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RETROFIT OF SOFT STORY LIGHT FRAME TIMBER BUILDINGS WITH THE DKB SYSTEM USING STEEL KNEE-BRACES

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Abstract

Recent research efforts focusing on timber structures with soft story deficiencies by NEES (Network for Earthquake Engineering Simulation) funded by the National Science Foundation resulted in experimental verification of a number of design methodologies and retrofit solutions. Objectives of the research included evaluation of the underlying assumptions of the guidelines for seismic retrofit of multi-story timber buildings with weak or soft story deficiencies (FEMA P-807) and validation of current and prospective retrofit design options. As part of this research, a particular retrofit strategy titled the *Distributed Knee Brace* (DKB) system was proposed.

The DKB system is categorized as an array of knee braces placed parallel to the soft story line of resistance that uses existing stud walls and floor joists reinforced with additional studs and strategically arranged knee braces. In a single line, the combination of two knee braces, a joist, and wall studs on each side creates lateral resistance out of existing gravity resisting elements. This retrofit strategy, through testing, demonstrated an ability to accommodate 6% drift while providing a highly redundant energy dissipation mechanism while capacity protecting upper stories. In the pilot research, the ductility of the system was derived from slipping/yielding of the nailed connections between the timber knee brace to floor joist and timber knee brace to wall stud connection.

The current research is an independent continuation of the work on the DKB system. However, this research focuses on the behavior of the DKB system when nail yielding is replaced with a brace comprised of a combination of elastic compression bar buckling and small diameter steel rod yielding as the energy absorption/ductile mechanism. Replacing the timber nailed connections with the proposed steel rod alternative results in a more predictable solution that allows more flexibility in retrofit optimization while offering a more slender and a spatially less intrusive physical profile.

To validate the steel brace version of the DKB system and confirm its viability, full scale, cyclic, pseudo-static physical testing was conducted. The tests confirmed that this system (similar to the previously tested DKB system) was able to support nearly full lateral load capacities beyond drifts of 6%. Furthermore, the steel version of the DKB system improved utilization of the existing timber walls resulting in a more optimum system performance.

Keywords: Knee-brace, weak-story, soft-story, seismic retrofit, steel brace

1. Introduction

In San Francisco, at the time of this publication, preliminary seismic investigation suggested that approximately 40 to 80 percent of multi-story timber buildings with weak and soft story deficiencies (broadly defined as "soft story") would become unsafe following a major earthquake event (magnitude 7.2 or greater) [1]. Further, CAPSS (Community Action Plan for Seismic Safety) has reported that upwards of 25 percent of the soft story buildings would exceed the collapse limit state [2]. As such, the policymakers recognized that soft story buildings represent a major post-disaster management concern and consequently, both the Cities of San Francisco and Los Angeles have recently adopted mandatory soft story retrofit ordinances.

Coincident with the work in community risk and hazard assessment were research efforts by NEES (Network for Earthquake Engineering Simulation) to specifically examine soft story deficiencies in low rise timber structures and experimental verifications of various retrofit design methodologies [3]. Out of this research came the *Distributed Knee Brace* (DKB) system [4].

The DKB system engages structural members that are part of an existing gravity load resisting system (wall studs and joists) and coverts them to lateral load resisting frames. The load fuse in the DKB frames (in the original work), was yielding of the nailed connections that fastened the knee braces to both the wall studs and floor joists. As a retrofit strategy, the application of the DKB system is, in general, a design method to strategically locate the knee braces to reduce torsional irregularities, increase over all weak story strength, and provide augmented ductility in the first story. As the DKB system can develop very large displacements without loss of vertical load carrying capacity; it creates, essentially, a seismically isolated story at the most lower lever. By properly balancing strength and ductility with a reduction in torsional irregularities, the first floor can be used to capacity protect the non-ductile/rigid stories above.

The research conducted and presented herein was motivated to provide a more reliable (less variable) type structural fuse within a DKB system than yielding of nailed connections between wood members. Moving to a less variable "yielding element" provided an opportunity for a more optimized and repeatable solution. Further, as an engineered solution (all in steel), a packaged product could be created with simple installation guidelines for homeowners wishing to strengthen their soft story structures.

The steel knee brace, as the DKB ductile mechanism, relies on a combination of steel components yielding in tension coupled with steel elastic buckling in compression. Both the yielding and elastic buckling mechanisms are numerically more predicable with less variability than the timber-to-timber nailed connection DKB counterpart. Component tensile and compression tests were conducted to determine realistic component failure properties of the steel knee brace. After, full scale pseudo-static testing of the steel knee brace was conducted in the structural laboratory of California Polytechnic State University, San Luis Obispo. The testing consisted of four identical steel knee braces installed in the test frame built to represent a typical U.S. residential/light commercial timber bearing wall type structural system.

Presented herein are details of the DKB retrofit strategy, properties of the steel knee brace, the DKB system in the test frame, test results, a nonlinear analysis of the system articulating recorded data with the numerical model, and finally, a comparison between the system proposed herein and data collected from previous research in this area.

