Trends of Internal Forces in Concrete Floor Diaphragms of Multi-storey Structures During Seismic Shaking

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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 (ESA) method, in some cases, provides an inadequate outcome for the estimation of forces that develop in floor diaphragms. This research investigates the magnitude and trends of forces in concrete floor diaphragms, with an emphasis on transfer (compatibility) forces, under seismic loading. The first part of this research investigates a new pseudo-Equivalent Static Analysis (pESA) method for determining inertial forces in floor diaphragms. The pESA method was found to provide adequate results for regular structures in comparison to the more sophisticated non-linear time history analysis results. The second part of this paper describes research that investigates the magnitude of transfer and total forces within reinforced concrete floor diaphragms of structures with vertical elements of varying stiffness (different sized frames or walls or a combination of walls and frames). These forces were investigated using non-linear time history analysis. The effects of: structural ductility, the ratio of the stiffness of vertical lateral force resisting elements, the strength of the connection element between the lateral force resisting systems, the structural height and various modeling sensitivities on the magnitude of transfer and total forces were investigated. The main outcomes of this research are identifying appropriate methods for the design of floor forces and determining trends associated with transfer floor forces.

KEYWORDS: Floor diaphragms, pESA method, inertial forces, transfer forces

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 transfer the forces from the structure to the ground. The magnitudes 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 frequently in 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 ([1], [10], [8], [13]), leading to poor predictions of the structural response. "Parts", NZS 1170.5 [14] overestimates floors accelerations for diaphragm design.

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 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. One of the problems with this type of system is the development of transfer or compatibility forces within the floor diaphragm.

A paper by Clough [7] 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 [11] has shown when a dual system is subjected to lateral loads 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 [7]. The frame will primarily deform in shear mode when subjected to lateral forces and the wall will deform in a bending mode (Figure 1).

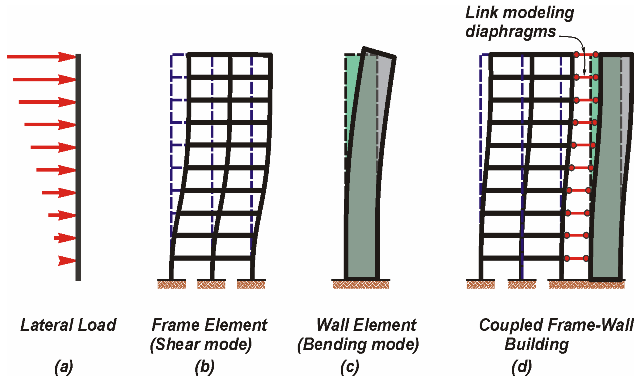

Figure 1 Deformation patterns for frame and wall elements[11]

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. This "fighting" causes an increase in forces that are present in the floor diaphragm. These forces have been found to be many times larger than the inertia forces in the diaphragms alone ([1], [16], [11]). The forces that develop from this fighting action are referred to as transfer or compatibility forces.

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 simultaneously. Therefore inertial and transfer forces should not be treated separately.

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.

The New Zealand loadings standard ([15]) 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. The American seismic standard [4] 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. The European seismic code [3] 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. This research is hoped to fill the knowledge gap on the forces within floor diaphragms.

2. DESCRIPTION OF ANALYSIS CARRIED OUT

The investigations of transfer and inertial forces in reinforced concrete floor diaphragms was carried out using a non-linear time history analysis program called Ruaumoko 2D [6]. The distributions of inertial forces in floor diaphragms were investigated by developing regular frame structures which meet the criteria of the ESA method and determining the floor forces in the structure by the use of the ESA method. These distributions were compared to the results from a new pseudo-Equivalent Static Analysis (pESA) method and time history analyses. The pESA method has been suggested as an alternative static design method. One of the earlier references to this work is the paper by Bull [2]. 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 the New Zealand loadings standard [14]. The PGA force is extended from the base of the structure to meet the pESA method force envelope (which is the ESA multiplied by the flexural overstrength of the structure). A graphical representation of this envelope is provided in Figure 2.

