Lessons learnt from the performance of buildings incorporating tilt-up construction in the Canterbury Earthquakes

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ABSTRACT: In this paper, buildings incorporating tilt-up construction are examined from several perspectives using relevant case studies. The behaviour of these buildings during the Canterbury Earthquakes is reviewed, and methods used to repair earthquake damage are then discussed. Specific aspects including grouted connections, bolted connections and panel reinforcing are examined in detail. Issues related to the design and construction of new buildings which incorporate tilt-up construction are presented and discussed.

Buildings incorporating tilt-up construction have generally performed well during the Canterbury Earthquakes. In most cases, earthquake damage to these buildings was repairable, even in situations where liquefaction-induced settlement occurred. Ductile panel connections exhibited good seismic performance, and the use of cold-drawn mesh has not resulted in any panel failures being initiated under face loading.

Figure 1. The seven-storey West Fitzroy Apartments building in central Christchurch is an example of a multi-storey tilt-up building which has performed exceptionally well during the Canterbury Earthquakes

INTRODUCTION

Tilt-up construction has been a popular method for erecting buildings in New Zealand since it was introduced in the late 1950s (Ellen, 1961). It has become the principal form of construction in Christchurch for industrial buildings, and has seen increasing utilisation in low-rise commercial and residential structures. More recently, tilt-up methods have been extended to allow the construction of multi-level buildings, particularly in Christchurch. Today, there are more than one thousand commercial buildings in Christchurch which were built using tilt-up construction techniques, and have since been subjected to the Canterbury earthquake sequence.

Typically, wall panels in single-level tilt-up buildings act primarily as cladding panels, and are often critical under face loading with low demand on in-plane resistance. In contrast, wall panels in multi-storey buildings which incorporate tilt-up construction often form the primary lateral load-resisting system and are thus subjected to significant in-plane loading. The Canterbury Earthquake sequence has demonstrated that, whilst there were some instances of undesirable behaviour, buildings which incorporate tilt-up construction have typically performed well (Henry and Ingham, 2011), with most single-level tilt-up buildings able to be occupied almost immediately following each of the major earthquakes.

Following the earthquakes, many engineers are now involved in the Detailed Engineering Evaluation (DEE) process, and with tilt-up buildings forming a significant portion of commercial buildings in Christchurch (in terms of total number, total floor area and total value), it is important that DEEs present realistic results in terms of observed building behaviour and predicted behaviour under future earthquakes. Further, as tilt-up construction appears to remain a popular choice, there may be scope for improvements to design and construction techniques.

To this end, structural engineering guidance published following the earthquakes has focussed on two key areas. Assessment of existing earthquake-affected buildings is addressed in documents such as the DEE guidelines, published by the former Department of Building and Housing's Engineering Advisory Group (EAG, 2012). This document contains a list of 'generic building issues' pertaining to both single-level and multi-storey tilt panel structures. Design of new buildings is covered by publications such as the Structural Engineering Society's Interim Design Guidance (SESOC, 2012), however 'industrial tilt panel structures' are beyond the scope of this document. Despite the prominence of tilt-up buildings in the Christchurch CBD, recommendations from the Canterbury Earthquakes Royal Commission's final reports (CERC, 2012) were limited to cladding panels on multi-storey commercial buildings.

Several relevant examples are given in this paper of the performance of tilt-up buildings in the Canterbury Earthquakes. Key issues in the guidance documents which relate to tilt-up construction include bolted panel connections, panel reinforcing, grouted panel connections floor-to-panel connections, and panel slenderness. These are presented and discussed in this paper in the context of the published guidance, and illustrated using the relevant case studies. Tilt-up construction referred to in this paper is, as per the Tilt-Up Concrete Association's definition, a method in which concrete wall panels are cast on-site and tilted into place (Brooks, 2000). However, many of the principles can be extended to panels cast off-site.

RELEVANT CASE STUDIES

Case study 1: West Fitzroy Apartments

The seven-storey West Fitzroy Apartments building (Fig. 1) located within the Christchurch CBD is a well-performing tilt-up concrete building. Constructed in 1998, the building received awards from the New Zealand Concrete Society, the Association of Consulting Engineers New Zealand, and the American Tilt-Up Concrete Association. The structure is constructed primarily of tilt-up concrete shear wall panels and a proprietary hollow-core flooring system with an in-situ concrete floor diaphragm. The building suffered relatively minor damage with the most significant relating to differential settlement of the shallow foundation system.