2. Retrofit Strategy and the DKB System

In general, the DKB system as a retrofit strategy for soft stories, introduces strategic lines of resistance to mitigate torsional irregularities while simultaneously strengthening a weak story and providing augmented ductility. By adding knee braces perpendicular to an existing gravity wall, lateral force resistance is created. However, once the knee braces are introduced and lateral forces are applied, the force demands on the gravity frame change dramatically and, as such, require reinforcement.

The changes in force demands due to the addition of the knee brace require: 1) introducing a tensile connection between the floor joist and the wall stud, 2) a wall stud strengthening because of the combined bending/compression and bending/tension loading introduced by the knee brace, 3) an out-of-plane shear connection between the wall stud and the bottom plate (and possibly, depending on the age and type of construction the bottom plate and the floor/foundation below) (Fig. 1 and 2).

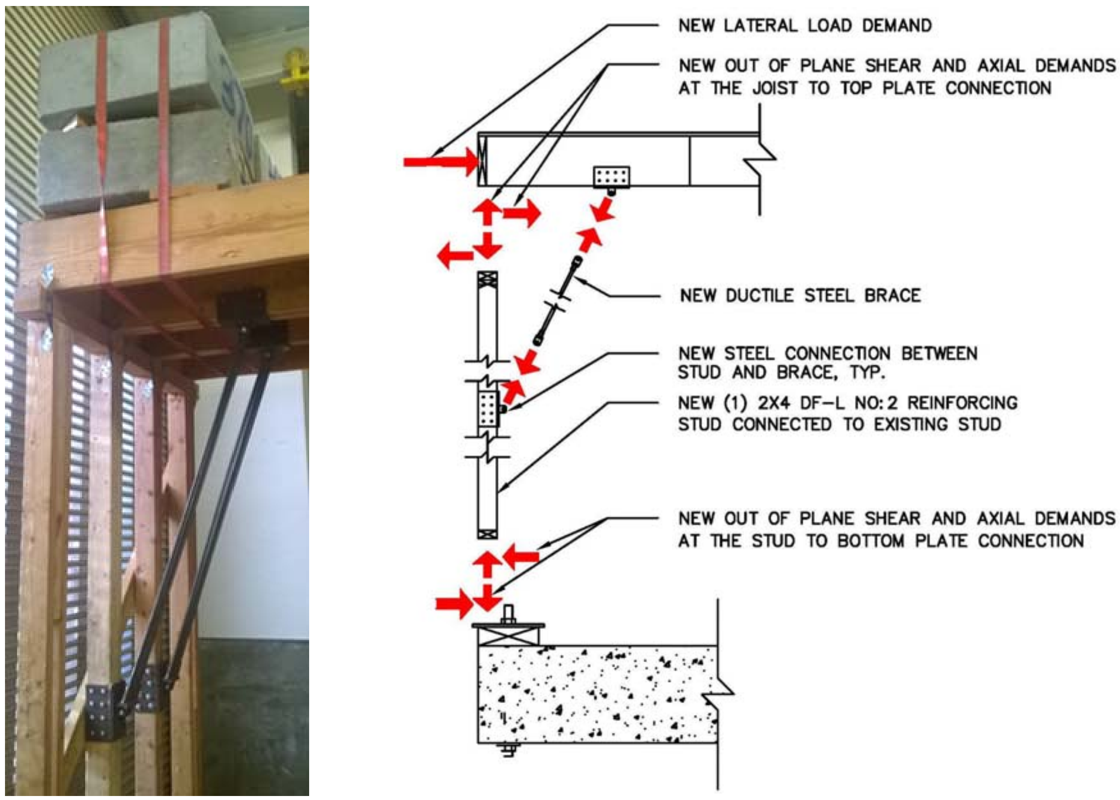

Fig. 1- DKB system test specimen.

Fig. 2 - Joist to stud connection reinforcement (left – stud to joist connection, right – added stud to bottom plate reinforcement)

For the knee brace system to be ductile, the yielding element must capacity protect against all other modes of failure. Analysis of different failure modes of the system establishes the upper bound design capacity of the steel knee brace for both tension (yielding) and compression (buckling). Thus, the design of the ductile knee brace must ensure that it is the weakest link in the system and will become the failure mechanism (Fig. 3)

