HANLON & PARTNERS Ltd.
CONSULTING STRUCTURAL ENGINEERS

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7 July 2015

Spectra Ltd
PO Box 613
Dunedin 9054

Attention: Bevan Meddings
bevan@spectra.net.nz

Dear Bevan

184 HIGH STREET, DUNEDIN – UPDATED DETAILED SEISMIC ANALYSIS

As requested we have reviewed our Detailed Seismic Assessment carried out in 2012.

We can confirm that the items discussed in our Report dated 13 November 2012 have been addressed.

The building's current seismic strength is at 100% NBS (New Building Standard)

Attached with this letter for your reference are the following documents:

a) Hanlon & Partners Ltd DSA Report dated 13 November 2012.
b) Hanlon & Partners Ltd drawings No. 13966 - S1 - to secure first & second floors.

If you require further information please call.

Yours faithfully
Hanlon & Partners Ltd

DI Hand

David Hand

Job No. 13699

HANLON AND PARTNERS LTD
CONSULTING STRUCTURAL & FIRE ENGINEERS
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Email: office @ hanlons.co.nz

13 November 2012

Spectra Limited
P.O. Box 613
Dunedin 9054

Attention: Bevan Meddings
bevan@spectra.net.nz

Dear Bevan

184 HIGH ST, DUNEDIN
DETAILED SEISMIC ANALYSIS

We have carried out a detailed structural analysis of 184 High St, Dunedin. Based on failure of the weakest members that will cause collapse of the building we estimate that the building achieves 75% of the strength to resist earthquake loading that would be required of a building built to today's Earthquake Standard.

Building and Analysis
Our review is based on obtained copies of the original 1974 structural drawings. The geotechnical conditions at the site have been assessed for seismic evaluation purposes as "Class C; shallow soil". The underlying rock is described on the Institute of Geological & Nuclear Sciences Geological Map of the area as Olivine Basaltic Rocks.

The building is relatively low rise consisting of four 3.5m high storeys' above a variable depth basement. The foundations are shallow, being cut into local rock and following the hillside. The building has a stiff lift and stairwell core on its north side with a moment resisting concrete frame attached to and surrounding it. The floors consist of precast prestressed 'double T' beams with a reinforced 50mm insitu concrete overlay which is tied into the beams that make up the concrete frame.

The structure was modelled using structural analysis software called ETABS and subjected to the loading regimes set out in AS/NZS1170, which the current New Zealand design loadings Standard.

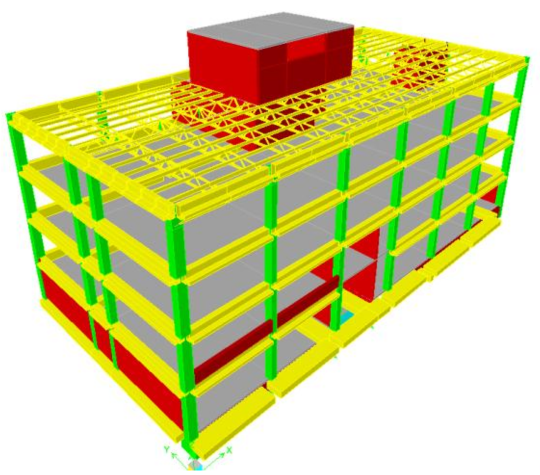



Figure 1: Showing analysis model of building viewed from the same angle as the accompanying photograph. The concrete perimeter panels at each floor were modelled as inertial weight but are not shown on the model.

The building was analysed by the 'modal response spectrum method' as set out in NZS1170.5 Earthquake Actions.

Material strengths and stiffnesses used were as specified in the New Zealand Society for Earthquake Engineering (NZSEE) document "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes", June 2006.

Dunedin shares the lowest probability in New Zealand of experiencing the design earthquake loading, and is thus assigned the lowest 'Z factor' value of Z = 0.13 allowed in NZS1170.5. This means that the accelerations and therefore the forces applied to buildings here are relatively low in a New Zealand context.

The capacity/demand ratio of the strength of various members throughout the building gives an indication of which members will fail before the others under a set loading, eg the 1170.5 design earthquake loading. A capacity/demand ratio greater than 1 means the member will not fail under that loading and less than 1 means that failure will occur. Members with a lower c/d ratio will fail before those with a higher c/d ratio.

