Periodica Polytechnica
Civil Engineering

Development of a Lateral Performance Prediction Equation (LPPE)

62(2), pp. 429–443, 2018
https://doi.org/10.3311/PPci.9050

Hakan Karaca[1*]

RESEARCH ARTICLE

Received 26 January 2016; Revised 02 October 2017; Accepted 14 November 2017

Abstract
The need to find out the behaviour of structures under the influence of lateral loading in the simplest and most accurate way encourages researchers to seek and develop different methods. The efficiency, ease of application, and time to reach the desired results are the factors which have influenced the way these methods are developed. In a similar stance, the aim of this study is to develop a method to evaluate the structures in terms of lateral performances without consuming too much time and effort. To this end, by using the analytical methods, it is planned to develop an equation to model the relationship between global drift ratio as the lateral performance variable and a number of selected geometrical and structural variables. This equation is named as lateral performance prediction equation (LPPE) by the author.

Keywords
lateral performance, number of storey, lateral stiffness index, lateral strength index, soft storey index, multiple regression analysis

[1] Nigde Directorate of Disaster and Emergency Management Presidency Nigde/Turkey
* Correspondig author, email: karaca26@hotmail.com

1 Introduction

It is clear that the structures are not mainly designed to reach a certain capacity or deformation level but to distribute the vertical and lateral forces safely to the ground and exhibit displacements within serviceability limit states. As long as the load-carrying mechanism of the structure successfully performs transferring of oncoming loads or energy safely to the ground, it is not the primary issue whether the transfer of loads is performed by elastic or inelastic behaviour. The reasons behind the existence of different types of responses as elastic or inelastic are the different design approaches and applied codes, which impose a certain type of behaviour onto the structure. These different approaches and codes contribute to the sizing of structural elements and the shaping of the geometry of the structure. Therefore, the structural system and geometry might show considerable variability from one structure to the other depending on the aforementioned reasons.

It should also be highlighted that depending on the applied codes in the design and the design approaches used, the lateral response of structures in terms of drifts and forces could be highly varied. For example, a structure might be designed to resist lateral forces within elastic limits; therefore, instead of exhibiting large lateral drifts, it absorbs and dissipates energy with very small lateral drifts but with considerably large forces carried by the system. Therefore, the drift predictions of such a structure would be very different from those of a structure designed to dissipate the earthquake energy by inelastic response. Consequently, it is not easy to estimate the structural response in terms of drifts or any other lateral performance variable with varying column sizes, height distribution characteristics, and the number of storeys. In addition, a structural behaviour might shift from highly ductile to lesser ductile behaviour if the column sizes change, which in turn complicates the matter even more. However, regardless of the design approaches or applied codes, it is intuitively deduced that with greater size of columns, with more uniform height distribution and higher number of storeys, lateral performance of a structure changes within an expected pattern. In other words, if a structure is modified to create new models with varying column sizes, height distribution, and number of storeys without changing other parameters and the geometrical orientation, the influence of this modification to the lateral performances would be in a predictable pattern.

This study is built on the idea of prediction of lateral performance analytically by using lateral strength and stiffness indexes which basically are the ratio of the total column area and total stiffness of columns in a single storey to the total floor area bounded by structural frame, the soft storey index, which is the ratio of the height of the first storey to the height of the second storey and the number of storey. As the engineering demand parameter for lateral performance, the global drift ratio is selected due to its good association with structural damage [2].

2 Review

It is a fact that relating the lateral performance, mostly in terms of displacement, vulnerability or damage, to the most basic structural and geometrical variables of buildings is not new and it is probably already known by instinct since old ages. For the historical structures, by intuition and experience, the pioneers of the field established the basic rules of thumb for building structurally sound and safe buildings. In brief, it was intuitively deduced that column sizes and distribution, and total height and height distribution have an impact on the overall performance. Explicitly speaking, lateral strength and stiffness, which are based on column size and distribution; the existence of soft storey, or significantly higher first storey; and the number of storeys have highly correlated relationships with lateral performance. The relationship, however, is not very explicit and it is indirectly expressed through more complex parameters such as ductility, strength reduction factor, fundamental period, lateral stiffness, beam-to-column stiffness ratio, reinforcement ratio, base shear etc. In fact, in the literature, there are numerous studies relating building performance to the mentioned complex parameters developed through nonlinear static procedures [3][4].

Since the simplest geometrical parameters do not have one-to-one or direct relationship with lateral performance, there are not many attempts to generate a closed form LPPE based on these parameters. However, as the analysis capability increased, empirical prediction equations started to flourish. In one of the earliest studies [5], researchers attempted to form an equation to predict the tip displacement by using a number of geometrical parameters and with simple regression analysis. With further effort to predict lateral performance and damage, artificial neural networks (ANN) were incorporated into the field [6]. The newly introduced method allowed processing a large number of inputs as predictor variables and outputs as predicted variables. Hence, an influx of parameters which are thought to influence lateral performance were introduced. The introduction of numerous parameters indicates the complex nature of the problem, which indeed requires using the right variables and demand parameters. However, as the whole point of the empirical procedure is to test whether the selected variables have any relationship with the demand parameters, no conclusive study has yet been performed, which would bring an end to further investigation. Moreover, since the distribution and the bias of the variables within the database is a strong determining factor in the end result, the variation of the parameters from one study to the other should not be worrying. Offering an opportunity to easily manipulate the input and output characteristics, the methods based on ANN continued attracting several researchers in the field and, studies were performed especially to predict lateral drift and displacement [6] [7][8]. The concept of associating the basic geometrical and structural variables with lateral performance had its way to Turkey a few decades ago [9]. The purpose of the initial study[9] was to associate certain structural features with structural vulnerability. Then, a wave of similar studies, which used the same earthquake damage databases of Turkey, focused on the assessment of the vulnerabilities of structures by using discriminant methods ([1][10][11][12]). In all these studies, a method which is based on the empirical relationship between the observed damage and the simple geometrical parameters of building has been proposed. They intended to generate an equation to predict the level of vulnerability and damage by introducing the predictor variables of lateral strength and stiffness indexes, soft storey index, the number of storeys, overhang ratio and redundancy score which is based on the number of continuous frames.

Indeed, the parameters mentioned in the above studies are considered for this study. Only four of the above listed parameters, as lateral strength and stiffness indexes, soft storey index and number of storey, are selected for the development of proposed LPPE. Among the parameters of the proposed LPPE, a considerable amount of information has been gathered for the number of storeys and the soft storey index, and thus, the existence of a correlation between lateral performance and these parameters has already been accepted by default. For the parameters of lateral stiffness and strength indexes, sufficient evidence on the scaling characteristics of these parameters with lateral performance was provided in a recent study [13]. There are also studies focusing on the same subject [14][15] which also proved the existence of dependency between the ratio of the shear wall area to the floor area, with lateral performance. All three studies showed a negatively exponential trend of lateral performance associated with the increase in the mentioned parameters.

3 The method

Figure 1 outlines the utilized procedure in order to develop the proposed LPPE . As the first step, a database is to be formed with the structures according to the initial prescriptions imposed by the study. Then follows the determination of the right predictor variables for the formation of the LPPE. Indeed, in order to identify the right predictor variable in the formation of an empirical equation and the determination of the outliers requires the pre-evaluation of the proposed parameters with respect to the predicted variables. In other words, supervised approach is always required before number crunching in order to make sense of the quantities which indeed have physical meaning. Even a simple visual screening of the database through simple plots guide the analyst as to whether the database needs an elimination scheme for the identification of the outliers both as predictor and predicted variables.

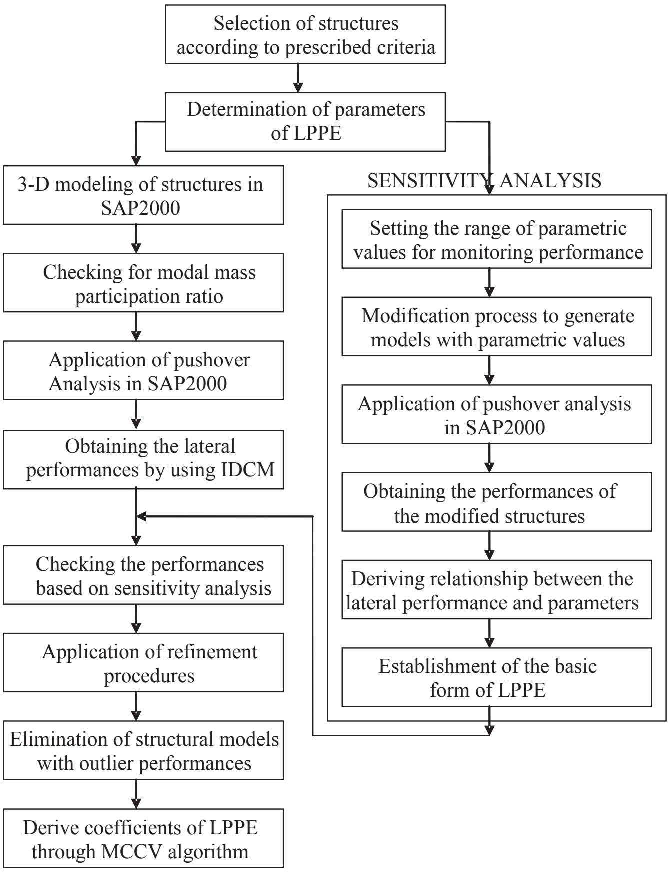


Fig. 1 Flowchart Describing the Development od Lateral Performance Prediction Equation (LPPE)

Generally, a pure empirical analysis does not allow the pre-formulation of the hypothesis; however, if the nature of the problem allows performing a sensitivity analysis, the basic scaling characteristics of the predictor variables (lateral stiffness and strength indexes, soft storey index and the number of storeys) with respect to the predicted variables, which is the global drift ratio in this case, could be determined. A range of predictor variables is generated within the physical limits allowed by the Turkish building code. Then, the structural models are modified to suit the range of the values of the predictors and the models are regenerated. These models then are analysed for structural stability and compliance before utilization for further analysis.

