PSEUDO-DYNAMIC TEST OF FULL-SCALE RCS FRAME: PART I – DESIGN, CONSTRUCTION, TESTING

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ABSTRACT

A full-scale three-story, three-bay composite steel-concrete frame was pseudo-dynamically tested under ground motions representing four earthquakes of varying hazard levels, with imposed story drift ratios up to about 5.5%. The frame was then subjected to a quasi-static pushover, which imposed story drifts up to 10%. Overall, the test demonstrates that the composite frame performs as implied by building provisions, exhibiting very little damage under frequent earthquakes, controlled and repairable damage under the design earthquake, and collapse prevention under the maximum considered earthquake. This paper summarizes the planning, design, construction, and execution of the frame test, including a summary of the test results. Comparisons with analytical simulations and design implications are discussed in a second companion paper.

INTRODUCTION

This is the first of a two-part paper describing a pseudo-dynamic test of a full-scale three-story three-bay composite steel concrete frame, conducted through international collaboration between researchers in Taiwan and the United States. The test specimen is an RCS moment frame consisting of reinforced concrete (RC) columns with composite steel (S) beams. The frame measures 12 meters tall and 21 meters long, making it among the largest frame tests of its type ever conducted. The three-story prototype structure is designed for a highly seismic location, representative of the earthquake hazard in coastal California and Taiwan. The design is based on provisions for composite structures in the *International Building Code* (IBC, ICC 2000) and other standards referenced by the IBC. The frame was loaded pseudo-dynamically using input ground motions from the 1999 Chi-Chi and 1989 Loma Prieta earthquakes, scaled to represent seismic hazard levels with 50%, 10%, and 2% chance of exceedence in 50years. Following the pseudo-dynamic tests, quasi-static loads were applied to push the frame to interstory story drifts up to 10 percent, which provides data to validate simulation models for large deformation, so-called "collapse prevention", response.

This paper reviews the design, construction, and testing of the composite RCS frame. Reported test results include overall load-deformation response and damage patterns observed under each of the earthquake loading events. The companion paper (Cordova et al. 2004) summarizes design implications and validation of the analytical simulation models using the frame test results.

Background on RCS System

RCS systems gained popularity in the United States in the late 1970's and 80's as a variation of conventional steel moment frames in mid- to high-rise buildings in low seismic zones (e.g. Houston, TX). During the same period, Japan also began to develop and utilize similar systems in low-rise structures as an alternative to conventional reinforced concrete construction. The primary motivation for RCS systems lies in their optimal usage of the structural steel and concrete. Reinforced concrete columns offer a significant cost-advantage over structural steel for resisting compressive column loads (Griffis 1992), while composite steel beam and composite slab-decks provide efficient floor framing systems for long spans desired in commercial buildings.

Innovative construction and staging operations add to the attractiveness of RCS construction by reducing the cost and construction time. A typical high-rise construction sequence utilizes small steel erection columns to advance steel framing several floors ahead of placing reinforced concrete columns. An alternative precast construction method has been applied to low-rise buildings in Japan. In this scheme, the steel beam is cast integral with the column and field spliced a short distance away from the column face. Variations to these methods, such as utilizing the column reinforcing bar cage as the erection column have been developed and used in Japan.

Beam-Column Connections

From the standpoint of seismic design, a significant advantage of RCS systems lies in the design and performance of the beam-column connection. Shown in Fig. 1 is a typical connection where the steel beam passes continuous through the joint, thereby avoiding interruption of the beam at the column face and eliminating the need for welding or bolting the beam at the point of maximum moment. This type of connection detail avoids the fracture problems of conventional steel moment frames that were encountered during the 1994 Northridge and 1995 Hanshin earthquakes. It also avoids reinforcing bar anchorage and congestion issues common in seismically-detailed reinforced concrete structures. Subassembly tests of composite connections have confirmed that, when properly detailed to provide force transfer between the steel and concrete, the composite RCS connections can provide sufficient strength to develop the beam plastic moment and reliable hysteretic behavior (Deierlein and Noguchi 2004).

