High-Rise Buildings with Transfer Floors: Linear Versus Nonlinear Seismic Analysis

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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 behaviour for frequent earthquake but suffer extensive crack in the vicinity of the transfer floors for rare earthquake. For design purposes, current numerical modelling of high-rise building adopts reduced stiffness for the vertical elements for strength analysis and full stiffness for serviceability and drift analysis: a tradition that needs to be verified. A 3-D numerical model is built-up for a high-rise building with such vertical irregularities and analysed using elastic response spectrum and nonlinear time-history analysis techniques. The effect of the transfer floors on the buildings' drift and seismic-generated internal forces is investigated where judgment for adopting a full or reduced stiffness for the vertical elements is scrutinized.

1 INTRODUCTION

Recently, innovative architectural design merged with the advanced and powerful structural numerical analysis stimulated a new generation of "super-tall" and "mega-tall" buildings. Furthermore, discontinued columns and shear walls within high-rise buildings are no more considered as a sin. Consequently, architectural demands for high-rise buildings in which columns may have different arrangement between certain levels become familiar. Many high-rise buildings are currently constructed with this kind of vertical irregularity where "transfer" floors are provided to account for the discontinuous columns and/or shear walls in order to accommodate the building's function (Figures 1). As such, the transfer floor is defined as the floor supporting the vertical and lateral load resisting elements and transfer their straining actions to a different underneath system. Different structural systems can be used below the transfer floor: moment-resisting frames and/or shear walls; while the floors above may be supported by shear walls or columns. The transfer floor system itself can take the form of transfer girders or slabs.

Recently, several investigations tacked the analysis of high-rise buildings with transfer systems. For example, Sullivan (2010) studied the capacity of reinforced concrete frame-wall structures where he argued that material overstrength, higher mode effects and secondary load paths associated with the 3-dimensional structural response affect the overall capacity of such system and pointed out to serious limitations with capacity design procedures included in current codes of practice for such buildings and the urgent need for further research on this subject. 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 for the model building to investigate the structural behavior of these buildings. Gomez-Bernal *et al.* (2013) also performed a 3D numerical analysis using ANSYS to investigate the interaction between shear walls and transfer-slabs, subjected to lateral and vertical loading. Qi and Zhong (2013) introduced an experimental study on seismic performance of transfer floor structure for frame-supported short-leg shear wall where one beam transfer floor and three inclined column-shaped transfer floor structures of frame-supported short-leg shear wall were tested under vertical loading and horizontal cyclic loading.

Once again, their research shows that further investigation is still generally needed to investigate the beahviour of buildings with transfer systems.

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 and imposes high ductility demands on the elements in the vicinity of the transfer floors (Zhang and Ling 2011, Abdelbasset et al. 2014). Therefore, if this irregularity is not taken into consideration during the design stages, it may become a major source of building damage particularly during rare earthquakes. A recent research (Li et al. 2006 and El-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. The said investigations along with another one by Yong et al. (1999) also 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 investigation of Al-Ali and Krawinkler (1998) and Su (2008) where drifts are pronounced by reducing the stiffness of the vertical elements (columns and shear walls); hence, revealing the importance of deciding to consider a reduced or a full stiffness for these structural elements in any numerical model of high-rise buildings with transfer floors.

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

Here, an analytical seismic study for the response of high-rise buildings with transfer floors is carried out via 3-D modelling of these buildings using the finite element technique. The numerical models are analysed using elastic response spectrum and time-history analysis techniques. The effect of transfer floors on the drift and seismic-generated forces in such structures is investigated. Adopting full or reduced stiffness in the numerical for the vertical elements in the numerical models is scrutinized to verify the previously mentioned strategy for drift calculations and/or suggest a suitable alternative for the reduction of the vertical elements stiffness when performing serviceability/drift design under seismic loads.

