Scotia Place – 12 Story Apartment Building
A Case Study of High-Rise Construction Using Wood and Steel – WCTE2000

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ABSTRACT

This paper describes the design of a 12-story apartment building on a single story basement, which has wood floor diaphragms, and structural steel gravity and lateral load resisting systems. The design objective was to develop the most cost-effective structural system while meeting building functionality goals and adhering to code requirements. The main structural and non-structural design issues considered in this all-wood floor building are reviewed: gravity loads, lateral loads imposed by wind and earthquake, floor vibration, acoustics, and changes in wood moisture content. The lightweight structural form proved to be a practical system to lower construction material cost and enable alternative construction techniques to be employed. A comparison with a concrete floor option is briefly made.

INTRODUCTION

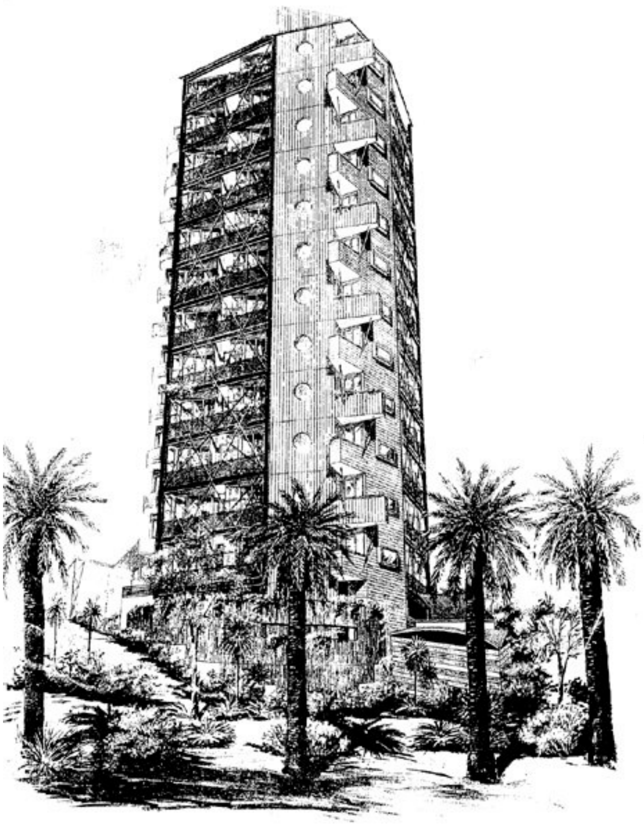
High-rise construction has been primarily limited to concrete and steel. (A successful exception is an all-wood five story structure on top of an existing five story concrete structure, designed by Banks) With a better understanding of other material properties, in particular wood, and the ability to automate design procedures through computers, the inclusion of wood as a construction material for high-rise building is feasible and provides an opportunity to explore alternative construction techniques. The use of lightweight construction in constrained sites, i.e. sites with limited access for cranes or vehicular traffic, can provide construction ease and consequently reduce costs. Without the exploration of use of different materials and construction techniques, the building industry will continue to refine conventional practices to advance techniques. While refinements may increase construction efficiency, they might be considered as incremental changes. In comparison, quantum advancements in construction techniques may develop from the application of different construction materials.

The Scotia Place project, situated on the edge of Myers Park in the center of Auckland, New Zealand, is ideal for a studio apartment building. A combination of the small lot, height restrictions, and day light indicators resulted in the building's diamond footprint shape (see Figures 1 and 2). Additionally, the architect was able to face nearly all the studios towards the park. At the start of the project the goal was to achieve a low per unit cost of less than $100k (NZ) per studio. Initial studies indicated that in comparison to a concrete floor option, constructing the building with wood floors provided a structural construction cost savings. This was a positive financial factor that enabled the project to proceed with a wood floor design, even though this type of lightweight construction was considered to be unprecedented at the time.

The design for Scotia Place was completed mid-1999 while construction was underway on the foundations. Building construction is essentially complete at the time of writing this paper, January 2000.

