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# Appendix B: Archetype Building 1

## Concrete Moment Frame with Concrete Shear Walls

### B.1 Motivation

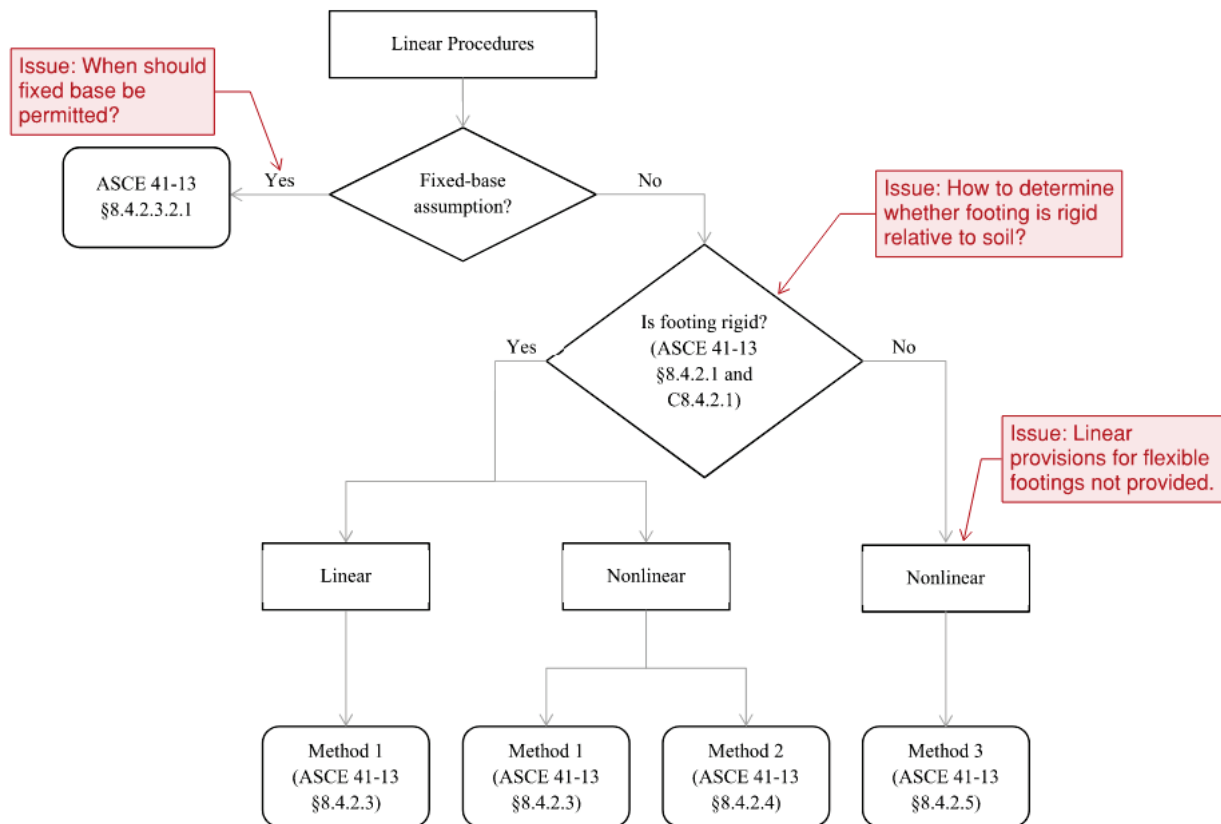
Previous seismic rehabilitation guidelines such as FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, and later the ASCE 41/SEI -06 *Seismic Rehabilitation of Existing Building* have both provided guidelines for foundation analysis and retrofit design. ASCE/SEI 41-13 and -17 sought to improve these guidelines and provide more accurate results. However, there are still a number of areas for improvement within these provisions. The WG-2 Foundation Working Group was tasked with evaluating the shallow foundation provisions in the ASCE/SEI 41-17 standard for clarity, usability, and technical content and providing recommendations and code change proposals as input for deliberation by the ASCE/SEI 41-23 committee to be incorporated in the next ASCE/SEI standard update.

To provide historical context, ASCE/SEI 41-06 foundation provisions utilized linear procedures to incorporate soil-structure interaction which included kinematic and foundation damping. However, for flexible base modeling, both FEMA 356 and ASCE/SEI 41-06 allowed for infinite ductility if a spring was added in modeling; the soil strength was not required to be evaluated. Research has shown that this soil bearing with infinite ductility assumption can be correct when the axial forces on the foundation (both gravity and earthquake) are low. However, it is not always the case, and can cause an underestimation of deformations (transient, during the earthquake, and permanent) in the superstructure when axial forces are higher. In addition, given the infinite ductility assumption, acceptance criteria was not provided for the flexible base modeling case. Lastly, the Method 1 soil stiffness assumed that the footing was rigid, and the soil remained elastic and in contact over the entire bottom of footing surface, which can overestimate the soil stiffness by a significant amount.

ASCE 41/SEI -06 also decoupled the rocking and yielding mechanisms and had separate checks for them, despite that they do not occur independently. In ASCE/SEI 41-13, the decoupled rocking issue was addressed with the addition of  $m$ -factor tables and nonlinear acceptance criteria for these actions that are a function of the soil stiffness and gravity loads on the foundations. In addition, ASCE/SEI 41-13 revised the soil-foundation-structure interaction provisions and added limitations. The fundamental concept of both the ASCE/SEI 41-13 revision and the ASCE/SEI 41-17 update is that if the acceptance criteria of the foundation chapter are satisfied, regardless of the methodology used (subject to limitations of each method), then the foundation deformations are accurate enough, and the analysis is suitable for determining the component level acceptance criteria of the superstructure. This philosophy is retained in the changes ATC is proposing for ASCE 41-23.

However, several issues with the ASCE/SEI 41-17 foundation chapter have been identified.

- There are large gaps in the ASCE/SEI 41-17 linear procedure process including:
  - A lack of clarity on when a fixed-base assumption is permitted leads to confusion about what analysis provisions to follow.
  - The rigidity of the footing relative to the soil must be determined, in order to establish the applicable analysis method. However, the method provided for the relative rigidity determination is in the commentary section and does not take into account that soil separates from the footing during rocking action.
  - Linear provisions for footings that are flexible relative to soil are not provided (Method 3 did not provide provisions for linear procedures). Therefore, the user has no guidance on evaluating strip (combined) footing or mat foundation conditions with a lack of structural footing stiffness or strength that would classify the structural component of the foundation as flexible relative to the soil.



**Figure B-1 ASCE/SEI 41-13 linear procedures flow chart with gaps identified.**

- Some of the provisions require a flexible-base analysis to be done in addition to a fixed-base analysis to determine if the results from the fixed-base procedures could be used.
- The prescriptive soil properties permitted to be used when soils information is not available are so low that it would most certainly require soil exploration to be conducted for evaluation of the foundations for almost all buildings regardless of their foundation capacity or level of seismicity. Further, the prescriptive bearing capacities are inconsistent between the two methods provided; expected bearing capacities based on the calculated gravity loads to existing footings per ASCE/SEI 41-17 Equation 8-3 are conservative when compared to expected bearing capacity based on Equations 8-1.
- Method 2 is a complicated process and it is unclear if the complexity provides more accurate results.
- Acceptance criteria for compression due to overturning in the absence of moment on the footing (for example, axial overturning action at ends of a brace frame supported by two independent footings) is not explicitly addressed.
- Soil bearing acceptance is expressed only in terms of ultimate bearing capacity for an isolated rectangular footing resisting axial load and uniaxial moment. This left out a lot of cases and necessitated the use of engineering judgement which potentially resulted in inconsistencies and misapplication in the use of the standard. The  $m$ -values provided in ASCE/SEI 41-17 are derived based on axial and overturning actions but were incorrectly applied to soil bearing. The intent is that these  $m$ -values should not be applied to soil bearing.
- The use of the fixed-base method can result in unusually large footings for a typical superstructure. The results can also be unusually large compared to footings using the other methods or ASCE/SEI 7-16 for similar-sized superstructures.
- Foundation acceptance is only provided for soil bearing with little guidance provided for evaluation of the foundation structural component. In fact, the provisions in the material chapters pertaining to foundations are inconsistent with Chapter 8. The material chapters for concrete and masonry require force-controlled foundation designs while the wood and steel chapter requires foundation design to be deformation controlled, see Table B-1. Further, it is unclear from Chapter 8 if certain items for nonconforming concrete beams can be treated as deformation-controlled actions.

**Table B-1 Material Specific Structural Foundation Requirements from FEMA P-2006**

Foundation Material	ASCE/SEI 41-13 Section	Action Type
Steel	§ 9.9.4	Deformation-controlled for steel pile; Force-controlled for connection from pile to pile cap
Concrete	§ 10.12.3	Force-controlled; the required capacity is limited by 125% of the capacity of the supported vertical component
Masonry	§ 11.6.2	Force-controlled and modeled as elastic with no inelastic deformation capacity unless demonstrated through ASCE/SEI 41-13 § 7.6
Wood	§ 12.6.2	Flexure and axial loads are considered deformation-controlled with <i>m</i> -factors per ASCE/SEI 41-13 Table 12-3. Acceptability of soil below wood footings determined per ASCE/SEI 41-13 Chapter 8.

- Bounding for stiffness and bearing capacities has been required because soil is inherently less homogeneous and has greater variations in material properties than other materials such as steel or concrete. However, the bounding in ASCE/SEI 41-17 is problematic for two main reasons. First, the high and low bounding requires extra analysis effort and thus should yield significantly different results. However, because the bounding and calibration is determined from ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991), the bounding does not result in significant changes in the superstructure response. Second, the ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) equations are based on a rigid structure and elastic soil response where the soil remains in contact with the footings, this overestimates the stiffness. Before applying bounding requirements to the more realistic lower stiffnesses, case studies need to be completed to ensure that the bounding yields significant results as well as not overreach and cause undue conservatism.
- Usability and clarity issues include:
  - Navigation through the foundations chapter in ASCE/SEI 41-17 is complicated as requirements for linear and nonlinear procedures were intermixed within the standard.
  - In some cases, acceptance criteria and direction for items such as bounding and stiffness are provided in narrative form. This led to confusion in applying the provisions and reduced the usability and clarity of the chapter. Tabulated acceptance criteria would be easier to follow.
- Further nuanced technical concerns:

- Definitions of select key terms, such as uplift, are not clear, leading to confusion and misuse of provisions. Uplift in the context of these provisions is the pure axial force causing the entire footing to separate from the soil as opposed to some soil separation on part of the footing as the footing rotates due to rocking action.
- Determination of the effective footing width ( $B_f$ ) for a mat foundation is missing,
- Where footing overturning action cannot be idealized as a rectangular or I-shaped footing, such as combined footings and mat foundations, the analysis method requires engineering judgement.
- Although based simply on statics, ASCE/SEI 41-17 Equation 8-10 for determining the upper bound moment capacity of a rigid shallow rectangular footing may be confusing to users without its derivation and therefore could lead to implementation issues.

$$M_{CE} = 0.5(L_f P_{UD})(1-q/q_c) \quad (\text{ASCE/SEI 41-17 Eq. 8-10})$$

- The overturning action is very dependent upon the transient axial load level, so when evaluating with pseudo seismic forces, determining a realistic seismic axial load is very difficult. This issue is not limited to foundations; it applies to linear procedures of other chapters as well.
- The intended definition of  $Q_g$  utilized in Chapter 8 is the expected dead load excluding live loads and snow loads and load factors. This is the  $Q_D$  definition in Chapter 7 (the action cause by dead loads).
- The FEMA P-2006 *Example Application Guide for ASCE/SEI 41-13* examples used the load combinations where the intent was just to use expected dead load, indicating that this misinterpretation is common.

These numerous issues with ASCE/SEI 41-17 were a catalyst for goals for the ATC WG-2 Foundation Working Group study to provide proposed provision and commentary changes for consideration by the ASCE/SEI 41-23 committee and subcommittee. The priorities were to derive a shallow foundation provision structure that was user friendly and to address all the above gaps as well as new items as discovered during the case study work. This ATC work utilized a case study to investigate hypotheses based on these highlighted issues with the provisions.

## B.2 Case Study Overview – Archetype 1

Two case study buildings were investigated as part of this project. Archetype 1, an existing concrete two-way slab and column moment frame building, was investigated to evaluate the use of ASCE/SEI 41-17 Chapter 8 for clarity, usability, and technical content as part of ATC 140 –Working Group 2 objectives. The structure in its original configuration and with concrete shear wall retrofit ASCE/SEI 41-17 were investigated during the study.

Archetype 2, a concrete moment frame building, and its analysis are described in a separate appendix.

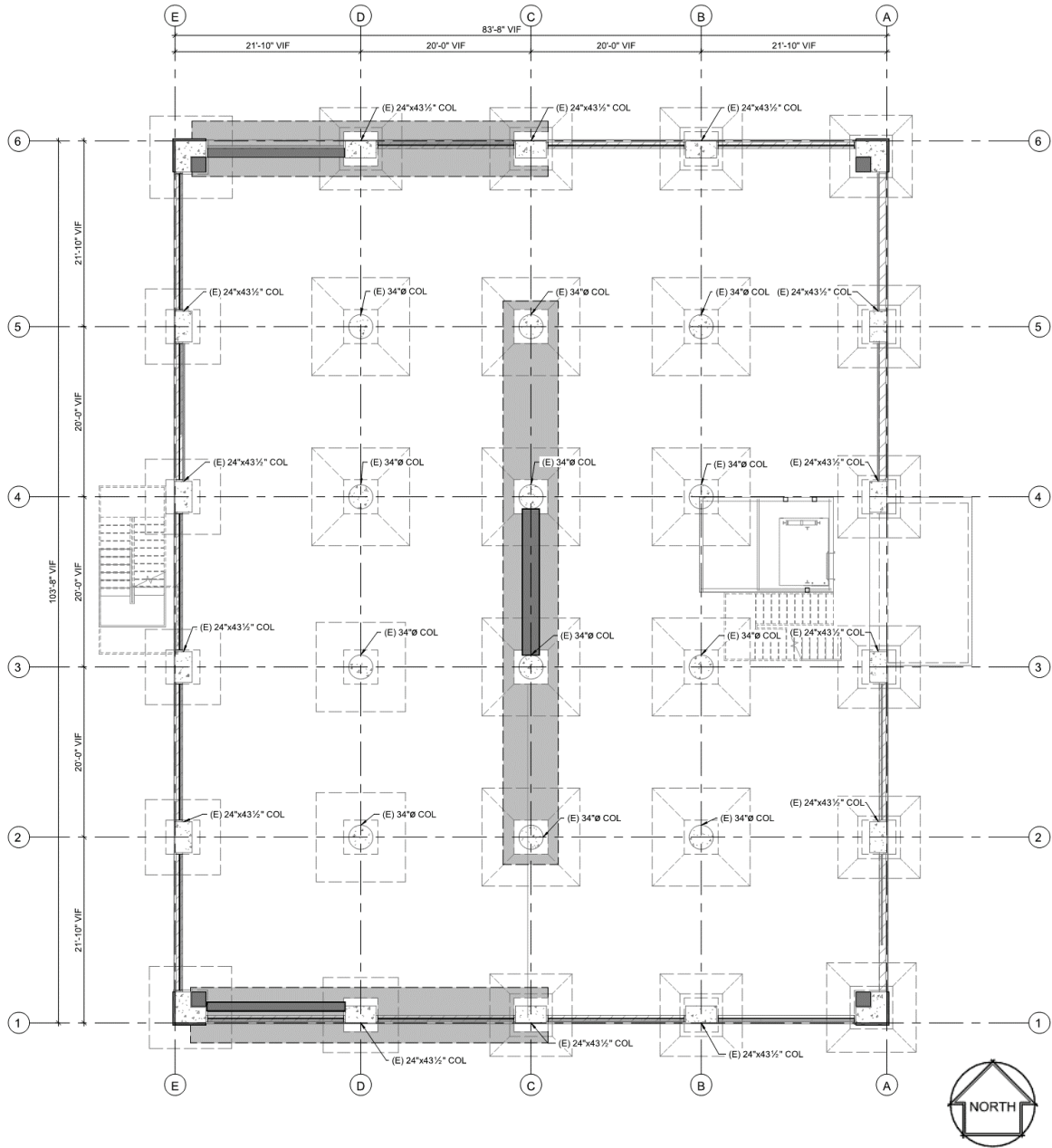
### **B.2.1 Building Description**

The 1920s existing building is a five-story, 55-foot-tall reinforced concrete structure that measures approximately 104 by 84 feet (5 by 4 bays) in plan. Concrete columns occur on an approximate 20-foot square grid throughout the building, and the structure is supported at its base on shallow isolated footings. Floor and roof slabs are reinforced concrete; the core for the existing elevator and stair are non-structural infill walls.

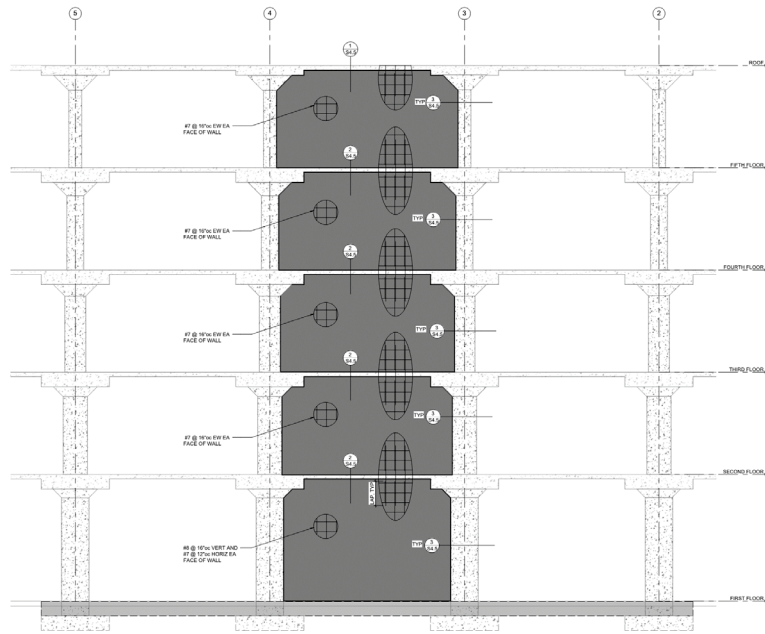
The gravity system consists of 5 ½-inch reinforced concrete flat slabs at floors 2 to 5 and 6-inch flat slabs at the roof. Drop caps at the columns are typically 7'-0" square and approximately 9-inches additional thickness. At the second floor, the interior columns are typically 34-inches in diameter and exterior columns are typically 24-inches by 43.5-inches. The interior columns decrease in diameter at the upper floors.

The existing lateral force-resisting system is slab-column moment resisting frame (Concrete Moment Frame, C1), and with new shear walls added, Concrete Shear Walls with Rigid Diaphragms (C2). The retrofit consists of adding shear walls at strategic locations; one in the center of the building for north-south loading, two in the orthogonal direction at the ends of the building to provide shear and plan torsion stability (See Figure B-1 and Figure B-2 below). The studies apply unidirectional loading in the north-south direction which generates essentially no seismic axial load at the shear wall and in the east-west direction to study the effects from significant seismic axial loads (both up and downward).

The single shear wall added in the center of the building for north-south loading has limited seismic axial load and allows for examination of a singular rectangular footing, and then further examination as it is expanded to reach the adjacent columns as shown in Figure B-2. This reduced the number of parameters being studied, and in particular it removed the sensitive seismic axial load component which can have a profound effect on the foundation's behavior. In contrast, the exterior shear walls see significant seismic axial force, both uplift and downforce, making them an ideal case to study the seismic axial effects without noise from other parameters.



**Figure B-2 Foundation plan.**



**Figure B-3 Retrofit wall and existing structure elevation (3 of 5 bays shown). The grey grade beam and shear wall were added in the retrofit.**

The building is located in a high seismic region and would be classified as Risk Category 2 per the 2019 IBC.

## B.2.2 Soil Conditions

A geotechnical investigation was performed on the site. The soil consists of medium stiff clayey fill underlain by stiff to very stiff clay and claystone bedrock. New and existing footings are founded on the claystone bedrock with a  $N_{60}$  (penetration blow count corrected to an equivalent hammer energy efficiency of 60%) equal to 25 per the geotechnical engineer. The initial shear modulus is calculated per ASCE/SEI 41-17 § 8.4.2.2, and the effective shear modulus is determined based on the ratio in ASCE/SEI 41-17 Table 8-2.

## B.2.3 General Modeling Assumptions

### B.2.3.1 ANALYSIS MODEL

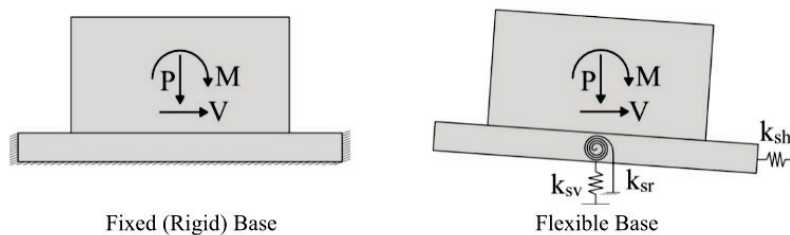
The finite element analysis program, ETABS by CSI, is widely used by the engineering community and is the analysis platform in this case study. The analysis model is three-dimensional for all cases and consists of analysis objects including joints, frames, and areas. ETABS automatically converts the object-based model into an element-based model in the analysis. The element-based model consists of finite elements and joints with lumped joint mass.

Where nonlinear characteristics are included in the analysis, lumped plasticity, user-defined hinge properties are input in ETABS and assigned to frame elements. Fiber modeling is not utilized in this

analysis as the nonlinear aspects of the elements are adequately captured by the nonlinear hinges applied to frame elements, though it is recognized that modelling to the frame elements at the center of the wall, as opposed to the neutral axis, does somewhat underestimate superstructure demands. Walls and slabs that are typically defined as shell elements in linear models are defined as frame elements in the nonlinear models for assignment of frame hinges.

When analyzed with a fixed-base assumptions, the base of each column (including each end of shear walls) are restrained against translation and rotation which is consistent with the foundation details. This assumption was compared to a model with base of the columns pinned (base of columns restrained against translation but not rotation). The fundamental period of the two models was within 5% of each other, indicating that, for this building, the column base fixity does not have a large effect on overall building response. However, the pinned-base analysis underestimates the strength and stiffness of the column frames, so the fixed base analysis is utilized for this case study.

In the analysis models with foundation components explicitly modeled (flexible base), overturning action on the soil is modeled as either a single rotational spring or coupled axial springs as discussed later.



**Figure B-4 Structure boundary conditions from FEMA P-2008.**

Multiple analysis models of the same building were used to analyze the different ASCE/SEI 41-17 foundation modeling provisions. Analysis models for all hypotheses are outlined in Table B-2. Each hypothesis (described in Section B.3) will reference the models utilized. These models were developed to cover multiple foundation modeling options in ASCE 41: linear and nonlinear static procedures, and fixed base and flexible base, Method 1 springs. Load cases studied vary by hypothesis.

Every analysis model is a 3-dimensional ETABS model with the following attributes:

- Unidirectional loading in the North-South direction (except for the design of the East-West retrofit foundation as described in Section B.6)
- Expected material properties used in each model are  $f'_{ce} = 4.8$  ksi and  $f_{ye} = 52.0$  ksi.
- The concrete effective stiffnesses are cracked properties per ASCE/SEI 41-17 Table 10.5

Rigid diaphragms, though slab and column frame actions are included. Accidental torsion is not investigated within these analyses except where bi-directional cases are noted. The building plan is symmetric in the north-south direction with the retrofit wall at the plan center of the building and thus torsional response is ignored for these investigations. The retrofit in the east-west direction provides walls on the west side on the north and south ends of the structure as shown in Figure B-2 and for the bi-directional analysis at these footings, the models include a 5% accidental torsion (ASCE 41-17 §7.2.3.2.1 & ASCE/SEI 7-10 §12.8.4.2).

**Table B-2 Analysis Models Utilized in the Case Study**

	<b>Model A</b>	<b>Model B</b>	<b>Model B.1</b>	<b>Model C</b>	<b>Model D</b>
Analysis Procedure	LSP (§ 7.4.1.3)	LSP (§ 7.4.1.3)	LSP (§ 7.4.1.3)	NSP (fundamental mode load application)	NSP (fundamental mode load application)
Soil Springs	N/A (Method 1 Fixed Base)	Method 1 linear springs with upper and lower bound stiffness values	Method 3, type varies, see Section B.3.6: Hypothesis 6	N/A (Fixed Base)	Method 1 nonlinear moment rotation springs compression-only vertical springs with expected values and no uplift capacity
Foundation Retrofit	Retrofit and No Retrofit cases modeled	Modeled	Modeled	Modeled	Modeled
Columns	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements with Nonlinear Hinges Top and Bottom	Elastic Frame Elements with Nonlinear Hinges Top and Bottom
Structural Slab	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements with Nonlinear Hinges Each End	Elastic Frame Elements with Nonlinear Hinges Each End
Shear Wall	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements with Nonlinear Flexural Hinges Top and Bottom and Shear Hinge at Center	Elastic Frame Elements with Nonlinear Flexural Hinges Top and Bottom and Shear Hinge at Center

A significant amount of time was invested ensuring accurate modeling and that the code interpretations used in this study were in keeping with the industry standard of practice. A peer review was completed of the modeling to ensure that the case study began with a highly reliable model. In creating the models, additional background foundation concepts in ASCE/SEI 41-17 were investigated including the expected bearing capacities, bearing capacity bounding as well as the expected restoring dead load, discussed in Sections B.4 and B.5 of this report. Understanding the interpretations of these concepts and their effects on results is necessary so that these factors do not convolute results aimed at examining other topics.

Note that when completing ASCE/SEI 7-10 analyses for comparison, the results can vary significantly due to the use of different redundancy factor  $\rho$  and torsional effects. For this study, the redundancy factor,  $\rho$ , is taken as 1.0 since the existing column and slab frames provide redundancy; however,  $\rho = 1.3$  could also be considered technically accurate. For this case, the ASCE/SEI 7-10 results would have been more similar to the ASCE/SEI 41-17 linear results had  $\rho = 1.3$  been used. Simplifications and assumptions such as these redundancy factors and torsional effects can change results significantly. Therefore, it is important to use engineering judgement when comparing and drawing conclusions about these analysis results.

To represent a realistic force level in a high seismicity area, these studies utilized a site with accelerations: for ASCE/SEI 7-10:  $S_{DS} = 1.0$  g and  $S_{D1} = 0.6$ , and for ASCE/SEI 41-17:  $S_{XS} = 1.0$ g and  $S_{X1} = 0.6$ g (BSE-1E), and  $S_{XS} = 1.5$ g and  $S_{X1} = 1.0$ g (BSE-2E). The vertical distribution of forces is derived from the base shear calculations using ASCE/SEI 7 and the pseudo seismic force demands using ASCE/SEI 41.

### **B.2.3.2 ANALYSIS PROCEDURES**

Linear and nonlinear analysis procedures from ASCE/SEI 41-17 Chapter 7 were utilized in this study. Many of the hypotheses compare linear results from the linear static procedure (LSP) to nonlinear results from the nonlinear static procedure (NSP). In these comparisons, the nonlinear results are utilized as the benchmark for calibration with linear procedures. This study assumes that the results of nonlinear analyses are reasonable for comparison with the results of the hypotheses related to linear analyses. Additionally, though fundamentally difficult to compare, a parallel assessment using ASCE/SEI 7-10 provisions was also performed. Note: Some aspects of building may not conform to the requirements of current code but are used for illustrative purposes to highlight use of the foundation provisions in ASCE/SEI 41-17 and compare outcomes with the provisions for new buildings using ASCE/SEI 7. As with this type of parametric study, engineering judgment is required when generating, reviewing, and drawing recommendations from the results.

### **B.2.3.3 GENERAL OVERVIEW OF ASCE/SEI 41-17 CHAPTER 8 SOIL MODELING METHODOLOGIES**

There are three “methods” for foundation modeling in ASCE/SEI 41-17. These different methods were utilized in the case study. There are two methodologies (fixed-base and flexible-base) included within Method 1. Method 1 fixed base models do not have soil springs and are restrained against

translation and global rotation at the soil-structure interface. Acceptance criteria for the fixed base models are per ASCE/SEI 41-17 §8.4.2.3.2.1 which includes provisions for soil bearing and overturning stability of individual foundation elements. Method 1 flexible base models use uncoupled moment, shear, and axial springs to model rigid foundations such that the moment and shear behaviors are independent of the axial load. Method 1 soil springs can be utilized for both linear and non-linear analysis methods but is only applicable to footings assumed rigid compared to the soil. Method 2 is also for shallow footings considered rigid compared to the soil but can only be utilized with nonlinear analysis methods. Method 2 provides an alternative approach for rigid footings that uses a bed of nonlinear springs that accounts for coupling between vertical loads and moment. Method 2 is the preferred approach when there is significant variation in axial load. The moment-rotation and vertical load-deformation characteristics are modeled as a beam on a nonlinear Winkler foundation with stiffer vertical springs at the end regions of the foundation to allow for tuning of the springs to approximately match the elastic vertical and rotational stiffness provided in Method 1. This Method 2, if applied in the NDP, may also be used to account for settlement and permanent deformations, though determination of those requires complicated combinations of plastic and gap elements in parallel and in series (Harden et al., 2005). Method 3 is the only method allowed for shallow foundations where the structural component (footing) is flexible (not rigid) relative to soil, and it is only applicable to nonlinear analysis procedures. Method 3 uses a similar methodology to Method 2 with Winkler springs beneath the foundations, except that a uniform distribution of soil stiffness and strength is applied. The differences between the different methods can be significant since Method 2 is meant to be calibrated with Method 1, which is based on low-strain, elastic soil response and assumes the soil remains in contact with the footing, whereas Method 3 can include geometric nonlinearity where the soil separates from the footing.

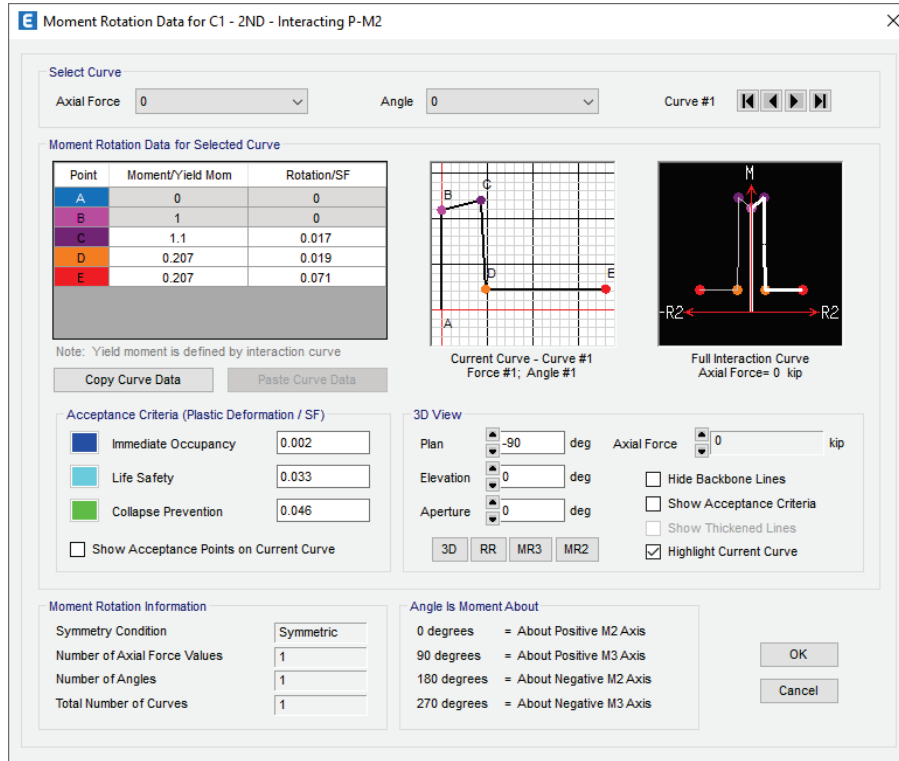
Deep foundation provisions are not investigated as part of this case study.

#### **B.2.3.4 SUPERSTRUCTURE PROPERTIES**

Concrete modeling and analysis procedures from ASCE/SEI 41-17 Chapter 10 are followed for the ETABS model superstructure.

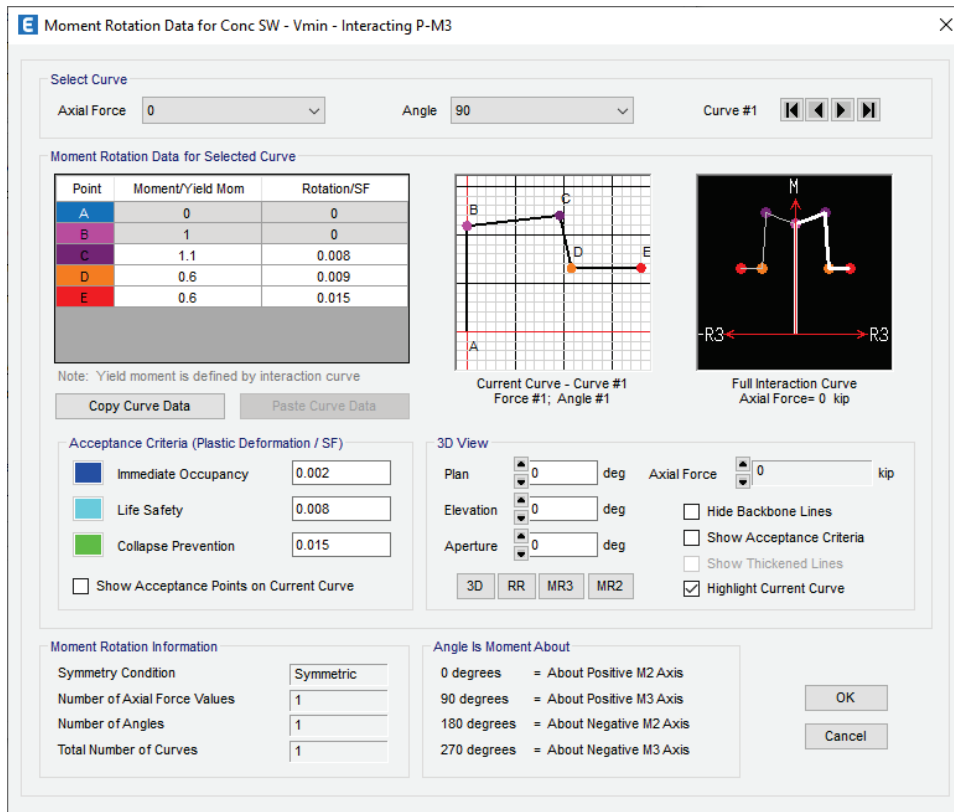
- Material Properties (ASCE/SEI 41-17 § 10.2) – Materials properties utilized in the analysis model are expected strengths based on usual data collection. Existing drawings of the building were available for review, although they did not specify design strengths of materials. Materials testing was performed to determine expected strengths.
- Modeling and Design (ASCE/SEI 41-17 § 10.3.1) – Elastic component effective stiffnesses are determined per ASCE/SEI 41-17 Table 10-5. When nonlinear models are utilized, hinge properties are defined per ASCE/SEI 41-17 Figure 10-1. Nonlinear hinges are assigned at appropriate locations on frame elements within the ETABS model where nonlinear behavior is expected.
  - For concrete column frame elements, a nonlinear P-M hinge is defined at the top of the column at the base of the capital, and at the base of the column at the connection to the

floor slab. For each hinge, the slope from A to B in the load-deformation relation is the same as defined for the linear models. The slope from B to C is taken as 10% of the initial slope. The deformation or rotation that defines point C is defined by other tables in ASCE/SEI 41-17 Chapter 10. An example column P-M hinge is included in Figure B-5.

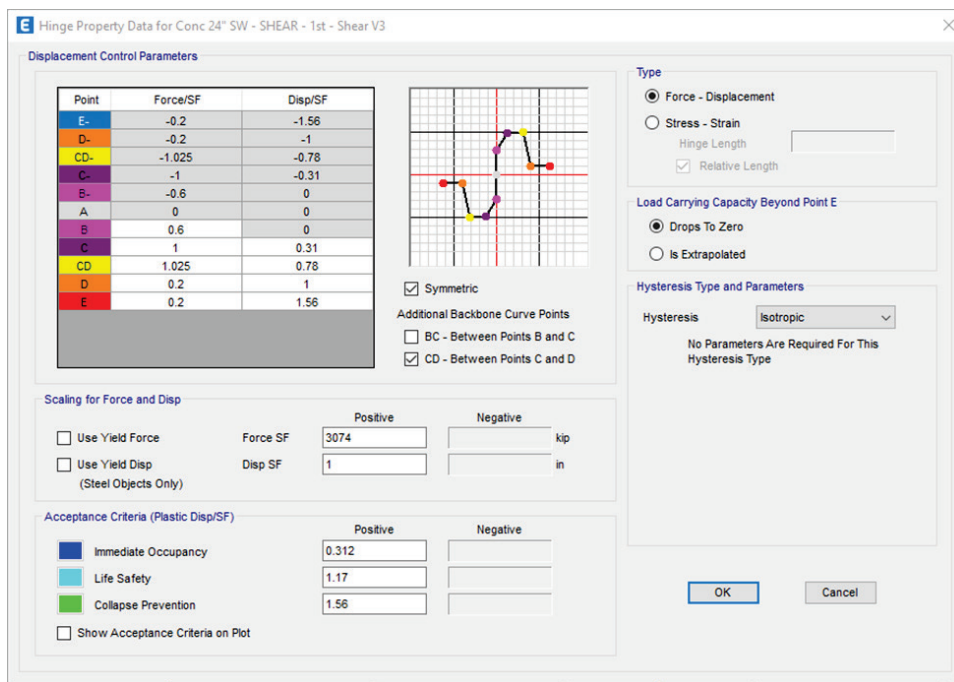


**Figure B-5 Sample column P-M hinge property.**

- Concrete shear walls are modeled as frame elements in all analysis models with the appropriate effective elastic stiffness values (ASCE/SEI 41-17 Table 10-5). For nonlinear analysis, the concrete shear wall frame elements have flexural hinges at top and bottom and a shear hinge at the center per ASCE/SEI 41-17 Tables 10-19 and 10-20. Note that the shear wall flexural hinges are moment only, not P-M hinges. See example shear wall hinge properties in Figure B-6 and Figure B-7.

