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Inelastic response 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 twenty-one steel roof deck diaphragms. This test program was initiated and designed to evaluate the seismic inelastic response of steel roof decks with different thicknesses 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. The tests included monotonic, quasi-static reversed cyclic inelastic deformation, and seismic motions. Shear performance and failure mode of the steel decks for both types of deck-to-frame connections were investigated. Lateral resistance and elastic stiffness of steel decks with different panel thicknesses and connector types were determined and compared with those available in the Diaphragm Design Manual. 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. These tests confirm that the response modification factor for structural systems built with steel decks with nail–screw connectors should be greater than the current value in the building codes.

1. Introduction

Steel roof deck diaphragms are widely used in low-rise industrial and commercial buildings in North America and Europe. They can provide large interior spaces needed for manufacturing and warehouse facilities. Light-gage steel decks are also used in other types of construction. This paper focuses on their application in single-storey buildings. The size and dimensions of these buildings typically vary from 5 to 20 m high, 6 to 45 m wide, and 18 to over 100 m long. A steel deck is commonly comprised of a number of corrugated steel sheets, fastened to one another in the side laps and to the perimeter frame members in the end connections, as well as to the interior joist beams. Screw, button punch or weld with washer is usually used for side laps and nail, rivet, weld or weld with washer for deck-to-frame connections at discrete points. To carry the vertical loads, the roof deck is designed to span between the steel joists, which are supported by either steel girders or wall panels. Lateral load due to wind or earthquake is transferred from the out-of-plane walls and roof into the frame or in-plane walls by the steel deck, which is designed to act as a diaphragm. Lateral loads are resisted by vertical braces or shear walls.

Considerable research efforts have been undertaken to study the in-plane behavior of the steel deck diaphragms with corrugated steel sheets in the 1960's. The foundation of the work was laid down by Nilson [1]. He provided the practical information for welding the panels and studied the effect of supporting beams, and also the span and the depth of the panel profile on the flexibility of a diaphragm. Luttrell extended previous works by investigating the effect of panel configuration, material properties, span length, and particularly the method of fastening the diaphragm. He also, developed a formula for estimating the shear stiffness of standard corrugated panels [2]. Bryan and El-Dakhakhni [2] conducted a series of tests on corrugated sheets and sheet fasteners and presented a method of calculating the shear flexibility and shear strength of a practical diaphragm by considering the effects of the separate components. Some more tests were undertaken on light-gage steel connections by Chong and Matlock [3] and more studies were performed for predicting the strength and stiffness of the corrugated steel sheets. Easley and McFarland [4]

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studied the problem of determining shear deformations and fastener forces in diaphragms, following a different approach, and developed formulas for both stiffness and fastener forces. Davies [5,6] modified Bryan's work and developed a simplified analysis application for regular and irregular diaphragms based on computer-oriented approaches. Many tests have been conducted during these years, though most have been performed under monotonic loading where the main emphasis was on shear strength and lateral stiffness.

In plastic design it is necessary to know the ultimate shear load of a panel of roof sheeting, and whether it can sustain the same order of deformation as the frames. In the late 1980's the ductility approach was established and included in the design codes. Seismic provisions allowed engineers to use reduced seismic loads in design to achieve satisfactory inelastic seismic performance and thus, the knowledge of inelastic behavior of structural components became of practical importance. On the other hand, reconnaissance reports of past earthquakes have stated that steel roof decks of single storey frames and tilt-up structures were severely damaged and showed that more attention should be paid in their seismic design [7]. The roof diaphragm has to be able to transfer the load to the steel braced frame through the direct connections between the deck and the frame. In most of the cases in the 1989 Loma Prieta earthquake problems were caused by these poor connections. Also, tilt-up systems have exhibited poor seismic response in a number of earthquakes, including the 1964 Alaska, 1971 San Fernando, and 1994 Northridge events. Most frequently, the connections between the wall panels and flexible diaphragms failed. Localized collapses of roof sections were common and often accompanied by collapses of the wall panels in the out-of plane direction. In structures designed before the 1971 San Fernando earthquake, the connections between the roof and wall panels did not have sufficient strength or deformation capacity to resist the out-of-plane inertial forces induced in the panels [8].