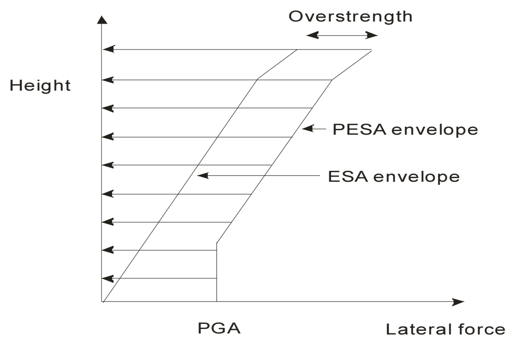

Figure 2 Static forces for ESA and pESA envelopes

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

2.1. The Model

The structures used to investigate the applicability of the pESA method were regular perimeter frame structures with internal gravity frames. The buildings were four bays wide, six bays deep (6.1m in length) and had heights of 3, 6, 9 and 12 stories with inter-storey heights of 3.6m. 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 and heights similar to the first model but excluding the 12 storey structure. A spring element was used to connect 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. These models assume lumped plastic hinges, no shear deformation and a rigid foundation. The effect of these modeling assumptions was investigated by a sensitivity study. The results from this study are presented at the end of this paper.

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 frame-wall structures was kept constant (9-storey 0.576s, 6-storey 0.321s and 3-storey 0.275s). 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 structural 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. Another case study on the effect of the strength of the connection element (diaphragm) on the magnitude of transfer forces was also investigated. 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 action, predominately in the floor diaphragm and torsion is not considered in the building behaviour. In some cases where the floor diaphragm is long and slender with the loading in the short direction of the floor, some inaccuracy may occur when using the current conclusions for design of such floors.

Sensitivity studies were carried out for these models. The effect of foundation compliance, magnitude of the earthquake, time period of the structure, shear deformations and lumped plastic hinge assumption were all investigated. The foundations were modelled using a series of springs which modelled the interaction between the piles and the soil. The foundation was modelled assuming the structure was in the Wellington (NZ) CBD area (a high seismicity region). The subgrade reaction coefficients used to model the pile-soil interaction were 4574kN/m3 for 0-5m, 9806kN/m3 for 5-10m and 15037kN/m3 for 10-20m. The Ramberg-Osgood hysteresis rule was used to represent the inelastic action of the soil-pile system. A study was also carried on the effect of the level of seismicity on total floor forces. The earthquake records for this study were scaled to match the design spectra for the Auckland (NZ) region (a low seismicity region). The magnitude of transfer and inertial forces were compared to the results from the Wellington study. In the initial study all the 9-storey structures were designed to have the same fundamental translational time period of 0.576s. Two other time periods with the same stiffness ratios, strengths and heights were investigated, these were 0.972s and 1.437s. The effect of the lumped plastic hinge assumption in the wall was investigated by comparing the total forces which develop from a two component hinge model to results from a distributed plasticity hinge model. The distributed hinge model was developed by connecting a beam plastic hinge element to a multi-spring element in Ruaumoko [6].

2.2. Modelling Parameters

The models were 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 hysteretic 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 Priestly et. al. [12]. A constant damping model was used in these analyses. The weights for this model were based on: Hollow-core flooring of 300kg/m2, 90mm topping slab, density of concrete of 23.5kN/m3, 1.3kPa super imposed dead loads, curtains walls and glazing's of 0.4kPa and live load of 3kPa. The time step used in the analysis was 0.002s. A sensitivity study was carried out to ensure this time step was of reasonably accuracy.

2.3. Time History Records

Four time history records were used to investigate the applicability of the pESA method. These records were; Lucerne (Landers, California), Izmit (Kocaeli, Turkey), La Union (Michoacan, Mexico), El Centro (Imperial Valley).

Twelve time history records (six, each with a north and south component) were used 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 have 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 [9]. The earthquake records chosen according to these recommendations were; Lucerne (1992), Izmit (1999), La Union (1985), El Centro (1940), Llolleo (1985), Tabas (1978). The records chosen for the Auckland region were; El Centro (1940), Delta (1979), Kalamata (1986), Karinthos (1981), Matahina (1987) and Bovino (1980).

The records were scaled according to the New Zealand Loadings Standard [14] to match the site specific design spectra. In this analysis the k_1 (local) factor was used, the k_2 (family) 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. Inertia 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.

