

FINITE ELEMENT ANALYSIS OF INTEGRAL ABUTMENT BRIDGES

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by

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TABLE OF CONTENTS

TABLE OF CONTENTS.....	ii
LIST OF TABLES.....	v
LIST OF FIGURES	x
1. INTRODUCTION	1
1.1 Problem Statement.....	1
1.2 Objectives	2
1.3 Scope and Limitations	2
2. BACKGROUND	4
2.1 General.....	4
2.2 Other Research.....	7
3. METHOD-STRUCTURAL MODELING.....	12
3.1 General.....	12
3.2 Sap2000 Modeling.....	12
3.3 Pile-Soil Interaction	15
3.3.1 General.....	15
3.3.2 P-y method	17
3.4 Soil-Abutment Interaction	23
3.4.1 General.....	23
3.4.2 Method	24

3.5	Load Modeling and Load Cases	26
4.	BRIDGE I	29
4.1	Introduction.....	29
4.2	Analysis	31
4.2.1	Transformed Section.....	32
4.2.2	Soil Model.....	36
4.2.3	Load cases.....	38
4.3	Analysis Results.....	41
4.3.1	Moment Reduction.....	43
4.3.2	Moment Reduction with Backfill Effect.....	46
4.3.3	Abutment-pile joint.....	50
4.3.4	Depth of Inflection Point	55
4.3.5	Depth of Zero Rotation	59
5.	BRIDGE II.....	63
5.1	Introduction.....	63
5.2	Analysis	66
5.2.1	Transformed Section.....	67
5.2.2	Soil Model.....	72
5.2.3	Load Cases	73
5.3	Analysis Results.....	76
5.3.1	Moment Reduction.....	78
5.3.2	Abutment-pile joint.....	81
5.3.3	Depth of Inflection Point	86
5.3.4	Depth of Zero Rotation	90

6.	BRIDGE III.....	94
6.1	Introduction.....	94
6.2	Analysis	96
6.2.1	Load cases.....	97
6.2.2	Soil Model.....	100
6.3	Analysis Results.....	101
6.3.1	Moment Reduction.....	102
6.3.2	Abutment-pile joint.....	105
6.3.3	Depth of Inflection Point	110
6.3.4	Depth of Zero Rotation	114
7.	SUMMARY, CONCLUSIONS and RECOMMENDATIONS	118
7.1	Summary.....	118
7.2	Conclusions.....	118
7.3	Recommendation	119
8.	LIST OF REFERENCES	120
	APPENDIX A.....	122
	APPENDIX B.....	131
	APPENDIX C	198

LIST OF TABLES

Table 2-1: Estimated Construction Savings for Phase III Integral Abutment Bridges (Allen, 2010)	11
Table 3-1: Representative Values of ϵ_{50} soft clay (Hannigan et al., 1997)	22
Table 4-1 : Properties of Bridge I	29
Table 4-2 : Material properties of Bridge I	33
Table 4-3 : 28” Channel section properties	34
Table 4-4 : Composite section parameters	35
Table 4-5 : Soil parameters used in the analysis	37
Table 4-6 : Steel HP pile section properties	37
Table 4-7: Superstructure moment reductions of Bridge I models relative to clay soil.	44
Table 4-8 : Superstructure moment reductions of Bridge I models relative to sand soil.	45
Table 4-9 : Abutment springs.	47
Table 4-10 : Pile head deformation, forces and computed stiffness values with backfill effect.	48
Table 4-11 : Moment comparison with and without backfill effect.	49
Table 4-12 : Pile head displacements and forces in soft clay, and computed stiffnesses for Bridge I abutments.	51

Table 4-13 : Pile head displacements and forces in medium clay, and computed stiffnesses for Bridge I abutments.....	51
Table 4-14 : Pile head displacements and forces in stiff clay, and computed stiffnesses for Bridge I abutments.	52
Table 4-15: Pile head displacements and forces in very stiff clay, and computed stiffnesses for Bridge I abutments.....	52
Table 4-16 : Pile head displacements and forces in loose sand, and computed stiffnesses for Bridge I abutments.....	53
Table 4-17: Pile head displacements and forces in medium sand, and computed stiffnesses for Bridge I abutment	53
Table 4-18 : Pile head displacements and forces in dense sand, and computed stiffnesses for Bridge I abutments.....	54
Table 4-19 : Physical properties of Bridge I.....	54
Table 4-20 : Depths of inflection points with regards to seven pile sizes and seven soil types.	55
Table 4-21 : Depths of first zero rotation.....	60
Table 4-22 : Depths of second zero rotation.....	60
Table 5-1: Properties of Bridge II.....	63
Table 5-2 : Material properties of Bridge II.....	68
Table 5-3 : Wisconsin precast-prestressed 72W” girder properties.....	69
Table 5-4 : Composite properties of the internal girders.	70
Table 5-5 : Composite properties of the external girders.	71

Table 5-6 : Superstructure moment reductions for Bridge II models relative to clay soil.....	79
Table 5-7 : Superstructure moment reductions for Bridge II models relative to sand soil.....	80
Table 5-8 : Pile head displacements and forces in soft clay, and computed stiffnesses for Bridge II abutments.....	82
Table 5-9: Pile head displacements and forces in medium clay, and computed stiffnesses for Bridge II abutments.	82
Table 5-10 : Pile head displacements and forces in stiff clay, and computed stiffnesses for Bridge II abutments.....	83
Table 5-11: Pile head displacements and forces in very stiff clay, and computed stiffnesses for Bridge II abutments.	83
Table 5-12 : Pile head displacements and forces in loose sand, and computed stiffnesses for Bridge II abutments.	84
Table 5-13: Pile head displacements and forces in medium sand, and computed stiffnesses for Bridge II abutments.	84
Table 5-14 : Pile head deformation and forces in dense sand, and computed stiffnesses for Bridge II abutments.	85
Table 5-15 : Bridge II physical properties	85
Table 5-16 : Depths of inflection points.	86
Table 5-17 : Depths of first zero rotation.....	91
Table 5-18 : Depths of second zero rotation.....	91
Table 6-1: Properties of Bridge III.....	94

Table 6-2 : Material properties of Bridge III.	96
Table 6-3 : Superstructure moment reductions for Bridge III models in clay soil. ..	103
Table 6-4 : Superstructure moment reductions for Bridge III models in sand soil...	104
Table 6-5 : Pile head displacements and forces in soft clay, and computed stiffnesses for Bridge III abutments.....	106
Table 6-6: Pile head displacements and forces in medium clay, and computed stiffnesses for Bridge III abutments.....	106
Table 6-7: Pile head displacements and forces in stiff clay, and computed stiffnesses for Bridge III abutments.....	107
Table 6-8: Pile head displacements and forces in very stiff clay, and computed stiffnesses for Bridge III abutments.....	107
Table 6-9: Pile head displacements and forces in loose sand, and computed stiffnesses for Bridge III abutments.....	108
Table 6-10: Pile head displacements and forces in medium sand, and computed stiffnesses for Bridge III abutments.....	108
Table 6-11: Pile head displacements and forces in dense sand, and computed stiffnesses for Bridge III abutments.....	109
Table 6-12 : Bridge III physical properties.....	109
Table 6-13 : Depths of inflection points for the piles of Bridge III.	110
Table 6-14 : Depths of first zero rotation.....	115
Table 6-15 : Depths of second zero rotation.	115
Table A-1: Spring Stiffnesses for HP10x42 Section, k (k/ft).	123
Table A-2: Spring Stiffnesses for HP10x57 Section, k (k/ft).	124

Table A-3: Spring Stiffnesses for HP12x53 Section, k (k/ft).	125
Table A-4: Spring Stiffnesses for HP12x63 Section, k (k/ft).	126
Table A-5: Spring Stiffnesses for HP12x74 Section, k (k/ft).	127
Table A-6: Spring Stiffnesses for HP14x73 Section, k (k/ft).	128
Table A-7: Spring Stiffnesses for HP14x89 Section, k (k/ft).	129
Table A-8: Spring Stiffnesses in Sand, k (k/ft).	130



LIST OF FIGURES

Figure 3-1: Deflection of pile and distribution of lateral resistance (Poulos & Davis, 1980)	16
Figure 3-2: Form of the results obtained from a complete solution (Reese & Van Impe, 2000). (M: Moment, y: lateral deflection of pile, S: Slope, E_p : Modulus of Elasticity of pile, I_p : Moment of Inertia of pile, V: Shear, p: soil pressure).....	18
Figure 3-3: Typical p-y curve (Greimann, 1987).....	19
Figure 3-4 : Soil Stiffness, adopted from Chovichien (Chovichien, 2004).	22
Figure 3-5: Typical Abutment Soil Spring, adopted from Talbott (Talbott, 2008)	24
Figure 3-6: Relationship between wall movement and earth pressure. (After Clough and Duncan, 1991)	26
Figure 3-7: a) AASHTO HL-93 Design Truck, (AASHTO, 2010) b) AASHTO HL-93Design Tandem.	27
Figure 4-1 : Elevation view of Bridge I	30
Figure 4-2 : Plan view of Bridge I	30
Figure 4-3 : 2-D Model of Bridge I in SAP2000	31
Figure 4-4 : Cross section of the superstructure of Bridge I.....	32
Figure 4-5 :Cross-sectional details of channel section.	34
Figure 4-6 : Cross-section details of composite section of Bridge I.....	35
Figure 4-7 : Illustration of soil-springs on structural-model of a bridge.	36

Figure 4-8 : An illustration of lane definition in SAP2000.	38
Figure 4-9 : Influence line display of Bridge I in SAP2000.	40
Figure 4-10: AASTHO HL-93 Design Truck axle loads.....	40
Figure 4-11 : Illustration of load definition for the frame model of Bridge I.	41
Figure 4-12 : Typical deformed shape of the frame model of Bridge I in SAP2000.	42
Figure 4-13 : Typical moment diagram of the frame model of Bridge I.	42
Figure 4-14 : Achieved superstructure moment reduction relative to clay foundation soil.....	44
Figure 4-15 : Achieved superstructure moment reduction relative to sand foundation soil.....	46
Figure 4-16 : Typical deformed shape of the frame model of Bridge I with backfill effect.	47
Figure 4-17: Illustration of pile as a rotational and a linear spring.....	54
Figure 4-18 : Depth of inflection points of Bridge I piles in soft clay.....	56
Figure 4-19 : Depth of inflection points of Bridge I piles in medium clay.	56
Figure 4-20 : Depth of inflection points of Bridge I piles in stiff clay.	57
Figure 4-21 : Depth of inflection points of Bridge I piles in very-stiff clay.....	57
Figure 4-22 : Depth of inflection points of Bridge I piles in loose sand.	58
Figure 4-23 : Depth of inflection points of Bridge I piles in medium sand.....	58
Figure 4-24 : Depth of inflection points of Bridge I piles in dense sand.....	59
Figure 4-25 : Chart of the depths of first zero rotation in clay soil.	61
Figure 4-26 : Chart of the second zero rotation in clay soil.....	61
Figure 4-27 : Chart of the depths first zero rotation in sand soil.	62

Figure 4-28 : Chart of the depths of second zero rotation in sand soil.	62
Figure 5-1 : Elevation view of Bridge II.....	64
Figure 5-2 : Elevation view of the one span of Bridge II.	65
Figure 5-3 : Plan view of the one half of the Bridge II.....	65
Figure 5-4 : Cross-section of the superstructure of Bridge II.	66
Figure 5-5 : 2-d frame model of Bridge II in SAP2000.....	66
Figure 5-6 : Wisconsin precast-prestressed 72W” girder cross-section details.	69
Figure 5-7 : Composite cross-section of the internal girders.....	70
Figure 5-8 : Composite cross-section of the external girders.	71
Figure 5-9 : An illustration of lane definition of Bridge II in SAP2000.....	74
Figure 5-10 : Influence line display of Bridge II in SAP2000.....	75
Figure 5-11 : Illustration of load definition for Bridge II.	76
Figure 5-12 : Typical deformed shape of the frame model of Bridge II.....	77
Figure 5-13 : Typical moment diagram of the frame model of Bridge II.....	77
Figure 5-14 : Achieved superstructure moment reduction relative to clay foundation soils.	79
Figure 5-15 : Achieved superstructure moment reduction relative to sand foundation soils.	81
Figure 5-16 : Depth of inflection points of Bridge II piles in soft clay.	87
Figure 5-17 : Depth of inflection points of Bridge II piles in medium clay.	87
Figure 5-18 : Depth of inflection points of Bridge II piles in stiff clay.....	88
Figure 5-19 : Depth of inflection points of Bridge II piles in very-stiff clay.	88
Figure 5-20 : Depth of inflection points of Bridge II piles in loose sand.	89

Figure 5-21 : Depth of inflection points of Bridge II piles in medium sand.	89
Figure 5-22 : Depth of inflection points of Bridge II piles in dense sand.	90
Figure 5-23 : Chart of the depths of first zero rotation in clay soil.	92
Figure 5-24 : Chart of the depths of second zero rotation in clay soil.	92
Figure 5-25 : Chart of the depths of first zero rotation in sand soil.	93
Figure 5-26 : Chart of the depths of second zero rotation in sand soil.	93
Figure 6-1 : Elevation view of Bridge III.	95
Figure 6-2 : Plan view of Bridge III.	95
Figure 6-3 : 2-d frame model of Bridge III in SAP2000.	97
Figure 6-4 : An illustration of lane definition of Bridge III in SAP2000.	98
Figure 6-5 : Influence line display of Bridge III in SAP2000.	99
Figure 6-6 : Illustration of load cases for the frame model of Bridge III.	100
Figure 6-7 : Typical deformed shape of the frame model of Bridge III in SAP2000.	101
Figure 6-8 : Typical moment diagram of the frame model of Bridge III.	102
Figure 6-9 : Achieved superstructure moment reduction relative to clay foundation soil.	103
Figure 6-10 : Achieved superstructure moment reduction relative to sand foundation soil.	105
Figure 6-11 : Depth of inflection points of Bridge III piles in soft clay.	111
Figure 6-12 : Depth of inflection points of Bridge III piles in medium clay.	111
Figure 6-13 : Depth of inflection points of Bridge III piles in stiff clay.	112
Figure 6-14 : Depth of inflection points of Bridge III piles in very-stiff clay.	112

Figure 6-15 : Depth of inflection points of Bridge III piles in loose sand.....	113
Figure 6-16 : Depth of inflection points of Bridge III piles in medium sand.	113
Figure 6-17 : Depth of inflection points of Bridge III piles in dense sand.	114
Figure 6-18 : Chart of the depths of first zero rotation in clay soil.	116
Figure 6-19 : Chart of the depths of second zero rotation in clay soil.....	116
Figure 6-20 : Chart of the depths of first zero rotation in sand soil.....	117
Figure 6-21 : Chart of the depths of second zero rotation in sand soil.	117
Figure B-1 : Lateral deflection of piles in medium clay under Truck LL	134
Figure B-2 : Lateral deflection of piles in stiff clay under Truck LL	135
Figure B-3 : Lateral deflection of piles in very stiff clay under Truck LL	136
Figure B-4 : Lateral deflection of piles in loose sand under Truck LL	137
Figure B-5 : Lateral deflection of piles in medium sand under Truck LL.....	138
Figure B-6 : Lateral deflection of piles in dense sand under Truck LL.....	139
Figure B-7 : Bridge I pile rotation in soft clay under Truck LL	140
Figure B-8 : Bridge I pile rotation in medium clay under Truck LL	141
Figure B-9 : Bridge I pile rotation in stiff clay under Truck LL.....	142
Figure B-10 : Bridge I pile rotation in very stiff clay under Truck LL	143
Figure B-11 : Bridge I pile rotation in loose sand under Truck LL	144
Figure B-12 : Bridge I pile rotation in medium sand under Truck LL	145
Figure B-13 : Bridge I pile rotation in dense sand under Truck LL	146
Figure B-14 : Bridge I pile moments in soft clay under Truck LL.....	147
Figure B-15 : Bridge I pile moments in medium clay under Truck LL.....	148
Figure B-16 : Bridge I pile moments in stiff clay under Truck LL	149

Figure B-17 : Bridge I pile moments in very stiff clay under Truck LL	150
Figure B-18 : Bridge I pile moments in loose sand under Truck LL.....	151
Figure B-19 : Bridge I pile moments in medium sand under Truck LL	152
Figure B-20 : Bridge I pile moments in dense sand under Truck LL	153
Figure B-21: Lateral deflection of piles of Bridge II in soft clay under Truck LL...	155
Figure B-22: Lateral deflection of piles of Bridge II in medium clay under Truck LL	156
Figure B-23: Lateral deflection of piles of Bridge II in stiff clay under Truck LL ..	157
Figure B-24: Lateral deflection of piles of Bridge II in very stiff clay under Truck LL	158
Figure B-25: Lateral deflection of piles of Bridge II in loose sand under Truck LL	159
Figure B-26: Lateral deflection of piles of Bridge II in medium sand under Truck LL	160
Figure B-27: Lateral deflection of piles of Bridge II in dense sand under Truck LL	161
Figure B-28 : Bridge II pile rotation in soft clay under Truck LL.....	162
Figure B-29 : Bridge II pile rotation in medium clay under Truck LL.....	163
Figure B-30 : Bridge II pile rotation in stiff clay under Truck LL	164
Figure B-31 : Bridge II pile rotation in very stiff clay under Truck LL	165
Figure B-32 : Bridge II pile rotation in loose sand under Truck LL.....	166
Figure B-33 : Bridge II pile rotation in medium sand under Truck LL	167
Figure B-34 : Bridge II pile rotation in dense sand under Truck LL	168
Figure B-35 : Bridge II pile moments in soft clay under Truck LL	169
Figure B-36 : Bridge II pile moments in medium clay under Truck LL	170

Figure B-37 : Bridge II pile moments in stiff clay under Truck LL	171
Figure B-38 : Bridge II pile moments in very stiff clay under Truck LL	172
Figure B-39 : Bridge II pile moments in loose sand under Truck LL	173
Figure B-40 : Bridge II pile moments in medium sand under Truck LL.....	174
Figure B-41 : Bridge II pile moments in dense sand under Truck LL.....	175
Figure B-42 : Lateral deflection of piles of Bridge III in soft clay under Truck LL	177
Figure B-43 : Lateral deflection of piles of Bridge III in medium clay under Truck LL	178
Figure B-44 : Lateral deflection of piles of Bridge III in stiff clay under Truck LL	179
Figure B-45 : Lateral deflection of piles of Bridge III in very stiff clay under Truck LL.....	180
Figure B-46 : Lateral deflection of piles of Bridge III in loose sand under Truck LL	181
Figure B-47 : Lateral deflection of piles of Bridge III in medium sand under Truck LL.....	182
Figure B-48 : Lateral deflection of piles of Bridge III in dense sand under Truck LL	183
Figure B-49 : Bridge III pile rotation in soft clay under Truck LL	184
Figure B-50 : Bridge III pile rotation in medium clay under Truck LL	185
Figure B-51 : Bridge III pile rotation in stiff clay under Truck LL	186
Figure B-52 : Bridge III pile rotation in very stiff clay under Truck LL.....	187
Figure B-53 : Bridge III pile rotation in loose sand under Truck LL	188
Figure B-54 : Bridge III pile rotation in medium sand under Truck LL.....	189

Figure B-55 : Bridge III pile rotation in dense sand under Truck LL.....	190
Figure B-56 : Bridge III pile moments in soft clay under Truck LL	191
Figure B-57 : Bridge III pile moments in medium clay under Truck LL	192
Figure B-58 : Bridge III pile moments in stiff clay under Truck LL.....	193
Figure B-59 : Bridge III pile moments in very stiff clay under Truck LL.....	194
Figure B-60 : Bridge III pile moments in loose sand under Truck LL	195
Figure B-61 : Bridge III pile moments in medium sand under Truck LL	196
Figure B-62 : Bridge III pile moments in dense sand under Truck LL	197

1. INTRODUCTION

1.1 Problem Statement

Integral abutment bridges have been a good alternative and promising one to conventional bridges because of several advantageous they provide. The primary advantageous are that the negative end moments created in the bridge girders reduce the mid span positive moments and deflections. However, in most cases the real efficiencies that an integral abutment brings into the design have not been certain and not well-defined yet, such as how much extra span can be achieved or how much can be saved in terms of a girder size. The difficulty in predicting the real capacity and contribution of an integral abutment stems mainly from the involvement of lots of effects in the performance of integral abutment bridges. Mainly, they are affected by foundation soil characteristic, pile type, load type (static, dynamic, cyclic), bridge skew and etc. Many other variables can also be derived as a function of these mentioned. The lack of guidelines on the design and construction of integral bridges also bring difficulties to bridge designers and owners to assess the real behavior and capacity of the structure. In light of these this study aimed to get a better insight of the integral abutment contribution by means of a parametric study; although not comprehensive, this study is intended to provide some data to contribute in the development of guidelines for selecting when integral abutments may be most effective.

1.2 Objectives

The primary objective of this study is to investigate the contribution of integral-abutments to conventional bridges in terms of the usage of lighter and less deep girder sections in the design for superstructure due to reduced positive moments. In conventional bridges the positive moment in the middle span can be reduced by creating negative moments at the either ends of the span which can be done by means of the integration of the girders with the abutments. In the literature, most of the research that was performed focuses on the behavior of integral abutment bridges (IABs) under thermal change and time dependent effects of creep and shrinkage. But the research concerning the live-load effects on integral bridge components are scarce. The study was aimed to create some data on the contribution of integral abutments and the effects of live loads.

In addition to the first objective, an investigation of the pile-abutment joint in an integral system, which is exposed to rotational stresses under live load, is the other goal. Because integral abutments inherently have a moment carrying mechanism, they are exposed to high rotational deformation and moment stresses in the service-life of the bridge. To be able to create an approximation for a range of integral-abutments characteristic with a certain types of bridges will enable more efficient pile and girder design.

1.3 Scope and Limitations

This research study is limited to short to medium length span of integral abutment bridges. In the study, the bridges are assumed to be symmetric with no skew. Abutments are

assumed to be supported by a single row of steel HP piles as commonly used in Wisconsin. A moment connection is assumed between the piles and abutments as well as between the superstructure and abutments. Typical pile lengths range from 40 to 120 feet and common HP-pile sizes vary between 10 and 14 inches (WisDOT, 2011). In this study, seven sizes of piles are considered and they are; HP10x42, HP10x53, HP12x53, HP12x63, HP12x74, HP14x73 and HP14x89. The pile lengths considered in the analysis for short length bridges is 30 ft. and 40 ft., and for the medium length bridge 50 ft. Piles are used with their strong axis parallel to the longitudinal span of the bridge. In terms of pile foundation, both cohesive and cohesionless soils are considered. Four different consistency of clay: soft, medium, stiff and very-stiff, and three types of sands: loose, medium, and dense are considered. In the analysis, the ground water level is assumed deeper than the bottom end of the pile, so, dry properties of the soils are used. In the analysis, self-weight or the dead loads of the components are neglected because the axial deformation of piles are negligible and not of interest. Although, superimposed dead loads are considered, the load is not used in the analysis of short length bridges but two span continuous bridge. Additionally, loads due to thermal effects, time dependent loads (shrinkage, creep), dynamic loads such as vehicle acceleration, braking or impact loads are not considered. Lastly, the behavior of pile foundation soil and the concrete of the structure are assumed to be within the linear-elastic range, that is, yielding of the piles and cracking of concrete is not anticipated.

2. BACKGROUND

2.1 General

One of the main behaviors of traditional bridges is their ability to have tolerance to movements which may be caused by the loads due to vehicles, winds, earthquakes, shrinkage, creep and temperature change. Forces from these effects can be accommodated by expansion joints allowing deformation. In this sense expansion joints are critical in terms of the serviceability of conventional bridges. On the other hand, bridges with jointed superstructures require extra care and budget in maintenance. According to reports by other authors, based on the feedback and past experience from the owners of bridges, high initial cost in the construction and future maintenance of joint components are the utmost and primary concern (Kunin & Alampalli, 2000).

Considering the drawbacks and extra cost of superstructure expansion joints, the integral abutment bridge concept emerged in the industry as a response to the demand of constructors and an alternative option for cost-effective bridges (Kunin & Alampalli, 2000). The primary benefit of integral abutments is that they enable the construction without expansion joints, therefore, initial construction cost and likely future maintenance cost are eliminated. Additional benefits are, simple design and construction procedures, rapid construction, ease in construction embankments, simple form, vertical piles, and improved ride quality. Further, design efficiency is significant, such that, by having superstructure and substructure integrated, it is rendered possible that the loading on superstructure can be better distributed to the substructure. Therefore much less design forces are apparent and thereby

smaller sections can be utilized (Maruri & Petro, 2005). In addition to the primary benefits, the IAB concept provides a number of indirect benefits as well, such as the protection of nature thanks to the construction method that enables less interaction with nature. Cost is saved due to the decrease in the number of hours that is spent on labor as a result of more rapid construction (Allen, 2010).

The employment of the integral abutment concept by the States has increased steadily since the 1980s (Maruri & Petro, 2005), and becoming popular across the United States, in particular at a time when States' budgets are limited and funding is scarce (Kunin & Alampalli, 2000). In spite of this, currently there are no guidelines or single adopted procedure available in the country on the design and construction of integral or semi integral bridges. Because of lack of a guideline and a common procedure, there are uncertainties regarding the capacity of integral abutment bridges. According to Maruri and Petro many States have taken steps towards improvement of the design and detailing of integral abutments, and came up with different techniques mostly based on their past experience and observations on the constructed bridges.

Commonly, the concept of Integral abutments is implemented on short to medium length bridges (Maruri & Petro, 2005). However longer lengths have been achieved by some states. According to survey result of those authors, the longest precast concrete girder structure was an 1175 ft. bridge built in Tennessee, a 1044 ft. long steel and a 953 ft. long concrete bridge in Colorado (Kunin & Alampalli, 2000). The state of Tennessee currently owns the longest bridge that employs integral abutment (Arsoy et al., 1999).

In IABs the girders and piles get integrated rigidly with abutments, which makes the whole structure like an indeterminate frame. That is, all the stiffness and flexibilities are distributed to the components of bridge and foundation soil. In more than one span continuous bridges, the intermediary piers can be constructed integrally or independently with the support (Maruri & Petro, 2005). The skew of the integral bridges are limited and mostly below 30 degrees. From 3 ft. to no limit is used in terms of abutment heights. Further, steel HP piles are mostly preferred due to their high ductile capacity along with the cast-in-place, prestressed, pipe and concrete piles (Kunin & Alampalli, 2000).

Burke (2009) states that, pile-abutment joints are exposed to dynamic forces due to vehicular movements as well as stresses from temperature change. Consequently, piles are exposed to both axial and flexural loading which makes the pile-abutment joint critical. Further, flexibility of the substructures is crucial due to the load distribution nature of integral abutments, such that the flexural resistance of the substructure has an influence on the design length of the span. The abutments are considered to have enough flexural capacity to use with a single row of piles under the dynamic vehicular loads. Steel piles were observed to be reaching yielding under these loadings (Burke, 2009).

The factors that have influence on the lateral pressure on pile foundations can be given as live loads, earthquake, wind loads, water pressure and earth pressure. It is stated in the report of other authors that pile foundations should be designed to accommodate these loadings and smaller deflection, and the tolerable deflection is the governing criterion that defines the capacity of laterally loaded vertical piles (Barker, Duncan, Rojiani, Ooi, Tan, & Kim, 1991a). There are procedures available to analyze the vertical piles under lateral

loading to measure the capacity. Elastic analysis, subgrade analysis, and p-y methods are commonly used. According to other authors, the elastic and subgrade reaction analysis approximates the soil behavior as linear (Poulos & Davis, 1980). However, in the p-y method the nonlinear behavior of soil is considered. The p-y method is developed by Reese to use with a computer program. Evans and Duncan developed a method for the model of nonlinear behavior of soil, but it does not require a computer analysis (Barker, Duncan, Rojiani, Ooi, Tan, & Kim, 1991a).

2.2 Other Research

Some authors used 2-D frame modeling techniques to analyze two bridges in Iowa to verify a design procedure for piles in integral abutment bridges (Girton, Hawkinson, & Greimann, 1991). In the study, piles were modeled as equivalent cantilever beam columns to model the real bridges in order to approximate longitudinal displacement under temperature change. Predictions of longitudinal thermal expansion of bridge structures and pile strains were developed. As a result, it was reported that good results were achieved between the analytical and experimental model.

Thippeswamy and Gangarao (1995) analyzed five in service jointless bridges under various load cases. In the modeling they idealized the bridges as 2-D frame models. Some of the findings of their analyses are summarized here. According to their results, live load stresses at the abutment top and bottom interface are negligible because of the flexibility of the structure where the system behaves like a simply supported structures (Thippeswamy &

Gangarao, 1995). Under dead loads, weak axis bending of piles shows 3 times less stresses than strong axis bending of piles. The authors suggest that bridges should be supported on piles that bend about their weak axis since this reduces the stresses at the superstructure-abutment joint. The stresses in concrete where the piles are fixed to the abutment are very small. Thus there is no fear of concrete cracking or separation of abutments from piles.

In their study, Dicleli and Erhan (2010) examined the soil-bridge interaction on the magnitude of internal forces in the components of IABs by a parametric study. In the analysis, short to medium lengths of integral abutment bridges are used, and they are modeled as 2-D frame, and analyzed with SAP2000 using a linear-elastic assumption. P-y method is used to model the soil-structure interaction. The HL-93 Design truck is used as a live load. Based on results of the study, the authors report that internal forces of IAB components are highly dependent on the soil stiffness, and therefore soil-structure behavior is quite important regarding distribution of internal forces between the bridge components. However, the soil-structure stiffness becomes insignificant in a case of multiple span bridges. The positive moments over the girders and the negative moments over the supports are affected less by the change in the soil properties. This is explained by the fact that the rotational stiffness over the intermediate supports is relatively larger compared to the rotational stiffnesses on either ends of the span, which makes the rotational deformation or change insignificant. Thus, small changes in the rotation in these ends don't contribute to the magnitude of the internal forces on the superstructure components. It is also reported that superstructure shear is not changed much as a function of soil properties, but, in the contrary, it is found out that the moments and shear forces on piles are highly dependent on soil

stiffness. Moreover, when the backfill soil is considered it causes a smaller span and pile moments and larger abutment moments due to larger abutment stiffness under backfill effects (Dicleli & Erhan, 2010).

Chovichian (2004) investigated the pile behavior of integral abutment bridges under thermal expansion and contraction. As part of that author's research, analytical models were developed to perform a parametric study to evaluate the effect of the primary variables involved in the soil-pile interaction. Some of the results which are related to this study are reproduced here as follows;

- Pile axial load does not have a significant effect on the behavior of piles in the soil. The deflected shape and moment distribution along the pile were not significantly affected.
- A minimum pile length must be provided below ground level in order to prevent displacement at the base of the pile.
- For the same soil type, as the pile stiffness increases due to size or rotation, the deflected shape of the pile is affected. The inflection point, the final zero deflection, and the final zero moment depths increase.
- For the same pile type and orientation, as the soil stiffness increases, the deflected shape of the pile is affected. The inflection point, the final zero moment depths decrease.

The Vermont Agency of Transportation (VTrans) has taken comprehensive research towards the development of procedures in the use of integral abutment bridge construction in Vermont. The State has preferred bridges with integral abutments as a first choice in the past because of the distinct benefits such as rapid and lower cost of construction, reduced environmental effect and the elimination of future maintenance. The lack of a single standard design procedure, and the uncertainty regarding how efficient and economical the design and construction of an IAB can be some of the primary drawbacks. In order to best approach

design of integral abutment bridges, VTrans embarked on a research project which was targeting to overcoming the uncertainties and to make use of the integral bridges in a much more efficient manner. The research project was divided into phases initially, to obtain tangible data based on complete analyses, performance monitoring and literature review on jointless bridges, and then to develop design guidelines, contract plans and specification (Allen, 2010).

In order to monitor performance and obtain tangible data to see the efficiency of integral abutment bridges on an economical scale, three integral abutment bridges were instrumented and monitored. The bridge named Middlesex is a straight bridge with a 43 meter (141 ft.) span, the bridge named East Montpelier is a 15 degree skewed bridge with a 37 meter (121 ft.) span and the third bridge named Stockbridge is two span, has curved girder with an 11.25 degree curvature and has a total length of 69 meters (226 ft.). The Vermont findings are compared in Table 2-1 with an estimated cost related to phases of conventional abutment bridges. The data on the table does not contain indirect savings such as reduced construction and maintenance.

As a result it was reported that the total construction savings are 16.8, 19.8 and 9.5 percentages for East Montpelier, Stockbridge and Middlesex bridges, respectively. These results are partial proof of significant savings of IABs against the conventional bridges.

Table 2-1: Estimated Construction Savings for Phase III Integral Abutment Bridges (Allen, 2010)

	East Montpelier	Stockbridge	Middlesex
Total project cost, as bid	\$2,369,907	\$4,155,879	\$2,254,458
Savings by category			
Cofferdam construction	\$100,000	\$150,000	—— ^a
Substructure concrete and and reinforcing	\$250,000	\$308,000	\$140,000
Steel Piling	\$25,000	\$310,000	—— ^b
Granular backfill	\$22,000	\$33,000	\$35,000
Excavation	—— ^c	\$20,000	\$40,000
Total Savings	\$397,000	\$821,000	\$215,000
Savings from project bid	16.8%	19.8%	9.5%

^a Conventional abutment did not require a cofferdam

^b Conventional abutment would have utilized a spread footing foundation

^c Excavation savings are included in the cofferdam construction savings, noted above

3. METHOD-STRUCTURAL MODELING

3.1 General

To be able to investigate pile behavior of integral abutment bridges (IABs), an entire modeling of the structure is required. This is because of the fact that an IAB exhibits monolithic behavior under loading. The behavior of superstructure and substructure is interdependent. Due to this dependency, loads from the superstructure, for example, are distributed through the substructure and the soil in proportional to the stiffnesses. Hence, an indeterminate frame behavior takes place that involves an overall structural analysis.

There are three bridges considered in this study to investigate the integral abutment contribution to increasing possible span length of conventional bridges by means of moment comparison, and to observe the pile behavior by measuring the pile deformation under live loading. The definitions of the three bridges are given in the following section. The general description of the analysis method and the analysis parameters are presented in the subsequent sections.

3.2 Sap2000 Modeling

Considering the objectives and the scope of the study, a two-dimensional modeling technique was adopted. Other authors describes the 2-D frame modeling as simpler, faster and reliable compared to 3-D modeling, because of the fact that 2-D frame models are easy to build and analyze since simple beam elements are used. In this sense, they are also

appropriate for a parametric study (Thippeswamy & Gangarao, 1995). 2-D modeling does not capture the exact distribution of forces across the width of the bridge. But since maximum girder moments are caused by loads applied near the center of the span, when forces reach the abutments the load effects have had an opportunity to spread out to create uniform distribution as assumed in 2-D modeling.

The 2-D frame models of the IABs were built and analyzed by using the finite element software SAP2000 V14. The bridges considered in the study were assumed as symmetric lengthwise and have no skews. All of the components were built using linear-elastic beam elements since cracking of concrete or failure or softening of the soil was not assumed. In the definition of the beam elements in the 2-D model, lumped properties of the girders and the piles were used. That is, the total number of girders for the superstructure and the total number of piles for the substructures were each simulated by a single beam element.

In IABs, loads from the superstructure are transferred to the substructure through the abutments. Abutments are the intermediate member in the model and critical in this sense. It was assumed that the heavy concrete abutment elements are rigidly connected to the girders and the piles, and to account for the high rigidity of abutments, higher than normal material properties were assigned to them to create higher flexural stiffness. In other words, the flexural stiffness of each abutment was increased through the use of higher modulus of elasticity that will create a rigid behavior for the concrete abutment in the 2-d frame model.

In terms of superstructure definition, fully composite action between the girders and the slab are assumed. The structural parameters that are considered in the two-dimensional model are the cross sectional area and the moment of inertia about the strong axis. A single

composite property of a girder and a single property of a pile were used to get lumped properties for the superstructure and substructure definition, respectively in the 2-D model in SAP2000.

In the modeling of the two-span continuous bridge, the pier that provides intermediate support to the superstructure is modeled as a roller support since girders were not assumed to be integrated with the pier. It is believed that this can be assumed regardless of the connection type since the piers are spaced far from the abutments and a change in the stiffness there, relative to the other stiffnesses in the structure, should not cause significant changes in abutment forces. However, this assumption is likely more valid when the bridge are symmetric from end to end. Modeling the pier as a roller support also makes the model simpler and the analysis less complicated.

Although there is a controversy as to whether piles should be used with their strong or weak axis parallel to the girders in the integral bridge system, it is considered in this study that piles are oriented with their strong axis parallel to the longitudinal position of the bridges. Further, piles are assumed to have enough embedded length in the abutments to create a rigid behavior in the connection.

In bridges, live loads such as vehicular loads can be distributed across the slab as a function of the properties of the slab and girders. In numerical analysis the distribution of live loads is considered by the use of live load distribution factors which are a function of the girder span, girder spacing, and a girder stiffness factor. Current AASTHO (2010) specifications contain correlations for live load distribution factors for conventional bridges. However, there is no guideline in the code that addresses the use of live load distribution

factors (LLDF) in the use of integral abutment bridges as well as no other guideless related to IABs. For this reason the method of analysis here was formed and performed in such a way that it excludes the use of LLDFs. Hence, the decision was taken that in the structural analysis of the bridges the whole properties of the bridge cross section is to be used, for example lumped flexural stiffness of all the composite girders that compose the superstructure will be used rather than a number of girders that are shared by a single lane.

3.3 Pile-Soil Interaction

3.3.1 General

Poulos and Davis (1980) state that if soil failure occurs before the failure of piles, piles are considered rigid, so that soil resistance determines the lateral capacity. Yielding moment of piles is reached first in the case of relatively long piles, and this determines the ultimate lateral resistance rather than the soil failure. Also as they indicate that in cohesive soils the lateral pressure does not increase continuously along the depth of the pile, rather it reaches an ultimate pressure somewhere near to three times the pile diameter from the ground surface and becomes constant or approximately it becomes constant when the lateral pressure reaches 8 to $12c_u$ where c_u is the undrained shear strength of cohesive soils (Poulos & Davis, 1980). The typical ultimate soil resistance is illustrated by those authors as in Figure 3-1.

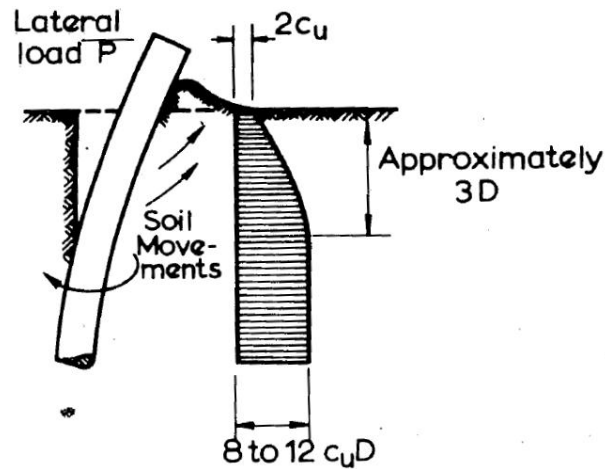


Figure 3-1: Deflection of pile and distribution of lateral resistance (Poulos & Davis, 1980)

The authors also describe the methods which are widely used for predicting the deflection of laterally loaded piles. One is the subgrade-reaction approach and other is the elastic approach. As it is described by the author, in the subgrade-reaction approach the soil is characterized as a series of unconnected linearly-elastic springs which does not allow continuity, so that the deformation at each spring is independent. However, the continuum nature of soil is taken account in the elastic approach which is a more accurate approach as far as the non-linear behavior of soil is considered.

In the present modeling phase of soil-pile interaction in SAP2000, a nonlinear state of the soil was neglected and it was assumed that yielding moments of piles are not reached. Because the study is concerned with the pile deformation under live loadings it is expected that only small deflections and the surrounding soil will not reach its ultimate strength. This assumption is also verified in the upcoming chapter where the analysis results are presented.

3.3.2 P-y method

The p-y method is developed by Reese to use with a computer program, and it has become a common procedure to model the nonlinear behavior of pile-soil interaction (Reese & Van Impe, 2000). It is stated by the author that moments along the length of the piles are calculated more correctly than the pile deflections, therefore field test is recommended as far as the pile deflection is concerned. Some of the important concepts are given by the author as follows: “in most cases, the properties of the soil between the ground surface and a depth of six to ten diameters will govern the behavior of piles subjected to lateral loading.”, “Experimental results, and elementary theory, show that p-y curves near the ground surface have a dominant effect on the computation of pile deflection and bending moment” “The p-y method is based on a Winkler approach that the soil is not treated as a continuum, but as a series of discrete, uncoupled resistance.” However the p-y method gives reasonable response. Figure 3-2 shows a set of curves that are the result of analysis of laterally loaded piles by a computer program. The derivations of these curves are based on basic beam theory. Such curves are also expected as a result of this study.

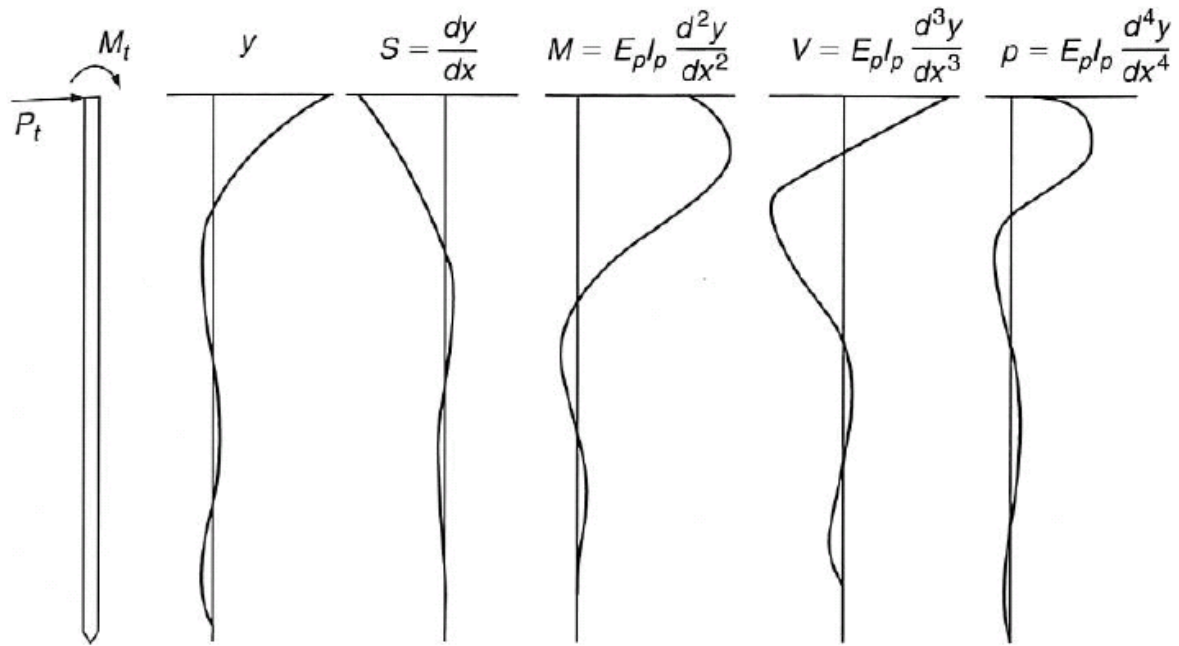


Figure 3-2: Form of the results obtained from a complete solution (Reese & Van Impe, 2000).
(M: Moment, y: lateral deflection of pile, S: Slope, E_p : Modulus of Elasticity of pile, I_p : Moment of Inertia of pile, V: Shear, p: soil pressure).

In the present analysis, the soil lateral resistance under vehicular loading is represented by linear-elastic springs, and the piles are modeled as linear elastic beam elements using the finite element software SAP2000. In the p-y method the soil resistance, p force per unit length of pile and pile deflection, y, are related to each other by a nonlinear curve analogous to the behavior of real soil. In the p-y method it is assumed that soil pressure is related to pile deflection by pile and soil stiffness. The relationship of a p-y curve is shown in Figure 3-3. Although the soil properties have the dominant effect on the formation of p-y curves, some other parameters that have influence are depth, pile width, water table location, loading condition (static or cyclic) and etc. (Hannigan, Goble, Thendean, Likins, & Rausche, 1997).

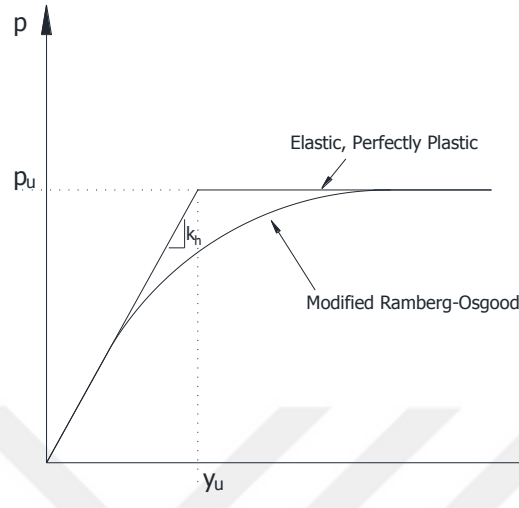


Figure 3-3: Typical p-y curve (Greimann, 1987).

In the development of p-y curves for the present analysis, Greimann's approximate method to the modified Ramberg-Osgood is used (Greimann, 1987). In this method Greimann has expressions that enable one to get the spring stiffness for soft and stiff clay calculated as follow:

For soft and stiff clay the soil stiffness is calculated as;

$$k_h = \frac{p_u}{y_{50}} \quad 3-1$$

Where the ultimate soil resistance is calculated from the following expression;

$$p_u = \min \left\{ \left(3 + \frac{\gamma}{c_u} * z + \frac{0.5}{B_p} * z \right) * c_u * B_p \right. \quad 3-2$$

For very stiff clay the soil stiffness is calculated as;

$$k_h = \frac{p_u}{2 * y_{50}} \quad 3-3$$

Where the ultimate soil resistance is calculated from the following expression;

$$p_u = \min \left\{ \begin{array}{l} 9 * c_u * B_p \\ \left(3 + \frac{\gamma}{c_u} * z + \frac{2}{B_p} * z \right) * c_u * B_p \end{array} \right\} \quad 3-4$$

Where;

P_u = ultimate soil resistance, kip/ft

c_u = undrained shear strength of clay, in TABLE

B_p = pile width, ft;

γ = effective, unit-weight of the soil, pcf;

z = depth of soil spring from ground surface, ft;

Y_{50} = displacement at one-half of the ultimate soil reaction, ft;

$2.5B_p \epsilon_{50}$ for soft and stiff clay and

$2.0B_p \epsilon_{50}$ for very stiff clay

ϵ_{50} = axial strain at one-half of the peak stress difference from a tri-axial test

0.02 for soft clay,

0.01 for stiff clay,

0.005 for very stiff clay

For sand the soil stiffness is obtained by the equation

$$k_h = \frac{J * \gamma * k}{1.35}$$

3-5

Where;

B_p = pile width, ft;

γ = effective, unit-weight of the soil, pcf;

J = soil parameter;

200 for loose sand

600 for medium sand

1500 for dense sand.

Using the foregoing correlations for the soil models, linear elastic spring stiffnesses can be obtained and defined over the length of the pile. Although medium clay is not mentioned in the correlations above, assuming medium consistency of clay is an intermediate of soft and stiff clay, the correlations were adopted in the development of medium clay. The “0.5” value in the correlation was changed to “0.25” based on the recommendation of Matlock (1970). A typical pattern of soil stiffnesses relative to the soil type is illustrated in Figure 3-4. In a sand foundation soil, there is a continuous linear increase of soil stiffness along with the depth. However, soil stiffness becomes constant after some depth from the ground surface for the clay foundation soil.

With regard to the Figure 3-4, the area belonging to a particular spring on the interface is taken as the tributary length times the pile diameter; where the tributary length is

taken as the sum of the half spacing above and below the spring, and the pile diameter is referred to as the flange width since piles are used about their strong axis.

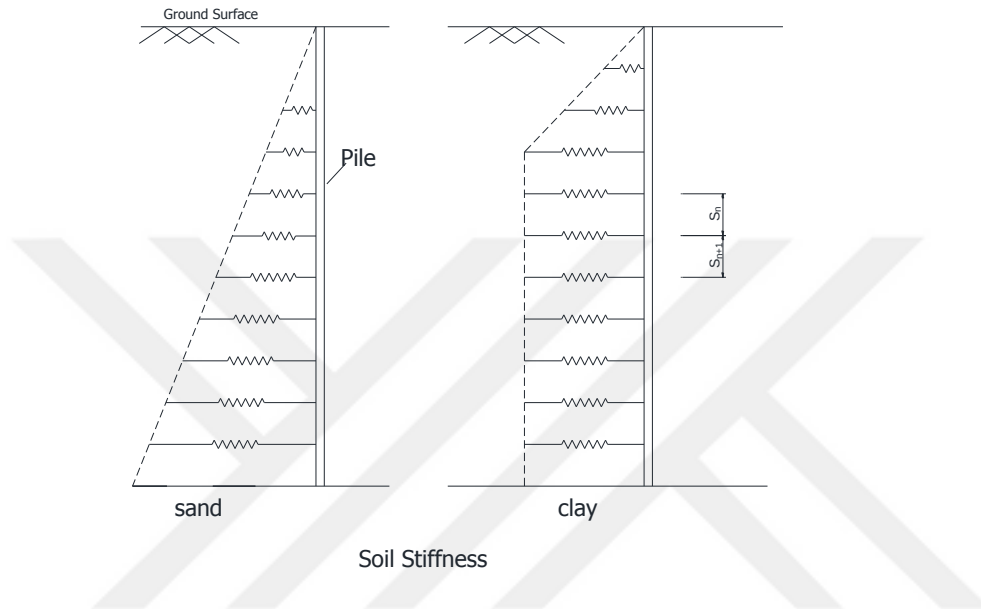


Figure 3-4 : Soil Stiffness, adopted from Chovichien (Chovichien, 2004).

Table 3-1: Representative Values of ϵ_{50} soft clay (Hannigan et al., 1997)

Clay Consistency	Average Undrained Shear Strength, c_u (kPa)	ϵ_{50}
Soft Clay	12 - 24	0.02
Medium Clay	24 - 48	0.01
Stiff Clay	48 - 96	0.007
Very Stiff Clay	96 - 192	0.005
Hard Clay	192 - 383	0.04

3.4 Soil-Abutment Interaction

3.4.1 General

The procedure of modeling the soil-abutment interaction is different than the modeling of soil-pile interaction. The abutments are considered to behave similar to retaining walls. The earth pressure exerted on a retaining wall can be classified into at-rest, active, and passive states. As stated in a report by Barker, the wall displacement has an effect on the magnitudes of soil pressures that are applied on the wall surface (Barker, Duncan, Rojiani, Ooi, Tan, & Kim, 1991). Active state occurs when the abutment moves away from the backfill, and in the same sense, passive pressure takes place when the abutment moves toward the backfill. The at-rest state is the stationary form of the abutment where there is no movement to either side. In terms of bridges, the active state develops during the temperature contraction of the superstructure which causes the abutments to move inwards. Conversely, the passive state develops during the expansion of the superstructure when abutments are pushed towards the backfill. AASTHO (2010) defines the required movement of the abutment to reach the passive state as 2% of the wall height. Taking the movement of abutment as 2% percent of the height is a common procedure in the design. Although, the lateral deflection of the abutment under live load effect is not significant, for only one case in the analysis backfill effect was considered to see its influence on the load distribution. One analysis case in Bridge I was made including the backfill effect. Except this, backfill effect is not considered in the analysis. In general the backfill effect is not considered in the present analysis, except for one case (for the 1st bridge to be investigated) where backfill influence will be investigated.

3.4.2 Method

A typical force-displacement relationship for the abutment-soil springs is shown in Figure 3-5. It is a common practice in the construction of bridges to use granular soil as a backfill. Considering this, it is assumed in this study that backfill consists of medium sand. In order to get the backfill pressure and accordingly the subgrade reaction, the model developed by other authors (Barker, Duncan, Rojiani, Ooi, Tan, & Kim, 1991) was used to get the soil parameters in the computation. In this model, the relationship of lateral earth pressure is given as a function of Δ/H values, where Δ is the lateral deflection and H is the abutment height. The relationship is shown in Figure 3-6.

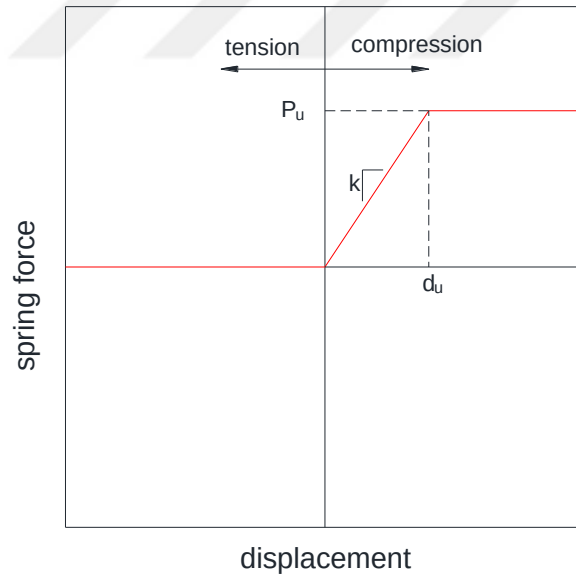


Figure 3-5: Typical Abutment Soil Spring, adopted from Talbott (Talbott, 2008)

Assuming a small abutment deflection under live load, the Δ/H value can be chosen between 0 and 0.001. The corresponding values for K_p , passive earth pressure, and K_o , at-rest earth pressure, are found to be 0.125 and 0.4, respectively, using the Figure 3-6. For simplicity, a uniform lateral abutment deflection can be adopted which is also a conservative assumption. Hence, abutment deflection can be taken as $0.001H$ for any spring level on the abutment-backfill surface. Assuming the unit weight of medium sand is 120 lb/ft^3 , the earth pressure exerted on the abutment surface by the backfill is computed as;

$$\Delta P = (K_p - K_o) * \gamma * z \quad 3-6$$

A spring force is obtained by multiplying the pressure by the tributary area of each spring along the interface of backfill-abutment. Having found the forces that are normal to the abutment surface, the stiffness of each spring can be obtained by dividing the force by the abutment displacement which was assumed $0.001H$ as uniform along the abutment height.

$$F = \Delta P * s * w \quad 3-7$$

$$k_i = \frac{F_i}{0.001 * H} \quad 3-8$$

Where;

K_p : passive earth pressure

K_o : at-rest earth pressure

γ : unit weight of sand

z : depth measured from ground surface

w : width of the abutment

s :spacing between the spring nodes.

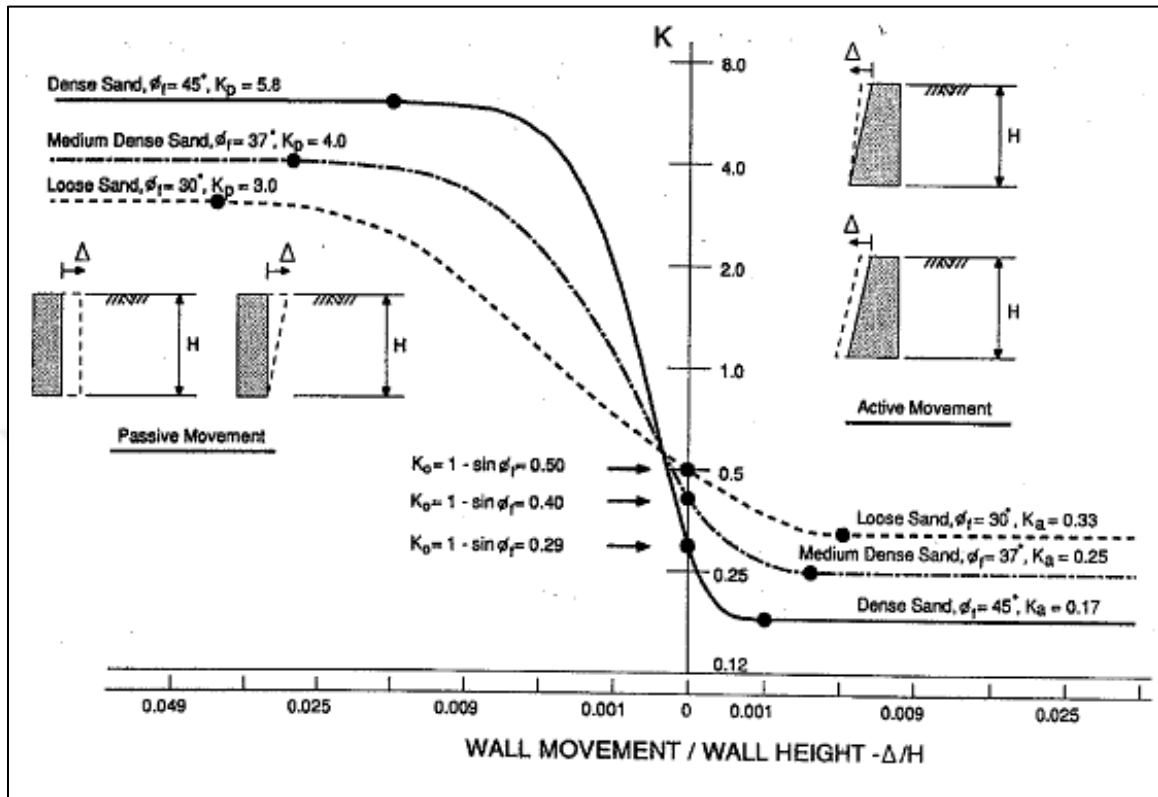


Figure 3-6: Relationship between wall movement and earth pressure. (After Clough and Duncan, 1991)

3.5 Load Modeling and Load Cases

Live loads and dead loads are considered primary loads in this investigation while temperature, creep and shrinkage loads are considered as secondary. It is a common approach in the design of integral bridges that creep and shrinkage are assumed opposite in nature and they may cancel out (Thippeswamy & Gangarao, 1995). Here the loads related to self-weight (which exist before the integral connection is made between superstructure and substructure) thermal change, and time dependent loads due to creep and shrinkage are not considered,

since the purpose of the research is to investigate live load effects on pile behavior and substructure to superstructure connection forces.

The live loads considered in this study are the vehicle loads based on the AASHTO (2010) load cases. AASHTO HL-93 live load consists of a design truck or design tandem and design lane load as shown in Figure 3-7. The design truck has three axles with the front axle load of 8 kip, middle axle load of 32 kip and rear axle load of 32 kip. The spacing between the two 32 kip axles should be varied between 14 ft. and 30 ft. The design tandem consists of a pair of 25 kip axles spaced 4 ft. apart. The design lane load consists of uniformly distributed load of 0.64 kip/ft. in the longitudinal direction, and distributed over 10 ft. width transversely. It is recommended by AASHTO (2010) that the LL cases which induce the larger effect must be employed.

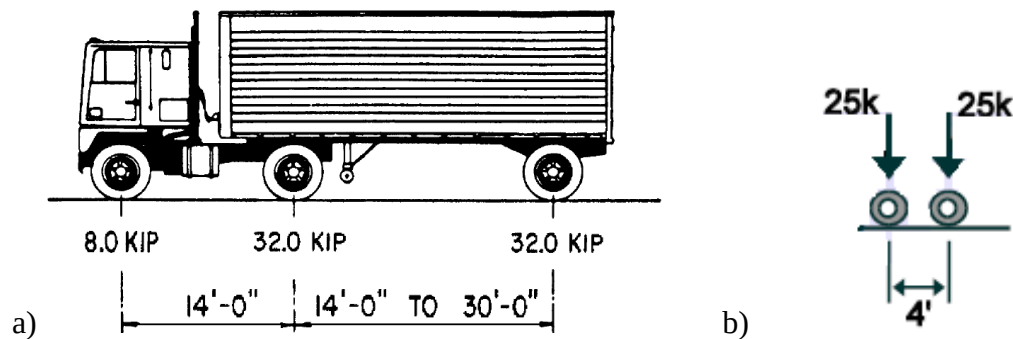


Figure 3-7: a) AASHTO HL-93 Design Truck, (AASHTO, 2010)

b) AASHTO HL-93 Design Tandem.

In all analyses of the bridges, the HL-93 truck with the lane load combination is used since the higher element forces and deformations occur under this load case with regards to bridge spans of interest in this study. Additionally, superimposed dead loads such as median, sidewalk and railing are taken into account. However, superimposed DLs were considered only for the two span continuous bridge which is a medium length bridge since the influence will be insignificant with the use of short length bridges.

Although the finite analysis program used in this study has the ability to define lanes and vehicle loads, this option is not used as a default. Rather, manually created truck load along with a uniform lane load is applied to the model. Influence analysis was made first to find out the location of the truck that causes the worst case condition at the abutment pile joint in terms of rotational deformation. More details are given about the loading cases in the analysis part of this report.

4. BRIDGE I

4.1 Introduction

Bridge I is a single span slab-on-girder bridge, it has a 45 ft. length of span and a width of 32 ft. as shown in Figure 4-1. The superstructure consists of precast girders and a concrete deck as shown in Figure 4-4. The girders are channel sections, and there is no spacing left between the girders when they are placed side by side along the width of the bridge. The deck is of 8 inch thick concrete. In terms of substructure, abutments are considered without wingwalls and steel HP piles were assumed for piling. The abutments are 5 by 3 ft. in dimensions for height and thickness, respectively, and they are assumed to have a rigid connection with both girders and piles. The pile length considered in the analysis of Bridge I is 30 and 40 ft. In terms of soil parameters, four consistencies of clay and three kinds of sand are considered. A summary of the bridge properties is given in Table 4-1.

Table 4-1 : Properties of Bridge I

Properties	Bridge I
Span length (ft)	45
Bridge width (ft)	32
Girder type	Channel
Girder width (ft)	4
Girder height (in)	28
Slab thickness (in)	8
Abutment height (ft)	5
Abutment thickness (ft)	3
Piles	HP Steel
Pile length (ft)	30, 40

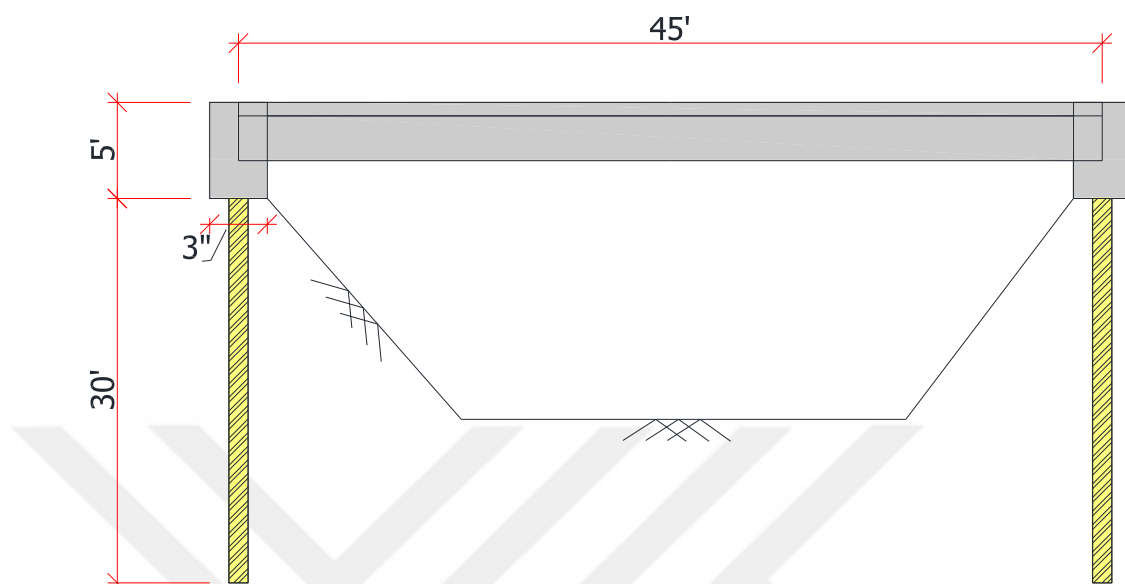


Figure 4-1 : Elevation view of Bridge I

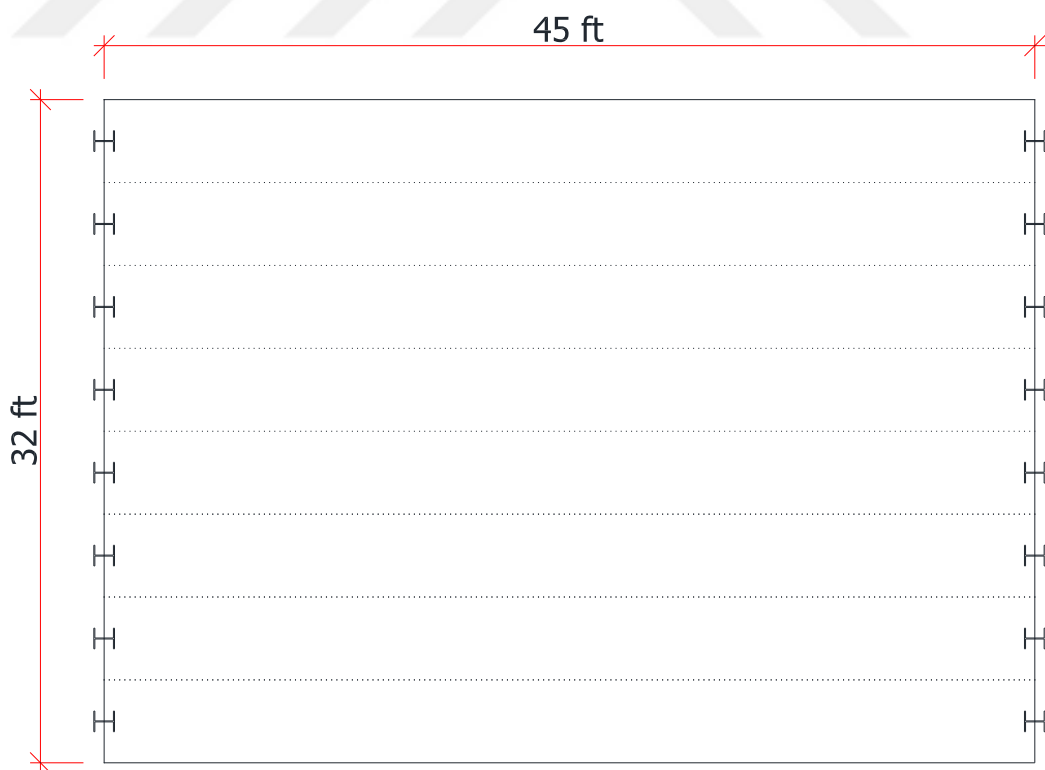


Figure 4-2 : Plan view of Bridge I

4.2 Analysis

The bridge was modeled using the structure analysis program Sap2000 V14. Two-dimensional modeling and analysis technique was adopted to model the bridge in the program. In the modeling, girders as well as piles and abutments are defined by using linear-elastic beam elements. By defining a rigid connection between each element, an indeterminate frame system was formed. Linear-elastic springs were used to model soil-structure interaction ignoring the non-linear behavior of soil. There are eight piles used in a single row, and 30 and 40 ft. length of pile is considered. Considering the highest diameter of a pile, which is the flange width of the HP14x89 section, the spacing limit is satisfactory considering the WisDOT (2011) requirement. The properties of the HP piles considered in this study are given in Table 4-6.