The significantly reduced building weight resulting from the wood floor option required review of the applicability of conventional design practices and code requirements. Conventionally, seismic loading would be the governing load case. However in this instance, due to the reduced building weight, wind was the governing load case for the lateral load resisting elements. Furthermore, the code evaluation procedures for occupancy comfort under wind loading were not applicable for structures with building weights as low as that used for this project. In addition to these structural design related issues, acoustic functionality of the building was also an important design factor for this lightweight construction project.

This paper focuses on these issues, and attempts to succinctly present major design considerations factoring into the resultant structural design for the project. It is important to note that, although outside the scope of this paper, complete assessment of the structural design for the Scotia Place project required consideration of other issues which factor into use of wood floors in high-rise construction.



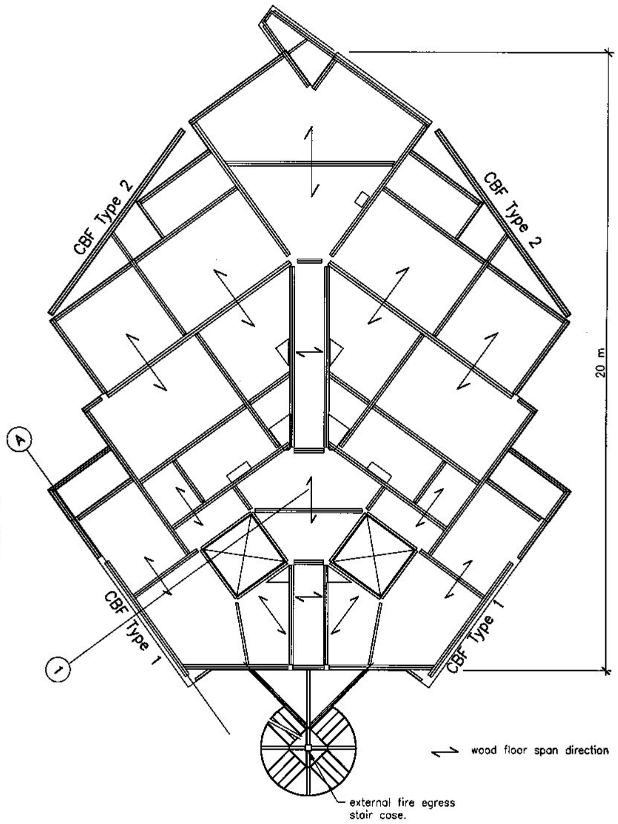


Figure 1: Artist Impression

Figure 2: Floor Plan, Steel Framing

STRUCTURAL FORM

Overview

The apartment tower consists of wood floor and steel framing, and is founded on a single level basement. The transfer level is a concrete slab at ground level, which distributes lateral loads to the cast-in-place and masonry perimeter basement walls below. Both the gravity and lateral load resisting systems have columns which are supported on 600-mm diameter piles. These reinforced concrete piles are cast-in-drilled-hole ("CIDH"), and provide tension and compression.

Gravity Load Resisting System

The glue laminated wood floor is formed from 30-mm by 65-mm laminates. Plank widths of 1,200-mm maximum were fabricated at the laminating workshop and spliced together on site to form a contiguous wood floor at each level. The wood floor is discontinuous at inter-tenancy wall locations, interrupting the diaphragm's load path and reducing impact vibration transmission (Refer Figure 3). Steel beams supporting the wood floor are raised higher than the primary steel beams to ensure the floor spans in one direction. This prevents two-way action, and therefore stresses perpendicular to the laminate joints are induced mainly from lateral loads. Where possible, the wood is continuous over the supports, enabling spans up to 2.9 m. Supporting the steel beam floor framing are tubular and "W" flanged steel columns. Some of the gravity columns are transferred at the ground floor, i.e. are discontinuous at the concrete transfer slab, to enable car parking and isles in the basement below.

Lateral Load Resisting System

The primary lateral load resisting system consists of wood floor diaphragms transferring loads to steel concentrically braced frames ("CBFs"). Collector beams at each floor deliver the loads to the node points of the CBFs. The elastically responding CBFs transfer the lateral loads at the transfer level, the top of the basement, to the perimeter reinforced concrete and masonry walls. The CBFs' overturning actions continue through the basement via the continuous columns to the CIDH piles. The pile design is governed by tension under lateral loads.