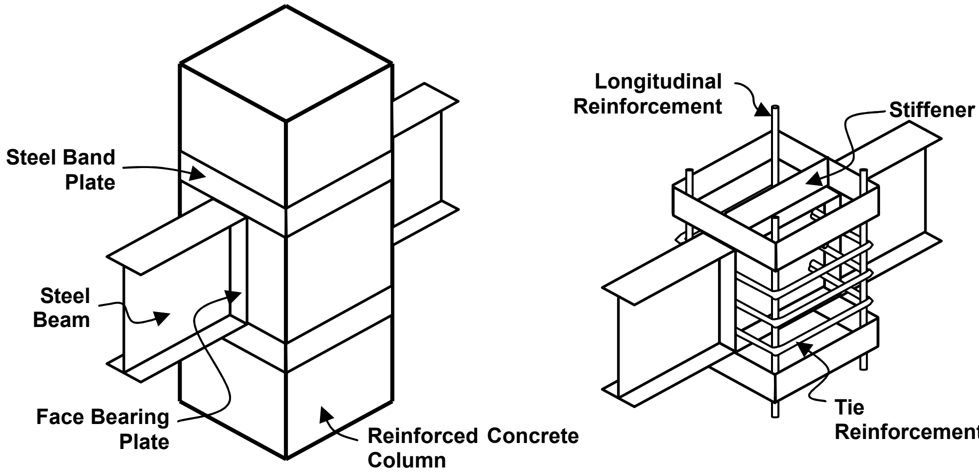


Figure 1 – Connection between Steel Beam and Reinforced Concrete Column

The detail in Fig. 1 utilizes several design features, which tests have shown to be particularly effective at mobilizing force transfer mechanisms between the steel beam and reinforced concrete column. These include face bearing plates, which develop a diagonal shear strut mechanism within the beam flanges, and steel band plates, which confine the concrete above and below the beam and help mobilize concrete outside the beam to resist joint shear. Ties within the joint provide concrete confinement and stabilize the vertical column reinforcement. Equations to quantify the strength and stiffness of composite connections were first presented in the 1994 ASCE guidelines for design of joints between RC columns and steel beams. Subsequent research has lead to proposals to improve the accuracy and robustness of the ASCE joint model (e.g., Kanno and Deierlein 2002 and Parra-Montesinos 2000).

System Design Guidelines

Apart from detailing of the composite joints, design requirements for composite frames follow directly from those of conventional reinforced concrete and structural steel construction. RC columns are designed per ACI-318 (2002) and the steel (or composite) beams are designed by the AISC-LRFD Specification (1999). General seismic loading and design requirements are provided in the IBC (ICC 2000), which adopts specific detailing requirements for composite special moment frames in Part II of the AISC Seismic Design Provisions (1997). Further details on the development of seismic building code provisions are summarized by Deierlein (2000).

Several groups of investigators have developed trial designs of RCS frames based on typical US building layouts (Mehanny et al 2001, 2002, Bugeja 1999). These trial designs have been an effective mechanism to exercise proposed seismic design provisions for RCS systems and then, through nonlinear analyses, to evaluate the performance of these systems. In general, these investigations have shown that the inelastic dynamic response of the RCS frames is similar to comparably designed steel moment frames.

Rationale behind Full-Scale Test

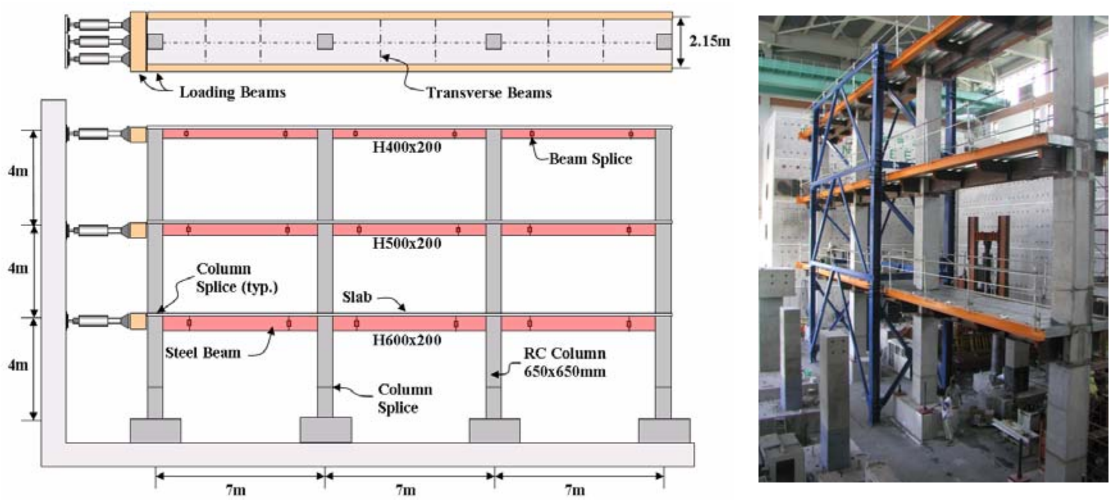
While RCS moment frame systems have been used for nearly twenty years, most applications in the U.S. have been limited to high-rise buildings in low seismic regions. There has been research (e.g., Kanno 1993, Mehanny 2001) to show this system to be equivalent, if not superior, to the seismic behavior of all-steel moment frames. Nevertheless, the engineering community has been hesitant to adopt this system in high seismic regions such as California. One of the primary motivations of the full-scale RCS frame test is to serve as a "proof of concept" for innovative composite moment frames as alternatives to conventional steel and concrete systems. In addition to this, the test will provide data to evaluate and validate important design provisions for composite moment frames. Particular topics of investigation include strong-column weak-beam criterion, composite action of concrete slab and steel beams, integrity of the precast column and composite beam-column connections, and overall system response characteristics that have not been evaluated in prior subassembly tests. The frame test also provides valuable data to validate models and computer codes for nonlinear simulation and performance assessment. In addition to these direct benefits, the large-scale test provides an opportunity to promote international collaboration and explore pseudo-dynamic test methods and data archiving envisioned for the NSF-NEES initiative (www.nees.org).

