Low Damage Design – A Case Study

D.R. Gardiner[1], B.D Galloway[1] & D. K Bull[1]

[1]*Holmes Consulting Group, New Zealand.*

2013 NZSEE Conference

ABSTRACT: There is a demand from some clients for low damage design as a result of the recent Canterbury earthquakes. Lengthy downtime and significant insurance costs that resulted from the sequence of earthquakes are contributors to this demand.

A case study is presented in this paper on a low damage design that was developed for an educational facility in Christchurch. The facility comprised of three interconnected buildings that were seismically separated with different lateral force resisting systems.

A variety of approaches to low damage design were adopted for the project. Consideration was given to both primary (e.g. floors and walls) and secondary structural elements (e.g. precast cladding panels) to ensure that a consistent level of performance was achieved for the system as a whole.

The structural systems adopted for the project included; post-tension concrete rocking walls, reinforced concrete slotted beams with removable dampers and conventional concrete systems designed for low levels of ductility demand. The use of non-standard systems required verification to be carried out to ensure the elements behaved as envisaged. Examples of items that were explicitly considered included; torsion demand on the slotted beam systems and low cycle fatigue of the removable dampers.

1 INTRODUCTION

This case study considers three separate buildings that are arranged in an L-shape and are interconnected via the floor levels. The brief for these buildings was based around creating low damage Importance Level 3 structures that accounted for the functional requirements for each of the buildings (NZS1170.5:2004). A plan of the geometry and relationship between each of the buildings is shown in Figure 1.

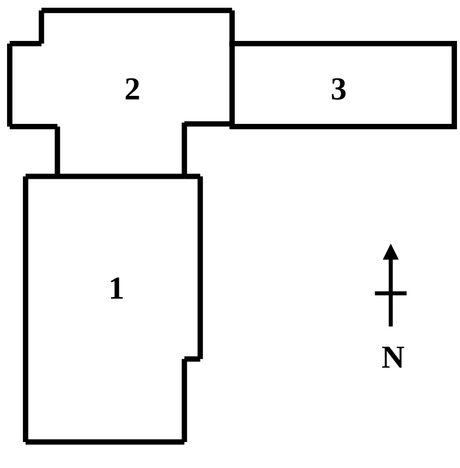

Figure 1 Plan View of Building Complex

Building 1 is a rectangular three storey building with a footprint of approximately 580m^2. This building is intended to be used as a large open plan function space at the ground floor level and as classrooms in the upper levels. The open-plan space at the ground floor required the lateral force resisting systems to be concentrated around the peripheral of the building. To account for this requirement, the structural system comprised of post-tensioned reinforced concrete rocking walls and frames that were located around the perimeter of the building.

Building 2 is located between buildings 1 and 3 with a footprint of around 350m^2. This building is connected to both Buildings 1 and 3 by link bridges. This building is typically three stories above ground; it has a below ground basement and a plant deck at the top level. This building has a number of different intended uses: with the basement for storage, level 1 as an open-plan staff facility, the plant deck space for the services for the new buildings and the remainder of the building classroom space. The open-plan staff facility at level 1 restricted the types of structural systems that could be employed for this building. The structural system that was chosen included reinforced concrete frames with slotted beams and post-tensioned columns in the north-south direction and conventionally reinforced concrete walls designed for low levels of ductility in the east-west direction. The structural system for the basement was insitu reinforced concrete walls designed for limited ductility. The floor diaphragms comprised of metal decking with an insitu concrete topping.

Building 3 is a long narrow three storey building with a footprint of approximately 200m^2. This building is primarily intended to be used for classrooms. Due to the geometry and functional requirements of this building, well distributed structural systems were chosen lending to a more conventional design to be employed. Reinforced concrete transverse walls were used for the short, north-south direction, and reinforced concrete frames that comprised of precast concrete columns and insitu spandrel beams were employed for the longitudinal, east-west direction of the building. Both the walls and the frames of this building were designed for low levels of ductility. The floor diaphragms for this building were precast rib and timber infill with insitu concrete topping.

Detailed description of a selection of aspects that should be considered when designing both rocking walls and slotted beam structural systems are discussed in the following paragraphs. Included, in these paragraphs, are comments relating to the benefits of these structural systems.

