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Preliminary Soil Structure Interaction Analysis of an Instrumented Wellington Building

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ABSTRACT

This paper presents the initial development and validation for a numerical model of a building in Wellington that was instrumented with strong motion recording devices during a number of large earthquakes and aftershocks. The model considers both the structural systems, the underlying soil, and makes consideration for soil structure interaction effects. This validation study aims to demonstrate the predictive capability of the model through comparison with the responses recorded by the instrumented structure and provide a quantitative benchmark for the level of confidence that is reasonable to assign to forward prediction analyses using similar models. This paper discusses ongoing preliminary work towards these goals.

1 INTRODUCTION

Learning from past earthquakes is a vital part of earthquake resilience research. In terms of structural response to strong earthquake shaking, instrumented buildings are a critical tool, as they are able to record quantitative data describing the actual response of structural systems. Such data offer significant gains towards understanding structural response relative to the required approach for non-instrumented structures, which relies upon strong motion recordings of the response at the ground surface alone (Naeim ,1996). The recognition of the importance of instrumented buildings in New Zealand led to the development of the GNS building instrumentation programme (Uma et al. 2011), and due to this initiative, five instrumented buildings in Wellington captured the structural response to the 2016 Kaikōura earthquake (Chandramohan et al., 2017) in addition to previous seismic events such as the 2013 Cook Strait earthquake sequence (Ma et al., 2014; Thomson and Bradley 2015; Simpkin et al., 2015).

While this data is invaluable in regard to learning from these previous earthquakes, it cannot directly answer critical questions related to the response of multi-storey Wellington buildings when subjected to stronger levels of shaking. One way of addressing this key question is to develop validated numerical models for use in forward prediction analysis. In order to enable forward prediction for multi-storey buildings founded on different soil deposits, it is important that the validated numerical models make consideration for soil structure interaction (SSI) effects, which can significantly change critical structural performance metrics such as the distribution of inter-storey drift throughout the building (Naeim et al., 2008).

Previous New Zealand-led activities by Thomson and Bradley (2014) and Simkin et al. (2015) have demonstrated the potential for better understanding of structural behaviour enabled by monitored buildings. This paper presents preliminary results towards the validation of soil structure interaction models of an instrumented building in central Wellington. This ongoing project aims to not only provide a benchmark for the ability of numerical SSI models to capture key aspects of the response of the instrumented building, but also offers great potential for investigating other research questions. For example, the simplified analysis approaches commonly used in practice to account for SSI effects have been shown to be ineffective (e.g. Naeim et al., 2008), and the insights gained through the current efforts can be used to guide future practical applications of SSI analysis in New Zealand.

2 INSTRUMENTED BUILDING DETAILS

Following observations of structural and geotechnical damage due to the 21 July 2013 Seddon earthquake, seven buildings in central Wellington were instrumented with inexpensive accelerometers by Simkin et al., 2015. The current work focuses on one of these buildings, a 38 m high, seven storey reinforced concrete structure with a single storey basement. Figure 1 shows the selected structure in plan. The building is approximately 36 m in the long direction and 16 m in the short direction, and is oriented about 55˚ from north as shown in Figure 1. The lateral load resisting system consists of shear walls in the y-direction and a moment resisting frame in the x-direction that was later retrofitted with additional shear walls (Gultom et al., 2019).

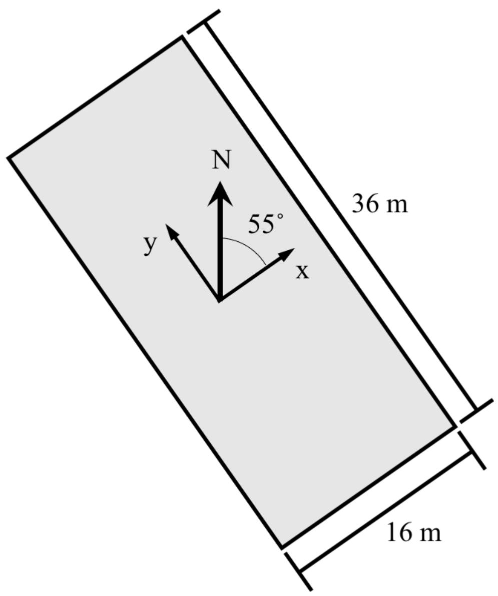

Figure 1: Plan view of instrumented building considered in this study.

The selected building was monitored during the 2013 Cook Strait earthquake sequence (Holden et al., 2013) from 24 July to 9 September 2013. During the 16 August M_w6.6 Lake Grassmere earthquake, the building was instrumented with three temporary accelerometers, with one each located on the basement slab, on the third floor, and on the roof (Gultom et al., 2019). Other events were recorded during the instrumentation period, but the Lake Grassmere earthquake caused the strongest response, thus all analysis considers only this earthquake. The results presented in this paper consider only shaking in the y-direction.