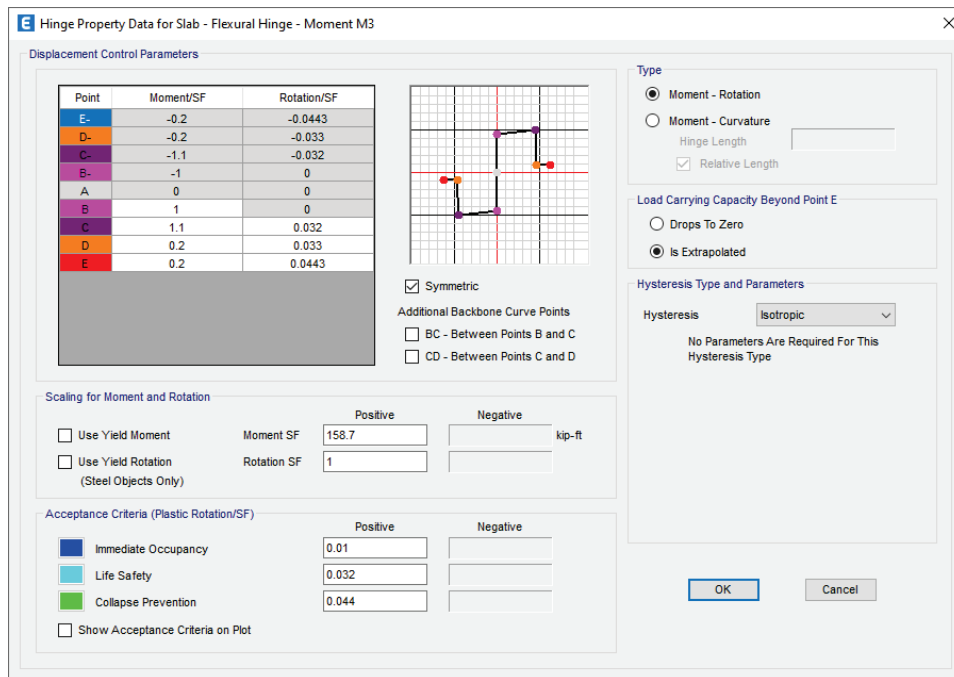


**Figure B-6** Sample concrete shear wall flexural hinge property.



**Figure B-7** Sample concrete shear wall shear hinge property.

- The concrete floor slab at each level is modeled with frame elements to capture frame action with the columns, and in-plane diaphragm action is modelled by slaving coordinates at each floor and roof together to form a rigid diaphragm. The ground floor slab-on-grade is omitted from all analysis models as it does not contribute to the behavior of the structure. In the nonlinear models, the effective beam width model per ASCE/SEI 41-17 § 10.4.4.1 is utilized to model the slab and the drop panels, both as frame elements that contribute to the moment resisting action of the frame. Hinges per ASCE/SEI 41-17 Table 10-15 are assigned to the slab frame elements at the edge of the drop panels in the nonlinear models. Sample hinge properties are included in Figure B-8 and Figure B-9. Rigid diaphragm constraints are applied for both linear and nonlinear analysis models.



**Figure B-8 Sample flexural slab hinge property.**

**Hinge Property Data for Slab - Torsion Hinge - Torsion T**

Displacement Control Parameters

Point	Moment/SF	Rotation/SF
E-	-0.2	-0.0443
D-	-0.2	-0.033
C-	-1.1	-0.032
B-	-1	0
A	0	0
B	1	0
C	1.1	0.032
D	0.2	0.033
E	0.2	0.0443

Symmetric

Additional Backbone Curve Points

BC - Between Points B and C

CD - Between Points C and D

Scaling for Moment and Rotation

Use Yield Moment      Moment SF:             kip-ft

Use Yield Rotation      Rotation SF:      

(Steel Objects Only)

Acceptance Criteria (Plastic Rotation/SF)

Immediate Occupancy      Positive:       Negative:

Life Safety      Positive:       Negative:

Collapse Prevention      Positive:       Negative:

Show Acceptance Criteria on Plot

Type

Moment - Rotation

Moment - Curvature

Hinge Length:

Relative Length

Load Carrying Capacity Beyond Point E

Drops To Zero

Is Extrapolated

Hysteresis Type and Parameters

Hysteresis:

No Parameters Are Required For This Hysteresis Type

**Figure B-9 Sample torsional slab hinge property.**

Exterior and interior staircases are not included in the analysis model. The elevator shaft is modeled as an opening in the slab at each floor level. The stair and shaft walls are not included in the model as they are nonstructural hollow clay tile walls that are removed as part of the retrofit.

## B.2.4 Building Retrofit

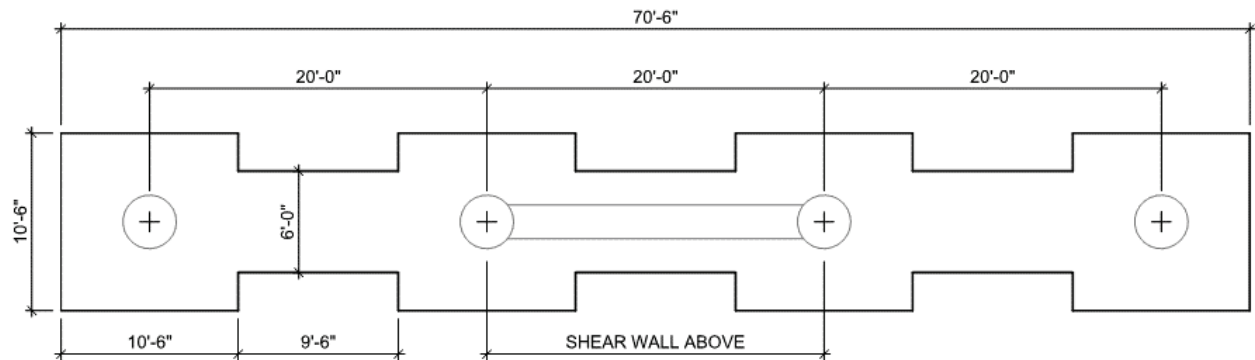
The investigations completed used ASCE/SEI 41-17 to examine the existing footings, but also to examine possible foundation retrofits in conjunction with the new proposed shear walls.

### B.2.4.1 PROPOSED FOUNDATION RETROFIT GEOMETRY (NORTH – SOUTH DIRECTION)

The existing foundations were evaluated using ASCE/SEI 41-17 linear static procedures with a fixed-base assumption, as well as ASCE/SEI 7-10, equivalent lateral force methodology with a fixed-base assumption for their capacity to support overturning forces due to lateral loading on the new concrete shear wall at the center of the building. These studies used an acceleration level  $S_a$  of  $1g$  to represent a realistic force level in a high seismicity area as well as site specific accelerations at a high seismicity site. These accelerations were also scaled to identify at what accelerations the foundation acceptance criteria and allowable bearing pressure are met. While further discussion of these studies is included in subsequent chapters, they all indicated that the existing foundation was not adequate to support overturning forces due to lateral loading on the new concrete shear wall. Therefore, new concrete foundations were proposed as shown in Figure B-2.

The retrofit footing at the central wall connects the existing pad footings at adjacent columns together to create one continuous footing. This engages additional load that reduces the uplift

at the foundation due to the lateral loading on the new shear wall. The retrofit requires continuous reinforcement through the existing footings for flexure. The proposed retrofit plan layout is shown in Figure B-10 with geometric properties in Table B-3. To simplify the analysis, the retrofit footing is approximated as a rectangular footing with an average footing width to account for the variations in footing width along its length.



**Figure B-10** Retrofit footing plan layout with dimensions.

**Table B-3** Retrofit Footing Geometric Properties

Retrofit Footing Geometric Properties	
Footing Area ( $A_f$ )	612 ft <sup>2</sup>
Average Footing Width (B)	8.7 ft

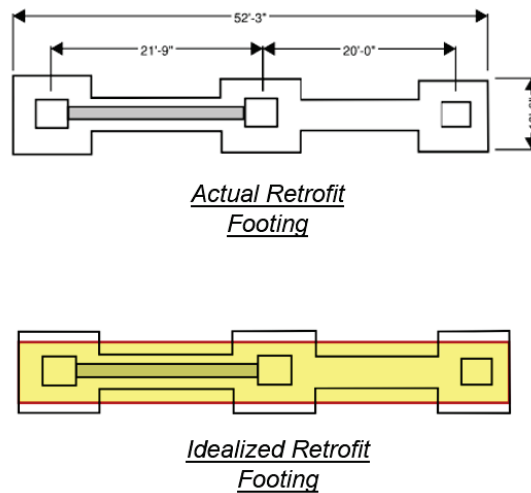
The footing retrofit was designed utilizing ASCE/SEI 7-10 provisions assuming the new footing is rigid compared to the soil and an elastic, triangular soil bearing pressure distribution. The retrofit footing was designed to meet bearing pressure requirements and for structural footing strength. It was determined that a 6-foot-wide by 4-foot-deep footing 6-foot-wide by 4-foot-deep footing with (30) #11 bars top and bottom is adequate for the design loads. This footing was then used for comparisons with ASCE/SEI 41-17 foundation designs. ASCE/SEI 41-17. It is noted that a Method 3 approach would be more appropriate for this footing configuration. However, as practice may treat this as rigid, we are exploring Methods 1 and 2 for comparison purposes.

#### B.2.4.2 PROPOSED FOUNDATION RETROFIT GEOMETRY (EAST – WEST DIRECTION)

Based on calculations performed as part of Section B.6, the existing foundation is not adequate to support overturning forces due to lateral loading on the new concrete shear walls.

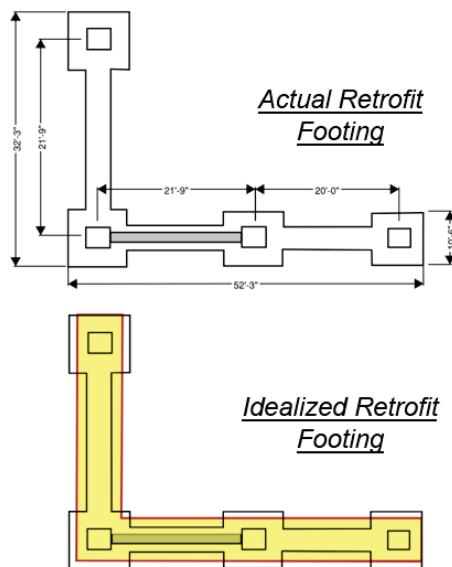
In the east-west direction, the proposed retrofit footing extends one bay beyond the shear wall towards the middle of the building. It connects three existing pad footings together to create one continuous footing. This engages additional dead load that reduces the uplift at the foundation. The retrofit requires continuous reinforcement through the existing footings for flexure. The proposed

retrofit plan layout is shown in Figure B-11. The retrofit footing was idealized as rectangular rather than three 10'-6" square footings connected by a narrower continuous footing.



**Figure B-11** Rectangular retrofit footing plan layout in east-west direction.

However, analysis indicated that this retrofit was not acceptable. Therefore, an alternative retrofit footing was also investigated in which an L-shaped footing extends one bay perpendicular to the retrofit shear wall as shown in Figure B-12. This engages additional dead load. Further discussion of these footing designs can be found in Section B.6.



**Figure B-12** L-shaped retrofit footing plan layout in east-west direction.

## B.3 Investigation Hypotheses

Hypotheses were developed that pose answers to questions arising from the highlighted issues with the ASCE/SEI 41-17 foundation guidelines, and served to guide the case study investigations. Each hypothesis is set to examine a technical point within the chapter that the working group sees as requiring clarification. Each hypothesis attempts to isolate one aspect to be quantitatively investigated. In some hypotheses, it is difficult to study one provision without understanding the implications of other assumptions. These are investigated within each hypothesis as required. This section summarizes the hypotheses and the resulting conclusions and recommendations at a high level. Subsequent sections (B.4 through B.12) provide more in-depth results and discussions of topics of interest that arose throughout the hypothesis work.

While the general results of each hypothesis are discussed within this section, the detailed results of the technical studies are organized in later sections within the framework of ASCE/SEI 41-17 Chapter 8.

### B.3.1 Hypothesis 1: Acceptance Criteria for Fixed Base Condition

Hypothesis 1 states that the use of the Method 1 linear, fixed base foundation approach with overturning action  $m$ -factors in accordance with ASCE/SEI 41-17 § 8.4.2.3.2.1 provides reasonable assurance that the overturning stability and forces are accurate and slightly conservative and the fixed base model may be used to evaluate superstructure components.

#### B.3.1.1 HYPOTHESIS 1 PROCESS

LSP (linear static procedure) was performed and compared to NSP (nonlinear static procedure) analysis. The acceptance ratios of the linear and nonlinear analyses were compared to test the hypothesis. For both the LSP and the NSP, the stiffnesses used were the best estimate expected values; the upper and lower bound stiffnesses were not modeled. The following analysis were done using Model A as described in Table B-2.

A pseudo lateral force was applied at every floor for the following base shear force levels and scenarios (linear load cases). For each case, the foundation soil acceptance ratios and foundation structure acceptance ratios were recorded (for retrofit footing cases).

1. Using ASCE/SEI 41-17 with an acceleration level  $S_a$  of 1g to represent a realistic force level in a high seismicity area without a foundation retrofit.
2. Using ASCE/SEI 41-17, scale the base shear until the overturning compression action demand balances with the capacity when an  $m$ -factor for CP of 4.0 is applied without a foundation retrofit using compression acceptance criteria per ASCE/SEI 41-17 § 8.4.2.3.2 ( $S_a = 0.62g$ ).
3. Using ASCE/SEI 7-10 with an acceleration level  $S_a$  of 1g, with  $R = 6$  (special concrete shear wall, though one could have used  $R = 5$ ), to represent a site-specific force level in a high seismicity area without a foundation retrofit.

4. Using ASCE/SEI 7-10, scale the base shear until the overturning compression action demand balances with the capacity without a foundation retrofit ( $S_a = 0.18g$ )
5. Using ASCE/SEI 41-17 with a site-specific acceleration level  $S_a$  for the same location as Linear Case 3 without a foundation retrofit.

All of the above cases indicated that a shear wall retrofit without a foundation retrofit is unacceptable for this building. The final three cases include a foundation retrofit.

6. Design of a retrofit foundation for site-specific loading of Linear Case 3 using ASCE/SEI 7-10.
7. Evaluation of foundation designed in Linear Case 6 using ASCE/SEI 41-17 and site-specific loading of Linear Case 5.
8. Evaluation of retrofit footing from Linear Case 6 with LSP Method 1 foundation springs.

Following completion of the LSP analysis cases, NSP was utilized for comparison and benchmark with Model D. Model C was also investigated to determine the superstructure behavior without displacement at the foundation/soil interface.

These results from Model C and Model D were recorded for comparison with Model A results:

1. Column demands (all actions) as well as the associated capacities
2. Slab demands (flexure action) as well as the associated capacities
3. Total base shear
4. Shear force action in shear wall
5. Soil Acceptance Criteria

Primarily, acceptance ratios are used to compare the LSP and NSP results:

- $Q_{UD} / kmQ_{CE}$  (ASCE/SEI 41-17 LSP Results)
- Target Displacement Rotation / Allowable Rotation (ASCE/SEI 41-17 NSP Results)
- Demand / Capacity (ASCE/SEI 7-10 Results)

### **B.3.1.2 HYPOTHESIS 1 RESULTS**

Table B-4 summarizes the findings from the first four linear analysis cases investigated. Based on the analysis results, the existing foundations are not adequate with a typical high seismicity site design acceleration of  $1g$  for both ASCE/SEI 41-17 and ASCE/SEI 7-10 analyses.

**Table B-4 Summary of Acceptance Ratios and DCRs for Linear Cases 1, 2, 3, and 4**

	ASCE/SEI 41-17			ASCE/SEI 7-10	
	Case 1	Case 2		Case 3	Case 4
	Realistic Force Level Sa = 1g	Acceptable w/o footing retrofit Sa = 0.30g	CP m-factor & ASCE/SEI 41-17 Section	Realistic Force Level Sa = 1g	Acceptable w/o footing retrofit Sa = 0.18g
Uplift acceptance ratio or DCR (Conventional Tributary Area Restoring Dead Load)	2.1	0.6	8 (8.4.2.3.2.1)	2.4	0.4
Uplift acceptance ratio or DCR (Capacity Based Design Restoring Dead Load)	1.6	0.5	8 (8.4.2.3.2.1)	0.9	0.1
Bearing Pressure acceptance ratio or DCR	3.3	1.0	4 (8.4.2.3.2.1)	2.1	1.0
Overall Overturning Stability DCR	1.7	0.5	10 (7.2.8.1)	3.2	0.5
Outcome	NG	OK		NG	OK

It was only when the seismic acceleration with ASCE/SEI 41-17 methodology was reduced to 0.30g that the existing footing was sufficient for the seismic loading. Similarly, an acceleration of 0.18g was required with ASCE/SEI 7-10. These findings indicate that in high seismic regions, a shear wall retrofit without a foundation retrofit is unlikely to be acceptable. While this is obvious for most readers, there has been a repeated theme discussed by some that “foundation retrofit is uneconomical to perform and that buildings don’t tip over”. This study numerically proves that foundation retrofit is required in order to ensure the shear wall performs as intended and does protect the existing components; therefore, the building is able to meet the targeted performance level.

If a foundation retrofit was not provided, the building would not meet the target performance objective. In that case, the retrofit would fall in the category of a Partial Retrofit in accordance with ASCE/SEI 41-17 Section 2.2.5. These limitations should be relayed to the stakeholders if this partial retrofit approach is taken.

These results were confirmed with an additional study at a specific high seismicity site (Linear Case 5). The ASCE/SEI 41-17 analysis was performed using the BSE-2E Seismic Hazard Level with

Collapse Prevention acceptance criteria, which is consistent with the ASCE/SEI 41-17 Basic Performance Objective for Existing Buildings (BPOE). The site-specific seismic parameters were  $S_{xs} = 1.5g$  (ASCE/SEI 41-17) and  $S_{DS} = 1.0 g$  (ASCE/SEI 7-1). The results for the ASCE/SEI 41-17 are described in Table B-5 and Table B-6.

**Table B-5 Summary of ASCE/SEI 41-17 Acceptance Ratios for Linear Case 5 and ASCE/SEI 7-10 DCRs for Linear Case 5 and 3**

	ASCE/SEI 41-17	ASCE/SEI 7-10
	Acceptance Ratio	DCR
Uplift (Conventional Tributary Area Restoring Dead Load)	3.0	2.4
Uplift (Capacity Based Design Restoring Dead Load)	2.1	0.9
Bearing Pressure	4.4	2.1
Overall Overturning Stability	2.2	3.2
Outcome	NG	NG

As expected, for this specific high-seismic site, the shear wall retrofit without a foundation retrofit is unacceptable and would not meet the target performance level. Therefore, further cases were analyzed to design and evaluate a retrofit footing. Alternatively, in practice, one could proceed on a partial retrofit basis.

The footing retrofit was designed utilizing ASCE/SEI 7-10 assuming the new footing is rigid compared to the soil (Linear Case 6). The resulting footing is described in Section B.2.4.1. For Linear Case 7, this footing was then used for evaluation with ASCE/SEI 41-17 using the same site-specific seismic hazard and performance level as Linear Case 5. This footing does not quite meet the acceptance criteria of ASCE/SEI 41-17, see Table B-6 below.

**Table B-6 Linear Case 7 Results: Summary of Footing Retrofit Acceptance Ratios and DCRs**

	ASCE/SEI 41-17			ASCE/SEI 7-10	
	Section	CP m-factor	Acceptance Ratio	Section	DCR
LSP, Bearing Pressure	8.4.2.3.2.1	4	1.32	12.13.4	0.95
LSP, Uplift	8.4.2.3.2.1	8	0.71	12.13.4	0.56
LSP, Overall Overturning	7.2.8.1	10	0.51	12.8.5	0.75
Outcome			NG		OK

However, note that the bearing pressure acceptance ratio for ASCE/SEI 41-17 is 1.32. As the ASCE/SEI 7-10 results are very dependent on the system Response Modification Coefficient,  $R$ , and the redundancy factor  $\rho$ , these ASCE/SEI 7-10 results could align with the ASCE/SEI 41-17 acceptance ratio of 1.32 had for example  $\rho = 1.3$  been used. Further, the footing strength is not adequate based on an ASCE/SEI 41-17 force-controlled analysis, with the acceptance ratio of 4.9, see Table B-41. It does meet ASCE/SEI 41-17 if the footing flexural action is treated as deformation controlled, though that is not permitted in accordance with ASCE/SEI 41-17 Section 10.12.3.

Following these analyses, the retrofit footing based on the ASCE/SEI 7-10 design was evaluated for site specific loading using the LSP with Method 1 foundation springs per ASCE/SEI 41-17 Figure 8-2. Spring derivation and methodology discussion can be found in Section B.8.3.1. In short, the retrofit footing is treated as a rigid body for the Method 1 spring stiffness derivations. Method 1 uses uncoupled moment and axial springs to model rigid foundations such that moment and shear behaviors are independent of axial load. Shear (sliding) springs may also be used. In this case, and for all methods in this investigation, lateral moment is restrained within the analysis model. The results of this flexible base analysis (Linear Case 8) are compared to the results of the fixed base study (Linear Case 7) in Table B-7.

**Table B-7 Comparison of Fixed Base (Linear Case 7) and Flexible Base Method 1 (Linear-Case 8) Acceptance Ratios**

	Fixed Base	Flexible Base
LSP, Bearing Pressure	1.32	1.16
LSP, Uplift	0.71	0.68
LSP, Overall Overturning Stability	0.51	0.50
Outcome	NG	NG

These analyses show similar results, with the flexible base ratio slightly less than that of the fixed base model. Following completion of the LSP analysis, NSP was utilized as comparison and benchmark for these studies. Multiple soil spring methodologies were also examined as a part of this NSP analyses, see Section B.10.3 for these derivations. The nonlinear superstructure hinge behavior was modeled in accordance with ASCE/SEI 41-17 Chapter 10. Following are the NSP cases evaluated and brief findings associated with each. Complete analysis findings can be found in Section B.10. Note that the acceptance ratios discussed in these findings are the rotations at the base of the shear wall at the target displacement compared to the allowable footing rotation from ASCE/SEI 41-17 Table 8-4.

1. Fixed base NSP as a baseline for comparison
  - The calculated target displacement is equal to 5.3 inches. The fundamental period of the structure is 0.45 seconds, which matches the LSP analysis.

2. Method 1 soil springs NSP – force-controlled foundation design

- The flexural demand at the target displacement was used to assess the footing as force-controlled with lower-bound strength as specified in ASCE/SEI 41-17 § 10.12.3. The footing flexural action has an acceptance ratio of 1.37, so it is not acceptable and would require additional strength with this analysis approach.
- ASCE/SEI 41-17 provides guidance in the commentary (§ C8.4.2.1) for determining when a foundation is rigid compared to soil by comparing the foundation stiffness to the soil stiffness in Equation C8-1. Based on this definition, the footing is not rigid compared to the soil; therefore, Method 1 is not applicable. Therefore, alternative Method soil springs are explored.

3. Method 2 (non-tuned) soil springs NSP – force-controlled foundation design

- ASCE/SEI 41-17 §8.4.2.4.1 and C8.4.2.4.1 state that Method 2 springs should be tuned to match the stiffness of Method 1 and provide a reference to Gajan et al (2010). In order to assess the affect of tuning or not tuning Method 2 springs, this model assumes no tuning and is compared to the subsequent model which does tune the springs.
- The footing rotation at the target displacement meets the acceptance criteria.
- The flexural action in the footing is also assessed at the target displacement to verify the footing strength. The footing is undersized for the force-controlled flexural demands, with an acceptance ratio of 2.29.
- During this investigation, it was found that the acceptance criteria in ASCE/SEI 41-17 Table 8-4 is highly dependent on the  $A_c/A_f$  factor and the  $b/L_c$  of the footing. The allowable rotation is highly sensitive to the footing area, and in this case the footing width, since the length is constrained. When the footing width doubles, the allowable rotation increases by a factor of 5.7, which demonstrates that the calculated rotation is highly sensitive. See further discussion in Section B.10.5.4.

4. Method 2 (tuned) soil springs NSP – force-controlled foundation design

- This method utilized tuned springs per Gajan et. Al. instead of ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991), see Section B.1.3.3. These revised springs negligibly change the response of the structure from Method 1 to tuned Method 2 as indicated in Section B.1.5.10, However, as noted in Section 5.8.3.4, using the  $K_{50}$  stiffness had a significant difference, and led to a more flexible system.
- The acceptance ratio for the footing rotation is similar to the initial Method 2 results. The flexural foundation acceptance ratio is 1.69.

5. Method 3 soil springs NSP – force-controlled foundation design

- The footing that was designed using ASCE/SEI 7-10 is then evaluated for force-controlled flexure in accordance with ASCE/SEI 41-17. and the structural footing design is not acceptable with an acceptance ratio of 2.39.

6. Method 3 soil springs NSP – deformation-controlled foundation design

- Although foundations are typically required to be evaluated as force-controlled in accordance with ASCE/SEI 41-17 § 10.12.3, the nonlinear modeling and acceptance criteria provisions for concrete beams within ASCE/SEI 41-17 Chapter 10 are applied to the foundation structure for this case, which is commonly done in practice. The ASCE/SEI 7-10 designed retrofit footing has flexural hinges assigned to each end of the footing beams between the existing footings. The hinges are assessed in accordance with the provisions of ASCE/SEI 41-17 Chapter 10 to the Collapse Prevention acceptance criteria.
- The fundamental period of this model is 0.63 seconds, which is more than the LSP Method 1 (0.54 seconds) and LSP fixed based procedures (0.45 seconds). The target displacement is 10.7 inches.
- The acceptance criteria per ASCE/SEI 41-17 Table 8-4 is dependent on the  $L_c$  (defined as the length of the contact area and equal to  $A_c/b$ ). In this case, that is taken as the full length of the footing. Judgement may be required in other foundation configurations.
- The acceptance ratio for the footing at the target displacement is 0.43. Therefore, the retrofit footing design is acceptable based on a deformation-controlled foundation design.

7. Method 3 soil springs NSP – deformation-controlled, acceptance criteria at inflection points

- As discussed in FEMA P-2006 § 5.7.6.1, a flexible footing could be assessed by evaluating individual sections separated at inflection points. For this case, the acceptance criteria is recalculated for a similar condition to NSP Case 6 but with the soil acceptance criteria evaluated with  $L_c$  defined for three individual segments based on flexural inflection point locations.
- All of the segments meet their acceptance criteria (acceptance ratios are 0.49, 0.34, and 0.20). The highest loaded segment also has the lowest rotation as the beam hinge adjacent to it is yielding which reduces the rotation demand.

A summary of all of the foundation acceptance criteria cases (excluding those without the retrofit footing) are included in Table B-41 in Section B.10.3. The only ASCE/SEI 41-17 analysis cases where the structural footing is acceptable are the cases where the footing is evaluated as deformation-controlled. The force-controlled design of footing is overly conservative when compared to the ASCE/SEI 7-10 results. In general, these NSP results were less conservative than the LSP results (except the fixed base case), which is expected.

### **B.3.1.2 HYPOTHESIS 1 CONCLUSIONS**

In conclusion, these analyses found:

- Retrofit of foundation is required with superstructure retrofit at high seismic sites regardless of the ASCE/SEI 7-10 or ASCE/SEI 41-17 approach, unless a partial retrofit goal is selected.
- It is difficult to compare ASCE/SEI 7-10 to ASCE/SEI 41-17 results due to fundamentally different approaches. The ASCE/SEI 7-10 approach is sensitive to the Response Modification Factor and the Redundancy Factor such that ASCE/SEI 7-10 solutions can vary significantly as a result.
- Further guidance is recommended to determine when the foundation is rigid compared to the soil, or if the superstructure is sensitive to foundation deformations. The notion of relative rigidity of the footing to the soil is only applicable to small strains, whereas large strains will likely lead to soil separation from footing as well as localized yielding of the soil and footing,
- The force-controlled design of the footing is overly conservative when compared to the ASCE/SEI 7-10 results, as well as based on judgement. See Section B.9.2.2.
- Guidance on the definition of  $L_c$  in ASCE/SEI 41-17 Table 8-4 is recommended. It is unclear if it should always be the full length of the footing or taken between inflection points at flexible footings as discussed in FEMA P-2006.
- Clarity is recommended in terms of stiffness derivation, capacity calculation and acceptance criteria definition for each of the Methods. Currently all the acceptance criteria are in ASCE/SEI 41-17 Table 8-4, which also includes the modeling parameters for Method 1 and 2. See Section B.8 for further discussion.
- The acceptance criteria in ASCE/SEI 41-17 Table 8-4 are sensitive to the axial load on the footing and the footing geometry. Slight changes to load or the footing dimensions significantly change the acceptance criteria. Investigation into this is recommended for future development of the acceptance criteria tables.
- Per ASCE/SEI 41-17 Section 8.4.2.3.2, there is an exception for fixed base foundations that states “Where a shallow foundation is subject to a seismic compression force that exceeds three times the gravity load or  $A_c/A$  exceeds 0.6, the foundation overturning demand shall be treated as force controlled...”. However, it was discovered that users were not treating the foundation overturning demand as force-controlled when it fell under this category. Therefore, in providing m-factors for overturning moment actions, the revisions recommended by this committee include that if  $A_c/A_f > 0.4$ , the m-factors are to be calculated from the Table 8-3.

Ultimately, based on the case study results, the use of the linear, fixed base foundation approach with overturning action m-factors in accordance with ASCE/SEI 41-17 § 8.4.2.3.2.1 provides reasonable assurance that the overturning stability and forces are accurate and the fixed base model may be used to evaluate superstructure components.

However, these results indicate that by using a linear fixed base foundation approach, the footing design may be overly conservative resulting in a massive footing, and as happened with this case, a new foundation that meets ASCE/SEI 7-10 may not meet the ASCE/SEI 41-17 acceptance criteria for the target performance objective.

### **B.3.2 Hypothesis 2: Bounding Requirements for LSP (Method 1)**

Hypothesis 2 states that the use of overturning action m-factors in accordance with Table 8-3 provides reasonable assurance that the overturning stability and forces are accurate and, therefore, structural components have reasonable demands from the flexible base model. Also, that this statement is independent of lower and upper bound conditions; i.e. that lower bound stiffness or stiffness derivations that account for separation between the footing and soil, will provide sufficient accuracy and that the upper bound need not be analyzed to obtain reasonable analysis results.

#### **B.3.2.1 HYPOTHESIS 2 PROCESS**

LSP (linear static procedure) was performed and compared to NSP (nonlinear static procedure) analysis. The ASCE/SEI 41-17 design approach was used and compared against the ASCE/SEI 7-10 design. The acceptance ratios of the linear and nonlinear analyses are compared to test the hypothesis. For the LSP, both upper and lower bound stiffness values are evaluated.

The following analyses were completed using Models A and B per Table B-2.

A pseudo lateral force was applied in accordance with ASCE/SEI 41-17 for a specific site with an ASCE/SEI 7-10  $S_{DS} = 1.0g$  to represent a realistic force level in a high seismicity area for the following scenarios (Linear Cases):

1. Fixed Base
2. ASCE/SEI 41-17 Method 1 Linear Soil Springs – upper and lower bound
3.  $K_{50}$  Linear Soil Springs – upper and lower bound.  $K_{50}$  springs are further explained in Section B.8.3.1.

Following completion of the LSP analysis, NSP was utilized for comparison and as a benchmark with Model D per Table B-2. The NSP Model D use Method 1 nonlinear moment rotation springs and compression-only vertical springs with expected values and no uplift capacity. These results from Model D were recorded for comparison with the above Model A and B results:

1. Total base shear
2. Foundation Soil Acceptance Ratios
3. Foundation Structure Acceptance Ratios
4. Maximum vertical deflection in retrofit footing
5. Column demands (all actions) as well as the associated capacities

6. Shear wall demands (all actions)
7. Slab demands (flexure action) as well as the associated capacities
8. Story Drift

As with Hypothesis 1, acceptance ratios are used to compare the LSP and NSP results:

- $Q_{UD} / kmQ_{CE}$  (ASCE/SEI 41-17 LSP Results)
- Target Displacement Rotation / Allowable Rotation (ASCE/SEI 41-17 NSP Results)
- Demand / Capacity (ASCE/SEI 7-10 Results)

### **B.3.2.2 HYPOTHESIS 2 RESULTS**

Brief findings associated with each linear analysis case are included below. Complete analysis findings can be found in Section B.8.

1. Linear Fixed Base: Hypothesis 2's Linear Case 1, is the same site -specific loading examined as Hypothesis 1's Linear Case 7. It utilizes ASCE/SEI 41-17 and a fixed base model that includes the retrofitted foundation. See Section B.8.3.3.1 for more detailed results.
  - The retrofit footing was then evaluated for bearing pressure due to overturning using ASCE/SEI 41-17 Equation 8-10 and the overturning moment capacity is calculated with the upper-bound soil bearing capacity in accordance with ASCE/SEI 41-17 § 8.4.2.3.2. The acceptance ratio is 1.32. The overturning moment capacity is dependent on the expected vertical load  $P_{UD}$ . Further discussion on the calculation of  $P_{UD}$  is provided in Section B.5. For this and subsequent calculations,  $P_{UD}$  is equal to the unfactored, expected vertical load including the self-weight of the footing.
  - These fixed base results are compared against the ASCE/SEI 7-10 allowable bearing pressure calculation. As with Hypothesis 1, for the purposes of this evaluation, the site-specific seismic acceleration of 1g is used. The redundancy factor,  $\rho$ , is taken as 1.0. The base shear is calculated including the R-factor for a special concrete shear wall ( $R=6$ ) and ASD load cases are utilized to evaluate the allowable bearing capacity for comparison. The footing is adequate for the ASCE/SEI 7-10 analysis, with a bearing pressure acceptance ratio of 0.98.
2. ASCE/SEI 41-17 Method 1 Linear Soil Springs – upper and lower bound (see Section B.8.3.3.2 for more detailed results)
  - Method 1 foundation springs are in accordance with ASCE/SEI 41-17 Figure 8-2. Spring derivation and methodology can be found in Section B.8. In accordance with ASCE/SEI 41-17 § 8.4.2, the lower bound stiffness is calculated as half of the expected value and the upper bound stiffness is calculated as twice the expected value.

- The moment demand at the base of the footing is determined from the reaction of the rotational soil spring. Resulting footing acceptance ratio for the lower bound case is 0.61 and 0.75 for upper bound stiffness.
3.  $K_{50}$  Linear Soil Springs – upper and lower bound (see Section B.8.3.3.3 for more detailed results)
- $K_{50}$  boundary rotation stiffness assumes that 50% of the moment capacity is mobilized and accounts for non-service level actions and displacements (EQ actions) and includes gapping between soil and footing. Spring derivation and methodology can be found in Section B.8.3.1. The  $300M_{c,foot}$  value is the expected rotational stiffness for a rectangular footing and  $550M_{c,foot}$  is applicable to an “I” shaped footing and is provided for comparison.
  - Acceptance ratios for the lower bound case are 0.38 and for the upper bound case are 0.50.

The results indicate that the ASCE/SEI 41-17 fixed base analysis provides reasonable correlation to the ASCE/SEI 7-10 foundation design. The flexible-base analysis procedures have lower acceptance ratios which is consistent with the reduced force attracted to the shear wall because of flexibility in the supporting foundation as well as higher m-factors permitted for the flexible-base analysis. The difference between acceptance ratios for lower and upper-bound analyses is relatively negligible for this case study. Following completion of the LSP analysis, NSP was utilized as comparison and benchmark for these studies. The nonlinear superstructure hinge behavior was modeled in accordance with ASCE/SEI 41-17 Chapter 10. Complete analysis findings can be found in Section B.8.3.3.4. Note that the acceptance ratios discussed in these findings are the rotations at the base of the shear wall at the target displacement compared to the allowable footing rotation from ASCE/SEI 41-17 Table 8-4.

#### 4. Flexible Base NSP Method 3 soil springs

- The effective fundamental period of this model is 0.70 seconds and the target displacement is 12.8 inches.
- The acceptance ratio for the footing at the target displacement is 0.78. Therefore, the retrofit footing design is acceptable. See Section B.8.3 and Table B-32.

