Carillon Tower Peer Review

National War Memorial, Taranaki Street
Mount Cook
Wellington 6021

Issues Register

Holmes Consulting

Issue 3
21 September 2020
Project 140675.12

Holmes Consulting

Level 2, 254 Montreal Street
Christchurch Central
PO Box 6718
Christchurch 8442
holmesconsulting.co.nz

Issues Register

Carillon Tower Peer Review

Prepared For:
Ministry for Culture and Heritage

Date: 21 September 2020
Project No: 140675.12
Issue No: 3

Prepared By:

[redacted]

[redacted]
PROJECT ENGINEER
Holmes Consulting LP

Reviewed By:

[redacted]

[redacted]
PROJECT DIRECTOR

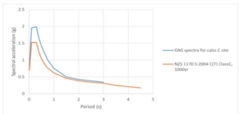
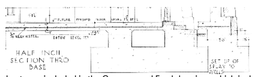
Holmes Consulting

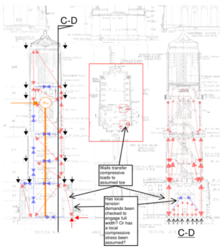
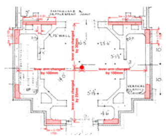
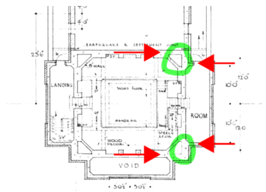
Comment Issue Register

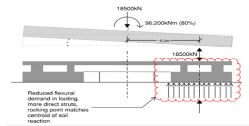
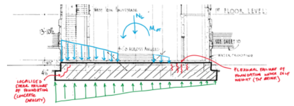
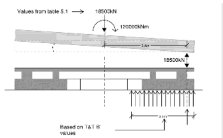
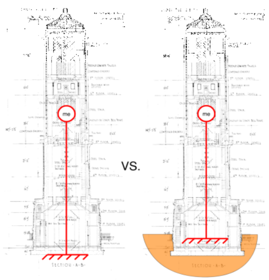
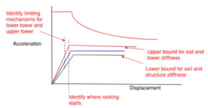
ISSUE	DATE	WHO	DESCRIPTION	Text Colour
1	27/07/2020	KDM	Issue 1 - Issue to DTC	Blue
2	20/08/2020	KDM	Issue 2 - Issue to DTC	Blue - New Comments, Black - Previous Comments, Grey - Closed Comments, Orange - Supporting Documents
3	21/09/2020	KDM	Issue 3 - Issue to MCH	Black

PEER REVIEW LOG

GENERAL		REVIEW STATUS	
PROJECT NAME:	Carillon Tower Peer Review	PROJECT NUMBER:	140675.12
COMPANY DESIGNING:	Dunning Thornton	NAME OF DESIGNER:	[redacted]
COMPANY REVIEWING:	Holmes Consulting	NAME OF REVIEWER:	[redacted]
		REVIEWED BY PRIOR TO TRANSMITTAL:	[redacted]