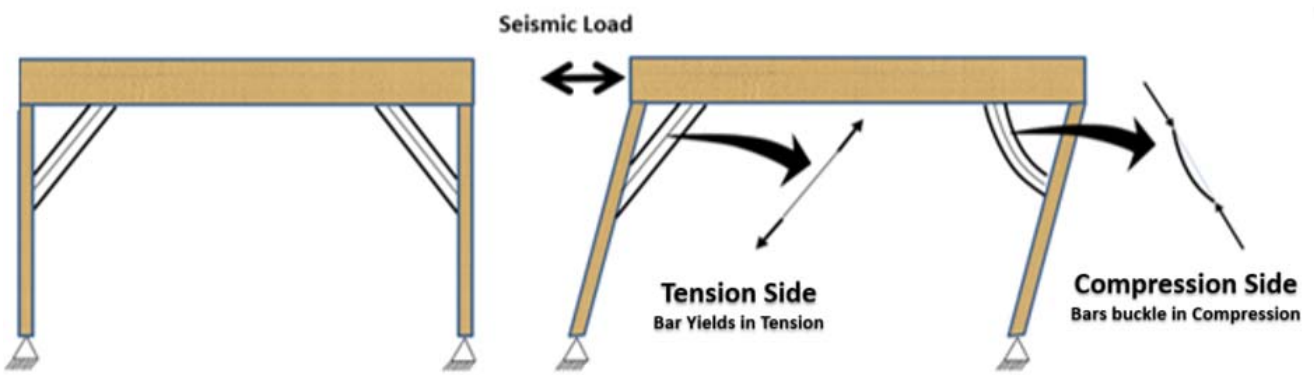

Fig. 3 - Schematic of the yielding mechanisms of the tensile and compression braces

The steel knee brace is fabricated by combining four US Standard #4 ASTM 615 Grade 60 rebar, designed to elastically buckle in compression, and a single concentric 0.25in (6.4mm) ASTM A36 steel rod, designed to yield in tension. The “compression only” rebar connection is achieved by attaching the rebar to one end of the steel brace and placing the other end in an oversized sleeve thereby allowing movement and minimizing friction generated forces (Fig. 4). The steel sleeve housing is two inches long and is capable of accommodate an 8% story drift at the floor level in its installed configuration. It is worthwhile to note that a longer sleeve can be provided if necessary. When the brace is in tension, the central rod yields and the compression rebar do not participate. In compression, the small diameter tensile bar has virtually no capacity and the compression rods bear against the backing plate. The compression bars continue to take additional force until elastic buckling occurs thereby limiting the transmissibility of the forces to the wall studs.

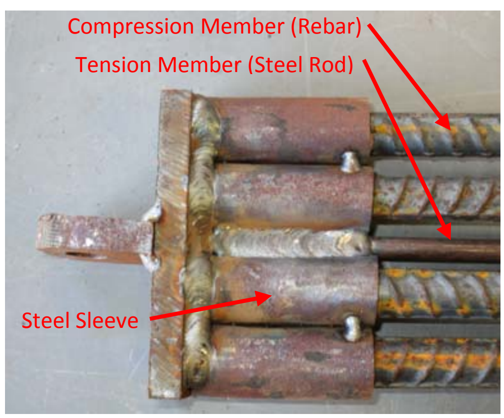

Fig. 4 - Steel brace tension and compression members.

The brace was designed such that the tension and compression capacities were theoretically similar. The upper limit of the brace capacity was chosen to capacity-protect the reinforced wall studs against combined compression/bending failure, the newly reinforced connections, and other possible failure modes. As part of the installation of the connection assemblies, the floor joist was furred to 3.0in (7.6cm) to match the width of the double stud consisting of the existing and new reinforcement stud. The knee brace was connected to both the floor joist and wall studs each with the same rigid steel connection assembly designed to insure all of the ductility and nonlinear behavior occurred in the brace. The connection assembly was constructed of a bolted steel bracket with (8) 0.25in (6.4mm) diameter ASTM A325 bolts (Fig. 5 and 6).

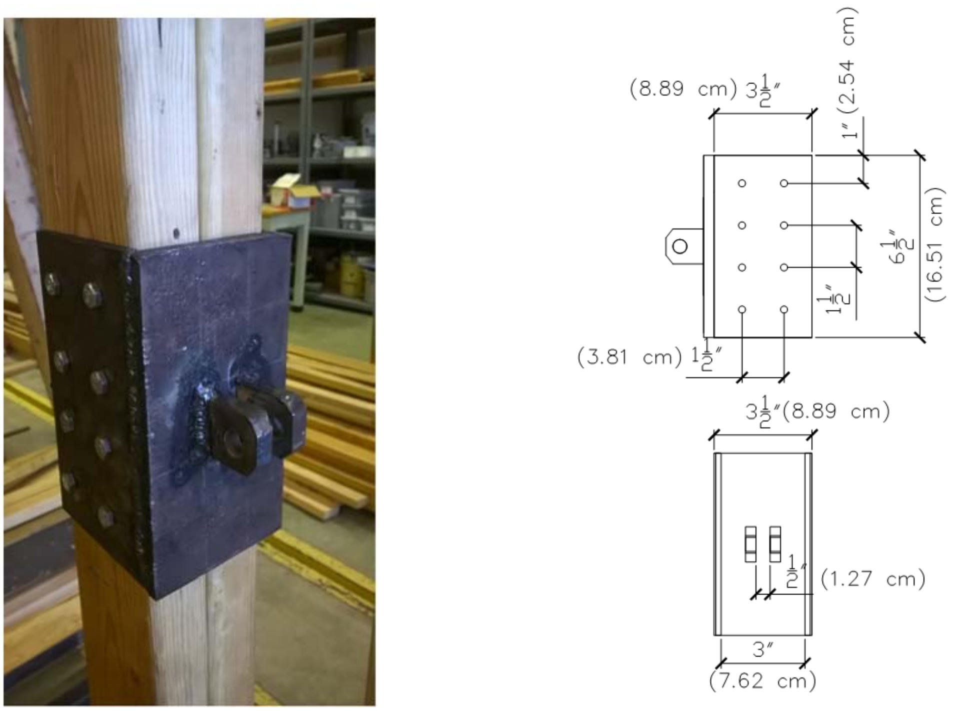

Fig. 5 - Stud wall to knee brace connection.

