EXPERIMENTAL INVESTIGATION OF PROGRESSIVE COLLAPSE OF STEEL FRAMES UNDER MULTI-HAZARD EXTREME LOADING

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ABSTRACT :

Experiments were performed to evaluate the progressive collapse resistance of steel buildings considering multi-hazard extreme loading. Two 1/3 scale three-story, two-bay steel frames, a special moment resisting frame and a post-tensioned energy dissipating frame designed and previously tested for seismic performance on a shaking table, were adapted for quasi static collapse testing. The experiments simulated the structural response after the sudden failure of a column. The objective of the tests was to evaluate the effectiveness of earthquake resistant design details in enhancing the resistance to progressive collapse. In these tests, an effort was made to document the load resisting mechanism, the sequence of damage in the frames and correlate observed damage with changes in the resisting strength. The experimental results demonstrate significant vertical displacement capacity for both frame systems and the capacity to redistribute loads after the failure of a single column. However, the steel moment frame appears to have significantly more ductile response by maintaining its yield strength up to the point of first connection fracture. The vertical load carrying capacity of the special moment frame appears to be dependent on the rotation capacity of plastic hinges before buckling and fractures occur. The vertical load carrying capacity of the post-tensioned energy dissipating frame depends heavily on the performance and ultimate strength of the tendons used for connecting the beams to the columns. The PTED frame lost a significant amount of strength after failure of one of the tendon strands.

KEYWORDS: Experimental study, progressive collapse, steel frames, post-tensioned frames, multi-hazard loading.

1. INTRODUCTION

Since the collapse of the Ronan Point Towers in London 1968 and more recently spurred by the September 11 2001 events at the World Trade Center in New York City, the structural engineering community has focused significant efforts towards better understanding of the phenomena of progressive collapse in building structures. The ultimate goal is to establish rational and reliable methods for the assessment and the enhancement of structural resistance to extreme accidental events. Although design methodologies (such as the alternative path method and the tie force method) and analysis procedures to enhance resistance to progressive collapse are proposed in guideline documents issued by the U.S. General Services Administration (2003) and the Department of Defense (2005), there is a scarcity of experimental data to support the numerical modeling of building structures under extreme loads, particularly to the point of failure.

