Whakatane Civic Building, 14 Commerce Street, Whakatane

Whakatane Civic Building Detailed Seismic Assessment Peer Review

Whakatane District Council

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Contents

1 Executive Summary **1**
2 Peer Review – Parallel ETABS Model Description **2**
3 Aurecon Parallel ETABS model Findings **3**
4 Structure Displacement Ductility **3**
5 Aurecon Independent Force Based Assessment **3**
6 Review of Ground Motion Scaling **4**
7 DSA Report and Calculations Peer Review **4**
8 Peer Review Summary **4**
9 Explanatory Note **5**

Appendices

Appendix A **6**

Figures

Figure 1: 3D ETABS Model 2

Tables

Table 1: Load Summary 2
Table 2. Comparison of Fundamental Parameters 3
Table 3. Summary of assumed ductility vs. NBS% rating 3

1 Executive Summary

Aurecon have been engaged by Whakatane District Council to complete a structural peer review of the Detailed Seismic Assessment (DSA) carried out by Holmes Consulting LP, for the Whakatane Civic building located at 14 Commerce Street, Whakatane.

The Peer Review also included producing a parallel ETABS model and carrying out a Force-Based Assessment in line with the Seismic Assessment Guideline (July 2017 version) to compare results against the Response History Analysis (RHA) method adopted by Holmes Consulting LP. The parallel assessment has provided agreement in the modelling and methodology provided by Holmes that the inelastic procedures are in general appropriate.

Aurecon reviewed the ground motion selection, scaling and suitability of the considered period range of interest and are in general agreement with the methodology.

The DSA draft report and subsequent revisions of the report in addition to some parts of calculations relevant to the queries were also reviewed by Aurecon.

Based on the extent of the analyses Aurecon agrees with the NBS% rating of the primary elements of the superstructure. However, considering the geotechnical characteristics and performance of the foundation, the building may not be appropriate to be used as Emergency Operations Centre (EOC) following a major earthquake event and a methodology to review the building condition after a major event has been provided by Holmes.

The assessment of the flooring system was out of the scope for Holmes for the DSA and at this stage an NBS% is not assigned to the precast flooring system of the building. We understand that the floor systems will be subject to further engineering design and the installation of support systems.

2 Peer Review – Parallel ETABS Model Description

A Parallel ETABS model has been produced to carry out a forced based assessment.

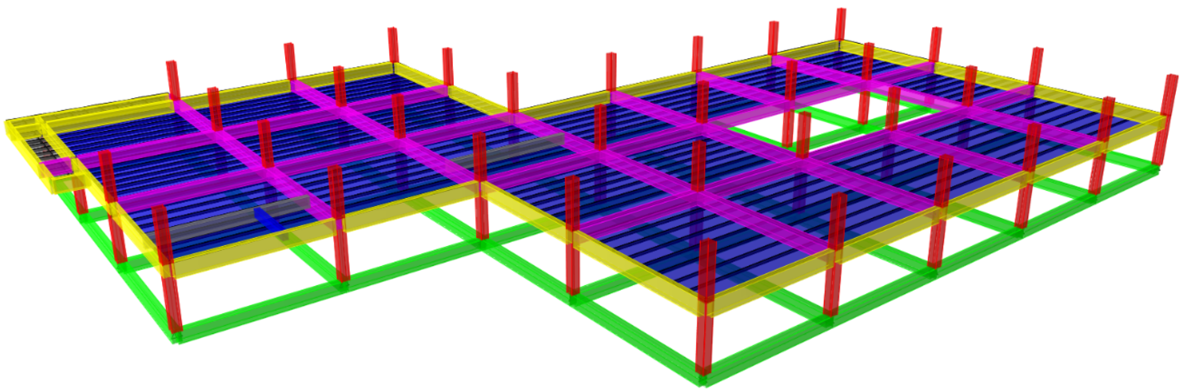

Figure 1: 3D ETABS Model

The Parallel ETABS model seismic coefficient is based on NZS1170.5, using the following parameters:
Importance Level: Considered for both IL2 and IL4
Return Period (ULS): 1/500 (for IL2) and 1/2500 (for IL4)
Site Soil Category, Ch(T): C
Hazard Factor, Z: 0.3
Return Period Factor, Ru: 1.0 (for IL2) and 1.8 (for IL4)
Near Fault Factor, N(T,D): 1.0
Ductility = 2.0, 3.0 and 6.0 considered for sensitivity analysis (column-sway with sufficient plastic hinge zone detailing)
It has been assumed that the building was constructed in accordance with the available original structural drawings, produced by Murray-North, dated November 1989.
Buildings loads used in the ETABS model are in accordance with AS/NZS 1170.1, a summary is shown in Table 1 below.

Table 1: Load Summary

Design Use	Nature of Load*	Uniformly Distributed Load (kPa)
Roof	LL SDL SW	0 0 0.6
First Floor -	LL SDL SW	3.0 1.0 3.6

*LL = Live Load, SDL = Superimposed Dead Load
A seismic load reduction factor of 0.3 was applied to the live load when computing the seismic weight.
The ETABS model is analysed using Response Spectrum Analysis (RSA) based on the following:

- "Cracked" frame stiffness reduction factors
- "Semi-Rigid" first floor diaphragm

- The timber roof is flexible; therefore, roof mass has been lumped at the top of the cantilevering first floor columns
- Building is assumed to have been built as per available original structural drawings, produced by Murray-North, dated November 1989.
- Young's modulus, E, value for concrete and steel of 25GPa and 200GPa respectively.

It is notable that the parallel ETABS model is not expected to provide identical results to the Response History Analysis (RHA) carried out by Holmes Consulting LP. However, it allows a comparable force-based assessment to be undertaken.

3 Aurecon Parallel ETABS model Findings

The findings of the parallel ETABS model are summarised in the Table below and compared against the corresponding values from the Holmes Consulting LP assessment.

