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EXPERIMENTAL INVESTIGATION OF THE PROGRESSIVE COLLAPSE OF A STEEL POST-TENSIONED ENERGY DISSIPATING FRAME

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ABSTRACT

Experiments were performed to evaluate the progressive collapse resistance of steel buildings considering multi-hazard extreme loading. As part of the experimental program a 1/3rd scale three-story, two-bay post-tensioned energy dissipating frame (PTED) designed and previously tested for seismic performance on a shaking table, was adapted for quasi static collapse testing. The experiment simulated the structural response of the building after the sudden failure of a base column. The scope of the test was to evaluate the effectiveness of earthquake resistant design details in the resistance to progressive collapse. An effort was made to document the sequence of damage in the frame and to correlate observed damage with changes in the resisting strength. The experimental results demonstrate significant vertical displacement capacity and the ability of the steel framing to redistribute loads after the failure of a single column. The vertical load carrying capacity of the post-tensioned energy dissipating frame depends primarily 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 the first failure in one of the tendon strands. A numerical simulation investigates the possibility to reproduce the experimentally measured response with commercial structural analysis software.

Introduction

The 1998 terrorist attacks against US embassies in Kenya and Tanzania and more recently the September 11 2001 events at the World Trade Center in New York City have motivated the structural engineering community to focus significant efforts towards better understanding the sequence of events leading to 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 already proposed in guideline

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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.

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 the 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 (Tsitos et al., 2008): (i) a special moment resisting frame (SMRF) and (ii) a post-tensioned energy dissipating frame (PTED). The latter test is presented in this study.

The concept of the PTED frame for seismic design 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. The ED bars are properly restrained so that they can yield both in tension and compression without buckling. A PTED frame was previously tested for seismic performance on the shake table at the University at Buffalo Structural Engineering and Earthquake Simulation Laboratory (SEESL). The results of the seismic tests are presented by Wang (2007) and by Wang and Filiatrault (2008). The seismic study concluded that the PTED frame had desirable re-centering capability after the maximum considered seismic event and was effective in terms of limiting accelerations and damage to easily replaceable components. The same frame with minimal seismic damage was adapted for this study, in order to conduct collapse experiments simulating the missing column scenario.

Experimental Setup

For the quasi-static "push-down" collapse testing, the PTED frame was installed in a horizontal position on the 24 in. thick reinforced concrete Reaction Wall and Strong Floor of the SEES Laboratory (http://nees.buffalo.edu/facilities/facilities.asp). The loading simulation was implemented by means of a MTS 244.51S servo-controlled actuator with a stroke of 40 in. and a force capacity of 220 kips. Due to the quasi-static character of the test, the actuator was used in its "static" configuration, using a low capacity servo-valve (SEESL, 2009). A 2 in. thick steel plate, attached to the strong floor, served as a base of the whole experimental setup. The specimen was 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 with relatively low friction. Geometric properties of the PTED frame, the loading method and a general view of the experimental setup are illustrated in Fig. 1.

Standard wide flange "W" sections were used, in order to maintain geometric similitude to the full scale structure. The DYWIDAG Mono-Strand post-tensioning system was used for connecting the beams to the columns of the PTED frame. Two tendons were used at the mid-depth of the beam of each floor with initial post-tensioning forces of approximately 20 kips, 15 kips and 12.5 kips per tendon for the 1^{st}, 2^{nd} and 3^{rd} floor, respectively. The tendons were equipped with load cells installed adjacent to their anchorage mechanism. The beam incorporated web flange reinforcing plates and reinforced flange segments at both ends, to prevent the beams from buckling under compression, when subjected to seismic cyclic loads.

The exterior columns of the PTED frame were connected to specially designed reaction blocks, consisting of a rectangular reinforced concrete base and an overlying steel block shaped as a triangular prism. Each reaction block was firmly attached to the strong floor with 4 post tensioned DYWIDAG Thread-Bars loaded to 100 kips each. The central (interior) column of each frame was left unsupported, in an attempt to simulate the "missing column" scenario (see Fig. 1). 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 installed specimen was fully instrumented with uni-axial strain gauges, displacement transducers (string potentiometers) and a 3D imaging Krypton system, operating with infrared LED's (SEESL, 2009). All instrumentation channels were connected to a Pacific Instruments 6000 Data Acquisition System.