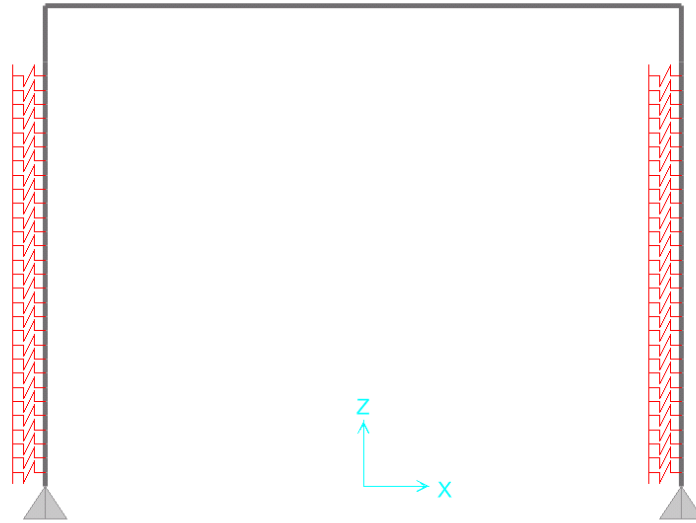


Figure 4-3 : 2-D Model of Bridge I in SAP2000

$$f_{cg} = 5000 \text{ psi}$$

$$f_{cd} = 4000 \text{ psi}$$

$$f_{ca} = 4000 \text{ psi}$$

$$E_d = 33000 * 0.15^{1.5} \sqrt{f_{cg}} = 4286 \text{ ksi} \quad 4-1$$

$$E_d = 33000 * 0.15^{1.5} \sqrt{f_{cd}} = 3834 \text{ ksi} \quad 4-2$$

$$E_a = 33000 * 0.15^{1.5} \sqrt{f_{ca}} = 3834 \text{ ksi} \quad 4-3$$

Where;

E_g = Modulus of elasticity of girder, ksi

E_d = Modulus of elasticity of deck, ksi

E_a = Modulus of elasticity of abutment, ksi

Table 4-2 : Material properties of Bridge I.

Component	f'_c, psi	E_c, ksi
Girders	5000	4286
Deck	4000	3834
Substructure	4000	3834

$$n = \frac{E_d}{E_g} = 0.89 \quad 4-4$$

The detailed cross-section profile of the precast channel is shown in Figure 4-5, and the properties of the section are given in Table 4-3.

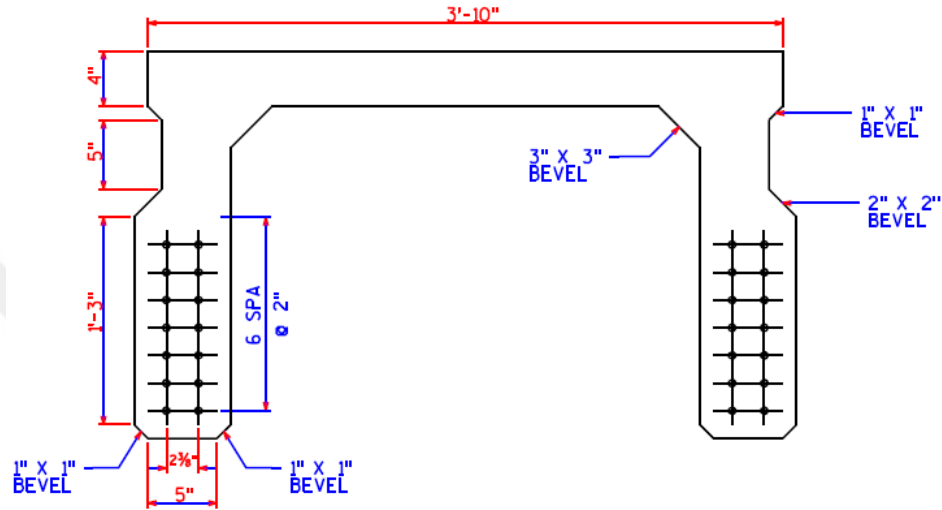


Figure 4-5 :Cross-sectional details of channel section.

Table 4-3 : 28" Channel section properties.

28" Channel Girder Properties		
Area	(in ²)	500
r^2		78.08
y_T	(in)	11.05
y_B	(in)	-16.95
I	(in ⁴)	39040
S_T	(in ³)	3533
S_B	(in ³)	-2303

The detailed cross-section of the composite section is shown in Figure 4-6. For convenience the values needed to determine composite properties are given in Table 4-4. In the table "Y" denotes the distance between the centroid of the component and the bottom,

and “ y_c ” denotes the centroid of the composite section. The centroid of the cross section is found by equation 4-5, and the gross moment of inertia is found by the 4-6.

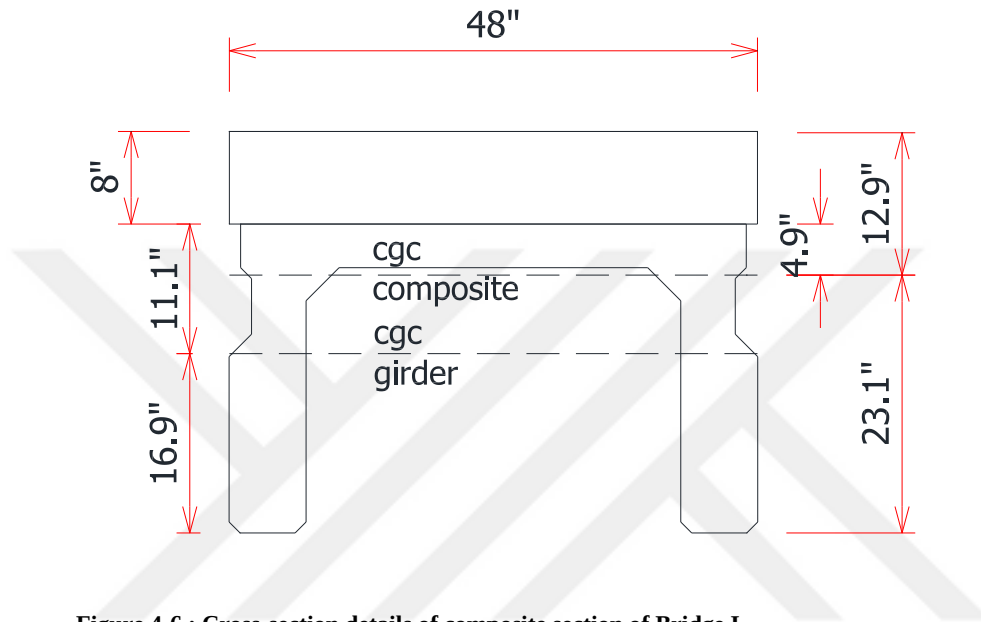


Figure 4-6 : Cross-section details of composite section of Bridge I.

Table 4-4 : Composite section parameters.

Component	Y (in)	A (in ²)	A*Y (in ³)	A*Y ² (in ⁴)	I (in ⁴)	I+A*Y ² (in ⁴)
Deck	32	343	10991	351702	1831.8	353534
Girder	16.95	500	8475	143651	39040	182691
Summation, Σ		843	19466			536225

$$y_c = \frac{\Sigma A * Y}{\Sigma A} \quad 4-5$$

$$y_c = 23.08 \text{ in}$$

$$I_{gross} = \Sigma(I + A * Y^2) - \Sigma(A * y_c^2) \quad 4-6$$

$$I_{gross} = 86988 \text{ in}^4$$

The composite properties are to be assigned as girder properties in the two-dimensional frame model in the program. Recalling that the superstructure consists of eight girders, the composite properties of a single girder is increased 8 times to get the lumped value to be able to define girder element properties in the program.

4.2.2 Soil Model

In order to model the soil-structure interaction in the program, the 30 ft. long beam elements for piles, for convenience, were meshed into 30 discrete elements having a 1 ft. length. Thus, linear-elastic springs could be assigned to the nodes of any two adjacent discrete elements to represent the soil response against piles. Each spring element will have unique p-y relations, depending on the depth, soil type and pile diameter. An illustration of the definition of springs along the length of the pile is shown in Figure 4-7.

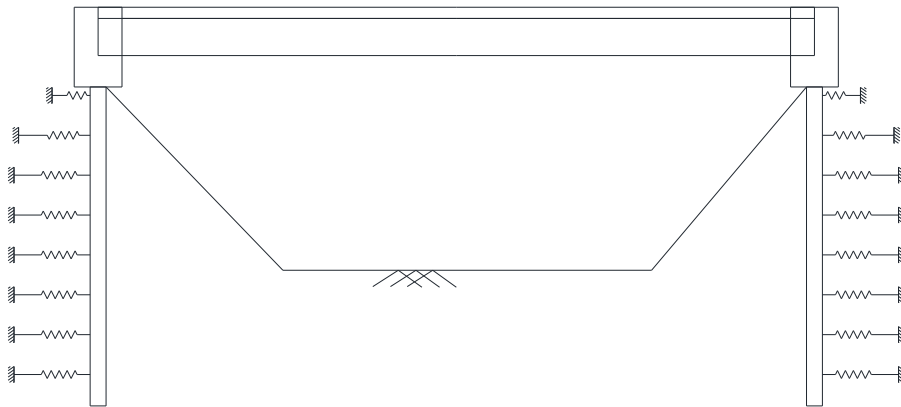


Figure 4-7 : Illustration of soil-springs on structural-model of a bridge.

Using the correlations in Section 3.3.2 spring stiffnesses for Sap2000 models were developed for the seven kinds of soil with varying sizes of piles. The soil properties considered in the analysis of all the bridge models as well as Bridge I are given in Table 4-5. The stiffnesses of soil springs are provided in APPENDIX A. Table 4-6 provides the pile sections and cross-sectional properties to use in the SAP2000 models.

Table 4-5 : Soil parameters used in the analysis.

Soil Type	γ (pcf)	c (psi)	ϕ (deg)	ϵ_{50}
Soft clay	120	2.6	-	0.02
Medium clay	120	5.2	-	0.01
Stiff clay	120	10.4	-	0.007
Very stiff clay	120	20.8	-	0.005
Loose Sand	120	-	30	-
Medium Sand	120	-	35	-
Dense Sand	120	-	40	-

Table 4-6 : Steel HP pile section properties.

Pile Section	Height (in)	Flange Width (in)	Area (in²)	Moment of Inertia (in⁴)
HP10x42	9.7	10.1	12.4	210
HP10x57	10	10.2	16.8	294
HP12x53	11.8	12	15.5	393
HP12x63	11.9	12.1	18.4	472
HP12x74	12.1	12.2	21.8	569
HP14x73	13.6	14.6	21.4	729
HP14x89	13.8	14.7	26.1	904

4.2.3 Load cases

Prior to load definition the number of lanes needs to be determined. The number of lanes can be defined such that the width of the bridge is divided by the width of a lane, which is 12 ft., and the result is used as a whole number for lanes. Therefore, the number of lanes is defined initially by equation 4-7. As a result, 2 lanes were assumed for the model of Bridge I, then live load definition can be made with the two lanes shown in Figure 4-8.

$$\# \text{ of lane} = \frac{32}{12} = 2.7 \rightarrow 2 \text{ Lane}$$

4-7

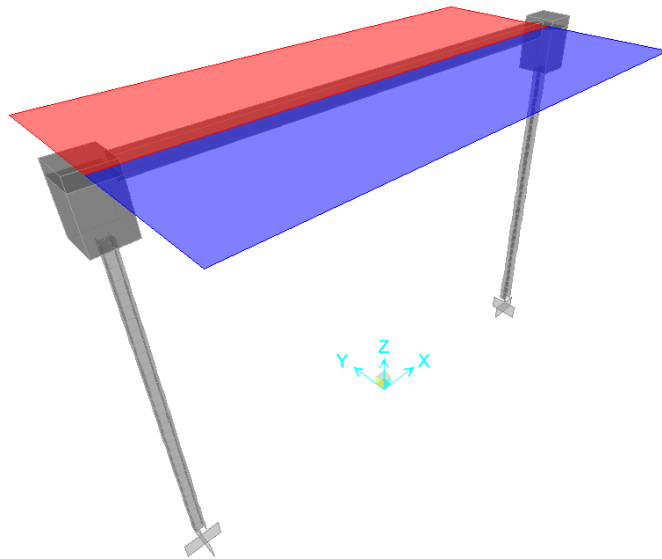


Figure 4-8 : An illustration of lane definition in SAP2000.

The AASHTO (2010) HL-93 Truck was used as a vehicle load with an axle spacing of 14 feet, since the larger effects occur with the lower axle spacing. In Sap2000, the HL-93 Truck includes lane load as a default option, and vehicle loads are considered as moving loads (CSI analysis Reference Manual, 2008). That is, every component of the structure is exposed to different stress levels and deformation depending on the position of the truck over the bridge lanes. By default vehicle loading, the program finds the worst case out of many. Making use of a default load case for Sap2000 is unnecessary for this study, because the interest is merely the analysis of a particular point which requires a static location of truck and analysis. In this sense, manual load cases were preferred.

Dicleli & Erhan (2010) proved that influence lines are independent of the soil type but its magnitude can be increased or reduced with regards to soil type. Accordingly, influence line analysis was performed to find out the location of the truck on the longitudinal length of the lane, which creates the largest influence on the rotational deformation of abutment-pile joint.

Figure 4-9 shows the influence lines for the left pile-abutment joint. The height of the vertical lines shows how much effect the truck has in that location with regards to the pile-abutment joint. Vertical line heights can vary with the different soil stiffness, and the point where the slope of the curve gets zero has the largest influence on the pile-abutment joint. With regards to influence line analysis, the most effective point appears at 18 ft. from the left end of the girder. Based on this location, the manual truck load will be applied along with the distributed lane load along the span.

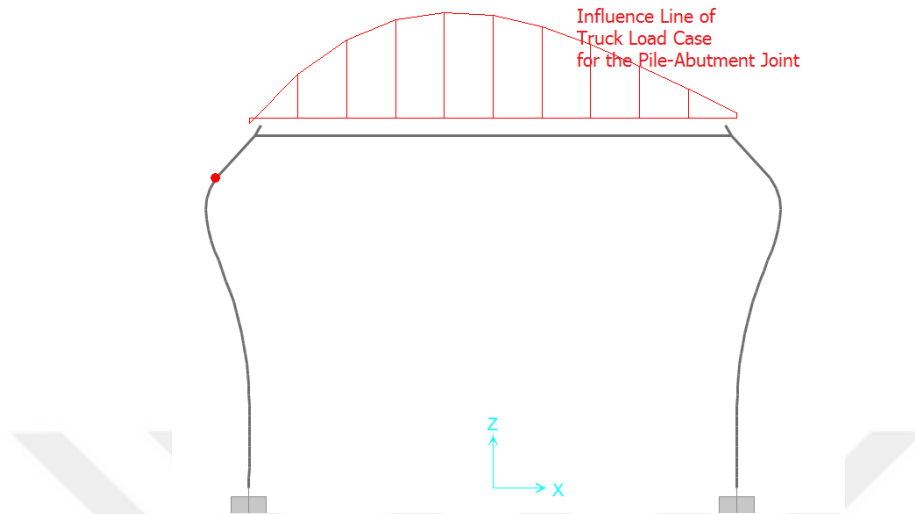


Figure 4-9 : Influence line display of Bridge I in SAP2000.

The truck load is assigned to the frame element in such a way that the center of gravity of the truck will match a distance of 18 ft. from the left end. While manual truck axle loads were applied as point loads, the lane load was applied as a distributed load along the length of the girder. Since there are two lanes assumed, the magnitude of manual truck loading as well as lane load was increased by a factor of two to create the worst case possible as far as the abutment-pile joint is considered. No superimposed DL is considered since it will be insignificant compared to other loads. The illustration of the load cases is shown in Figure 4-11.

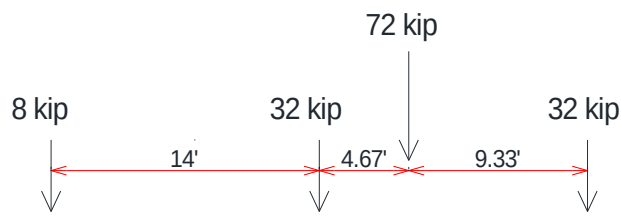


Figure 4-10: AASTHO HL-93 Design Truck axle loads.

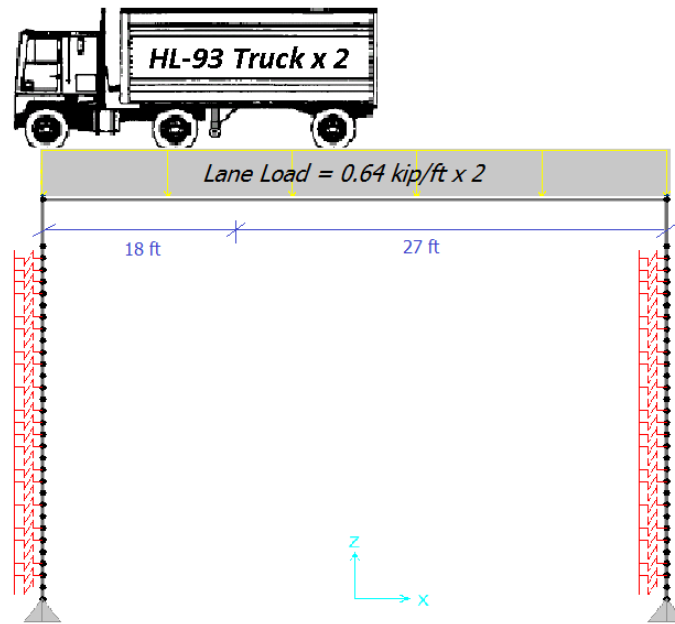


Figure 4-11 : Illustration of load definition for the frame model of Bridge I.

4.3 Analysis Results

The typical deformed shape and moment diagram of Bridge I, from the SAP2000 analysis, are shown in Figure 4-12 and Figure 4-13 with an amplified scale. The results are based on the truck location where the center of gravity of the truck is placed 18 feet from the left end of the girder. This case is not to create maximum moment, rather, to create the maximum rotation at the abutment-pile junction on the left side.

Although, 30 and 40 ft. pile lengths are considered in the analysis, it was observed based on the data that the results of the two cases are the same. Therefore, only one resultant case in terms of pile length will be given, however, results are valid both for 30 and 40 ft pile cases. Total number of cases considered in the analysis of Bridge I is 49 with the use of seven different sizes of pile having 30 ft. length, and seven kinds of soil.

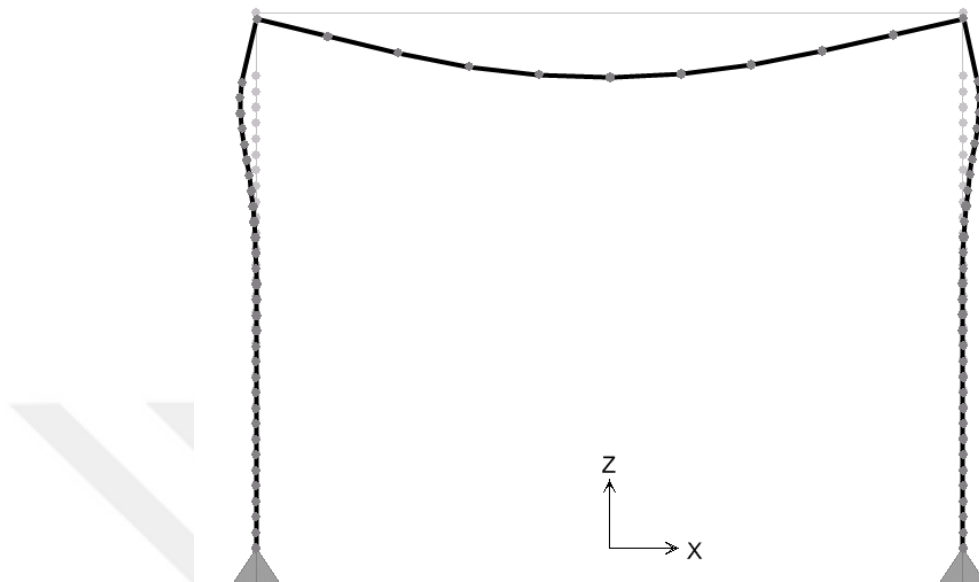


Figure 4-12 : Typical deformed shape of the frame model of Bridge I in SAP2000.

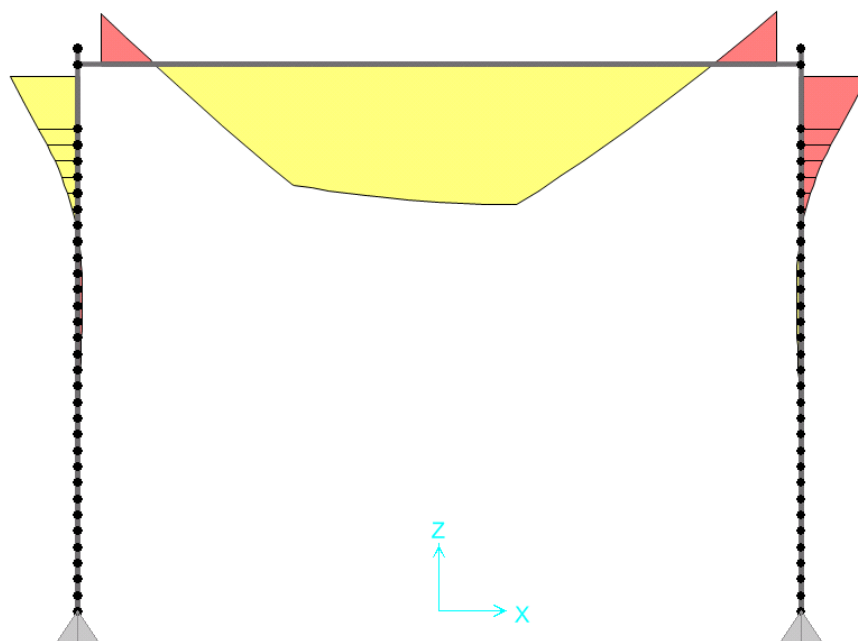


Figure 4-13 : Typical moment diagram of the frame model of Bridge I.

4.3.1 Moment Reduction

Analyses were made to measure the superstructure moment reduction in bridges with integral abutments versus the conventional type where the superstructure works as a simple supported beam. In order to see the moment reduction capacity of integral abutments for a range of pile sizes and soil types, comparisons were made after obtaining the girder moments for each case. With seven different sizes of piles and seven kinds of soil considered, results for forty nine cases were obtained.

In simply supported bridges, the moment forces as well other parameters become independent of the soil type. The forces can be obtained by static equilibrium regardless of the flexural stiffness of the beams. Therefore, the moment force becomes a constant value for the simply supported case of the superstructure since only one load case considered. The loading definition must stay the same in order to make a comparison between an integral abutment 2-d frame model and the simply supported beam model. In this sense, the manually applied truck loading was used in the same location and type for each analysis. The results were then gathered in Table 4-7.

With regards to the data obtained with the use of lightest pile, which is HP10x42, at least 13 percent of moment reduction capacity is achieved and this value increases as high as 14 percent with the use of HP14x89 piles in soft clay. In the case of medium clay use as a foundation soil, the reduction capacities are 18 and 31 percent for the range of piling between HP10x42 and HP14x89. In terms of stiff clay soil, the moment reduction capacity ranges between 26 percent and 39 percent as lowest and highest level, respectively. For the very stiff clay considered, 33 percent moment reduction capacity can be achieved, and this goes to as

high as 45 percent with the use of the highest stiffness for piling and foundation soil. The reduction capacities of the cases are shown as a chart in Figure 4-14.

Table 4-7: Superstructure moment reductions of Bridge I models relative to clay soil.

Pile Section	Regular Bridge Moment	Soft Clay		Medium Clay		Stiff Clay		Very Stiff Clay	
		Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %
HP10X42	1331	1160	13%	1089	18%	989	26%	887	33%
HP10X57	1331	1130	15%	1052	21%	948	29%	847	36%
HP12X53	1331	1104	17%	1024	23%	921	31%	823	38%
HP12X63	1331	1085	18%	1002	25%	898	33%	801	40%
HP12X74	1331	1064	20%	978	27%	873	34%	778	42%
HP14X73	1331	1038	22%	951	29%	850	36%	761	43%
HP14X89	1331	1012	24%	921	31%	812	39%	735	45%

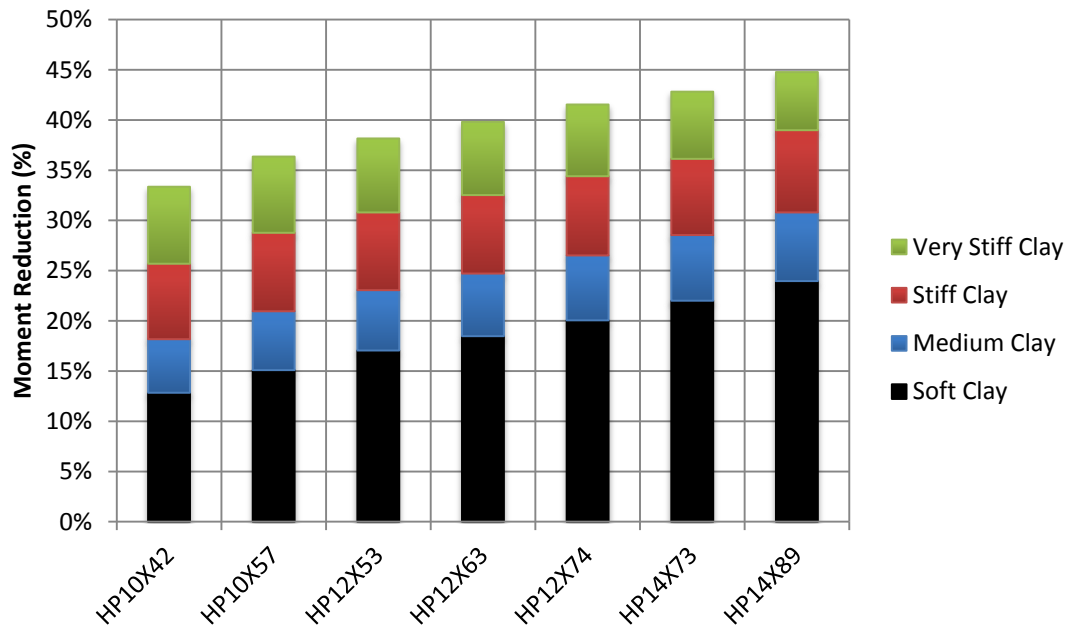


Figure 4-14 : Achieved superstructure moment reduction relative to clay foundation soil.

The data obtained for the sand soils are given in Table 4-8. The same trend takes form as in the previous cases. With the use of loose sand as a foundation soil, 15 percent moment reduction capacity can be achieved as the least for the lightest pile section considered and 28 percent as the highest for the largest section considered in the analyses. For foundation with medium sand the ranges of reduction capacities are 20 and 34 percent, as lowest and highest respectively. In terms of dens sand considered, 25 percent moment reduction capacity is achieved and this capacity rises to 40 percent with the use of largest section. The illustration of the moment reduction capacities for cohesionless soil is shown in Figure 4-15.

Table 4-8 : Superstructure moment reductions of Bridge I models relative to sand soil.

Pile Section	Regular Bridge Moment	Loose Sand		Medium Sand		Dense Sand	
		Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %
HP10X42	1331	1136	15%	1064	20%	995	25%
HP10X57	1331	1100	17%	1023	23%	950	29%
HP12X53	1331	1067	20%	988	26%	916	31%
HP12X63	1331	1044	22%	963	28%	890	33%
HP12X74	1331	1019	23%	936	30%	864	35%
HP14X73	1331	986	26%	904	32%	833	37%
HP14X89	1331	954	28%	872	34%	803	40%

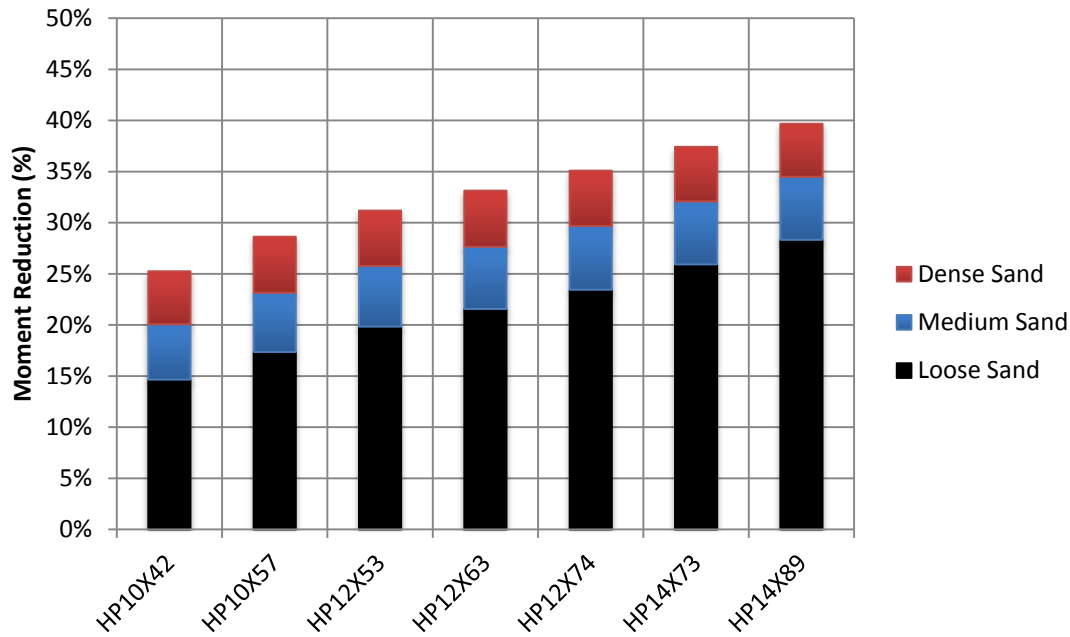


Figure 4-15 : Achieved superstructure moment reduction relative to sand foundation soil.

4.3.2 Moment Reduction with Backfill Effect

In terms of the abutment-soil interaction, structural analysis was made to see the influence of backfill on the internal moments of the clear span. In this analysis, soft clay is considered as a foundation soil along with the seven different pile sections for piling, and medium sand assumed as backfill soil. Using the method described in 3.4.2, abutment spring stiffnesses were developed as given in Table 4-9. These springs were attached along the length of abutments of Bridge I which is discretized to five 1 ft. elements. Because of the connection of girders and the abutments, no springs were used for the top 2 ft. length since these joints move inward under loading which creates an active soil pressure. The illustration of the spring definition is shown in Figure 4-16.

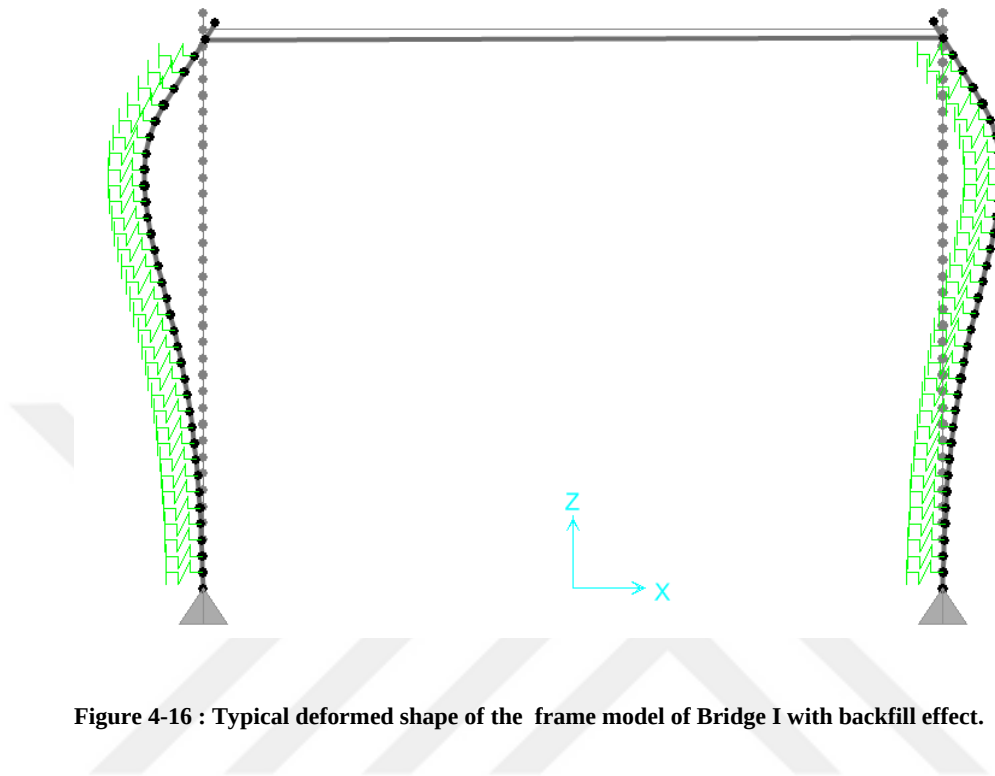


Figure 4-16 : Typical deformed shape of the frame model of Bridge I with backfill effect.

Table 4-9 : Abutment springs.

γ (kcf)	Depth z (ft)	Abut. Heigh (ft)	Passive pressure K_p (ksf)	At-rest Eart Press.	Uniform Load (ksf)	Abut. Width- w (ft)	spacing (ft)	Spring stiffness, K (kip/ft)
0.12	1	5	1.125	0.4	17.4	45	1	783
0.12	2	5	1.125	0.4	34.8	45	1	1566
0.12	3	5	1.125	0.4	52.2	45	1	2349
0.12	4	5	1.125	0.4	69.6	45	1	3132
0.12	5	5	1.125	0.4	87	45	1	3915

After getting the pile profile that includes backfill effect in the analysis, conclusions can be made as follows: Inclusion of the backfill stiffness into the model results in a moment reduction both in the span and at the pile head. It also causes more restraint in terms of structure deformation. Compared to an analysis without backfill effect, pile head rotation and lateral deflection are reduced because of the stiffer states of the abutments. However, in

contrast to decrease in moment forces, shear forces increase at the pile head. The data related to pile head condition are given in Table 4-10.

Table 4-10 : Pile head deformation, forces and computed stiffness values with backfill effect.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (k/ft)
HP10x42	0.00071	-0.0331	97	-26.5	136793	9633
HP10x57	0.00068	-0.0318	118	-28.0	172984	10572
HP12x53	0.00066	-0.0307	138	-28.9	208650	11308
HP12x63	0.00064	-0.0298	152	-29.7	237011	11952
HP12x74	0.00062	-0.0289	168	-30.5	270026	12663
HP14x73	0.00060	-0.0278	190	-31.1	316133	13459
HP14x89	0.00058	-0.0266	212	-32.0	366868	14431

The clear span moment forces that include the backfill presence were obtained and provided in Table 4-11 along with the values which do not consider the backfill effect. As a result it is observed that with the incorporation of abutment-backfill interaction into the model, the span moments of the superstructure are reduced 4.5 percent in average with all the pile sections considered. This is due to the stiffening effect of backfill over abutments. In brief, girder moments are reduced while abutment moments are increased.

Table 4-11 : Moment comparison with and without backfill effect.

Pile Section	Moments w/o Backfill (ft-kip)	Moments w/Backfill (ft-kip)	Moment Reduction
HP10X42	1160	1104	-5.1%
HP10X57	1130	1078	-4.8%
HP12X53	1104	1056	-4.5%
HP12X63	1085	1040	-4.3%
HP12X74	1064	1022	-4.1%
HP14X73	1038	999	-3.9%
HP14X89	1012	976	-3.7%

As a result, it can be concluded that neglecting the abutment-soil interaction is a conservative approach as far as pile head deformation and bridge span moments are concerned (span moments overestimated by 3.5-5%). The influence of backfill is small and conservative in terms of the stiffness of rotational spring which is approximated in the preceding chapter. But, there is an increase in terms of the stiffness of linear spring which was also approximated to represent the lateral resistance of piles. Having observed the influence of abutment backfill, the abutment-soil interaction is not considered in the analysis of other two bridges as a choice. This will result in an overestimate of bridge span moments and pile head deformation.

4.3.3 Abutment-pile joint

In order to approximate the pile-abutment behavior as a rotational and linear spring for a given parameters, pile head displacements were obtained along with the moment and shear forces. Pile profiles in terms of lateral deflection, rotation and moment are provided in APPENDIX B. Additionally, span moments of the integral bridge and the moments as if the bridge is conventional were obtained to make a comparison. In order to develop rotational springs that could be used to represent the pile in a simple model, the obtained pile head moment forces were divided by pile head rotation, as in the equation 4-7, and to get linear springs, pile head shear forces were divided by the lateral deflection of the pile head, as in the equation 4-9. The analysis results as well as stiffness values are given in the seven tables below with regards to soil types. In the tables, positive rotation is clockwise, and negative deflection means that pile deflects outward of the bridge span. Negative shear values show the soil (behind the piles) forces against the pile.

$$K_{\theta} = \frac{M}{\theta} \quad 4-8$$

$$K_{\delta} = \frac{V}{\delta} \quad 4-9$$

For bridges that have similar properties to Bridge I (Table 4-19), the pile foundation can be represented by a single rotational and linear spring as shown in Figure 4-17, where the values can be based on the data obtained by the result of this parametric study and given in the Table 4-12 to Table 4-18.

Table 4-12 : Pile head displacements and forces in soft clay, and computed stiffnesses for Bridge I abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.00077	-0.0352	104	-16.9	135770	5762
HP10x57	0.00074	-0.0337	127	-18.9	172554	6721
HP12x53	0.00071	-0.0337	147	-20.1	207042	7182
HP12x63	0.00069	-0.0324	163	-21.2	236232	7878
HP12x74	0.00067	-0.0313	179	-22.3	267564	8559
HP14x73	0.00064	-0.0289	202	-22.3	314642	9280
HP14x89	0.00061	-0.0275	224	-24.5	364821	10703

Table 4-13 : Pile head displacements and forces in medium clay, and computed stiffnesses for Bridge I abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.00069	-0.0318	131	-28.2	189033	10630
HP10x57	0.00066	-0.0299	157	-30.9	238965	12373
HP12x53	0.00063	-0.0285	179	-32.4	285486	13644
HP12x63	0.00060	-0.0273	196	-33.8	324503	14845
HP12x74	0.00058	-0.0261	215	-35.2	370690	16189
HP14x73	0.00055	-0.0247	238	-36.3	431159	17606
HP14x89	0.00052	-0.0232	262	-37.8	502879	19522

Table 4-14 : Pile head displacements and forces in stiff clay, and computed stiffnesses for Bridge I abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.00059	-0.0269	160	-46.1	271186	20596
HP10x57	0.00055	-0.0248	187	-49.6	340619	24009
HP12x53	0.00052	-0.0234	209	-50.8	401152	26013
HP12x63	0.00050	-0.0222	226	-52.4	454728	28310
HP12x74	0.00047	-0.0210	245	-54.0	519068	30869
HP14x73	0.00045	-0.0198	267	-54.4	595982	32970
HP14x89	0.00041	-0.0179	291	-58.0	713235	38964

Table 4-15: Pile head displacements and forces in very stiff clay, and computed stiffnesses for Bridge I abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.00049	-0.0218	178	-66.9	366255	36846
HP10x57	0.00045	-0.0197	205	-70.6	460674	42952
HP12x53	0.00042	-0.0185	226	-71.3	536817	46132
HP12x63	0.00040	-0.0174	241	-72.9	605528	50235
HP12x74	0.00038	-0.0163	258	-74.3	688000	54774
HP14x73	0.00036	-0.0153	280	-73.8	786517	57725
HP14x89	0.00033	-0.0141	300	-75.1	909091	64034

Table 4-16 : Pile head displacements and forces in loose sand, and computed stiffnesses for Bridge I abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.00074	-0.0340	118	-19.6	159030	6895
HP10x57	0.00071	-0.0321	144	-22.2	204255	8287
HP12x53	0.00067	-0.0305	168	-24.3	250373	9583
HP12x63	0.00065	-0.0293	186	-25.9	287481	10607
HP12x74	0.00062	-0.0279	205	-27.4	329582	11768
HP14x73	0.00059	-0.0263	231	-29.2	393526	13357
HP14x89	0.00056	-0.0246	256	-31.0	461261	15094

Table 4-17: Pile head displacements and forces in medium sand, and computed stiffnesses for Bridge I abutment

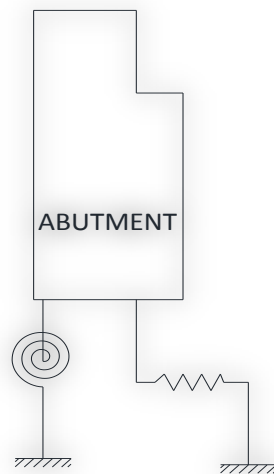
Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.00067	-0.0306	142	-31.4	212575	12339
HP10x57	0.00063	-0.0284	171	-34.8	273163	14701
HP12x53	0.00059	-0.0267	196	-37.3	332203	16781
HP12x63	0.00056	-0.0254	214	-39.1	379433	18502
HP12x74	0.00054	-0.0240	234	-40.9	435754	20447
HP14x73	0.00050	-0.0224	259	-42.8	514911	22969
HP14x89	0.00047	-0.0207	284	-44.7	604255	25855

Table 4-18 : Pile head displacements and forces in dense sand, and computed stiffnesses for Bridge I abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.00060	-0.0271	162	-44.0	271357	19453
HP10x57	0.00055	-0.0249	191	-48.0	346642	23152
HP12x53	0.00052	-0.0231	215	-50.5	416667	26205
HP12x63	0.00049	-0.0218	233	-52.5	476483	28882
HP12x74	0.00046	-0.0205	253	-54.4	547619	31906
HP14x73	0.00043	-0.0189	277	-56.1	642691	35588
HP14x89	0.00040	-0.0174	300	-58.0	751880	40068

Table 4-19 : Physical properties of Bridge I

Bridge Length	(ft)	45
Lumped Moment of Inertia of superstructure	(in ⁴)	695900
Lumped Area of superstructure	(in ²)	843
Abutment Height	(ft)	5
Abutment Thickness	(ft)	3

**Figure 4-17: Illustration of pile as a rotational and a linear spring.**

4.3.4 Depth of Inflection Point

In the design of pile foundations it is a common procedure that piles are considered as a cantilever beam having a fixed end somewhere at depth below ground surface. In the determination of the cantilever beam length the distance between abutment and inflection point is considered. Commonly, piles are assumed to get fixed about ten times the diameter of pile section from the ground surface. In this study the pile end fixity related to all cases were obtained based on the location of inflection points and provided in the following tables and figures.

Table 4-20 : Depths of inflection points with regards to seven pile sizes and seven soil types.

Depth of Inflection Point , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	8.92	6.75	4.93	3.8	7.52	5.86	4.79
HP10x57	9.79	7.37	5.41	4.17	8.09	6.35	5.17
HP12x53	10.65	8.05	5.94	4.63	8.69	6.81	5.58
HP12x63	11.18	8.47	6.26	4.85	9.03	7.08	5.8
HP12x74	11.77	8.86	6.58	5.1	9.45	7.41	6.03
HP14x73	12.71	9.61	7.17	5.6	9.99	7.87	6.47
HP14x89	13.58	10.11	7.38	5.92	10.5	8.27	6.78

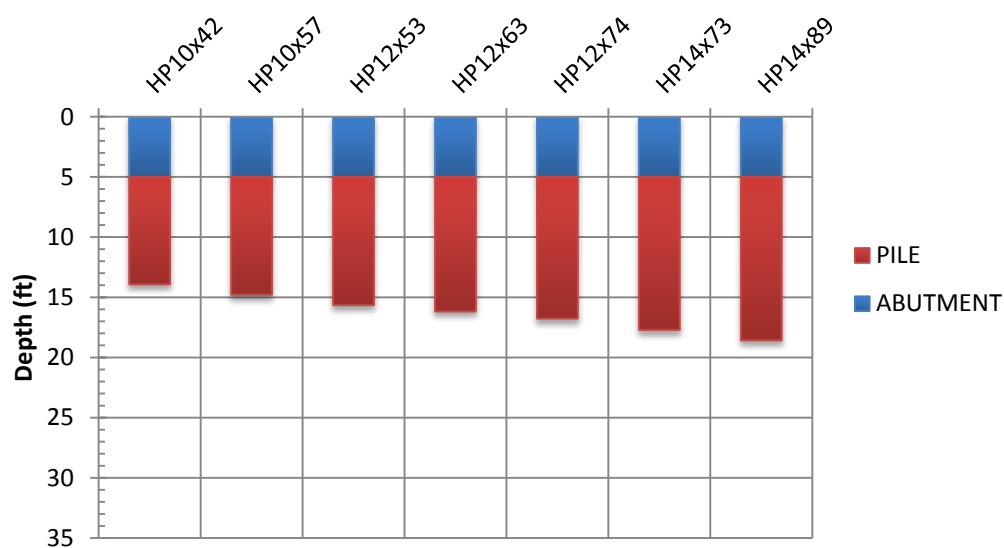


Figure 4-18 : Depth of inflection points of Bridge I piles in soft clay.

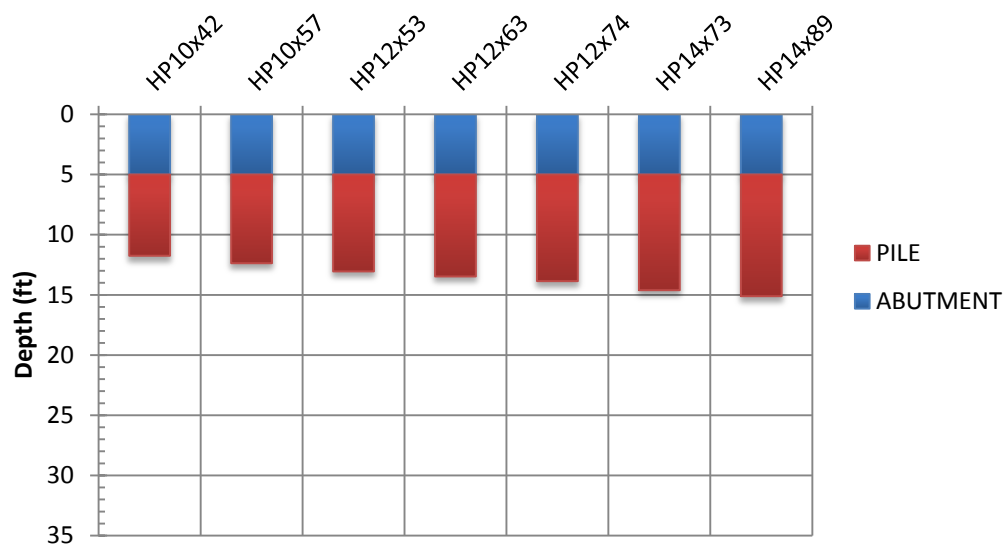


Figure 4-19 : Depth of inflection points of Bridge I piles in medium clay.

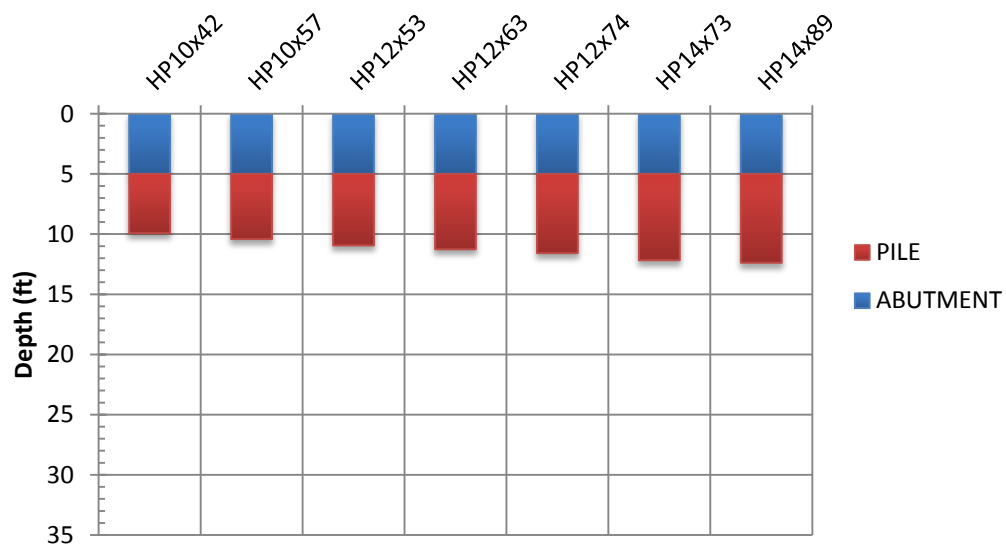


Figure 4-20 : Depth of inflection points of Bridge I piles in stiff clay.

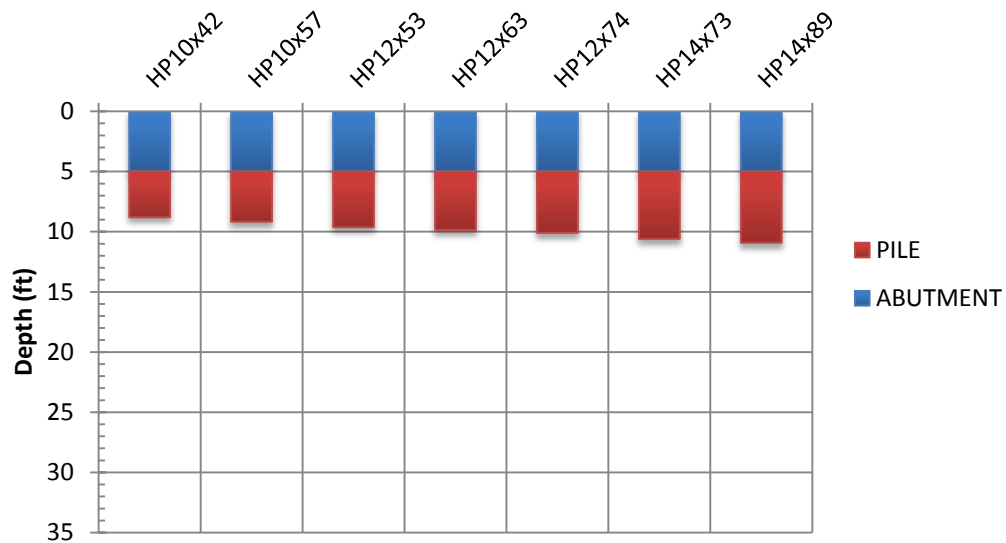


Figure 4-21 : Depth of inflection points of Bridge I piles in very-stiff clay.

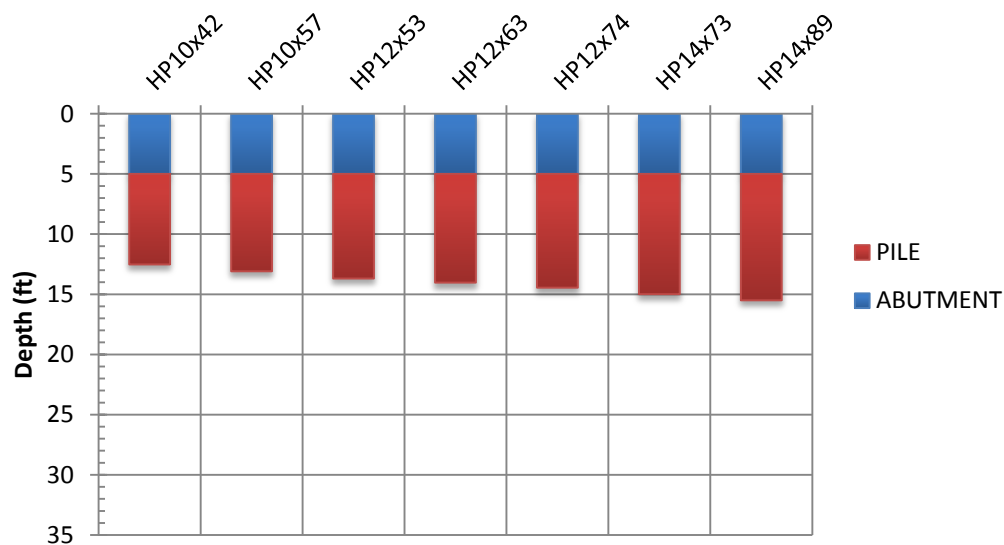


Figure 4-22 : Depth of inflection points of Bridge I piles in loose sand.

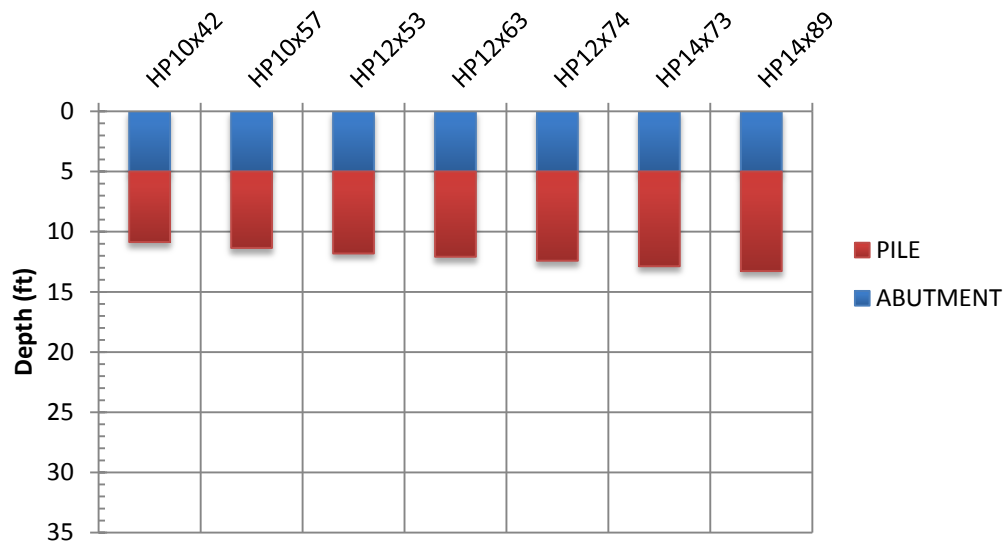


Figure 4-23 : Depth of inflection points of Bridge I piles in medium sand.

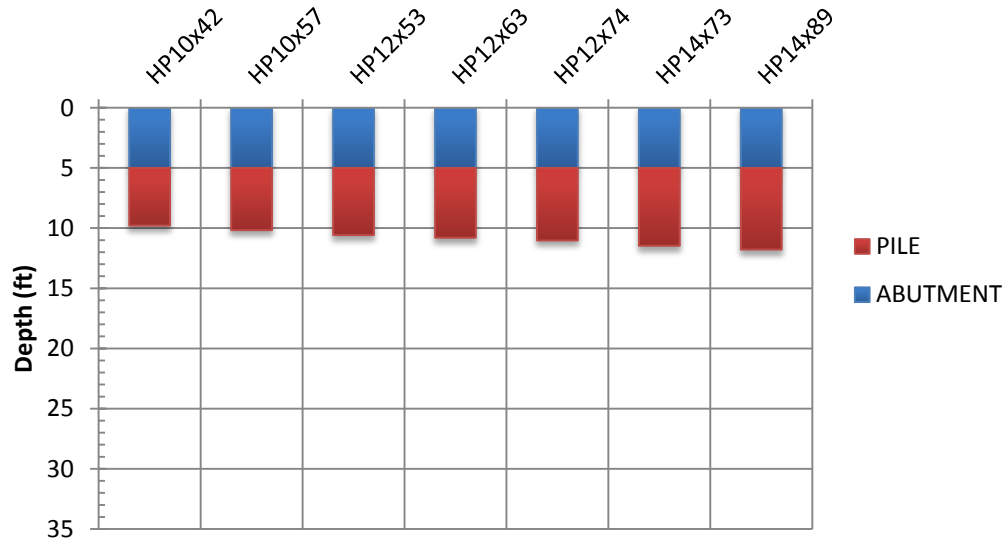


Figure 4-24 : Depth of inflection points of Bridge I piles in dense sand.

4.3.5 Depth of Zero Rotation

To get a good insight about the pile behavior, it is essential to observe the pile deformation as a whole. Based on the analysis it was observed that piles exhibit double curvatures under live loading. At some distance below the ground surface the pile rotation becomes zero and then starts to get reverse curvature. Typically another zero rotation state is achieved depending on the stiffnesses of soils and piles. Table 4-21 and Table 4-22 provide the depths of first and second point of zero rotation along the depth of the piles of Bridge I. Figure 4-25 to Figure 4-28 illustrate the increase and decrease of the point of zero rotation relative to the soil types and pile sections.

Table 4-21 : Depths of first zero rotation.

Point of Zero Rotation , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	3.4	2.4	1.7	1.2	2.8	2	1.4
HP10x57	3.8	2.7	1.9	1.4	3	2.3	1.8
HP12x53	4.2	3	2.1	1.6	3.4	2.5	2
HP12x63	4.5	3.2	2.3	1.7	3.5	2.7	2.1
HP12x74	4.8	3.4	2.4	1.8	3.7	2.8	2.2
HP14x73	5.3	3.8	2.7	2	4	3	2.4
HP14x89	5.7	4	2.8	2.2	4.2	3.2	2.6

Table 4-22 : Depths of second zero rotation.

Second Point of Zero Rotation, ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	30	25	20	18	26	22	19
HP10x57	32	26	21	19	27	23	20
HP12x53	35	28	23	20	28	24	21
HP12x63	35	29	23	20	29	25	22
HP12x74	35	29	24	21	30	25	22
HP14x73	35	30	26	22	31	27	23
HP14x89	35	31	26	23	32	28	24

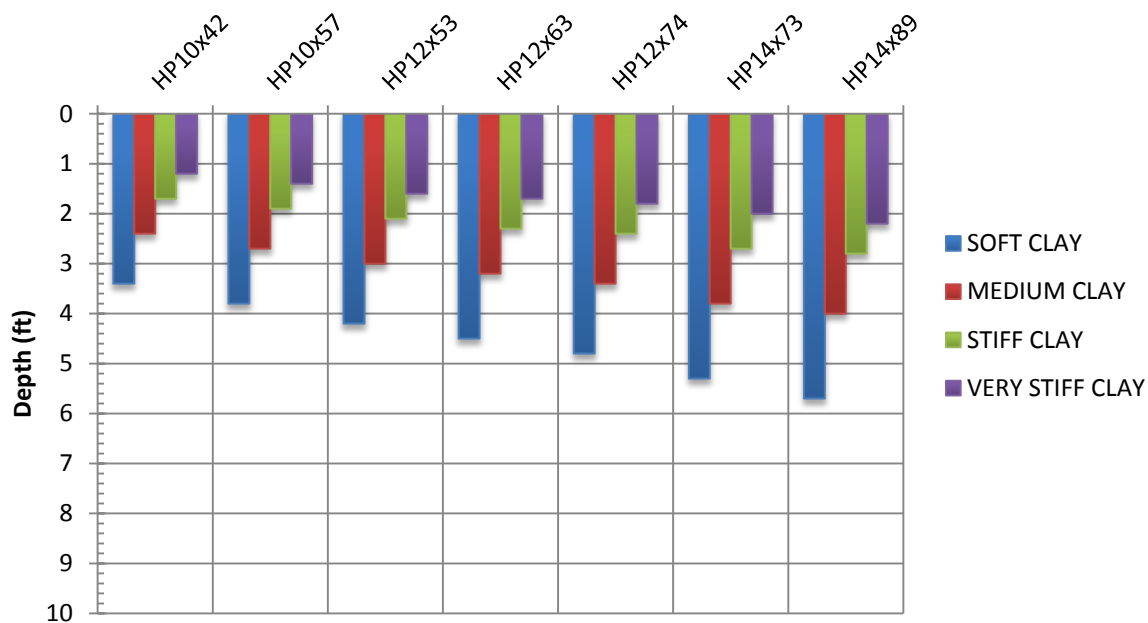


Figure 4-25 : Chart of the depths of first zero rotation in clay soil.

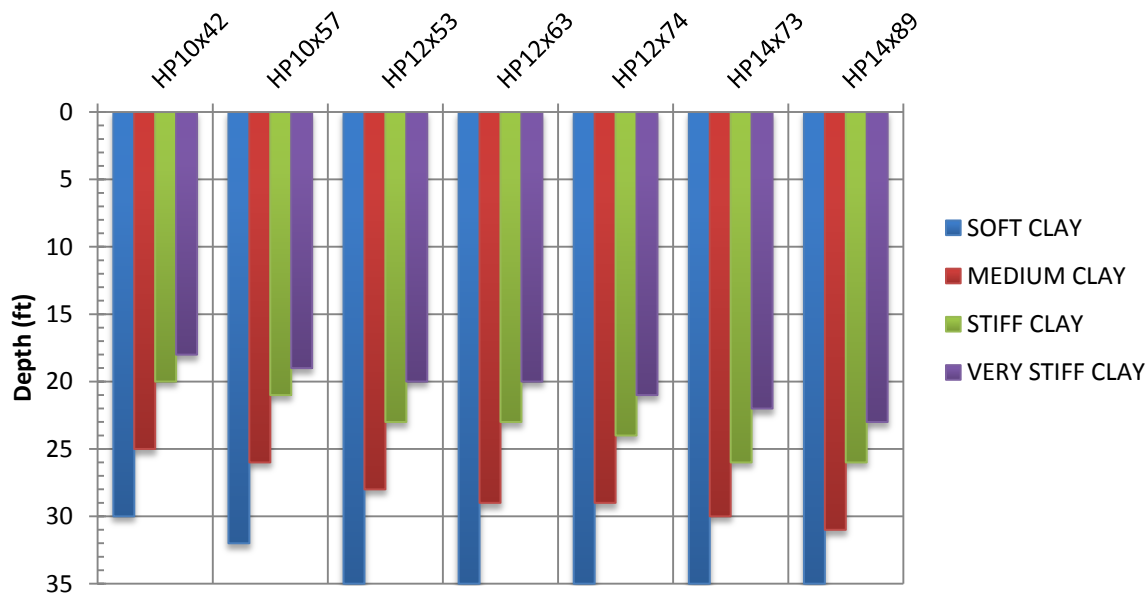


Figure 4-26 : Chart of the second zero rotation in clay soil.

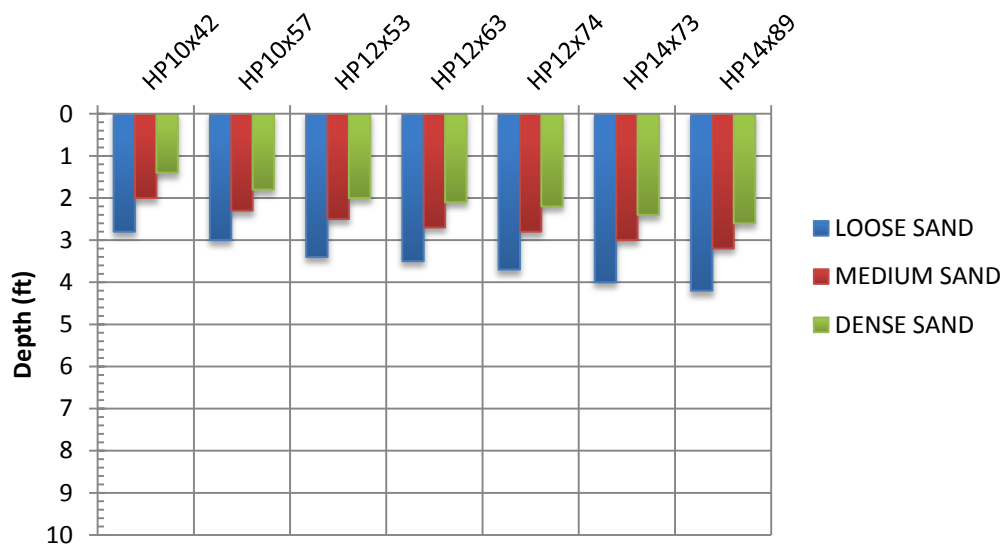


Figure 4-27 : Chart of the depths first zero rotation in sand soil.

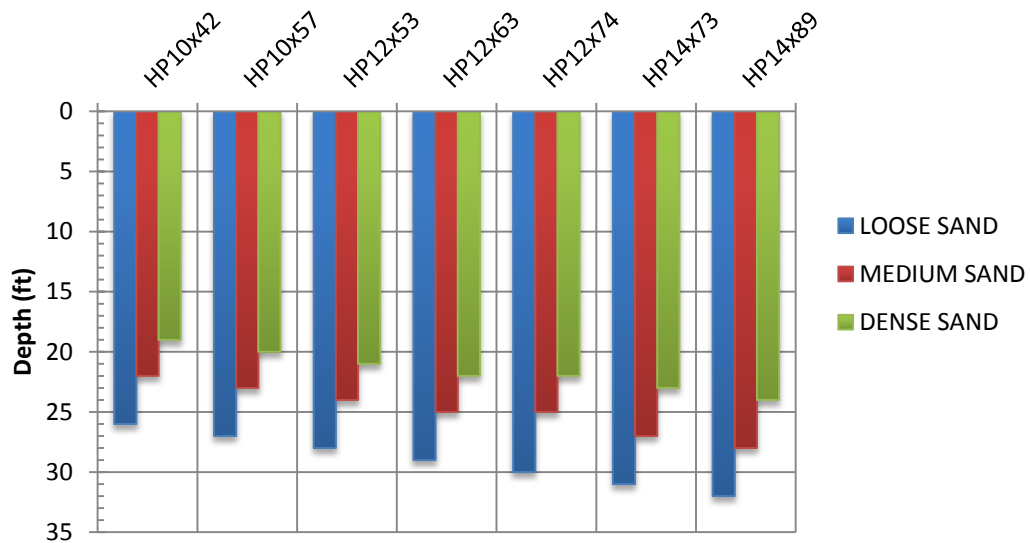


Figure 4-28 : Chart of the depths of second zero rotation in sand soil.

