33rd Annual General Conference of the Canadian Society for Civil Engineering
33e Congrès général annuel de la Société canadienne de génie civil

Toronto, Ontario, Canada
June 2-4, 2005 / *2-4 juin 2005*

ANALYTICAL INVESTIGATION OF SEMI-RIGID FLOOR BEAMS CONNECTION IN MODULAR STEEL STRUCTURES

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ABSTRACT: The Modular steel building technique is fast evolving as an effective alternative to conventional on-site construction. A modular steel building, while generally designed using conventional methods, is unique in its method of construction as a result of special connections and details required to facilitate lifting and other construction handling operations. An analytical investigation using the finite element method is conducted on a stringer-to-beam connection of a typical floor-system of a modular steel school, designed using the Canadian steel design code. The results of the analysis revealed a number of issues that would need to be considered in any reliable prediction of structural response of modular steel floor framing. The rigidity of the connection partially restrains the rotation of the supported beam. This leads to force distribution between adjoined beams different from the case in conventional steel construction. Observations made from these results are expected to be incorporated into design guidelines that can be used by designers for an optimal design of modular steel buildings.

1.0 INTRODUCTION

Highly dynamic market forces and demands by industry clients for speedy, flexible, and cost-effective products have increased the complexity of traditional on-site building construction processes. The modular construction technique, which involves the design of structures to be built and finished at one location and be used at another, is fast evolving as an effective alternative. The technique is widely used in North America, Japan, and in parts of Europe. The rational for the use of modular construction technique is largely speed of construction, while combining the design flexibility of traditional methods with the quality of controlled manufacturing. Clearly, however, economy of scale of production may be an important factor to be considered against these benefits. In other words, it may be very expensive using modular construction for one-off buildings, which are non-repetitive than by traditional construction.

The volumetric concept of modular construction may be similar to temporary or relocatable buildings but differ greatly in terms of quality, structural design and general performance criteria. Application of modular construction within the civil engineering sector is found mainly in general building construction.

Light steel framing is widely used as the structural form of choice in modular construction due to its efficiency in terms of material use and also the ability to integrate lighter weight materials into a sophisticated manufacturing process. The Steel Construction Institute (SCI) of the United Kingdom has conducted some studies on performance specification for modular construction using light steel framing (Lawson, 1999). The limitations of the use of light steel framing in modular buildings are evident in its range of applications. For heavily loaded structures, hot rolled steel members, such as I-sections, are more useful and efficient. The use of these sections in modular building construction is what has been described in this paper as modular steel buildings.

Modular steel buildings have been typically used for one-to-six storey schools, apartments, hotels, correctional facilities and dormitories. The modules are assembled in a manufacturing plant and this allows excavation and foundation works to be carried out simultaneously with the building of the modules. Moreover, fabricating the modules in a controlled indoor environment ensures higher quality standards. Currently, there are no studies on the behaviour of modular steel buildings and there are no documented design provisions to address their unique requirements. Conventional building methods and codes are followed in the design of modular steel buildings. The current study is part of an extensive research program to address various design issues in modular steel buildings and to develop guidelines for the optimum design of modular steel buildings.

In modular steel buildings, the floor-system is typically designed as a grid structure consisting of floor stringers and beams. Floor stringer-to-beam connections, in conventional construction, are usually achieved using clip angles, which are generally shop welded to the web of the supported beam and site bolted to the web of the supporting beam. It is assumed that this connection will transfer reactions from the ends of the stringers to the floor beams through shear action, while allowing for rotation. In modular steel floor framing, however, such connections are achieved in a controlled factory environment by direct welding of the webs of the joining members. Structurally, these two methods of connecting beams may have substantially different effects on the strength and behaviour of the floor framing.