Experimental Procedures and Results

Low amplitude identification tests (low-level displacement "ramp"- and "saw-tooth"-shaped histories) were performed on the specimen for equipment check and calibration of system performance (e.g. calculation of "elastic" stiffness and comparison with analytic predictions). These preliminary tests were followed by a final "push to collapse", using a slow displacement ramp with a loading rate ranging of 0.2 in./min. The data acquisition rate was set at 10 Hz, which was considered more than sufficient, given the quasi-static nature of the experiments.

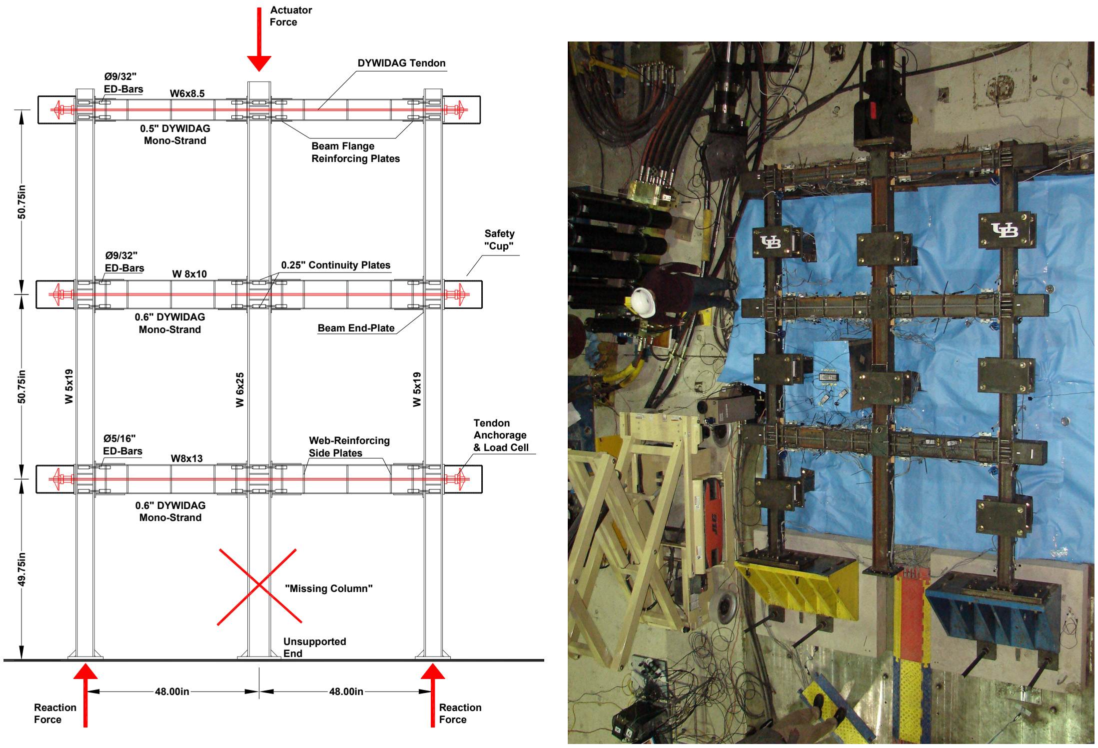

Figure 1. Geometry, loading method and basic features of the PTED specimen (left); general view of the experimental setup on the strong floor of SEESL before the testing (right).

The most representative results of the "push-down" test is the plot of the force applied by the actuator versus the displacement of the central column of the PTED. This plot is presented in the top half of Fig. 2 It should be noted that the applied displacement was controlled automatically up to 13.75 in. and manually – at increments of 0.2 in. – thereafter. Control issues associated with the manual approach are the cause of the oscillations observed in the plot after 13.75 in., likely due to stress relaxation of the yielded material in between load steps when the displacement is held constant.

