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EFFECTS OF OUT OF PLANE STRENGTH AND STIFFNESS OF COMPOSITE FLOOR SLABS ON THE INELASTIC RESPONSE OF ECCENTRICALLY BRACED FRAME

A. Momtahan[1] and C. Clifton[2]

ABSTRACT

The Eccentrically Braced Frame (EBF) structure is a cost-effective and commonly used seismic resisting framing system that can satisfy strength, stiffness and ductility requirements for high seismic zones. The high elastic stiffness is provided by the bracing members and high ductility capacity is achieved by transmitting the brace axial forces to another brace, or to an adjacent column, through a short length of beam known as an active link that is typically designed and detailed to undergo inelastic response in shear in a severe earthquake. The inelastic deformation of the active link can result in permanent deformation and building residual drift. The Christchurch earthquake series from 4 September 2010 to 23 November 2011 comprised six damaging earthquakes. In particular, the Mw 6.3 Earthquake on 22 February 2011 severely impacted the city, with significant damage to residential and commercial structures. Detailed analyses of the strong motion data recorded indicated that the intensity of this event was 2 to 3 times more than the Ultimate Limit State (ULS) values specified by the New Zealand seismic loading standard (i.e. NZS 1170.5) over the period range of 0.5 to 4 seconds.

These large seismic forces were able to load EBF structures into their inelastic range; the first time worldwide that this has happened. Despite this, the EBF building structures have performed significantly better than would have been expected given the intensity of loadings. From the total of four EBF building structures that were investigated in detail after the earthquakes, it was evident that almost all the active links were strained into their inelastic range. The active links exhibited paint flaking and Lüders lines. However, they were free of residual distortions. It was also reported that in most cases less than 50% of the cumulative strain capacity of the yielded active links was utilised and therefore the active links had sufficient post-earthquake capacity not to require replacement. Comparison of the measured inelastic response of the 12-storey Club Tower building for 22 February 2011 event (based on sliding marks on the stairs) with the predicted response of the design models, showed that this building exhibited an apparent stiffness increase of between 2 to 2.5 times that of the structural model. The results have also indicated that the building has post-earthquake residual drift of only 0.14%. This indicated that this building has self-centred to within construction tolerances, despite the severity of the event. That self centring is particularly interesting as there was specialist self centring systems installed in the building. All investigated EBF structures had been constructed compositely with the floor slabs. Floor slabs have also shown very minor damage in the form of hairline cracking in the areas directly above active links; no repair of these floors has been required. This led to consideration that the observed increase in the lateral stiffness in the EBFs in Christchurch and enhancement of their active links in the resistance to yielding, is in part due to the out-of-plane stiffness of the floor slabs to which the collector beams are compositely connected.

In this study, the slab contribution has been quantified through an advanced numerical model of a single

[1]Senior Structural Engineer, Stiles and Hooker Ltd, Hamilton New Zealand
[2] Professor, Department of Civil, and Environmental Engineering, University of Auckland, New Zealand

EBF frame with composite floor slab. Numerical time-history analyses of a 10-storey prototype building with and without slab contribution have also been carried out to establish the increase in lateral stiffness and level of reduction in residual drift of the EBF buildings when composite slab contribution is taken into account. This contribution is significant, especially when considered in conjunction with the beneficial impact of Soil Foundation Structure Interaction (SFSI) also reducing the open field intensity of seismic actions into the building superstructure. The slab contribution to seismic loading levels with and without SFSI is presented.

Introduction

Expected building performance and the benefits of self-centring

In the past few decades, the seismic design concept has focused on developing structural systems, which are capable of maintaining a specified level of inelastic deformations, as well as developing stable ductile mechanisms through inelastic response of specific regions in the structural systems. The main objective of this concept is to ensure the occupants' life safety during a severe earthquake, while allowing controlled damage to occur. It avoids the need to design for the very high level of seismic loads associated with elastic response to such an event, which would require very large member sizes in the superstructure and foundations for an event with a very low probability of occurrence in the lifetime of the building.
While controlled damage is acceptable in the low probability Ultimate Limit State (ULS) event, to meet the objectives of the current seismic codes for medium to high-rise buildings, in a much higher probability Serviceability Limit State (SLS) event, non-structural components should suffer no damage and the building must remain functional. Structural damage is typically avoided by providing the structure with sufficient strength at the SLS to remain elastic and with sufficient lateral stiffness to prevent deformation that would cause non-structural damage. In the ULS, under very infrequent and severe earthquakes, structural and non-structural damage is expected, but life safety must be preserved through prevention of collapse of the structure and preservation of the means of egress.

While controlled damage is acceptable in the low probability Ultimate Limit State (ULS) event, to meet the objectives of the current seismic codes for medium to high-rise buildings, in a much higher probability Serviceability Limit State (SLS) event, non-structural components should suffer no damage and the building must remain functional. Structural damage is typically avoided by providing the structure with sufficient strength at the SLS to remain elastic and with sufficient lateral stiffness to prevent deformation that would cause non-structural damage. In the ULS, under very infrequent and severe earthquakes, structural and non-structural damage is expected, but life safety must be preserved through prevention of collapse of the structure and preservation of the means of egress. In practice, as demonstrated with the initial September 4th 2010 earthquake in the Christchurch 2010/2011 earthquake series and within the Wellington CBD in the 2013 Seddon earthquake, an event may be greater than the SLS level and less than the ULS level. This may cause non-structural damage requiring repair, and/or minor structural damage, which will not generally require repair. Finally, there is a very low probability of an event greater than the ULS level, called a Maximum Considered Event (MCE), which will cause considerable structural damage for which the building should not collapse. The 22 February 2011 earthquake was such an event within Christchurch CBD, although of shorter duration than typically expected for an MCE, while the entire earthquake series delivered the duration of an MCE event.

