Wickham Woolstores

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Wickham Woolstores
DA Report
Structural Engineering

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Issue and Revision Record

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Contents

Executive Summary and Certification

This Structural Engineering Report is provided to support the Development Application submission for Building 1.

The report addresses the primary structural engineering issues associated with building condition and the structural engineering approach to adaptive re-use.

The report covers the structural engineering approach to;

- Condition and remediation of condition defects.
- Strength and capacity for loads imparted by the proposed new use.
- Approach to resistance to lateral loads from wind and seismic actions.
- Fire resistance of structural elements.
- Footings and foundations

Subject to appropriate remedial and strengthening works in detailed design we certify that the building has adequate structural capacity to be adaptively re-used for the purpose proposed.

1 Building Description

The building is one of three buildings on the site that were originally constructed as wool stores.

The site is a level site at the corner of Annie and Milford Streets. The Annie St façade is the front façade and faces south.

The building has had several tenants since it was a wool store, however these occupants have not apparently made any major changes to structure.

The building is rectangular in plan and has four levels.

The roof is a steel framed saw-tooth roof with south-light windows.

The external walls are cavity brickwork housed within a reinforced concrete frame.

All internal floors are timber boarded and timber framed, however there are localised slabs or toppings in toilet areas.

There are three access and egress stairs housed in brick shafts, two passenger and goods lifts, several open timber stairs between levels and several timber framed goods hoists (assumed to originally have been wool bale hoists).

The front (south) of the building has had office partitioning and toilet facilities installed at ground floor (Level 1) and at Level 3.

2 Building Condition

2.1 Roof Structure

The roof is clad in fibre cement corrugated cladding. The cladding was not inspected in detail but appears to be predominantly watertight with only localised leaks. These localised leaks are however significant and appear to have been leaking for some time. It is considered likely that the cladding contains asbestos.

The rafter or purlin system was not visible at the time of inspection since raking ceilings are in place. The ceiling material has not identified but may have the potential to contain asbestos.

There are 13 primary rectangular steel trusses running east/west across the building which also form the framing for the south-light glazing.

These are supported at mid-span by steel columns fabricated from two 175mm x 75mm channels sections stitched with welded plates to form 175mm x 150mm box sections.

There is a single series of triangular steel trusses running north/south down the centre of the building linking the steel columns.

The column grid in the north/south direction is at double the spacing of the grid of the timber columns on the floors below.

Most of the glazing is intact in the south-lights, however some panels have been replaced with polycarbonate sheeting or other cladding.

The roof appears to be level and well aligned with no major distortions and, where the steel framing is visible, it is generally lightly corroded. There are however sections of mullions that have corroded through and failed in recent storms.

2.2 External Brickwork

The external walls are infilled with cavity brickwork. For most of the building the infill is two skins of 110mm with a 50mm cavity. In the front façade and at the upper floor this varies. It is not clear if the brick skins become thicker or if a wider cavity is used in these locations.

The bricks appear to be uniform, well fired, durable, dry pressed bricks with little sign of degradation.

The mortar appears to be a cementitious mortar (rather than a pure lime mortar) in good condition with joints filled and pointed and little mortar loss.

There are areas of brick joint cracking associated with differential in-plane movement in the external walls. Crack width is generally small (less than 2mm). It appears likely that the movement is settlement related.

There are also areas of vertical brick cracking near the building corners which may have a brick growth component or may be the result of impact damage. These appear to be relatively minor.

The cavities were not inspected. Bearing in mind the condition of adjacent concrete and the proximity to the harbour, there is the potential for cavity ties to have corroded and this will be checked. Installation of remedial cavity ties is a relatively straightforward process if necessary, although maybe a little more complicated if there are wide cavities at the upper floor.

2.3 Perimeter Concrete Frame

The external concrete is in a highly-deteriorated condition. The west façade is in the worst condition, closely followed by the east façade. The north façade is a little better and the south façade is in the best condition, although it also shows signs of spalling.

The exposed concrete shows an angular basalt aggregate but there are wide variations in the density and permeability of the sand/cement paste matrix.

It is apparent that there was aggregate segregation at the base of columns at each lift, as the concrete was placed. This occurs when concrete is dropped into place from the column head (i.e. without using a delivery pipe) and/or is insufficiently vibrated.

At these locations, the spalling of concrete and corrosion of reinforcement is at its worst.

