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Optimisation through an Alternative Solution approach incorporating soil-structure interaction

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ABSTRACT

The New Zealand Building Code is performance based. However, the most commonly adopted design methods utilise capacity-based Verification Methods such as B1/VM1. Although considerable time and cost is typically associated with design approaches which deviate from the Verification Methods, such as an Alternative Solution, it does present designers with the opportunity to assess the response of buildings with greater certainty. This allows for more explicit consideration of the stress and strain response in individual elements and in the founding soils, and as such an Alternative Solution approach can be particularly advantageous for the design of atypical buildings, buildings where foundation response is of particular interest, or buildings that integrate existing elements.

A case study is presented in this paper where nonlinear time history analysis was implemented within an Alternative Solution approach to validate reinstatement of a building that suffered earthquake damage by reusing the existing foundation system. The uncertainty in ground conditions and foundation response was able to be incorporated into the design through detailed ground modelling including the effects of liquefaction, allowing nonlinear pile springs to be developed and conservatism reduced through the assessment of individual pile performance. The performance of individual elements of the lateral load resisting system was also assessed, allowing for an efficient superstructure design.

The paper will discuss the modelling and analysis techniques adopted in application of the Alternative Solutions approach, including amalgamation of international best practice with NZ standards and modelling of and performance criteria adopted for piles and critical superstructure elements. The paper will also comment on the advantages of this approach in relation to conventional design approaches.

1 INTRODUCTION

The design of buildings in New Zealand is required to comply with the requirements of the New Zealand Building Code (New Zealand Government 1992). The stated objectives of the Building Code include ensuring that structures have a low probability of damage during frequent seismic shaking and life safety during infrequent seismic shaking.

The Building Code is a performance-based, rather than prescriptive, document. That is, it specifies the performance a building must achieve, rather than prescribing the way a building should be built. Compliance with these performance requirements can be demonstrated through the application of approved Verification Methods (such as “B1/VM1” (MBIE 2021) for general structural performance and “B1/VM4” for foundation performance), or the adoption of an Alternative Solution. An Alternative Solution refers to a design where compliance (in whole or part) with the relevant Building Code clause is demonstrated by a means other than a Verification Method or Acceptable Solution. This may comprise physical testing, or by Standards or other guidance that is not cited in the relevant Verification Method.

Approved verification methods are predicated on assessing building performance at the Ultimate Limit State (**ULS**) for common structural typologies. Verification methods have been developed to inherently consider the performance of a structure at levels of shaking greater than the ULS, and performance measures have been set to have an adequate margin against failure at the ULS with the resulting risk of collapse during seismic events of greater intensity deemed acceptable.

However, the assumptions inherent in approved verification methods result in an inconsistent life safety risk depending on the structural form and site, among many factors. Furthermore, application of the verification methods cited in the Building Code is only appropriate for the design of a subset of buildings, with other approaches often being more appropriate in the design of atypical buildings, buildings where foundation response is of particular interest, or buildings which integrate existing elements.

This paper presents a case study where the performance of a building reinstatement has been demonstrated to comply with the Building Code objectives via an Alternate Solution. The proposed building reinstatement included reuse of the existing foundation system. Therefore, the performance of the foundation system and the influence of the resulting soil-structure interaction on building performance were key design considerations. The use of nonlinear time-history analyses within the Alternative Solution framework allowed for direct verification of the performance of both the foundation and superstructure elements in accordance with the objectives of the Building Code and optimisation of the design outcome.

2 PERFORMANCE VERIFICATION OF BUILDING REINSTATEMENT RESPONSE

The superstructure of the original building in the case study presented in this paper comprised of precast flooring with insitu topping supported on steel framing, with the lateral load resisting system comprising of two levels of Eccentrically Braced Frames (**EBF**s) and three levels of Concentrically Braced Frames (**CBF**s). The foundation system comprised of a shallow reinforced concrete slab supported on a series of screwpiles.

Reinstatement of the building required deconstruction of the damaged superstructure and replacement compliant with contemporary Building Code design requirements. Damage to the original building during the earthquake was deemed to be isolated to the superstructure, with no discernible damage to the foundation system following extraction and inspection of screw piles. Therefore, the proposed reinstatement included re-use of the existing foundation system with modification as required.

