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When old meets new: Monitoring load sharing in a concrete building extended upwards

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ABSTRACT: To meet increased demand for office and residential space in developed cities, many sites are being redeveloped. This usually involves the demolition of existing structures and the building new higher and more modern structures. However, restructuring is also possible, and this can potentially lead to significant time and cost savings. The environmental impact of the construction may also be reduced. This study examines the case of Condor Tower, an existing 10 storey building in Perth, Western Australia, to which 18 floors are being added. Efficient design of the restructure requires understanding the load sharing between the existing concrete structure and the new infill structure. Because the redeveloped structure effectively has two vertical load paths, each of which has significantly different shrinkage and creep characteristics, the load sharing behavior is difficult to predict. In order to investigate this behavior, several columns of the concrete frame of the original 10 storey building were strain gauged prior to the installation of infill concrete walls and new floors. Data is being collected from these strain gauges as the additional floors are added to the building. This paper presents some of the results obtained so far from this monitoring, along with some preliminary analysis. These results suggest that the building behaves in a satisfactory manner, and the behavior limits suggested by conventional engineering mechanics envelope the behavior well.

Keywords: Reinforced concrete structures; monitoring; retrofitting.

1 INTRODUCTION

The restructuring and reuse of existing buildings is becoming of increasing interest. Sustainability and green development have become catch-cries of the current age. Demolition of buildings which have significant remaining design life is increasingly being frowned upon. However, as cities develop and grow, and inner city living becomes more fashionable in western countries, the pressure for redevelopment of inner city properties is becoming greater.

Restructuring of an existing building maximises reuse of the structural materials in that building, particularly if the building is constructed of reinforced concrete. While the majority of the structural components of steel framed buildings can be recycled, this is seldom the case for concrete. However, the restructuring of existing buildings can pose significant engineering challenges, particularly when increasing the height of the structure is desired. In such cases the existing foundations and structural system are unlikely to be able to safely support the load of the additional floors without considerable modification. The interaction between the original (old) structural system components and the retrofitted (new) structural components is likely to be complex, particularly when the structural material involved is reinforced concrete. Concrete shrinks and creeps over time, with the rate of shrinkage and creep being time dependent. The behaviour of old concrete is significantly different from new concrete, notably with respect to shrinkage. Consider the case of a building containing a structural frame sufficient to carry the loads of the existing building.

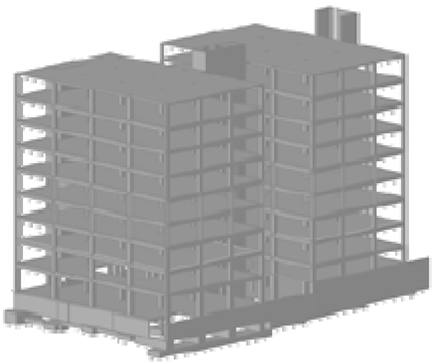
When further storeys are added to the building additional structural members (representing an additional load path) must be added to the original building. The old section of the building then contains both old and new load paths. When the new load path is added, the old load path carries all of the dead load of the original structure, and the new load path carries at most only its own self weight. As each additional storey is added, the extra load is shared between the old and new load paths. However, the new load path shrinks and creeps at a higher rate than the old load path, and this is likely to cause load to be shed from the new load path to the old load path. The old load path may not have sufficient capacity to carry significant extra load, and at some point may start to deform plastically, shedding load back to the new load path. This is a complex process which is difficult to quantify. Condor Tower in Perth, Western Australia, started life as a 10 storey concrete framed office building. It is in the process of being transformed into a 28 storey tower of luxury apartments through the addition of 18 storeys. The additional 18 storeys are being constructed using a combination of precast panels and cast in-situ concrete. Prior to the addition of these floors, the foundation was converted from pad spread footings to an array of micro-piles topped by a concrete raft. The concrete frame was strengthened by the addition of in-situ concrete in-fill walls. A monitoring program is being undertaken in order to obtain a better understanding of the load sharing between the old load path (the concrete frame) and the new load path (the infill walls) in the original part of the building. Strain gauges and data loggers were installed on a number of the concrete columns in the basement, on the first floor and on the eighth floor before the pouring of the infill walls. Data has been collected from the commencement of the installation of the infill walls until the present time. This paper reports the preliminary results of this study. Monitoring is anticipated to continue throughout the remaining construction period and for some time after completion.

2 THE PROJECT

Condor Tower is located in St Georges Terrace in the Perth Central Business District. It was originally known as the Oakleigh Building, and was built in the 1960's. It is currently being restructured to provide inner city residential apartments. The restructuring has been designed and engineered by Pritchard Francis Pty Ltd. Fig. 1 contrasts the Oakleigh Building (left) with Condor Tower (right). The Oakleigh Building was 43 m high, while Condor Tower will be 101 m, almost two and a half times taller.

