Seismic rehabilitation of a concrete encased steel riveted frame building - Adelphi House, Wellington

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ABSTRACT: This paper describes the complex seismic rehabilitation design of Adelphi House, a 7 storey 1920's heritage building in Courtney Place, Wellington. The existing building structure comprises concrete encased steel sections with riveted connections. The brief from the client was to prepare an economical strengthening scheme that minimised interference with the building internal layout and maintained the heritage aspects of the building.

The design process included intrusive investigations of the structure and laboratory testing of the materials used for the original construction. The behaviour of riveted structural steel connections under seismic load conditions was also researched from international papers to provide data for the structural analysis

Rehabilitation techniques were developed to strengthen and preserve the heritage fabric of the building to meet 100% of the current earthquake code provisions. A unique structural engineering solution was developed including the application of 'steel jackets' to the existing columns with a bespoke solution for the stiffening of the beam-column joints, two new internal steel composite columns, the introduction of cross bracing in the longitudinal direction, and the strengthening of the foundations. The design also addressed the existing 'weak-column strong-beam' configuration and a 'soft storey' failure mechanism found during the analysis of the existing structure.

The analysis of the existing building and the subsequent strengthening design has been performed using the non-linear static procedure, known as 'push over analysis'. This analysis has been undertaken in line with the FEMA 356, FEMA 440 and ASCE41-06 documents and a number of published papers.

The design is complete with construction presently underway with completion expected in early 2014.

1 INTRODUCTION

Adelphi House is a 7 storey building originally constructed in 1928 and is located in Wellington's Courtenay Place Heritage Area. Harrison Grierson Consultants Limited were commissioned by the owners to develop an economical strengthening scheme to satisfy 100% of New Building Standard (100% NBS). A previous scheme prior to our commission was deemed too expensive and intrusive on the floor space and heritage aspects.

As we see many existing buildings throughout the country undergoing structural strengthening works it is becoming evident that there are two categories in to which these strengthening designs can be grouped. The first would be the addition of new primary framing systems, potentially ignoring the strength of the existing structure. The second would be the method of adding strength to the existing structural elements, seen as a more discrete or harmonious solution. The aim for Adelphi House was to provide a solution in line with the latter option.

Our approach for the project was to perform a non-linear static analysis ("Push Over") of the existing structure to model the likely behaviour in a seismic event. This method enabled us to focus on the critical weaknesses of the existing structure and develop a strengthening design that added strength and resilience whilst utilising the existing inherent capacity of the existing building.

2 BUILDING DESCRIPTION

2.1 General

The building is known as 'Adelphi Finance House' and is located at 15 Courtenay Place, Te Aro, Wellington.

The building was erected in 1928 and was formally the Courtenay Chambers. The building has seven levels comprised of commercial space, with retail tenancy at its ground level frontage with Courtenay Place.

An extract from the Old Shoreline Heritage Trail Guide describes the history of the Courtenay Chambers building:

".... a handsome neo-Classical building, erected in 1928 and another Llewellyn E. Williams design. Although not excessively tall by Wellington standards, it is enormous in the context of the low-rise buildings on Courtenay Place. The retention of the street's modest scale can be partly attributed to a public seminar on the future of Courtenay Place, held in 1986 and sponsored by Wellington's Civic Trust, which strongly supported limiting the street's buildings to two storeys"

The building is rectangular on plan with approximate plan dimensions of 10m width and 20m length. The structural layout is relatively regular in both plan and elevation (See Figure 1).