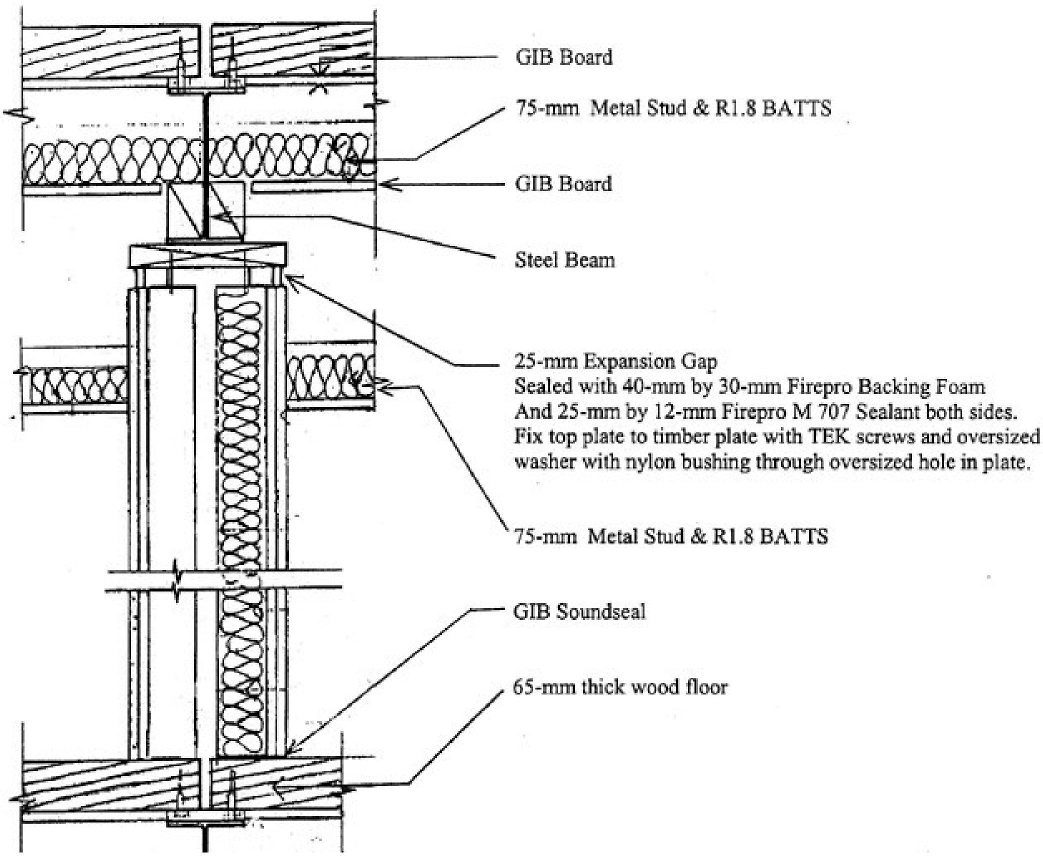


Figure 3: Floor, Ceiling and Wall Connection with 15-mm Wide Gap in Wood Floor

DESIGN BASIS: GLOBAL LATERAL LOADING

Overview

The design of the global lateral load resisting system for a building of this configuration with a concrete floor would normally be governed by earthquake design actions. However, use of the wood floor resulted in a lower building weight. This caused wind in the transverse direction to govern design of the structural steel lateral load resisting system, specifically, the lateral resisting system elements were sized to provide sufficient transverse stiffness. The wood floor design was governed by seismic loading.

Lateral analyses were considered for the building's two sets of axes, the principle geometric axis and the principle strength axis. The geometric axis was used for wind analysis and provided the maximum width of building to be affected. The principle strength axis, the line of CBFs, was used for seismic analysis, in accordance with local code requirements. Wind actions were assessed using static loads, and a dynamic modal analysis was performed for the seismic assessment.