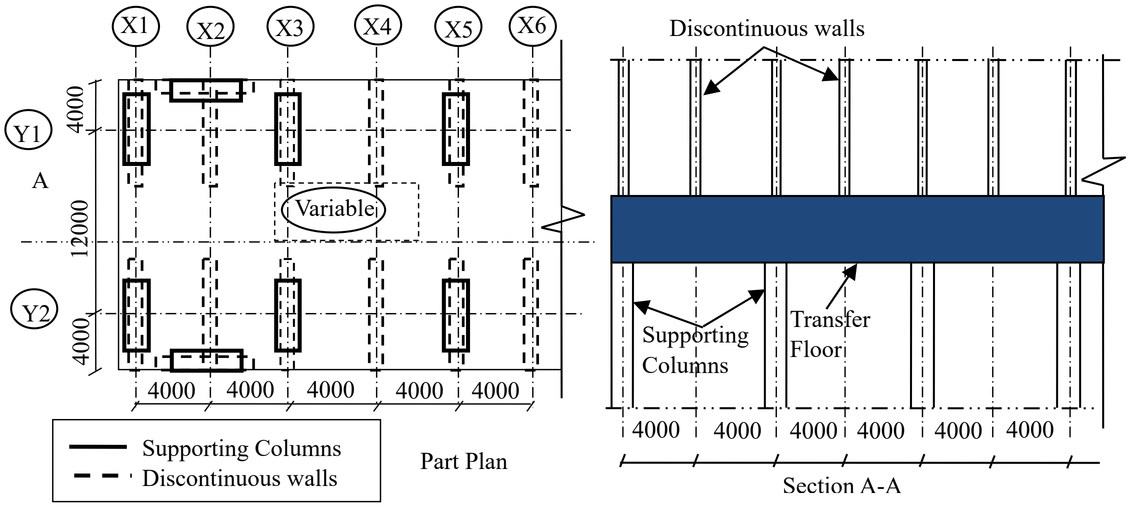

Fig. 1 Schematic part plan and part elevation for a building with transfer floor.

2 METHODOLOGY

Comprehensive literature review on seismic performance of high-rise buildings with transfer floors is presented elsewhere (Abdelbasset et al. 2016) which also includes a comparative study between different provisions of the most commonly used codes of practice dealing with design of high-rise building with the vertical irregularity resulting from transfer floors.

Here, linear and nonlinear three-dimensional finite element seismic analyses are performed on a high-rise buildings with a transfer slab. The seismic response of such buildings is investigated using equivalent static loads, elastic response spectrum, and linear and nonlinear time history analysis techniques. Story-shear distribution, bending moment distribution, inter-storey drift, and floor displacements are numerically evaluated and presented. It was shown by Adbar et al. (2014) that a nonlinear numerical analysis is highly recommended to investigate the seismic displacement demands for any gravity-load resisting frames; which is emphasized for buildings with severe discontinuities such as those created by the existence of a transfer system.

3 THE ANALYZED BUILDING

The current numerical analyses for buildings with transfer floors consider one type of transfer: solid transfer plates (slabs). This system easily accommodates the difference in the location of the vertical load bearing elements above and below its level.

The study is conducted using a building (Figure 2) which comprises of a 50 story tower with a total height of 175 m: this building is slightly modified from its real design. A similar building was previously adopted by El-Awady et al. (2014) to scrutinize the effect (among other parameters) of changing the level of the transfer system on the seismic behaviour of the building. Based on said analysis, the location of the transfer floor is chosen to be at 20% of the total building height measured from the foundation and fixed throughout the investigation: this is also matching the approximate practical level at which transfer system may be required.

The building adopted in this study is symmetric in plan, and as such, avoids any torsional effects. The floor area is 20.0 m × 48.0 m where the spacing between the columns below the transfer floor is 8.0 m, and shear walls above it is 4.0 m. The typical story height is 3.5 m. The 2000 mm thick transfer slab is supported on 1.0 m × 4.0 m columns underneath the slab and supports 0.30 m × 8.0 m shear walls above it.

The analysis considers various configurations for the stiffness of the different building's structural elements; the transfer floor system and its location within the building height were kept fixed during the whole investigation. The results cover the global behaviour of the structures i.e., story-shear distribution, base-shear, story-moment distribution, drift behaviour, lateral displacement distribution, and mass participation ratio.