Client experiences from recent severe earthquakes, such as Northridge 1994, Kobe 1995, Loma Prieta 1998 and Christchurch 2010/2011, are that the controlled damage philosophy is not preferred and that they want rapid return to use of their buildings. If it is multi-storey residential then occupants may want to stay in their buildings. This requires a low damage design solution and to keep the residual drift of the buildings below a specified level. This concern has led the researchers to focus on developing structural systems with active self-centring characteristics allowing the structure to return to its original position after an earthquake, recognising that self-centring is important for the post-earthquake functionality of the building. Without effective self-centring the building is likely to require demolition even if the damage is repairable.

Eccentrically braced frames (EBFs)

The eccentrically braced frame structure was introduced as a framing system that is highly ductile, having full hysteresis loops similar to Moment Resisting system Frames (MRF) and an overall plastic mechanism of response, and stiff similar to concentrically braced frames (CBF). This beneficial combination inspired researchers to continue their focus on EBF structures as preferred structural systems, for satisfying the objectives of the modern seismic design philosophy at a moderate expense.

As discussed above, for successful re-occupancy of a building, it is advantageous for the lateral load resisting system to re-centre. However, there has not been mention in the literature regarding the tendency of EBFs to return to its initial position after a seismic event. This is owing to the fact that seismic energy is dissipated through the large plastic deformations of active links in EBF building structures. This potentially results in large residual deformation in link beams and potentially severe damage to the floor slab directly above the links after earthquakes. Therefore the post-earthquake repair of EBFs with traditional active links has been deemed difficult and expensive. Mansour, Christopoulos, and Tremblay (2011) however reported that designing EBFs with replaceable shear links could mitigate these drawbacks.

Most EBFs are supporting a floor slab that is integral across the EBF bay. Experimental studies on EBF subassemblies with strips of floor slab attached (Prinz & de Castro-e-Sousa, 2014; Ricles & Popov, 1989) have shown that they increase the shear resistance of the inelastically responding active link and this is incorporated into the overstrength actions generated by the active links. However, research into the out of plane strength and stiffness of the floor slab and its influence on the overall system response have not been undertaken until after the Christchurch EQ series, commencing in 2011 as discussed in the subsequent sections.

Christchurch earthquake series and impact on multi-storey steel buildings

The construction of modern steel multi-storey buildings in Christchurch area goes back only to 1987. This means that modern steel buildings in that region were designed with the recent seismic provisions. However, detailed analyses of the strong motion data recorded showed that the intensity of the 4 Sep 2010. event was approximately 0.7 times the ULS specified by the NZ seismic loading standard over the period range of 0.5 to 4 seconds. The 22 Feb 2011 event was between 1.5 and 3 times the ULS and the largest 13 Jun 2011 earthquake was 0.9 times the ULS (G. Clifton et al., 2012). The recent Christchurch EQ series was the most intense worldwide that pushed the modern EBF building structures into their inelastic range and had an impact on recently built EBFs from 3 to 22-storey in height. Their response has been generally better than expected (G. Clifton et al., 2012). Investigation by Bruneau, Clifton, MacRae, Leon, and Fussell (2011) revealed that damages was confined to the active links of EBFs. There was reported that almost all active links were strained to the inelastic range. Evidence of inelastic deformation was flaking of the brittle intumescent paint and Luder's lines on the active links. In couple of cases fracture of the active links were also observed. The general performance of the EBF building structures in Christchurch and possible reasons of active link failures were identified and discussed in detail by Bruneau et al. (2011).


Figure 1. (Left) Paint flaking of the most heavily strained EBF link in Club Tower, Christchurch, (Right) Fractured link at level six of Pacific Tower, Christchurch

The investigation by Bruneau et al. (2011) also identified an increase in stiffness of the EBFs as well as in their resistance to yielding, when they are compositely connected to the floor system. G. Clifton et al. (2012) explains this as when an EBF system deforms inelastically it pushes the floor slab out of plane. However, the concrete floor slab, compositely connected to the beam, resists this movement owing to a high out of plane stiffness and reasonably high elastic threshold. They also reported that a comparison of measured inelastic response of the 12-storey Club Tower building (based on sliding marks on the stairs) with the predicted response of the design models showed that the building apparent stiffness was between 2 to 2.5 times that of the models. Post-earthquake measurements of the building showed residual drift of only 0.14%. This led to a consideration that the out of plane resistance of the floor slab might be a significant source of this stiffness and unexpected ability to self-centre. This is could be caused by the slab forcing the EBF's collector beams back to being horizontal at the end of the earthquake and hence the columns vertical leading to the almost complete self-centring.

This paper investigates the dynamic response of a hypothetical modern multi-storey EBF building with composite floor slabs and correlates the results with field evidences and findings from the 22 Feb 2011 and subsequent severe earthquakes.