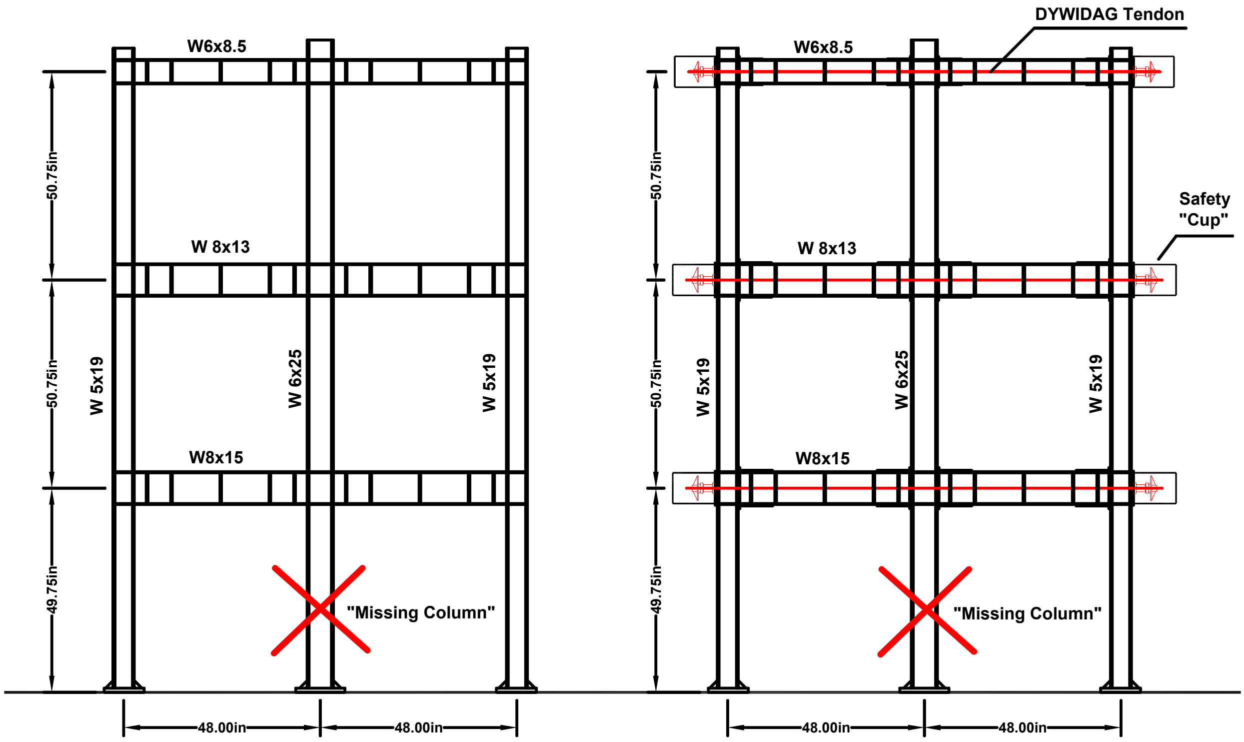
Several numerical studies have been published investigating the progressive collapse of buildings (Kaewkulchai and Williamson, 2004; Bazant and Verdure, 2007) and the adequacy of commercially available structural analysis software to perform collapse analyses (Marjanishvili and Agnew, 2006). The dynamics of impact of collapsing failed elements has also been investigated by Kaewkulchai and Williamson (2006). In a different approach to capture the dynamics of impact and other phenomena involved in a progressive collapse, Sivaselvan and Reinhorn (2006) proposed a method using a Mixed Lagrangian Formulation (MLF). In most cases, it is attempted to predict the global response of the building using simplifying assumptions without explicitly considering detailed but important phenomena, such as axial-flexure-shear force interaction in the beams, joint failure, local buckling and panel zone deformation (for steel structures). To address this need, Khandelwal et al. (2008) and Bao et al. (2008) proposed macro-models of beam-column subassemblies that capture the key local and global response characteristics by calibrating their models to results from detailed finite element analyses. Such detailed analyses, presented by Luccioni et al. (2004) or Khandelwal and El-Tawil (2007), enable the simulation of minute component effects but cannot be applied to whole buildings due to the large computational demand.

Experimental investigations of large scale building models or sub-assemblies are scarce in the literature. Karns et al. (2006, 2007) evaluated the resistance of different types of steel frame connections, initially subjected to blast loads and then pushed using monotonic loading, in order to determine their post-blast integrity for the purpose of mitigating progressive collapse. Recently, Sasani et al. (2008) designed and performed a series of quasi static collapse tests on 3/8 scale models of concrete frame beams. In-situ full scale tests were performed on existing concrete buildings by Sasani and Sagiroglu (2008), on steel buildings by Sezen (2007) and on masonry buildings by Zapata and Weggel (2008). All of the above followed the standard approach of the sudden loss of one or more exterior columns ("missing column scenario"), simulating an extreme accidental load, such as bomb blast or impact of a heavy vehicle. For the cases involving real buildings, the structures were able to redistribute the loads without the propagation of failure to additional members, thus the margin of safety against collapse could not be directly determined.

