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Influence of Non-Structural Components on Roof Diaphragm Stiffness of Single-Storey Steel Buildings

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Abstract

Single-storey steel buildings can be found in regions of active and moderate seismicity levels in Canada. The SFRS in these structures typically includes a cold-formed steel roof deck diaphragm that transfers horizontal loads to vertical steel bracing bents. Steel deck roof diaphragms can be relatively flexible compared with the vertical bracing. The seismic loads at a given site, calculated using the 2005 NBCC, depend on the fundamental period of vibration of the structure, which can be estimated using empirical expressions. Past studies have shown that the dynamic response of buildings is affected by the flexibility of the roof diaphragm. Although it is possible to incorporate the diaphragm flexibility in the calculation for the period of vibration, deviations exist between field tests and numerical simulations for the building period, which are likely due to the stiffening effect of the non-structural components. The objective of this research was to provide a better understanding of the influence of the non-structural roofing components on the performance of single-storey steel buildings subjected to seismic loading. The scope of study involved the determination by testing of material properties for the non-structural roofing materials, including:

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gypsum board; fibreboard and polyisocyanurate (ISO) insulation, all of which are used in the SBS-34 roof configuration. It was also necessary to complete testing of the typical mechanical connectors found in the roof diaphragm. A linear elastic finite element model of a roof deck diaphragm that accounts for the steel panels, the non-structural components and the various mechanical connections was developed. The analytical results obtained from the model were compared with past test results. The model was then relied on to establish stiffness values for additional diaphragm configurations, in which the deck thickness and connector pattern were varied.

Introduction

A large proportion of the single-storey steel buildings in Canada are located in regions of active and moderate seismicity levels, such as on the Pacific coast and in the St-Lawrence and Ottawa River Valleys. The seismic force resisting system (SFRS) in these structures typically includes a cold-formed steel roof deck diaphragm that transfers horizontal loads to vertical steel bracing bents. Steel deck roof diaphragms are relatively flexible compared with the vertical bracing, and hence, in-plane roof deformations due to lateral loads can exceed the horizontal deformation of a building's walls *(Tremblay et al. 2004)*. The seismic loads at a given site, calculated using the 2005 National Building Code of Canada (NBCC) *(NRCC 2005)*, depend on the fundamental period of vibration of the structure, which can be estimated using empirical expressions. These formulae have been typically derived for multi-storey structures with rigid floor and roof diaphragms *(Goel and Chopra 1997; Tremblay 2005)*, and therefore, do not necessarily reflect the behaviour of low-rise steel buildings with flexible roof diaphragms. Past studies have shown that the dynamic response of such structures can be affected by the flexibility of the roof diaphragm *(Adebar et al. 2004; Jain and Jennings 1985; Medhekar 1997; Tena-Colunga and Abrams 1996; Tremblay and Stiemer 1996)*. In particular, the fundamental period of vibration is generally lengthened when compared to similar structures with rigid diaphragms. An expression to predict the period of a building that accounts for the flexibility of the roof diaphragm was proposed by Medhekar *(1997)* and validated by shake table testing by Tremblay and Bérair *(1999)* and Tremblay *et al. (2000)*. Furthermore, FEMA-356 *(2000)* allows for the introduction of the in-planeflexibility of the roof diaphragm in the estimate of the fundamental period of vibration. Tremblay and Rogers *(2005)* have described the possible impact of building period on the design and cost of the seismic force resisting system.

Although it is possible to incorporate the diaphragm flexibility in the calculation of the period of vibration, it has been observed that deviations exist between field tests and numerical simulations for the building period *(Medhekar 1997; Ventura 1995; Tremblay et al. 2002; Paultre et al. 2004; Lamarche 2005).* Furthermore, Rogers *et al. (2004)* and Yang *(2003)* have shown that the non-structural roofing components can increase the stiffness of the roof diaphragm, thus changing the period of vibration.

In view of this background information, there existed a need to examine further and to assess the possible influence of non-structural roofing components on diaphragm shear stiffness and on the performance of single-storey steel buildings subjected to seismic loading. The scope of study involved the determination by testing of material properties for the non-structural roofing materials, including: 12.7 mm Type X gypsum board; Matériaux Cascades Securpan 25.4 mm fibreboard and 63.5 mm polyisocyanurate (ISO) insulation; all of which are used in the Association des Maîtres Couvreurs du Québec (AMCQ) SBS-34 roof configuration. This common and conventional roof configuration was chosen after consulting with the AMCQ and the Ontario Industrial Roofing Contractors Association (OIRCA). It was also necessary to complete testing of the typical mechanical connectors found in the roof diaphragm. A linear elastic finite element model of a roof deck diaphragm that accounts for the steel panels, the non-structural components and the various mechanical connections was developed. The analytical results obtained from the model were compared with the test based findings of Yang *(2003)* and Essa *et al. (2001)*. Since only a limited number of tests have been carried out on diaphragm specimens with non-structural components, the model was then relied on to establish stiffness values for additional diaphragm configurations, in which the deck thickness and connector pattern were varied.

Non-Structural Roofing Components

The choice of a roof system was based on a literature review and on advice received from AMCQ, OIRCA, as well as from various roofing contractors. The roof configuration known as SBS-34, commonly found in Canada, was previously selected by Yang *(2003)* for two large-scale cantilever diaphragm tests; thus there existed test results to aid in the validation of the finite element model. It was anticipated that this particular roof configuration would have adequate stiffness to augment the in-plane stiffness, G’, of the overall diaphragm. The large-scale diaphragm tests by Yang showed an average increase in G’ of 49% due to the additional in-plane shear stiffness of the non-structural roofing components, as well as the reduced warping of the steel deck cross-section *(Rogers et al. 2004)*. This hot bitumen adhered roof system is composed of the following layers (from top to bottom) (Figure 1): i) Two layers (4 mm + 2.2 mm) of synthetic rubber SBS (Styrene-Butadiene-Styrene) waterproof membrane; ii) One layer of 25.4 mm thick flame resistant Matériaux Cascades Securpan wood fibreboard, hot bitumen adhered; iii) One layer of 63.5 mm thick polyisocyanurate (ISO) insulation, hot bitumen adhered; iv) Two layers of paper vapour retarder (No. 15 asphalt felt), hot bitumen adhered; v) One layer of 12.7 mm thick Type X gypsum board, 12 screws per 2.4 x 1.2 m panel mechanically fastened; and vi) Steel roof deck, *e.g.* 0.76 mm Canam P3615B ASTM A653 *(2005)*.

