Ductility Factor of RC Building Frames with Different Infill Wall Configurations under Low Intensity Far Field Earthquake Effects

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SUMMARY: (10 pt)
It is not a well accepted practice for engineers to incorporate seismic resistance into structural design in regions of low earthquake intensity such as Malaysia. Introducing ductility design criteria into structural plan is less favoured by the nation's industry. The increment in construction cost is another main reason behind this impediment. Therefore, it is rather important to understand the behaviour and capacity of non-earthquake designed structure under local earthquake condition, despite being minor. This paper investigates the ductility performance of non-ductile reinforced concrete frames with different precast infill locations. Using localized soil response spectrums, pushover analysis for the frame-infill system is performed at both serviceability limit state (SLS) and ultimate limit state (ULS) of vertical loading, respectively. The behaviour factor (q_0) and displacement behaviour factor (q_A) are presented, and implication of using these factors in judging the suitability of DCL structure in resisting low to moderate seismic demand is included.

Keywords: ductility factor; precast; infill wall; far-field earthquake; displacement behaviour factor

1. INTRODUCTION

Increasing cost of construction, large involvement of labour skill at site as well as chaotic building process, are some of the many reasons causing the government of Malaysia to strongly encourage the usage of precast concrete structures particularly for large construction projects. While the history of precast concrete may be long, its implementation in the country is considerably rather young. The wet and high humidity of the country at all times have impeded usage of dry joint system (Hamid, 2009). Unless additional water proofing technique is employed, leakage problem might become apparent. This means the only choice left is wet connection, where considerably amount of cast-in-situ concreting works still needs to be done at site. As a result, the cost of timber formwork needs to be absorbed into the building cost, as in conventional cast-in-situ method.

A type of reusable mould was invented by the local industry, in replacing the costly timber formworks (Tiong et al, 2011). The moulds were used for concreting work which connected two, three or four precast concrete panels. Besides serving as connection, these joints also acted as columns. The whole structural system consists of discrete precast infill panel and column. No beams were required. With recent major earthquakes felt in Sumatran and the Philippines, Malaysia is now in the midst of developing local seismic design provisions. As mentioned earlier, expensive construction to resist for rare seismic threats is not a well-accepted practice by the industry throughout the nation, at least up to the present. Therefore, there is a strong need for a study to be carried out in analyzing the optimum structural component or building method that best suits the need of the nation.

A study by Divan and Madhkhan (2011) revealed that the behaviour factor (q_0) or also-known-as response modification factor (R_I) could easily been affected by the presence of wall panels, and also the height of structure. Eurocode 8 (CEN 1998, 2003) does recommend two approaches in determining these behaviour factors. One of which is to use the readily recommended values, while the second is from nonlinear pushover analysis. This paper addresses the behaviour factor of such wall-frame system, influenced by different location of infill panels.

2. STRUCTURAL MODELS

2.1. Wall Configurations

A total of four different types of wall configuration were used in this study, as shown in Fig. 2.1. These different configurations of wall position were chosen to represent different infill locations, due to the possibility of such concrete wall panels being replaced by ceiling-to-floor glass panels. As a matter of such, the bracing due to presence of glass was negligible.