Results

1. The response spectrum analysis has shown that the reinforcing that ties the 2nd storey floor into the lift and stair well will start to fail in shear when the building is subject to shaking that is 75% of what the building would be subjected to under current Standard requirements. As the overstressed ties break other ties around the stairwell/lift shaft become more stressed until they also become overstressed and break leaving the floor with fewer ties as shaking continues.
The building gets some of its resistance to lateral forces from the floor diaphragms transferring load. As the building loses its diaphragm connections it becomes less stiff and lateral displacements become greater with continued shaking. The floors are also supported vertically on a 75mm wide rebate on the beams and until the displacements exceed 75mm the floors should remain in place.

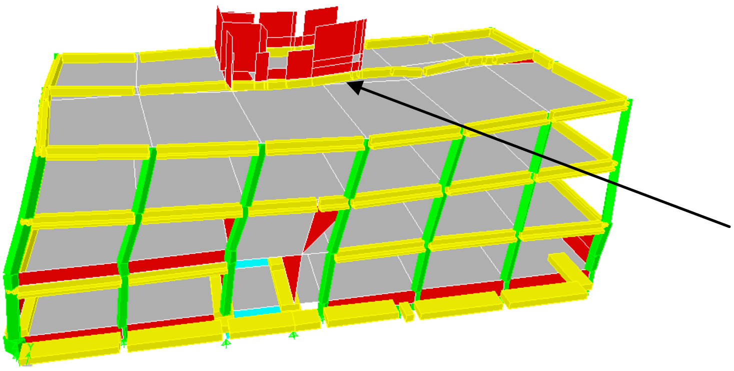


Figure 2: Showing location where floor ties start to fail under 75% current earthquake loading. The roof has been removed from the view for clarity.

2. Under predominantly north/south direction shaking the ends of the building try to bend around the stiff core like wings. Most of the perimeter beams of the building are tied to the floor diaphragms and are thus restrained to the displacement of the stiff diaphragms. At roof level however the beams do not have a diaphragm to restrain them and are subject to lateral flexure about their weaker axis.
The roof perimeter beams are most stressed where they join to the stiff central core as they bend about it. The beams at this location have the capacity to resist up to 80% of the transverse flexural demand of the current earthquake Standard loading.

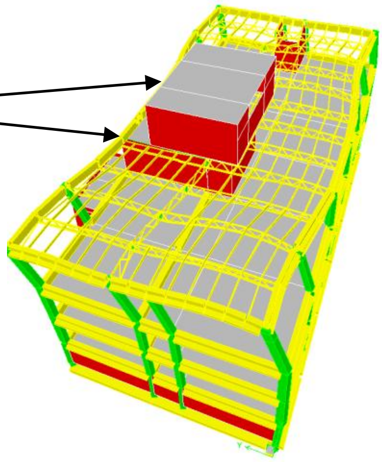


Figure 3: Showing building under north-south earthquake loading with exaggerated displacement and sections of perimeter beams which fail in lateral flexure as the building bends about the stiff central core.

3. Inter-story drift is the difference in lateral deflection between two adjacent stories of a building subjected to lateral loads.The accurate estimation of inter-story drift ratio and its distribution along the height of the structure is very critical for seismic performance evaluation purposes since the structural damage is directly related to the inter-story drift ratio.

   The current provisions in NZS1170.5 limit inter-story drift to 2.5% of the storey height between any two adjacent floor levels.

   The interstorey drifts in 184 High St, under current Standard earthquake loading are around 0.1%; well within the 2.5% limit between any two adjacent floor levels from NZS1170.5.

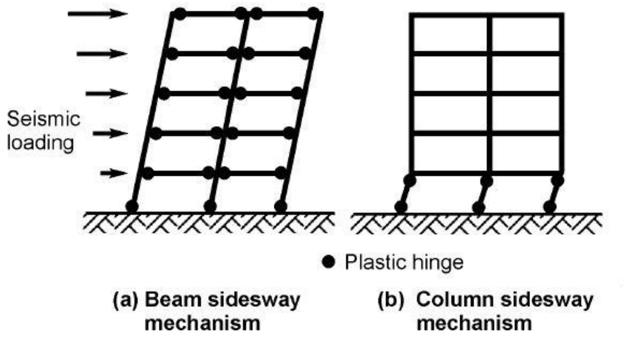
4. One of the discrepancies with current code requirements is the spacing of the stirrup ties around the longitudinal reinforcing bars in the columns, which are around twice the distance apart than stirrups of similar size under current Standard requirements.