Table 2. Comparison of Fundamental Parameters

	Holmes Consulting LP	Aurecon
Period (X-Direction)	0.6 s	0.4 s
Period (Y-Direction)	0.6 s	0.4 s
Drift (X-Direction) IL2	1.00%	0.80%
Drift (Y-Direction) IL2	1.00%	0.90%
Drift (X-Direction) IL4	1.80%	1.50%
Drift (Y-Direction) IL4	1.80%	1.60%
Seismic Weight	16,800 kN	17,187 kN

The summarised results obtained from the independent ETABS model are comparable with those from the original assessment carried out by Holmes Consulting LP.

4 Structure Displacement Ductility

The structure displacement ductility factor is an important consideration in a forced based assessment. While The July 2017 version of the Seismic Assessment Guideline proposed the SLaMA method, it provides limited guidance on the selection of the appropriate ductility factor based on the element detailing. The previous NZSEE guidelines, dated June 2006, provide prescriptive rules and equations to determine the ductility, taking member detailing into account. Based on the June 2006 guidelines, the building easily meets the ductility 2 requirements, however, falls short of the ductility 6 requirements, therefore, the ductility is expected to be somewhere between 2-6. This Peer Review has included a sensitivity analysis with forces assessed for ductility 2, 3 and 6.

5 Aurecon Independent Force Based Assessment

Aurecon have carried out an independent force-based assessment in line with the NZSEE Guidelines (July 2017 Version), a summary of the overall seismic rating found for both IL2 and IL4 assessed for ductility 2, 3 and 6 systems is summarised below, expressed in terms of Percentage of New Building Standard (% NBS).

Table 3. Summary of assumed ductility vs. NBS% rating

Importance Level	Ductility	%NBS
IL2	$\mu = 2$	70
IL2	$\mu = 3$	110
IL2	$\mu = 6$	200
IL4	$\mu = 2$	40
IL4	$\mu = 3$	60
IL4	$\mu = 6$	110

Holmes Consulting LP assessed the building to have a seismic rating of 90% NBS and 110% NBS when assessed as an IL4 and IL2 building respectively. The Holmes values of 90% NBS for IL4 is consistent with the results from the force-based assessment using a ductility value in the order of 5. The Holmes value of 110% for IL2 is consistent with the results from the force-based assessment with a ductility demand value of 3.

6 Review of Ground Motion Scaling

Holmes engaged Bradley Seismic Ltd. to advise on the selection of the ground motions. Suite of the 13 selected ground motions were scaled in accordance with the ASCE41-17 advised method. While the lower bound of the period range (0.24 Sec) is selected based on NZS1170.5, the upper bound (1 Sec) is selected based on ASCE41-17 advises.

Aurecon has carried out independent scaling of the selected ground motions based on NZS1170.5 method over the above period range of interest. While the scaling methods of ASCE41-17 and NZS1170.5 have a fundamental difference, it was observed that in most of the cases the scaling factors are comparable. Therefore, Aurecon agreed with the period range of interest and adopted scaling method. Further details of the queries related to the ground motion scaling are available in Appendix A.

7 DSA Report and Calculations Peer Review

Aurecon peer reviewed the DSA report and some parts of calculations related to the peer review queries for the building at 14 Commerce Street, Whakatane. The initial draft of the report was dated 9th August 2018 and revised three times based on the peer review queries. The latest draft of the DSA report is dated 18th January 2019. The complete set of the peer review queries is attached as Appendix A to this report.

8 Peer Review Summary

Summary of the key points of the peer review are as follows:

- Based on the extent of the analyses performed by Holmes Consulting Group the primary lateral resisting system of the superstructure has a rating of >100% NBS (IL2) and 90% NBS (IL4).
- The continued functionality (SLS2) requirements was outside of the initial scope of engagement of Holmes Consulting LP. The building may not be able to be used as Emergency Operation Centre (EOC) following a major earthquake event. Whakatane District Council is advised to consider an alternative EOC building.
- Following a major earthquake event, the building should not be re-occupied immediately. It is required that the building be assessed for the state of damage. The required scope of the building inspection and assessment is described in section 4.3.2 of Holmes DSA report.
- An assessment of the precast flooring system was excluded from Holmes scope of the work. While a limited commentary is provided in Section 5.3 of the latest draft report, no NBS% rating is assigned to the precast flooring system. Appendix C5E of the latest revision of the Section C5 of the assessment guideline published on 31 November 2018 has provided clarification on the assessment method of the precast flooring system. It is recommended to assess the performance of the units.
- It is understood that catcher frames are being installed for the precast flooring units. While this may not directly affect the NBS% rating of the flooring units, it potentially educes the life safety risk for the building occupants resulting from loss of support for the floor units.
- A major part of the queries was related to geotechnical information and performance of the foundation. It is notable that the pile performance and soil-structure interaction were not assessed in detail. Aurecon believes that the primary structural components of the superstructure have sufficient capacity to provide life safety for the building occupants during major earthquake events. However, based on the geotechnical information provided, consideration of soil-structure interaction and liquefaction potential

beyond IL2 ULS level, may result in change in the reported structurally dominant behaviour of the building.

- We appear to have a philosophical disagreement with Holmes Consulting LP on the effect of liquefaction. While Holmes classify the liquefaction as post-earthquake phenomenon, Aurecon believes that liquefaction may occur during the earthquakes with sufficient long duration and certain characteristics. It should be notices that a certain number of the suite of the ground motions considered for analysis of the building are subduction zone earthquakes. These earthquakes may have long duration that trigger liquefaction during the event.

9 Explanatory Note

- This assessment contains the professional opinion of Aurecon as to the matters set out herein, in the light of the information available to it during preparation, using its professional judgment and acting in accordance with the standard of care and skill normally exercised by professional engineers providing similar services in similar circumstances. No other express or implied warranty is made as to the professional advice contained in this report.

- We have prepared this report in accordance with the brief as provided and our terms of engagement. The information contained in this report has been prepared by Aurecon at the request of its client Whakatane District Council and is exclusively for its client's use and reliance. It is not possible to make a proper assessment of this assessment without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Aurecon. The assessment will not address issues which would ned to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this assessment by any third party.

- The assessment is also based on information that has been provided to Aurecon from other sources or by other parties. The assessment has been prepared strictly on the basis that the information that has been provided is accurate, complete and adequate. To the extent that any information is inaccurate, incomplete or inadequate, Aurecon takes no responsibility and disclaims all liability whatsoever for any loss or damage that resulting from any conclusions based on information that has been provided to Aurecon.

