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Report

Middlemore Hospital – Galbraith Building

Independent Structural Review

Prepared for Counties Manukau District Health Board

Prepared by Beca Limited

26 April 2018

Revision History

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	Sheila Karimi **Hamish McCormick**	Independent Review	24 April 2018

Document Acceptance

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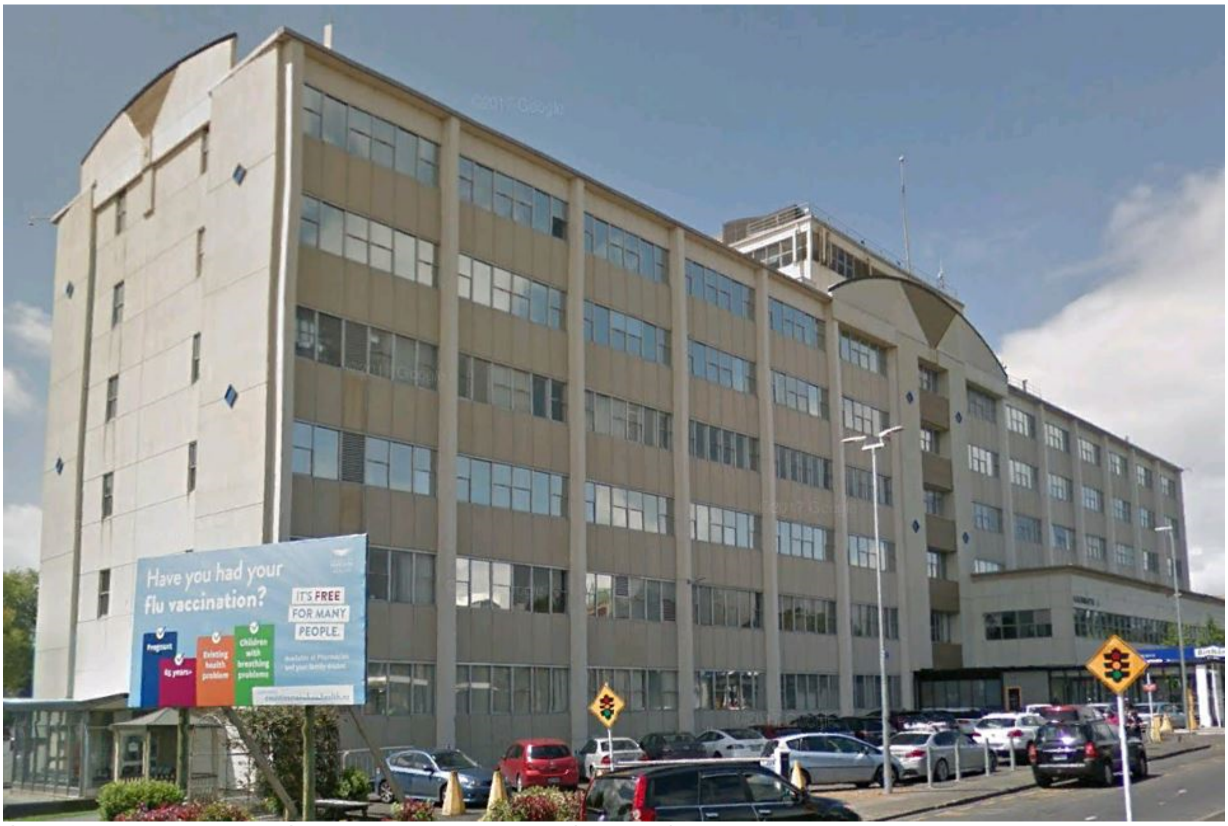
on behalf of	**Beca Limited**		

Executive Summary

Background

Beca Ltd (Beca) has been engaged by Counties Manukau District Health Board (CMDHB), to undertake an independent review of the recently completed Detailed Seismic Assessment undertaken by Holmes Consulting Ltd (Holmes) of the two structures comprising the Galbraith Building complex at Middlemore Hospital. Beca's independent review includes an independent Detailed Seismic Assessment (DSA) of the primary RC frames and infills.



Galbraith Block Buildings, Stage 1 (background) and Stage 2 (foreground)

The Galbraith Block at Middlemore Hospital was constructed in two stages early in the 1960's. Stage 1 and 2 buildings are seismically separated with Stage 1 designed and constructed first, at the Southern end of the site, and Stage 2 a few years later at the Northern end of the site. The two buildings are reinforced concrete frame buildings with reinforced concrete floor slabs which are supported on a raft foundation.

Summary of Beca Structural Review Findings

The result of Beca's independent review of the Galbraith Building indicates the building's earthquake rating to be 20*%NBS* (IL4) assessed in accordance with the guidelines document "*The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessment*" dated July 2017 (the Technical Guidelines). Therefore the building is a Grade D building in accordance with the Technical Guidelines grading scheme.

%NBS is defined as the ratio of the ultimate capacity of a building as a whole and the ULS shaking demand for a similar new building on the same site, expressed as a percentage.

- We have undertaken an independent Detailed Seismic Assessment for the primary frames, and assessed the overall score for the primary frames to be 20*%NBS* (IL4).
- We consider there are a number of localised structural elements (infill walls, stairs, Galbraith – Bray Link) that have unpredictable and undesirable structural behaviour in an earthquake. We have qualitatively assessed these as <34%NBS (and thus Grade D).
- Since the building rating is less than 34*%NBS*, we consider it is likely the Auckland City Council will determine the building's status as an Earthquake-prone Building upon receipt of this report and the Holmes Report.

By comparison, the Holmes Assessment of the Galbraith Building can be summarised as follows:

- Holmes rate the building as 20*%NBS* (IL4).
- Their report categorises the building as Grade D.
- Their report similarly identifies a range of poorly scoring structural elements within the building.
  - These include the primary lateral load resisting system (the reinforced concrete frames), and it particularly identifies the infill walls as having an adverse impact on the behaviour of these frames.
  - These also include a number of localised elements that are individually scored at <34*%NBS.*

We provide the following comparative commentary between the reports

- Following Beca's review of the building, we consider the Beca opinion and Holmes opinion on the building to be broadly in agreement.
- The Beca independent DSA intentionally followed a different methodology to the methodology adopted by Holmes. These different methodologies have resulted in similar fundamental conclusions and building *%NBS* ratings, but they give visibility to different aspects of the building behaviour. These differences have been discussed with Holmes and we are in general agreement as to the reasons behind these differences and are in general agreement that these differences do not have a substantive impact on the overall *%NBS* rating for the building.
- Our evaluation of the infill walls highlights the high uncertainty related to assessment of wall infills, and specifically the high uncertainty (and significant consequence) of the presence and/or extent of the gap around the infill walls. We therefore consider it more appropriate to provide a ranged score for the infill walls. We consider it most appropriate to give the *expected* range for these infill walls as 20*%NBS* (IL4) – 30*%NBS* (IL4). We note however that due to the high uncertainty, the actual performance of the walls may be outside this expected range. We have discussed this uncertainty with Holmes who concur with our views on the high levels of uncertainty pertaining to the assessment of wall infills.
- We highlight a key difference in the methodologies, in that Beca considers the structural issues with the primary reinforced concrete frames are likely to be more widespread than is readily apparent using Holmes' methodology. We note that this does not impact the overall *%NBS* rating but would impact the extent of possible strengthening. From our conversations with Holmes, we understand that they are broadly in agreement with our observations.

Details of our independent assessment and review is provided in the body of this report. Further details summarising the findings of the quantitative assessment of our review is appended to this report.

Summary of Other Post-Disaster Considerations

Beca has reviewed the Holmes commentary on Post-Disaster Considerations and are broadly in agreement with it, though we provide some additional commentary on it here. Specifically the Holmes Report discusses a number of other issues related to the Post-Disaster building behaviour. These can broadly be summarised as:

- Holmes has assessed the structure against the SLS2 (operational functionality) condition for the building, and concluded this is 25*%NBS.* We note this assessment was focused on structural elements.
- Holmes discusses the effects of non-structural elements, including ceilings, services and basement infrastructure and highlight the risks around these items.
- Holmes discusses the impacts of damage to, or collapse of, the Galbraith Building on the adjacent Acute Services Building.

We highlight that the accepted seismic assessment methodology for presenting the structural and life safety impacts of assessments is based on the Ultimate Limit State (ULS) and not on the SLS2 condition. This commentary relating to SLS2 behaviour and post-disaster considerations does not therefor impact the *%NBS* rating for the building, but is non-the-less an important consideration for a hospital.

Based on our experience of other post-disaster situations, we provide the following additional commentary.

Structural SLS Condition Behaviour

- Beca has undertaken a high-level review of the Holmes' assessment of the structure at the SLS2 state.
- In our opinion, the assessment of the structure in the SLS2 case as 25*%NBS* is likely to be somewhat conservative, however there are other, non-structural, aspects of the Galbraith Building that are likely to be of significant importance to CMDHB.

Utilities and Infrastructure:

- We agree with the Holmes assessment that utilities and infrastructure within the Galbraith building basement may sustain relatively little damage in any event short of full structural collapse.
- However, we highlight that the loss of maintenance access to these utilities post-disaster is significantly more likely, and, coupled with relatively minor damage, (for example, loss of supports to non-structural elements) may lead to more widespread disruption for the campus.
- We note a similar situation exists for utilities feeding the Acute Services Building from plantrooms within the Galbraith Building.

Acute Services Building:

- We agree with the Holmes assessment, the Galbraith building is unlikely to be able to cause collapse of the adjacent Acute Services Building.
- We highlight that from our experience, cordons or similar access restrictions that are likely to result in the case of significant damage to Galbraith can result in significant operational disruption to nearby buildings and recommend this be actively planned for.

Non-Structural Elements:

- We note that Holmes have not included non-structural elements (ceilings, services, etc.) in the assessment of the building *%NBS* rating, and agree this is in accordance with the Seismic Assessment Guidelines.

- We highlight the historic deficiencies with restraint of non-structural elements and the significant disruption to continuing operations that these can have of facilities (which does not form part of a %NBS rating). We recommend further evaluation into this is undertaken.

Beca considers that access to utilities and infrastructure, impacts of potential cordons, and damage to non-structural elements are aspects of the post-disaster functionality that may have a notable impact on hospital operations and recommend further investigation into the likely extent of these impacts, along with possible mitigation, is undertaken.

Commentary on Retrofit Options

Beca has reviewed the range of possible retrofit and strengthening options presented by Holmes in their report. We are broadly in agreement that these retrofitting options represent appropriate types of retrofitting techniques for the building, noting that there is still significant work required to develop a full strengthening scheme. We provide the following summary comments:

Localised strengthening to unpredictable structural elements (infill walls, stairs, Galbraith-Bray link etc.)

- Beca broadly agrees with the sort of local strengthening approaches set out by Holmes.
- Due to the unpredictable nature of these items, Beca consider retrofitting of all these elements will be necessary for the building.

Global reinforced concrete frame strengthening

- Beca broadly agrees with the sort of local strengthening approaches set out by Holmes.
- We consider the structural issues with the primary reinforced concrete frames are likely to be more widespread than is readily apparent using the Holmes methodology. We expect that the extent of this strengthening should be more widespread than that suggested in the Holmes report.

Down rating of the building from IL4 to IL3

- Beca agrees that this could be a pragmatic way to move many parts of the structure above 34*%NBS*.
- We highlight the disruptive impact variable importance levels can have on an interconnected hospital campus. The interconnectedness of access, egress, utilities, and infrastructure can mean that damage sustained to lower importance level buildings can adversely impact on nearby IL4 facilities.
- We note that even with a down rating to IL3, some strengthening works (especially the localised works) would still likely be required, and that the building function may likely need to be changed.

Next Steps

We recommend that CMDHB undertake the steps set out by Holmes Consulting in their report with regard to potentially earthquake prone buildings, including being aware of their legal obligation in that regard.

A determination will need to be made on the approach for the Galbraith Building, broadly being:

- Down-rating its importance level (and likely changing its function)
- Undertaking strengthening
- Replacement
- And/or a combination of the above

The determination of the approach for the Galbraith Building will need to be considered in light of the wider campus. Particular consideration should be given to the inter-relationship of the Galbraith Building to the rest of the campus, specifically the impact of utilities and plant within Galbraith on other facilities, and the impact of restricted access or cordons in a post-disaster scenario should be evaluated.

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Appendices

Introduction

Beca Ltd (Beca) has been engaged by Counties Manukau District Health Board (CMDHB), to undertake an independent review of the recently completed Detailed Seismic Assessment undertaken by Holmes Consulting Ltd (Holmes) of the two structures comprising the Galbraith Building complex at Middlemore Hospital. Beca's independent review includes an independent Detailed Seismic Assessment (DSA) of the primary RC frames and infills.

The Galbraith Building complex is on the Northwest corner of the Middlemore campus and consists of two primary structures and a number of smaller structural parts. The general arrangement is shown below:

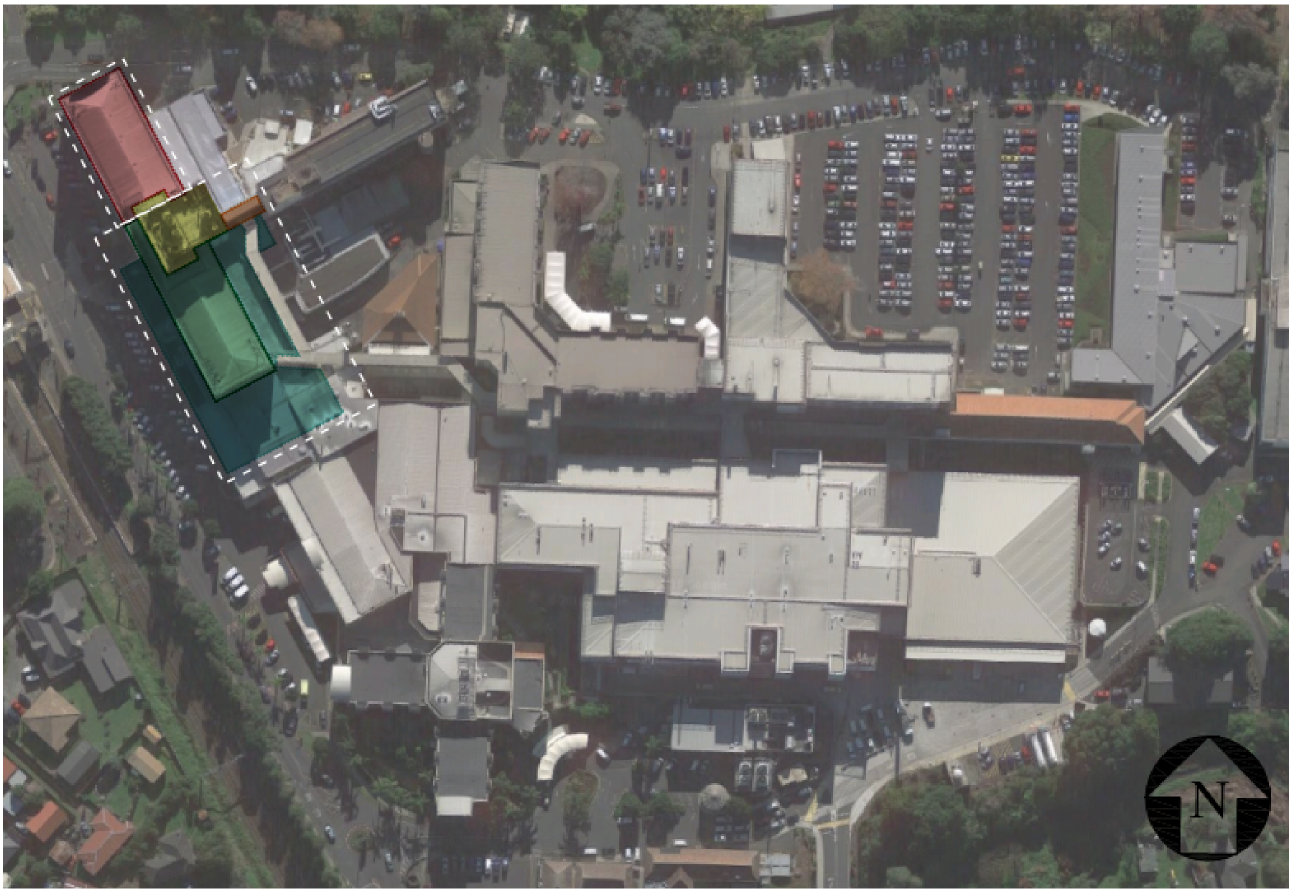


Figure 1.1 – Showing Galbraith Building complex. Stage 1 is in green to the south. Stage 2 is in red to the north. The level 7 plant room (part of Stage 1) is in yellow. The Galbraith-Bray link bridge is in orange.

This report summarises the findings of this independent review.

1.1 Scope of Assessment

The primary focus of the review is on the major building elements impacting on global building behaviour. Our review is split into two portions:

- An independent Detailed Seismic Assessment of the building for significant structural elements (the primary moment frames which provide overall stability to the buildings, and the infill walls which cause a considerable modification to their behaviour, and
- Reviews of the Holmes assessment of more localised building elements.

For the major building elements, our independent assessment uses a different analytical approach than the Non-Linear Time History Analysis (NLTHA) used by Holmes Consulting. We consider using an independent and different analytical assessment approach provides the DHB with increased confidence as to the seismic rating of the building complex and the issues associated with it.

Our assessment was undertaken in accordance with the recently published “Seismic Assessment of Existing Buildings Technical Guidelines for Engineering Assessments issued July 2017 by MBIE (Engineering Assessment Guidelines). We understand that Holmes also followed these Guidelines, though used a different analysis technique within them.

1.2 Assessment Methodology

The techniques used in undertaking the independent assessment of the primary structure are generally as outlined in the guideline document *The Seismic Assessment of Existing Buildings - Technical Guidelines for Engineering Assessments*, dated July 2017 *(*the *Engineering Assessment Guidelines).*

Our methodology is briefly summarised below, which generally follows the key steps of the Simple Lateral Mechanism Analysis (SLaMA) technique described in Appendix 2A of the *Engineering Assessment Guidelines*:

- Review of the available structural drawings to identify the main structural elements (primary and secondary). A near full set of structural drawings was available for both Stage 1 and Stage 2. A limited number of architectural drawings were also made available.
- Visual inspection of key elements of the building including general arrangement, building modifications, stairs and relationship to adjacent buildings, including identification of non-structural elements that may present a significant life-safety hazard. A site visit was undertaken by Beca staff to the site.
- Meet with Holmes Consulting to gather and gain an understanding of the information Holmes Consulting Ltd have collected about the Galbraith Building complex.
- A review of the geotechnical reports available for the site.
- Selection of appropriate member properties and determination of structural element probable capacities. Probable material strengths were taken from the appropriate chapter of the Engineering Assessment Guidelines.
- Calculation of the expected seismic actions on the building following the current New Zealand loading standards (NZS 1170) for an equivalent new building.
- Hand calculations of selected key elements of the building to determine the probable capacities and failure mechanisms of these subassemblies and the whole building. Frames were simplified where possible. At each joint in a given frame direction, the failure hierarchy was identified by comparing the strengths of the elements around the joint.
- Based on the detected failure mechanism from the previous step, it was identified that limited to no ductility could reasonably be achieved by the structural system, thus an elastic 3D analysis of the structure was undertaken. This analysis provided scores for the dominant brittle mechanisms.
- Determination of the likely earthquake rating (*%NBS*) based on the 3D analysis of the structure. Elastic member demands were extracted and post-processed to provide scores for the various non-ductile mechanisms. Where required, engineering judgement was applied to provide a ‘global’ score rather than conservatively reporting on an individual element.
- Meet with Holmes Consulting to compare and discuss our results compared with theirs with the aim of identifying any differences and gaining an understanding of why these may have occurred.

For the review of other elements, we have generally undertaken quantitative assessment. For some elements where uncertainty of behaviour is high (such as the stairs), and where we consider attempts to quantify would be prone to inaccuracy, we have undertaken a qualitative review and provide appropriate commentary. Similarly, for elements where the consequences of failure are minimal we have provided a qualitative review of the Holmes Consulting assessment.

2 Building Description

The Galbraith Block at Middlemore Hospital is a reinforced concrete frame building constructed in the early 1960's. The Galbraith Block actually consists of two separate buildings, Stage 1 was designed and constructed first, at the Southern end of the site, and Stage 2 a few years later at the Northern end of the site. The Galbraith Block is currently being used for maternity and birthing, gynaecology radiology, with some general office and file storage.

The building is described in further detail in the Holmes Consulting report and that information is therefore not replicated here.

We make the following comments regarding the Holmes interpretation of the Structural System.

- Beca are in general agreement with Holmes' assessment of the structural system as a two way moment frame.
- We note that the infill walls impact the seismic behaviour of the frames. The infill has an impact on the frames which is relatively significant, but difficult to quantify. There are some differences between how Beca and Holmes have assessed these infill walls, which is discussed in further detail later.
- There are some areas of uncertainty about how the moment frames interact with the ground. These include:
  - There is some uncertainty about whether the floor slab between Stage 1 and Stage 2 is seismically separated at ground level. Both Beca and Holmes have assumed it is separated based on the information available, though this could not be confirmed on site. Should the buildings not be separated at ground level, the impacts on the building *%NBS* rating would likely be modest.
  - We have undertaken our analysis considering that lowest structural level of Stage 1 is the ground floor (due to relatively extensive basement walls and ground buttressing) and the lowest structural level of Stage 2 is the basement floor (due to minimal basement walls and ground buttressing).
  - We note this is different to Holmes' assumption that the lowest structural level of Stage 1 is at basement level (with some soil interaction at ground level). Following discussion, both Beca and Holmes were in general agreement that the structure below ground level in Stage 1 is not critical to building behaviour and we do not consider this difference in analysis likely to make an appreciable difference to the Building *%NBS* rating.

Beca undertook a site visit on 22 February 2018 accompanied by CMDHB personnel. This site visit included Stages 1 and 2, basement levels, typical levels, roof, upper plant rooms, and linking structure. We subsequently met with Holmes to discuss the findings of our site visit and to compare understanding of the building. From this meeting we concluded that Beca's overall understanding of the structural system was comparable to Holmes understanding.

From Beca's review of the available information on the adjacent structures and associated secondary structural elements, and subsequent discussion on these with Holmes, our understanding of these is as follows:

- The Galbraith-Bray Link is an independent structure (albeit buttressed by Galbraith Stage 1)
- The Level 2 Maternity link is a later addition, contributing mass, but not strength, to Galbraith Stage 1.
- The stairs are largely built-in, and have been assessed qualitatively.

3 Independent Detailed Seismic Assessment of Primary Structure

3.1 Overview of Review Findings

As part of this review, Beca has undertaken an independent Detailed Seismic Assessment of the primary structural elements using an alternative methodology, most notably the primary structural frames and the infill walls that affect the behaviour of these frames.

For less critical structural elements we have undertaken a simplified quantitative review to enable comparison to the Holmes assessment, or where appropriate undertaken a qualitative review of the Holmes approach.

3.1.1 Understanding the Beca Independent Assessment compared to the Holmes Assessment

It is important to note that due to the different assessment methodologies used, the *%NBS* determined by Beca are in some cases different from that given by Holmes. This is reflective of the inherent uncertainty in assessment of existing buildings.

Beca has discussed with Holmes the areas where the assessments provide different *%NBS*, with a view to gaining a joint understanding of where the assessments differ.

