The Newcastle Workers Club in retrospect - what can we still learn?

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Abstract

The collapse of the Newcastle Workers Club (NWC) during the earthquake in 1989 eventually led, in 1991, to an independent investigation of the causes likely to have been involved. The technical details have been described previously. A broader perspective is taken here, noting that in many failures there are non-technical reasons that ultimately lead to technical failures. Such investigations often require a degree of mental dexterity or lateral thinking, sometimes characterized as being able to 'think outside the square'. Herein three cases are examined, concentrating on the NWC. The over-riding outcome is that checking of design, drawings, documentation and the actual construction process often is, with hindsight, inadequate and mostly likely would have increased the likelihood of achieving adequate structural safety and performance. Importantly, such checking should be done by engineers with considerable practical experience.

1. Introduction

One of the consequences of the 1989 earthquake that occurred during the morning of 28 December, 1989 in the Newcastle region was the partial collapse the Newcastle Workers Club, a 17 year-old reinforced concrete and brick masonry building fronting the south side of the east-west-oriented King Street in the Newcastle inner city zone. The building was severely damaged mainly along its western side. There the reinforced columns are considered to have failed progressively under the loads imposed as a result of structural inertia and ground shaking. The technical aspects of the failure mechanisms that most likely were involved have been investigated in considerable detail during 1992. However for legal reasons the outcomes were not made available publicly until much later, first at the 2009 Earthquake Engineering Conference (Melchers 2009) and subsequently in the Australian Journal of Structural Engineering (Melchers 2011).

The coronial inquest into the death of the 9 people killed by the collapse of the car-park floor slabs on the western side of the building spent some time on possible causes for the collapse but was unable to present conclusive findings. The inquest did note the absence of significant numbers of bricks inside the building as compared with the large quantity outside, indicating that the upper storey brick walls had not collapse through oscillation as would be expected under the predominantly east-west ground shaking at the site. In a sense this was a warning that all was not as would be expected from conventional notions of building response under earthquake ground motions. Typically these are predominantly horizontal shaking, with, generally, lesser vertical shaking. At the time there was considerable speculation that the presence of old mine-workings under the city had contributed to the collapse, but that was eventually shown to be inconsistent with the generally good behaviour of neighbouring and other buildings in the area and elsewhere, even in undermined areas. During that time also, a number of buildings were demolished, usually on the basis that they were unsafe, even though it was clear from simple observation that considerable effort was required for many to demolish them. Other cases, such as the collapse of some shop awnings in Beaumont Street, were soon identified as the result of their retrofitted support systems (to deal with the removal of the original kerb-side support posts), being unsuited to cope with earthquake ground motions. It is noted that at that time and prior, there was no requirement (in Australian standards) for buildings and other structures to be designed to deal with earthquake ground motion. The city and surrounds were rated zero. Standing back already in early 1990 it was apparent that despite the considerable damage to the city's infrastructure and buildings, it was remarkable that there were so few cases of building collapse or serious structural damage as would normally be associated with an earthquake (Conference 1990). The case that stood out was the Newcastle Workers Club (NWC).

As recorded elsewhere (Melchers 2011), early in 1992 the author accepted an invitation by solicitors acting for an unnamed client to undertake an independent assessment of the reason(s) for the collapse of the NWC. Access was provided to professional photographs taken for the NSW Police Service and for the Newcastle Herald, to architectural, structural design and construction drawings and to a consulting engineering report prepared for the coronial inquest. As noted earlier, even at that time it was clear "... that investigation of a failure may require a circular process that involves knowing what to look for before that is likely to be observed, and also that preconceived ideas about the likely failure mode ... should be put aside when looking at the available evidence." (Melchers 2011).

2. Overviews

The first action on receiving the drawings and the photographs was simply one of immersing oneself in them. There were several hundred photographs, many showing very similar things, from slightly different angles and the 'trick' was to try to understand what it was one was actually looking at, and what that might mean. But if you do not know what to look for you will not see it, usually. So the photos were gone through many times as the investigation went on. Similarly, working through the drawings initially did not appear to show anything unusual, other than an early observation that the footings on the west side (the collapsed side) were different to those on the east side of the building, but were all very similar in size. The other (east) side of the building was complicated by the presence of the (older) neighbouring part of the club. That could have had an effect, but there was no visual evidence of any load transfer between the buildings. Also, it was noted that not the whole of the west side had collapsed, even though that facade appeared highly repetitive (Fig. 1).