   It is not possible to give this feature a %NBS value as the current standard requirements are based on experimental data rather than a direct correlation with the theoretical reinforcing buckling load demand and the capacity of the reinforcing to carry that load.

   The longitudinal reinforcing bars in the columns will only be called upon to be resistant to buckling from axial load if the concrete core of the columns inside the perimeter of the longitudinal reinforcing is crushing and the reinforcing is required to carry the axial load. This will occur where ‘hinges’ form in columns and beams where the available flexural strength has been exceeded.
   From our calculations the flexural strength of the columns will not be exceeded and ‘hinges’ will not form in the columns under the earthquake loading for Dunedin for building founded on rock.

5. It is evident from the earthquake demand / capacity ratios for the various elements of the building that a form of ‘capacity design’ has been carried out in the design of the building members.

   In the capacity design of structures design ‘hinges’ are allowed to form in places that will not cause building collapse and the members are made stronger in the places where hinge formation would lead to collapse of the building. Where the hinges are allowed to form special measures are taken to ensure the members stay intact and hold up the building, allowing the occupants to escape.
   In the diagram below the frame with the beam sidesway mechanism will not collapse with the formation of plastic hinges where they are indicated but the column sidesway mechanism will.



At 184 High St the beams have a lower c/d ratio than the columns meaning that a beam sidesway mechanism will be likely to form rather than the column mechanism. This means that although members in the building may have failed, collapse is avoided and the occupants are able to escape.

The analysis shows that the beams do have sufficient strength to resist the current earthquake loading (around 150%) and their c/d ratio is lower than most of the columns which all have a c/d ratio greater than 1.9, ie 190% NBS.

6. A potential critical structural weakness that buildings may possess, that has been brought to the forefront by the Christchurch earthquake is the vulnerability of the stairs to collapse preventing egress from the building even though it may remain standing post-earthquake.

   We have assessed the performance of the stairs as recommended by the Department of Building & Housing in accordance with the Report to the Royal Commission on Stairs and Access Ramps between Floors in Multi-storey Buildings.

   The stair stringers are steel RHS sections which are welded to 'C' sections that are set into the landings with reinforcing lugs. Under lateral loading there is sufficient weld such that the C section will be bent before the weld capacity is exceeded and will thus be ductile enough to remain intact during earthquake shaking. This building is relatively low rise and the interstorey displacements (difference in displacement between two adjacent storeys) under earthquake loading should be sufficiently small not to cause failure at the stair connection to the landing.

   The stairs in this building have been found to meet the requirements of the report and should not therefore represent a critical structural weakness in the building. This can only be verified by detailed structural analysis.

Conclusion

The building has an assessed %NBS score of 75%NBS, which is regarded as exposing the occupants to **low risk** of earthquake damage.

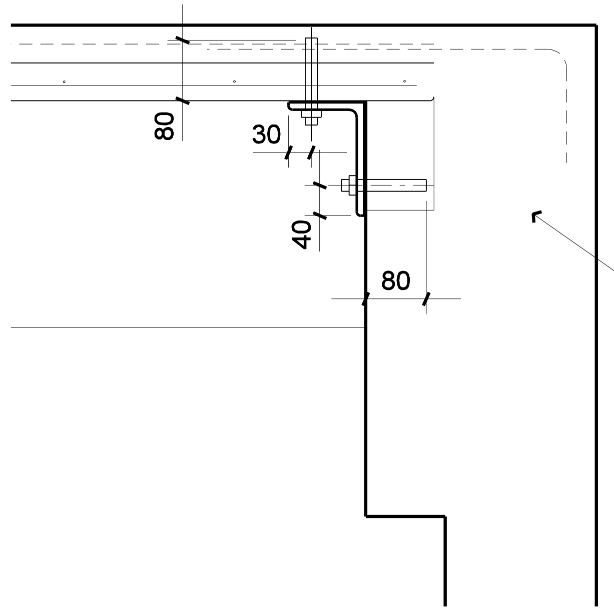
Our opinion of the stair details is that they do not represent a critical structural weakness in the building.