Areas where the assessments are notably different can be seen as reflective of areas of high uncertainty, and thus of potential risk of unpredictable behaviour. This is particularly true of the infill panels which are discussed further below.

The Beca assessment approach is:

- Understanding the elemental capacities.
- From there, establishing a likely failure hierarchy and consequence of failure.
- Converting this into an understanding of overall system behaviour.

The different approaches used by Beca and Holmes give different insights in to building behaviour, and we consider that a greater understanding of likely building behaviour is gained from having both approaches available.

For the purposes of aiding understanding of the two documents, we provide some examples of how the various results could be interpreted:

Results with similar *%NBS*

- Example: Columns supporting Level 7 plant. Beca have scored this element at 25*%NBS* (IL4), while Holmes have scored this 20*%NBS* (IL4).

In examples such as this, the relatively small difference between 20*%NBS* and 25*%NBS* can generally be accounted to the under certainty inherent in assessing historic structures. Both of these score lie within the same banding (‘Grade D – High Risk’). This small variability in *%NBS* should be seen as aligned scores, and the Beca review score can be seen as generally in agreement with the Holmes score.

Results with different *%NBS*

- Example: Infill walls to the Stage 2 building. Beca and Holmes are both in agreement and have scored the out-of-plane capacity of the infills to 20*%NBS* (IL4). However, Beca have scored the effect of the infill walls inplane on the reinforced concrete columns as 20% - 40*%NBS* (IL4), while Holmes have scored this 50*%NBS* (IL4).

In examples such as this, the relatively significant difference is generally due to significant uncertainty in the available knowledge of the element. In some cases it may be because the method of construction is unknown (and possibly not able to be adequately determined even with further investigative works), or it may be that engineering assessment of that element only allows an inexact estimation of the element. The Engineering Assessment Guidelines note the difficulty in assessing infills due to the various geometries that the compression strut in the panels can form. This is further complicated with the presence of openings within the infills that can form complex load paths, which are not easily assessed.

Generally speaking in these situations Holmes have chosen to adopt a particular value for the *%NBS*, whereas Beca has chosen to convey the uncertainty with a range.

These sort of results can generally be attributable to the high uncertainty associated with assessment of any given element. Given that unpredictability is seen as undesirable for structural systems, it is often still possible to provide a clear recommendations in these instances.

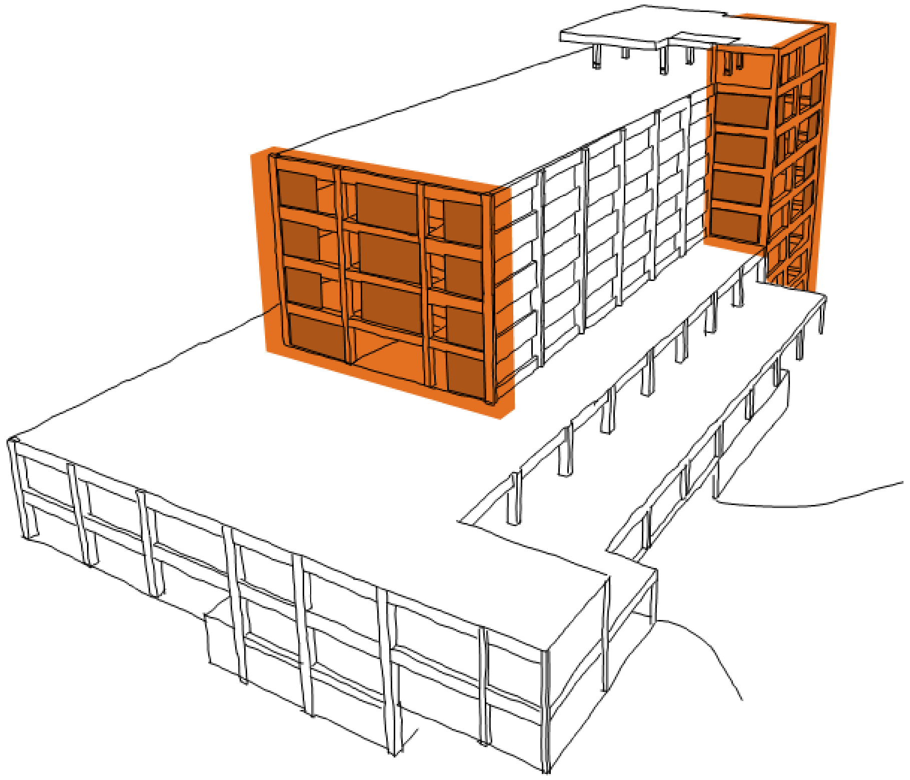
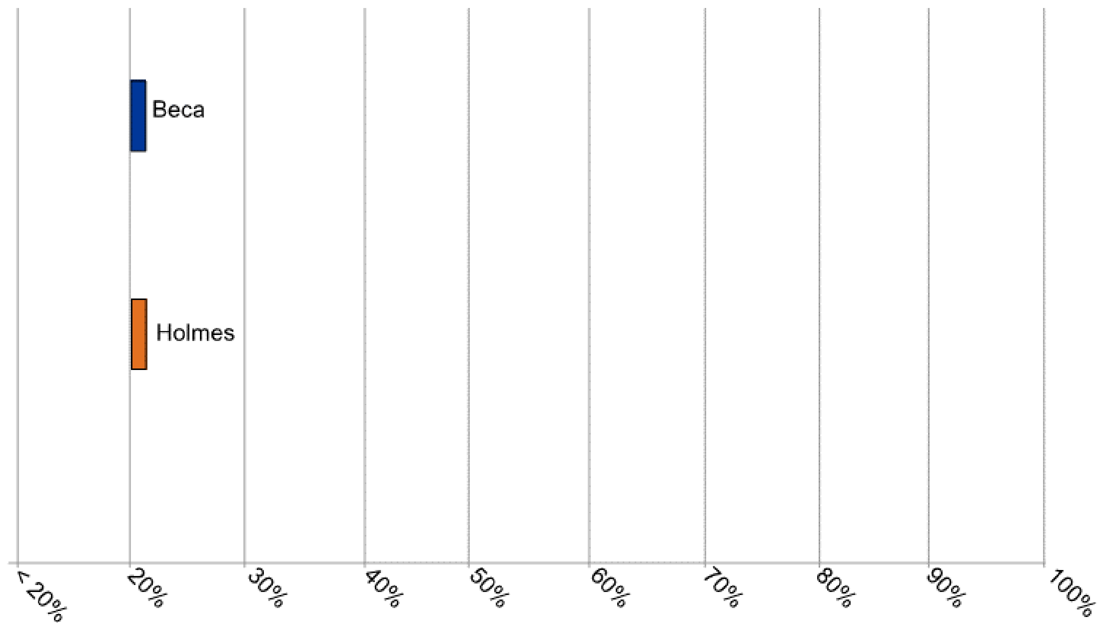
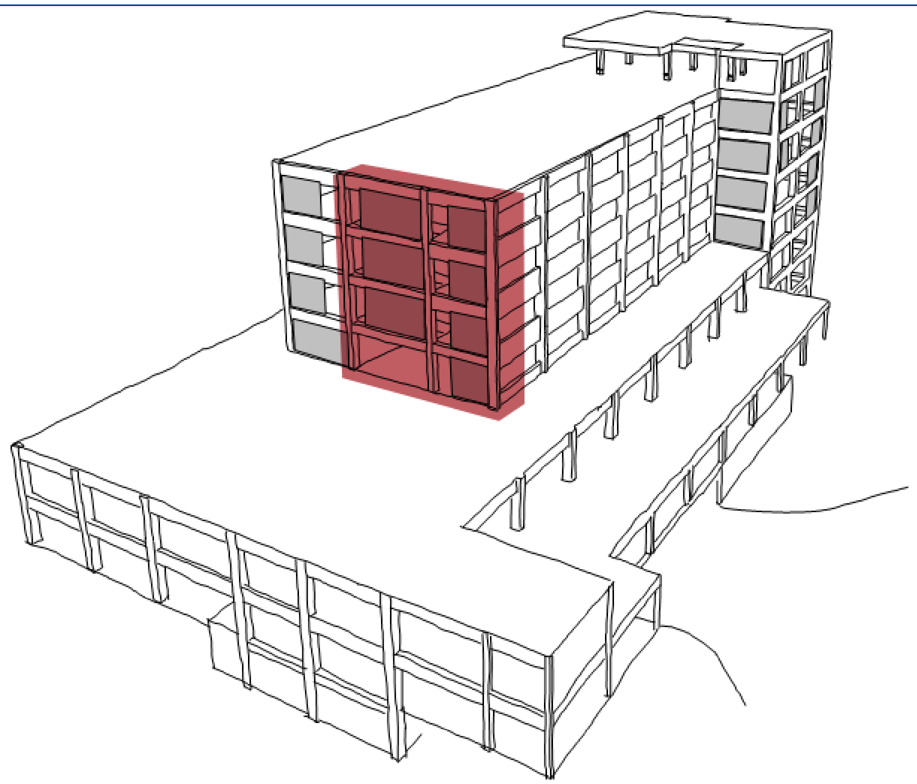
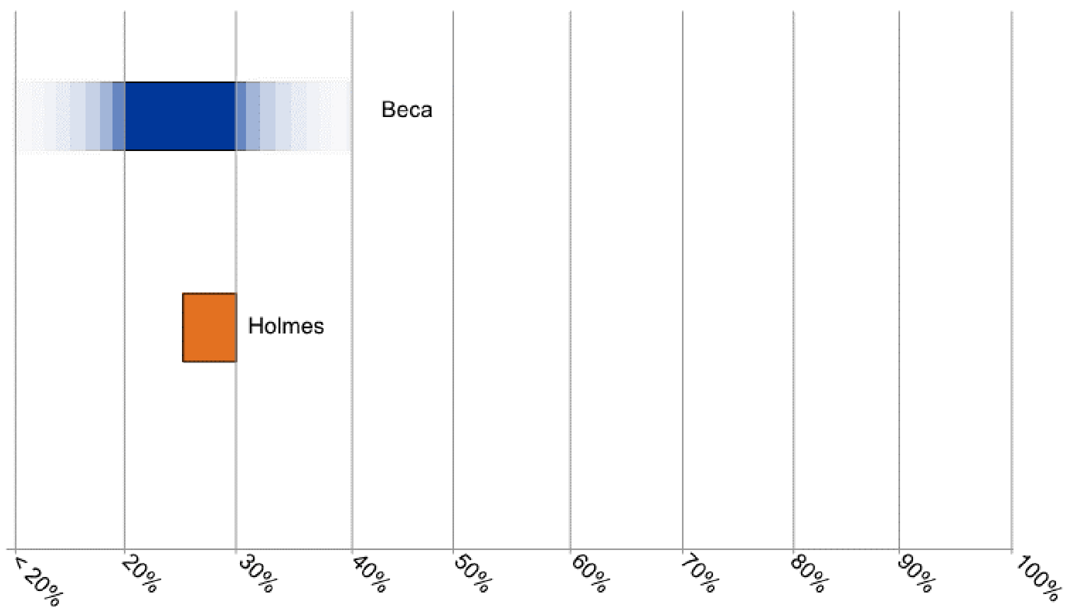
3.1.2 Seismic Risk

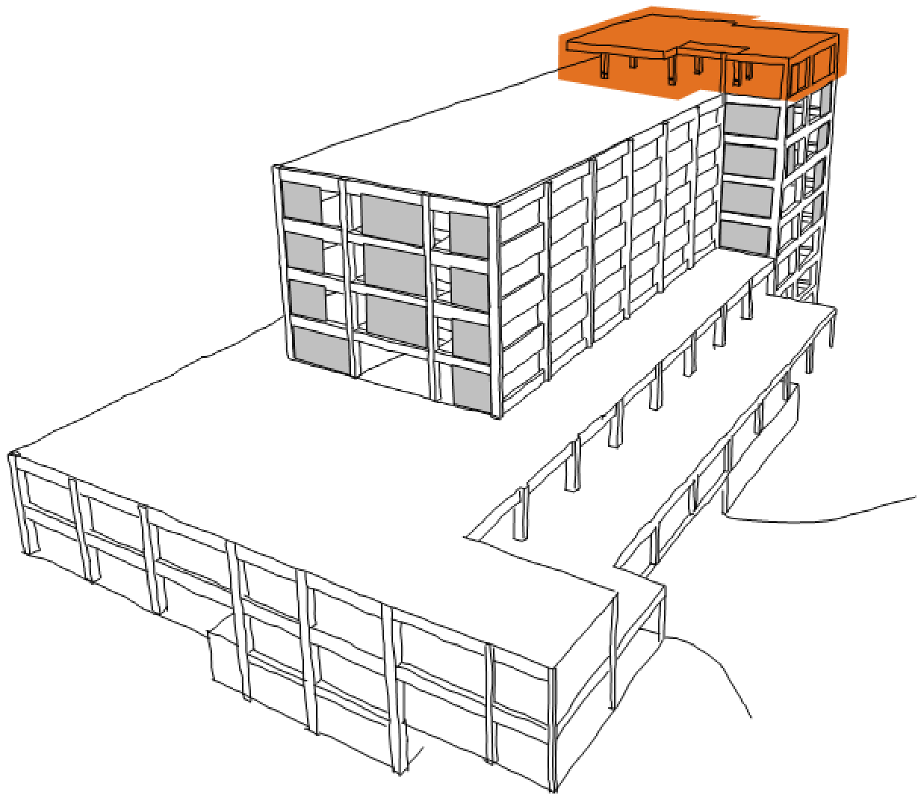
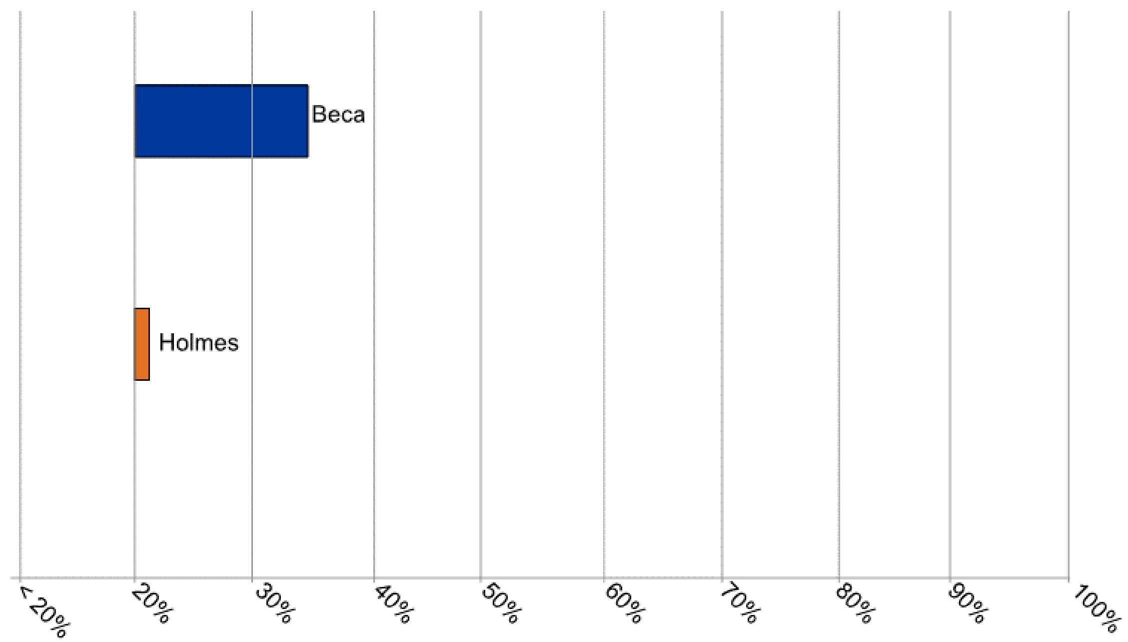
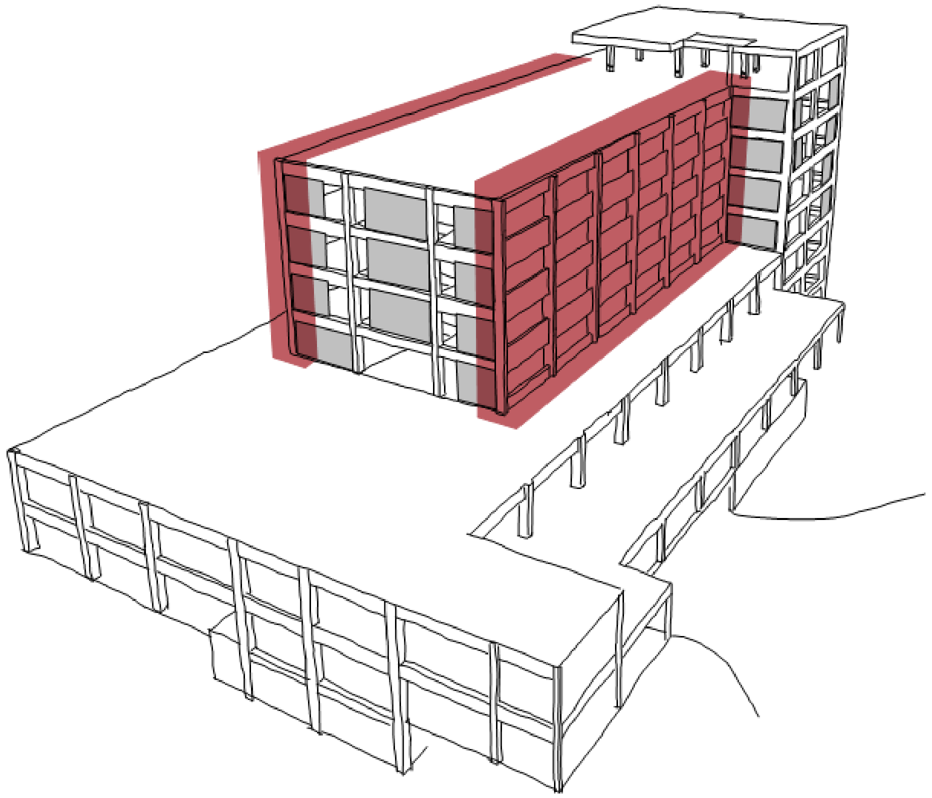
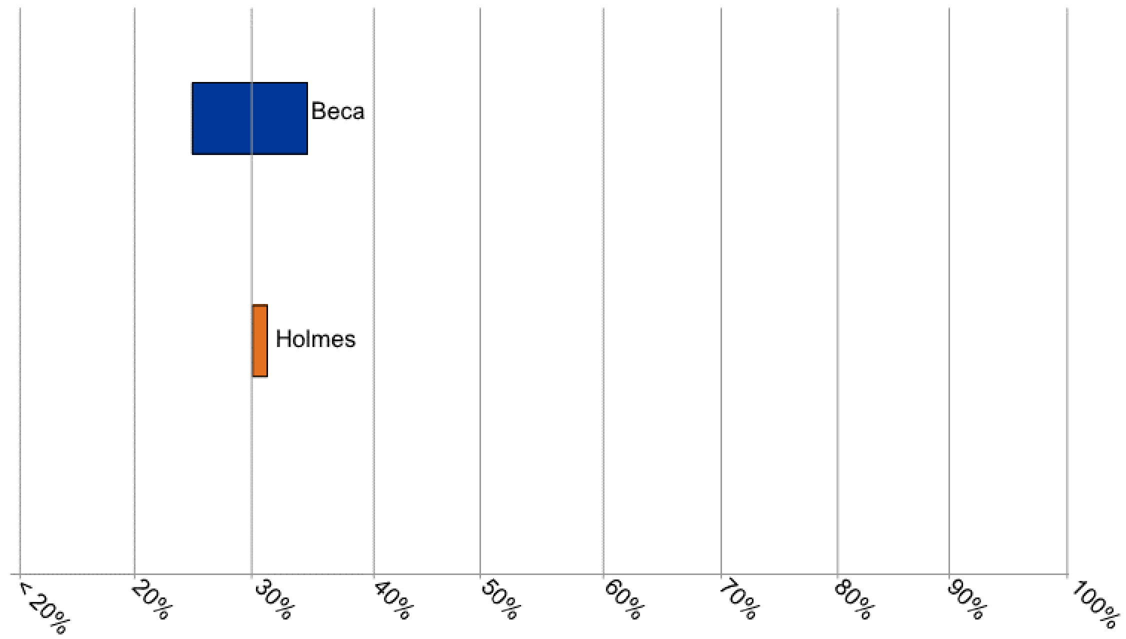
The New Building Standard requires an IL4 building to have a low probability of collapse in a 1 in 2500-year “design level” earthquake (i.e. an earthquake with a probability of exceedance of approximately 2% over the assumed 50 year design life of a building). These grades and *%NBS* are referred to often in the following sections, so are shown here for convenience. A fuller explanation of seismic risk, *%NBS*, and the legislative environment related to earthquake prone buildings is giving in the Holmes Report, so is not further repeated here.

