Six-Level Timber Apartment Building in a High Seismic Zone

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Summary

This six-level timber framed apartment building is under construction in New Zealand. The building, in Wellington, is in the highest seismic zone in the country and is predominately braced by plywood-lined walls. Specifically developed details address issues of acoustic performance, vertical load and lateral load resistance, and ductility under seismic attack. Use of such a system simplified the construction of a transfer structure between the apartments and car parking levels as well as providing an economical form of construction.

Introduction

Townscape Developments specialise in providing student accommodation in two cities in New Zealand. They also act as the main contractor to build their projects. Typically such buildings would comprise two or three levels of lightweight construction, sometimes over a concrete level above the ground floor. Whilst the quality of the buildings is appropriate to student use, New Zealand building regulations require that the standards for acoustic performance, as well as fire rating and structural performance are maintained.

This project was initiated when Townscape purchased an inner city site, requiring six levels of apartments to provide the required 130 beds, in addition to a car parking level. Townscape requested that a lightweight option be developed for this building if possible, rather than the more conventional concrete structure, because this is the construction system that they understand. The detailed design of the building was carried out by Holmes Consulting Group project engineer John Cuthbert, under the direction of the authors.

Design Considerations

The area of the site, at about 670 square metres dictated a building that used the entire site, with a footprint of approximately 21 metres by 31 metres in plan. The Wellington district scheme imposes a height limit on the site, which meant that to construct the required number of levels, the floor-to-floor height would have to be minimised and hence the structural depth at each floor level minimised. Also, as is common with residential construction over car parking, the structural grid for the residential levels would not match an efficient grid for the car parking level. This meant that a transfer structure was likely, one level above the foundations in order to maximise the numbers of car parks which could be provided.

The layout of the apartments also offered many walls, which could be used for both bracing and providing vertical support for the structure. This is because most of the apartments are studios, having the same proportions as a typical hotel room. This provides bracing wall lines at approximately four metre centres in each direction.

It became clear that a light weight super-structure could be designed for the apartment, and would achieve the requirements of minimising floor-to-floor heights both in the typical apartment floors, and to the transfer floor at level 1. A concept design utilising plywood floors on proprietory ‘I’ section joists, and predominately plywood-braced walls was produced, and subsequently developed into working drawings.

Figure 1

Design Loadings and Considerations

New Zealand is a seismically active country and Wellington is in the region with the highest design seismic loadings, the region being traversed by several active faults. Buildings in New Zealand are required to be designed to resist the gravity, seismic and wind loads specified in the New Zealand Loading Standard [1]. For earthquake loading, the standard specifies a design lateral load, as a proportion of total building weight, required to be applied to the building. This load depends on the location within the country, importance attached to the building, fundamental lateral periods of the building, site subsoil conditions and the ductility provided by the completed structure. This last aspect, the ductility, is a measure of the buildings ability to continue to support lateral and vertical loads when the building structure is loaded beyond the elastic range. New Zealand standards specify a minimum required ductility for buildings to achieve as a function of their height, as well as the maximum ductility that specific systems may be assumed to provide. Design of ductile structures is achieved in New Zealand using capacity design, whereby specific elements which can dissipate seismic energy through yielding are permitted to yield under design earthquake loading. Other elements of the structure are then designed to resist the so called “over strength” loads from those yielding elements, so that they in turn will remain elastic. The ability of various elements to dissipate seismic energy has been established over many years by a great number of large-scale tests.

New Zealand standards also specify the required wind design loading for buildings. Wind loads for New Zealand in general, and Wellington in particular, are higher than for many other countries in the world due to its geographic location. However, the design for wind is carried out using conventional limit state design and structures must be proportioned for strength to resist the greater of wind or earthquake loading.

Timber Super-structure Concept

The 6-level timber super-structure consisted of an approximately rectangular floor plate, braced by predominately plywood-lined walls in each direction. These walls are typically on the line between adjacent apartments, where they can run 6 levels through the super-structure without penetrations. In two cases, where walls adjacent the boundary have no penetrations, these are used as bracing walls. In addition, there are some walls between studios and the corridor which are used as bracing walls. Refer to figure 2 for a plan lay out of the walls.

