

Seismic Upgrade of Wellington City Council's Newtown Park Flats

A.G. Cattanach

Dunning Thornton Consultants Ltd, Wellington.

S. Palmer

Tonkin and Taylor Ltd., Wellington.



2013 NZSEE
Conference

ABSTRACT: Wellington City Council is the second biggest landlord in the country behind Housing New Zealand, and their stock includes many medium and high-rise apartment blocks constructed in the 1960's and 70's. The Housing Upgrade Project involves seismic, architectural and services upgrade of all buildings to a varying degree, in a \$200M project over 10 years. Newtown Park is the largest complex, with five structures housing over 250 units. Though all five blocks were designed by the same Architect and Engineer, the depth to rock, piling method, era of construction and refurbishment philosophy has required different engineering responses to each.

The buildings were supported by bored piles with and without belled bases and driven expanded base piles (Franki). The historic borehole data was limited and the pile founding depths were unknown. Initial assessments identified that foundation uplift capacity was critical to the strengthening design. An investigation programme including boreholes and cyclic load testing of existing piles was developed. The testing method, the results and how this information was coordinated with the structural design is discussed: especially the importance of soil-structure interaction for stiff, limited ductile structures.

The structural solutions range from augmenting the existing strength, and two methods of superstructure de-stiffening with added energy absorption. The building typology (many stiff transverse walls, and perforated longitudinal walls) is common in similar government and private developments of the era that are now reaching the end of their first "design life". The range of solutions shows that with care and innovation, the existing fabric can be re-used in an economic manner.

1 INTRODUCTION

The Wellington City Council's Housing Upgrade Project (HUP) is intended to significantly improve the social environment within the Council-subsidised accommodation. The 30-40-year-old projects had degraded in social, seismic and safety terms. With a significant accumulation of deferred maintenance, an upgrade sponsored by both local and central government was planned.

From a structural point of view, this required addressing any significant degradation issues (as the buildings were intended to last for at least another 30 to 50 years without maintenance), and increasing the seismic capacity to greater than 70% of new building standard (NBS). This target was conceived within the Council before the HUP began, and was generally supported by the various consultants as they were appointed to the projects, given the scale and nature of the accommodation.

A project budget and programme over the subsequent 10 years needed to be established quickly in order to secure funding. This required Phase 1 of the HUP, which is described below, and was carried out by a group of consultants. Once funding was secured, these projects were staged over a 10-year period and tendered out to architect-led design teams. This paper deals with the Newtown Park project, which comprised six existing buildings and over 250 existing units. Built in a 1970s semi-brutalist style, the blocks had a poor reputation in terms of amenity, appearance and crime/safety.

2 PHASE ONE

Phase 1 of the HUP involved assessments of over eight major complexes in a period of less than nine months in 2008. This was a time when many Wellington consultants were busy on new building projects and resources with experience in seismic retrofit were scarce. Dunning Thornton and Holmes advocated for appointing a group of engineers to review the portfolio rather than one to two firms as originally envisaged. In this way, the assessment load was spread and there was an ability to cross-review the results of each investigation.

The assessments were based on original drawings, a walk-over survey and any IEP (or the occasional detailed) assessments to date.

The scope involved a reduced Detailed Engineering Evaluation, in which the approximate yield coefficient and ductility was established and any critical structural weaknesses identified. For the larger blocks, the study went on to identify possible retrofit schemes to achieve the 70% NBS target. These were intended to establish approximate budget rather than to be the definitive solution. Feedback from the Wellington City Council has suggested, with some exceptions, that this process did give a reasonable estimate of the structural upgrade budget required.

2.1 Phase One Results - Newtown Park

Newtown Park comprises three towers, two medium-rise block structures and a sixth block which, L-shaped in plan, was irregular and stepped up the adjacent West bank. In general the blocks conformed to the expected structural pattern for concrete residential buildings: transversely, the many inter-tenancy walls provide significant strength and robustness, but longitudinally, the arrangement, perforated by doorways and service rises, was typically poor.

In the towers, a Critical Structural Weakness (CSW) existed where the walls stepped in plan position above the ground floor. The blocks relied on their shear walls being held down by Franki piles: the way these are constructed brings into question their reliable tension capacity considering the termination of bars into the bulb base. The sixth block had structural, insulation, urban, security and services issues. It was decided that refurbishment of this block was uneconomic.