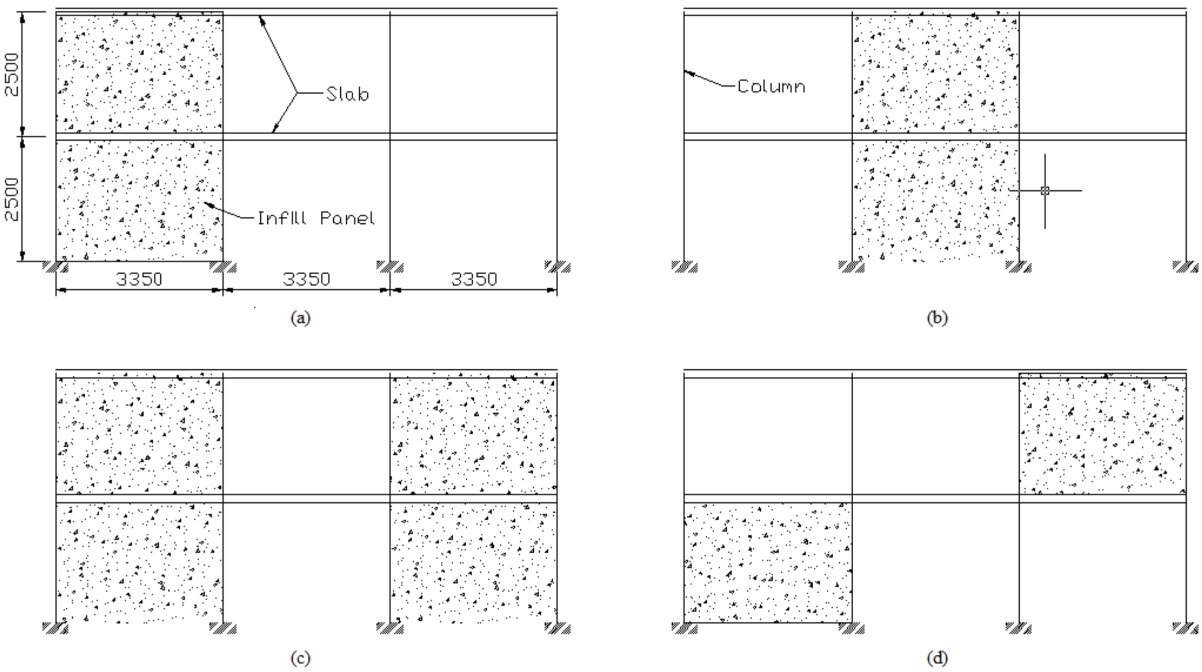

Figure 2.1. Different wall configurations adopted in the study

2.2. Structural Design

The structure was designed as a low ductile one, (DCL) based on the design provisions stated in Eurocode 2 (CEN 1992, 2002). Until the national annex has been finalized and revealed, a higher ductility design which of course, involves relatively higher building cost will not be the preferred choice. As a result, this study focused on DCL structure, instead of the DCM (medium class ductility) or DCH (high ductility class) building. The structure was only designed for vertical load carrying capacity only. In other words, no lateral load resistance was provided. The vertical load came from the structure self-weight itself only (permanent action), without any variable actions. Since the structure was designed as DCL, the wall-column connection was designed as semi-rigid.

2.3. Seismic Loading

The walls are firstly subjected to pushover analysis, under nominal serviceability limit states (SLS) gravity loading condition, represented as Model 1, 3, 5 and 7 (the odd numbers). Model 2, 4, 6 and 8 (even numbers) basically repeated the previous four models respectively; with the only different was the increment of vertical load from SLS to ultimate limit states (ULS). Five local design spectrums representing all five soil types (from hard rock to soft soil) were used in determining the demand curves, as shown in Fig. 2.2.

