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IN-PLANE MONOTONIC AND CYCLIC TESTING OF STEEL ROOF DECK DIAPHRAGMS WITH NAILED AND WELDED CONNECTIONS

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ABSTRACT

A series of in-plane shear tests were carried out on a number of large-scale steel roof deck diaphragms. This test program was initiated and designed to evaluate the seismic inelastic response of steel roof decks with different thickness and different types of deck-to-frame connections: nails and arc spot welds. Self-drilled screws were used for sheet side lap fasteners in all specimens. All diaphragm specimens were tested using a cantilever steel test frame with pinned corner connections and intermediate joist beams. The tests included monotonic and quasi-static reversed cyclic inelastic deformation. Shear performance and failure mode of the steel decks for both types of deck-to-frame connections were investigated in this series of tests. Resistance of steel decks with different panel thickness and connector type was determined. The inelastic response and hysteretic behavior of the decks under large deformation condition at cyclic loading was studied. Testing of all specimens confirmed that the inelastic deformation of a deck is mainly concentrated on the edge of the diaphragms parallel to the lateral loading. The cyclic tests showed a pinched hysteretic behavior for all the specimens. Nail specimens sustained large inelastic deformation cycles with progressive strength degradation. In contrast, weld specimens showed very significant deterioration and very rapid strength reduction after the peak load was reached. It seems from these tests that the response modification factor for steel deck systems with nail-screw connections should be greater than the current value in the building codes.

Introduction

Steel roof deck diaphragms are commonly used for single-story buildings in North America which are typically occupied for industrial and commercial purposes. The structure of these buildings is composed of steel decks and vertical braced frames. Steel deck is made of corrugated steel sheets connected to one another in side laps and to the perimeter beams in end labs, as well as to the joist beams. Screw, button punch or weld with washer is usually used for side laps and nail, weld or weld with washer for deck-to-frame connections. Lateral loads due to wind or earthquake

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motion is resisted by steel deck and vertical braces. Current design codes, e.g. Canadian Standards Association, S16 Design of Steel Structures (SCA 2009), recommend that inelastic demand should be limited to the vertical braces of the frame, while the other elements including the roof decks remain in elastic range. An alternative approach for capacity based design allows inelastic response to occur in the roof decks while the braces retain elastic behavior (Tremblay 2004). In such cases, the roof deck is considered to act as a ductile fuse for the lateral load path instead of vertical braces and should sustain large inelastic deformation cycles without significant strength degradation. It results in thinner deck panels with reduced number of fasteners that makes the structure more cost-effective compared to the current approach for seismic design. Several studies have been conducted to evaluate the inelastic performance of steel roof decks (Rogers 2003), (Essa 2003) and (Tremblay 2004) but more research is needed to practicize the idea of using inelastic capacity of roof decks as an alternative approach in seismic design.

This paper presents the results of a series of in-plane shear tests to investigate the seismic inelastic response of steel roof deck diaphragms with different deck thickness and different types of deck-to-frame connections. The present study focuses on the evaluation of elastic stiffness, shear strength, initial yield and failure drift of deck specimens under monotonic and reversed cyclic loading.

Inelastic Response of Roof Diaphragms

Generally, the behavior of structures in major earthquakes is inelastic. Seismic codes allow structures to respond in the inelastic range in order: (1) to reduce the design seismic loads so that structures are cost-effective and economical and (2) to dissipate seismic energy through plastic deformations. This strategy is acceptable for buildings with steel roof decks while satisfies the life safety requirements. The expected and reported inelastic deformation of the steel roof diaphragms in typical single-storey structures during an earthquake is illustrated in Fig. 1. Lateral inertia load is generated due to the earthquake motion and roof mass and transferred to the ends of the deck towards the lateral resistant supports. In-plane shear force tends to be maximum near the end beams and causes the shear deformation at the edges of the roof.