The reinstatement was required to comply with the performance objectives of the Building Code including, where appropriate, the provisions of the relevant NZ material and loading standards. The re-use of existing screwpiles in soils which were expected to experience liquefaction for earthquake shaking equivalent to

ULS, and greater, introduced significant nonlinearity into the foundation response. Therefore, the overall response of the reinstatement building when subject to significant shaking was expected to be significantly influenced by the foundation response. The project team determined that it would be overly onerous to assess the response of the building using approved verification methods due to spatial variability in the ground conditions across the site, significant nonlinearity in the foundation pile response and the significant change in lateral stiffness up the height of the building. The adopted Alternative Solution approach was deemed appropriate for addressing the structural response of this building including soil-structure interaction as it allows for direct consideration of the nonlinear response of the piles.

When applying an Alternative Solution to building design, the risk of collapse during seismic shaking of greater intensity than that corresponding to ULS requires explicit consideration. Common international practice is to explicitly assess building performance at a seismic event of significantly greater intensity than the ULS, termed the Maximum Considered Earthquake (**MCE**) to ensure that an acceptable margin of safety to collapse is achieved.

The primary performance criteria of interest with respect to the seismic performance of the reinstatement were:

- Building displacement to adjacent existing structures and boundaries,
- Rotation in the EBF active links,
- Displacement in the existing, and any new, screwpiles, and
- Tensile force in pile-to-pile cap connections for existing screwpiles.

The alternative solution approach comprised a series of nonlinear time-history analyses applied to a finite-element model of the building. The model was developed in ETABS v20 to represent the primary gravity and lateral load resisting systems of the building, and a 3-D isometric view of the ETABS object model is presented in Figure 1.

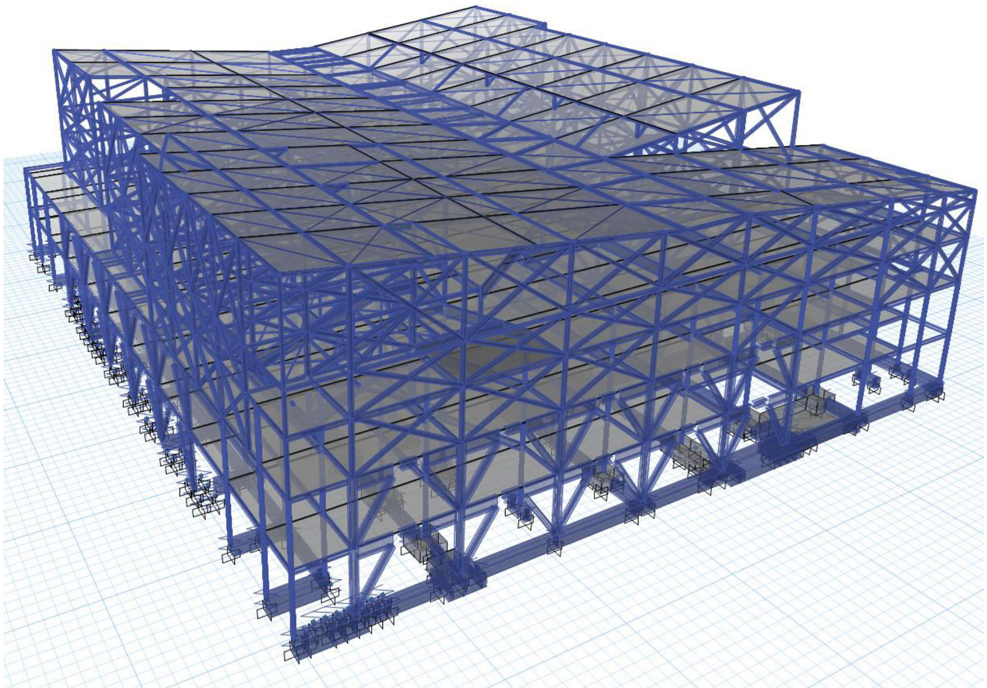

Figure 1: 3D Analysis Model

The analysis model included nonlinear elements to represent the EBF active links and the screwpiles. The EBF active links were defined as ductile shear hinges to model the plasticity in the active links. The screwpiles were modelled as multilinear elastic links with a series of pile-deformation curves to represent individual pile response as discussed below and presented in Figure 4.