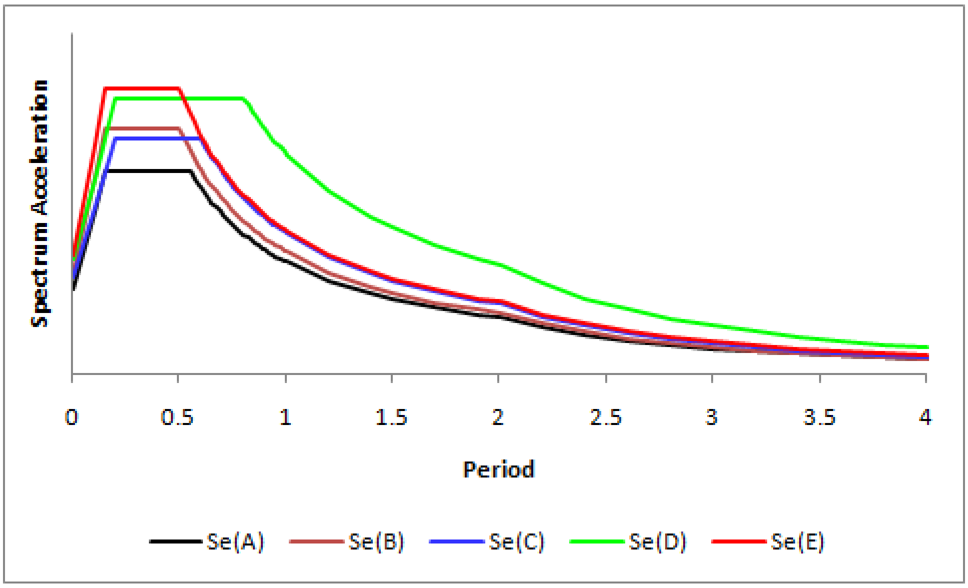

Figure 2.2. Design spectra for Malaysia used in pushover analysis for different soil conditions

3. RESULTS AND DISCUSSION

The monotonic pushover responses of all 8 models are presented in Fig. 3.1. It was clearly observed that all models exhibited a certain degree of ductility, as they deform into their nonlinear regions. The effect of varying vertical loading only affected the global response of Model 1 and 2, while the others did not reflect such results. This nonlinear lateral response was governed by the semi-rigid connection between column-wall interface, rather than the nonlinearity of frame elements.

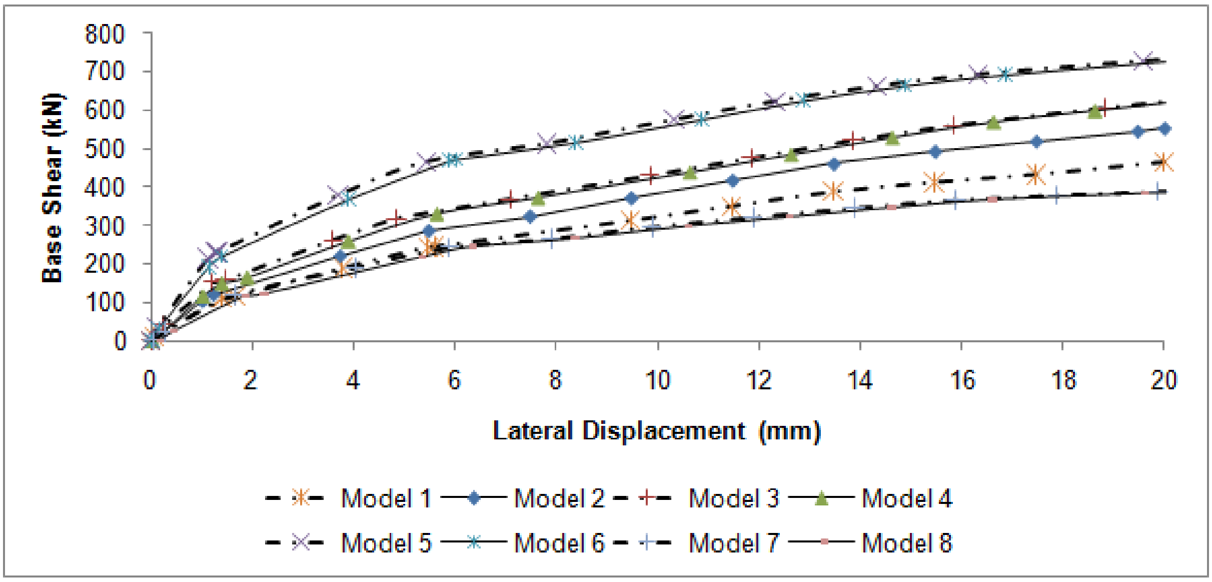

Figure 3.1. Pushover curves obtained for Model 1 to Model 8

The ductility or behaviour factors for all models are presented in Fig. 3.2. The dotted line in red indicated the average value. It was observed that although the ductility seemed to be changing when moving between models, it was not a random movement. Except for Model 1 and 2, the ductility of remaining models was not sensitive to vertical loading. It seems that irregularity in the plane direction (in this case the X-direction) is governing the global ductility of the structure should the vertical load changes, instead of vertical irregularity.

