

SOIL-STRUCTURE-INTERACTION EFFECT ON SEISMIC RESPONSE OF BRIDGES

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ABSTRACT

Efforts have been made in the past to investigate and quantify the behaviour of bridge piers on pile foundation when subjected to lateral loading such as that occurring during earthquakes. These studies were motivated from field observations where it was found that damage in the bridge superstructure could be mitigated in strong ground motion events when the structure is supported on compliant, shallow pile foundations. The effectiveness of this mechanism of behavior is owing to the reduction of rotation demand in the piers due to the foundation compliance: rotation of the pile-head accounts for part of the bridge lateral deformation response, through a “spring-in-series” arrangement with the pier (i.e. $K_{pier} = K_{found} + K_{column}$). Some benefit to design can be obtained from this behavior if a dependable estimation of the equivalent foundation stiffness can be established as a function of rotation demand. By modeling the foundation as a rigid pile-head supported by a group of deformable piles, it is the objective of this thesis is to study its characteristic resistance curve; in this regard several pushover analyses of a single pile under kinematic constraint at the point of connection with the overbearing pile-head were conducted, in order to observe the intensity and influence of the soil-pile interaction on the superstructure deformation demands when excited by ground motion events of increasing intensity. A sensitivity investigation is conducted to support the development of quantitative models for the coordinates of the milestone points of the resistance curve in terms of the important design variables (soil profile, pile aspect ratio, etc.) which are then scaled up to integrate into the resistance curve of a complete pile-group. In this study an example bridge overpass for an Ontario Highway is considered in relevance to the Canadian practice; the case study comprises a prestressed Reinforced concrete two span superstructure. It is examined under a suite of strong ground motions extracted from the PEER and the Earthquakes-Canada Databases in order to illustrate the effectiveness of this design concept in mitigating seismic demands from the vulnerable bridge piers. Several conditions are combined in terms of soil compliance, depth and boundary conditions of the piles, support conditions at bridge abutments, and connectivity between the cap beam of the intermediate pier with the superstructure. It is shown that a significant share of pier drift demands is mitigated by a compliant foundation, however optimal performance is obtained when combined with non-compliant supports at the abutments, in order to also moderate the total flexibility of the system and the magnitude of the sway periods, that could cause significant damage in the bridge due to large displacement demands.

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LIST OF SYMBOLS AND ACRONYMS

A_c	Cyclic load factor
A_s	Static load factor
BH	Borehole
c_r	Damping of substructure for rocking motion
c_s	Damping of substructure for sway motion
c_x	Horizontal damping of substructure
c_y	Vertical damping of substructure
c'	Soil shear strength
d_e	Equivalent diameter of a steel H-Beam
Δ_p	Plastic displacement of pier top
Δ_e	Elastic displacement of pier top
Δx	Depth increment along pile
Δy	Increment of pile lateral displacement
Δy_1	Head lateral displacement at first yield of pile
Δy_2	Head lateral displacement at second yield of pile
Δu	Ultimate displacement at pilecap
φ'	Soil angle of friction
$\ddot{\varphi}_1$	Substructure angular acceleration in response to free-field motion
Φ	Fundamental mode shape of translational vibration of the pier-foundation
Φ_{CN}	Total displacement of bridge top point due to SSI action
Φ_i	Curvature
Φ_{ul}	Curvature upon formation of first plastic hinge
E	Modulus of Elasticity of a single pile
E_c	Modulus of Elasticity of a column

E_s	Elastic modulus of soil
EI_e	Pile flexural rigidity
E_cJ_{eff}	Effective stiffness of pier column
ε	Soil strain
f_y	Yield strength of structural steel
G	Soil shear modulus
γ	Soil density
χ	Angle of the piles with the horizontal plane in a pile group
H	Height of superstructure for SDOF/MDOF model
H_{pier}	Height of bridge pier
H_B	Depth to inflection point of pile
h	Height of pile
h_θ	Lateral displacement of MDOF superstructure at top point due to rotation of pilecap
I	Moment of inertia of a single pile
I_0	Moment of inertia of a foundation for SDOF model
K_a	Coefficient of active earth pressure
K_{column}	Lateral stiffness of column
K_{pier}	Lateral stiffness of pier
K_{pile}	Lateral stiffness of pile
K_{11}	Foundation shear stiffness against pilecap lateral displacement
K_{12}	Foundation shear stiffness against pilecap rotation
K_{21}	Foundation bending stiffness against pilecap lateral displacement
K_{22}	Foundation bending stiffness against pilecap rotation
$K_{found}^{transl.}$	Equivalent translational stiffness of foundation
$K_{found}^{rotat.}$	Equivalent rotational stiffness of foundation

k_x	Horizontal (translational) stiffness of foundation spring
k_y	Vertical stiffness of foundation spring
k_h	Coefficient of horizontal subgrade reaction
L	Clear height of pier column
L_m	Depth for second plastic hinge for a concrete pile
L_p	Length of plastic hinge
μ_Δ	Displacement ductility
μ_ϕ	Curvature ductility
$M_{k,fix}$	Moment reaction at pilecap with fixed bearing at tip
$M_{k,flex}$	Moment reaction at pilecap with flexible bearing at tip
$MDOF$	Multi degree of freedom
m_0	Lumped mass for substructure
m_1	Lumped mass for superstructure
N	Number of cycles for cyclic load
P	Axial load on pile
p	Soil resistance per unit length
p_u	Ultimate soil resistance
RQD	Rock Quality Designation
r_0	Radius of circular disc
$SDOF$	Single degree of freedom
SSI	Soil-Structure-Interaction
t_w	Web and flange thickness
θ	System lateral drift
θ_0	Rotation of foundation for SDOF model
θ_c	Pier chord rotation angle

θ_s	Relative rotation of bridge to vertical about pile turning point
$\dot{\theta}_0$	Angular velocity of substructure
θ_s	Rotation of pilecap connected to a single pile at top point
θ_p^*	First plastic rotation of pile
θ_p^{**}	Second plastic rotation of pile
U_l	Total lateral displacement of superstructure at top point for SDOF model
$\dot{U}_0$	Substructure velocity for SDOF model
$\ddot{U}_0$	Substructure acceleration for SDOF model
$\dot{U}_1$	Superstructure velocity at lumped mass for SDOF model
$\ddot{U}_1$	Superstructure acceleration at lumped mass for SDOF model
u	Lateral displacement of MDOF superstructure at top point
u_0	Lateral displacement of substructure at top point
u_g	Ground displacement
u_t	Total lateral displacement of MDOF superstructure at top point
u_t	Total lateral displacement of superstructure at top point
$\dot{u}_g$	Ground velocity
$V_{k,fix}$	Shear reaction at pilecap with fixed bearing at tip
$V_{k,flex}$	Shear reaction at pilecap with flexible bearing at tip
w_s	Lateral displacement of pilecap connected to a single pile at top point
$\ddot{Y}_G$	Ground acceleration for free-field motion
Z	Plastic modulus of cross section

CHAPTER ONE: INTRODUCTION

1.1. Background

In recent years several studies have reported on the effect of soil structure interaction on the response of superstructures such as buildings and bridges to static and/or dynamic ground excitation (Mylonakis and Gazetas 2000, Wolf 1985, Gupta and Trifunac 1991), and some design recommendations have been developed in this regard (Stewart et al. 2003), (Sanayei et al. 2016). The interaction between the foundation (substructure) with the surrounding soil impacts two important dynamic characteristics of the system, namely the effective stiffness (Veletsos et al. 1971) and the equivalent or effective damping (Novak and El Hifnawy 1983). During strong ground excitation the implication of stiffness modification resulting from the foundation compliance is the reduction of seismic demands from the vulnerable components of the superstructure (e.g. piers) through participation of the foundation in the development of lateral deformations. This concept has attracted attention from researchers in the bridge-engineering field, as it presents an opportunity in bridge design to limit the design requirements for pier detailing at the ultimate limit state. Relevant research and illustration of concepts is through back analysis of field records from instrumented existing bridges (with deep or shallow foundations) (Wolf and Deeks 2004). A viable example of such beneficial effect for mitigation of seismic demands owing to soil foundation interaction is the Painter Street Overcrossing in California (PSO, Figure 1-1) which was monitored since the 1950's and has been cited in several publications (Goel and Chopra 1994), (Inel and Aschheim 2004). The PSO is an older structure, which, according to detailed investigation should have endured significant damage in past earthquakes due to old-type detailing (Kotsoglou and Pantazopoulou 2013); its good performance, which is attributed to the favorable contribution of its foundation compliance has been referred to as a paradigm for assessing of older bridge structures, and also as a concept in bridge design through the use of short pile foundations.



Figure 1-1 - Instrumented Painter Street Overcrossing (PSO) bridge (Goel and Chopra 1994)

In addition to the research on field data, some conceptual approaches have been attempted in order to formulate methods by which the beneficial aspects of Soil-Structure-Interaction (SSI) may be explicitly accounted for in design to lead to an improved structural system. For example, considering the embankment participation in monolithic bridges, where the deck is driven by the embankment displacements, a bent with stiff foundation undergoes the total deck displacement demand through flexural deformation and commensurate damage in its piers. improved pier performance may be envisioned by effecting stiffness reduction of the individual bent columns through a simultaneous increase in their number and reduced sectional size of columns per bent, so as to obtain increased deformation capacity (Kotsoglou and Pantazopoulou 2009). Another alternative to achieve reduced pier damage and to exacerbate the impact of the pile contribution is to secure increasing foundation compliance through deformation of the piles, preferably controlled to increase in moderation with increasing ground motion intensity.

Several parameters define and govern the extent of benefit or detriment effected by SSI on superstructure and pile-foundation. An indicator of the beneficial effect of soil-foundation interaction on superstructure seismic response is the reduction of deformation drift demand on pier columns. By the term deformation drift reference is made to the rotation of the chord of the deforming pier from the tangent to its deformed shape at the end support. This deformation index results from accumulation of curvatures along the pier, and therefore it is linked to the pier damage. Thus, of the total drift ratios which is defined by the relative displacement of the pier ends divided by the pier height, the fraction that results from rotation of the tangent depends on the deformation outside of the pier – such as, for example, in the foundation. These terms are depicted in Figure 1-2 below:

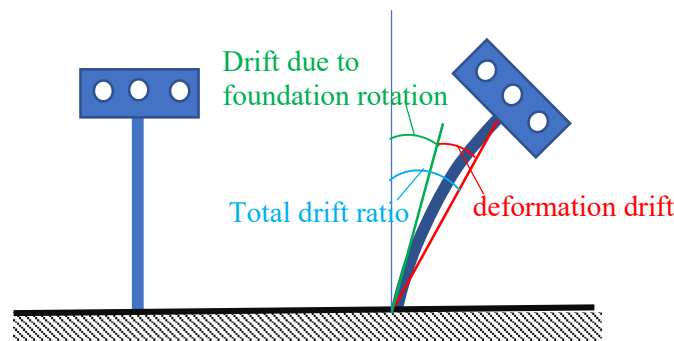


Figure 1-2 - Definition of deformation drift and illustration of foundation-related drift ratio

The same concept extended to building systems is modelled in Figure 1-3 illustrating how rotation of pilecap foundation can reduce the amount of relative displacement that occurs in the critical storey.

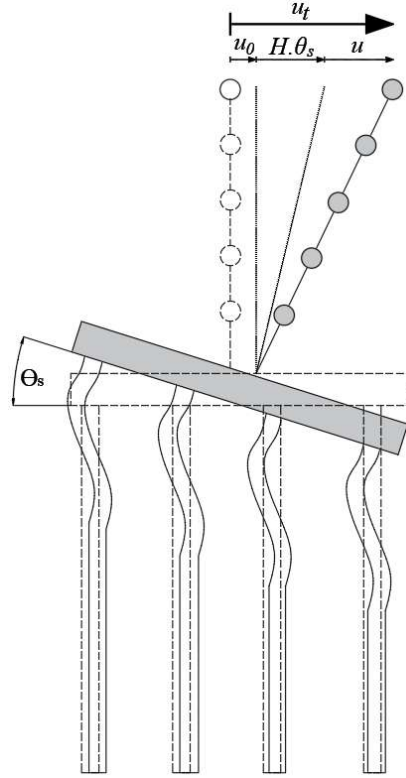


Figure 1-3 – Deformed shape of pile groups and superstructure lateral displacement considering dynamic soil pile interaction due to sway motion for buildings from (Hokmabadi et al. 2014)

It is clear from the above examples that several considerations are in order before the effectiveness of SSI in reducing demands of the superstructure may be generalized. One such consideration is the increased compliance of the system will lead to a longer effective structural period and therefore larger total displacements. It should be noted that the displacement demand is proportional to the square of the fundamental structural period, T (Clough and Penzien 2013). Therefore, there are bounds to the benefit that may be gained by merely elongating the effective period. The degree of damage in the soil that can be tolerated is another aspect of caution when calling on SSI to mitigate damage to the superstructure. In fact, this is why engineering of the SSI engagement is necessary so that it is scaled up with the intensity of the ground motion. This is all a matter of careful balance between the stiffnesses of foundation and superstructure, the two components acting as springs in series; disproportionately increasing one over the other will effectively lump the deformation in one of the two systems thereby also causing damage localization. Thus, SSI analysis approaches for bridge and building foundation should encompass several topics such as nonlinearity of soil, substructure, and superstructure in response to induced seismic excitations. These aspects will be discussed in the present thesis.

1.2. Objectives of the thesis

As mentioned in the preceding, soil structure interaction during a seismic event can be potentially beneficial in the context of seismic response of the bridge superstructure. For example, if a bridge pier is designed to resist earthquake forces, the demand on columns can be reduced at the design performance limit state by delegating a fraction of the deformation demand to the foundation. In conventional design of bridge structures, the prevailing approach is to establish a rigid foundation at both pier and embankments, especially with penetration of the pile tips in bedrock (Abdelsalam et al. 2010), (FHWA 2016). Thus, it is assumed that the imposed displacement at the base of pier is entirely sustained by the columns.

Two concerns may be raised for the current design approach. First, it is unclear whether the foundation actually remains elastic and performs as intended during seismic events. A stiff pier support means that the pier generally attracts a high moment at the base; this moment is resisted by the foundation and eventually by the soil – it is therefore likely that some interaction actually will inevitably occur regardless of whether it is considered in the design. Second, shear strength of soil can be exceeded by the sway and rocking motions of pile group and pilecap, which could compromise the vertical resistance of the piles (skin friction). An example for the importance of considering the loss in soil resistance is the case of progressive collapse of superstructure as would occur as a result of large soil movements or liquefaction (National Research Council 1985). Therefore, soil foundation interaction and its implications for foundation design should be critically considered. Moreover, it is evident that for conventional design with an increasing earthquake intensity larger damage is expected at pier as depicted in Figure 1-4-(a). Pantazopoulou and co-workers (Pantazopoulou et al. 2014) proposed a novel methodology indicating how and when to benefit from a compliant foundation in order to reduce drift demand on pier columns. This is based on the virtual work principle that is used to define the work-equivalent stiffness of the structure, from the strain energy stored in the structures, individual components, when it deforms into a specific displacement pattern (e.g. lateral sway). It was observed that disproportionately stiff components that do not deform, do not store strain energy, therefore, do not contribute to the effective stiffness of the structure. Instead the effective stiffness is dominated by the contribution of the most compliant elements, where deformation develops. Therefore, by moderating the stiffness of the foundation relative to that of the pier, not only lowers the effective stiffness of the structure, but also results in better distribution of deformation, encompassing more members. Thus, it can be argued that contrary to older bridge design approaches, which preclude any damage from occurring in the foundation, here, contribution of all components should be encouraged, allowing for some minimal soil damage at high intensity events so as to alleviate excessive damage in the superstructure. In this context, performance limits have been proposed for the soil strain, at the operational, significant damage and collapse prevention limit, where foundation is not repairable anymore or a collapse

can occur due to foundation failure (see Figure 1-4-(b)). The intent is to regulate the effective relative stiffnesses of all subcomponents of the system, to ensure that they are simultaneously engaged in deformation thereby moderating the level of damage in any single subcomponent.

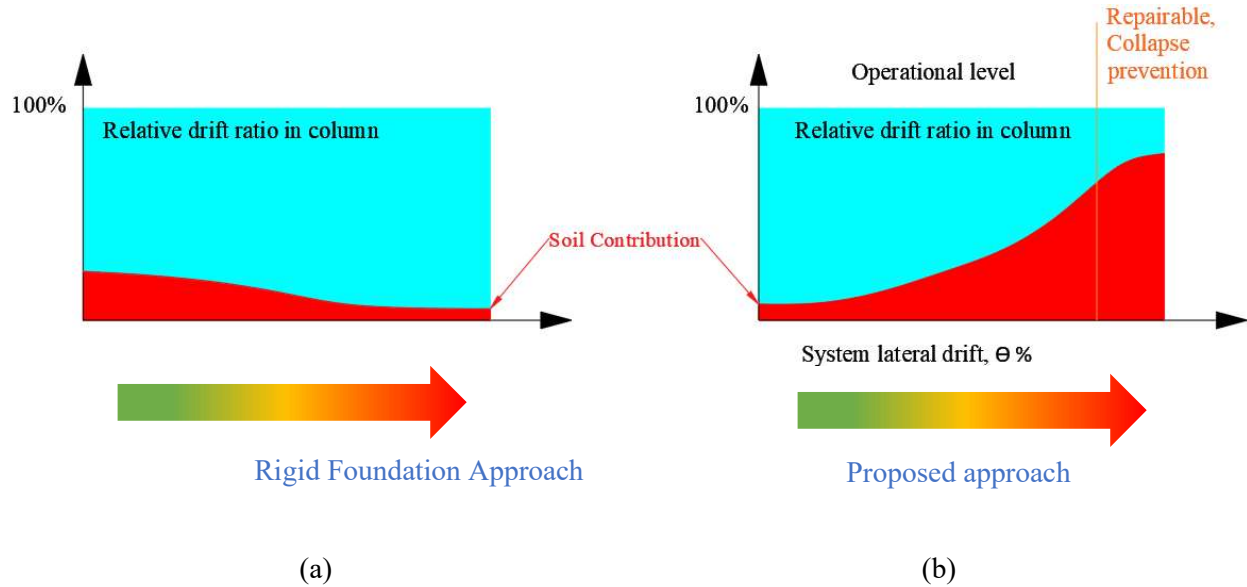


Figure 1-4 - Comparison of pier design approaches as earthquake resisting component for (a) conventional design method (b) novel design method proposed by (Pantazopoulou et al. 2014) considering beneficial soil foundation interaction effect.

Therefore, the main objective of the present work is to contribute to the development of the proposed design concept, by exploring the sensitivity of the SSI-Bridge Superstructure problem to its important design variables, such as the type of soil media (surrounding the foundation), the depth of the pile foundation, the type of foundation (shallow or deep), and the overbearing superstructure forces that are transferred as axial forces on the foundation.

In the context of practical modelling the SSI effects at the bridge supports, an additional objective of the research is to investigate the nonlinear response of the substructure consisting of piles and pilecap to lateral displacement simulating ground excitation effects. To represent these effects in simple superstructure models using equivalent springs at the base, a detailed analysis is conducted in order to relate the properties of springs to the contribution of a single pile subjected to the lateral displacement and rotation of the pilecap.

For this investigation two different pile tip fixities were assumed for embedded piles as depicted in Figure 1-5. These represent two opposite extremes in actual bridge construction. Figure 1-5-(a) models the case of a restrained pile tip (non-rotational and non-translational fixity at the tip, effected by embedding the pile in

bedrock); conducting pushover analysis for this example is achieved by applying gradually increasing values to the two degree of freedom at the top node of the pile (lateral translation, w_s , and chord rotation, θ_s) so as to obtain the resistance curve of the pile. In Figure 1-5-(b) the opposite case is depicted, where the pile tip does not extend to bedrock and therefore that point is free of stress (zero moment and zero shear).

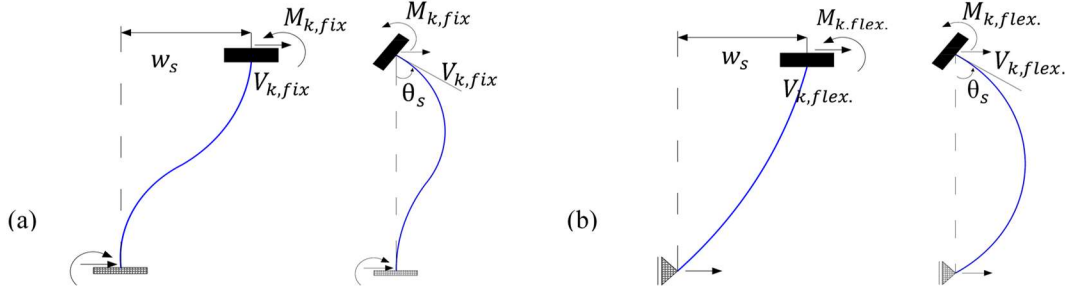


Figure 1-5 - Displaced and rotated pilecap for (a) fixed pile tip (b) compliant pile tip (Hojjat et al. 2019)

For the elastic case (Bletzinger 2015) the case with fixed tip develops twice as large bending moment and four times larger shear force reaction to rotation or translation respectively at the connection point with the pilecap as compared to the non-fixed tip case (see Eqns. 1-1 and 1-2 respectively).

$$\begin{Bmatrix} M_{k,fix} \\ V_{k,fix} \end{Bmatrix} = \frac{2EI}{l} \begin{bmatrix} 3 & 2 \\ 6/l & 3/l \end{bmatrix} \cdot \begin{Bmatrix} w_s/l \\ \theta_s \end{Bmatrix} = \begin{pmatrix} \frac{6EI}{l^2} \cdot w_s + \frac{4EI}{l} \cdot \theta_s \\ \frac{12EI}{l^3} \cdot w_s + \frac{6EI}{l^2} \cdot \theta_s \end{pmatrix} \quad (1-1)$$

$$\begin{Bmatrix} M_{k,flex} \\ V_{k,flex} \end{Bmatrix} = \frac{3EI}{l} \begin{bmatrix} 1 & 1 \\ 1/l & 1/l \end{bmatrix} \cdot \begin{Bmatrix} w_s/l \\ \theta_s \end{Bmatrix} = \begin{pmatrix} \frac{3EI}{l^2} \cdot w_s + \frac{3EI}{l} \cdot \theta_s \\ \frac{3EI}{l^3} \cdot w_s + \frac{3EI}{l^2} \cdot \theta_s \end{pmatrix} \quad (1-2)$$

Equations (1-1) and (1-2), qualitatively (nonlinearity of piles also to be considered) estimate the force reactions acting at the connection of a single pile with the pile-head in response to horizontal displacement. These forces are transferred to the superstructure bents through the pile-head. For an inelastic pile embedded in a nonlinear soil medium the stiffness of the pile is a function of the imposed displacement. To determine this relationship numerically, a pushover analysis is conducted for the pile-soil assembly under increasing displacement acting at one of the degrees of freedom at the pile head each time. To isolate the effect on the individual stiffness terms all other degrees of freedom are restrained when conducting the pushover analysis for any one of them on the pile-soil system. It will be seen that the nonlinear response of system to the aforementioned excitations deviates from the linear equations used in mechanics.

1.3. Scope and organization of thesis

The thesis is developed in six chapters representing results from a multitude of simulation experiments conducted in order to better understand the implications of SSI, first between a single pile and its pilecap, and then between the foundation and the bridge pier seismic response. Chapter 2 reviews the mechanics of the underlying physical problem and reviews the background literature. In Chapter 3, a single pile representing the foundation conditions of the selected reference bridge was modelled and studied per pushover analysis (Reese and Wang 2006). The reference structural model studied (pilecap interacting with individual pile) is part of an actual overpass bridge on Hwy 401 located in Eastern Ontario. The case study considered consists of a two-span continuous, 64 m long deck, supported through bearings on abutments and a bent cap. The bent is a planar frame comprising two columns, which are anchored into the pilecap. The original bent foundation system consists of eighteen driven steel H-Piles that are arranged in two groups of nine (one group per bent column), all connected to a massive reinforced concrete pilecap and driven to the bedrock. The bridge replaced an older structure that had different foundation elements in terms of foundation material and pile diameter. Here the old piles were treated as a benchmark point, set to be modelled and contrasted with the replaced piles, which will be reflected as a part of the evaluation of results.

To perform the sensitivity analysis, piles in the model were extended to various depths above bedrock so as to reproduce various degrees of compliance in the boundary conditions of the pile member, whereby the pile tip (free end of the pile) can displace and/or rotate. Therefore, in the scope of this thesis, first, a complete sensitivity analysis was performed to study the effect of varying pile geometry and subsurface materials on response of a single pile to imposed lateral displacements. The focus of Chapter 3 is on the interaction of a single pile with its pilecap, when subjected to an incrementally increasing horizontal translation and a rotation imposed at the centroid of a massive (rigid) pilecap. Pilecap response of the arrangement to induced displacements is then plotted in the form of force displacement diagrams for several soil types, and different pile geometries. The resistance of piles to a pilecap transverse translation or rotation at the point of connection was calculated for increasing magnitude of kinematic disturbance under displacement control and is represented herein in the form of pile pushover resistance envelopes. In order to assemble the complete foundation stiffness under the bridge bents comprising the contributions of several piles, reduction coefficients are applied to the single-pile resistance curve, so as to account for the interaction between piles within a pile-group (Foye et al. 2011), (Viggiani et al. 2012). Using these values equivalent spring properties are derived in order to account for the foundation compliance in simple bridge models under lateral load. These depend on relevant characteristics of the foundation problem (ground type, pile inclination, length, pile dimension and shape.)

Different types of soil are found in the provided geotechnical study report of case study bridge (Golder Associates 2016). In Chapter 4, for consistency of the results and to facilitate generalization of the findings, four uniform soil types, namely a soft clay, a stiff clay, a loose sand and a dense sand and three combination of stiff clay and dense sand were adopted in the models. This was done in order to examine a broad range of possible ground conditions. First, stiff clay, soft clay and dense sand in uniform layers were considered as the embedment layer. In consecutive soil models, a sand layer gradually replaced stiff clay, increasing in thickness from pile-tip to the top of the pilecap, in 3 increments, in order to observe the effect on the system's response under the imposed pile-head displacements. Moreover, the axial force on the pile, representing the self-weight of superstructure was varied in the sensitivity analysis considering its effect on foundation's pushover resistance. Finally, the pile-length was also varied, with the maximum length extending down to bedrock. Analyses was carried out using two different software employing algorithms for spatial integration of the governing equations either using with the finite difference method or the finite element method. This aspect of the analysis also enabled a brief comparison of two methods for modelling soil-pile interaction.

As mentioned in the preceding, one objective of the thesis was to simulate the seismic response of substructure at the base of the bridge by means of nonlinear springs and thereupon study the seismic response of bridge pier when it forms part of a complete structure system. This part of the study is discussed in Chapter 5. The selected bridge pier used in the sensitivity analysis consisted of a bent cap and two columns connected to a single pilecap supported by a group of H-section steel piles. Similar to a single pile, the response of pile group and pilecap to cyclic excitations was computed. Properties of nonlinear springs were obtained from force displacement results from group pile analyses. Pile group analysis were limited to a certain length of piles and Axial Load on pilecap which were later adopted to superstructure analysis. Four uniform soil types were considered for the pile group sensitivity analysis.

As a benchmark for compliant base bridge seismic analyses, the superstructure of the selected bridge was modelled with a fixed base foundation in a pertinent software platform (CSiBridge) and was excited with several seismic events. An important objective of this study was also to compare the bridge seismic response with two connection types (monolithic and elastomeric bearing) of bent cap to deck. Drifts in bridge columns were calculated and compared for different scenarios.

Subsequently, the case of compliant foundation was modelled at the pier base and the same analyses as for fixed base bridge were performed, using two different bent-cap to deck connection arrangements. The compliant base foundation was analysed for four different substructure stiffness in addition to the fixed base case. Also, four different levels of damping were considered (representing the different soil energy

dissipation characteristics) and same analyses were conducted, in order to understand the effect of damping on the seismic response of the bridge pier when a foundation is compliant.

Chapter 6 reviews the results obtained with reference to the different parameters investigated. Comparisons are made with respect to the two different methods of simulating the substructure contribution at the pier base in terms of the calculated pier seismic response. The contribution of compliant base abutments in the longitudinal direction are also discussed based on the results of Chapter 5. The chapter summarizes the salient findings of the study, along with key points to be considered in future research.

CHAPTER TWO: LITERATURE REVIEW

2.1. Soil structure interaction evaluation methods

In practice, the effects of soil structure interaction is neglected during design of many conventional structures on grounds that consideration of a rigid foundation may be more conservative in the estimation of demands in a force-based design framework. Of course, in such a case, displacement demands are underestimated which may be unconservative in the context of seismic assessment or may obscure convenient retrofitting options that could exploit on this concept. Several past studies have showcased the beneficial effect of soil foundation interaction for the superstructure, particularly the piers (Goel and Chopra 1994). Moreover, the collective SSI effects on the seismic response of the substructure should be closely considered for design, where foundation failure can cause progressive collapse or disproportional damage to superstructure (De Carlo et al. 2004). A variety of modes of failure may occur, localizing in piles, connections to the pilecap, or permanent settlements and rotations to the body of the pilecap (see Figure 2-1).



Figure 2-1 - Failure of pile to pilecap connection due to earthquake imposed load and soil pile interaction (Wei et al. 2008)

An essential element of all SSI analysis methods is the representation of the compliances of the foundation (NEHRP Consultants Joint Venture 2012) in the analytical model of the entire system. Different degrees of complexity can be introduced in the model in this regard: SSI effects are classified into inertial interaction, kinematic interaction and soil foundation compliancy influence (FEMA 2009). These effects are caused by specific foundation and superstructure properties. For kinematic interaction, the stiffness of substructure

can increase or reduce the magnitude of transferred seismic forces from the soil to the superstructure. Inertial interaction refers to the soil mass engaged in the vibration of the system; vibration may be partially damped by underlying soil layers, which highlights the significance of substructure damping for SSI and superstructure dynamic response. During early SSI studies the effect of stiffness and damping of substructure on response of shallow foundations were elaborated. It was proposed that the overall damping of underlying soil layers and stiffness may be represented by dashpots connected to a circular disk located on the surface of an elastic soil medium as in Figure 2-2 (Veletsos et al. 1971) and (Zhang and Makris 2002).

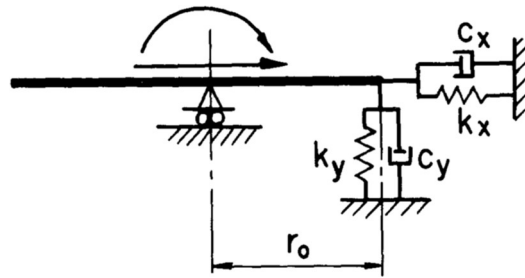


Figure 2-2 - Simplified model for modelling of overall soil stiffness and damping contribution of the foundation from (Veletsos et al. 1971)

A fixed base structure modelled with the illustrated model in Figure 2-2, would require infinitely large values for the lateral and vertical stiffness of the springs shown (k_y and k_x). With respect to the properties representing SSI, it was mentioned that the stiffness and damping of dashpots are not only dependent of soil properties such as the Poisson ratio (q), but also the circular frequency of the excitation (ω), and the size of the foundation (Veletsos et al. 1971). The above-mentioned modelling approach for soil is the backbone of the modelling attempts in the current work.

SSI methods evaluate the structure seismic response to a free field motion; however kinematic interaction considers deviations of ground input motions from free field motions owing to the modification of the ground response owing to the participation of the bridge mass; this aspect is controlled by several parameters such as the shape and embedment depth of foundation (NEHRP Consultants Joint Venture 2012).

For the design of deep foundations such as piles it is relevant to also consider the nonlinearity of the soil but also the formation of plastic hinges in piles as a result of the ground deformation, as this also affects crucially the performance of the superstructure during excitation (NEHRP Consultants Joint Venture 2012).

Methods of modelling structural systems for SSI analysis are divided to two categories. First a direct analysis is feasible so that the substructure and soil are discretized with contact nodes at the interface boundaries. This method is called *model in continuum* (FEMA 2009). All displacements of the excited soil mass are transferred to the substructure based on coupling restrains between nodal pairs as shown in Figure 2-3 (Wolf and Deeks 2004). The practical versatility of the direct modelling method is controversial, because the required input motion need be the modified record rather than the free field as described before, whereas variations in foundation properties (nonlinearities) due to induced forces need be also considered (NEHRP Consultants Joint Venture 2012).

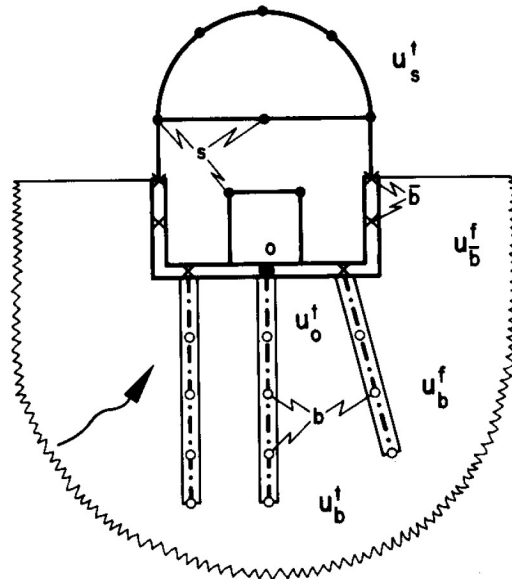


Figure 2-3 - Model of structure supported on piles in continuum (Wolf and Deeks 2004)

A second, alternative approach for modelling SSI is known as the *substructure* method. In this procedure, free field ground motion is the primary input. In the present work, the ground motion was adopted from simulated records generated according with Atkinson (Atkinson 2009). A uniform soil layer or combination of only two soil layers was used in all analyses of the soil-foundation interaction for the sake of simplicity. Furthermore, the overall substructure and soil seismic response are evaluated and represented as springs and dashpots at the base of superstructure. For this purpose, substructure and soil are excited with the selected ground free field motion, whereas the system response is evaluated at the base of superstructure in terms of a base shear vs lateral displacement resistance curve that is the composite effect of the sequence of substructure layers working in series in this model.

To illustrate this approach a simplified model of a bridge swaying in the transverse direction is depicted in Figure 2-4; the deck's tributary mass is modelled as lumped mass (m_1) supported by a pier (having translational stiffness (k) and damping (c), which is excited by a ground acceleration ($\ddot{u}_g$) (Ghannad 2017),

(NEHRP Consultants Joint Venture 2012). The height of the pier column is H ; foundation mass and moment of inertia are represented by m_0 and I_0 respectively. The figure shows that the lateral displacement (U_0) and rotation (θ_0) of foundation due to sway and rocking motion result in lumped mass displacement of U_1 . The dynamic response of the system may be calculated with the following matrix Equation (2-1), where the equivalent stiffness and damping of foundation have been computed to k_s and c_s for sway and k_r and c_r for rocking motion. The expression below is a statement of dynamic equilibrium and represents the equation of motion of the system shown in Fig. 2-4 when subjected to a time dependent excitation such as an earthquake, where $\ddot{U}$ and $\dot{U}$ represent acceleration and velocity of the lumped mass, respectively.

$$\begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_0 & 0 \\ m_1 H & 0 & I_0 \end{bmatrix} \begin{Bmatrix} \ddot{U}_1 \\ \ddot{U}_0 \\ \ddot{\theta}_0 \end{Bmatrix} + \begin{bmatrix} k & -k & -kH \\ -k & k + k_s & kH \\ 0 & 0 & k_r \end{bmatrix} \begin{Bmatrix} U_1 \\ U_0 \\ \theta_0 \end{Bmatrix} + \begin{bmatrix} c & -c & -cH \\ -c & c + c_s & cH \\ 0 & 0 & c_r \end{bmatrix} \begin{Bmatrix} \dot{U}_1 \\ \dot{U}_0 \\ \dot{\theta}_0 \end{Bmatrix} = \begin{Bmatrix} 0 \\ k_s u_g + c_s \dot{u}_g \\ 0 \end{Bmatrix} \quad (2-1)$$

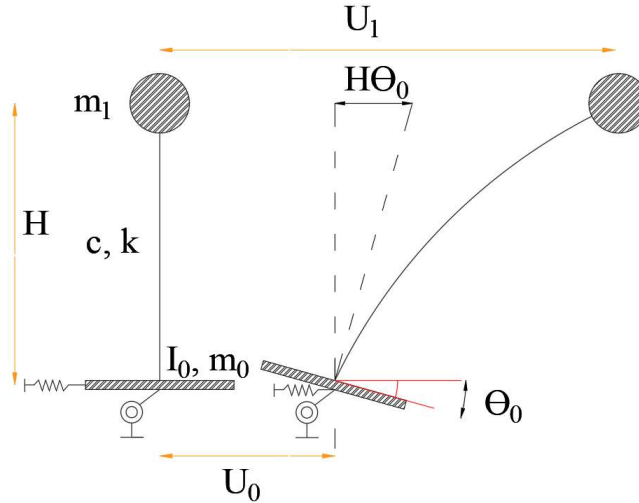


Figure 2-4 - Simplified model of a lumped mass with spring and dashpots at base after (Ghannad 2017), (NEHRP Consultants Joint Venture 2012)

2.2. Analysis approach for single pile and pile groups

The Canadian Bridge Design Code (CSA S6-19) dedicates Section 6.11.2.2.1 to the calculation of pile lateral resistance; Several research and field investigations were performed to find an optimized solution for modelling the interacting deep foundation with surrounding and undelaying compliant soil during dynamic excitation. The two principal methods for modeling SSI (Substructure method and continuum) for

application in practice ((El Naggar and Bentley 2000), (Zhang et al. 2016), (Reese 1997)) were discussed in the preceding section. In particular, with regards to the substructureing approach, an alternative method for the SSI problem is depicted in Figure 2-5 (Kausel et al. 1978): a sequential approach is used by first considering the wave propagation and reflection from the soil layers using computer software such as ProShake (EduPro Civil Systems 2017). The calculated motion at the foundation level is used as input excitation for the foundation to calculate the response at the interface nodes (at the points of contact with the superstructure). Ultimately, interface nodes are replaced with springs and dashpots to replace the substructure. This time the superstructure is excited with the same motion.

2.2.1. Substructure method

This approach is based on Winkler's original method of subgrade reaction (Taghavi et al. 2017). In the current research, the substructure method was preferred over the modelling in continuum due to possibility of application of one-dimensional elastoplastic springs, whose stiffness can be computed directly from developed methods and can be monitored for diverse range of parameters. As discussed earlier, a section of this work was dedicated to performing an extensive sensitivity analysis on foundation parameters which influence the seismic response of the superstructure due to dynamic soil foundation interaction.

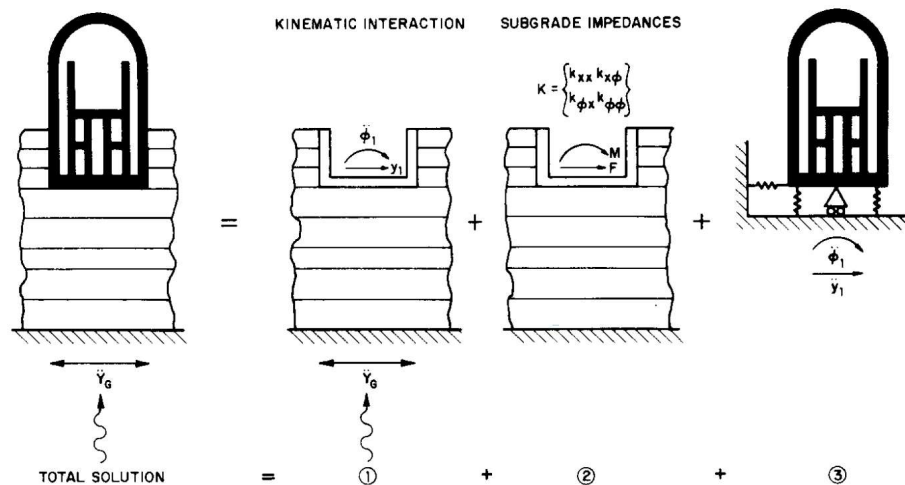


Figure 2-5 - Problem solving for soil structure interaction (Kausel et al. 1978)

The steps summarized in the preceding were adopted for modelling of pile foundation at the base of bridge structures. Two techniques are mainly applied for application of springs according with the substructure method. In the first option, springs and dashpots are applied directly at the base of superstructure, which represent the entire substructure as shown in Figure 2-6.

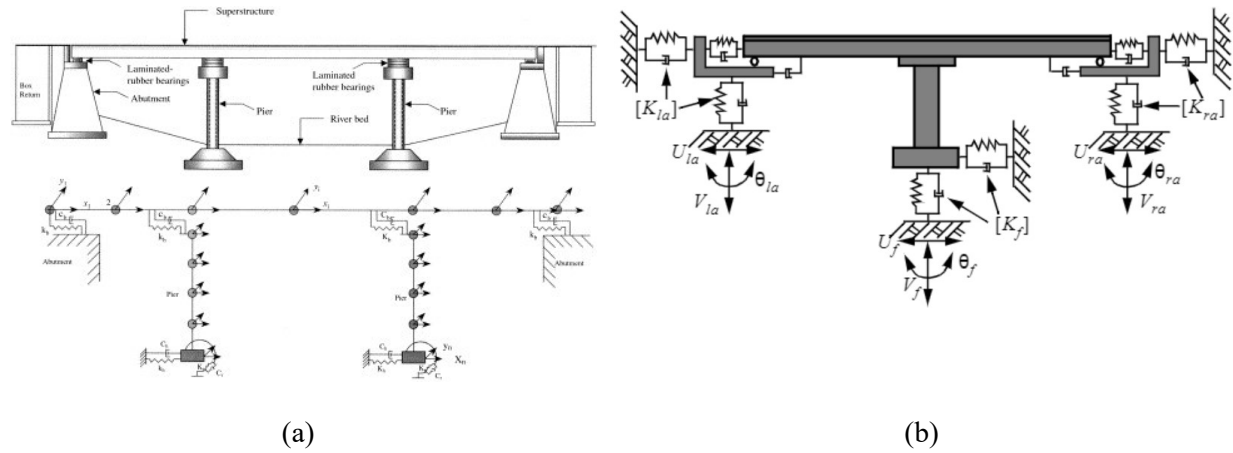


Figure 2-6 - Methods of modelling of pile foundation bridges with springs and dashpots at pier and embankment base for (a) Modelling deck and columns with lumped masses (Tongaonkar and Jangid 2003) (b) Realistic model of superstructure (Zhang and Makris 2002)

In the second option, lateral translational springs are attached along the pile skin to represent the response of soil at several elevations of piles as shown in Figure 2-7 (Rahmani 2014).

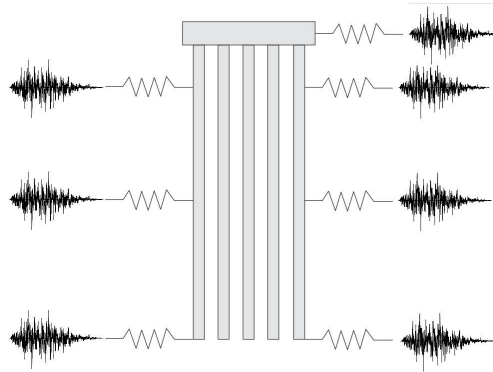


Figure 2-7 - Soil springs attached to pile-skin from (Rahmani 2014)

The undelaying soil could consist of many layers or of a uniform layer. Response of each soil layer to dynamic excitation is different than that of the other layers based on the soil properties such as shear strength (c'), friction angle (ϕ'), shear-modulus (G), etc. The so-called equivalent linear stiffness for the springs has been used in this approach (Rahmani et al. 2016). But in reality, soil behaviour in response to seismic excitations consists of a nonlinear force-deformation relationship. Thus, the use of nonlinear stiffness and pertinent nonlinear algorithms have also been considered (Wolf 1991).

2.2.2. Pushover analysis variants and soil resistance

For estimating the force deformation resistance curves of the pile skin springs dynamic or static loads may be used (NEHRP Consultants Joint Venture 2012). In the first scenario (dynamic approach), a foundation soil interface node is excited with a free field motion. Hysteresis response loops represent the input value for calculation of soil spring stiffness (Rahmani et al. 2012), (Nogami et al. 1992). Computed stiffness values obtained from this method depend on the seismic event intensity as shown in Figure 2-8. The envelope of the hysteresis loops at each depth level would be required in order to construct the secant stiffness-deformation plot in this approach.

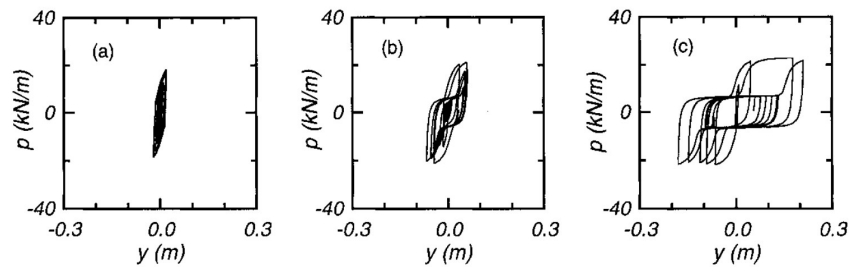


Figure 2-8 - Force deformation behaviour at depth of x of a single pile embedded in soft clay (Ross Boulanger et al. 1999)

When using a static approach, empirical equations are developed based on field pile pushover tests. Field tests were carried out by means of imposing static lateral loads on single pile head and measuring force displacement at several depths along the length of the pile as depicted in Figure 2-9 (Reese et al. 1975). Soil resistance (p) is measured per length of pile. This value is measured against displacement of soil over a specified increment of tributary depth (known as p - y curve).

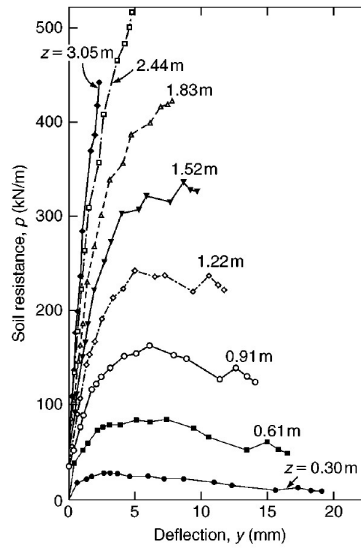


Figure 2-9 - Soil resistance versus deflection at several depths along an embedded pile in stiff clay from field static push over tests (Reese et al. 1975)

As mentioned earlier soil resistance at the interface, interacting with pile skin, is dependent on both pile and soil material strength. Moreover, pile dimensions, the presence of water and the depth from the top influence the obtained resistance curve. In addition, the load type (i.e., either monotonic or cyclic) is important and ought to be considered in the context of modeling, in order to account for soil resistance degradation due to repeated cycles of lateral load reversals.

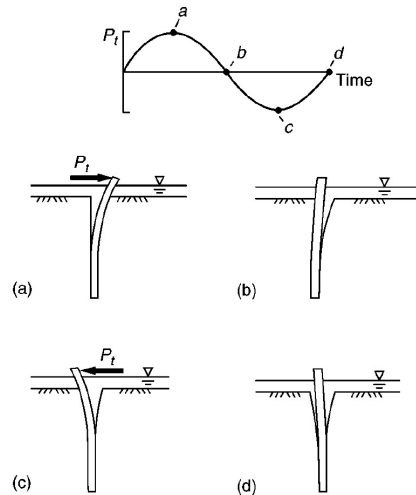


Figure 2-10 – Schematic soil reaction to lateral top displacement of an embedded pile in stiff clay (Long and Reese 1984)

For instance, in Figure 2-10 displacement of soil around the pile head due to cyclic lateral movement is illustrated in four steps that comprise a full cycle. It is clear that the material surrounding the pile head is

consolidated in each half cycle and therefore a decreased soil resistance occurs for subsequent displacement reversals (Reese et al. 2002). Widely used empirical equations for soil resistance versus displacement have been recommended for soft clay (Matlock 1970), stiff clay (Reese and Welch 1975), and sand (Reese et al. 1974). In the present study, cyclic loading was applied for computation of the p-y curves used to represent the pile-skin resistance in the bridge system, as well as for estimation of the force displacement response of the pile head. Further details with regards to the dependency of soil resistance on other influencing parameters will be provided in the following chapters.

The underlying physical model of soil-pile interaction is described by the governing differential Equation (2-2) (Reese and Matlock 1960); this is used for computing soil resistance as a function of displacement. In the present analysis, nonlinear p-y curves are used to represent the response of horizontal springs attached to the pile skin as the pile undergoes lateral translation or head rotation; Equation (2-2) is solved using finite-differences after discretization of the independent variable (x) (Reese et al. 2006).

$$EI \frac{d^4 y}{dx^4} - P \frac{d^2 y}{dx^2} - E_s y + q = 0 \quad (2-2)$$

In Equation (2-2), E and I are modulus of elasticity and moment of inertia of the pile cross-section, respectively. P is the axial load acting on pile top, E_s is the elastic modulus of soil and q is the lateral distributed load, imposed along the length of the pile. Also, Δx is the depth increment and Δy is the lateral displacement. The finite difference approximation is organized as follows: Considering a length h of the column subdivided to n discrete segments, by sequential sections, the ith section, the governing equation is expressed using finite differences according with Equation (2-3) (Isenhower and Wang 2016).

$$a_i y_{i-2} + b_i y_{i-1} + c_i y_i + d_i y_{i+1} + e_i y_{i+2} = f_i \quad (2-3)$$

Above equation is the backbone of calculations performed by computer program LPILE and GROUP, which were used for the substructure analysis of this thesis. Terms in equation (2-3) are defined as follows:

$$a_i = EI_{i-1}$$

$$b_i = -2EI_i - 2EI_{i-1} + Ph^2$$

$$c_i = EI_{i+1} + 4EI_i + EI_{i-1} - 2Ph^2 + 2E_s h^4$$

$$d_i = -2EI_{i+1} - 2EI_i + Ph^2$$

$$e_i = EI_{i+1}$$

$$f_i = qh^4$$

To account for the soil pile group interaction, the same method as in the case of a single pile is used, to compute the cumulative soil resistance to lateral excitation (Figure 2-11). For the application in the present work, the overall response of pile and pilecap arrangement was needed as input to the base of the superstructure model. The soil and pile resistance and interaction against imposed lateral loads are typically computed analogous to a single pile. If piles are located closely, reduction factors are applied in order to modify the calculated resistance of the individual piles. More details in this context are available in the following chapters.

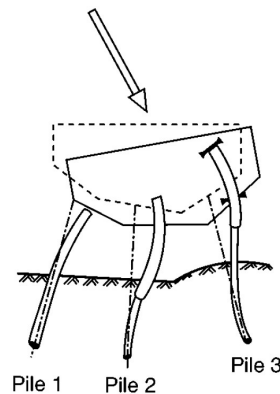


Figure 2-11 - Group of piles connected to pile head, with different types of connection (Reese et al. 2002)

In the current thesis four types of foundation soil, namely stiff clay, soft clay, loose sand and dense sand are used in the pile pushover analysis; results were plotted in the form of shear displacement ($V-\Delta$) and moment rotation ($M-\theta$) curves; secant values for the stiffness terms that couple the associated degrees of freedom and the corresponding actions shown in Figure 2-13-(b) are also given in plots against the imposed displacement / rotation magnitude. Results from the analysis can be used to develop resultant foundation springs at the base of a F.E. bridge superstructure model to represent the overall nonlinear foundation compliance (De Carlo et al. 2004), (Ghannad et al. 1998), (Singh and Vinayak 2015), without the need for detailed modeling of the substructure.

2.3. Implication of foundation deformation for the bridge structure

2.3.1. Mechanics of SSI participation in superstructure response

In the earlier sections the advantages of partial contribution of the pile foundation to stiffness of the bridge system were considered as a vehicle to reduce the seismic demand on piers (Farantakis et al. 2014). This concept was understood after evaluation of field data in instrumented bridges in the US (e.g. Painter street overcrossing, California), illustrating that damage to the bridge bents was mitigated owing to a rotational compliance at the pier supports during moderate seismic events. Recently the number of instrumented

bridges has increased on account of this observation, while several analytical and numerical studies have been used to correlate the field records with the beneficial effect of SSI on short pile foundation bridges. For instance, Figure 2-12 compares the observed damage at foundation and pier of a numerically analysed bridge for a fixed and a compliant foundation (Antonellis et al. 2014).

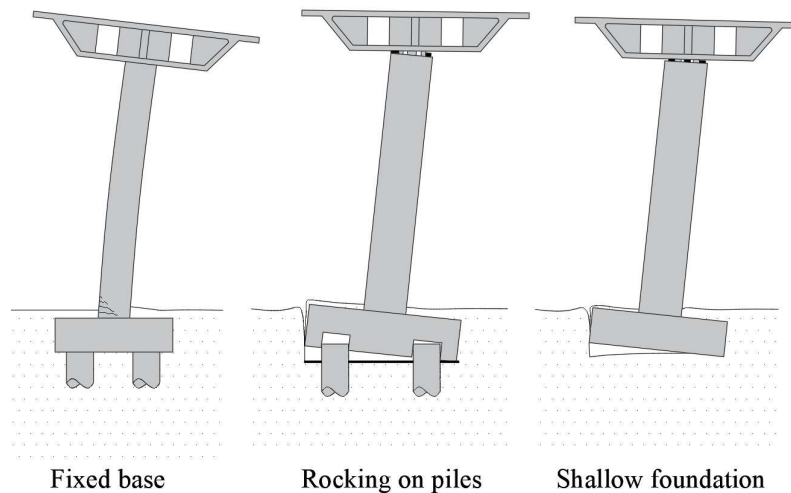


Figure 2-12 – Deformed shape of a foundation, column and deck subjected to rocking motion for fixed base, pile base and shallow foundation (Antonellis et al. 2014)

Initially the emphasis was placed on the abutments, since it was determined that due to their being supported on short pile foundations, they are driven by the embankment response during earthquake motion carrying along the bridge deck in rigid-body translation to the same displacements ((Inel and Aschheim 2004) and (Goel and Chopra 1994)). In monolithic structures this renders the piers the most vulnerable component of the bridge, owing to the excessive drift demands; significant pier damage is expected in that case if the pier is stiff and the foundation non-compliant. But a compliant foundation, in such cases, relieves partly the drift demands, since part of the drift is developed inside the foundation, by translation and rotation of the pilecap, through deformation of the piles (Kotsoglou and Pantazopoulou 2009).

Figure 2-13-(b) depicts the base of the pier shown to the left (Figure 2-13-(a)), with the definition of the degrees of freedom considered. The effective translational stiffness of the bent is modified from the reference value associated with pier deformation alone ($K_{pier} = 12EI/H^3$), when soil is compliant, as follows: If Φ is the fundamental mode shape of translational vibration of the pier-foundation system, and it includes nonzero share of deformation below grade as depicted in the figure, then the total effective stiffness (from Virtual Work principle) is:

$$K_{pier-foundation}^{eq} = K_{pier} \cdot \Delta\phi_{pier}^2 + K_{found}^{transl.} \cdot \Delta\phi_{found}^2 + K_{found}^{rotat} \cdot \left(\frac{\partial\phi}{\partial z} \Big|_{z=H_B} \right)^2 \quad (2-4)$$

where the first term represents work done through pier deformation, the second through translation of the foundation and the third represents work done by rocking of the pilecap (here $\Delta\Phi_{found} = \theta_s \cdot H_B$) (Farantakis et al. 2014).

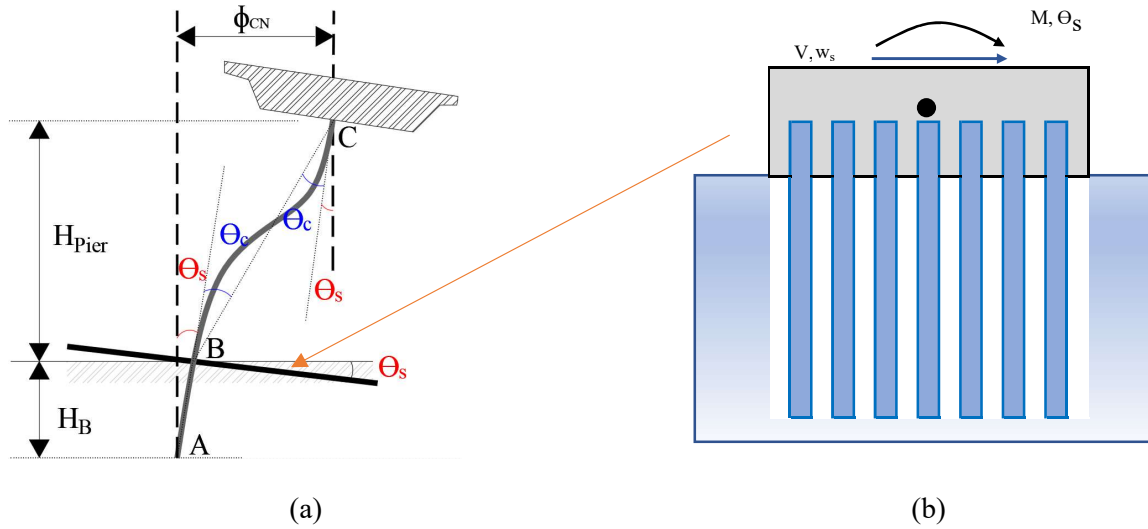


Figure 2-13 - Deck-Pier-Foundation connection: (a) Definition of terms after (Farantakis et al. 2014), (b) Degrees of freedom and corresponding actions at pilecap. Note that demands in individual subcomponents are partly mitigated if all such elements of the system are engaged in deformation

Note that with increasing translational and rotational stiffness of the foundation and surrounding soil ($K_{found}^{transl.} \rightarrow \infty$ and $K_{found}^{rotat} \rightarrow \infty$), lateral movement and rotation of pilecap tends to zero, therefore, the second and third terms of Equation (2-4) disappear. This means, if foundation is rigid, stiffness of the entire arrangement equals the stiffness of pier, in other words the relative displacement between pier top and base is identical to the displacement of the top of pier (Farantakis et al. 2014). On the other hand, a compliant foundation reduces the share of lateral drift ratio undertaken by pier deformation according with: $(\theta_c = (\Phi_{CN}/H_{pier}) - \theta_s)$.

2.3.2. Nonlinear deformation of piles and columns

Boundary conditions for deep foundation such as fixity of pile tip in bedrock or embedment in soft soil, as well as connection type to pilecap determine the lateral and rotational stiffness of substructure against pilecap displacement and rotation. Earlier it was discussed how a rigid pile tip bearing magnifies the rigidity over the scenario where pile tip can rotate. These were discussed for elastic stiffness of the member. It was

also mentioned that pile head is sufficiently embedded in pilecap so that a rigid connection between both is assumed.

In this context, particular attention should be given to inelastic deformation of the piles. If the cap translates beyond a certain displacement limit (Δy_1), the pile may yield at the connection point to the pilecap as it can be seen in Figure 2-14(a). Prior to this point, the pile deflection is elastic, and the stiffness of system could be computed from Equation (1-1). Equations have been developed for linear stiffness (K_I) of piles in cohesive soil in connection with horizontal subgrade reaction (k_h). For a concrete pile $K_I = \sqrt{2}EI_e / (\sqrt[4]{\frac{EI_e}{k_h}})^3$ (Poulos and Davis 1981), where EI_e is the flexural rigidity of the pile.

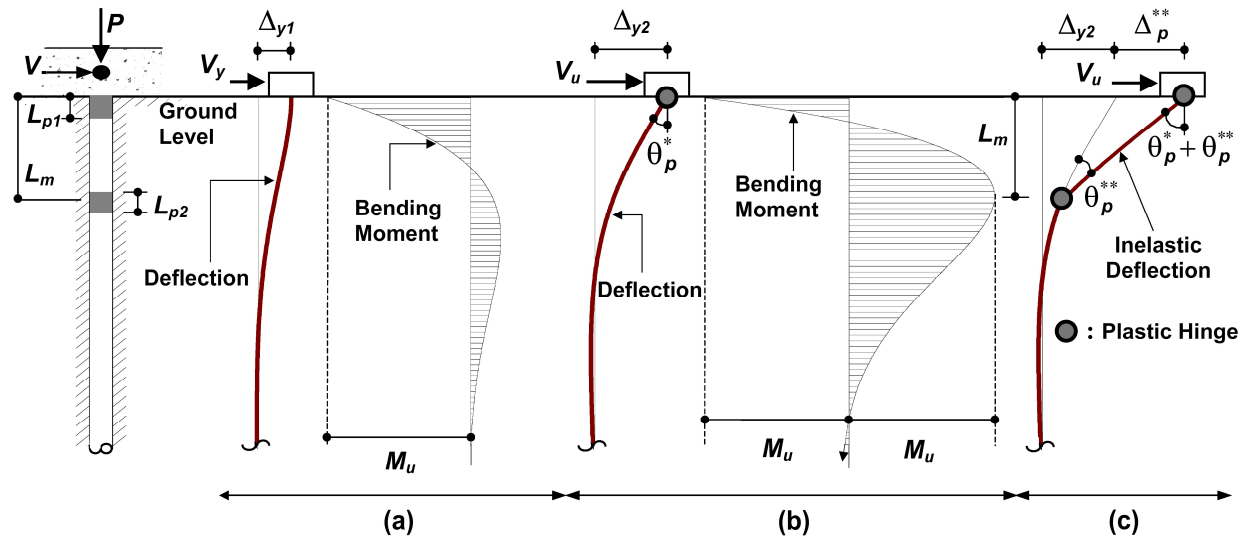


Figure 2-14 – Elasto-plastic deformation of pile with fix connection to pilecap due to lateral cap displacement for (a) first yield (b) second yield and (c) ultimate state (Song et al. 2004)

Ultimate moment at yielding of piles is, $M_u = \Delta y_1 \cdot EI_e / \sqrt[4]{\frac{EI_e}{k_h}}$. From Figure 2-14-(b) it is observed that after first yielding, the pile head can rotate, enforcing significant rotation demand in the plastic hinge of the pile adjacent to the connection with the pilecap. Further lateral translation of the pilecap results in a second yield of the pile at depth L_m . The term for calculation of plastic rotation at surface after first yield, for a concrete pile is given $\theta = (\Delta - \Delta y_1) / (\sqrt{2} \sqrt[4]{\frac{EI_e}{k_h}})$ after (Song et al. 2004), where Δ (generally larger than Δy_1) is an induced deflection at pilecap. Similar equations are also available for embedded piles in cohesionless soils (Matlock and Reese 2002).

In Figure 2-15, the push-over resistance curve of the discussed pile is depicted. An idealized trilinear curve is fitted to the plotted envelope. K_1 represents the stiffness to elastic deformation whereas K_2 corresponds to the stiffness of yielded pile at connection to cap. The importance of pile behaviour during sway and rocking motion for superstructure is the fact that after the second yield, while further displacement occurs at pile level, the magnitude of sustained shear force at the cap remains nearly constant. Obviously after displacement Δ_u failure of pile and collapse of the superstructure is possible. Therefore, it is suggested that the curvature ductility demand in piles is monitored and restricted to within safe limits. Moreover, set limits for Serviceability Limit States (SLS) need to be considered in regard with irreversible lateral movement of structure, and permanent dislocation of its soil.

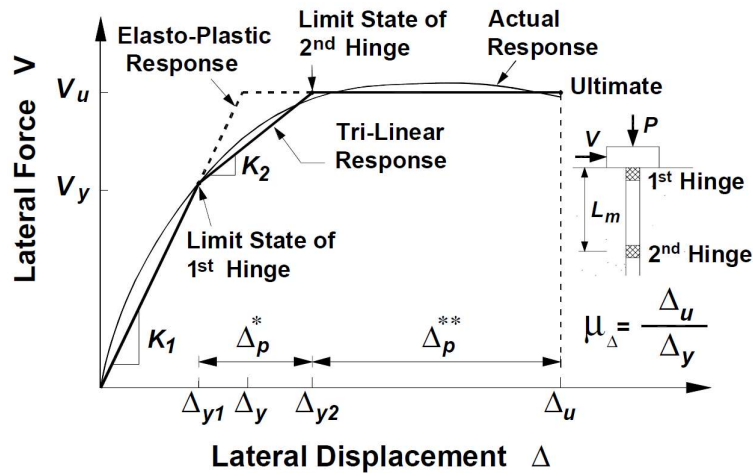


Figure 2-15 – Idealized push-over curve for pile head displacement with fixed head connection to cap
(Song et al. 2004)

Displacement ductility capacity of the pile is defined as the ratio of ultimate demanded displacement to the yield displacement, which is controlled by the plastic deformation (Δp^{**}) after second yield. The second plastic rotation is determined by, $\theta_p^{**} = \Delta p^{**} / L_m$, which is equal $(\Phi_{ul} - \Phi_i) / L_{pl}$, where Φ_i and Φ_{ul} are the curvature demand and the ultimate curvature capacity for the first plastic hinge, respectively, and L_{pl} is the length of first plastic hinge (the defining equation is given in the following chapters as in (Aviram et al. 2008) and is the same as for columns). The curvature ductility demand at first hinge is defined as $\mu_{\phi i} = \Phi_i / \Phi_y$. Figure 2-16 illustrates how the magnitude of damage can be moderated by reducing curvature ductility demand in the pier (Song et al. 2004). The concept is similar to the paradigm explored here, where now the kinematic relief occurs at the pier top: In Figure 2-16-(a) the fixed connection of pier to deck is illustrated. This type of connection corresponds the old approach of designing bridge bents, versus the illustration in Figure 2-16-(b), where the deformed shape of a pier with flexible connection to the deck is depicted.



Figure 2-16 - Deformed shape of pier for (a) monolithic connection to deck (b) connection through bearing (Buckle et al. 2006)

The latter design approach, which corresponds to a reduced deformation drift demand on pier is recently adopted in practice where embankment movement drives the lateral demand in the bridge (FHWA 2014). A flexible connection of pier to deck can be provisioned by means of elastomeric bearings (Buckle et al. 2006). Similar to piles, plastic hinges can form at either the base and/or the top of pier columns due to lateral movement. Expressions for the bottom hinge are given with reference to Figure 2-17 in the following paragraphs (FHWA 2014) and (Panagiotou 2008).

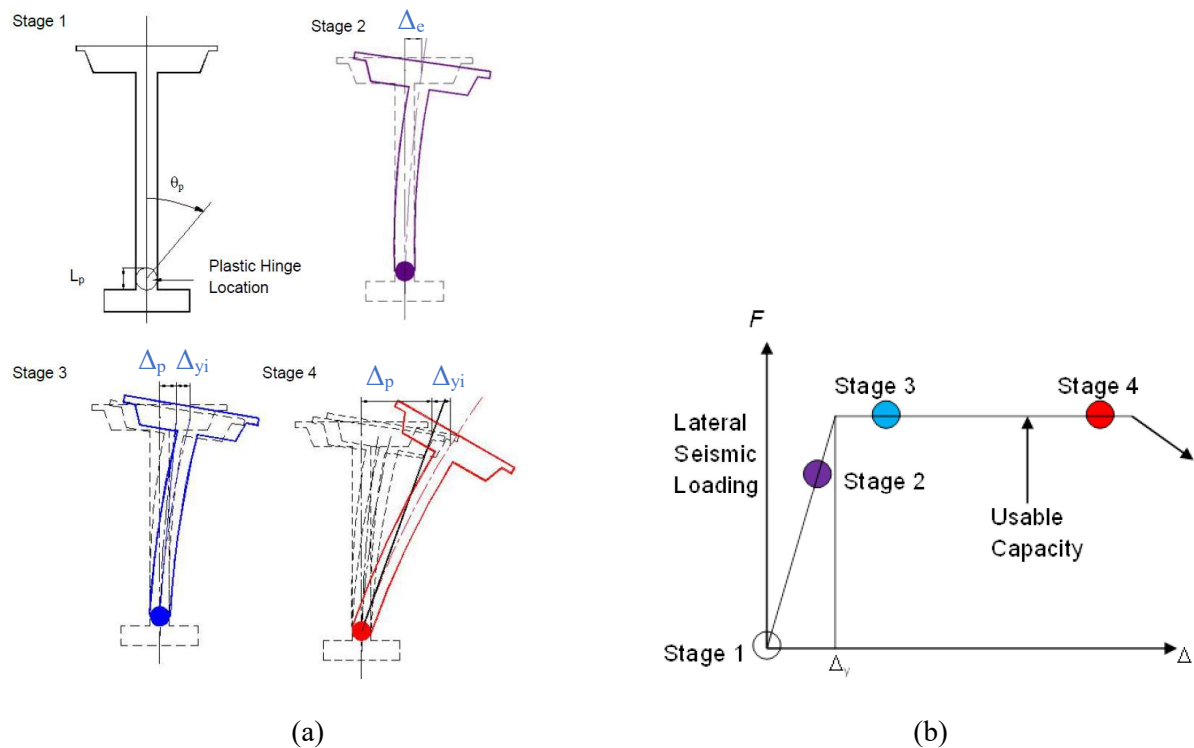


Figure 2-17 - Stages of plastic hinge formation for a cantilever column (a) deformed shape (b) force-displacement plot (FHWA 2014)

It is assumed that the pier columns undergo four stages of deformation from the initial condition to failure, which can be due to concrete crushing, reinforcement buckling, web shear cracking or degradation of the lateral resistance of column with increasing number of lateral displacement reversals. Equations (2-5), (2-6), and (2-7) are given for the displacement of column top at four stages with reference to Figure 2-17:

$$\Delta_e = \frac{FL^3}{3E_c I_{eff}} \quad (2-5)$$

$$\Delta_{yi} = \frac{F_p L^3}{3E_c I_{eff}} = \frac{1}{3} \Phi_{yi} L^2 \quad (2-6)$$

$$\Delta_p = \Phi_p L_p (L - 0.5L_p) \quad (2-7)$$

Where L is the clear height of the column, $E_c I_{eff}$ is the effective flexural stiffness of the column. Numerous expressions for plastic curvature in relation with limit state (failure modes) for reinforced concrete columns are recommended in the literature (Ian Buckle et al. 2006).

CHAPTER THREE: FOUNDATION RESPONSE TO LATERAL LOAD SIMULATING EARTHQUAKE ACTION ON BRIDGES

3.1. Introduction

In this section the response of the substructure to several types of loading is presented and discussed. A bridge structure was selected as a reference case study, that complies with the requirements of structural design in Canada and especially in the Province of Ontario. The chosen structure is also representative of recent structural design practice in Canada. The selected bridge is an overpass connecting East to West of Avonmore Road over Hwy 401. It was considered for rehabilitation as a part of the periodical repair assessments program by Ministry of Transportation of Ontario (Ontario Ministry of Transportation 2016). Therefore, a repair design was provided which was made available for the research purpose of this study (Golder Associates 2016). Based on the available drawings and specifications, extensive sensitivity studies were performed to evaluate the response of the substructure (in this chapter) and superstructure (in following chapters) to seismic excitation.

The objective of the current study was to assess the effect of seismic demands on compliant-base foundations such as piled raft foundations on soft ground. In order to generalize the sensitivity analysis, the superstructure of Avonmore overpass was used for analyses in the following chapters, however, the geometry of piles and site geology were subjects of sensitivity analysis, to assess the response of similar structures in diverse ground conditions due to soil structure interaction (SSI).

In the current chapter, first the geotechnical aspects of the foundation and soil are explained. Next, a single H-pile and the respective reinforced concrete pilecap of Avonmore bridge are modelled in program LPile (Isenhower and Wang 2015). This is a computer program that uses the Finite Differences method to solve the nonlinear governing differential equations (see Chapter 2, Equation (2-3)). A set of pushover analyses were performed to obtain pile-head response to a cyclic lateral load, as well as to generate the response curve for soil lateral resistance along the pile depth at defined increments. Results are presented in graphs of varying soil stiffness versus pile head displacement, in addition to p-y curves that are used as the main indicators of soil resistance in this work (Rajapakse 2016). Following this, the piles and pilecap under the bridge central pier were modelled in program GROUP in order to account for pile-to-pile interaction. To this end, pushover analyses were performed to compute the response of the group of piles and pilecap to cyclic displacement. As a point of independent evaluation of the LPile results, the computer program PLAXIS 2D was also used to conduct single pile push-over analysis; the latter program uses the finite element method to solve the numerical model. A comparison between push-over analysis, carried out by LPile and PLAXIS 2D, is also presented. Results of the current chapter are subsequently used as input to

a structural analysis program for further study of seismic response of the bridge system to the imposed excitations.

3.2. The case study - Avonmore bridge overpass

Selected structure to serve as a reference case study for this work is a reinforced concrete overpass located in south east Ontario between the cities of Ottawa and Montreal. The four span overpass was constructed in 1960s to provide access from South to North over the 401 Highway providing connection to the County Road 15, Figure 3-1. Partial repair and replacement of bridge foundation as well as deck and abutments was planned and completed in 2018 (Ontario Ministry of Transportation 2016).



Figure 3-1 - Avonmore overpass (Old structure) [Google map]

The old structure consisted of three sets of bents including three circular columns per setting, which were monolithically connected to the deck. The superstructure was supported on deep foundations including several rows of H-piles driven to bedrock. For the rehabilitation process, the three bents were replaced with a single one consisting of two circular columns connected to deck through the bent cap as shown in Figure 3-2. The abutments, wingwalls and deck were adjusted to conform to geometric and standard requirements. Furthermore, the H-piles were replaced with new sets of driven 310 x 110 H-piles piles.



Figure 3-2 - Avonmore overpass (New structure) [Google map]

As usual, prior to planning for restoration of foundation an extensive geological investigation was performed by the geotechnical consultant for the project (Golder Associates 2016). According to their report the study area is known to be comprised of comparatively undrained clay to non-cohesive sand layers that are sedimented above glacial till plains. Ground investigations were performed by means of several field and lab tests. Among them, three boreholes were driven to bedrock as can be seen in Figure A-1, from left to right, labelled as BH 14-731, BH 14-733, and BH 14-732. Foundation soil layers and their elevations are summarized in Figure A-1 of the Appendix.

3.3. Matters and material

3.3.1. Selection of appropriate soil profiles

With the intention of generalizing the outcome of this work, four sets of uniform and three sets of nonuniform soil layers were selected as in Figure 3-3 for numerical modelling over the multilayer Avonmore bridge ground site.

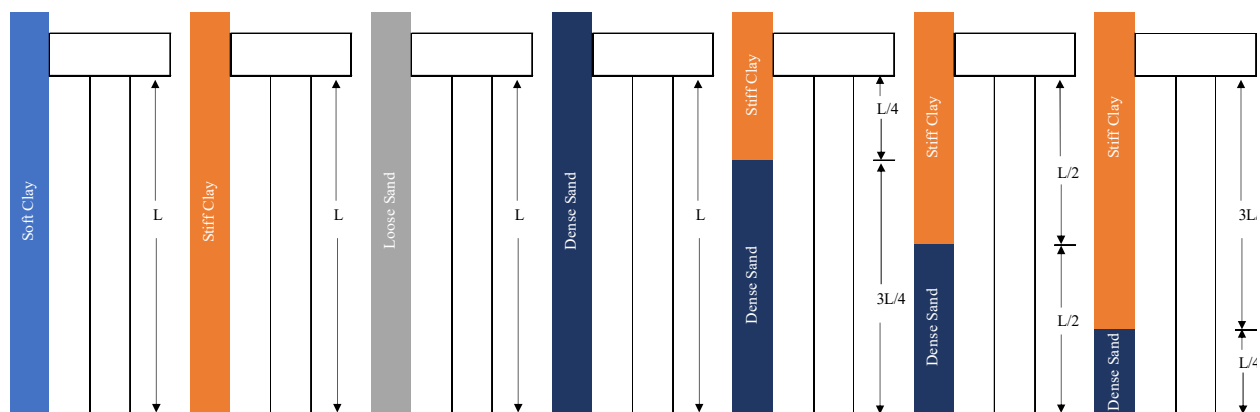


Figure 3-3 – Selection of appropriate soil profiles for numerical analysis

Required geotechnical properties of the selected soil types for numerical analyses are presented in Table 3-1. These soil types consist of loose and dense sand, as well as stiff and soft clay. Assumed soil properties for the numerical analyses are mean values of corresponding soil properties found at Avonmore bridge site based on the geotechnical analysis (through bore holes) of the site. For instance, a range of densities was reported for stiff clay, but only the average value was used as input to computer program. Geotechnical properties of soil such as unit weight (γ), undrained shear strength (S_u) and angle of friction (ϕ'), and coefficient of lateral earth pressure were estimated from SPT-N values obtained from field tests using well established relationships (Bowles 2001), (Lindeburg 2008), (Kumar et al. 2016), (Bowles 1992), (Clayton et al. 1995), (Canadian Geotechnical Society. 1992). A thick bedrock layer (10m) was added to the bottom

of soil layers for all analyses. The Rock Quality Designation (RQD) measurements on the recovered shale limestone bedrock core samples of Avonmore can be found in Table A-4 of the Appendix.

Table 3-1 – Selected soil properties for numerical analysis

Soil Type	Cohesive soil		Cohesionless soil	
	Loose Sand	Dense Sand	Soft Clay	Stiff Clay
γ (kN/m ³)	14.2	17.1	11.2	16.6
S_u (kPa)	-	-	18	75
ϕ' (°)	30	40	-	-

3.3.2. Reinforced concrete pilecap

Geometry and material of pilecap at pier and abutments were adopted from drawings of the Avonmore bridge as depicted in Figure 3-4 and Figure 3-5. As can be seen in Figure 3-4, the pilecap is resting on eighteen 310 x 110 H-piles. All piles under the pier were driven vertically.

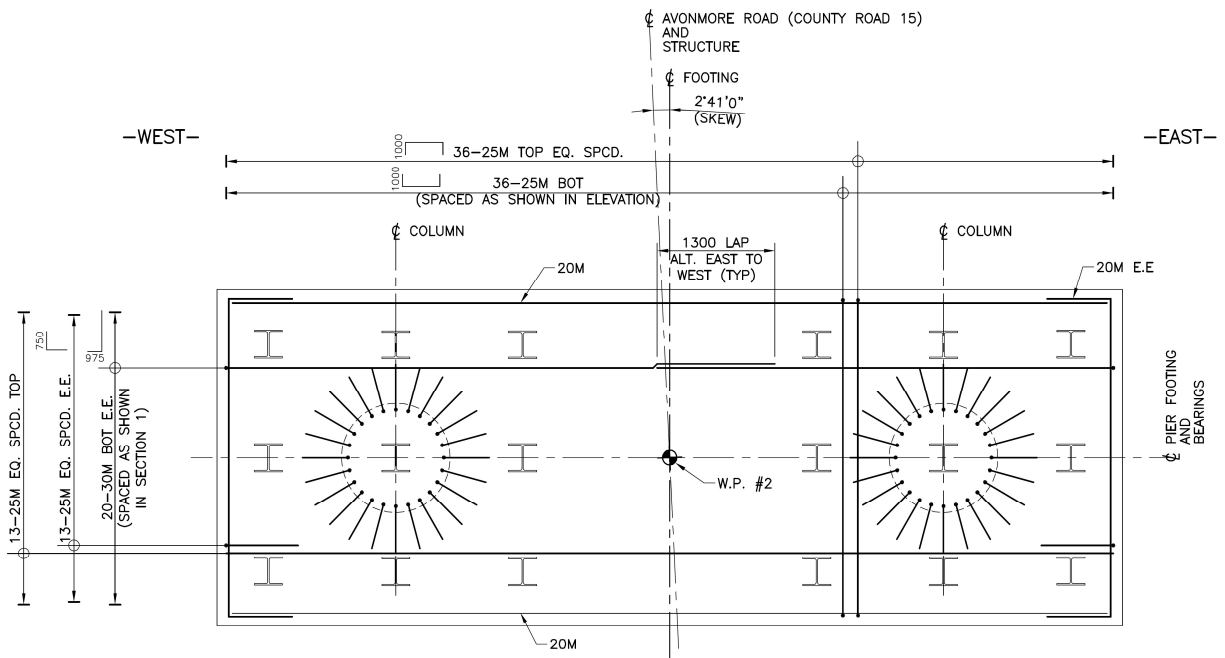


Figure 3-4 - Pier footing plan view (MMM GROUP 2013)

Pier foundation consisted of a reinforced concrete pilecap with 10m length and 3.7m width. The shorter edge of the pilecap was along the bridge layout line. The pilecap under the pier was a 1.00 m thick plate

with longitudinal 30M reinforcing bars on the tension side and 25M bars in the compression side as shown in Figure 3-5 (MMM GROUP 2013).

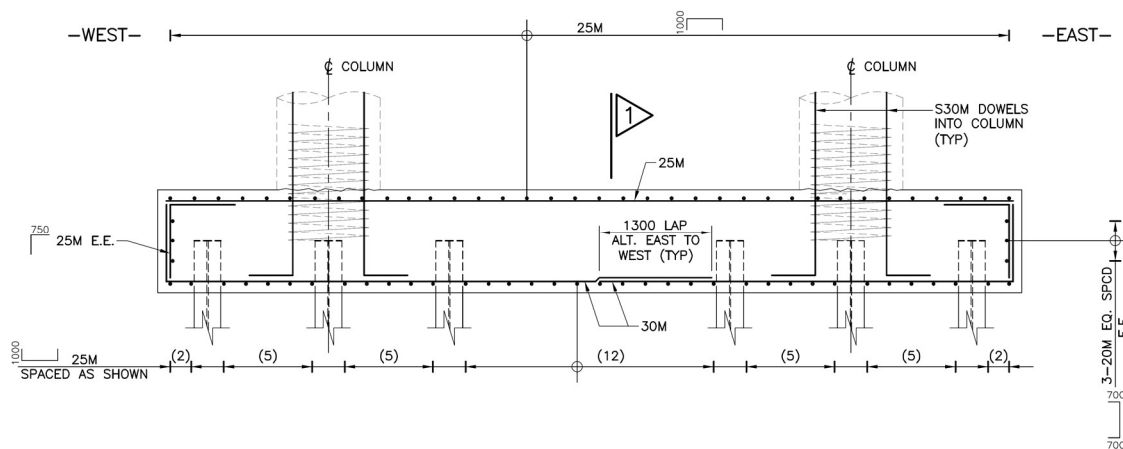


Figure 3-5 - Pier footing side view (MMM GROUP 2013)

Abutments were reinforced concrete members 10.912 m long by 1.2 m thick at both ends of the bridge. For modelling, the cross section of the abutments was idealized to be rectangle. Abutments were founded on a set of eight 310 x 110 H-piles, whereby two piles out of eight on both sides of the abutments were battered (1 to 10) as in Figure B-1 of the Appendix. The thickness of the abutments from the given elevations is 3.22m as shown in Figure B-1 (MMM GROUP 2013). The top surface of the pilecap replacing foundation at the pier was about 0.8 m below the ground surface. Pile heads were embedded in the pilecap over a length of 0.5m. Thus, the relative displacement and rotation of pile heads and pilecap was taken equal to zero and the connection between pile and pilecap was modelled as having fixed displacement and rotation (FHWA 2006).

3.3.3. Driven steel piles

Two different pile sections were used for the analyses. The first section was a BP 10 × 42 steel section, which represents the new piles that replaced the old foundation of Avonmore bridge. The second pile section was HP 310 × 110, which was considered to be loaded in the directions of both, the weak and strong axis to take in to account the fact that seismic load could potentially excite the structure in any of the two directions. The properties of both sections are summarized in Table 3-2. Yield strength of structural steel used (type: A572Gr50) was taken equal to 345 MPa with the elastic modulus of 200 GPa.

Table 3-2 - Properties of pile sections - Avonmore overpass

Section	Weight (kg/m)	Depth, (mm)	Flange Width, (mm)	Thickness		I_{xx} (mm ⁴)	I_{yy} (mm ⁴)	F_y (MPa)	d_e (mm)
				Flange (mm)	Web (mm)				
HP 310 × 110	110	308	310	15.5	15.5	23700	7740	290	349
BP 10 × 42	63	246	256	10.7	10.7	8740	2980	203	283

3.4. Simulation Study

Extensive numerical analyses were performed to understand the substructure response to incrementally applied cyclic lateral displacement. In this chapter p-y curves generated by the program LPile are presented, and a brief outline of the method used by the program to produce the curves is discussed. This is done in order to provide insight into the role of important parameters for computing soil response to lateral displacement. Introducing soil response in relation with lateral displacement to pile skin in program CSiBridge in form of nonlinear springs, is an effective method to study the effect of soil structure interaction on the response of the superstructure to seismic excitation. Subsequently, the response of pile head and all piles to cyclic pilecap displacement is presented in the form of push-over response curves. These curves are introduced later at the base of bridge piers in the form of a lateral and a rotational spring (CSiBridge 2017a). Results from LPile were also correlated against those obtained from the finite element analysis.

3.4.1. Introduction to sensitivity analysis

The main parameters that potentially affect the response of foundation and subsequently the response of the superstructure to seismic forces include vertical Axial Load on the foundation, soil properties, geometry of foundation and foundation stiffness. The effect of these parameters on the response of substructure to the imposed pilecap lateral movement and rotation was investigated through extensive sensitivity studies. Initially, axial loads of magnitudes of 500, 1000 and 2000kN on a pile length of 10m and HP 310 × 110 section were considered. Additional pile lengths of 4.2, and 6m were considered in separate analyses. A different pile section (write section here) was also considered to study the effect of foundation stiffness. Figure 3-6 outlines the relevant details and scope of the sensitivity studies.

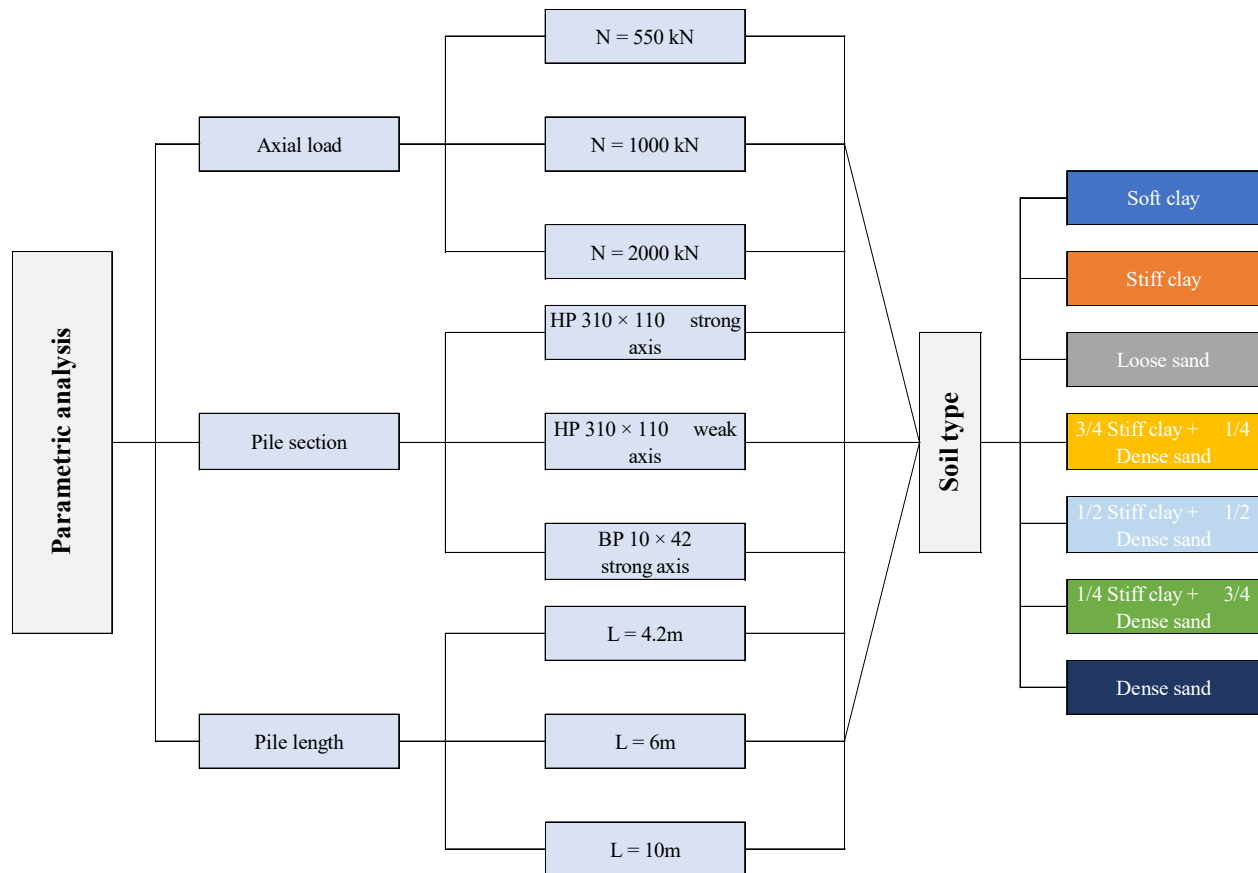


Figure 3-6 - Outline of substructure sensitivity analysis

3.4.2. Provisions for numerical modelling of a single pile

For the current research, computer program LPILE version 2018.10.05 with Data Format 10 was used to model a single pile and its connection with the pilecap. The conventional mode of analysis was chosen among several other options. This mode of analysis enables pile push-over analysis, auto computes the pile-head stiffness matrix, and accounts for a nonlinearity in pile and soil material properties. Furthermore, p-y curves at specified elevations were also obtained as part of the output. The p-y curves were estimated at 11 elevations along the pile length with 1m spacing. An additional curve was estimated at the mid-height of the pilecap. Much of this information is required to model the soil reaction in the structural model. For the purpose of analysing a more general case, the length of piles at Avonmore bridge were extended from 4.2m to 10 m, which is more usual in practice.