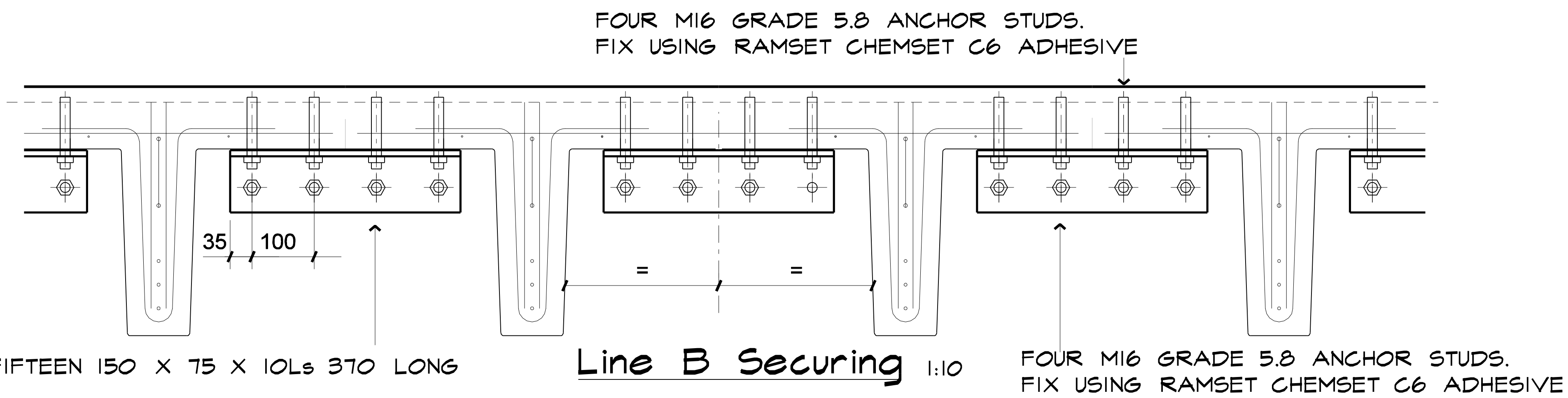
For further information please do not hesitate to call me.