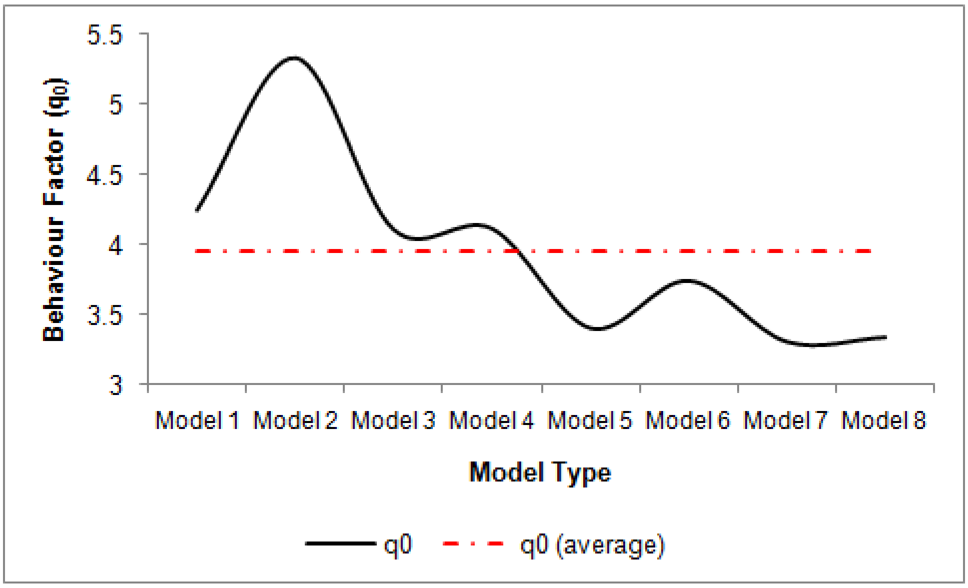

Figure 3.2. Behaviour factors of Model 1 to Model 8

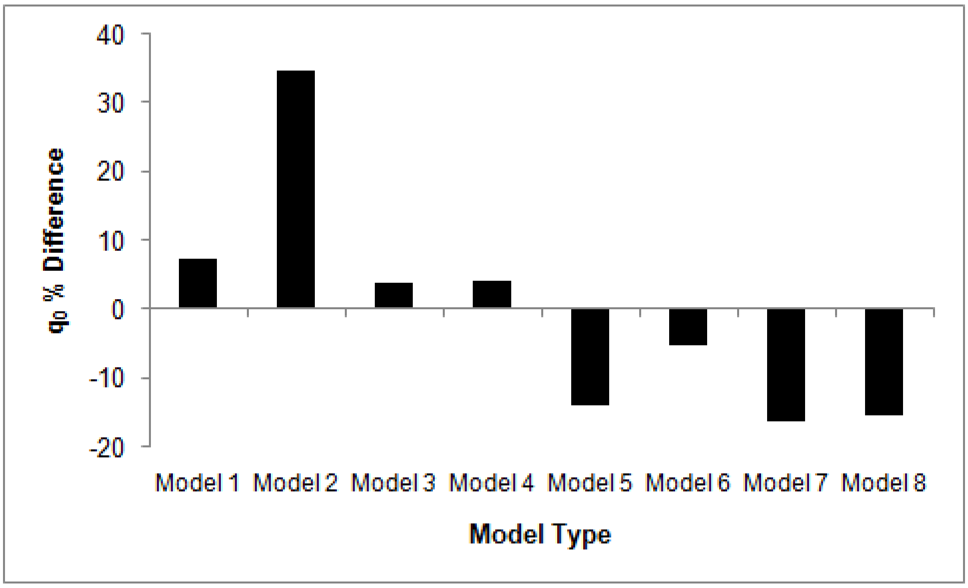

Figure 3.3. Percentage of difference for behaviour factor q_0

Studying the changes of behaviour factor based on the model types on average basis, the ductility was noted to be decreasing from Model 1 to Model 8. This was found to be logical as Model 5 and 6 possessed the highest relative stiffness in the lateral direction due to the additional infill panels as compared to the others. However interestingly, the ductility between Model 5 and 7 was observed to be rather similar. Model 5 comprised four infill panels, while Model 7 only possessed two panels. The arrangement of infill panels in Model 7 allows for more opening space between frames, while demonstrating the same ductility as Model 5 where 4 out of 6 spaces need to contain infill panels.