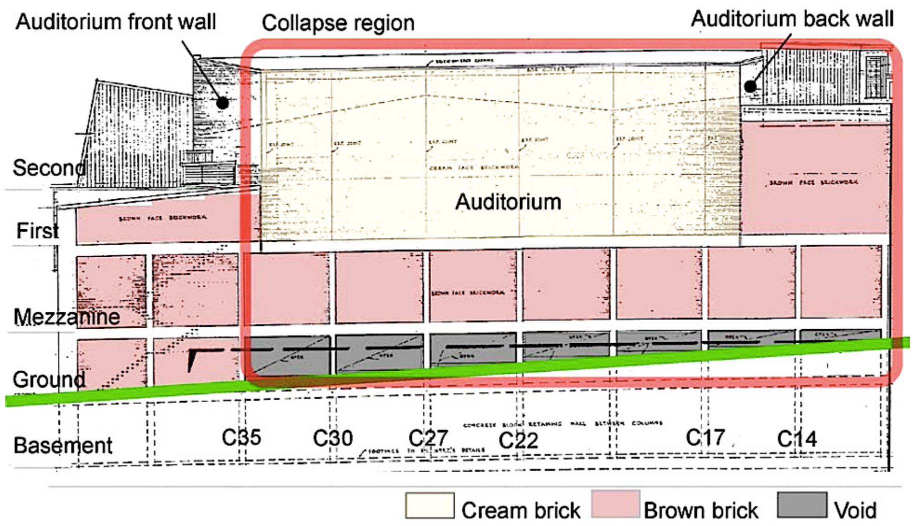


Figure 1. Diagram of the west wall of the NWC based on design drawings, showing column and slab layout and the collapse region (redrawn from Melchers 2011).

A check on the ultimate capacity of one of the number of apparently identical columns, considered typical for the west wall columns showed its capacity to be adequate, even with a nominal allowance for bending caused by the presence of the slabs, and using simple tributary areas and crowd loading (Melchers 2011). By comparison, the interior columns were much larger, consistent with their larger floor tributary areas. These columns were larger in cross-section than the west wall columns and were shown on the drawings as having significantly more reinforcement bars. As evidenced by the photographs, these columns had remained intact, including the large drop-panels. However, the slabs had separated from the drop-panels during the collapse (Fig. 2). There were suggestions at that time that this indicated shear or similar failure in the region of the drop-panels, but close examination of the photos shows the slabs had pulled away or that the reinforcement, located at the top of the slabs, had been ripped out by the sudden downward movement of the slabs (Fig. 2). This was consistent with the collapse of (part of) the west side of the building. At this stage there appeared to be no obvious cause for the collapse.

Figure 2. Reinforcement exposed at the drop panels (note column and drop panel at back).

3. What happened to the bricks?

The photographs tendered in the coronial inquest showed clearly that very few bricks had fallen into the building onto the slabs from what were cavity brick infill panels. They did show many bricks in the laneway next to the west wall, with a considerable number scattered over the mobile crane that was parked there. Taken together, this is inconsistent with the notion of unsupported swaying brick walls collapsing under earthquake ground motion with the expectation that this would leave (roughly) equal numbers of bricks on each side. It also raised the question about the impact and damage that would be caused by even a single brick falling say 2-3 storeys from the upper, most swaying part of an unsupported skin of brickwork. It would be considerable. However, as noted in the report and subsequent paper (Melchers 2011), enquiries with the owners of the crane, Hunter Water Corporation (HWC), revealed that there was little damage to the crane and in particular its relatively thin steel (and therefore relatively weak) cabin roof and its windows. Their observations were supported by a close examination of the photographs of the crane (Fig. 3) The only possible conclusion was that the bricks had not fallen 2-3 storeys. That, together with the relatively large number of bricks on the crane and in the laneway indicated a different mode of failure altogether. But what?

Figure 3. Close-up view of crane showing lack of damage from bricks to cab and to cab windows (part of photo courtesy of NSW Police Service).