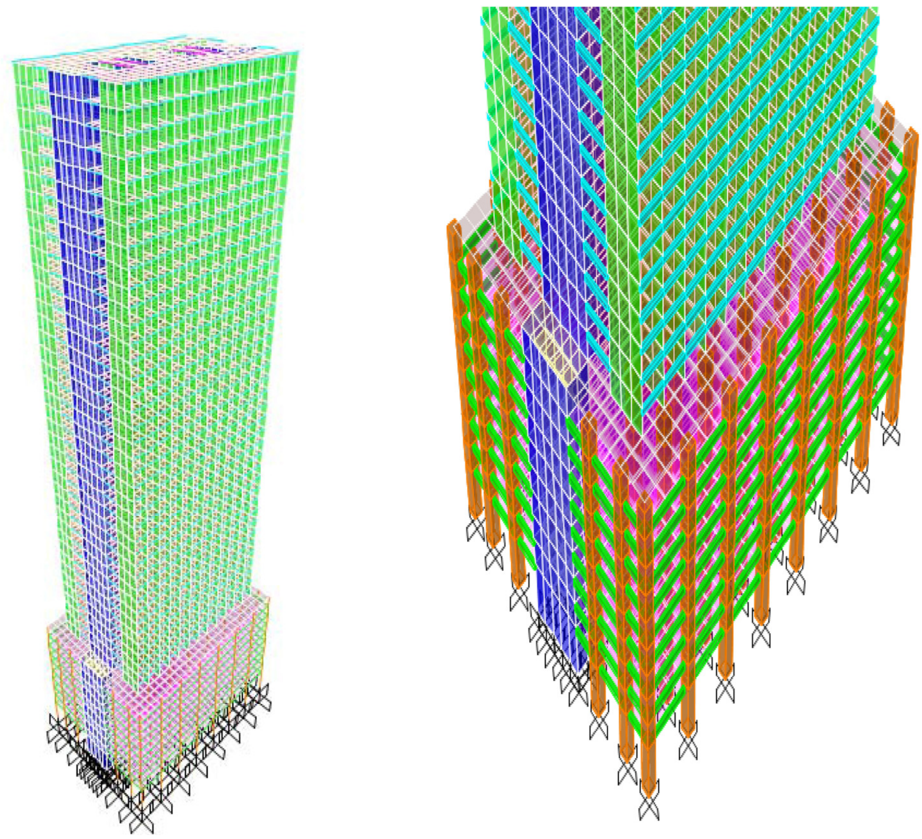

Fig. 2 Three dimensional view of the FE model adopted for a 50 storey tower with a transfer slab at 20% of the building height.

4 SEISMIC INPUT

4.1 *The Response Spectrum Analysis*

Response spectrum analysis was conducted on the models to evaluate the behaviour of the building incorporating the first twelve vibrational modes using the CQC (complete quadratic combination - a method that is an improvement on SRSS for closely spaced modes combining sequence, UBC 1997). Figure 3 shows the design and maximum considered response spectra chosen for the conducted analyses according to ASCE/SEI 07-10: Cairo, Egypt is the place considered to be the location of the models currently investigated.

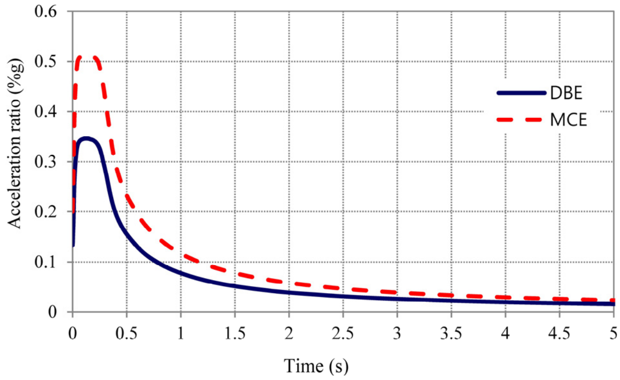

Fig. 3 Response spectrum according to the ASCE/SEI 07-10 code.

