16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 3410

Registration Code: S-Z1462905453

Seismic Response of a Model Moment-Frame Steel Building with Ductile-Anchor Uplifting Baseplate Connections

C.A. Trautner[1] and T.C. Hutchinson[2]

[1] *Ph.D Candidate, University of California, San Diego, ctrautne@ucsd.edu*
[2] *Professor, University of California, San Diego, tahutchinson@ucsd.edu*

Abstract

Several previous studies have shown that steel column baseplate connections that allow the structure to undergo rocking associated with column uplift may reduce building damage caused by strong earthquake motions. The use of concrete anchors as the ductile "fuse" to allow the uplift motion is an attractive option due to the relative ease with which the anchors may be replaced following a seismic event. More importantly, ductile-anchor column baseplate connections impart an inherently self-centering tendency to the system. In this study, we present results illustrating the robustness and practicality of the reusable anchor concept, as well as preliminary results from a program of dynamic shake-table testing of a miniature, three-story moment-frame building. The building was tested with different configurations of column baseplate connections and traditional frame fuses to allow behavior ranging from uplift-dominated to structural-hinge dominated. Configurations with ductile, uplifting baseplate connections experienced reduced seismic demands and damage, exhibiting well-delineated self-centering behavior. Under intense motions, the performance of the building was improved substantially, with significantly reduced residual drift. Mechanisms for this improved behavior, including frequency shift associated with boundary condition changes caused by anchor elongation, are discussed.

Keywords: seismic demand reduction; rocking system; ductile concrete anchorage; seismic demand; dynamic testing

1. Introduction

1.1 Background

Previous research and observations from recent earthquakes have indicated that the use of ductile concrete anchorages or baseplate connections, particularly those allowing uplift, can significantly reduce demands in both building and nonbuilding structures. The theoretical basis for demand reduction via building uplift caused by rocking motion was first introduced by Housner [1]. Since this seminal effort, there has been well-supported evidence of damage reduction due to building uplift or foundation rocking in strong motion events [2]. In addition, several experimental studies have demonstrated the capability of such systems to significantly lower both the magnitude of typical design parameters such as total base shear and the intensity of observed damage. Such studies have incorporated base connections ranging from those that are relatively exotic, for exampe connections that match the theoretical kinematics of a vertical roller via the use of large guides, to connections which merely incorporate a very thin baseplate to allow yielding and uplift [3,4,5]. The thin baseplate approach, in particular, has several appealing practical advantages, as it requires relatively little departure from traditional construction practices in many countries, and it does not require a separate shear transfer mechanism such as a shear key. However, there are some disadvantages to this approach as well, including requiring demand-critical welds, which may undergo significant plastic deformations. Additionally, the baseplate cannot be easily replaced following a seismic event, and very large anchors may be required to resist the combined tension and shear demands.

As a separate but related development, there has been renewed interest in utilizing stretch length in the design of anchorage as a mechanism to promote ductility. Stretch length may be defined as the region in which significant plastic deformation is expected to occur in an anchor during earthquake loading. For simplicity, it is often defined in terms of a multiple of the nominal anchor diameter, D [6]. This length may extend into the concrete in which the anchor is embedded if measures are taken to debond the anchor. Although some form of minimum anchor embedment or stretch length has historical antecedents in the building codes of the United States [7], the most recent interest in this topic stems from observations following the 2010 Maule, Chile earthquake. In this earthquake, connections that incorporated ductile anchors, specifically anchors with a well-defined stretch length greater than 8D, exhibited fewer anchor fractures and were associated with generally improved connection performance [8]. More importantly, large, massive structures such as tanks and chimneys designed with anchors with very large stretch lengths were observed to undergo rocking motion. Several structures which had undergone large base rotations were found to be undamaged following the earthquake, except for elongated anchors. In at least one instance, the anchors themselves were designed to be replaceable, with the anchors consisting of bars locked into embedded inserts in the foundation [8]. With this type of design, the structure could be returned to service as soon as the old anchors could be unthreaded and new anchors installed.

1.2 Scope of paper

This paper presents preliminary results of an experimental program designed to probe the effectiveness of uplifting base connections incorporating ductile, replaceable steel anchors in moment-framed building structures. The removable/replaceable anchor connection concept is first discussed in the context of pseudo-static testing of traditional, steel moment frame column baseplate connections subjected to lateral load and uplift. Numerical modeling and select dynamic test results from a three-story, 3.7m-tall miniature building with replaceable beam-column fuses and anchorages are then presented. Salient behavioral improvements in the uplifing-column configurations are discussed.

2. Experimental specimen configuration and design

2.1 Design and preliminary testing of ductile uplifting connections with replaceable anchors

In the 2010 Maule earthquake, some of the best-performing connections for large industrial structures incorporated relatively large-diameter anchors with a very long stretch length (on the order of 16D or higher). In some cases, particularly for elevated tanks and chimneys, evidence indicated that elongation of these anchors allowed significant connection uplift during the earthquake. One connection design investigated following the earthquake included a replaceable anchor with a specialized T-head. This head fit into a socket embedded in the concrete, such that the anchor could be loosened (if needed), twisted, and removed from the structure following significant plastic deformation [8]. Similar anchors could then be re-installed. While offering very robust performance in a replaceable connection, such connections have the downside of requiring both specialized anchors and embedments. As a preliminary test of a removable anchor system, the authors investigate herein the performance of two-part replaceable anchors made with off the shelf components: ductile all-thread rod and an embedded anchor body comprised of steel tubing, a bearing plate, and a hex nut [9]. The use of such a connection is illustrated schematically in Fig. 1.