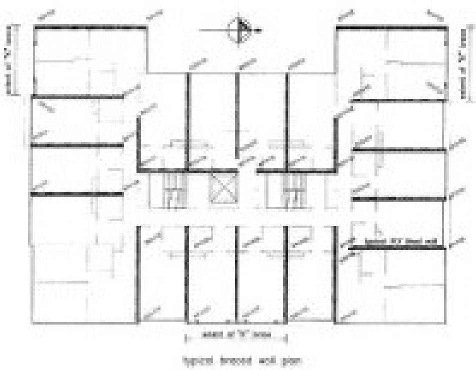

Figure 2

In the North-South direction, there are nine plywood-lined walls bracing the structure, in addition to one steel k-brace. In the East-West direction there are 10 plywood-lined walls bracing the structure in addition to two steel ‘k’ braces.

These k-braces were used in order to maintain an even distribution of bracing elements across the floor plate, and to provide a high degree of torsional restraint to the structure. This required incorporating bracing into the perimeter face of the building, which is pierced regularly by window openings. Plywood-lined walls would not be effective in such situations, where as the steel k-braces are.

The k-brace lateral stiffness was set to approximately match that of a typical plywood lined wall. This entailed spacing the diagonal braces further apart than usual, to reduce the brace stiffness. This also allowed the braces to accommodate typical window openings.

The floors for each level consist of 19-mm plywood fixed to proprietary timber I section joists. These joists, which are now commonly used in New Zealand, consist of mechanically graded timber chords glued to a plywood web. They achieve superior engineering properties compared with conventional sawn timber and may be handled and fixed in the same way as sawn timber joists. Various penetrations through the floor are present, allowing for 2 stairwells and a lift shaft, but also to provide services ducts distributed over the floor area.

Concrete Sub Structure

The timber super-structure is connected to a suspended reinforced concrete slab and beam structure at level 1. This level is in turn supported by a series of reinforced concrete columns, and perimeter concrete basement walls. Because the lay out of columns in the bottom level is chosen to optimise car circulation and parking, the columns do not line up with the load bearing elements from the timber super-structure above. Accordingly the floor slab and beams are designed to resist the vertical loads arising from gravity and earthquake action. The height limits on the building meant that this structure had to be constructed within an overall depth of 500mm. By keeping the beam and slab spans down to the range of five – six metres, and with the use of relatively wide beams, this was achieved. This would not have been possible had the super-structure not been of lightweight construction.

The slab also acts as a diaphragm and distributes the shear forces from the plywood-lined walls and k-braces into the perimeter concrete walls and some internal steel braces in the basement.

The foundation system for the building consists of a series of pads and strip footings founded in an alluvial material at shallow depth. Significant uplift loads are generated by the plywood bracing walls and k-braces, and where these exceed the dead load of the building they are resisted with passive soil anchors grouted into the ground.

Details of Bracing Walls

The ply bracing walls range in length from 4 – 8 metres long, although most walls are approximately 6.5 metres long. At this length they have an aspect ratio of about 2.4. Plywood thickness over the lower 2 levels is 19-mm, reducing to 15-mm over the central two levels and 12-mm over the top 2 levels. The plywood is nailed to timber studs using bright (i.e. non-galvanised) steel nails. Nail centres vary from 40-mm at the base of walls to 150-mm at the top of the building. The timber studs, grade F8, are 140 x 45 over the lower two levels and 90 x 45 over the upper 4 levels. Studs are designed to transfer the vertical loads from gravity down to the level 1 concrete deck and the plywood and nailing is designed to transfer the earthquake shear forces similarly.