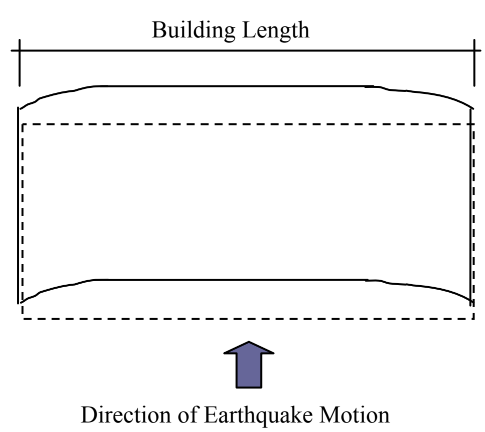

Figure 1. Inelastic deformation of steel roof diaphragms in typical single-story structures during a severe earthquake.

Testing Program

A testing program was conducted at the University of British Columbia to study the response of steel roof deck diaphragms during earthquakes (Ventura and Motamedi 2008). This test program was initiated and designed to evaluate the response of nineteen deck specimens with three different types of deck panels (0.75, 0.91 and 1.2 mm thick) and two types of deck-to-frame connections (nail fastener and welded connection). Previous studies have shown that severe deformation zones are concentrated near the end supports of the roof. Therefore, the test program was designed to represent a half portion of a roof diaphragm and replicate the observed behavior. The objectives of this test program were: (1) determine the shear performance and failure mode of the steel decks and the connectors; (2) evaluate and compare the resistance of steel deck roofs with different panel thickness and connector type; and (3) review the inelastic response and hysteretic behavior of the steel roof decks under large deformation condition at reversed cyclic loading.

Description of Specimens

Nineteen specimens with dimension of 6.15m x 2.75m were constructed on a steel test frame. They differed in the type of fasteners and the thickness of the panel. Each one was made of six 0.75, 0.91 or 1.2 mm thick corrugated steel panels with a depth of 38 mm and flutes spaced at 152 mm o/c. Deck panels were 0.94 m width and 3.2 m long with an end lap connection at the specimen midpoint. They were connected to one another in side laps and to the perimeter test frame members, as well as to the joist beams spaced at 1.52 m. A schematic plan view layout of deck-to-frame and side lap connections is illustrated in Fig. 2.

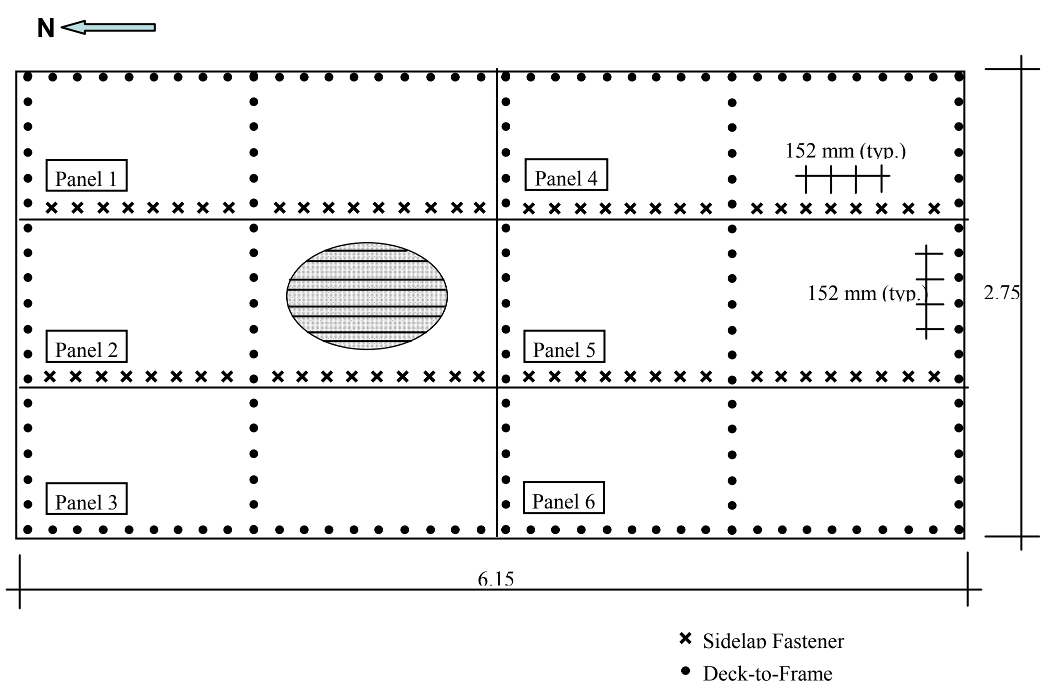

Figure 2. Schematic plan view of test specimens and layout of deck-to-frame and sidelap connections.