More recently, several tests have been conducted incorporating cyclic loading. In one study [9], eighteen large scale tests were carried out on steel diaphragm assemblies made with different deck thicknesses using various types of fasteners in various configurations. The tests were performed using a cantilever type configuration for the test setup, with the steel deck diaphragm in a horizontal plane. Both cyclic and monotonic testing was conducted. The results showed the load capacity of roof diaphragms, subjected to lateral loads, is directly dependent on the performance of the connections. Rogers and Tremblay [10,11] carried out an experimental program to investigate the seismic inelastic behavior of deck-to-frame fasteners and side lap fasteners. They tested about two hundred light-gage sheet connections and provided information on inelastic cyclic response of the various types of connectors, including load-deformation hysteresis and energy dissipation capacity.

Current design codes, e.g. Canadian Standards Association, S16 Design of Steel Structures [12], 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 [13] allows inelastic response to occur in the roof decks while the braces retain elastic behavior. Thus, the roof deck is considered to act as the ductile fuse element in the lateral load path instead of vertical braces and should sustain large inelastic deformation cycles without significant strength degradation. It causes the thinner deck panels with more spacing for the fasteners that makes the structure more cost-effective compared to the current approach for seismic design. Several studies [10,11,13] have been conducted on inelastic performance of steel roof decks and more are needed to implement in practice 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 thicknesses and different types of deck-to-frame connections. Twenty-one large-scale specimens with 0.75, 0.91 and 1.2 mm thick deck panels were tested using a cantilever steel test frame. Pneumatic fastened nails and arc spot welds were used for deck-to-frame and self-drilled screws for sheet side lap fasteners. The present study focuses on the evaluation of elastic stiffness, shear strength, initial yield and failure drift of decks under monotonic, reversed cyclic loading, and seismic motions. Load-deformation hysteresis response and ductility of the decks with various configurations are studied and damage distribution along the deck and the failure modes of the connectors are explored.

2. Seismic behavior of light-gage steel decks

Generally, the behavior of structures in major earthquakes is inelastic. Seismic codes allow structures to behave in an inelastic range to reduce the design seismic loads that make the structures cost effective. The reduced seismic loads are permitted to be used for systems that exhibit stable energy dissipation capacity through plastic deformations. This strategy is acceptable for buildings with steel roof decks while satisfying life safety requirements. Specifically, due to the large demands expected at soft-soil sites, the seismic performance of structures is not likely to improve unless the roof is incorporated as part of the lateral-force-resisting system in these structures.

The expected lateral deformation of the steel roof decks in typical single-storey structures during an earthquake is illustrated in Fig. 1(a). This figure shows the basic concept for a group of the roof plane diaphragms. Lateral inertia load is generated due to the earthquake motion, and roof and side walls mass, and is transferred to the ends of the deck toward the lateral resistant supports. The diaphragm acts as a short-deep beam with simple end supports. From the typical shape of the shear diaphragm, the following observations are apparent [14]: (1) The maximum average in-plane shear is near the end beams and causes the shear deformation at the edges of the roof; (2) Zones nearer the mid-span may have smaller shear and thus less diaphragm strength is required; (3) The larger design shear may be resisted using heavy panels and fewer connections or by more frequently connected lighter panels; (4) Efficient use of materials may not be met by using a single diaphragm design for the entire roof area. General arrangement of a simple diaphragm is illustrated in Fig. 1(b). The intermediate joist beams and perimeter girders as roof supporting elements and the roof deck panels are shown schematically in this figure. The corrugations serve to stiffen the panel and can have a variety of different shapes.

Roof supporting elements are important components of the diaphragm in that they act as stiffeners, similar to stiffeners in thin-web girders. Such elements protect the zone from general buckling. In addition, members with suitable connections to carry the flange forces must always restrain diaphragms. If a light-gage deck contains a sufficient number of closely spaced fasteners but with large spans, elastic buckling is its usual mode of failure. Easley and McFarland [4], and Easley [15] have presented equations describing the buckling behavior of light-gage diaphragms using approximate analytical methods and carried out experimental studies for their verification. However, in most applications of light-gage diaphragms in buildings, the joists and rafters span is short and diaphragm failure occurs at loads below buckling, caused by failure at the fasteners, due either to shear failure of the fasteners themselves or to localized bearing failure of the panel material around the fasteners. Typical exaggerated deformations, forces and couples on the deck panel in this mode of failure are shown in