3 STRUCTURAL MODEL DEVELOPMENT AND FIXED-BASE VERIFICATION

A simplified structural model is developed and analysed for the y-direction of the structure using the OpenSees finite element analysis framework (McKenna, 2011). This simplified model does not attempt to capture the true complexity of the structure, but instead focuses on capturing the fundamental period and matching the vibration characteristics of the first mode. It is unlikely that higher modes are appropriately considered, but this approach provides a first-order model of the structure adequate for the current purposes.

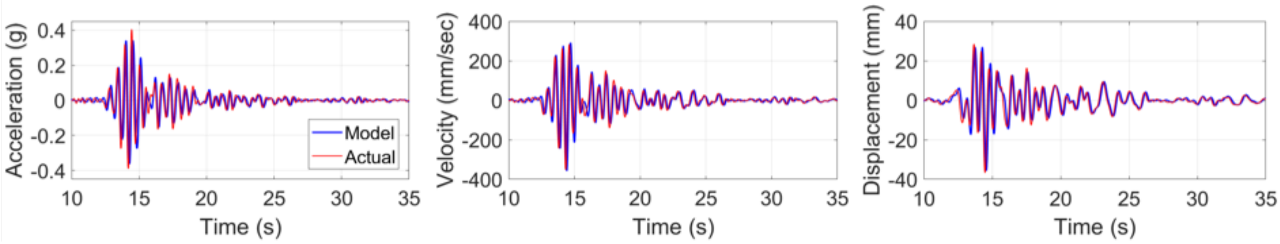

Figure 2: Comparison of fixed-base model and actual response of structure for Lake Grassmere earthquake.

A constant distribution of mass is assumed throughout the structure. Lumped nodal masses are based on tributary areas and simple dead load calculations. Linear elastic beam elements are used to model the beams and columns, with a single element used for each. The nodes in each storey are constrained to share equal horizontal degrees of freedom. Beam and column stiffness values are assigned to match the fundamental period to that reported by Gultom et al. (2019). Rayleigh damping is applied such that the damping ratio is 5% at the first and third modal periods. This damping scheme is carried across to the SSI models.

Figure 2 compares the acceleration, velocity, and displacement response recorded at the roof level during the Lake Grassmere earthquake to that obtained at the roof level in a fixed-base analysis of the simplified model when the acceleration recorded at the basement level is used as the input excitation. As shown, there is good agreement between the simplified model and the recorded behaviour. The acceleration response is slightly off, but this is likely due to differences in the higher mode periods. Gultom et al. (2019) reported that the fundamental period of the structure varied between about 0.4-0.45 s during the monitoring period for ambient vibration analysis and lengthened up to about 0.54 s during the Lake Grassmere earthquake. For the fixed based analysis, a fundamental period of 0.508 s produced the best match to the recorded data.

This fixed-base model is used to examine the likelihood of soil structure interaction effects in the recorded response for the Lake Grassmere earthquake by comparing the roof acceleration in the previous verification analysis to that obtained using the surface ground motion recorded at a nearby strong motion station (about 50 m from the building) as the input excitation rather than the recorded basement acceleration. The results of this comparison are shown in Figure 3. The clear difference between the model and observed accelerations using this nearby surface motion as input, coupled with the clear similarity of Figure 2 when the basement input motion is used, indicates that the basement motion has been altered relative to the free-field. The observation that the amplitude of the actual structure is less than the model is consistent with the hypothesis of SSI effects at this site, as such effects would serve to reduce the demands transmitted into the structure relative to the free-field demands. These results correspond to the findings of Gultom et al. (2019), who used spectral acceleration ratios to come to the same conclusion.

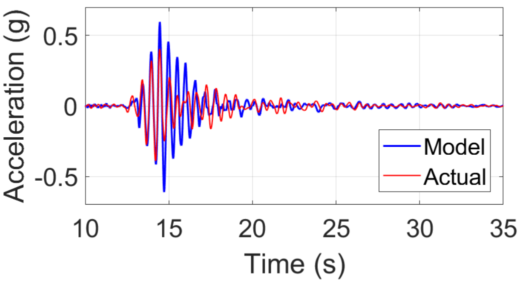

Figure 3: Roof acceleration in fixed base model using nearby strong motion record compared to observed roof acceleration.