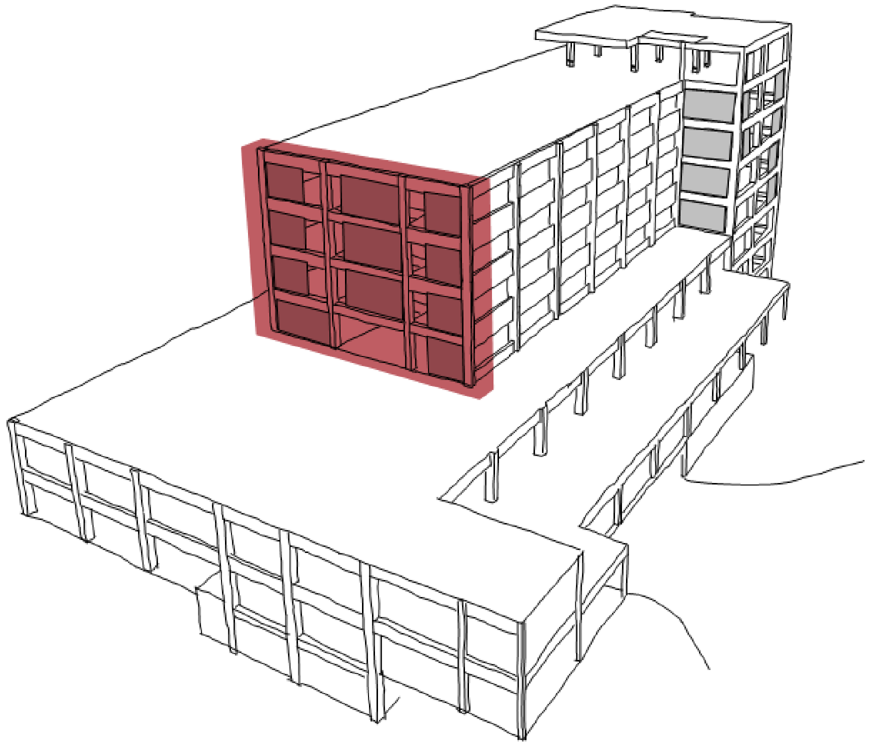
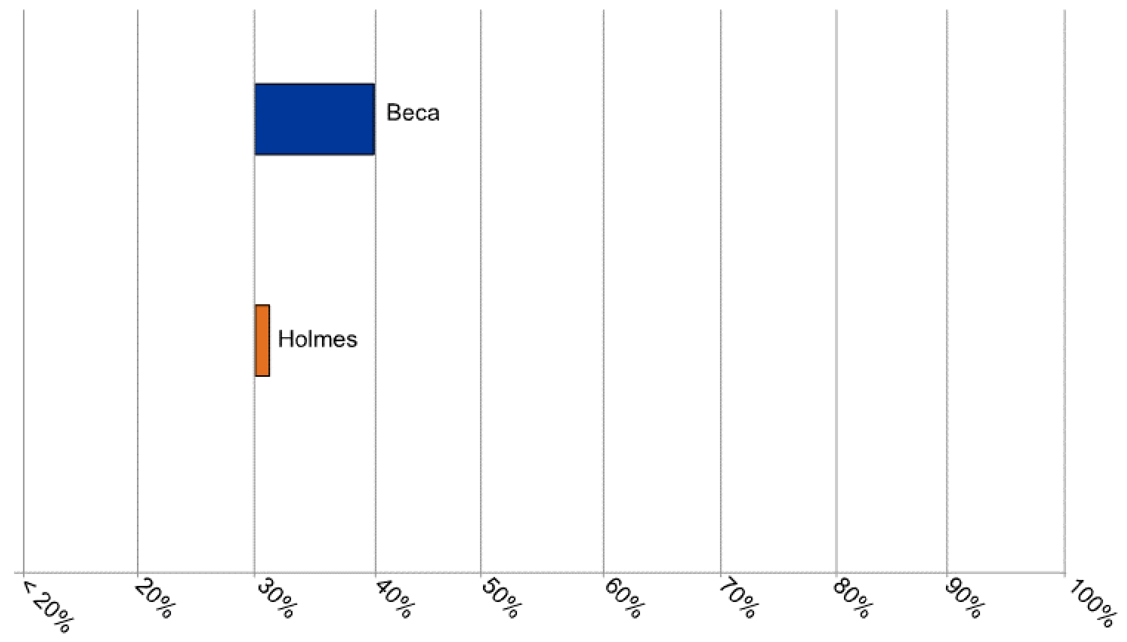
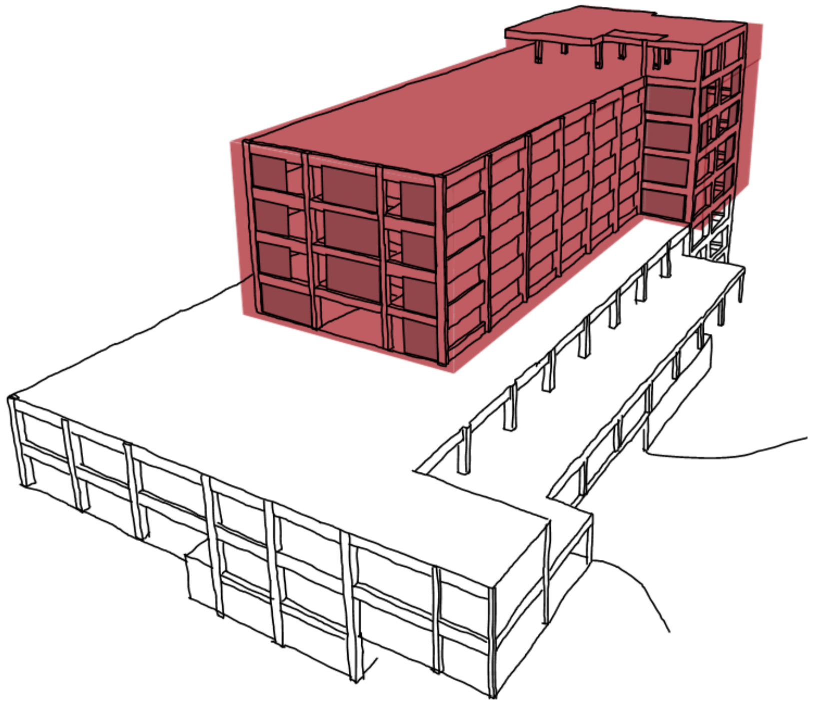
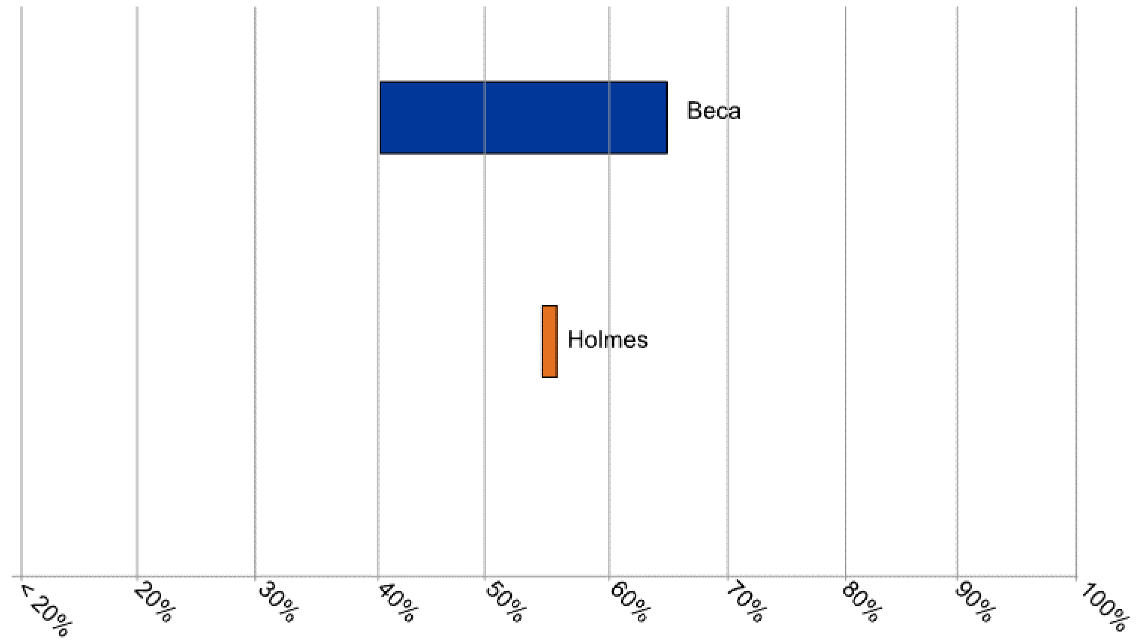
Table 4: Relative Earthquake Risk

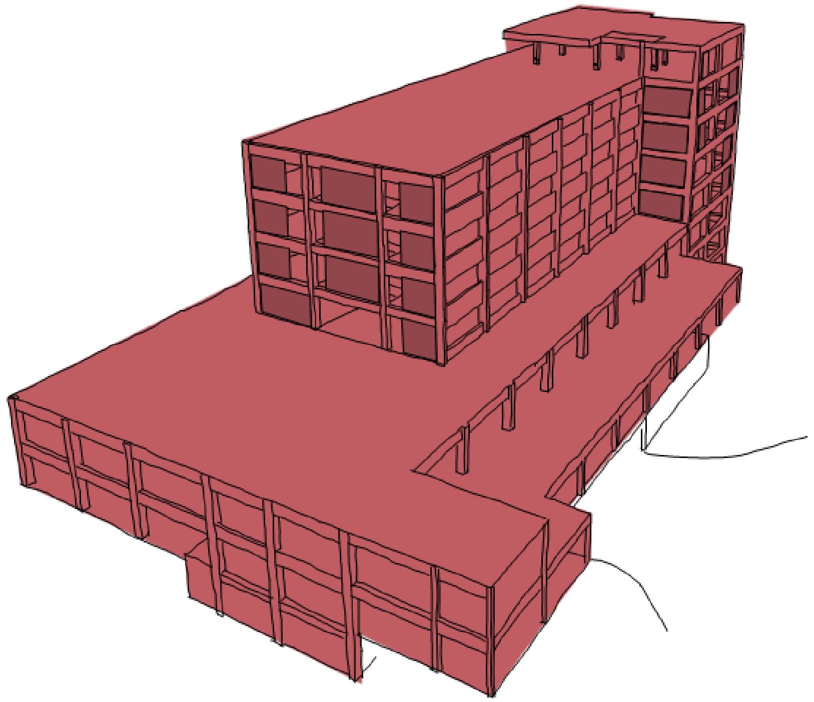
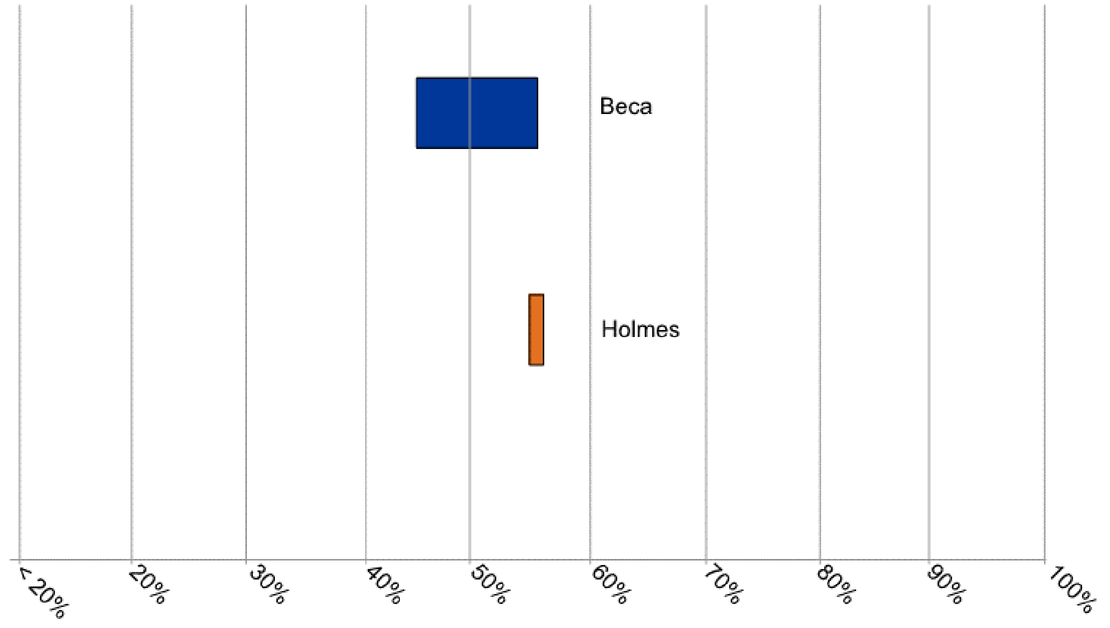
Building Grade	Percentage of New Building Strength (*%NBS*)	Approx. Risk Relative to a New Building	Risk Description
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 80	2 to 5 times	low to medium risk
C	33 to 67	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

3.2 Summary of Stage 1 Behaviour

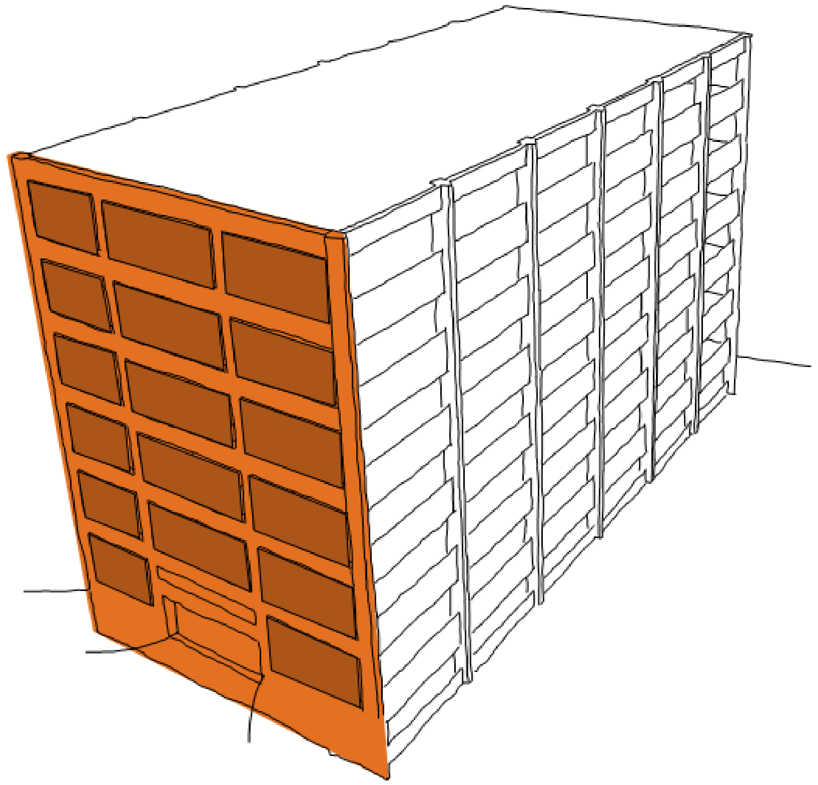
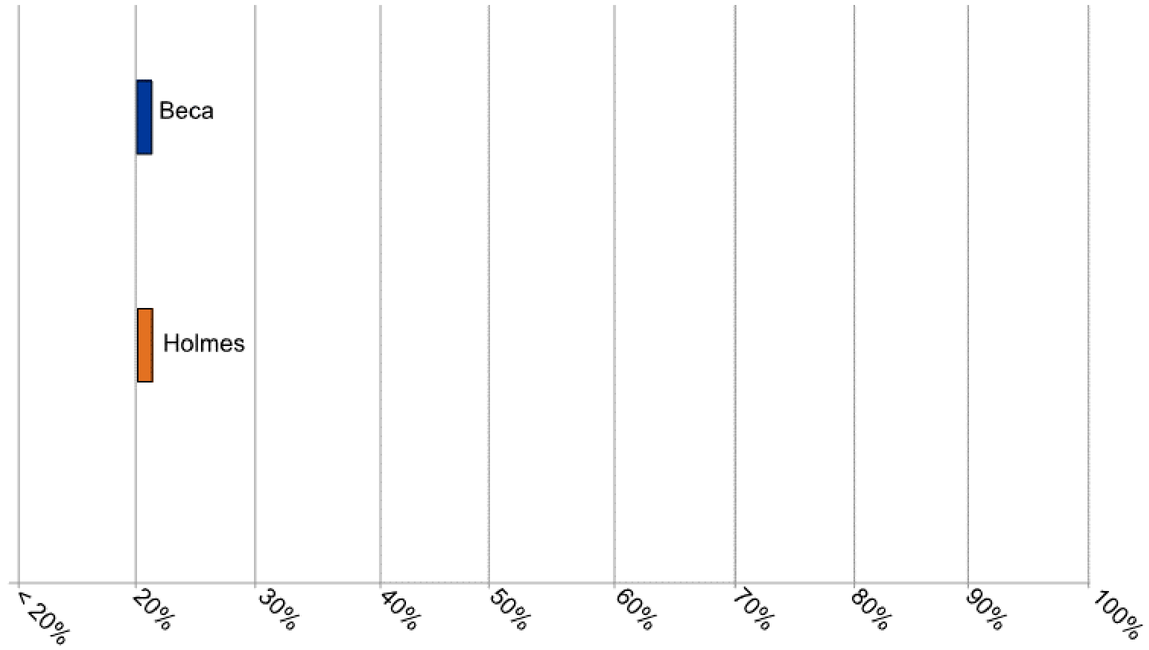
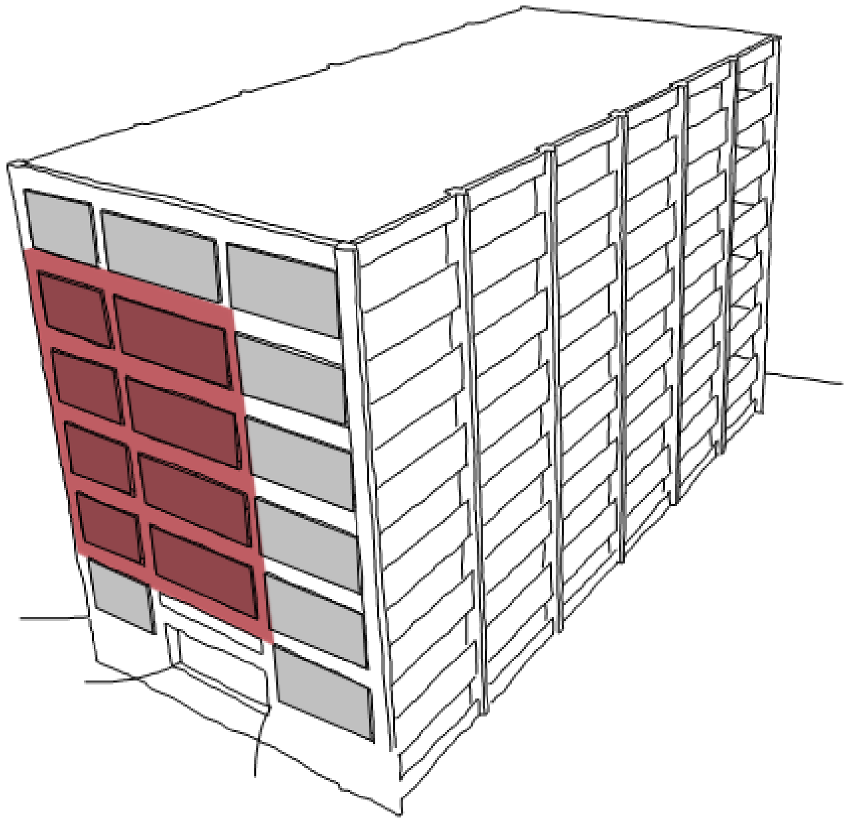
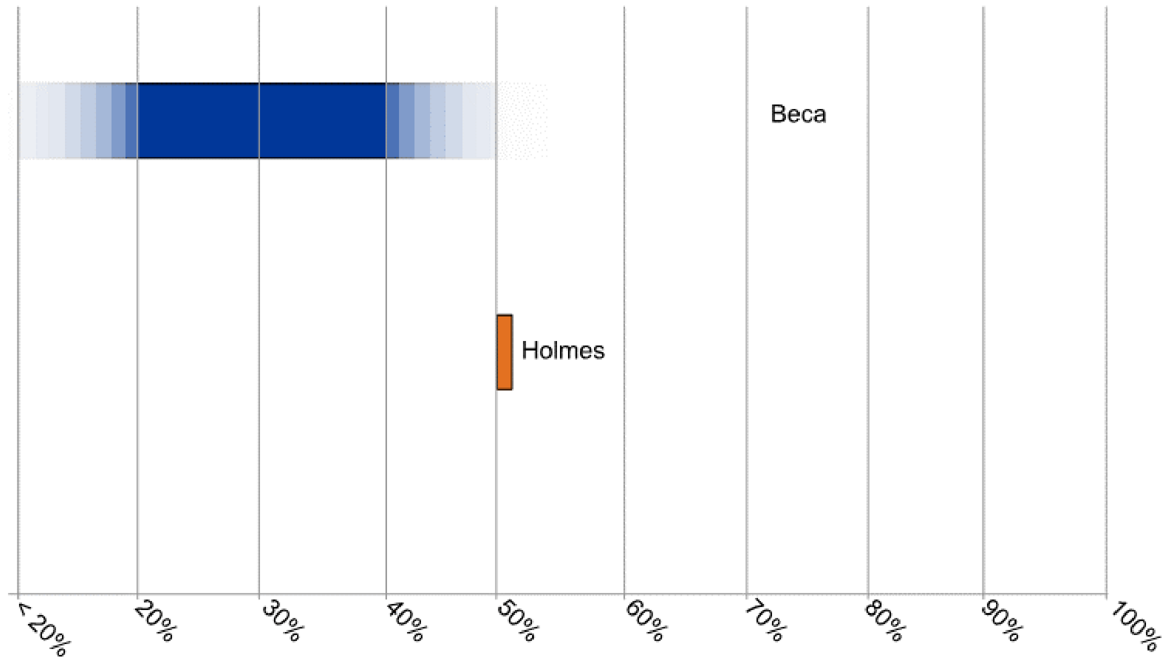
Element	Location	Consequence	
Infills walls out-of-plane.		Damage to infill panels and risk of collapse to the ground or onto structure below due to lack of restraint of the panels. Damage to structure below, particularly the Level 2 plant room. Life safety risk to those outside the structure. **20%*NBS* (IL4)**	 This infill behaviour is governed by out-of-plane actions, hence the higher certainty than the infill in-plane interaction with the frames.
Infill walls causing column shear failure		Damage to frame due to infill panel interaction with frame, causing local shear failure to the columns. Occurs to south elevation of tower and around the northern stair and lift core area. Possible damage to nearby structure. Life safety risk to those inside the structure. **<20% – 30%*NBS* (IL4)**	 Differences in score due to modelling of infills and assumptions around behaviour, however we are both in conclusion that the infill panels interacting with the structure impacts significantly on the structural performance of Stage 1. Spread of scores covers both uncertainty in the gap present between the infills and the bounding frame, as well as critical scores from different locations in the structure.