Out of plane stiffness of composite slab

When an EBF deforms laterally in plane, the collector beams rotate in the vertical plane and cause the floor slab above to be displaced vertically. The floor slab resists this deformation, thereby increasing the systems' effective strength and stiffness. The resistance of the floor slab comes from the mesh and concrete in the slab as well as the compositely connected secondary beams (G Charles Clifton, 2006). Up until recently, there has been no research to determine the contribution of the out of plane resistance of a composite slab to the lateral stiffness of an EBF and to quantify the nature of that contribution. Ricles and Popov (1989) investigated the cyclic behaviour of composite links and participation of the floor slab. Results from this research confirmed a greater initial elastic stiffness and increased shear capacity of the links. However the contribution of the slab degraded over successive cycles of inelastic loading. The EBF tested by Ricles and Popov (1989), however had only a narrow strip of floor slab over the collector beam/active link and the full and 3D slab contribution to the out of plane resistance was not considered. The narrow strip also made the floor slab very susceptible to localised slab damage, which was much more significant than that seen in the Christchurch buildings.

An undergraduate study at University of Auckland in 2011 was carried out by Mathieson (2011) and Volynkin (2011), and initial quantification of slab out of plane resistance based on simple yield line theory was provided. In this study it was shown that the presence of slab decreases the peak residual displacement of a hypothetical, 10-storey EBF, designed to current NZ design practice (NZS3404) under a range of 10 representative EQ records, to approximately less than one third of that without the slab effect, but the peak displacement remained approximately the same. In this study the out of plane stiffness of the floor slab was overestimated by assuming that the considered yield lines fully develop within and around the perimeter of the panel zone. Preliminary investigations by the author however showed that this is highly unlikely, as secondary beams act as a stiff boundary element, which delays the propagation of the slab yielding under moderate loading.

More recently, Prinz and de Castro-e-Sousa (2014) also carried out numerical investigations on the dynamic performance of an EBF structure to quantify the composite slab contribution to the stiffness of EBF systems. The results from this study showed that the structures incorporating the slab experienced lower inter-storey drifts compared to bare steel models. It was also reported that elastic out of plane stiffness of the slab outside the link causes reduction in residual drifts and provides a kind of self-centring behaviour. They also stated that demand on link's rotation in an EBF structure is magnified by shortening of the frame bay width. Therefore, added stiffness by slab membrane action may help reduce link's rotation demand. It was however discussed by Khan and Clifton (2011) following a series of experimental tests that the links elongate as they deform in the vertical plane and this elongation becomes larger as the links plastic deformation angle increases. However, this elongation does not exceed 1.5mm in total. This level of deformation would not have any significant effect in stiffening the concrete slab and reducing link's rotation demand. Also, since the active link beam is much stiffer than the floor slab, in practice the elongation would not be quite as large as the bay width would shorten, meaning that the total amount of extension is likely to be around 1mm. This is insignificant in terms of generating the internal forces, especially there will be slab damage with this movement, which will tend to reduce the compression that can be carried by the slab across the shortening region. Hence, the shortening of the bay width has insignificance effect on increasing the shear resistance of the EBF active link. Similar to Ricles and Popov (1989), Prinz and de Castro-e-Sousa (2014) did also consider a narrow strip of composite slab above the collector beams in their investigations. This assumption would not lead to an accurate out of plane stiffness of the slab, which is provided by the 3D nature of the slab panel. While numerous researches and studies have investigated the seismic performance of EBF structures, the majority of them either neglect the concrete slabs contribution or replace the concrete slabs by rigid-diaphragm constraints.

Modelling of the composite slab

To obtain the out of plane stiffness of a floor slab, two typical slab panels situated over the northern and eastern braced bays of a hypothetical 10-storey EBF building were modelled in 3D in Vulcan. Vulcan is a FEA program developed in the University of Sheffield, England, to mainly model the behaviour of composite floor slabs. Vulcan allows a rapid input of composite floor slab comprising steel beam network. The inelastic behaviour of composite floor slab in Vulcan has been validated through number of experimental tests and numerical modelling in both overseas and NZ.

A typical plan of this building is shown in Figure 3. The composite concrete floor slab comprised of a 130mm

thick trapezoidal slab with 0.95mm thick metal decking.
The link beams were omitted from the models in Vulcan, to solely capture the out of plane stiffness of the concrete slab. Different number/spacing of the secondary beams as well as different mesh sizes in the floor slab were considered in the analyses to explore the effects of these parameters on the response of the slab to out of plane deflection. To achieve the cyclic response and behaviour of the floor slab, a saw-tooth loading regime was applied to the models.

Cyclic response of composite floor slab to out of plane deflection

The analysis results from Vulcan revealed that a slab when the secondary beams span perpendicular to the active link, provides a greater resistance to vertical deformation compared to when the secondary beams span parallel to the active link. This resistance for the secondary beam parallel case also increases when the spacing between the secondary beams decreases. Due to the composite action between the steel beams and floor slab, when the collector beams are moved out of plane the secondary beams and the slab will be also moved, with maximum vertical movement at the end of the active link and minimum movement at the supporting column line.