The target or roof displacements are computed by the Improved Displacement Coefficient Method (IDCM), and performance global drift ratios are obtained by normalizing the roof displacements by the total height. After that, the process is repeated for all the values of the predictor variable within the established ranges and a number of global drift ratios are obtained. Consequently, through the sensitivity analysis ,the relative scaling of the lateral performance with respect to the predictor variables are obtained.

Together with the sensitivity analysis, all the models in the database are subjected to pushover analysis, the models with unacceptable modal mass participation ratio are eliminated and for the remaining structural models the target displacements are computed by using the IDCM. By using the relationships obtained by the sensitivity analysis, the structural models with the outlier performances are quickly identified. Then, together with the prescribed limits imposed by the IDCM, the models with the outliers are evaluated to check whether there is a real physical reason for elimination.

After the elimination scheme, the remaining database, then, is subjected to a procedure which is based on Monte-Carlo cross validation (MCCV) scheme. The application of the cross validation scheme requires the random partition of the database into two as training and validation parts. The number of samples in each part is determined in such a way that secures precision and prevents the loss of meaning. Since both training and validation samples are randomly selected, the procedure is repeated as many times as desired. In each run, a set of coefficients are derived for the LPPE and the derived coefficients are tested with the validation sample. The success of each round is determined by the correlation coefficient and significance test results. Then, the rounds with the highest overall correlation coefficients above a predetermined threshold with acceptable significance test results are selected. The significance of each coefficient is also checked for each round in order to make sure that there is not a loss of meaning. The median values and the standard deviations of the coefficients are then assigned as the coefficients of the LPPE.

4 The database

A total of 37 structures from Eskisehir were gathered for the development of the LPPE. These structures were arbitrarily chosen to avoid any bias that could arise due to the similarities between certain structural parameters. In the selection process, a few restrictions were imposed such as the number of storeys was limited between 4 and 8 storeys, and the vertical uniformity of the storeys was satisfied. Moreover, only moment resisting frame and shear walled frame structures were considered.

All the structures were modelled in three dimensions using the SAP2000 program [16]. In the modelling, conventional beam-column frame modelling assumptions were used. Primary and secondary structural elements were identified, and secondary structural elements such as infill walls were excluded from the models. Primary structural elements of beams, columns, shear walls, and slabs were modelled as line and area elements. Plastic hinges were assigned to the member ends to model the flexural response in beams and to model the biaxial flexural response under axial loads in columns. In the pushover analyses, the default force-displacement characteristics of plastic hinges that are based on criteria listed in [17] were utilized.

Most shear walls are designed to resist high shear forces and designed to fail first in the flexural mode and then in shear mode to avoid the undesired consequences in an earthquake. Consequently, after necessary checks, it was decided that it is reasonable to model these walls only with biaxial flexural hinges. The slabs were modelled as diaphragms that transfer axial loads between the adjacent frames. The interaction of the neighbouring frames was allowed with this model; however, the in-plane and out-of-plane bending was restricted for the slab elements. The same vertical loads were assigned to all the structures to reduce the variability in lateral performance that could be caused by different vertical loads. Dead and live loads were assumed as 300 and 200 kg/m^2, respectively. A wall load of 1050 kg/m was assigned to each beam element.

The 3-D models of the structures were assessed as each direction of the models were treated as different structures. Therefore, 74 structural models were generated out of 37 structures. Before application of the pushover analyses, the modal mass participation ratios of these structural models were checked. The models with modal mass participation ratio less than 60% were eliminated from the database. Moreover, both the uniform and mode shaped lateral loading were applied while the translational roof displacements at the centre of mass were monitored in the pushover analysis.

4.1 The response spectrum curves

As a part of a probabilistic seismic hazard study that involves development of response spectrum curves for the city of Eskisehir, the elastics response spectrum curves were developed for 5% damping coefficient as shown in Figure 2 [13]. The curves were developed for different annual exceedence rate of earthquake occurrences in 50 years and the site condition was assumed as NEHRP B/C boundary site with V_s = 760 m/s. As the IDCM requires demand displacements, these curves were utilized in this study for the computation of the lateral performances of the structural models gathered from the city of Eskisehir.

5 Sensitivity analysis

The intuitively deduced relationship between the size of the columns, height distribution, the number of storeys and the lateral performance in terms of displacements and global drift ratios has to be proved by sensitivity analysis.

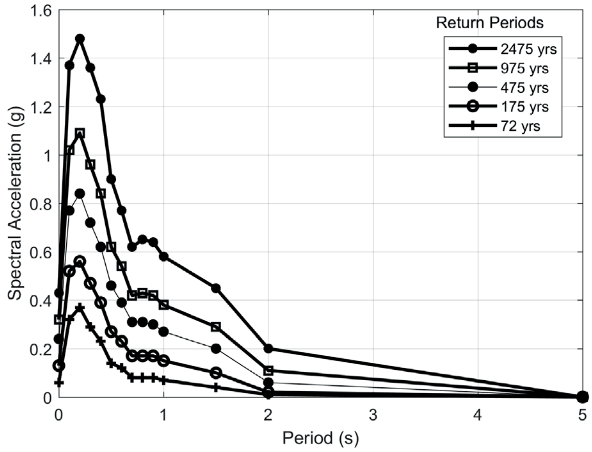


Fig. 2 Elastic Response Spectrum Curve Developed for the City of Eskisehir (NEHRP B/C Boundary Site, Response at 5% Damping)

The influence of the sizing of the columns over lateral performance in terms of flexure and shear can be expressed by the introduction of lateral strength index (*lsiA*) and lateral stiffness index (*lsiI*) as shown in the following equation

$$lsiA = \sum A_{col} / A_{sf}, lsiI = \sum I_{col} / A_{sf} \quad (1)$$

where A_{col} is the total cross-sectional area of columns in a single storey, I_{col} is the total moment of inertia of the columns in a single storey, and A_{sf} is the total floor area bounded by the structural frame. The uniformity of the column sizes, floor area and the orientation has to be satisfied for the application of these expressions for a single structure. Soft storey index (*ssi*), or the ratio of the height of the first storey to the second storey, is introduced to model the influence of soft storey, if exists, over the lateral performance. Lastly, the number of storeys (*n*) is included in the proposed LPPE to model the relationship between lateral performance and the number of storeys, which is self-explanatory. The following equation was written to model this relationship. (2)

$$LP(lsiA, ssi, lsiI, n) = f(lsiA) + f(ssi) + f(lsiI) + f(n) + \sigma$$

where *LP* represents the target or roof displacement for the considered response spectrum curve and *f* represents the proposed parametric functions. In the development of the LPPE, the most appropriate lateral performance variable that correlates with the mentioned structural parameters is sought by considering the fact that all buildings are designed for the best performance, not in accordance with the criteria like highest capacity or minimum deformation. Consequently, there is a possibility that different lateral performance variables relate to the structural parameters in different ways. Therefore, different methods with different lateral performance variables should be assessed with the proposed structural parameters. In this study, global drift ratio was selected as the lateral performance variable, and the IDCM was employed to obtain the global drift ratio of the structures.

