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Seismic Analysis of High-Rise Buildings with Transfer Slabs: State-of-the-Art-Review

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ABSTRACT: In many high-rise buildings, architectural requirements may result in a variable configuration for the vertical structural elements between the stories of the building. To accommodate such vertical elements' discontinuity, a "transfer" floor conveying vertical and lateral loads between upper and lower stories must be introduced. A drawback of the transfer floor is the sudden change in the building's lateral stiffness at its level: the structure becomes susceptible to the formation of a soft-storey mechanism under moderate to severe earthquakes. These buildings generally showed conventional elastic behavior for frequent earthquake but suffer extensive crack in the vicinity of transfer floors for rare earthquake. In this paper, a state of the art review is presented on recent publications dealing with the seismic behavior of high rise buildings with transfer floor.

1 INTRODUCTION

Recently, innovative architectural design merged with the advanced and powerful structural numerical analysis stimulated a new generation of super- and mega-tall buildings. Furthermore, discontinued vertical elements (columns and shear walls) within high-rise buildings are no longer considered as a design mistake. Consequently, the architectural demands for high-rise buildings in which columns may have different arrangement between levels become familiar. Many high-rise buildings are currently constructed with this kind of vertical irregularity where a "transfer" floor is provided to account for the discontinuous columns and/or shear walls (Figure 1).

As such, a transfer floor is known to be the floor system which supports the vertical and lateral load resisting elements and transfer their straining actions to a different underneath structural system. The transfer system itself may take the form of a transfer girders or slabs.

A major drawback of any transfer floor is the abrupt change in the building's lateral stiffness in the vicinity of its level; a direct consequence of such irregularity is that the deformation of a soft-storey mechanism under moderate to severe earthquakes or lateral wind loads imposes high ductility demands on the elements in the vicinity of the transfer floors (Zhang and Ling 2011 and Abdelbasset et al. 2014). Therefore, if this irregularity is not taken into consideration during the design stages, it becomes a major source of damage during strong earthquakes. A recent research (Al-Awady et al. 2014) pointed out to the severity of the drift in the vicinity of the transfer floor on the level of damage occurring to these buildings. This investigation showed the significant effect of the lateral flexure and shear stiffness of the vertical elements above/below the transfer level on the drift values. These findings can be correlated to the outcomes of older investigations (Li et al. 2006, Yong et al. 1999) where drifts are pronounced by reducing the stiffness of these vertical element; hence, revealing the importance of considering a reduced or a full stiffness for the columns and the shear walls in any numerical model of high-rise buildings with transfer floors.

High-rise buildings with transfer floors generally suffer no/minor cracks (conventional elastic behaviour) when subject to frequent (minor) earthquake (Li et al. 2008). However, severe cracking in the vicinity of the transfer floor is encountered when these buildings are subjected to rare (medium to major) earthquakes. Currently, reduced stiffness for cracked columns and walls is normally adopted for strength design of these buildings while full stiffness adopted for serviceability and drift design.

Here, a state-of-the art review on structural and seismic behavior of high rise buildings with transfer floors is presented. It covers the effect of transfer floor systems types and the structural irregularity classification. It also covers some codes of practice limitations for such irregular buildings. The review discusses the transfer structures local deformations and stresses concentration.

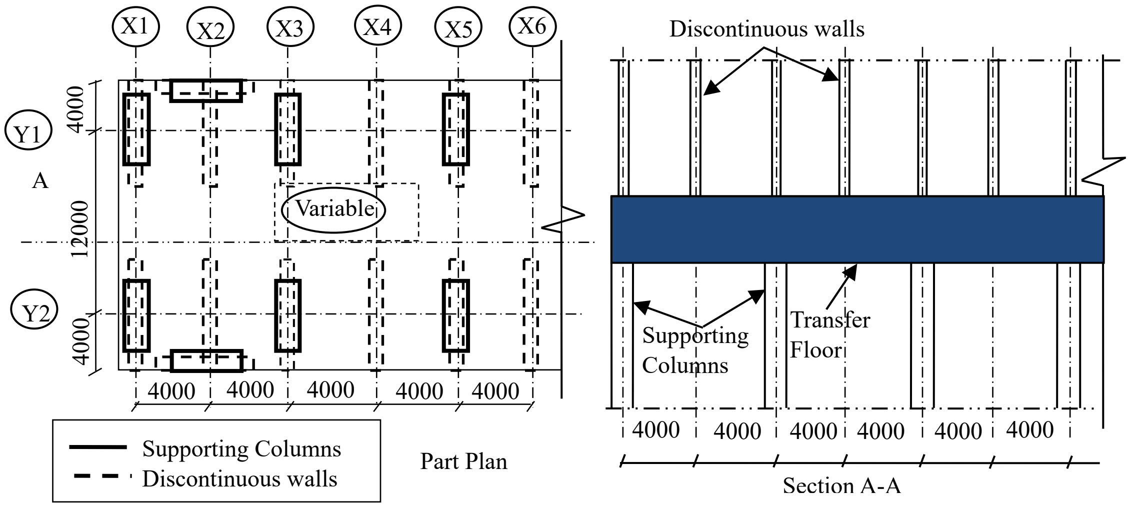

Fig. 1 Schematic part plan and part elevation for a building with transfer floor.

The review discusses the effect of the sudden change in the building stiffness on the seismic behavior of the building and outline the story drifts distribution along the building height. The commonly adopted numerical models for these irregular building are briefly outlined and the effect of the vertical location of the transfer floor with respect to the building height is presented.

2 TYPES AND EFFECT OF TRANSFER FLOOR SYSTEMS

A transfer floor transmits vertical and lateral loads from columns or shear walls above it to a different structural system underneath it. These transfer structures may be in form of transfer deep girders or transfer thick slabs. Depending on the distribution of the loads above the transfer system and the chosen type of the transfer floor system, the building is classified as either a flexural or shear structure.

The complexity of the structural behavior, in this case, is magnified by the presence of the resisting systems irregularities: an abrupt change in the lateral stiffness at the transfer floor level from a stiff system above it to a relatively flexible one below it. This creates a soft-story and violates the seismic design concept of “strong column-weak beam”. Therefore, a high-rise building with a transfer floor could be vulnerable to earthquakes. It was concluded (Yoshimura 1997) that “if first storey mechanism might occur, the collapse could be unavoidable even for buildings with base shear strength of as much as 60% of the total weight”.

In case of high-rise buildings, the severity of the collapse increases with the increase of the number of stories. This is because of the plastic energy that accumulates at the weak story of the building which increases with the increase of number of stories. Thus, control of the collapse mechanism in irregular buildings under earthquake excitation is needed especially in high-rise buildings (Dinh and Ichinose 2004).

As a result of the expected total collapse of such buildings due to its vulnerability to the lateral excitation, design codes modified the earthquake force reduction factor on the basis of the irregularity that is classified according to the geometric profile and the distribution of over strength and masses along the structure height.