Appendix A

Whakatane Civic Building Peer Review Queries

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Project: Whakatane Civic Building DSA
Job No: 502828-V01-01

Date: Reviewers Comments: 28/01/2019
Designers Response: Click or tap to enter a date.

Review By: Mehrdad Seifi (aurecon)

Response By: Jeff Clendon (Holmes)

Review Query Sheet No.: 6

No.	Document Reference	Reviewers Comment	Designers Responses(s)	Agreed Close-Out Action	Status
1	Whakatane CB-DSA GMs Selection and Scaling P.4	As per the designer comments the period range of interest is understood to have been selected based on NZS 1170.5. The upper bound used by Holmes appears to be 1.5T. Referring to NZS1170.5 Cl.5.5.2 (iii) the upper bound of the period range of interest is 1.3T1. **Please clarify why 1.5T is used, and whether scaling over this extended range is significant given that neither ASCE41-17 or NZS1170.5 have strictly been followed.**	Scaling is generally in accordance with ASCE 41-17. This is to maintain consistency with the assessment approach utilising NLTHA using ASCE 41 tier 3 assessment procedures. The lower bound value for the period range of interest was adjusted from 0.2T as specified in ASC41-17, to 0.4T as specified by NZS1170.5. This was in recognition of the excessively conservative effect that scaling to the NZS1170.5 target spectra introduced to the scaled records, given the shape of the low period corner of the NZS1170.5 spectral shape. This approach is outlined in our letter to Aurecon of 23 July 2018, to which agreement in principal was given by Aurecon at the time.	In the letter dated 23rd of July it is mentioned “NZS1170 defined the period range of interest as between 0.4T and 1.5T” while in fact NZS1170.5 considers 1.3T1 as the upper bound. It is clarified that you selected the lower bound from 1170.5 and the upper bound is selected from ASCE41-17. Considering my earlier discussion at the strengthening phase (Review dated 22nd June 2018) I am	Closed.

				happy with shifting the lower bound to 0.4 Sec.	
2	Whakatane CB-DSA GMs Selection and Scaling P.2	We have calculated the scaling of the ground motions as per the requirements of NZS1170.5. A comparison of the relative scale factors is shown attached. While in most of cases scaling factors are comparable, there are some instances where the values of the scaling factors from NZS1170.5 are larger than the adopted values. Please confirm that the rating of the building remains unchanged using the scaling factors based on 1170.5 and considering the acceptance criteria of ASCE41-17. (we note that the acceptance requirements of ASCE41-17 requires in assessment that the mean of the maximum design actions from the suit of records is considered.	Our assessment has utilised NLTHA using ASCE 41-13 tier 3 assessment procedures, as per clause C1.6.2 of the NZSEE guidelines. Scaling of the selected ground motion records, with the period range of interest modified as discussed previously, has been undertaken to the alternative rational procedures outlined in ASCE41-17, assessed at the average response from 11 sets of ground motion records. Scaling to the procedures of NZS 1170.5 should be used in conjunction with the “maximum of three” ground motion records. This is a different approach and is not compatible with the current assessment.	noted	Closed
3	Whakatane CB-DSA GMs Selection and Scaling Ps.3 and 4.	Despite that it is mentioned that the upper bound is considered as 1.5T, In Figures 1 and 2 the dashed line of the upper bound is shown on 1.0 Seconds. (0.6sec x 1.5 = 0.9sec). **Please Justify why 1.0 second is considered as the upper bound.**	As the scaling procedure is generally in accordance with ASCE 41-17, the upper bound of the period of interest is set by the provisions of ASCE 41-17. ASCE 41-17 notes that this upper bound period shall be at least 1.5T, but not less than 1 second	As per ASCE41-17 Clause 2.4.3. (3). Agreed, Closed	Closed.
4	DSA Report, Executive Summary and Assessed Seismic Rating (Page i)	**IL4 usage** We understand that SLS2 review is not part of the brief. But we would comment that if the building is to be adopted as an IL4 importance structure, then this should be undertaken. There are concerns of the performance of the slabs and piles where the liquefaction may occur at an IL2 ULS event.	Agreed in principal. The client is aware of the performance issues in relation to SLS2 loading and IL4 functionality. Refer also to comment below in relation to foundations and geotechnical concerns; these aspects are being reviewed. The status of SLS2 performance will be clarified in the DSA report executive summary and within the body of the report. Agreed at the meeting on 17/12/18 as follows: - The continued functionality (SLS2) requirements fall outside of the %NBS rating.	Based on the information provided in Clause 4.3 of the T&T geotech report and insufficient data on SLS2 performance Aurecon is not able to comment on SLS2 performance of the building. [Rev 6]. Executive summary, Page iv and section 4.3 have been	Closed

			- Report to be updated to more clearly convey to the Council potential circumstances under which the building may not be re-occupiable following significant earthquake shaking. - Reference is to be made in the report to the need for operational protocols and arrangements to be prepared in relation to review for evidence of foundation damage resulting from ground movements, such as liquefaction, displaced ground etc. Report updated to reflect the above.	updated addressing the geotechnical concerns. Sections 4.3.1 on liquefaction and associated Effects on Foundation performance and 4.3.2 on Post-Earthquake Building Review are added to Rev. 4 of the report. As stated earlier in Q30, we do not consider the liquefaction as Post-Earthquake phenomenon. Apart from that considering, adding sections 4.3.1 and 4.3.2 on the building performance as IL4 aurecon is happy to close this query and others related to geotechnical issues.	
5	DSA Building description (Page ii)	**Flooring System** We understand that the performance of Double-Tee units and flooring system has been excluded from the brief. However, we would have thought that this should be included to inform the client of the floor performance for different importance levels.	The performance of the flooring has been previously highlighted to the client. As a separate exercise, secondary floor catcher frames are being installed to mitigate risks associated with floor seating. This strengthening work is reiterated in the building description section of the DSA report. Agreed at the meeting on 17/12/18 as follows: - the assessment of the precast flooring system has been excluded from Holmes' scope of work.	Even though we understand that there will be a catch frame system, we would have thought that comprehensive statement of the precast floor performance addressing the different potential failure mechanisms	Closed