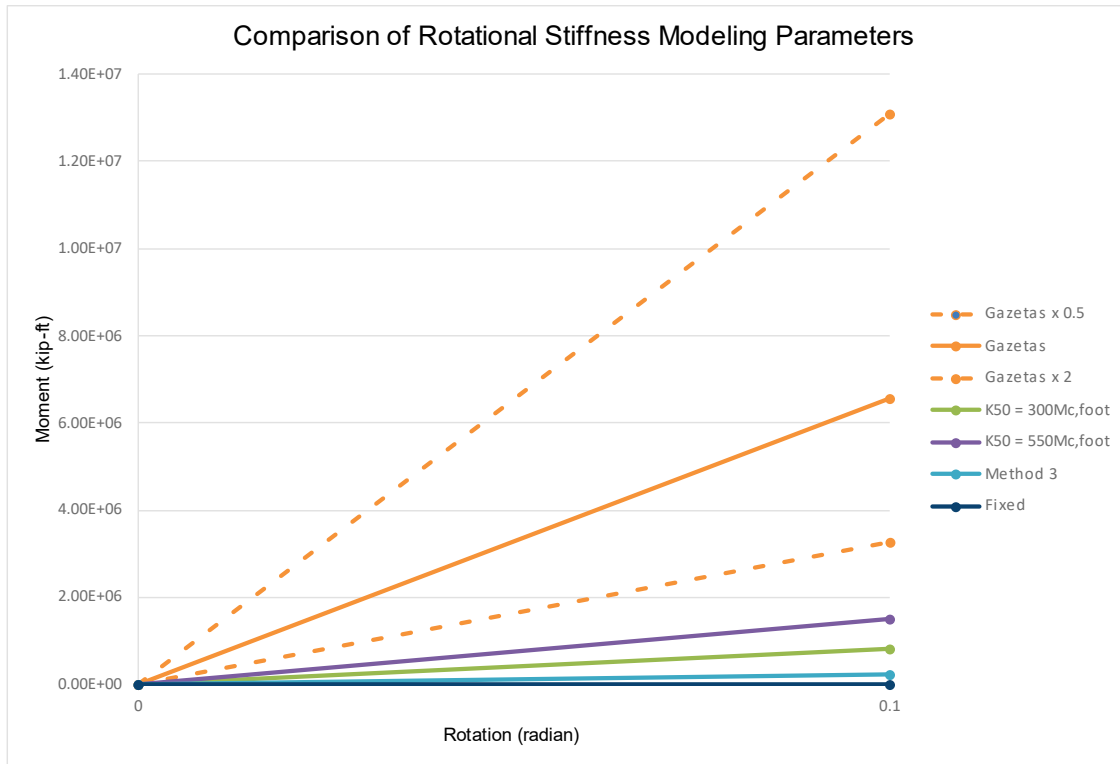
### **B.3.2.3 HYPOTHESIS 2 CONCLUSIONS**

In conclusion, these analyses found:

- Stiffness bounding conclusions confirm the hypothesis:
  - For linear analyses, because the ASCE/SEI 41-17 Figure 8-2 used for spring stiffness assumes that the soil remains in contact with the footing, the results are reasonable only if the soil remains in contact with the footing. Based on the soil bearing and superstructure results, if ASCE/SEI 41-17 Figure 8-2 must be used for stiffness derivation, then the lower bound Method 1 stiffness provides a fairly reasonable approach to modeling flexibility for linear procedures and is recommended to be used for the LSP. The lower-bound stiffness

provided reasonable results relative to the ASCE 7 foundation design while still including the effect of some foundation displacement in the super structure.

- Upper-bound stiffness does not yield sufficiently different results (superstructure component actions and foundation overturning acceptance ratios) to warrant the additional effort to include in the analysis procedures, and therefore need not be evaluated.
- $K_{50}$  effective stiffness (with gapping) correlates better with nonlinear analysis methods (Method 3) and is considered more realistic and more accurate than the Method 1 Lower Bound solution (see Section B.8.3.1.2 for more information on  $K_{50}$  effective stiffness).
- There are discrepancies between the stiffness values of ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) equations which are based on a rigid footing and elastic soil response where the soil remains in contact with the footings, and those stiffness values derived using the modulus of subgrade reaction and methods that embrace and incorporate soil separation from the footing as well as flexible and yielding structural footings. There is potential to bound and calibrate springs on the wrong (too stiff) solution. This leads to Hypotheses 3 and 5.
- See Figure B-13 for a comparison of the stiffness modeling parameters based on the different methods. Note that the ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) parameters, even the lower-bound, are significantly higher than that of the  $K_{50}$  method. The ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) method overestimates the stiffness. See Section B.8.3.1.4 for further information.



**Figure B-13 Comparison of stiffness modeling parameters based on different methods.**

While investigating Hypothesis 2, additional issues within the foundation provisions of ASCE/SEI 41-17 came to light. The investigations into these issues include:

- For this archetype building, the superstructure failure mechanism changes from fixed base analysis to the flexible base analysis. For the fixed base analysis, the columns remained elastic whereas in the flexible base case, the columns were failing in flexure. See section B.8.3.4 for more information.
- For this archetype building, when flexibility is introduced in a nonlinear system, when flexibility is introduced in a nonlinear system, the effective fundamental period of the model shifted significantly from 0.45 seconds for fixed base to 0.7 seconds for the flexible base NSP Method 3 soil springs proving that the fixed base model is very approximate.
- In addition to bounding of soil stiffness, the Hypothesis 2 models were used in investigating bearing strength bounding's effect on overturning moment capacity acceptance ratios. Further details of these calculations are included in Section B.4.1.2. The conclusions of that analysis include:
  - The use of upper-bound soil bearing strength for fixed-base analysis provides reasonable results compared to ASCE/SEI 7-10. For soil assessment, it is recommended that the terminology be revised to specify the use of the expected soil bearing strength with a factor

of 2 to account for transient, seismic loading effects in lieu of referring to “upper-bound” strength. Using the expected soil bearing without a factor of 2 to design the structural footing provides a more reasonable structural footing size/configuration. The use of lower bound stiffness combined with upper bound soil bearing is difficult for users to follow.

- The use of lower-bound soil bearing strength does not provide acceptable results for flexible-base analyses relative to ASCE 7 foundation design.
- Further guidance and clarity are recommended for defining  $P_{UF}$ , the expected vertical load on soil at the footing interface caused by gravity and seismic loads (formerly  $P_{UD}$ ). See Section B.5 for detailed calculations.
  - Provide user further guidance on the calculation of  $P_{UD}$  (factored, unfactored, include footing weight, etc.).
  - Recommend clarification that  $P_{UD}$  be expected (unfactored) load with footing weight included as was the original intention of this calculation.
- There are multiple approaches to determining allowable rotations for an atypical foundation configuration. See Section B.6 for more information.
  - I-shaped vs. rectangular footings provide numerically different allowable rotations.
  - Guidance should be provided to the user for cases where I-shaped footings when  $b/L_c$  is not between 1 and 10.
  - Rotation demand can be determined as rotation between end points of wall or between points of contraflexure.

As a part of Hypothesis 1 and 2, the scope of Hypotheses 4 (Force versus Deformation Control) and 5 (Calibration of Springs for Method 2) were also completed, see the corresponding sections for those results.

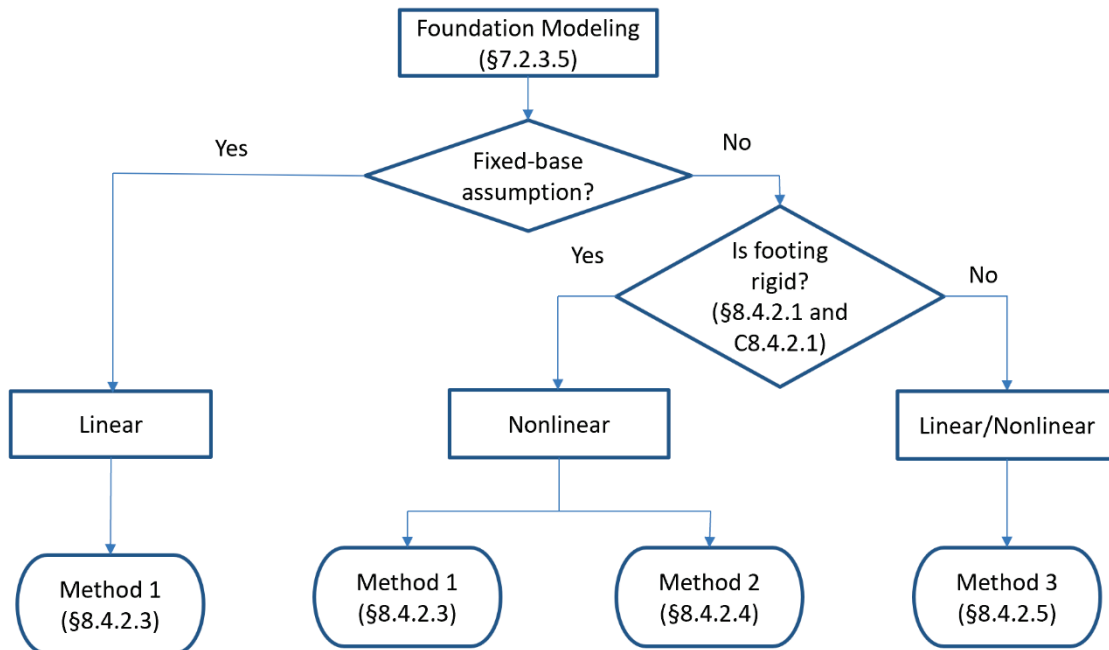
### **B.3.3 Hypothesis 3: Stiffness and strength relativity between structural footing and soil**

The ASCE/SEI 41-17 standard provides three methods of modeling and evaluation of shallow, flexible base foundations. The selection of the appropriate method is dependent on whether the footing is modeled using a rigid base or flexible base (building’s boundary condition) assumption which is based on the relative flexibility and strength of the structural footing and the soil foundation. The footing flexibility assessment should take into account the soil bearing pressure distribution, for instance whether uplift occurs, as well as the strength of the foundation element. Where the structural footing is flexible relative to the soil foundation or yielding of the structural footing or slab occurs, the footing is classified as flexible. Methods 1 and 2 are intended for footings that are stiff

and Method 3 is used for footings that are flexible relative to the soil. Figure B-14 provides a flowchart to assist the user with understanding the existing ASCE/SEI 41-17 process.

Contrary to the current ASCE/SEI 41-17 approach, the interaction between the structural system and its foundation should be considered at a high level before the nuance of structural footing to soil comparison is made, if at all. At this higher-level perspective, the decision-making process should determine if the foundation flexibility should be included or not (fixed base), and if so, should just the soil or both the structural footing and the soil be included. The hypothesis below was developed with this in mind.

As indicated in the flowchart, ASCE/SEI 41-17 § 8.4.2.1 requires that Method 3 be used where either stiffer soil relative to structural footing or a yielding foundation occurs. Equations C8-2 and C8-3 provided in ASCE/SEI 41-17 commentary should not be used to determine relative stiffness between soil and footing where uplift occurs or where the footing yields, and when used will provide incorrect results. Hypothesis 3 states that the determination of when Method 1 or 2 is acceptable to be used as opposed to Method 3 should be based on the engineer’s judgement, which should include an assessment of whether the integrated curvature of the structural footing (rotations and associated vertical deformations) is significant as compared to the nonlinear soil action. It is not practical to provide an all-encompassing numerical determination of when flexible foundation modelling is required.



**Figure B-14** Various foundation modeling approaches in ASCE/SEI 41-17.

This hypothesis was investigated through the analysis performed in other hypotheses. Based on those results, the following was determined:

- Fixed-base and flexible-base assumptions are not trivial to the performance of the superstructure. Depending on the building type and configuration, an “incorrect” base fixity assumption can incorrectly demonstrate that displacements in the superstructure are acceptable. Therefore, an assessment of foundation flexibility should be included in the provisions.
- The equations provided in ASCE/SEI 41-17 commentary for determining relative flexibility between soil and foundations are not particularly useful to the practicing engineer as they are typically oversimplified for foundation applications and they do not account for any soil gapping or foundation yielding, which is common in rocking foundations under seismic loading.
- General guidance in narrative format regarding how to assess when fixed foundations are permitted should be provided to assist the user.

### **B.3.4 Hypothesis 4: Force- versus deformation-controlled footing assessment**

As seen in Hypotheses 1 and 2, foundations designed using force-controlled provisions were overly conservative relative to the ASCE/SEI 7-10 designed results. Therefore, further examination of the foundation design being force- versus deformation-controlled was warranted. In ASCE/SEI 41-17, concrete foundations are typically required to be evaluated as force-controlled actions in accordance with ASCE/SEI 41-17 § 10.12.3, which requires the structural component of the foundation to remain essentially elastic. Hypothesis 4 posits that foundation component yielding that meets the deformation-controlled acceptance criteria defined for actions of that component does not preclude the structure meeting the target performance objective. In some cases, controlled yielding (defined as meeting the deformation-controlled acceptance criteria) will enable the target performance objective to be met with less retrofit scope. The goal is to investigate two scenarios: one where the retrofitted footing is allowed to yield and one where it remains essentially elastic utilizing LSP and NSP.

#### **B.3.4.1 HYPOTHESIS 4 PROCESS**

The models created for Hypotheses 1 and 2 were used in this study, see below. The structural foundations were modeled as elastic concrete beams on either elastic foundation (Method 1 and  $K_{50}$ ) springs or nonlinear compression-only foundation springs (Method 3).

- ASCE/SEI 7-10 (for comparison)
- Models with footing designs based on elastic beam methodology with lower-bound soil springs:
  - LSP - Fixed Base

- LSP - Method 1 Lower Bound (Rigid Footing)
  - LSP - Method 1 Upper Bound (Rigid Footing)
  - LSP -  $K_{50} 300M_{c,foot}$  (Rigid Footing)
  - LSP -  $K_{50} 550M_{c,foot}$  (Rigid Footing)
- NSP - Method 3

Using these models, the structural foundation components are evaluated for each model and compared to the ASCE/SEI 7-10 calculations. Foundation strength is evaluated as both force-controlled and deformation-controlled for comparison.

#### **B.3.4.2 HYPOTHESIS 4 RESULTS**

For linear analyses, the case study shows reasonable correlation with ASCE/SEI 7 foundation design where the foundation is modelled as an elastic structural foundation on elastic soil springs (tension/compression) with unreduced elastic loads applied and the structural foundation is assessed as deformation-controlled.

Footing design based on deformation-controlled actions using acceptance criteria from the appropriate material chapters should be permitted. The use of an elastic beam modeling approach utilizing elastic springs with expected stiffness provides a reasonable approach to foundation design and evaluation. As an alternative, reduction of pseudo-elastic forces by an  $m$ -factor or  $DCR$  may provide reasonable results depending on stability of the compression-only soil spring analysis model.

There are inconsistencies between the various material chapters that will need to be resolved as some require actions to be treated as force-controlled, and others as deformation-controlled. These will need to be reexamined with these foundation provisions for consistency.

See Section B.7 and for further investigation into the modeling and assessment of flexible foundations.

#### **B.3.5 Hypothesis 5: Calibration of Springs for Method 2**

During the Hypothesis 1 and 2 studies, calibrating Method 2 to Method 1 was difficult to implement. Hypothesis 5 states that further guidance is necessary for calibration of Method 2 springs to Method 1 spring stiffness.

Method 2 provides an alternative approach for rigid foundations that uses a bed of nonlinear springs that accounts for coupling between vertical loads and moment; therefore, Method 2 is preferred over Method 1 when there is significant variation in vertical load. Based on the commentary and provisions provided in ASCE/SEI 41-17, Method 2 is recommended for nonlinear analysis procedures where yielding and gapping springs are used to represent the soil-footing interaction.

Method 2 is not intended for use with linear procedures and is expected to be too complicated for linear procedures due to the unreduced axial forces that are the basis of the linear procedures.

The moment-rotations and vertical load-deformation characteristics are modeled as a beam on a nonlinear Winkler foundation with stiffer vertical springs at the end regions of the foundation to allow for tuning of the springs to approximately match the elastic vertical and rotational stiffness (see Figure B-56). While this distribution of spring stiffness does allow accurate modeling of rotational and vertical stiffness and moment capacity, it does not ensure that the settlements are accurately predicted with Method 2 nonlinear dynamic analyses. Method 2 allows for soil acceptance criteria to be based on those of the superstructure or those of ASCE/SEI 41-17 Table 8-4 for foundation rotations for NDP.

The stiffness equations for sliding and rocking foundation stiffness are accurate for lightly loaded foundations, loaded in the elastic range. But if the foundations produce significant nonlinear actions, these stiffnesses tend to overestimate the effective foundation stiffness. This stiffness issue also affects the stiffness, strength and capacity distribution of vertical springs. The existing procedure in ASCE/SEI 41-17 attempts to select distributed springs in a way that approximates the vertical and rocking stiffnesses from elastic solutions that do not apply when nonlinearity occurs.

ASCE/SEI 41-17 implies the user should calibrate the Method 2 Winkler spring equations (Figure 8-5) to match the vertical and rotational stiffnesses from the elastic solutions (Method 1) in Figure 8-2. However, the stiffness equations noted in Figure 8-5 do not have variables to calibrate. The commentary C8.4.2.4.1 refers to Gajan et al. (2010) for reference on calibration, which provides a method of calibrating the equations. There is a possible order of magnitude difference between the two calculations. The most reasonably accurate calibration procedure is contained in the reference as it is dependent on the footing dimensions and can more closely be tuned to Method 1 findings. However, as noted in previous sections, Method 1 assumes that the soil remains in contact with the footing which would lead Method 2 to overestimate the stiffness if calibrated to Method 1. As soon as there is some yielding and footing uplift, there is a reduction in effective rotational stiffness. Therefore, calibrating Method 2 to Method 1 is not rational. Instead, Method 2 could be calibrated to the  $K_{50}$  stiffness.  $K_{50}$  boundary rotation stiffness assumes that 50% of the moment capacity is mobilized and accounts for non-service level actions and displacements (EQ actions) and includes gapping between soil and footing. Studies have shown that there is a direct correlation between moment capacity and  $K_{50}$  and therefore that it can be an accurate method for determining the secant stiffness of a rocking foundation for the point at which half of the moment capacity is mobilized. See Section B.8.3.1.2 for further information on the  $K_{50}$  stiffness derivation.

### **B.3.5.2 HYPOTHESIS 5 RESULTS**

If Method 2 is kept, it is recommended that Method 2 be calibrated to the  $K_{50}$  stiffness. A procedure described in Gavras et al., 2015 could also be explored as a calibration method to improve the spring parameter selection for rocking and axial loading in Method 2. However, it is recommended that Method 2 is removed until a consensus is reached and provisions updated to be consistent between Method 1 and Method 3 or  $K_{50}$ . The goal of this calibration should be to more realistically

model the capacity, stiffness and energy dissipation in moment-rotation hysteresis loops while sacrificing some of the accuracy of the vertical stiffness.

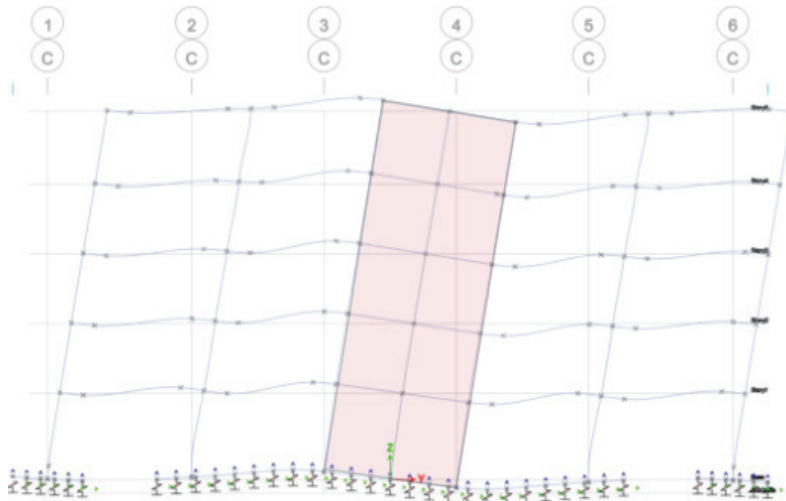
### **B.3.6 Hypothesis 6: Method 3 not intended to be used with LSP and LDP**

ASCE/SEI 41-17 § 8.4.2.1 requires Method 3, which includes geometric nonlinearity at the soil-structure interface, to be used where structural foundation flexibility or yielding is significant. However, ASCE/SEI 41-17 lacks clear provisions on how to use Method 3 with LSP and LDP. Nonsensical instability can occur at the foundation interface when linear, pseudo seismic (unreduced) forces are applied to a linear model. Therefore, Hypotheses 6 states that LSP and LDP should not be used with Method 3 in the current form. However, linear Method 3 provisions are proposed in the 2023 edition to reflect industry practice, and are developed to permit a linear approach for flexible foundations relative to soil.

#### **B.3.6.1 HYPOTHESIS 6 PROCESS – NORTH SOUTH DIRECTION**

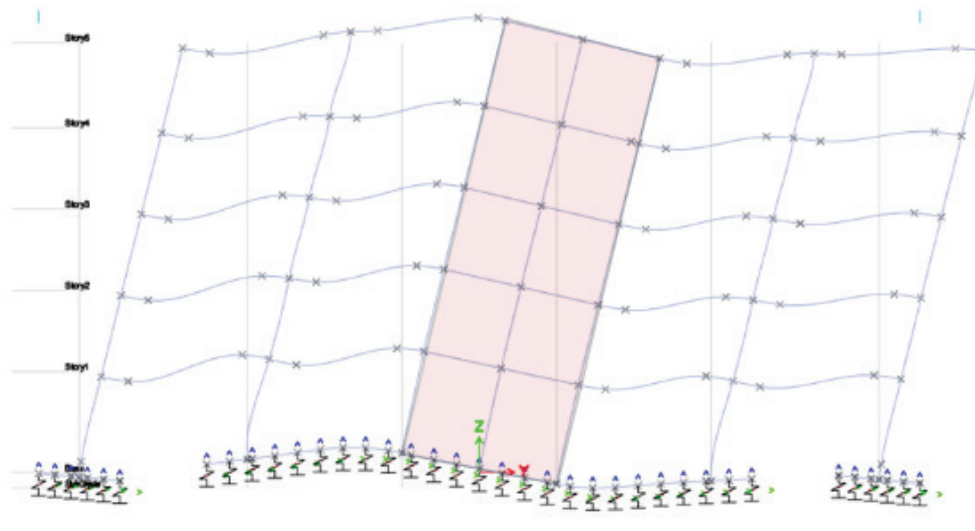
Multiple models were used for comparison in this evaluation, and the structure was examined in both the North-South direction, and the East-West direction. While all superstructures were linear elastic (Model B.1 from Table B-2), modeling varied as described below:

1. North-South LSP Method 3 model with linear elastic soil springs (ASCE/SEI 41-17 Eq. 8-11)
  - Applied Loads: unreduced pseudo elastic loads
  - Soil springs: linear-elastic (resists tension/compression)
  - Soil spring stiffness:
    - Modulus of subgrade reaction,  $k_{sv}$  (Method 3, ASCE/SEI 41-17 Eq. 8-11)
    - Expected and lower bound stiffness evaluated



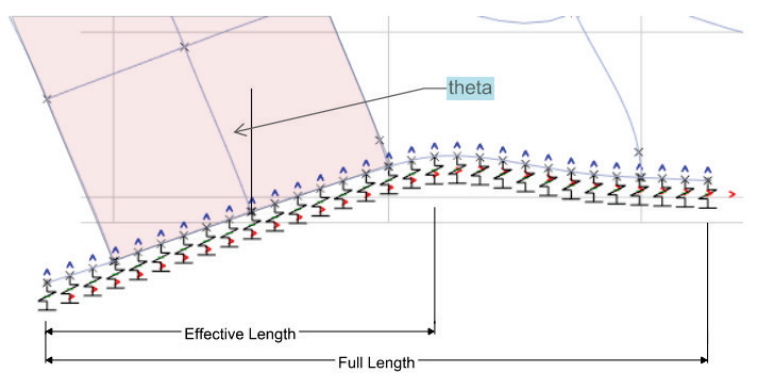
**Figure B-15: North-South LSP model with Method 3 linear elastic soil springs**

2. North-South LSP Method 3 model with nonlinear soil springs (compression-only) capped at expected bearing capacity
  - Pseudo elastic loads reduced by  $m = 4$
  - Stiffness:
    - Compression stiffness = modulus of subgrade reaction,  $k_{sv}$  (Method 3, Eq. 8-11)
    - Expected stiffness evaluated



**Figure B-16: North-South LSP Method 3 model nonlinear soil springs (compression-only) capped at expected bearing capacity.**

These analyses require calculation of the Effective Length and Total Axial Load  $P_{UD}$ . The effective length as shown in Figure B-17 is the distance between the inflection points.

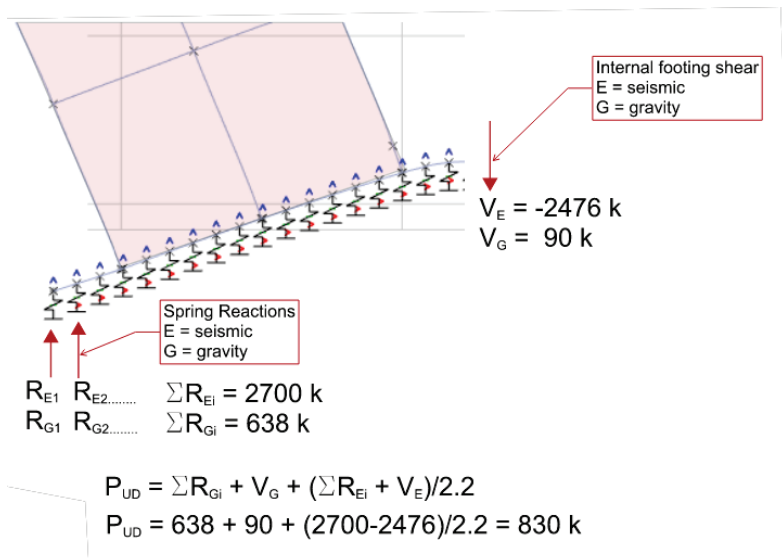


**Figure B-17 Effective Length.**

The total  $P_{UD}$  can be determined using statics to sum the forces, the spring reactions and the internal footing shears, as shown in Figure B-18. The total  $P_{UD}$  is based on the equation provided in ASCE/SEI 41-17 Section 8.4.2.3.1:

$$P_{UD} = P_G + P_E/DCR$$

In this case, the spring reactions and internal footing shears are used to determine the gravity and seismic axial forces. Per ASCE/SEI 41-17 Section 8.4.2.3.1, the DCR need not be taken as less than  $C_1C_2$  or greater than  $2C_1C_2$ . In this case, the maximum is used so the DCR is  $2(1.1) = 2.2$ .



**Figure B-18  $P_{UD}$  determination.**

Using the Effective Length and Total  $P_{UD}$ , the allowable rotation can be determined. An example calculation is included below in Figure B-19.

$b =$	9.5 ft
$L =$	32.3 ft
$A_f =$	307 ft <sup>2</sup>
$P_{UD} =$	830 kips
$q_c =$	10.5 ksf
$A_c =$	79 ft <sup>2</sup>
$A_f =$	307 ft <sup>2</sup>
$A_{rect} =$	307 ft <sup>2</sup>
$A_{rect} - A_f / A_{rect} =$	0.00
$A_c / A_f =$	0.26
$L_c = A_c / b =$	8.31 ft
$b / L_c =$	1.14
allow. rotation =	0.0353 radians, 41-17 Table 8-4
rotation =	0.017
acceptance ratio =	0.62

**Figure B-19 Allowable rotation determination.**

#### B.3.6.4 HYPOTHESIS 6 – NORTH SOUTH RESULTS AND CONCLUSIONS

The results of each North-South analysis are included in Table B-9 below. For the linear elastic approach, the rotation demand is determined at the base of the shear wall. It is compared to the allowable rotation determined using ASCE/SEI 41-17 Table 8-4 (as for nonlinear procedures), but modified by a factor of 0.75 to convert to a linear analysis per ASCE/SEI 41-17 Section 7.6.3.7. This factor may not be applicable for this application but was used in an endeavor to remain consistent with Chapter 7. For the nonlinear spring approach, the bearing demand is determined from analysis and compared to an expected bearing capacity with no additional m-factors applied.

**Table B-8 Summary of Foundation Soil Results (N-S) Direction from Hypotheses 2 and 6**

		Foundation Soil Acceptance Ratio
Hypothesis 2 LSP (See Section B.8.3.3 for calculations)	LSP - ASCE/SEI 7-10 (Section B.8.3.3.1)	0.98
	LSP - Fixed Base (Section B.8.3.3.1)	1.32
	LSP - Method 1 Lower Bound (Rigid Footing) (Section B.8.3.3.2)	0.61
	LSP - Method 1 Upper Bound (Rigid Footing) (Section B.8.3.3.2)	0.75
	LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing) (Section B.8.3.3.3)	0.38
	LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing) (Section B.8.3.3.2)	0.50
Hypothesis 6 (North-South)	LSP - Method 3 Expected Stiffness (Full length, Total $P_{UD}$ )	0.58
	LSP - Method 3 Expected Stiffness (Effective length, Effective $P_{UD}$ )	0.45
	LSP - Method 3 Lower Bound Stiffness <sup>1</sup> (Full length, Total $P_{UD}$ )	0.84
	LSP - Method 3 Lower Bound Stiffness <sup>(1)</sup> (Effective length, Effective $P_{UD}$ )	0.64
	*LSP - Nonlinear Method 3 Springs Expected Stiffness ( $m = 4$ )	0.79
Hypothesis 2 NSP	NSP - Method 3 Expected Stiffness (Full length, Total $P_{UD}$ ) (Section B.8.3.3.4)	0.78
	NSP - Method 3 Expected Stiffness (Effective length, Effective $P_{UD}$ )	0.47

<sup>(1)</sup> Lower Bound stiffness is  $k_{sv}$  (ASCE/SEI 41-17 Eq. 8-11) multiplied by 0.5.

As can be seen in the table, the proposed linear elastic approach, using lower-bound stiffness and effective footing length and axial loads, provides reasonable acceptance ratios when compared to Method 1 and NSP Method 3 analyses. However, the application of  $m = 4$  may be conservative or unconservative depending on the strength capacity of the structural system of any given building and the distribution of forces therein. Therefore, this nonlinear springs approach could require further guidance.

For linear procedures, the recommended methodology from what is currently developed is the LSP Method 3 Lower Bound Stiffness with effective length and effective  $P_{UD}$  highlighted in Table B-9

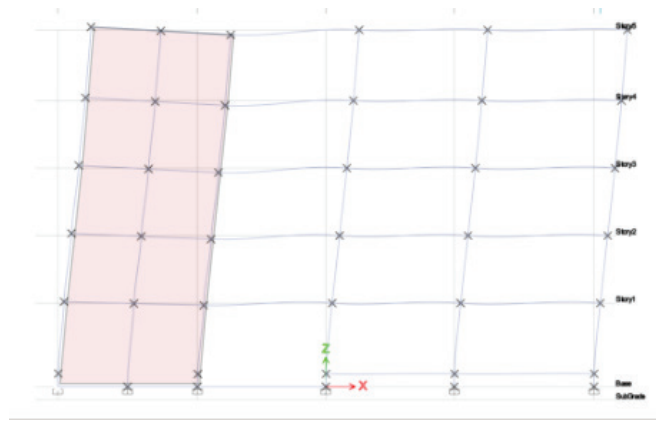
above. This is more liberal than ASCE/SEI 7-10 and more conservative than ASCE/SEI 41-17 NSP, which deems it acceptable for this case.

### B.3.6.3 HYPOTHESIS 6 PROCESS – EAST WEST DIRECTION

A similar process was completed in the East-West direction where the shear wall and retrofit rectangular footing are located at the corner edge of the structure. Multiple models were used for comparison in this evaluation and while all superstructures were linear elastic, modeling varied as described below:

#### 1. East-West LSP Fixed Base

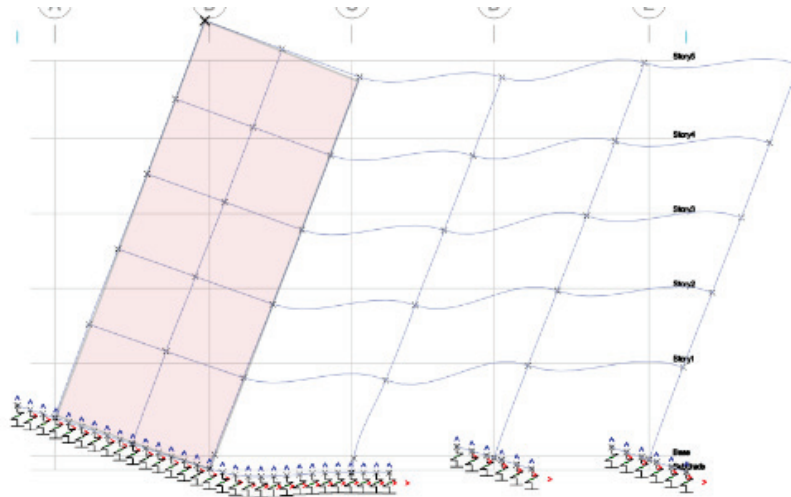
- Applied Loads: unreduced pseudo elastic loads
- Soil springs: N/A – fixed base



**Figure B-20 East-West LSP analysis model with fixed base.**

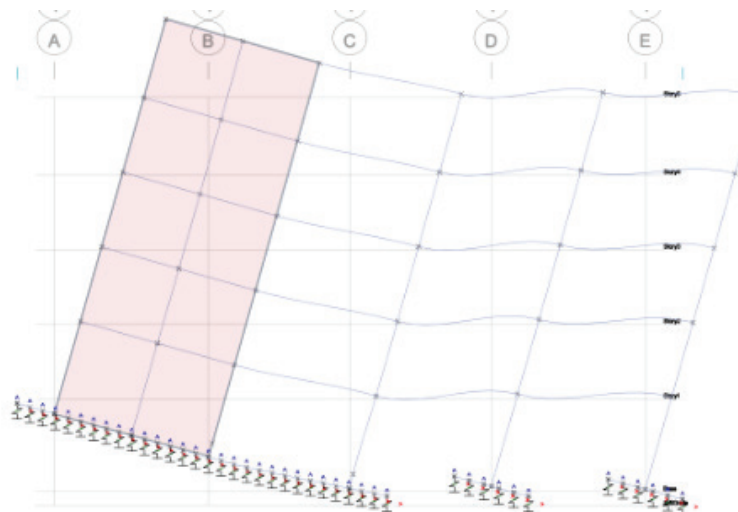
#### 2. East-West LSP Method 3 model with linear elastic soil springs (ASCE/SEI 41-17 Eq. 8-11)

- Applied Loads: unreduced pseudo elastic loads
- Soil springs: linear-elastic (resists tension/compression)
- Soil spring stiffness:
  - Modulus of subgrade reaction,  $k_{sv}$  (Method 3, ASCE/SEI 41-17 Eq. 8-11)
  - Expected and lower bound stiffness evaluated



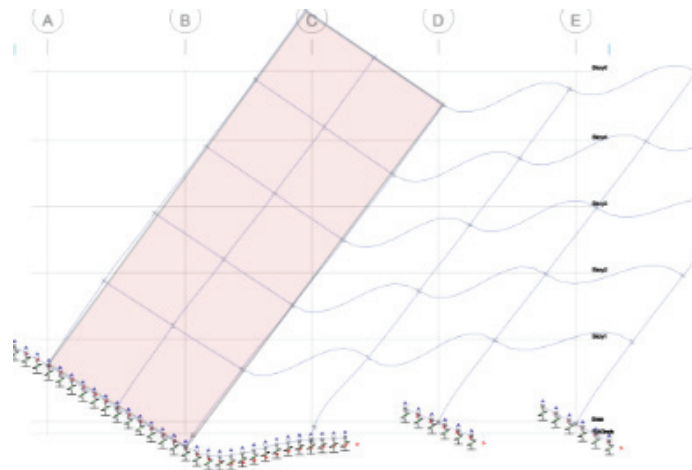
**Figure B-21 East-West LSP model with Method 3 linear elastic soil springs.**

3. East-West LSP Method 3 model with linear elastic soil springs (ASCE/SEI 41-17 Fig. 8-2)
  - Applied Loads: unreduced pseudo elastic loads
  - Soil springs: linear-elastic (resists tension/compression)
  - Soil spring stiffness:
    - Vertical translation stiffness from ASCE/SEI 41-17 Fig. 8-2
    - Expected stiffness evaluated



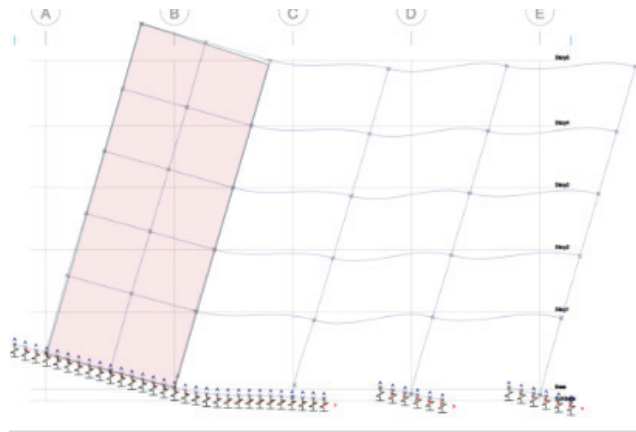
**Figure B-22 East-West LSP model Method 3 linear elastic soil springs.**

4. East-West LSP Method 3 model with nonlinear soil springs (compression-only) capped at expected bearing capacity
  - Pseudo elastic loads reduced by  $m = 4$
  - Stiffness:
    - Compression stiffness = modulus of subgrade reaction,  $k_{sv}$  (Method 3, Eq. 8-11)
    - Expected stiffness evaluated



**Figure B-23 East-West LSP Method 3 model nonlinear soil springs (compression-only) capped at expected bearing capacity.**

5. East-West LSP Method 3 model with nonlinear soil springs (compression-only) NOT capped at expected bearing capacity
  - Pseudo elastic loads reduced by  $m = 4$
  - Stiffness:
    - Compression stiffness = modulus of subgrade reaction,  $k_{sv}$  (Method 3, Eq. 8-11)
    - Expected stiffness evaluated



**Figure B-24 East-west LSP Method 3 model nonlinear soil springs (compression-only) not capped at expected bearing capacity.**

These results were also compared with an NSP push-over analysis in which the compression-only, nonlinear soil springs were capped at the expected bearing capacity.