5. BRIDGE II

5.1 Introduction

Bridge II is a concrete-deck, prestressed girder bridge, it has a 244 ft. total length of span and a width of 59.25 ft. as shown in Figure 5-2 and Figure 5-3. The bridge is a continuous two-span bridge. The girders are Wisconsin precast-prestressed 72W” sections, and the center-to-center spacing between girders is 10.75 ft. The concrete-deck is 9 inch thick and there is a 2 inch haunch between the deck and the girders. In terms of substructure, abutments were considered without wingwalls, and HP steel sections were used for piling with their strong axis parallel to the longitudinal direction of the structure. Abutments are 10 by 3 ft. in dimensions for height and thickness, respectively, and the pile length assumed is 40 and 50 ft. in the analysis. An intermediate pier is located in the center of the bridge where two spans get connected, and it was assumed that girders were not integral with the pier. Table 5-1 provides the summary of the properties of Bridge II.

Table 5-1: Properties of Bridge II

Properties	Bridge II
Span Length (ft)	122
Bridge width (ft)	59.25
Girder type	Wisconsin 72W”
g-g spacing (ft)	10.75
Slab thickness (in)	9
Abutment height (ft)	10
Abutment thickness (ft)	3
Pile length (ft)	40, 50

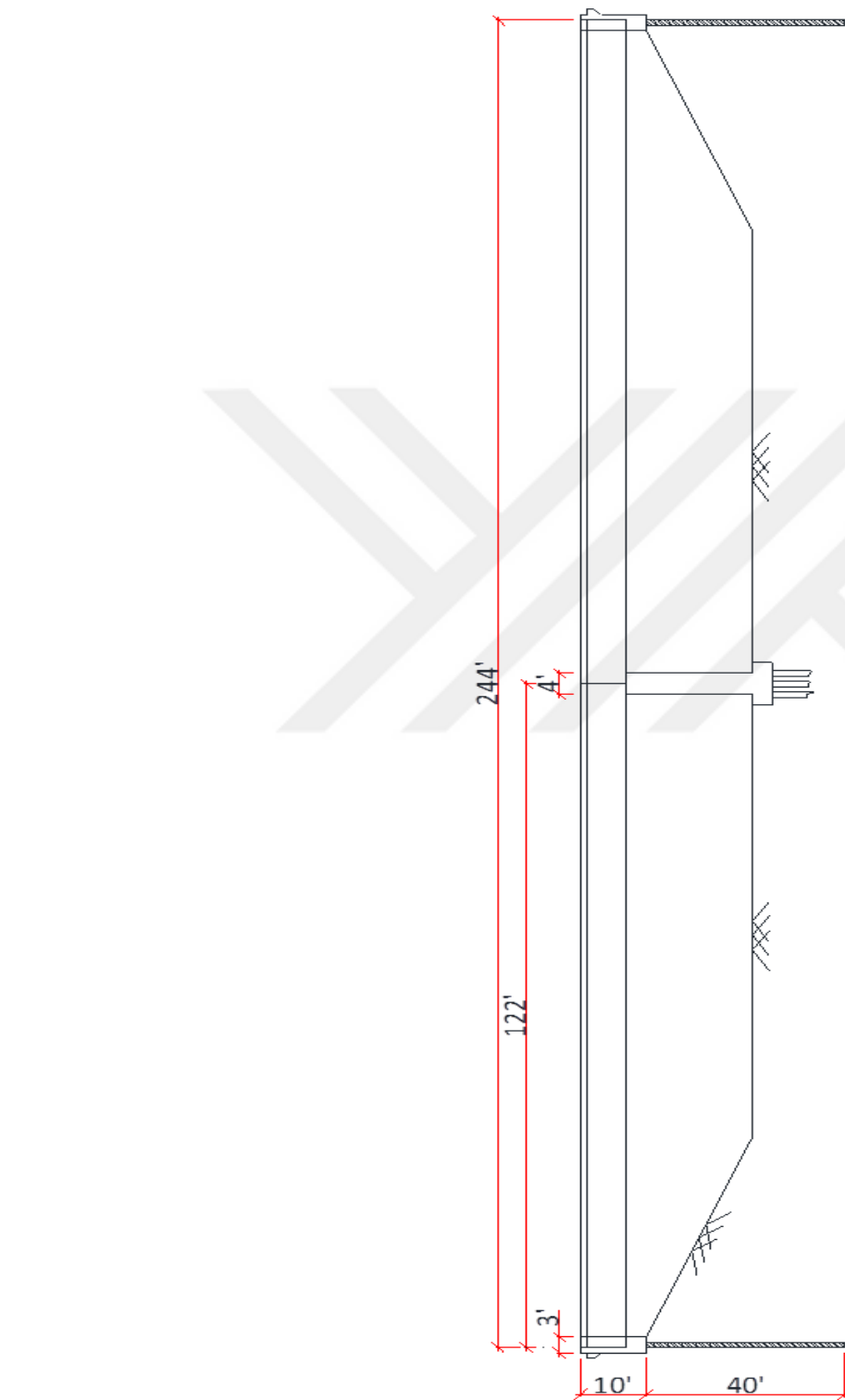


Figure 5-1 : Elevation view of Bridge II

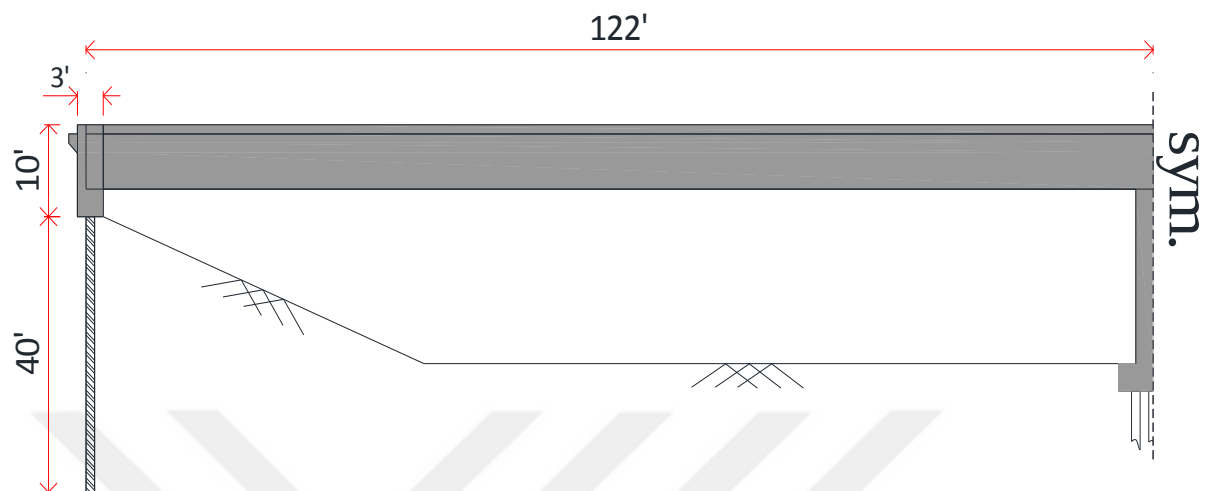


Figure 5-2 : Elevation view of the one span of Bridge II.



Figure 5-3 : Plan view of the one half of the Bridge II

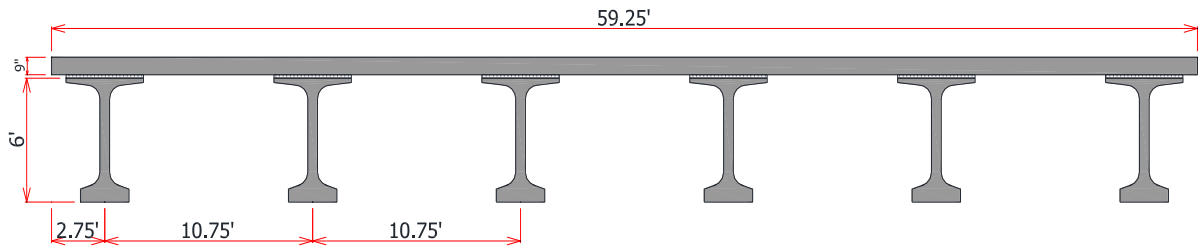


Figure 5-4 : Cross-section of the superstructure of Bridge II.

5.2 Analysis

The bridge was modeled using the structure analysis program Sap2000 V14. Two-dimensional modeling and analysis technique was adopted as for Bridge I. In the model, girders as well as other components of the bridge are modeled by using linear-elastic beams. By defining a rigid connection between each element, an indeterminate frame system was formed.

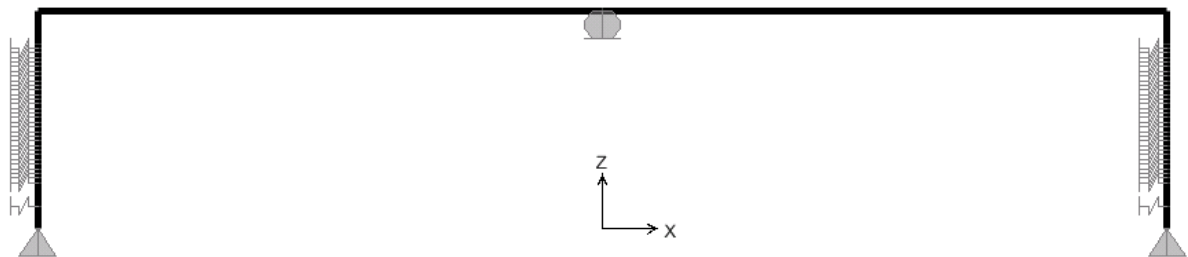


Figure 5-5 : 2-d frame model of Bridge II in SAP2000.

5.2.1 Transformed Section

In the definition of the girder elements in SAP2000, composite action is considered, therefore the flexural stiffness of the beam element in the model includes both the deck and girders properties. However, material properties vary between components, for this reason transformed area method was used to get the stiffness of the composite section. Therefore, the effective area of the deck was transformed into an equivalent area by using modular ratio n , to consider it as a girder material. The factor n is the ratio of Modulus of Elasticity of deck and girder. Although, the effective flange width can be taken as the tributary width perpendicular to the girder based on the AASTHO 2008, the width of the effective width was chosen as a function of thickness. For the flange width of external girders, overhang and g-g spacing were considered. Figure 5-2 shows the side-view of one half of the bridge because it is symmetric on the other side, and the cross section of the superstructure is given in Figure 5-4. The material properties of the superstructure and substructure are given in Table 5-2. The modular ratio is computed by equation 5-4, to transform the deck section into equivalent dimensions to use in a gross section calculation.

$$f_{cg} = 8000 \text{ psi}$$

$$f_{cd} = 4000 \text{ ksi}$$

$$f_{ca} = 3500 \text{ psi}$$

$$E_g = 1820 \sqrt{f_{cg}} = 5148 \text{ ksi} \quad 5-1$$

$$E_d = 33000 * 0.15^{1.5} \sqrt{\frac{f_{cd}}{4000}} = 3834 \text{ ksi} \quad 5-2$$

$$E_a = 33000 * 0.15^{1.5} \sqrt{\frac{f_{ca}}{4000}} = 3587 \text{ ksi} \quad 5-3$$

E_g = Modulus of elasticity of girder, ksi

E_d = Modulus of elasticity of deck, ksi

E_a = Modulus of elasticity of abutment, ksi

Table 5-2 : Material properties of Bridge II.

Component	f'_c , psi	E_c , ksi
Girders	8000	5148
Deck	4000	3834
Substructure	3500	3587

$$n = \frac{E_d}{E_g} = 0.74 \quad 5-4$$

The detailed cross-section with of the 72" W precast girder section is shown in Figure 5-6 and the properties of the section are given in Table 5-3.

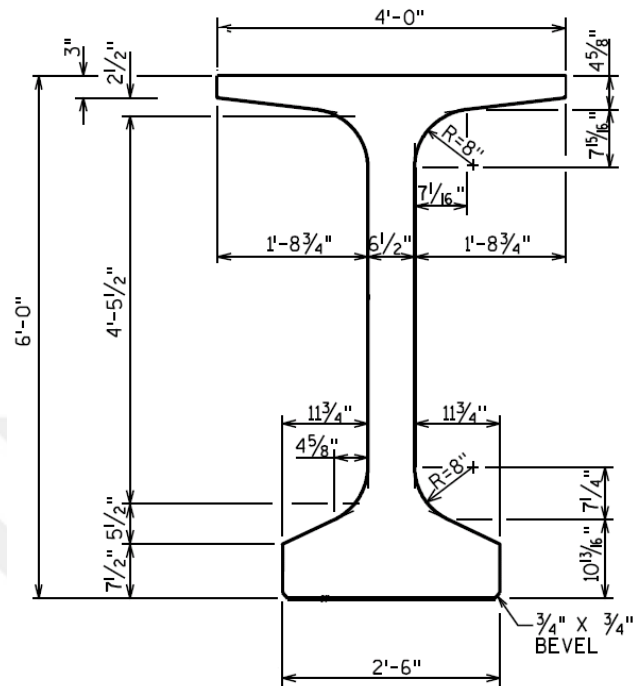


Figure 5-6 : Wisconsin precast-prestressed 72W'' girder cross-section details.

Table 5-3 : Wisconsin precast-prestressed 72W'' girder properties.

72W'' Girder Properties		
Area	(in ²)	915
r ²		717.5
y _T	(in)	37.13
y _B	(in)	-34.87
I	(in ⁴)	656426
S _T	(in ³)	17680
S _B	(in ³)	-18825

The details of the composite section for an interior girder are shown in Figure 5-7 which includes the values of centroid of the girder as well as composite section. For convenience the values needed to compute composite properties are given in Table 5-4, where “Y” denotes the distance between the centroid of the component and the bottom, and “ y_c ” denotes the centroid of the composite section. The centroid of the cross section is found in equation 5-4, and the gross moment of inertia is found by the 5-6.

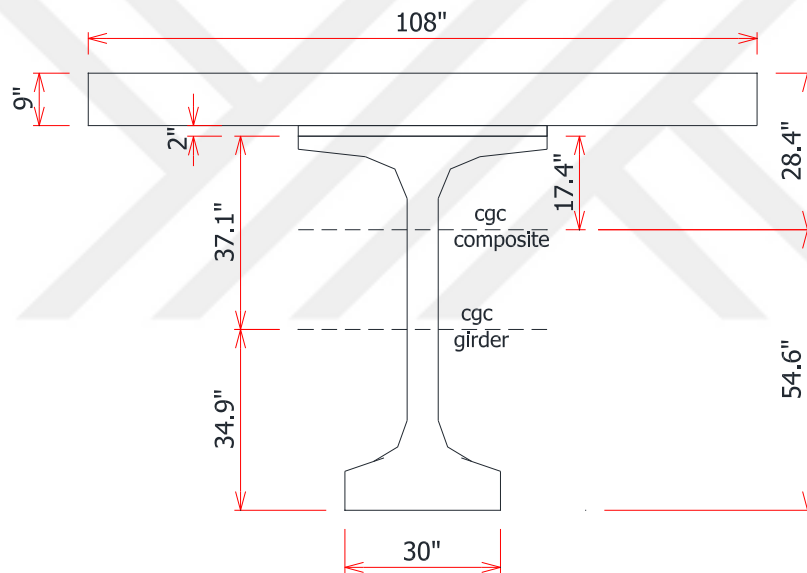


Figure 5-7 : Composite cross-section of the internal girders.

Table 5-4 : Composite properties of the internal girders.

Component	Y (in)	A (in ²)	A*Y (in ³)	A*Y ² (in ⁴)	I (in ⁴)	I+A*Y ² (in ⁴)
Deck	78.5	724	56834	4461469	4886	4466355
Girder	34.87	915	31906	1112564	656426	1768990
Summation, Σ		1639	88740			6235345

$$y_c = \frac{\sum A * Y}{\sum A} \quad 5-5$$

$$y_c = 54.14 \text{ in}$$

$$I_{gross} = \sum(I + A * Y^2) - \sum(A * y_c^2) \quad 5-6$$

$$I_{gross} = 1430710 \text{ in}^4$$

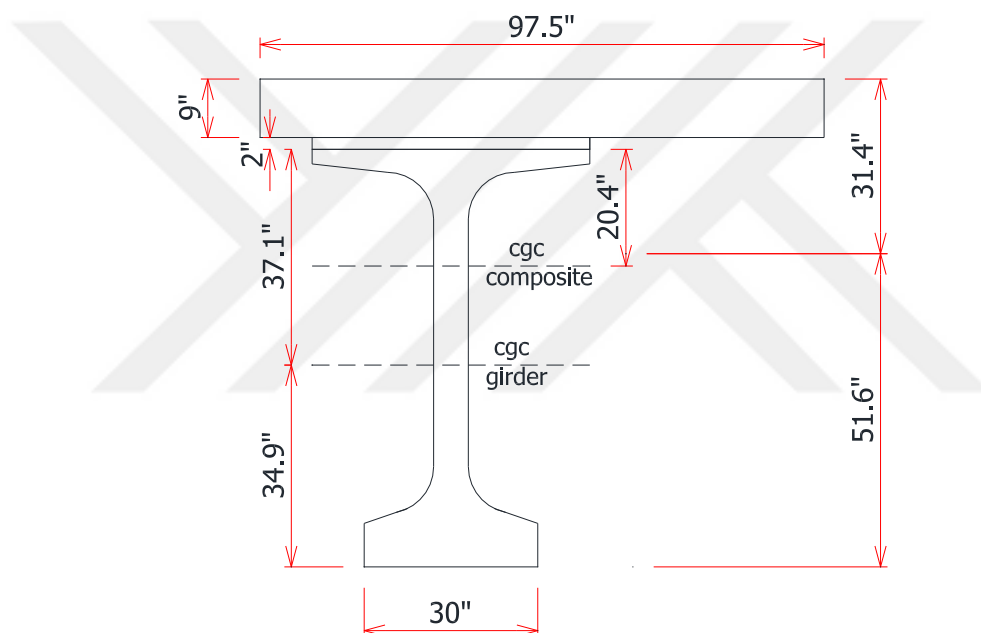


Figure 5-8 : Composite cross-section of the external girders.

Table 5-5 : Composite properties of the external girders.

Component	Y (in)	A (in ²)	A*Y (in ³)	A*Y ² (in ⁴)	I (in ⁴)	I+A*Y ² (in ⁴)
Deck	78.5	654	51339	4030112	4412	4034524
Girder	34.87	915	31906	1112564	656426	1768990
Summation, Σ		1569	83245			5803513

$$y_c = \frac{\sum A * Y}{\sum A} \quad 5-7$$

$$y_c = 53.06 \text{ in}$$

$$I_{gross} = \sum(I + A * Y^2) - \sum(A * y_c^2) \quad 5-8$$

$$I_{gross} = 1386854 \text{ in}^4$$

$$I_{total} = 2 * I_{exterior} + 4 * I_{interior} \quad 5-9$$

$$I_{total} = 8496550 \text{ in}^4$$

$$A_{total} = 2 * A_{interior} + 4 * A_{exterior} \quad 5-10$$

$$A_{total} = 9694 \text{ in}^2$$

The composite properties are to be assigned as girder properties in the two-dimensional frame model in the program. Because the superstructure consists of six girders, the properties of an internal and external girder are lumped to get a single value to be able to define girder element properties in the program.

5.2.2 Soil Model

The same geotechnical parameters and pile sections are employed as in the analysis of Bridge I which considers four different consistency of clay and three kinds of sand. Hence, using seven different piles in the analysis with a single soil type, there are 49 cases defined

with the seven kinds of soil. In order to model the soil-structure interaction in the program, the beam elements for piles were meshed into 30 discrete elements that have a 1 ft. length for the first 30 ft. from the ground surface, and the remaining 10 foot length was meshed into 2 discrete elements with a 5 ft. length to each. Thus, linear-elastic springs could be assigned to the nodes of any two adjacent discrete elements to represent the soil response against piles. Using the correlations given in Section 3.3.2, the stiffness values for each spring were computed for the seven kinds of soil considered and the seven sizes of piles. The stiffness of the springs are the same as in the other Bridges except that there are more springs used now since the pile length is longer.

5.2.3 Load Cases

To be able to define load cases, the number of lanes needs to be determined first. In the definition of number of lanes, the width of the bridge is divided by the width of the lane, which is 12 ft., and the result is used as a whole number which will be the number of lane for which the bridge will be designed. The number of lanes for Bridge II is found by the equation 5-11. As a result, 4 lanes are assumed on Bridge II. Figure 5-9 illustrates the lane definition of the program.

$$\# \text{ of lane} = \frac{59}{12} = 4.2 \rightarrow 4 \text{ Lane}$$

5-11

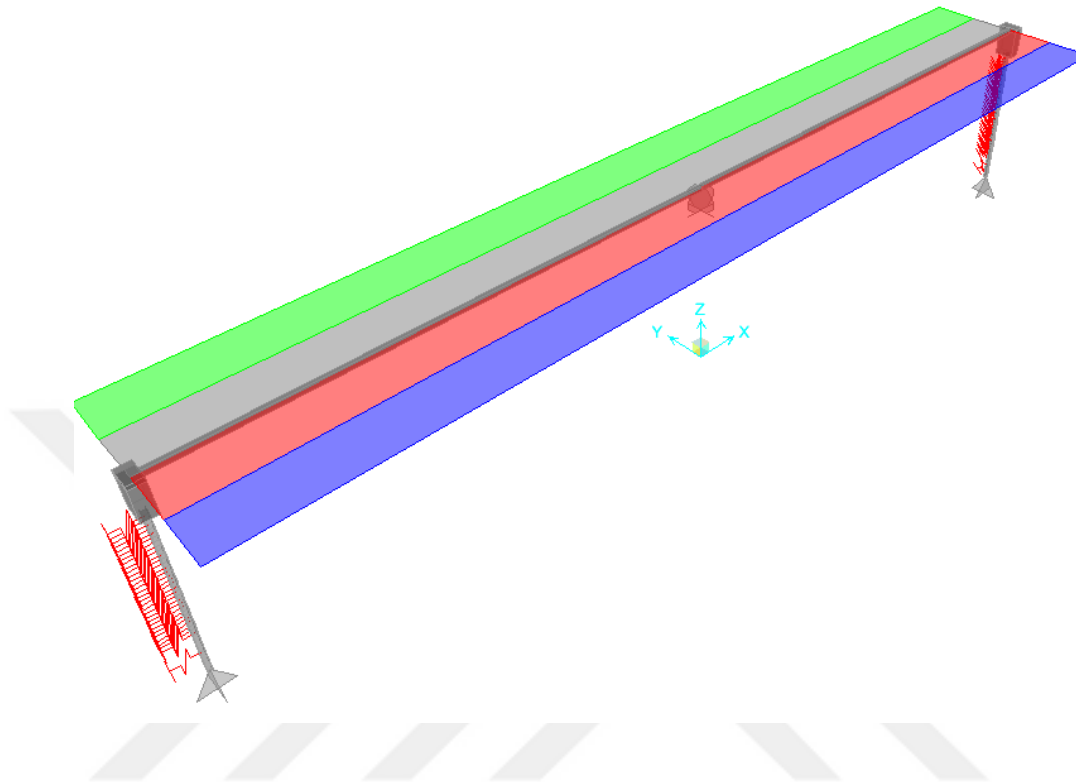


Figure 5-9 : An illustration of lane definition of Bridge II in SAP2000.

An AASHTO HL-93 Truck was used as the vehicle load with an axle spacing of 14 feet, since the larger effects occur with the lower axle spacing. In addition to the vehicle load, lane loads were also considered. Although the program has built-in vehicle and lane definition, manually created loads were used. The explanations were made in the analysis of Bridge I for the use of manual loadings. Therefore, influence line analysis was performed to find out the location of truck on the longitudinal length of the lane, which has the largest influence on the rotational deformation of abutment-pile joint. Based on the influence line analysis, the most effective point appears at 47 ft. from the left end of the girder. Therefore, the manual truck load will be applied with regards to this location. Figure 5-10 shows the influence lines for the pile-abutment joint under truck load.

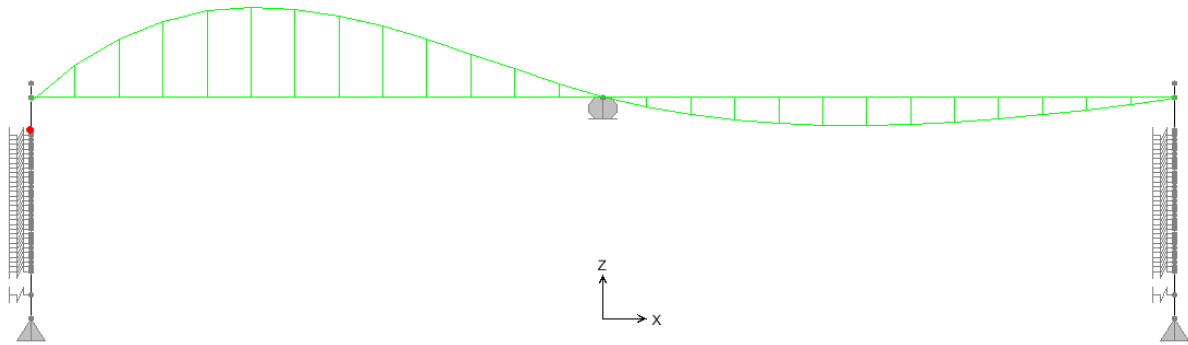


Figure 5-10 : Influence line display of Bridge II in SAP2000.

The truck load is assigned to the girder element in such a way that the center of gravity of the truck will match the location on the girder that is 47 ft. from the left end. The axle loads for a HL-93 are assigned as points loads while lane loads were assigned as distributed load to the first span. Considering the influence as shown in the figure, using lane load just for the first span will be a conservative approach. Since there are four lanes assumed, the magnitude of the load was increased by a factor of four to account for the all lanes. A similar approach should be considered for the manual vehicle load, so the axle loads were increased by a factor of four, as well. Because the spans of the Bridge II are relatively long, superimposed dead loads were also considered since their effects on span moments may be noteworthy. For simplicity 1 kip load per foot is assumed as superimposed DLs. The illustration of the load cases are shown in Figure 5-11.

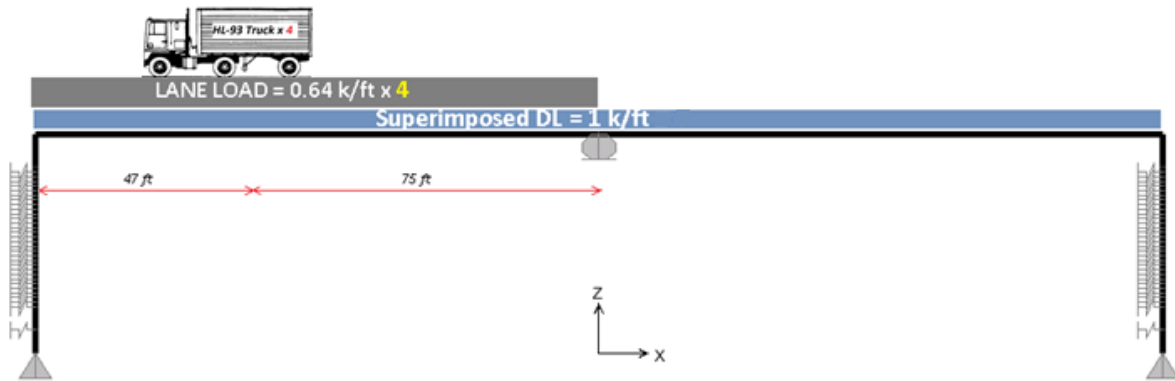


Figure 5-11 : Illustration of load definition for Bridge II.

5.3 Analysis Results

Although, 30 and 40 ft. pile lengths are considered in the analysis, it was observed, based on the data, that the results of the two cases are almost the same. Therefore, only one result case in terms of pile length will be given here, however, results are valid both for 40 and 50 ft pile lengths. Analyzes were made for 49 cases by using finite element program SAP2000. As a result, element forces and pile deformations were obtained for each case under live loading. The typical deformed shape and the moment diagram of Bridge II are shown in Figure 5-12 and Figure 5-13 with an exaggerated scale. The results are based on the truck location where the center of gravity of the truck is placed 47 feet from the left end of the girder. This case creates the maximum rotation at the abutment-pile joint on the left side.

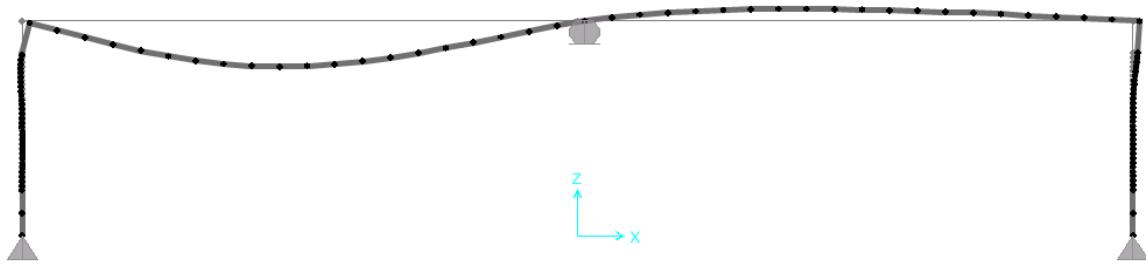


Figure 5-12 : Typical deformed shape of the frame model of Bridge II.

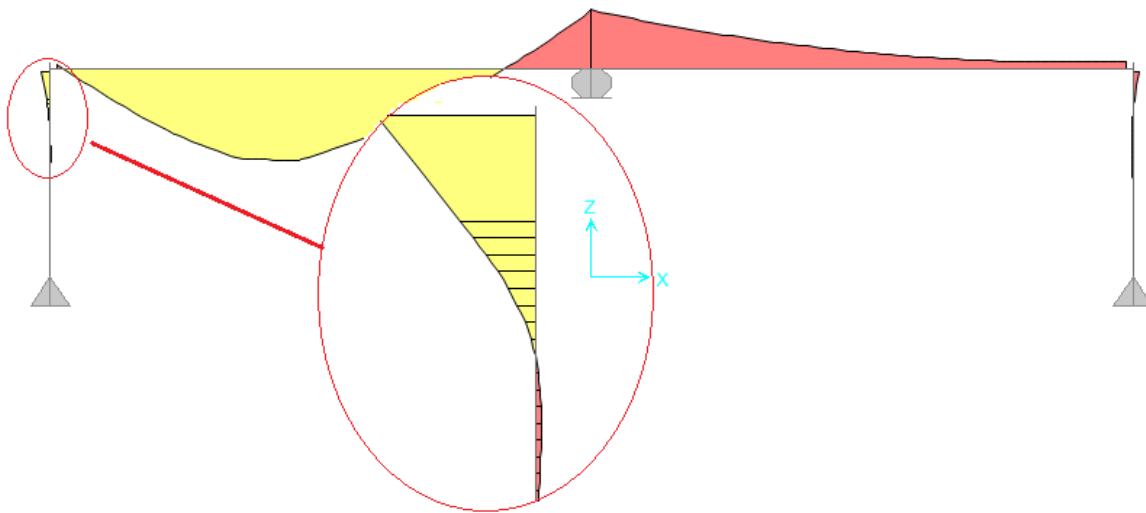


Figure 5-13 : Typical moment diagram of the frame model of Bridge II.

5.3.1 Moment Reduction

In order to see the positive moment reduction due to integral abutments related to Bridge II for a range of pile sizes and soil types, comparisons were made after obtaining the girder moments both with and without integral abutments. The results are presented in Table 5-6 and Table 5-7, for clay and sand soils, respectively.

According to these results, it is evident that high positive moment reduction could not be achieved for the bridges with more than one span and longer span lengths. This ineffectiveness of integral abutments stems from the use of an intermediate support and the stiffer girders, relative to pile stiffness, required for the long spans.

With regards to data obtained after the analyzes, with the use the of lightest pile, HP10x42, 1 percent of moment reduction capacity is achieved at least and this value reaches 2 percent with the use of HP14x89 piling surrounded with soft clay. In the case of the use of medium clay as a foundation soil, the reduction capacities are 2 and 3 percent for the range of piling between HP10x42 and HP14x89. For stiff clay soil type, the moment reduction capacity ranges between 3 percent and 5 percent as lowest and highest level, respectively. For the very stiff clay considered, 4 percent moment reduction capacity can be achieved, and this goes to as high as 6 percent with the use of the highest stiffness for piling and foundation soil. The reduction capacities of the cases considered are illustrated in Figure 5-14.

Table 5-6 : Superstructure moment reductions for Bridge II models relative to clay soil.

Pile Section	Regular Bridge Moment	Soft Clay		Medium Clay		Stiff Clay		Very Stiff Clay	
		Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %
HP10X42	10751	10592	1%	10531	2%	10430	3%	10306	4%
HP10X57	10751	10583	2%	10511	2%	10398	3%	10261	5%
HP12X53	10751	10560	2%	10482	3%	10364	4%	10224	5%
HP12X63	10751	10550	2%	10466	3%	10340	4%	10195	5%
HP12X74	10751	10534	2%	10446	3%	10312	4%	10161	5%
HP14X73	10751	10512	2%	10416	3%	10276	4%	10126	6%
HP14X89	10751	10489	2%	10385	3%	10237	5%	10079	6%

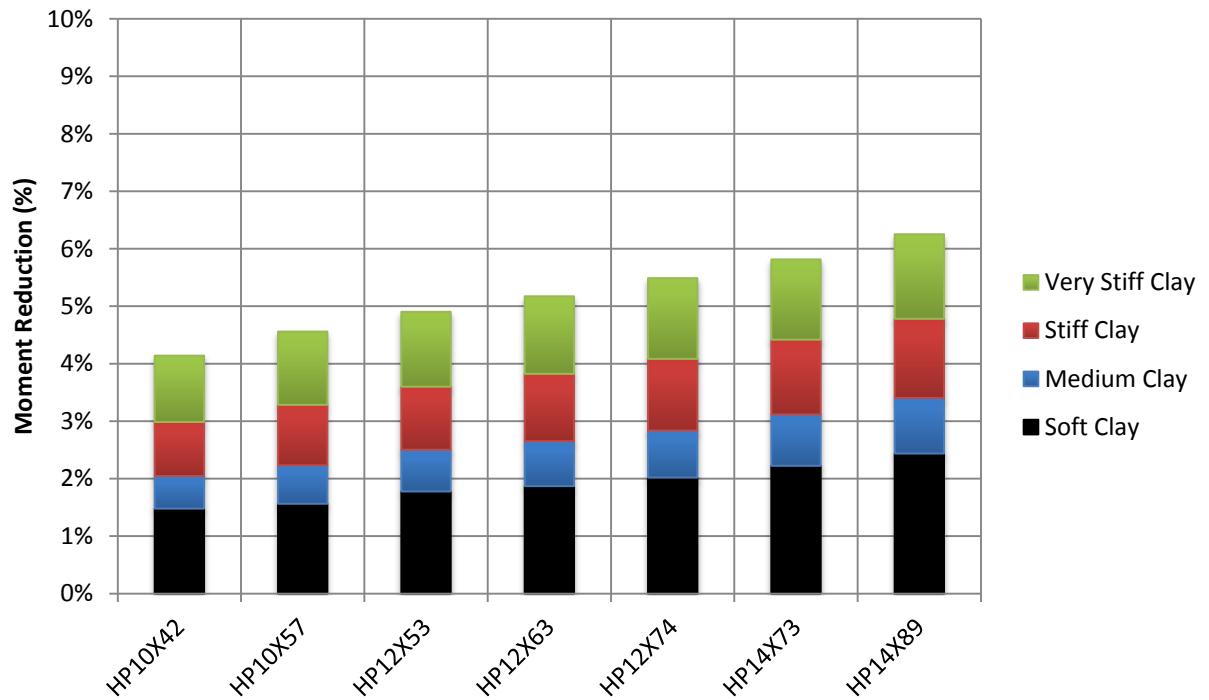


Figure 5-14 : Achieved superstructure moment reduction relative to clay foundation soils.

The data obtained for sand soils are given in Table 5-7. The same pattern takes place as in the previous case. With the use of loose sand as a foundation soil, 2 percent moment reduction capacity can be achieved as the least for the lightest pile section considered and 3 percent as the highest for the largest section considered in the analyses. For foundation with medium sand the ranges of reduction capacities are 2 and 4 percent, as lowest and highest respectively. In terms of dense sand considered, 3 percent moment reduction capacity is achieved and this capacity rises to 5 percent with the use of largest section. The illustration of the moment reduction capacities for cohesionless soil is shown in Figure 5-15.

Table 5-7 : Superstructure moment reductions for Bridge II models relative to sand soil.

Pile Section	Regular Bridge Moment	Loose Sand		Medium Sand		Dense Sand	
		Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %
HP10X42	10751	10578	2%	10512	2%	10440	3%
HP10X57	10751	10562	2%	10485	2%	10405	3%
HP12X53	10751	10533	2%	10449	3%	10363	4%
HP12X63	10751	10518	2%	10429	3%	10336	4%
HP12X74	10751	10499	2%	10404	3%	10304	4%
HP14X73	10751	10466	3%	10364	4%	10258	5%
HP14X89	10751	10437	3%	10325	4%	10214	5%

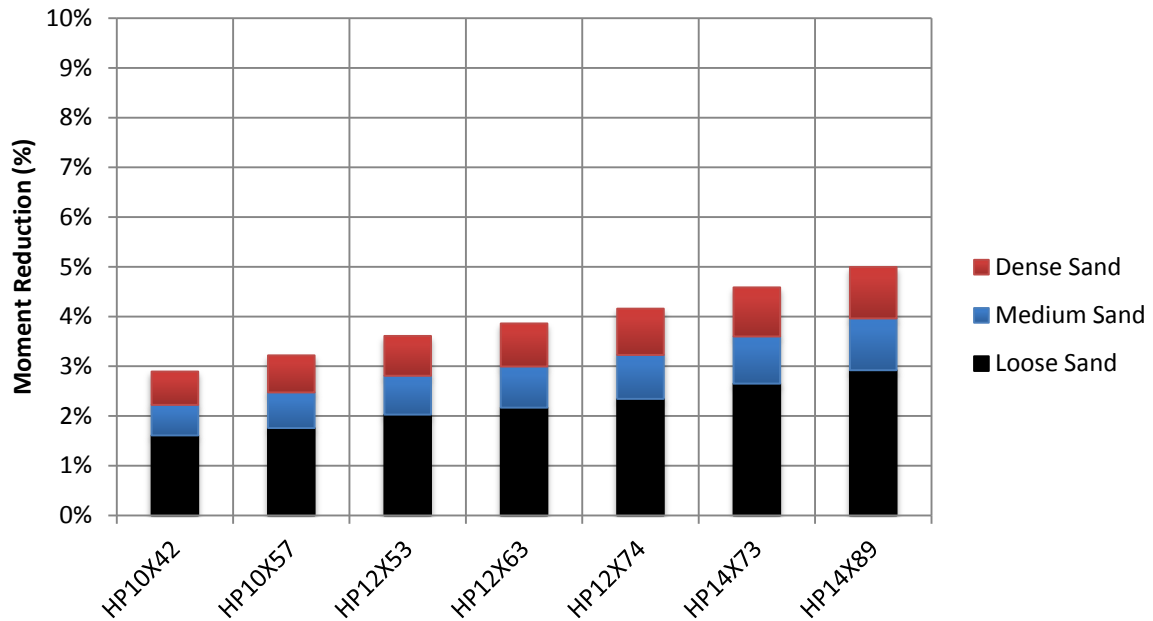


Figure 5-15 : Achieved superstructure moment reduction relative to sand foundation soils.

5.3.2 Abutment-pile joint

In order to approximate abutment-pile behavior as rotational and linear springs for a given set of parameters, analysis results related to the pile head deformation were obtained along with the element forces. To develop rotational and linear spring stiffnesses, equation 4-7 and 4-9 were used. The data belongs for pile head deformation and forces are provided in Table 5-8 to Table 5-14.

Table 5-8 : Pile head displacements and forces in soft clay, and computed stiffnesses for Bridge II abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0013	-0.004	132	-18	105516	53672
HP10x57	0.0013	-0.001	168	-20	133865	299407
HP12x53	0.0012	0.003	200	-22	160514	-96845
HP12x63	0.0012	0.005	227	-24	182183	-59744
HP12x74	0.0012	0.006	261	-26	209976	-55771
HP14x73	0.0012	0.010	301	-27	243528	-31087
HP14x89	0.0012	0.013	348	-30	282697	-26616

Table 5-9: Pile head displacements and forces in medium clay, and computed stiffnesses for Bridge II abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0012	-0.011	180	-32	146223	35080
HP10x57	0.0012	-0.008	227	-37	184253	53243
HP12x53	0.0012	-0.005	268	-40	219492	86459
HP12x63	0.0012	-0.004	304	-42	249385	137216
HP12x74	0.0012	-0.002	344	-45	283128	308340
HP14x73	0.0012	0.001	397	-48	329734	-572222
HP14x89	0.0012	0.004	458	-52	382943	-175757

Table 5-10 : Pile head displacements and forces in stiff clay, and computed stiffnesses for Bridge II abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0012	-0.015	244	-60	203503	49251
HP10x57	0.0012	-0.012	305	-67	255444	67509
HP12x53	0.0012	-0.010	355	-71	300338	88834
HP12x63	0.0012	-0.008	399	-75	338997	114174
HP12x74	0.0012	-0.006	450	-80	384615	156638
HP14x73	0.0012	-0.004	511	-83	441659	262161
HP14x89	0.0011	-0.001	586	-89	511344	712510

Table 5-11: Pile head displacements and forces in very stiff clay, and computed stiffnesses for Bridge II abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0012	-0.015	306	-97	264477	77964
HP10x57	0.0012	-0.013	377	-109	325843	104236
HP12x53	0.0011	-0.010	434	-112	382716	131315
HP12x63	0.0011	-0.009	485	-119	430346	163629
HP12x74	0.0011	-0.007	543	-125	486124	213402
HP14x73	0.0011	-0.005	611	-128	552941	312169
HP14x89	0.0011	-0.003	694	-135	636114	583243

Table 5-12 : Pile head displacements and forces in loose sand, and computed stiffnesses for Bridge II abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0012	-0.080	148	-21	122720	259
HP10x57	0.0012	-0.076	190	-24	158201	320
HP12x53	0.0012	-0.073	231	-28	195101	380
HP12x63	0.0012	-0.070	263	-30	223260	429
HP12x74	0.0012	-0.068	301	-33	257265	486
HP14x73	0.0012	-0.064	354	-36	307292	569
HP14x89	0.0011	-0.060	412	-40	361721	664

Table 5-13: Pile head displacements and forces in medium sand, and computed stiffnesses for Bridge II abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0012	-0.081	197	-37	170121	454
HP10x57	0.0011	-0.078	251	-43	219214	550
HP12x53	0.0011	-0.074	300	-47	267142	635
HP12x63	0.0011	-0.072	340	-51	305481	708
HP12x74	0.0011	-0.070	387	-55	351818	792
HP14x73	0.0011	-0.067	450	-60	417440	903
HP14x89	0.0011	-0.063	520	-65	491029	1035

Table 5-14 : Pile head deformation and forces in dense sand, and computed stiffnesses for Bridge II abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0011	-0.079	243	-56	220308	715
HP10x57	0.0011	-0.075	306	-65	282288	862
HP12x53	0.0011	-0.072	362	-70	342479	982
HP12x63	0.0010	-0.070	410	-76	393097	1091
HP12x74	0.0010	-0.067	463	-81	451267	1216
HP14x73	0.0010	-0.064	534	-87	534000	1369
HP14x89	0.0010	-0.060	613	-94	628074	1560

For the physical parameters of bridges similar to Bridge II, the steel pile foundation can be represented by a single rotational and linear spring where the values can be based on the data obtained by the result of this parametric study and given in the Table 5-8 through Table 5-14.

Table 5-15 : Bridge II physical properties

Total Bridge Length	(ft)	244
Number of spans		2
Lumped Moment of Inertia of superstructure	(in ⁴)	8496550
Lumped Area of superstructure	(in ²)	9694
Abutment Height	(ft)	10
Abutment Thickness	(ft)	3

5.3.3 Depth of Inflection Point

The pile end fixity values related to the cases of Bridge II were obtained based on the location of inflection points and provided in the following tables and figures.

Table 5-16 : Depths of inflection points.

Depth of Inflection Point , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	8.92	6.75	4.93	3.8	7.52	5.86	4.79
HP10x57	9.79	7.37	5.41	4.17	8.09	6.35	5.17
HP12x53	10.65	8.05	5.94	4.63	8.69	6.81	5.58
HP12x63	11.18	8.47	6.26	4.85	9.03	7.08	5.8
HP12x74	11.77	8.86	6.58	5.1	9.45	7.41	6.03
HP14x73	12.71	9.61	7.17	5.6	9.99	7.87	6.47
HP14x89	13.58	10.11	7.38	5.92	10.5	8.27	6.78

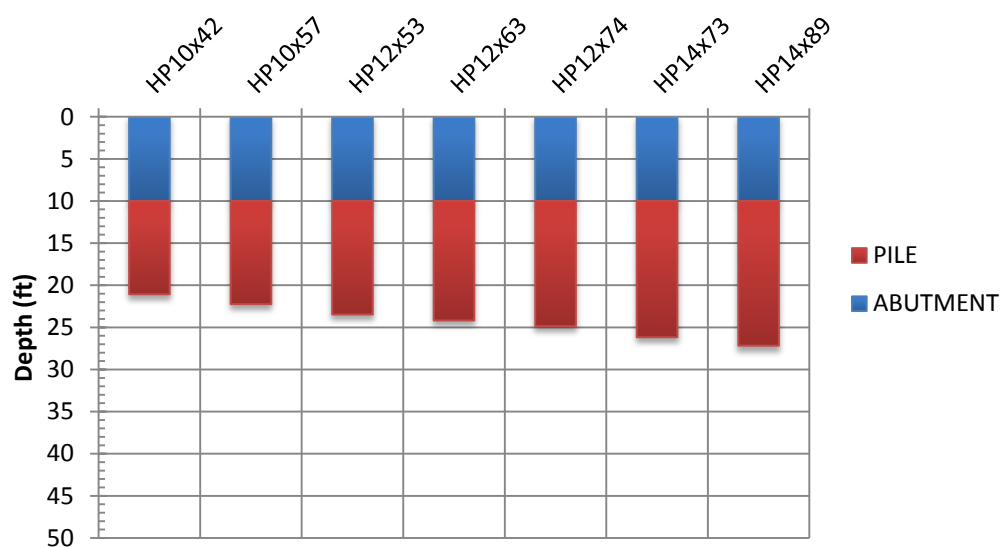


Figure 5-16 : Depth of inflection points of Bridge II piles in soft clay.

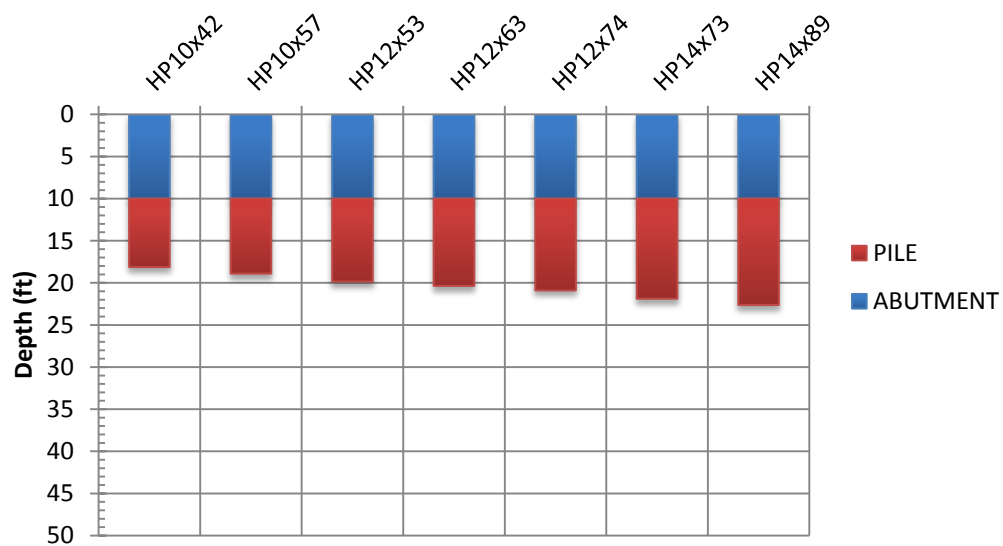


Figure 5-17 : Depth of inflection points of Bridge II piles in medium clay.

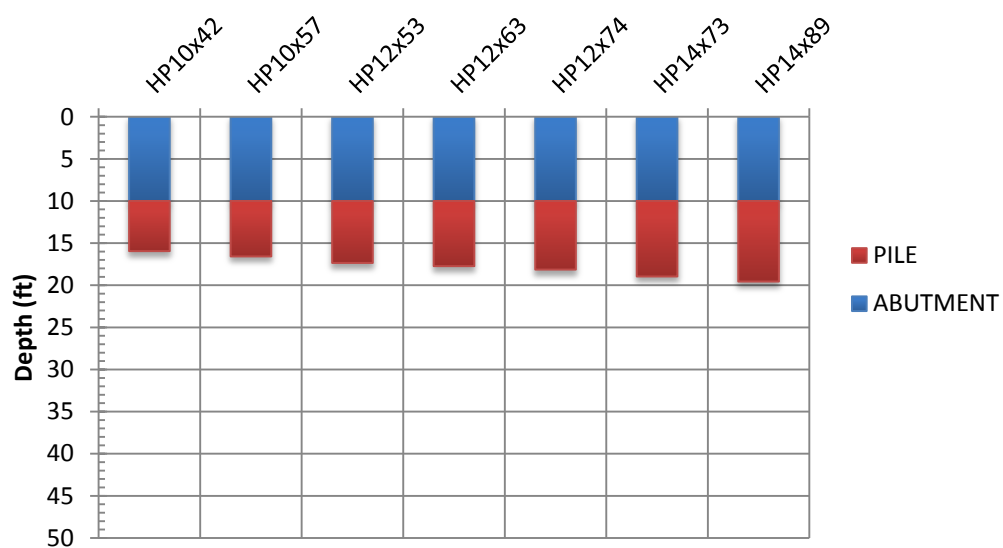


Figure 5-18 : Depth of inflection points of Bridge II piles in stiff clay.

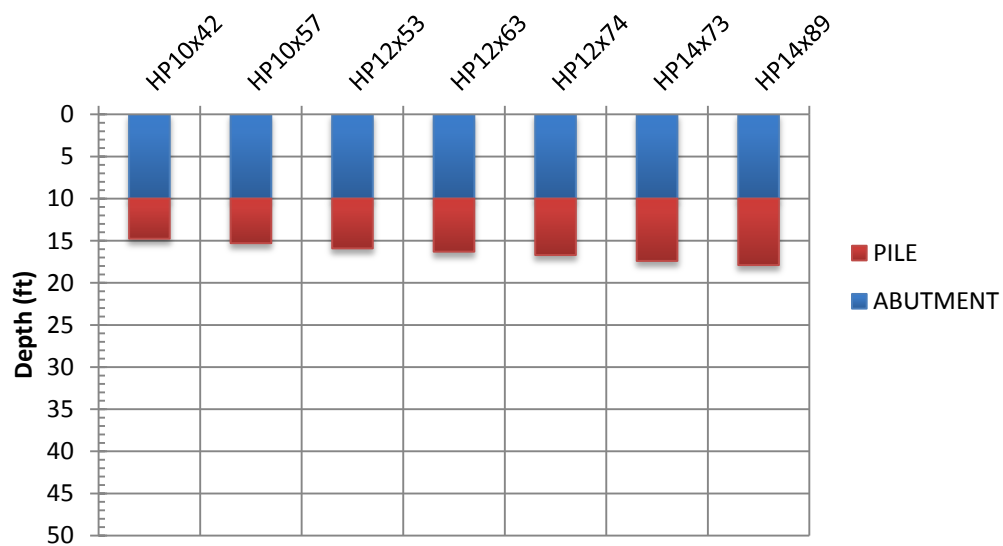


Figure 5-19 : Depth of inflection points of Bridge II piles in very-stiff clay.

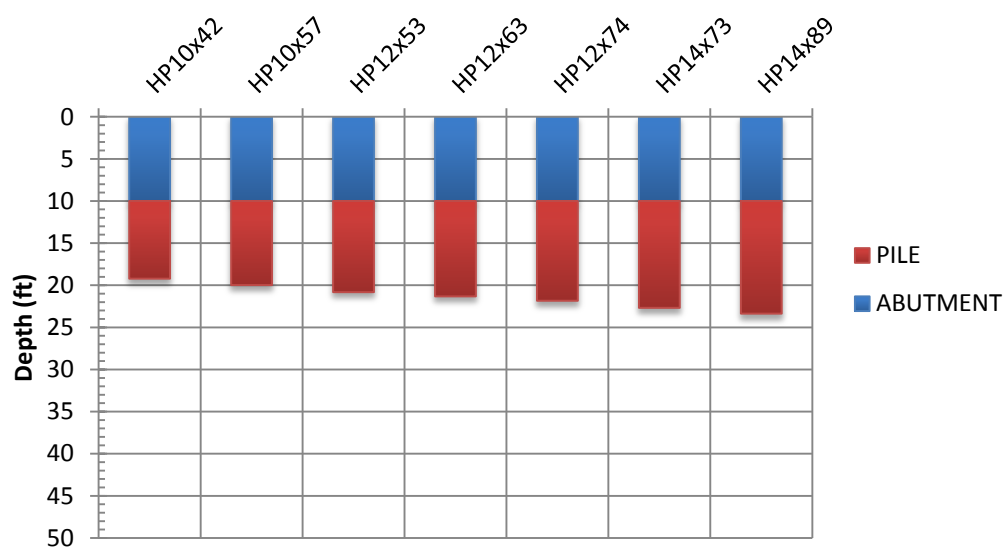


Figure 5-20 : Depth of inflection points of Bridge II piles in loose sand.

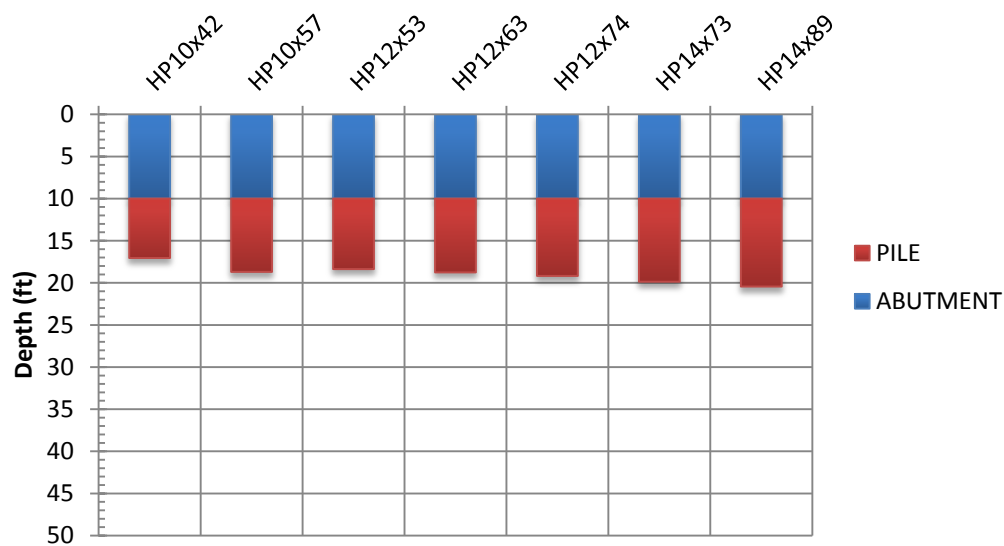


Figure 5-21 : Depth of inflection points of Bridge II piles in medium sand.

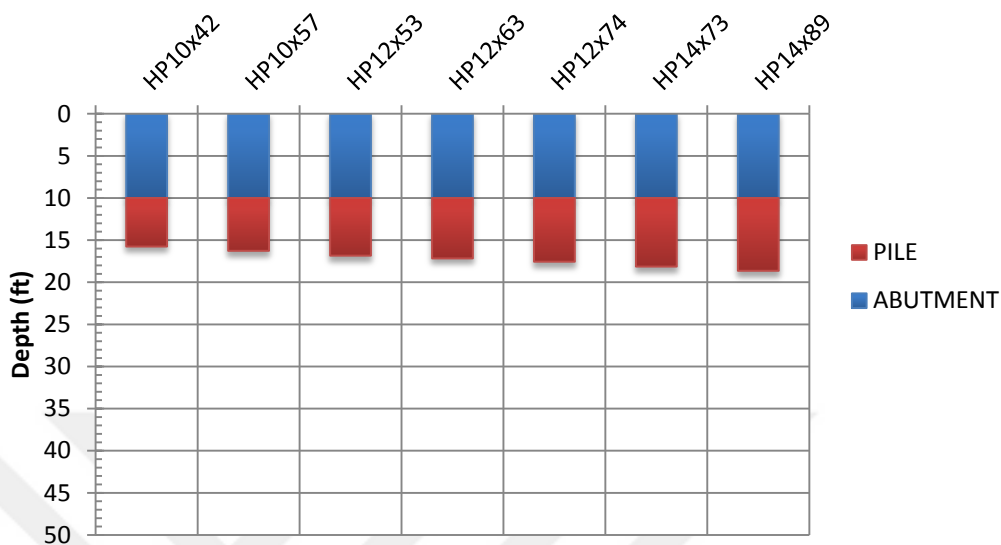


Figure 5-22 : Depth of inflection points of Bridge II piles in dense sand.

5.3.4 Depth of Zero Rotation

Table 5-17 and Table 5-18 provide the first and second points of zero rotation along the depth of the piles of Bridge II. Figure 5-23 to Figure 5-24 illustrate the increase and decrease of the point of zero rotation relative to the soil types and pile sections.

Table 5-17 : Depths of first zero rotation.

Point of Zero Rotation , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	5.6	3.9	2.8	2.2	2.3	1.7	1.2
HP10x57	6.2	4.4	3.1	2.5	2.6	1.8	1.4
HP12x53	7	5	3.7	2.9	2.9	2.1	1.6
HP12x63	7.4	5.3	3.9	3.1	3.1	2.2	1.7
HP12x74	7.8	5.6	4.1	3.4	3.4	2.4	1.8
HP14x73	8.7	6.3	4.7	3.9	3.7	2.6	2
HP14x89	9.3	6.7	5	4.2	4	2.8	2.2

Table 5-18 : Depths of second zero rotation.

Second Point of Zero Rotation , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	39	31	26	24	30	26	24
HP10x57	41	33	27	25	32	28	25
HP12x53	43	35	29	26	33	29	26
HP12x63	44	36	30	27	34	29	26
HP12x74	45	37	31	28	35	30	27
HP14x73	46	38	32	29	37	31	28
HP14x89	47	40	34	30	38	32	29

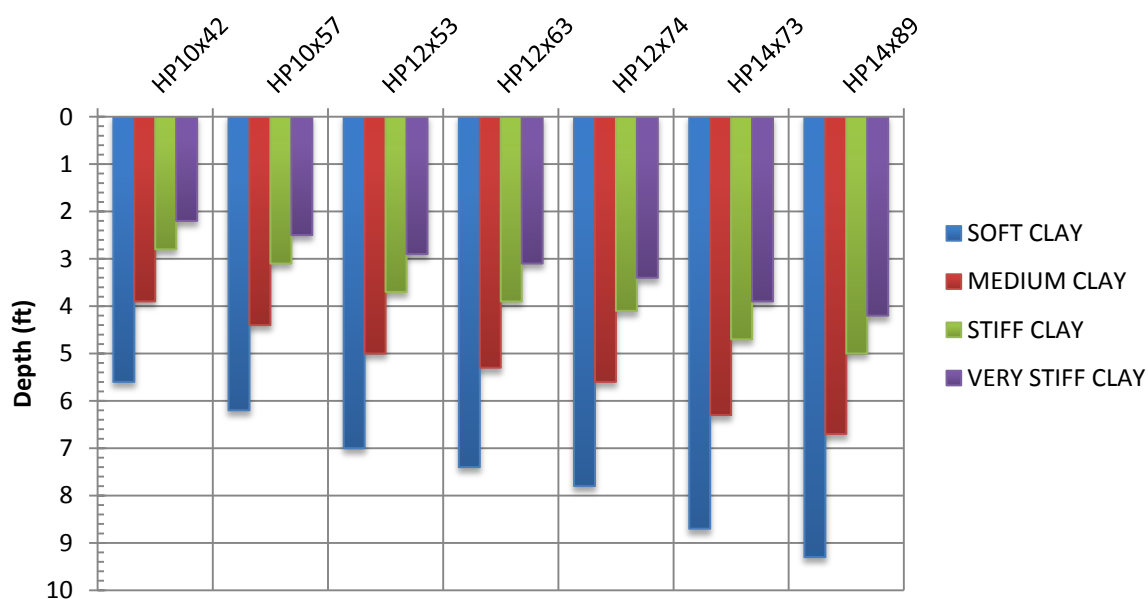


Figure 5-23 : Chart of the depths of first zero rotation in clay soil.

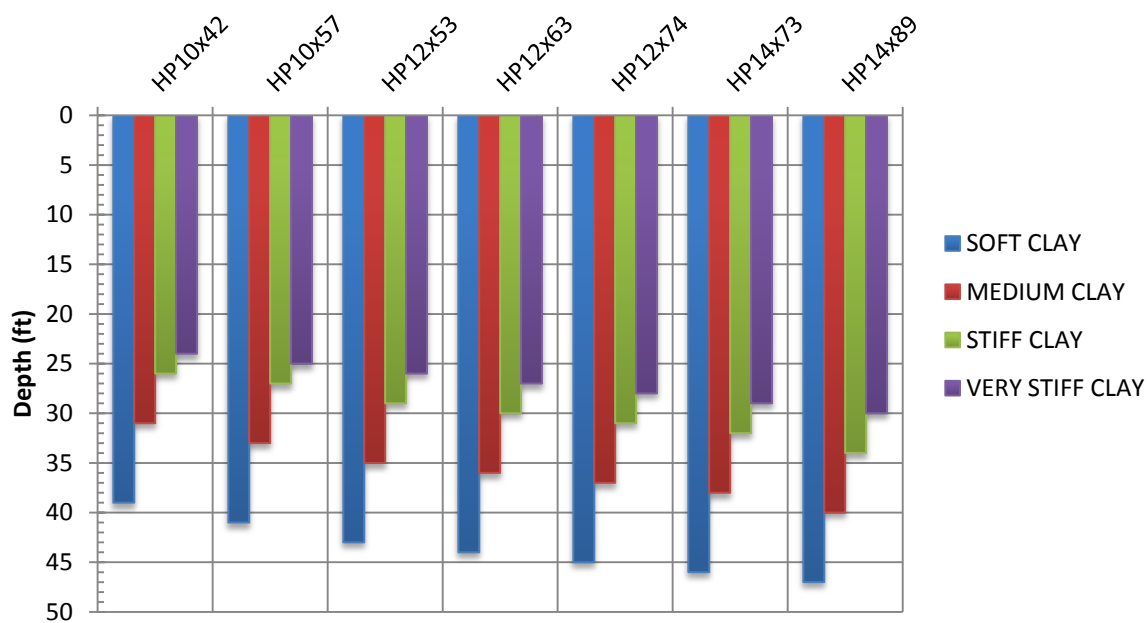


Figure 5-24 : Chart of the depths of second zero rotation in clay soil.

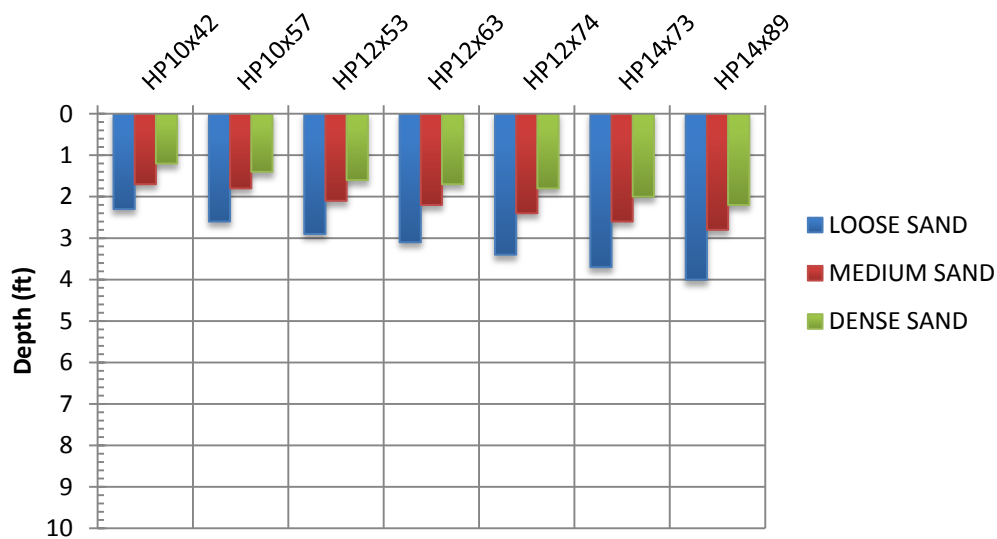


Figure 5-25 : Chart of the depths of first zero rotation in sand soil.

