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The Effect of Reducing Superimposed Dead Load on the Lateral Seismic Deformations of Structures

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Abstract**—**The vast majority of the Middle East countries are prone to earthquakes. Despite that and from a seismic hazard point of view, the higher values of the superimposed dead load intensity of partitions and wearing materials of the constructed reinforced concrete slabs in these countries can increase the earthquake vulnerability of the structures. The primary objective of this paper is to investigate the effect of reducing superimposed dead load on the lateral seismic deformations of structures, the inter-story drifts and the seismic pounding damages. The study utilizes a group of three reinforced concrete structures at three different site conditions. These structures are assumed to be constructed in Nablus city of Palestine, and having superimposed dead load value as 1 kN/m^2, 3 kN/m^2, and 5 kN/m^2, respectively. SAP2000 program, Version 18.1.1, is used to perform the response spectrum analysis to obtain the potential lateral seismic deformations of the studied models. Amazingly, the study points that, at the same site, superimposed dead load has a minor effect on the lateral deflections of the models. This, however, promotes the hypothesis that buildings failed during earthquakes mainly because they were not designed appropriately against gravity loads.

Keywords**—**Gravity loads, inter-story drifts, lateral seismic deformations, reinforced concrete slabs, response spectrum method, SAP2000, seismic design, seismic pounding, superimposed dead load.

I. Introduction

EARTHQUAKES are natural disasters that whenever strongly strike, they often result in considerable economic and human losses. Annually, about 60,000 people die throughout the world in natural disasters [1]. Most of fatalities are because of collapse of buildings during earthquakes [2]. Hence, the majority of the death could be reduced by improving of design concepts and construction methodologies.

When an earthquake occurs, buildings have a tendency to resist the ground motion by moving in an opposite direction to the shaking. The inertia of building's mass and its contents is responsible for the reversed motion [3] as shown in Fig. 1.

The developed inertia forces are hereby designated as seismic loads [5], and directly proportional to the mass of the affected system. The concentration of the mass of building systems at roofs [5] permits the assumption is that seismic loads are most effective on roofs and floors levels as idealized in Fig. 2. Hence, a lightweight roof is a key factor for providing an overall stability of the structure.

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Admittedly, heavyweight roofs have revealed unsatisfactory results during recent earthquakes. The increase in the mass of roofs stimulates considerable seismic loads which may cause substantial lateral seismic deformations. These deformations may be sufficient to damage the structural and non-structural components of the building. As a comparison, in the 1995 Kobe earthquake in Japan, most of the buildings were utterly collapsed due to the overweighting of roofs by large tiles of Japanese industry [6]. On the other side, dwellings roofed by lightweight materials in Haiti played a positive factor in mitigating losses and the number of damaged buildings during the 2010 earthquake of Haiti [7].

Regardless the fact that the Middle East region is an active seismic region [8], the construction practice followed in many of the Middle East counties like Palestine, Jordan…etc., does not sufficiently consider the seismic risk in building design and construction. Most of the builders prefer the use of the reinforced concrete slabs in the construction industry. Although reinforced concrete slabs are massive by nature, they are overloaded by a high superimposed dead load (SDL), that may reach six times the SDL used in the United States, for example [9], [10]. In these countries, SDL includes the weights of the partition walls, tiles, infill aggregate beneath the tiles…etc. In this method of construction, the systems of plumbing and water pipes are installed within the space between the top face of the slab and the cement mortar fastens the tiles. Besides the adverse effect of the high SDL, water and wastewater leakage may be not readily observed. Wastes attack the concrete and the steel bars leading to premature corrosion of steel. Indeed, the previously mentioned construction method needs to be seriously revised. This paper, however, focuses on the lateral seismic vulnerability of the structure as a negative consequence of high SDL rather than other effects.