Cracking of the walls was limited to the base of the cantilever concrete shear wall panels in the predetermined plastic hinge zone. Well detailed confined regions exhibited no spalling in the cracked regions and cracks were well distributed over the potential hinge zone. Ducted panel splices are used at panel joints throughout the building and no instances of joint failure or damage have been identified. In general the building has performed very well and has been occupied throughout the earthquake sequence with the exception of mandatory downtime enforced by the initial CBD red zoning. Whilst repairs are necessary, the damage is considered minimal with respect to the building seismic performance. Some of the key features of the West Fitzroy Apartments building which contributed to its successful performance include well distributed shear walls, well detailed yielding regions with confinement steel, confined ducted splice connections and a well tied foundation system. Floor-to-wall connections utilised proprietary TAC20 seating systems (Fig. 3a), and no failure of this system has occurred.

Case study 2: Statistics New Zealand

The Statistics New Zealand building is a 4000m^2 three-storey tilt-up concrete office building located within the Christchurch CBD. The structure is constructed primarily of precast concrete tilt-up shear wall panels and a proprietary hollow-core flooring system with in-situ concrete floor diaphragm. Although the building performed well in terms of life safety objectives, building damage meant that the building could not be occupied after the February 22nd, 2011 earthquake. Damage included cracked shear walls with damaged reinforcing steel, liquefaction-induced settlement and extensive damage to non-structural elements.

Following the earthquakes, an economically viable repair methodology was implemented. New screw piles and enlarged foundation beams were constructed, and one corner of the building was raised 60mm back to level using hydraulic jacks. New in-situ reinforced concrete shear walls were poured adjacent to the existing ones (Fig. 2a), and significant cracks were repaired using epoxy injection in redundant walls. The floor diaphragm was strengthened using steel bracing to the underside of the floors (Fig. 2b), and new roof bracing was added. This bracing also formed part of the temporary works. Seismic bracing of all suspended services was upgraded from the original construction. Not only was the repair work able to reinstate the building substantially to its pre-earthquake condition, but it was also sufficient to upgrade the building to 100% of New Building Standard.

Figure 2. Repair and upgrade of Statistics New Zealand: a) New in situ reinforced concrete shear walls; b) Diaphragm strengthening using steel bracing to underside of floors

Case study 3: St James Court

The St James Court building in Gloucester Street, central Christchurch is an 1100m^2 three-storey tilt-up concrete building with commercial and residential units. The structure is constructed of precast concrete tilt-up shear wall panels and a proprietary hollow-core flooring system with in-situ concrete floor diaphragm. In particular, floor-to-wall connections consisted of 20mm rebates in the panels with panel reinforcing tied into the floor topping (Fig. 3b). This detail has historically been used for buildings with very low drift demands. The building performed well in the earthquakes with only superficial damage occurring. No significant repairs have been required. Particular inspection of the floor-to-wall connections revealed that these connections have not exhibited any evidence of movement between the connected elements.

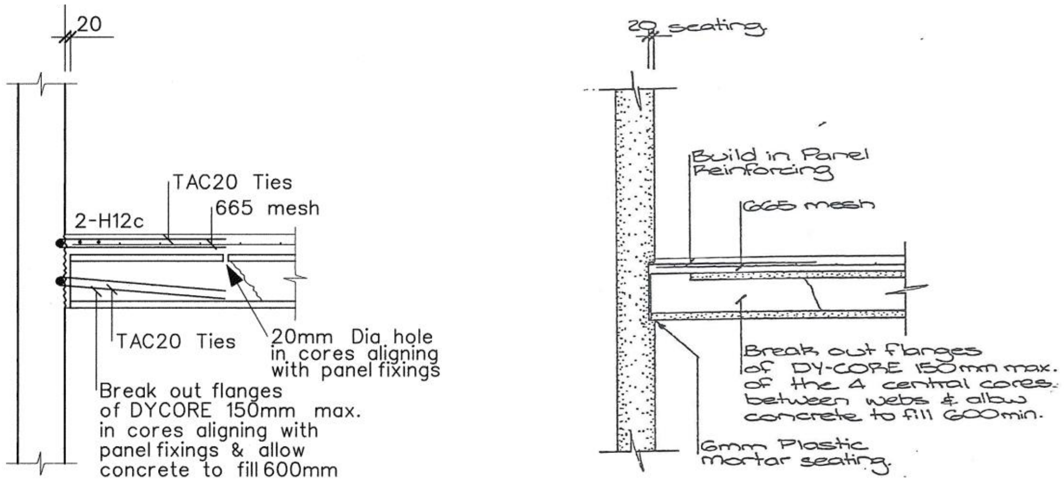

Figure 3. Common existing floor-to-panel connections: a) Proprietary TAC20 seating system; b) Rebate in panel with panel reinforcing tied in to topping