3 STRUCTURAL IRREGULARITY CLASSIFICATION

Code provisions and many researchers had clearly defined the structural irregularity. Codes of practice such as UBC (1997), ASCE 7-10 (2010) and IBC (2009) share a general definition for these buildings as the structures which have significant physical discontinuities in configuration or in their lateral force resisting systems. However, ASCE 7-10 provides more details in describing this irregularity, and the corresponding instructions of dealing with these cases. According to both UBC 1997 and ASCE 7-10, vertical irregularity has to be considered in many cases; the following discussion is only related to this vertical irregularities.

Inter-storey stiffness reduction (soft-storey) is one of the major vertical irregularity sources; it exists if the lateral stiffness of one storey is less than 70% of that in the storey above it, or less than 80% of the average stiffness of the three stories above it (UBC 1997). In addition, the ASCE 7-10 defines extreme soft story irregularity in which the lateral stiffness of one story is less than 60% of that of the story above it, or less than 70% of the average stiffness of the three stories above it.

Regarding the strength, UBC (1997) provisions defines a weak story as the story with a strength that is less than 80% of that in the story above it; the story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration. ASCE 7-10 provisions are more conservative in describing the weak story aspects by adding one more ratio for the extreme weak story in which the lateral strength is less than 65% of that in the story above it.

Mass irregularity is another source which exists when the effective mass of any storey is more than 150% of the effective mass of any adjacent story; a roof that is lighter than the floor below need not to be considered as per both codes (UBC 1997 and ASCE 7-10).

Vertical geometric irregularity should be considered to exist when the horizontal dimensions of the lateral load resisting system in any story is more than 130% of that in an adjacent story. An in-plane offset of any lateral load resisting element which is greater than the length of this element should also be considered as one of the major sources of vertical irregularity (UBC 1997 and ASCE7-10).

4 CODES LIMITATIONS AND CURRENT DESIGN GUIDELINES

This section discusses the different provisions for vertically irregular buildings in both UBC 1997 and ASCE7-10; the IBC 2009 provisions correlates to the ASCE7-10. For example, UBC 1997 specifies a storey drift ratio between stories which is less than or equal to 1.3 in order to ignore the structural vertical irregularity, noting that this exception is not valid for some irregular buildings. The same ratio is found in ASCE 7-10 only when no storey drift ratio is bigger than 1.3 of any two successive storeys.

4.1 *Load Combinations*

UBC 1997 and ASCE7-10 provisions define basic load combinations for all conventional buildings; however, irregular buildings have other special load combinations for both allowable stress and strength design methods as follow:

$$\begin{aligned} UBC\,1997: \quad & 1.2D + f_1 L + 1.0E_m \\ & 0.9D + 1.0E_m \end{aligned} \tag{1}$$

where f_1 is *1.0* for public assembly and/or garage floors and for live loads in excess of *4.79* kN/m^2 and 0.5 for other live loads. *D* is the dead load and E_m is the estimated maximum earthquake force that can be developed in the structure; $E_m=\Omega_o E_h$ with Ω_o being the seismic force amplification factor that is required to account for the structural over-strength ($\Omega_o=2.0$ *to* *2.8*) and E_h is the earthquake load.

$$\begin{aligned} ASCE7\text{-}10: \quad & (1.2 + 0.2S_{DS})D + E_{mh} + L + 0.2S \\ & (0.9 - 0.2S_{DS})D + E_{mh} + 1.6H \end{aligned} \tag{2}$$

where D, S and H are the dead load, snow load and lateral earth or water pressure, respectively. E_{mh} is the horizontal seismic forces effect including the structural over-strength factor; $E_{mh}=\Omega_o Q_E$ with Ω_o being the seismic force amplification factor ($\Omega_o=1.25$ *to* *3.0*) and Q_E is the horizontal seismic forces from V or FP (equivalent lateral force procedure). S_{DS} is the design spectral response acceleration parameter at short periods.

4.2 *Equivalent Static Load Procedures*

Equivalent static lateral force procedures are permitted by both UBC 1997 and ASCE7-10 for some of the irregular structures. UBC 1997 allows using the equivalent static force procedures to irregular structures whose heights are less than 19.8 m and for all regular structures which exist in Seismic Zone 1 and Occupancy 4 and 5 in Seismic Zone 2 (A seismic zone is a structure classification which is based on risk category which is adopted by UBC 1997 and ASCE7-10 for the determination of flood, wind, snow, and earthquake loads based on the risk associated with any unacceptable performance). ASCE7-10 allows using equivalent static force procedures to irregular structures with time period $T < 3.5T_s$ and having limited horizontal irregularities where T_s is the time period at which the horizontal and descending parts of the response spectrum ($T_s = S_{D1}/S_{DS}$) which also depends on the Site Class as S_{DS} and S_{Da} are the design spectral response acceleration parameter in the short period range and at a time period of 1 s corresponding to a specific Site Class. Horizontal irregularity may be encountered in structures having one or more of the irregularity types listed in ASCE7-10 and should be assigned to a specific seismic design categories defined in the same specifications in order to comply with certain design requirements.

Structures having a flexible upper portion supported on a rigid lower portion where the average storey stiffness of the lower portion is at least 10 times the average storey stiffness of the upper portion, and the period of the entire structure is not greater than 1.1 times the period of the upper portion may be considered as two separate structures each one fixed at its base. Both portions of the structure considered separately can be classified as being a regular structure, therefore equivalent static force procedures may be adopted.

For irregular structures, both UBC 1997 and ASCE 7-10 scale the response resulting from a linear dynamic analysis such that the resulting based shear from such analysis should not be less than 100% (UBC 1997) or 85% (ASCE7-10) of the base shear resulting from the equivalent static load analysis regardless of the ground motion representation. Thus, these codes of practice require that the elastic response base shear divided by R (a numerical coefficient representing the inherent over-strength and global ductility capacity of the lateral force-resisting systems) should not be less than the base shear determined for the same building case using static force procedures. It is worth mentioning that this ratio is reduced to 80-90% for regular structures.

4.3 *System and Height Limitations*

It was agreed that structural systems are not equally responding to the same earthquake induced forces (Paulay and Priestly 1992). Structural system configuration, symmetry, mass distribution, and vertical irregularity must be considered. Structural strength, stiffness and ductility in relation to an acceptable response should be also considered. As such, design codes of practice specify many limitations to control the building systems in case of having vertical irregularities features. However, each code permits exceeding these limits under the condition of ensuring the building safety using any type of sophisticated and detailed analysis.

Columns, beams, and/or slabs supporting discontinuous walls or frames of structures with vertical irregularity should have adequate design strength to resist the maximum axial force that can develop in accordance with the load combinations including the over-strength factor (UBC 1997 and ASCE 7-10). The connections of any discontinuous elements to the supporting system should also be adequate to transmit the forces for which the discontinuous system is required to be designed.

Building's height limit for the various structural systems in significant seismic zones depends on the type of the structural system resisting the lateral loads. UBC 1997 and ASCE 7-10 allow designers to exceed these limits by a maximum of 50% only for regular unoccupied structures (i.e. inaccessible to the general public). Referring to these two codes limits, the maximum height for the irregular structures with shear walls as lateral load resisting system is 175 m if the building lies in Seismic Zones 1 or 2: this limit imposed by UBC 1997 and ASCE7-10 to the irregular structures is less than the limit of similar regular structure.