However, even in locations where there is a dense concrete matrix (e.g. in the beams), there is still spalling and corrosion.

The likely contributing factors are: permeable concrete, insufficient cover to the reinforcement and problematic concrete chemistry.

The majority of the damage is to the exposed external concrete faces, however there are localised areas of internal spalling which may indicate that the problem goes deep within the concrete matrix.

The concrete has had a cementitious render placed over it. It is not known at this point whether this was always in place or whether this was installed as a protection measure when the concrete was noted to be deteriorating.

A full concrete condition analysis is being carried out. At this stage it appears likelythat the west, north, and south facades would require at least complete removal of render and removal and re-build of the external concrete faces to a depth of around 70mm. The south façade might need less work.

In addition to general condition there are localised areas of beam cracking because of stresses set up by, what appears to be, differential settlement in external walls. The movement is relatively small but has generated crack widths in beams of up to 2mm, which may have been sufficient for localised yielding of reinforcement.

This damage is however relatively localised and infrequent and is readily repairable.

There is a flashing below the brickwork at the bottom of each infill panel and weep holes in the brickwork. At most levels this flashing over-flashes the render. However, at first floor the flashing tucks behind the render which causes a weakness leading to render delamination. This is readily repairable.

At the building corners, there is minor cracking at the beam/column interfaces. This does not appear to be major but warrants further investigation.

The entire façade on all sides will have a level survey to check location and quantum of any settlement. Settlement does not currently appear to be a major issue for the facades.

2.4 Timber Structure

The timber structure consists of hardwood tongue and groove floorboards of typical dimension 85mm wide x 22mm thick laid east/west.

These boards are generally sound but are heavily degraded by abrasion and wear. There are local failures. There are areas of distortion and cupping where differential settlement has taken place or moisture has affected the floors.

There is little evidence of termite or borer damage.

For a warehouse the boards are relatively thin, however quite adequate for residential use. This may relate to the general wool storage role, which probably did not impart heavy point loads.

The floor boards are supported by high quality sawn hardwood joists spanning north/south. The joists are typically 240 mm deep x 60 mm wide and span single spans, butt jointed over the main bearers. They are typically at between 380 and 400mm centres (possibly nominally at 15inch centres). They appear visually to be quite sound and an initial estimate of stress grade would be at least F22.

The joists have herring-bone blocking at mid-span.

The floor joists are supported by bearers running east/west at approximately 4000mm centres.

The bearers are hardwood and are predominantly hand adzed and so vary dimensionally. A typical dimension is 290mm deep x 220mm wide. Considering drying shrinkage this would equate to an original 12 inches deep by 9 inches wide.

As an initial estimate a grade of F22 would be appropriate. This will be confirmed by inspection by a timber grader.

There are some localised deeper bearers. It is not clear if this was for loading reasons or if these are just replacements which happened to be a little larger. They appear to be later replacements because the steel corbels have been cut to allow for their installation.

There are also some circular sawn bearers. It is assumed that these are not original.

The bearers are supported on steel corbels which sit onto the timber column heads. These corbels are formed from rolled steel joists laid on their sides. Approximate RSJ dimensions are 305 mm deep x 130 mm wide across the flange x 760 mm long.

There is a single vertical bolt securing bearer to corbel.

The bearers are secured at the external walls with a seating onto a projecting concrete corbel and by a steel plate and bolt arrangement. The steel plates are anchored into the external concrete.

Consequently, there is a continuous tie across the building at each line of bearers (i.e. at each grid). This tie is limited in strength by the bolt and timber capacity.

There are some bearers with splits from drying shrinkage, however in general they appear to be in good condition.

The timber columns are also hardwood and predominantly hand adzed with some circular sawn columns, which are expected to be non-original.

Column dimensions appear to reduce slightly up the height of the building. From ground to first floor they are typically 250mm x 250mm, from first to second they are 225 x 225, and from second to third they are 215 x 215. Dimensional variations are +/- 10mm in each direction on each floor but generally +10mm rather than -10mm.

There is some column splitting, some bowing and there are some minor tilts, however for the most part, columns are straight within reasonable tolerances.

The columns have suffered some borer attack (very localised) and some impact damage on corners but appear generally sound.

Column bases are below the suspended timber ground floor and could not be inspected in all cases. Where columns had been exposed for test pits, they were supported clear of the ground on pile caps and appeared to be reasonably sound. These bearing areas will all be drill tested to check condition.