For the building's global lateral load analyses, a rigid diaphragm was assumed for the determination of deflections and element actions. Consideration of the wood floor's flexibility, which is shown to be negligible, is addressed in the following section ("Wood Floor Design Considerations").

Wind Analysis

The allowable roof lateral deflection under transverse wind loading is limited to the height of the building divided by 500, as dictated by code. This limit on the building's deflection when subjected to service limit state wind loading determined the building's minimum lateral stiffness, i.e. the bracing and column sizes of the CBFs. The service limit state wind loading is an event defined by a 1-year return period and a 5% probability of exceedance. During this design event, inter-story drifts of 5-mm were shown to be likely. The building cladding system, and the acoustic and fire sealed internal partition walls required detailing to accommodate this movement.

Since the building was not classified by the local loadings code as a wind sensitive structure, a wind dynamic procedure in accordance with the Australian code was not required. However, because the building's shape and weight was significantly atypical, a dynamic wind analysis assessment for determining floor acceleration, a measure of motion one feels in the building from lateral loads, was attempted to ensure occupancy comfort was satisfied. It was determined that the approach outlined by the code was not applicable because the data that the code is based on was not within the range of this building. Specifically, the building density (defined as the total weight divided by the total volume) was 50% lower than the lowest value used in the data to develop the code. Therefore, the degree of interpolation required to obtain the floor acceleration could only be classified as an educated guess.

However, the code procedure indicated that floor accelerations would be inversely proportional to building density. Specifically, half the building density results in approximately twice the floor acceleration. This indicated that Scotia Place, having a lower building density due to wood floors, would be susceptible to higher floor accelerations, and therefore potentially more discomfort to the occupants. The code also indicated that floor accelerations would be inversely proportional to the square root of the damping of the building. Therefore, to offset the factor of increase in floor acceleration due to the low building density, detailing measures were made to provide significant damping. Optimally, the wood floor to structural steel connection detailing as designed would provide four times the 0.5% to 1% of damping normally exhibited with a conventional building. Although not quantifiable, due to the number of inter-tenancy walls and elastomeric sealant between the walls, ceilings and floors, it was anticipated that >5% damping was provided. Lastly, the shape of the building was such that reattachment of wind vortices would be unlikely and therefore the cross-wind acceleration component would not contribute to the total floor acceleration. Based on the added damping and the shape of the building, the floor accelerations anticipated were expected to be below that recommended for occupancy comfort.

Seismic Analysis
In the longitudinal direction of the building, seismic actions applied in the direction of the CBF line were able to be developed in the elastic domain by the members sized for the wind loading in the transverse direction. A dynamic (modal superposition) analysis was used to determine member actions and deflections. A hybrid analysis was performed to include the effects of the wood diaphragm. A mathematical model created with ETABS software performed a modal dynamic analysis and wind static elastic analysis, assuming rigid floor diaphragms. A model of the wood diaphragm, including the effects of the connections, using plate elements for the diaphragm with 65-mm thick wood properties, was developed with the structural software SAP2000. The maximum loads from the global analysis with a rigid diaphragm were applied in the local analysis of the wood diaphragm model. Refer to the following section for details of this analysis.

Wood floor DESIGN CONSIDERATIONS

Overview
Use of a wood floor diaphragm concept resulted in numerous unique design considerations. With regard to the building's structural performance, the simplifying assumption of a rigid diaphragm in the global analysis had to be assessed considering the demand actions from the aforementioned seismic analysis and the properties of a wood floor. The earthquake demand actions generated the highest stresses in the wood floor and lateral loads on the screws. The structural effect of implementing a wood floor in the global analysis was incorporated by performing a local assessment of the wood floor diaphragm and interpolating how the local analysis results would affect the global response. In addition to the structural diaphragm issue, use of a wood floor required special considerations to be given to floor vibration, acoustic transmission, movements from changes in wood moisture content, and fire rating.

