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Characterizing the Load Deformation Behaviour of Steel Deck Diaphragms

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Abstract

Lateral loads flow through a building's horizontal roof and floor diaphragms before being transferred to the vertical lateral force resisting system (e.g. braced frames, moment frames or shear walls). These diaphragms are therefore a critical structural component in the resistance of lateral loads. A review of the literature shows that a large number of experimental programs have been performed to obtain the in-plane load-deformation behavior of steel deck and concrete on steel deck diaphragms. The tested diaphragm behavior was found to be dependent on a set of factors including loading protocol, fastener type, fastener size and spacing, and more. There does not currently exist a single, unifying review of these diaphragm tests and their relevant results. A research program is being conducted to collect and consolidate the available literature about tested steel deck diaphragms and their results. A database has been created that includes over 450 tested specimens with more than 130 cyclic tests. In addition, an effort is made to characterize diaphragms' load-deformation response as grouped by sidelap and support fastener type. The test programs and results collected into this database as well as the characterization of diaphragm behavior are discussed in this paper.

1.0 Introduction

There is strong evidence that diaphragms designed to current U.S. building codes undergo inelastic deformations during large earthquakes. Partial collapse of precast concrete parking garages during the 1994 Northridge earthquake were tied to inelasticity in diaphragm components that led to the failure of non-ductile gravity columns (EERI 1996). Subsequently shake table tests (e.g. Rodriguez et

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al. 2007) and computational simulations (e.g. Fleischman and Farrow 2001) have shown that current code level diaphragm forces can be significantly smaller than the elastic forces actually developed during large earthquakes. While some engineers and researchers in North America propose to increase diaphragm forces to ensure that diaphragms remain elastic (e.g. DeVall 2003, Nakaki 2000), others suggest that in some cases it may be more economical to design the diaphragm as the energy-dissipating element (Tremblay and Rogers 2005). An update to U.S. building code has been proposed through the Building Seismic Safety Council - Provisions Update Committee (BSSC-PUC) which significantly increases diaphragm design demands, but also allows reduced force design via a new force reduction factor, R_s, accounting for diaphragm ductility (NEHRP 2015).

The behavior of real three-dimensional buildings during earthquakes is complex, especially if the vertical lateral force resisting system (LFRS) and the horizontal LFRS (diaphragms) are both experiencing inelastic deformations. To understand the seismic performance of buildings including the interaction of vertical LFRS and horizontal LFRS inelasticities, it is crucial to have a clear understanding and characterization of the inelastic behavior of diaphragms. Although a large number of early experimental programs on steel deck diaphragms focused only on capturing stiffness and peak strength, more recent research programs also captured the post-peak behavior. These research programs on steel deck and concrete on steel deck diaphragms studied a large range of variables but no consolidated review of post-peak behavior exists.

A research project known as the Steel Diaphragm Innovation Initiative (SDII), a joint industry / NSF funded collaboration between Johns Hopkins University, Virginia Tech, and Northeastern University, aims to understand and improve the seismic behavior of steel framed buildings with steel deck diaphragms. As part of that effort, this paper has the following objectives: 1) the collection of experimental diaphragm research information including test setups, loading protocols, and results, into one comprehensive database, and 2) characterizing the behavior of the diaphragms including inelastic response and ductility. The database currently comprises 468 specimens obtained from research reports and papers. A subset of 86 specimens for which post peak behavior was available is briefly analyzed in this paper and future work to further analyze the dataset is discussed.

2. Diaphragm Database

2.1 Typical Diaphragm Components and Test Setup

Figure 1 demonstrates some of the structural components that are part of a typical steel deck diaphragm. The steel deck panels are corrugated and fastened to the structural frame using perimeter member fasteners and interior member fasteners such as arc spot welds, powder actuated fasteners, self-drilling screws and in cases of concrete on metal deck diaphragms, headed shear studs. Sidelap fasteners, such as screws, welds, or mechanical crimping (e.g. button punch) connect adjacent panels to each other. Similarly, end lap fasteners connect the ends of steel decking sheets to each other, often at interior members.

The most common testing methodology for diaphragms is the American Institute of Steel Construction, AISI, cantilever test method (AISI, 2013). This test method subjects a cantilevered diaphragm to a specified displacement protocol applied at its free end. Note that the length or span of an experimentally tested diaphragm is defined as transverse to the applied load, while the depth is defined as parallel to the load (see Figure 1).