The exposed concrete walls of the blocks were typically covered in a thin cement render. Signs of degradation were limited to one or two discrete places. Although no chemical testing was carried out at the time, subsequent investigations showed that chloride and carbonation penetration was minimal. However, as shown on other projects, it is prudent to investigate this when considering refurbishment of concrete buildings, as it has a significant bearing on both upgrade cost and the future maintainable life of the structure.

3 PHASE TWO - REVITALISATION

Dunning Thornton were appointed as sub-consultant to Studio Pacific Architecture for Phase Two of the Newtown project. The scope of the work involved:

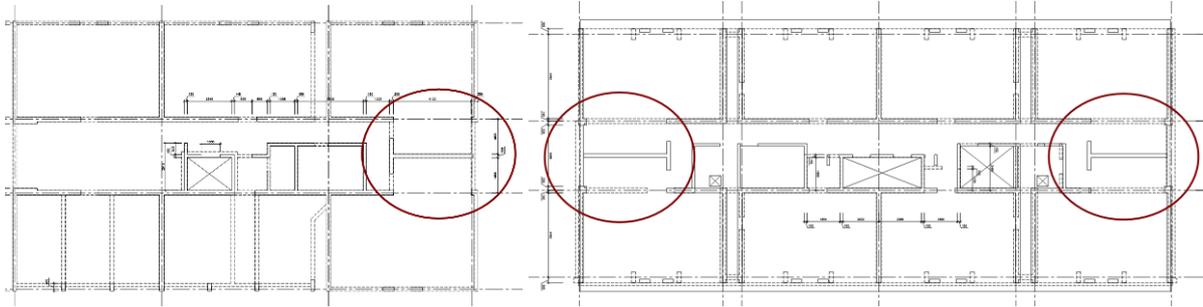
- Structural upgrade to greater than 70% NBS
- Improvements to circulation and fire escape
- Improvements to insulation, fixtures and fittings
- Alterations to the mix of units (studio, one-bed, two-bed, three-bed) to retain a mixed and secure social environment
- Threading the services required for the above through the existing structures

Where new holes (typically for doorways for unit amalgamations) were required in the existing tower walls, thickenings were added around these penetrations to provide the same or greater shear and flexural capacity. Where several of these openings occurred above one another, diagonal reinforcement was used to provide additional ductility to account for any minor redistribution of forces

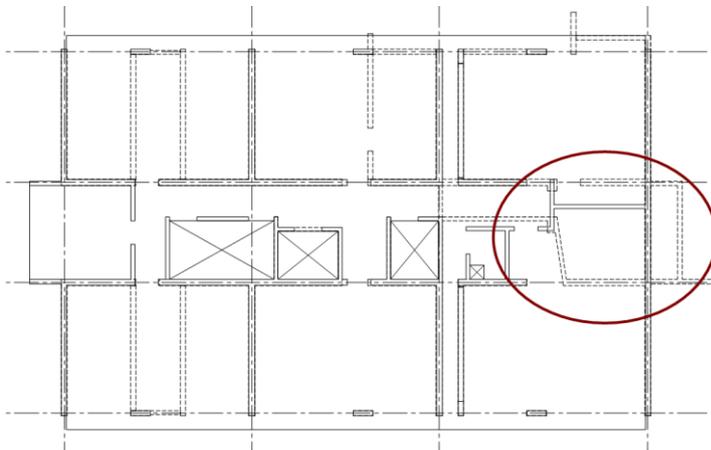
between floors.