Fig. 6 - Knee brace to stud wall and furred joist connection.

The steel brace was connected to the stud at mid height to optimize the stud wall capacity by minimizing the unbraced length of the stud wall in the direction of loading action. Static analysis demonstrates that when the brace is in compression, the maximum compression demand is on the lower half of the stud, and when the brace is in tension, the maximum compression demand is on the upper half of the stud. Thus, the compression plus bending demands will be the same for both the upper and lower half of the stud. The compression demand is equally magnified above and below the brace connection by the presence of gravity loads from the floors above.

3. Test Frame and Test Set Up

The test structure consisted of a two-bay continuous joist floor system supported by two outer walls (each supporting approximately 1600lbs (7.12 kN) representing wall loading of 400lbs/ft (4.84 kN/m) of wall length) and an intermediate non-bearing wall framed at the floor center line (Fig. 7). Two knee braces were installed in each of two walls (four braces total) (Fig. 8). Within the test frame, the test specimen was braced laterally using rollers oriented parallel to the direction of motion to prevent any accidental out-of-plane torsional response.

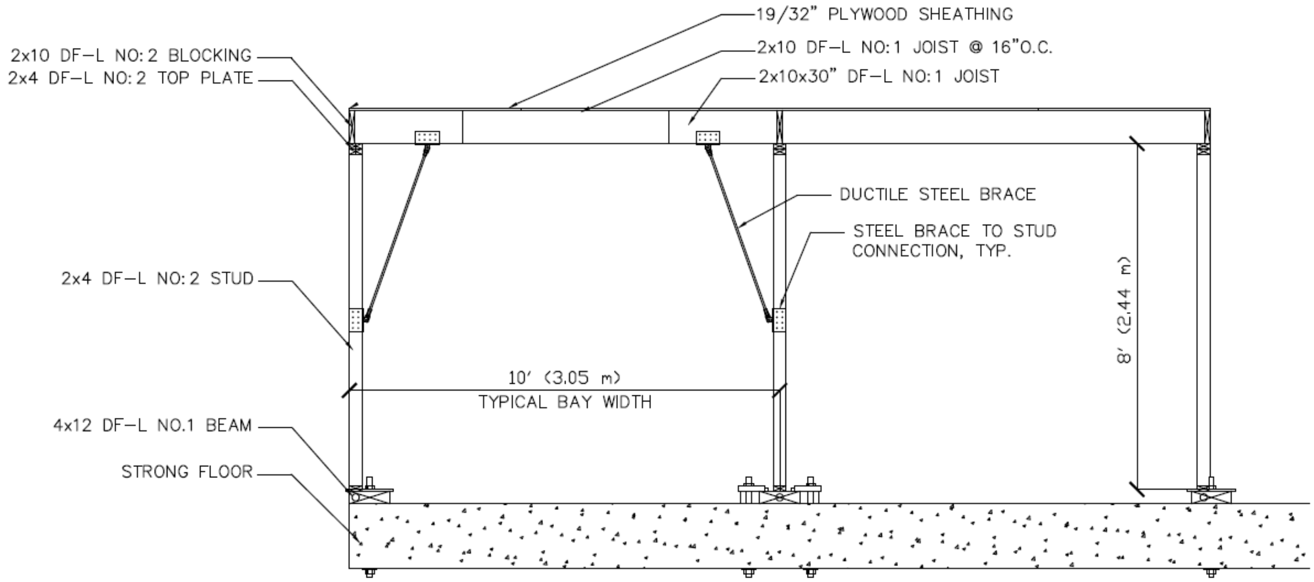

Fig. 7 - DKB retrofit frame system.

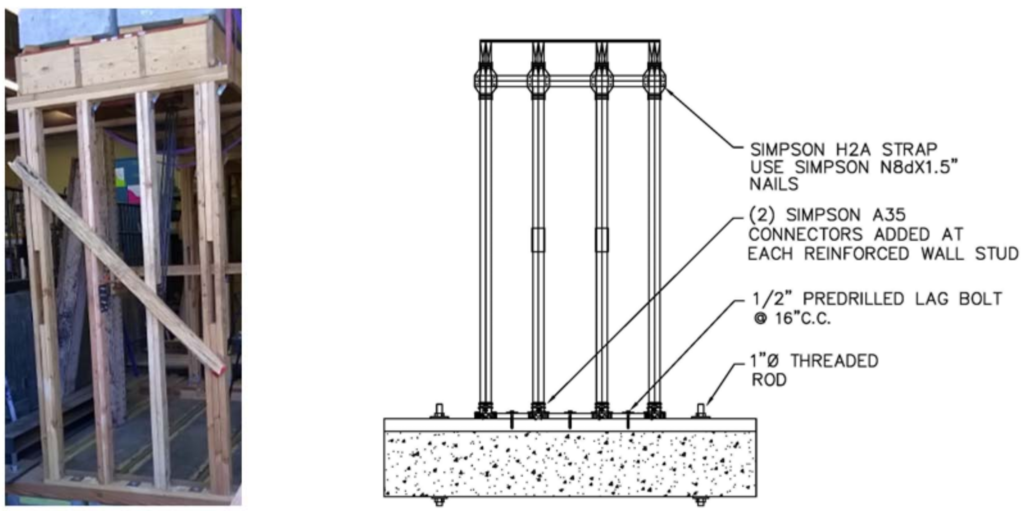

Fig. 8 - Stud wall elevation with stud reinforcement and knee braces in place. Note that the diagonal wall brace was temporarily placed for lateral stability.

