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Modeling of damping effects in soil – Structure interaction in the pushover analysis

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Abstract

The paper shows aspects of damping modelling in nonlinear static pushover analysis (NSPA) of structures through systematization of damping types and formed flow diagram, depending on the type of applied target displacement analysis. By applying the developed flow diagram, in the process of creating and analysing numerical models of structures, it is possible to very efficiently consider which type of damping should be selected and how to introduce damping in nonlinear static pushover analysis. Generally speaking, in nonlinear static pushover analysis (NSPA) damping is introduced indirectly by reducing the response spectrum. The problem of introducing damping is considered taking into account the soil-structure interaction, modelling the kinematic effects and the damping effects of the foundation, i.e., the ground. Parametric analysis was performed by varying the parameters: the effect of foundation depth, i.e., the impact of the existence of underground floors and the impact of damping. Based on the developed pushover curves, the levels of target displacements for each individual condition were determined, and then these discrete values were classified according to soil categories. By connecting such discrete values of the target displacement, a cumulative curve was constructed, i.e., an envelope of possible states of drifts and forces. By applying the proposed procedure, it is possible to consider the possible level of nonlinear deformations of the system, taking into account the effect of interaction with the soil and damping. The research established that the introduction of the soil-structure interaction can significantly affect the values of global drift, and thus partially the correction of the relevant total lateral seismic force. The sensitivity of the change in the relevant total lateral seismic force is much lower than the displacement, because in the nonlinear domain the system has much less stiffness, even in certain situations the stiffness is zero, so a small increase in load can produce much greater deformation. Also, the research found that for different types of soil and different damping values, the fragments of the pushover curves, obtained by interpolating the target displacements for the same soil types, overlap at certain intervals.

Key words: damping, flowchart, NSPA, pushover curve, drift, seismic

1 Introduction

A more realistic description of the behaviour of structures during the earthquake requires the modelling and introduction of the soil structure interaction (SSI). This introduces the influence of the flexibility of the foundation structure and the soil. For the earthquake effect, deformation and displacement of the structure are the function of the interaction of three connected systems: the structure, the foundation structure and the geological environment in which the structure is founded. Methods that introduce the influence of structure-soil interaction into the nonlinear static pushover analysis (NSPA) are defined in FEMA 440 [1]. When determining the level of target displacement (TD) in NSPA, the effects of SSI and damping due to interaction with the ground are introduced indirectly.

When the displacement of the system is caused by the action of an earthquake, it is necessary, in addition to the viscous one, to consider hysteretic damping, which occurs due to the development of nonlinear deformations. Damping is most often introduced in the analysis of structures as an element of the critical damping whose values are a function of the type of material, and independent of the mass and stiffness of the system [2]. On the other hand, by applying an equivalent relative damping coefficient, damping can be considered on different types of materials, introducing it in the form of composite damping [3]. Also, by implementing a single equivalent relative damping coefficient, it is possible to take into account both viscous and hysteretic damping in the nonlinear analysis of structures [4]. In [5], the damping was analysed on the basis of the material nonlinear response of the system under cyclic action with a heterogeneous composition of the mechanical characteristics of the material. The research on the identification and analysis of building damping coefficients based on earthquake accelerometers is presented in [6], while the effect of viscous damping modelling on nonlinear seismic performance of multi-storey frame systems was considered in [7]. The introduction of damping effects in SSI and the analysis of the seismic response of the system were presented in [8]. Different soil characteristics, design seismic levels and SSI modelling techniques indicate a lower seismic response compared to the classical fixed structure model (restrained in foundations). The aim of the research presented in this paper is to consider in more detail the behaviour of the system during SSI for different levels of target displacement in NSPA and for different values of damping introduced into the system. A seismic performance analysis can show the level of sensitivity of variations of drift and seismic forces transmitted to the structure at SSI during an earthquake.

2 Damping modelling in NSPA

In the process of structure modelling and preparation of seismic analysis, according to which the structure calculation will be performed, damping can be introduced through: damping of materials, damping originating from link elements and damping which is directly defined in the analysis [9]. Material damping can be introduced as: modal damping, viscous damping and hysteretic damping. Link element damping can be introduced as: effective damping, nonlinear behaviour damping and damping of frequency dependent link elements. Depending on the analysis type, the general classification of damping can be: modal, viscous and hysteretic.

In NSPA, generally speaking, damping is not introduced prior to calculation, but it is subsequently defined after the structural design, in the target displacement analysis (TDA). Figure 1 shows the flowchart of introduction of damping in NSPA-TDA [10].