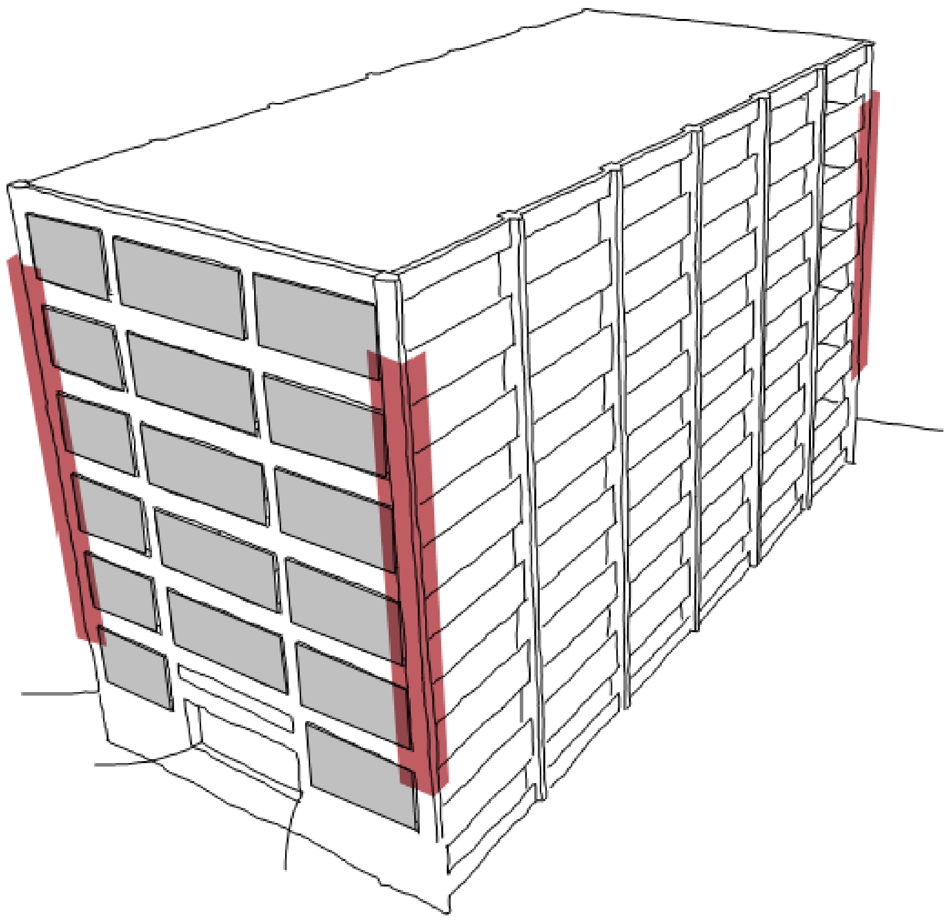
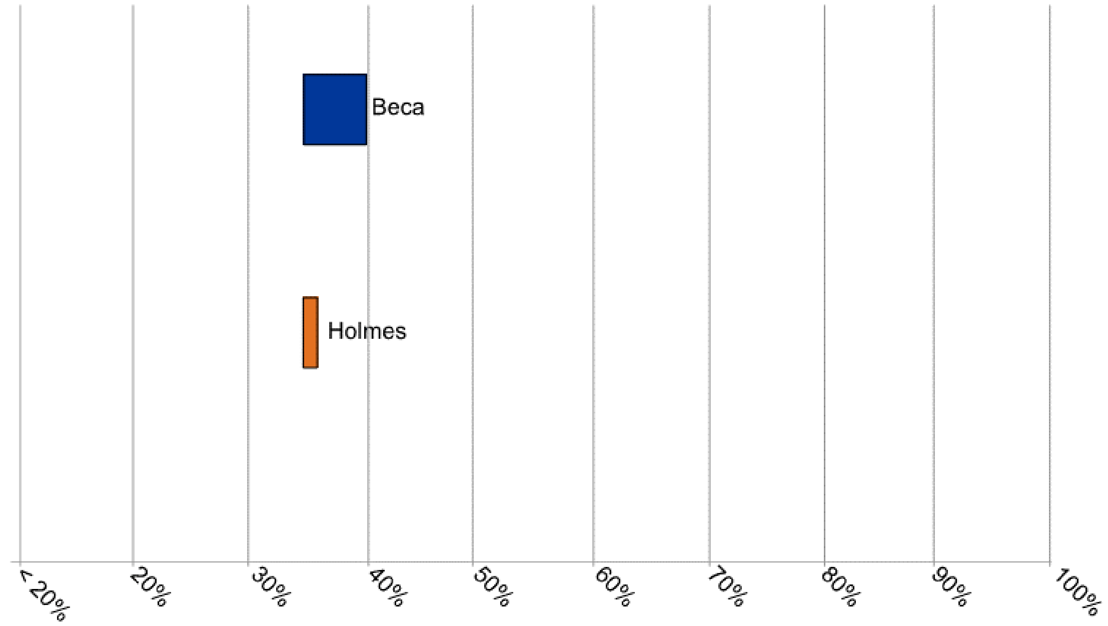
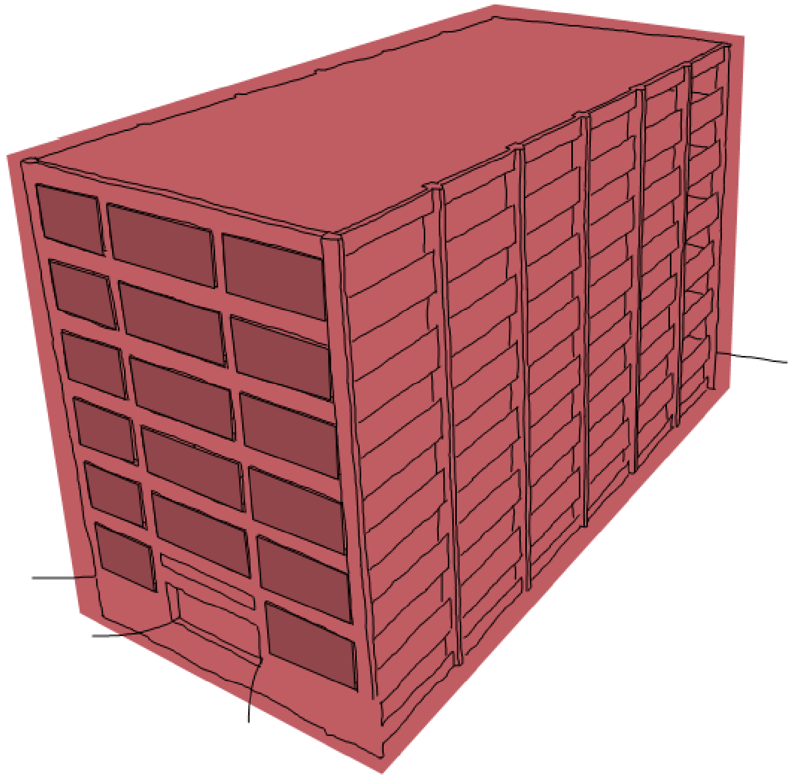
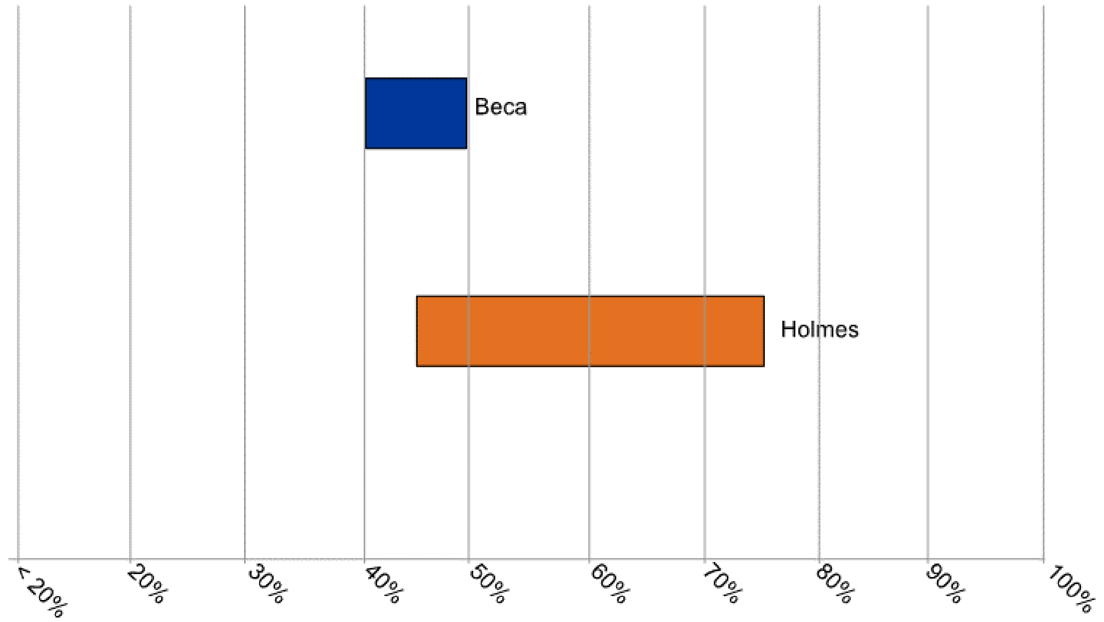
Element	Location	Consequence	
Level 7 Plant Room		Flexure-shear failure in columns below, and beam-column joint shear failure at Level 7, leading to potentially widespread damage to plantroom structure. Possible damage to structure below. Life safety risk to those inside and potentially outside the structure. **20% - 34%*NBS* (IL4)**	 Both Beca and Holmes are in agreement that the columns supporting the Level 7 plant room have insufficient lateral capacity, however the exact mechanisms responsible for failure vary slightly. Beca presents a range of scores to indicate the level at which initial failure of the column occurs, and when a sufficient number of columns and joints are damaged for there to be a high risk of collapse.
External longitudinal frames to tower		Flexure-shear failure in perimeter columns due to interaction with half-height infill panels, with damage concentrated to Levels 3-5. Potential widespread damage to perimeter frames. Possible damage to structure below. Life safety risk to those inside and potentially outside the structure. **25% - 34%*NBS* (IL4)**	 Both are in agreement that the half-height concrete panels are detrimental to the tower frames performance. Beca presents a range of scores to cover the initial column element failure, to when there are sufficient columns to constitute a high risk of multiple column failure in the frame line.

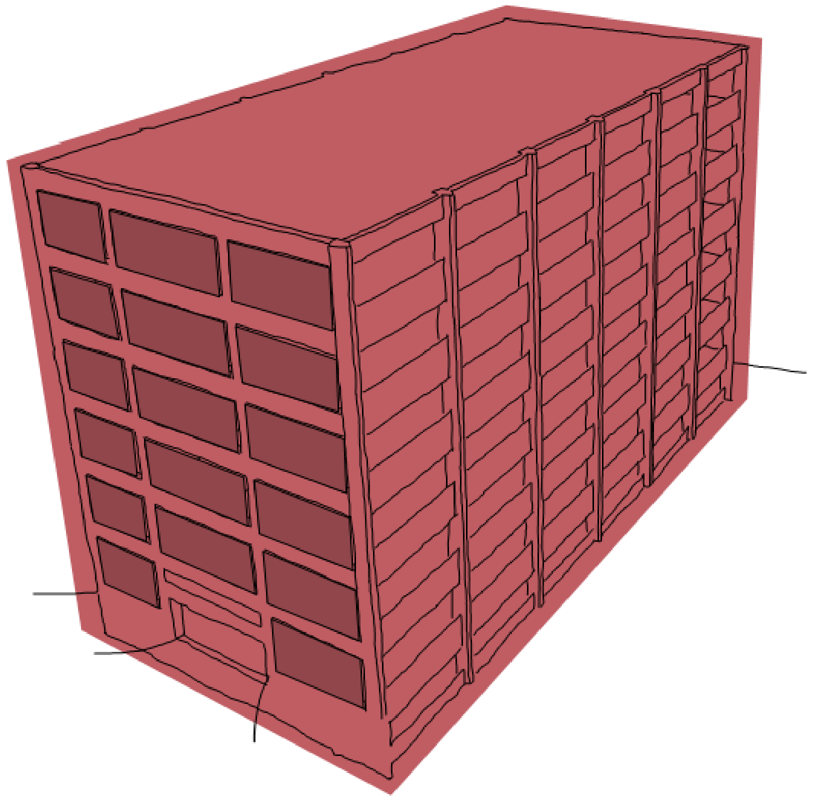
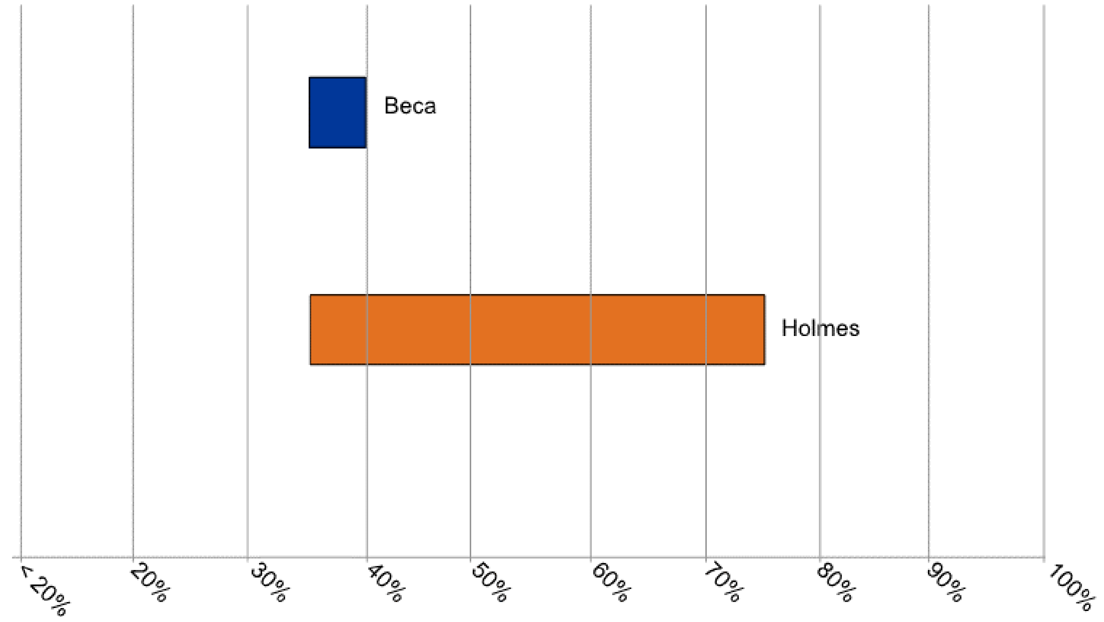
Element	Location	Consequence	
Transverse end frame to south elevation		Beam column joint failure of frames and some shear failure of beams, leading to potential widespread damage to end frames. Possible damage to structure below, notably the Level 2 plant room. Life safety risk to those inside the structure. **30% – 40%*NBS* (IL4)**	 Both are in agreement that the transverse end frame to the south elevation has insufficient lateral capacity, however the expected failure mechanisms and scores along this frame line differ. Beca expects there to be beam-column joint shear failure, critical at Level 3, leading to the failure of the frame line between 30%-40%NBS *(IL4)*. Holmes has found the columns failing in shear along this gridline. During discussions with Holmes, they noted that this area was sensitive to the type of analysis undertaken.
Transverse and longitudinal direction tower frame failure		Failure is expected to be a variety of brittle mechanisms, but dominated primarily by beam-column joint failure and column flexure shear mechanisms. Widespread failures throughout tower structure. Significant life safety risk to those inside and outside the structure. **40% – 65%*NBS* (IL4)**	 Both are in agreement that the tower structure as a system sits somewhere between 34%NBS *(IL4)* and 67%NBS *(IL4)*, however the expected mechanisms differ. Difference in presented scores and expected mechanisms due to analysis type and ability to report on a singular figure. Beca has found the response is dominated by the transverse frames, with beam column joint shear failure dominant at some levels, and column flexure-shear at others. Scores are presented based on a spread from the lowest scoring internal frames, to the highest. The overall structural mechanism is expected to fall within these values.

Element	Location	Consequence	
Transverse and longitudinal direction podium frame failure		Failure is a variety of mechanisms, but dominated by column flexure-shear failure. Scores are governed by the western podium level columns with partial height infill. Short columns cause premature shear failure, leading to widespread failures throughout whole structure. Significant life safety risk to those inside and outside the structure. **45% - 55%*NBS* (IL4)**	

3.3 Summary of Stage 2 Behaviour

Element	Location	Consequence	Estimated %NBS
Infills walls out-of-plane.		Damage to infill panels and risk of collapse to the ground or onto structure below due to lack of restraint of the panels. Damage to structure below, particularly the Level 2 plant room. Life safety risk to those outside the structure. **20%*NBS* (IL4)**	 This infill behaviour is governed by out-of-plane actions, hence the higher certainty than the infill in-plane interaction with the frames.
Infill walls causing column shear failure		Damage to frame due to infill panel interaction with frame, causing local shear failure to the columns. Occurs to south elevation of tower and around the northern stair and lift core area. Possible damage to nearby structure. Life safety risk to those inside the structure. **20% – 40%*NBS* (IL4)**	 Differences in score due to modelling of infills and assumptions around behaviour. Spread of scores covers both uncertainty in the gap present between the infills and the bounding frame.

Element	Location	Consequence	Estimated %NBS
North-South elevation beam-column joint failure		Beam column joint shear failure in north and south elevations of structure. Because of the use of deformed bars, we note some load redistribution to the internal frames causing beam column joint failure to the internal transverse frames. The score reported assumes this redistribution. Life safety risk to those inside and outside the structure **35% – 40%*NBS* (IL4)**	 Both are in general agreement that the beam-column joints to the north and south elevations have insufficient strength. Difference in scores likely due to analysis techniques used – Beca has undertaken an elastic analysis model with joint failure simulated by pinning damaged joints, whilst Holmes have used an inelastic model that considers redistribution of loads based on the non-linear properties of the elements.
Longitudinal direction frame failure		Beam column joint failure in internal and external joints, leading to loss of stiffness of system and excessive displacement demands on columns. Flexure-shear failure in perimeter columns due to interaction with half-height infill panels. Widespread damage throughout structure. Shear failure of internal beams, leading to widespread damage throughout structure. Life safety risk to those inside and outside the structure **40% - 50%*NBS* (IL4)**	 Note that it is difficult to compare directly the findings of Beca and Holmes, due to the difference in assessment methodology. Beca's methodology provides scores for the building in each principal frame direction, whereas Holmes' dynamic analysis inherently provides a score in both directions. Holmes' range of scores for elements is provided above, and does not represent an uncertainty range.

Element	Location	Consequence	Estimated %NBS
Transverse direction frame failure		Beam column joint failure in internal and external frames, leading to loss of stiffness to frames, excessive displacements and widespread damage throughout structure. **35% - 40%*NBS* (IL4)**	 As noted above, Holmes' range of scores for individual elements is provided above, and does not represent an uncertainty range.

3.4 Behaviour of Primary Frames and Expected Behaviour Hierarchy

Fundamental to the Beca assessment, is understanding the likely failure mechanism of the building. This is explained below.

3.4.1 Primary Frame Capacity

The primary frame failures represent the ULS failure state of the building.

The two intermediate frame failure modes – the high level plant failure and the corner beam-column joint failure – represent potentially significant structural damage to specific areas of the frame. The transverse and longitudinal frame failures represent the onset of building wide structural failures.

As all of these primary frame failure modes are associated with significant damage and risk or loss of life, they can all be consider as critical to the overall *%NBS* rating of the building, they are separately conveyed here to illustrate the different extent of their impact, which may be important for assessing remedial options.

The impact of the infill walls on the primary frames is considered in the following section.

3.4.1.1 Stage 1 Building

The Stage 1 building was found to primarily have issues with column flexure-shear strength as well as beam-column joint shear strength, however the strength hierarchy calculations undertaken also indicated that there are a number of undesirable brittle mechanisms possible.

The primary frames score is limited by the performance of the tower structure, in particular the perimeter frames in both directions. The eastern and western elevation frames are limited by their flexural and shear strength from two influences the reinforced concrete partial height walls have on the frames. Firstly the concrete upstands increase the lateral stiffness of the perimeter frames, which cause them to resist proportionately more load than the internal frames. The second impact is the 'shortening' of the columns. The concrete upstand reduces the effective length of the column, meaning that any hinges must form between the clear length of the beam above, and the upstand below. This significantly increases the shear demand on the column, leading to a premature shear failure of the column. The primary longitudinal frame has thus been scored 25*%NBS* (IL4) based on the performance of these columns.

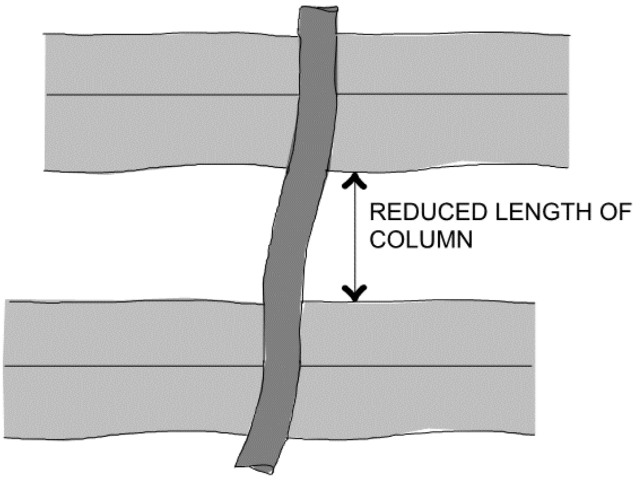


Figure 3.1: Example of shear failure of a short column, caused by influence of adjacent partial height infill (Kam, 2011) (L) and illustration of a 'short column' (R).

In the transverse direction, the southern elevation is expected to develop beam-column joint failure at approximately 30*%NBS* (IL4), with the tower frames as a whole scoring between 40% - 65*%NBS* (IL4)*.* The structural drawings show no transverse reinforcement in the beam column joint which, although typical for buildings of this era, has a significant impact on the ability of the joint to develop the full strength of the beam and column, and hence generate a ductile response. The use of plain round bars anchored into the joint with 180 degree hooks is also a detail that has significant detrimental implications on the joint performance, as there is a risk of a concrete wedge being ‘blown’ out the back of the joint when subjected to significant flexural demands leading to a risk of loss of gravity support. The joints at the end of the building, particularly the corners, will also be subjected to bi-directional loading, and consequently score particularly low.

3.4.1.2 Stage 2 Building

The Stage 2 Building uses deformed bars, and consequently has some different failure hierarchies when compared to the Stage 1 Building.

The critical failures of the primary frames in the transverse direction is due to beam-column joint failure of both interior and exterior beam-column joints. The perimeter frames are expected to fail in the order of 25-30*%NBS* (IL4) while the internal frames are expected to fail at approximately 40*%NBS* (IL4). The beam reinforcement is deformed and typically turned down into the joint with a 90 degree bend, a similar detail that would be used in modern construction. This may suppress a sudden brittle failure of the joint allowing it to maintain a gravity load path, but will result in a softening of the joint and redistribution of the loads to the internal frames, thus the expected score in approximately 35%-40*%NBS* (IL4).

In the longitudinal direction, the critical frames are the east and west elevation frames failing in shear due to the shortening effects described in the previous section, which have been scored at 40*%NBS* (IL4). There is also beam column joint failure of the internal longitudinal frames scoring approximately 50*%NBS* (IL4)*,*but as mentioned above, this may not constitute an abrupt loss of gravity support, but will lead to significant softening of the frame and large displacement demands on the columns *.* The critical storey for these mechanisms is typically Level 3 and Level 2.

3.4.2 Infill walls

The early onset failures are related to the infill walls. Infill walls were common in 1960’s construction, but they can have undesirable impacts on the primary structure. This is the case with the Galbraith Building.

There are two likely failure modes for the infill walls. The uncertainty in assessing these walls means it is not possible to identify which mode would form first, hence both modes are presented above.

<u>Possible collapse of Infill to the ground below:</u> This is the more desirable failure mode. In this instance the infill panels detach from the building and may or may not collapse to the ground below. This poses a direct life safety risk to any nearby pedestrians, but is considered a preference to the alternative mode as it does not significantly damage the primary structure of the Galbraith Building.

<u>Infill walls causing failure of end frame:</u> In this mode, the infill walls do not fall out, and instead ‘lock up’ against the end frames. This serves to stiffen the end frame and concentrate seismic load into this frame. The resulting end frame damage is therefore triggered much earlier that would be the case without the infill walls. This could in turn lead to wider failures in the end-most bay of the building, resulting in life safety risk to occupants.

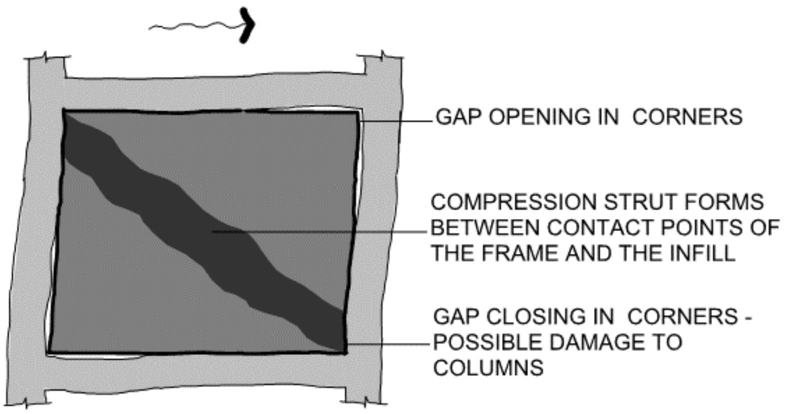


Figure 3.2: Illustration of compression strut forming between corners of frame due to interaction between frame and infill. Note that a column to column mechanism is shown.

The assessment of the infill walls has a very high level of uncertainty. This is because of the construction gap that has been built around the infill walls. Our understanding is that this gap has been formed by plasterboard at the time of construction, which is not a reliable method for forming a seismic gap, but will non-the-less provide some level of separation.