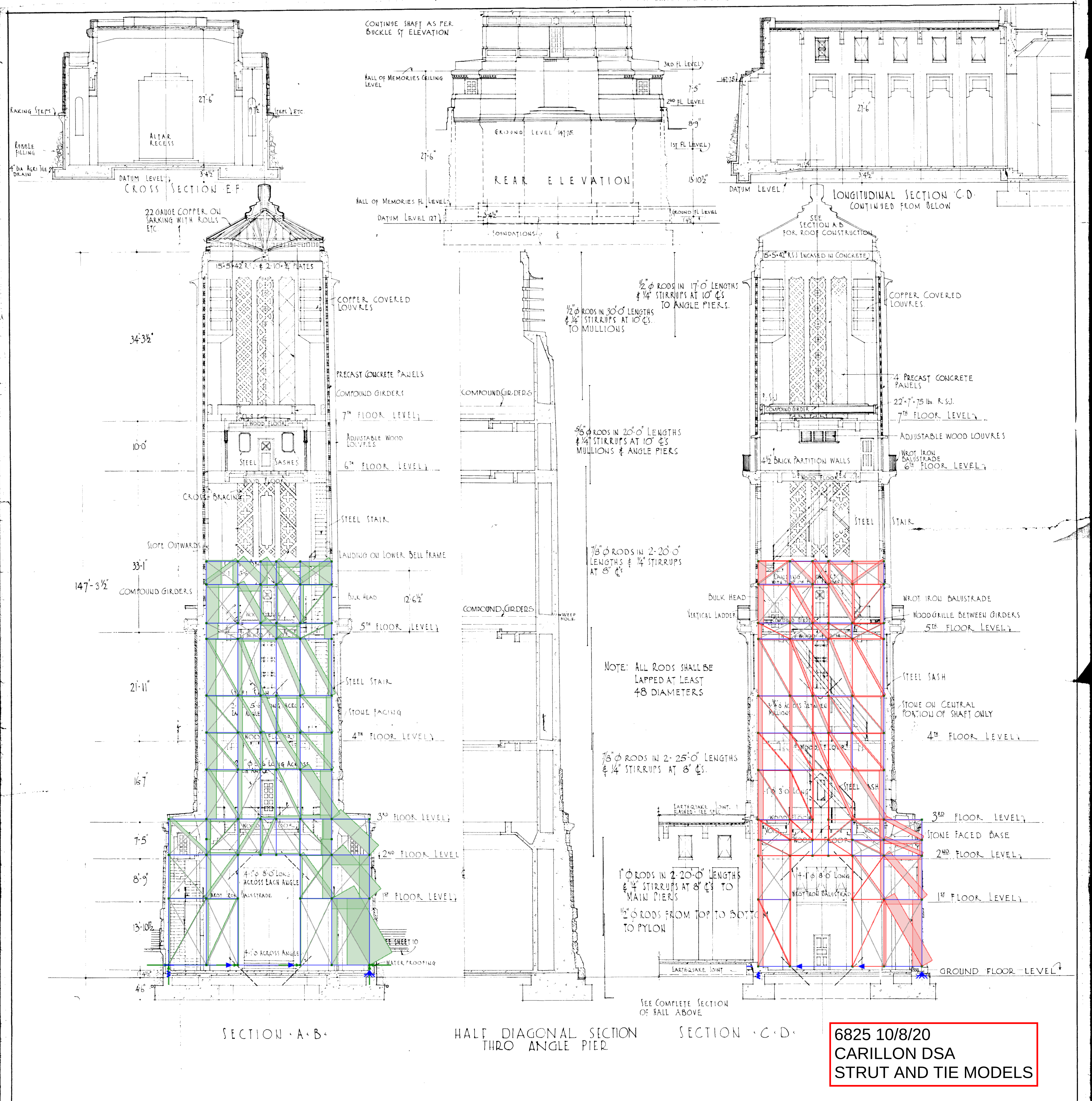
Item No.	Document/ Drawing reference	Item	Reviewer Comment	Designer Response	Reviewer Comment	Designer Response	Item Resolved
1	n/a	Note	"*The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments, July 2017, Version 1*" (NZSEE, 2017) is referred to as the "*Engineering Assessment Guidelines*" within this review register.	Noted.			Closed
2	-DT DSA Report - Section 5.4.2. -Appendix D - Section 1.2	Note	Holmes understands that a full DSA of the Carillon tower to the Assessment Guidelines was not within scope, and that the assessment has primarily been limited to the performance of the steel frames supporting the bells with results of the previous assessment and completed strengthening reconsidered in this report. In 2011, a "*Seismic Assessment of the Carillon Tower*" was limited to "simplified assessments of the tower to ascertain its ultimate limit state seismic performance relative to an equivalent new building".	Noted.			Closed
3	n/a	Note	Proposed strengthening will impact tower stiffness, therefore floor response spectras. To be considered as part of strengthening.	Noted. The bell mass will also be lower in the tower due to the removal of the outriggers, so the NLTHA will be re-run as part of the strengthening design.			Closed
4	n/a	Recommendations	We recommend the DSA be extended to incorporate the full performance of the Tower.	Agreed, and we have discussed this with the client as being an early part of the design phase of the strengthening. Completing the DSA at this stage isn't going to uplift the %NBS and benefit the client.	We acknowledge that completing a full DSA of the Carillon Tower may not uplift the %NBS of the structure. If the objectives of the study was to assist TA's in establishing whether the building is earthquake prone there is potential justification for no further refinement. In line with C1.3 of the Assessment Guidelines, "many buildings will not require, of justify, the use of lengthy and detailed analyses... effort may even be better spent in completing an appropriate retrofit rather than necessarily understanding fully how the existing building configuration may perform". Our understanding is that the client is expecting a comprehensive assessment of the risks for the building (Carillon Tower as a whole), in particular, to a point where a rough order of cost can be established on the basis of strengthening to 100%NBS(IL3). For a complex building like the Carillon Tower, proceeding to the strengthening scheme without completing the DSA (nor closing out open comments in this review) adds risk and should be communicated clearly to the client with respect to impact on potential strengthening costs. It should also be clearly communicated to the client that this is an unfinished DSA, with the results of work to date identifying whether the structure is earthquake prone with the understanding that the client has already decided to strengthen it seismically.		Open