3.1 Structural General Arrangement

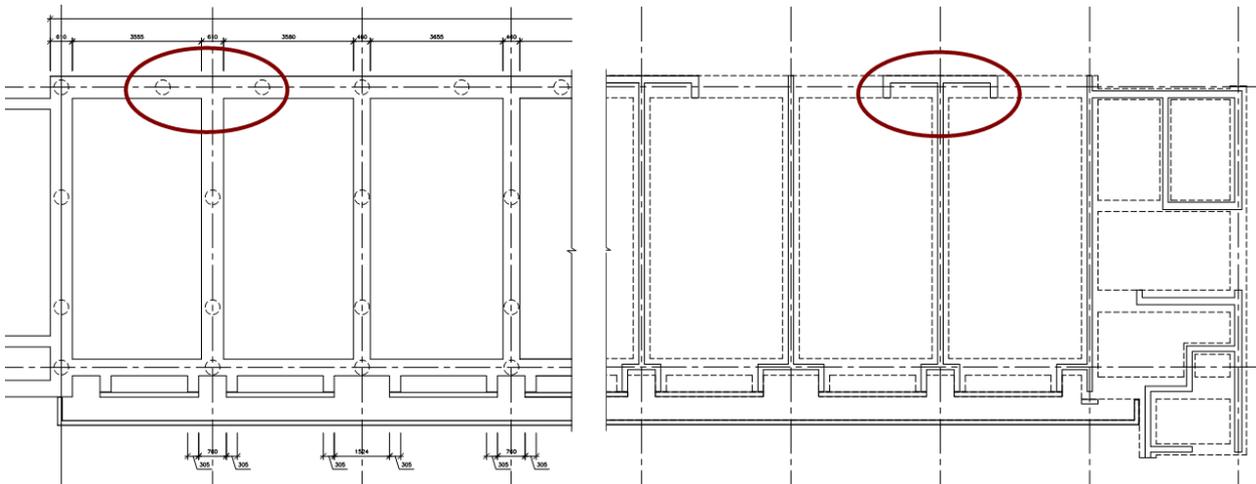
Plan extracts highlighting the critical structural items are shown below



Fire and Mid Towers



Stadium Tower:



Russell and Mansfield Block composite drawing:

4 SITE GEOLOGY

The site was the old terminus for the Wellington tramway, and had both geotechnical and geoenvironmental issues. The Northern part of the site has shallow depths (2-5 metres) to Wellington weathered greywacke rock, whereas the Southern areas have more than 15 of silt sand and gravel alluvium over the rock. This posed complexities both for the seismic soil class attributed to each

building, and in the likely hold-down capacity of the piles below the shear walls, with some piles founded in rock and others in alluvium.

Because the structures predominantly comprised shear walls, foundations were quickly determined to be critical elements in the overturning capacity of the buildings. No as-built piling records existed, and the only information available other than the design drawings was the vague recollections of a renowned Wellington piler who worked on the project early in his career.

5 FOUNDATION INVESTIGATIONS

Desktop investigations suggested that each building or group of buildings had a different piling system. Piles include:

- 400mm diameter bored piles in alluvium and in some instances into rock.
- 450mm diameter bored belled (700mm) piles socketed into rock.
- 400mm diameter driven expanded base, cast insitu ('Franki') pile in alluvium and in some instances onto rock

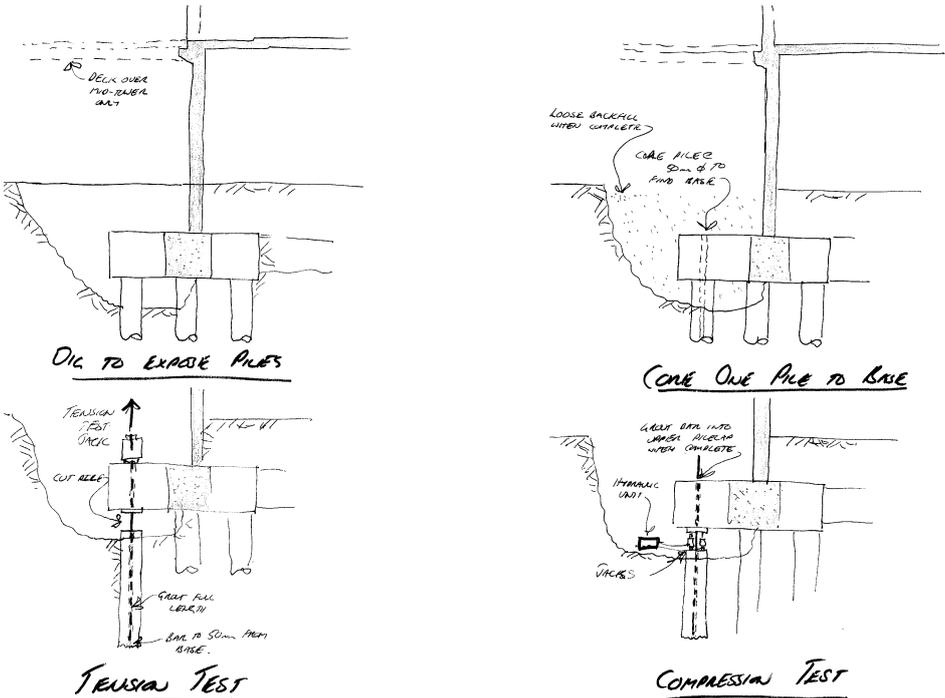
Typical bearing and skin friction capacities were quickly shown to be inadequate for the loads imposed. The failure mechanisms were uncertain, and of particular concern was the structural tension capacity of the Franki piles. It was not seen as practical to carry out extensive superstructure upgrading to reduce foundation loads: pile testing to ascertain the real available capacities was expected to provide a significant benefit to the project.

