underestimates initial stiffness. In terms of global responses in the wall direction both approaches gave nearly identical results.

Rayleigh damping of 2.5% at $0.2T_1$ and $1.5T_1$, where $T_1$ is the calculated first mode period, were used for the nonlinear response history analyses based on the recommendation of PEER/ATC Report 72 (2010). The seismic masses were based on the weight of the structures reported by [6]. Axial load ratios at the base of the walls were estimated to be about $0.04A_g f_c$ and less than $0.01A_g f_c$ for the RC and PT buildings, respectively.

**Comparisons of the Analytical Results with Test Results**

Figure 9 displays comparisons of the analytical results with test results for the RC and the PT models in terms of base moment vs. roof drift for the 25%, 50%, and 100% Kobe records. It is noted that only the global responses are presented here due to space limitations. Results indicate that for the RC building, for all three records, the overall load-displacement relation is reasonably captured, although overall stiffness is slightly underestimated for the 25% Kobe record, whereas strength degradation and peak lateral displacement are modestly overestimated for the 100% Kobe record. Additional factors that might address the discrepancies were identified as: (i) the parameters used to model strength deterioration and cyclic degradation; and (ii) effects of biaxial responses and torsion. Current modeling involves two-dimensional analysis for the shear wall direction; however, three-dimensional analysis is needed to investigate these issues. Future studies will also focus on refining the sliding models. A bilinear model could be used to account for the near rigid behavior prior to initiation of shear sliding (e.g. in 25% Kobe record). In addition, interpretation of the actual test data indicated that sliding stiffness significantly dropped once the concrete crushed and rebars buckled at the wall boundaries; however, Perform 3D is not capable of modeling sliding behavior that is coupled with wall bending behavior. An alternative computational platform might be used to overcome this issue.

Fig. 9(d) indicates that the analytical model for the PT building appears to accurately predict the initial stiffness of the building. The model also provides a good estimate of the hysteretic response under the 50% Kobe record (Fig. 9(e)). In particular, stiffness, strength and peak displacements are reasonably well predicted. Finally, for the 100% Kobe record (Fig. 9(f)),...
peak displacements are accurately predicted, and the model also captures the softening and stiffness degradation that is apparent in the test results. However, the model tends to recover the initial stiffness at small drifts and thus exhibits a more pronounced flag-shaped response compared to the test results. A possible explanation is related to bidirectional effects that are not captured in the 2-D model which only considers excitation and response in the wall direction. It is noted that under the 100% Kobe record, concrete crushing was observed at the base of the PT columns, and was mainly associated with the frame direction response where interstory drift ratios close to 4.0% were measured in the first story (as opposed to 1.5% in the wall direction).

![Graphs showing comparisons between model and test results for different Kobe records.](image)

**Figure 9.** Comparison of analytical results with the test results at (a) 25%, (b) 50%, (c) 100% Kobe for the RC building; (d) 25%, (e) 50%, (f) 100% Kobe for the PT building.

**Conclusions**

Detailed modeling studies related to the December 2010 tests of two, full-scale, four-story buildings that were tested on the NIED E-Defense shake table are presented along with a brief summary of the tests. Ability of current nonlinear modeling techniques to capture the lateral load versus roof displacement relations were assessed by comparing experimental and analytical results. Analytical results revealed that the RC model was capable of adequately capturing the responses at the service-level events, although some discrepancies were observed for the 100% Kobe record. Future studies will focus on (i) refining the sliding models, (ii) sensitivity of the parameters used to model strength degradation to better capture responses at collapse-level events where significant strength loss and stiffness degradation were observed, (iii) effects of biaxial responses and torsion, and (iv) assessment of local responses. For the PT building, it was found that a model consisting of inelastic fiber cross sections for the walls, beams and columns provided reasonably accurate predictions of the measured global response in the wall direction. Future studies related to the PT building will focus on assessing local responses, modeling of the frame direction to assess bidirectional effects, and comparisons of experimental and analytical results for the Takatori records.
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References