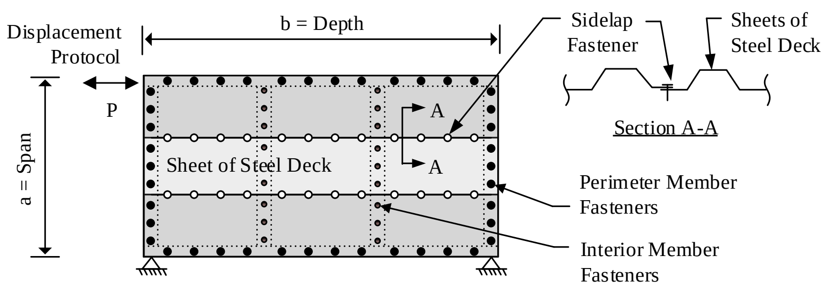


Figure 1. Cantilever Test Layout with Fastener Locations

2.2 Diaphragm Database

The objective of the diaphragm test database is to consolidate all pertinent data related to experimental test specimens to allow comparison between groups of specimens across multiple research programs and analyze resulting load-deformation behavior as a group rather than as individual tests. Categories of data collected includes geometry, materials, monotonic or cyclic loading protocol, fastener configuration, and results. Some information was unavailable for some specimens or testing programs while other references included complete data. Test setup data was deemed relevant if it might have contributed to the load-deformation behavior of the specimen, and is thus described in the database.

The geometry of the diaphragm specimens includes the dimensions of the diaphragm and the size of perimeter members. Some diaphragm tests utilized large framing sections to allow reuse of the testing frame, but may not be representative of steel framed building construction. Geometric properties of the steel deck such as profile, thickness, length and cover width of steel deck panels have been shown to have substantial effect on diaphragm behavior and were thus documented in the database. Luttrell and Winter (1965) showed that deck warping at panel ends is independent of panel length and therefore concluded that longer steel deck panels considerably increase diaphragm stiffness with minimal effects on diaphragm strength. Increasing cover width resulted in similar results with increasing strength, but proved to contribute less drastically to the behavior than increasing panel length. Material properties of the steel decking (e.g. yield strength and ultimate strength) also have been shown to affect diaphragm behavior (Ellifritt and Luttrell 1970) and thus nominal and measured material properties were input in the database wherever available.

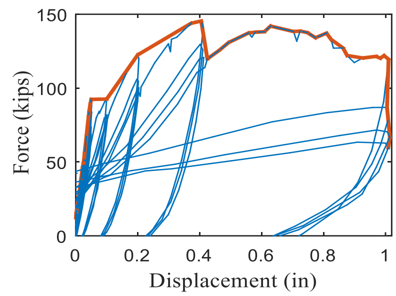


Figure 2. Cyclic Envelope

Although monotonic loading protocols (e.g. loading rate) may have less influence on load-deformation behavior than cyclic loading protocol, time-dependent relaxation effects and residual displacements in the diaphragm supports can affect results (AISI 2013). Conversely, diaphragm load deformation behavior can be heavily dependent on cyclic loading protocols. Cyclic loading protocols demonstrate the effects of strength degradation in the inelastic response range, observed as smaller load deformation envelopes or backbone curves than their monotonically loaded counterparts (Essa 2003). Some cyclic loading protocols can have extensive deformations in a single cycle (e.g. see Figure 2). For cyclic loading with large displacement steps, an envelope as shown in Figure 2 is more appropriate than a backbone curve to characterize the post-peak behavior, since a backbone curve only captures the peak data points from each cycle. For cyclic curves with closely spaced intermediate displacement cycles, it was deemed appropriate to capture the behavior of the diaphragm using backbone curves. Quasi-static or dynamic loading protocols and their respective load deformation data, when made available in the literature, are reported in the database.

Perhaps the most important factor in diaphragm behavior is the fastener type, spacing, and configuration. Diaphragm construction can include a variety of fastener types and patterns. For the early diaphragm test programs, common construction practice for steel framed buildings at the time was to button punch (BP) or weld sidelaps while welding the deck to the perimeter and interior members. As construction technology progressed, it has become increasingly common to use self-drilling screws and powder actuated fasteners (PAF) as sidelap and structural frame fasteners respectively. (Essa 2003) showed that the screwed sidelap and PAF support fasteners demonstrated more ductility than a diaphragm with support welds and button punched sidelaps. Deck to frame welds-with-washers also yielded ductile behavior, but are not yet common in the construction industry. Decreasing the spacing of interior supports increases the strength of a diaphragm, due to a larger number interior support fasteners reducing the probability of the deck buckling (Ellifritt 1970). The key fastener system variables logged in the database are location, type, size and spacing.