Thus, according to the ASCE/SEI 07-10 provisions, the seismic parameters, related to this location, are:

- 0.2 sec spectral acceleration $S_s = 0.5$ m/s^2
- sec spectral acceleration $S_1 = 0.117$ m/s^2
- Long period transition period $T_l = 20$ s
- Soil type B (very dense soil).
- The inherent over strength and global ductility capacity of lateral force-resisting systems R = 5.0.
- The system over-strength factor $\Omega_o = 2.5$
- The deflection amplification factor $C_d = 4.5$.
- The building importance factor $I = 1.25$.
- The numerical coefficient $C_t = 0.02$.
- For mass source, the dead loads factor is taken 1.0 and the live loads factor is taken 0.5.

For all building's floors above and at the transfer floor level a super imposed dead load SID of 3.0 kN/m^2 and a live load LL of 2.0 kN/m^2 are considered. On the other hand, for all floors below the transfer floor level, a super imposed dead load SID of 4.5 kN/m^2 and a live load of 5.0 kN/m^2 are considered. These loads matche the actual values adopted in the design of the actual building from which the current model is slightly modified.

4.2 *Time History*

ASCE/SEI 07-10 provisions provide a guidance on the development of ground motion acceleration time histories for linear and nonlinear response history analyses. If a suite of not less than three appropriate ground motions is in the analysis; the design member forces and design story drift are to be taken as the maximum value determined from the analyses. However, if at least seven ground motions are analyzed, the average forces and drifts resulting from the analyses may be used.

If possible, each considered ground motion consists of a horizontal acceleration history (pair of orthogonal components), selected from an actual recorded event. The records should be deducted from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the MCE (maximum considered earthquake) response spectrum. If the numbers of appropriate recorded ground motion pairs are not available, ASCE/SEI 07-10 allows to use simulated ground motion pairs. For 3D analysis, ground motions are to be scaled such that the average of the 5% damped response spectra for the suite of motions is not less than the design response spectrum for the site for periods ranging between 0.2T and 1.5T, where T is the natural period of the structure in the fundamental mode for the direction of response being analysed. For three-dimensional analysis, the square root of the sum of the squares (SRSS) spectrum of the 5% damped response spectra is used for each pair of the horizontal ground motion components: each pair of motions is to be scaled such that for each period between 0.2T and 1.5T, the average of the SRSS spectra for all horizontal component pairs does not fall below 1.3 times the corresponding ordinate of the design response spectrum by more than 10%.

From a practical point of view, matching by arithmetic scaling of the time histories in an attempt to match the target design spectrum is extremely difficult because of the nature of the MCE response spectrum. The MCE spectrum for a region like Cairo does not represent any singular earthquake event; rather, it is the blending of multiple events with smaller and larger earthquake magnitudes, occurring on different types of faults, at varying distances from the site of interest. Because of the breadth of the 0.2T to 1.5T period range in which the target spectrum has to meet, arithmetic scaling often results in time histories being so amplified that the energy content of the matched time histories is unrealistically high at most periods.

The selection of candidate time histories to match the target MCE spectrum can be challenging where records should have magnitudes, fault distance, and source mechanisms that are consistent with those of the MCE. Despite the increase in number of available time histories with more earthquakes and more databases such as the NGA database (Reyes and Kalkan 2012), there are still gaps in the available records in meeting the code's requirements. There are still deficiencies in the quantity of records for large magnitude events in the near and far field. Many earlier time histories do not have reliable information for longer periods beyond 2 to 5 seconds. Significant modifications of the seed time histories are needed such that the spectra match the MCE target spectrum for tall buildings which may have fundamental periods greater than 5 seconds.

In this regard, seven records were chosen (Table 1): records have the same mechanism with a magnitude ranged from 6.5 to 7.5 and a distance from the epicentre almost the same and frequency near the one results from the spectrum analysis.

Table 1. Ground motion records adopted in the current analysis.