PLANNING AND DESIGN OF TEST FRAME

The design, construction, and loading of this test frame is meant to be as realistic as possible, within the constraints of the laboratory facility at the National Center for Research in Earthquake Engineering (NCREE) in Taipei. The size of the test frame is restricted by the 15-meter tall strong wall and the general space limitations of the lab. Based on these considerations, along with the project goals and budget, the frame proportions are based on a rectangular three-story office building with plan dimensions of 42 x 28 meters and a framing grid of 7-meter bays in each direction and a 4-meter story height. Lateral loads are assumed to be resisted by two framing lines in each orthogonal direction.

Referring to Fig. 2, the three-bay three-story prototype frame represents one of two lateral-load resisting frames in the short framing direction. Columns are pre-cast with the beam stubs and field spliced using grouted sleeve couplers. Column splices are located 1000 mm above the footing and directly above the 1st and 2nd floor slabs. Steel beams are spliced with bolted moment connections 1500 mm away from the column face. Bolted moment splices are designed per recommendations in FEMA 350 (2000) to develop 1.2 times the expected moment strength at the column face. Further discussion of design considerations for the column and beam splices and their effect on behavior is discussed in the companion paper (Cordova et al., 2004).

To model composite beam and floor diaphragm behavior, a 2150mm wide slab is integrated with the floor beams on each level of the frame. Shear studs are provided



(a) (b)

Figure 2 – (a) Plan and elevation of the full-scale composite frame specimen.
(b) Photo of RCS test frame and external steel brace frame.

on the longitudinal and transverse framing beams. Loads are introduced at each framing level through stiff loading beams, which extend the full length of the frame and are connected to the slab through horizontal shear studs. As shown in Fig. 2a, three actuators (each with a force and stroke capacity of 1000kN and $\pm$500 mm, respectively) are installed at each floor level.

The frame was intentionally designed to the minimum limits of the IBC provisions so as to represent the minimum expected performance and to interrogate system design parameters. As Taiwan's seismic design codes adopt similar requirements to those in the United States, the frame is equally representative of design standards in both countries.

Design Loading

The loading conditions for the design of the test frame are obtained from the IBC. Typical office dead and live loads are assumed to be 4.4 and 2.4 kN/m^2 (92 and 50psf), respectively. Seismic design forces are based on a highly seismic site with mapped spectral accelerations of S_s = 1.5g and S_1 = 0.72g using IBC 2000. The soil condition at the building site is assumed to be that of site class D (F_a = 1.0), while the building itself is assigned a Seismic Use Group I and a Seismic Design Category D according to IBC. The fundamental period of the frame as defined by the code is 0.56 second, whereas the theoretical period is roughly 1 second. Using T = 0.56 second, the frame falls within the constant acceleration portion of the design response spectra. Therefore, the total base shear is computed as follows:

$$V = \left(\frac{2/3\, F_a S_S}{R/I} \right) W \tag{1}$$

where the importance factor, I, is equal to 1.0 and W is equal to the effective seismic

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mass of the building. The IBC categorizes this system as a composite special moment resisting frame, which has an R-value of 8. Coupled with the required accidental torsional moment, the frame has a design base shear coefficient of 0.13, which equates to a total base shear of 1160kN.

Member Design

The steel beam sizes are controlled by strength in negative bending and, therefore, are sized as steel sections only. Shear studs are provided to develop full composite action in positive bending. The column design is governed by the strong-column weak-beam criterion of ACI 318 (2002), which requires that the sum of the nominal flexural strengths of the columns framing into a joint should be greater than the 1.2 times the sum of the beam strengths. This is an important design issue for this frame and is discussed further in the companion paper (Cordova et al., 2004). The *IBC* interstory drift criterion [$C_d \Delta_d = \Delta_{in} \leq 0.02$(story height), with C_d = *5.5*] is met using the members designed for strength. Therefore, the drift criterion does not govern the design. The final member sizes and properties are presented in Table 1.

Table 1 – Test frame member properties.