Crack patterns over the floor slab, in the case where the secondary beams span parallel to the active link, indicated that the majority of the slab yielding occurs between the collector beams and the adjacent secondary beam and only minor yielding developed beyond that. This means that the virtual work done by the slab due to out of plane deformation is concentrated within this zone. In this case, the curvature of the yieldlines developed in the floor slab is larger than when slab spans parallel to the active link. However, this

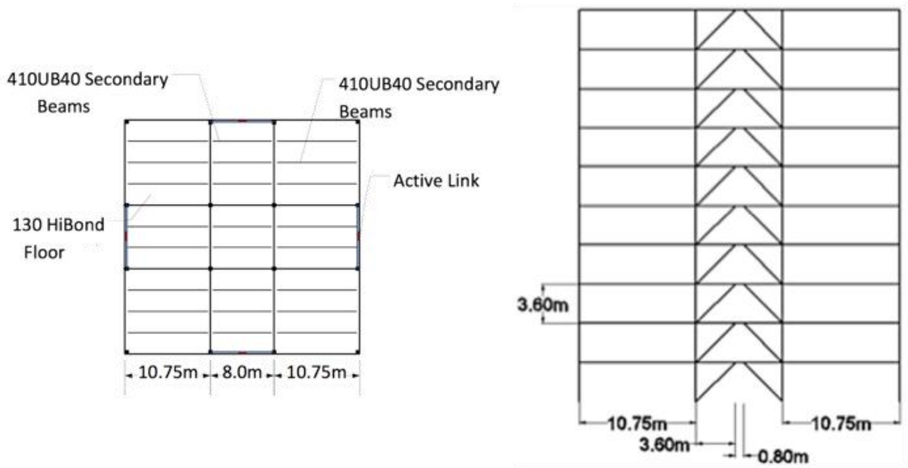


Figure 3. Floor plan and elevation of hypothetical 10-Storey EBF building

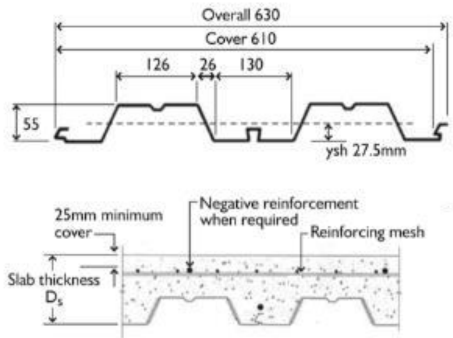


Figure 4. Metal decking properties

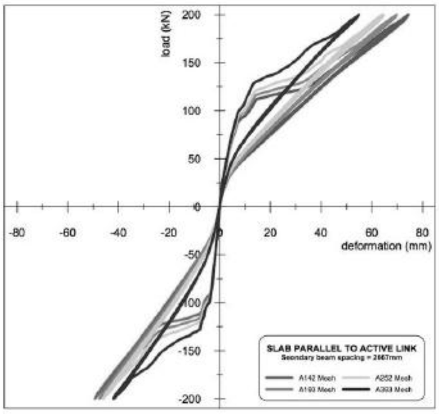
does not result in additional resistance, as the flexural strength of a yieldline does not increase by curvature at least for a simple biaxial reinforcement stress-strain relationship, which is appropriate to use. This is because when a floor slab yields there is a minimal strain hardening associated with the steel reinforcing mesh and therefore as curvature of the slab increases the flexural strength will not significantly increase to a greater extent. (Reinforcement has been engineered to have a ratio of ($f_y/f_u \geq 0.9$).

A total number of 16 cases were modelled using Vulcan to generate the cyclic response of floor slab to out of plane deflection and its sensitivity to the spacing and direction of secondary beams as well as reinforcement ratio in the slab. The results from these analyses are plotted in Figure 5. The results suggest that increase in mesh sizes in the slab increases it's out of plane stiffness. This increase is however slight compared to when the number of secondary beams increases. It is evident that elastic stiffness of the slab remained unchanged when different mesh sizes were used. An increase in the out of plane stiffness, even slight, occurred after the floor slab yielded. This is attributed to the fact that a greater force was required to bring a mesh with a larger area into yield. The inelastic responses of the slab obtained from Vulcan were used in Numerical Integration Time History Analysis (NITHA) of the hypothetical 10-storey EBF building to investigate its response when the out of plane stiffness of the slab is taken into consideration.

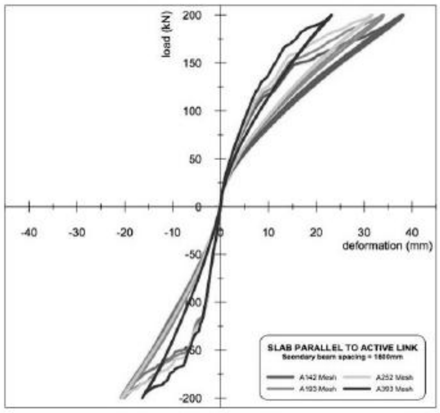
Effect of the floor slab resistance on dynamic behaviour of EBF building structures

Details of the analysis model

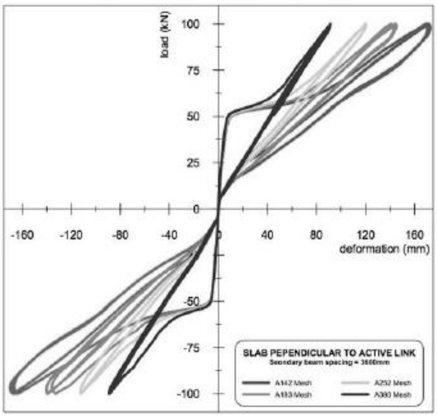
The main objective of this study was to make an assessment on the effects of the out of plane stiffness of the composite floor slab on entire building performance. As such, peak and residual drifts, peak and residual displacements were used as performance indicators of the hypothetical 10-storey EBF building structure were compared to determine the influence of the slabs on general building performance and the tendency to self-centre.