In order to accommodate the addition of new storeys to the building, significant modifications to the existing structure were required. The original foundations consisted of pad and strip footings approximately 4000 mm wide and 900 mm deep under the columns and load bearing walls. These foundations were not adequate to support the additional loads that will be generated by the new floors. A staged construction process was adopted in which the foundations were modified progressively across the building. Micro cement grout was injected underneath the existing footings to a depth of 1600 mm. Concrete piles of 300 mm diameter were installed in between the original pad footings. The pad footings were then cut back in size and a new 1400 mm deep raft foundation was constructed, incorporating the existing pad footings and capping the piles.

To assist in the transfer of the additional load from the new floors to the foundations, new load bearing concrete in-fill walls were installed. These walls were cast in place between the existing colum



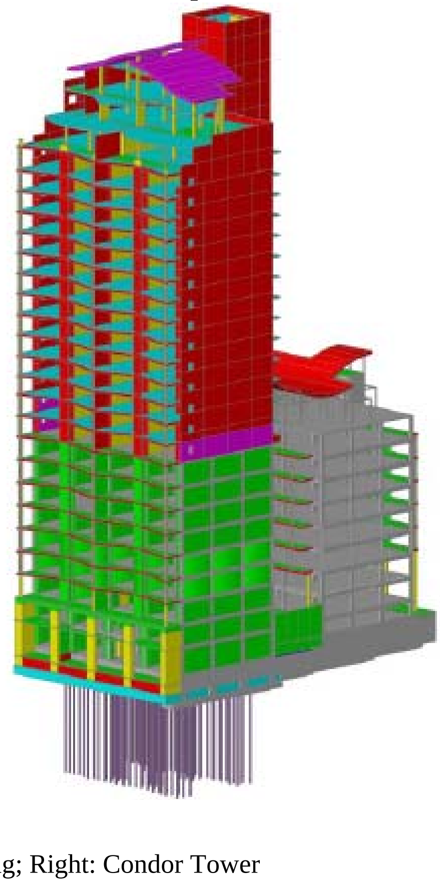


Fig. 1 Left: Oakleigh Building; Right: Condor Tower

the building along each north-south gridline, spanning between the existing slabs of the concrete frames. The new infill walls are not continuous over the height of the building, but are separated by the existing concrete slabs at each floor level. Connection between the walls and the existing frame has been achieved through the installation of shear connectors both horizontally through the existing columns and vertically through the floor slabs. The column layout for the Oakleigh building is shown in Fig. 2. The grid spacing in the north-south direction is approximately 6.2 m and in the east-west direction 7.3 m. Only the Block A is being extended upwards. The floor to floor height in the Oakleigh building is approximately 3.35 m, while the floor to floor height in the added floors is 3m. The original floor slabs are 9 inches thick (approx. 230 mm) and the new floor slabs are 200 mm thick. The column sizes ranged from approximately 710x530mm in the basement to 460x460mm in the fourth to eighth floors, while the in-fill walls varied in thickness from 530mm in the basement to 250mm in the fifth to eighth floors (interior frames).

3 THE MONITORING PROGRAM

Prior to the commencement of the casting of the infill walls, strain gauges and data loggers were installed on the columns on grid lines A and B at the basement level (except for grid line 1, where access was limited and the gauges were installed on the first floor level) and the eighth floor floor level. At each location the strain gauges were configured in a half bridge to measure axial strain and to provide temperature compensation. Temperature sensors were also installed. Each strain gauge bridge was covered by a metal plate to protect against damage, and was connected to a battery powered data-logger, which was installed close to the underside of the floor slab for protection. The data loggers were custom built at the University of Western Australia. Each data logger includes a wireless transmitter and receiver, enabling the data to be collected wirelessly using a standard laptop and the data logger to be reset remotely. The strain gauge locations are shown in Fig. 3. Readings are taken every 20 minutes, and the datacollected from the data loggers every 2-3 months.

4 PRELIMINARY RESULTS AND ANALYSIS

After the completion of the foundation strengthening works, modification of the structural system of the building commenced with the casting of the infill walls between the columns of the existing concrete frame. Naturally the basement infill walls were poured first, followed by the ground floor, the first floor, and so on. The infill walls on each floor extended to the bottom of the existing concrete slab.