Making contact with HWC turned out to be fortuitous. It revealed two HWC operatives had been on-site servicing the crane at the time of the earthquake. One of them recalled what he had seen, even if only in an instant. That was that the west wall had collapsed by what he described as 'horizontal folding' of the wall, about half way up. Both commented on the considerable loss of bricks from the wall, sufficient to drag one of them under the crane, and sufficient for the other to consider his workmate had been killed. The dust created by the collapse then obliterating all vision. But what was reported as seen in that instant was sufficient to focus attention back on the capacity of the part of the west wall that had 'folded horizontally'. This did not immediately fit in with the usual expectations for structural response under earthquake ground-shaking, i.e. mainly lateral movement of each storey, typically with greater amplitude for the higher storeys.

Under the conventionally-accepted response of framed structures to ground-shaking it would be expected that the upper and lower ends of columns framed into beams and slabs would be prone to plastic hinge-type failure. The rest of the column(s) would not be under the same moment conditions and therefor unlikely to be subject to plastic rotation failure. Alternatively, for strong-column-weak-beam designs, the plastic hinges would be in the beams (and slabs). If either of these were the mode of failure there should be whole (or nearly whole) lengths of columns that should have survived. And the photographs should show pieces of column - that is, the pieces between the upper and lower plastic hinges. But this is not what the photos showed. There was no evidence of pieces of column, even as the sequence of photographs recorded the progression of the clean-up of the site. So what had happened?

4. Re-thinking the failure mode

While columns under earthquake loading conditions can be expected to show plastic hinges at the top and bottom or in the immediately adjacent beams, columns subject to axial loads and to bending can fail in other ways as well. The usual design approach is to design the columns to 'fail' in the relatively ductile bending mode, that is, in the high M, low P part of the usual column interaction diagram (Fig. 4) with the capacity and ductility of the reinforcement bars in tension governing behaviour. This is consistent with the classical weak-beam, strong-column approach and underlies the conventional thinking and philosophy for the design of rigid frames (Ferguson 1962). The alternative, for the low M, high P part of the interaction diagram, is failure by concrete compression failure, perhaps with reinforcement buckling. This implies brittle failure, including of the concrete. Typically this is explosive and destructive of the concrete. If it occurs little would be left of the column, apart from reinforcing bars. In practice such a mode of column failure would have major consequences for structural safety, and is completely inconsistent with the long-established notion that structures should have a degree of ductility and deformation capacity as they approach ultimate capacity (Ferguson 1962). Explosive, brittle failure is not a subject uppermost in the minds of most structural engineers when they think about the structural behaviour of reinforced columns, but for the NWC, the question gradually became whether such a mode of failure was possible for the west wall columns? Could this be possible, given that the (M-P) combinations at ultimate load as estimated for these columns appeared to be well inside the capacity curve for the columns with their specified size and with the specified reinforcement (Fig. 4)?

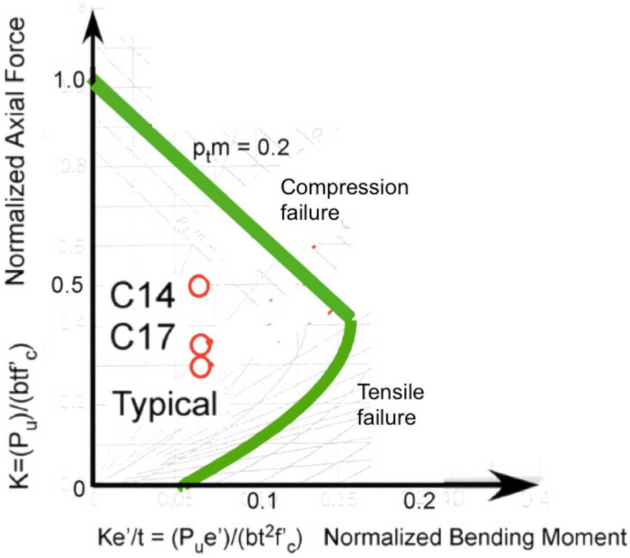
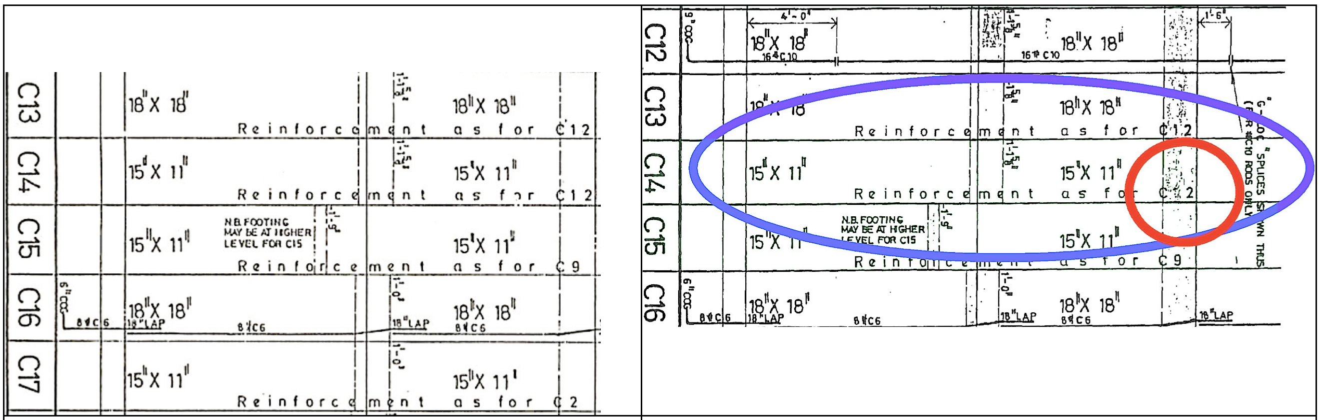