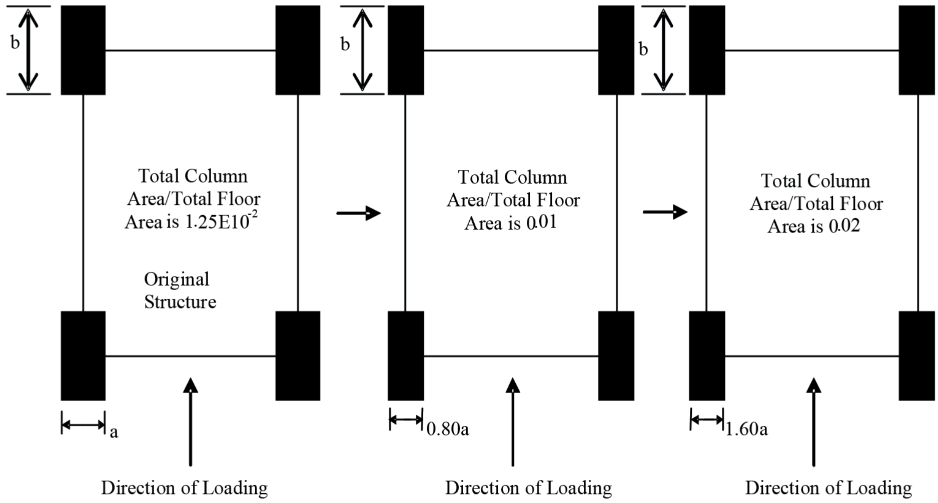


Fig. 3 Lateral Strength Index Modification Scheme

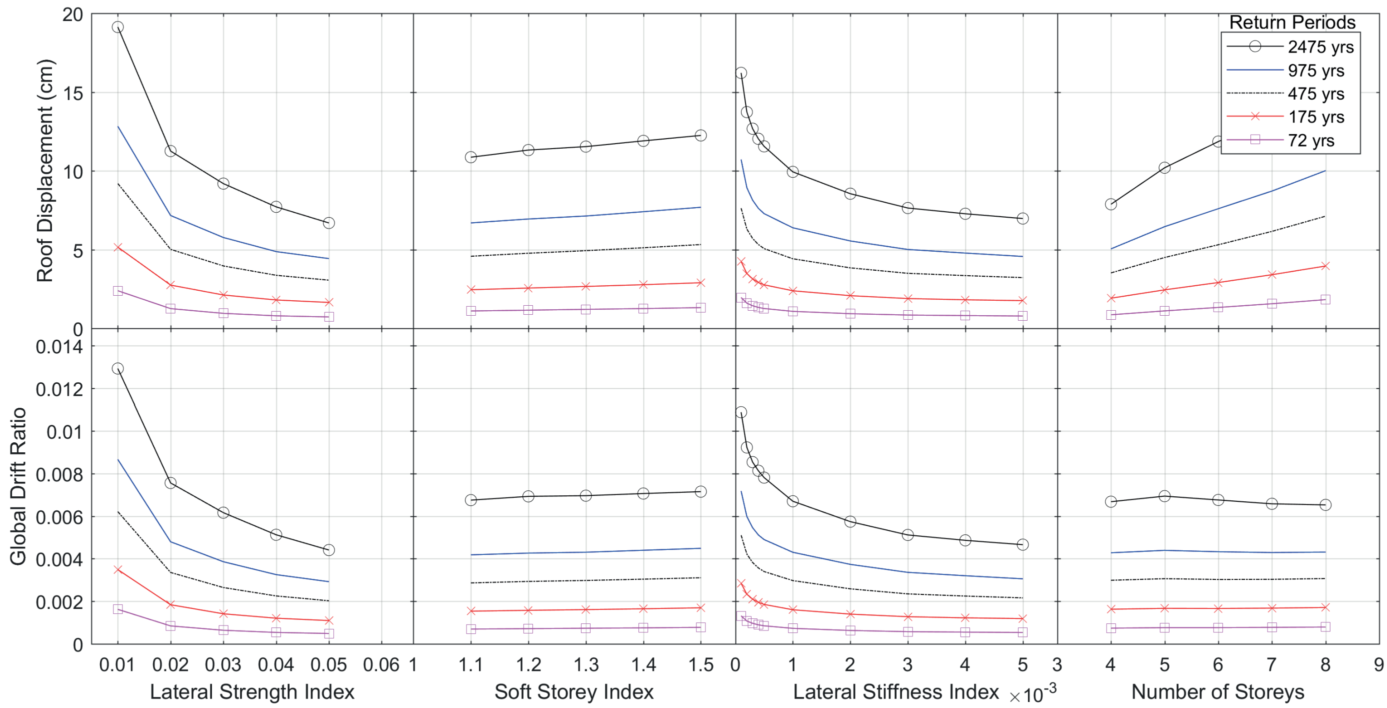


Fig. 4 The Scaling of Lateral Performance with respect to the Proposed Predictor Variables

The sensitivity analysis is conducted to examine the scaling of the lateral performance of a structure with respect to its structural parameters. Parameters of lateral strength and stiffness index, soft storey index and the number of storeys are modified for each structural model in each direction, and the modified structural models are generated with predefined values of the proposed structural parameters. The ranges of values are decided after checking for the distribution of these indexes within the sample and the criteria of the related standards about the column sizes and height distribution. Figure 3 explains the modification scheme for structures with varying lateral strength indexes. The predefined range of lateral strength indexes of 0.01 to 0.05, lateral stiffness indexes of (1, 2, 3, 4, 5, 10, 20, 30, 40, 50) × 10^{-4} m^2, soft storey indexes of 1, 1.1, 1.2, 1.3, 1.4, 1.5, and the number of storeys of 4, 5, 6, 7 and 8 were specified and the structures in the database were modified to generate the models with these values. After generation of the artificial models, their performances in terms of roof displacements and global drift ratios were computed by using the IDCM, and the relationship between the proposed structural parameters and roof displacements and the global drift ratio were obtained.

Confirming the previous studies, the nonlinear scaling of the lateral performance with respect to the lateral strength and stiffness indexes is very well modelled in Figure 4. The negatively exponential reduction in lateral performance is observed for both parameters, but with a varying degree of curvatures. According to the results of the sensitivity analysis, the final form of LPPE would look like the following equation:

$$\delta_{GDR}(lsiA, ssi, lsiI, n) = a_1 + a_2(lsiA)^{a_6} + a_3(ssi) + a_4(lsiI)^{a_7} + a_5(n) + \sigma \quad (3)$$

where δ_{GDR} is the global drift ratio, a_1, a_2, a_3, a_4 and a_5 are the coefficients of LPPE, a_6 and a_7 are the power coefficients to model the nonlinear scaling between the lateral performance and lateral strength and stiffness indexes, and σ is the standard deviation. The power coefficients can be selected from a narrow range of values since the sensitivity analysis provides sufficient clues about the degree of nonlinearity. It should be mentioned that there is an issue of correlation among the parameters of Equation 3. The lateral strength and stiffness indexes are related to each other and both the number of storeys and the soft storey index are related to the lateral performance through height. The strong correlation between two predictor variables in the multiple regression analysis, in other words, the multicollinearity, can severely affect the outcome of the regression analysis. Therefore, unless the inclusion of both parameters negatively influences the regression analysis, a solution must be implemented before progressing with a healthy multiple regression analysis. In most cases, the most straightforward solution to the multicollinearity problem is the elimination of one of the correlated parameters.

The important determining factor in the type of lateral response of a structure, as shear or flexure dominated, is the column-beam stiffness ratios of a structure. In general, the columns are designed to be stiffer than beams so that the structure would perform in a flexure-dominated response and would not fail in a disastrous fashion. However, depending on the date of design and construction, as the design approach differs, some of the structures do not comply with the mentioned approach. Therefore, both type of behaviour might dominate the lateral response in the compiled sample. Consequently, it is concluded that inclusion of both indexes is necessary, as lateral displacement cannot be modelled with the exclusion of either of the flexural and shear displacements.

6 Refinement of the database

By using the response spectrum curves developed for Eskisehir and the structural models of the database, the performance roof displacements and global drift ratios are obtained. Both directions of structures are evaluated as a different model in the evaluation. The site class is accepted as C according to the NEHRP specifications, and the corresponding site amplification factors are applied to obtain the elastic response spectrum curves for the NEHRP C site class.

In the evaluation, only the roof displacements that are computed by using response spectrum curves developed for earthquakes with return period of 475, 175 and 72 years are considered since the calculated roof displacements by using response spectrum curves developed for earthquakes with 2475 and 975-year return periods are bigger than the ultimate displacements obtained by the structural analysis. The roof displacements are converted into global drift ratios through normalization with respect to height. As presented in Figure 5, the distribution of these performances are plotted to see if the performances follow the relationships derived by the sensitivity analysis. The fitted curves are based on the results of the sensitivity analysis, and coefficients are specifically assigned to model the lateral performances of the structures in the sample. As the first impression, the dispersion of lateral performances is quite noticeable in the plots. The existence of too many outlier values obviously causes a loss of pattern. Therefore, without any further evaluation, and as the principles of supervised learning prescribes, a refinement of the database is deemed necessary.

The outlier structures are checked for certain structural features that create the wide dispersion in the lateral performance distribution. Among the several features, the fundamental period, relative sway ratio, and ductility are identified as potential features that could create such a dispersion. In addition, the prescriptions about the use of IDCM are checked in the related reports [17,18]. Indeed, a re-examination is suggested about the structures with high relative sways, which is defined as the ratio of the maximum displacement at the roof level to the displacement at the center of mass of roof storey [17]. If this ratio is above 1.2, then the pushover analysis approach needs to be changed through the assignment of the loads at the center of mass, and the displacement at the center of mass should be recorded. However, in order to prove the existence of a relationship between the lateral performance of the structures and the structural parameters, the structures should not be prone to the influences of rotational displacements, which is very difficult to foresee with the proposed parameters. Therefore, as the first step of elimination, the structures with high relative sway ratio are excluded.

Another suggestion prescribed in the reports pertaining to the use of IDCM is related with the fundamental periods of the structures [17,18]. Both reports tried to set a higher and lower limit value on the fundamental period to prescribe the applicability limits to IDCM. For example, according to the earlier report [17], for fundamental periods greater than 1.0 s, the higher modal effects should be included to estimate the roof displacements. Indeed, the structures with fundamental periods higher than 1.0 s are considered within the boundaries where equal displacement rule applies. The latter report [18] has the same limit value of fundamental period as 1.0 s for the application of the inelastic to elastic displacement ratio, which is more than unity in IDCM. Consequently, the coefficient of inelastic to elastic displacement ratio is set as 1.0 for the structures with fundamental periods higher than 1.0 s.