3 GROUND CONDITIONS AND FOUNDATION RESPONSE

To develop the pile-deformation curves required in the performance-based assessment of the structure, a comprehensive understanding of the ground conditions and foundation response was required. As mentioned, the original building in this case study was founded on screw piles that were not damaged in the earthquake. The screw piles are founded in a dense gravel layer within a deep alluvial soil profile in an area of New Zealand with potential for strong seismic shaking. Below the dense gravel layer was a looser sand and silt layer which was assessed to trigger liquefaction during ULS design seismic shaking. Liquefaction was not predicted to trigger in the dense gravel layer. Therefore, understanding the variability in depth and thickness of the founding gravel layer for the screw piles was critical for the assessment of the screw pile seismic response.

A 3D ground model was developed for the site to understand the spatial variability of the soil layers, particularly the founding gravel layer. Figure 2 presents contours of founding gravel thickness across the site, which generally started between 5 m and 7 m below the ground surface. The gravel layer is between 2.4 m and 4 m thick and typically 3 m thick across most of the site, with a thicker region in the north-west and a thinner channel feature in the south-west. The gravel thickness information was combined with screw pile as-built records to determine the thickness of the gravel beneath the helix, as shown in Figure 3, as this was critical to the extent that liquefaction of the layer below would affect seismic pile capacity and stiffness.