As formerly noted in this chapter, for the comprehensive investigation of SSI effect on the superstructure two methods will be used to estimate the reaction of the substructure against lateral movement, at the base of the superstructure: first as two springs which represent the total translational and rotational reaction of the substructure, and second, as distributed springs attached to the pile skin. In this chapter it is explained

that how the substructure reaction per both methods is computed by relevant analyses. Particular attention was paid to modelling of the reinforced concrete pilecap, which mobilizes larger soil reactions compared to a pile, at the same depth, due to greater equivalent diameter. For the second method, two concerns were raised. First, it was interpreted from preliminary analyses that the soil reaction (p) to movement (y) changes for the two scenarios depicted in Figure 3-7, just under the pilecap (approximately 50cm for the 310 x 110 H-piles and pilecap of the case study). Further to this matter, a single pile connected to the pilecap was modelled to compute the substructure response as mentioned above. Since there were 18 piles connected to the pilecap, it was assumed that each pile interacts with $1/18^{\text{th}}$ of the pilecap as in Figure 3-8. In Figure 3-7-(a), $1/18^{\text{th}}$ of the pilecap and the H-pile connected to the pilecap is depicted. As briefly discussed before, soil in proximity of the pile and pilecap can be modelled as nonlinear springs, which simulate the reaction of soil to the movement of pile and pilecap (p - y curves). For instance, the soil reaction to displacement at pilecap level is indicated as $(p-y)_{pc/18}$, and at 50cm below the pilecap as $(p-y)_{p-1}$. Furthermore, the same pile was modelled without the pilecap, where the soil spring 10cm under the pile top is shown as $(p-y)'_{p-1}$ (Figure 3-7-(b)). The objective of the latter model was so as to illustrate through comparison with the first model, the effect of the pilecap restraint of the p - y spring properties. For both scenarios soil reaction-displacement curves were generated by the program, LPILE, in response to the pilecap displacement, where it was observed that the soil reaction for the first case is different from the second setting.

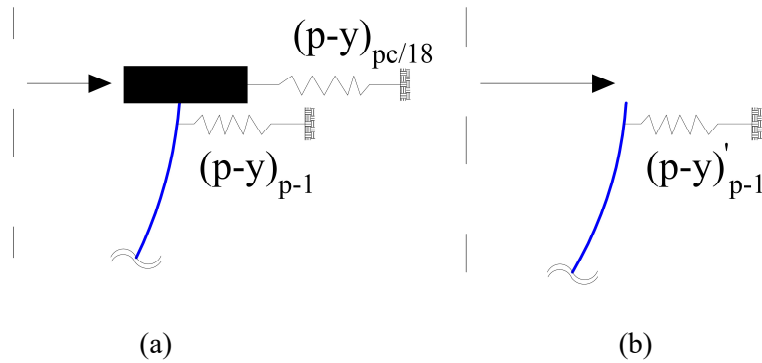


Figure 3-7 - Single pile modelled with pilecap (a) and without pilecap (b)

Therefore, it was deemed necessary to include the pilecap in to the model in order to represent a realistic correlation between soil reaction and soil movement for Point 1 as shown in Figure 3-7. On the other hand, particular attention should be paid to the dimensions of the pilecap. The pile group under the pier consisted of eighteen H-piles. As noted above, it was assumed that the sphere of influence of each pile is one-eighteenth of pilecap area as it is depicted in Figure 3-8 for half of pilecap. For instance, the area marked in red was considered to act together with a pile as modelled in LPILE.

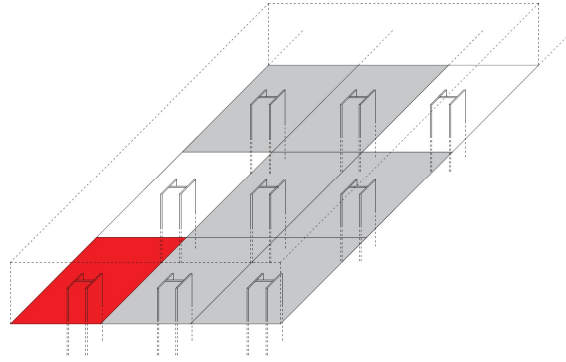


Figure 3-8 - Idealized pile and pilecap setting

Similar to piles, soil reaction due to movement of the pilecap was required for use with the CSiBridge model. Three scenarios were considered within the context of modelling the pilecap in LPILE. These are to generate the p-y curve for: a) One-eighteenth of pilecap connected to single pile, b) Complete pilecap connected to single pile, c) pilecap is independently modelled and pushed over. From the first scenario, as depicted in Figure 3-9 (a) p-y curves at several depth along the single pile were obtained. It may be argued that the first scenario is not realistic for computing the p-y curve in proximity of the pilecap, due to different size of modelled pilecap ($1/18^{\text{th}}$ of full member) than the reference pilecap. Therefore, from the second scenario the p-y curve in proximity of the pilecap was obtained, while effect of size of the reference pilecap was considered. It was determined that the obtained p-y curve for the third scenario, could also effectively satisfy the requirements of the bridge-system to model pilecap spring in CSiBridge, since soil-displacement curve was similar to the one obtained from the second scenario.

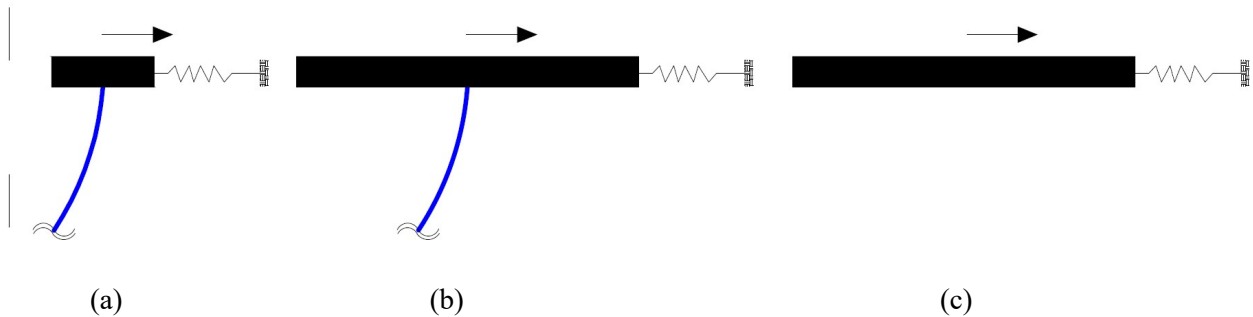


Figure 3-9 - Scenarios for modelling the translational function of pilecap in LPILE (a) 1/18 with pile (b) 1/1 with pile (c) 1/1 without pile

When the pilecap was modeled as a single entity as depicted in scenario (c) in Figure 3-9, with the same boundary conditions and loading, the generated p-y curve at the centroid of pilecap, (which will be the location of attached spring to the base in the structural model), remained unchanged compared to scenario (b) in Figure 3-9. Additionally, analyses were performed to examine the effect of loading type (lateral

movement or rotation) on the p-y curves of the pilecap, generated at mid-height of the footing for the second and third scenarios. It was found that pilecap lateral displacement or rotation have no effect on soil reaction p-y curves for both scenarios. In other words, existence of single pile does not affect the p-y curve for the pilecap at its mid-height.

Therefore, for this study, first a tributary portion of pilecap connected to a single pile (see Figure 3-7 (a)) was modelled in LPILE for all soil layer variants as shown in Figure 3-6. Next, the entire pilecap was separately modelled, where the p-y curves for further analysis in this research were generated.

3.4.3. Generating p-y curves and discussion of the results

3.4.3.1. Soft clay subjected to cyclic load

In this section the method to develop the p-y curve for soft clay is explained as a representative sample for the remainder of the study. Curves are functions of several parameters, reflecting the dependencies of the stiffness of soil whereby the effective stiffness of the substructure results from deep foundation's stiffness and the reaction of soil to lateral movements.

In order to establish the p-y curve for cyclic loading the following values are defined (Matlock 1970):

γ'_{ave} = average effective unit weight of soil, varying with the depth of the foundation with distance from ground surface.

x = depth from the ground surface to the point where a p-y curve is established.

c = soil shear strength at depth x , whereby in this work this value remained unchanged for a particular soil type.

b = equivalent diameter/diameter of the pile at the point of establishing the curve.

J = Coefficient for soil type, assumed to be 0.5 for soft clay; and

y_{50} = Deflection at one-half of the ultimate soil resistance which is equal: $2.5 \epsilon_{50} b$ or from Table 3-3.

Table 3-3 - ϵ_{50} vs. soil consistency (Matlock 1970)

Soil Consistency	ϵ_{50}
Soft	0.020

Medium	0.010
Stiff	0.005

p_u is the ultimate soil resistance per unit length of pile and can be calculated as following by (3-1):

$$p_u = \min \left(\left[3 + \frac{\gamma'_{avg}}{c} x + \frac{J}{b} x \right] cb \text{ \& } 9cb \right) \quad (3-1)$$

$$x_r = \frac{6cb}{\gamma'_{ave} b + Jc} \geq 2.5b \quad (3-2)$$

A typical p-y curve along the pile length consists of three parts; p values are normalized with respect to p_u , whereas y values are normalized with respect to y_{50} . The ascending part of the curve is limited to p/p_u values ≤ 0.72 and is described by the equation:

$$p = \frac{p_u}{2} \left(\frac{y}{y_{50}} \right)^{\frac{1}{3}} \quad (3-3)$$

The second part of the curve is a linear descend, ranging from $p/p_u = 0.72$ to a residual value of $0.72(x/x_r)$. The corresponding coordinates for the milestone points in the horizontal axis are, $(y/y_{50}) = 3$ and 15 respectively. Beyond the latter point the ratio of p/p_u remains constant as shown in Figure 3-10 (Isenhower and Wang 2016).

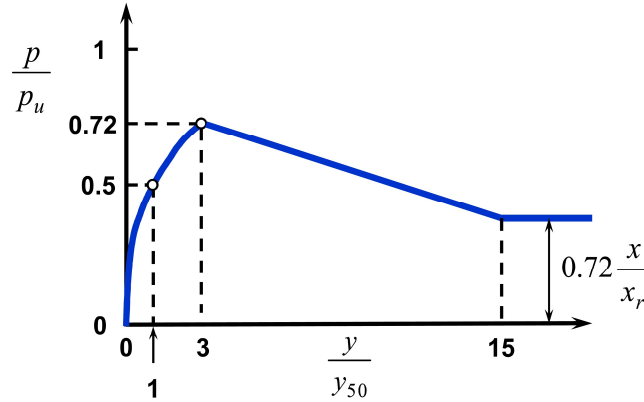


Figure 3-10 - Schematic p and y correlation in soft clay for cyclic loading (Isenhower and Wang 2016)

Some of above-mentioned equations indicate that the soil reaction is a function of soil properties like shear strength as well as foundation dimensions such as depth and diameter. Results from the LPile analysis (Isenhower et al. 2018) for soft clay at 11 different locations along the pile length are presented in Figure 3-11. It is evident that the soil response at elevation of 0.6m under the pilecap depth has the highest values

due to larger equivalent diameter of the pilecap as compared to a single pile. At 1.2m from top of the pile is the transition region from pilecap to pile, the lowest soil response values were obtained here. This can be related to the occurrence of stress concentration in this area. It is also seen that for the depths between 2.2m and 10.2 the soil response to the lateral displacement is almost the same (corresponding p-y curves are all overlapping), which indicates that soil resistance against lateral deflection of embedded pile in soft clay is not a function of depth.

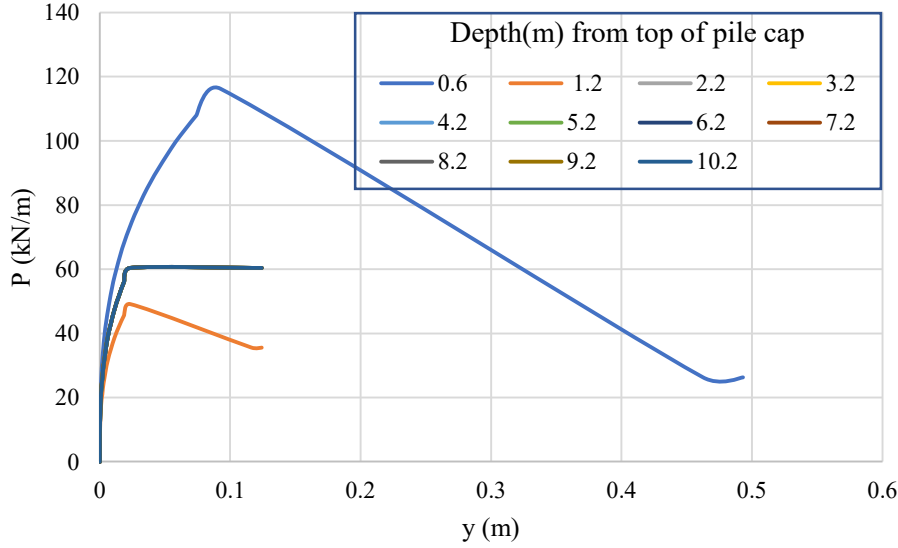


Figure 3-11 - p-y curves for soft clay and lateral loading at several points along the pile length (LPile)

3.4.3.2. Stiff Clay above Water Table Subjected to Cyclic Load

A p-y curve for stiff clay subjected to cyclic loading without free water can be defined by the relationship:

$$y = y_{50} \left[2 \left(\frac{p}{p_u} \right) \right]^4 + y_{50} \left[9.6 \left(\frac{p}{p_u} \right)^4 \right] \log |N| \quad (3-4)$$

Parameters in the equation are defined as follows:

N : is the number of cycles for cyclic loading.

ε_{50} from Table 3-3 is equal 0.005.

Here p_u occurs at a y value equal to $16y_{50}$. It is understood that the additional number of cycles extends the curve towards higher lateral movement as depicted in Figure 3-12, however lower soil resistance is observed at any given displacement value.

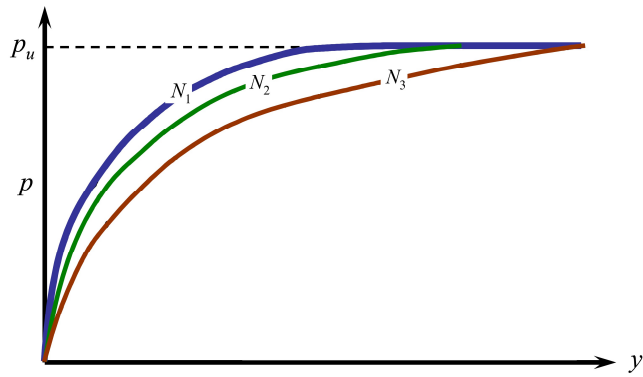


Figure 3-12 - p-y curve for stiff clay under cyclic loading (Isenhower and Wang 2016)

According to the theoretical background of program LPile, p-y curves were generated for different depths under the pilecap as shown in Figure 3-13. The points are defined at the mid height of the pilecap, at 0.6m below, and after that, one meter apart along the pile length. It was observed that as in the case of soft clay, soil response at the elevation of the pilecap shows the highest value, which is again related to larger diameter of the pilecap. On the other hand, the soil response remains almost unchanged to lateral displacement along the pile length except at 1.2m below the pilecap top, which can be interpreted as a stress concentration point, whereby loosening of the soil could result in reduction of unit length resistance of soil to lateral movement.

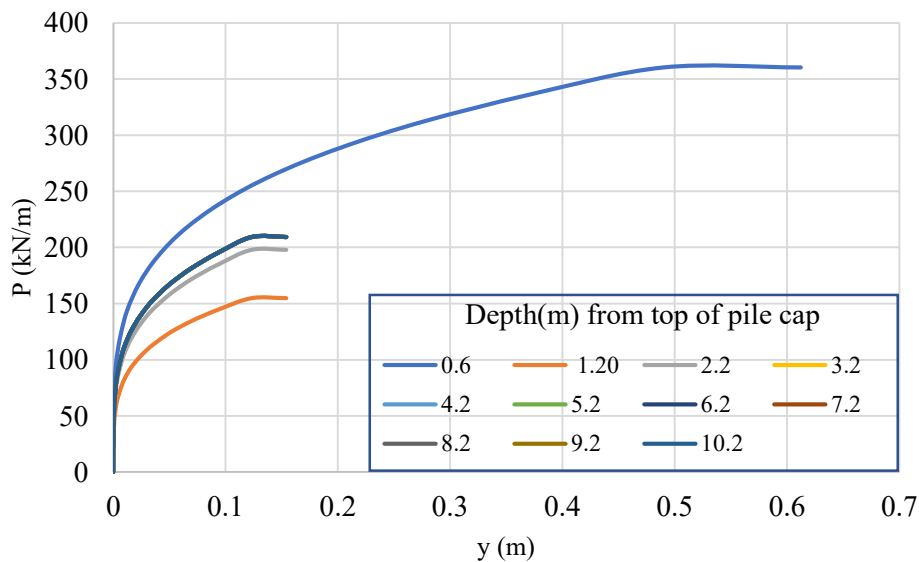


Figure 3-13 - p-y curves for stiff clay and cyclic loading at several point along pile skin (LPile)

For the depth 3.2m and greater the curves indicate same p and y values for the pushover analysis, as can be seen in Figure 3-13. It is also seen that for the depths between 2.2m and 10.2 the soil response to the lateral

displacement is almost the same (corresponding p-y curves are all overlapping), which indicates that soil resistance against lateral deflection of embedded pile in stiff clay is not a function of depth.

3.4.3.3. Sand above water level subjected to cyclic load

P-y curves can be estimated from several different empirical equations. Many of them are supported by field experiments. However, there is limited information about the response of sandy soils under cyclic lateral loading. Based on limited knowledge, empirical equations are developed to reflect the relationship between foundation movement and soil mobilized pressure in this context (Reese 1997).

In this work the approach developed by Reese et al. (1974) is adopted and solved using a Finite Differences Approach with program LPile. The assumption in this method is that the ground surface is horizontal, and piles are not battered. Equations are defined and used for establishing target p-y curves at points of interest along pile skin. Parameters angle of internal friction (ϕ), unit weight of soil (γ), and coefficient of lateral earth pressure at rest (K_0) are relevant parameters to establish p-y curves. Program LPile assumes the conservative value of 0.4 for K_0 , i.e., the coefficient of lateral earth pressure at rest.

$$\alpha = 45^\circ + \frac{\phi}{2} \quad (3-5)$$

$$K_a = \tan^2(45^\circ - \frac{\phi}{2}) \quad (3-6)$$

$$p_s = \min \begin{cases} p_{st} = \gamma x \left[\frac{K_0 x \tan \phi \sin \beta}{\tan(\beta - \phi) \cos \alpha} + \frac{\tan \beta}{\tan(\beta - \phi)} (b + x \tan \beta \tan \alpha) \right. \\ \quad \left. + K_0 x \tan \beta (\tan \phi \sin \beta - \tan \alpha) - K_a b \right] \\ p_{sd} = K_a b \gamma x (\tan^8 \beta - 1) + K_0 b \gamma x \tan \phi \tan^4 \beta \end{cases} \quad (3-7)$$

$$y_u = \frac{3b}{80} \quad (3-8)$$

$$p_u = \overline{A_c} p_s \quad (3-9)$$

whereby $\overline{A_c}$ for cyclic load is function of pile depth and diameter and is estimated from relationship from Isenhower and Wang (2016) as shown in Figure 3-14:

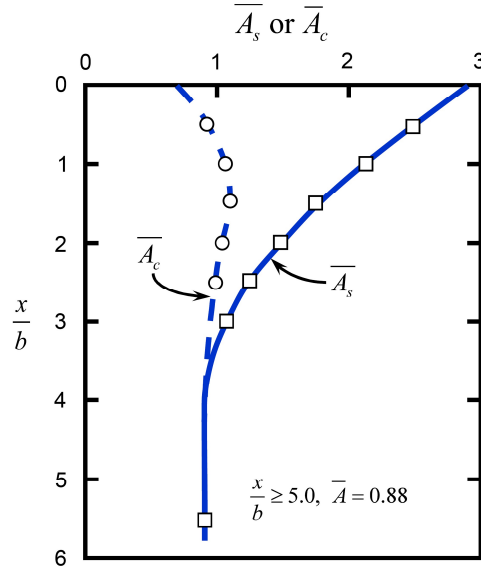


Figure 3-14 - Factor A_c for cyclic loading in sand in relation to depth and pile diameter (Isenhower and Wang 2016)

$$y_m = \frac{b}{60} \quad (3-10)$$

$$p_m = B_c p_c \quad (3-11)$$

Also B_c for cyclic load is function of pile depth and diameter and is estimated from relationship from Isenhower and Wang (2016) as shown in Figure 3-15. In order to estimate the p-y curve based on the above-mentioned parameters and equations, there are three regions to be considered as shown in Figure 3-16. The p-y relation is given by Equation (3-12), where the linear ascending portion from zero displacement to y_k is calculated from Equation (3-13):

$$p = (kx)y \quad (3-12)$$

The recommended values of k by Isenhower and Wang (2016) are presented Table 3-4, where k is a fixed empirical value giving the changing of soil modulus with depth (Isenhower and Wang 2016)

Table 3-4 - Recommended values of k for sand for cyclic loading (Isenhower and Wang 2016)

(k) MN/m ³	Relative Density		
	Loose	Medium	Dense
	6.8	24.4	61

The y coordinate of the first point of the nonlinear part is defined by Equation (3-13):

$$y_k = \left(\left(\frac{p_m}{y_m^{1/n}} \right) / kx \right)^{\frac{n}{n-1}} \quad (3-13)$$

in which, terms n and m are determined as follows:

$$m = \frac{p_u - p_m}{y_u - y_m} \quad (3-14)$$

$$n = \frac{p_m}{my_m} \quad (3-15)$$

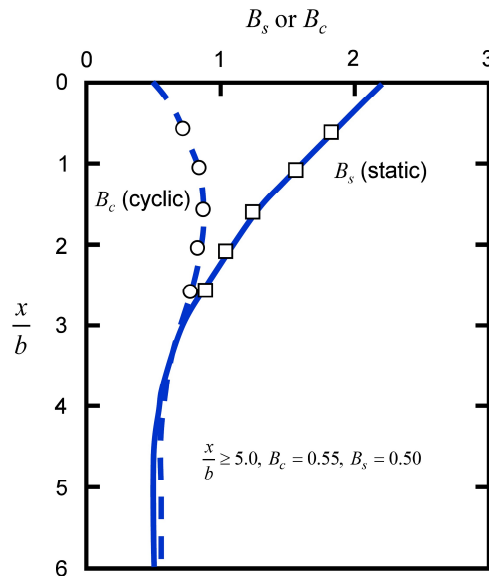


Figure 3-15 - Factor B_c for cyclic loading in sand in relation with depth and pile diameter (Isenhower and Wang 2016)

The relationship between p and y from point (y_k, p_k) to (y_m, p_m) is defined by Equation (3-16):

$$p = p_m \left(\frac{y}{y_m} \right)^{\frac{1}{n}} \quad (3-16)$$

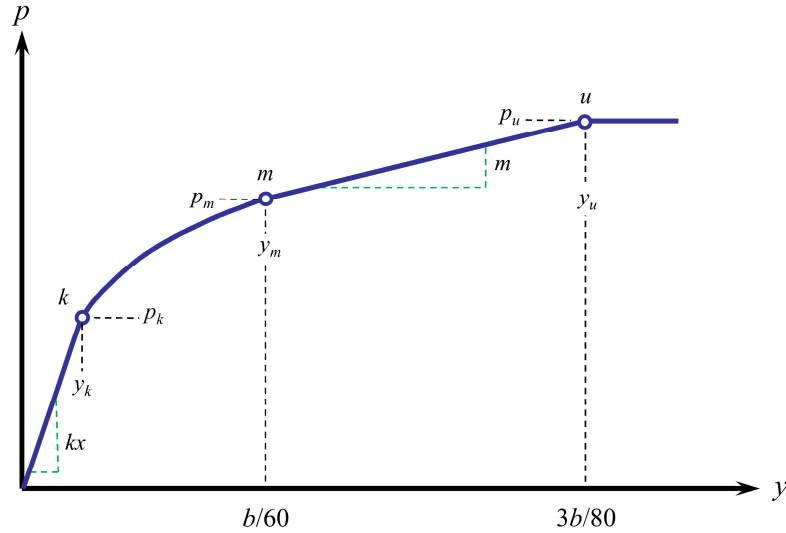


Figure 3-16 - Characteristic p-y curve for sand (Isenhower and Wang 2016)

Same as noted before, the p-y curves were generated for loose sand and dense sand as shown in Figure 3-17 and Figure 3-18 respectively; these curves are for 10 m HP 310×110 pile (loaded in direction of its strong axis) and a pilecap with 1.2 m thickness.

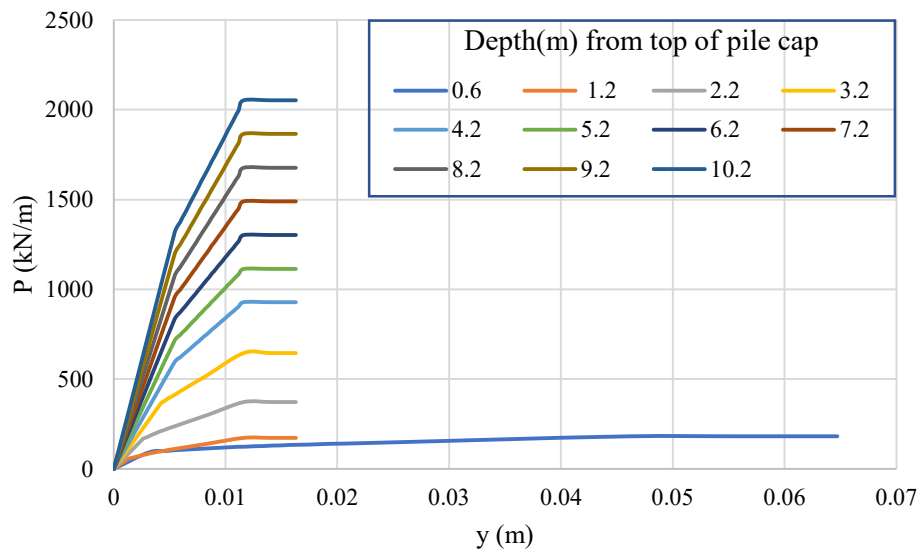


Figure 3-17 - p-y curves for loose sand under cyclic loading at several points along the pile length

From both Figure 3-17 for loose sand and Figure 3-18 for dense sand it may be noted that the soil resistance to lateral displacement increases with depth. This is also clearly stated in the equations used to model this part. For instance, in Equation (3-12), with increasing k from loose sand to the dense sand, soil reaction, p , grows. The main parameter for the nonlinear part of the p-y curve is the ratio of depth to pile diameter

which in turn defines the A_c and B_c factors. These factors in turn define the magnitude of the peak response p_k at the end of the linear portion of the curve as well as the maximum value p_m of the parabolic segment. Further they affect the slope of the line between y_m and y_u . Values of A_c and B_c are lowest close to the ground surface. From the p-y curves appears that the soil resistance close to the ground surface is significantly lower than the soil resistance against displacements along the pile in deeper elevations. The reason for this is the gapping that occurs in proximity of the pilecap (or the pile-head in the absence of a pilecap), close to the ground surface. Cohesionless soil fails and leaves a void when subjected to cyclic loading, therefore in this case a very loose soil and low soil response is expected (Isenhower and Wang 2016). This phenomena was not observed for the soft and stiff clay.

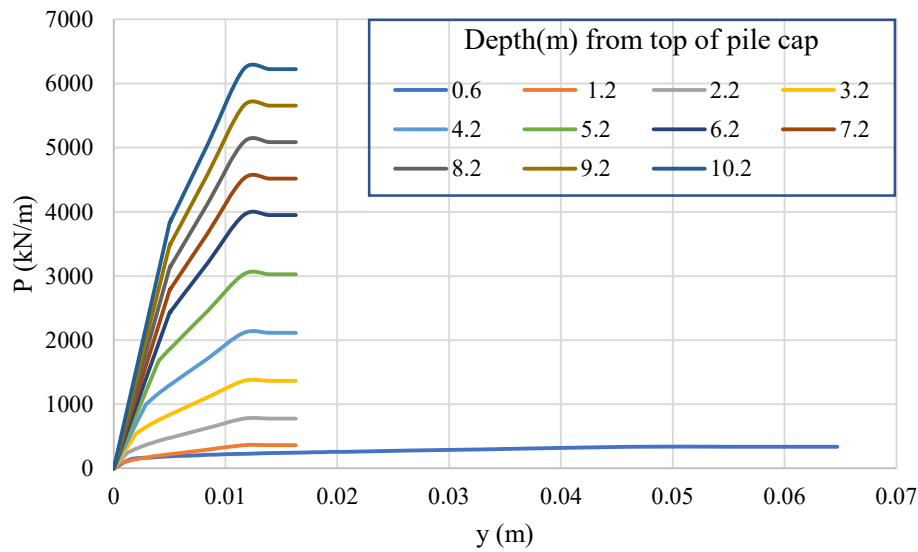


Figure 3-18 - p-y curves for dense sand under cyclic loading at several points along the pile skin (LPile)

3.4.4. Push-over analysis for a single pile and pilecap

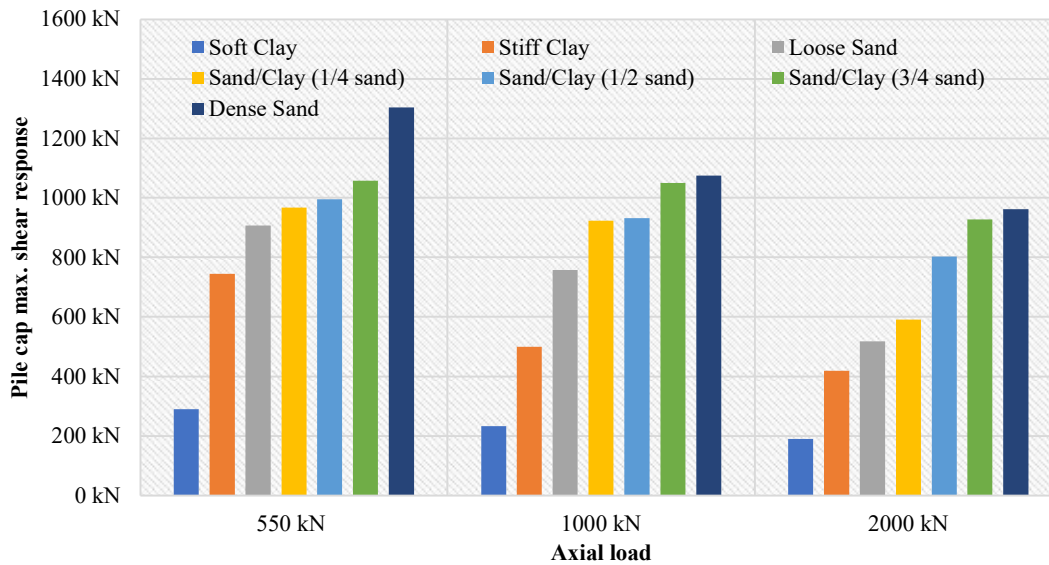
In this section results of several single pile and pilecap push-over analyses are discussed. In this context, the effect of three parameters, namely, the pile axial load, pile section and pile length, on the response of single pile and pilecap to the imposed cyclic lateral displacement as well as rotation are investigated. Meanwhile, sensitivity of the response to the embedment soil is examined as illustrated in Figure 3-6. As discussed in Chapter 2, the stiffness of substructure will be presented in form of a stiffness matrix (Isenhower and Wang 2016) where K_{11} is the shear stiffness versus pilecap lateral displacement, K_{12} is shear stiffness versus pile head rotation, and K_{21} and K_{22} are bending stiffness versus pilecap displacement and rotation, respectively.

$$[K] = \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix}, \quad [F] = [K] \cdot [X] \quad (3-17)$$

3.4.4.1. Effect of axial load on response of single pile & pilecap

The applied Axial Load at the centroid of the pilecap tributary area was set equal to magnitudes of 550, 1000 and 2000kN acting on the pile axis in three separate analyses. This force represents the overbearing forces transferred from superstructure to the foundation. In this section, the length of pile is fixed to 10m and pile section is HP 310×110 is considered. Pilecap is assumed to be a reinforced concrete plate, as described before, with 1.2m thickness, width of 1.233m and a length of 1.667m.

Push-over analyses was performed on a single pile, which is monolithically connected to the pilecap. First, the pilecap was displaced incrementally in the horizontal direction to the point that the program stopped converging as a result of zero effective stiffness. Meanwhile, seven different types of homogenous and heterogenous ground conditions were examined. As shown in Figure 3-19, with increasing axial load the maximum obtained lateral load resistance reduces for all soil types. This means that for the same soil type, higher Axial Load results in a lower resistance of the foundation to lateral displacement as shown in Figure 3-19-(a). It is also evident that soft clay shows lower lateral resistance as compared to stiff clay. This order is followed by loose sand, non-uniform soil 1, 2, 3 and dense sand. Results suggest that larger ratio of dense sand in ground conditions results in higher shear stiffness of the substructure (Hojjat et al. 2019).



(a)

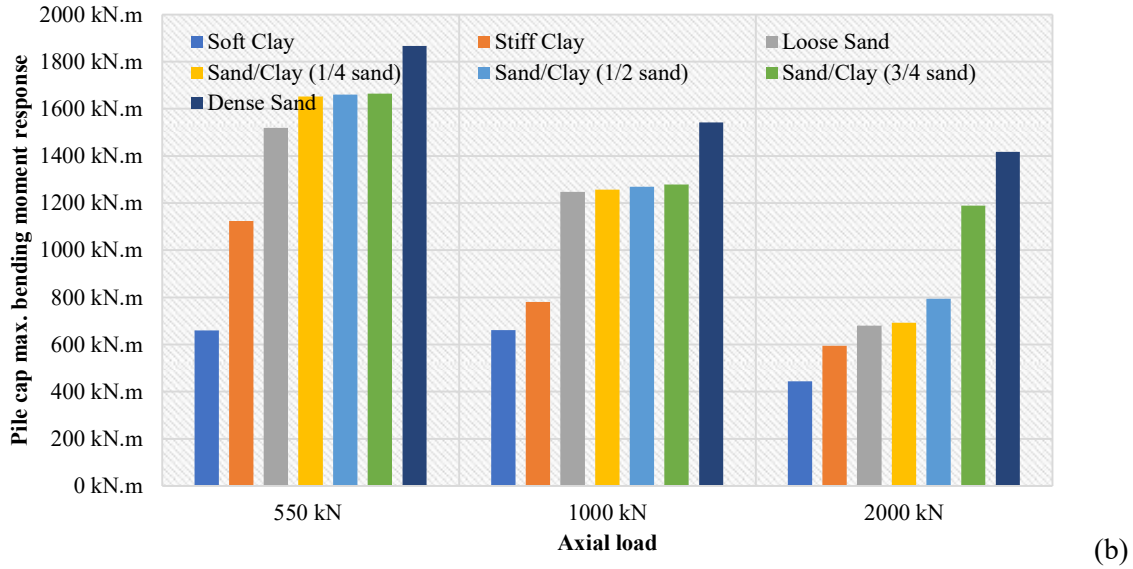


Figure 3-19 - (a) Peak lateral resistance (b) bending moment resistance to pile head displacement (under varying Axial Load)

Similarly, it may also be observed that the peak bending moment developed in the pile to pilecap connection under lateral displacement increases with varying type of soil from soft clay to dense sand as shown in Figure 3-19-(b).

It may be observed that with increasing axial load on the foundation, a lower maximum flexural resistance was obtained. This can be partially related to local buckling in the compression zone of the critical pile cross-section, which negatively impacts the plastic moment strength of pile as evident in the second term on the right hand side of Equation by (Horne 1978) :

$$M_{p,bukl.} = F_y \cdot Z - t_w a^2 f_y \quad (3-18)$$

where F_y is the yield stress, Z is the plastic modulus, t_w is the web and flange thickness of the beam and a is give by the following Equation (3-19), where P is the applied axial load on pile.

$$a = \frac{P}{2t_w f_y} \quad (3-19)$$

In Figure 3-20 maximum shear and bending moment resistance values for a single pile and pilecap to an applied rotation of increasing magnitude in seven soil arrangements are presented. From Figure 3-20-(a) it can be interpreted that shear stiffness in sand at the time of sway motion has the highest value. However, the shear resistance drops by fifty percent when the axial load increases from 1000kN to 2000kN. For other

soil compositions, the shear response decreases with increasing Axial Load value, but is less sensitive to the magnitude of axial load. Foundation in soft clay has the lowest shear resistance as compared to the others.

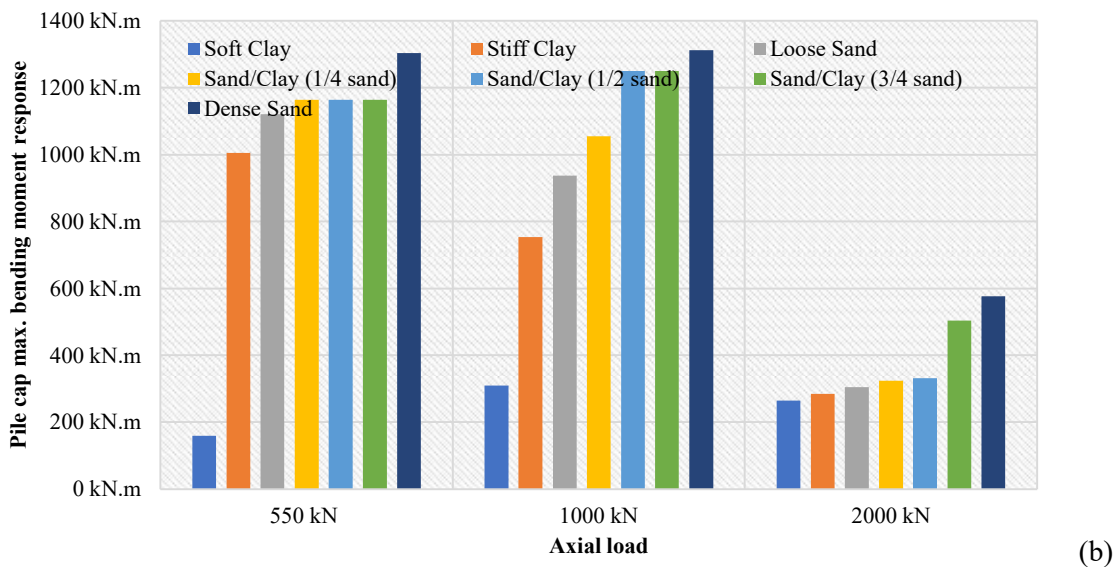
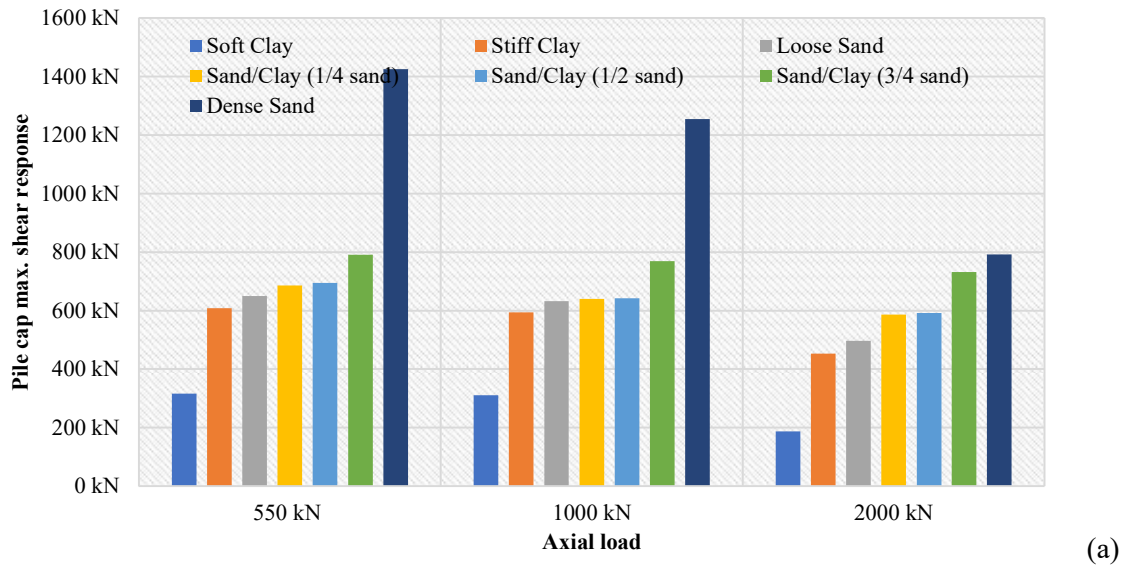
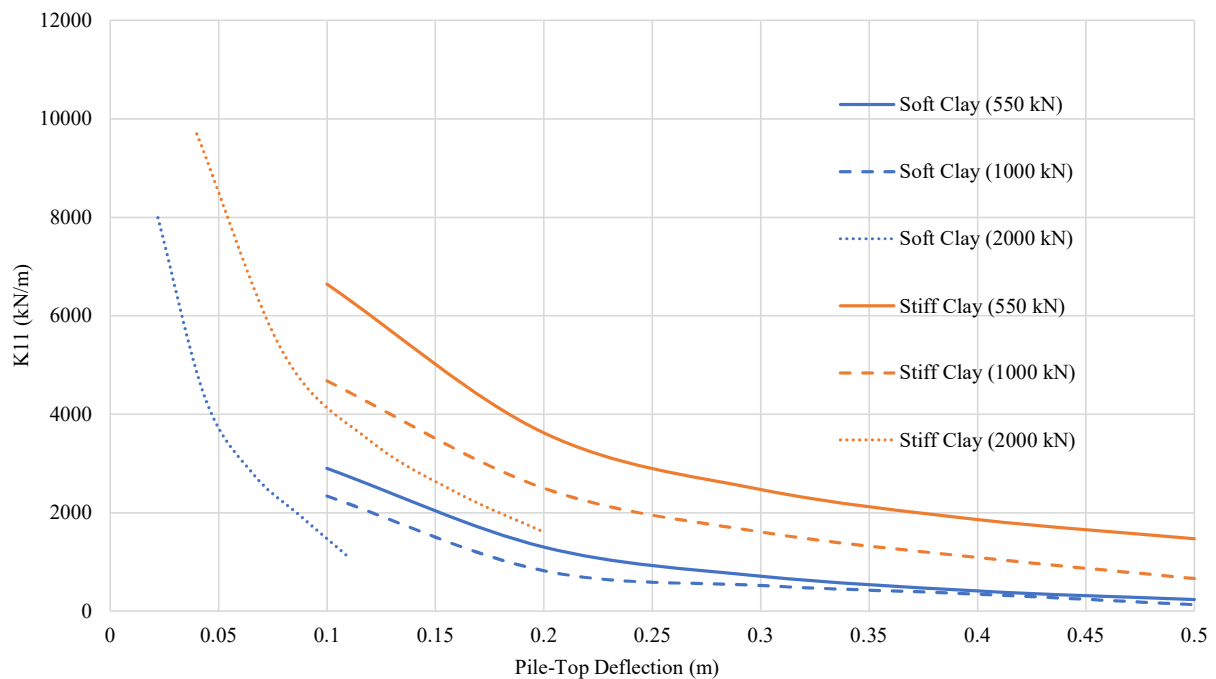


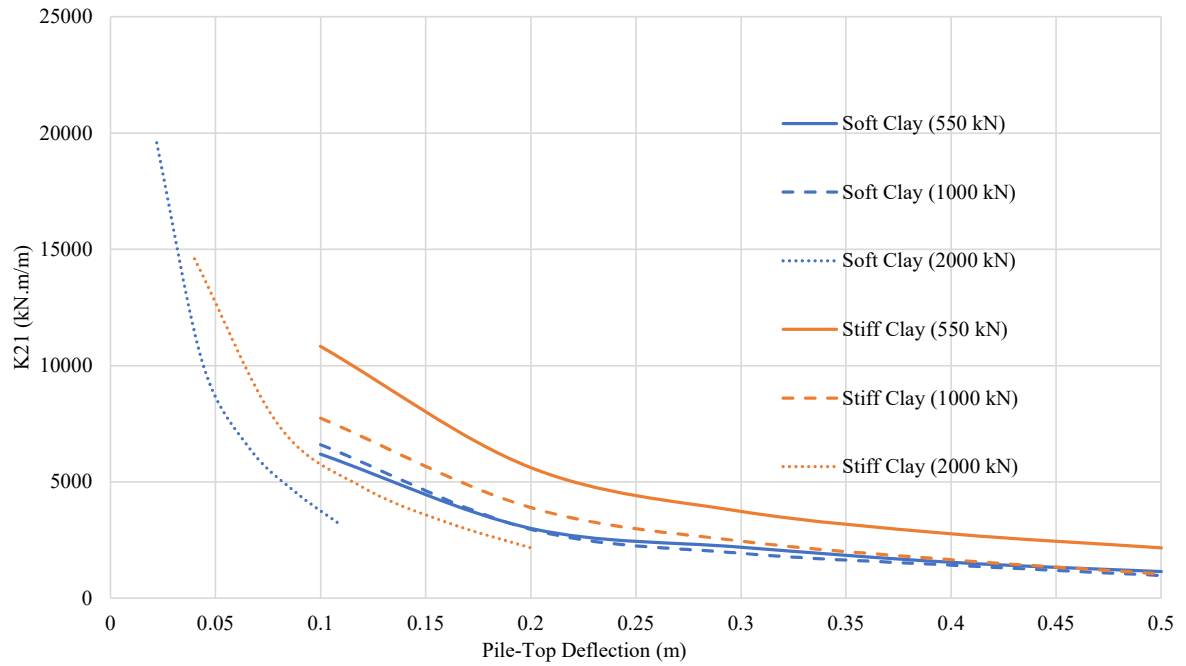
Figure 3-20 - Max. lateral force (a) and bending moment (b) response to pile head rotation (varying Axial Load)

In this research intent was to find a correlation between stiffness of a foundation subjected to cyclic loading and its performance under a diverse range of conditions such as varying axial load on pilecap. The applied displacements of the pilecap resulted in a nonlinear displacement in both the soil and the pile. It is common in practical analysis of the bridges to replace the substructure stiffness at the base of superstructure, with

linear springs, for instance at a bridge pier. Clearly, this assumption is overly simplified. Push-over analyses were carried out to plot the force-displacement curves. For the substructure, stiffness was defined as the ratio of force to sustained displacement at each increment of the pilecap deflection. The variation of stiffness with increasing applied displacement is studied, for embedded pile and pilecap in Figure 3-21. Plot (a) presents the effective shear stiffness values of the substructure versus lateral displacement of the pilecap. Increasing the pile axial load reduced the stiffness for embedded pile both in soft and stiff clay. The shear stiffness in stiff clay was obviously higher than in the soft clay. For the highest axial load of 2000 kN the LPILE program did not converge for displacements larger than 0.12 m in soft clay and 0.2 m in stiff clay. In Figure 3-21-(b) the bending stiffness of the substructure due to lateral displacement is shown. Note that in stiff clay, a higher stiffness is reached but under axial load of 550kN the entire arrangement in soft clay is stiffer than the embedment in stiff clay under 2000 kN. This means that the effective stiffness is not always a function of soil type but also of external variables.

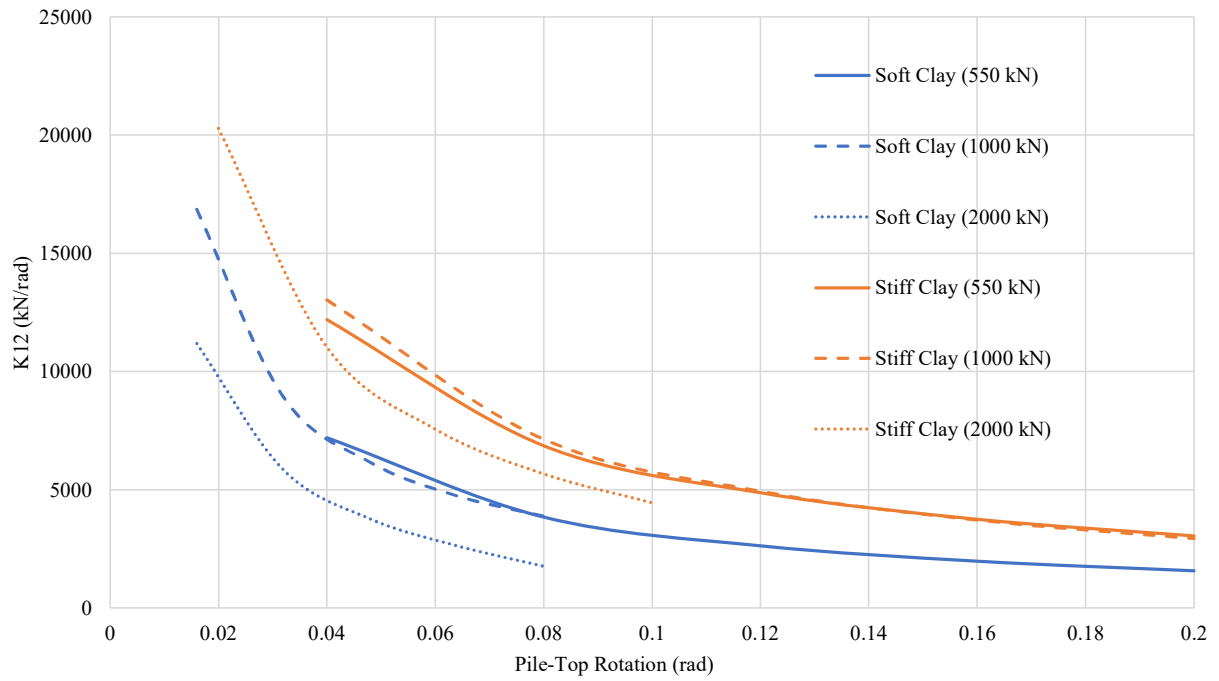


(a)

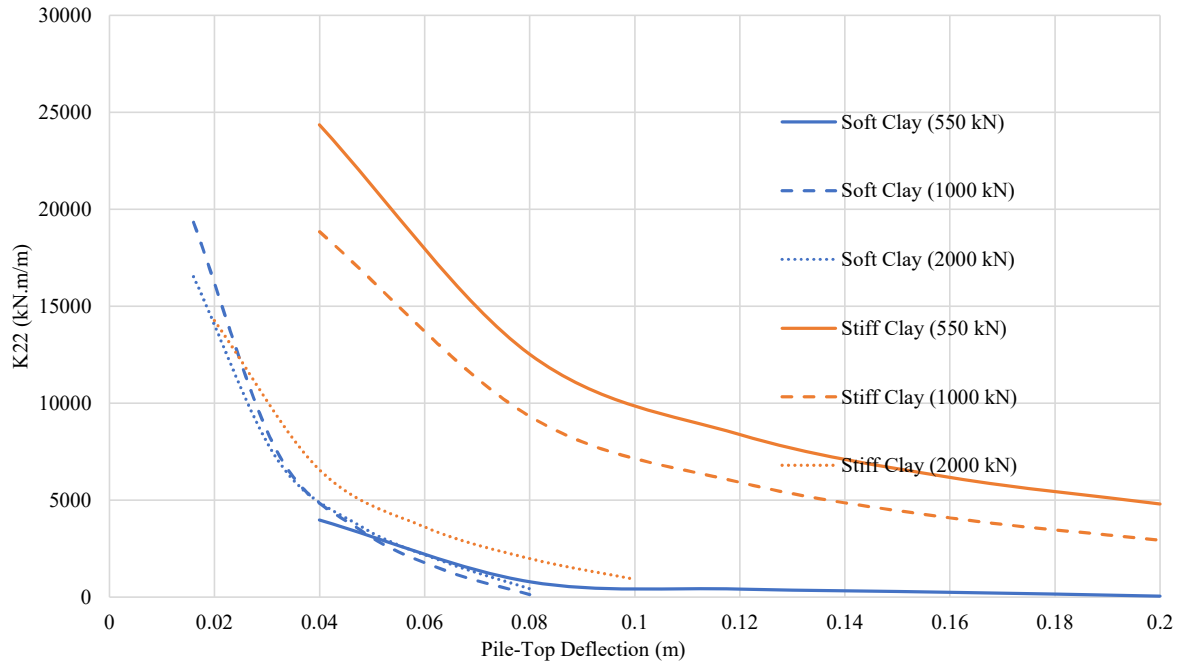


(b)

Figure 3-21 - K_{11} Displacement (top) and K_{21} Displacement (bottom) for clay – varying axial load



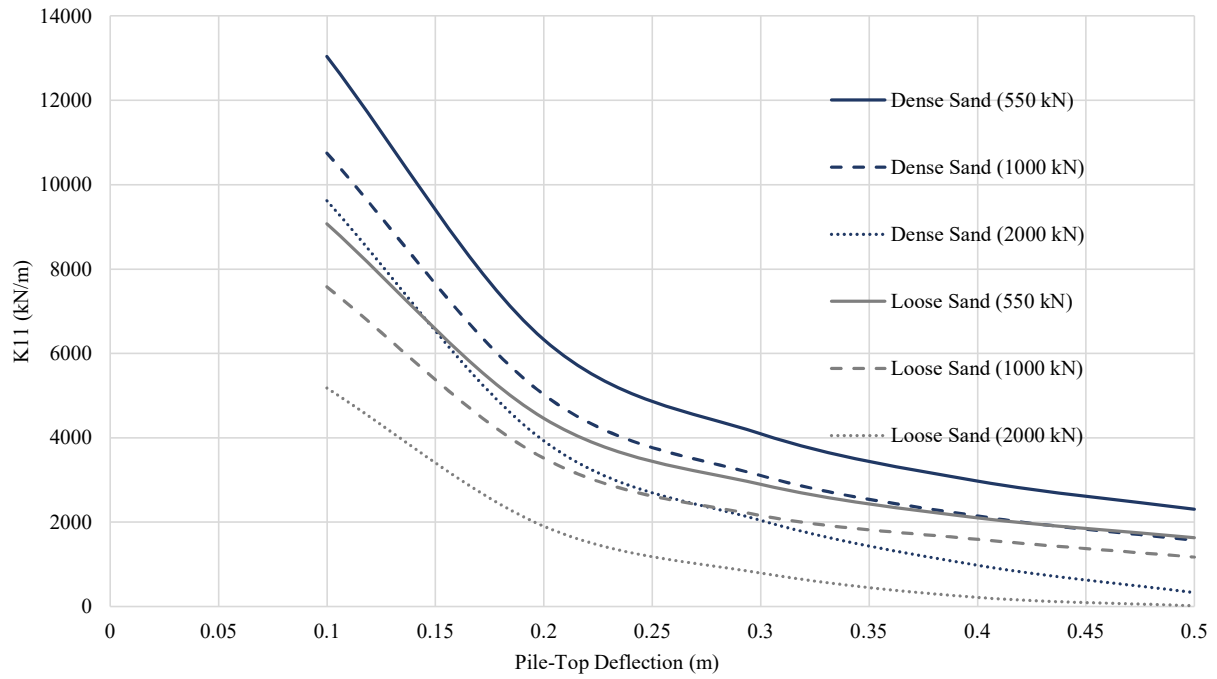
(a)



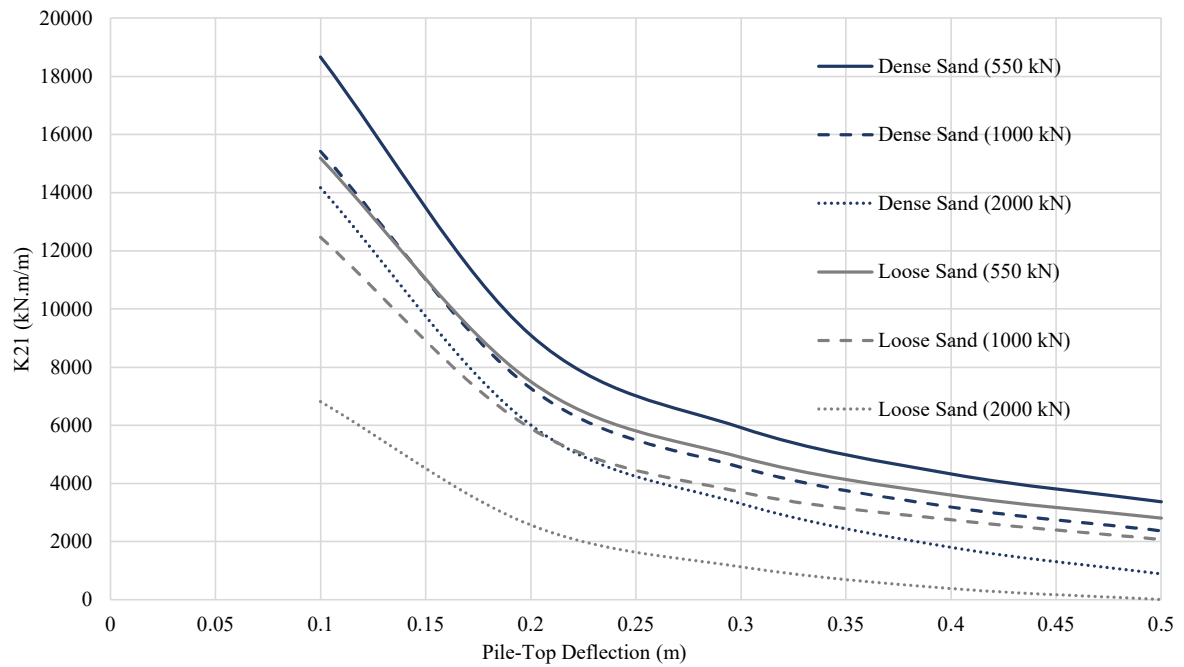
(b)

Figure 3-22 - K12 Rotation (top) and K22 Rotation (bottom) for clay – for varying axial load

Figure 3-22-(a) confirms that higher shear stiffness in stiff clay due pilecap rotation. It also shows that increase of lateral load up to a certain level (here 1000 kN) has no effect on lateral stiffness. In Figure 3-22-(b) it can be observed that bending stiffness of soft clay against rotation larger than 0.08 rad is negligible. The same is valid for embedded foundation in stiff clay under a 2000kN Axial Load.



(a)

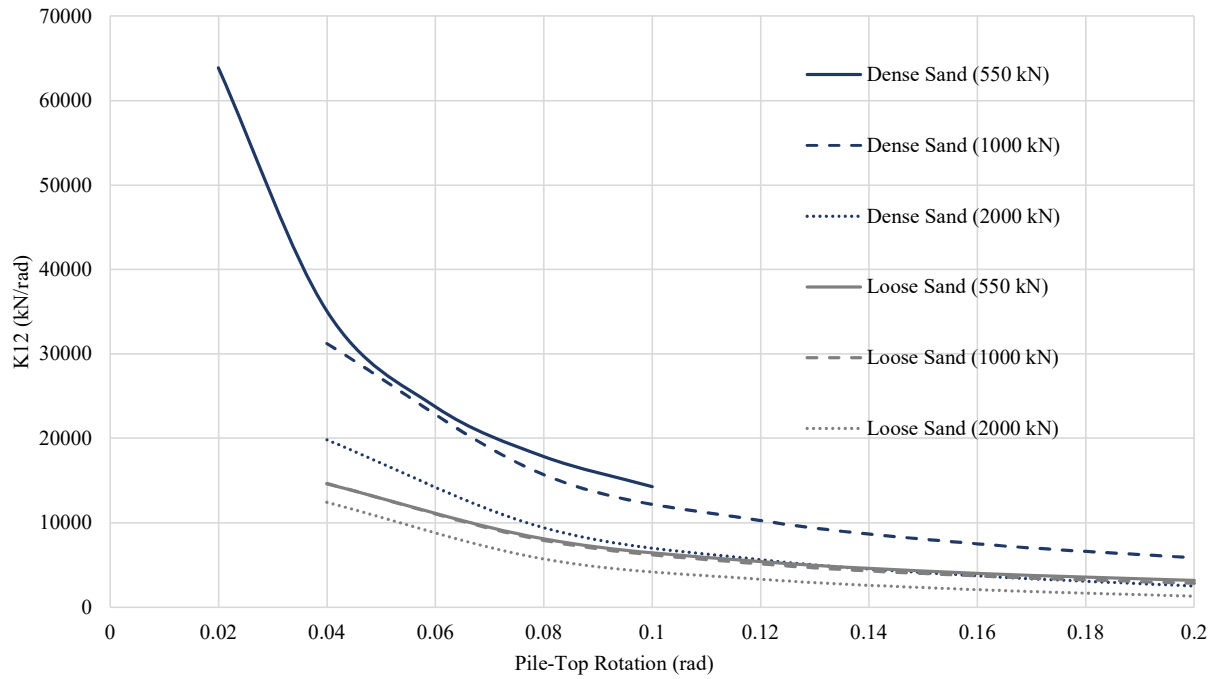


(b)

Figure 3-23 - K11 Displacement (top) and K21 Displacement (bottom) for sand – varying axial load

Figure 3-23-(a) and (b) plots the shear and bending stiffness of embedded foundation in sand versus pilecap lateral displacement. It is seen that increasing the axial load magnitude negatively affects the lateral and

rotational stiffness regardless of soil type. Although the pilecap carries higher shear and bending forces in dense sand under 1000kN Axial Load, its performance is pretty similar to that of loose sand under lower applied axial force (500kN). K_{11} and K_{21} stiffness terms in loose sand that sustained 2000kN is the least among other scenarios and tend to zero for displacements greater than 450 mm.



(a)

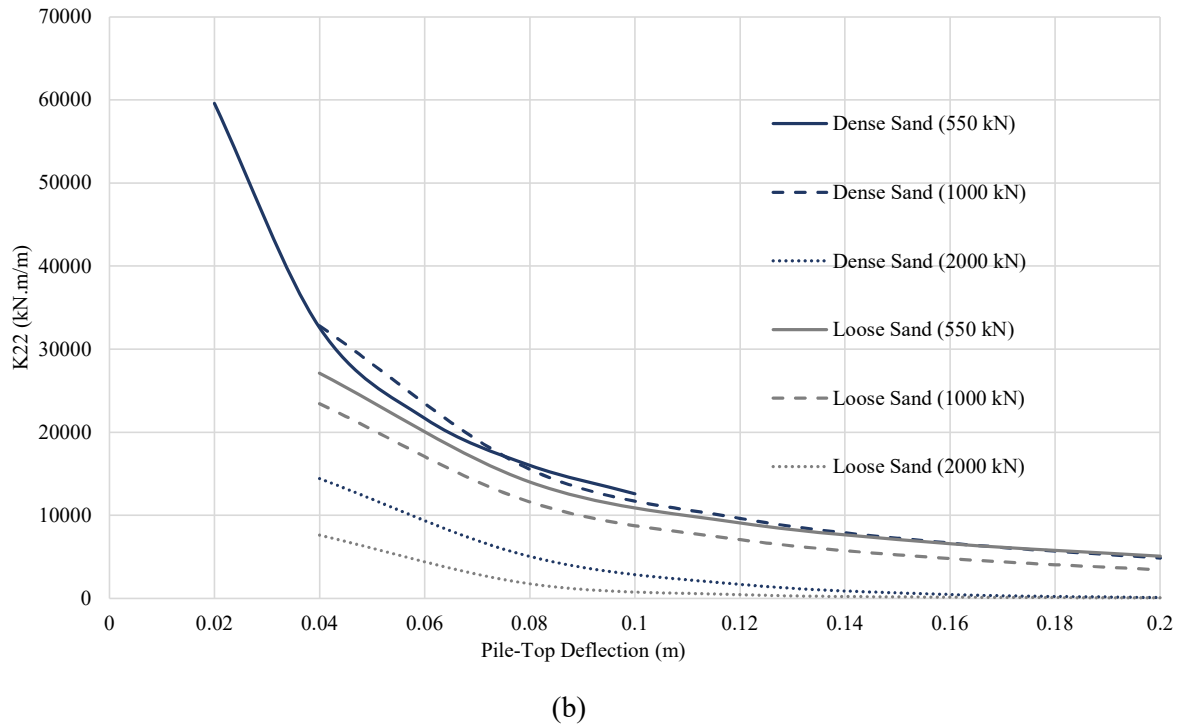


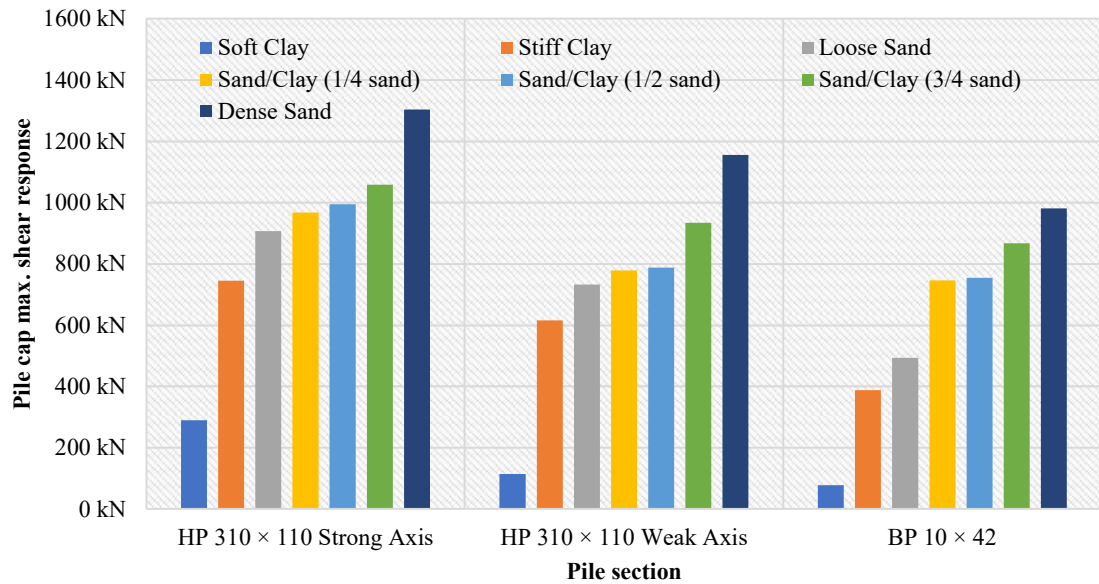
Figure 3-24 - K12 Rotation (top) and K22 Rotation (bottom) for sand – varying axial load

Two other components of the stiffness matrix (see Equation (3-17)) for embedded pile and pilecap in sand are plotted in Figure 3-24-(a) and (b). Increasing the axial load has almost no effect on shear stiffness in loose sand due to pilecap rotation. Axial load larger than 1000 kN compromises the shear stiffness in dense sand reducing it by 50%, but lower sustained axial load has no effect on this value. Bending stiffness in both loose and dense sand due to pilecap rotation and under higher Axial Load (2000kN) dramatically drops, whereas for pilecap rotation values exceeding 0.12 rad, it tends to zero.

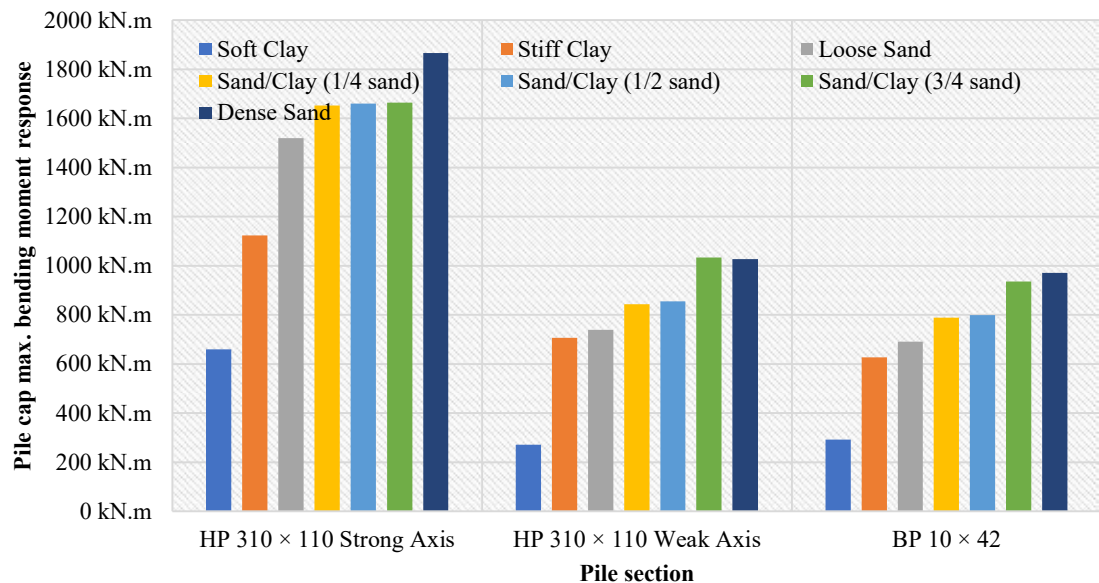
3.4.4.2. Effect of pile section on response of single pile & pilecap

In this section three different piles are simulated using the program LPile in the first trial a HP 310 × 110 pile is used. This pile was pushed over in the direction of its strong axis. In a second model the same section was used, but it was pushed towards its weak axis. For the third trial a BP10×42 pile (strong axis) was considered. The length of the pile in all cases was set to 10m. It was found that shear resistance of the pilecap to lateral movement increased from soft clay to dense sand. With a growing ratio of dense sand in the ground profiles the shear reaction increases for all three pile sections. The same trend was observed for the pilecap bending moment reaction to sway. The shear and bending moment reaction were considerably higher for the HP 310 x 110 pile section, pushed along its strong axis. Even the bending moment response of foundation was twice as large for this section as compared to the other sections. Despite the equivalence

in pile diameters, a significant difference was obtained between responses of the pushed over HP 310 x 110 pile along the strong and weak axis (Figure 3-25). This is related to the higher stiffness of the pile along its strong axis. Maximum shear and moment reaction of laterally pushed-over HP 310 x 110 along weak axis, and BP10×42 along strong axis have comparatively similar range of values.



(a)



(b)

Figure 3-25 - (a) Max. lateral force and (b) bending moment response to pile head displacement (for different pile sections)

Analysis of pile and pilecap bending moment responses to pile-head rotation illustrates that with increasing ratio of sand in the (sand/clay) soil mix (Figure 3-3) higher shear and moment response to pilecap rotation

is achieved. Highest bending moment reaction is observed for HP 310 × 110 (strong axis) in dense sand as shown in Figure 3-26. On the other hand, a pile embedded in soft clay is capable of sustaining very low bending moments.

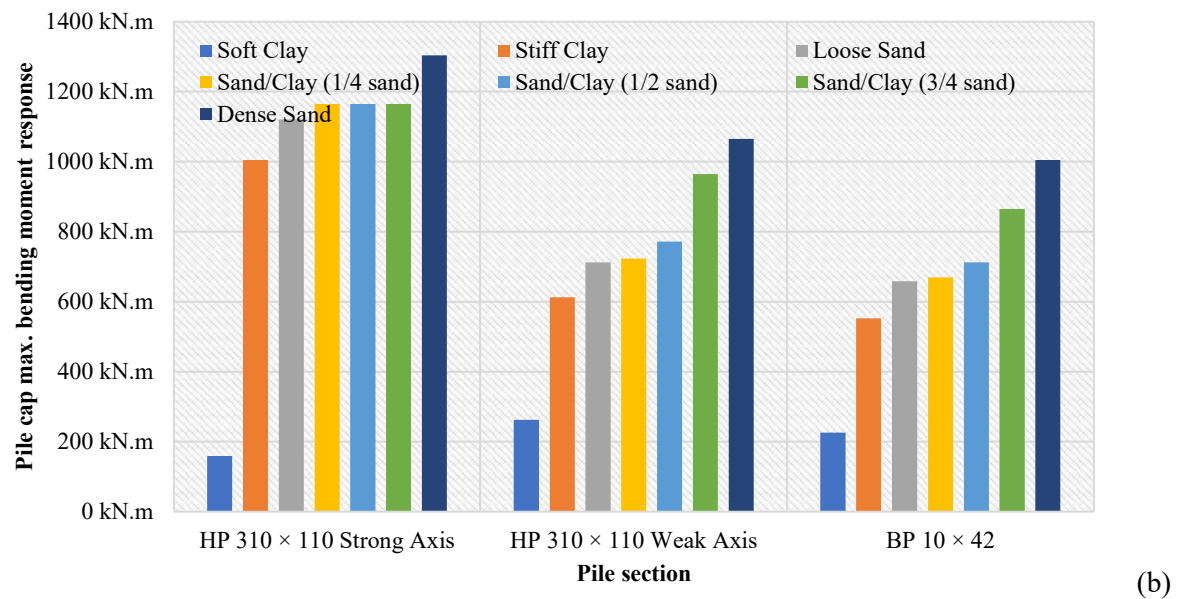
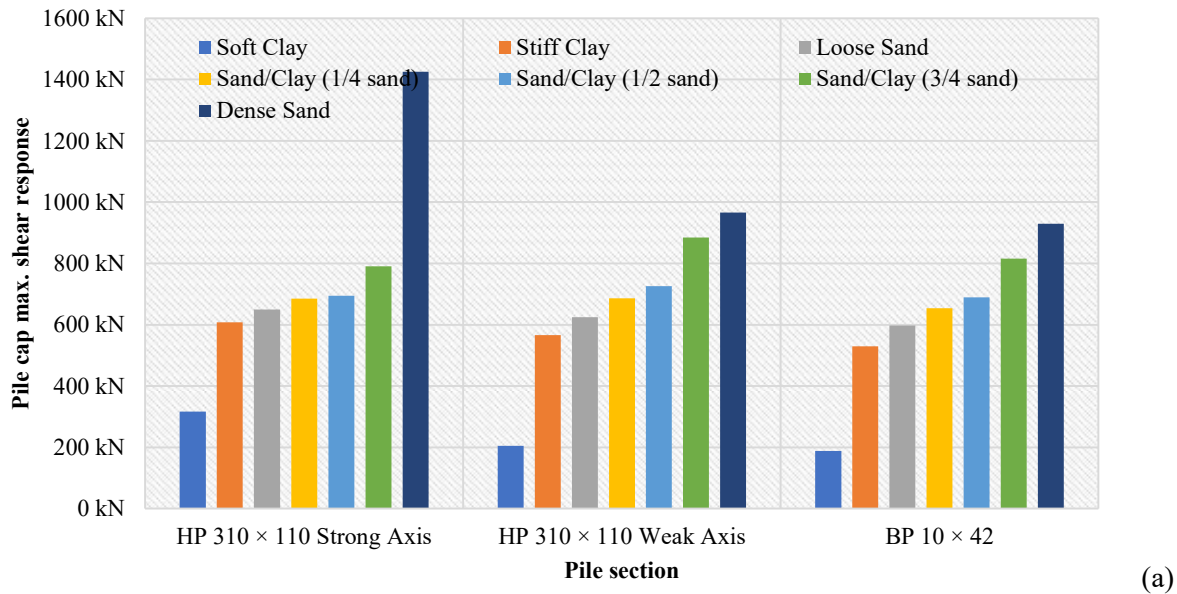
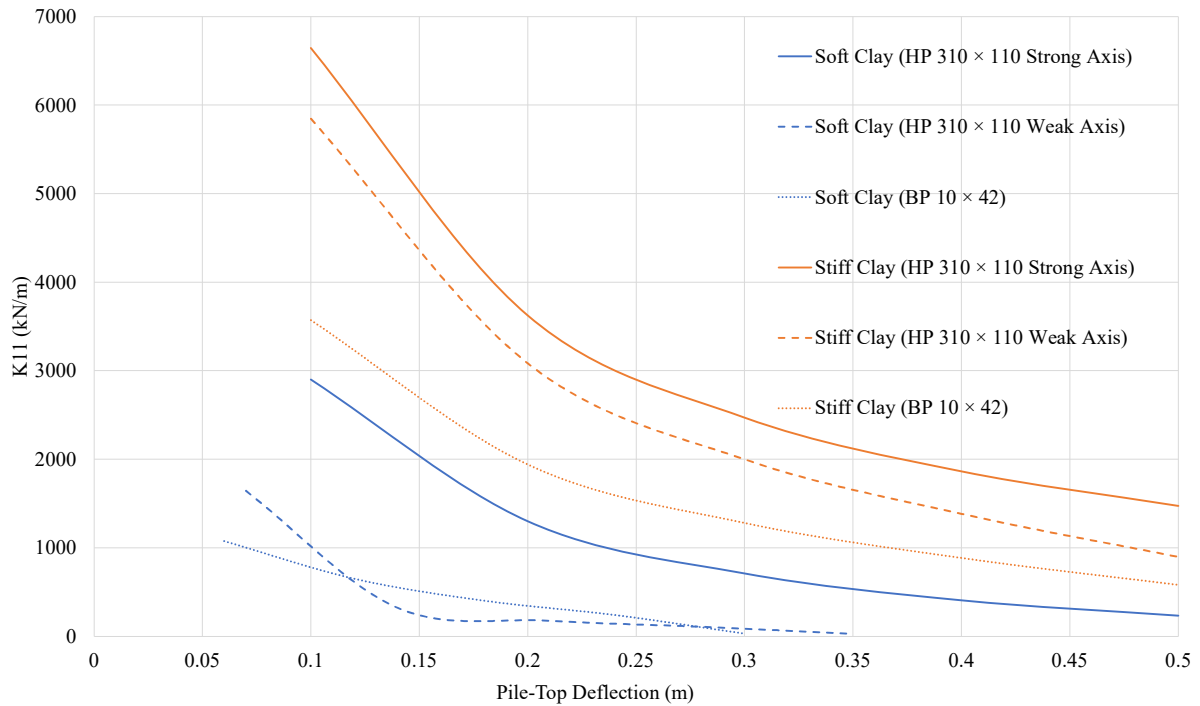


Figure 3-26 - (a) Max. lateral force and (b) bending moment response to pile head rotation (for different pile sections)

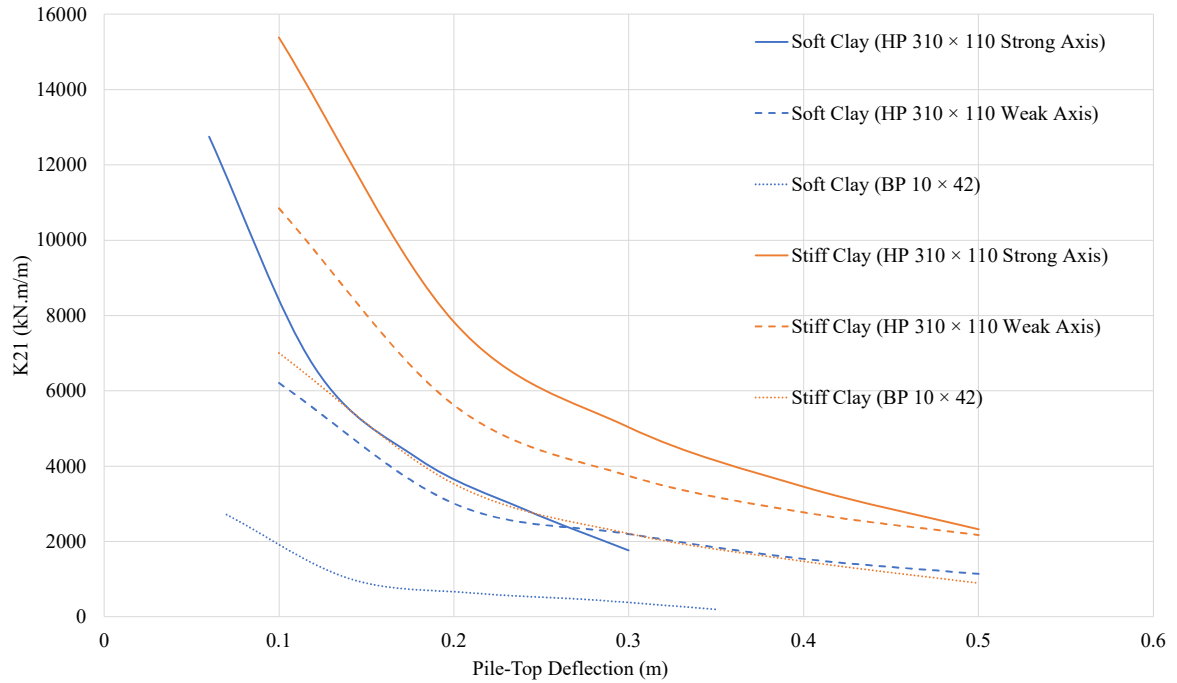
For the embedded HP 310 × 110 pile in all soil types except in dense sand, the maximum shear reaction is lower than nominal shear strength of pile (830.7 kN). It is concluded that the failure for those arrangements can occur due to soil failure prior to pile failure, while the pile maximum strength is not reached (Figure

3-26). On the other hand, with the exception of foundation in soft clay, the nominal flexural strength (439.1 kN) is exceeded.

In Figure 3-27-(a) and (b) shear and bending stiffness of foundation versus pilecap lateral movement is plotted. From the deformed shape of the pile and the fact that the pile tip rotation is not manually restricted in the program, it is concluded that the pile tip behaves as a pinned support. Accordingly, the elastic shear and bending stiffness of HP 310×110 piles along the strong axis were calculated as per Equation (1-2) to 658 kN/m and 3950 kN-m/m, respectively. For lateral displacements up to 0.40m, the stiffness of foundation is larger than the pile shear stiffness. This underlines the contribution of soil resistance to the overall performance of the substructure. However, the additional resistance by soil is larger for stiff clay compared to soft clay. It was observed that up to 150 mm pilecap displacement, for stiff clay, K_{2I} is controlled by the soil (bending stiffness of the substructure is larger than that of the pile) whereas for soft clay, this limit is 180 mm. After the maximum displacement capacity of the pile-soil arrangement, the soil does not provide anymore resistance and the pile gradually fails.



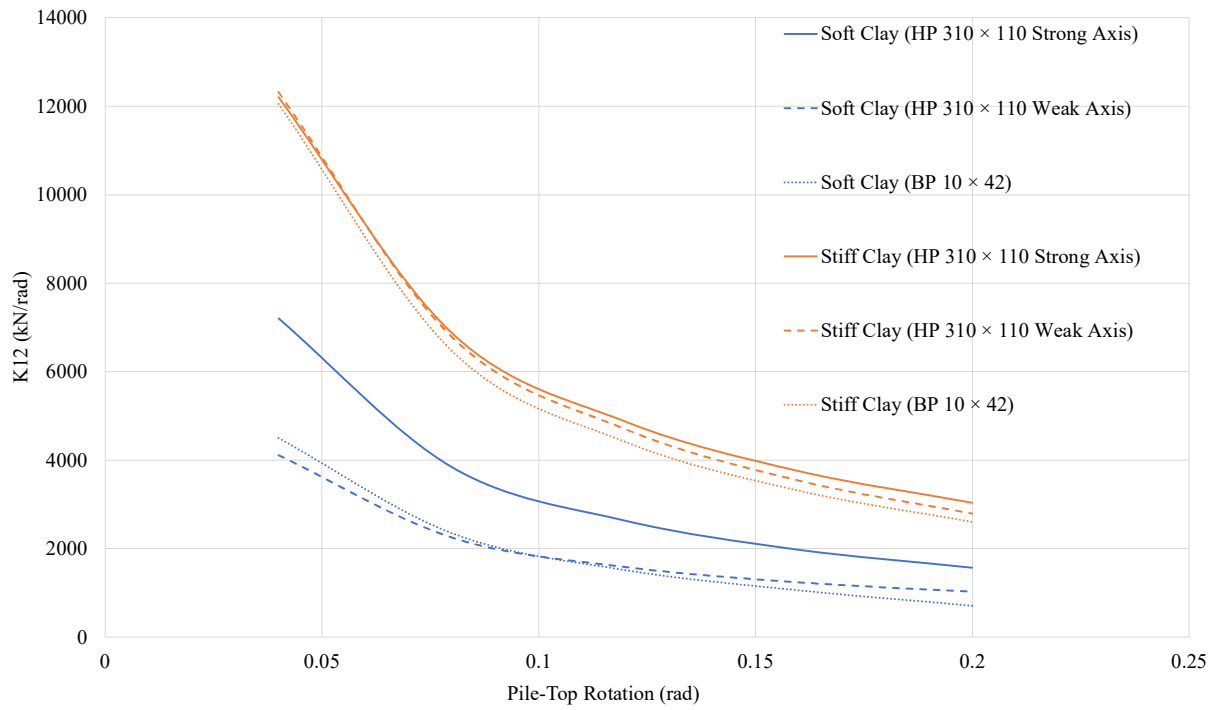
(a)



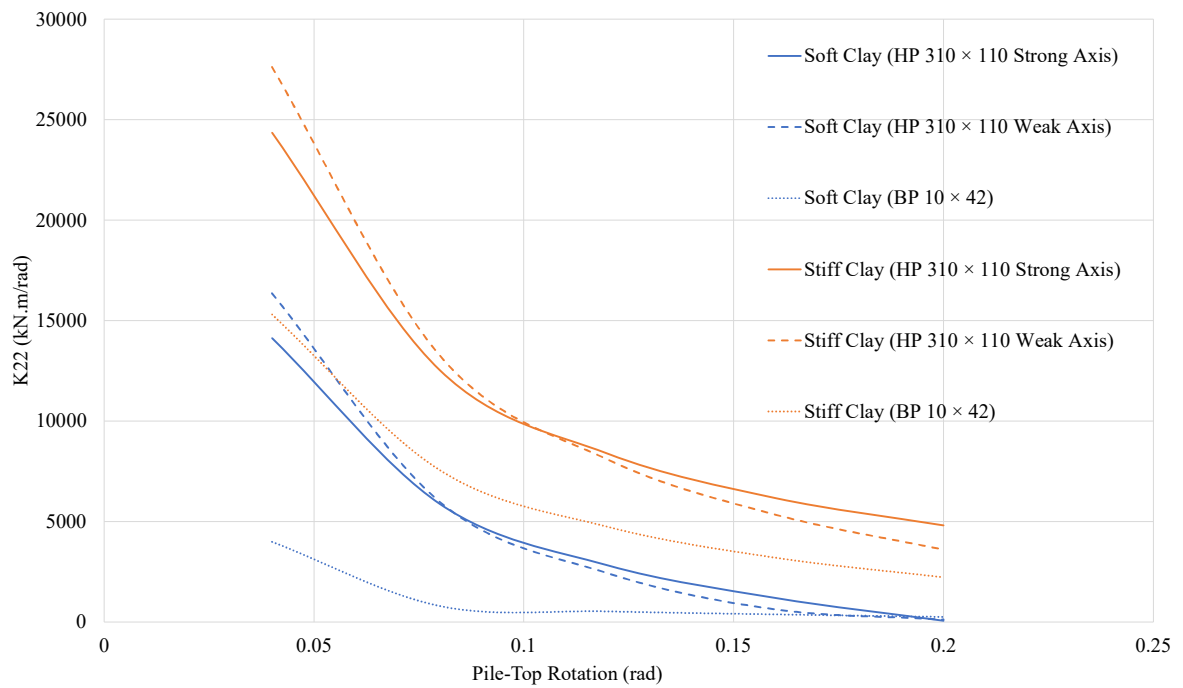
(b)

Figure 3-27 – Relationship between (a) K_{1I} and Displacement and (b) K_{2I} and Displacement for clay – (pile section is a variable)

Stiffness versus pilecap rotation is illustrated for clay in Figure 3-28. Stiff clay shows less sensitivity to direction of loading, in terms of shear stiffness. On the other hand, bending stiffness in soft clay tends to zero for rotation larger than 0.5 rad. Stiffness in stiff clay is almost everywhere larger than the value in soft clay.