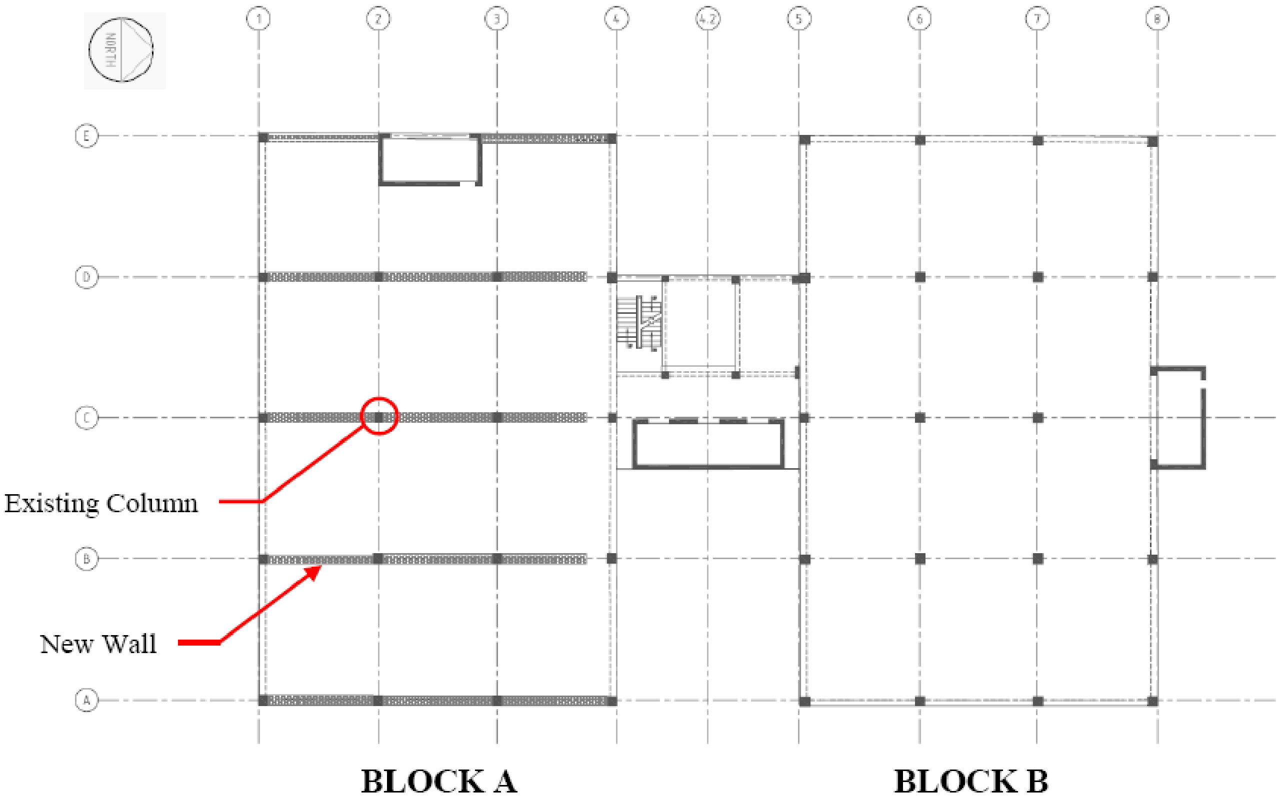


Fig. 2 Typical floor layout.

As mentioned above, the new walls are not continuous, but are interleaved with the original structure. Once the infill walls were completed up to the 8th floor, the construction of the additional floors began. These consist of a combination of pre-cast reinforced concrete panels and in-situ concrete. At the time of writing construction is continuing and the 20th floor has been reached. In this paper the behaviour of the end frame, grid line A in Fig. 2 and the left elevation in Fig. 3, will be considered. The end frame has the advantage of the infill walls being complete from top to bottom. (Due to the architectural requirements for the entry foyer, the frame on grid line B contains some bays without infill walls, as illustrated in the right elevation in Fig. 3. This complicates interpretation of the results.) As the monitoring program is not yet complete, only a relatively crude analysis of the results will be provided here. The strains measured across the columns on grid line A at the basement will be averaged for interpretation. The strains were observed to be quite consistent across the four columns, implying that plane horizontal sections of the building were remaining plane. There was, however, some "breathing" of the building with temperature change, where the distribution of loading between the columns fluctuated with the changing temperature, with some columns exhibiting higher strains at higher temperatures and others lower strains at higher temperatures. Averaging the column values across the floor helped to counter this effect.

The basement columns exhibited strain increases from the time the infill wall on the ground floor above was poured. These strain increases increased at an ever decreasing rate as the construction continued, as illustrated in Fig.4. In order to perform some rather crude analysis of the behaviour, a quadratic equation can be fitted to the data. This equation is also plotted in Fig. 4, and it can be seen that the fit is surprisingly good.