Due to the non-linear elastic behavior (gap opening) of the PTED frame at low deformation levels, the elastic region could not be well-defined near the origin of the global curve. The resistance of the frame increased monotonically to 121 kips at a displacement of 3.5 in. with a slight stiffness softening. Up to that point, only minor slippage of the beams with respect to the columns and yielding of the flanges of the central sections of beams had been observed. With increasing displacement, a series of tendon ruptures, failures of the energy dissipating bars (not very important for the global response) and buckling at the beams of the 2nd and 3rd floor led to a rapid decrease of the frame resistance. The residual "strength" of the PTED frame after the application of 19 in. of displacement was insignificant; less than 15 kips. Fig. 3 shows a schematic of the PTED frame and notes (using various symbols) the distribution of major types of damage observed. A general view of the PTED specimen after the conclusion of the "push-down" test is presented in Fig. 4, which can be viewed in conjunction with Fig. 3 for better understanding of the damaged state. It is noted that the buckling of the post-tensioned beams at the left bay of the 2nd and 3rd floors occurred at the end of the reinforced zones, far from the interface with the column flange.

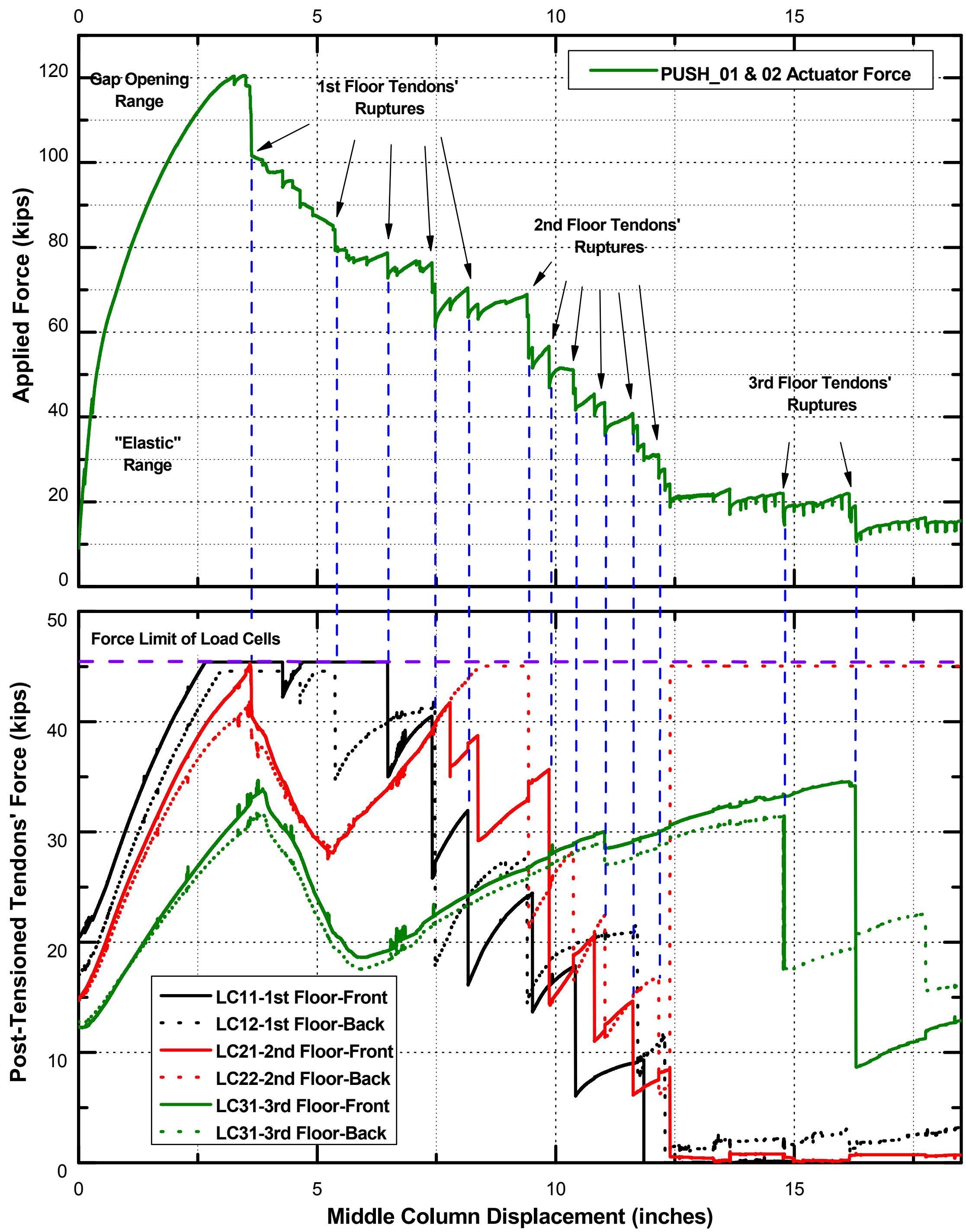

Figure 2. Combined plot of the global force vs. interior column E-W displacement response of the PTED specimen (top) and measured post-tensioning force vs. interior column E-W displacement for all six tendons of the PTED (bottom) correlating the global degradation behavior with single rupture events in the Mono-Strand wire bundles.

By comparing the top plot on Fig. 2 (global behavior) with plots of the tendon forces vs. the interior column's displacement in the bottom half of Fig. 2, 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 tendon consists of seven bundled wires, all encased in a protective PVC pipe). Ruptures of single wires are represented by vertical force drops and can help trace damage events observed in the global plot back to the approximate area where the damaging event occurred. In addition to the tendons' tensile fractures, the local minima of the force traces for the 2nd and 3rd floors (in the vicinity of 5 in. global displacement) are explained by the simultaneous apparition of heavy buckling in the respective beams (see Fig. 3). The sequence of damage events becomes evident by the study of the tendons' tension level in conjunction with strain data and direct observation and is summarized in Table 1.

Table 1. Sequence of damage events during the PTED collapse test.