A key concept to consider when trying to carryout a low damage design is to ensure that both the primary and secondary structural elements perform in a low damage manner to ensure that a similar level of performance is achieved for the building as a whole. Comment on this is given in Section 4.

2 ROCKING WALLS

Rocking walls typically comprise a conventional wall with un-bonded post-tensioned bars that are anchored at both the top and bottom of the wall. Figure 2 provides a simplified image of a rocking wall system. The rocking wall configuration typically results in non-linear elastic force-deformation behaviour characteristics. A force-displacement plot that represents this behaviour is shown in Figure 3.

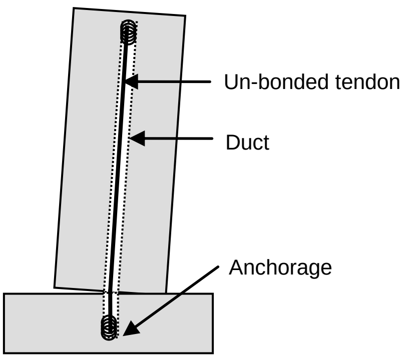

Figure 2 General components of a rocking wall

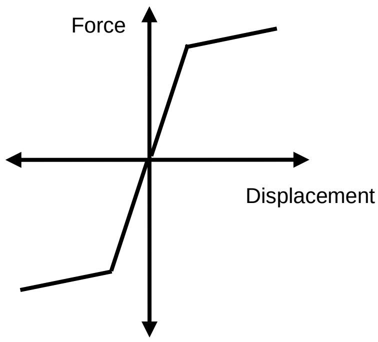

Figure 3 Non-linear elastic behaviour

There are a number of items that need to be considered in the design of rocking wall structures.

Selections of some of these items are discussed in the following sub-sections.

2.1 *Damping Mechanisms for Rocking Walls*

External or internal energy dissipaters may be utilised to provide damping for rocking wall systems. One problem related to internal dissipation is that if an earthquake occurs and causes damage to the dissipaters, the dissipaters will need to be removed and replaced. This therefore would not constitute a low-damage system. A possible, but more costly, solution to this could be to use internal lead dampers.

External dissipaters provide an alternative solution where the damper may be replaced after an earthquake, allowing the damper element of the structure to be returned to its pre-earthquake condition after replacement. Durability can be an issue for this type of solution, especially if the rocking walls are to be located around the perimeter of the building exposing the dissipaters to an exterior environment.

Additional dissipation was not included in the rocking walls for Building 1. The primary reason for this was related to architectural requirements. From the experience of this project, it is recommended that some form of damping is employed for rocking wall systems. One of the key reasons for making this recommendation is related to the behaviour of a non-damped rocking wall system after initiation of rocking; after the wall begins to rock, the displacements rapidly increase due to the reduction of stiffness. The presence of damping helps to mitigate this by reducing the forces due to absorbing some of the energy. If a system with no additional damping mechanism can not be avoided, the initial post-tensioning applied to the tendons should be reduced to such a level that avoids initiation of rocking at Ultimate Limit State (ULS) design loads. This would prevent large displacements from developing by maintaining large stiffness of the wall.

2.2 *Durability of Rocking Interface*

Extra care needs to be given to durability issues associated with rocking walls. This is due to the vulnerability of the exposed rocking interface that the wall rocks on. The low damage philosophy of the rocking wall system needs to be incorporated in the considerations of the durability for the system to ensure that similar performance levels of both the seismic system and durability of the structure are met. The rocking surface is generally not accessible for easy replacement; this should be taken into account during the design.

The approach that was used for the rocking walls in Building 1 was to employ a stainless steel shoe for the wall and a stainless steel set down pocket into the foundation for the shoe to sit within. The wall was set down into a pocket to provide a secondary mechanism to resist the shear actions on the wall. The shoe was used to prevent spalling of the edges of the wall as it rocked onto its edges. The walls for Building 1 were located around the perimeter of the building and therefore the rocking surfaces were located in exterior environments. For this reason, stainless steel was chosen as the metal for the shoes and pockets.