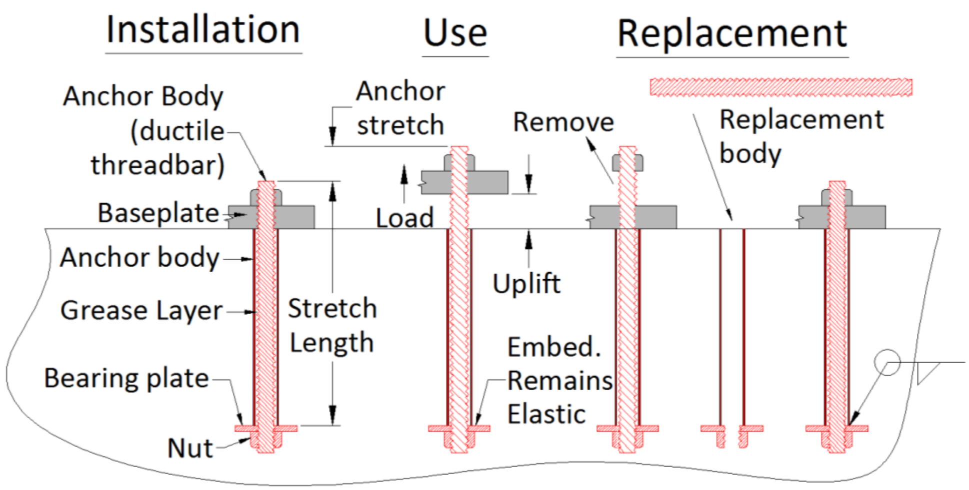
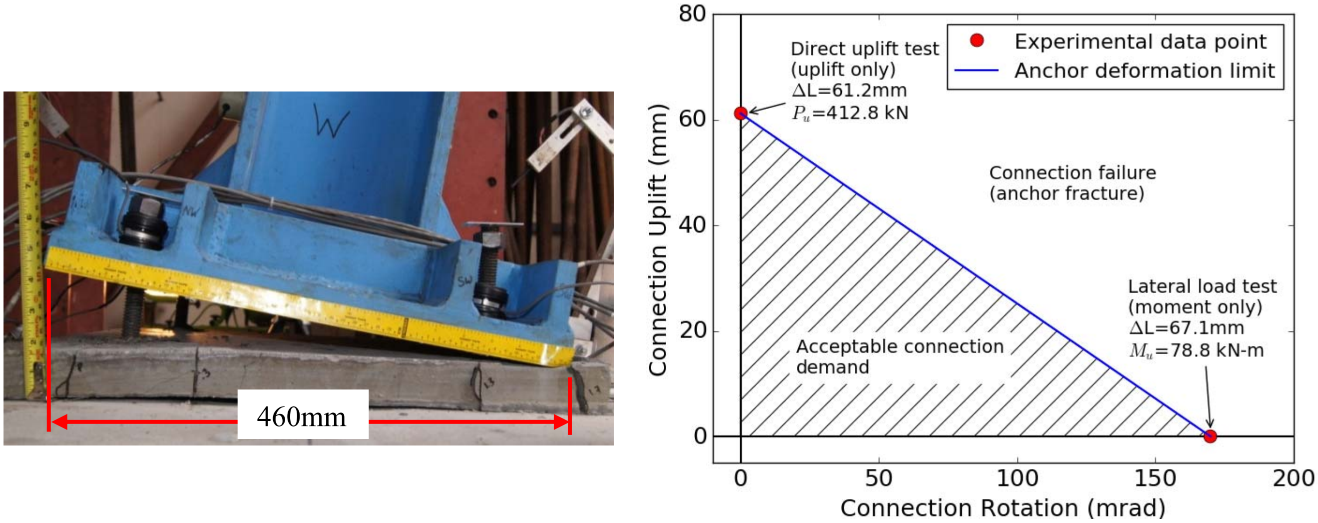


Fig. 1 – Uplifting base connection with removable anchor concept

As a proof-of-concept test for connections used in later dynamic testing, full-scale removable anchors were tested using the test setup shown in Fig. 2a. This connection consisted of a US W8x48 column with a stiffened, 460mm-square baseplate with a 330mm-square, four anchor pattern. The anchors in this connection utilized a 19mm diameter replaceable anchor insert and a 250mm (13D) deep cast-in sleeved embedment similar to the one shown in Fig. 1. Additional details of the connection have been described elsewhere [9]. This connection was first tested under displacement-controlled, cyclic, pseudo-static lateral loads without axial load. Following this, the anchors were replaced and tested individually under direct tension. The lateral load testing showed that such connections are capable of withstanding very large rotations, above 160mrad if ductile, mild steel threaded rod inserts are provided. This level of rotation is likely far in excess of the rotation of the connection associated with drift-induced failure of the superstructure of typical steel moment frames with ductile detailing. Importantly, anchor-dominated connection behavior under lateral loads is inherently self-centering; there is no resistance to the baseplate returning to its original position under the influence of gravity once it is displaced.

Similar anchor elongations, approximately 60 mm, were observed at fracture when the repaired anchors were tested under direct tension. Fig. 2b shows the potential combinations of rotation and direct uplift that may be resisted by this system based on the testing performed. Anchor elongation associated with the direct-uplift testing greatly exceeds the vertical displacement of column bases observed in uplifting systems with thin baseplates, and is comparable to values observed in vertically unrestrained systems under very strong motions [3,4]. Taken with the large rotation capacity, the large direct-uplift capacity indicates that this system is appropriate for the range of behaviors that may be observed in a moment-frame structure during earthquake excitation, ranging from frame-dominated flexure to global uplift motion. Importantly, the successful removal and replacement of the anchors during testing provided proof of the practicality of repairing and re-using these connections.



