Internal forces of concrete floor diaphragms in multi-storey buildings

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ABSTRACT: Simplistic design methods are commonly employed by design engineers to determine the approximate magnitude and distribution of inertial forces in reinforced concrete floor diaphragms of multi-storey buildings. Various researchers have identified that the commonly employed simplistic design method, the Equivalent Static Analysis method, in some cases, provides a poor representation of the true structural response. This research investigates the magnitude and trends of forces in concrete floor diaphragms, with an emphasis on transfer forces, under seismic loading. This research considers the following items: inertial forces which develop from the acceleration of the floor mass; transfer forces which develop from the interaction of lateral force resisting elements with different deformation patterns, such as wall and frame elements; and variation of transfer forces due to different strengths and stiffness of the structural elements. The magnitude and trends of forces in the floor diaphragms have been determined using 2-dimensional inelastic time history analysis. Trends have been identified which will aid the improvement of seismic floor diaphragm design methods

1 INTRODUCTION

Reinforced concrete diaphragms (floors and roofs) of a structure tie the vertical structural elements (such as walls and frames) together to allow buildings to resist external loads such as gravity and lateral forces from seismic events or wind action. Floor diaphragms play an important role of transferring forces from the structure to the lateral force resisting elements which then transfers the forces from the structure to the ground.

The magnitudes and paths of internal forces within concrete floor diaphragms are considerably more complex than those assumed by some simplistic methods employed in current design practice, such as the Equivalent Static Analysis (ESA) method. The ESA method is used around 90% of the time for structural design, due to its simplicity and efficiency. The ESA method has been found by various researchers to under-estimate the acceleration of floors, particularly in the lower levels of the buildings (Bull 1997, Nakaki 2000, Fleischman et al 2002, Rodriguez et al 2002), leading to poor predictions of the structural response.

1.1 Diaphragm forces

There are two main types of forces that exist in diaphragms, namely; inertial forces from the accelerations of the floors and transfer forces which result from incompatible deformation patterns from different lateral force resisting systems within the structure. The type of lateral force resisting systems and the geometry of the structure will dictate which of these forces, inertia or transfer, will dominate.

A dual or a hybrid structure (combination of walls and frames) provides lateral force resistance through both frames and structural wall systems. A dual system is a favourable system to use due to the ability to dissipate large amounts of energy. The wall element in the system will provide an increase of stiffness which will be beneficial in terms of drift control.

The issues of size and location of inertia and transfer diaphragm forces are not limited to “dual” systems. Tower-to-podium floors and ground floors over basements are further examples. Also buildings with a series of walls or frames of varying geometries and strengths will have similar diaphragm issues.

A paper by Clough (1982) discussed the existence of two types of internal forces in diaphragms. These are described as equilibrium forces which are associated with externally applied loads and compatibility forces associated with the deformation of the structure.

Research by Paulay and Priestley (1992) has shown when a dual system is subjected to lateral loads, such as a seismic event, a form of “fighting” exists between both the wall and frame elements. This “fighting” comes from the contrasting deformation patterns for both frames and walls; this is similar to that described by Clough. The frame will primarily deform in shear mode when subjected to lateral forces and the wall will deform in a bending mode, see Figure 1.

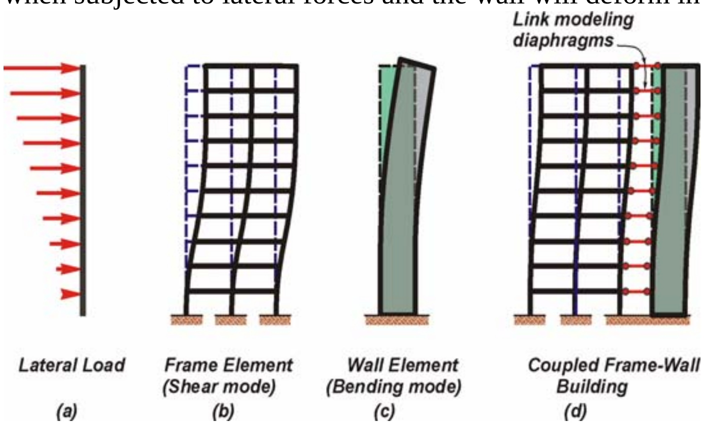

Figure 1 Deformation patterns for frame and wall elements (Paulay et al, 1992)