5 STRUCTURAL SYSTEM CHOICE

The selection of building configuration is one of the most important aspects of the overall design, if not the most important one, which may impose severe limitations on the structure ability to provide seismic protection (Paulay and Priestly 1992). In addition, it is found that for buildings with discontinuous distributions in mass, stiffness, and strength, the effect of strength irregularity has more significant effect than the effect of stiffness irregularity (Soni and Mistry 2006). For any two systems with the same values of the periodic time T_n and the damping ratio ξ it was agreed that both systems will have the same deformation response $u(t)$ even if one system is massive or stiffer than the other (Paulay and Priestly 1992): this argument may be not valid for irregular structures with the same periodic time and damping ratio.

For setback structures, Li *et al.* (2003) observed that for low-rise buildings with edge columns supporting the long transfer beam (Figure 2a), gravity load usually controls the design of the buildings. Even though the structural walls do not extend below the transfer structure, the column frame structure alone below the transfer structure designed to resist the gravity load is strong and stiff enough to resist the seismic load. However, using a setback columns to support the transfer beam (Figure 2b), less unbalanced end moment due to gravity load is induced in the columns supporting the transfer structure.

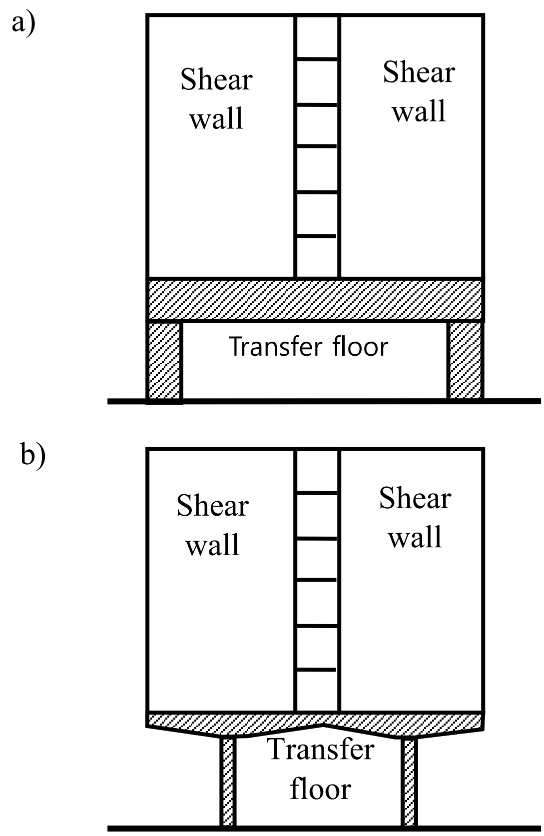

Fig 2 Low-rise buildings with columns supporting transfer beams (Soni and Mistry 2006).

Hence, columns designed to resist gravity load may not be strong enough to resist the additional seismic load. A soft-storey mechanism could be generated below the transfer storey when the building is subjected to seismic loads. So, even for a low-rise building with such irregularity, special attention must be paid to design these setback columns (Soni and Mistry 2006, Li *et al.* 2003).

Setback structures actually represent a case of combined mass and stiffness irregularities in which roof drift are larger than that for building with only one of these two irregularities (Al-Ali and Krawinkler 1998). Storey drift for the setback part of the structure is usually larger than that of floors above the transfer level; a significant increase in the drift also occurs in the vicinity of the transfer floor level.

An abrupt change in elevation may also be called a setback which normally results in concentration of structural straining actions near this level of discontinuity during the dynamic response of the building. The magnitudes of these straining actions can only be predicted via sophisticated analytical methods such as the nonlinear time history analysis and linear the push-over analysis (Paulay and Priestly 1992). It is recommended to avoid such irregularities which interfere with the natural flow of the gravity or the lateral load at the center of the building. Constant or gradually reducing stiffness of the building's storeys with height would reduce the plastic deformation concentration during severe seismic events beyond the capacities of the affected members. Furthermore, in addition to avoiding such discontinuity, it is generally advised that members' strength for undesirable plastic deformation is amplified in comparison with that for desired one: for example, shear strength of a concrete member must exceed its flexural strength to ensure that plastic shear deformations, associated with large deterioration of stiffness and strength and leading to failure, shall not occur.

Simple concepts that may summarize the plan configuration selection can be given as follow: selection of a suitable structural configuration for inelastic response and an appropriately detailed locations for inelastic deformations. Thus, capacity design philosophy for the seismic design of buildings can be adopted where potentially plastic hinge regions are identified while the remaining structure is strengthened via the adoption of the dynamic magnification factors in order to maintain elastic behaviour in these other parts (Paulay and Priestly 1992).

Xu *et al.* (2000) investigated the effects of the vertical positioning of the transfer structure on the seismic response behavior of the frame-supported shear wall structures. It was found that the degree of abrupt change in the inter-storey drift and shear concentration of the frame-supported shear walls increased with increasing the height of the transfer structure within the building. In addition, for a high-level transfer storey supported by a frame with a full elevation center core wall, more cracks and damage were found on the exterior shear walls above the transfer structure.

Further studies were performed for a hypothetical tube structures and real coupled shear wall-core wall buildings with transfer stories at various levels under earthquake loads (Geng and Xu 2002, Wu *et al.* 2007, and Rong and Wang 2004). The soft-storey phenomenon was found to be more dominant with increasing the difference in the equivalent lateral stiffness between the superstructures above and below the transfer floor in addition to the effect of the above-mentioned higher position of the transfer floor.

When transfer floor is located at a level close to 40% of the building height, soft storey phenomena may occur with the maximum inter-storey drift taking place at the transfer floor level. In this case, the mass participating in the first mode may be reduced as the upper portion of the building excites higher order modes. Su (2008) agreed with the previous conclusion and argued that the neighboring floors of a transfer storey is highly affected by the transfer floor location especially in the drift: drift demands and high mode effects are generally higher as the transfer floor is positioned at a higher level. As such, more vibration modes must be considered in any response spectrum analysis in order to improve the analysis results.

This conclusion contradicts the fact that transfer floor's huge masses located at high levels forces the structure's model to act as a SDOF system; the contribution of the first mode in such case should have been higher. However, it is agreed that abrupt changes in the inter-storey drift for the stories adjacent to the transfer floor level could be reduced with decreasing the height of the transfer floor level. Similar reduction trends in the drift is expected when the difference in the lateral stiffness between the substructures above and below the transfer floor decreases. Rotation of the resisting elements below the transfer floor can be further controlled by arranging the transfer floor located at lower floor (below 10% of the total building height) so that shear transfer above the transfer floor can be effectively reduced.