2.3 *Foundations: Rocking Walls vs. Conventional Walls*

One of the benefits of a rocking wall system over a conventional wall system relates to the size of the foundation that is required. Rocking wall structures require smaller (typically shallow) foundations as there is no need to resist the generally large overturning loads of the building as resistance to the overturning is created by the post-tensioning. It should be noted though, the footings for rocking wall buildings needs to be large enough to ensure that the foundation does not rock before the wall rocks.

3 **SLOTTED BEAMS**

A primary driver for the development of the slotted beam system relates to how slotted beams minimises the effects of beam elongation. A description regarding what beam elongation is and some of the consequences related to it, is provided in Section 3.1. The general configuration of the slotted beam, along with how slotted beams minimise beam elongation is provided in Section 3.2. Comparisons are also made in this Section between the performance of conventional beams and slotted beams.

3.1 *Beam Elongation*

The mechanics and consequences of beam elongation have been studied by a number of researchers over the past few decades (Fenwick and Fong (1979), Fenwick and Megget (1993), Fenwick and Davidson (1995), Matthews (2004), Lindsay (2004) and MacPherson (2005)). Beam elongation is initiated due to inelastic strains which developed in the reinforcement during the cyclic behaviour of an earthquake; typically at the ends of beams. These regions are called "hinge zones" or hinges". As the beam column joint is rotated, cracks develop in the hinge location causing yielding of the reinforcement which leads to irrecoverable inelastic strain in the reinforcement (see Figure 4). On the reverse cycle, the cracks can not be fully closed by the compression forces due to the internal compression force being resisted by both steel reinforcement and concrete. The bars therefore do not recover. The previous tension strains and the particles of dislodged concrete within the crack, contribute to the cracks not closing under compression. This behaviour results in the beam growing (elongating) in length. Consequences of this behaviour can include the corner columns of the structure being pushed outwards, resulting in significant cracking and possible tearing of the connection between the column and the floor plate. The pushing out of the column also leads to the beam being subjected to significant torsions which in turn cause the beam to rotate out of plane and subsequently lead to the possible loss of seating for the floor diaphragm units, causing these to collapse on the floor below.

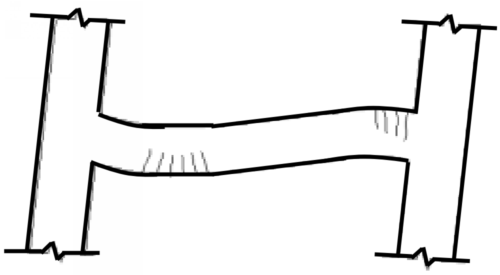

Figure 4 Cracking pattern in beam due to cyclic action from an earthquake (at "hinge" locations)

3.2 *Geometry and Mechanics of Slotted Beams*

The performance of reinforced concrete slotted beams has been considered by a number of researchers (Au 2010, Leslie 2010, Bryne et. al. 2012, Muir et. al. 2012) at the University of Canterbury over the past few years. The original concept for the slotted beam came from Japan around 15 years ago. The slotted beam is a modified version of a conventional reinforced concrete beam. A typical layout of the reinforcement of a slotted beam is shown in Figure 5. The reinforcement of a slotted beam is arranged so that all the yielding of the reinforcement is concentrated in the bottom bars and not the top bars, resulting in a hinge forming at the top of the beam during cyclic rotations from an earthquake. A wedge of concrete is removed from the bottom of the beam, at the beam-column joint interface, to allow the beam to rotate without contacting the face of the column and causing damage to the beam; this also ensure the hinge mechanism can form at the top of the beam. Diagonal hanger bars are additional bars which provide resistance to both shear and torsion forces of the beam. Further information regarding the mechanics of slotted beams can be found from the research that has been carried out at the University of Canterbury.

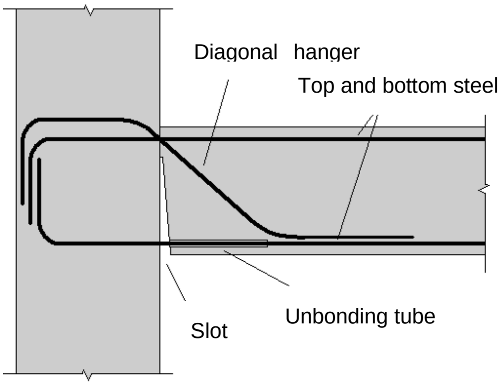

Figure 5 General arrangement of a slotted beam

The slotted beam system has been developed to minimise beam elongation and consequently improve the performance of the structural system. Beam elongation is minimised by focusing the yielding to occur in one location, the top hinge. This minimises elongation in the beams.