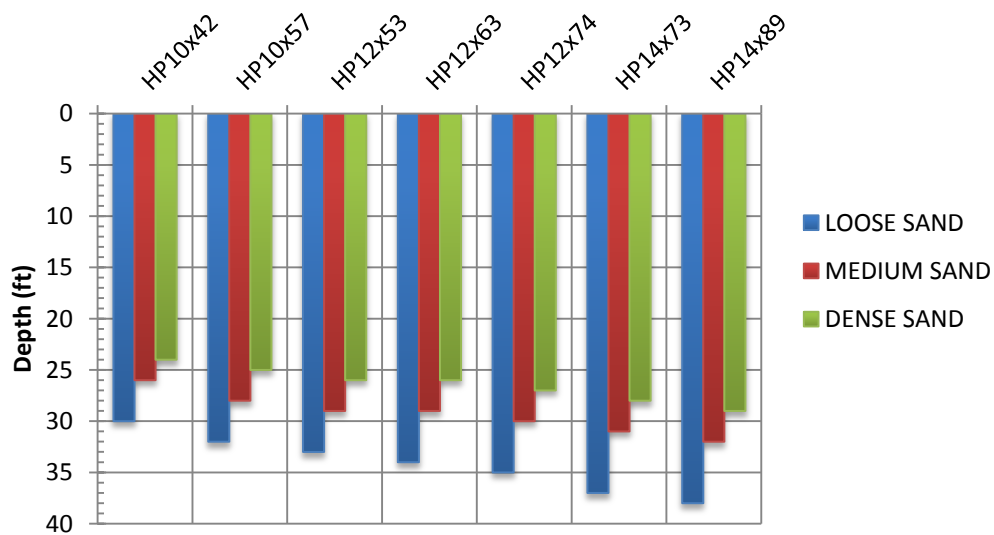


Figure 5-26 : Chart of the depths of second zero rotation in sand soil.

6. BRIDGE III

6.1 Introduction

Bridge III is a reinforced concrete slab bridge. This bridge has a 50 ft length of span and a width of 46 ft as shown in Figure 6-2 and Figure 6-3. The thickness of the slab is 26.4 in. In terms of substructure, 10 steel HP piles were assumed for piling, and abutments are 10 by 2.5 ft in dimensions for height and thickness, respectively. The same soil parameters and pile sections are assumed as in the Bridges I and II. A summary of the structure parameters of Bridge III is given in Table 6-1.

Table 6-1: Properties of Bridge III.

Properties		Bridge III
Span length	(ft)	50
Bridge width	(ft)	46
Slab thickness	(in)	26.4
Abutment height	(ft)	10
Abutment thickness	(ft)	2.5
Number of piles		10
Pile length	(ft)	30

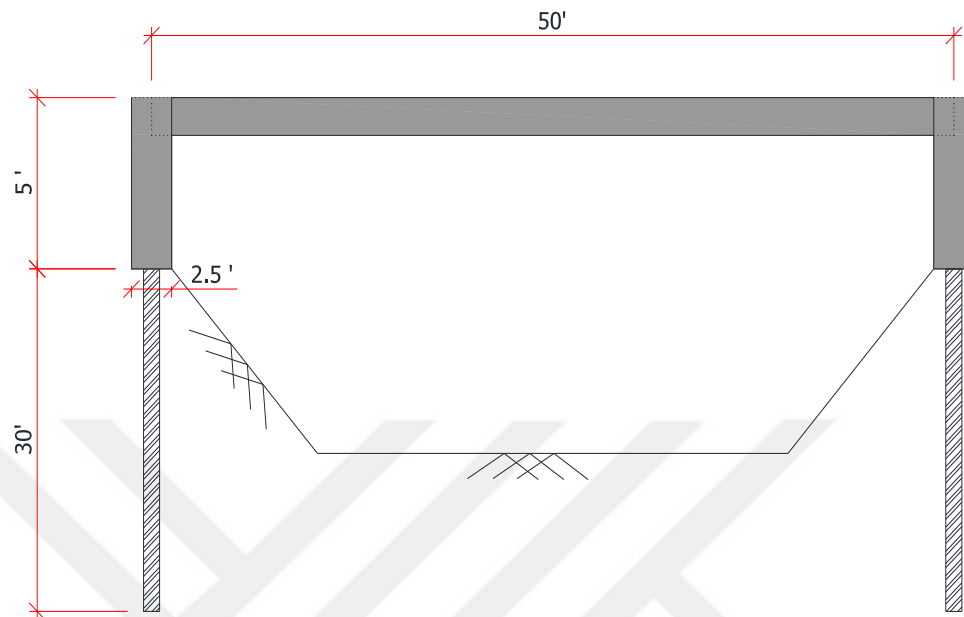


Figure 6-1 : Elevation view of Bridge III.

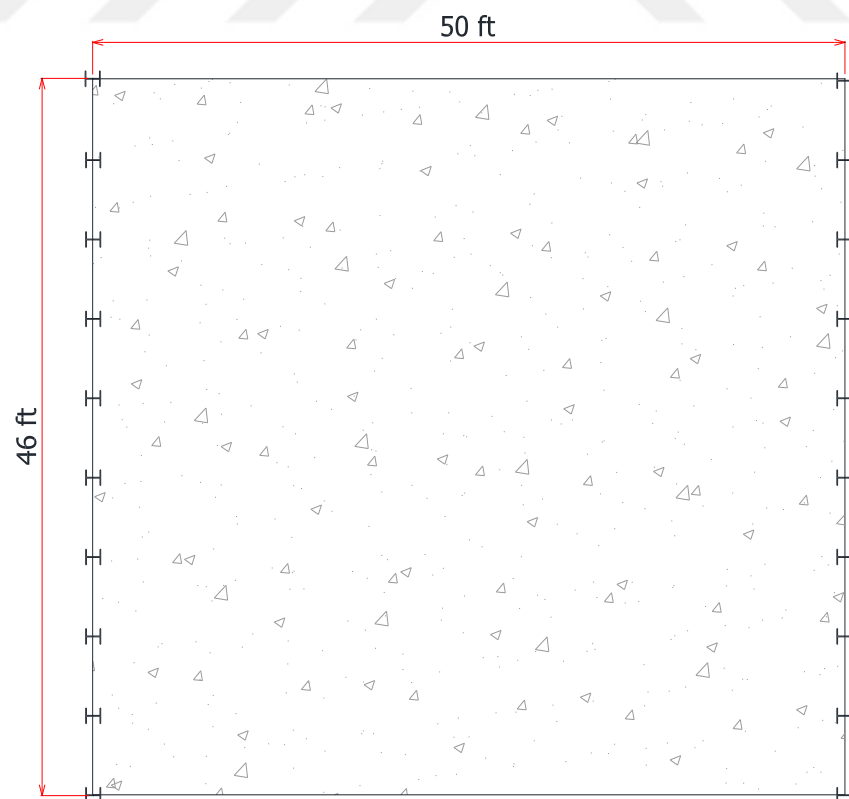


Figure 6-2 : Plan view of Bridge III.

6.2 Analysis

The bridge was modeled using the structure analysis program Sap2000 V14. Two-dimensional modeling and analysis technique was adopted for Bridge III as in the analysis of Bridge I and II. Because the bridge has no girders, the properties of the slab section were used instead of modeling of girder elements in the program. All components of the bridge including slab were modeled by using linear-elastic beam elements. However, material properties vary for different components. Table 6-2 gives the material properties of the components.

$$f_{cd} = 4000 \text{ ksi}$$

$$f_{ca} = 3500 \text{ psi}$$

$$E_d = 33000 * 0.15^{1.5} \sqrt{\frac{f_{cd}}{4000}} = 3834 \text{ ksi} \quad 6-1$$

$$E_a = 33000 * 0.15^{1.5} \sqrt{\frac{f_{ca}}{4000}} = 3587 \text{ ksi} \quad 6-2$$

Where;

E_d = Modulus of elasticity of deck, ksi

E_a = Modulus of elasticity of abutment, ksi

Table 6-2 : Material properties of Bridge III.

Component	f'_c , psi	E_c , ksi
Deck	4000	3834
Substructure	3500	3587

6.2.1 Load cases

AASHTO HL-93 Truck was used as a vehicle load with an axle spacing of 14 feet, since the larger effects occur with the lower axle spacing. In addition to the vehicle load, lane loads are also considered. In the definition of number of lanes, the width of the bridge is divided by the width of the lane, which is 12 feet, and the result is used as a whole number which then defines number of lanes. However, such procedure is not taken account for Bridge III since its properties are taken from a real structure which was designed for two lanes, therefore, two lanes will be assumed in the study.

Figure 6-3 shows the modeling of Bridge III in the program. Figure 6-4 illustrates the lane definition of the program.

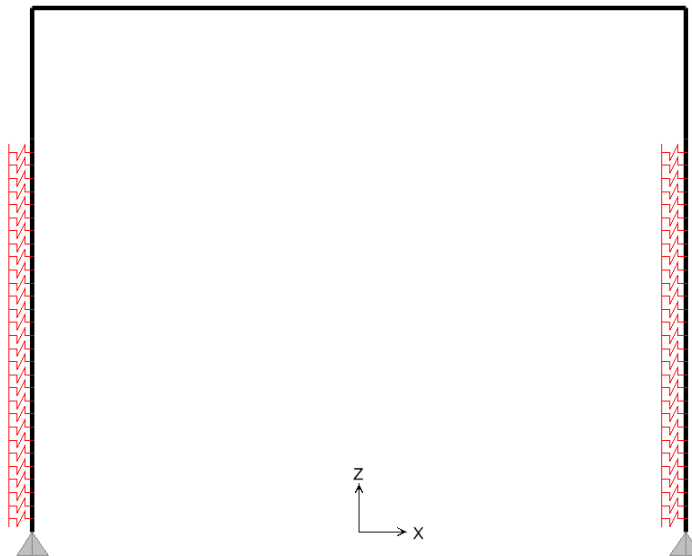


Figure 6-3 : 2-d frame model of Bridge III in SAP2000.

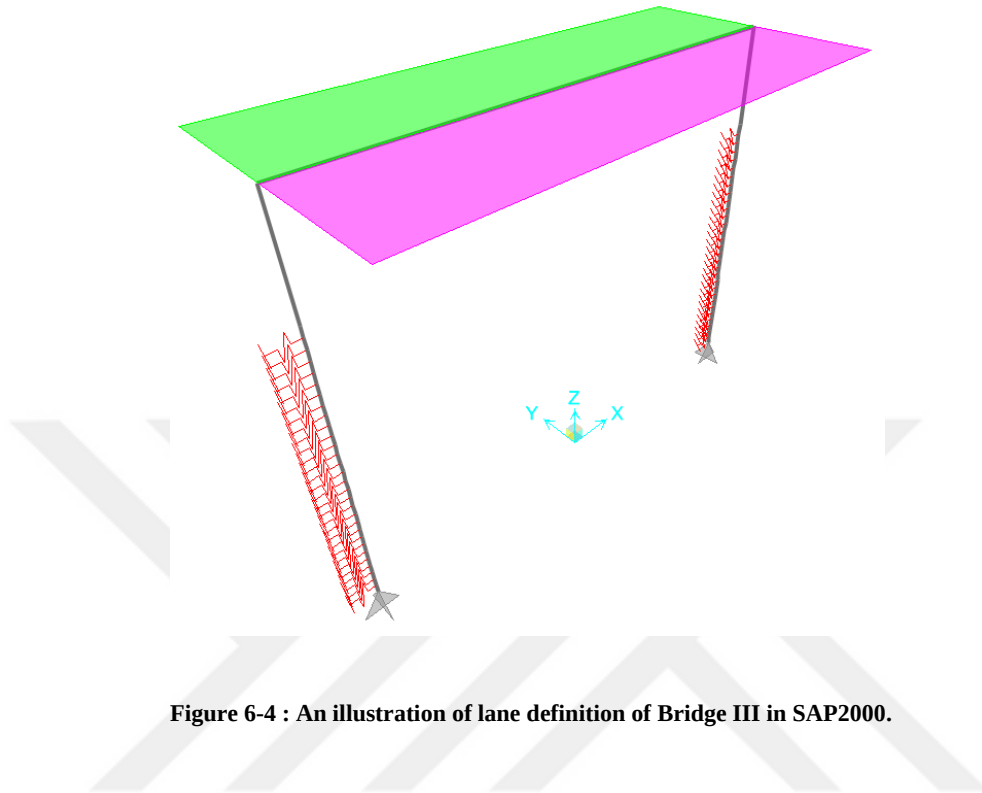


Figure 6-4 : An illustration of lane definition of Bridge III in SAP2000.

Manually created Truck load was used as a live load case in combination with lane loads. Influence line analysis was performed to find out the location of Truck load on the longitudinal length of the lanes that causes the largest influence on the rotational deformation of abutment-pile joint. Based on the influence line analysis, the most effective point appears at 15 ft from the left end of the girder. Therefore, the manually defined Truck load will be applied with regards to this point. Figure 6-5 shows the influence lines for the pile-abutment joint under truck load.

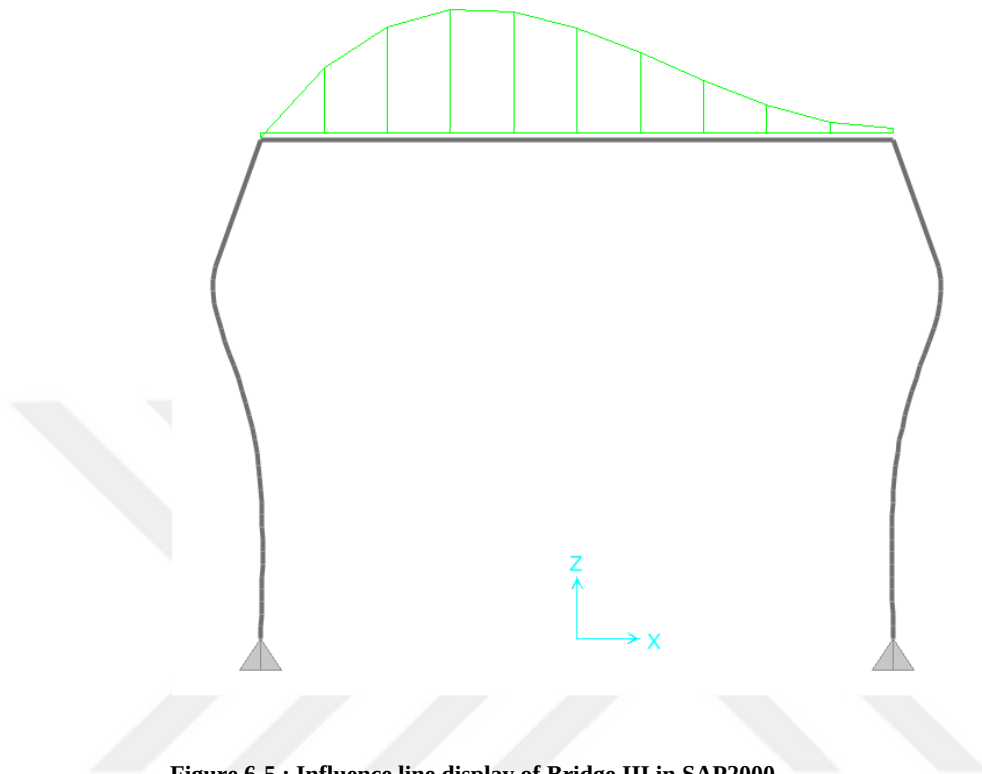


Figure 6-5 : Influence line display of Bridge III in SAP2000.

The truck load is assigned to the girder element in such a way that the center of gravity of the truck will match with the distance of 15 ft from the left end. The axle loads for a HL-93 truck were imitated by using point loads in the program, and the lane loads were considered as distributed load over the length of the girder. Since there are two lanes assumed, the magnitude of both manual Truck and lane load was increased by a factor of two to create the worst case possible as far as the abutment-pile joint concerned. No superimposed DL considered since it will be insignificant compared to other loads. The illustration of the load cases are shown in Figure 6-6.

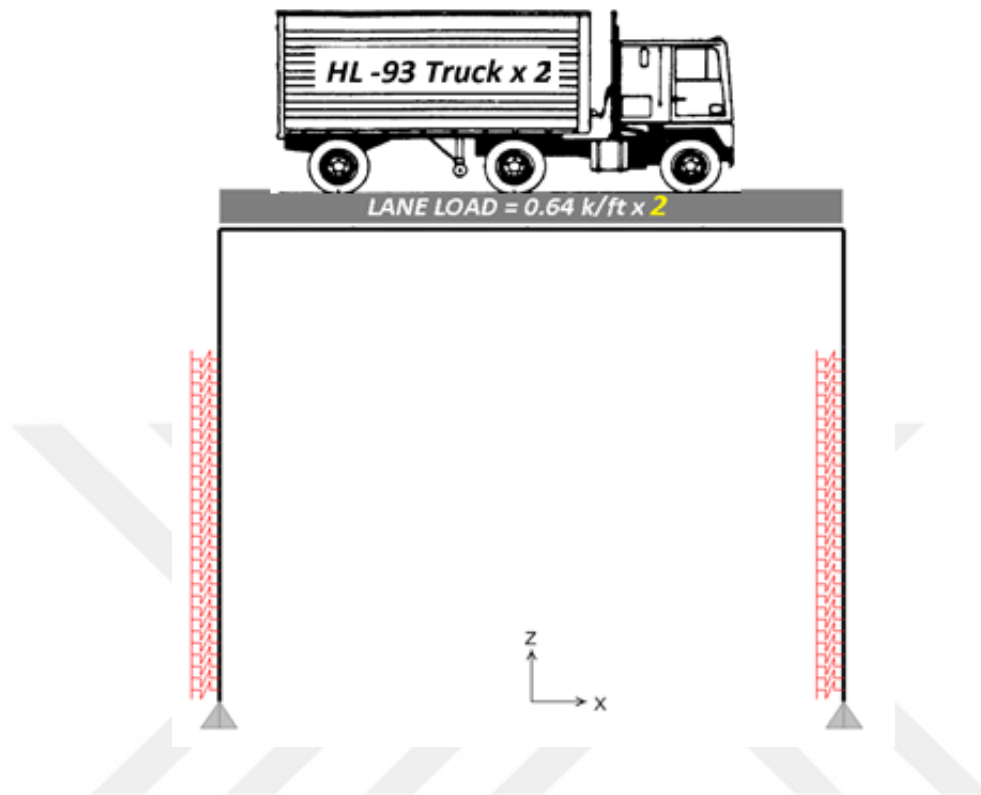


Figure 6-6 : Illustration of load cases for the frame model of Bridge III.

6.2.2 Soil Model

In order to model soil-structure interaction in the program, the beam element used for the pile was meshed into 1 foot discrete elements for the first 30 ft from the ground surface, and the remaining 10 foot pile length was meshed into 2 discrete elements having a 5 foot length. Thus, linear-elastic springs could be assigned to the nodes of two adjacent elements to represent the soil response against piles. Using the correlations given in section 3.3.2, the stiffness values for each spring were computed for the seven kinds of soil considered and the seven sizes of piles. The other stiffness tables are provided in APPENDIX A

6.3 Analysis Results

The bridge was analyzed under 46 cases by using SAP2000. The girder moments of the 2-d frame models and the deformed shape of the pile elements due to live loading were obtained. The typical deformed shape and the moment diagram are shown in Figure 6-7 and Figure 6-8, respectively, with an exaggerated scale. The results are based on the truck location where the center of gravity of the truck placed 15 ft from the left end of the bridge. This case creates the maximum rotation at the abutment-pile joint on the left side.

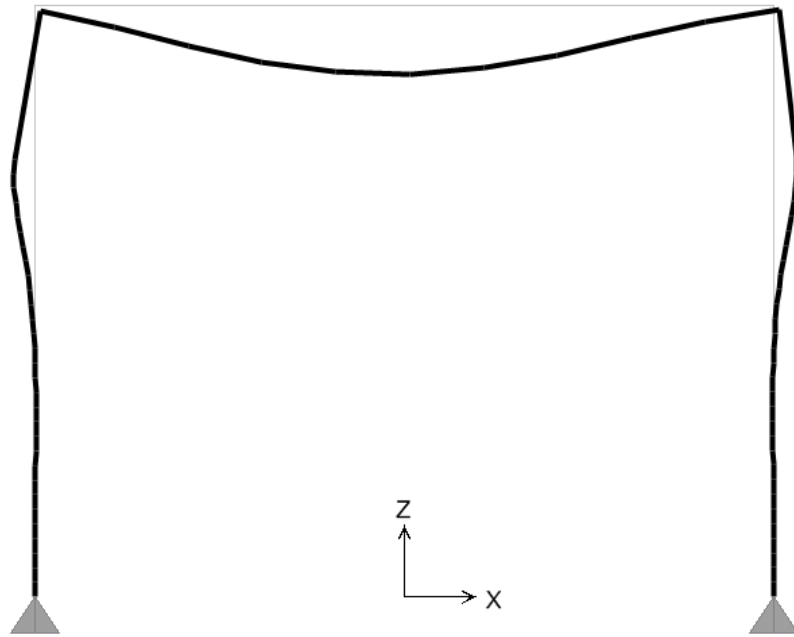


Figure 6-7 : Typical deformed shape of the frame model of Bridge III in SAP2000.

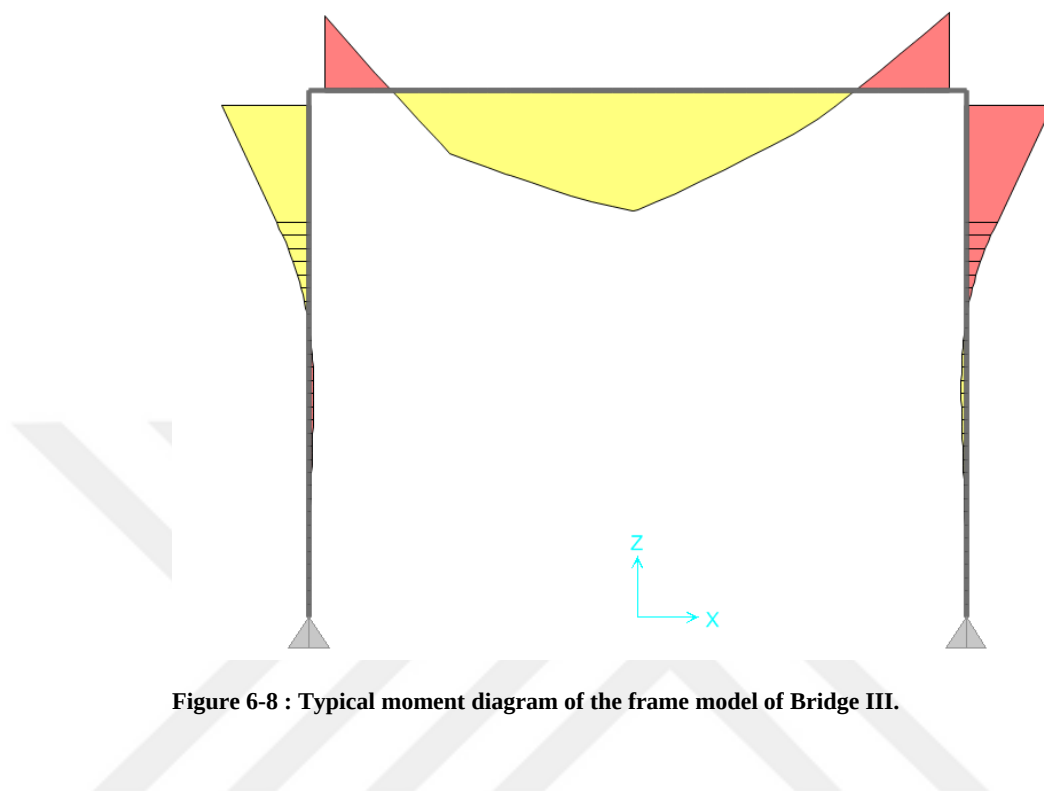


Figure 6-8 : Typical moment diagram of the frame model of Bridge III.

6.3.1 Moment Reduction

In order to see the positive moment reduction provided by of integral abutments related to Bridge III for a range of pile sizes and soil types, comparisons were made after obtaining the girder moments with and without integral abutment bridges. The results are presented in Table 6-3 and Table 6-4, for clay and sand, respectively.

With regards to data obtained after the analyses, with the use of the lightest pile, HP10x42, 28 percent of moment reduction capacity is achieved at least and this value increases as high as 39 percent with the use of HP14x89 piles in soft clay. In the case of the use of medium clay as a foundation soil, the reduction capacities are 38 and 46 percent for the range of piles between HP10x42 and HP14x89. For stiff clay foundation soil, the moment

reduction capacity ranges between 47 percent and 52 percent as lowest and highest level, respectively. For the very stiff clay considered, 53 percent moment reduction capacity can be achieved, and this goes to as high as 56 percent with the use of the highest stiffness for piling and foundation soil. The reductions of the cases considered are illustrated in Figure 6-9.

Table 6-3 : Superstructure moment reductions for Bridge III models in clay soil.

Pile Section	Regular Bridge Moment	Soft Clay		Medium Clay		Stiff Clay		Very Stiff Clay	
		Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %
HP10X42	1629	1168	28%	1016	38%	869	47%	770	53%
HP10X57	1629	1127	31%	979	40%	841	48%	751	54%
HP12X53	1629	1097	33%	956	41%	827	49%	744	54%
HP12X63	1629	1074	34%	936	43%	813	50%	735	55%
HP12X74	1629	1051	35%	917	44%	800	51%	726	55%
HP14X73	1629	1026	37%	898	45%	790	52%	721	56%
HP14X89	1629	1000	39%	877	46%	775	52%	711	56%

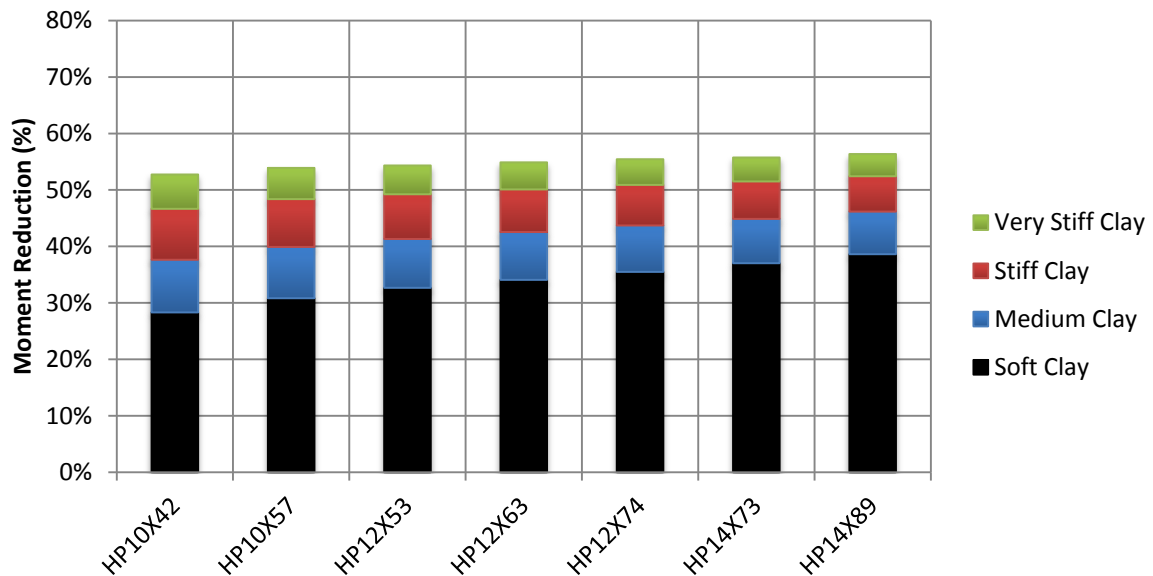


Figure 6-9 : Achieved superstructure moment reduction relative to clay foundation soil.

The same pattern takes place as in the previous cases. With the use of loose sand as a foundation soil, 31 percent moment reduction capacity can be achieved as the least for the lightest pile section considered and 43 percent as the highest for the largest section considered in the analyses. For foundation with medium sand the ranges of reduction capacities are 39 and 49 percent, as lowest and highest respectively. In terms of dense sand considered, 45 percent moment reduction capacity is achieved and this capacity rises to 53 percent with the use of largest section. The illustration of the moment reduction capacities for cohesionless soil is shown in Figure 6-10.

Table 6-4 : Superstructure moment reductions for Bridge III models in sand soil.

Pile Section	Regular Bridge Moment	Loose Sand		Medium Sand		Dense Sand	
		Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %	Integral Bridge Moment	Moment Reduct. %
HP10X42	1629	1130	31%	992	39%	893	45%
HP10X57	1629	1082	34%	950	42%	858	47%
HP12X53	1629	1043	36%	919	44%	828	49%
HP12X63	1629	1017	38%	998	39%	818	50%
HP12X74	1629	991	39%	878	46%	802	51%
HP14X73	1629	960	41%	854	48%	785	52%
HP14X89	1629	931	43%	833	49%	769	53%

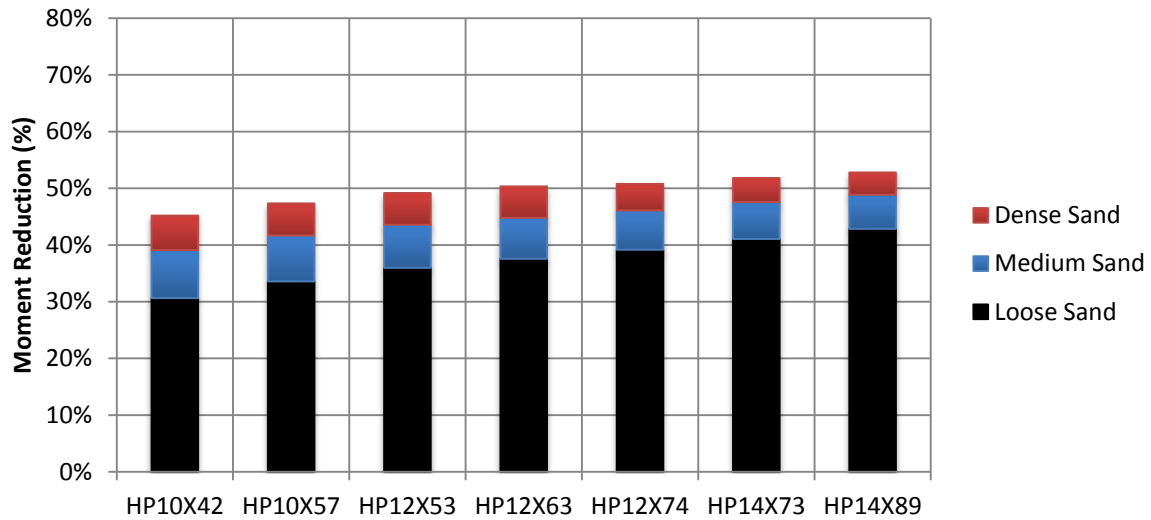


Figure 6-10 : Achieved superstructure moment reduction relative to sand foundation soil.

6.3.2 Abutment-pile joint

In order to approximate the pile-abutment behavior as a rotational and linear spring for a given set of parameters, analysis results related to the pile head deformation were obtained along with the element forces. To develop the rotational and linear spring stiffnesses equations 4-3 and 4-4 were used. Data for to head displacements and forces are presented in from Table 6-5 to Table 6-11.

Table 6-5 : Pile head displacements and forces in soft clay, and computed stiffnesses for Bridge III abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0006	-0.0712	161	-30	251170	5102
HP10x57	0.0006	-0.0656	186	-32	311558	5844
HP12x53	0.0006	-0.0617	209	-33	370567	6375
HP12x63	0.0005	-0.0586	223	-34	413729	6870
HP12x74	0.0005	-0.0555	240	-34	466926	7414
HP14x73	0.0005	-0.0521	261	-35	537037	7984
HP14x89	0.0005	-0.0486	282	-35	615721	8719

Table 6-6: Pile head displacements and forces in medium clay, and computed stiffnesses for Bridge III abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0005	-0.0520	174	-44	364780	10196
HP10x57	0.0004	-0.0471	197	-46	450801	11622
HP12x53	0.0004	-0.0439	217	-46	529268	12591
HP12x63	0.0004	-0.0413	231	-47	593830	13548
HP12x74	0.0004	-0.0388	246	-47	668478	14609
HP14x73	0.0003	-0.0362	266	-47	766571	15653
HP14x89	0.0003	-0.0333	284	-48	879257	17138

Table 6-7: Pile head displacements and forces in stiff clay, and computed stiffnesses for Bridge III abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.000317	-0.033342	173	-59.076	545741	21262
HP10x57	0.0003	-0.0296	193	-60	670139	24290
HP12x53	0.0003	-0.0278	209	-60	771218	25859
HP12x63	0.0003	-0.0259	221	-60	863281	27840
HP12x74	0.0002	-0.0241	234	-60	966942	30031
HP14x73	0.0002	-0.0227	250	-60	1091703	31597
HP14x89	0.0002	-0.0207	266	-60	1248826	34596

Table 6-8: Pile head displacements and forces in very stiff clay, and computed stiffnesses for Bridge III abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0002	-0.0209	159	-70	753555	40483
HP10x57	0.0002	-0.0183	175	-71	916230	46361
HP12x53	0.0002	-0.0173	190	-70	1049724	48776
HP12x63	0.0002	-0.0160	200	-70	1169591	52610
HP12x74	0.0002	-0.0148	210	-70	1296296	56815
HP14x73	0.0002	-0.0141	226	-69	1458065	58821
HP14x89	0.0001	-0.0128	239	-69	1659722	64544

Table 6-9: Pile head displacements and forces in loose sand, and computed stiffnesses for Bridge III abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0006	-0.0662	177	-32	295000	5883
HP10x57	0.0005	-0.0597	203	-35	370438	6978
HP12x53	0.0005	-0.0546	227	-36	449505	7982
HP12x63	0.0005	-0.0511	243	-37	509434	8772
HP12x74	0.0004	-0.0476	260	-38	580357	9664
HP14x73	0.0004	-0.0435	282	-39	682809	10870
HP14x89	0.0004	-0.0396	302	-40	790576	12187

Table 6-10: Pile head displacements and forces in medium sand, and computed stiffnesses for Bridge III abutments

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.0005	-0.0488	185	-46	411111	11199
HP10x57	0.0004	-0.0432	209	-47	516049	13170
HP12x53	0.0004	-0.0392	229	-49	617251	14871
HP12x63	0.0003	-0.0364	243	-49	698276	16283
HP12x74	0.0003	-0.0336	257	-50	790769	17869
HP14x73	0.0003	-0.0305	276	-51	923077	19892
HP14x89	0.0003	-0.0276	293	-51	1065455	22219

Table 6-11: Pile head displacements and forces in dense sand, and computed stiffnesses for Bridge III abutments.

Pile Section	Θ (rad)	δ (in)	M (kip-ft)	V (kip)	K_{Θ} , M/ Θ (ft-kip/rad)	K_{δ} , V/ δ (kip/ft)
HP10x42	0.000343	-0.03626	184	-56	536443	18383
HP10x57	0.0003	-0.0317	205	-57	669935	21624
HP12x53	0.0003	-0.0302	226	-59	846442	23586
HP12x63	0.0003	-0.0264	234	-58	896552	26546
HP12x74	0.0002	-0.0242	246	-59	1008197	29142
HP14x73	0.0002	-0.0220	262	-59	1169643	32238
HP14x89	0.0002	-0.0197	277	-59	1344660	36046

For the physical parameters of bridges similar to Bridge III, the steel pile foundation can be represented by a single rotational and linear spring where the values can be based on the data obtained by the result of this parametric study and given in the Table 6-5 through Table 6-11.

Table 6-12 : Bridge III physical properties.

Bridge Type	Slab
Bridge Length (ft)	50
Bridge Width (ft)	46
Slab Thickness (in)	26.4
Abutment Height (ft)	10
Abutment Thickness (ft)	2.5
Number of Piles	10

6.3.3 Depth of Inflection Point

The pile end fixity values related to the cases of Bridge III were obtained based on the location of inflection points and provided in the following tables and figures.

Table 6-13 : Depths of inflection points for the piles of Bridge III.

Depth of Inflection Point , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	8.92	6.75	4.93	3.8	7.52	5.86	4.79
HP10x57	9.79	7.37	5.41	4.17	8.09	6.35	5.17
HP12x53	10.65	8.05	5.94	4.63	8.69	6.81	5.58
HP12x63	11.18	8.47	6.26	4.85	9.03	7.08	5.8
HP12x74	11.77	8.86	6.58	5.1	9.45	7.41	6.03
HP14x73	12.71	9.61	7.17	5.6	9.99	7.87	6.47
HP14x89	13.58	10.11	7.38	5.92	10.5	8.27	6.78

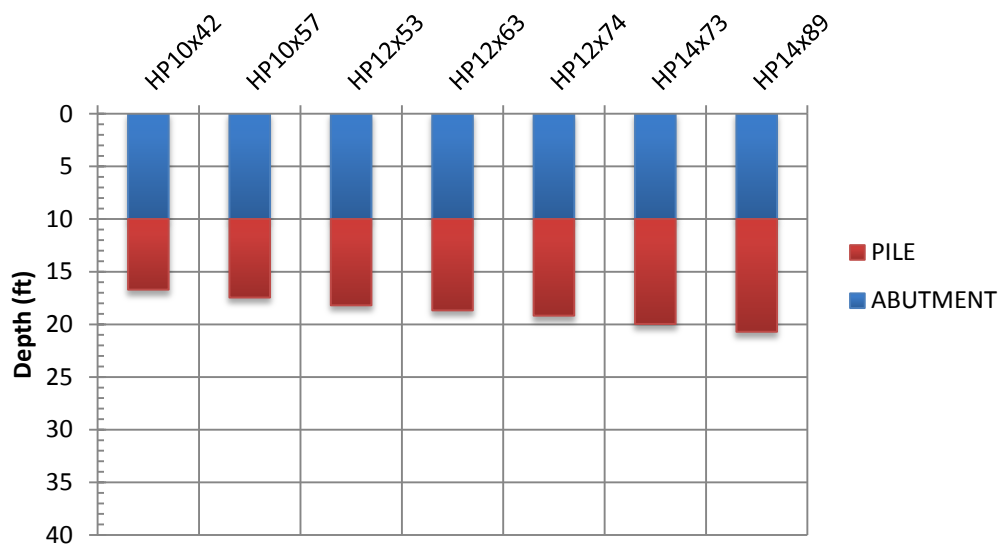


Figure 6-11 : Depth of inflection points of Bridge III piles in soft clay.

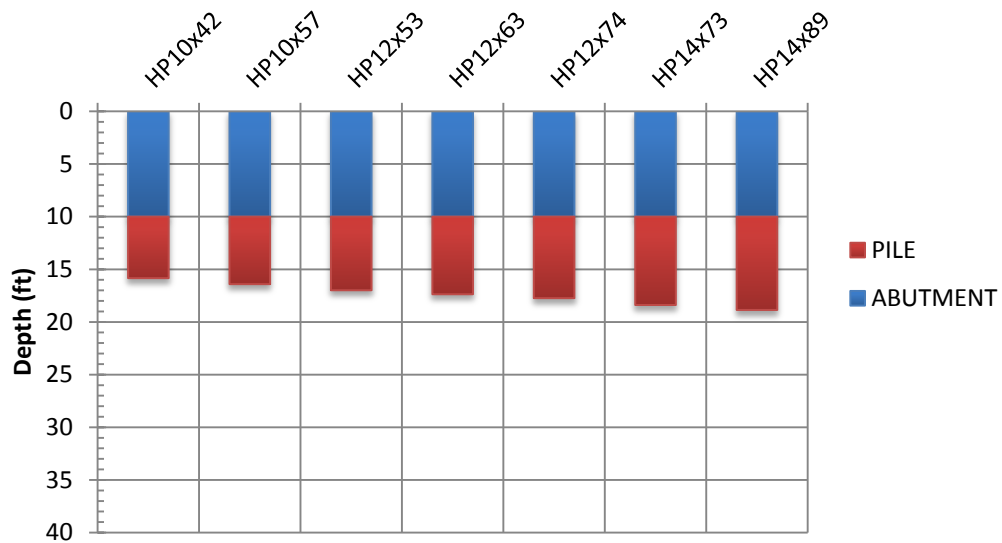


Figure 6-12 : Depth of inflection points of Bridge III piles in medium clay.

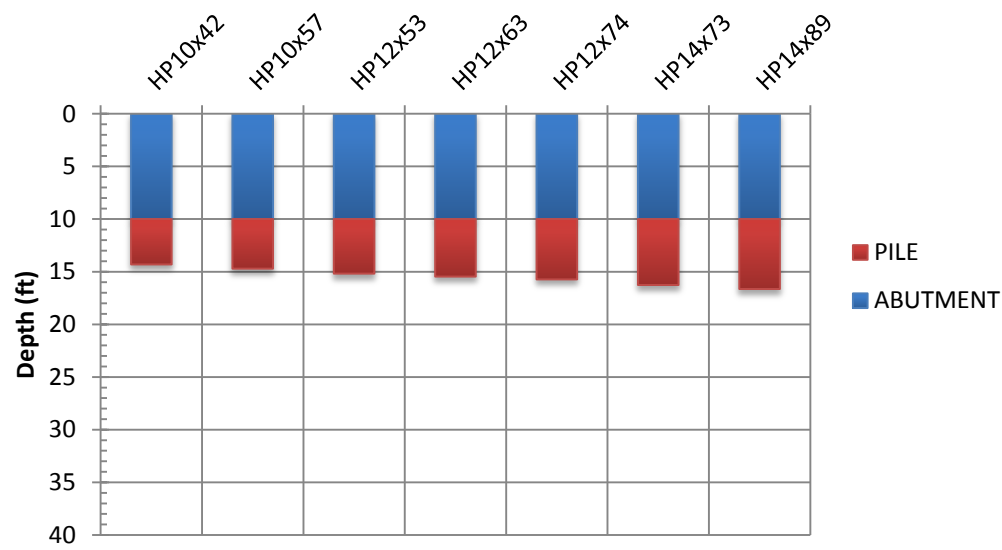


Figure 6-13 : Depth of inflection points of Bridge III piles in stiff clay.

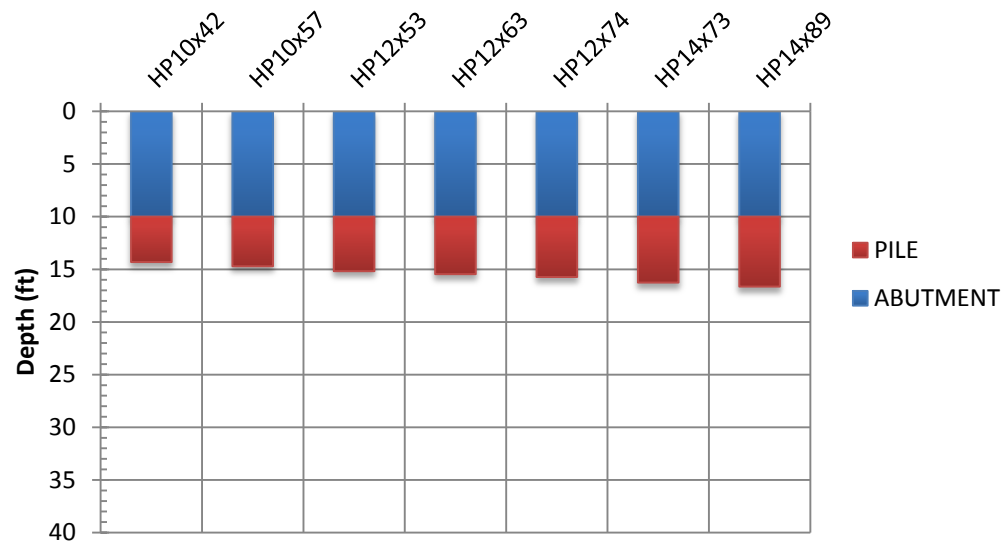


Figure 6-14 : Depth of inflection points of Bridge III piles in very-stiff clay.

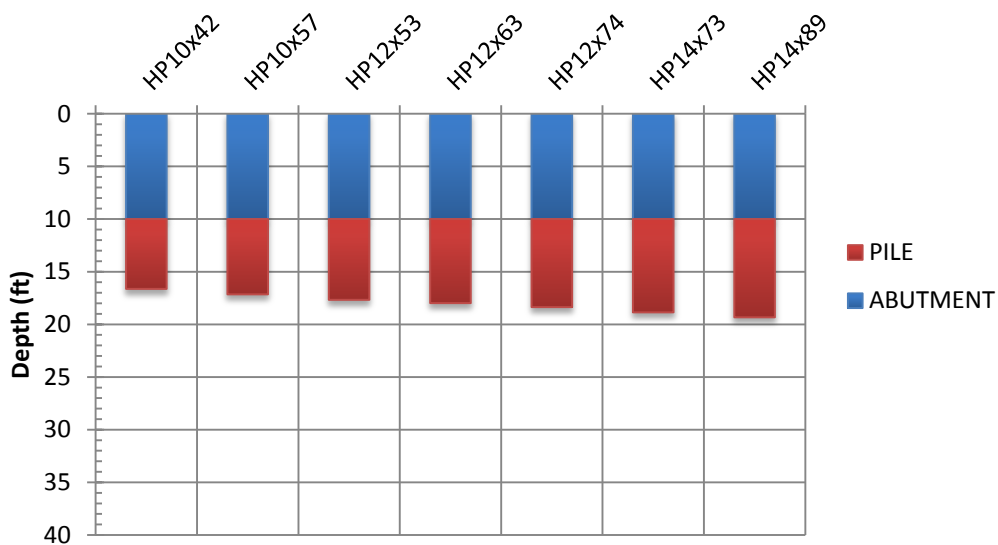


Figure 6-15 : Depth of inflection points of Bridge III piles in loose sand.

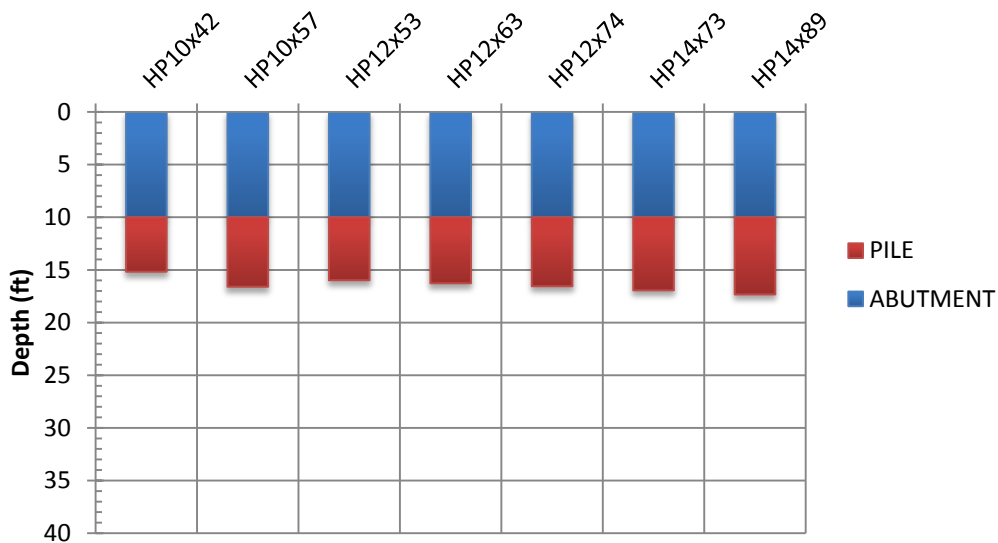


Figure 6-16 : Depth of inflection points of Bridge III piles in medium sand.

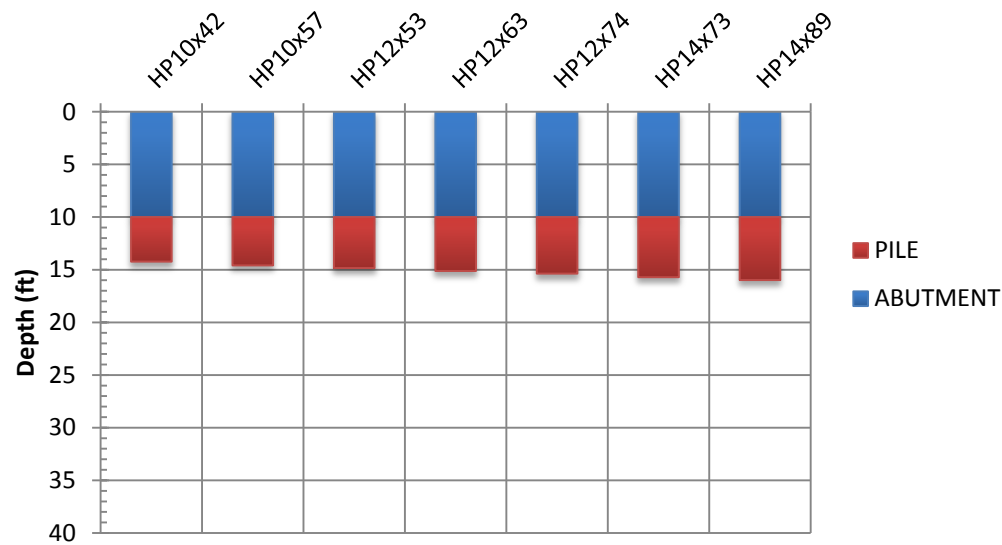


Figure 6-17 : Depth of inflection points of Bridge III piles in dense sand.

6.3.4 Depth of Zero Rotation

Table 6-14 and

Table 6-15 present the first and second point of zero rotation along the depth of the piles of Bridge II. Figure 6-18 to Figure 6-21 illustrate the increase and decrease of the point of zero rotation relative to the soil types and pile sections.

Table 6-14 : Depths of first zero rotation.

Point of Zero Rotation , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	2.1	1.4	0.9	0.7	1.7	1.2	0.9
HP10x57	2.4	1.6	1.1	0.8	1.9	1.5	1
HP12x53	2.7	1.9	1.3	0.9	2.1	1.6	1.1
HP12x63	2.9	2	1.4	1	2.3	1.7	1.3
HP12x74	3.2	2.2	1.5	1.1	2.4	1.8	1.4
HP14x73	3.6	2.5	1.7	1.3	2.7	2	1.5
HP14x89	3.9	2.7	1.8	1.4	2.9	2.1	1.7

Table 6-15 : Depths of second zero rotation.

Point of Zero Rotation , ft							
Pile Section	Soft Clay	Medium Clay	Stiff Clay	Very Stiff Clay	Loose Sand	Medium Sand	Dense Sand
HP10x42	34	29	25	22	30	26	24
HP10x57	35	30	26	23	31	27	25
HP12x53	38	32	27	24	33	28	25
HP12x63	40	33	28	25	33	29	26
HP12x74	40	33	29	25	34	30	27
HP14x73	40	35	30	27	35	31	28
HP14x89	40	36	31	28	36	32	28

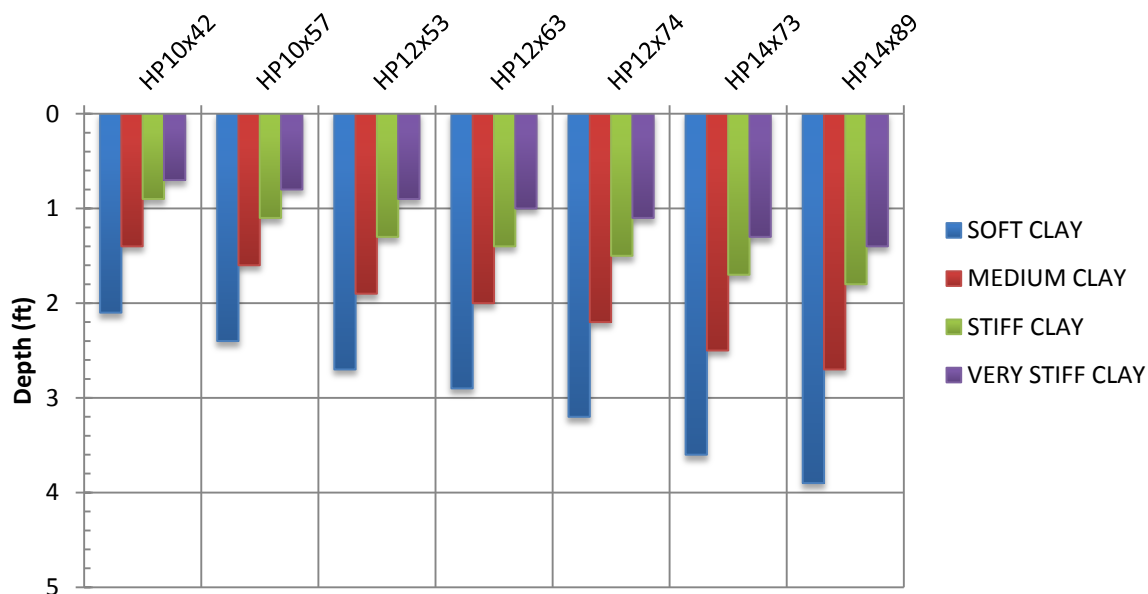


Figure 6-18 : Chart of the depths of first zero rotation in clay soil.

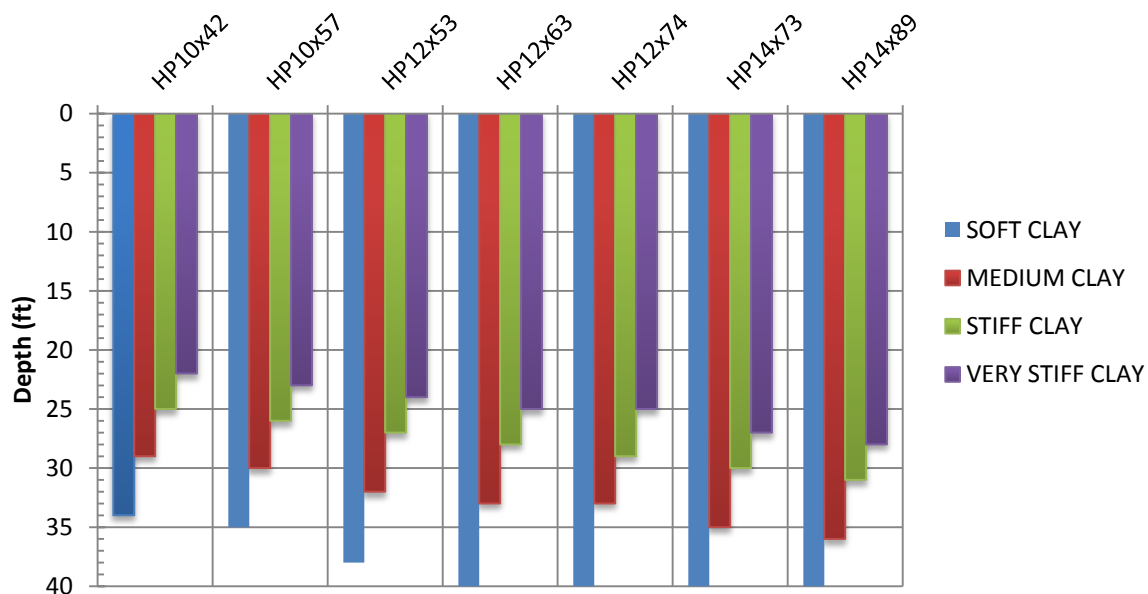


Figure 6-19 : Chart of the depths of second zero rotation in clay soil.

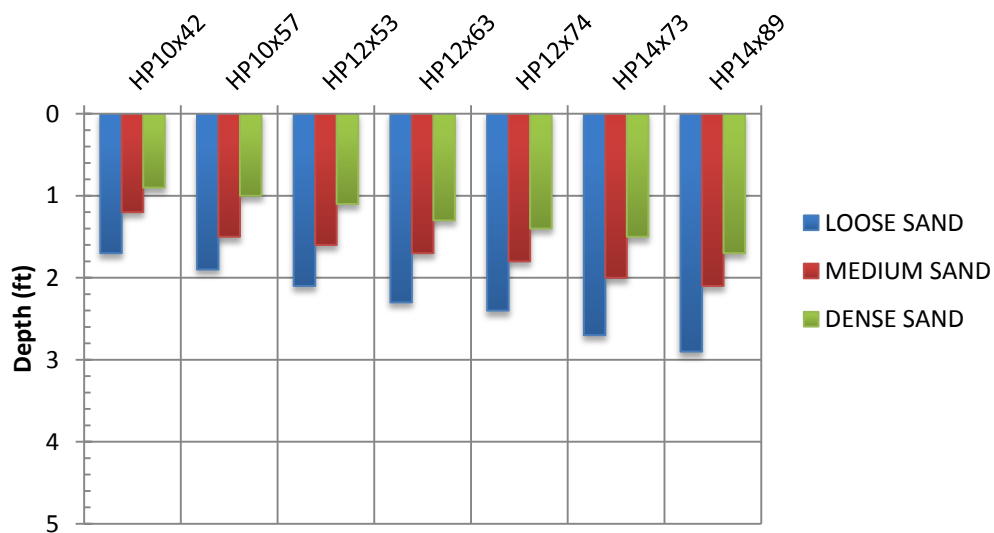


Figure 6-20 : Chart of the depths of first zero rotation in sand soil.

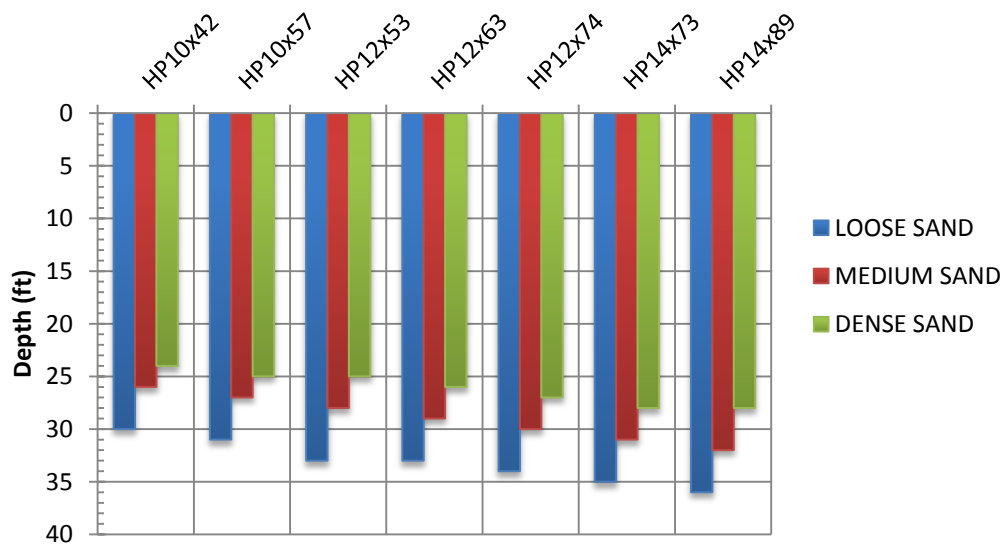


Figure 6-21 : Chart of the depths of second zero rotation in sand soil.

7. SUMMARY, CONCLUSIONS and RECOMMENDATIONS

7.1 Summary

In this study, the effects of integral abutments compared to conventional bridges were investigated through the study of pile behavior and element forces. Three types of integral bridges were analyzed with 2-d frame modeling techniques using finite element analysis and a computer program SAP2000 V14. A wide range of pile sizes and soil types were considered in the study which resulted in about two hundred and fifty different cases.

For the same pile section, with the stiffer soils, the depths of inflection points and zero rotation decrease, but for the same soil type with the stiffer pile sections, the depths of inflection points and zero rotation increase. The inflection point location position is often considered as an effective rigid support location in the analysis of piles.

7.2 Conclusions

When using an integral abutment, soil and pile stiffnesses have direct and significant effects on the magnitude of positive mid-span moments in the superstructure of bridges. These effects are particularly significant and beneficial in reducing moments for short span single span bridges. The influence of an integral abutment and its substructure stiffnesses on the superstructure decreases, however, in a continuous bridge with more than one span, and particularly if the spans are long ($>60\text{ft.}$). This mainly stems from the high flexural rigidity of the intermediate support and the high flexural rigidity in girders with long span bridges.

Using a 30ft. pile length, in lieu of much longer piles, is satisfactory relative to obtaining beneficial effects of reducing superstructure positive moments with integral abutments. It was observed that a longer pile length does not contribute to the flexural stiffness of integral abutment substructures. In this study a 30 ft. length was used as the least pile length. Longer piles demonstrated that in effect the pile became “fixed” in the soil at a depth less than 30ft. so the extra length did not improve behavior.

Excluding the backfill effect behind the concrete abutment in an analysis is conservative in terms of predicting positive moment reduction created in the superstructure by use of integral abutments. Inclusion of the backfill effect into analysis makes the integral abutment stiffer. In this case abutment moments are higher and superstructure span moments are lower compared to the integral abutments without backfill effect.

7.3 Recommendation

In analysis of bridges with integral abutments, with characteristics (span lengths and materials) similar to those examined here, the effect of the integral abutment should be simulated by supporting the bridge superstructure on translational and rotational springs to simplify the analysis. The necessary spring constants for such simulations have been identified in this study. Pile behavior is approximated by these rotational and linear springs as shown for the three bridges considered. These approximations can be used for quick analysis of integral abutment bridges that have similar relative flexural stiffness distribution.

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A. APPENDIX A

SPRING STIFFNESS TABLES

Table A-1: Spring Stiffnesses for HP10x42 Section, k (k/ft).

Depth below ground surface (ft)	Spring Stiffness for HP10x42 Section, k (k/ft)			
	Soft Clay Soil Springs	Medium Clay Soil Springs	Stiff Clay Soil Springs	Very-stiff Clay Soil Springs
0	0	0	0	0
1	29	104	314	811
2	36	117	372	1173
3	43	131	430	1348
4	50	145	488	1348
5	57	158	545	1348
6	64	172	603	1348
7	67	186	661	1348
8	67	199	718	1348
9	67	213	770	1348
10	67	227	770	1348
11	67	241	770	1348
12	67	254	770	1348
13	67	268	770	1348
14	67	270	770	1348
15	67	270	770	1348
16	67	270	770	1348
17	67	270	770	1348
18	67	270	770	1348
19	67	270	770	1348
20	67	270	770	1348
21	67	270	770	1348
22	67	270	770	1348
23	67	270	770	1348
24	67	270	770	1348
25	67	270	770	1348
26	67	270	770	1348
27	67	270	770	1348
28	67	270	770	1348
29	67	270	770	1348
30	202	809	2311	4044
35	337	1348	3851	6739
40	337	1348	3851	6739
45	337	1348	3851	6739
50	337	1348	3851	6739

Table A-2: Spring Stiffnesses for HP10x57 Section, k (k/ft).

Depth below ground surface (ft)	Spring Stiffness for HP10x57 Section, k (k/ft)			
	Soft Clay Soil Springs	Medium Clay Soil Springs	Stiff Clay Soil Springs	Very-stiff Clay Soil Springs
0	0	0	0	0
1	29	103	314	808
2	36	117	371	1166
3	43	131	428	1348
4	50	144	486	1348
5	56	158	543	1348
6	63	172	600	1348
7	67	185	657	1348
8	67	199	714	1348
9	67	212	770	1348
10	67	226	770	1348
11	67	240	770	1348
12	67	253	770	1348
13	67	267	770	1348
14	67	270	770	1348
15	67	270	770	1348
16	67	270	770	1348
17	67	270	770	1348
18	67	270	770	1348
19	67	270	770	1348
20	67	270	770	1348
21	67	270	770	1348
22	67	270	770	1348
23	67	270	770	1348
24	67	270	770	1348
25	67	270	770	1348
26	67	270	770	1348
27	67	270	770	1348
28	67	270	770	1348
29	67	270	770	1348
30	202	809	2311	4044
35	337	1348	3851	6739
40	337	1348	3851	6739
45	337	1348	3851	6739
50	337	1348	3851	6739

Table A-3: Spring Stiffnesses for HP12x53 Section, k (k/ft).

Depth below ground surface (ft)	Spring Stiffness for HP12x53 Section, k (k/ft)			
	Soft Clay Soil Springs	Medium Clay Soil Springs	Stiff Clay Soil Springs	Very-stiff Clay Soil Springs
0	0	0	0	0
1	29	102	306	755
2	35	114	356	1060
3	41	127	406	1348
4	47	139	455	1348
5	53	151	505	1348
6	59	164	555	1348
7	65	176	604	1348
8	67	188	654	1348
9	67	200	704	1348
10	67	213	753	1348
11	67	225	770	1348
12	67	237	770	1348
13	67	250	770	1348
14	67	262	770	1348
15	67	270	770	1348
16	67	270	770	1348
17	67	270	770	1348
18	67	270	770	1348
19	67	270	770	1348
20	67	270	770	1348
21	67	270	770	1348
22	67	270	770	1348
23	67	270	770	1348
24	67	270	770	1348
25	67	270	770	1348
26	67	270	770	1348
27	67	270	770	1348
28	67	270	770	1348
29	67	270	770	1348
30	202	809	2311	4044
35	337	1348	3851	6739
40	337	1348	3851	6739
45	337	1348	3851	6739
50	337	1348	3851	6739

Table A-4: Spring Stiffnesses for HP12x63 Section, k (k/ft).

Depth below ground surface (ft)	Spring Stiffness for HP12x63 Section, k (k/ft)			
	Soft Clay Soil Springs	Medium Clay Soil Springs	Stiff Clay Soil Springs	Very-stiff Clay Soil Springs
0	0	0	0	0
1	29	102	306	752
2	35	114	355	1055
3	41	127	405	1348
4	47	139	454	1348
5	53	151	503	1348
6	59	163	552	1348
7	65	175	602	1348
8	67	188	651	1348
9	67	200	700	1348
10	67	212	750	1348
11	67	224	770	1348
12	67	237	770	1348
13	67	249	770	1348
14	67	261	770	1348
15	67	270	770	1348
16	67	270	770	1348
17	67	270	770	1348
18	67	270	770	1348
19	67	270	770	1348
20	67	270	770	1348
21	67	270	770	1348
22	67	270	770	1348
23	67	270	770	1348
24	67	270	770	1348
25	67	270	770	1348
26	67	270	770	1348
27	67	270	770	1348
28	67	270	770	1348
29	67	270	770	1348
30	202	809	2311	4044
35	337	1348	3851	6739
40	337	1348	3851	6739
45	337	1348	3851	6739
50	337	1348	3851	6739

Table A-5: Spring Stiffnesses for HP12x74 Section, k (k/ft).

