Trends regarding the structural enhancement of URM buildings (including case studies)

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ABSTRACT: This paper presents case studies that include estimations of costs for typical strengthening schemes for URM buildings (with flexible diaphragms) for both simple and complex structural configurations for the purpose of exploring when minimum cost strengthening measures may be employed in New Zealand.

New Zealand and California (and other west coast USA states) have Unreinforced Masonry Building (URM) stocks that are similar in both construction practice and design. Therefore it is feasible to study data from both locations in order to seek to align best engineering and business practice on both sides of the Pacific for seismic strengthening. Research by John Kariotis et al of ABK (1981, 1986,) Los Angeles, and more recently by Professor Jason Ingham et al (2000 to 2013), University of Auckland, has advanced a new “liberal” approach to URM strengthening. This research indicates that the out-of-plane URM wall inertial forces are reduced for flexible diaphragms to approximately one-quarter and these URM walls are somewhat more stable than previously anticipated.

This new approach lead to an adoption of a “bolts-only” retrofit ordinance by many Californian cities when the wall slenderness (based on storey height) is less than 14 for top storey walls of double wythe brick, and less than 9 for lower storey three wythe walls. Such strengthening programs merely involve the installation of anchorages around the building perimeter along with parapet bracing, and is deemed sufficient in all but the most irregular or “complex” buildings. Cost-saving measures have since been extended further.

1 INTRODUCTION

Cost is a significant factor affecting property owners’ decisions to seismically rehabilitate their URMs according to Egbelakin et al. (2011) and has been generally high. Hidden costs associated with strengthening URMs is regarded in California as one of the main contributors to the high cost of retrofitting URMs (EERI, 2003.) Local NZ contractors have confirmed in private communication that opening-up and making-good work adds disproportionately to the costs for this type of project.

URM’s are often identified as ‘earthquake prone.’ However, relatively “simple” URM buildings can be economically strengthened for costs in the range of $60 to $150 per m2. “Complex” URM buildings often require more expensive strengthening design, generally costing in the range of $300 to $700 per m2. The more expensive approach is necessary for “complex” URMs where there are walls with dominant window openings, rigid or long-span diaphragms, storey heights are large, walls are thin or of low strength, and/or cross walls are insufficient. This more expensive strengthening often involves one or more of the following actions in conjunction with anchoring walls to floor diaphragms, (and installing parapet braces and cavity ties where appropriate):

- The installation of portal or braced frames, sometimes with soldier beams (purlins) spanning between the frames spaced to suit the particular building demand, say 5.0m to 8.0m.
- Application of a Fibre Reinforced Polymer (FRP) to one or both surfaces of the URM walls.
- The application of a thin coating of reinforced shot-crete dowelled to the URM interior face.
- Insertion of steel or fibre reinforcement strips.

- Post-tensioning the walls with vertical cables to pre-compress the brick.

The results from the research by Kariotis et al and Ingham et al suggest that the transverse (out-of-plane) URM seismic wall forces are reduced for flexible diaphragms compared with rigid diaphragms, to approximately one-quarter, and work-energy principles demonstrate greater out-of-plane stability of URM walls than previously anticipated (Moore 1983, Kariotis 1986, Blaikie 1999,2002, Ingham 2013). The consequence of these benefits is that many California Cities have adopted a more “liberal” Bolts-only URM Retrofit Ordinance at a cost of less than $100 per m2. Such an ordinance may only require the installation of anchor bolts at approximately 800mm spacing (and not more than 1400mm) around the building perimeter (along with parapet bracing.)

2 TECHNICAL ISSUES

With regards to the out-of-plane assessment of a URM wall, ductility is not a factor because the work-energy stability assessment method developed by Ingham (2013) is based on the displacement method.