Figure 1 : SBS-34 roofing cross-section as tested by Yang *(2003)*

Roofing Component Experiments

The material properties of the non-structural roofing components were not readily available in the literature; hence, physical testing was necessary to determine representative values for use with the finite element model. A total of four different test setups were fabricated to measure the initial stiffness of the materials and connections in the linear elastic range. The first test was a centre point load flexural test, which was necessary to determine the flexural stiffness of the gypsum and fibreboard. This test was carried out because Yang *(2003)* observed that the warping deformations of the steel roof deck panels, which affect the shear stiffness of the diaphragm, were restrained by the flexural stiffness of the gypsum board in particular. The second test setup was a simple two-sided shear assembly in which the shear stiffness of the gypsum and fibreboard was measured on a local scale. A four-sided shear test was then used to measure the shear stiffness of the gypsum and fibreboard on a larger scale. Specimens that incorporated combinations of the other non-structural roofing components were also tested with the four-sided shear setup. The final test setup was of the screw connection between the gypsum and underlying steel deck, as well as the screw sidelap connections between the steel deck panels and the nailed deck-to-frame connections. An MTS Sintech 30/G universal testing machine with a 150 kN load cell was used for all tests. The LVDTs and load cell were connected to a Vishay Model 5100B scanner, which was used to record the data with the Vishay System 5000 StrainSmart software. A detailed account of the test program is available in the work of Mastrogiuseppe *(2006)*.

Flexural Tests

The flexural tests were conducted in order to obtain the flexural stiffness of the fibreboard and gypsum board panels. The flexural test setup was a simple centre-point flexure test (Figure 2a), which was based on ASTM Standard C473 *(2003)*. Each test was conducted in displacement control at a crosshead speed of 6.35 mm/sec until failure of the specimen. In all, 24 fibreboard specimens were tested. This included specimens that were cut from a single panel but without a specific orientation with respect to the grain. Eight additional specimens were cut from the same panel: Four were cut in direction ‘A’ and the other four were cut perpendicular to the previous specimens, direction ‘B’. This approach was used to investigate the hypothesis that any existing directionality of the wood fibres would affect the flexural properties. Directions A and B have no precise meaning other than they are perpendicular to one another. A total of 44 flexural gypsum board tests, comprising two series of specimens, were performed. The first series contained specimens parallel to the long side of the panel, while the second series was oriented perpendicular to the long side. Gypsum board is typically fabricated with a finishing layer of paper on one side of the panel. It was felt that this layer may have an effect on the flexural stiffness and strength of the panel depending on whether the paper was placed in tension or compression during testing. The white finishing paper was placed in compression for 22 of the test specimens, and in tension for the remaining flexural specimens. Young’s modulus in flexure, E, could then be calculated given that all other variables were known, such as I in the flexural stiffness expression EI.

The average Young’s modulus obtained from the flexural stiffness of the fibreboard specimens was 250 MPa. However, when the data from the ‘A’ and ‘B’ data sets were compared, there were two different values of stiffness: 298 MPa for ‘A’ and

241 MPa for ‘B’. This represents a difference of approximately 20%, which is much larger than the calculated coefficient of variation of the data set. Nonetheless, the general shape of the load vs. deformation curve was the same for the two sets of data. The flexural stiffness of the gypsum panel was much higher than that of the fibreboard panel. Mean values of E = 2850 MPa in the parallel direction and 2410 MPa in the perpendicular direction were determined. The gypsum board was found to be roughly ten times stiffer in flexure than the fibreboard. The flexural stiffness results were similar for the gypsum board specimens for which the finishing paper was in tension and compression. In the development of the finite element model it was decided to use an average Young’s modulus for flexural stiffness of 250 MPa for the fibreboard and 2625 MPa for the gypsum panels.

Two-Sided Shear Tests

The two-sided shear tests were carried out in accordance with ASTM D1037 *(1999)*. The shear deformation of the gypsum and fibreboard specimens was directly measured by an LVDT placed in line with the loading plates (Figure 2b). The displacement controlled rate of loading was taken as 0.2% of the length of the specimen per minute, *i.e.* 0.508 mm/min. A total of five fibreboard specimens and four gypsum board specimens were tested. The fibreboard and gypsum board shear specimens behaved linearly up to approximately 40% and 50% of the ultimate load, respectively. The ASTM D1037 Standard contains a method for the calculation of shear strength from the test results, however, there is no recommended equation given to determine the stiffness of the specimen for this specific test setup. It was therefore necessary to rely on equations for the interlaminar shear stiffness from ASTM D1037 and the through thickness shear from ASTM D1037 and D2719 *(2001)*. The ASTM D2719 Standard contains a factor to compensate for the nonuniform stress distribution in small test specimens. The specific equations have been documented by Mastrogiuseppe *(2006)*. The resulting values for stiffness of the fibreboard and gypsum board panels were 175 and 1290 MPa, respectively, with the interlaminar shear stiffness equation and 60 and 437 MPa, respectively, using the through thickness shear equation. The two equations give very different results for local shear rigidity although they are both taken from the ASTM D1037 standard. The interlaminar shear equation gives a rigidity almost three times higher than the though thickness shear equation. The average shear stiffness for the gypsum board was over seven times greater than that of the fibreboard. However, the results for both the fibreboard and the gypsum stiffness were scattered, as can be seen in the coefficient of variation of 36.7% and 16.6%, for the two materials, respectively. One possible cause of the scatter of results may be the small scale and localized loading of the test setup. Furthermore, the test setup was originally not developed to determine stiffness, but rather the shear strength properties of a material. The four-sided shear test setup was required to provide additional information on the shear stiffness of the non-structural components prior to the selection of representative values for use with the finite element model.