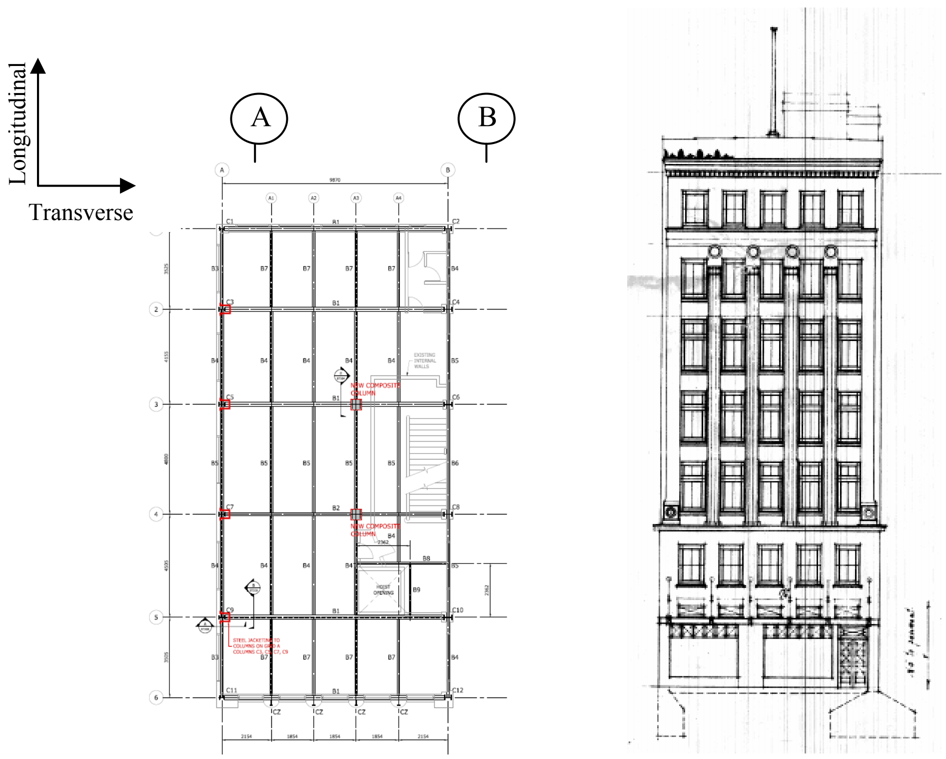


Figure 1. Typical Floor Plan and Original Facade Elevation

The existing gravity resisting structure comprises the following:

- In-situ reinforced concrete suspended floors and roof.
- Concrete encased steel beams and columns.
- Lightly reinforced concrete side walls (containing window openings above Level 3)
- Concrete foundations comprising a narrow ground beam under the columns on GL A and pad footings under the columns on GL B.
- In-situ reinforced concrete stairs.

The existing lateral load resisting structure comprises the following:

- In-situ RC suspended floors act as floor diaphragms.
- In the transverse direction concrete encased steel frames act as moment resisting frames (the riveted beam-column connections are the weak point).
- In the longitudinal direction the RC side walls act as compression struts between the columns where no openings are present in the bay (from level 3 down to ground only). For the levels above level 3 the spandrels and piers of the side walls will provide some lateral resistance, however due to the low level of reinforcing reported from intrusive investigation these are not a reliable lateral resisting system and have therefore been ignored.

2.2 **Material Properties**

The existing drawings for the building were obtained from the council archives. Given the age of the building these provided reasonable detail. However, there were areas of the drawings which were unclear (poor copies, etc) and some assumptions had to be made during the initial stages. There was also as-built information missing which was required for the detailed analysis we were to undertake that was probably not included in the original design documents of the time. This information was therefore obtained by laboratory testing of extracted samples of the building materials.

The testing schedule included compressive tests of the concrete cores using both crushing and Schmidt hammer tests, reinforcing sonar scanning, steel tensile, chemical and hardness tests, and concrete carbonation testing.

2.3 **Geotechnical Conditions**

From our initial investigations using GNS maps it appeared that the site was on the boundary between class C and D subsoil designations. This is likely due to the fact that the area was historically a swamp prior to the 1855 earthquake which raised and drained the area. We therefore recommended that a geotechnical engineer perform a site specific study to determine both the site sub soil class and the bearing capacity of the soil for use to check the existing foundations and design the any new foundations.

RDCL geotechnical engineers performed the site investigations to provide the required information.

The site subsoil class was determined as Type C (shallow soil) based on shear wave test results, the preferred method for classifying sub soil class according to NZS1170. The bearing capacity was determined as 435kPa based on SPT results from existing bore logs. Further STGT testing has been carried out in the shallow zone to confirm this value during the site works.