4 SOIL STRUCTURE INTERACTION MODEL DEVELOPMENT

Figure 4 shows the soil structure interaction model considered in this study. The above-ground structural model corresponds exactly to the fixed-base model discussed in the previous section. The only difference is the addition of the basement slab and walls, which are defined with multiple elements to facilitate node-to-node links with the adjacent soil elements. The mass and stiffness of these basement elements are assumed to be the same as the remaining beams and columns of the model. For these preliminary analyses, the basement nodes are linked directly to the soil nodes such that the bodies share equal degrees of freedom at these points.

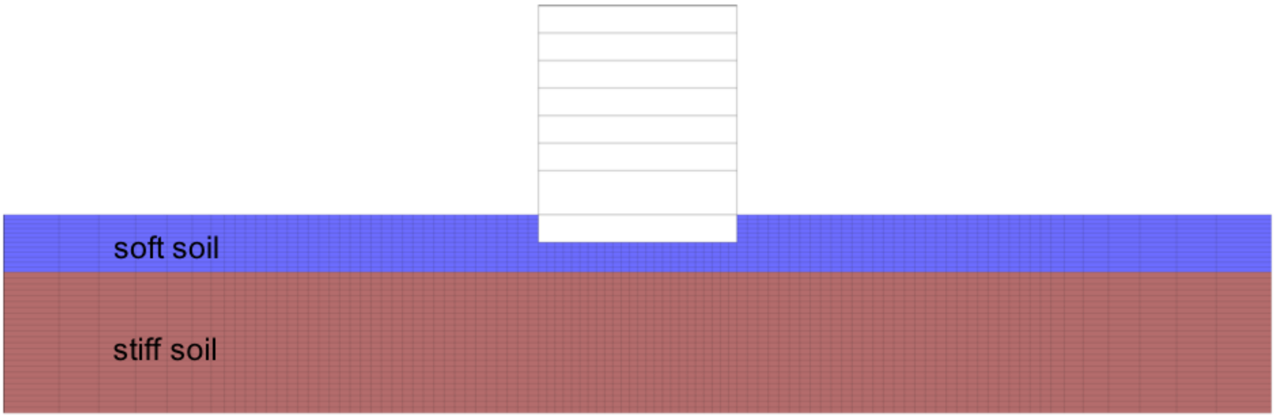

Figure 4: Finite element mesh for soil structure interaction model.

The site soils are roughly divided into two layers based on nearby CPT and SPT logs, an approximately 10 m thick layer of softer soil and a deeper layer of stiff soil. An idealised depth-dependent shear wave velocity profile is assumed where $V_s = 160z^{0.25}$ in the soft soil layer and $190z^{0.25}$ in the stiff soil layer for each element mid-point depth z. These simplified velocity profiles are based on the site characterisation work of Cox and Vantassel (2018) and the typical values assigned to different geologic deposits by Semmens et al. (2010). Figure 4 shows the layout of the soil mesh in the SSI model. The soil is modelled using stabilised reduced-integration quadrilateral elements (McGann et al., 2015) assigned an elastic isotropic constitutive response. Mass densities of 1.7 and 1.8 Mg/m^3 are assumed for the soft and stiff soil layers, respectively, and a Poisson's ratio of 0.3 is assumed for all soil materials. The soil mesh is 36 m deep and 230 m wide and the structural model in centred within this domain.

The base of the soil mesh is fixed against vertical translation, and the input ground motion is applied using the method of Joyner and Chen (1975) to account for a compliant base. Selective mesh refinement is used such that the elements are smaller near the structure than near the boundaries, and the boundaries are extended out horizontally in an attempt to minimise boundary effects on the structural response. Free-field boundaries are applied by assigning a very large thickness in the out-of-plane direction to the final column of elements at either horizontal extent and assigning periodic boundaries to the nodes for these elements. In this manner, the response of these exterior columns of soil is unaffected by the structure and true free-field motions are transmitted into the central part of the model. The interior soil elements are assigned an out-of-plane thickness of 20 m to account for incompatibility between the plane strain treatment of the model and the three-dimensional reality of the structure. This increased thickness provides better mass compatibility between the soil and structural portions of the model. In this preliminary SSI model, the solid and beam element nodes representing the soil and structure, respectively, are enforced to share equal translational degrees of freedom and no relative movement between the two bodies is possible.