Yours faithfully
Hanlon and Partners Ltd

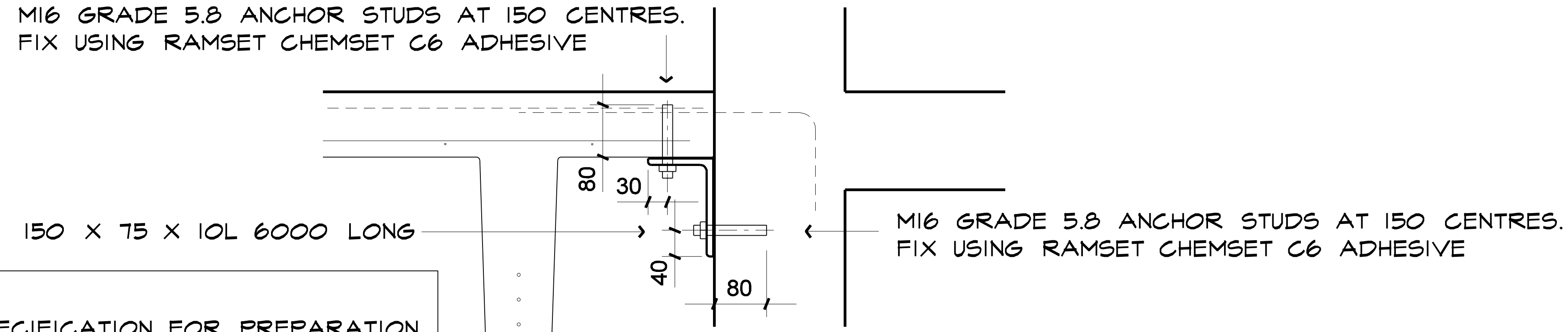
Lyndsay M^c Grannachan

Lyndsay McGrannachan
BE (Hons), BSc, PG Dip, CPEng, MIPENZ



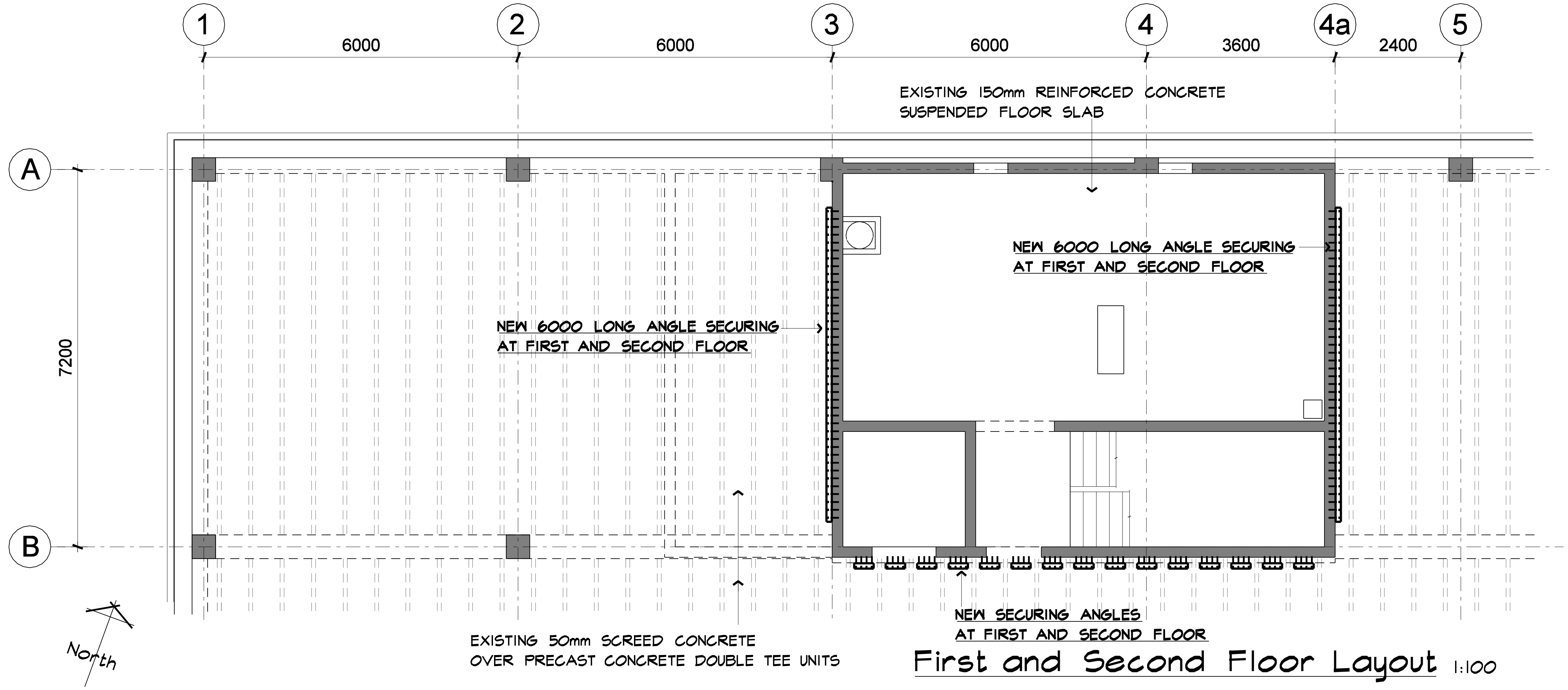


Line B Securing 1:10



Lines 3 and 4a Securing 1:10

CONFIRM DIMENSIONS ON SITE BEFORE COMMENCING FABRICATION.
ALL STRUCTURAL STEEL ANGLES TO BE SHOP PRIMED. REFER SPECIFICATION FOR PREPARATION.
ALL BOLTING TO COMPLY WITH SUPPLIERS SPECIFICATIONS AND REQUIREMENTS



First and Second Floor Layout 1:100

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184 High Street Dunedin. First and Second Floor Securing

DESIGNED	L McGrannachan	DATE	January 2013	SHEET NO. 1
DRAWN	L McG. M T.	SCALE	as shown	
ISSUED	30. 01. 2013	JOB NO.	13699	Revision