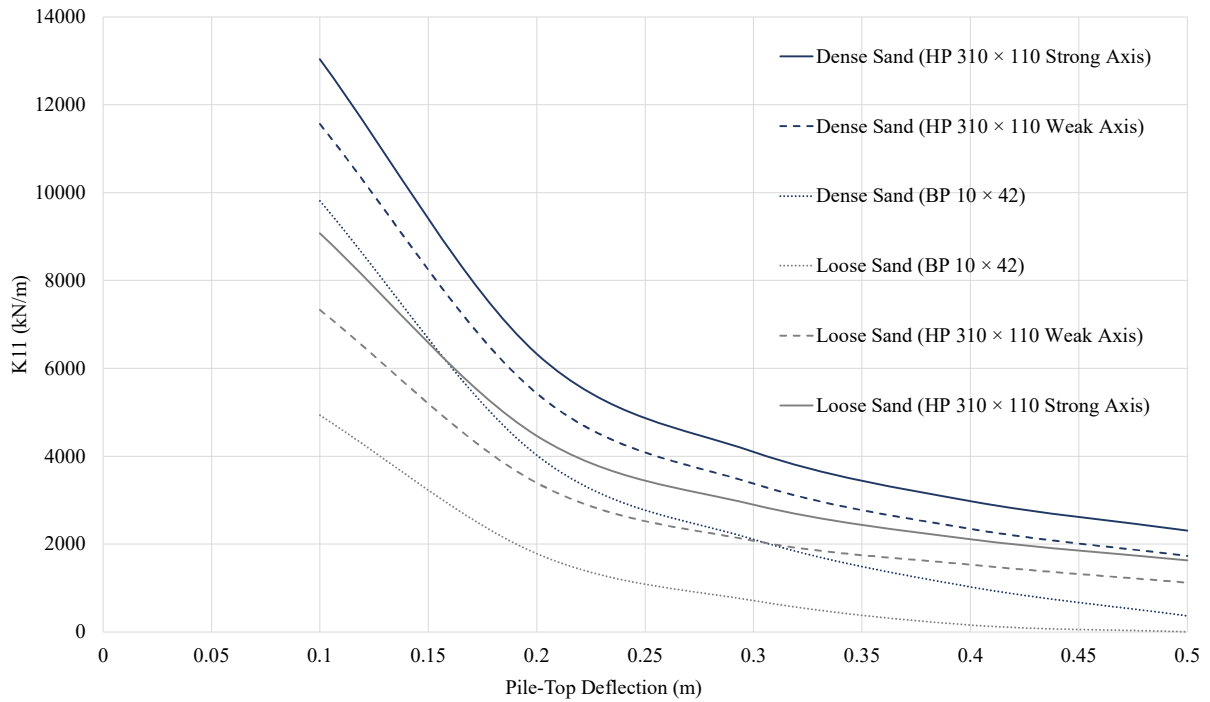
(a)



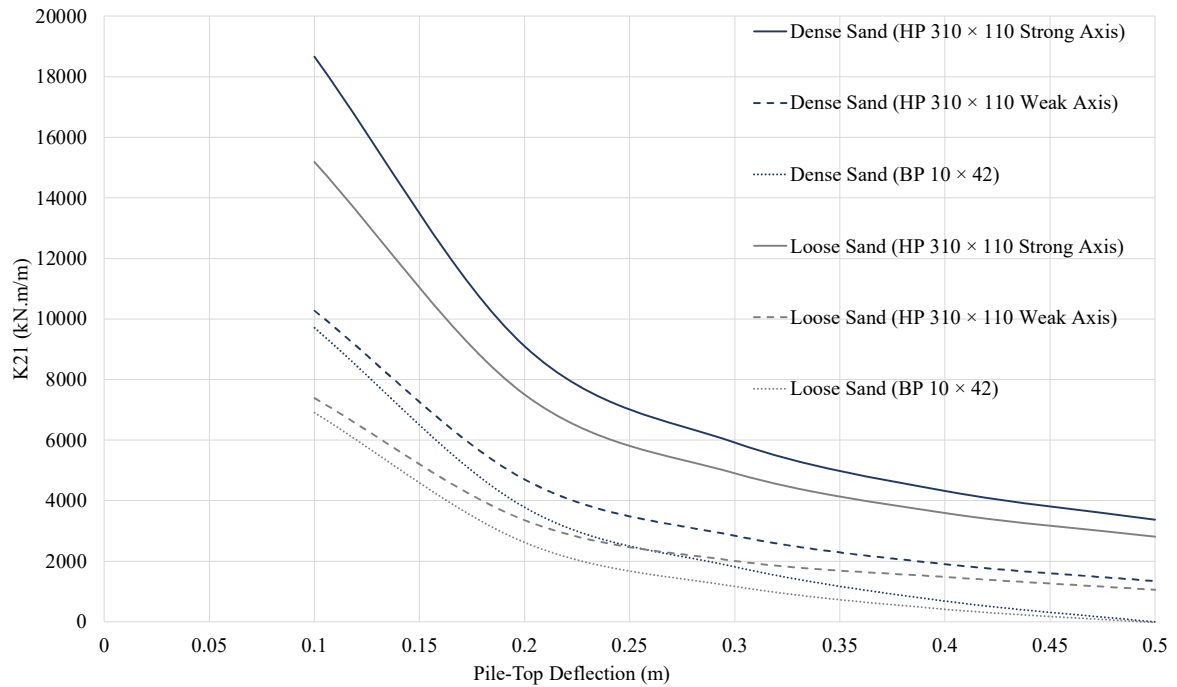
(b)

Figure 3-28 - K_{12} vs. Rotation (top) and K_{22} vs. Rotation (bottom) for clay – (pile section is a variable)

In dense sand, shear stiffness has the highest value for deflected HP 310×110 along the strong and weak axis. Stiffness in loose sand for BP10×42 due to pile-head lateral displacement is very small and for lateral displacement larger than 360 mm it tends to zero as shown in Figure 3-29. It is concluded that the shear stiffness of substructure due to lateral displacement depends on soil type rather than pile section. For bending stiffness, however, the HP 310×110 strong axis was highest in dense sand, followed by loose sand. For two other selected sections the same trend was observed. This shows that bending stiffness due to lateral displacement, primarily relies on pile stiffness.



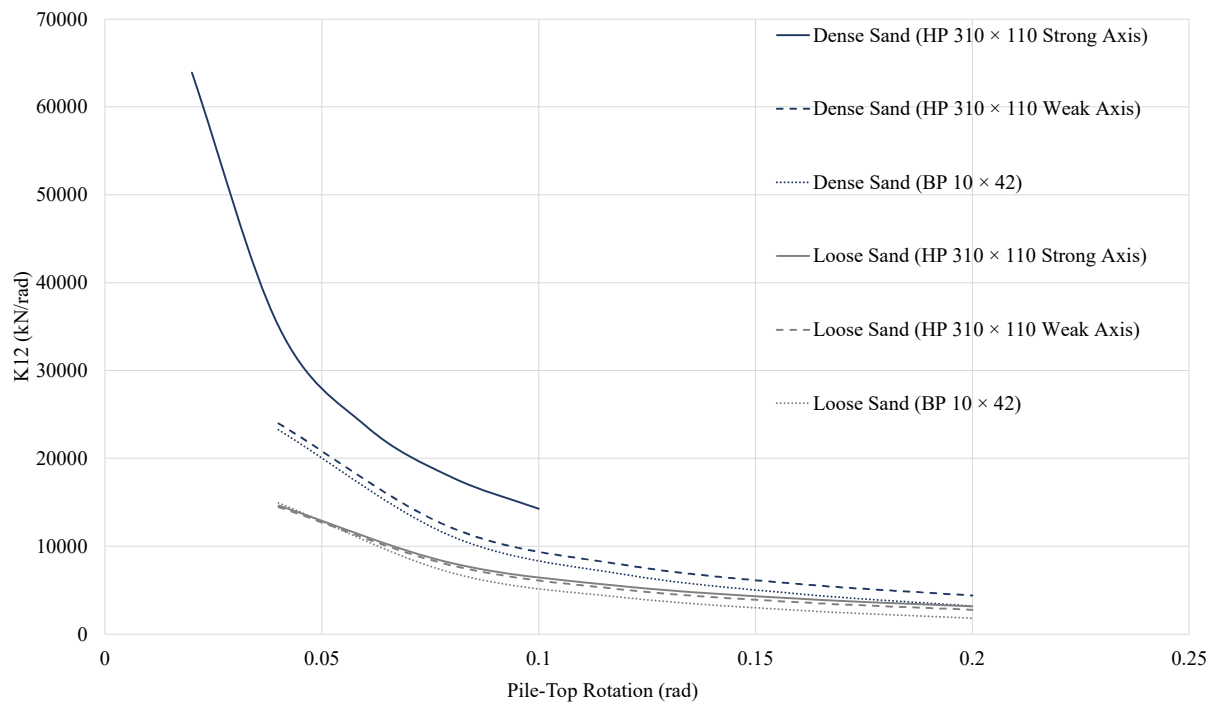
(a)



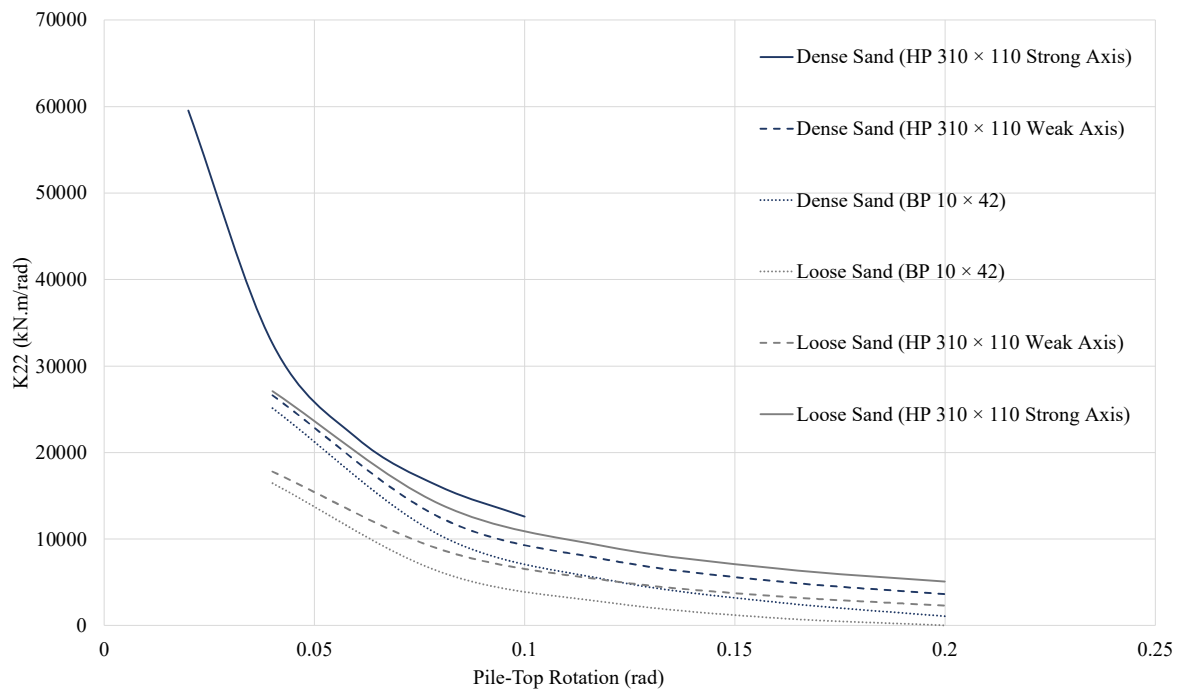
(b)

Figure 3-29 - (a) K/I vs. Pile-top displacement and (b) K_2/I vs. Pile-top displacement for sand – (pile section is a variable)

Shear stiffness due to pilecap rotation in dense sand for all sections is higher than in loose sand. Stiffness of the arrangement with HP 310×110 along the strong axis is evidently higher than in the others (Figure 3-30). For all other sections and sand type, stiffness does not considerably change. Bending stiffness of selected sections due to pilecap rotation (same as for lateral movement) shows dependency on pile section, so that stiffness for piles with higher moment capacity have higher bending stiffness against rotation.



(a)



(b)

Figure 3-30 - K_{12} vs. Rotation (top) and K_{22} vs. Rotation (bottom) for sand – varying pile section

3.4.4.3. Effect of pile length on response of single pile & pilecap

In this section results of push-over analyses, performed on a single pile and pilecap with different lengths is discussed. Three pile lengths of 4.2, 6 and 10m were selected. Axial load of 550kN was applied to the pilecap. The same pilecap dimensions as in the previous sections was used. Uniform soil profiles of soft and stiff clay, as well as loose and dense sand were considered for these analyses. It is illustrated in Figure 3-31 that maximum shear response to lateral displacement occurs in denser soils. As expected, with a larger pile length, the shear response increases for all selected pile lengths.

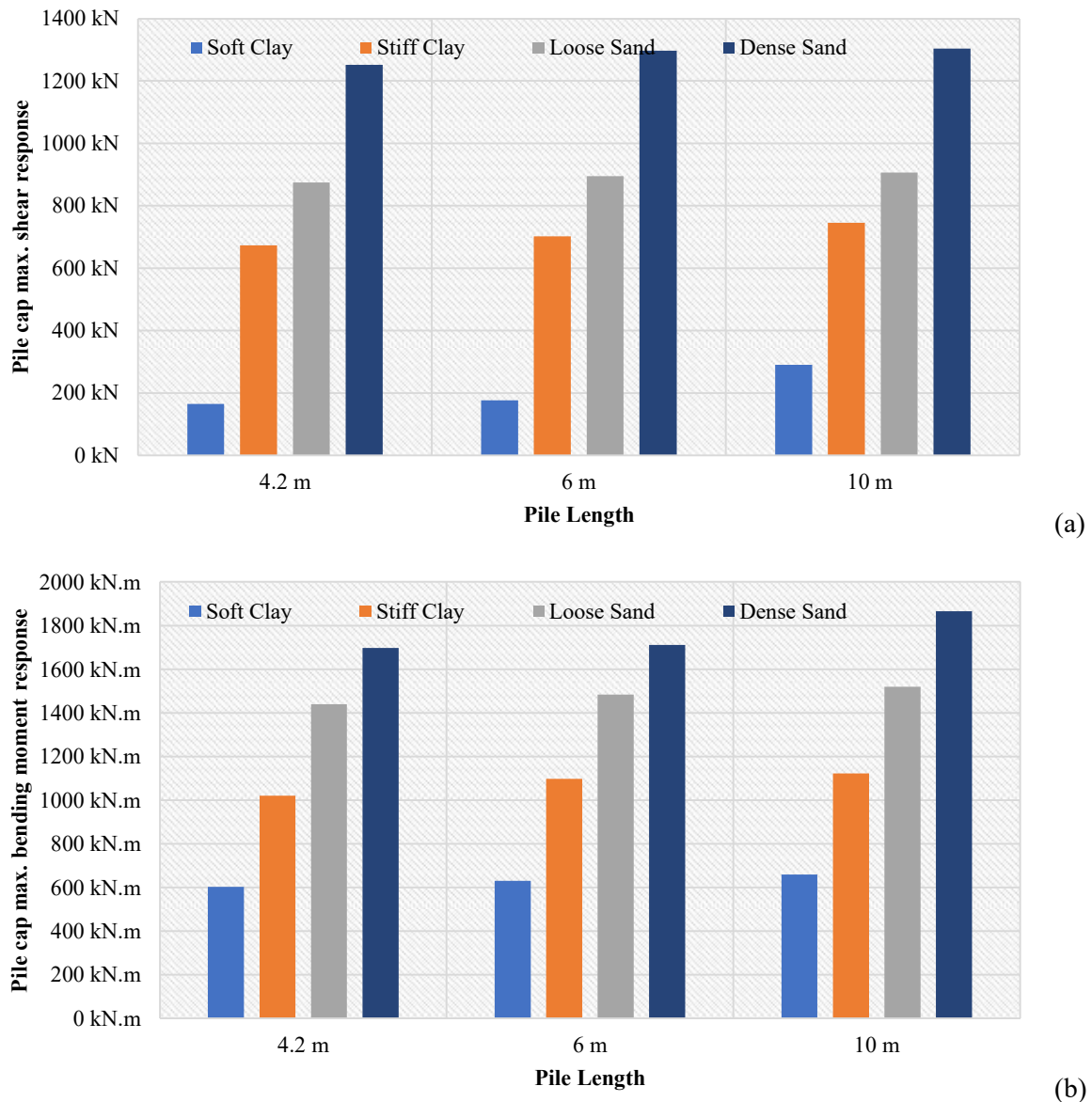


Figure 3-31 - (a) Peak Lateral resistance and (b) Peak bending resistance to pile head displacement (for different pile lengths)

From Figure 3-31- (b) it can be seen that the bending resistance of the pilecap increases from soft/loose to stiff/dense material. Also, piles with a deeper embedment length develop higher bending and shear resistance. Therefore, longer piles mobilize larger soil resistance. Considering a beam subjected to lateral displacement, shear and bending reaction at the deflecting end decreases with increasing length, according to Equations (1-1) and (1-2). However, the observed results indicate the contrary, which further emphasizes the significance of considering soil substructure interaction.

Observations on the pile deformed shape illustrates that rotation of pile tip is averted by soil resistance up to a certain pile head lateral displacement. It is assumed that pile tip is at a fixed support. Beyond this displacement limit, soil resistance against pile tip rotation is exceeded, and inevitably pile tip rotation occurs. The pilecap displacement, in cases where pile-tip rotation happens, varies for different pile lengths as depicted in Figure 3-32. The figure compares the deformed shape assumed by a short (in red) and a long (in gray) pile with increasing displacement of the pilehead. Near failure the soil clamping at the pile tip is gradually degraded and rotation of the tip occurs freely.

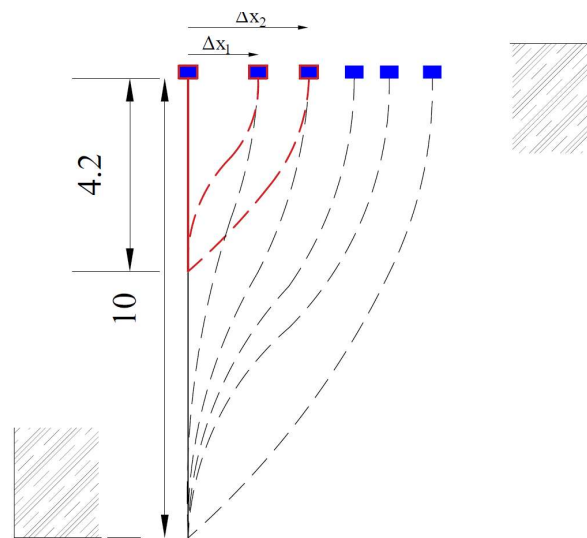


Figure 3-32 - Pile tip support type in accordance with pile length and pilecap displacement

For deeper embedded piles, larger displacement is sustained to the point that pile tip rotates. Assuming a linear elastic deformation for the pile, the following conclusions may be drawn. It appears from Figure 3-32 that at pilecap displacement of Δx_1 , the pile-head shear force reaction for 10m and 4.2m long pile may be calculated from Equation (1-1). In this case, shear force reaction of longer pile is $((4.2/10)^3 =) 0.075$ times that of the shorter pile. For displacement Δx_2 , top reaction of 10m pile is calculated from Equation (1-1), but in this case, for 4.2m long pile, the reaction force is computed by Equation (1-2). The ratio of the reaction forces for 10m pile to that of the 4.2m pile is $((12/4) * (4.2/10)^3 =) 0.022$. Further to the linear

elastic response of the pile to lateral displacement or pilecap rotation, soil resistance contribution and nonlinearity of pile and soil material should be considered in terms computing of pile head force reaction. Nonlinearity (plastic response past yielding) of H-pile and soil are accounted for in program LPile.

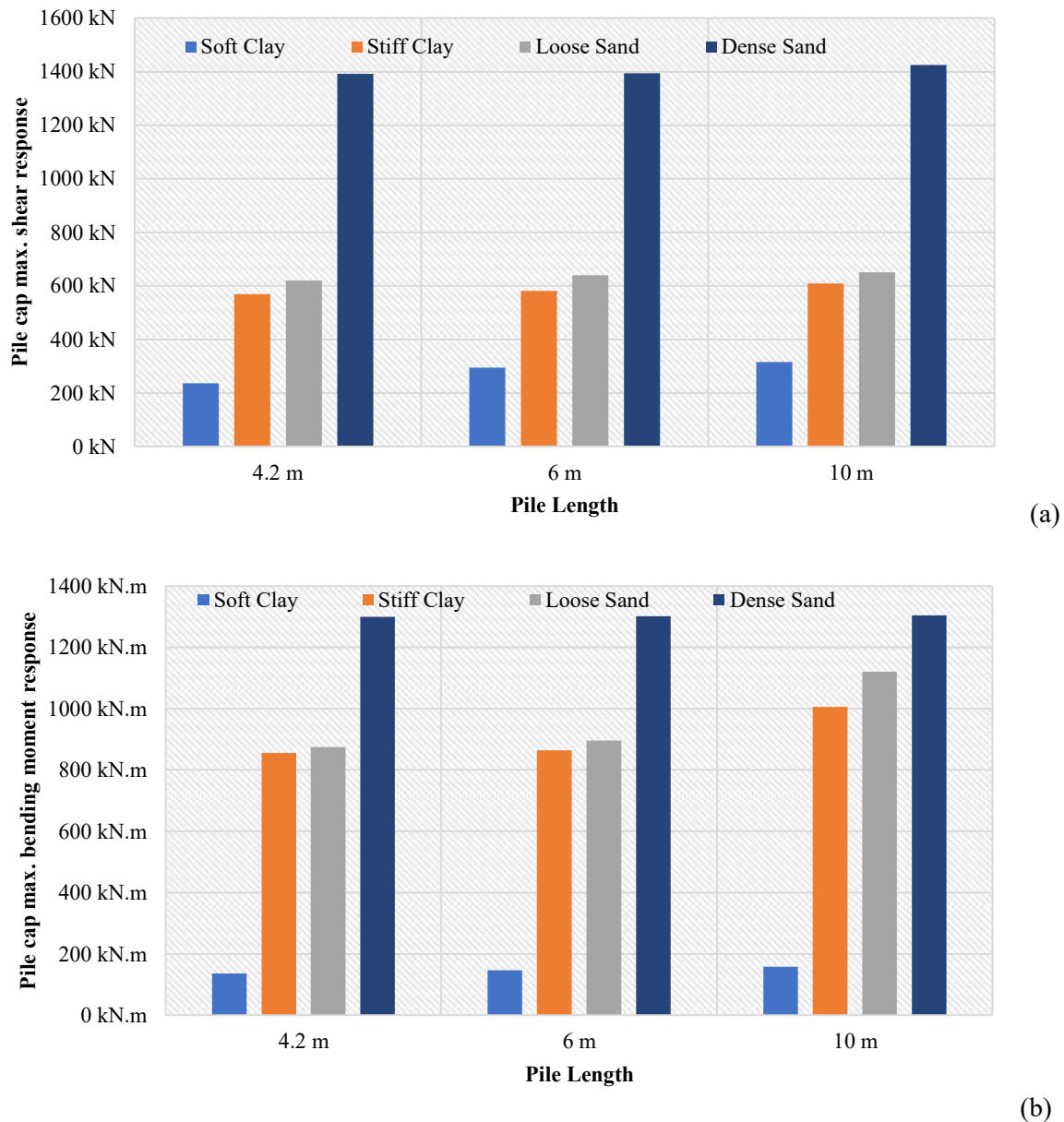


Figure 3-33 - (a) Peak lateral force and (b) bending moment response to pile head rotation (varying pile length)

Maximum shear and bending force reaction of pilecap to rotation is shown in Figure 3-33. The magnitude of both reaction forces increases with the length of piles. Evidently dense and loose sand show a stiffer behavior than either stiff or soft clay in response to pilecap rotation.

Similar to the former case of pilecap lateral displacement, investigations are carried out regarding the pile deformed shape to pile-head rotation. This time the pilecap rotation did not result in pile tip rotation for all the selected pile lengths. Thus, elastic shear force and bending moment response due to pile head rotation could be calculated from Equation (1-1) respectively. The ratio of pile head elastic shear force response of 10m long to 4.2m long pile is $((4/10)^2 =) 0.17$. For the pile head rotation range of elastic pile and soil deformation, the shear and bending moment response of the rotated pile head was larger for shorter piles. For instance, the response of embedded pile in stiff clay to pilecap rotation is plotted in Figure 3-34.

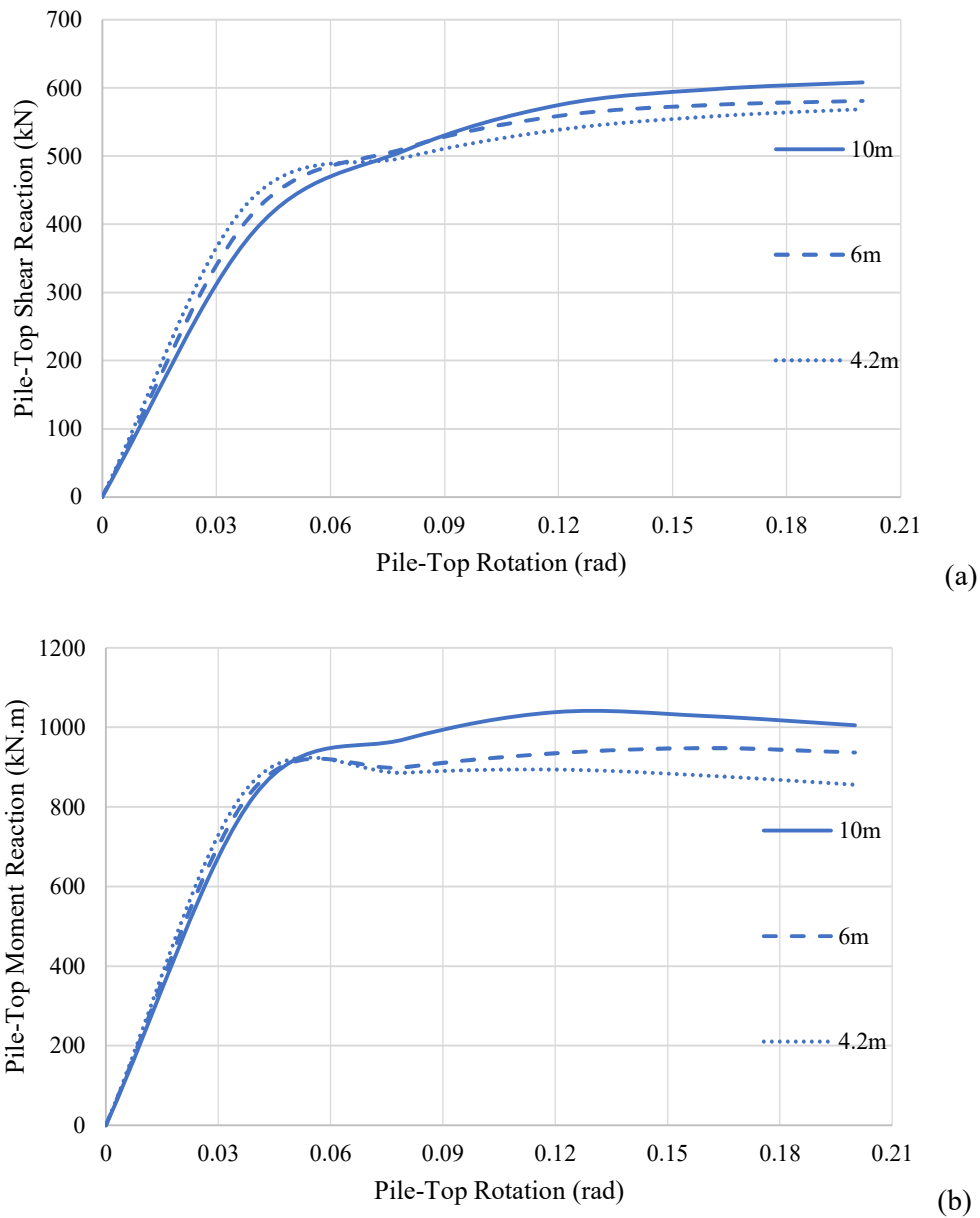


Figure 3-34 - (a) Pile head shear and (b) bending moment reaction to pilecap rotation in stiff clay (for different values of pile length)

For small values of pilecap rotation (0 to 0.034 rad), both reaction forces in Figure 3-34 were found larger for the shorter pile. After this limiting rotation, the resistance of longer piles gradually increased. Nonlinearity of response could be due either to pile or to soil nonlinear response. Particular attention should be paid to the peak resistance value. This does not necessarily occur for elastic deformations.

Furthermore, cyclic loading of piles cause a comparatively non predictable soil resistance, especially when large axial loads are applied due to gradual compaction of some part of damaged soil near the ground surface known as gapping effect (Isenhower and Wang 2016) Figure 3-35 shows the difference of soil contact pressure against local displacement when a one way loading and while a cyclic loading acts on pile (Pranjoto and Pender 2003). Although, LPile does not use reversed cyclic loading, nevertheless the nature of gapping effect exists when especially clayey soil type is available. There is no considerable number of tests performed on the effect of cyclic loading on sandy soils particularly fort lateral cyclic loading of piles, but it can be concluded that the frequent loading of pile head in same course result in flow of sand in the open gap behind the pile head, which in turn cause constant pile head deformation (Isenhower and Wang 2016).

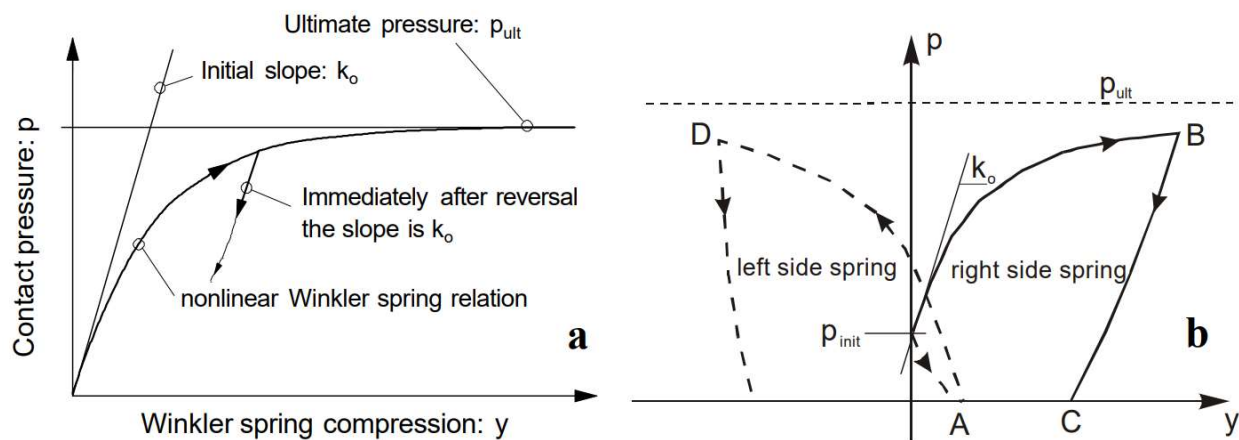


Figure 3-35 - (a) Winkler springs (b) De/Attaching springs on both sides of pile (Pranjoto and Pender 2003)

3.4.5. Push-over analysis for a group of piles and pilecap

Performance and resistance of piles in a pile group can be similar but also lower than the combined effect of individual piles subjected to lateral displacement and rotation. In case piles are located close to each other a reduction factor needs to be applied to the resistance curve of piles (Cox et al. 2008), (Prakash 1962), (Lieng 1989). Some researchers have provided recommendations to specify this reduction (Focht and Koch 1978), (Bogard and Matlock 1983). However, this seems to be a complex phenomenon because the reduction factor is deemed to be dependent on several parameters, and not only on the spacing of piles. For instance, the pile arrangement and magnitude of loading are important considerations while

recommending a reduction factor for performance of each pile in a pile group, thus the factor is applied to soil resistance (Brown et al. 1987). One reason for reduction of soil resistance in the vicinity of nearly located piles in a pile group can be due to overlapping of the shear fault surfaces of the neighboured piles and therefore additional weakening of soil known as the shadow effect may occur (Reese et al. 2018).

In order to consider the mutual effect of neighbouring piles on foundation response to sway and rocking motions, computer program GROUP (Reese et al. 2019) was used to model one half of the pilecap, studied in this work, with a set of 9 piles. It should be noted that the pilecap under Avonmore bridge pier consisted of a 1.2 m thick plate supported on 18 HP 310×110 piles. The bridge deck is connected to a pier-cap through a frame-type pier with two columns. As mentioned before, a method to introduce foundation response to the base of columns is to compute the response of the complete foundation and obtain the input in the form of a matrix in the superstructure program. Since there were two columns provided, it was decided to consider half of foundation to be adopted at the base of each column.

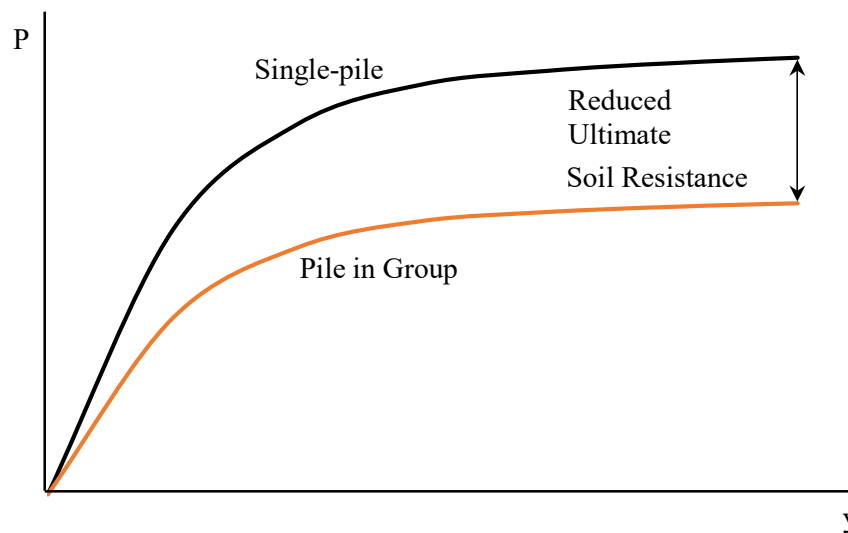


Figure 3-36 - Characteristic p-y curve for reduced pile performance in pile group (Reese et al. 2002)

The computer program GROUP (Reese et al. 2018) accounts for reduced efficiency of the piles acting in a group setting. There are three effects considered for calculation of the required modification factor (f_m), namely, the side-by-side effect, the trailing, and the leading effect (Reese et al. 2002). Several researchers have made extensive effort on finding generalized reduction factors for any type of soil, diameter (b) and spacing of the piles (s) in a foundation. Below is the approach used by program GROUP (Cox et al. 2008). Curves were fitted to recommended partial modification factors (β) by different authors (Cox et al. 2008), (Prakash 1962), (Schmidt 1981).

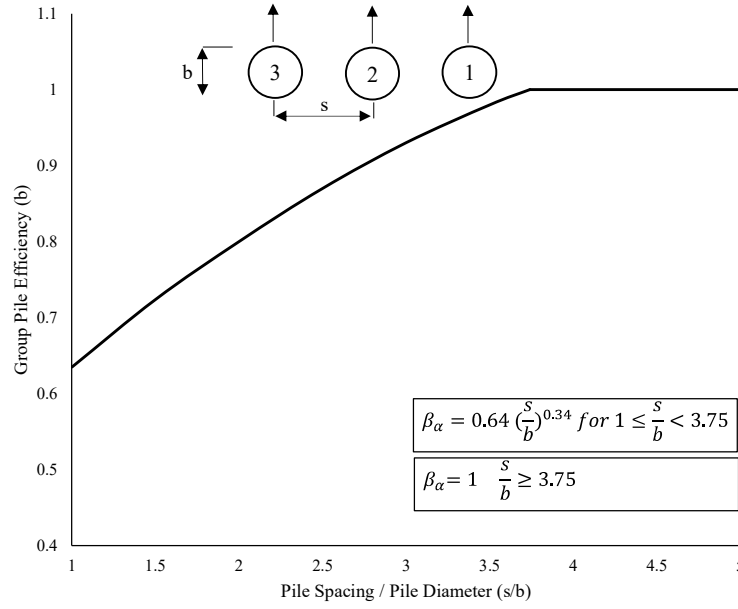


Figure 3-37 - Partial reduction factor for piles in a row after (Reese et al. 2002)

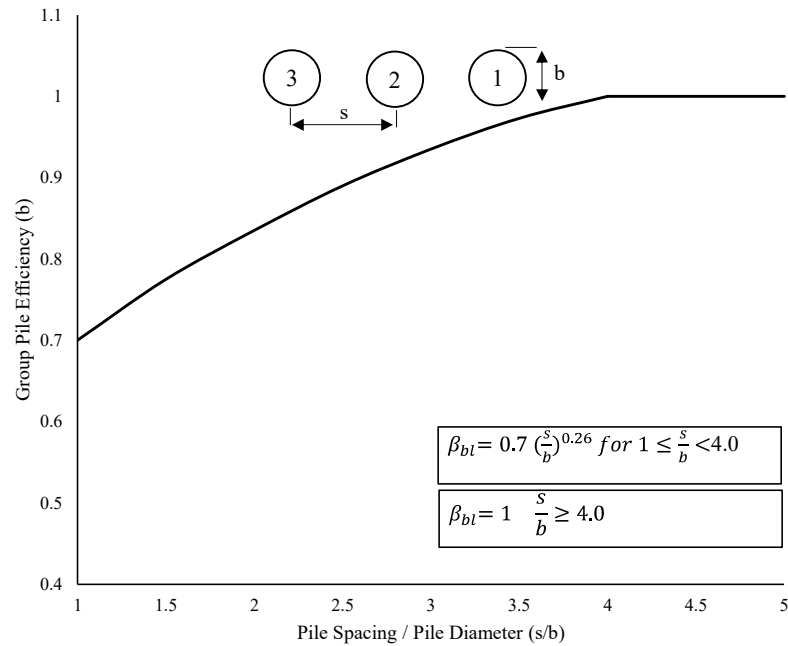
The modification factor to be applied to the p-y curves of single pile within a pile group can be obtained from multiplication of the partial modification factors as in Equation (3-20) after Reese et al. (2018):

$$f_m = \beta_\alpha \cdot \beta_{bl} \cdot \beta_{bt} \cdot \prod \beta_s \quad (3-20)$$

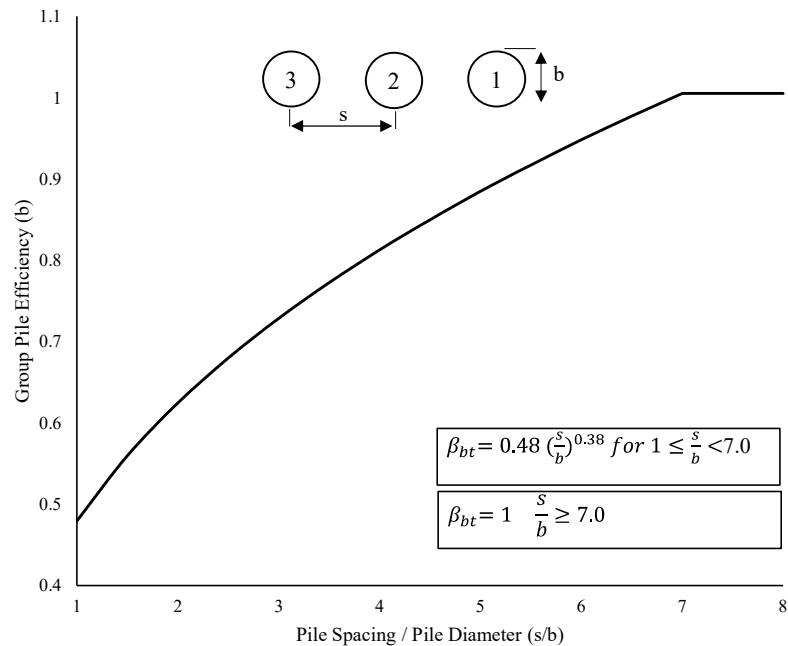
The partial modification factors for pile spacings in a row and in a line (leading and trailing piles) are obtained from Figure 3-37 and Figure 3-38 respectively. The last component of Equation (3-20) considers the effect of skewed piles which is derived from the Equation (3-21), whereby β_b and β_α are calculated with side by side and in-line equations, assuming that the pile spacing is the diagonal between piles and χ is the angle of the piles with the horizontal plane:

$$\beta_s = (\beta_b^2 \cos^2 \varphi + \beta_\alpha^2 \sin^2 \chi)^{\frac{1}{2}} \quad (3-21)$$

The last term in Equation (3-20) accounts for the interaction of a single pile with several other skewed piles. Note that the developed equations for reducing the soil response are all conservative assumptions due to lack of experiments on all soil types in addition to construction type of piles. For instance the implemented efforts do not differentiate between a bored and a rammed pile, which can be accompanied with loosening or strengthening effect on ground (Cox et al. 2008).



(a)



(b)

Figure 3-38 - Partial reduction factor for piles in a line – (a) leading & (b) trailing after (Reese et al. 2002)

Figure 3-39 and Figure 3-40 show the output of the performed analyses in program GROUP. It is evident that the shear force or bending moment response of pilecap to either lateral displacement or pile head rotation is not equal to 9 times the response of a single pile response as presented in previous section, but considerably smaller values were obtained from the program.

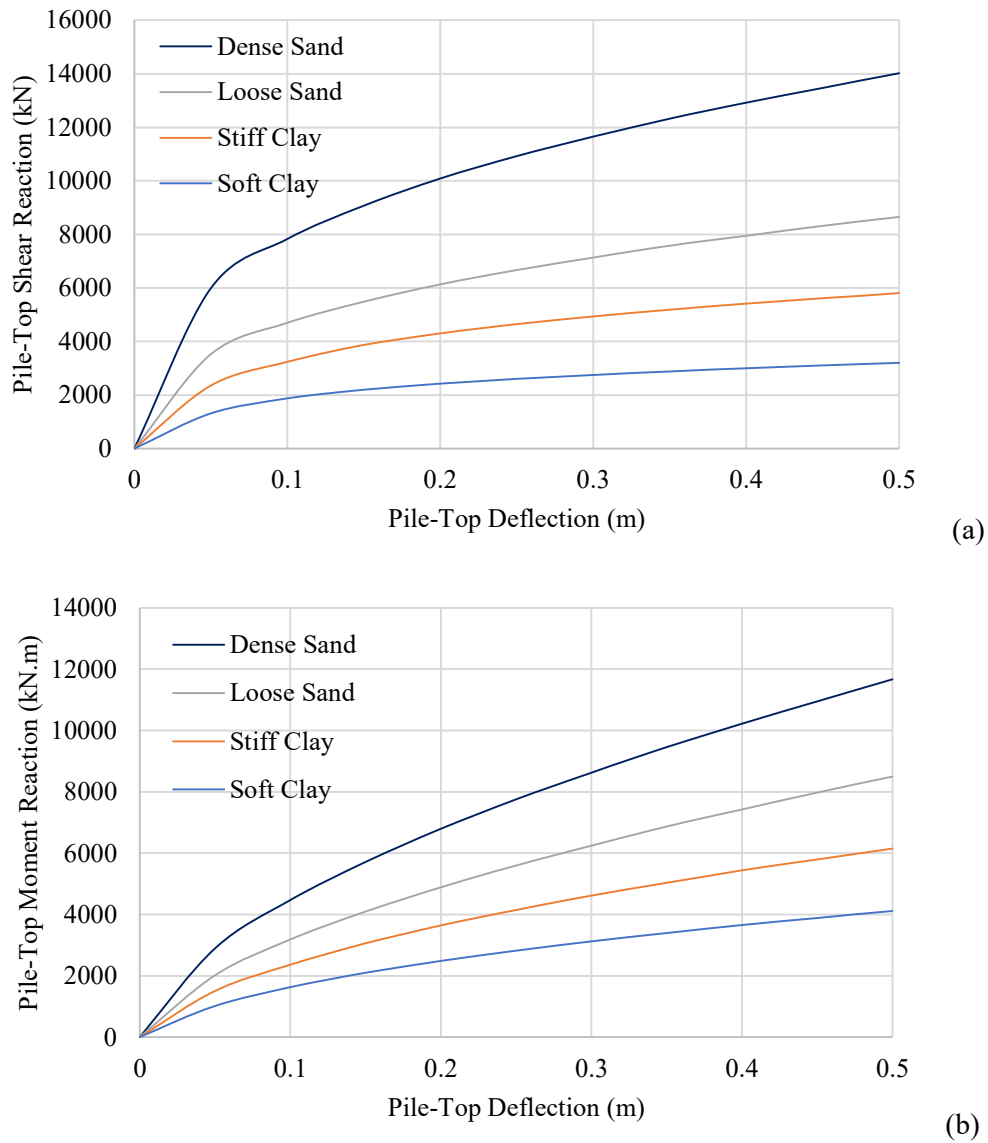


Figure 3-39 - Push-Over Curves for group piles: (a) Shear Force Displacement and (b) Bending Moment Displacement

The spacing of piles in the longitudinal direction (perpendicular to the bridge axis) is about 4 times the pile diameter ($s/b=4$) whereas in the shorter side of pilecap the spacing of piles is about 3.5 times the pile diameter ($s/b=3.5$). The direction of imposed displacement in program GROUP is along the shorter side of pilecap; here consideration of the reduction factors is necessary as the spacing of piles is less than the provided limit used in deriving from Figure 3-37 and Figure 3-38 (limit for $\beta_\alpha = 3.75$, for $\beta_{bl} = 4$ and for $\beta_{bt} = 7$). Accordingly, the program reduces the resistance of foundation. As expected, embedded piles in dense sand have stiffest setting and piles in soft clay the least stiffness. But embedded foundation

in stiff clay and soft clay have almost similar bending moment resistance to rotation. Other than that, foundation in four soil types represent an increasing stiffness from clayey ground to sandy soil.

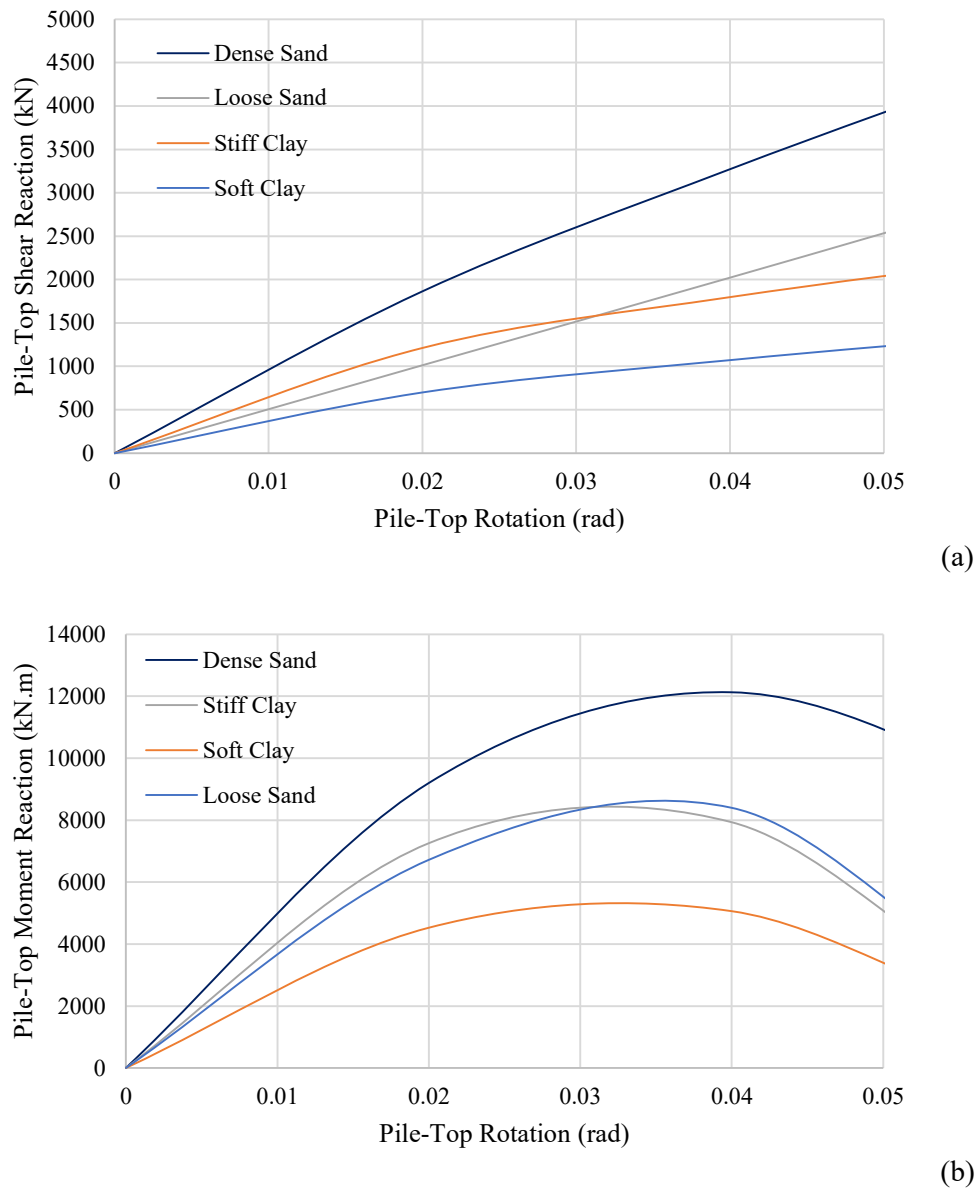


Figure 3-40 - Push-Over Curves for group piles: (a) Shear Force Rotation (b) and Bending Moment Rotation

In this chapter the endeavour was to identify and understand the magnitude of foundation translation and rotation under lateral loads such as those occurring during an earthquake, and the implications these might have on the superstructure. In the program GROUP – following the same procedure as LPile - first a lateral displacement was applied to pilecap where other movements (rotation of the pilecap) were restricted; the resistance of the pilecap and group of piles was calculated and a characteristic resistance curve was

obtained. In the next step, the process was repeated by applying a gradually increasing rotation on the pilecap while restraining the lateral displacement. These two deflection types correspond to the sway and rocking motion of the system of foundation in response to earthquake excitation as shown in Figure 3-41. The force displacement envelopes thus obtained, were input directly into the program CSiBridge as properties of the equivalent rotational and translational springs that can model the compliance of the supports. In the following chapters will be shown that for further understanding of how structural elements like pile and pilecap interact in a FE model, piles are remodelled with soil reaction represented explicitly by distributed springs with p-y properties applied along pile length. Results of this section are used in the structural program (CSiBridge) to compare the two alternative methods of accounting for the soil compliance (by equivalent springs at the base of the piers and by explicit modeling of p-y springs along the pile length). Comparison of the two modeling approaches will be conducted in the forthcoming chapters considering stiff clay soil properties for foundation material (Table 3-1).

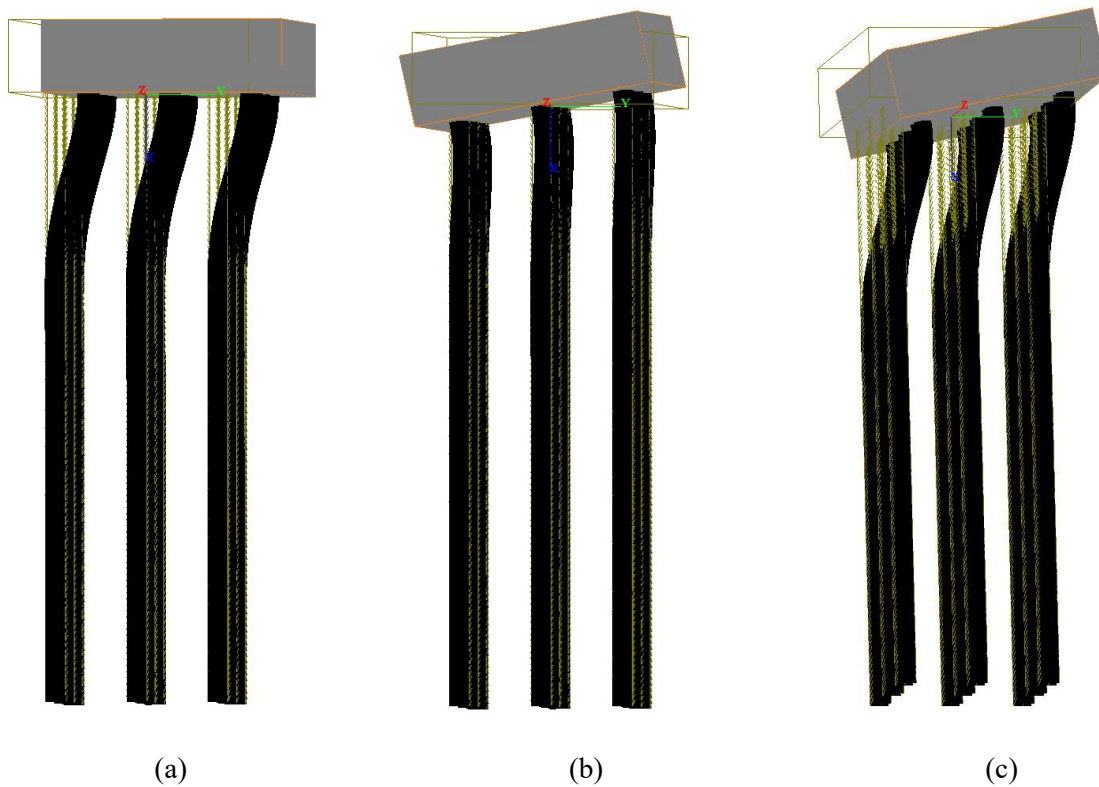


Figure 3-41 - Deformed group pile shape (from program GROUP): (a) lateral displacement (b) Pilecap rotation (c) combined displacement & rotation -

3.4.6. Alternative method for push-over analysis of a single pile and pilecap

In order to compare the response of pile head obtained from the LPile program and FE (finite element) analysis a model was setup in the program PLAXIS 2D. A pushover analysis was performed with a pilecap with the same dimensions as those used in the previous analyses. The embedded pile with a length of 10 m was modelled as a nonlinear beam element. The soil medium was modelled using Mohr-Coulomb failure criteria with the same properties as those used before for the stiff and soft clay, as well as for loose and dense sand. As noted before, two sets of analyses were performed to estimate the four elements of the stiffness matrix as in equation (3-17), which are in fact are force displacement relationships. Therefore, the necessary boundary conditions were introduced to the system for each case, respectively. Connection of the pile to the pilecap occurred at a point of 500 mm into the depth of the pilecap, while the 90 degree angle between the two was fixed (FHWA 2016). Since the effect of settlement was ignored in the previous cases, vertical movement of pile top and tip was set to zero in the FE model. In the first case, the lateral displacement at the centroid of the pilecap was the controlling variable in the pushover analysis. In the second scenario, a rotation at the centroid of the pilecap while restraining lateral displacement was applied. The deformed shape of both cases is illustrated in Figure 3-42.

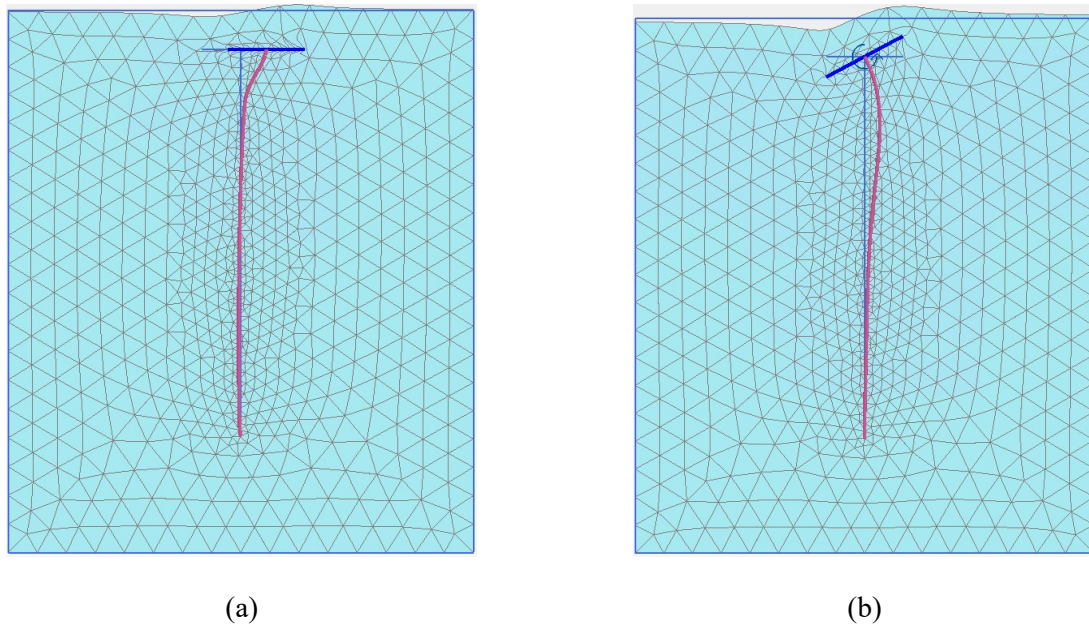
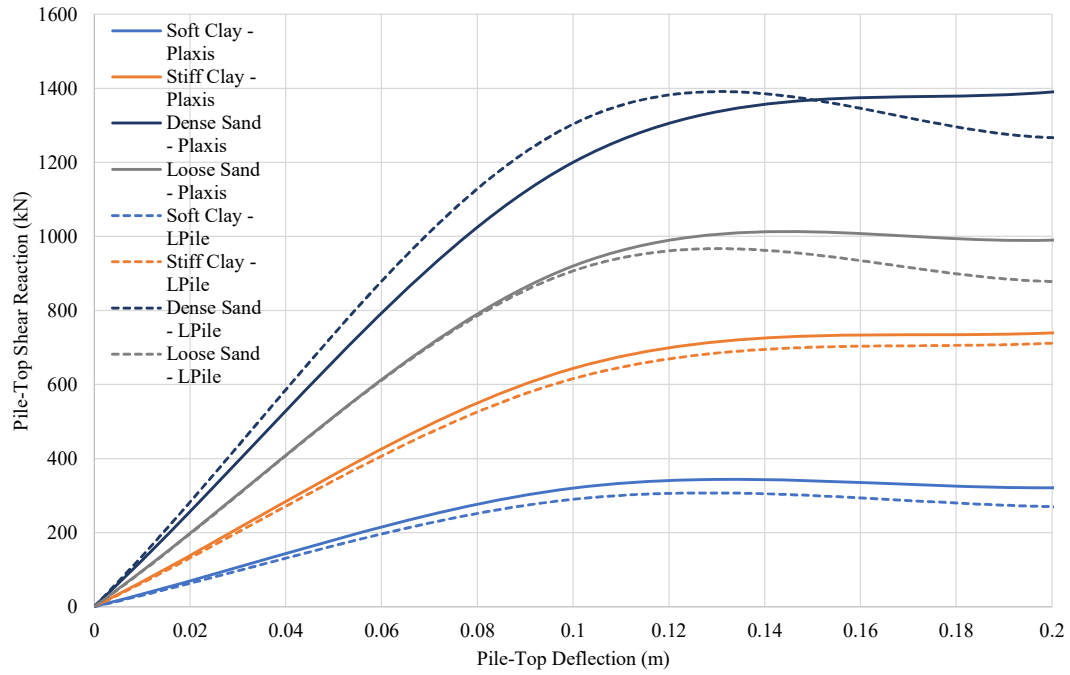
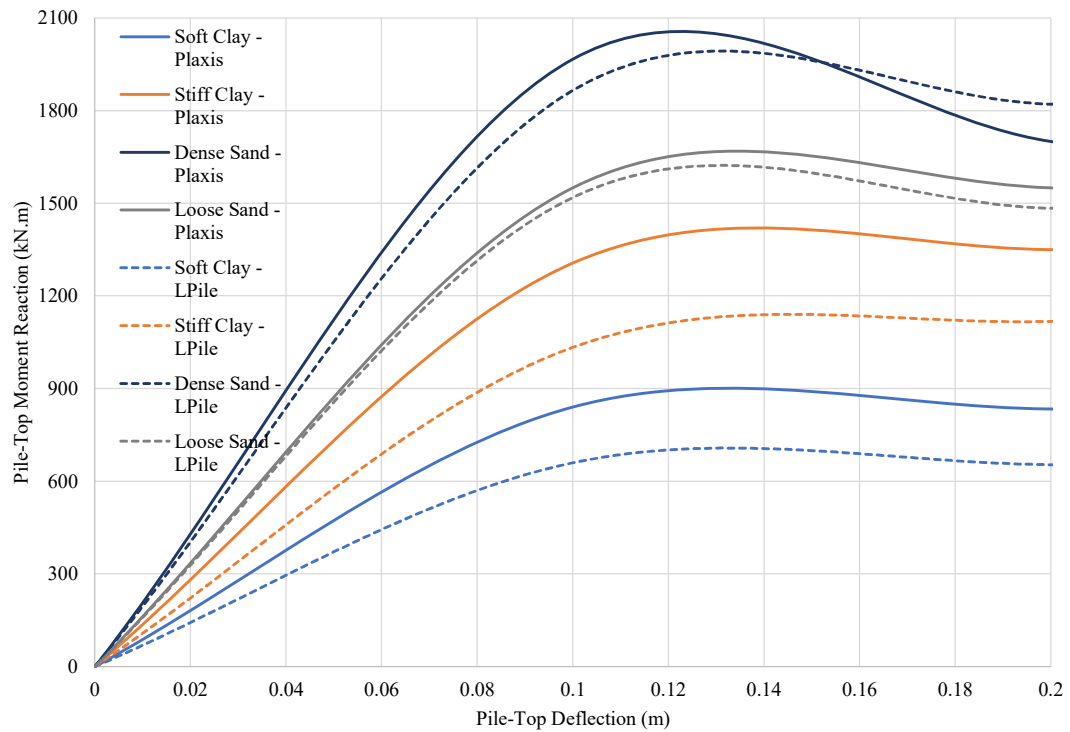


Figure 3-42 - Single pile deformed shape using PLAXIS 2D due to (a) pilehead lateral displacement (b) pile head rotation–



(a)

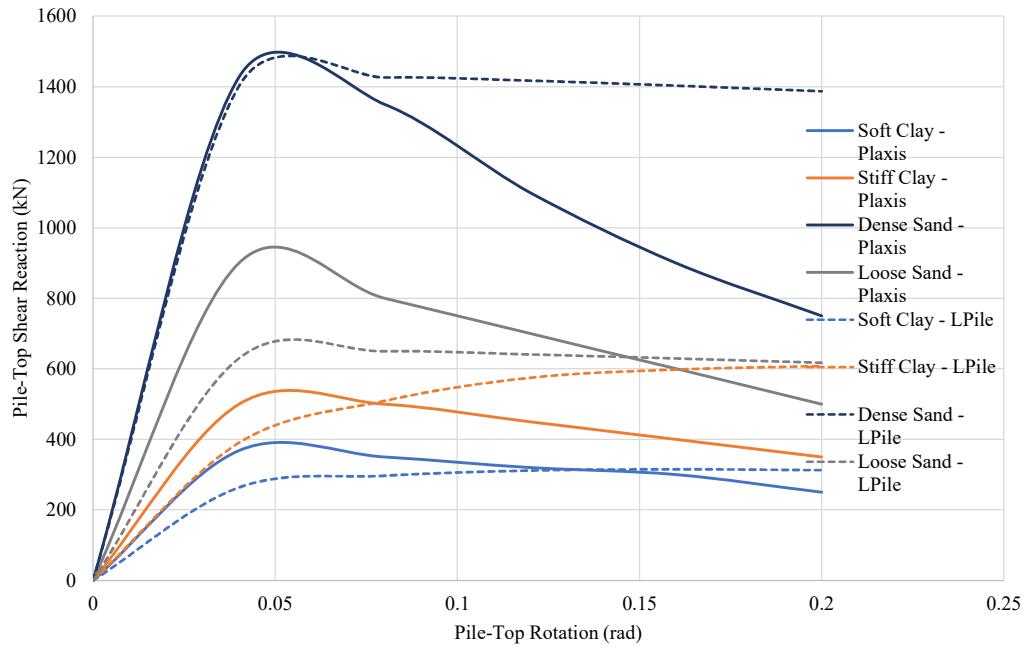


(b)

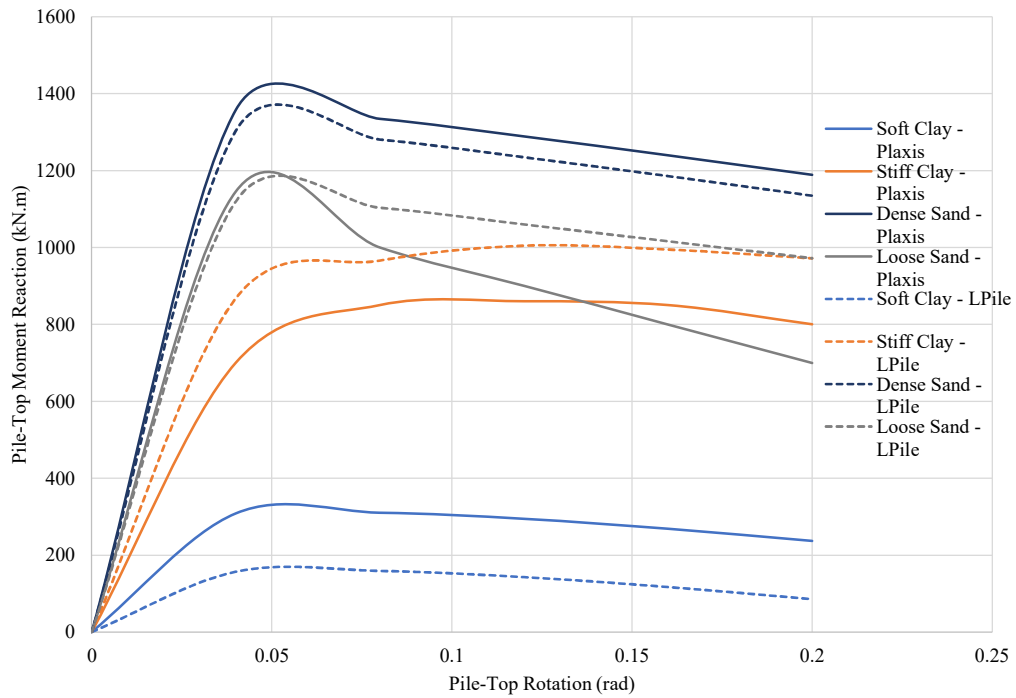
Figure 3-43 – (a) Shear force displacement and (b) Bending moment displacement - Pushover curves - PLAXIS 2D vs. LPile.

Comparing the results obtained from FE analysis with results from the substructure method using LPile as in Figure 3-34, it is evident that a slightly higher maximum pile head shear and bending moment resistance was estimated from the FE model for pilecap lateral displacement. However, a steeper post peak descent for the bending moment resistance for embedded pile in dense and loose sand according to the FE analysis was observed for pile head displacement larger than 200 mm (Figure 3-43). It may also be observed that shear failure for foundation in soft clay occurs at a larger displacement. The respective higher stiffness of foundation in FE compared to finite difference analysis can be related to the different numerical approach used to solve the field equations; the effect of a finer discretization, but also to the different way of modeling the pilecap to pile connection.

Similarly, higher maximum shear and bending moment resistance was observed as a result of pile-head rotation from FE analysis, except for maximum bending moment reaction of stiff clay, which is slightly lower than in the preceding comparison. In this regard pile head response differs from substructure method so that shear force reduces more drastically after 0.05 rad pile head rotation especially for cohesionless soil types. Similar observation can be made for the bending moment response of embedded piles in loose sand.



(a)



(b)

Figure 3-44 – (a) Shear force rotation (b) and Bending moment rotation - Pushover curves - PLAXIS 2D vs. LPILE

In the future chapters the impact of soil structure interaction on response of superstructure to seismic excitation will be discussed. In this context, nonlinear time history analyses were performed on a bridge with compliant base. To account for soil structure interaction in the structural analysis of the bridge, two approaches were adopted to simulate the soil. In first scenario, foundation components such as piles, pilecap and abutments were modelled independently, and soil was modelled using attached spring to piles and pilecap. Stiffness of the springs were obtained from LPile analysis. This model is referred to hereon as “substructure method with springs along the pile length”. Alternative to this is the detailed modelling in continuum, however this would yield an excessively large model of the entire system of bridge / embankments & abutments and mobilized soil. A second scenario considered modelling of the superstructure independently with springs at the base of pier which represents the soil compliance. Stiffness of springs were obtained from the GROUP type analysis (where interaction effects within the group of piles was also considered). Results from PLAXIS were not used any further because they could not be incorporated in the 3D computer model of the bridge system.

CHAPTER FOUR: SEISMIC ANALYSIS OF THE AVONMORE BRIDGE: FIXED BASE CASE

4.1. Introduction

In this chapter first the process of modelling the selected two span bridge for seismic analysis is described in detail (MMM GROUP 2013), with reference to structural details and proposed idealizations in the numerical analysis. The main bridge components include deck, abutments, bent-cap, columns, and foundation. In this chapter the focus is on response of the superstructure assuming a fixed base; several seismic excitations were considered acting in the longitudinal axis of the bridge where columns deflected without influence from the framing action of the cap-beam. Applicable time history records for the selected bridge site at Cornwall were generated from simulated records by (Atkinson 2009). Two scenarios were considered for the type of connection between bent-cap and deck girders, namely a monolithic connection and a bearing type support. Analysis results will be presented in form of bridge column drifts, in accordance with two scenarios and in response to selected time history excitations. Analysis of the bridge system was conducted using the nonlinear finite element code CSiBridge (CSiBridge 2017a). This is a platform that has many common elements with SAP2000, the main difference being that the reference cross sections in the program library uses the established prefabricated prestressed beam sections used in the superstructure of North American bridges, as well as several standardized elements and properties related to bridge hardware.

4.2. Bridge superstructure and numerical modelling

4.2.1. Bridge deck

The bridge deck consists of a 64m long and 11m wide reinforced concrete slab with thickness of 225mm as in Figure A-2 and Figure C-1. The deck slab has a two percent transverse slope for draining purposes. Longitudinal reinforcement bars at top of section are interchangeably 30M and 20M bars, and 20M bars at the bottom of section (Figure C-2). Transvers reinforcement bars are 15M spaced at 300mm. In the model top longitudinal reinforcement was modelled across the deck cross-section width using 63 - 30M bars and 60 - 20M bottom bars; reinforcement was arranged symmetrically in the cross- section. Reinforcing steel was specified as Grade 400w in selected bridge drawings. Therefore, the minimum tensile yield strength and ultimate strength were taken equal to 400 MPa and 540 MPa respectively. Unit weight of the bar was taken as 77 kN/m³ with elastic modulus (E) of 200,000 MPa. Compressive strength of applied concrete for deck was 30MPa. On top of the deck there was a 90mm layer of asphalt and waterproofing system, which were accordingly modelled as permanent load. Reinforced concrete parapets were designed for both sides of the deck having 825mm height and a thickness of 250 mm. On each side of the parapet wall 5 – 15 M

bars were placed longitudinally as shown in Figure 4-1. Two 15M anchors, outer to longitudinal bars, connected the parapet to the deck slab. Cover to anchors was specified to 50mm.

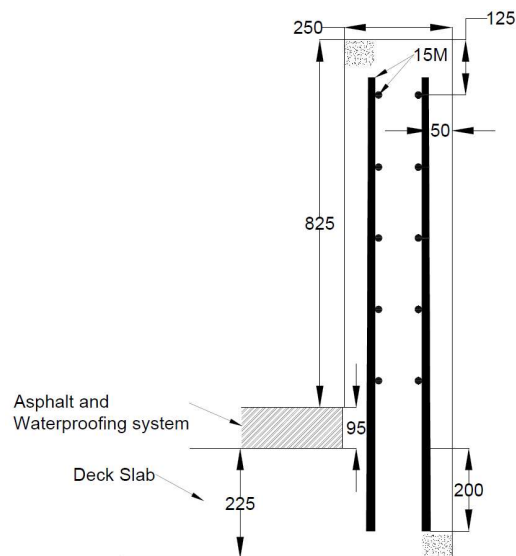


Figure 4-1 - Idealized section of reinforced concrete parapet and deck asphalt

Five rows of prestressed NU-1200 girders run longitudinally under the deck slab (Hanna et al. 2010). Seven low relaxation wire strands with size 15 and Grade 1860 were used in this girder type. Cover was 50mm and the concrete class was 60MPa specified in design. In the absence of a design report for pretension loss parameters, the default tendon loads for both tendon ends (see Table 4-1) as in the CSiBridge analysis program were applied. Two straight tendons and two harped tendons were used as in Figure 4-3. The same superstructure cross-section was used for the entire bridge length. Cross sectional dimensions of the girder NU-1200 are shown in Figure 4-2.

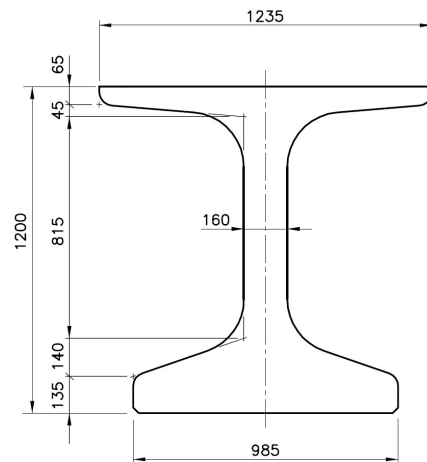


Figure 4-2 - Cross section of NU 1200 girder (MMM GROUP 2013)

Intermediate vertical diaphragms were modelled at middle of each span, between girders and with monolithic connection to the deck. Diaphragms consisted of 16 – 20M longitudinal bars, confined with 10M bars at 300mm. Rectangular diaphragm sections were accordingly modelled in CSiBridge.

Table 4-1 - Tendon loads for both ends of tendons

Curvature coefficient	Wobble Coefficient (1/mm)	Anchorage Set (mm)	Slip (mm)	Elastic Shortening Stress (MPa)	Creep Stress (MPa)	Shrinkage Stress (MPa)	Steel Relaxation Stress (MPa)
0.15	6.56E-07	6.4		0	34.47	48.26	34.47

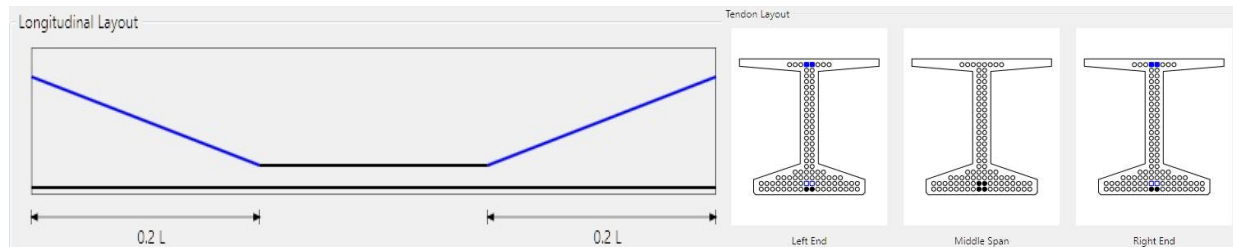


Figure 4-3 - Layout of modelled tendons in program CSiBridge (Black spots correspond to the straight tendons and blue spots are the harped tendons)

Two scenarios were considered for connection of bent cap to NU-girders: The first option was a fixed connection between the components, and a second option was connection of pier cap to girders through elastomeric bearings. In the absence of detailed information about the stiffness of the bearings in the available design for the studied bridge, several commercial references were inspected in order to specify the shear stiffness of elastomeric bearing (EKSPAN 2016), (TRELLEBORG 2016), (Standards-Australia 2004), (GOODCO 2010), (NOR-Rubber 2016). It was understood that the applied elastomers should meet the resistance capacity requirements of CAN/CSA-S6-06 and OPSS 1202. For the specified bearing thickness of 20mm and sectional dimensions of 400mm×350mm, the shear stiffness of 4120 N/mm was initially estimated. However, control analysis for the robustness of the model was conducted with the selected bearing stiffness, showing failure of time-history analysis just at the beginning of the earthquake record due to excessive displacements (lack of robustness). Therefore, the shear stiffness of the bearing was increased to 6000 N/mm to enable convergence, preventing early failure of bearings and of the dynamic

analysis. This enabled realistic study of the behaviour of columns in response to seismic demand. In Figure 4-4 the idealized section of the modelled deck components is depicted.

Since the design earthquake combination is examined, the applied vertical loads exclude the weight of vehicles. Therefore, the self weight of deck slab, NU girders, asphalt, and parapets were imposed as vertical load and transferred to the columns and foundation.

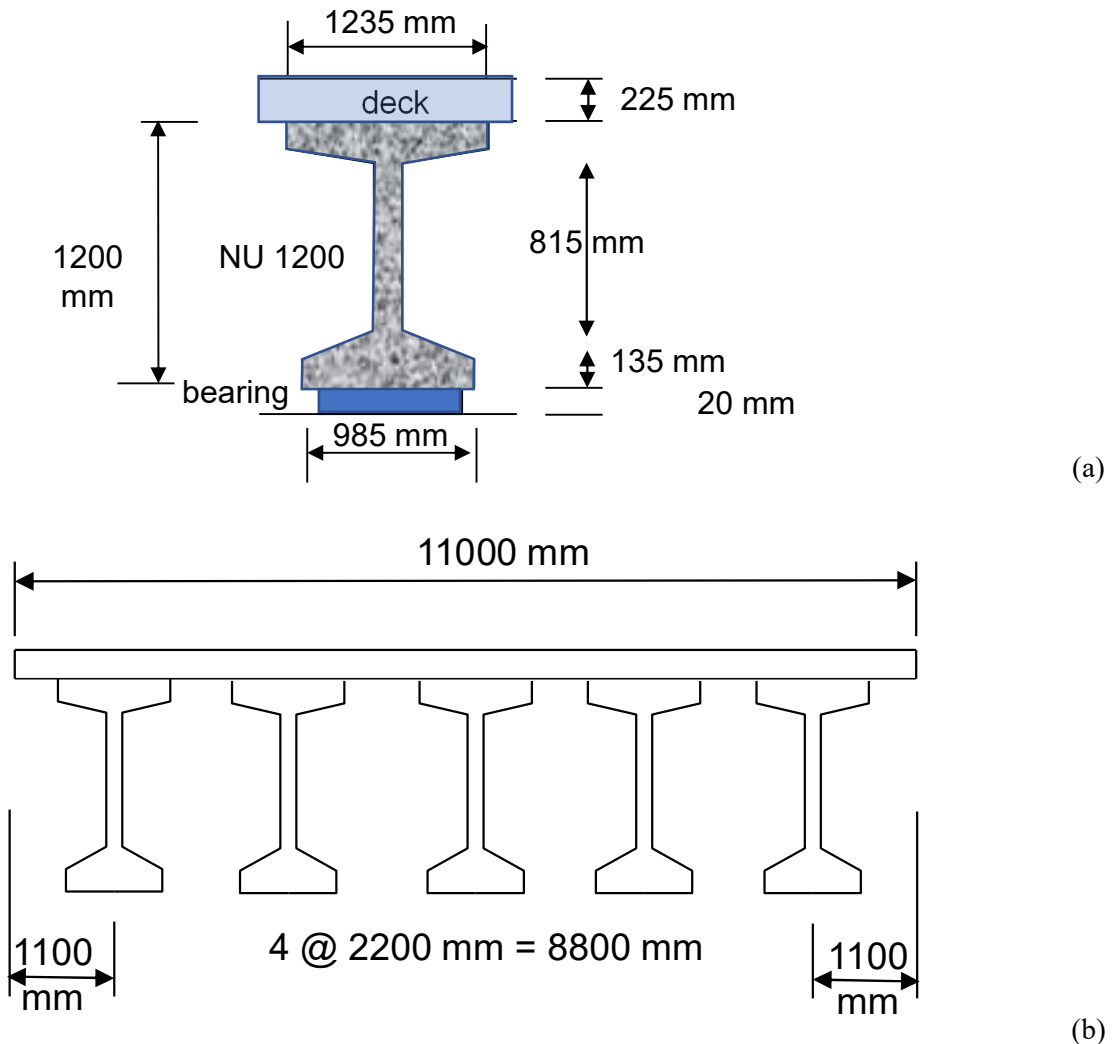


Figure 4-4 – (a) Idealized deck, girder and bearing and (b) idealized deck and girders section for modelling in CSiBridge

4.2.2. Bridge pier

The pier consists of a reinforced concrete cap with rectangular cross-section and two reinforced concrete columns with circular sections. The clear height of column was 4850mm, with 600mm embedment length in bent cap and pilecap. Diameter of the column was 1200mm. Column longitudinal reinforcements from

pilecap and bent cap were overlapped over 2200mm. Longitudinal bars were evenly distributed in the column perimeter; 24 – S30M bars, confined with S15M spirals at 70 mm spacing and with clear cover of 70mm (transverse reinforcement was used to calculate the confined concrete properties according to the confinement model of (Mander et al. 1988)). Designed concrete class for pier cap and columns was 30MPa. Columns were distanced at 6050 mm from each other and 2431mm from each end of the pilecap. The bent cap height and width were 1500mm and 1200mm (see Figure C-4), respectively. In the available drawings 9 – 20M longitudinal bars were provided on each side of the cross-section. Furthermore, 9 – 30M bars were included at top and bottom, as well as seven additional 30M longitudinal bars at top. In addition, 20M stirrups at 250 mm were applied as shown in Figure 4-5.

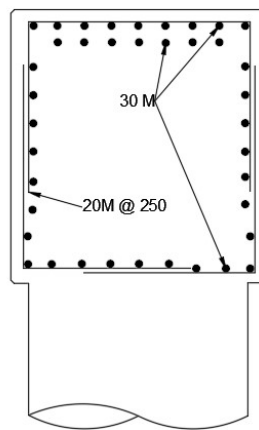


Figure 4-5 - Idealized cross section of pier-cap for modelling

Due to relative complexity, this section was modelled with the section designer form in program CSiBridge. Length of pier cap was 10912mm (Figure 4-8). Particular attention was paid to the translational degrees of freedom between column and pier cap.

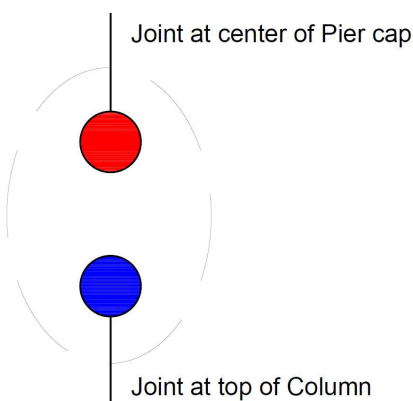


Figure 4-6 – Schematic joint restraint for fixed connection of column and pier cap after (CSiBridge 2017b)

To account for fixed connection between the columns and the pier cap, in the bridge bent data form of the program, all rotation and translational degrees of freedom at the top of column were set to be fixed to those of the end node of the cap-beam (rigid connection). This setting is interpreted as a common joint between bent cap and column despite the different elevation coordinates of the respective two points. Therefore, all relative rotations and displacements between top of columns and pier cap are suppressed. In the Figure 4-7 joints of columns at the connection point to bent cap is illustrated.

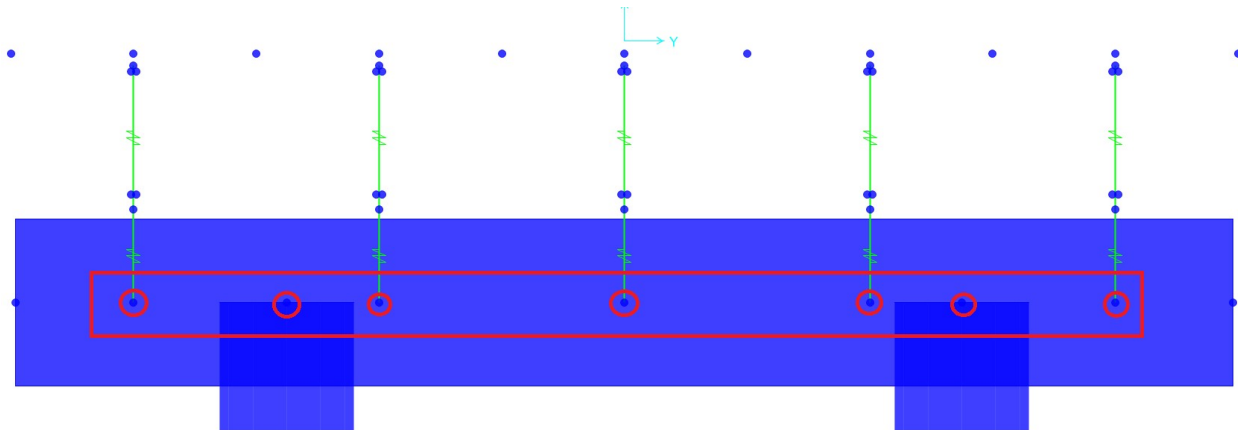


Figure 4-7 – Group joints at top of columns and bent cap

In order to fix the relative displacement between column top and bent, joints at top of columns and joints at middle of bent cap were constrained in the same joint group. As a result, all joints displace together.

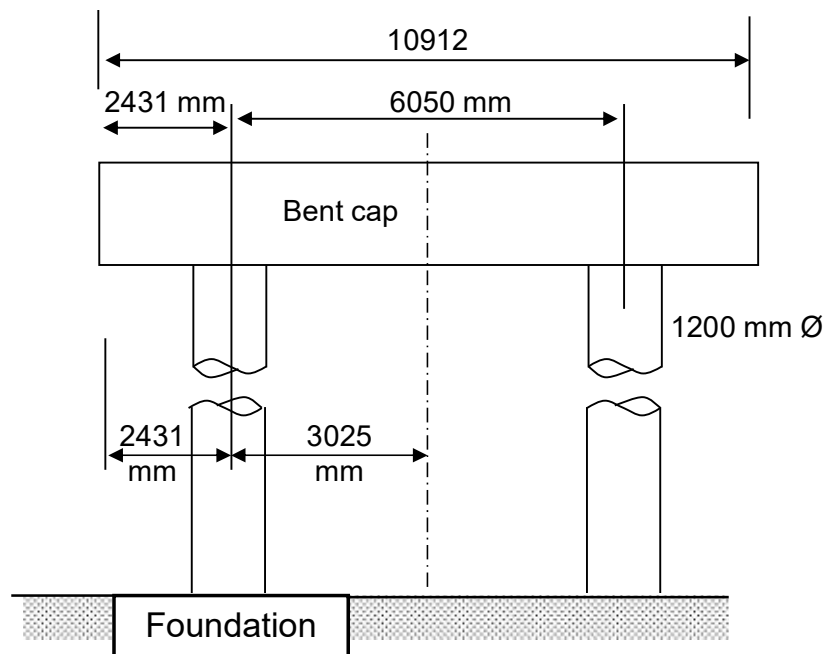


Figure 4-8 - Dimensions of modelled columns and pier cap

In program CSiBridge a window is dedicated to assign the bent object to the bridge. Bent is located at the middle of the deck at station 32000mm.