In general, large deflections do not affect the structure itself, but they affect the stability of the adjacent structures especially in the crowded urban areas where buildings may collide. Seismic pounding between closely spaced buildings is one of the major causes of significant building structures damages across the globe [11]. This type of destruction was evident through history in the 1999 Kocaeli earthquake, the 1995 Kobe earthquake, the 1992 Cairo earthquake, the 1989 Loma Prieta earthquake [12]. Figs. 3 (a) and (b) are some instances of ponding effects [13].

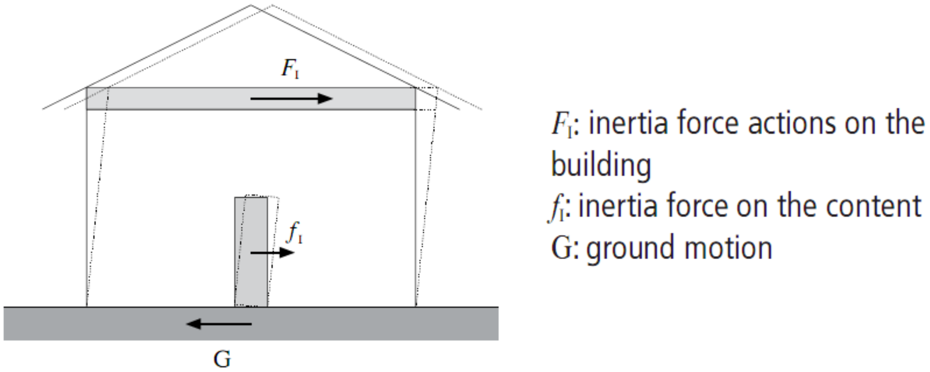

Fig. 1 Inertia forces affect a building and its content [4]

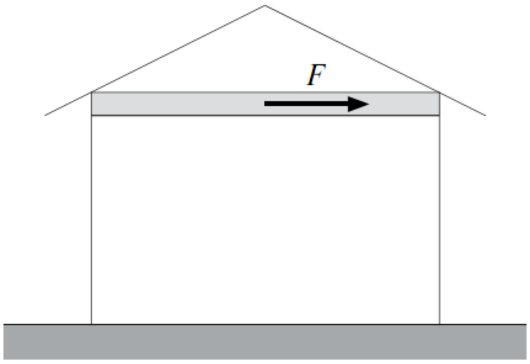

Fig. 2 The resultant of earthquake loads [4]

II. Materials and Methods

Nine structural models are constructed to represent the proposed multi-story commercial building. The building is three bays with three bays in a plan as shown in Fig. 4, with a height of ten stories.

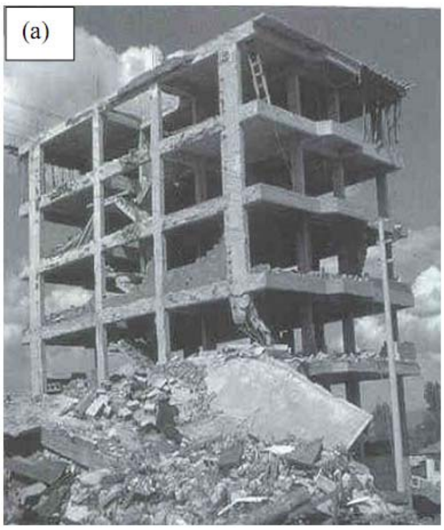

Fig. 3 (a) Column failure dates in 1999 Kocaeli earthquake

Fig. 3 (b) Front elevation damage dates in 1989 Loma Prieta earthquake

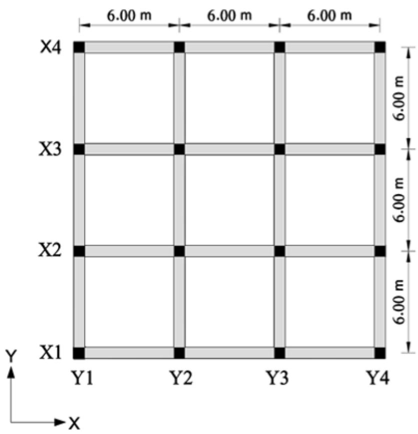

Fig. 4 Typical floor plan of the assumed models

The difference in the applied gravity loads results in variations in the dimensions of beams and columns constituting models. These accredited dimensions in Table I are finals, as they record satisfactory results in the checks shown later.