Structural Diaphragm Analyses
The structural performance of the wood diaphragm was analyzed in the following two ways:

<u>Method 1: Statics using seismic parts loading</u>
Hand calculations were performed assuming a continuous wood floor of 65-mm thickness, with design loading derived for seismic parts from the New Zealand loadings code. The diaphragm was assumed to be simply supported between the CBFs. The stresses within the floor were assumed to resist plank bending, that is, plane sections remained plane. Additionally, it was conservatively assumed that there were no chord forces. The demand acceleration from this method was 0.45 g. See Table 1 for a summary of demand stresses and lateral screw forces.

<u>Method 2: Hybrid analysis using ETABS and SAP2000</u>
A hybrid analysis was performed to approximate the impact of the magnification of diaphragm accelerations as a result of diaphragm flexibility. The first part of the two parts of this analysis was to obtain the demand floor accelerations from the global analysis of the building assuming rigid diaphragm action. The top floor acceleration was used as this would be largest acceleration anticipated. (Note: No account was made for dynamic magnification of this acceleration due to floor flexibility; this assumption was verified as valid after obtaining the local analysis results.) The second part

of the hybrid analysis was to apply this demand acceleration to an elastic finite element model, modeling the effects of the flexible diaphragm action introduced to the system by a wood floor. The model consisted of shell elements capturing the wood floor and beam elements capturing the wood floor to steel beam connection, that is, the flexibility introduced by the gap beneath the inter-tenancy walls. The floor to steel beam joint flexibility was included in the model by using a lower bound secant stiffness approximation. (Refer to Figure 4 for the simplified stiffness developed from the actual predicted non-linear response.)

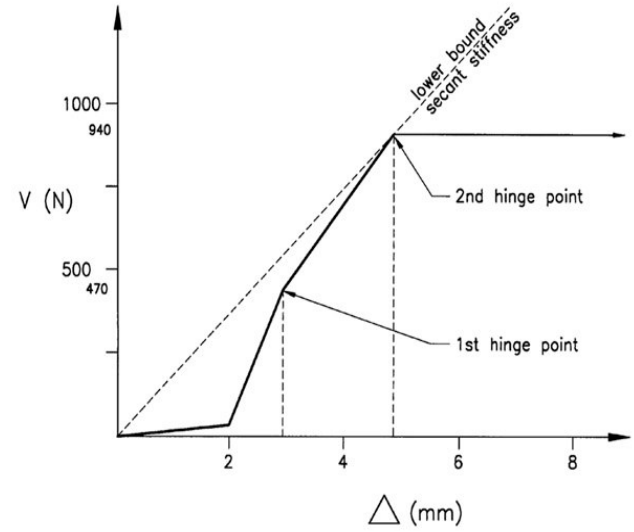
The simplified response depicted in Figure 4 was developed by considering the non-linear response of the connection as the wood floor moved laterally relative to the steel beam. The following sequence of events summarizes this movement. First, the gap closes between the screw and the oversized hole in steel beam flange, followed by the first of two flexural hinges forming in the screw, at the soffit of the wood floor. With additional lateral deformation, the second and last hinge would be formed. At this point a mechanism would be formed and the full lateral capacity of the connection would be developed. Although the design goal to not develop this force level or deflection was achieved, a ductile response in the connection would be exhibited should the actual demand actions be greater than the designed actions.

From this analysis, maximum principle tensile stresses within the wood floor and lateral loads on the screws were obtained. The diaphragm deflection and first fundamental mode of the non-linear response of the beam to floor connection was also captured to quantify the effect of the flexibility, hence, the expected amount of diaphragm acceleration magnification. The result of this hybrid analysis was a demand acceleration of 0.23 g, a factor of ~2.0 *smaller than the first method.*

Table 1: Summary of Structural Diaphragm Analyses

	Method 1: Statics Using Seismic Parts Loading	**Method 2: Hybrid Analysis Using ETABS & SAP2000**	**Capacity**
Demand diaphragm acceleration	0.45 g	0.23 g	N/A
Maximum wood tensile stress	0.41 Mpa	0.21 MPa	0.6 MPa
Maximum Screw Lateral Load	1,980 N	560 N	2,140 N w/out gap 940 N w/ gap

In order to use a demand diaphragm acceleration other than that derived from seismic parts loading (Method 1), the dynamic effects of the diaphragm and diaphragm connection flexibility needed to be included with the dynamics of the building to capture any magnification. The period of the diaphragm and its connections, based on lower bound secant stiffness, was 0.13 seconds. The period of the building was 1.8 seconds. Based on the ratio of these two periods, the deflection obtained from the local analysis of the diaphragm, and the fact that the actual amount of mass being excited and transferred by the diaphragm was less than that used in the both methods, the overall building deflections and wood stresses obtained from the maximum acceleration level based on a rigid diaphragm assumption was judged to be reasonable for design purposes.