For the in-plane action it has been suggested that it is satisfactory to use in a force-method analysis with a ductility μ=2, and structural performance Sp=0.7. (Reference: private communications with Jason Ingham and Ernesto de Peralta of Opus.) Others argue for a displacement ductility μ=1.5, and NZSEE now recommends this value of 1.5 for an IEP assessment. If a displacement method is used it has been argued that a damping of 15% in-plane be used, although recent European research indicates that a lower damping of 5% may be more appropriate. (Reference: private communications with Jason Ingham and Ernesto de Peralta of Opus.)

The NZSEE committee attending to the re-write of NZSEE has effectively agreed that the in-plane walls (generally independent of their specific material) are sufficiently stiff that the excitation at the diaphragm is comparable to the excitation at the ground. Hence we don’t need to use the Parts and Components spectra on the diaphragm, but rather use regular spectra. However we should use the Parts and Components spectra for the out-of-plane walls. The NZSEE committee have signed-off on the work-energy equation procedure (Ingham 2013) for assessing the out-of-plane capacity of URM walls, except that delta 2 should be 0.25 x instability displacement and the critical displacement should be 50% of the wall thickness. The selection of an appropriate period for the out-of-plane wall is a key step.

For the tension connections the committee has determined that the period in the Parts and Components method is the period of the out-of-plane wall and the loads from the Parts and Components spectra, but for diaphragm shear connections use the period of the diaphragm and the normal spectra. (Private correspondence with Jason Ingham; and Ted Blaikie, Ernesto de Peralta, and Robert Davey of Opus.)

3 CALCULATIONS

Elastic URM strengthening design practice often puts undue emphasis on masonry stress levels, but seismic capacity is likely to be governed by stability and energy considerations so it is desirable to limit the total building drift to between 0.4% and 0.5%. Reference: Kariotis, Priestley, EERI, and Ingham.

Each flexible diaphragm responds at its own natural frequency based on its geometry, stiffness and tributary mass, and is considered uncoupled from the response of adjacent stories. Referring to published figures from Priestley (1985,) the flexible diaphragm response to seismic excitation will have a period of vibration much greater than the in-plane walls. This floor response becomes the input acceleration for the face-loaded OOP walls. To fully understand the URM behaviour it is important to perform 3-D dynamic nonlinear analysis… which indicates that floor accelerations at different levels will be of different magnitudes to ground and in-plane wall accelerations – and may be out-of-phase. For very flexible floor diaphragms their accelerations will be small. So out–of-plane wall action of walls perpendicular to direction of seismic action, governed by a larger diaphragm period of approx. 2.0 seconds so C(Td) seismic coefficient for this component is typically less than 0.05G versus Cd =0.2 at seismic zone C (1/4 reduction.) Consequently the calculated in-plane shear loads on the URM walls using SRSS with Vd and Vw will be much less than what the traditional method of using the basic Cd for out-of-plane (OOP) action a generalisation applicable to all seismic zones:

However, the SRSS method for combining modes is not always appropriate: given the large difference in the period of the walls responding in plane and the period of the diaphragm, the peak in-plane response will probably coincide with near the peak response of the diaphragm. The approach of giving the building credit for the longer fundamental period of vibration of the flexible timber diaphragms has been proposed in NZSEE2006 per the 2011 amendment:

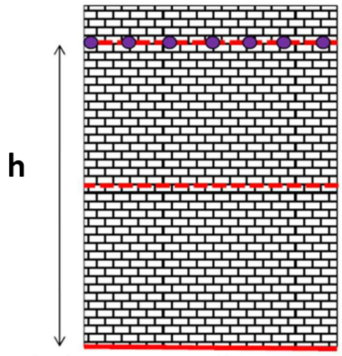
4 IN-PLANE ANALYSIS

Consider the weight tributary to the diaphragm = Wd (including out-of-plane walls) so in-plane shear: Vd = C1.C3.C(Td).Wd where C(Td) is per 3.1.1 of ANZS1170.5 where Td is diaphragm period, approx. 2.0 seconds so C(Td) approx. = 0.05G
Vw (in-plane) = Cd .Ww where Ww = weight of in-plane wall.
Tests by Ingham et al (2011) indicate the diaphragm period is not reduced significantly when the required anchor bolts connecting the OOP walls to the diaphragms are installed. (Beware adding plywood to an old floor or roof in that it may result in a relatively rigid diaphragm and increase seismic in-plane demand.) It is suggested by some engineers not to use Cp parts coefficient to calculate anchorage demand. Whatever, the appropriate demand coefficient may well be smaller than Cd - and significantly larger than C(Td.) Section 8.1.1 of NZS1170.5 (for Parts) has a upper limit of 20% for the mass of the part relative to the total structure mass before it should be considered a part. This is because it is meant to apply in the “pimple on the pumpkin” situation where the pimple is responding to an input motion corresponding to the response of the pumpkin. It is not possible for the pimple to drive the response of the pumpkin. Dominant in-plane seismic loads are coming from the URM In-Plane walls. Consider C(Ti) = 0.05 at Td = 2.5 seconds: NZSEE commentary suggests using Cd(Ti) = 0.05 x Sp/Kμ = 0.025

5 OUT-OF-PLANE WALLS

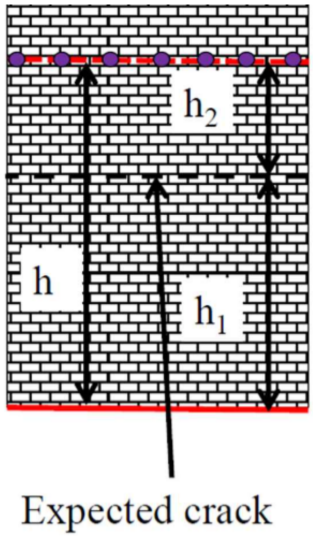
The work-energy method was initially developed by Tom Moore et al, then focussed upon URM by Ted Blaikie, and extended by Ernesto de Peralta of Opus and Professor Jason Ingham at the University of Auckland, and is employed here to calculate a 44% New Building Standard (44%NBS) capacity for a 210mm top storey wall of a two storey building with seismic zone C and soil type C: (Note that this method has been adapted by Blaikie to include the effect of door and window openings, or two-way action may be assumed locally to span around smaller openings.)

WALL (supported by floor or diaphragm at top)	Case Study #1B strengthen	
Masonry density (ρ_m) =	1800	kg/m^3
Wall height (h) =	2.8	m
Total wall thickness (b_{wnom}) =	0.21	m
Depth of mortar pointing (p) =	0.01	m
Effective wall thickness (b_w) =	0.19	m
Applied axial force (O) =	0	N
Ecentricity (e) =	0	m
Centre of mass coeff (c) =	0.5	
Ht of 1st wall segment (h_1) =	1.87	m
Ht of 2nd wall segment (h_2) =	0.93	m
Mass of 1st wall segment (m_1) =	706	kg
Mass of 2nd wall segment (m_2) =	353	kg
Weight of 1st wall segment (W_1) =	6,922	N
Weight of 2nd wall segment (W_2) =	3,461	N
Total weight (W) =	10,383	N
Calculate max wall resistance (F_0) =	**3.00**	kN
Instability displacement (Δ_{ins}) =	**0.190**	m