To distinguish models, every model has a specific designation such that it begins with the value of the carried SDL and ends with the type of the supporting ground. The clearance of all stories composing models is identical and equal to 2.95 m. It is also worth mentioning that the SDL values in Table I (1 kN/m^2, 3 kN/m^2, and 5 kN/m^2), are assumed by researchers to examine the effect of gradual reduction in SDL on the lateral movement of buildings.

TABLE I
DESIGNATION OF MODELS AND DIMENSIONS OF THE CONSTITUTING STRUCTURAL MEMBERS

No.	SDL (kN/m^2)	Ground Layer	Model Designation	Vertical Height (m)		Slab Depth (mm)	Beams Sections (mm)		Columns Sections (mm)	
				Story	Structure		Width	Depth	Length	Width
1	1	Rock	1-R	3.4	34	130	650	400	650	650
2	3	Rock	3-R	3.55	35.5	130	700	450	700	700
3	5	Rock	5-R	3.7	37	130	750	500	750	750
4	1	Soft Rock	1-SR	3.4	34	130	650	400	650	650
5	3	Soft Rock	3-SR	3.55	35.5	130	700	450	700	700
6	5	Soft Rock	5-SR	3.7	37	130	750	500	750	750
7	1	Soft Soil	1-SS	3.4	34	130	650	400	650	650
8	3	Soft Soil	3-SS	3.55	35.5	130	700	450	700	700
9	5	Soft Soil	5-SS	3.7	37	130	750	500	750	750

kN= kilonewton, m = meter, R = rock, SR = soft rock, SS = soft soil, mm = millimeter.

A. The Referenced Codes

The following codes are used in different stages of work.

1. The Jordanian Code [14].
2. The 2015 International Building Code (IBC 2015) [15].
3. The Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) [16].
4. The 2014 Building Code Requirements for Structural Concrete and Commentary (ACI 318-14) [17].

B. Definition of the Applied Loads

Dead loads (DLs), live loads (LLs), and horizontal earthquake loads will be only included in the work. It should be noted that due to the symmetry and uniformity of floor plans of models, horizontal earthquake loads are applied in one direction, i.e. the X-direction is considered.

DLs are the own weight of the structure added to the SDL. Own weight of the building is calculated by assuming that the unit weight of the reinforced concrete material constituting structural members is 25 kN/m^3.

Live loads on floors comprise all loads produced by the use and occupancy of the building other than dead loads. The nominal live load intensity used for the structural design is specified as the weight per unit area and taken as 4 kN/m^2 [14].

The seismic calculations and the analysis procedures are performed according to the IBC 2015 Code, and the ASCE 7-10 Code.

C. Scope of Analysis

Due to the constraints in space in this paper, the calculations are presented for one model only (Model 1-R).

D. Estimation of the Trial Sections

For all models, the depths above of slabs and beams were determined according to the deflection requirements stated by the ACI 318-14 Code. For instance, the provided 400 mm depths for beams in model 1-R, satisfies the requirements of Section 9.3.1.1 in the ACI 318-14 Code.

Finally, columns sections are designed to carry loads determined according to the tributary area method.

E. Mathematical Modeling

3D models of the buildings are built. The beams and columns are modelled using a space frame element, and the slab is modeled using a thin shell element. Shell mesh size is 0.75 m by 0.75 m. This size was approved through a series of mesh sensitivity analysis procedures such that when the difference between two subsequent results is worthless, the smaller mesh size is selected. The models are analyzed using the finite element program, SAP2000, Version 18.1.1 [18]. The followings are the major assumptions made in constructing the models.