The connection of the frames and walls to a diaphragm requires deformation compatibility to exist for the entire structure. This compatibility restraint alters the overall deformation of the structure forming a combination of shear and flexural deformation modes which result in the “fighting” action described. Paulay and Priestley (1992) found that the wall will dominate the structural behaviour in the lower levels of the structure and the frame will control the behaviour in the upper levels of the structure. The “fighting” causes an increase in forces present in the diaphragm. These forces can be many times larger than the inertia forces on the diaphragms alone (Paulay and Priestley, 1992, Stewart, 1995, Bull, 1997).

Traditionally inertial and transfer (compatibility) forces have been treated separately. This is not correct, both inertial and transfer forces should be considered simultaneously. As the structure accelerates from the inertial forces it also deforms; hence inertial and transfer forces are not separate entities. The ESA method only considers inertial forces and neglects to account for these transfer forces. In situations where these transfer forces may be present the ESA method should not be used to determine the forces in the diaphragm.

NZS1170.5 (2004) identifies that mixed systems (frames and walls) may induce large seismic actions in the structure. This Standard requires rational analysis to be carried out for these types of structures but does not give any specific details on how to deal with transfer forces. FEMA450 (2003) briefly discusses dual systems, criteria is provided with regards to the percentage of resistance that the frames must provide, but no details are given on transfer forces. Eurocode 8 (2004) recognises that dual systems will affect the structural behaviour and accounts for this by providing different behaviour factors for different structural types including dual systems as a type of structure. No details could be found regarding the development of these behaviour factors though.

2 DESCRIPTION OF ANALYSIS CARRIED OUT

The investigations of transfer and inertial forces in reinforced concrete floor diaphragms were carried out using a non-linear time history analysis program, Ruaumoko 2D (Carr 1981-2007).

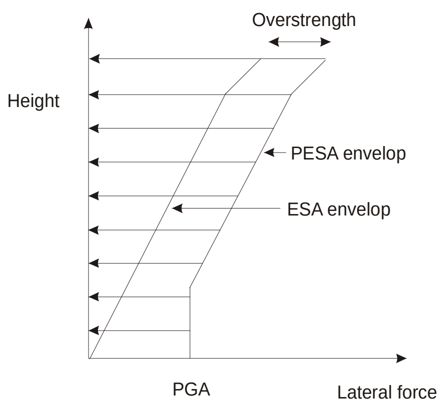

Figure 2 static forces at levels of the structure for the pESA and ESA

The distributions of inertial forces were investigated by developing regular frame structures which meet the criteria of the ESA method. These distributions were compared to a new pseudo-Equivalent Static Analysis (pESA) envelope which has been suggested as an alternative static design method. One of the earlier references to this work is the paper by Bull (2004). This envelope accounts for both peak ground acceleration (PGA) and overstrength of the structure which is not accounted for in the ESA method. The PGA force is calculated by multiplying the seismic weight of the floor by the elastic site hazard spectrum for horizontal loading from NZS1170.5 (2004). The PGA force is extended from the base of the structure to meet the ESA method force envelope which is multiplied by the overstrength of the structure. A graphical representation of the envelope is provided in Figure 2.

The investigation of transfer forces was carried out using various models which investigated the effects of varying stiffness, strengths and different strengths of the connection element (floor) between the wall and the frame elements.

2.1 The model

The structures used to investigate inertial forces were regular perimeter frame structures with internal gravity frames. The buildings were four bays wide by six bays deep (6.1m in length) and the heights of the structures were 3, 6, 9 and 12 stories with each inter-storey height being 3.5m.