5	n/a	Recommendations	Seismic hazard. Would a site specific hazard study benefit the strengthening scheme?	The site specific hazard for the neighbouring Arras tunnel (PSHA completed in 2012 by GNS) shows a greater than code hazard. This PSHA also did not incorporate the increased hazard from the Hikurangi Subduction Zone event. We would expect the hazard at the Carillon site to be greater than NZS1170.5 soil class C. We have had intial discussions around this with the client as part of developing the brief for the strengthening.  *Figure 4-1: Target response spectra for time history scaling, 1000 year return period event derived from PSHA from GNS report*	We recommend progressing discussions with the client during the early stages of strengthening to discuss the potential impact of NZS1170.5 vs a site specific hazard study.		Open
6	n/a	General	What elements, not considered, limits this assessment from being a "Full" DSA? Can the designer please provide commentary as to why excluding these items does not over-simplify the assessment of the steel frames supporting the bells to the extent that poor behaviour is not identified and/or captured.	We have completed a general assessment of the load path through the tower under overturning. Items we have assessed are: - Corner piers and bracing from L5 to roof, including moment capacity of piers, axial capacity of braces, axial capacity of ring beam at roof level. - Capacity of the lower tower using a simplified (large scale) strut and tie model considering openings. Assessment of critical bottle struts and tension ties was completed. - Mullions between L5 and roof under accelerations from NLTHA. - Global overturning of the tower. We completed a parametric analysis to test the sensitivity of the bell frames to our assumptions. This determined there was not a significant difference in accelerations at the bell frame levels when the tower stiffness was altered, e.g. cracked or stiffer than expected.	Refer commentary in item 4.		Open
7	n/a	General	A rocking response of the superstructure is a fundamental assumption in the assessment. Can the designer please provide drawing/calculation reference confirming the plane in which rocking occurs (i.e. at the base of the footing, or the top of the footing)? Has there been any intrusive invesitgations to confirm the location in space of the joint? Given the fundamental nature of this mechanism, reccomend intrusive investigations to ensure as built condition in accordance with potential design philosophy.	The rocking interface is shown on original Gummer and Ford 1928 drawings. Drawing 4 shows a double slab with Neuchatel layer separating the walls from the footing. Drawing 7 shows the Neuchatel layer wrapping up the tower side at the inerface with the Hall of Memories, and a detail of the foundations.  Two unnumbered sheets are included in the Gummer and Ford drawings which include calculations of the tower's lumped masses. These calculations use Dr. Omori's formula to determine the fracturing acceleration which we understand to be the onset of rocking. Omori investigated the rocking phenomenon in Japan in the early 1900's. From Dr. Omori's formula $a = \frac{I g t}{x h w V}$ Agreed that investigation of this is required of the interface to ensure the rocking interface is present, before committing to detailed design.	Recommend investigation to occur during early strengthening works.		Open