Figure 4. Interaction diagram showing estimated M,P values for selected columns and capacity curve for typical west wall column size and reinforcement.

The preliminary analysis of the capacity of the west wall columns noted they were all of the same cross-sectional size and had found they were well within acceptable bounds for the typical floor tributary areas and crowd loading and with nominal bending action from the floors that framed into these columns. But was this assessment valid for all west side columns?

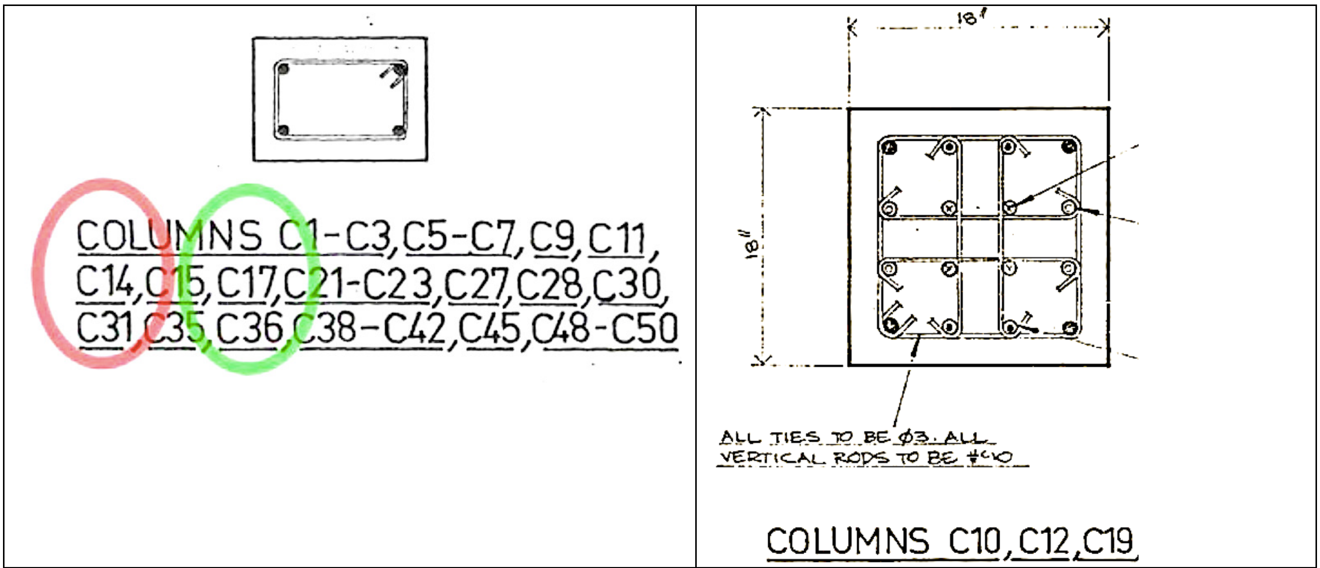
A reworking of the imposed loads revealed that there was a difference, namely at the interior end wall of the auditorium (south wall of the auditorium). In that region the auditorium wall transferred some part of its very considerable load mainly to west wall column C14 and also some to its northern neighbour C17 (Fig. 1). According to the column reinforcement schedule shown on the design drawings the overall dimensions of these two adjacent columns were the same as all other columns in the west wall. And, as noted, the footing sizes were essentially the same for every column along the west wall, including C14 and C17. In other words, superficially at least, there was nothing obvious to distinguish these columns from any of the others in the west wall (Fig. 1).

