CAPITAL UNIVERSITY OF SCIENCE AND TECHNOLOGY, ISLAMABAD

Assessment of Suitable Modelling and Design Techniques of Raft Foundation Using Soil Structure Interaction

by

Tariq Mahmood

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This effort is devoted to my respected and cherishing parents, who helped me through each troublesome of my life and yielded every one of the comforts of their lives for my brilliant future. This is likewise a tribute to my best teachers who guided me to go up against the troubles of presence with ingenuity and boldness, and who made me what I am today.

CERTIFICATE OF APPROVAL

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They said,"Exalted are You; we have no knowledge except what You have taught us. Indeed, it is You who is the Knowing, the Wise."

I offer thanks to the Almighty Allah (swt) Who guided me and Who is the source of every magnificent bounty and knowledge I have. Allah (swt) is indeed the Beneficent, the Merciful, and to Him belongs all praise.

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Abstract

Concrete mat foundations design has advanced substantially in the last twenty five years. These advancements are happening in field of finite element analysis of reinforced concrete design and soil structure interaction (SSI) together with exponential growth of computing power. Current design practice and philosophy does not take advantage of such development and rely on assumption which although simplifies the design workflow and productivity but renders an uneconomical and in some cases erroneous design of raft foundation. Raft being the most expensive element of structural system adversely affects the economic feasibility of the project. This study attempts to assess current design practice of raft foundation and compares the results with analysis performed considering soil structure interaction.

Current modeling approach for multistory RCC frame building is based on two stage analysis. In stage-1 building is assumed to be fixed at the ground level and no SSI effects are considered. After analysis reactions are calculated with fixed end condition at the grade. These results are transferred to a different model for foundation design. This approach is used mostly by the structural engineers. Ignoring the SSI effects in stage-1 will produce an upper bound solution, which may be used for preliminary design of raft foundation but needs further analysis to reach final design.

To compare the results of modeling approach of current practice which ignores SSI effects another model is developed which incorporates the SSI effects. This model is based on guide lines provided in a report published by National Institute of Standards and Technology titled as NIST GCR 12-917-21 "Soil-Structure Interaction of Building Structures". The report helps practising engineer to eliminate the problems associated with practical application of SSI principles. This report enhances the understanding of SSI principles and synthesis body of knowledge for practising engineers making it possible for practising engineers to analyse structures accordingly for better understanding of response and required performance of structure.

A regular mid-rise concrete frame building of G+7 stories founded over fine grained soil of varying consistencies from very soft to very stiff in moderate to high seismic zone is modeled using above mentioned two approaches. Six models of each modeling approach are analyzed over six different soil consistencies. Different soil were chosen from UBC-97 table 16-K. In both approaches soil media is modeled primarily as Winkler soil model. In first approach, a uniform modulus of subgrade reaction is used to model the soil behavior ,while in the 2^{nd} approach, a pseudo coupling method is used to capture the dishing effect of settlement profile which is closely related to field observations.

After finalizing the model, parametric study on raft foundation analysis of above mentioned building is conducted and bending moments, settlement profiles and pressure distribution are compared with different approaches. Approach-II analysis produced a concave upward profile for all soil types which was also anticipated by elastic settlement analysis, while Approach-I profile produced a concave down profile. The corresponding bending moment diagram changed the placement configuration and quantity of reinforcement in the final design. The interaction of column with raft is also captured as Approach-II combines the structure, foundation and soil in one model. The effect of flexible raft on column forces was also calculated which cannot be calculated using Approach-I. Shear force in raft foundation is found to be not sensitive to the two approaches used for analysis. The study also indicated that a close collaboration is required between structural and geotechnical engineer during design process to achieve required strength and economy of structures.

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Abbreviations

Ab	Area of Foundation
B andd L	Half Width and Length of Foundation
DoF	Degree of Freedom
ESA	Equivalent Static Analysis
E-SDoF	Equivalent Single Degree of Freedom
FEM	Finite Element Method
$\mathbf{I}_{bx}$	Moment of Inertia About X-Axis
$\mathbf{I}_{by}$	Moment of Inertia about Y-Axis
$\boldsymbol{k}$	Modulus of Subgrade Reaction
MDoF	Multi Degree of Freedom
P	Pressure of Structure
R	Response Modification Factor
RC	Reinforced Concrete
SDoF	Single Degree of Freedom
SSI	Soil Structure Interaction
SB	Rock Soil
SD	Stiff Soil
$\mathbf{T}_{FIX}$	First Mode Period of Fixed Base
$\mathbf{T}_{STR}$	Period of Structure in Fixed Base
$\mathbf{V}_s$	Shear Wave Velocity
V	Poison's Ratio
$\boldsymbol{w}$	Deflection
Δ	Target Inter Story Displacement Ductility Ratio

Chapter 1

Introduction

1.1 Background

The rate of urbanization in Pakistan is highest in south Asia together with its high population growth. This high rate of urbanization causes housing crisis in urban areas where precious urban land is short in supply. Truly this is a great challenge but comes with great opportunities. One of the solutions of this problem is to develop midrise apartment buildings.

Major cities of Pakistan lie on plains along five rivers. These landforms generally consist fine grain soil of varying consistencies with varying layer thickness. Therefore, foundation of midrise building will mostly be on these types of soil. Also, some of these cities are situated close to mountain ranges and fault lines which also poses yet another hazard of high seismic activity. Therefore, it urges a need for a greater understanding of performance of midrise building's mat foundation design practice on such type of soil. This urgent need motivates design engineers to revisit current design practice in the light of latest knowledge of soil structure interaction (SSI) in order to achieve required strength and greater economy.

In current analysis and design practice, the building is assumed to be fixed at the ground level, and no SSI effects are considered. Using commercially available software based on finite element method reactions are calculated at the base of the model at grade. These reactions are imported without superstructure, into another model where mat slab is modeled over linear springs according to Winkler subgrade model. The model is then analyzed using finite element method. After analysis, reinforced cement concrete design of mat is followed, which is based on ACI 318.

The procedures of ACI 318 for the design of mats are based primarily on results of tests reported by ACI-ASCE Committee 326 (1962,1974). ACI-ASCE Committee 326 published a report in 1966 and then 1988 which was again reappraised in 2002. This report, which is titled as Suggested Analysis and Design Procedure for Combined Footings and Mats" ACI 336.2R-88 (Reappraised 2002). Although, research in the field of SSI had advanced at that time but still it was difficult to implement in regular design practice. Special and very important structure such as nuclear power plants were used to designed according to SSI principles. But to encourage the designer to incorporate SSI principles in regular design practice, ACI 336.2R-88 gave guidelines regarding important factors that must be considered during design of mat foundation.

It was well acknowledged that a building structure is an interlinked system of three components, the super structure, the foundation element and subgrade soil [4]. These components interact with each other collectively to generate response against imposed loading conditions. Attempts have made to develop a framework for practising engineers to practice SSI in their routine design practice. NIST report [1], published in 2012, titled "Soil Structure Interaction for Building Structures" NIST GCR 12-917-21 synthesis the knowledge of SSI, and provide guidelines to practising engineer for implementing SSI principles. Although, its emphasis is on response of superstructure against seismic loading but comprehensively considering the interlinked behavior of structural components. These guidelines can be used to design the raft foundation which will make it possible to incorporate the important factors in one model in order to determine the structural response.

1.2 Problem Statement

The methods described in the ACI 336.2R-1998 reappraised in 2002 tries to address raft foundation design problems considering available resources of hardware and software capability of computers. Therefore, it oversimplifies the design procedure which leaves no room for structural engineer but to design raft foundation conservatively. This makes the cost of the building to rise considerably and in some cases erroneous design. The procedures described above are all based on assuming a uniform thickness mat foundation. However, in practice structural engineer often come across situation where it is often desirable to use mats of varying thicknesses. Furthermore, ACI committee reports did not addressed the performance against earthquake loads. The effect of structural stiffness of superstructure cannot be captured on contact pressure distribution and mat design as only loads are transferred at the time of mat foundation modeling. It is a usual practice to use linear Winkler soil model with a constant modulus of subgrade reaction using Bowles empirical formula which ignores dishing settlement profile under foundation as observed in field and also indicated by Boussinque Theory for stresses calculations. Therefore, considering above mentioned factors and benefitting from advancement in computer hardware and efficient and easy to use three-dimensional finite element analysis software, requires to upgrade current mat foundation design practice in light of soil foundation structure interaction knowledge.

1.3 Objective

The main objective of this study is to compare the current mat foundation design procedure with analysis model developed according to the guidelines given in NIST[1] to incorporate SSI effects.

1.4 Research Methodology

In order to achieve above mentioned objective, a regular midrise moment resisting frame (MRF) is modeled and analyzed according the current design practice without considering SSI. Another model of same building is modeled and analyzed according to the principles of SSI. Comparative study is presented on results of both approaches and conclusions are drawn.

A multistory moment resisting frame (MRF) building is analyzed with two different approaches. Following tasks are performed to achieve the objectives of this research work.

1. Approach-1

1.1 In Approach-1 a multistory MRF building (Ground +7 story) is analyzed as per current design practice considering the two soil profile types SD & SE with six varying soil consistencies from very soft to very stiff.

1.2. RCC frame model is developed in ETABS with fixed base condition and reactions are calculated. These reactions are exported to SAFE 2016, where a mat foundation is modeled over linear Winkler soil subgrade model with constant modulus of subgrade reaction. Bowles empirical equation is used for determining the modulus of subgrade reaction to determine spring constant.

1.3. Six models for mat foundation design are analyzed for six soils of varying consistencies from very soft to very stiff.

2. Approach-II

2.1 In Approach-II, a multistory MRF frame model is developed in ETABS with three interlinked system i.e. Superstructure, mat foundation and soil model as per guidelines provided in a NIST report [1] . Linear Winkler soil subgrade model is used as per proposed scheme which incorporates stiffness and damping.

2.2. Six models for mat foundation design are analyzed for six soils of varying consistencies from very soft to very stiff.

3. Results of the both approaches are interpreted in terms of mat foundation design parameters such as settlement profiles, bending moments and shear.

4. Comparisons of settlement profiles, bending moments, and shear of both design approaches are developed.

Total twelve numerical models are prepared using structural analysis commercial softwares, ETABS 17.1 and SAFE 2016, as shown in Table 1.1.

Table 1.1: Numerical models generated for this study

Sr. no.	**Soil type**	**Soil Profile Types**	
		SD(stiff soil)	**SE (Soft)**
01	Approach-I, ETABs model with fixed supports and SAFE Models with Mat Foundation	03	03
02	Approach-II, Flexible base model in ETABS	03	03

1.5 Limitations of Study

The limitations of this study are:

1. Only the numerical modeling, analysis and design are performed.
2. No kinematic effect has been considered; only inertial effects have taken into account.
3. Only dead load and lateral load effects of earthquake zone 2B have been studied in this report.

1.6 Organization of the Thesis

Chapter 1: Introduction: In this chapter, the research gap has been identified and outline of research methodology are presented.

Chapter 2: Soil Foundation Structure Interaction & Literature Review: This chapter presents detail literature review of soil foundation structure interaction analysis and mat foundation design procedure with and without SSI leading to research gap.

Chapter 3: Modeling And Analysis Methodology: In this chapter, different modeling techniques for mat foundation design are discussed. The formulation of Winkler's subgrade modal has been discussed. Calculation of soil springs through different methods are obtained and compared. Details of 08 stories MRF case study building, modeling and design methodology are presented.

Chapter 4: Analysis And Results: In this chapter, Different parameters i.e. settlement profile, bending moment and shear force has been compared.

Chapter 5: Conclusions And Recommendations: This chapter summarizes the whole research work. Conclusions of the research work have been described. Also, recommendations for future research are enlightened.

Chapter 2

Soil Foundation Structure Interaction & Literature Review

2.1 Introduction

The foundation of building takes the load from superstructure and distribute it to soil media underneath it. This makes the foundation a unique element in the whole system consisting of superstructure, foundation and soil media. This requires special design procedures to address complexities in order to figure out response of soil media and foundation element.

Due to the complex interaction between structure, foundation and soil media assumption are made to simplify the design procedure. One such important assumption is to consider the base as rigid, which effective neglects any deformation that may occur in in soil media, contrary to actual behavior of soil media. Schematic representation for such assumption can be shown in figure 2.1 (a), (b). Another assumption is to consider the foundation element to be rigid which neglects the deformation characteristics of foundation material whatsoever. When two above assumptions are made before analysis then a fixed base condition is effectively implemented which neglects any loss of energy due deformation of soil and foundation. This loss of energy is considered to be beneficial and lowers the demand force in case of seismic loading Raychowdhury [1]. SSI body of knowledge tries to develop modeling techniques that may accounts for compatible deformation at the interface of soil and foundation.

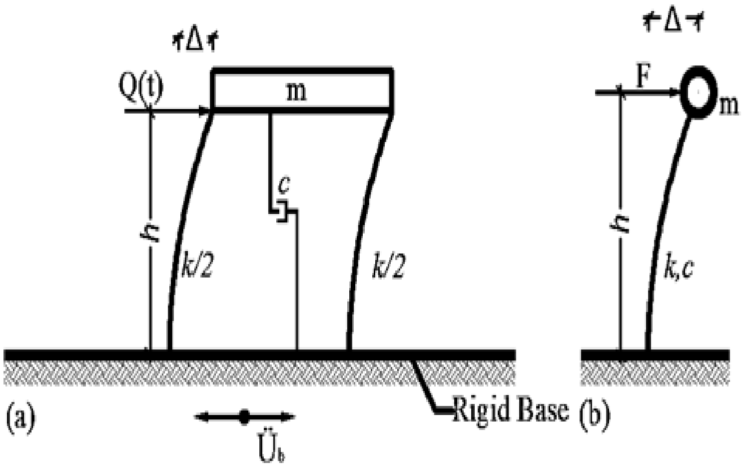


FIGURE 2.1: Structural response due to latral force F; (a) Fixed-base Frame (b) Fixed-base Stick model [2]

Therefore SSI considers six degree of freedom at the base of the structure which consist of vertical translation in z-direction, two horizontal translation in x and y direction, two rocking mode about x and y direction and on rotation about z direction, which is shown in figure 2.2

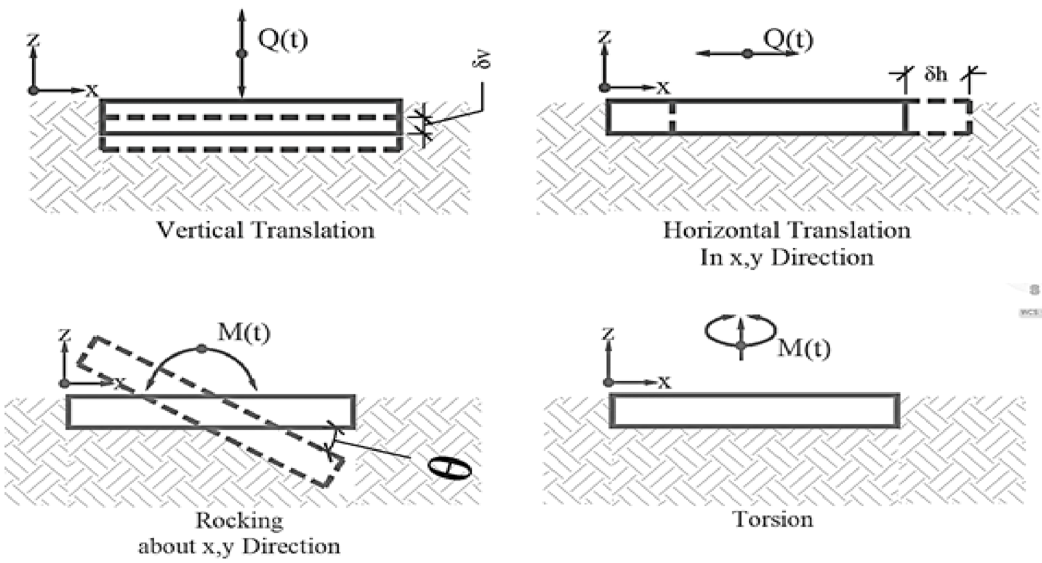


FIGURE 2.2: SSI model with stiffness and damping coefficient [2]

The six degrees of freedom as shown in figure 2.2 only include stiffness, but if one has to incorporate the damping, system gets complicated and can be visualized as shown in Figure 2.3.

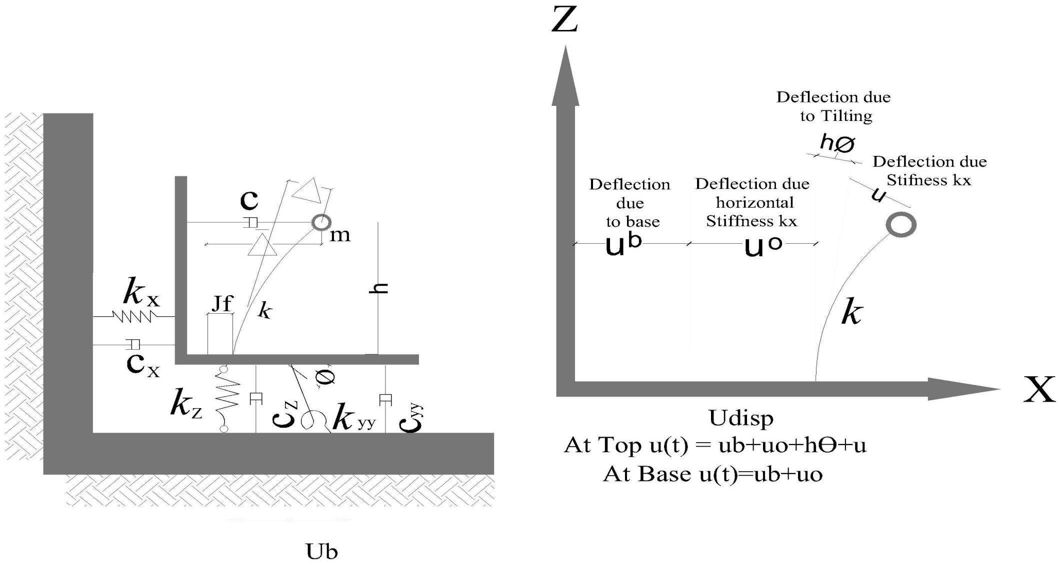


FIGURE 2.3: SSI model with stifness and damping coefficent [2]

FIGURE 2.3: SSI model with stiffness and damping coefficient [2]

Consider a very big rigid wall sitting on rigid base and another smaller rigid wall where structure is rested on and in between two wall soil is modeled using stiffness and damping parameters.

In this way displacement at the top and at base can be calculated.

To accurately describe this model impedance functions are used. Impedance function are developed on basis of frequency dependent stiffness and damping characteristics of soils-foundation interaction. Solutions were first worked out by (Luco and Westman [3]; Veletsos and Wei [4]) as shown in equation 2.1.

$$\overline{k_j} = k_j + i\omega c_j \tag{2.1}$$

Impedance functions solution are available for rigid circular or rectangular foundation placed on surface of, or embedded within, a uniform, elastic, or visco-elastic half-space. For a rigid rectangular foundation resting on a surface of half space with shear wave velocity V_s, Pais and Kausel [5] presented equations for determining the stiffness and damping term. These solutions used shear wave velocity of soils for determining the stiffness and damping characteristics. As soil is non-uniform material and presence of structural load complicates the process of selection of V_s. For a single effective V_s correction are necessary.

The solutions for impedance functions are also based on rigid foundation element assumption. To incorporate the flexibility of foundation into the model, springs representing stiffness are distributed across the foundation in such a manner that allow the foundation to deform in a natural way as per imposed load from super-structure. This effect can be incorporated in by calculating vertical impedance, as described above, normalizing it by the foundation area as compute a stiffness and damping intensity as equations 2.2 and 2.3 respectively (after Luco and Westman [3]; Veletsos and Wei [4]).

$$k'_z = k_z/4BL \tag{2.2}$$

$$c'_z = c_z/4BL \tag{2.3}$$

The distribution of spring can be shown in Figure 2.4. The value of interior spring can be taken as k'_z times tributary area dA. If the same spring is used across the entire length then the rotational stiffness would generally be underestimated. The reason for this that vertical spring does not behave in a uniform manner across the foundation and tend to produce more reaction along the edge of foundation as compared to centre of foundation. This is also true for damping coefficient c'_z, if implemented uniformly across the foundation would result in overestimated radiation damping from rocking. This because the translation vibration mode are much more effective radiation damping sources than rocking modes.

To overcome this difficulty foundation is divided into different strip along edges and stiffer springs are used along the edges or may arranged as per settlement profile provided by geotechnical engineer Harden and Hutchinson [6] worked out expression for spring stiffness and end length ratios and for as a function of L/B using static stiffness from Gazetas [7].

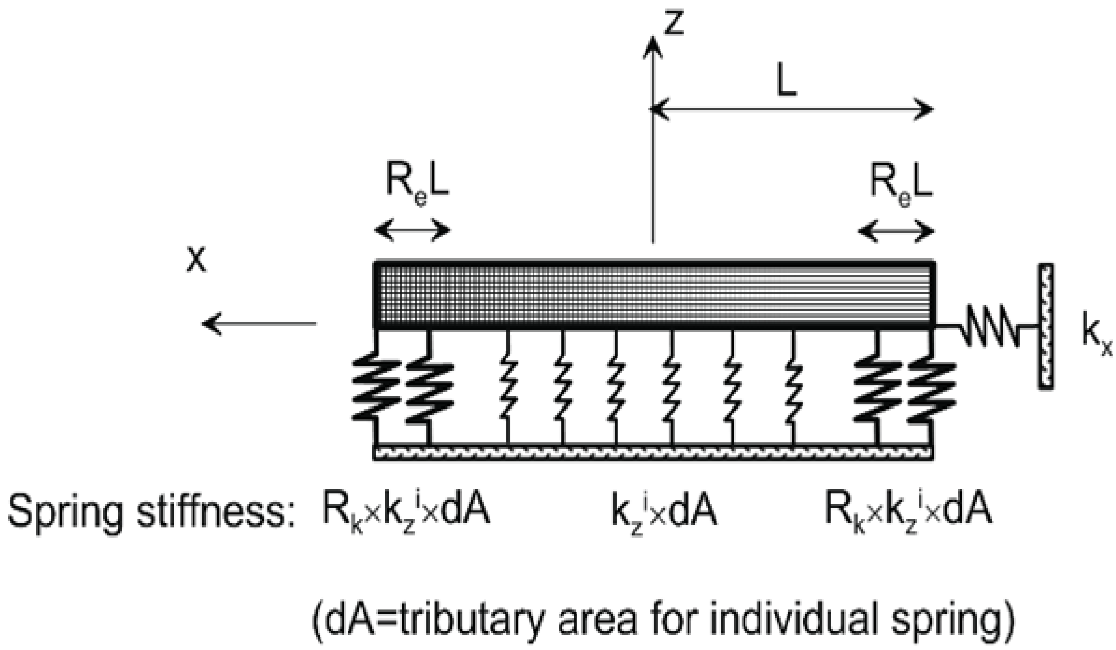


FIGURE 2.4: Vertical spring distribution for foundation[2]

The base of the structure can be termed as rigid when theoretical rigid behavior can be assumed during modeling process by making column node at grade level to be fixed in six degree of freedom, as compared to actual deformable behavior. These motions consumes energy and also serve to increase overall damping of the system and decrease in the predicted damage to the structure

2.2 Non-Linear Modeling of SSI

There are two sources of non-linearity in SSI phenomena i.e geometric non-linearity and material non-linearities in the whole system of superstructure, foundation and soil. To incorporate non-linearity in to model is a very difficult task despite current computational capabilities.

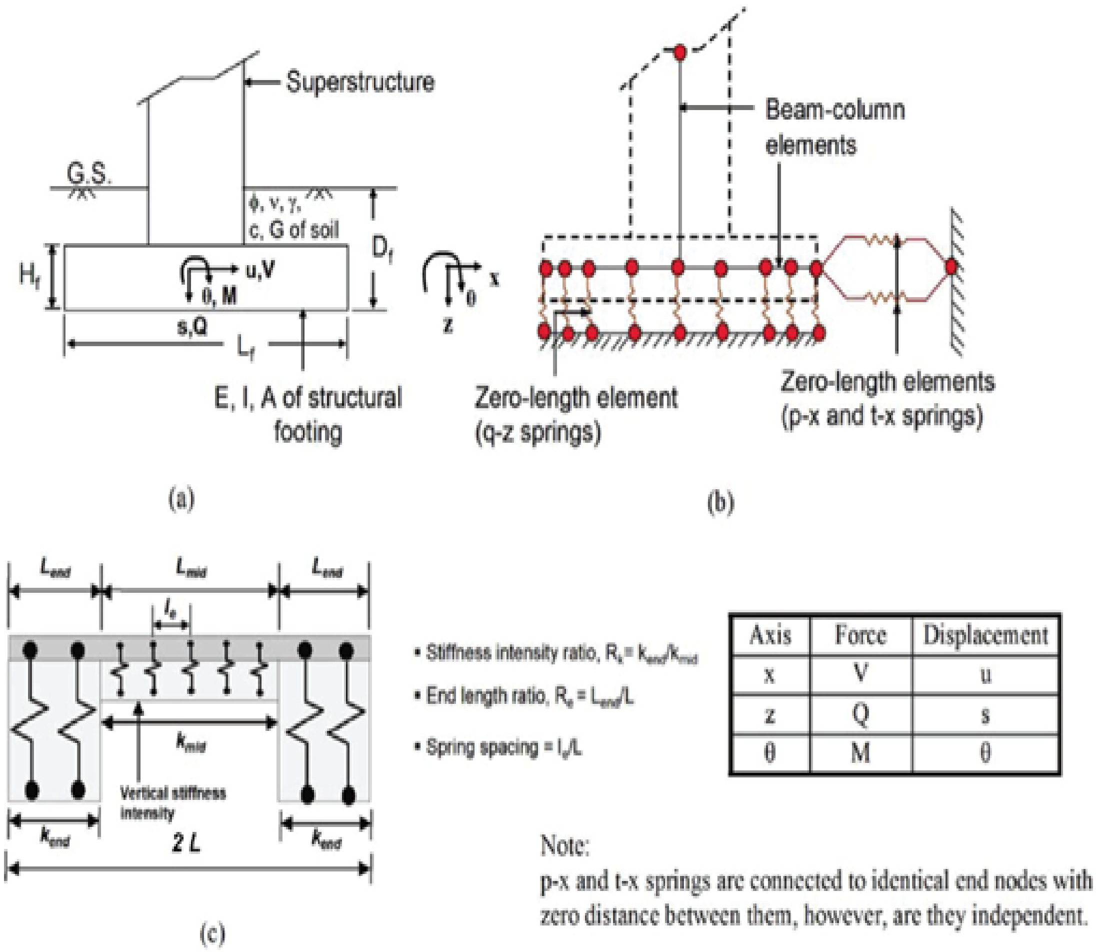
Research have categorized the research in two categories. In category-1 structure is allowed to yield with a linear soil or equivalent linear soil. In category-2 type of model a yielding or gapping soil is used with a linear structures.

Ciapoli and Pinto[8], Mylonakis and Gazetas [9], Perez-Rocha and Avlis [10] concluded that non-linearly in superstructure generally cause a reduction in ductility demand in superstructures.

Studies on non-linear foundation and soil indicates there may beneficial effects in seismic response in terms of lower seismic demand due to dissipation of energy in yielding of foundation and soil. Gazetas [11]; Gajan and Kutter [12] even proposed to revisit the foundation design procedure while allowing soil to yield with in a permissible range of settlement and tilting of structure.

There are three broader categories of modeling non-linear foundation and soil behavior (1) Plasticity based macro models (2) Continuum model (3) Beam on non-linear-Winkler foundation models.

The continuum model requires exceptional computing powers and therefore has limited application. Beam on non-linear Winkler foundation BNWF) models have been used for foundation since 1958 due to work of McClelland Focht [13]. BNWF is most simplest modeling technique for soil modeling till date. Raychowdhury and Hutchinson [14] have used non-linear springs for shallow foundation design as shown in figure 2.5.



Axis	Force	Displacement
x	V	u
z	Q	s
θ	M	θ

FIGURE 2.5: Beam on non-linear Winkler Foundation (a) Hypothesized foundation-structure system (b) Idealized model (c) Variable vertical stiffness distribution [14]

Plasticity based macro element (PBM) models are new [15] in the modeling of non-linear response of rigid foundation. It basically uses element from both continuum and BNWF formulation as shown in figure 2.6. Although PBM are quite rational in translating the soil behavior but have certain drawbacks too. It can not incorporate foundation flexibility, effects of stress-induced inhomogeneity on radian damping and failure mode other that shear failure. Also there are limited experimental validation for these type of experimental models.

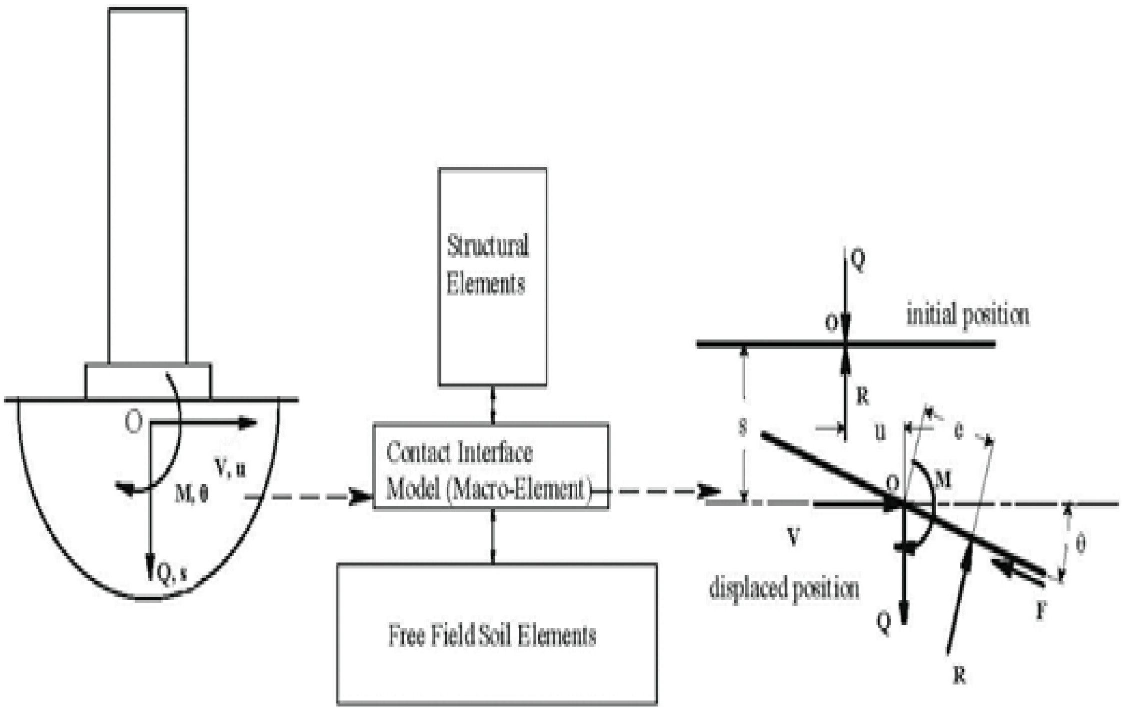


FIGURE 2.6: Plasticity Based Macro Element Model (Contact Interface Model) [12]

Chapter 3

Modelling And Analysis Methodology

3.1 Introduction

In this Chapter, Modeling approaches are discussed in details. Computers and Structures Inc. (CSI) softwares, ETABS v17.0.1 and SAFE v16.0.1 are used for modeling and analysis.

3.2 Summary of Results from Prior Studies

Naeim et al. (2008) and Tileylioglu et al. (2010) utilized typical engineering tools such as SAP2000, Integrated Software for Structural Analysis and Design (Computers and Structures, Incorporated) and as ETABS, Extended Three Dimensional Analysis of Building Systems (Computers and Structures, Inc.), to model the soil-structure interface of two buildings. Elastic springs with no compression capacity limit, and zero tension capacity are used. Analysis initially employed the full substructure modeling approach, designated the Baseline Model. Dependency of the structural elements is also tested by finding their effect by eliminating them from the model and it is observed that application of a zero-tension condition in the foundation springs and consideration of multi-support excitation along basement walls has no significant effect on results.

3.3 Description of Case Study Building

The observed model is an 08 story (G+07) frame structure as shown in figure 3.1. The building plan is symmetrical along both x-axis and y-axis with length and width of 60' and three equal bays in both directions. The sizes of beams and columns are shown in table 3.1. Beams are placed on all grids typical height of each story is taken as 12 ft. The building is mix-used commercial building with shops, departmental store and residential apartments. For the case study building, the size of the footing (raft footing) is 64' x 64' with thickness of 24" for all six soil conditions mentioned in table 3.3. The structure is analyzed with full flexure stiffness of mat and full stiffness of building superstructure.

Table 3.1: Cross Sectional Details Of Building

Structural Member	**Beam**	**Column**
Cross Section Area(in^2)	18x18	18x18
Floors Assigned	All	All

Slab thickness is taken as 6 inches. Total twelve models are prepared, six models for each approach. The building structural elements are first designed according to gravity load. For gravity load design, dead load consists of self-weight of the structure and superimposed load (finishes and partition wall loads). According to UBC-97 live load for shop floors is taken as 100psf and 50 psf for offices and 40 psf for apartment floors and roof. Two load cases are defined as mass source (including self-weight) and 25% for live load of shops. Building is situated in seismic zone 2B. Importance factor is 1 as defined in UBC-97 for standard occupied buildings. For linear static analysis UBS -97 based value of "R" factor is taken as 8.5 considering the moment resisting frame building.

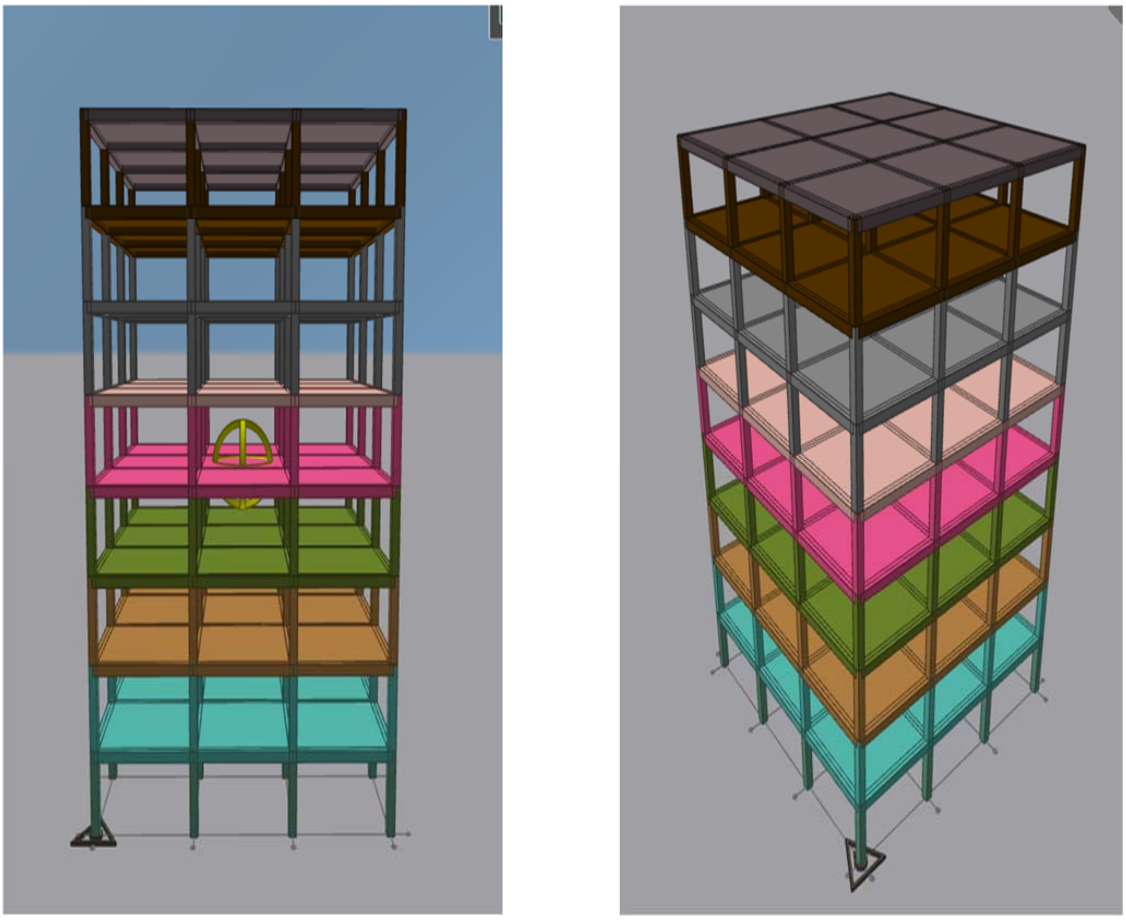


FIGURE 3.1: Elevation & 3D View of case study building

3.4 Soil Classification

The classification of soils is based upon Uniform Building Code (1997) as shown in table 3.2. Soft soil with Vs < 600 ft./sec and Stiff soil with 600 ft./sec ≤ Vs≤ 1,200 ft./sec. Soft soils are classified further into three category i.e. Soft soil 1, Soft soil 2 & Soft soil 3 depending upon the shear wave velocity. Similarly, Stiff soil are also further divided into three types i.e. Stiff soil 1, Stiff soil 2 & Stiff soil 3 depending upon their shear wave velocity Detail of soil parameters used are shown in table 3.3.

Table 3.2: Soil Classification

Soil Profile Type	Soil Profile Name	Shear Wave Velocity(ft/sec)
S_D	Stiff Soil	600-1200
S_E	Soft Soil	<600

Table 3.3: Soil Parameters considered in current study

Soil Type	G(psf)	B(ft)	L(ft)	μ	Cu(Psf)	Vs(ft/sec)
Soft 1	95036	64	64	0.4	250.75	200
Soft 2	443504	64	64	0.35	386.38	400
Soft 3	791975	64	64	0.3	522	500
Stiff 1	1600830	64	64	0.3	1000	700
Stiff 2	2726460	64	64	0.25	1500	900
Stiff 3	4312198	64	64	0.2	2000	1100

3.5 Modeling

In total, twelve (12) models are analyzed by using two different approaches of analysis and design, six model for each approach.