Floor	Steel Beams (F_y = *345 MPa*) (d, b_f, t_w, t_f)	RC Columns (f'_c= *40 MPa*)		
		Section	Rein. Bars (F_y = *410 MPa*)	
1^{st}	H600x200x11x17mm	650x650mm	Exterior Interior	8-#11bars 12-#11bars
2^{nd}	H500x200x10x16mm	650x650mm	Exterior Interior	4-#11bars 12-#11bars
3^{rd}	H396x199x7x11mm	650x650mm	Exterior Int. Lower Int. Upper	4-#11bars 12-#11bars 8#11bars

Joint Design

The composite joints were designed using the strength model of ASCE Joint Recommendations (1994) to develop the nominal moment capacity of the composite beams. The joint details are similar to the design shown in Fig. 1, except that the joints had moment-resisting beams framing in the two orthogonal directions. The orthogonal (transverse) members were provided to (a) mimic space-frame construction practice that is common in Japan and Taiwan, and (b) to provide support for the composite slab and longitudinal loading beams. As shown in Fig. 4, the transverse beam detail provides additional concrete confinement to the joint region and increases the strength and stiffness of this connection. However, the transverse

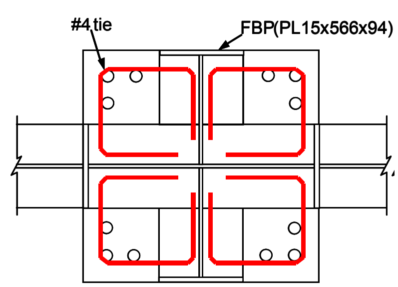

Figure 4 – Joint tie configuration.

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beam also complicates tie detailing in the joint. Prior to finalizing the frame design, effectiveness of the tie configuration shown in Fig. 4 was demonstrated through composite joint subassembly tests at NCREE, which confirmed that the detail provided sufficient anchorage to the transverse ties. The top floor joints posed another complication, since there are no design guidelines or tests of roof joints where the column does not extend above the joint. This challenge was met with a straightforward modification to the joint detail that is described in the companion paper (Cordova et al. 2004) and was verified by subassembly tests conducted at NCREE prior to the frame test.

Test Loading Protocol

Using the pseudo-dynamic methodology, lateral loading is applied to simulate four earthquake loading events, based on the following two strong motion records: TCU082-EW from the 1999 ChiChi earthquake and LP89G04-NS from the 1989 Loma Prieta earthquake. These records are used in a series of four loading events, scaled according to the following hazard levels: a frequent earthquake with a 50% chance of exceedence in 50 years (50/50) using the TCU082 record, a 10/50 design level earthquake with the LP89G04 record, a 2/50 maximum considered earthquake with the TCU082 record, and, finally, a repeat of the 10/50 LP89G04 design level event. After the four earthquakes, a final pushover using a triangular loading pattern is applied out to a static roof drift ratio of 8 percent.

For pseudo-dynamic loadings, the input ground motion records are scaled based on the spectral acceleration at the theoretical first mode period of the building (T = 1 second) for the specified hazard levels. Spectra for the three earthquakes are compared in Fig. 5. Note that because of differences in the hazard curve for the assumed building sites in the US and Taiwan, the spectrum for the 2/50 record lies below the 2/50 hazard curve implied by the IBC design loads. The hazard level corresponding to the maximum considered earthquake for a site in the US is about equal to a 4/50 (versus 2/50) hazard level. Aside from the earthquake ground motion, the other input to the pseudo-dynamic algorithm relates to the seismic mass and gravity loads from the prototype building. The seismic mass is the same as that assumed for design, and the gravity loads are included through the geometric stiffness (P-Δ) terms of the pseudo-dynamic algorithm.

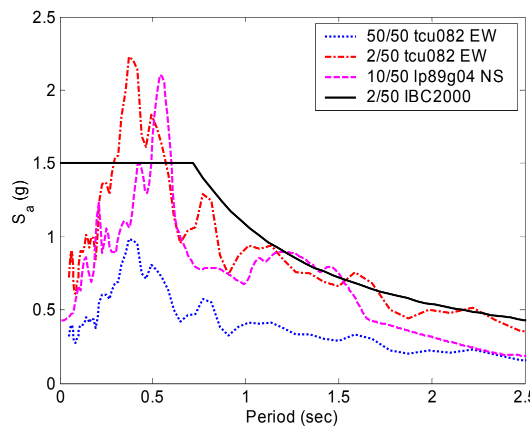

Figure 5 – Scaled response curves for earthquakes used in test simulation.