3 INITIAL ASSESSMENT

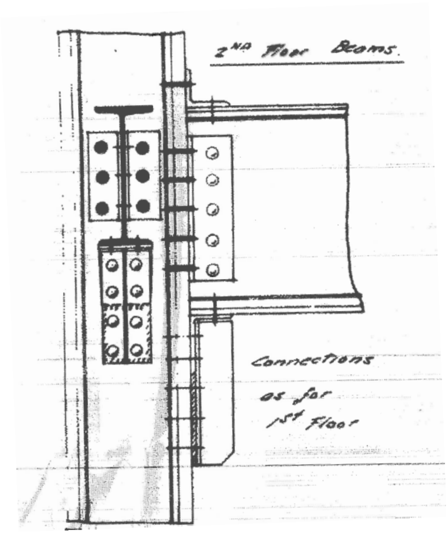
3.1 Modelling the Existing Steel Riveted Frame

3.1.1 *Steel Riveted Clip Angle Connections*

The existing steel frame is connected together using steel riveted connections with clip angle brackets (Refer Figure 2). The entire frame was also encased in concrete, most likely intended for fire protection rather than to add stiffness to the frame. This concrete is however lightly reinforced.

Prior to performing the analysis we studied many international papers relating to non-linear modelling of existing steel framed building, particular those constructed using riveted connections.

This led us to a paper titled Seismic Behaviour of Older Steel Structures' (American Journal of Structural Engineering - Roeder et al - April 1996). This paper covers the research and testing of the strength, stiffness, hysteretic behaviour and ductility of steel riveted sub assemblages and provided detailed input in to how we modelled the clip angle connections as non-linear hinges. The clip angle connections exhibited behaviour similar to partially restrained (PR) connections, meaning that the majority of the frame displacement would be due to local deformation of the connection.



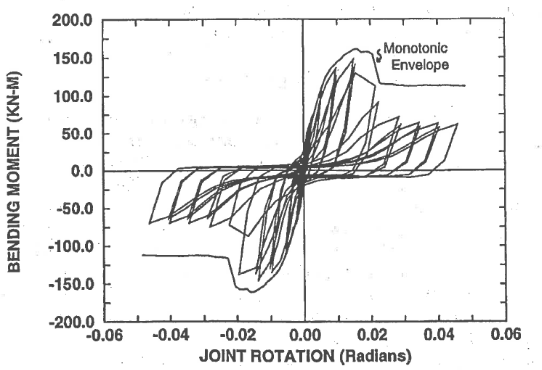


FIG. 4. Comparison of Monotonic and Cyclic Moment—Rotation Behavior

Figure 2. Typical Rivet Connection (Excerpt from Archive Drawings)

Figure 3. Monotonic and Cyclic Moment-Rotation Behaviour

Various yield mechanisms are possible in the connections, and good rotational capacity can be displayed depending on which yield mechanism governs. Tensile yielding of the rivets gives the least ductility (unfortunately the case for the connections in Adelphi House), shear yielding of the rivets gives intermediate ductility levels, and flexural yielding of the flange clip angles provided the highest level of ductility. Additionally, the concrete encasement provided relatively increased strength, stiffness, and joint rotation of the connections with the weaker more flexible connections (clip angle assemblies such as those in Adelphi House).

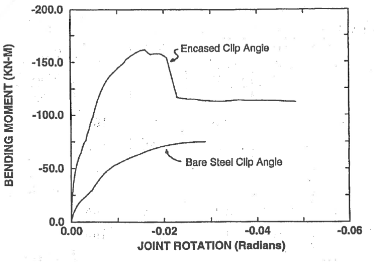


Figure 4. Bare steel vs encased joint moment rotation behaviour

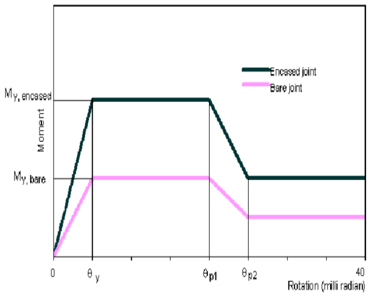


Figure 5. Idealised Moment-Rotation Behaviour (NZSEE Excerpt)

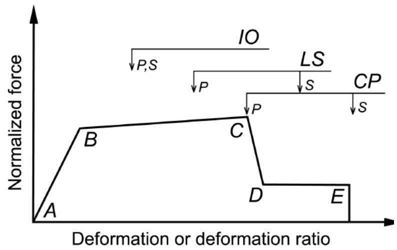


Figure 6. Basic Non-linear Force-Deformation Relationship

3.1.2 *Column sizes variance*

Despite a relatively regular structural layout there is a difference in the strength and stiffness between the columns on opposite sides of the building. The columns on GL B are larger to allow for an extension that never occurred. Although not obviously much larger than the columns on GL A the larger plates used result in the columns on GL B being approximately 3 times stiffer and twice as strong.