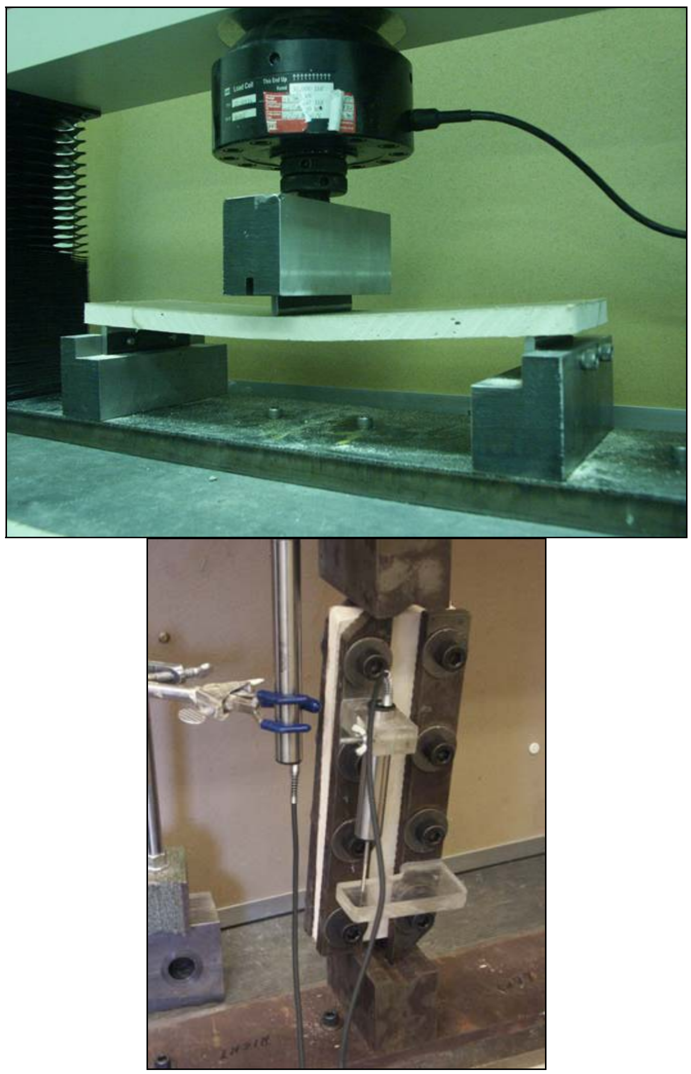

Figure 2 : Test setups: a) flexural test setup; b) two-sided shear test setup

Four-Sided Shear Tests

The four-sided shear tests were conducted in order to obtain the shear stiffness of the gypsum, fibreboard and combinations of other non-structural roofing components. This test setup, which was based on ASTM D2719 *(2001)*, was necessary because of the type and size of the non-structural roofing elements. A specimen having a square shear area was loaded along all four edges by a system of hinges and rails (Figure 3a). As the cross head of the loading machine moved vertically upwards at a constant rate of 2.1 mm/min, bearing forces were applied at the reinforced corners of the panel, resulting in shear forces along the four sides of the panel. The diagonal elongation of the specimen was measured with LVDTs placed on both faces of the specimen.

In all, 22 specimens were tested, comprising fibreboard panels, gypsum panels, as well as combinations of other non-structural components. Stiffeners were installed on most specimens to ensure that flexural deformations of the panel were minimized. In addition to the single panel specimens, it was necessary to fabricate specimens that consisted of combinations of fibreboard, ISO insulation, felt vapour retarder and gypsum (Figure 3b). These test specimens were similar to the diaphragm specimens with non-structural components tested by Yang *(2003)*. For three tests a 25.4 mm ISO board was hot bitumen adhered to a fibreboard panel. A total of four "full component" specimens were fabricated in an attempt to represent all of the non-structural elements of a roof. A sheet of felt vapour retarder was first hot bitumen adhered to the fibreboard/ISO section. As a second step in the fabrication, a gypsum layer was then hot bitumen adhered to the vapour retarder.

The fibreboard was used as the base material in all the "sandwich" construction specimens because it has lower shear stiffness than the gypsum board. This facilitated the measurement of any change in stiffness as the additional non-structural layers were added. If gypsum had been used as the base material, the relative increase in stiffness due to the added layers, would have been much lower than the stiffness of the gypsum itself, perhaps even negligible. Also, note that only the fibreboard was connected to the loading rails; the other non-structural layers were located within the central portion of the test specimen. These specimens were tested in the same test setup as the plain gypsum board and fibreboard specimens. The ISO insulation board could not be tested by itself because the test frame as fabricated could not accommodate for its thickness and flexibility.

The average shear stiffness of the gypsum board was 1284 MPa, which was 5.5 times higher than that of the fibreboard 235 MPa. An increase in shear stiffness of 30%, compared with the fibreboard alone, was measured when the ISO board was added to the fibreboard. A total shear stiffness increase of almost 70% compared to the fibreboard alone was realised when the gypsum board and vapour retarder layers were added to the fibreboard and ISO board. These results provide the shear stiffness of the roofing section with the load applied to the fibreboard. However, in the actual roof the shear load / deformation would first be applied to the gypsum board from the steel roof deck panels. Screw fasteners are typically used to connect the gypsum board to the deck panels. Hence, it was necessary to identify the increase in stiffness to the gypsum board because of the addition of the vapour retarder, ISO and fibreboard panels. The stiffness of the fibreboard and gypsum board panels was known, as well as the “full component” and the fibreboard / ISO section. The only individual non-structural component for which the shear stiffness was not known was the ISO board, excluding the vapour retarder which was assumed to have negligible in-plane shear stiffness.

Significant out-of-plane bending response developed during the “full component” tests, which made it difficult to rely on the direct displacement measurements to determine the sought after shear stiffness values. To overcome this shortcoming Mastrogiuseppe *(2006)* developed a finite element model of the “full component” test setup with which the ISO insulation was determined to have a shear stiffness of 4 MPa. With this value, the shear stiffness of the non-structural roofing components, with the gypsum board as the base element was found to be 1380 MPa. The shear stiffness of the fibreboard alone was defined as 235 MPa, and that of the gypsum panels was 1284 MPa. The values from the four-sided test frame were used to define the shear properties of the non-structural components for use in the finite element model.