Testing techniques – ISEE

Internet-based Simulations for Earthquake Engineering (ISEE) has been developed by the researchers in NCREE as a prototype of Internet-based cooperative structural

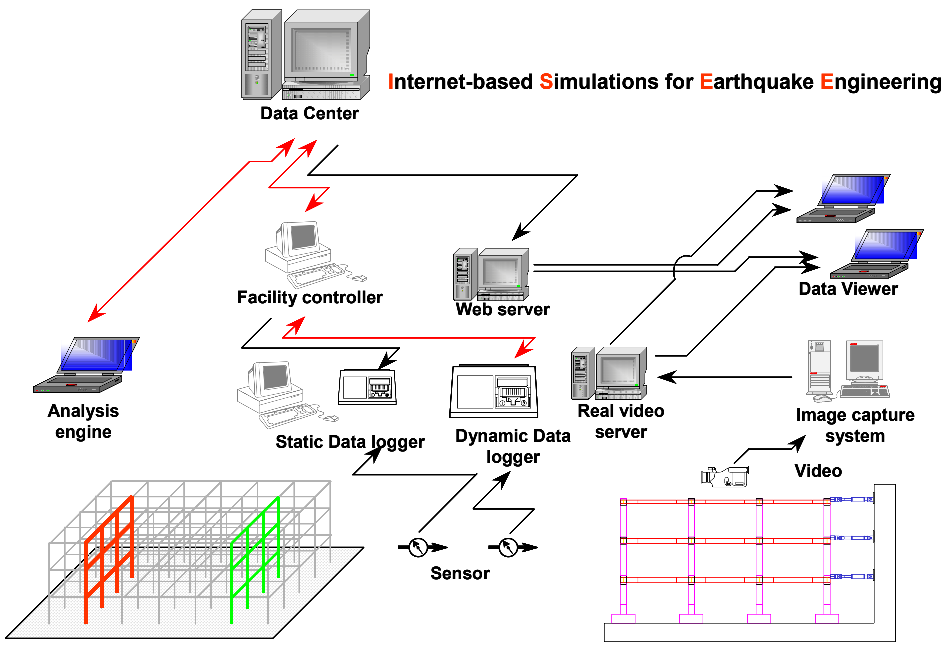

Figure 6 – ISEE framework employed in the pseudo-dynamic test

experimental environment [Yang et al. 2002]. During the full-scale RCS frame test, ISEE was activated to allow remote participants witness the real time video images of the specimen in the laboratory as well as the digital response data through the Internet. The schematic of the ISEE framework configured for this study is shown in Fig. 6. There are three major components in this framework: the Data Center, the Analysis Engine, and the Facility Controller. The Data Center is a database server, which processes all the prescribed data sent from the Analysis Engine and the Facility Controller. The Analysis Engine handles the numerical integration of the dynamic responses of the entire system (analytical plus physical components) using Newark explicit scheme with a time step size of 0.02 second. The Facility Controller is the software bridging the experimental facilities and the Data Center. When the target displacement is satisfactorily imposed on the specimen, instruments record and send the data (about 300 channels) to the data logger while concurrently sending prescribed response data (floor displacements, story shears, etc.) back to and the Data Center for real-time Internet distribution. The appropriate information is also sent to the Analysis Engine to compute the target displacement for the next time step. An independent Real Video Server broadcasts video images which allow data viewers anywhere in the world to witness the experiment via the internet.

CONSTRUCTION OF TEST SPECIMEN

The test frame was detailed and built in the precast method of construction in conformance with standard industry practice. This type of precast construction is commonplace in Taiwan and Japan, and therefore was the method of choice for this test frame. The choice of precast construction also provided the opportunity to test strength and durability of the precast splice connections, which have not been widely investigated in RCS frames prior to this experiment.

Construction Process

At the precast fabrication plant, single story precast column assemblies were integrally cast with steel beam stubs to create a beam-column tree that can be seen in Fig. 7a. The beam stubs are fabricated to accommodate the bolted splice connections, and the precast column is outfitted with standard grouted splice couplers. The upper region contains the embedded steel beam as well as the standard details shown in Fig. 1 (including a transverse beam). Longitudinal reinforcing bars protrude beyond the top of the column to allow for the grouted connection of the upper column. The foundation-column stub assemblies were also prefabricated in the same manner.

(a) (b)

Figure 7 – Construction photos of (a) a typical pre-cast beam-column module and (b) the completion of the first floor.

The construction process began with placement and anchorage of the foundation blocks, followed by (1) installation of the beam-column trees, (2) erection and splicing of the steel beam spans, (3) erection and attachment of longitudinal loading beams to the transverse beams, (4) plumbing of the story, followed by tightening of the steel bolts and grouting of the column connectors, and (5) lay out and attachment of the steel deck. This process was repeated until all three floors were completed, with each floor requiring about 2 days worth of work. Figure 7b is a photo of the finished first floor. After the full frame was constructed, shear studs were installed and the slab reinforcement and concrete was placed at each floor.

It is worth noting that during construction, shear studs were inadvertently placed within the expected beam hinge regions on the 2nd and 3rd floor beams. This was a mistake, which violated the design intent to follow the FEMA 350 requirements that no shear studs be placed within the hinge zone (one-half beam depth from column face). After considering remedial options, it was designed to leave the studs in place, recognizing that attempted removal may cause more negative effects than leaving the studs in place. In the end, it was reassuring and interesting to see that the shear studs did not impact the hinge behavior, in spite of the 3rd floor beams undergoing rather large hinge rotations.