Depth below ground surface (ft)	Spring Stiffness for HP12x74 Section, k (k/ft)			
	Soft Clay Soil Springs	Medium Clay Soil Springs	Stiff Clay Soil Springs	Very-stiff Clay Soil Springs
0	0	0	0	0
1	29	102	306	750
2	35	114	355	1050
3	41	126	404	1348
4	47	139	453	1348
5	53	151	501	1348
6	59	163	550	1348
7	65	175	599	1348
8	67	187	648	1348
9	67	199	697	1348
10	67	212	746	1348
11	67	224	770	1348
12	67	236	770	1348
13	67	248	770	1348
14	67	260	770	1348
15	67	270	770	1348
16	67	270	770	1348
17	67	270	770	1348
18	67	270	770	1348
19	67	270	770	1348
20	67	270	770	1348
21	67	270	770	1348
22	67	270	770	1348
23	67	270	770	1348
24	67	270	770	1348
25	67	270	770	1348
26	67	270	770	1348
27	67	270	770	1348
28	67	270	770	1348
29	67	270	770	1348
30	202	809	2311	4044
35	337	1348	3851	6739
40	337	1348	3851	6739
45	337	1348	3851	6739
50	337	1348	3851	6739

Table A-6: Spring Stiffnesses for HP14x73 Section, k (k/ft).

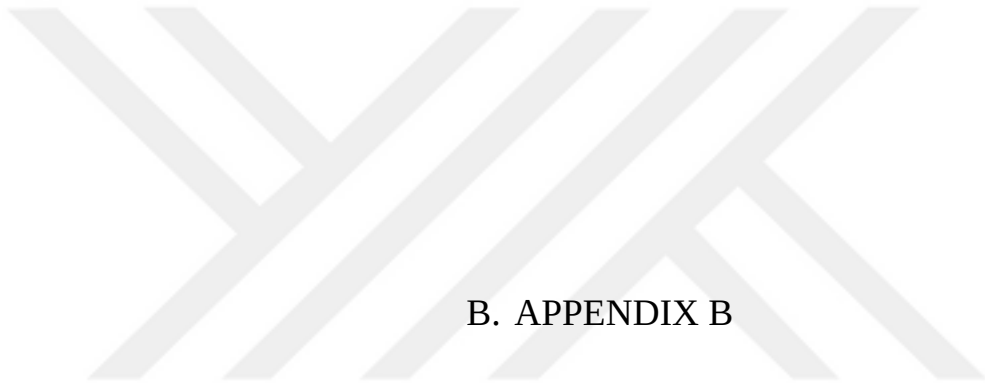
Depth below ground surface (ft)	Spring Stiffness for HP14x73 Section, k (k/ft)			
	Soft Clay Soil Springs	Medium Clay Soil Springs	Stiff Clay Soil Springs	Very-stiff Clay Soil Springs
0	0	0	0	0
1	28	101	299	701
2	33	112	341	954
3	39	123	383	1206
4	44	134	425	1348
5	50	145	467	1348
6	55	156	509	1348
7	61	167	551	1348
8	66	177	593	1348
9	67	188	635	1348
10	67	199	677	1348
11	67	210	719	1348
12	67	221	761	1348
13	67	232	770	1348
14	67	243	770	1348
15	67	254	770	1348
16	67	265	770	1348
17	67	270	770	1348
18	67	270	770	1348
19	67	270	770	1348
20	67	270	770	1348
21	67	270	770	1348
22	67	270	770	1348
23	67	270	770	1348
24	67	270	770	1348
25	67	270	770	1348
26	67	270	770	1348
27	67	270	770	1348
28	67	270	770	1348
29	67	270	770	1348
30	202	809	2311	4044
35	337	1348	3851	6739
40	337	1348	3851	6739
45	337	1348	3851	6739
50	337	1348	3851	6739

Table A-7: Spring Stiffnesses for HP14x89 Section, k (k/ft).

Depth below ground surface (ft)	Spring Stiffness for HP14x89 Section, k (k/ft)			
	Soft Clay Soil Springs	Medium Clay Soil Springs	Stiff Clay Soil Springs	Very-stiff Clay Soil Springs
0	0	0	0	0
1	28	101	299	700
2	33	112	340	950
3	39	123	382	1201
4	44	134	424	1348
5	50	144	466	1348
6	55	155	507	1348
7	61	166	549	1348
8	66	177	591	1348
9	67	188	633	1348
10	67	199	675	1348
11	67	210	716	1348
12	67	221	758	1348
13	67	232	770	1348
14	67	243	770	1348
15	67	254	770	1348
16	67	264	770	1348
17	67	270	770	1348
18	67	270	770	1348
19	67	270	770	1348
20	67	270	770	1348
21	67	270	770	1348
22	67	270	770	1348
23	67	270	770	1348
24	67	270	770	1348
25	67	270	770	1348
26	67	270	770	1348
27	67	270	770	1348
28	67	270	770	1348
29	67	270	770	1348
30	202	809	2311	4044
35	337	1348	3851	6739
40	337	1348	3851	6739
45	337	1348	3851	6739
50	337	1348	3851	6739

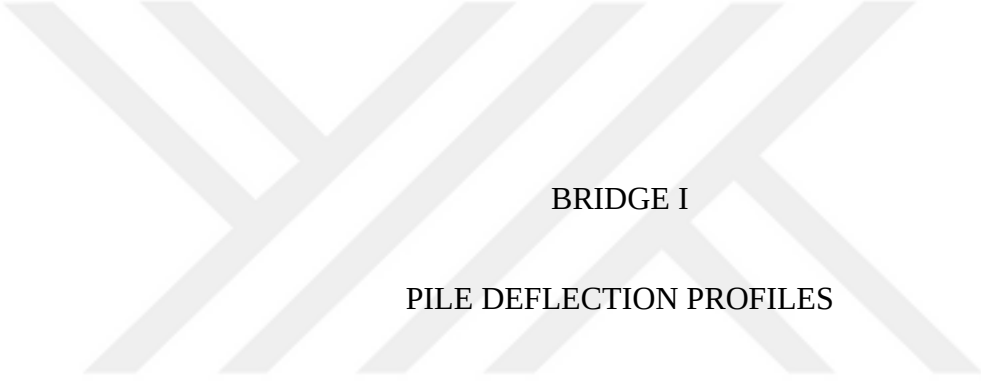
Table A-8: Spring Stiffnesses in Sand, k (k/ft).

Depth below ground surface (ft)	Spring Stiffness, k (k/ft)		
	Loose Sand Soil Springs	Medium Sand Soil Springs	Dense Sand Soil Springs
0	0	0	0
1	12	80	200
2	16	107	267
3	53	160	400
4	71	213	533
5	89	267	667
6	107	320	800
7	124	373	933
8	142	427	1067
9	160	480	1200
10	178	533	1333
11	196	587	1467
12	213	640	1600
13	231	693	1733
14	249	747	1867
15	267	800	2000
16	284	853	2133
17	302	907	2267
18	320	960	2400
19	338	1013	2533
20	356	1067	2667
21	373	1120	2800
22	391	1173	2933
23	409	1227	3067
24	427	1280	3200
25	444	1333	3333
26	462	1387	3467
27	480	1440	3600
28	498	1493	3733
29	516	1547	3867
30	1600	4800	12000
31	3111	9333	23333
32	3556	10667	26667
33	4000	12000	30000
34	4444	13333	33333