			- An assessment to the recently revised Appendix C5E of the Assessment Guidelines would have highlighted potential issues and raised awareness of the rating of these elements. This is worth a further but limited review. Appendix C5E.6.2 (loss of support) indicates that the primary causes of loss of support are related to beam elongation and spalling from the support ledges and the back face of the floor units. Other effects such as construction tolerance, creep, shrinkage, and thermal movements contribute less than a third of the potential total loss of seating length. Given that the lateral system of the building behaves with a column sway mechanism, there is no effective beam plasticity or beam elongation, and the rotational demand between the beam and the seating is relatively modest (elastic only) over the 115mm depth of the floor seating detail. An initial review of the seating capacity indicates that loss of support is not a risk at the lateral capacity of the building. No details of the precast floor unit reinforcing are currently available to allow a flexural check to C5E.6.3, although again, there is nominal rotational demand on the floor seating detail. Some commentary has been added to the report.	prior to strengthening would be beneficial. [Rev 6.]. while the column sway mechanism is reported as the governing behaviour mechanism, Figures 26 and 27 shows the local formation of the hinges in a beam beyond CP level. Page S12 of the drawings also shows the direction of the flooring units are parallel to the beam with hinge formation. Section 5.3 of the report is revised to address concerns on the performance of precast flooring system. While aurecon is unable to confirm the NBS% rating reported in section 5.3.1, installation of catch frames may minimize the Life safety risk for the building occupants.	
6	DSA Assessed Seismic Rating (Page iv)	**Typo** The typo shows the rating for IL2 to be 1100% NBS. Please amend it to 110%.	Will be corrected.	Checked, Corrected	Closed
7	DSA Assessed Seismic Rating (Page iv)	**Structural Weakness** According to Assessment Guideline, Structural weakness is an aspect of building that scores less than 100%NBS. So, when the building is scored 110% NBS we do not believe it needs	Agreed. The 110% IL2 rating of the column-sway mechanism has been included to provide context to that mechanism under the IL4 scenario. The executive summary of the DSA report will be revised to clarify both the IL2 and IL4 scenarios.	Corrected on page v of Rev.2 of the report.	closed

		to be classified as a structural weakness. Please Comment.			
8	DSA Assessed Seismic Rating and method of assessment, (Page v)	**Reference Documents** While the Assessment Guideline is referring to ASCE 41-13, the document is superseded by its successor ASCE 41-17. Also, the earlier discussion for GM scaling was based on ASCE41-17. In Page v of DSA the ASCE41-13 is mentioned as the reference. Please clarify which version of the code is used. In the method of assessment again ASCE 41-17 is being referred to. The DSA makes reference to 41-13 and 41-17 and appears to "mix and match" the two standards. Can you please advise the impacts of not consistently utilising one standard all the way through?	The NLTHA ASCE41-13 tier 3 assessment approach has been used as outlined in clause C1.6.2 of the NZSEE guidelines. Assessment procedures, including backbone curves and inelastic rotation limits, are based on the provisions of ASCE41-13. Only the GM selection and scaling is based on the provisions of ASCE41-17 as an alternative procedure with rational basis. The ASCE41-17 scaling procedure is preferred due to the better representation of demand from the mean of 11 records, its use of maximum direction, and in our opinion is "best practice". This will be clarified and updated in the DSA report.	While the assessment guideline refers to ASCE41-13, we believe that considering the changes in the acceptance criteria in the latest version of ASCE41, the "best practice" always will be referring to the latest standard versions.	Closed
9	DSA Section 4.3, Appendix B Section 7.3.1 Appendix D Sections D.2.5 (Pages 6,20 and page 9 of the T&T geotech Report)	**Foundation and Geotechnical Information** On page 6 it is stated that "*Their review has identified several geotechnical risks that do not directly impact the performance of the primary structure under the ULS loading, but that do have potentially significant implications to the building's performance"* Several factors including liquefiable soil underneath the building and limited capacity of piles under the liquefied condition are addressed. 1. The building performance is investigated for the MCE IL2 and IL4. 2. The geotechnical statement indicates that beyond IL2 level the performance of the structure is not "structurally dominant". *3.* In Section 7.3.1 of T&T report it is stated that: "*Without mitigation measures, the building foundations are very likely to meet ULS (IL2) performance criteria and are unlikely to meet SLS2 (IL4) performance unless the deformations can be accommodated, and cyclic*	Section 7.3.1 of the geotechnical report states that the foundations are *very likely* to meet the IL2 ULS strength demands but are *unlikely* to meet the IL4 SLS2 performance requirements. Although SLS2 performance is currently outside the scope of the DSA, the DSA report notes the various geotechnical issues raised by the geotechnical report and indicates that the building will not be suitable as an IL4 facility following significant earthquake shaking. Section 7.3.1 of the geotechnical report goes on to say that post-shaking foundation performance *is likely to allow the building to be evacuated without collapse (i.e. meet ULS (IL2) performance requirements), provided the superstructure performs adequately* Given that assessment to the NZSEE Guidelines is concerned with significant life safety hazards, and SLS2 performance is not directly related to life safety hazards at all, we don't feel there is a contradiction between the building being structurally dominated, and the statement that the building is unlikely to meet SLS2 performance requirements due to geotechnical effects. Agreed at the meeting on 17/12/18 as follows:	Considering the likelihood of the damage to the piles beyond DBE (IL2), the liquefaction potential and insufficient data about pile performance, we are unable to confirm that the building performance for DBE/MCE(IL4) is structurally dominated. While we agree that life safety issues for IL2 are realistic, we still believe that the performance of the foundations and in particular the piles has not been established. [Rev 10]. Refer to Q4	Closed