2.5 Footings and Foundations

Some opening-up works have been carried out to provide access for geotechnical investigation of the footings and to understand the existing footing system.

At the column-base locations and projecting above ground level there are rectangular concrete plinths. Typical dimensions measured were 1550mm long by 710mm wide. These plinths support the column bases as well as the ends of the ground floor timber bearers (310mm deep x 250mm wide).

Around the ground floor loading docks there are brick walls supported on reinforced concrete beams.

The perimeter external walls sit onto reinforced concrete beams.

The concrete pile caps are each supported by a pair of 300mm diameter timber piles.

The perimeter concrete edge beams are supported by closely spaced timber piles.

The perimeter of the building does show some in-plane deflection which is expected to be due to differential settlement. This shows up as cracking in the infill brickwork in certain bays and racking of the beams at floor level leading to cracking in the beams.

Within the building there is a very distinctive apparent upward bulge in the floor structure in the north-east corner. This apparent uplift is also noticeable at the first internal row of columns from the east wall down the length of the building, although the amount of uplift appears to reduce moving towards the south.

Although there is some minor differential movement in the external wall at the north-east corner of the building, it is not consistent in magnitude with the magnitude of the movement in the adjacent floor. The cause is unclear.

Potential causes or contributions could include:

- Flotation of some underground structure (e.g. a pipe or a tank). The survey indicates a drainage easement in the area of uplift.
- Heave caused by swelling of the timber floor (e.g. due to water ingress)
- Uniform settlement of the remainder of the building with this section being supported on a more rigid bearing material (i.e. rather than having lifted, this section has not settled as much as the rest of the building).
- Swell in underlying clays (unlikely since the high-water table would indicate that the subsurface clays would be in a saturated condition).

- A man-made effect because of a building alteration (it is noted that the three columns in the worst affected area may not be original, since they are circular sawn, not adzed).

Further investigation will be carried out to diagnose the cause. Notwithstanding this localised area, it appears that there is differential movement between column bases throughout the building.

A floor level survey is being carried out to confirm the extent and significance of this.

Possible causes of differential settlement include:

- Differential loading history (quite possible in a warehouse)
- Differential pile performance due to variability in bearing material
- Degradation in piles (e.g. fungal decay)

2.6 Storm Water Drainage

At each roof gutter line, there is a rainwater head and downpipe each side of the building (east and west facades). On the west, these discharge to kerb, via an in-ground pipe in the verge. On the east, these appear to discharge into a piped system running to the south.

Several of the downpipes are disconnected and therefore saturating the ground. This may be contributing to differential settlement.

The in-verge pipes appear to have insufficient diameter when compared with downpipe cross section. This may have been causing surcharge and failure in the downpipes. The gutters, downpipes, and in-ground system will all be renewed to comply with capacity requirements.

3 Vertical Load Capacity

In determining the vertical load capacity of the structure, the following values were assumed;

Table 1: Adopted values

Property	Adopted value
Tmber grade	F22
Timber density	800 kg/m^3
Timber type	Hardwood
Concrete topping depth	200 mm
Concrete unit weight	25 kN/m^3

3.1 Timber Columns

Table 2: Vertical load capacity of columns summarises the adopted column dimensions and the associated load capacity.

Table 2: Vertical load capacity of columns

Column	Section (mm)	Length (mm)	Load capacity (kN)
Level 1	250x250	3370	1630
Level 2	225x225	3700	1240
Level 3	215x215	3770	1070

Under the adopted loading conditions, the existing timber columns have sufficient capacity for the structure to be repurposed for residential use.

3.2 Timber Floors

Table 3: Vertical load capacity of floor members

Column	Section (mm)	Length (mm)	Spacing (mm)	Load capacity (kPa)
Bearers	290x220	4250	4000	14.3
Joists	240x60	4000	400	26.9

Under the adopted loading conditions, the existing timber bearers and joists have sufficient capacity for the structure to be repurposed for residential use.

4 Seismic Capacity

The building structure will be designed to comply with current Australian Standards for Earthquake design.

This will involve the addition of new bracing elements (shear walls and bracing frames) installed in both the east/west directions and the north/south directions.

These bracing elements will align up the full height of the building and will be positioned in lift walls, stair walls and suitable inter-tenancy walls.