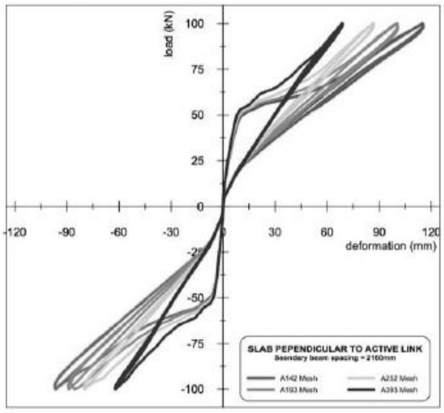
(a) Slab parallel to link beam, 2 secondary beams



(b) Slab parallel to link beam, 4 secondary beams



(c) Slab perpendicular to link beam, 2 secondary beams



(d) Slab perpendicular to link beam, 4 secondary beams

Figure 5. Out of plane response of composite slab in an EBF system

The building was designed according to NZS1170.5 (SNZ, 2005), NZS3404 (SNZ, 2007) and HERA R4-76 (G C. Clifton & Feeney, 1995), which are the current NZ design provisions, using capacity design principles. The secondary members were sized to withstand the forces generated by fully rotated and strain hardened active links. In this way the inelastic demand was confined to the links with the rest of the elements remaining essentially elastic. The building was assumed to be located in the high seismic zone of Wellington with soil type C and was designed for a ductility factor of 3. The building was modelled in 2D in RUAUMOKO (Carr, 2004) due to the programme's flexibility in performing NITHA. The slab's out of plane resistance to vertical movement was modelled using translational springs and a dummy beam (Figure 6). The springs represented the resistance to the upward and downward motions at either end of the link, and were attached from the ends of the collector beam to a dummy beam. The dummy beam provided a node for the slab spring to connect to and also transferred loads to the columns. The dummy beam was attached to the columns with pinned connections and given a very high stiffness so that it did not itself contribute to the

structural performance of the system. Note that nodes 1 and 2 shown in Figure 6 are coincided but are shown apart herein for clarity.
The results Vulcan (Figure 5) used to model the relationship between load and displacement of the floor slab springs. Results obtained from Vulcan suggested that except from the first cycle, the response of the modelled slab was almost bi-linear. This is because once the slab has yielded in the first cycle the post-yielding curve maintains two levels stiffness with the initial stiffness being greater up to a relatively low level of load. This will have a significant influence on dynamic self-centring. However the higher strength and stiffness of the first cycle of loading needs also to be included.

Seven EQ records, as listed in Table 1, were used in the NITHA. The EQ records were selected based on their similar seismological signature for the given building location (Oyarzo-Vera, McVerry, & Ingham, 2012). In accordance to procedure outlined in NZ1170.5 (SNZ, 2005), the scale factors worked out to ensure the records match the design spectrum and that the total energy of at least one of the records is greater than the spectrum's over the period range of interest.

Results and discussion

The period of the building with the slab considered as a numerical diaphragm (infinitely stiff in plane) was T1 = 2.36sec. With Slabs, the period was T1 = 2.32sec. This was sensible as the building was slightly stiffer with the links effectively stronger.

Force resisted by the floor slab

The maximum force resisted by the floor slab when spans parallel to the active link was 590kN when deflected downwards and 400kN when deflected upwards. These values were 100kN and 90kN respectively when slab spans perpendicular to the active link. The shear-yield capacity of the active links in the building ranges from 1800kN at the bottom floor to 620kN at the top. This indicates that the floor slab contributes more effectively to seismic resistance towards the top levels of the building where the ratio of the link shear stiffness to floor slab resistance is the lowest. It is interesting to note that this is consistent with the pattern of links plastic demand in the 12-storey HSBC building in Christchurch. This building comprises 10-storeys of open plan office on top of a two storey massive raft foundation. The inelastic demand was greater from levels 3 to 5 and reduced from level 8 to the top, despite the uniform and minimal contribution from the non-structural walls. This will be due at least in part to the proportionally higher lateral resistance in the top levels generated by the floor slab.