The ductility of the wall is achieved by the bright steel nails bending when the load exceeds the design loading. This system provides a structural element which can achieve a displacement ductility factor of 4 as permitted in the New Zealand Timber Standard [2] and noted in various timber design guidelines [3] and [4]. To design assuming such ductility requires that other failure modes in the wall are suppressed. Accordingly, the over-turning moment on the wall and the resulting tension and compression chord forces at the wall ends are derived from the nail over strength loads. For bright steel nails, tests have shown the over strength loads may be taken as double the nominal yield load on the nail. The resulting tension and compression chord forces are too high to be comfortably resisted by timber members. Accordingly, steel posts were introduced to the ends of each wall. These posts are 125-mm square over the lower 2 levels and 89-mm square over the upper 4 levels. In addition to resisting the required loads, these post sizes could be concealed within the wall stud sizes. The plywood is nailed into the end timber stud member which in turn is bolted through the steel post to provide the required shear transfer. The timber plate at the lowest level of the wall is bolted into the concrete slab and the plywood nailed into that to transfer the shear into the concrete diaphragm.

The wall-to-floor junction which occurs at every level required special consideration. At this junction, horizontal floor shears must be transferred into the wall and the floor supported for gravity loads. In addition, the horizontal wall shears must be transferred through the joint to the level below, and the vertical loads in the studs transferred through the junction to the level below. Tension and shear forces in floors must sometimes be transferred across the joint to ensure the floor is tied back into walls at right angles to the wall under consideration. Finally, there are acoustic and fire rating issues to be satisfied, all in the minimum structural depth possible.

The acoustic requirements in particular are significant with lightweight structures. New Zealand building regulations require minimum Sound Transmission Class ratings to be satisfied through inter-tenancy walls, and vertically through floors. In addition, there are Impact Isolation Class requirements, to prevent excessive noise being transferred through the structure into adjacent spaces. The Sound Transmission Class requirements may be satisfied by using proprietary resilient fixings to fix the ceiling to the underside of the floor joist system and to fix one of the layers of the plasterboard wall linings to the studs (not the same side as the plywood bracing element is fixed to). However, the Impact Isolation Class requirements mean that the flooring material must not continue through an inter-tenancy wall, but must be separated by a saw cut immediately on 1 side of inter-tenancy walls. To ensure that diaphragm shears and tensions can be transmitted across the joint, as noted above, the joists are fixed using proprietary brackets into a ribbon plate which in turn is well nailed to the studs, and sections of steel strapping are placed across the joint from joist to joist.

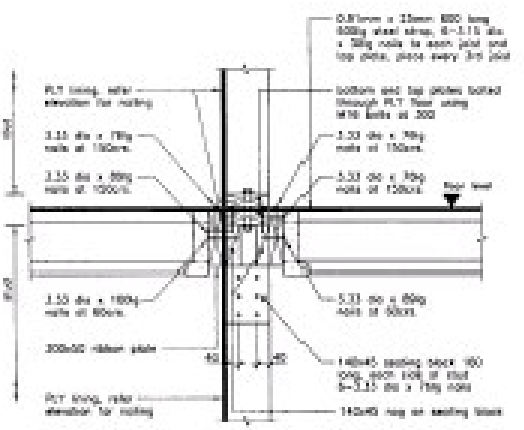

Figure 3

The usual Fig 3 practice of introducing boundary joist members in between the timber wall top and bottom plates can not be used in multi storey construction like this. This is because the modulus of elasticity of timber across the grain is only a small fraction of that along the grain and the deformation of the timber over the depth of boundary joist plus plates will cause damage to wall linings and cladding. To avoid this, the top and bottom wall plates are connected directly together and bolted to transfer wall shears. The floor joists are fixed to a ribbon plate, as described above, on the face of the studs. To provide adequate fixing into the studs for both horizontal and vertical shears, the ribbon plate is fixed into timber blocking, which itself is supported from seating blocks nailed into the side of the studs.
Figure 3 shows the arrangement where the floor joists are perpendicular to the wall on both sides. A similar detail is used where the joists are parallel to the wall on one side.