4 Testing

A cyclic pseudo-static test was performed in the structures laboratory at California Polytechnic State University (San Luis Obispo) using the CUREE-Caltech protocol for displacement controlled quasi-static cyclic testing [5]. However, rather than using a monotonic pushover to categorized the reference deformation as an option suggested by the literature, a reference deformation of 2.5% was selected to represent the life safety drift. The CUREE-Caltech protocol was followed between the initiation cycles and 100% of the reference displacement. However, beyond the life safety displacement of 2.5% drift, in an effort to better characterize the hysteretic properties of the test model, displacements where incremented in each primary cycle by 20% with two trailing cycles of 75% of the primary cycle in accordance with the protocol. Eight additional primary cycles followed each by two secondary cycles (at 75% of the primary) continued beyond the reference displacement terminating in a maximum drift demand of 6.5%. The test was concluded due to safety concerns at the large displacements.

The test structure was instrumented using high resolution displacement string potentiometers. Displacements were recorded at three locations on the stud bearing stud wall, the diaphragm, and along the knee brace. The instrumentation diagram (Fig. 9), shows the location of the sensors. In addition, forces and displacements were taken as part of the actuator data collection.

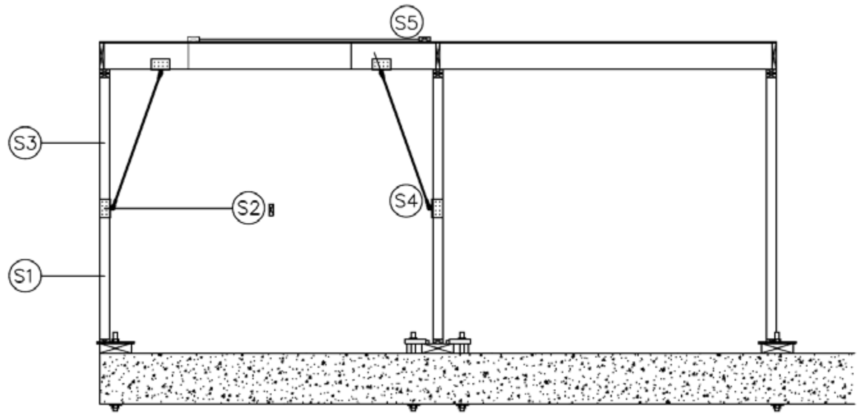

Fig. 9 - Instrumentation Diagram.

Data was recorded at 10 Hz throughout testing and a signal-to-noise ratio scheme was used to filter and smooth the recorded test data. The Sacitzky-Golay filter used is characterized by a moving average of least squares fits that calculates the new smoothed data point [6]. The peak frame lateral force resistance (for two frame lines) was approximately 1500lbs (6.62kN) occurring at roughly 4.3% drift. The ultimate drift of 6.5% showed a load of 1450lbs (6.45 kN). The smoothed hysteresis (showing only primary cycles) for two frame lines (four braces total) and an accompanying annotated backbone curve are provided in Fig. 10.