In this study, the semi-rigidity of welded stringer-to-beam connections in a typical module floor of a modular steel school building is investigated. The floor-system is designed as a grid structure of stringers and floor beams using the Canadian steel design code (CISC, 1997). It was then analysed using the finite element (FE) method. A sound engineering judgment is applied to verify the finite element model. The results of the FE results are used in reassessing the behaviour of the floor system

2.0 DESIGN OF A TYPICAL MODULE FLOOR FRAMING IN MODULAR STEEL SCHOOL

A four-storey structure is considered for the modular steel school building, which consists of classrooms separated by corridors. A typical storey is made up of six identical modules. Each module consists of a floor and a ceiling separated by a number of columns. The corridor on each storey floor runs through the middle portion of all the modules, between the internal columns. Figure 1 shows a plan view of a typical storey floor. It shows the arrangement of the modules and the positioning of stringers, beams and columns in each module. The modules are labelled M#1 to M#6. Horizontal and vertical connections of different modules are made on site. A horizontal module connection usually involves field bolting of clip angles, which are shop-welded to the floor beams. A typical vertical module connection is achieved by field-welding base plates of an upper module column to cap plates of a lower module column. The floor framing in each module is composed of two floor beams (FB), a number of floor stringers (FS) and a floor metal deck with concrete topping. Similarly, the ceiling framing will include two ceiling beams (CB) and a number of ceiling stringers (CS).

Figure 2 shows sections of a typical floor assembly. The sections show general arrangements of first and second floor. The second floor assembly (top section) depicts floor arrangements of two modules of the second storey and ceiling arrangements of two modules of the first storey. A typical floor or ceiling assembly is composed of a floor or ceiling beam and a floor or ceiling stringer. In addition, the floor assembly consists of a steel deck and a concrete floor and the ceiling finishes. The first floor assembly (bottom section) depicts floor arrangements of two modules of the first storey and their connection to the footing. The composition of the floor assembly is similar to the second floor described above. The connection of the floor assembly to the foundation is unconventional. In conventional steel construction, the steel columns bear directly on the foundations. As shown in the figure, in modular steel buildings, the interior columns bear on the floor beams and these beams are welded to steel base plates, which are connected to the foundations. The floor beams are stiffened to provide sufficient rigidity at the joint.