1. Members have uncracked sections.
2. Diaphragms are rigid.
3. Columns are fixed at the base level.

F. Verification of the Static Analysis

It is important to verify answers obtained through SAP2000 analysis. The verification process, however, starts with the static analysis results.

The researchers work to verify the most critical SAP2000 results that could significantly affect the topic of this paper. The verifications are mainly expressed as to the percentage error between the two obtained values. The maximum accepted percentages of errors are specified inside SAP2000 manuals [19]. Results of the static analysis, however, are proofed according to:

1. The Condition of Compatibility.

This condition is satisfied as seen in Fig. 5.

2. The Condition of Equilibrium

This condition is satisfied as shown in Table II.

TABLE II
CHECK FOR THE BALANCE OF FORCES FOR MODEL 1-R

Item	SAP2000	Hand Calculations	Error (%)
Weight of the Structure (kN)	21918	21918	0.00
SDLs (kN)	3240	3240	0.00
LLs (kN)	12960	12960	0.00

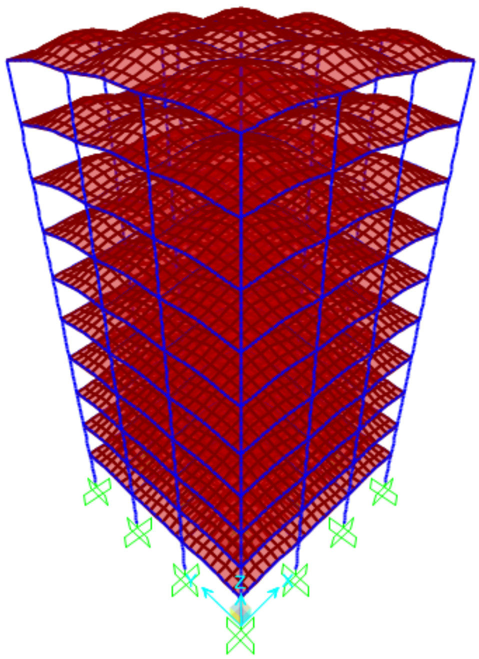

Fig. 5 Three dimensional deflection shape of Model 1-R

3. The Stress-Strain Relationship

The Direct Design Method (DDM) described in the ACI 318-14 Code is used to verify the moment values generated in the slab and the beam of the internal span (Y2-Y3) of frame X2 at the fourth-floor slab (Level: +17.00 m). It is worth mentioning that Model 1-R satisfied all the limitations of the DDM. This verification process involved moments generated under the load situation 1.2DL+1.6LL given by the ACI 318-14 Code. The accepted level of error in this check is 25% [19]. The verifications of the total factored moment values obtained by SAP2000 are shown in Table III. It shall be reminded that the other models were verified against these three checks.

TABLE III
VERIFICATION OF TOTAL FACTORED MOMENTS OBTAINED BY SAP2000

Item	Value
Total factored moment of the beam (kN.m)- DDM	177
Total factored moment of the beam (kN.m)- SAP2000	178
Error (%)	0.565
Level of Error	OK
Total moment of the slab in the column strip (kN.m/m)- DDM	15.4
Total moment of the slab in the column strip (kN.m/m)- SAP2000	13.5
Error (%)	14.1
Level of Error	OK
Total moment of the slab in the middle strip (kN.m/m)- DDM	20.6
Total moment of the slab in the middle strip (kN.m/m)- SAP2000	18.7
Error (%)	10.2
Level of Error	OK

G. Factors Affecting the Seismic Behavior of Structures

In the event of an earthquake shaking, the response behavior of structures is distinguished through the natural periods (T_n) and the damping ratios (ζ) they have.

Despite the fact that structure is damped in nature, the damping ratio ζ = 5% [20] assigned for concrete structures has no measurable effect on the periods of vibration. Therefore, T_n almost equals the damped period [20]. In the final consideration, ζ = 5% is automatically adopted by SAP2000 program and set to affect all modes of vibrations.