Similarly, structures with lower fundamental periods are also considered as a different group that requires attention. Both reports accept the dispersion of the estimated displacement from the equal displacement approximation at shorter periods. The earlier report [17] defines the short period range as 0-0.50 s, whereas in the latter report [18], no definite limits

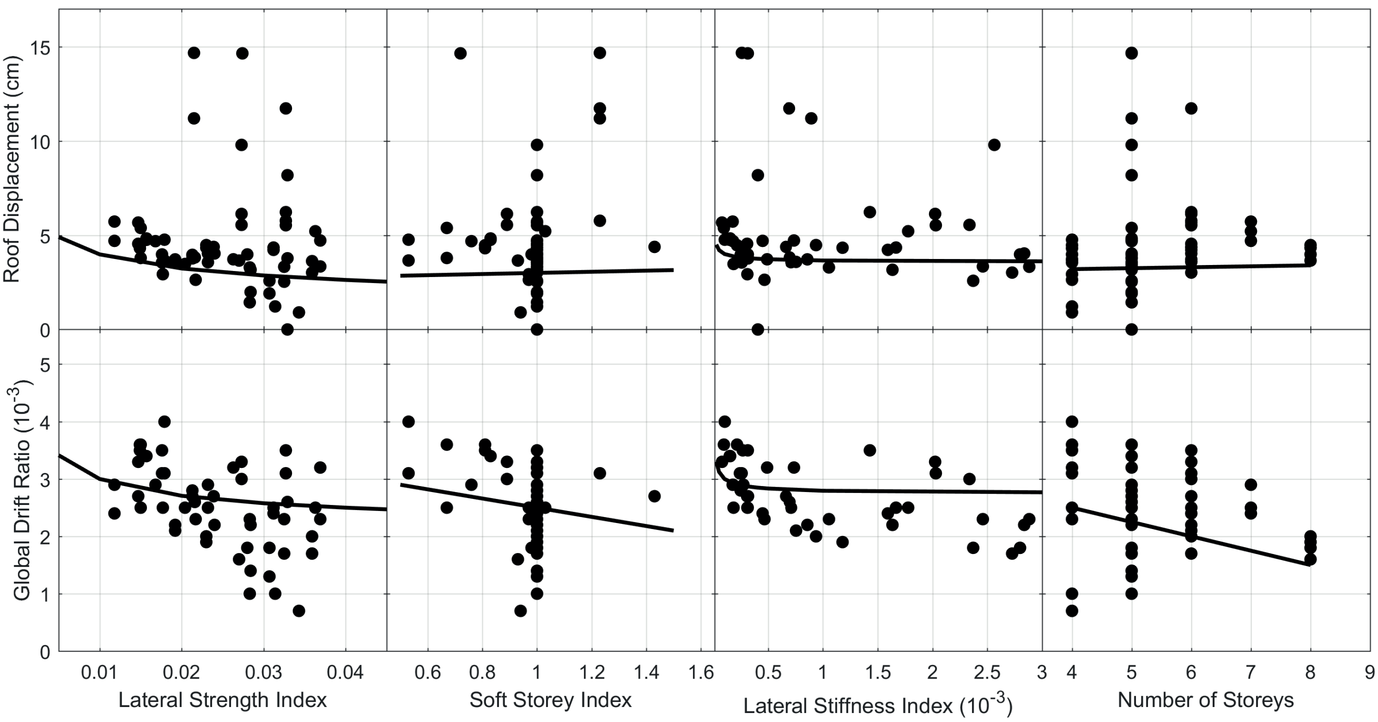


Fig. 5 Variation of Lateral Performance Values with Respect to the Functions Obtained by Sensitivity Analysis (NEHRP Site Class C, Lateral Performances of 62 Structural Models for a 475-year Return Period Earthqauke are Presented

are proposed. In fact, in the report, high sensitivity of the target displacement to the low fundamental periods is mentioned, and for the structures with lower than 0.20 s, the use of modification factor is suggested. After all, the structures with low fundamental periods are evaluated considering the suggestions of both sources [17,18]. Indeed, considering the fact that these limits are developed by using a large database of structural models and that they are mostly valid for all types and ranges of structural models, the evaluation of the limit values for a specific sample could generate reliable results for that specific sample. In addition, lower limits are not clearly defined in the related documents, which triggered a sample investigation to reveal these limiting fundamental period values.

For such an investigation, to define the upper and lower limit fundamental periods, a number of structural models are required with a very wide range of coverage, and the lateral performances of these models must be estimated. In such an attempt, all the 62 original structural models are used to generate the lateral performance versus fundamental period plot in Figure 6. Obviously, the models have a wide range of fundamental periods and displayed varying lateral performances that are computed by using the IDCM. The response of the roof displacements with respect to the changing fundamental periods provide valuable information in setting the upper and lower limits of fundamental periods, in which a reliable pattern emerges between these limits.

Knowing that the outliers in terms of fundamental periods could be one of the prime reasons for the dispersion in Figure 5, and considering the emerging patterns of lateral performances in Figure 6, a lower limit of 0.35 s and an upper limit of 0.80 s are imposed for the elimination of outliers. Only between these two fundamental periods, a pattern emerges for each of the response spectrum curves developed for Eskisehir. The regions with different patterns of behaviour are associated with different regions of the response spectrum curves in Figure 2 where the curves display different patterns.

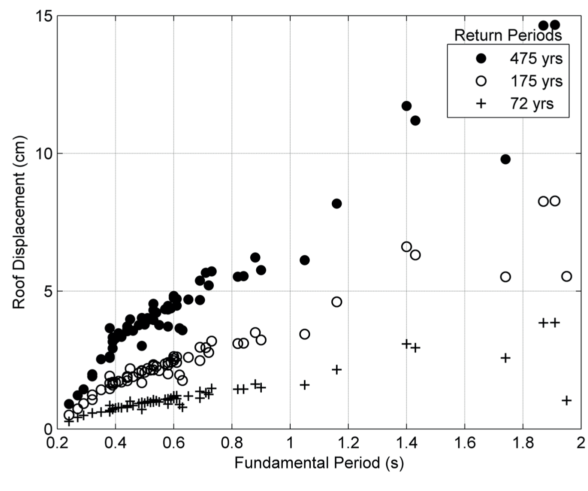


Fig. 6 Variation of Roof Displacement with Fundamental Period (62 Structural Models)

The main disadvantage of limiting the fundamental periods between 0.35 s and 0.80 s is that the estimated displacements form a narrow band for each response spectrum curve. The estimated roof displacements, and hence the global drift ratios, would limit the capability of a LPPE that is developed by using these lateral performances. However, these limitations could be imposed as a prescription in the application criteria of the LPPE. Moreover, it should be mentioned that the shape of the elastic response spectrum curves is a strong determining factor in the establishment of these limits as well. Therefore, the seismic hazard and seismic risk studies must be conducted as part of the LPPE development procedure in order to set the prescriptions of a developed LPPE for a specific region.

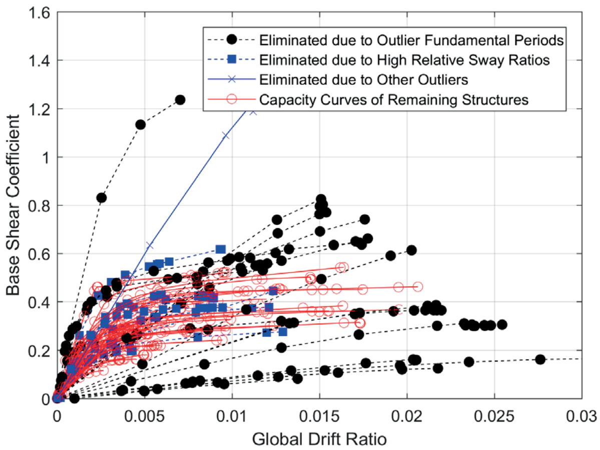


Fig. 7 Capacity Curves of Structures

Considering the technical background of the reasons for the dispersion in Figure 5, a two-stage elimination was proposed for technical and visual elimination schemes. Technical elimination required the assessment of structures with respect to fundamental periods and relative sway ratios. Then, visual elimination is performed by removing the outstanding pushover curves with very high values of yield or ultimate global drift ratios, yield or ultimate base shear coefficients or negative post elastic stiffness (Figure 7). Indeed, the sole purpose of visually eliminating the outstanding pushover curves is to create more uniformly distributed pushover curves with certain characteristics such as yield drift ratios, plastic to elastic stiffness ratios, and ultimate values. At the end of the elimination process, the analysis proceeded with the remaining structural models. In summary, out of the 62 models, 26 structural models are eliminated with the technical elimination scheme, and only one structure is eliminated by the visual elimination scheme; and thus, 35 structural models remained for further analysis.

7 Analysis

The refinement procedure is followed by the establishment of the cross validation scheme that is formulated for the verification of the relationships identified by the sensitivity analysis. The MCCV scheme that involves the selection of 28 samples for the development of LPPE and 7 samples for the validation in each round is given in Figure 8. These two stages that are utilized in the development of LPPE are referred to as development and validation stages in the following paragraphs.

The selection of structures for training and validation samples is performed randomly with a MATLAB program developed specifically for this study. Therefore, the generation of hundreds of possible scenarios, indeed 500 times in this study, becomes possible with the formulated cross validation scheme.

The coefficients of the structural parameters are determined through the development stage of the cross validation rounds, while the power coefficients are determined through a trial procedure due to the limited amount of data. In fact, since the nonlinear scaling characteristics of lateral strength and stiffness indexes are easily identifiable and can easily be modelled, probable power coefficients are selected from a narrow range.