Diaphragm-to-wall restraints
Roof diaphragm only

Site Location factors:		
Hazard Factor (Z) =	0.4	*NZS 1170.5 Table 3.3*
	C	*NZS 1170.5 Table 3.2*
Return Period Factor (R) =	1.0	*NZS 1170.5 Table 3.5*
Near Fault Factor (N(T,D) =	1.0	*NZS 1170.5 Cl 3.1.6.2 & Table 3.7*
Natural wall period (T_p) =	0.745	sec
Spectral Shape Factor ($C_h(0)$) =	1.33	*NZS 1170.5 Table 3.1*
Site Hazard Coefficient (C0)) =	0.51	$C_h(0)$ x Z x R x N(T,D)
Height to mid-height of wall (h_i) =	1.4	m
Floor Height Coefficient (C_{Hi}) =	1.23	*NZS 1170.5 Cl 8.3 (for all h_i < 12m)*
Part Spectral Shape Factor ($C_i(T_p)$) =	2.0	$C_i(T_p) = 2.0$ for $T_p \leq 0.75$ sec
		$C_i(T_p) = 0.5$ for $T_p \geq$ 1.5 sec
		$C_i(T_p) = 2(1.75 - T_p)$ for $0.75 < T_p < 1.5$ sec
ontal Acceleration Coefficient ($C_p(T_p)$) =	1.25	C(0) x C_{Hi} x $C_i(T_p)$
Wall Displacement Response (D) =	**0.260**	m
Percent of New Building Standard =	**44%**	%NBS



6 ANCHOR BOLT DESIGN

A typical anchorage detail is shown in Figure 1 below.