4.2.3. Bridge abutments

The NU-girders at both ends of deck are supported on 1200mm thick reinforced concrete abutments with a height of 3400mm and a width of 10900mm. Abutments are supported on eight H-piles as explained in Chapter 3 of the thesis. H-piles are embedded in the abutment over a length of 500mm, thus, a fixed head connection between piles and abutment is assumed (FHWA 2006). Abutments serve as supports of the deck at both ends. Modelling and configuration of abutments are performed considering the lateral displacement and certainly the overbearing vertical loads. In order to sustain lateral and transverse displacements at abutments for many abutment configurations a fusible shear key is added to the end of abutment as in Figure 4-9.

The shear key prevents yielding to abutment for induced displacements, however for displacements exceeding the fuse gap, yielding of abutment is likely. Another approach is to design the component to sustain nonlinear stress-deformation. Meanwhile, flexibility of the foundation to lateral movements should be considered. This is a soil structure interaction problem. Similar to the compliant foundation at pier base, flexible foundation can be assumed at the base of abutment. It was in Chapter 1 and 2 discussed that the magnitude of relative displacement of the superstructure can be reduced when established on a compliant substructure. To examine this aspect, two types of abutments are introduced as shown in Figure 4-9, namely a stub and a seat abutment. Note that the resistance of stub abutment to induced lateral forces relies on soil resistance.

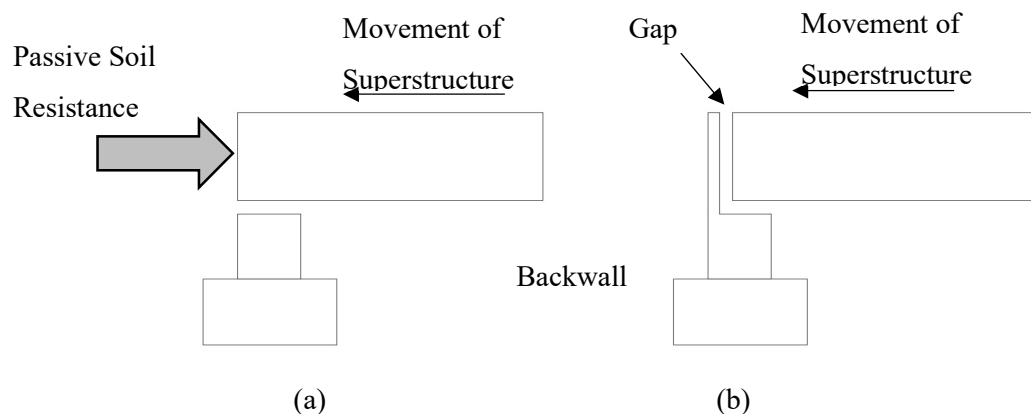


Figure 4-9 - Different abutment configuration based on lateral resistance (a) Stub (b) Seat abutment (FHWA 2014)

For modelling of abutments of the selected bridge, roller supports were applied. A roller support sustains vertical loads but provides no resistance to longitudinal translation. From the available abutment drawings (Figure C-3) it appears that a shear key setting was provisioned for the abutment of the example bridge. To model this arrangement, first a roller bearing was introduced into the model, whereas vertical translation, as well as translation normal to the layout line was set to fixed. Also, the translation along the layout line, and the rotation about all three axis were set to free. Preliminary trials in program CSiBridge showed that with such rollers on the abutments an early failure occurs. To avoid the early numerical failure and to comply with the provisions of the design of example bridge, identical elastomers as those used at the bents with a lateral stiffness of 6000 N/mm were modelled (form 'Bridge Bearing Data' of program CSiBridge is used for this setting) at both ends between abutments and NU-girders. This way the goal of avoiding yield at bridge deck and abutments was achieved. This configuration is also recommended in literature (American Association of State Highway and Transportation Officials 2002). The final model of the bridge in program CSiBridge is shown in Figure 4-10.

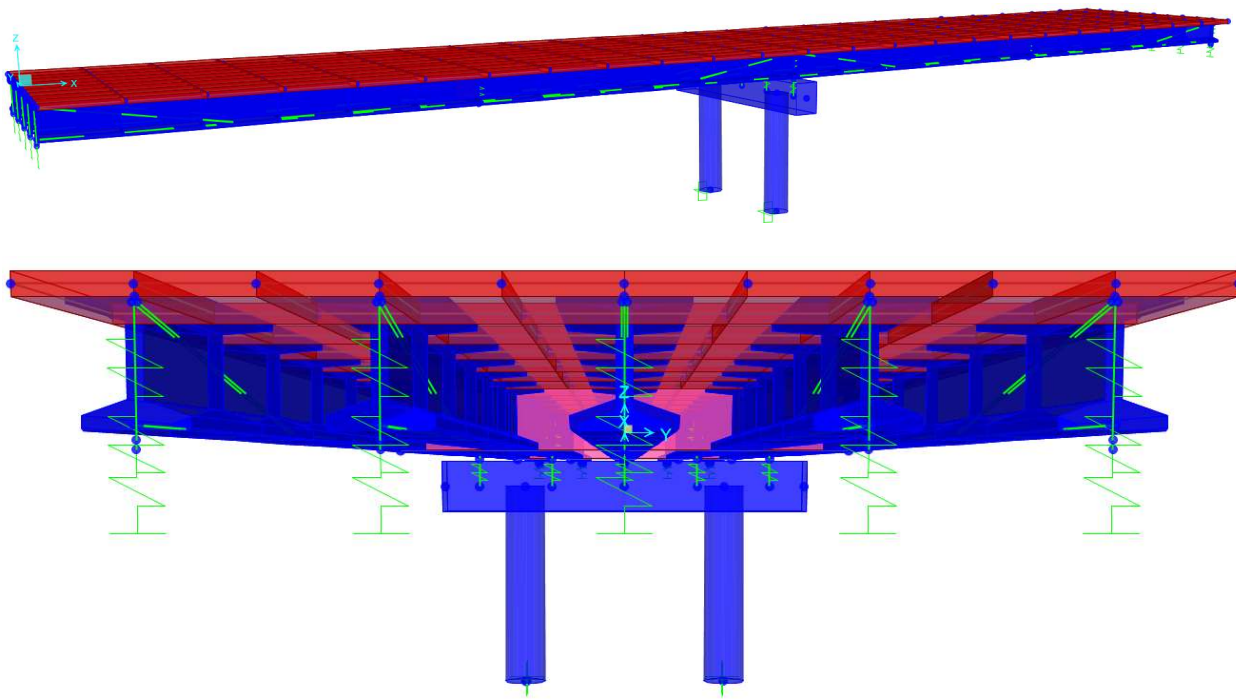


Figure 4-10 – Fixed-base bridge model in CSiBridge

Several different levels of complexity could be adopted in modeling the dynamic response of the bridge system. The model developed herein was rather extensive and detailed so as to enable the observation of the many parameters that affect the longitudinal dynamic response of the bridge (e.g. modes of vibration of the continuous deck, effect of sliding in the abutments, effect of soil compliance in the abutments, sliding

of the deck over the central pier (over the elastomer, etc.). A simpler model such as a pier with a tributary mass at the top and the compliant foundation model at the base would be another, simple alternative, in which case, however, the rotatory inertial effects of the tributary length of the deck that is excited in the longitudinal vibration would need to be estimated and the kinetic energy associated with that motion added to the effective mass of the system in order for the solution to be entirely relevant.

4.3. Preparation of seismic excitation record

In order to pursue seismic nonlinear analysis, simulated time history ground accelerations from www.seismotoolbox.ca was used to excite the bridge superstructure. Use of simulated ground motions generated by the current seismological model over historical records was done as required by the Canadian Bridge Design Code to produce design-spectrum compatible strong records which could sufficiently move the structure. Recommendations of the National Building Code Canada 2015 (NBCC 2015) in division B was followed to factor the selected acceleration time history records so that the account for regional specifications of the provisions. These include the site soil classification for seismic site response which affect shear wave velocity, geometric effects, topographic irregularities which affect spread of wave propagation and intensity (Research Council Canada 2015), (Kramer 1996). Historically, it was suggested that to account for the mentioned effects modification factors should be applied to time history records in order to use them for a site other than the site of origin (Krinitzsky and Chang 1977), however a careful modification of the record rather than only multiplication by modification factor was also recommended (Vanmarcke 1979).

As discussed before four type of soft ground was used for purpose of the sensitivity analysis. Selected ground types represent stiff and soft soil (E) considering their shear wave velocity (V_{s30}) and average standard penetration (N) values (Research Council Canada 2015). Greater modification factors $F(T)$ are recommended to scale the site-specific spectral response acceleration, $S_a(T)$, according to NBCC 2015. Thus, site class E was selected for further matching the Uniform Hazard Spectrum (UHS) from code to design spectral acceleration values, $S(T)$. UHS was derived from Appendix C of the NBCC 2015, which represent horizontal spectral acceleration values with 2% likelihood of exceedance in 50 years. In the code it is maintained that the UHS values are recommended according to statistical analysis of recorded hazards in either the target regions or in neighbourhood areas. For the selected bridge UHS values for Cornwall east of Ontario was chosen.

A procedure was recommended to select the ground motions and to scale them to reach the spectral response acceleration and also, to apply to acceleration time history respectively (Tremblay et al. 2015). The steps followed to generate the scaled time history records are described in detail in the next paragraph.

Two sets of ground motions for two scenarios low-magnitude (M6) and high-magnitude (M7) were selected for the ground class E from www.seisimotoolbox.ca which were generated to represent the major Magnitude-Distance (M-R) seismic scenarios in Canada (Tremblay et al. 2015). Each set includes 45 simulated response spectra and time history records. It is recommended that for different period ranges those records to be matched with the target response spectrum that contributes to the definition of the design hazard (Halchuk et al. 2007), for instance M6 for short- to intermediate-period structures and M7 for long period structures. Target period is built within a specified minimum and maximum value, defined as T_{min} that equals as a minimum 20% of the fundamental mode of vibration and a highest period which accumulates not less than 90% of mass participation ($T_{90\%}$) (Tremblay et al. 2015). For the example bridge the fundamental mode of vibration and $T_{90\%}$ when the base is fixed were computed as 0.498s and 0.291s respectively. Therefore, T_{min} of the range is specified as 0.1s. Besides, the maximum period of range of interest is defined as the highest value of twice the fundamental period and 1.5s (Tremblay et al. 2015). In the latter case, the maximum period calculated was 1.5s. Thus, two period ranges were set to 0.1s to 1s and from 1s to 1.5s. For the first range M6 records were important and for the second case M7 records. For the first range 5 case scenarios were selected from simulated record directory and for the second range 6 cases for the purpose of seismic nonlinear analysis to comply with the requirement of NBCC 2015.

Two fault distances were used to subdivide 90 simulated records for M6 in two groups of 45. The first set were simulated for radius of 10 to 20 km about the fault and second set 20 to 30km distanced from the fault (Atkinson 2009). Five records were required to be selected from M6 database. Criteria for assortment of records are to select records with smallest standard deviation of ratio of recommended spectral acceleration by code, $S(T)$, to spectral acceleration of simulated record $S_a(T)$ over the period range of interest per each record. Furthermore, the mean value of the acceleration ratio should remain between 0.5 and 2 (Atkinson 2009). After this practice for all 90 records were performed, the following records were rated as the best match for the above criteria as in Table 4-2.

Table 4-2 - Rated and selected records for group M6

Record Nr.	East6e1 - 2	East6e1 - 15	East6e1 - 34	East6e2 - 3	East6e2 - 5
Fdist (km)	12.8	10.7	16.6	19.8	19.5
Mean $S(T)/S_a(T)$	1.29	1.04	1.49	1.66	1.81
SDEV $S(T)/S_a(T)$	0.438	0.489	0.569	0.66	0.906

Similarly, 90 records were rated and selected for the Scenario 2 from M7.0 simulated records as listed in Table 4-3.

Table 4-3 - Rated and selected records for group M7

Record Nr.	East7e1 - 3	East7e1 - 45	East7e1 - 10	East7e2 - 7	East7e2 - 8	East7e2 - 9
Fdist (km)	20.8	24	24.9	42.2	45.3	47.2
Mean $S(T)/S_a(T)$	0.53	0.54	0.53	1.09	1.3	1.36
SDEV $S(T)/S_a(T)$	0.171	0.157	0.190	0.456	0.516	0.479

The mean value of all selected records per period of interest was computed, for instance for the scenario specific periods between 0.1s and 1s, desired records were selected from M6 category and for periods between 1s and 1.5 s preferred records were chosen from the M7 category. Response spectra of preferred records are shown in Figure 4-11.

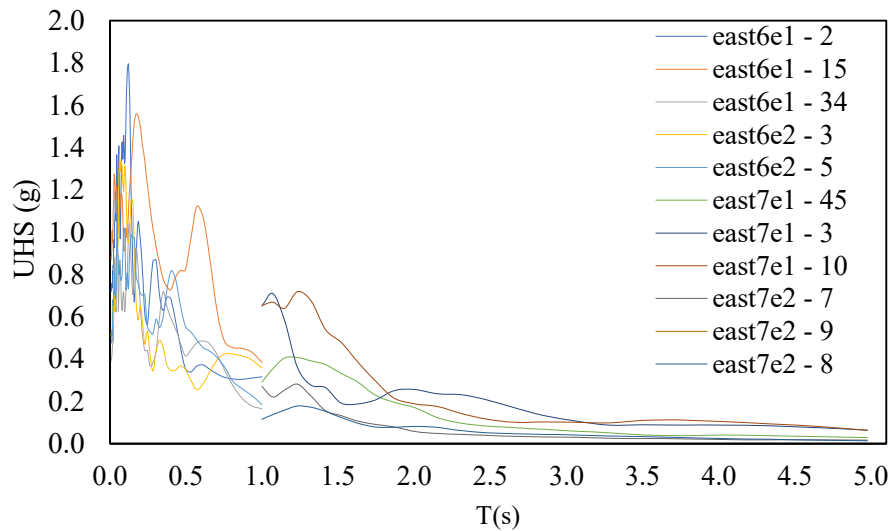


Figure 4-11 - Response spectra of preferred records for magnitude category M6 and M7 (Atkinson 2009)

Afterwards, the mean spectra of selected records within the period range 0.1s to 1s and 1s to 5s were established. In order to match the mean spectrum with the target spectrum, for the first range, the factor of 1.24, and for the second range the factor of 1.18 were multiplied by the mean spectrum, so that all acceleration values are not less than 90% of the design spectral acceleration values, $S(T)$ as in Figure 4-12.

In the following, the corresponding simulated time history records were scaled up with the abovementioned factors for M6 and M7 categories as instructed by (Atkinson 2009).

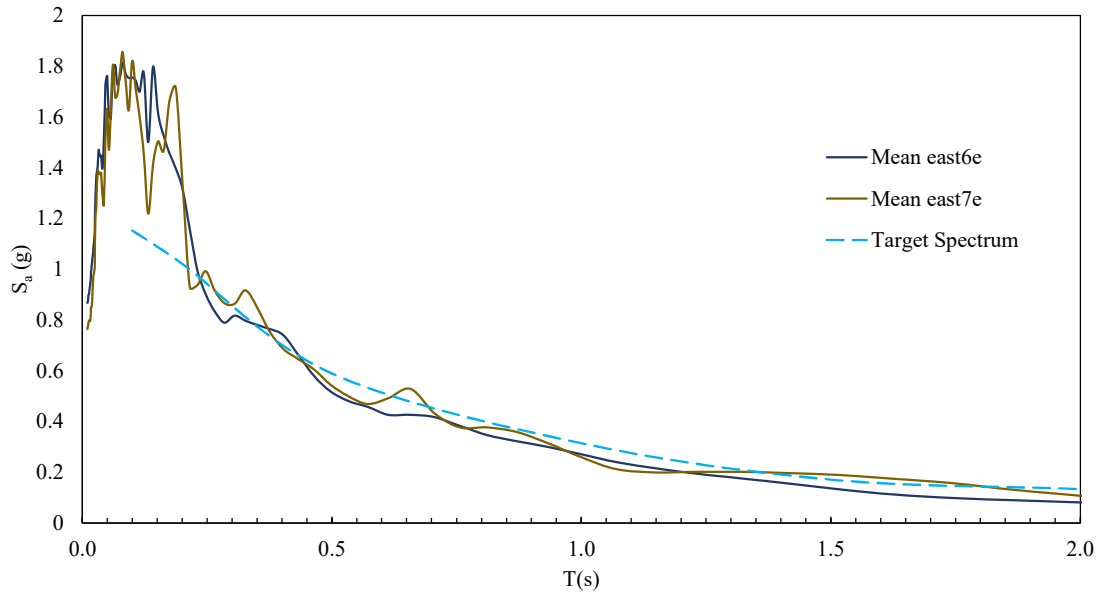


Figure 4-12 – Scaled response mean spectra of scenario specific preferred records to match the target spectrum

Scaled time history records were later applied to the bridge model for seismic analysis. By the first trials it was observed that the response of selected bridge to the applied excitations does not sufficiently reflect the soil structure interaction effect. In other words, response of superstructure did not significantly change between a compliant and a fixed-based scenario. Therefore, the second scaling factor of 1.5 was applied to all time history records to intensify the damage to bent and enable more accurate assessing of soil structure interaction effect on bridge columns response as in Figure 4-13 and Figure 4-14.

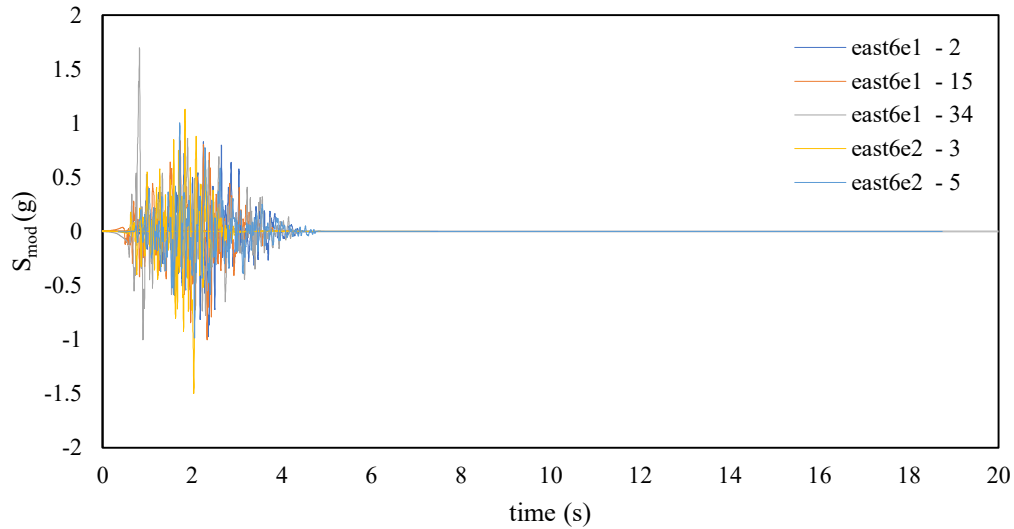


Figure 4-13 – Scaled preferred time history records for scenario 1

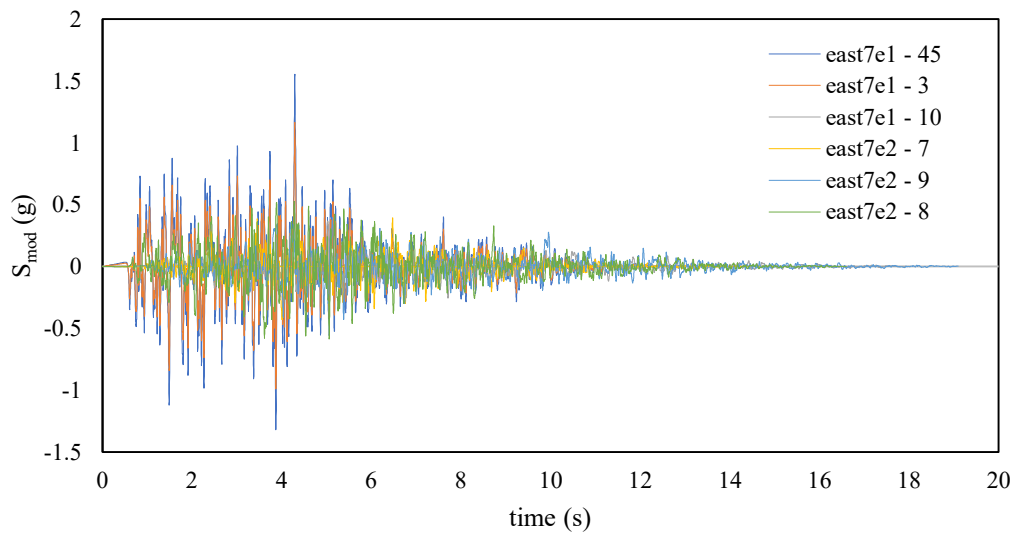


Figure 4-14 - Scaled preferred time history records for scenario 2

4.4. Time history analysis input

All 11 scaled records were entered in program CSiBridge V.20 to perform seismic analysis of the bridge. Records were prepared in ASCII file format to be input in the analysis. An input form in the program is designated to incorporate time history records. Time steps were set to 0.002 in ACSII file, as well as in program time history function window.

For the analysis, 3 load cases were defined. First a nonlinear dead load case which consist of the bridge self weight including deck, girders, haunch, barriers, and side walks. In this analysis no live load was considered

in order to avoid additional parameters that could potentially affect the bridge response. While for the nonlinear dead load case a static load type was selected, the analysis type was set to nonlinear and geometric nonlinearity was set to P-Delta, which accounts for nonlinear buckling of the columns (CSiBridge 2017a) and for second order effects. A modal load type was also defined, whereby this load case adopted the stiffness from the nonlinear dead load case. Modal analysis was conducted using Ritz vectors over Eigen vectors, due to the fact that global convergence is of prime interest, whereas convergence of localized response is not important in connection with the number of modes that are requested for analysis (CSiBridge 2017b). Furthermore, this option facilitates a faster analysis, whereby duration of each time history analysis was shortened from 15 hours to 10 hours for 198 trials. Load type was set to acceleration along the bridge layout, since analyses were all performed along bridge longitudinal axis. Mass source is taken from bridge mass.

Time history records can be input as in program predefined time history load type. Nonlinear analysis time history with direct integration over modal solution was selected to account for nonlinear response of the structure. Transient history type was the only available option for time history load case. Applied load type was set to acceleration in longitudinal direction. Since the selected units of measurement for force and displacement in the program were originally set to N and mm respectively, whereas the acceleration values of time history records had units of cm/s^2 , the scale factor for the time history record was set to 10. Output step was set the same as that of the record's discretization, to 0.002s. Number of output steps were chosen as high as 27000. The reason was to avoid altering this value per analysis per record. Although most of the time history records lasted up to 55 seconds, non-zero acceleration values occurred in most cases over a small timespan of about 30 seconds. Note that the strong motion duration is not same for all records. For ease of creating ASCII files, some portion of the time range was omitted, where acceleration was zero. However, this adjustment varies among records. Therefore, to speed up the large number of analysis that had to be conducted, a fixed number of analysis steps was determined. Constant modal damping of 5% was considered for all modes. P-Delta geometric nonlinearity was considered in the analyses.

It is assumed that capacity design was performed for the selected bridge to resist lateral forces, which is usual for highway two span overcrossings. Accordingly, two plastic hinges form at top and bottom of columns while being subjected to lateral displacement whereas overstrength factors are usually applied to other bridge components like bent cap and deck so that they remain elastic during lateral excitations (FHWA 2014). Therefore, a nonlinear resistance mechanism is activated in the columns as shown in Figure 4-15 for longitudinal displacement of the bridge with the formation of plastic hinges in critical locations of the pier (plastic hinges are zones where plastic moments are developed (FHWA 2014), marked by energy dissipation through nonlinear deformation). Initially hinges occur at top of the columns of the frame pier

(A), then hinges form at the bottom of columns (B), and eventually deformation capacity of the pier-frame system is reached (C). Note that the intensity of deformation in the plastic hinges may be reduced if alternative sources of deformability may account for part of the displacement demand. For example, sway and rocking motion at column base create relative lateral displacement at top to bottom of columns that can account for a share of the total demand.

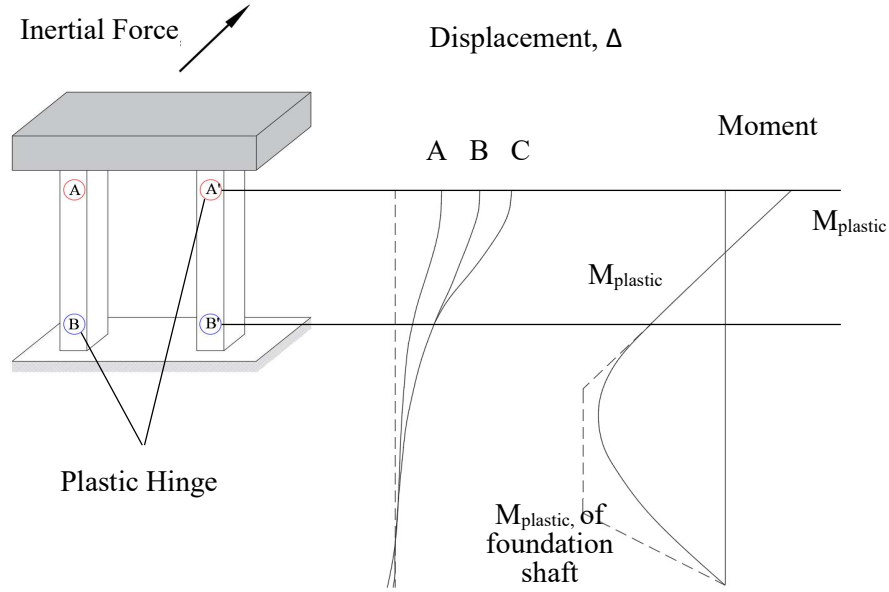


Figure 4-15 – Formation of plastic hinges in a two column bent in longitudinal direction, after (FHWA 2014)

In order to obtain nonlinear response of the structure from program CSiBridge to an induced time acceleration load, it is necessary to define the plastic hinge properties on the members that develop the deformation in the structure (i.e. piers). For bridges it is assumed that the intermediate bents resist seismic loads (FHWA 2014). Depending on the degree of kinematic restraint, plastic hinges may occur at top and bottom of columns over a critical length that is defined by Equation (4-1) (Priestley et al. 1996):

$$l_p = 0.08l + 0.022f_y d_{bl} \quad (4-1)$$

where l is the shear span (i.e., the length of column from the point where largest moment occurs to the point of contraflexure (zero moment)); f_y is the yield stress of column reinforcement and d_{bl} is the diameter of longitudinal reinforcing bar. For the bridge columns considered in the present study the length l_p was estimated equal to 0.67 m. Two degrees of freedom about vertical axis (column axis), and perpendicular axis to the bridge layout line were defined. Column failure condition was also set to flexure or shear, whichever occurs first.

4.5. Numerical analysis of fixed based bridge

For each record, a nonlinear time history analysis was performed. The goal was to calculate the imposed drift demands on the bridge columns as a result of seismic excitation along the longitudinal axis of structure, in order to estimate the damage potential of each motion in yielding of the piers. As discussed earlier, the base of the structure is considered fixed in the present chapter, so that all the earthquake energy is absorbed by the bent and elastomeric bearings at both ends of deck.

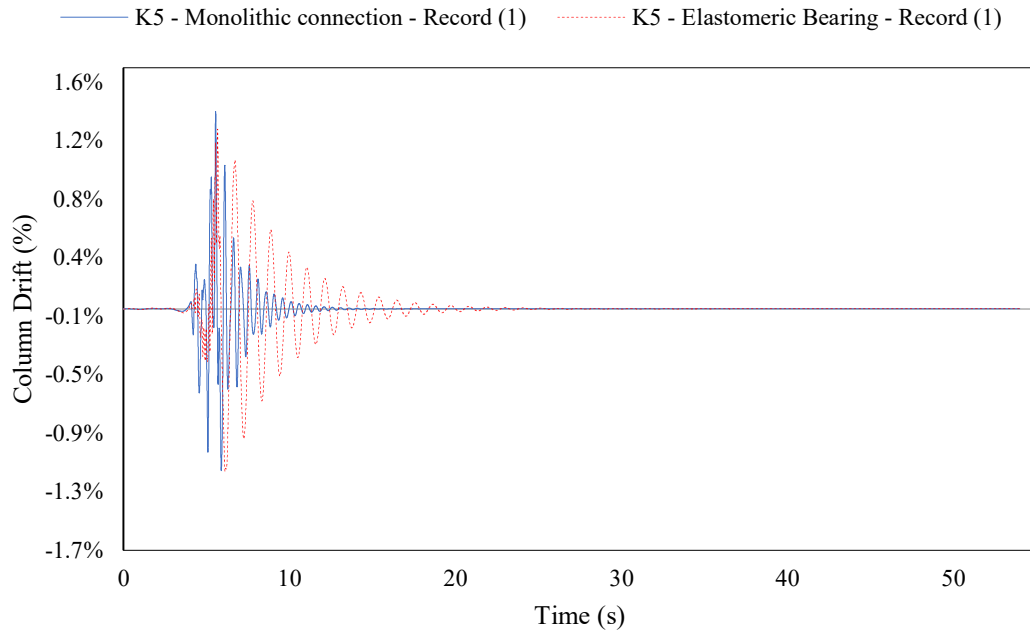
Two different types were considered for the connection of bent cap to the I-girders under the deck. In the first case, the cap was monolithically connected to the girders. In the second case, the bent cap was connected to the girders with elastomeric bearings. In this model, all three rotational degrees of freedom were unrestrained, and a stiffness value was designated to the translation in the direction of the longitudinal axis of the bridge, which is specified as partial fixity in the program. The selection of shear stiffness of bearings was discussed in the preceding sections of the chapter.

Detailed results of the time-history analyses are presented in the following chapter together with the responses of the bridge model with a compliant base, in order to obtain insights regarding the SSI effect on the response of bridges to induced excitations.

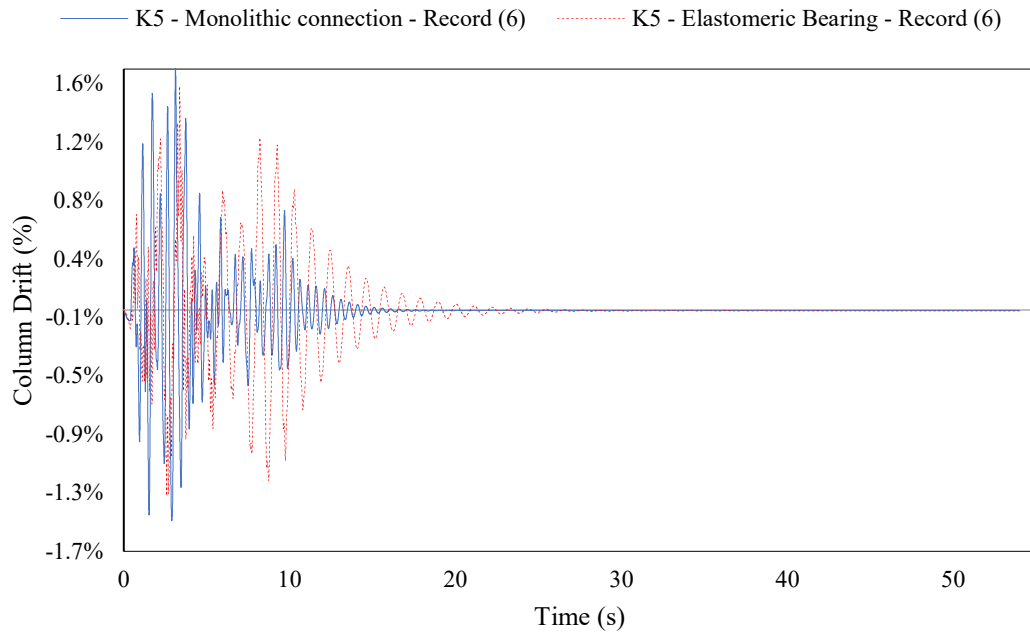
Overall, twenty-two analysis were carried out for the fixed base structure. Per analysis, longitudinal time history displacement response of the highest joint of columns at connection point to the bent cap was generated.

The clear height of column was 4850mm. This height was used to calculate the maximum occurred drift ratio in columns corresponding to the maximum absolute displacement at the top of the column. Both columns developed a symmetric lateral response shape, and due to the symmetry of the structure in the transverse direction, no difference between the response of the two columns was observed. In Figure 4-16, the response of the bridge to the first and sixth time-history input is depicted (K is the indicator of foundation stiffness, for instance K5 corresponds a fixed base pier. ‘Elastomeric Bearing’ corresponds to connection of cap/bearing to the girders through elastomeric bearings, and ‘Monolithic connection’ means fixed connection of bent cap to the girders) – the first case corresponds record **east6e1-2** and the latter case corresponds to record **east7e1 – 45**. These cases induced the largest column drifts, where the first record was selected for the lower-magnitude and second case was selected for the higher-magnitude scenarios as per the previous section. It is observed that 0.1% higher drift ratio has occurred in columns when bent cap is rigidly connected to the NU-girders; however the damage scenario would correspond to twice the ductility demand in the fixed case due to the shape of lateral deflection on the column in the two modelling options (see insert in Figure 4-17). For the first scenario the translational period of the structure was found

to be 0.49s, however the period for the second scenario was 1.08s. This is consistent with evidence from the literature according to which usual values for the period of highway bridges varies between 0.5 s and 1.5s (FHWA 2014).



(a)



(b)

Figure 4-16 – Time History response for bent cap connection to girders with/out elastomeric bearing for
(a) record 1 and (b) record 2

In Figure 4-18, values of the maximum drift response of columns for eleven records and two different connection types of bent cap to NU-girders are collected. For all records, a reduction of column drift for elastomeric bearing is observed. It seems that in eighty percent of scenarios drift decrease is about 0.1%, except for the two last cases of low magnitude scenarios, where this values drops by about 0.2%.

In Figure 4-17-(a) the deformed shape of the bridge is shown for scenario 1, where connection of bent cap to NU-girders is monolithic, however abutments are connected to NU-girders through elastomeric bearings. This shape clearly illustrates the lateral force path. As noted, columns resist longitudinal forces, and the rotation release at abutments as well as elastic deflection of elastomers considerably reduce transferred forces to deck. Columns are fixed at both ends, and plastic hinges are formed at top and bottom of column as explained before.

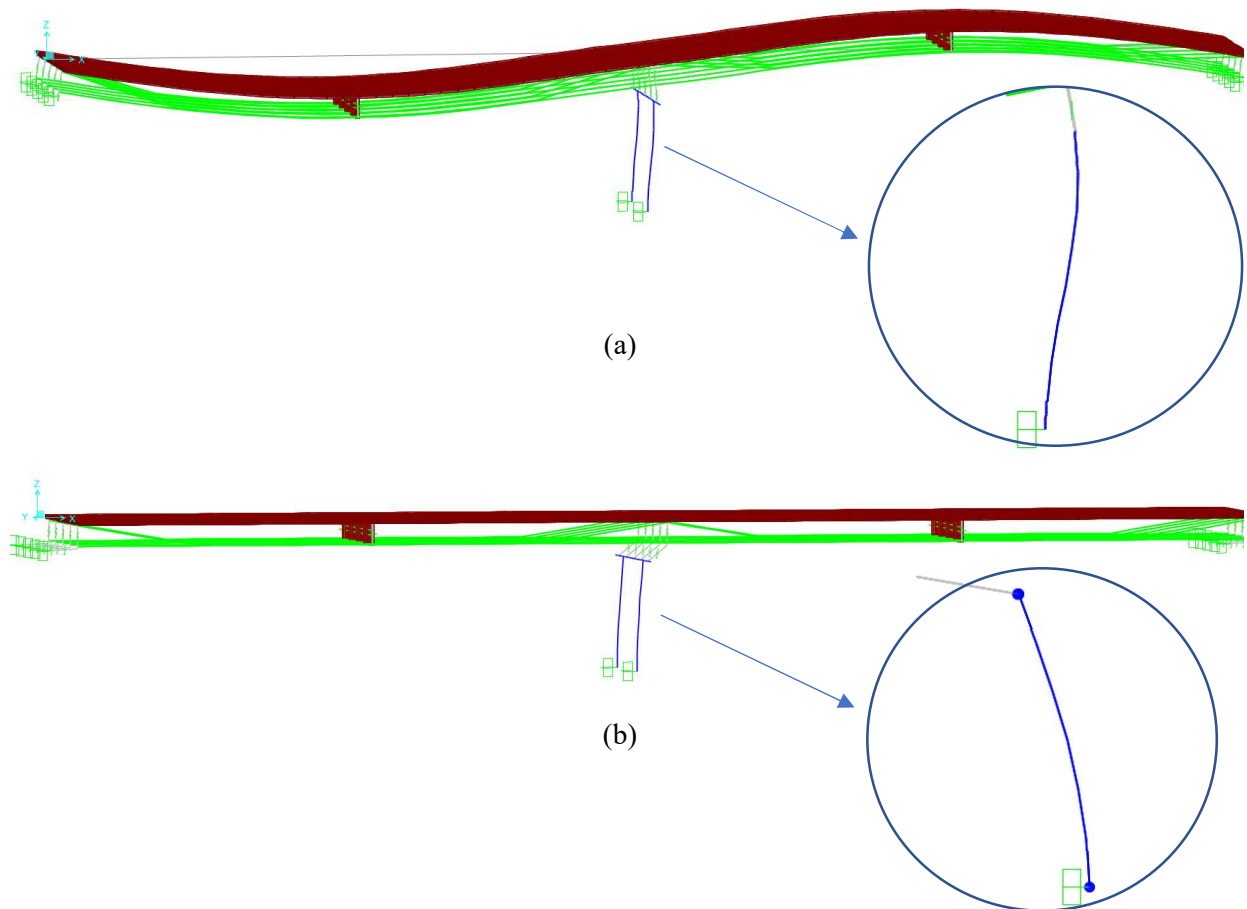


Figure 4-17 - Deformed shape - (a) Monolithic connection (b) flexible connection of bent cap to NU-girders

Curvature in the elastic case for (a) and (b) for equal lateral displacement Δ is as follows: Case (a): $\varphi=(6\Delta/H^2)$; Case (b): $\varphi=(3\Delta/H^2)$ → Note that the curvature demand in (a) is twice that in (b), leading to double ductility demand and damage level in the monolithic case as compared to the case with the elastomeric bearing.

Introducing of elastomeric bearing between bent cap and the NU-girders considerably reduced the force transfer from columns to the deck. Firstly, rotation of elastomeric bearing to the girders are released, thus no rotation is transferred to the deck. Furthermore, lateral displacements are sustained by the bearings. Meanwhile, a smaller deformation compared to the first scenario is induced to columns even for the same drift ratio on account of the different fixity conditions at the ends.

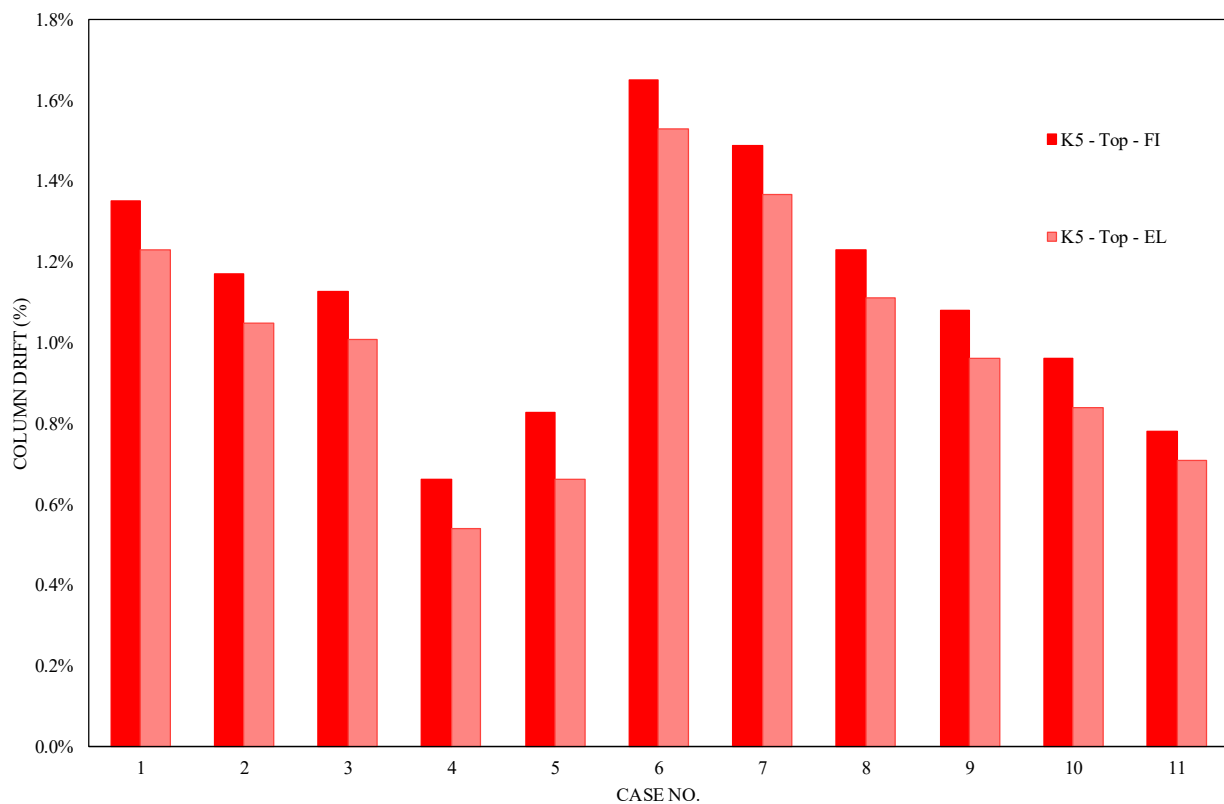


Figure 4-18 - Maximum column drift response - Fixed base bridge

CHAPTER FIVE: RESPONSE OF COMPLIANT BASE BRIDGE TO SEISMIC EXCITATION

5.1. Introduction

Different strategies are applied for seismic design of bridges. The corresponding configurations are categorized based on behaviour of bridge components in response to seismic excitation. For instance, in a reference guide three types of design scenarios were introduced. In the first scenario ductile piers resist the earthquake forces, while the deck remains elastic. The second scenario comprises a ductile deck with an elastic pier, and in the third scenario the superstructure consists of an elastic deck and elastic pier with fusing keys (American Association of State Highway and Transportation Officials 2002). Usually, design approaches are not mixed because the resulting response of the system can be complex so that common methods cannot be applied to estimate the seismic response and the sequence of failure may be less transparent for application of capacity design principles in such a situation.

A compliant foundation resists earthquake forces in sequence to the superstructure. The substructure can behave linearly elastic or can be ductile in response to seismic excitation. Due to interaction of foundation with the soil, it is likely that nonlinearity occurs even at very low intensity earthquakes. The approach in the current research is to account for the nonlinear response of soil as well as the ductility of the supported pier. The response of the substructure to lateral translations as would occur during ground excitation was studied extensively in Chapter Three. Accordingly, two methods of modelling the substructure were investigated. The first method used distributed nonlinear springs, attached to pile's skin, in order to represent the interaction with the soil. In the second method, the entire substructure was replaced with a lateral translational and rotational spring. In this chapter the first method is applied for piles at pier base as well as abutments.

Same as in the case of a bridge with fixed base, here, two scenarios are considered for connection of bent cap to the NU-girders (i.e., monolithic connection, or elastomeric seating). Meanwhile, four soil types were used as subsurface material. Thus, analyses were performed for four different substructure stiffnesses. Moreover, seismic analyses were carried out for five various damping values for the superstructure. Eleven simulated records were used for analyses. Results of 176 runs in terms of damage to pier columns will be subsequently discussed.

5.2. Modelling of substructure

Foundation of the selected bridge consists of a pilecap and eighteen piles at pier base, as well as abutments supported on eight H-piles at each end of the deck. Geometry and structural properties of the pilecap and abutments were adopted from prototype, explained in the previous chapter.

5.2.1. Modelling of pilecap

To model the foundation, the arrangement of piles at pier base were idealized to a symmetric distribution relative to the pilecap area, and accordingly the coordinates of each pile were specified. Since the software's bridge wizard can not generate these components automatically, the pilecap and piles were manually modelled. First, the pilecap was modelled with shell elements as an area object at the base of the pier. Particular attention was paid to the connection of pilecap to the columns. The nodal points of the shell were located on a plane at the mid-height of the pilecap. In the current case the pilecap thickness was 1200mm, thus the nodes were spaced at 600mm from top and bottom surface. To create a fixed connection between columns and pilecap, the last node at the bottom of the columns had to be kinematically connected to the shell nodes. Thus, the column's length was extended by a rigid end zone to the mid-height of the shell elements so that the base node would lie in the plane defined by the shell nodes. The same configuration was required for piles to form a fixed connection with the pilecap. Although in the prototype the embedment length of piles into the pilecap was 500mm, here the pile-head was extended by 100mm so that the highest node of all piles would lie in the plane of the shell-nodes. The pilecap was 3700mm long and 10000mm wide.

5.2.2. Modelling of abutments

The impact of soil pile interaction at the abutments and its effect on the superstructure's seismic response was also studied in this work. Therefore, abutments were modelled for two scenarios. In the first, the substructure was modelled as a partially restrained roller, with the lateral stiffness of an elastomeric bearing (i.e., rigid abutment behavior). In this scenario no SSI is considered for the abutment. In the second, a realistic arrangement was used to model the substructure. A thick shell was chosen to model the abutment. The Height of abutment was 3220mm by width of 10912mm and a thickness of 1200mm. Piles were embedded by 1610mm into the abutment shell (Figure 5-1), in order to create a fixed connection using "Body" constraints in CSiBridge. Supplemental lateral springs were attached to the abutment shell nodes, with the stiffness of elastomeric bearing, used in the first scenario to simulate the partial restraint to translation effected by the bearings.

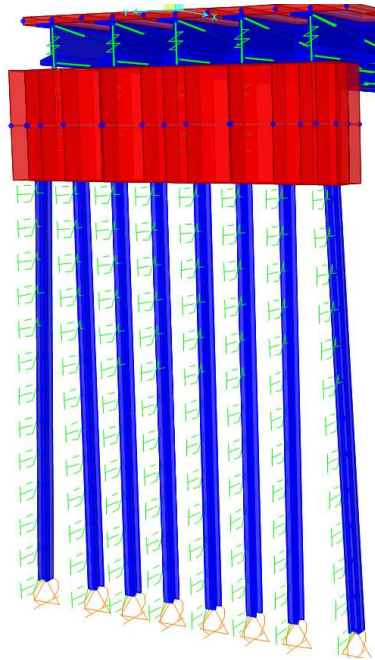


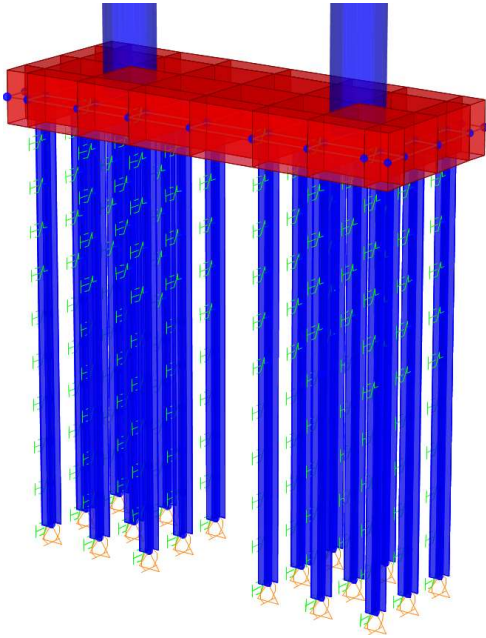
Figure 5-1 - Abutment established on H-Piles

5.2.3. Modelling of piles

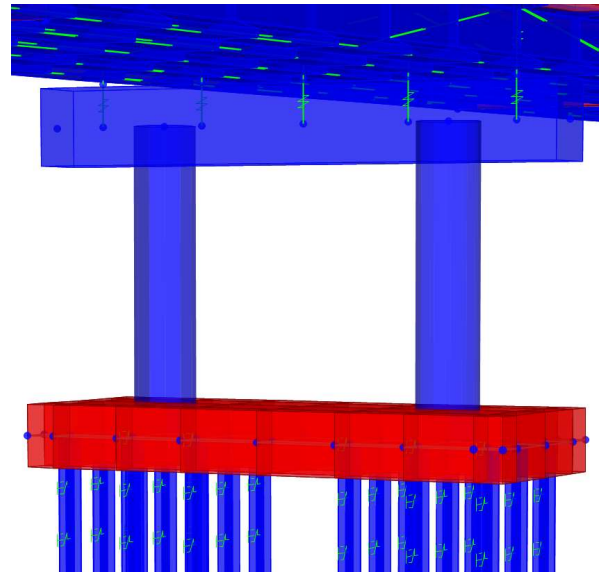
Piles were modelled as frame objects under pilecap and abutments. While the length of the piles at the selected bridge pier is about 4m, longer piles (10m) were modelled at pier base (Figure 5-2). The short length was due to the fact that in the area of Cornwall, bedrock is sedimented very close to the surface. Thus, short piles were sufficient in order to penetrate the bedrock. However, bedrock is usually formed deeper under the ground surface. To generalize the results of this study to a wider range of bridges, the deeper piles were also considered. Accordingly, in investigating that case, the lengths of piles at abutments were extended to 14m so that the elevation of pile tips at pier and abutments would be the same.

5.2.4. Simulating soil response to lateral movements

Soil reaction to the lateral movements is the essential aspect of modelling the substructure. As explained in Chapter three of this thesis, a method to account for soil foundation interaction while exciting the superstructure is to simulate the soil behaviour along the pile-skins by means of lateral springs. For this objective, single-joint links were attached to every meter length of each pile. A single joint-link behaves as a spring attached to a fixed support. The link was defined in the corresponding data sheet as a multilinear support type. The input property is given in the form of a force displacement envelope. The corresponding values were obtained from p-y curve output of program LPILE as shown in Figure 3-11, Figure 3-13, Figure 3-17, and Figure 3-18, for soft clay, stiff clay, loose sand and dense sand, respectively.

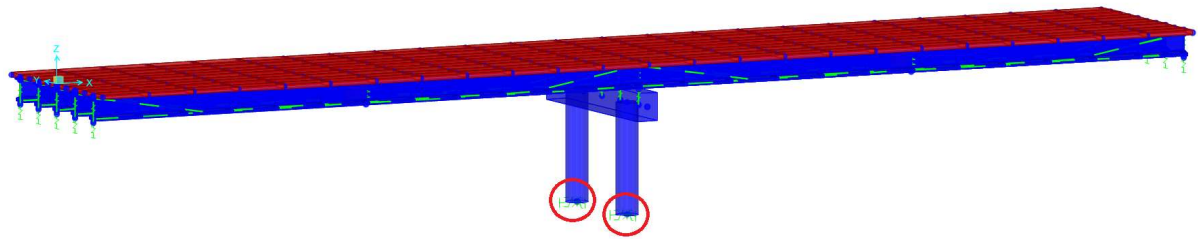


(a)

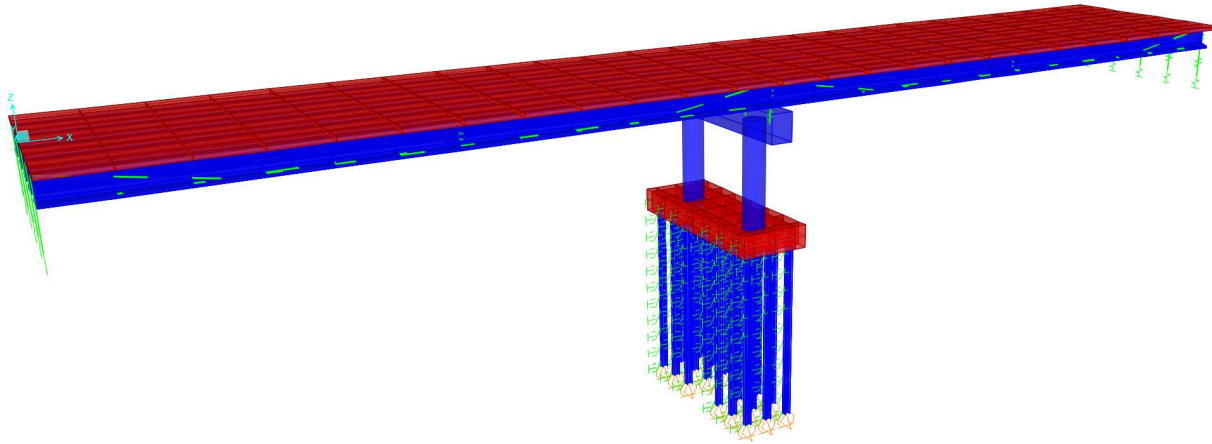


(b)

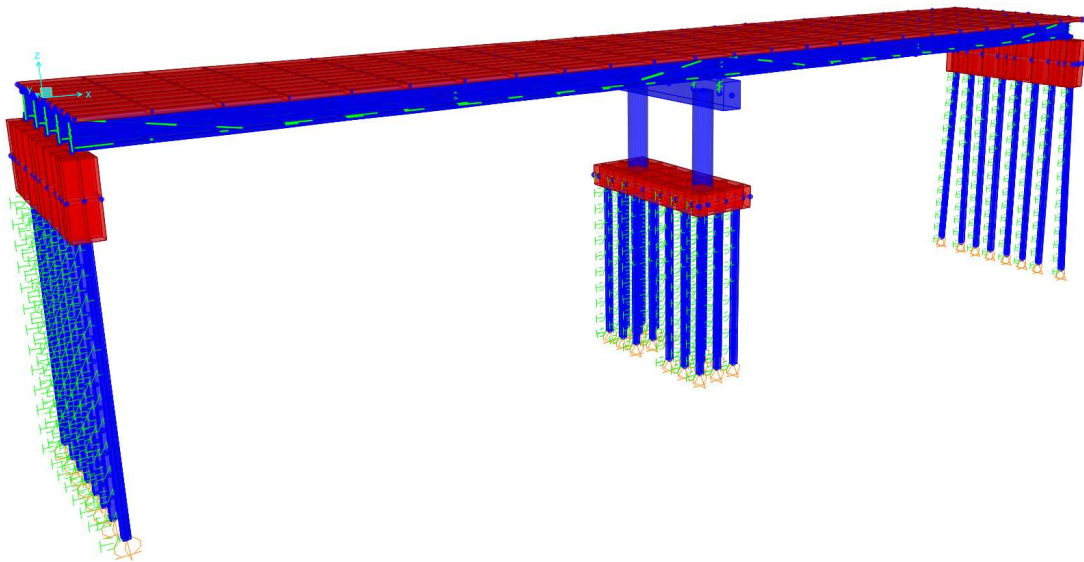
Figure 5-2 - Model of connection of (a) Columns, piles and pilecap at pier (b) Bent cap, columns and pilecap



(a)



(b)



(c)

Figure 5-3 - Compliant base bridge model with (a) pier supported on foundation springs and abutments on roller (b) pier supported on H-piles and abutments on partially restrained rollers (c) pier and abutments supported on H-piles

Force deformation curves depend on parameters such as pile depth for sand or diameter of embedded component underground as discussed in Chapter Three. Therefore, as required, several springs for each type of soil were defined to represent the proper soil behaviour at each row (pile elevation). Link hysteresis-type was set to kinematic as recommended for soil-structure-interaction application (CSiBridge 2017b). Note that for the kinematic type hysteresis consists of multilinear curves, which represent the nonlinear behaviour of material for each degree of freedom, in an independent manner (CSiBridge 2017b). The complete model of bridge with piles supporting abutments and pier as well as with roller at abutments is illustrated in Figure 5-3. Attached springs to piles are shown in green color. As explained before, a separate analysis had been carried out to generate soil force deformation reaction to lateral movement for the pilecap. Therefore, a specific spring that simulates soil response at the pilecap level was attached in the middle and at mid-height of the pilecap. It was assumed that the soil resistance to deflection of abutments is negligible. As a result, no link was attached to the abutments. It was also assumed that the soil under and around piles sufficiently resists vertical forces, therefore all pile tips were fixed against vertical movement. Current analyses were concentrated on longitudinal movement of superstructure, therefore transverse movement, and rotation about longitudinal axis of piles tips was restrained. As a result, the first mode of vibration is focused on the longitudinal displacement of the bridge.

5.3. Sensitivity analysis

5.3.1. Input parameter

As discussed before, substructure stiffness is dependent of several factors, where soil type is one of the most important parameters. For the purpose of sensitivity analysis, four uniform ground types were selected. In the forthcoming discussion, each substructure stiffness is indicated with a letter as follows: Identifiers K1 to K4 represent stiffness of substructure in the order of soft clay, stiff clay, loose sand, and dense sand. Geotechnical properties of soil types were elaborated in Chapter Three. Stiffness K5 corresponds to a fixed base.

Meanwhile, two connection types between pier bent and NU-girders were considered. Scenarios identified with letters FI represent the monolithic connection of bent cap to girders, whereas scenarios identified with EL stand for elastomeric bearing between girders and cap.

Furthermore, damping of bridge was varied in order to investigate the effect of damping on seismic response of superstructure while a compliant base is available. For this arrangement, stiffness K2 for stiff clay was adopted for all analyses, while the damping ratio was changed from 0.01 to 0.05. D1 shows the least and D5 the largest damping value considered (Figure 5-4).

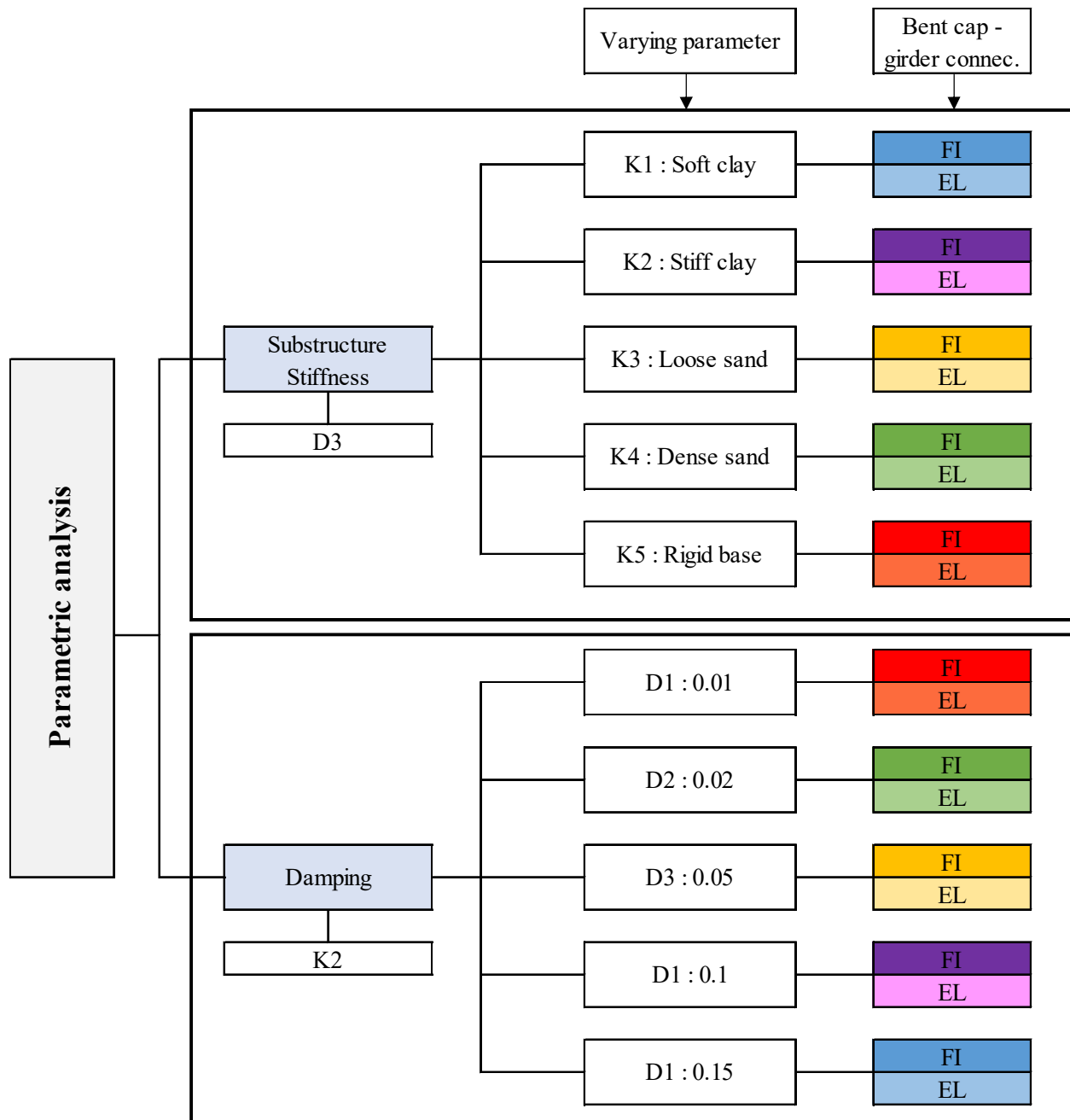


Figure 5-4 - Outline of sensitivity analysis on implication of soil structure interaction for foundation of a two-span bridge at pier and abutment

5.3.2. Variation of foundation stiffness

The selected bridge was excited with eleven simulated events for four different soil stiffness values. It was discussed in the preceding that with increasing foundation stiffness, the induced forces to superstructure also increase. In order to quantify the damage to the superstructure and particularly to the components that undertake the deformations (i.e., the pier columns and the elastomeric bearings), the peak values of column

drift ratios were calculated. Drift is generated by the relative displacement of column top to base, after adding the end support rotations that could relieve or intensify the column deformation depending on the direction of its action (Farantakis et al. 2014). The applied sign convention for pilecap rotation, and longitudinal displacement of pilecap and bent cap is illustrated in Figure 5-5.

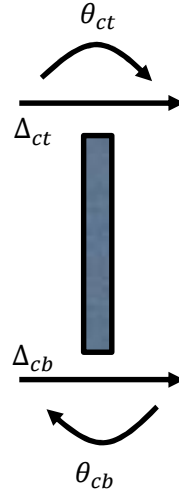


Figure 5-5 - Sign convention for displacements at top and bottom of pier

Based on the indicated deformation in aforementioned Figure 5-5, the column drift ratio is computed per Equation (5-1):

$$\theta_c = (\Delta_{ct} - \Delta_{cb})/H_{pier} - \theta_{ct} - \theta_{cb} \quad (5-1)$$

Where:

θ_c : Column drift ratio; H_{pier} : Height of Pier

Δ_{ct} : Pier top long. Displacement; Δ_{cb} : Pier bottom long. displacement

θ_{ct} : Pier top rotation; θ_{cb} : Pier bottom rotation

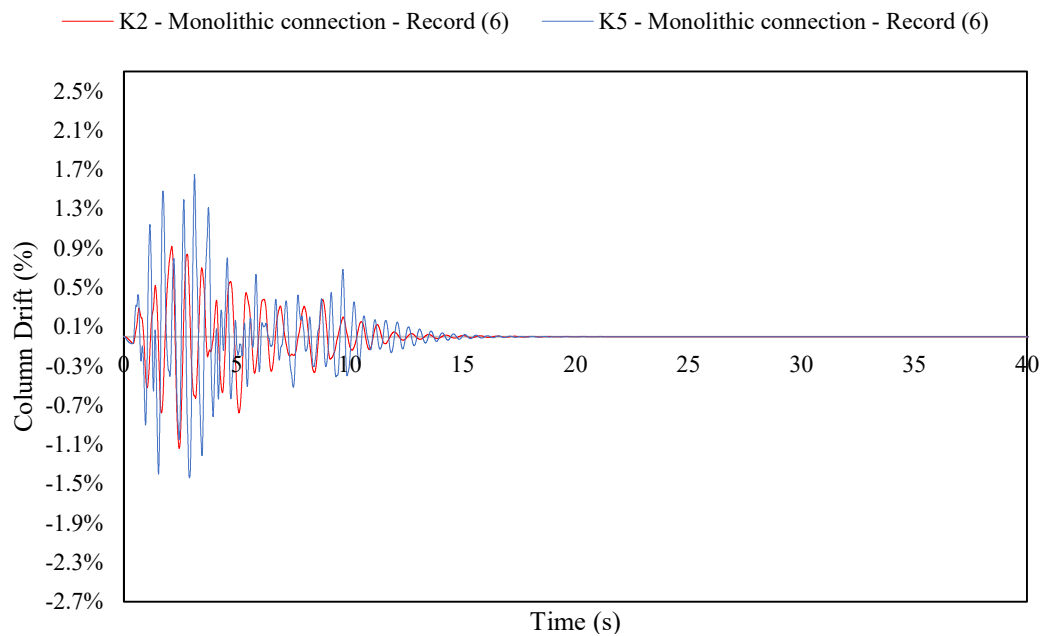
In the present study, pier column drift ratios were computed from the difference of top and bottom column displacement divided by column clear height, minus the rotation of the pilecap (see Figure 2-13). Table 4 lists the effective structural period for the first translational mode of vibration in correspondence with five different substructure stiffness values. Evidently, the structure oscillates with longer periods when the substructure has lower stiffness. This effectively increases the total displacement demand but reduces the deformation in the critical components of the superstructure; this is considered favorable, but under circumstances it could be considered problematic (P-D effects and user-anxiety if the permanent drift by naked eye is visible in case of large permanent drifts for example). However, moderation of the phenomena

is key to the design; this is why adaptation of this procedure in practical design ought to be accompanied with acceptable limits to the magnitude of the pile-rotation (i.e., performance limits).

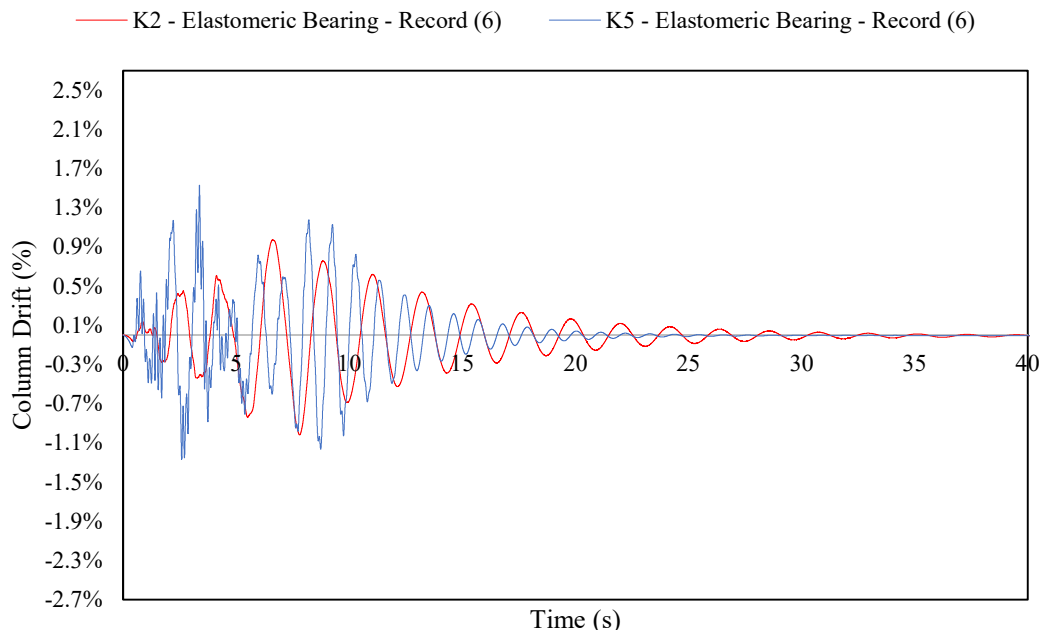
Table 4 - Period of vibration in correlation with substructure stiffness

Substructure Stiffness	Fixed connection of bent cap - girder					Elastomeric bearing on bent cap				
	K1	K2	K3	K4	K5	K1	K2	K3	K4	K5
T (s)	0.81	0.76	0.73	0.72	0.49	2.59	2.18	1.92	1.73	1.08

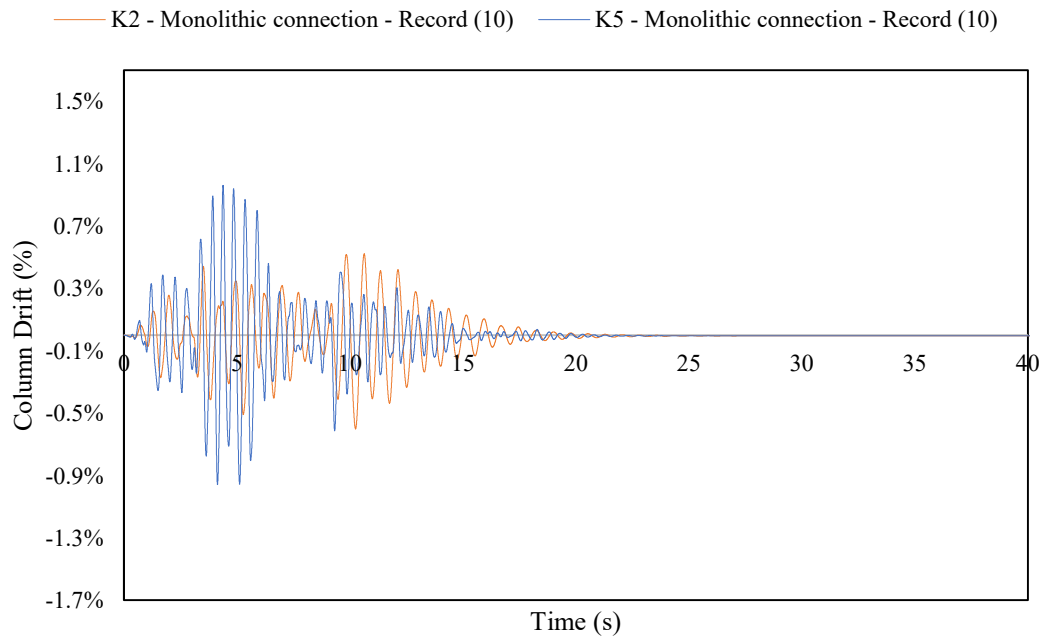
Estimation of extent of damage to soil could be performed by detailed analysis of the pile displacement at several elevations, especially at those elevations where nonlinear springs were defined to represent soil behavior. Drift time history responses of columns for two records and two foundation stiffness are shown in Figure 5-6. Also, the pilecap rotation time history is plotted in Figure 5-6 for the pier with flexible foundation. K2 is related to the embedded piles in stiff clay and case K5 represents a fixed base bridge.



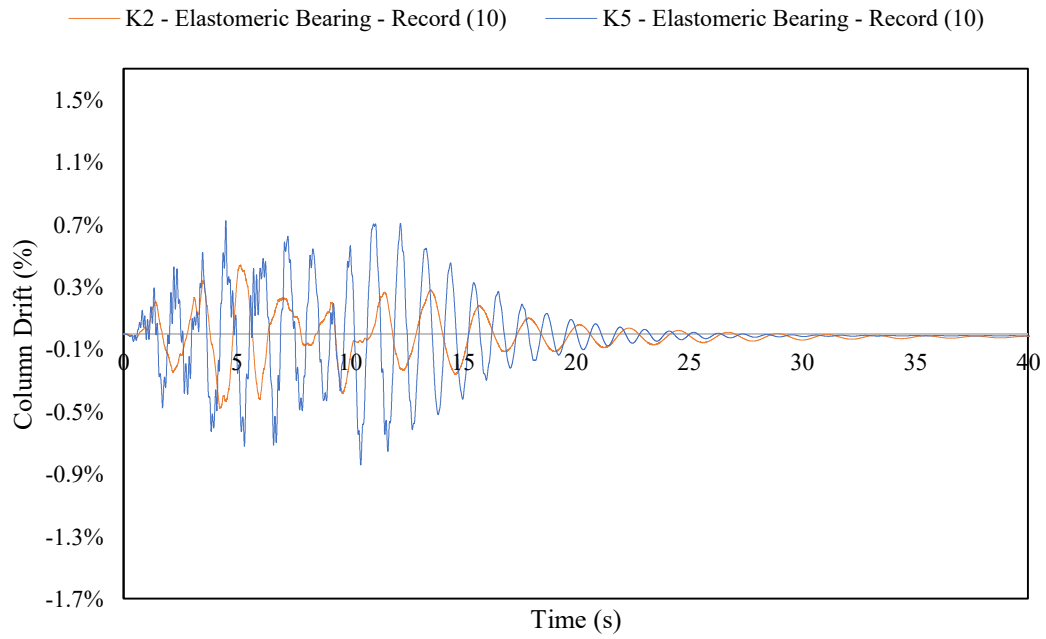
(a)



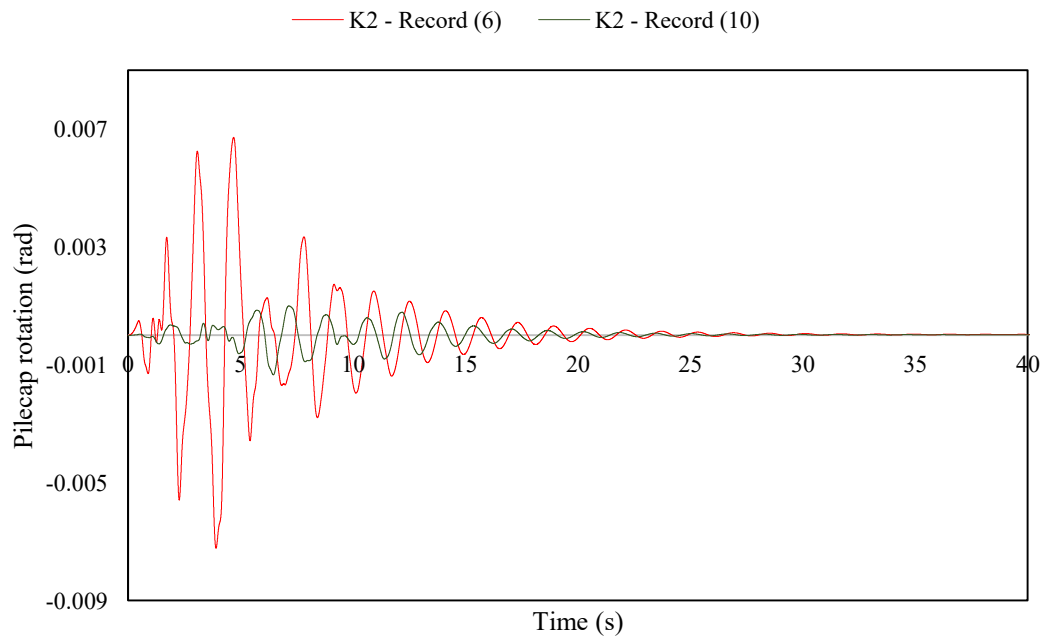
(b)



(c)



(d)



(e)

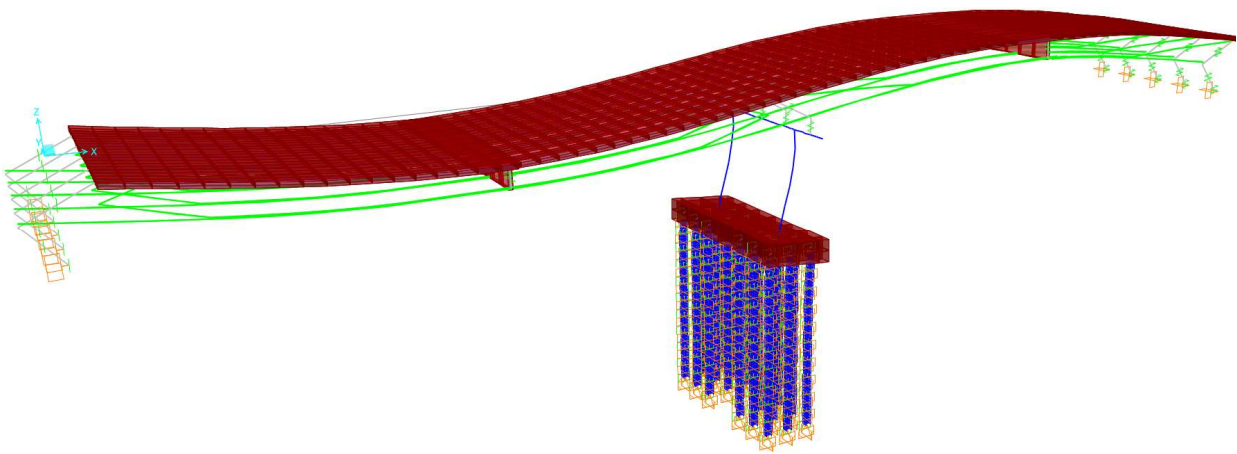
Figure 5-6 - Column drift response time history response for stiffness K2 and K5 for (a) bent cap connection to girders without elastomeric bearing for event #6 (b) with elastomeric bearing for event #6

- (c) bent cap connection to girders without elastomeric bearing for event #10 (d) with elastomeric bearing
for event #10 (c) Foundation rotation for the foundation stiffness K2 for event #6 and #10

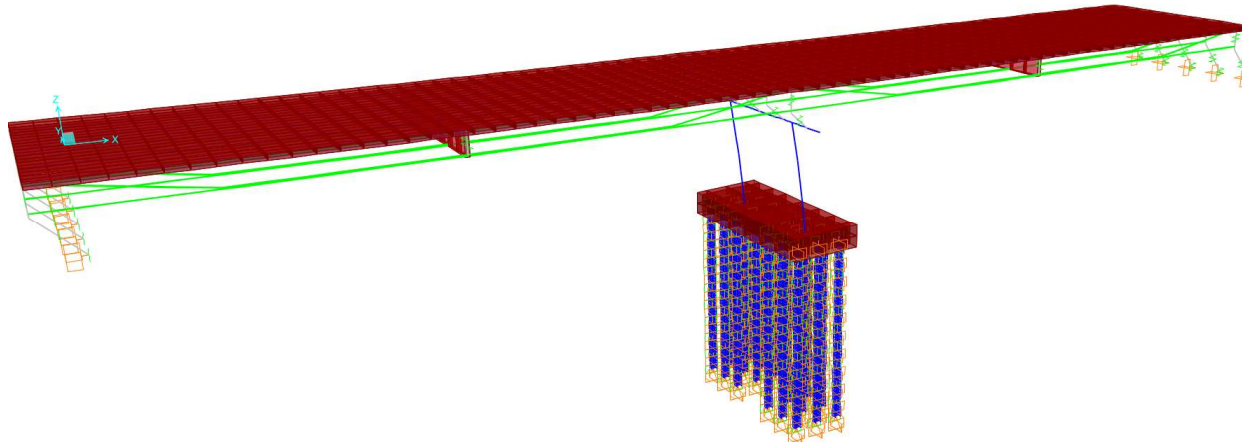
In Figure 5-7 the deformed shape of bridge for the maximum column top displacement for substructure stiffness K2 is illustrated. These shapes are representative of all deformed shapes obtained from the sensitivity analysis of this chapter. It appears from the deformed shape for monolithic connection of bent cap to girders that rotation of columns caused bending in deck. Pilecap has rotated and displaced horizontally, and lateral displacement and rotation of deck at supports is evident. Comparing the deformed shape of the monolithic bridge with that of the bridge on elastomeric bearings suggests that for the latter case no bending is transferred from columns to the deck. Also, shear forces at deck are sustained by elastomeric bearings at bent-cap and supports.

Comparison of maximum column drift for four compliant base bridge and one fixed base with two connection types between bent cap and girders in

Figure 5-8 shows that the maximum drift occurs for pier based on rigid footing. In general, maximum drift is smaller for scenarios with softer soil and elastomeric bearing on bent-cap. For example, the excited bridge with event 6 and a fixed base shows maximum drift of 1.64%, whereas the bearing type on bent-cap is fixed. However, for the same event the induced drift ratio in the pier columns drops to 1.47% for K4, 1.38% for K3, 1.14% for K2 and 0.78% for K1. It is observed that the magnitude of drift reduction for softer ground is not consistent between cases. For instance, from K5 to K4, the difference between induced drift ratios is 10.4%, whereas between K2 and K1 the maximum difference is 31.6%. This proves that that estimation of pier seismic response for compliant base bridge is contingent upon knowing geotechnical properties of soil and subsequently stiffness of foundation, as well as other factors discussed before.



(a)



(b)

Figure 5-7 - Deformed shape - (a) fixed connection (b) flexible connection of bent cap to NU-girders for compliant base pier

Further analysis of results illustrates that maximum column drift ratio is also dependent of the bearing type between bent cap and girders. For event number 6, column drift ratio drops by 15.4% for the case of elastomeric bearing compared to a fixed bearing and stiffness K1. Apparently, the variation in the values of column drift for the different bearing types for all substructure stiffness cases ranges from 0.09% to 0.26%. However, for over 90% of all cases drift reduction due to stress relief at elastomeric bearing is about 0.12% - that means that it is much more significant for the bridge response to engage the foundation than to relieve the monolithicity at the top of the pier support by means of introducing an elastomeric bearing: note that a reduction by 50% is observed between the fixed base and the most compliant base case

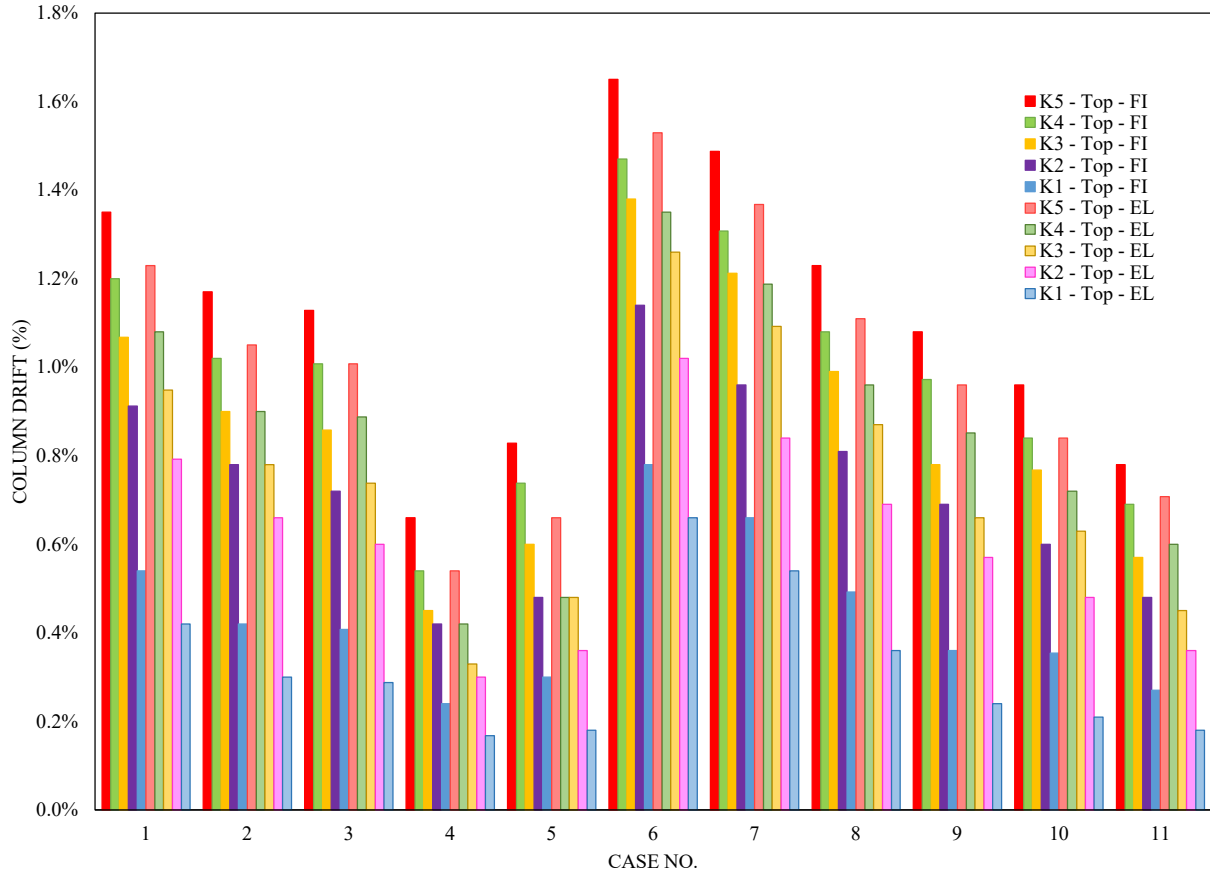


Figure 5-8 - Maximum column drift ratio due to longitudinal movement of bridge with variable substructure stiffness

5.3.3. Variation of damping

Maximum column drift ratios for different values of damping are illustrated in Figure 5-9. Evidently, lower damping has resulted in larger relative displacement at top and bottom of columns. For the events #6, #7, #1, and #8 with a fixed bearing at bent-cap and initial damping ratios of 1% and 2%, the highest drift demand on pier was observed. For damping of 5%, 10% and 15% a large drift reduction happens in comparison to the two values of damping considered initially. This reduction ranges between 5.5% and 67.5%. Meanwhile the variation of drift for the latter values of damping considered and for all eleven events ranged between 0.0% and 38.5%. It is concluded that for a pier with fixed connection of bent cap to girder, the induced drift ratio is sensitive to seismic events with larger ground acceleration and low damping in the range of 1% to 2%.