ID	Earthquake	Station	Year	Magnitude	Distance (km)	Frequency (Hz)
1	Imperial Valley 06, Mxico	Cerro Prieto	1979	6.53	15.19	0.1125
2	Duzce, Turkey	Lamont 1061	1999	7.14	11.46	0.0875
3	Manjil, Iran	Abbar	1990	7.37	12.55	0.13
4	Landers, USA	N. Palm Springs Fire, Sta 36	1992	7.28	26.95	0.1125
5	Tottori, Japan	OKYH08	2000	6.61	24.84	0.0375
6	Tottori, Japan	OKYH09	2000	6.61	21.22	0.02625
7	Darfield, New Zealand	Heathcote Valley School	2010	7.00	24.36	0.075

Four methods are currently adopted for scaling the magnitude of ground motion records using a multiplying factor, so that the response spectrum of the modified records fits the target spectrum defined in the corresponding design regulations. More details about these methods are given elsewhere (ASCE/SEI 07-10, Euro code 08, Egyptian code of loads 2012, Abdelbasset et.al. 2015). The scale factor adopted here is based on the method defined in ASCE/SEI 07-10 (and IBC 2009) which is considered to be one of the most common codes of practice used for analysis of structures worldwide.
The ground motions selected for the analysis correspond to those recommended as default set of records for soil type B which is a soft rock that satisfies the seismological signature requirements presented in the response spectrum curve are shown in Figure 4. Seven records were chosen in order to use the mean values of the responses resulting from the analysis.

The study examined strong motion records available from the PEER strong motion database (http://peer.berkeley.edu/smcat/). Table 2 shows the scale factors that were calculated based on the ASCE/SEI 07-10 provisions for the considered records. The scaling factors are then multiplied by the importance factor of 1.25. Figure 4 shows the scaled spectra as a result of using these factors on each of the records chosen for the analyses.

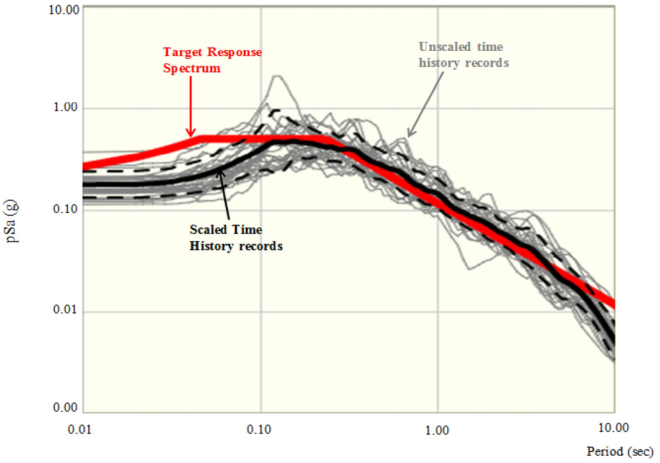

Fig. 4 Scaled time history spectra.

Two methods for the nonlinear time history analysis are commonly adopted: the first one is the direct integration method and the second is the modal analysis. The method used in this analysis is the direct-integration time history which is a nonlinear, dynamic analysis method in which the equilibrium equations of motion are fully integrated as the structure is subjected to dynamic loading. Analysis involves the integration of structural properties and behaviour at a series of time steps which are small relative to loading duration. Integration is performed at every time step of the input record, regardless of the output increment.

4.3 *Finite Element Simulation*

The finite element software package ETABS is adopted in the current numerical analysis. This program is deliberately chosen since it is one of the most widely adopted packages in design offices. A typical numerical model for the prototype building is shown in Figure 2.

A three-dimensional model is adopted for each of the investigated analysis with different stiffness for the structural elements of the prototype building: slabs, walls and columns. Four cases are considered for the stiffness of the building's structural elements:

1. All structural elements have full stiffness (gross inertia – I_g).
2. All horizontal elements (slabs) have full stiffness (gross inertia – I_g) with all vertical elements (columns and walls) have reduced stiffness (cracked inertia – I_{cr}).
3. The transfer slab has full stiffness (gross inertia – Ig) with all other horizontal elements and vertical elements having reduced stiffness (cracked inertia – I_{cr}).
4. All elements have reduced stiffness (cracked inertia – I_{cr}).

The main purpose of the current stage of analysis is to compare the building's response for each of the above mentioned case. In this concern, analysis was performed using one of the following seismic analysis techniques: linear elastic response spectrum (LERS), linear time history (LTH) and non-linear time history analysis (NLTH). The above mentioned four types of inertia were checked in the numerical analysis adopting LERS and LTH analyses but only the cases of gross and cracked inertia for all elements were checked for LTH analysis.