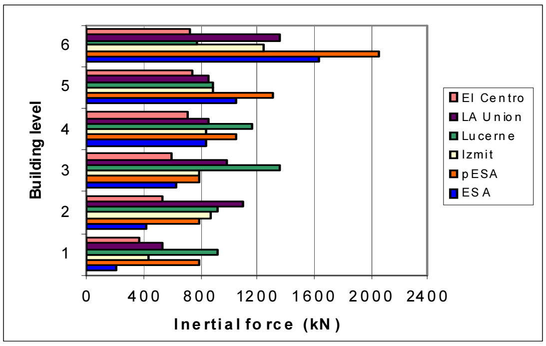

Figure 3 pESA and ESA results compared to inelastic time history results for a 6-storey structure

This figure indicates that the ESA method underestimates the forces in the lower levels of the structure. This result is in accordance with what past researchers have found. Further analyses were carried out on 3, 9 and 12 storey structures. The results for these structures indicated that the pESA method 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.

3.2 Transfer forces investigation

The magnitude of transfer forces for these analyses was measured by the magnitude of axial force in the spring element between the wall and the frame. This axial force provides a measure of transfer forces as this element provides the only connection between the frame and the wall elements. Comparisons were made for a variety of different parameters to determine how transfer forces are affected by these parameters.

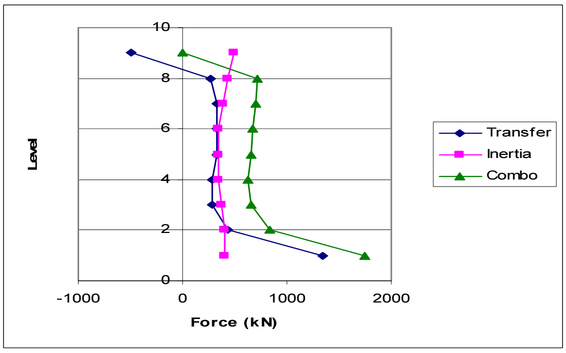

Figure 4 Comparison of inertia and transfer forces for SR1:43 and structural ductility of 3

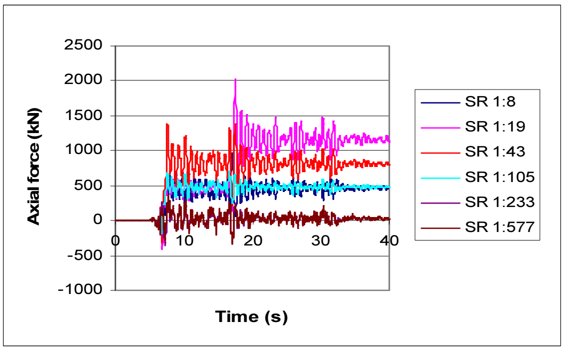

Figure 5 Transfer forces for various frame-wall stiffness ratios at level 1 for 9-storey structure

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 shows that for this structure transfer forces are many times larger than inertial forces at the base and the top 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 shapes which occur for the wall and the frame elements. For the lower levels of the structure the wall is restraining the frame to deform like the wall whereas for level 9 the frame is pulling the wall. This change in sign occurs due to the change of the dominant (stiffer) lateral force resisting element with height of the structure.

The results shown in Figure 5 are results indicating the trends and affects on transfer forces for an inelastic structure due to different frame-wall 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 SR 1:43. This occurs as both the wall and the frame elements are of similar stiffness; therefore 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 with little resistance and therefore lower transfer forces are observed.

This figure also indicates for stiffness ratios less 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 which results from the formation of a plastic hinge at the base of the wall. These forces arise due to the frame continually trying to resist the permanent drift which is imposed by the wall. These forces are extremely important and they should be incorporated into the design of the structure. It was also observed that smaller transfer forces existed for the upper levels of the structure. This is because at these levels little inelastic action occurs, therefore only small transfer forces develop.

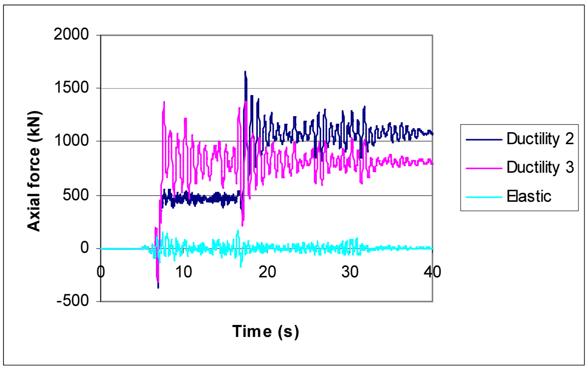

Figure 6 Variation of transfer forces for different structural ductility levels at level 1 of the structure