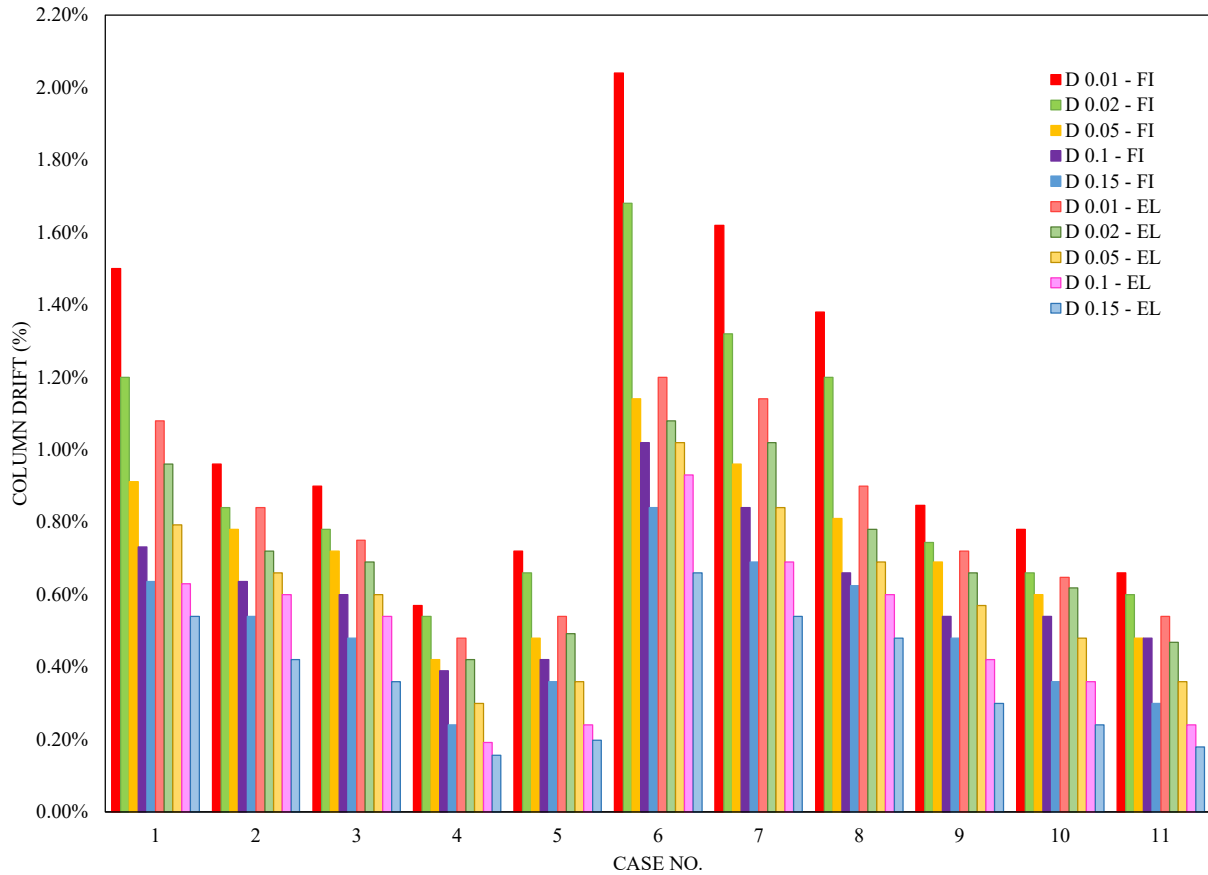


Figure 5-9 - Maximum column drift due to longitudinal movement of bridge with different damping ratios

In general, the use of elastomeric bearing at bent-cap helps to mitigate damage to the pier columns. It appears from Figure 5-9 that for the event number 6 the largest drop of drift demand occurs between the two scenarios of monolithic and elastomeric connection between pier-cap and the deck for 1% and 2% damping. This trend is also seen for other large intensity events like #1, #7 and #8, ranging between 20% and 35%. With increasing damping, the difference drops to a range of 9% to 17.8%. For scenarios with elastomeric bearing connection, it is understood that column drift ratio is not sensitive to damping as explained for the fixed bearing. A gradual reduction of drift is observed from lowest damping to the highest for each event, which ranges between 4.6% and 36%. It is concluded that by adding elastomeric bearing in the cap-beam to deck connection, it is possible to minimize the effect of lower damping on drift demands at higher intensity seismic events, compared to the scenarios with a monolithic connection.

5.4. Contribution of compliant base abutments

The model used for sensitivity analysis consisted of piers supported on H-piles, while abutments were supported on elastomeric bearings connected to a roller. This arrangement provided firm support against vertical movement and partial fixity against lateral movement, while rotation of the deck at supports was

unrestrained. Indeed, the abutments of the case-study are supported on piles with penetrated tip in bedrock, and these conditions may be considered as a fixed support. However, piles at abutments can also resist the seismic forces in the same manner as in the pier base, provided that they are flexible or more accurately, supported in compliant soil. In order to examine the effect of compliant base support at both ends of deck (Figure 5-3-(c)) on seismic response of pier columns, abutment and H-piles were modelled (explained in Chapter 4), and excited with eleven spectrum-compatible artificial records for embedded piles in soft clay, which correspond to substructure stiffness type K1. For instance, in Figure 5-10, the time history response of two different abutment arrangements (Figure 5-3-(b) and (c)) to seismic event #6 is plotted. From the plot and deformed shape of piles (Figure 5-11) it is evident that resistance of piles at abutment against longitudinal movement of deck provides additional shear resistance at supports to that of the elastomeric bearing.

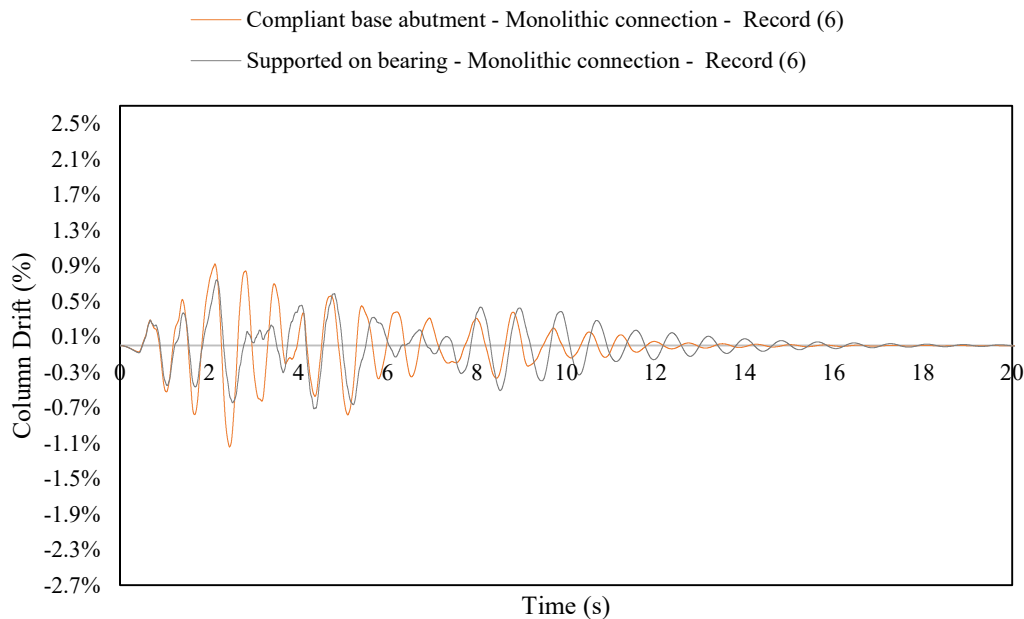


Figure 5-10 – Comparison of column drift history response for bridge with abutment supported on roller bearing and with abutment supported on piles

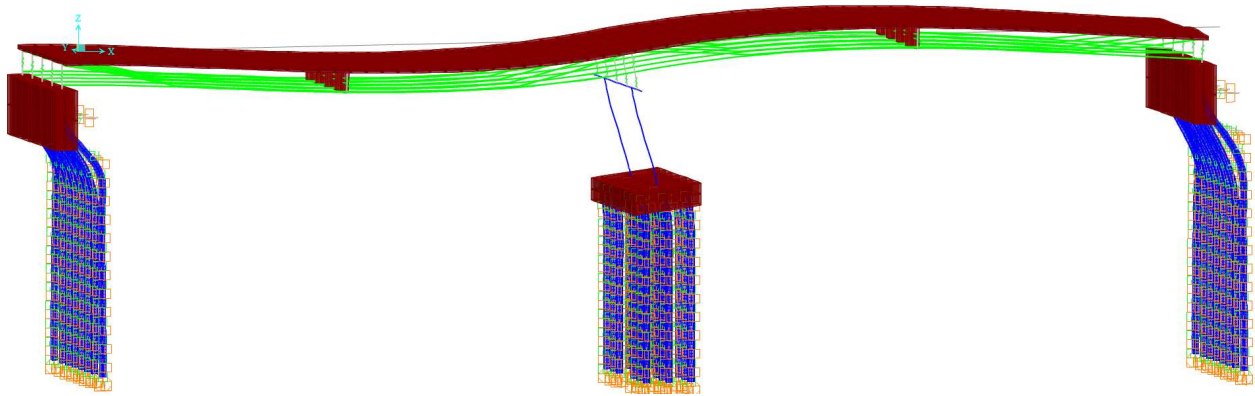


Figure 5-11 – Deformed shape of bridge with abutment supported on piles

The maximum drift demand for the pier columns for the case considered where abutment-piles form a compliant end support are summarized in Figure 5-12. It is observed that contribution of compliant base abutment in resisting the seismic excitation considerably increases the maximum column drifts. The increase of damage is larger for events with larger ground acceleration such as event #6, #7, and #1. In general, the maximum drift values were decreased between 24% to 42%. In this case a corollary is also worth noting: Where the abutment compliance is unavoidable, a fixed base pier would undergo significant drift demands and consequent damage.

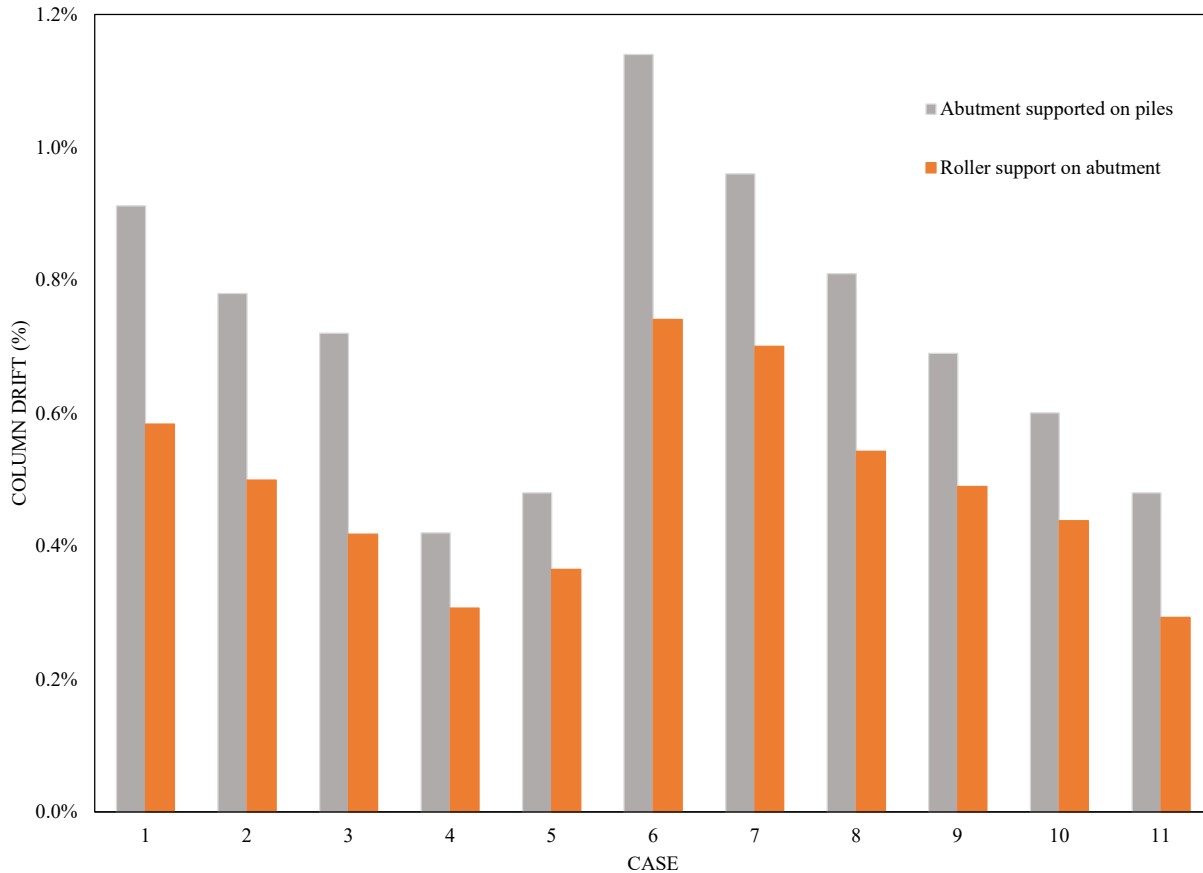


Figure 5-12 – Maximum column drift response for bridge with abutment supported on elastomeric bearing and with abutment supported on embedded piles in soft clay

5.5. Comparison of substructure modelling methods

Additional trials were performed to compare the effect of modelling methods of substructure at bridge pier. Previously, the substructure was modelled at the pier location by means of steel H-piles and soil was simulated as attached springs along pile skin (Figure 5-3-(b)). This method was termed as “substructure method with springs at pile skin” according to the literature (Rahmani et al. 2016). Another approach for modelling the foundation was termed “substructure method with springs at base”. In this method substructure is introduced to pier foundation as a lateral and a rotational spring. Stiffness of the springs are representative of substructure overall stiffness. Force deformation correlations obtained from GROUP analysis in previous chapters for a set of 9 piles and half of pilecap length were used to define the required spring properties at the base of each pier column. For this purpose, in the Link/Support Property Data form of the program CSiBridge, multilinear plastic links were defined for translation along the longitudinal direction and rotation about the transverse axis (i.e., orthogonal to the longitudinal direction). Modelling

was carried out for stiff clay, constant damping of 5%, and a fixed connection of bent-cap to NU-girders; time history analysis was carried out for the eleven selected spectrum-compatible events.

In Figure 5-13 seismic response of column to event 6 for both methods is shown. It was observed that maximum induced drift in pier for modelled foundation with springs at base is comparable but smaller than the modelled foundation with attached springs to pile skin. This was also the case for all other seismic events (Figure 5-14)

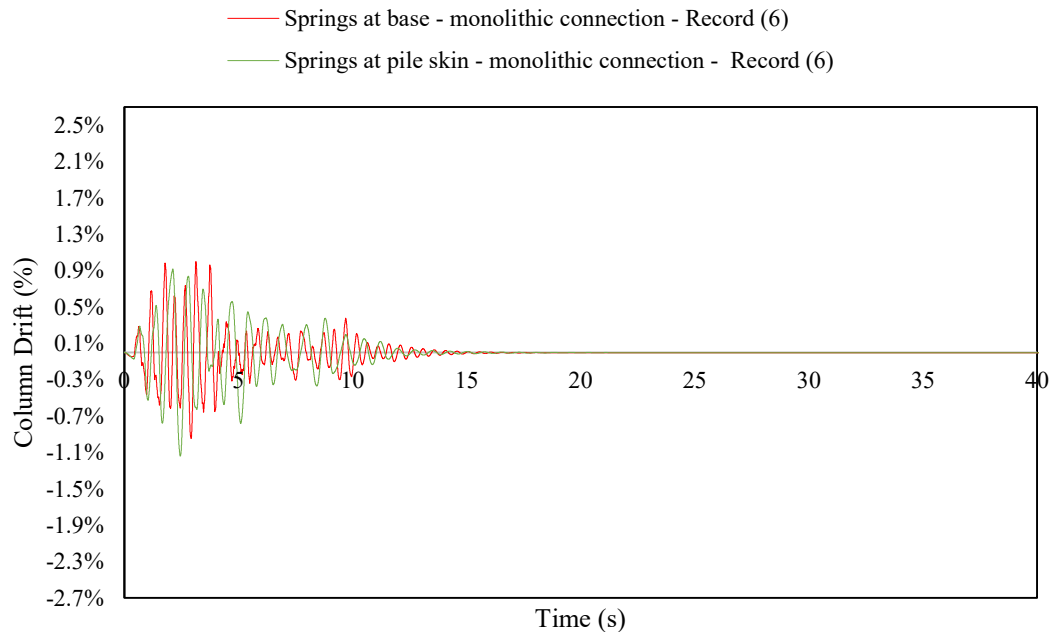


Figure 5-13 - Comparison of column drift history response for substructure method for overall springs attached to the base and springs attached to pile skin

A reason for the slightly lower demand in pier for the substructure method with springs at base could be the fact that the two coupling components of the stiffness matrix were not added to the model. These two components are lateral stiffness due to pilecap rotation as well as rotational stiffness due to pilecap lateral displacement. It is reminded that this is a limitation of the software (that coupled springs cannot be introduced) – as independent moment rotation and force – translation relationships would be input for the rotational and translational springs, respectively. Therefore, it is concluded that the application of the substructure method with equivalent springs might underestimate the magnitude of damage to the superstructure. However, a choice of preferred method of modelling in practice could be left to the designer due to negligible difference between the results from analysis of both substructureing methods, and high

degree of effort pertinent to modelling the deep foundation, as well as soil response springs at several elevations along the pile skin.

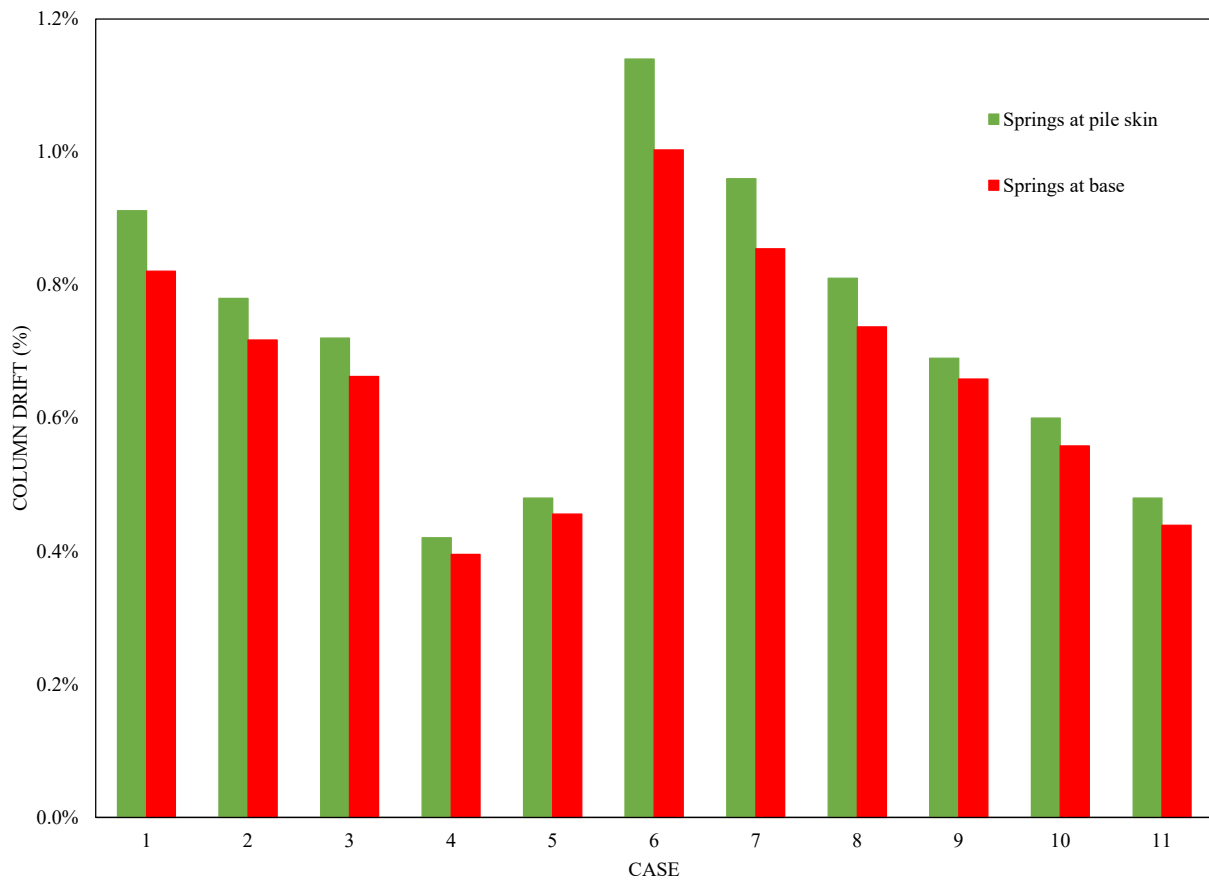


Figure 5-14 - Maximum column drift response for two types of substructure method - embedded piles in stiff clay

CHAPTER SIX: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1. Background and motivation

This thesis explored the effect of foundation soil compliance in partly alleviating deformation demands in the piers of bridge systems and essentially functioning as a mechanism of damage-mitigation during severe seismic events. This phenomenon has been first reported in instrumented short bridges in California (Meloland Road Overcrossing – MRO, and Painter Street Overcrossing - PSO) which were located near El Centro and were subjected to the Imperial Valley Earthquake in 1979 (ML=6.6, PGA = 0.32g). It was found that for the intensity of this earthquake the damage of the piers was much less than expected, particularly considering that these structures were lightly reinforced (based on former standards).

The extraordinary response of these bridges, demonstrating operational level damage where much more severe demands were estimated triggered an extensive investigation in the bridge community trying to understand and possibly benefit from these observations in practical bridge design. It was first found that owing to the shallow foundation of the abutments on the relatively soft embankment soil deposits, the motion of the bridge was controlled by the response of the end embankments. This would render the intermediate pylons susceptible to the displacement demands imposed by the embankments, in a “follower” type behavior (a follower component is driven in displacement by a controlling part of the structure – here the embankments). However, the response of the pylons in both the MRO and the PSO, also being founded on compliant soil with relatively short piles, was compliant to the applied overturning moments and lateral forces transferred, thereby alleviating, through uplift and deformation of the soil surrounding the piles, part of the resistance and the fixity condition at the base of the piers. Farantakis et al. (2014) explored further this behavior and found that the greatest effect in reducing the damage to piers is the rotational response of the pile-head, since a rotation by 1 degree ($=\pi/180$ radians), accounts for a lateral displacement of $H \cdot \pi/180$ at the top of the pylon, where H is the height of the piers; thus, of the total displacement demand, Δ , that the pier is subjected to, only the difference $\Delta_{\text{pier}} = \Delta - (H \cdot \alpha_{cb} \cdot \pi/180)$ would be undertaken by deformation, where α_{cb} is the rotation of the pilecap in degrees.

This concept has been the motivating consideration of the present research. Several bridges are now instrumented in north America to further investigate the SSI effect and how it may be exploited in better design of structures – with the potential to mitigate significant damage to the bridge superstructure in the ensuing earthquake, particularly in the case of seismic assessment of existing bridges. Note that this approach could be criticized as less conservative than a stiff foundation that allows no damage to the soil, on account of the fact that foundation is considered a higher priority in terms of accessibility for repair in a bridge structure.

Application of compliant foundation requires careful studies in terms of failure due to severe deflections. Excessive foundation movements can cause unnecessary and unrepairable damage to the underlying and surrounding soil. This damage can cause failure of foundation and superstructure as a result, while superstructure is still intact. Therefore, it was argued in the present work that a reasonable solution is needed to involve the foundation in resisting seismic forces in addition to earthquake resisting components of superstructure, such as the piers. Thus, incorporating actively the SSI effects in bridge seismic assessment or design is therefore explored as a method for staged or staggered acceptable performance, where allowable damage for foundation soil and superstructure are adjusted in reference to the intensity of the ground motion. This led to the proposal by (Pantazopoulou et al. 2014), where performance criteria were established for the average soil shear deformation and pier drift under three different performance limit states (operational or damage-limitation, significant damage, and life-safety).

6.2. Methodology developed

Through Virtual Work Principles expressions were derived for the effective stiffness of the foundation and pier system. With reference to elastic systems, it was shown that the foundation and pier behave as a system of springs in series. To derive an expression, it was first necessary to calculate the lateral normalized response shape of the system (e.g. the fundamental mode from an eigenvalue analysis or a simple lateral pushover analysis). Thus, the effective stiffness is equal to the deformation work that is stored in the foundation-pier system when a unit displacement occurs at the top of the pier (translational, in the direction of the earthquake). This work is the sum of work done in each sub-system (here, foundation, pier, or even elastomer bearing at the top). By definition work of deformation or strain energy of each member or subcomponent is equal to the product of the individual member stiffness times the deformation occurring in the normalized lateral response shape of the system within the ends of the member, squared. In this manner the effective lateral stiffness of a pylon on compliant foundation comprises additive contributions of the lateral stiffness of the foundation and the lateral stiffness of the pier. Having explicitly introduced the stiffness of the foundation in the pylons' effective stiffness, it is evident that the structural period and corresponding seismic demands are affected.

The resistance and stiffness of the foundation system to applied lateral translation and rotation at the point of contact with the pier (i.e., at the base), depends on the relative stiffness of the piles and the surrounding soil. To quantify proper values by which to establish the composite pylon stiffness, two common methods of analysing SSI problems were considered, namely substructureing and treating the system as a continuum (i.e., solving the governing differential equations of the problem along the piles that support the pile-head). The latter case was deemed suitable for application in the present study, as it provided the opportunity to vary substructure geometry and material and enabled detailed study of those parameters on the seismic

response of the substructure considering soil-foundation-interaction. In the substructuring method the structure is separated into the superstructure and the substructure. The substructure is excited with free field motion such as the waveform of an earthquake event. Subsequently, the response of the substructure is idealized at the base of the superstructure using springs with nonlinear stiffness. Two approaches were elaborated for modelling of the springs: 1) using equivalent springs that represent the overall compliance of the foundation at the base of the superstructure or 2) discrete springs attached to the piles' skin. After incorporating the springs, the superstructure is excited with the same free field ground motion.

The failure mechanisms of piles and columns due to dynamic movements were studied and interpreted through nonlinear analysis. Results pertained to the location of the plastic hinges and the significance of the magnitude of plastic deformation in piles and pier columns under lateral pushover analysis of SSI problems. It was understood that optimizing the deformation of pile head in favor of reduction to column displacement demand in response to seismic excitation is an iterative process. Excessive pile head deformation can cause failure of foundation or early (i.e. at low intensity motion) exceedance of the serviceability limit state.

Several parameters influence behaviour of deep foundation and superstructure, which underscores the importance of sensitivity investigation on most important variables. Parameters are pile geometry such as length, and section, pile material, axial load imposed on pile head, and underlying and surrounding soil. In this work emphasis was placed on parameters more relevant to SSI that control the mobilization of soil resistance, such as pile geometry and axial load. A sample bridge from east Ontario was selected and adopted to the study for exploration of the concepts with reference to a realistic case. Superstructure and pile sections were adopted for the analyses. Extensive sensitivity studies were performed to understand the effect of above-mentioned variables on the response of a pile and pilecap to a cyclic lateral displacement, imposed on the pilecap, as well as a cyclic rotation in the direction of earthquake action. Four soil types, namely: soft clay, stiff clay, loose sand, and dense sand, were used for analysis. In addition, mixed soil layers from clay and sand were utilized for further extension of the sensitivity analysis. From the performed pushover analysis in program LPILE the overall response of the foundation arrangement was quantified in terms of the secant stiffness versus pilecap displacement and rotation. Considering that the actions (i.e., the developed moment and shear forces at pilecap due to induced displacement) can be expressed as product of the stiffness matrix by the deflection vector, four equivalent stiffness entries were defined: These were, the translational stiffness associated with lateral displacement, and the rotational stiffness associated with pile-head rotation, as primary terms of the equivalent stiffness matrix at the pylon base representing the SSI effects. Coupling terms in the off diagonal linking translational forces with the rotational degree of freedom and vice-versa are often neglected in practical analysis where separate equivalent springs (translational and

rotational) are used at the base of the pier to model SSI; however these terms were estimated in the study so as to also give a measure of the error resulting from neglecting of these coupling effects. Nonlinear stiffness curves obtained facilitate accurate simulation of foundation seismic response at base of the superstructure, over the typical application of elastic stiffness in practice.

6.3. Results and conclusions

Larger axial loads on pilecap reduced the effective stiffness of the foundation with increasing lateral displacement and pile head rotation for all seven soil combinations. Sensitivity to increasing axial load for pilecap displacement and rotation was different. Stiffness reduction for embedded piles in different types of soil also varied. For instance, amplifying the axial load from 1000 kN to 2000 kN had no effect on the stiffness of embedded pile in soft clay for displacements larger than 35 cm, whereas the stiffness of embedded pile in stiff clay almost dropped to half, for the same Axial Load and displacement magnitudes. It was concluded that for each soil type and axial load the cause of strength loss should be identified, i.e., whether the pile fails first or soil, and the same is needed for the cause of failure. Subsequently, three different pile sections were tested with the fixed axial load magnitude and variable soil types as done earlier. Generally, larger pile diameters sustained large inelastic displacements showing a milder rate of stiffness reduction. For the selected soil properties, cohesionless soil demonstrated larger stiffness over cohesive ground. This conclusion was also relevant for increasing the Axial Load on the pilecap. Another scenario considered was exciting piles with different embedment lengths. It was observed that longer piles have larger stiffness against imposed displacements. It is known that for elastic deflections, longer columns have smaller stiffness. It was concluded that deflecting the head of an embedded pile mobilizes the soil resistance. Thus, the longer the pile embedment, the larger soil resistance is mobilized. Therefore, lengthier piles correspond to stiffer foundation and reduced SSI. Another observation referred to the rotation of pile tip. The tip of longer piles tends to rotate at larger imposed displacements at the pile-head, as compared to shorter piles. This can alter the boundary conditions and therefore affect the stiffness of the pile. Additionally, p-y curves (as an indicator of soil resistance p , to lateral deflections y), were introduced and generated for four soil types at different depths of piles. Relevant empirical equations were given, and their important variables were discussed.

Likewise, pushover analyses were performed on a group of piles and pilecap in program GROUP for four different soil types. Force-displacement curves were generated for further modelling of foundation springs at the base of the superstructure. Recommended reduction factors for interaction closely located piles were addressed.

The structural components of selected bridge superstructure considered as a case study in the research, was also used to conduct a sensitivity analysis. Deck, NU-girders, pier cap and columns were modelled in the computer software CSiBridge, whereas several idealization details were addressed in the model. Considering that the primary earthquake resisting component of the analysed bridge is the central pylon that comprises two piers with a cap-beam, two types of connection between bent cap and NU-girders in the longitudinal axis of the bridge were examined in the sensitivity investigation. These alternatives are meant to consider vastly different translational stiffness of the pylon in the longitudinal direction of the bridge by using a monolithic connection in one case, and elastomeric bearings with a finite translational stiffness in the other case. Abutments were modelled using elastomeric bearing seats with finite longitudinal translational stiffness. Plastic hinges were added to top and bottom of columns for nonlinear time history analysis. Eleven design-spectrum compatible time histories of synthetic ground motions were generated from simulated records and in accordance with recommendations of NBCC for soft soil conditions. From the time history analysis, it was concluded that connection of bent cap to girders through elastomers result in 0.1% reduction of drift demand in the pier, but the magnitude of the moments transferred to the foundation are significantly less. Seismic analysis of rigid base bridge established the benchmark for compliant base analyses.

For the compliant base bridge, accurate modelling of foundation in the software was a critical milestone in analysis of the bridge system. First, H-piles and pilecap were modelled using soil springs attached to the piles' skin. Soil springs were assigned the force- displacement values of the p-y curves calculated earlier for soft clay, stiff clay, loose sand, and dense sand for each analysis. As was shown from the LPILE nonlinear analysis, the force displacement values varied with depth along the pile length. Subsequently, time-history dynamic analyses were carried out for each of four compliant substructure scenarios independently. Again, two types of connection between bent cap and girders were considered (i.e. monolithic or through elastomer).

A main conclusion of the thesis is that with increasing stiffness of substructure the share of drift ratio demand developed in the pier increased. Pier drift was maximum for the fixed base structure. It was also shown that for higher intensity events the effective reduction of pier drift ratio is larger for cohesive soil types. But a fixed order of magnitude for the relative displacement reduction could not be determined conclusively. Connection of bent cap to girder by elastomers resulted in further reduction of imposed drift ratios in the pier, but significantly increased the period of vibration. Generally, for soft soils the ground motion had long period on the surface and the likelihood of resonance increases. Therefore, it is reasonable to recommend that selection of the bridge structural system should either combine a compliant substructure

with rigid connection of columns to the deck or alternatively, the use of elastomeric bearings with a stiff or even rigid substructure. This would avoid damage to deck and abutments due to excessive displacements.

In addition to substructure stiffness, the effect of damping of the system on seismic response of the pier was investigated, with five different values of damping for the case of the compliant base bridge. It was observed that lower damping resulted in larger relative displacement between top and bottom of the pier (i.e. the pier drift ratio increased).

In an alternative model, abutments were modelled using reinforced concrete shell elements, supported on H-piles embedded on embankments. Same as in the case of the pier foundation, soil springs were added along the piles' skin, assuming stiff clay soil. This modelling approach added soil resistance to the lateral stiffness of abutment, which was previously governed by the stiffness of the elastomeric bearing. In this trial deck was monolithically connected to abutments, in order to investigate the effect of contribution of soil resistance on reduction of drift demand at pier. The foundation was excited with eleven ground motions and the response of the structure demonstrated a considerable increase in drift ratio demand at the pier. Thus, the compliant abutment support proved not beneficial of the demands in the pylon compared to the case that the deck sits on the elastomeric bearing.

The two methods of modelling substructure springs at pier were compared. It is recalled that the first method comprises application of springs at pile skin whereas the second method utilize a rotational and a lateral spring as representative of substructure partial constraints to translation and rotation, attached to the base of the pylon columns. The difficulty of modeling of base springs (second method) was mentioned earlier, namely consideration of the coupling terms in the stiffness matrix associated with the two degrees of freedom mentioned; this omission led to larger drifts at the pier with application of the base springs idealization.

6.4. Recommendations for further extension of the investigation

A disclaimer is in order at this point. It is noted that many parameters are involved in determining the effective stiffness of a compliant substructure, thereby affecting the soil foundation interaction. In this work some soil material properties were considered as part of the sensitivity analysis, however there are still several others that might need be considered in an expanded future study using the same context of investigation.

Application of pushover analysis for single pile assessment can be a realistic method to quantify the effective stiffness contribution of the pile to a foundation system, but also to derive proper p-y springs for response to lateral loads. The methodology with the equivalent soil replacement springs to account collectively for the substructure compliance needs further development to also reflect correctly the effective

damping of the substructure considering that the coupling terms of the true stiffness matrix are suppressed in this modeling approach thereby underestimating the effective stiffness of the foundation in the system response.

As was done here for the substructure it is recommended that, sensitivity studies should be extended to consider variable bridge geometries and material characteristics. As per NBCC 2015 there are modification factors to be applied on spectrum accelerations according with the soil type. In this work values corresponding to soft clay had been used. In future extensions of the study this attribute of the design spectrum should be adjusted in spectrum-compatible record generation.

BIBLIOGRAPHY

- American Association of State Highway and Transportation Officials. 2002. Standard Specifications for Highway Bridges. Washington, D.C.
- Atkinson, Gail M. 2009. "Earthquake Time Histories Compatible with the 2005 National Building Code of Canada Uniform Hazard Spectrum." *Canadian Journal of Civil Engineering* 36(6):991–1000. doi: 10.1139/L09-044.
- Bletzinger, Kai-Uwe. 2015. *Aufgabensammlung Zur Baustatik : Übungsaufgaben Zur Berechnung Ebener Stabtragwerke*. Hanser.
- Bogard, Dawaine, and Hudson Matlock. 1983. "Procedures for Analysis of Laterally Loaded Pile Groups in Soft Clay." 499–535.
- Bowles, Joseph E. 1992. *Engineering Properties of Soils and Their Measurement*. McGraw-Hill.
- Bowles, Joseph E. 2001. *Foundation Analysis and Design*. Vol. 1. McGraw-Hill.
- Canadian Geotechnical Society. 1992. *Canadian Foundation Engineering Manual*. Canadian Geotechnical Society.
- Clough, Ray W., and Joseph Penzien. 2013. *Dynamics of Structures*, Third Edition.
- Committee on Earthquake Engineering Commission on Engineering and Technical Systems National Research Council. 1985. *Liquefaction of Soils During Earthquakes*. Washington.
- CSiBridge. 2017a. "Bridge Seismic Design."
- CSiBridge. 2017b. "Reference Manual."
- EduPro Civil Systems. 2017. "ProShake."
- EKSPAN. 2016. "Elastomeric Bearings & Industrial Products."
- Federal Highway Administration, and Office of Highway Information Management. 1995. *Highway Statistics Summary To 1995*. Springfield.
- FEMA. 2009. FEMA P-751: 2009 NEHRP Recommended Seismic Provisions: Design Examples.
- FHWA. 2006. "Design and Construction of Driven Pile Foundations Reference Manual – Volume I." I(132021):968.

- FHWA. 2014. LRFD Seismic Analysis and Design of Bridges Reference Manual.
- FHWA. 2016. Design and Construction of Driven Pile Foundations-Volume I.
- Focht, John A., and Kenneth J. Koch. 1978. "Rational Analysis of the Lateral Performance of Offshore Pile Groups." in Proceedings of the Annual Offshore Technology Conference.
- Ghannad, Mohammadali. 2017. "Soil-Structure-Interaction (Course Script)."
- Goel, Rakesh K., and Anil Kumar Chopra. 1994. "Seismic Response Study of the US 101/Painter Street Overpass Using Strong Motion Records."
- Golder Associates. 2016. Foundation Investigation and Design Replacement of Highway 401 Underpass at Avonmore Road, Site No. 31-177 Highway 401, 8.5 Km West of Highway 138 Township of Cornwall W.P. 4382-01-01 G.W.P. 4064-12-00.
- GOODCO. 2010. "Elastomeric Bearings." 1–20.
- Gupta, V. K., and M. D. Trifunac. 1991. "Seismic Response of Multistoried Buildings Including the Effects of Soil-Structure Interaction." *Soil Dynamics and Earthquake Engineering* 10(8):414–22. doi: 10.1016/0267-7261(91)90056-6.
- Horne, M. 1978. *Plastic Theory of Structures*. New York: Pergamon Press.
- Inel, Mehmet, and Mark A. Aschheim. 2004. "Seismic Design of Columns of Short Bridges Accounting for Embankment Flexibility." *Journal of Structural Engineering*. doi: 10.1061/(ASCE)0733-9445(2004)130:10(1515).
- Isenhower, William M., and Shin-tower Wang. 2015. "User's Manual for LPILE 2015."
- Isenhower, William M., and Shin-tower Wang. 2016. "Technical Manual for LPILE 2016." LPILE (October).
- Karol, Reuben Hirsh. 1960. "Soils and Soil Engineering." Prentice-Hall Civil Engineering and Engineering Mechanics Series.
- Kotsoglou, A., and S. J. Pantazopoulou. 2009. "Assessment and Modelling of Embankment Participation in the Seismic Response of Integral Abutment Bridges." *Bulletin of Earthquake Engineering* 7(2):343–61. doi: 10.1007/s10518-009-9103-z.
- Kotsoglou, Anastasios, and Stavroula Pantazopoulou. 2013. "Exploitation of Foundation Flexibility on the Seismic Performance of Bridge Piers." *Structural Engineering International* 23(2):167–75. doi: 10.2749/101686613X13439149156688.

- Kramer, Steven Lawrence. 1996. *Geotechnical Earthquake Engineering*. Washington: Pearson.
- Krinitzsky, E. ..., and F. ... Chang. 1977. *State-Of-the-Art for Assessing Earthquake Hazards in the United States Report 7 Specifying Peak Motions for Design Earthquakes*.
- Lieng, J. T. 1989. "A MODEL FOR GROUP BEHAVIOUR OF LATERALLY LOADED PILES."
- Lindeburg, Michael R. 2008. *Civil Engineering Reference Manual for the PE Exam*. Professional Publications.
- Long, James H., and Lymon C. Reese. 1984. "Testing and Analysis of Two Offshore Drilled Shafts Subjected To Lateral Loads." *ASTM Special Technical Publication* 214–28.
- Matlock, Hudson. 1970. "Correlations for Design of Laterally Loaded Piles in Soft Clay." in *Proceedings of the Annual Offshore Technology Conference*.
- Matlock, Hudson, and Lymon C. Reese. 2002. "Generalized Solutions for Laterally Loaded Piles." in *Geotechnical Special Publication*.
- MMM GROUP. 2013. *Structural Design - Hwy 401 Underpass at Avonmore Road - Site No. 31-177*.
- Mylonakis, George, and George Gazetas. 2000. "Seismic Soil-Structure Interaction: Beneficial or Detrimental?" *Journal of Earthquake Engineering* 4(3):277–301. doi: 10.1080/13632460009350372.
- El Naggar, M. H., and K. J. Bentley. 2000. "Dynamic Analysis for Laterally Loaded Piles and Dynamic P-y Curves." *Canadian Geotechnical Journal*. doi: 10.1139/cgj-37-6-1166.
- NEHRP Consultants Joint Venture. 2012. "Soil-Structure Interaction for Building Structures." *Nist Gcr* 12:917–21. doi: 12-917-21.
- NOR-Rubber. 2016. "Bridge Elastomeric Bearing Pads."
- Novak, M., and L. El Hifnawy. 1983. "Damping of Structures Due to Soil-Structure Interaction." *Journal of Wind Engineering and Industrial Aerodynamics* 11(1–3):295–306. doi: 10.1016/0167-6105(83)90108-3.
- Ontario Ministry of Transportation. 2016. "Southern Highways Program 2016-2020."
- Panagiotou, Marios. 2008. "Seismic Design, Testing and Analysis of Reinforced Concrete Wall Buildings." UNIVERSITY OF CALIFORNIA, SAN DIEGO, San Diego.
- Poulos, HG, and EH Davis. 1981. "Pile Foundation Analysis and Design." *International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts*. doi: 10.1016/0148-9062(81)90191-1.

- Prakash, Shamsheer. 1962. Behavior of Pile Groups Subjected to Lateral Loads - Shamsheer Prakash.
- Pranjoto, Satyawati, and M. J. Pender. 2003. Gapping Effects on the Lateral Stiffness of Piles in Cohesive Soil.
- Rahmani, Amin. 2014. "Three-Dimensional Nonlinear Analysis of Dynamic Soil-Pile-Structure Interaction for Bridge Systems under Earthquake Shakings." The University of British Columbia (November).
- Rajapakse, Ruwan. 2016. "Laterally Loaded Piles." Pp. 235–40 in Pile Design and Construction Rules of Thumb. Elsevier.
- Reese, L. C., and H. Matlock. 1960. "Numerical Analysis of Laterally Loaded Piles." in Second Structural Division Conference on Electronic Computation . Pittsburgh.
- Reese, Lymon C. 1997. "Analysis of Laterally Loaded Piles in Weak Rock." Journal of Geotechnical and Geoenvironmental Engineering 123(11):1010–17. doi: 10.1061/(ASCE)1090-0241(1997)123:11(1010).
- Reese, Lymon C., and Shin-Tower Wang. 2006. Verification of Computer Program LPILE as a Valid Tool for Design of a Single Pile under Lateral Loading.
- Reese, Lymon C., and Robert C. Welch. 1975. "Lateral Loading of Deep Foundations in Stiff Clay." ASCE J Geotech Eng Div 101(7):633–49.
- Research Council Canada, National. 2015. National Building Code of Canada 2015.
- Schmidt, H. G. 1981. "Group Action of Laterally Loaded Bored Piles."
- Singh, R. N. P., and Hemant Kumar Vinayak. 2015. "Assessment of Soil-Structure Interaction in Seismic Bridge Pier Analysis Using Force and Displacement Based Approaches." Selected Scientific Papers - Journal of Civil Engineering 10(2). doi: 10.2478/sspjce-2015-0023.
- Standards-Australia. 2004. "Bridge Design - Bearings and Deck Joints - AS 5100.4-2004."
- Tongaonkar, N. P., and R. S. Jangid. 2003. "Seismic Response of Isolated Bridges with Soil-Structure Interaction." Soil Dynamics and Earthquake Engineering 23(4):287–302. doi: 10.1016/S0267-7261(03)00020-4.
- TRELLEBORG. 2016. "Elastomeric Bearing Pads & Strips."
- Vanmarcke, Erik H. 1979. "State-of-the-Art for Assessing Earthquake Hazards in the United States. Report

14. Representation of Earthquake Ground Motion: Scaled Accelerograms and Equivalent Response Spectra.”

- Wolf, John P. 1985. “Dynamic Soil-Structure Interaction.” Prentice-Hall Civil Engineering and Engineering Mechanics 1.
- Wolf, John P. 1991. Classification of Classification of Analysis Methods for Dynamic Soil-Structure Interaction Interaction Classification of Analysis Methods for Dynamic Soil-Structure Interaction. Missouri.
- Wolf, John P., and Andrew Deeks. 2004. Foundation Vibration Analysis - A Strength of Materials Approach. Elsevier.
- Zhang, Jian, and Nicos Makris. 2002. “Seismic Response Analysis of Highway Overcrossing Including Soil-Structure Interaction.” Earthquake Engineering and Structural Dynamics. doi: 10.1002/eqe.197.
- Veletsos, Anestis S. et al. 1971. “Lateral and Rocking Vibration of Footings.” Soil Mechanincs and Foundation Divison, Preceedings of the American Society of Civil Engineers 1227–48.
- Reese, Lymon C. et al. 1974. “Analysis of Laterally Loaded Piles in Sand.” in Proceedings of the Annual Offshore Technology Conference.
- Reese, Lymon C. et al. 1975. “Field Testing and Analysis of Laterally Loaded Piles in Stiffclay.” in Proceedings of the Annual Offshore Technology Conference.
- Kausel, Eduardo et al. 1978. “The Spring Method for Embedded Foundations.” Nuclear Engineering and Design. doi: 10.1016/0029-5493(78)90085-7.
- Brown, Dan A. et al. 1987. “Cyclic Lateral Loading of a Large-Scale Pile Group.” Journal of Geotechnical Engineering 113(11):1326–43. doi: 10.1061/(ASCE)0733-9410(1987)113:11(1326).
- Mander, J. B. et al. 1988. “Theoretical Stress-Strain Model for Confined Concrete.” Journal of Structural Engineering 114(8):1804–26. doi: 10.1061/(ASCE)0733-9445(1988)114:8(1804).
- Nogami, Toyoaki et al. 1992. “Nonlinear Soil-Pile Interaction Model for Dynamic Lateral Motion.” Journal of Geotechnical Engineering 118(1):89–106. doi: 10.1061/(ASCE)0733-9410(1992)118:1(89).
- Clayton, C. R. I. et al. 1995. “Terzaghi’s Theory of Consolidation, and the Discovery of Effective Stress. (Compiled from the Work of k. Terzaghi and A.W. Skempton).” Proceedings of the Institution of Civil Engineers - Geotechnical Engineering 113(4):191–205. doi: 10.1680/igeng.1995.28015.

- Priestley, M. J. N. et al. 1996. Seismic Design and Retrofit of Bridges. Hoboken, NJ, USA: John Wiley & Sons, Inc.
- Ghannad, M. Ali et al. 1998. "A Study on the Frequency and Damping of Soil-Structure -Interaction Using a Simplified Model." 85–93.
- Ross Boulanger, By W. et al. 1999. SEISMIC SOIL-PILE-STRUCTURE INTERACTION EXPERIMENTS AND ANALYSES.
- Reese, LC et al. 2002. "Single Piles and Pile Groups Under Lateral Loading." Applied Mechanics Reviews. doi: 10.1115/1.1445326.
- Stewart, Jonathan P. et al. 2003. "Revisions to Soil-Structure Interaction Procedures in NEHRP Design Provisions." Earthquake Spectra 19(3):677–96.
- De Carlo, Giuseppe et al. 2004. INFLUENCE OF SOIL-STRUCTURE INTERACTION ON THE SEISMIC RESPONSE OF BRIDGE PIERS.
- Song, S. T. et al. 2004. "LIMIT STATE ANALYSIS OF FIXED-HEAD CONCRETE PILES UNDER LATERAL LOADS."
- Reese, Lymon C. et al. 2006. Analysis and Design of Shallow and Deep Foundations. Hoboken, New Jersey: John Wiley & Sons.
- Buckle, I. G. et al. 2006. Seismic Isolation of Highway Bridges. Buffalo.
- Ian Buckle, Author et al. 2006. Title and Subtitle SEISMIC RETROFITTING MANUAL FOR HIGHWAY STRUCTURES: PART 1-BRIDGES 6. Performing Organization Code.
- Halchuk, Stephen et al. 2007. "Revised Deaggregation of Seismic Hazard for Selected Canadian Cities." 9th Canadian Conference on Earthquake Engineering 432(June):420–32. doi: 10.4095/223221.
- Wei, Xiao et al. 2008. DAMAGE PATTERNS AND FAILURE MECHANISMS OF BRIDGE PILE FOUNDATION UNDER EARTHQUAKE.
- Cox, WR et al. 2008. "Lateral-Load Tests on 25.4-Mm (1-in.) Diameter Piles in Very Soft Clay in Side-by-Side and In-Line Groups." Pp. 122-122–18 in Laterally Loaded Deep Foundations: Analysis and Performance. ASTM International.
- Aviram, Ady et al. 2008. "PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER Guidelines for Nonlinear Analysis of Bridge Structures in California."

- Abdelsalam, Sherif S. et al. 2010. "Current Design and Construction Practices of Bridge Pile Foundations with Emphasis on Implementation of LRFD." *Journal of Bridge Engineering* 15(6):749–58. doi: 10.1061/(ASCE)BE.1943-5592.0000118.
- Hanna, Kromel E. et al. 2010. Design Aids of NUI-Girder Bridges Nebraska Department of Roads (NDOR) Project Number: P322 FINAL REPORT PRINCIPAL INVESTIGATORS.
- Foye, K. C. et al. 2011. "Resistance Factors for Design of Piles in Sand: Tools to Understand Design Reliability." 435(1).
- Rahmani, A. et al. 2012. "Determination of Dynamic P-y Curves for Pile Foundations Under Seismic Loading."
- Viggiani, Carlo et al. 2012. *Piles and Pile Groups*. New York: Spon Press.
- Farantakis, S. et al. 2014. "Exploiting SSI to Mitigate Seismic Demands in Bridge Piers." Tenth U.S. National Conference on Earthquake Engineering, Alaska, U.S. doi: 10.4231/D31Z41T56.
- Pantazopoulou, S. et al. 2014. "Exploiting SSI to Mitigate Seismic Demands in Bridge Piers." 10NCEE - Frontiers of Earthquake Engineering.
- Hokmabadi, Aslan S. et al. 2014. "Physical Modeling of Seismic Soil-Pile-Structure Interaction for Buildings on Soft Soils." doi: 10.1061/(ASCE)GM.1943.
- Antonellis, Grigorios et al. 2014. "Seismic Response of Bridges with Rocking Foundations Compared to Fixed-Base Bridges at a Near-Fault Site." doi: 10.1061/(ASCE).
- Tremblay, Robert et al. 2015. "Selection and Scaling of Ground Motion Time Histories for Seismic Analysis Using NBCC 2015." 11th Canadian Conference on Earthquake Engineering (11CCEE) 16.
- Liu, Jinyuan et al. 2015. Statistical Correlations between SPT N-Values and Soil Parameters.
- Kumar, Ranjan et al. 2016. "Estimation of Engineering Properties of Soils from Field SPT Using Random Number Generation." *INAE Letters* 1(3–4):77–84. doi: 10.1007/s41403-016-0012-6.
- Rahmani, Amin et al. 2016. "Evaluation of Substructuring Method for Seismic Soil-Structure Interaction Analysis of Bridges." *Soil Dynamics and Earthquake Engineering* 90:112–27. doi: 10.1016/j.soildyn.2016.08.013.
- Sanayei, Masoud et al. 2016. "Load Rating of a Fully Instrumented Bridge: Comparison of LRFR Approaches." *Journal of Performance of Constructed Facilities* 30(2):04015019. doi:

10.1061/(ASCE)CF.1943-5509.0000752.

Zhang, Chenrong et al. 2016. “Winkler Load-Transfer Analysis for Laterally Loaded Piles.” *Canadian Geotechnical Journal*. doi: 10.1139/cgj-2015-0394.

Taghavi, Amirata et al. 2017. “Design-Oriented Seismic Soil-Pile-Superstructure Interaction Analysis Using a Dynamic p-y Method.” *Bridge Structures* 13(2–3):57–67. doi: 10.3233/BRS-170113.

Reese, Lymon C. et al. 2018. “Technical Manual for GROUP 2018.”

Isenhower, W. M. et al. 2018. “User’s Manual for LPILE 2018.” LPILE.

Hojjat, Sina et al. 2019. “Soil-Structure-Interaction Effect on Seismic Response of a Pile Group at Bridge Pier Supports.” Toronto: 12th Canadian Conference on Earthquake Engineering.

Reese, Lymon C. et al. 2019. “User’s Manual for GROUP 2018.”

APPENDICES

APPENDIX A: Soil Layers at Avonmore bridge site

Table A-1- Range of SPT N value correlation with angle of friction for non cohesive soils

Location	Elevation	Soil Type	$n_h \left(\frac{MN}{m^3} \right)$	s_u (kPa)
North Abutment	81.9 – Pilecap level	Compact to Very Dense Gravelly, Silty Sand (Fill)	6 to 15	28–30
	80.5 – 81.9	Compact Gravelly, Silty Sand	3 to 6	-
	79.8 – 80.5	Stiff Silty Clay	-	75 kPa
	76.4 – 79.8	Compact to Dense Silty Sand (Till)	4 to 11	-
	76.4	Bedrock	-	-
Central Pier	79.6 – pilecap level	Stiff Silty Clay	-	75 kPa
	78.7 – 79.6	Compact Sand and Gravel (Till)	3 to 6	-
	78.0 – 78.7	Loose Sand and Gravel (Till)	1 to	-
	76.5 – 78	Compact to Dense Sand and Gravel (Till)	4 to 11	-
	76.5	Bedrock	-	-
South Abutment	81.9 – pilecap level	Compact to Very Dense Gravelly, Silty Sand (Fill)	6 to 15	-

	80.7 – 81.9	Compact Sand and Gravel	3 to 6	-
	80.0 – 80.7	Stiff Silty Clay	-	75 kPa
	76.0 – 80.0	Dense to Very Dense Gravelly, Silty Sand (Till)	8 to 11	-
	74.9 – 76.0	Compact Silty Sand	3 to 6	-
	74.9	Bedrock	-	-

Table A-2 - Range of SPT N value correlation with angle of friction for non cohesive soils (Karol 1960), (Liu et al. 2015)

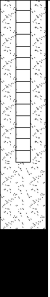
SPT N value	>50	30–50	10–30	4–10	0–4
Angle of friction, degree	>41	36–41	30–36	28–30	<28
Soil conditions	Very good	Good	Fair	Poor	Very poor

Table A-3 - SPT N value correlation with cohesion for cohesive soils (Karol 1960)

SPT N value	>30	15–30	8–15	4–8	2–4	<2
Cohesion, kPa	192	96–192	48–96	24–48	12–24	12
Soil conditions	Hard	Very stiff	Stiff	Firm	Soft	Very soft

Table A-4 - Rock Quality Designation (Karol 1960)

RQD (%)	0 -25	25 - 50	50 - 75	75 - 90	90 - 100
Rock Quality	Very poor	Poor	Fair	Good	Excellent

RECORD OF BOREHOLE No 14-731 SHEET 2 OF 3 METRIC														
PROJECT 12-1121-0099-1730		LOCATION N 4991810.3 :E 196424.1				ORIGINATED BY DWM								
G.W.P. 4382-01-01		BOREHOLE TYPE Power Auger 200 mm Diam., (Hollow Stem), Rotary Drill NQ Core				COMPILED BY JM								
DIST Eastern HWY 401		DATE October 24-27, 2014				CHECKED BY MJK								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						
— CONTINUED FROM PREVIOUS PAGE —														
76.4 12.5	Silty SAND, some gravel, contains cobbles and boulders (TILL) Compact to dense Grey Wet		13	SS	13		78							14 52 26 8
			14	SS	39									
			15	SS	34									
			16	SS	50/0.2									
73.3 15.6	Limestone (BEDROCK) Bedrock cored from depths of 12.5 m to 15.6 m For bedrock coring details refer to Record of Drillhole 14-731		1	RC	REC 90%		76							RQD = 90%
			2	RC	REC 100%									
			3	RC	REC 100%									
END OF BOREHOLE														
NOTES: 1. Water level in well screen at a depth of 7.0 m below ground surface (Elev. 81.9 m), measured on November 26, 2014.														

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT: 12-1121-0099-1730

LOCATION: N 4991810.3 ;E 196424.1

INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: 14-731

DRILLING DATE: October 24-27, 2014

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC Drilling

SHEET 3 OF 3

DATUM: Geodetic

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY										FEATURES	
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX					
							TOTAL CORE %	SOLID CORE %										
							DP W/L CORE AXIS	TYPE AND SURFACE DESCRIPTION						Jr	Ja	10 °C		
		Continued from Record of Borehole 14-731		76.43														
		Limestone (BEDROCK), with thin black shale interbeds and partings Fresh Thinly to medium bedded Grey Fine to medium grained Non-porous Strong to very strong - Broken core from 12.5 m to 12.6 m		12.51														
13	Rotary Drill NQ Core			1	0													
			2	0														
			3	0														
14																		
15																		
16		END OF DRILLHOLE		73.38														
17				15.56														
18																		
19																		
20																		
21																		
22																		

DEPTH SCALE

1 : 50

LOGGED: DWM

CHECKED: MJK



PROJECT 12-1121-0099-1730			RECORD OF BOREHOLE No 14-732			SHEET 1 OF 3			METRIC														
G.W.P. 4382-01-01			LOCATION N 4991739.5 ; E 196461.4			ORIGINATED BY DWM																	
DIST Eastern HWY 401			BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill HQ Core			COMPILED BY JM																	
DATUM Geodetic			DATE October 23, 2014			CHECKED BY MJK																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
89.1	GROUND SURFACE																						
0.0	ASPHALTIC CONCRETE																						
0.1	PORTLAND CEMENT CONCRETE																						
88.6																							
0.5	Sand and gravel, trace silt (FILL) Dense to compact Grey-brown Dry		1	SS	37																		
			2	SS	21																		
			3	SS	13																		
86.4																							
2.7	Sand and silt, some gravel (FILL) Brown Moist																						
86.1																							
3.1	Gravelly silty sand, contains cobbles (FILL) Dense Grey-brown Moist		4	SS	45																		
85.3																							
3.8	Sand and silt, trace to some gravel, contains cobbles and organic matter (FILL) Very dense to loose Brown Moist		5	SS	17																		
			6	SS	7																		
			7	SS	112/0.2																		
			8	SS	29																		
			9	SS	57																		
81.5																							
7.6	SAND and GRAVEL, some silt, contains white shells Compact Grey Wet		10	SS	14																		
80.7																							
8.4	SILTY CLAY to CLAY Stiff Grey Wet		11	SS	9																		
80.0																							
9.1	Gravelly SAND, some silt to silty, contains cobbles (FILL) Very dense to dense Grey Moist to wet		12	SS	79																		

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT 12-1121-0099-1730		RECORD OF BOREHOLE No 14-732				SHEET 2 OF 3		METRIC						
G.W.P. 4382-01-01		LOCATION N 4991739.5 ; E 196461.4				ORIGINATED BY DWM								
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill HQ Core				COMPILED BY JM								
DATUM Geodetic		DATE October 23, 2014				CHECKED BY MJK								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						
— CONTINUED FROM PREVIOUS PAGE —														
			13	SS	40		79							
			14	SS	62		78							30 40 23 7
77.7 11.4	Gravelly SAND, some silt to silty, contains cobbles (TILL) Very dense to dense Grey Moist to wet		15	SS	84		77							
			16	SS	53									8 34 41 17
76.0 13.1	SILT and SAND, some clay, trace gravel, contains cobbles and boulders (TILL) Very dense Grey Moist		17	SS	16		76							
74.9 14.2	Silty SAND, some gravel, occasional sand seams (TILL) Compact Grey Wet		1	RC	REC 100%		75							RQD = 100%
	Shaley Limestone (BEDROCK) Bedrock cored from depths of 14.2 m to 17.2 m For bedrock coring details refer to Record of Drillhole 14-732		2	RC	REC 100%		74							RQD = 100%
71.9 17.2	END OF BOREHOLE		3	RC	REC 100%		73							RQD = 100%
							72							

GTA-MTO 001 N:ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IM\GINT\121121\0099.GPJ GAL-GTA.GDT 01/12/16 JM

+ 3, × 3, Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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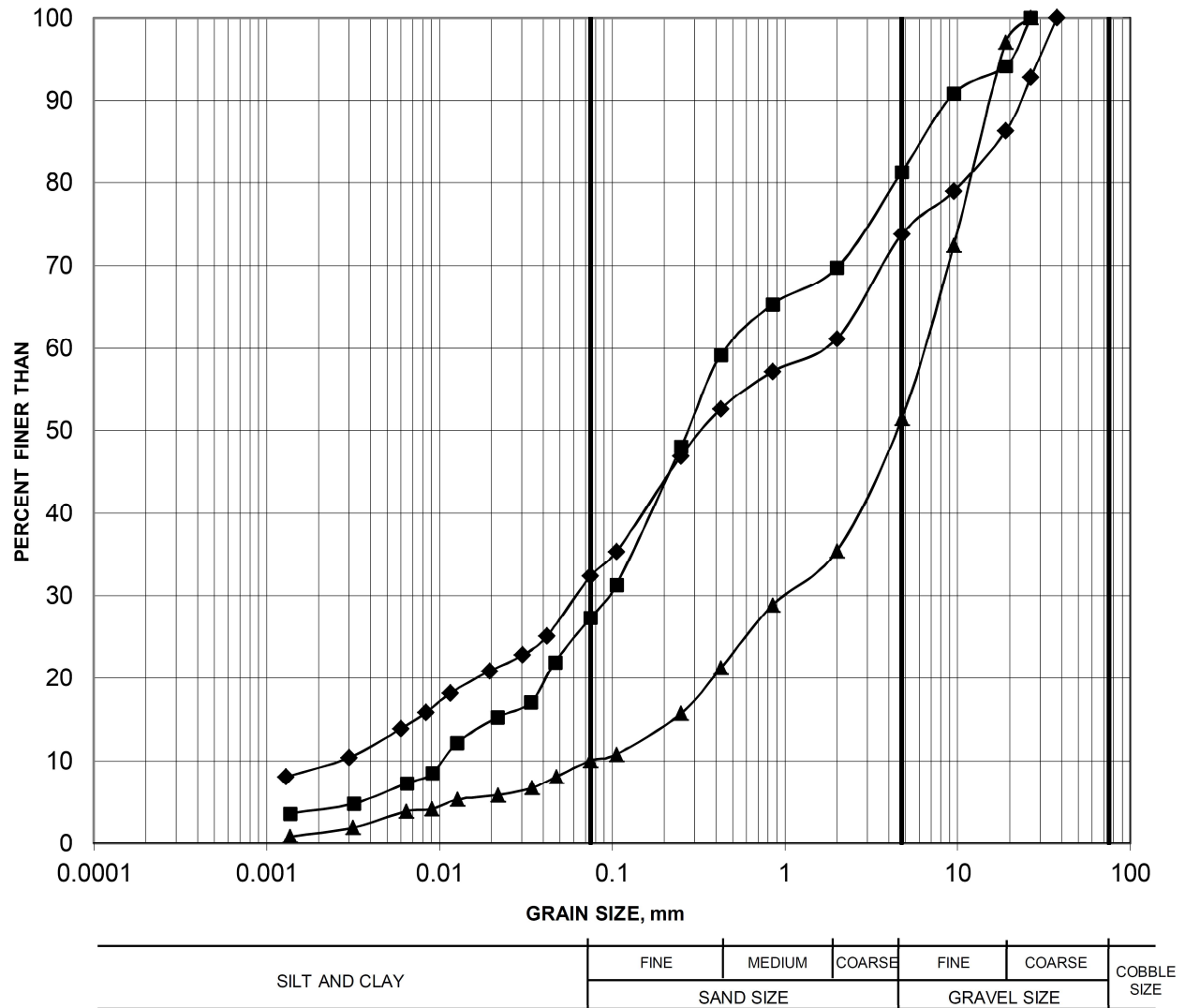


PROJECT 12-1121-0099-1730			RECORD OF BOREHOLE No 14-733			SHEET 1 OF 2			METRIC			
G.W.P. 4382-01-01			LOCATION N 4991770.1 :E 196432.6			ORIGINATED BY DWM						
DIST Eastern HWY 401			BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), Rotary Drill NQ Core			COMPILED BY JM						
DATUM Geodetic			DATE October 30-31, 2014			CHECKED BY MJK						
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES					
82.5	GROUND SURFACE											
0.0	TOPSOIL											
0.2	Silt and sand, some gravel, contains organic matter (FILL) Loose Brown to black Wet		1	SS	5							
81.0												
1.5	Silty SAND, some gravel, contains shells Compact Brown Wet		2	SS	10							
80.7												
1.8	SILTY CLAY to CLAY, trace sand, trace gravel Stiff Grey Wet		3	SS	10							
79.6												
2.9	Gravelly SAND, some silt (TILL) Compact Grey Wet		4	SS	23							
78.7												
3.8	Sandy GRAVEL, contains silt seams, cobbles and boulders (TILL) Loose to compact Grey Wet		5	SS	8							
			6	SS	22							
			7	SS	76/0.3							
76.5			8	SS	50/0.0							
6.0	Shaley Limestone (BEDROCK) Bedrock cored from depths of 6.0 m to 9.4 m For bedrock coring details refer to Record of Drillhole 14-733		1	RC	REC 97%							
			2	RC	REC 100%							
73.1												
9.4	END OF BOREHOLE											

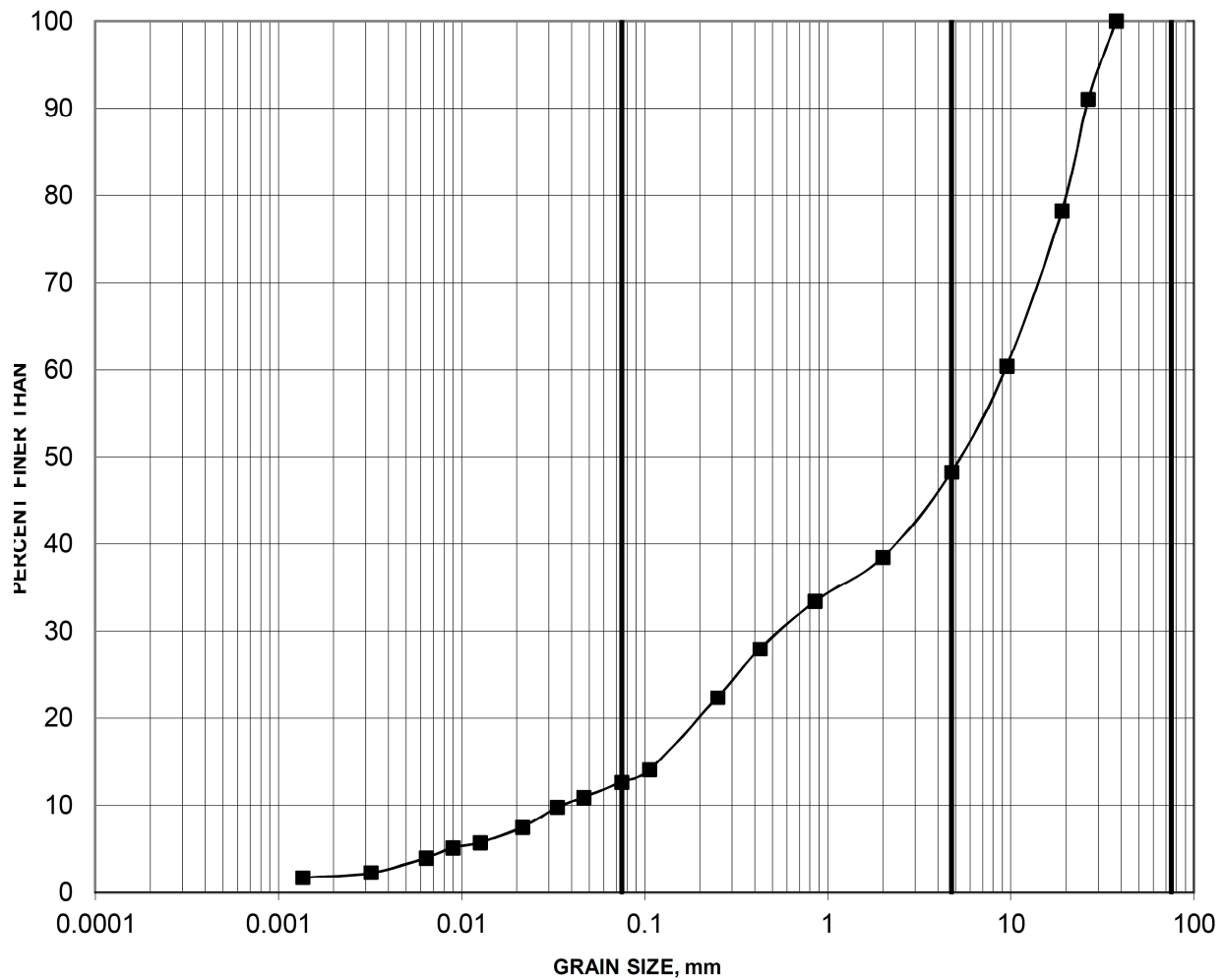
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GTA-RCK 031 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0099 MRC 22 STRUCTURES EASTERN REGION\SPATIAL IMAGING\1211210099.GPJ GAL-MISS.GDT 01/12/16 JM

**SILTY, GRAVELLY SAND to SAND and GRAVEL
(EMBANKMENT FILL)**



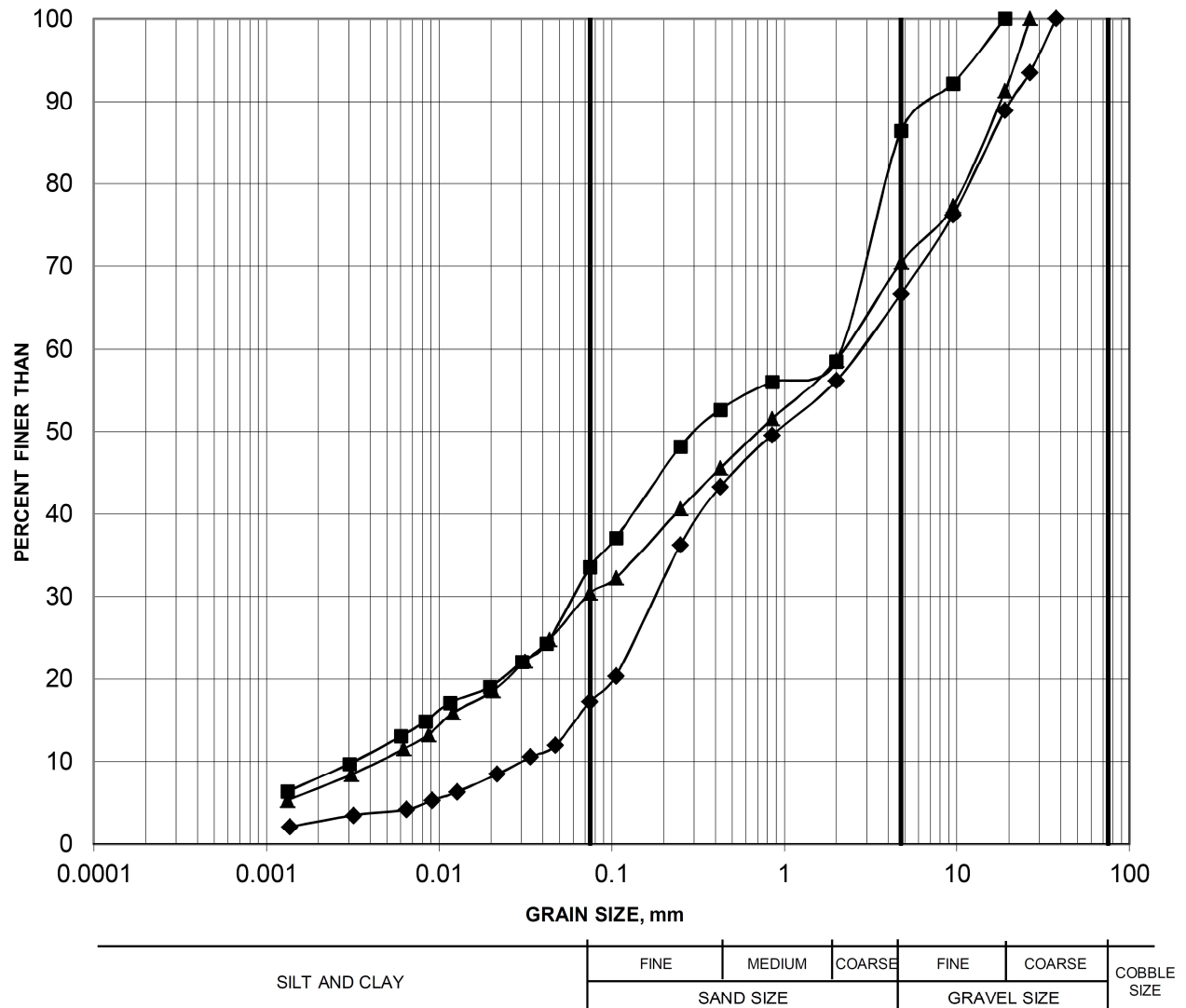
SAND AND GRAVEL



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

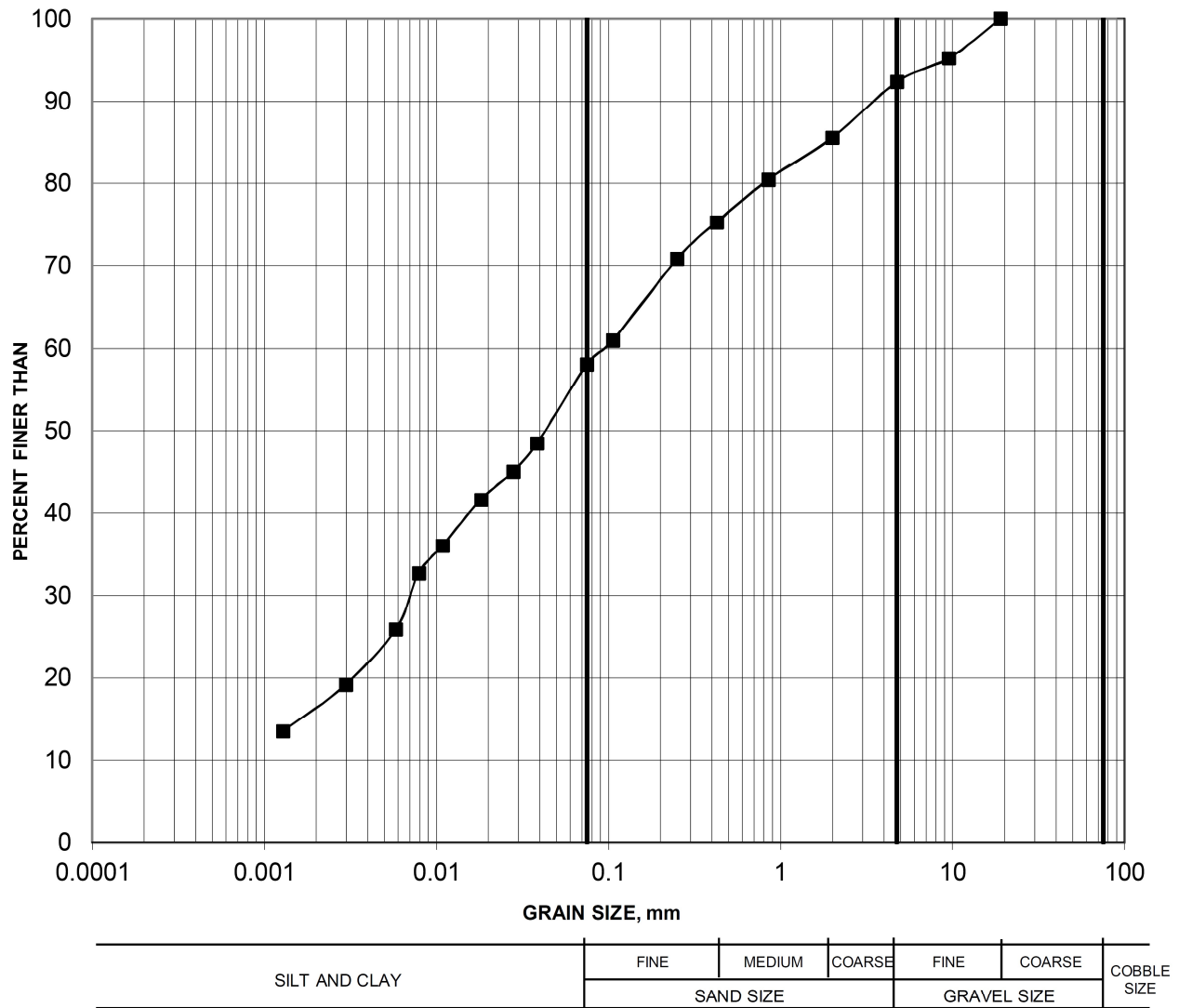
Borehole	Sample	Depth (m)
—■— 14-732	10	7.62-8.23

SILTY SAND to GRAVELLY SILTY SAND (GLACIAL TILL)



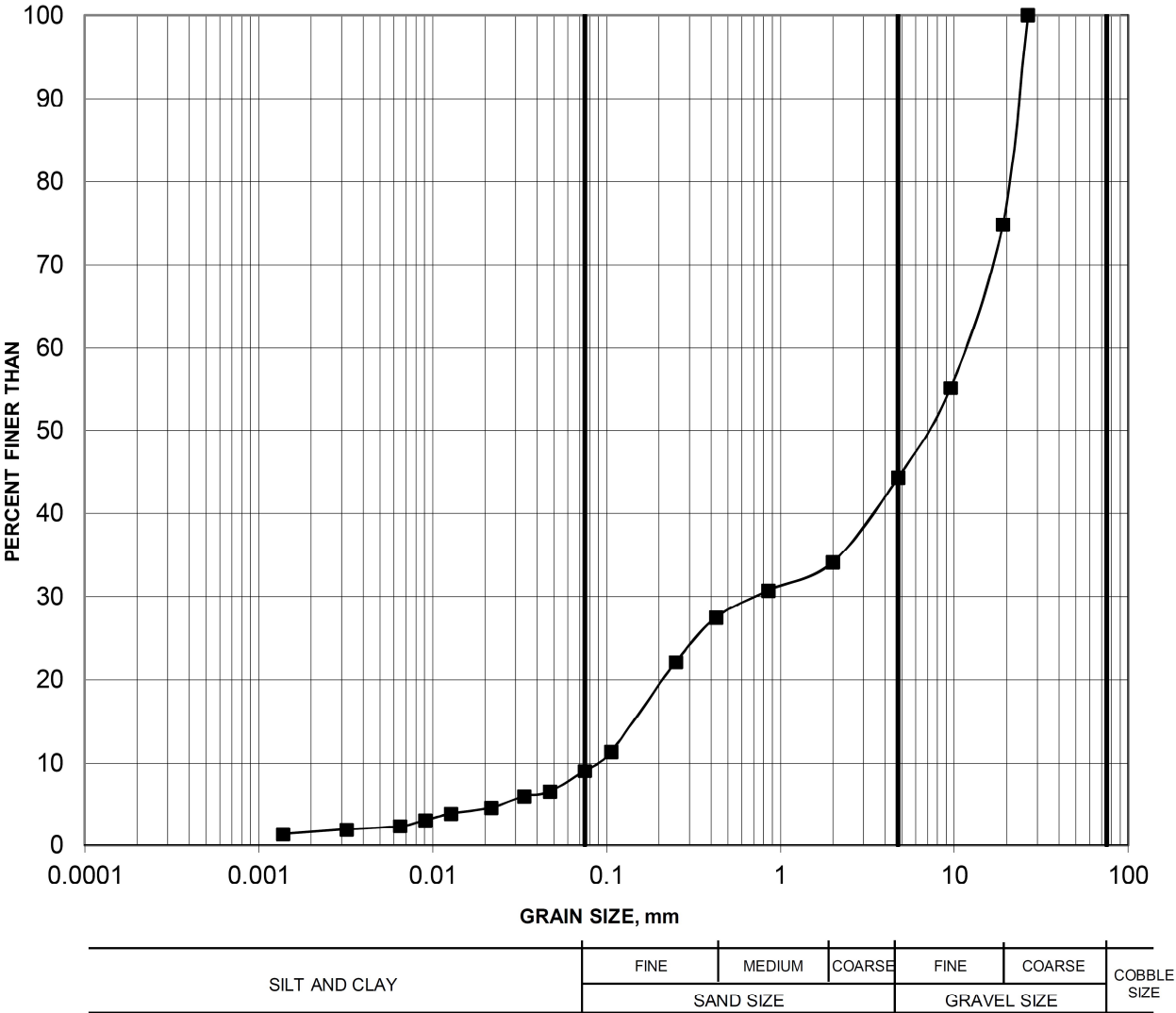
Borehole	Sample	Depth (m)
14-731	14	10.67-11.28
14-732	14	10.67-11.28
14-733	4	3.05-3.66

SILT AND SAND (GLACIAL TILL)

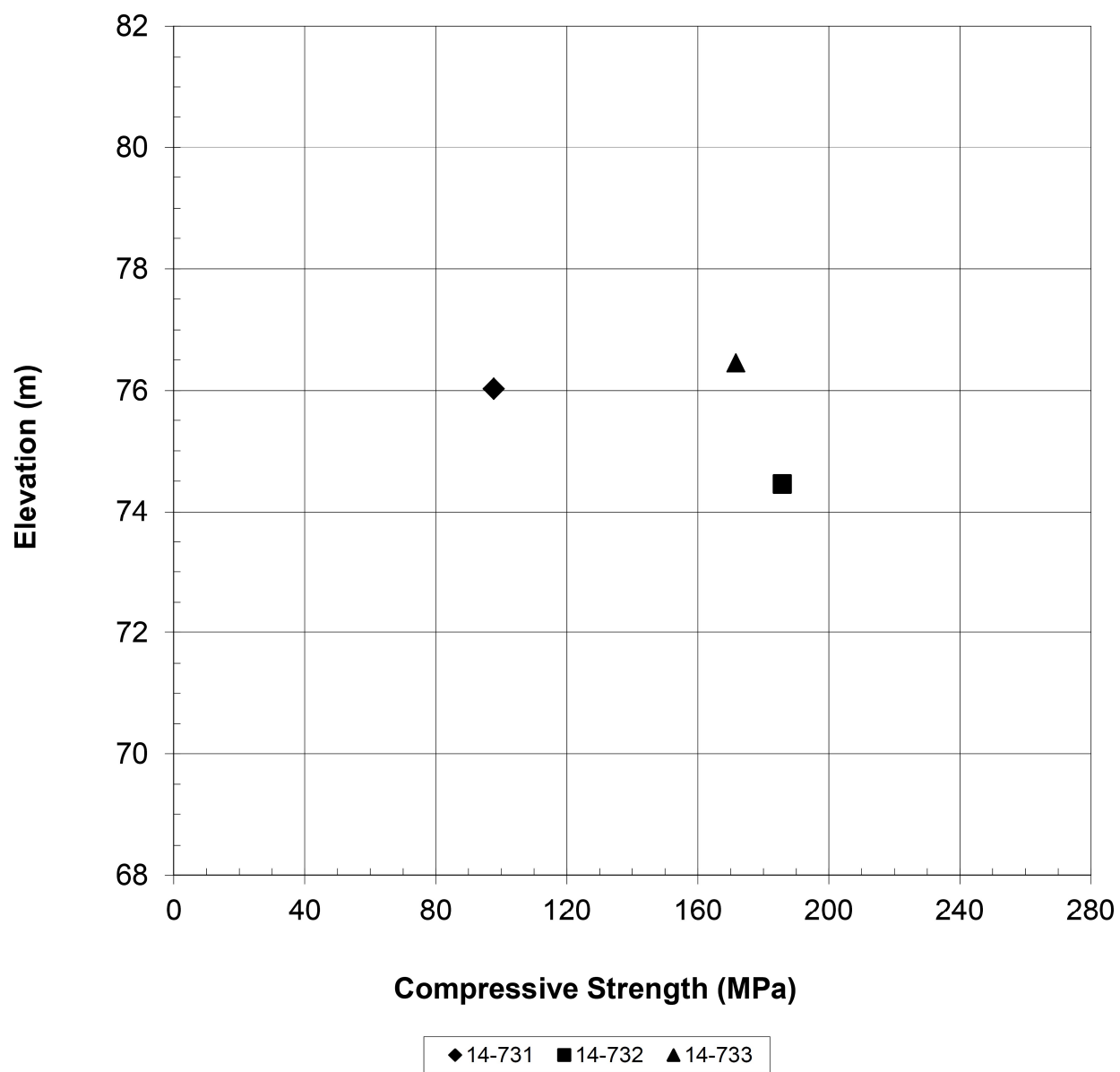


Borehole	Sample	Depth (m)
—■— 14-732	16	12.20-12.80

SAND and GRAVEL (GLACIAL TILL)