2.3 Review of Test Programs Included in the Database

A total of 468 specimens from 28 references and 11 research programs were reviewed, and input in the database as described in Table 1. A total of 329 specimens subjected to monotonic loading and 137 subjected to cyclic are included. Table 2 summarizes the fastener configurations for specimens included in the database. Populating the database is an ongoing effort and data is still being extracted from additional references not yet listed here.

Table 1. Overview of Research Programs in Experimental Diaphragm Database

Testing Program	Reference	Number of Specimens
Cornell University	Nilson 1960	39
West Virginia University	Ellifritt and Luttrell 1970, Apparao 1966, Luttrell 1967, Luttrell 1965, Luttrell 1971	205
University of Salford	Davies and Fisher, 1979	4
ABK, A Joint Venture	ABK 1981	3
Iowa State University	Porter and Greimann 1980, Neilson 1984, Easterling 1987	32
Virginia Tech	Hankins et al. 1992, Earls and Murray 1991, Pugh and Murray 1991, Bagwell 2007,	61
University of Montreal, McGill University	Martin 2002, Essa 2003, Yang 2003, Tremblay et al., 2004, Tremblay et al., 2008, Franquet 2009, Masseralli 2009, Masseralli et al., 2012	82
Tongji University	Liu et al. 2007	6
Hilti Corporation	Beck 2008, Beck 2013a, Beck 2013b	19
Tokyo Institute of Tech.	Shimizu et al. 2013	15
	TOTAL =	**468**

The first published research program on light gage steel diaphragms was conducted at Cornell University and included tests on 39 specimens (Nilson 1960). Nilson concluded that it is economical and sufficient to replicate diaphragm behavior through a cantilevered setup which would become the standard for diaphragm testing. Luttrell continued research on light gage steel decking at West Virginia University in the 1960's-70's, and focused on evaluating the effect of deck profile and geometry, material properties, and fastener type, size and spacing on a series of over 200 tests (e.g. Ellifritt 1970). Later testing investigated the effects of lightweight concrete on shear diaphragms (Luttrell 1971). Luttrell's research led to the development of SDI's Diaphragm Design Manual (Luttrell 2015), the most widely utilized design document for steel deck diaphragms.

Table 2. Number of Experimental Tests with Fastener Types

Deck to Frame Fasteners		Sidelap Fasteners	
Welds	87	Welds	56
Screws	70	Screws	139
PAF	82	BP	26
Other/Unavailable	233	Other/Unavailable	251

A series of public and proprietary research programs from the late 70's to late 80's further examined the influence of composite slab steel deck systems. Notably, the first, and one of the few, research programs with cyclic tests on composite concrete on steel deck diaphragms were performed at Iowa State University (Easterling 1987). Virginia Tech performed a series of industry tests on roofing systems and deck profile types in the 1990's and 2000's. Programs at the University of Montreal and McGill University focused on the inelastic performance of steel deck diaphragms subjected to both quasistatic and dynamic cyclic loading. Full scale test from Hilti Corporation and Tongji University investigated the ductile behavior of PAFs and self-drilling screws.

3. Discussion of Load-Deformation Behavior by Fastener Type

3.1 Introduction

Available load-deformation plots from the literature were digitized to allow unification of units, comparison between groups of specimens, and further analysis. A subset of 86 specimens for which post-peak data was available are presented in the following sections split into groups based on sidelap and support fastener type. All specimens were tested in a cantilever diaphragm configuration similar to Figure 1. Shear stiffness, *G'*, was obtained by connecting the first data point (displacement vs. unit shear load) to the data point at 40% of the ultimate test load, P_{ult}. In the following tables, the value of *G'* is multiplied by the aspect ratio, *a*/*b*, which adjusts for specimen geometry (AISI 2013). Ductility was calculated as the ratio of the displacement where the specimen strength degrades to 80% of the ultimate load to the yield displacement of the diaphragm. In this case, the yield displacement is defined as P_{ult} / G'. Also tabulated in the following sections are the ultimate unit shear strength, S_{ult}=P_{ult}/b, and ultimate shear angle, γ_{ult} = max displacement / a.