5 SEISMIC RESPONSE OF THE ANALYZED BUILDING

The global behaviour of the prototype building is presented. The results of the model include the shear distribution, base-shear relative to periodic time, transfer floor shear force, moment distribution, drift distribution, displacement and mass participation ratios. A sample of the results is plotted in the Y-direction of the building with almost a typical behaviour recorded in the X-direction.

5.1 *Storey Drift*

Figure 5 shows a plot of the inter-story drift distribution over the building height in Y-direction. The figure shows that for all cases of analyses; linear and nonlinear, the drift below the transfer floor reaches a maximum value midway between the foundation and transfer floor level and then decreases gradually up to the transfer floor location.

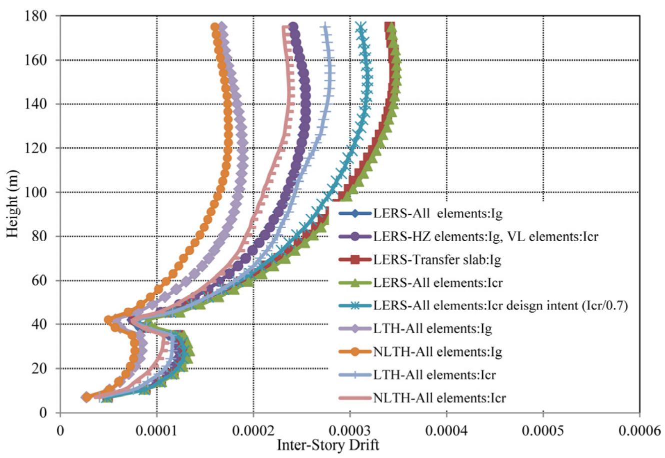

Fig. 5 Story drift resulting from the different numerical analysis techniques.

Above the transfer floor, the drift begins to increase till it reaches a maximum value in vicinity of the top third of the building height and decreases till the roof level; however, values of drift at roof level are between the maximum values and values at transfer level. It is evident from Figure 5 that the NLTH analysis gives values for drift less than those obtained by the LERS analysis by about 30-40%. Furthermore, LTH analysis may be considered as a transition (average value) between LERS and NLTH analyses; drift values resulting from LTH analysis is also less than LERS analysis by 20% but larger than those of the NLTH by 10%.

5.2 *Lateral Displacement*

A plot of the lateral displacement distribution in the Y-direction over the building height is shown in Figure 6. It is obvious that for all types of analyses; linear and nonlinear, the lateral building displacement matches a flexural behaviour mode till the transfer floor level where a large force hit the building due to the huge mass of the transfer floor. Displacement from foundation up to the transfer floor level simulates a fixed-fixed flexural member, where above the transfer floor level, the building acts as a free cantilever with its base fixed at the transfer floor. It is evident from Figure 6 that the NLTH analysis with the direct integration method gives values for displacement similar in pattern but less in values by about 30-50% compared to that resulting from LERS analysis with reduced inertia for all elements, as per most design code requirements. Once again, LTH analysis results lies between those of the NLTH and LERS analyses; displacement values resulting from LTH analysis are larger than those resulting from the NLTH analysis by 10-15% and less than those resulting from LERS analysis by 20-25%.

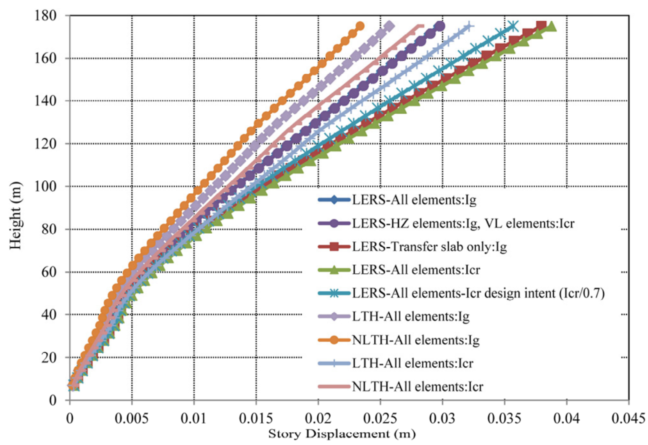

Fig. 6 Story displacement resulting from the different numerical analysis techniques.