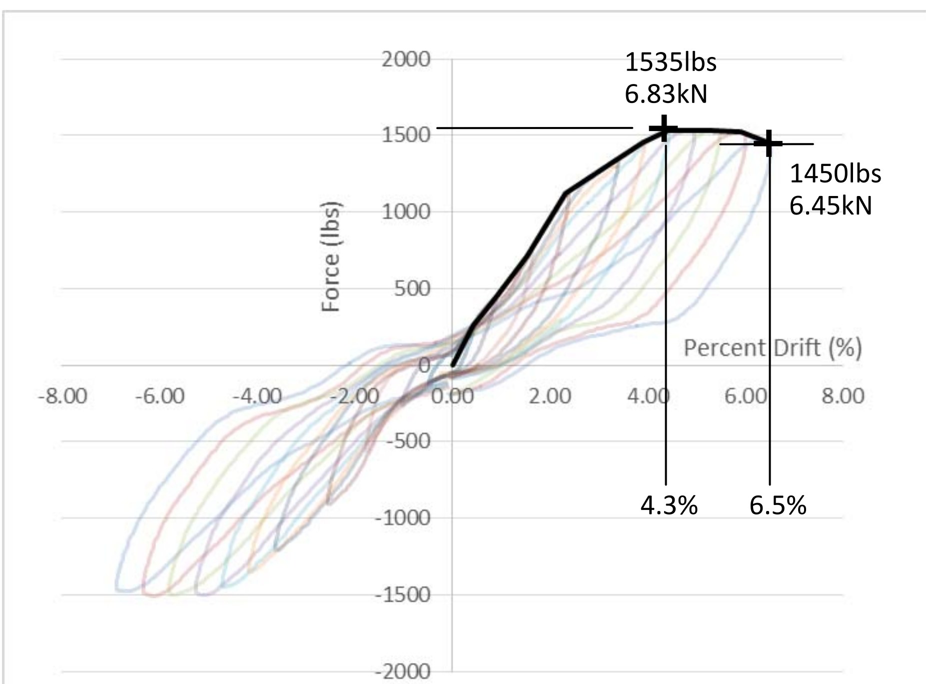

Fig. 10 - Load deformation and Backbone Curves

From observation, the ductility of the system, as expected, was predominantly from the steel braces. Further, the brace, as intended, acted as a capacity fuse protecting the rest of the system. Post-test investigation showed some signs of stress in the steel connectors but this was due directly to the large rotations experienced by the connections at large demand drift cycles. Further, the plastic deformation in the tension rod was the primary source of energy dissipation in the brace coupled with some compression yielding of the buckled bar at large compressive deformations (only in the last cycle of loading). This was consistent with the design. At the last large drift cycle, the shortening of the compression brace was 1.2in (3.05cm) resulting nearly 6in (7.6cm) of out-of-plane lateral buckling displacement (Fig. 11). The maximum elongation of the tension rod was measured to be approximately 0.9in (2.29 cm) (this value includes slip in the tension brace).

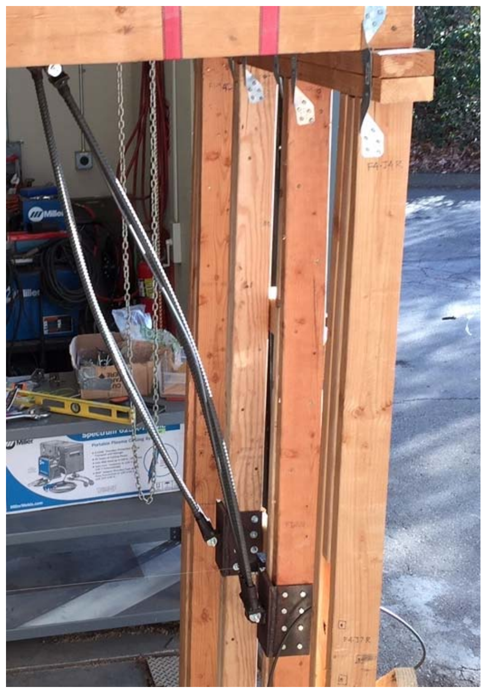

Fig. 11 - Compression buckled steel knee braces.

5. Brace Testing and System Analysis

As part of this research, a simple nonlinear (geometric and material) pushover analysis was conducted on the test specimen in an effort to compare simulation data with data captured during physical testing. The system was modelled as a 2D single frame consisting of inelastic longitudinal springs with asymmetric behavior representing the braces. All other elements were assumed elastic and analyzed as such. Rigid links were used to account for rigid offsets for the brace-to-joist and brace-to-stud connections. The applied vertical gravity loads in both the simulation model and the physical model was taken to be 1600lbs (7.2kN). Loads were applied on the outermost frames representing 400lbs/ft (5.84kN/m).

The nonlinear compression (buckling) and nonlinear tension (yielding) brace models used were monotonic phenomenological models captured directly from component testing. The component testing for the compression model was performed on a single US Standard #4, Grade 60, ASTM A615 rebar with the same unbraced length (51in, 129.54cm) as the compression bars in each steel brace. Test data taken from the component test was a perfect match, within 1.0lb (5N) of the theoretical Euler buckling load (338lbs, 1.5kN). In the simulation model, the data was scaled by a factor of four to represent the four physical bars located in each of the braces. In addition, a 0.125in (0.32cm) gap was introduced into the phenomenological model representing the tolerances of the fabricated brace.

The component testing for the tension model was done using an ASTM standard tension test on a 0.25in (0.64cm) ASTM A36 tension rod 8in (20.32cm) long. The data collected was length-scaled to represent the physical 51in specimen. The coupon test was performed using the same steel rod stock as was used in the steel brace. While the component test did show some strain hardening in the steel, this was not seen in the data collected. Further, it is likely that there was slip at the tension rod connection thus preventing any strain hardening from occurring. For this reason, strain hardening was not considered in the simulation model to better match with the data collected. The force displacement nonlinear spring model used in numerical simulation is provided in Fig. 12.