8	n/a	General	Has the designer considered whether it is possible for the bells to fall through into the foyer once the frame fails? Following initial discussions we understand this was considered but discounted, can this discussion be provided? Has an assessment into potential brittle fracture of the bells been done? We note that the bells cannot disconnect from the frame given the number of bolts. We also note a tight tolerance of bell diameter to opening size for the bells immediately above the foyer (anectodally <5mm). From discussions on site, careful coordination is required during installtion to avoid bells impacting the sides of the voids. In service, steel framing spans across the voids, which will limit the ability for large piece(s) of bell(s) to fall though.	Under the EPB methodology, the bell frames have exceeded their ULS capacity at the reported %NBS. Collapse scenarios where the bell frame failure does not pose a significant life safety hazard are possible, however we do not believe there is a sufficient body of evidence to demostrate these are the probable collapse scenarios. Therefore it would be inconsistent with the EPB methodology to not consider the bell frames to be Structural Weakness. No mechanical testing of the bell material has been completed. Agreed that the largest bells (#1 and #2) have minimal clearance, however these bells are not above the opening. The bell directly above the opening is bell #6 (4tonne) which is considerably smaller than the opening size (1.9m diameter bell vs. 3.2m square opening). Some of the small bells also exceed 25kg which is the minimum weight for a part to be considered as a Significant Life Safety Hazard when over a Space Class I under the Assessment Guidelines (Table A4.1).			Open
9	n/a	Recommendations	Aligning with the philosophy above, is it possible to explore strengthening measures that prevent the bells from falling into the foyer? This could include a strengthened slab which could support total weight of bells and associated impact loads. This could avoid strengthening to the bell frames which does not appear feasible due to clashes of braces with bells.	Yes, we consider there is value in a strengthened floor because the bells are castings so there is a degree of uncertainty. This needs to be balanced against access requirements to get the bells in and out. We have discussed with the client that all of these parameters can be weighted up during preliminary design of the strengthening.	Ongoing opportunities to be explored during strengthening schemes.		Open
10.1	n/a	Level of detail in model (1 of x) - Note	Sufficient detail should be modelled in the superstructure (or assessed in post-processing) where further refinement would not affect the decision. The following items are those identified to date that may impact analysis (both for the superstructure and bell frame support).	We have varied and challenged our assumptions such that we don't believe further refinement would affect the outcome (of the bell frames).			Open
10.2		Level of detail in model (2 of x)	How has the out of plane performance of walls been incorporated/assessed? - In particular, over double-height spaces adjacent floor penetrations.	These have not been checked in full detail, as these are doubly reinfroced. We have included ring beams in our concept strengthening so that there is some allowance for improving diaphragms, out-of-plane walls, etc.	Refer discussion in item 4 regarding understanding the building as a whole.		Open
10.3		Level of detail in model (3 of x)	How has the diaphragms been incorporated/assesed? - In particular, the ability to transfer out-of-plane demands to in-plane walls (including connections).	As above.	Refer discussion in item 4 regarding understanding the building as a whole.		Open
10.4		Level of detail in model (4 of x)	Can the designer please provide simplified strut-tie diagragms of the superstructure in both orthogonal directions which demostrates how the rocking mechanism is realised? An example is shown below for the E-W direction. This suggests local stress concentrations in walls as opposed to a uniform stress distribution at the toe across the building width. Has the impact of these local stress concentrations been assessed (for example, toe crushing of confined concrete in Table 2) and how does this impact the assumed rocking leverarms? 	Please find attached SpaceGass strut-and-tie models overlaid on the building elevations (not exactly to scale). This uses compression only elements to form the diagonal struts. **[6825 200810 Response to 140675.12-Carillon Tower-HC Peer Review Comments-Issue01.pdf]** We have reviewed the transfer of the compression force to the full flange width using strut-and-tie. The vertical reinforcement in the corners does not have sufficient capacity to achieve equal stress across the full flange. We have reviewed the compression block depth using a reduced flange (bw + 0.15h). This would cause a reduction in lever arm of 100mm resulting in a negligible reduction (<2%) in overturning moment, i.e. rocking would occur slightly earlier. 	Noted.		Closed
10.5		Level of detail in model (5 of x)	How has transfer forces on level 3 been assessed? - Refer figure in item 10.4. To achieve rocking about the point as adopted in analysis, we expect local stress concentrations at level 3, as lateral shears in the in-plane walls above level 3 are transferred across to the buttresses.	Tension tie reinforcement in walls as part of strut and tie.	As shear is transferred from the in-plane walls in the E-W direction above level 3, to the in-plane buttresses below level 3 (to the point of assumed rocking), the loadpath is unclear. We expect significant transfer forces through this joint where existing tie reinforcement is inadequate. Reccommend a detailed assessment of this level (as well as other transfer diaphragms) as part of the early design phase of strengthening. In line with discussion in item 4, our understanding is that this report is not a completed DSA of the Carillon Tower, with results to date identifying to the client whether the structure is Earthquake Prone. Recommend potential weakness be assessed as part of early design strengthening. 	Noted. The performance of this transfer area will be assessed as part of the strengthening design phase, as well as diaphragms as per 10.3 above. The strengthening concept provides a conceptual load path through this area, so that there is cost allowance to address this potential structural weakness.	Open
10.6		Level of detail in model (6 of x)	There appears to be potential for a secondary rocking mechanism between lower and upper sections of the tower? How has this been currently modelled/assessed and/or discounted?	Assessment Guidelines do not allow ductility due to the lapped round bars so the capacity is limited to the debonding of the lapped bars and this is not a "dependable mechanism". We expect the piers to rock after the debonding of the reinforcement occurs but this has been considered as the beyond ULS resilience required so that the step-change loss of moment capacity in the piers is not considered a Severe Structural Weakness (refer A6.6 of Assessment Guidelines).	Noted.		Closed

