Earthquakes and historical buildings — it isn't all bad news

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Abstract

Historical buildings, particularly those built principally from unreinforced masonry (URM), have had a "bad press" when it comes to behaviour under seismic loading. We continue to hear explicit or implicit statements from engineers to the effect that lime mortar and wrought iron have no redeeming properties or that masonry has no ductility. Experience and research, both in Australia and overseas, shows this not to be the case. By using examples I hope to be able to demonstrate that historical buildings have a great deal to teach us about seismic loading. I also take the opportunity of describing a number of seismic strengthening projects.

Whilst the paper attempts to give an overview, it deals in most detail with URM buildings and their near cousins, adobe and other forms of earth construction.

1 What are the principal behavioural differences?

1.1 Structural forms

Almost without exception, historical buildings are load bearing structures or have significant load bearing components. Even the great railways' and industrial buildings from the 19th century, which may have had large wrought iron or steel frames, usually relied on load bearing masonry for their lateral stability. In general it was not until after World War I that clad framed buildings started to gain the ascendancy.

From an engineer's perspective, the significant difference with most historical buildings is the need to resist seismic loading without the presence of easily analysable moment resisting frames.

Although not a large consideration in Australia, the other significant structural form is arches and domes and their derivative vaults. These have a mixed history in resisting seismic loading, but the survival of so many ancient examples in earthquake-prone Italy suggests that we have more to learn.

The most important aspect of structural form is continuity and its maintenance. Roofs and floors must be tied to walls and the connections maintained. This is obvious to the present audience, but it has been a principal source of failure in historical buildings and is often difficult to check.

1.2 Materials

1.2.1 Masonry with lime mortar v. cement mortar

In the recently issued Practice Note No. 18 from the Association of Consulting Structural Engineers of NSW, on the inspection of hanging awnings, we find, with no further qualification, the statement:

> older masonry walls may have lime mortar rather than cementitious mortar or may have a combination of both if repairs have been undertaken previously (ACSE, 2008).

This implies a perceived wisdom that all lime mortar will be weak and probably of no structural merit. It also implies that lime mortar is not cementitious and assumes that "cementitious" equals Portland cement. I find it disappointing that an elite body, which arguably has the cream (in NSW at least) of structural engineers as its members, could be so ignorant.

Many historical buildings, unfortunately, have very poorly made lime mortar, and modern buildings are still being built with very poorly made cement mortar. Each has to be assessed separately, as the best lime mortars, found in government infrastructure, were made using hydraulic lime, and have flexural properties far exceeding what is allowed for Portland cement mortar under the current Masonry Structures Code, AS 3700. Flexural tests carried out on core samples for projects for NSW RailCorp have found a number of structures built from the 1870s to the 1920s where the characteristic flexural tensile strength (f'_{mt}) exceeds 1 MPa, with individual cores testing at greater than 6 MPa. Such brickwork has significant capacity to resist seismic loading.

Planned research in the UK is seeking to demonstrate that masonry with lime mortar has significant capacity to absorb seismic energy, but funding is still to be found (Taylor, 2008).

1.2.2 Timber v. steel and concrete

Although not explicitly stated in Codes and texts, timber appears to have significant ductility as it was used in typical historical structures. It is also subject to attack from insects and fungi: in most cases, however, the damage is easily detectable. The performance of major timber structures, such as Asian temples, in earthquakes has been well documented (Zhiping, 2000).

Bolted joints in timber and, indeed, mortice and tenon joints found in older structures, can experience significant joint movement, with corresponding energy absorption, while maintaining structural integrity and leaving little or no permanent damage. By contrast, the stiffness found in most steel and concrete framing joints may only absorb energy in the post-elastic range with accompanying permanent damage.

If significant deterioration has not occurred in timber framed structures, it is likely that they will survive earthquakes better than similar steel or concrete framed structures.

1.2.3 Iron v. steel

There is little difference between the behaviour of iron and steel under seismic loading, with the possible exception of crack propagation characteristics under failure conditions. There is, however, one incidental advantage of wrought iron over steel: it is usually less prone to corrosion.

1.2.4 Load bearing masonry v. cladding

A frame clad with masonry behaves differently, and often markedly so, from an all-masonry building, due to the different stiffnesses of the two elements, unless they have been connected sufficiently to act compositely. Fortunately, very few historical buildings of significance use masonry clad frames; masonry clad frames in this context should not be confused with timber reinforced masonry best typified by the buildings of mediaeval England.

1.2.5 Other materials and combinations

This section would be incomplete without mentioning earth buildings, both those types which have had a good history of survival in earthquakes and those which may now be retrofitted by simple means following the work of Dominic Dowling at UTS (Dowling, 2004 etc.).

Of particular note for traditional masonry buildings is the superior performance of those which have been built with embedded timbers either as lacing or simply as horizontal members, such as in the Himalayas (Langenbach, 2006). Similar performance enhancement has been found in stone buildings with horizontal embedded material and this performance enhancement has been demonstrated with discrete element analysis (Brookes and Swift, 2000 and Jordan and Brookes, 2004).