1. Approach I
2. Approach II

3.5.1 Approach I

Modeling Approach I is basically one used by practising engineers now a days for moment resisting frame design. This modeling approach for multistory RCC frame building is based on two stage analysis. In stage-1, building is assumed to be fixed at the ground level as shown in fig 3.1. and no SSI effects are considered. After analysis reactions are calculated with fixed end condition at the grade. In stage II, the results from stage I analysis are transferred to a different model for foundation design as shown in fig 3.2.

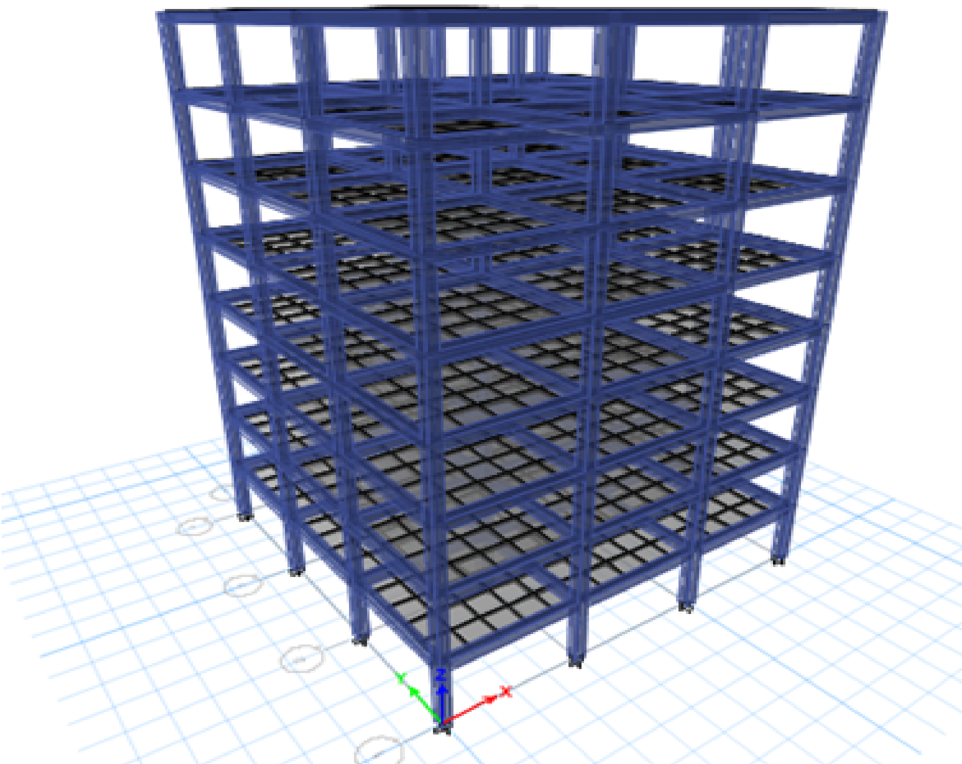


Figure 3.2: Fixed base modal at ground level

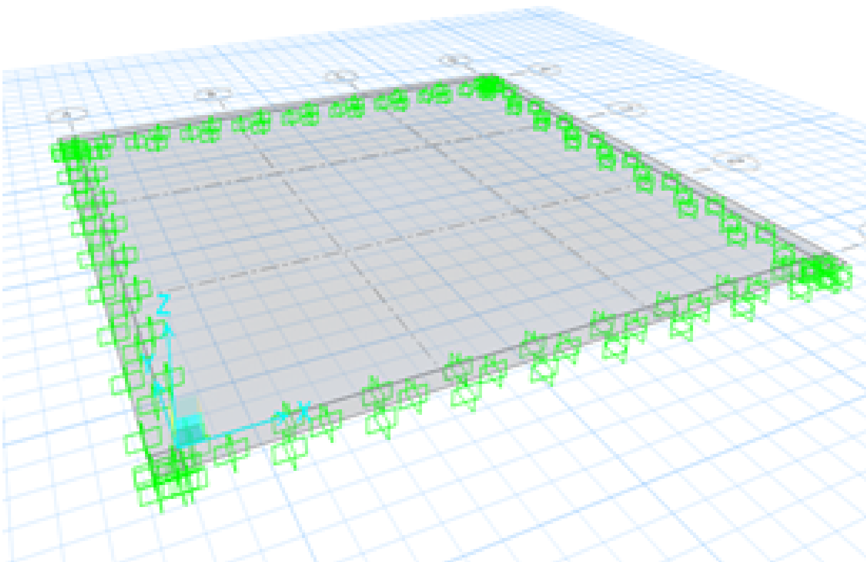


Figure 3.3: Model for foundation design

For modeling approach I, a constant Winkler's hypothesis is used in which modulus of subgrade is calculated by Bowles Empirical Formula. Joseph E. Bowles [16] has also presented an empirical relation between subgrade reaction and maximum allowable bearing pressure as shown in equation 3.1

$$SI : k_s = 40\,(SF)\,q_a, \quad Fps: \; k_s = 12\,(SF)\,q_a \tag{3.1}$$

where q_a is in ksi or KPa.

The equation 3.1 is based upon $q_a = q_{ult}/SF$ and the ultimate soil pressure is at the settlement of ΔH= 1 inch. The values calculated are shown in table 3.4. The highlighted values of Modulus of subgrade Reaction have been used for Approach-I. The different parameters settlements, moments and shear force that can impact mat foundation design were evaluated.

Table 3.4: Modulus of Subgrade Reaction For Approach-I

Soil Type	**Bowles Empirical Formula**			
	1" Settlement		**2" Settlement**	
	***SF = 3 Constant Spring (pcf)**	***SF= 2 Constant Spring (pcf)**	***SF = 3 Constant Spring (pcf)**	***SF= 2 Constant Spring (pcf)**
Soft-1	18000	12000	9000	6000
Soft-2	27826.56	18551.04	13913.28	9275.52
Soft-3	37594.08	25062.72	18797.04	12531.36
Stiff-1	72000	48000	36000	24000
Stiff-2	108000	72000	54000	36000
Stiff-3	144000	96000	72000	48000

3.5.2 Approach II

To model soil structure interaction (SSI), direct approach has been used, in which superstructure, foundation and soil are modeled as single unit. Soil Modeling is based upon concept of pseudo coupling. This model is based on guide lines provided in a report published by National Institute of Standards and Technology titled as NIST GCR 12-917-21 "Soil-Structure Interaction of Building Structures". Using the table 2.2a by Pais and Kausel (1988), modulus of subgrade reaction, has been calculated for all degrees of freedom by incorporating the effects of SSI as shown in table 3.5. A Schematic illustration of a building foundation with the soil spring is shown in Figure 3.4.Calculation of horizontal and vertical stiffness of shallow foundation for springs under the base slab have been shown in table 3.6.

By using the vertical stiffness intensity, vertical springs were spread onto the foundation as shown in Figure 3.5 whereas, the Figure 3.6 illustrates the 3D model of case study building having raft foundation with application of vertical stiffness intensity and vertical springs. The color variation in figure 3.5 ans figure 3.6 shows the intensity of vertical sicknesses as presented in table 3.6. The modification of stiffness intensities was done near the corner and edge of foot print of foundation to incorporate the sarcasm of rotational/rocking stiffness. Similarly, calculation of horizontal and vertical dashpot intensities of shallow foundation was done by using Pais and Kausel (1988) (NIST GCR 12-917-21) equations. Similarly, they are also spread like vertical springs over the footprint of foundation. The modification for dashpot intensities was also done near the edge to incorporate overestimation of rotational damping. The vertical spring and dashpot intensity distribution over the foundation for different soil types corresponding to the areas are shown in table 3.6. The different parameters settlements, moments and shear force that can impact mat foundation design were evaluated. The settlements, bending moments and shear forces are calculated on sections A-A' and B-B' as shown in figure 3.7.

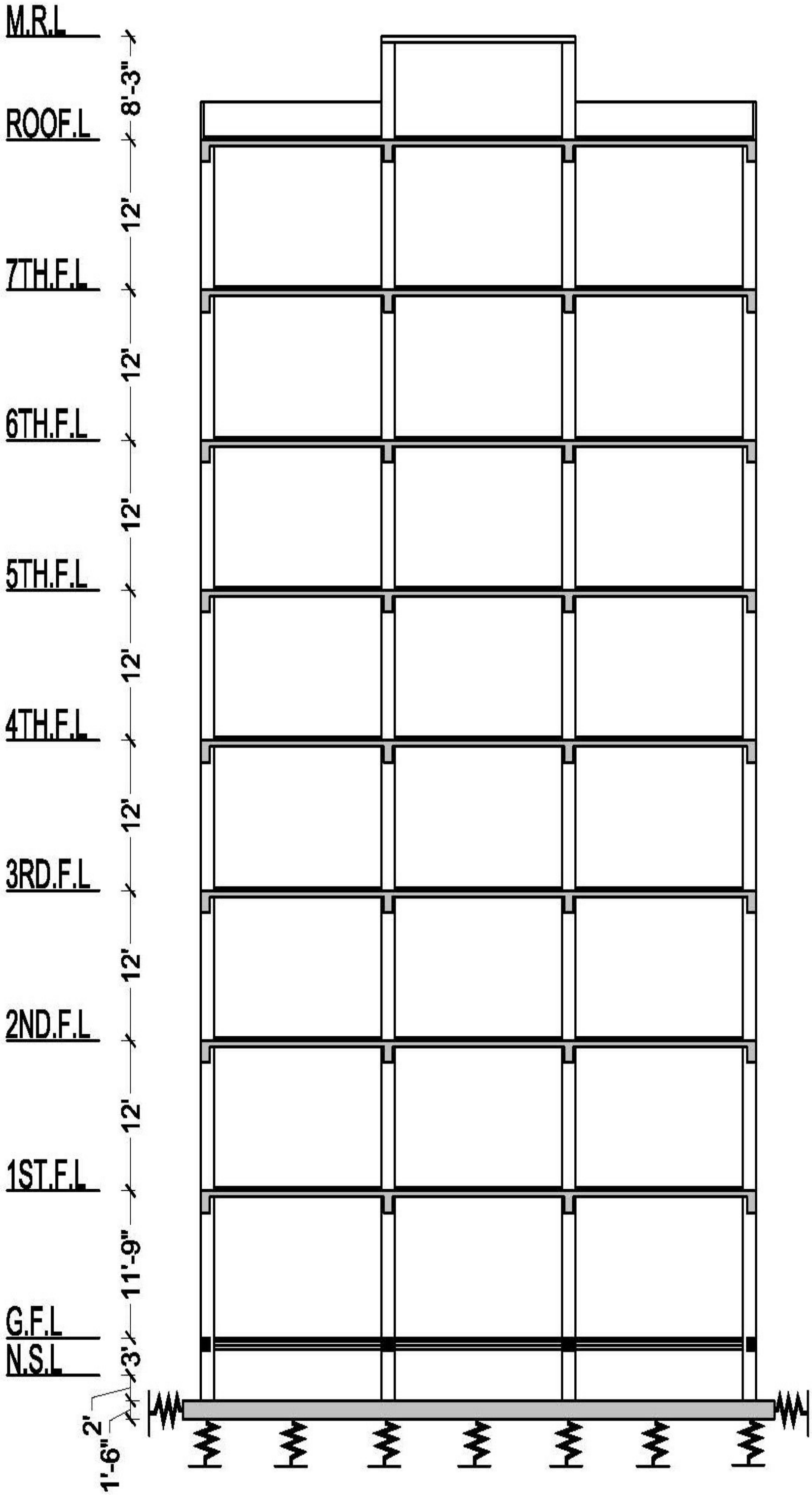


FIGURE 3.4: Schematic illustration of a building foundation with the soil spring

Table 3.5: Calculation of Stiffnessess For Approach-II

Soil Type	**G(psf)**	**Kx(lb/ft)**	**Ky(lb/ft)**	**Kz(lb/ft)**	**Kxx(lb/ft)**	**Kyy(lb/ft)**	**Kzz(lb/ft)**
Soft 1	9.5E+04	1.7E+07	1.7E+07	2.9E+07	2.0E+10	2.3E+10	2.9E+10
Soft 2	4.4E+05	1.3E+08	1.3E+08	1.3E+08	8.1E+10	1.1E+11	1.8E+11
Soft 3	7.9E+05	2.3E+08	2.3E+08	2.2E+08	1.3E+11	1.8E+11	3.3E+11
Stiff 1	1.6E+06	4.2E+08	4.2E+08	4.4E+08	2.42E+11	3.3E+11	5.6E+11
Stiff 2	2.7E+06	7.7E+08	7.7E+08	7.0E+08	4.3E+11	5.8E+11	1.2E+12
Stiff 3	4.3E+06	1.2E+09	1.2E+09	1.0E+09	6.3E+11	8.6E+11	1.8E+12

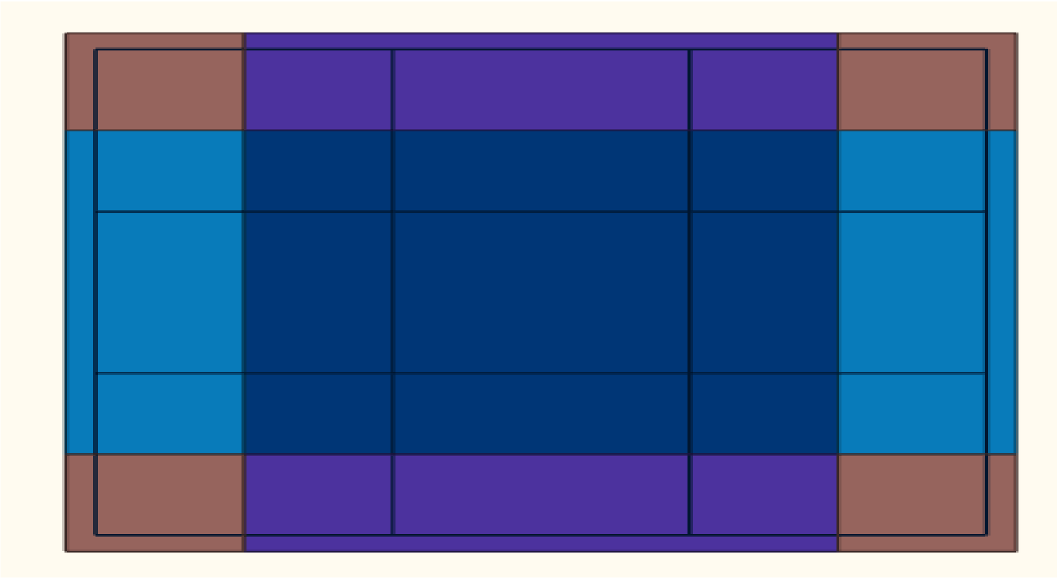


FIGURE 3.5: Vertical spring and dashpot intensity distribution over the foundation

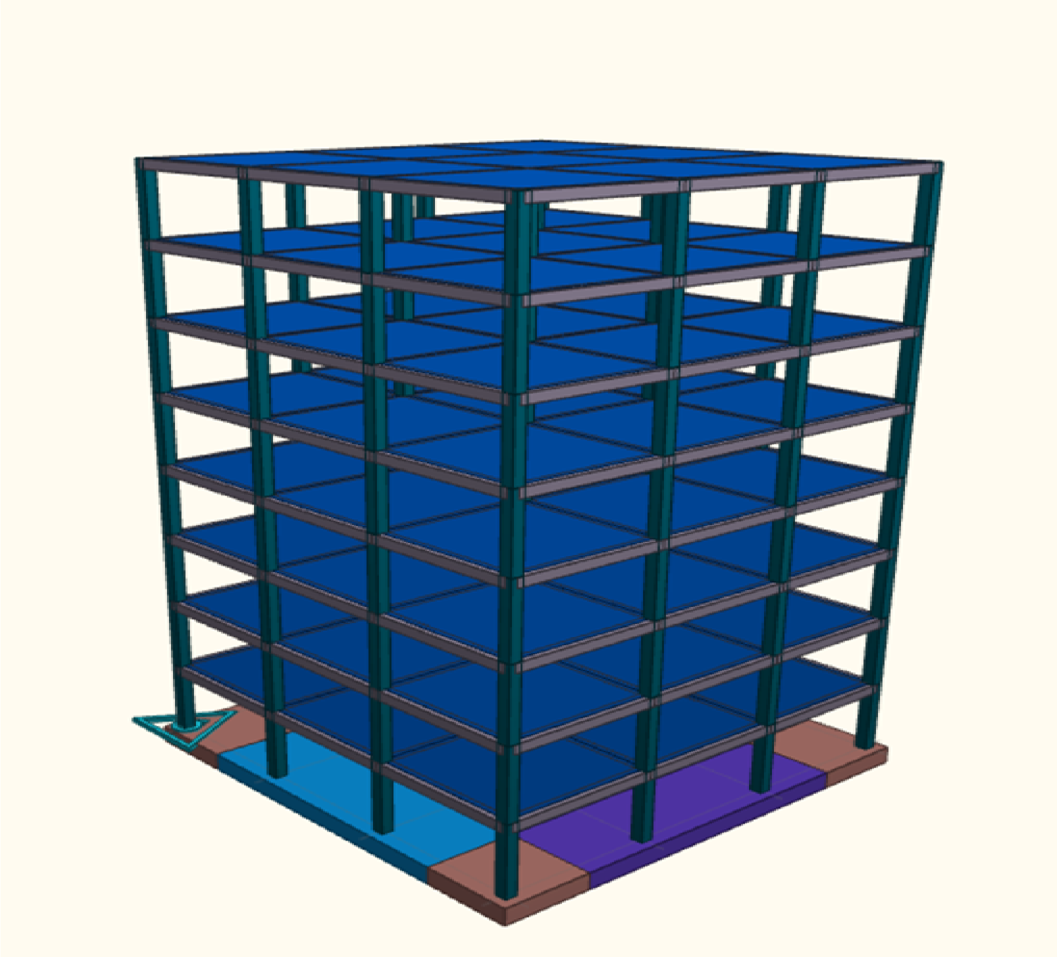


FIGURE 3.6: 3D View & Elevation of case study building with SSI

TABLE 3.6: Vertical spring and dashpot intensity distribution over the foundation for Different soil types corresponding to the areas in Fig 3.5.

Soil Type	G	Kz	Cz	Kz	Cz	Kz	Cz	Kz	Cz
	lbf/ft^2	lb/ft^3	lbf.sec/ft^3	lbf/ft^3	lb.sec/ft^3	lbf/ft^3	lbf.sec/ft^3	lbf/ft^3	lbf.sec/ft^3
Soft 1	9.50E+04	7.13E+03	1.01E+03	1.62E+04	2.30E+03	1.62E+04	2.30E+03	1.62E+04	2.30E+03
Soft 2	4.44E+05	3.19E+04	3.51E+03	6.55E+04	7.21E+03	6.55E+04	7.21E+03	6.55E+04	7.21E+03
Soft 3	7.92E+05	5.30E+04	5.25E+03	1.08E+05	1.07E+04	1.08E+05	1.07E+04	1.08E+05	1.07E+04
Stiff 1	1.60E+06	1.06E+05	8.37E+03	1.91E+05	1.51E+04	1.91E+05	1.51E+04	1.91E+05	1.51E+04
Stiff 2	2.73E+06	1.71E+05	1.28E+04	3.42E+05	2.56E+04	3.42E+05	2.56E+04	3.42E+05	2.56E+04
Stiff 3	4.31E+06	2.54E+05	1.78E+04	5.06E+05	3.55E+04	5.06E+05	3.55E+04	5.06E+05	3.55E+04

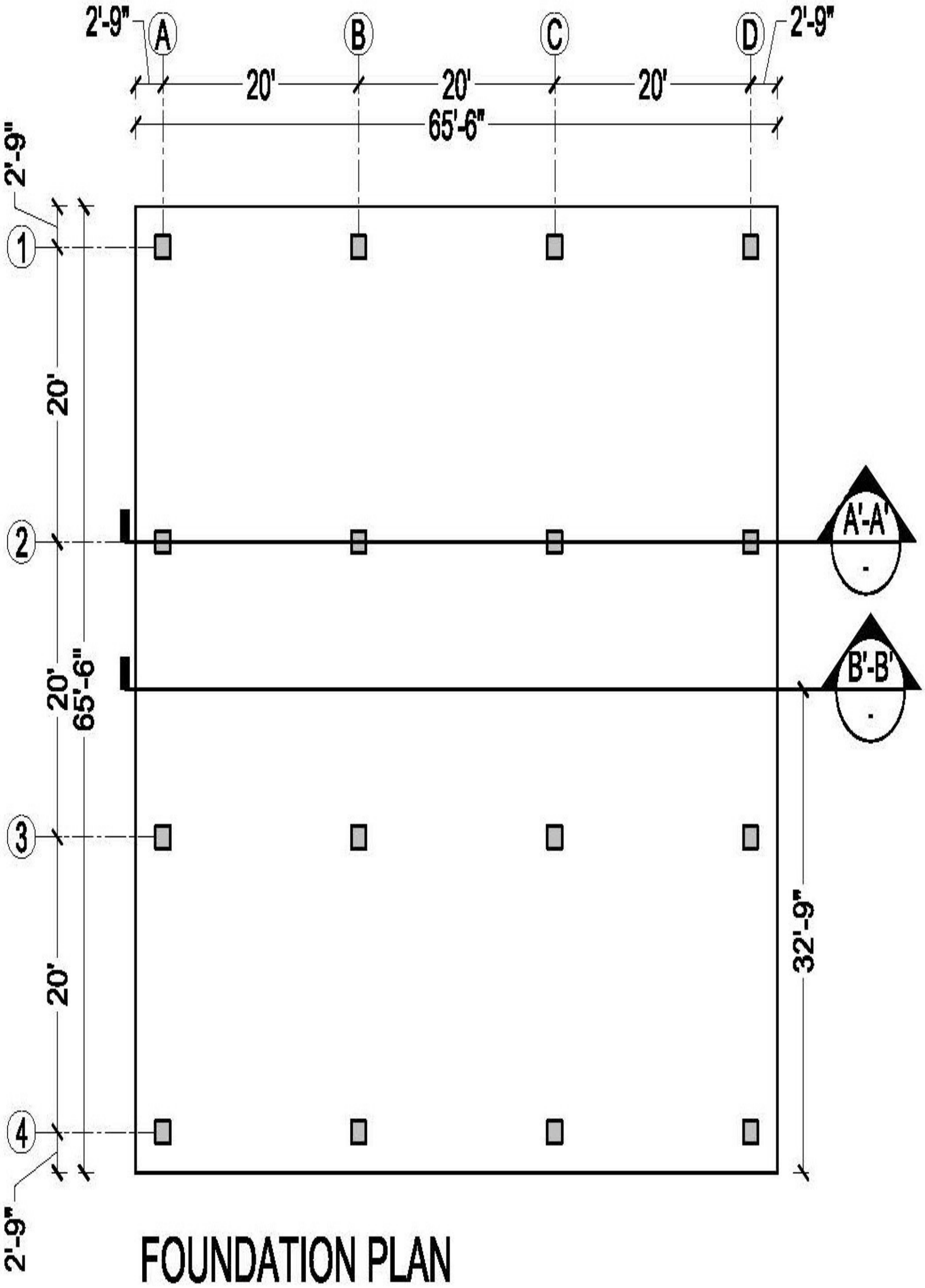


FIGURE 3.7: Vertical spring and dashpot intensity distribution over the foundation

Chapter 4

Analysis And Results

4.1 Introduction

The case study building as described in chapter 3 was modeled as per Approach - I, which is conventional design method without considering SSI effects. The same building is also modeled as per Approach - II, which incorporates SSI effects as per guide lines given in report published by National Institute of Standards and Technology titled as **NIST GCR 12-917-21 "Soil-Structure Interaction of Building Structures"**. After, modeling, analysis is carried out on twelve models, six from each approach. The different parameters, such as settlements, bending moments and shear force, that can impact mat foundation design are evaluated using both approaches and compared in this study. Apart from this a settlement analysis using elastic method is also conducted for all type of soil considered in this study. The settlement values computed from elastic analysis are compared with settlement profiles obtained from analysis conducted as per Approach - I and Approach - II.

4.2 Elastic Settlement Analysis

The increase of stress in soil layers due to the load imposed by structures at the foundation level is always be accompanied by some strain, which results in the settlement of the structures. The immediate settlement is sometimes referred to as the elastic settlement .For current study, calculation of settlement is done on the basis of theory of elasticity. This theory was first formulated by Timoshenko and Goodier (1951) and is represented by the following equation:

$$\Delta H = q_o B'(\frac{1-\mu^2}{E_s})(I_1 + \frac{1-2\mu}{1-\mu} I_2) I_F............(4.1)$$

And whereas Equation 4.1 was simplified by Joseph E. Bowles to equation 4.2

$$\Delta H = q_o B'(\frac{1-\mu^2}{E_s}) m I_s I_F...........................(4.2)$$

and also made a program called FFACTOR that performs the tedious calculations to calculate the values of Steinsbrenner's Influence Factor I_s and Fox's Influence Factor I_f. Using above formula, settlement values for different soil consistencies are evaluated and shown in Table 4.1

Table 4.1: Immediate Settlement (Elastic Method)

Soil Type	**Shear ModulusG(psf)**	**ΔH(in)**	**ΔH(in)**
		Edge	**Centre**
Soft 1	95036.950	1.271	1.589
Soft 2	443504.000	0.326	0.387
Soft 3	791975.000	0.205	0.243
Stiff 1	1600830.000	0.101	0.120
Stiff 2	2726460.000	0.066	0.078
Stiff 3	4312198.000	0.046	0.054

It can be seen in Table 4.1 that settlement values at center is greater than at edge, indicating dishing settlement profile of soil.This dishing settlement profile suggests that pressure at the centre of mat, takes more time to dissipate because of loaded soil around it as compared to the edge, where there is less pressure in adjacent soil.

4.3 Pressure Distribution and Settlement

In this section pressure distribution and settlement profile as obtained from analysis results of Approach - I and Approach - II are discussed and compared.

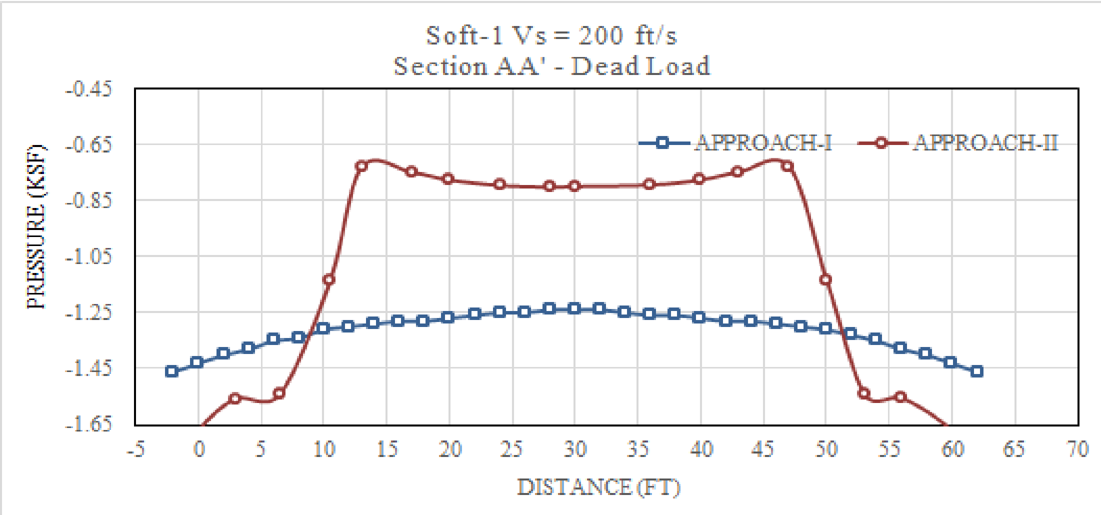


FIGURE 4.1: Comparison of Pressure Distribution Profile of Approach - I and Approach - II at Section AA' for dead load case

Fig. 4.1 compare the pressure distribution profile of Approach - I and Approach - II over soft soil - 1 along section A-A' due to dead load case. Due to use of constant value of subgrade reaction in Approach - I analysis a concave downward graph obtained for the pressure profile beneath the raft. The pressure at the edges are greater than at center of raft. The variation of pressure across section is gradual except column locations. The pressure distribution profile of Approach - II is also concave downward across section but effect of choosing different values of subgrade reaction across the section can be seen clearly with the peak edge stresses as compared to relative to quite low stresses at centre of raft.

Fig. 4.2 compares the results of settlement profile of raft foundation over soft soil using Approach-I and Approach-II along section A-A' due to dead load case. Approach - I analysis results has concave down profile with settlement value more at the edge as compared to the value at the center of the raft. The settlement computed at the edge is around -0.9 inches and at the center is around -0.65 inches.

Approach - II analysis results has concave up profile with settlement values more at the center as compared to the values at the edge of raft. The settlement computed at the edge is around -1.2 inches and at the center is around -1.38 inches.

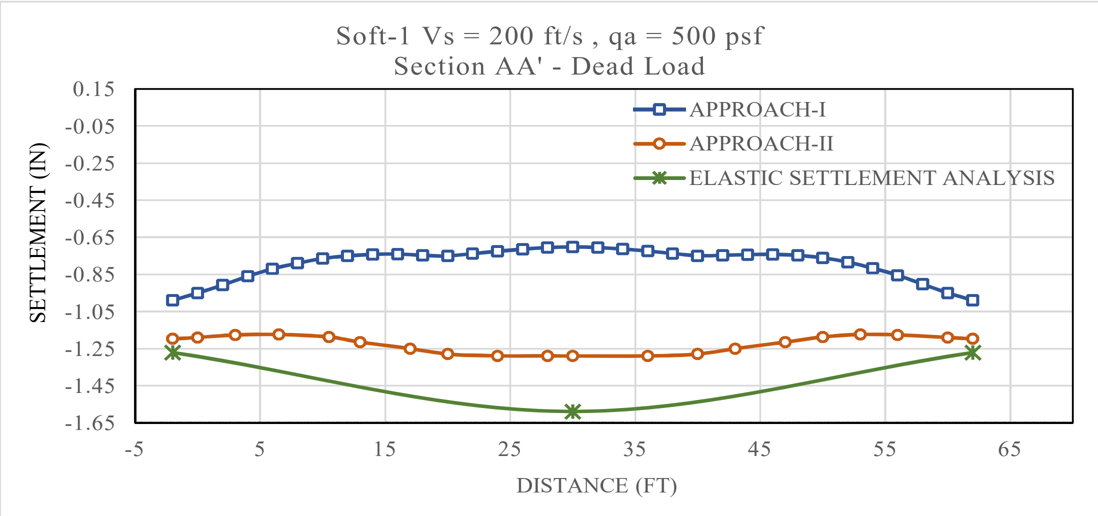


FIGURE 4.2: Comparison of Settlement Profile of Approach - I and Approach - II at Section AA' for dead load case

The difference between the values from Approach - I and Approach - II increases as one move from edge to center, indicative of opposite behavior of settlement. The difference in values in two profiles at edge and at center is 0.3 inches and 0.48 inches respectively. When these results are compared to elastic analysis results as shown in Table 4.1, one can find a closer match with Approach - II as compared to Approach - I. The values at edge is -1.27 inches from elastic analysis for soft soil is quite close to value of -1.2 inches at same point obtained from Approach-II analysis. While at center of raft elastic analysis shows a value of -1.58 inches as compared to -1.38 inches calculated through Approach - II analysis. Although, the difference increases as one move from edge to center which is 0.07 inches to

0.2 inches but the fact that settlement at the edge is less than the settlement at the center remains the same in the both analysis.

Approach - I profile is contrary to Approach - II profile and values obtained from elastic analysis as can be seen in Fig 4.2. The difference in values between Approach - I and elastic also increases as one move from edge to center quite appreciably which is approximately 0.3 inches at the edge and 0.68 inches at the center. The results of Approach-II follows the same pattern of dishing profile as mentioned in [17].

From Fig 1 to Fig 5 in appendix A, comparison of Approach - I and Approach - II continues for stiffer consistencies of soil. As soil gets stiffer, the difference between settlement values getting less and in close agreement with each other but the shape of profile continues to be concave down and concave up for Approach - I and Approach - II respectively. The values obtained from elastic analysis as per Boussinesq's analysis presented in Table 4.1, are also in close agreement with Approach - II values i.e. less values of settlement at edges than the settlement values at center. This shows that effect of SSI is getting less pronounced as soil gets stiffer but the behavior of raft is better predicted through Approach - II as compared to Approach - I for all types of soils.

The same behavior is observed along section B-B' for dead load case which can be seen in Fig 6 to Fig 8 given in appendix A for all soil consistencies.

Fig 4.3 illustrates the settlement comparison of raft foundation for all soils in Approach - I for dead load case at section A-A'. It can be observed that soil with higher values of modulus of subgrade reaction showed the lesser deformation or settlement, whereas, the soil with lower values of modulus of subgrade reaction gave the higher settlement. The overall settlement profiles are similar as presented by the Boussinesq's theory for stress distribution under the foundations.

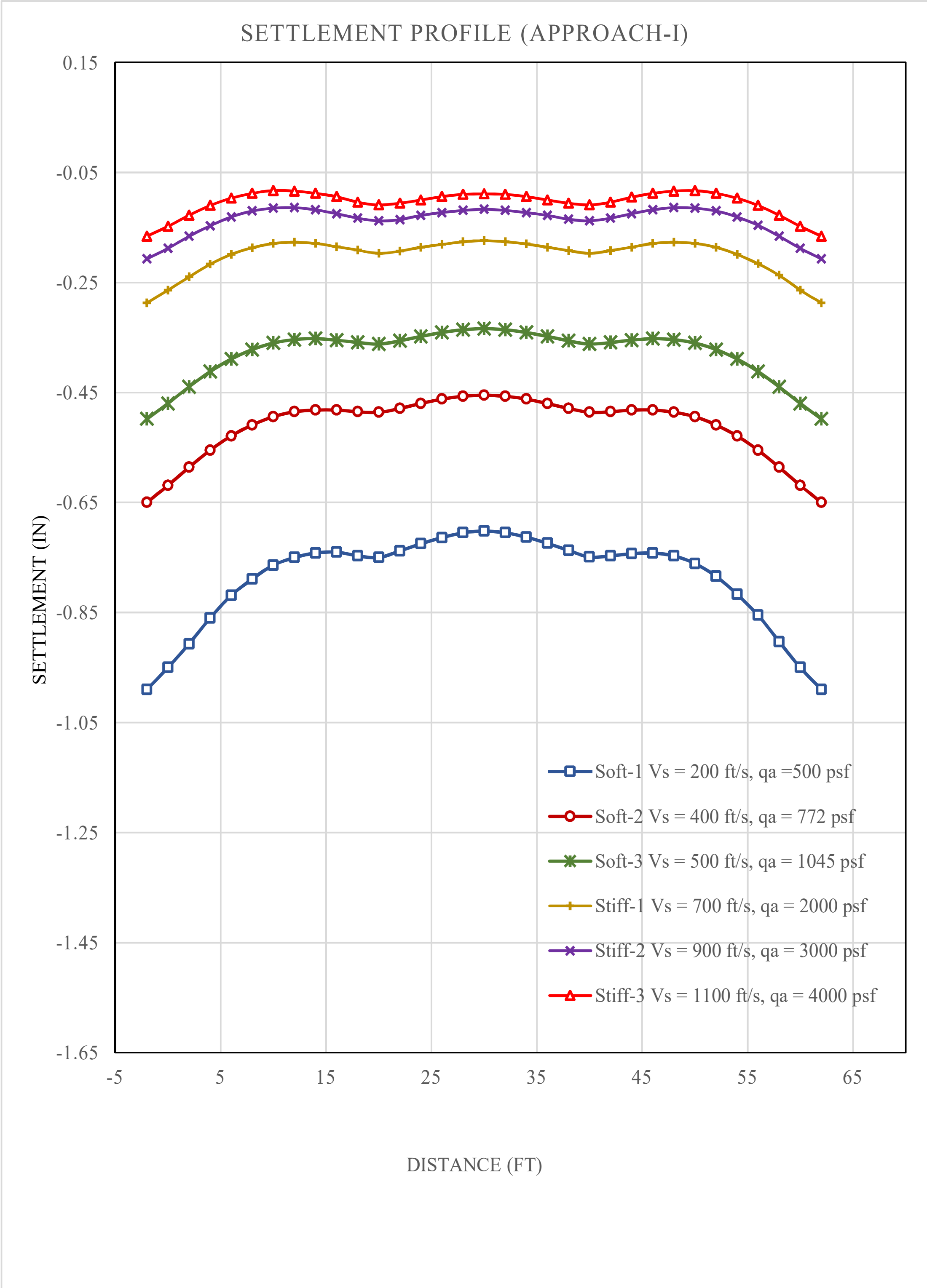


FIGURE 4.3: Settlement Profile of Approach - I at Section AA' for dead load case

Fig 4.4 shows the settlement comparison of raft foundation using Approach - II for all six types of soils at section A-A'. Fig 4.5 and Fig 4.6 illustrates the comparison of raft settlement for all soil types using Approach - I and Approach - II at section B-B' for dead load case respectively.