10.7		Level of detail in model (7 of x)	How has the foundations below the assumed rocking plane (but above the base of the footing) been assessed? - T&T have provided required bearing areas/stiffnesses for vertical loads outlined in Table 5.1 (T&T Geotechnical Report). The footnotes of Table 5.2 (T&T Geotechnical Report) state assessed bearing pressures are based on a rectangular stress block. Comparing the centroid of the bearing area, with the centroid of the predicted rocking point of the superstructure highlights an eccentricity. We would expect little to no eccentricity. How has this eccentricity been assessed/can the designer please demonstrate that eccentricity between assumed rocking point, and centroid of bearing area can be resisted in the foundation?	There is an eccentricity between the point of rocking in the tower and the point of support from the soil on the foundations, which is balanced by the self-weight of the foundations (6000kN). This has been accounted for in the modelling of the rocking by adjusting the spring stiffnesses provided by T&T to suit the modelled lever arm. We have reviewed the eccentricity through the foundations and the moments this induces in the foundations using a beam model with a series of springs. The foundation does not have adequate capacity to resist the moment or shear induced at full rocking (100%NBS (IL3)). We have also reviewed the design actions at 25%*NBS* (IL3) when the tower is not rocking. The foundations do have adequate moment and shear capacity for these actions, so the overall rating of the tower is not changed. This will require further analysis during the next phase of work, and consideration in the strengthening scheme.	-What base shear (kN) is the onset of failure in the foundation, and does this occur prior to other failure modes identified, noting that premature failure of the footing could cause a step function response in the building? -Is it reasonable to treat the two systems in isolation (the superstructure and the foundations)? Is the stiffer loadpath rocking about a point in line with the centroid of the bearing area as opposed to rocking at the toe of the superstructure and subsequent flexural action of the footing? Refer below, if rocking point is in line with soil reaction, then flexural action is reduced in the footing. This reduces the leveram and associated moment before rocking by approximately 20% (leveram approx 5.2m).  	The foundations capacity is approximately 25 to 30%NBS(IL3). The failure of the foundations occurs at approximately the same base shear as the failure of the lower tower, but after the failure of the bell frames. This failure is the yielding of the top reinforcement in the foundation as the tower rocks over so we don't believe this is a step-change type of failure. The flexure failure of the foundation occurs under the self-weight of the foundation behind the area of bearing of the tower above. At the point of flexure failure the shear demands at the toe of the foundation equals 0.85Vc,prob. This includes allowance for degradation from curvature due to the yielding reinforcement. The failure of the foundations at 25 to 30%NBS(IL3) is the dependable performance of the foundation, assuming upper bound rocking response (longest lever arm and stiffest soil), and this can be demonstrated through calculation. Beyond this point, the failure of the foundation is less certain. There may be gradual destiffening and reduction of the lever arm but the performance is highly dependent on ground parameters and as-built foundations. This could be verified through extensive modelling, but we believe we should instead design out this uncertainty by allowing to improve the foundation capacity. 	Open
10.8		Level of detail in model (8 of x)	How has the internal forces in the foundations been assessed? Does the capacity of the footing impact the point of rocking assumed. - We note that the foundations contain voids, and the footing appears to indicate low reinforcement quantities.  	Refer item 10.7 above.	Refer item 10.7 above.		Open
10.9		Level of detail in model (9 of x)	Is the rocking performance sensitive to the retained soil on the east and west sides? How has this been assessed/discounted?	We have reviewed the potential hold-down force provided by the friction of the soil bearing on the walls. This friction would add less than 5% to the building overturning so is within the accuracy of the building weight and other assumptions.	Acknowledged additional hold down forces provide nominal increase in overturning resistance. We are more interested on the impact of base shear is taken out, potentially reducing the effective height of the system. How has this been captured and/or assessed? Recommend detailed sensitivity studies be conducted as part of early seismic strengthening.  	Base shear is assumed to be taken out as passive pressure on the sides of the foundation pad and as friction on the underside of the pad, as per T&T's report. This requires geotechnical input to see if there is enough displacement to activate the passive pressure of the soil to determine the effect of the retaining on the structure.	Open
11.1		ADRS (1 of x)	Please provide ADRS plot for each of the principle directions based on a triangular distribution of applied load. In the plot clearly show the effect of upper and lower bound stiffness assessments and show where the identified failure mechanism will occur.  	We have compiled an ADRS plot for the upper bound east-west rocking response. This is attached. The assumed SDOF response is shown in black. The maximum displacements from the NLTHA are shown as data points, and averaged as a dashed line. The maximum base shears are also shown as data points, and averaged as a dashed line. Due to the significant higher mode effects and the "poor isolation" of the tower (not significant difference in the rocking mode to the tower modes), we don't believe a SDOF ADRS accurately represents the response of the rocking of the tower. **[6825 200810 Response to 140675.12-Carillon Tower-HC Peer Review Comments-Issue01.pdf]**	Noted.		Closed
11.2		ADRS (2 of x)	Provide information as to how the effective periods were calculated (Table 13)?	As per formula for SDOF system (DBD)	Noted.		Closed