Three types of connectors were used for connecting the panels: The self-drilled screws for side lap fasteners and K64062 nails or 20 mm diameter arc spot welds for deck-to-frame fasteners. For simplicity, the fastener types are referred as “Nail” and “Weld” in the text and figures. For all these specimens, the spacing of the fasteners was 152 mm in direction perpendicular and every flute in the direction parallel to the loading. Table 1 provides the characteristics of the entire diaphragm tested.

Table 1. Characteristics of the specimens, test program and the results of the tests.

Specimen No.	**Deck Thickness (mm)**	**Type of Deck-to-Frame Connections**	**Loading Type**	**Peak Shear Resistance (KN)**	**Elastic Stiffness (KN/mm**
Specimen 1	0.91 (20 Gage)	Nail	Monotonic/Cyclic	85	1.35
Specimen 2	0.91 (20 Gage)	Nail	Monotonic	88	1.4
Specimen 3	0.91 (20 Gage)	Nail	Cyclic	93	1.45
Specimen 4	1.2 (18 Gage)	Nail	Monotonic	118	1.5
Specimen 5	0.91 (20 Gage)	Weld	Monotonic	92	1.4
Specimen 6	0.91 (20 Gage)	Weld	Cyclic	60	1.4
Specimen 7	0.75 (22 Gage)	Nail	Monotonic	64	1.3
Specimen 8	0.75 (22 Gage)	Nail	Cyclic	56	1.3
Specimen 9	0.75 (22 Gage)	Nail	Cyclic	50	1.15
Specimen 10	0.75 (22 Gage)	Nail	Cyclic	52	1.2
Specimen 11	0.91 (20 Gage)	Nail	Cyclic	65	1.5
Specimen 12	0.91 (20 Gage)	Nail	Cyclic	67	1.35
Specimen 13	1.2 (18 Gage)	Nail	Cyclic	67	1.45
Specimen 14	1.2 (18 Gage)	Nail	Cyclic	77	1.55
Specimen 15	1.2 (18 Gage)	Nail	Cyclic	65	1.0
Specimen 16	1.2 (18 Gage)	Nail	Cyclic	115	1.6
Specimen 17	0.91 (20 Gage)	Nail	Cyclic	80	1.45
Specimen 18	0.91 (20 Gage)	Weld	Cyclic	55	1.45
Specimen 19	0.91 (20 Gage)	Weld	Cyclic	64	1.6

Test Set up

All diaphragm specimens were tested within a cantilever, 6.1m x 2.8m steel test frame with pinned corner connections. This large-scale test frame represents a half portion of the overall roof diaphragm in the typical single-storey structures.

An out-of-plane support system was used to support the north side of the test frame vertically. It consisted of two devices attached to the rigid floor and was equipped with four rollers located on both top and bottom sides of a frame member. A reaction beam with two strong supports was used to connect the south side of the test frame to the rigid floor and prevent any movement. The load was applied by actuator of a 3m x 4m uni-axial shake table. The actuator of the shake table is displacement controlled and an MTS digital servo-controller is used to operate the hydraulic actuator with a two-way capacity of ±260 KN force and a stroke limit of ±450 mm. The loading beam was used to transfer the load from the shake table to the test frame and a load cell was mounted on it to measure the applied load signal directly. The load cell connectors were pinned to rotate freely in the horizontal direction at both the test frame connection and the loading connector beam joint at the shake table.