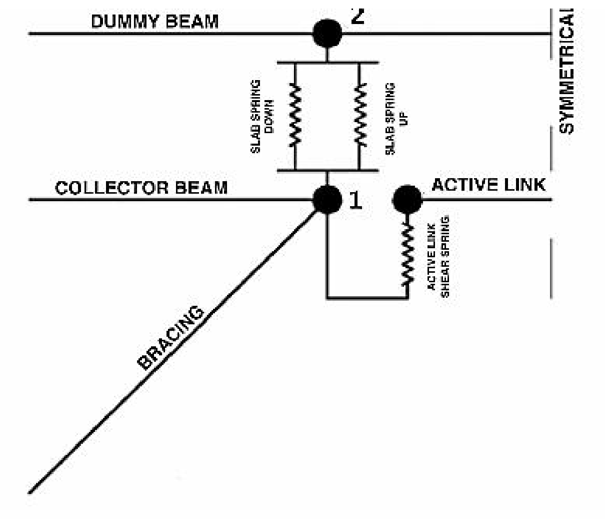


Figure 6. Active link junction used in model

Table 1. Earthquake ground motion record used in NITHA

Record Name	Country	Date	Magnitude (Mw)	Near Fault Effect	Scale Factor
Arcelik	Turkey	17/08/99	7.5	Yes	3.00
Duzce	Turkey	12/11/99	7.2	Yes	0.59

El Centro	Imperial Valley	19/05/40	7.0	No	1.21
Hokkaido	Japan	26/09/03	8.3	No	0.78
La Union	Mexico	19/09/85	8.1	No	1.92
Lucerne	USA	28/06/92	7.3	Yes	0.75
Tabas	Iran	16/09/78	7.4	Yes	0.47

Demands on secondary members

An assessment of the demands on the secondary members was made by observing the member actions that were more than 5% higher than it was with “No Slabs” to avoid interpreting small fluctuations as an increase in force. The analysis results suggested that demands on the secondary members increased when the slab’s out of plane contribution was considered. This was much higher at the lower levels for all cases. Since the active link and floor slab act compositely together to resist the out of plane movement of the collector beams, their associated stiffnesses are additive. The increase in stiffness of the primary seismic resisting system therefore causes additional force to be distributed to the secondary members. Among all secondary members, braces were the most common members on which the highest increase observed.

It is therefore recommended that a magnification factor be developed for use in EBF design to take into account the additional stiffness of the active links due to out of plane resistance of the floor slab. This factor should be applied in conjunction with the overstrength factor, which incorporates variability in material strength as well as strain hardening.

Peak displacements

The results showed that the maximum floor displacements in all cases are approximately comparable for both “Slabs” and “No Slabs” conditions. This means that the building’s peak displacement during an earthquake is not affected by the out of plane stiffness of the floor slab. The slight difference in maximum floor displacements was due to the increased axial demand on the columns and bracing elements when the slab contribution is taken into account. This reduced column respective moment capacities and therefore increased peak rotations. The peak displacements for the EBF building shown in Figure 7 and 8.

Residual drift and displacement

Residual drift is an important parameter to decide if a structure is able to be repaired or demolished after the occurrence of an intense earthquake. According to the literature repairing damaged structures with residual drift of greater than 0.5% is not financially viable. Experience from the Christchurch earthquake however puts this desirable residual limit to less than 0.3% and, even at a residual drift of 0.1%, some repair such as lift guide rail realignment is required (C. Clifton, Bruneau, MacRae, Leon, & Fussell, 2011). Therefore, for successful re-occupancy of a building, it is advantageous for the lateral load resisting system to re-centre.

A comparison of the results between the “No Slabs” and “Slabs” conditions showed a decrease in the residual drift and displacement of the building for most ground motion records. The reduction was generally minor or even none in the first few levels. This is attributed to the fact that at these floors the contribution of the slabs is relatively minor because the added stiffness from the slab springs is small compared to the stiffness of the active links. For the floors above, the relative amount of added stiffness due to the floor slabs is greater which causes a greater reduction in residual drift at these levels. Comparing the results suggested a larger reduction in residual drift when slab runs parallel to the link beams. This is clearly because the out of plane resistance of the slab was greater for this condition. This increase in the slab spring stiffness generated a more noticeable drop in the residual drift at the first few storeys. These results also confirmed that influence of the mesh sizes, used in the floor slab, is negligible on the out of plane stiffness and hence on seismic response of the building. In contrast, the spacing of the secondary beams had a substantial influence in reducing the residual drift and displacement of the building.

It is should be noted that since the slab spring stiffness on every level is the same, the reduction in the residual drift per story is closely proportional to the displacement experienced at that floor.

For some records, the “Slabs” case showed a different pattern of residual drift/displacement to other. This is due to the nature of the inelastic action that occurs in the model during a numerical integration time history analysis. The response of the structure is largely dependent on when the inelastic action first occurs, as the building will respond differently from that point on. The residual displacement for the hypothetical EBF building shown in Figure 9 and 10 clearly displays the tendency of the building to go back to its original position when the out of plane stiffness of the floors lab is considered. This pattern of decreasing residual displacements was observed for all EQ records when transitioning from “No Slabs” to “Slabs”.

Building performance and SFSI effects

It is well recognised that the seismic response of a structure is significantly influenced by the interaction between the structure, foundation and underlying soil. This interaction is referred to as SFSI in the literature and used to evaluate the collective response of a structure during a ground shaking.

This interaction assumes that the structure and supporting foundation form a single entity. For so long, soil-structure interaction (SSI) is often adopted to investigate the seismic response of structure-foundation systems. This is based on the assumption that the interaction between the soil and the foundation is linear elastic (Storie & Pender, 2013). To better see the real benefit of interaction between structure, foundation and soil some nonlinear soil behaviour must be mobilised (Pender, 2014). For shallow foundations, SFSI may involve some uplift of the foundation as well as yielding of the foundation soil during large earthquake shaking. Uplift and soil yielding can have a significant influence on the earthquake response of buildings on shallow foundations.