In order to interpret the strain changes correctly, knowledge of the amount of weight being added to the structure is required. This information is quite difficult to obtain, as construction is a continuous process. Based on the amount of material coming on site and the site records, an approximate relationship between added load and time has been constructed. For consideration of the averaged basement results, it is convenient to reference both the load and the strain to the commencement of the infill walls on the ground floor, with time being measured in days from this event, and strain and load being considered as changes from this date. Both Fig. 4 (showing averaged basement strain against time) and Fig. 5a (showing added load against time) are plotted in this way. Fig. 5a shows that as construction proceeded,

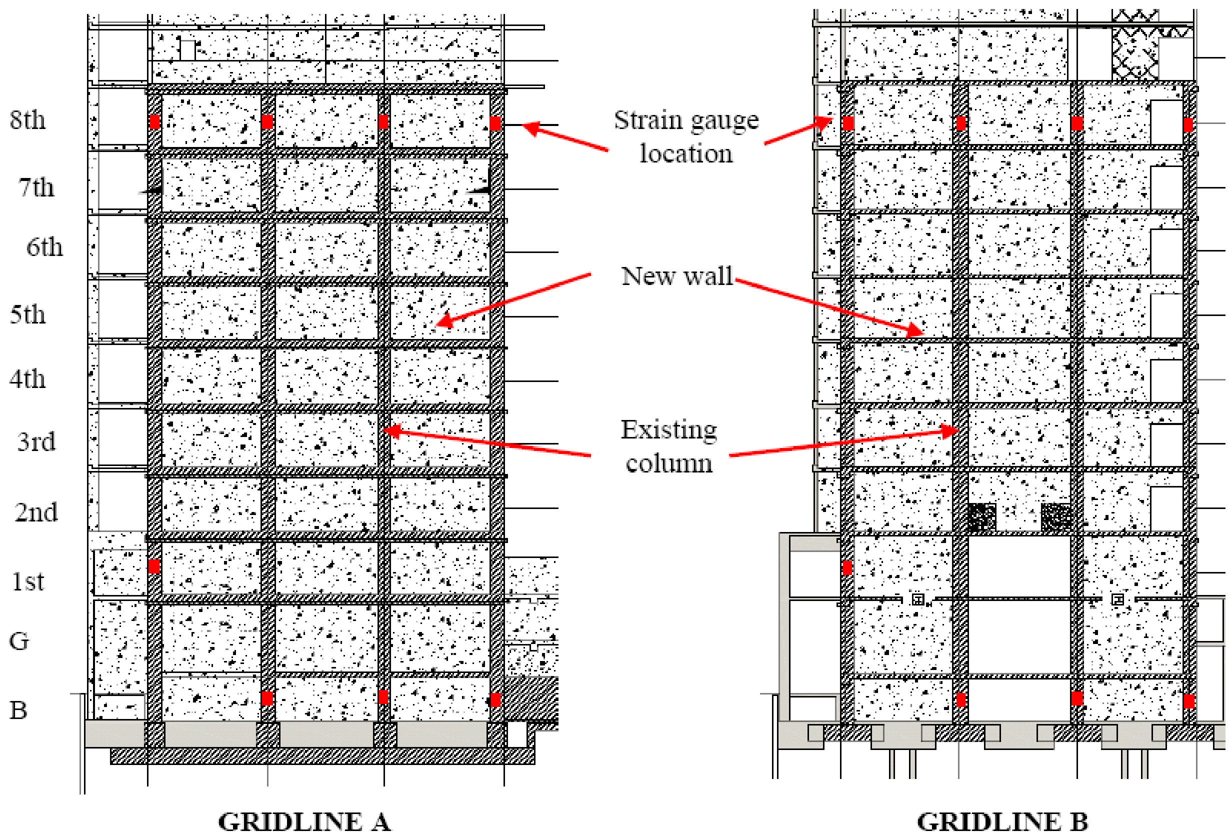


Fig. 3 Strain gauge locations.

the rate at which weight has been added to the structure has increased slightly as the construction moved from infill walls to complete floors, and as the floor to floor construction times improved. In this figure and in the analysis, the total load added to the building has been distributed between the five frames on the basis of tributary areas.

increase. However, this is clearly not the case here. It is instructive to compute the effective axial stiffness of the combined column/wall system. Neglecting the effect of creep and shrinkage and assuming an average Young's modulus for all the concrete (30 GPa), the product of Young's modulus and cross-sectional area (*EA*) for the wall and columns can be computed.