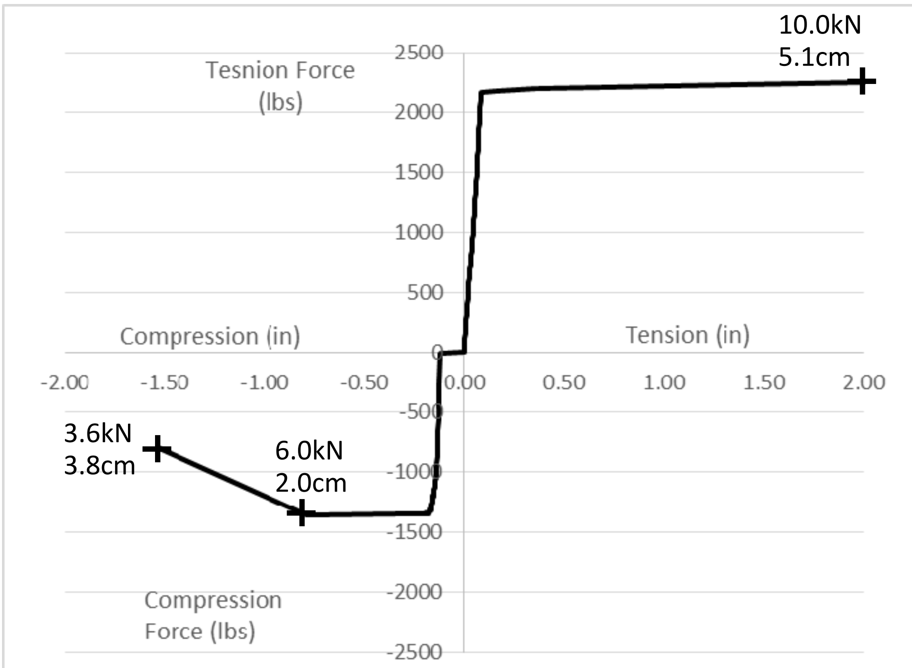

Fig. 12 - Brace Phenomenological Component Model.

The physical and simulation data showed fair articulation (Fig. 13). Data collected during the physical testing at various points along the stud height (sensors S1, S2, and S3 – Fig. 9) showed a good match with the horizontal data collected at the floor level. However, for the last three large displacement cycles, there was a substantial slip recorded between floor system and the top of the stud wall when the brace was loaded in tension at the outside wall. This slip is a clear indication of the out-of-plane shear capacity being exceeded at the joist/rim joist-to-double top plate connection. This slip reduced the effectiveness of the tension side brace and will be addressed in the subsequent testing.

As the identified ductile failure mechanism, the steel brace was designed to capacity protect the other failure modes. Using the results of the component testing as the actual demand, a comparison was made against the calculated capacities of the protected failure modes (stud bending/tension, stud bending/compression, shear at the stud to ground floor connection (taken by the Simpson A35 clips), and tension between joist and stud (taken by the Simpson H2 clip). The demands on the protected failure mechanisms associated with the observed collapse mechanism (brace buckling on one side and brace yielding on the other) are provided in Table 1.

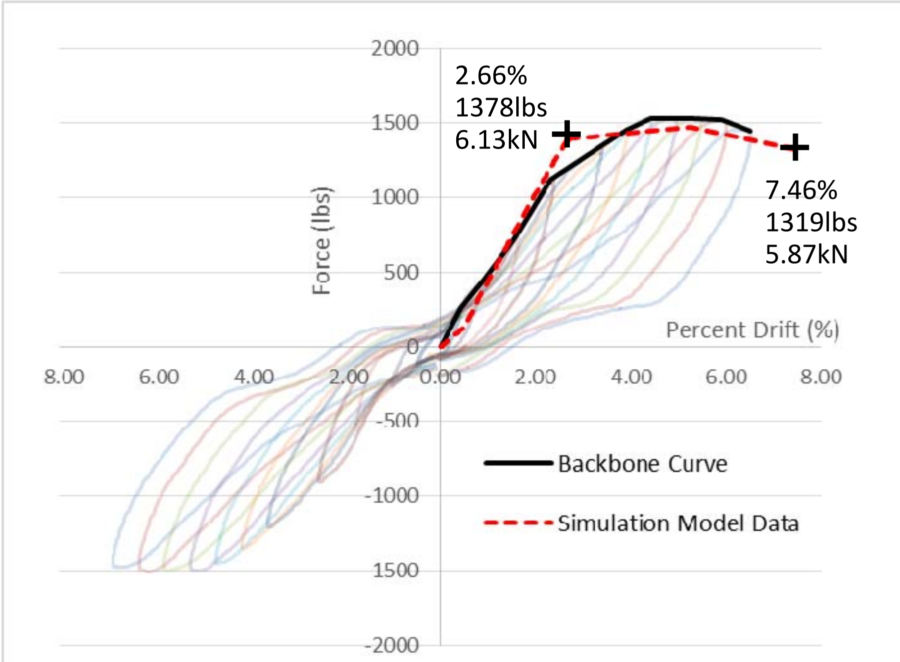

Fig. 13 - Physical and Simulation Test Comparison

Table 1 - Demands from Analysis

Type Of Demand	Demand
Worst Case Stud Demands (Bending + Compression)	21.62kip-in (2.44kN-m) (**Moment**) with 2538lbs (11.4kN) (**Compression**)
Worst Case Stud Demands (Bending + Tension)	11.52kip-in (1.30kN-m) (**Moment**) with 1206lbs (5.37kN) (**Tension**)
Tension At H2 Connection	1206 lbs (5.37kN)
Shear At A35 Connection	478 lbs (2.13kN)