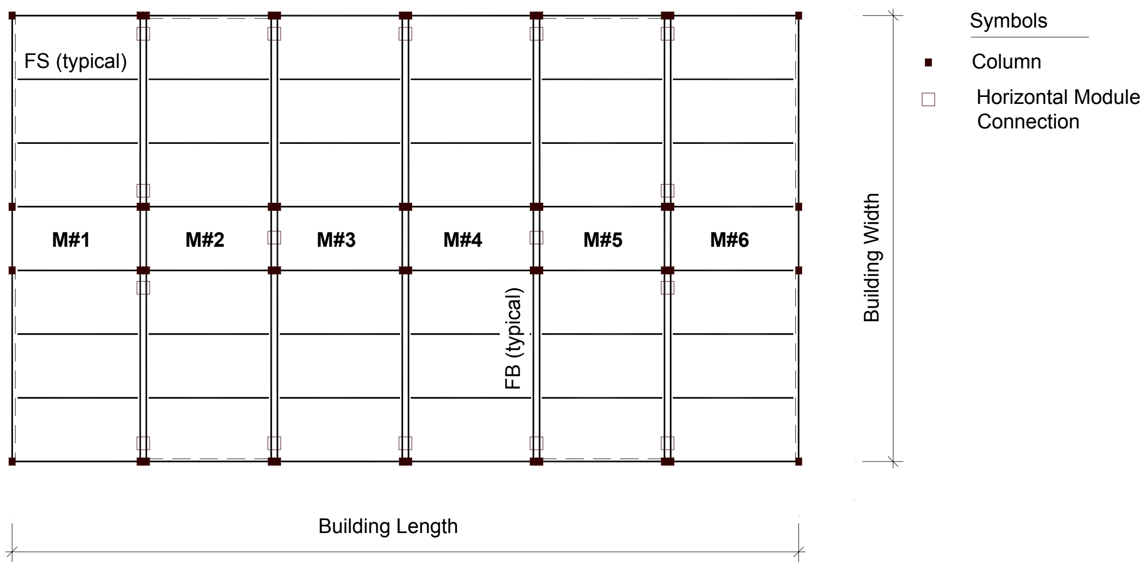

Figure 1: Plan View of a Typical Floor of a Four-Storey Modular Steel Structure

A typical module floor framing was designed as a grid structure consisting of floor stringers and beams using the conventional Canadian steel design code (CISC, 1997) in accordance with the Load and Resistance-Factor Design requirements. Figure 3 shows a general arrangement of the module floor framing considered in the design. Each stringer has a span of 3600 mm and the total span of each floor beam is 16500 mm. The stringers are labelled as SB1 to SB6, with the module symmetric about SB6. The floor stringers were designed as simply supported beams to withstand a mid-span bending moment of $wl^2/8$ for a uniformly distributed load w. The floor beams were designed as continuous beams. The dead load used for the design includes a superimposed load of 1.4 kN/m^2 accounting for finishes and service installations. The dead load (DL) is composed of the weights of the concrete floor, walls, steel deck as well as the self weight of the member. The live loads (LL) used for the design were 3.6 kN/m^2 for the classroom floors and 4.8 kN/m^2 for the corridor. The resulting steel sections for the floor stringers and beams are, respectively, W200X21 (W8X14 imperial designation) and W310X39 (W12X26 imperial designation). The yield strength of both steel sections is 350MPa (50 ksi imperial designation).