Case study 4: Single-level tilt-up warehouse building in Hillsborough

An example of the superior behaviour of well designed and constructed ductile panel connections is demonstrated in a single-level tilt-up warehouse building in Hillsborough in Southeast Christchurch. This was located in an area subjected to some of the highest accelerations during the February 22nd, 2011 earthquake. Accelerations at the nearby Heathcote Valley School recording station were measured at 1.41g horizontally, and 2.21g vertically (Bradley & Cubrinovski, 2011). The building considered consists of a 93m by 64m steel portal frame warehouse with low-height concrete walls on three sides. Along one boundary parallel to the portal frames, there is a 12.3m high, 175mm thick tilt-up wall. This wall is connected to ductile cantilevered end-wall columns using ductile cast-in anchors which are fully integrated with the panel reinforcing. The wall-and-column system forms the main lateral load-resisting system in the direction perpendicular to the portal frames, supporting the full 93m length of the roof.

Despite the extremely high accelerations, the majority of the panel connections performed well and did not result in the collapse of any tilt-up panels (Fig. 4). The building was able to be reoccupied within days following some minor remedial works.

Figure 4. Example of ductile cast-in anchors as part of the main lateral load-resisting system demonstrating good performance under seismic actions. Inset: spalled vermiculite fire-rating material showing evidence of movement of the connection under seismic actions

BOLTED TILT-UP PANEL CONNECTIONS

The DEE guidelines identify panel connections as a ‘generic building issue’ for single- and multi-level tilt-up buildings. The guidelines note that connection details often use stiff, brittle connections that do not allow for shrinkage or thermal actions, form cracks in the vicinity, and result in a non-ductile connection which is prone to failure in the event of movement.

Panel connections have long been an area of interest in several previous North American earthquakes, including Anchorage 1964, San Fernando 1971, Whittier Narrows 1987, Loma Prieta 1989 and Northridge 1994. (Henry and Ingham, 2011; Hamburger et al., 1988; Adham et al., 1990; Shepherd et al., 1990; Adham et al., 1996). In most cases, changes in connection design provisions resulted from each of these events, with a view to improving strength and ductility of panel connections for face loading. Following the 1994 Northridge Earthquake, Restrepo et al. (1996) carried out research at the University of Canterbury on embedded connections in thin sections. This work formed the basis for commercial development of ductile panel connections which are fully integrated with the panel reinforcing. Such connections have been used in many recent single-level tilt-up buildings; however connections with less ductility are still commonly encountered, particularly in older buildings.

Behaviour of bolted connections

There have been several instances of panels failing at anchorages (Henry and Ingham, 2011; Clifton et al., 2011). Poor performance appears to have been associated with expansion anchors (Fig. 5a & b) and older single-bolt ‘clip-plate’ connections, although no major panel failures are known to have been caused by the latter. Bolted panel connections which incorporate ductile cast-in inserts (Fig. 5c) have typically displayed superior performance. Where these have been installed correctly, no failures have been observed as a result of seismic damage. In some cases, connection configuration has caused less desirable behaviour, particularly connections with no allowance for movement arising from drying shrinkage, thermal actions, differential settlement and prying action on bolts to panels.

Figure 5. Examples of performance of bolted panel connections: a, b) Pullout of expansion anchor; c) Ductile cast-in anchor showing surface panel spalling

Design of new tilt-up panels, as well as repair and retrofit of existing tilt-up panels, should have a three-fold aim with respect to panel connections:

- A strength hierarchy for face loads, whereby the panel connections are stronger than the panels themselves – this ensures that damage occurs along yield lines, rather than being focussed at single connection points;
- Ductile panel connections, for example by utilising yielding elements within the concrete or alternatively by having cast-in inserts which are able to engage panel reinforcing steel on pull-out, to prevent sudden failure if the connections are overloaded;
- Adequate allowance for movement arising from shrinkage and thermal effects, to prevent pre-earthquake weakening of the connection.

Where these have been implemented in design of tilt-up panels, the connections have generally performed well in the earthquakes. An example of the superior behaviour of well designed and constructed ductile panel connections is demonstrated in Case study 4 above. Whilst repair and retrofit of existing panels may seem straightforward, the objectives above should be borne in mind to minimise damage under future earthquakes and ensure that the panel remains firmly connected to the main structure.