#### B.3.6.4 HYPOTHESIS 6 - EAST WEST RESULTS AND CONCLUSIONS

The results of each East-West analysis are included in Table B-9 below. As with the North-South cases, for the linear elastic approach, the rotation demand is determined at the base of the shear wall. It is compared to the allowable rotation determined using ASCE/SEI 41-17 Table 8-4 (as for nonlinear procedures), but modified by a factor of 0.75 to convert to a linear analysis per ASCE/SEI 41-17 Section 7.6.3.7. This factor may not be applicable for this application but was used in an endeavor to remain consistent with Chapter 7. For the nonlinear spring approach, the bearing demand is determined from analysis and compared to an expected bearing capacity with no additional m-factors applied.

**Table B-9: Summary of Foundation Soil Results (E-W) Direction from Hypotheses 2 and 6**

	Foundation Soil Acceptance Ratio
LSP - ASCE 7-10	1.34
LSP - Fixed Base <sup>(1)</sup>	2.03
LSP - Method 3 ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) Stiffness Expected	0.76
LSP - Method 3 Expected Stiffness (Full length, Total $P_{UD}$ )	0.44
LSP - Method 3 Expected Stiffness (Effective length, Effective $P_{UD}$ )	0.42

LSP - Method 3 Lower Bound Stiffness (Full length, Total $P_{UD}$ )	0.66
LSP - Method 3 Lower Bound Stiffness (Effective length, Effective $P_{UD}$ )	0.62
LSP - Nonlinear Method 3 Springs Capped/( $m = 4$ )	N/A <sup>(2)</sup>
LSP - Nonlinear Method 3 Springs Uncapped/( $m = 4$ )	1.27

<sup>(1)</sup> Overturning capacity includes restraint from slabs framing into structure above footing.

<sup>(2)</sup> N/A indicates soil is yielding and the soil acceptance is determined from the numerical stability of the subject model. Foundation springs are capped at expected bearing capacity of soil.

As can be seen in the table, the proposed linear elastic approach, using lower-bound stiffness and effective footing length and axial loads, provides reasonable acceptance ratios. It is unclear how to do a comparison between the uncapped and capped conditions.

Again, the application of  $m = 4$  may be conservative or unconservative depending on the strength capacity of the structural system of any given building and the distribution of forces therein. Therefore, this nonlinear springs approach could require further guidance.

For linear procedures, the recommended methodology from what is currently developed is the LSP Method 3 Lower Bound Stiffness with effective length and effective  $P_{UD}$  highlighted in Table B-9 above. This is more liberal than ASCE/SEI 7-10 and more conservative than ASCE/SEI 41-17 NSP, which deems it acceptable for this case.

A summary of the superstructure results for the E-W cases, LSP Fixed Base, Method 3 Lower Bound, and NL Springs uncapped, is included in the tables below. The fixed base analysis significantly underestimates the superstructure demand compared to flexible base procedures. Depending on structural system, acceptance ratios using a uniform  $m$ -factor could be conservative, unconservative or accurate. Therefore, again it is not recommended that a uniform value be used for all structural systems.

**Table B-10 East-West Acceptance Ratios for Interior Columns for different LSP Procedures**

Existing Interior Columns - Moment Acceptance Ratios (CP Limit State)							
Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Max.	Outcome
LSP - Fixed Base	0.39	0.15	0.20	0.19	0.20	0.39	OK
LSP - Method 3 Lower Bound	0.63	0.62	0.74	1.00	1.08	1.08	NG
*LSP - NL Springs Uncapped Demand/( $m = 4$ )	0.85	0.58	0.67	0.95	1.85	1.85	NG
NSP - Method 3	0.00	0.00	0.00	0.00	0.05	0.05	OK

Existing Interior Columns - Shear Acceptance Ratios (CP Limit State)							
Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Max.	Outcome
LSP - Fixed Base	0.19	0.12	0.17	0.17	0.15	0.19	OK
LSP - Method 3 Lower Bound	0.36	0.36	0.58	0.86	0.87	0.87	OK
*LSP - NL Springs Uncapped Demand/(m = 4)	0.44	0.32	0.56	0.82	1.39	1.39	NG
NSP - Method 3							

Note: No additional m-factor applied to component capacity. Actual m-factor for structural component ranges from 4.2-2.7

**Table B-11 East-West Acceptance Ratios for Exterior Columns for different LSP Procedures**

Existing Exterior Columns - Moment Acceptance Ratios (CP Limit State)							
Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Max.	Outcome
LSP - Fixed Base	1.04	0.27	0.27	0.53	0.14	1.04	NG
LSP - Method 3 Lower Bound	5.15	0.71	1.11	1.14	0.41	5.15	NG
*LSP - NL Springs Uncapped Demand/(m = 4)	1.41	0.20	0.43	0.87	0.55	1.41	NG
NSP - Method 3	0.33	0.04	0.04	0.00	0.00	0.33	OK
Existing Interior Columns - Shear Acceptance Ratios (CP Limit State)							
Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Max.	Outcome
LSP - Fixed Base	1.16	0.29	0.33	0.36	0.09	1.16	NG
LSP - Method 3 Lower Bound	5.33	1.11	1.18	1.28	0.31	5.33	NG
*LSP - NL Springs Uncapped Demand/(m = 4)	1.41	0.32	0.32	0.76	0.44	1.41	NG
NSP - Method 3							

NOTE: No additional m-factor applied to component capacity. Actual m-factor for structural component ranges from 4.2-2.7

**Table B-12 East-West Acceptance Ratios for Retrofit Shear Walls for different LSP Procedures**

Retrofit Shear Wall - Shear Acceptance Ratios (CP Limit State)							
Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Max.	Outcome
LSP - Fixed Base	0.49	1.03	0.83	0.60	0.25	1.03	NG

LSP - Method 3 Lower Bound	0.30	0.80	0.58	0.01	0.10	0.80	OK
*LSP - NL Springs Uncapped Demand/( $m = 4$ )	0.22	0.80	0.57	0.34	0.14	0.80	OK
NSP - Method 3	0.01	0.01	0.11	0.05	0.01	0.11	OK
Retrofit Shear Wall - Moment Acceptance Ratios (CP Limit State)							
Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Max.	Outcome
LSP - Fixed Base	0.56	0.53	0.46	0.42	0.24	0.56	OK
LSP - Method 3 Lower Bound	0.82	1.24	0.77	0.32	0.17	1.24	NG
*LSP - NL Springs Uncapped Demand/( $m = 4$ )	0.56	1.04	0.66	0.38	0.21	1.04	NG
NSP - Method 3	0.00	0.00	0.00	0.00	0.00	0.00	OK

Note: No additional m-factor applied to component capacity. Actual m-factor for structural component is 4.0

**Table B-13 East-West Acceptance Ratios for the Concrete Slab for different LSP Procedures**

Existing Slab - Flexure Acceptance Ratios (CP Limit State)							
Analysis Model	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Roof	Max.	Outcome
LSP - Fixed Base	0.37	0.37	0.35	0.29	0.15	0.37	OK
LSP - Method 3 Lower Bound	1.32	1.32	1.19	1.00	0.60	1.32	NG
LSP - NL Springs Uncapped Demand/( $m = 4$ )	1.52	1.49	1.38	1.27	0.77	1.52	NG
NSP - Method 3	0.25	0.22	0.20	0.16	0.00	0.25	OK

Note: No additional m-factor applied to component capacity. Actual m-factor for structural component ranges from 3.3-3.4

**Table B-14 East-West Story Drift Results for different LSP Procedures**

Story Drift - drift per story (in)						
Analysis Model	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Roof	Max.
LSP - Fixed Base	0.59	0.69	0.68	0.62	0.54	0.69
LSP - Method 3 Lower Bound	2.86	2.47	2.46	2.41	2.46	2.86
LSP - NL Springs Uncapped Demand/( $m = 4$ )	0.98	0.85	0.85	0.83	0.86	0.98
NSP - Method 3	1.95	1.72	1.68	1.61	1.64	1.95

Story Drift - drift ratio per story						
Analysis Model	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Roof	Max.
LSP - Fixed Base	0.004	0.005	0.005	0.005	0.004	0.005
LSP - Method 3 Lower Bound	0.018	0.020	0.020	0.019	0.019	0.020
LSP - NL Springs Uncapped Demand/( $m = 4$ )	0.006	0.007	0.007	0.007	0.007	0.007
NSP - Method 3	0.013	0.014	0.013	0.013	0.012	0.014

## B.4 Prescriptive Expected Bearing Capacities (Proposed ASCE/SEI 41-23 Section 8.4.2.1): Bearing Capacity Determination and Bounding

During the Hypothesis 1 analysis and in setting up the case study models, multiple approaches to defining bearing capacity and its upper and lower bounds were investigated.

### B.4.1 Bearing Capacity Investigation

#### B.4.1.1 METHODOLOGY

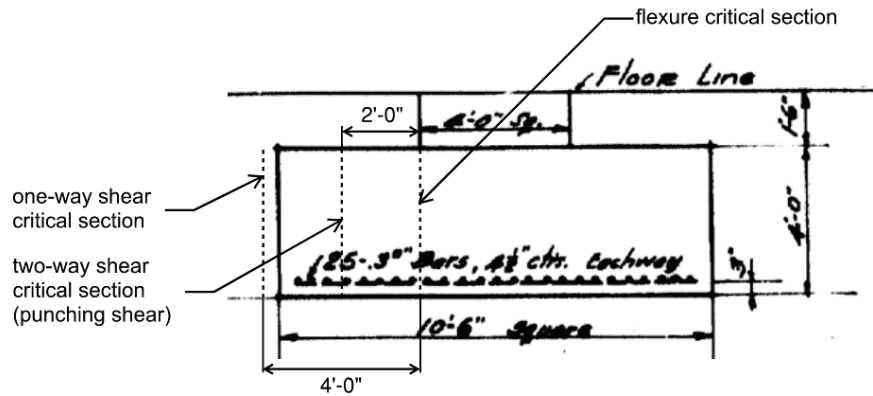
The expected soil bearing capacity was evaluated with the following different approaches and then compared.

- Based on allowable bearing pressure specified on the original construction drawings (ASCE/SEI 41-17 Equation 8-1).
- Based on the calculated gravity loads to the existing footing (ASCE/SEI 41-17 Equation 8-3).
- Based on the maximum force that can be transferred to the soil through the structure as limited by the deformation-controlled structural footing capacity (expected strength with  $m = 1$ ).
- Based on the maximum force that can be transferred to the soil through the structure based on the force-controlled structural footing capacity as specified in ASCE/SEI 41-17 § 10.12.3.

#### B.4.1.2 EXISTING FOOTING DESCRIPTION

The existing building was designed and built in the early 1920s for storage. Design loading noted on the existing drawings is 300 psf live load at each floor level and 40 psf live load at the roof. Due to the high loading, the existing footing below each column is robust as shown in Figure B-25. It is important to note that footings of other buildings with more typical design live loads would likely be

18" to 24" deep. The expected rebar yield strength and concrete compressive strength are shown in Table B-15 and are based on usual testing; therefore, the knowledge factor is equal to 1.0.



**Figure B-25 Existing footing with critical sections identified.**

**Table B-15 Existing Footing Expected Strength**

	Expected Strength	Knowledge Factor
Rebar Expected Yield Strength, $f_{ye}$	38.6 ksi	1.0
Concrete Expected Strength, $f'_{ce}$	3.2 ksi	1.0

#### B.4.1.3 EXPECTED SOIL BEARING CAPACITY BASED ON ALLOWABLE BEARING PRESSURE

A geotechnical report was developed for the building retrofit. The allowable bearing pressure provided in the report for dead plus live loads is 3.5 kips-per-square-foot (ksf). This is converted to a prescriptive expected bearing capacity in accordance with ASCE/SEI 41-17 Equation 8-1 and an upper-bound bearing capacity in accordance with ASCE/SEI 41-17 Figure 8-1a as shown below.

The prescriptive expected allowable bearing capacity for this spread footing is:

$$q_c = 3q_{allow} \quad (\text{ASCE/SEI 41-17 Eq. 8-1})$$

where:

$$q_{allow, D+L} = 3.5 \text{ ksf} \quad (\text{allowable bearing capacity from geotechnical report for D+L})$$

$$q_{ce} = 3(3.5 \text{ ksf}) = 10.5 \text{ ksf}$$

Then the upper bound soil bearing capacity based on the allowable bearing pressure can be determined:

$$q_{c, upperbound} = 2q_{ce} \quad (\text{ASCE/SEI 41-17 Figure 8-1a})$$

$$= 2(10.5 \text{ ksf})$$

$$= 21 \text{ ksf}$$

#### **B.4.1.4 EXPECTED SOIL BEARING CAPACITY BASED ON CALCULATED GRAVITY LOAD**

The expected bearing capacity for shallow footings can be determined based on the gravity load action on the soil using ASCE/SEI 41-17 Equation 8-3. This results in slightly lower expected bearing capacity than as calculated based on allowable bearing pressure as shown in the calculations below. The calculated bearing demand under the original dead plus live loading in this case is higher than the allowable bearing pressure of 3,500 psf provided by the geotechnical engineer. This discrepancy is attributed to the typically conservative determination of foundation design parameters in current design. Coordination with the geotechnical engineer is recommended to determine a reasonable expected bearing capacity that is consistent with the original foundation design. For the purposes of this study, expected bearing strength determined from the allowable bearing capacity provided in the geotechnical report is utilized in the calculations (the greater 21 ksf value), and recommendations are made to modify the approach to determining expected bearing capacity from calculated gravity load.

The prescriptive expected allowable bearing capacity for this spread footing is:

$$q_{ce} = 1.5Q_G \quad (\text{ASCE/SEI 41-17 Eq. 8-3})$$

where:

$$Q_G = 1.1(Q_D + Q_L + Q_S) \quad (\text{ASCE/SEI 41-17 Eq. 7-1})$$

Note that this is in accordance with the ASCE/SEI 41-17 equations but it would have been more appropriate to use  $Q_D + Q_L$  as that would have been used in the original design.

where:

$$Q_D = 371.5 \text{ kips} \quad (\text{calculated using Tributary Area})$$

$$Q_L = 0.25(496 \text{ kips}) \quad (\text{ASCE/SEI 41-17 § 7.2.2, 300 psf floors, 40 psf roof})$$

$$Q_G = 1.1(371.5 \text{ kips} + 0.25(496 \text{ kips})) = 545 \text{ kips}$$

Then to convert that into a pressure, use the area of the footing

$$\text{Footing Area} = (L_{\text{footing}})(b_{\text{footing}}) = (10.5 \text{ ft})(10.5 \text{ ft}) = 110.25 \text{ sq ft}$$

$$Q_G = (545 \text{ kips}) / (110.25 \text{ sq ft}) = 4.9 \text{ ksf}$$

$$q_{ce} = 1.5 (4.9 \text{ ksf}) = 7.4 \text{ ksf}$$

Then the upper bound soil bearing capacity based on the calculated gravity load can be determined:

$$\begin{aligned}
 q_{c, \text{upperbound}} &= 2q_{ce} && \text{(ASCE/SEI 41-17 Figure 8-1a)} \\
 &= 2(7.4 \text{ ksf}) \\
 &= 14.8 \text{ ksf}
 \end{aligned}$$

#### B.4.1.5 STRUCTURAL FOOTING CAPACITY – EXPECTED STRENGTH

The structural footing capacity was determined by investigating these mechanisms: two-way shear, beam shear, and the flexural capacity which includes assessing reinforcement development. The critical sections for these mechanisms are shown in Figure B-25. Due to the depth of the footing, one-way shear does not apply as the critical section is outside of the footing. The calculations for each mechanism below are based on expected strength of the concrete footing. The limiting mechanism is the flexural capacity, which would limit the maximum bearing delivered to the soil to 20.0 ksf assuming expected material strengths.

First, the flexural capacity of the footing was determined:

$$M_{ce} = A_s f_{ye} (d - a/2) \quad \text{(Flexural strength per ACI 318-14 § 22.3)}$$

where:

$$A_s = 7.75 \text{ sq in} \quad \text{(footing contains (25) \#5 bars at 4.5 inches-on-center)}$$

$$f'_{ce} = 3.21 \text{ ksi} \quad \text{(expected compressive strength of the concrete per testing)}$$

$$f_{ye} = 38.6 \text{ ksi} \quad \text{(expected yield strength of the reinforcement per testing)}$$

$$d = 45 \text{ in} \quad \text{(effective depth per ACI 318-14)}$$

$$a = (A_s f_{ye}) / (0.85 f'_{ce} b_{\text{footing}}) \quad \text{(depth of equivalent rectangular stress block per ACI 318-14)}$$

$$= (7.75 \text{ sq in})(38.6 \text{ ksi}) / [(0.85)(3.21 \text{ ksi})(10.5 \text{ ft})] = 0.9$$

$$M_{ce} = (7.75 \text{ sq in})(38.6 \text{ ksi})(45 - 0.9/2)$$

$$= 1110 \text{ kip ft}$$

The flexural capacity was then used to determine the maximum allowable bearing capacity to the footing (if another mechanism controlled, that mechanism would have been used in this determination instead):

$$M_u = q_u [(L_{\text{footing}} - c) / 2]^2 (b_{\text{footing}}) / 2 \quad \text{(flexural demand at the critical section of a square footing)}$$

where:

$$c = 4 \text{ ft} \quad (\text{per Figure B-25})$$

which can be rearranged to solve for  $q_u$  at  $M_u = M_{ce}$ :

$$\begin{aligned} q_u &= 2M_{ce}/\{[(L_{\text{footing}}-c)/2]^2(b_{\text{footing}})\} \\ &= 2(1110 \text{ kip ft})/\{[(10.5 \text{ ft} - 4 \text{ ft})/2]^2(10.5 \text{ ft})\} \\ &= 20.0 \text{ ksf} \end{aligned}$$

The other mechanisms are checked at this bearing capacity to ensure that the flexural mechanism controls. The two-way shear calculations follows:

The two-way shear demand at this soil pressure can be calculated:

$$V_u = q_u[(L_{\text{footing}})^2-(c+d)^2] \quad (\text{two-way shear at the critical section of a square footing})$$

where:

$$c = 4 \text{ ft} \quad (\text{per Figure B-25})$$

$$d = 4 \text{ ft} \quad (\text{per Figure B-25})$$

$$\begin{aligned} V_u &= (20 \text{ ksf})[(10.5 \text{ ft})^2-(4 \text{ ft} + 4 \text{ ft})^2] \\ &= 925 \text{ kips} \end{aligned}$$

The shear capacity is:

$$V_c = 4\lambda(f'_{ce})^{0.5}b_o d \quad (\text{ACI 318-14})$$

Where:

$$b_o = 4(c+d) = 4(4 \text{ ft} + 4 \text{ ft}) = 32 \text{ ft}$$

$$V_c = 4\lambda(3.21 \text{ ksi})^{0.5}(32 \text{ ft})(4 \text{ ft}) = 4178 \text{ kips}$$

Which is much higher than the demand, so the flexural mechanism controls over the two-way shear.

Finally, the development length is checked per ACI 318-14 Table 25.4.2.2:

$$\begin{aligned} l_d &= d_b(f_{ye}\Psi_t\Psi_e)/[25\lambda(f'_{ce})^{0.5}] \quad (\text{ACI 318-14 Table 25.4.2.2}) \\ &= (0.625 \text{ in}) (38.6 \text{ ksi})(1)(1)/[25(1) (3.21 \text{ ksi})^{0.5}] \\ &= 17 \text{ inches} \end{aligned}$$

Therefore, the maximum bearing pressure delivered to the soil based on the deformation-controlled footing capacity is 20 ksf assuming expected material strengths.

#### **B.4.1.6 STRUCTURAL FOOTING CAPACITY – FORCE-CONTROLLED**

An approach to determining the maximum force that can be delivered to the soil can be based on ASCE/SEI 41-17 § 10.12.3. This section states that the foundation shall be evaluated as force-controlled; however, the capacity of the foundation components need not exceed 1.25 times the capacity of the supported vertical structural component or element (column or wall). Performing a limit state analysis, where the retrofit wall above is designed to meet the capacity of the existing footing, the flexural strength of the footing is investigated in the calculations below with force-controlled capacities. Based on this approach, the maximum bearing pressure based on these provisions is 16.7 ksf, which is also less than the upper bound soil bearing capacity based on the geotechnical recommendations.

First, the flexural capacity of the footing was determined:

$$M_n = A_s f_y (d - a/2) \quad \text{(Flexural strength per ACI 318-14 § 22.3)}$$

where:

$$A_s = 7.75 \text{ sq in} \quad \text{(footing contains (25) \#5 bars at 4.5 inches-on-center)}$$

$$f'_c = 2.14 \text{ ksi} \quad \text{(lower bound compressive strength of concrete)}$$

$$f_y = 25.7 \text{ ksi} \quad \text{(lower bound yield strength of the reinforcement)}$$

$$d = 45 \text{ in} \quad \text{(effective depth per ACI 318-14)}$$

$$a = (A_s f_y) / (0.85 f'_c b_{\text{footing}}) \quad \text{(depth of equivalent rectangular stress block per ACI 318-14)}$$

$$= (7.75 \text{ sq in})(25.7 \text{ ksi}) / [(0.85)(2.14 \text{ ksi})(10.5 \text{ ft})] = 0.9$$

$$M_n = (7.75 \text{ sq in})(25.7 \text{ ksi})(45 - 0.9/2)$$

$$= 740 \text{ kip ft}$$

$$1.25 M_n = 925 \text{ kip ft} \quad \text{(ASCE/SEI 41-17 § 10.12.3)}$$

This flexural capacity was then used to determine the maximum allowable bearing capacity to the footing:

$$M_u = q_u [(L_{\text{footing}} - c) / 2]^2 (b_{\text{footing}}) / 2 \quad \text{(flexural demand at the critical section of a square footing)}$$

where:

$$c = 4 \text{ ft} \quad \text{(per Figure B-25)}$$

which can be rearranged to solve for  $q_u$  at  $M_u = 1.25M_n$ :

$$\begin{aligned} q_u &= 2M_{ce} / \{[(L_{\text{footing}} - c) / 2]^2 (b_{\text{footing}})\} \\ &= 2(925 \text{ kip ft}) / \{[(10.5 \text{ ft} - 4 \text{ ft}) / 2]^2 (10.5 \text{ ft})\} \\ &= 16.7 \text{ ksf} \end{aligned}$$

#### B.4.1.7 CONCLUSIONS

These results demonstrate that the determination of expected bearing capacity using gravity loading can provide conservative values compared to the use of allowable bearing pressure from geotechnical studies. It also shows that it is important to evaluate the structural footing strength in addition to the soil bearing capacity as it may be the governing mechanism in the load path, particularly if the footing strength is evaluated as force-controlled.

**Table B-16 Summary of Bearing Pressure Capacities**

Methodology	Bearing Capacity
ASCE/SEI 41-17 Equation 8-1	21.0 ksf (Upper-bound)
ASCE/SEI 41-17 Equation 8-3	14.8 ksf (Upper-bound)
Footing Capacity (Deformation-Controlled)	20 ksf (limit state analysis)
Footing Capacity (Force-Controlled)	16.7 ksf (limit state analysis)

#### B.4.1.8 RECOMMENDED CHANGES

Where information on soil bearing capacity is not available in either the construction documents or a geotechnical report, prescriptive expected bearing capacity may be calculated with a 2.5 factor applied to the calculated design gravity loads.

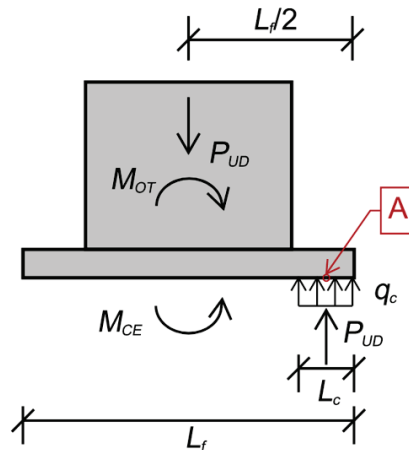
### B.4.2 Investigation of Soil Bearing Capacity Bounding

Soil bearing capacity bounding was investigated as a part of Hypothesis 2.

#### B.4.2.1 METHODOLOGY

Strength bounding was investigated to determine its effect on overturning moment capacity acceptance ratios for the retrofit footing designed using ASCE/SEI 7-10. Current ASCE/SEI 41-17 provisions permit the use of upper-bound bearing capacity for both fixed base (ASCE/SEI 41-17 § 8.4.2.3.2.1) and flexible base (ASCE/SEI 41-17 § 8.4.2.3.2.2) procedures. Table B-17 and

Table B-18 summarize the results of utilizing lower and upper bound bearing capacity when calculating overturning moment capacity. The expected moment capacity is determined using statics to sum forces about point A in Figure B-26. An example calculation for the LSP Fixed Base utilizing upper-bound bearing capacity is included below.



**Figure B-26** Overturning and resisting forces on an isolated footing.

The expected moment capacity is calculated:

$$M_{CE} = 0.5(L_f P_{UD})(1-q/q_c) \quad (\text{ASCE/SEI 41-17 Eq. 8-10})$$

where:

$$L_f = 70.5 \text{ ft} \quad (\text{footing length per Figure B-10})$$

$$A_f = 612 \text{ sq ft} \quad (\text{footing area per Table B-3})$$

$$q_{c, \text{upperbound}} = 2q_{ce} = 21 \text{ ksf} \quad (\text{ASCE/SEI 41-17 Figure 8-1a})$$

$$P_{UD} = 1662 \text{ kips} \quad (\text{Table B-20, } P_{UD} = P_G + P_E/\text{DCR})$$

$$q = P_{UD}/A_f = (1662 \text{ kips})/(612 \text{ sq ft}) = 2.71 \text{ ksf}$$

$$M_{CE} = 0.5[(70.5 \text{ ft})(1662 \text{ kips})][1-(2.71 \text{ ksf})/(21 \text{ ksf})]$$

$$= 50,964 \text{ kip ft}$$

Then, by comparing this capacity to the demand per the ETABS model, the acceptance ratio can be identified:

$$M_{\text{base}} = 269,427 \text{ kip ft}$$

$$\text{Required } m = M_{\text{base}} / M_{\text{CE}} = (269,427 \text{ kip ft}) / (50964 \text{ kip ft}) = 5.3$$

$$\text{Allowable } m = 4 \quad (\text{ASCE/SEI 41-17 Section 8.4.2.3.2.1})$$

$$\text{Acceptance Ratio} = \text{Required } m / \text{Allowable } m = 5.3/4 = 1.32$$

#### B.4.2.2 RESULTS AND CONCLUSIONS

For the fixed base case, the  $m$ -values are determined per ASCE/SEI 41-17 Section 8.4.2.3.2.1. For all remaining cases, the  $m$ -values are linearly interpolated based on the  $b/L_c$  and  $A_c/A_f$  values as described in ASCE/SEI 41-17 Table 8-3. The use of upper-bound soil bearing strength for fixed-base analysis provides reasonable results compared to ASCE/SEI 7-10 with an acceptance ratio relatively close to 1.0 as shown in Table B-17. The use of lower-bound soil bearing strength does not provide acceptable results for fixed base or flexible-base analyses, with acceptance ratios greater than 1.0 for the LSP – Method 1, using both upper or lower bound spring stiffnesses properties as shown in Table B-18.

**Table B-17 Summary of Bearing Capacity Bounding to Determine Moment Capacity**

Model	$P_{UD}$ (kip)	$q$ (ksf)	$M_{\text{base}}$ (k-ft)	Upper Bound Strength				
				$q_c$ (ksf)	$M_{\text{CE}}$ Upper (k-ft)	required $m$	allowable $m$	Acceptance Ratio
LSP - Fixed Base	1,660	2.71	269,427	21.0	50,948	5.29	4.00	1.32
LSP - Method 1 Lower Bound Stiffness (Rigid Footing)	1,586	2.59	177,978	21.0	49,002	3.63	6.00	0.61
LSP - Method 1 Upper Bound Stiffness (Rigid Footing)	1,611	2.63	224,538	21.0	49,672	4.52	6.00	0.75
LSP - $K_{50}$ 300 $M_{c,\text{foot}}$ (Rigid Footing)	1,586	2.59	111,809	21.0	49,003	2.28	6.00	0.38
LSP - $K_{50}$ 550 $M_{c,\text{foot}}$ (Rigid Footing)	1,586	2.59	146,138	21.0	49,002	2.98	6.00	0.50

(1) For more information on the  $K_{50}$  stiffnesses, see Section B.8.3.1.2.

(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

**Table B-18 Summary of Bearing Capacity Bounding to Determine Moment Capacity**

Model	P <sub>UD</sub> (kip)	q (ksf)	M <sub>base</sub> (k-ft)	Lower Bound Strength				
				q <sub>c</sub> (ksf)	M <sub>CE</sub> Lower (k-ft)	required m	allowable m	Acceptance Ratio
LSP - Fixed Base	1,660	2.71	269,427	5.25	28,284	9.53	4.00	2.38
LSP - Method 1 Lower Bound (Rigid Footing)	1,586	2.59	177,978	5.25	28,310	6.29	2.06	3.05
LSP - Method 1 Upper Bound (Rigid Footing)	1,611	2.63	224,538	5.25	28,314	7.93	2.00	3.97
LSP - K <sub>50</sub> 300M <sub>c,foot</sub> (Rigid Footing)	1,586	2.59	111,809	5.25	28,310	3.95	2.06	1.92
LSP - K <sub>50</sub> 550M <sub>c,foot</sub> (Rigid Footing)	1,586	2.59	146,138	5.25	28,310	5.16	2.09	2.50

(1) For more information on the K<sub>50</sub> stiffnesses, see Section B.8.3.1.2.

(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

For the fixed-base analysis procedure, the use of upper-bound soil bearing strength provides reasonable results compared to ASCE/SEI 7-10. If the expected or lower-bound soil bearing strength were to be used instead, the *m*-factor or footing size would have to be increased to provide comparable results. Increasing the footing size was deemed too conservative. Note that if the *m*-factors for the fixed-base procedures are increased, they could become equal to or greater than the *m*-factors used for the flexible-base procedure (ASCE/SEI 41-17 Table 8-3), which is counterintuitive to the general concept that the fixed-base procedure should provide a more conservative design. Therefore, we suggest that the soil bearing strength equivalent to the upper-bound strength continue to be used for the fixed-base procedure, in which case the terminology will be revised to specify the use of the expected soil bearing strength with a factor of 2 to account for transient, seismic loading effects. Discussions with the geotechnical community have agreed that between 1.5 and 2 would be a more accurate estimate of the overstrength due to the short transient nature of the earthquake loading.

These results can also be compared to those of the Nonlinear Static Procedure as presented in Section B.10 with acceptance ratios in Table B-41. All of the soil acceptance ratios for the NSP methods explored ranged from 0.29 to 0.74 which is more in line with the upper-bound strength for the LSP flexible-base procedures. Lower-bound soil bearing strengths do not provide acceptable results for linear procedures. In any case, design of the structural foundation should be performed with the expected bearing capacity and bounding is not needed.

### **B.4.2.3 RECOMMENDED CHANGES**

The use of upper and lower bound properties for soil bearing should be eliminated and the expected ultimate capacity,  $q_{cDA}$ , which includes a factor of 2 for short duration seismic loading, should be utilized for linear analysis procedures. The proposed commentary changes delete the statement “To allow for soil variability or uncertainty, an upper- and lower-bound approach to defining stiffness and capacity is required to evaluate the sensitivity of the structural response to these parameters.” Instead, it stresses that it is “important that geotechnical engineers report the average expected results obtained and the actual factor of safety applied to arrive at design values for soil strength and stiffness. In the past, design values recommended by geotechnical engineers were often consistent with lower-bound strengths.”

## **B.5. Foundation Overturning Capacity (Proposed ASCE/SEI 41-23 Section 8.4.4.1.1.1): Expected Vertical Load $P_U$**

### **B.5.1 Motivation**

During the investigation of Hypothesis 2, it was found that there is a lack of clarity in how to calculate the expected vertical load,  $P_{UD}$  (which is proposed to be  $P_U$  in the ballots for the upcoming ASCE/SEI 41-23), in determining the overturning moment capacity,  $M_{CE}$ , in Equation 8-10. The  $P_{UD}$  is defined in ASCE/SEI 41-17 Section 1.2.2.1 as the expected vertical load on soil at the footing interface caused by gravity and seismic loads, however further guidance on if this includes load factors or the footing dead load is unclear.

### **B.5.2 Technical Studies**

ASCE/SEI 41-17 provides multiple definitions of  $P_{UD}$ . The footing acceptance ratio is sensitive to the  $P_{UD}$  value.  $P_{UD}$  is defined in ASCE/SEI 41-17 §1.2.2.1 as both the expected vertical load on soil as well as the deformation-controlled axial force. The latter definition includes load combination factors. The following  $P_{UD}$  calculations were investigated to determine the effect on the calculation of  $M_{CE}$ .

1. Expected load (without retrofit footing dead load and no load factors)
2. Expected load (with retrofit footing dead load and no load factors)
3.  $P_{UD}$  Factored –  $0.9P_G$  (without retrofit footing dead load)
4.  $P_{UD}$  Factored –  $1.1P_G$  (without retrofit footing dead load)

While there are other conditions that could be examined, these analyses were deemed adequate to form a conclusion. Often in new construction, the weight of the footing is not included and this assumption was used for the factored load scenarios.

### **B.5.2.1 $P_{UD}$ - EXPECTED LOAD**

ASCE/SEI 41-17 § 8.4.2.3.1 states that “the expected vertical load  $P_{UD}$  is taken as the maximum action that can be developed based on a limit-state analysis considering the expected strength of the components delivering force to the footing; alternatively, the expected vertical load is determined by dividing the seismic linear elastic load by the maximum demand capacity ratio (DCR) of the components in the load path and summing with the gravity loads.” In addition, the following equation is provided:

$$P_{UD} = P_G \pm P_E/DCR$$

Expected loads were calculated based on this equation with and without the dead load of the footing included. For this case study, vertical seismic loads are essentially zero, so  $P_{UD}$  is equal to the gravity load.

### **B.5.2.2 $P_{UD}$ - FACTORED LOAD**

$P_{UD}$  is also calculated using 0.9 and 1.1 load factors in accordance with ASCE/SEI 41-17 § 7.2.2.

### **B.5.2.3 RESULTS AND CONCLUSIONS**

The models created for Hypothesis 2 were used in the parameter study.

The results of this parameter study are shown in Table B-19 (expected load without retrofit footing dead load and no load factors), Table B-20 (expected load with retrofit footing dead load and no load factors), and Table B-21 ( $P_{UD}$  factored two ways without retrofit footing dead load). The calculation of  $P_{UD}$  affects the overturning soil bearing acceptance ratio by 20% to 30%. The  $P_G$  used in these calculations does not include live load. For clarity for users and to be consistent with the original intent of these provisions, we recommend that the text be revised to clarify the nomenclature and calculation of the expected axial load to exclude any load factors (1.1 or 0.9 in ASCE/SEI 41-17 Equations 7-1 and 7-2) and to include the self-weight of the footing. No live load is to be included.

**Table B-19 Summary of Methods for Calculating  $P_{UD}$  – Expected Load (without Footing Dead Load)**

$P_{UD}$ Summary – Expected Load (without Footing Dead Load)								
Model	Initial Fundamental Period (seconds)	Expected Load (w/ DCR, without Footing DL)						
		$P_G^2$ (kips)	$P_E$ (kips)	$C_1C_2$	DCR	DCR Used	$P_{UD}$ (kips)	Footing Acceptance Ratio
LSP - Fixed Base	0.43	1295	-6.00	1.10	3.00	2.20	1,292	1.64
LSP - Method 1 Lower Bound (Rigid Footing)	0.58	1219	0.09	1.10	2.38	2.20	1,219	0.65
LSP - Method 1 Upper Bound (Rigid Footing)	0.50	1244	0.06	1.10	2.89	2.20	1,244	0.81
LSP - $K_{50}$ $300M_{lc,foot}$ (Rigid Footing)	0.65	1219	0.22	1.10	6.23	2.20	1,219	0.41
LSP - $K_{50}$ $550M_{lc,foot}$ (Rigid Footing)	0.60	1219	0.17	1.10	5.24	2.20	1219	0.54

(1) Expected Load (w/ DCR) = gravity load combined with axial seismic forces divided by maximum DCR (either with or without footing DL)

(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

**Table B-20 Summary of Methods for Calculating  $P_{UD}$  – Expected Load (including Footing Dead Load)**

$P_{UD}$ Summary – Expected Load (including Footing Dead Load)								
Model	Initial Fundamental Period (seconds)	Expected Load (w/ DCR, w/ Footing DL)						
		$P_G$ (kips)	$P_E$ (kips)	$C_{(1)}C_{(2)}$	DCR	DCR Used	$P_{UD}$ (kips)	Footing Acceptance Ratio
LSP - Fixed Base	0.43	1662	-6.00	1.10	3.00	2.20	1660	1.32
LSP - Method 1 Lower Bound (Rigid Footing)	0.58	1586	0.09	1.10	2.38	2.20	1586	0.52
LSP - Method 1 Upper Bound (Rigid Footing)	0.50	1611	0.06	1.10	2.89	2.20	1611	0.65
LSP - $K_{50}$ 300Mc,foot (Rigid Footing)	0.65	1586	0.22	1.10	6.23	2.20	1586	0.33
LSP - $K_{50}$ 550Mc,foot (Rigid Footing)	0.60	1586	0.17	1.10	5.24	2.20	1586	0.43

(1) Expected Load (w/ DCR) = gravity load combined with axial seismic forces divided by maximum DCR (either with or without footing DL).