3.2 Bare Deck Specimens Subjected to Monotonic Loading

Table 3 presents the results for bare deck diaphragm specimens subjected to monotonic loading as grouped by support fastener type / sidelap fastener. Figure 3 and Figure 4 show plots of the associated data. The unit shear strength of the diaphragm specimens, S_{ult}, were mostly in the range of 0.396 k/ft (5.78 kN/m) to 1.88 k/ft (27.5 kN/m). Two research programs tested higher capacity diaphragms including Martin (2002) and Beck (2008, 2013a, 2013b) which included specimens with unit shear capacity as large as 6.07 k/ft (88.6 kN/m). Obviously, the strength and stiffness of diaphragms is highly dependent on the fastener spacing and deck type. Due to space restrictions, it was not possible to present all specimen information, nor is it the intent of this paper to study strength and stiffness which have been previously characterized (Luttrell 2015).

There is a marked difference in ductility between specimens with mechanical fasteners to the support as compared to specimens with welds to the support. Figure 3a shows load-deformation behavior of diaphragm specimens with PAF to the support. The average ductility for this group was 4.50 although the variation was especially large as demonstrated by the scatter in Figure 3a and a standard deviation of 3.46.

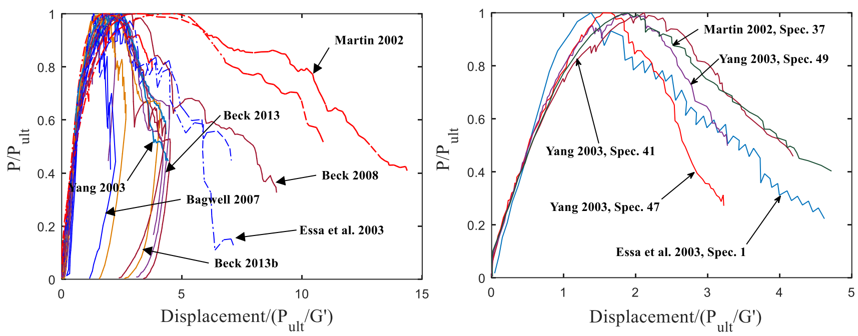
Martin (2002) specimens 32 and 19 were identical except PAF fasteners were at 6 in. (152 mm) vs. the more typical 12 in. (305 mm) which led to a substantial increase in ductility, (7.12 vs. 3.76, respectively). Martin (2002) specimen 30 was identical to specimen 32 but used thinner 0.030 in. (0.76 mm) B type roof deck vs. 0.036 in. (0.91 mm) thick and resulted in even larger ductility of 9.68. Bagwell (2007) studied deep deck and cellular deck wherein specimens 10 and 11 were 7.5 in. deep cellular deck with a steel sheet along bottom. Although these are not typical deck sections, they demonstrate that cellular deck can have extremely large ductility (13.6 and 13.8) because they mitigate limit states associated with deck deformations in favor of deformations at the support fasteners.

Table 3. Bare Deck Specimens Tested Monotonically
Grouped by Support Fastener Type / Sidelap Fastener Type