Re-inspection of the Table with the column reinforcement schedule on the design drawings showed that the west wall columns were all marked as having the same size 15 x 11 inch (375 x 275 mm) and had just 4 reinforcing bars, typically referred to ‘as for C2’ or ‘as for C9’, both of which were essentially the same. However, close reading of the schedule showed that columns C14 and C17 were to have reinforcement ‘as for C12’, where C12 was one of the interior, much larger columns, measuring 18 x 18 inch (450 x 450 mm) and with 16 reinforcement bars (Fig. 5). How these 16 bars were supposed to fit inside the smaller (15 x 11 inch) exterior columns was not clear - the only cross-sections shown on the drawings did not refer specifically to columns C14 and C17 but included them with other west wall columns (Fig. 6). And, as noted earlier, the footing sizes for all the west wall columns, including C14 and C17 were essentially the same, and overall much smaller than the interior column pads, such as for C12.



(a) (b)

Figure 5. (a) Part of reinforcement schedule and cross-section from design drawings, (b) ditto from construction drawings, showing unclear notation (circled).



(a) (b)

Figure 6. Cross-sections for columns on construction drawings (a) west wall columns including C14 and C17, (b) interior columns.

The drawings available to that point had been the design drawings. The question now arose whether the drawings available for the investigation represented what had happened in practice - i.e. on-site. Had the inconsistencies been found during construction? As noted, by the time the investigation was under way the building had largely been demolished and it was not feasible to ascertain, for example, the footing sizes, and certainly not the sizes of the columns. The only course of action was to examine the drawings the builder had used for construction, and these were subpoenaed from the builder. In that process it became apparent that there had been a number of design changes of the west wall columns.

The column reinforcement schedule on the construction drawings for columns C14 and C17 are shown in Fig. 5b and might be compared with the details on the design drawings (Fig. 5a). On the construction drawing in question the notation for C14 is not particularly clear. Spacing of the text suggests it might have been meant to read ‘C12’, but it reads more like ‘as C 2’ - had the ‘1’ simply been removed? Why? As noted, column C2 was one of the standard west wall columns with just 4 reinforcement bars, as also shown on the construction drawings (Fig. 5b). For column C17 it is clearly 'as for C2' in both cases.

Evidently, there are important inconsistencies both in the design drawings and in the construction drawings. The reasons for this cannot now (or even in 1991 after the event) be ascertained. Unfortunately, experience over many years suggests that this case of inconsistencies is not an isolated incident in the cause of building collapses (Feld 1965). More unfortunate, perhaps, is that the inconsistencies apparently were not detected, or worse, if detected were not reported.

As far as the investigation was concerned, once the inconsistencies between the reinforcement schedules, the sizes of the column cross-sections and the sizes of the corresponding footings were recognized, it was relatively straight-forward to develop the likely sequence of a very rapid 'progressive collapse' of most of the west wall, commencing at column C14, and moving both towards the back of the building and forward until it hit the stiffer panels at column C35 (Melchers, 2011). That also showed consistency with various other observations made immediately after the collapse, such as the survival of the two panels closest to King Street, which showed only some relatively minor cracking between the brick infill panels and the reinforced concrete frame.

5. The structural design

At the time the building was designed there was, as noted, no requirement in Australian Standards to allow for seismic loading. Only structural designs to cope with dead, live and wind loadings were required. Assuming that all wind loads were transferred to the interior columns through diaphragm action of the reinforced concrete slabs, it is of interest, now, to consider the capacity of the typical west wall columns. The estimates made during the investigation show that in the period prior to the collapse, the critical columns on the west side of the building would have been very close to their ultimate strength had full crowd loading been imposed on all public areas, using the value of such loads as specified at the time (Melchers 2011). The critical condition for these critical west wall columns was that of compression failure under high axial load and some bending moment. From full scale column testing experimental work and from theoretical analyses this is known to offer little or no warning (Ferguson 1962), unlike the more usual design case that tries to stay within the ductile zone of column behaviour - this corresponds to a column failure mode that provides warning through a gradual increase in deformation afforded by the ductility of that mode. On the other hand, the typical west wall columns were close to balanced design and thus less likely to have suffered sudden compression failure. Again taken together this indicates that for the cross-sectional column sizes as shown on the drawings C14, and to a lesser extent C17, would have been very much under-designed. Overall, the indications are that for reasons that cannot now be ascertained the higher loading on these columns, even without earthquake loading, was not consistent with the structural behaviour that could reasonably have been expected.