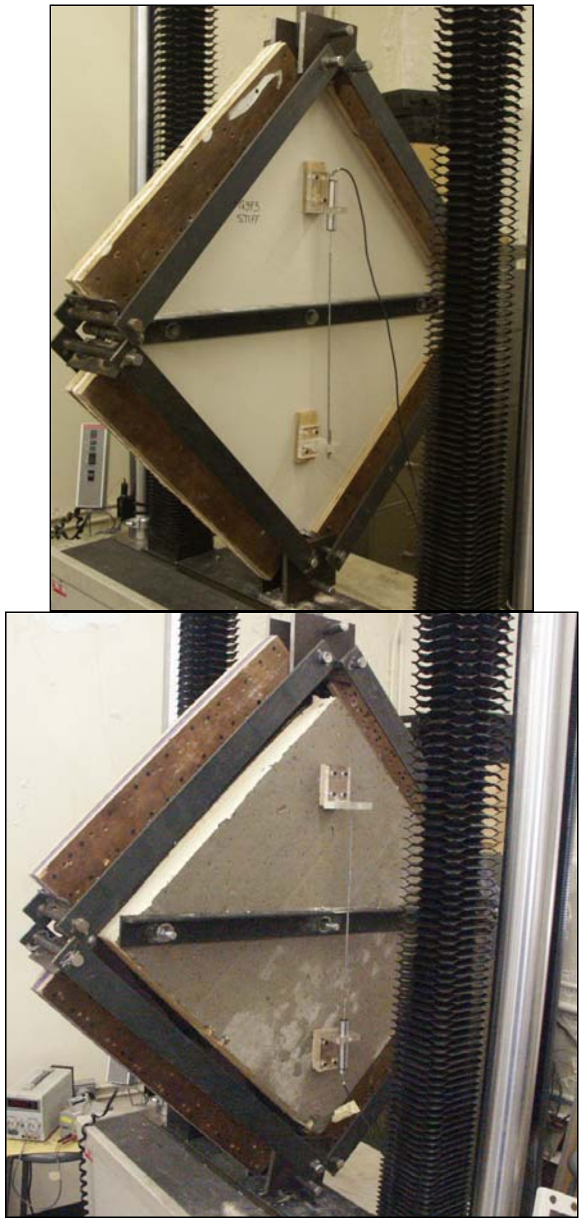

Figure 3 : Four-sided shear test setup: a) gypsum board specimen; b) “full component” specimen

Connection Tests

The connection tests were carried out to determine the stiffness of the typical screw and nail (powder actuated fastener) connections that are present in roof deck diaphragms: gypsum board to steel deck; steel deck sidelap connections; and frame-to-deck connections. A single overlap / single shear setup was used for the testing of all individual connections (Figure 4). Each specimen was composed of two pieces (gypsum, steel deck or steel plate) that were attached by a single fastener. The free ends of the two pieces were then installed in a gripping device that was fastened to the testing frame. Each test was conducted in displacement control at a crosshead speed of 1 mm/min. Detailed information on the sidelap and deck-to-frame connection tests can be found in Nedisan *et al. (2006)*.

Test specimens were constructed of 0.76 mm, 0.91 mm, 1.22 mm and 1.51 mm ASTM A-653 *(2002)* Grade 230 MPa sheet steel. The gypsum board was 12.7 mm Type X, and the steel plates were 4.8 mm Grade 300W CSA G40.20/G40.21 *(2004)* material. The steel plates were used to represent the supporting flange of a frame member beneath the steel roof deck. Hilti X-ENDK22-THQ12 powder actuated (nail) fasteners were used to connect the deck elements to the frame. Sidelap connections were made of two deck panels fastened with a Hilti S-MD 12-14 X 1 HWH #1 screw. The gypsum-to-deck screw connectors were #12 Hex with Round Galvalume Plate Dekfast™ products, made by SFS intec.

The stiffness of the gypsum-to-deck connections for the first three steel thicknesses were all very similar, hence, an average value of 3.14 kN/mm was determined for these specimens as a group. The connection stiffness for these specimens was mainly dependent on the tightness of the screw/washer combination. It was clear that if the connector was not well installed, or if a washer was not used, the connection stiffness would be much lower than this average value. The thickness of the sheet steel did not seem to have an impact on the stiffness of the connection; rather the placement of the washer was critical. However, the 1.51 mm thick sheet steel specimens possessed a much higher stiffness than the others, with an average value of 6.30 kN/mm. The thicker deck prevented the screw from rotating, thus reducing the dependence of the connection performance on the washer tightness.

Initial shear stiffness of the deck-to-frame fasteners was measured as 32.3 kN/mm, 31.7 kN/mm, 46.6 kN/mm and 50.3 kN/mm for the 0.76 mm, 0.91mm, 1.22 mm and 1.51 mm sheet steel specimens, respectively. The sidelap connection stiffness was 11.9 kN/mm, 14.7 kN/mm, 18.6 kN/mm and 21.2 kN/mm for the same sheet steels. Stiffness parameters for various sidelap and deck-to-frame connections have also been provided by Rogers and Tremblay *(2003a,b)* and the Steel Deck Institute (SDI) *(1991)*. These supplementary connection stiffness values were also incorporated in the finite element model for comparison purposes.