The purpose of this study is to experimentally investigate the progress of damage through collapse in steel frames when subjected to a missing column scenario and determine the mechanisms and capacity of the frames to transfer the loads to neighboring columns. The idea of investigating the influence of seismic detailing of a building on its progressive collapse resistance in a multi-hazard framework was presented by Hayes et al. (2005), in a case study of the 1995 terrorist attack on Alfred P. Murrah Federal Building in Oklahoma City. A key finding of that study was that strengthening the perimeter elements of the building using current seismic detailing techniques could have greatly reduced damage to the structure. In this perspective, a series of tests were conducted, following the missing column scenario, on two 1/3 scale three-story, two-bay frames: (i) a special moment resisting frame (SMRF) and (ii) a post-tensioned energy dissipating frame (PTED). The concept of the PTED frame has been developed and experimentally investigated by Christopoulos et al. (2002). The PTED frame has beams that are not welded or bolted to the column flanges and relies instead on the post-tensioning force provided at each floor by two tendons located at mid-depth of the beam for shear force transfer and for re-centering when the frame is subjected to lateral seismic loads. Four symmetrically placed energy dissipating (ED) bars located at each connection provide energy dissipation under cyclic loading as gaps at the beam-column interface open and close due to the rocking of the beam relative to the column. Both frames used in these experiments were previously tested for seismic performance on the shake table at the University at Buffalo Structural Engineering and Earthquake Simulation Laboratory (SEESL). The design of the frames and results of the seismic tests are presented by Wang (2007) and by Wang and Filiatrault (2008). The seismic study concluded that both SMRF and the PTED frames had similar performance in terms of displacements, with the PTED frame being superior in terms of limiting accelerations and damage to easily replaceable components. The same frames were used for this study to conduct experiments simulating the missing column scenario, in order to evaluate the progressive collapse resistance of steel frames with different seismic detailing.

2. EXPERIMENTAL SETUP

For the quasi-static collapse testing, the frames were installed on the 24 in. (60 cm) thick Reaction Wall and Strong Floor of SEESL (http://nees.buffalo.edu/facilities/facilities.asp). The loading simulation was implemented by means of a MTS servo-controlled actuator with a stroke of 40 in. (1016 mm) and a force capacity of 220 kips (978.6 kN). Due to the quasi-static character of the tests the actuator was used in its "static" configuration, using a low capacity servo-valve (SEESL, 2008). A 2 in. (5 cm) thick steel plate, attached to the strong floor, served as a base of the whole experimental setup. The two frames were equipped with thin steel sliding plates at the columns. Additionally, a support and sliding mechanism, consisting of eight steel pedestals with TEFLON sliding pads, was designed, in order to restrict the out-of-plane motion of the frames, while allowing the development of very large in-plane deformations. Schematics of the two frames tested are presented in Figure 1. A typical plan view and an elevation view of the experimental setup, as well as a detail of a single pedestal with its major components are shown in Figure 2.



(a) SMR Frame (b) PTED Frame

Figure 1: Schematics of the two frames tested.