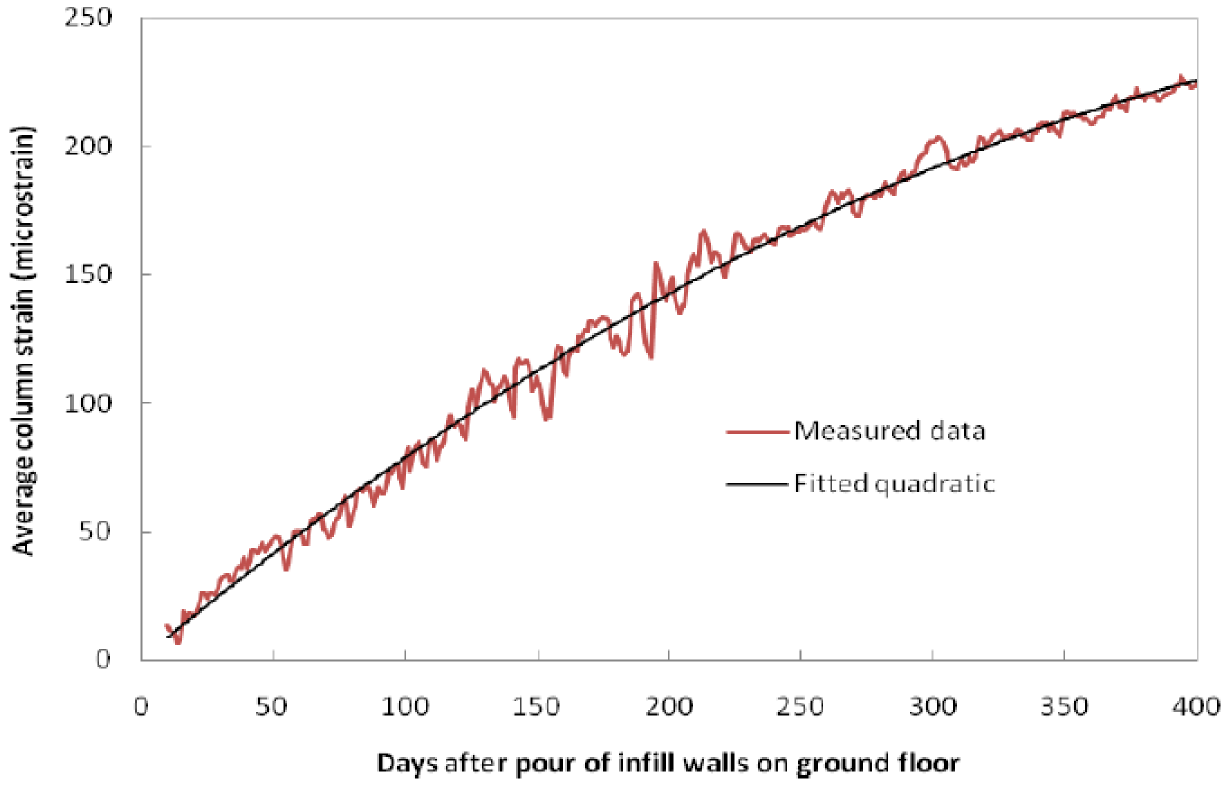


Fig 4. Average column strains at basement level.

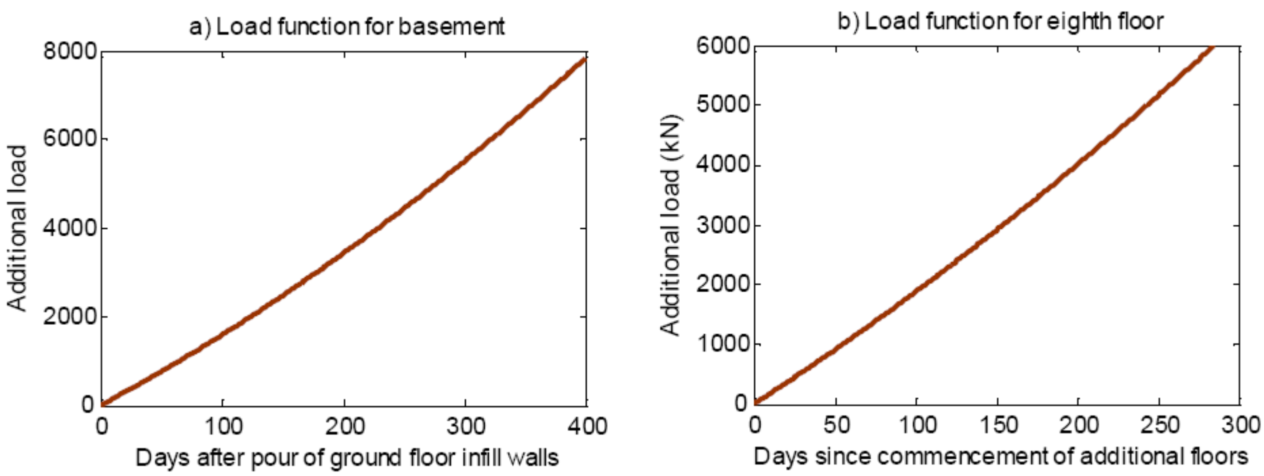


Fig. 5 Additional loading on basement and eighth floors from selected time origin (frame on grid A).

From the figures it is clear that the behaviour of the building is non-linear. A simplistic analysis neglecting the effect of the different ages of the components would suggest that as the rate of load addition increases, the rate of strain increase should also

The reinforcing steel has been taken into account in calculating these stiffnesses. The relative contributions of the columns and the walls are shown in Table 1.