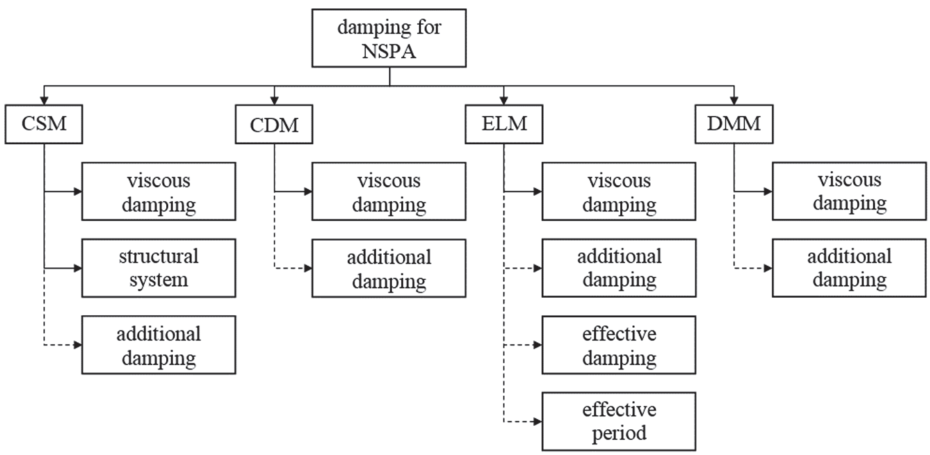

Figure 1. Flowchart of introduction of damping in NSPA [10]

The process of introducing damping is carried out via a single global coefficient which can take into account both viscous and hysteretic damping. Depending on the type of target displacement analysis, the following options are possible:

- Capacity Spectrum Method (CSM) [11]:
  Damping is introduced via the global damping coefficient as *inherent and additional* damping, but additional influence can be made through the type of the structural system.
- Displacement Coefficient Method (DCM) [12]:
  Damping is introduced via the effective damping coefficient which is used for generation of response spectra. In essence, this is a viscous damping, while the hysteretic damping is determined from the calculation, though an additional damping can be introduced by this coefficient.
- Equivalent Linearization Method (ELM) [1]:
  Damping is introduced via the global damping coefficient (inherent and additional damping), but effective damping can be defined as alternative, presented via the relative damping coefficient for the hysteretic system response.
- Displacement Modification Method (DMM) [1]:
  Damping is introduced in a similar fashion to DCM.

3 SSI in NSPA-TDA with damping effects

Problems of SSI analysis refer to the definition of: seismic action, dynamic soil characteristics, foundation stability in seismic conditions and SSI modelling. There are three key parameters that must be considered when introducing the effects of SSI according to FEMA 440 [1]:

- introduction of flexible foundation effects (FFE),
- kinematic interaction effects (KIE),
- dissipation of energy from the soil-structure system by radiation and hysteretic soil damping - foundation damping effects (FDE).

The classical model, in which no SSI is introduced, defines the connection between the foundation structure and the ground as an absolutely rigid base model (RBM). Such a system is excited by free field motion (FFM) with conventional damping (foundation input motion). Structural systems that take into account vertical stiffening elements (bearing walls) can be particularly sensitive, even to small rotations and translations, which are not taken into account when assuming a rigid base. According to FEMA 440 [1], for NSPA, SSI is modelled by introducing flexibility into the soil-foundation structure. Such an interaction model is called the flexible base model (FBM), whereby the impact of structural and geotechnical components of the foundations is introduced. The first component is introduced by modelling the flexible structure of the foundation, while the second component is introduced by modelling springs with associated stiffness components that substitute the soil effects. In this model too, the resulting recording of free field motion with 5 % of damping is used as a conventional initial value. Comparison of this model with the model which has an absolutely rigid foundation structure reveals that there is an increase of the vibration period of the structure and change in force distribution in cross-sections [13]. Additional improvement of SSI reflects in the introduction of kinematic interaction model (KIM) effects, so the filtered recording of the foundations, foundation input motion with (FIM) is taken into consideration. The final step in the improvement of SSI interaction in NSPA-TDA is introduced through the effects of the foundation damping model (FDM), and the ground acceleration recording is generated taking into account the damping of the foundation structure, too. This damping is obtained from the relative displacement of foundations and the soil, so an affective reduction of the spectral curve ordinate is achieved. The last model defined in this way contains both the KIM and FDM effects.
In practical analyses, foundation damping model is introduced via the coefficient which represents the ratio of the basic vibration period of the flexible base model and of the rigid base model. Other factors that affect the damping of the foundation are the dimensions of the foundation structure and the influence of the underground floors. Foundation damping is combined with conventional initial damping of the structure, in order to correct the damping coefficient of the entire system including the structure, founda-