B. APPENDIX B

PILE PROFILES OF BRIDGES



BRIDGE I

PILE DEFLECTION PROFILES

PILE ROTATION PROFILES

PILE MOMENT PROFILES

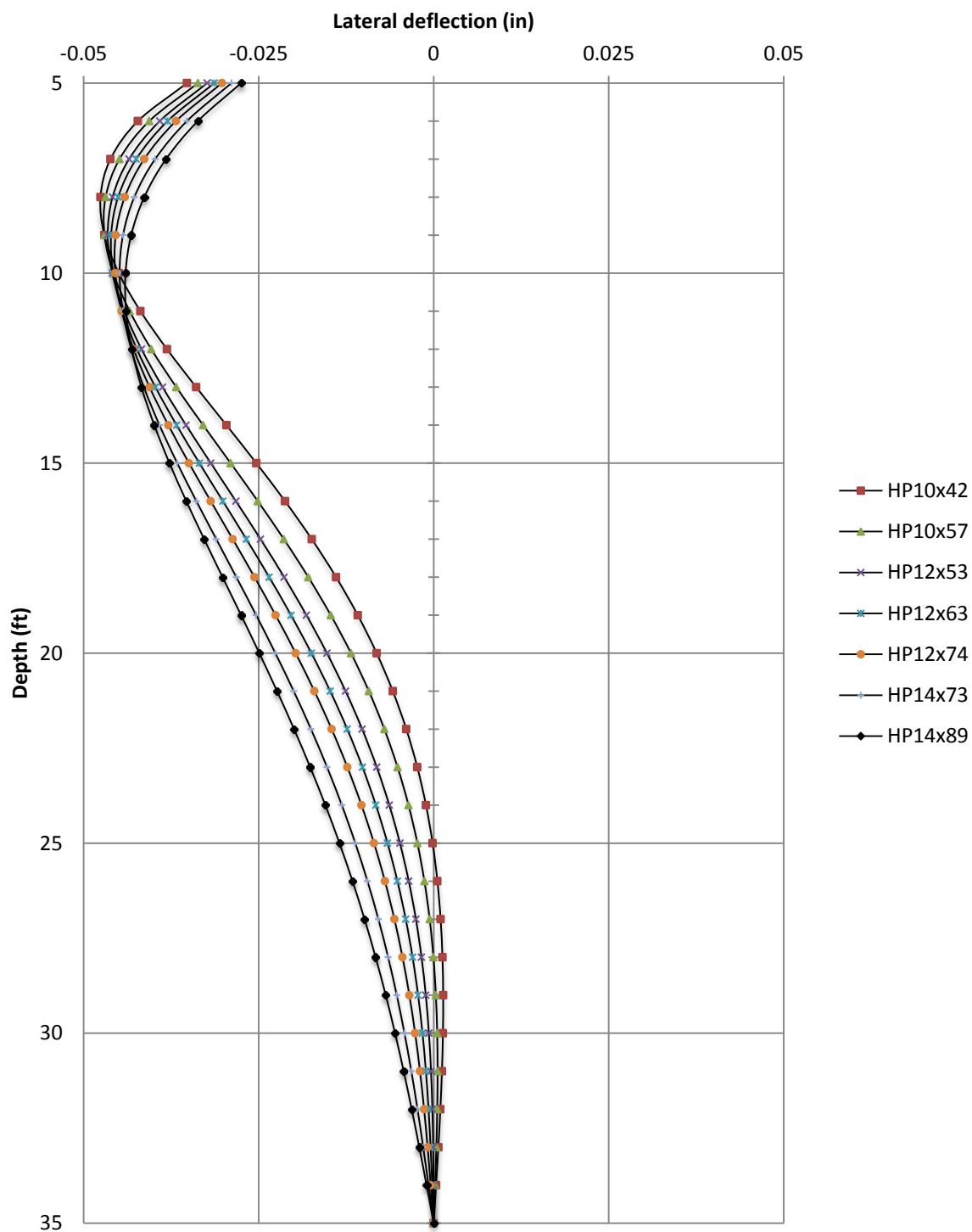


Figure B-1: Lateral deflection of piles in soft clay under Truck LL

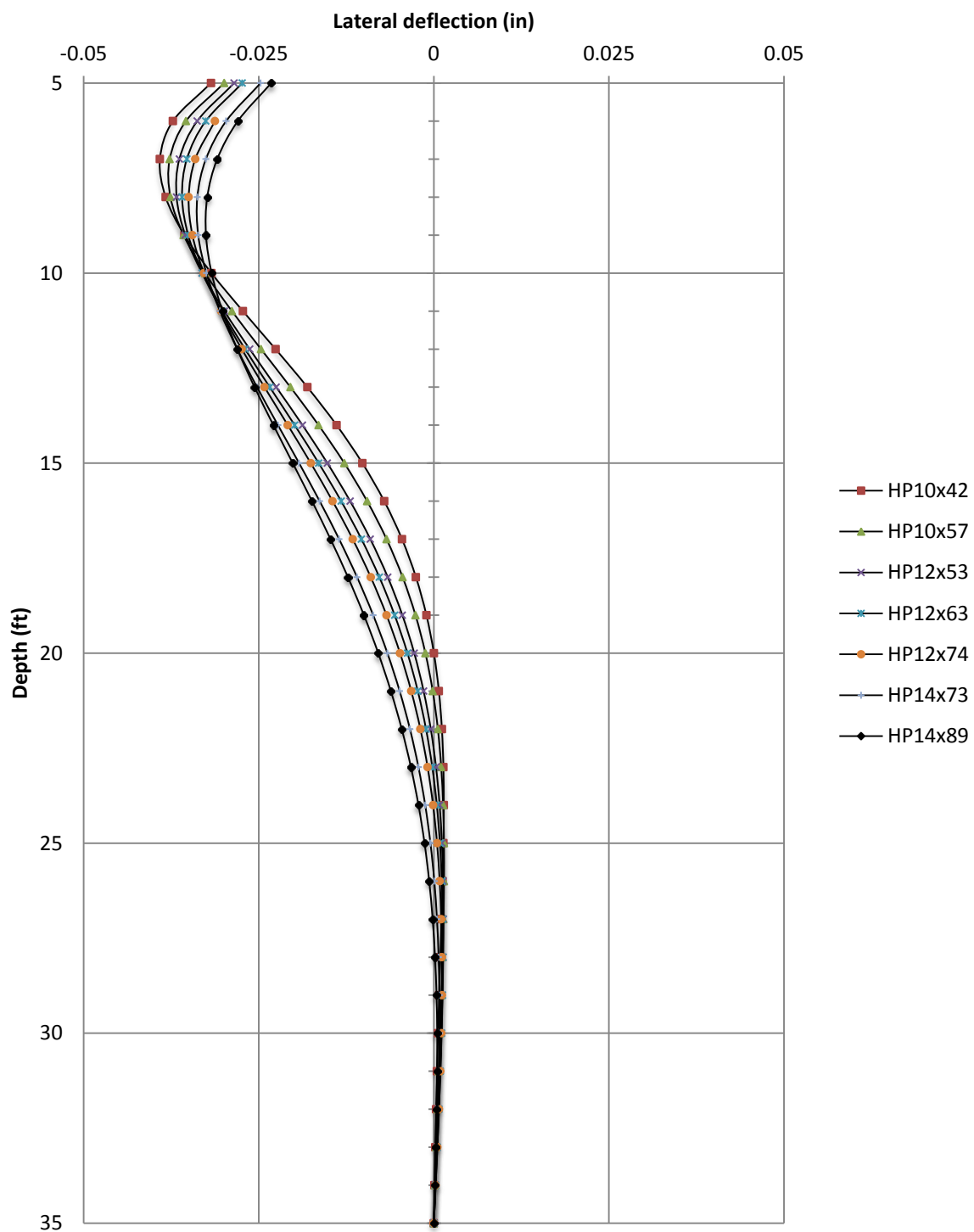


Figure B-1 : Lateral deflection of piles in medium clay under Truck LL

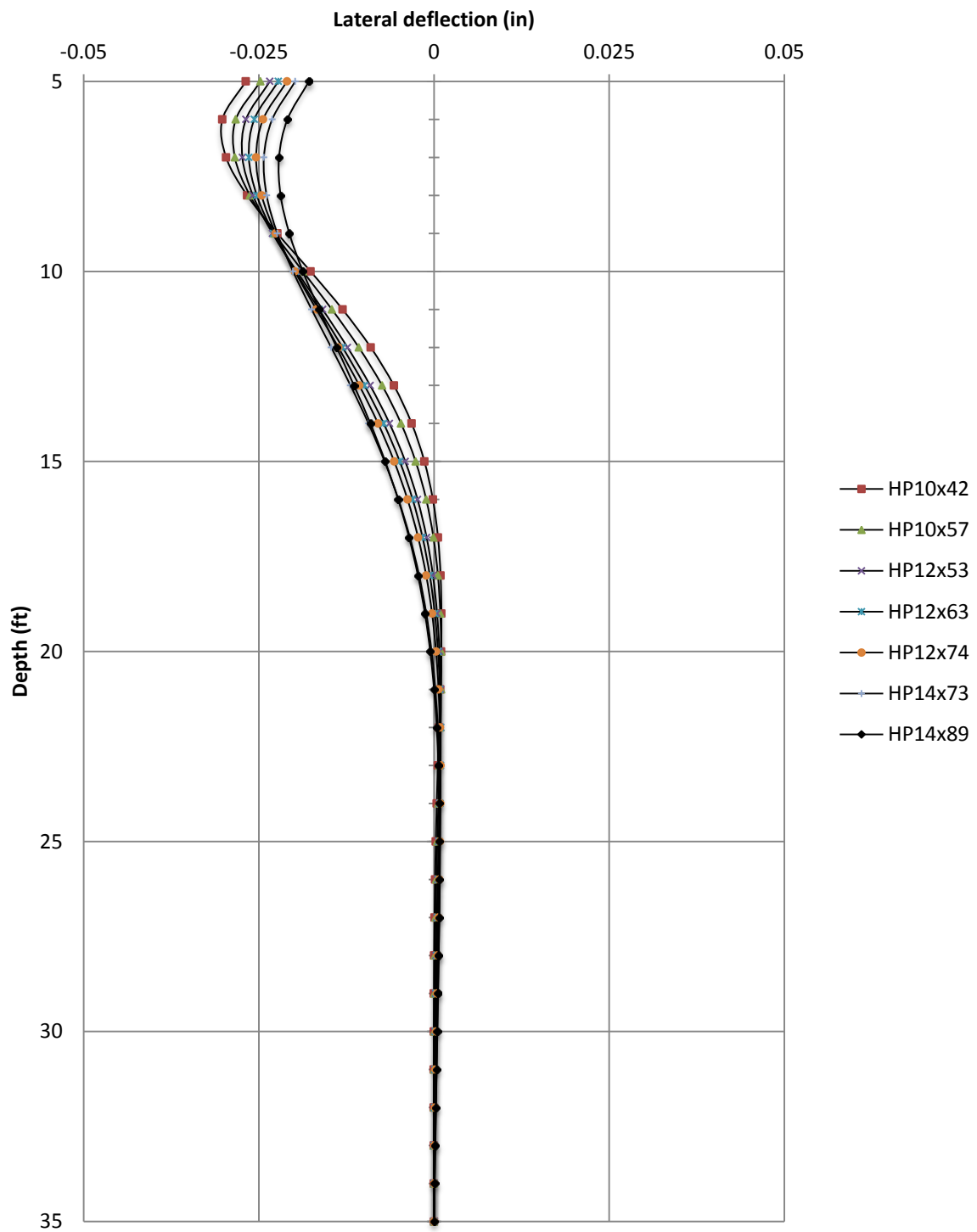


Figure B-2 : Lateral deflection of piles in stiff clay under Truck LL

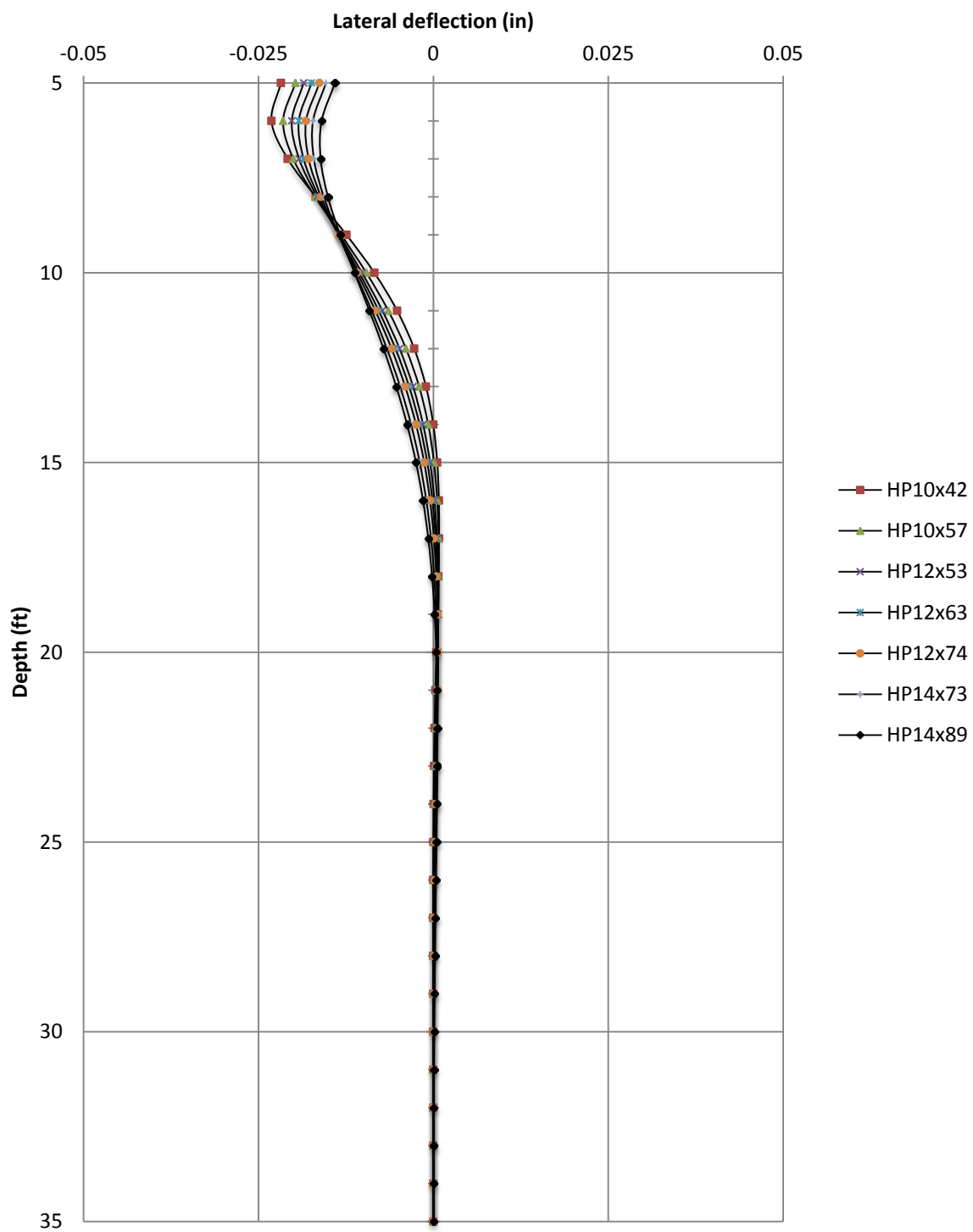


Figure B-3 : Lateral deflection of piles in very stiff clay under Truck LL

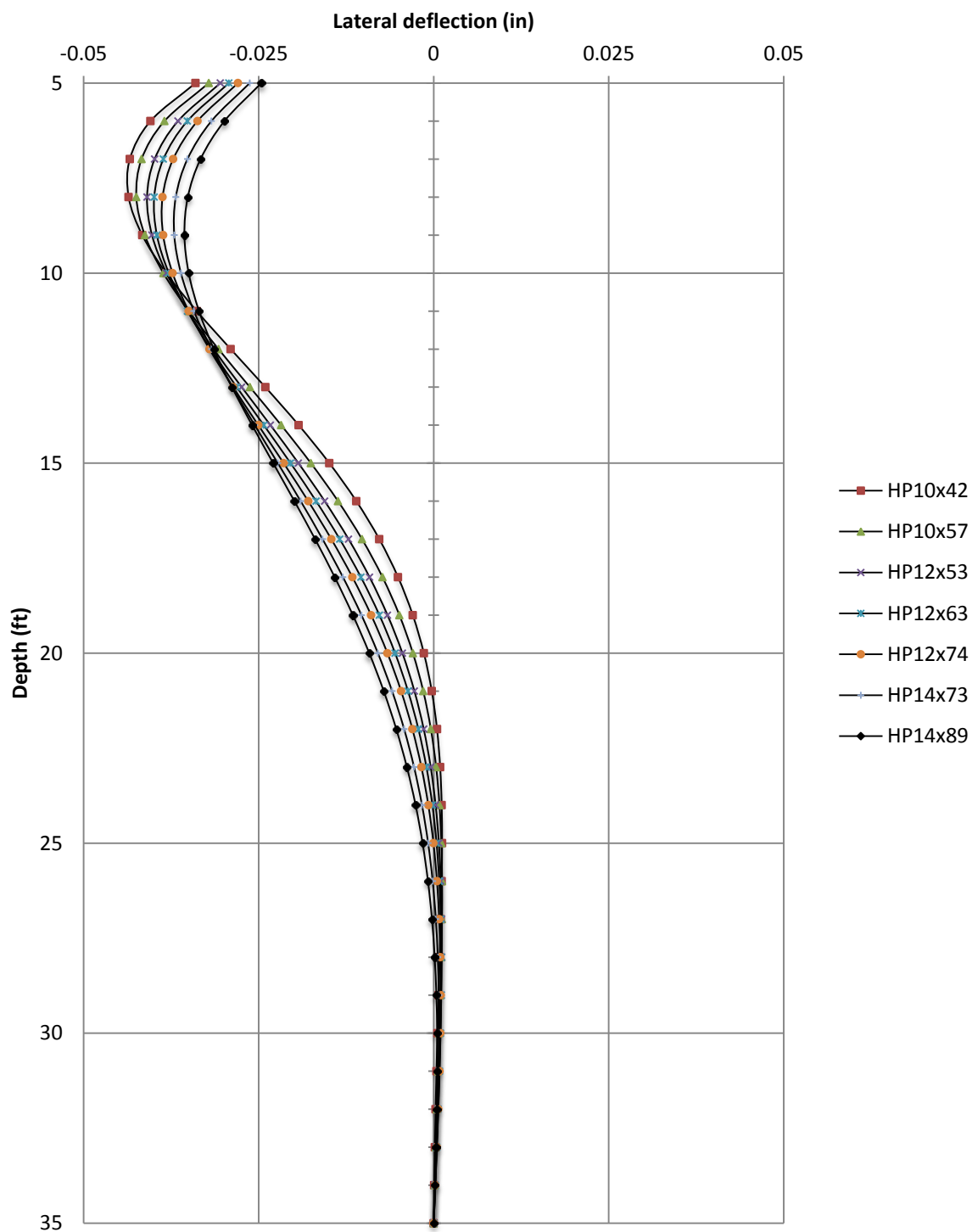


Figure B-4 : Lateral deflection of piles in loose sand under Truck LL

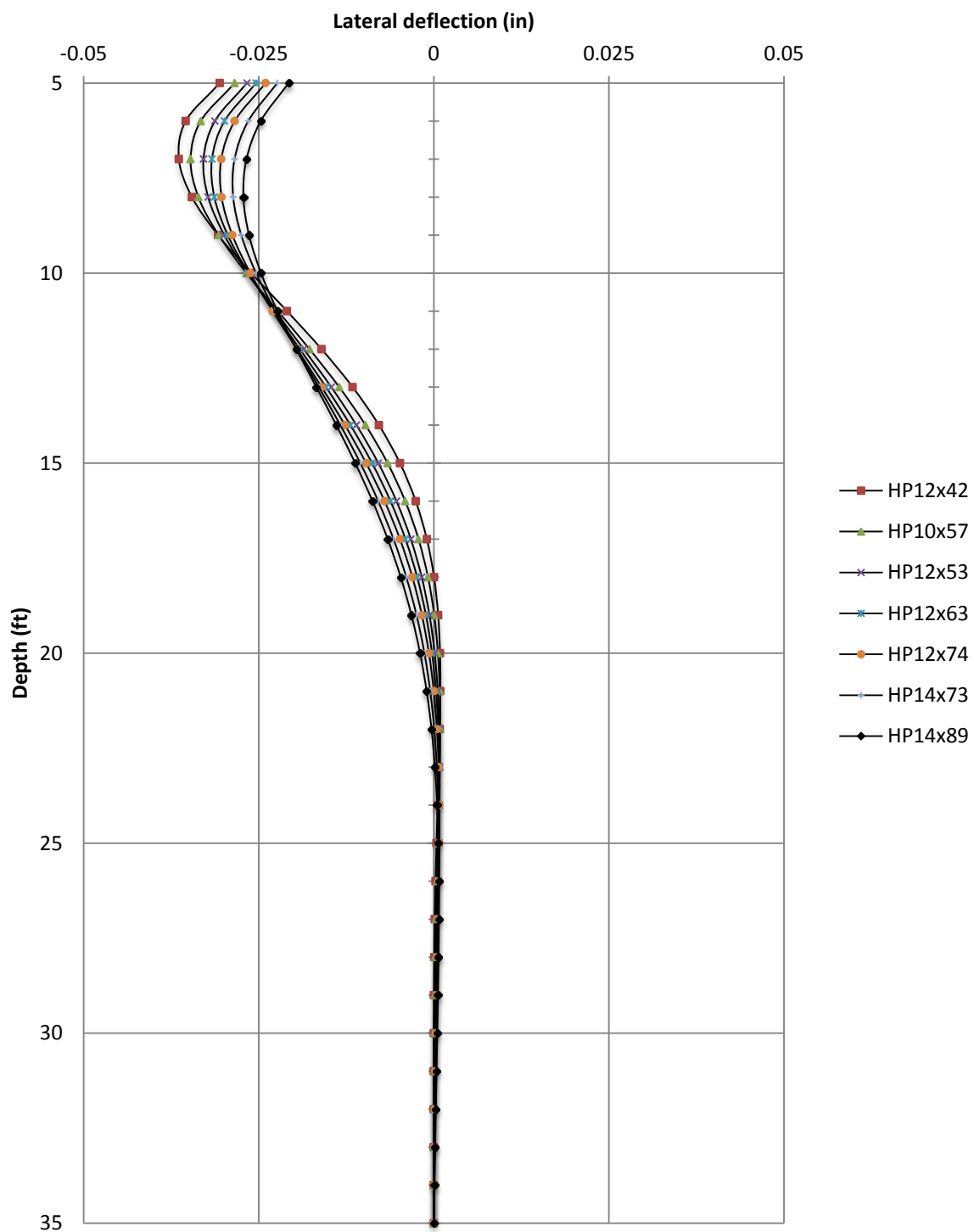


Figure B-5 : Lateral deflection of piles in medium sand under Truck LL

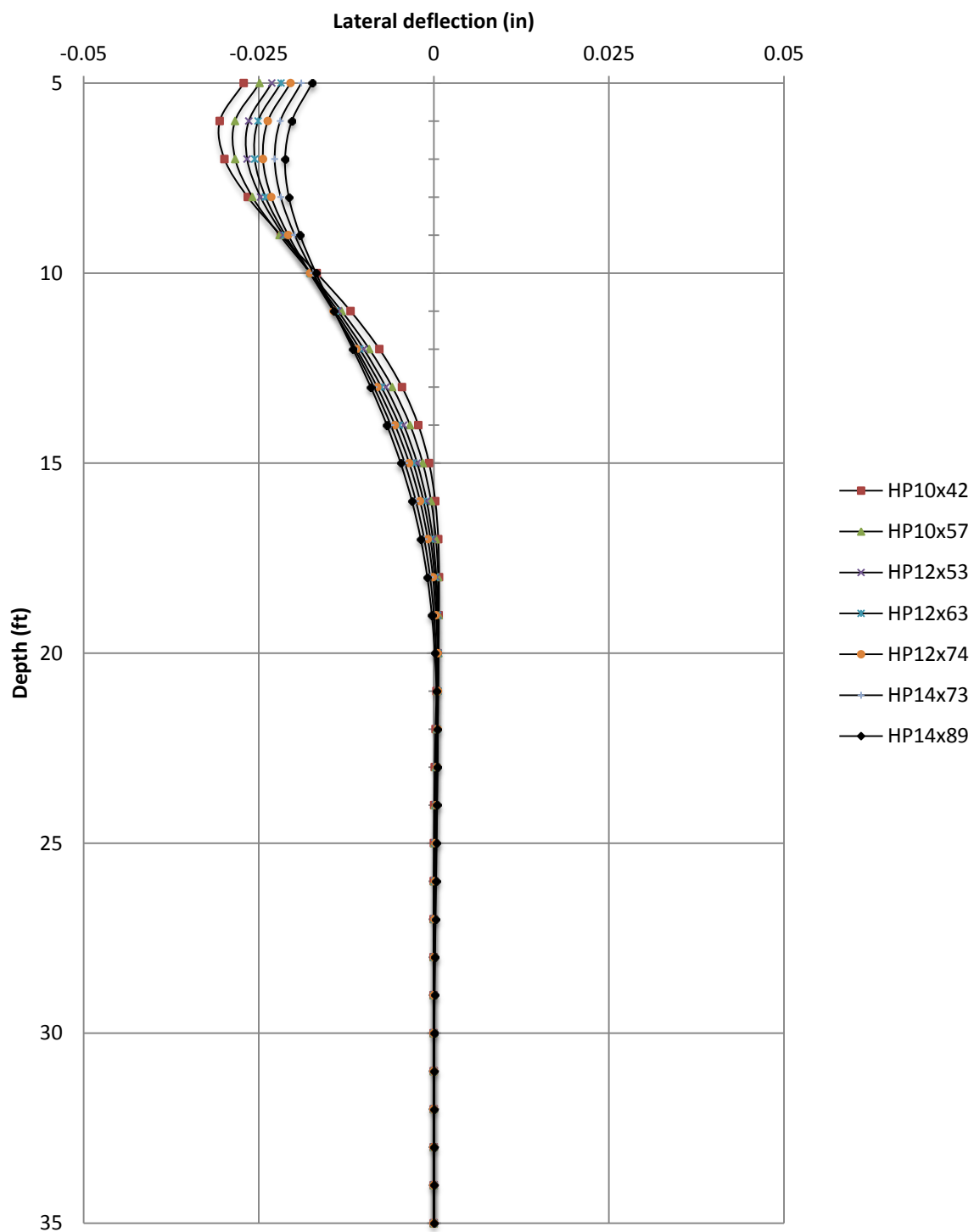


Figure B-6 : Lateral deflection of piles in dense sand under Truck LL

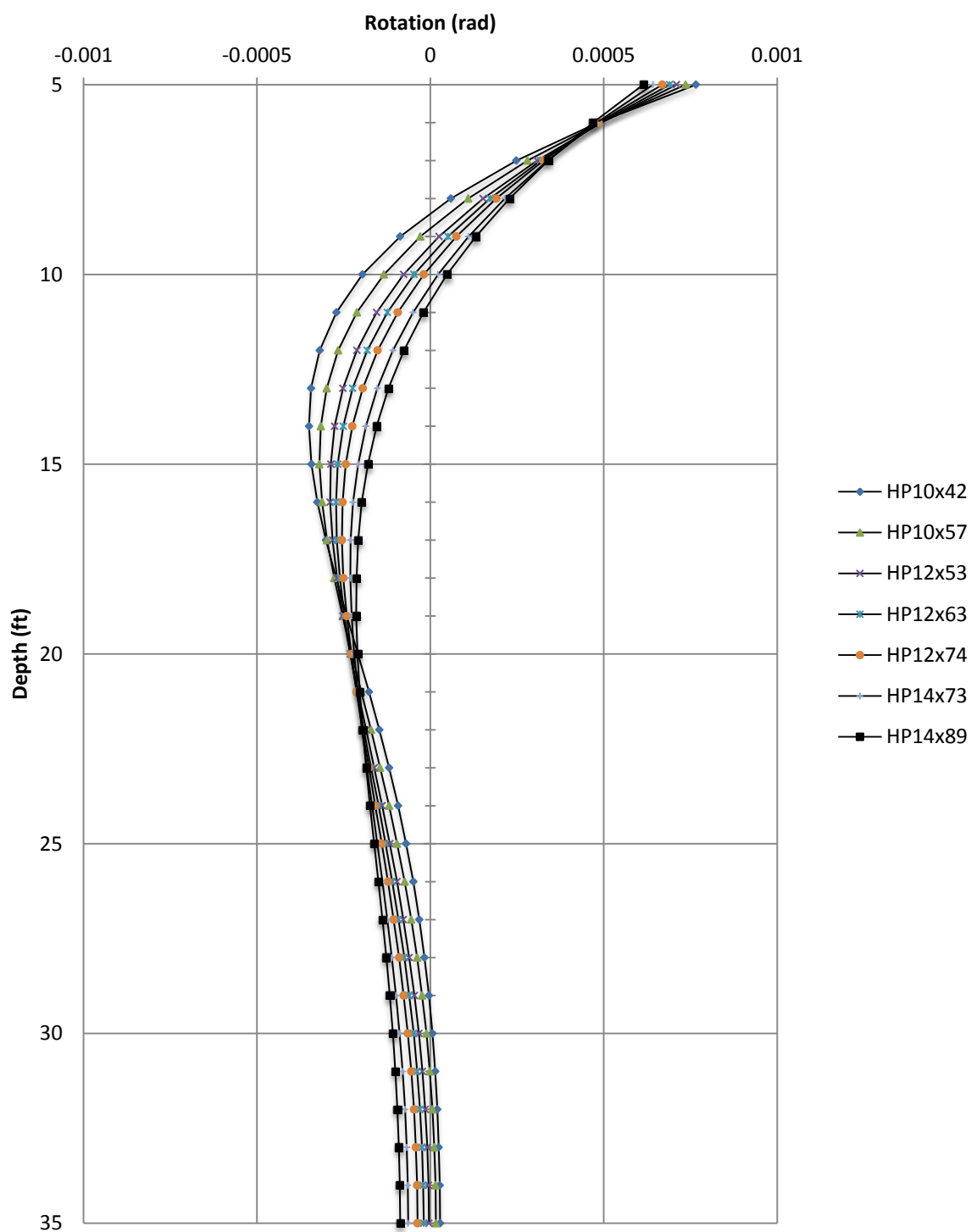


Figure B-7 : Bridge I pile rotation in soft clay under Truck LL

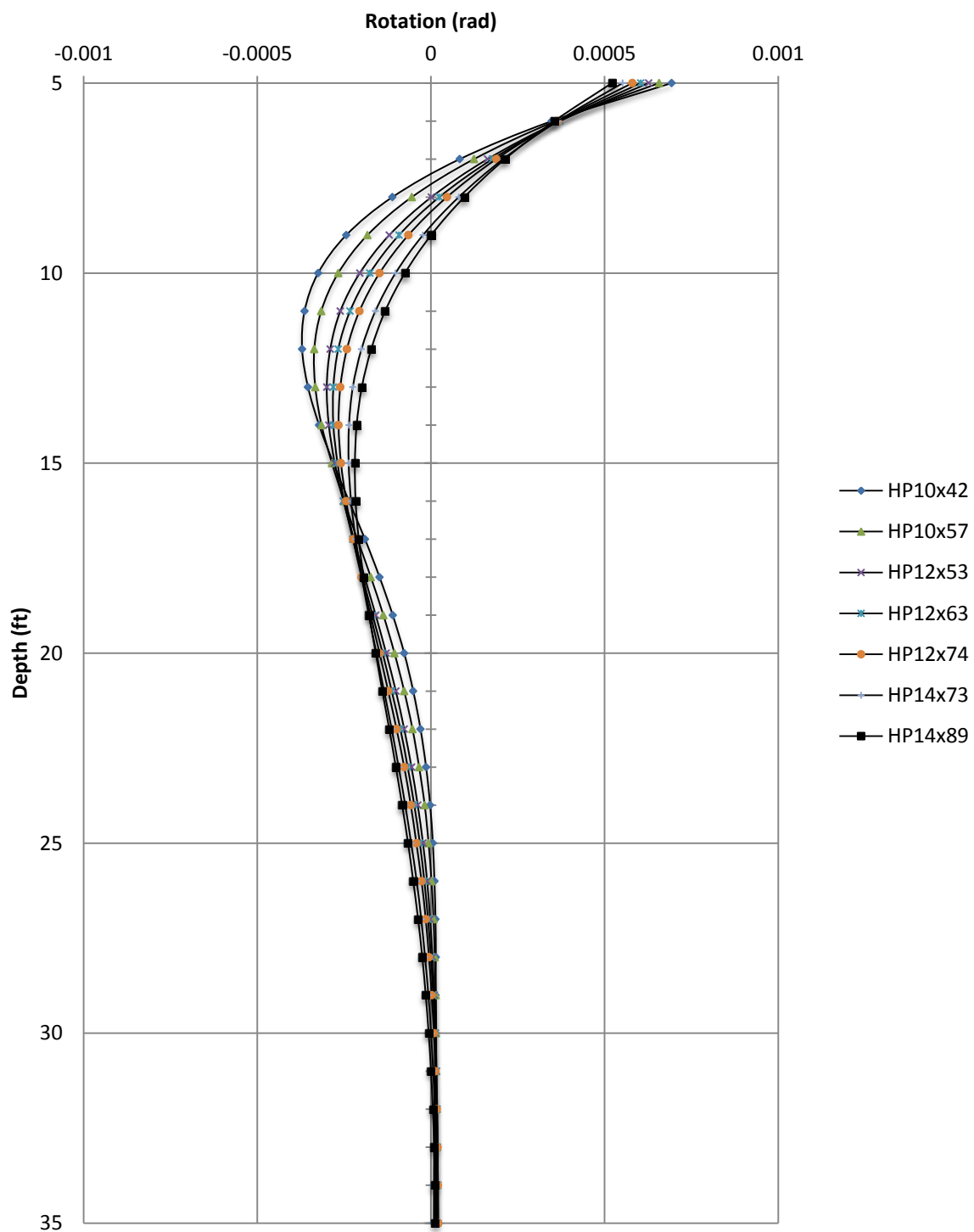


Figure B-8 : Bridge I pile rotation in medium clay under Truck LL

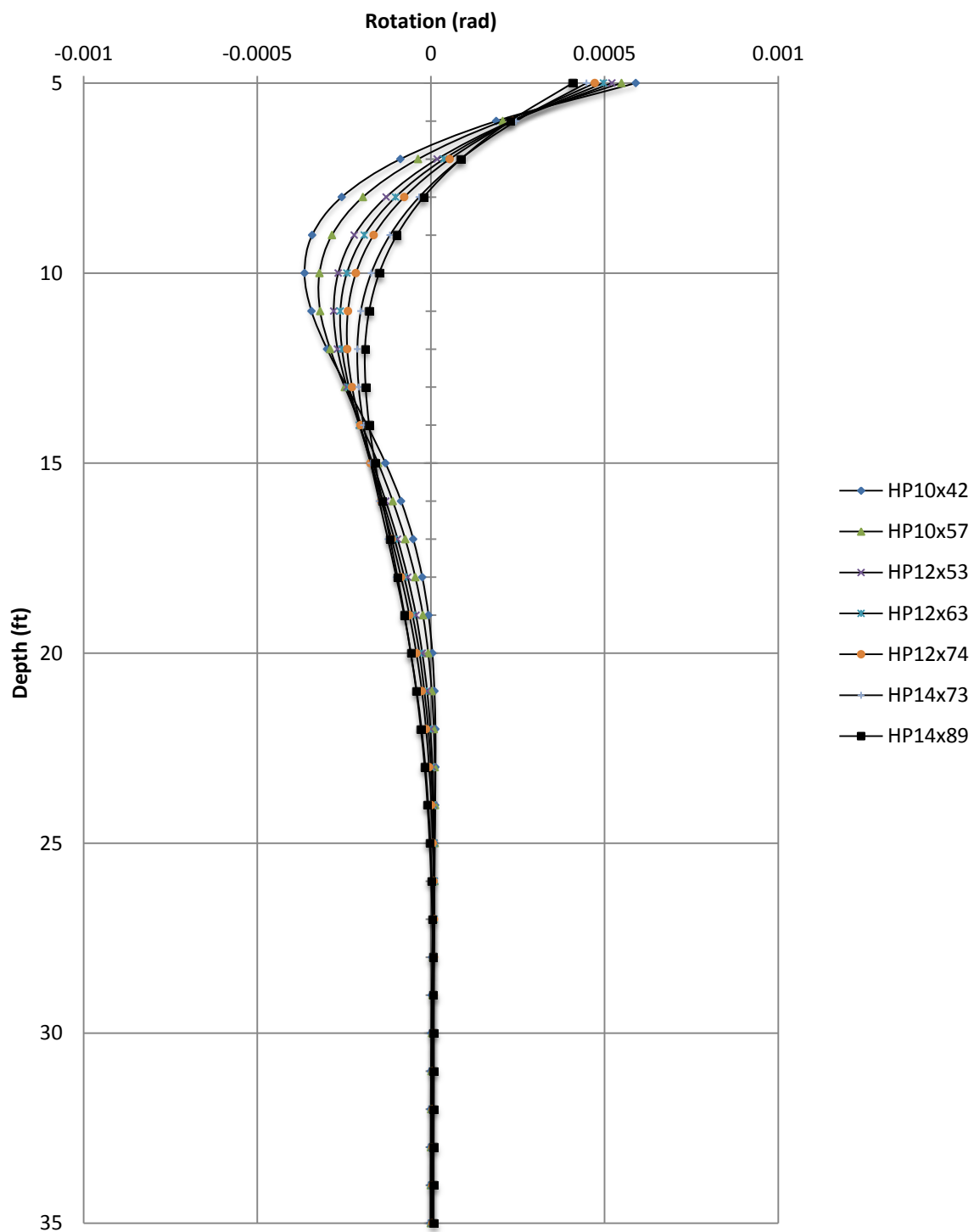


Figure B-9 : Bridge I pile rotation in stiff clay under Truck LL

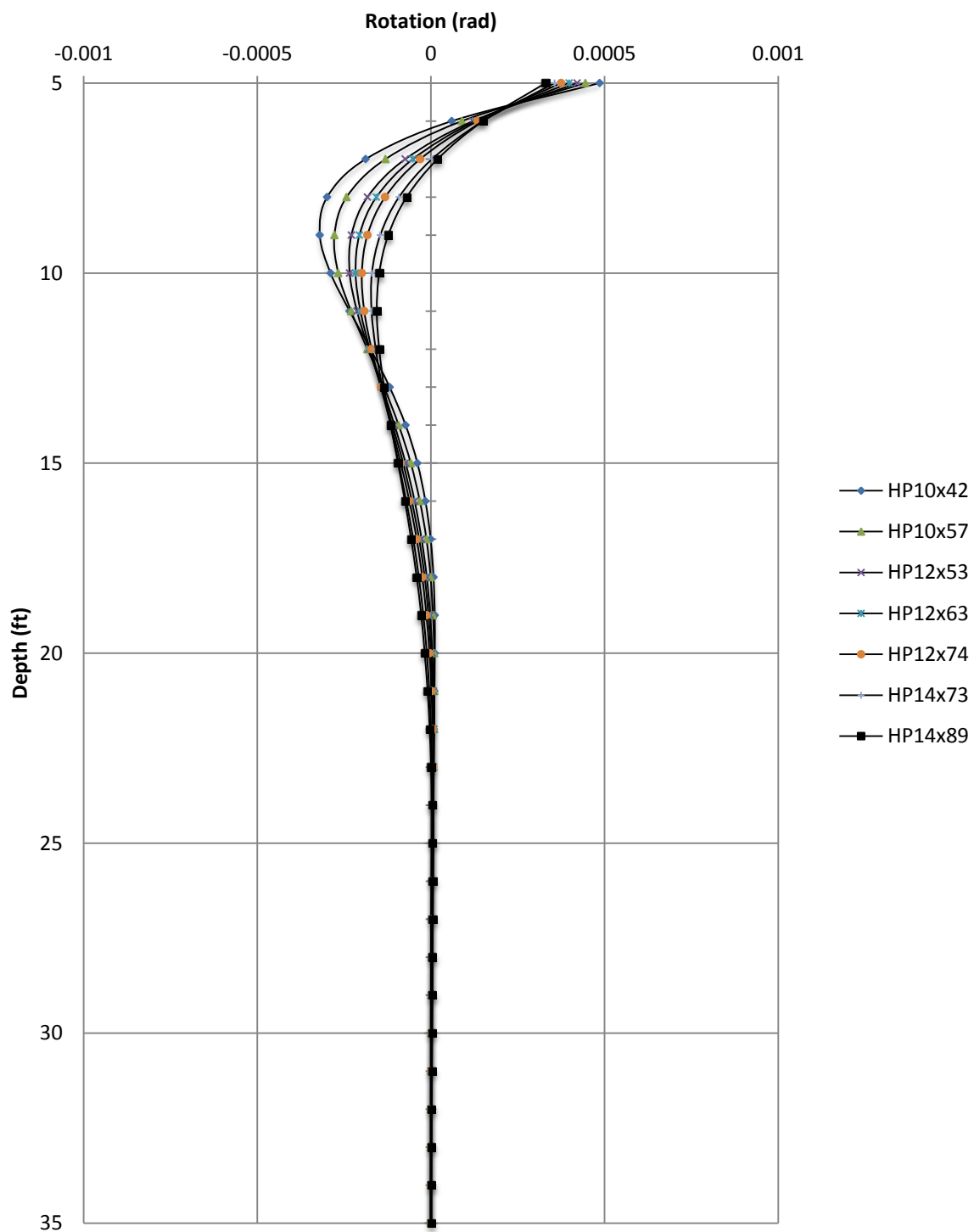


Figure B-10 : Bridge I pile rotation in very stiff clay under Truck LL

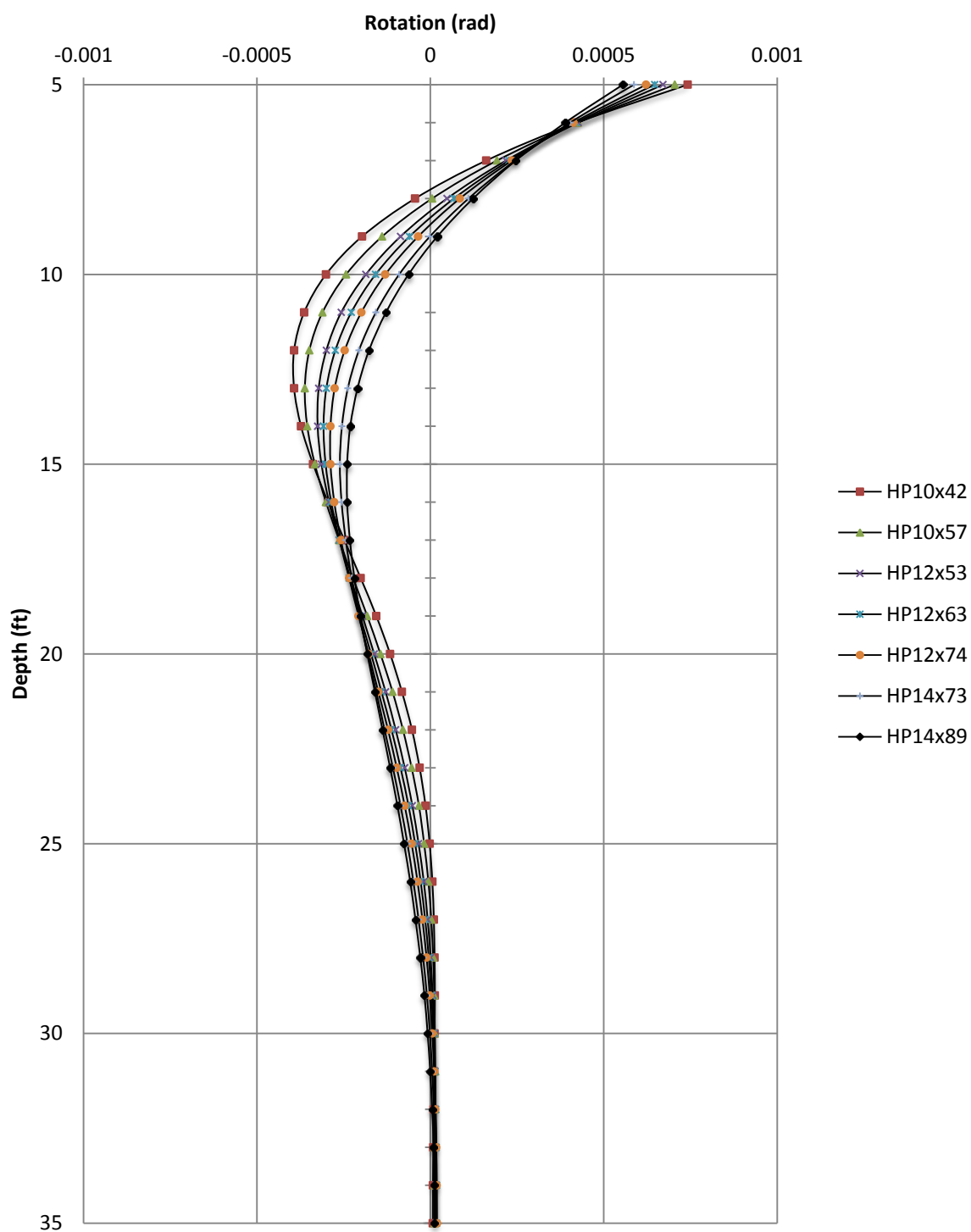


Figure B-11 : Bridge I pile rotation in loose sand under Truck LL

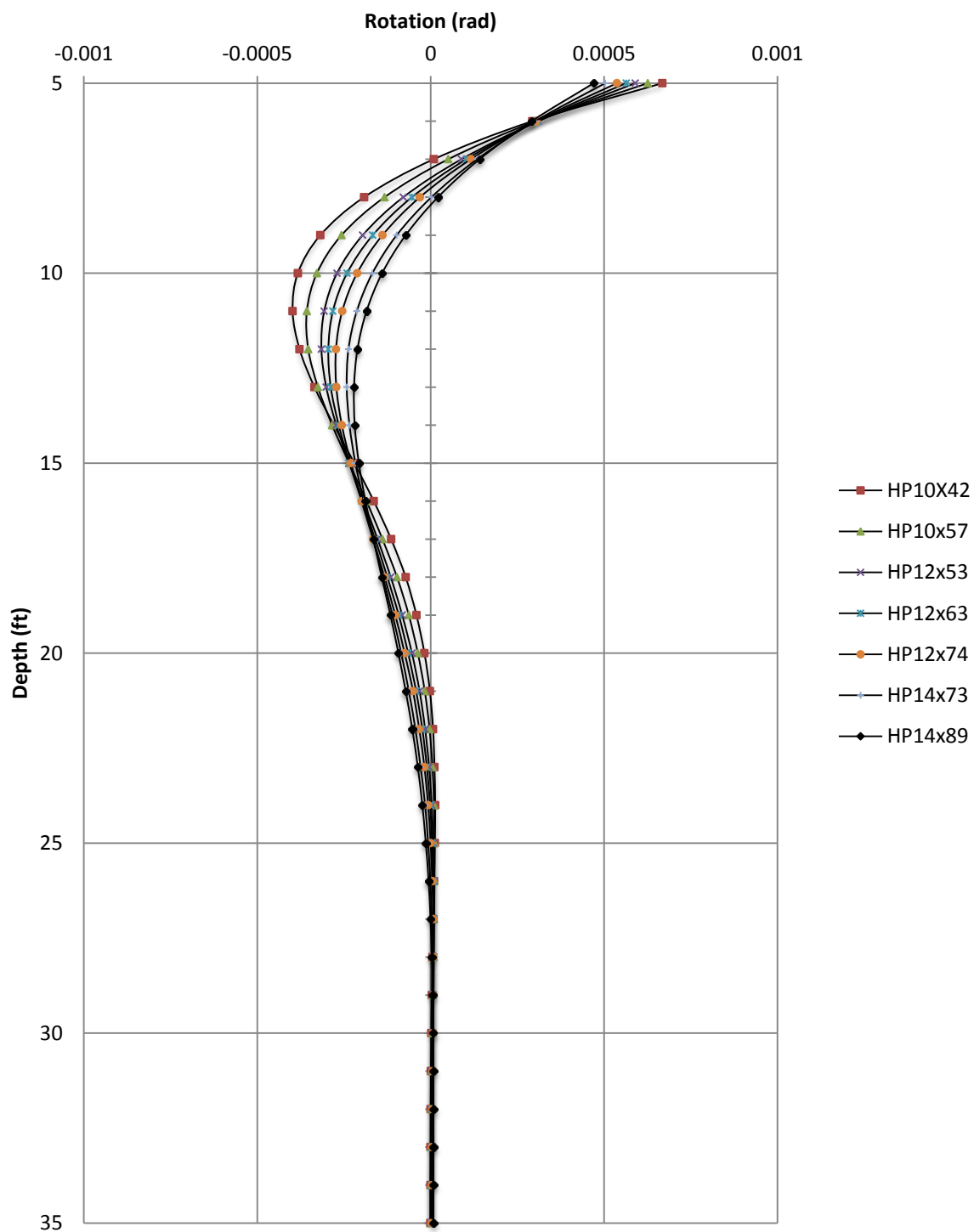


Figure B-12 : Bridge I pile rotation in medium sand under Truck LL

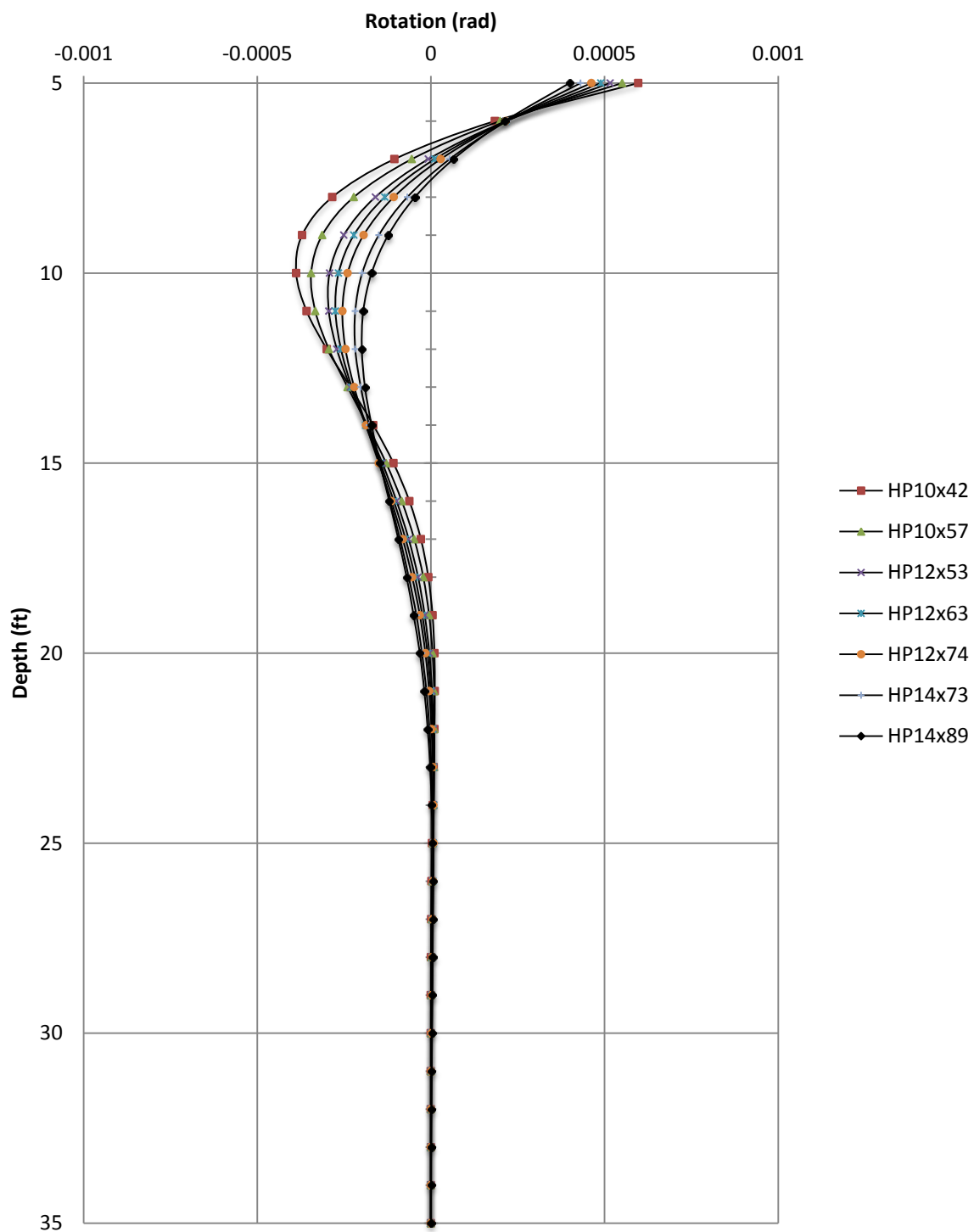


Figure B-13 : Bridge I pile rotation in dense sand under Truck LL

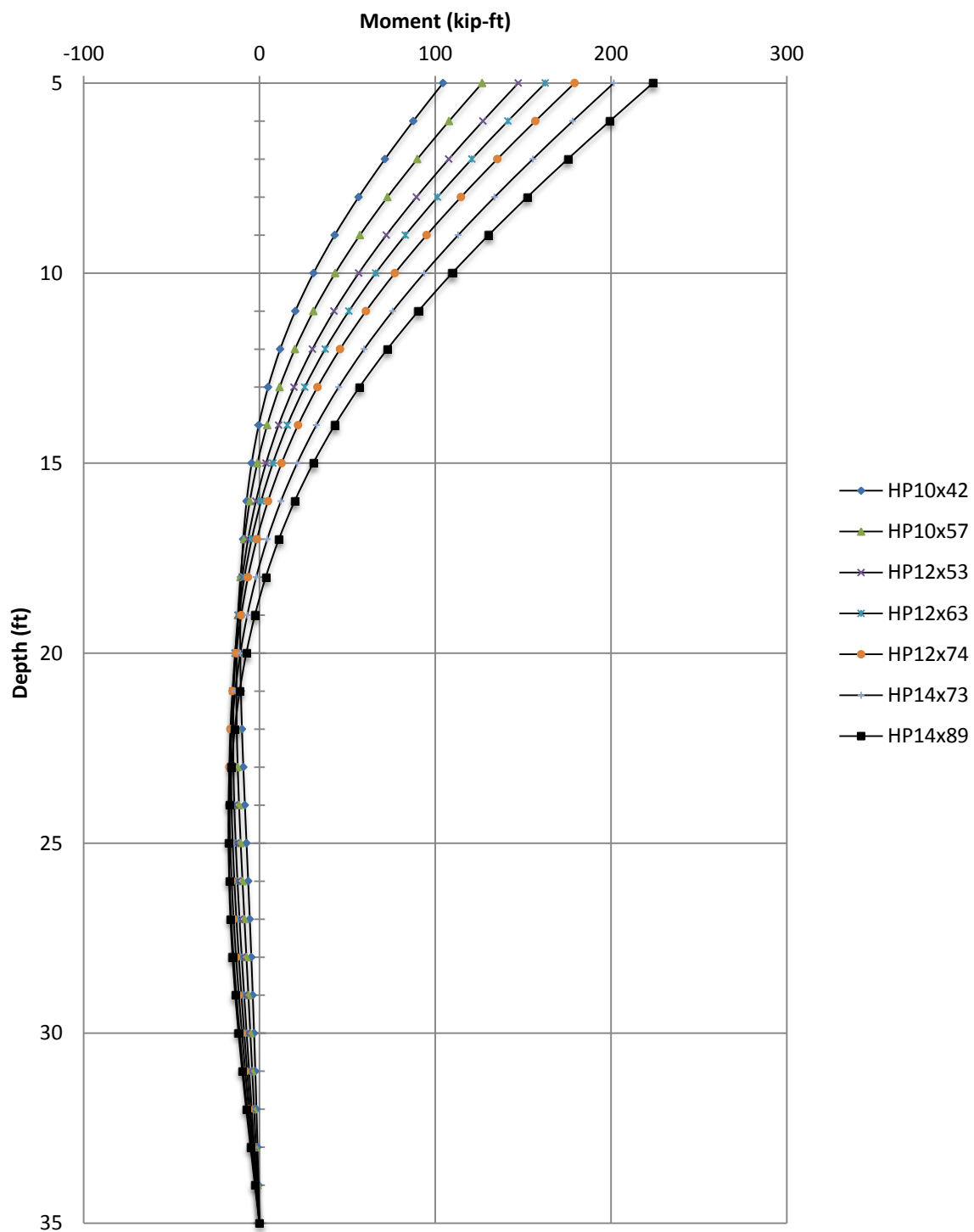


Figure B-14 : Bridge I pile moments in soft clay under Truck LL

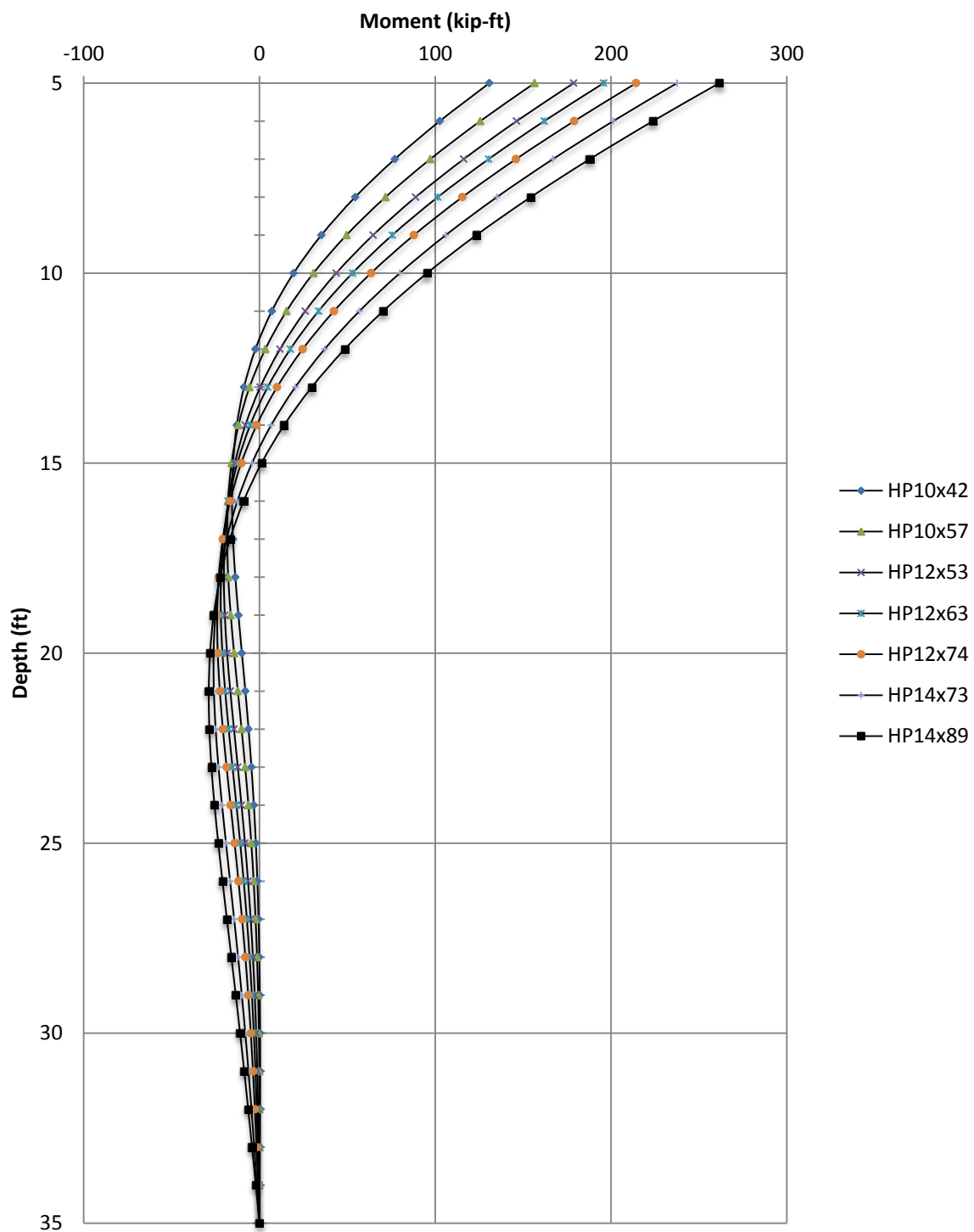


Figure B-15 : Bridge I pile moments in medium clay under Truck LL

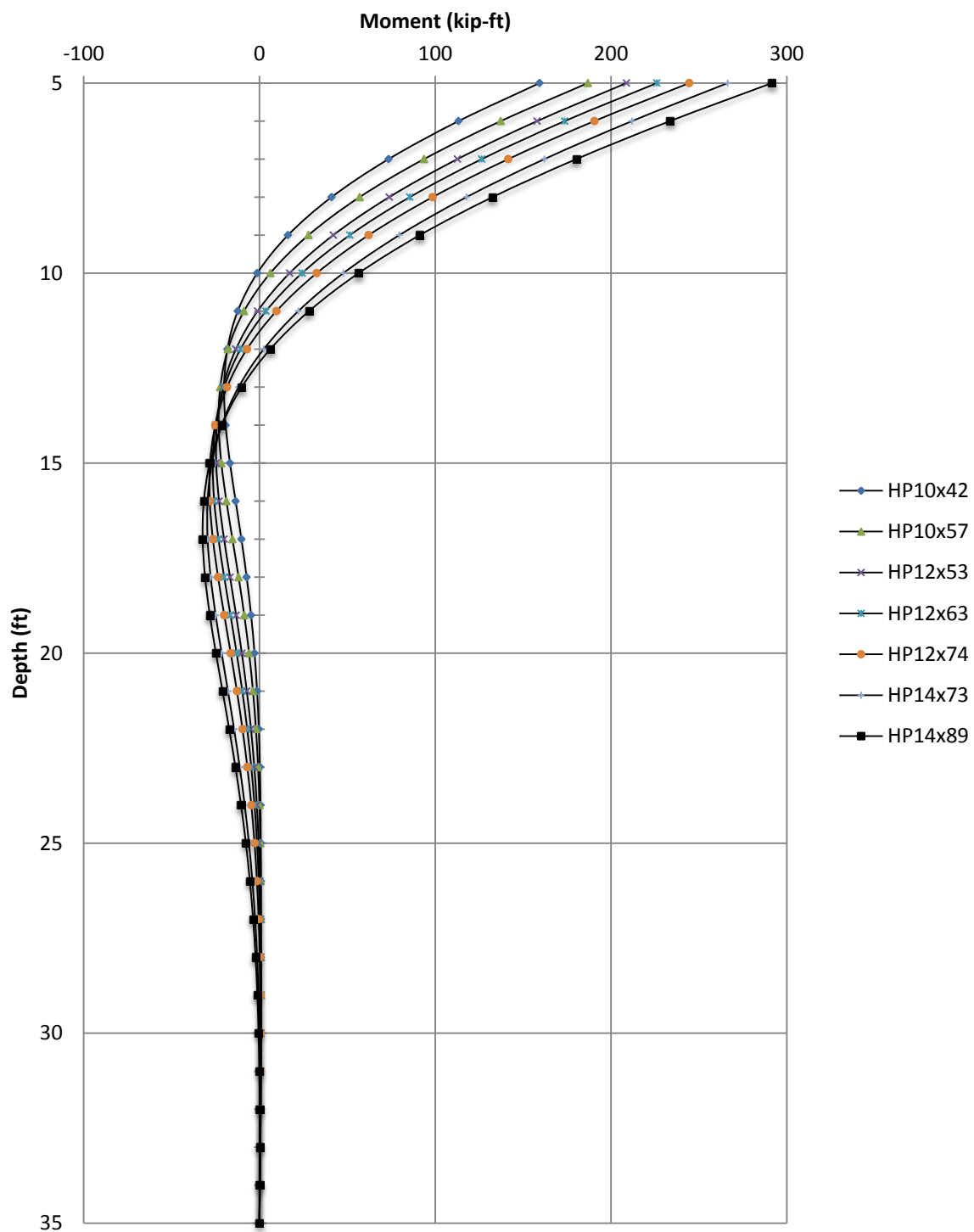


Figure B-16 : Bridge I pile moments in stiff clay under Truck LL

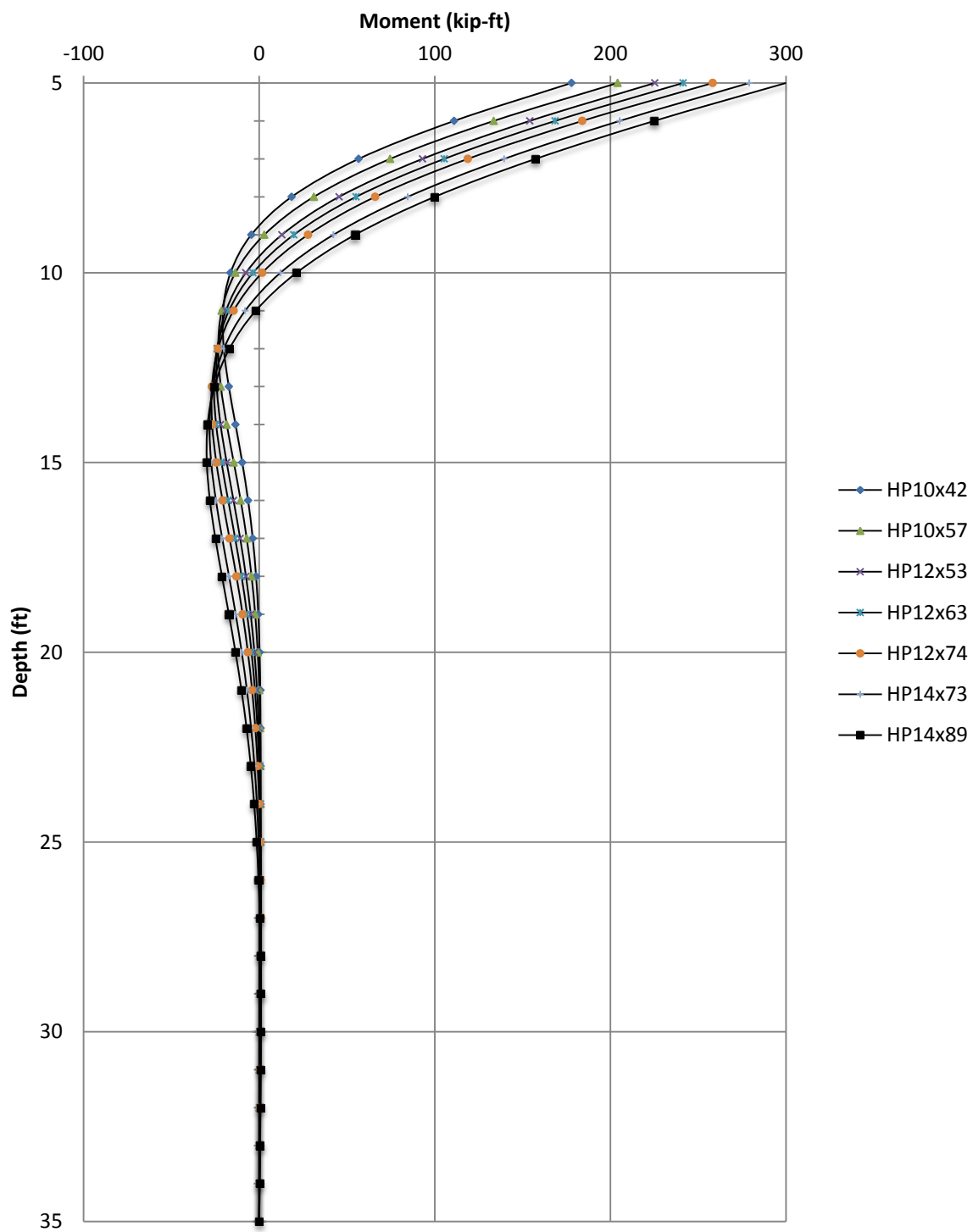


Figure B-17 : Bridge I pile moments in very stiff clay under Truck LL

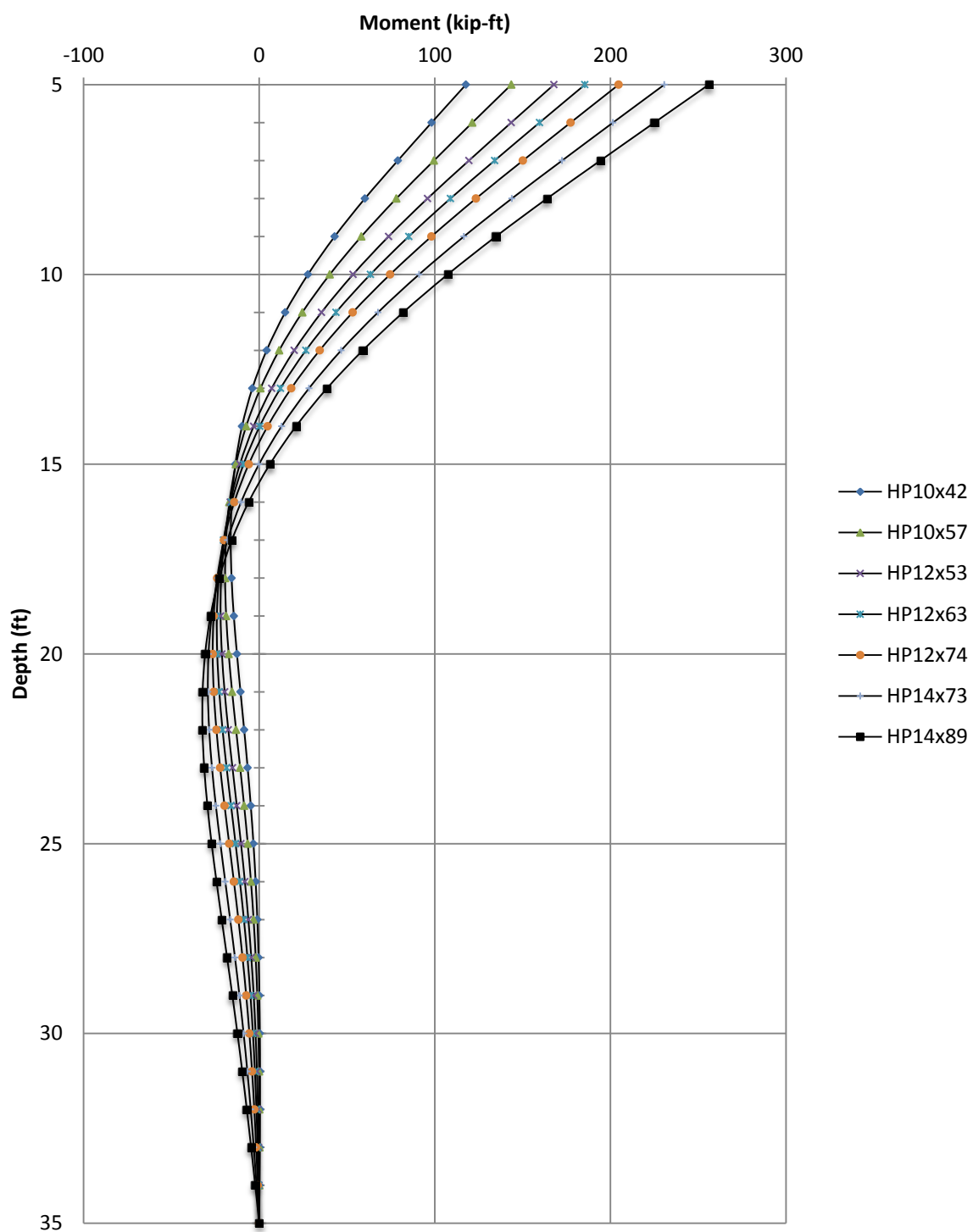


Figure B-18 : Bridge I pile moments in loose sand under Truck LL

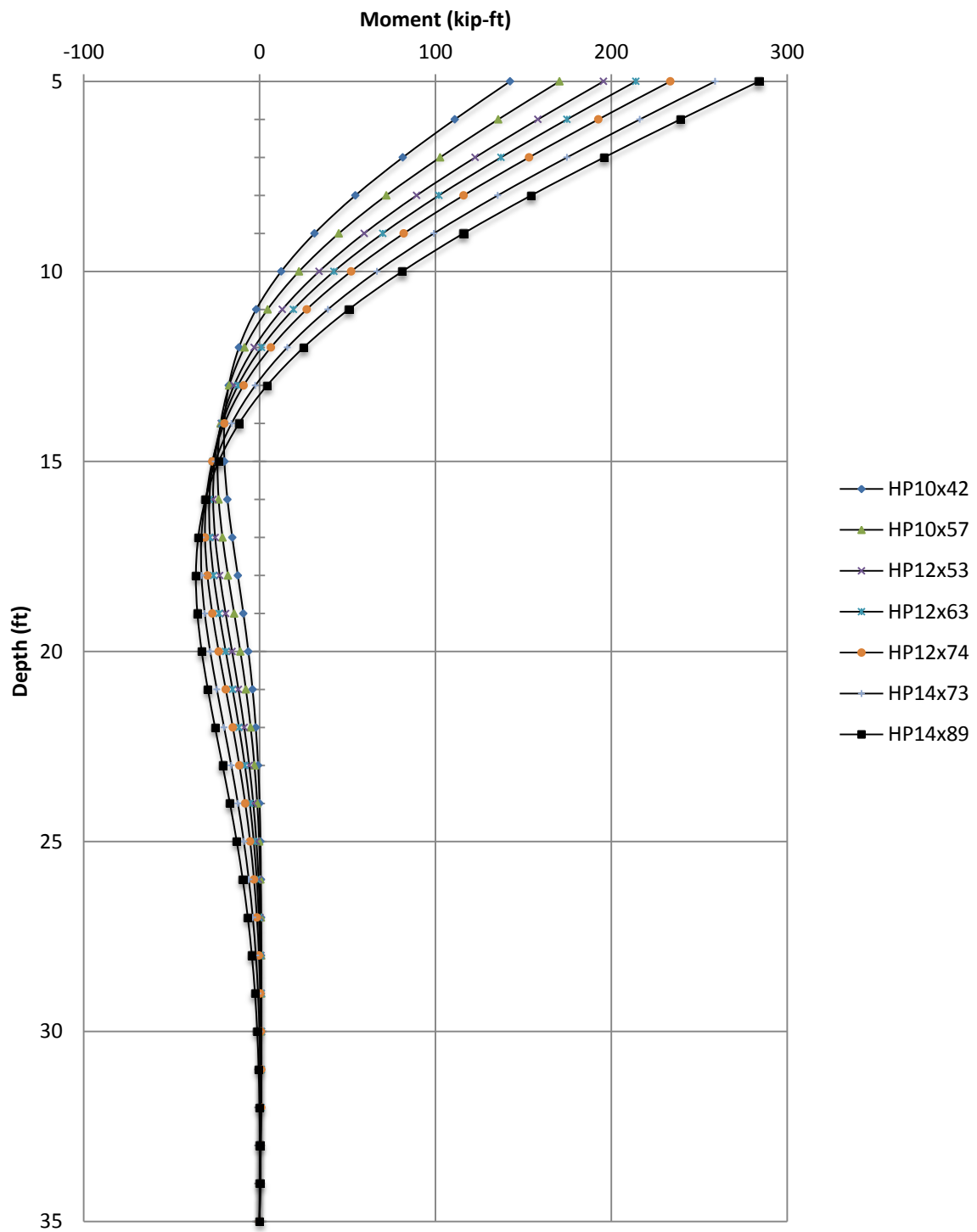


Figure B-19 : Bridge I pile moments in medium sand under Truck LL

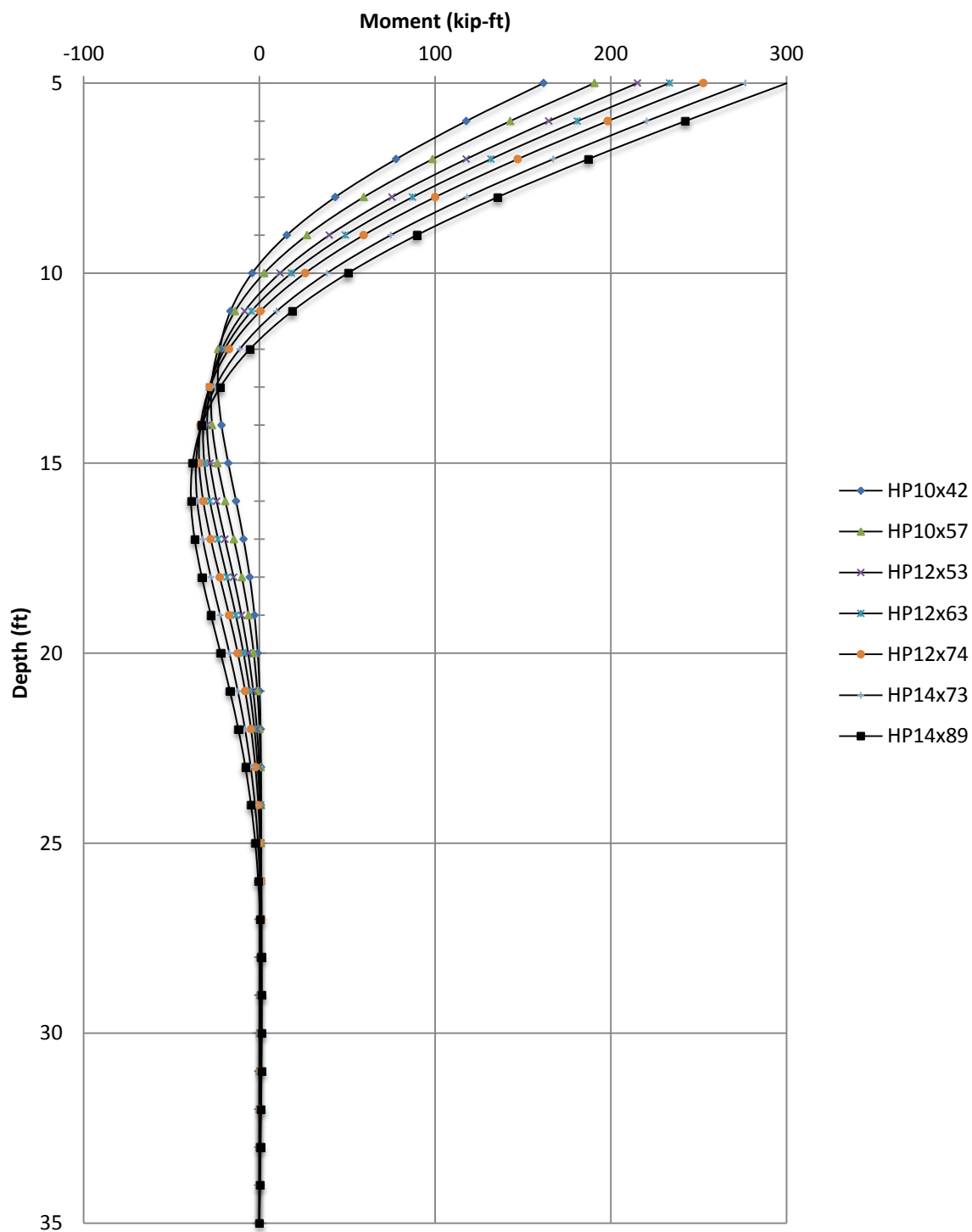


Figure B-20 : Bridge I pile moments in dense sand under Truck LL

BRIDGE II

PILE DEFLECTION PROFILES

PILE ROTATION PROFILES

PILE MOMENT PROFILES

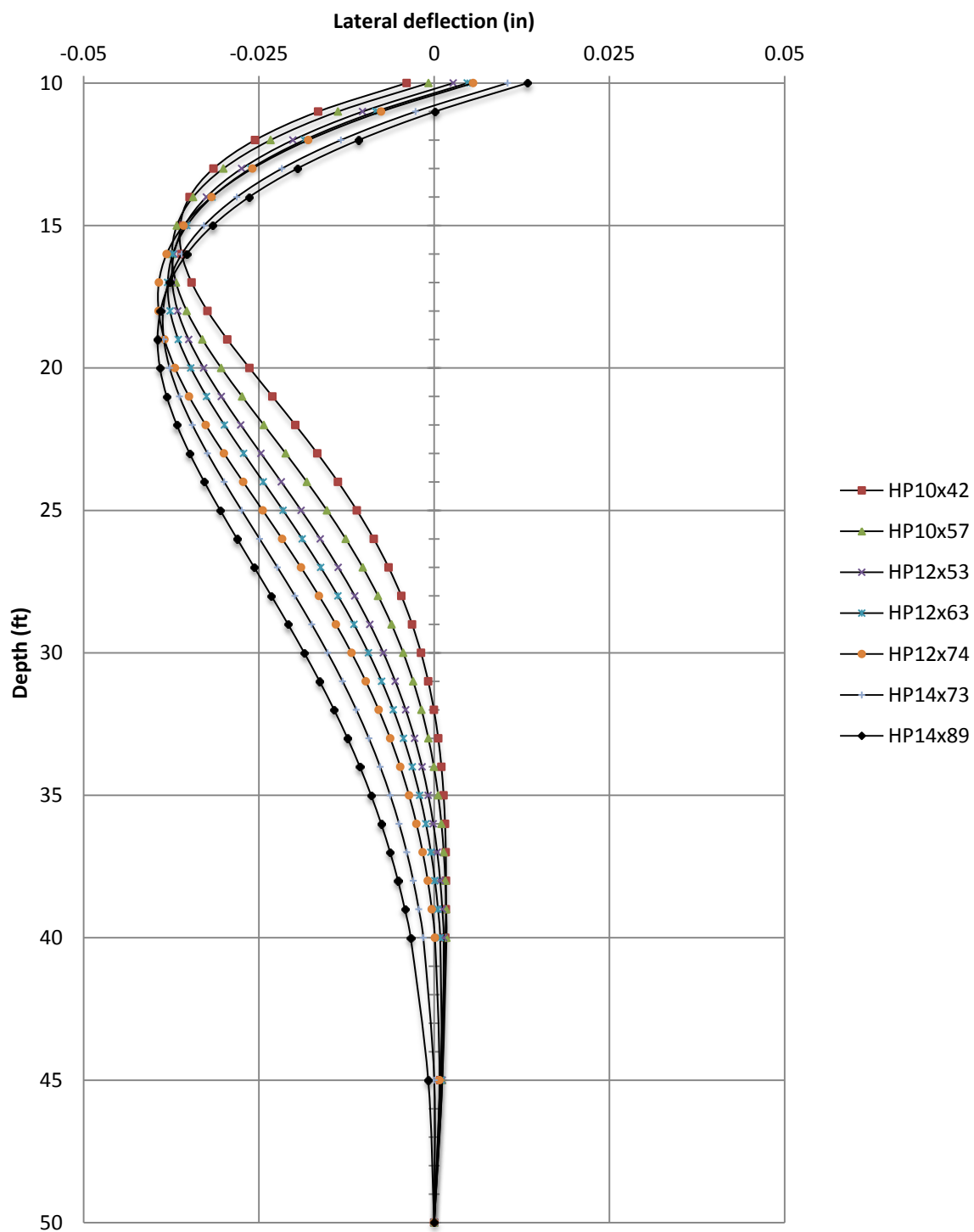


Figure B-21: Lateral deflection of piles of Bridge II in soft clay under Truck LL

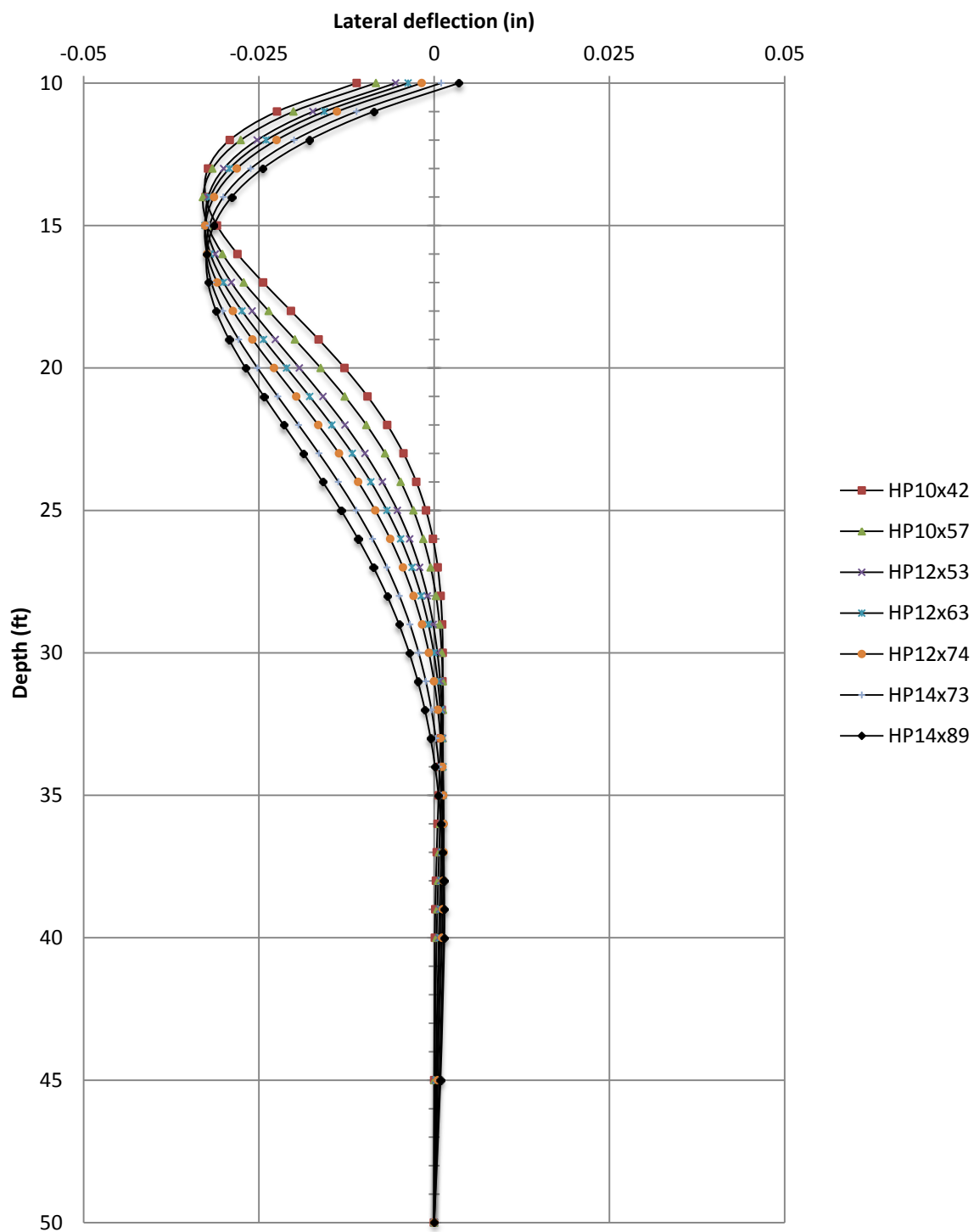


Figure B-22: Lateral deflection of piles of Bridge II in medium clay under Truck LL

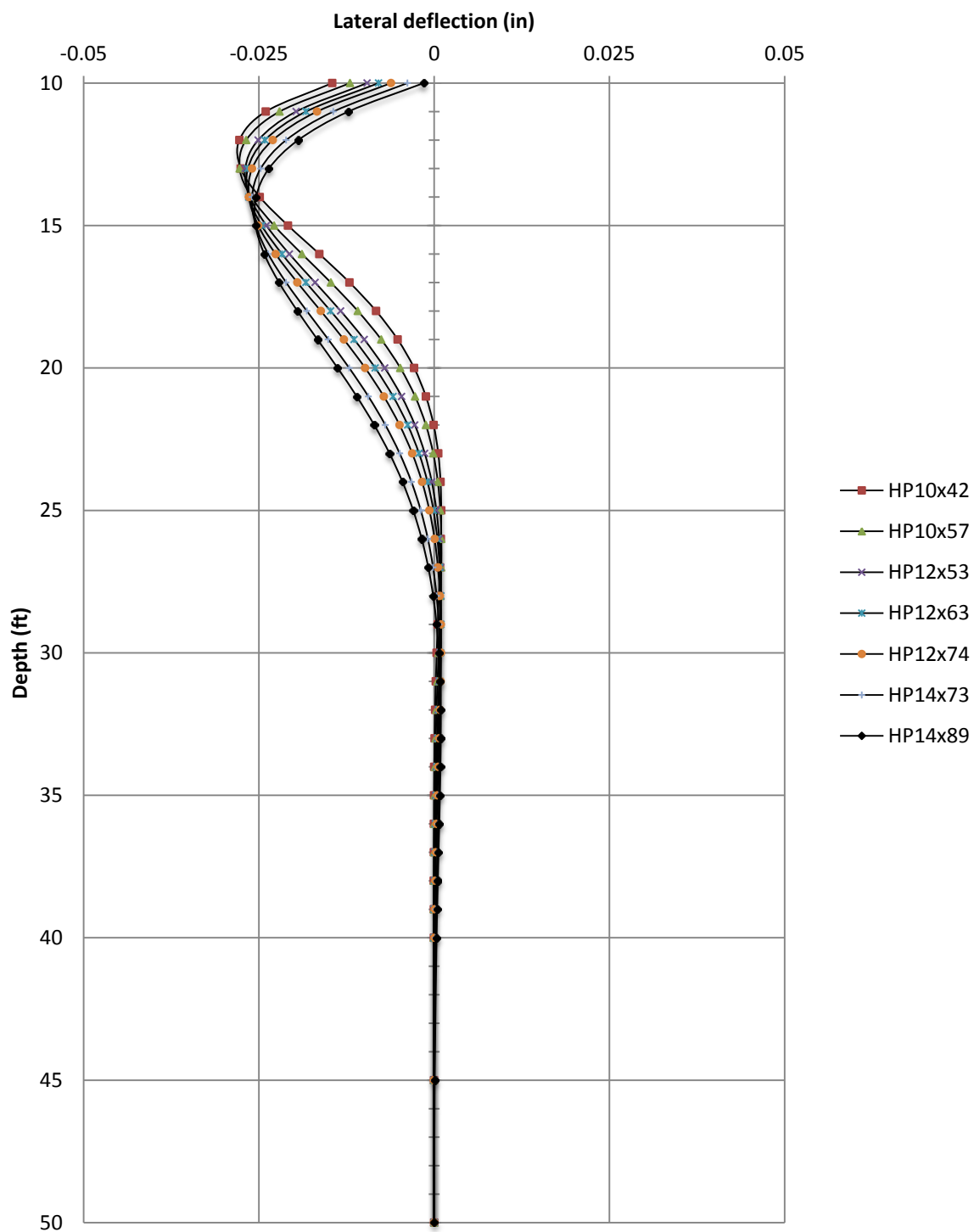


Figure B-23: Lateral deflection of piles of Bridge II in stiff clay under Truck LL

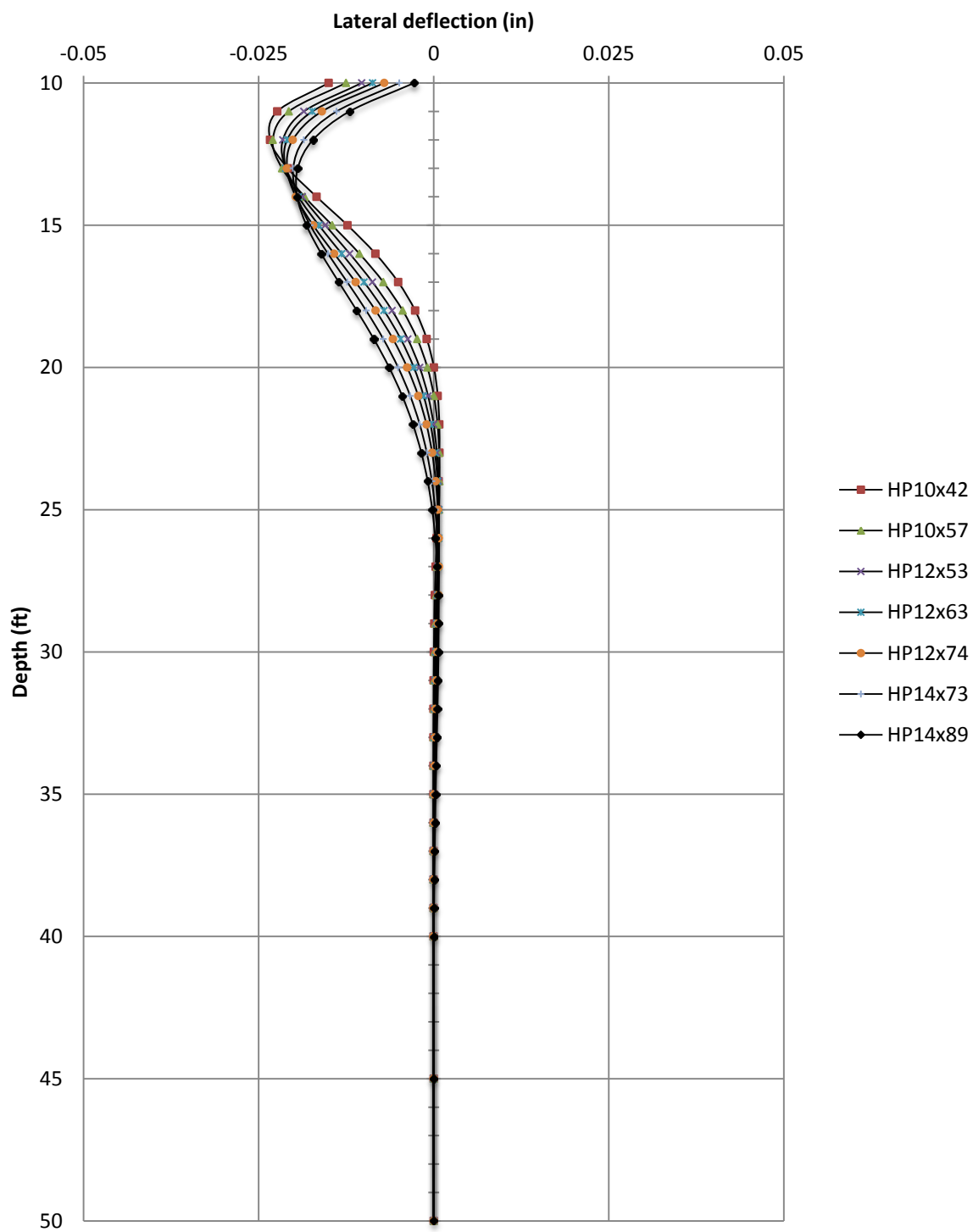


Figure B-24: Lateral deflection of piles of Bridge II in very stiff clay under Truck LL

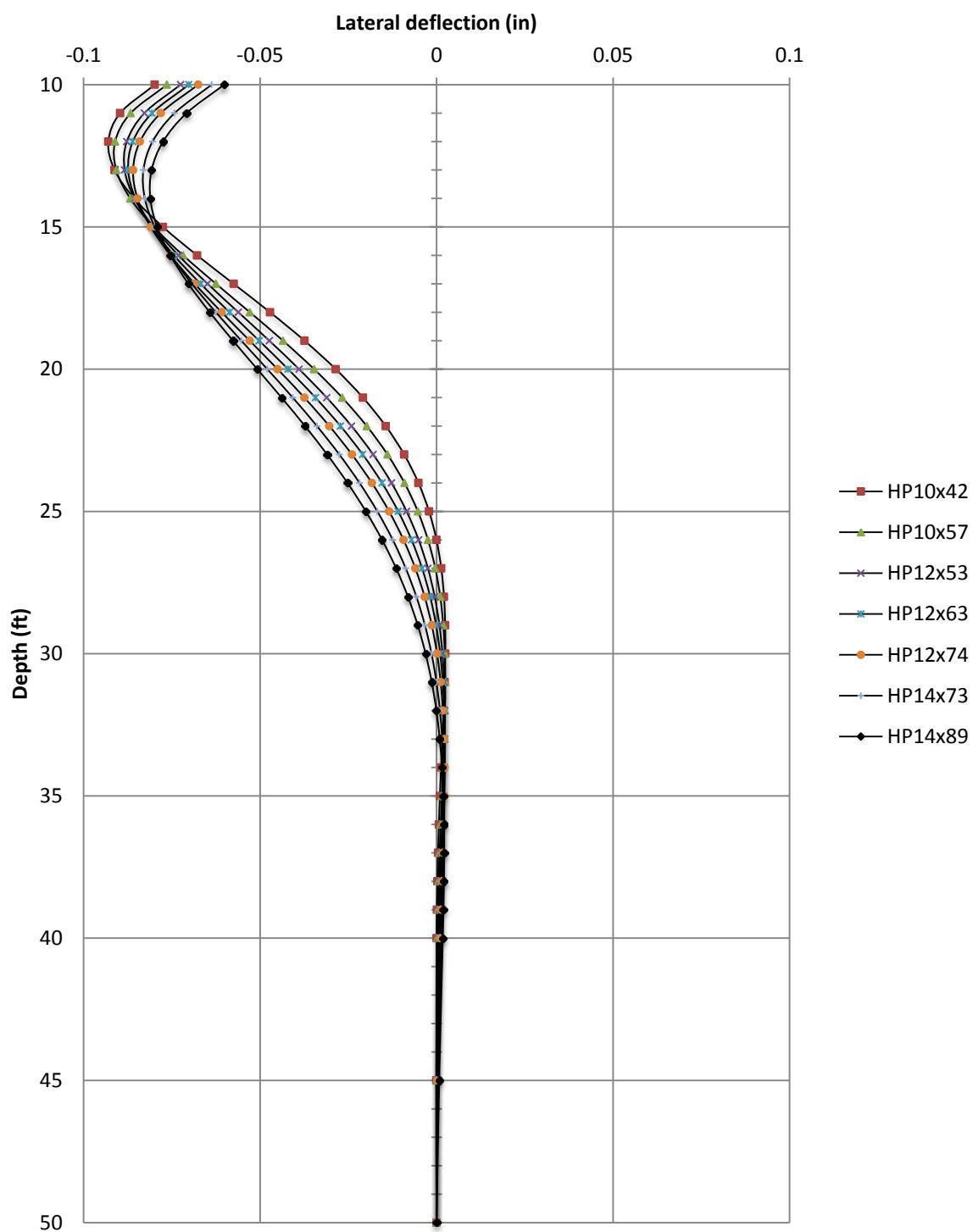


Figure B-25: Lateral deflection of piles of Bridge II in loose sand under Truck LL

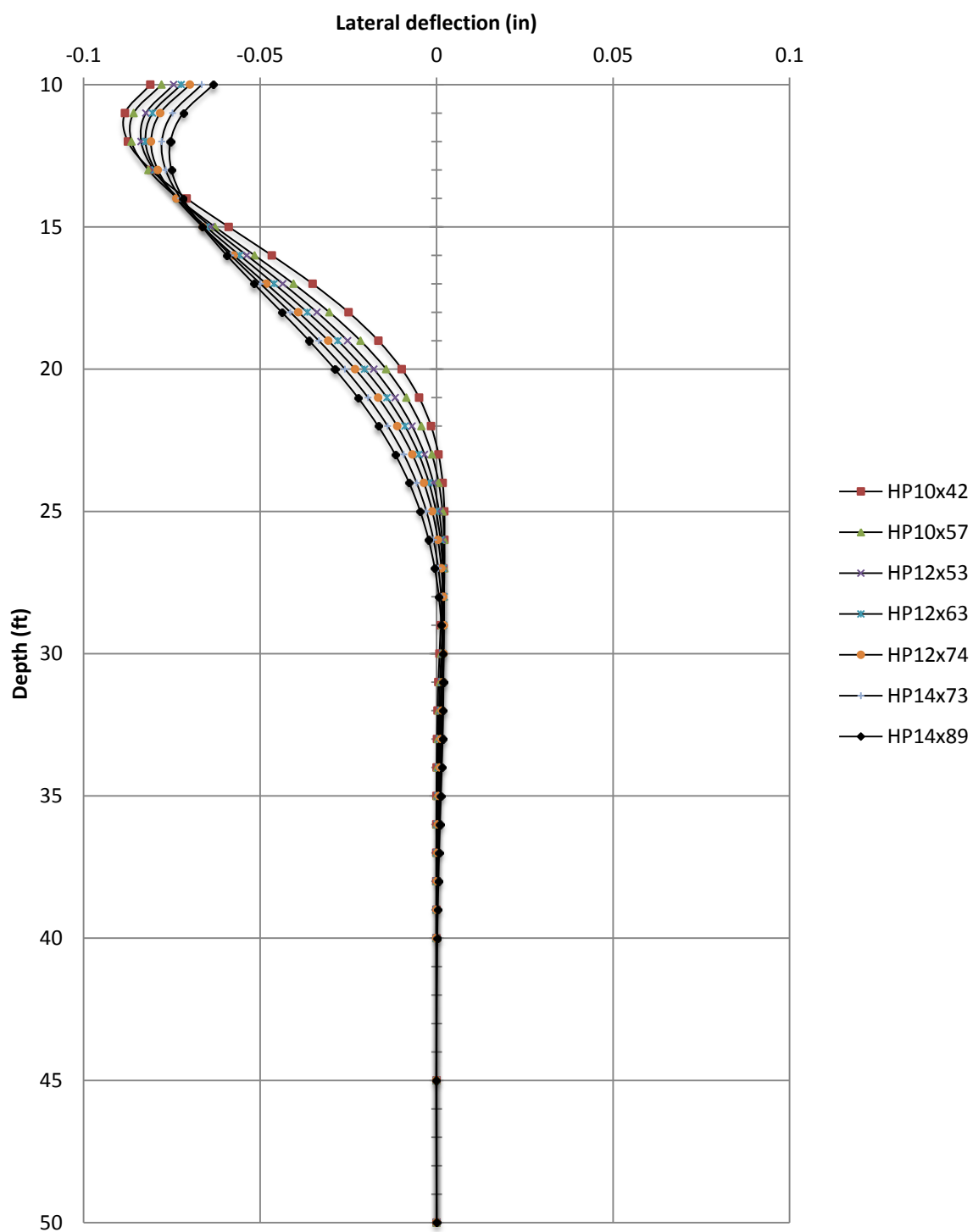


Figure B-26: Lateral deflection of piles of Bridge II in medium sand under Truck LL

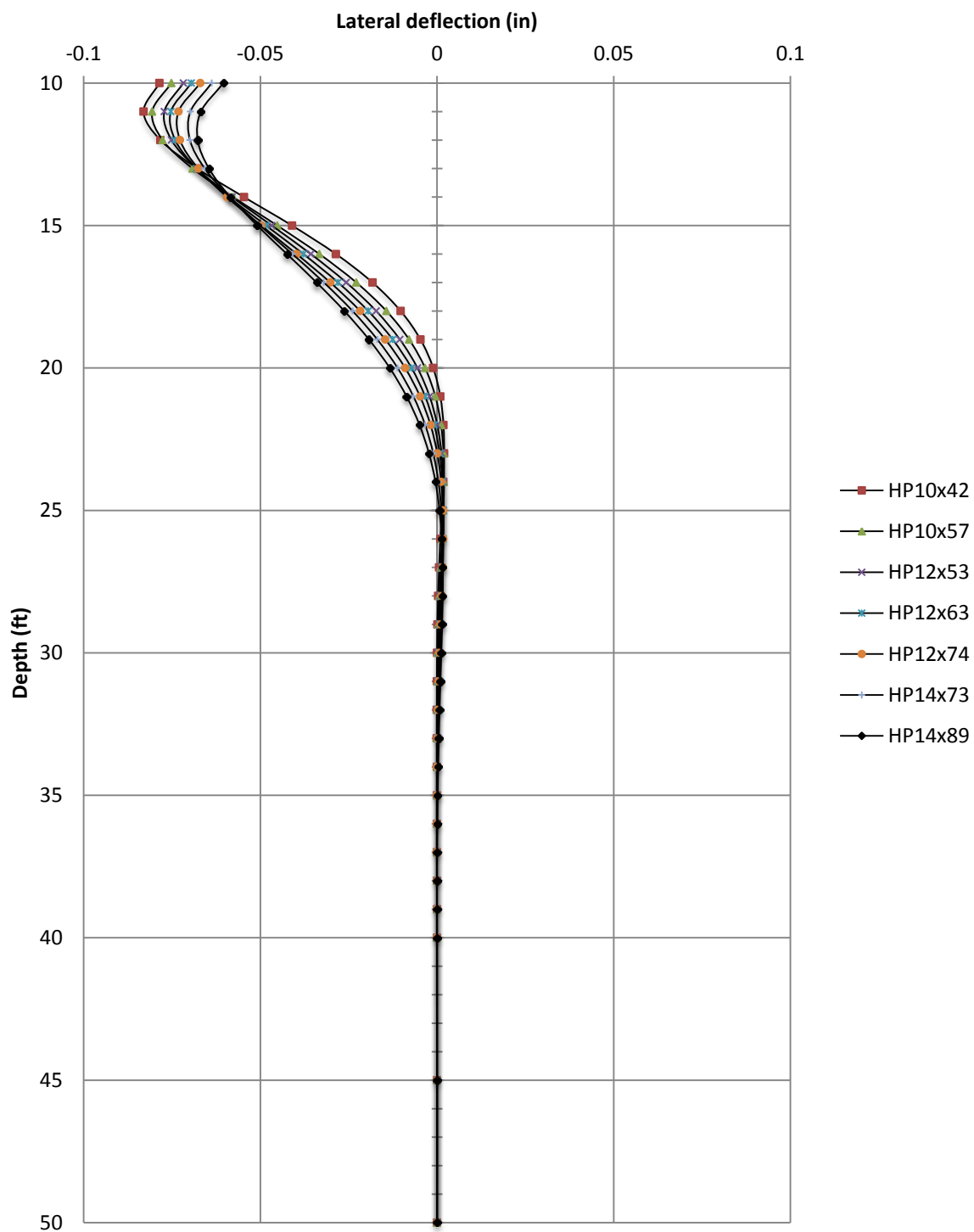


Figure B-27: Lateral deflection of piles of Bridge II in dense sand under Truck LL