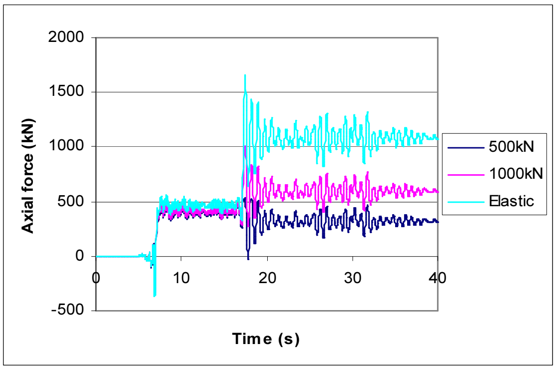

Figure 7 Affect of different floor diaphragm strengths on transfer forces

Figure 6 indicates there is a major difference in the magnitude of transfer forces between elastic and in-elastically responding structures. The forces observed for the inelastic structure are of the order of ten times larger than the forces obtained from elastic analysis at level 1. Under most reasonably sized earthquakes, inelastic behaviour will occur in the structure. Also it is shown that large residual forces develop for in-elastically but not for elastic responding structures. Therefore, this highlights the importance of considering the inelastic action within the structure when considering transfer forces. The differences in transfer forces for level 9 of the structure were found to be much less as little inelastic action occurs at this level.

Figure 7 indicates the trends found due to changing the strength of the frame-wall connection element (floor diaphragm). This figure indicates that magnitude of transfer forces is found to reduce with reducing floor diaphragm strength or increase inelastic action within the floor diaphragm. When the connection becomes inelastic between the wall and the frame elements the elements can deform more independently. This therefore results in lower transfer of forces between the elements. 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.

3.4 Findings from Sensitivity Study for the Transfer Force Analysis

The sensitivity study on foundation compliance highlighted the importance of including foundation effects when studying the magnitude of floor forces. Figure 8 shows the differences in the maximum average envelope floor force values for a model with and without foundation considerations.

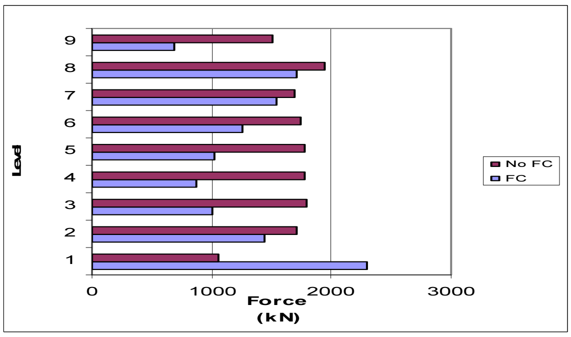

Figure 8 Comparison between maximum total floor forces for a rigid and complex foundation assumption

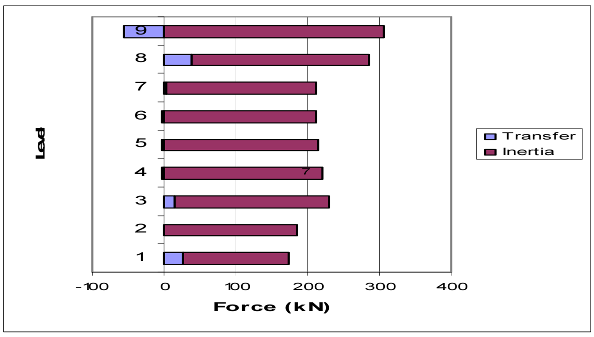

Figure 9 Transfer and inertial forces for low seismic level earthquake

Figure 8 shows a comparison of the total floor force results for structures with a rigid (NFC) and a complex foundation (FC) model. It can be clearly seen that the total forces at level 1 for the complex foundation model are much greater than for the rigid model; these forces are less for the other levels in the structure. The difference in the magnitudes of floor forces occurs due to the larger displacements which occur in the complex foundation model compared to the rigid foundation model.

Figure 9 provides a comparison between the transfer and inertial forces for a 9-sotrey structure with a frame-wall stiffness ratio of 1:43 in a low seismicity region. These results indicated that the size of the earthquake has a large effect on the type and magnitude of forces present in the floor diaphragm. This figure clearly shows that the transfer forces are reasonably negligible in comparison to inertial forces. This is due to the flexible elements used in the design of low seismic region structures.