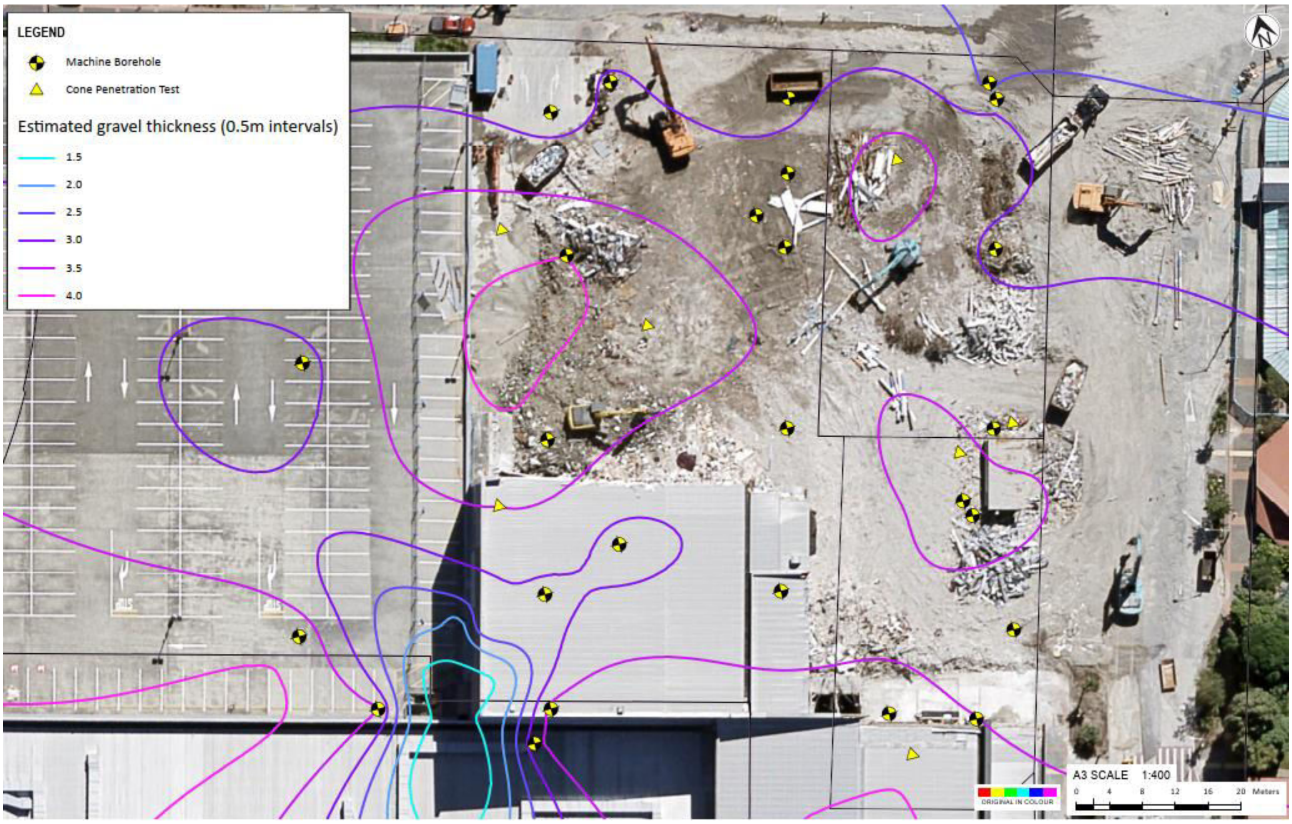

Figure 2 Contours of the thickness of the founding gravel layer at the site.

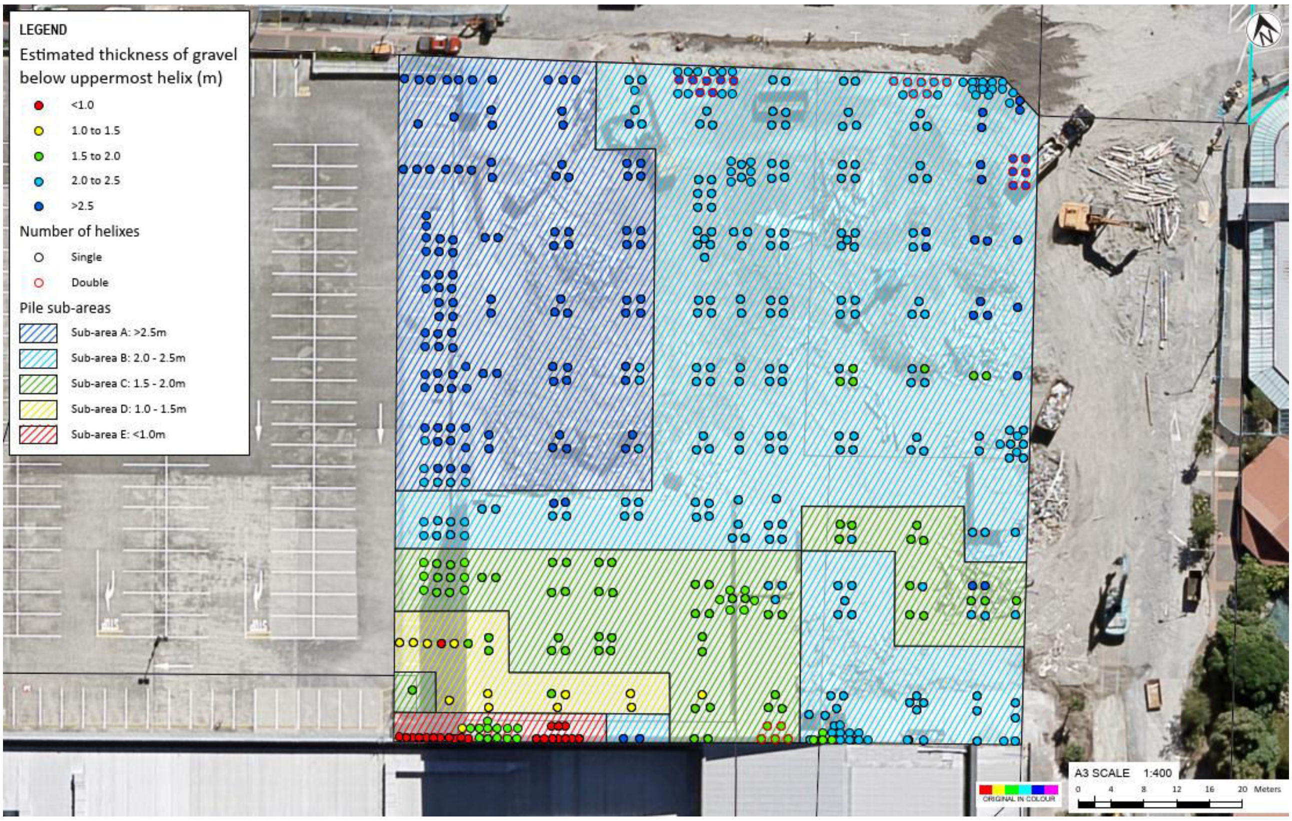

Figure 3 Thickness of gravel beneath the screw pile helices and pile foundation sub-areas used in design.