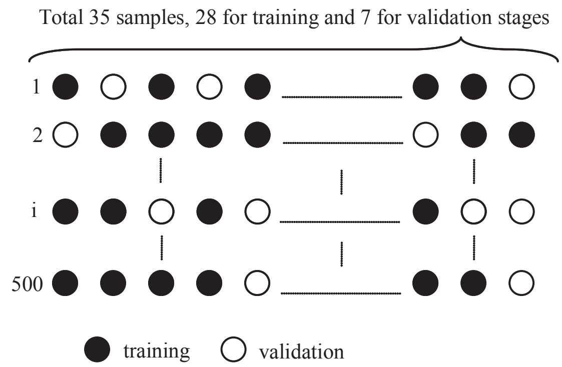


Fig. 8 MCCV Random Selection Scheme, Ramdom Selection of Training and Validation Samples is Repeated 500 Times

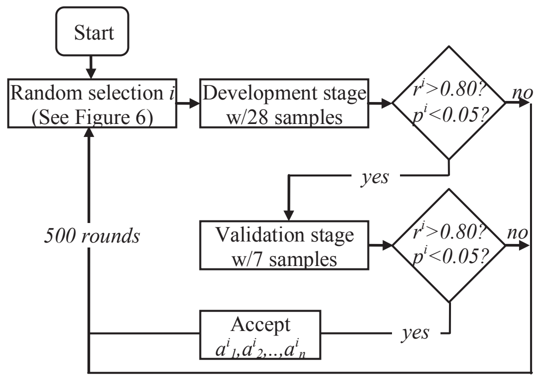
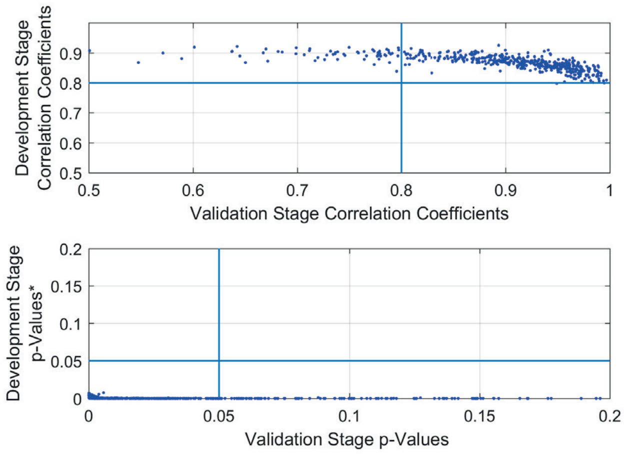


Fig. 9 MCCV Scheme for the Development and Validation Stages and the Determination of the Valid Coefficients for LPPE

Figure 9 presents the schematics of the MCCV scheme formulated to derive of the coefficients of the LPPE. As summarized in the figure, the final coefficients of equations are obtained from several incidences of statistically acceptable cross validation rounds. Both development and validation stage analysis requires correlation coefficient (r) above 0.80 and 95% confidence interval (p < 0.05). Then, the final coefficients of the LPPE are computed by averaging the coefficients obtained from the successful cross-validation rounds. The generated statistical parameters of these cross validation rounds are presented in Figure 10 for 475-year return period earthquake. The correlation coefficients (r) for development and validation stages are provided in the upper subplot, while the significance test results (p) of these rounds are given in the bottom subplot.

One of the prerequisites for the acceptance of the proposed method is to conduct significance tests for each parameter. Figure 11 is developed for the verification of whether each coefficient of the LPPE is statistically meaningful.



Due to very low t-test results, p values are scaled to display t-test results of validation stage

Fig. 10 Correlation Coefficients Computed for the Development and Validation Stages and Overall Significance Test Results in terms of *p* Values (500 Validation Rounds, 475 years Return Period)

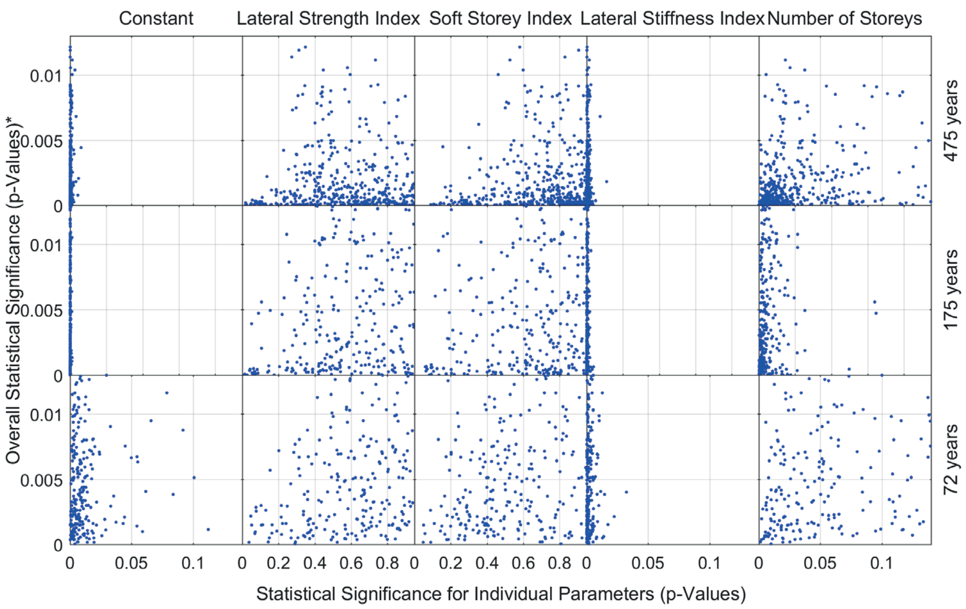
Technically, the p-values of the test results should remain below 0.05, which means that the reliability of the derived coefficients for the parameters is above 95%; in other words, the derived coefficients are statistically meaningful. After examining Figure 11, it is observed that the test results for the p values are clustered between 0 and 0.05 for the constant coefficient of the equation, lateral strength index, and the number of storeys for each considered earthquake. The p values of lateral strength index and soft storey index vary between 0 and 1.0 for the considered earthquake.

Knowing the physical background of the claims and the emerging patterns after the sensitivity analysis and together with the performance of the structures with the relationship characteristics obtained through the sensitivity analysis, the significance test results could be questioned. Since multi-collinearity is an issue in the regression analysis, and the significance test results are very prone to the mentioned issue, the significance test results of each individual parameter might be attributed to this issue. Moreover, knowing that the results of the significance tests could be improved by increasing the sample size, the same significance test results of each individual parameter could also be attributed to the small sample size. Consequently, it is decided to adopt the developed LPPE for further use.

As the proposed method proved to be promising, the selection of the coefficients from amongst the several set of coefficients generated by the cross validation rounds remains an issue. Generally, when a MCCV scheme is applied, the coefficients of the validation round with the highest correlation coefficient are selected.

However, it is a fact that the most statistically meaningful validation rounds with highest correlation coefficients might not be associated with the most successful development stage correlation coefficients. Therefore, for the selection of the coefficients of the LPPE, firstly, the distribution characteristics of the coefficients of the successful rounds must be investigated. Since, the coefficients are derived for each response spectrum curve that was developed using earthquakes with different return periods, for each structural parameter, five sets of coefficients are derived for each response spectrum curve.

After the evaluation, it was realized that the characteristics of the distribution of all the coefficients are similar to the distribution in Figure 12, the mean of the coefficients is favoured to the coefficients obtained by only a single round. Hence, with the introduction of distribution characteristics of the computed coefficients, the variability of the coefficients in terms of standard deviation could be incorporated into the LPPE as well. Consequently, the mean of the coefficients of statistically acceptable cross validation rounds is selected to



** Due to very low t-test results, p values are scaled to display t-test results of individual parameters*

Fig. 11 Significance Test Results of Individual Structural Parameters form the LPPE. The resultant mean and the standard deviations of the coefficients for each response spectrum curve are compiled in Table 1. Out of 500 validation rounds performed for the cross validation, 291 statistically validated rounds are used to derive the mean and the standard deviation of the coefficients of the LPPE.

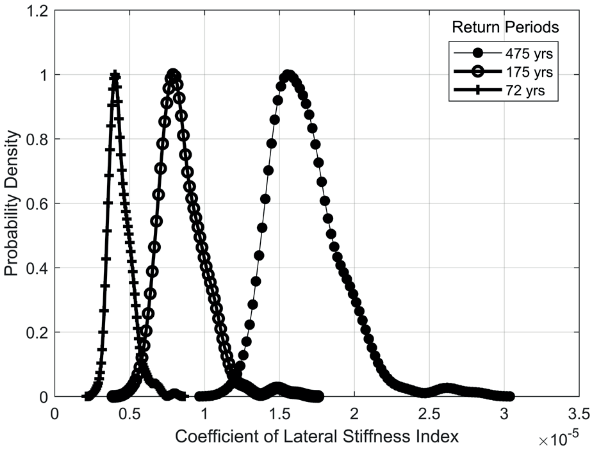


Fig. 12 Distribution of Coefficients for Lateral Stiffness Index (Distribution of Values Obtained by 291 Validated Rounds of Cross Validation Scheme out of 500)

Table 1 Mean and Standard Deviations of Coefficients of Lateral Performance Prediction Equation for Each Response Spectrum Curve (291 Validation Rounds)*

Coefficients		Return Period** 475	175	72
a_1		301.17	160.56	58.19
σ_{a1}		30.58	19.74	8.35
a_2		–4.21	–1.70	–0.37
σ_{a2}		4.37	2.84	1.34
a_3		–5.32	–4.20	6.35
σ_{a3}	$(\times 10^{-5})$	23.16	15.82	6.83
a_4		1.66	0.84	0.43
σ_{a4}		0.23	0.15	0.07
a_5		–15.07	–7.99	–3.46
σ_{a5}		2.86	1.54	0.86
σ_{all}		0.01	0.00	0.00

* *a_6 and a_7 are obtained as –0.5 by trial and error method*

** *Return periods of the considered earthquake in the derivation of coefficients*

The type of the relationship of the global drift ratios with each individual parameter is crucial for the verification of the reliability of coefficients in reflecting the real physical behaviour. A close examination of the coefficients in Table 1 reveals that the lateral performance in terms of global drift ratio correlates positively with the parameter of the lateral stiffness index, while it does so negatively with the parameters of lateral strength index, the number of storeys, and the soft storey index.