		displacement does not cause significant damage to piles." This contradicts with the assumption mentioned in the last paragraph on page 7. Please clarify.	- Aurecon as peer reviewers are satisfied that the ratings of >100%NBS at IL2 and 90%NBS at IL4 are appropriate from a life safety perspective. - No further assessment of foundations to be undertaken. - Clarification to be provided around post-shaking foundation performance. Refer query item 4 regarding increase clarity in report around potential impact on reoccupation. Report updated to reflect the above.		
10	DSA Section 4.3 and Appendix D Sections D.2.5 (Pages 6 and 20)	**Foundation and Geotechnical Information** It is stated that piles are modelled "pinned top and bottom". Based on Figure 8 the reinforcement of the piles is well developed into the ground beam and development length is provided. Also, it is mentioned that *"The geotechnical engineer has identified the piles are likely to experience cyclic loading induced bending with resulting post-earthquake reduction in pile carrying capacity"* Therefore, the piles cannot be considered as pinned at the top. We expect that these should be modelled and performance of the piles under cyclic loading determined.	The analysis model has not included lateral soil structure interaction. Lateral load takeout from the model is via the ground floor slab level, with only vertical loading taken through the piles into vertical restraints at the bottom of the assumed pile depth. The assumption of pins at the top of the piles was to transfer all bending actions from the column base into the foundation beam system. This provides a conservative demand on the foundations beam for their assessment. We agree that some of the column base moment will be transferred into the top of the piles. As noted above, the geotechnical report states that the foundations are likely to allow the building to be evacuated without collapse. The piles below the ground floor columns are octagonal in shape, with a maximum dimension of 400mm, are reinforced with D16 bars longitudinally, and have spiral reinforcing at a 50mm pitch throughout and at 25mm pitch at the top and bottom ends. We don't believe that additional modelling of the pile behaviour will identify any significant life safety hazards, and don't propose to carry out any additional modelling. As for query item 9 above - considered closed.	With the status of the information provided, we are unable to provide comment on the piles' performance during a DBE/MCE level earthquake. The response provided to Q24 indicates formation of hinges next to the ground beam-pile cap joints. Full modelling which incorporates the piles will result in different load distribution and the location of hinge formation may change. [Rev 10]. Refer to Q4	Closed
11	DSA Section 4.3 and Appendix D	**Foundation and Geotechnical Information** It is understood that the soil-structure interaction is not considered. It is mentioned the building response is structurally dominant,	The limiting mechanism of the building superstructure is the column-sway mechanism that develops in the ground floor columns.	We disagree on this matter of the liquefaction and effect on the building.	Closed

	Section D.2.5 (Pages 6 and 20)	Consistent with the above comment, please show that the performance of the piles at MCE level (IL2 and IL4) is satisfactory considering the risk of liquefaction.	This mechanism limits the building's lateral earthquake strength for both the IL2 and IL4 levels of shaking, based on plastic rotation limits in the plastic hinges of the ground floor columns under MCE levels of earthquake shaking. Liquefaction is caused by the increase in pore water pressure due to earthquake shaking, which in turn causes the soil particles to lose contact with each other and effectively behave like a liquid. This process takes time to develop within the soil structure and is generally considered to be an issue following earthquake shaking rather than during earthquake shaking. Given the statements in the geotechnical report in relation to ULS performance at the IL2 level of shaking, and given the ductile detailing of the piles themselves, we believe that the pile performance will be satisfactory during the IL2 and IL4 levels of earthquake shaking, up to the limit of the superstructure performance as noted in the DSA report. As for query item 9 above - considered closed.	Liquefaction may occur during the earthquakes with long durations. The Kaikoura earthquake lasted for nearly two minutes. In addition, the subduction zone earthquakes may have quite long durations of shaking. We note that the location of the hinges currently formed may change by proper modelling of the piles and consideration of the unsupported length of the piles due to the impact of liquefaction in the soil strata. [Rev 10]. Refer to Q4	
12	DSA site observation (Page 10)	**Data Collection Compliance** With reference to Section 6.2.4.3 ASCE41-17, where nonlinear procedures are used data collection consistent with either the usual or comprehensive levels of knowledge shall be performed. While Response History Analysis (RHA) is used for structural analyses the data collection requirements are minimum as per the table 6.1 of ASCE41-17. This level of data collection is only applicable for LSP or LDP analysis. Please comment.	(As noted elsewhere, the assessment is based on the provisions of ASCE 41-13) We acknowledge the prescriptive requirements for data collection in ASCE 41, however note that as this assessment is being completed under the NZSEE framework, we have applied the NZSEE recommendations as to what comprises an appropriate level of investigation to accompany a DSA. Probable material properties in accordance with the recommendations of the NZSEE guidelines are used for the assessment.	Noted.	closed
13	DSA Section 7.3 and Appendix D (Page 10 and	**Discrepancy of Reference Documents** It is understood that the ASCE 41 is used for the assessment.	The NZSEE guidelines are used for the assessment, utilising the NLTHA ASCE41-13 tier 3 assessment approach as outlined in clause C1.6.2 of the guidelines.	We believe that there is a discrepancy between the	Closed

	Appendix D information)	On the other hand, it is mentioned that MCE/ULS ratio is in accordance with Table C1.1 of Engineering Assessment Guideline. There is inconsistency between Engineering Assessment Guideline and the ASCE41 methodology. If the backbone curves are defined for concrete members based on ASCE tables then the ratio of CP to LS is approximately 1.5 and not 1.8. As per the Assessment Guideline MCE performance levels for IL2 and IL4 refers to 2500-year and 10,000-year return periods respectively. Please clarify how the two sets of records are utilised for analyses.	Clause C1.6.2, and in particular table C1.1 within this clause, provides the ground motion scaling factors to be applied to the ground motions used for the life safety assessment. Table C1.1 sets the reference return periods defining the target earthquake shaking for 100% NBS life safety performance. For IL2, 500-year return period, and for IL4, 2500-year return period. For our assessment, two completely separate assessments were carried out. One for IL2 utilising earthquake records appropriate for a 500-year return period for life safety assessment, scaled by 1.8 for collapse prevention assessment. The second for IL4 utilising earthquake records appropriate for a 2500-year return period for life safety assessment, scaled by 1.5 for collapse prevention assessment. Both assessments comply with the recommendations of clause C1.6.2 and table C1.1 of the NZSEE guidelines. We note that the ratio of LS to CP limits is not immediately relevant to the ratio of demand input to be paired with each performance level. In fact, they are expected to be different, which is why the two performance levels need to be considered. LS and CP limits are on the capacity side, LS simply giving acceptance limits with a higher level of reliability than CP. The limits from the appropriate performance level are to be compared against the paired demands as set by C1.6.2. There is no inconsistency.	documents. As discussed in the meeting on 23rd Nov. 2018, Jeff Clendon also agreed that 1.5 better represents the margin between the Life Safety and Collapse Prevention. Clause C1.5.1 of the assessment guideline defines the %NBS based on DBE(ULS) level. While in general there is a margin in the order of 1.5 between LS and CP in elements governed mainly by flexural behaviour, it is evident that increasing the demand by 80% results in MCE level to govern the performance of the building. Refer to Page 15 of Boys et al paper written by Senior Technical Staff at Holmes. Considering that currently a bigger demand is being applied on the building we propose to conclude the	