6 TRANSFER STRUCTURES' LOCAL DEFORMATION EFFECT

Transfer structures are usually idealized as deep beams or thick plates. As such, the flexural stiffness and strength of the transfer structure are much higher than those of the supporting columns or shear walls of the superstructure above (Eggert el al. 1997). Furthermore, it was argued that rotations of the irregular building's columns are higher than those of the regular one having the same plan and height dimensions and the same elements dimensions (Figure 3). It is also argued that local flexural rotations of transfer structures (Figure 4) exist and in many cases cannot be ignored. This deformation directly affect the undesirable shear concentration (Su 2008). However, the deformations of transfer structures is still being ignored and a rigid plate/diaphragm assumption is adopted in routine structural analysis of buildings with transfer structures.

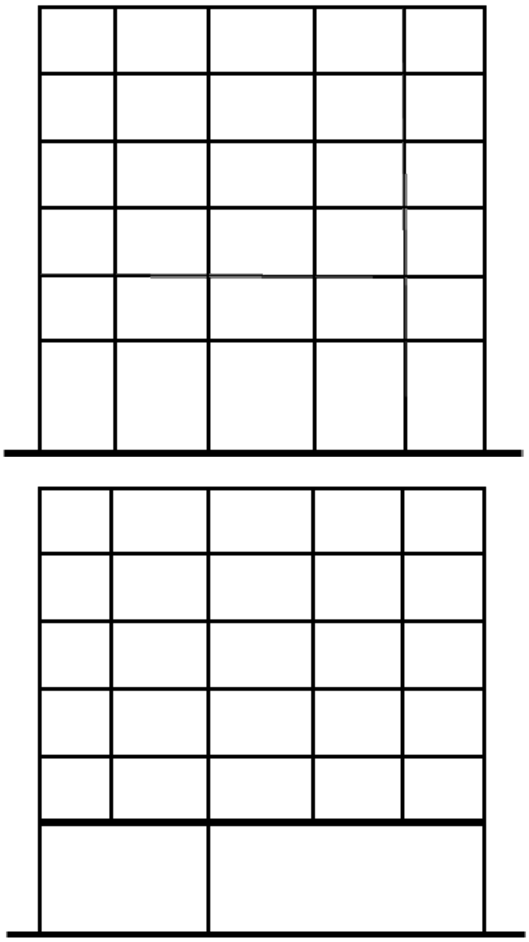

Fig. 3 Regular (left) and irregular columns (right) in a building with transfer floor.

Wu *et al.* (2007) performed a shaking table test on a 12-storey building model and found out that the actual shear forces in the walls or columns under the transfer structure are six to eight times greater than those if the transfer structure is assumed to be a rigid diaphragm. Therefore, for better prediction of the interactions between the exterior shear walls, columns and core walls, elements such as shell, beam or 3D solid elements should be adopted to model the transfer floor instead of considering it as a rigid diaphragms. In this respect, Su (2008) provided precautions to reduce the misleading effects due to the local deformation of the transfer floor. Stiff transfer structure with high flexural and shear stiffness is recommended to reduce the local deformation of the transfer system under lateral loads; thus, decreases the abrupt change in shear forces in the exterior walls. However, in this manner, higher seismic forces are harnessed and affect the transfer structure and the overall behavior of the building. Despite this measure, shear force concentrations in the exterior walls above the transfer structure can still be observed. This fact demonstrates that the effect of shear concentration is partially due to the major behaviour and interaction of a coupled core wall and shear wall structure on a restraint boundary; in other words, this effect cannot be completely eliminated.

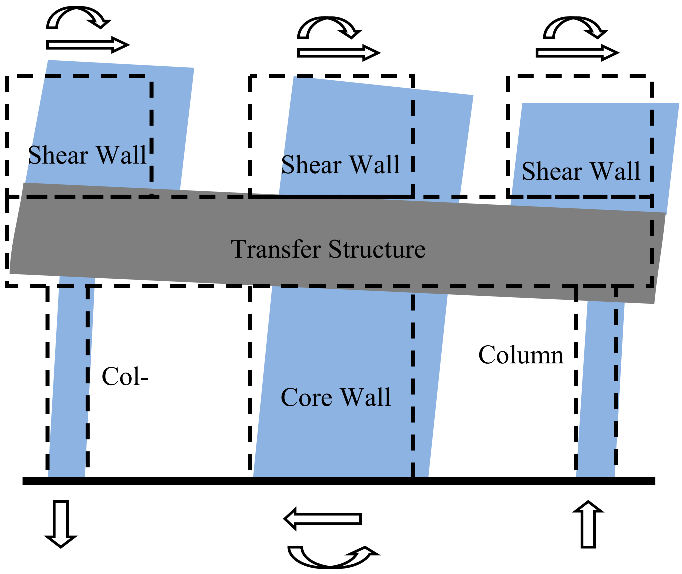

Fig. 4 Regular and irregular columns (above) and deformation of transfer structure and shear concentration at the transfer structure (below) (Su 2008).

Similarly, a stiff central core wall below the transfer floor can slightly limit the local rotations at the transfer level. Thus, the inter-storey drifts and the difference in rotations between the exterior walls and the core wall can be slightly reduced. The amount of shear force transfer from the core wall to the exterior walls, which is proportional to the difference in rotations, can also be limited. In addition, local rotation of the core wall can be further controlled by arranging the transfer floor at a lower floor level (below 10% of the total height) so that shear transfer above the transfer structure can be effectively reduced.

Chen and Fu (2004) suggested that flexural stiffness of the shear walls above the transfer floor which is much higher than that of the transfer beam can also decrease shear force transfer from the center wall to the edge walls. Furthermore, Ye *et al.* (2004) reported that providing floor openings above the transfer structure, which could break the essential load path for transferring shear forces, could also effectively reduce the shear concentration effect on the shear walls above the transfer structure and, hence, improve the seismic performance of building. However, diaphragm action may be adversely affected if openings significantly reduce the ability of the diaphragm to resist in-plane flexure and shear. As general rule, diaphragms should be designed to respond elastically, as they are not suitable to dissipate energy through the formation of plastic regions.

As such, it is evident that shear concentration effect due to transfer floor deformations must be taken into account. Su (2008) recommended to increase the design shear load at the shear walls above the transfer structure by about 20%.

Referring to Figure 4, the core wall deflects under the effect of earthquake load as a vertical cantilever. Since the transfer structure and core wall are monolithically jointed together, the joint region between the transfer structure and core wall is rotated in a similar manner because of the displacement compatibility. Thus, a pair of push-and-pull forces from the columns below the transfer causes deflection of this transfer structure. The rotation of the exterior walls above the transfer structure is therefore different from that of the core wall, and the difference in rotations can be as high as 0.0005 rad (Su 2008).

In order to reduce the rotation incompatibility between the core wall and the shear walls above the transfer structure, high in-plane compressive and tensile restraining forces will develop in the slabs just above the transfer floor. These horizontal reactions cause shear force transfer from the core wall to the exterior walls. In addition, the transfer floor itself is highly affected and, thus, should be accurately studied due to the huge horizontal forces needed to be transferred from the superstructure above the transfer floor resisting system to the one below it: a significant shear forces and bending moments are expected to affect the diaphragms and the transfer floor especially if located in a lower height.