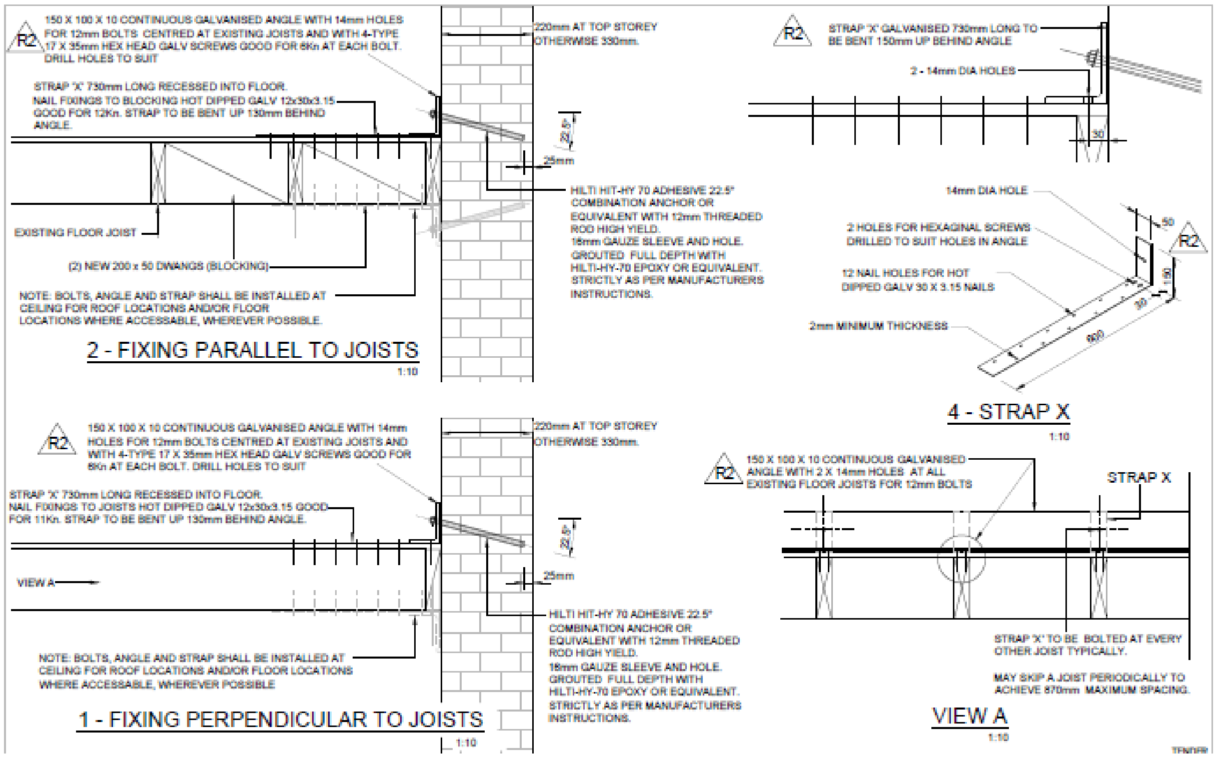


Figure 1. Anchor bolt connection

Note that some local engineers prefer not to use epoxy injection because they argue that epoxy is not as effective as cement grout injected into proposed 60mm diameter holes drilled in URM walls. However, many other practitioners in NZ and overseas prefer to install epoxy with a sleeve for cavity walls and for walls that may have substantial internal cavities/large cracks. Ingham tests show that they are equally effective. In California an industry grew up around URM adhesive anchorage and the URM industry evolved to standardised connections for majority practice – as shown in Figure 1 above with special inspection by the engineer during installation. Recent cost-saving developments involve the use of anchor details that avoid the breaking into the floor or attic space such as with coach screws from angles to joists, or installing Helifix type Bow-ties and even Dryfix wall ties from URM to joists.

7 URM STRENGTHENING COST DATA

See Figure 2 for a graph of unit costs of strengthening URMs versus %NBS for both simple and complex buildings in seismic zones A and C, derived from the case studies presented below.

A) Simple buildings using "liberal" strengthening method:

Case Study #1 Standard two-storey URM 2.8m tall, with no parapet Zone A and C

Consider a typical 40m x 20m x 2.8m storey height URM building with no parapet located in NZ seismic zone A or C on soil type C. Repair cost =$40 per m2 of floor area.

7.1 Note that at seismic Zone C: %NBS un-strengthened = 30%NBS, and strengthened %NBS = 100%NBS.

Note that at seismic Zone A: %NBS un-strengthened = 10%NBS, and strengthened %NBS = 44%NBS (per work-energy spreadsheet to check URM wall stability by Ingham 2013, see above.)

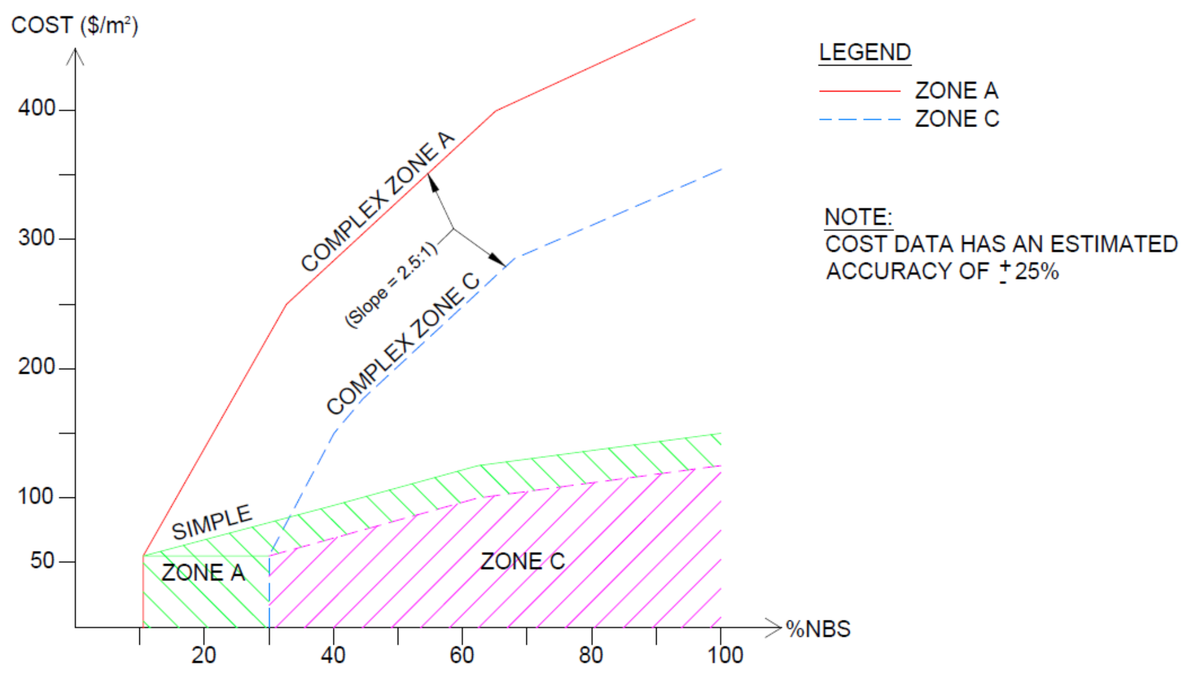


Figure 2. Unit costs of strengthening URMs versus %NBS for both simple and complex buildings in seismic zones A and C