5.3 *Storey Shear*

Figure 7 shows a plot of the story-shear distribution over the building height in the Y-direction for all the cases of the seismic analyses (linear response spectrum, linear time history and nonlinear time history). The figure shows that a significant reduction in the story-shear above the transfer slab level takes place due to the sudden change in overall building stiffness at the slab level. The NLTH analysis results-in less story-shear compared to both LTH and LERS analyses by 10-15% and 25-30%, respectively. The results for LTH analysis can be considered as transition between the LERS and NLTH analyses.

5.4 *Storey Moment*

A plot of the story-moment distribution over the building height in the Y-direction is shown in Figure 8, for all cases of seismic analyses; linear response spectrum, linear time history and nonlinear time history methods. The figure reveals that the story-moment distribution has an inflection point at the vicinity of the transfer floor level for case of NLTH analysis; the building is behaving as a cantilevering for all cases of linear analysis. The NLTH analysis results less values for the story-moment than those resulting from the LERS and LTH analyses for both cases of using full or reduced stiffness for all elements. NLTH analysis results values for the moments which are less than those resulting from the LERS and LTH analyses by 40-50% and 15-25%, respectively. Once again, the LTH can consider as an analysis that results values lie the NLTH and LERS analyses. Values of story-moment are almost the same in the upper floors of the building for all cases of analyses. The pattern of results is almost the same for all cases of analyses especially above transfer floor.

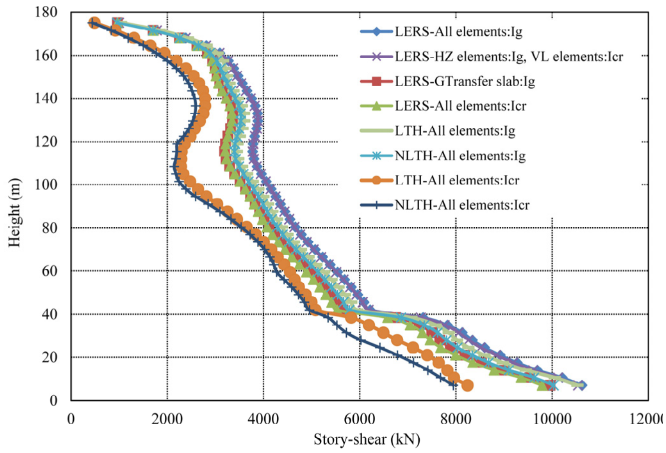

Fig. 7 Story-shear resulting from the different numerical analysis techniques.

5.5 *Base Shear*

Figure 9 shows a plot for the base-shear for the several cases of analyses (LERS, LTH and NLTH) in the Y-direction which is the strong direction of the building. It is evident from the figure that the base-shear resulting from NLTH analysis is less by 10-20% for gross inertia of all elements and 40% for cracked inertia of all structural elements than the values resulting from the LTH analysis. Furthermore, base-shear values resulting from the LTH and NLTH analyses are less than those resulting from the LERS analysis by 10% and 20%, respectively. It is also noted that values obtained from the equivalent static load method is extremely conservative and give values more than any dynamic analysis.

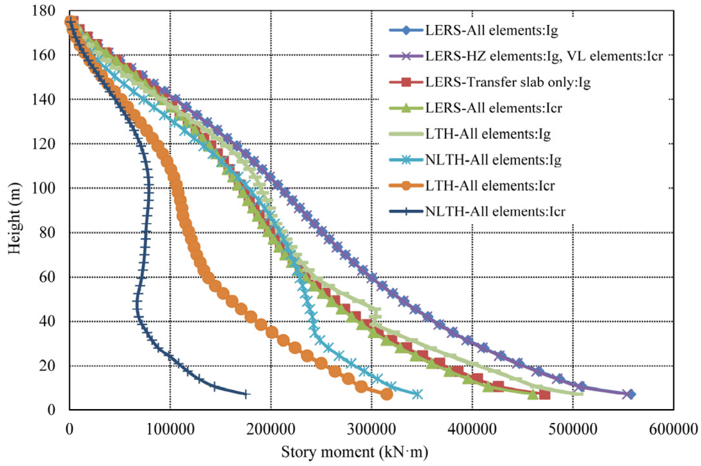

Fig. 8 story-moment resulting from the different numerical analysis techniques.