7 STRESS CONCENTRATION IN BUILDING WITH TRANSFER FLOORS

Seismic forces concentrate in diaphragms where a huge mass of the building is located, i.e., where a vertical discontinuity or stiffness variation causes a redistribution of forces between the vertical elements of the lateral force resisting system. Thus, a sophisticated analysis should be performed for vertical irregular structures to get a better understanding for its behavior.

Based on his shaking table tests, Su (2008) argued that under minor earthquake attacks, most of the buildings remained elastic; no cracks were detected and the natural frequencies did not decrease. When the models were subjected to moderate earthquakes, cracks began to occur at top of columns below transfer floor and at the base of first floor columns. After major earthquakes, the models were severely damaged: damage was found in the peripheral shear walls above the transfer floor either when the transfer structure system was composed of beams or thick plates. Tension failure was observed on the end shear walls above the transfer floor. Further, shear and central core wall structures in the middle and upper floors were also damaged by shear. Floor slabs and beam-wall joints were all cracked. In other words, a weak floor formed at the floor above the transfer structure.

Xu *et al.* (2000) conducted an elastic dynamic analysis on a 27-storey building with transfer beams at the 7th floor and reported an abrupt change in shear forces of walls above the transfer floor. This effect became more severe when the building was subjected to rare earthquakes and the stiffness of the shear walls below the transfer structures was decreased. Their work indicated that if such building does not collapse after major earthquakes, it will lose most of its lateral load-carrying capacity and may need substantial retrofitting work.

In addition, Rong and Wang (2004) argued that during a shaking table test on irregular frames, damage tended to concentrate at the lower storey and severe spalling of concrete and buckling of reinforcement were observed in first storey columns at both ends, which indicates an existence of a soft-storey collapse mechanism. Furthermore, they recorded that under the same earthquake intensity, the crack width of the irregular frames was found to be double that for buildings with regular frame system. The load carrying capacity of irregular frame system was found to be 15% less than that for regular frame system.

The over-strength is a characteristic of structures where the actual strength is larger than the design strength: the degree of over-strength is material and system dependent. Due to the stress concentration accompanying vertical irregularities, the over-strength factor of irregular structures are lower than that for regular structures. Based on testing regular- and irregular-frame buildings, Valmundsson (1997) predicted base shear over-strength factors of 3.0 and 2.0, respectively. However, in their numerical analysis, the over-strength factor for both cases was found to be about 1.7. The distribution of the over-strength along the height of the tested buildings is shown in Figure 5: the concept of varying the over-strength along the building height is adopted by different code provisions as a good indicator for the structural irregularity.

8 LATERAL STIFFNESS DEGRADATION MEASUREMENT AND EFFECT

8.1 *Equivalent Stiffness Estimation*

Li et al. (2006) argued that buildings with transfer floors may be dividing it into two parts as shown in Figure 6. The lateral stiffness of each part can be estimated by applying a horizontal force on each part separately and determining the corresponding lateral deformation.

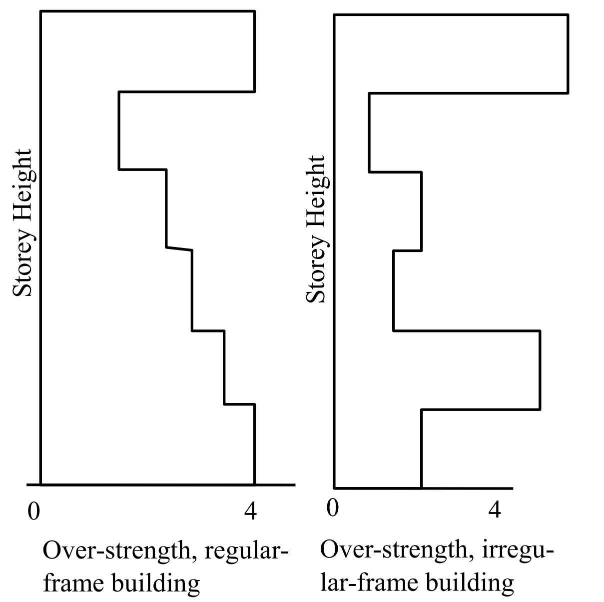

Fig. 5 Over-strength profiles of tested frames (Valmundsson 1997).

Despite the importance of the flexural stiffness below the transfer structure in controlling the soft-story effect, Chinese National Specification (2002) and Geng and Xu (2002) considered the lateral shear stiffness below and above the transfer structure separately (Figure 6) and argued that equivalent lateral stiffness ratio γ_e less than 1.3 results-in a seismically resistant structures where

$$\gamma_e = \frac{\Phi_1\left(\dfrac{\Delta_1}{H_1}\right)}{\Phi_2\left(\dfrac{\Delta_2}{H_2}\right)+\Phi_1\theta_b} \leq 1.3 \qquad (3)$$

Figure 6 shows that the concept of equivalent lateral stiffness ratio adopted by the Chinese National Specification (2002) which is modified in Equation 3 to take into consideration the effect of rotation of the structure above the transfer level due to the flexural rotation θ_b below the transfer level and the inelastic response of structures under a rare earthquake attack. In Equation 3, Φ_1 and Φ_2 are the displacement magnification factors due to stiffness degradation for the substructure below and the superstructure above the transfer level, which may be taken as *2.0* and*1.5*, respectively based on results from shaking table tests. This equation reflects the fact that when the lateral drift angle due to flexure $\Phi_1\ \theta_b$ is larger than that due to shear $\Phi_1\ (\Delta_1\ /H_1)$, the soft-story phenomenon vanishes (Su 2000).

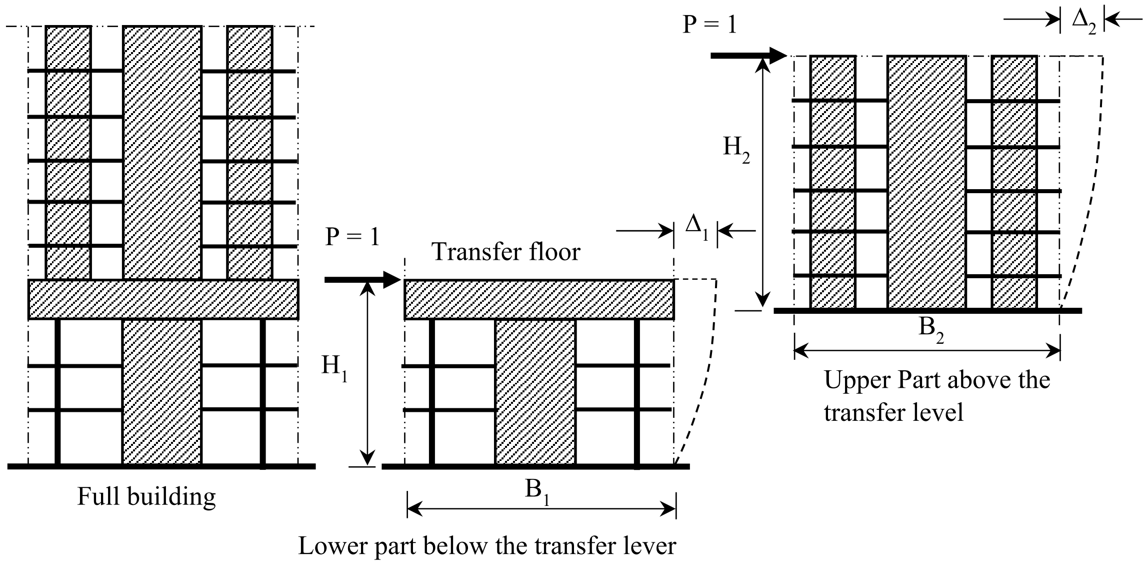

Fig. 6 Numerical models for calculating the equivalent stiffness below and above the transfer structure (Li et al. 2006).