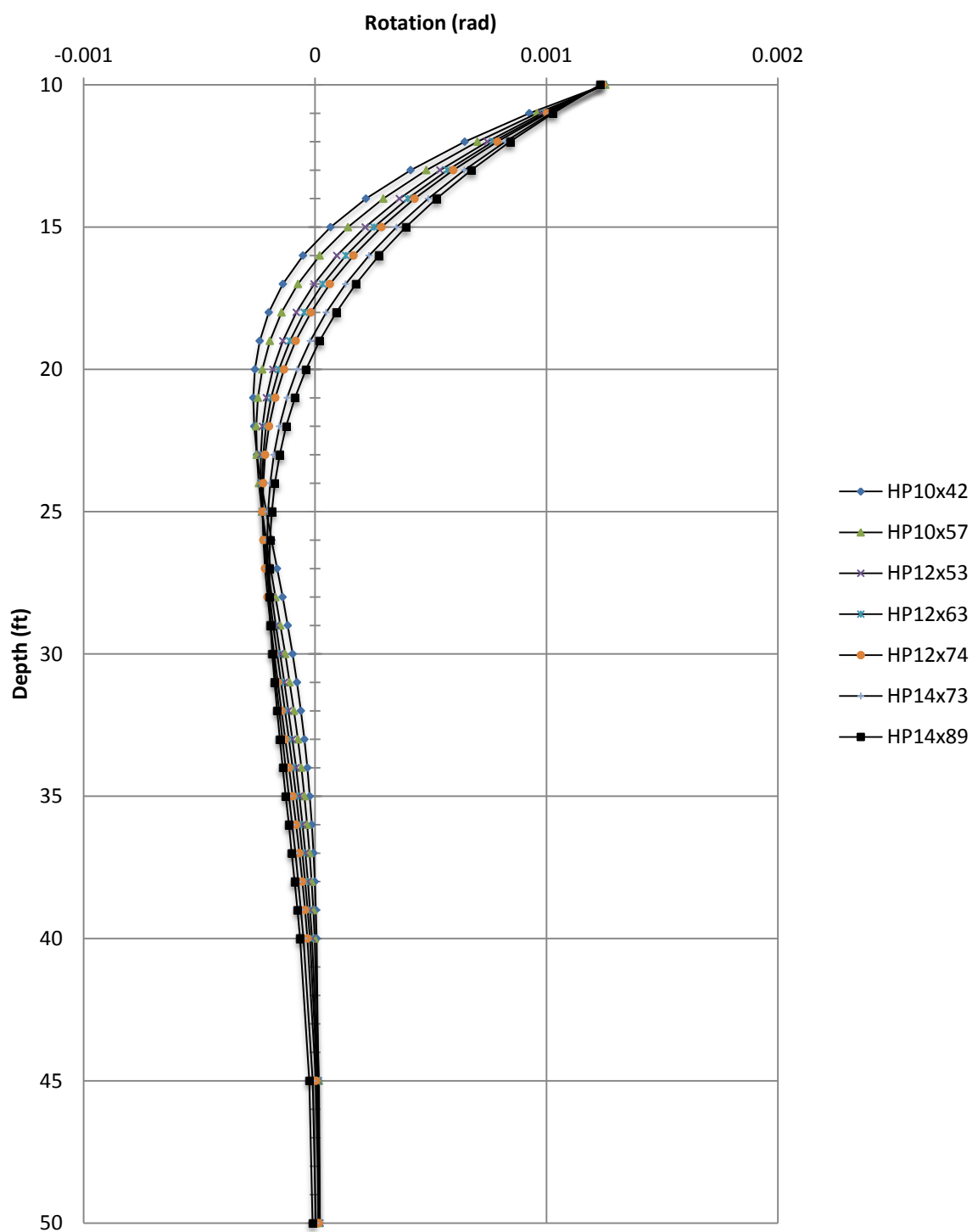


Figure B-28 : Bridge II pile rotation in soft clay under Truck LL

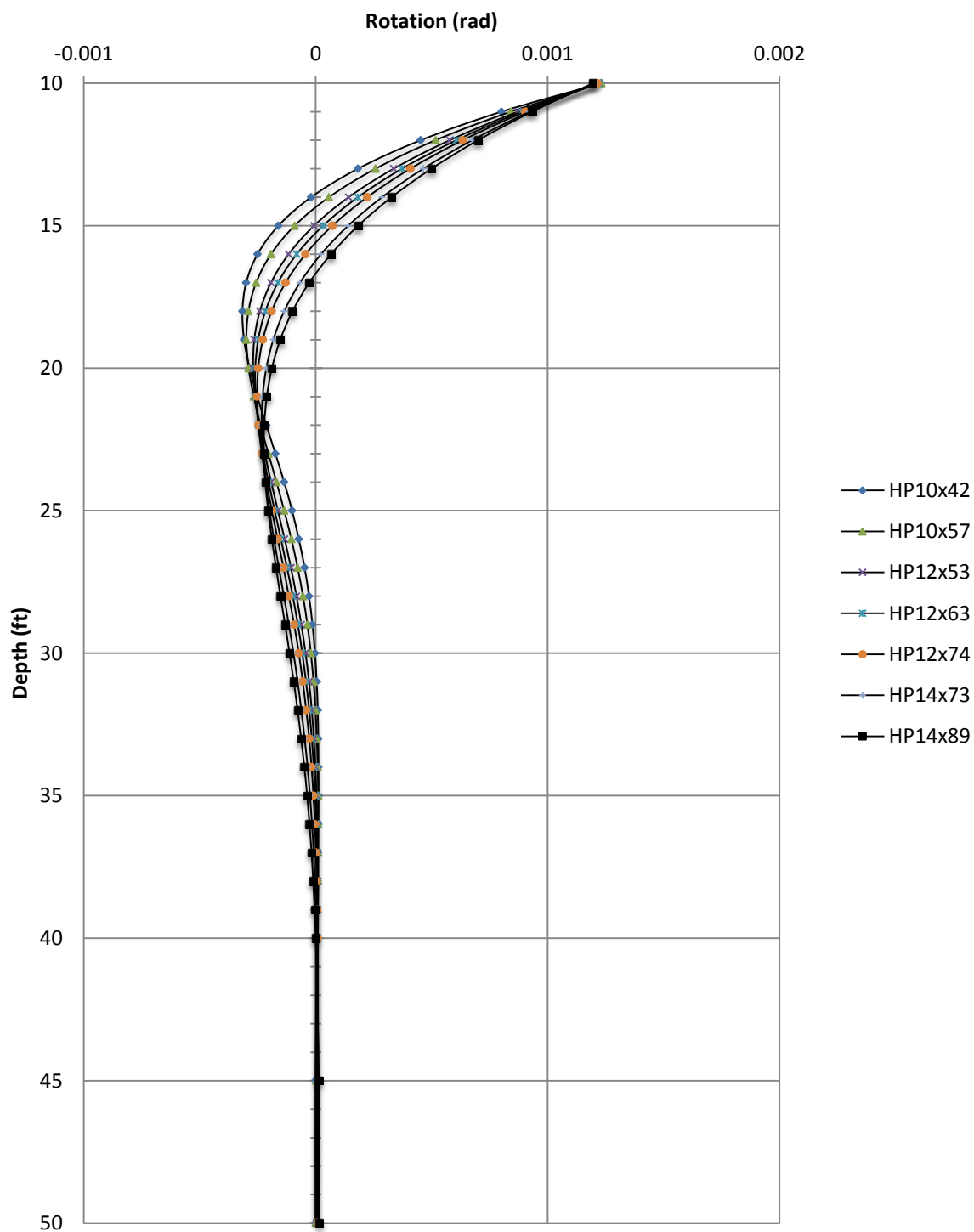


Figure B-29 : Bridge II pile rotation in medium clay under Truck LL

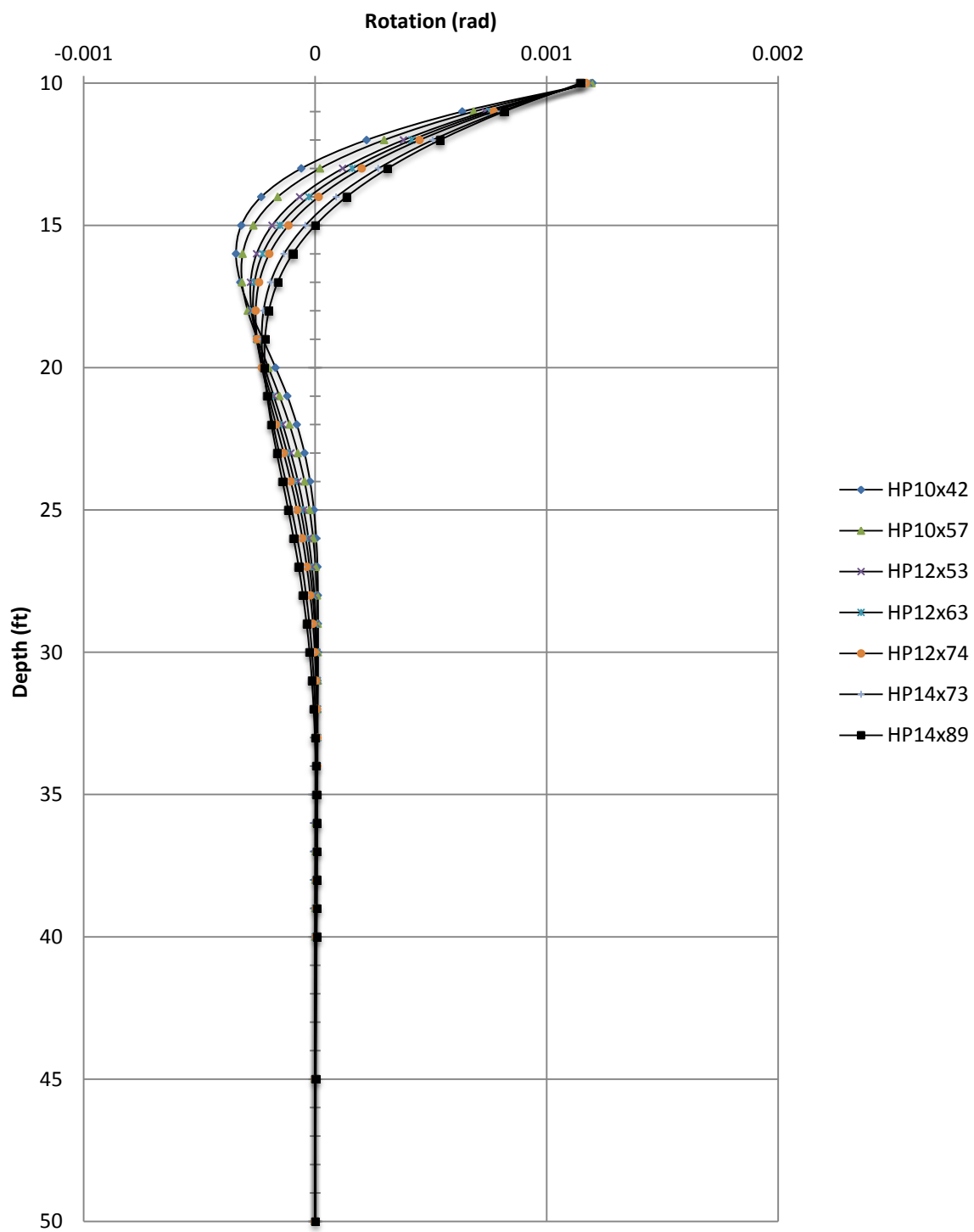


Figure B-30 : Bridge II pile rotation in stiff clay under Truck LL

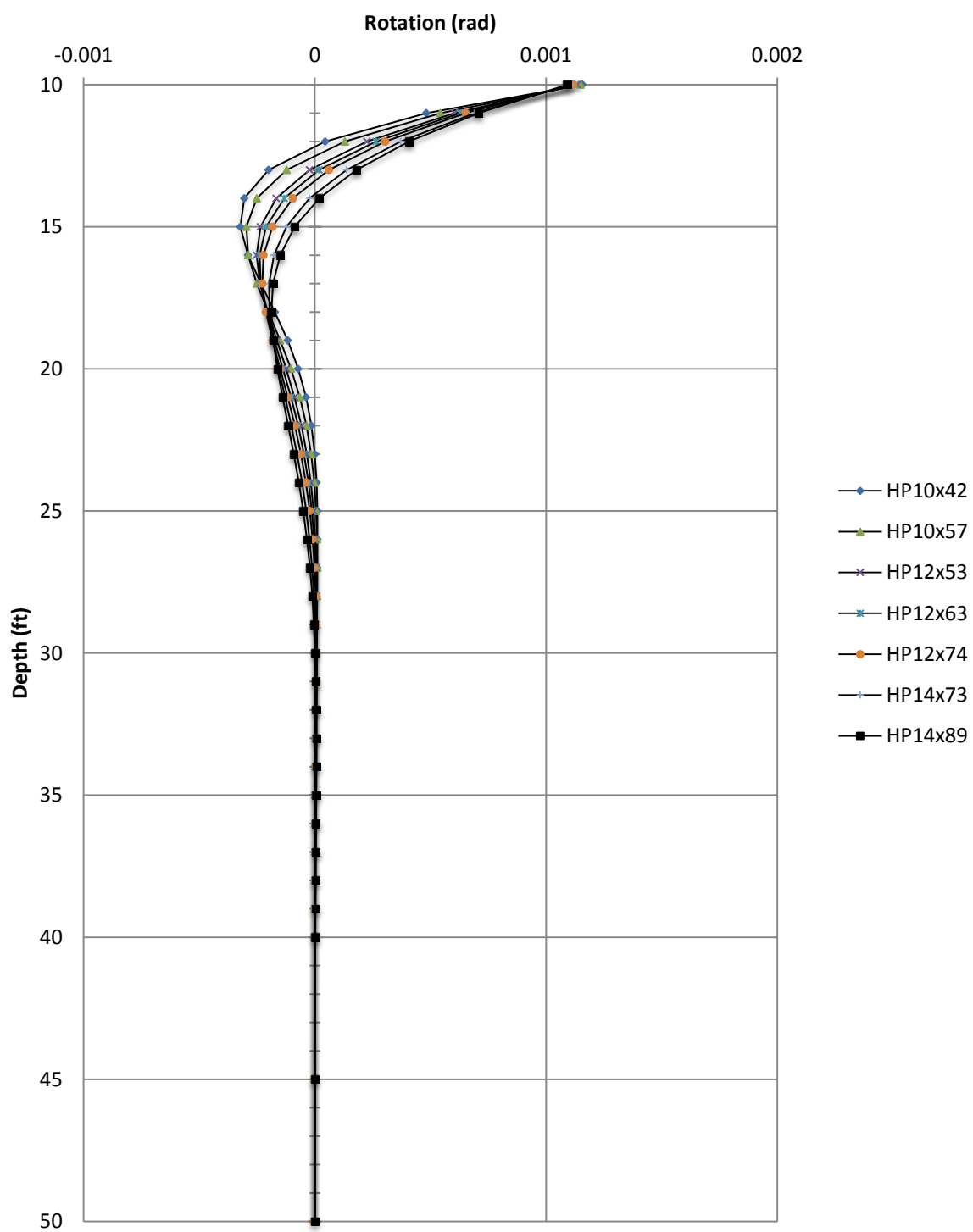


Figure B-31 : Bridge II pile rotation in very stiff clay under Truck LL

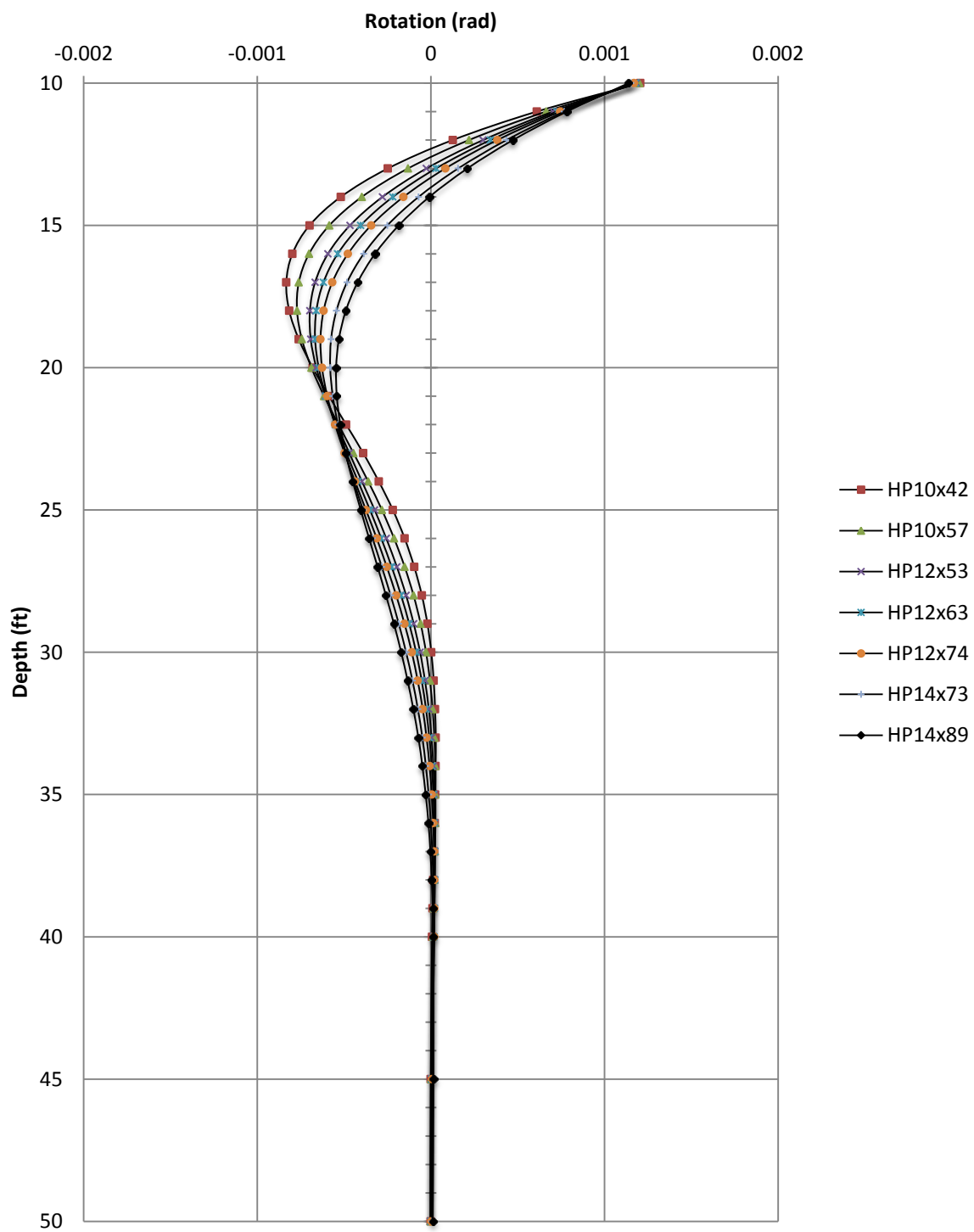


Figure B-32 : Bridge II pile rotation in loose sand under Truck LL

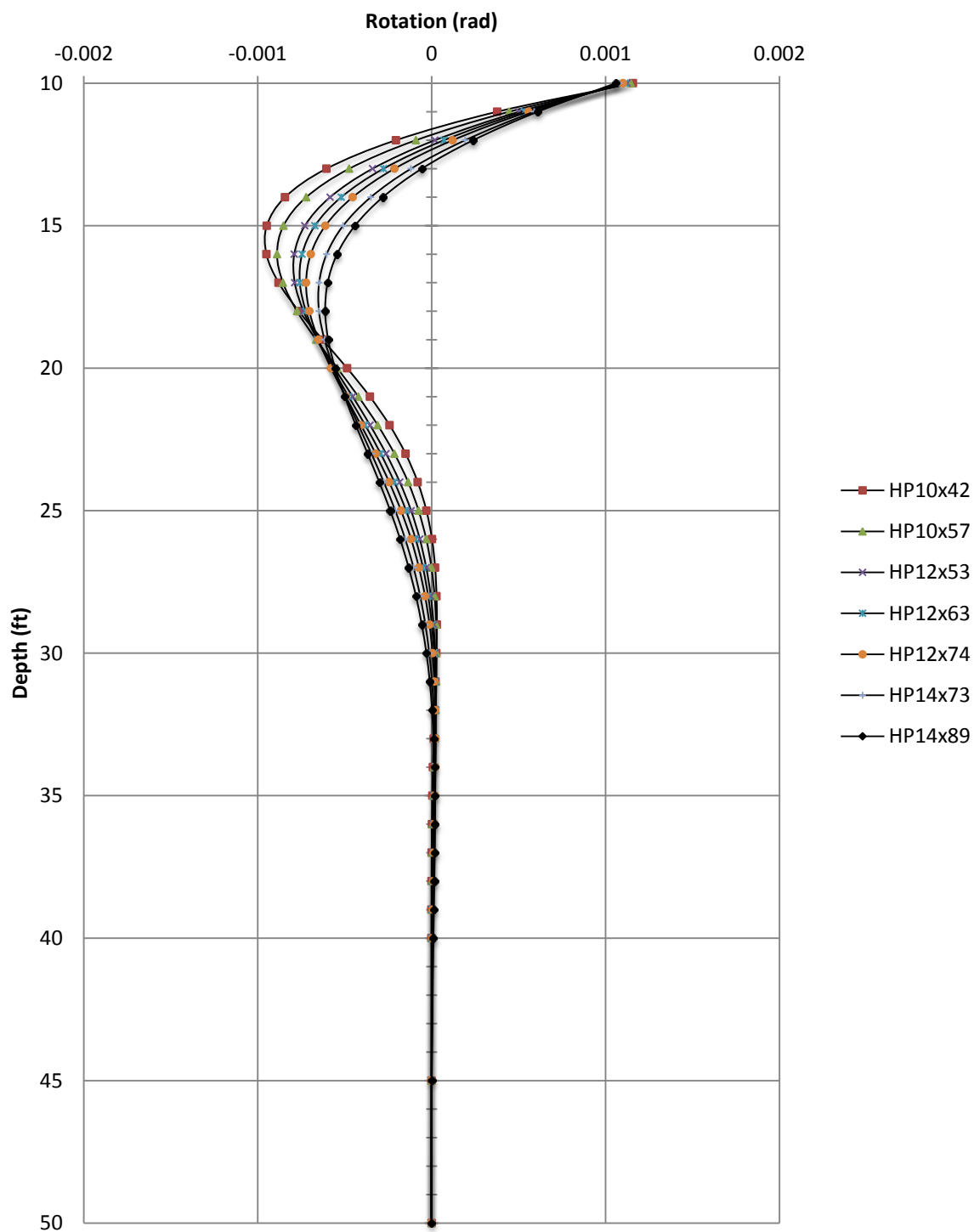


Figure B-33 : Bridge II pile rotation in medium sand under Truck LL

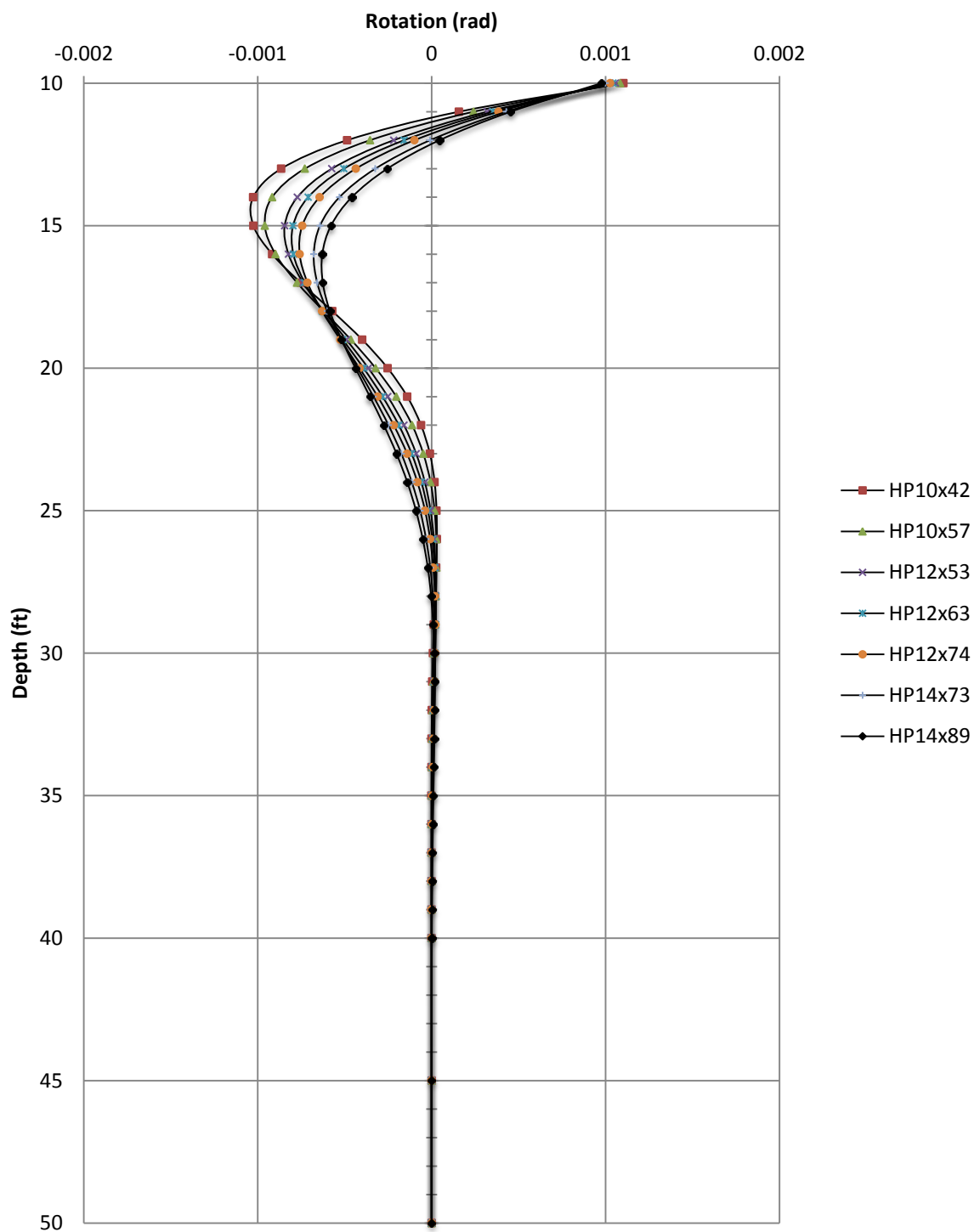


Figure B-34 : Bridge II pile rotation in dense sand under Truck LL

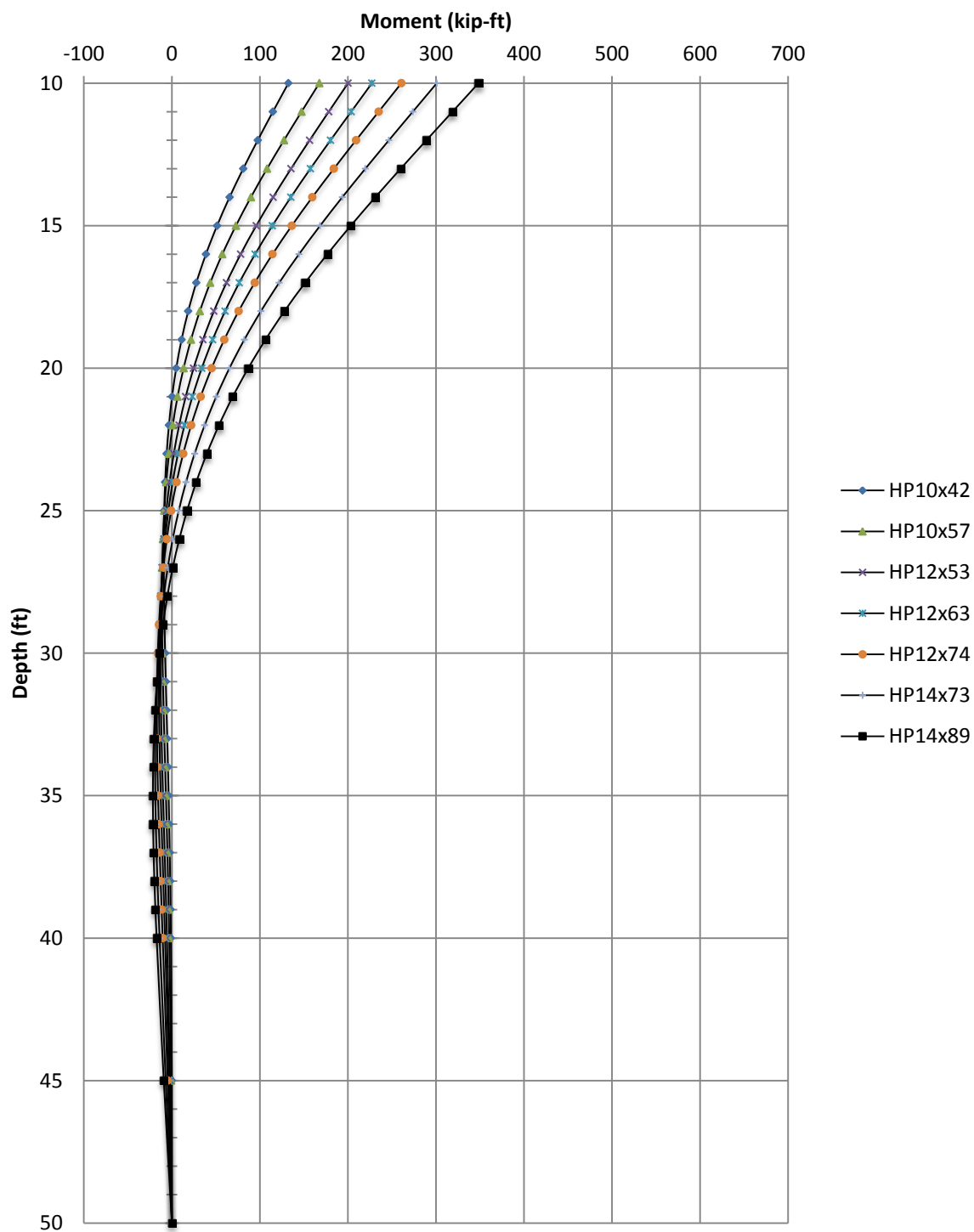


Figure B-35 : Bridge II pile moments in soft clay under Truck LL

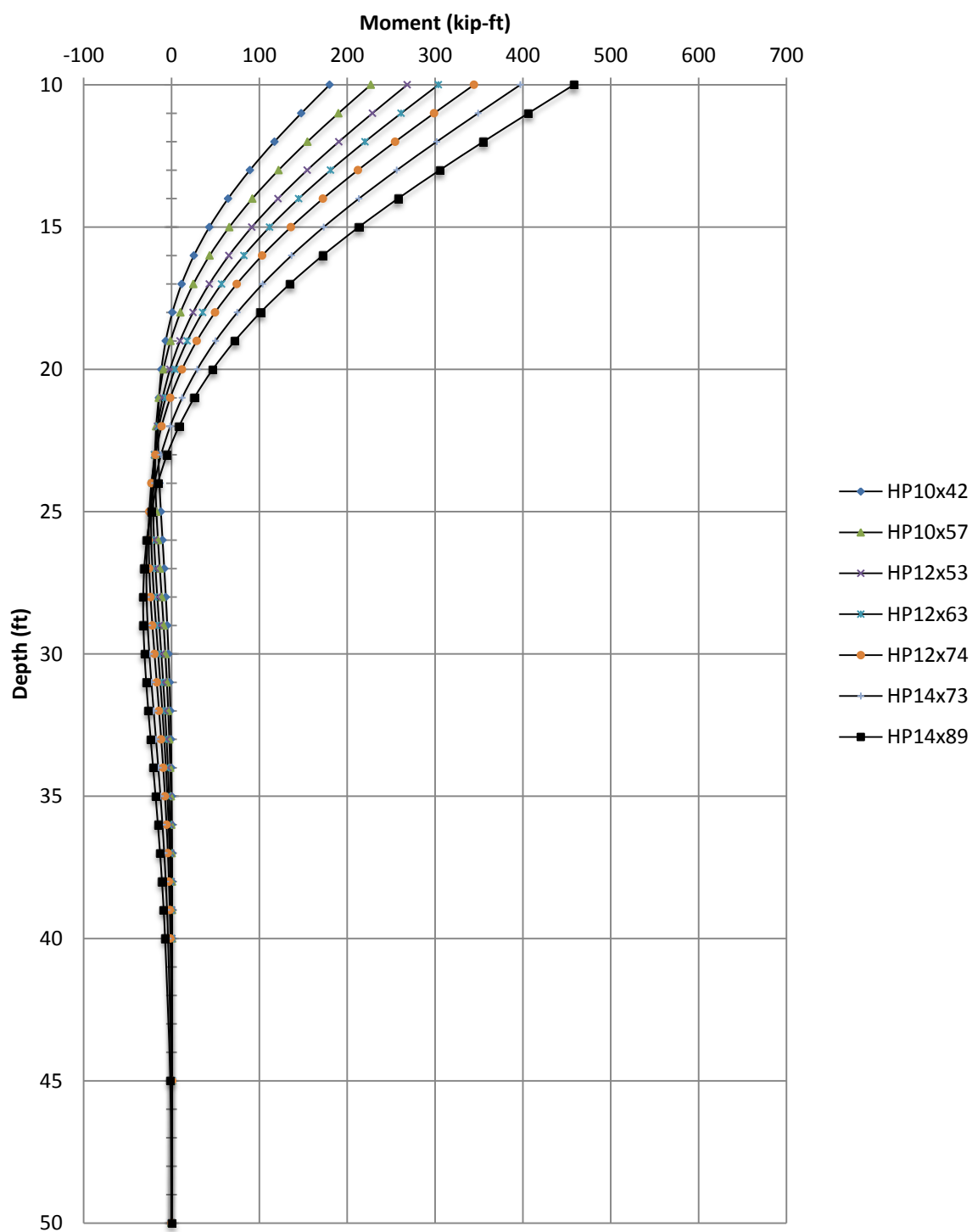


Figure B-36 : Bridge II pile moments in medium clay under Truck LL

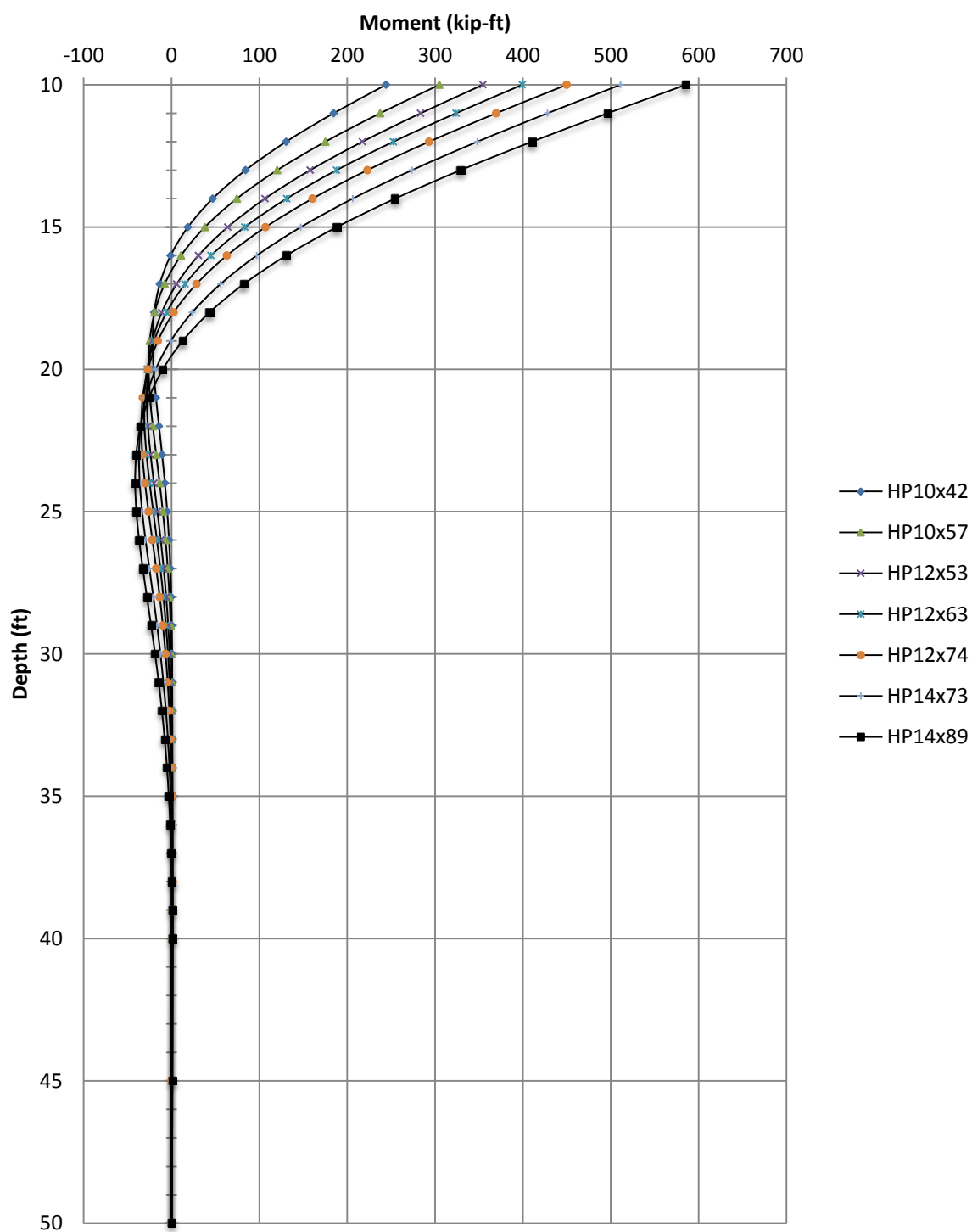


Figure B-37 : Bridge II pile moments in stiff clay under Truck LL

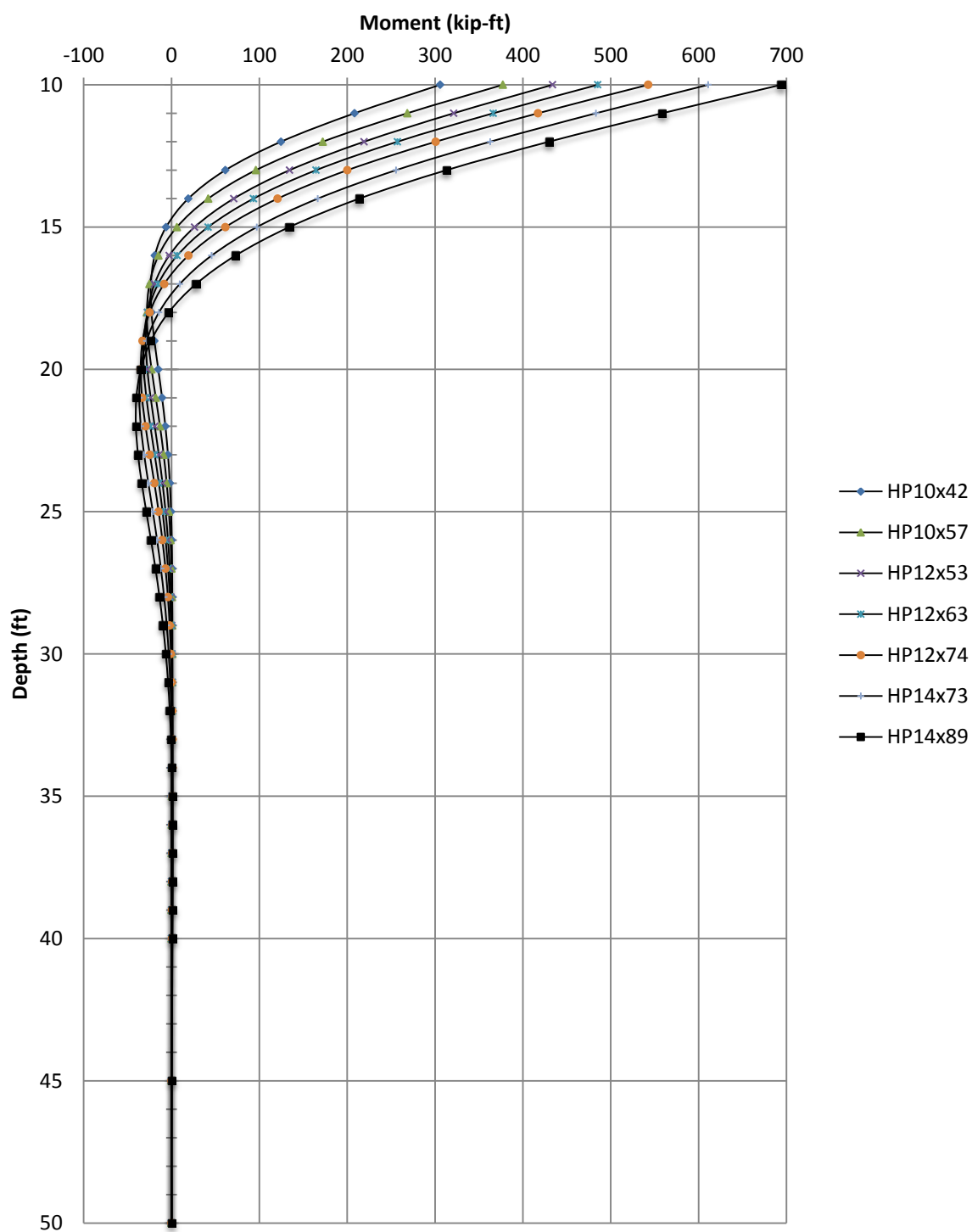


Figure B-38 : Bridge II pile moments in very stiff clay under Truck LL

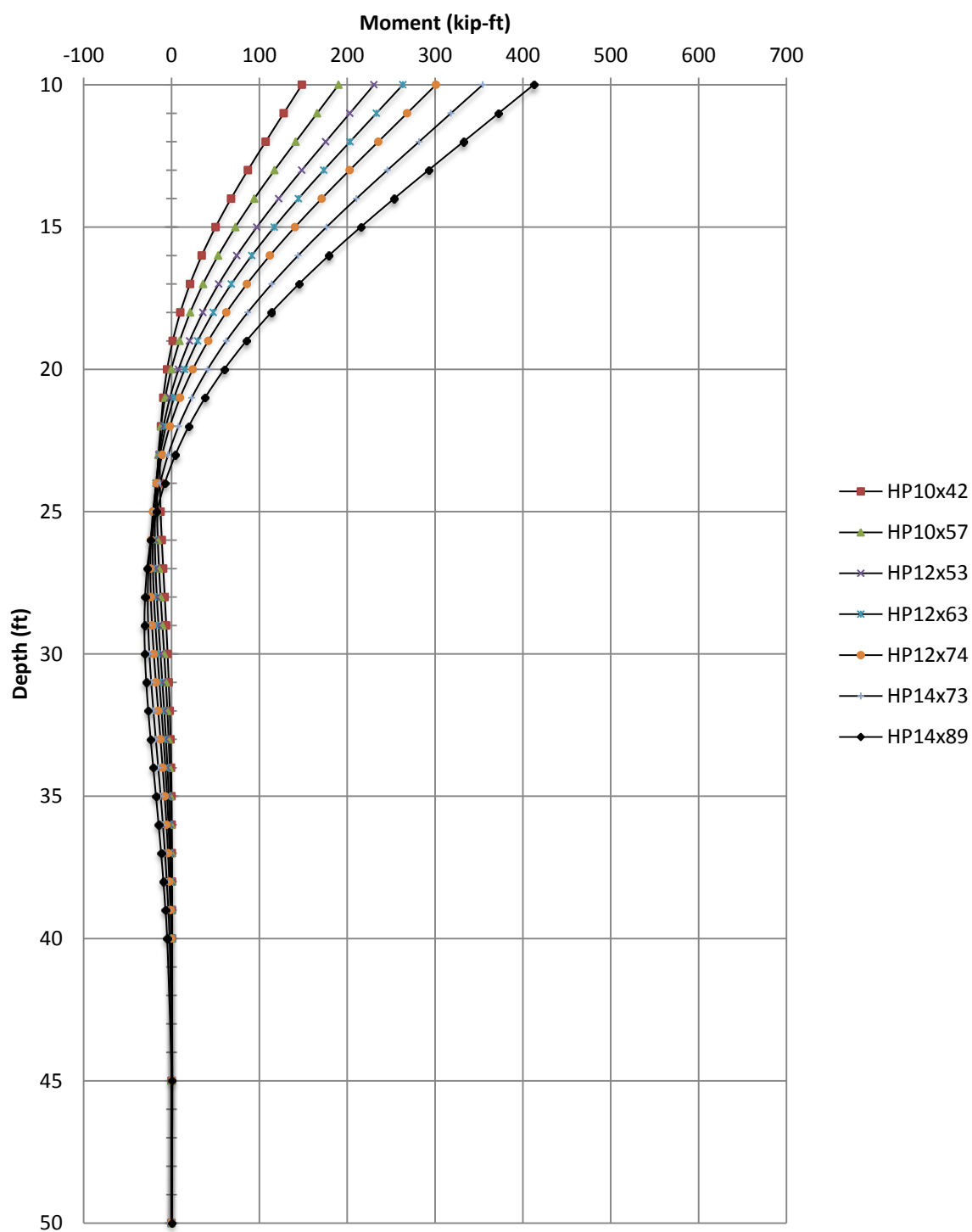


Figure B-39 : Bridge II pile moments in loose sand under Truck LL

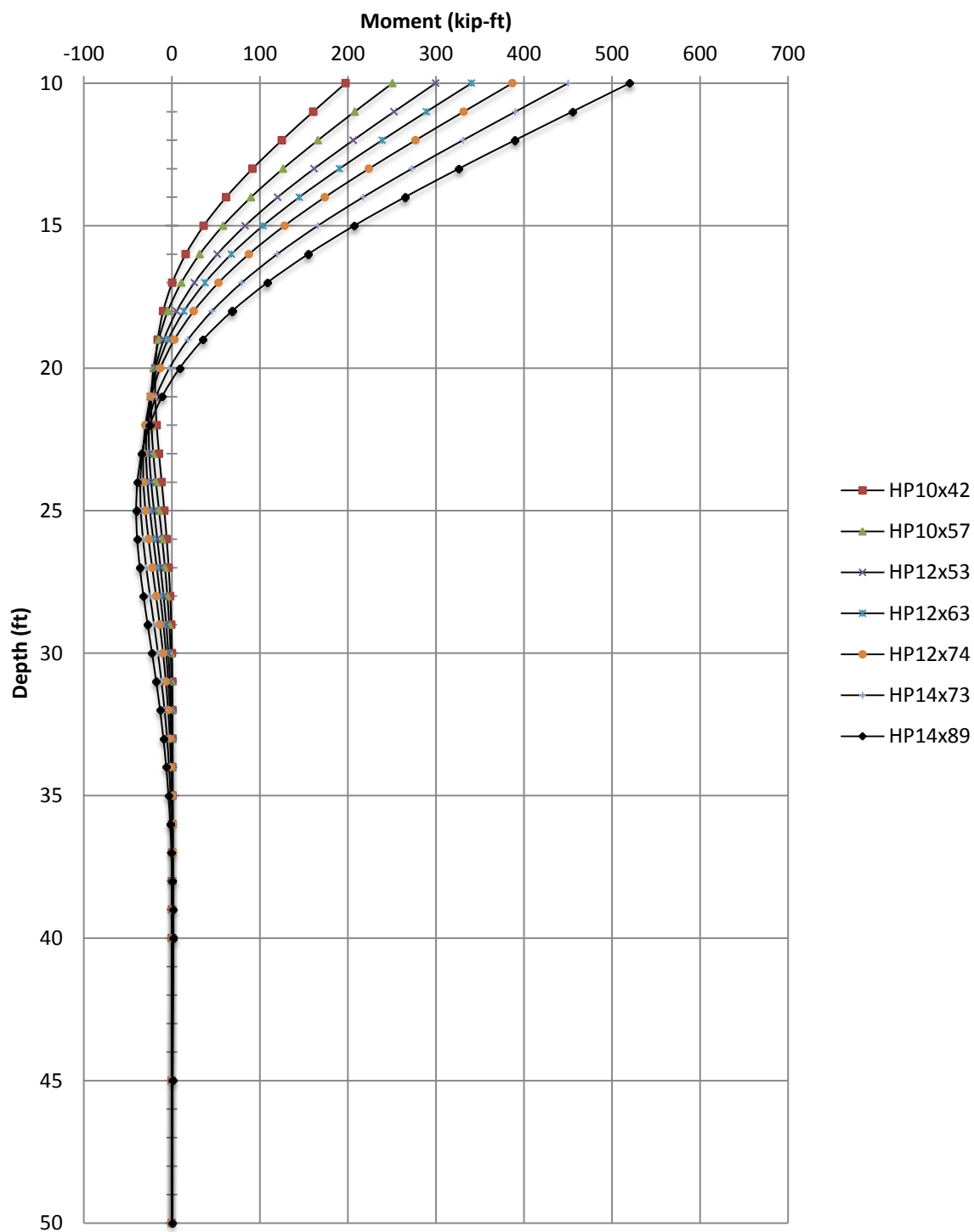


Figure B-40 : Bridge II pile moments in medium sand under Truck LL

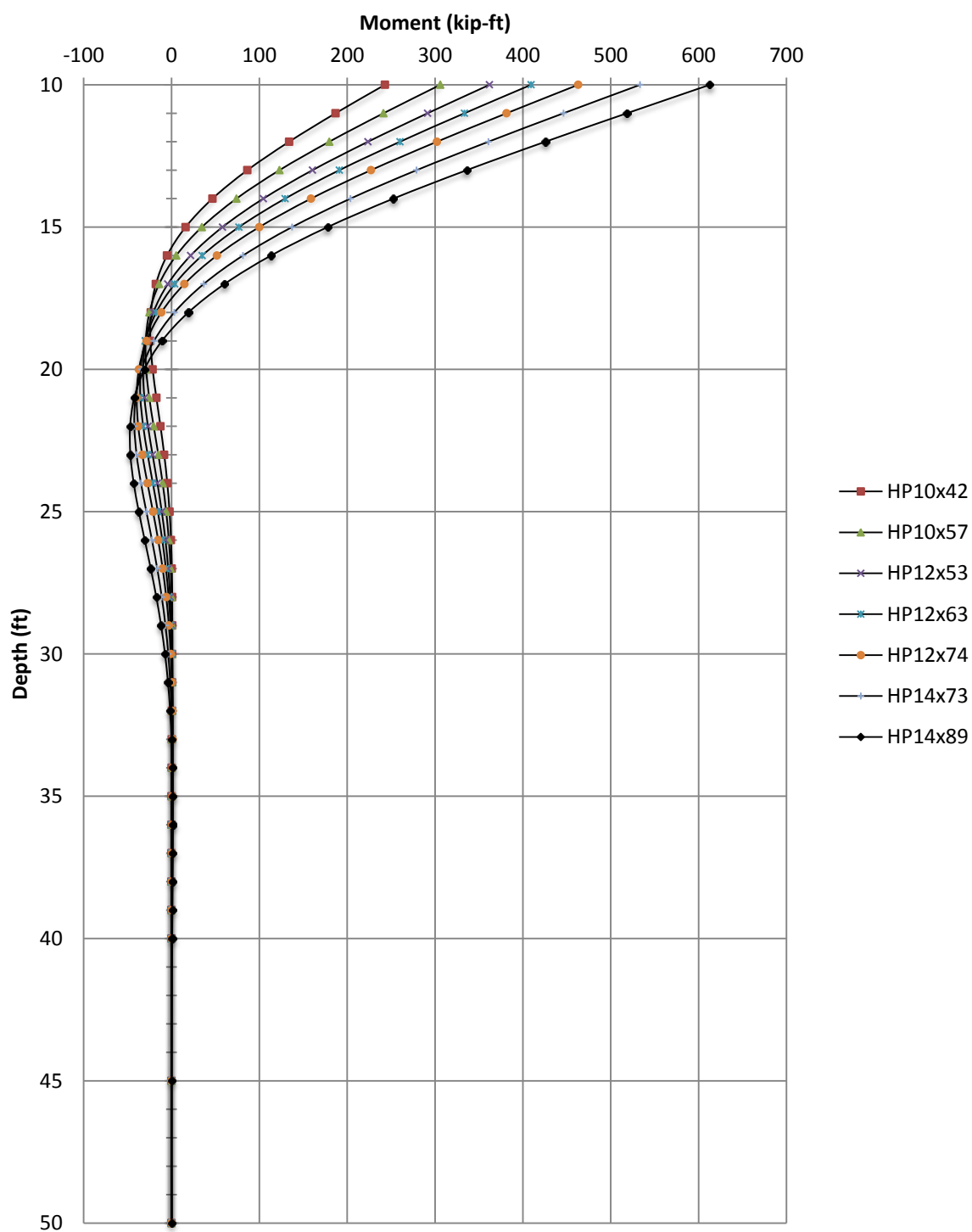


Figure B-41 : Bridge II pile moments in dense sand under Truck LL

BRIDGE III

PILE DEFLECTION PROFILES

PILE ROTATION PROFILES

PILE MOMENT PROFILES

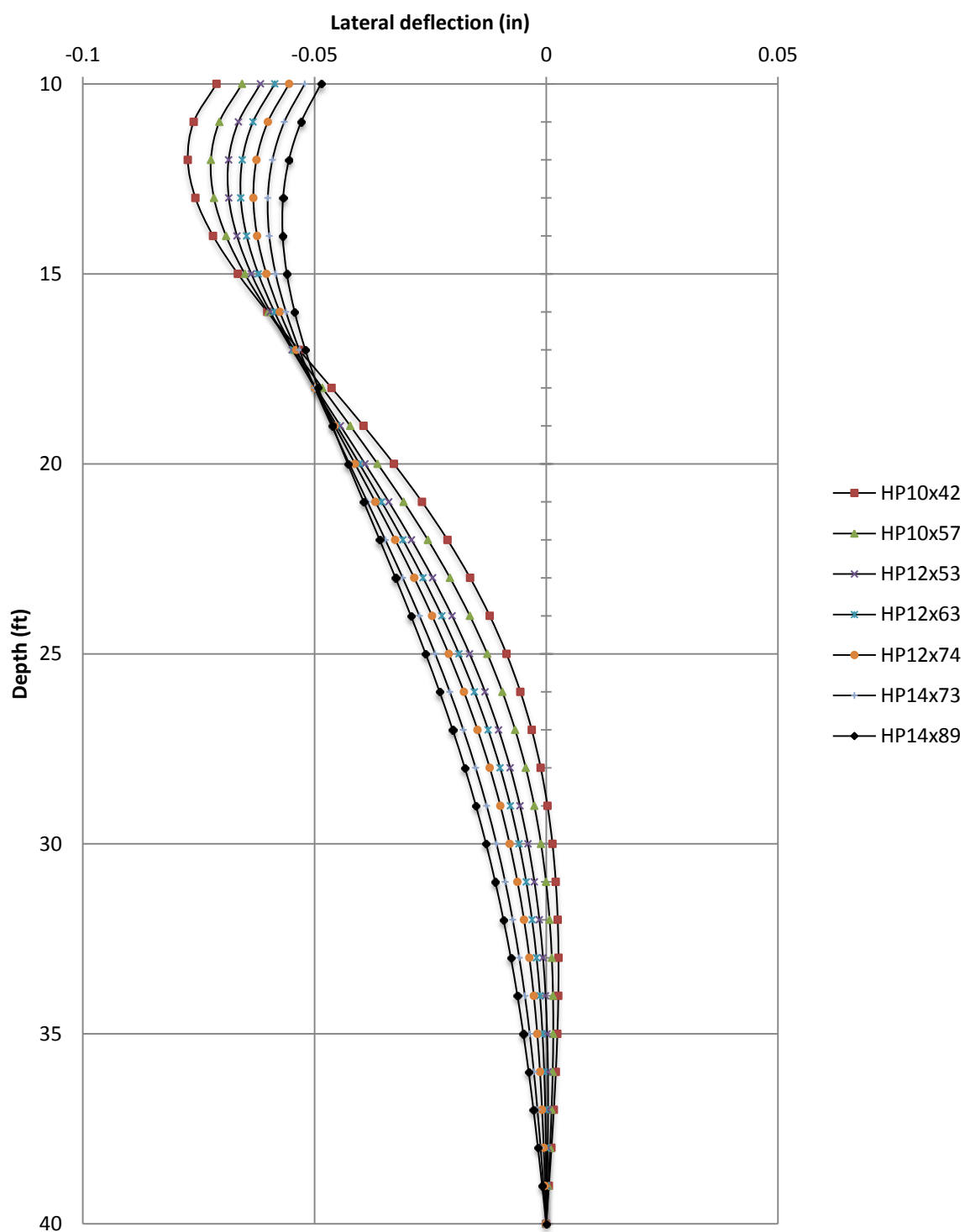


Figure B-42 : Lateral deflection of piles of Bridge III in soft clay under Truck LL

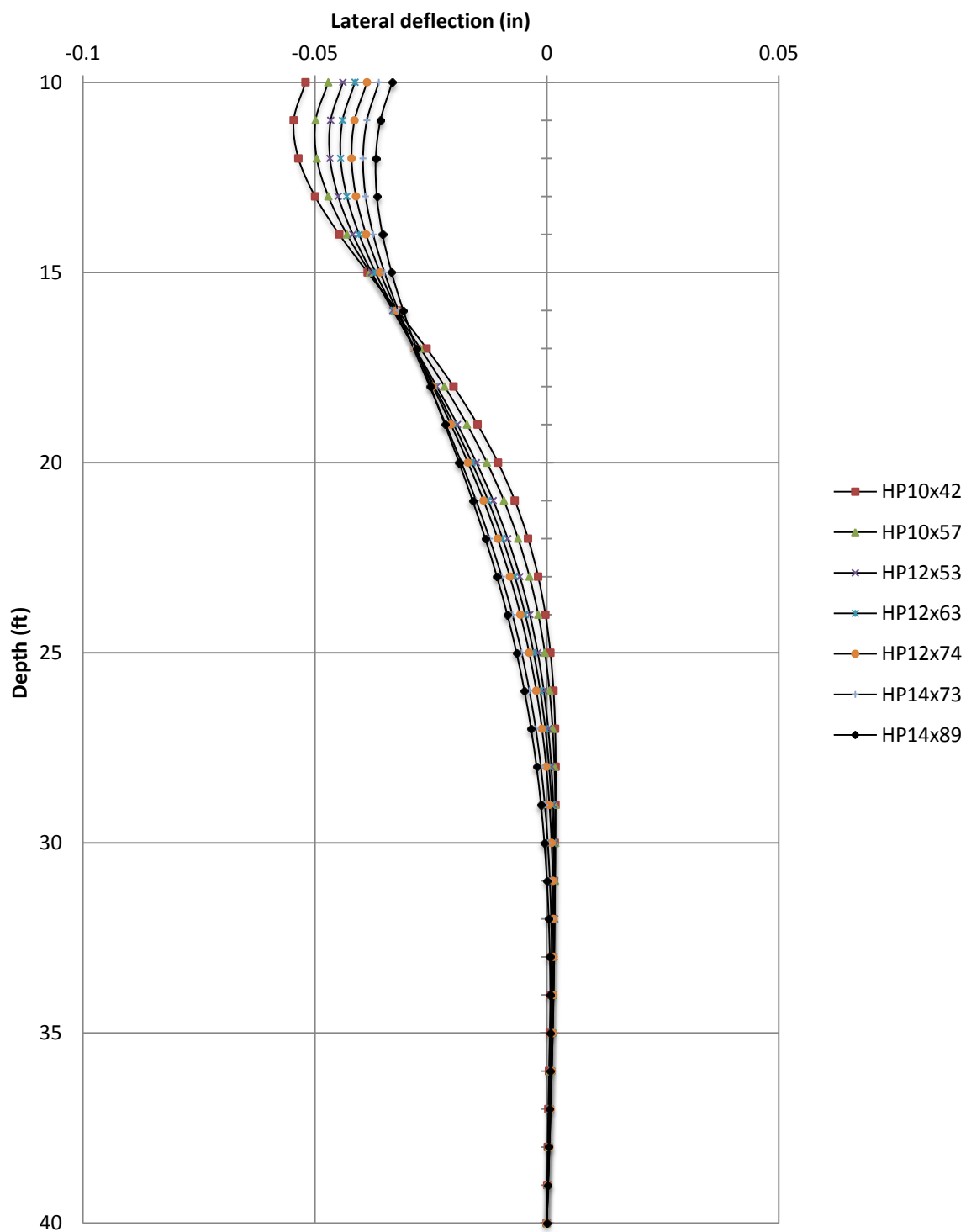


Figure B-43 : Lateral deflection of piles of Bridge III in medium clay under Truck LL

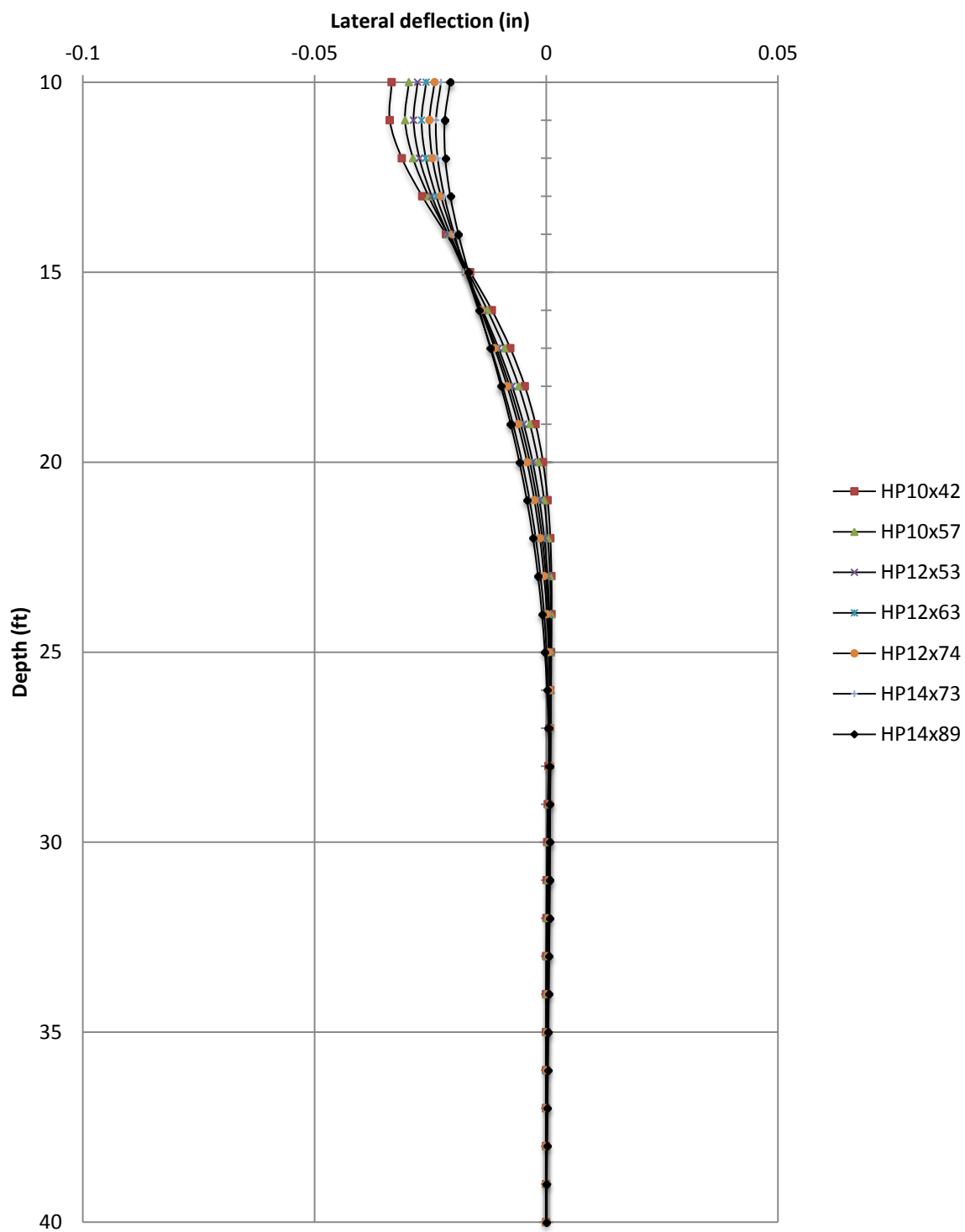


Figure B-44 : Lateral deflection of piles of Bridge III in stiff clay under Truck LL

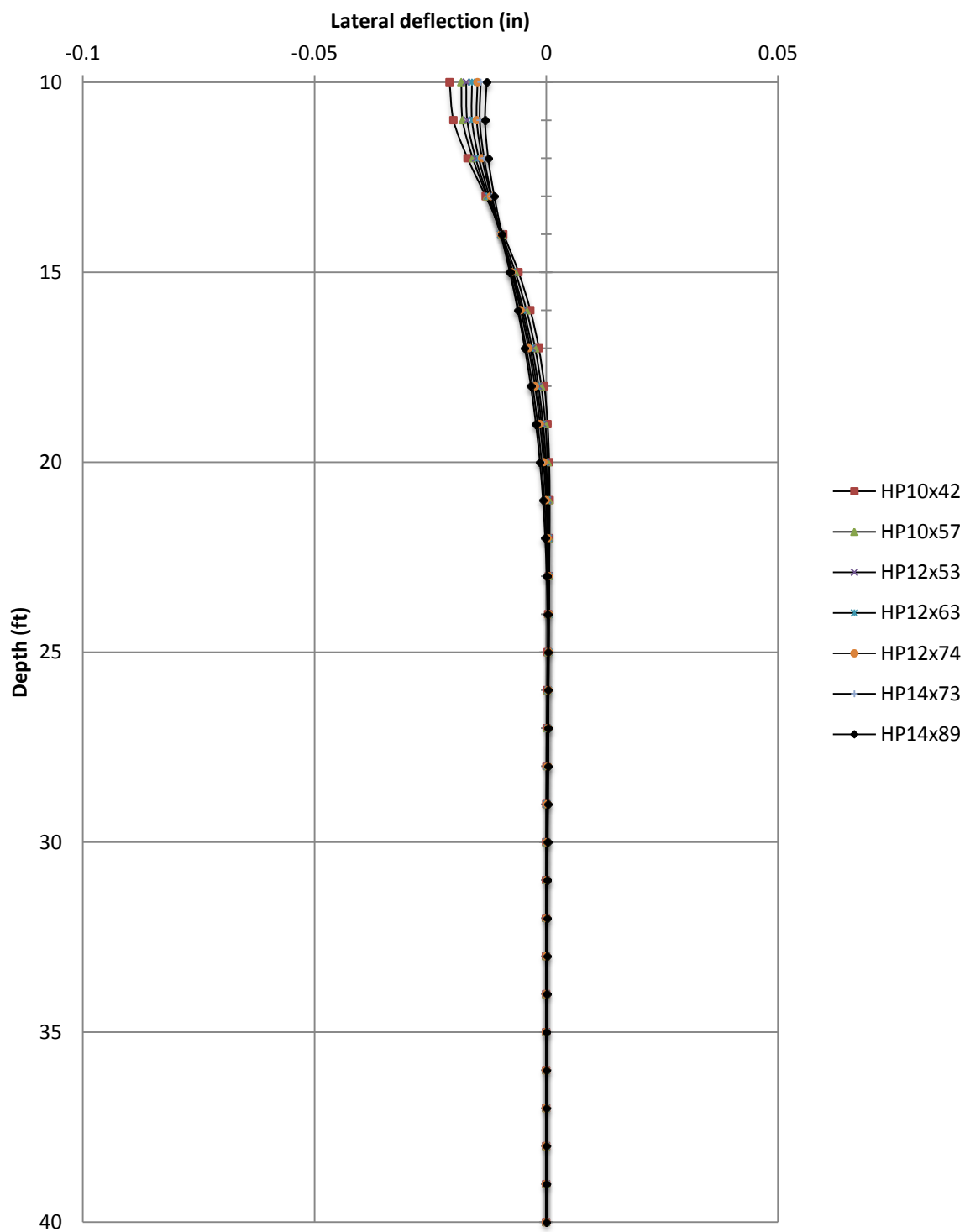


Figure B-45 : Lateral deflection of piles of Bridge III in very stiff clay under Truck LL

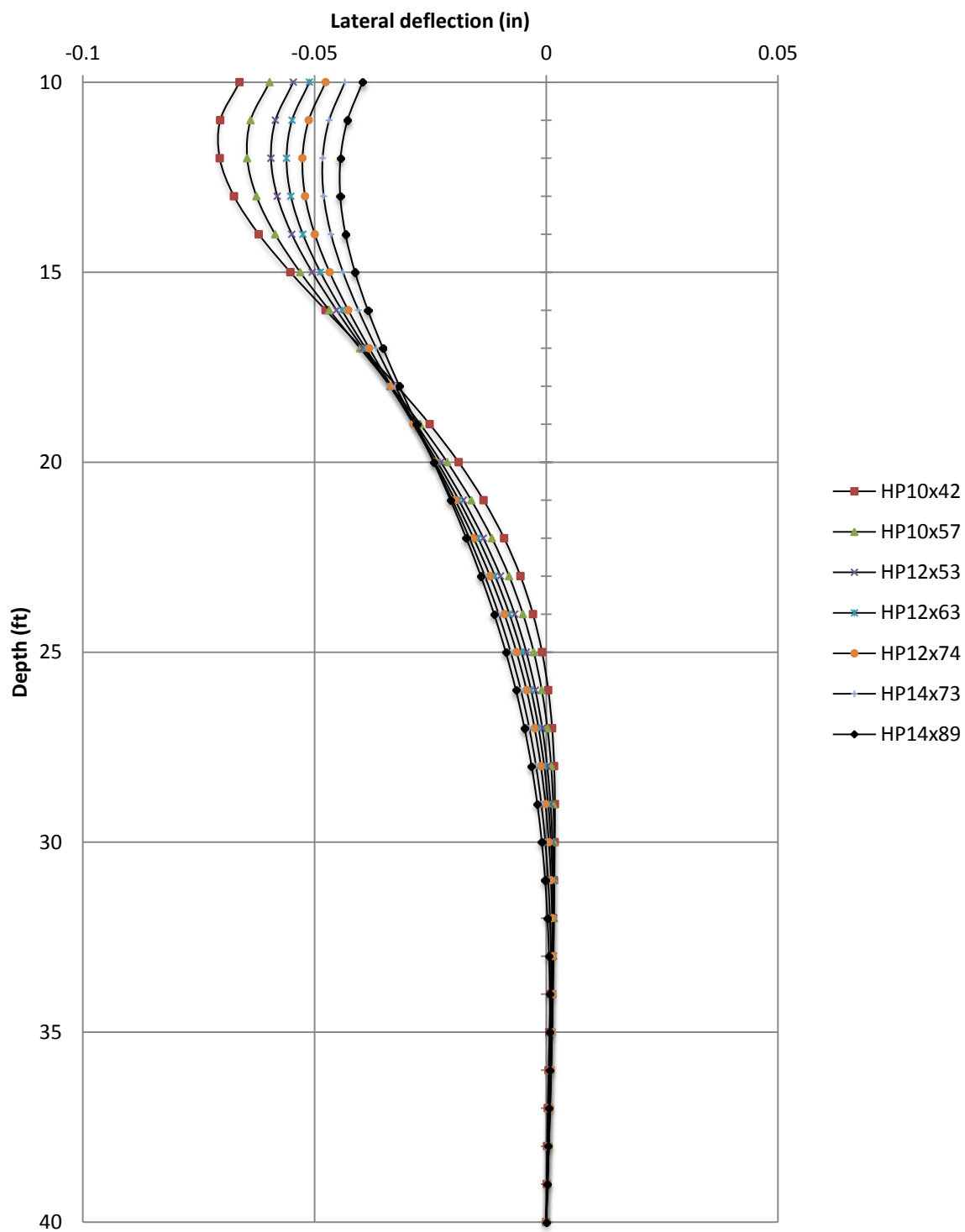


Figure B-46 : Lateral deflection of piles of Bridge III in loose sand under Truck LL

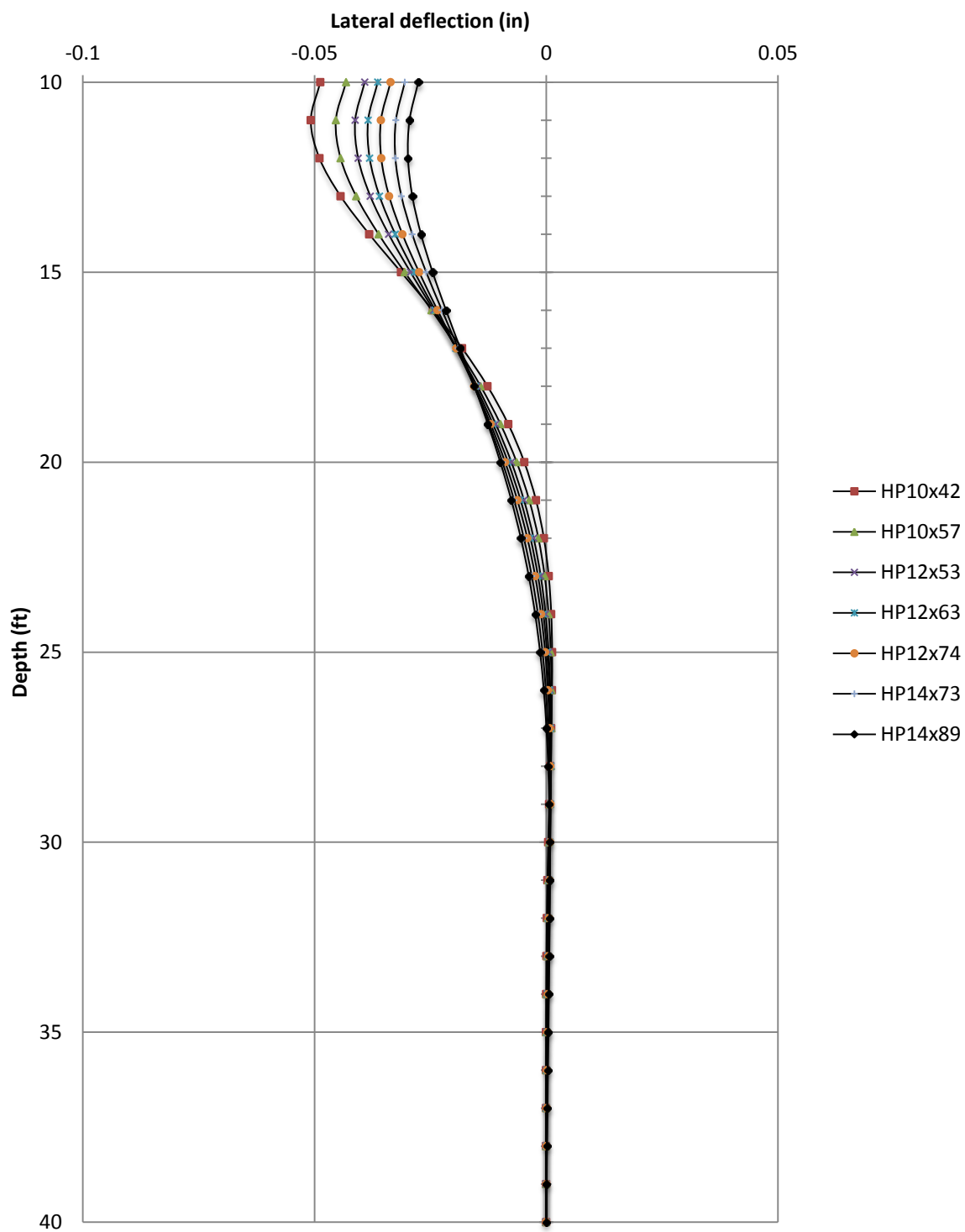


Figure B-47 : Lateral deflection of piles of Bridge III in medium sand under Truck LL

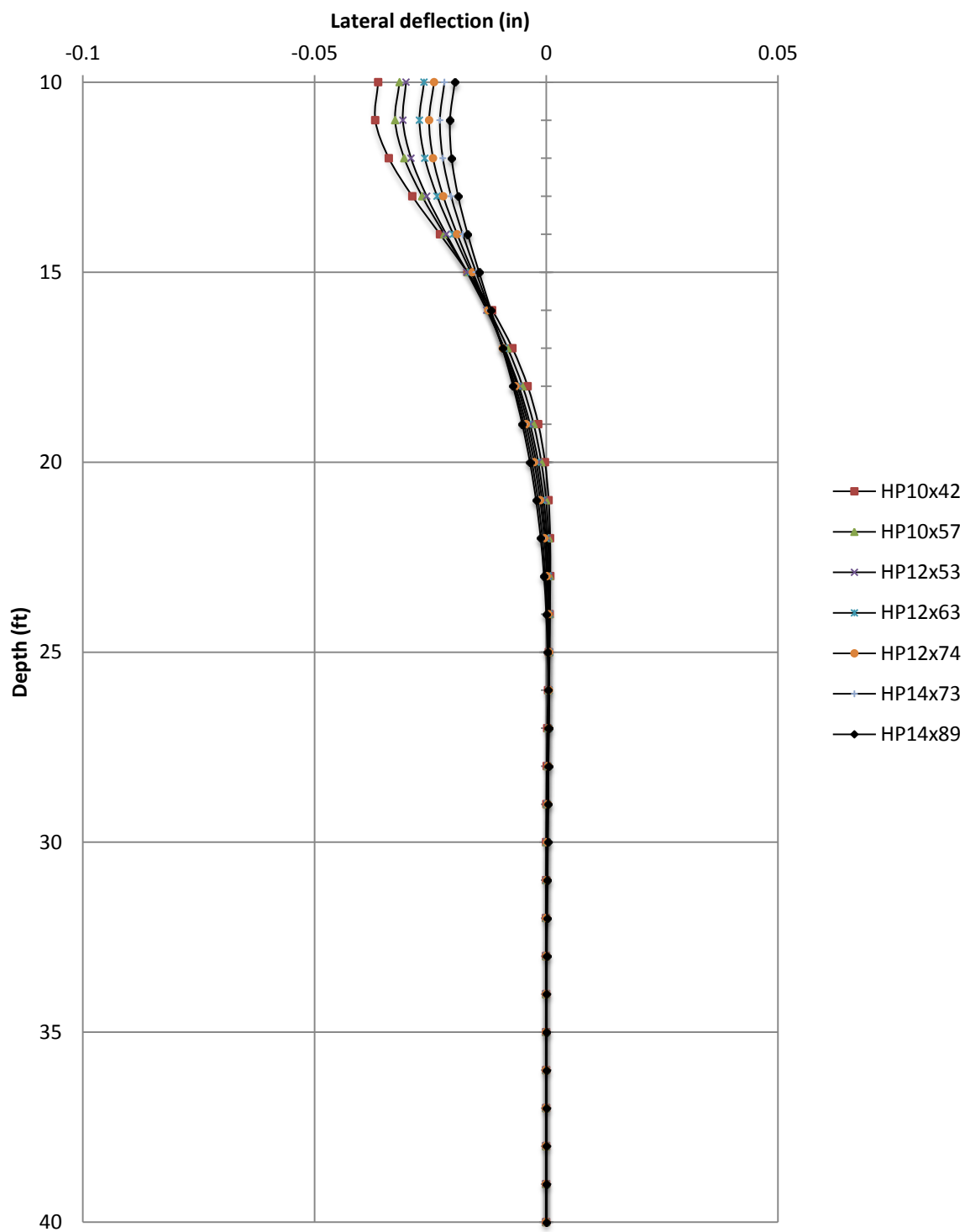


Figure B-48 : Lateral deflection of piles of Bridge III in dense sand under Truck LL