(a) (b)

Fig. 2 − 19-mm diameter removable anchor connection concept test: (a) connection subjected to large (150mrad) rotation (b) connection uplift-rotation response test data and range of acceptable connection demand

2.2 Miniature building design, testing, and pre-test Analysis

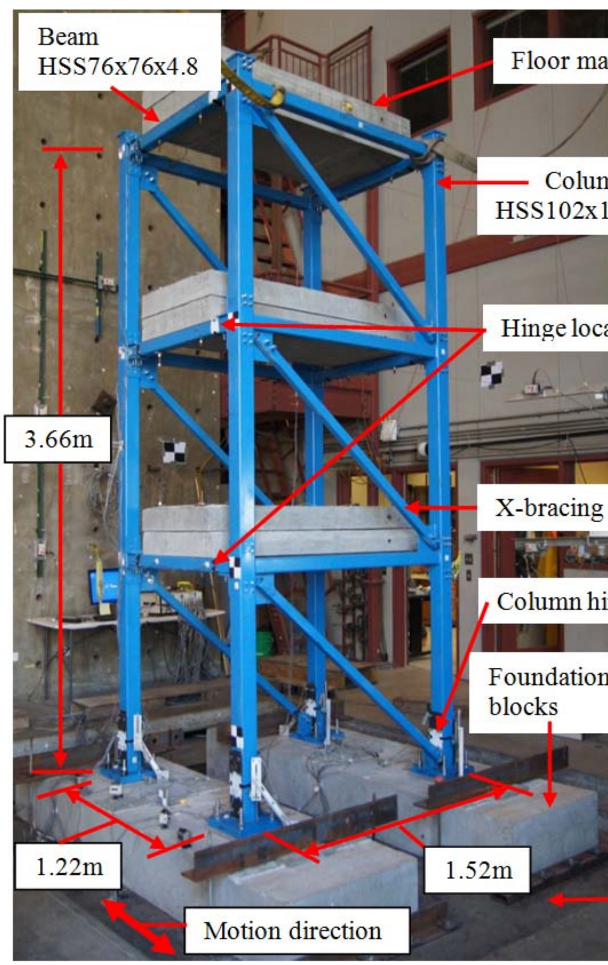
To investigate the system-level behavior of structures with ductile anchor uplifting connections, an experimental program and complimentary numerical analyses were conducted. The goals were to:

1. Compare the overall structural behavior of a building designed with uplifting baseplate connections incorporating ductile anchors to a "traditional" superstructure beam and column hinge design
2. Investigate the possible reduction of global structural demands (maximum drift, residual drift, maximum floor accelerations, etc.) for a ductile "anchor-dominated" design
3. Provide experimental, system-level dynamic response data documenting the distribution of demands from service to extreme earthquakes for use in numerical model validation

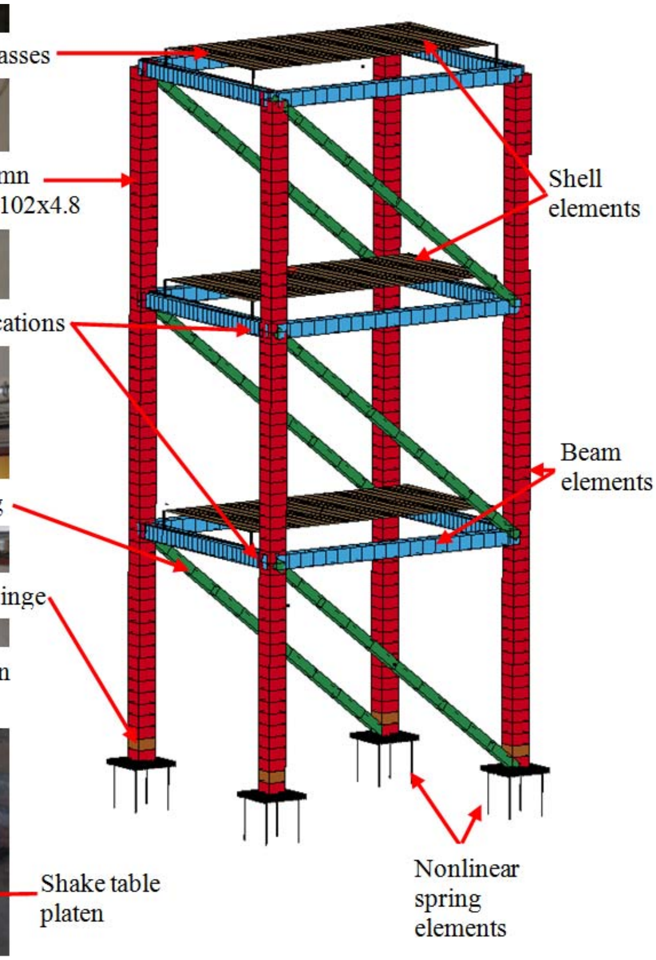
To these ends, an experimental structure consisting of a representation of a moment-frame building structure was constructed. The structure was designed as a "miniature building" – an approach that attempts to elicit the basic physical responses of a structure during an earthquake without providing exact time, force, and geometric similitude [10]. The primary advantage of this approach is that it allows multiple strong-motion tests with sacrificial elements at a reasonable cost. The building was approximately 1.2x1.5m in plan, with three stories of approximately 1.2m each (Fig 3a). The building was placed on two concrete footings, which were securely mounted on a shake table platen. The structure was comprised entirely of ASTM A500 Gr. B HSS sections (tested F_y=347 MPa), with double clip-angle connections at the beam-column joints. A 280-mm square, 38-mm thick ASTM A36 (tested F_y=290MPa) baseplate with a 200-mm 2x2 anchor pattern was welded to the bottom of each column. Concrete slabs were placed at each floor for a total mass of 26.7 kN. Diagonal bracing was provided for lateral stability.