Eurocode 8 (CEN 1998, 2003) also permits an estimation of inelastic deformation by applying a certain amplification factor onto the displacement based on elastic analysis. This amplification factor is also termed as displacement behaviour factor (represented by q_A in this paper). Although the code does provide some default values for designers to choose from should the displacement factor is unknown, the selection criteria for a suitable value remains unclear. Presented in Fig. 3.4 is the plotting of q_A for all models. Although the ductility factor demonstrated a decreasing pattern between the models, the displacement factor on the other hand is changing randomly.

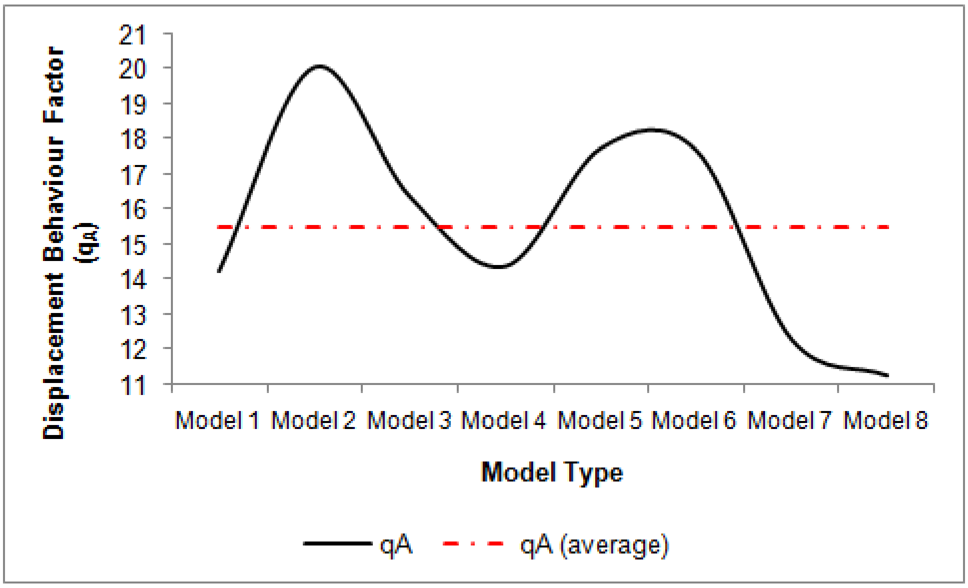

Figure 3.4. Displacement behaviour factors of Model 1 to Model 8

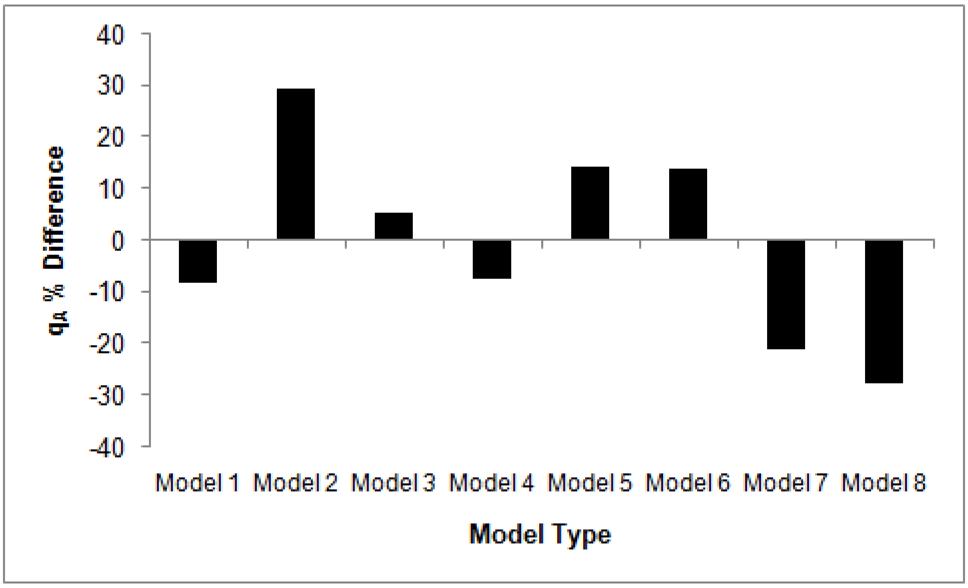

Figure 3.5. Percentage of difference for displacement behaviour factor q_A

There is no correspondence or any relationship noticed between the types of wall configuration (location), as well as vertical loading values with the displayed displacement behaviour factor. In some models, the difference could somehow be as small as lesser than 6 % while it can be up to 30 % (in Model 8). Such inconsistency reflected that should the structure is to be designed by