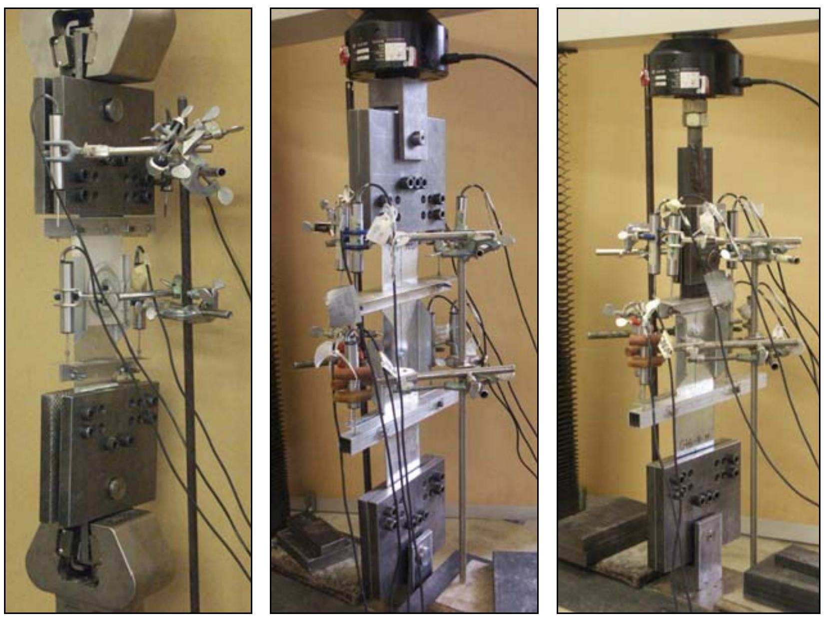

Figure 4: Connection test setups : a) gypsum-to-deck test; b) sidelap test; c) deck-to-frame test

Contribution of Roofing Components to Diaphragm Shear Stiffness

Yang *(2003)* carried out twelve large-scale roof diaphragm tests (3.658 m x 6.096 m in plan), two of which were constructed with non-structural components. The Group 3 tests, characterized by a 0.76 mm thick Canam P3615 type steel deck, as well as nailed deck-to-frame and screwed sidelap connections, are relevant to this paper.

The test specimens were constructed of three full-width steel panels and one half-width panel along the north and south edges of the frame. Deck-to-frame and sidelap connectors were placed at a spacing of 305 mm. The deck-to-frame connectors were Hilti X-ENDK223-THQ12 powder actuated fasteners and the sidelap connectors were Hilti S-MD 12-14x1 HWH #1 screws. The AMCQ SBS-34 roofing configuration, as described previously, was used for the diaphragm specimens constructed with non-structural components. There were six full-size gypsum panels and three half panels screwed directly to the top of the steel roof deck. A total of twelve screws per full panel and nine per half panel were placed. Once the gypsum panels were attached to the deck, hot bitumen was applied and the felt paper was rolled onto the gypsum. Bitumen was then mopped onto the felt paper such that the ISO insulation board could be adhered. The fibreboard was attached to the insulation following the same procedure. Finally, two layers of SBS water proof membrane were installed by means of hot bitumen and an open flame propane torch.

The Group 3 monotonically loaded diaphragm specimens 43 and 45 by Yang were used for this study. The first was composed of a bare sheet steel deck diaphragm, while the second included the non-structural roofing components. An in-plane initial shear stiffness of G' = 2.58 kN/mm was measured for the bare test diaphragm, whereas the clad specimen was able to reach a stiffness of 4.17 kN/m, an increase of 62%. If the results of the cyclically loaded tests 44 (bare) and 46 (clad) were included in the comparison, an average increase in shear stiffness of 49% would be attained. During the loading process the gypsum board, which is quite rigid in its own plane, and also possesses a flexural stiffness, greatly restrained the warping of the panels (Figure 5). In addition to the added shear stiffness of the gypsum, the warping restraint directly increased the shear stiffness of the complete diaphragm. Based on observations made during testing, it was concluded that the gypsum layer added to the stiffness of the steel deck, while the other non-structural layers contributed very little. This conclusion was reached mainly because of the significant damage that occurred in the gypsum layer (Figure 5), whereas the other non-structural components exhibited no visible distortion.

Figure 5: Warping deformation of steel deck restrained by non-structural components – test 45 *(Yang 2003)*

Linear Elastic Finite Element Diaphragm Models and Analyses

The objective of the analytical phase of this research project was to develop linear elastic finite element models that would adequately reproduce the initial stages of the roof diaphragm in-plane shear behaviour. The analytical models, based on the large-scale diaphragm tests conducted by Yang *(2003)*, were built using the SAP2000 v.8.2.3 software *(CSI 2002)*. The models were developed to reproduce the test results obtained by Yang of a bare steel diaphragm specimen (test 43), as well as a diaphragm that was constructed with non-structural roofing components (test 45). In addition, a bare steel specimen with 0.91 mm thick panels (test 17) that was tested by Essa *et al. (2003)* was also used for comparative purposes. Material properties of the various finite elements were selected based on the data acquired in the experimental stages of this project. The models were assembled according to the dimensions and specifications of the diaphragm specimens tested by Yang. Initially, eight models were created: four bare sheet steel roof diaphragms with a panel depth of 38 mm and a thickness of 0.76, 0.91, 1.22 and 1.51 mm; and four roof deck diaphragms with the same series of steel panels but clad with non-structural components. Screw fasteners were used for all sidelap connections, while powder actuated nails were modeled at all deck-to-frame connection locations. A 305/305 connector configuration was used throughout, *i.e.* a deck-to-frame nail spacing of 305 mm and a sidelap fastener spacing of 152 mm.

Cantilever diaphragm models (3.658 m x 6.096 m) were built according to the specifications of the Group 3 test specimens, as cited in Yang *(2003)* and described by Mastrogiuseppe *(2006)*. The 20456 node bare sheet steel models had three full

6.096 m long deck panels and two half width panels which were represented with 17812 four-node flat shell elements capable of developing bending and membrane behaviour (Table 1) (Figure 6a). Each steel panel was modelled separately, which required link elements to connect the various panels and framing members. The underlying pin connected frame was composed of 600 six d.o.f. beam elements that were assigned material properties such that the shearing deformation of the model would only take place in the sheet steel and its connections. A total of 1999 link elements were used to represent the sidelap and deck-to-frame connections (Table 2), as well as contact surfaces; this includes multi-link elements that restricted movement of the steel deck into the frame at contact locations away from connections.