The structure was constructed with strategically located areas of reduced cross section, herein termed *fuse elements*. In this case fuse elements were placed at locations of anticipated large moment and shear forces, namely the beam-column joints and at the column near the baseplate. Additionally, the baseplate could be re-configured to accept anchors of different sizes, ranging from 6.4mm diameter to 19mm, as shown in Fig. 4. The structure was configured to accommodate different ratios of beam or column fuse strength to anchor group strength. Two of the configurations that were used during testing, representing the extremes of intended behavior, are summarized in Table 1. Configuration 1 was intended to represent the design of a building with uplifting connections, and no special detailing in the building superstructure (i.e. there was no fuse cut into the beam and column elements). In this case, the response of the building was dominated by uplift and plasticity in the base connection, and therefore it is referred to as the base connection dominated (BD) model herein. Configuration 2 was a "traditionally designed structure", with beam hinges at beam-column connections and column hinges at the base of the structure. This structure was dominated by the behavior of these superstructure hinges, and is therefore referred to as the superstructure-dominated (SD) model.

In concert with the experimental program, a numerical model was developed using LS-DYNA [10] to assist in the design of the structure and allow prediction of the test structures response using nonlinear time-history analysis. The main features of this model are shown in Fig 3b. Note that the beams, columns, and diagonal bracing members were modeled with beam elements, while the floor slabs and column baseplates were modeled using shell elements. Based on the material behavior observed in tension-coupon tests, an elastic-plastic material model with kinematic hardening was selected for the beam and column elements away from the fuses, however observed plastic behavior in these regions was minimal. The elastic modulus, yield stress, and tangent modulus for each member is reported in Table 1. A resultant-type beam element with lumped sectional behavior was used to model the fuses. The moment-rotation response of these elements was defined based on the calculated expected moment-rotation response of the reduced sections. The force-displacement behavior of the anchors was similarly defined by the results of tensile testing of the anchor inserts. The support provided by the grout pads beneath the baseplates was modeled using compression-only springs. A damping ratio of approximately 2% was targeted at frequencies of about 3 Hz and 15 Hz, corresponding to the first and third modes through a combination of stiffness- and mass-weighted (Rayleigh) damping. This model was intended to reflect the level of detail commonly utilized in practice for the design of framed building structures, and is therefore referred to as the "design" model herein.



(a)



(b)

Fig. 3 − Test structure: (a) photograph of experimental setup and (b) "design" numerical model

Table 1 – Summary of material model parameters

Member	E (GPa)	F_y (MPa)	E_t (GPa)
Column and beam		347	0
Column baseplate	200	290	10
Column fuse		381	10

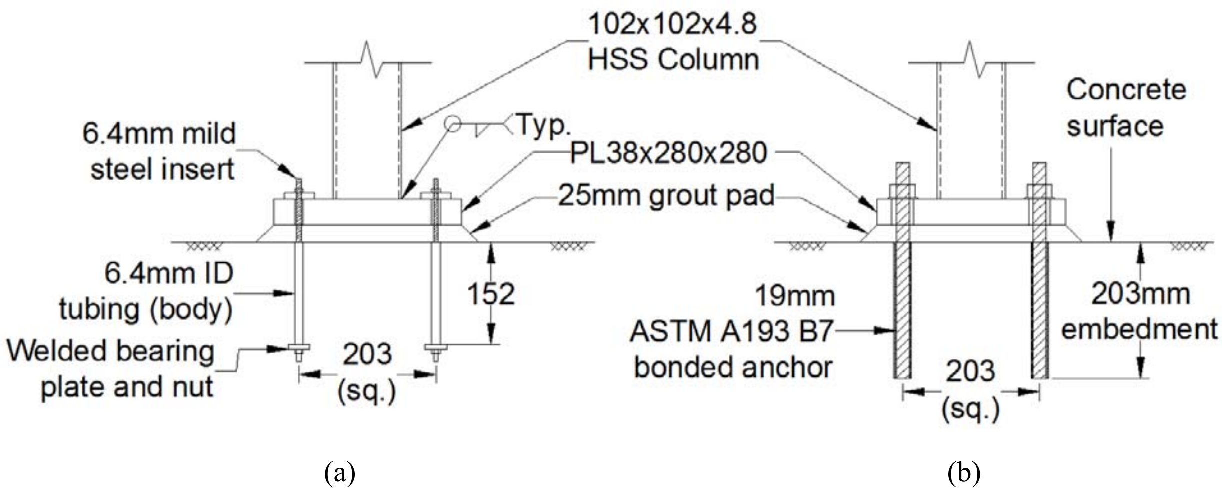


Fig. 4 − Details of the base connections utilized in the structure: (a) base connection dominated [BD] test structure and (b) structure dominated [SD] test structure.