11.3		ADRS (3 of x)	A simplified overturning check in the E-W direction, based on a seismic weight of 18500kN (T&T report table 5.1), an effective width of 9.75m, a bearing capacity of 1000kPa, and an effective wall length of ~12m suggests a base shear of ~0.2-0.3g before onset of rocking. We acknowledge the crudeness/simplification of this calculation, but provides an approximate view into the base shear of the structure at point of rocking. Table 13 reports acceleration of the SDOF model as ~0.4g in the EW direction. How does this corelate to calculated acceleration at onset of rocking, and if greater, what is the reason for the amplification? Masses were taken from Figure 16 to determine an effective height (25.3m) assuming a linear deflected shape above the assumed point of rocking.	Preliminary information was given to T&T for them to complete their works. The detailed weight and rocking accelerations were derived after this so there is a slight discrepancy. $T = 2\pi\sqrt{\frac{displacement\ mm}{acceleration\ mm/s^2}}$	Can hand calculation of lateral base shear in the EW direction utilising updated inputs be provided? With boundary conditions, as in column D (based on preliminary information), with updated building mass in accordance with Figure 16 of Appendix A, suggests a base shear between 3800-4200kN at onset of rocking.	Derivation of base shear as below: Detailed seismic mass = 20198kN Rocking lever arm = 6.37m (centre of mass to centroid of bearing area) so, overturning moment = 128,660kNm SDOF system: me = 1310tonne (64%) he = 25.177m V = M/He = 5110kN = 0.4g (using effective mass as applied at effective height)	Closed
11.4		ADRS (4 of x)	Based on our meeting you stated that rocking does not occur before the failure mechanisms that currently limit the structural capacity. Is it then reasonable to scale records for the time history on the basis of a rocking effective period or should the period be based on the effective period at 30%NBS (limiting capacity).	This was discussed internally, however we did not believe rescaling of the records would lift the building's %NBS above the EPB threshold. The period of the tower response at the limiting capacity is approximately 0.6s (and is linear, pre-rocking). From figure 20 of Appendix A, the current scaling is approximately 10% over the NZS1170 spectra at 0.6s, so would over-estimate the bell and tower actions by the same, i.e. no change to 15%NBS, and 25%NBS might increase to max 30%NBS. Therefore we did not believe it was of benefit to the client to commission a variation to T&T's scope.	Noted. A comparison of lateral force coefficient in accordance with NZS1170.5 for an elastically responding structure with a fundamental period of 0.55 seconds (as indicated by SDOF response in supporting documentation in item 11.1) corellates with expected accelerations.		Closed
11.5		ADRS (5 of x)	Please identify how the failure mechanisms of the tower have been determined from the simplified 2D models undertaken.	The average (of 7 records) storey forces have been applied in more detailed linear models. The piers and bracing were modelled in Microstran in the previous partial DSA, so this was updated with the revised storey forces. The lower portion of the tower (to level 5) has been modelled as a 3-D strut-and-tie in SpaceGass with storey forces applied at each diaphragm level and actions from the piers and braces applied at level 5. Failure mechanisms were then determined using the local element demands from these models and capacities derived according to the assessment Guidelines.	Which of these failure modes occur during the predominately elastic response of the superstructure, i.e. before rocking occurs?	In the east-west direction rocking occurs between 40 to 45%NBS(IL3). The bell frame failure, pier lap failure at level 6, and lower tower shear failure all occur significantly prior to rocking. The roof bond beam failure occurs around the on-set of rocking.	Closed
12		Soil Stiffness	How has the designer derived upper and lower bound soil stiffnesses? Does the centroid of the bearing area corellate with point of rocking, and how does this impact the point springs modelled in the analysis? Appendix A - Table 9 spring stiffnesses vary slightly from section 5.1 of T&T report.	**T+T:** Soil stiffness has been assessed based on the assessed ground conditions. The range considered to select the upper and lower bound soil stiffnesses are based on C4- guidelines. The range considers the uncertainty in the foundation ground conditions and uncertainty in the assessment of the stiffness parameter **DTC:** Stiffnesses in Table 9 vary from T&T's report as this is taking into account the offset of the centroid of the bearing on the soil and the assumed point of rocking in the tower.	Recommend discussions from item 10.7 be incorporated into early design strengthening with respect to stiffness bounds in the foundation, i.e. whether the building actually will rock at an eccentricity to the centroid of bearing area.		Open
13	Section 10.1.2	Tower Properties	Can the designer conduct/provide a sensitivity study that investigates varying strengths of concrete? Concrete strength is assumed to be 15MPa. There is anecdotal evidence to suggest concrete strengths significantly higher. We suggest limiting initial sensitivity study to stuctural weaknesses identifies in Table 2 (page 12) as well as impact on tower stiffness/effective period/base shear. We note a higher concrete strength may assist in shear strength and lap lengths assessed as being critical. If deemed sensitive, recommend core samples taken at different levels.	All Structural Weaknesses are governed by the reinforcement. An increase in concrete capacity may have a slight increase in bar lap capacity for piers but this is insignificant. The strut-and-tie analysis of the lower tower uses full steel capacity as they have hooks (stirrups). We have considered the merits of doing specific testing and do not believe more data will change the conclusions of the assessment.			Closed
14		General	Is a concept design report available for the 2020 concept strengthening scheme?	No, the intent of the sketches was allow a rough order of cost to be established on the basis of strengthening to 100%NBS(IL3).	Noted. Refer discussion in item 4. Concept strengthening scheme should make clear this addresses critical items identified in reports to date. It should also note that detailed analysis of full Tower as part of early design strengthening is required to ascertain full extent of strengthening costs.		Open
15		General	Is a design report available for the 2012 strengthening scheme?	No, a Design Features Report was not completed for the access improvement works.	Noted. Refer discussion in item 4. Concept strengthening scheme should make clear this addresses critical items identified in reports to date. It should also note that detailed analysis of full Tower as part of early design strengthening is required to ascertain full extent of strengthening costs.		Open
16		General	What is the expected lateral movement in the North-South Direction? Site observations of western elevation appears to suggest between Hall of Memories and Carillon structure is a 'sealant' with a downpipe embedded (<80mm gap t.b.c). Has an on-site review of '120mm' seismic gap detailing been conducted?	Table 15 of the Appendix shows the expected displacements at the interface with the Hall of Memories for each record at 100%*NBS* (IL3). The average displacement of the 7 records is 70mm. The expected displacement of the Hall of Memories is less than 10mm at 100%*NBS* (IL3, 100year design life). Extensive on-site investigation has not been completed, but the presence of the gap was observed during the works completed to the Hall of Memories.	Recommend sufficient verification of movement allowance (via inspection and intrusive investigation where necessary) be carried out in enough locations to provide confidence in assumptions being made. In the case of exterior elevation, a 'sealant' with a downpipe embeded was observed between the Hall of Memories and the Carillon Tower (see below). 		Open
17	Section 9.0	Recommendations	Please outline concerns with the bell frame structure with regard safety/ usability and why you believe replacing the frames needs to be explored. *Serviceability of the bell frames* Similar to above, the strengthening works could present an opportunity to prolong the life of the bell frames and make improvements to safety/usability of the instrument. The concept design presented with this report involves adding new steel elements to the existing frame. However, the option of replacing the existing bell frames with new bell frames should be explored to assess costs vs. value.	This has come about from discussions with client in terms of what options need to be considered in developing the scope of any future projects.	Noted.		Closed
18	Section 9.0	General	Removing the lift does not appear practical. Has this been discussed with the operator?	No, the intent of the sketches was allow a rough order of cost to be established on the basis of strengthening to 100%NBS(IL3).	Recommend including issues relating to the "removal of the lift" into a risk register during early design strengthening.		Open
19		T&T Report	Can you confirm whether this report has been externally peer reviewed, in particular ground motion selection and scaling?	**T&T:** The report, including ground motion selection has not been externally peer reviewed.	Recommend a geotechnical engineer with experience in ground motion selection and scaling, as well as site specific hazard study (pending outcome of discussions in item 5) be completed.	This needs to be a client decision around the consequences vs. costs. If the building is strengthened such that there is a dependable rocking mechanism the importance of precision in calculating the hazard is less.	Open