TEST FRAME RESULTS

The frame was tested at the NCREE lab in October of 2002 and witnessed by about thirty researchers from the US, Taiwan, and Japan. Testing was broadcasted live via the Internet (http://rcs.ncree.gov.tw), including real-time data plots and live video. The response of the frame was monitored and documented with over 300 data channels, visual inspections, and photographic images.

General Response

Examples of the recorded test results during the pseudo-dynamic earthquake loading are shown in Fig. 8, which includes plots of the roof displacement history throughout all events (Fig. 8a), the maximum and minimum interstory drift ratios (Fig. 8b), and the maximum and minimum story shears (Fig. 8c). During the 50/50 TCU082 event the roof experienced a maximum displacement of about 200mm, with fairly uniform interstory drift ratios that ranged from 1.5 to 2.0%. A maximum base shear of about

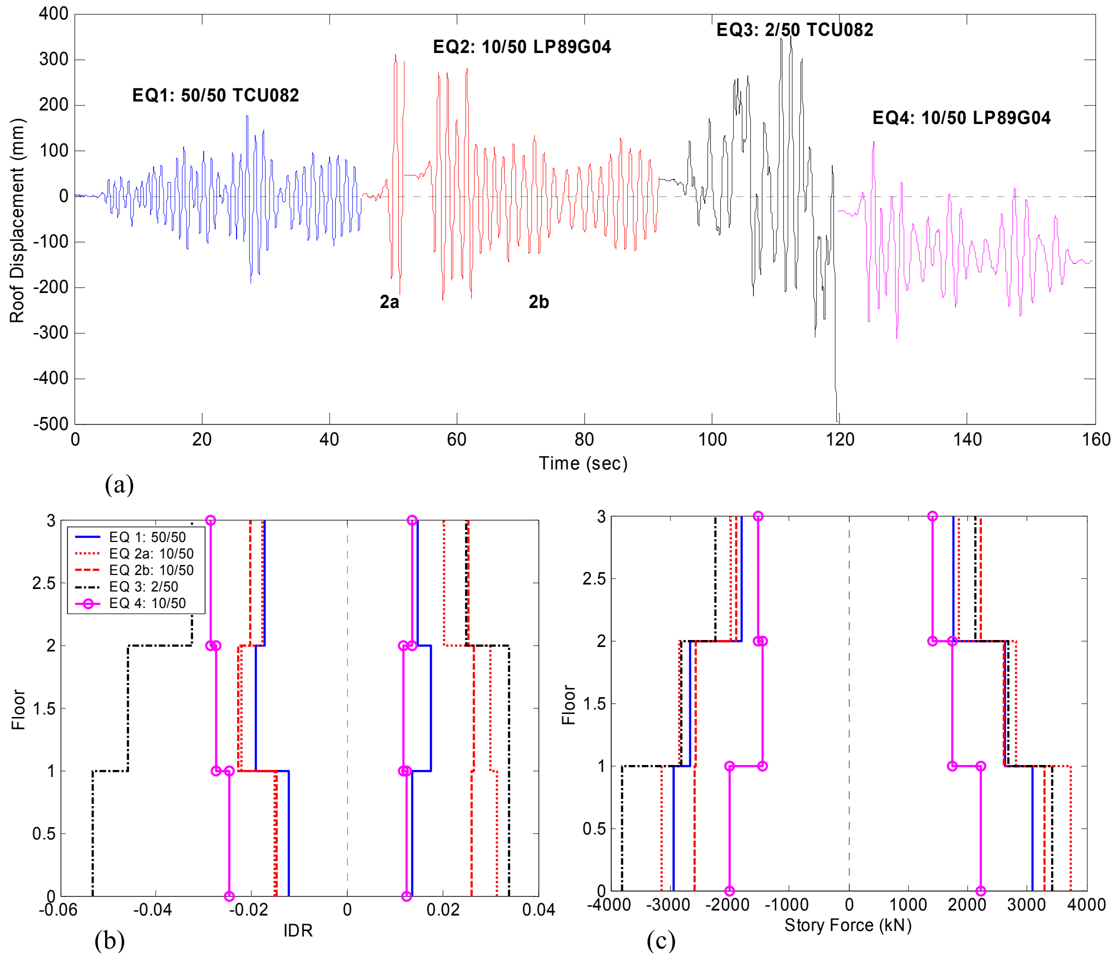

Figure 8 – Global response of frame to earthquake excitations. (a) Roof displacement, (b) Max/min IDR, and (c) Story force.

3000kN occurred during the 50/50 event, which is about 2.6 times that of the design value (1160kN). The residual drifts resulting from this event were negligible.