6. Observations

While superficially the NWC failure was the result of the occurrence of the Newcastle earthquake, the analysis of the evidence suggests strongly that it is more realistic to consider the earthquake ground motions as simply "the straw that broke the camel's back".

In this context it is observed that when designed in compliance with design codes and also properly executed most structures have very high levels of safety against collapse (Brown et al. 2008). Most structures can tolerate modest errors in design and construction, and others can tolerate more serious errors simply because in the normal lifetime of the structure the nominal code-specified design loads are almost never fully applied. Thus, the fact that errors in the critical columns of the NWC were not revealed throughout the 17 years before the earthquake is likely to be simply because the full design-code-specified crowd-loading, over all the public areas, had never been applied. It took an unusual loading condition, one for which the building as a whole, and the critical west wall columns in particular, had not been designed, for the deficiencies to be revealed. Unfortunately, this is not the first time such fortuitous history has occurred, with eventual tragic circumstances (Feld 1965).

In the case of the NWC, despite what we now know about the deficiencies, it is reasonably to assume that it is likely that its service life would have been much longer than it was, had it not been for the extra loading and hence stresses from the earthquake. This says something about the overall conservativeness of the underlying safety margins in conventional structural design codes, and may well explain why, in the overall population of structures, significant failure is so rare, despite the fact that errors in design and in construction are much less rare, and, for smaller errors, even common. This is a virtue, and one on which the structural engineering profession has much relied, perhaps not always wittingly. Importantly, it should not be abused, such as cutting corners in design and analysis, or, in design, dismissing what might appear to be relatively unlikely events. One of the reminders the NWC case provides is that there can be a significant gap between design intent and final execution.

Well outside the brief for the original NWC investigation (as initially constrained by legal limitations), but interesting nevertheless, is whether the original design, and in particularly the drawings, had been checked in-house before going out for construction. The comment that there had been several revisions in the design of the west wall columns (Melchers 2009) could be read as indicating there were issues that needed to be resolved. Whether these were matters of detail or were more substantial obviously cannot now be determined. But even if the changes themselves were minor, the question arises whether the drawings that were issued for construction had been fully checked by an engineer independent from the design team. And even then, the fact that all columns on the west wall were shown to be of the same size, with similar footing sizes, could easily have led to an oversight in checking the reinforcements in C14 and C17. Overall, perhaps the most important question is why those columns were sized, on the design drawings, the same as the others, considering the much greater loading any reasonable load path and load distribution would suggest they were meant to support and hence the greater amount of reinforcement they had to contain.

The sizes of the footings on the west wall adds further dimension. As noted, they all appear to be similar in size, including those for C14 and C17. This would appear to be inconsistent with the column loadings and the reinforcement for the columns. No details of computations for these footings were in the set of extensive documents made available for the 1991 investigation. Eventually those held by Council were obtained. The only footing design information consisted of a single sheet of paper with what was interpreted in 1991 as a simple computation allowing only for vertical forces. These were those for a typical west wall column. It can only be presumed that Council 'checked' or overviewed these computations.

From the records made available for the investigation, it was unclear whether the on-site construction team noticed the unclear notation for column C14 and if so whether they alerted construction management. This aspect also cannot now be ascertained and was not part of the 1991 investigation, but from a broader perspective it is another interesting question.

Similarly, there was no evidence during the investigation whether 'proof-checking' or similar had been performed for the final design, and by whom. In the eventual out-of-court legal settlement Newcastle City Council was reported as having taken responsibility for 'checking/approving' the design. Although not part of the investigation, it is unclear whether there was compliance checking of what was actually constructed against the design drawings. Although such checking appears largely to have gone 'out of fashion' in modern Australian construction, it is likely that at the time of construction of the HWC it should have applied. If so, would it have detected the type of inconsistency in drawings, particularly such as for the critical column C14?