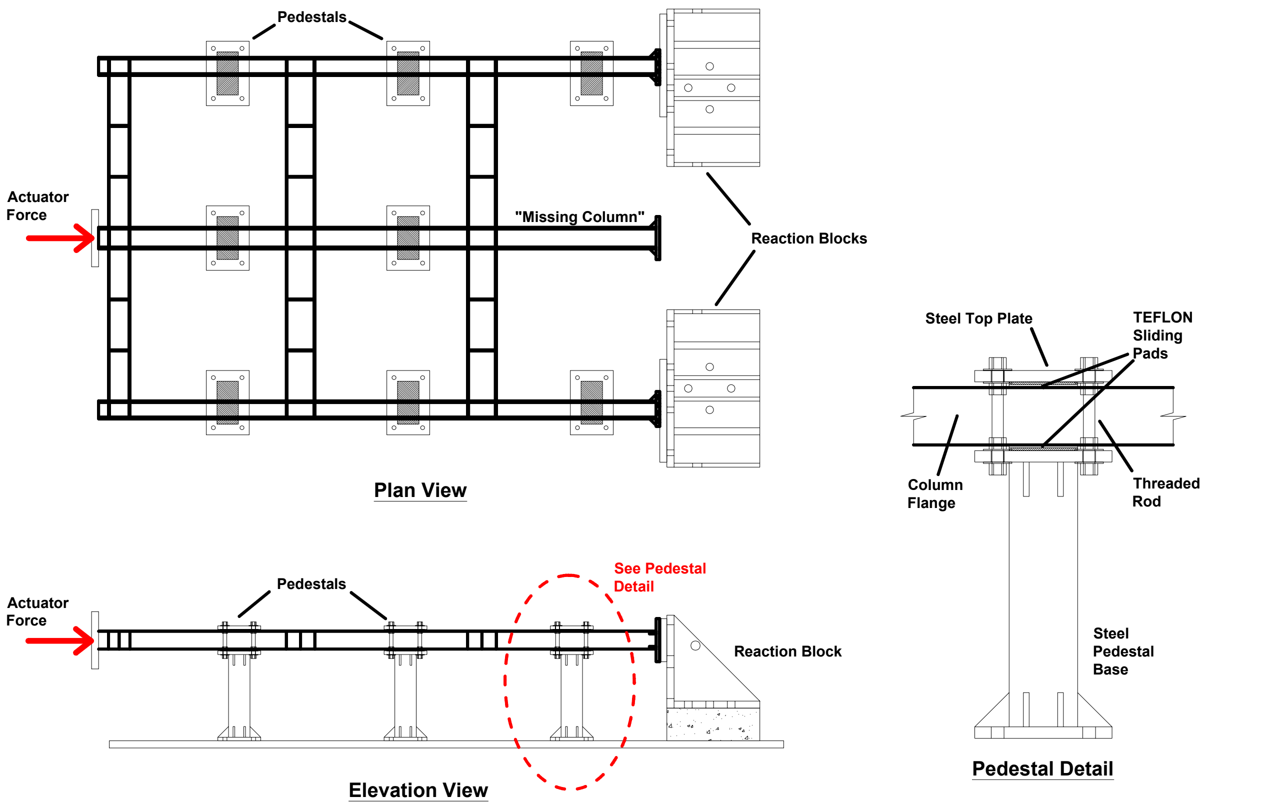


Figure 2: Plan (top) and elevation (bottom) views of the experimental setup. Sliding and support mechanisms are shown. A typical pedestal detail with the major components marked is shown on the right.

Standard wide flange "W" sections were used, in order to maintain geometric similitude to the full scale structure. In the case of the SMRF the beam to column connections were fully welded with fillet welds. No panel zone reinforcement plates were used. DYWIDAG Mono-Strand tendons were used for connecting the beams to the columns of the PTED frame. Two tendons were used at each floor with initial post-tensioning forces of approximately 20 kips (88.9 kN), 15 kips (66.7 kN) and 12.5 kips (55.6 kN) per tendon for the 1st, 2nd and 3rd floor, respectively. The beams of both frames incorporated web reinforcement plates and reinforced flange segments at both ends.

The exterior columns of each frame were connected to specially designed reaction blocks, consisting of a rectangular reinforced concrete base and an overlying steel block shaped as triangular prism. Each reaction block was firmly attached to the strong floor with 4 post tensioned DYWIDAG Threadbars. The central (interior) column of each frame was left unsupported, in an attempt to simulate the "missing column" scenario. In the case of the PTED frame, special safety "cups" were designed, in order to arrest the components of the tendons' anchorages, in the event of sudden failure of the tendons during testing. The frames were fully instrumented with uni-axial strain gauges, displacement transducers (string potentiometers) and a 3D imaging Krypton system, operating with infrared LED's (SEESL, 2008). All instrumentation channels were connected to a Pacific 6000 Data Acquisition System.

3. EXPERIMENTAL RESULTS

Low amplitude identification tests were initially performed on each frame for equipment check and calibration of system performance (e.g. calculation of initial "elastic" stiffness and comparison with analytic predictions). These preliminary tests were followed by a final "push to collapse", using slow displacement ramps with rates ranging from 0.1 to 0.2 in./min (2.54 to 5.08 mm/min). The data acquisition rate was set at 10 Hz, which was considered sufficient, given the quasi-static nature of the experiments. The most representative results of the "push-down" tests are the plots of the force applied by the actuator versus the displacement of the central columns of the frames. These plots are presented in Figure 3, both for the SMRF and the PTED frames. Figure 4 shows the change in the post-tensioning forces of the front tendons at each floor during the "push-down" test of the PTED specimen.