The models used in the 2-D analysis to investigate transfer forces were three bay 7m span frames connected to a 400mm thick wall of varying lengths. The height of each level of the structure was 3.6m. A spring element was used to connect the lateral force resisting elements, the frame and the wall, together. This element represented the typical properties of floor diaphragms, such as small out-of-plane stiffness compared to in-plane stiffness.

The investigation of the effect of stiffness variation on transfer forces was carried out by changing the stiffness ratio of the wall and the frame element. The stiffness ratio was obtained by finding the stiffness of the wall and frame elements at the base of the structure and comparing them as a ratio. The frame-wall stiffness ratios investigated were; 1:8, 1:19, 1:43, 1:105, 1:233 and 1:577. To ensure each of the structures of different stiffness had the same dynamic properties the fundamental translational period of all the structures was kept constant. The fundamental translational time period for the wall-frame structures was 0.576s.

A further case study was carried out to investigate the effect of the strength of the structure on transfer forces. This was carried out by investigating three different ductility levels; elastic, ductility of 2 and ductility of 3. The ductility level of the structure was determined by performing pushover analysis. Equal energy principles were employed due to the short time period of the structure to determine the level of structural ductility.

The final case studied was associated with determining the effect of the strength of the connection element on transfer forces. Various strength levels for the connection spring were considered, these were; 300kN, 500kN, 800kN, 1000kN, 1200kN, 1500kN and elastic.

The 2-D analysis carried out is representative of 3-D structures as long as the transfer of forces is by shear in the floor diaphragm and torsion is not present. In some cases where the floor diaphragm is long and slender, some inaccuracy may occur.

2.2 Modelling parameters

The model was designed in accordance with the weak beam-strong column principle. The structural members were modelled as reinforced concrete members. Effective section properties were used to account for stiffness degradation after section cracking and rigid end blocks were incorporated to account for additional stiffness in the section joints. The hysteresis model employed to represent the inelastic action in the concrete members was the Revised Takeda Model. This model was used because it represents typical hysteretic behaviour of reinforced concrete members. The plastic hinge lengths, representing the location of inelastic action, used in the model were determined using the guidelines given in Priestley et al 2007. The damping model used in the analysis was a damping model where the damping varied linearly with elastic natural frequencies of the structure. The distribution of weights for this model represents the typical weights expected to be carried by a three bay frame; the weight was distributed evenly to each bay. The time step used in the analysis was 0.002s. A sensitivity study was carried out to ensure the time step was small enough. The over-strength factors for the buildings were determined from pushover analyses.

2.3 Time-history records

Twelve time history records (six, each with a north and south component) were employed to investigate the trends of inertial and transfer forces. The records chosen for this analysis were records with similar motion characteristics to that expected of a seismic event in Wellington. There are three major types of fault motion that has been identified to possibly occur in Wellington; Strong-forward directivity caused by rupture of the Wellington fault (active right lateral strike-slip fault), near-neutral directivity due to rupture on the Wellington fault and motion which is related to a large subduction zone type event (McVerry, 2003). The earthquake records chosen were; Lucerne (Landers, California), Izmit (Kocaeli, Turkey), La Union (Michoacan, Mexico), El Centro (Imperial Valley), Llolleo (Chile), Tabas (Iran).
The records were scaled according to NZS1170.5 (2004) to match the site specific design spectra. In this analysis the k_1 (local) factor was used, the k_2 (family) factor was discarded. The k_2 factor was discarded as this factor introduces unnecessary conservatism in the results. Omitting this factor will ensure more realistic results are obtained.

3 RESULTS

3.1 Inertial force investigation

Figure 3 provides a graphical comparison of both the pESA and ESA results compared to the results from four different seismic events for a six storey structure. This figure indicates that the ESA method underestimates the forces in the lower levels of the structure. This result is in accordance which what past researchers have found. Other analyses were carried out on 3, 9 and 12 storey structures. The results for these structures indicated that the pESA method again predicted inertial forces more accurately than the ESA method for the 3 and 9 storey structures. It was found that both static methods over predicted the inertial forces for the 12 storey structure.