The testing regime was determined by identifying critical pile types and founding conditions. Four bored piles were selected for testing in tension compression and cyclic loading. Three in alluvium, one to rock.

The cost of the testing was \$200k and with the results, was significantly less than the additional structure that would have been required without the testing.

5.1 Pile Testing Methodology

Individual piles from pile groups were selected for testing which allowed access for testing, adequate continued support to the building by the other piles in the group while the selected pile is tested, and reaction from the pile cap and structure in tension and compression



The pile caps and top 1.0m of the test piles were uncovered by excavation. A concrete core was drilled along the length of the test pile. A 500mm long section of the pile was cut out. The compression load jack was positioned in the cut out section of the pile. A Macalloy bar was grouted into the full length of the pile. The tension load jack was attached to the Macalloy bar above the pile cap.

Compression, tension and cyclic testing then proceeded in accordance with AS1259 Piling – design and installation.

5.2 Pile Testing Results

Testing indicated variable conditions of the existing piles:

- Piles that were detailed on historic drawings as straight shafted were found to be belled.
- A thin silt layer below one of the tested piles resulted in large (50mm) displacement on compression loading.
- In one pile the concrete was found not to be continuous. Collapse of the pile (necking) is expected to have occurred during construction. This pile failed in tension during the cyclic load testing.