In the case of the SMRF, the frame behaved essentially elastic up to a force level of 135 kips (600.5 kN) and vertical displacement of 0.65 in. (17 mm). The elastic region was followed by a well defined yield plateau with minor hardening, up to the peak force. The maximum force resistance was 146 kips (649.4 kN) corresponding to a displacement of 6.35 in. (161 mm), resulting in a displacement ductility on the order of 10 for the SMRF. It is obvious from the plot that little or no significant hardening behavior occurred after the initiation of plastic hinge formation. A possible explanation for this is that the yielding of the connections' panel zones and the absence of restraints for the lateral movement of the side columns (geometric condition) provided the flexibility to accommodate these deformations. Following the peak force, major failures occurred at the central beam to column connections, starting from the sides of the central connection on the first floor and subsequently progressing to the second floor. The connection failures initiated as tensile cracks at the bottom beam flanges that propagated into the web. Weld failures were also observed. As deformations became larger, significant beam web buckling and panel zone shear deformations on the exterior columns were observed. The test continued until a final displacement of 19 in. (483 mm) approximately, at which the residual strength of the frame was 45 kips (200.2 kN) (i.e. 30% of the maximum). It should be noted that the applied displacement was controlled automatically up to 10 in. and manually – at increments of 0.2 in. (5.08 mm) – thereafter. Control issues associated with the manual approach are the cause of the oscillations observed in the plot after 10 in. (254 mm), likely due to stress relaxation of the yielded material in between load steps when the displacement is held constant. The condition of the SMRF after the test is displayed in Figure 5a. Detailed views of a central beam to column connection and a deformed panel zone are shown in Figure 5b and Figure 5c, respectively.