				discussion about the MCE/ULS factor.	
14	DSA Section 7.3 and Appendix D Section D.3 (Pages 10, 23 and 24)	**Discrepancy of Reference Documents** Please confirm that to study the 100% MCE performance of the building, the collapse prevention factor, as recommended by table C1.1 of the Seismic Assessment Guideline is multiplied by the values reported as Scale Factor in Tables 23 and 24.	Correct. The collapse prevention assessment was undertaken by scaling the life safety earthquake records by the scale factors from table C1.1 as noted above. 1.8 for the IL2 assessment, and 1.5 for the IL4 assessment.	Noted	Closed
15	DSA Section 7.3 and Appendix D Section D.2. 3 (Pages 10, 14-19)	**Discrepancy of Reference Documents** The Assessment Guideline Table C1.1 and The ASCE41 do not have the same ratio between the life safety and collapse prevention acceptance criteria. As ASCE 41 values are utilised to define the backbone curves, and Table C1.1 is used for the MCE/ULS ratio, please justify their inconsistency?	As noted above, the assessment has been undertaken to the provisions of the NZSEE guidelines, which outline the recommendations to be adopted when utilising NLTHA using ASCE 41-13 tier 3 assessment procedures. These recommendations have been incorporated in the assessment, except for full consideration of SLS2 performance, which was specifically excluded from the assessment brief by the client. There is no inconsistency with this approach. The ratios should not be expected to be the same.	As noted for Q13	Closed
16	DSA Section D.3 (Pages 23 and 24)	Considering that the ground motions suites recommended for 500-year and 2500-year return periods are different please advise how these ground motions are appropriate to be used for other return periods resulting from applying the table C1.1 Scale factors.	Table C1.1 of the NZSEE guidelines outlines the scaling factors that are to be applied to the ground motions used for life safety assessment. These factors have been applied in accordance with the NZSEE guidelines.	Noted	Closed
17	DSA Section 7.3 and Appendix D Section D.3 (Pages 10, 23 and 24)	**Concurrency of seismic actions** Please confirm that concurrent seismic actions are being considered in application of the suite of the ground motions.	Yes, the two perpendicular components of each earthquake shaking record pair are applied simultaneously for each of the eccentricity runs. No. There are four sets of analysis runs for each earthquake record with loads applied at the COM offset by the specified mass eccentricity, i.e. lateral shaking loads applied at the four points offset from the calculated COM by +/- the mass eccentricity in each direction. Thus, there are four eccentricity runs for each pair of earthquake shaking records.	Please clarify the “Eccentricity Run” Term. Is this referring to Concurrency? [Rev 6]. Noted	Closed

18	Appendix A Page A-4, recommendations	**Necessity of Strengthening** While the building is rated as 110%(IL2) it is stated that "*strengthening to 100%IL2 may be impracticable*", This appears that should be 100%(IL4).	Agreed, this statement should be made regarding the IL4 level of shaking. Corrected.	Noted	Closed
19	DSA Section 8 and Appendix A (Pages 11 and C-9)	Considering considerable difference between the results of SLaMA and RHA analyses can please explain the reasons of the discrepancy.	The SLaMA provides a simplified check of the results of the more detailed and accurate NLTHA assessment approach. Simplifications and conservative assumptions in the SLaMA have led to the lower and more conservative results.	Noted. We believe that where sophisticated analysis like RHA or pushover is being used the SLaMA does not provide valuable insight.	Closed
20	Appendix C Page C-7 and C-9	**Ultimate Displacement for SLaMA** Please clarify how the ultimate displacement capacity is calculated for the structure.	The ultimate displacement capacity was determined as the minimum of 2.5% drift as per AS1170.5 drift limits, or the deflection based on a maximum plastic rotation demand of 0.045 radians, being the LS performance limit form ASCE41-13 table 10-8 for a condition i column with P/Agf'c <0.1 and ρ>0.006	Please refer to the comment above. It is noticeable that Table 10-8 of ASCE41 is updated in ASCE41-17.	Closed
21	Appendix C Page C-8	**Hysteretic Damping** The hysteretic damping level adopted is "Medium". The bars in the drawings are deformed. Please clarify why Medium damping is utilised? Please refer to Figure 13.	"High" damping may be justified based on deformed bars within concrete frames as per Table C2D.1 of the NZSEE guidelines. "Medium" damping provides a conservative result (lower capacity). Given the assessment is based on the NLTHA analysis, and the SLaMA is just providing a global check on the results, this conservative approach has no effect on the building strength reported in the DSA.	Noted	Closed
22	Appendix D Section D.1.1 (Page 11)	**Secondary Structural Elements** It's stated that "*Secondary structural elements are modelled, and their performance directly assessed and evaluated. Secondary structural elements are not directly modelled"*, Please clarify as this appears contradictory.	Agreed, this statement has been reworded to read "Primary structural elements and key secondary structural elements are modelled, and their performance directly assessed and evaluated. Other secondary structural and non-structural elements are not directly modelled."	Noted	Closed
23	Appendix D Sections D.2.2 (Page 14)	**Material Properties** The value of Modulus of Elasticity for concrete appear to be non-compliant with the NZS3101 Equation 5-1. It is understood that these values are used to represent the effective stiffness of the elements based on Table 10-5 ASCE41. Can you please advise?	Agreed. The E value entered in the material sheet is that for an assumed compressive strength of 30MPa, calculated using the older formula E=3320$\sqrt{f'c}$ + 6900. This is an automatic calculation within the input workbook and is in the process of being updated. I am undertaking a sensitivity analysis on the assessments using an E value based on Eq. 5.1 of	Calculations provided by Holmes for this query titled "*Whakatane DC Peer Review-DSA-Q23 reponce_2018Dec04*" are checked. It	Closed