According to the sensitivity analysis, the global drift ratios positively scale with the parameters of the number of storeys and soft storey index, whereas negatively scale with the lateral stiffness and strength index as expected. However, according to the resultant coefficients in Table 1, the global drift ratios positively scale with lateral strength index, which is not consistent with the sensitivity analysis results. This inconsistency between the scaling of global drift ratios must be explained in order to prove the validity of the proposed LPPE. In order to investigate this inconsistency, the values of lateral strength index, lateral stiffness index, soft storey index and the number of storeys are classified into subintervals, and the lateral performances estimated by the IDCM for an earthquake with 475-year return period are plotted with respect to the mentioned structural parameters. Since the distribution of the lateral strength index and the estimated displacements have a very significant effect on the outcome of the regression analysis, the distribution of the lateral performances with respect to the lateral strength indexes of the structures in the sample is examined. As a result of the examination of the distribution characteristics of the estimated global drift ratios in Figure 13, it is observed that the global drift ratios decrease with the increasing lateral strength index with strong tendency. However, it should be known that randomly selected values might not yield the expected scaling characteristics. Adding on top of that, the existence of a parameter that models the inverse nonlinear scaling of the lateral performance values complicates the regression analysis more.

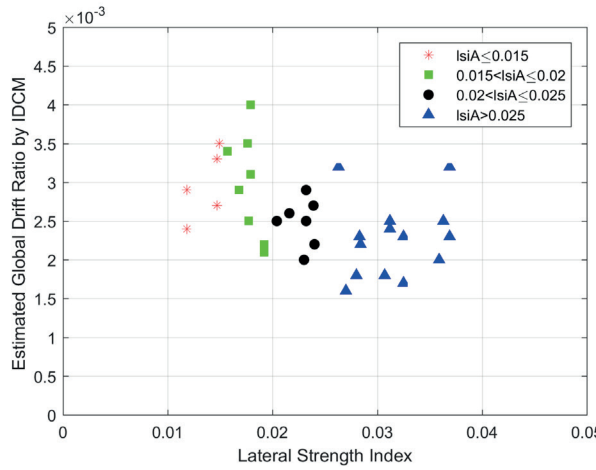


Fig. 13 Distribution of the Estimated Global Drift Ratios by IDCM with Respect to Lateral Strength Index

The aforementioned issues in the regression analysis challenged the development of the LPPE and have a potential to create a bias in the coefficient estimations. These issues also strongly influence the derived coefficients for lateral strength index. As shown in Figure 14, the distribution of the derived coefficients for lateral strength index displays varying mean and standard deviations. Both negative and positive coefficients are obtained as a reflection of the aforementioned issues.

Similarly, the relationship between the soft storey index and the global drift ratios is evaluated. The distribution of soft storey indexes and the lateral performance might follow the expected pattern as shown in Figure 15. It can be clearly concluded from the figure that as the soft storey index increases, the estimated drifts by IDCM decrease. Hence, the LPPE successfully reflects the type of scaling of lateral performances with the soft storey index.

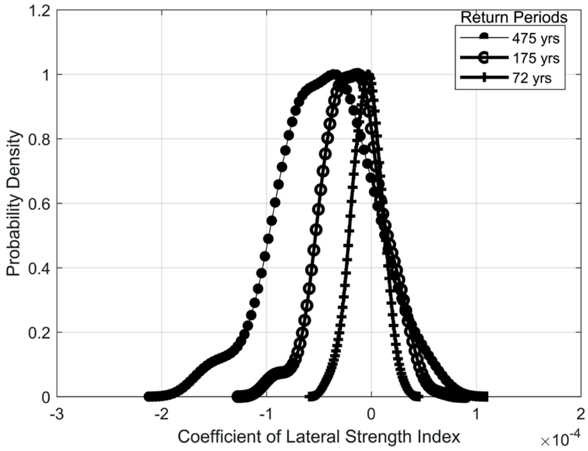


Fig. 14 Distribution of Coefficients of Lateral Strength Index (291 Validated Rounds of Cross Validation Scheme out of 500)

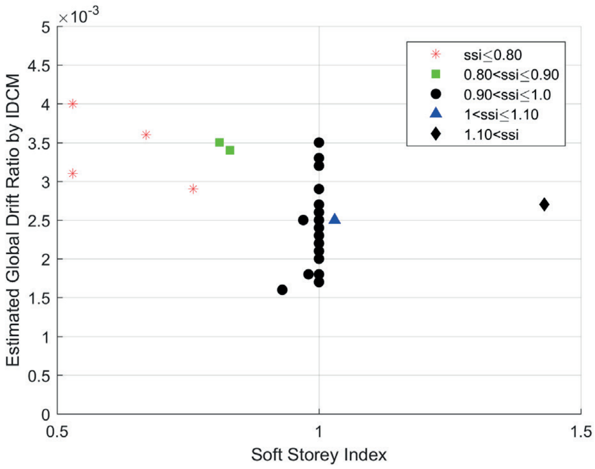


Fig. 15 Distribution of the Estimated Global Drift Ratios by IDCM with Respect to Soft Storey Index

The relationship between the lateral stiffness index and the nonlinear scaling of the global drift ratios captures attention in Figure 16. Obviously, a pattern emerges between the lateral stiffness index and the estimated global drift ratios, which justifies the obtained scaling characteristics in the sensitivity analysis. It is strongly expected that with the increasing number of storeys, the target or roof displacements automatically increase since the height increases in proportion to the number of storeys. Even with a few cases where this assumption is not true, the distribution of the number of storeys with the estimated and predicted target displacements follows this assumption. However, as presented in Figure 17, the global drift consistently decreases as the number of storeys increases, which indicates that the slope of roof displacement versus the number of storeys is less than unity as expected.

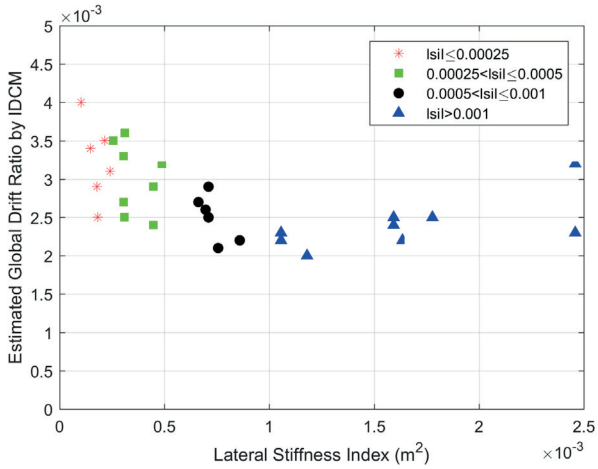


Fig. 16 Distribution of the Estimated Global Drift Ratios by IDCM with Respect to Lateral Stiffness Index

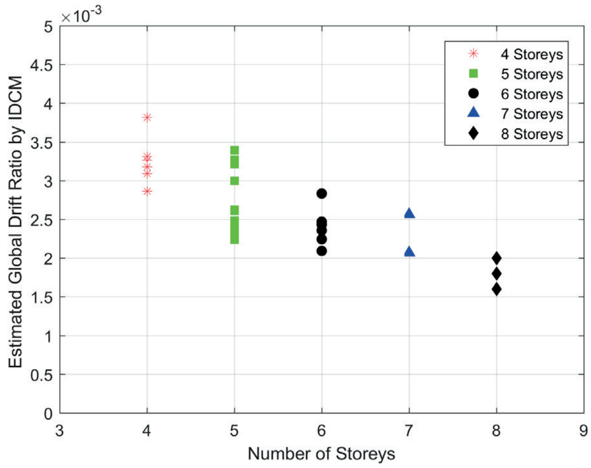


Fig. 17 Distribution of the Estimated Global Drift Ratios with Respect to Number of Storeys

Following the evaluation of the distribution of lateral performances with respect to the structural parameters, it is concluded that the distribution characteristics of the estimated global drift ratios with respect to the structural parameters are the determining factors in the development of the LPPE.

Except for the lateral strength index, the relationship of the structural parameters with the lateral performance is very well reflected in the LPPE. The issue of scaling of global drift ratio with respect to the lateral strength index, which also could originate from multi-collinearity, must be emphasized in addition to the low significance test results of lateral strength index and soft storey index. However, considering the physical background that explains the scaling of the global drift ratio with respect to the lateral strength index, the results of the sensitivity analysis, and the distribution of the performances of original structures with respect to the lateral strength index, which strongly indicates the relationship; the developed LPPE could be a valid one. It should be noted that the complex relationship between the lateral strength and stiffness indexes and the lateral performance could yield different results if the same procedures in this study are applied with a different sample, depending on the structural parameters dominating the lateral response.