Table 2 – Ratio of beam and column fuse strength

Configuration	Description	Beam Fuse Yield Moment (% of virgin)	Column Fuse Yield Moment (% of virgin)	Anchor Group Plastic Moment (% of Column Fuse)
1	Base connection dominated (BD)	100%	100%	17%
2	Structure dominated (SD)	15%	17%	2500%

3. Numerical analysis and test results

Prior to executing the experiments, the design model was used to predict the behavior of the structure in each configuration under a large suite of ground motions scaled to a variety of target intensities. Ground motions from this numerical analysis suite were then selected for use in the shake table tests. One of the motions utilized was the 1989 Loma Prieta event recorded at the Gilroy Array #1, 090 direction (record G01090 from the PEER Database [11]). This motion was scaled by 150%. The motion acheieved on the shake table is shown in Fig. 5. In general, the SD analysis results were characterized by broad hysteretic response, consistent with the typical response of a moment frame with ductile detailing. In contrast, the BD model responded nearly linearly, regardless of motion intensity. As a result, the SD structure was often left with significant residual drifts driven by significant plastic rotation at the beam-column connection hinges, although this result was dependent on the individual motion characteristics. In general, the maximum drift in the BD case was also somewhat larger. A representative analysis, in this case from the 150% scaled Loma Prieta-Gilroy motion, is shown in Fig. 6. Although indicative of the overall shape of the hysteretic curves in most cases, this analysis was somewhat unusual in that the total base shear was significantly higher in the BD configuration. However, this higher transient base shear did not translate into higher maximum roof drift. More importantly, the residual roof drift was approximately 0.2% for the SD case and essentially negligible for the BD case. This behavior was found for the majority of ground motions examined in the numerical analysis suite. Although some ground motions resulted in small residual drift for the SD configuration, the majority resulted in significant residual deformation. For the BD configuration, no motions resulted in residual drifts higher than 0.05%, indicative of self-centering, damage-free behavior.

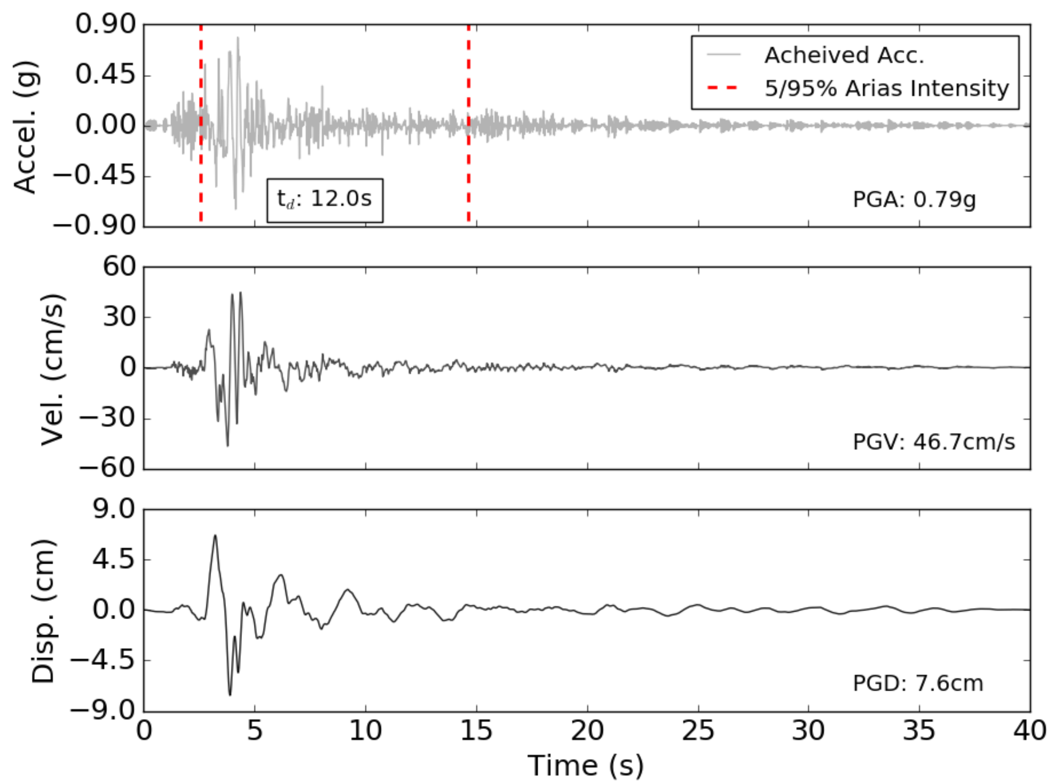
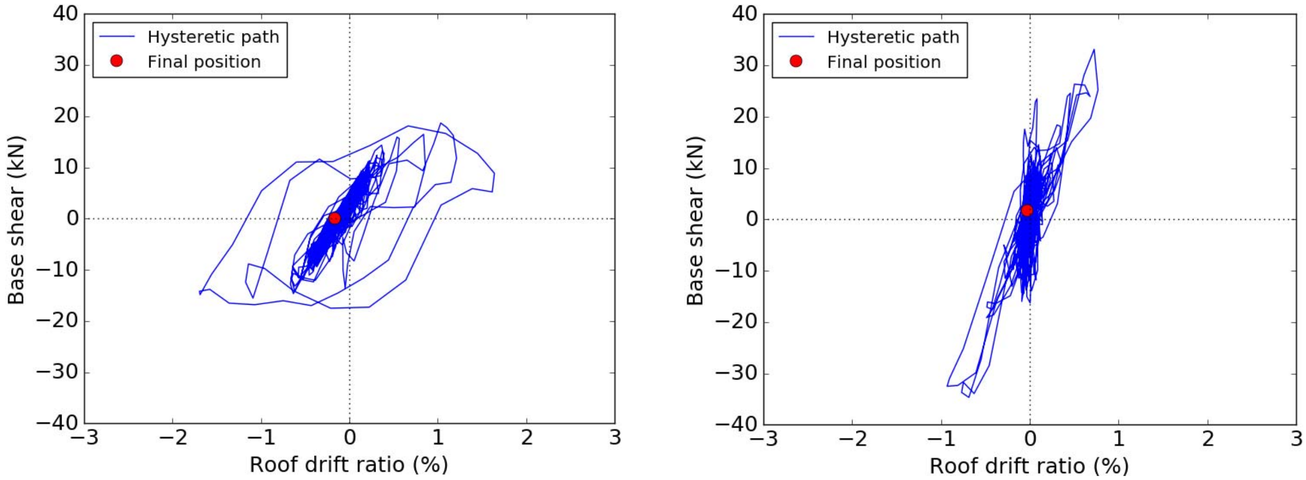