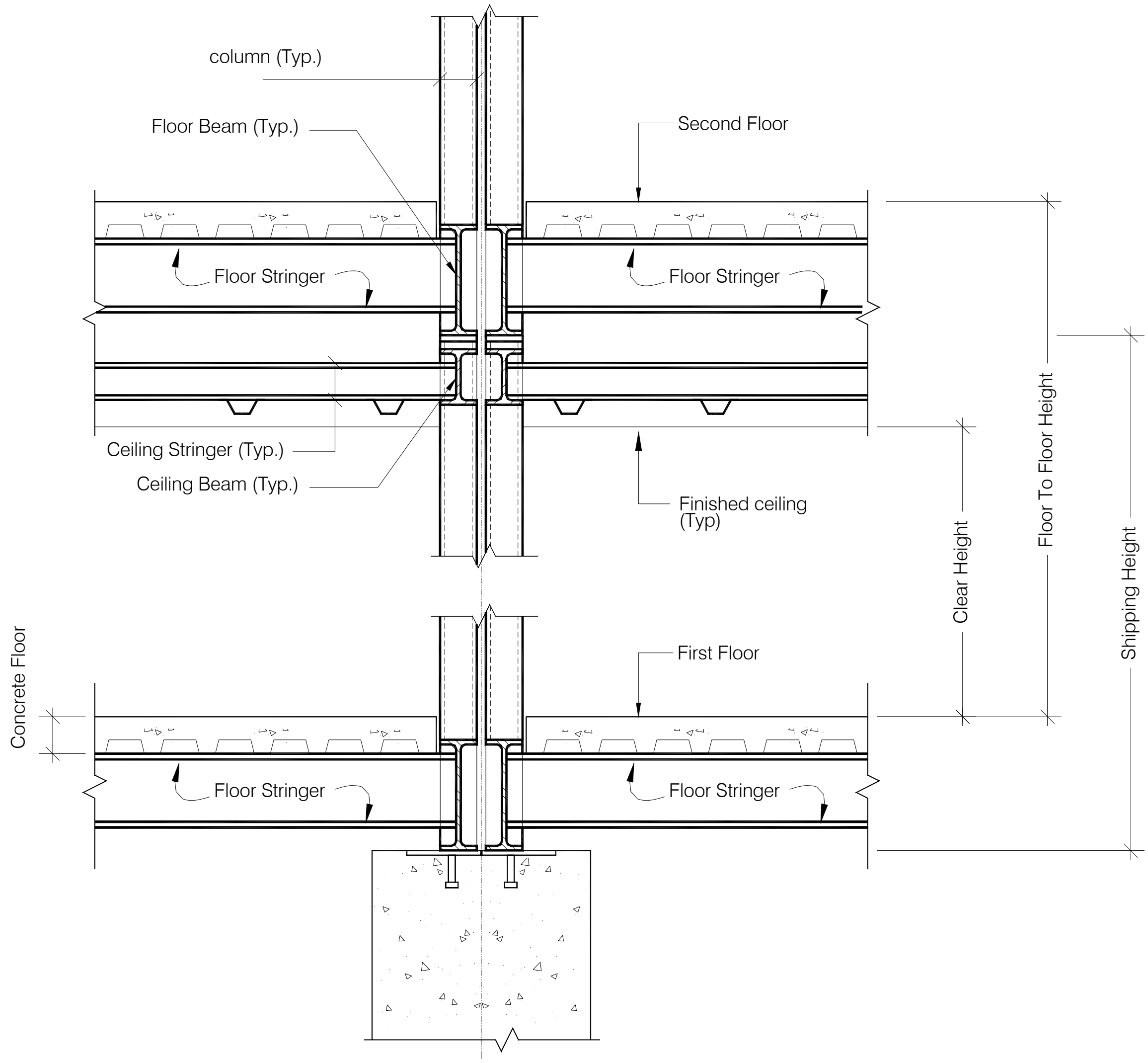

Figure 2: Typical Section of a Modular Steel Building

3.0 STRINGER-TO-BEAM WELDED CONNECTION

The connections in a structural framework can influence the behaviour of the structure in many ways, depending on the strength, stiffness and deformation capacity (Bijlaard and Steenhuis, 1991). The behaviour of connections may significantly affect the internal force distribution of steel framed structures. Nonetheless, the actual behaviour of connections is traditionally disregarded in the analysis and design of steel frames. These have been based on the idealization of joints as either a perfect hinge or fully rigid. These assumptions suggest, respectively, no restraint for rotation of connection for the hinge and no relative rotation of connection for the rigid joint. The use of these idealizations in many routine design practices may be effective and efficient in analyzing a large number of structures, but for many others, the true semi-rigid behaviour of joints would need to be considered in order to correctly assess their reliability and integrity (Hadianfard and Razani, 2003). In reality, any structural connection will deform to some extent and resists a certain amount of bending moment. This is also supported by experimental evidence that has shown that joints considered as pins often exhibit some rotational stiffness and strength, while on the other hand those considered fully rigid develop some bending deformation (Kishi and Chen, 1986).

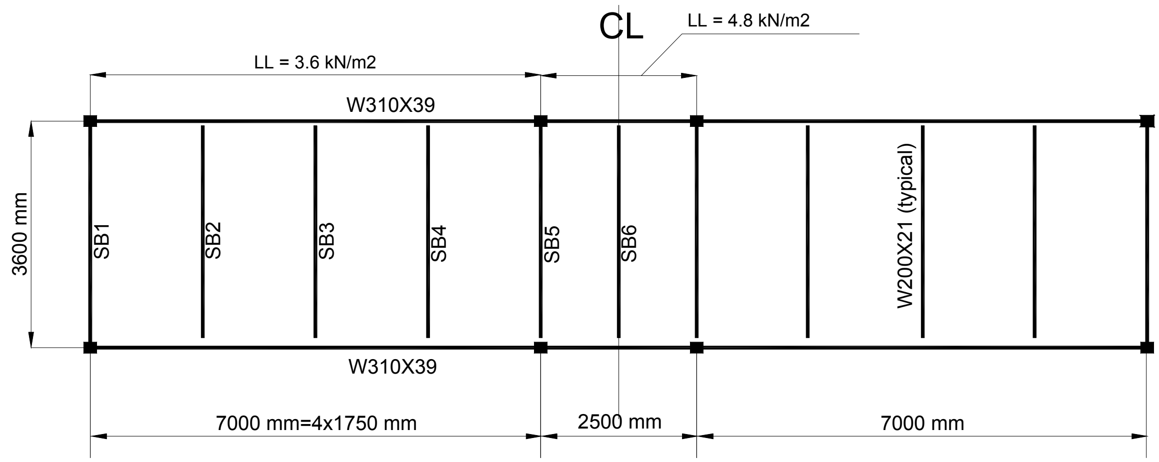

Figure 3: Layout of Modular Floor Framing adopted for FE Model

Rotational springs have been used to model semi-rigid restraint conditions. Most semi-rigid connections have highly nonlinear behaviour and the analysis and design of frames using them are often difficult and cumbersome. From a practical point of view, however, it is important to identify both the structural situations where the rotational behaviour of joints needs to be accounted for and those allowing either the hinge or the rigid model to be assumed. This will lead to an optimal design of steel structures.