PANEL REINFORCING

One of the ‘generic building issues’ noted for both single- and multi-level tilt-up buildings in the DEE guidelines is the presence of hard-drawn wire mesh reinforcement. The DEE guidelines state that the mesh has very low ductility, to the extent that a crack in the panel may be sufficient to fracture the mesh and cause the panel to fail dramatically under face loading. However it should be noted that the Canterbury earthquake sequence did not generally cause any failures of this type in tilt-up construction. Where catastrophic collapse of cladding panels did occur, this was initiated by failure of non-ductile panel connections (Henry and Ingham, 2011; Kam et al., 2010; Clifton et al., 2011; Kam et al., 2011). During the September 4 earthquake, many industrial buildings had instances of racking and stock collapsing against cladding panels, yet this was not sufficient to cause panels to collapse to the authors’ knowledge.

Currently, precast panels under face loading are usually designed to the parts and components provisions in NZS1170.5:2004. Under these provisions, the ductility of the part can reduce the seismic load demand on the part substantially. However the definition of ‘ductility of the part’ is vaguely defined, and NZS1170.5 somewhat arbitrarily suggests that a part ductility of 3.0 may be used for precast panels. When such panels have been built using cold-drawn mesh, the situation may arise whereby many buildings with undamaged tilt-up panels are deemed “earthquake prone”. However this appears to be on the basis of an assumed part ductility of 1.0 without an investigation of the actual available ductility. As such, there is a need to devise a methodology to assess the actual ductility capacity of tilt-up panels.

Assessment of panel ductility

By adopting a first-principles approach in conjunction with the guidelines in NZSEE (2006) for the assessment of existing buildings, the actual available ductility in a panel can be estimated to give values which better explain observed behaviour. Such an approach may involve the following steps:

1. Perform a moment-curvature analysis of a section of panel using a strength reduction factor of 1.0, and probable strengths or actual material properties where available. Cold-drawn mesh typically has an ultimate strength considerably higher than the stated tensile strength, and can often achieve a fracture strain of up to 3%.

2. From the moment-curvature analysis, establish the moment and curvature at yield of the reinforcing steel, and at ultimate. Priestley et al. (2007) recommend limiting steel strains to $0.6\varepsilon_{su}$, where ε_{su} is the steel strain at maximum stress. However since cold-drawn mesh does not typically exhibit strain-hardening, and ε_{su} is essentially the fracture strain, the ultimate moment and curvature may be more appropriately taken at $0.5\varepsilon_{su}$.

3. To determine the plastic rotation, an assumption about the yielding length must be made. Cold-drawn mesh that is not deformed will achieve poor bond with the concrete between cross-wires. As such, the pitch of the mesh can be used as the yielding length. If the weld on the mesh breaks, the yielding length would become longer.

4. The ultimate displacement capacity is obtained using moment-area theorems. An upper bound elastic deflection will tend to provide a conservative estimate of the available displacement ductility. For this step, the panel geometry must be simplified to a one-way strip along the region of highest bending moment demand.

5. The demand moments can be determined using simplified methods, or a finite element analysis of the panel allowing for full geometry, loading and potentially period reduction for very large panels.
6. The likelihood and consequences of mesh rupturing should be considered in the context of factors such as the presence of other reinforcing in the panel, the bending moment distribution on the panel, whether the panel forms part of the main lateral load-resisting system.

Assessment and repair of damaged panels

Panels which have become damaged under face loading need to be assessed with the understanding that the mesh may have yielded and used up some of its strain ductility capacity. In such cases, where axial loads are small, the crack width may be used to estimate a residual strain in the mesh, once again using an assumption of the yielding length. Where cracks appear to indicate high strains in the mesh, it may be necessary to expose the mesh and measure the level of reduced strain capacity.

A common repair technique for cracked cladding panels has been to inject the cracks with epoxy. However this technique may lock in residual strains and experience has shown that cracking under future earthquakes tends to occur along cracks which have been previously repaired. As such, damaged panels repaired in this manner may require replacement, or alternatively, a retrofit to limit face loading to elastic levels only.

Example: single-level tilt-up commercial building

As an illustration of the above analysis, a single-level tilt-up panel was examined. The panel is 120mm thick and reinforced with cold-drawn 663 mesh. It spans 7.4m across, and is 4.4m high. The panel is undamaged, and had previously been assessed as being 27% of New Building Standard using the DEE guidelines, due to the presence of the cold-drawn mesh. Considering the actual ductility capacity of the panel and allowing for probable material strengths, a moment curvature analysis of a critical section indicated that a curvature ductility of 3.9 could be obtained before the steel strain reached 1.5% ($0.5\varepsilon_{sf}$).