(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

**Table B-21 Summary of Methods for Calculating  $P_{UD}$  – Factored Load (ASCE/SEI 41-17 Load Combinations without footing dead load)**

Effects of $P_{UD}$ on Overturning Action to Assess Soil Bearing Capacity					
Model	Initial Fundamental Period (seconds)	$P_{UD}$ Factored: $0.9P_G$		$P_{UD}$ Factored: $1.1P_G$	
		$P_{UD}$ (kips)	Footing Acceptance Ratio	$P_{UD}$ (kips)	Footing Acceptance Ratio
LSP - Fixed Base	0.43	1159	1.81	1530	1.42
LSP - Method 1 Lower Bound (Rigid Footing)	0.58	1097	0.72	1447	0.56
LSP - Method 1 Upper Bound (Rigid Footing)	0.50	1119	0.89	1477	0.70
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	0.65	1097	0.45	1447	0.35
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing)	0.60	1096	0.59	1447	0.46

Notes:  $P_{UD}$  factored = maximum vertical elastic forces delivered to the retrofit footing with load combinations per ASCE/SEI 41-17 7.2.2

### B.5.3 Recommended Changes

Based on the results of this study,  $P_{UD}$  used in this  $M_{CE}$  equation is recommended to be redefined as  $P_U$  to avoid confusion with other  $P_{UD}$  definitions as it includes a reduction in the earthquake axial load demand by a limit state analysis which is similar to demands to force controlled elements. For clarity for users and to be consistent with the original intent of these provisions, we recommend that the text be revised to clarify the nomenclature and calculation of the expected gravity axial load to exclude any load factors (1.1 or 0.9 in ASCE/SEI 41-17 Equations 7-1 and 7-2) and to include the self-weight of the footing. Therefore, we recommend that in defining the gravity load  $P_G$  used in this equation, it is not recommended to reference ASCE/SEI 41-17 Equation 7-1 which includes a load factor. No live load is to be included.

## **B.6. Seismic Overturning Resisted by Axial and Moment Action (Proposed ASCE/SEI 41-23 Section 8.4.4.1.1.3): Bi-Directional Loading**

### **B.6.1 Motivation**

Currently in the standard, overturning acceptance is addressed only for unidirectional moment for a rectangular or I-shaped footing. Provisions are required to allow for assessment of footings under bi-directional loading.

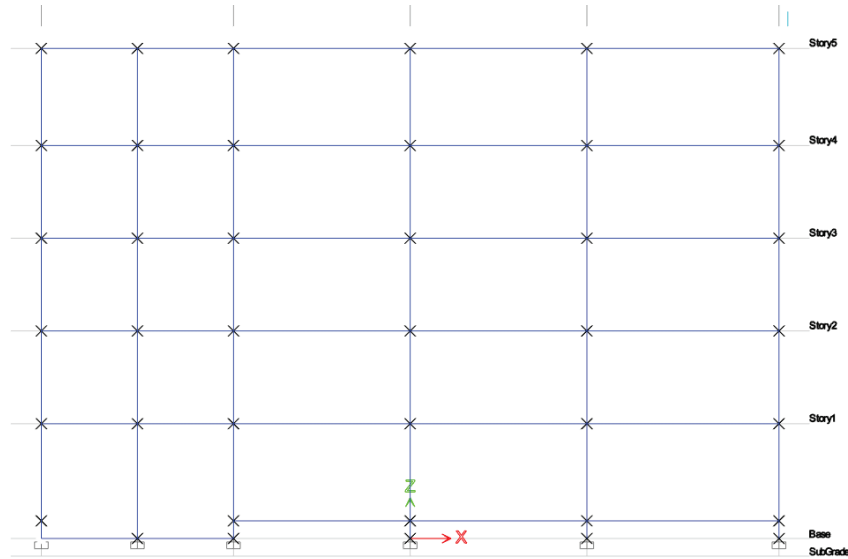
### **B.6.3 Technical Studies**

The design of a retrofit footing for loading in the east-west direction was used to investigate ASCE/SEI 41-17 foundation analysis provisions for a corner wall condition with axial earthquake load from frame action including L-shaped foundation and other complexities. Retrofit footings designed using ASCE/SEI 7-10 and ASCE/SEI 41-17 were compared similar to other hypotheses.

#### **B.6.3.1 METHODOLOGY AND ASSUMPTIONS**

Model Summary:

- E-W Direction Seismic Loading
- Fixed Base Model
- Columns fixed at base
- Retrofit shear walls modeled as frame element fixed at base with rigid links to adjacent columns
- Includes 5% accidental torsion (ASCE/SEI 41-17 §7.2.3.2.1 & ASCE/SEI 7-10 §12.8.4.2)
- For ASCE/SEI 7-10,  $\rho = 1.0$  consistent with previous case studies
- Bi-directional loading
  - ASCE/SEI 7-10 §12.5.4 requires design for 100% in primary direction and 30% in perpendicular direction
  - ASCE/SEI 41-17 §7.2.5 only requires multidirectional effects for certain conditions.

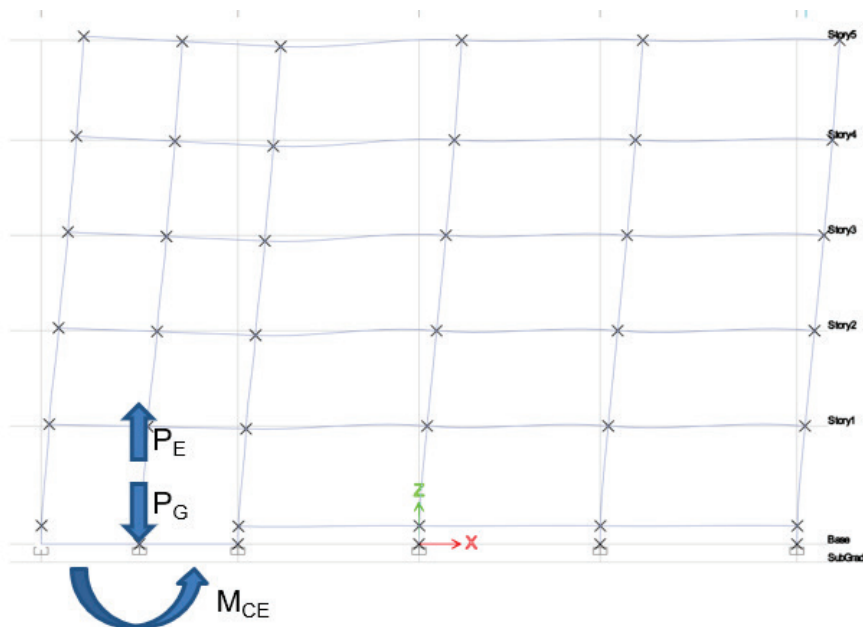


**Figure B-27** Elevation from fixed Base model used for Bi-Directional loading analysis, the corner wall is between Gridlines A and A.5.

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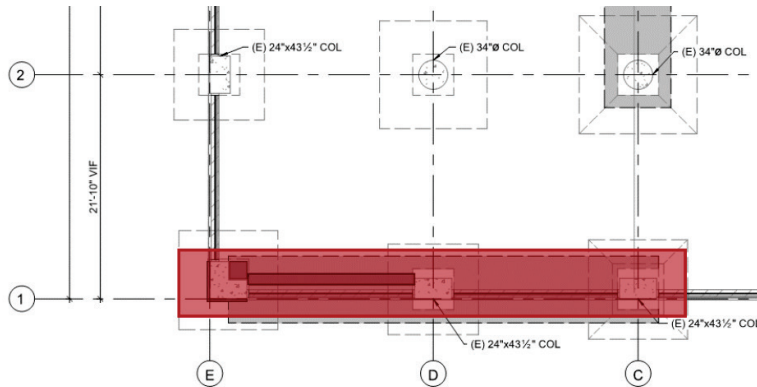
### B.6.3.2 RECTANGULAR RETROFIT FOOTING

For loading in east direction, the net earthquake reaction is upward due to frame action, so a retrofit footing extending to adjacent bays is investigated by evaluating and comparing the retrofit footing using ASCE/SEI 7-10 and ASCE/SEI 41-17.



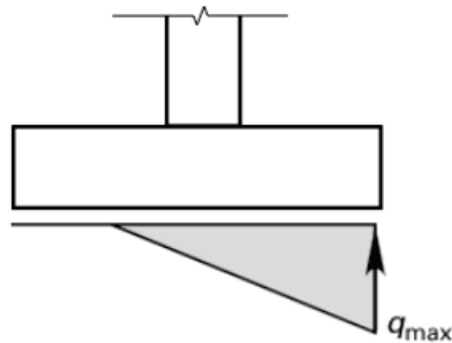
**Figure B-28** Frame elevation showing forces for loading in the east direction.

The rectangular footing extends one bay beyond the shear wall, see Figure B-29. This engages additional dead load that reduces the uplift at the foundation. The actual footing, comprised of existing 10'-6" square footings and new connecting grade beams, is idealized as rectangular, see Figure B-11.



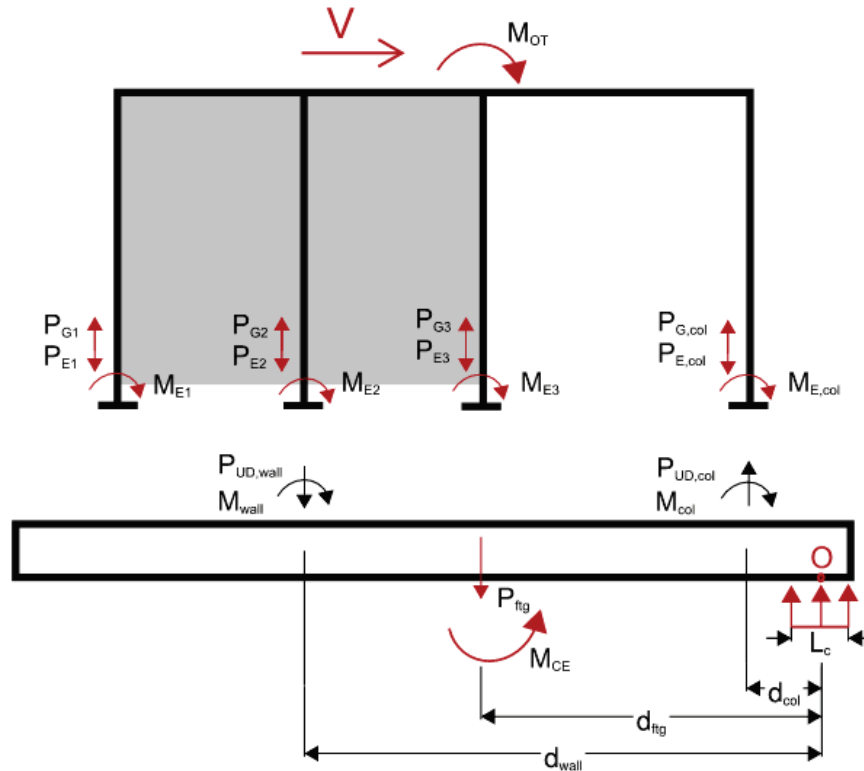
**Figure B-29 Foundation plan showing Idealized proposed rectangular foundation at corner wall.**

For ASCE/SEI 7-10 design, the bearing pressure is evaluated using an elastic triangular distribution assuming the new footing is rigid compared to the soil.



**Figure B-30 Elastic triangular bearing pressure distribution at rectangular footing.**

For ASCE/SEI 41-17, the expected moment capacity  $M_{CE}$  can be derived by summing moments about the center of the resulting compression block from  $P_{UD}$ , while neglecting the restoring force from perpendicular slabs at each floor as shown in Figure B- 31 and in the following equations. The footing would transfer the loads if designed as elastic/force-controlled so while there are other possible cases to study, the behavior described is one realistic possibility.



**Figure B- 31 Forces acting at concrete footing for expected moment capacity derivation that neglects the restoring force from perpendicular slabs at each floor.**

Using statics, the expected moment capacity can be determined:

$$P_{G,wall} = P_{G1} + P_{G2} + P_{G3}$$

$$P_{E,wall} = P_{E1} + P_{E2} + P_{E3}$$

$$P_{UD,wall} = P_{G,wall} \pm P_{E,wall} / DCR$$

$$M_{wall} = M_{E1} + M_{E2} + M_{E3} + (P_{E1} - P_{E3})L_{wall} / 2$$

$$P_{UD,col} = P_{G4} \pm P_{E4} / DCR$$

$$M_{col} = M_{E4}$$

$$M_{CE} = \sum P_{UD,i}d_i = P_{UD,wall}d_{wall} + P_{ftg}d_{ftg} + P_{UD,col}d_{col}$$

$$M_{OT} = M_{wall} + M_{col}$$

Using the equations determined above, we can determine the expected moment capacity:

$$M_{CE} = \sum P_{UD,i}d_i = P_{UD,wall}d_{wall} + P_{ftg}d_{ftg} + P_{UD,col}d_{col}$$

Where:

$$P_{UD,wall} = P_{G,wall} +/- P_{E,wall}/DCR$$

Where:

$$P_{G,wall} = P_{G1}+P_{G2}+P_{G3} = 533 \text{ kips}$$

$$P_{E,wall} = P_{E1}+P_{E2}+P_{E3} = -1285 \text{ kips (this is due to the frame action or the coupling between the wall and column)}$$

$$P_{UD,wall} = (533 \text{ kips}) +/- (1285 \text{ kips}/2.2) = -51 \text{ kips}$$

$$P_{UD,col} = P_{G4} +/- P_{E4}/DCR = (221 \text{ kips}) +/- (-44 \text{ kips}/2.2) = 201 \text{ kips}$$

$$M_{col} = M_{E4} = 2036 \text{ kip ft}$$

$$M_{CE} = (-51 \text{ kips})(36.1 \text{ ft}) +(201 \text{ kips})(4.1 \text{ ft}) +(298 \text{ kips})(25 \text{ ft}) = 6440 \text{ kip ft}$$

And the overturning moment demand:

$$M_{OT} = M_{wall} + M_{col}$$

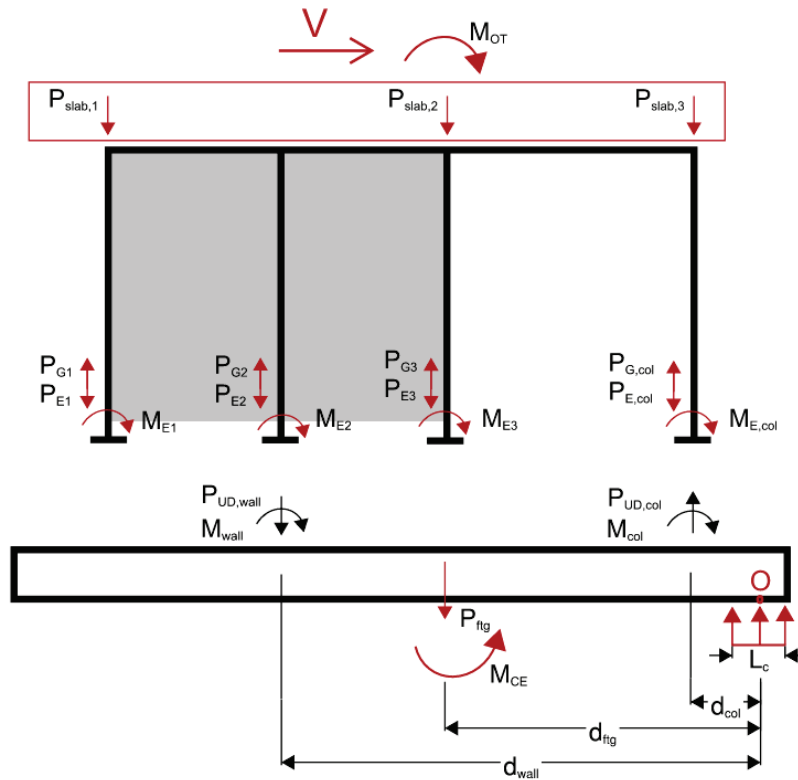
Where:

$$\begin{aligned} M_{wall} &= M_{E1}+M_{E2}+M_{E3}+(P_{E1}-P_{E3})L_{wall}/2 \\ &= 154,385 \text{ kip ft} \end{aligned}$$

$$M_{OT} = M_{wall} + M_{col} = (154,385 \text{ kip ft}) + (2036 \text{ kip ft}) = 156,421 \text{ kip ft}$$

As  $M_{OT}$  is greater than  $M_{CE}$ , this retrofit footing is not adequate.

Alternatively,  $M_{CE}$  can be derived by summing moments about the center of the resulting compression block from  $P_{UD}$  and including the restoring force from perpendicular slabs at each floor.

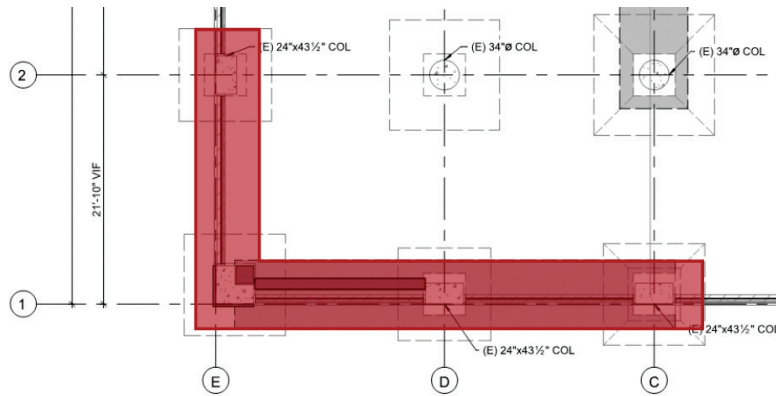


**Figure B-32** Forces acting at concrete footing for expected moment capacity derivation that includes the restoring force from perpendicular slabs at each floor.

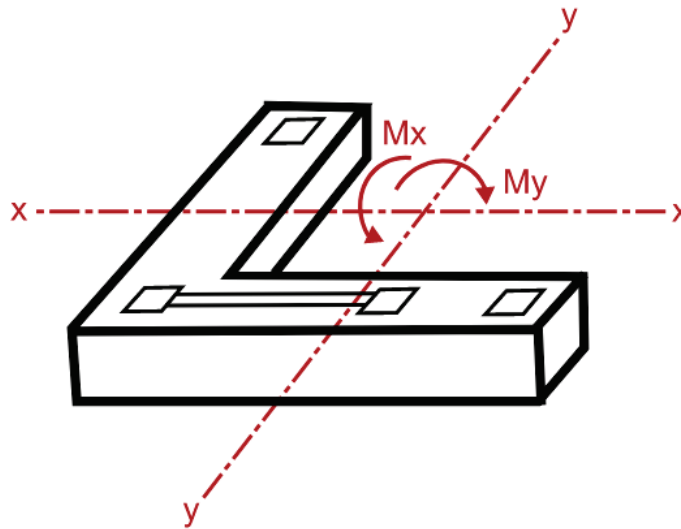
Details calculations are not provided for this option but would give similar results to the calculations above. For all scenarios with ASCE/SEI 7-10 or ASCE/SEI 41-17, a rectangular retrofit footing is not acceptable, so an L-shaped footing, which extends to adjacent bay perpendicular to retrofit shear wall, is investigated.

### B.6.3.3 L-SHAPED RETROFIT FOOTING

The retrofit L-shaped footing extends one bay perpendicular to the retrofit shear wall and is evaluated for the effects of frame action as well as concentrated moment. The actual footing, comprised of existing 10'-6" square footings and new connecting grade beams, is idealized as two rectangular shapes forming an L, see Figure B-12.



**Figure B-33** Foundation plan showing idealized proposed L-shaped foundation at corner wall.



**Figure B-34** L-shaped footing with axes and moment shown.

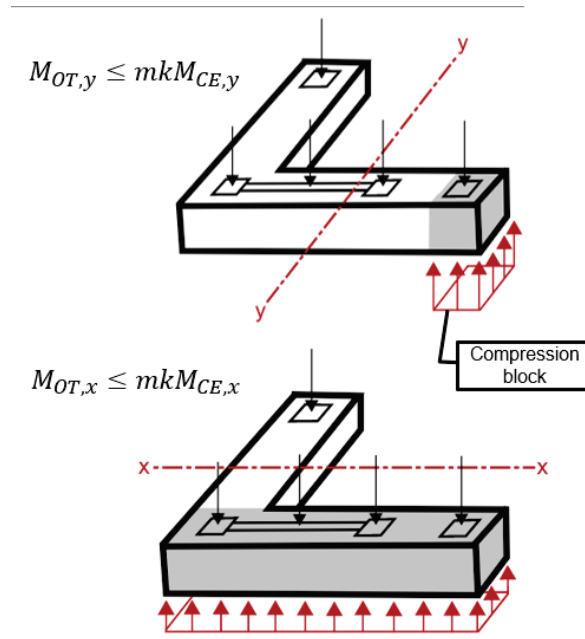
For ASCE/SEI 7-10, the footing is assumed to be rigid and loading includes moment in each direction as well as axial loads, which are combined to determine the maximum bearing pressure described in the equation below:

$$M_{OT} = \left(\frac{P}{A}\right) + \left(\frac{M_y x}{I_y}\right) + \left(\frac{M_x y}{I_x}\right)$$

The acceptance ratio represents the applied bearing pressure compared to the allowable of 4.66 ksf.

For ASCE/SEI 41-17, the expected moment capacity  $M_{CE}$  is calculated in each direction with  $P_{UD}$  determined similar to a rectangular footing but including the column in the perpendicular direction. Based on the current provisions which dictate that overturning is based on a rectangular

compression block, and provided that the footing is strong enough to engage the adjacent column weight, this approach is reasonable.



**Figure B-35** Compression blocks for each direction at the L-Shaped Footing.

$M_{CE}$  is calculated as the sum of moments about center of calculated compression block from  $P_{UD}$ . The combined acceptance ratio is determined by square root sum of squares below per previous direction. However, note that the ASCE/SEI 41-23 committee is currently removing the square root as described in Section B.6.4.

$$\sqrt{\left(\frac{M_{OT,y}}{mkM_{CE,y}}\right)^2 + \left(\frac{M_{OT,x}}{mkM_{CE,x}}\right)^2} \leq 1.0$$

▪

#### B.6.3.4 RESULTS AND CONCLUSIONS

The results of these studies are summarized in Table B-22 and Table B-23 below. These studies used linear static procedure with fixed base assumptions for loading in the East-West direction. Further discussion of East-West loading can be found in section B.3.6.3. Conditions were evaluated with and without the contributions of slabs and/or perpendicular frames to resist overturning in addition to the bidirectional load cases described in Section B.6.3.1.

**Table B-22 Retrofit footing uplift acceptance ratios for comparison of bi-directional load of effects for East-West Direction**

Retrofit Footing Uplift Acceptance Ratios (Linear Static Procedure, E-W, Fixed Base)					
Foundation Type	ASCE/SEI 7-10 <sup>(2)</sup>	ASCE/SEI 41-17 <sup>(1)(2)</sup>	ASCE/SEI 41-17 (w/ slab) <sup>(2)</sup>	ASCE/SEI 41-17 (100%+30%) <sup>(1)</sup>	ASCE/SEI 41-17 (100%+30% w/ grade beam)
Rectangular	1.34	5.93	2.03	7.21	4.27
L-Shaped	0.86	1.88	1.07	2.00	1.68

<sup>(1)</sup> This excludes the contribution of perpendicular frames to resist overturning

<sup>(2)</sup> This does not include bi-directional load affects.

**Table B-23 Retrofit footing compression acceptance ratios for comparison of bi-directional load of effects for East-West Direction**

Retrofit Footing Compression Acceptance Ratios (Linear Static Procedure, E-W, Fixed Base)					
Foundation Type	ASCE/SEI 7-10 <sup>2</sup>	ASCE/SEI 41-17 <sup>(1)(2)</sup>	ASCE/SEI 41-17 (w/ slab) <sup>(2)</sup>	ASCE/SEI 41-17 (100%+30%) <sup>(1)</sup>	ASCE/SEI 41-17 (100%+30% w/ grade beam)
Rectangular	1.16	2.43	1.17	2.57	2.06
L-Shaped	0.44	1.76	0.86	1.85	1.51

<sup>(1)</sup> This excludes the contribution of perpendicular frames to resist overturning

<sup>(2)</sup> This does not include bi-directional load affects.

These results indicate the following:

- ASCE/SEI 41-17 provisions result in larger footing sizes (or higher acceptance ratios) compared with ASCE/SEI 7-10. As can be seen in Table B-22 and Table B-23, the Acceptance Ratios for the ASCE/SEI 41-17 case without the slab or adjacent frames is higher than that of the ASCE/SEI 7-10 results. Some of these results are also repeated and further explored in Table B-9.
- Uplift at the corner column condition, where seismic axial loads contribute to uplift, is significant with linear ASCE 41 procedures. The effect of perpendicular framing to resist uplift should be included to develop a reasonable retrofit foundation design. This is evident in the comparison between the ASCE/SEI 41 results that exclude versus include the slab in Table B-22. Further, from Table B-22, it can be seen that the L-shaped footing acceptance ratios are lower than those of the rectangular footings, indicating that there are restorative effects from including the three-dimensional structure.
- Guidance should be provided for the ASCE/SEI 41-17 user to assist with  $M_{CE}$  determination for different foundation configurations modeled as fixed-base (statics with a soil bearing compression block due to rocking).

## **B.6.4 Recommended Changes**

The recommended changes include adding new provisions for determining foundation overturning capacity where the footing is non-rectangular and for bi-directional moments on the footing. Currently in the standard, overturning acceptance is addressed simplistically only for unidirectional moment for a rectangular or I-shaped footing. In addition, a new methodology is outlined in the proposed commentary for evaluating foundations where the footing is required to resist overturning moments simultaneously about the two horizontal principal axes of the footing. This methodology is applicable to isolated footings of any plan geometry.

## **B.7 Acceptance Criteria for the Structural Footing (Proposed ASCE/SEI 41-23 Section 8.4.4.1.1.2.2)**

### **B.7.1 Motivation**

There is currently no specific requirement or acceptance criteria for checking the structural footing in Chapter 8. Evaluation of the concrete foundation structural component is specified in the concrete chapter (§10.12.3) where demands to the foundation are treated as force-controlled. Case studies have shown that this requirement could be overly conservative and as a result, leaves the possibility that the adequacy and strength of the footings may not be checked. Specific requirements have been introduced specifying the magnitude and application of the soil pressures as loads to the footing.

### **B.7.3 Technical Studies**

See discussion and results of Hypothesis 1 and 2.

### **B.7.4 Recommended Changes**

The recommended change clarifies what the acceptance criteria is for a structural footing. The proposed language points to evaluation per the material chapters and defines the appropriate demands. Alternatively, it allows for evaluation of the footing as force controlled for the soil pressure distribution under specific conditions.

## **B.8 Soil Stiffness for Shallow Foundations (Proposed ASCE/SEI 41-23 Sections 8.4.5.1, 8.4.5.2.1.2)**

### **B.8.1 Motivation**

ASCE/SEI 41-17 has three methods (Methods 1, 2 and 3) for determining and modelling soil spring stiffness as well as lower and upper bounding requirements. The goal is to simplify modelling approaches and eliminate redundant or unused options.

## B.8.3 Technical Studies

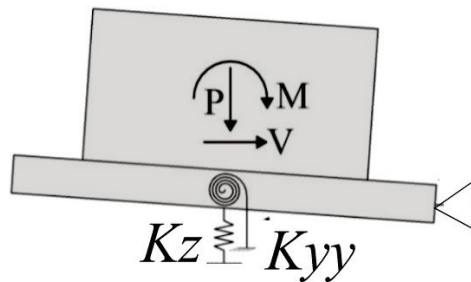
### B.8.3.1 COMPARISON OF METHODOLOGIES FOR DERIVING SOIL SPRINGS

The following soil spring methodologies are investigated. Studies conducted under Hypothesis 2 indicated that ASCE/SEI 41-17 Method 2 was not practical nor necessary since it must be calibrated to Method 1.

- Method 1 – ASCE/SEI 41-17 Figure 8-2
- $K_{50}$  Stiffness (see Section B.8.3.1.2)
- Method 3 – ASCE/SEI 41-17 Equation 8-11

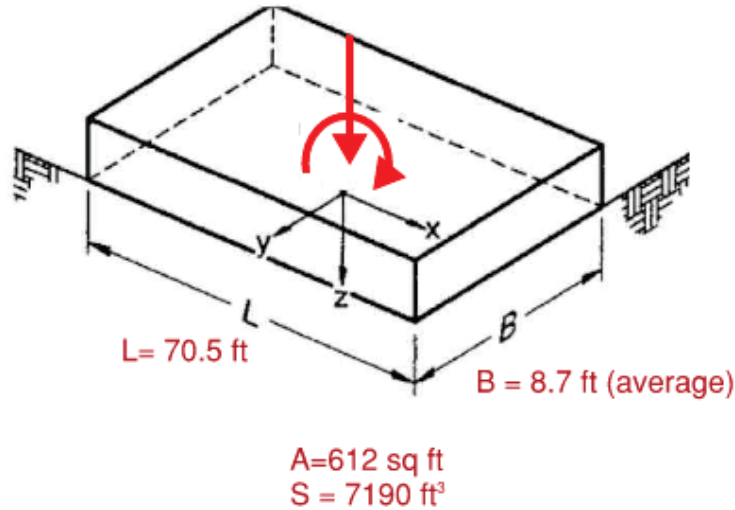
#### B.8.3.1.1 Method 1 Soil Stiffness Derivation

The existing, new, or retrofit footing is treated as a rigid body for the Method 1 spring stiffness derivations. Method 1 uses uncoupled moment and axial springs to model rigid foundations such that moment and shear behaviors are independent of axial load. Shear springs may also be used. However, in this case, and for all methods in this investigation, lateral movement is restrained within the analysis model. A graphical representation of the Method 1 springs is shown in Figure B-36. The parameters described in this figure,  $K_{yy}$  and  $K_z$ , correspond to the uncoupled spring stiffness coefficients in overturning (rotation) and in the vertical direction, respectively per ASCE/SEI 41-17 Figure 8-2.



**Figure B-36 Method 1 foundation springs (adapted from FEMA P-2006).**

The footing geometric parameters are shown in Figure B-37.



**Figure B-37 Retrofit footing dimensions for soil spring calculations.**

The shear modulus is calculated utilizing ASCE/SEI 41-17 Equation 8-5 with the standard penetration test blow count provided by the geotechnical engineer. The axial spring,  $K_z$ , and rotational spring,  $K_{yy}$ , are calculated using the equations in ASCE/SEI 41-17 Figure 8-2. The example calculations below are for the retrofit footing; a similar procedure is done for the individual existing footings supporting existing columns throughout the rest of the building. The stiffness coefficients used in ETABS are summarized in Table B-24. Upper and lower bound stiffness is defined as twice the expected stiffness and one-half the calculated stiffness respectively in accordance with ASCE/SEI 41-17 § 8.4.2.

The initial shear modulus was determined to be:

$$G_0 = \sim 120 p_a (N_{60})^{0.77}$$

Where:

$$N_{60} = 25.0 \quad \text{(per the geotechnical report)}$$

$$p_a = 2.12 \text{ ksf} \quad \text{(atmospheric pressure)}$$

$$G_0 = \sim 120 (2.12)(25)^{0.77} = 3028 \text{ ksf}$$

Then, the shear modulus can be determined using ASCE/SEI 41-17 Table 8-2:

$$G/G_0 = 0.3 \quad \text{(Site Class D, } S_{xs}/2.5 = 0.6)$$

$$G = 0.3G_0$$

$$= 0.3 (3028 \text{ ksf}) = 908 \text{ ksf}$$

The spring stiffness coefficients can then be determined from ASCE/SEI 41-17 Figure 8-2:

$$K_z = GB/(1-\nu)[1.55(L/B)^{0.75} + 0.8]$$

Where:

$$\nu = 0.35 \quad \text{(per the geotechnical report)}$$

$$L = 70.5 \text{ ft}$$

$$B = 8.7 \text{ ft (average)}$$

$$K_z = (908 \text{ ksf})(8.7 \text{ ft})/(1-0.36)[1.55(70.5/8.7)^{0.75} + 0.8]$$

$$= 100167 \text{ kip/ft} = 8347 \text{ kip/in}$$

And

$$K_{yy} = GB^3/(1-\nu)[0.47(L/B)^{2.4} + 0.034]$$

$$= (908 \text{ ksf})(8.7 \text{ ft})^3/(1-0.36)[0.47(70.5/8.7)^{2.4} + 0.034]$$

$$= 786,390,000 \text{ kip-in/radian}$$

**Table B-24 Method 1 Soil Spring Stiffnesses for ETABs for the Retrofit and Existing Footings**

Method 1 Soil Spring Stiffnesses (Kz in kip/in and Kyy in kip-in/radian)				
		Lower Bound (0.5x)	Expected	Upper Bound (2x)
Retrofit	K <sub>z</sub>	4174	8347	16694
	K <sub>yy</sub>	393,174,556	786,349,112	1,572,698,224
(E) Ftg	K <sub>z</sub>	1437	2874	5747
	K <sub>yy</sub>	4,892,313	9,784,627	19,569,254

### B.8.3.1.2 K<sub>50</sub> Soil Stiffness Derivation

K<sub>50</sub> boundary rotation stiffness assumes that 50% of the moment capacity is mobilized and accounts for non-service level actions and displacements (EQ actions) and includes gapping between soil and footing. The calculation discussed in this section is based on the findings provided in “*Validation of ASCE/SEI 41-13 Modeling Parameters and Acceptance Criteria for Rocking Shallow Foundations*” (by Hakhamaneshi et al. dated May 2016).

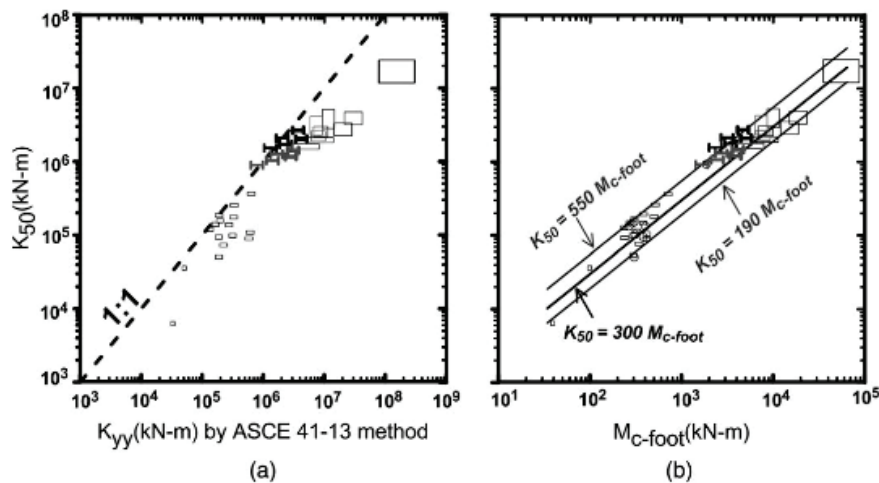
Figure B-38 shows the derivation of the rocking moment capacity used to determine stiffness parameters.

$$M_{c-foot} = \frac{PL}{2} \left( 1 - \frac{q}{q_c} \right)$$

from the two methods would be more similar. [Deng et al. \(2014\)](#) observed that the stiffness,  $K_{50}$ , was approximately proportional to the rocking moment capacity when 50% of the capacity is mobilized, and that the ratio  $K_{50}/M_{c-foot}$  ranged from 230 to 460. For design of rectangular rocking footings, they suggested that  $K_{50} \approx 300M_{c-foot}$ . Figure 3b plots the measured  $K_{50}$  and the  $M_{c-foot}$  values from the data generated by [Johnson \(2012\)](#) and the MAHS test series. It is noted that about 68% of the data points (mean plus/minus one standard deviation) lie between the  $K_{50}/M_{c-foot}$  ratios of 190 and 550. As it will be shown later, I-shaped footings with a larger MAR were found to have larger  $K_{50}/M_{c-foot}$  ratios than other footings.

**Figure B-38 Rocking moment capacity equation.**

The test data within this report determined that the rotation stiffness of I-shaped footings largely fell within  $K_{50}/M_{c-foot}$  ratios of 190 and 550 depending on the missing area ratio (MAR). Based on the configuration of the retrofit footing investigated within this study,  $K_{50} = 300M_{c-foot}$  is used for lower bound stiffness and  $K_{50} = 550M_{c-foot}$  for upper bound stiffness values. The correlation between rocking moment capacity and stiffness based on testing is shown in Figure B-39.



**Figure 3.** (a) Measured  $K_{50}$  vs. ASCE 41-13 method, (b) measured  $K_{50}$  vs. measured  $M_{c-foot}$ .

**Figure B-39 Rocking moment to stiffness correlation.**

Table B-25 shows the calculations for the derivation of both upper and lower bound  $K_{50}$  values used in the proceeding analyses. The expected  $q_{CE}$  is used for the stiffness calculations.

**Table B-25 Lower and Upper Bound  $K_{50}$  Soil Spring Derivations**

	Lower-Bound		Upper Bound	
	Retrofit	(E) Footing	Retrofit	(E) Footing
$P_{UD}$ (kips)	1,440	680	1,440	605
$q_{CE}$ (ksf)	10.5	10.5	10.5	10.5
$A_c/A$	0.22	0.59	0.22	0.52
$M_{c,foot}$ (k-ft)	39,385	1,473	39,385	1,516
$K_{50} = 300M_{c,foot}$ (kip-ft/radian)	$11.8 \times 10^6$	442,000	$21.7 \times 10^6$	834,000
$K_{yy}$ (kip-in/radian)	$142 \times 10^6$	$5.3 \times 10^6$	$260 \times 10^6$	$10 \times 10^6$
$K_z$ (kip-in)	4,174	1,437	4,174	1,437

**B.8.3.1.3 Method 3 Soil Stiffness Derivation**

Method 3 soil springs utilize decoupled Winkler springs. Method 3 diverges from Method 2 because it is intended for shallow foundations not rigid relative to the soil with uniform springs beneath a footing. The stiffness derivation in accordance with ASCE/SEI 41-17 Equation 8-11 is shown in the calculations below. Note that this stiffness calculation requires the footing width,  $B_f$ , which is not defined for a mat foundation.