The input motion for these analyses is taken from plane strain models of central Wellington developed and analysed for the Lake Grassmere earthquake by McGann et al. (2019). These plane strain models were developed for a vertical slice through the Thorndon basin that is aligned with the long direction of the instrumented structure, and the horizontal acceleration recorded in the plane strain model 36 m below the site of interest is used as the input ground motion in the current study. McGann et al. (2019) showed that the spectral acceleration in the plane strain model at the site of interest matched observations at the nearby strong motion station significantly better than corresponding 1D site response analyses. Though the current model is constrained to consider only 1D wave propagation up from the base, and the use of an input motion from the plane strain model is not strictly required, future iterations of the SSI model will incorporate the horizontal and vertical accelerations recorded in the plane strain model at all nodes on the boundary of the soil domain in the SSI model to consider the effects of more than just vertically propagating SH-waves.

5 RESULTS AND DISCUSSION

The performance of this preliminary SSI model is assessed through comparisons of the acceleration and displacement response at the basement, third floor, and roof of the structure. Because the SSI model undergoes global rigid body displacements and accelerations when applying the ground motion using the Joyner and Chen (1975) method, all quantities discussed in this section are relative to either the basement of the structure (for roof and third floor responses) or to the base of the soil mesh (for basement responses).

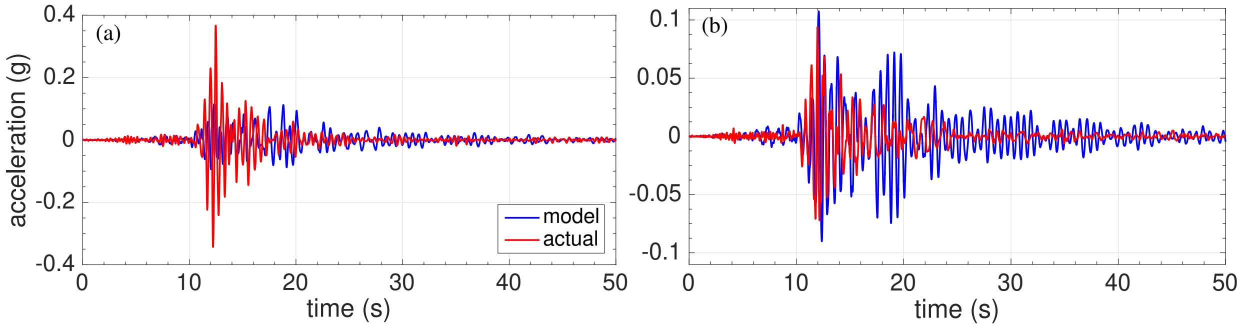

Figure 5: Acceleration time history response of structure in SSI model. (a) roof; (b) basement.

The horizontal acceleration response of the model structure is compared to the instrument recordings in Figure 5. Two key observations are evident in these results. The amplitude of the roof acceleration is significantly less than the actual building response, and while the peak acceleration at the basement is similar, the levels of shaking beyond about 15 seconds are far too large. An examination of the global deformation modes in the SSI model reveals the cause of this discrepancy for the roof response. As the soil domain is excited by the input ground motion, the structure undergoes largely rigid body horizontal translations, and little inter-storey deformation develops. Because the internal structural deformation is small, the acceleration of the roof relative to the basement shown in Figure 5(a) is small both relative to the observed response and in an absolute sense, despite the fact that the model basement acceleration is actually larger than what was observed in reality.

The displacement response of the model structure reveals a similar story. As shown in Figure 6, there is little difference between the horizontal displacements at the roof and basement of the model, and though the amplitudes are similar to the observed displacements, too many large amplitude oscillations occur. These repeated larger oscillations both before and after the strongest shaking in the observed response are due to the development of a rotational deformation mode in the structure, essentially rocking, during the analysis. As indicated by the acceleration response in Figure 5, this rocking response is largely a rigid body rotational mode for the structure and most of the deformation occurs in the soil. Figure 7 shows the deformed shape of the SSI model at the approximate time steps where the peak roof displacement response occurs in either horizontal direction. The essentially rigid body rocking of the structure is evident from these plots.

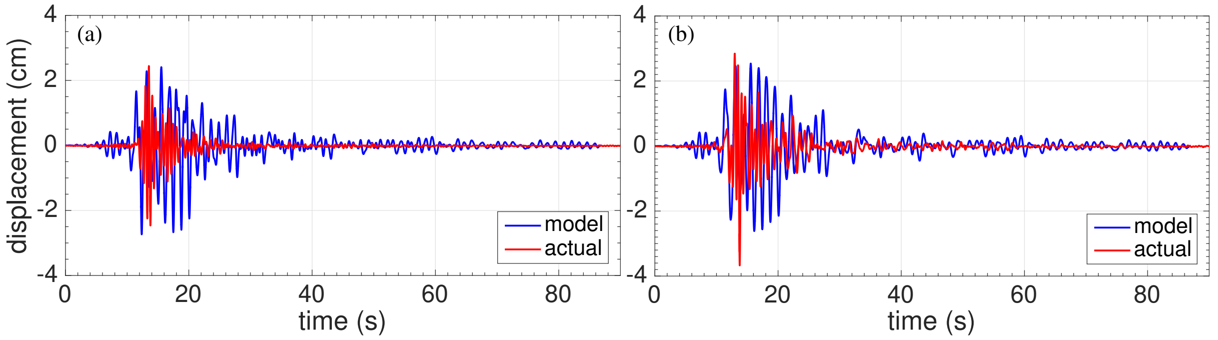

Figure 6: Displacement time history response of structure in SSI model. (a) roof; (b) basement.