Welds are able to withstand very limited deformations, so they exhibit a brittle failure mode. For this reason, weld fracture has to be absolutely avoided. Consequently, manufacturers of modular steel buildings typically weld a minimum of 80% of the stringer depth to the main beam. Designers often check this weld length against the shear transferred from the floor stringer to the beam, assuming no restraint for rotation at the ends of the stringer. This assumption fails to predict the true behaviour or response of the floor system.

4.0 FINITE ELEMENT MODEL

A 3-D finite element (FE) model is used to represent the structural elements and connections. The model was developed using the commercial finite element computer program, SAP2000 Nonlinear (CSI, 2000). The webs and flanges of both the stringers and the floor beams were meshed using shell elements. In total, 63,012 shell elements with reasonable aspect ratios were formed. A few assumptions and simplifications were made in order to facilitate the construction of the model and to simplify the post-processing of the results. Knife-edge restraint along an array of nodes was assumed for each column location. The effect of the slab on the steel framing was simulated by restraining the lateral movement of the top flanges of the floor stringers and beams to prevent lateral torsional buckling of the flanges. Also, the fillets at the corners of the steel beam sections were neglected in the model as it is expected to have no significant effect on the results.

The ability of the model to reproduce the expected design bending moment in the floor beams was checked against finite element results at some specific sections in the beam to verify the finite element model.

5.0 RESULTS AND DISCUSSION

The internal actions that beams and joints have to withstand depend on the end joints rotational stiffness, which, in turn, affects the flexural resistance the beams and joints are able to provide. For a perfect hinge joint, the flexural resistance of the beam is designed to withstand a mid-span bending moment of $wl^2/8$ for a beam subjected to a uniformly distributed load w. There is no restraint for rotation at the joints and the connection moment is assumed to be zero. In the case of a fixed-end restraint, the maximum bending moment is developed at the supports and can be evaluated as $wl^2/12$, while the moment at the mid-span is given as $wl^2/24$. There is full transfer of end moment from the supported member to the supporting member and thus no relative rotation exists in the connection. Under the above transversely loaded beams, there is no axial force developed in the beams. The distribution of bending moments in beams as outlined above is significantly affected if the true semi-rigid behaviour of the joint is taken into account.

Table 1 shows a comparison of the distribution of forces and bending moments between the finite element results and design values for the various floor stringers. It can be observed that the bending moments at the mid-span of the stringer obtained from the finite element analysis (M_{FE}) are only about 90% of the design mid-span moments for a simply supported beam (M_d). This percentage is not significantly affected by the variation in the magnitudes of the applied load on the various floor stringers. In other words, floor stringer SB6 produced almost the same percentage of M_{FE}/M_d as floor stringer SB5, which is subjected to 70% more loading. Further investigations, however, revealed that the percentage of M_{FE}/M_d is significantly affected by the ratio of the beam web thickness (t_w^b) to the stringer web thickness t_w^s and the beam-to-stringer depth ratio (d_b/d_s). As shown in Figure 4, the mid-span moment of the floor stringer obtained from the FE analysis decreases with an increase in the ratio of the beam web thickness to the stringer web thickness (t_w^b / t_w^s). As the thickness of the supporting beam increases in relation to the thickness of the stringer, the joint becomes more rigid and its capacity to restrain rotation is enhanced. However, as the depth of the supporting beam increases in relation to the depth of the supported stringer, the joint capacity to restrain rotation is reduced and consequently, the magnitude of the mid-span moment is increased.