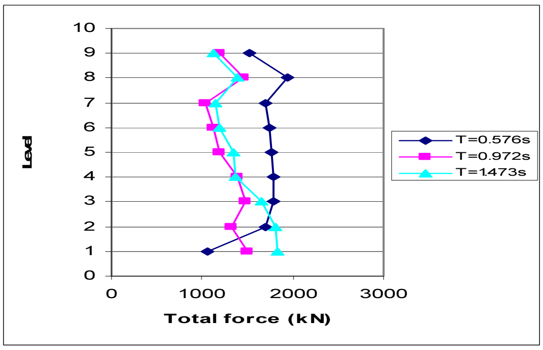

Figure 10 Comparison of total floor forces for frame to wall structures with the same stiffness ratios but different fundamental time periods

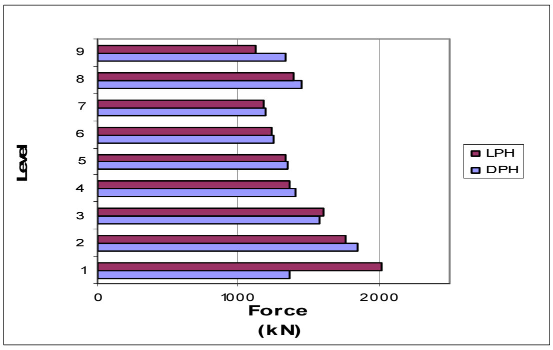

Figure 11 Comparison between total floor forces for models with lumped and distributed plastic hinge representations

Figure 10 provides a comparison between the floor force envelopes for structures with different fundamental time periods and similar stiffness ratios. This figure indicates that the distribution of floor forces changes for structures with different fundamental time periods. For the large period structures the forces were found to be more similar. This is due to lower relative change in the spectral accelerations. Larger forces are observed at level 1 for the structures with larger periods. This is due to the structures with larger periods deforming more which results in the development of larger transfer forces at this level.

Figure 11 provides results from the investigation of the sensitivity of the lumped plastic hinge assumption. These results indicated that it is very important to consider the distributed plasticity in the wall for these structures. Transfer forces develop from the different displacement patterns of the wall and the frame elements for these structures. These two plastic hinge models (lumped and distributed) for the wall result in different deflection patterns, therefore affect the magnitude of total forces in the floor diaphragm. The comparison shown in Figure 12 is reasonably typical for all structures with a reduction of total forces for the model with the distributed plastic hinge.

The sensitivity study on shear deformations indicated that when analysing these types of structures shear deformations should be taken into account if the structure is susceptible to shear deformations. When shear deformations occur at the base of the wall two situations can eventuate. The first being that the wall softens which inturn reduces the magnitude of transfer forces which develop in the floor diaphragms especially at level 1 of the structure. The second being that when the wall softens larger wall displacements occur which induces larger transfer forces at the lower levels of the structure. Generally if small shear deformations develop the first situation will occur and if large shear deformations develop the second situation will occur.

4. CONCLUSIONS

This research has lead to some interesting findings with regards to the magnitude and type of forces which develop within floor diaphragms. A summary of the main findings of this research is provided below;

- pESA method is found to adequately predict the inertial forces which develop within floor diaphragms.
- Transfer forces are found to be many times larger than inertial forces.

- Transfer forces are largest for frame-wall structures where the frame and wall elements are of similar stiffness as each element can resist the deformations by the other element. The forces are found to be smaller when the stiffness ratio for the frame-wall structure is either small or large as the more flexible element provides little resistances to induce the transfer forces.
- Inelastic analyses give very different results to elastic analysis in terms of transfer forces and residual forces in the structure. This indicates that inelastic analyses should be carried out for these structures.
- Transfer forces are found to be smaller for the upper levels of structures where little inelastic action occurs.
- It is found when the inelastic action is allowed to occur in the connection elements between the frame and wall (the diaphragm) the magnitude of transfer forces and residual forces reduce.
- The sensitivity study indicated that it is very important to consider the flexibility of the foundation to accurately determine the floor diaphragm forces.
- It was shown that the magnitude of transfer forces for low seismic regions are negligible due to the flexible nature of the structure.
- It is found that in structures with walls shear deformations should be accounted for in the model.
- It was found the lumped plasticity assumption for wall elements leads to inaccurate results. This assumption affects the displacements of the wall element which in turn affects the development of transfer forces.

This research has provided information on the trends of forces in floor diaphragms and also information on the effects that some modelling assumptions have on the magnitude of forces which develop within floor diaphragms.

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