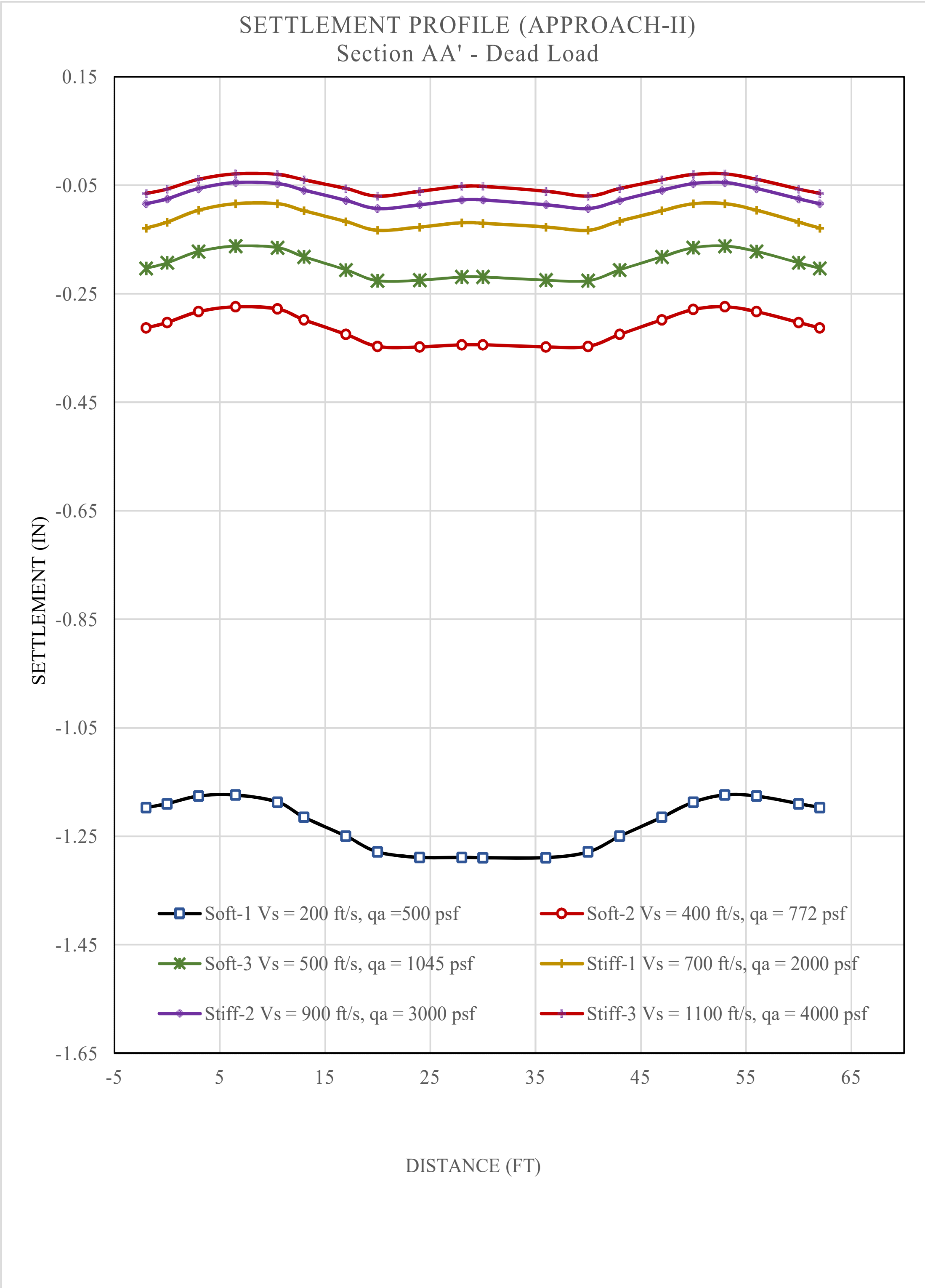


FIGURE 4.4: Settlement Profile of Approach - II at Section AA' for dead load case

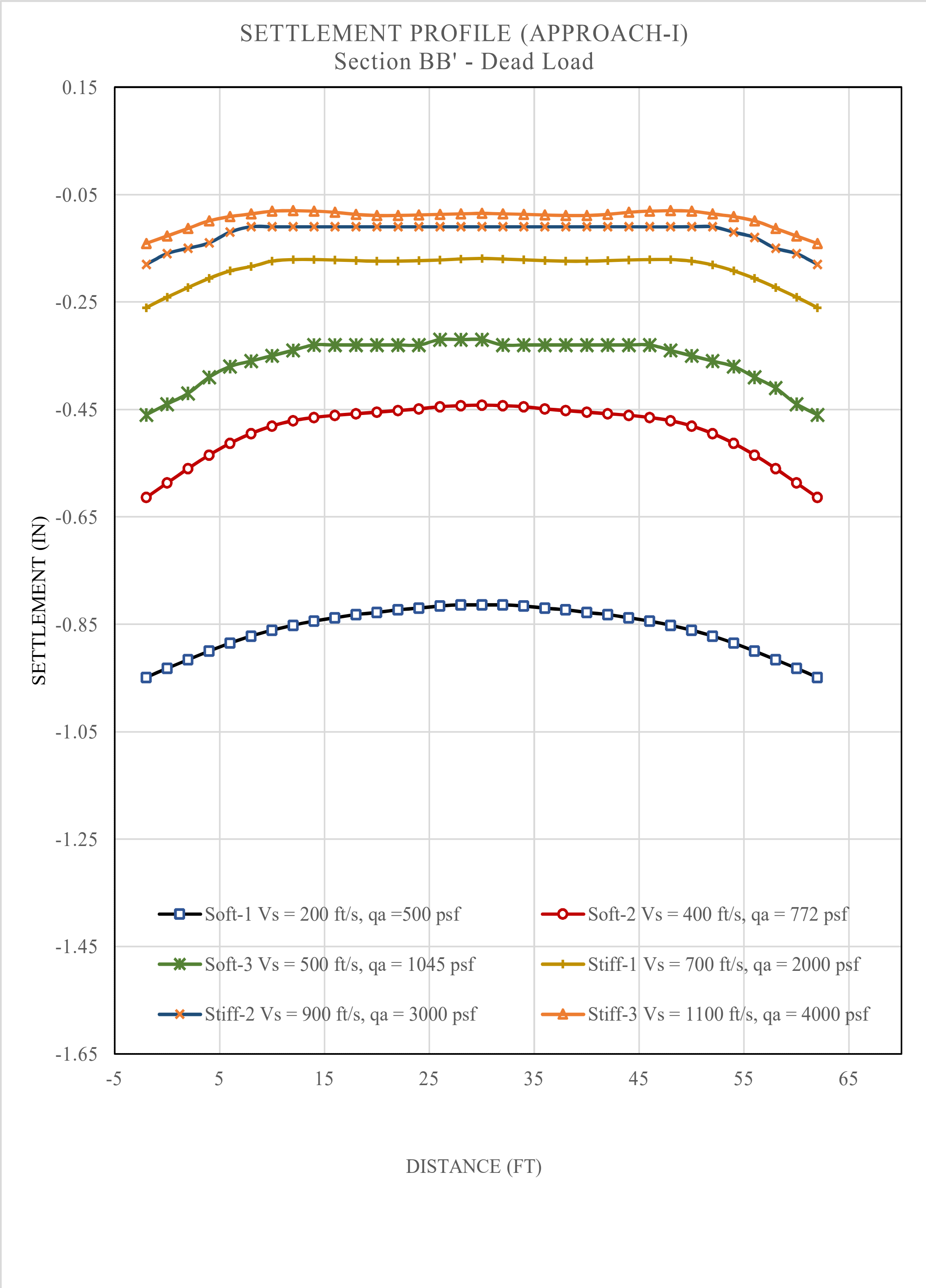


FIGURE 4.5: Comparison of Settlement Profile of Approach - I at Section BB' for dead load case

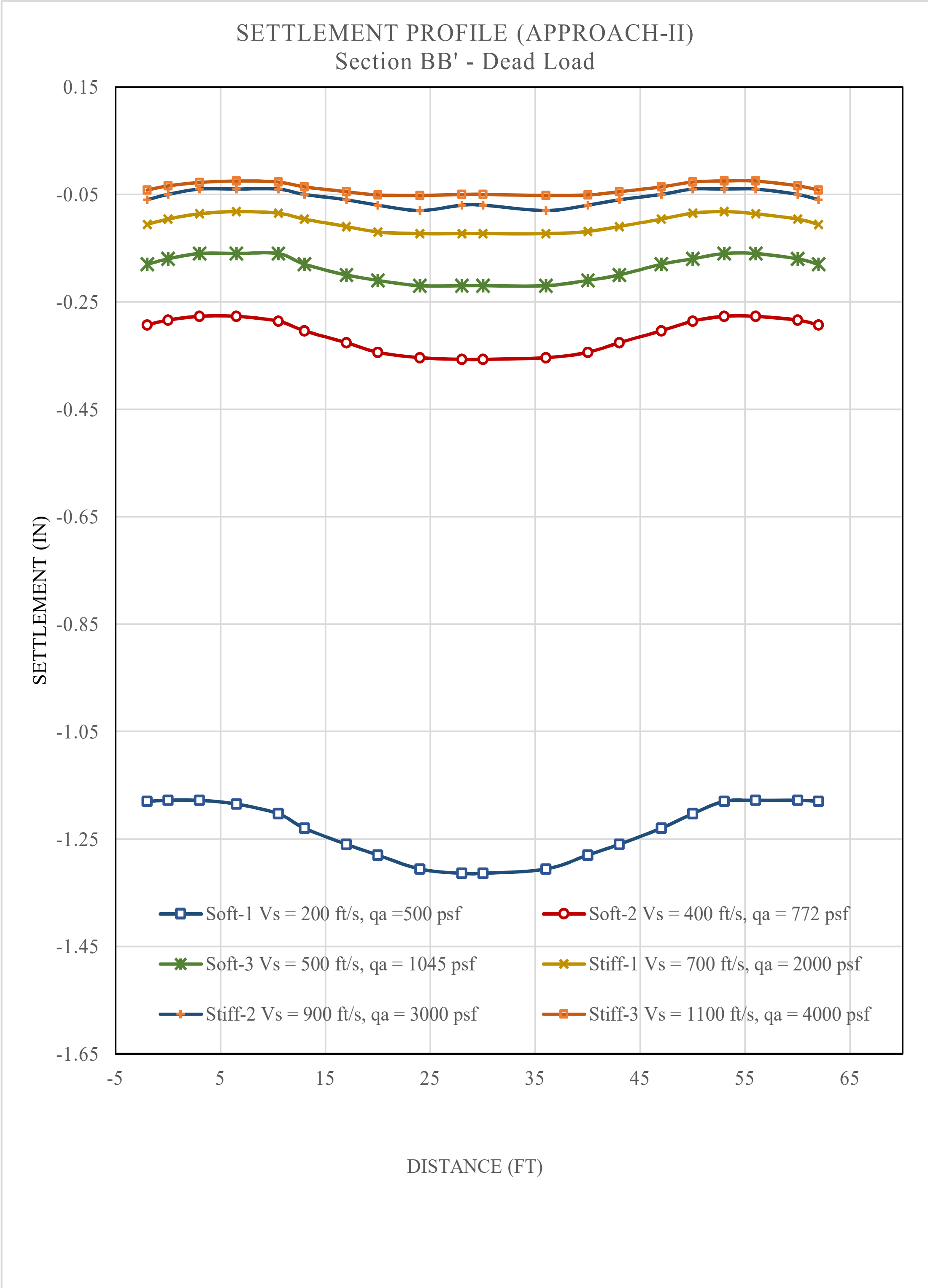


FIGURE 4.6: Comparison of Settlement Profile of Approach - II at Section BB' for dead load case

The same raft was also analyzed for lateral load case and results for section A-A' and B-B' is shown The comparison between the settlement profiles of different soil consistencies are illustrated from Fig 4.7 to 4.10. The comparison between the both approaches are shown in appendix A from Fig 9 to Fig 14. Same behavior was observed as observed in dead load case.

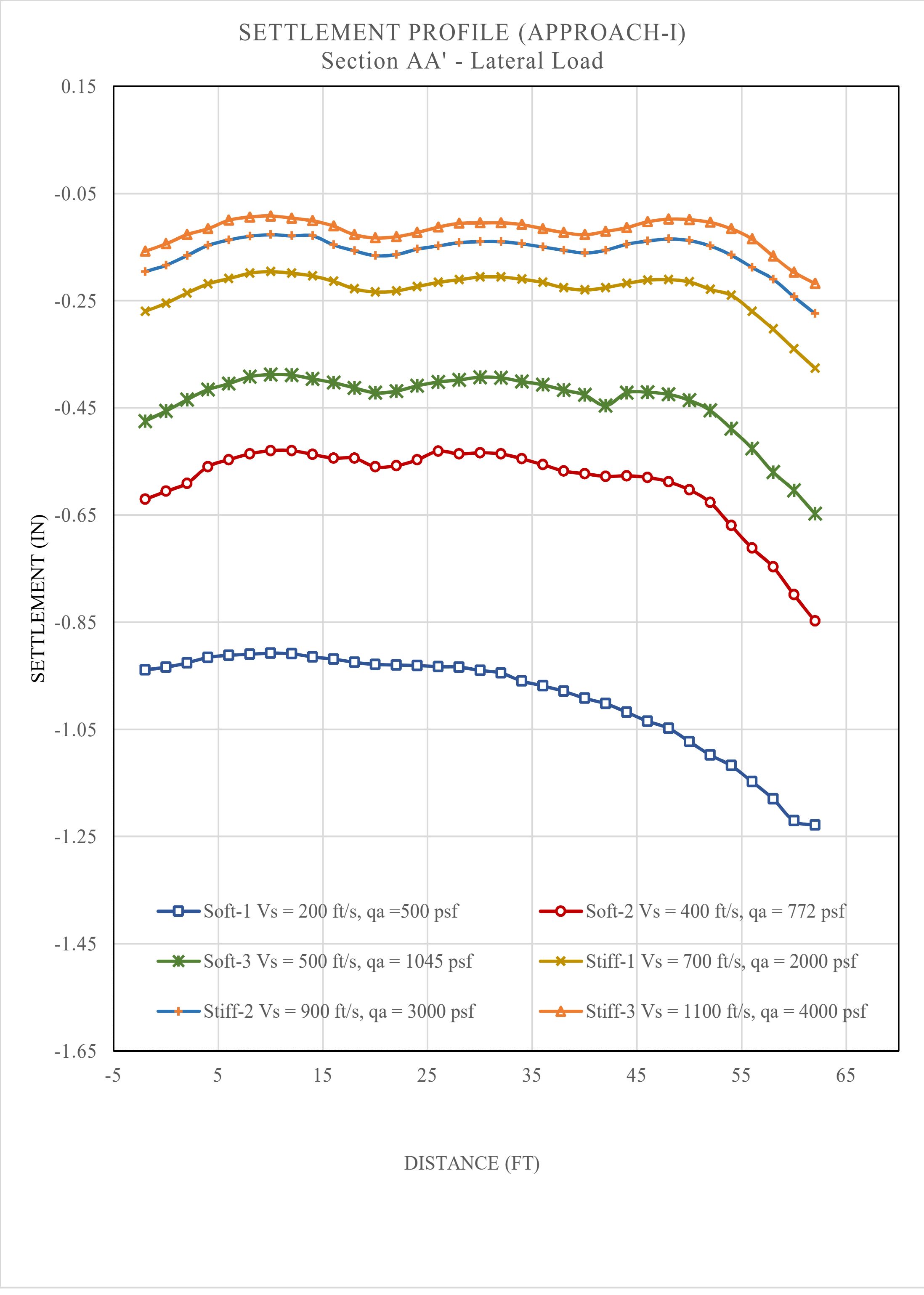


FIGURE 4.7: Comparison of Settlement Profile of Approach - I at Section AA' For Lateral Load

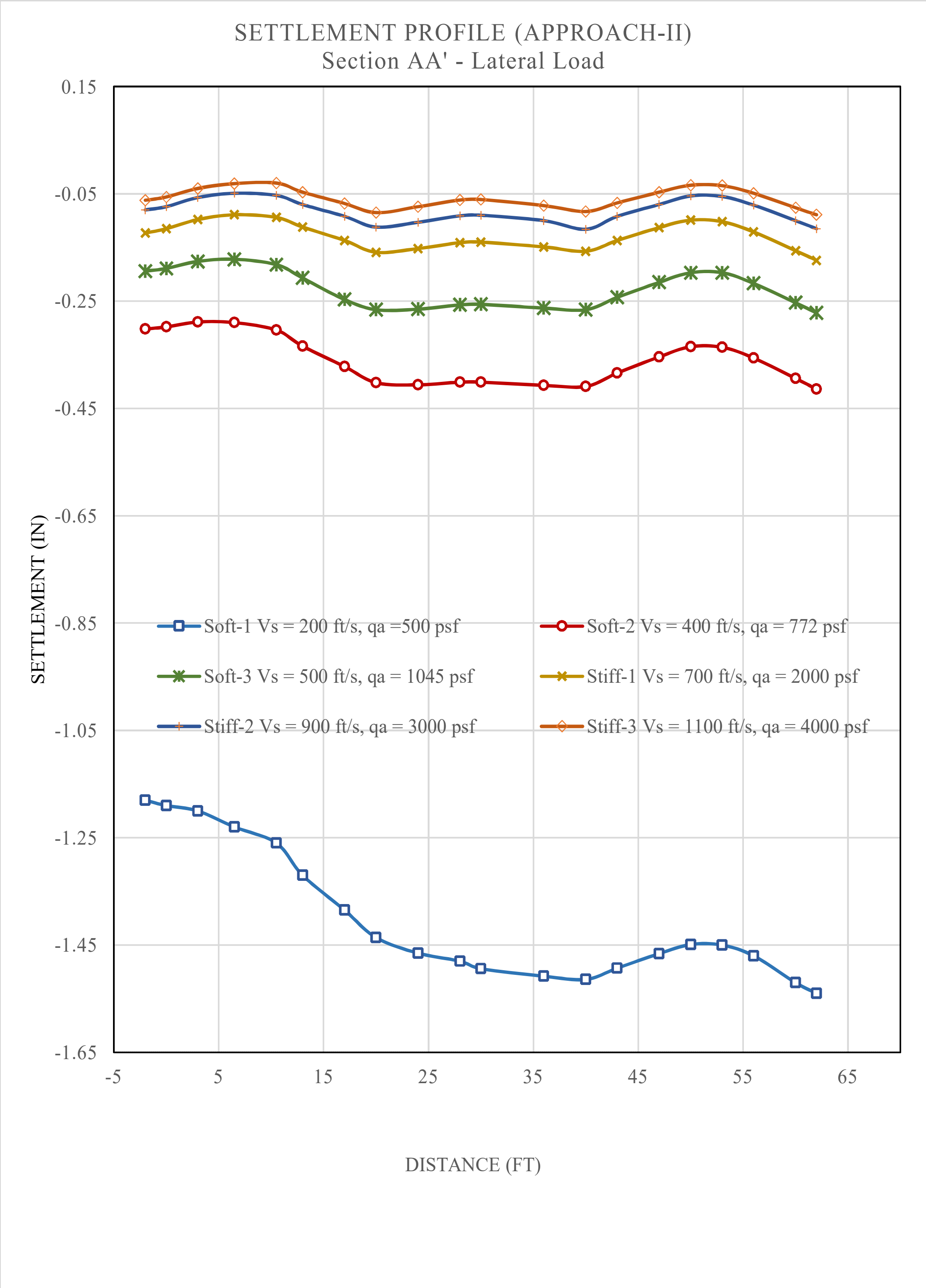


FIGURE 4.8: Comparison of Settlement Profile of Approach - II at Section AA' For Lateral Load

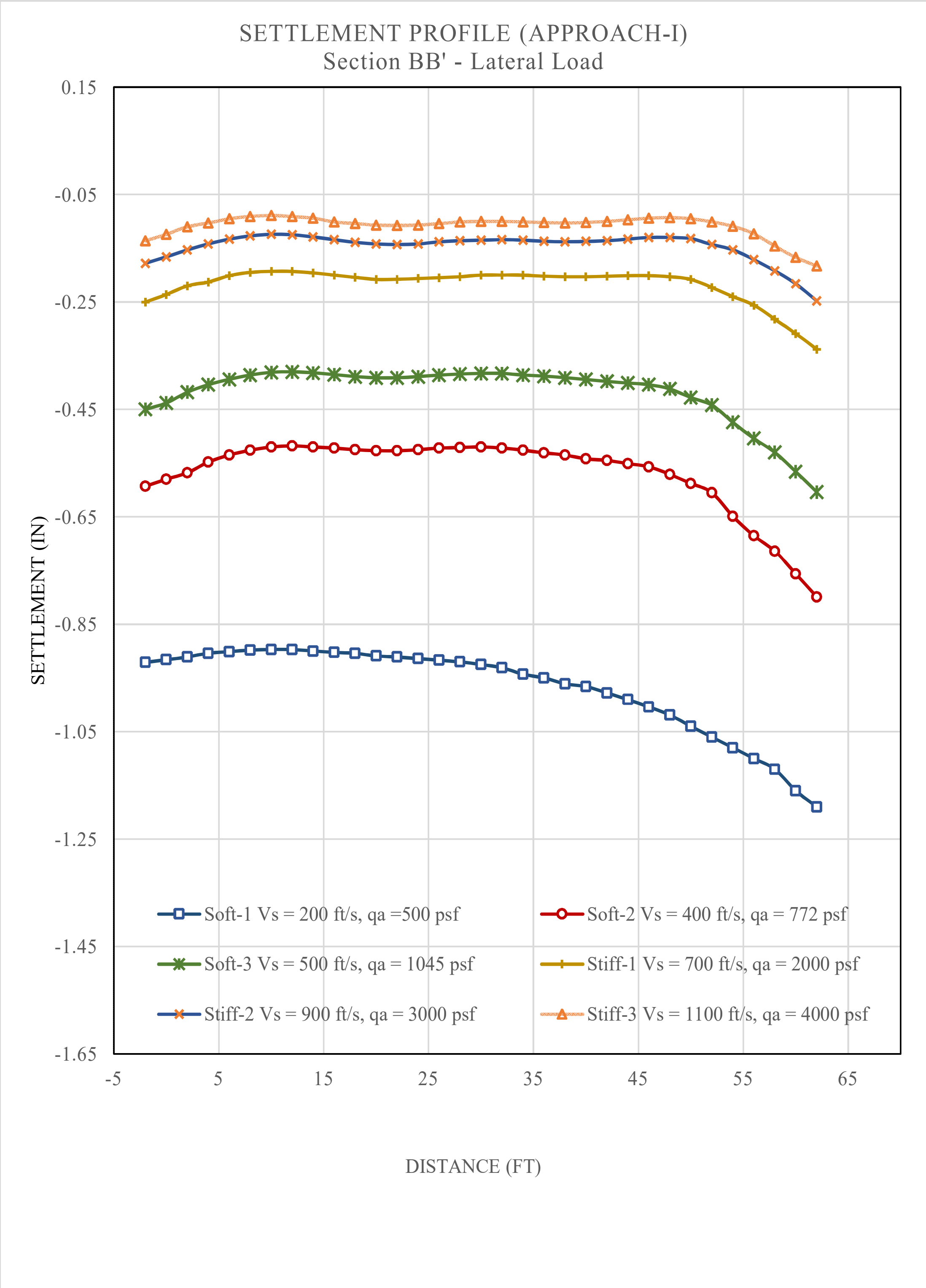


FIGURE 4.9: Comparison of Settlement Profile of Approach - I at Section BB' For Lateral Load

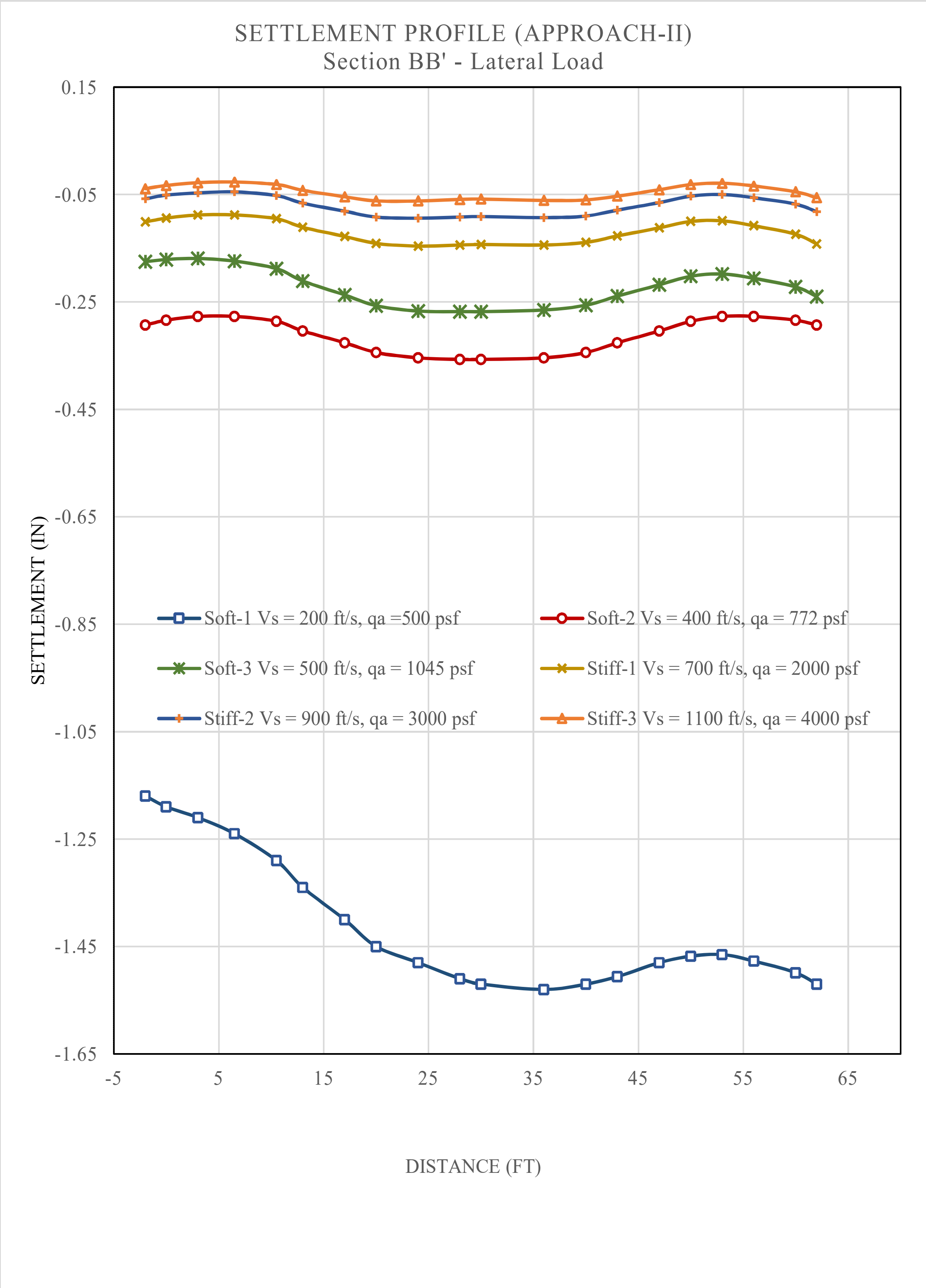


FIGURE 4.10: Comparison of Settlement Profile of Approach - II at Section BB' For Lateral Load

4.4 Bending Moment

In mat foundation design prediction of bending moment is one of the most important parameter, in order to correctly place reinforcement. The raft bending moment was obtained through Approach - I and Approach - II at section A-A' (along column line) and at section B-B' (through mid of raft) for dead and lateral load cases and compared accordingly. The results are shown through graphs in Fig 4.11 to Fig 4.19.

Fig. 4.11 compares the results of bending moment of raft foundation over soft soil as calculated through Approach - I and Approach - II along section A-A' due to dead load case. In this analysis, positive moments are defined as moments which cause tension at bottom and negative moment which cause tension at the top. Positive moments are recorded under all columns due to heavy concentrated loads while negative moments are recorded between columns.

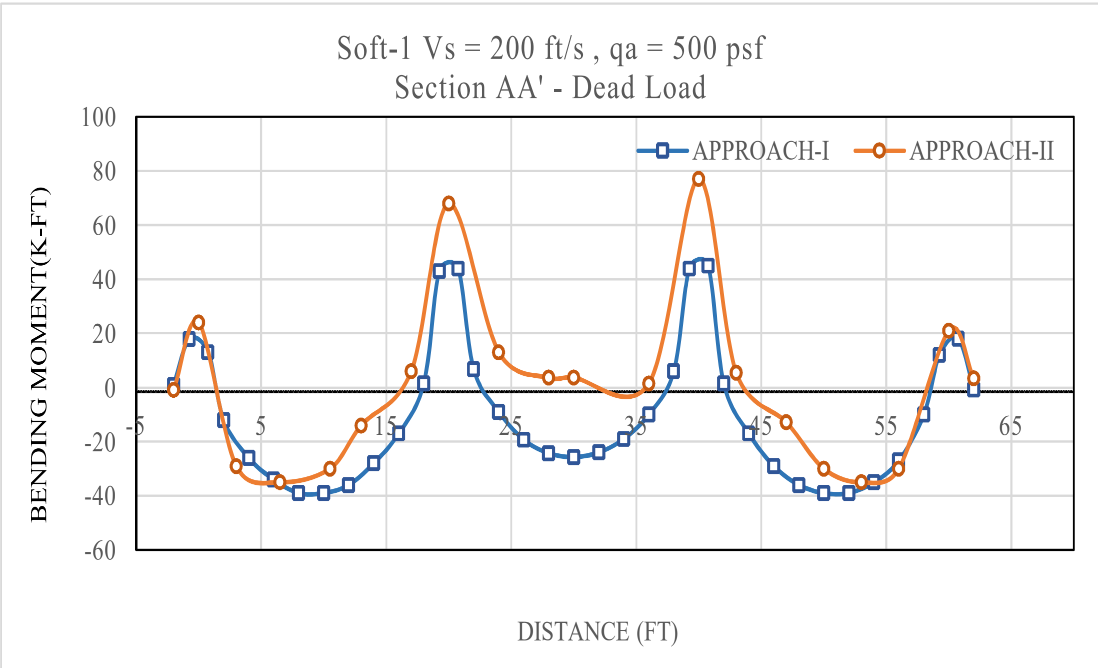


FIGURE 4.11: Comparison of Bending Moment of Approach - I and Approach - II at Section AA' for dead load case

Although settlement profile as discussed in previous section, is quite contrary to each other for both approaches but in the exterior span regions shape of settlement profile is similar. That is why the negative moment are recorded in both cases. The value of this negative bending moment as calculated by Approach - II is approximately 20% less than value calculated through Approach - I. The interior span is the region where settlement profile differs significantly. In Approach - I, due to concave downward profile, a negative bending moment appears in bending moment diagram of order -25 kip-ft /ft and in Approach-II, concave upward profile produces a small positive bending moment indicating a flatter region in the middle of raft. The difference in the values of bending moment is quite appreciable and also due to positive bending moment, the reinforcement configuration will be opposite in the final raft design of Approach - II as compared to Approach - I. This pattern of bending moment continues as the soil gets stiffer and difference in values are getting less.

The positive bending moment under exterior columns are same for the both approaches but bending moment under interior columns are around 60% more recorded in Approach - II as compared to Approach - I. Therefore, more reinforcement is required under all the interior columns in case of Approach - II. This illustrates that by using guidelines given in NIST report regarding determination and application of modulus of subgrade reaction under raft foundation will give a profile closer to elastic settlement analysis and thus produce a distribution of bending moment at critical section opposite to the Approach - I analysis. This fact can be substantiated with the help of monitoring the response of structure through instrumentation.

Fig 15 to Fig 19 in appendix B shows the bending moment diagram for remaining five soil consistencies. These graphs indicate a similar pattern as seen for the Soft-1 soil but as the soil get stiffer, SSI effect gets diminished supporting the fact that SSI effect will not be significant in stiffer soils. Beside this, one can observe from the graphs that negative moment starts appearing in Approach - II in interior span region as soil gets stiffer and eventually become almost equal for Stiff-3 soil to the values obtained from the Approach - I analysis.

For Section B-B' the comparison of bending moment of Approach - I and Approach - II considering dead load case for all six soil types as mentioned in this study is given in Fig 20 to Fig 22 of appendix B. In Soft-1 case the negative bending moment of interior span region as calculated by Approach - I is contrary to positive bending moment as calculated by Approach - II which greatly affect the final design of raft foundation.

Fig 4.12 combines the results of bending moment at section A-A' for dead load case as calculated with Approach - I while Fig 4.13 displays the results of Approach - II.

Fig 4.14 and 4.15 showed combined results of Approach - I and Approach - II respectively for dead load case at section B-B' for all soil types as mentioned in this study. The results of lateral load case showed similar behaviour expect that there is a shift towards the right side in the direction of earthquake.

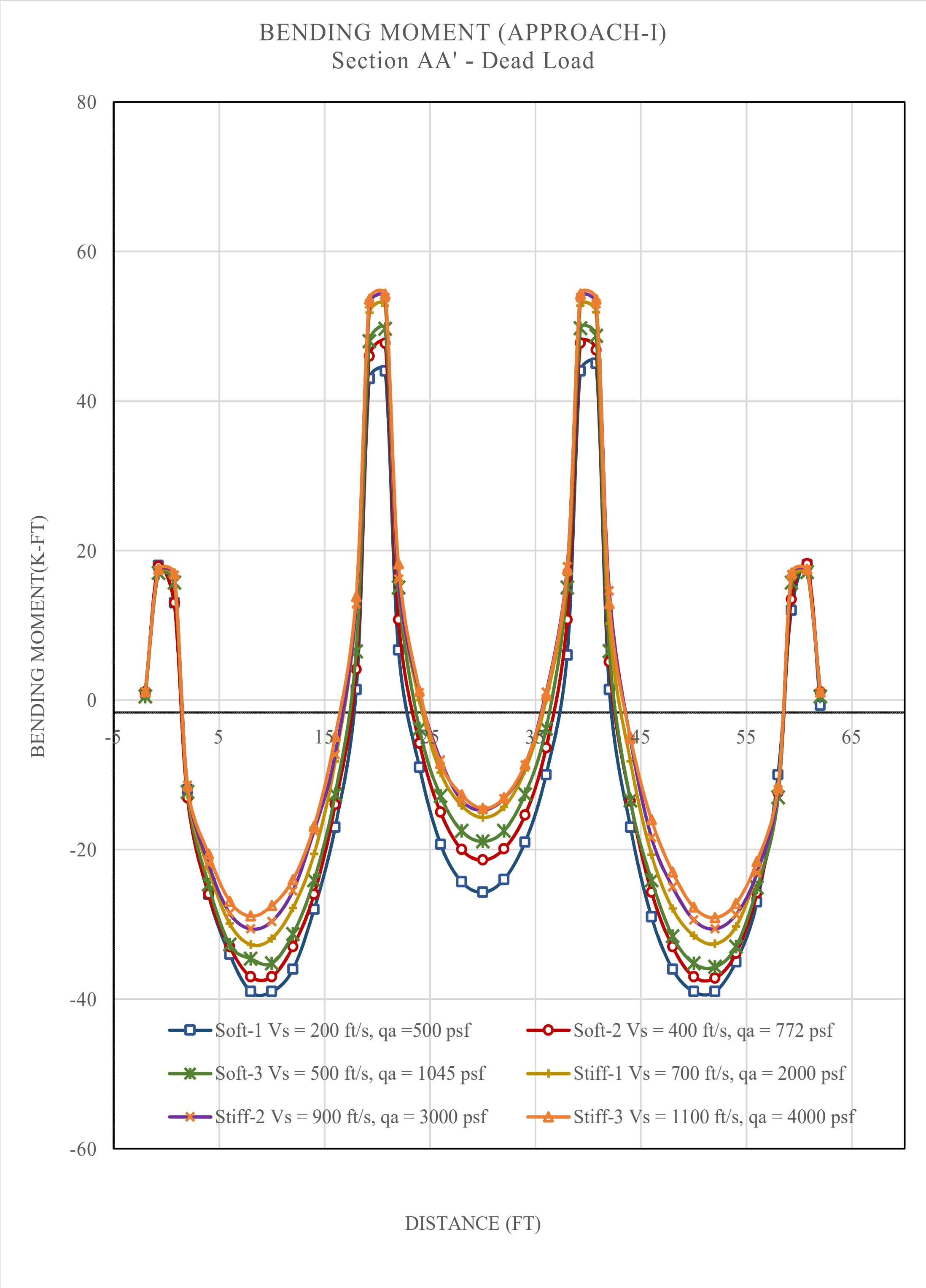


FIGURE 4.12: Bending Moment profile of Approach - I at Section AA' for dead load case

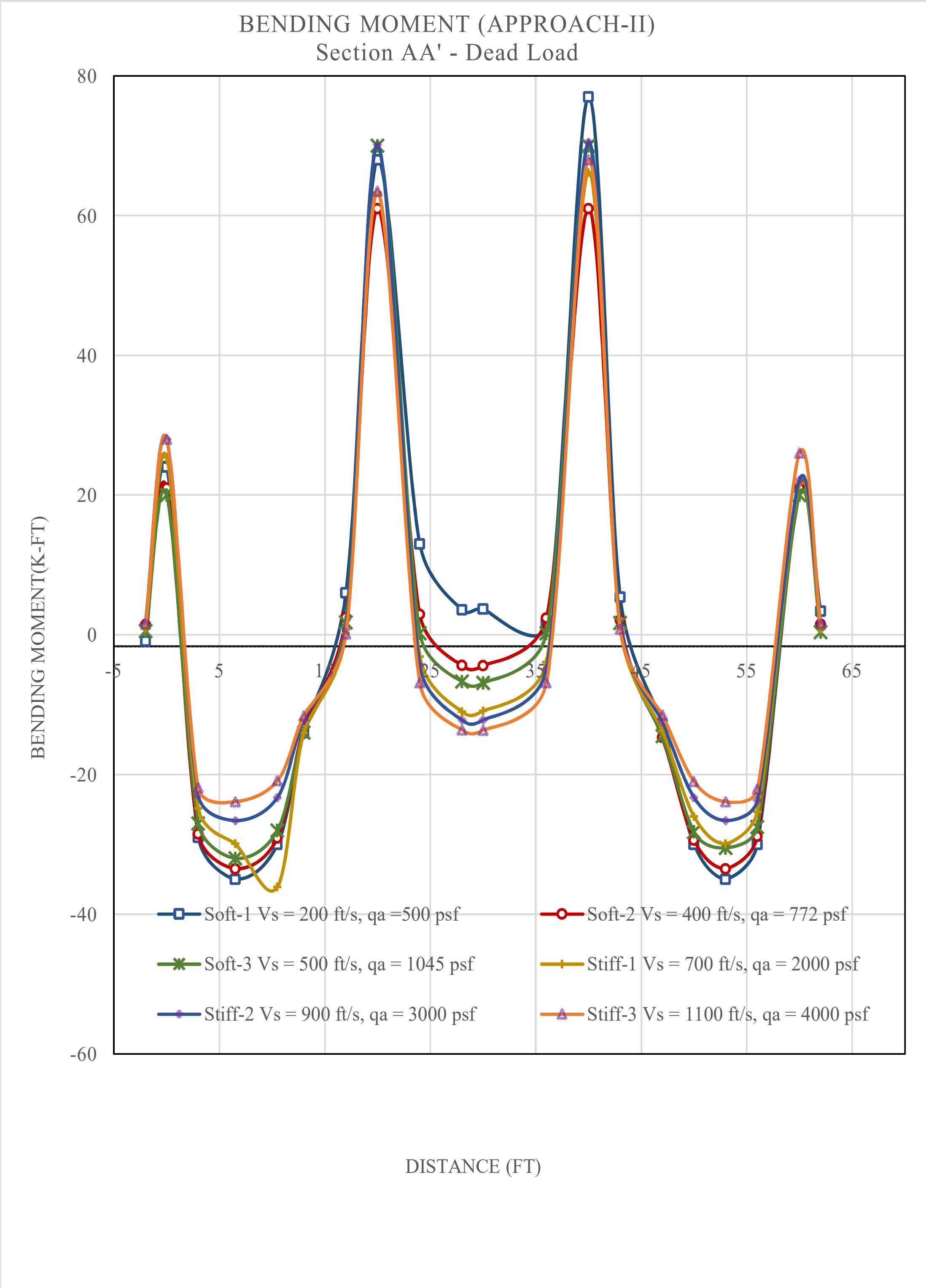


FIGURE 4.13: Comparison of Bending Moment profile of Approach - II at Section AA' for dead load