The capacity of the Bending/Compression in the wall stud and Bending/Tension in the wall stud was calculated in accordance with the National Design Specifications (NDS) for Wood Construction [7]. All applicable NDS adjustment factors were used except for the Bucking Resistance Factor (C_p) and the LRFD resistance factor (ϕ). The additional moment demand from localized P-δ effects was accounted on the wall studs in the Compression/Bending limit state as part of a simple, member level second order analysis; as such, this capacity side reduction factor for Column Stability Factor was set to 1.0. The LRFD resistance factor was set to 1.0 to represent the probabilistic average for ultimate capacities. The ultimate capacities for the H2 and A35 Simpson Strong Tie connectors were taken from ESR report 3096 [8] and ESR report 2613 [9]. The ratio of calculated demands to capacities (DCR) is provided in Table 2.

Table 2 – Demand/Capacity Ratio

Components	DCR
Bending + Compression in the Stud Wall	0.96
Bending + Tension in the Stud Wall	0.53
Tension At H2 Connection	0.87
Shear At A35 Connection	0.48

5. Comparison with Previous Testing

The thrust of the research presented herein was focused on furthering and refining the concept of the Distributed Knee Brace. The DKB system engages gravity walls through out-of-plane diagonal bracing that contribute energy dissipation and provide additional protection to seismically substandard structures. Data collected from the testing performed as part of the research described herein is compared with previous DKB (timber version) tests in the form of backbone curves (Fig. 14). For consistency in reporting, the strength data provided is for two lines of braces for each of the timber and steel versions of the DKB system braces. The results show that the walls were better engaged (thus providing additional strength), while large drift capacities were preserved.

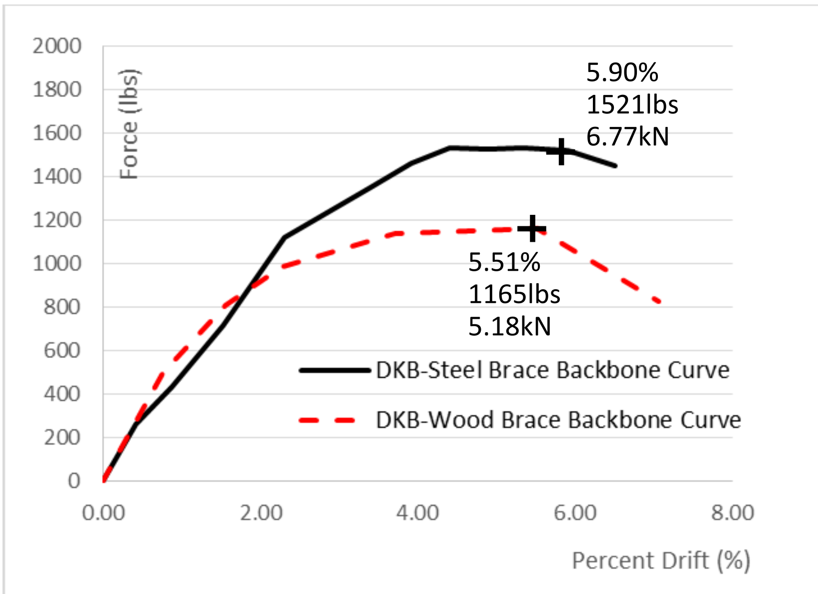

Fig. 14 – Steel and Wood Knee Brace Test Comparison (Data shown for two frame lines)

6. Conclusions and Future Work

The strategy using the DKB system is to locate the brace lines to best eliminate unwanted torsional response, increase lateral load resistance/ductility in a cost effective way, and reduce demands on the diaphragms above by providing supports as distributed resistance. A single brace line (or frame line) is defined as a line parallel to loading consisting of a wall stud on each side of a floor span, a floor joist, and two knee braces connected together to provide lateral resistance. The tested and simulated work presented here focused on specific steel designed knee brace for use within the DKB system.

The tested results from the steel version of the DKB retrofit system demonstrates that the proposed brace configuration offers a viable alternative to the DKB (wood version) system retrofit strategy for soft-story wood-framed buildings. In addition, the reduced profile creates a more desirable configuration while providing higher lateral load capacities. The test data showed that a single frame line (created from an existing "gravity only") constructed with the steel knee brace within a DKB system was capable of resisting 750lbs (3.34kN) at 4.3% drift and 725lbs (3.23kN) at 6.5%. With the more predicable capacities of the steel yielding/buckling system, the demand/capacity ratios of the protected failure modes can be increased while maintaining the same margin of safety. This directly results in these higher capacities.

For the testing described herein, there was an unaccounted (and unanticipated) slip failure at the connection between the floor diaphragm and the outer wall as confirmed by the collected data. The capacity exceedance of this out-of-plane shear connection will be addressed in the future testing. Further, as part of this testing/research sequence, different brace capacities, configurations, and distributions will be modeled numerically from a system level perspective using specific benchmark soft-story timber buildings. The end goal of this work is to create a feasible and standardized retrofit solution the low rise, timber soft story class of existing structures.

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