An additional irregularity in the columns was the reduction in size and stiffness from Level 1 to Level 2, and Level 2 to Level 3, and remaining the same size from Level 3 and above. The stiffness and strength reduction was in the order of 30-50%, the effects of which are discussed in the results section.

3.2 **Push over analysis**

The push over method was used in order to capture the inelastic behaviour of the structure and model the post yielding behaviour of the individual elements and their effect on the overall response of the structure.

Our assessment of Adelphi House followed the FEMA 356 ("Prestandard and Commentary for the Seismic Rehabilitation of Buildings") procedure for the push over analysis. This document outlines the various parameters and objectives of the procedure. In particular the following parameters and acceptance criteria were assessed:

- Target displacement for the analysis – The target displacement is factored by η to allow for any torsional effects of the structure (e.g. - max displacement/average displacement).
  - $\delta_t = C_0\, C_1\, C_2\, C_3\, S_a\, (T_e^2/4\pi^2)\, g \times \eta$
- Yield rotation value – the rotation, in radians, that the individual hinges yield at.
- Two load patterns checked; inverted triangle and vertical distribution relative to floor mass, using the elastic site spectra for soil type C
- Non-linear load cases and patterns are defined. Concurrent load directions are still applicable (100% + 30%).
- Backbone curves based on the strain hardening slopes in accordance with the material tensile test results.
- Levels of acceptance criteria for different performance levels of the hinges (Immediate Occupancy, Life Safety, Collapse Prevention).

For the non-linear hinges we only specified bi-linear back bone curves. For the rivet connections we used the data from the test conducted by Roeder et al (Refer section3.1.1). This was because we didn't want to push the hinges to a point where they underwent excessive yielding or rotation given the age and potential lack of resilience in the structure.

For the steel encased elements the stiffness of the members was calculated using only the concrete enclosed by 3 sides of the steel (i.e. between the flanges and web) as per ASCE 41-06 clause 5.4.2.2

We used ETABS Non-Linear software to analyse the structure.

4 ANALYSIS RESULTS OF EXISTING STRUCTURE

Upon our assessment of the riveted connections, we decided that the very basic strengthening would involve remediating these weak zones. However, when using the full moment capacity of the beams for the connections the existing structure exhibits a soft storey mechanism between the Level 2 and Level 3 in the transverse direction (Refer Figure7). This is a potential collapse mechanism and is due to a 'weak column strong beam' arrangement. The location of the soft storey is due to the combination of the storey shears increasing towards the bottom of the building and the stiffness increase of the columns below Level 3.

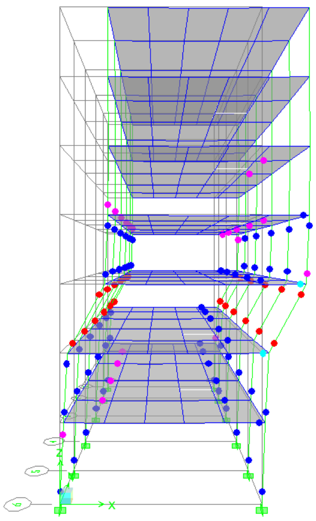


Figure 7. Existing Soft Storey Failure Mechanism of the Un-strengthened Structure

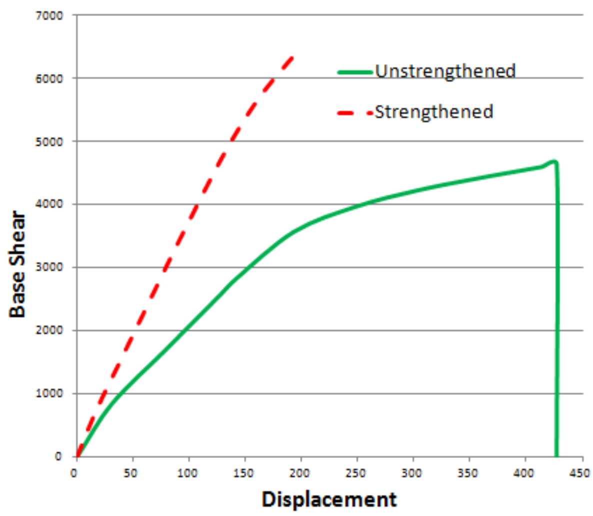


Figure 8. Push Over Curves of the Un-strengthened and Strengthened Structure

Another observation of the existing structure was the excessive drift prior to the soft storey failure mechanism.