Should a robust gap be available, then the wall infill mechanisms would only eventuate at high *%NBS.* Should there be no gap, then the lower bound *%NBS* would be appropriate. While Holmes have based their analysis on a selected gap size, for the Beca assessment we have chosen to reflect the uncertainty of the size of the gap (and the high consequence of the gap) in the uncertainty of the *%NBS* score.

3.4.2.1 Differences in Wall Assessment Methodology

Both Beca and Holmes have used equivalent strut members for assessing the influence of the full-height concrete infill panels in the analysis models. Beca has followed Section C7 of the Assessment Guidelines to develop the section properties of the struts, and have modelled them as a ‘gap’ link element (the strut has no stiffness until the gap is closed and the frame engages with the infill) – with the gap ranging from 5 mm – 20 mm for sensitivity purposes.

The difference in scores between Beca and Holmes can be attributed to a few factors:

1. Geometry of struts – the exact geometry of the compression strut that develops in an infill panel from interaction with a bounding frame are difficult to locate precisely. For longer panels there is a possibility that multiple struts develop, as well as multiple struts that form around openings in panels. Beca has assumed that the strut forms either column to column, or column to beam, with the strut slightly offset from its nodal position. This causes the strut to have a shallower angle and consequently an increased stiffness, and more loads while Holmes have typically located struts nodal to the frames (central on beam and column).
2. Analysis methods – there is likely to be a significant amount of variability in load distribution around the structure between the different analysis methods used by Beca and Holmes. We have taken the approach of scoring only those columns that have a hierarchy that allows column shear failure prior to failure of the infill.
3. Stiffness of struts – Beca has followed the recommendation of Section C7 of the Assessment Guidelines to calculate the equivalent strut section properties, and then modelled a non-linear link element with an axial stiffness based on the these section properties. The gap around the infill is accounted for by using a ‘gap’ compression only link element, which means that the infill takes no load until the frame displaces enough to engage the link. In contrast Holmes have worked backwards from the shear strength of the

adjacent columns, to calculate a compression only strut with a stiffness based on the frame drifts required to 'yield' the column. The gap around the infills is accounted for by increasing the required drift, and thus decreasing the stiffness of the equivalent strut.

3.4.3 Overall Assessment

The result of Beca's independent review of the Galbraith Building indicates the building's earthquake rating to be 20*%NBS* (IL4) assessed in accordance with the guidelines document "*The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessment*" dated July 2017 (the Technical Guidelines). Therefore the building is a Grade D building in accordance with the Technical Guidelines grading scheme.

The summary of Beca's independent review of the Galbraith Building primary structure is:

- Beca has undertaken an independent Detailed Seismic Assessment for the primary frames, and determined the overall score for the primary frames to be 20*%NBS* (IL4).
- We consider there are a number of localised structural elements (infill walls, stairs, Galbraith – Bray Link) that have unpredictable and undesirable structural behaviour in an earthquake. We have qualitatively assessed these as <34%NBS (and thus Grade D).
- Since the building rating is less than 34*%NBS*, we consider it is likely Auckland City Council will determine the building's status as an Earthquake-prone Building upon receipt of this report and the Holmes Report.

By comparison, the Holmes Assessment of the Galbraith Building can be summarised as follows:

- Holmes rate the building as 20*%NBS* (IL4).
- Their report categorises the building as Grade D.
- Their report similarly identifies a range of poorly scoring structural elements within the building. These include the primary lateral load resisting system (the reinforced concrete frames), and it particularly identifies the infill walls as having an adverse impact on the behaviour of these frames.

We provide the following comparative commentary between the reports

- Following Beca's review of the building, we consider the Beca opinion and Holmes opinion of the building to be broadly in agreement.
- The Beca independent DSA intentionally followed a different methodology to the methodology adopted by Holmes. These different methodologies have resulted in similar fundamental conclusions and building *%NBS* ratings, but they give visibility to different aspects of the building behaviour. These differences have been discussed with Holmes and we are in general agreement as to the reasons behind these differences, and in general agreement that these differences do not have a substantive impact on the overall *%NBS* rating for the building.
- Our evaluation of the infill walls highlights the high uncertainty related to infill assessment, and specifically the high uncertainty (and significant consequence) of the presence and/or extent of the gap around the infill walls. We therefore consider it more appropriate to provide a ranged score for the infill walls. Following conversation with Holmes, we consider it most appropriate to give the *expected* range for these infill walls as 20*%NBS* (IL4) – 30*%NBS* (IL4). We note however that due to the high uncertainty the actual performance of the walls may be outside this expected range.
- We highlight a key difference in the methodologies, in that Beca considers the structural issues with the primary reinforced concrete frames are likely to be more widespread that is readily apparent using Holmes' methodology. We note that this does not impact the overall *%NBS* rating but would impact the extent of possible strengthening. From our conversations with Holmes, we understand that they are broadly in agreement with our observations.

4 Review of Identified Structural Weaknesses

A structural Weakness (SW) is any element of the building that limits the earthquake rating to less than 100%*NBS*. The identified SWs in Galbraith building both Stage 1 and 2 buildings and the results are listed in table 3.1.1

4.1 Summary of Review – Stage 1 Building

Table 3.1.1 - Identified Structural Weaknesses - Stage 1 Building

Galbraith Block Stage 1 Building Element	Structural Weakness(SW)	Beca Review & Assessment	Holmes Consulting %NBS(IL4)
Columns supporting Level 7 plant floor	Insufficient storey shear capacity leading to shear failure of columns, excessive lateral displacement and partial or complete collapse of plant floor	Column flexure-shear mechanism. **25%*NBS* (IL4)**	20%
Link Structure between Galbraith Stage 1 building and Building 2(Bray)	Insufficient in-plane displacement capacity in seismic separations to Galbraith and Bray building, leading to connection damage and loss of out-of-plane support	Lack of an identifiable lateral load resisting system in longitudinal direction has resulted in rating the link structure **<34%*NBS* (IL4).** For more details refer section 4.5.	20-30%
Infill walls to south elevation and core area(not including stair core walls)	Insufficient restraint against panels falling out of building onto roof or public areas below	Out-of-plane failure of concrete panels due to insufficient out-of-plane flexural strength **20%*NBS* (IL4)**	20%
Reinforced concrete columns to south tower elevation, adjacent full height concrete infill walls	Shear failure induced by contact with infill wall leading to loss of support of floors	Strength hierarchy shows the infills typically fail prior to a concrete shear failure along this elevation. Critical columns from panel interaction are located around the lift core area. **20% - 30%*NBS* (IL4)**	25%
Reinforced concrete columns to south tower elevation	Insufficient shear capacity leading to loss of support of floors	Beam-column joint shear failure from insufficient reinforcement in beam column joints. **30 %*NBS* (IL4)**	25%
Lightweight roof over level 7 plant floor	Insufficient lateral bracing to roof structure	Connection failure between portal beam and column connection. **25%*NBS* (IL4)**	25%
Southern stairwell	Insufficient sliding capacity to cope with building movement leading to damage to concrete stairs and walls and potential loss of support of stair flight	Not readily quantifiable, but considered **<34%*NBS* (IL4).** For more details refer to section 3.2.	25%
Reinforced concrete columns to central core area, adjacent full height concrete infill walls to stairs and lifts	Shear failure induced by contact with infill wall leading to loss of support of floors	Column shear failure due to interaction with infills. Critical frames located around lift core. **< 20% - 40%*NBS* (IL4)**	30%

Galbraith Block Stage 1 Building Element	Structural Weakness(SW)	Beca Review & Assessment	Holmes Consulting %NBS(IL4)
Short perimeter columns to east and west tower elevations, level 2 to 6	Premature shear failure leading to excessive lateral displacements and loss of support of floors	Column flexure-shear mechanism develops due to half-height concrete panels shortening the columns effective height. **20%*NBS* (IL4)**	30%
Beam Column Joints	Insufficient confinement to some beam-column joints, leading to loss of support o floors	This is not expected to be the score of the global collapse mechanism as this is an isolated joint. **30%*NBS* (IL4)**	30%
Link Structure between Galbraith Stage 1 building and Building 2(Bray)	Insufficient out-of-plane capacity of link structure leading to loss of support of link structure floors	The seismic response is governed by flexural failure of the supporting column, leading to excessive displacement demands on the columns and loss of gravity support **25%*NBS* (IL4)**	30%
Stairs to entrance and core area	Insufficient sliding capacity to cope with building movement leading to damage to concrete stairs and walls and potential loss of support of stair flight	Not readily quantifiable, but considered **<34%*NBS* (IL4).** For more details refer to section 3.2.	30%
Raft Foundation	Punching shear failure leading to loss of superstructure lateral stiffness, excessive lateral deformations and potential instability or loss of vertical support/excessive vertical displacements	Low risk of failure or excessive displacement under seismic loading **60-100%*NBS* (IL4)**	45-100%
Level 2 plant plat form roof	Insufficient capacity of steel portal frames and connections	Level 2 plant roof score **100%*NBS* (IL4)**	50%
All other reinforced concrete columns (except south elevation and short perimeter columns)	Insufficient shear capacity leading to loss of support of floors. Most columns have some risk (<100%NBS) however the score is governed by internal tower columns level 2 to 5.	Internal joints to tower are typically governed by actions in the transverse direction, with either beam-column joint shear failure or column flexure-hinging being the critical mechanisms. **40% - 65%*NBS* (IL4)**	55%
All reinforced concrete columns	Round bar limits strength and deformation capacity in columns and beam-columns joints, leading to increased lateral building displacements and higher risk of loss of support of floor from individual column failure.	Beca does not have a score that is directly comparable to this line item from Holmes.	55%
Short perimeter columns to west podium elevations, ground level to level 2	Premature shear failure leading to excessive lateral displacements and loss of support of floors	Shear failure of columns to western elevation of core area **45%*NBS* (IL4)**	55%
Reinforced concrete beams	Insufficient shear capacity of the beams, leading to progressive loss of lateral building capacity (and excessive lateral displacements)and local hazard /risks to floor support	Insufficient shear capacity of reinforced concrete beams due to lack of transverse reinforcement, **35% – 100%*NBS* (IL4)**	60-100%

4.2 Summary of Review – Stage 2 Building

Table 3.1.2 - Identified Structural Weaknesses - Stage 2 Building

Galbraith Block Stage 2 Building Element	Structural Weakness(SW)	Beca Review %NBS(IL4)	Holmes Consulting %NBS(IL4)
Infill walls to north elevations (not including stair core walls)	Insufficient restraint against panels falling out of building onto public area below	Out-of-plane failure of concrete panels **20%*NBS* (IL4)** due to insufficient out-of-plane flexural strength	20%
Northern stairwell	Insufficient sliding capacity to cope with building movement leading to damage to concrete stairs and walls and potential loss of support of stair flight	Not readily quantifiable, but considered **<34%*NBS* (IL4).** For more details refer to section 3.2.	30%
Beam column joints – north and south elevations	Insufficient confinement to some beam-column joints, leading to loss of support o floors	Failure of joint due to insufficient horizontal joint reinforcement. **35% - 40%*NBS* (IL4)**	35%
Beam-column joints – typical frame joints except north and south elevation	Insufficient confinement to some beam-column joints, leading to loss of support of floors	**35% - 40%*NBS* (IL4)**	45%
Reinforced concrete columns to north tower elevation, adjacent full height concrete infill walls	Shear failure induced by contact with infill wall leading to loss of support of floors	**20% - 40%*NBS* (IL4)** (depending on assumed gap width)	50%
Raft foundation	Punching shear failure leading to loss of superstructure lateral stiffness, excessive lateral deformations and potential instability or loss of vertical support/excessive vertical displacements	Low risk of failure or excessive displacement under seismic loading **60% - 100%*NBS* (IL4)**	55-100%
Reinforced concrete beams	Insufficient shear capacity of the beams, leading to progressive loss of lateral building capacity (and excessive lateral displacements)and local hazard /risks to floor support	**40% - 100%*NBS* (IL4)** Different score likely to be based on assessment methodology used, and its inability to consider inelastic behaviour.	60-100%
Reinforced concrete columns to north and south elevations	Insufficient shear capacity leading to loss of support of floors	Critical element on northern elevation at entrance level (ground floor). Beam offset from BC joint leading to significant shear demands on column **55% - 75%*NBS* (IL4)** South elevation, **75%*NBS* (IL4)** Difference in score likely to be based on assessment methodology used, and its inability to consider inelastic behaviour.	75%

Galbraith Block Stage 2 Building Element	Structural Weakness(SW)	Beca Review %NBS(IL4)	Holmes Consulting %NBS(IL4)
Short perimeter columns to east and west elevations; all levels at risk, but governed by mid-levels	Premature shear failure leading to excessive lateral displacements and loss of support of floors	Critical to columns Level 3 and below. **45% - 55%*NBS* (IL4)** Difference in score likely to be based on assessment methodology used, and its inability to consider inelastic behaviour.	75%
Interior reinforced concrete columns	Insufficient shear capacity leading to loss of support of floors	Governed by transverse direction frames internal columns. Note that Beca does not consider this a governing global failure mechanism. **80% - 90%*NBS* (IL4)**	75%

4.3 General Commentary

Beca has reviewed a range of primary and secondary structural elements and consider there are a number of elements in the building, both localised and more global, that are scored in the 20%*NBS* (IL4)-25%*NBS* (IL4) range. There are some differences in the Beca and Holmes assessments as to exactly which structural elements would trigger first, but this is explained by the inherent uncertainty that is part of seismic assessment.

Beca has also reviewed the building for Severe Structural Weaknesses (SSW) as defined by the Technical Guidelines and consider that no SSW are present. This is in general agreement with the Holmes findings.

A SSW is defined a structural weakness that is potentially associated with sudden catastrophic collapse and significant loss of life. We note that the lack of clearly definable longitudinal stability system in the Galbraith-Bray link, meets some of the criteria for a SSW, but the transiently occupied nature of the link would mean it would not be formally categorised as a SSW. This issues with this link should none-the-less be addressed.

4.4 Staircase and Safe Egress Review

Following the 2011 Canterbury earthquakes, the Department of Building and Housing (now MBIE) issued Practice Advisory 13 in response to concerns about stair collapse and damage observed in the earthquakes. The primary concern of this Practice Advisory is staircases with sliding support details in mid to high-rise multi-storey buildings.

The staircases in Galbraith consist of two flights between each floor as shown in Fig. 3.2.1. The stair flights appear to be constructed from in situ reinforced concrete; and are continuously supported by the reinforced concrete walls that enclose the stairwell. The original structural drawings indicate that the stair flights supported on the edge of the reinforced concrete beams at each floor level.

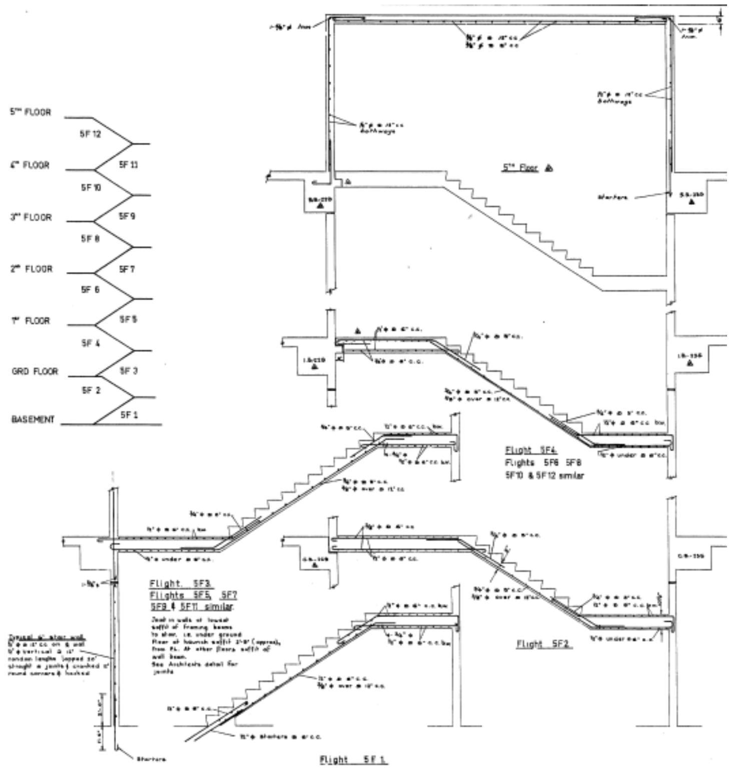


Fig.3.2.1 Stair Details - Galbraith Block Stage 2 Building

Because the stair cases are built in with no sliding connection, it is likely that stairs won't be able to accommodate the building displacement during the shaking leading to damage to concrete stairs, walls and potential loss of support of stair flights at each floor level.

Beca's view is that the stair structural system is such that it is very difficult to reliably quantify it as a *%NBS*. As the stairs are integrally connected to their surrounding walls they are very likely to be susceptible to early onset damage. On the flip side, the integral connections make them somewhat less susceptible to collapse than some other stairs may be.

Holmes has assessed the stairs as within a range of 25*%NBS* (IL4) to 30*%NBS* (IL4). We understand from Holmes that this range is based on a qualitative rather than quantitative assessment.

The Holmes rating of 25*%NBS* (IL4) to 30*%NBS* (IL4) suggests the following behaviour:

- The stair are potentially earthquake prone
- The stairs are not the lowest graded element in the building.

Beca agrees that both of these points are reasonable conclusions to draw about the stairs. Though we would highlight that the quantification of these point into a *%NBS* rating provides a somewhat misleading level of precision.

4.5 Galbraith-Bray Link Structure Review

Our site visit and review of the available drawings lead us to find the link structure does not include a clearly identifiable defined lateral load resting system and a robust load path in the longitudinal direction. Our discussions with Holmes Consulting confirm this finding. During earthquake shaking the link structure will displace longitudinally and may well "bounce" between the floors of Galbraith building and Bray building.

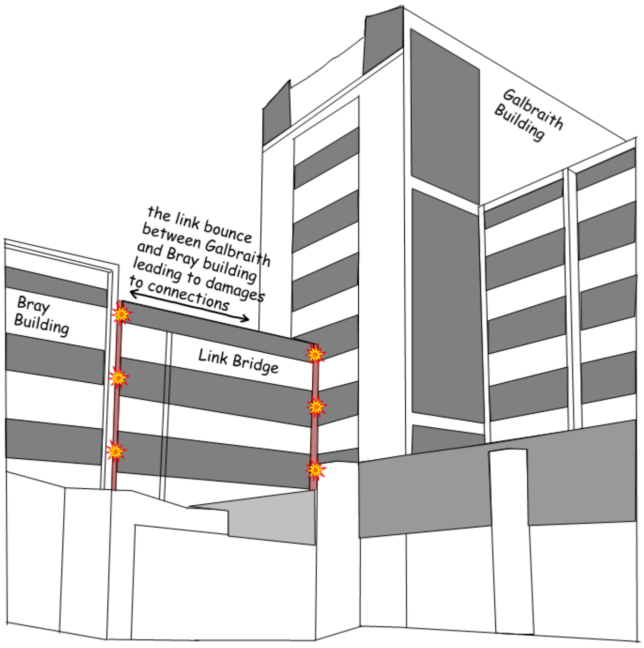


Fig. 4.5.1 Galbraith – Bray Link

It is possible that this 'bouncing' would lead to damage to the supporting connections. Of particular concern is the risk of damage to the bolt connecting the link bridge to the Galbraith Building. Should that bolt shear, there is a risk of loss of support at the Galbraith end of the link.

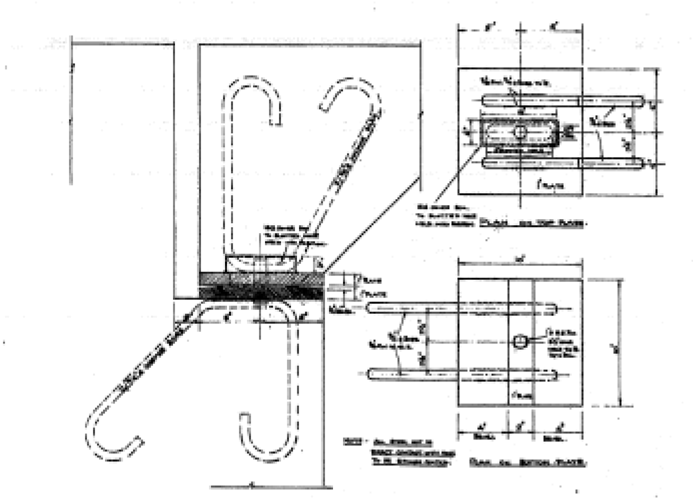


Fig. 4.5.1 Connection Details – Link Structure to Galbraith Building

In our view, it is difficult to assess and quantify a reliable *%NBS* in the absence of a clear lateral load resisting system. While the 'bouncing' between Galbraith and Bray may well contain the link bridge, this is not a reliable load path.

We identify this as a structural weakness and consider this would categorise the element as <34*%NBS*.

Holmes Consulting has assessed the link structure as within a range of 20-30*%NBS* (IL4) in the longitudinal direction. We understand from Holmes that this range is based on a qualitative rather than quantitative assessment and is reflective of the Galbraith-Bray longitudinal actions as being difficult to quantify, but generally a poor structural system. This is therefore similar to Beca's view on this link bridge. However we would highlight that providing a *%NBS* score to the link structure in longitudinal direction provides a somewhat misleading level of precision.

The lateral load resisting system in transverse direction comprises a slender reinforced concrete frame as shown below. Beca has undertaken an independent assessment of the frame which has resulted in 25*%NBS* (IL4) in the transverse direction. The seismic response of the link structure in these directions is governed by ground floor column flexural failure, leading to large storey displacements and eventual loss of gravity support.

Holmes Consulting has quantified the rating of the transverse frame as 30*%NBS* (IL4).