No.	Event Description:	Global Displ. (in)
1	Elastic, non-linear deformation of the frame.	0 to 0.4
2	Onset of yielding of the ED bars. Plastic hinges form at the central un-reinforced sections of the beams.	0.4 to 3.5
3	First major rupture of tendons at 1st floor. Buckling of beams at 2nd and 3rd floors.	3.5
4	Sequence of ruptures at the tendons of the 1st floor.	3.5 to 12.5
5	Succession of failures of the tendons of the 2nd floor	8.0 to 12.5
6	Ruptures at the tendons of the 3d floor	15.0 to 16.5
7	End of test with residual strength of ~ 15 kips.	19

The PTED push-down test demonstrated that a single column failure is not sufficient to cause progressive collapse of a frame designed with this level of seismic detailing. 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), which is three times less than the maximum measured capacity of the PTED. 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.

Numerical Simulation

A numerical model of the PTED specimen was constructed using SAP2000 with a macro-model approach in which the PTED connection was modeled by an assembly of beam and rigid elements connected by non-linear springs and "gap" elements. The model explicitly accounted for the PTED beams' depth and the gap opening. The ED bars were modeled as bi-linear springs with a smooth transition zone (Bouc-Wen model). The tendons (PT bars) were modeled as linear elements with axial stiffness and ability to deform when loaded perpendicularly to their axis. Concentrated plasticity approach was used both for the beams and the tendons. A more comprehensive description of the model is given by Tsitos (2009).

A schematic of the deformed state of the numerical model with marked locations of plastic hinges is presented in Fig. 5 (left). A comparative plot of the experimental and numerical global applied force versus the interior column displacement of the PTED frame is also shown in Fig. 5 (right). The numerical model captures the observed behavior with sufficient accuracy, particularly the peak strength. A deviation from the experimental data in the softening branch just before the peak is related to modeling assumptions for the ED bars and the gap elements. The strength degradation is more gradual in the experiment because it was caused by a sequence of failure of individual strands in tendons. In contrasts, the complete tendon at a floor level failed instantaneously since all strands were assigned the same nominal properties. Further development of the model used for the post-tensioning cables will increase the detail of simulation of the tendons' damage sequence that was observed during the push-down test.