Sub-areas of gravel thickness below the screw pile helices were developed across the site, as shown in Figure 3, to facilitate development of vertical screw pile deformation curves (springs) to be used in the nonlinear time history analysis. Available screw pile testing was used to calibrate 3D finite element models of the screw piles using Plaxis software and then upper and lower bound pile springs were developed using these models with consideration of the variation in gravel thickness and liquefaction of the layer beneath the gravel. Figure 4 presents the upper and lower bound springs for the case where liquefaction occurs beneath the founding gravel for the different sub-areas.

In Figure 4 the dark blue curve represents the stiffest pile response including liquefaction triggered in the layer below the founding gravel, where the gravel layer is greater than 2.5 m thick beneath the screw pile helix. The red curve represents the softest pile response with liquefaction for locations where the gravel beneath the helix is less than 1 m thick. The black line (both compression and tension) represents the upper bound case where liquefaction has not triggered, which is likely early in the earthquake time history before excess pore water pressures significantly generate to trigger liquefaction. On the tension side there are no coloured lines because the pile helices are sitting at the top of the gravel and all piles are expected to respond similarly in tension. The lower bound tension case (light grey line) represents when liquefaction has been triggered in the layers above the founding gravel. These springs could then be directly used in the nonlinear time history analysis of the building structure.

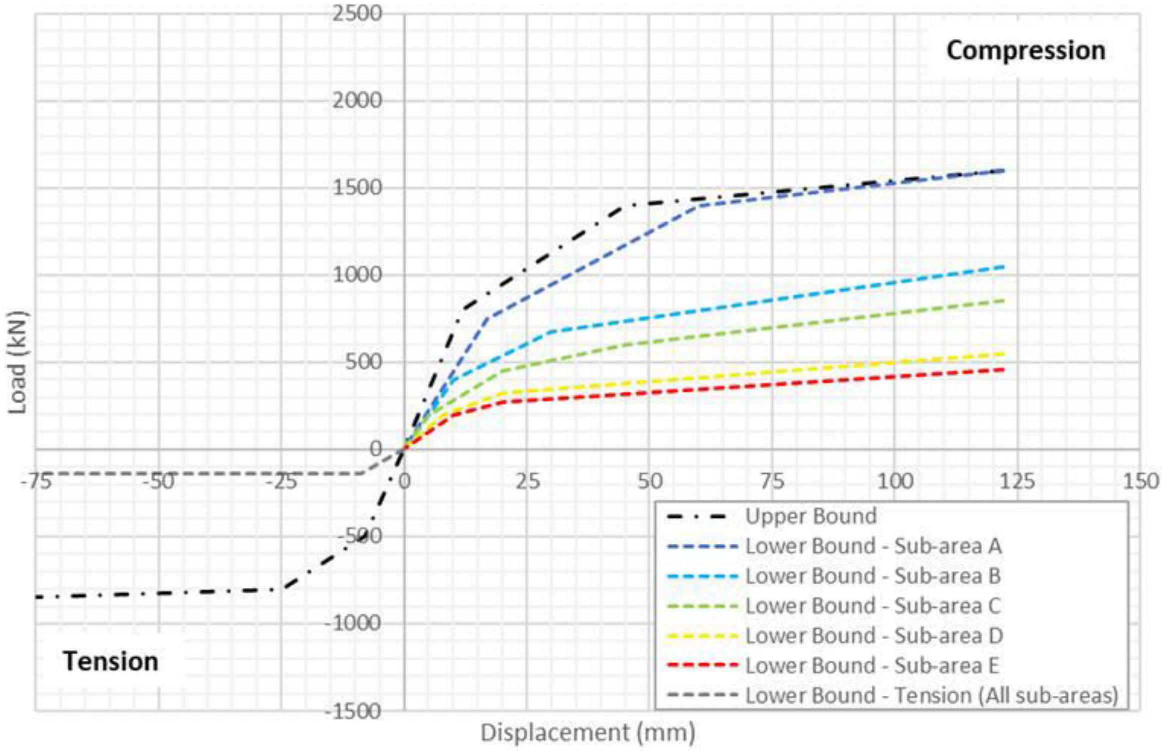

Figure 4 Upper and lower bound liquefied design case vertical pile springs.