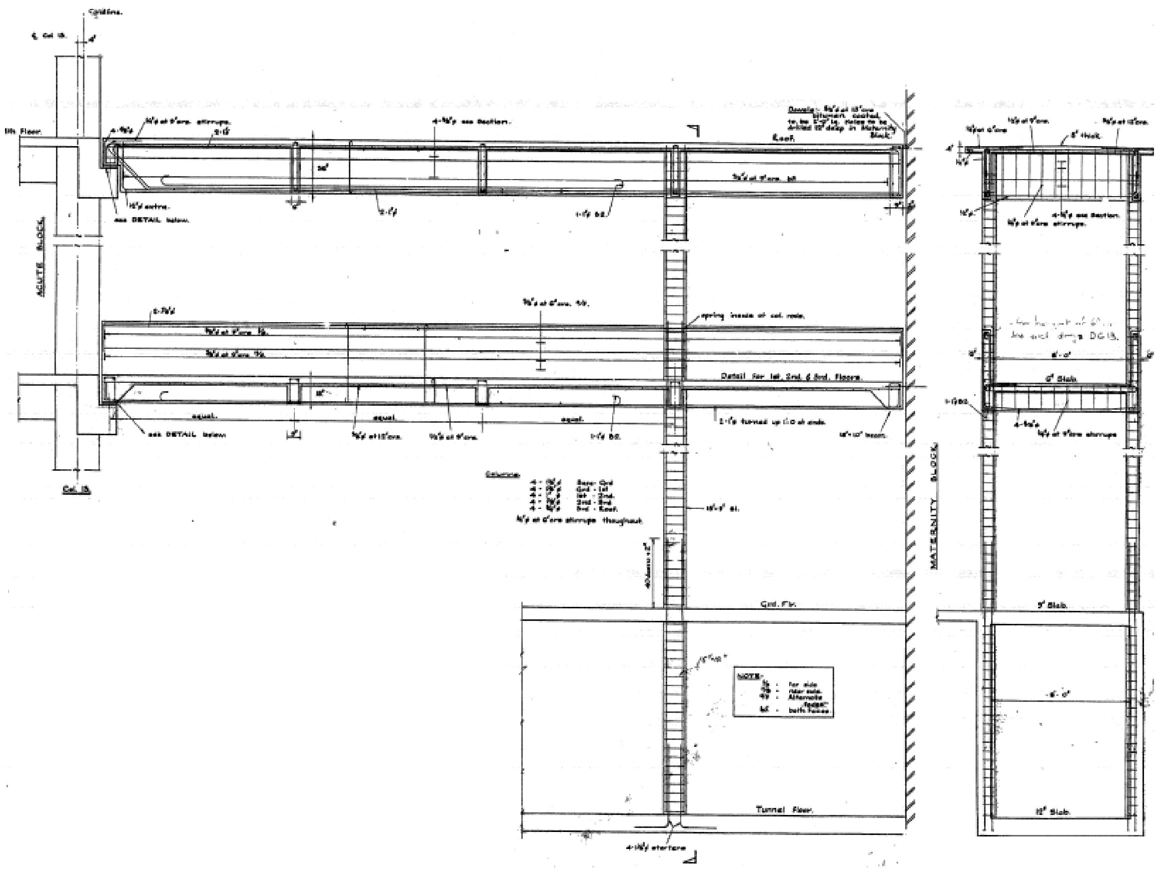


Fig.4.5.1 Link Structure Section

4.6 Foundation System Review

4.6.1 Site Conditions and Potential Geohazards Review

Tonkin and Taylor provided the geotechnical assessment of the Galbraith building. According to the geotechnical report, the site has generally a consistent profile. The geological profile consists of Puketoka formation of the Tauranga Group comprising alluvium with sand layers. This layer overlies Kaawa sand at a depth of 20-30m below ground level. Based on this assessment, the potential for liquefaction to have a consequential effect on the Galbraith building is low to very low. The assessed risk of failure under seismic loading is considered to be very low.

Beca has undertaken a high level review of the geotechnical report. We note that this report it does not include the specific information they used in the Holmes Consulting assessment, however according to our general experience of the location and the limited information in the report, we believe the outcomes are reasonable.

- The geotechnical assessment appears to be largely based on earlier work Tonkin and Taylor carried out in 2010/2012 rather than a specific review for this particular site. For a building of this importance this could be considered slightly light, though we would anticipate that further investigation might be undertaken as necessary for further works. We also note that there appears to be no geotechnical information under the footprint and all data is to the south and east, so there is potentially a risk (we consider small) that the conditions could be different for this building.
- The profile seems to generally match what would be expected but the “upper alluvium” may not all be as strong as Tonkin and Taylor have assumed. We would expect it to be variable with some stronger and some weaker zones/layers. We would note that the raft foundation would likely provide some ability to average this out.
- As noted in the report, the edges/corners are likely to attract the greatest loads and the Holmes analysis has taken account of this. T&T say that yielding will not take place but we believe some consideration of a localised reduction in modulus would be useful.

4.6.2 Raft Foundation Review

In Beca’s view the building is structurally dominated and the seismic response of the superstructure is the governing failure mode. However we have undertaken a simple qualitative assessment of the punching shear capacity of the raft foundation under maximum axial loads and rated the raft as of 60-100%*NBS* (IL4). The lower end score is based on the corners of the raft, with notably better scores being expected for interior columns.

By comparison, Holmes Consulting has evaluated the raft foundation being not overly sensitive to increased seismic loadings and rated that as 45-100%*NBS* (IL4). This is similarly reflective of the raft as being non-critical.

5 Commentary on Associated Seismic Risks

5.1 Serviceability Limit State (SLS2) Criteria for Structure

Beca has undertaken a high level review of the likely SLS2 (operational functionality) condition of the building. Beca's view is that the structural system is generally only capable of modest levels of ductility, and while this is not considered desirable in a ULS assessment, when evaluating against SLS2 it does means that the damage occurring within the structural system is likely to be relatively modest prior to structural failure.

As such, we consider that the SLS2 limit of the structure is likely to be closer to the ULS limit than would be common for many buildings, and we would not consider the SLS2 condition for the structure to be of a primary concern.

The Holmes report sets out an evaluation of the structural performance against SLS2 criteria for continued operability in a moderate event. These *%NBS* are provided for structural elements, which generally give the performance of the non-structural-elements as somewhat higher scores in the SLS2 case than in the ULS case.

It is our view that the SLS2 evaluation of the structural elements given by Holmes may be somewhat conservative, but would not in any case be the primary driver for any works required to the building.

We note that *%NBS* is intended by the Earthquake Prone Buildings methodology to be an evaluation against life safety concerns, so is not intended to be applied to SLS2 conditions. None-the-less, given the SLS2 requirements are highly important for a hospital, it is valuable to provide commentary on the likely performance.

Further, from our experience, the SLS2 condition is likely to be governed by the behaviour of non-structural elements, and we would expect that these may be critical in evaluating the impact on hospital-wide operations.

5.2 Serviceability Limit State (SLS2) Criteria for Non-Structural Building Elements

Maintaining operational continuity following an earthquake is not only dependent on primary structural elements but on secondary structural and non-structural elements. From our recent experience in evaluating similar buildings following the recent earthquakes in Christchurch and Wellington regions, non-structural building elements (façade glass, ceilings, internal walls, overhead services) constitute a significant portion of the repair / reinstatement cost following an earthquake. In a moderate seismic event, non-structural element damage will likely contribute heavily to downtime and the repair costs.

While the review of non-structural elements is not part of the scope of this report, we provide the following commentary.

Individual non-structural elements such as suspended ceilings, lightweight glazing façade panels, partition walls, lightweight cladding elements and building services are excluded from our review. We note however that there are historic concerns with the installation of seismic restraint for non-structural elements in buildings in New Zealand and that these have been found to perform worse than might be expected in significant seismic events.

We would recommended that a separate assessment of non-structural elements, and especially of building services is undertaken.

The façade of the Galbraith Building consists of infill concrete panels and individually framed windows. The infill panels have been discussed in some detail above from a structural perspective, and highlighted as a poorly performing element, through there is uncertainty as to the exact failure mechanism of these panels. Through any failure mechanism, the generally undesirable behaviour of the panels means we would consider it unlikely that the infill panels would maintain their weather-tightness function after a major seismic event.

The glazed portion of the façade of Galbraith building is individual framed windows. These are unlikely to have been detailed to accommodate any seismic movement in the building and may become damaged. However as the individual panes of glass only present a localised hazard to individuals they are not considered to present a significant non-structural risk (as per the *Engineering Assessment Guidelines* methodology). We would note that individually glazed windows have been found to perform better than expected in the recent earthquakes in Christchurch and Wellington.

5.3 Galbraith Building Infrastructure (Basement and Plantrooms)

Beca understand that the infrastructure in the Galbraith building feeds other facilities on the campus. Most notably the basement of the Galbraith Building feeds through to the wider hospital, and the Galbraith level 2 plantroom feeds the Acute Services Building.

We consider it would take high level of shaking to for structural failures to damage this infrastructure. However there is a risks that insufficient restraint of this infrastructure could lead to damage in a major seismic event.

We note however that the low *%NBS* of the building is likely to present a significant risk to continued access and ongoing use of the basement infrastructure. In a major seismic event large enough to damage the superstructure, it would be likely that there would be some level of damage to the basement infrastructure, even if the extent of that damage was relatively modest.

From our experience, any event resulting in damage to the superstructure may result in Health and Safety protocols making the building un-enterable for an extended period of time (days or weeks at the short end of the spectrum, to indefinitely at the long end). If this were coupled with some infrastructure damage, or simply ongoing infrastructure maintenance requirements, this could have a significant operational impact on the wider hospital campus.

The potential impact of Galbraith structural damage, coupled with infrastructure repair requirements, should be assessed at a hospital campus level to determine the overall campus operational risk.

5.4 Risks from, and to, Adjacent Buildings

Beca have reviewed the portion of the Holmes Consulting Report in relation to adjacent buildings. This section states that the Galbraith Building is a separate structure to the acute services building, highlights the distance of the Galbraith Stage 1 tower from the Acute Hub building and states *"In our opinion it is therefore unlikely that a tower collapse would progress into a collapse risk to the Acute Hub building, particularly given the alignment of concrete floors, the separate extension structure between, and the modern detailing in Acute Hub. There could still be significant life safety hazards to people in the Acute Hub building from debris, and so in such a scenario, restricted access in the grid bays nearest the Galbraith Extension would be prudent."*

Beca are in general agreement with this evaluation as regards likelihood of direct damage to the Acute Hub building from a Galbraith building failure. We highlight two additional considerations that we believe should be considered by CMDHB.

- It is our experience that following major seismic events there can be a period of considerable uncertainty as to the state and safety of damaged buildings. There can also be considerable uncertainty as to the appropriate precautions that need to be put in place. This is especially true if advance emergency planning has not covered damaged building scenarios (as it generally hasn't), and consider the extensive operation disruption caused by a major event. We have found that in practice, this means conservatively large cordons will often be put in place by decision-makers on the ground at the time. In our experience, these practicalities of post-disaster management may mean that damage to Galbraith impacts the Acute Hub building, and we recommend that CMDHB mitigate this by building into their advance emergency planning
- We understand from our Building Services work on Middlemore that the Galbraith L2 plantroom feeds the Acute Hub building. The structural stability (and access to) the L2 plant room will be determined by the Galbraith Building behaviour. This may mean that the services in the Acute Hub building (and thus Acute Hub building operations) are directly impacted by damage to the Galbraith Building. Building Services advice should be sought on this matter.

6 Commentary on Seismic Retrofit and Strengthening

6.1 Retrofit and Strengthening Options

Beca has reviewed the range of possible retrofit and strengthening options presented by Holmes in their report. We are broadly in agreement that these retrofitting options represent appropriate types of retrofitting techniques for the building, noting that there is still significant work required to develop a full strengthening scheme.

We do however provide a brief commentary on the options set out by Holmes as an aid to future decision making.

Broadly, Holmes present three types of retrofit strategies, with the intention that some or all may be implemented – that is, they are not mutually exclusive. Each type is commented on below:

Local Strengthening

There are a number of options put forward for localised repairs to parts of the building that are particularly poorly detailing, unpredictable, or may have disproportionate impacts. These include:

- Infill walls (poorly detailed, unpredictable, may lead to disproportionate failures)
- Galbraith-Bray link structure (poorly detailed, unpredictable, may lead to loss of emergency egress)
- Stairs (poorly detailed, unpredictable, may lead to loss of emergency egress)
- L7 plant and roof (localised low capacity)

It is Beca's view that these localised strengthening would all be essential works for the Galbraith Building. They address particularly unpredictable behaviour, and are all relatively localised in their extent, making them much more feasible to undertake than the global strengthening options.

Global Strengthening

The Holmes report similarly sets out a number of strengthening options for the more global deficiencies – that is those associated with the primary moment frames.

Beca agree with Holmes assessment that these are low capacity structural elements and that that they would require strengthening should CMDHB seek to raise the *%NBS* of the Galbraith Building. However we highlight an important difference in the outputs of the assessments in relation to the *extent* of these repairs.

The Beca assessment approach includes evaluating a wide range of frames and shows that the capacities of a significant proportion of the frames is relatively low. This means that the Beca assessment identifies a significant proportion of the frames that would require remedial works should strengthening be undertaken.

The Holmes NLTHA models failure sequences of individual elements. This means that after initial structural element failures it can be difficult to discern the subsequent behaviour of other structural elements. It is somewhat analogous to these initial structural failures acting a 'fuse' and making determination of post-fuse behaviour unclear. Beca believe that this means the extent of structural issues in the frames is more widespread than is visible in the Holmes model. We have discussed this issue with Holmes and it is our understanding that they are broadly in agreement with the view.

We stress that this difference in analysis does not fundamentally change the grade given to the building (which is based on initial failures, for which both analysis methods give similar results), but we believe it does impact the extent of likely strengthening works to the frames, and thus is likely to impact feasibility of significant building strengthening.

Downgrading of importance Level to IL3

The idea of downgrading the building to IL3 is presented in the Holmes report. Beca agree that this would be a pragmatic step towards shifting the building above *34%NBS.* We highlight two key points in relation to this.

- Most significantly, classifying the building as IL3 is based on changing the use of the building. Specifically this means that the building could not be designated for IL4 “Medical emergency or surgical facilities”, “emergency services facilities”, “post-disaster function”, or “essential facilities”. We note that IL3 use includes “health care facilities with a capacity of 50 or more residential patients but not having surgery or emergency treatment facilities”, or “Emergency medical or other emergency facilities not designated as post-disaster”.
- We highlight the disruptive impact variable importance levels can have on an interconnected hospital campus. The interconnectedness of access, egress, utilities, and infrastructure can mean that damage sustained to lower importance level buildings can adversely impact on nearby IL4 facilities. Beca’s general experience is that the limitation of an IL3 building in a major hospital campus can be constraining for the hospital and would need to be carefully evaluated as part of the long term operational plan. This can be further constrained by concerns about utilities connecting between buildings of different importance level.
- We would expect that concerns around the more unpredictable elements of the building (infill walls, Galbraith-Bray link, stairs) would still likely need to be addressed through retrofitting work even following any downgrade to IL3.

Significant Interventions

The Holmes report sets out some options for more significant interventions and notes additional damping as the likely most effective method. Beca consider the options put forward by Holmes to be a reasonable approach to providing step-change improvement in the structure, though we note that we would consider these to be major strengthening works and agree they would also need to be coupled with addressing local deficiencies.

7 Explanatory Statement

- This report has been prepared by Beca at the request of our Client and is exclusively for our Client's use for the purpose for which it is intended in accordance with the agreed scope of work. Beca accepts no responsibility or liability to any third party for any loss or damage whatsoever arising out of the use of or reliance on this report by that party or any party other than our Client.
- The inspections of the building discussed in this report have been undertaken to assist in the structural assessment of the building structure for seismic loads only. This assessment does not consider gravity or wind loading or cover building services or fire safety systems, or the building finishes, glazing system or the weather tightness envelope.
- This assessment does not include an assessment of the building condition or repairs that may be required.
- No geotechnical ground investigations, subsurface or slope stability assessments have been undertaken by Beca. The geotechnical review was limited to a very high level review of the available geotechnical report by Tonkin and Taylor dated 8 February 2018.
- Beca is not able to give any warranty or guarantee that all possible damage, defects, conditions or qualities have been identified. The work done by Beca and the advice given is therefore on a reasonable endeavours basis.
- Except to the extent that Beca expressly indicates in the report, no assessment has been made to determine whether or not the building complies with the building codes or other relevant codes, standards, guidelines, legislation, plans, etc.
- The assessment is based on the information available to Beca at the time of the assessment and assumes the construction drawings supplied are an accurate record of the building. Further information may affect the results and conclusion of this assessment.
- Beca has not considered any environmental matters and accepts no liability, whether in contract, tort, or otherwise for any environmental issues.
- The basis of Beca's advice and our responsibility to our Client is set out above and in the terms of engagement with our Client.

Appendix A

Quantitative Calculation Summary

Galbraith Building Stage 1 and Stage 2 Calculation Summary

1 Introduction

The purpose of this document is to provide a concise summary of the key assessment calculation steps and methodology used.

2 Summary

The buildings are 1962 IL4 hospital buildings, consisting of reinforced concrete moment frames with some reinforced concrete infills. The site is located in Manukau, south of Auckland. The structures sit on reinforced concrete raft foundations. Notable features include:

- Infill panels cantilever from the tops of beams up the end elevations of the buildings, and around the core area of Stage 1. An approximately 16 mm gap is detailed to surround the panels (filled with plasterboard), with the only physical connection at the base of the panels via starter bars from the beam. The infill panels are reinforced with non-ductile HRC mesh.
- Stage 1 has plain round bars with bars typically anchored into the unreinforced beam column joints using 180 degree hooks. For Stage 2, there is also no beam column joint reinforcement, however bars are deformed and turned down into the joint with 90 degree bends
- Lap lengths are called up on the structural drawing as typically being $40d_b$. This is approximately that of a modern design code for the deformed bars of Stage 2, thus no reduction in strength has been applied. For Stage 1, this is typically less lap length than modern design codes require, and the allowable stresses in the lap splices has been reduced accordingly. We note that the column schedules note that lap splices are to occur mid-height of each storey, whilst the drawings show these splices occurring above floor level. We have assumed that the cross-sections take precedence over the note on the column schedule.

Critical assumptions and conclusions include:

- Infill panels. Scores have been reported only for those columns where the shear strength of the infill exceeds that of the column (i.e hierarchy allows for column shear failure prior to the infill failing). A sensitivity analysis has checked the scores of the building based on a 5 mm, 10 mm and 20 mm gap around the infill.
- A 'bare frame' analysis was undertaken separate from infill analysis. This is to prevent the scores of the frame being supressed by the governing
- End columns of frames were assumed to have 0 axial load for joint shear calculations. Internal columns were not expected to have significant variable axial loads and thus were assessed with their seismic gravity loads applied.
- Haunched beams flexural strength and modelled stiffness was based on the deepest section of the member adjacent to columns.
- Part C5 of the Assessment Guidelines notes that if plain round bars are lapped in PPHZ then a structural ductility factor of 1.0 should be used. This recommendation has been followed, however we note this is peak conservatism. There are many different mechanisms occurring within the structure (column hinge – column shear – BCJ failure), none of which are expected to develop any significant structural ductility, thus an elastic loading (mu = 1 and Sp = 1) has been used to score the structure.

- The gamma factor from Part C5 of the Assessment Guidelines for shear degradation of the beams and columns has been taken as 0.29 based on the low expected curvature ductility demands expected from the columns. That is, we do not expect to get any ductility out a column hinge mechanism due to the lap splice failures, thus significant curvature demands are not expected to be imposed, and thus failure has occurred prior to the degradation of the shear strength. A sensitivity check of the gamma factor was completed to gauge the residual shear capacity of the columns following the flexural , taking a factor value of $\gamma = 0.10$ (corresponding approximately to a curvature ductility of 10), the probable residual column shear capacity is very close to the scores for flexural hinging.

3 Analysis Procedure

The detailed seismic assessment generally follows recommendations of the Engineering Assessment Guidelines (July 2017).

Below is a summary of the calculation process:

- Review of structural drawings undertaken to assess the structural configuration and potential load paths. Identified possible issue with column shear, and lack of joint reinforcement causing beam column joint shear failure.
- Probable member capacities calculated using the Assessment Guidelines Part C5: Concrete Buildings.
- Joint hierarchy checked by comparing the probable capacities of elements framing into a joint. Each joint could then be checked for what its expected failure mechanism was. This formed the basis for proceeding with an elastic force based analysis.
- 3D analysis model created in ETABS. Modal response spectrum analysis used to ‘score’ critical elements
- Engineering judgement applied to mechanism scores to assess inelastic behaviour and develop scores for local elements, entire frames and the global structure

4 Loads

4.1 Gravity Loads

a. Dead Loads

Unit weights for materials are as follows:

Concrete unit weight	=	24	kNm^{-3}
Steel unit weight	=	80	kNm^{-3}
Concrete masonry unit weight	=	20	kNm^{-3}

Uniformly distributed area loads:

External concrete panels (4” thick)	=	2.45	kPa
Internal concrete panels (6” thick)	=	3.65	kPa
Concrete floor (6” thick)	=	3.65	kPa
Concrete floor (7” thick)	=	4.30	kPa

Light weight roofing	=	0.05	kPa	
Purlins and roof framing	=	0.05	kPa	
Uniformly distributed line loads:				
External spandrels (1000 high, 100 thick)	=	2.45	kN/m	
Roof parapet (900 long, 100 thk)	=	2.15	kN/m	
Point loads				
Stair weight per stair	=	30.0	kN	per stair

b. Superimposed Dead Loads

Uniformly distributed area loads:

Suspended ceiling	=	0.20	kPa
Services (general floors)	=	0.50	kPa
Services (basement)	=	1.00	kPa
Finishes	=	0.10	kPa
Cladding panels	=	0.50	kPa
Partitions and non-structural walls	=	0.20	kPa (over the floor plate)

Uniformly distributed line loads:

Glazing (allowance for 10 thk, 1/3 height)	=	0.25	kN/m
Cladding panels(allowance for heavy cladding)	=	0.40	kN/m

c. LIVE LOADS

Uniformly distributed area loads:

General areas	=	3.0	kPa

3.0 kPa has been used as some areas are offices (3.0 kPa), some areas are hospital wards (2.0 kPa), some are operating theatres (3.0 kPa) and some are hallways and corridors (4.0 kPa). So on average, approximately 3.0 kPa is a fair assumption over the entire floor plate.

Roof	=	0.25 kPa

Note that there is no plant room in Stage 2 thus no requirement for any additional live load considerations.

Note that for seismic weight calculations, an area live load reduction factor of 0.5 has been used in conjunction with the combination factor.

4.2 Seismic Weight

A summary of the results is as follows:

Stage 1

Storey	DL (kN)	SDL (kN)	LL (reduced)	Ws (kN)	ms (t)	Average
7th Floor	3387	408	1006	4801	489.447	13.2
6th Floor	4889	1029	822	6740	687.009	5.9
5th Floor	9435	1116	463	11014	1122.7	10.7
4th Floor	10097	1135	463	11695	1192.12	11.4
3rd Floor	10479	1135	486	12100	1233.4	11.8
2nd Floor	18530	1968	485	20983	2138.97	9.0
1st Floor	22432	2712	1188	26332	2684.22	10.2
Ground	21116	3570	1032	25717	2621.52	11.2
				119382	12169	10.0 kPa

Stage 2

Storey	DL (kN)	SDL (kN)	LL (reduced)	Ws (kN)	ms (t)	Average
6th Floor	1224	685	0	1909	194.585	2.5
5th Floor	5927	801	331	7060	719.645	9.3
4th Floor	6077	801	331	7209	734.859	9.5
3rd Floor	6077	801	331	7209	734.859	9.5
2nd Floor	6661	801	331	7793	794.401	10.2
1st Floor	6862	801	331	7994	814.89	10.5
Ground	7912	1169	331	9412	959.462	12.4
				48586	4953	9.1 kPa

For a reinforced concrete building, an average distributed weight of approximately 8-12 kPa is expected, so these values are reasonable.

Note that these include additional mass from attached items such as the Maternity Link Building.

4.3 Seismic Loads

The following data was used for seismic loads:

- Soil Class D
- Z=0.13
- R_U = 1.8 (IL4) with corresponding SLS.
- N(T,d) = 1.0

Generally, as non-ductile mechanisms were identified, S_p = 1.0 and k_u = 1.0.

5 Analysis Model

A modal response spectrum analysis (MRSA) using ETABS 2016 was used for the structural analysis of the buildings. The MRSA was used to account for possible higher mode effects of the buildings.

The MRS was scaled up to 100% of the equivalent static loads to account for the vertical irregularity in Stage 1. The response spectrum was also scaled up to 100% for the Stage 2 building to account for possible torsional irregularity caused by the infills, the vertical stiffness irregularity caused by the basement shear walls. We note that this is a conservative assumption, however it does not fundamentally change the outcome of the seismic assessment.