tions and soil. The improved FIM recording of soil acceleration to which the foundation structure is exposed, of the KIM+FDM model, differs from the FFM recording, among other things, due to the statistics averaging of different ground acceleration recordings [14]. These effects belong to the group of KIM effects and they are important for the buildings with relatively short vibration periods (<0.5s), of large floor plan dimensions having underground floors. The ratio of response spectra (RRS) is used for presentation of KIM effects, via the ration of the FIM response spectra ordinate and the FFM response spectra ordinate. The effects of the existing foundation structure and underground floors deeper than 3m participate in the determination of RRS. KIM effects can be efficiently included by the procedure defined in [15]:

- determine the effective foundation size $b_e = \sqrt{ab}$, where a and b are dimensions of the foundation layout,
- determine RRS_{bsa} as a function of the vibration period T, where:

$$RRS_{bsa} = 1 - \frac{1}{14.1}\left(\frac{b_e}{T}\right)^{1.2} \text{ za T} \geq 0.2 \text{ s} \tag{1}$$

- if e is the depth to which there are underground floors, it is necessary to calculate the additional effects to RRS_e caused by the existence of underground floors, which are the function of the vibration period T, where:

$$RRS_e = \cos\left(\frac{2\pi e}{Tnv_s}\right) \text{za T} \geq 0.2 \text{ s} \tag{2}$$

where v_s is the velocity of the shear wave for the local soil conditions, taken as an averaged value of velocity at the depth e, n factor of shear wave reduction for the expected peak ground acceleration (PGA): PGA = 0.1g: n = 0.9, PGA = 0.15g: n = 0.8, PGA = 0.2g: n = 0.7 and PGA = 0.3g: n = 0.65,

- multiply RRS_{bsa} and RRS_e in order to obtain the final value of RRS for the required vibration period. The spectral acceleration recording ordinate is the product of free field recording spectrum and RRS.
- in order to obtain the complete recording spectrum for the foundation structure, it is necessary to iterate the previous steps for different vibration periods.

The velocity of shear waves which are the function of the soil classes according to FEMA 273 are [16]: A: v_s>1524m/s², B: 762<v_s<1524, C: 366<v_s<762, D: 183<v_s<366, E: v_s<183m/s². Foundation damping effects are presented through modified system damping coefficient. In the initial structure dumping coefficient β_i the foundation damping β_f, is not taken into consideration, and generally speaking β_i is taken to be 5 %. The final value of the system damping coefficient β_0 takes into consideration SSI, so that change from β_i over to β_0 has an effect on the correction of the elastic response spectra. Determining the foundation damping coefficient is conducted according to:

- period of structural vibrations is determined comprising that the foundation structure is fixed T_{fix}, and afterwards, flexibility in determination of the period of vibrations T_{flex} is taken into consideration, whereby values of stiffness components are determined according to FEMA 356 [12]:
- foundation damping coefficient is determined from:

$$\beta_f = a_1\left(\frac{T_{flex,eff}}{T_{fix,eff}} - 1\right) + a_2\left(\frac{T_{flex,eff}}{T_{fix,eff}} - 1\right)^2 \tag{3}$$

- while the system damping coefficient β_0 which takes into consideration SSI is determined from:

$$\beta_0 = \beta_f + \frac{\beta_i}{\left(\frac{T_{flex,eff}}{T_{fix,eff}}\right)^3} \tag{4}$$

The research presented in this paper is based on the application of the DMM method to determine the target displacement in NSPA. The DMM method is a newer generation of the DCM method, where certain coefficients that participate in the calculation were corrected, and parts of the calculation related to hysteretic behaviour models were further improved [10].