Borehole	Sample	Depth (m)
14-733	6	4.57-5.18



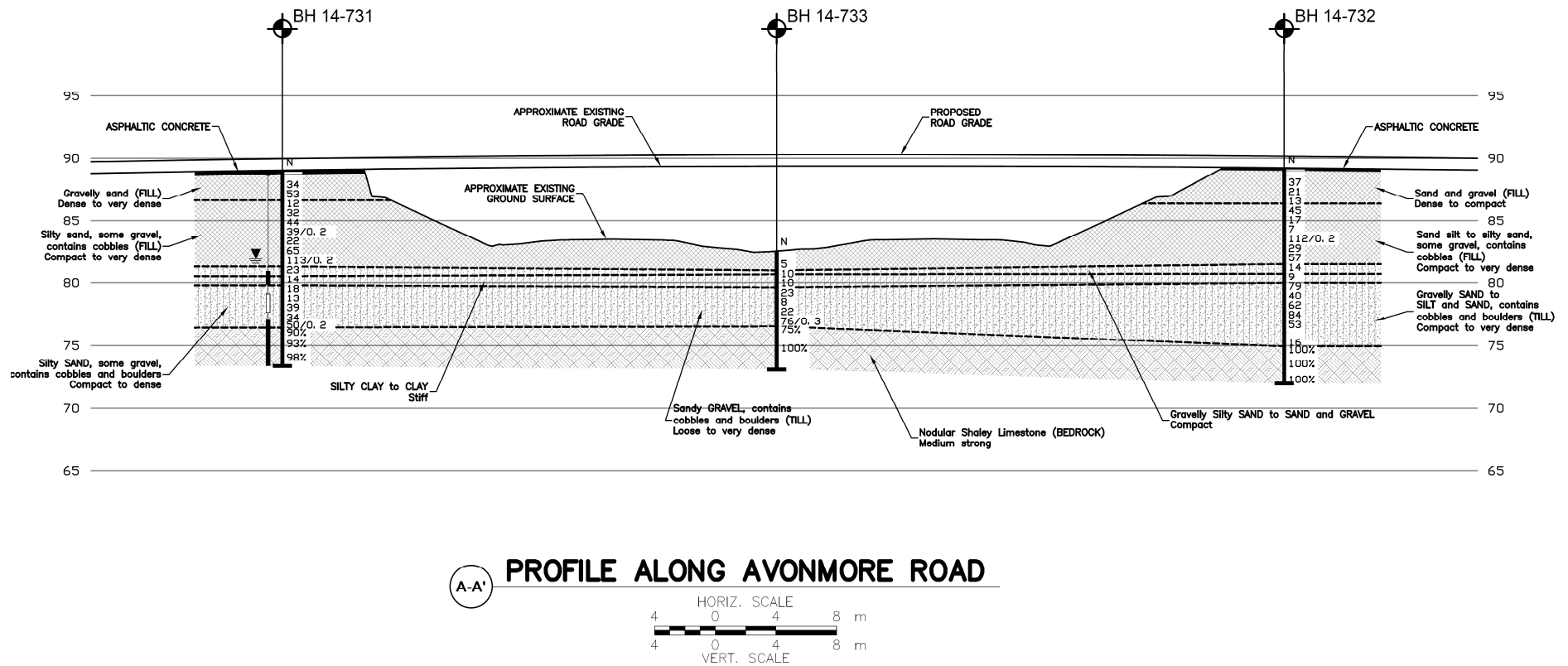


Figure A-1 – Site geology at under Avonmore bridge

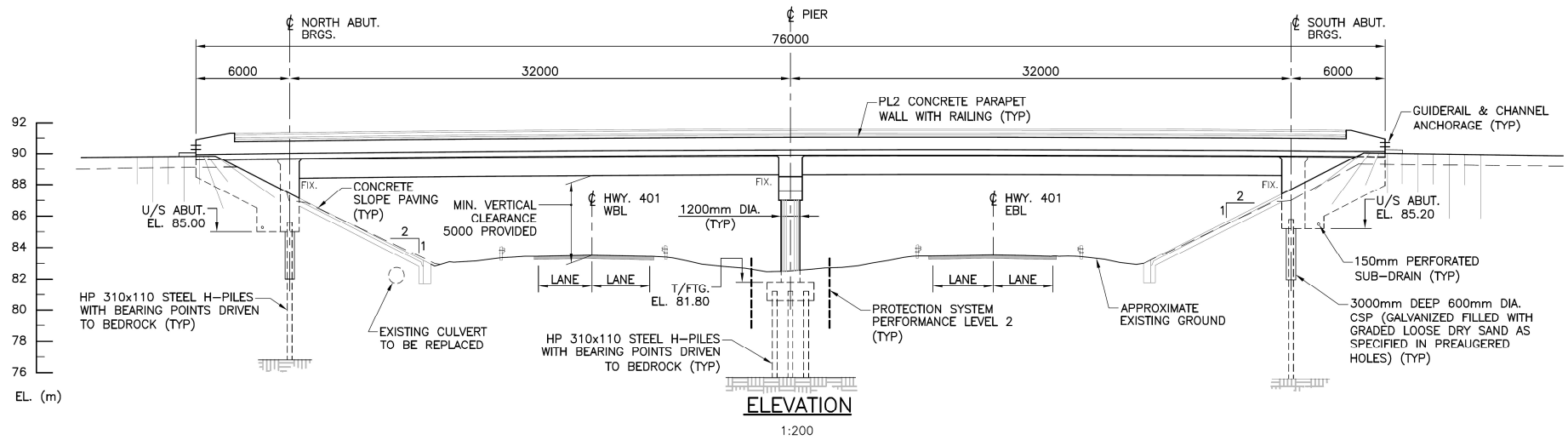


Figure A-2 - Restored bridge side view

APPENDIX B: Pilecap and abutment cross sections

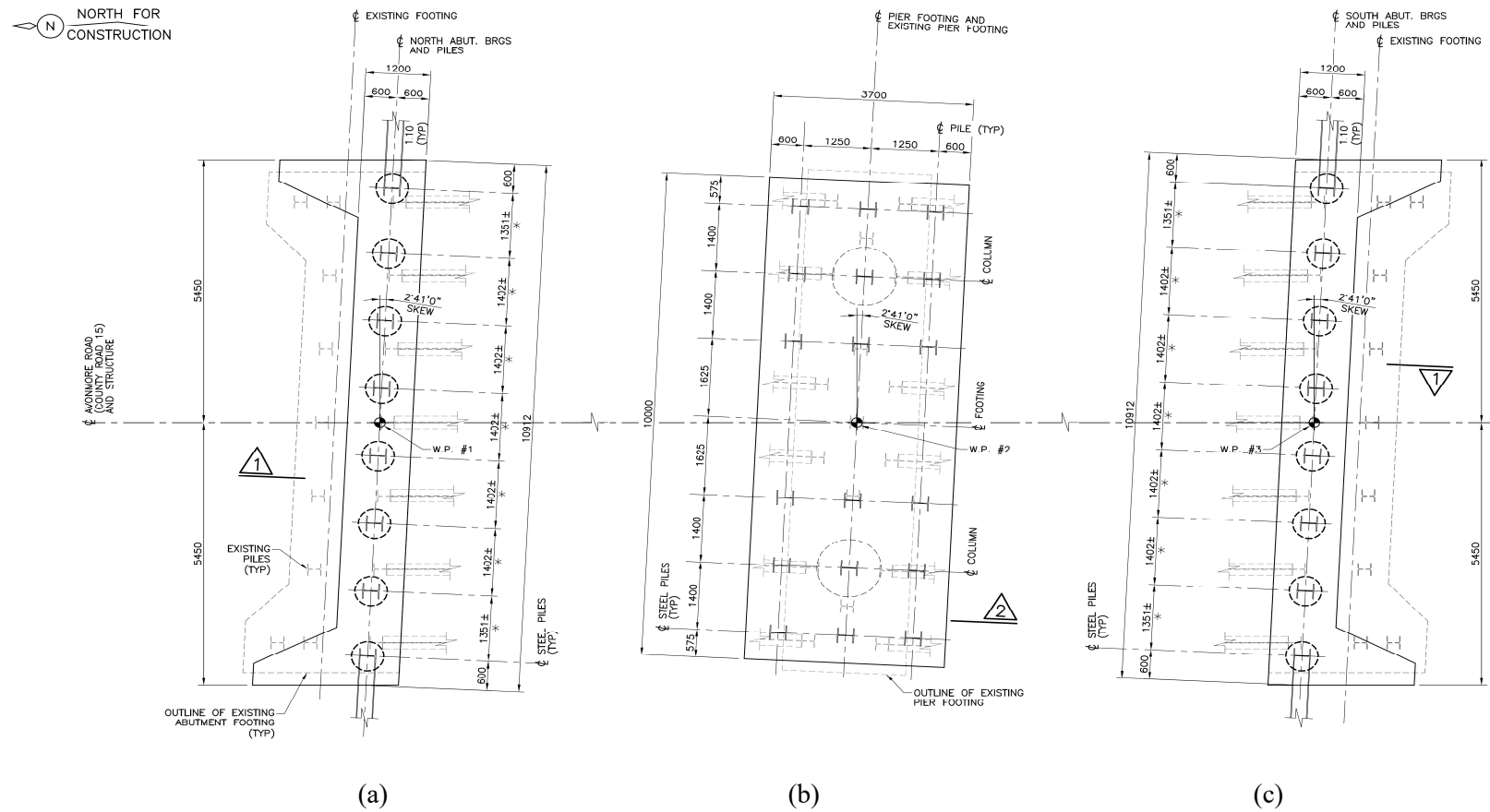


Figure B-1 - Pilecap (a) and abutments (b) & (c)

APPENDIX C: Avonmore Bridge Structural components

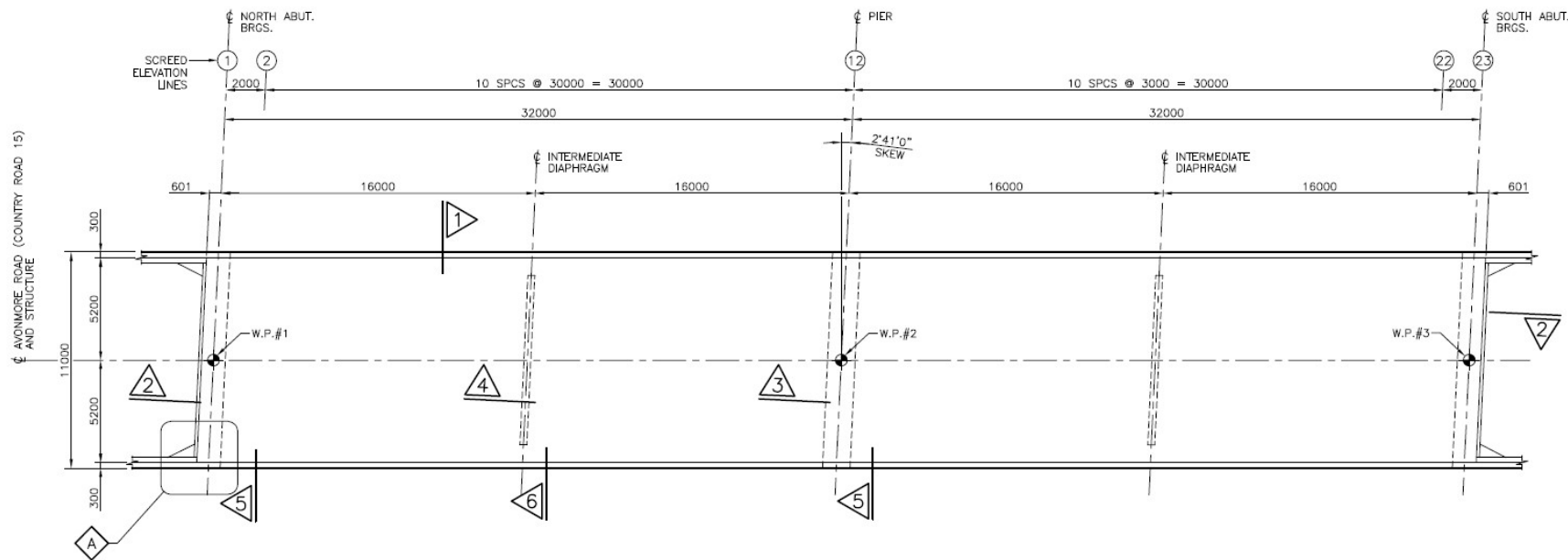


Figure C-1 - Avonmore bridge deck slab

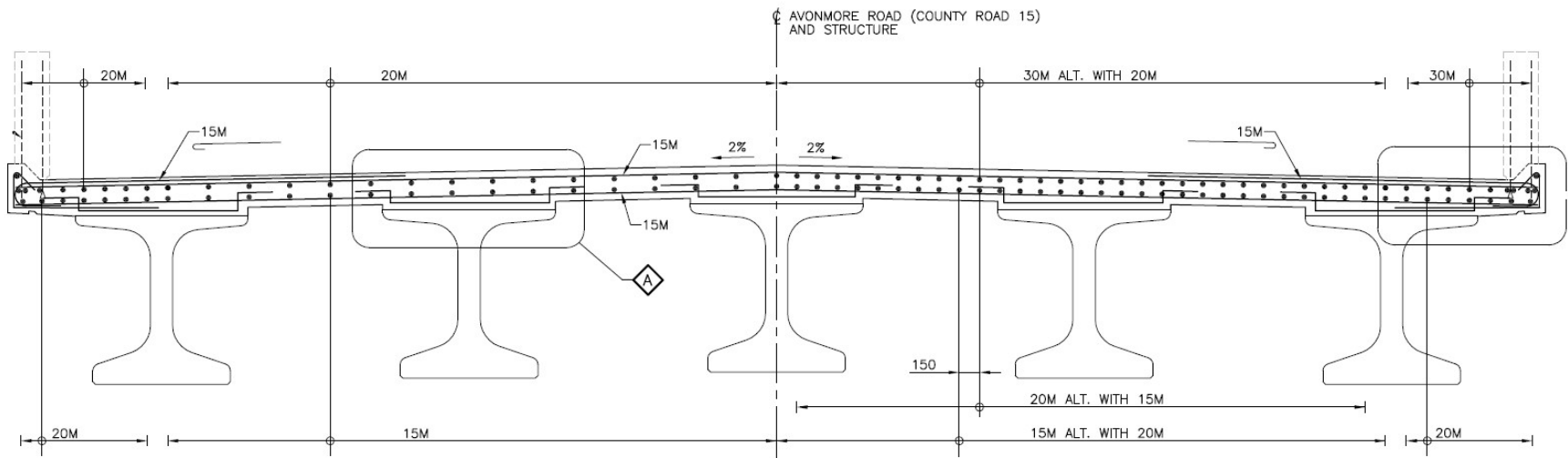


Figure C-2 - Avonmore bridge deck reinforcement

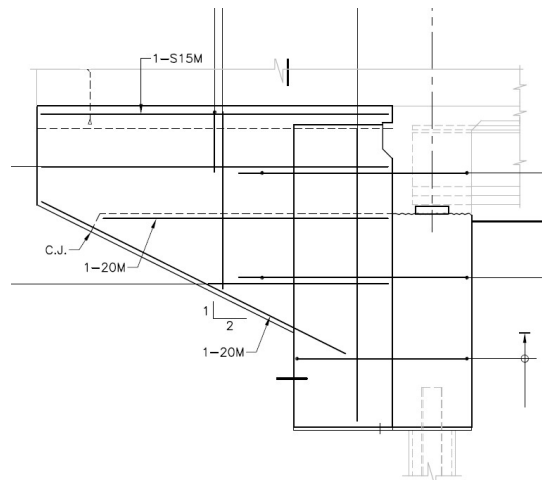


Figure C-3 - Avonmore bridge abutment connection to the deck

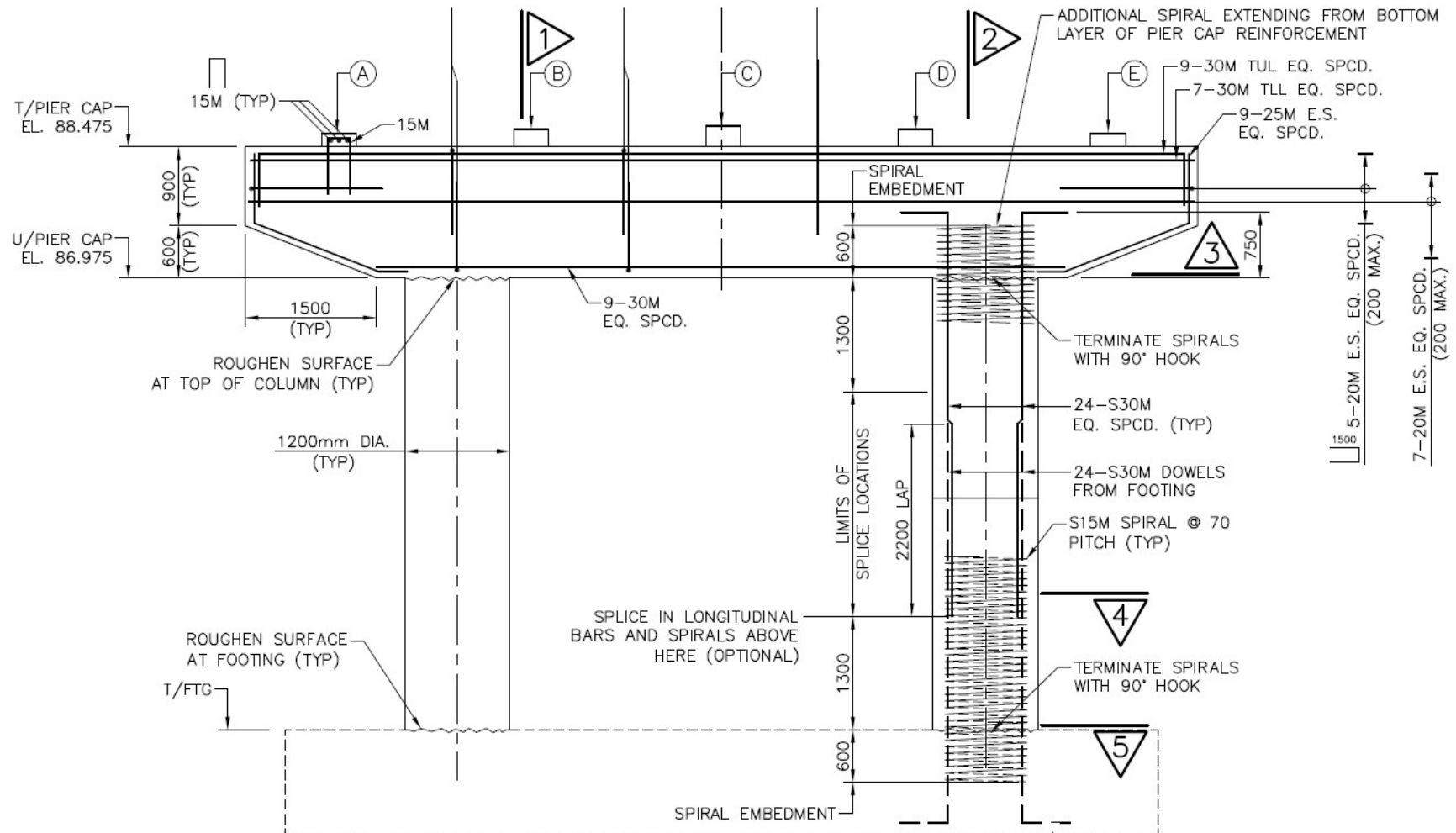


Figure C-4 - Avonmore bridge pier

APPENDIX D: Foundation modes of failure due to SSI effect

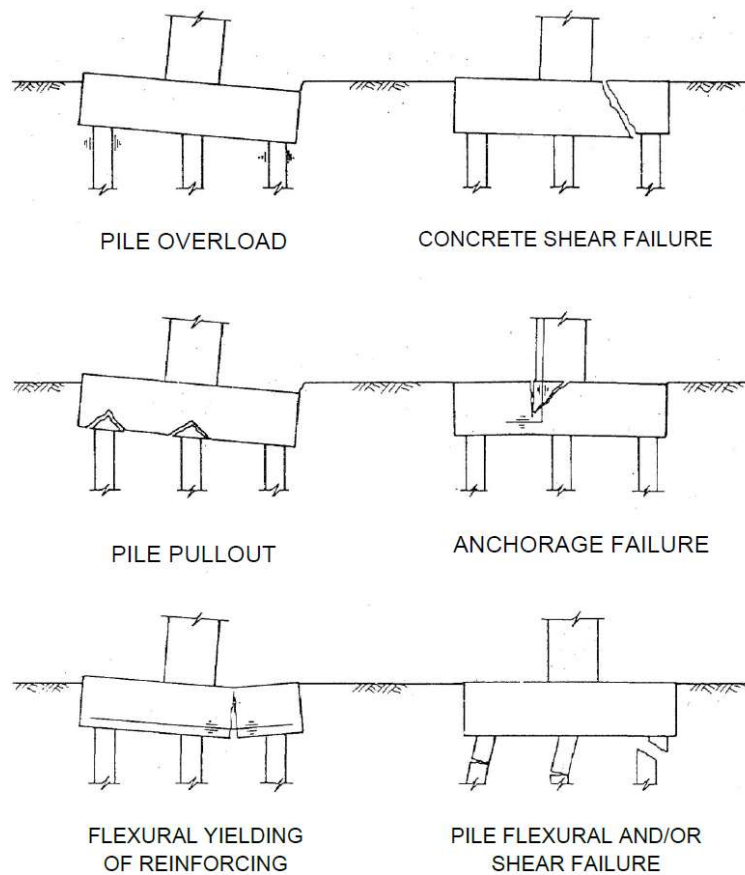
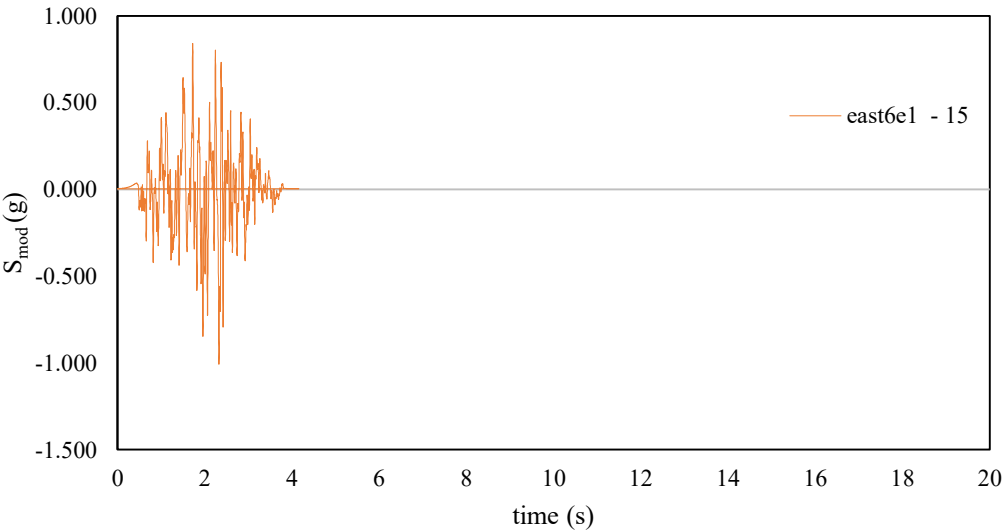
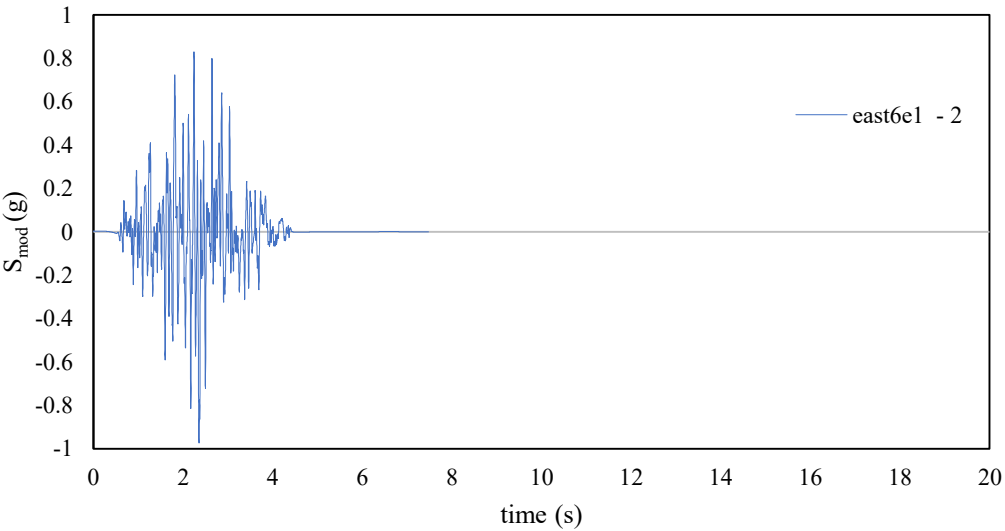
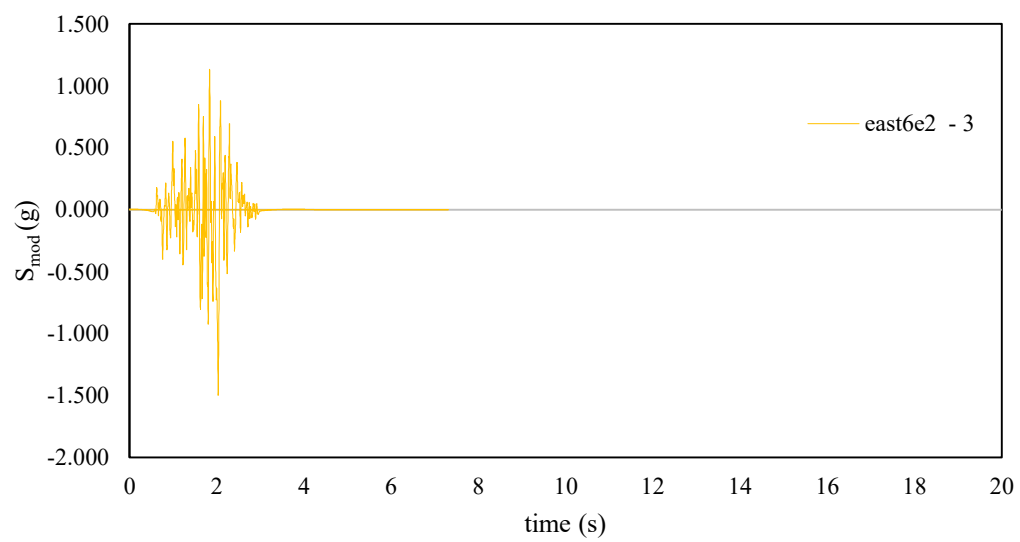
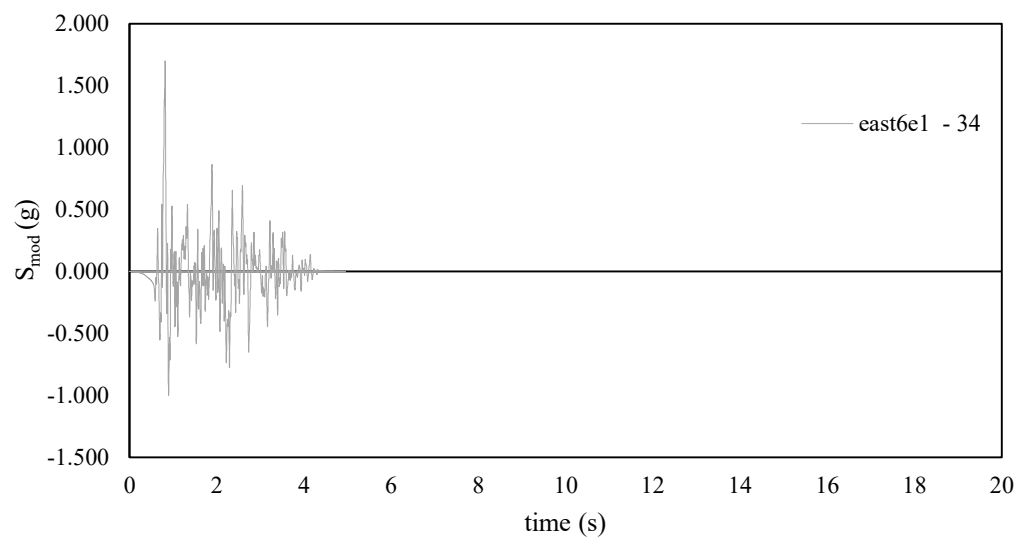
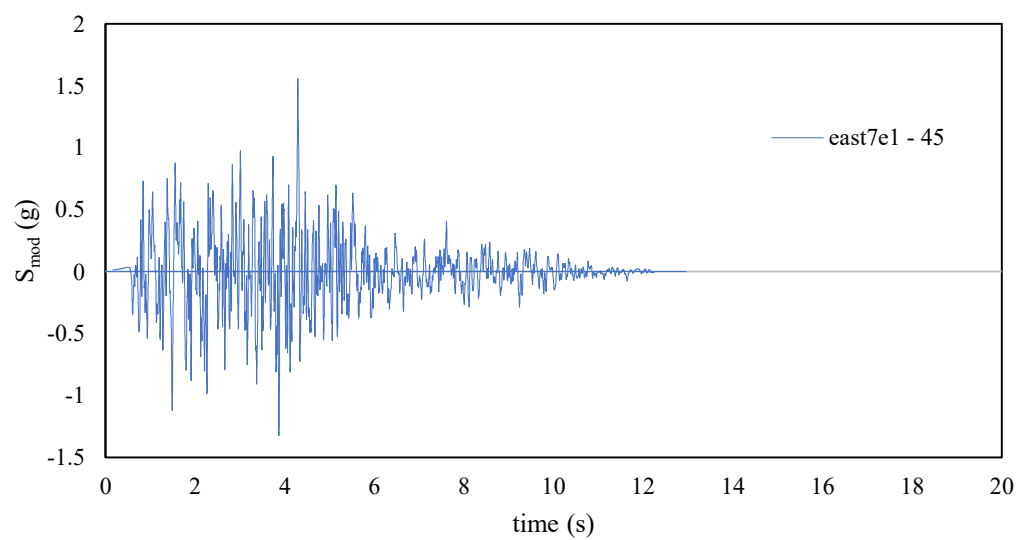
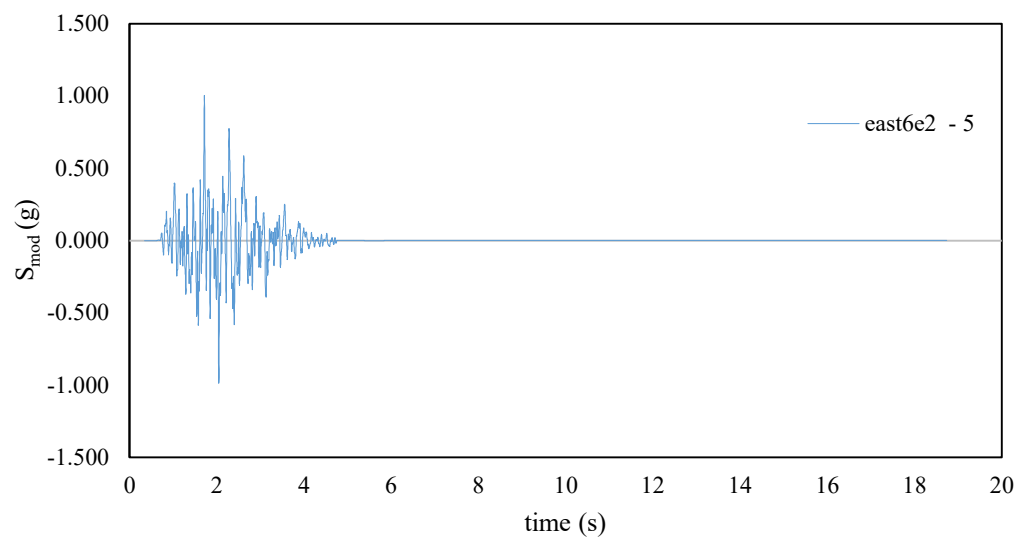


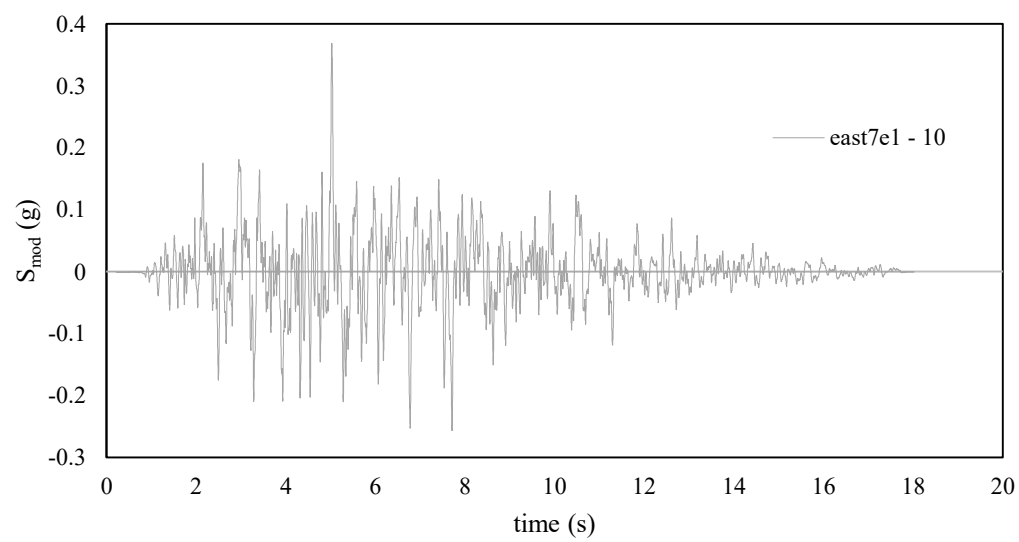
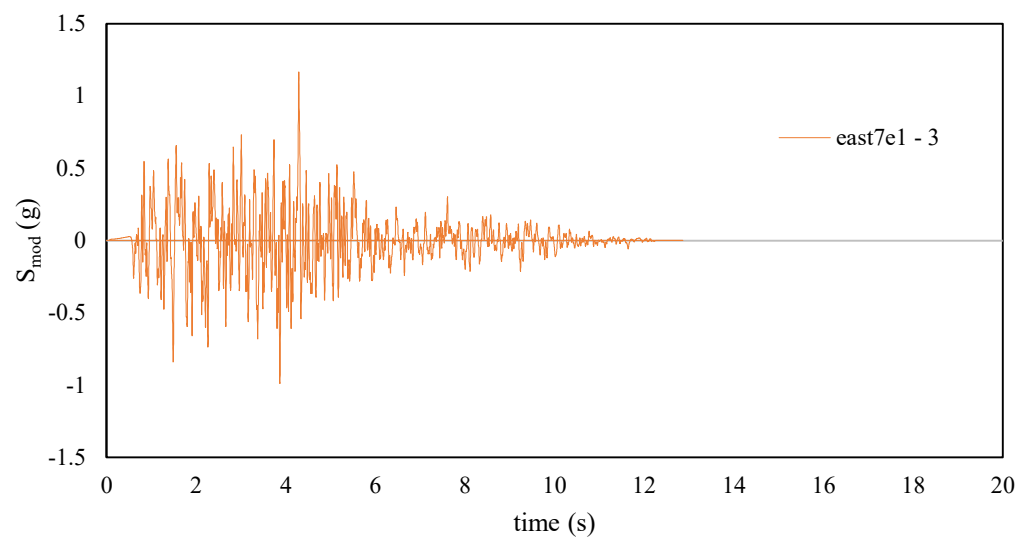
Figure D-1 – Structural failure modes of foundation due to SSI (Federal Highway Administration and Office of Highway Information Management 1995)

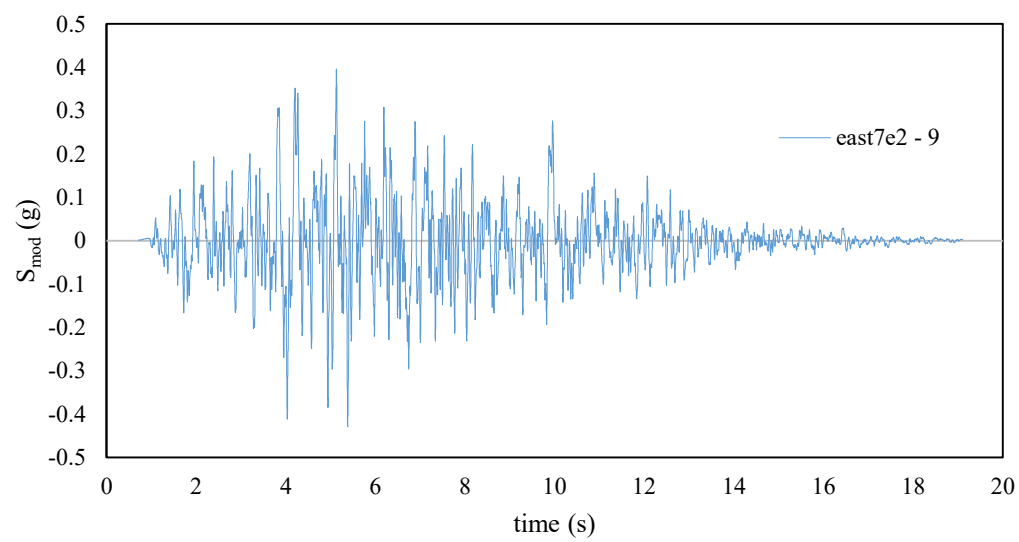
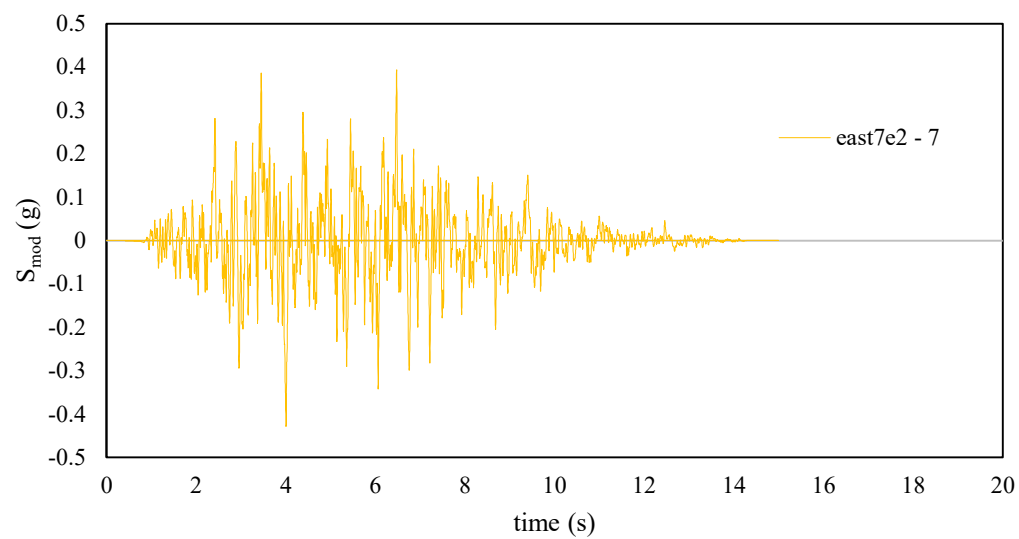
APPENDIX E: Scaled time history records

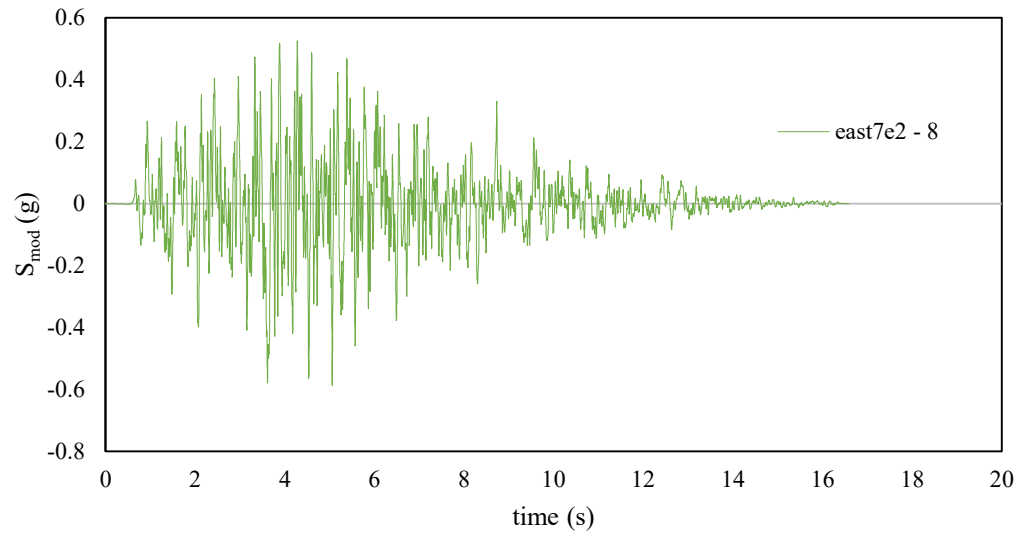




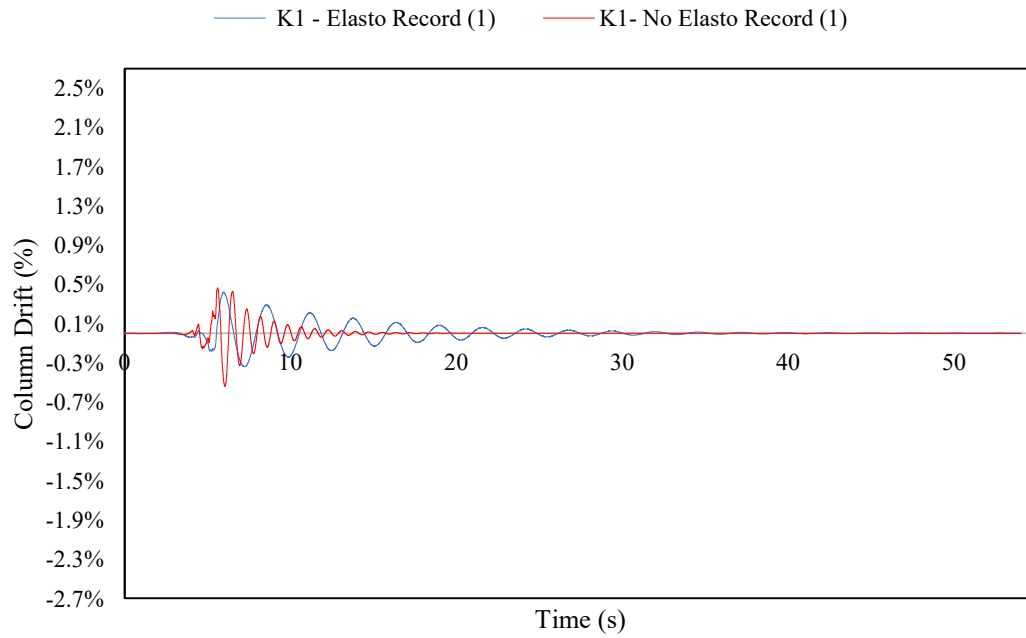


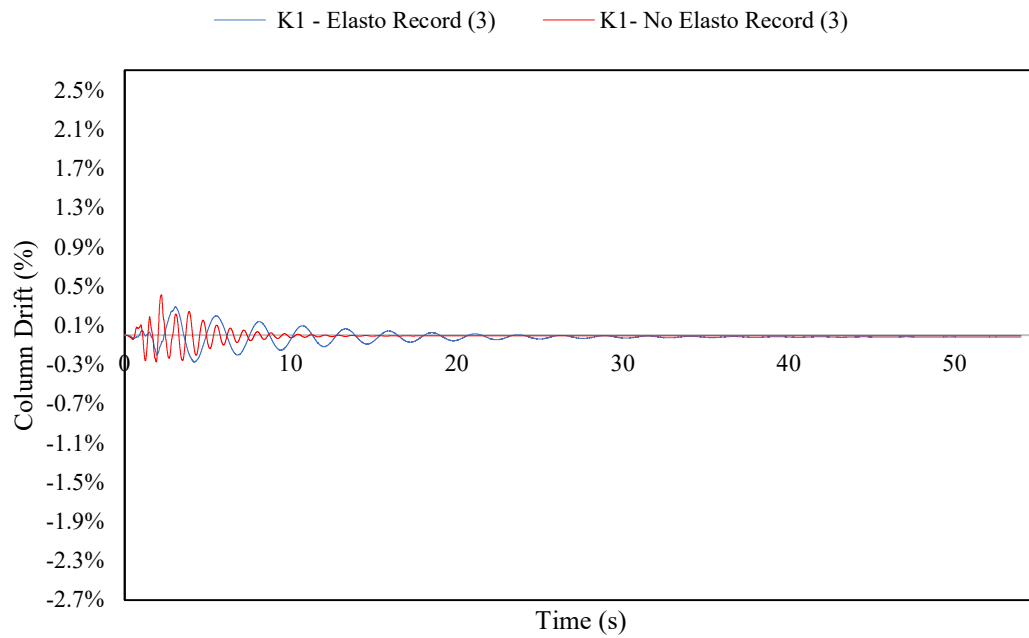
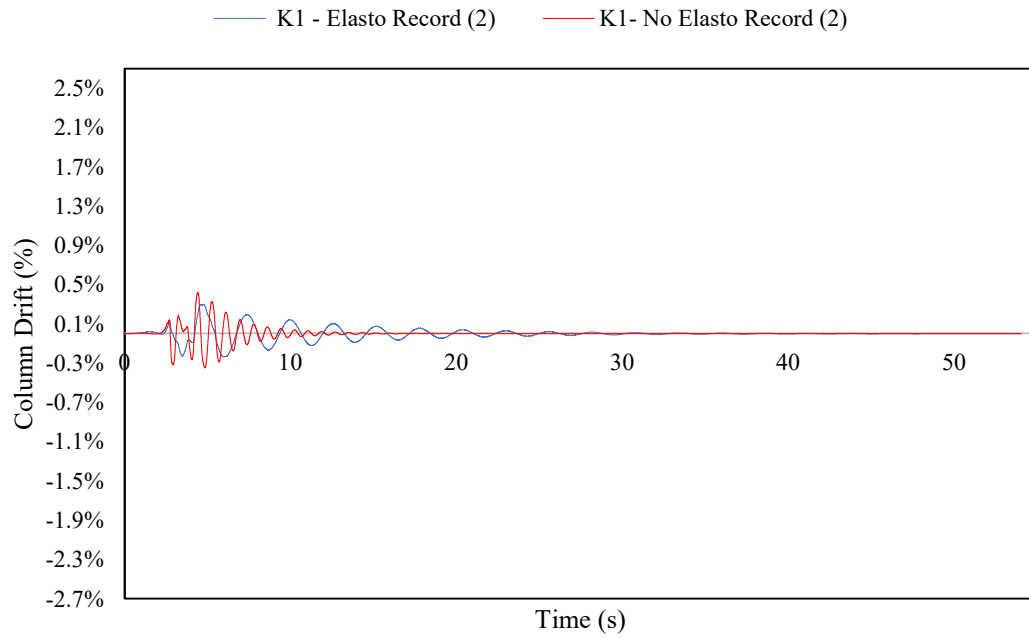


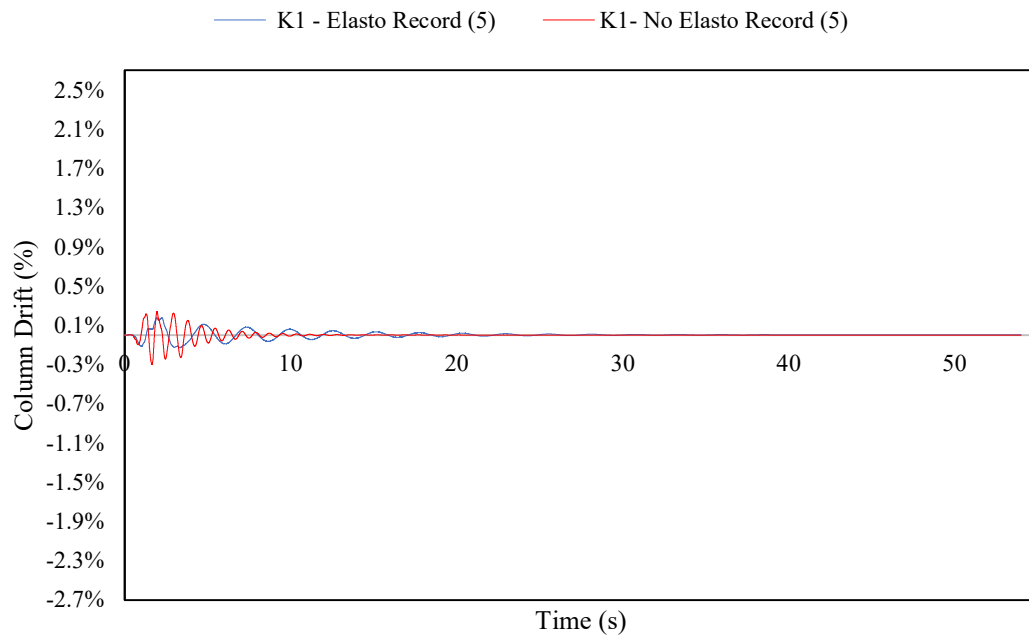
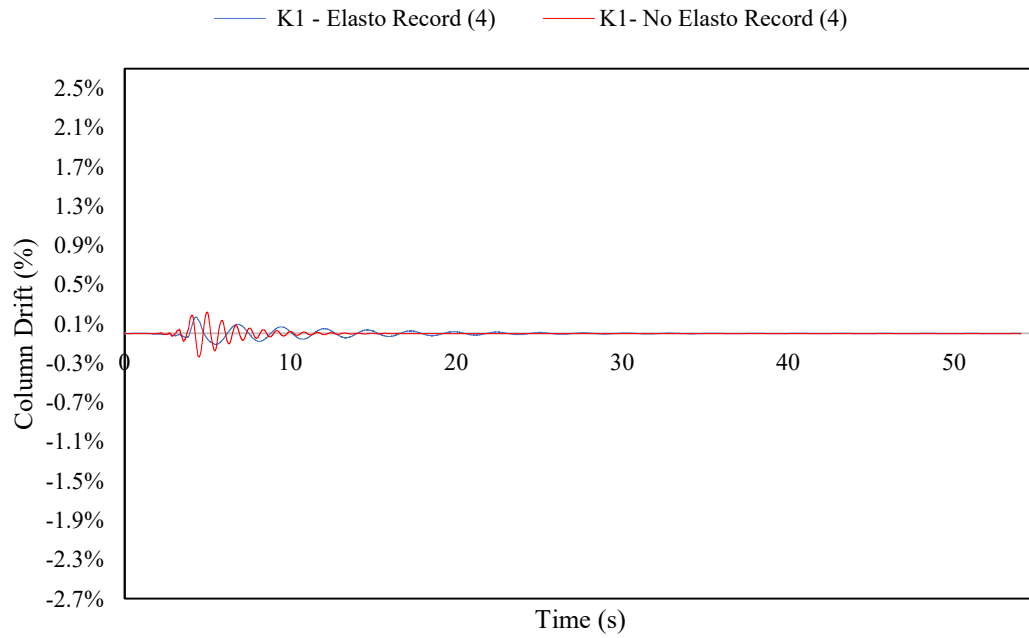


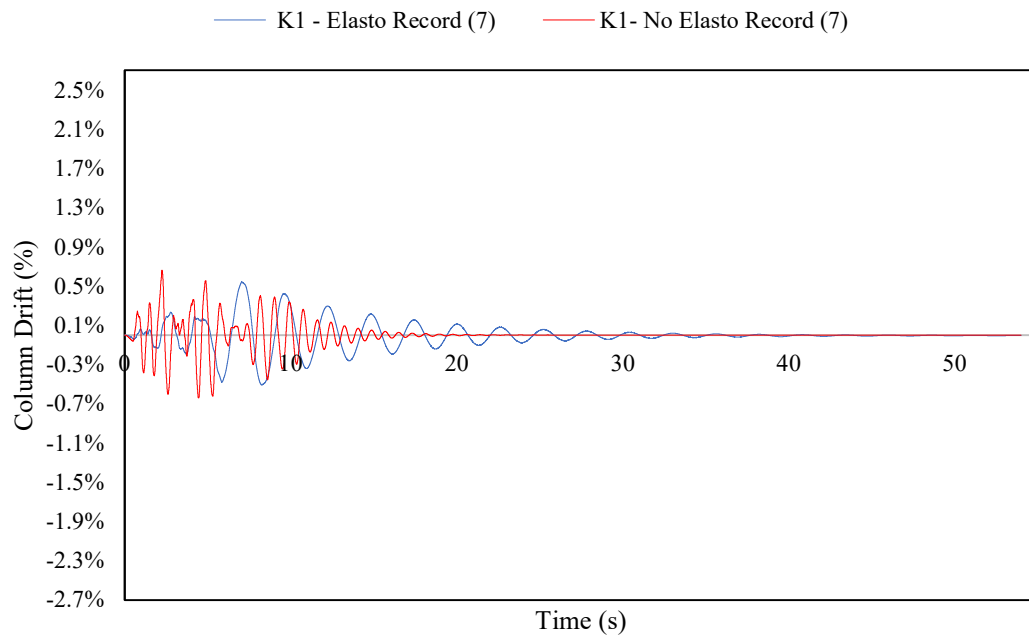
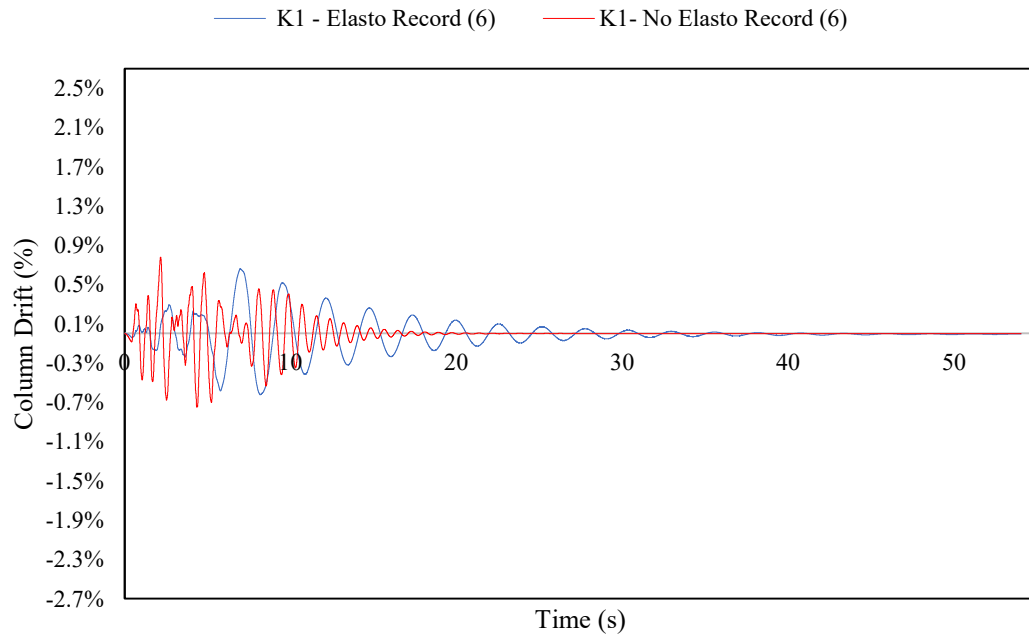


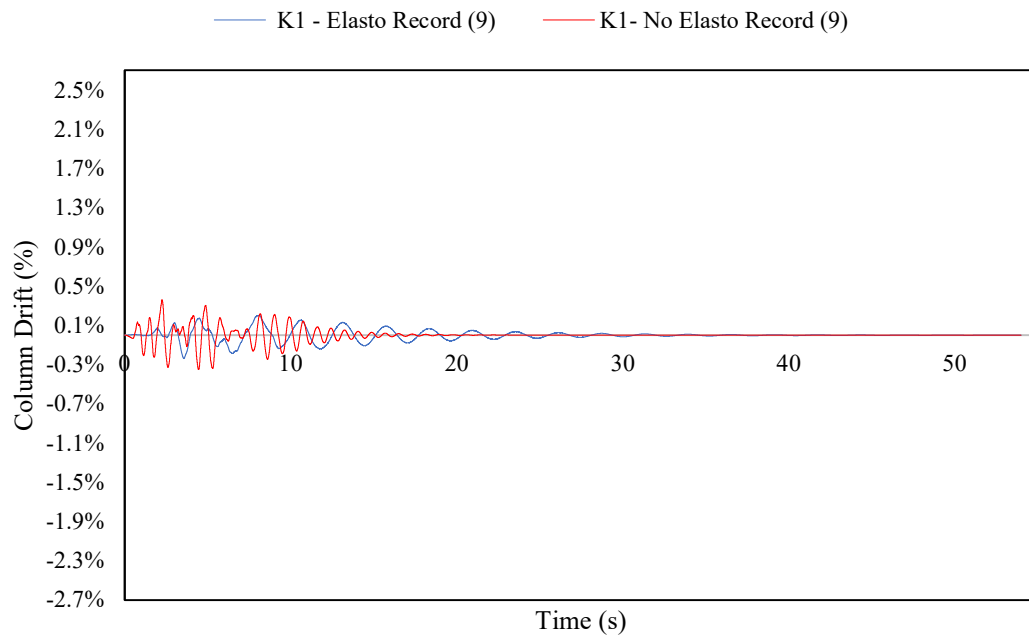
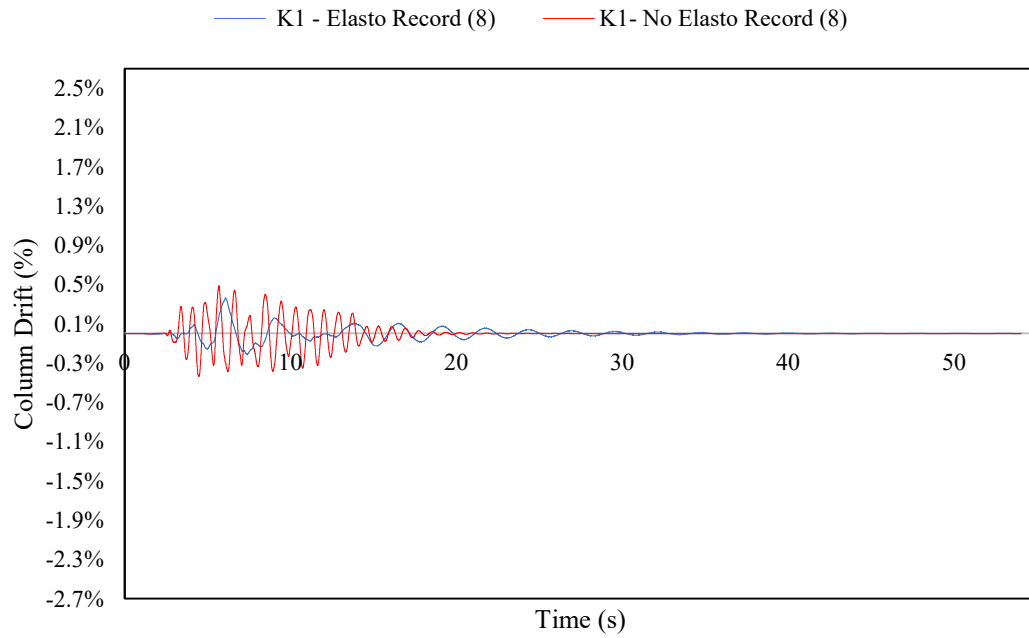
APPENDIX F: Drift history response of bridge pier varying substructure stiffness

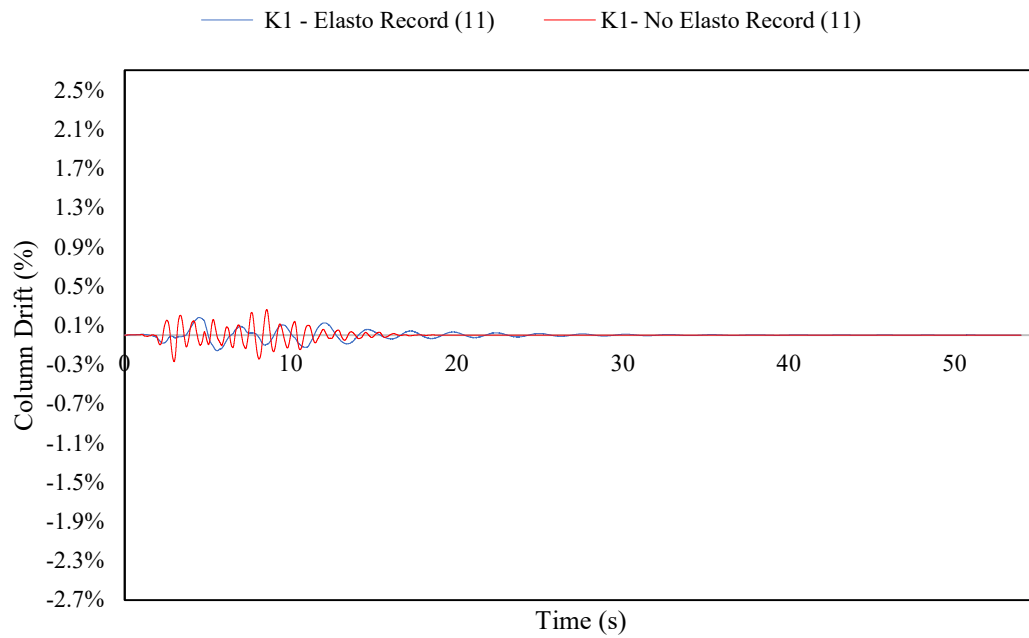
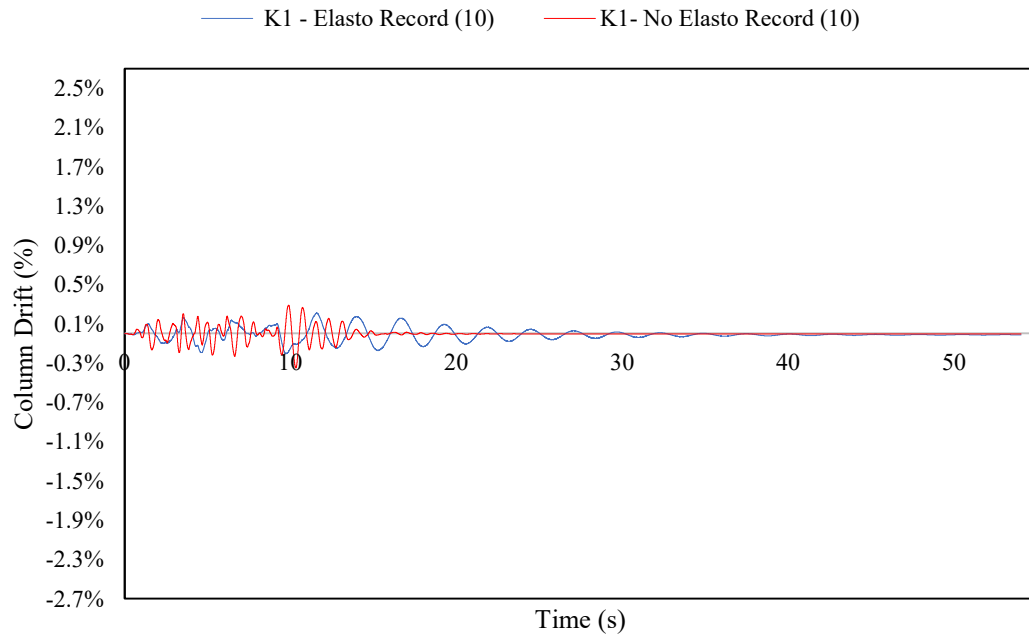


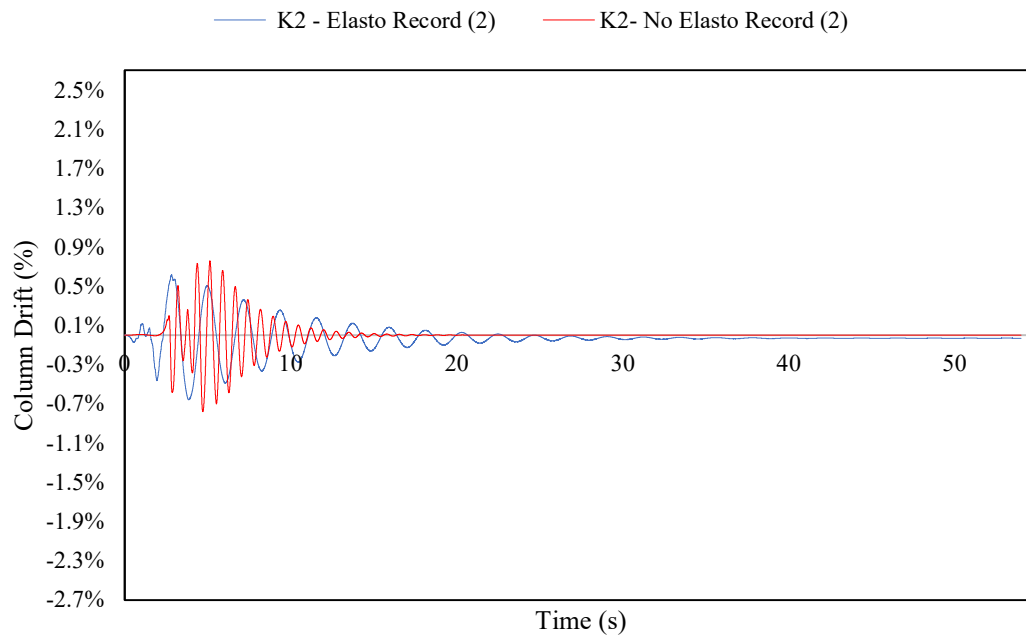
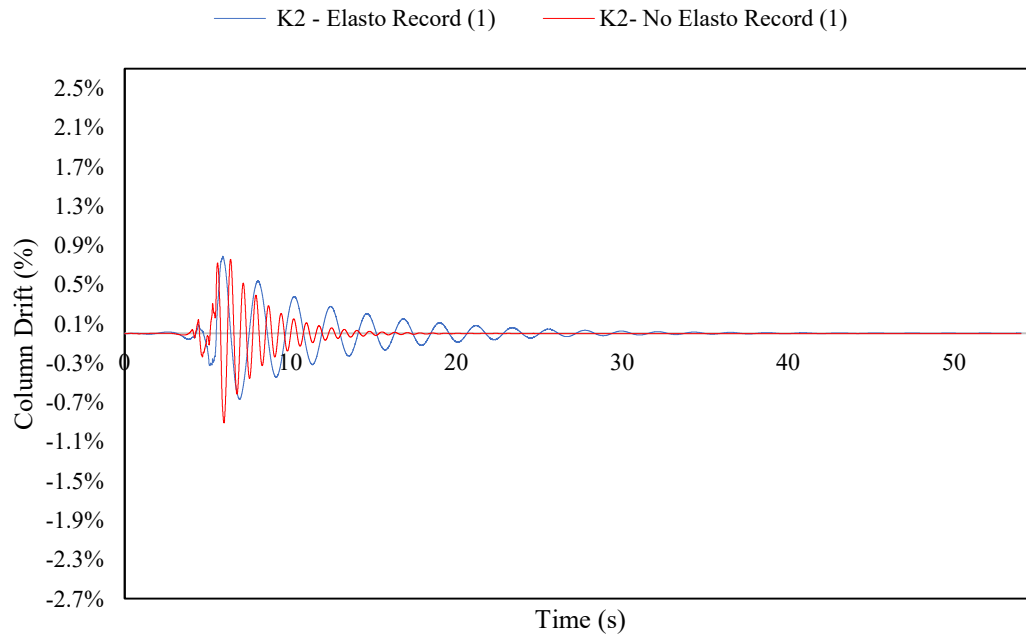


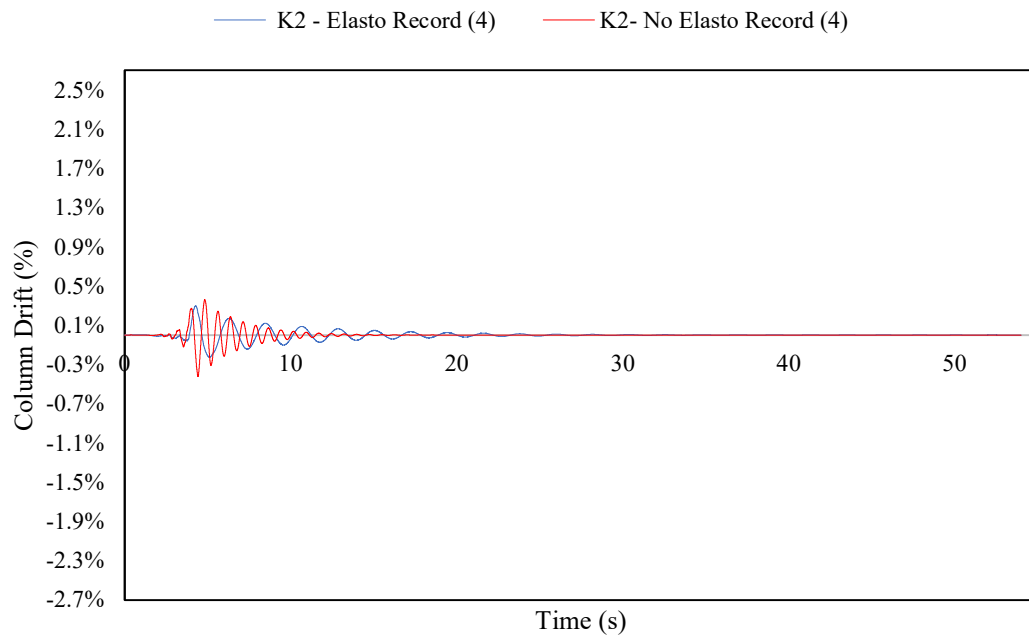
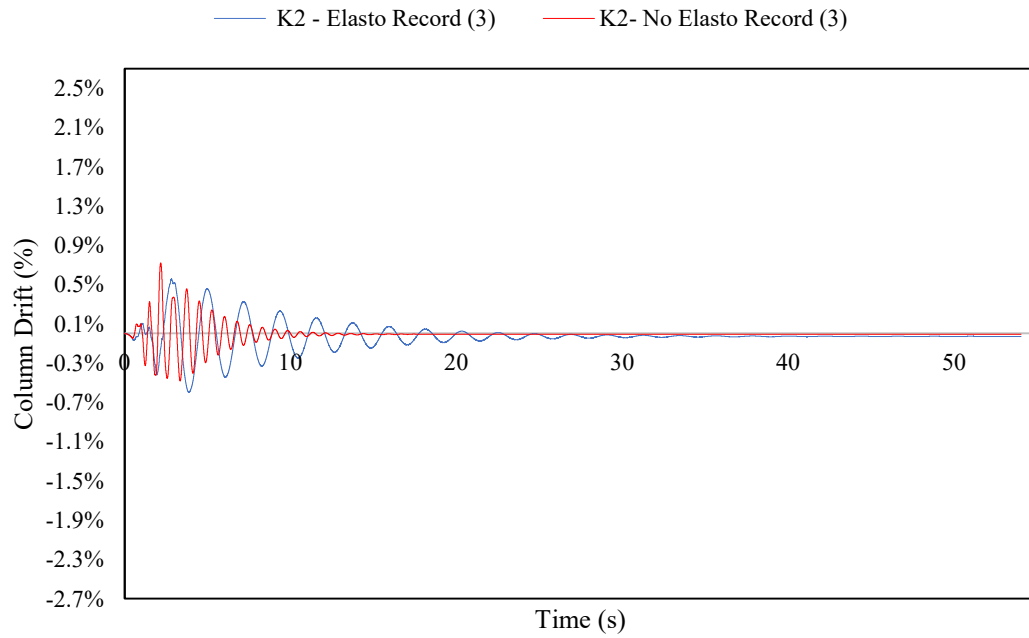


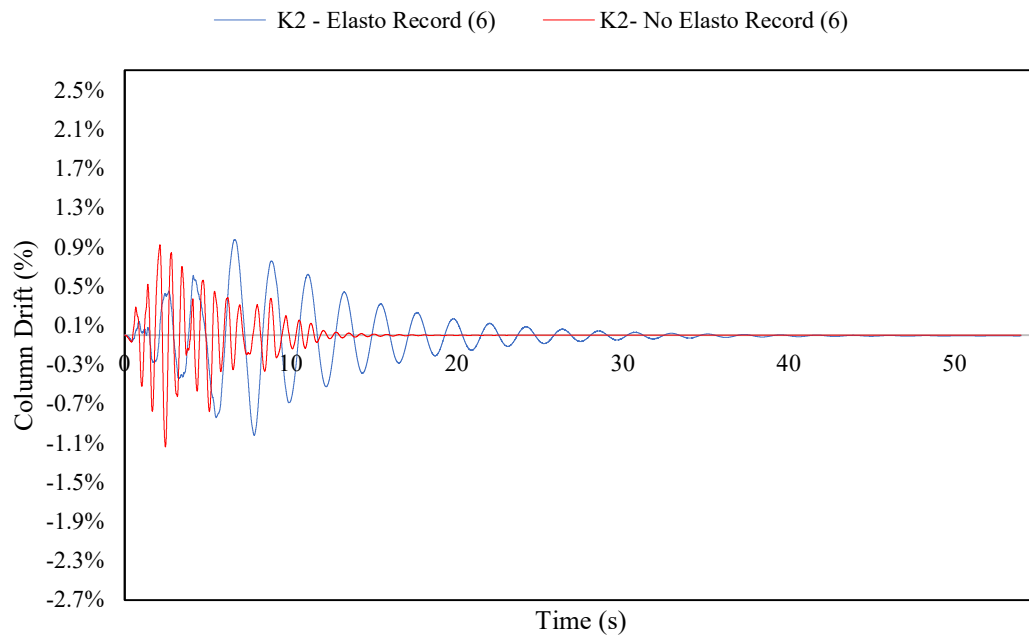
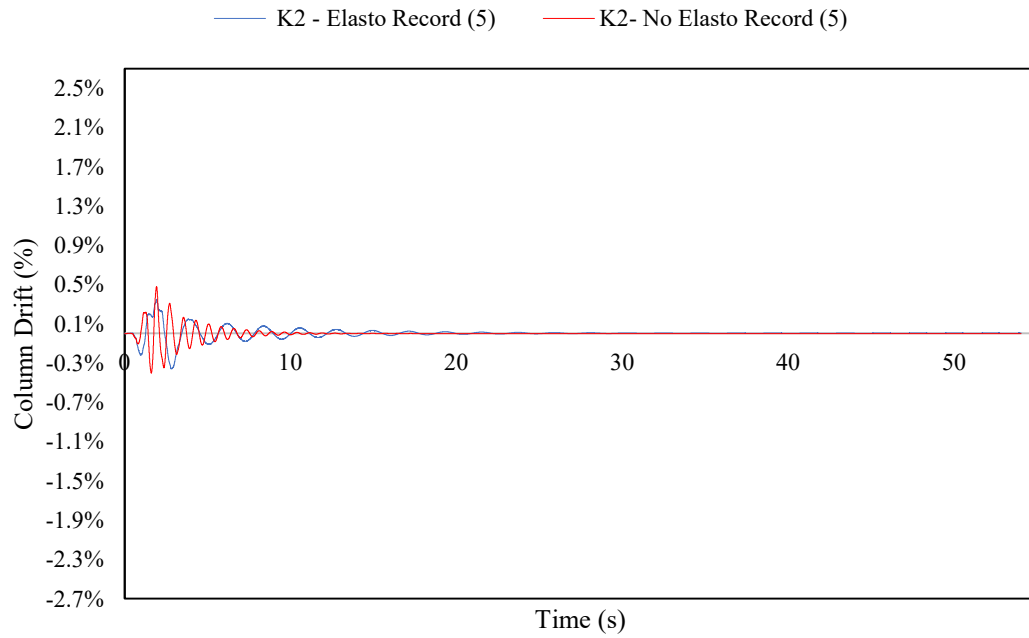


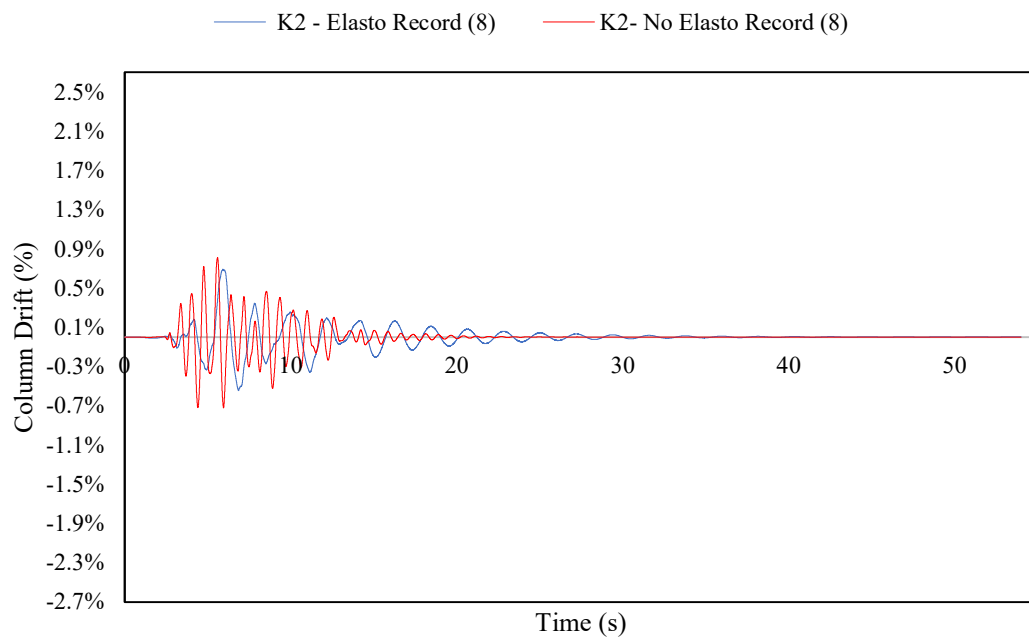
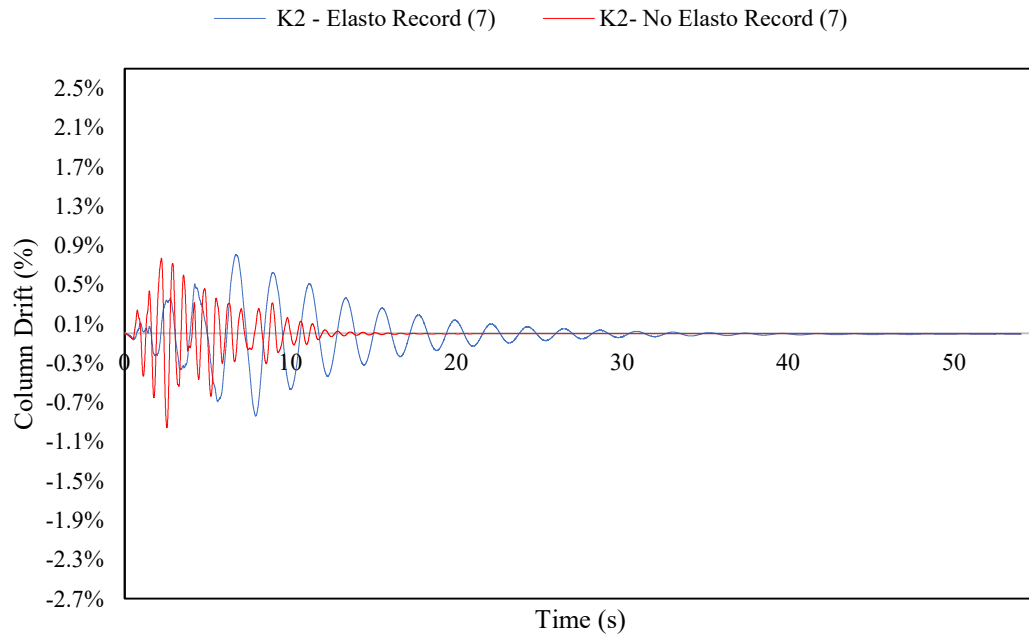


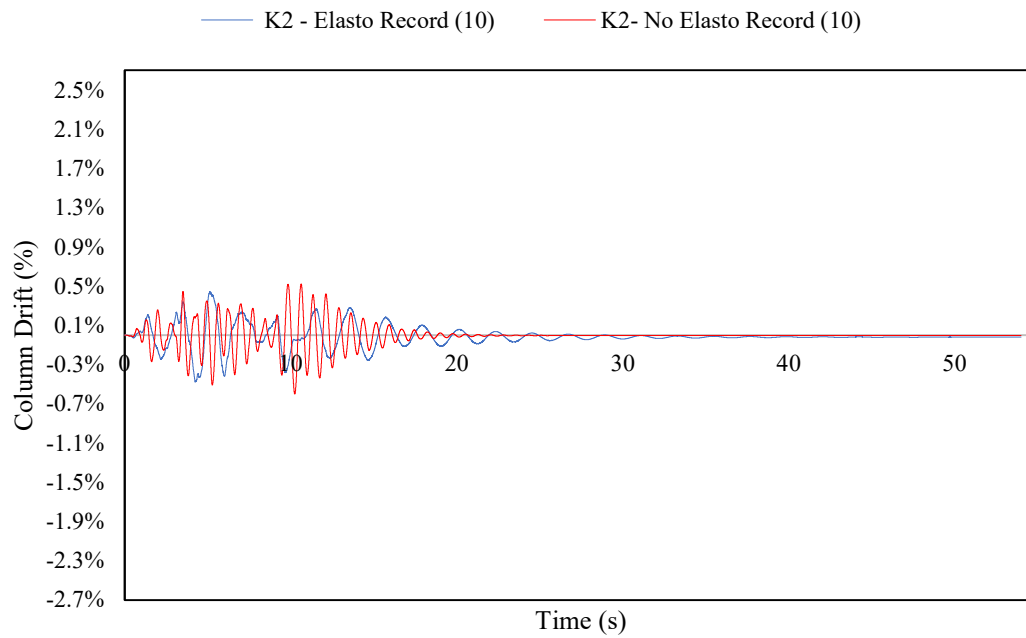
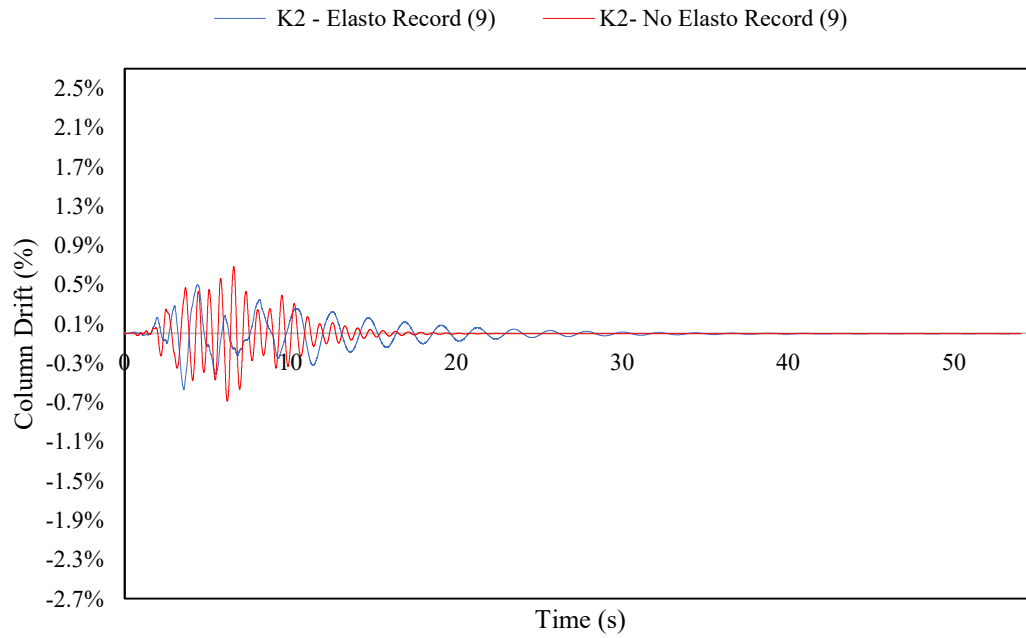


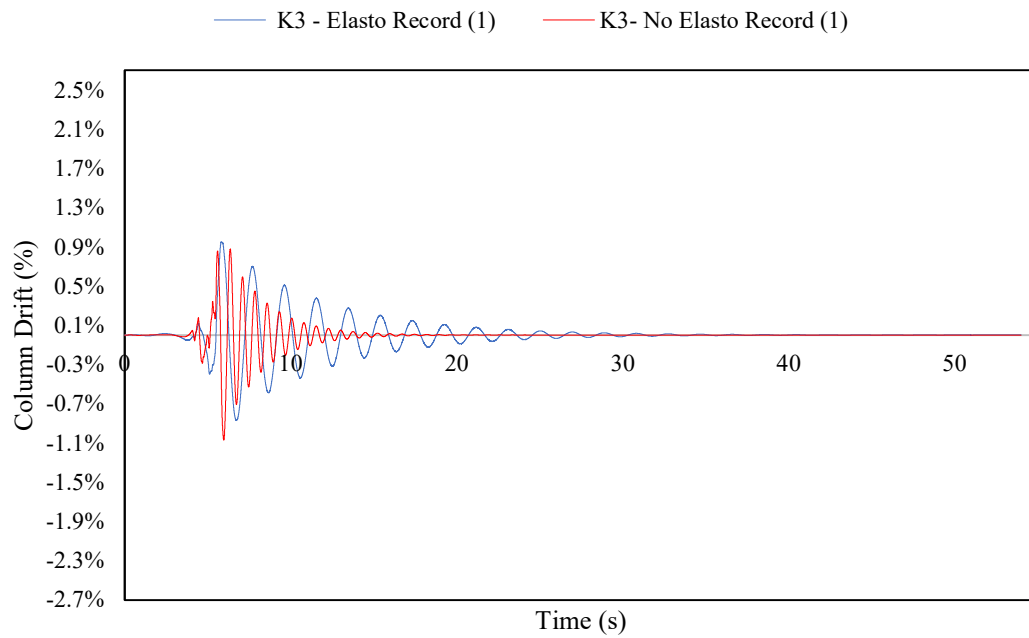
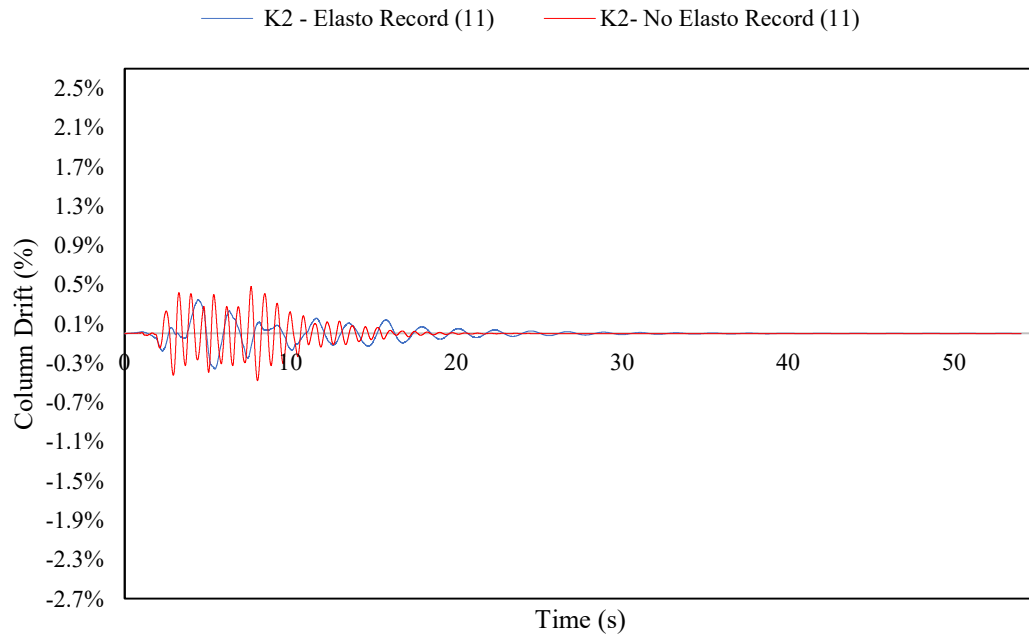


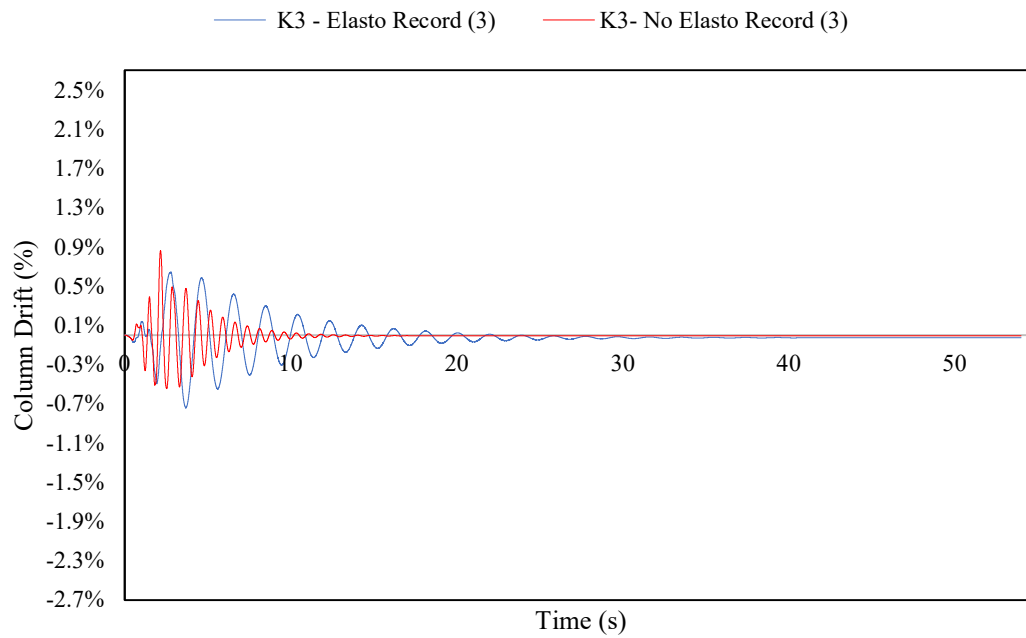
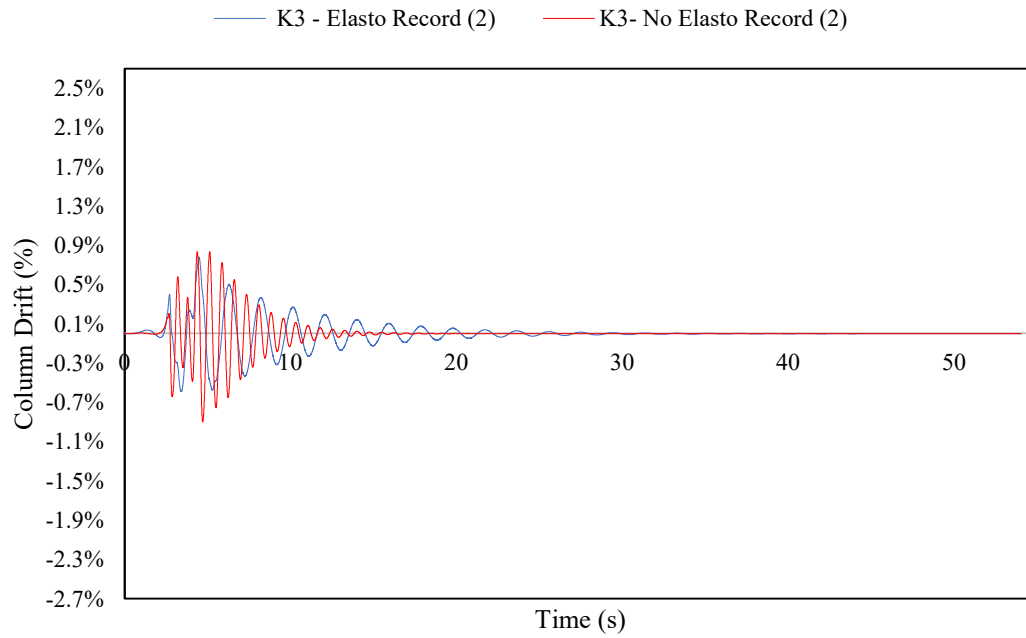