Reference	Spec. #	G'(a/b) kips/in	(kN/mm)	S_{ult} kips/ft	(kN/m)	γ_{ult} Rad*1000	Duc-tility, μ
PAF / Screw							
Martin 2002	19	24.2	(4.24)	1.14	(16.7)	14.8	3.76
Martin 2002	30	99.4	(17.4)	1.60	(23.3)	19.2	9.68
Martin 2002	32	130	(22.8)	2.36	(34.4)	16.5	7.12
Essa et al. 2003	5	15.7	(2.76)	0.759	(11.1)	28.7	3.11
Essa et al. 2003	17	22.9	(4.01)	0.991	(14.5)	25.5	3.22
Yang 2003	43	15.4	(2.71)	0.915	(13.4)	21.4	3.20
Yang 2003	44	14.9	(2.61)	0.718	(12.5)	17.7	3.25
Bagwell 2007	7	12.0	(2.10)	0.492	(7.18)	10.2	2.98
Bagwell 2007	8	13.5	(2.37)	0.533	(7.77)	6.68	1.56
Bagwell 2007	9	3.05	(0.533)	0.396	(5.78)	36.9	3.05
Bagwell 2007	10	35.5	(6.22)	0.495	(7.22)	20.4	13.8
Bagwell 2007	11	44.7	(7.82)	0.447	(6.53)	15.4	13.6
Bagwell 2007	17	89.2	(15.6)	2.50	(36.5)	5.24	1.79
Beck 2008	63	60.7	(10.6)	2.04	(29.8)	25.0	4.39
Beck 2008	64	67.8	(11.9)	3.06	(44.7)	17	3.20
Beck 2008	65	85.2	(14.9)	3.95	(57.7)	16.7	2.93
Beck 2013a	1	70.1	(12.3)	4.05	(59.1)	20.3	3.16
Beck 2013a	2	70.4	(12.3)	3.81	(55.6)	20.2	3.20
Beck 2013a	3	54.9	(9.62)	6.07	(88.6)	20.5	2.22
Beck 2013b	2	61.1	(10.7)	3.45	(50.3)	19.2	2.91
Beck 2013b	3	51.3	(8.99)	4.05	(59.1)	17.6	2.25
Average		49.6	(8.69)	2.09	(30.5)	18.8	4.50
Std. dev.		33.3	(5.83)	1.60	(23.3)	6.81	3.46
Weld / BP							
Martin 2002	37	24.9	(4.37)	0.858	(12.5)	13.5	2.81
Essa et al. 2003	1	11.8	(2.07)	0.542	(7.92)	17.6	1.96
Yang 2003	41	10.5	(1.84)	0.627	(9.15)	20.8	3.03
Yang 2003	47	5.24	(0.918)	0.496	(7.24)	25.4	2.23
Yang 2003	49	7.07	(1.24)	0.585	(8.53)	22.6	2.59
Average		11.9	(2.09)	0.622	(9.07)	20.0	2.52
Std. dev.		6.92	(1.21)	0.126	(1.84)	4.09	0.384
Weld / Screw							
Essa et al. 2003	11	19.1	(3.35)	1.23	(17.9)	30.0	2.32
Essa et al. 2003	15	22.0	(3.85)	1.30	(19.0)	29.0	3.81
Bagwell 2007	12	10.3	(1.80)	1.41	(10.5)	15.7	1.30
Bagwell 2007	13	57.4	(10.1)	1.05	(15.3)	6.55	N/A*
Bagwell 2007	14	32.3	(5.66)	1.88	(27.5)	9.00	1.84
Average		28.2	(4.94)	1.37	(20.0)	18.1	2.32
Std. dev.		16.2	(2.84)	0.281	(4.10)	9.84	0.935
Weld / Weld							
Martin 2002	22	27.0	(4.74)	2.21	(32.2)	14.8	1.79
Essa et al. 2003	9	13.1	(2.29)	0.811	(11.8)	33.4	2.99
Essa et al. 2003	10	13.1	(2.29)	0.985	(14.4)	28.1	2.01

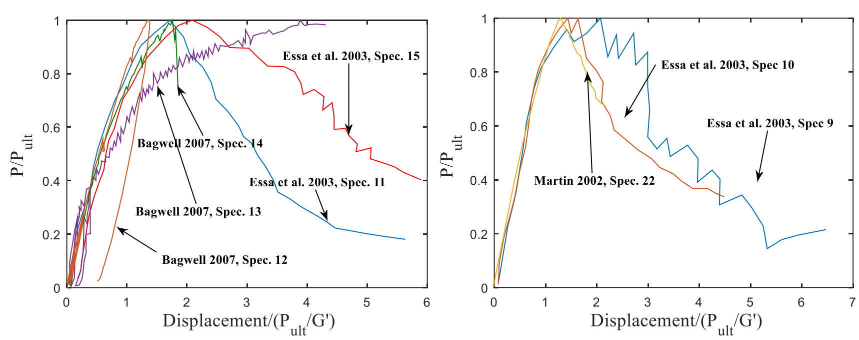
*Post peak-force deformations did not reach 80% of S_u
PAF = Power actuated fastener, BP = Button Punch

Specimens with welds to the supports (see Figures 3b, 4a, and 4b) experienced limit states such as distortion of the deck sheet ends, fracture at weld connections, often occurring in rapid succession, and slip at the sidelaps. Once failure of the deck support attachments occurred, there was often loss of load carrying capacity. It is shown, therefore, that ductility is not nearly as sensitive to the type of sidelap fastener as it is to support fastener type. Although there are slight gains in ductility with mechanical sidelap fasteners, once failure occurs at support welds, sidelap fasteners are often not as relevant.