Case Study #2 Standard One-Storey URM 5.6m tall, with no parapet

Consider a typical 40m x 20m x 5.6m storey height URM building with no parapet located on soil type C. Repair cost > $100 per m2

This is a larger unit cost because we must install and anchor a new mid-height diaphragm to reduce the out-of-plane span of the 5.6m tall URM walls. This can be achieved by installing a partial mezzanine floor around the inside of the building perimeter or installing a fire-escape-type landing structure or stiff beams with tie rods around the outside or inside of the perimeter at mid-height to function like the stiffener of a plate girder or bulkhead of a ship hull.

Case Study #3 Standard one-storey URM 2.8m storey height, with parapet

Repair cost = $40 per m2 plus cost of parapet at say $30 per m2 = $70 per m2

Case Study #4, California typical, pre-Kariotis research.

Approximately 25,900 URM buildings with an average size of 1,000 square metres have been inventoried in California Zone 4’s 365 territorial authority jurisdictions. In the early 1980’s, it was estimated that the URM Law would result in roughly $4 billion in retrofit expenditures in California seismic zone 4 with activity well into the twenty-first century. This cost, at $4000,000,000/(25,900 x 1,000m2) = US$155 per m2, 1980’s cost.

Case study #5, California typical, post-Kariotis research

The estimated repair cost for simple buildings is less than $100 per m2 and has been reduced from case study #5 by research developments, but for complex buildings the cost is in excess of $250 per m2.

Case Study #6 Dunedin Gym, Opus “Liberal” repair of simple building with parapets.

Total Area = 3000m2, and most of the building is four storey so the substantial cost of parapet bracing is diluted by the three floors of diaphragm anchorages.

Cost of retrofit = $500,000/3000sq.m. = $167 per m2, subsequently reduced to an estimate of $125 per m^2 by value engineering and reducing the tenant impact.

Case study #7 Linton House, Dunedin

Strengthening cost for this two storey URM is $88 per m2.

Repairs for this building, and also for case study #8 below, merely consisted of installing Helifix ties from the URM walls to the timber joists at diaphragm levels.

Case study #8 Cumberland Hall, Dunedin

Repair cost of this two storey is approximately $85 per m2 and consisted of installing tie rods to confine the URM walls at diaphragm levels.

B) Complex Buildings (data from Christchurch indicates a factor of 2.5 x cost to increase capacity from 33%NBS to 67%NBS):

Case Study #9 Napier, Miyamoto “Cautious” or substantial repair to 95%NBS.

This two storey 25m x 7m URM has an aspect ratio of almost four so required, in addition to the standard anchor bolt details, the addition of a steel braced frame at the storefront and roof diagonals to increase the capacity from 11%NBS to 95%NBS. The repair cost was $94k/360m2 = $260 per m2. (Low because a relatively simple building.)

Case study #10, 6-storey Dunedin Brewery, Beca

Case study #10 Dunedin Brewery, Beca

The strengthening scheme cost approximately $350 per m2 to improve the capacity to 67%NBS and involved strengthening the in-plane and out-of plane capacity of the un-reinforced masonry walls by installing concrete overlay (200mm shot-crete at top storey and 300mm at lower storeys with YD12 bars at 200mm E.W.) to the external walls and providing Helifix cavity ties at 600mm E.W. with 50mm embedment minimum to the outer brickwork layers (wythes).