The fundamental period shall not exceed a prescribed limit given by the ASCE 7-10 Code such that:

$$T_1 \leq C_u T_a \tag{1}$$

where T_1 is the fundamental period in seconds (s), and C_u is a factor for the upper limit on the approximate fundamental period (T_a) and determined according to the ASCE 7-10 Code.

The approximate period (T_a) is calculated according to the ASCE 7-10 Code as:

$$T_a = C_t h_n^x \tag{2}$$

where C_t and x are parameters rely on the structural system and the structural material respectively and determined from the ASCE 7-10 Code, and h_n is the total height of the building in meters. In the final analysis:

$$T_1 \leq C_u C_t h_n^x \tag{3}$$

$C_u = 1.45, C_t = 0.0466, x = 0.900, and\ h_n = 34.0\ m.$ Then, the maximum permitted value of T_1 in (3) is $T_1 = 1.61\ s$. With referring to SAP2000 analysis, $T_{1_{SAP2000}} = 1.49\ s$ is less than $T_1 = 1.61\ s$ which is acceptable.

H. Verification of T_1 Value Obtained by SAP2000

The fundamental period, $T_1 = 1.49\ s$ obtained from SAP2000 is checked against the value computed according to Rayleigh's method of analysis [20]:

$$T_1 = \sqrt{\frac{4\pi^2}{g}\left(\frac{\sum_{i=1}^{n} w_i\, \delta_i^2}{\sum_{i=1}^{n} P_i\, \delta_i}\right)} \tag{4}$$

where n is the number of floors which equals 10, w_i is the seismic weight of story i, δ_i is the lateral movement of each story under the effect of P_i, g is the gravitational acceleration and equals 9.81 m/s^2, and P_i is the total static load distributed over the area of the diaphragms. This distributed load is taken as 15 kN/m^2. Table IV, however, shows verification of T_1 value obtained by SAP2000. It should be mentioned that the accepted level of error in this check is 10% [19]. Thus, T_1 in (4) is:

$$T_1 = \sqrt{\frac{4\pi^2}{9.81}\left(\frac{22911}{42252}\right)} = 1.48\ s.$$

$$Error\ (\%) = \frac{1.49-1.48}{1.48} \times 100\% = 0.676\% \leq 10\%,$$
$$which\ is\ OK.$$

It shall be recalled that the fundamental periods of the other models were verified similarly.

TABLE IV
VERIFICATION OF T_1 VALUE OF MODEL I-R BY RAYLEIGH'S ANALYSIS

# of Story	w_i (kN)	P_i (kN)	δ_i (m)	$w_i\delta_i^2$ (kN.m^2)	$P_i\delta_i$ (kN.m)
10	2262	4860	1.36	4189	6613
9	2516	4860	1.32	4372	6406
8	2516	4860	1.25	3949	6089
7	2516	4860	1.16	3390	5642
6	2516	4860	1.04	2729	5061
5	2516	4860	0.894	2013	4347
4	2516	4860	0.721	1307	3503
3	2516	4860	0.524	689	2544
2	2516	4860	0.311	243	1512
1	2516	4860	0.1010	30.4	534
			$\sum$	22911	42252

TABLE V
THE DETERMINATION OF S_{DS} AND S_{D1} VALUES

Model	Site Class	S_s	S_1	F_a	F_v	S_{DS}	S_{D1}
1-R	B	0.500	0.250	1	1	0.500	0.250
3-R	B	0.500	0.250	1	1	0.500	0.250
5-R	B	0.500	0.250	1	1	0.500	0.250
1-SR	C	0.500	0.250	1.2	1.55	0.600	0.388
3-SR	C	0.500	0.250	1.2	1.55	0.600	0.388
5-SR	C	0.500	0.250	1.2	1.55	0.600	0.388
1-SS	D	0.500	0.250	1.4	1.9	0.700	0.475
3-SS	D	0.500	0.250	1.4	1.9	0.700	0.475
5-SS	D	0.500	0.250	1.4	1.9	0.700	0.475