The unit subgrade spring coefficient was determined:

$$k_{sv} = 1.3G/[B_f(1-\nu)] \quad (\text{ASCE/SEI 41-17 Equation 8-11})$$

Where:

$$G = 908 \text{ ksf} \quad (\text{calculated in Section B.8.3.1.1})$$

$$\nu = 0.35 \text{ ksf} \quad (\text{Poisson's Ratio})$$

$$B_f = 8.7 \text{ ft (average)}$$

$$\begin{aligned} k_{sv} &= 1.3(908 \text{ ksf})/[(8.7 \text{ ft})(1-0.35)] \\ &= 209 \text{ kips/ cubic ft} \end{aligned}$$

Then, the stiffness per spring can be determined:

$$k = k_{sv} B_f l_i$$

where

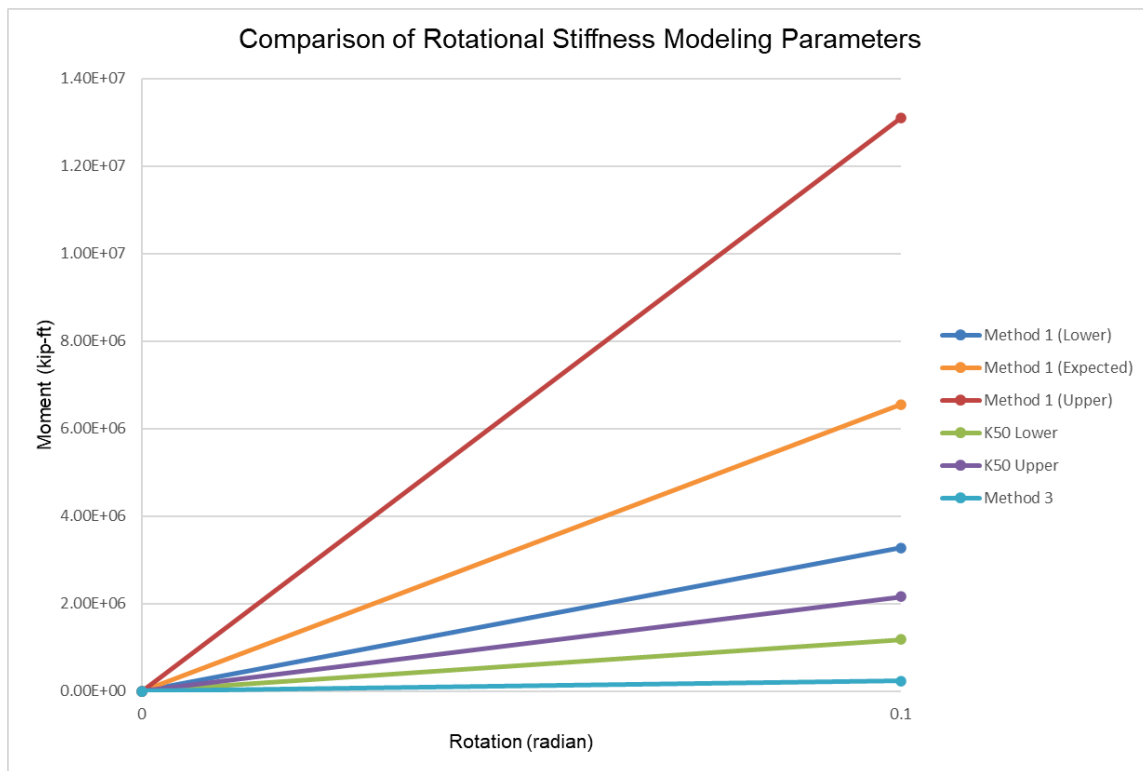
$$l_i = 2.94 \text{ ft} \quad (\text{distance between springs})$$

$$k = (209 \text{ kips/cubic foot})(8.7 \text{ ft})(2.94 \text{ ft})$$

$$= 5337 \text{ k/ft}$$

### B.8.3.1.4 Stiffness Comparison

Figure B-40 provides a comparison of the rotational stiffness values based on the derivations summarized in the previous sections. For comparison purposes, the rotations of 0 and 0.1 radians were selected (x-axis). The Method 1 moments at the selected rotations are determined by using the rotational spring stiffness values,  $K_{yy}$ , in Table B-24 (as the moment is the product of the  $K_{yy}$  and the rotation). The  $K_{50}$  moments are the product of the retrofit condition  $K_{50}$  stiffnesses in Table B-25 and the selected rotations. As previously described, Method 3 Equation 8-11 provides a translational stiffness, which is applied over point springs along the footing. To compare with Method 1 and  $K_{50}$  values, the resulting rotational stiffness was calculated based on the overturning moment and rotation measured at the ends of the shear wall.



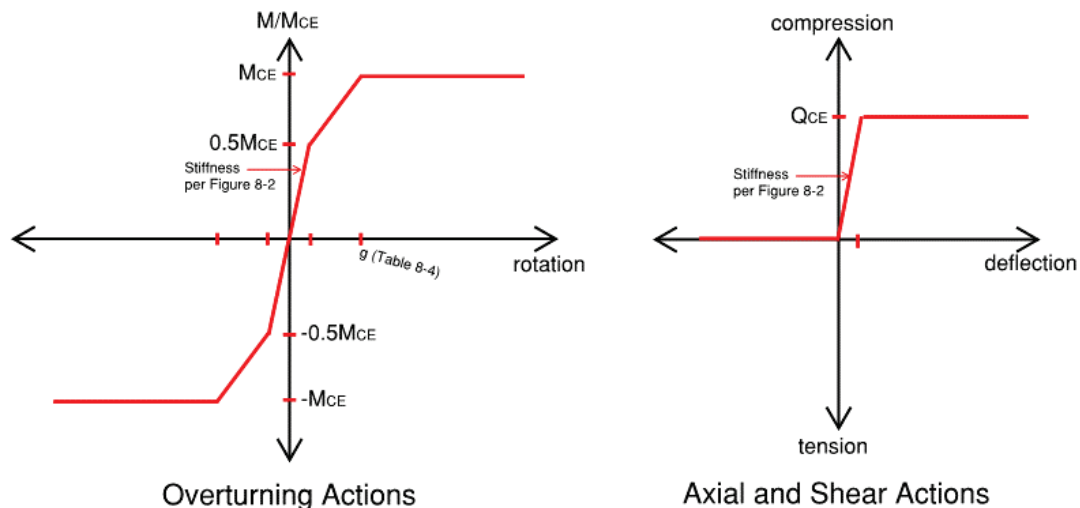
**Figure B-40 Comparison of rotational stiffness modeling parameters.**

## B.8.3.2 MODELING PARAMETERS AND ACCEPTANCE CRITERIA FOR COMPARISON OF METHODOLOGIES FOR DERIVING SOIL SPRINGS

### B.8.3.2.1 Method 1 – Modeling Parameters and Acceptance Criteria

For the linear application of Method 1, the springs are defined as linear with the stiffness calculated as discussed above. The capacity of the soil is then evaluated in accordance with ASCE 41-17 § 8.4.2.3.2. The upper-bound capacity is permitted for compression in accordance with ASCE 41-17 § 8.4.2.3.2 and  $m$ -factors are applied as specified in ASCE 41-17 Table 8-3.

For an NSP analysis, ASCE 41-17 § 8.4.2.3.3 specifies the modeling parameters for the soil springs and references Figure 8-4 and Table 8-4, which are used to define the springs shown in Figure B-41. The moment capacity,  $M_{CE}$ , is calculated in accordance with ASCE 41-17 Equation 8-10 and the expected soil bearing capacity is utilized for axial compression actions. Nonlinear acceptance criteria is based on overall footing rotation at the target displacement as specified in ASCE 41-17 Table 8-4.



**Figure B-41 Method 1 NSP modeling parameters.**

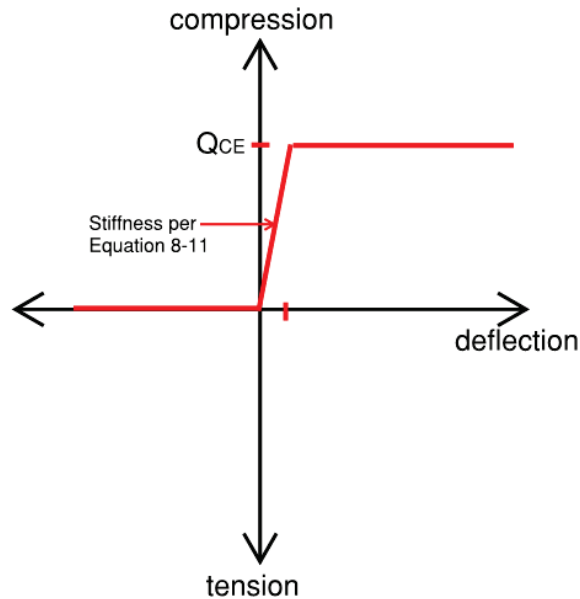
### B.8.3.2.2 $K_{50}$ Stiffness – Modeling Parameters and Acceptance Criteria

$K_{50}$  boundary rotation stiffness is applied in the models similarly to Method 1. Acceptance criteria provided in ASCE/SEI 41-17 Table 8-3 is used to evaluate the footing acceptance ratio similar to the Method 1 approach.

### B.8.3.2.3 Method 3 Stiffness – Modeling Parameters and Acceptance Criteria

The capacity of the soil springs for Method 3 is noted in ASCE/SEI 41-17 § 8.4.2.5.2 as equal to the expected bearing capacity of the soil in compression and zero in tension. A representative Method 3 soil spring is shown in Figure B-42. The acceptance criteria is based on the rotation limits of ASCE/SEI 41-17 Table 8-4. The rotation modeling parameters noted in ASCE/SEI 41-17 Table 8-4

are not utilized in Method 3, since the springs are for axial actions. The rotation is dependent on the axial soil stiffness and the rigidity of the footing.



**Figure B-42 Method 3 NSP modeling parameters.**

The nonlinear static analysis results are discussed in the following sections and compared against the linear analysis.

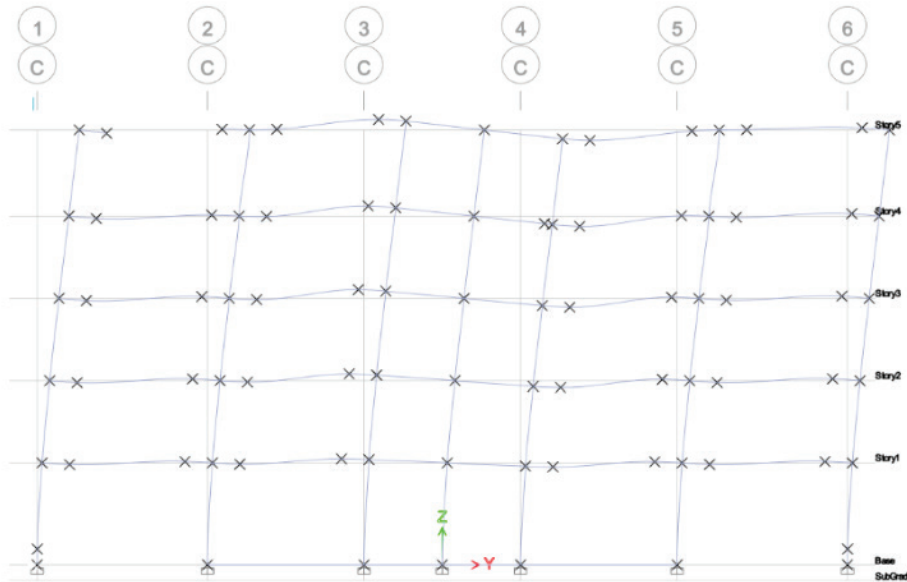
### **B.8.3.3 EFFECTS ON SUPERSTRUCTURE AND FOUNDATION METHODOLOGIES FOR DERIVING SOIL SPRINGS**

This section evaluates the effect of lower or upper-bound soil stiffness on soil bearing due to overturning and forces in the superstructure. The following analysis models are analyzed with the ASCE/SEI 7-10 designed retrofit footing. The Case 1 and Case 2 models are analyzed for both lower and upper-bound stiffness. Upper-bound bearing strength capacity is used for all cases.

- Case 1: LSP – Fixed Base Condition
- Case 2: LSP – Flexible Base Condition (Method 1)
- Case 3: LSP – Flexible Base Condition ( $K_{50}$  Stiffness)
- Case 4: NSP – Flexible Base Condition (Method 3)

#### **B.8.3.3.1 Case 1: LSP – Fixed Base Condition**

The seismic base shear based on the same site-specific design criteria used in Hypothesis 1, which corresponds to an  $S_{XS}$  of 1.5 and a base shear of 1.32 times the seismic weight of the building (7,200 kips). This is also true for all other LSP models contained within this hypothesis.



**Figure B-43 LSP analysis model with fixed base.**

The retrofit footing was then evaluated for bearing pressure due to overturning using ASCE/SEI 41-17 Equation 8-10 as shown in the calculations in Section B.4.2.1 and summarized below.

The expected moment capacity is calculated:

$$M_{CE} = 0.5(L_f P_{UD})(1-q/q_c) \quad (\text{ASCE/SEI 41-17 Eq. 8-10})$$

$$= 50,964 \text{ kip ft}$$

Then, by comparing this capacity to the demand per the ETABS model, the acceptance ratio can be identified:

$$M_{\text{base}} = 269,427 \text{ kip ft}$$

$$\text{Required } m = M_{\text{base}} / M_{CE} = (269,427 \text{ kip ft}) / (50964 \text{ kip ft}) = 5.3$$

$$\text{Allowable } m = 4 \quad (\text{ASCE/SEI 41-17 Section 8.4.2.3.2.1})$$

$$\text{Acceptance Ratio} = \text{Required } m / \text{Allowable } m = 5.3/4 = 1.32$$

The overturning moment capacity is dependent on the expected vertical load  $P_{UD}$ . Further discussion on the calculation of  $P_{UD}$  is provided in a Section B.5. For this and subsequent calculations,  $P_{UD}$  is equal to the unfactored, expected vertical load including the self-weight of the footing.

These fixed base results are compared against the ASCE/SEI 7-10 allowable bearing pressure calculation. For the purposes of this evaluation, the site-specific seismic  $S_{DS}$  of 1g is used. The redundancy factor,  $\rho$ , is taken as 1.0. The base shear is calculated including the R-factor for a special concrete shear wall ( $R=6$ ). ASD load cases are utilized to evaluate the allowable bearing capacity for comparison.

The acceptance ratio for the LSP Analysis Results with Fixed base using ASCE/SEI 7-10:

$$q_{max} = 2P_u / (3 B_f e') \quad (\text{footing pressure with } e > L/6)$$

where

$$B_f = 8.7 \text{ ft (average)} \quad (\text{footing width per Figure B-10})$$

$$L_f = 70.5 \text{ ft} \quad (\text{footing length per Figure B-10})$$

$$M_u = 18,365 \text{ kip ft} \quad (\text{ASD D+L Load Case from ETABS with 25\% reduction in LL})$$

$$P_u = 997 \text{ kips} \quad (\text{Load from ETABS, } 0.6D+0.7E)$$

$$e' = L_f / 2 - e$$

where

$$L_f / 6 = 11.8 \text{ ft}$$

$$e = M_u / P_u = 18.4 \text{ ft} > L/6$$

$$e' = L_f / 2 - e = (70.5 \text{ ft}) / 2 - 18.4 \text{ ft} = 16.85 \text{ ft}$$

$$q_{max} = 2(997 \text{ kips}) / (3 (8.7 \text{ ft})(16.85 \text{ ft}))$$

$$= 4.55 \text{ ksf}$$

$$\text{Acceptance ratio} = q_{max} / q_{allow}$$

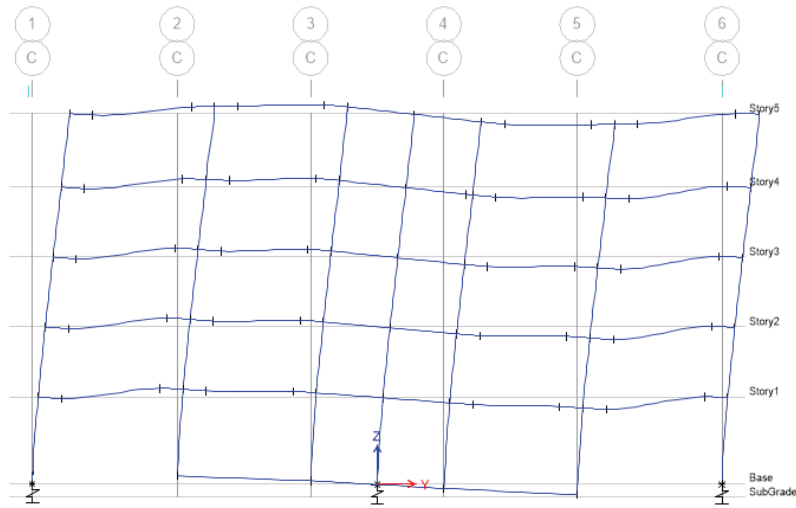
$$= 4.55 \text{ ksf} / 4.66 \text{ ksf}$$

$$= 0.98$$

When evaluated with ASCE/SEI 41-17, the footing is not adequate with an overturning soil bearing acceptance ratio of 1.32. The footing is acceptable based on an ASCE/SEI 7-10 analysis with a bearing pressure acceptance ratio of 0.98.

### B.8.3.3.2 Case 2: LSP – Flexible Base Condition (Method 1)

This analysis model includes the addition of a rigid retrofit footing and Method 1 linear soil springs. A single rotational and axial spring is assigned directly below the shear wall. Method 1 rotational and axial springs are also added beneath each existing footing at each column. The deflected shape under dead load and earthquake load is shown in Figure B-44.



**Figure B-44** LSP analysis model with Method 1 soil springs.

The moment demand at the base of the footing is determined from the rotational soil spring. Resulting footing acceptance ratios are shown in below for lower bound and for upper bound stiffness. In accordance with ASCE/SEI 41-17 § 8.4.2, the lower bound stiffness is calculated as half of the expected value and the upper bound stiffness is calculated as twice the expected value.

ASCE 41-17 with footing retrofit, check overturning compression			
$P_{UD}$ =	1586 kips	(expected load w/ footing DL)	
$q$ =	2.59 ksf		
$q_c$ =	21.0 ksf, upper-bound in accordance with 8.4.2.3.2		
$A_f$ =	612 ft <sup>2</sup>		
$L_f$ =	70.5 ft		
$A_c = P_{UD}/q_c$ =	75.5 ft <sup>2</sup>		
$b$ =	8.7 ft		
$L_c = A_c/b$ =	8.7 ft		
$b/L_c$ =	1.00		
$A_c/A_f$ =	0.12		
MCE =	49,002 k-ft		
Mbase =	177,978 k-ft (ETABS)	(corresponding moment at spring)	
required m =	3.6		
allowable m =	6.0 Section 8.4.2.3.2.1		
Acceptance ratio =	0.61		

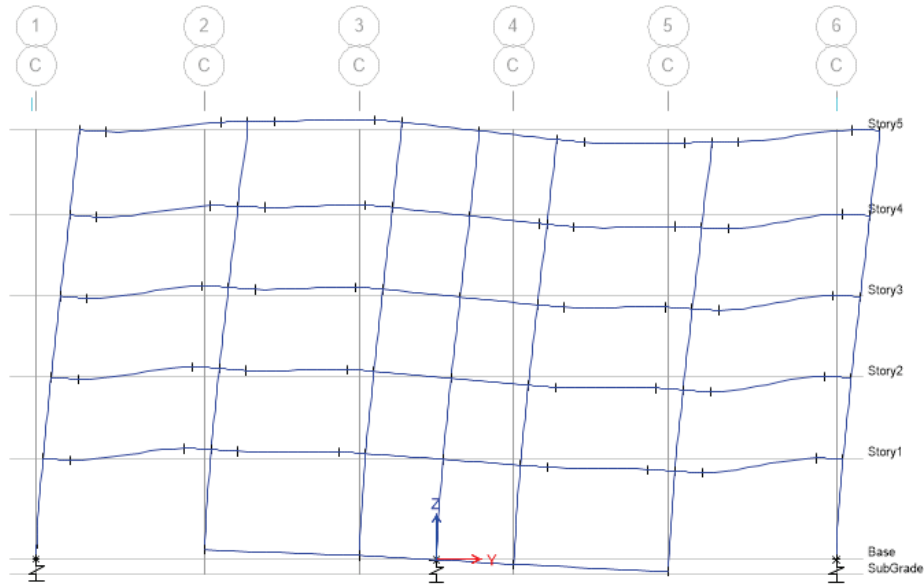
**Figure B-45 LSP analysis overturning check for Method 1 flexible base (lower bound).**

ASCE 41-17 with footing retrofit, check overturning compression			
$P_{UD}$ =	1611 kips	(expected load w/ footing DL)	
$q$ =	2.63 ksf		
$q_c$ =	21.0 ksf, upper-bound in accordance with 8.4.2.3.2		
$A_f$ =	612 ft <sup>2</sup>		
$L_f$ =	70.5 ft		
$A_c = P_{UD}/q_c$ =	76.7 ft <sup>2</sup>		
$b$ =	8.7 ft		
$L_c = A_c/b$ =	8.8 ft		
$b/L_c$ =	0.99		
$A_c/A_f$ =	0.13		
MCE =	49,672 k-ft		
Mbase =	224,538 k-ft (ETABS)	(corresponding moment at spring)	
required m =	4.5		
allowable m =	6.0 Section 8.4.2.3.2.1		
Acceptance ratio =	0.75		

**Figure B-46 LSP analysis overturning check for Method 1 flexible base (lower bound).**

### B.8.3.3.3 Case 3: LSP – Flexible Base Condition ( $K_{50}$ Stiffness)

This case is modeled as explained in Case 2 above with a single spring at the base of the shear wall with  $K_{50}$  rotational spring stiffness. The  $300M_{c,foot}$  value is the expected rotational stiffness for a rectangular footing and  $550M_{c,foot}$  is applicable to an “I” shaped footing and is provided for comparison.



**Figure B-47 LSP analysis model with  $K_{50}$  soil springs (lower and upper bound).**

Resulting footing acceptance ratio determinations are shown below for lower bound (Figure B-48) and upper bound stiffness (Figure B-49).

ASCE 41-17 with footing retrofit, check overturning compression			
Retrofit Footing			
$P_{UD} =$	1586 kips	(expected load w/ footing DL)	
$q =$	2.59 ksf		
$q_c =$	21.0 ksf, upper-bound in accordance with 8.4.2.3.2		
$A_f =$	612 ft <sup>2</sup>		
$L_f =$	70.5 ft		
$A_c = P_{UD}/q_c =$	75.5 ft <sup>2</sup>		
$b =$	8.7 ft		
$L_c = A_c/b =$	8.7 ft		
$b/L_c =$	1.00		
$A_c/A_f =$	0.12		
$M_{CE} =$	49,003 k-ft		
$M_{base} =$	111,809 k-ft (ETABS)	(corresponding moment at spring)	
required $m =$	2.3		
allowable $m =$	6.0	Section 8.4.2.3.2.1	
Acceptance ratio =	0.38		

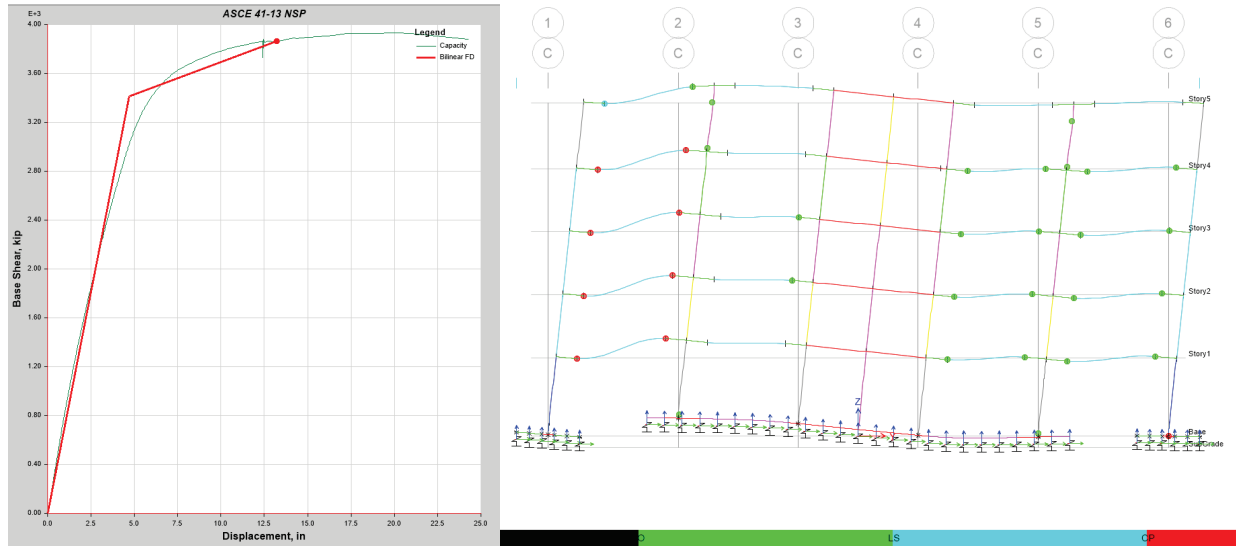
**Figure B-48 LSP analysis results with  $K_{50}$  flexible base (lower bound  $300M_{c,foot}$ ).**

<b>ASCE 41-17 with footing retrofit, check overturning compression</b>			
Retrofit Footing			
$P_{UD} =$	1586 kips	(expected load w/ footing DL)	
$q =$	2.59 ksf		
$q_c =$	21.0 ksf, upper-bound in accordance with 8.4.2.3.2		
$A_f =$	612 ft <sup>2</sup>		
$L_f =$	70.5 ft		
$A_c = P_{UD}/q_c =$	75.5 ft <sup>2</sup>		
$b =$	8.7 ft		
$L_c = A_c/b =$	8.7 ft		
$b/L_c =$	1.00		
$A_c/A_f =$	0.12		
$M_{CE} =$	49,002 k-ft		
$M_{base} =$	146,138 k-ft (ETABS)	(corresponding moment at spring)	
required $m =$	3.0		
allowable $m =$	6.0	Section 8.4.2.3.2.1	
Acceptance ratio =	0.50		

**Figure B-49 LSP analysis results with  $K_{50}$  flexible base (upper bound  $500M_{c,foot}$ ).**

#### **B.8.3.3.4 Case 4: NSP – Flexible Base Condition (Method 3)**

The Method 3 soil springs (Section B.8.3.2.3) are incorporated into the model and the same footing is assessed for rotation acceptance criteria. The Method 3 pushover analysis is shown in Figure B-50 and Figure B-74. Nonlinear hinge definitions for structural components are not outlined herein as our focus is on ASCE/SEI 41-17 Chapter 8.



**Figure B-50 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model.**

The pushover curve and deflected shape at the target displacement are shown in Figure B-50. The effective fundamental period of this model is 0.70 seconds (calculated from the pushover curve) and the target displacement is 12.8 inches.

The rotation at the base of the shear wall at the target displacement is compared to the acceptance criteria for footing rotation from ASCE/SEI 41-17 Table 8-4. The acceptance criteria calculations are included below.

$P_{UD} =$	1769 kips		
$q_c =$	10.5 ksf		
$A_c =$	168 ft <sup>2</sup>		
$A_f =$	613 ft <sup>2</sup>		
$A_c/A_f =$	0.27		
$L_c = A_c/b =$	19.41 ft		
$b/L_c =$	0.45		
allowable rotation =	0.0224 radians	41-17 Table 8-4	
rotation at target =	0.0176 radians		
acceptance ratio =	0.78		

**Figure B-51 ASCE/SEI 41-17 Method 3, footing acceptance criteria.**

### B.8.3.4 RESULTS

The resulting effects of the soil modeling assumptions on the superstructure were captured for each analysis case. Acceptance ratios were calculated for columns, shear walls, and slabs as shown in Table B-26 through Table B-31. The LSP acceptance ratios compare the analysis demand to the deformation-controlled capacity in accordance with ASCE/SEI 41-17 Equation 7-36. None of the superstructure elements shown in the tables below have been evaluated as force-controlled. The

acceptance ratios for the NSP analyses compare the hinge rotation to the acceptance criteria for Collapse Prevention as specified in ASCE/SEI 41-17 Chapter 10. If there is no inelastic rotation in the hinge at the target displacement, the acceptance criteria is listed as 0.00. Maximum values for each action and analysis have been highlighted.

Similar to the soil foundation acceptance ratios, the superstructure results indicate a nominal difference in forces in the superstructure between lower and upper-bound stiffness for each flexible foundation analysis. The  $K_{50}$  analysis procedures have higher acceptance ratios than the Method 1 analyses, because of the increased flexibility in the soil springs.

**Table B-26 Existing Interior Column - Moment Acceptance Ratios (CP Limit State)**

Existing Interior Columns – Moment Acceptance Ratios by Story						
Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP - Fixed Base	0.77	0.62	0.26	0.41	0.60	OK
LSP - Method 1 Lower Bound (Rigid Footing)	1.22	0.86	0.39	0.60	0.82	NG
LSP - Method 1 Upper Bound (Rigid Footing)	1.06	0.76	0.33	0.51	0.75	NG
LSP - $K_{50}$ 300M <sub>c,foot</sub> (Rigid Footing)	1.57	0.76	0.50	0.51	0.77	NG
LSP - $K_{50}$ 550M <sub>c,foot</sub> (Rigid Footing)	1.52	0.73	0.45	0.45	0.70	NG
NSP - Method 3	0.54	0.00	0.15	0.35	0.66	OK

Note: a DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement

**Table B-27 Existing Interior Column - Shear Acceptance Ratios (CP Limit State)  
(For Nonlinear Cases Acceptance Ratio is the Same as the Moment Acceptance Ratio)**

Existing Interior Columns – Shear Acceptance Ratios by Story						
Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP - Fixed Base	0.34	0.45	0.20	0.34	0.49	OK
LSP - Method 1 Lower Bound (Rigid Footing)	0.64	0.72	0.31	0.51	0.67	OK
LSP - Method 1 Upper Bound (Rigid Footing)	0.51	0.59	0.26	0.43	0.62	OK
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	0.89	0.80	0.41	0.43	0.63	OK
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing)	0.83	0.68	0.36	0.38	0.57	OK
NSP - Method 3	See moment acceptance ratios for nonlinear cases above					

**Table B-28 Retrofit Shear Wall - Shear Acceptance Ratios (CP Limit State)**

Retrofit Shear Walls – Shear Acceptance Ratios by Story						
Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP - Fixed Base	0.85	0.93	0.75	0.05	0.29	OK
LSP - Method 1 Lower Bound (Rigid Footing)	0.53	0.82	0.63	0.05	0.20	OK
LSP - Method 1 Upper Bound (Rigid Footing)	0.74	0.85	0.69	0.05	0.25	OK
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	0.14	0.80	0.57	0.05	0.15	OK
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing)	0.28	0.83	0.61	0.04	0.18	OK
NSP - Method 3	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: a DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement

**Table B-29 Retrofit Shear Wall - Moment Acceptance Ratios (CP Limit State)**

Retrofit Shear Walls – Moment Acceptance Ratios by Story						
Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP - Fixed Base	1.00	0.83	0.55	0.78	0.30	OK
LSP - Method 1 Lower Bound (Rigid Footing)	0.79	0.76	0.46	0.59	0.20	OK
LSP - Method 1 Upper Bound (Rigid Footing)	0.96	0.82	0.51	0.69	0.25	OK
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	0.52	0.71	0.40	0.48	0.14	OK
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing)	0.64	0.76	0.44	0.56	0.18	OK
NSP - Method 3	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: a DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement

**Table B-30 Existing Slab – Flexure Acceptance Ratios (CP Limit State)**

Existing Slab – Flexure Acceptance Ratios by Story						
Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP - Fixed Base	0.42	0.59	0.68	0.77	0.49	OK
LSP - Method 1 Lower Bound (Rigid Footing)	1.28	1.38	1.39	1.32	0.75	NG
LSP - Method 1 Upper Bound (Rigid Footing)	0.80	0.92	0.95	0.94	0.51	OK
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	1.83	1.89	1.89	1.77	1.00	NG
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing)	1.46	1.53	1.54	1.44	0.83	NG
NSP - Method 3	1.33	1.29	1.29	1.17	0.83	NG

**Table B-31 Story Drift – Drift Ratio per Story**

Story Drift – Ratios by Story					
Analysis Model	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Roof
LSP - Fixed Base	0.003	0.006	0.007	0.008	0.008
LSP - Method 1 Lower Bound (Rigid Footing)	0.008	0.012	0.013	0.013	0.013
LSP - Method 1 Upper Bound (Rigid Footing)	0.005	0.009	0.010	0.011	0.011
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	0.011	0.015	0.016	0.016	0.016
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing)	0.009	0.013	0.014	0.014	0.014
NSP - Method 3	0.012	0.013	0.013	0.013	0.013

Table B-32 and Table B-33 summarize the previous analyses and the foundation acceptance ratios. The results indicate that the ASCE/SEI 41-17 fixed base analysis provides reasonable correlation to the ASCE/SEI 7-10 foundation design. The flexible-base analysis procedures have lower acceptance ratios which is consistent with the reduced force attracted to the shear wall because of flexibility in the supporting foundation as well as higher m-factors permitted for the flexible-base analysis. The difference between acceptance ratios for lower and upper-bound analyses is relatively negligible for this case study.

**Table B-32 Summary of Analysis Results for the Retrofit Structure Foundation**

Assessment of Soil Bearing due to Overturning Action						
Model	Initial Fundamental Period (seconds)	Effective Fundamental Period (seconds)	Target Displacement (inches)	Base Shear (kips) <sup>(1)</sup>	Max. Vertical Uplift at Shear wall <sup>(2)</sup> (in)	Max. Vertical Uplift <sup>(3)</sup> (in)
LSP - ASCE/SEI 7-10	0.43	-	-	0.17W	-	-
LSP - Fixed Base	0.43	-	-	1.3W	-	-
LSP Method 1 Lower Bound (Rigid Footing)	0.58	-	-	1.3W	0.88	2.26
LSP Method 1 Upper Bound (Rigid Footing)	0.50	-	-	1.3W	0.49	1.12
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	0.65	-	-	1.3W	1.29	3.46
LSP $K_{50} 550M_{c,foot}$ (Rigid Footing)	0.60	-	-	1.3W	1.01	2.64
NSP Method 3	0.70	0.76	12.8	0.54W	4.30	6.30

Note: Analysis includes retrofit footing sized previously in hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6-feet wide by 4-feet deep with (30) #11 bars top and bottom and (4) #5 stirrups at 6"oc.

(1) W is the effective seismic weight of the superstructure equal to 7,200 kips.

(2) Maximum vertical uplift taken at face of shear wall

(3) Maximum vertical uplift at any location along the retrofit footing

**Table B-33 Summary of Analysis Results for the Retrofit Structure Foundation**

Assessment of Soil Bearing due to Overturning Action							
Model	Allowable Rotation <sup>(2)</sup> (radians)	Rotation at Target - at shear wall <sup>(3)</sup> (radians)	Rotation at Target - at ends of ftg <sup>(4)</sup> (radians)	Rotation at Target - at inflection pts <sup>(5)</sup> (radians)	Rotational stiffness (kip-ft/rad)	<i>m</i> -factor	Footing Acceptance Ratio <sup>(2,8)</sup>
LSP - ASCE/SEI 7-10	-	-	-	-	-	-	0.98
LSP - Fixed Base	-	-	-	-	-	4.0	1.32
LSP Method 1 Lower Bound (Rigid Footing)	-	-	-	-	32,764,546	6.0	0.61
LSP Method 1 Upper Bound (Rigid Footing)	-	-	-	-	131,058,185	6.0	0.75
LSP - K <sub>50</sub> 300M <sub>c,foot</sub> (Rigid Footing)	-	-	-	-	11,815,563	6.0	0.38
LSP K <sub>50</sub> 550M <sub>c,foot</sub> (Rigid Footing)	-	-	-	-	21,661,866	6.0	0.50
NSP Method 3	0.022	0.018	0.004	0.008	2,411,500	-	0.78

Note: Analysis includes retrofit footing sized previously in hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6-feet wide by 4-feet deep with (30) #11 bars top and bottom and (4) #5 stirrups at 6 inches on center.

- (1) *W* is the effective seismic weight of the superstructure equal to 7,200 kips.
- (2) Allowable rotation and footing acceptance ratios calculated using entire footing length and effective footing width, rotation demand taken at end of shear wall. *P<sub>UD</sub>* used to calculate footing acceptance ration is expected load including footing dead load.
- (3) Rotation at target displacement taken at ends of shear wall.
- (4) Rotation at target displacement taken at ends of retrofit footing.
- (5) Rotation at target displacement taken at inflection points of footing by using entire length of retrofit footing but measuring the rotation demand between inflection points of the deformed footing (see Section B.10.5.8).
- (6) Maximum vertical uplift taken at face of shear wall
- (7) Maximum vertical uplift at any location along the retrofit footing
- (8) Expected moment capacity used to determine Footing Acceptance Ratios based on upper-bound bearing capacities per 8.4.2.3.2 for LSP models.