			NZS3101 and the assumed compressive strength of 37.5 MPa. In progress. Yes, the upper and lower bound periods of interest were adjusted to match the updated fundamental periods follow the scaling procedures used, i.e. the upper bound remained at 1.0 seconds and the lower bound period was 0.2 seconds.	indicates that the period of the structure is reduced but the scaling factors also are slightly changed. Therefore, Holmes state that the changes in the results based on MCE for IL2 building checks were negligible. Please confirm that the updated lower bound period of interest is 0.2 Sec and the upper bound remained at 1 sec. [Rev 6]. Noted	
24	Appendix D Sections D.2, D2.3.1 and D.3.2 (Pages 14 and 26)	**Plastic Hinge Definition** It's been mentioned that the in-house software is using Lumped/Concentrated Plasticity Model. In figures 25 and 26, the location of plastic hinges is not clear. Please clarify the location of the defined hinges and plastic hinge status.	Plastic hinges are assumed to form in the frame elements at the faces of the adjoining members. For the columns, plastic hinge locations are assumed and modelled at the top of bottom of the column. The red colour shown in the plots indicate that one or both hinges have exceeded the appropriate CP rotation limits for the section. Please find attached hand marked plots showing a more focussed representation of the plastic hinge locations where ASCE41-13 rotation limits are exceeded.	Noted	Closed
25	Appendix D Section D.2.3 (Page 14)	**Plastic Hinge Definition** We understand that concentrated plasticity model is used for modelling the PPHZ. For biaxial loading, the stiffness/strength degradation and pinching etc should be considered concurrently. Most of the literature, uses "fibre modelling" for biaxially loaded columns. Please provide a peer reviewed journal paper where concentrated plasticity model is used to capture the biaxial interaction for RHA analysis.	Correct. For the column elements, biaxial performance at the hinge location is evaluated for concurrent rotations about each principal axis in conjunction with the concurrent axial load. Fibre element modelling is not consistent or available with the ASCE 41 RHA approach, which uses concentrated plasticity. An NZSEE conference paper is attached addressing how biaxial effects are considered in acceptance criteria. (Nonlinear Analysis Acceptance Criteria for the	According to building's behaviour and your reply to query No. 20, the columns' condition is i as per Table 10-8 of ASCE41-13. In this case there is no difference between the values provided by Boys et al. as the amendment to	Closed

			Seismic Performance of Existing Reinforced Concrete Buildings: Oliver et. al. 2012) Note that ASCE 41 acceptance criteria do not require any such reduction for biaxial bending.	ASCE41-06 and values of Table 10-8 of ASCE41-13. The above was discussed in the meeting on 23rd Nov. 2018. Please note that the acceptance criteria in Table 10-8 of ASCE41-17 has completely changed. Therefore, your evaluation is compliant with ASCE41-13, which does not consider the biaxial effect on reduction of plastic rotation limits.	
26	Appendix D, Table 15 (Page 15)	**Referenced Document** The report concludes that the column failure mechanism is flexural and column sway mechanism governs the performance of the structure. Therefore, the columns are controlled by condition i based on ASCE 41. Table 10-8 of ASCE 41-13 has been superseded by Tables 10-8 and 10-9 of ASCE41-17. Please provide information/calculations showing that the acceptance criteria used to define the backbone curves are still compliant with the ASCE41-17.	As noted above, the assessment has been completed to the provisions of ASCE41-13. Only the GM selection and scaling is based on the provisions of ASCE41-17 as an alternative procedure with rational basis. The ASCE41-17 performance limits have not been used.	As per the above comment for Q20 and 25.	Closed
27	Appendix D Table 18 (Page 18)	**Referenced Document** It appears that Table 18 presents the information in the format of Chapter 6 ASCE41-06 and classifies the elements to Primary and Secondary but is references as ASCE41-17 classification. It is not clear which version of	Agreed. The table heading does not represent the table contents particularly well. This table contains the default values of the modelling and assessment parameters for the various ASCE component types which are available within our assessment workbook. All the component types used in our assessment include automatic determination of the modelling parameters	It's noticed that this table is removed from revision 3.	Closed

		the code has been utilised for assessment and what are the acceptance criteria. Considering that classification of elements to Primary and Secondary is superseded, can you please explain where these values are applied and what the highlighted values are being referred to.	and acceptance criteria from ASCE41-13 tables 10-7 and 10-8, based on the axial load and shear reinforcing ratio for columns, and on the shear demand ratio for beams. These tables in the report shall be revised to better reflect the parameters for only the element types used in our assessment.		
28	Appendix D Table 19 (Page 18)	**Further Explanation** Please clarify how the information provided in this table are used in the model.	Table 19 defines the strength parameters of the various moment frame beam and column sections represented in the NLTHA model. These parameters are used during the analysis to automatically determine the appropriate modelling parameters for strength and stiffness degradation and acceptance criteria at each time step of the analysis. Section D.2.3 describes the frame element modelling in more detail.	Noted	Closed
29	Appendix D Section D.2.3.2 (Page 19)	**Further Explanation** It is understood that the Beam-column joints are not explicitly modelled. Please provide the assessment results for the beam column joints.	Attached a supplementary calculation for beam-column joint capacity vs. demand generated from columns overstrength demand.	Checked	Closed
30	Appendix D, Section D.2.5 (Page 20)	**Further Explanation** Please explain the foundation performance for different importance levels. The report indicates a base shear takeout by passive earth pressure on the sides of the ground beams. Can you please advise the impact of the liquefaction on fixed ended embedded piles where they connect to the ground beams?	As noted above, liquefaction is generally considered to be an issue following earthquake shaking rather than during earthquake shaking. Lateral load transfer from the building to the ground, or more literally from the ground into the building, is via a sliding shear plane at the underside of the foundation beams. The area of the sliding shear plane is approximately 1878 m2 (neglecting the courtyard area). Section 5.5 of the geotechnical report gives a ULS lateral shear capacity of 4 kPa. This implies a lateral load transfer capacity of 7510 kN. The lateral base shear capacity of the building superstructure for both IL2 and IL4 levels of load, based on the assessment analysis, is between 0.32 and 0.35g. Using the building weight of 16775 kN, this implies a maximum building lateral load of 5871 kN, which can be transferred by the soil shear plane mechanism. With lateral load transfer occurring across the shear plane interface, minimal lateral movements of the	We disagree on this matter. A certain number of records used for assessment are subduction zone earthquakes. Subduction zone earthquakes normally have quite long duration and liquefaction may happen during the earthquake shaking. Please confirm if the 4kPa lateral capacity reported by T&T is the full shearing capacity of the soil at the 600mm depth?	Closed