4 Multi-parameter numerical analysis

Applying the previously described mathematical formulation of the damping problem in SSI, NSPA analyses were performed first, and then the target displacements were determined using the DMM method. An 8-storey 4-span reinforced concrete (RC) frame modelled by linear finite elements was considered as a representative model, with nonlinear effects being involved in the development of geometric and material nonlinearity. The dimensions of the span are 5 m, and the height of the floor is 3 m. The columns are 40x60 cm on the first and second floors, 40x55 cm on the third and fourth floors, 40x50 cm on the fifth and sixth floors and 40x40 cm on the seventh and eighth floors. The reinforcement of the columns of the first and second floors is 10RØ19, while the reinforcement of the remaining columns is 6RØ19. The beams are 30x60 cm on the first and second floors, 30x55 cm on the third and fourth floors, 30x45 cm on the fifth and sixth floors and 30x40 cm on the seventh and eighth floors. The class of concrete is MB 30. The reinforcement of the beams from the first to the sixth floor is 11RØ19 at the ends and 6RØ19 in the middle, while the reinforcement of the beams of the seventh and eighth floors is 9RØ19 at the ends and 4RØ19 in the middle.
The design elastic response spectra, according to FEMA 273 [16], with the ordinate normalized to value 1 is presented in figure 2. In relation to these response spectra, the

acceleration spectra with KIE and FDE are generated. The impact of the coefficient e, which introduces the existence of underground floors, can be analysed by comparing the spectra developed for different values e = 0 and e = 9 m with the given constants: soil type C, v_s = 600m/s², PGA = 0.3g, n = 0.65, β_0 = 0.05. Reduction of values in the field of constant accelerations is up to 50 % in case of the development of response spectra for KIE and FDE.

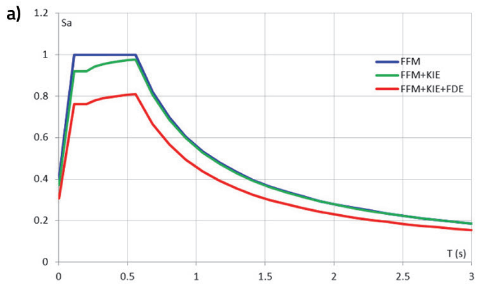

Figure 2. Diagrams of elastic response spectra for FFM, response spectra corrected by KIE and response spectra corrected with KIE and FDE for type C soil, v_s = 600m/s², β_0 = 0.05: a) e = 0, b) e = 9 m

Based on the conducted NSPAs, pushover curves are developed which represent the total horizontal shear force for the corresponding displacements of the highest node of the structure. Then, target displacements for different values of parameters were analysed, separating KIE from FDE and by finally joining them. Taking into consideration KIE only, the variation of the following parameters is considered: e = (0, 3, 9, 15) m, n = (0.65, 0.7, 0.8, 0.9), soil = (A, B, C, D, E), while taking into consideration FDE only, the variation of the following parameters is considered: β_0 = (0, 0.05, 0.1, 0.15, 0.2, 0.25, 0.3), soil = (A, B, C, D, E). Taking into consideration both KIE and FDE, variation of the following parameters is considered: β_0 = (0, 0.05, 0.1, 0.15, 0.2, 0.25, 0.3), soil = (A, B, C, D, E), for e = 3 m, PGA = 0.3g, n = 0.65. In figure 3 is presented the impact of introduction of KIE by analysing the global drift of DR structure. The global drift parameter represents the ratio of the horizontal target displacement of the highest structural node, from NSPA and DMM analyses, and the building height (expressed in percentiles). By increasing the coefficient e from e = 0 to e = 9 m the global drift is minimally reduced, while at e = 15m there is a considerable reduction of the global drift. The lowest values of the global drift are obtained for the type A soil, while the highest values are obtained for type E soil, which is to be expected, because the first soil type has the highest values of shear wave velocity v_s >1524 m/s². In case of the higher values of the coefficient e ≥15 m, which is the effect of deep underground floors, the global drift is more considerably reduced for C, D and E soil types.

The impact of FDE is considered via the coefficient β_0, which is varied within the limits of possible values for the reinforced concrete frame system. The systems in which, conditionally speaking, there is no damping, develop global drifts several times higher than the systems which have only 5 % damping. Figure 4 presents the variation of the global drift in the function of the variation of the coefficient β_0. The multi-parameter research can confirm that, within the considered values, is dominant in relation to KIE of the frame system.

Joint effects (KIE and FDE) for *PGA* = 0.3g, *n* = 0.65 and *e* = 3 m are considered via pushover curves and target displacements for all types of soil in the function of the total damping. Figure 5 shows developed pushover curves, levels of target displacements for all types of soil and damping coefficients, and separately considered are the target displacements which do not take into account the SSI effect.