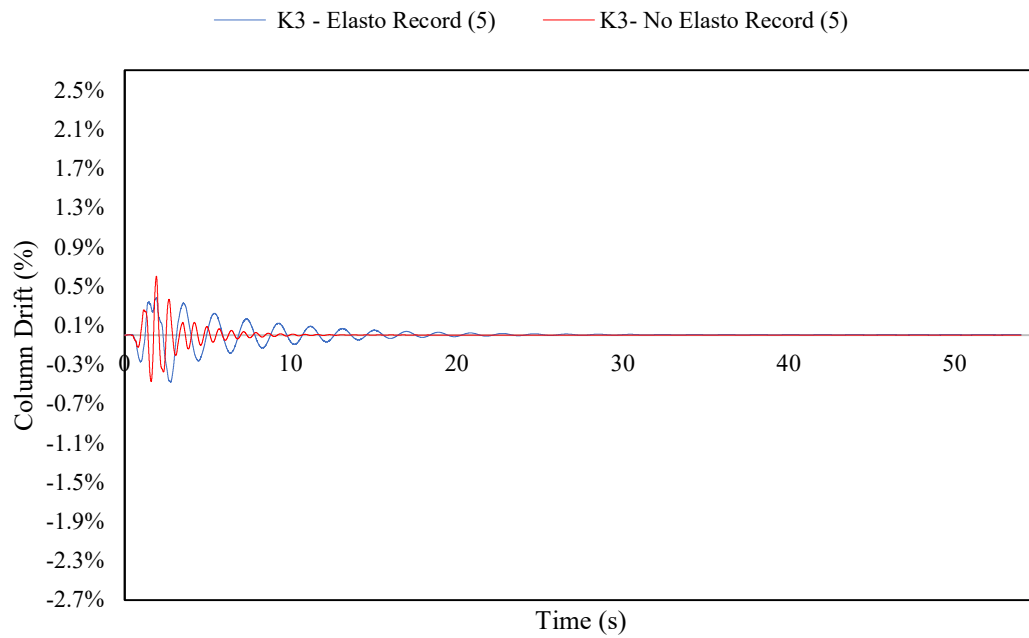
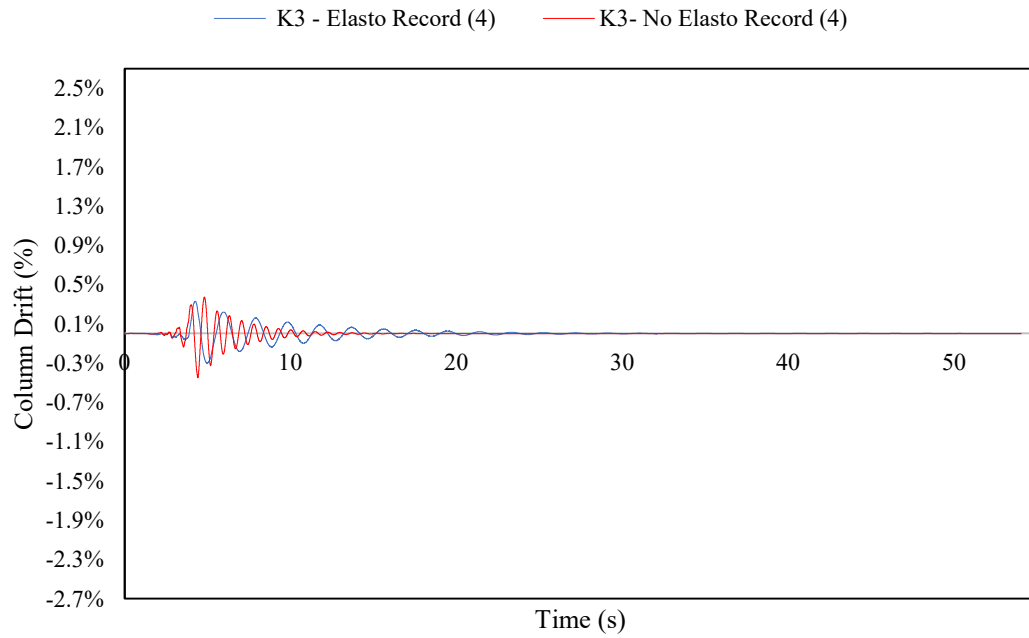


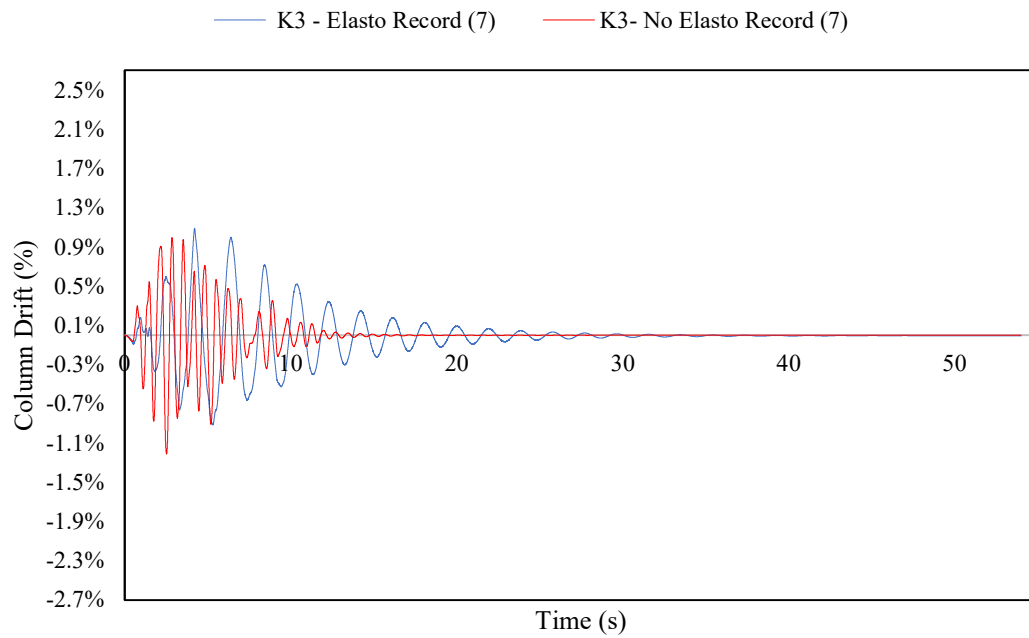
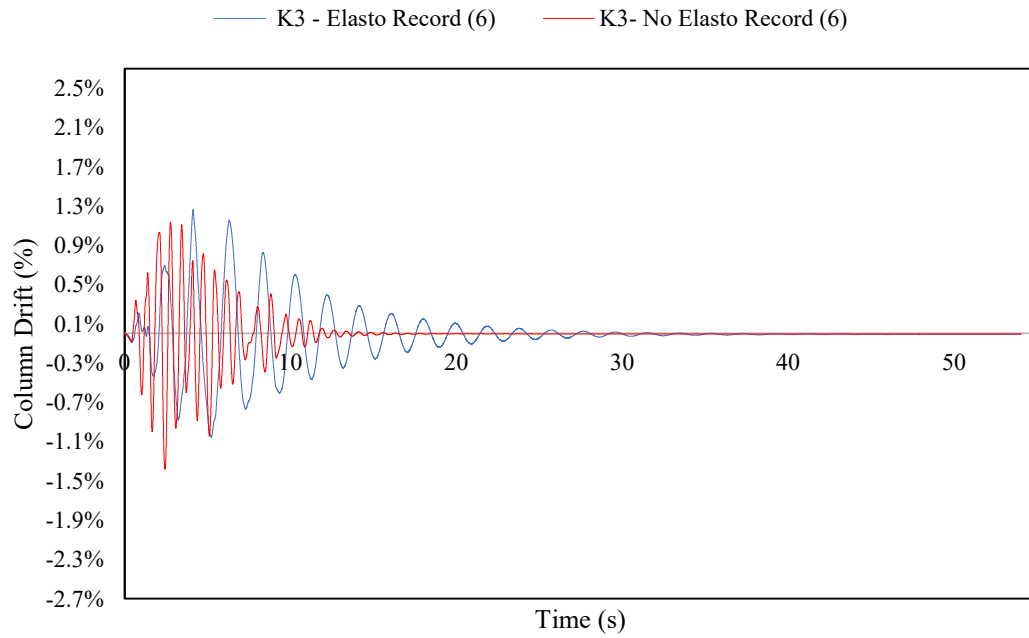


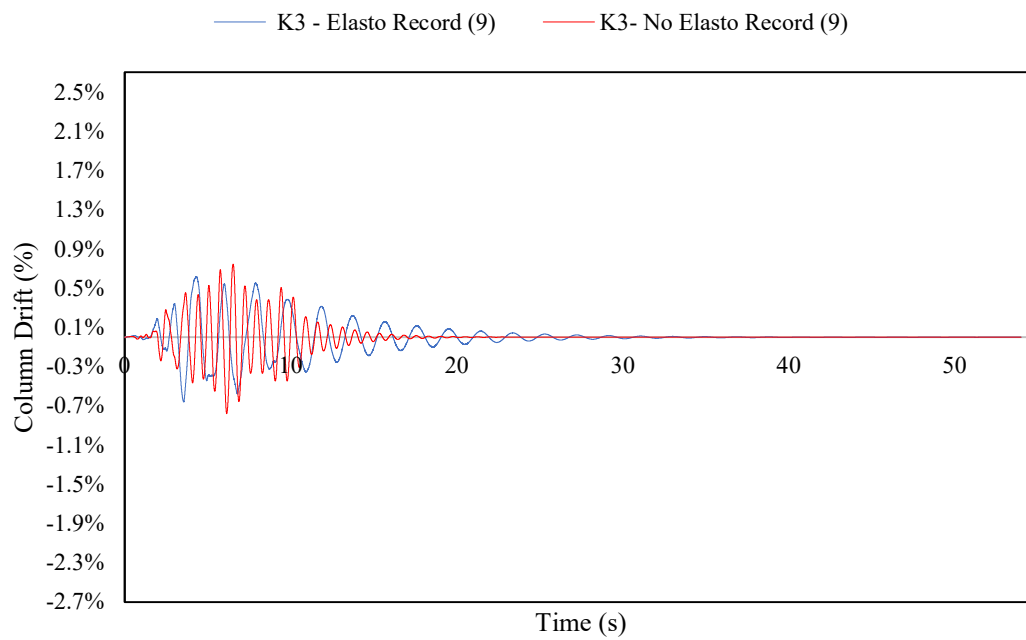
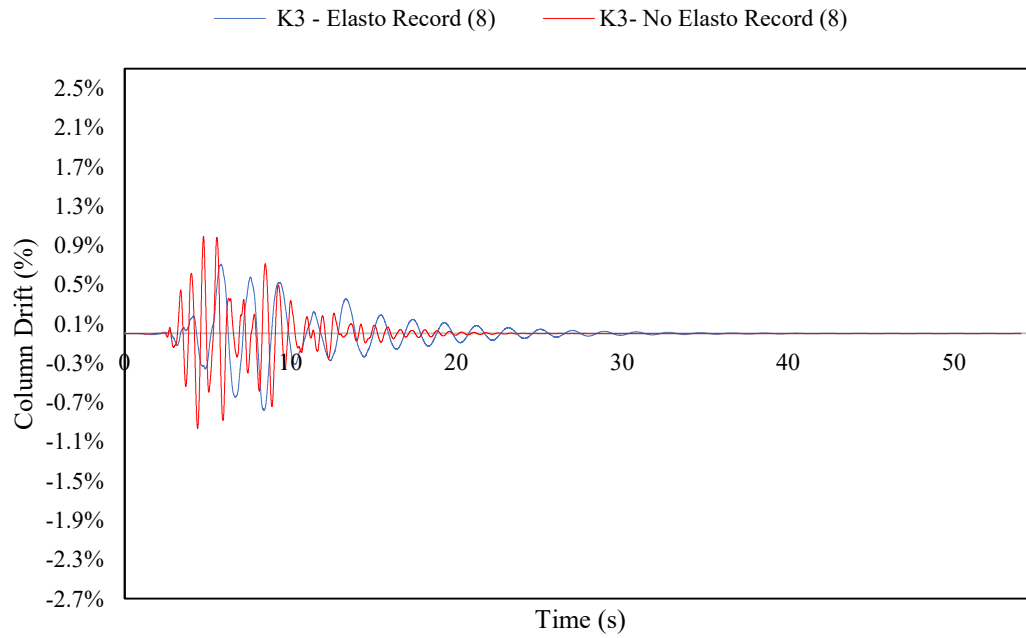


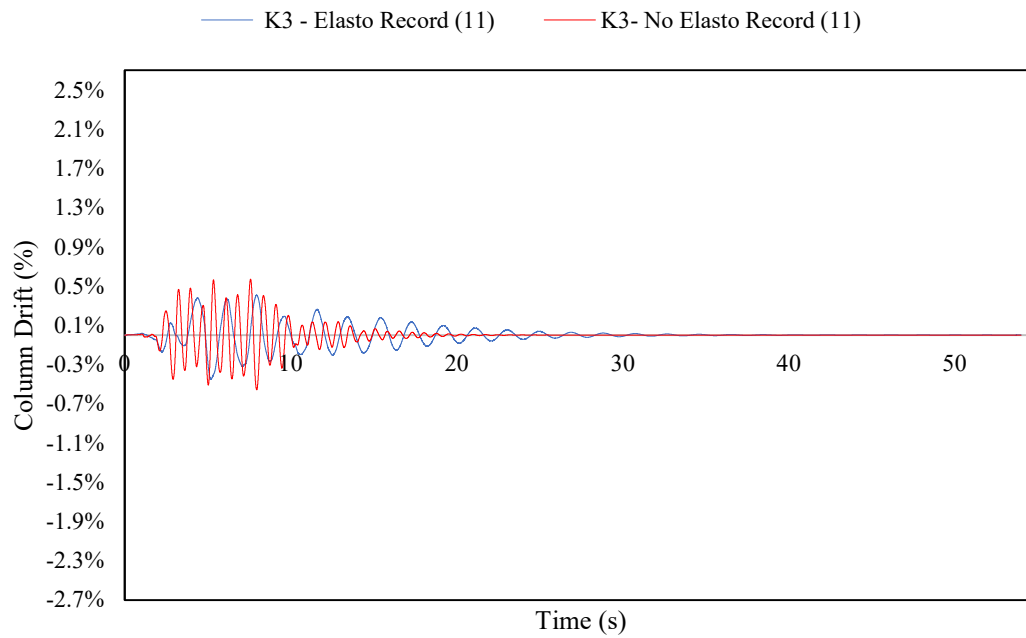
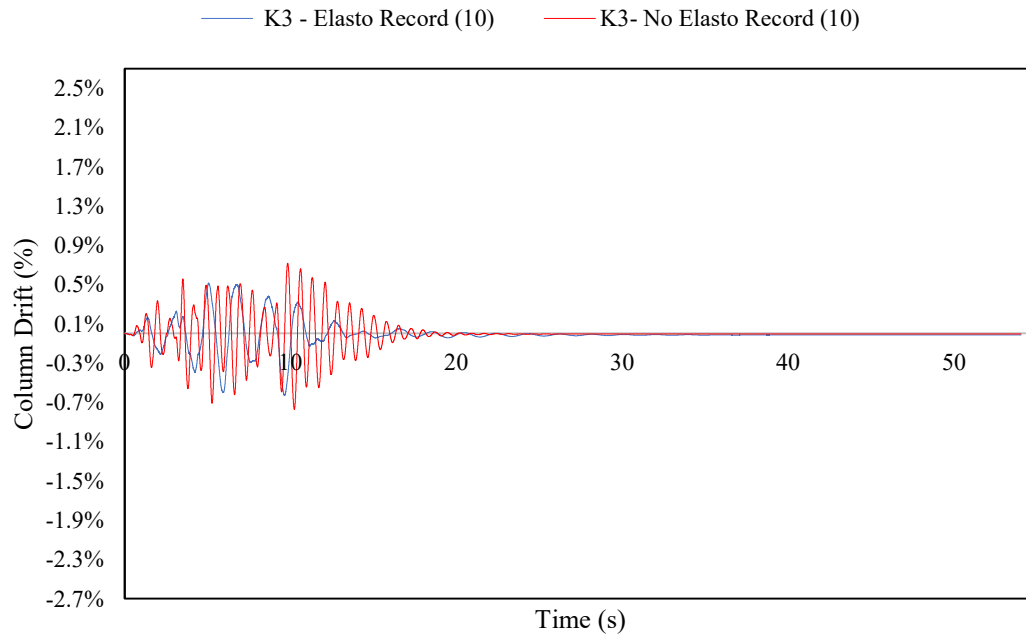


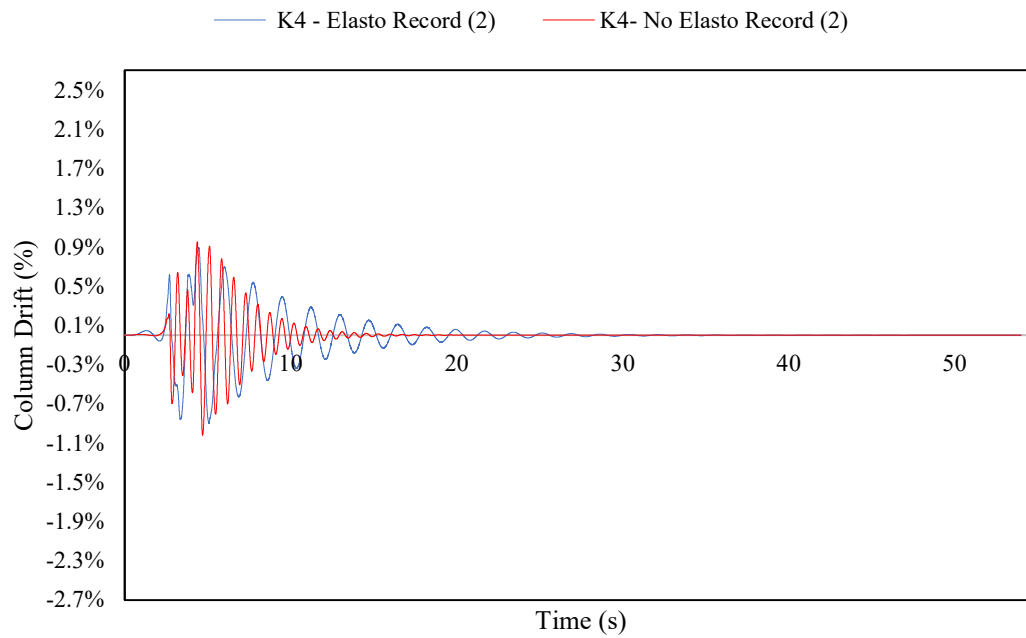
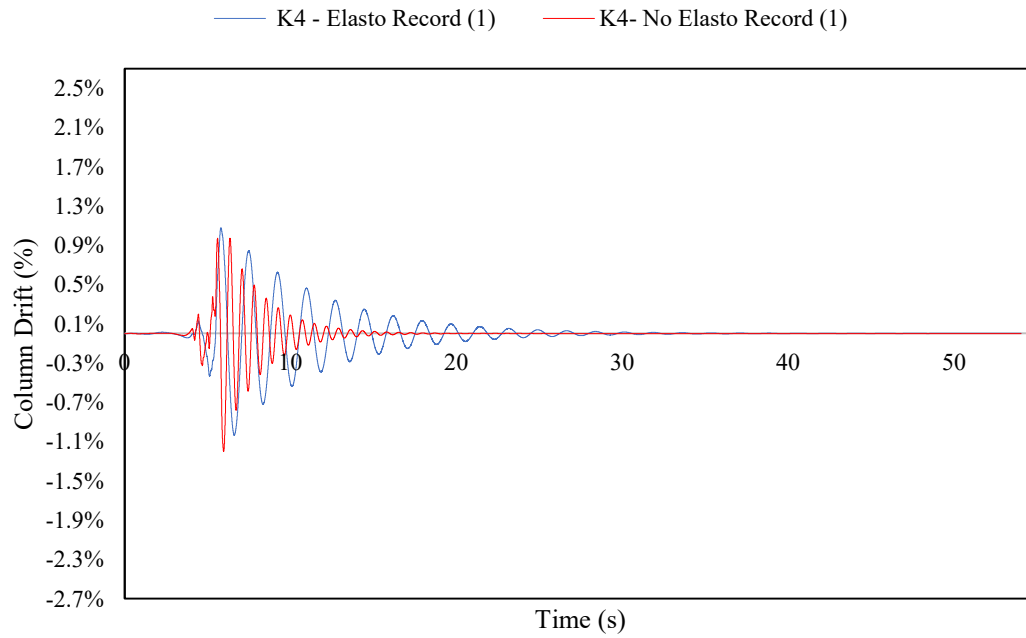


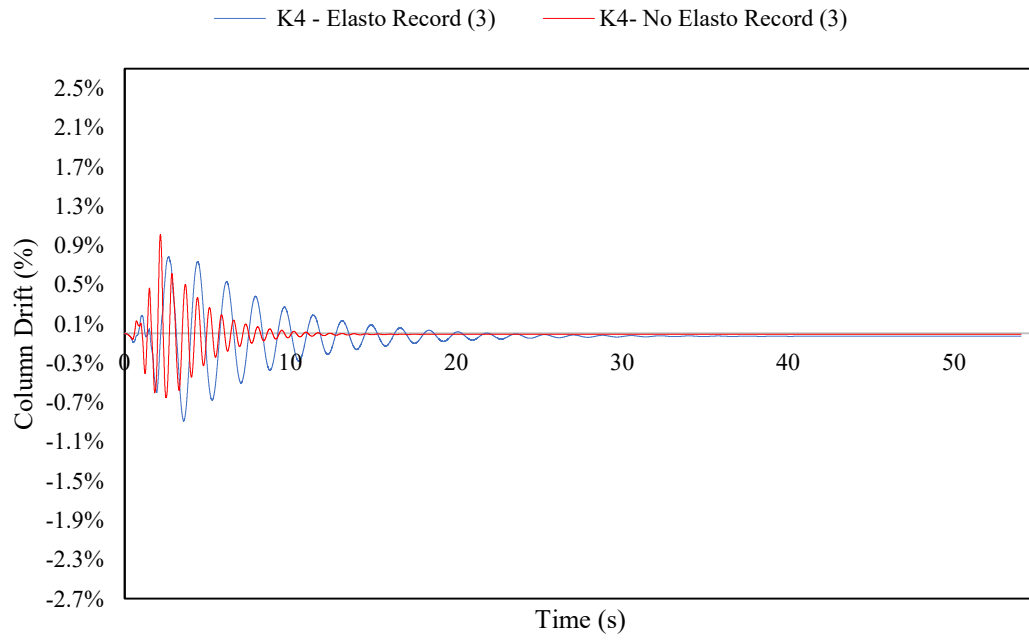


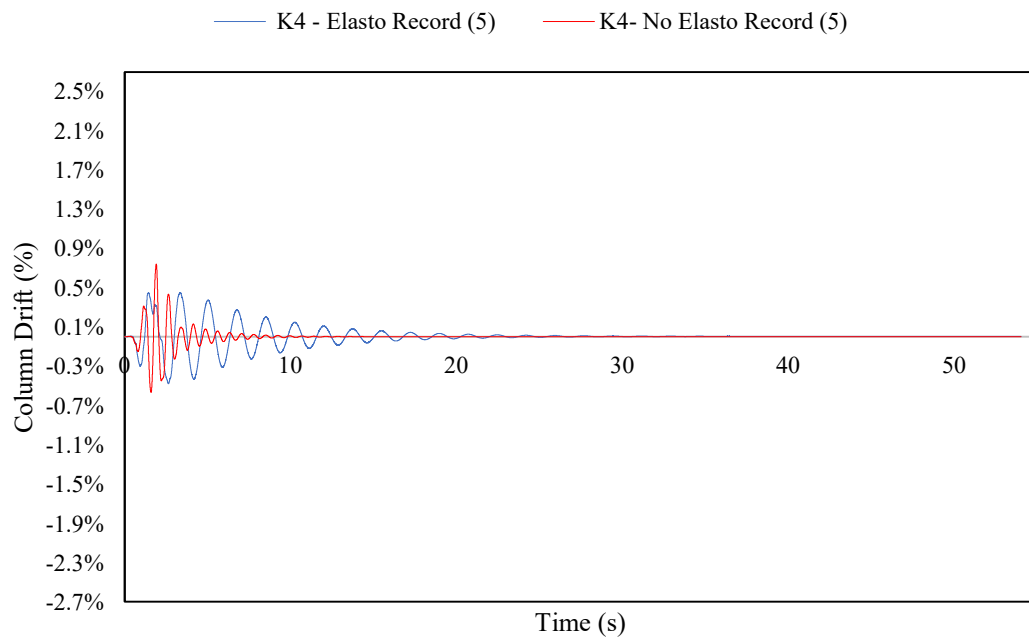
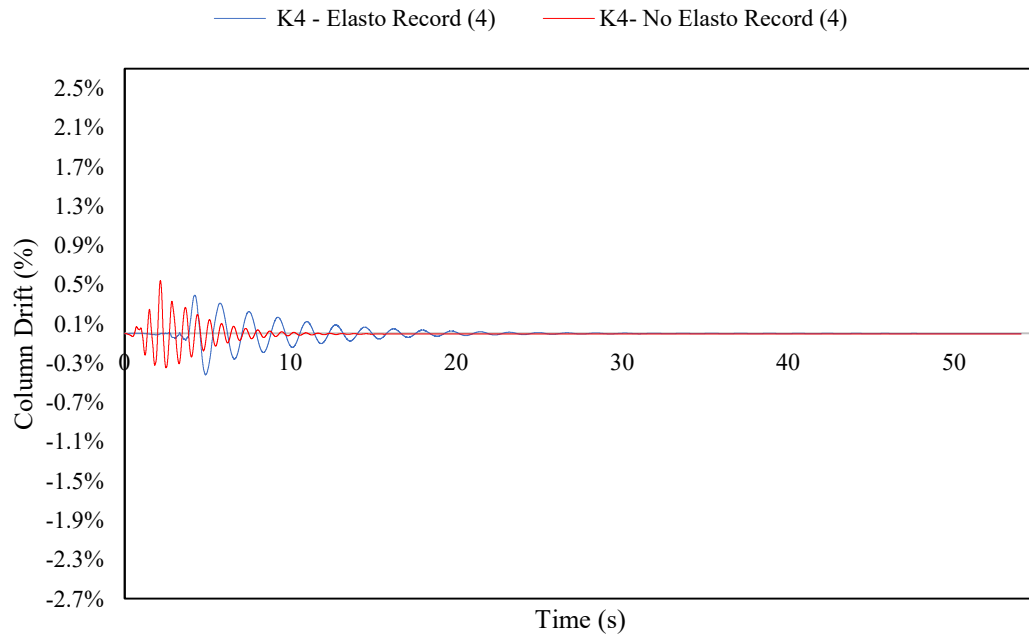


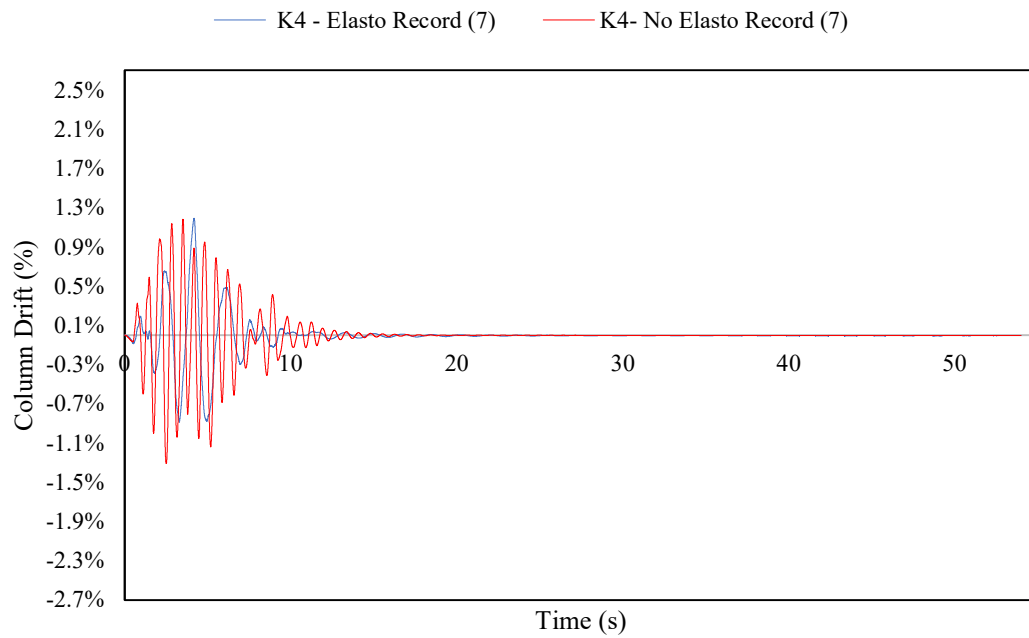
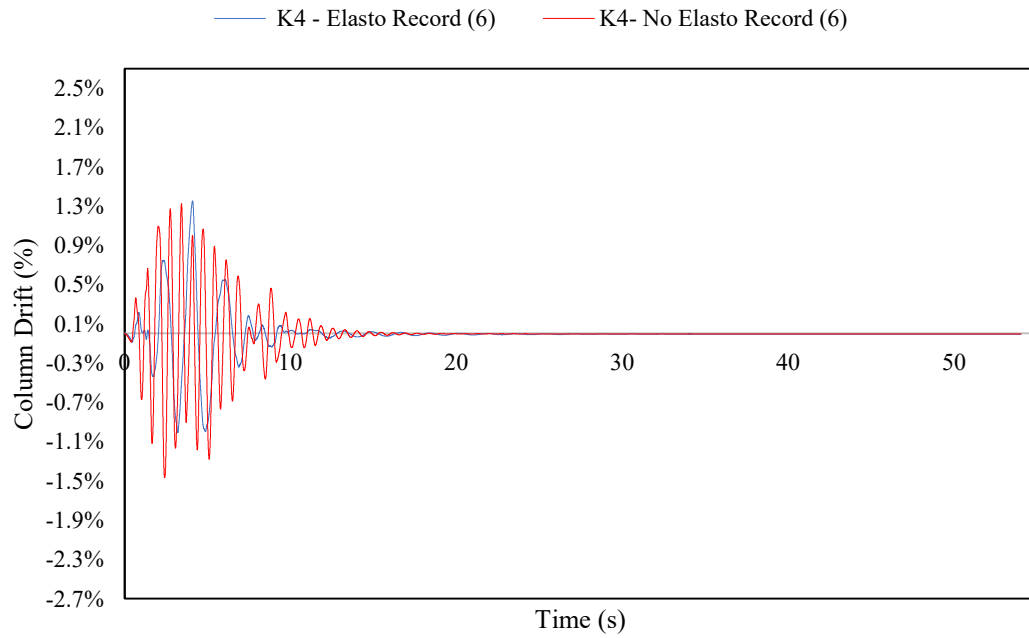


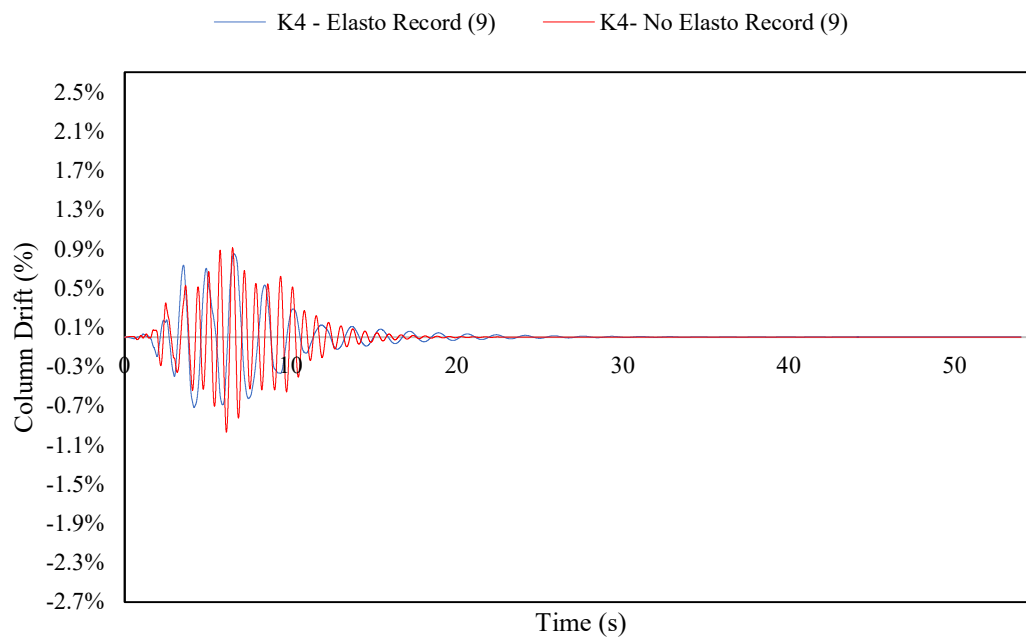
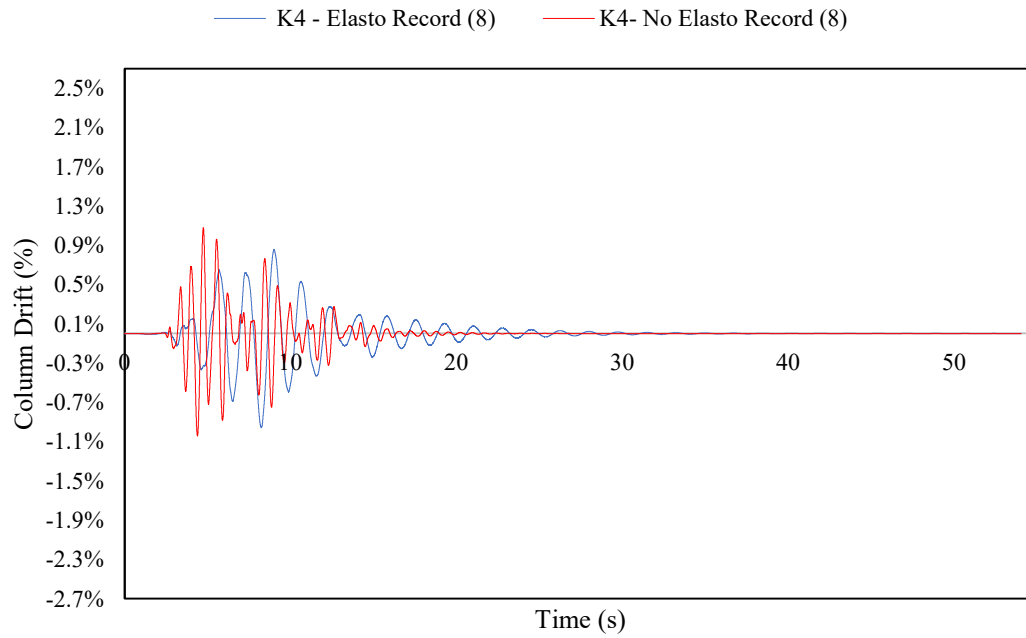


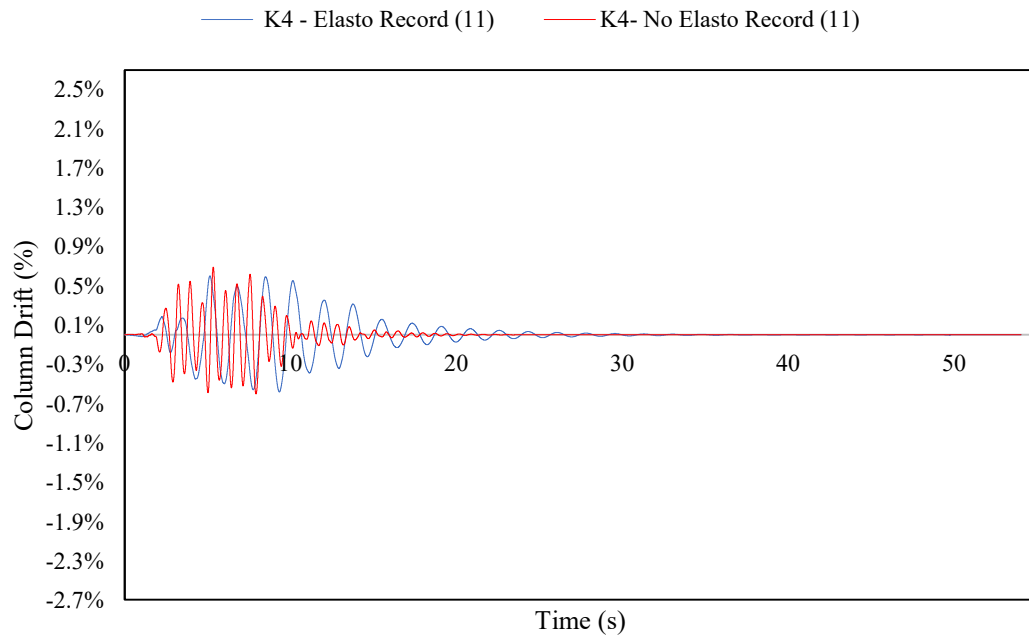
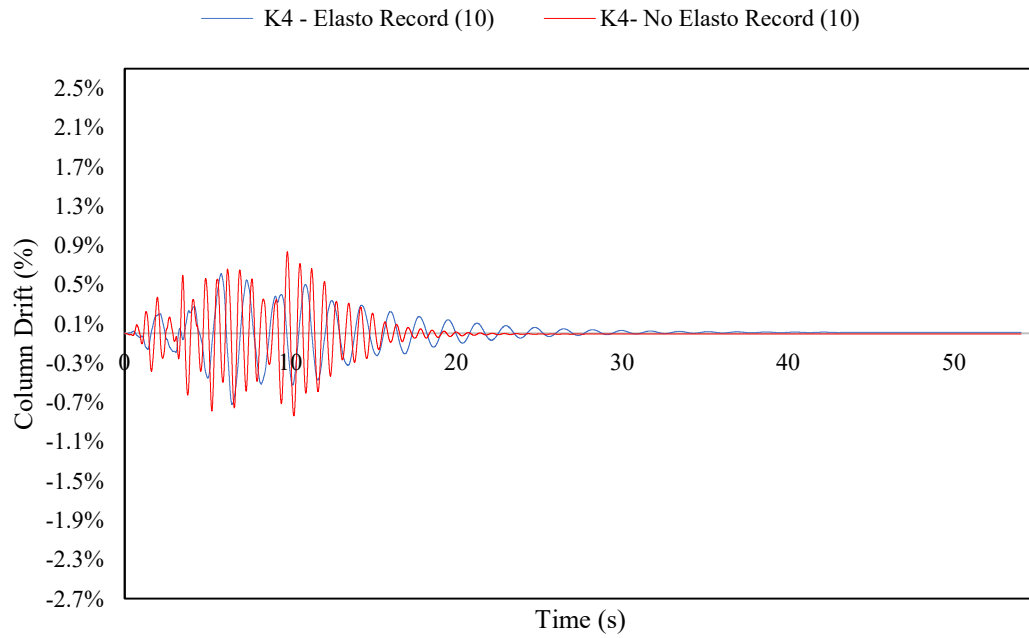


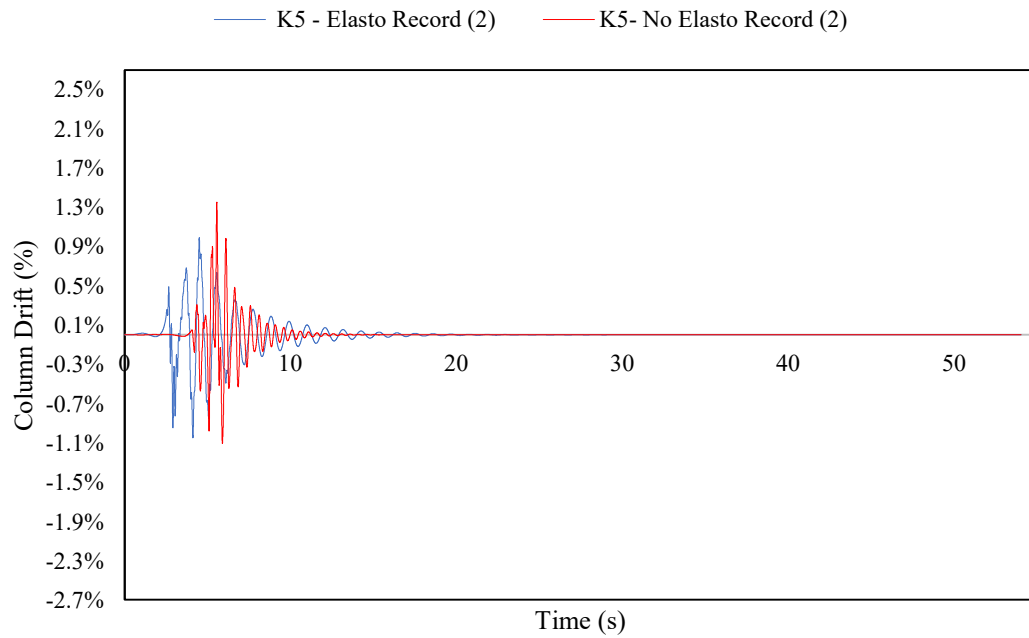
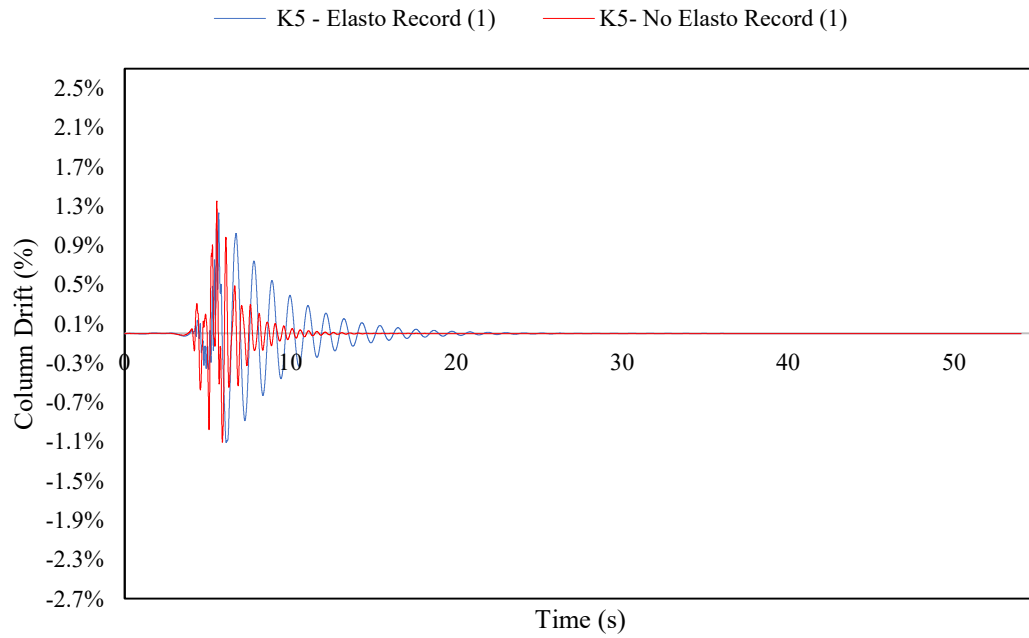


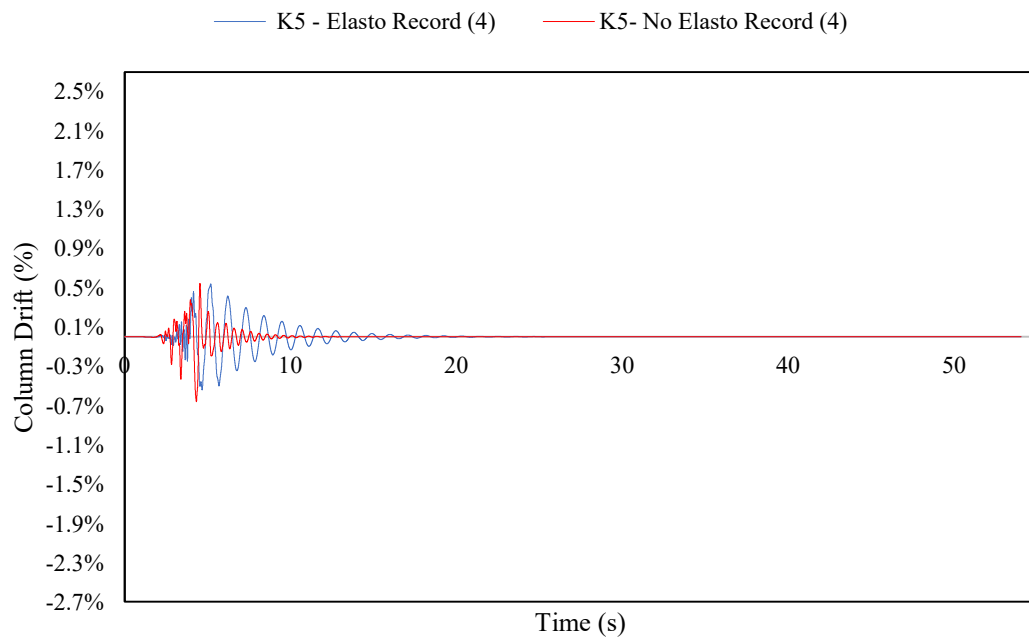
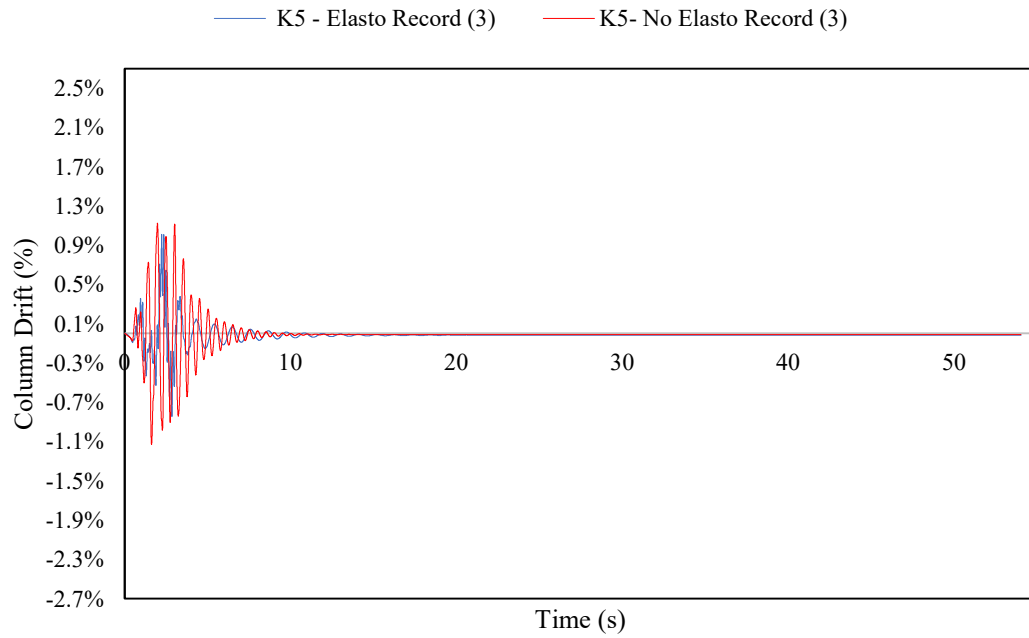


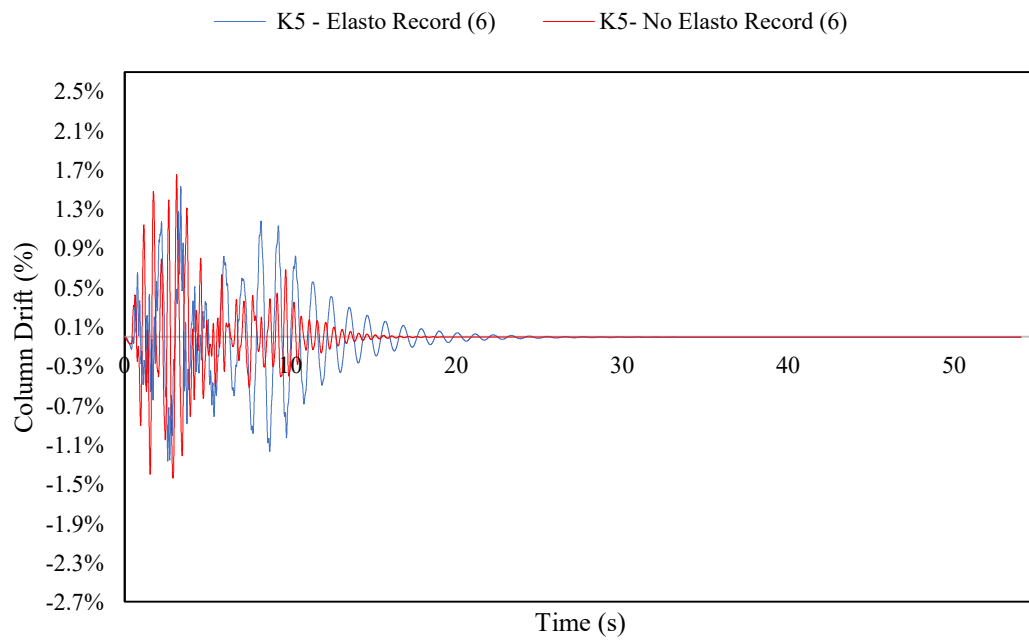
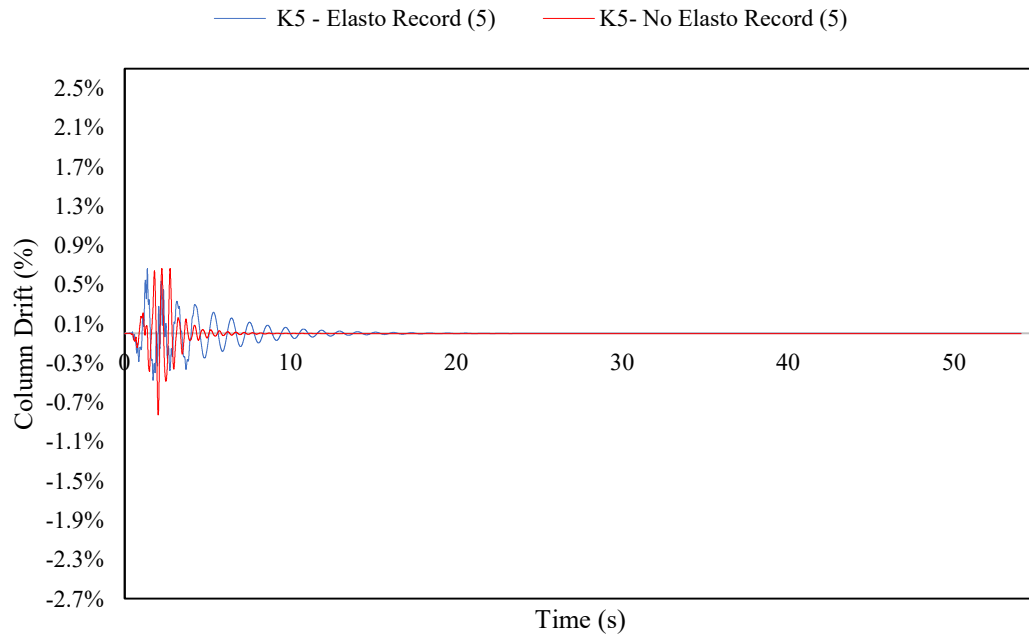


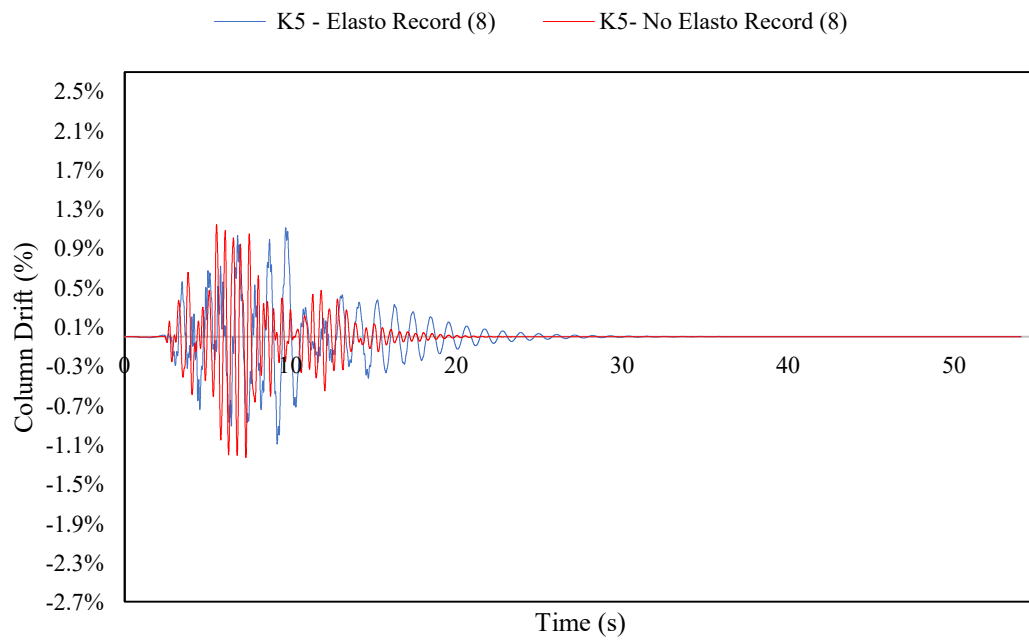
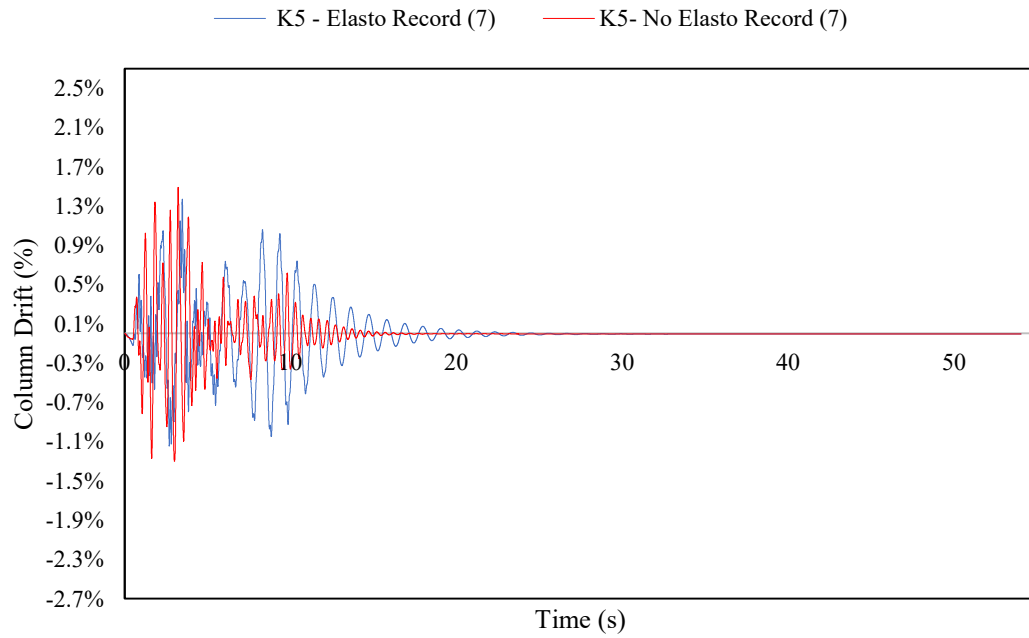


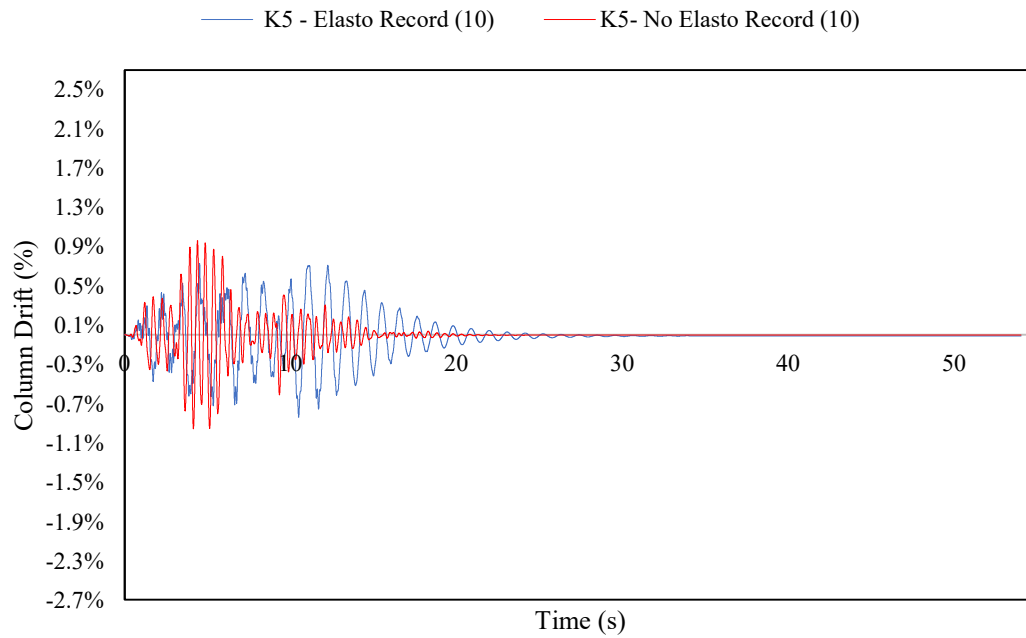
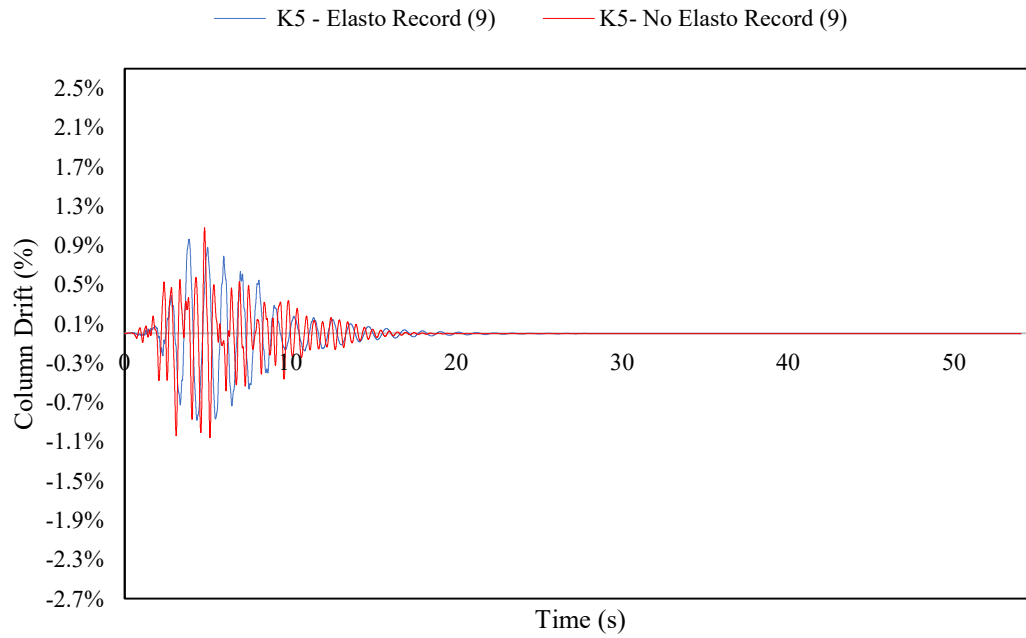


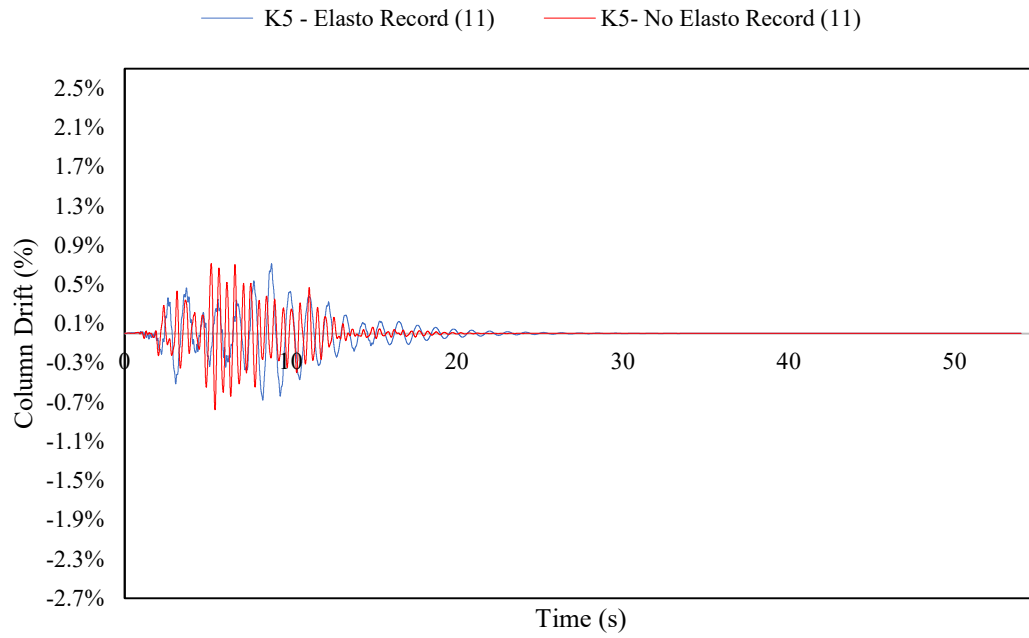




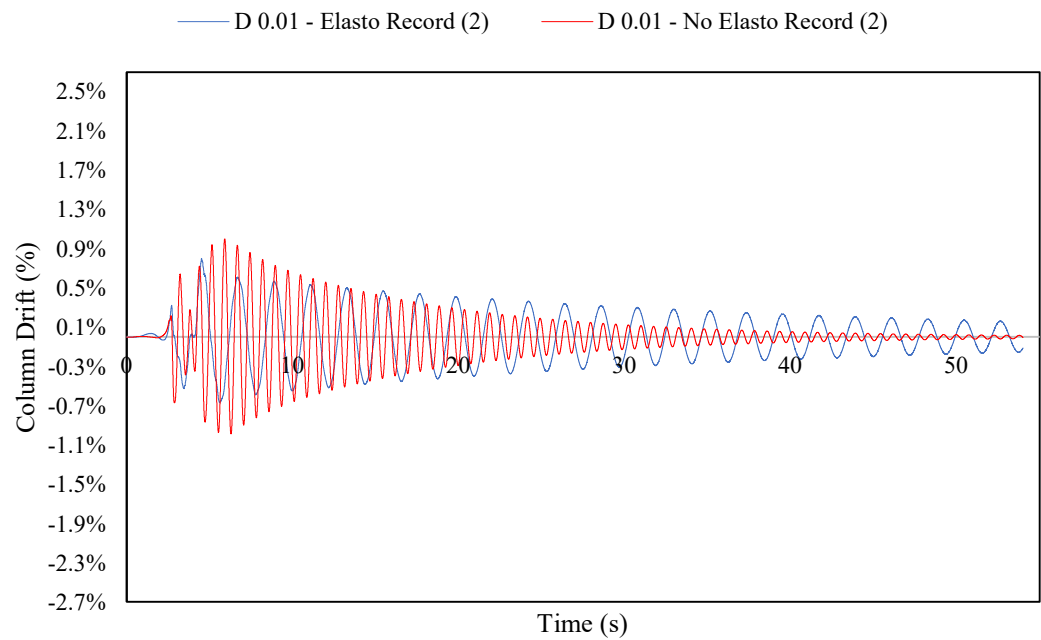
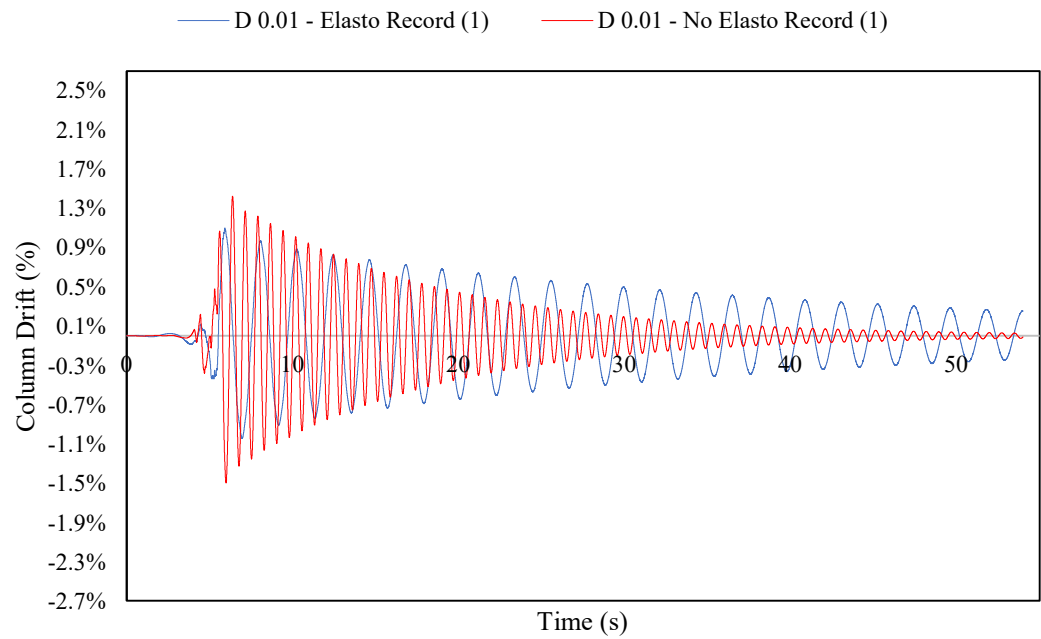


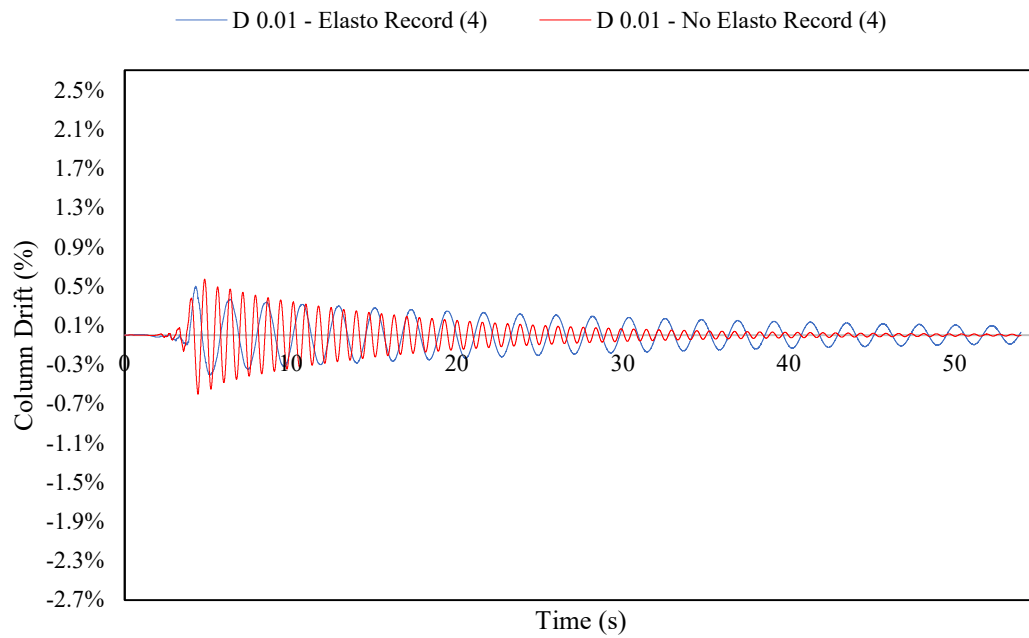
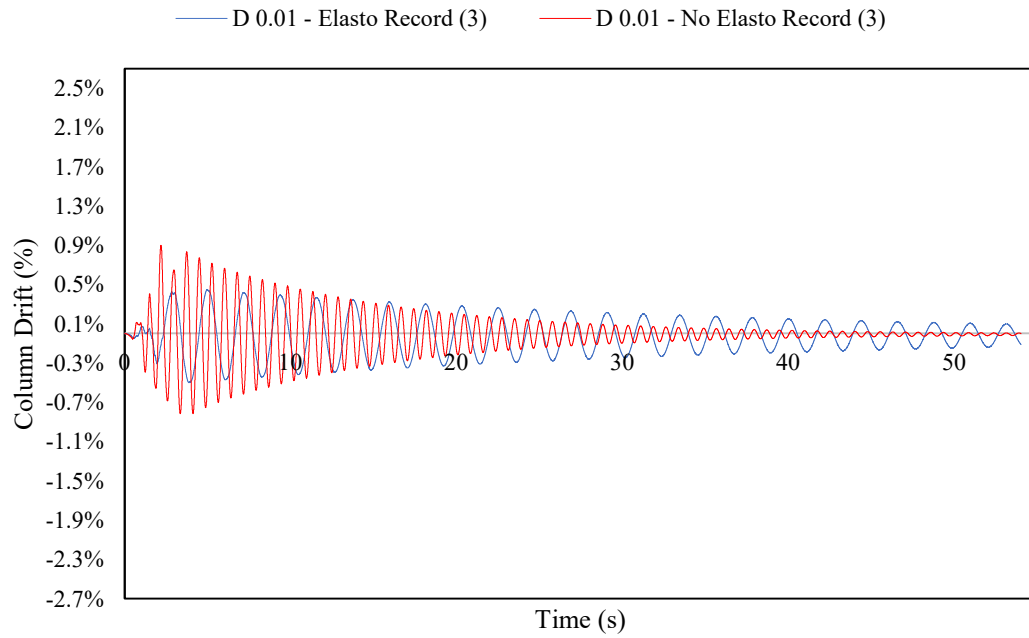


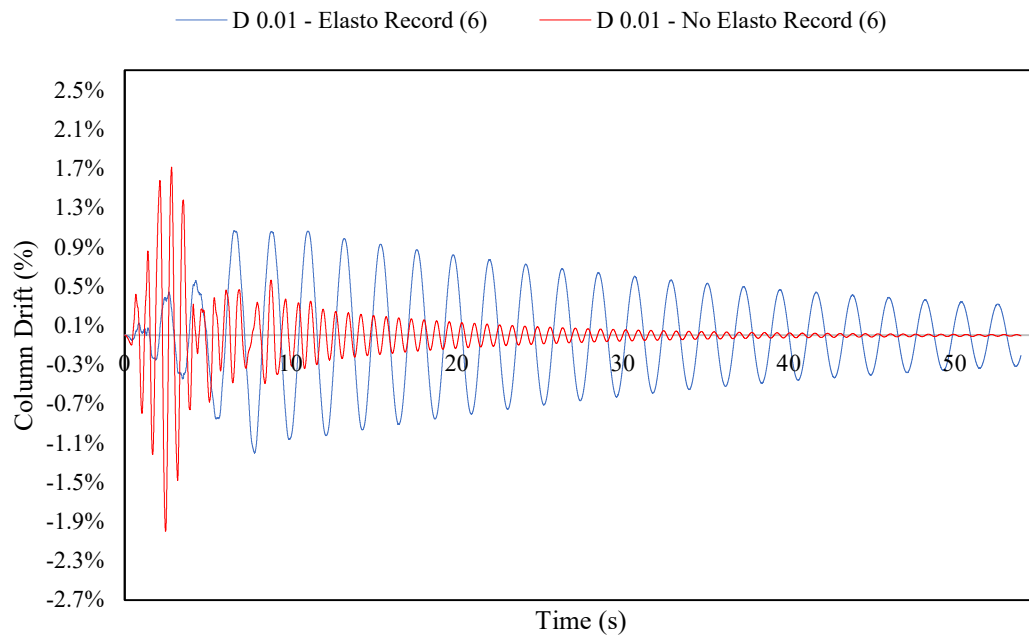
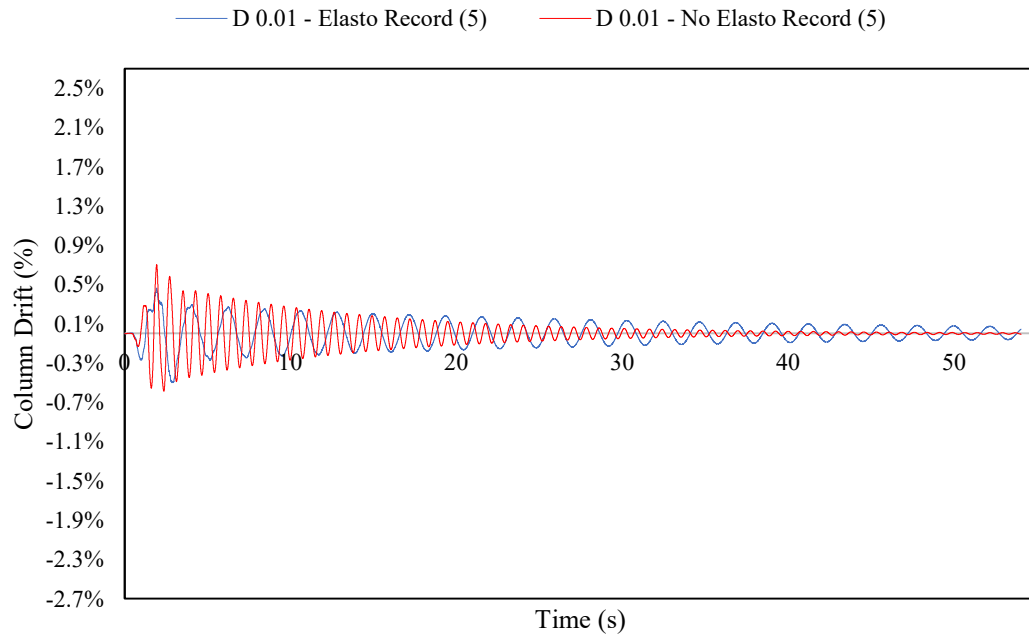


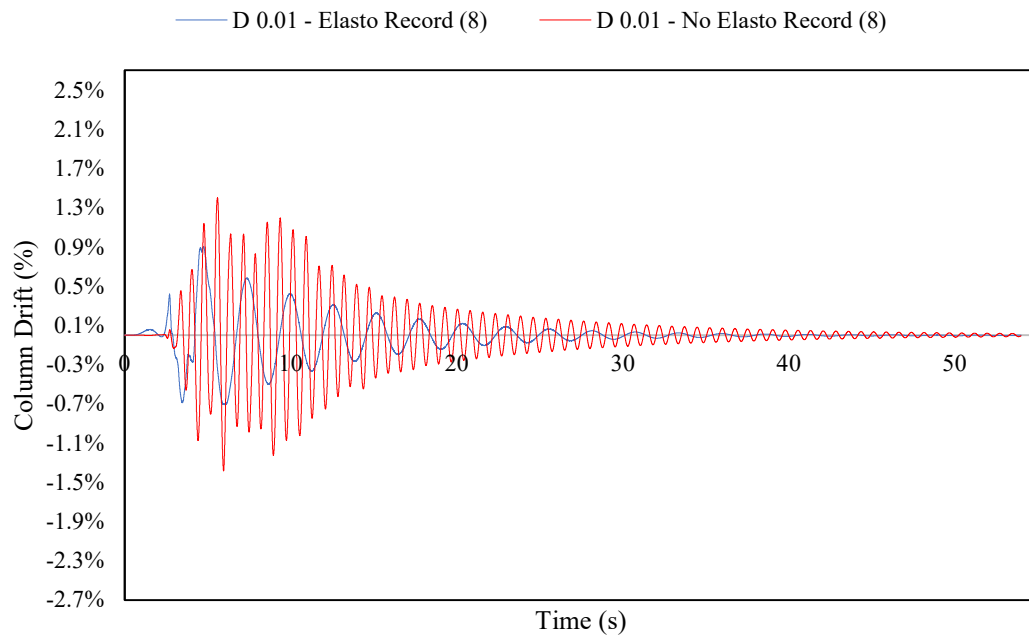
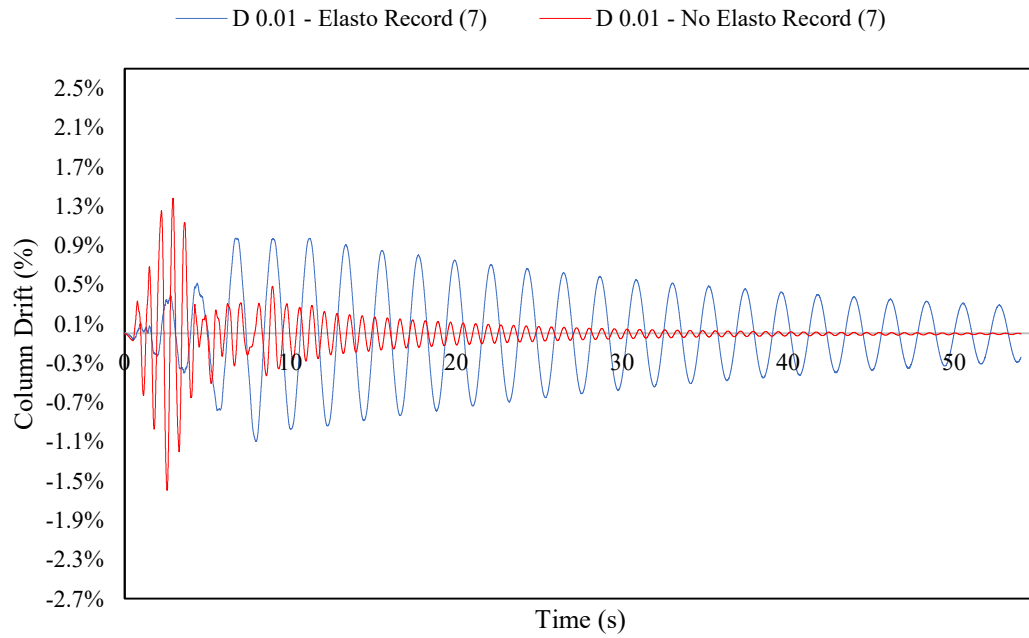


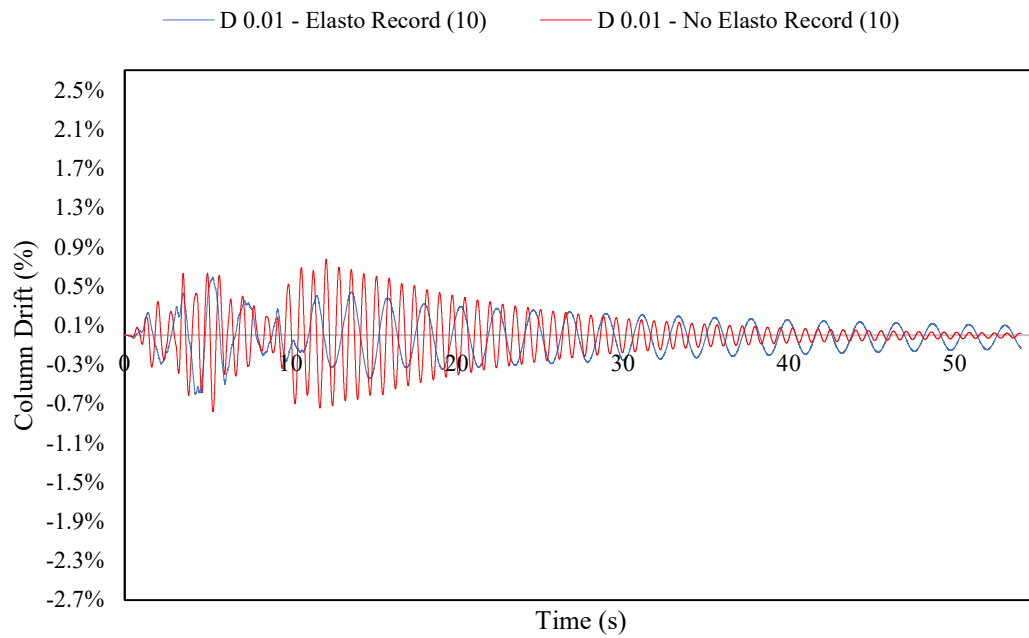
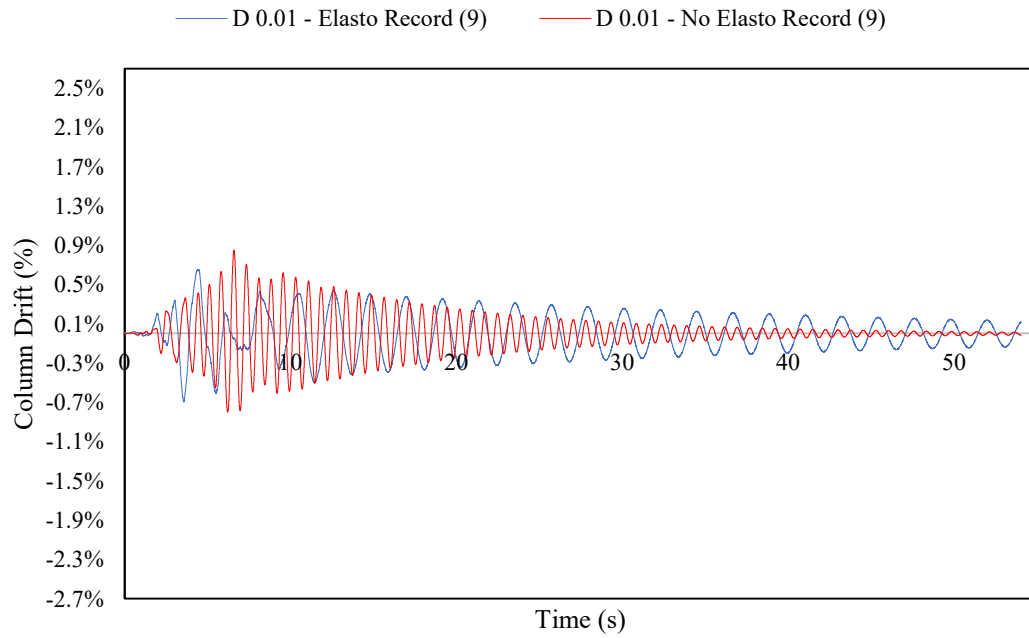
APPENDIX G: Drift history response of bridge pier varying damping

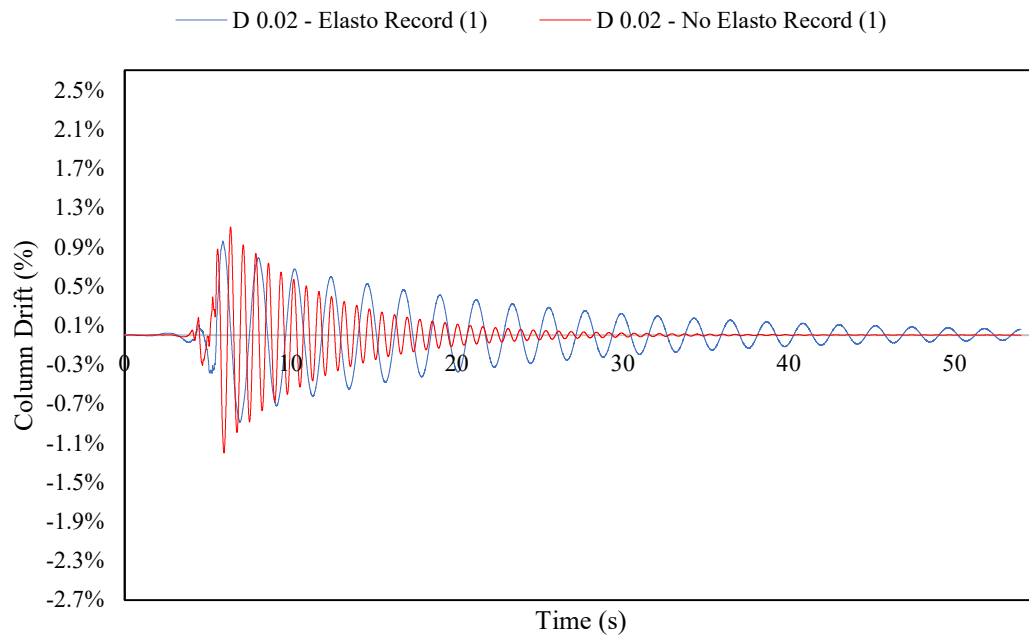
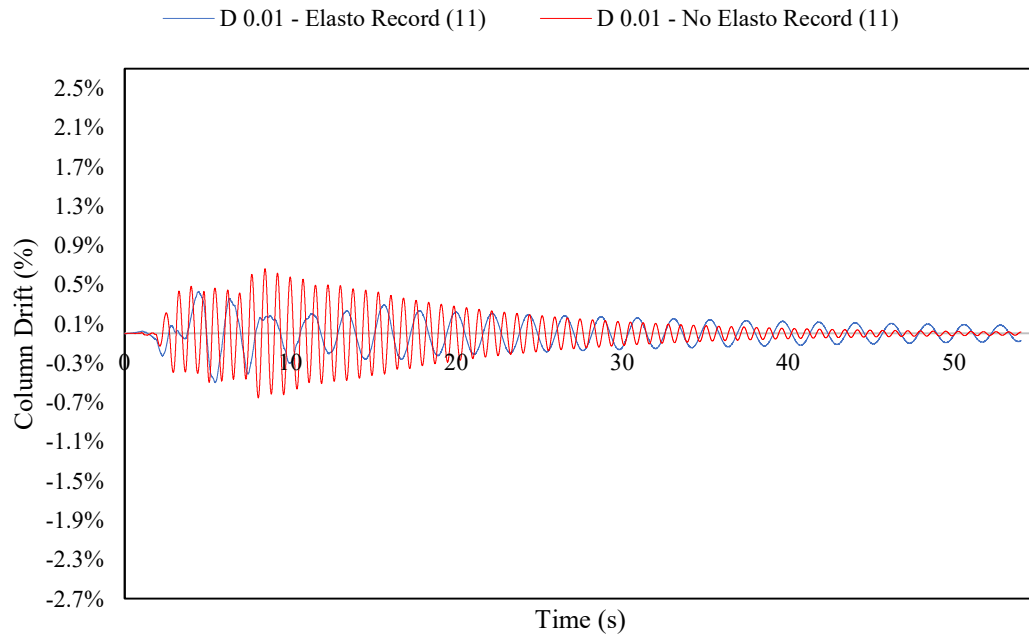