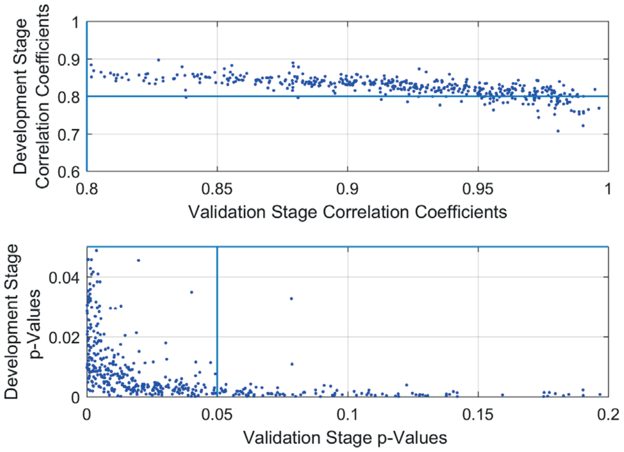
8 Adjustment of the LPPE

As the existence of multicollinearity causes a loss of meaning in the significance tests, the sign of the coefficients of the related parameters would usually be opposite. This observation is proven true in the case study performed to develop a LPPE. As a solution to the multicollinearity problem, lateral strength and soft storey indexes are removed and the proposed LPPE is adjusted. Consequently, after the removal of the lateral strength and soft storey indexes as a solution to solve the multicollinearity issue, Equation 2 could be rewritten to model this relationship.

$$LP(lsiI, n) = f(lsiI) + f(n) + \sigma \quad (4)$$

where LP represents lateral performance for the considered response spectrum. In the open form, Equation 4 could be rewritten as

$$\delta_{GDR}(lsiI, n) = a_1 + a_4 lsiI^{a7} + a_5 n + \sigma \quad (5)$$



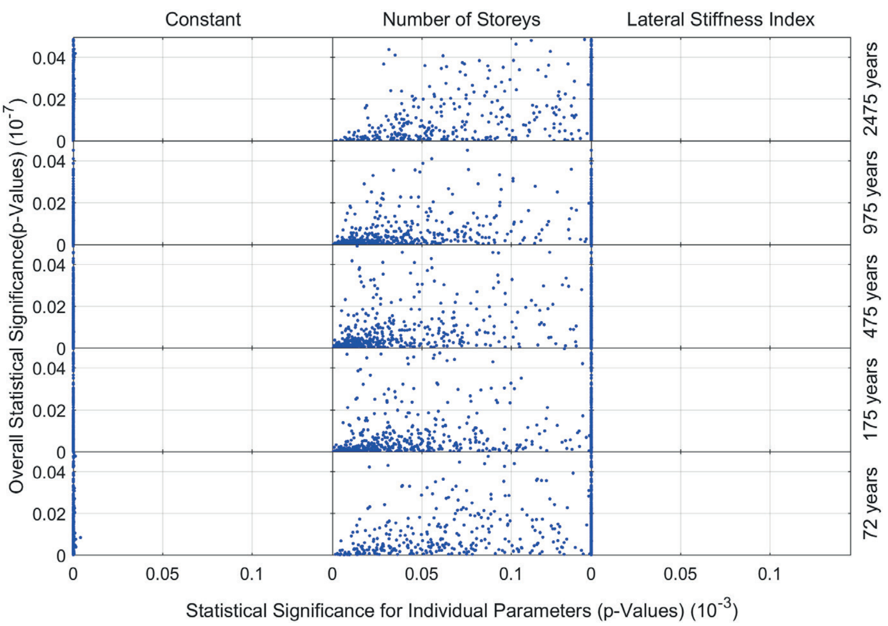
** Due to very low t-test results, p values are scaled to display t-test results of validation stage*

Fig. 18 Correlation Coefficients Computed for the Development and Validation Stages and Overall Significance Test Results

The high correlation coefficients associated with the very low significance test results in Figure 18 indicate the existence of a very strong relationship between the lateral performances and the structural parameters in Equation 5. It also proves the success of the adjusted LPPE in modelling the sample data. The significance test result for individual structural parameters in Figure 19 also support the success of this adjustment. It can be clearly claimed that both the parameters of lateral stiffness index and the number of storeys are highly correlated with the estimated global drift ratios. However, this does not mean that the lateral performance of the structures can be predicted only by using these two parameters. As stated earlier, if the sample size is increased, a definite improvement in the significance test results could be observed. Moreover, with improvements in the formation of the LPPE, which might contain both lateral stiffness and lateral strength indexes without creating a multicollinearity issue, the LPPE might perform better in modelling the lateral performance of the structures. The coefficients of the LPPE are derived with the application of the same procedures as the mean and standard deviation of the statistically successful cross validation rounds are determined.

The coefficients of the adjusted LPPE in Table 2 display a consistent behaviour with the results of the sensitivity analysis and the scaling of lateral performances of original structures in the sample.

The performance of the derived LPPE is best observed in trellis plots. Both the lateral stiffness index and the number of storeys are assigned with a range of values. After that, the global drift ratios are derived by using the developed LPPE for the assigned range of values. The scaling of the lateral performance with the lateral stiffness index and the number of storeys is clearly presented Figure 20.



** Due to very low t-test results, p values are scaled to display t-test results of individual parameters*

Fig. 19 Significance Test Results of Individual Structural Parameters

Table 2 Mean and Standard Deviations of Coefficients of LPPE for Each Response Spectrum Curve *

Coefficients	Return Period*		
	475	175	72
a_1	281.35	150.61	62.63
σ_{a1}	15.97	8.81	4.69
a_4	–16.45	–8.58	–3.38
σ_{a4} $(\times 10^{-5})$	2.20	1.18	0.66
a_5	1.53	0.80	0.41
σ_{a5}	0.11	0.07	0.03
a_{7*}	–0.5	–0.5	–0.5
σ_{all}	0.01	0.00	0.00

** Return periods of the considered earthquake in the derivation of the coefficients*

*** Obtained by trial and error*

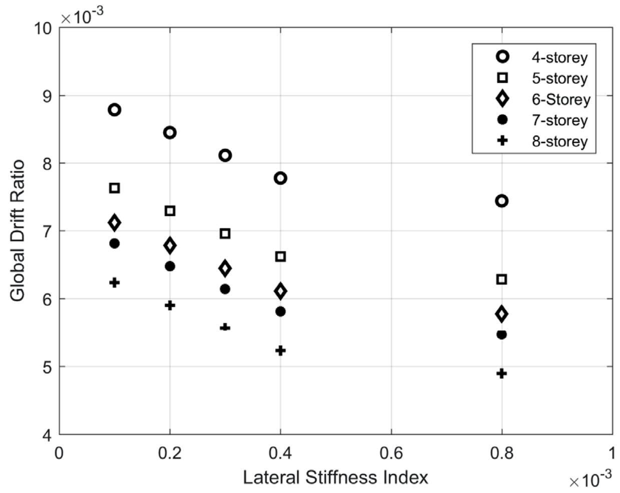


Fig. 20 Trellis Plot of Global Drift Ratios Obtained by IDCM for 2475-year Return Period Earthquake

9 Evaluation of the results

37 structures in both transverse directions are subjected to a number of compliance criteria and sensitivity analysis procedures for the development and validation of LPPE. The sensitivity analysis, which indeed provided an insight to the basic behavioural patterns of structures, guided in the formation of the LPPE and helped to sort out the outlier behaviour of the structures. Only after the determination of the pattern of behaviour, the structures in the sample were subjected to the elimination scheme.

In summary, out of 74 structural models, 39 structural models were eliminated due to the criteria of modal mass participation ratio, fundamental period, relative sway ratio, and visual elimination. It should be mentioned that only a single structural model was eliminated due to the visual elimination criteria, while the rest of the models are eliminated due to the imposed technical criteria.

Several criteria of elimination are imposed to have a predictable pattern of lateral performances and to serve the purpose of the study. It is not surprising that the pattern of lateral performances obtained by the IDCM is similar to the fundamental period patterns. The similarity is justified through the investigation of the basic relationships of fundamental period, demanded spectral displacement, height, mass, and lateral stiffness of the structures. The influence of the basic structural properties on the lateral performances is clearly identified in the application of the elimination procedures. It should also be mentioned that fundamental period has a very strong influence over the type of behaviour of the structures, as the capacity curves with the most distinct patterns of behaviour in Figure 7 are associated with very high or low fundamental periods. The high relative sway ratio is another factor which cannot be foreseen by using the proposed structural parameters; hence, structures with high relative sway ratios are also removed.

In the end, it is shown that there is a requirement of tuning the lateral performances into a predictable pattern, which initiated the imposition of several criteria. In order to obtain a predictable pattern of lateral performances, the elastic response spectrum curves as well as the pushover analysis of the models were investigated. If the elastic response spectrum curves allow a smoother pattern, then there is a better chance of obtaining more uniformly distributed lateral performances with demand spectral displacements which are within a predictable pattern.

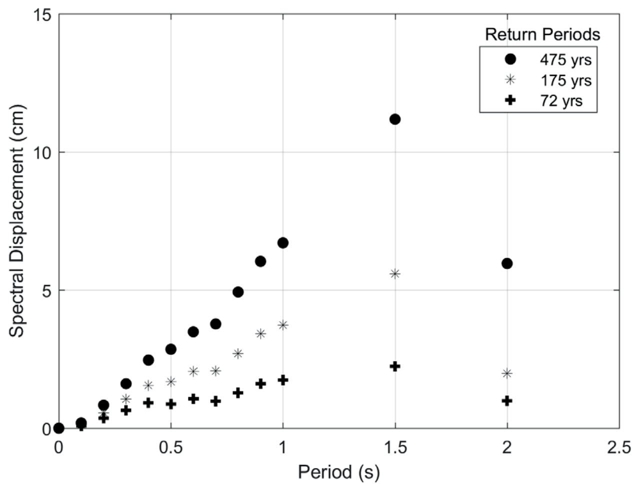


Fig. 21 Displacement Response Spectrum Curves Developed for Eskisehir

Unfortunately, the displacement response spectrum curves that were developed for Eskisehir do not allow such a uniform behaviour for the related period ranges. Indeed, limits between 0.35 s and 0.80 s seem to be hardly justified considering the variance in the displacement response spectrum curves in Figure 21.