Taking a strip of panel in the region of highest moment, a higher estimate of the elastic deflection was obtained by allowing for cracked section properties, and ignoring two-way action. Concentrating all the inelastic action at one point, on a line of mesh between cross-wires, a displacement ductility of 3.0 was obtained. Using this value, the panel could be considered closer to 90% of New Building Standard.

To put observed panel damage into context, it is worth noting that, for a yielding length equal to the pitch of the mesh (150mm), the crack width required to cause $0.5\varepsilon_{su}$ is of the order of 5mm. This fact helps to illustrate why no failures of tilt-up panels under face loading have been observed.

OTHER 'GENERIC BUILDING ISSUES'

Grouted tilt-up panel connections

Several issues with the grouted duct system have been highlighted as a result of the earthquake. Ducted splices have been identified in the DEE guidelines as a potential 'generic building issue' noting instances of un-grouted ducts, over-confinement of the reinforcing bar causing high strains over the panel joint zone, and loss of cover concrete due to limited or no confinement in the duct region.

Multi-storey buildings which incorporate tilt-up construction are usually built from taller panels spanning several floors, rather than multiple single-level elements spliced together. This reduces the need for grouted splice joints. As there are usually multiple panels in plan, the stresses in horizontal ducted splices are relatively low compared with other forms of construction. In newer multi-storey tilt-up buildings, ducted splices at ground level are often designed to the overstrength of the plastic hinge region above, and as such, damage to the ducted splices can be effectively controlled.

No grouted duct failures in tilt-up buildings have occurred to the authors' knowledge. In general, moment-resisting connections at the base of wall panels have performed well, with only minor cracking and spalling damage (Henry & Ingham, 2011). SESOC (2012) have issued specific design guidance for the detailing of confinement steel around panel splices, and debonding of reinforcing at panel joints to limit strains. Further emphasis is required around the quality control of the precast panel fabrication, and the erection and filling of ducted splice connections.

Floor-to-panel connections

The DEE guidelines identify seating for precast floor systems as a potential 'generic building issue' for multi-storey tilt-up buildings, noting that some panels have small rebates for seating of the precast units, with panel reinforcing tied into the topping. The guidelines note that units may lose seating and delaminate from the toppings, while other proprietary connection details may initiate a break in flooring units away from the supports. To the authors' knowledge, no such failures were identified in tilt-up buildings in the Canterbury Earthquakes, and these connection details often performed better than those with more 'conventional' seating details in other forms of construction. Case studies 1, 2 and 3 above identify instances of satisfactory behaviour.

Panel slenderness

Panel slenderness is identified as a 'generic building issue' for single-level tilt-up buildings. The DEE guidelines note that panels have the possibility of buckling in diagonal compression. However, Henry & Ingham (2011) note that this was not observed in any buildings during the Canterbury Earthquakes. Panel slenderness ratios do not often exceed 60 in single-level tilt-up buildings. Testing and research summarised by Beattie (2007) has demonstrated that typical examples of warehouse panels, with this slenderness and loaded in plane, are unlikely to experience Euler buckling or Vlasov/Timoshenko lateral-torsional buckling.

CONCLUSIONS

Many buildings in Christchurch have been built incorporating tilt-up construction. Over the last two and a half years, these buildings have been subjected to the Canterbury Earthquake sequence and in the majority of cases have performed well. Guidance on assessment of existing earthquake-affected buildings (EAG, 2012) and the design of new buildings (SESOC, 2012) contain some recommendations pertinent to buildings incorporating tilt-up construction. In addition, the following conclusions relate specifically to issues regarding tilt-up buildings:

- Ductile anchorage systems and cast-in bolted inserts which are fully integrated with the panel reinforcing have exhibited superior performance to other connections such as post-fixed expansion anchors. Where these have been correctly installed, no failures are known to have occurred. The three-fold aim of bolted panel connections is that they are stronger than the panel itself, ductile, and that they allow for expected panel movements. These aims should be borne in mind for repairs and retrofits of existing panels

- By adopting a rational first-principles approach to assessing the ductility capacity of face-loaded panels reinforced with cold-drawn mesh, a realistic estimate of the panel's % New Building Standard can be obtained. Such an approach yields answers that are consistent with the observation that no panels have failed catastrophically under face loading initiating due to fracture of mesh reinforcement

- An appropriate level of quality control is required from contractors to ensure that buildings are constructed according to the drawings and specification. In most cases, contractually this responsibility belongs to the contractor, and it is imperative that this responsibility is actively practised.

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