Thus, the rigidity of the connection partially restrains the rotation of the supported beam. Consequently, hogging moments are developed at the ends of the stringers. As shown on table 1, the ends of the stringers are capable of developing hogging moment, M_h, of about 10% of the mid-span moment of a simply supported beam, M_d. This moment is significantly increased with an increase in the ratio of the beam web thickness to the stringer web thickness, t_w^b / t_w^s, and decreased with an increase in the beam-to-stringer depth ratio, d_b/d_s (Figure 5).

In addition to the observed bending moments that are unaccounted for in a typical design of modular steel buildings, considerable tensile forces are developed in the stringers. As shown in Table 1, the magnitude of the axial (tensile) force developed in the stringers (N_{FE}) could be as high as 39.5% of the total load (W) supported by the stringers. The horizontal restraint provided by the supported slab caused this axial force to develop.

The welded connection in the modular floor framing thus experiences an appreciable moment because of the hogging moments developed at the ends of the stringers. In addition, these connections are subjected to the effect of axial forces in the stringers. It therefore implies that the weld is not subjected only to shear forces as assumed in the conventional design practice but also to the effect of end moments and axial forces in the stringers. Reassessing the capacity of the weld under the combined axial, shear and moment revealed a significant increase in its stresses. The percentage increase in the weld stress from the design assumption (shear only) to the actual case (shear, bending and normal force) was about 226%.

For the floor beams, the bending moments obtained from the FE analysis matched that of the design moments. The hogging moments at the ends of the stringers resulted in some torsional moments in the floor beams but these were of negligible value in the analysed floor.

The stringers were redesigned accounting for the axial force and the reduced mid-span moment from the FE analysis. The design results revealed that a lighter section (W200X15) would have been adequate thus providing some savings in materials.

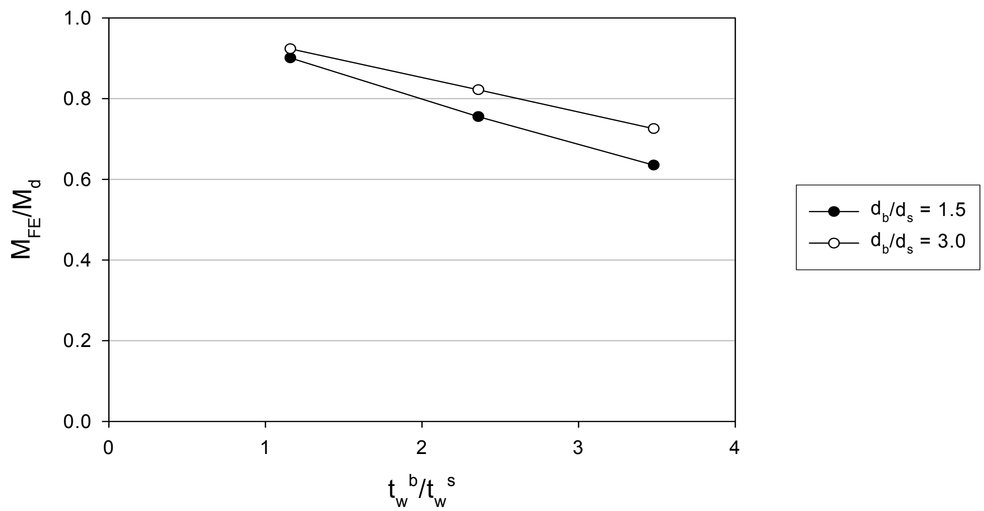

Figure 4: Variation of FE-to-Design Mid-Span Moment Ratio with Beam-to-Stringer Web Thickness Ratio