Due to a problem with the out-of-plane bracing, the first design level event (10/50 LP89G04) had to be stopped at about 7 seconds into the record and then restarted from the beginning (see point 2a in Fig. 8a). The maximum drift of 3% observed in this segment (2a) caused significant hinging at the column bases. The resulting loss in stiffness triggered an unexpected failure in the lateral bracing frame, which permitted the RCS frame to rock out-of-plane to a roof drift ratio of about 1.5%. Thus, in addition to base hinging associated with the in-plane drift of about 3%, the column bases experienced some additional out-of-plane hinging associated with the 1.5% out-of-plane drift. After surveying the damage, the out-of-plane bracing was repaired to permit restarting the 10/50 earthquake event. However, being as the frame had already experienced the major excursions of the first 10/50 event (segment 2a), it was decided to rerun the 10/50 event scaled down to 80% of its original intensity. This was done with the assumption that the two events (segments 2a and 2b) would represent the intensity and damage equivalent to that which would be imposed by the full 10/50 design level event. Ultimately, the maximum roof displacement that occurred during this design level event was about 300mm, with maximum interstory drifts ranging from 1.5% to 3.0%. The base shear was approximately 3800kN, which is about 3.3 times larger than the design base shear; and the residual roof drift was approximately 0.3%.

Under the maximum considered earthquake (EQ 3: 2/50) the frame experienced a maximum roof displacement of 500mm at about 28 seconds into the record. This displacement exhausted the maximum stroke of the actuators, so the pseudo-dynamic event was again halted. Examining the pseudo-velocities and accelerations of the frame at this time step, it was determined that the frame had reached its maximum drift and was beginning to unload. Analytical simulations (described in the companion paper, Cordova et al. 2004) further confirmed that this was the maximum excursion of the 2/50 event and that subsequent cycles were smaller. Given this information, it was decided that the loading up to this point was a reasonable representation of the maximum considered earthquake, even though the frame was subjected to only 28 of the full 45 seconds of the record. During the 2/50 loading, deformations began to concentrate in the first two stories of the frame, with a maximum IDR of 5.5% occurring in the first floor. The maximum base shear remained about 3800kN, and the residual roof drift was about 2.7%, with a largest contribution from the first floor (3.4%). Given the large amount of residual drift, there was concern that the frame would hit the maximum stroke of the actuators during the final 10/50 pseudo-dynamic test. Therefore, following the 2/50 event, the frame was straightened using the actuators to reduce the residual drift to approximately 0.3%.

The final pseudo-dynamic test was a repeat of the 10/50 earthquake, using the 80% scaled value so as to enable a direct comparison with the prior loading, represented by segment 2b in Fig. 8a. Aside from offering insight on the effect of accumulated damage, this test was intended to provide some information related to performance (safety) to aftershocks. The maximum roof drift attained during this last pseudo-dynamic loading was about 300mm, with interstory drifts remaining within 3%. The base shear was considerably lower in this event than in the previous three, with values peaking at approximately 2200kN. The residual roof drift was about 1.1%.

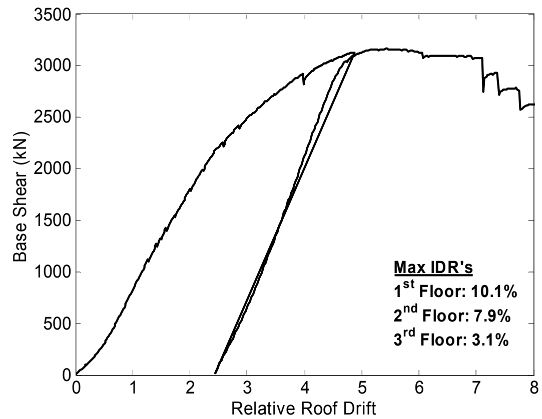

Figure 9 – Final pushover

The final pushover of the frame resulted in a maximum base shear of 3200kN (Fig. 9), which is about 2.8 times that of the design base shear. This reveals that even after sustaining significant damage through four major earthquakes, the frame still maintained a very large overstrength value. Deformations continued to concentrate in the first and second floors, creating a fairly pronounced two-story mechanism (see Fig. 8b), characterized by column hinging at the base and beneath the second floor beams and flexural yielding of the first floor beams. Hinging in the second floor beams was limited, due in large part to the fact that the measured material yield strengths came in 40% larger than the specified minimum strength (484 MPa versus the minimum specified yield strength of 345 MPa). This behavior is discussed further in the companion paper by Cordova et al. 2004.