I. Characteristic Parameters of an Earthquake Excitation

According to the conventional practice of Palestine and Jordan, the design earthquake is expected to have a 10% probability of exceedance in 50 years; having a return period of 475 years. This shaking is well defined regarding the Z-factor such that [21]:

$$Z = PGA/g \tag{5}$$

where Z is a constant given for each different location in the country, and PGA is the peak ground acceleration. Z = 0.2 is assigned for Nablus city of Palestine [22].

For every construction site, it is essential to go with the coefficients of the design spectral accelerations at short and long periods, i.e. S_{DS}, and S_{D1} respectively. The coefficients of the design spectral acceleration are determined according to the ASCE 7-10 Code as:

$$S_{DS} = S_s F_a \tag{6}$$

$$S_{D1} = S_1 F_v \tag{7}$$

where S_S, and S_1 are the coefficients of spectral accelerations, and determined as [22]:

$$S_S = 2.50Z \tag{8}$$

$$S_1 = 1.25Z \tag{9}$$

F_a and F_v are constants to express the effect of sites other than rocks and determined according to the ASCE 7-10 Code as a function of the site class and both S_S and S_1 values. For the studied models, S_{DS} and S_{D1} values are shown in Table V.

J. The Method of Seismic Analysis

The modal response spectrum method will be used in determining the maximum lateral deflections of the studied models. In this method, the displacements, which are the focus of the research team, depend primarily on the expected spectral accelerations of the buildings during their modes of vibration. This acceleration, however, is represented as a function of the vibrational modal periods on a graph known as the acceleration response spectrum.

Fig. 6 shows the acceleration response spectrum provided by the ASCE 7-10 Code. This spectrum has been considered in SAP2000 tools for the analysis of the models.

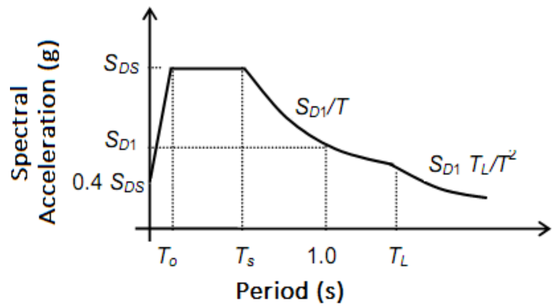

Fig. 6 The acceleration response spectrum [16]

S_{DS} is the coefficient of the design spectral acceleration at the short period, S_{D1} is the coefficient of the design spectral acceleration at the long period. T_0, T_s, and T_L are controlled periods in seconds. T_0 and T_s could be calculated according to the ASCE 7-10 Code as:

$$T_0 = 0.20T_S \tag{10}$$

$$T_S = S_{D1}/S_{DS} \tag{11}$$

$T_L = 4.00\ s$ [21]. It should also be noted that the spectral acceleration values are normalized to g. Table VI, however, shows the calculated values of the important coordinates on the horizontal and vertical axes of the acceleration response spectrum shown in Fig. 6.

TABLE VI
VALUES REQUIRED TO DEFINE THE ACCELERATION RESPONSE SPECTRUM FOR DIFFERENT MODELS

Model	T_0	T_S	T_L	$0.4S_{DS}$	S_{D1}	S_{DS}
1-R	0.100	0.500	4.00	0.200	0.250	0.500
3-R	0.100	0.500	4.00	0.200	0.250	0.500
5-R	0.100	0.500	4.00	0.200	0.250	0.500
1-SR	0.129	0.647	4.00	0.240	0.388	0.600
3-SR	0.129	0.647	4.00	0.240	0.388	0.600
5-SR	0.129	0.647	4.00	0.240	0.388	0.600
1-SS	0.136	0.679	4.00	0.280	0.475	0.700
3-SS	0.136	0.679	4.00	0.280	0.475	0.700
5-SS	0.136	0.679	4.00	0.280	0.475	0.700

III. Simulation Results

To demonstrate the effect of the SDL on the lateral seismic deformations, side deflections of the studied models have been read from SAP2000 program, and the following configurations are produced.