8.2 *Acceptable Loss in Story Strength*

When buildings are subjected to occasional earthquakes and damage occurs, both the natural frequencies and the damping ratios started to change. The natural frequencies of the structure may drop by 10% to 20% due to the degradation in both lateral stiffness and strength of the structure. In a case study (Su 2000), the natural frequency of the structures was decreased by 20-46% due to degradation of stiffness and strength. The damping ratio was also increased from 2% after frequent earthquakes to 4.5%-7.5% after a rare earthquake.

Thus, severe damage and collapse would occur if the first fundamental frequency is reduced by approximately 20% and 50%, respectively after earthquake attack. The significant reduction in the spectral frequencies also indicates that structural safety is seriously affected at the level above the transfer floor (Li *et al.* 2006).

In any nonlinear response history analysis, deformation imposed at any storey should not result in a loss of total story strength that exceeds 20% of the initial strength. This is due to the fact that the component deterioration will lead to a loss in lateral- and gravity-load resistance, even if deterioration occurs only in deformation-controlled actions. Since no absolute limit is placed on the deformations that can be tolerated in any one component, it is important to check that the loss in story resistance does not become excessive. As a general target, the loss in the lateral story resistance at maximum drift should not be more than about 20% of the un-deteriorated resistance (TBI 2010).

8.3 *Soft Story Formation*

Under a seismic load, the reduction in the lateral resisting elements stiffness will attract much higher lateral deformations, and in many cases, high torsional deformations. The excessive inter-storey drift (along with the P-δ effect arising from gravity loads) may cause plastic hinges to form at the ends of vertical structural elements. If the elements are not ductile enough, failure of individual vertical supports will results-in a progressive collapse of the whole storey; so, it can be concluded that abrupt changes in the inter-storey drift may lead to a soft-storey formation. Since lateral flexural and shear stiffness often change abruptly near the transfer level, it is essential to prevent the formation of the soft-storey in buildings with transfer floors.

8.4 *Lateral Deformation below the Transfer Structure*

Lateral deformations below a transfer structure can be separated into shear and flexural modes (Figure 7): the lateral deformation of the transfer structure Δ_l is the sum of the shear deformation Δ_{sl} and the flexural deformation Δ_{fl}; i.e,

$$\Delta_l = \Delta_{s1} + \Delta_{f1} \tag{4a}$$

and the rotation of the transfer structure may be expressed as

$$\theta_b = \frac{(\Delta_{a1} + \Delta_{b1})}{B_1} \tag{4b}$$

where, Δ_{a1} and Δ_{a2} are the vertical movements at the left and right edges of the transfer structure and B_1 is the width of the substructure below the transfer structure (Su 2008).

Transfer floor level may be classified as a soft-story if excessive reduction in the lateral stiffness took place. To compute the lateral stiffness of the n story, vertical members at the *(n−1)* story are fixed, and a horizontal force F is applied at the n story. With Δ being the lateral drift of the n story, the lateral stiffness K_n will be F/Δ. The change in the lateral stiffness R_s (Xu *et al.* 2000) is defined by

$$R_s = \frac{K_n}{K_{n+1}} \tag{5}$$

where K_n is the n[th] floor stiffness and K_{n+1} the stiffness of the n+1 floor.

It was argued that the ratio of lateral stiffness may not give an accurate indicator to the existence of a soft-story for high-rise buildings since flexural stiffness plays an important role and it is recommended to consider the change in both the flexural stiffness and shear stiffness when assessing the presence of a soft story (Li *et al.* 2006, 2008). Thus, an alternative to R_s is proposed (Li *et al.* 2008) to express the change in lateral stiffness where

$$R_u = \frac{U^*}{U} \qquad and \qquad R_{\Delta U} = \frac{\Delta U^*}{\Delta U} \tag{6}$$

where, U is the displacement, ΔU is the inter-storey drift, U^* and ΔU^* are reference displacement and reference inter-storey drift of a reference building in which all vertical members above the transfer level extends to the foundation.

Lu *et al.* (1999) considered the effect of the vertical irregularity on the lateral behaviour of a building and the soft story formation where a comparative study was conducted on two six-storey, three bays, reinforced concrete frames; one having a tall first story called (BF) and the other with discontinuous interior columns (DCF). A 1:5.5 scale models were tested on an earthquake simulator. Because of the irregularity, trial design analysis was performed (according to Euro code 8) to determine the appropriate q-factor (Earthquake force reduction factor) for the two frames and it was found to be 3.50 and 2.70 for the BF and DCF frames, respectively; the DCF frames has a lower ductile response. Equivalent static force procedures were adopted in computing the base shear of the BF frame while response spectrum procedures were adopted for the DCF frame. The BF frame has lower base shear than that calculated for the DCF frame by about 20% (Yoshimura 1997).

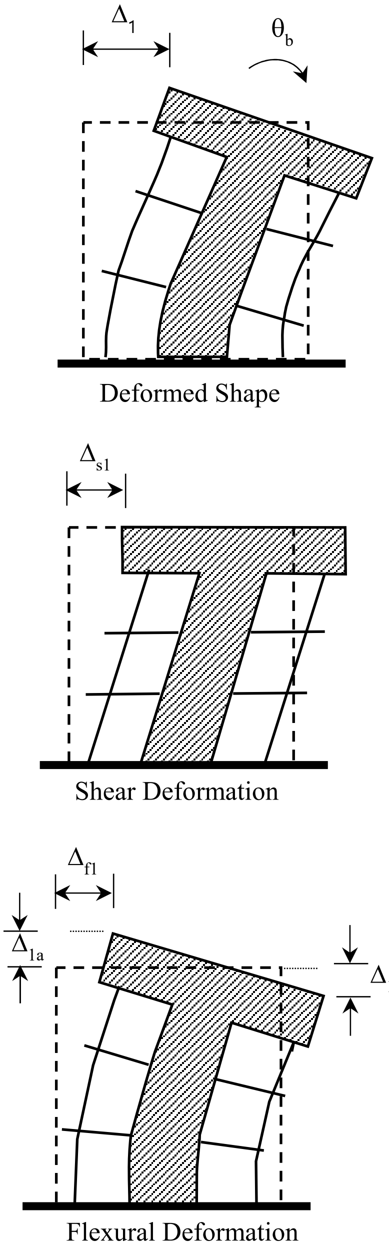

Fig. 7 Typical shear and flexural deformations of a substructure below a transfer level (Su 2008).