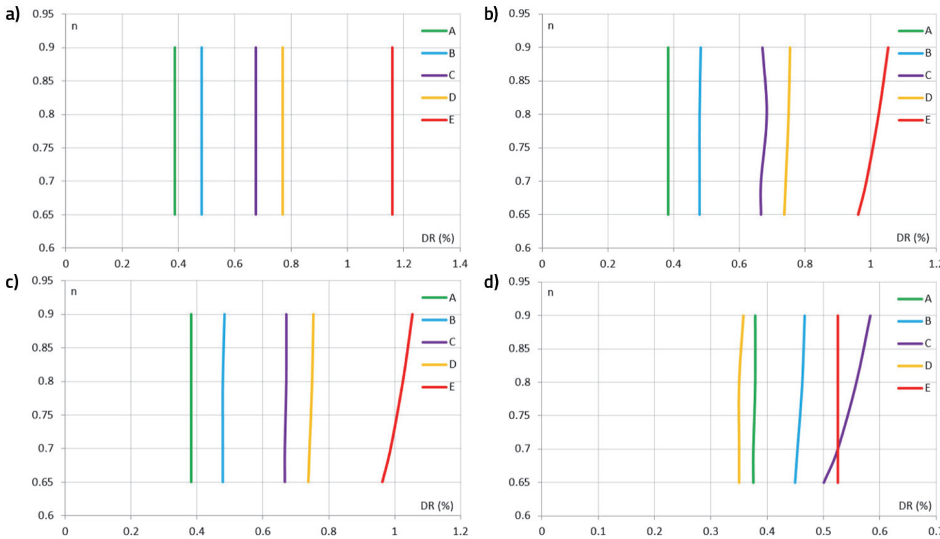

Figure 3. Variation of the global drift for the level of target displacement in the function of different types of soil A, B, C, D and E – introduction of KIE: a) *e* = 0, b) *e* = 3 m, c) *e* = 9 m, d) *e* = 15 m

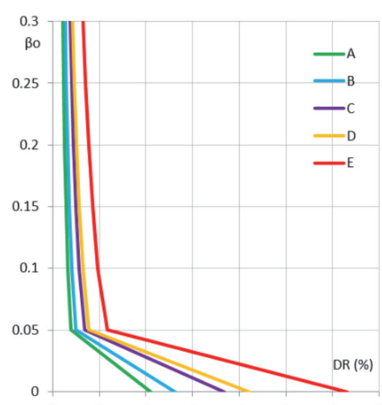

Figure 4. Global drift variation depending on the coefficient β_0

The values on the abscissa are shown as a function of the global drift DR, while the values on the ordinate are shown as a function of the ratio of the total shear force at the base of the structure and the dead weight of the structure *P/W*. The model of the pushover curve, obtained by NSPA, consists of elastic and a nonlinear part. The stiffness in the nonlinear domain is significantly lower than the stiffness in the elastic domain, so a small increase in seismic force is sufficient to cause considerable deformations of the system. Realized levels of target displacement, in all soil types, except for certain situations in type E soil, are located on the pushover curve. The construction was designed according to the regulations, but even for the conditions of foundation in type A soil, a global drift was achieved, for the level of the target displacement, bordering with the nonlinear part of the pushover curve. In soils with poorer physical and mechanical characteristics, the global drift, for the level of the target displacement, moves along the pushover curve, increasing. A special situation was obtained in the case of type E soil, where, in fact, certain target displacements were not achieved, because they are greater than the displacements defined by the push curve. This means that in these situations there is no sufficient available capacity of the structure in relation to the set seismic requirement. However, it should be noted that when the structure is modelled without SSI or with low damping, then this situation occurs, but when KIE and FDE are introduced into the system, the capacity of the structure is satisfactory in relation to the seismic requirement. The contribution of the effects of SSI with the introduction of the system damping reduces the global drift by up to 50 %.

Figure 6 shows isolated fragments of pushover curves only for the levels of target displacements. By connecting such discrete values of target displacements, in the form of fragments, a cumulative curve is constructed, i.e., an envelope of possible states of drifts and the total shear force at the base of the structure. By applying the proposed procedure, it is possible to analyse the possible available level of nonlinear deformations of the system, taking into account the effect of interaction with the soil and damping. For soil type A the minimum value of global drift is 0.2 %, while for soil type E the maximum value of global drift is up to 1 %. Such a wide range of values indicates differences in the behaviour of the structure founded in different types of soil, regardless of the type of structural system. Considering that the envelope of target displacements is designed for various types of soil and various levels of damping β_0, in certain situation the values of pushover curve fragments coincide.