4 PERFORMANCE VERIFICATION OF BUILDING REINSTATEMENT

4.1 Nonlinear Time History Analysis for performance verification

The building is an Importance Level 3 (IL3) structure in accordance with AS/NZS 1170.0, with a 50-year design life. The resulting ULS demand corresponds to shaking with a recurrence interval of 1,000-years. The MCE event has been defined, with reference to NZS 1170.5 (SNZ 2016, 5), as the lower of the 1 in 2,500-year event, and the deterministic 84th-percentile shaking for the Alpine Fault.

Selection and scaling of ground motion records was carried out in accordance with the provisions of ASCE 7 (ASCE 7 2016, 7), to align with international best practice. A suite of eleven acceleration time history records were scaled to characterise the seismic demand at both the ULS and MCE.

The adopted performance verification approach allowed for direct assessment of the performance criteria, through explicit consideration of EBF active link rotations, pile displacements, building displacements and pile connection forces. The approach also allowed direct consideration of several areas of uncertainty with influence on the response and performance of the reinstatement building, including:

- Uncertainty in ground conditions and soil response, which was addressed through a suite of lower and upper bound analyses based on geotechnical guidance as discussed above.
- Variation in the response of the EBF active links due to variability in material properties. This was addressed through the application of both probable strength and upper characteristic properties in definition of the active link elements, resulting in two suites of analyses.

4.2 Considerations for adopting NLTHA within the Alternative Solutions Framework

The adopted performance verification approach required the consideration of a number of technical issues to ensure the objectives of the Building Code were addressed and verified with sufficient rigour. This included appropriate ground modelling and structural analysis techniques, identification of suitable performance criteria limits at the ULS and MCE, and the implementation of design procedures to preserve capacity design principles.

A uniform gravel thickness may have been assumed across the site to align with the simplified analysis within a B1/VM1 approach, leading to potential over or under conservatism. If sub-areas were established, pile capacities would have been set in each sub-area and reduction factors applied. Secant stiffnesses would be adopted with an assumed magnitude of pile displacement and a displacement limit would likely be imposed on each individual pile. Then the load in each pile and the displacement under that load would have to be checked against the capacities and displacement limits, and where these were exceeded reassessment and iteration would be required. A B1/VM1 approach would have required a significant number of iterations between geotechnical engineer and structural engineer due to the complexity of considering the variability in the ground and nonlinearity of the pile response, and a much more expensive foundation solution may have been proposed because of this difficulty.

The Alternative Solution method adopted enabled a level of granularity in the geotechnical analysis which is not typically viable within the framework of B1/VM1. The uncertainty and spatial variability in the gravel thickness and the effect of liquefaction could be captured in the foundation modelling and then applied directly in the structural analysis to determine whether the existing screw piles could be reused for a new structure. Similarly, the differing response of the piles in tension and compression was able to be directly incorporated in the modelling, which could not be implemented within a B1/VM1 approach. If the same pile springs had been developed for use with a verification method framework, a significant number of iterations would likely be required along with conservative assumptions on capacity and stiffness.

New Zealand guidance for the performance verification of building performance outside of B1/VM1 is limited. Codified provisions for the implementation of non-linear time history analysis are not in line with international best practice, whilst the response of, and performance criteria for deformation-controlled elements beyond the ULS is generally not addressed due to the focus of B1/VM1 approaches on ULS performance. Therefore, several provisions from international standards were incorporated into the analysis and design. This included reference to ASCE 7 for guidance on the number and application of ground motion records for analysis, as well as for the determination of demand parameters for design. The nonlinear force-displacement backbone curves for the EBF active links were developed through reference to a combination of NZS 3404 (NZS 3404 1997) and ASCE 41 (ASCE 41 2017, 41) to allow for inclusion of response and performance criteria at the ULS and MCE. Two separate backbone curves were developed for each active link size, one based on probable material properties, and another based on upper characteristic material properties. Sizing of EBF elements, and associated connections was undertaken in accordance with the requirements and procedures of HERA P4001 (HERA 2013), maintaining the principles of capacity design within the final solution. Use of the upper bound active link strength ensured that the design of the column and foundation elements would be undertaken in line with the principles of capacity design.