8.5 *Effects of Lateral Stiffness Variation of Structure above and below a Transfer floor*

Su 2008 found out an abrupt change in the inter-storey drift below the transfer level which is more severe when (i) lateral shear stiffness below the transfer structure is small, (ii) lateral flexural stiffness below the transfer structure is small and/or (iii) lateral flexural and shear stiffness above the transfer structure are small.

For the elastic seismic demands, increasing the stiffness of one story does not cause significant changes to the relative contributions of the different modes to the elastic story shear and the overturning moment as compared to the base case. However, decreasing the stiffness of one story can considerably change these contributions (Al-Ali and Krawinkler 1998). For the inelastic seismic demands, the effects of stiffness irregularities on the inelastic response of structures are larger than the effects of mass irregularities for similar stiffness and mass modifications.

9 STOREY DRIFTS

The changes in the shear and flexural stiffness of the structures above and below the transfer structure affect the lateral deflection and inter-story drift for the whole structure. Due to the vertical irregularity, a non-uniform and concentrated story drift occur at floors in the vicinity of the transfer floor. Structural members at the transfer structure should be designed with enhanced ductility and strength. Other requirements may include the need to carry out sophisticated analyses, as well as experimental verifications.

As a result of the stiffness variation, the effect of transfer floor to the inter-storey drift extends to one to two floors above it. It is also observed that the stories below the transfer floor also is adversely affected by the presence of the transfer floor as the drift also increases below the transfer floor due to the difference in the story stiffness.

For a building with a transfer floor located at approximately 10% of the total building height and subject to moderate to major earthquakes, Li *et al.* (2006) showed that the story drifts of a typical floors above the transfer level is about 3.3 times the story drifts at the transfer floor; below the transfer floor, the drift increase is about 1.2. These two ratios indicate that majority of the damage will commonly occur at the floors above the transfer level. This observation is reversed when the transfer floor is located at approximately at 50% of the total building height as the maximum drift is encountered below the transfer floor.

Another experimental study on the drifts values (Lu al. 1999) revealed that irregular frame structure have a gradual shift towards the soft first story mechanism while regular frame structure remained almost unchanged during the test: the deformation of the irregular frame structure was concentrated at the first story which about 20% larger than that of regular frame structure.

In addition, Li *et al.* (2008) argued that an increase in the lateral displacement at the floor above transfer floor occurs due to the cracking of the transfer system. As the reduction in the flexural stiffness of the transfer plate increases, the angle of rotation of the shear wall during seismic excitation at the upper floors increases which leads to the said increase in the lateral displacement.

Another experimental data (Li *et al.* 2006) revealed that deformation measurements can provide a good indication in assessing damage; in particular, story drift well relates to the degree of structural damage. For a structural system containing shear walls, slight, moderate, or severe damage would occur if the story drift approaches 1/ 1000, in the range of 1/300 – 1/700, or in the range of 1/80 – 1/200, respectively: Table 1 lists more details about the correlation between the damage and the lateral story drift.

Table 1. Damage and drift relation (Li et al. 2006).

Structural damage	Storey drift	Transfer floor damage	Inter-storey drift above the transfer floor
Small cracks on columns in frames	1/1000–1/1300	None	1/1500
A few number of small cracks on shear walls	1/1100–1/1200	Slight	1/750
Many through-cracks on shear walls	1/300–1/700	Moderate	1/360
Shear walls damaged with concrete crushed and reinforcement exposed	1/80–1/200	Severe	1/180

9.1 *Drift Demand*

For the cases with stiffness irregularities, story drift demands increases in the soft-story and decreases in most of the other stories. On the other hand, ductility and normalized hysteretic energy demands decrease in the soft-story and increase in the other stories (Al-Ali and Krawinkler 1998).

Soni and Mistry (2006) showed that introducing a soft and/or weak story increases the story drift demands in the modified and neighboring stories and decreases the drift demands in other stories. On the other hand, a stiff and/or strong story decreases the drift demands in the modified and neighboring stories and increases the drift demands in other stories. Generally, irregularity in upper stories would have a little influence on the floor displacements, and in contrast, irregularity in lower stories would have a significant influence on the height-wise distribution of floor displacements.

9.2 *Ductility Requirements*

The ability of the structure or its components, or the materials used to offer resistance in the inelastic domain of response, is described by the general term ductility. It includes the ability to sustain large deformation, and a capacity to absorb energy by hysteretic behavior. So, it is the single most important property that must be carefully studied by the designer in buildings located in regions of significant seismicity.

Static equivalent forces is not enough for assessing earthquake-induced structural actions, it must be appreciated that the actual seismic response is dynamic and related primarily to impose deformations than forces. To accommodate large seismically induced deformations, most structures need to be ductile. It is preferable, in structural design for earthquake resistance, to consider forces generated by earthquake induced displacements rather than traditional loads.

For ductile response of earthquakes, high compression strains are expected from the combined effect of the axial force and bending moment. Unless adequate, closely spaced, well detailed transverse reinforcement is placed in the potential plastic hinge region, spalling of concrete followed by instability of the compression reinforcement will follow. Even with strong column/weak beam design philosophy which seeks to dissipate seismic energy primarily in well confined beam plastic hinges, a column plastic hinge must still form at the base of the column: many structures have collapsed as a result of inadequate confinement of this hinge.

As a result, it has become accepted that seismic design should encourage structural forms that are more likely to possess ductility than those that do not (Paulay and Priestly 1992). Generally, this relates to aspects of structural irregularity locations carful choice, often termed plastic hinges, where inelastic deformations may occur.

Chen *et al.* (2000) concluded that it is conservative to predict the motion of masses based on the elastic modal analysis as the maximum displacement of these masses generally decreases with increasing the R factor (the numerical coefficient representing the inherent over-strength and global ductility capacity of lateral force-resisting systems as defined in UBC 97).

10 NUMERICAL MODELING OF BUILDINGS WITH TRANSFER FLOORS

Irregularity features clearance depends on the structural system used, the location and amount of the irregularity, and the design analysis method used (Sadashiva *et al.* 2008). 3-D computer models should be constructed to simulate the vertical irregular buildings and, if applicable, should be compared with the results obtained from shaking table tests.

Ye *et al.* (2003) performed a 3-D elastic analysis of a building model and reported that the differences in natural frequencies of the first and second modes between shaking table test results and the numerical analysis were within 10% for frequent earthquakes. The displacements of the top floor obtained from the tests and the numerical model under different seismic intensities were all within 3 to 7%.

Diaphragms are commonly used to model the transfer floor and it neighboring floors' slabs (Su 2008) to get better prediction of the interactions between the exterior shear walls and the flexible shell or beam elements of the transfer floor.