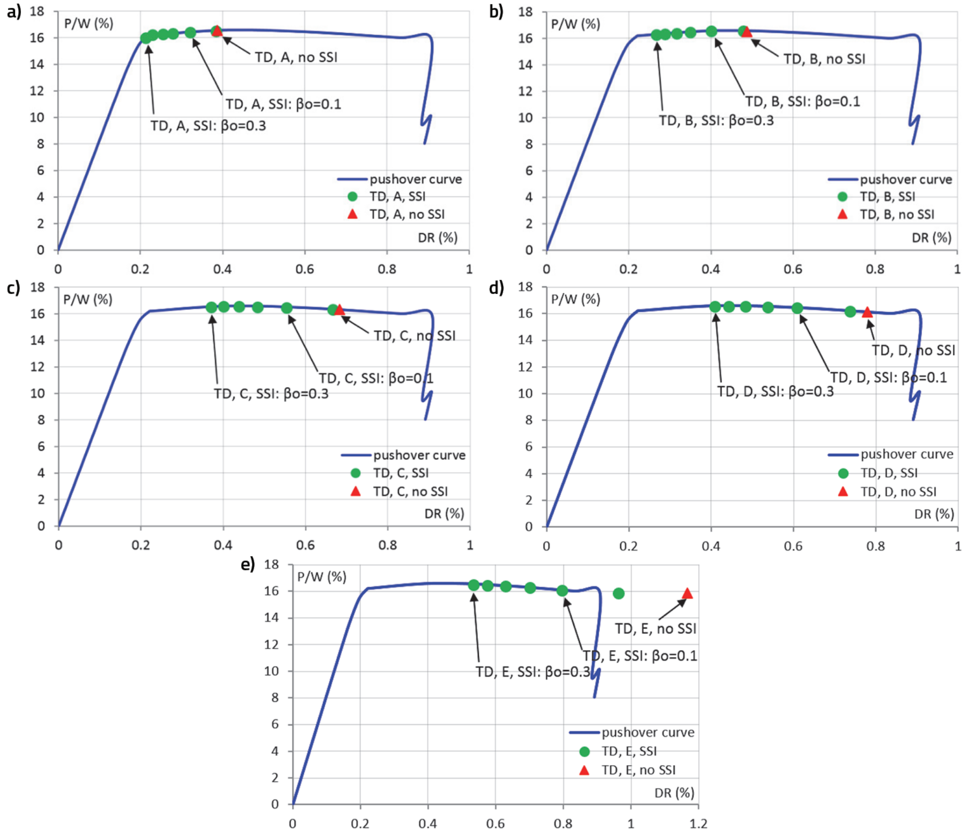

Figure 5. NSPA pushover curves and target displacements (TD) for soil types: a) A, b) B, c) C, d) D, e) E

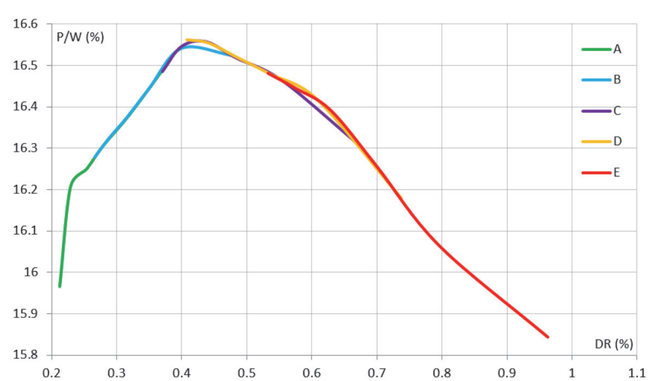

Figure 6. Envelope of possible states of drifts and total shear force at the structure base: *PGA* = 0.3g, *n* = 0.65, *e* = 3 m

5 Conclusion

The research, presented through the multi-parameter analysis, found that the introduction of SSI significantly increases the global drift, and thus significantly less affects the overall shear force at the base of the structure. The sensitivity of the variation in the relevant total shear force is significantly lower than the displacement, because in the nonlinear domain the system has significantly less stiffness, so that a small increase in load can produce significantly greater deformations. At higher values of the coefficient *e*, the law of the variation of global drifts, as a function of soil types, is not unambiguous. In the case of dampening reduction, the values of global drifts increase significantly. It is recommended that the value of the damping involved in KIE and FDM be considered in detail before design.

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