In the longitudinal direction the solid infill walls of the first 3 floors provide an adequate lateral resisting load path. However, above this the building has only marginal frame action comprising relatively small steel beams and weak riveted connections. This also resulted in a soft storey mechanism forming between Level 2 and Level 3 in the longitudinal direction.

5 STRENGTHENING SCHEME

Following the analysis of the existing structure it was crucial to develop a strengthening scheme that would address the soft storeys in both directions.

To do this would require us to consider the clients requirements (maximise leasable area whilst creating a strengthened structure) and the wider communities and local authorities interests in the heritage aspects of the area (minimise any changes to the exterior of the building fabric).

5.1 Transverse Direction

In the transverse direction we concentrated on solving the "weak column-strong beam" problem. We did this by providing steel plate jackets bonded to the columns, increasing their strength above the strength of the existing beams. The bond strength was verified by pull off testing prior to construction. The connections were strengthened further by providing a haunch and welding the beams to the new steel jackets (Refer Figure 7)

A new composite steel column was also added to two of the internal grids to add additional stiffness and strength in the transverse direction. These new columns were to be fully welded to the existing transverse beams to create an additional moment frame, with added strength provided by detailing the reinforcing bars to be continuous through the existing floors.

In the strengthened state the hinges formed only at the end of the beams and at the base of the columns, thus solving the soft storey system. We ensured that the residual beam connections had adequate gravity load carrying capacity in the post yield condition.

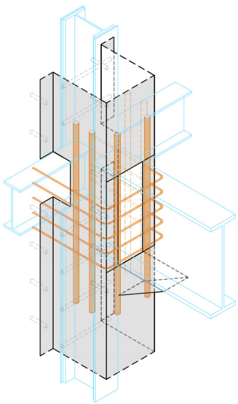


Figure 9. Isometric view of the steel jacket detail at the beam junction

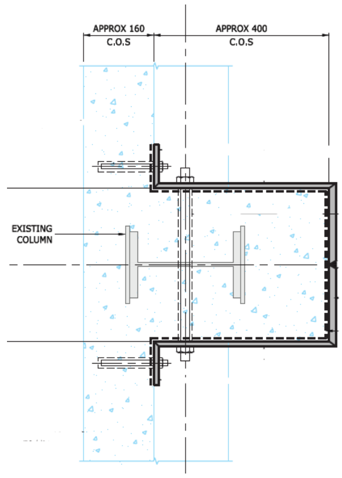


Figure 10. Plan view of the steel jacket showing the shear dowels to enable composite action