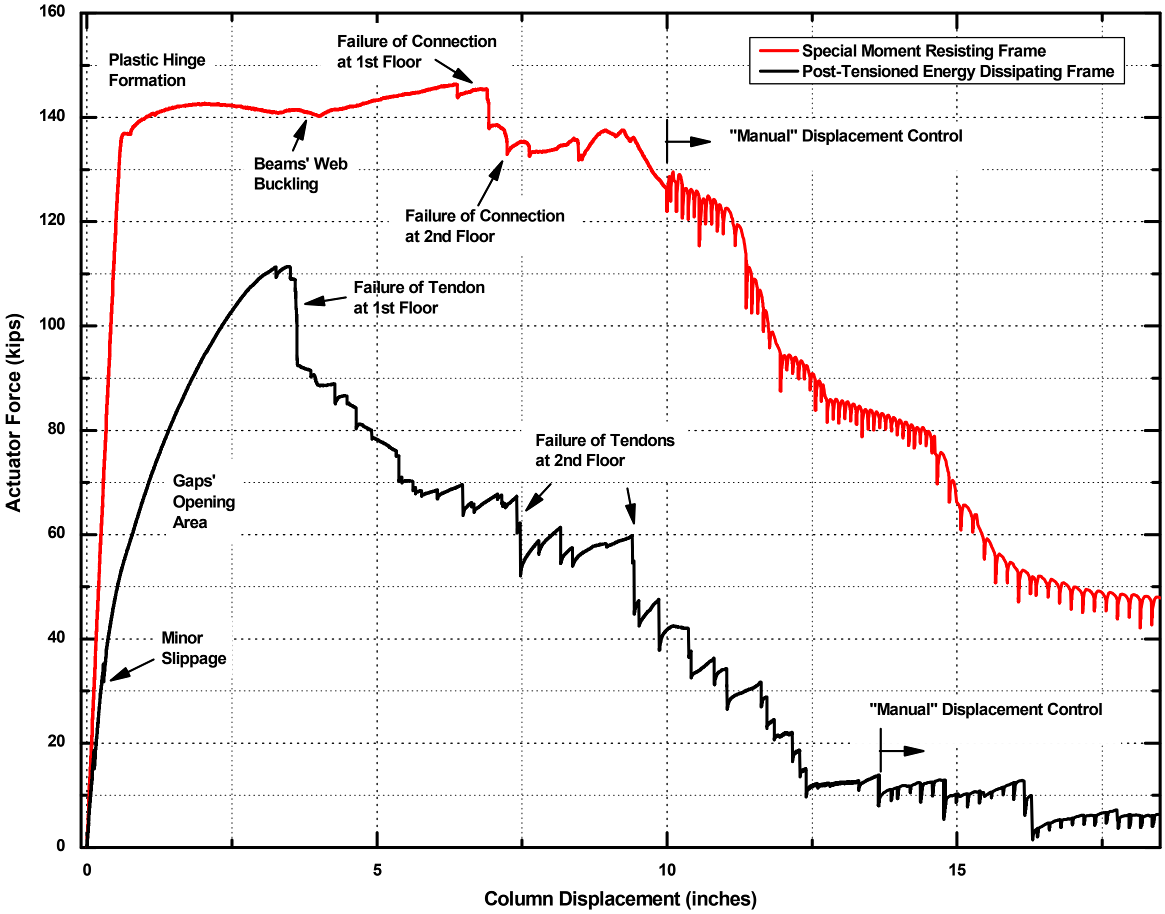


Figure 3: Comparative plot of applied force vs. displacement of the central column for the SMRF and PTED specimens and identification of key damage states.

