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2-S8400.02

Dear Bob

Gisborne District Council: 2002 Office Extension

Thank you for your letter dated 29 April 2013 which included a number of additional calculations and raises further items for consideration. We write to respond to your comments and calculations as follows:

We note that you consider that the design complies with NZS4203 which allows for rocking with a ductility of μ=2.0. However the current code NZS1170 states that rocking structures and structural elements require special study, beyond the scope of the standard (Clause 6.6). As indicated in our previous letter we have already considered the effects of rocking using the approach outlined in the SESOC Journal (Vol. 24, No.1-April 2011) and found that the increased period did not result in a reduction of the seismic demand.

From your additional calculations it is apparent that you have taken the period of the building to be 0.6 seconds when calculating the seismic demand to NZS1170. However all of our calculations, including an ETABS model of the whole building and the additional rocking checks, show the period to be less than 0.4seconds.

For the μ=3.0 case we note that the Structural Performance factor has been taken as 0.4 when NZS1170.5 indicates a minimum value of 0.7. (See clause 4.4.2).

The distribution of seismic loads to the shear walls (2001 calculations, pages 5-7) underestimates the eccentricity of the applied horizontal load by taking either the 10% accidental eccentricity or the distance from the centre of mass to the centre of rigidity. NZS1170.5 clause 5.3.2 states that “The accidental eccentricity shall be measured from the nominal centre of mass” resulting in a larger maximum overall eccentricity.

It should also be noted that the centre of mass in the transverse direction (Grids A-E) does not match the centre of rigidity due to the projection on Grids A-B/1-3 and the cut out at Grids D-E/1-2.

As such the seismic demand taken in your additional calculations appears to have been underestimated in both directions.

The nominal shear capacity of the wall used in the additional calculations as the shear demand for checking the wall/diaphragm connection and as the basis of your foundation uplift checks significantly underestimates the demand required by the overstrength factor $\varnothing_o = M^o/M_E$. A full capacity design approach remains necessary for any assumed ductility above $\mu=1.25$.

The calculation of the wall/diaphragm shear transfer on page 6 of the additional calculations overestimates the capacity of the joint and should be based on the Shear-friction provisions of NZS3101.1 section 7.7. On the assumption that the wall and slab were poured separately without intentional roughening of the junction, as seems likely without specific instruction to the contractor, we calculate the shear capacity of the joint to be approximately 1670kN.

Also, the additional H16 bars indicated on variation No.6 appear to apply only to the shear walls on Grids 3 and 7. There is no indication on the variation or construction drawings that this also applies to the walls on Grids C and D.

We are unclear how bearing of the wall onto the face of the floor beams as indicated on page 6 of the additional calculations can transfer additional shear from the floor diaphragm. Please provide further information on how this is to be achieved, including proposed load paths, the effects on the precast floor beams and consideration of the effects of successive loading cycles.

We feel it is prudent to ignore the bottom 56mm of concrete in the first floor slab where the traydec projects into the depth of the slab. The individual traydec units are not positively connected together and could potentially slip past each other under horizontal loading. As the 665 mesh provided in the slab is a brittle element it is necessary to check the diaphragm for the demand produced at $\mu=1.0$ to ensure premature failure does not occur.

The foundation stability checks on page 7 of the additional calculations utilise an additional horizontal force from the ground floor slab to provide a restoring moment. It would appear inconsistent to rely on the separation of the floor slab and top of foundations as a lever arm to produce a restoring moment while at the same time ignoring it in the calculation of the applied overturning moment (demand).

The calculation of this horizontal force at ground floor level assumes that the full width of the columns at both ends of the shear wall are available at the same time (2x400mm) to provide a bearing contact. This is clearly not the case as only one end of the wall can be pushing against the floor slab for any given load direction, while the other end of the wall tries to pull away.

The calculation of any shear resistance between ground floor slab and shear wall/ground beam should be also based on NZS3101.1, section 7.7. Given the details presented on drawing sheet 5 it is unclear if any shear-friction reinforcement is available.

We are also unclear as to how such a horizontal compression force would be resisted by a 100mm thick floor slab and how the load would ultimately be transferred into the foundations.

The validity of the Space Gass model used as part of your additional calculations cannot be readily checked. On the assumption that nodes 1,3,5,7 represent Grid Lines and hence column/pad base positions it would appear that additional supports have been added at nodes 4 and 6 (within the ground beams?) while the support provided by the foundation at node 1 has been omitted. We cannot check the figures used from the Space Gass analysis without the full input data and results, including reactions and bending moment diagrams etc.

It is also apparent from the additional calculations that only the shear walls in one direction have been checked. It will also be necessary to justify the walls on grids 3 and 7, which due to their longer lengths will have higher Overstrength Moments of Resistance affecting the shear demands and foundation uplifts. And the wall on Grid 7, in particular, will be affected by its position at the end of the building and have less vertical load in the columns from the weight of the building.

Please provide further justification on how the columns can provide additional lateral load resistance. The shear walls will be many times stiffer than the columns and attract almost all of the seismic loads even after rocking commences. Also, the column base detail on drawing sheet 6 appears to have been intended to act as a pinned joint.

While the additional calculations include ductility checks on the shorter shear walls they do not show how the the required ductility (μ=3.0) can be reached before a shear/brittle failure occurs. The shear wall overstrength moment is more than 4 times your demand moment based on a ductility of 3.

This means that the wall will remain elastic at the point of shear failure and we remain convinced that considering the building to be nominally ductile is the correct approach.

If you should have any queries or require further clarification in connection with the above please do not hesitate to contact us.

Regards

Graeme Salter
Senior Design Engineer (Structures)