5.1 Modal Analysis

A summary of the modal analysis of the structures are included below. For conciseness, only the first 12 modes are included. Sufficient modes were included to

Stage 1

TABLE: Modal Participating Mass Ratios								
Case	Mode	Period	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
		sec						
Modal	1	1.185	0.013	0.4145	0	0.013	0.4145	0
Modal	2	1.07	0.6116	0.0163	0	0.6246	0.4308	0
Modal	3	0.841	0.0032	0.2055	0	0.6278	0.6364	0
Modal	4	0.613	0.103	0.0042	0	0.7308	0.6406	0
Modal	5	0.524	0.0242	0.0601	0	0.755	0.7006	0
Modal	6	0.509	0.0304	0.0001	0	0.7854	0.7007	0
Modal	7	0.429	0.0025	0.1766	0	0.7879	0.8773	0
Modal	8	0.384	0.1019	0.0004	0	0.8899	0.8777	0
Modal	9	0.341	0.0385	0.0009	0	0.9283	0.8786	0
Modal	10	0.33	0.0008	0.0215	0	0.9291	0.9001	0
Modal	11	0.305	0.0006	0.0002	0	0.9297	0.9004	0
Modal	12	0.287	0.000003253	0.0223	0	0.9297	0.9227	0

Stage 2

TABLE: Modal Participating Mass Ratios								
Case	Mode	Period	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
		sec						
Modal	1	1.125	0.0011	0.7559	0	0.0011	0.7559	0
Modal	2	1	0.7136	0.0017	0	0.7147	0.7576	0
Modal	3	0.969	0.0877	0.0013	0	0.8024	0.7589	0
Modal	4	0.492	0.0227	0.00002824	0	0.8251	0.7589	0
Modal	5	0.437	0.000001374	0.1124	0	0.8251	0.8713	0
Modal	6	0.364	0.00002987	0.0034	0	0.8251	0.8748	0
Modal	7	0.34	0.0293	0.0002	0	0.8545	0.875	0
Modal	8	0.33	0.0705	0.0005	0	0.925	0.8754	0
Modal	9	0.307	0.0005	0.0511	0	0.9255	0.9265	0
Modal	10	0.282	0.0002	0	0	0.9257	0.9265	0
Modal	11	0.269	6.385E-07	0.004	0	0.9257	0.9305	0
Modal	12	0.238	6.235E-07	0.0001	0	0.9257	0.9306	0

5.2 Modelling Parameters

The following parameters were used within ETABS:

- Beams and columns were modelled as frame elements
- Rigid zone factor of 0.5 applied to all frame joints
- Infills were modelled as non-linear gap elements, with varying gap sizes and a stiffness based on the equivalent strut properties (AE/L). A separate non-linear static analysis was undertaken for the assessment of the infills using equivalent static demands in each direction. Struts were assumed to form from the column rather than the centre of the beam column joint.

- Beam cracked stiffness used $0.40I_g$ based on NZS3101 (Amnd 3) Table C6.5 for T and L beams. Beams with no contribution from the slab (i.e rectangular) were modelled with the same factor.
- Column cracked stiffness varied with axial load contribution, but generally ranged between $0.40I_g$ and $0.5I_g$
- The mass from the basement structure was not included in the mass source for either building
- Connecting structures and/or secondary structures attached to the buildings such as the Link Building, plant room roofs, etc were not modelled but their mass was included
- The half-height reinforced concrete upstand walls on the east and west perimeter of both buildings were modelled using shell elements. A stiffness reduction factor of 0.25 was used for these elements.
- Basement walls were modelled as shell elements with stiffness reduction factors based on their aspect ratios. These vary between 0.25 and 0.45.
- Floors were assigned rigid diaphragms, except for the roof which was modelled flexible.

6 Element Capacities

6.1 Infill Walls

The infill walls for Galbraith Building cantilever from the beam below. A packer of plasterboard or similar compressive material provides a gap between the panel and the frame.

Equivalent struts are calculated using recommendations from the Engineering Guidelines Part C7. There are a number of configurations of the struts, however the most likely mechanisms have been modelled in the ETABS model. There is a variation of what governs between shear capacity of the columns and the strut capacity, up the heights of the building and brace to brace.

An example of how the equivalent brace strength is included below for a 4” thick wall.

Equations from Part C7: Moment Resisting Frames with Infill Panels

hcol	=	3886	mm	
hinf	=	2793.8	mm	
Ef	=	23500	MPa	
Ec	=	23500	MPa	
Ibc	=	4854923348	mm4	
Linf	=	6146.6	mm	
rinf	=	6751.74	mm	
t	=	101.6	mm	
theta	=	0.427	rad	
lamda1	=	0.00109	-	
a	=	847.2	mm	
P	=	41.9	kN	axial load taken as base of wall (h x L x t x γ)
AE/L	=	299581.9		

The equivalent struts were modelled in ETABS using non-linear gap elements. These elements require a ‘closing’ of a gap prior to their stiffness being engaged, and act in compression only as a result.

6.2 Sub-assembly Capacities

The probable material properties used for calculation of member properties are as follows:

Material Properties				
Material		**Design Strength (MPa)**	**Strength Mod Factor**	**Assessment Strength (MPa)**
Reinforcing Steel (High tensile)		*317*	*1.08*	*342.36*
Reinforcing Steel (Medium tensilel)		*270*	*1.25*	*337.5*
Reinforcing Steel (Mild steel)		*240*	*1.25*	*300*
Concrete	**Foundations**	*17.5*	*1.5*	*26*
	Slab on Grade	*17.5*	*1.5*	*26*
	Suspended slab, beams, precast panels	*17.5*	*1.5*	*26*
	Columns	*17.5*	*1.5*	*26*
Structural Steel	**Beams**	*236*	*1.15*	*271*
	Columns	*236*	*1.15*	*271*
	CHS	*236*	*1.1*	*260*
	Plate	*236*	*1.15*	*271*
	Other members	*236*	*1.15*	*271*
Bolts				*4.6 (uno)*
Weld Strength				*E41XX GP (uno)*

The reinforcement was assumed to be mild steel with a probable yield strength of 300 MPa.

For moment curvature analysis of members, recommendations from the Engineering Assessment Guidelines Part C5 Concrete Buildings have been used:

- Steel reinforcement - bi-linear approximation
- Concrete – an unadjusted Mander concrete model

6.2.1 Columns

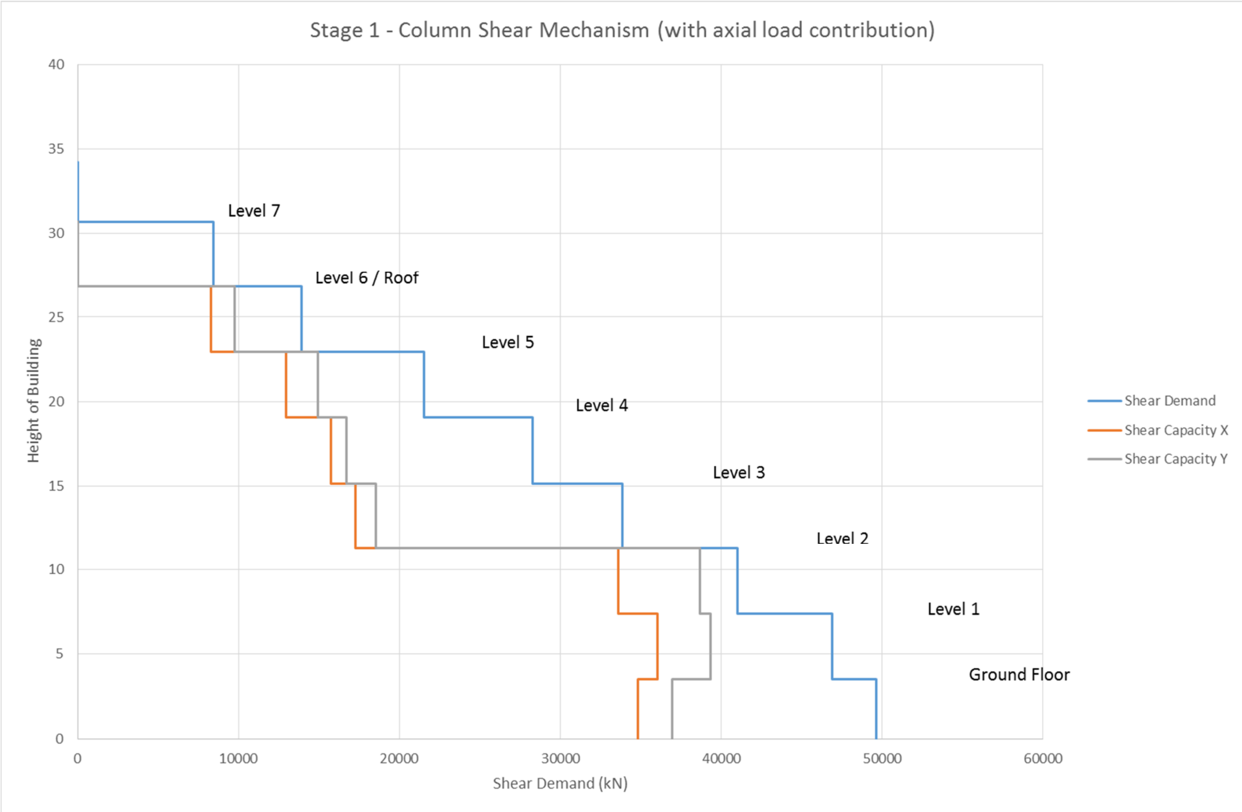
Column capacity was calculated using the recommendations in Part C5. Columns had shear capacity calculated for both zero axial load and seismic gravity load axial load. Typically external columns had their axial load contribution to shear strength taken as 0 to account for frame action causing low or zero axial load on the end columns.

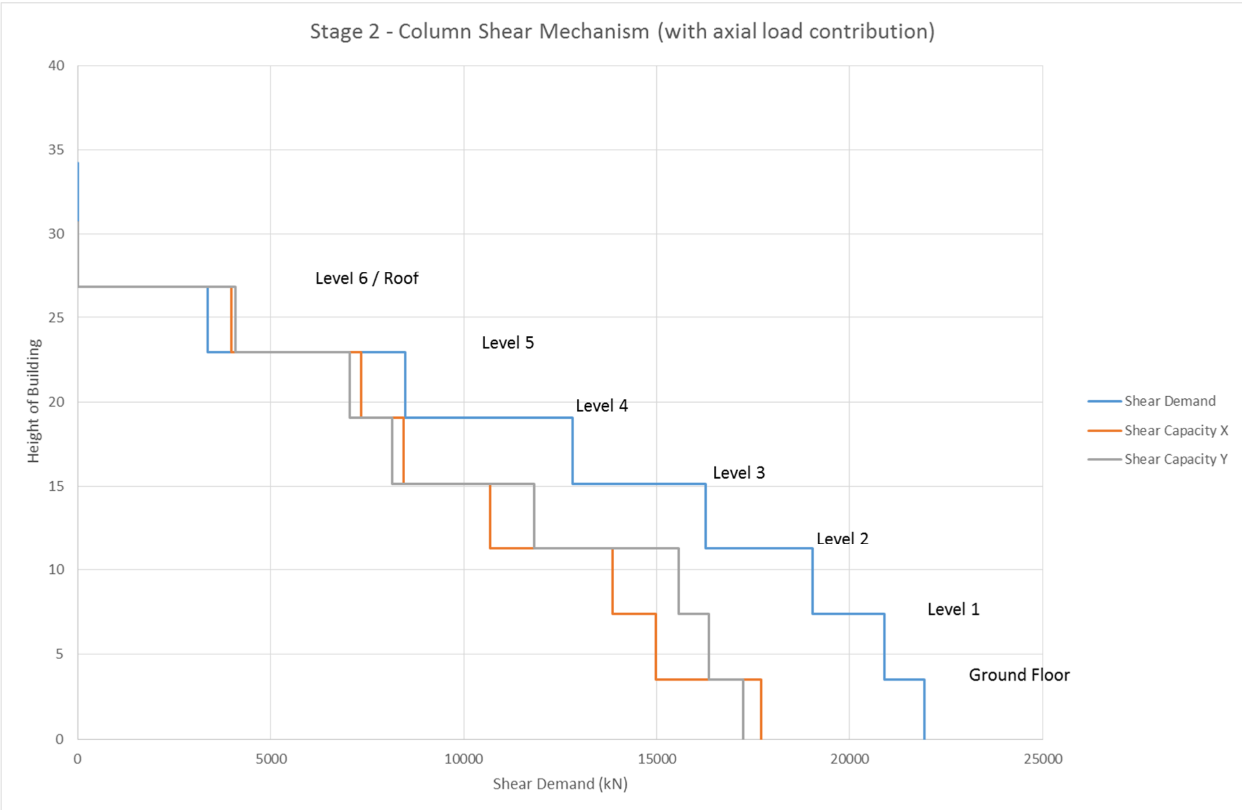
The storey shear capacity neglects any contribution from the stair well walls (gaps present above thus no positive anchorage for shear transfer), but does include the basement perimeter and internal reinforced concrete walls.

Stirrups are typically at 12” (305 mm) centres through the central portion of columns, with closer stirrup centres of 6” (152 mm) closer to joint faces.

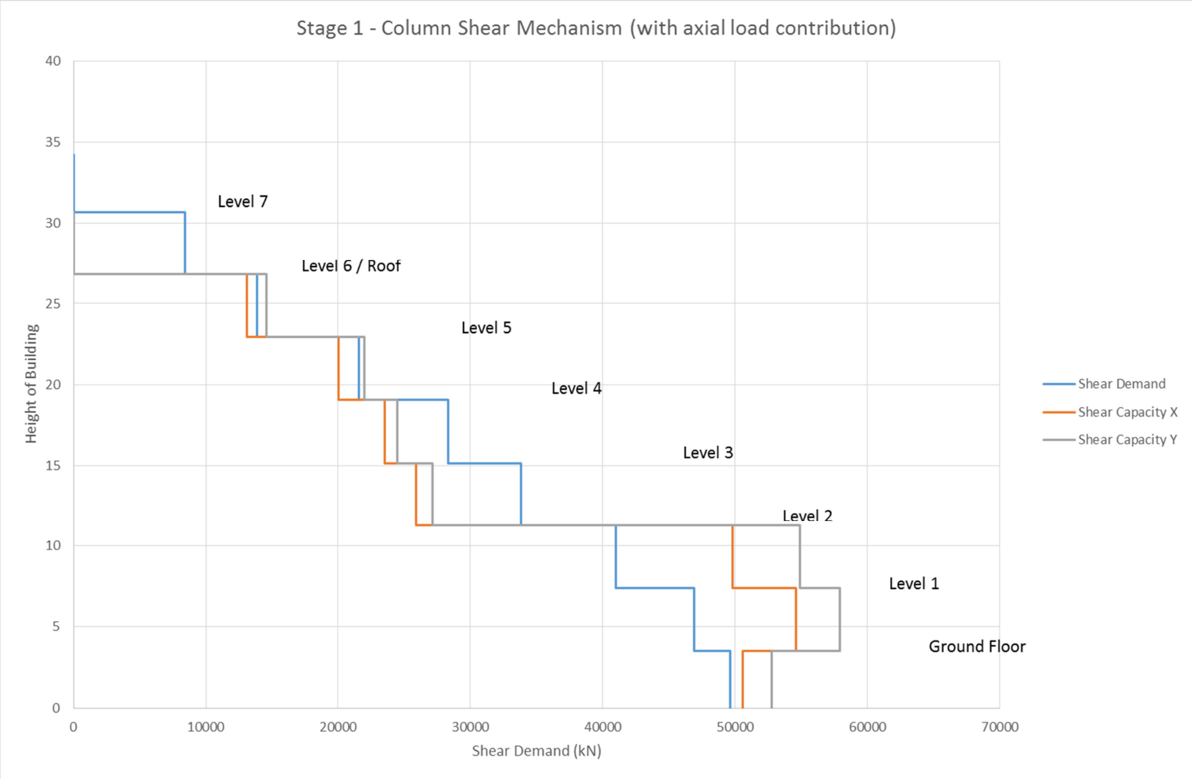
To provide an estimate of the upper bound scores for the structure, a storey shear check was undertaken with the column shear capacity. The graphs below summarise the results:

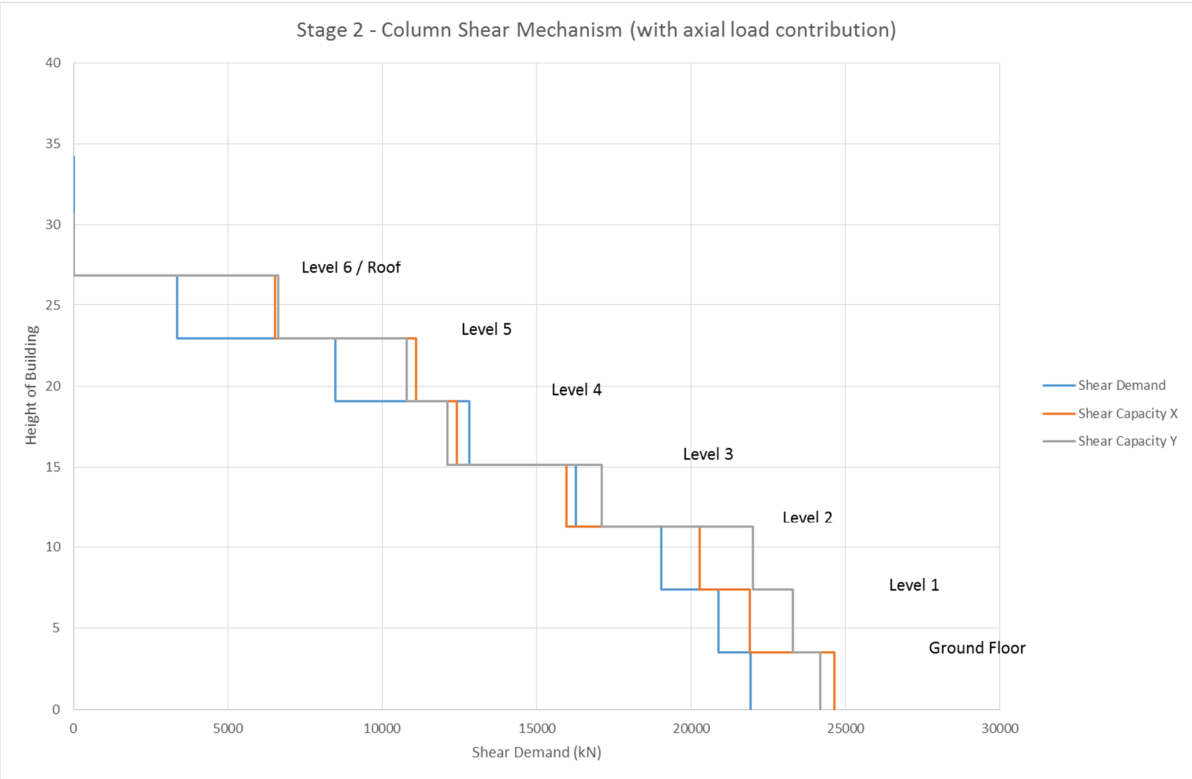
a. Gamma = 0.1


Stage 1 - Column Shear Mechanism (with axial load contribution)
Height of Building
Shear Demand (kN)
Level 7
Level 6 / Roof
Level 5
Level 4
Level 3
Level 2
Level 1
Ground Floor
Shear Demand
Shear Capacity X
Shear Capacity Y
0
5
10
15
20
25
30
35
40
0
10000
20000
30000
40000
50000
60000


Stage 2 - Column Shear Mechanism (with axial load contribution)
Height of Building
Shear Demand (kN)
Level 6 / Roof
Level 5
Level 4
Level 3
Level 2
Level 1
Ground Floor
Shear Demand
Shear Capacity X
Shear Capacity Y
0
5
10
15
20
25
30
35
40
0
5000
10000
15000
20000
25000

b. Gamma = 0.29





6.2.2 Beams

Beam capacity was calculated using the recommendations in Part C5. Contribution from the slab cast integral with the beams was included in the moment-curvature analysis.

Stirrups are typically at 12” (305 mm) centres through the central portion of beams, with closer stirrup centres of 6” (152 mm) closer to joint faces.

For haunched beams, the minimum cross sectional area at the centre of the beam was used as the critical section for shear.

6.2.3 Beam Column Joints

Beam column joints were assessed using recommendations in the Assessment Guidelines C5: Concrete Buildings. To account for variation in axial loads, the external joints were checked for both maximum axial loads and zero axial loads and the expected hierarchy decided based on comparisons with the surrounding elements.

The joint capacities use the revised equations (currently out for comment) in the Assessment Guidelines as these are typically revised based on errata.

The joints for Stage 1 which use plain round bars with hooked ends use k_j = 0.2.

For Stage 2, more joint deformation was expected to be possible due to the use of deformed bars and hence some redistribution of load from the lowest scoring joints was expected. A joint factor of k_j = 0.4 was used for this.

7 Joint Hierarchy

Joint hierarchies were checked based on the probable capacities of the elements framing into the joint as well as the joint itself. Variation in axial load was checked to identify all possible mechanisms.

Generally, the prevailing critical mechanisms within the structure are joint shear and column shear, with some isolated instances of beam shear. No significant ductility was expected to be achieved due to the use of round bar, lack of adequate confinement, and lack of joint reinforcement.