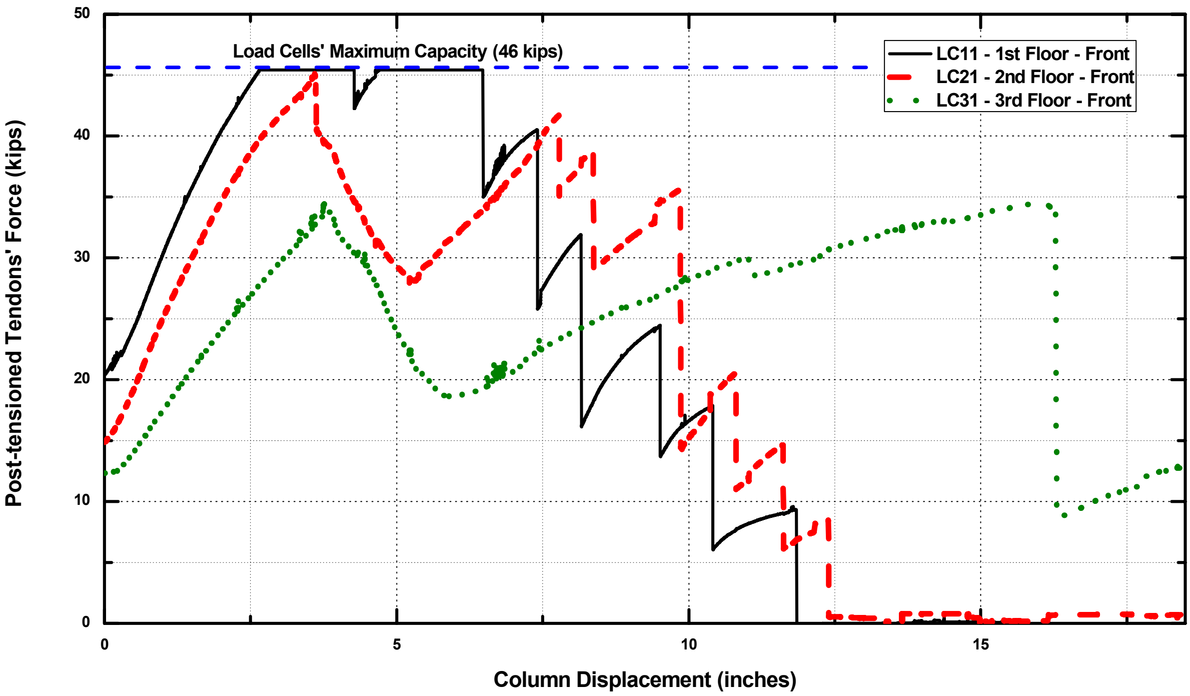
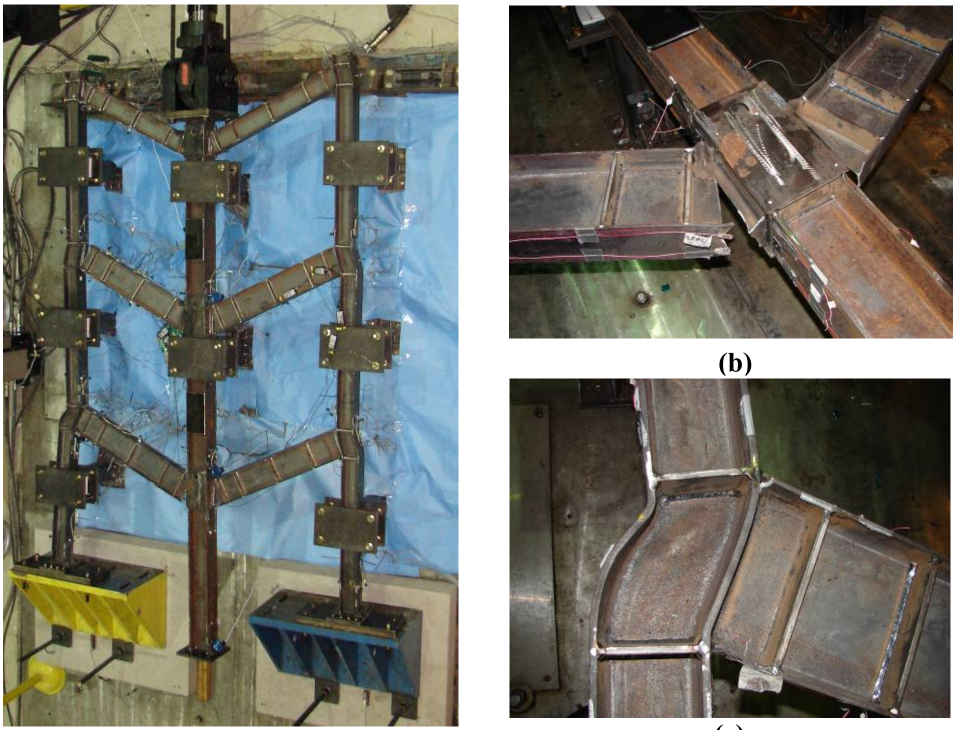
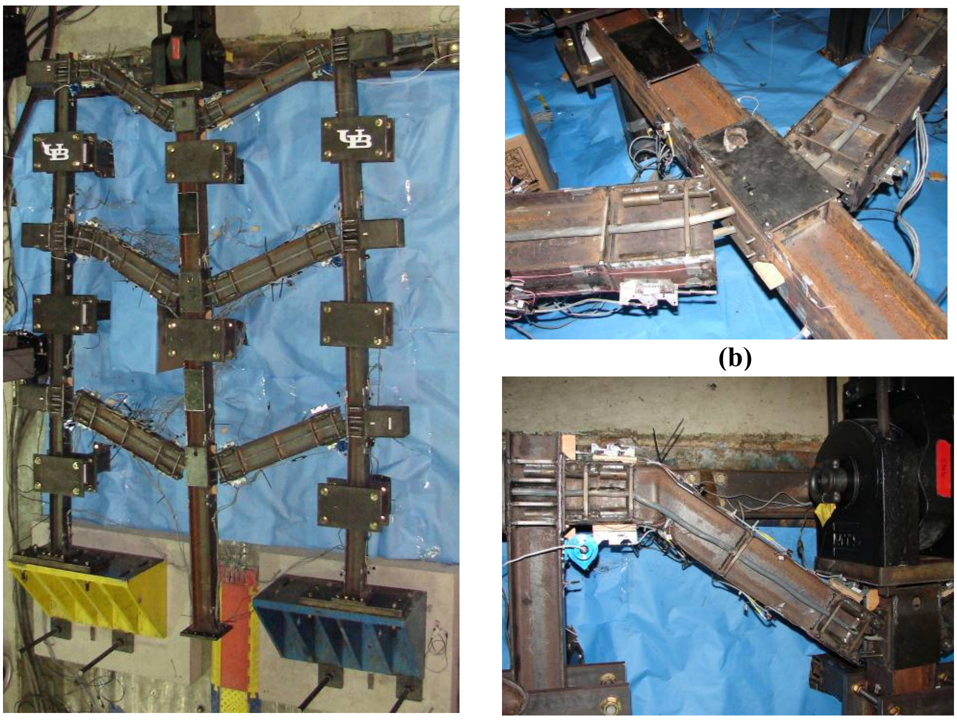