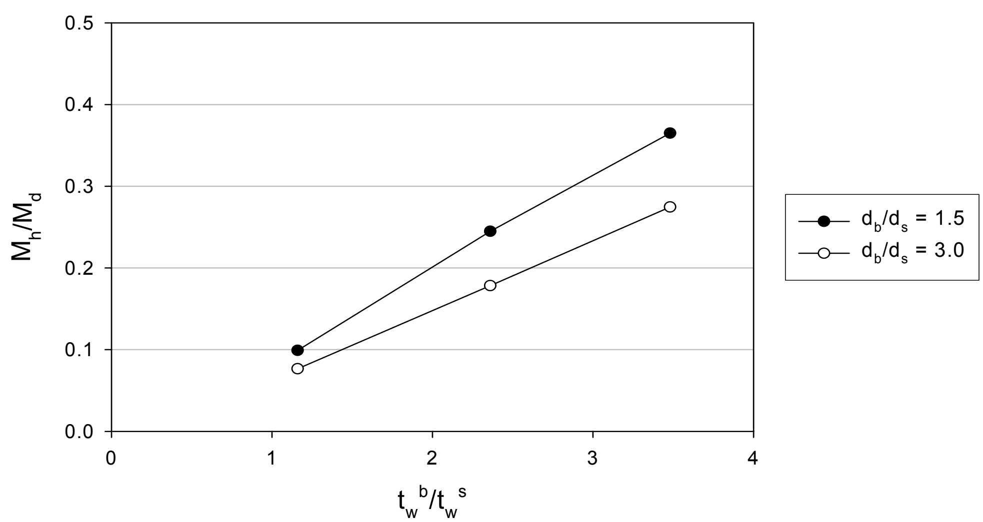

Figure 5: Variation of the ratio of FE Hogging Moments to Design Mid-Span Moment with Beam-to-Stringer Web Thickness Ratio

Table 1: Finite Element Results Compared to Design Values for Floor Stringers

	FLOOR STRINGERS					
	SB1	**SB2**	**SB3**	**SB4**	**SB5**	**SB6**
Mid-Span Moment (Design) M_d, (KNm)	22.28	22.03	22.03	22.03	30.8	18.26
Mid-Span Moment (FE) M_{FE}, (KNm)	20.2	20.09	20.14	20.05	27.75	16.54
M_{FE} as a percentage of M_d (%)	**90.66**	**91.19**	**91.42**	**91.00**	**90.10**	**90.58**
Hogging Moment at end of span (Design) M_h, (KNm)	0	0	0	0	0	0
Hogging Moment at end of span (FE) M_n, (KNm)	2.08	1.94	1.89	1.98	3.05	1.72
M_n as a percentage of M_d (%)	**9.34**	**8.81**	**8.58**	**9.00**	**9.90**	**9.42**
Axial Force (Design) N_d, (KN)	0	0	0	0	0	0
Axial Tensile Force (FE) N_{FE}, (KN)	15.16	18.55	19.08	18.14	21.79	13.84
Total Load on Beams excl. self wt. W (KN)	48.74	48.2	48.2	48.2	67.68	39.82
N_{FE} as a percentage of W (%)	**31.10**	**38.49**	**39.59**	**37.63**	**32.20**	**34.76**

6.0 CONCLUSIONS AND RECOMMENDATION

Direct economic benefits in terms of time and cost savings present a viable case for modular steel school buildings as an effective alternative to conventional on-site steel construction. The paper describes the process and features of the modular steel building; and presents a typical modular steel floor framing. The design of the floor framing was described and the finite element (FE) model used to represent the system explained. Results of the FE analysis revealed that consideration of the true behaviour or semi-rigidity of the welded connection leads to distribution of forces and moments that is different from the case of conventional steel construction. The axial forces and end hogging moments in the stringers observed in this study are not accounted for in a typical design of modular steel buildings. These forces and moments will affect the design of the stringers in a floor framing and will also affect the design of the weld connecting the floor beams to the stringers. This study has demonstrated that, the welds must have the capacity to transfer significant bending moment and axial force in addition to the vertical shear force from the stringer to the supporting beam. Clearly, the traditional rotational spring used to represent semi-rigidity of connections will not be representative of modular steel framing connections since it cannot predict the axial forces developed in the stringers. The results of this study therefore show that a realistic model of semi-rigid behaviour of connections needs to be developed to enable accurate prediction of the structural response of modular steel floor framing.

Acknowledgements

The authors would like to express their deep gratitude for the financial support provided by the National Sciences and Engineering Research Council of Canada (NSERC). The second author would also like to acknowledge the immense impartation of knowledge from Mr. Robert J. Murray the principal of Murray Engineering, P.C., New York City.

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