(a) PAF to Support, Screw Sidelap (b) Weld to Support, BP Sidelap

Figure 3. Behavior of Monotonically Loaded Bare Deck Specimens



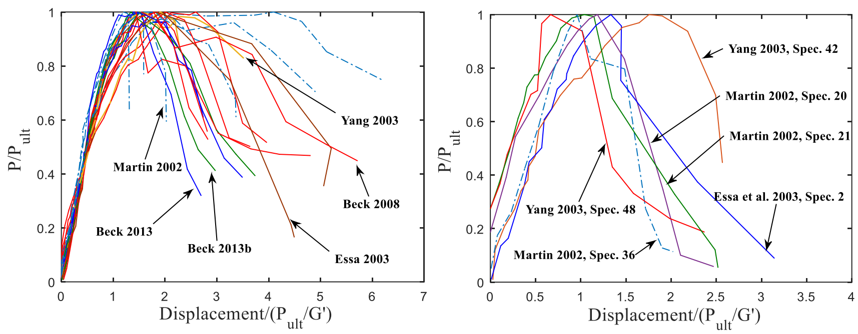
(a) Weld to Support, Screw Sidelap (b) Weld to Support, Weld Sidelap

Figure 4. Behavior of Monotonically Loaded Bare Deck Specimens

3.3 Bare Deck Specimens Subjected to Cyclic Loading

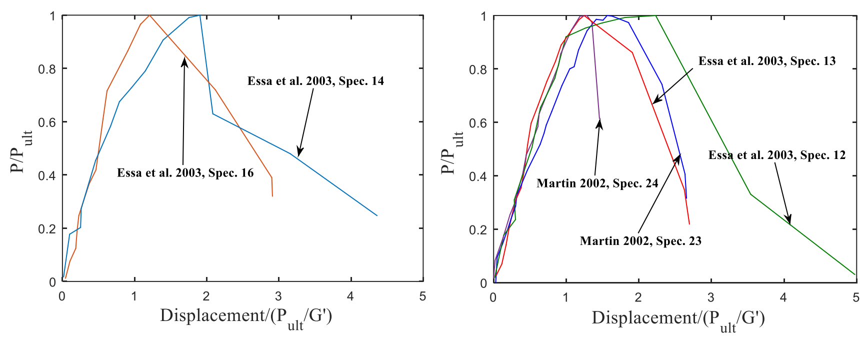
Table 4, Figure 5 and Figure 6 show data from similar specimens as the previous section, but subjected to cyclic loading. The average ductility value for PAF to support and weld support reduced by 39% and 23% to 2.75 and 1.83, respectively.

Strength degradation associated with cyclic loading causes a reduction in the available ductility of the diaphragm system. However, the trends described above are still applicable in that specimens with PAF to the support demonstrate more ductility than specimens with welds to the support. The standard deviation in ductility is shown to be smaller for the set of cyclically loaded specimens than the monotonically loaded group, although it is possible that is related to which specimens were selected to be in the group. This will be studied further in the future.



(a) PAF to Support, Screw Sidelap (b) Weld to Support, BP Sidelap

Figure 5. Behavior of Cyclically Loaded Bare Deck Specimens



(a) Weld to Support, Screw Sidelap (b) Weld to Support, Weld Sidelap

Figure 6. Behavior of Cyclically Loaded Bare Deck Specimens

Table 4. Bare Deck Specimens Tested Cyclically
Grouped by Support Fastener Type / Sidelap Fastener Type