performance-based procedure, careful attention should be paid in properly determining the nonlinear displacement that the structure could exhibit, given such a vast percentage of difference between models.

The mode of structural failure was also studied for Model 2, 4, 6 and 8 respectively. The structural failures of column element are denoted in red lines (see Fig. 3.6).

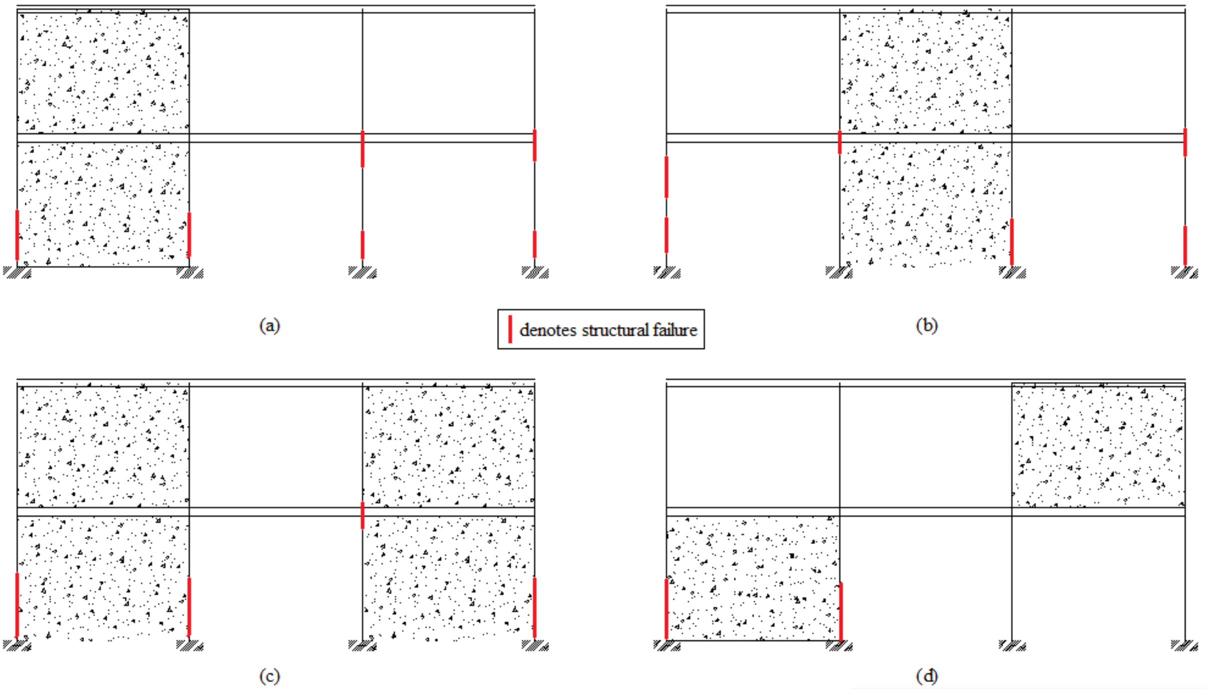

Figure 3.6. Structural failure: (a) Model 2; (b) Model 4; (c) Model 6 and; (d) Model 8

Besides displaying rather dissimilar ductility factor as well as displacement behaviour factor, the wall configurations were also observed to have affected different force demand onto supporting column elements. As denoted in Fig. 3.6(a), shear failure occurred at interstory level without infill panels, and all columns at ground floor failed near the bottom. Fig. 3.6(b) clearly illustrated the failure of columns different to the previous case, followed by the remaining two other figures. Studying the mode of failure in the first three cases, the additional wall panels as in Fig. 3.6(c) did somehow decrease the length of column failure, compared to the previous two figures. Interestingly, the wall configuration as denoted in Fig. 3.6(d) revealed the least column failure.