Moreover, the prerequisites in the related reports [17,18] helped to create a group of structures with a predictable pattern of behaviour. The pattern of behaviour could be perfectly followed in Figure 22, which presents the scaling of the roof displacements through the fundamental periods. The strong dependence of the estimated roof displacements on the fundamental period of the structures in the refined database implies that there is a strong connection between the proposed structural parameters and the lateral performance considering the height, mass and lateral stiffness dependence of the fundamental periods.

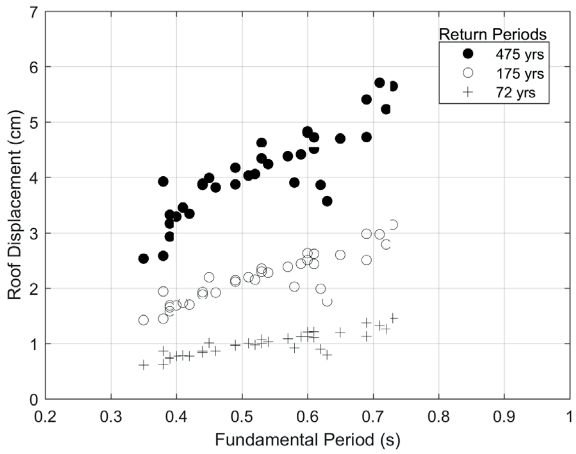


Fig. 22 Narrow Band of Roof Displacements Obtained by IDCM and the Fundamental Period Relationship between 0.35 s and 0.80 s (35 Structural Models)

Unfortunately, imposing a limitation on the fundamental periods limits the capability of the LPPE. However, it should be known that if the performance of the LPPE is successful in the prediction of the lateral performances for a certain range of fundamental period, then it is anticipated that the capability of the LPPE could be extended to cover the whole range of periods of the response spectrum curve.

10 Final recommendations

After several stages of obtaining a valid LPPE, two equations are proposed. If the statistical concepts are taken into consideration, only equation 5 should be employed with the coefficients provided in Table 2. However, if structural concepts are also taken into consideration, equation 3 should be employed with the coefficients given in Table 1. The following equations provide the expanded forms of equation 5, which include lateral stiffness index and the number of storeys as parameters.

$$\delta_{GDR}(475) = \left(281.35 + 1.53\ lsiI^{-0,5} - 16.45\ n\ \pm 0.01\right) \times 10^{-5} \quad (6)$$

$$\delta_{GDR}(175) = \left(150.61 + 0.80\ lsiI^{-0,5} - 8.58\ n \pm 0.00\right) \times 10^{-5} \quad (7)$$

$$\delta_{GDR}(72) = \left(62.63 + 0.41\ lsiI^{-0,5} - 3.38n \pm 0.00\right) \times 10^{-5} \quad (8)$$

However, if Equation 3 is preferred, which is the case in this study, the following forms of equations with the coefficients should be employed:

$$\delta_{GDR}(475) = \begin{pmatrix} 301 - 4.21\ lsiA^{-0,5} - 5.32\ ssi + \\ 1.66\ lsiI^{-0,5} - 15.07n \pm 0,01 \end{pmatrix} \times 10^{-5} \quad (9)$$

$$\delta_{GDR}(175) = \begin{pmatrix} 160.6 - 1.70\ lsiA^{-0,5} - 4.20\ ssi + \\ 0.84\ lsiI^{-0,5} - 7.99\ n \pm 0.00 \end{pmatrix} \times 10^{-5} \quad (10)$$

$$\delta_{GDR}(72) = \begin{pmatrix} 58.19 - 0.37\ lsiA^{-0,5} + 6.35\ ssi + \\ 0.43\ lsiI^{-0,5} - 3.46\ n \pm 0.00 \end{pmatrix} \times 10^{-5} \quad (11)$$

The equations are developed considering the different return periods. Thus, the discrete lateral performances in terms of global drift ratios can be interpolated to obtain the lateral performances for the earthquakes with return periods other than the ones employed in this study.

References

[1] Pay, A. C. "*A New Methodology for the Seismic Vulnerability Assessment of Existing Buildings in Turkey*". M.Sc. Thesis, METU, Ankara, Turkey. 2001

[2] Pitilakis, K., Weatherill, G., Crowley, H., Pinho, R. "*Systematic Seismic Vulnerability and Risk Analysis for Buildings, Lifeline Networks and Infrastructures Safety Gain*." Deliverable D2. 12. 2011

[3] Pinho, R., Marques, M., Monteiro. R., Casarotti, C., Delgado, R. "Evaluation of Nonlinear Static Procedures in the Assessment of Building Frames". *Earthquake Spectra*, 29(4), pp. 1459–1476, 2013. https://doi.org/10.1193/100910EQS169M

[4] Fragiadakis, M.,M., Vamvatsikos, D., Aschheim, M. "Application of Nonlinear Static Procedures for the Seismic Assessment of Regular RC Moment Frame Buildings". *Earthquake Spectra*, 30(2), pp. 767–794. 2014. https://doi.org/10.1193/111511EQS281M

[5] Brun, J., Kostem, C. "*Lateral Deflection Prediction of Concrete Frame-Shear Wall System*". Technical Report No. 433-6 Fritz Engineering Laboratory, Lehigh Univ., USA. 1986. http://digital.lib.lehigh.edu/fritz/pdf/433_6.pdf

[6] Wu, X., Ghaboussi, J., Garrett, Jr J. H. "Use of neural networks in detection of structural damage". *Computers and Structures*, 42(4), pp. 649–659, 1992. https://doi.org/10.1016/0045-7949(92)90132-J

[7] Arslan, H. "Application of ANN to Evaluate Effective Parameters Affecting Failure Load and Displacement of RC Buildings". *Natural Hazards and Earth System Science*, 9, pp. 967–977, 2009. https://doi.org/10.5194/nhess-9-967-2009

[8] Kameli, I, Miri, M., Raji, A. "Prediction of Target Displacement of Reinforced Concrete Frames Using Artificial Neural Networks". *Advanced Materials Research*, 255–260, pp. 2345–2349, 2011. https://doi.org/10.4028/www.scientific.net/AMR.255-260.2345

[9] Hassan, A. F., Sozen, M. A. "Seismic Vulnerability Assessment of Low Rise Buildings in Regions with Infrequent Earthquakes". *ACI Structural Journal*, (94)1, pp. 31–39, 1997.

[10] Ozcebe, G., Yucemen, M. S., Aydogan, V., Yakut, A. "Preliminary Seismic Vulnerability Assessment of Existing Reinforced Concrete Buildings in Turkey. Part I: Statistical Model Based on Structural Characteristics". In: *Seismic Assessment and Rehabilitation of Existing Buildings*". (Wasti, S. T., Ozcebe, G. (Eds)). pp. 29–42, NATO Science Series (Series IV: Earth and Environmental Sciences), vol. 29. Springer, Dordrecht. 2003.

[11] Yucemen, M. S., Ozcebe, G., Pay, A. C. "Prediction of Potential Damage due to Severe Earthquakes". *Structural Safety*, 26(3), pp. 349–366, 2004. https://doi.org/10.1016/j.strusafe.2003.09.002

[12] Yakut, A., Ozcebe, G., Yucemen, M. S. "Seismic Vulnerability Assessment Using Regional Empirical Data". *Earthquake Engineering and Structural Dynamics*, 35, pp. 1187–1202. 2006. https://doi.org/10.1016/j.strusafe.2003.09.002

[13] Karaca, H. "*Estimation of Potential Earthquake Losses for the Evaluation of Earthquake Insurance Risks*". Ph.D. Thesis, METU Civil Engineering Dept., Ankara, Turkey, 2014. http://etd.lib.metu.edu.tr/upload/12618203/index.pdf

[14] Burak, B., Comlekoglu, H. G. "Effect of Shear Wall Area to Floor Area Ratio on the Seismic Behavior of Reinforced Concrete Buildings". *Journal of Structural Enginering ASCE*, 139(11), 1928–1937, 2012. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000785

[15] Gunel, A. O. "*Influence of the Shear Wall Area to Floor Area Ratio on the Seismic Performance of Existing Reinforced Concrete Buildings*". M.Sc. Thesis, METU, Ankara, Turkey, 2013.

[16] *Computers and Structures*, Inc. Sap2000 Version 15, Structural Analysis Program, Berkeley, California, 2013.

[17] ATC-40. "*Seismic Evaluation and Retrofit of Concrete Buildings*". Applied Technology Council, California, USA, 1996. https://www.atcouncil.org/pdfs/atc40toc.pdf

[18] FEMA-440. "*Improvement of Nonlinear Static Seismic Analysis Procedures*", Federal Emergency Management Agency Report, Washington D.C., USA, 2005. https://www.fema.gov/media-library-data/20130726-1445-20490-9603/fema-440.pdf