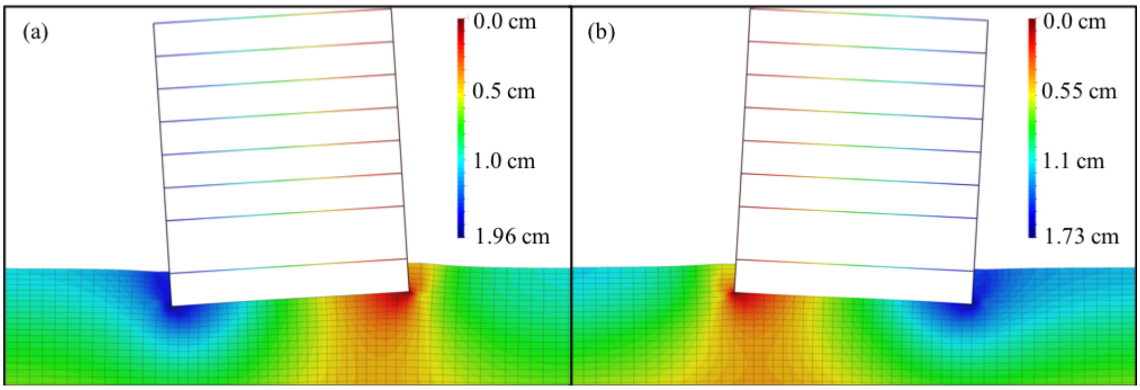

Figure 7: Snapshots of deformation at peak roof displacements during SSI analysis (magnified 100 times). Contours show magnitude of vertical displacement. (a) 12.3 seconds; (b) 15.5 seconds.

It is possible that a rocking response developed in the actual structure, but the number and layout of the instruments cannot allow this to be definitively determined. In any case, the model rocking amplitude is consistently too large with time, and the model rocking period is too long relative to the observations. Despite the relatively good performance in terms of wave propagation demonstrated by McGann et al. (2019) for the same shear wave velocity profile at this site, the near-surface soil may not be stiff enough relative to the structure in the SSI model. The actual structure has a piled raft foundation, and the fact that the piles are not considered in this preliminary analysis may also be exacerbating issues related to the soil-structure stiffness ratio.

6 CONCLUSIONS AND FUTURE WORK

A preliminary soil structure interaction model was developed and analysed for a building in central Wellington that was monitored with accelerometers during the 2013 Lake Grassmere earthquake. The SSI model was based on a simplified treatment of the structure, soil, foundations, and loading conditions, in which only linear elastic response was considered, the pile foundations were ignored and the structure was rigidly linked to the soil, and only 1D wave propagation was considered within the soil domain. Despite the clear limitations of this simplified treatment, the preliminary model successfully captured SSI effects, as the near-field acceleration response was quite different to the free-field response experienced in the soil near the model boundaries. The model also demonstrated the development of a rocking response that cannot be considered in a traditional fixed-base analysis, and overall serves as a solid proof-of-concept analysis that will be refined and expanded moving forward.

Future work will consider a more realistic interface condition between the basement and surrounding soil as well as the pile foundations that exist in the real structure, and will eventually make consideration for nonlinear soil and structural behaviour. Based on the results presented and discussed here, several aspects of the preliminary model have been identified as worthy of a further, closer examination. These aspects include: (1) the size of the soil domain relative to the structure, as it may currently be too small to eliminate boundary effects and wave trapping within the soil domain; (2) the stiffness of the soil near the basement, which may be too soft to adequately transmit the seismic demands to the structure in a manner similar to that observed in the field; (3) the damping in the preliminary model is currently based on structural vibration modes only, and may need to be amended to properly account for damping in the soil.

Once the issues with the preliminary model have been sorted out, the resulting SSI model will be used in a more extensive validation study using the multiple strong-shaking events recorded by the building. The validated SSI model can then be used in future forward prediction analyses to study the effects of different seismic demands and ground conditions (including ground improvement) on the structural response.

7 ACKNOWLEDGEMENTS

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