For this testing program, the test frame was instrumented to capture its lateral displacements. The instrumentation for these experiments consisted of linear variable differential transformer and position transducers to control the small movement of fixed end beam at south side and measure the lateral displacements of the frame, respectively. A general view of the test setup, a test specimen during testing and the schematic plan view and the location of the instruments are illustrated in Fig. 3.

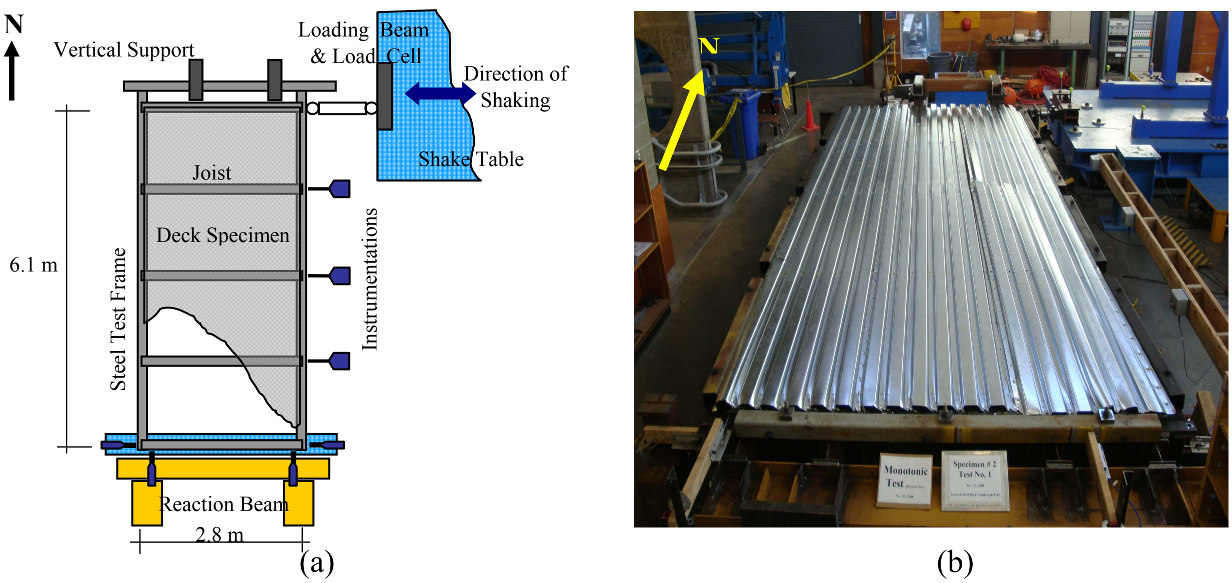

Figure 3. Steel Diaphragm test set-up: a) Schematic plan view and instrumentation layout; b) General view.

Loading Protocol

A loading protocol was developed for performing reversed cyclic tests on the steel diaphragm specimens based on ATC-24 guidelines for cyclic testing of steel structural components (ATC 1992). Monotonic load-deformation response was needed to determine the deformation parameter, yielding displacement required for defining the amplitudes of the loading sequences. These tests were displacement-controlled using a gradually increasing displacement as 7.5 mm/min, since it allows for a better comparison of the results among the specimens. The low velocity of testing protocol avoids the effect of strain rate on the inelastic response results.

Test Results and Discussion

Monotonic Tests

Monotonic tests were first performed to determine the shear performance and deformation parameters of the different diaphragm types studied. Three of these four specimens are made with 0.75, 0.91 and 1.2 mm thick panels with nail fasteners and one another with 0.91 mm and weld connections. Measured elastic stiffness and peak shear resistance of the specimens are presented in Table 1. These tests also allowed the observation of behavior and failure modes of the specimens. Inelastic deformation of a deck is mainly concentrated near the end support on the edge of the diaphragms parallel to the lateral loading.