Laboratory research that has been carried out at the University of Canterbury has included testing of a full super-assembly of a conventional reinforced concrete frame (Mathews 2004, Lindsay 2004 and MacPherson 2005) and a slotted beam frame (Muir et. al, 2012) over the past decade. Comparisons between the cracking patterns at the beam column joint at similar levels of lateral displacement are shown for both a conventional system (MacPherson, 2005) and a slotted beam system (Muir et. al, 2012) in Figure 6 and Figure 7 respectively for similar maximum displacements.

Figure 6 Monolithic beam column joint (MacPherson, 2005)

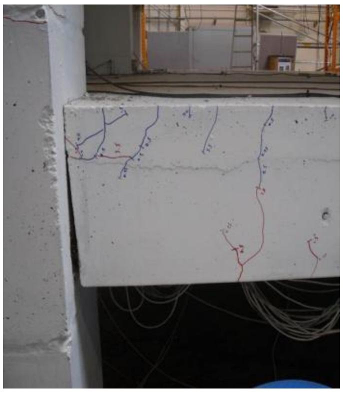

Figure 7 Slotted beam column joint (Muir et.al, 2012)

Comparisons between the cracking shown in Figure 6 and Figure 7, indicate that there is significantly less damage to the slotted beam system. The crack pattern shown for the slotted beam system also indicates that cracks (minor and typically closing) have predominantly formed in the top of the beam rather than the bottom. This is correlates to the location of the hinge for the slotted beam at the top of the beam.

3.3 *Low Cycle Fatigue*

Replaceable hysterectic dampers were employed in place of bottom steel for the reinforced concrete slotted beams of Building 2 (see Figure 8 and Figure 9). Replaceable dampers were chosen as these can be replaced after a certain number and sizes of earthquakes. One of the key issues identified during the process of designing the external dampers is related to low cycle fatigue.

Low cycle fatigue corresponds to the relationship between the number of cycles that a member experiences and the force exerted to the member at each cycle. Low cycle fatigue generally becomes more problematic when fewer cycles are exerted, but the force is large – this is characteristic of the actions that are typically felt in beams during earthquakes. This is primarily a problem for unbonded steel as the actions can not be distributed throughout the member like in a conventional beam.

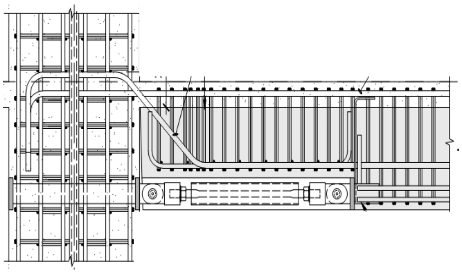

Figure 8 Elevation of replaceable damper

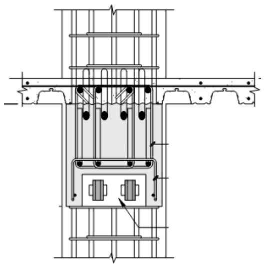

Figure 9 Section through slotted beam showing replaceable dampers

An approach to dealing with low cycle fatigue is to consider the cyclic fatigue life of the unbonded steel. A study was carried out by Mander et al. (1994) on the fatigue behaviour of reinforcing steel. The correlation obtained from the results of this study indicated that the plastic strain amplitude could be related to fatigue life of reinforcing steel. This relationship is show in Equation 1. Where ε_p is the plastic strain amplitude and N_f correlates to the number of cycles to failure.

$$\varepsilon_p = 0.08\left(2N_f\right)^{-0.5}$$ **Equation 1**

This equation may be used to determine the cumulative fatigue of the member by considering the plastic strains, which the member has been subjected to, at different imposed drifts through the earthquake. The impose drifts may be obtained by considering the magnitude and number of earthquakes that the designer believes is acceptable for the member to sustain. An accepted loading protocol (ACI440) should be employed and scaled to represent the expected drifts for the structure. This will provide a history of drifts and also a history of the plastic strains that the member should be able to sustain. For each of the plastic strains, the effective number of cycles that could occur at this strain rate can be determined by the use of Equation 1. From this, a Miners summation method could then be used to determine the cumulative fatigue for the member. This should indicate whether low cycle fatigue will be an issue for the imposed drift level. If low cycle fatigue is found to be an issue, a longer unbonded length may be required.