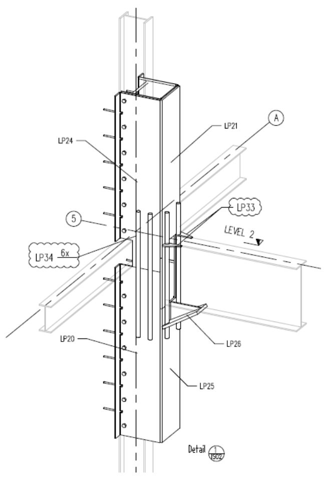


Figure 11. Isometric shop drawing of the steel jacket

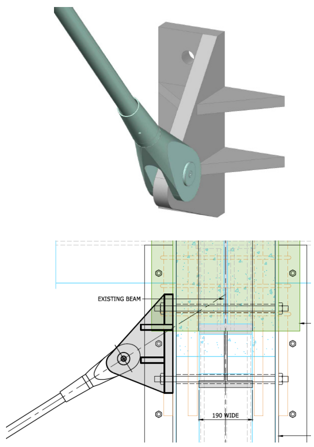


Figure 12. 3D render and Cross section of the bespoke Macalloy connection

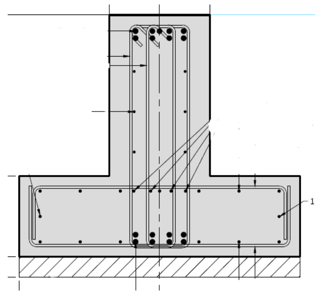


Figure 13. Cross section of the new foundation beams

5.2 Longitudinal Direction

We decided that providing vertical bracing would be the most efficient solution in the longitudinal direction. Initially we opted to locate this new bracing between he new internal composite columns, away from crossing the windows and altering the external aesthetics of the building. However, this option would also require strengthening of the Level 3 diaphragm which would then act as a transfer diaphragm when the stiffer infill walls came in to affect at this level. It was therefore decided to move the vertical bracing out to the side walls and spread it over several bays if required to reduce the section size to reduce overall costs.

The final design involved the installation of Macalloy tension bracing using bespoke connections to the existing columns on one side and the steel jackets on the strengthened side of the building.

5.3 Foundations

To reduce the foundation pressures to within allowable limits we designed new stiff deep ground beams. These would help to spread the overturning seismic actions in the longitudinal direction, especially given the original narrow ground beam under GL A.

The foundations were modelled using FEM software Cedrus. This enabled us to accurately check the local ground pressures and to check if any uplift occurred and if so what magnitude.

5.4 Residual capacity of the strengthened structure

To verify that the strengthened structure can withstand a potentially larger than code level earthquake the push over analysis was also taken to 150% of the target displacement. This method checks for both the case of a larger earthquake occurring than expected or required (a pseudo MCE case) but also assesses for any redundancy and residual strength.

In the case of the strengthened Adelphi House it is likely that the new tension braces may experience tension yielding. This would be easy to remove and replace the Macalloy braces, even with temporary Reid braces given the lead in time of Macalloy elements. Additionally the frame beams may experience yielding which would also require some local repairs following a large earthquake.

5.5 Potential for Pounding

In accordance with NZSEE guidelines ''Assessment and Improvement of the structural Performance of Buildings in Earthquakes'' (Appendix 4D: Potential of Pounding), the building was checked for two situations. The first checks for when adjacent floor levels align and applies increased design actions (125% and 175%) to the columns on the potential collision side of the building. The second checks for when floor levels do not align (greater than 20% of the storey height) meaning a potential for mid-storey pounding of the columns. This check requires a pseudo displacement to be applied to the columns at the potential height of impact.

Despite the fact that the above procedures are approximate and quite conservative, the strengthened columns of Adelphi House were capable and robust enough to resist all the above extreme forces and with significant residual strength remaining.

6 COST OF THE STRENGTHENING WORKS

The construction work is now in to its final stages, awaiting the delivery of the Macalloy Braces from the UK.

The reported construction budget for the works was approximately $800,000. This is substantially less than the $2million estimated cost of the previous scheme by the original consultant engaged by the client.

This saving can be attributed to both performing a non-linear analysis and by utilising the inherent strength of the existing structure. Our strengthening scheme addressed the weak points of the existing structure without ignoring its positive attributes; strong beams and a regular layout which lent itself well to distributed lateral resisting frame systems.

The constructed strengthening scheme also provided an almost imperceptible reduction in leasable space, especially in comparison to the original scheme which comprised new EBF frames, shear walls and ground anchored piles.

7 CONCLUSION

There was a potential that this building could have been left unstrengthened and under utilised due to the substantial cost of the original strengthening scheme. Based on our assessment this building could have been at substantial risk of collapse if left un-strengthened.

Extensive intrusive investigations led to a good understanding of the existing structure which enabled more advanced analysis to be carried out, methods of which usually lead to economo-technical optimum solutions.

The design addressed the positive impact of the steel jacketing as a cost effective solution for this type of structure, where the aim is to increase both the strength and stiffness via the transformation of the original columns to steel and concrete composite sections.

The final strengthening scheme to Adelphi House has resulted in a cost effective, bespoke and successful solution for the owner.

Given the prime location in Courtenay Place the client should be able to look forward to successfully marketing a building that meets 100% of New Building Standard in a desirable city centre and heritage area. The money saved by the strengthening scheme has enabled us to provide a design for the owner to build new balcony overlooking Courtenay Place, well suited for the frequent red carpet events.

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