Table 1 : Material properties of shell elements

Property	Steel Deck				Non-Struc. Components
	0.76 mm	**0.91 mm**	**1.22 mm**	**1.51 mm**	
t (mm)	0.72	0.905	1.22	1.51	12.7
E (MPa)	195.2	197	203	203	3.07
G (MPa)	75.1	75.8	78.1	78.1	1.38
μ	0.3	0.3	0.3	0.3	0.11

Table 2 : Stiffness properties of link (connection) elements (kN/mm)

Connection	Steel Deck Thickness			
	0.76 mm	**0.91 mm**	**1.22 mm**	**1.51 mm**
Deck-to-frame	32.0	32.0	46.6	50.3
Sidelap	11.6	14.7	18.6	21.2
Gypsum-to-deck	3.14	3.14	3.14	6.28

The non-structural roofing component model consisted of the same 600 beam elements that represented the frame, as well as 35092 shell elements for the steel deck panels and the non-structural panels (Table 1), 1870 link elements for connections (Table 2) and contact areas, as well as 37264 nodes (Figure 6b). There were fewer link elements than with the bare sheet steel model in order for a converged solution to be reached. A single layer of 12.7 mm thick material that represented the complete non-structural section (gypsum, ISO insulation & fibreboard) was connected to the sheet steel shell elements through links that represented the gypsum screw fasteners.

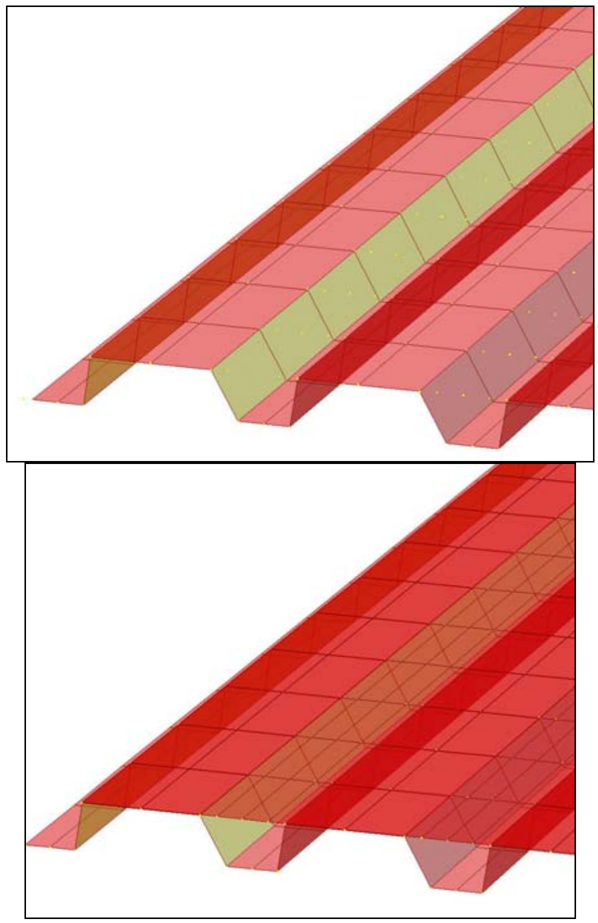

Figure 6: Undeformed shape of full-scale diaphragm model; a) bare steel; b) bare steel with non-structural roofing

Material properties for the shell elements were assigned based on a combination of measured and codified values. Properties for the steel deck shell elements were taken from the test data compiled by Yang *(2003)* for the 0.76 and 0.91 mm thick deck. Since testing of the two thicker deck types had not been carried out, the material properties were obtained from the CSA S136 Standard *(2001)*. The properties of the non-structural components were taken from the results of the material tests presented above. A value for Poisson's ratio of 0.11 was found for the gypsum and assumed for use with the full cross-section of non-structural components. The thickness of the gypsum, 12.7 mm, was also used, however the Young's and shear moduli of the gypsum were increased to account for the additional stiffening effect of the ISO insulation and fibreboard layers (Table 1).

Table 3 lists the computed stiffness of the models, both bare steel and clad, as well as the measured in-plane shear stiffness of the corresponding diaphragm tests by Yang *(2003)* and Essa *et al. (2003)*. The test-to-predicted ratios of the three diaphragm specimens vary from 0.94 to 0.96, which indicates that the FE model can be considered as relatively accurate. The model was then used to evaluate the stiffness of the remaining configurations for which diaphragm test data was not available. As expected, the overall stiffness of the bare sheet diaphragm increased as the thickness of the panels increased. The shear stiffness of the diaphragm with 1.51 mm thick panels was 4.5 times that obtained for the diaphragm with 0.76 mm panels. A significant increase in the elastic shear stiffness was recorded when the non-structural components were added to the model. This result was most evident for the diaphragm with the thinnest steel deck panels. The effect of the non-structural components diminished as the sheet steel thickness increased, *i.e.* a 58.6% increase in stiffness was calculated for the 0.76 mm steel, whereas only a 16.9% increase was obtained for the 1.51 mm panels.

Table 3 : Comparison of test based and analytical shear stiffness G'

Deck Thickness (mm)	**Fastener Pattern (mm/mm)**	**Cladding**	**G' Test (kN/mm)**	**G' FE Model (kN/mm)**	**% Inc *vs.* Prev.**	**% Inc *vs.* Bare**	**Test/FE**
0.76	305/305	Bare steel	2.58[1]	2.74	N/A	N/A	0.94
0.91	305/305	Bare steel	4.22[2]	4.49	63.5%	N/A	0.94
1.22	305/305	Bare steel	N/A	8.30	85.0%	N/A	N/A
1.51	305/305	Bare steel	N/A	13.0	57.1%	N/A	N/A
0.76	305/305	SBS-34	4.17[3]	4.35	N/A	58.6%	0.96
0.91	305/305	SBS-34	N/A	6.42	47.7%	43.2%	N/A
1.22	305/305	SBS-34	N/A	10.8	68.9%	30.8%	N/A
1.51	305/305	SBS-34	N/A	15.2	40.4%	16.9%	N/A

[1]Test 43 by Yang *(2003)* [2]Test 17 by Essa *et al. (2003)* [3]Test 45 by Yang *(2003)*

The SAP model was able to reproduce the elastic load-deformation behaviour of the diaphragm tests accurately. Figures 7 and 8 illustrate the deformed bare steel deck diaphragm and the deformed shape of the deck with the gypsum board models, respectively. The warping of the steel deck corresponds to that observed during testing of the diaphragm specimens, which was much less apparent in the model and test with the non-structural components. Figure 9 shows the flexural deformation in the non-structural components, as was observed in the diaphragm specimens tested by Yang.