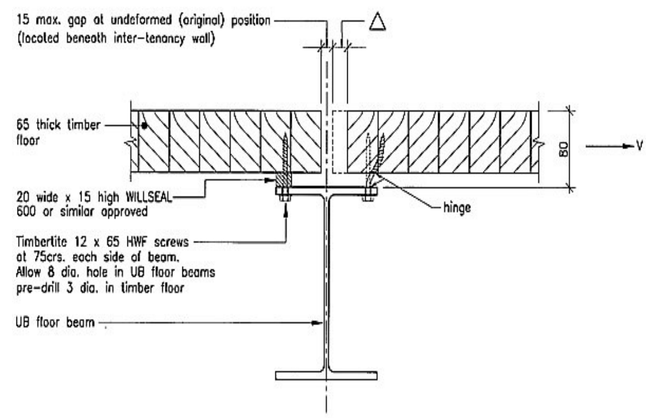


Figure 4: Non-linear Wood Floor to Beam Connection

Figure 5: Wood Floor – Beam Connection

Floor Gravity Deflections and Vibration

Determining the span lengths as well as the sizes and configurations of supporting beams was based on limiting deflections from applying dead, superimposed dead, and live loads. The accumulative deflection affect of the supporting steel beams was included. Floor vibration for the steel beams and the wood floor were assessed separately. The steel beam satisfied the recommended frequency limits. Floor vibration was assessed using Chien's recommendations for the steel beams and checking the deflection from a 1-kN point load applied mid-span on the wood floor, in accordance with NZS 4203:1992. The deflection of the wood floor exceeded the 1-mm limit, failing the code provision of using floor stiffness as a measure for satisfying floor vibration. However, the wood floor design was deemed acceptable since similar types of construction, i.e. using similar wood thickness and span, have been used in low-rise buildings. Furthermore, since a wood floor span was contained within a single apartment unit, vibration across an inter-tenancy wall could not occur.

Acoustic Affects

The wood floor was designed to have a 15-mm wide maximum gap beneath the inter-tenancy walls. (Refer to Figures 3 and 5). This gap would prevent impact vibrations from passing from one apartment to another, thereby limiting horizontal communication between apartments. Impact vibration transmission is transferred through solid mediums. The interruption of the medium and introduction of other mediums, which have a different natural frequency, would act to intercept and filter out the driving vibration, respectively. This detail provided an Impact Insulation Coefficient ("IIC") of 55, on site. While the exact science of this phenomenon is outside the scope of this paper, the essence has been introduced to reflect the importance of acoustic issues with wood floor structural design and construction, particularly for apartment buildings. The vertical IIC of 55 was assured by testing at Auckland University on a full–scale model of the floor and ceiling between apartments. To provide the IIC of 55, a false ceiling and introduction of acoustic resilient mediums were used. The Sound Transmission Coefficient ("STC") required to minimize air borne noise was assured by virtue of accomplishing the IIC.

Moisture Content

Wood floor changes in moisture content are likely, and are likely to be seasonal. The greatest change is expected immediately following construction. Unfortunately, construction occurred during winter. With proper handling of the planks, including applying a water resistant coat and plastic at the laminating factory, the moisture content is kept below ~12% and changes are expected to be on the order of 2%. A designed moisture content change of 2% was adopted. Using moisture-expansion coefficients recommended by Breyer et al, the expected movement of a 3.5 m wide module was expected to be accommodated with the gap in the wood floor at the inter-tenancy walls and flexibility of the wood floor to steel beam connection.