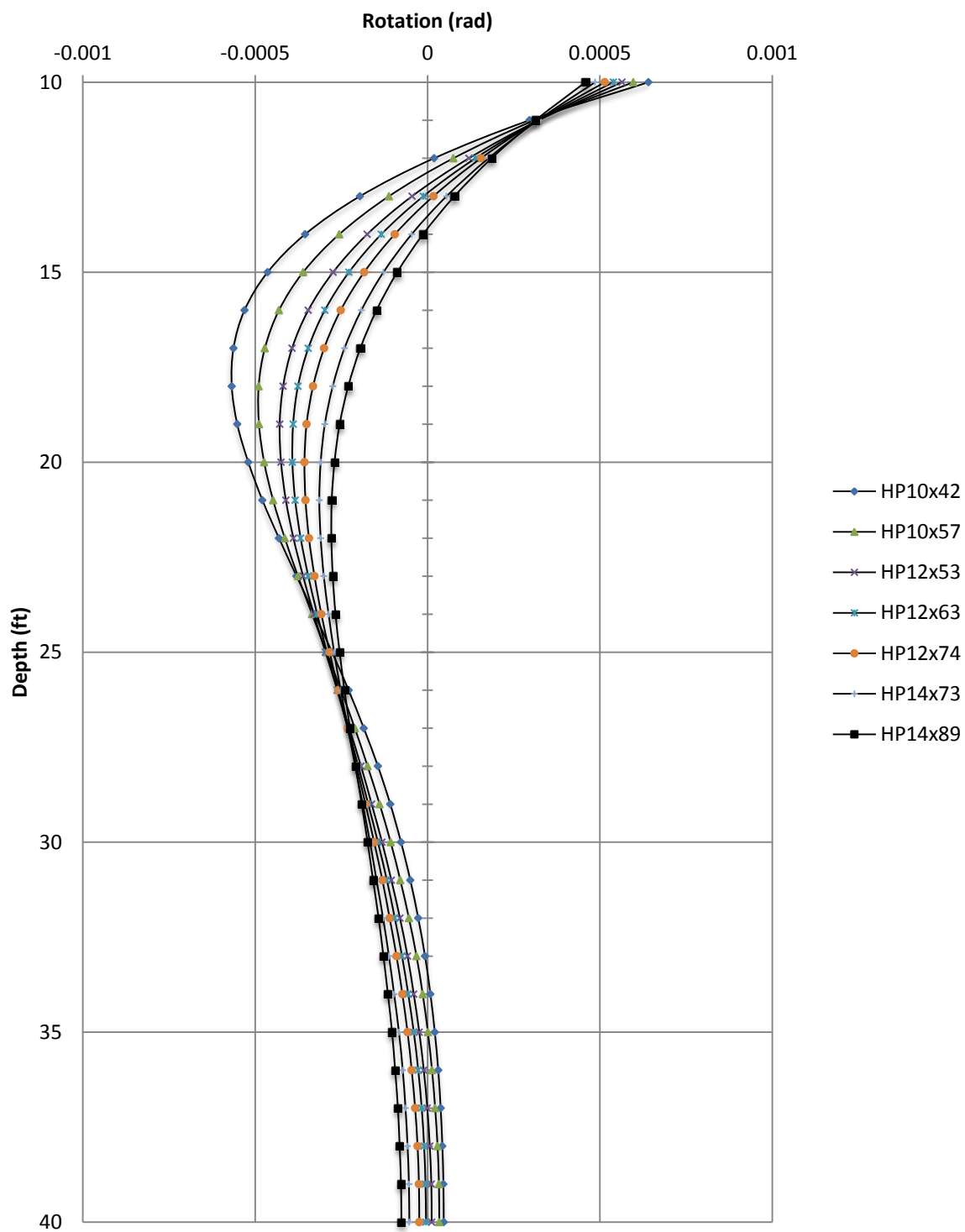


Figure B-49 : Bridge III pile rotation in soft clay under Truck LL

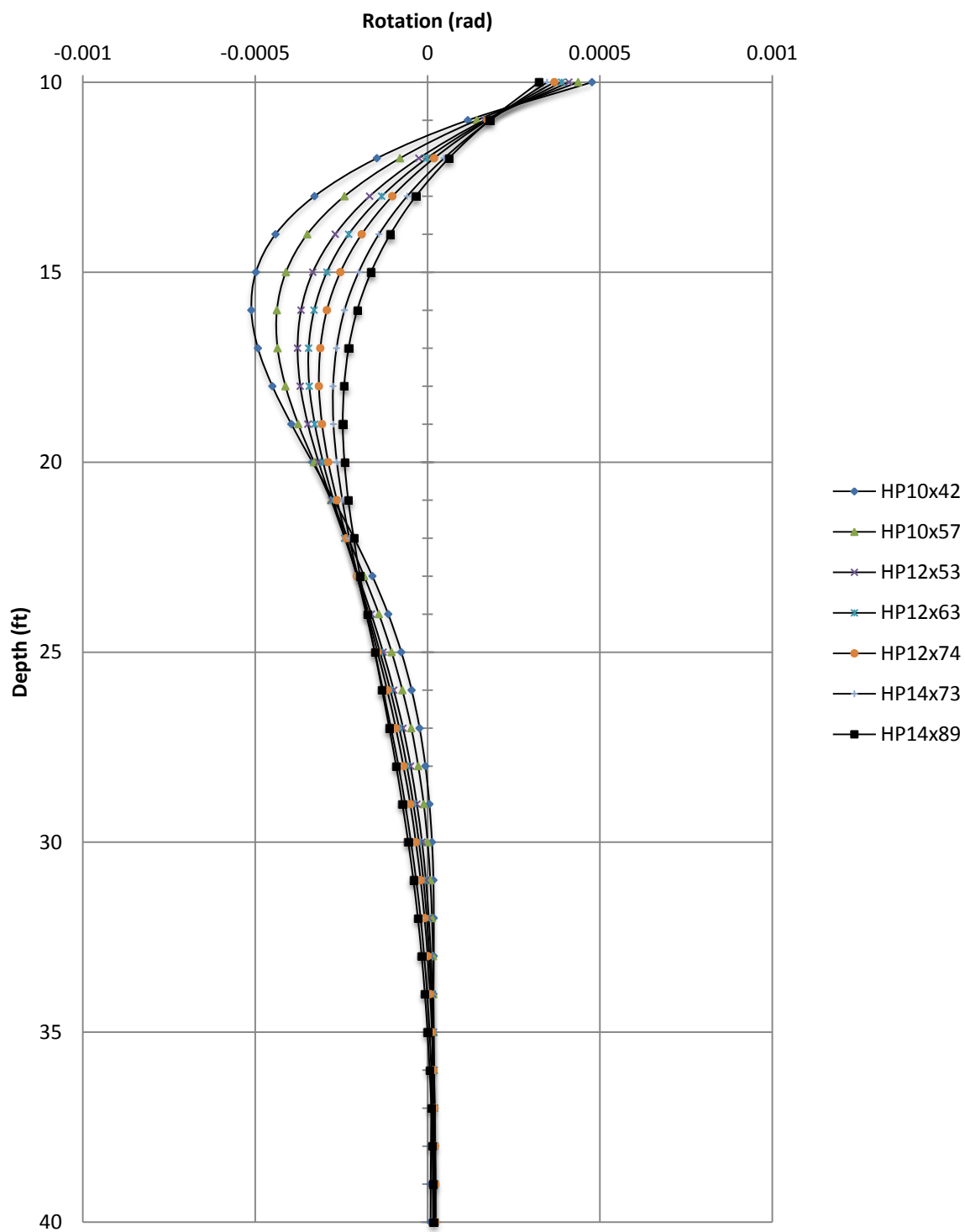


Figure B-50 : Bridge III pile rotation in medium clay under Truck LL

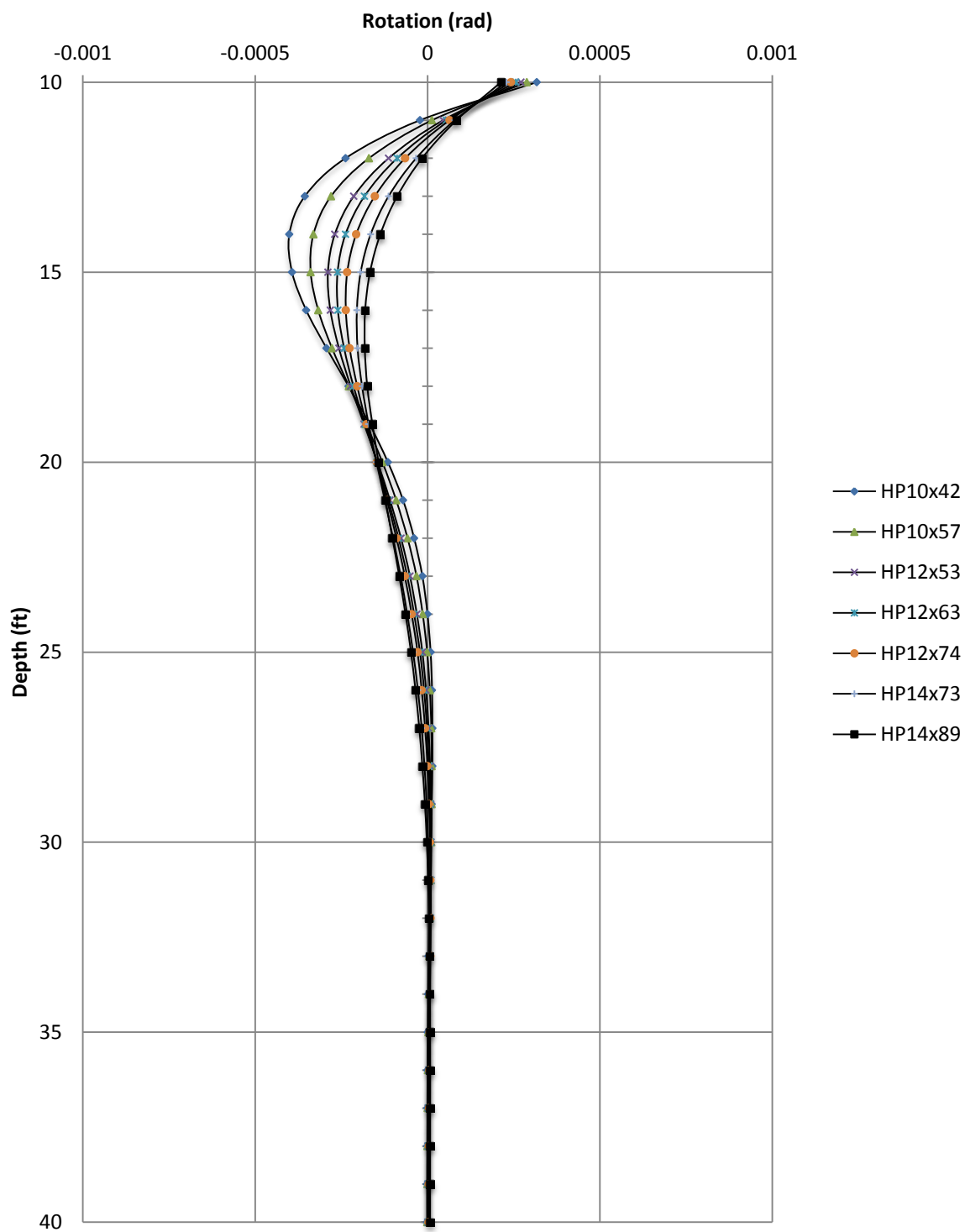


Figure B-51 : Bridge III pile rotation in stiff clay under Truck LL

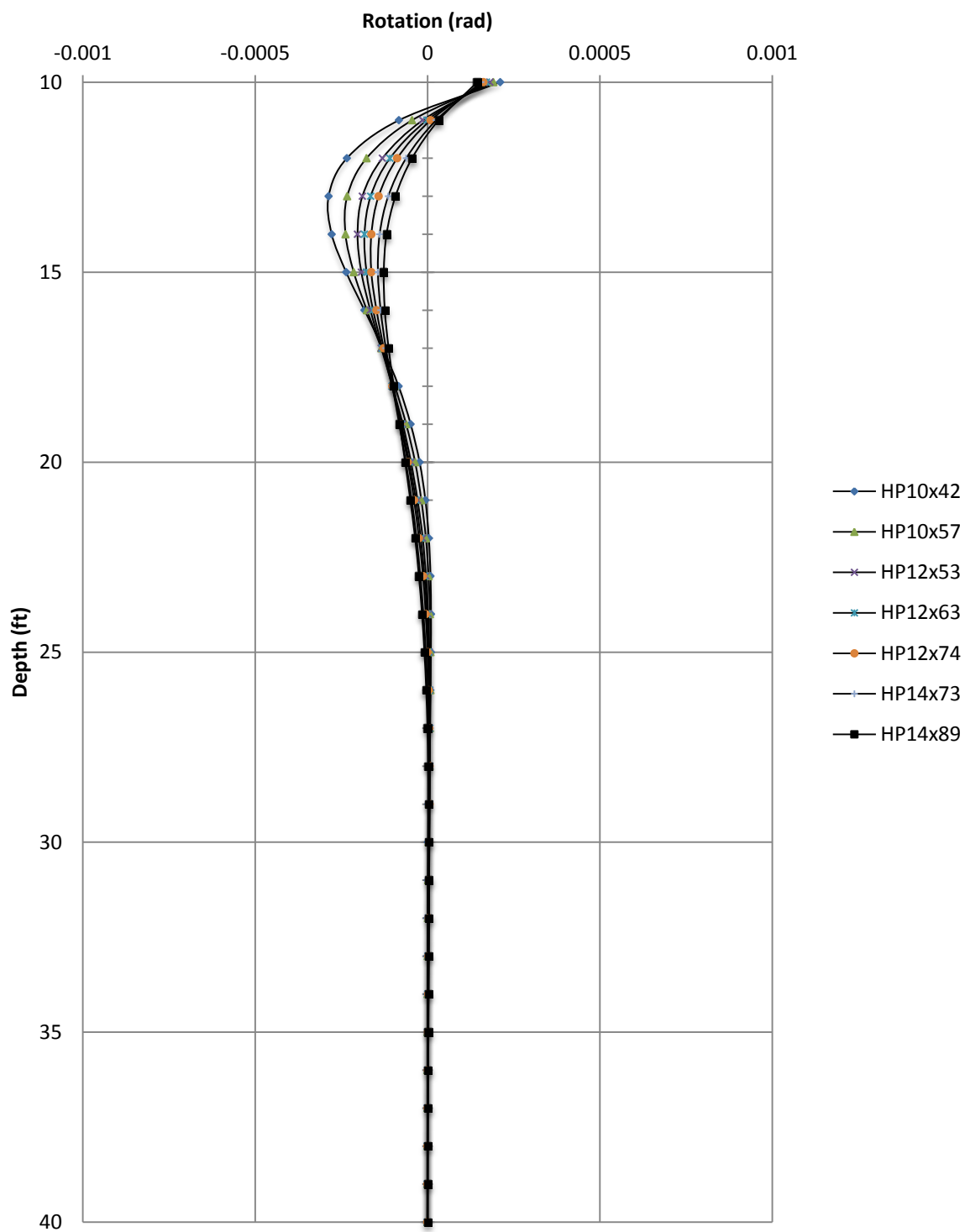


Figure B-52 : Bridge III pile rotation in very stiff clay under Truck LL

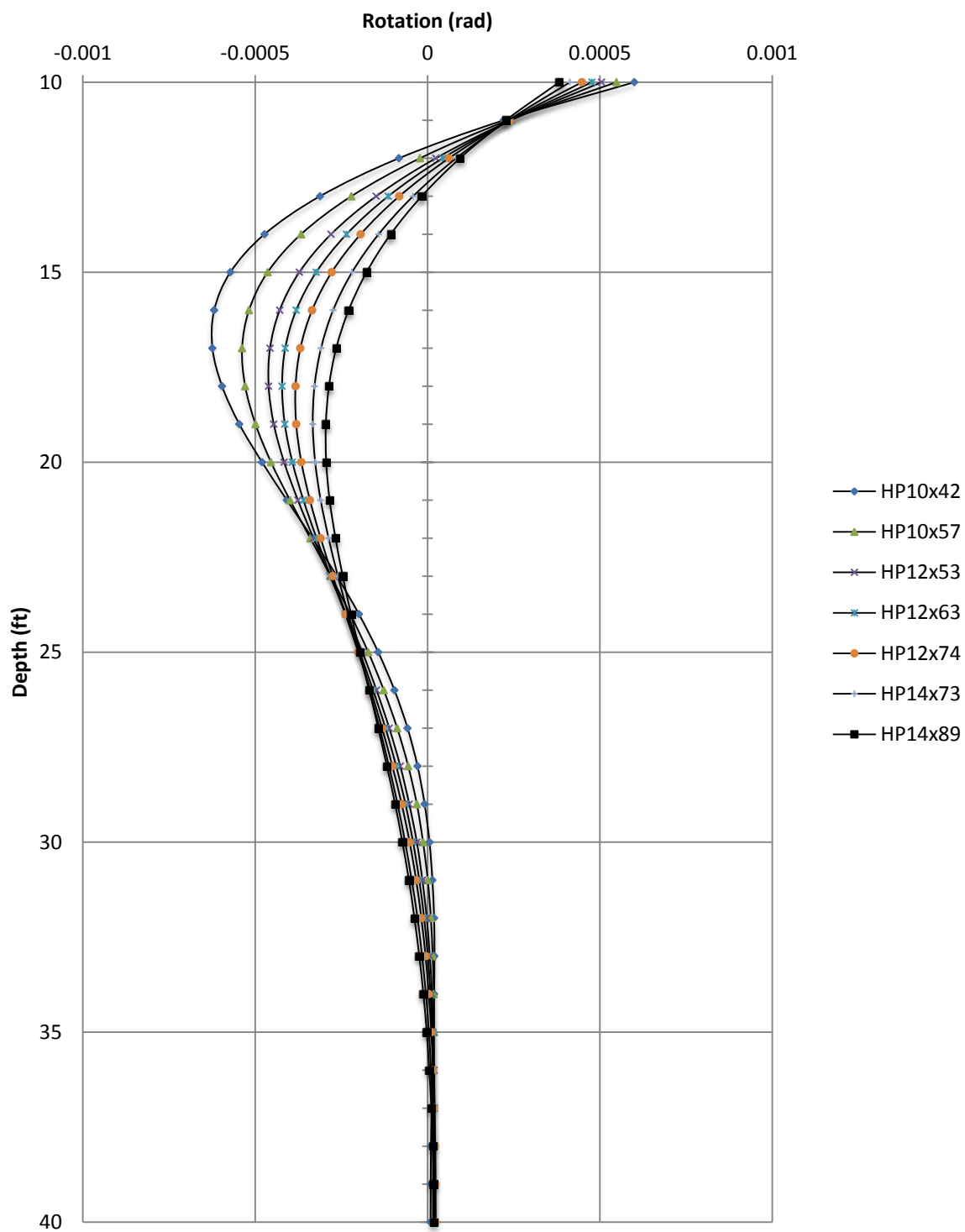


Figure B-53 : Bridge III pile rotation in loose sand under Truck LL

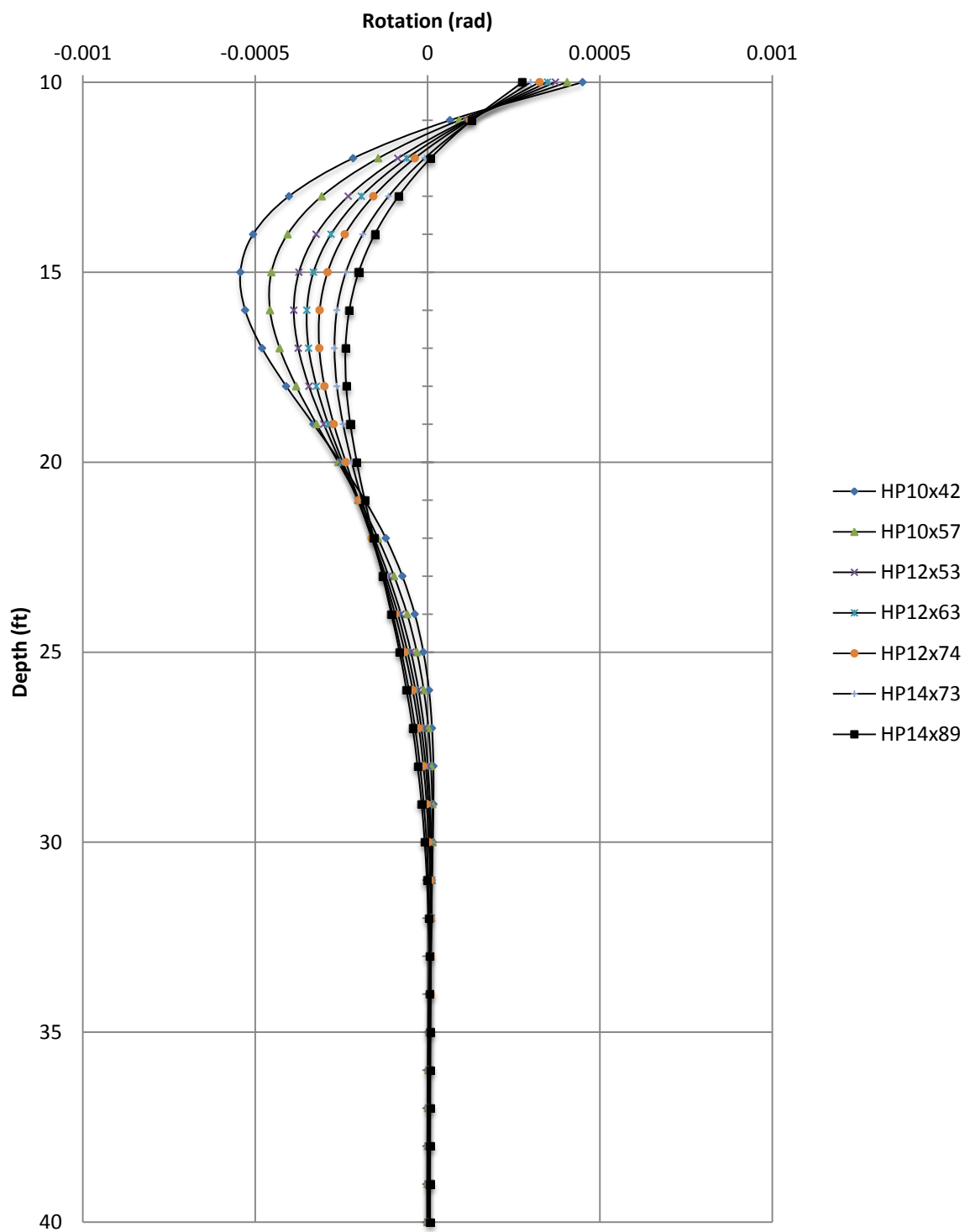


Figure B-54 : Bridge III pile rotation in medium sand under Truck LL

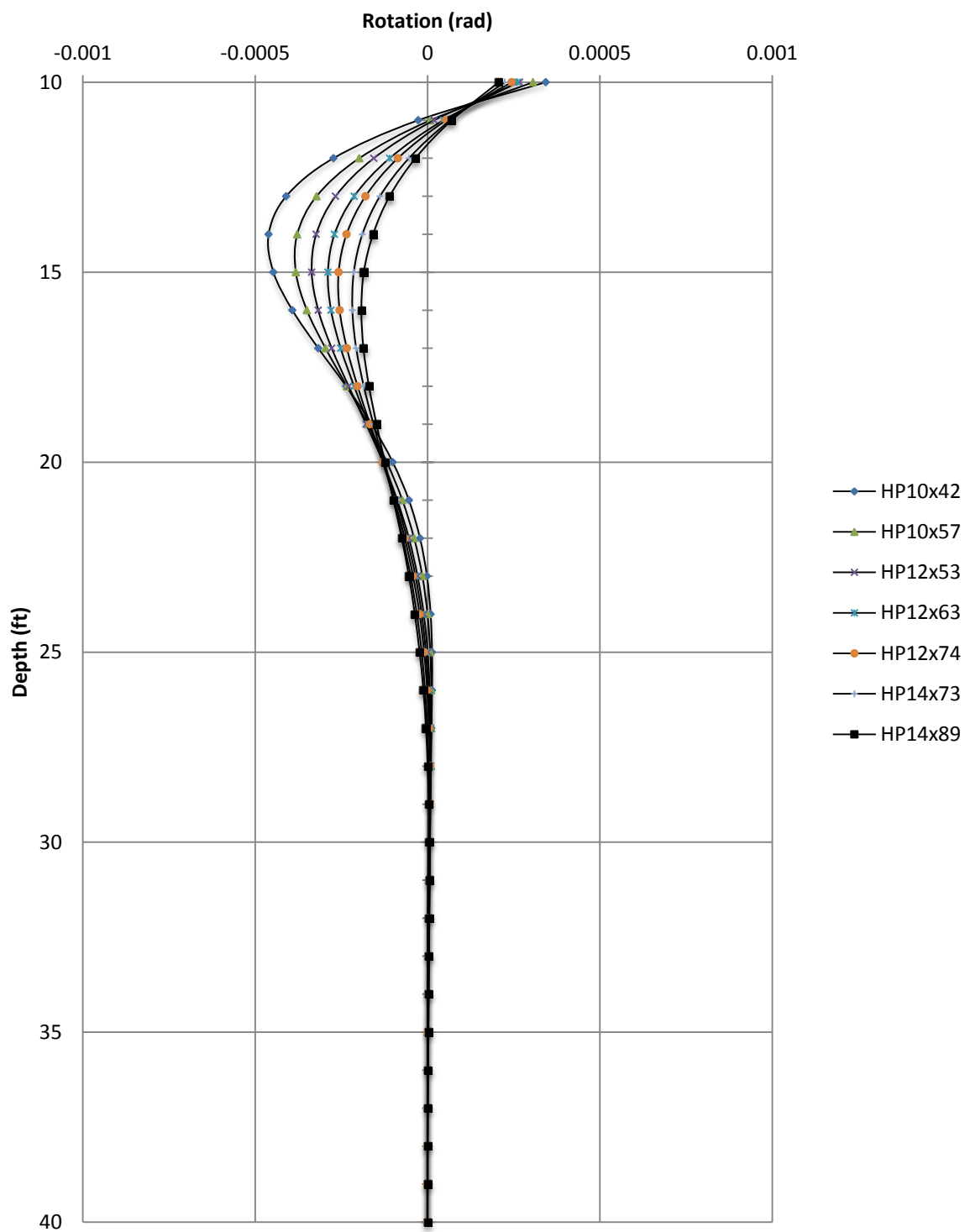


Figure B-55 : Bridge III pile rotation in dense sand under Truck LL

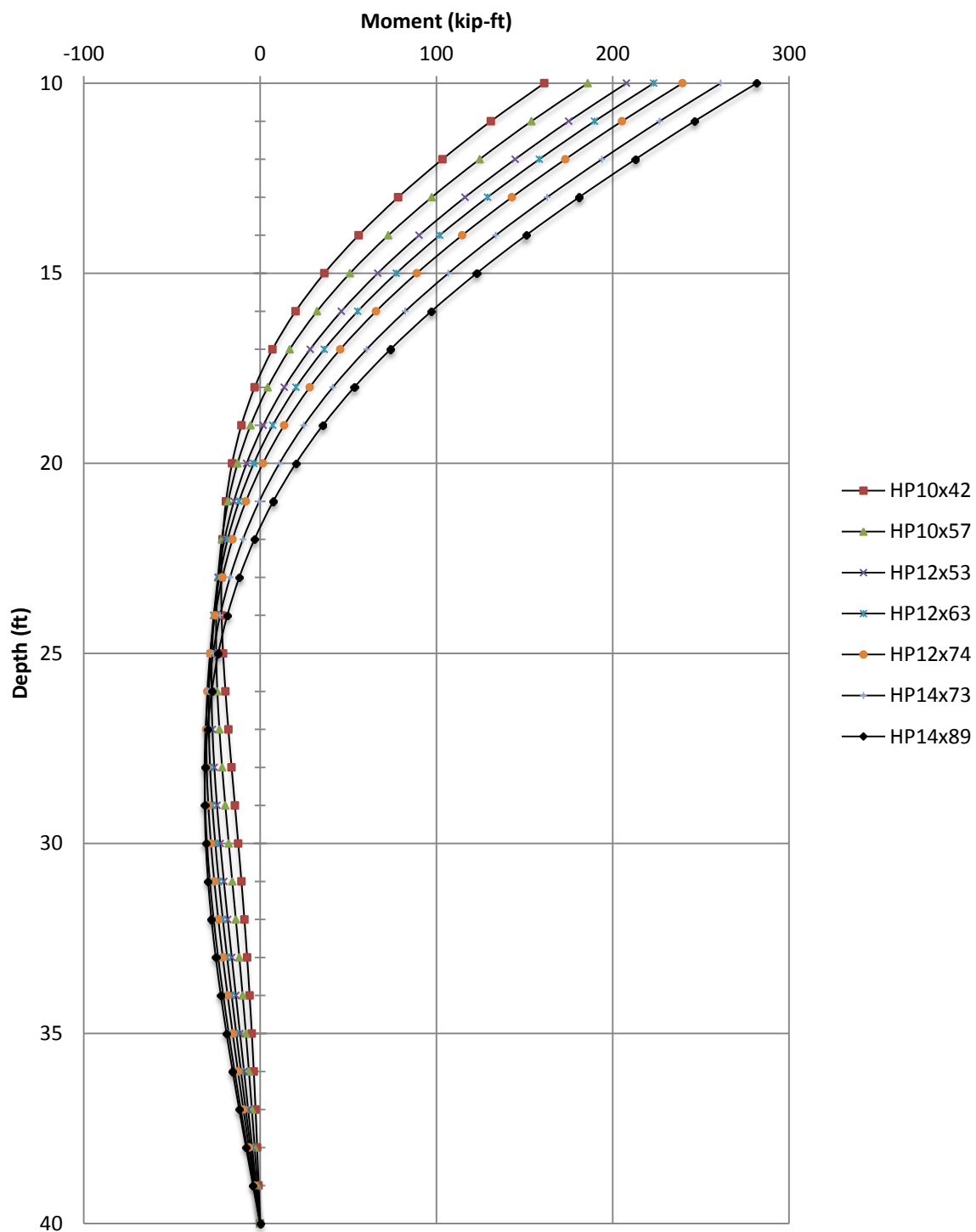


Figure B-56 : Bridge III pile moments in soft clay under Truck LL

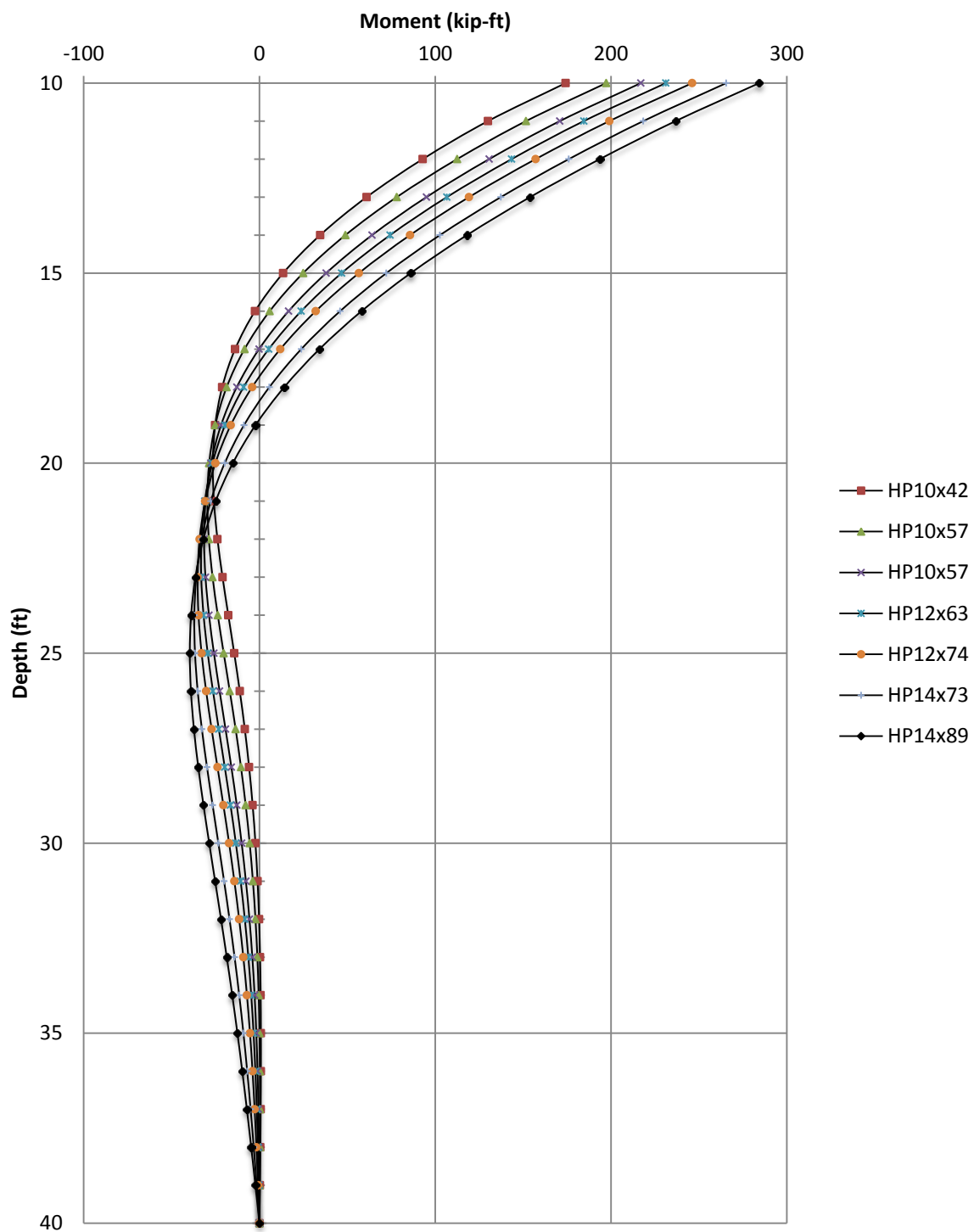


Figure B-57 : Bridge III pile moments in medium clay under Truck LL

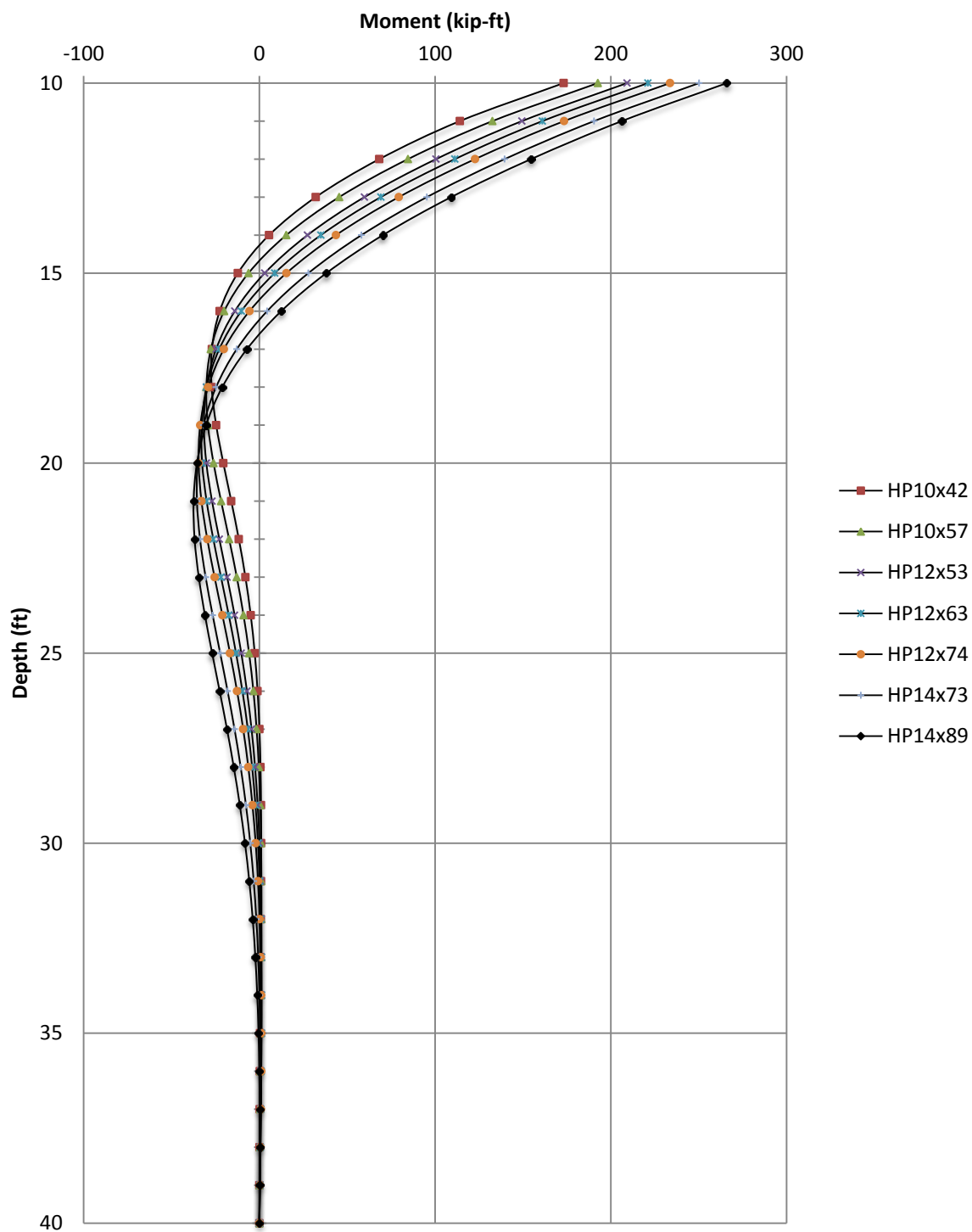


Figure B-58 : Bridge III pile moments in stiff clay under Truck LL

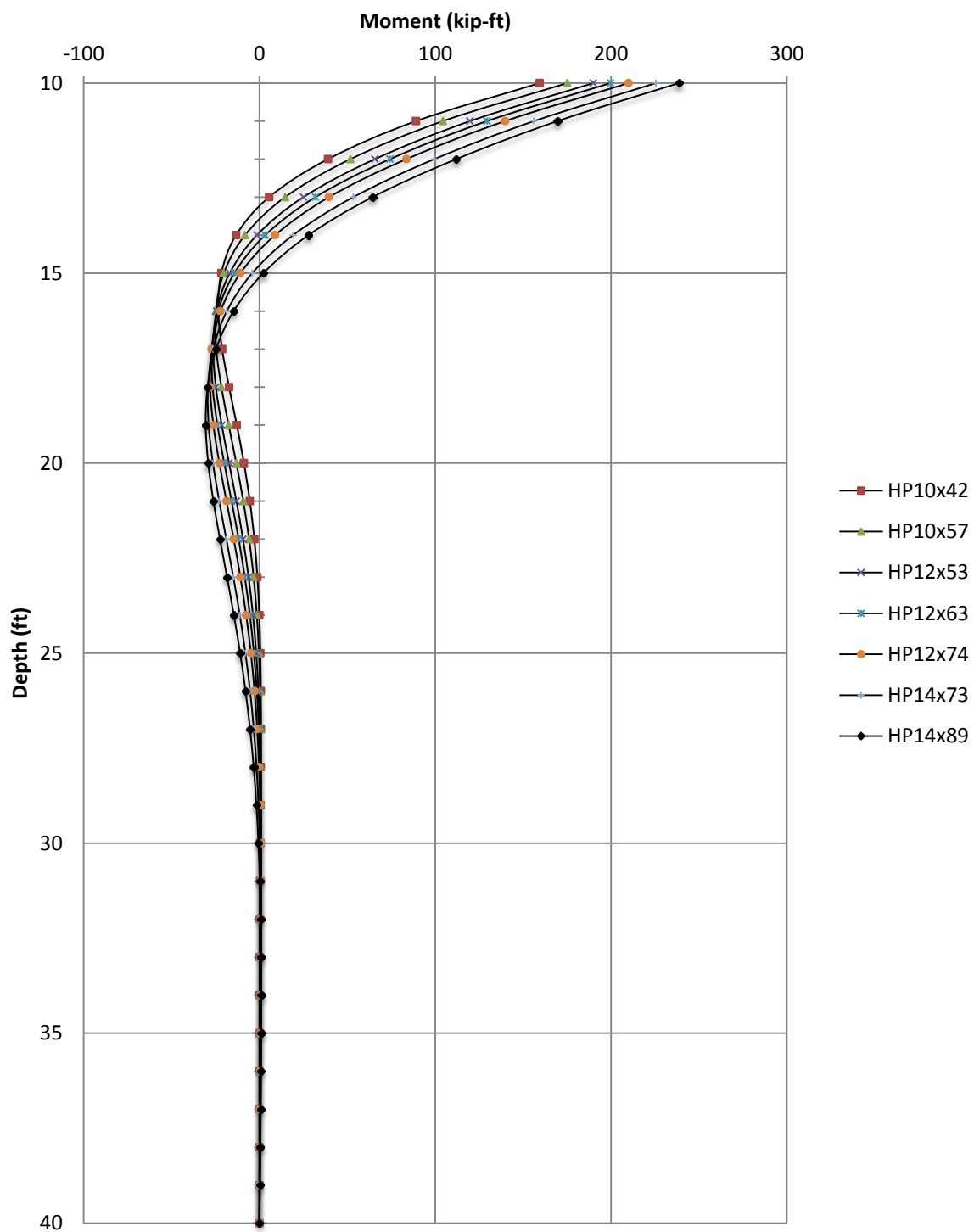


Figure B-59 : Bridge III pile moments in very stiff clay under Truck LL

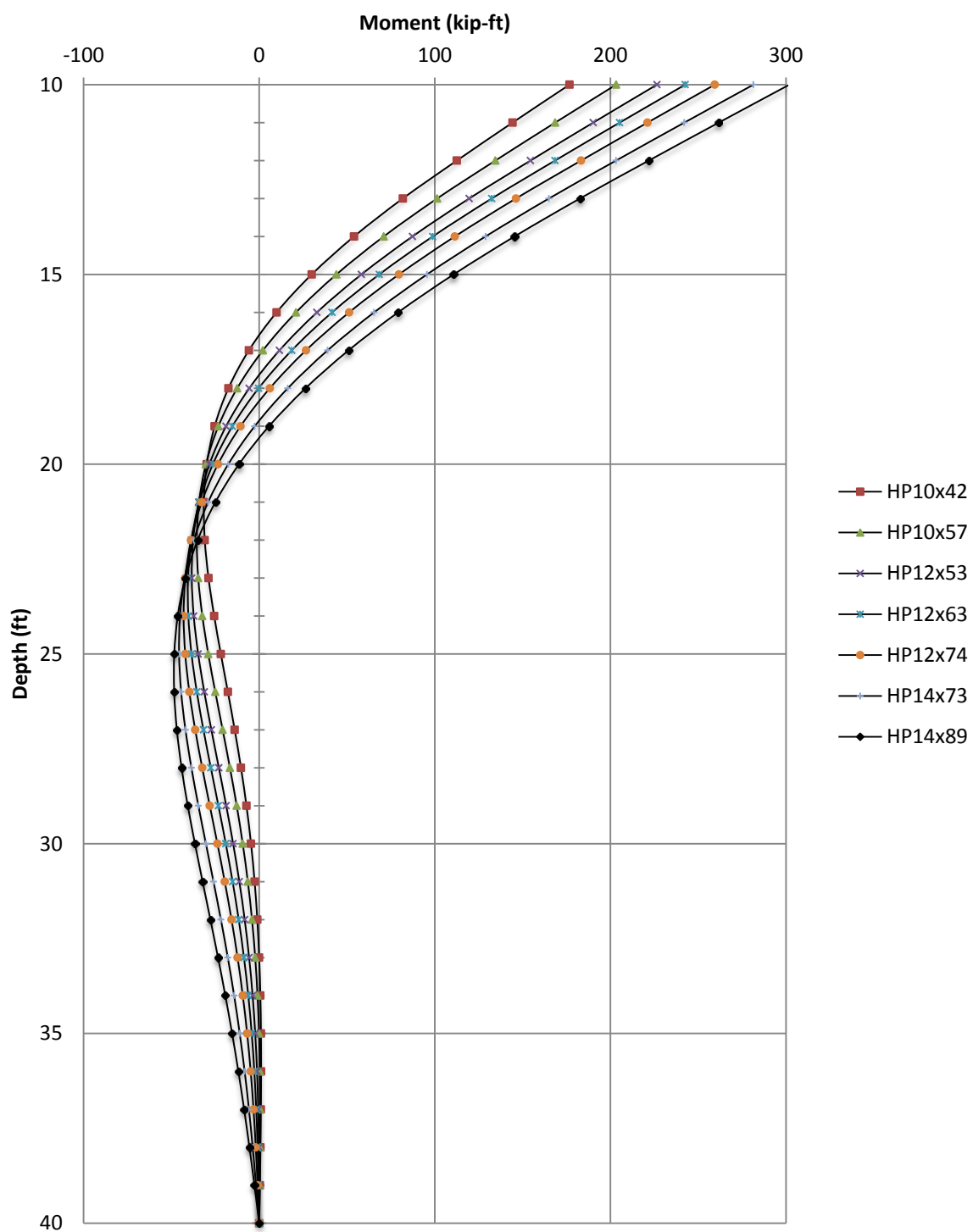


Figure B-60 : Bridge III pile moments in loose sand under Truck LL

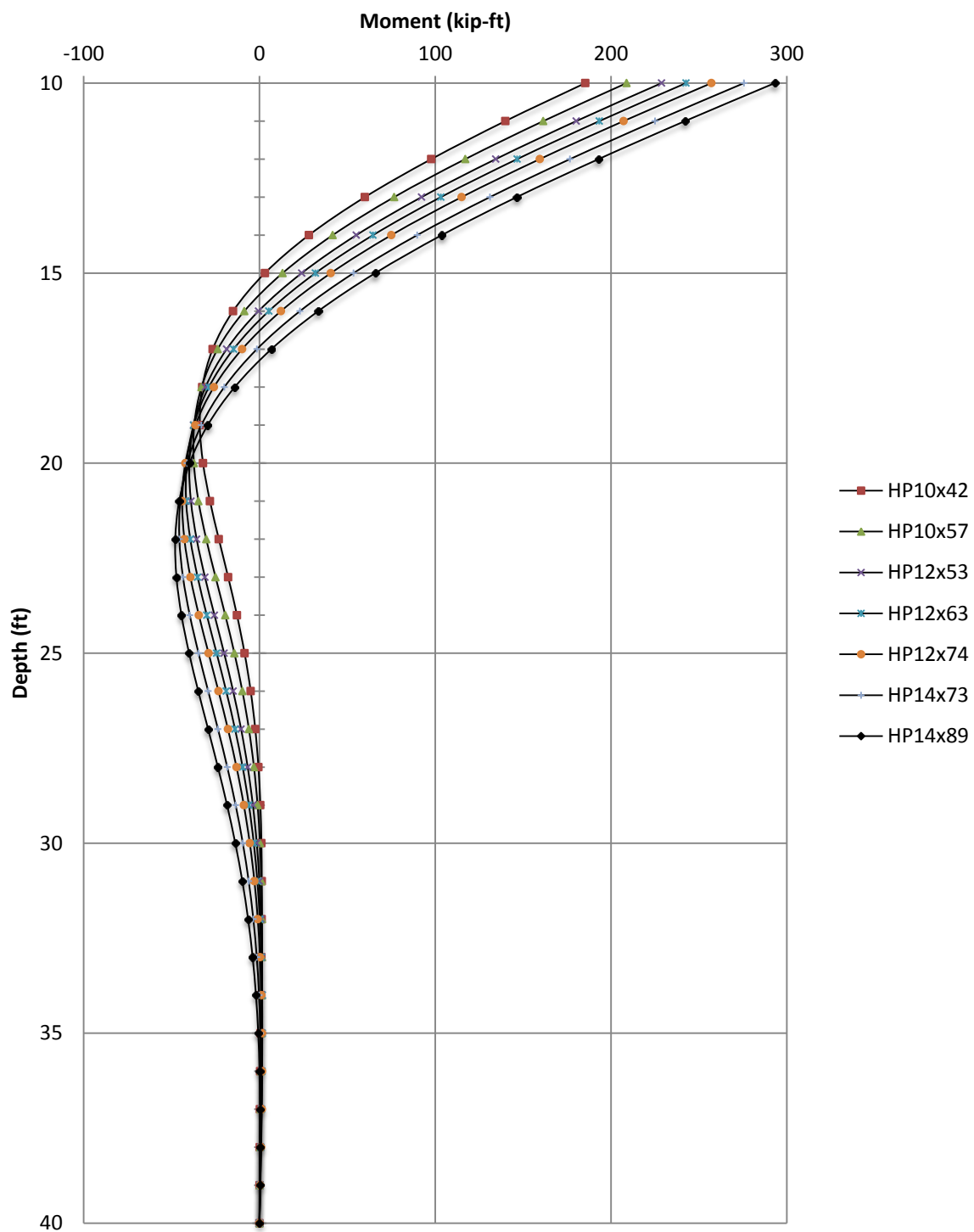


Figure B-61 : Bridge III pile moments in medium sand under Truck LL

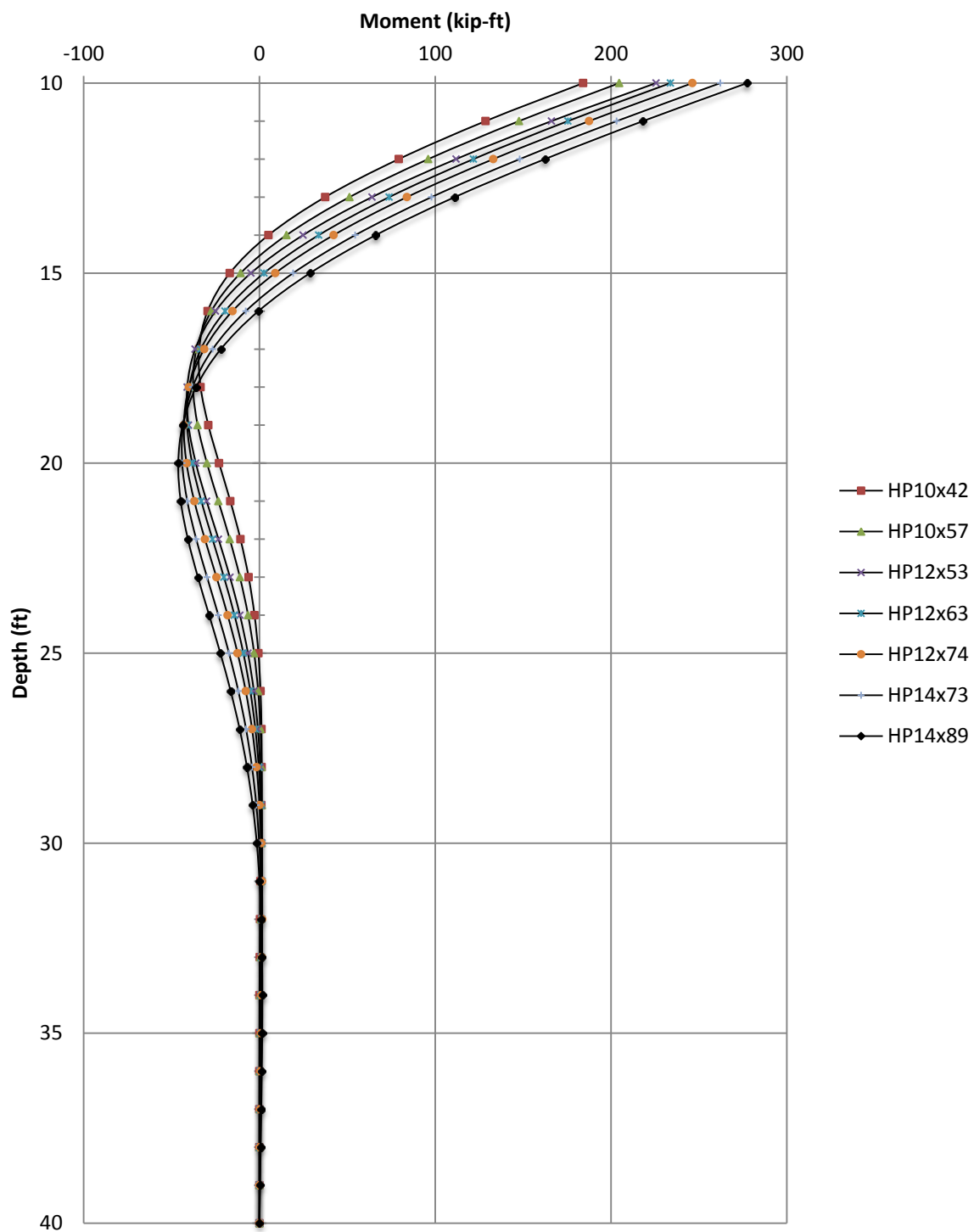
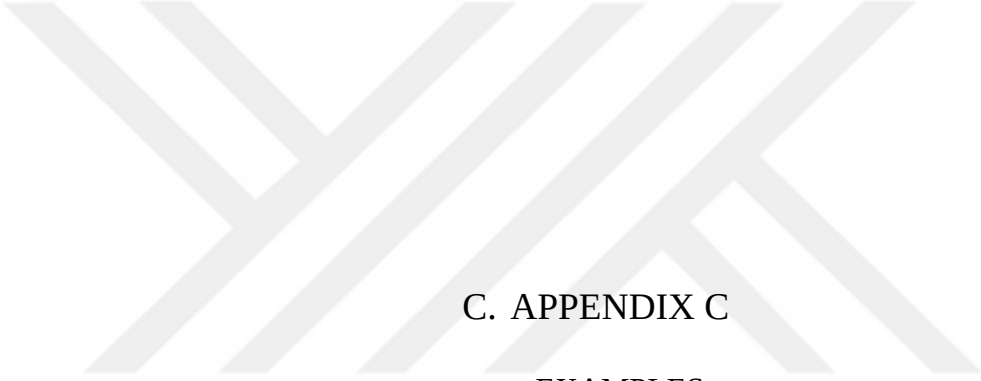


Figure B-62 : Bridge III pile moments in dense sand under Truck LL



C. APPENDIX C

EXAMPLES

Appendix C : Example 1, Use of springs in SAP2000

To illustrate the use of pile springs in SAP2000, a 2-d frame model was created in the program as shown in Figure C-1. To use in this example, very stiff clay soil foundation and HP14x89 pile section was assumed in terms of the properties of the springs that will be defined in the program.

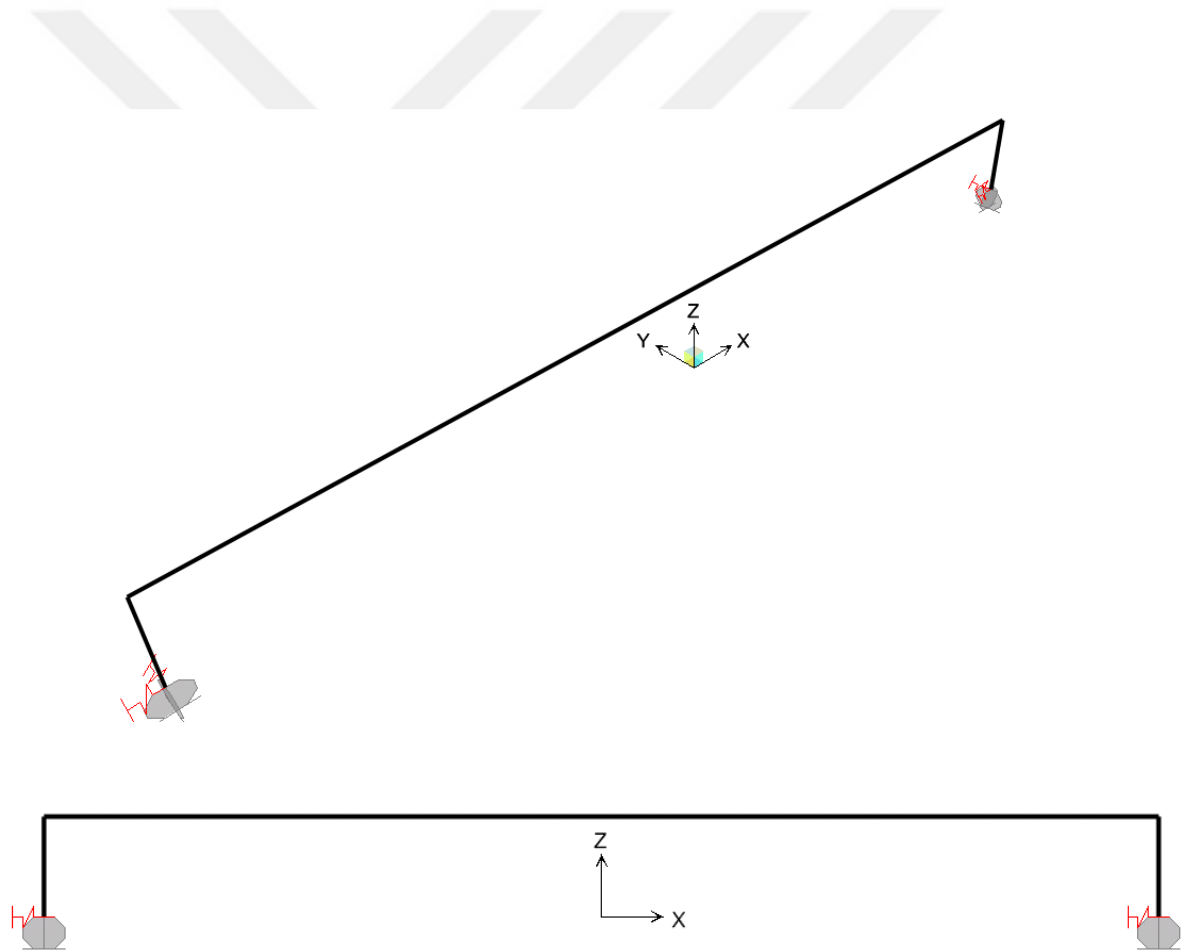


Figure C-1: 2-d frame model with the rotational and translational springs attached at abutments.

Based on the data provided in the report in Table 4-15, the stiffnesses are 564032 kip/ft and 909091 kip-in/rad for translational and rotational springs, respectively. Having known the stiffnesses, the springs were then assigned to the abutment bottom joints in the 2-d frame model in the program. Translational springs were assigned to joints in the direction of X axis, which is symbolized as “Translational 1” in the program. Rotational springs were assigned to the joints in the rotation about Y axis which is symbolized as “2” in the program. The same load cases that were used to derive the spring stiffnesses in the report were considered in order to make a superstructure positive moment comparison and to check the accuracy of the data that were provided in the report.

Based on Table 4-7 the superstructure positive moment is given as 732 kip-ft relative to very stiff clay and HP14x89 pile section. In this example, with the use of springs in place of piles, the maximum positive moment appears to be 780 ft-kip. The deformed shape and moment diagram are shown in Figure C-2 . As it is noticed that higher than actual superstructure positive moment was obtained. This is because of the definition of the same springs to both abutments in either ends of the span, which is not same in the actual model. The stiffness values provided in the report and used in this example belongs to the single pile which undergoes the worst deformation. On the other hand the other pile has less deformation and therefore should have higher stiffness.

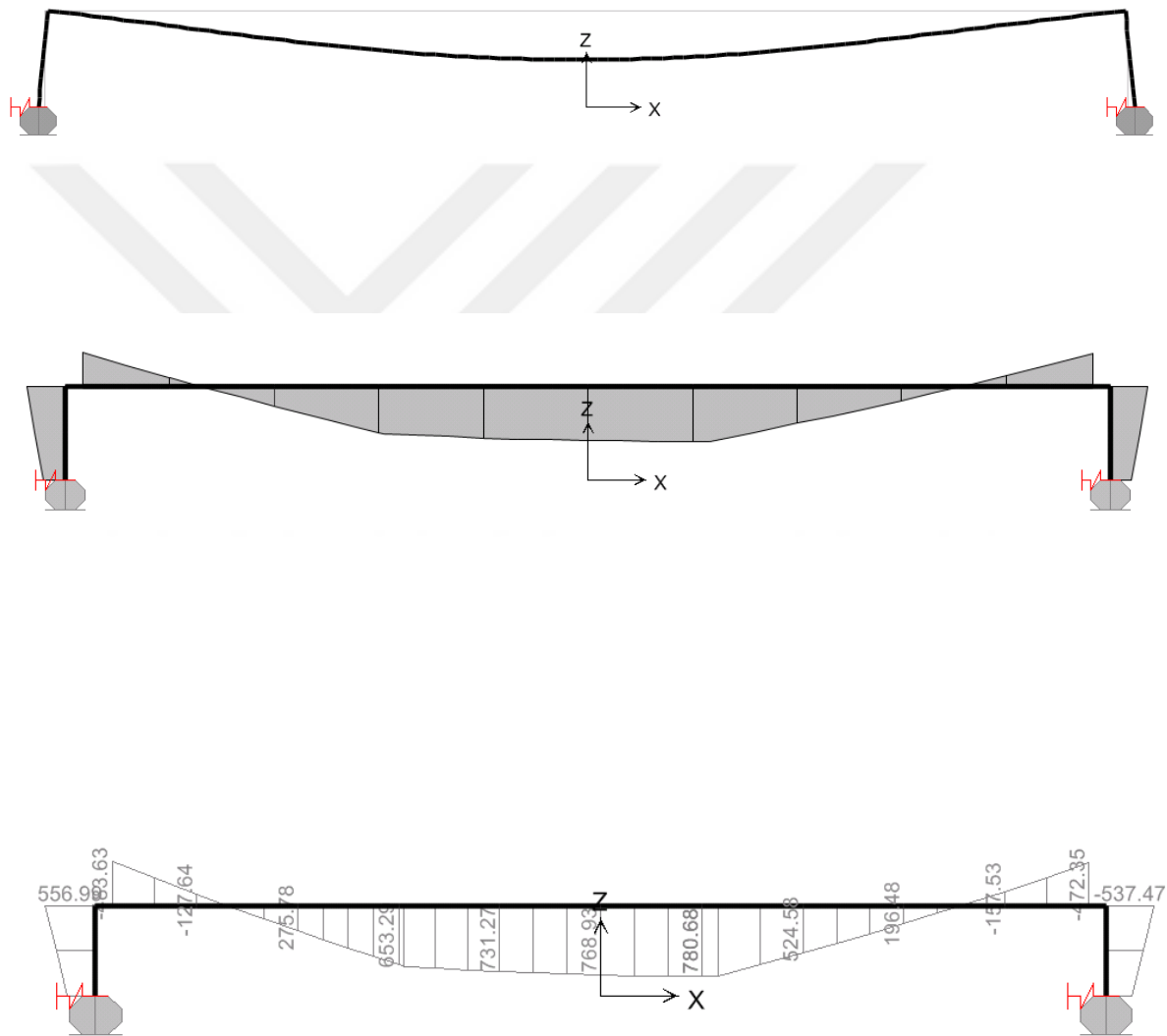


Figure C-2: Deformed shape and moment diagram of the 2-d frame model with springs attached at abutment bottoms.

Appendix C : Example2; Equivalent column stiffness

In this example, a method that enables the use of springs as an equivalent column stiffness is shown. To use in this example, 1in long columns were chosen to represent piles. These columns will be assigned an equivalent stiffness which will be derived from the spring stiffness data provided in the report. As in the first example, very stiff clay foundation soil and HP14x89 pile section was assumed in terms of spring properties.

A 2-d frame model was created in the program as shown in Figure C-3 . In the model, 1in long beam elements are attached rigidly to abutment bottom joints.

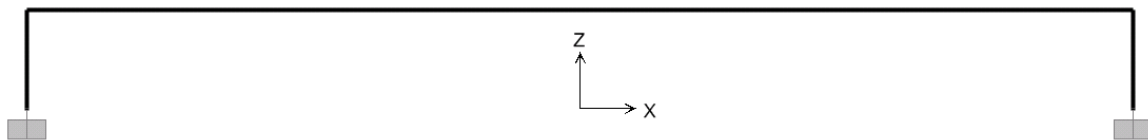


Figure C-3: 2-d frame model with 1in long columns are attached to abutment bottom joints.

Initially, individual deformation of a 1in long column under a lateral unit force and moment were obtained assuming 1 unit column stiffness. Using the equations below, the 1unit colum stiffness will be modified based on the stiffnesses of rotational and translational springs.

Column head rotation and deflection under 1 kip lateral load :



$$P := 1 \text{ kip}$$

$$L := 1 \text{ in}$$

$$E := 1 \text{ ksi}$$

$$I := 1 \text{ in}^4$$

$$\Delta_v := \frac{P \cdot L^3}{3E \cdot I}$$

$$\Delta_v = 0.333 \cdot \text{in}$$

$$\theta_v := \frac{P \cdot L^2}{2 \cdot E \cdot I}$$

$$\theta_v = 0.5 \cdot \text{rad}$$

Column head rotation and deflection under 1 kip-in moment load :



$$M := 1 \text{ kip-in}$$

$$L := 1 \text{ in}$$

$$E := 1 \text{ ksi}$$

$$I := 1 \text{ in}^4$$

$$\Delta_m := \frac{M \cdot L^2}{2 \cdot E \cdot I}$$

$$\Delta_m = 0.5 \cdot \text{in}$$

$$\theta_m := \frac{M \cdot L}{E \cdot I}$$

$$\theta_m = 1 \cdot \text{rad}$$

Pile head rotation and deflection from the SAP2000 models will be matched to the summed rotation and deflections of the 1in long column that analyzed under unit loads. The solution is as follows;

$$\theta_t := \theta_m + \theta_v$$

$$\Delta_t := \Delta_m + \Delta_v$$

$$\frac{M}{K_\theta} := \left(\frac{\theta_v}{E \cdot I} \cdot \frac{\text{rad}}{\text{kip}} \right) \cdot V + \left(\frac{\theta_m}{EI} \cdot \frac{\text{rad}}{\text{kip} \cdot \text{in}} \right) \cdot M$$

$$\frac{V}{K_\Delta} := \left(\frac{\Delta_v}{E \cdot I} \cdot \frac{\text{in}}{\text{kip}} \right) \cdot V + \left(\frac{\Delta_m}{E \cdot I} \cdot \frac{\text{in}}{\text{in} \cdot \text{kip}} \right) \cdot M$$

$$\frac{1}{K_\theta} := \left(\frac{0.5}{E \cdot I} \cdot \frac{\text{rad}}{\text{kip}} \right) \cdot \frac{V}{M} + \frac{1}{EI} \cdot \frac{\text{rad}}{\text{kip} \cdot \text{in}}$$

$$\frac{1}{K_\Delta} := \left(\frac{0.333}{E \cdot I} \cdot \frac{\text{in}}{\text{kip}} \right) + \left(\frac{0.5}{E \cdot I} \cdot \frac{\text{in}}{\text{in} \cdot \text{kip}} \right) \cdot \frac{M}{V}$$

$$\frac{M}{V} := \left(\frac{1}{K_\Delta} - \frac{0.333}{E \cdot I} \cdot \frac{\text{in}}{\text{kip}} \right) \cdot \left(\frac{E \cdot I}{0.5} \cdot \frac{\text{in} \cdot \text{kip}}{\text{in}} \right)$$

Stiffnesses of the translational and rotational springs are:

$$K_{\Delta} = 64032 \frac{\text{kip}}{\text{ft}}$$

$$K_{\theta} = 909091 \frac{\text{kip} \cdot \text{ft}}{\text{rad}}$$

$$\left(\frac{1}{K_{\Delta}} - \frac{0.333}{E \cdot I} \cdot \frac{\text{in}}{\text{kip}} \right) \cdot \left(\frac{1}{K_{\theta}} - \frac{1 \text{ rad}}{E \cdot I} \right) \cdot \left(\frac{E \cdot I}{0.5} \cdot \frac{\text{in} \cdot \text{kip}}{\text{in}} \right) \cdot \left(\frac{E \cdot I}{0.5} \cdot \frac{\text{in} \cdot \text{kip}}{\text{rad}} \right) := 1$$

$$\left(\frac{1}{5336 \frac{\text{kip}}{\text{in}}} - \frac{0.333}{E \cdot I} \cdot \frac{\text{in}}{\text{kip}} \right) \cdot \left(\frac{1}{10909092 \frac{\text{kip} \cdot \text{in}}{\text{rad}}} - \frac{1}{E \cdot I} \cdot \frac{\text{rad}}{\text{kip} \cdot \text{in}} \right) \cdot \left(\frac{E \cdot I}{0.5} \cdot \frac{\text{in} \cdot \text{kip}}{\text{in}} \right) \cdot \left(\frac{E \cdot I}{0.5} \cdot \frac{\text{in} \cdot \text{kip}}{\text{rad}} \right) := 1$$

$$\left(0.0001874 - \frac{0.33}{E \cdot I} \right) \cdot \left(0.000000091 - \frac{1}{E \cdot I} \right) := \frac{0.25}{(EI)^2}$$

$$(0.0001874 \cdot E \cdot I - 0.33) \cdot (0.000000091 \cdot E \cdot I - 1) := \frac{0.25}{EI}$$

$$1.7 \cdot 10^{-11} \cdot (E \cdot I)^3 - 0.000187 \cdot (EI)^2 + 0.3333EI - 0.25 := 0$$

$$EI_1 := 0.75$$

$$EI_2 := 10998217$$

$$EI_3 := 1782$$

As shown above, a cubic equation was obtained in the derivation of the new stiffness coefficient. The solution of the cubic equation results in three roots; the root EI_3 will be used to modify the initial unit stiffness of the 1in long beam elements, or columns, in the frame model in the program.

After the modification of the column stiffness by an equivalent value, the 2-d model was analyzed under the same load cases as in the report. Based on the results, the maximum superstructure positive moment is obtained as 782 kip-ft as can be seen in Figure C-4 . Considering the result that was obtained in the first example, it can be seen that similar result can be achieved with the use of equivalent column method.

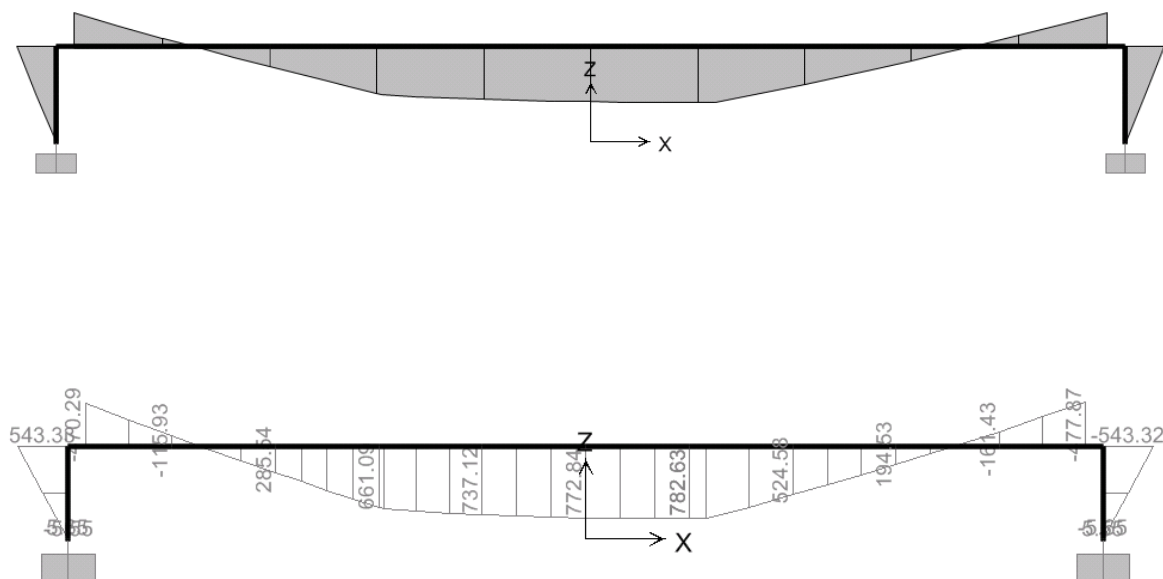


Figure C-4: Moment diagram of the frame model with 1in equivalent columns.