The load-deformation curves obtained from all monotonic tests performed for each of the specimens are presented in Fig. 4. All systems have a comparable initial stiffness but exhibited significantly different ductility, resistance and post peak resistance response. Specimens with nail fasteners exhibited a ductile behavior with progressive failure. The diaphragm with welded connections showed brittle failure and limited ductility. However, the maximum load capacity for each configuration was similar. The diaphragm strength decreased rapidly after the peak load was reached. All the specimens showed a reserved capacity up to approximately 50% of the peak strength after failure of the connectors started

For nail specimens, the initial stiffness was about 1.3, 1.4 and 1.5 KN/mm for deck panels with 0.75, 0.91 and 1.2 mm thickness, respectively. The first yielding occurred at 0.5, 0.6 and 0.7% drift and the failure happened at 1.9, 2.2 and 2.3% drift for these diaphragms. The failure of the 0.75 mm deck panel happened progressively but mainly at 1.9% drift. For the welded specimen with 0.91 mm thick panels, the initial stiffness was 1.4 KN/mm and first yielding and failure took placed at 0.65 and 1.3% drift, respectively. The tests were continued until 200 mm (about 3.5% of the length of the specimen) of displacement was reached at the loaded end.

The load capacity of the specimens increased by increasing the thickness of the panels, but the ductility decreased for the thicker panels. The deck specimens with thicker panel showed higher yielding and failure drift.

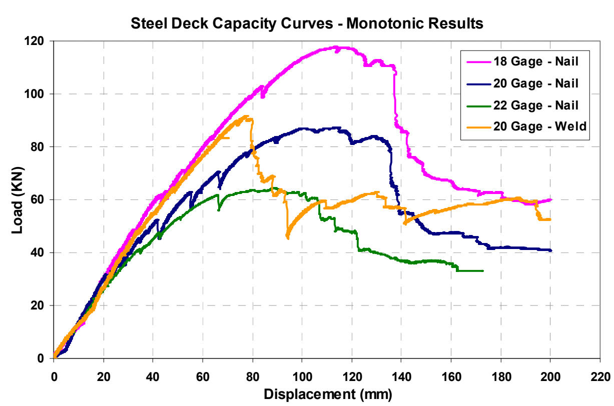

Figure 4. Monotonic load-deformation response of diaphragm specimens 2, 4, 5 and 7.

Cyclic Tests

The reversed cyclic tests were then performed on specimens with the various panel thickness and different connector types to assess the cyclic behavior of roof deck diaphragms. These tests were repeated on each specific type of specimen for three times to validate the response and testing observation. The results obtained from repeated tests were statistically analyzed and the mean values were calculated and presented. The measured performance of some of these specimens is presented in Fig. 5. Load-deformation response of the specimens under cyclic and monotonic condition is compared in this figure.

Minor sliding was observed at the starting point of each test while the load was distributed throughout the specimen. Elastic stiffness and initial yielding drift of each type of deck specimen were similar in the monotonic and cyclic tests. In all cyclic tests a pinched hysteretic behavior was observed. The Nail specimens sustained large inelastic deformation cycles with progressive strength degradation. However, the welded specimens showed very significant deterioration during the cycles. The peak resistance of welded specimens in the cyclic loading was substantially less than the resistance under monotonic loading. This clearly indicates that a sudden brittle failure of the welded connections is likely to occur during actual earthquake induced motions. This difference in resistance was not observed in the nail specimens. The measured shear resistance, elastic stiffness and deformation parameters for different diaphragm types under monotonic and cyclic loading are summarized in Table 2.