To investigate the potential influence of SFSI in the earthquake performance of multi-storey buildings supporting by shallow foundations, Storie and Pender (2014) modelled generic 5, 10 and 15storey buildings. The buildings were intended to represent typical multi-storey buildings (supported on shallow foundations) that found in the Christchurch CBD to perform satisfactorily during the Christchurch earthquakes. These building were modelled as equivalent SDOF systems, with 5% damping, using the recommendations outlined by Priestley (2007). A bed of nonlinear vertical springs was used to capture interaction between the foundation and the underlying soil during earthquake.

Since the peak acceleration of the lumped mass in the generic SDOF systems during an earthquake time history could be a very good indication for the order of magnitude of force that the systems are subjected to Storie and Pender (2014) compared this earmark for the fixed-base structures with that of the structures where nonlinear SFSI was included for the Christchurch EQ records. The comparison showed that SFSI significantly reduces the peak acceleration of the structure in all cases.

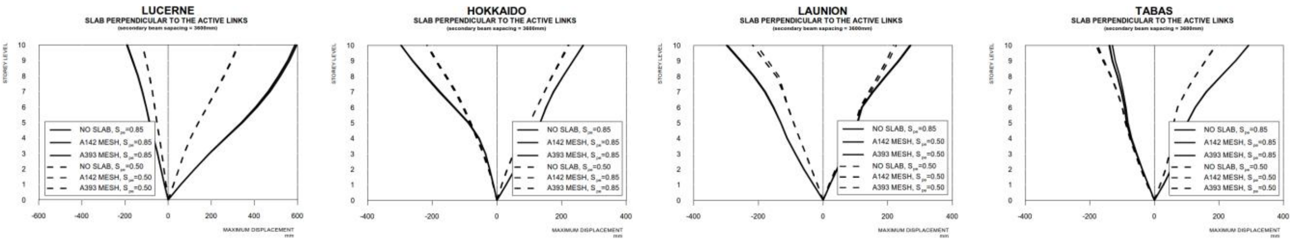


Figure 7. Comparison of peak displacement for slab perpendicular to the active link, 2 secondary beams, with and without the SFSI effects (i.e. s_p=0.5 and s_p=0.85)

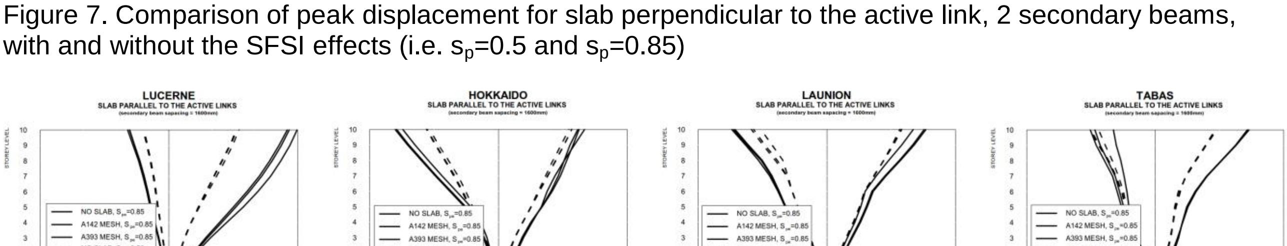


Figure 8. Comparison of peak displacement for slab parallel to the active link, 4 secondary beams, with and without the SFSI effects (i.e. s_p=0.5 and s_p=0.85)

A comparison between the observed interstorey drift in the North-South direction of the HSBC tower in the 22 Feb 2011 Christchurch earthquake and the expected response of the structural model to this earthquake showed that the actual structure had an apparent stiffness 2.3 times that of the model (C. Clifton et al., 2011). The results from the analyses by Storie and Pender (2014) indicated that the SFSI influence on structures of this type could have reduced the seismic demands significantly. Results for the closest match (i.e. 10-storey superstructure) showed that the peak acceleration of the structural system is reduced from 0.78g for the fixed base case to 0.22g for the SFSI case.

The main objective in this section was to determine the influence of the out of plane slab strength and stiffness on the response of the superstructure with and without the effect of SFSI. Since quantifying the SFSI influence on superstructures founded on shallow foundation is at an early stage and the superstructure model was elastic, compared with the actual building in which the seismic resisting system underwent yielding of the primary elements on every level, an expected conservative allowance for the reduction of seismic loads into the superstructure through SFSI was made. This allowance was made by comparing the reduction in peak acceleration for the structure generated by Storie and Pender (2014) with that specified in NZS1170.5 (SNZ, 2005) for NITH through the Sp factor, and taking 50% of this difference as the effective Sp. The reduction in peak roof displacement incorporating SFSI alone was between 30% and 50%, compared with the reduction of nearly 60% observed in practice.

The effect of slab's out of plane stiffness on superstructure peak displacement was similar for the fixed base and for the SFSI cases; however the reduction in residual deflections was greater for the SFSI case than for the non-SFSI case. The results indicated that the slab out of plane strength and stiffness contribution to the building overall response was increasingly beneficial as the overall response of the superstructure decreases and so better quantification of the SFSI effects should show an enhanced contribution from the slab towards the self centring of the building compared with that found from this study.