Fig. 3.7 shows the base shear demand obtained for different soil classification (S_A to S_E) from pushover analysis.

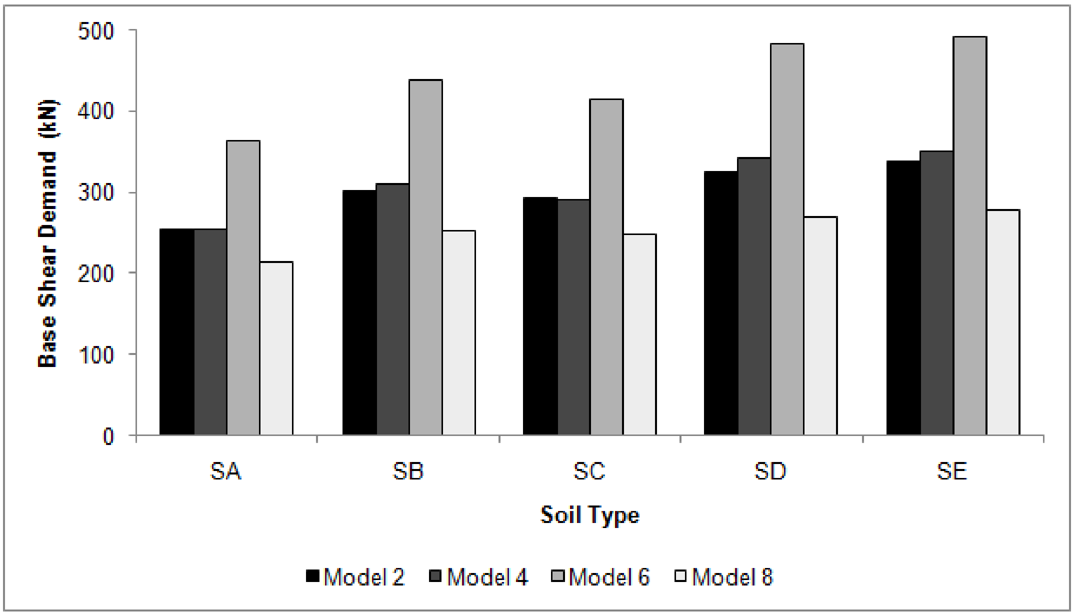

Figure 3.7. Base shear demand for different soil condition

Observing the trend of seismic demand from the above figure, the seismic responses for Model 6 (which possessed the highest lateral stiffness compared to others) were rather sensitive to the soil condition. Moving from hard to soft soil, the base shear demand was seemed to be increasing correspondingly. Despite demonstrating the lowest ductility factor, Model 8 was noted to be insensitive to the types of soil excitation. Its base shear demand seemed to be rather consistence regardless of the soil characteristics.

4. CONCLUSIONS

Nonlinear pushover analyses were successfully carried out in analysing the ductility factor as well as displacement behaviour factor of four different types of precast concrete infill frames, under both SLS and ULS vertical loading condition. It was observed from the study that the locations of infill wall affected significantly the global ductility of the frame structure, and also the displacement behaviour factor. Unless the structure is irregular in the plane direction, SLS and ULS vertical loading did not have significant impact onto the global ductility behaviour. However, the displacement behaviour factors were observed to have no correspondence with the parameters being studied (i.e. wall locations, as well as types of vertical loading). As demonstrated by Model 8, lower ductility and displacement behaviour factor should not be used as the only parameters in ruling out the adequacy of DCL structure in resisting seismic force. Despite having lower factors, Model 8 was having the lowest seismic demand compared to the remaining seven other models, and also suffering least structural failure.

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