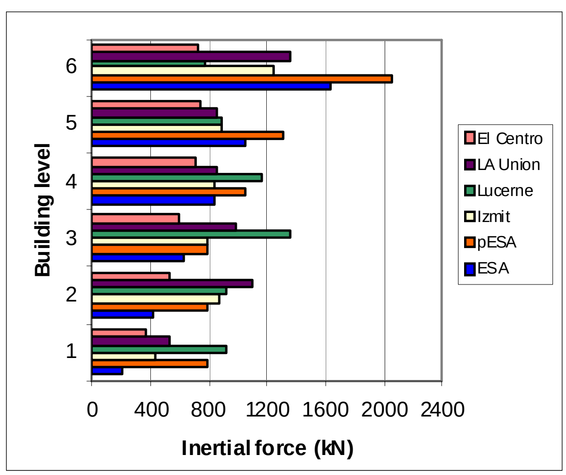

Figure 3 pESA and ESA results compared to inelastic time history analysis results for a 6 storey structure

3.2 Comparison of transfer forces to inertial forces

Comparisons were made between the magnitudes of inertial and transfer forces. Figure 4 provides a comparison between these forces at a point in time where the combination of forces at level 1 of the structure are greatest. This figure clearly shows that transfer forces are many times larger than inertial forces; especially at the base of the structure. It is interesting to note the different sign of force for inertia and transfer force at the top of the structure. This is due to the different deformation patterns of the wall and the frame elements. For the lower levels of the structure the wall is pulling the frame (ie, the wall is stiffer) and for level 9 the frame is pulling the wall. This change in sign occurs due to the change of the stiffness of the dominant lateral force resisting element with height of the structure. This pattern is typical for structures with SR values of less than 1:105.

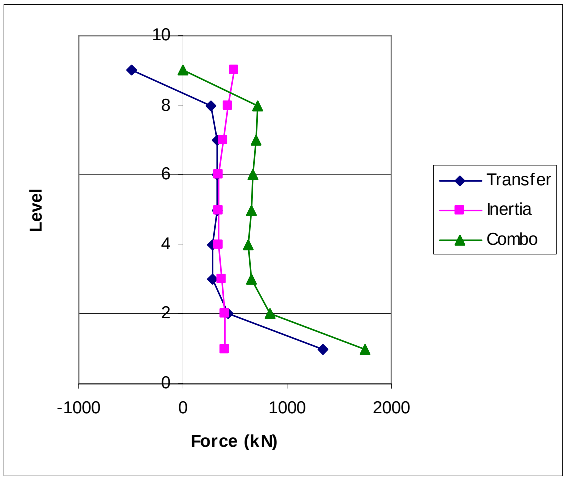

Figure 4 Comparison of inertia and transfer forces at an actual time step of non-linear time history analysis

3.3 Stiffness variation results for transfer forces

The magnitude of transfer forces in this and the following models was measured by the magnitude of axial force in the spring element between the wall and the frame. The axial force provides a measure of transfer forces as this spring element provides the only connection between the frame and the wall elements.

The results shown in Figure 5 are results indicating the trends and affects on transfer forces for an inelastic structure due to different wall-frame stiffness combinations in level 1.

Figure 5 indicates that the greatest transfer forces generally occur in structures with medium frame-wall stiffness ratios such as SR 1:19 and SR1:43. This occurs as both the wall and the frame elements are of similar stiffness, hence both elements are stiff enough to actively resist the deformations imposed by the other element. When the stiffness levels of both the wall and the frame element differ significantly the more flexible element is unable to resist the deformations imposed by the stiff element, hence the flexible element follows the deformed shape of the stiffer element and therefore lower transfer forces are observed.