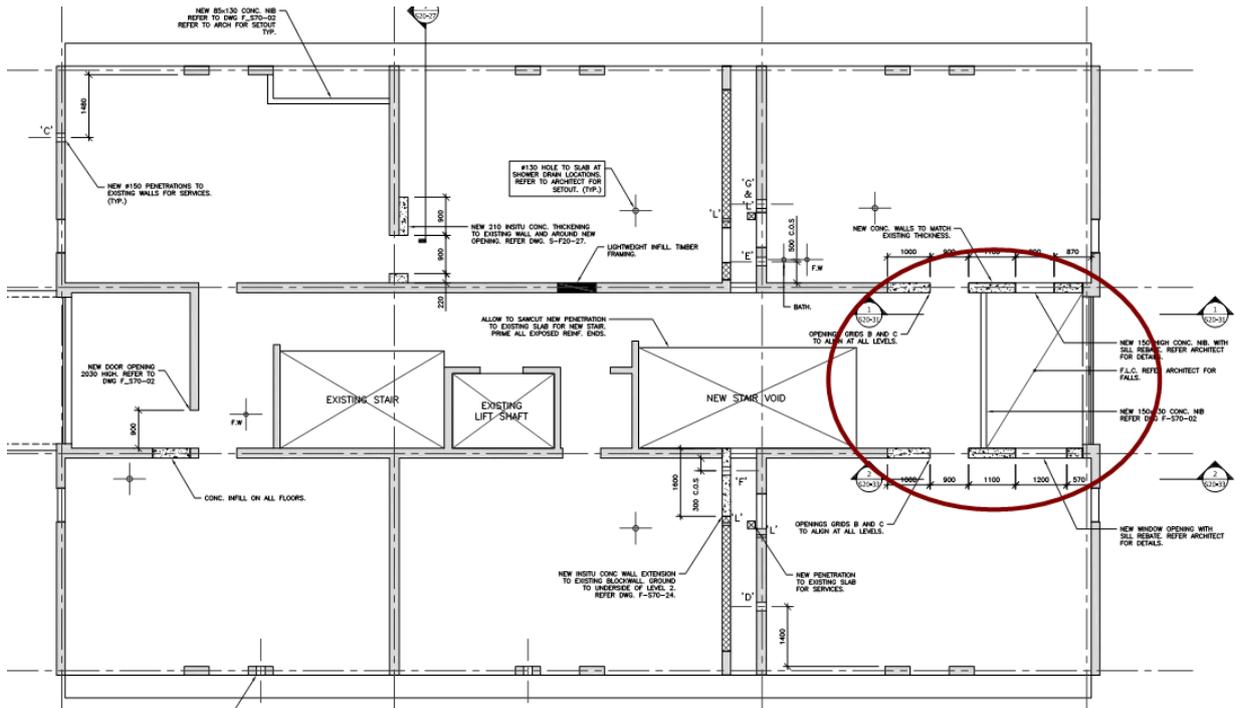


The tension failure of one of the test piles due to a structural defect threw into question the robustness of a primary structure reliant on piles only. A series of “what if” scenarios were investigated, considering a defect in one, two or three piles in a building. The total loss in overturning capacity was only 10-25%, and as such, the risk was considered acceptable without further investigation.

6 SEISMIC UPGRADES - TOWER BLOCKS

6.1 Towers

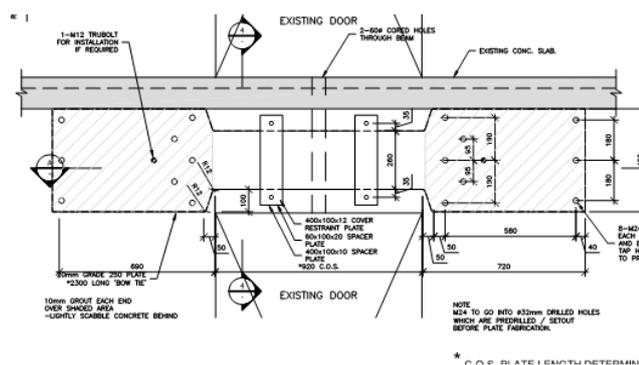
Fire Tower was strengthened by the removal of the CSW through rearrangement of the structural walls so that they were regular top-to-bottom of the building and ran through to the outside wall. This is shown in the diagram below.



Plan on Fire Tower showing longitudinal walls continued to the end of the building

Similarly, for Mid Tower, this CSW was removed at both ends of the building. The additional number of storeys and poorer soil on this site resulted in a slightly lower final capacity.

In architectural terms, Stadium Tower retained far more of the existing joinery and finishes. As such, the structural intervention was required to be far less severe. De-stiffening the end of the building with the CSW meant its brittleness was removed. However, this significantly reduced the overturning capacity for the building. To counter this, the existing spandrels (with limited ductility) were cut and new steel elements were added with less strength but significantly greater energy absorption capacity. These “bow ties” were flexural yielding plate elements as shown below. The cut in the existing spandrels was achieved without compromising the fire resistance of the spandrel by coring two holes vertically down through the doorways as shown in the detail below.



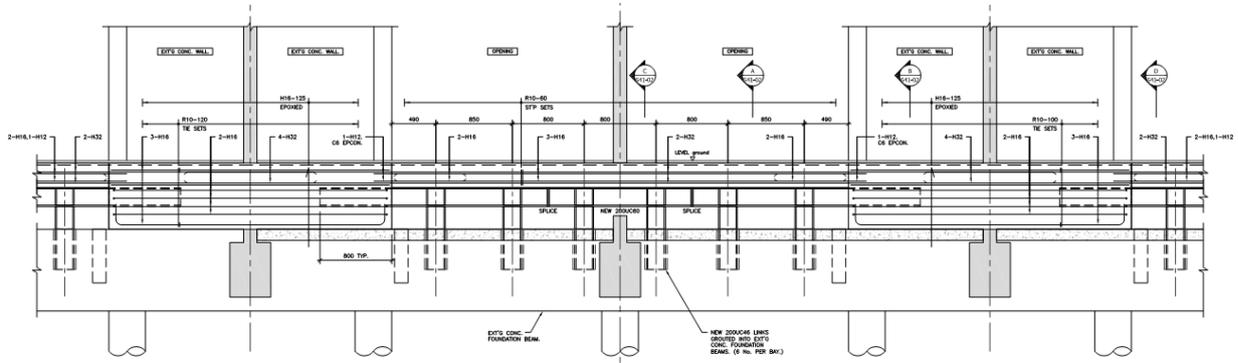
By utilising this system, foundation failure was precluded by limiting the input loads to the over-strength of the bow tie elements. These were inserted in the main corridors (areas with significant architectural rework), requiring little disturbance to the units behind.