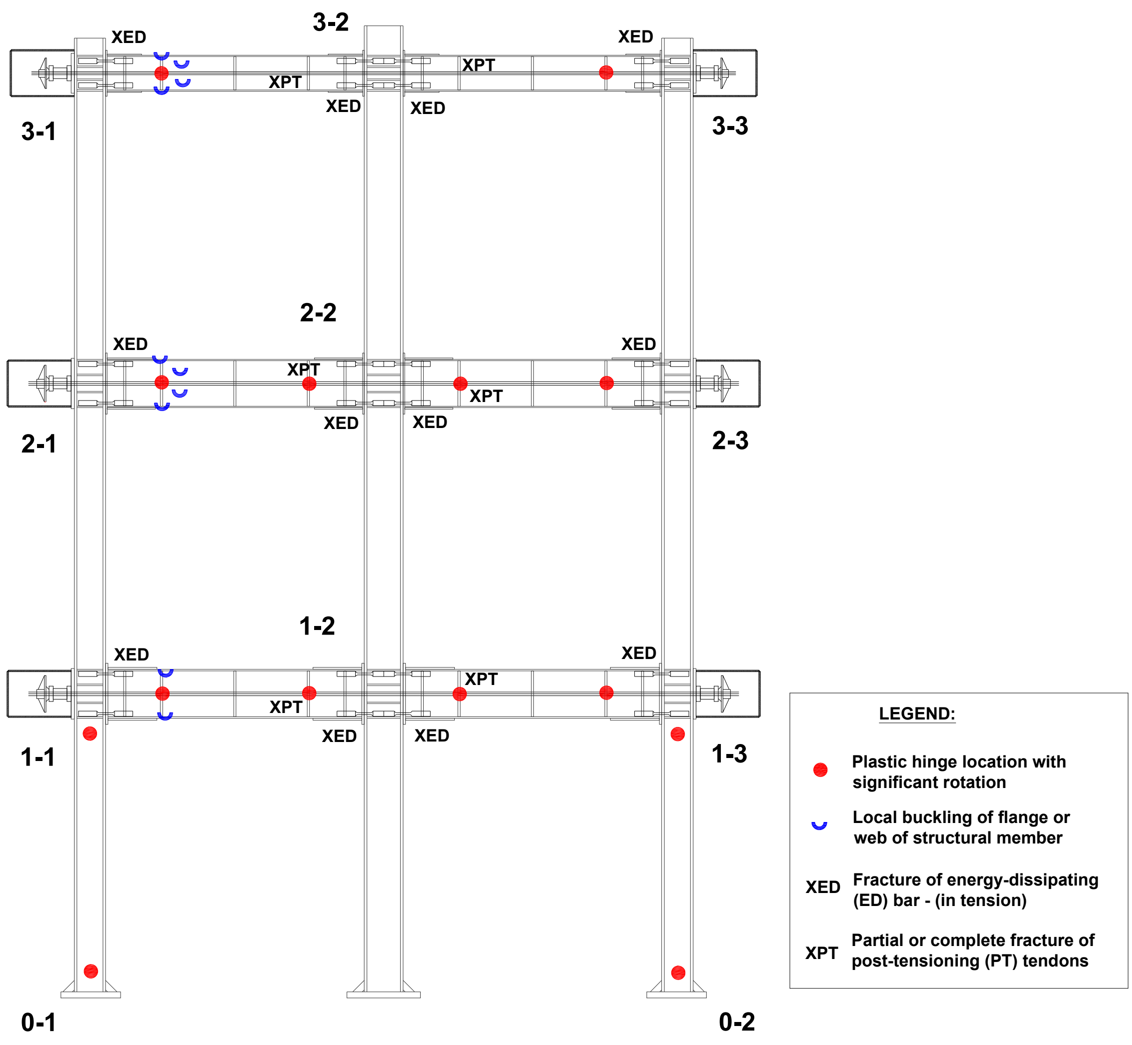

Figure 3. Schematic of the damaged state of the PTED specimen at the end of the collapse test with the spatial distribution of major types of damage (the frame is shown in the original un-deformed geometry for clarity); heavy buckling is observed at the un-reinforced beam sections at the north side (left) of the 2nd and 3rd floor beams; tendon failures are random with tendency to occur near the interior beam column connections; ED-bars break in tension.