As mentioned, the value of *EA* varies in time in non-linear fashion. This complex time variation is a result of the interacting effects of elastic shortening and creep (occurring in both the old and new concrete but at different rates) and shrinkage (only occurring in the new concrete). When analysing the creep and shrinkage of new high rise concrete buildings occurring during and after construction, the effect of each phenomenon is commonly treated independently and combined linearly, despite the obvious non-linearity of the system [1-3]. The system is effectively statically determinate with respect to vertical loading. Since plane sections remain plane, and each floor shrinks, creeps and undergoes elastic shortening independently of the behaviour of every other floor, this does not lead to severe errors. However, in the situation here, the differential behaviour of the two structural systems leads to significant interaction. Unlike normal high rise buildings (which contain a single vertical load path), the two vertical load paths present in the Condor Tower render the structure indeterminate with respect to vertical loading. Applying the conventional model for shrinkage and ignoring the interaction leads to the generation of significant tensions in the new concrete members and corresponding significant compression increases in the old members which are unrealistic. As a first step towards generating a suitable model, in this paper the variation of the effective *EA* with time is investigated. Comparison is made with the equivalent values of *EA* generated by conventional elastic approaches. Since the response of the structure varies in time (Fig. 4), there is no unique value of *EA*. There are two common ways of evaluating *EA* for a non-linear structure, namely the secant approach and the tangent approach. In the secant approach, at any value in time the ratio of the total load to the total strain is determined. In the tangent approach the derivative of load with respect to strain is determined at each particular value in time.

Here both methods are used in order to investigate the behaviour of *EA*. The variation of these moduli with time is approximated by using the quadratic equations fitted to the observed variation of strain with time and the deduced variation of load with time. Fig. 6a shows the variation of the secant *EA* with time. Compared to the elastic values presented in Table 1, the secant *EA* starts below the elastic *EA* for the columns alone at early time, and progresses to a value close to the elastic *EA* for the columns. The tangent *EA* (Fig. 6b) commences at the same value, but at large time reaches a value which is closer to the elastic *EA* of the columns and the walls together. That the initial value of *EA* is lower than the elastic stiffness of the columns alone is unexpected. However, the value is quite sensitive to the rate of loading at early times, and the accuracy to which this is known is low. To within the accuracy of the data, the initial tangent *EA* can reasonably be taken as the elastic stiffness of the pre-existing frame. This means that at small time virtually all the load is carried by the old frame. The role of the infill walls (apart from adding load) is to provide bracing which is effectively continuous, so the columns can be considered stocky. Consequently their load carrying capacity is significantly enhanced. At larger times the effective tangent *EA* increases at an increasing rate. So far it has not reached the *EA* of the combined section including both the walls and the columns. It is predicted that this value will be the limit at large time. The ongoing monitoring program will allow this prediction to be verified or disproved in the near future. The implication of the increasing tangent *EA* at larger times is that the load is increasingly being shared between the old and the new concrete. It will be interesting to see whether, after the loading sequence has finished, the larger shrinkage and creep in the new concrete will cause some of this load to be shed from the new concrete into the old columns. A similar analysis can be performed

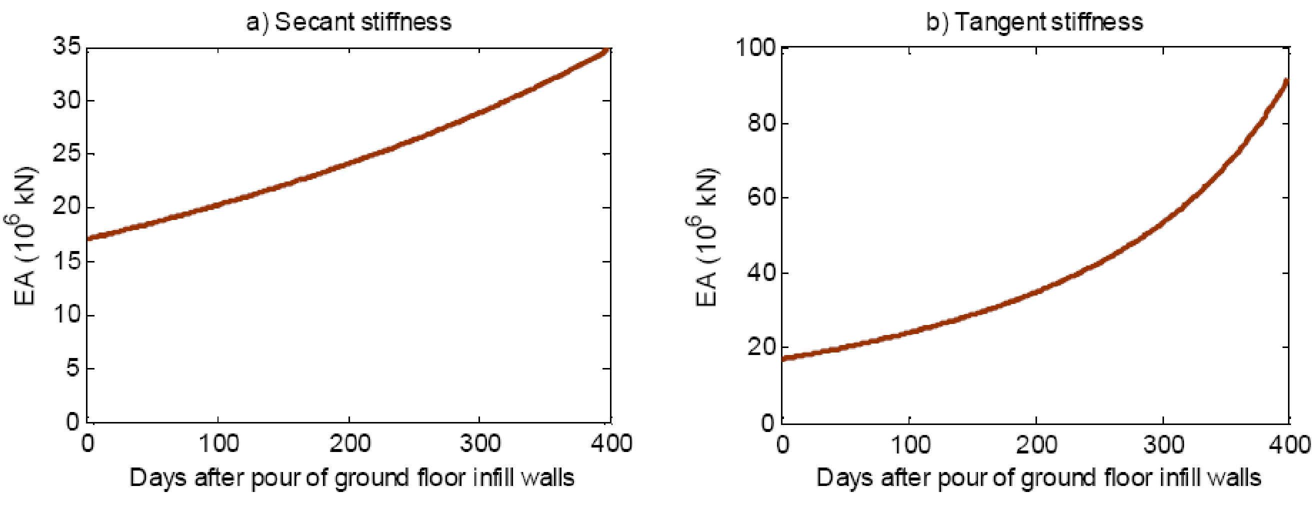


Fig. 6 Effective secant and tangent stiffness of basement columns and walls over time.