Case study #11 1932 Church, Auckland, Beca

This church is a Category I historic place in the traditional layout with bell tower, tall narrow nave, aisles, transept, ceiling vaulting, etc. The era of construction spanned the Napier earthquake in 1931. After the earthquake the architect introduced a gravity concrete frame system in response to concerns at the time, but essentially the bulk of the building comprises load-bearing unreinforced brick. This church was strengthened in 2008 after it was deemed earthquake- prone due to a fundamental load path deficiency across the nave, and poor face load performance of some large brick wall panels.

The church was strengthened to 50% NBS by Beca as it was believed that this had no real impact on the historic fabric and it would deliver a substantial reduction in risk to occupant safety. The church was strengthened in the following way: Ground beams were installed between existing concrete pilasters in the floor cavity beneath the aisles to create upside-down portal frames. Similarly, the existing steel roof trusses were connected to the concrete pilasters at eaves level to ensure they also became portals. Steel rod roof bracing was installed to stop the building components moving independently (including bell tower). The bracing served to tie back the top of the four tall gable walls. Steel "strong- back" beams were installed behind high brick wall panels to improve their face load performance. Stainless helically wound drill-in brick cavity ties were installed from the exterior to tie the thin outer brick skin to the much thicker inner one (large gable brick wall panels treated only). Selected brittle stone ornamentations were discretely "pinned" back. The cost of this work in 2008 was $300,000 for a floor area of 700m2, which equates to $430/m2 which is a high unit cost because this URM structure is highly complex.

Case study #12 1972 Church, Hawke's Bay, Beca

This is a small single level building comprising largely unreinforced concrete block masonry walls with a high stud height and a timber-framed roof. The church was strengthened by adding a plywood ceiling diaphragm to provide a load path for lateral loadings. A 100mm thick reinforced concrete overlay (shot-crete) was added to the front gable wall to improve the face load performance of this unreinforced element. An existing Fibrolite lined timber gable wall was re-lined in plywood to improve its in-place resistance and connection to the new ceiling diaphragm and existing foundation. The cost of this work was $200,000 for a floor area of 260m2. This equates to $770 per m2.

Case study #13 Five Christchurch strengthening projects, Opus

- Ivey Hall, Lincoln University;
- Christchurch Boys' High School, Main Block
- Christchurch Family Court.
- The Christchurch Environment Court.
- Ivey Hall, Lincoln University

Opus International Consultants Ltd designed strengthening for these five buildings to 67%NBS. These buildings are all complex and the main repair techniques used were to tie back gable walls and facades and add steel bracing and/or shot-crete. All of the strengthened buildings in Christchurch performed well with the exception of the Environment Court, which has now been demolished. The strengthening cost was approximately equal to 100%-120% of the shell replacement cost.

Case study #14 Knox College, Dunedin

Strengthening to approximately 100%NBS consisted of adding anchorages, parapet braces, and steel/concrete frames and cost approximately $350 per m2.

Case study #15 Christchurch URMs

Christchurch City Council has published information on the projected cost of seismic improvement of (complex) URM buildings that identifies that the cost to strengthen such a URM building to 33% NBS is approximately $400 per m2. The URM building damage statistics indicate that 85% of those 224 buildings demolished following the Christchurch earthquake swarm were constructed of unreinforced masonry, suggesting that this URM class of building suffered the most extensive damage in the earthquakes. 'Loss of façade' (out-of-plane failure) is the failure mode responsible for greatest number of fatalities and subsequent demolitions. 55% of unbraced parapets collapsed, and the performance of braced parapets was disappointing, 25% collapsed. Reference: "Demolition statistics and information on the cost of seismic improvement." By Ingham et al., NZSEE presentation seminars, 2013.

8 RECOMMENDATIONS

Laboratory tests need to be performed on the newer generation of URM wall to diaphragm connections, such as Helifix-type bow-tie, HD, and dryfix helical screws from URM walls to joists and with anchor bolts attached to URM walls and say 450 long x 100mm x 50mm angle with 12mm coach-screws to the joists.