Description of Damage

The following is a brief overview of the observed damage after each test:

- **EQ#1 – 50% in 50 year:** There were very minor cracks observed within the 1st floor reinforced concrete columns at the base, some minor yielding of the steel beams, and virtually no damage to the concrete slab or composite joints. During this test, and throughout all subsequent tests, there was loud bolt-banging, associated with slippage of bolts in the beam splices. Aside from the loud sound, the bolt slippage and banging did not detrimentally affect the frame. Upon thorough inspection, researchers and engineers witnessing the test agreed that the frame met performance target for "immediate occupancy", where the structural stability was not compromised and the frame required little if any repair.

- **EQ#2 – 10% in 50 year:** Crack widths near the base of the 1st floor columns opened to about 2mm and were accompanied by some minor spalling of the cover concrete (see Fig. 10b). Minor cracks also appeared in the upper portion of the 2nd floor columns. The steel beams in all floors yielded (see Fig. 10a), although the 2nd floor beams experienced much less yielding than the other stories. Local buckles appeared in the 1st and 3rd story beams in the lower flanges and slightly into the web. The largest buckling distortions at the flange tips measured up to 15 to 25mm. The upper flanges did locally buckle due to restraint from the composite

(a) (b) (c) (d)

Figure 10 – Typical conditions of beams, composite beam-column connections, and base columns following (a, b) design level and (c, d) maximum considered earthquakes.

slab. Local buckling was more significant for beams adjacent to the exterior columns since there was less longitudinal restraint than provided around the interior columns. The composite joints experienced very minor cracking. At this stage, the building was considered to meet the "life safety" performance target, where the level of damage required repairs but had not significantly affected the stability of the structure. Repairs to the structure would likely involve epoxy injection of cracks, patching of spalled concrete, heat straightening of local flange buckles, and plumbing to reduce residual interstory drift.

- **EQ#3 – 2% in 50 year:** The base of the 1st floor columns had distributed cracks up to 5mm in width and significant spalling of the cover concrete (Fig. 10d). Large cracks measuring about 10mm opened up between the bottom of the column and the footing - presumably due to yield penetration of the longitudinal bars in the column footing. The cracks within the upper region of the 2nd floor columns grew to widths of 4mm and were accompanied by minor spalling just below the beam-column joint. The 1st and 3rd floor beams experienced extensive yielding and large local buckles of the bottom flange and web, with flange tip distortions up to 70mm (Fig. 10c). The 2nd floor beams exhibited only slight yielding beyond that observed under EQ#2 as the inelastic deformations began to concentrate in the columns directly beneath the beams. The 1st and 2nd floor slab experienced some local crushing on the interface of the slab and the column as well as cracking on all three floors. The composite joint regions were relatively undamaged, experiencing only minor cracking (Fig. 10c). With significant local damage and a residual drift of 3.4 % (140mm) in the first story, the consensus of observers was that the frame had reached its "collapse prevention" performance level, implying that the structure would need significant repairs (probably uneconomical and perhaps infeasible) to restore its strength and stiffness.

- **EQ#4 – 10% in 50 year:** The final pseudo-dynamic loading (EQ#4 – 80% of the 10/50 event) exhibited the same damage patterns as the maximum consider earthquake (2/50) and did not significantly intensify this damage.

- **Final Pushover:** As shown in Fig. 9, during the final pushover the deformations concentrated in the lower two stories of the frame, with a peak interstory drift ratio of 10% reached in the first story. By the conclusion of the test, there was severe hinging at the column bases, significant yielding and local buckling in the 1st floor beams, and significant flexural cracking at the top of the 2nd floor columns. The sudden strength drop apparent in Fig. 9 at a roof drift ratio of 7% (corresponding to about 9% interstory drift in the first story) was caused by a net section rupture in one of the lower beam flange splice plates for one of the 1st floor beam splices. This first rupture precipitated subsequent ruptures in neighboring splices, which are evident in the strength drops under continued loading (Fig. 9). Note that the beam splices were designed to develop the shear and moment associated with the expected strength of the beams at the column face, which turned out to just about equal the actual plastic beam moment based on the measured steel yield strength.

CONCLUSIONS

Past studies have demonstrated that when designed to current standards, composite RCS systems have the strength, stiffness, and ductility required to safely resist large earthquakes; and their performance is comparable to that of steel or reinforced concrete frames. Prior subassembly tests have shown that the beam-column connections can be detailed in a very simple and practical manner to provide sufficient strength and reliable seismic behavior. The full-scale RCS frame test described herein further validates the reliability of this innovative system. Designed to just meet the minimum requirements of current building code standards, the full-scale frame performed very well under four earthquake loading scenarios and a final pushover test out to an interstory drift ratio of 10%. Composite beam-column joints designed with standard details exhibited excellent behavior, as did the precast column splices. Overall, the test frame clearly demonstrates the capabilities of composite moment frames to meet and exceed the seismic performance expectations implied by modern building codes. The sequel paper (Cordova et al. 2004) further explores the design implications of this test as well as validation of the analytical models used within this study.

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