Reference	Spec. #	G' (a/b) kips/in	(kN/mm)	S_{ult} kips/ft	(kN/m)	γ Rad*1000	Duc-tility, μ
PAF/Screw							
Martin 2002	28	12.1	(2.11)	0.959	(14.0)	13.4	1.97
Martin 2002	29	15.3	(2.67)	0.919	(13.4)	6.58	1.30
Martin 2002	31	65.4	(11.4)	1.81	(26.4)	11.3	4.37
Martin 2002	33	114	(20.0)	2.40	(35.0)	10.8	5.66
Martin 2002	34	24.7	(4.33)	1.16	(16.9)	13.1	3.04
Martin 2002	35	26.5	(4.63)	1.18	(17.2)	5.90	1.59
Essa et al. 2003	8	16.2	(2.83)	0.850	(12.4)	19.7	2.98
Essa et al. 2003	18	26.3	(4.60)	1.07	(15.6)	17.7	4.00
Yang 2003	38	23.1	(4.05)	1.04	(15.1)	13.1	N/A*
Yang 2003	40	10.6	(1.86)	0.884	(12.9)	15.8	N/A*
Beck 2008	3	72.3	(12.7)	3.96	(17.8)	18.1	3.20
Beck 2008	4	44.9	(7.86)	3.43	(50.0)	17.9	2.41
Beck 2008	5	46.1	(8.07)	3.48	(50.8)	17.8	2.26
Beck 2008	6	73.4	(12.9)	4.33	(63.2)	17.9	2.76
Beck 2008	7	59.6	(10.4)	2.08	(30.3)	16.6	3.79
Beck 2008	8	45.6	(7.99)	1.93	(28.2)	16.9	1.65
Beck 2013a	1	48.7	(8.54)	4.11	(60.0)	18.9	1.88
Beck 2013a	2	61.6	(10.8)	3.93	(57.3)	18.5	2.42
Beck 2013a	3	57.2	(10.0)	5.77	(84.3)	23.0	2.40
Beck 2013b	2	58.4	(10.2)	3.47	(50.6)	18.5	2.50
Beck 2013b	3	49.5	(8.67)	4.09	(59.7)	20.4	2.08
Average		45.3	(7.94)	2.52	(36.7)	15.8	2.75
Std. dev.		25.1	(4.4)	1.47	(21.4)	4.28	1.06
Weld/BP							
Martin 2002	20	16.8	2.95	0.674	9.83	8.23	1.51
Martin 2002	21	15.2	2.66	0.932	13.6	12.9	N/A*
Martin 2002	36	14.0	2.46	0.672	9.81	8.08	1.46
Essa et al. 2003	2	12.3	2.15	0.517	7.54	11.0	1.45
Yang 2003	42	11.2	1.96	0.696	10.2	13.3	2.36
Yang 2003	48	4.02	0.705	0.449	6.56	23.4	1.25
Average		12.3	2.15	0.657	9.58	12.8	1.60
Std. dev.		4.12	0.721	0.153	2.23	5.14	0.389
Weld/Screw							
Essa et al. 2003	14	18.3	3.21	0.884	12.9	17.5	2.00
Essa et al. 2003	16	16.0	2.80	1.30	19.0	19.7	1.86
Weld/Weld							
Martin 2002	23	164	(28.7)	2.35	(34.3)	15.8	2.20
Martin 2002	24	26.7	(4.67)	2.27	(33.1)	10.3	1.41
Essa et al. 2003	12	14.0	(2.45)	0.712	(10.4)	21.1	2.62
Essa et al. 2003	13	11.2	(1.97)	0.888	(13.0)	17.8	2.00
Average		54.0	(9.45)	1.56	(22.7)	16.2	2.06
Std. dev.		63.8	(11.2)	0.757	(11.1)	3.91	0.439

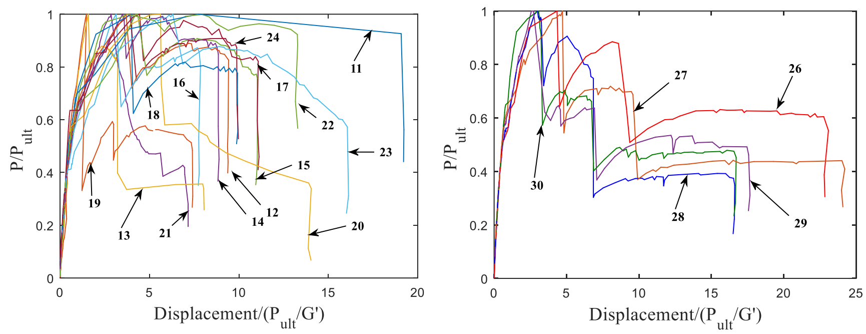
*Post peak-force deformations did not reach 80% of S_u
PAF = Power actuated fastener, BP = Button Punch