### B.8.3.5 CONCLUSIONS

The results indicate that the ASCE/SEI 41-17 fixed base analysis provides reasonable correlation to the ASCE/SEI 7-10 foundation design, though the ASCE/SEI 41/-17 fixed base analysis indicated that the footing is not adequate due to an overturning soil bearing acceptance ratio of 1.32 while that of the ASCE/SEI 7-10 analysis was 0.98.

The flexible-base analysis procedures have lower acceptance ratios than the fixed-base and ASCE/SEI 7-10 which is consistent with the reduced force attracted to the shear wall because of flexibility in the supporting foundation as well as higher m-factors permitted for the flexible-base analysis.

Similar to the soil foundation acceptance ratios, the superstructure results indicate a nominal difference in forces in the superstructure between lower and upper-bound stiffness for each flexible foundation analysis. The  $K_{50}$  analysis procedures have higher acceptance ratios than the Method 1 analyses, because of the increased flexibility in the soil springs. In general, the  $K_{50}$  stiffness models (with gapping) correlate better with nonlinear analysis methods (Method 3). In addition, the upper-bound stiffness does not yield sufficiently different results (superstructure component actions and foundation overturning acceptance ratios) to warrant the additional effort to include in the analysis procedures,

#### **B.8.4 Recommended Changes**

The recommended change clarifies how to calculate the soil stiffness for shallow foundations by providing a specific equation that is a function of the shear modulus and footing length and width (specified for mat foundation also) and Poisson's ratio. It also allows for this modulus to be provided by the geotechnical engineer.

### **B.9. Acceptance Criteria for Isolated Spread Footings with Foundation Interface Modeled as a Flexible Base Footing Not Rigid Relative to Soil (Proposed ASCE/SEI 41-23 Section 8.4.5.2.1.3)**

#### **B.9.1 Motivation**

ASCE/SEI 41-17 has limited guidance on modeling and assessment of foundations that are flexible relative to the soil. Method 3 only provides provisions for nonlinear analysis and acceptance criteria. The goal is to provide better guidance and provisions for linear flexible foundation modeling and acceptance criteria for the user.

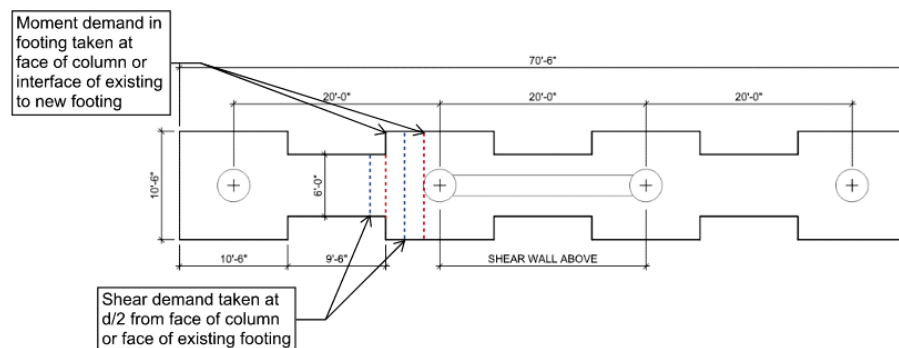
#### **B.9.2 Technical Studies**

In addition to evaluating the effect of soil stiffness bounding on foundation design and superstructure performance, the structural foundation components are also evaluated for each model and compared to each other and the ASCE/SEI 7-10 calculations. The goal is to assess different options for how to evaluate foundations using linear procedures that includes flexibility of the foundation itself. Although ASCE/SEI 41-17 specifies that structural foundations be evaluated as

force-controlled components, foundation strength is also evaluated as deformation-controlled in this case study for comparison as yielding of the foundation contributes to its flexibility.

### B.9.2.1 STRUCTURAL FOUNDATION DESIGN

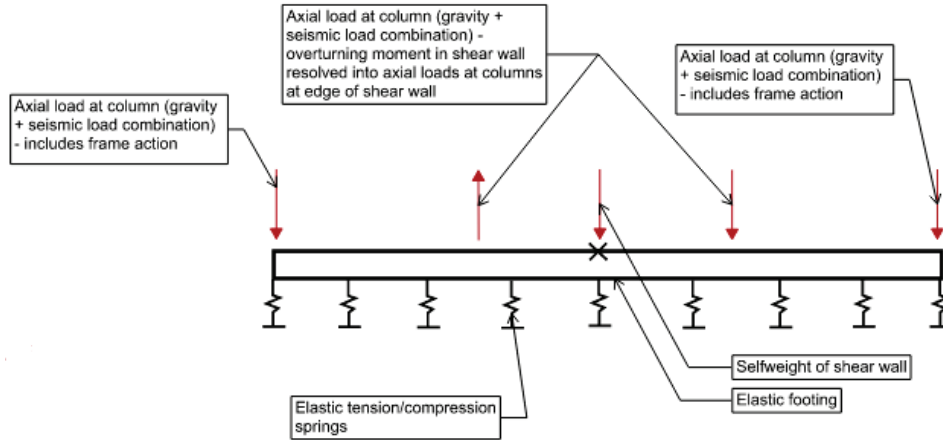
The structural foundation designed using ASCE/SEI 7-10 as discussed in previous sections is evaluated for the ASCE/SEI 41-17 analysis models. The ASCE/SEI 7-10 foundation design was based on an elastic beam on elastic soil analysis. For the ASCE/SEI 41-17 linear procedures (fixed base, Method 1 and  $K_{50}$ ), the loads on the retrofit footing are obtained from the analysis models and applied to an elastic beam supported by soil springs to determine the internal forces in the footing. We note that a thorough analysis of the footing would include evaluation at multiple locations (existing footing at face of column, new footing at face of existing footing, etc.) as shown in Figure B-52. For simplicity, the results shown herein are determined at the new footing section at the face of the existing footing.



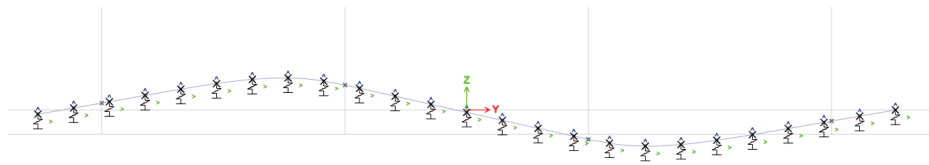
**Figure B-52 Critical sections for foundation design.**

Two approaches to modeling of the structural foundation and supporting soil were investigated for the linear fixed-base procedure:

1. The structural foundation is modeled as an elastic concrete beam on elastic (tension/compression) foundation springs with vertical stiffness calculated from ASCE/SEI 41-17 Fig. 8-2. Unreduced, pseudo-elastic forces are applied to the foundation as determined from the analysis model and as shown in Figure B-53. See Figure B-44 for deformed shape of footing using this approach.

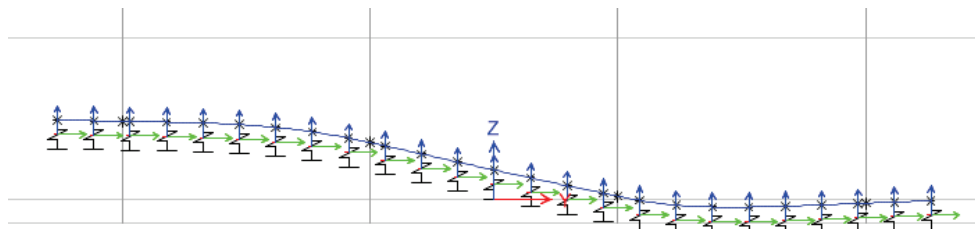


**Figure B-53 Load application on elastic foundation model.**



**Figure B-54 Elastic tension and compression springs.**

- The structural foundation is modeled as an elastic concrete beam on nonlinear (compression-only) foundation springs with vertical compression stiffness calculated from ASCE/SEI 41-17 Fig. 8-2. This approach is equivalent to the conventional foundation design approach using SAFE with compression-only springs. In order to create a stable analysis model, the pseudo-elastic forces are reduced by an  $m$ -factor or DCR from the superstructure above. These reduced loads are applied to the foundation similar to the approach shown in Figure B-53. A representative deformed shape of the foundation using this approach is shown in Figure B-55.



**Figure B-55 Elastic compression-only springs.**

The compression-only spring analysis was considered for three cases:

1. Reducing pseudo-elastic forces by  $m = 4$ , which is equivalent to the m-factor for soil bearing pressure compression due to overturning. The use of this m-factor is somewhat arbitrary, but we understand that this approach is often used for similar SAFE-type analyses. For this example, the reduction by  $m$  did not provide a stable analysis model and at the end of the uplift side of the footing, the displacement was significant (~30 inches). This indicates that the footing is undersized, or the m-factors are too small. This confirms the fixed base procedure may be overly conservative relative to ASCE/SEI 7-10 and the results of the NSP.
2. Reducing pseudo-elastic forces by DCR calculated from superstructure delivering load to the foundation. The DCR was calculated for the shear wall and was determined to be approximately 3. This DCR is less than  $m = 4$  and therefore also provides an unstable analysis model.
3. Reducing pseudo-elastic forces by  $m = 7.76$ , which is equal to the ratio of ASCE/SEI 41-17 (1.32 W per Table B-41) to ASCE/SEI 7-10 base shear (0.17W per Table B-41) for comparison between ASCE 7 and ASCE 41. Note that this is not a realistic m or DCR factor and this analysis is only performed to investigate the feasibility of the compression-only analysis approach.

For the fixed-base analysis, the comparisons of the following approaches are evaluated for the elastic beam supported by springs with results shown in Table B-34 and Table B-35 for both lower and upper-bound soil stiffness:

1. Elastic tension/compression springs with concrete foundation analyzed for deformation-controlled actions (expected strength with m-factors from ASCE/SEI 41-17 Chapter 10).
2. Nonlinear, compression-only springs with concrete foundation analyzed as force-controlled (lower bound strength with no additional  $m$ -factors).

**Table B-34 Comparison of Fixed-Base Approaches**

Utilizing m-factors for LSP Fixed Base Strength Design					
Model	Lower-Bound Stiffness				
	Deflection (in)	Moment (k-ft)	Moment Acceptance Ratio	Shear (kip)	Shear Acceptance Ratio
Tens/Comp Spring w/ component m-factor	6.45	62,128	1.89	5,209	1.29
Compression-only Demand/( $m = 4$ )	Unstable				
Compression-only Demand/( $m = 7.76$ )	3.84	12,450	1.52	1,235	0.89

**Table B-35 Comparison of Fixed-Base Approaches**

Utilizing $m$ -factors for LSP Fixed Base Strength Design					
Model	Upper-Bound Stiffness				
	Deflection (in)	Moment (k-ft)	Moment Acceptance Ratio	Shear (kip)	Shear Acceptance Ratio
Tens/Comp Spring w/ component $m$ -factor	2.49	47,266	1.44	7,000	1.26
Compression-only Demand/( $m = 4$ )	Unstable				
Compression-only Demand/( $m = 7.76$ )	3.80	12,369	1.51	1,235	0.89

The results of this comparison show that the compression-only approach yields similar results to the tension/compression analysis when a large  $m$  (7.76) or  $DCR$  factor is used to reduce pseudo-elastic forces. Reduction of pseudo-elastic forces by an  $m$ -factor equal to 4 (which is also equivalent to the  $m$ -factor for overturning soil bearing and the  $m$ -factor for the structural concrete beam in Chapter 10) does not provide realistic results and would result in a larger footing.

The results of this case study indicate that the compression-only spring approach could be used provided that the applied  $m$ -factor or  $DCR$  is such that the reduced, applied loads do not make the model unstable. However, for this case study, with a shear wall designed using the fixed-base analysis procedure, this approach is not feasible. Given that we understand that this approach has been used successfully on building design by practicing engineers, we recommend additional case studies to investigate the use of this approach.

For the purposes of this case study and comparison between linear analysis procedures, the structural foundation will be evaluated using the elastic beam on elastic (tensions/compression) springs for the remainder of this section. The analyses utilize lower-bound spring stiffness. A comparison between lower and upper-bound stiffness is provided at the end of this section. For the nonlinear static model, the forces in the foundation are taken directly from the model since the footing is explicitly modeled.

For each analysis model, the structural foundation is evaluated as force-controlled and deformation-controlled for comparison as discussed below.

### B.9.2.2 FORCE-CONTROLLED – EVALUATION

Structural foundation evaluation in accordance with ASCE/SEI 41-17 provisions requires that concrete structural components be evaluated assuming force-controlled actions. The loads on the foundation are applied as discussed above and the internal forces in the footing are compared to the

calculated strength using lower bound strength properties with no  $m$ -factors applied. Two approaches to the force-controlled evaluation were performed.

1. Unreduced, pseudo-elastic forces are applied to the foundation based on the superstructure analysis model.
2. Alternatively, in accordance with ASCE/SEI 41-17 § 10.12.3, the capacity of the foundation components need not exceed 1.25 times the capacity of the supported vertical structural component or element (column or wall). In this case, the overturning forces applied to the footing are based on 1.25 times the flexural strength of the shear wall.

The acceptance ratios for the force-controlled structural foundation evaluations are summarized in Table B-36 and Table B-37. Based on these results, it is clear for all analysis procedures that the force-controlled analysis yields significantly higher acceptance ratios than the ASCE/SEI 7-10 foundation design and therefore significantly more conservative structural foundation designs.

### **B.9.2.3 DEFORMATION-CONTROLLED – EVALUATION**

A similar evaluation of the structural foundation using deformation-controlled methodology was also performed with loading applied as described above. For linear procedures, the internal forces in the footings were compared to flexure and shear capacities calculated using expected strength properties and  $m$ -factors from the concrete material chapter (Chapter 10), specifically for concrete beams. For the NSP model, rotation demand in the structural foundation was compared to allowable rotation in Chapter 10.

The results of the deformation-controlled analysis are also shown in Table B-36 and Table B-37. Typically, this approach yields results more similar to the ASCE/SEI 7-10 design results.

### **B.9.2.4 SUMMARY OF FOUNDATION EVALUATION RESULTS**

Included below are the structural foundation results for multiple cases showing their results for both force-controlled and deformation-controlled cases. For simplicity, the results shown herein are determined at the new footing section at the face of the existing footing.

**Table B-36 Summary of Retrofit Structure Foundation Design – Unreduced Loading<sup>(1)</sup>**

Retrofit Footing Design Comparison - Unreduced, Pseudo-Elastic Forces <sup>(2)</sup>					
Model	Design Moment in Footing (k-ft)	Moment Acceptance Ratio	Design Shear in Footing (k)	Shear Acceptance Ratio	Action Classification
ASCE/SEI 7-10 (for comparison)	7,734	0.99	797	0.73	Force-controlled
LSP - Fixed Base	62,128	7.56	7,163	5.16	Force-controlled
		1.89		1.29	Deformation-Controlled <sup>(4)</sup>
LSP - Method 1 Lower Bound (Rigid Footing) <sup>(3)</sup>	45,240	5.51	3,546	2.55	Force-controlled
		1.07		0.51	Deformation-Controlled <sup>(4)</sup>
LSP - Method 1 Upper Bound (Rigid Footing) <sup>(3)</sup>	56,462	6.87	4,386	3.16	Force-controlled
		1.33		0.51	Deformation-Controlled <sup>(4)</sup>
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing) <sup>(3)</sup>	28,195	3.25	2,268	1.56	Force-controlled
		0.67		0.33	Deformation-Controlled <sup>(4)</sup>
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing) <sup>(3)</sup>	36,780	4.23	2,932	2.02	Force-controlled
		0.87		0.43	Deformation-Controlled <sup>(4)</sup>
NSP - Method 3	16,180	1.86	1,547	1.06	Force-controlled
	N/A	0.00	N/A	0.00	Deformation-Controlled <sup>(5)</sup>

<sup>(1)</sup> Analysis includes retrofit footing sized previously in Hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6 feet wide by 4 feet deep with (30) #11 top and bottom and (4) #5 stirrups at 6"oc.

<sup>(2)</sup> Design moment and shear for LSP models are amplified elastic forces.

<sup>(3)</sup> Footing designed based on elastic beam methodology with lower-bound soil springs.

<sup>(4)</sup> Footing strength is equal to the expected strength multiplied by a m-factor of 4 (determined from Chapter 10).

<sup>(5)</sup> There was no plastic rotation in the footing, therefore acceptance ratio is 0

**Table B-37 Summary of Retrofit Structure Foundation Design – Capped Loading<sup>1</sup>**

Retrofit Footing Design Comparison - Forces Limited by 1.25 x Expected Force to Footing <sup>2</sup>					
Model	Design Moment in Footing (k-ft)	Moment Acceptance Ratio	Design Shear in Footing (k)	Shear Acceptance Ratio	Action Classification
ASCE/SEI 7-10 (for comparison)	7,734	0.99	797	0.73	Force-controlled
LSP - Fixed Base	37,209	4.53	4,286	3.09	Force-controlled
		0.88		0.62	Deformation-Controlled <sup>4</sup>
LSP - Method 1 Lower Bound (Rigid Footing) <sup>3</sup>	12,275	1.49	1,079	0.78	Force-controlled
LSP - Method 1 Upper Bound (Rigid Footing) <sup>3</sup>					
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing) <sup>3</sup>		0.29		0.16	Deformation-Controlled <sup>4</sup>
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing) <sup>3</sup>					
NSP - Method 3	16,180	1.86	1,547	1.06	Force-controlled
	N/A	0.00	N/A	0.00	Deformation-controlled <sup>5</sup>

(1) Analysis includes retrofit footing sized previously in Hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6 feet wide by 4 feet deep with (30) #11 top and bottom and (4) #5 stirrups at 6"oc.

(2) Design forces for footing capped at 1.25x the maximum expected strength (moment and shear) of the retrofit shear wall provided above the footing per ASCE/Sei 41-17 Section 10.12.3.

(3) Footing designed based on elastic beam methodology with lower-bound soil springs.

(4) Footing strength is equal to the expected strength multiplied by a m-factor of 4 (determined from Chapter 10)

(5) There was no plastic rotation in the footing, therefore acceptance ratio is 0.

### B.9.2.5 STIFFNESS BOUNDING COMPARISON OF LINEAR PROCEDURES

The effects of lower and upper-bound linear stiffness assumptions on the supporting springs were also investigated for the linear procedures. Results are shown in Table B-38. All analyses are based on evaluation of force-controlled actions.

**Table B-38 Summary of Retrofit Structure Foundation Design – Unreduced Loading**

Stiffness Bounding for Footing Strength Design								
Model	Lower-Bound Stiffness				Upper-Bound Stiffness			
	Moment (k-ft)	Moment Acceptance Ratio	Shear (kip)	Shear Acceptance Ratio	Moment (k-ft)	Moment Acceptance Ratio	Shear (kip)	Shear Acceptance Ratio
LSP - Fixed Base	62,128	7.56	7,163	5.16	47,266	5.75	7,000	5.04
LSP - Method 1 Lower Bound (Rigid Footing)	45,240	5.51	3,546	2.55	-	-	-	-
LSP - Method 1 Upper Bound (Rigid Footing)	-	-	-	-	56,462	6.87	4,386	3.16
LSP - $K_{50} 300M_{c,foot}$ (Rigid Footing)	28,195	3.25	2,268	1.56	-	-	-	-
LSP - $K_{50} 550M_{c,foot}$ (Rigid Footing)	-	-	-	-	29,124	3.54	3,074	2.21

Note: Moment and Shear design forces represent unreduced pseudo-elastic forces. Moment and shear acceptance ratios reflect force-controlled methodology.

### B.9.2.6 CONCLUSIONS

Design of the footing using ASCE/SEI 41-17 force-controlled actions is significantly conservative compared to ASCE/SEI 7-10 design. Footing design based on deformation-controlled actions using acceptance criteria from the appropriate material chapters should be permitted. The use of an elastic beam modeling approach utilizing expected stiffness, elastic springs provides a reasonable approach to foundation design and evaluation. Note that the use of upper- or lower-bound stiffness may yield unconservative results depending on the foundation condition; therefore expected stiffness is recommended. As an alternative, reduction of pseudo-elastic forces by an  $m$ -factor or  $DCR$  may provide reasonable results depending on stability of the compression-only soil spring analysis model.

### B.9.3 Recommended Changes

In the ASCE/SEI 41-23 code, footing design based on deformation-controlled actions will be included explicitly and foundation compression action  $m$ -factors will be included in a new table.

## **B.10 Nonlinear Static Procedure**

### **B.10.1 Motivation**

Many of the hypotheses compare linear results from the linear static procedure (LSP) to nonlinear results from the nonlinear static procedure (NSP). In these comparisons, the nonlinear results are utilized as the benchmark to calibrate linear procedures to. However, nonlinear procedures are not presumed to precisely estimate the real building performance, nor does this study seek to prove accuracy of nonlinear modeling compared to true building performance. The nonlinear methodology of ASCE 41 has been calibrated to research-based testing data; therefore, for this study, it is assumed the nonlinear analyses are more accurate in determining structural response and provide sufficient data to examine the hypotheses related to linear analyses. In the process of performing the NSP analyses for calibration with linear procedures, some interesting topics and interpretations related to the NSP were discovered and are discussed herein for reference.

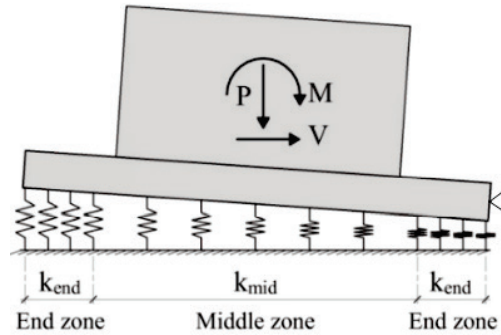
### **B.10.3 Technical Studies**

#### **B.10.3.1 METHOD 1 - STIFFNESS DERIVATION**

The retrofit footing is treated as a rigid body for the Method 1 spring stiffness derivations. Method 1 uses uncoupled moment and axial springs to model rigid foundations such that moment and shear behaviors are independent of axial load. Shear springs may also be used, in this case, and for all methods in this investigation, lateral moment is restrained within the analysis model. See Section B.8.3.1.1 for more information on the derivation of these springs for this case study.

#### **B.10.3.2 METHOD 2 - STIFFNESS DERIVATION (ASCE/SEI 41-17 FIGURE 8-5)**

Method 2 provides an alternative approach for rigid foundations that uses a bed of nonlinear springs that accounts for coupling between vertical loads and moments. The moment-rotation and vertical load deformation characteristics are modeled as a beam on a nonlinear Winkler foundation with stiffer vertical springs at the end regions to allow for tuning of the springs to approximately match the elastic vertical and rotational stiffness from Method 1. A graphical representation of the Method 2 springs is shown in Figure B-56.



**Figure B-56 Method 2 foundation springs (FEMA P-2006).**

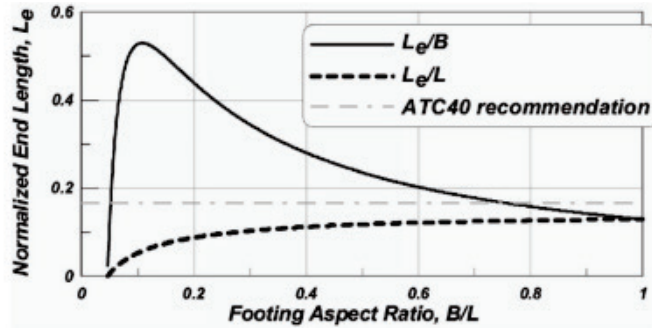
ASCE/SEI 41-17 § 8.4.2.4.1 requires that the Method 2 springs be tuned to approximately match the stiffness from Method 1 but does not provide a variable for tuning in the equations provided in Figure 8-4. There is a reference to Gajan et al. (2010) in ASCE/SEI 41-17 § C8.4.2.4.1 which provides a methodology for tuning. For the derivation shown below, no tuning is performed on the calculated stiffness. The next section utilizes the Gajan et al. approach for tuning. The results of the analysis with both approaches are compared. The un-tuned Method 2 springs are derived in accordance with ASCE/SEI 41-17 Figure 8-5.

$k_{end}$ =	9545	kip/ft/unit length		factor =	6.83
$k_{middle}$ =	1020	kip/ft/unit length		factor =	0.73
$l_{end}$ =	1.45	ft (B/6)			
$l_i$ =	3.38	ft			
# springs =	20.0	middle zone			
$K_{end}$ =	13810	kip/ft	1150.8	kip/in	<- go into etabs
$K_{middle}$ =	3449	kip/ft	287.4	kip/in	<- go into etabs

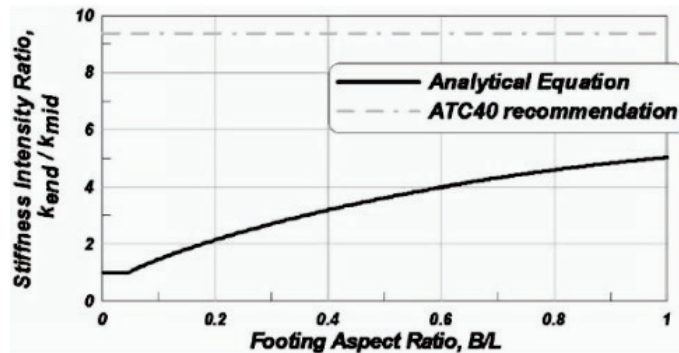
**Figure B-57 Derivation of Method 2 soil springs – ASCE/SEI 41-17 Figure 8-5.**

### B.10.3.3 METHOD 2 SOIL STIFFNESS DERIVATION – TUNED PER GAJAN ET AL. (2010)

Gajan et al. provides a methodology for tuning the middle and end Winkler springs to match the Method 1 stiffness values. This is done by first determining the length tributary to the end spring based on aspect ratio, then determining the factors for each of the springs based on the footing aspect ratio. The relationship of these parameters to the aspect ratio are shown in Figure B-58 and Figure B-59.



**Figure B-58** End length versus aspect ratio (Gajan et al, from Harden and Hutchinson, 2009).



**Figure B-59** Stiffness intensity ratio versus aspect ratio (Gajan et al, from Harden and Hutchinson, 2009).

The end length and the intensity ratio are determined in accordance with the figures above. The factors on the middle and end springs are then determined by comparing the total and end deflection to the results from Method 1. The Method 1 test loads and deflections are shown below.

Method 1	
test loads for tuning	
M =	500 kip-ft
P =	1000 kips
Deflection at end of footing	
$x_z =$	0.1198 inch (axial spring)
$x_{yy} =$	0.0032 inch (rotational spring)
$x_{total} =$	0.1230 inch

**Figure B-60** Method 1 test loads and associated deflections.

The Method 2 soil springs, tuned to the Method 1 deflections, are shown below. Further explanation of soil spring tuning is provided in FEMA P-2006.

B/L =	0.12				
Le/B =	0.50				
$k_{end}/k_{middle}$ =	1.53				
$k_{end}$ =	2096 kip/ft/unit length			factor =	1.5
$k_{middle}$ =	1370 kip/ft/unit length			factor =	1.0
lend =	2.50 ft				
li =	3.28 ft				
# springs =	20.0 middle zone				
$K_{end}$ =	5241 kip/ft	436.7 kip/in		<-	go into etabs
$K_{middle}$ =	4485 kip/ft	373.8 kip/in		<-	go into etabs
$K_{total}$ =	100189 kip/ft				
$x_z$ =	0.1198 inch	OK			
$F_{end}$ =	1.5 kips				
$x_{yy}$ =	0.0033 inch	OK			
$x_{total}$ =	0.1231 inch				

**Figure B-61 Derivation of Method 2 soil springs – tuned per Gajan et al.**

#### B.10.3.4 METHOD 3 - STIFFNESS DERIVATION (ASCE/SEI 41-17 EQUATION 8-11)

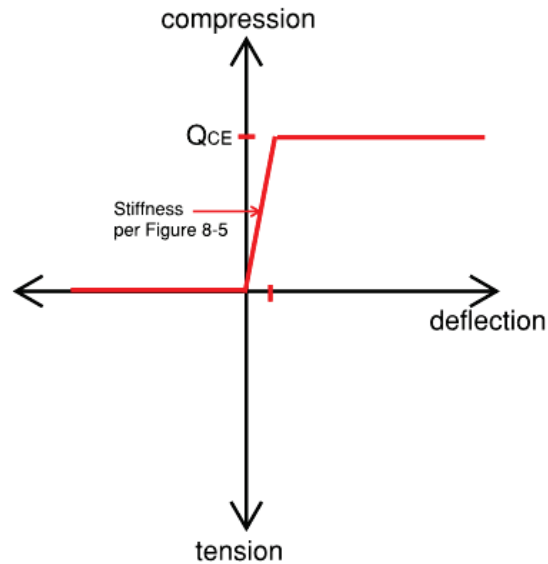
Similar to Method 2 soil springs, Method 3 soil springs utilize decoupled Winkler springs. Method 3 diverges from Method 2 because it is intended for shallow foundations not rigid relative to the soil. It also has uniform springs beneath a footing, whereas the Method 2 springs have stiffer end bearing springs. Method 3 soil stiffness is derived in Section B.8.3.1.3.

### B.10.4 Modeling Parameters and Acceptance Criteria

#### B.10.4.1 METHOD 1 – MODELING PARAMETERS AND ACCEPTANCE CRITERIA

Method 1 modeling parameters and acceptance criteria are described in Section B.8.3.2.1.

The capacity of the soil springs for Method 2 is noted in ASCE/SEI 41-17 § 8.4.2.4.2 as equal to the expected bearing capacity of the soil in compression and equal to zero in tension. A representative Method 2 soil spring is shown in Figure B-62. The acceptance criteria is based on the rotation limits of ASCE/SEI 41-17 Table 8-4. The rotation modeling parameters noted in ASCE/SEI 41-17 Table 8-4 are not utilized in Method 2, since the springs are for axial actions.



**Figure B-62 Method 2 NSP modeling parameters.**

### **B.10.4.3 METHOD 3 - MODELING PARAMETERS AND ACCEPTANCE CRITERIA**

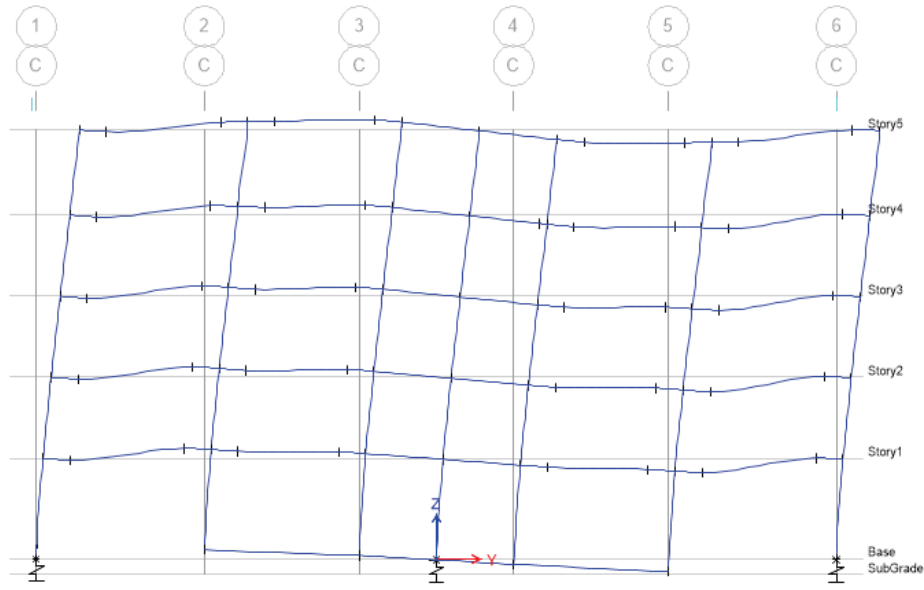
Method 3 modeling parameters and acceptance criteria are described in Section B.8.3.2.3.

Within ASCE/SEI 41-17, it is noted that the Winkler springs should be tuned to the Method 1 Springs for Method 2 (§ 8.4.2.4.1) yet the equations do not include variables that can be tuned. Specifying a tuning approach, if required for Method 2, is recommended in the future development of ASCE 41 Chapter 8.

## **B.10.5 Results**

### **B.10.5.1 HYPOTHESIS 1 LSP CASE 8: FLEXIBLE BASE CONDITION (METHOD 1), WITH FOUNDATION RETROFIT**

For comparison, Case 8 as described in Hypothesis 1 in Section B.3.1.1 is the Linear Case 5 analysis model from Hypothesis 1 utilized with the addition of a rigid retrofit footing and Method 1 linear soil springs. A single rotational and axial spring is assigned directly below the shear wall. Method 1 rotational and axial springs associated with the existing pad foundations at each column are also added beneath each existing footing. The deflected shape under dead load and earthquake load is shown in Figure B-63.

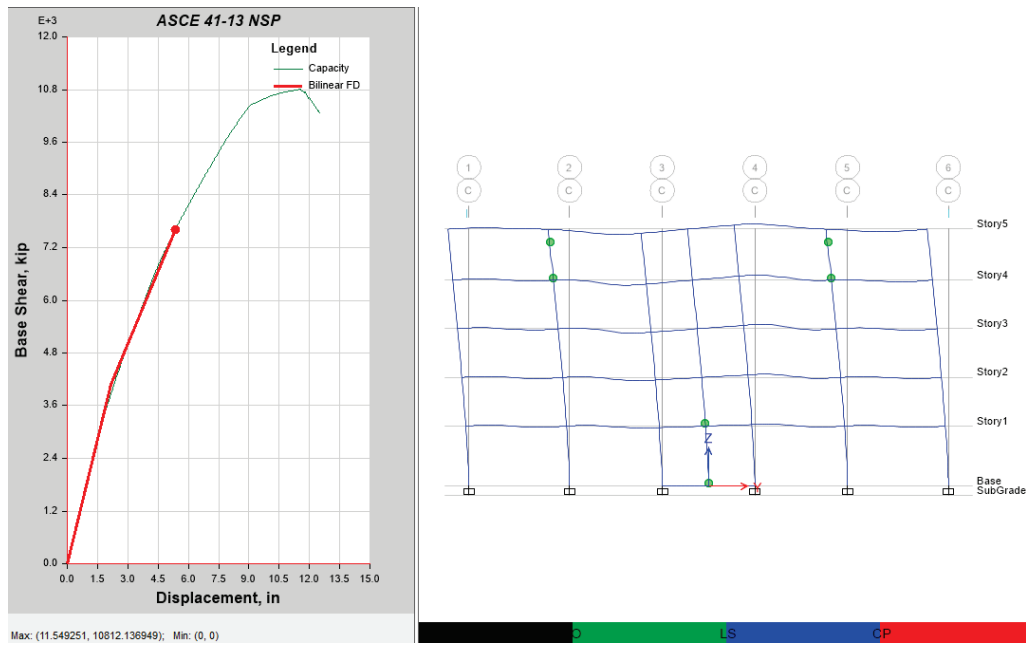


**Figure B-63 LSP Analysis model with Method 1 soil springs.**

The foundation acceptance criteria for this analysis can be seen in Table B-7. The moment demand at the base of the footing is the output from the rotational soil spring. The acceptance ratios from Method 1 flexible base and Method 1 fixed base in this table show similar results for bearing pressure, uplift, and overall overturning stability.

#### **B.10.5.2 NSP CASE 1: NSP ANALYSIS – FIXED BASE**

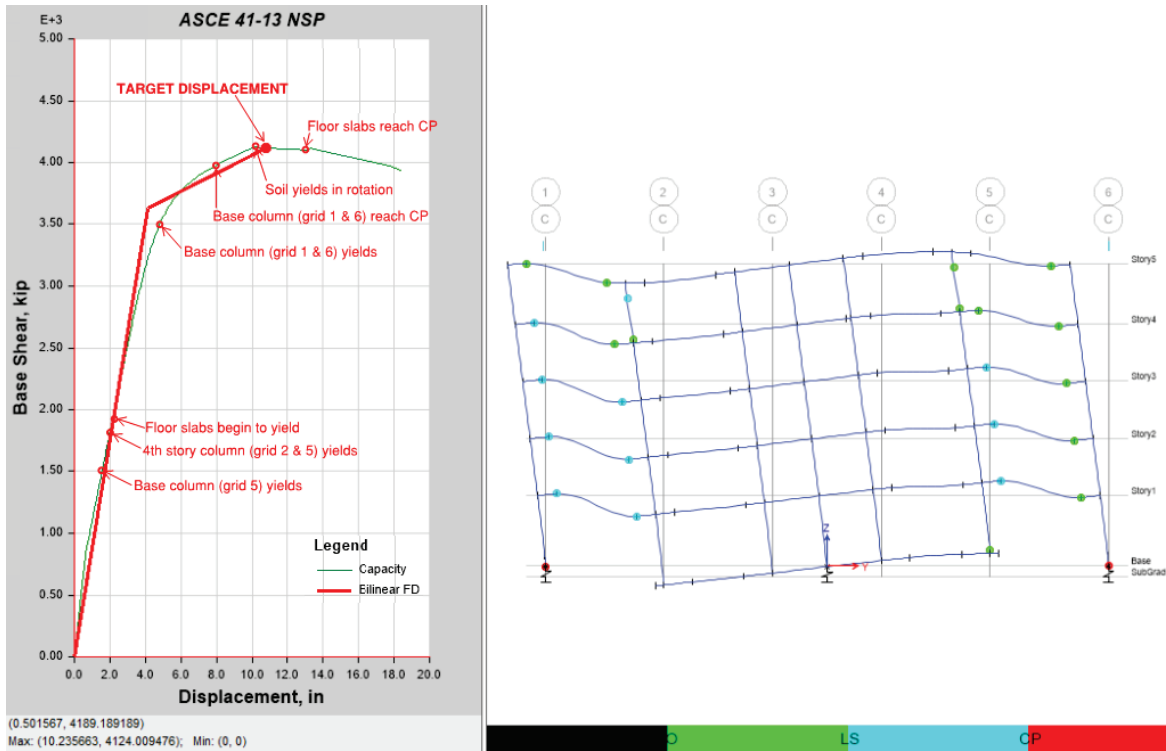
Figure B-64 shows the fixed base nonlinear pushover analysis force-displacement curve and deformed shape of the structure in elevation. Note that the shear walls are modeled as frame elements with flexural hinges top and bottom and a shear hinge at the center of each wall element. The wall frame element is located at the center of the elevation between gridlines 3 and 4. The calculated target displacement is equal to 5.3 inches. The fundamental period of the structure is 0.45 seconds, which matches the LSP analysis. There is no acceptance criteria in ASCE/SEI 41-17 for fixed base nonlinear procedures; these results are used as a comparison to the following flexible base analyses.