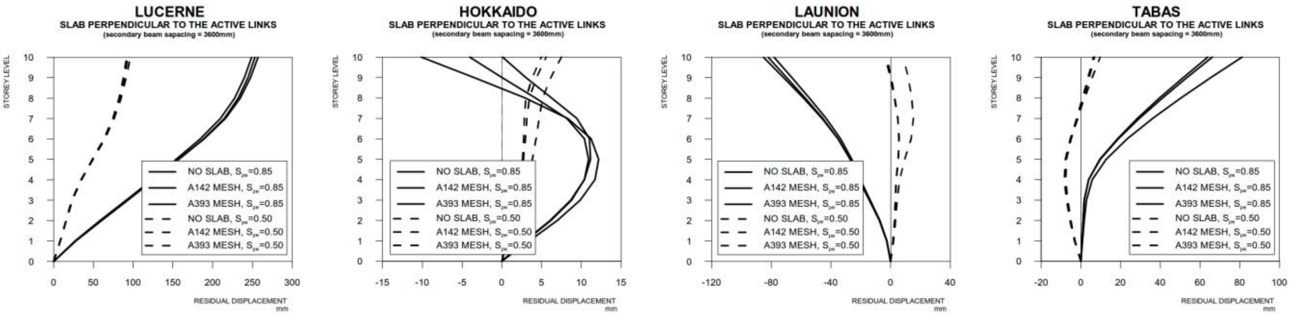


Figure 9. Comparison of residual displacement for slab perpendicular to the active link, 2 secondary beams, with and without the SFSI effects (i.e. s_p=0.5 and s_p=0.85)

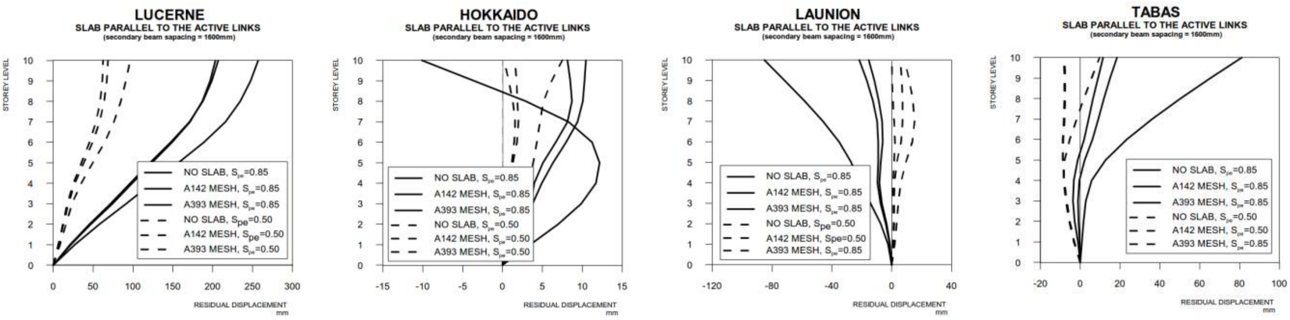


Figure 10. Comparison of residual displacement for slab parallel to the active link, 4 secondary beams, with and without the SFSI effects (i.e. s_p=0.5 and s_p=0.85)

Conclusions

From the results presented and discussed in this study, when the out of plane stiffness of the floor slabs is taken into account, the EBF building exhibits self-centring by showing a tendency to go back to its original position. The reduction in residual drift is caused by the floor slab pushing and pulling the ends of the collector beam, which force the entire structural system back towards its former alignment after the earthquake. For the majority of EQ records used in in this research, the ratio of the reduced displacement between "No Slabs" and "Slabs" cases was greater at the upper stories. This is because the influence of the slab springs was more prominent towards the upper levels of the structure where the added stiffness from the slab is comparable to the stiffness of the active links.

Observations from Northridge 1994, Kobe 1995, Kocaeli 1999 and Christchurch 2010-11 earthquakes have shown that significant nonlinear action in the soil and soil-foundation interface can be expected due to high levels of seismic excitation. Therefore, it is very important to consider the influence of soil-foundation interface nonlinearity on the response of the structures. Neglecting such effect prohibits the influence of energy dissipation due to soil yielding, large foundation deformation and foundation toppling on the structural response (Moghaddasi Kuchaksarai, 2012). It was however discussed in (Pender, 2014; Storie & Pender, 2014) that soil nonlinear deformation alone is not a significant influence on the response of the structure-foundation system, the main effect is the changes in the foundation stiffness that are induced by the detaching and reattaching from the underlying soil. This means that uplift of the foundation can have a significant influence on the earthquake response of buildings on shallow foundations. Comparison between seismic responses of fixed-base structures with that of the structures with nonlinear SFSI showed that SFSI significantly reduces the peak acceleration of the structure in all cases. A reduction in peak acceleration means that the forces transmitted to the structure are reduced and suggests improved structural performance.

In conclusion, results from this study indicate that the slab out of plane strength and stiffness beneficially contribute to the building overall response and hence building's self-centring. Also, enhanced contribution from the slab towards the self-centring of the building is expected when SFSI effects are considered. However, quantifying the SFSI influence on superstructures founded on shallow foundation is still at an early stage and more numerical and experimental analyses are yet to be implemented.

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