Huang *et al.* (2004) and Wu *et al.* (2007) constructed 3-D numerical models to compare the structural responses of buildings with transfer structures under frequent earthquake loads. The comparisons showed that the numerical analysis results for accelerations and inter-story drift ratios of bare frame models were similar, and these results generally agreed with each other for the first few vibration modes. Although their numerical studies could satisfactorily reflect the real dynamic response of buildings under frequent earthquakes, seismic responses of buildings under rare earthquakes could not be accurately simulated as the effects of stiffness and strength degradations of concrete elements were not considered.

Al-Ali and Krawinkler (1998) argued that for elastic seismic demands, spectral SRSS analyses provide reasonable approximations to elastic time history analyses for the studied response parameters (The investigated parameters were story drift, and ratio of displacement/total height of the building). For many investigated cases, there is no advantage in using the CQC over the SRSS method in combining the modal responses. For the cases in which several modes contribute in similar amounts to a response parameter, spectral analysis (SRSS or CQC) underestimates the time history response of this parameter.

In addition, Chintanapakdee and Chopra (2004) argued that the bias in the modal pushover analysis procedure does not increase, in case of vertical irregularity, relative to nonlinear response history analysis, i.e., its accuracy does not deteriorate, in spite of irregularity in stiffness, strength, or stiffness and strength. Furthermore for vertical movements, the difference between the cases considering and neglecting vertical motions may be very small in analysis results; this was probably because if the vertical mass was assumed to be lumped at each beam-column joint and column axial stiffness was assumed elastic, such assumptions might lead to the estimation of building vertical period being short and as a result of it, the building was considered to have responded to vertical motions like a rigid body (Yoshimura 1997).

The inelastic analysis methods have advantages over the elastic analysis methods in predicting the effects of structural discontinuities. This because buildings are usually designed for seismic resistance using an elastic analysis while most of them, especially high rise buildings subjected to rare earthquakes, experience significant inelastic deformations. Thus, nonlinear analyses provide the means for calculating the structural response beyond the elastic range, including strength and stiffness deterioration associated with the inelastic material behavior and the large displacements. It was showed by Yoshimura (1997) that the story ductility demands on multistory buildings with weak/soft stories vary depending on the relative yield strengths.

Thuat *et al.* (2004) suggested that the use of equivalent lateral force procedure is not necessarily restricted for certain types of vertically irregular buildings, such as buildings with a taller or heavier storey. This may be right as the equivalent lateral force procedure is more conservative in such designs as it totally ignores the effect of higher modes.

On another frontier, Lu et al. (2012) investigated the seismic behaviour of a 53-storey tower with the height of 250 m having discontinuous columns at the 37th and 38th storey. They performed a shaking table on 1/30 scaled model of the building in order to evaluate its seismic resistance capacity. Then, they introduced a 3D numerical model using the finite element method to which was produced according to the mass distribution of test model. They used the numerical model to investigate the structural behaviour of these buildings and compared the test results to the numerical analysis outcomes.

Gomez-Bernal et al. (2013) performed a 3D numerical analysis to investigate the interaction between shear walls and transfer-slabs subjected to lateral and vertical loading. In their investigation, they checked the behavior of shear wall/slab buildings and they argued that further investigations are still needed to investigate the behaviour of buildings with transfer systems to cover variables such as the slab type (solid waffle, plane), the walls' position on slab, the anchor between slab and wall, etc.

Though the vertically irregular high-rise building can resist strong wind, it brings an abrupt change in the lateral stiffness and mass so that it may be very vulnerable to earthquakes. In regions of low to moderate seismicity and subjected to strong winds, both the design wind and the credible frequent earthquakes control the design of the building in terms of deformation and inter-storey shear force. Based on this fact, it may not be true that high-rise buildings in regions of moderate seismicity and subjected to strong winds can provide enough seismic resistance without using explicit seismic design procedures. Seismic design may be needed for these vertically irregular high-rise buildings (Zhou and Xu 2007).

Finally, structural irregularity is argued to be a major concern in earthquake engineering society.

Depending on the building configuration and arrangement of structural members, the structural irregularity is a combined concept of irregular distribution of stiffness, strength and mass within the structure. If appropriate measures are not taken, the structural irregularity can become a major source of building damage during strong earthquake.

11 EFFECT OF TRANSFER FLOOR VERTICAL LOCATION

Vertical location of transfer floors with respect to total height of the building has a significant effect on high rise buildings performance (Paulay and Priestly 1992). Buildings with transfer floors at higher levels tend to deform and respond primarily as a single-degree-of-freedom structure with the fundamental mode dominating the response of the structure. As such, buildings with lower level transfer floor locations need to be analyzed using more modes contributions to obtain the required mass participation ratios.

A previous study by El-Awady et al. (2014) investigated the effect (among other parameters) of changing the level of the transfer system, and it was concluded that, buildings with transfer floors at higher levels tend to deform and respond primarily as a single-degree-of-freedom structure with the fundamental mode dominating the response of the structure. This investigation confirms the above argument stating that buildings with lower level transfer floor locations need to be analyzed using more modes contributions. Furthermore, the resulting shear distribution showed that buildings with lower level transfer floor have a higher base shear and lower base moment compared to similar buildings with transfer floor at higher level due to higher stiffness of the lower part of the structure. The investigated also revealed that roof drift of buildings with transfer floor at height of 10% of total building height is more than roof drift of buildings with transfer floor at height of 50% of total building height. This is due to the huge mass above the transfer floor in case of 10% of total height relative to the small mass above the transfer floor in case of 50% of total height types. As such, lower level transfer system would produce higher roof drift regardless of the transfer system type. Generally, placing the transfer floor in the lower part of the structure (in a ratio varying from 20-30% of the height of the structure) is commonly required by the architectural design.

12 CONCLUSIONS

In many high-rise buildings, architectural requirements may result in a variable configuration for the vertical structural elements between the stories of the building. In order to accommodate such discontinuity of the vertical structural elements, a "transfer" floor conveying the vertical and lateral loads between upper and lower stories must be introduced. The transfer system is divided to deep girders or thick plates while structural irregularity are divided into mass, stiffness and geometrical irregularity; where limitations for each of the type of these irregularities are specified in codes of practice to prevent the phenomena of soft story mechanism. Transfer system deformation is still ignored and assumption of rigid diaphragm is adopted in design, this concept is not quite correct and simulation in 3-D model should be done using solid element which will lead to stiff transfer system with high shear and flexural stiffness which reduces the deformation of the transfer system under lateral loads. However, the effect of solid element can be accommodated by increasing the design shear load at the shear walls above the transfer structure by about 20%.

Drift limitations is one of the governing concern in design of high rise building with transfer floor; where stiffness irregularities are presented, story drift demands increases in the soft-story and decreases in most of the other stories. Irregularity in upper stories would have a little effect on the floor displacements, while, irregularity in lower stories would have a significant effect on the height-wise distribution of floor displacements. Vertical location of transfer floors with respect to total height of the building has a significant effect on high rise buildings performance; introduction of the transfer floor in the lower part of the structure (20-30% of the total height of the structure from its foundation) is better than having it in a higher location.

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