Loads will be transferred via floor diaphragms and via the external structural frame to these bracing locations.

The seismic design criteria are as follows:

<u>Seismic Design Criteria</u>

Structural Importance Level 2	(AS 1170.0 Table F1)
Annual probability of exceedance 1/500	(AS 1170.0 Table F2)
Probability factor, $k_p = 1$	(AS 1170.4 Table 3.1)
Hazard factor, Z = 0.11	(AS 1170.4 Table 3.2)
Site sub-soil class E_e	(AS 1170.4 Clause 4.2)
Earthquake design category = II	

Cavity wall ties will be inspected for condition and, where necessary, remedial wall ties will be installed.

Brick panel to concrete frame connections will be inspected and, where necessary, additional restraint ties will be installed.

Existing bearer to wall ties will be analysed and will be strengthened where required.

Additional joist to wall restraints will be installed where needed as per the recommendations of AS 3826.

5 Wind Load Capacity

The lateral load resistance elements incorporated for seismic loads will also cater for lateral wind loads.

The wind design criteria are as follows:

Wind Loading Criteria

Region A2 (AS 1170.2 Figure 3.1(A))

Structural Importance Level 2 (AS 1170.0 Table F1)

Terrain Category 3 (AS 1170.2 Clause 4.2.1)

Design life 50 years

Structure height = 18.5m

Annual probability of exceedance 1/500 (AS 1170.0 Table F2)

$V_{R.ULS}$ 45 ms^{-1}

$V_{R.SLS}$ 37 ms^{-1}

6 Fire Resistance

6.1 Timber Structure

The structure, defined by the NCC as a Class 2 building, is required to be of Type A construction which dictates a 90-minute fire resistance period (FRP) for structural adequacy. Structural adequacy means the ability of a structure to maintain its stability and loadbearing capacity under the load condition stipulated by AS 1170.0 Clause 4.2.4. The assessment of the fire resistance for structural adequacy of a timber member is based on the concept of a loss in timber section due to a notional charring of any wood surfaces exposed to a standard fire. This notional charring results in an effective residual section and strength capacity of this section determines the fire resistance for structural adequacy.

AS 1720.4 stipulates the required fire resistance for structural adequacy of timber members. Adopting a timber density of 800 kg/m^3, a notional charring rate of 0.52 mm/min was determined in accordance with AS 1720.4 Clause 2.4. For the stipulated FRP, this equates to an effective depth of charring of 54.5mm. This depth includes an allowance of 7.5mm for a layer of uncharred timber that has attained a high temperature, and is assumed as having no mechanical properties contributing to the strength of the residual section.

Load condition $G+\psi_l Q$

$\psi_l = 0.4$ (AS1170.0 Table 4.1)

6.1.1 Timber Columns

The reduced capacity of the timber columns, for a load duration of 5 hours as stipulated by AS 1720.4 Clause 2.2, is summarised in Table 4: Fire resistance of columns.

Table 4: Fire resistance of columns

Column	Effective residual section (mm)	Reduced capacity (kN)
Level 1	140x140	385
Level 2	115x115	150
Level 3	105x105	100

The fire resistance of typical timber columns in the structure is sufficient to maintain structural adequacy for a 90-minute FRP. Therefore, the installation of additional fire resisting measures for these columns is not required.

The effective residual section timber columns that are supporting the roof loads is insufficient to resist failure under the applied design load, $G+\psi_l Q$. Therefore, these columns will require some additional fire protection at each level, to ensure that the necessary FRP is met, or an alternative fire engineered solution will be required.

6.1.2 Timber Floors

Without additional fire protection, the effective residual sections of the timber bearers and timber joists are insufficient to maintain structural adequacy for the required FRP. Consequently fire protection will be needed or an alternative fire engineered solution will be required.

7 Footings

A geotechnical investigation has been carried out. This is referred to in more detail in the Geotechnical Engineering Report.

The general finding is that the building is supported on driven timber piles founded into underlying clay and sands which are primarily reliant on friction (ie friction piles).

Condition and capacity assessments on these piles are currently taking place.

If the piles prove to be adequate they will be re-used to carry the building in its new adaptively re-used condition. The loads applied by the new development will be significantly less than those imparted by the original wool store usage.

If the piles have insufficient capacity or insufficient long term reliability in performance, they will be enhanced by a new footing system, which is likely to also involve piles.

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