on the data collected for the columns at the top of the original structure (the eighth floor). For these columns, the relevant time to commence the analysis is the time of construction of the first new floor. The installation of the in-fill walls below the eighth floor was found to have relatively minor impact on the strains in these columns. In contrast, temperature variation was found to have a much larger effect on the eighth floor columns than the basement columns. As might be expected, the temperature fluctuations themselves were smaller in the basement. Nevertheless, even allowing for this, the variations of strain with temperature (after averaging across the four columns) on the eighth floor are significantly larger than the basement columns. It is postulated that the "breathing" effect between the columns observed *within* the frame at grid location A also happens *between* the five concrete frames, and that as the temperature changes the strain in one frame may fall while the other frames may rise to compensate. The variation of the average column strain on the eighth floor with respect to time from the installation of the first new floor is shown in Fig. 7. The larger temperature effect is clearly evident,
but the overall variation with time can be represented very well by a quadratic function, as the fig-

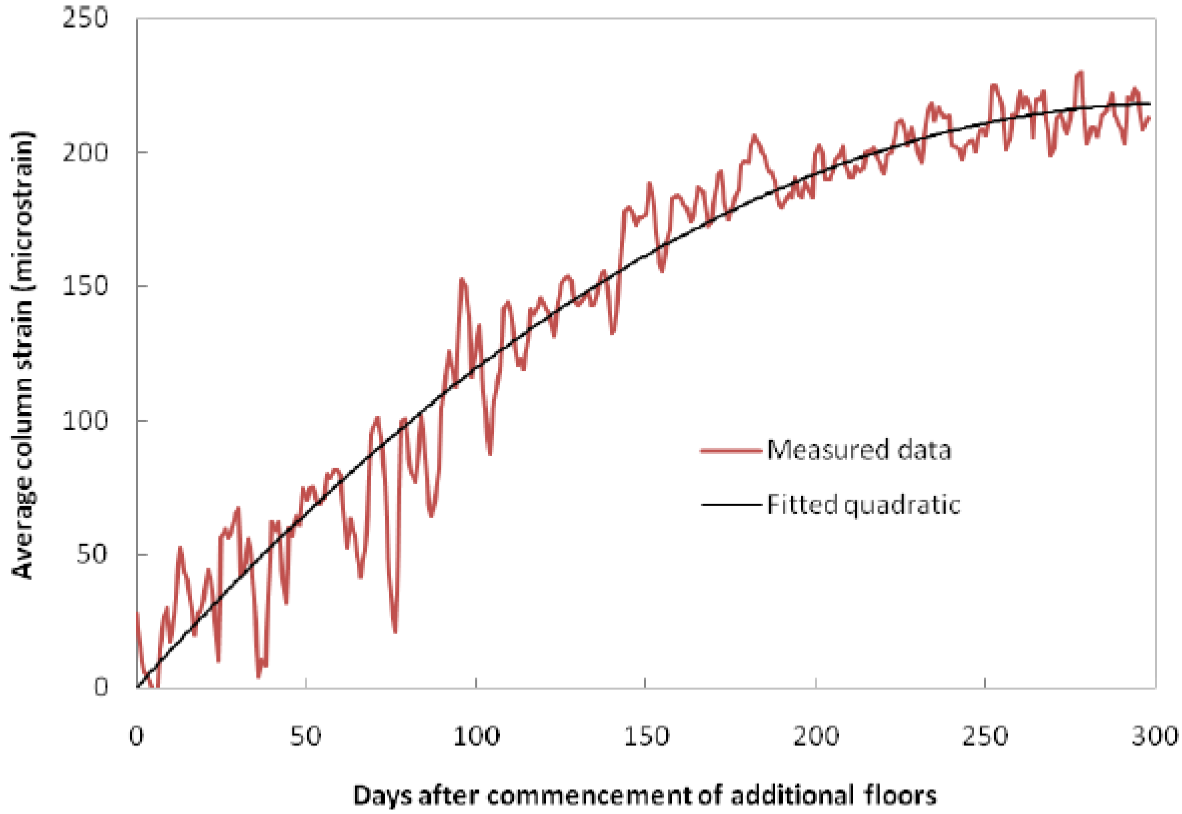


Fig. 7 Average column strains at eighth floor level.