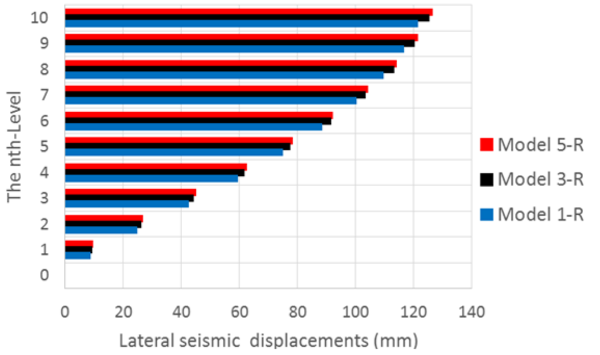

Fig. 7 Seismic deformations of models built over rock

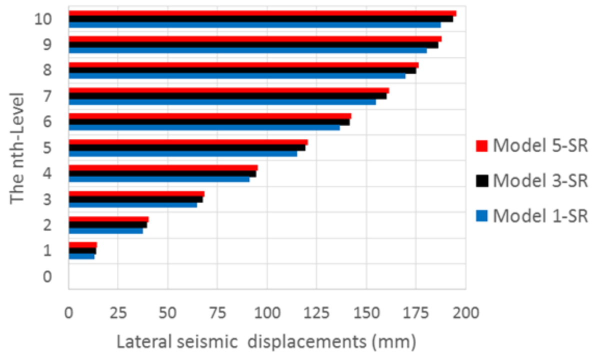

Fig. 8 Seismic deformations of models built over soft rock

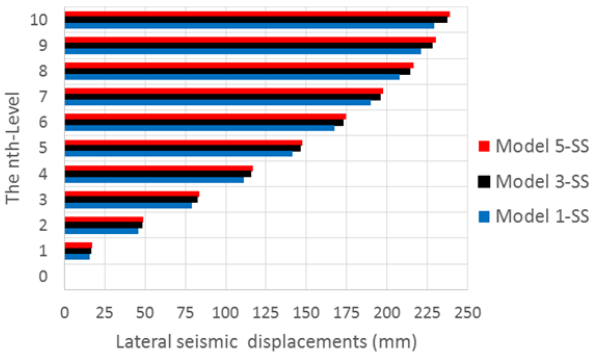

Fig. 9 Seismic deformations of models built over soft soil

In Figs. 7-9, the displacement response of the floors in different models is plotted against the floor levels. The unit mm, however, is used to compare the enlargement of response versus an increase in the SDL of typical cases in this research. It can be noted that when the SDL increases, the lateral motion of floors is always improved. On the other hand, it can also be seen that lateral deflections of any three models at the same site location are very close to each other, such that only using one displacement envelope of a model in a particular graph may be sufficient to describe the total response of the two different models in the chart.

IV. Conclusions

Despite the expected effect of the dimensions of structural systems, the total height of the structure, and the value of the SDL on the lateral deflection of buildings, the research team observed that the lateral movements of any three models at the same site are much like each other.

Because the deformation shape is a main key parameter to describe the structure status, Figs. 7-9 led the researchers to place and defend the following two premises:

1. The damage of buildings in the event of earthquakes does not essentially mean that their design does not follow prescriptive seismic code standards, but more likely because buildings were poorly engineered for gravity load effects.
2. Providing buildings with structural systems having an adequate strength capable of transmitting gravity loads and having first periods within the permitted range, is a vital part of the right seismic design.

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