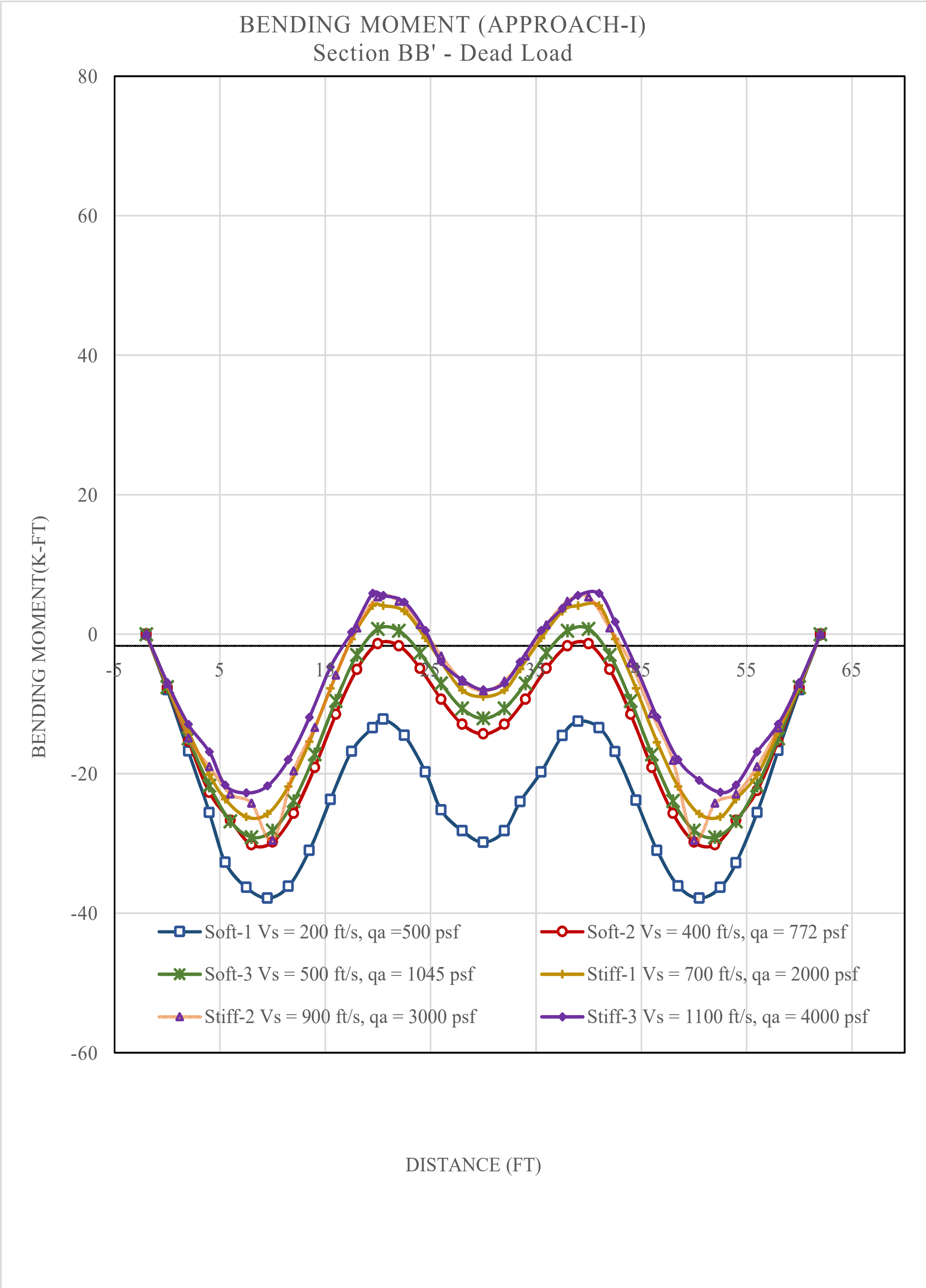


FIGURE 4.14: Comparison of Bending Moment profile of Approach - I at Section BB' for dead load case

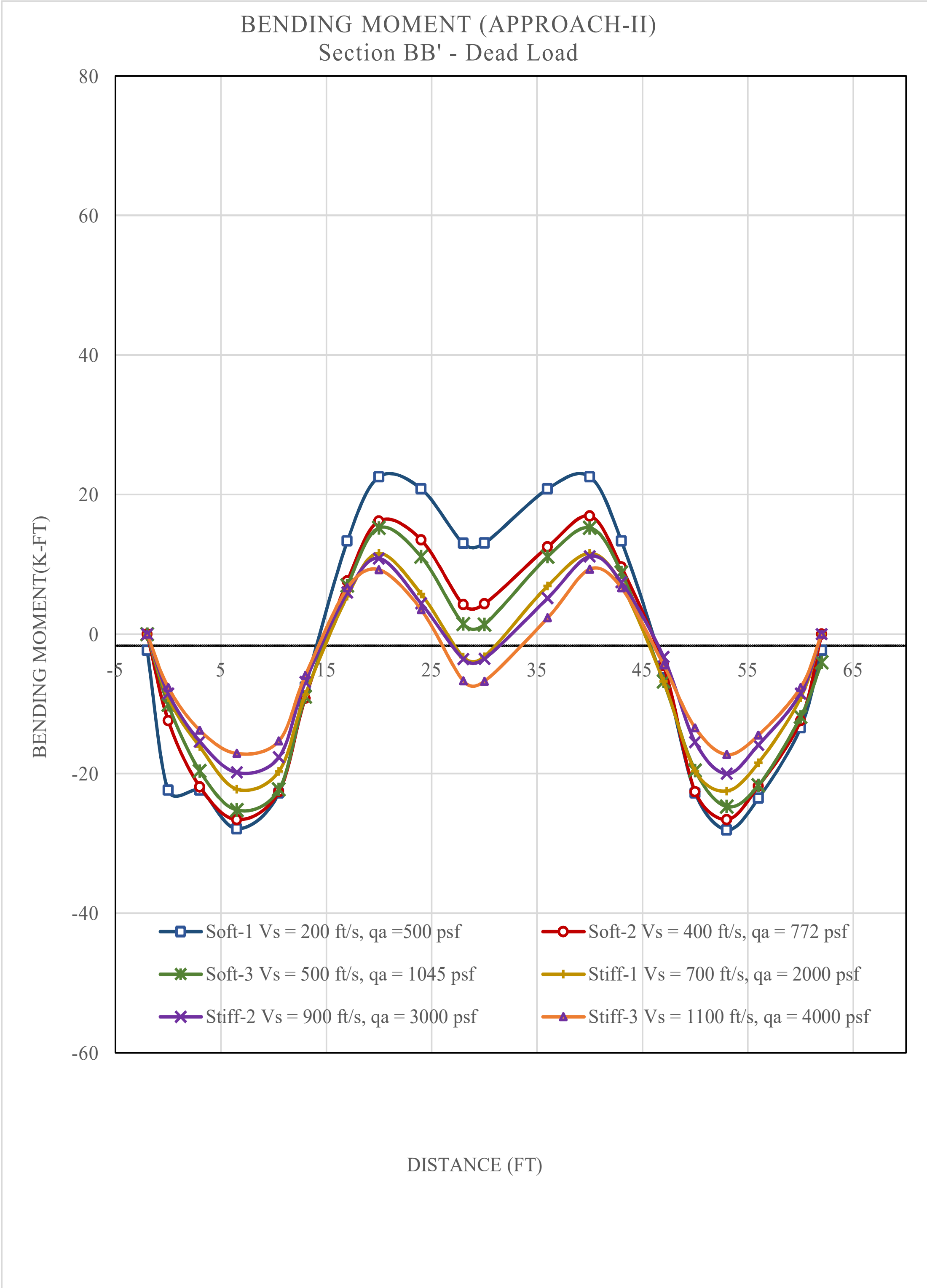


FIGURE 4.15: Comparison of Bending Moment profile of Approach - II at Section BB' for dead load

The results for the lateral load case at section A-A' and B-B' for all soil types as mentioned in this study are shown in Fig 4.16 to 4.19 and in Fig 23 to Fig

28 of appendix B . Although, the positive bending moment under columns as per Approach - II is 60% greater than as per Approach - I but a 20% lesser negative bending moment calculated as per Approach - II in interior and exterior span over a greater area, does offset this cost. Beside this, one can achieve a correct distribution of reinforcement as the elastic settlement analysis also predicting the same.

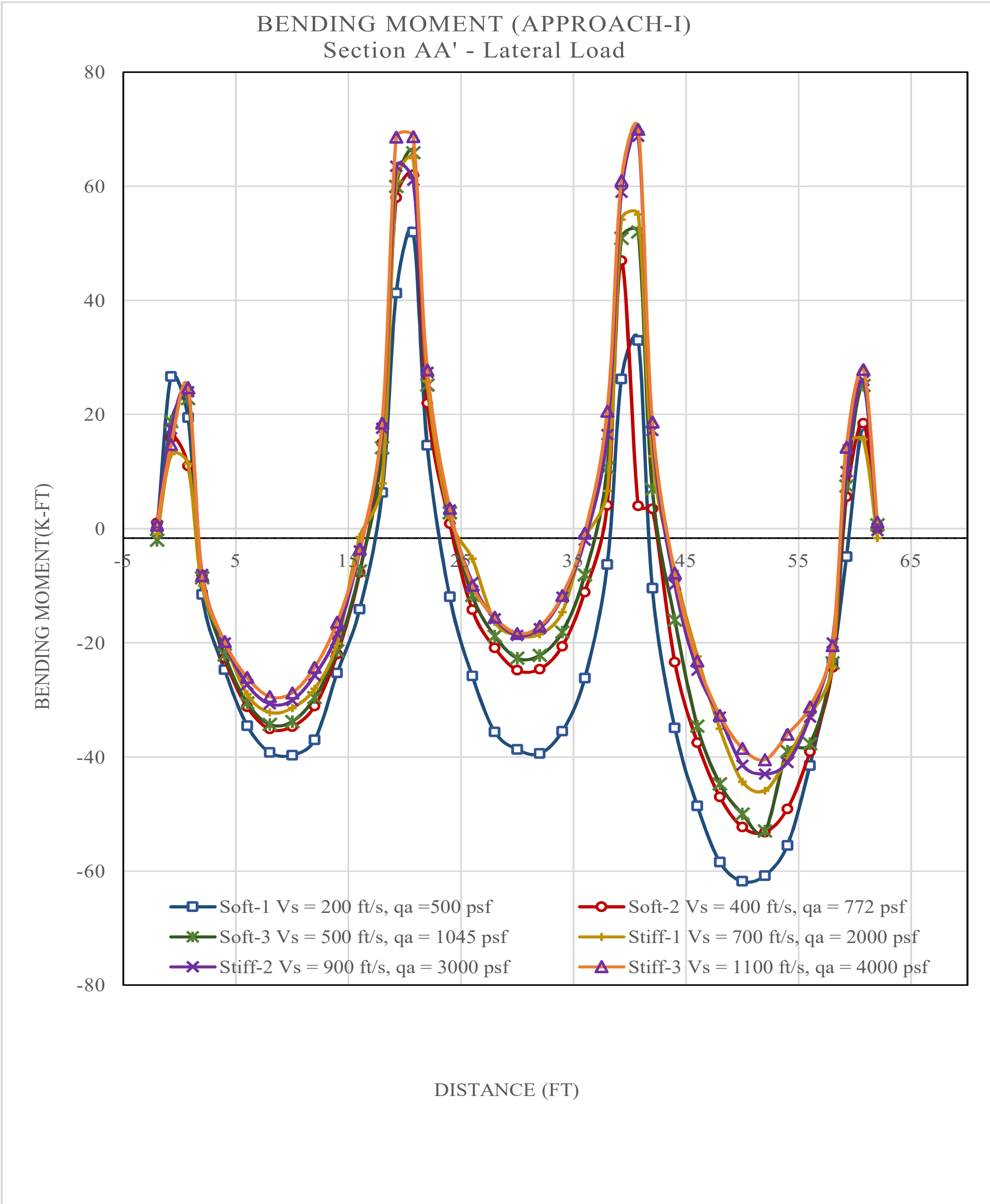


FIGURE 4.16: Bending Moment Profile of Approach - I at Section AA’ for lateral load

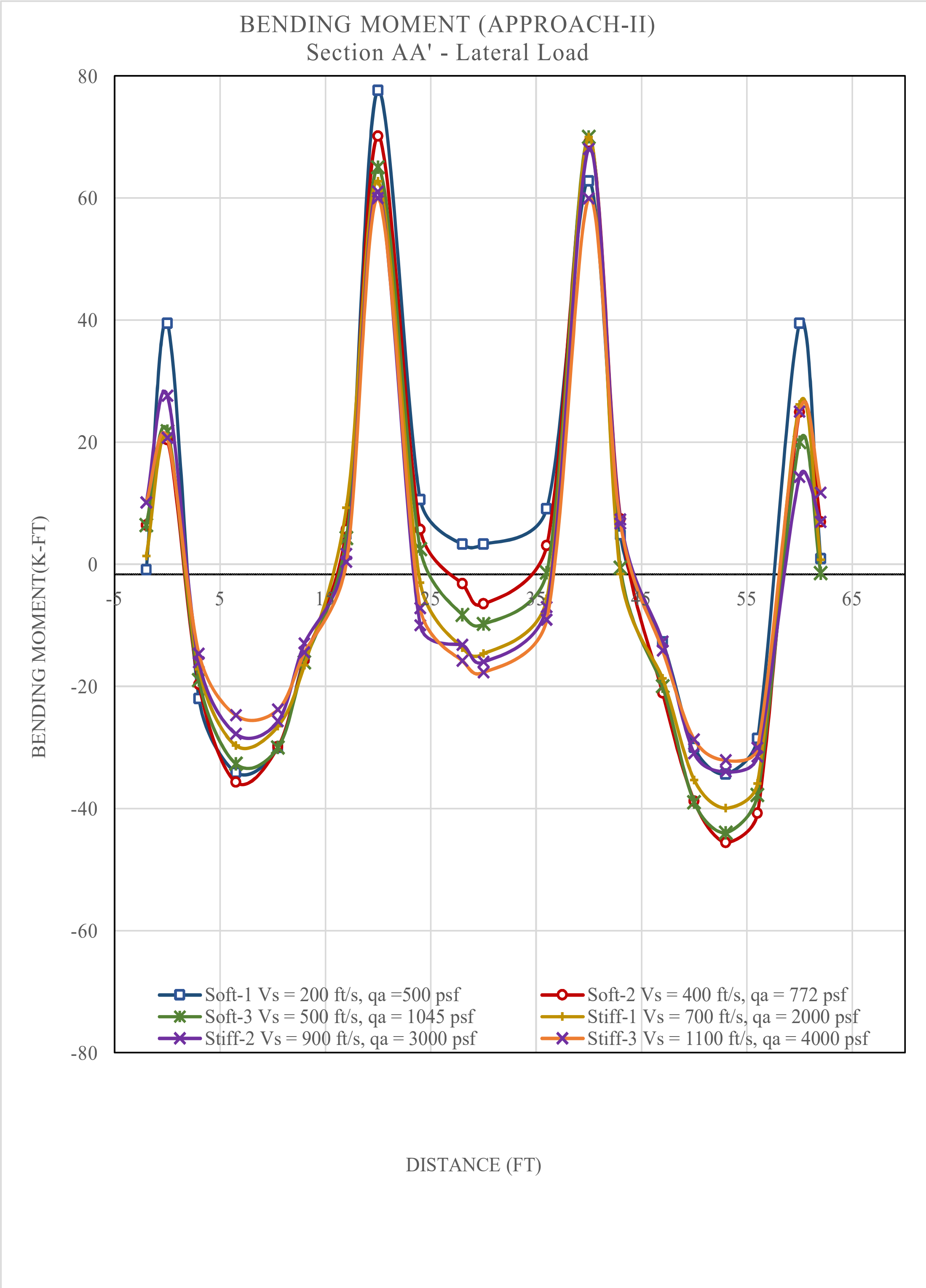


FIGURE 4.17: Bending Moment Profile of Approach - II at Section AA' for lateral load

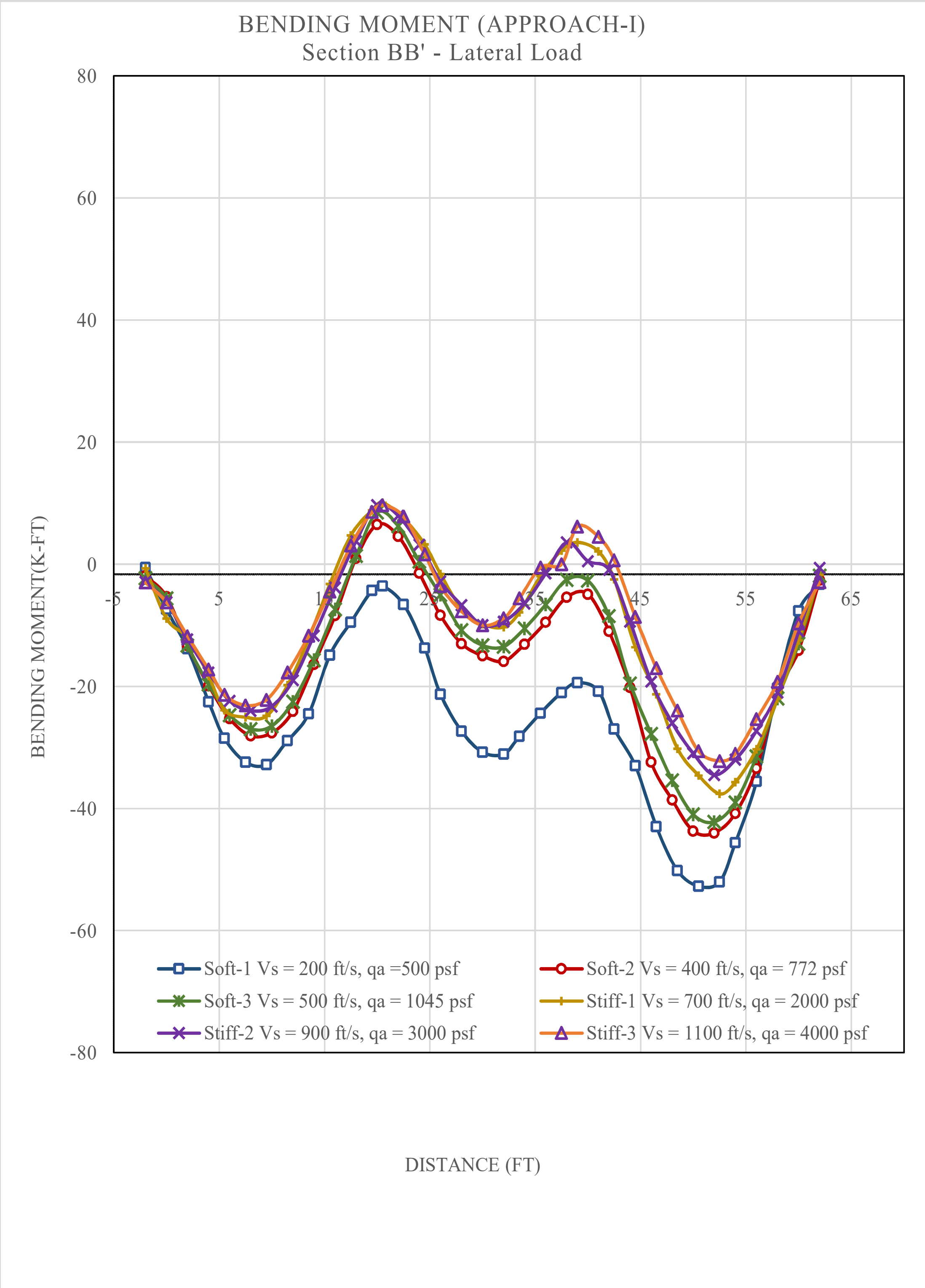


FIGURE 4.18: Bending Moment Profile of Approach - I at Section BB' for lateral load

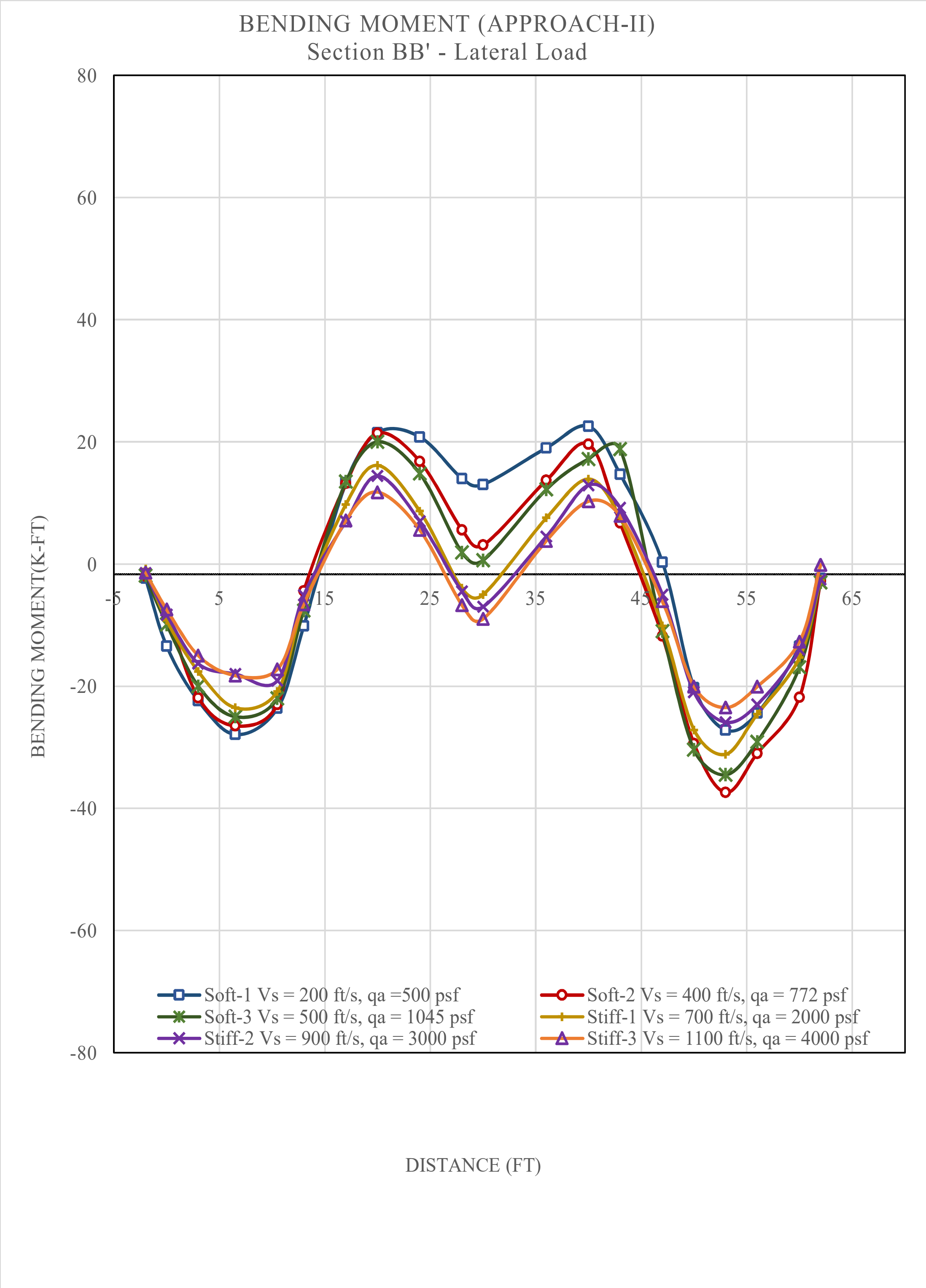


FIGURE 4.19: Bending Moment Profile of Approach - II at Section BB'

4.5 Shear Force

In current study, the profile of shear force is evaluated for Approach - I and Approach - II. The comparison of shear force profile obtained from Approach - I and Approach - II are shown in Figure 4.20 for soft soil 1 whereas, the results of remaining five soil types are shown in Fig 29 to Fig 33 of appendix C.

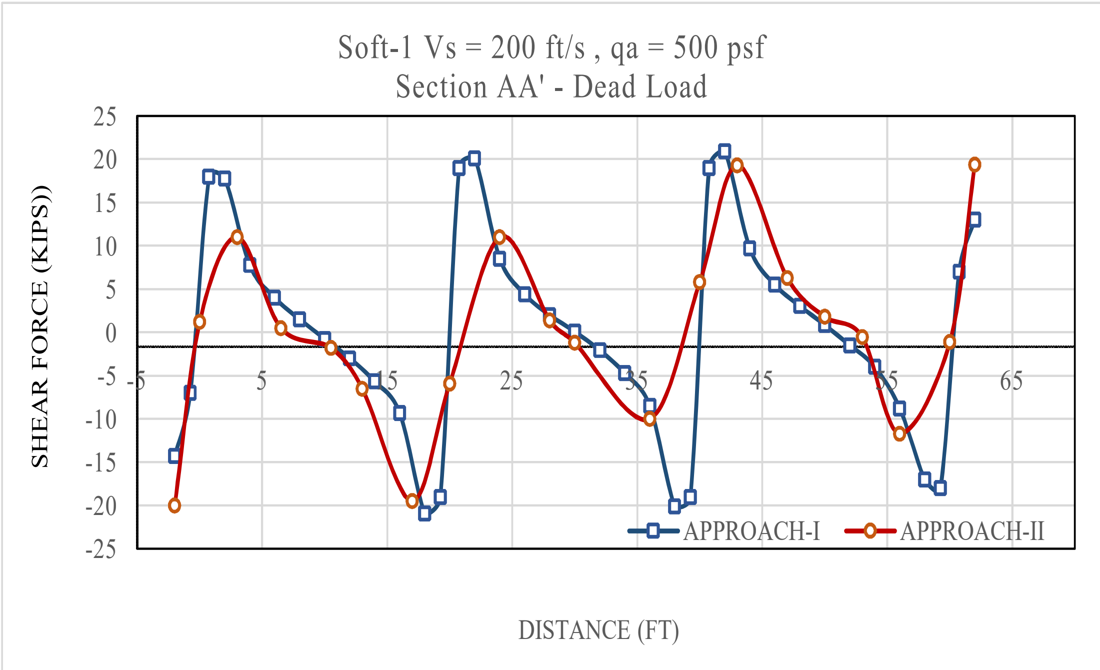


FIGURE 4.20: Comparison of Shear Force of Approach - I and Approach - II at Section AA' for dead load case

Shear Force at column line calculated from Approach - I is shown in Figure 4.21 whereas from Approach - II is shown in Figure 4.21. In case of Approach - I, greater values of shear force at some location are obtained as compared to Approach - II.

Shear Force at centre, calculated from Approach - I is shown in Figure 4.23 whereas from Approach - II is shown in Figure 4.24 for all soil types as mentioned in this study. The comparison for both the approaches are shown in Fig 34 to Fig 36 of appendix C.

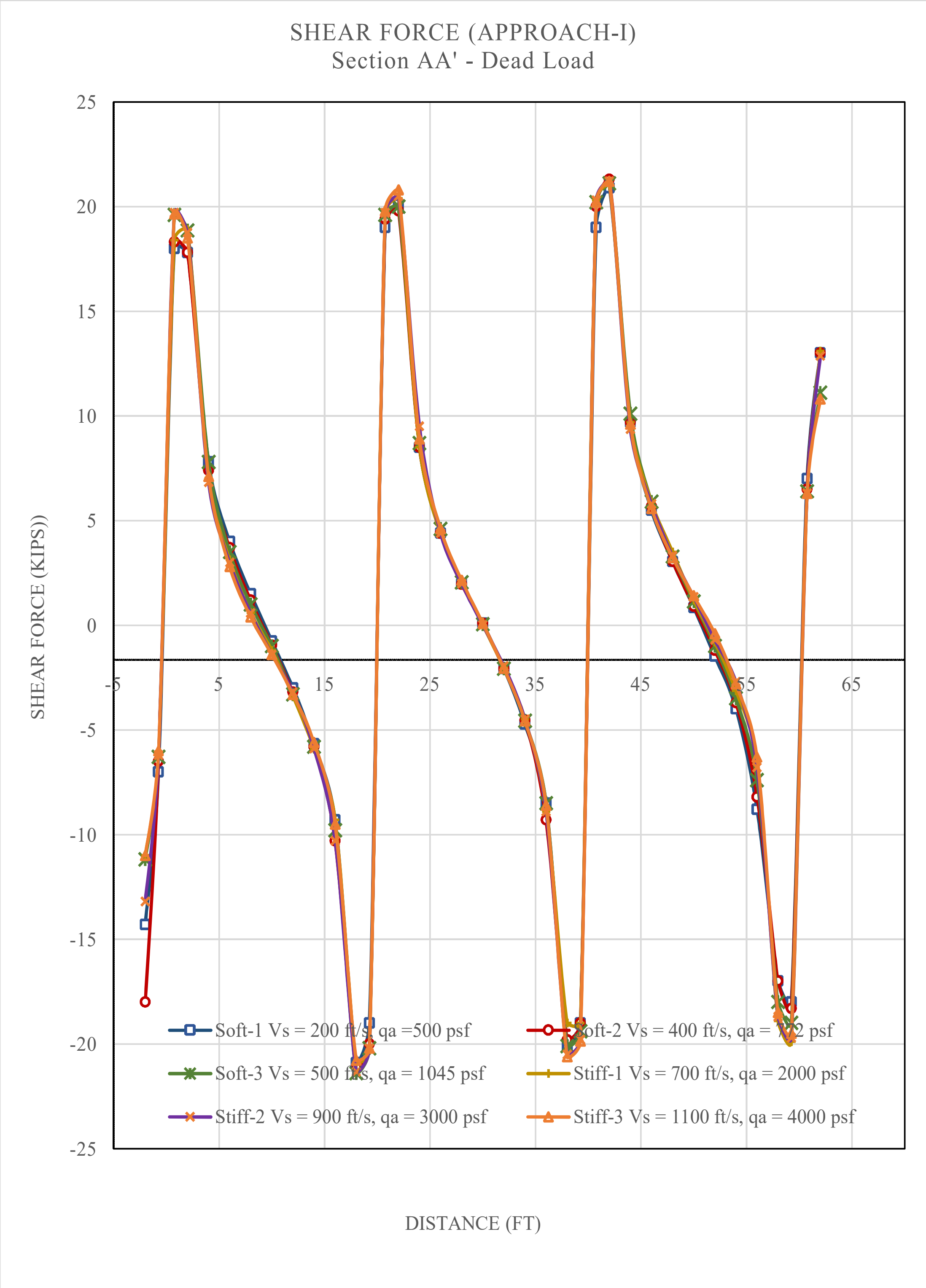


FIGURE 4.21: Shear Force Profile of Approach - I at Section AA' for dead load case

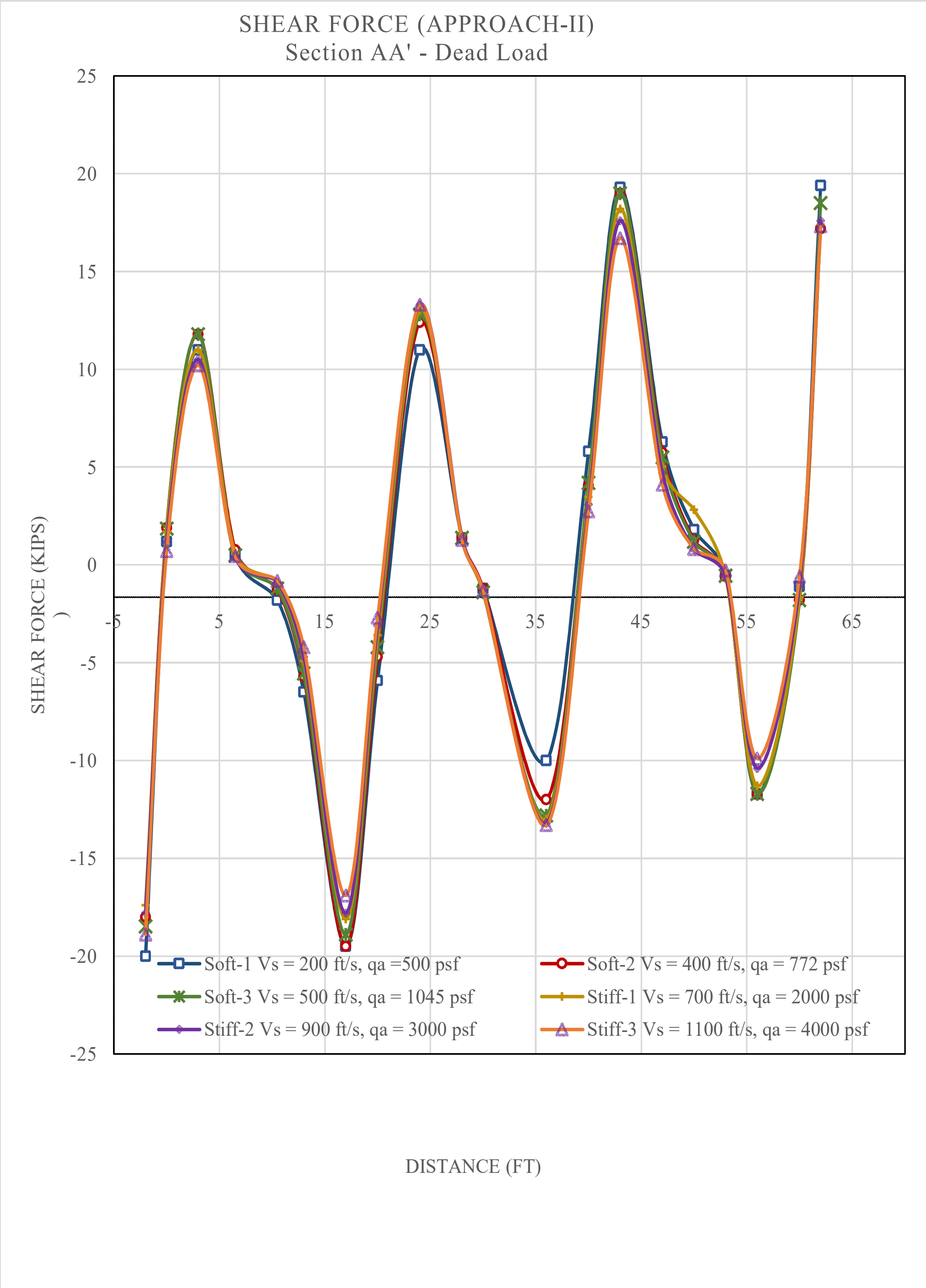


FIGURE 4.22: Shear Force Profile of Approach - II at Section AA' for dead load case

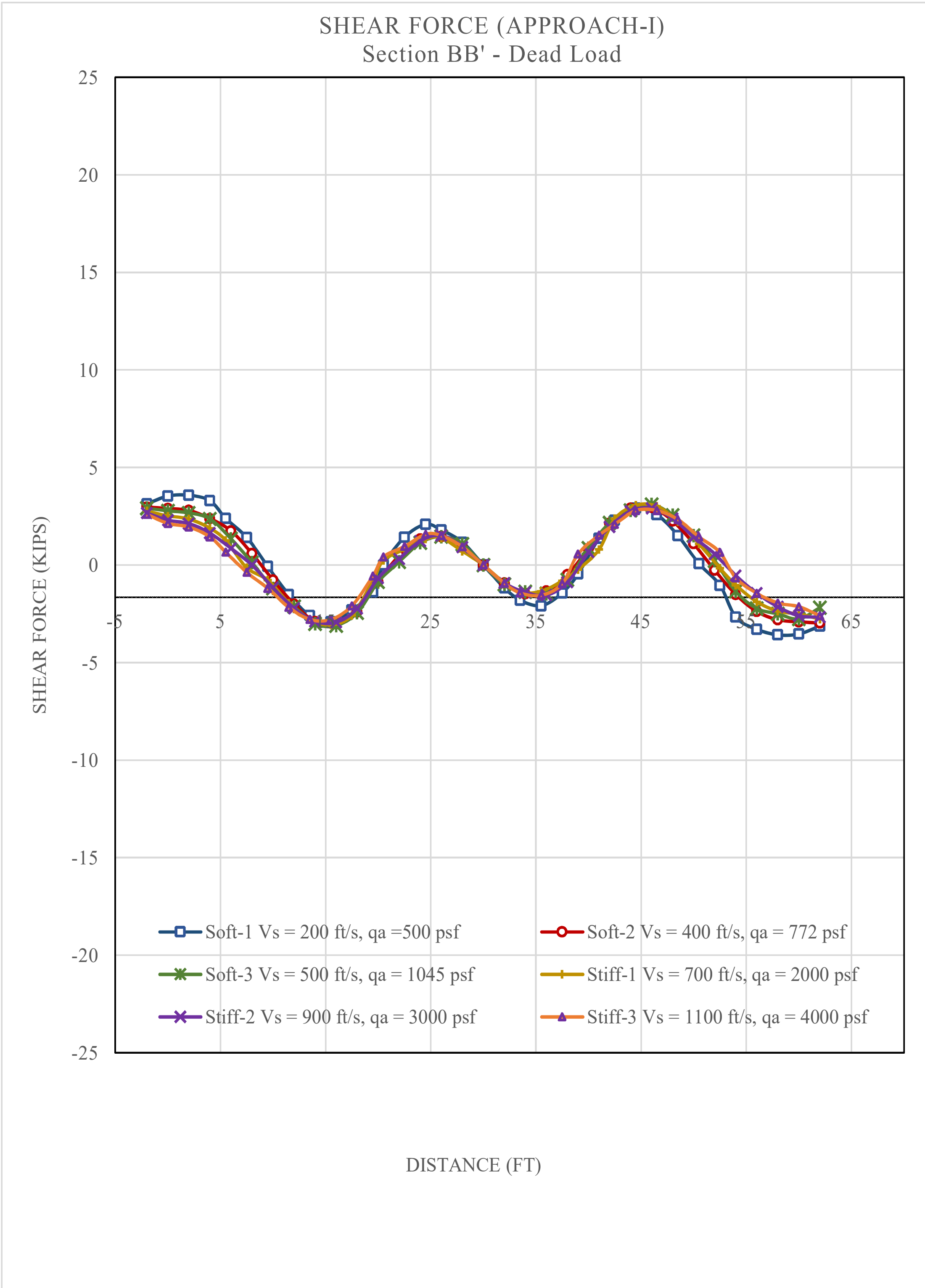


FIGURE 4.23: Shear Force Profile of Approach - I at Section BB' for dead load case

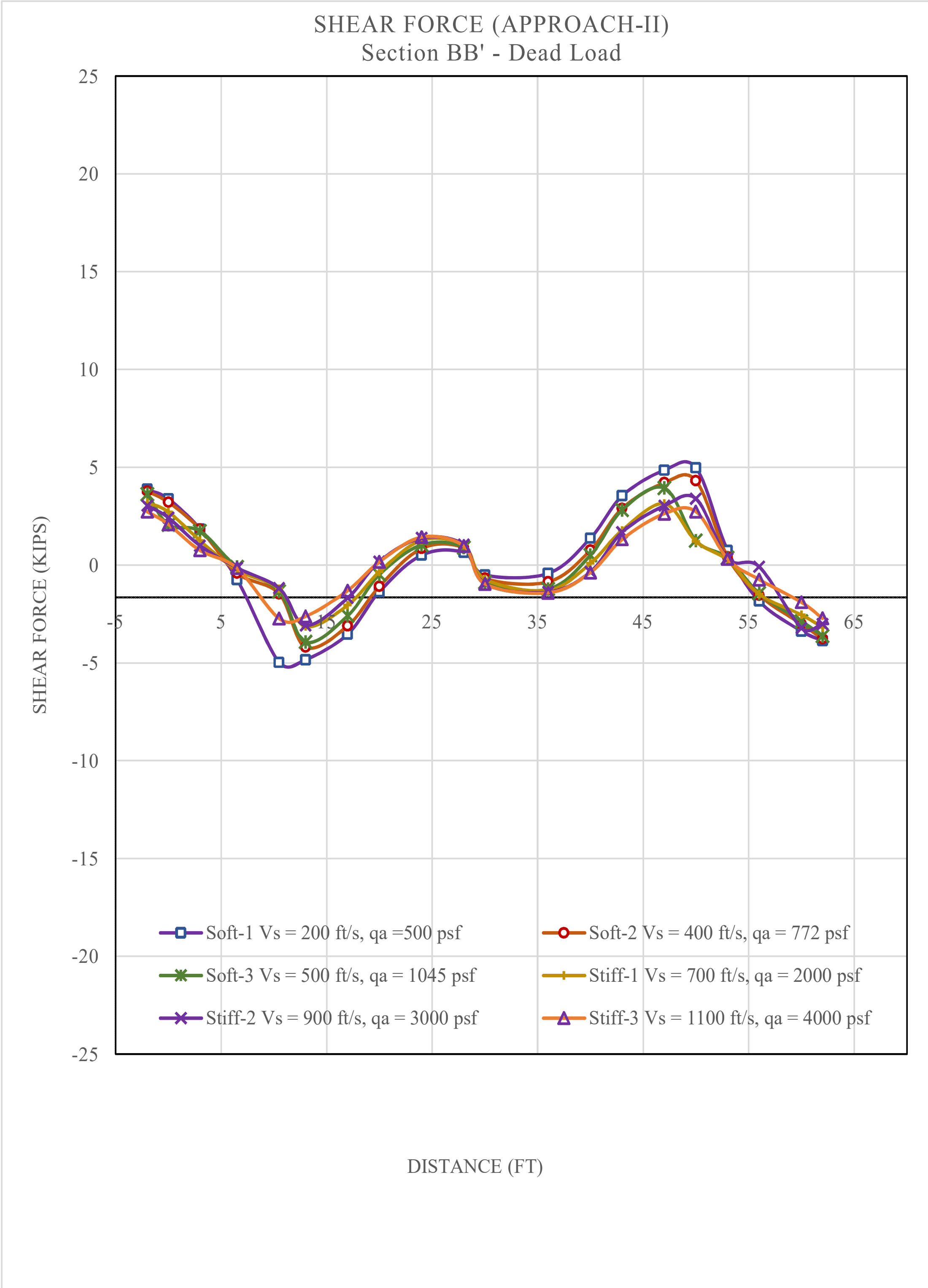


FIGURE 4.24: Shear Force Profile of Approach - II at Section BB' for dead load case

Shear force values calculated from Approach - I at column line for lateral load case for three different soil consistencies are shown in Fig 4.25.The shear values are calculated for three different soil consistencies i.e. Soft soil 1,Stiff Soil 1 and Stiff Soil 3.Similarly, Approach - II at column line for different soil consistencies are shown in Fig 4.26. The comparison of shear force calculated from Approach - I and Approach - II at column line are shown in Fig 37 to Fig 39 of appendix C. In case of Approach - I, greater values of shear force at some location are obtained as compared to Approach - II.