			foundation system are expected to occur, and thus minimum lateral loads and bending are expected in the piles. The T&T report states "Using a strength reduction factor of ϕ = 0.67 for the slab resistance, the ULS lateral capacity of the slab is 4 kPa at 600 mm depth." As for query item 9 above - considered closed.	We believe that for IL4 performance capacity of the piles with fixity into the ground beams and considering liquefaction needs to be assessed. While we have no concern on Life Safety at IL2 we do believe that the performance of the piles should be established. [Rev 10]. Refer to Q4	
31	Appendix D Section D.3.2 (Page 26)	**Rating at IL2** The 80% NBS for IL2 appears to be a mistake.	Agreed. Corrected in the DSA report. Oops, now updated in the current draft revision!	Section D.3.2 is now C3.2 and no correction has been made in Rev. 3. Please correct. [Rev 6]. Checked, Corrected.	Closed
32	General	**Further Explanation** Please provide the assessment of the Ground beam/ pile connection.	Refer to the foundation comments above. Lateral load transfer from ground to building is via a lateral shear transfer on the interface plane formed at the underside of the foundation beams. This transfer plane mechanism has sufficient capacity to transfer the maximum building base shear generated by the soft-story mechanism of the ground floor columns at IL2 and IL4 levels of shaking. The piles are not expected to be subject to any significant lateral displacements during the earthquake shaking but are well detailed for ductility if they are. No specific assessment of the pile and pile/foundation beam connection has been undertaken as part of our DSA.	Please refer to my comment for Q30. [Rev 10]. Refer to Q4	Closed.

			As for query item 9 above - considered closed.		
33	General	**Further Explanation** There does not appear to be any roof calculations. Please provide calculations confirming the roof bracing and connection from roof to column are adequate.	Please find attached sample calculations of the roof connection at the top of the concrete columns. The 1.6g used in these roof connection checks was based on an initial conservative pESA calculation. Updates to this conservative pESA were subsequently undertaken showing lower accelerations at roof level. Copy attached. Considered closed.	Please provide pESA calculations. Capacity check calculations are reviewed. Despite minor errors results are acceptable. [Rev 6] pESA calculations shows 1.16g instead of 1.6g. Therefore, calculations are conservative.	Closed
34	General	**Further Explanation** There does not appear to be any explanation on the difference between the performance of the building as existing and providing the separation between the two blocks. Please provide information on the key performance differences in a table format.	We did not provide any specific description of the differences in behaviour between the two models as this seemed irrelevant. The building as existing achieved a higher %NBS rating. Please see the attached damage representation of the separated building model subject to 110% IL2 shaking demand and compare to the damage representations in the DSA report. The smaller portion of the building is exhibiting a fully developed column-sway failure mechanism at the IL2 load level that was satisfactory for the joined building model. Apologies, it looks like the damage representation wasn't attached first time round! Please see the attached damage representation.	Please advise which attachment you are referring to. In the email dated 29/11 the two attachments are related to Q24 and Q33. [Rev 6] Comparison of the performance with the joined building at 110% IL2 MCE provided on Figure 26 confirms that separation of the building lowers the rating.	Closed
35	Cover page	**Date and Revision no** Pease update the revision no. and date according to the Report issue register	Noted, that one slipped through in the rush. Updated to revision 4 for the current draft issue.	[Rev 6] Correction is done. Please update the footer on the first	Closed

				couple of pages as well.	
36	Executive Summary, Overview, Page i	**Statement** In the last paragraph it is mentioned that "*Our Assessment and DSA report have been peer reviewed by Aurecon and all peer review queries have been closed out prior to the issue of the final version of this DSA report*". Please note this hasn't happened yet especially for the performance of the piles.	Noted. That was in anticipation of a successful conclusion prior to a final issue. This statement has been removed	[Rev 6] Noted.	Closed.

OVERALL SCALE FACTORS - IL2														
GM1	GM2	GM3	GM4	GM5	GM6	GM7	GM8	GM9	GM10	GM11	GM12	GM13		
3.17	1.20	2.18	2.05	1.76	1.36	3.80	1.55	1.25	6.10	1.01	3.78	1.95	aurecon	NZS1170.5
4.62	0.91	2.01	3.09	2.6	1.35	4.34	1.58	1.61	7.26	1.04	6.1	2.98	HCG	ASCE41-17
HCG	aurecon	aurecon	HCG	HCG	aurecon	HCG	HCG	HCG	HCG	HCG	HCG	HCG	<------	Largerk1xk2

OVERALL SCALE FACTORS - IL4														
GM1	GM2	GM3	GM4	GM5	GM6	GM7	GM8	GM9	GM10	GM11	GM12	GM13		
1.04	2.26	1.87	3.74	0.86	2.25	1.34	5.25	1.54	0.84	6.80	1.40	3.50	aurecon	NZS1170.5
0.87	2.01	2.47	3.65	0.91	2.78	1.61	4.81	1.51	0.98	9.44	1.3	4.61	HCG	ASCE41-17
aurecon	aurecon	HCG	aurecon	HCG	HCG	HCG	aurecon	aurecon	HCG	HCG	aurecon	HCG	<------	Largerk1xk2

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