Technical Details

A Muto analysis was carried out for the timber super-structure, including modelling the plywood lined walls and the steel k-braces. This analysis assumes a rigid diaphragm at each floor level, and assigns lateral load to the resisting elements depending on their stiffness and their location within the structure. In assigning the stiffness to the plywood-lined walls the recommendation in the New Zealand Timber Structure Standard and Timber Design Guidelines were followed. These require deflection to be considered as a result of base rotation, plywood shear deformation, nail slip and flexural deformation of the wall to be considered. The displacements arising from the Muto analysis were also used to calculate the fundamental period of the super-structure in other direction, which is required to derive the earthquake loading. An alternative analysis was undertaken, whereby lateral load was assigned to each bracing element according to the tributary area of floor around that element. This provides for the situation whereby the floors behave as a flexible rather than a rigid diaphragm. As expected, this produces higher design loadings for elements near the centre of the structure, whilst the Muto analysis produces higher design loadings for elements around the perimeter of the structure due to the torsional effects on the building. In the final design of elements, the greater of the two values was used. This approach was considered prudent in view of the fact that a structure of this type had not been completed in New Zealand previously. The fundamental period of the timber structure in each direction was found to be approximately 1.0 seconds. This, combined with the use of a displacement ductility factor, $\mu = 4.0$ and the site subsoil properties led to a design base shear coefficient for the building of $C = 0.15$. This means that the lateral resisting system of the building is required to assist approximately 15% the total weight of the building, taken as the dead load plus approximately 40% of the live load. This gives a design base shear for the timber structure of 1650 kN. By way of comparison, the design wind loads, for the ultimate limit state condition, are 440 kN in the north-south direction and 640 kN in the east-west direction. It can be seen that the seismic loads considerably exceed the design wind loads, as expected.
The New Zealand Loading Standard also requires that serviceability limit states be considered. The serviceability earthquake load for this building, taken as 1/6 of the elastic load (1/6 of the load corresponding to a displacement ductility factor of $\mu = 1.0$) is equal to 1100 kN. Again, by way of comparison the serviceability limit state wind loads are 280 kN in the North-South direction and 420 kN in the East-West direction. Again, the earthquake considerations are the governing ones.
The maximum design shear forces in the plywood are 30 kN per metre under the ultimate limit state. This load could be resisted using 19-mm F11 plywood nailed with 3.33-mm diameter annular-grooved nails at 40-mm centres. The components of total wall deformation were typically as follows:

- Base Rotation 28%
- Plywood Shear less than 1%
- Nail Slip 59%
- Flexural Deformation 12%

New Zealand Standards also specify peak inter-storey displacements which buildings must not exceed under the level of earthquake loading corresponding to our displacement ductility factor equal to 1.0. This allowable inter-storey displacement is 2% of the storey height for building structures up to 15 metres in height, and reduces lineally to 1.5% for structures up to 30 metres in height. For the timber super-structure, the allowable inter-storey displacement is 1.98%. The actual peak inter-storey drift was 1.3% of storey height.
The concrete sub structure was designed for a total earthquake force derived from the over strength shears of the timber super-structure plus the earthquake loading on the concrete structure itself corresponding to a displacement ductility factor $\mu = 1.0$. The total design base shear at foundation level was 4060 kN. In strong earthquake shaking, the concrete sub-structure will remain elastic and energy dissipation will occur in the timber super-structure.

Construction Program

Erection of the timber frames onto the level 1 concrete slab began in mid February 2004, and is expected to progress at the rate of about two weeks per floor level. The pre-nailed timber frames and steelwork are lifted into place by mobile crane, and then the bolted and nailed connections are made. The building is intended to be open for Student accommodation in June 2004.

References

1. NZS 4203: 1992 – Code of practice for General Structural Design And Design Loadings For Buildings
2. NZS 3603: 1993 – Timber Structures Standard
3. The Feasibility of Multistorey Light Timber Frame Buildings by Geoffrey C Thomas, June 1991
4. Feasibility of Using Timber for Medium Rise Office Structures by Mary Ann Halliday, June 1991