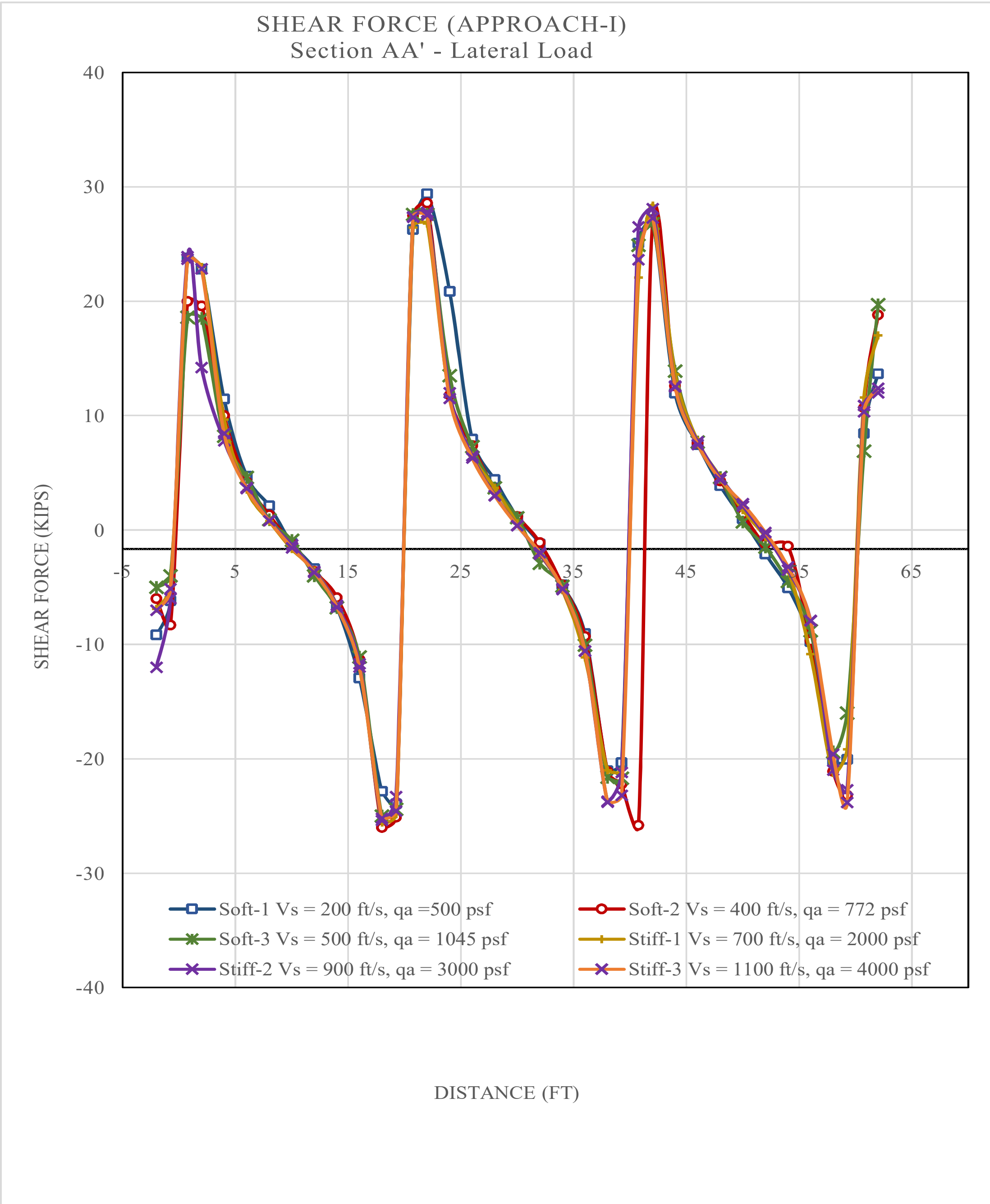


FIGURE 4.25: Comparison of Shear Force Profile of Approach - I at Section AA' for lateral load case

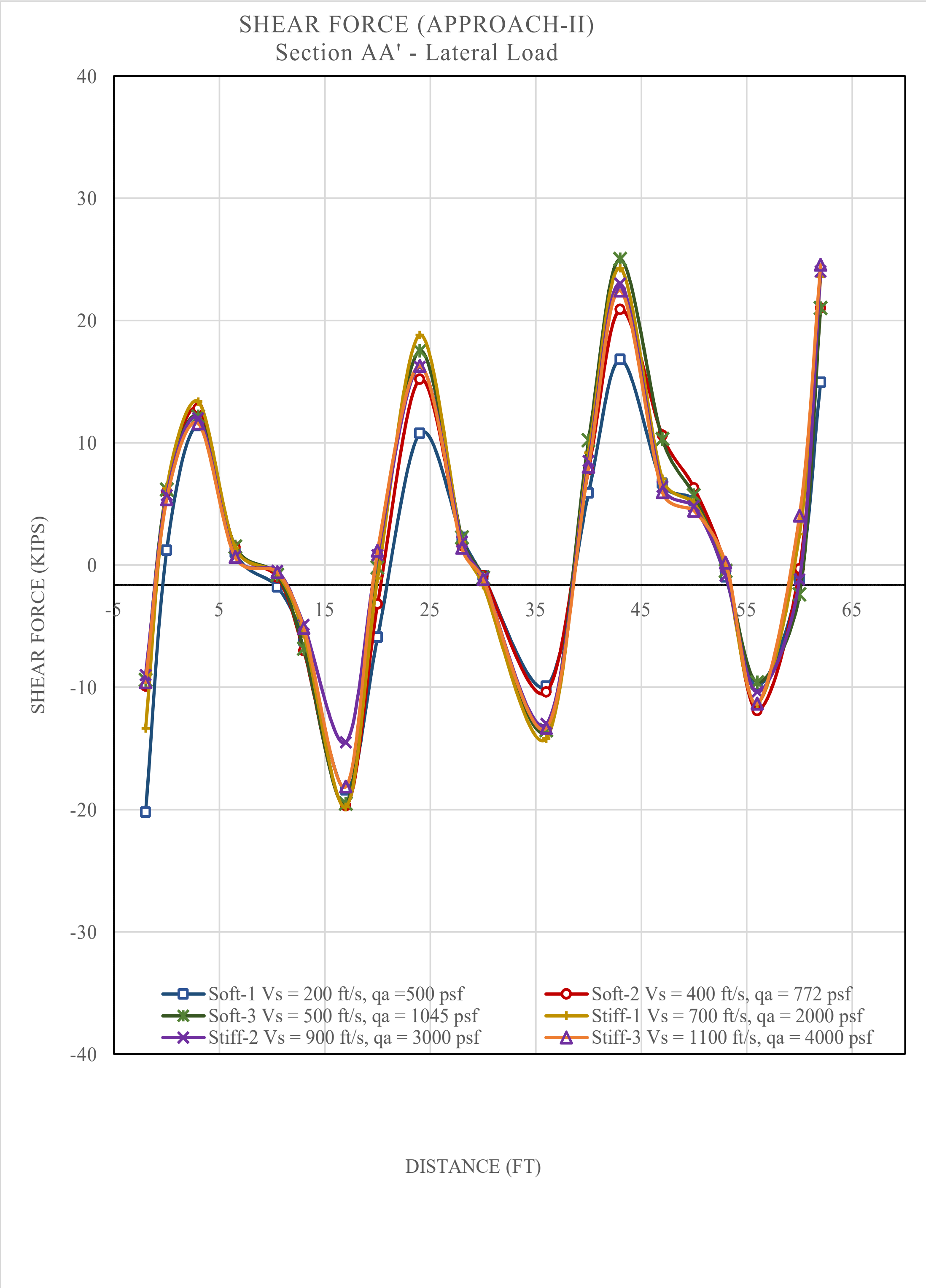


FIGURE 4.26: Comparison of Shear Force Profile of Approach - II at Section AA' for lateral load case

Shear force values calculated from Approach - I at centre for lateral load case for three different soil consistencies are shown in Fig 4.27. The Shear Force values are calculated for three different soil consistencies i.e. Soft soil1,Stiff Soil1 and Stiff Soil3. Similarly, Approach - II at centre line for different soil consistencies are shown in Fig 4.27. The comparison of shear force calculated from Approach - I and Approach - II at centre line are shown in Fig 40 to Fig 42 of appendix C.

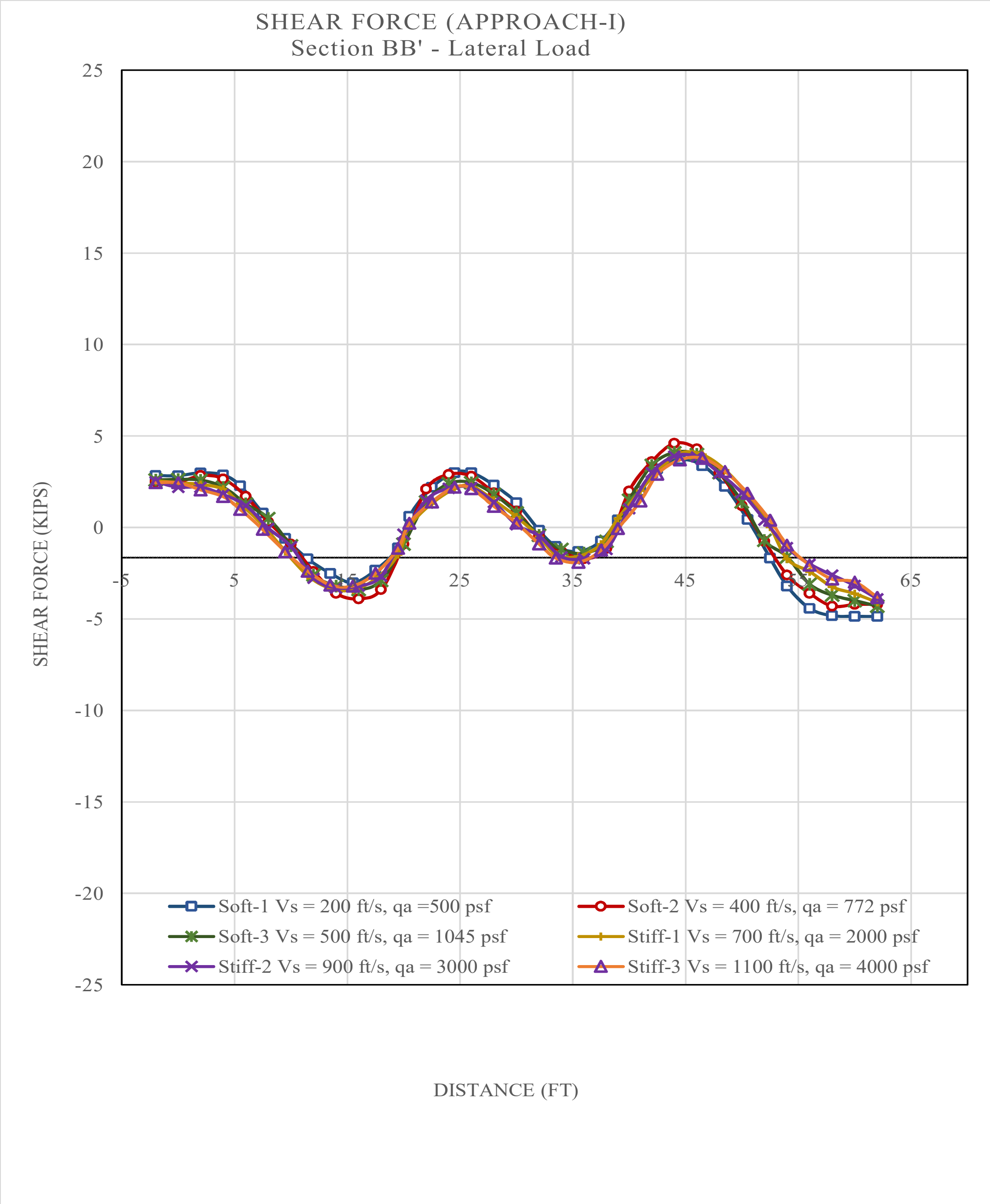


FIGURE 4.27: Comparison of Shear Force Profile of Approach - I at Section BB' for lateral load case

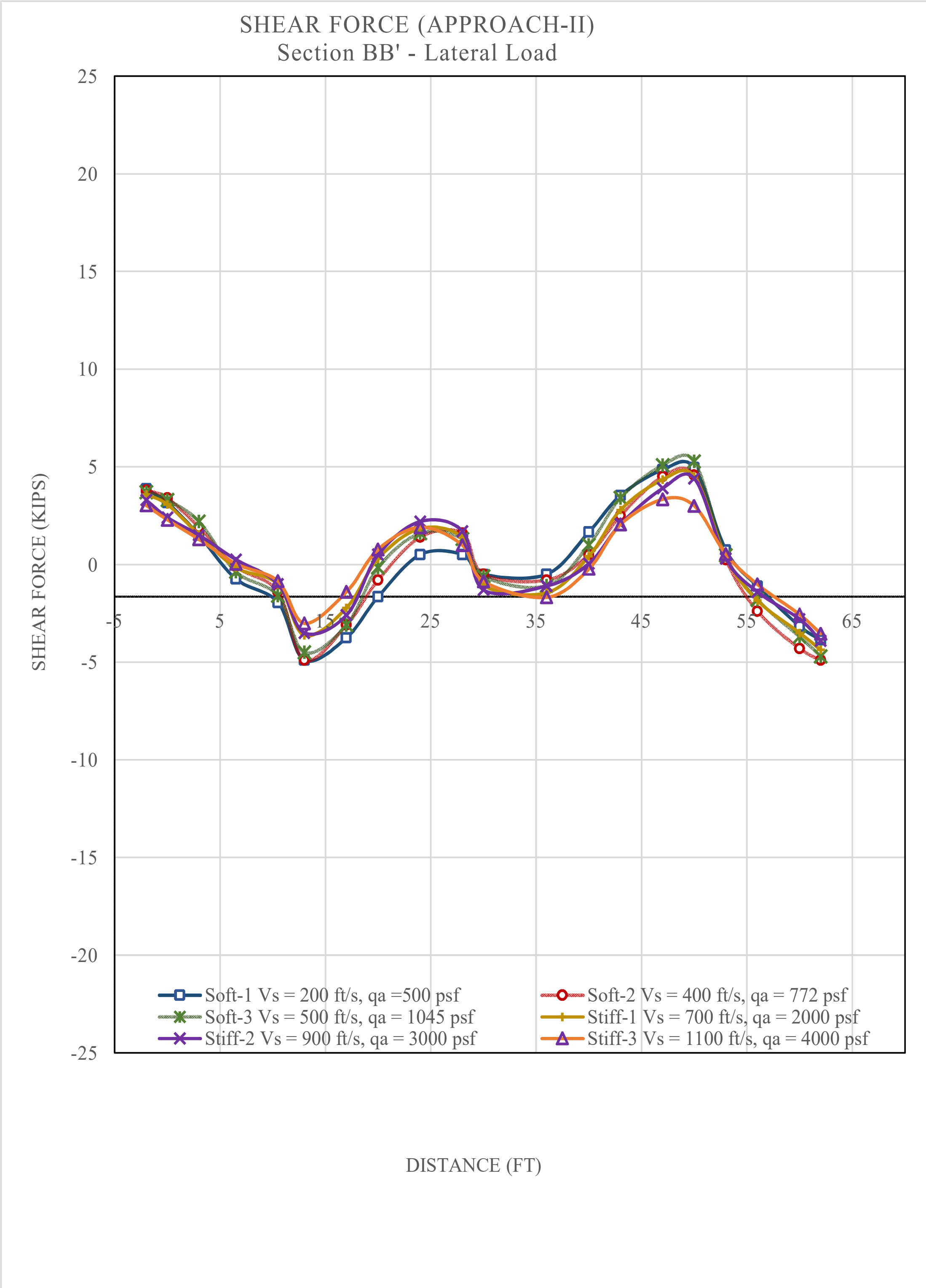


Figure 4.28: Comparison of Shear Force Profile of Approach - II at Section BB' for lateral load case

4.6 Effect of SSI on Super-Structure

In this section, the results of axial forces and bending moments against gravity loads, while in addition to axial force and bending moments, the shear force is also compared in lateral loads for Approach - I and Approach - II at base of the columns are discussed. Figure 4.29 and figure 4.30 shows the typical elevation of case study building with values of axial force and bending moments in superstructure respectively.

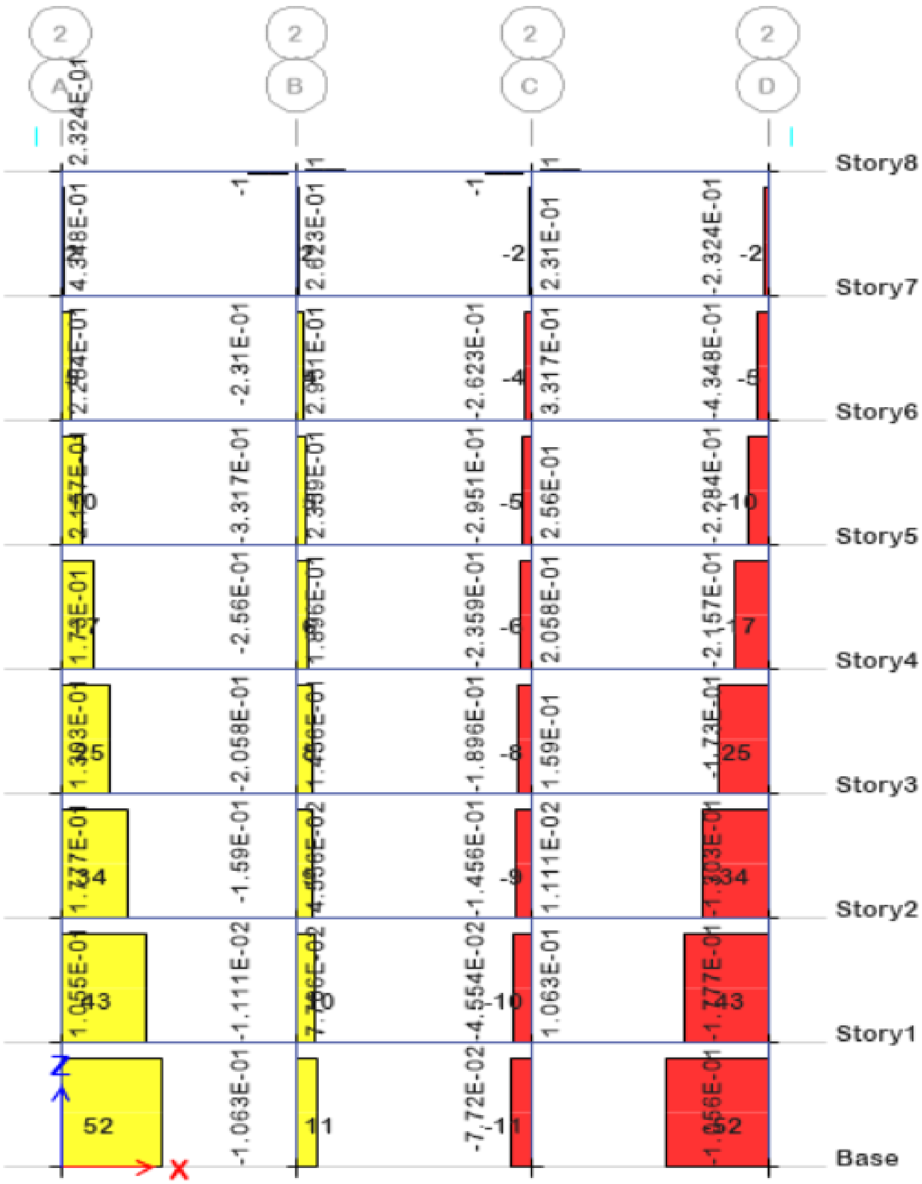


FIGURE 4.29: Typical Elevation of case study building showing the axial forces in superstructure

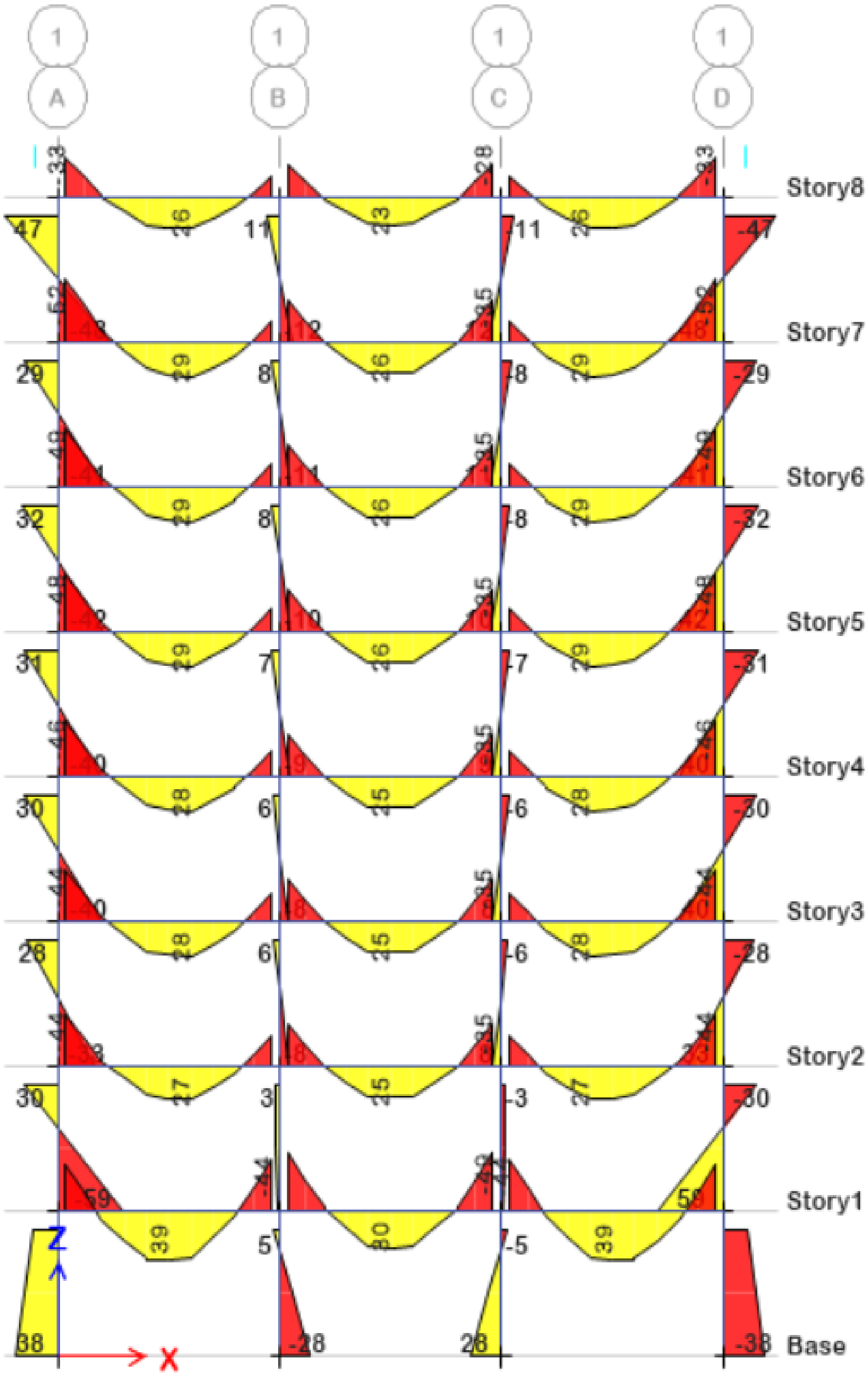


FIGURE 4.30: Typical Elevation of case study building showing the bending moments in superstructure

Figure 4.31 illustrates the results of axial forces and bending moments in Approach - I and Approach - II using gravity loads on corner column and interior column of exterior column line. The axial force on corner columns in Approach - I is calculated as 237 kips for all soil types, whereas, using Approach - II incorporating the SSI effects, the axial force in corner column increased from 237 kips to 255 kips, 240 kips, 238 kips and 240 kips in soil soft 1, soft 2, soft 3 and stiff 3 respectively, while at interior column, axial force in Approach - I is calculated as 402 kips for all soil types, whereas, using Approach - II incorporating the SSI effects, the axial force is increased from 402 kips to 436 kips, 426 kips and 423 kips in soil soft 1, soft 2 and soft 3 respectively. However, in stiff soils, the values of axial force is decreased. These values show that there is no significant increase in axial forces in raft foundation because of foundation dead load. However, a significant variations are observed in bending moments. It can be seen that, at corner column, using Approach-I, the bending moment is 16 k-ft, while in Approach - II, incorporating SSI effects, using Approach - I, the bending moment is 16 k-ft, while in Approach - II, incorporating SSI effects, the bending moment increased from 16 k-ft to 38 k-ft, 47 k-ft, 46 k-ft, 43.2 k-ft, 35.7 k-ft and 31.8 k-ft for soil soft 1, soft 2, soft 3, stiff 1, stiff 2 and stiff 3 respectively. The analysis results of Approach-I underestimates the values of bending moments which shows that ignoring the SSI effect for structural analysis may not assure structural safety of regular mid-rise moments resisting frames placed on soft soils.

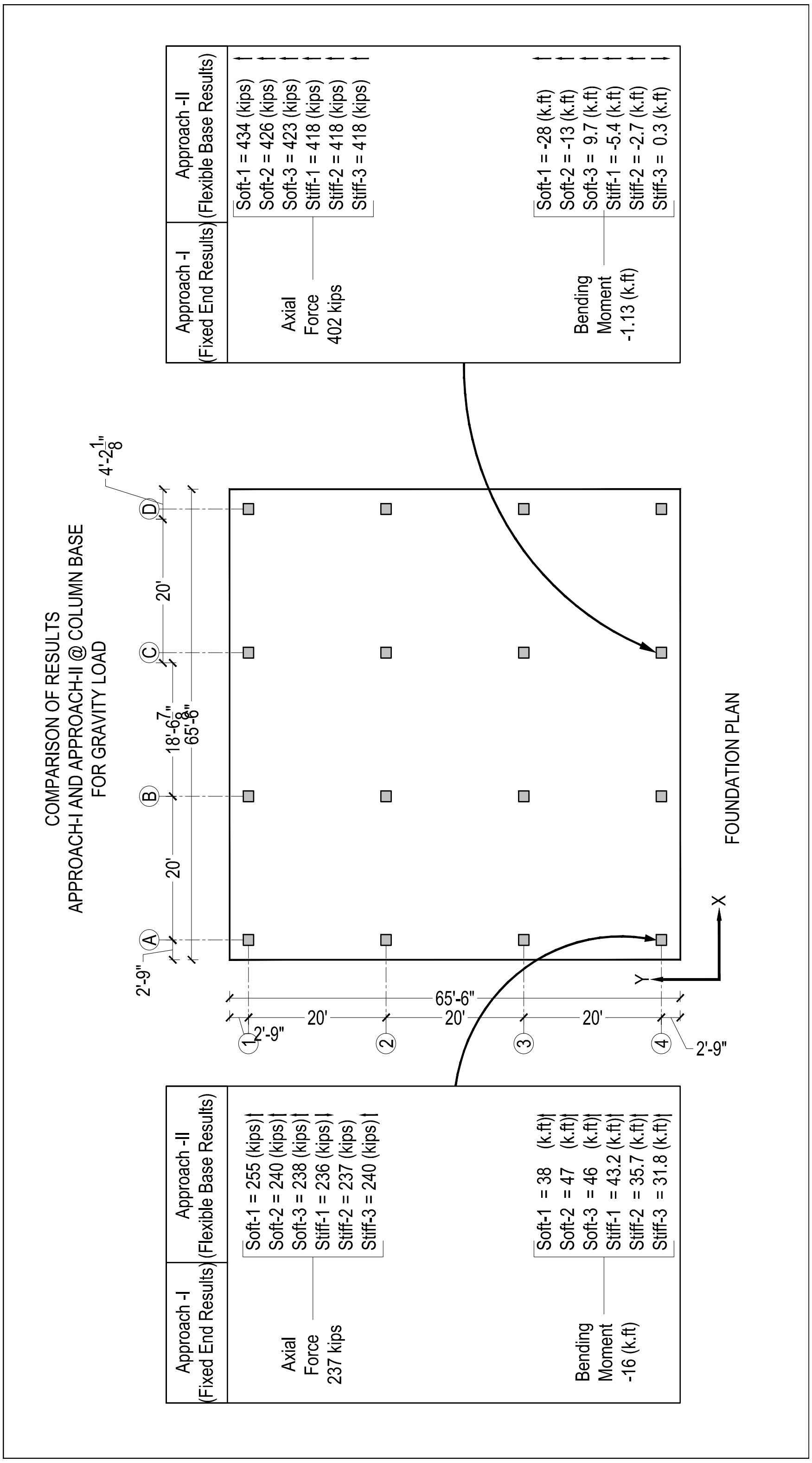


FIGURE 4.31: Comparison of Results of Axial forces and Bending moments at column base for exterior column line against gravity loads

Similarly, figure 4.32 illustrates the results of axial forces and bending moments in Approach - I and Approach - II using gravity loads on interior column line. The axial force on exterior column in Approach - I is calculated as 402 kips for all soil types, whereas, using Approach - II incorporation the SSI effects, the axial force in exterior column increased from 402 kips to 436 kips, 426 kips and 423 kips in soil soft 1, soft 2 and soft 3 respectively. Furthermore, in Approach - II, the values of axial force in interior column is decreased in soft soils from 696 kips to 652 kips, as compared to the Approach - I. However, a significant variations are observed in bending moments. It can be seen that, using Approach - I, the bending moment is 16 k-ft, while in Approach - II, incorporating SSI effects, the bending moment increased from 1.92 k-ft to 61 k-ft, 36 k-ft, 29 k-ft, 17 k-ft, 13 k-ft and 4.28 k-ft for soil soft 1, soft 2, soft 3, stiff 1, stiff 2 and stiff 3 respectively. This is due to the base flexibility. The stiff soils predict the rigid behavior as compared to the soft soils due to which the lesser values of bending moments are obtained, while soft soils shows the flexible base behavior.

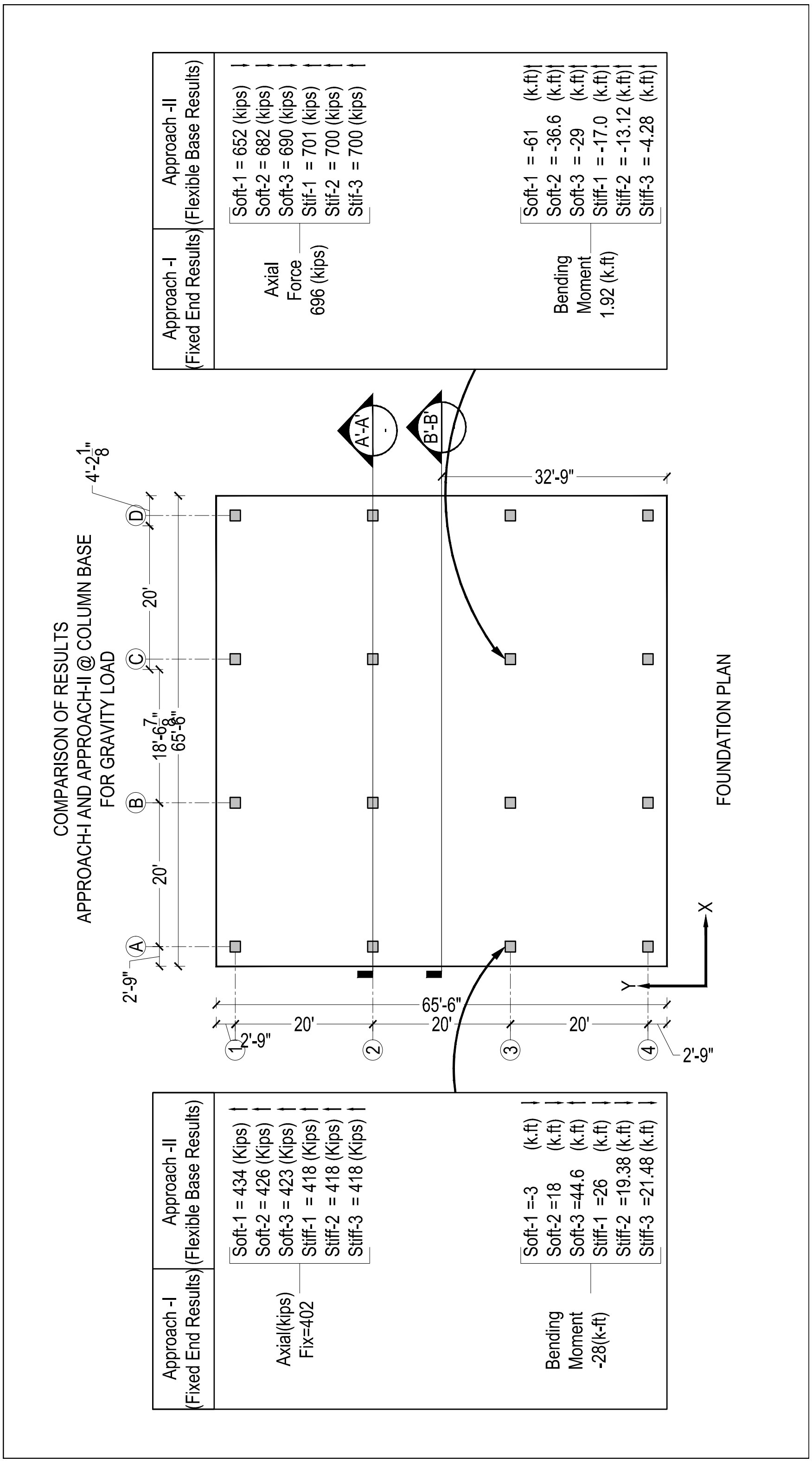


FIGURE 4.32: Comparison of Results of Axial forces and Bending moments at column base for interior column line against gravity loads

Figure 4.33 illustrates the results of axial forces and bending moments in Approach - I and Approach - II using lateral loads on corner column and interior column of exterior column line.