The steel that was used in the study by Mander et. al. (1994) was American reinforcing steel from 1994. It is believed that the steel, which is typically used in New Zealand today, would perform better than the steel that was tested and reported in this study. It would be beneficial for similar tests to be carried out on readily available New Zealand steel to determine how New Zealand steel compares and further to provide information for future designs where low cycle fatigue needs to be considered.

3.4 *Torsion: Slotted Beam vs. Conventional Beam*

Differences exist between how a conventional beam compared to a slotted beam behaves when subjected to torsion actions. The difference in behaviour is primarily related to the difference in the configuration of the end connections for the two types of beams.

Torsion actions, due to orthogonal drifts, which develop in beams of framed structures, are generally referred to as “secondary torsion” or “compatibility torsion”. The reason for this is due to the continuity of the beam to the floor slab, where the floor slab could provide a secondary mechanism to resist the beam torsion actions through redistribution.

The torsion demands in a conventional beam are resisted by the torque couple that develops between the longitudinal and shear reinforcement within the beam. The torque couple generally initiates spiral cracking along the length of the beam, causing a reduction in the torsional stiffness of the beam. The reduction in torsion stiffness could subsequently lead to the torsion actions being redistributed to the adjacent floor slab (secondary mechanism). As a result of this behaviour, generally torsion demands in conventional beams are typically not critical in the design of the beam.

The mechanism to resist torsion differs for a slotted beam, compared to a conventional beam, due to differences in the end connection. Slotted beams are designed to resist both shear and torsion by the diagonal hanger bars. The high position of the diagonal hanger bars, in the beam, at the end connection is the primary cause of the difference between the torsion resistance mechanisms for a slotted compared to conventional beam. This arrangement affects both: the development of a torque couple and consequently the development of spiral cracking along the entire length of the beam. A consequence of this is that the redistribution to the floor slab is not initiated and therefore the beam needs to resist all of the torsion. Due to this, care needs to be taken in the design to ensure that imposed torsion actions onto slotted beams are minimised, and are accounted for in the design of the hangers.

Another form of torsion which also needs consideration is stability torsion. This occurs under gravity loads and can be an issue when floors are seated on only one side of the beam.

3.5 *Diagonal Hanger Bars*

Care needs to be taken when considering the detailing of slotted beams to ensure the beams behave as envisaged by design. Most of the detailing issues that differ from a conventional beam are associated with the detailing corresponding to the diagonal hanger bars.

The "elbow" (location where the bar changes from horizontal to diagonal) region of the diagonal hanger bars induce a different force distribution than what is typically encountered in a conventionally reinforced beam. The function of the diagonal hanger bars are to resist both shear and torsion actions which are induced in the beam. These actions are resisted by axial forces developing in the hanger bars. When compression forces exist in the diagonal hanger bars (green arrows), additional tension ties (red arrow) to the shear reinforcement are required at the "elbow" to obtain equilibrium (Figure 10) and prevent bursting.

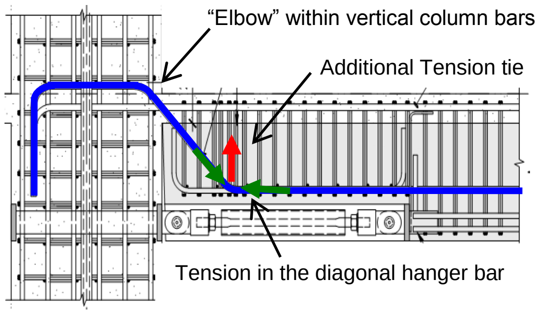

Figure 10 Force components at the "elbow" of the diagonal hanger bar

The position of the top "elbow" relative to the vertical column bars is also important. The start of the curve for the elbow needs to be located on the inside of the outermost vertical column bar. This is to ensure that the actions which develop at the elbow are well confined and can be resisted by the horizontal column stirrups and vertical column bars. If the elbow is not positioned on the inside the vertical column bars, the cover concrete could potentially spall during an earthquake and possibly buckle the hanger bar at the elbow.