Figure 4: Comparative plot of the post-tensioned tendons' forces vs. displacement of the central column for the PTED specimen.



(a) (b) (c)

Figure 5: The SMRF after testing: (a) general view, (b) detail of 1st floor central connection, (c) close-up view of 1st floor panel zone deformation.



(a) (b) (c)

Figure 6: The PTED frame after testing: (a) general view, (b) detail of 1st floor central connection, (c) close-up view of 3rd floor post-tensioned beam buckling.

The force vs. displacement curve for the PTED frame showed significantly different behavior. Due to the non-linear behavior (gap opening) of the PTED frame, the elastic region cannot be well-defined. The resistance of the frame increased monotonically to 111 kips (493.7 kN) at a displacement of 3.5 in. (88.9 mm) with a slight stiffness softening. Up to that point, only minor slippage of the beams with respect to the columns was observed. With increasing displacement, a series of tendon ruptures, failures of the energy dissipating bars and formation of plastic hinges at the beams' ends led to a rapid decrease of the frame resistance. The residual "strength" of the PTED frame after the application of 19 in. (483 mm) of displacement was insignificant; less than 5 kips (22.2 kN). Figure 6a shows a general view of the PTED frame after the conclusion of the "push-down" test. A completely destroyed connection is displayed in Figure 6b. Finally, a close-up view of a buckled beam is shown in Figure 6c. It is noted that the buckling of the post-tensioned beam has occurred at the end of the reinforced zone, next to the interface with the column flange.

By comparing the second plot on Figure 3 (PTED test) with the development of the tendons' forces in Figure 4 it can be observed that all major drops in the strength resistance of the frame correspond to failures of single wires of the front tendons (each DYWIDAG Mono-Strand consists of seven bundled wires). Ruptures of single wires are represented by vertical force drops in Figure 4. The rear tendons followed similar force "paths" with minor deviations due to (i) different initial tension and (ii) rotations of the beams with respect to their longitudinal axes during the tests and (iii) random slippage of the beams.

Independent of the differences in the behavior of the SMRF with respect to the PTED, it is estimated that a single column failure is not sufficient to cause progressive collapse of these frames. The most severe dead plus live load combination for the specific frame geometry would be of the order of 30 kips (133.4 kN) (scaled). This is 3 times less than the capacity of the PTED and 4 times less than the maximum resistance of the SMRF – this means that a dynamic amplification factor (as the real phenomenon is not quasi-static) of at least 3 can be accommodated without danger for the structure.

4. CONCLUSIONS

Two seismically resistant frame systems, a SMRF and a PTED frame, were subjected to quasi-static push-down tests for determining their resistance against progressive collapse. Both frames behaved well in terms of being able to resist loads three times larger than the most severe dead and live load combination without significant damage. In the case of the sudden loss of a central column, both frames would have been able to support the remaining building gravity load and also have significant remaining deformation capacity. In this multi-hazard extreme loading case, the seismic design and detailing provided the strength and ductility needed to prevent a progressive catastrophic failure of the tested structures.

Compared to the SMRF, the PTED frame turned out to be weaker and less ductile, although both frames were designed for similar earthquake load resistance. This is primarily due to the very high quality of the fully welded beam to column connections of the SMRF, a detail that would not be typical of real-world construction. Another noteworthy aspect is the virtually complete absence of hardening in the case of the SMRF, after the onset of yielding. This may be attributed to the frame's boundary conditions that permitted the in-ward deformations of the columns and panel zone deformations to accommodate the beam rotations.

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