The use of a displacement ductility $\mu=1.5$ is appropriate in a force-method analysis of URMs.

Use the Parts and Components spectra for the out-of-plane walls along with the work-energy equation procedure (Ingham 2013) for assessing the out-of-plane capacity of URM walls.

For the tension anchorage connections the appropriate period in the Parts and Components method is the period of the out-of-plane wall, and the loads should be derived from the Parts and Components spectra, but for diaphragm shear connections use the period of the diaphragm and the normal spectra.

It is suggested that a more comprehensive assessment of the data available for URM buildings be performed, so as to build a robust and transparent cost-benefit model for considering retrofit strategies for URM buildings. Unit costs of volume buildings such as large churches and civic facilities could have their floor area normalized by a factor of (building height/typical storey height of say 2.8m.)

The proposed URM seismic strengthening program for NZ is likely to have a significant impact on public safety (e.g. due to the risk of falling hazards.) Strategically important buildings (e.g. alongside critical transport routes,) are intended to be prioritized for assessment and strengthening; and in contrast, some exemptions will be available for lesser buildings where the impact of failure is low, such as farm outbuildings and some rural halls and churches. But there is no requirement to immediately prioritize old buildings on busy street frontages with unreinforced parapets and elevated appendages – elements which the Canterbury earthquake swarm experience has shown to have the potential to cause loss of life.

Complex URM buildings may be strengthened in stages to spread costs: URM buildings may be strengthened in one to four stage steps if it is desired to spread costs:

- 1st stage: improve public safety by eliminating falling hazards. This is done by securing/strengthening URM building elements that are located at height (e.g., chimneys, parapets, ornaments, gable end walls).
- 2nd stage: strengthen URM walls to prevent out-of-plane failures; by installing anchorage connections between the walls and the roof and floors.
- 3rd stage: if further capacity is required to survive earthquake loading, then the in-plane shear strength of URM walls can be increased by post-tensioning or by the insertion of steel and/or concrete frames or adding reinforced shotcrete or FRP material to the walls to supplement or take over the seismic resisting role from the original unreinforced masonry structure.
- 4th stage: ensure adequate connection between all structural elements of the building so that it responds as a cohesive unit rather than individual, isolated building components. In some situations it may be necessary to stiffen the roof and floor diaphragms, although this could increase seismic demand.

An affordable approach to the imperative of strengthening to a greater capacity of 67%NBS of our massive historic buildings such as cathedrals, law courts, school buildings, and civic centres is in need of further research. Initial indications are that reversible methods such as post-tensioning of slender columns and walls, could be employed in conjunction with displacement controls such as installation of diaphragms/bulkheads or cross-ties. Embedding a polymer textile or steel layer within or coating the URM surfaces with FRP or shot-crete can also be effective in providing a larger building capacity, but are less reversible methods.

Simple URM buildings do not contain large amounts of invasive window openings and so can generally be adequately strengthened for significantly less than $100 per m2 based on the presented case studies and as shown in Figure 2, for both seismic zones A and C. However, URM buildings can become unstable under large lateral deformation, as a result of P-delta effects. Because flexible substructures such as timber diaphragms and out-of-plane walls tend to have relatively long fundamental periods of vibration (>1.5seconds,) such structures tend to perform adequately when located on sites with firm soils because the energy content of ground shaking transmitted by such sites to the structures is relatively limited. However, flexible structures located on sites with deep soft soils can experience large displacement demands. Excessive drifts may occur on long span timber diaphragms when they are used in conjunction with heavy URM walled structures. Diaphragm drifts need to be checked for these types of structures.

Several improvements can be implemented should drifts exceed acceptable levels, including: reducing the diaphragm span by adding frame sub-structures and/or timber framed cross walls; or by increasing the stiffness of the diaphragm; or modifying structural and non-structural elements in such a way that larger potential drifts can be managed.

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