6.2 Blocks

Mansfield and Russell blocks were both strengthened longitudinally by addressing the rocking of the existing shear walls on the potentially brittle Franki piles. A new energy-absorbing foundation beam was introduced to provide the base hold-down that could not be relied upon from the Franki piles (see

diagram below).

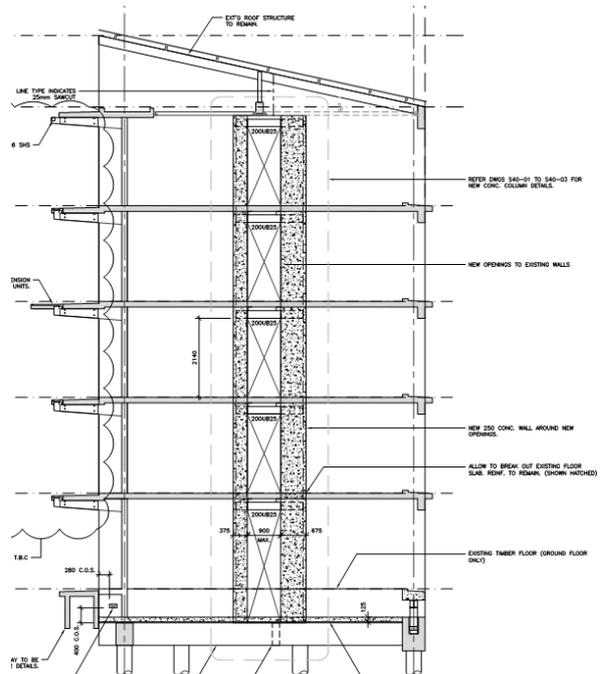
In order not to overload the existing walls in shear, the new foundation beam was designed to yield such that the dynamic overstrength actions it resisted were less than both the shear and flexural capacity of the existing walls above. This was achieved by using an innovative steel/concrete composite arrangement as shown in the diagram below.



To allow for the additional displacements imposed by this flexible and ductile system, all longitudinal beams that framed into rotating elements were “pinned” as below.

Late in the project a rearrangement of the social ambitions by the client required that both blocks also incorporate inter-tenancy doorways to allow a broad range of unit mixes. The existing transverse walls were meshed reinforced, and so had previously only been relied on to resist elastic loads. Creating doorways in these was a concern for brittle failure of the structure.

Instead the opportunity was taken to turn the existing full-length walls into a pair of ductile coupled walls joined by a steel beam in single curvature. This flexural element could therefore act as a fuse to ensure that dynamic overstrength shears would not exceed the strength of the mesh-reinforced walls. This work included using cored holes to sever the existing foundation beam to ensure deformation compatibility.



7 PEER REVIEW

After discussions with Wellington City Council, it was agreed that the buildings where de-stiffening/energy dissipation elements were added should be subject to peer review. Holmes Consulting Group in Wellington provided this review. The deformation compatibility mechanism of the existing foundation beam was questioned. Dunning Thornton made a physical card-and-string model of the mechanism to better understand/explain it. Illustrating the rotational deformations at this extreme scale helped both parties understand the curvature ductilities required, though these were shown to be sufficient.

An independent displacement-based analysis was carried out of the Stadium Tower and the curvature ductility demands on the bow ties was agreed. However, the loads on the fixings at each end of the bow ties were queried, and given their criticality, these were increased to add robustness to the system.