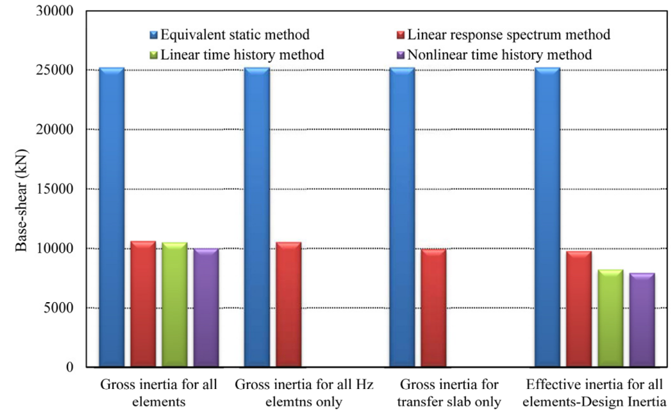

Fig. 9 Base-shear resulting from the different numerical analysis techniques.

5.6 *Mass Participation Ratio*

A plot of the mass participation ratio (Y-direction) for every mode is shown in Figure 10. It is evident from this figure that for all cases of analyses; the governing mode with the largest mass participation percentage is the translation mode. It is also noted that nonlinear time history analysis has a mass participation percentage almost similar to that for the response spectrum analysis for all cases of stiffness for the first modes.

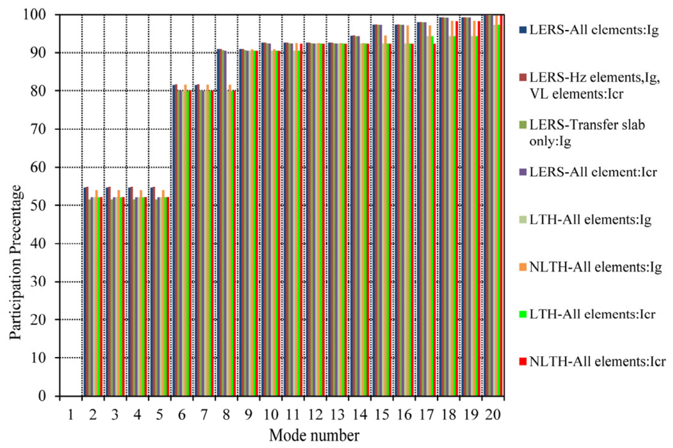

Fig. 10 Modal mass participation ratio considered in the different numerical analysis techniques.

6 CONCLUSIONS

Numerical seismic analyses for a building with a transfer floor have been performed. An equivalent static loading, a response spectrum and a time history were adopted in the analyses. A comparative study is presented on the effect of using reduced stiffness for the different structural elements of the building; as currently recommended by most codes of practice. The investigated stiffness reduction followed four schemes: the vertical elements, transfer slabs and horizontal elements in addition to using full stiffness for all elements. The following conclusions are deducted from the current investigations:

- Non-linear time history analysis results story-shear forces, story-moment and base-shear which are less than linear time history and elastic response spectrum analysis (reduced stiffness as per design code) by 10 to 30%.
- Non-linear time history analysis results drift and lateral displacement which are less than linear time history and elastic response spectrum analysis (reduced stiffness as per design code) by about 20 to 35%.
- Non-linear time history analysis results mass participation ratio similar to elastic response spectrum analysis (reduced stiffness as per design code).
- Linear time history analysis can be considered an alternative analysis method which is less conservative than elastic response spectrum analysis but still results a resendable factory of safety compared to the more accurate case which is based on the non-linear time history analysis and with less analysis complexities.
- For drift and lateral displacement checks, in case of high rise buildings with transfer floors, gross inertia may be used in the analysis and yet results reasonable and conservative values with an appropriate factors of safety.
- For strength design of high rise buildings with transfer floors, cracked inertia of (1.2 I_{cr}), where I_{cr} is the cracked inertia specified by codes of practice, can be used for and yet results values with an appropriate factors of safety.

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