In this context it is relevant that most European practice is still for structural design engineers to have on-site periodic (if not continuous) oversight of the construction process, usually by people with much experience in the construction industry. Discussion with them indicates that much of what is observed on-site is mundane and largely relates to misfits between drawings and what is actually feasible. As might be expected, only rarely are major problems, such as misalignments, detected and this, perhaps, adds a dimension of relaxation that is not necessarily warranted. As in Australia, local council 'approval' does not imply checking of the designs, as once was the case, using either Council staff or independent engineers. Concern has been expressed that often any checks are not carried out by engineers with sufficient practical experience. This may explain some quite recent failures (BBC News 2012, 2019; ABC News 2019). In these, changes in the designs also were involved.

7. A completely different structure

A completely different structure investigated some years ago has some parallels with the NWC, even though earthquake loading was not involved at all. In brief, a prestige multi-storey apartment building, still within the guarantee period, was subject to a restriction on the rate of sewage discharge to the local sewerage authority system. Peak flows had to be ameliorated. To do this the developer had installed a large holding tank within the underground car-park to even out the flow over each 24 hour period. Superficially this appeared to work, except that there were periodic blockages and overflows of sewage into the surrounding and lower car-park spaces, always with sewage leaking out through the cast iron-framed concrete covers being lifted upwards by water (i.e. sewage) pressure. This was, of course, unacceptable to the owners and the body corporate.

To help throw some light on this problem and also to assess any potential structural safety implications an on-site inspection was carried out. This suggested that the structure was unlikely to be at risk unless the situation was not rectified in a timely manner. It also found that the interim tank system appeared to behave like a septic tank, with inflow at one end and pumped outflow at the other, with a considerable build-up of solids that caused both nuisance and the occasional blockages and overflows. It was clear that the tank design and operation were unsuitable for the intended purpose. After the owners commenced legal action the developer agreed to modify the system by installing an external holding tank and diverting flows to it and from it to the city sewerage system. (The external tank eventually was unnecessary as the sewerage authority agreed to lift its restrictions before its construction). Throughout this process the question why the tank had been built (and presumably designed) with inlet and outlets at opposite ends was not explored. All emphasis was on rectifying the problem.

The reason for the poor design of the holding tank did not become clear until the developers took legal action against the subcontractor who had installed that system. Closer investigations (inside the tank, with associated H_2S gas and ventilation issues) showed that the contractor had installed pipework and electrical wiring for the pumps and for monitoring that would not have been necessary had the inlet and outlets been close together, as would have been the normal design. Almost as an afterthought the pipeline and the wiring drawings were compared with the structural drawings - these showed the openings in the concrete slab for installation and maintenance of the pumps. Although comparison was complicated because of the different referencing systems, it soon became clear that the pump access openings on the structural drawings were at the opposite end of the tank compared to the pipework on the mechanical/electrical drawings. How this came to be was not determined at the time. However, it can be considered the direct cause of the odd arrangement of pipework and electrical wring.

There was some suggestion that the location of the pump openings had been moved away from the area most trafficked in the car-park floor immediately above, so as to reduce inconvenience to apartment owners whenever the tank had to be pumped-out.

As for the NWC case, the inconsistency should have been picked up in cross-checking of the various drawings. It could have been missed because of the highly regular layout of the car-park at this level. In the present case it was clear that the pipework the contractor had installed within the inside and around the tank had been to overcome the mis-match between the incoming pipework relative to the tank orientation and similarly for the outgoing pipework, and for electrical wiring for the pumps. Irrespective of the precise cause, the pipework and the electrical wiring as installed can only be considered unusual. It suggests the contractor had noted the problem but did what many plumbers do - did a ‘work-around’, not realizing the implications this would have for the functioning of the tank system. This case indicates, again, issues with design, documentation and construction checking.

8. Another case

In this case there was concern about the corrosion of the roof space for a swimming complex and the potential structural safety issues. Although it started as a structural deterioration problem, it ended up, after much circular work, to be that of a problem with the air-conditioning system as installed, following cost-cutting decisions that did not appear to have fully considered the implications. And again, evidence of checking and cross-checking was missing.