Fig. 5 − 1989 Loma Prieta recording from Gilroy Array #1, scaled by 150% (G01090 150%) – as achieved as base input.



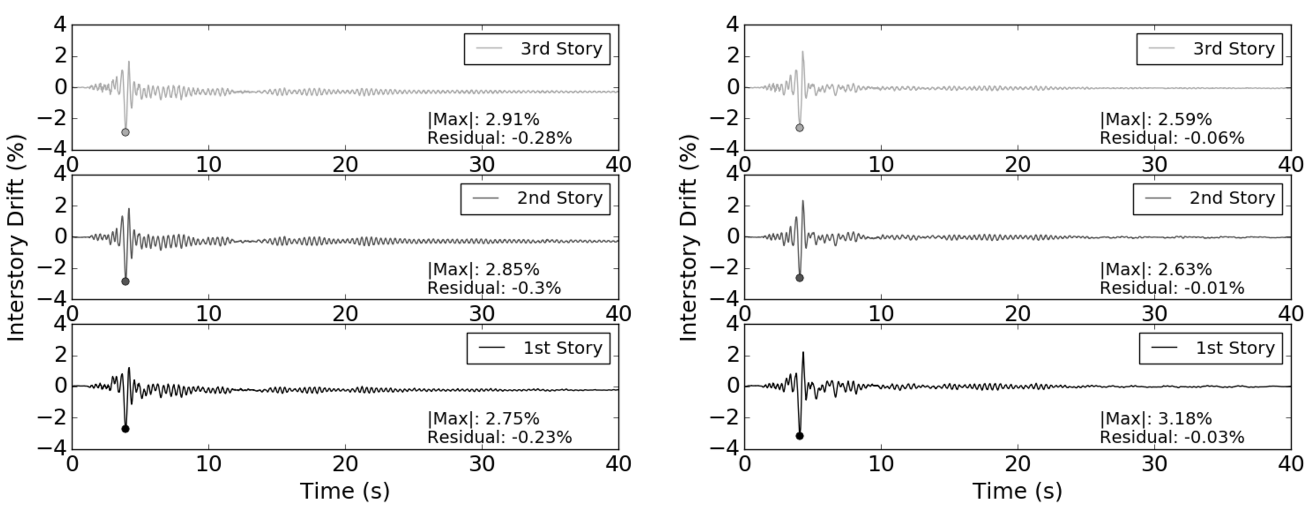
(a) **(b)**

Maximium base shear: 19.8kN
Maximum drift: 1.85%
Maximum residual drift: 0.21%

Maximum base shear: 34.7kN
Maximum drift: 0.93%
Maximum residual drift: 0.04%

Fig. 6 − Calculated base shear versus roof drift response and maximum response parameters under 150% G01090 motion: (a) structure-dominated configuration [SD], (b) BP-connection dominated configuration [BD]

A complete description of the experimental program is outside the scope of the current paper. However, it is noted that the results of the test program, particularly the maximum and residual drifts, were consistent with the numerical analyses performed as part of the initial design. The measured interstory drift ratios from the SD and BD configurations under the 150% G01090 motion are presented in Fig. 7. The maximum interstory drifts were very similar between the two tests. However, the average residual drift was approximately 0.27% in the SD configuration and about 0.03% in the BD configuration. Given that residual drift is often used as a measure of the repair and/or replacement cost of a structure, these results support the potential for uplifting connections with ductile anchors to reduce repair costs associated with earthquake damage.



(a) (b)

Fig. 7 − Interstory drift history under 150% G01090 motion: (a) structure-dominated [SD] configuration, (b) BP-connection dominated [BD] configuration

A partial explanation for the lower residual drift of the structure in the BD configuration may be traced to changes in the structure boundary conditions caused by yielding of the anchors. In addition to strong-motion excitation tests, the structure was subjected to 120 seconds of 0.75%g RMS white noise (WN) in order to determine the natural frequencies and mode shapes of the structure. The WN tests were conducted strategically before and after each earthquake motion in order to determine whether the earthquake test caused appreciable changes to the dynamic characteristics of the structure. Analysis of these tests indicates that the modes and mode shapes of the SD configuration were essentially unchanged throughout testing, regardless of the prior motion intensity or residual drift. The first natural frequency of this configuration was about 2.91 Hz (0.344s). However, the natural frequency of the BD configuration changed significantly when measured immediately before and after testing for all earthquake tests. Typical shifts in the first two measured transfer function peaks, corresponding to the first and second modes in the direction of shaking, were on the order of 15%. A representative plot is shown in Fig. 8. This reduction in natural frequency is caused by the loss of rotational restraint after the anchors elongate during the earthquake strong motion.