1.3 The quandaries of observed behaviour

Those of us who were involved in the aftermath of the Newcastle earthquake, and similar events, are aware of many structures which received little or no damage when nearby buildings were severely damaged: many of the buildings in question were built of load-bearing URM. The types of damage found in Newcastle have been well documented and that in heritage buildings in a previous paper co-authored by me (Jordan, Ludlow and Trueman, 1992). An overview of all damage can be found in the Newcastle Earthquake Study (IEAust, 1990). Unfortunately, in all the papers little or no attention has been given to the undamaged buildings and the reasons why they survived so well.

Many of the buildings to which I refer, such as that seen in Figure 1 in which not even a hairline crack was found, despite it being in an MMVI to MMVII area, were well built and did not have laterally unsupported elements such as tall parapets and gables; neither did such buildings have soft understoreys, even though some were certainly less stiff in the commercially-oriented ground floors compared to the upper floors which had many small rooms. However, the lack of or very minor cosmetic damage suggests other factors which have not been properly explored:

- the natural period of the building was such that it was not excited by the seismic event (as can be deduced from AS 1170.4);
- there was substantial energy absorption capacity in the structure.

Figure 1: Typical undamaged building in Newcastle

2 Maintenance

One of the principal causes of damage to historical masonry buildings in the 1989 Newcastle earthquake was failure due to corrosion of embedded iron and steel ties. The most common such failures were ties in cavity brickwork.

There have been many published photographs of the typical tie failures in Newcastle, but as memories fade and new practitioners start out it needs to be emphasized that ties do not need to show any signs of corrosion in the cavity: it starts in the outer brick skin. I still hear of engineers using adapted medical equipment to view cavities to detect tie corrosion! It is usually more effective to look for signs of cavity movement, such as at window reveals.

3 Retrofitting

3.1 Reinforcement or base isolation

Two means of protecting historical masonry buildings from earthquake damage have been used: strengthening by internal or external reinforcing and base isolation. There has been considerable retrofitting of buildings with various types of reinforcement in Australia, but there are no known examples of base isolation except where used for damping railway-induced vibration.

Work on heritage buildings requires a different approach from that on new buildings. This has now been recognized by the National Engineering Registration Board who approved the new registration discipline of Heritage and Conservation Engineering in early 2008.

In the examples which follow, three Australian seismic strengthening projects are described, each of which shows how the work can be carried out without compromising important heritage criteria and which uses an innovative approach.

3.2 Reinforcement – Newcastle Cathedral

The largest retrofitting project ever carried out in Australia was the conservation and strengthening of Christ Church Cathedral, Newcastle, following the 1989 Newcastle earthquake. This project has been the subject of a separate paper (Jordan and Collins, 1997) which describes the project in considerable detail, including a large part devoted to the heritage conservation issues which are integral to such a project.

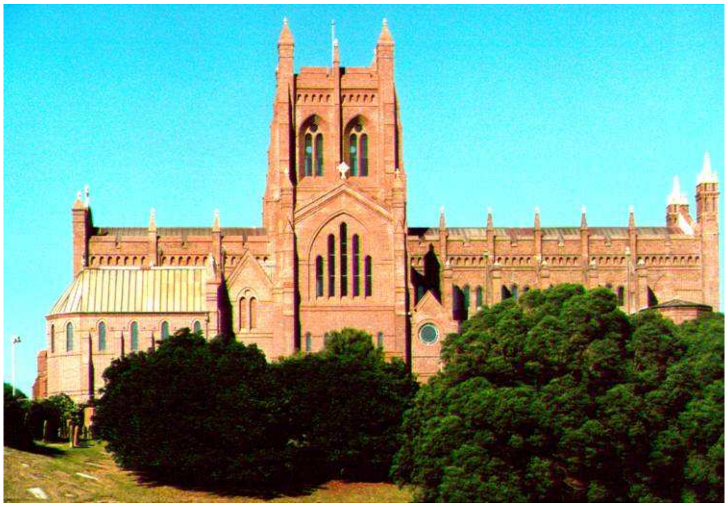

Figure 2: Christ Church Cathedral Newcastle, now a reinforced masonry structure.

Newcastle Cathedral uses mainly internal reinforcement as it was necessary to maintain the appearance: both outside and inside wall finishes are exposed face brickwork; a small amount of external framing was used in locations which could not be seen. Some 4000 metres of internal reinforcement using the "Cintec" system was used, with anchors made from stainless steel deformed bar up to 30 m long, and with bar up to 32 mm diameter, which were placed in drilled holes in the brickwork. It was the first project using the system in Australia and has the longest anchors ever installed.

Until the reinforcement system was "discovered" in Australia, the unacceptable strengthening design, from a heritage perspective, required large areas of demolition and rebuilding around a reinforced concrete core.

3.3 Reinforcement – Adelaide High School

A project with similar heritage constraints in which the author was involved was the seismic strengthening of Adelaide High School where, again, heritage considerations precluded any alteration to the appearance of the external brickwork or the internal wall finishes. In this case the cavity brickwork external walls were strengthened by grouting anchors into the wall cavities to form a series of reinforced bands: the walls were then connected within the roof space by hidden steel framework (see Figures 3 and 4.).