4 SECONDARY STRUCTURAL ELEMENTS

Consideration was given to the behaviour of both the primary and secondary structural elements in this design project. This is important to ensure a similar level of performance is achieved by both the primary and secondary structural elements.

Seismic gaps were used in a number of places to ensure that the secondary elements were not going to be damaged by the displacements of the primary elements. Some of the locations that seismic gaps were employed included between: the adjacent buildings, precast panel elements and the columns and walls of the building and between the flexible roof diaphragm and the primary structural elements.

5 CONCLUDING REMARKS

The purpose of this paper was to discuss aspects of a low damage design that was carried out on a new structure in Christchurch. Descriptions were provided of some of the learning's that were gained from this design.

With regards to the design of a rocking wall system, it is recommended that careful consideration is given to the type of damping mechanism that is employed for the rocking wall. The type of damping can have a significant affect on the performance of the system. Due consideration should be given to the durability of the exposed rocking surface as this surface would typically be extremely difficult to replace.

Unbonded dampers were used for the yielding elements of the slotted beams in this building. Consideration to low cycle fatigue was given and it was found to affect the length of the damper. A method was presented on how to deal with low cycle fatigue.

Slotted beams have a greater torsional susceptibility compared to conventional beams. Discussion on the differences in torsional resistance for slotted beams compared to conventional beams was provided.

Careful consideration needs to be given to the detailing of slotted beams. The differences in geometry of slotted beams compared to conventional beams affect the detailing requirements.

REFERENCES:

Au, E., (2010). The mechanics and design of a non tearing floor connection using slotted reinforced concrete beams, *Department of Civil and Natural Resources Engineering*. Christchurch, New Zealand, University of Canterbury, Master of Engineering.

Bryne, J. D. R. and Bull, D K (2012). Design and testing of reinforced concrete frames incorporating the slotted beam detail, *New Zealand Society for Earthquake Engineering.* Christchurch, New Zealand.

Leslie, B. J. (2010). The development and validation of a non-tearing floor precast concrete structural system for seismic regions, *Department of Civil and Natural Resources Engineering*. Christchurch, New Zealand, University of Canterbury. Master of Engineering.

Lindsay, R. (2004). Experiments on the Seismic Performance of Hollow-core Floor Systems in Precast Concrete Buildings. *Department of Civil Engineering*. Christchurch, New Zealand, University of Canterbury. Master of Engineering.

Fenwick, R. C. and Davidson, B. J. (1995). Elongation in Ductile Seismic-Resistant Reinforced Concrete Frames. *ACI SP-157 Recent Developments in Lateral Force Transfer in Buildings*: 143-170.

Fenwick, R. C. and Fong, A. (1979). The Behaviour of Reinforced Concrete Beams under Cyclic Loading. Report No 176, *University of Auckland*, Auckland, New Zealand.

Fenwick, R. C. and Megget, L. M. (1993). Elongation and Load Deflection Characteristics of Reinforced Concrete Members Containing Plastic Hinges. *Bulletin of the New Zealand National Society for Earthquake Engineering* 26(1): 28-41.

MacPherson, C. (2005). Seismic Performance and Forensic Analysis of a Precast Concrete Hollow-Core Floor Super-Assemblage. *Department of Civil Engineering*. Christchurch, University of Canterbury. Masters of Engineering.

Mander, J, B., Panthaki, F. D., and Kasalanati, A., Low Cycle Fatigue Behaviour of Reinforcing Steel, *Journal of Materials in Civil Engineering*, 6(4): 452-468.

Matthews, J. (2004). Hollow-core Floor Sab Performance Following a Severe Earthquake. *Department of Civil Engineering*. Christchurch, University of Canterbury. Doctor of Philosophy.

Muir, C. Bull, D. K. and Pampanin, S. (2012), Preliminary observations from biaxial testing, of a two storey, two by one bay, reinforced concrete slotted beam super assembly. *New Zealand Society for Earthquake Engineering*. Christchurch, New Zealand.

Standards New Zealand (2004). NZS1170 Structural Design Actions Volume 1. *Standards New Zealand*, Wellington, New Zealand.