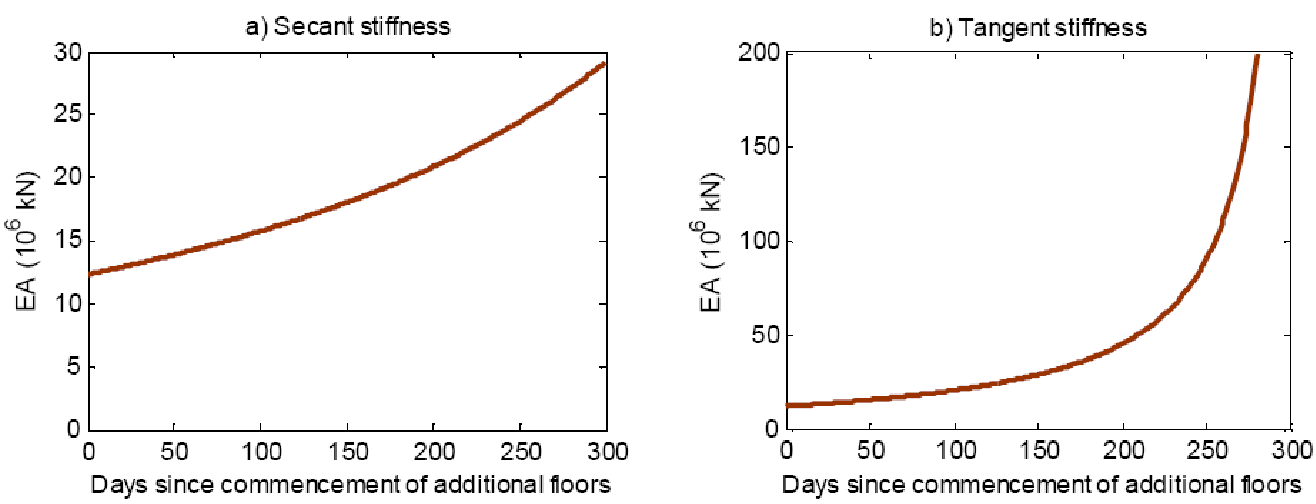


Fig. 8 Effective secant and tangent stiffness of eighth floor columns and walls over time.

ure illustrates. In order to evaluate the variation of the effective *EA* with time, the variation of load with time is re-configured as a function commencing at the time of the installation of the first new floor, and representing the additional load added from that time forward (Fig. 5b). Like the load function for the basement, this load variation can be approximated by a quadratic function. Using the same definitions as used earlier, the variation of the secant and tangent *EA*'s with time are calculated. These are plotted in Fig. 8. Once again the initial tangent *EA* computed using the fitted quadratic functions is lower than the elastic *EA* calculated for the old columns alone. However, the accuracy of the fit of the quadratic function to the strain variation at lower times may be rather inaccurate, and to within the accuracy of the method it would be reasonable to assume that at small times the effective *EA* is the *EA* of the old columns alone.

These observations support the notion that the original concrete frame carries most of the load at early time. Once again, over time the effective tangent *EA* increases. Unlike the basement, however, at large time the *EA* exceeds the nominal elastic *EA* for the whole section including the old columns and the new walls. This is not expected. Inspection of Fig. 7 suggests that at large times the strain in the old columns reaches a limit and does not increase any further. It is tempting to suggest that the column has reached a failure load and cannot support any more load. Such behaviour would require a plastic section to develop elsewhere within the column, allowing strain to occur with no further increase in stress. However, the presence of the new walls (which are tied to the old columns over their depth by shear connectors) makes this unlikely. Furthermore, if one section of the column is undergoing significant inelastic strains, spalling of the concrete cover is expected to occur. No such spalling has been observed anywhere within the building. The behaviour of these columns as the load continues to increase will be of significant interest. The plateau of the strains evident in the data collected to date does not appear to have a satisfactory explanation at this stage, and the collection of more data is necessary.

5 CONCLUSIONS

Although the interaction between the old and new vertical load paths in the Condor Tower is complex, data collected so far indicates that the structure is well behaved. Initially the load was carried almost completely by the original concrete frame, but as the loads on the structure have increased the load is being shared to an increasing extent by the two load paths without any obvious distress being exhibited by the old frame. At this stage a satisfactory model of the interaction including the effects of shrinkage and creep has not been constructed. However, from a practical engineering point of view, the results support the approximation of the tangent stiffness of the structure at small times by the stiffness of the original frame and at large times by the stiffness of the combined old frame and new walls computed assuming that plane horizontal sections remain plane. In a similar manner, it is reasonable to assume that the shrinkage and creep deformation of the combined structure will be bounded by the lesser deflection of a one dimensional model consisting of the properties of the old columns alone and a one-dimensional model consisting of the new concrete walls alone. Continued monitoring of the building during the remaining construction period and beyond will allow a clearer understanding of the behaviour and potentially allow a more accurate model of the behaviour of the combined system to be derived. Monitoring of the column stresses after the completion of construction will allow the effect of shrinkage and creep on the load sharing to be better understood, as one of the variables (the elastic shortening under additional loading) is removed. The interaction between the shrinkage, load sharing and creep is still likely to be complex.

6 ACKNOWLEDGEMENTS

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