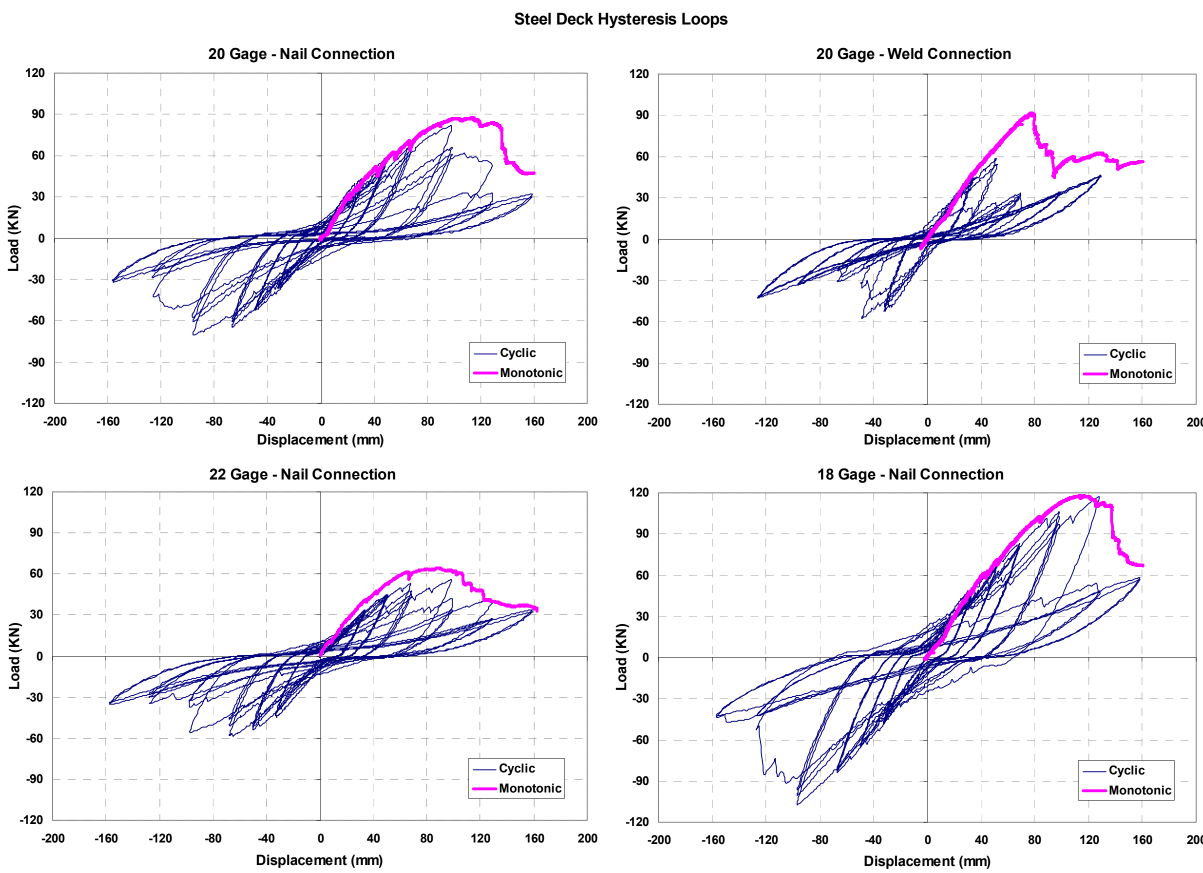

Figure 5. Load-deformation response of deck specimens under monotonic versus cyclic loading.

Failure of the Fasteners

Figures 6 to 8 show details of the behavior and failure modes of the connectors used in these test specimens during monotonic or cyclic tests. For specimens with nail fasteners inelastic response was developed by tilting of the screws at the side laps (Fig. 6) and ductile inelastic deformation of the panel where it is attached to the joists and to the end beam (Fig. 7). Limited damage was observed elsewhere in the deck panels. For the specimens with welded connections bonding failure of the welds happened along the end beam (Fig. 8-b) shortly after local buckling and distortion of the steel panel occurred near the welds (Fig. 8-a). The rest of each specimen, including side laps and perimeter frame members, showed no damage or evidence of inelastic action.

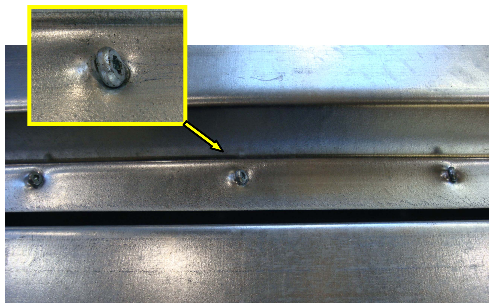

Figure 6. Screw tilting failure at side lap.