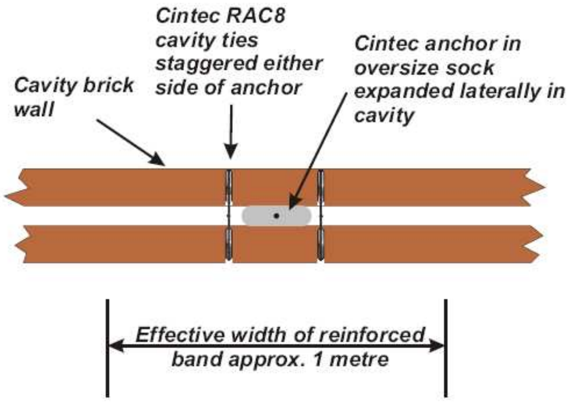

Figure 3: Strengthening system used at Adelaide High School

Figure 4: Bricks removed to demonstrate expanded grout sock

3.4 Reinforcement – Canberra Brickworks chimney

Where steel or concrete frames are not precluded because of heritage constraints, it is often significantly less expensive to use a steel frame. An example of this approach was the tall brick chimney at the former Canberra Brickworks at Yarralumla. When the brickworks was originally built, all chimneys were short with forced draught, in accordance with Burley Griffin's guideline that they should not be taller than the pine trees. Following World War II this principle was not followed when the brickworks was expanded to cater for the building boom, and a 150 ft (45.7 m) stack was built: after the closure of the brickworks the stack was recognized for its heritage significance.

Unfortunately, without considering all the implications, much of the brickworks' land was subdivided, and town houses were built within 15 m of the base of the stack. As happens, another arm of government then questioned the safety of the stack under wind and earthquake actions: the author was engaged firstly to assess the stack and then some time later to design a strengthening system. With a more detailed analysis than was first done using AS 1170.2, it was shown that the stack was safe against wind, but its apparent lack of ductility made it unable to comply with AS 1170.4 for earthquake.

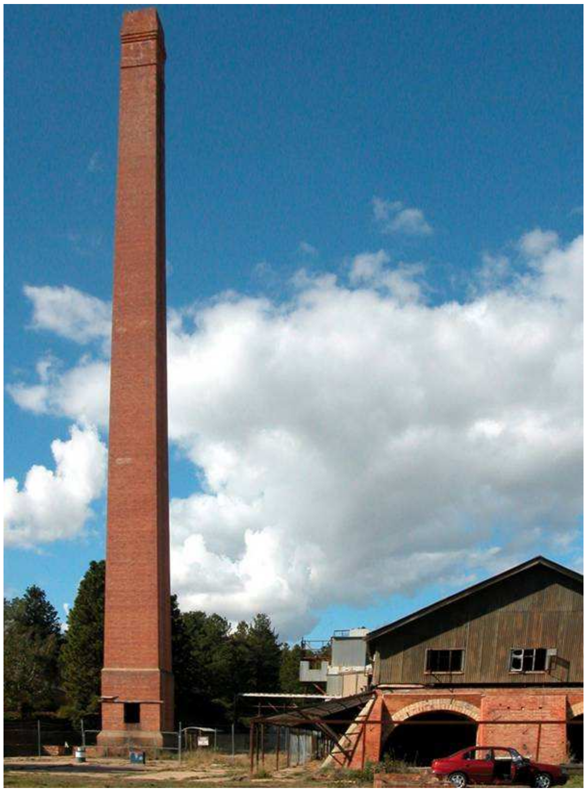

Figure 5: Stack S3 at Yarralumla. The new townhouses are behind the fence on the left.

Various strengthening schemes were considered, and rejected, as follows:

- reinforced concrete filling — would have overloaded the foundations and been very expensive to build;
- drilling and internally reinforcing the walls — extremely expensive and near the known limits of feasibility;
- a heavy steel internal frame with stiffness similar to the brickwork — high cost and construction difficulty;
- a light steel internal frame acting compositely with the brickwork — the chosen solution.

The light steel frame also had the potential of being its own scaffolding as it was erected inside the chimney. However, survey difficulties precluded fabrication of the frame for a "tight fit" which would have allowed shear connectors to be made from plates inserted into bed joints to give the required composite action connection.

The problem was solved by using a cementitious masonry anchor with moment capacity which allowed the frame to be up to 120 mm away from the brick face. In Figure 6 the steel frame at one panel point can be seen with one

Figure 6: Composite attachment of frame to brickwork using cantilevered SHS anchors

anchor fitted and the other about to be. The work, which included rock bolting the base of the frame to a depth of 6 m was successfully completed without scaffolding. All steelwork was carried in through an opening near the base of the stack and the angle sections used were able to safely carry scaffolding planks as the frame rose. Once the frame was completed the anchors were installed and bolted to the frame.

4. Conclusions

Historical buildings using traditional materials have survived well in seismic zones for thousands of years and a better understanding of why some survive could be of great benefit for seismic design. However, our knowledge gaps require strengthening of buildings to meet current risk criteria and some different approaches to strengthening design are described which have had good structural and heritage outcomes.

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