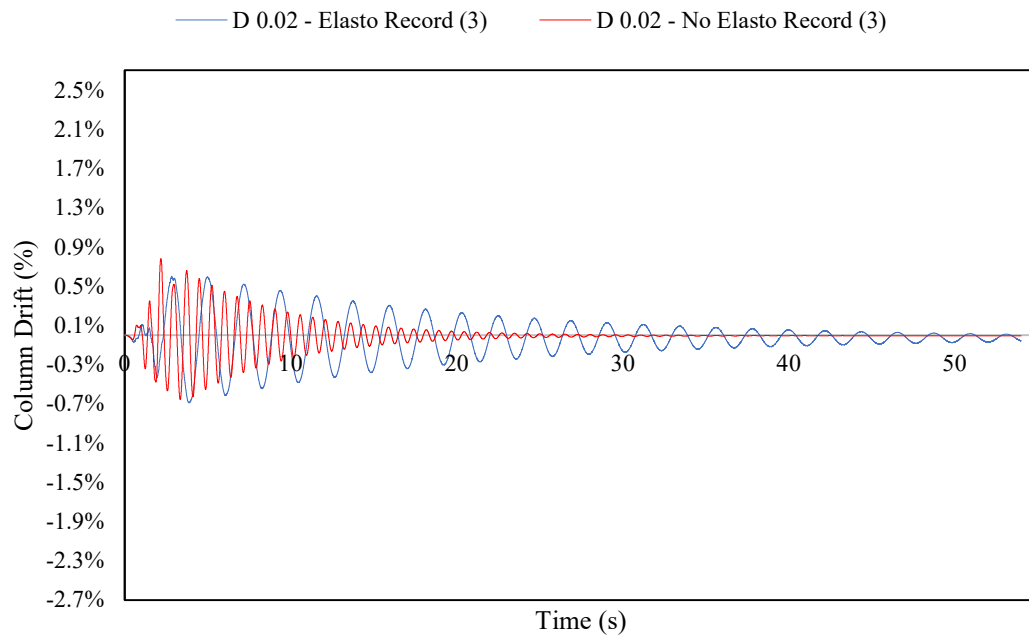
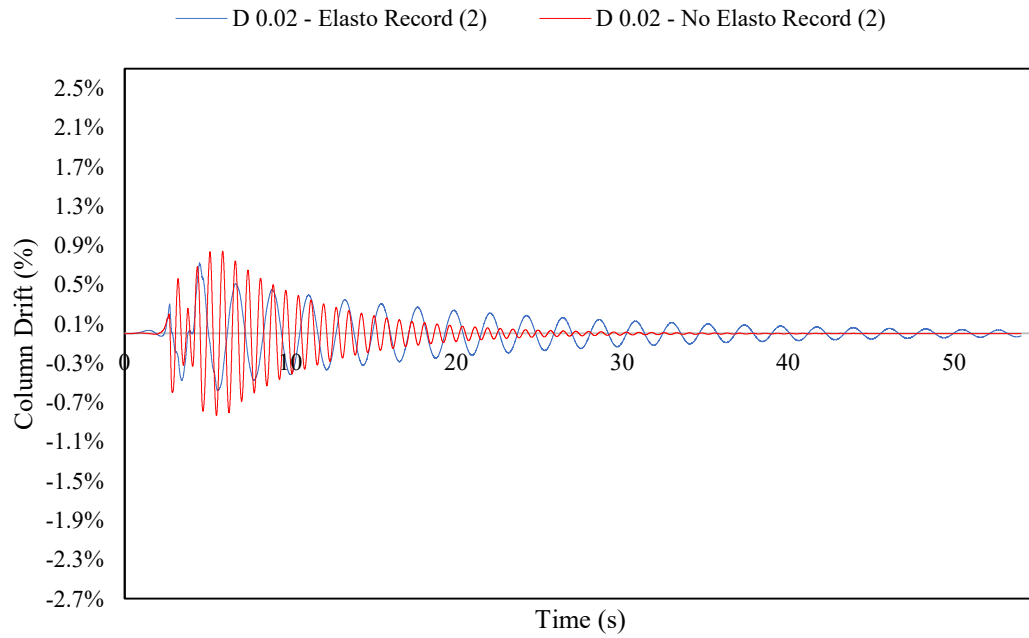


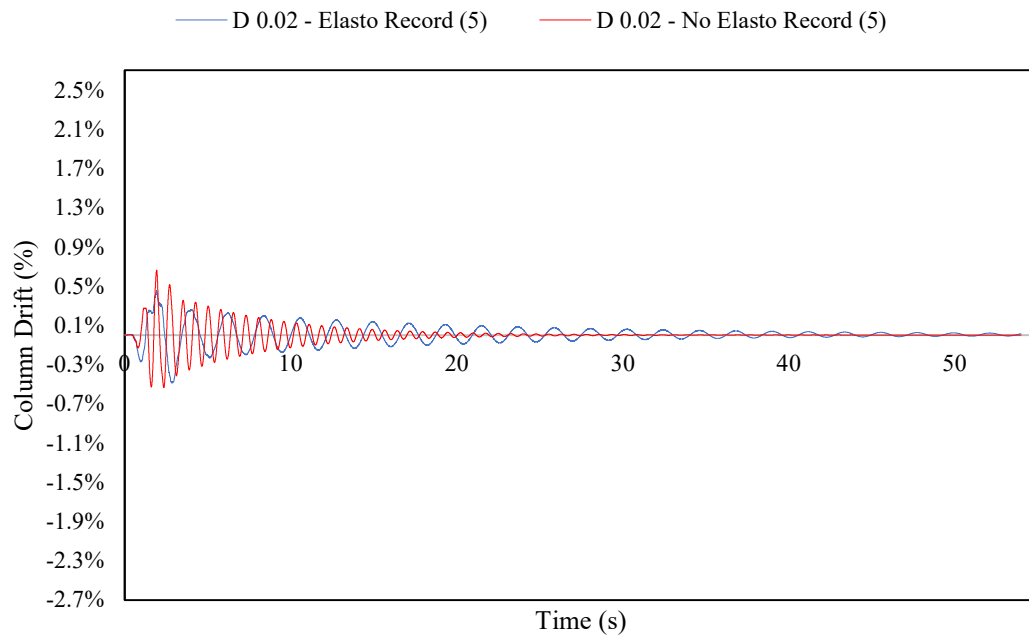
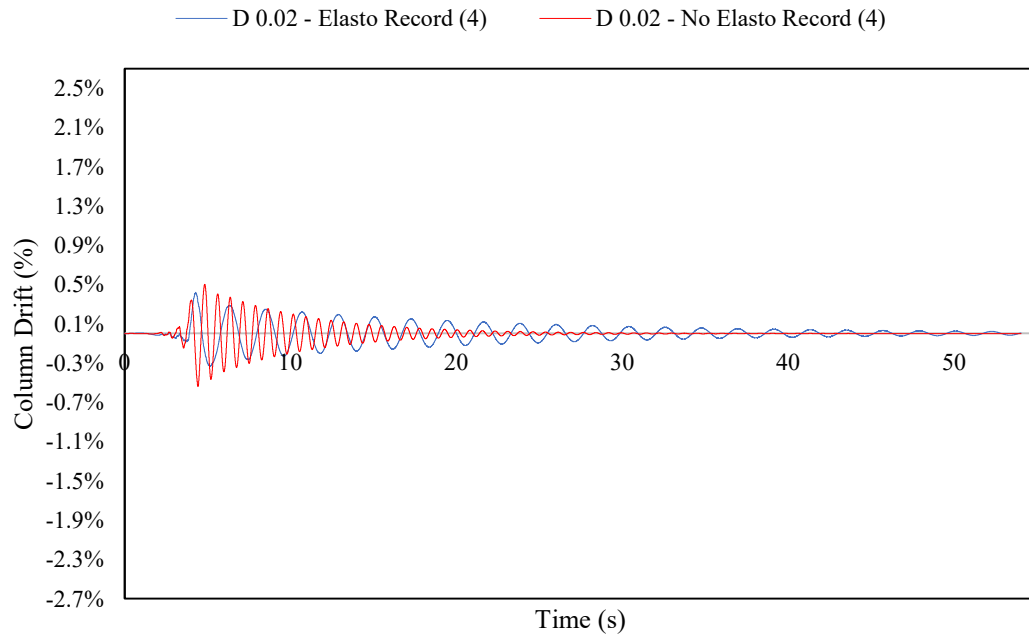


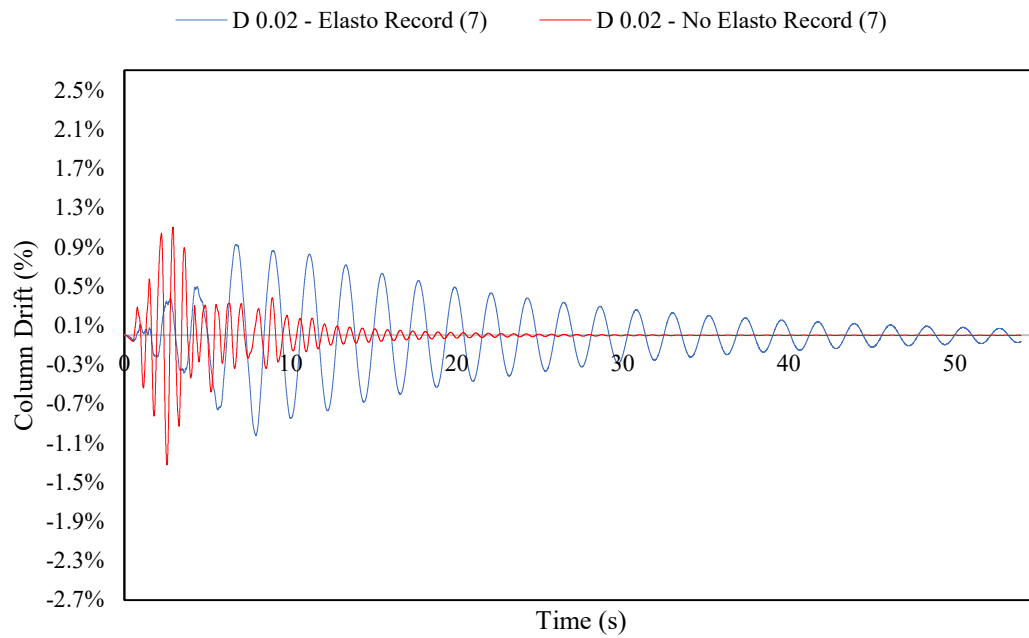
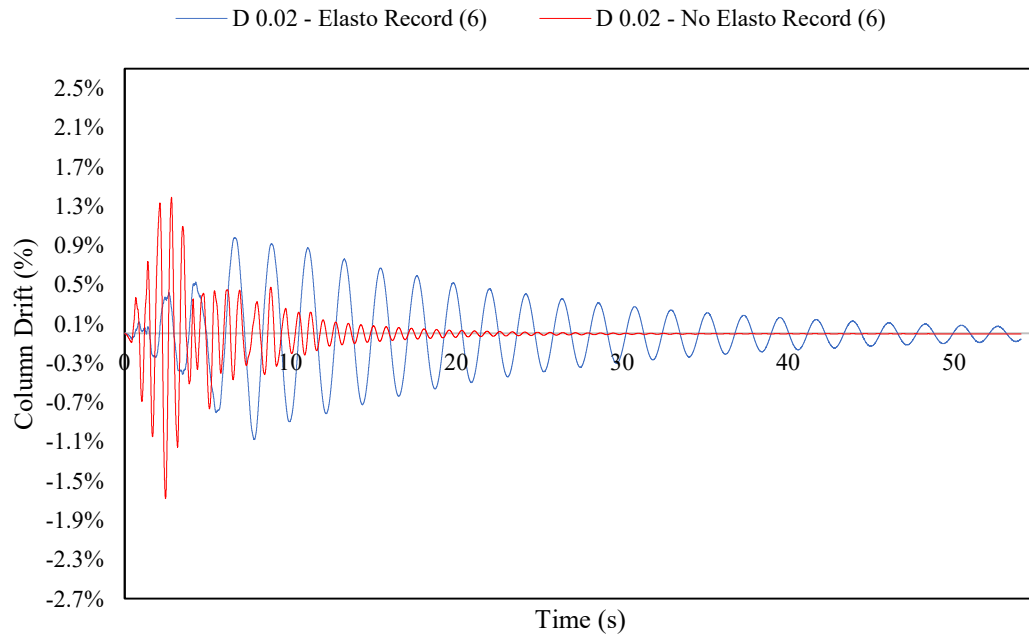


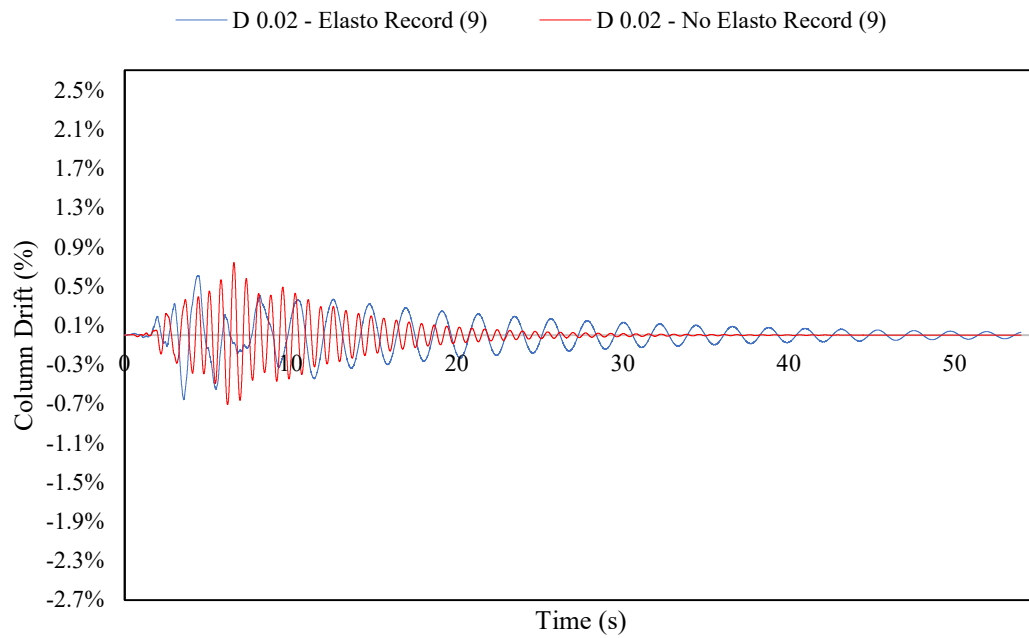
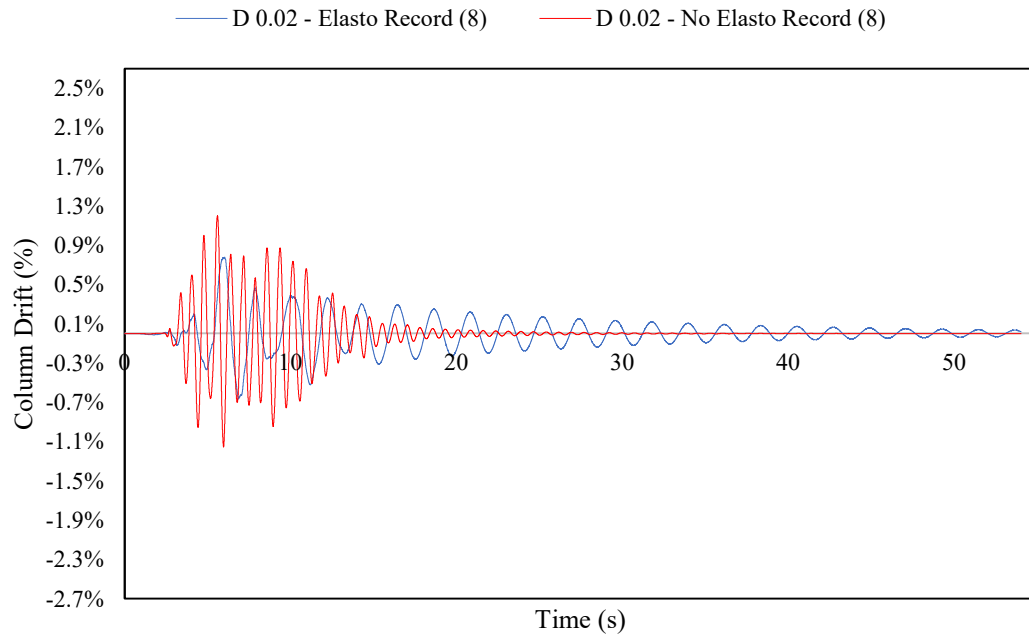


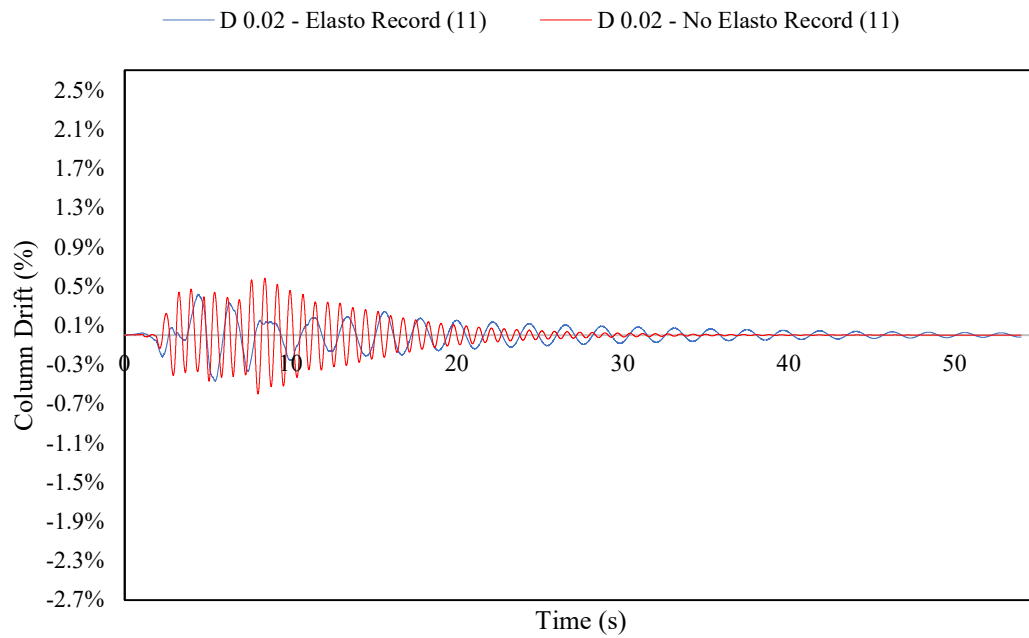
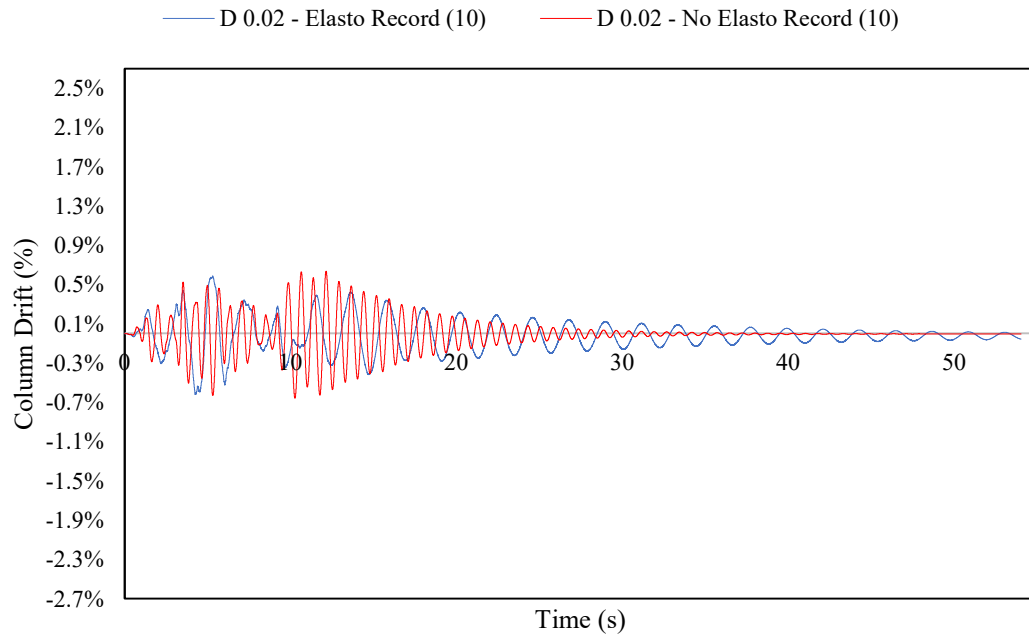


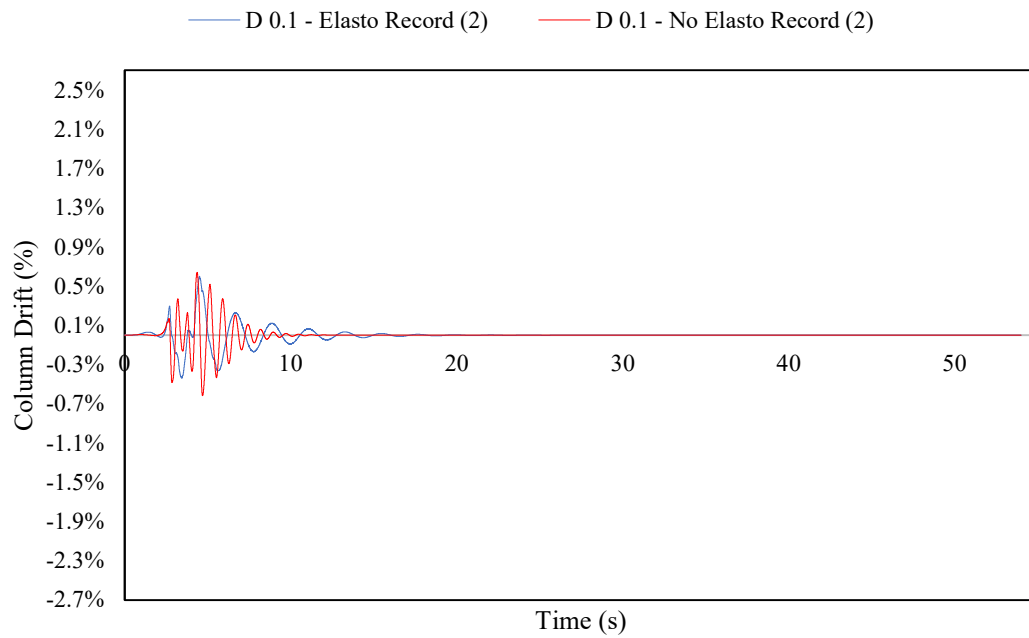
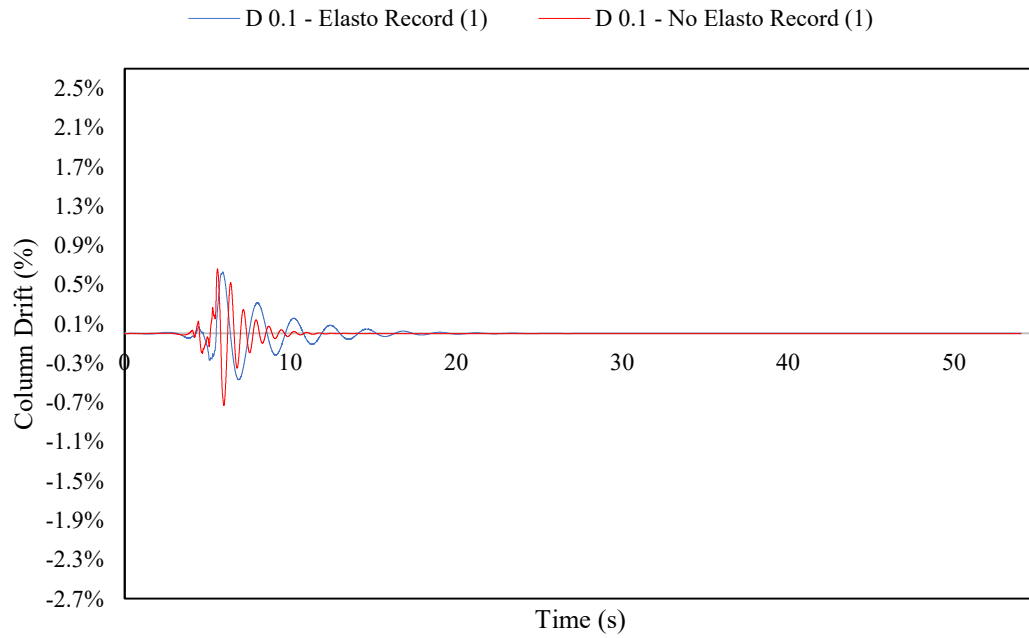


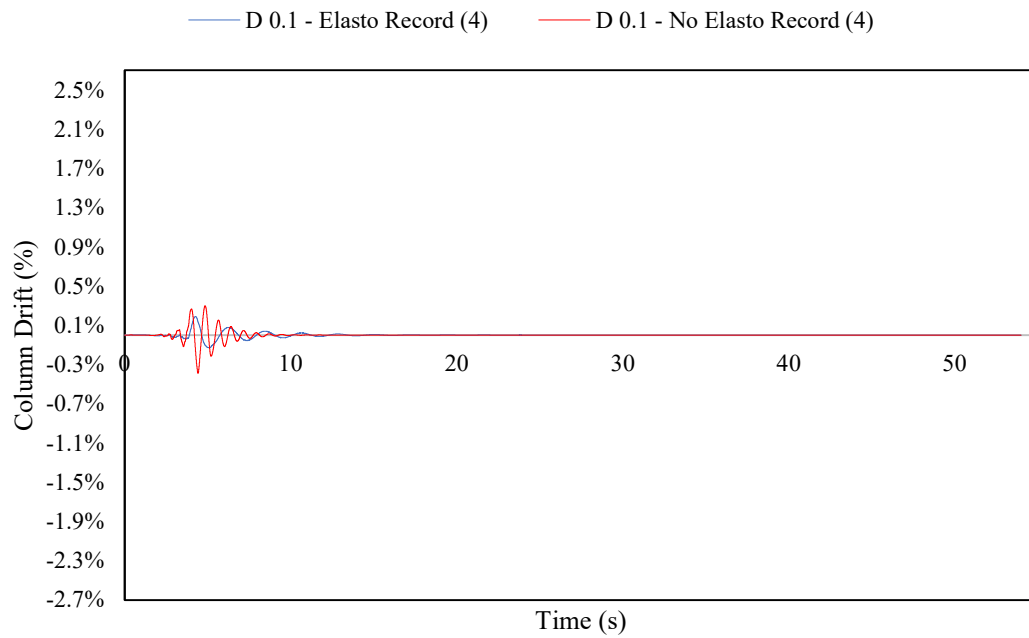
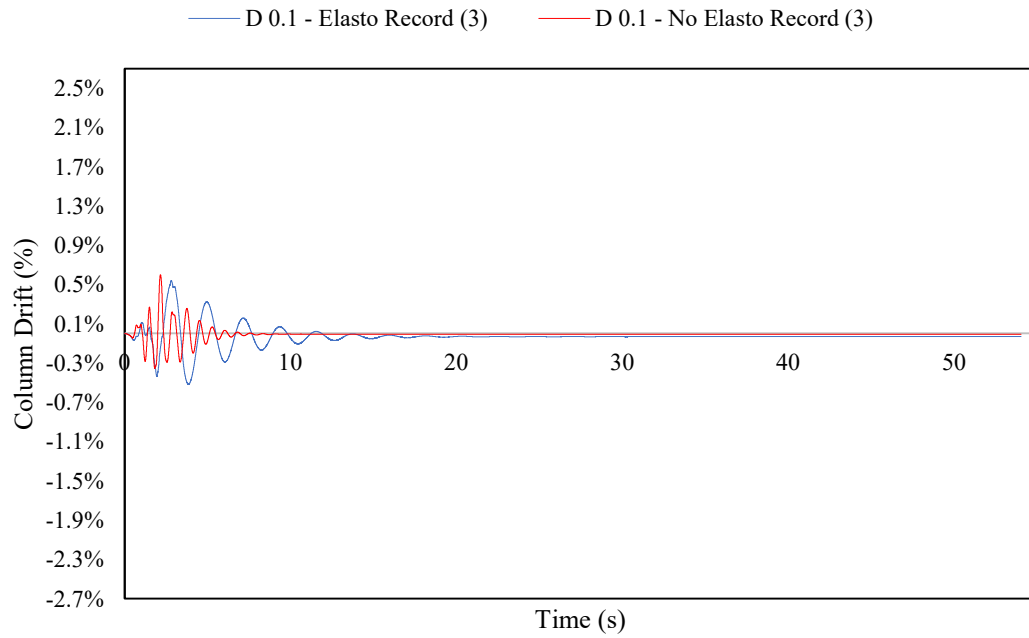


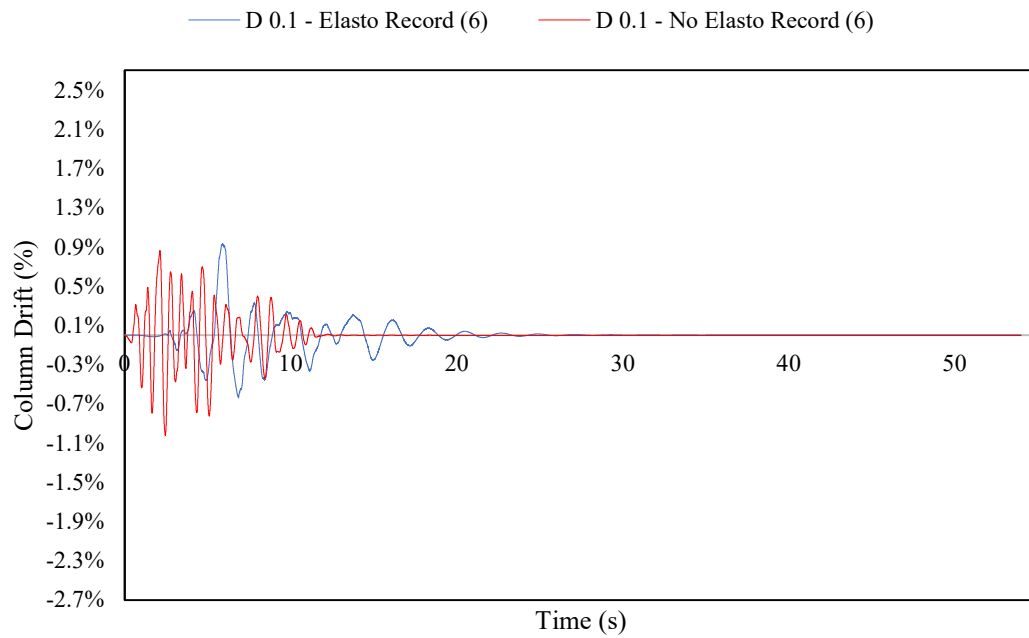
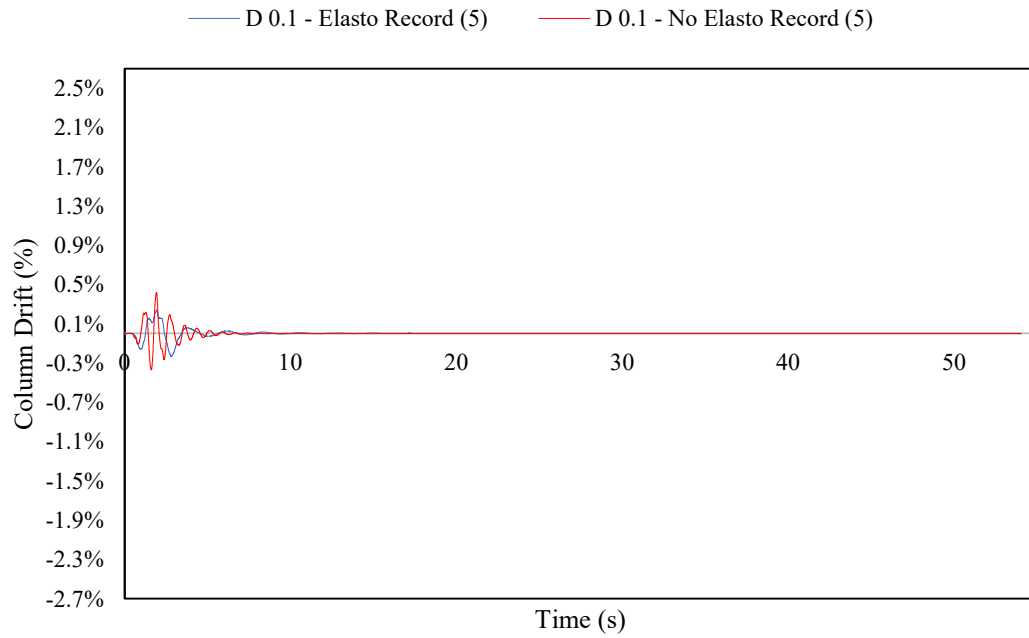


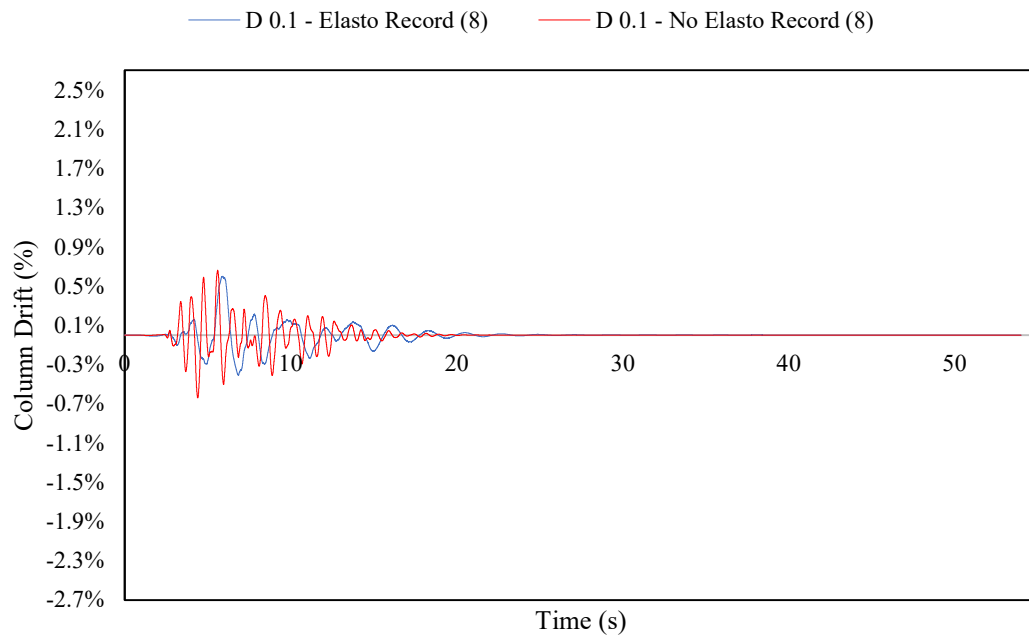
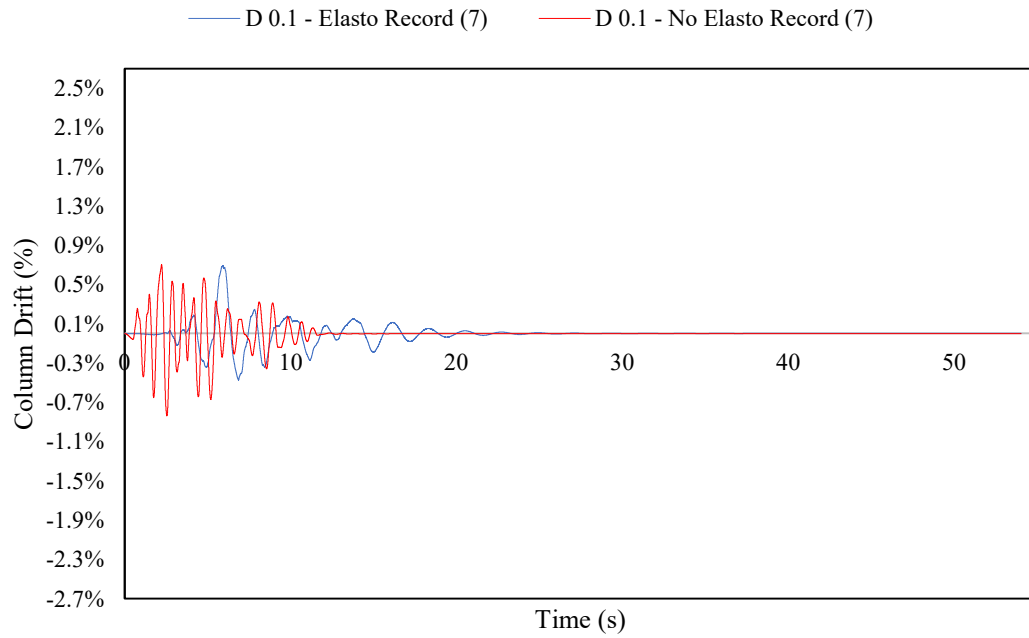


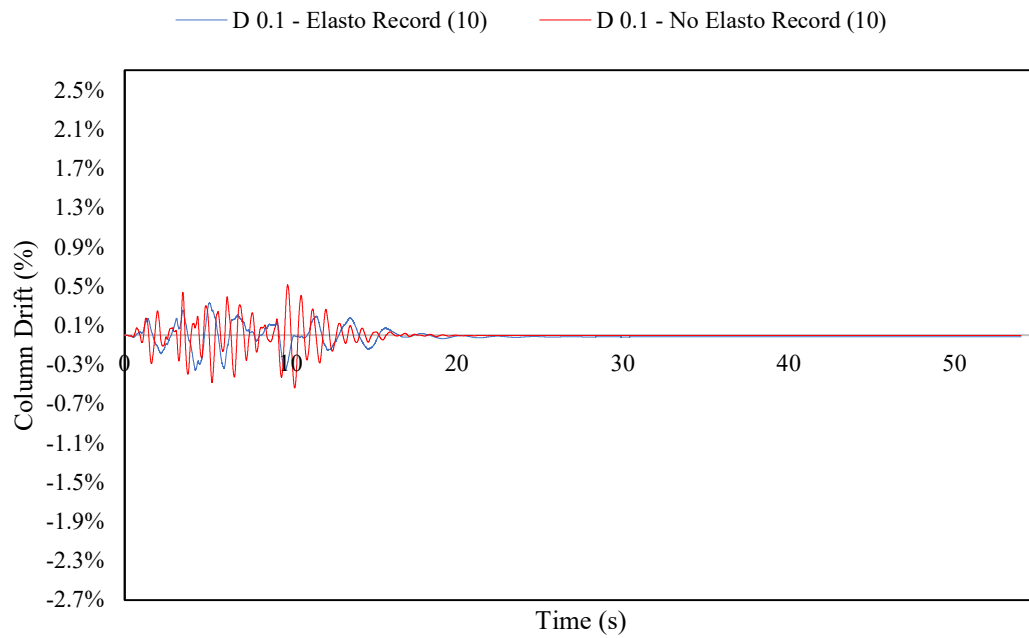
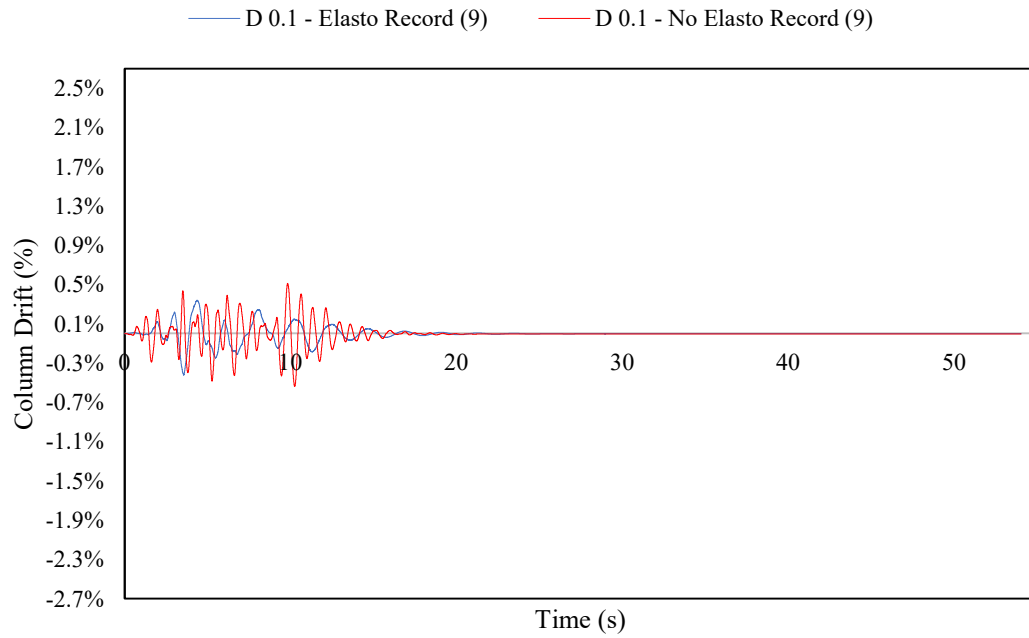


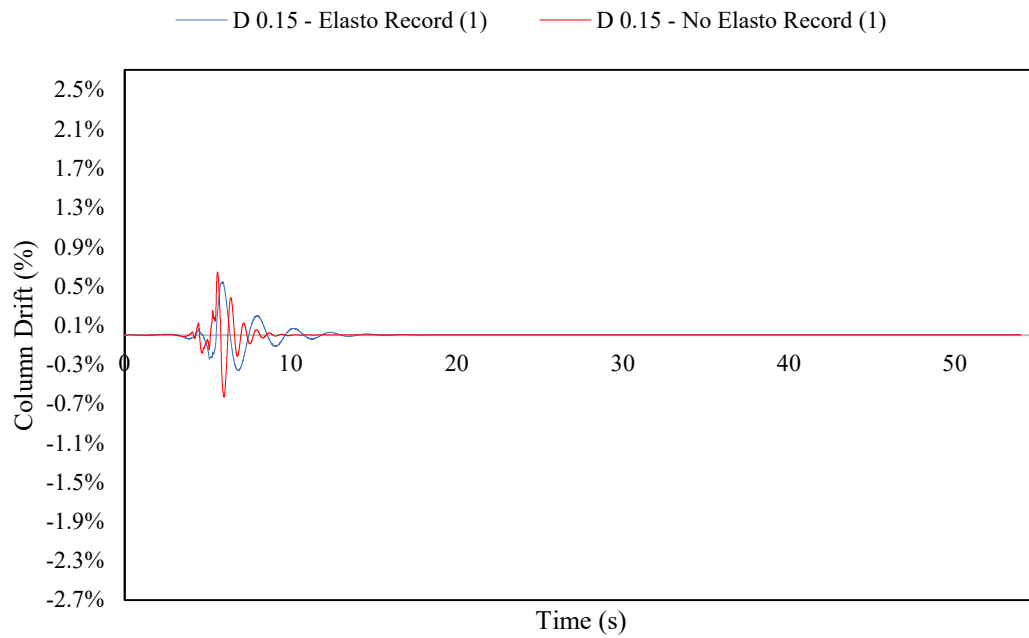
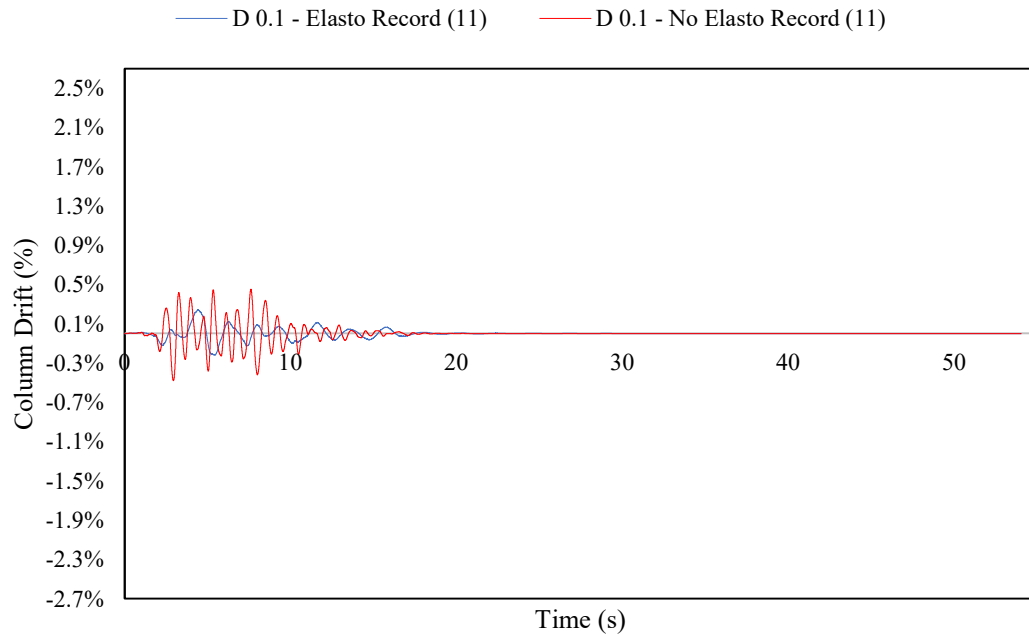


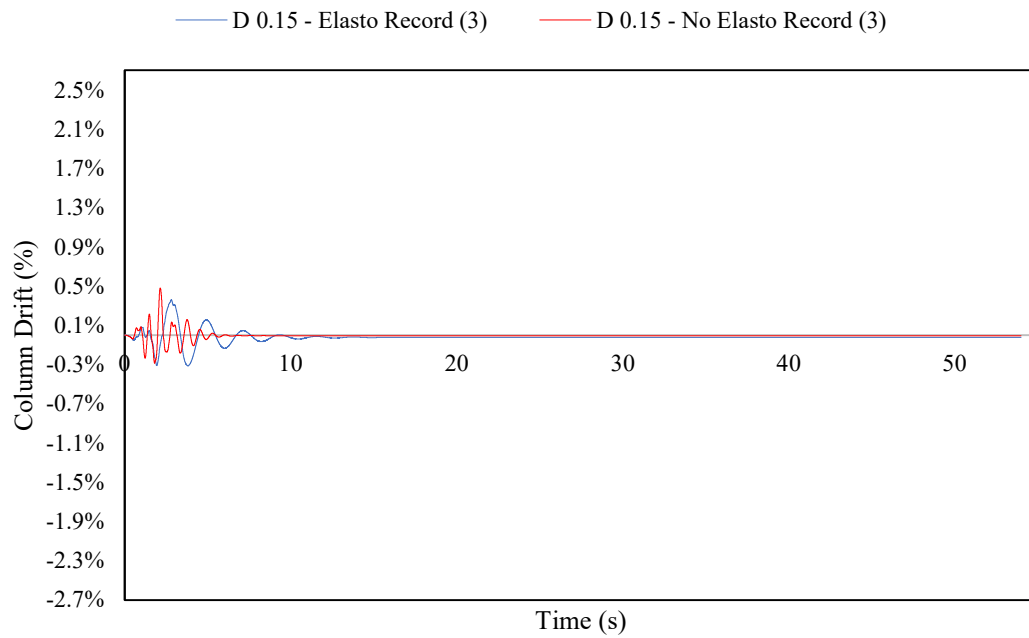
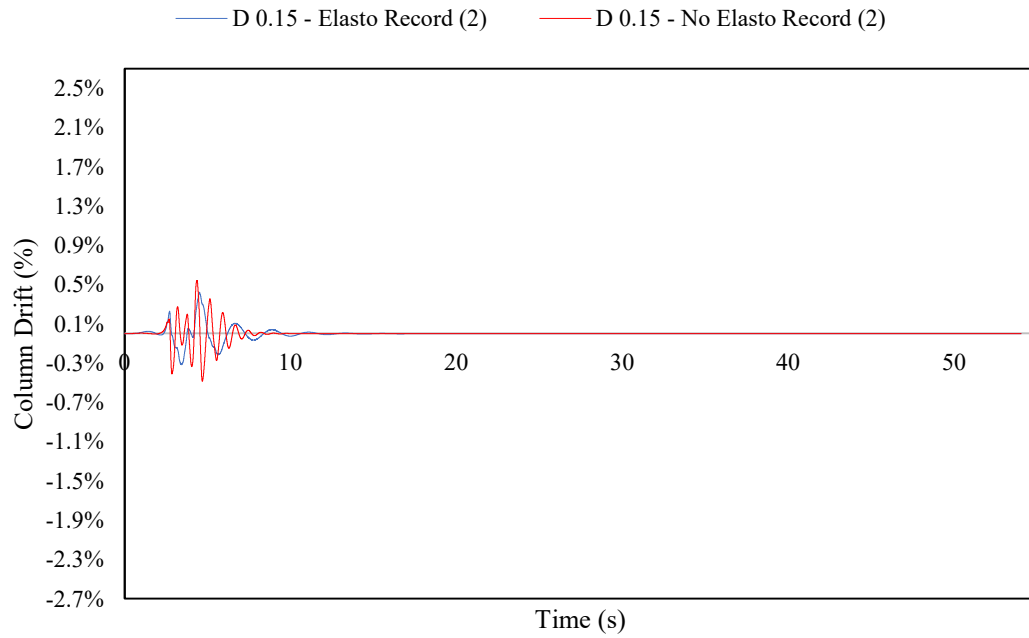


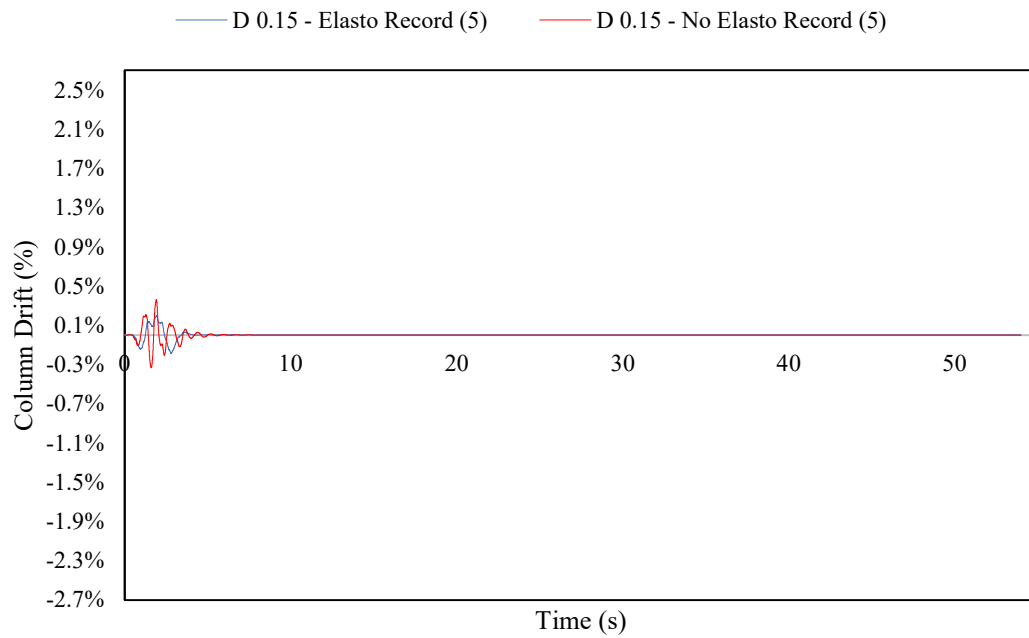
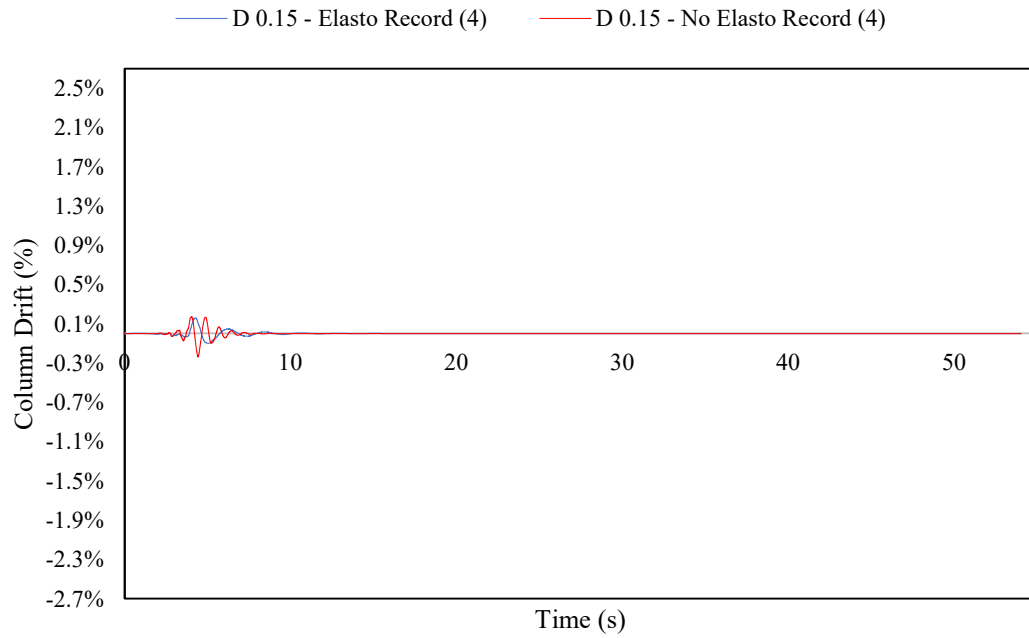


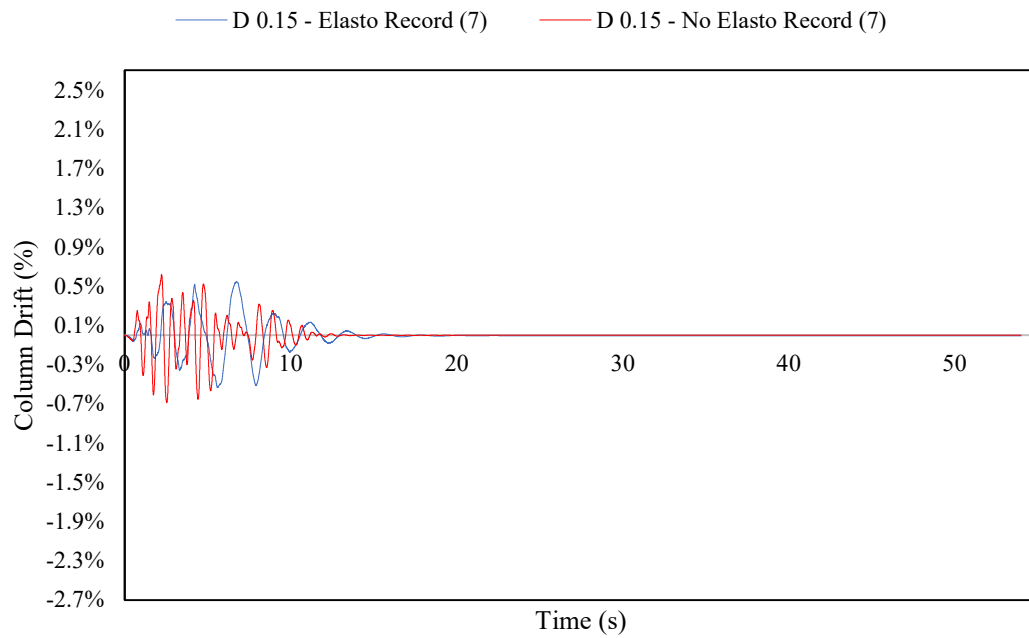
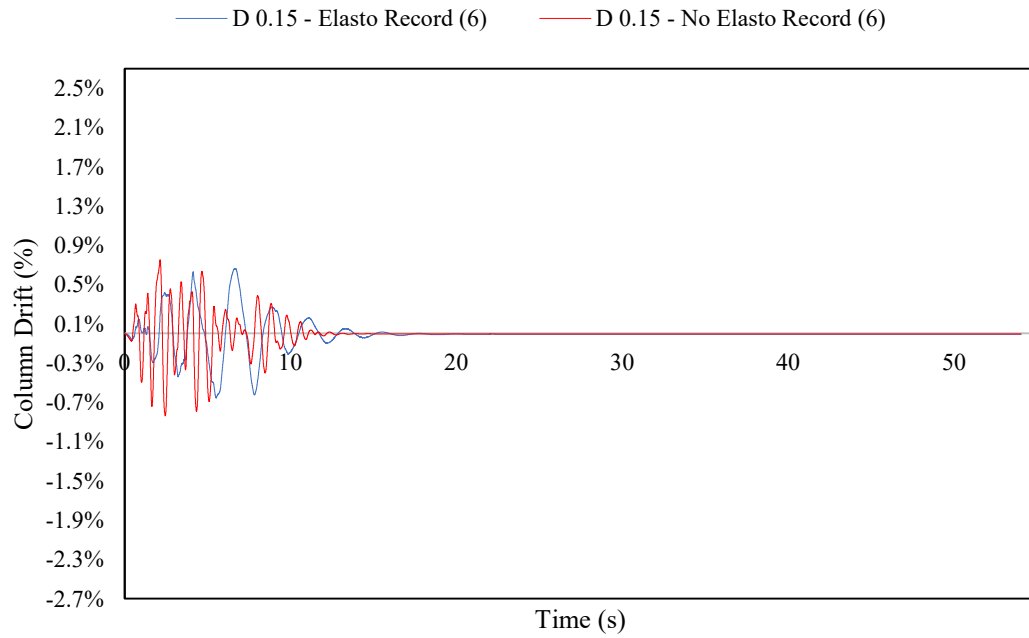


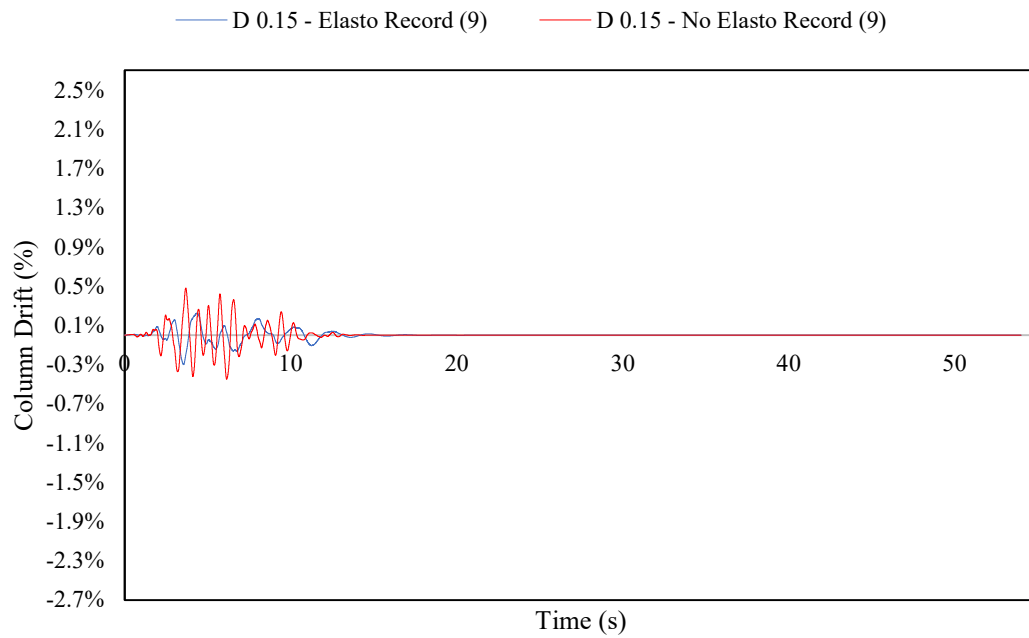
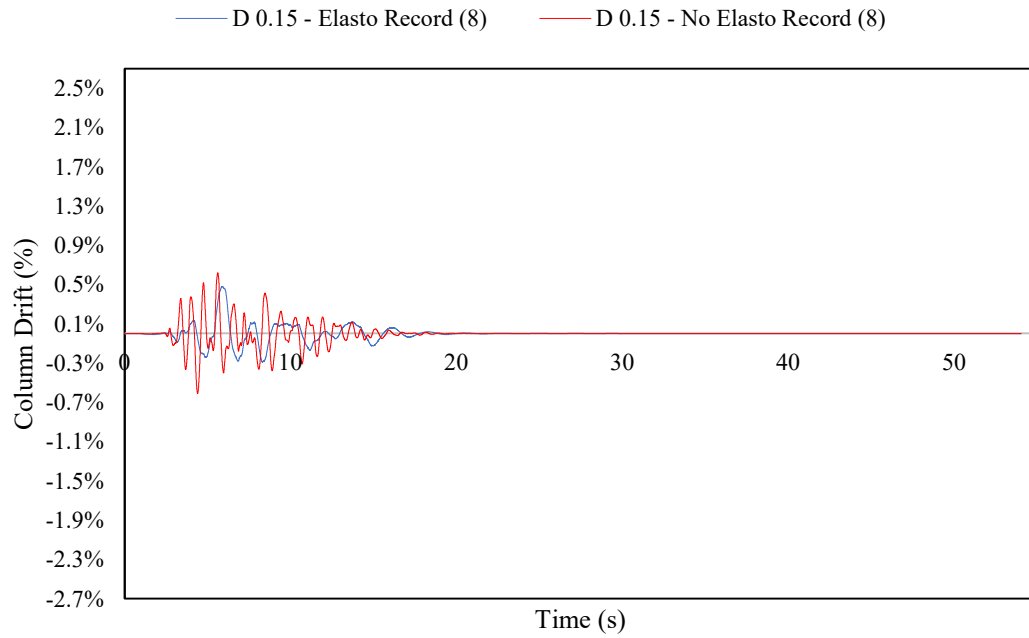


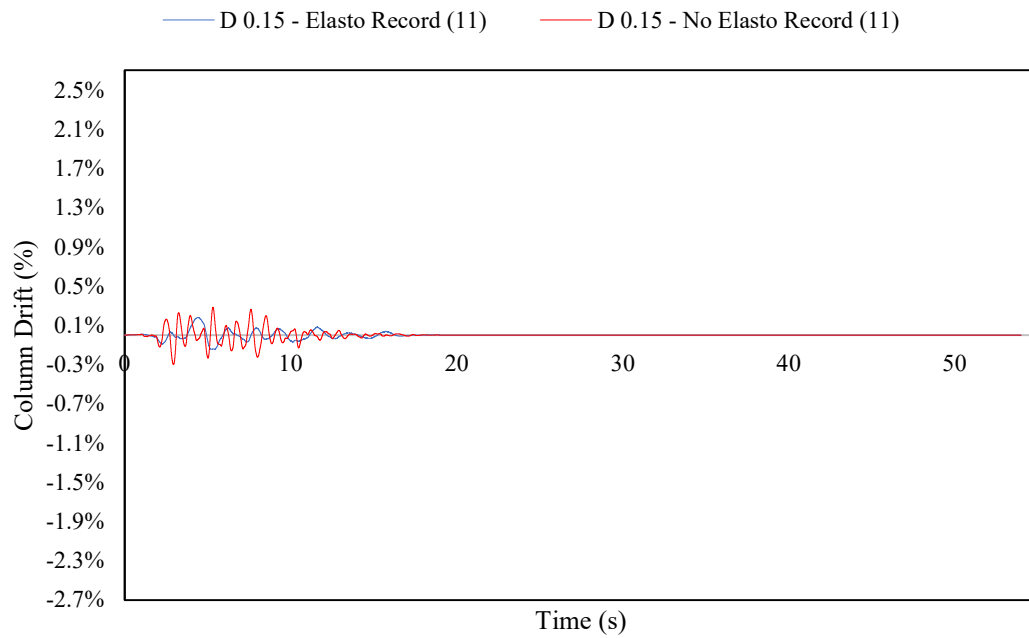
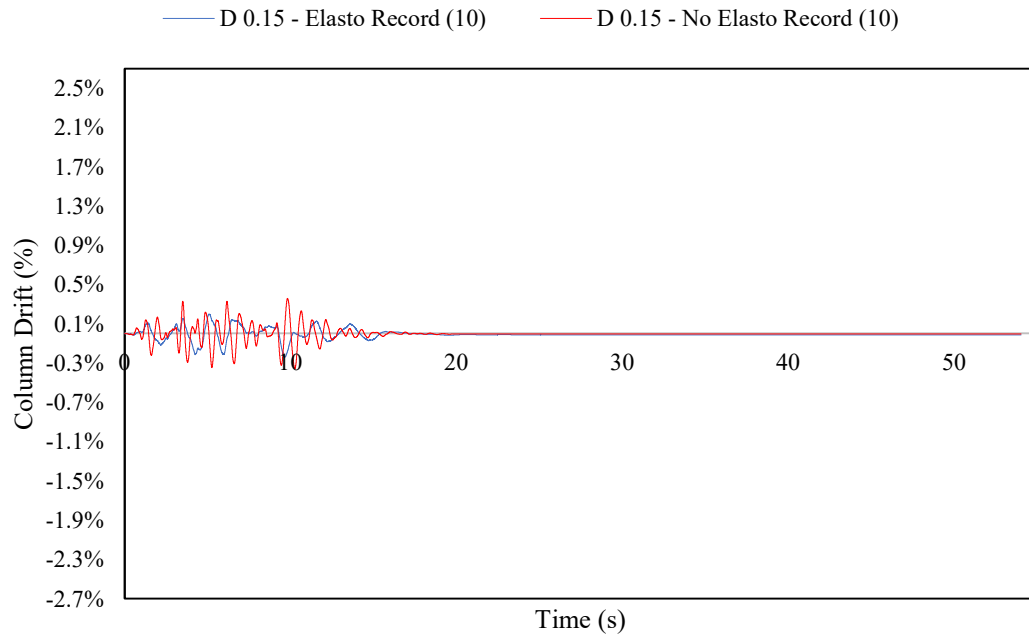




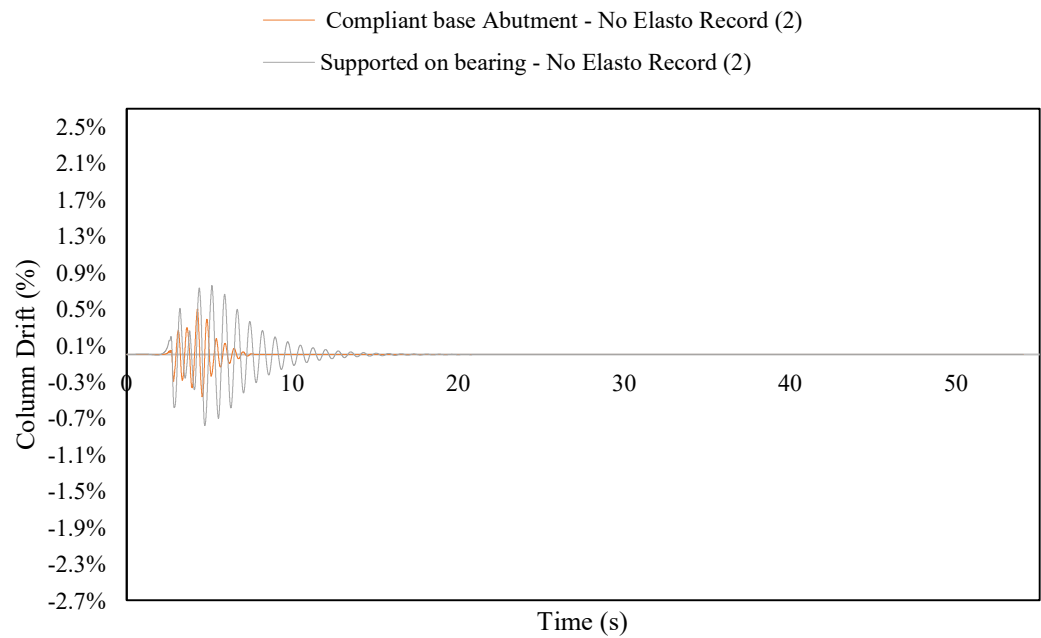
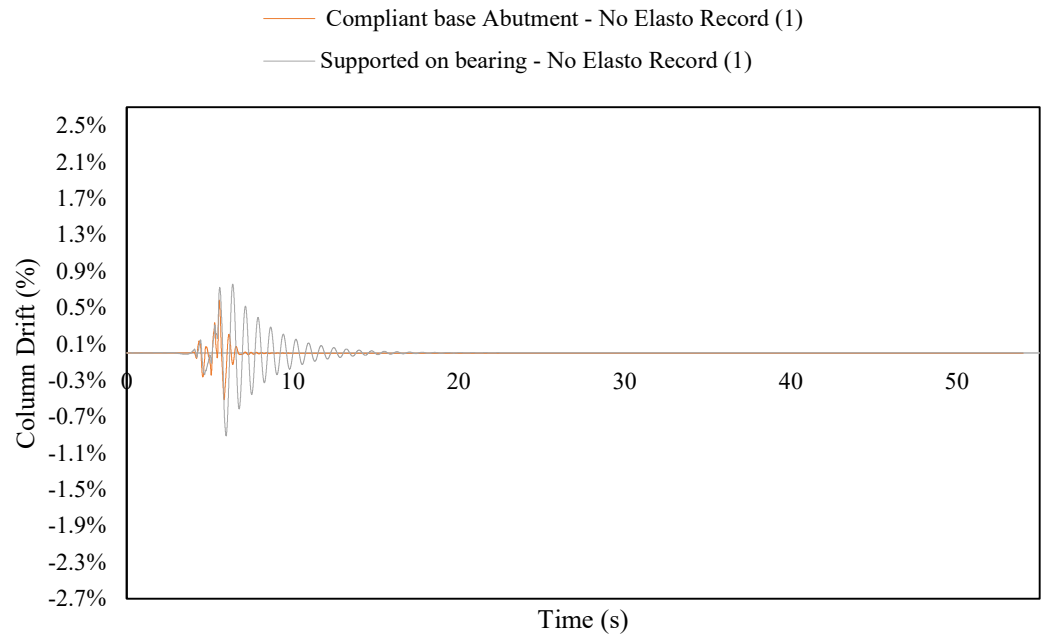


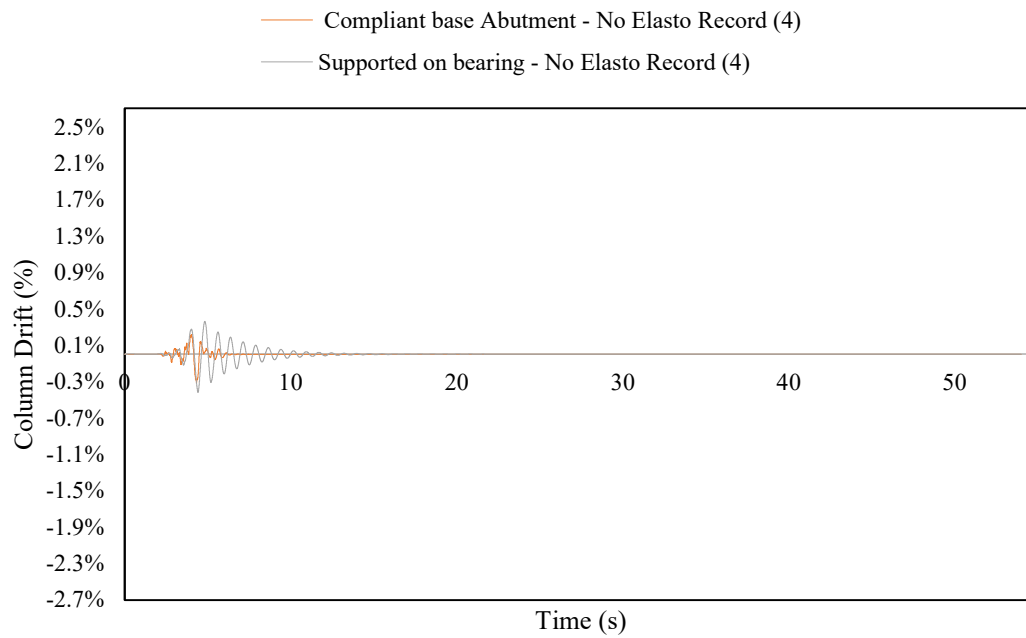
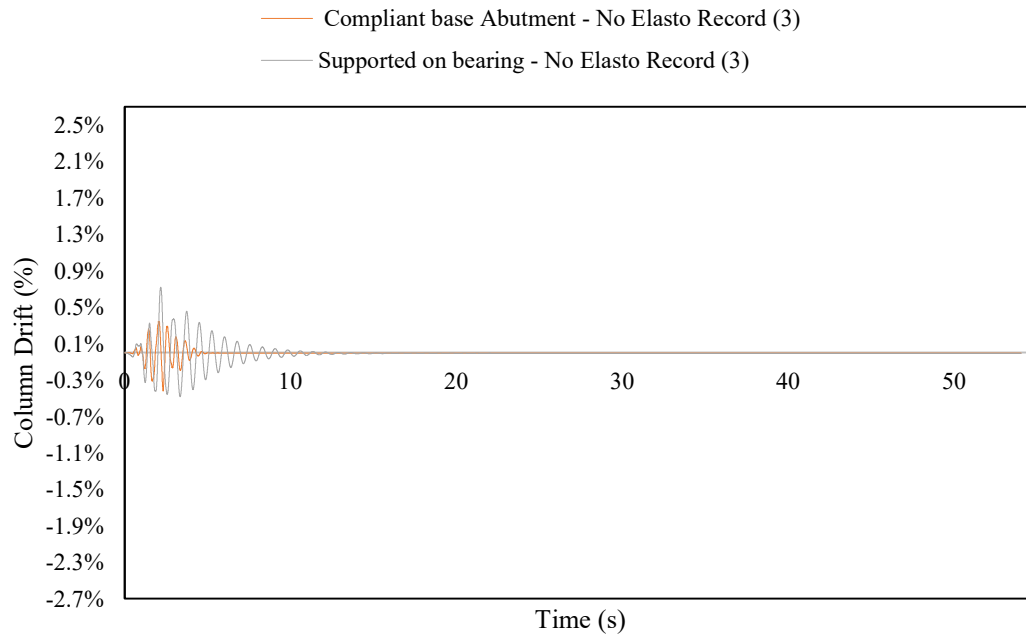


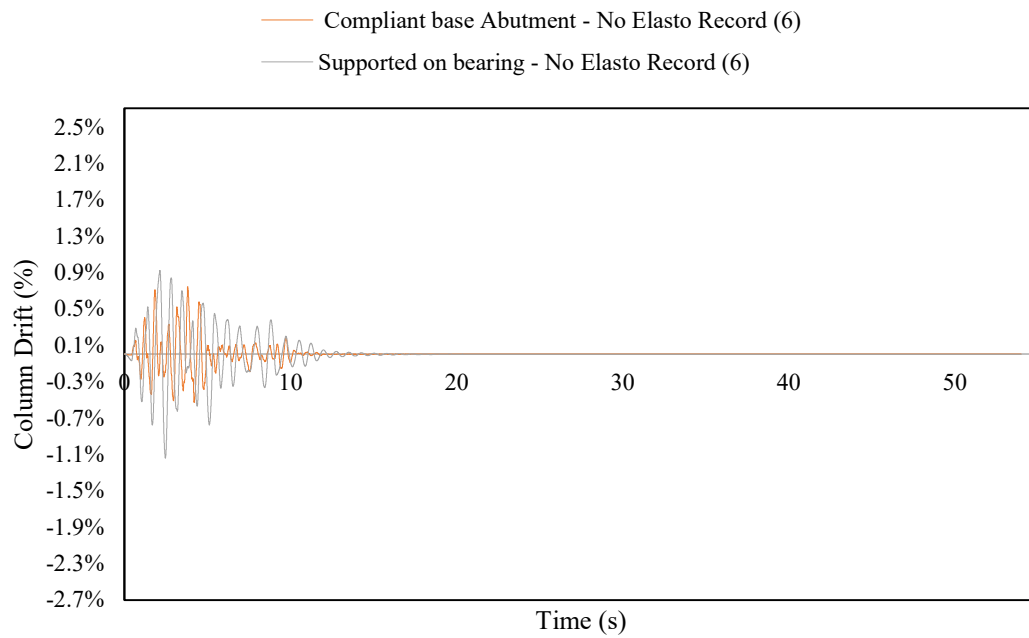
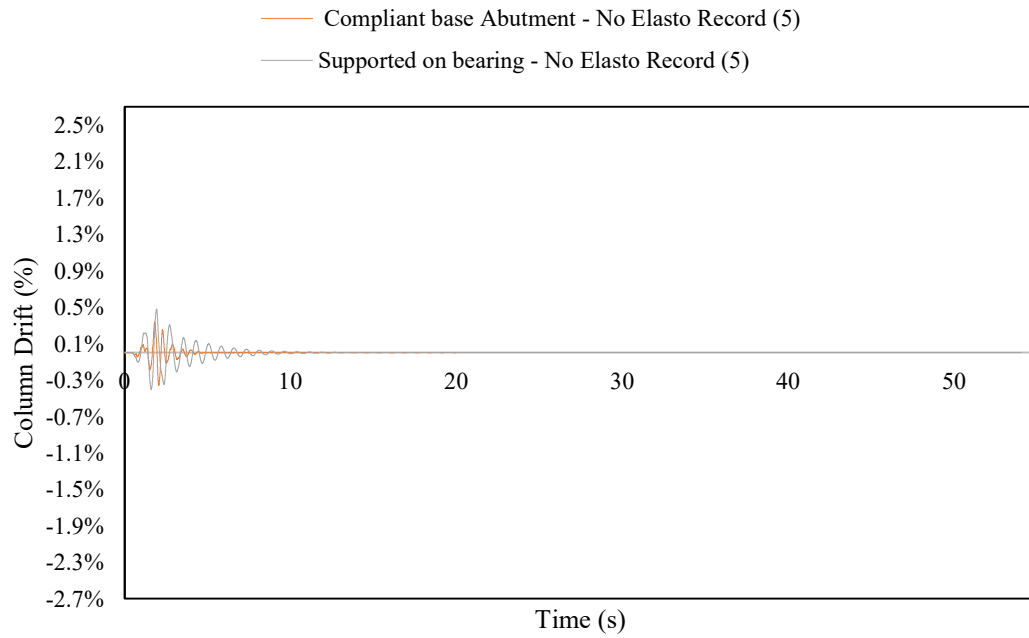


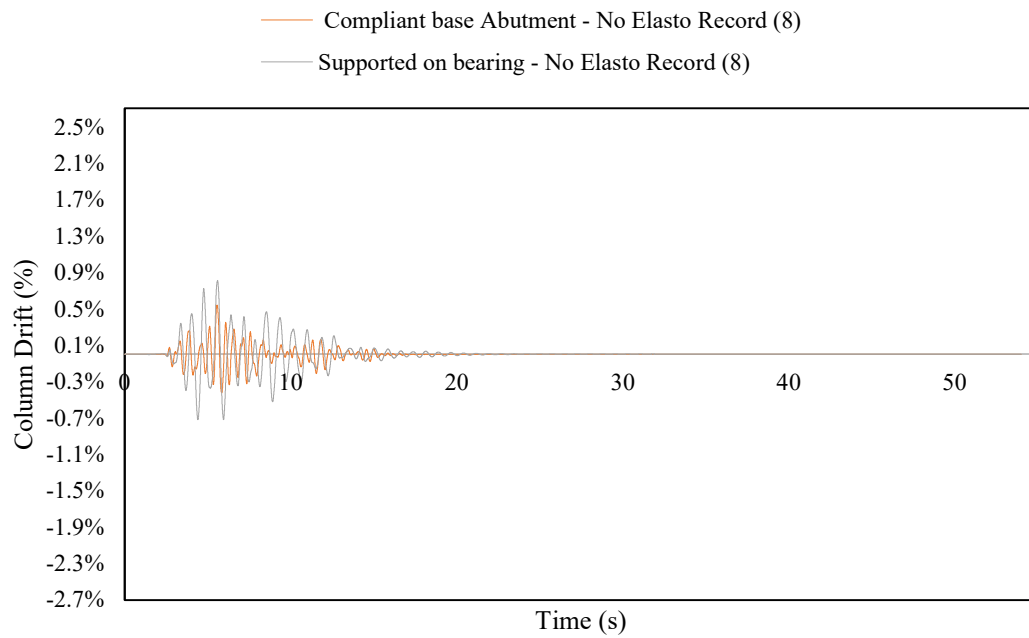
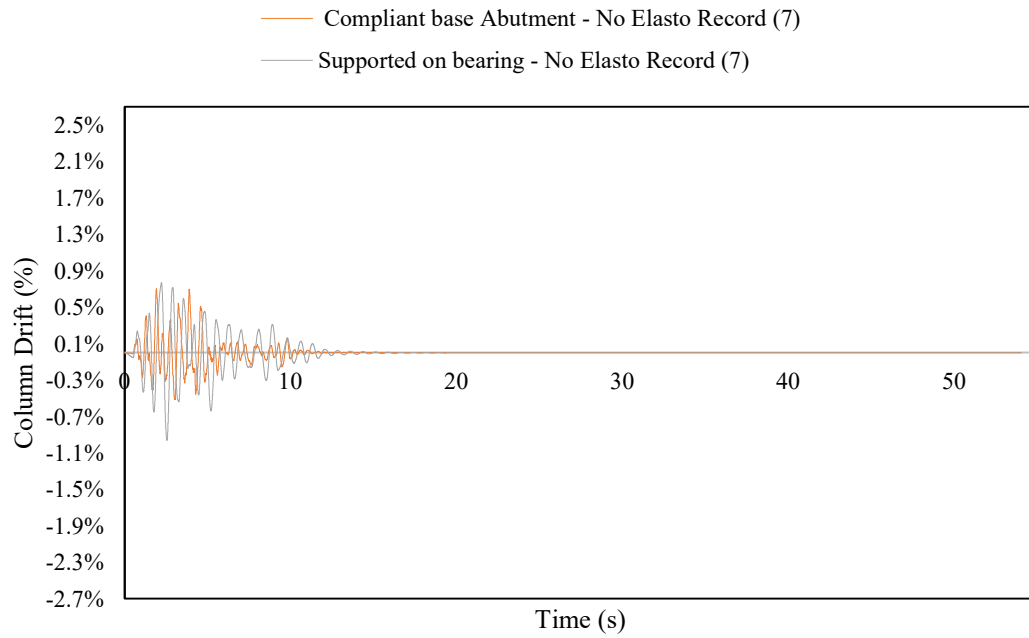


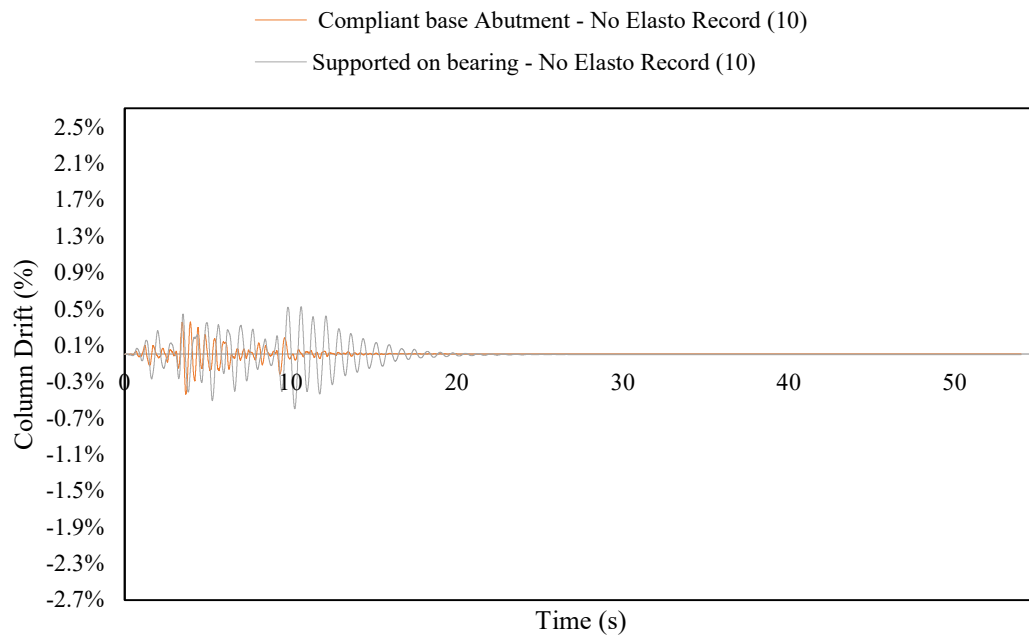
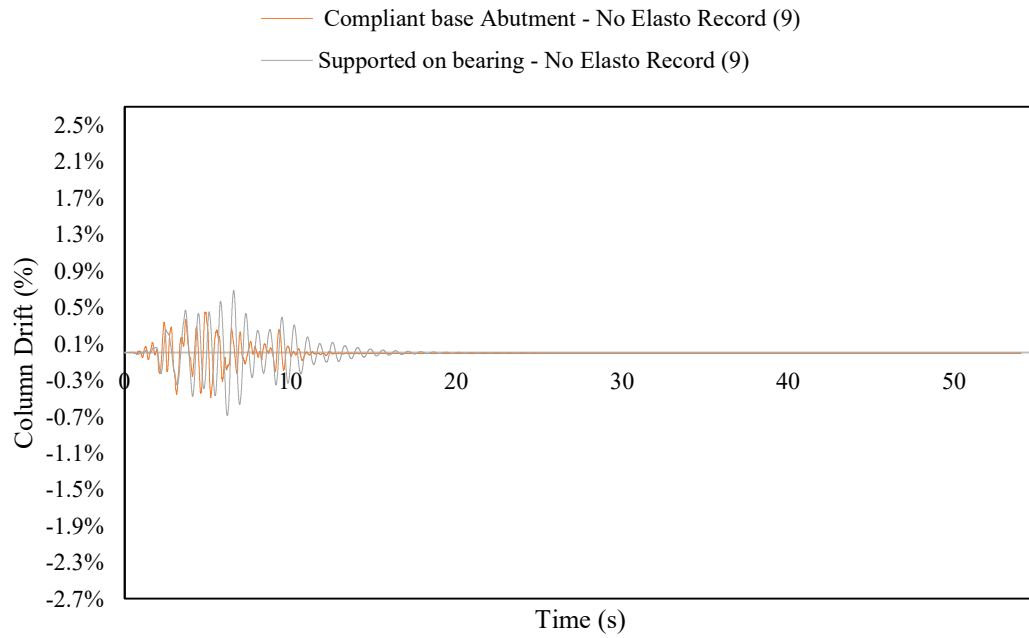
APPENDIX H: Drift history response of bridge due to abutment contribution

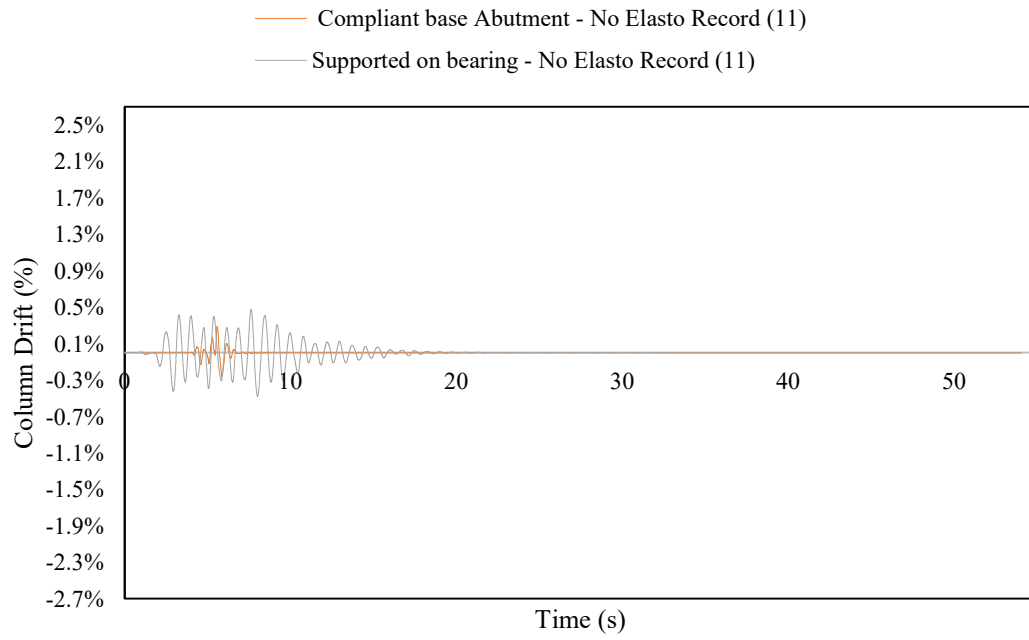












APPENDIX I: Drift history response of bridge for overall foundation springs at pier base