(a) (b)

Figure 7. Failure mode at deck-to-frame nail fasteners: Panel sliding and bearing failure: a) At joist; b) At end beam.

(a) (b)

Figure 8. Failure mode at deck-to-frame welded connections: a) Buckling and distortion at joist; b) Bonding failure at end beam.

Table 2. Results of experimental study.

Type of Connection	Deck Thickness (mm)	Monotonic Tests				Cyclic Tests (Average of three samples)			
		Elastic Stiffness (KN/mm)	First Yield Drift (%)	Failure Drift (%)	Max. Shear Resistance (KN)	Elastic Stiffness (KN/mm)	First Yield Drift (%)	Failure Drift (%)	Max. Shear Resistance (KN)
Nail	0.75 (22 Gage)	1.3	0.5	1.9	64	1.25	0.5	1.4	53
	0.91 (20 Gage)	1.4	0.6	2.2	88	1.4	0.45	1.65	71
	1.2 (18 Gage)	1.5	0.7	2.3	118	1.55	0.45	1.8	86
Weld	0.91 (20 Gage)	1.4	0.65	1.3	92	1.45	0.55	0.9	60

Conclusions

Inelastic response, hysteretic behavior and shear performance of nineteen steel roof deck diaphragms with various deck thickness and connection types were studied by performing large-scale monotonic and cyclic reversed tests. Important observations from these tests include:

1. The validity of responses and testing observations of the steel deck diaphragms backed by results obtain from repeated tests. The results of each specific type of specimen were statistically analyzed and the mean values were calculated and presented.
2. Monotonic and cyclic testing of all specimens confirmed that the inelastic deformation of a deck is mainly concentrated on the edge of the diaphragms parallel to the lateral loading.
3. For the specimens with nail fasteners inelastic response was developed by tilting of the screws at the side laps and ductile inelastic deformation of the panel where it is attached to the joists and to the end beam. Limited damage was observed elsewhere in the specimens.
4. For the specimens with weld connections bonding failure of the welds happened along the end beam shortly after by local buckling and distortion of the steel panel near the welds. The rest of each specimen, including side laps and perimeter frame members, showed no damage or evidence of inelastic action.
5. The results of monotonic tests of diaphragms with nail fasteners exhibited a ductile behavior with progressive failure. The diaphragm with weld connections showed brittle failure and limited ductility. However, the maximum load capacity for each configuration was similar.
6. The monotonic load-deformation curves show that the diaphragm strength decreased rapidly after the peak load was reached. All the specimens showed a reserved capacity up to approximately 50% of the peak strength after failure of the connectors started.
7. The load capacity of the specimens was improved by increasing the thickness of the panels. In contrast, the ductility decreased in thicker panels. All systems have a comparable initial stiffness but exhibited significantly different ductility, resistance and post peak resistance response.

8. The cyclic tests showed a pinched hysteretic behavior. Nail specimens sustained large inelastic deformation cycles with progressive strength degradation. In contrast, weld specimens showed very significant deterioration and very rapid strength degradation after the peak load was reached.
9. Under cyclic loading, the peak resistance of the specimens with weld connections was substantially less than the resistance under monotonic loading. This clearly indicates that a sudden brittle failure of weld connections is likely to occur during actual earthquake induced motions. This difference in resistance was not observed in the specimens with nail fasteners.
10. Elastic stiffness and initial yielding drift of each type of deck specimen were similar in the monotonic and cyclic tests. Minor sliding was observed at starting point of each test while the load distributed.

In conclusion, the results from these series of tests show that the nail connections show a more ductile behavior than the weld connections. Although the initial stiffness of both systems is practically the same, the resistance under cyclic loading of the nail connections is about 20% higher than that of the weld connections. It is clear from these tests that the response modification factor for steel deck systems with nail connections should be greater than the current value in the design code (NBCC 2005).

Acknowledgments

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