Individual pile response was modelled using elastic multilinear link elements. Consideration was given to the effects of soil plasticity and the ratcheting associated with this plasticity on the structural response. Difficulties associated with incorporating plasticity included the incorporation of gapping behaviour of the ground beams in conjunction with uncertainties with the hysteretic form of the pile response. A comparative study was undertaken between two sets of models which included.

- soil plasticity incorporating various pinched hysteresis models, and 2.5% viscous damping, versus
- no soil plasticity, and 5% viscous damping.

Results of this study indicated that building response could be bounded using an approach that does not incorporate soil plasticity. It was determined that excluding the contribution of vertical soil stiffness at the ground beams on compressive resistance produced conservative design actions.

4.3 Optimisation of design through direct performance verification

Whilst the adopted performance verification incorporated a number of assumptions and some conservatism where deemed appropriate, the approach also proved advantageous in several respects. Key advantages included efficiencies in the design workflow, optimisation of design, and enhanced confidence in the rigour applied to the final design solution.

The reinstatement building included a large number of discrete elements, including EBF active links and piles, which were expected to respond in a nonlinear manner. Accounting for this nonlinear response, particularly that associated with the piles, within a B1/VM1 approach would require the application of secant stiffnesses in the modelling and result in sufficiently excessive iteration to render the process practically unviable. Alternatively, conservative assumptions for pile stiffness and capacity would need to be adopted and may influence the overall viability of the design solution. The adopted performance verification approach allowed for direct assessment of the nonlinear response of individual piles, obviating the need for iteration or excessive conservatism.

The modelling of individual pile elements and direct assessment of element response, including both pile displacements and EBF active link rotations, was a key component of the approach. This allowed the project team to introduce additional piles at locations for which pile displacements were deemed unacceptable, and each EBF active link could be sized based on rotations determined in the analysis. As the remainder of the lateral load resisting system was sized in accordance with capacity design principles, optimisation of EBF active links also flowed through to optimisation of the remainder of the steelwork and connections.

The response of some building typologies, particularly those for which the soil and foundation response is significantly influential to the overall building response, may include significant events that result in a step change in seismic response at shaking intensities between the ULS and MCE. This step change may be due to events such as liquefaction, rocking, or ground stability. The influence of such events with respect to life safety risk beyond the ULS is not adequately assessed within B1/VM1 approaches. Therefore, verification of building performance at the MCE allowed for direct assessment of structural integrity at this limit state and provided enhanced confidence in a sufficiently low life safety risk.

5 CONCLUSIONS AND RECOMMENDATIONS

Application of an Alternative Solution approach, in the form of nonlinear time history analysis and direct assessment of building performance, allowed for optimisation of design and re-use of the existing screw piles for the reinstatement of a building that suffered earthquake damage. This was because the adopted compliance approach allowed for:

- direct assessment of element response, particularly EBF active link rotations and pile displacements,
- explicit consideration of individual element response, including variation of pile response based on geology and on tension/compression, without the onerous iteration required if using B1/VM1 approaches, and
- direct assessment of performance at MCE, whilst accounting for step changes in behaviour beyond the ULS associated with foundation response that would not be considered within B1/VM1 approaches. Additional performance levels, such as intermediary damage limit states, could be incorporated and readily assessed within this approach as required.

Demonstration of Building Code compliance through the use of Verification Methods may have resulted in the existing, undamaged piles being abandoned due to an onerous analysis and design framework and the resulting assumptions introducing sufficient conservatism to result in re-use of the piles being unviable. The variability in the ground conditions at the site was critical to seismic pile capacity and stiffness. The adopted approach allowed the development of pile-deformation curves that could be directly applied in the time history analysis without the need for onerous iteration and resulted in optimisation of the structural design with consideration of these affects.

New Zealand does not have a ratified procedure for performance verification outside of B1/VM1. This is an oversight in the Building Code framework as B1/VM1 does not, and cannot, address all building typologies and associated design solutions. To remedy this, an understanding of building and elemental performance at levels other than the ULS and the development of a corresponding philosophy and guidance is essential for consistency of outcome within the Building Code framework for both Acceptable Solutions and Verification Methods.

6 REFERENCES

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