This figure also indicates for stiffness ratios lower than 1:105 the transfer force seems to drift in one direction resulting in residual forces. This is due to the permanent drift of the wall element

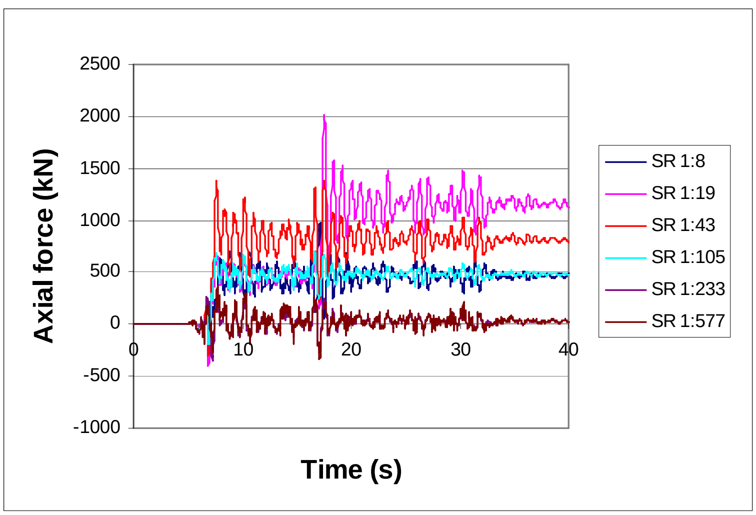

Figure 5 Indication of transfer forces which developed in level 1 of the 9 storey structure for various stiffness ratios (SR)

which results from the formation of a plastic hinge at the base of the wall. These forces arise due to the frame resisting the permanent drift which is imposed by the wall. These forces remain after the earthquake.
It was also observed that smaller transfer forces existed for Level 9 of the structure. At level 9 not much inelastic action occurred; hence the transfer forces are not apparent.
Smaller transfer forces are observed for the middle levels of the structure. This is because the difference between the deformation patterns of the wall and the frame elements at the middle levels is less than that at level 1 of the structure.

3.4 Strength variation results for transfer forces

A selection of strength variation results has been presented in the following figures. These figures provide comparisons between the magnitudes of transfer forces for different structural ductilities.

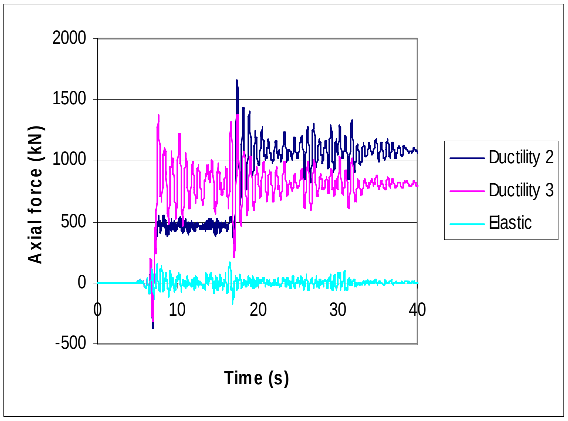

Figure 6 Variation of transfer forces due to different ductility levels in the structure at level 1 of the structure

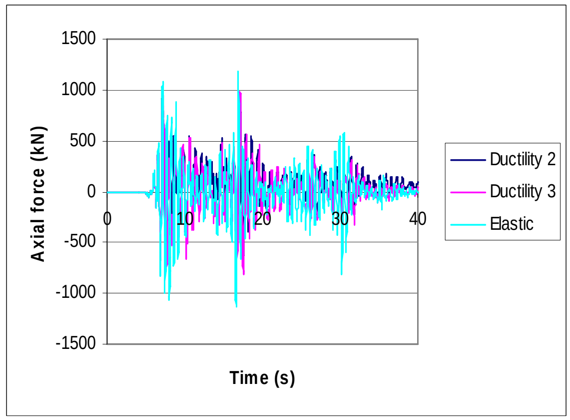

Figure 7 Variation of transfer forces due to different ductility levels in the structure at level 9 of the structure