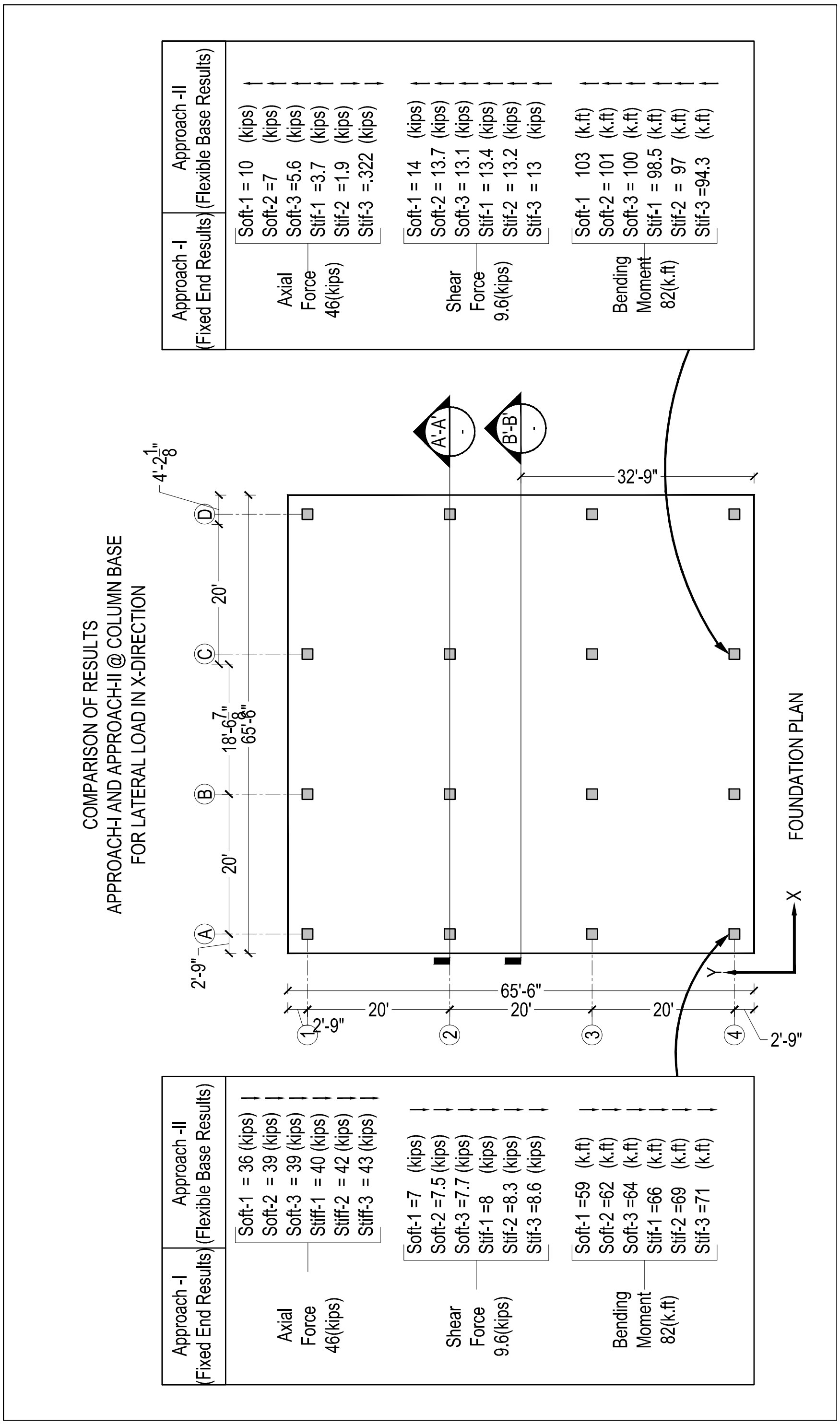


FIGURE 4.33: Comparison of Results of Axial forces and Bending moments at column base for interior column line against lateral loads

Similarly, Figure 4.34 illustrates the results of axial forces and bending moments in Approach - I and Approach - II using lateral loads on interior column line.

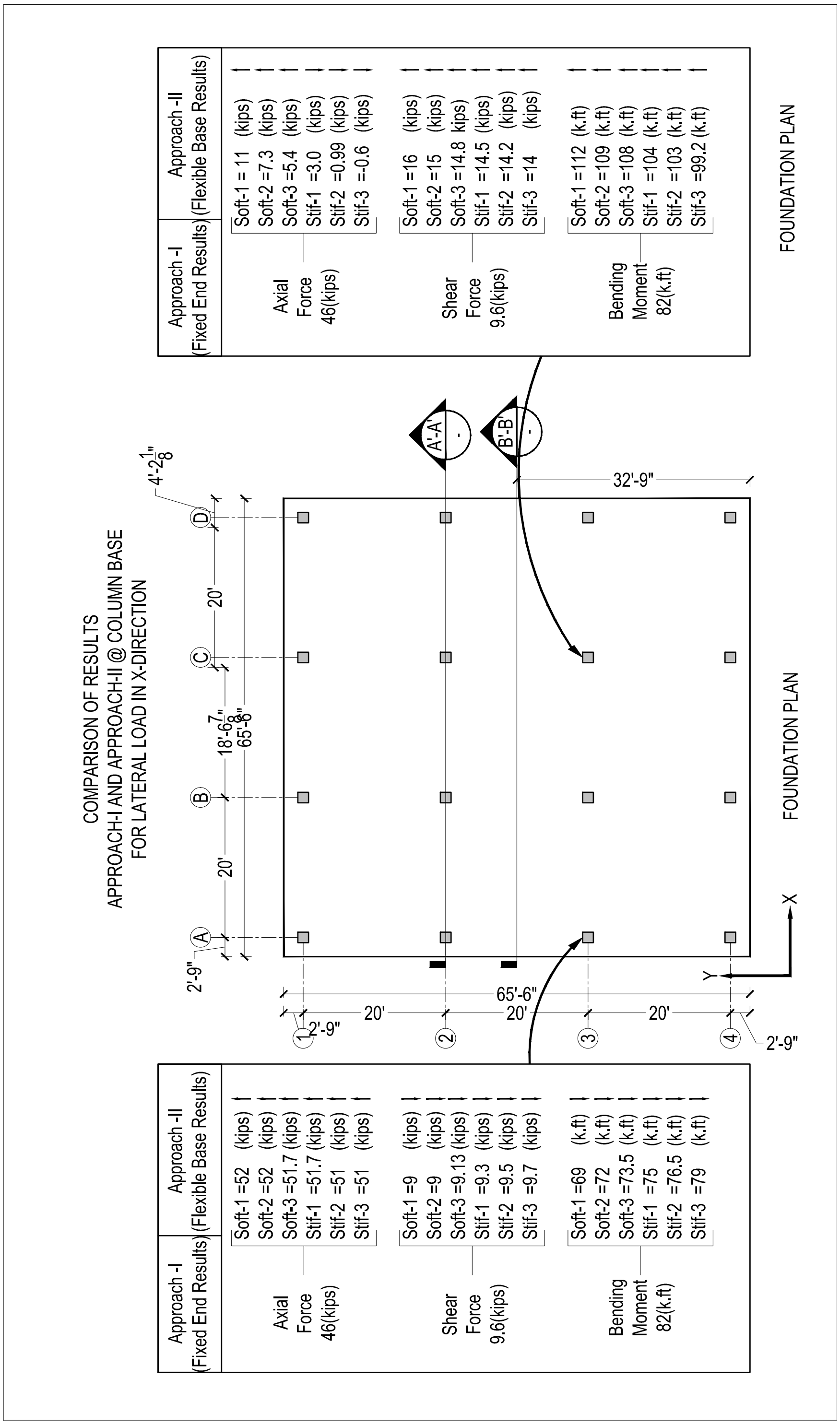


FIGURE 4.34: Comparison of Results of Axial forces and Bending moments at column base for interior column line against lateral loads

Chapter 5

Conclusions And Recommendations

5.1 Summary

The importance of soil structure interaction (SSI) for design of substructure has been discussed and the related literature review to use soil structure interaction principles in solution of raft foundation design has been presented. Since 1990s, great effort has been made for substituting the conventional method of deign of raft foundation by the new approaches of design incorporating SSI .In conventional/current design practice, referred here as Approach-I, is still being used by practising engineers now a days for raft foundation design of midrise moment resisting frame building and is based on two stage analysis. In stage-1, building is assumed to be fixed at the ground level and no SSI effects are considered. After analysis reactions are calculated with fixed end condition at the grade. These results are transferred to a different model for foundation design. Where subgrade soil is modeled according to Winkler's hypothesis with constant value of modulus of subgrade reaction. This approach ignores the SSI effects. Whereas, Approach-II in this study is based on the guidelines provided in a report published by National Institute of Standards and Technology titled as NIST GCR 12-917-21 "Soil-Structure Interaction of Building Structures". Using the table 2.2a by Pais and Kausel (1988) (NIST GCR 12-917-21), modulus of subgrade reaction was calculated for all degrees of freedom to incorporate the effects of SSI.

The case studied model is 8 story (G+7) moment resisting frame structure with raft foundation. The building plan is symmetrical along both x-axis and y-axis with length and width of 64' and three equal bays in both directions. Beams are placed on all grids typical height of each story is taken as 12 ft. The six different types of fine grained soils are used to determine SSI effects on foundation design. These six different soils are based upon Uniform Building Code (1997) Table 16-J. Soft soil with Vs $<$ 600 ft./sec and Stiff soil with 600 ft./sec $\leq$ Vs $\leq$ 1,200 ft./sec. . Soft soil are classified further into three category i.e. Soft soil 1,Soft soil 2 & Soft soil 3 selecting different values of shear wave velocity for each soil. Similarly Stiff soil are also further divided into three types i.e. Stiff soil 1, Stiff soil 2 & Stiff soil 3. This study endeavour to assess current design practice of raft foundation and compares the results with analysis performed considering soil structure interaction principles.

Different critical parameters for raft foundation design i.e. raft settlement profile, bending moment and shear force calculated as per Approach-I and Approach-II have been compared under dead load and lateral load conditions. A geotechnical elastic settlement analysis was also performed to determine shape of settlement profile which is one of most important parameter of raft foundation design.

5.2 Conclusions:

Based on the comparative study done in this report on current design practice of raft foundation referred as Approach-I with the design procedure provided in the NIST, N. (2012) considering SSI principles, following conclusions are made:

- The design based on Approach-II incorporating SSI is highly desirable from consideration of computational accuracy and feasible from practical point of view. This can easily be seen by comparing the results from approach-II and elastic settlement analysis as shown in table 4.1.
- The adopted distributed vertical springs have been adjusted near edges (as implemented in Approach-II) should be used as these tend to deform foundation in realistic manner.
- Settlement profile of Approach-II is quite close to elastic settlement analysis values, proving the accuracy of this approach.
- Implementation of inaccurate uniform spring constant can lead to major errors in raft foundation design moments.
- Significant saving in terms of reinforcement can be achieved in raft foundation design over soft soil as compared to stiff soil where SSI effects are not very much pronounced.

5.3 Future Recommendations

The aim of current study was to assess current design practice of raft foundation and compare the results with analysis performed considering soil foundation structure interaction. In this study, two soil types SD and SE with different soil consistencies have been considered. Future research work should be carried out in order to evaluate the different parameters of substructure by taking into account the kinematic effects along with the inertial effects. Furthermore, studies should be carried out to provide tangible insights into the benefits of SSI analysis for owners and practising engineers and to explore the benefits and limitations of SSI response history analysis procedures, possibly leading to improved procedures. Also future studies should be conducted to address fundamental limitations in the state of SSI knowledge, which limit the accuracy and reliability of SSI models available for use in engineering practice.

Bibliography

[1] Raychowdhury, P. (2008). Nonlinear winkler-based shallow foundation model for performance assessment of seismically loaded structures (Doctoral dissertation, UC San Diego).

[2] NIST, G. (2012). GCR 12-917-21 (2012) Soil-structure interaction for building structures. Gaithersburg, National Institute of Standards and Technology.

[3] Luco, J.E., and Westmann, R.A., 1971, "Dynamic response of circular footings," Journal of Engineering Mechanics, Vol. 97, No. 5, pp. 1381-1395.

[4] Veletsos, A.S., and Wei, Y.T., 1971, "Lateral and rocking vibrations of footings," Journal of Soil Mechanics and Foundations Division, Vol. 97, No. 9, pp. 1227-1248.

[5] Pais, A., & Kausel, E. (1988). Approximate formulas for dynamic stiffnesses of rigid foundations. Soil Dynamics and Earthquake Engineering, 7(4), 213-227.

[6] Harden, C.W., Hutchinson, T., Martin, G. R., and Kutter, B. L. (2005). "Numerical modeling of the nonlinear cyclic response of shallow foundations." Report No. 2005/04, Pacific Earthquake Engineering Research Center, PEER.

[7] Gazetas, G. (1991b). "Formulas and charts for impedances of surface and embedded foundations." Journal of Geotechnical Engineering, 117(9), 1363–1381.

[8] Ciampoli, M., and Pinto, P.E., 1995, "Effects of soil-structure interaction on inelastic seismic response of bridge piers," Journal of Structural Engineering, Vol. 121, No. 5, pp. 806-814.

[9] Mylonakis, G., and Gazetas, G., 2000, "Seismic soil-structure interaction: Beneficial or detrimental," Journal of Earthquake Engineering, Vol. 4, pp. 377-401.

[10] Perez-Rocha, L.E., and Aviles, J., 2003, "The evaluation of interaction effects in inflexible resistances," Revista de Ingenieria Sismica, No. 69 (in Spanish).

[11] Gazetas, G., 2006, "Seismic design of foundations and soil-structure interaction," Proceedings, First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland.

[12] Gajan, S., and Kutter, B.L., 2008, "Capacity, settlement, and energy dissipation of shallow footings subjected to rocking," Journal Geotechnical and Geoenvironmental Engineering, Vol. 134, No. 8, pp. 1129-1141.

[13] McClelland, B., and Focht, J.A., Jr., 1958, "Soil modulus for laterally loaded piles," Transactions, Vol. 123, pp. 1049-1063.

[14] Raychowdhury, P., and Hutchinson, T.C., 2009, "Performance evaluation of a nonlinear Winkler-based shallow foundation model using centrifuge test results," Earthquake Engineering and Structural Dynamics, Vol. 38, No. 5, pp. 679-698.

[15] Nova, R., and Montrasio, L., 1991, "Settlements of shallow foundations on sand," Geotechnique, Vol. 41, No. 2, pp. 243-256.

[16] Bowles, J. E. (1982). Foundation design and analysis.

[17] Ulrich, E.J., 1995. Introduction to the State-of-the-Art Mat Foundation Design and Construction. Special Publication, 152, pp.1-260.

Appendix - A Settlement Profiles

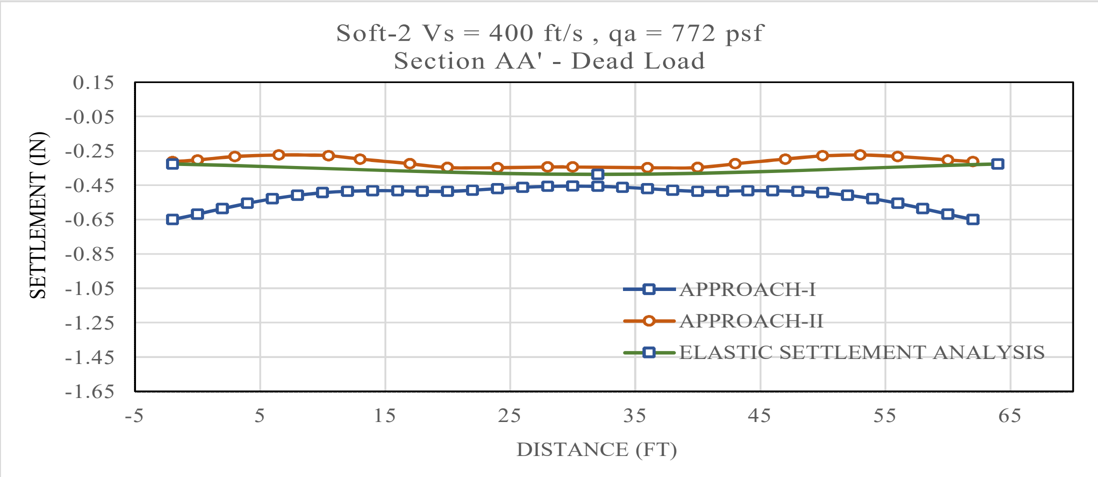


FIGURE 1: Comparison of Settlement Profile of Approach I and Approach II at Section AA' for dead load case

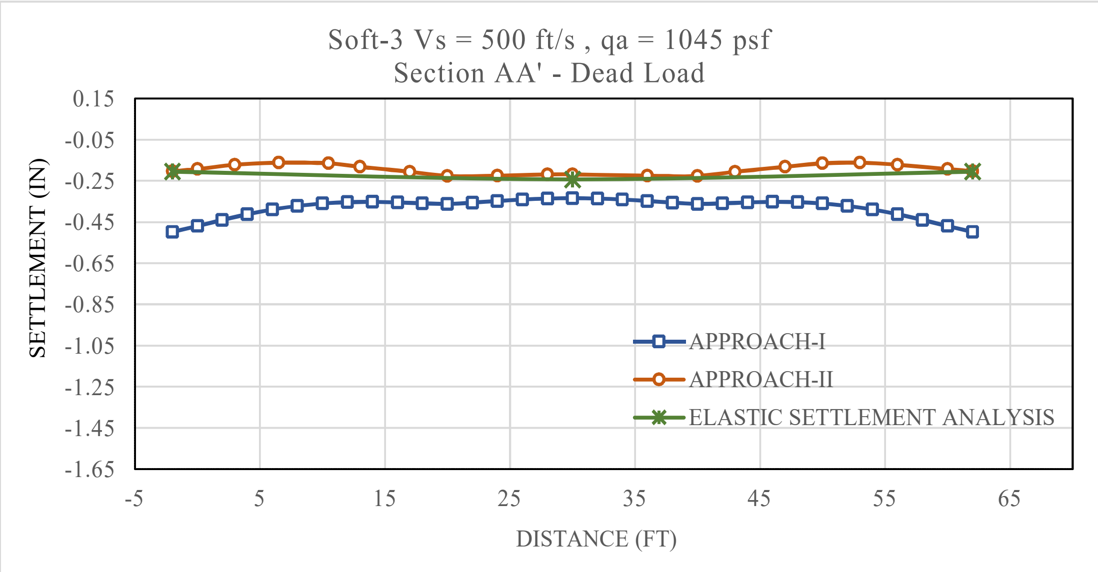


FIGURE 2: Comparison of Settlement Profile of Approach I and Approach II at Section AA' for dead load case

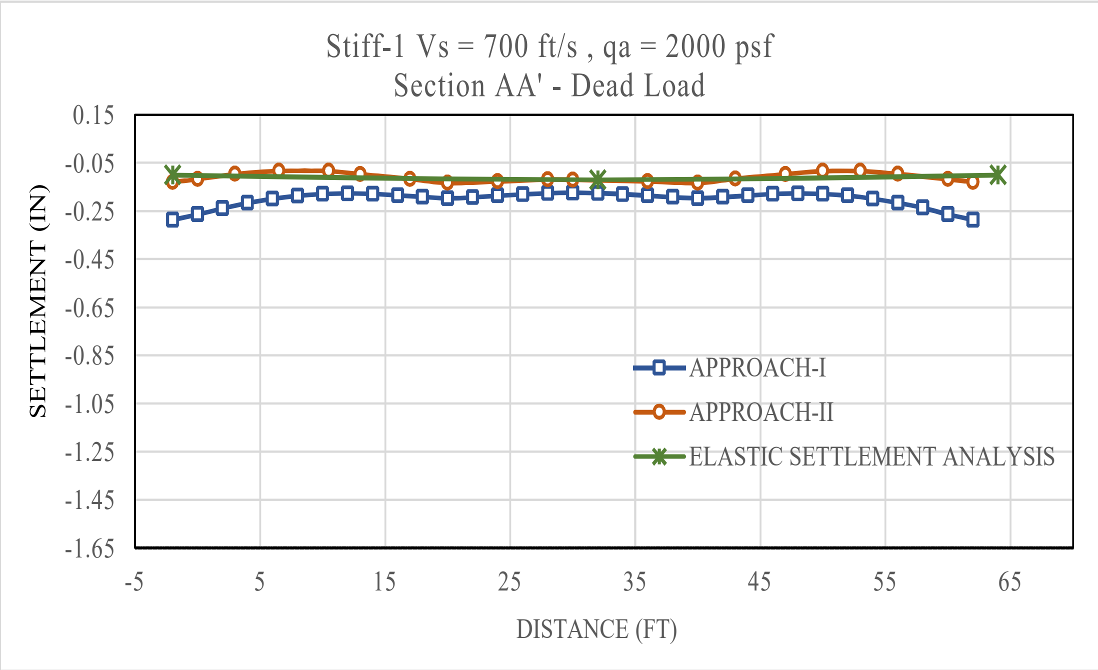


Figure 3: Comparison of Settlement Profile of Approach I and Approach II at Section AA for dead load case

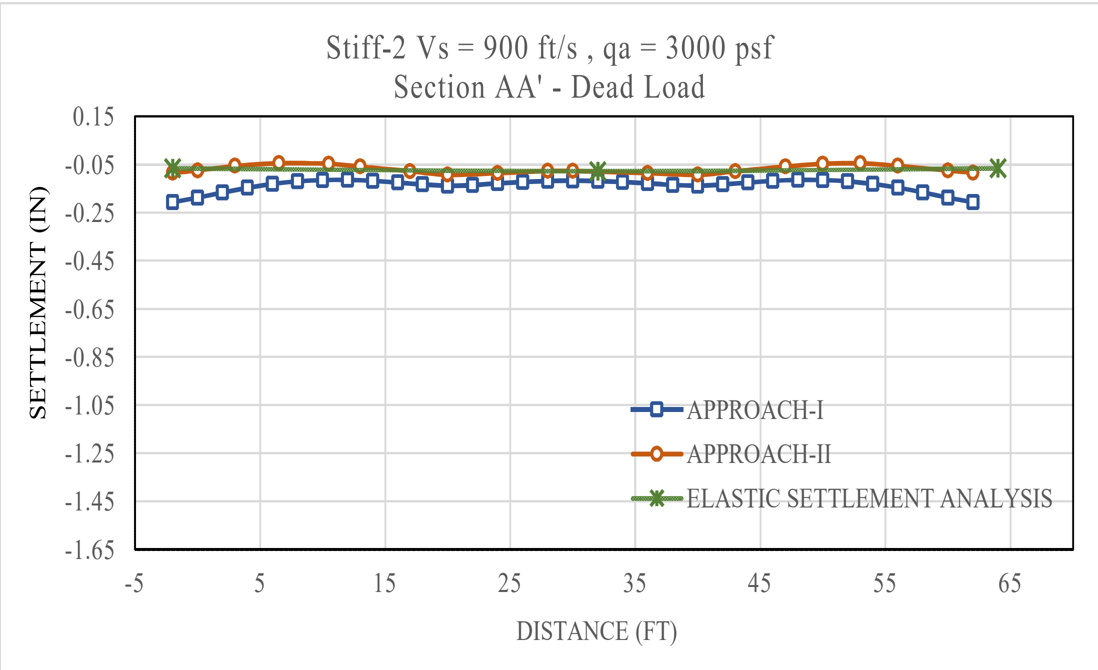


Figure 4: Comparison of Settlement Profile of Approach I and Approach II at Section AA’ for dead load case

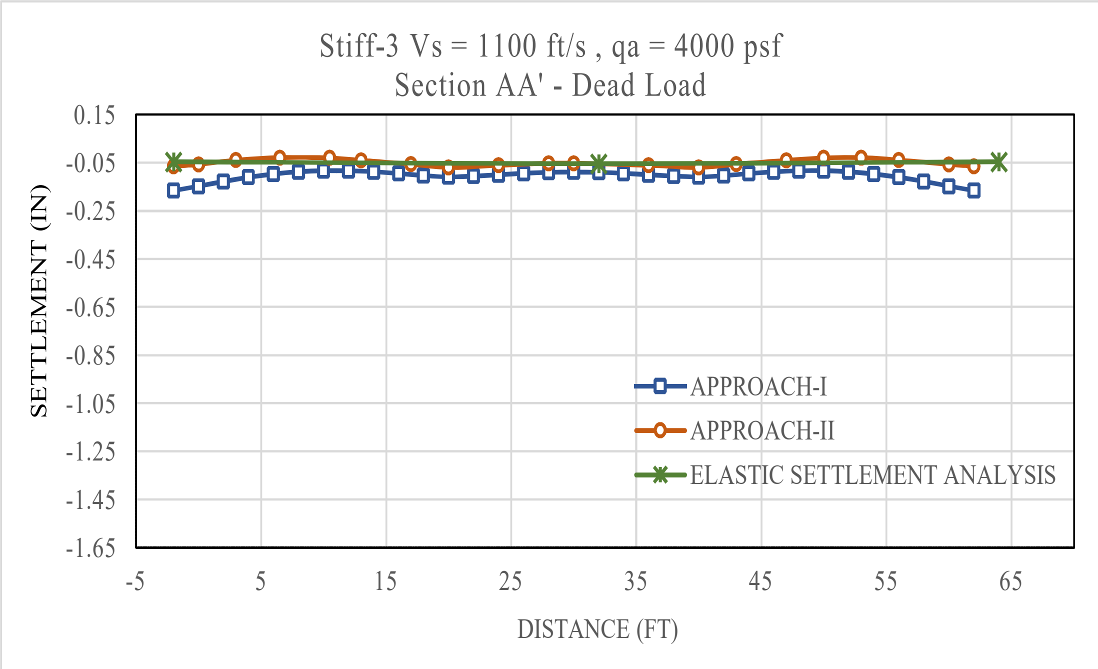


FIGURE 5: Comparison of Settlement Profile of Approach I and Approach II at Section AA' for dead load case

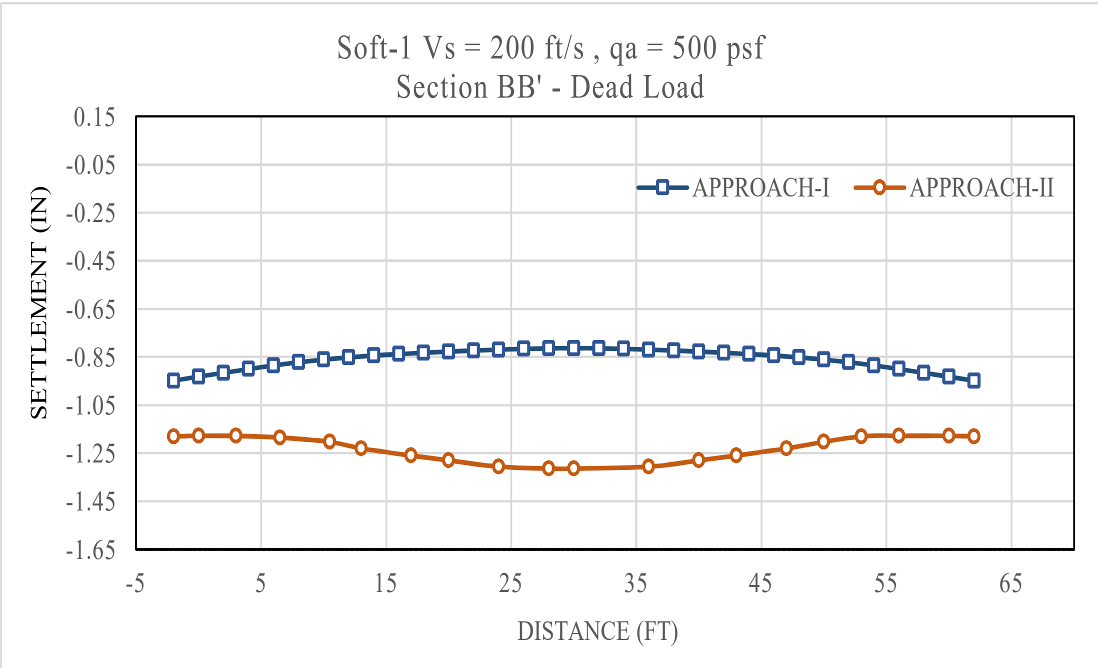


FIGURE 6: Comparison of Settlement Profile of Approach I and Approach II at Section BB' for dead load case

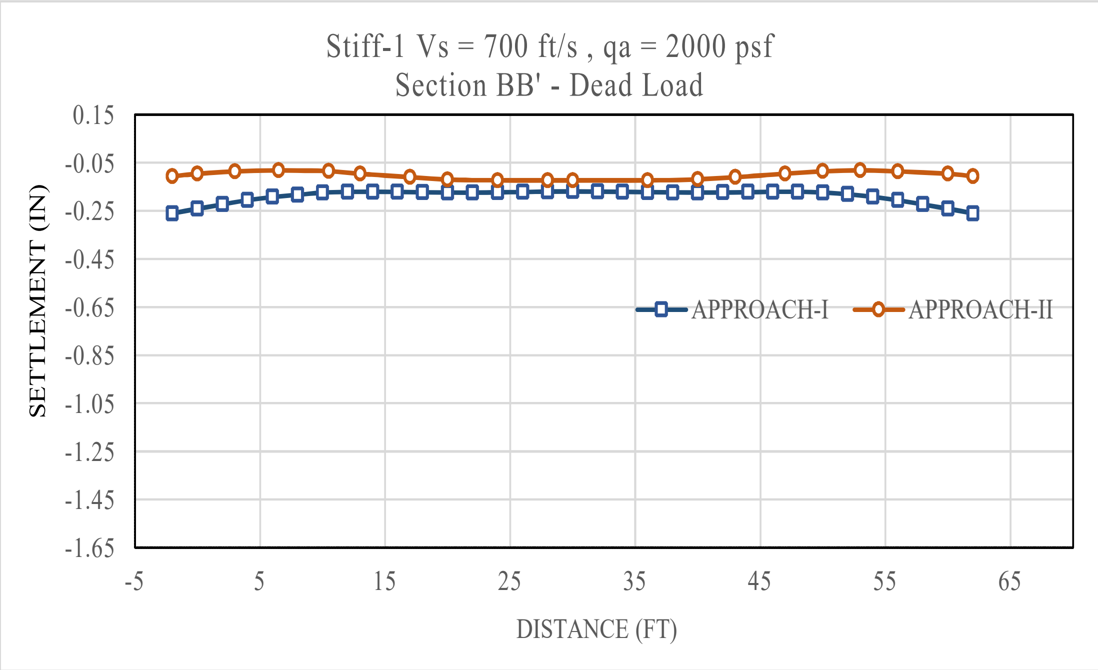


FIGURE 7: Comparison of Settlement Profile of Approach I and Approach II at Section BB' for dead load case

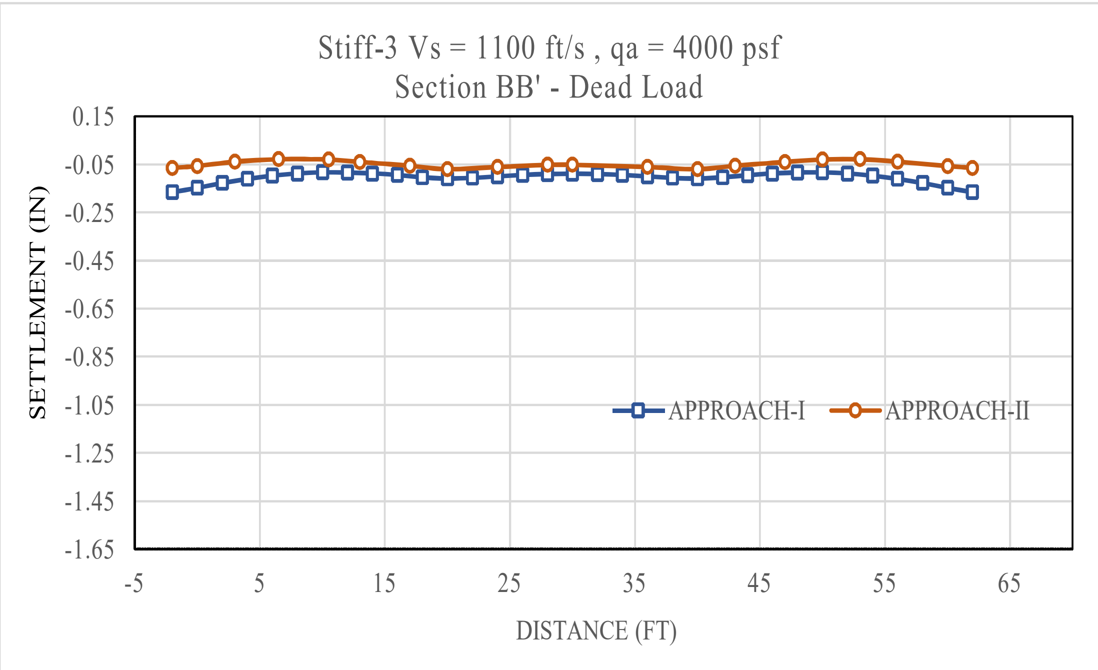


FIGURE 8: Comparison of Settlement Profile of Approach I and Approach II at Section BB' for dead load case

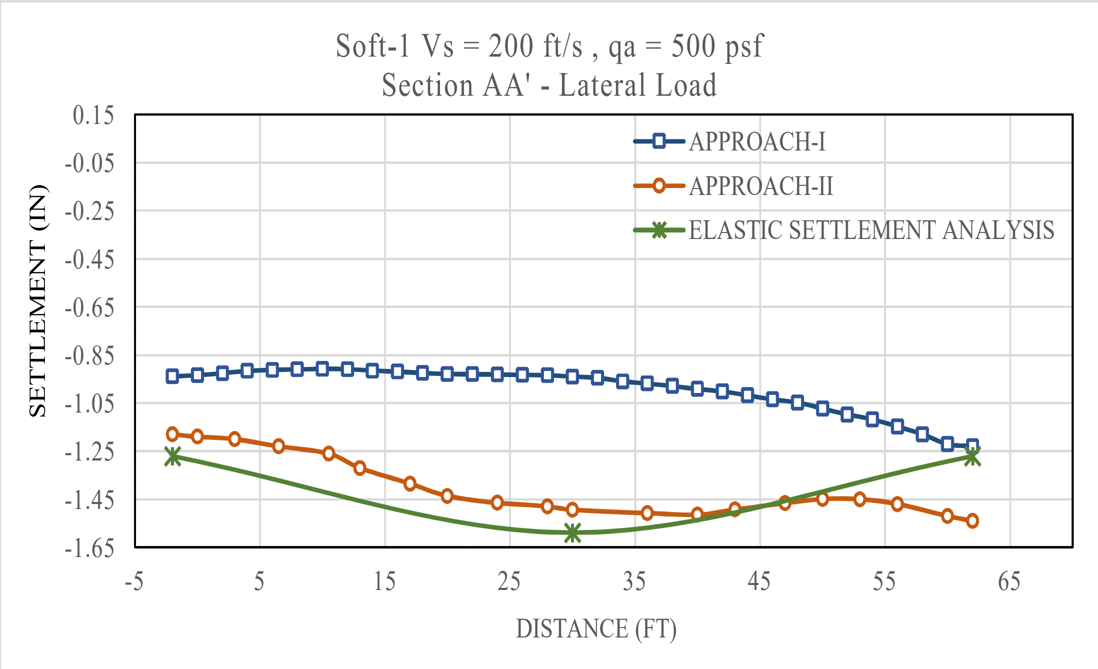


FIGURE 9: Comparison of Settlement Profile of Approach I and Approach II at Section AA' For Lateral Load Case

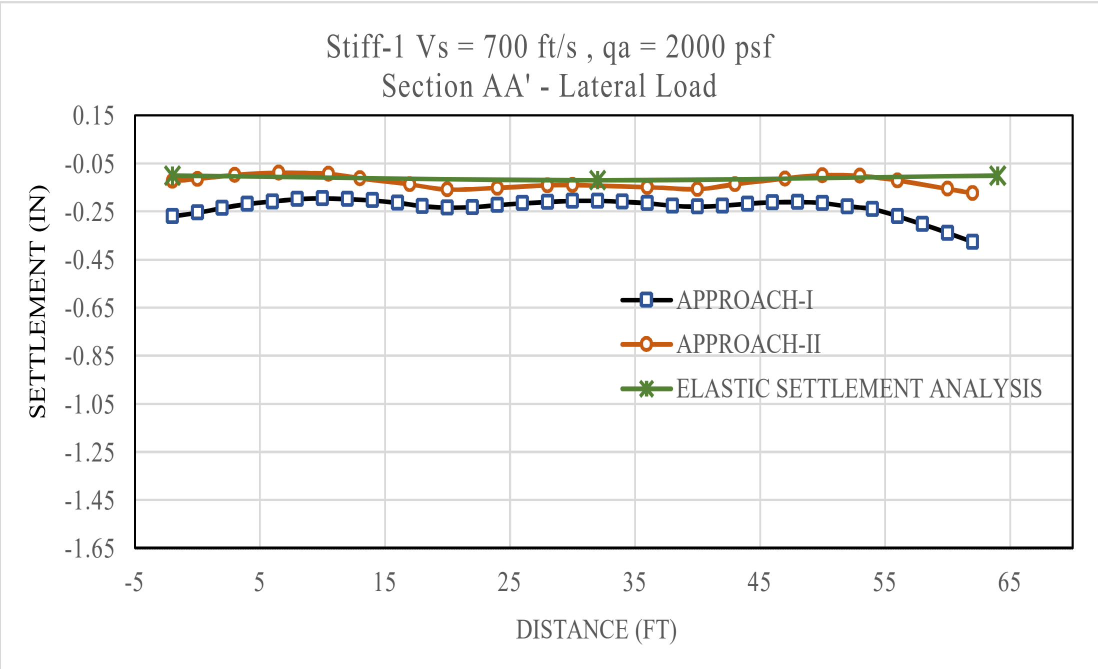


FIGURE 10: Comparison of Settlement Profile of Approach I and Approach II at Section AA' For Lateral Load Case