Fire Rating

The twelve levels of apartments have a single external escape staircase accessed from an internal horizontal safe path. The building is fully sprinkler protected, with smoke detectors in the corridors, lift pressurization, fire hose reels, and a charged hydrant riser mains for Fire Service use.

The wood floors were not included as part of the fire separation system, although it could be shown that due to their thickness they would contribute favorably to the fire rating of the floor system. The problem with trying to incorporate the floor into the fire cell, was the need to also provide an adequate separation between apartments on the same level. This resulted in the apartment fire cell being enclosed by the inter-tenancy walls and the finished (visible) ceiling level, with the ceiling void that contained the structural steel beams and acoustic materials being considered as the floor/ceiling system. The fire separations thus defined (i.e. excluding the wood floors) achieved a 30-minute fire resistance rating for integrity. The fire ratings were achieved by using gypsum plaster boards as part of an approved and tested system. Insulation ratings as applicable to the element under consideration were achieved. Finally, it was determined that the steel structure would perform adequately under fire conditions without passive fire resistance.

Construction Methodology

The wood floor planks were made up of 1,200-mm wide wood laminated modules. Each laminate was 30-mm wide by the floor depth of 65-mm. Upon placement on site the 1,200-mm wide modules were spliced together by using 12-gauge screws at 75-mm spacing and plywood. The original design intended for 3,600-mm wide modules. Hence, the labor component on site was significantly increased, although crane time was reduced as the 1,200-mm wide modules could be handled manually. A temporary water-resistant sealant was used and each plank was wrapped in plastic to minimize the absorption of water.

Since the finished floor surface was the wood floor itself, protection of the floor during construction proved to be difficult. Although the design did not require site welding, construction corrections were made with site welding and some damage to the floor occurred. Sanding and then applying three coats of a hard clear sealant attained the finished wood surface. This sealant provided a protective coating that prevented heel imprints on the floor from foot traffic when in final use.

COMPARISON TO CONCRETE FLOOR

A preliminary design of a concrete floor with similar structural steel lateral resisting systems and foundations was performed during the conceptual design phase, prior to a contractor being appointed to the project. The contractor considered both options and explored different construction techniques with the wood floor scheme. Following a comparison to the conventional concrete floor construction methodology, and a detailed take-off of construction materials, the contractor elected to proceed with the wood floor option. The scrutiny that the wood floor was put through and the result that the contractor chose to use the wood floor is a testament to the potential construction savings that can be made with the system.

Structural material differences between the use of a wood versus a concrete floor, with similar configuration of steel framing, are as follows:

Table 2: Wood versus Concrete Floor

Structural Item	Wood Floor	Concrete Floor
Steel Frames: Diagonals	125 SHS and 150 SHS	150 SHS and 200 SHS
Floor Depth	330-mm	460-mm
Tower Steel Cols	200 SHS and 250 UC	250 SHS and 310 UC
Steel Beam/Col Joint	No Stiffeners	Stiffeners Required
Basement Conc Cols	450-mm dia	500-mm and 650-mm dia
Piles	600-mm dia – 2.5 m embedment	750-mm dia – 3.5 m embedment

CONCLUSION

This paper has presented the primary design issues for successful implementation of wood floor and steel framing as a construction system for high-rise buildings. The main issues presented for the Scotia Place project which one may use to expedite designs of future wood floor diaphragm buildings are: validation of the assumption of a rigid diaphragm in design of the lateral load resisting system; verification that wood floor to beam connections successfully accommodate wood movement from changes in moisture content; and provision of sufficient design details to address acoustic issues and to satisfy building functionality requirements. Additionally, the use of lightweight high-rise construction requires a higher amount of damping to keep floor accelerations from wind loading at an acceptable level. Damping can be readily added by strategically including elastomeric sealants in the design, which serve as acoustic and fire seals as well.

In comparison with concrete floor structural systems, the use of wood floor diaphragms reduces structural material cost. Additional cost saving is also achieved as the structural wood floor provides an attractive floor finish. Finally, use of lightweight construction in constrained sites can provide construction ease and consequently reduce costs.

This paper is presented in the spirit of contributing to the process of gathering and distributing knowledge to advance the implementation of wood as a structural construction material.

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