The reduction in natural frequency may be recognized as a "fuse" behavior, as there is a significant, sudden change in the dynamic properties of the system -- in this case, the change is expected *a priori* and designed at a strategic location. This particular fuse behavior has a number of attractive attributes. First, reduction in natural frequency is often associated with smaller spectral accelerations in both observed and design response spectra. This particular structure's response was below the absolute peak of the response spectrum of this particular motion, therefore only a small reduction in spectral acceleration was realized for this test (from about 2.5g to about 2.4g as shown in Fig. 9). However, depending on overall site and motion characteristics, a 15% reduction in the natural frequency of a structure could lead to significantly smaller inertial forces, reduced initial structure cost, and reduced repair costs. This is true whether or not the most favorable case of being on the "downslope" of the response spectrum occurs, as evidenced by the much smaller residual drifts in the dynamic test results presented above). Second, this mechanism is only "activated" once the anchors have yielded, which would occur due to strong forces. A structure could potentially be designed for (presumably lower) inertial forces associated a strong motion event with the lower, "yielded" natural frequency. At the same time, the structure would behave in a stiffer, more desirable manner under loads that did not yield the anchors, such as wind and small seismic events.

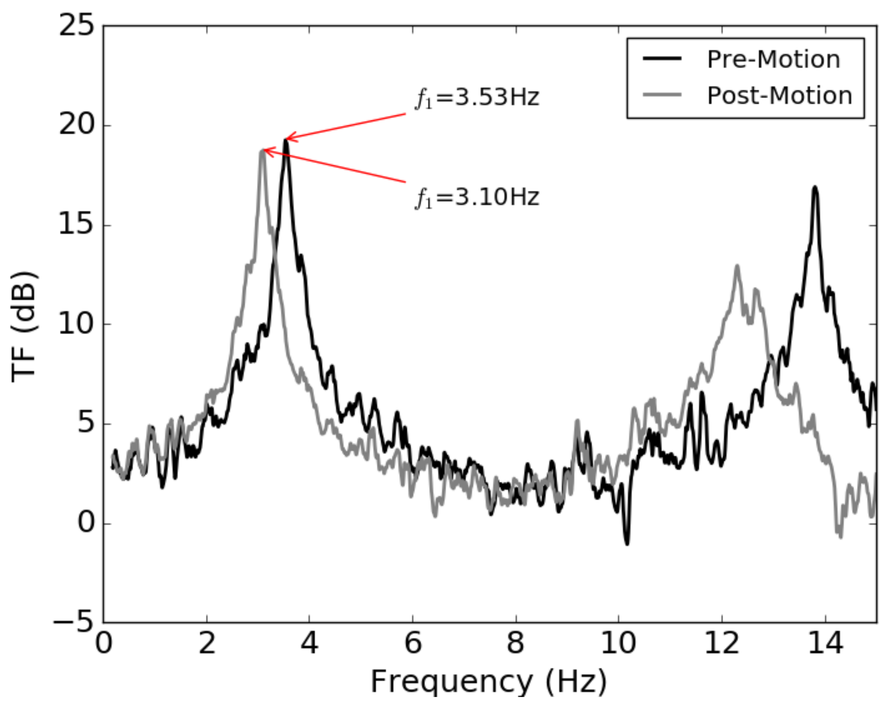


Fig. 8 – Example of shift in natural frequency associated with yielding of anchors during earthquake test (base-connection dominated configuration, 150% G01090 motion)

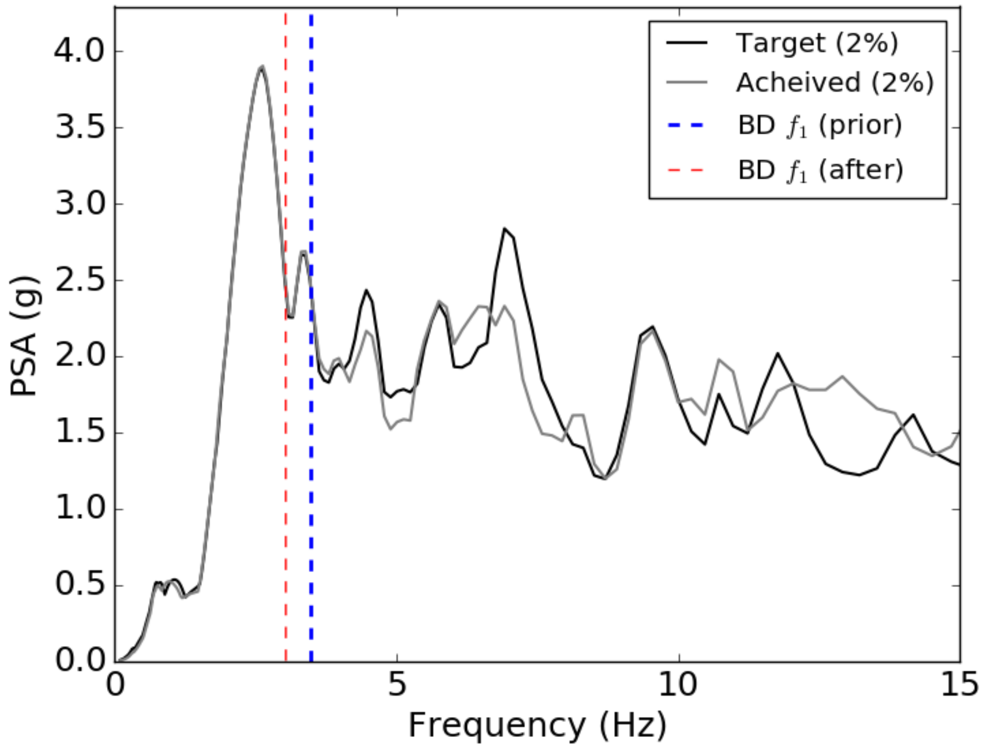


Fig. 9 – Elastic 2%-damped response spectrum of achieved motion with virgin and post-motion first natural frequency for the BD model identified.