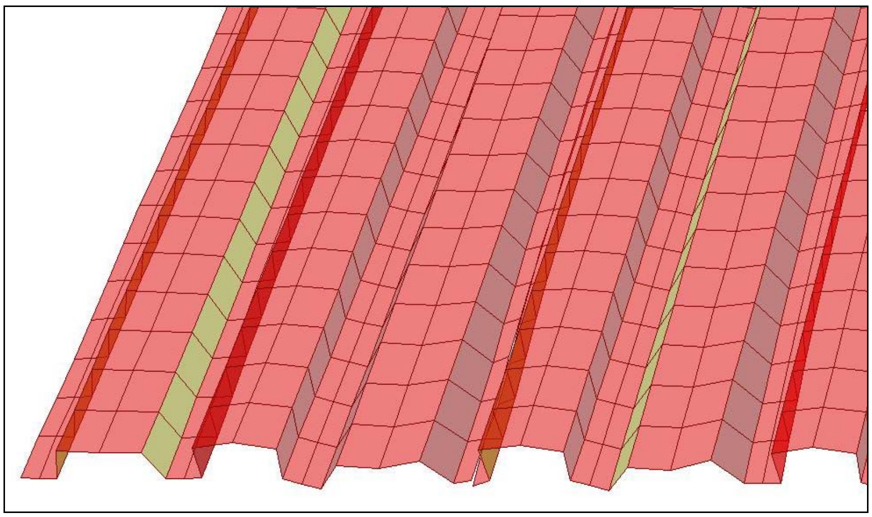

Figure 7: Warping and shear deformation of bare steel model

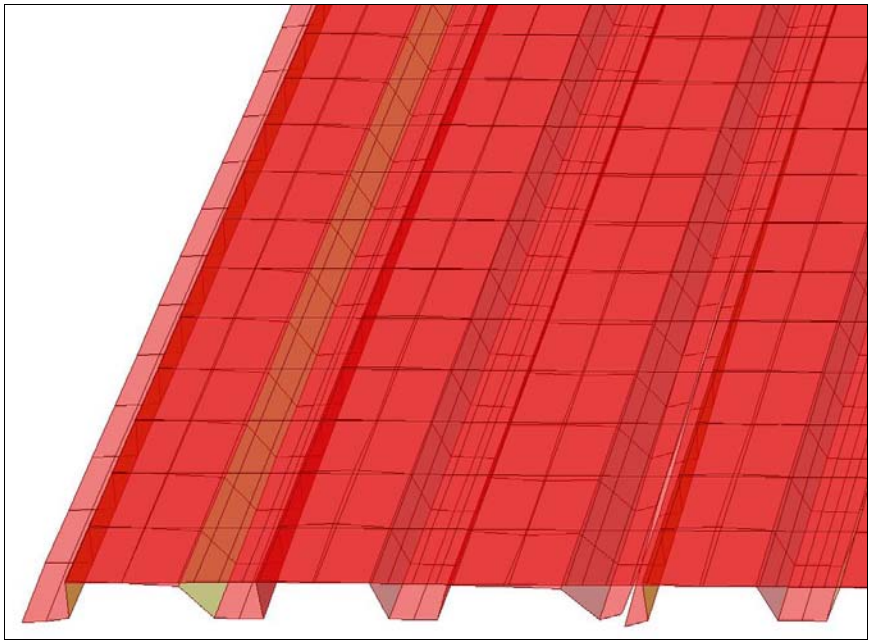

Figure 8: Restrained warping and shear deformations of model with roofing components

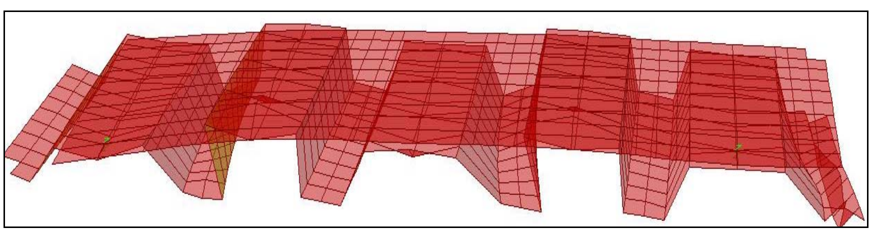

Figure 9: Deformations of steel deck and non-structural components

The stiffness values obtained with the FE models were approximately 5% higher than the values measured during testing, for both the bare sheet steel and clad diaphragms. This could possibly be due to material non-uniformity or irregularities that occurred during the construction of the test specimen. It is possible that in the diaphragms tested by Yang *(2003)* the quality of installation of the fasteners was not consistent, and hence in some locations the connection stiffness may have been lower than used in the FE models. The overall stiffness of a steel roof deck diaphragm is highly dependent on the performance of the individual deck-to-frame and sidelap connections. This would have led to a decrease in the measured shear stiffness of the test diaphragm. To verify whether the 5% discrepancy between the test and FE derived stiffness was due to poor connector quality an additional model was created in which 10% of the connectors had their stiffness reduced by 10%. Note, this decrease in stiffness was arbitrarily selected. The results of the analysis gave a shear stiffness of 2.63 kN/mm, which resulted in a test-to-predicted ratio of 0.98. This indicates that only a slight change in the connection stiffness, perhaps due to a lack of quality control during construction, for a small number of fasteners can change the overall diaphragm stiffness, which could explain the variation in test and FE stiffness values listed in Table 3.