20	T&T Report - Section 3.5, page 4	T&T Report - Slope Stability	Seismic slope stability has not been analysed as part of this assessment. What is the justification? In particular, the potential failure plane as the building rocks to the north, down the slope (concentrated bearing and lateral stresses).	**T&T:** We do not expect global instability under the assessed seismic conditions based on the ground conditions at the site and the slope of the ground to the north of Carillion tower. The additional load at the crest of slope from rocking of the tower in the N-S direction is expected to be small compared to the weight of the soil in a global failure plane. The bearing capacity is expected to be more critical. The rocking of the tower and the sloping ground in front of the foundation has been considered in the bearing capacity calculations. We have checked the bearing pressures are less than the bearing capacity.	Noted.		Closed
21	T&T Report - Section 3.5, page 4	T&T Report - Slope Stability	What were the boundary conditions when assessing the influence of slope on bearing capacity, was this under static or dynamic loads?	**T&T:** The influence of slope has been allowed for using inclination factors (Meyerhof 1963, Hansen 1970). The edge of the slope from the north side of the foundation is 7.5 m. The bearing capacity has been calculated under pseudo static loads as presented in Table 5.1 of the T+T geotechnical assessment report. The bearing capacity provided is intended for pseudo static or dynamic analysis. It allows for the combination of horizontal and vertical loading (inclined load) and the affective width of bearing (due to overturning moment). We have checked the bearing pressures are less than the bearing capacity.	Noted.		Closed
22		T&T Report	Is there reduced lateral stiffness of the soil when the structure rocks about toe towards the North (concentrated vertical and lateral stresses in the soil towards the slope), and what is its significance on the rocking response?	**T&T:** We expect the soil will lose some stiffness from the dynamic soil structure interaction between rocking foundation and the soil in the N-S direction. We expect any reduction is soil stiffness from cyclic loading to be captured in the range of soil stiffnesses provided in section 5.1 of the T+T report. We have checked the bearing pressures are less than the bearing capacity allowing for the slope.	Noted.		Closed
23	Appendix F, SK-C-01	Concept Strengthening	As noted in item 10.7, the foundations do not appear to have sufficient capacity to support demands from the superstructure at the rocking point modelled. The failure mode of the superstructure appears to be limited to the performance of the foundations which occurs prior to any rocking, Based on the assumed SDOF rocking response in the ADRS plots supporting response to item 11.1 this would limit the performance of the superstructure to <45%NBS(IL3). In line with discussion in item 4, concept strengthening should include some cost implications to ensure premature failure does not occur in the foundations prior to rocking. Access limitations should be considered as part of the costing exercise. Recommend potential weakness be assessed as part of early design strengthening. Recommend treating the superstructure and foundation system together, as opposed to in isolation (as discussed in item 10.7), to determine whether redistribution of loads occur that prevent premature failure of the foundation.			Refer also to response to 10.7. We note that the failure of the foundation does not change the reported seismic rating of the tower (25 to 30%*NBS* (IL3)). Agreed that the concept strengthening should include some cost for strengthening the foundations. We will provide a description of the works to the client as part of our peer review summary to allow a rough order of cost to be determined.	Open