4. Summary and conclusions

This paper presents preliminary experimental and numerical simulation results from a test program considering uplifting-baseplate connections constructed with ductile, replaceable anchors. Specifically, herein we present preliminary proof-of-concept testing of replaceable anchors, nonlinear time history analysis for the design of a miniature building system, and dynamic shake-table testing of a three-story, miniature experimental moment-frame structure. The following are the main conclusions of this work:

1. Full-scale testing and observations of anchorages in previous earthquakes indicate that ductile concrete anchors can provide highly robust, self-centering connection performance, allowing very large rotations and/or connection uplift that is consistent with the levels of deformation required by other previously-tested column uplift systems. Anchors constructed with threaded inserts can be quickly and easily repaired by replacing the damaged anchor components.
2. Nonlinear time history simulation of moment-frame building structures indicates that residual frame drift may be reduced significantly in structures that incorporate ductile anchors. Depending on the particular motion characteristics, this may be accompanied by a modest increase or decrease in the total base shear.
3. Shake-table testing indicates that residual interstory drift can be significantly reduced through the use of uplifting connections with ductile anchors. For a relatively intense motion, residual interstory drift was reduced from approximately 0.25% in a structure with beam-column connection hinges to a negligible amplitude of 0.03% in a similarly-designed structure with no hinges and a uplifting base connections
4. Performance improvements are partially due to the reduction in rotational restraint caused by yielding of the anchors, which causes a significant, instantaneous reduction in the natural frequency of the structure. In the current work, this reduction was found to be about 15% for the first two modes of the structure, but this could be larger or smaller depending on the exact structural configuration.

The qualitative conclusions of the paper are limited to experimental specimens considered, although field observations have confirmed the beneficial effects of uplifting behavior in some industrial structures. Additional, ongoing work of the authors attempts to expand the relevance of the current findings to specific types of structures and help to formulate overall design guidelines.

5. Acknowledgements

This research was funded by Hilti Corporation and the University of California San Diego. Material and technical assistance was provided by Hilti North America. The authors gratefully acknowledge the support of these organizations. The authors would also like to thank Mr. John Silva, Dr. Roberto Piccinin, Dr. Philipp Grosser, and Dr. Ulrich Bourgund for their input, as well as the staff of the UCSD Powell Labs for their assistance during testing. Opinions, findings, and conclusions are those of the authors, and do not necessarily reflect views of the sponsoring organizations.

6. References

[1] Housner, G. W. (1963). The behavior of inverted pendulum structures during earthquakes. *Bulletin of the Seismological Society of America* 53:2, 403-417.

[2] Rutenberg A, Jennings PC, Housner GW. (1982). The response of Veterans Hospital Building 41 in the San Fernando Earthquake. *Earthquake Engineering and Structural Dynamics*; 103:359–379.

[3] Clough RW, Huckelbridge AA. (1977). Preliminary experimental study of seismic uplift of a steel frame. Report No.UBC/EERC-77/22, EERC, University of California, Berkeley, CA.

[4] Midorikawa, M., Azuhata, T., Ishihara, T., and Wada, A., (2006). "Shaking Table Tests on Seismic Response of Steel Braced Frames with Column Uplift," *Earthquake Engineering and Structural Dynamics*, pp. 1767-1785.

[5] Midorikawa, M., Toyomaki, S., Hori, H., Asari, T., Azuhhata, T., and Ishihara, T., (2008). "Seismic Response of Six-Story Eccentrically Braced Steel Frames with Columns Partially Allowed to Uplift," 14th World Conference on Earthquake Engineering, October 12-17th, 2008, Beijing, China.

[6] American Concrete Institute (ACI) 318 Committee, (2014). *Building Code Requirements for Structural Concrete (ACI 318) and Commentary (ACI 318R)*, ACI, Detroit, MI.

[7] International Council of Building Officials, (1997). "Uniform Building Code," ICBO, Whittier, CA.

[8] Soules, J., Bachman, R., and Silva, J. (2012), "Report of ASCE/SEI Industrial Assessment Team on Earthquake Damage to Industrial Facilities in the Conception Area Resulting from the Chile Earthquake of February 27, 2011", American Society of Civil Engineers, Reston, VA.

[9] Trautner, C., Hutchinson, T.C., Grosser, P, and Silva, J. (2015). "Effects of Connection Detailing on the Cyclic Behavior of Steel Baseplate Connections," ASCE J. Struct. Eng., 10.1061/(ASCE)ST.1943-541X.0001361.

[10] Restrepo, J.I., Rodriguez, M.E., Kao-Stocker, G., and Carr, A. (2014). "Seismic Response of a Four-Story Miniature Building with Replaceable Plastic Hinges, Journal of Earthquake Engineering, 18:8, 1217-1240.

[11] Pacific Earthquake Engineering Research (PEER) Center, NGA-West2 Database, http://ngawest2.berkeley.edu/