Parametric study: Stiffening influence due to gypsum board

A parametric study involving the FE analyses of thirty-two nail-screw diaphragm models was carried out to determine the contribution of the non-structural components to overall roof diaphragm in-plane shear stiffness. The scope of study comprised four different steel deck thicknesses (0.76, 0.91, 1.22 and 1.51 mm) and four structural connector configurations (305/305, 305/152, 152/305 and 152/152), with and without gypsum board. Designers commonly rely on the SDI design method *(1991)* to calculate overall shear stiffness and capacity of a roof diaphragm; therefore, the FE analyses were conducted using the basic model described previously with SDI defined frame and sidelap stiffness properties instead of the test based values (Table 4). This approach was taken in an attempt to identify the possible increase in stiffness of the bare steel diaphragm, as calculated using the SDI method, due to the presence of non-structural roofing components. For this study the gypsum board alone was incorporated in the models due to the finding that the remaining non-structural components (ISO insulation & fibreboard) augmented the stiffness of the gypsum by only 5%. The material properties of the gypsum board were as follows; $t = 12.7$ mm, $E = 2625$ MPa, $G = 1284$ MPa and $\nu = 0.11$.

Shear stiffness values obtained from the FE analyses for the bare steel deck diaphragm models and for the diaphragm models with a gypsum board layer are provided in Table 5. Note that these values do not match those previously listed (Table 3) because SDI connection stiffness values were incorporated in the models instead of test based values. The percentage increase in diaphragm stiffness due to the addition of the gypsum board is tabulated in Table 6. The results clearly indicate that as the steel diaphragm becomes stiffer due to either the use of a thicker deck or more closely spaced structural connections, the contribution of the gypsum board to overall diaphragm stiffness decreases on a percentage basis. For the 0.76 mm specimen with a 305/305 connector spacing, a significant increase in G' (46.4 %) was caused by the addition of the gypsum layer. Conversely, for the 1.51 mm specimen with a 152/152 spacing, the increase was less than 5%. However, when comparing G' values for bare diaphragms versus diaphragms with the gypsum board, the actual contribution of the non-structural layer is very similar in absolute terms for all of the configurations modeled. The increase in shear stiffness between the diaphragms with gypsum board and those consisting of bare sheet steel varied between 1.27 and 1.65 kN/mm, with an average value of 1.41 kN/mm and a CoV of 0.076. These results indicate that the deck thickness and structural connector layout do not substantially influence the contribution of the non-structural components to in-plane shear stiffness of a roof diaphragm in absolute terms. Furthermore, it is plausible that in the analysis of a building model, which accounts for the behaviour of the diaphragm, G' can be increased to address the influence of the non-structural gypsum layer.

Table 4 : SDI stiffness properties of link (connection) elements (kN/mm)

Connection	**Steel Deck Thickness**			
	0.76 mm	**0.91 mm**	**1.22 mm**	**1.51 mm**
Deck-to-frame	19.4	21.2	24.6	27.4
Sidelap	10.1	11.0	12.8	14.2
Gypsum-to-deck	3.14	3.14	3.14	6.28

Table 5 : Finite element analysis linear elastic diaphragm stiffness G' (kN/mm)

Deck Thickness	**Bare Steel Model Fastener Pattern**				**Bare Steel + Gypsum Model Fastener Pattern**			
(mm)	**305/ 305**	**305/ 152**	**152/ 305**	**152/ 152**	**305/ 305**	**305/ 152**	**152/ 305**	**152/ 152**
0.76	3.26	4.05	9.29	11.8	4.78	5.59	10.7	13.2
0.91	5.17	5.40	12.3	15.5	6.46	7.06	13.7	16.9
1.22	8.51	8.70	17.4	23.1	10.0	10.1	18.7	24.4
1.51	12.5	13.1	22.3	30.0	13.8	14.4	23.8	31.5

Note: SDI connection stiffness values used in model.

Table 6 : Percent increase in diaphragm stiffness G' due to gypsum board

Deck Thickness (mm)	**Bare Steel Model Fastener Pattern**			
	305/305	**305/152**	**152/305**	**152/152**
0.76	46.4%	38.1%	15.3%	11.9%
0.91	25.0%	30.6%	11.0%	8.8%
1.22	18.1%	16.7%	7.8%	5.6%
1.51	10.1%	10.0%	6.5%	4.7%

Note: SDI connection stiffness values used in model.

Conclusions

The overall goal of this research was to provide a better understanding of the effect of non-structural roofing components on the performance of single-storey steel buildings subjected to seismic loading, specifically on roof diaphragm behaviour. This has been achieved by means of materials tests, finite element analyses, as well as a parametric study of the effect of deck thickness and connection configuration on the stiffening effect of gypsum panels. The gypsum board was found to be the stiffest element of the non-structural components, and because of this had the greatest influence on the in-plane force-deformation behaviour of the steel roof deck diaphragm. The other non-structural elements, either due to their low in-plane shear stiffness or lack of a direct connection to the steel deck, did not have as much of an effect. A finite element model was developed using SAP2000 to analyse the linear elastic behaviour of bare sheet steel deck diaphragms and diaphragms constructed with non-structural roofing components. A comparison of the measured stiffness of three diaphragm specimens provided test-to-predicted ratios in the range of 0.94 to 0.96. Given the close agreement of the test and analytical results it was concluded that the finite element model is adequate for the prediction of the linear elastic behaviour of roof deck diaphragms. A study was then carried out in which the elastic stiffness of five additional roof diaphragms was evaluated with the finite element model. Test data for diaphragms of these configurations was not available. In general, the diaphragm stiffness increased as the thickness of the steel roof deck panels increased. Furthermore, the contribution of the non-structural components, in terms of an increase in in-plane shear stiffness, was apparent for all deck thicknesses. This increase in stiffness became less on a percentage basis as the deck thickness was increased. A parametric study was then conducted in order to determine the contribution of gypsum board to overall in-plane shear stiffness of the steel deck diaphragm, with multiple deck thicknesses and connector layouts. For these FE models the stiffness of the sidelap and deck-to-frame connectors was based on the SDI predicted values, not test results. The study showed that the contribution of the gypsum board remained relatively constant in absolute terms regardless of deck thickness and connector spacing. On average, the diaphragms with gypsum board were 1.41 kN/mm stiffer in shear than the equivalent bare roof deck structure. Moreover, the percentage increase in shear stiffness of the diaphragm became less as the deck thickness was increased and as the structural connectors were placed at a closer spacing.

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