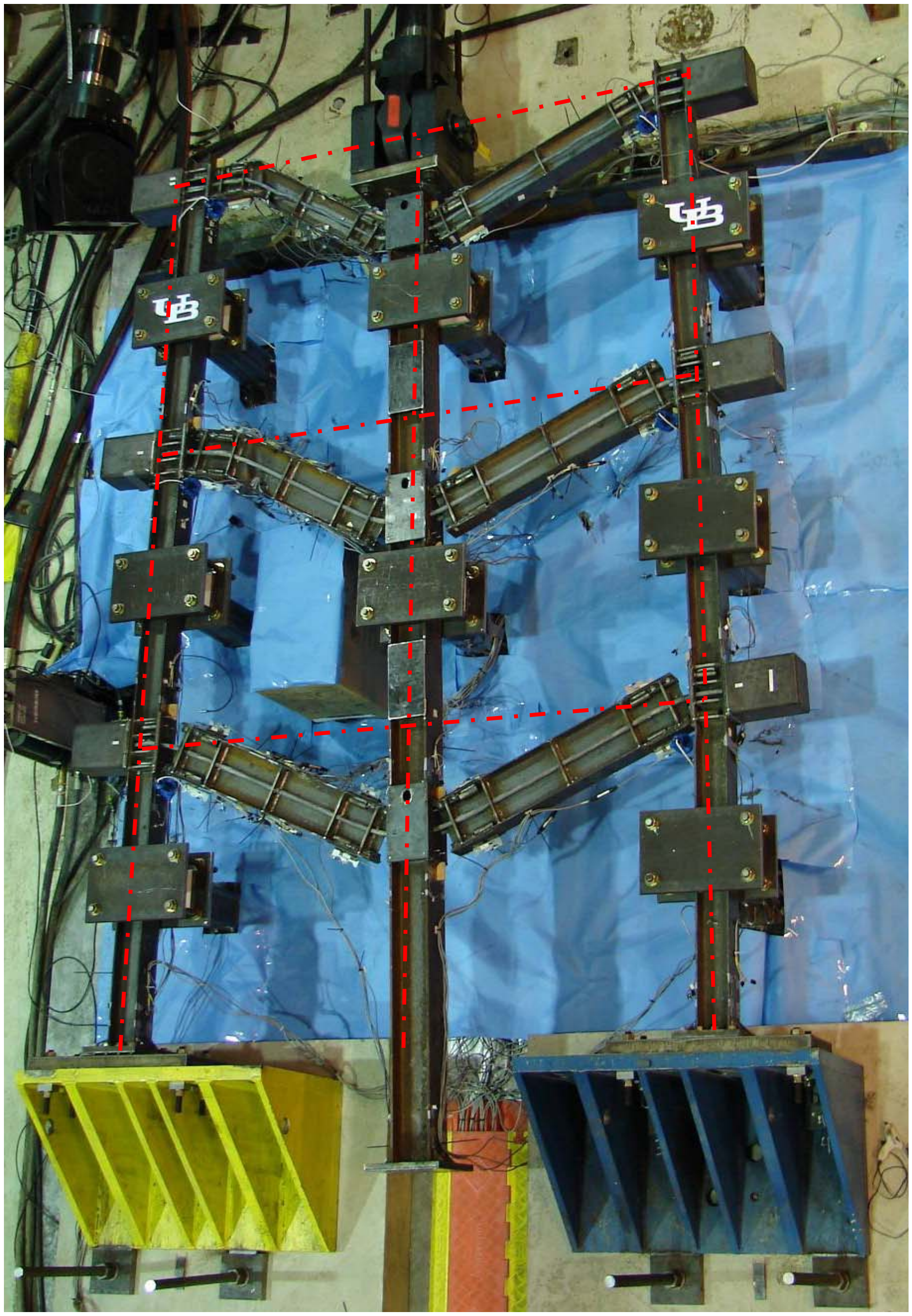

Figure 4. Photograph of the damaged state of the PTED specimen after the end of testing; approximate locations of the un-deformed members' centerlines are marked in red; the current viewpoint is from an oblique angle and has some distortion due to perspective.

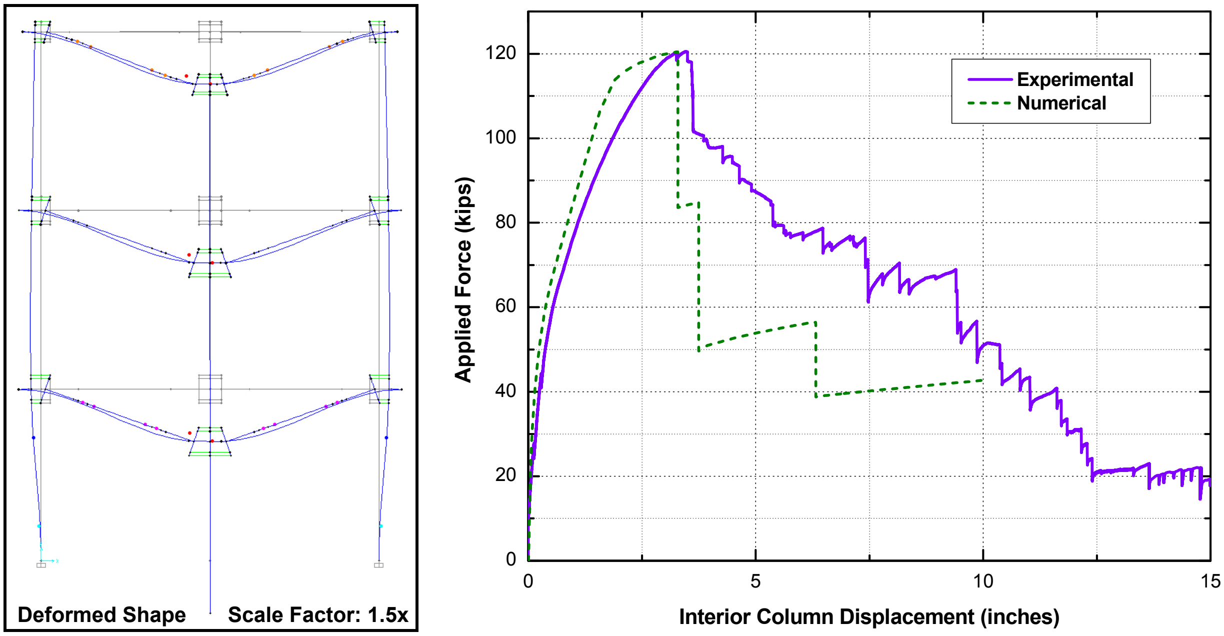

Figure 5. Deformed view of numerical model of the PTED frame (left) and comparison of experimental and numerical global force vs. interior column displacement plots (right).

Conclusions

Within the framework of a broader experimental program to investigate the resistance of steel buildings against progressive collapse, a PTED frame was subjected to a quasi-static push-down test following a missing-column scenario. The frame 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.

The experimental data recorded from the test provided useful insight into the types and propagation of damage would be expected when a post-tensioned frame is subjected to extreme displacements, such as those expected in a collapse situation. The critical role of the tendons' strength and failure mode is clearly reflected on the global behavior of the PTED and underlines the need for using adequate models of this crucial part of post-tensioned frames, when attempting numerical simulations. Numerical modeling of a progressive collapse scenario needs to be further researched and coded, with the ultimate objective of providing a simple yet reliable tool to the professional structural engineering community.

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