3.4 Concrete on Steel Deck Specimens

Only 20 concrete fill on steel deck diaphragm specimens found in the literature included post-peak load-deformation behavior (Easterling 1987). Deck to frame fasteners were grouped into two categories: welded and welded with headed shear studs. Table 5 and Figure 7 present some of the results. Easterling (1987) identified three limit states of practical significance: 1) diagonal tension cracking of the slab, 2) interface failure between deck and concrete (does not apply when headed shear studs are present), and 3) edge connector failure. Specimens 11 through 24 shown below were reported to experience all three limit states. Specimens reported as failing in interface failure (e.g. 11, 14, 17) exhibited some of the largest ductilities. Conversely, specimens reported as experiencing diagonal tension cracking exhibited some of the smallest ductilities (e.g. 12, 13, 16, 18, 19, 24). Specimens with headed shear studs experienced either diagonal tension cracking (specimens 26 and 29) or edge connector failure (specimens 27, 28, 30), although the difference in terms of ductility was not substantial.

Table 5. Specimens with Concrete on Metal Deck Tested Cyclically Grouped by Support Fastener Type (Easterling 1987)

Spec. #	G' (a/b)		S_{ult}		γ	μ
	kips/in	(kN/mm)	kips/ft	(kN/m)	Rad*1000	
Welded						
11	1770	310	6.11	89.2	5.53	19.1
12	1710	300	12.1	176	5.53	3.92
13	2020	354	16.8	245	5.57	3.23
14	1840	322	14.1	205	5.66	8.85
15	1130	198	6.84	99.8	5.56	4.78
16	920	162	8.01	117	5.69	3.29
17	1600	279	9.70	141	5.63	11.1
18	1580	277	10.7	156	5.61	4.03
19	1820	319	16.5	241	5.61	1.40
20	1300	228	6.21	90.6	5.58	5.65
21	870	152	8.16	119	5.61	3.27
22	1650	290	10.5	153	7.02	13.2
23	1370	240	7.09	103	6.97	12.3
24	1330	232	11.2	164	7.03	4.20
Average	1490	262	10.3	150	5.9	7.02
Std. dev.	338	59	3.43	50.2	0.58	4.93
Welds with Headed Shear Studs						
26	1590	279	5.80	84.7	7.01	4.45
27	1751	307	6.07	88.6	7.00	4.76
28	1580	277	7.98	116	6.98	3.37
29	1890	331	9.00	131	7.02	3.13
30	1530	269	7.69	112	6.98	3.27
Average	1670	292	7.31	107	7.00	3.80
Std. dev	131	23.0	1.20	17.6	0.016	0.673



(a) Welds to Support (b) Headed Shear Studs

Figure 7. Behavior of Cyclically Loaded Specimens Having Concrete Fill

4. Summary, Conclusions and Ongoing Work

As our design methods evolve to better predict diaphragm demands during seismic events, it is increasingly important to understand the full load-deformation behavior of steel deck diaphragms. This understanding is also critical for accurate assessment of building behavior and associated performance based earthquake engineering. In this paper, a database of past tests on steel deck diaphragms was described. Results from monotonic and cyclic tests on steel deck diaphragms and concrete filled steel deck diaphragms were plotted in groups based on support fastener type and sidelap fasteners type. Ductility was calculated for each specimen and compared between groups. The average ductility of monotonically loaded bare deck specimens with PAF and welds to the support was 4.50 and 2.39, respectively. Cyclically loaded bare deck specimens exhibited average ductility of 2.75 and 1.83 for PAF and welds to the support, respectively. Concrete on metal deck specimens produced ductility of 7.02 and 3.80 for welds to the support and headed shear studs, respectively. This demonstrates that steel deck and concrete on metal deck diaphragms can exhibit substantial post-peak inelastic load carrying capacity. This could be a very important factor as to why steel-framed buildings with these types of diaphragms survive large earthquakes without the types of collapses observed in precast concrete diaphragms.

The database and preliminary analysis of ductility is an important first step toward characterizing steel deck and concrete on metal deck diaphragm inelastic behavior. Ongoing work includes examining the diaphragm parameters and limit states that affect ductility and the variability in ductility. Load-deformation behavior will be characterized including backbone and pinching behavior. Overstrength will be examined by comparing strength with capacities calculated using the SDI Design Manual (Luttrell 2015). Finally, appropriate diaphragm force reduction factors, R_s, consistent with recently proposed design procedures (NEHRP 2015) will be proposed.

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