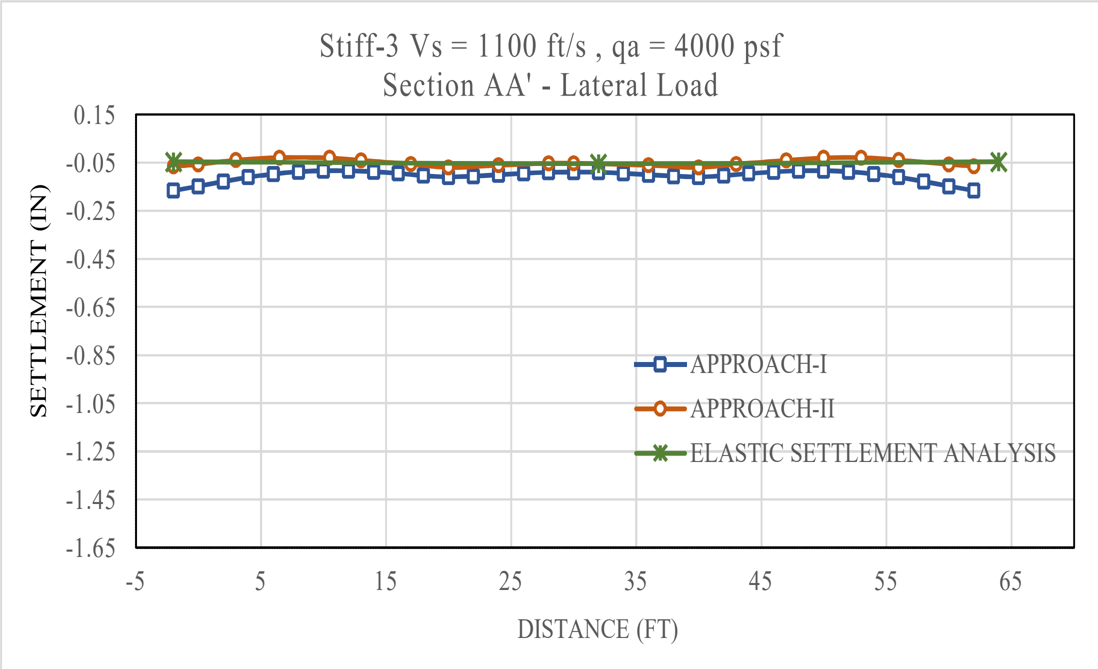


FIGURE 11: Comparison of Settlement Profile of Approach I and Approach II at Section AA' For Lateral Load Case

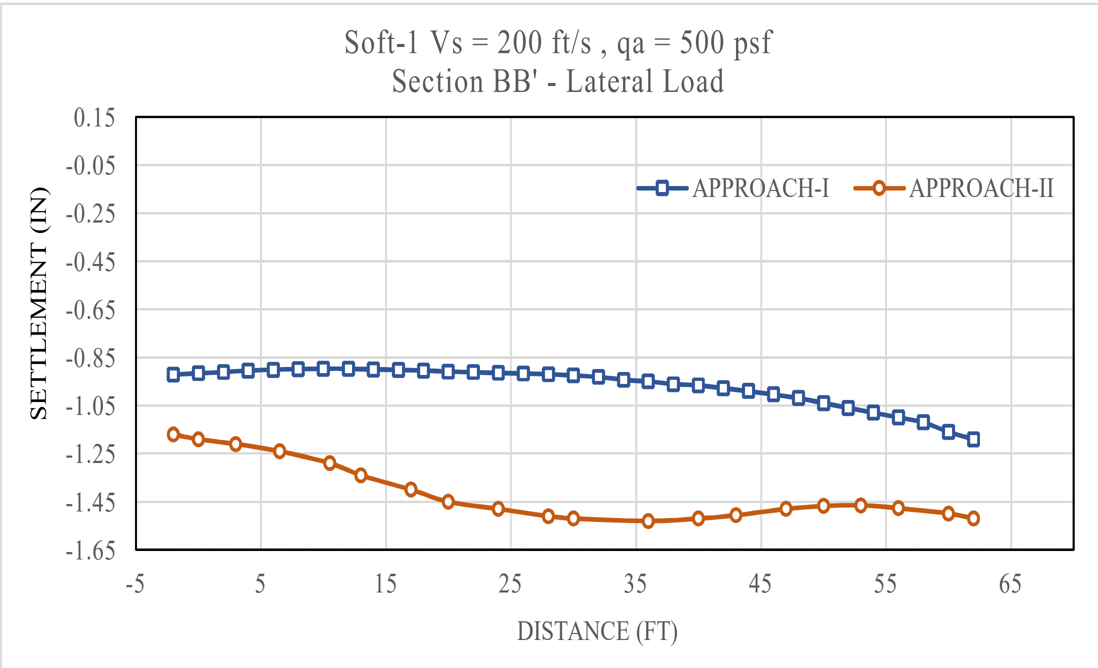


FIGURE 12: Comparison of Settlement Profile of Approach I and Approach II at Section BB' For Lateral Load Case

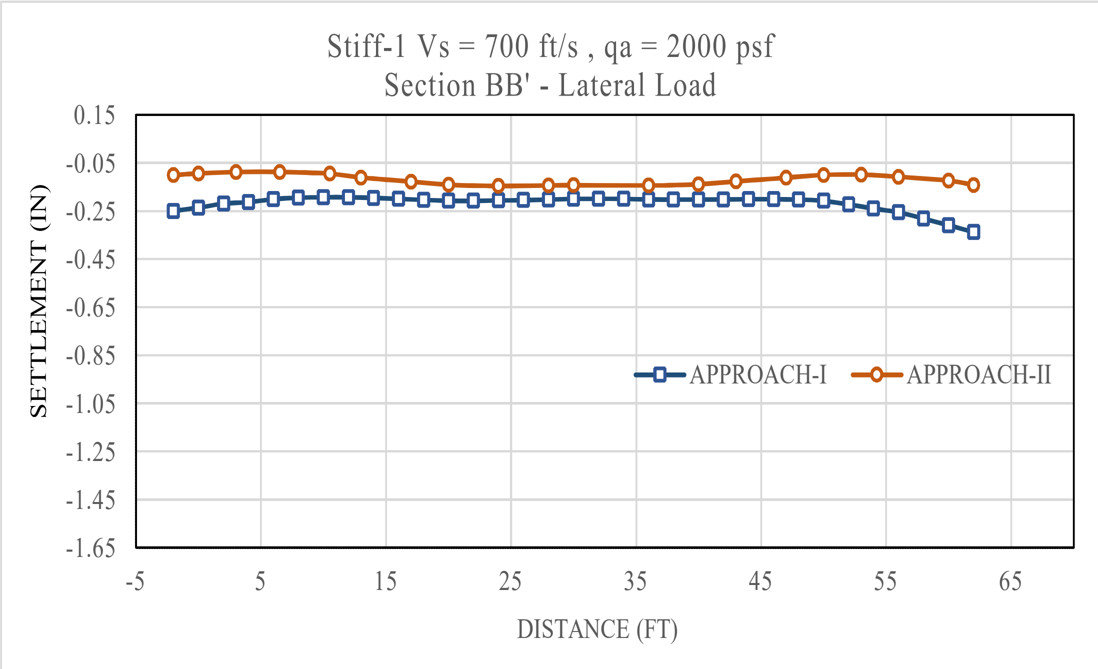


FIGURE 13: Comparison of Settlement Profile of Approach I and Approach II at Section BB' For Lateral Load Case

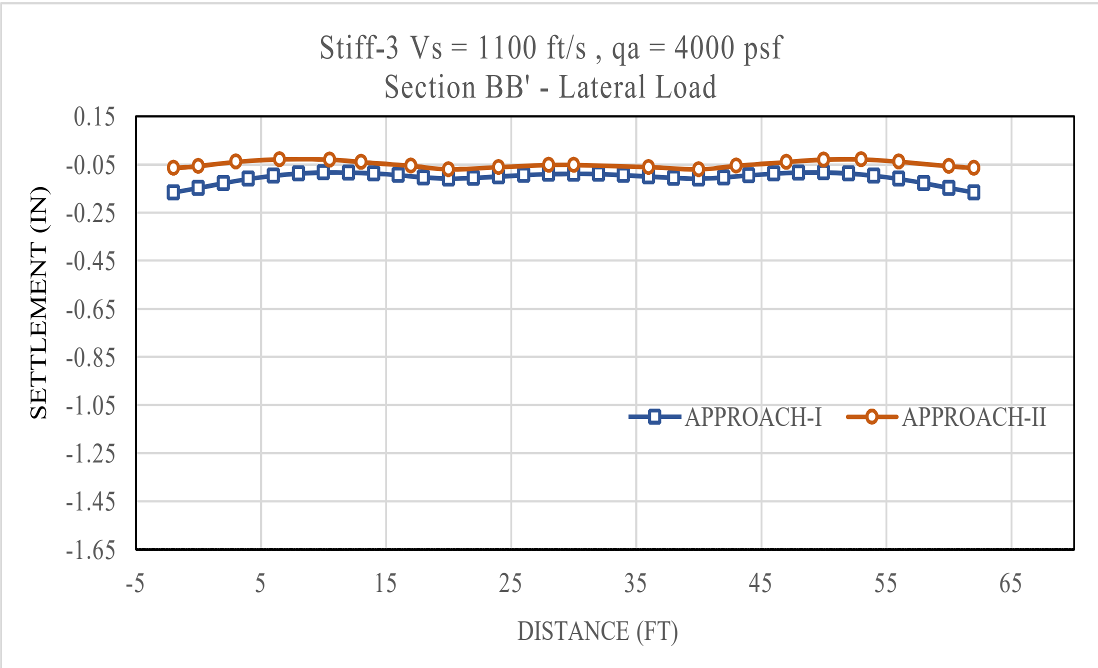


FIGURE 14: Comparison of Settlement Profile of Approach I and Approach II at Section BB' For Lateral Load Case

Appendix - B Bending Moments Profiles

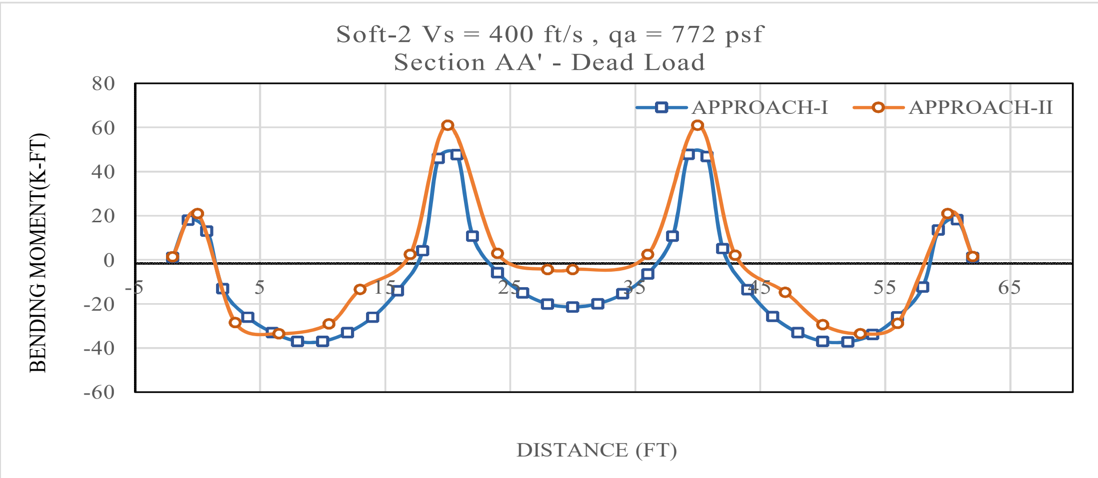


FIGURE 15: Comparison of Bending Moment of Approach I and Approach II at Section AA' for dead load case

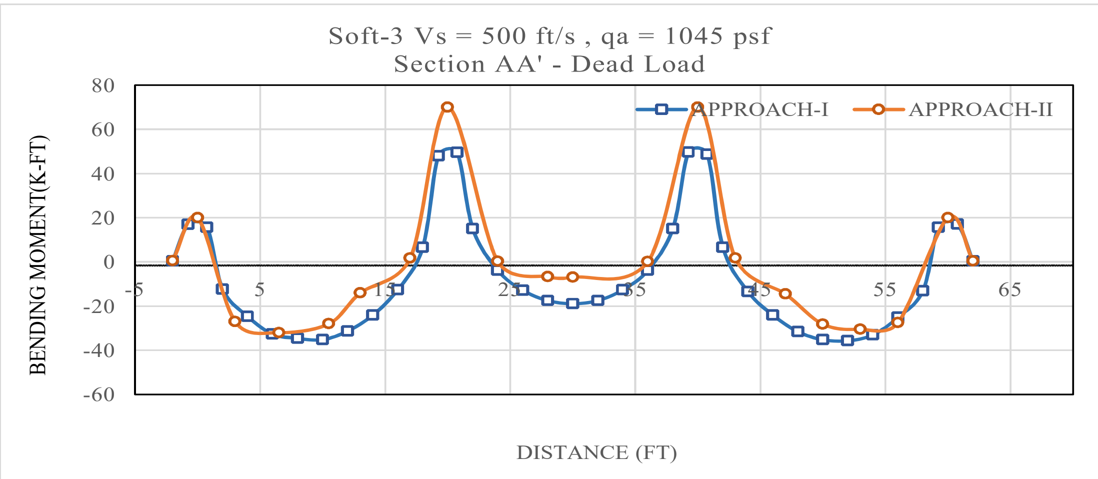


FIGURE 16: Comparison of Bending Moment of Approach I and Approach II at Section AA' for dead load case

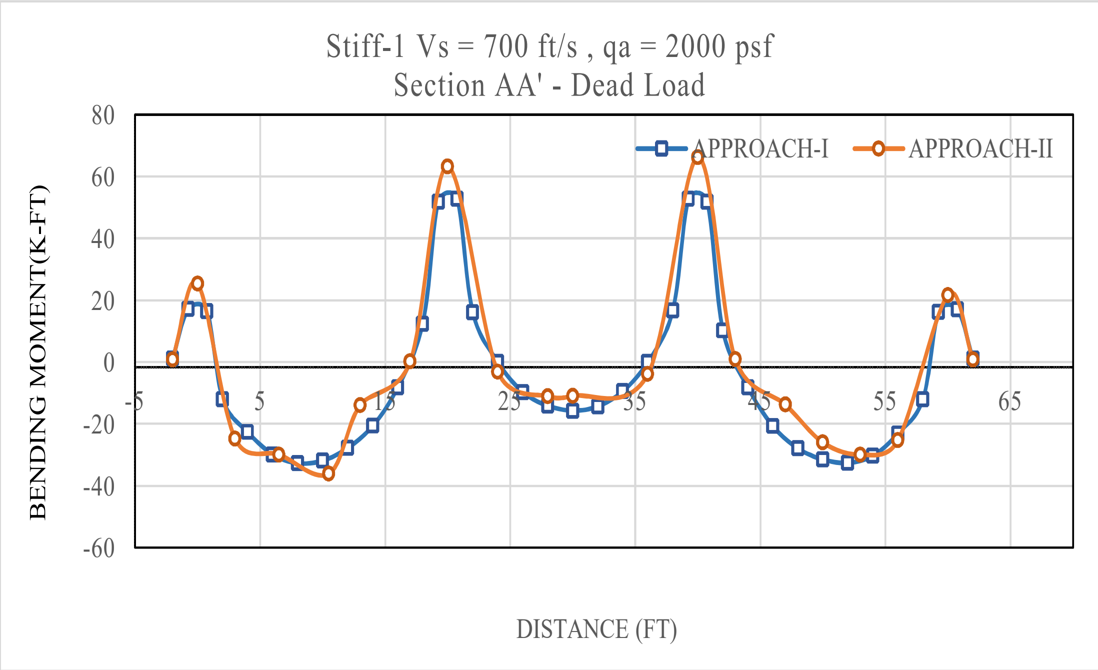


Figure 17: Comparison of Bending Moment of Approach I and Approach II at Section AA' dead load case

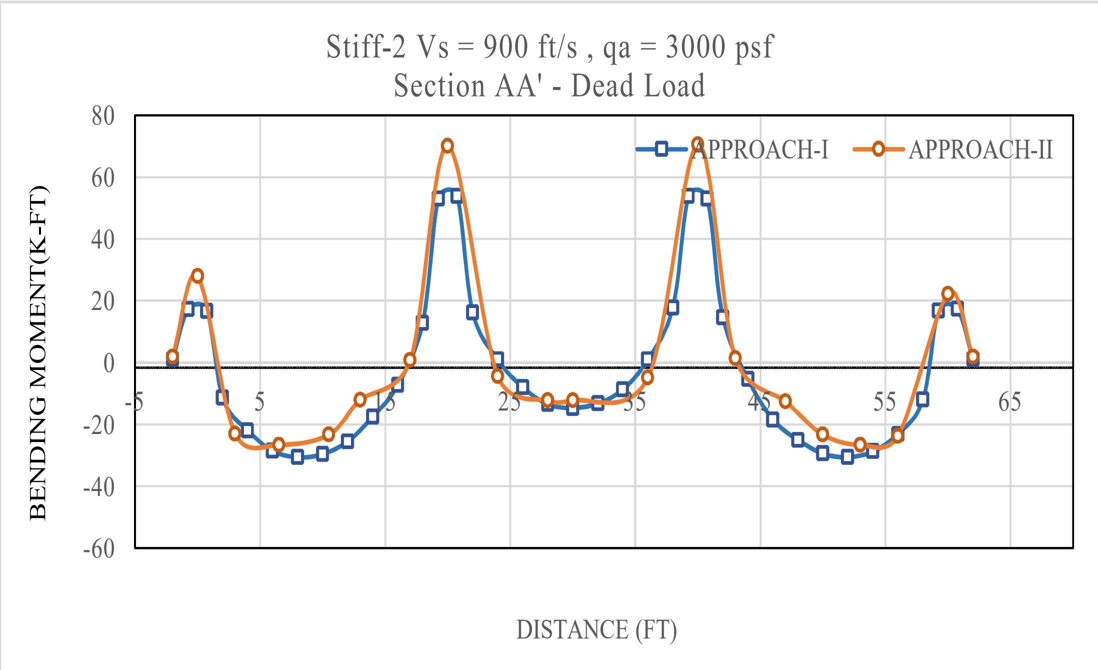


Figure 18: Comparison of Bending Moment of Approach I and Approach II at Section AA' dead load case

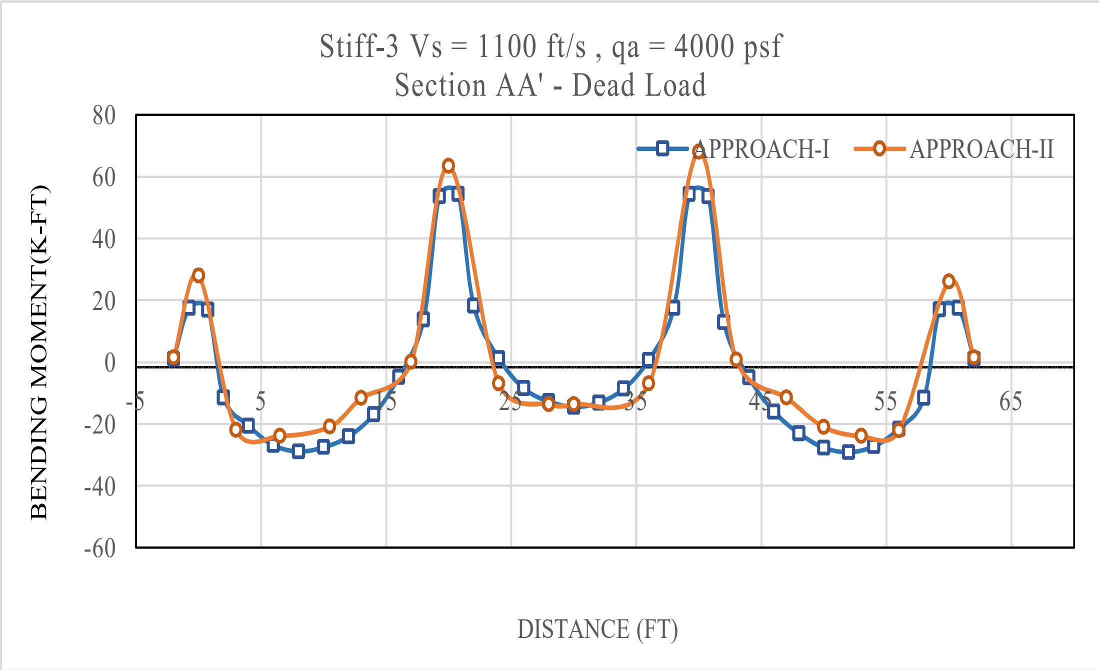


FIGURE 19: Comparison of Bending Moment of Approach I and Approach II at Section AA' dead load case

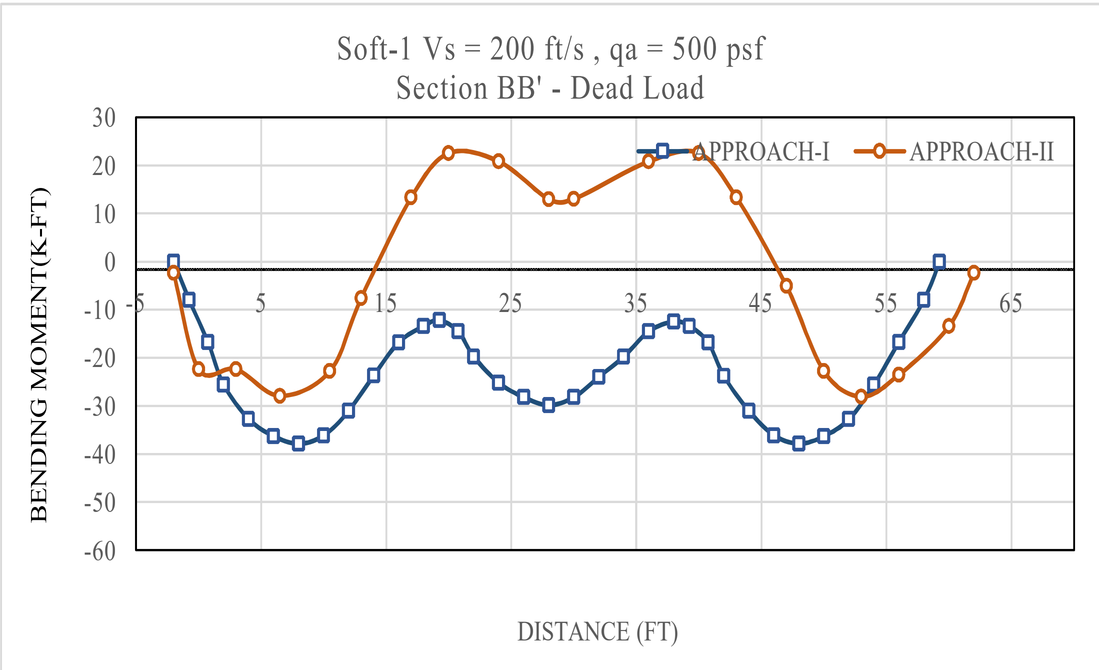


FIGURE 20: Comparison of Bending Moment of Approach I and Approach II at Section BB' dead load case

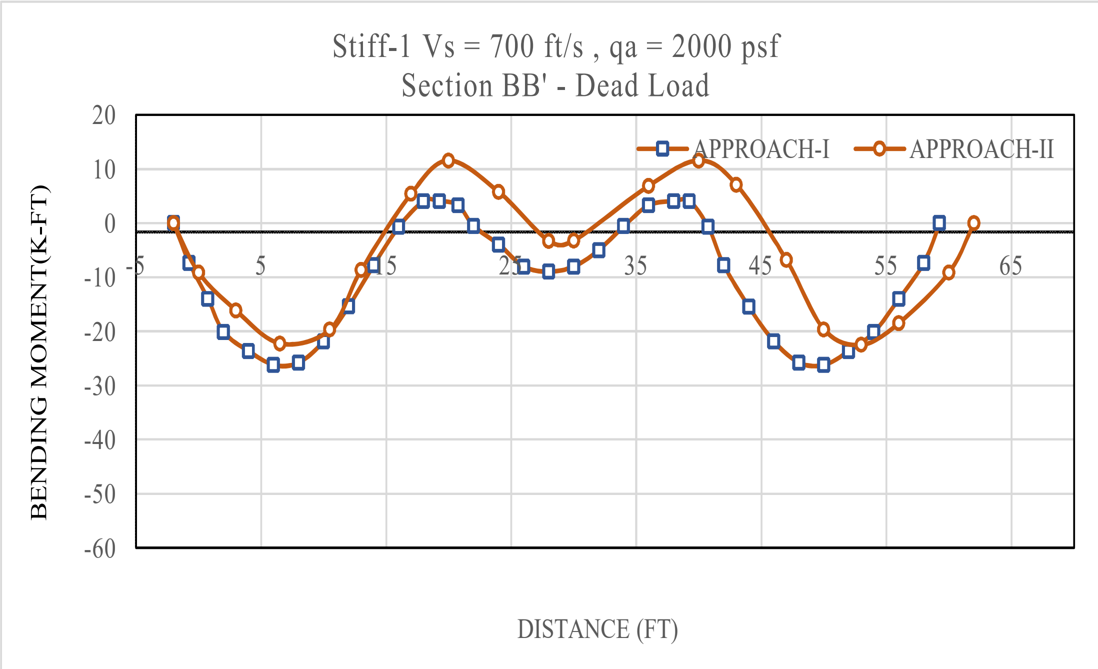


FIGURE 21: Comparison of Bending Moment of Approach I and Approach II at Section BB' dead load case

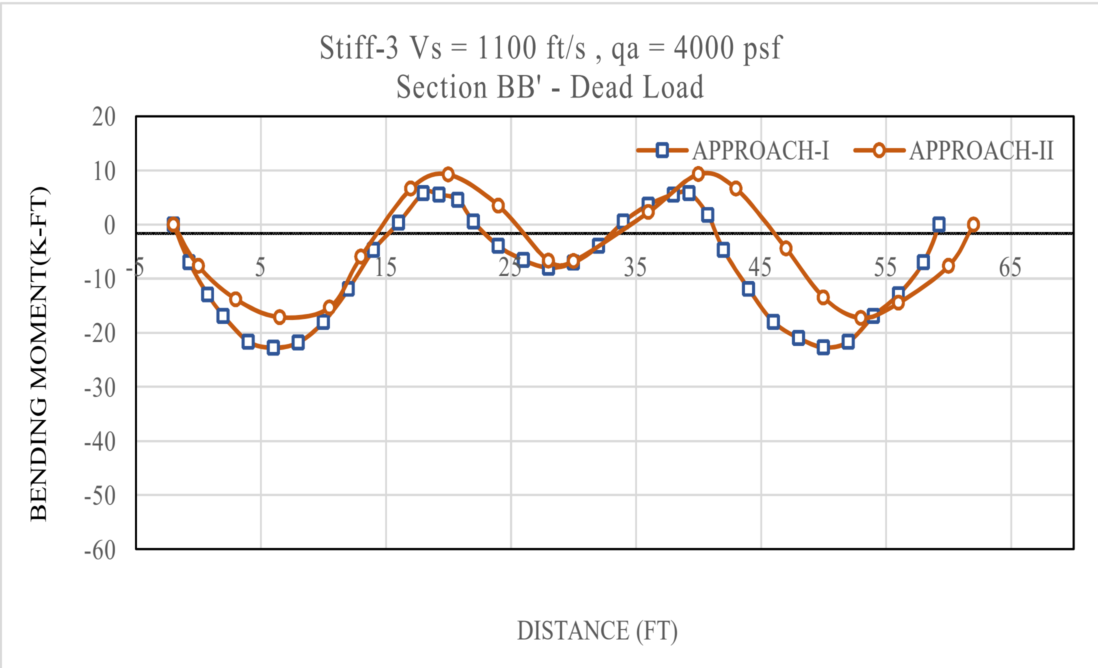


FIGURE 22: Comparison of Bending Moment of Approach I and Approach II at Section BB' dead load case

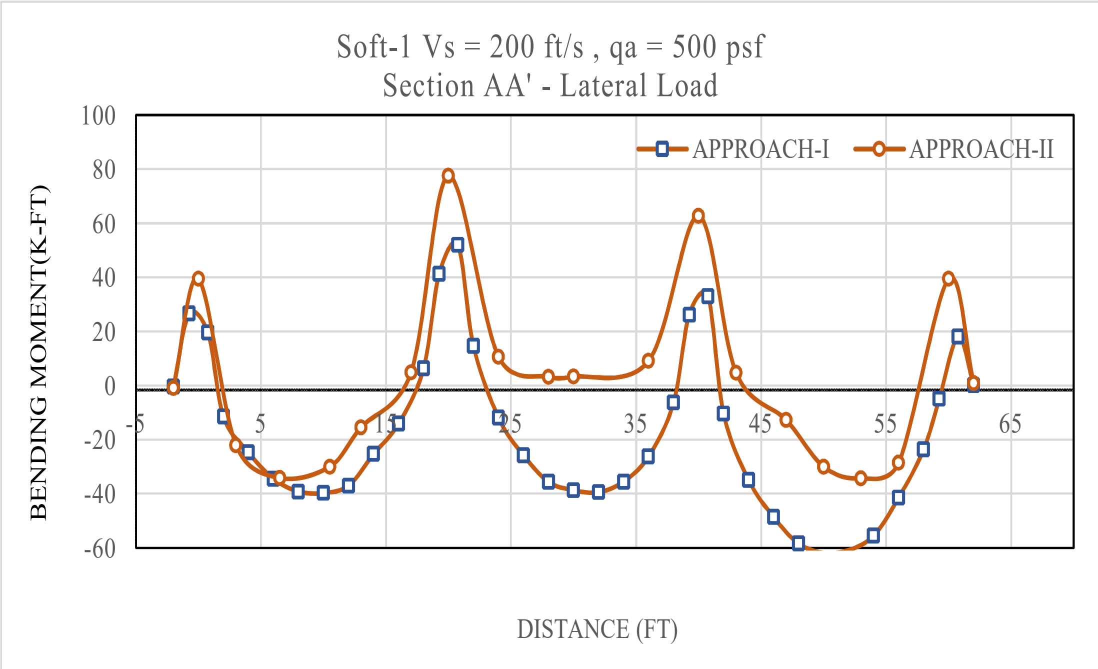


FIGURE 23: Comparison of Bending Moment of Approach I and Approach II at Section AA' for lateral load

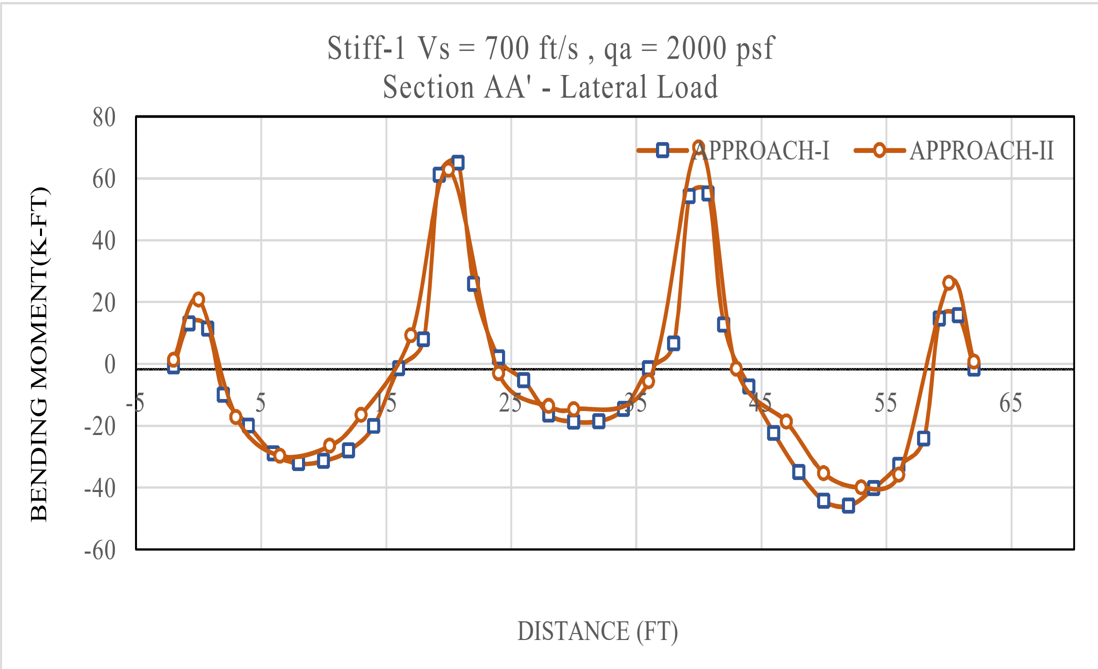


FIGURE 24: Comparison of Bending Moment of Approach I and Approach II at Section AA' for lateral load

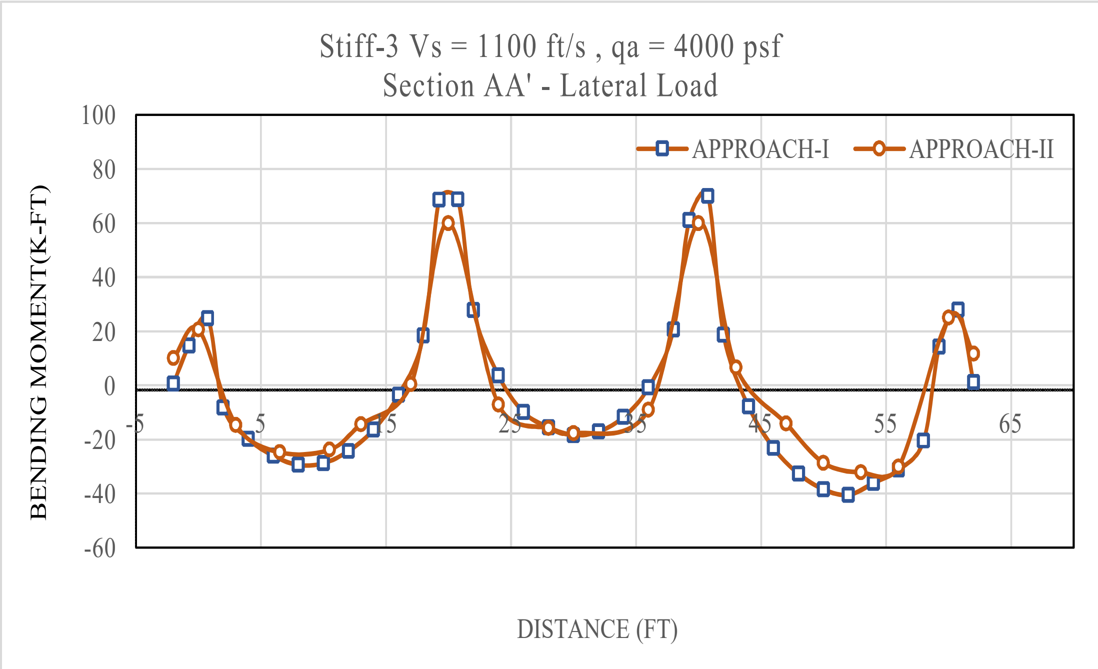


FIGURE 25: Comparison of Bending Moment of Approach I and Approach II at Section AA' for lateral load

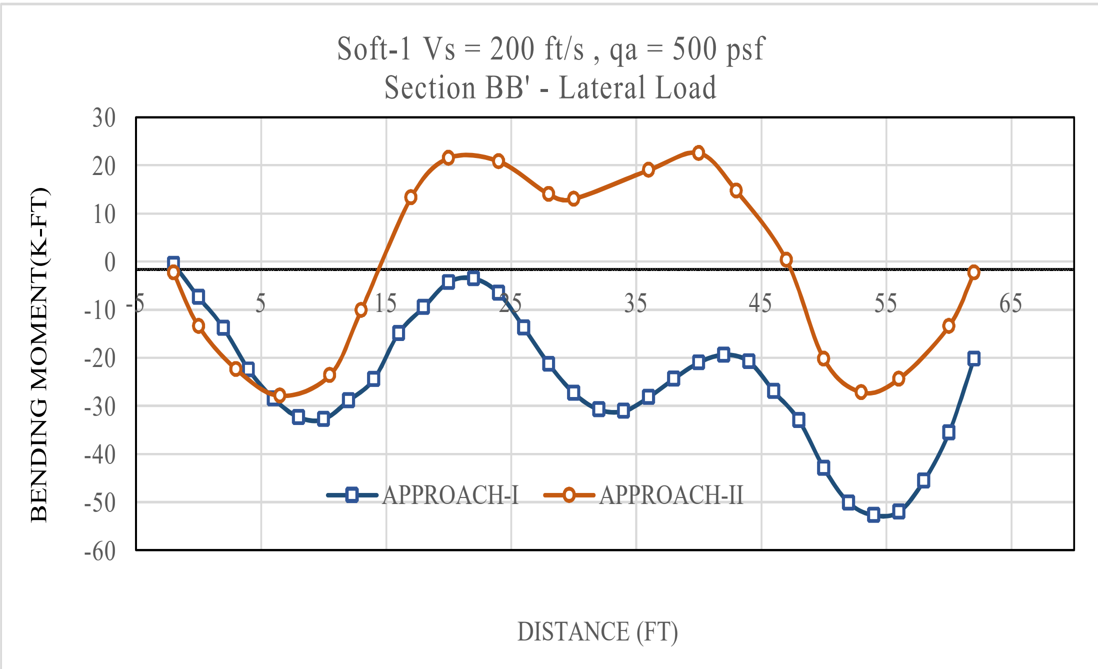


FIGURE 26: Comparison of Bending Moment of Approach I and Approach II at Section BB' for lateral load