Figure 6 indicates there is a major difference in relation to the magnitude of transfer forces between elastic and in-elastically responding structures. The forces observed, due to the increase of ductility, in level 1 are of the order of ten times larger than the forces obtained from elastic analysis. Under most reasonably sized earthquakes, inelastic behaviour will occur in the structure. Therefore, this highlights the importance of considering the inelastic action within the structure when considering transfer forces.
Figure 7 shows the transfer forces for level 9 obtained from elastic analysis are slightly greater than the forces determined from considering the inelastic action of the structure. This is because when the inelastic action is considered, a hinge forms at the base of the wall which in turn results in the deformation pattern of the wall being linear. The transfer forces are slightly smaller as the linear deformation pattern (compared to the typical wall deformation pattern) of the wall is more similar to the deformation pattern of the frame.

3.5 Connection strength variation results

Figure 8 indicates the trends found due to changing the strength of the connection element (floor diaphragm). This figure is for a 9-storey structure with ductility of 2.
It should be noted from the results that the connection strengths of 1000kN and 500kN both results in inelastic action in the spring member. These results show that when the connection between the wall and the frame element is inelastic, smaller transfer forces are obtained compared to when the connection is elastic. When the connection becomes inelastic between the wall and the frame elements some of the deformation is taken by the connection element. This allows the

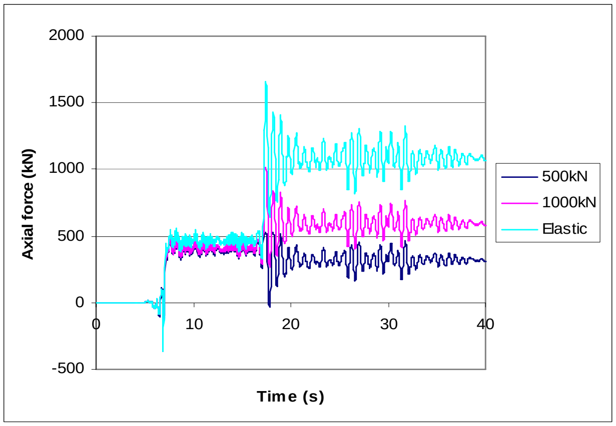

Figure 8 Comparison of the magnitude of transfer force for different connection strength

frame and wall element to act in a more independent way; hence producing lower transfer forces resistance between the elements. When the connection is stronger, an increase in transfer of forces in the connection element due to the incompatible deformations occurs. It should be noted here that as inelastic action occurs in the connection element, the level of residual force in the floor reduces but still exists. The level of inelastic action and hence realistic residual forces which exist in the structure will be researched in the near future.

4 CONCLUSIONS

Various conclusions can be drawn from the above analysis. These conclusions are summarised in the bullet points provided below:

- The pESA method was found to predict the magnitude of inertial forces better than the ESA method especially at the lower levels of the structure
- Transfer forces were found to be many times larger than inertial forces at the base of the structure
- The direction of inertial and transfer forces have been found to be in opposite directions for the upper levels of the structure where the relative stiffness of each lateral forces resisting elements changes
- Median frame-wall stiffness ratios produce the largest transfer forces as each lateral force resisting element can actively resist the lateral deformation
- Large or small stiffness ratios result in smaller transfer forces as the flexible element goes along for the ride and does not resist the other elements deformation
- Higher levels in the structure are found to have lower transfer forces due to less inelastic action occurring in the higher levels
- Residual forces were found to remain in the structure at the completion of the seismic event. These residual forces were found to result from the plastic hinge formation in the wall element. The plastic hinge cause the wall to permanently drift which resulted in permanent diaphragm forces
- Transfer forces were found to be up to ten times larger when inelastic action in the building elements was considered rather than elastic action. This highlights that for dual structures elastic analysis is inappropriate
- Inelastic action in the connection element (floor diaphragm) reduces transfer forces but does not remove residual floor forces

These results show that the pESA method could be used as an alternative to the ESA method for simple regular structures. These results also indicate that transfer forces can be reasonably large and should be considered when designing structures with both wall and frame lateral force resisting elements or combinations of walls with different geometries and strengths or for buildings with frames of different geometries and strengths.

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