8 IMPORTANCE OF SOIL/STRUCTURE INTERACTION

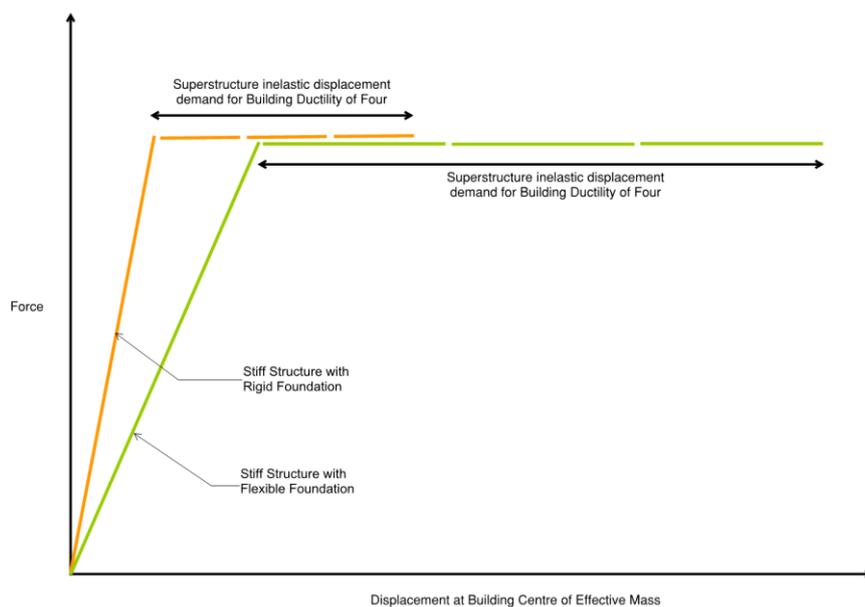
Both the blocks and the Stadium Tower involve adding ductility to the structure using steel energy-dissipating devices. The capacity of these devices is limited by their dependable plastic curvature capacities. It is essential that proper soil/structure interaction is accounted for in assessing these curvatures.

As the superstructures have many walls, there is little/no period shift benefit in de-stiffening the buildings unless significant structural demolitions were to occur. Keeping the layouts as was required to minimise cost, gains could only be made from the added damping and robustness provided by the yielding elements.

If analysis is carried out using a traditional rigid-based model, yield would occur under very small displacements and the required ductility would be achieved with very little curvature of the dissipation elements.

HOWEVER

Considering the initial elastic deformation of the sub-soils, the yield point of the structure is pushed to the right to a significantly greater displacement (see graph below). This requires a potentially very large increase in the curvature ductility demand on these elements to achieve the same overall building ductility. This essential soil/structure interaction principle is illustrated in the graph below. Under-estimation of the foundation flexibility can result in significant over-estimation of the seismic capacity of a stiff, ductile building.



9 CONCLUSION

1970's Multi-unit residential buildings exist throughout New Zealand and most have similar issues to the buildings described here. Typically they are very robust in their transverse direction, but their longitudinal performance is complex, relying on the coupling of any longitudinal elements around doorways and/or services risers.

For the Newtown Park flats, the existing buildings were reused in what was seen to be the most economic manner: retaining as much structure as possible and seismically strengthening either by geometrically removing CSWs or by adding energy dissipation to existing poorly-performing elements. A significant regime of insitu pile testing was carried out, as it was deemed to be a critical performance element. This pile testing more than paid for itself, in the understanding it provided to the structural solution.

Energy-dissipating elements were added to existing structural systems to improve their performance. It is emphasised that proper understanding of soil/structure interaction is essential for a proper appraisal of the ductility demands on rigid ductile systems.

The systems, peer reviewed to give additional surety as to their performance, provided extremely economic solutions to the seismic retrofit. It is suggested that options such as these may be considered for the large number of multi-unit residential buildings in New Zealand that pre-date modern codes that may require retrofit.

For solutions to be economic, it is important that the existing robust elements (insitu floor diaphragms, long inter-tenancy walls) are reused by clever de-stiffening and discrete placement of new structural details which can add energy absorption to the building's seismic resistance.

In considering retrofit solutions to stiff structures, soil-structure interaction **must** be considered as a significant proportion of the building's movement can result from foundation flexibility. Where buildings have ductility, not properly assessing the proportion of elastic foundation rotation can significantly underestimate the local ductility demands on these elements.

10 ACKNOWLEDGEMENTS:

Our client WCC and Studio Pacific Architecture (Marc Woodbury and Andrew Banks), Hawkins Construction and DTC Project Engineers Nick Carman and Chris Speed.