In brief, a significant indoor swimming complex had been designed to have air-conditioning with moist, used air being drawn, roughly at pool level, using standard ventilation equipment, to the outside from the main hall space. The roof of the building was designed as steel decking over a plenum with a vapour barrier between the underside of the (insulated) steel roof sheeting, supported on painted steel roof trusses, with the suspended ceiling carrying also the vapour barrier and the suspended light-weight tile ceiling. Apart from the suspended ceiling (with vapour barrier) this can be considered to be reasonably standard design for indoor pools. More usually the underside of the roof and the steel trusses would be fully exposed, protected with high quality protective coatings. However, the suspended ceiling was considered architecturally much superior. With the intended provision of a truly moisture-proof vapour barrier the design should have been adequate. And at no point was this original design seriously disputed in subsequent legal arguments.

On-site inspection of the roof space showed that what was reported to be a leak in the roof was actually caused by (unexpected) condensation in the roof space. This space also showed evidence of (rather modest) corrosion of the structural steel trusses and other components in that space, despite their protective coatings. Interestingly, to allow inspection of the roof space a small number of ceilings tiles were removed from underneath. Although not immediately obvious at the time, there was no vapour barrier at that location. Subsequently it was observed that the vapour barrier had not been installed anywhere in the suspended ceiling. It also was observed that the structure as constructed had large (unpowered) vents in the roof. Both observations were at odds with the design as given in the (then) available drawings.

Eventual examination of the correspondence and other documentation showed that sometime during the construction process there were significant changes to the air-conditioning system. It appeared that owing to project cost blow-out the cost of the (expensive) air-condition system had to be reduced. The system in place at the time of inspection was the modified system.

It worked in reverse to what is normally the case - namely it draw in fresh air from the outside at pool level, circulated it through the pool space and then through the roof space and expelled the by now very moist air to the outside through the roof vents. To make this possible had necessitated leaving off the vapour barrier in the ceiling (but not the perforated false ceiling tiles) and also to install exhaust vents on the roof. This mode of operation had the apparently unforeseen effect of bringing in high humidity air into the roof space. In turn this produced a highly corrosive, wet, chloride-enriched, environment in that space, causing the relatively high rates of corrosion of some parts of the roofing system. In this case changes in design, errors in the revised design concept, and inadequate (if any) overview and/or cross-checking of non-structural components, led, ultimately to structural problems.

9. Discussion and conclusion

The investigations of the NWC and the other cases were unlikely to have made much progress if thinking had been confined to the obvious, direct, effects one expects. In all cases the facts simply did not fit what normally would be expected, such as, for the NWC, from ground shaking, or under-mining effects, or quite localized geological conditions, including saturated soils (Conference 1999). In the case of the NWC it was necessary to consider other possibilities, but even the notion that the columns were somehow insufficient did not fit the facts until, moving even further away from the starting point, the notion that one or more of the columns had failed in compression started to make some sense. As noted, column compression failure is almost excluded from conventional design and design thinking, owing to design codes attempting to move designers away - because of the adverse, explosive loss of concrete in compression failure under sustained loading. Even in the usual concrete compression tests such a mode of failure is rarely seen in most (stiff) concrete compression testing machines. For these the loading releases when the specimen fails. But this is not the case under dead-load - for this the loading is sustained and as the specimen fails in compression the loading remains, and the specimen outer surfaces spall off under triaxial stress, leaving s smaller net area and thus increasing stress for the remainder. This scenario explains the lack of sizeable pieces of concrete (in particular pieces of column) in the debris recorded at the site.

Comparing to the slowly growing literature on structural failures, and the reasons for them occurring, shows that there has been and continues to be much emphasis on what happened to the structure and how structural components had failed (e.g. Feld 1965; 1997). A number of publications have considered failures related to changes in design philosophy and new techniques or new ways of doing structural engineering or construction (Sibly & Walker 1977; Stewart & Melchers 1989; Petroski 1993, 2012; Melchers 1994; Brady 2015). Often these types of problems become apparent in practice in the very process of construction. This is one reason for the relatively high rate of construction failures (Feld 1997). The occurrence of human error, the lack or poor performance of checking and oversighting processes largely has been overlooked in these analyses although there is a body of work that has considered them (Stewart & Melchers 1989; Melchers 1994) including in the overall context of structural failure (Stewart & Melchers 1997). Recent examination of failure cases in the UK's CROSS (Confidential Reporting on Structural Safety) scheme has highlighted the importance of such issues as revealed particularly in the more numerous 'near-miss' cases (Wainwight, 2018): the underlying issues ... nearly always come down to ... "communication, competence and regulation". The three cases considered herein may be considered to be examples. Nevertheless the implications for structural engineering (if not engineering more widely) are obvious.

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