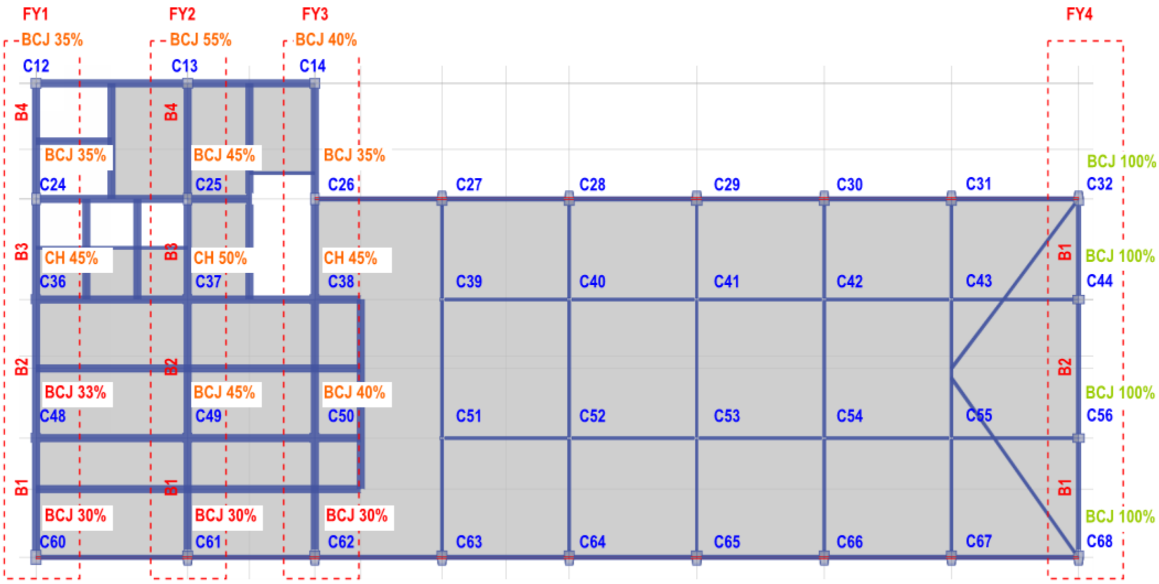
7.1 Stage 1

Stage 1 was more difficult to simplify and split into simple frames than Stage 2, however the same procedure was undertaken. The images below show the governing mechanism for each joint, as well as a score from the modal response spectrum analysis. **Note** these scores do not necessarily represent the final scores documented in the report for the following reasons:

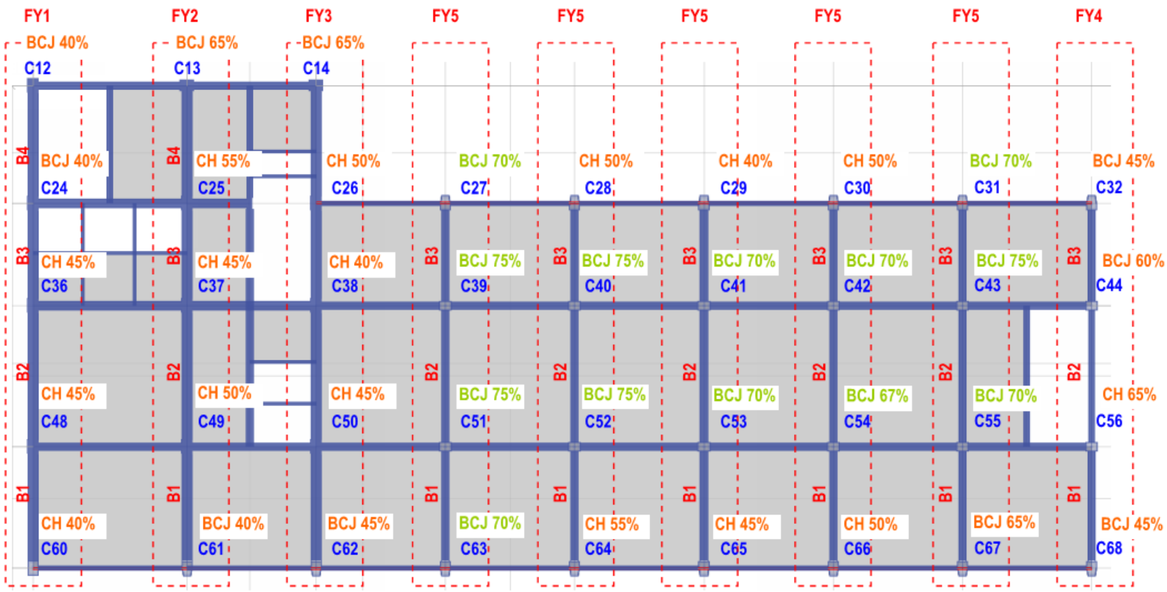
- Where necessary, engineering judgement has been applied to check whether the element has some deformation capacity to allow redistribution to other elements. This is to prevent an unnecessary amount of conservatism to be applied (i.e a beam column joint failure to one joint on a storey is not expected to constitute a complete failure of the building, rather it represents a local failure that may or may not represent partial collapse of the structure).
- Minor changes in score from subsequent modifications to the structural model have not been included in the below images
- Reporting is typically per the recommendations of the Assessment Guidelines (i.e 22 *%NBS* (IL4) = *20%NBS* (IL4)).
- In some instances, the column flexural hinge, and the consequent residual probable shear capacity are similar, and thus CH can be expected to also constitute a flexural-shear failure.

7.1.1 Transverse Direction

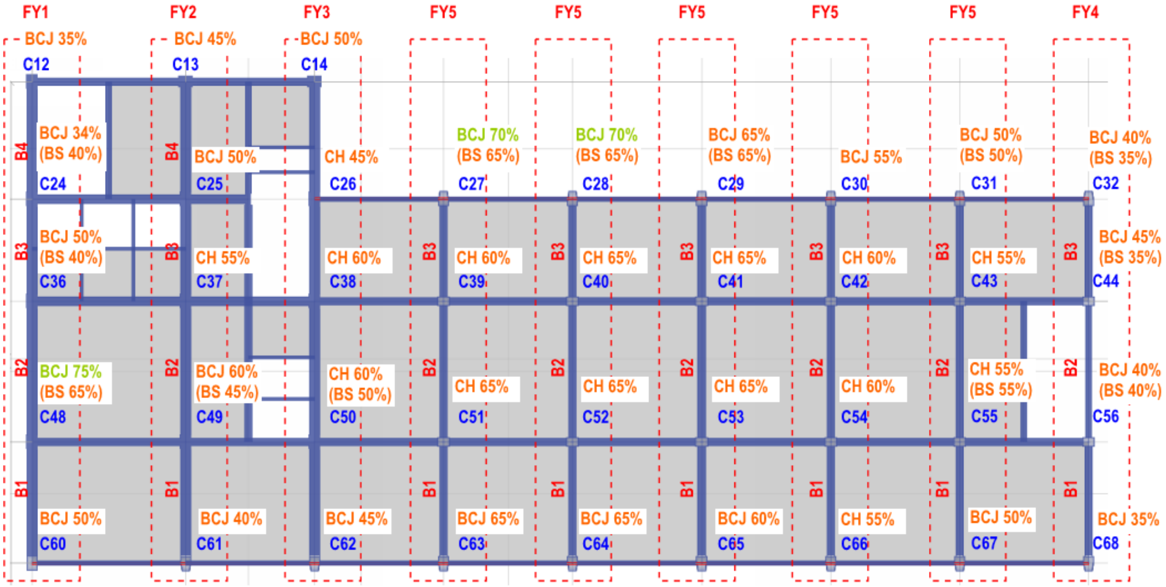
LEVEL 6 - Y DIRECTION



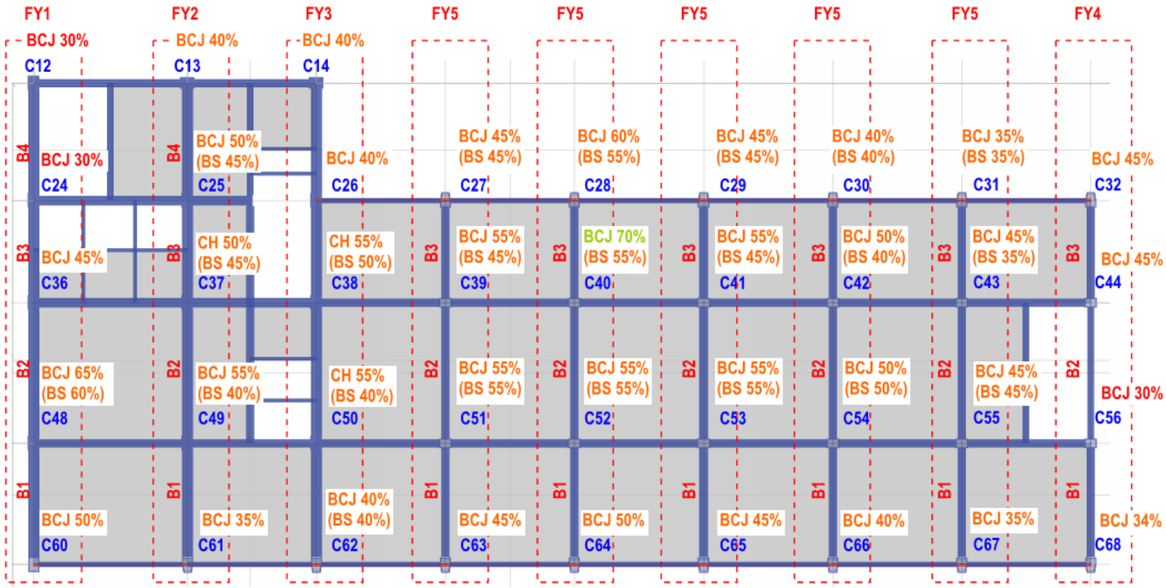
LEVEL 5 - Y DIRECTION



LEVEL 4 - Y-DIRECTION

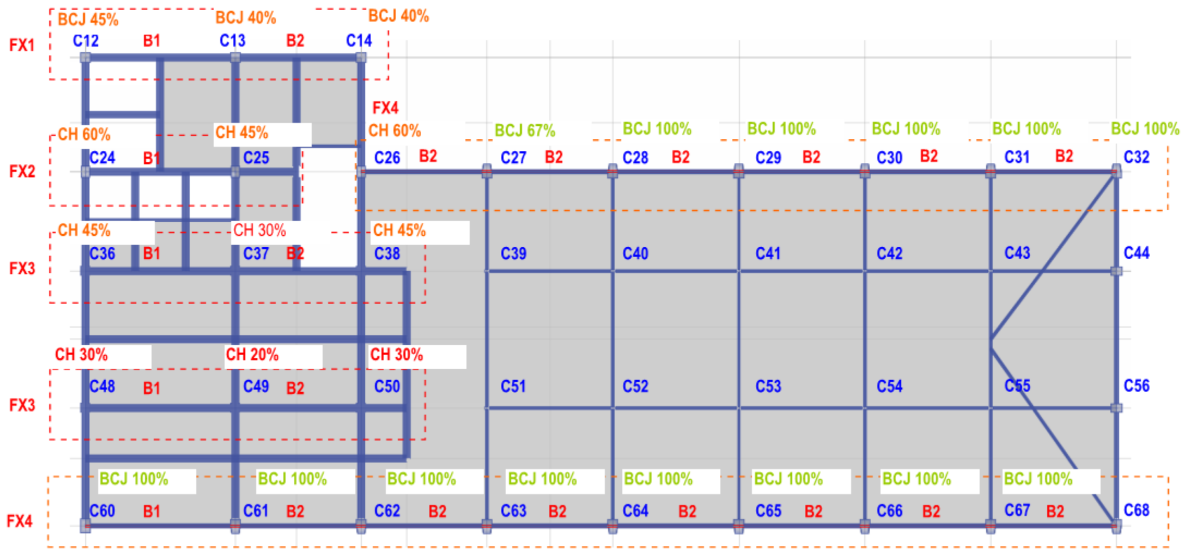


LEVEL 3 - Y-DIRECTION

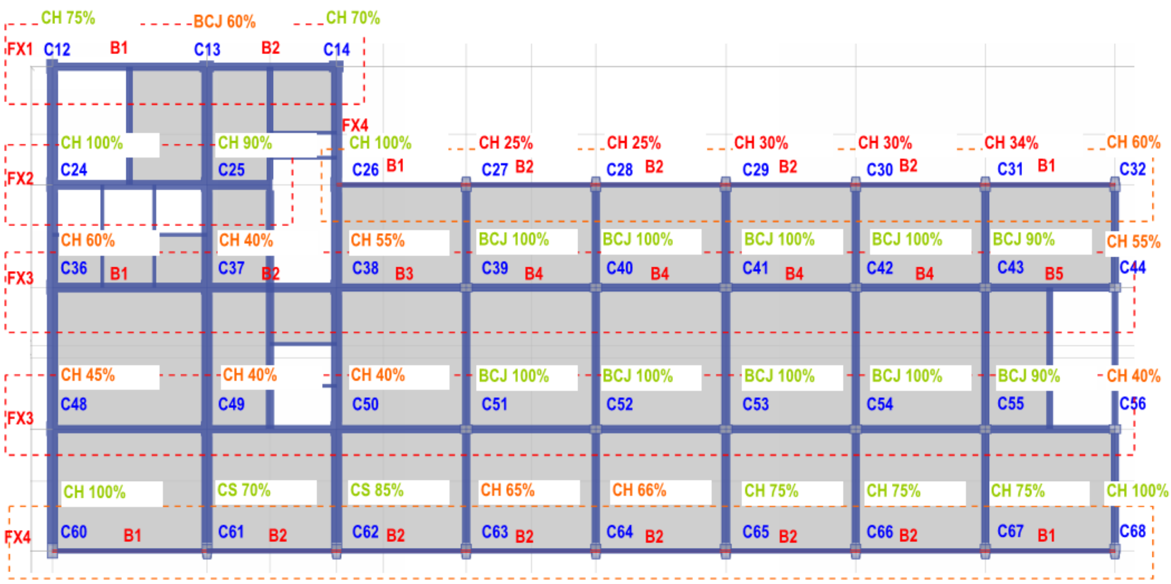


7.1.2 Longitudinal Direction

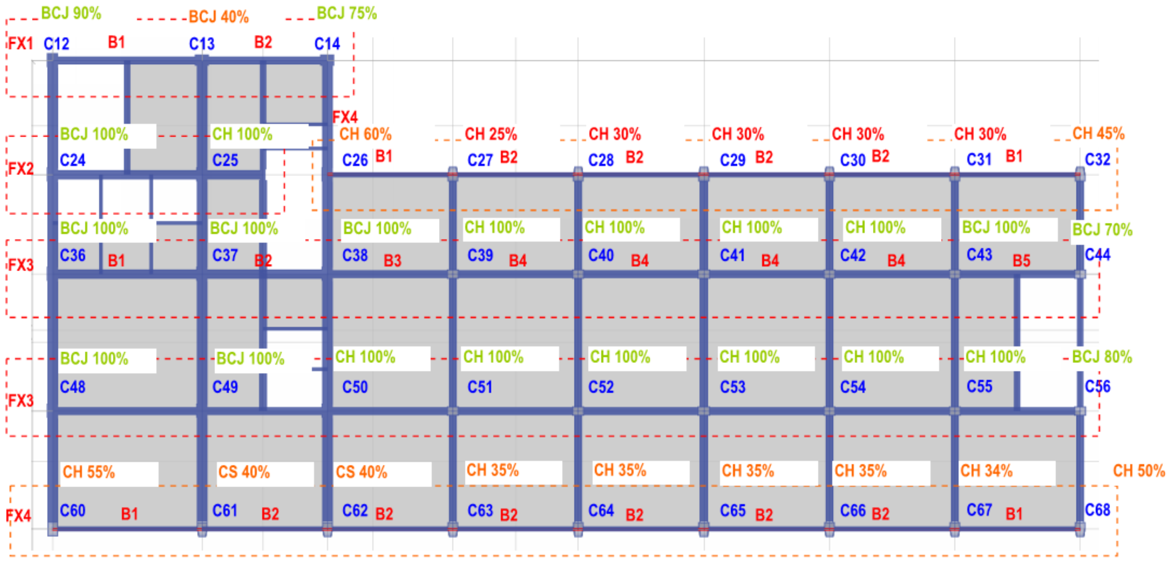
LEVEL 6 - X DIRECTION



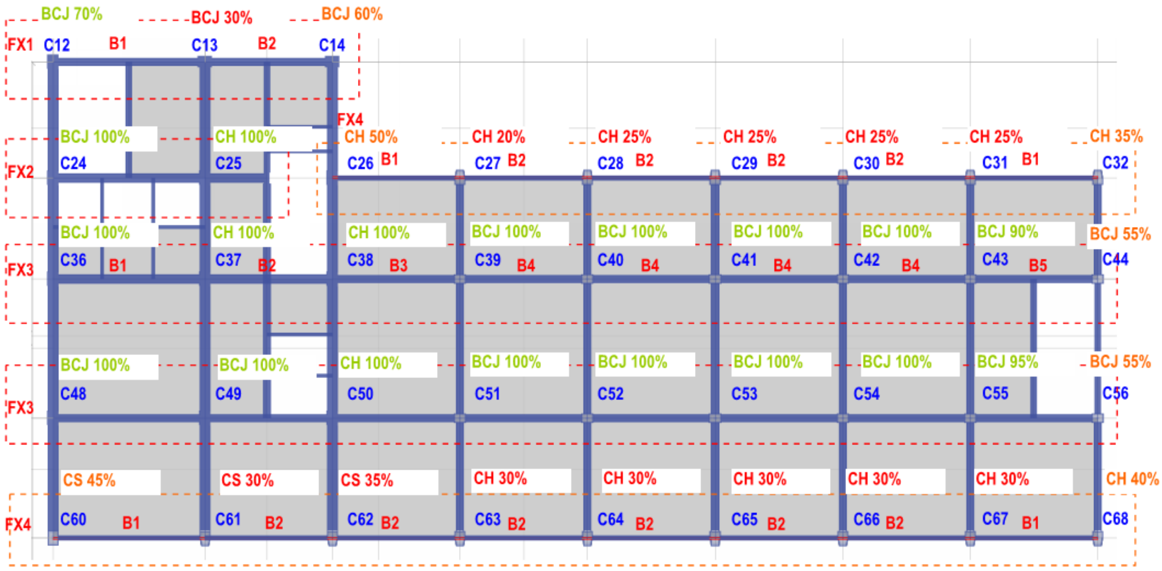
LEVEL 5 - X DIRECTION



LEVEL 4 - X DIRECTION

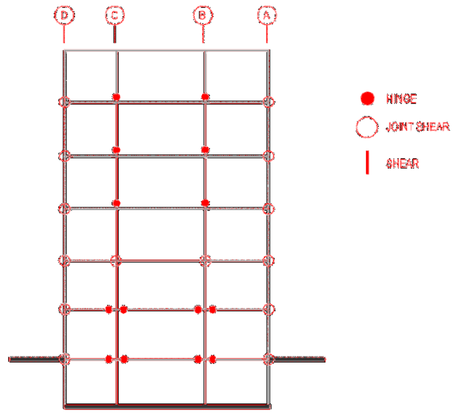


LEVEL 3 - X DIRECTION

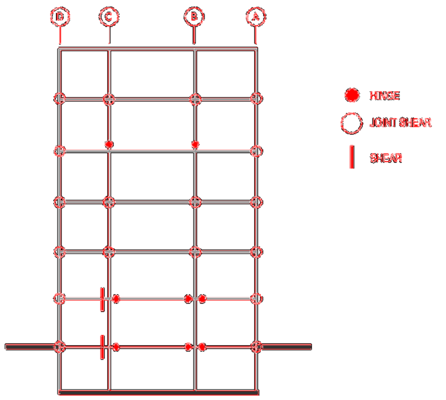


7.2 Stage 2

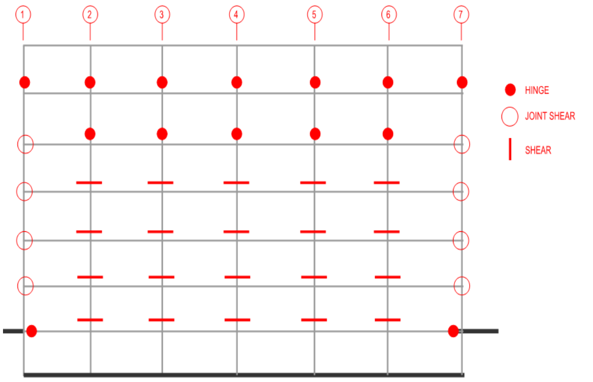
An example of how the structural frame for Stage 2 was split into different frame lines, and then different mechanisms identified is shown below.



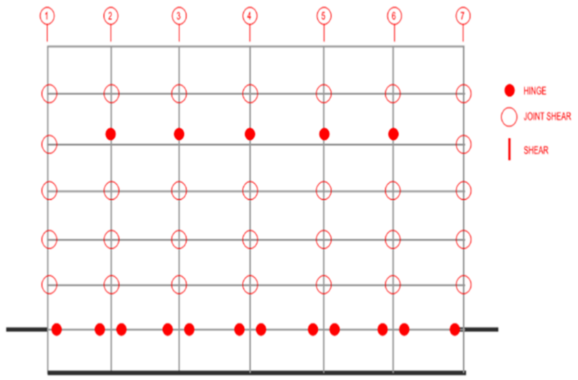
Transverse External Frames



Transverse Internal Frames



Longitudinal External Frames



Longitudinal Internal Frames

Appendix B

Specific Review Comments

Specific Review Comments

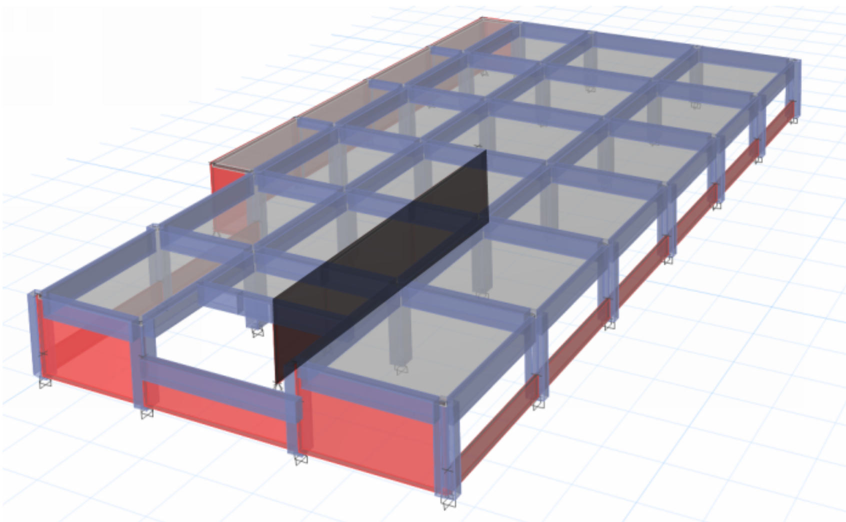
This section highlights some specific technical modelling assumptions arising from our review.

It is intended for Holmes consideration but we do not consider these items to materially impact the building %NBS rating.

Stage 2 Basement Wall

Beca has identified a long reinforced concrete basement wall within the structural drawings. From inspection of the Holmes analysis model, this does not appear to be accounted for.

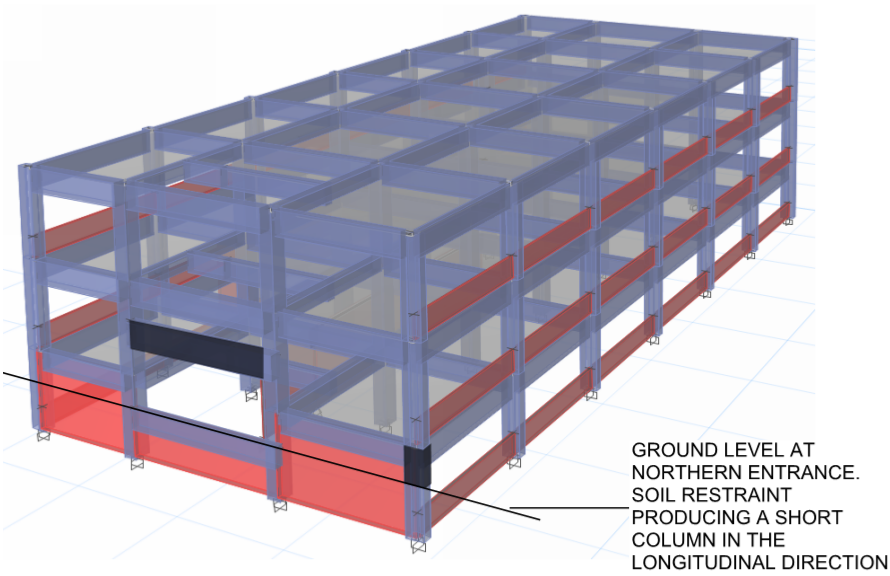
This is not expected to change the reported scores of the structure, however this load path should be considered for any strengthening works undertaken for the building.



Stage 2 Basement Short Columns

The Holmes report does not specifically mention any impact from the possible short column effects on the north end of the building. The analysis model does not appear to account for the structural configuration or boundary conditions at this end.

This is not expected to change the overall score of the structure, however the implications from the as-built geometry and the boundary conditions should be checked for any strengthening works undertaken for the building to ensure undesirable mechanisms are not occurring.


GROUND LEVEL AT
NORTHERN ENTRANCE.
SOIL RESTRAINT
PRODUCING A SHORT
COLUMN IN THE
LONGITUDINAL DIRECTION