INDICATIONS	BY WHOM			
CONCRETE IN SECTION	DRAWN L.M.W			
STONE IN	TRACED L.M.W & L.B.H.			
RUBBLE FILLING	CHECKED			
	DATE: 24-10-30.			

WELLINGTON MEMORIAL CARILLON TOWER

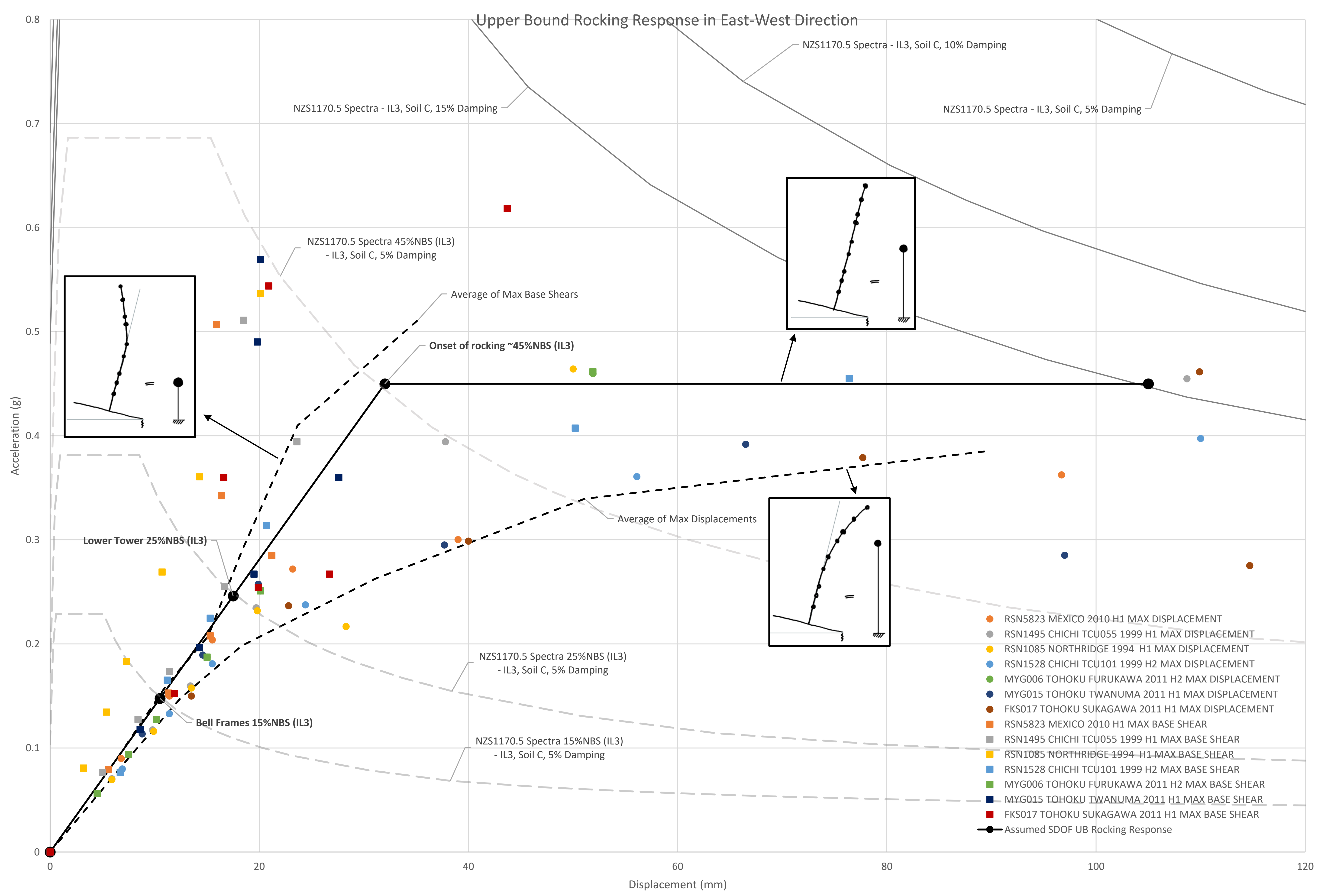
GUMMER & FORD
ARCHITECTS
STRUCTURAL ENGINEERS
AUCKLAND

SCALE: ONE EIGHTH INCH EQUALS ONE FOOT

NAME: VERTICAL SECTIONS

CONTRACT No 209

DRAWING No 2


Upper Bound Rocking Response in East-West Direction
NZS1170.5 Spectra - IL3, Soil C, 10% Damping
NZS1170.5 Spectra - IL3, Soil C, 15% Damping
NZS1170.5 Spectra - IL3, Soil C, 5% Damping
NZS1170.5 Spectra 45%NBS (IL3) - IL3, Soil C, 5% Damping
Average of Max Base Shears
Onset of rocking ~45%NBS (IL3)
Average of Max Displacements
Lower Tower 25%NBS (IL3)
NZS1170.5 Spectra 25%NBS (IL3) - IL3, Soil C, 5% Damping
Bell Frames 15%NBS (IL3)
NZS1170.5 Spectra 15%NBS (IL3) - IL3, Soil C, 5% Damping
Acceleration (g)
Displacement (mm)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
20
40
60
80
100
120
RSN5823 MEXICO 2010 H1 MAX DISPLACEMENT
RSN1495 CHICHI TCU055 1999 H1 MAX DISPLACEMENT
RSN1085 NORTHRIDGE 1994 H1 MAX DISPLACEMENT
RSN1528 CHICHI TCU101 1999 H2 MAX DISPLACEMENT
MYG006 TOHOKU FURUKAWA 2011 H2 MAX DISPLACEMENT
MYG015 TOHOKU TWANUMA 2011 H1 MAX DISPLACEMENT
FKS017 TOHOKU SUKAGAWA 2011 H1 MAX DISPLACEMENT
RSN5823 MEXICO 2010 H1 MAX BASE SHEAR
RSN1495 CHICHI TCU055 1999 H1 MAX BASE SHEAR
RSN1085 NORTHRIDGE 1994 H1 MAX BASE SHEAR
RSN1528 CHICHI TCU101 1999 H2 MAX BASE SHEAR
MYG006 TOHOKU FURUKAWA 2011 H2 MAX BASE SHEAR
MYG015 TOHOKU TWANUMA 2011 H1 MAX BASE SHEAR
FKS017 TOHOKU SUKAGAWA 2011 H1 MAX BASE SHEAR
Assumed SDOF UB Rocking Response