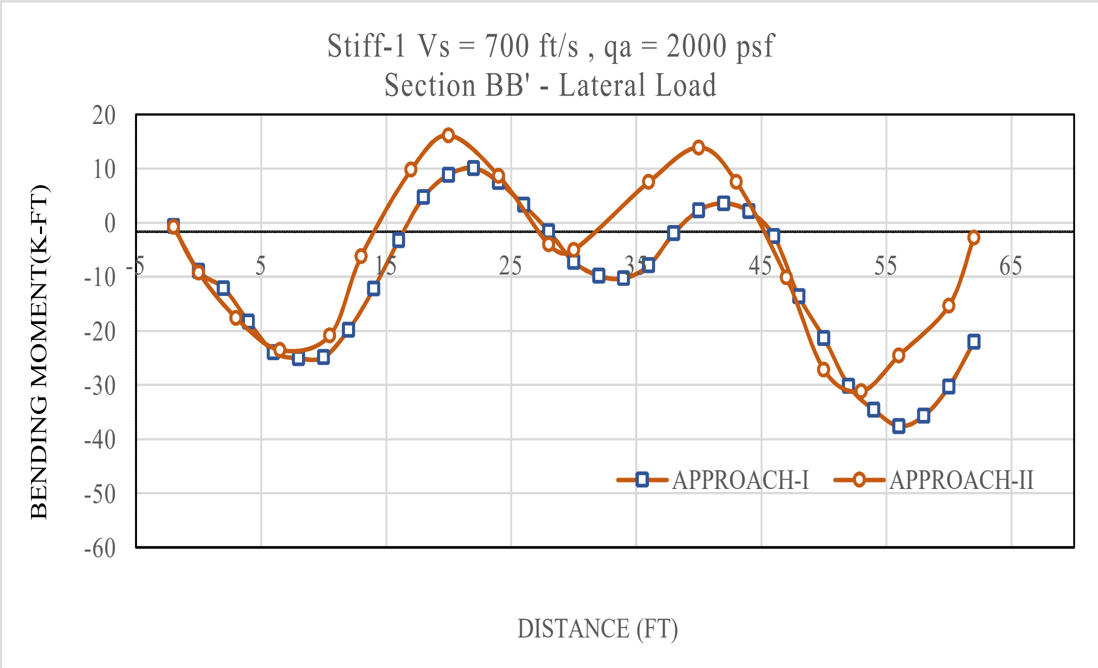


Figure 27: Comparison of Bending Moment of Approach I and Approach II at Section BB' for lateral load

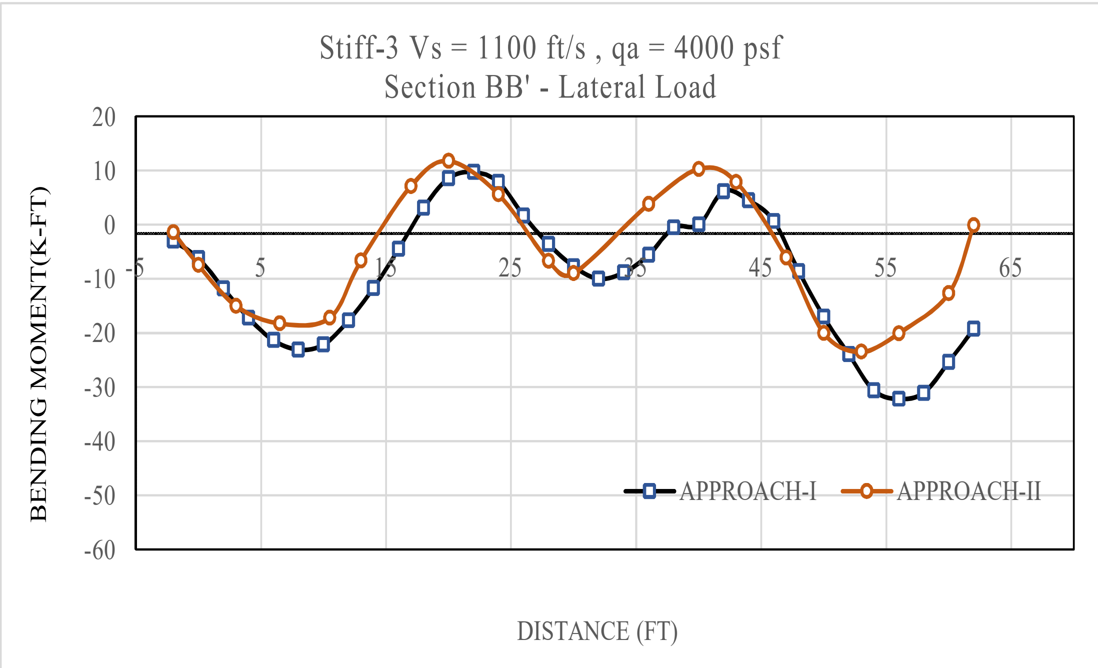


Figure 28: Comparison of Bending Moment of Approach I and Approach II at Section BB' for lateral load

Appendix - C Shear Force Profiles

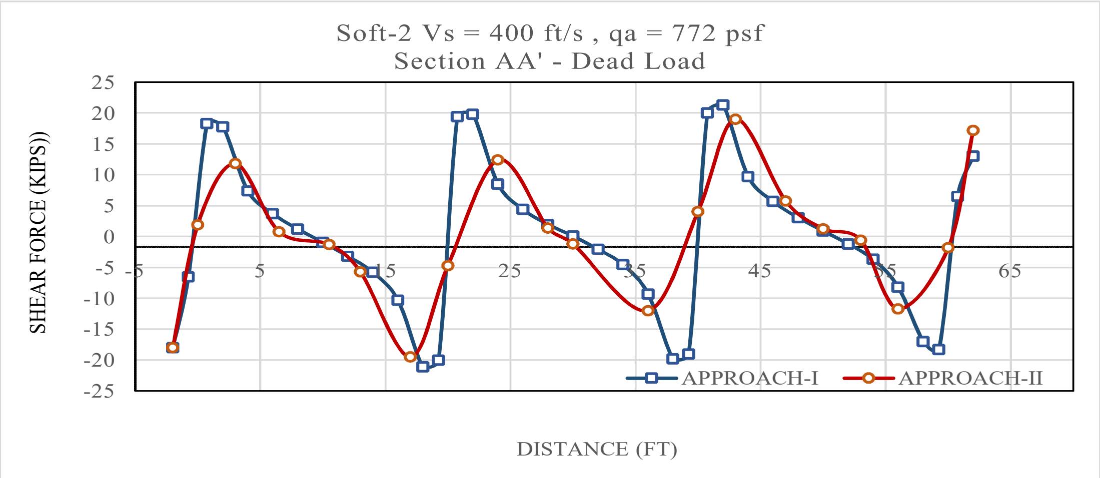


FIGURE 29: Comparison of Shear Force of Approach I and Approach II at Section AA' for dead load case

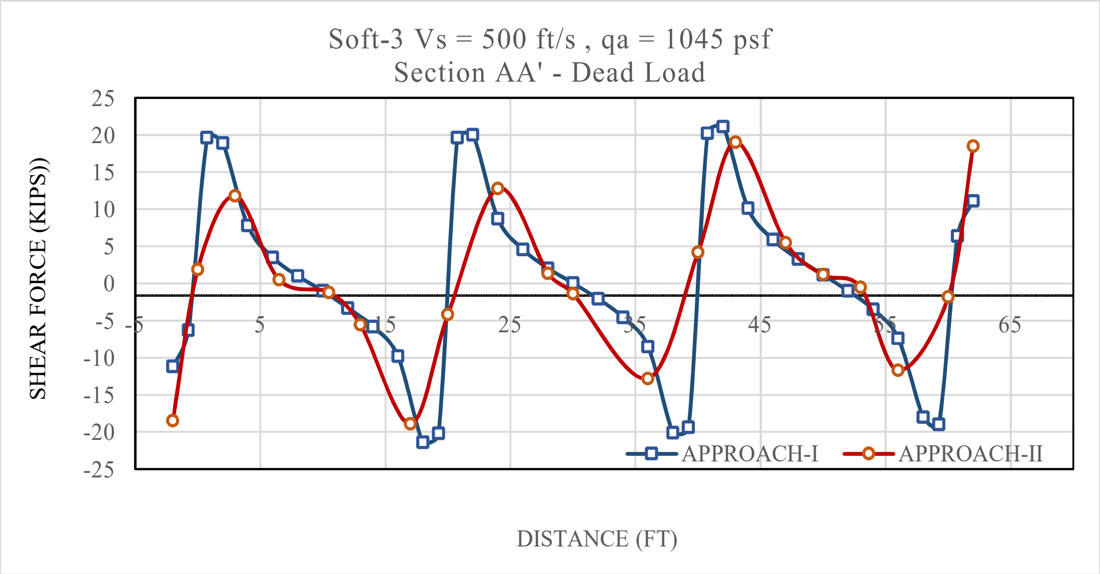


FIGURE 30: Comparison of Shear Force of Approach I and Approach II at Section AA' for dead load case

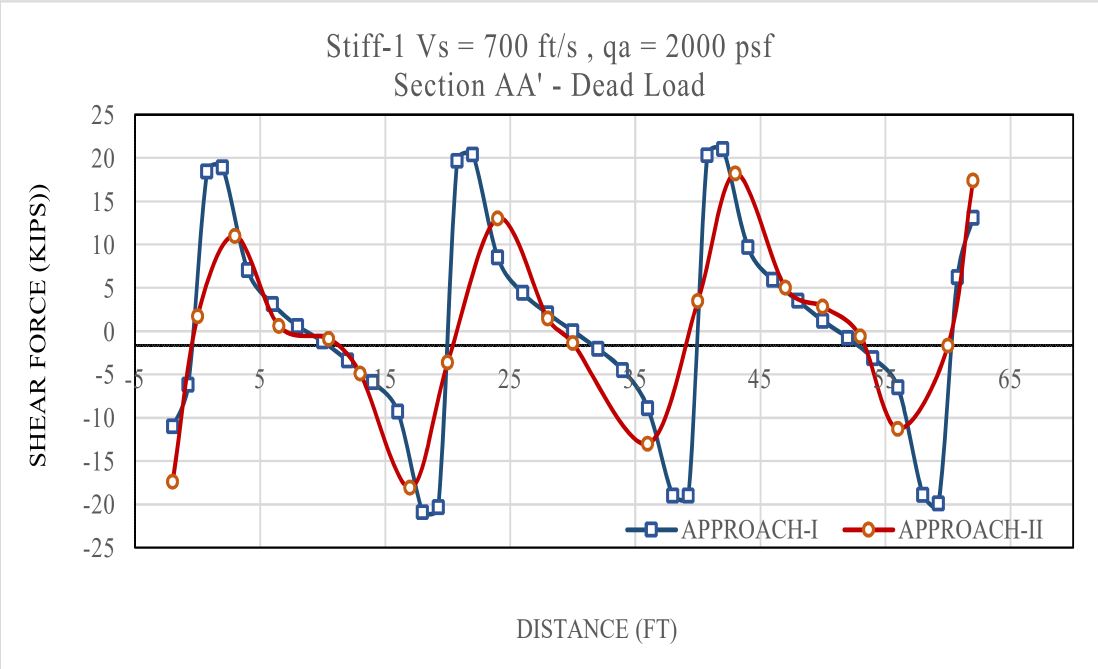


FIGURE 31: Comparison of Shear Force of Approach I and Approach II at Section AA' for dead load case

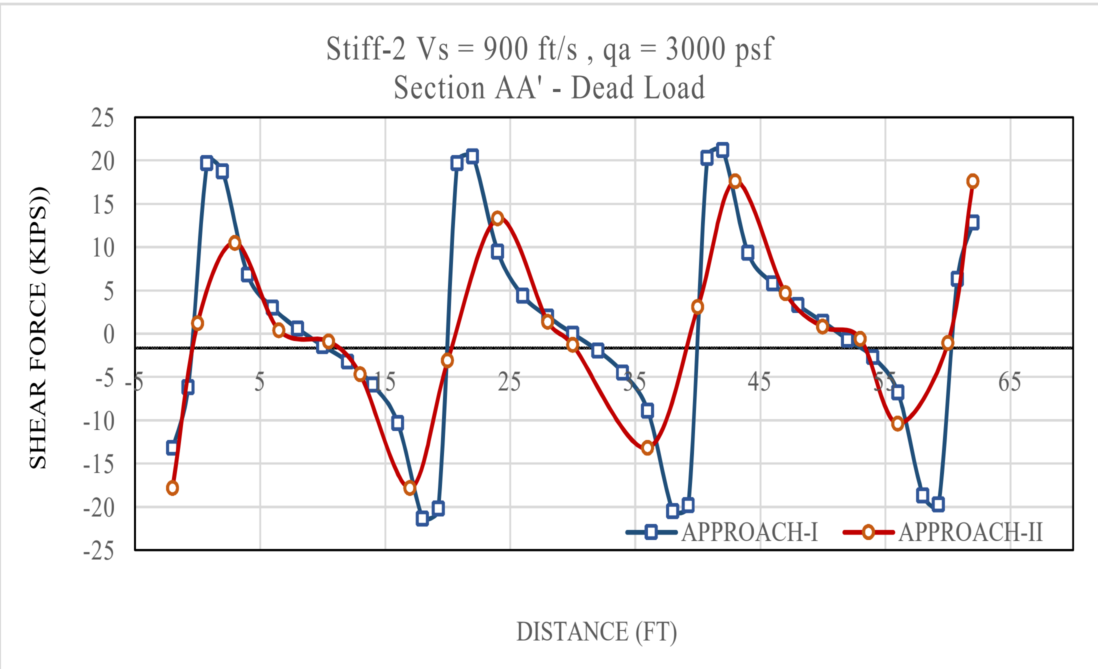


FIGURE 32: Comparison of Shear Force of Approach I and Approach II at Section AA' for dead load case

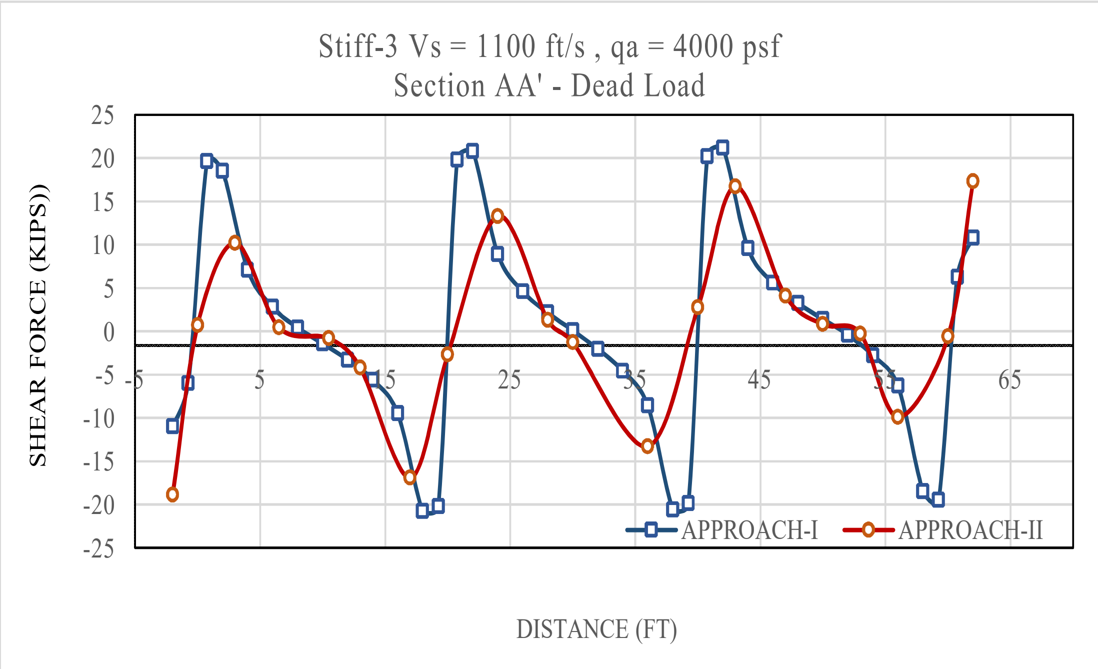


FIGURE 33: Comparison of Shear Force of Approach I and Approach II at Section AA' for dead load case

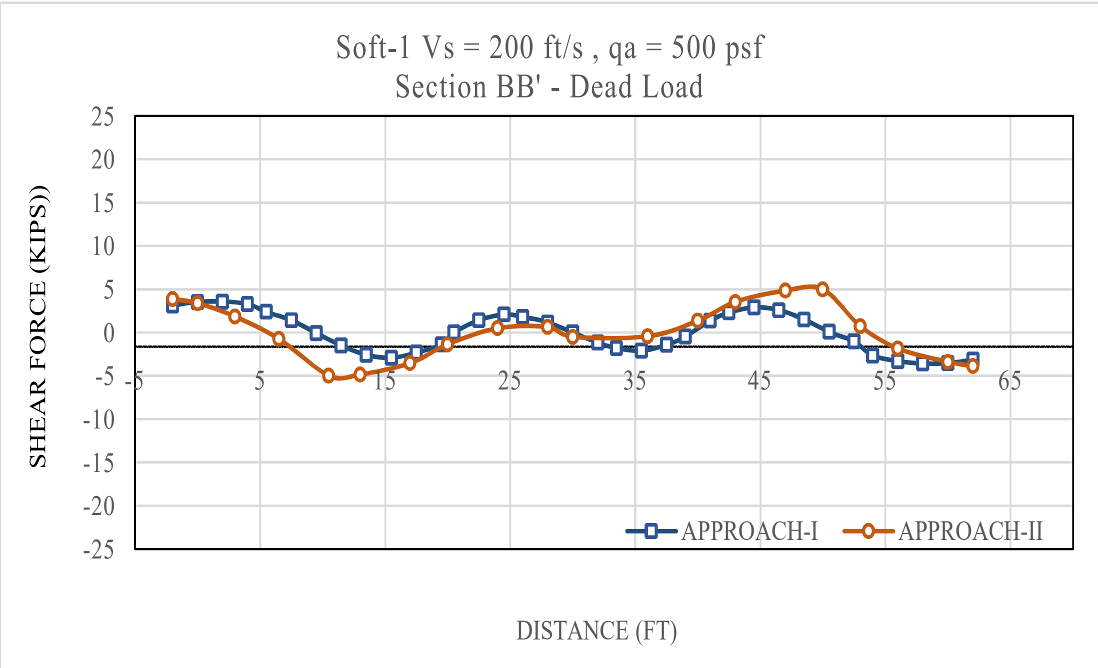


FIGURE 34: Comparison of Shear Force of Approach I and Approach II at Section BB' for dead load case

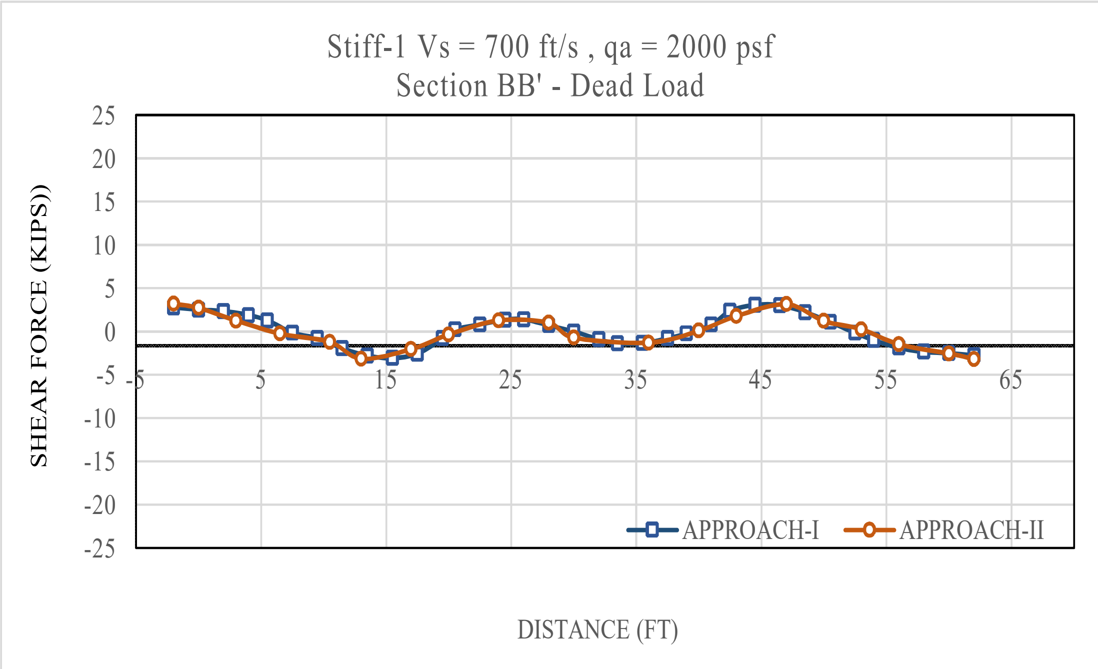


FIGURE 35: Comparison of Shear Force of Approach I and Approach II at Section BB' for dead load case

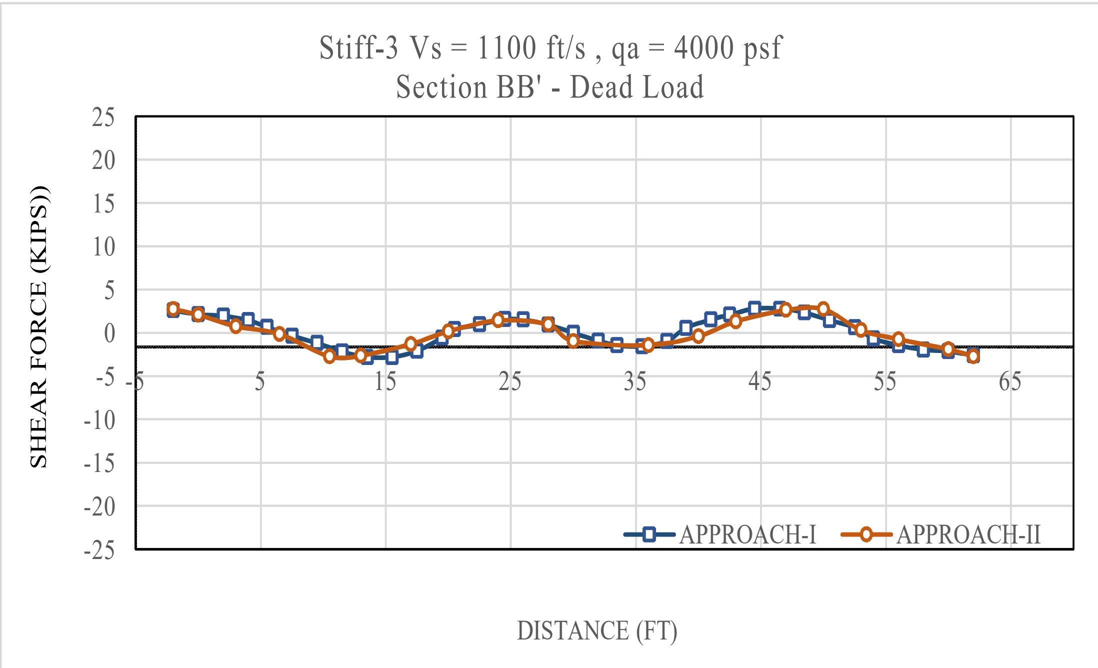


FIGURE 36: Comparison of Shear Force of Approach I and Approach II at Section BB' for dead load case

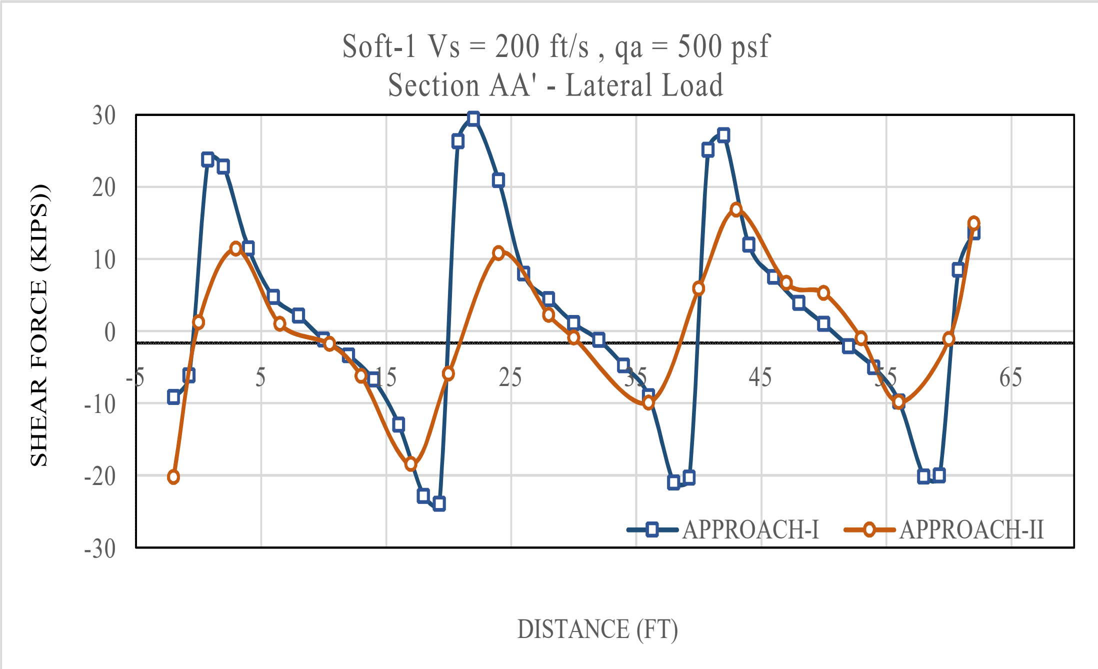


FIGURE 37: Comparison of Shear Force of Approach I and Approach II at Section AA' for lateral load case

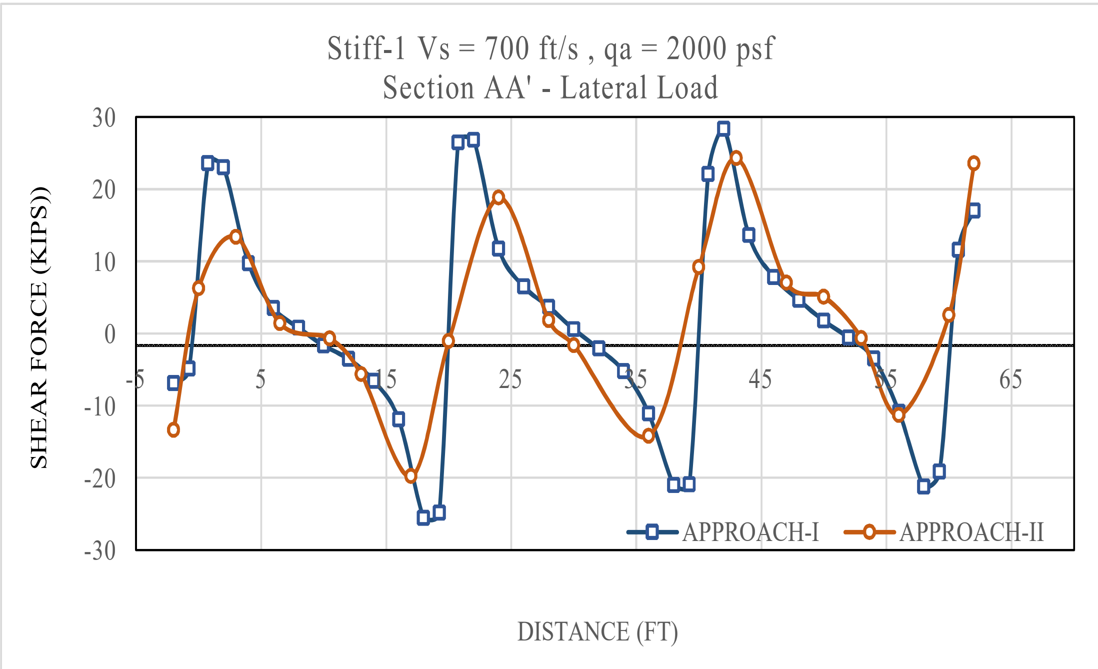


FIGURE 38: Comparison of Shear Force of Approach I and Approach II at Section AA' for lateral load case

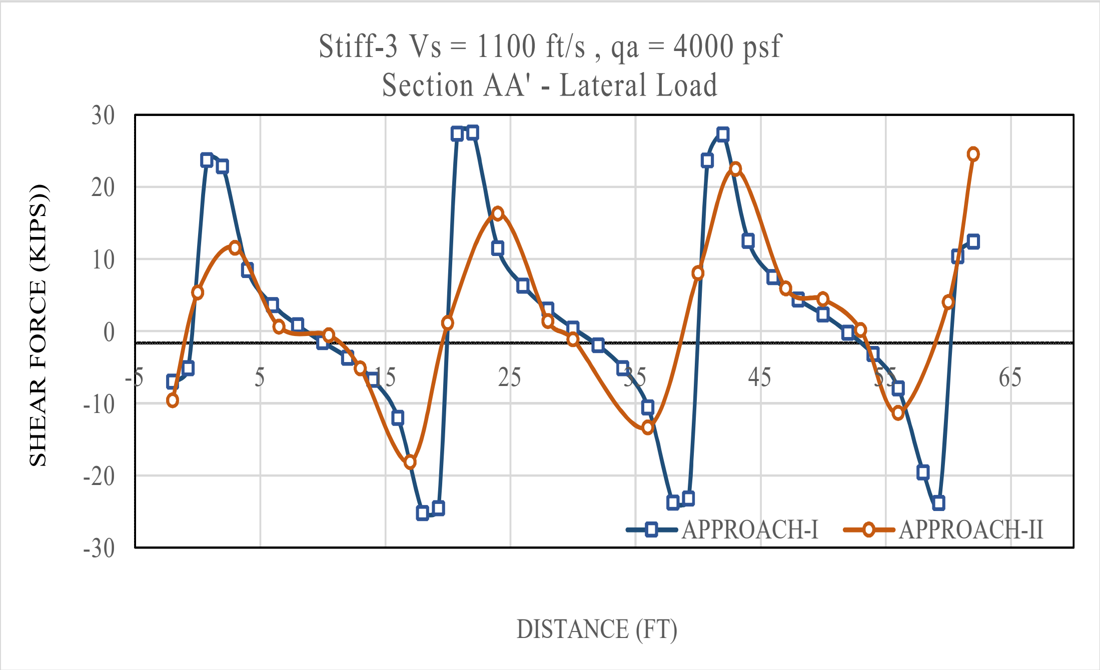


FIGURE 39: Comparison of Shear Force of Approach I and Approach II at Section AA' for lateral load case

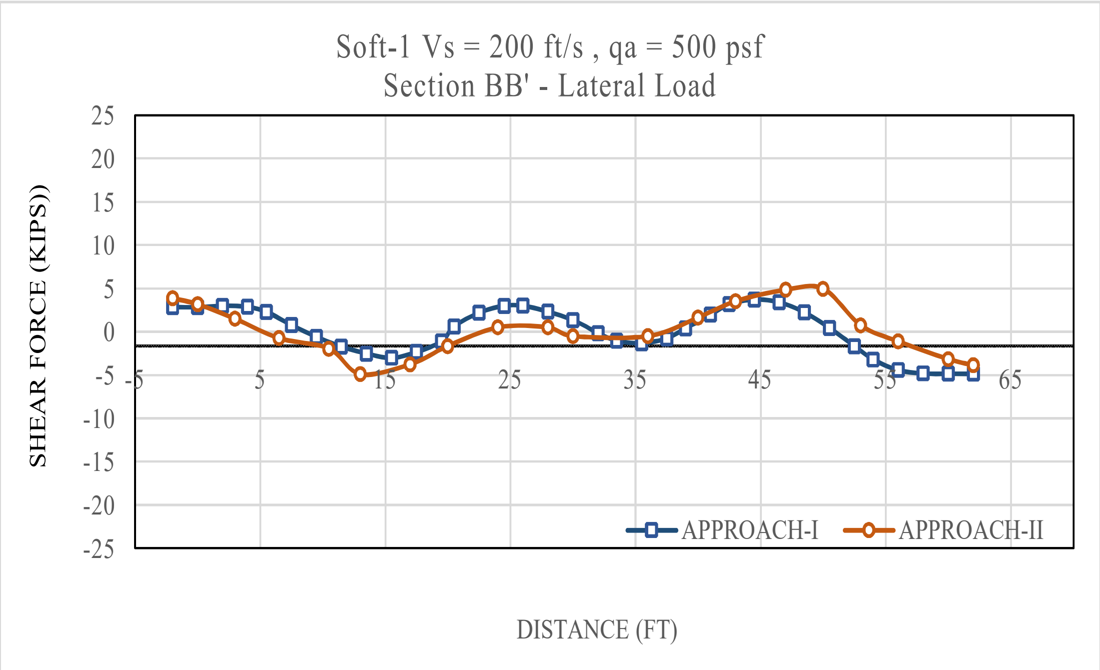


FIGURE 40: Comparison of Shear Force of Approach I and Approach II at Section BB' for lateral load case

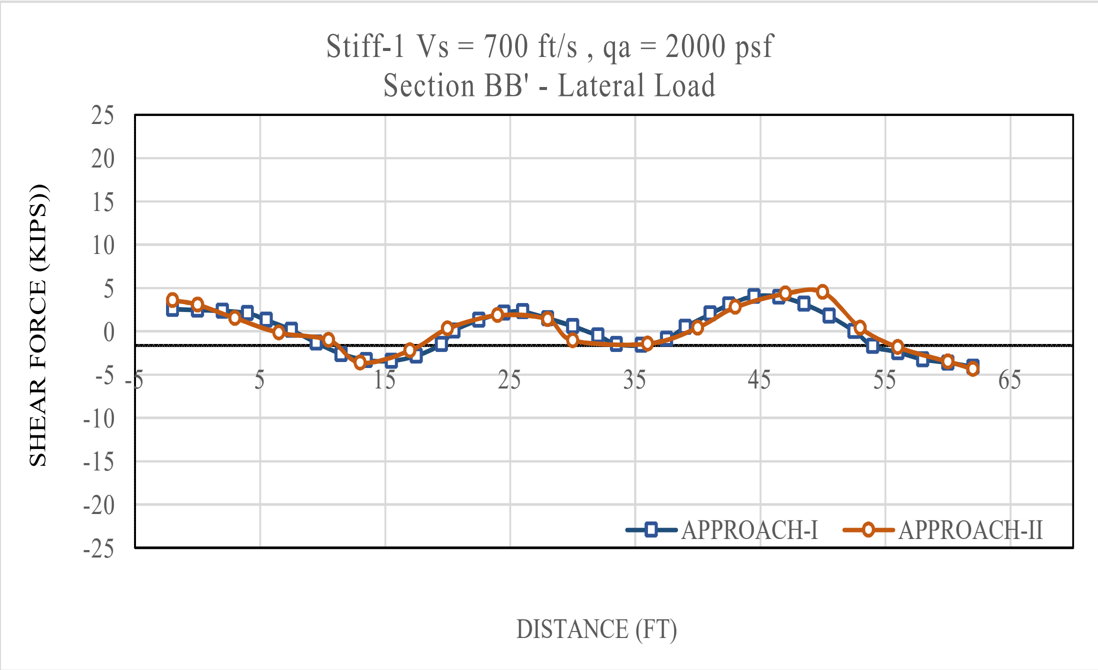


FIGURE 41: Comparison of Shear Force of Approach I and Approach II at Section BB' for lateral load case

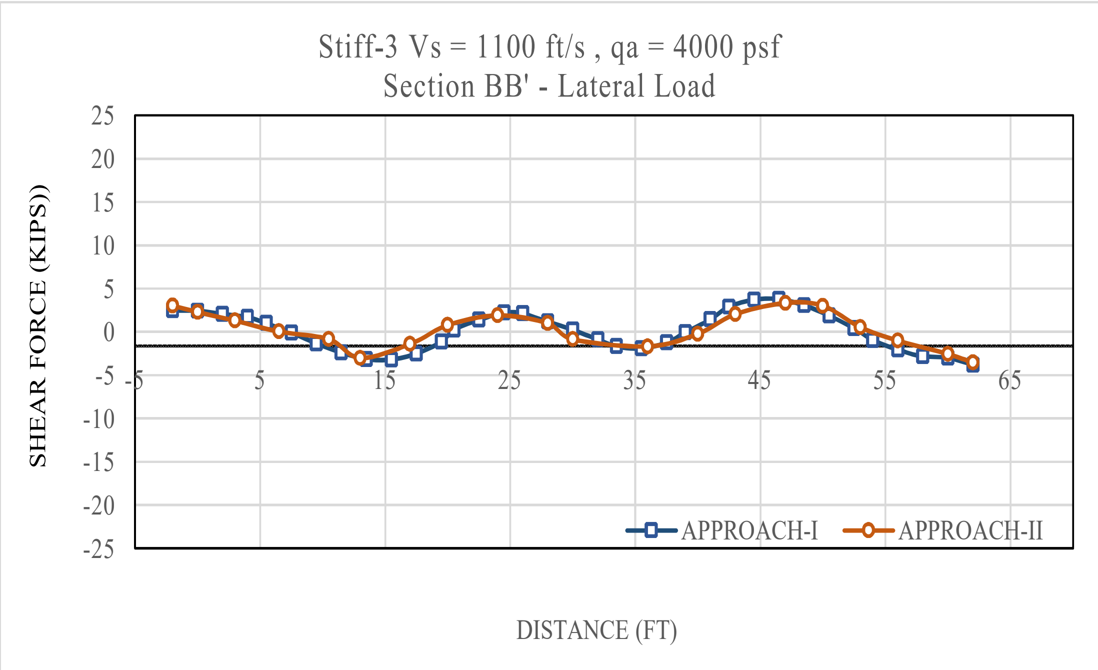


FIGURE 42: Comparison of Shear Force of Approach I and Approach II at Section BB' for lateral load case