**Figure B-64 ASCE/SEI 41-17 Nonlinear static pushover, fixed base model.**

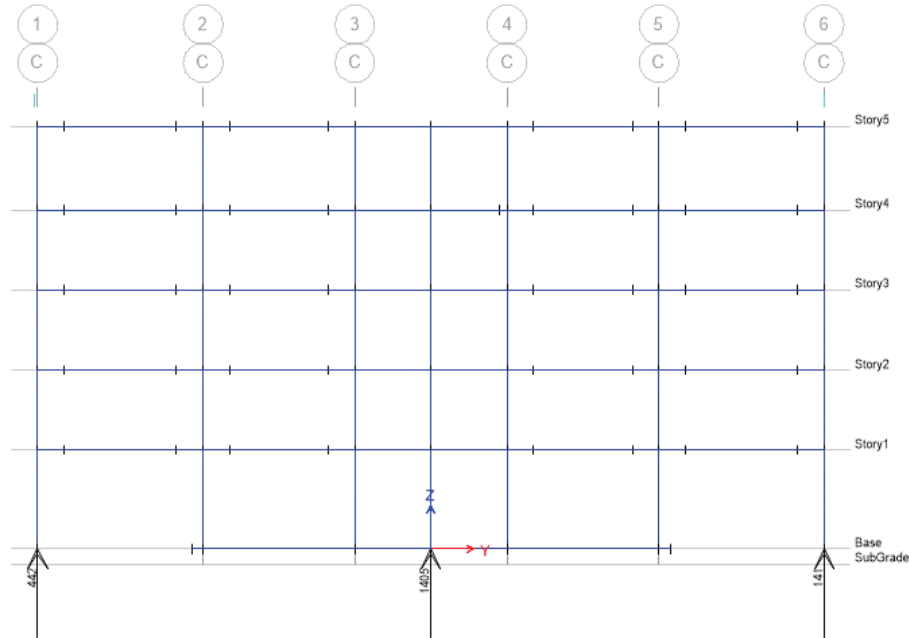
### B.10.5.3 NSP CASE 2: NSP ANALYSIS – METHOD 1

Figure B-65 shows the nonlinear pushover force-displacement curve and displacements for the nonlinear model with Method 1 foundation springs. The relative superstructure hinge acceptance ratios are shown graphically, but are investigated in a later section. The horizontal displacement at each node at the base of the structure is restrained. Rotational and compression-only springs are applied at each individual existing footing. At the retrofit foundation, a single rotational and compression-only spring is located at the center of the base of the shear wall to capture the stiffness of the entire footing. The footing designed using ASCE/SEI 7-10 procedures (6-foot-wide by 4-foot-deep with (30) #11s top and bottom) is used in the analysis model. The calculated target displacement for this model is 10.8 inches and the fundamental period is 0.54 seconds.



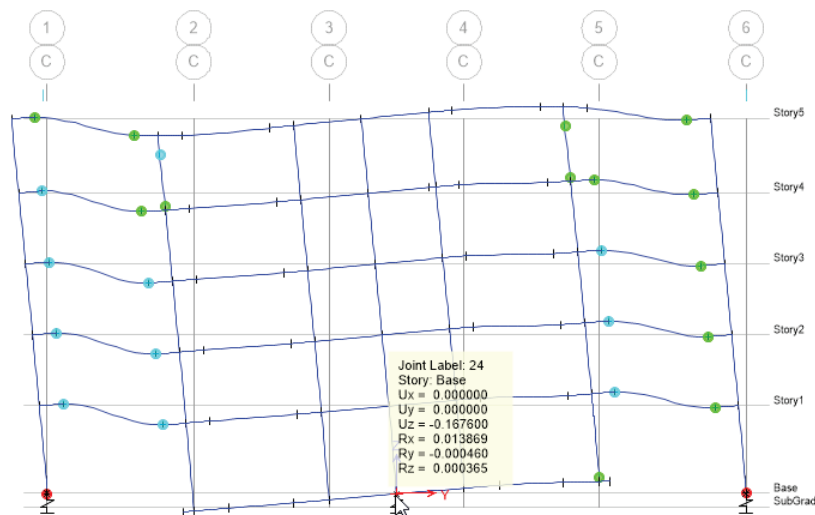
**Figure B-65 ASCE/SEI 41-17 Nonlinear static pushover, Method 1 flexible base model.**

The base reactions of the flexible base (Method 1) model are displayed in Figure B-66 at the target displacement. The rotation demand in the spring at the base of the shear wall is utilized to evaluate the acceptance criteria for the foundation rotation.



**Figure B-66 ASCE/SEI 41-17 Nonlinear static pushover, Method 1 flexible base model, base reactions at target displacement.**

The rotation of the retrofit footing at the target displacement is displayed in Figure B-67.



**Figure B-67 ASCE/SEI 41-17 Nonlinear static pushover, Method 1 flexible base model foundation rotation at target displacement (Rotation units are radians; displacement units are inches).**

The axial load shown in Figure B-66 is defined as  $P_{UD}$  in the calculations shown below. The footing rotation (Figure B-67) is then compared to the allowable rotation from ASCE/SEI 41-17 Table 8-4 to determine an acceptance ratio.

$P_{UD} =$	1405 kips		
$q =$	2.30 ksf		
$q_c =$	10.5 ksf, expected strength		
$A_c =$	134 ft <sup>2</sup>		
$A_f =$	612 ft <sup>2</sup>		
$A_c/A_f =$	0.22		
$b/L_c =$	0.12		
allowable rotation =	0.0238 radians, 41-17 Table 8-4		
rotation at target =	0.0139 radians		
acceptance ratio =	0.58		

**Figure B-68 ASCE/SEI 41-17 Nonlinear static procedure soil acceptance criteria and acceptance ratio.**

The flexural demand in the retrofit footing is reported at the target displacement to assess the ASCE/SEI 7-10 designed footing. The footing is evaluated as force-controlled with the lower-bound strength as specified in ASCE/SEI 41-17 § 10.12.3. The footing flexural action has an acceptance ratio of 1.37, so it is not acceptable and would require additional strength with this analysis approach.

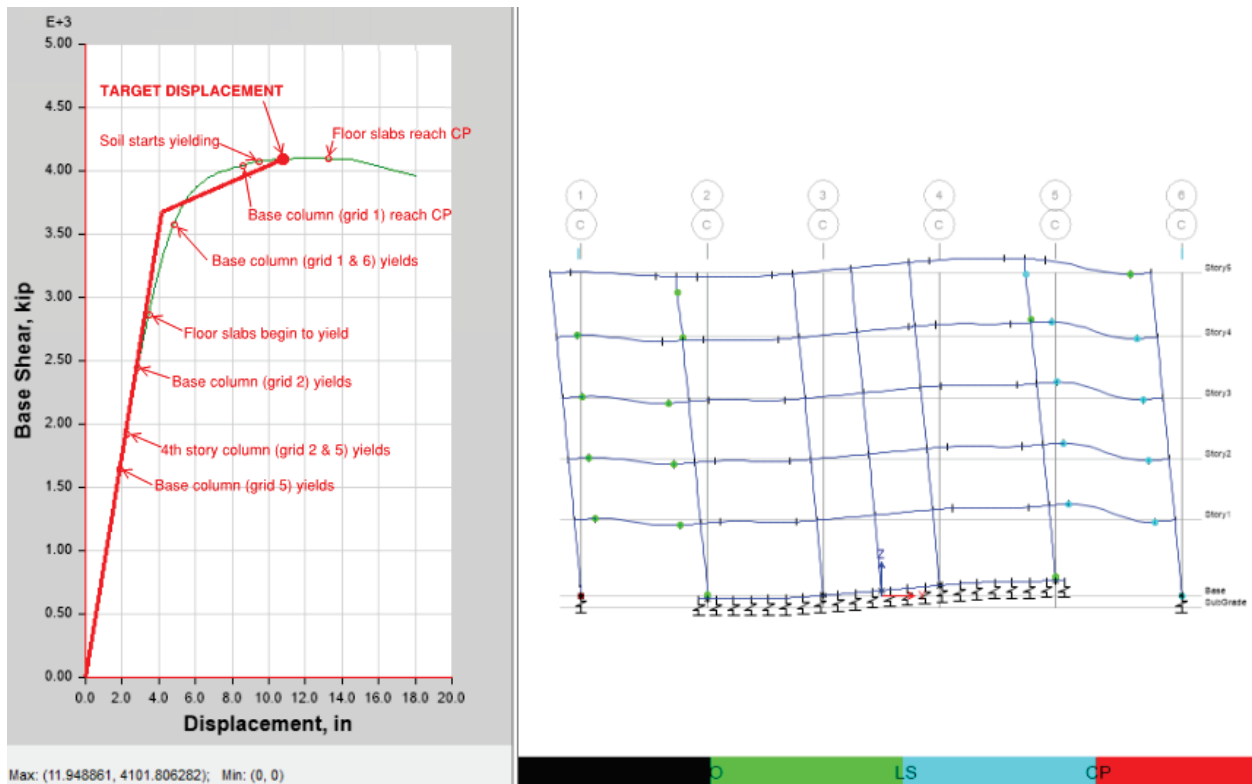
ASCE/SEI 41-17 provides guidance for determining when a foundation is rigid compared to soil in the commentary, § C8.4.2.1 by comparing the foundation stiffness to the soil stiffness in Equation C8-1. The calculations for this footing are shown below. Based on this definition, the footing is not rigid compared to the soil; therefore, Method 1 is not applicable.

$k_{sv} =$	0.09 kip/in <sup>3</sup> (stiffness / (width x length))	
$B =$	8.7 ft (average width)	
$I =$	960033 in <sup>4</sup> (8.7 feet wide x 4 feet deep)	
$E =$	4030.5 ksi 57000*sqrt(f'c)	
$EI/L^4 =$	0.008 ksi	
$2/3k_{sv}B =$	6.6 ksi >>EI/L <sup>4</sup> , not rigid	

**Figure B-69 ASCE/SEI 41-17 § C8.4.2.1 flexibility of shallow foundation.**

#### B.10.5.4 NSP CASE 3: NSP ANALYSIS – METHOD 2 – NON-TUNED SPRING

The soil springs derived in Section B.10.3.2 are utilized in this analysis model and the same footing is used in this analysis case. The flexural demand is then assessed at the target displacement to verify the footing. The footing is undersized for the force-controlled flexural demands, with an acceptance ratio of 2.29.



**Figure B-70 ASCE/SEI 41-17 Nonlinear static pushover, Method 2 flexible base model, ASCE/SEI 41-17 Figure 8-5.**

The pushover is shown in Figure B-70 and the acceptance criteria is determined in the calculations below. The footing rotation at the target displacement meets the acceptance criteria. The flexural action in the footing is also assessed at the target displacement to verify the footing strength. Based on a force-controlled evaluation, the footing strength is inadequate with an acceptance ratio of 2.29.

$P_{UD}$	=	1793 kips		
$q_c$	=	10500 psf		
$A_c$	=	171 ft <sup>2</sup>		
$A_f$	=	612 ft <sup>2</sup>		
$A_c/A_f$	=	0.28		
$b/L_c$	=	0.12		
allowable rotation	=	0.0195 radians, 41-17 Table 8-4		
rotation at target	=	0.0144 radians		
acceptance ratio	=	0.74		

**Figure B-71 ASCE/SEI 41-17 Method 2, Figure 8-5 – soil acceptance criteria.**

One discussion point to note, the acceptance criteria in Table 8-4 is highly dependent on the  $A_c/A_f$  factor and the  $b/L_c$  of the footing. The allowable rotation is highly sensitive to the footing area, and in this case the footing width since the length is constrained. The sensitivity of the acceptance criteria

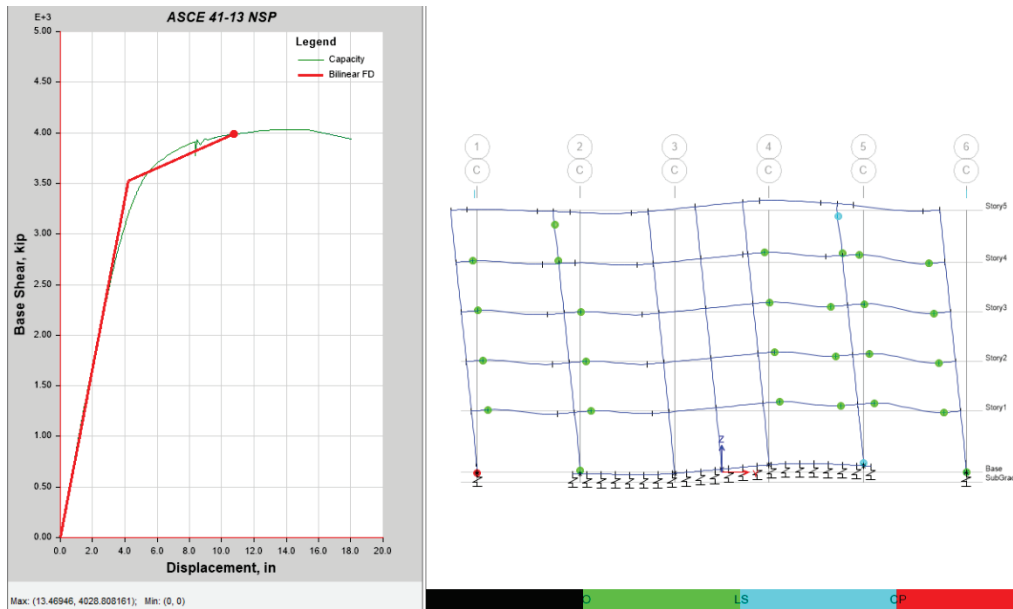
to the footing width is shown in Table B-39; when the footing width doubles, the allowable rotation increases by a factor of 5.7. Further investigation into the allowable rotation sensitivity to footing geometry and the  $A_c/A_f$  factor in accordance with ASCE/SEI 41-17 Table 8-4 is recommended.

**Table B-39 Summary of Footing Retrofit Allowable Rotations from ASCE/SEI 41-17 Table 8-4 (for rectangular footing)**

Comparison of Footing Size and Allowable Rotation	
Footing Width	Allowable Rotation
4'-0"	0.0030 radians
5'-0"	0.0040 radians
6'-0"	0.0100 radians
7'-0"	0.0141 radians
8'-0"	0.0172 radians

**B.10.5.5 NSP CASE 4: NSP ANALYSIS – METHOD 2 –TUNED SPRING, GAJAN ET AL.**

The analysis model is then updated with the Method 2 tuned springs derived in Section B.10.3.3. The same ASCE/SEI 7-10 designed footing is used in this analysis and is evaluated for force-controlled flexure at the target displacement. The revised soil springs negligibly change the response of the structure, shown in Figure B-72.



**Figure B-72 ASCE/SEI 41-17 Nonlinear static pushover, Method 2 flexible base model, Gajan tuned springs.**

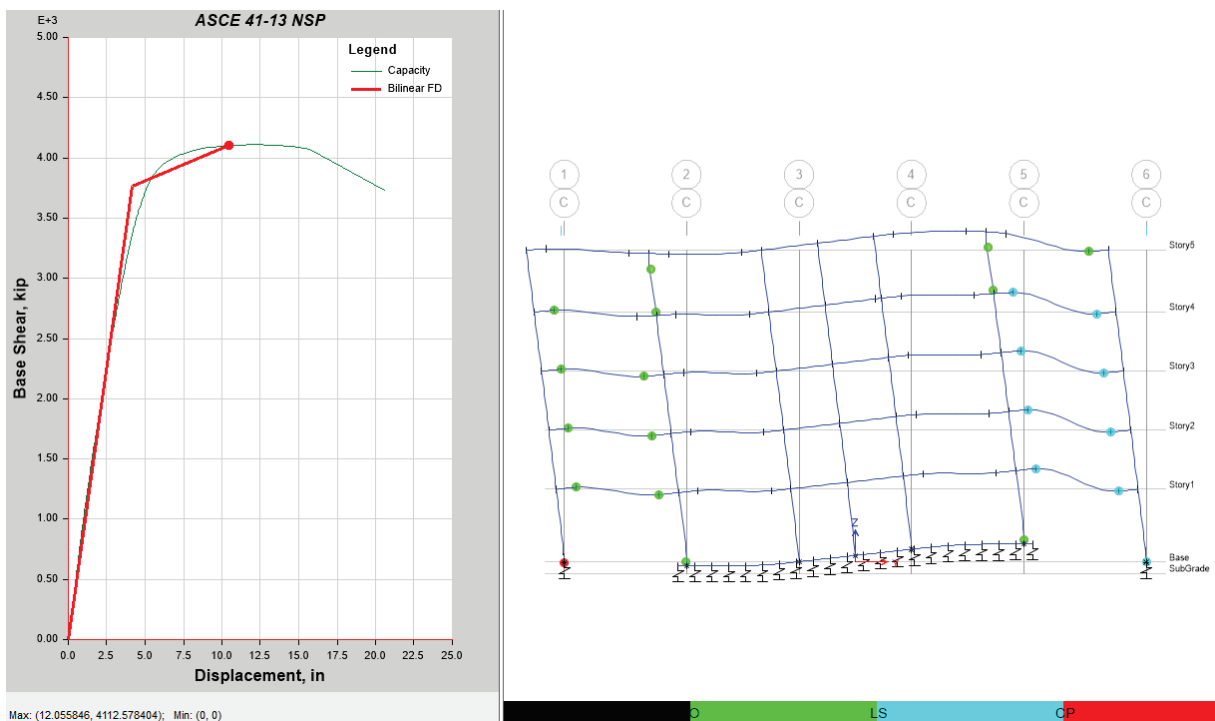
The acceptance ratio for the footing rotation is similar to the initial Method 2 results as shown in the calculations below. The flexural foundation acceptance ratio is 1.69.

$P_{UD} =$	1810 kips
$q_c =$	10500 psf
$A_c =$	172 ft <sup>2</sup>
$A_f =$	612 ft <sup>2</sup>
$A_c/A_f =$	0.28
$b/L_c =$	0.12
allowable rotation =	0.0193 radians, 41-17 Table 8-4
rotation at target =	0.0131 radians
acceptance ratio =	0.68

**Figure B-73 ASCE/SEI 41-17 Method 2, Gajan tuned – soil acceptance criteria.**

### B.10.5.6 NSP CASE 5: NSP ANALYSIS – METHOD 3 – FORCE-CONTROLLED FOUNDATION

The Method 3 soil springs (Section B.8.3.1.3) are incorporated into the model and the same footing is assessed for the force-controlled flexure at the target displacement. The Method 3 pushover analysis is shown in Figure B-74.



**Figure B-74 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model.**

The ASCE/SEI 7-10 footing is evaluated for force-controlled flexure and is not acceptable with an acceptance ratio of 2.39. The acceptance criteria calculations for the footing rotation are shown below.

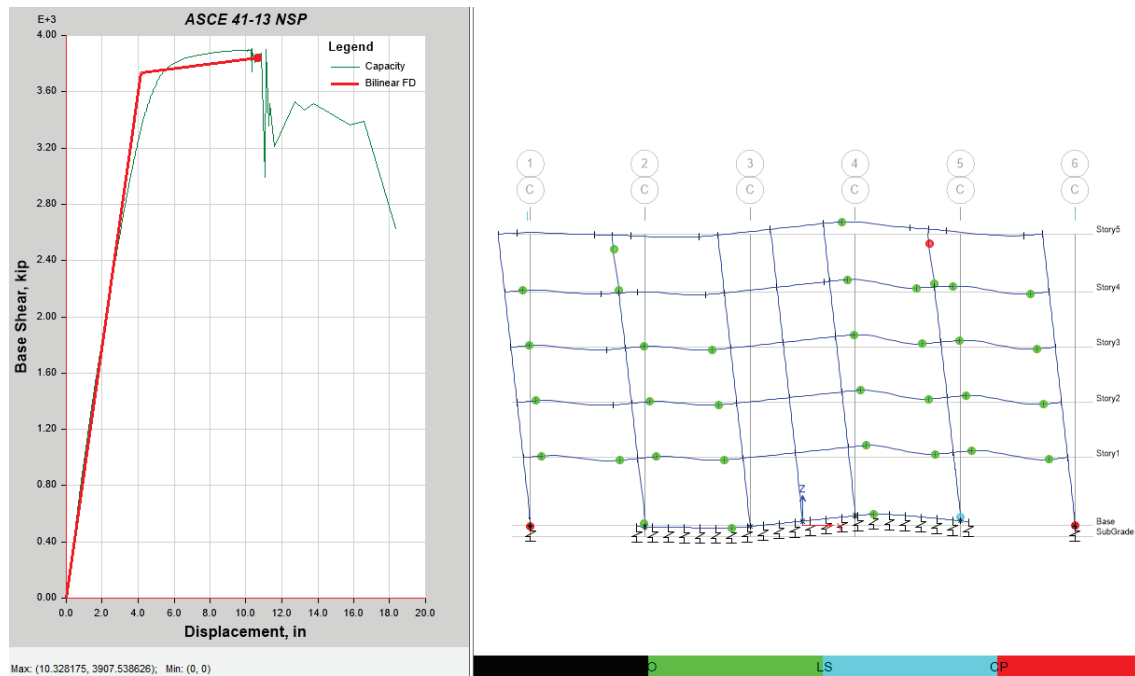
$P_{UD} =$	1842	klps		
$q_c =$	10500	psf		
$A_c =$	175	ft <sup>2</sup>		
$A_f =$	612	ft <sup>2</sup>		
$A_c/A_f =$	0.29			
$b/L_c =$	0.12			
allowable rotation =	0.0190	radians, 41-17 Table 8-4		
rotation at target =	0.0137	radians		
acceptance ratio =	0.72			

**Figure B-75 ASCE/SEI 41-17 Method 3, force-controlled – soil acceptance criteria.**

#### **B.10.5.7 NSP CASE 6: NSP ANALYSIS – METHOD 3 – DEFORMATION-CONTROLLED FOUNDATION**

A separate analysis using Method 3 soil springs is performed with the foundation structure evaluated as deformation-controlled. Although foundations are typically required to be evaluated as force-controlled in accordance with ASCE/SEI 41-17 § 10.12.3, the nonlinear modeling and acceptance criteria provisions for concrete beams within ASCE/SEI 41-17 Chapter 10 are applied to the foundation structure for this case. The ASCE/SEI 7-10 designed retrofit footing (6-foot-wide by 4-foot-deep with (30) #11 bars top and bottom) is incorporated into the analysis model with flexural hinges assigned to each end of the footing beams between the existing footings. The footing beam hinge collapse prevention acceptance criteria is assessed in accordance with the provisions of ASCE/SEI 41-17 Chapter 10. The acceptance ratio for the footing at the target displacement is 0.43. Therefore, the retrofit footing design is acceptable based on a deformation-controlled foundation design.

The pushover curve and deflected shape at the target displacement are shown in Figure B-76. The fundamental period of this model is 0.63 seconds and the target displacement is 10.7 inches.



**Figure B-76 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model.**

The rotation at the base of the shear wall at the target displacement is compared to the acceptance criteria for footing rotation from ASCE/SEI 41-17 Table 8-4. The acceptance criteria calculations are shown in Figure B-77.  $L_c$  is taken as the full length of the footing, similar to previous calculations. In the next analysis case, a different interpretation of  $L_c$  is explored.

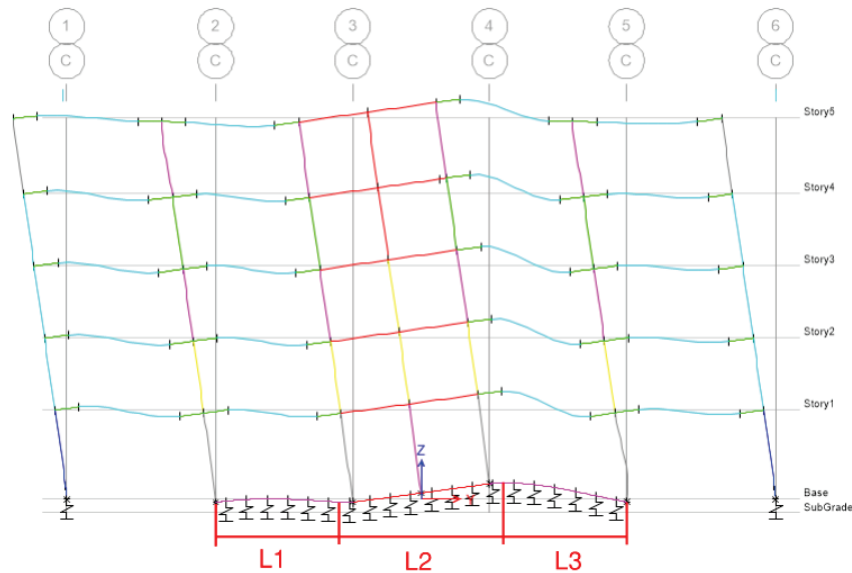
$P_{UD} =$	1768 kips		
$q_c =$	10500 psf		
$A_c =$	168 ft <sup>2</sup>		
$A_f =$	612 ft <sup>2</sup>		
$A_c/A_f =$	0.28		
$b/L_c =$	0.12		
allowable rotation =	0.0198 radians, 41-17 Table 8-4		
rotation at target =	0.0145 radians		
acceptance ratio =	0.73		

**Figure B-77 ASCE/SEI 41-17 Method 3, deformation-controlled – soil acceptance criteria.**

### B.10.5.8 NSP CASE 7: NSP ANALYSIS – METHOD 3 – DEFORMATION-CONTROLLED FOUNDATION, ACCEPTANCE CRITERIA DETERMINED AT INFLECTION POINTS

FEMA P-2006 is an Example Application Guide for ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings with Additional Commentary for ASCE/SEI 41-17. As discussed in FEMA P-2006 § 5.7.6.1, a flexible footing could be assessed by evaluating individual sections separated at inflection points. See FEMA P-2006 for more information. For this case, the acceptance criteria is recalculated for a similar condition to NSP Case 6 but with the soil acceptance criteria evaluated

with  $L_c$  defined for individual segments based on flexural inflection point locations. The three footing portions are shown in Figure B-78.



**Figure B-78 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model (Equation 8-11). Footing divided into three sections for acceptance criteria calculations.**

The results of the acceptance criteria calculations and acceptance ratios are shown in Table B-40. For each footing segment, the axial load and footing dimensions are used to calculate allowable rotations. All of the segments meet their acceptance criteria. The highest loaded segment, L1, also has the lowest rotation as the beam hinge adjacent to it is yielding which reduces the rotation demand.

**Table B-40 Summary of Analysis Results for the Retrofit Structure Foundation**

	$A_c/A_f$	Allowable Rotation, CP (radians)	Actual Rotation at Target (radians)	Acceptance Ratio
L1	0.78	0.0018	0.0009	0.49
L2	0.06	0.0424	0.0145	0.34
L3	0.00	0.0500	0.0143	0.20

Further investigation into the determination of  $L_c$  should be used for flexible foundations in accordance with ASCE/SEI 41-17 Table 8-4 is recommended.

**B.10.5.9 SUMMARY OF FOUNDATION ACCEPTANCE CRITERIA**

Table B-41 summarizes the previous seven nonlinear analyses, along with LSP for comparison, and the foundation acceptance criteria. For each analysis case, the soil acceptance ratio and the footing acceptance ratio are summarized. The footing acceptance ratio is calculated for each case utilizing the retrofit footing designed to ASCE/SEI 7-10 provisions. The only ASCE/SEI 41-17 analysis case where the footing is acceptable is the case where the footing is evaluated as deformation-controlled.

**Table B-41 Summary of Analysis Results for the Retrofit Structure Foundation**

Model	Initial Fundamental Period (seconds)	Target Displacement (inches)	Base Shear (kips) <sup>(1)</sup>	$P_{UD}$ (kips)	$A_c/A_f$	Allowable rotation (radians)	Rotation at target (radians)	Soil Acceptance Ratio	Footing <sup>(2)</sup> Acceptance Ratio	Overall Outcome
LSP – ASCE/SEI 7-10	0.45	-	0.17W	1304	-	-	-	0.95 (max)	0.97	OK
LSP – fixed base	0.45	-	1.3W	1304	-	-	-	1.32 (max)	4.90	NG
LSP – method 1 <sup>(3)</sup>	0.54	-	1.3W	1304	-	-	-	1.16	1.37	NG
NSP – fixed base	0.45	5.3	1.0W	-	-	-	-	-	-	
NSP – method 1 <sup>(3)</sup>	0.54	10.8	0.57W	1405	0.22	0.0238	0.0138	0.55	1.37	NG

<sup>(1)</sup> W is equal to the effective seismic weight of the superstructure equal to 7,200 kips,

<sup>(2)</sup> Footing is 6 ft wide by 4 feet deep with (30) #11 top and bottom. The acceptance ratio is based on a force-controlled design of the footing in flexure unless stated otherwise,

<sup>(3)</sup> Method 1 and Method 2 are not applicable as the footing is not rigid relative to the soil, but are investigated here for comparison.

**Table B-41 Summary of Analysis Results for the Retrofit Structure Foundation (Continued)**

Model	Initial Fundamental Period (seconds)	Target Displacement (inches)	Base Shear (kips) <sup>(1)</sup>	$P_{UD}$ (kips)	$A_c/A_f$	Allowable rotation (radians)	Rotation at target (radians)	Soil Acceptance Ratio	Footing <sup>(2)</sup> Acceptance Ratio	Overall Outcome
NSP – Method 2 (non-tuned) <sup>(3)</sup>	0.66	10.7	0.57W	1793	0.28	0.195	0.0144	0.74	2.29	NG
NSP – Method 2 (tuned) <sup>(3)</sup>	0.68	10.7	0.55W	1810	0.28	0.0193	0.0131	0.68	1.69	NG
NSP – method 3 (force-controlled)	0.63	10.5	0.57W	1842	0.29	0.0190	0.0137	0.72	2.39	NG
NSP – method 3 (deformation-controlled)	0.63	10.7	0.53W	1768	0.28	0.0198	0.0145	0.73	0.43	OK
NSP – method 3 Acceptance Criteria in sections	“	“	“	1633 135 0	0.78 0.06 0.00	0.0018 0.0424 0.0143	0.0009 0.0145 0.0143	0.49 0.34 0.29	“	OK

<sup>(1)</sup> W is equal to the effective seismic weight of the superstructure equal to 7,200 kips,

<sup>(2)</sup> Footing is 6 ft wide by 4 feet deep with (30) #11 top and bottom. The acceptance ratio is based on a force-controlled design of the footing in flexure unless stated otherwise,

<sup>(3)</sup> Method 1 and Method 2 are not applicable as the footing is not rigid relative to the soil, but are investigated here for comparison.

**B.10.5.10 SUPERSTRUCTURE RESULTS**

The superstructure results are determined for columns, slabs and the shear wall for each analysis model. The acceptance ratios from the superstructure are shown in Table B-42 through Table B-46. The LSP acceptance ratios compare the demand from ETABS to the deformation-controlled capacity in accordance with ASCE/SEI 41-17 equation 7-36. None of the superstructure elements investigated here are evaluated as force-controlled. The acceptance ratios for the NSP analyses compare the hinge rotation in the member under consideration to the acceptance criteria for collapse prevention as specified in ASCE/SEI 41-17 Chapter 10. If there is no inelastic rotation in the hinge at the target displacement, the acceptance criteria is listed as 0.00.

The same retrofit shear walls were utilized for all cases for effective comparison. Based on the results, the shear wall design could be optimized for the nonlinear design. In the nonlinear analysis no inelastic behavior occurs in the shear wall, the deformations are pushed into the surrounding structure which can be seen in the acceptance ratios of the existing columns and slabs.

The exterior columns do not meet the acceptance criteria as shown in the NSP pushover deformed shape with hinge acceptance figures in the previous results. These columns are intended to be retrofit and are not shown in the tables below. The existing structure was modeled as-is for all the analysis, however additional retrofit scope is likely required depending on the approach.

**Table B-42 Existing Interior Column - Moment Acceptance Ratios (CP Limit State)**

Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP – Fixed Base	0.99	0.55	0.37	0.56	1.35	NG
NSP – Method 1 Springs	0.20	0.00*	0.00*	0.00*	0.81	OK
NSP – Method 2 Springs Tuned	0.66	0.00*	0.00*	0.00*	0.82	OK
NSP – Method 3 Deformation Controlled	0.91	0.00*	0.00*	0.00*	1.00	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

**Table B-43 Existing Column - Shear Acceptance Ratios (CP Limit State)**  
**(For Nonlinear Cases Acceptance Ratio is the Same as the Moment Acceptance Ratio)**

Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP – Fixed Base	0.49	0.43	0.29	0.47	1.13	NG
NSP – Method 1 Springs	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 2 Springs Tuned	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 3 Deformation Controlled	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

**Table B-44 Retrofit Shear Wall - Shear Acceptance Ratios (CP Limit State)**

Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP – Fixed Base	0.48	0.96	0.87	0.72	0.34	NG
NSP – Method 1 Springs	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 2 Springs Tuned	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 3 Deformation Controlled	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

**Table B-45 Retrofit Shear Wall - Moment Acceptance Ratios (CP Limit State)**

Analysis Model	1 <sup>st</sup> Story	2 <sup>nd</sup> Story	3 <sup>rd</sup> Story	4 <sup>th</sup> Story	5 <sup>th</sup> Story	Outcome
LSP – Fixed Base	0.87	0.99	0.65	0.93	0.36	OK
NSP – Method 1 Springs	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 2 Springs Tuned	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 3 Deformation Controlled	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

**Table B-46 Existing Slab – Flexure Acceptance Ratios (CP Limit State)**

Analysis Model	2 <sup>nd</sup> Floor	3 <sup>rd</sup> Floor	4 <sup>th</sup> Floor	5 <sup>th</sup> Floor	Roof	Outcome
LSP – Fixed Base	0.48	0.62	0.72	0.73	0.48	OK
NSP – Method 1 Springs	0.82	0.98	0.98	0.97	0.68	OK
NSP – Method 2 Springs Tuned	0.58	0.57	0.55	0.42	0.15	OK
NSP – Method 3 Deformation Controlled	0.56	0.58	0.59	0.56	0.28	OK

### B.10.6 Methods for Determining Allowable Rotation at Footing-Soil Interface

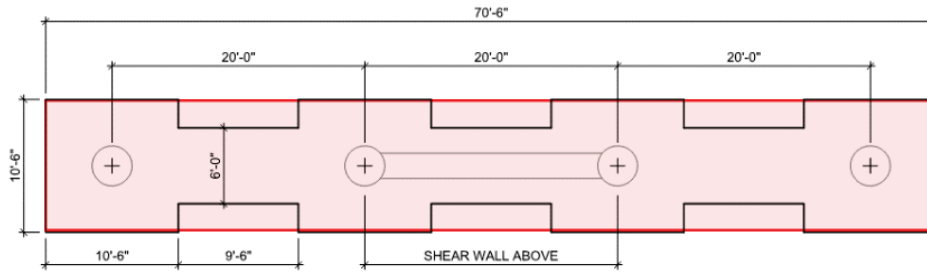
Tables 8-3 and 8-4 in Chapter 8 provide acceptance criteria for I-shaped footings when  $b/L_c$  is between 1 and 10. The retrofit footing used in this study was determined to have a  $b/L_c$  ratio outside of 1 and 10. Further investigation into determining footing allowable rotation was completed with the following methodologies for comparison:

- Entire retrofit footing length and width
- Entire retrofit footing length and effective footing width
- Equivalent I-shaped footing
- Inflection points
- I-shaped footing neglecting interior existing footings

Resulting values for comparison are provided in Table B-47.

#### B.10.6.1 ENTIRE RETROFIT FOOTING LENGTH AND WIDTH

The first method uses the entire retrofit footing length and width, neglecting the missing footing area at the locations of new foundation between existing footings. This method is consistent with the guidance provided in Figure 8-3 for an idealized footing. The idealized flange thickness is taken as the width of the existing isolated footing. The total area is the product of the idealized flange width and footing length.

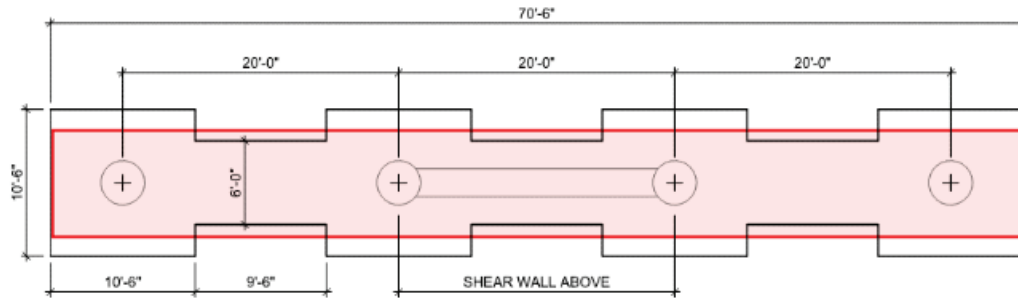


	$b =$	10.5 ft		
	$L =$	70.5 ft		
	$A_f =$	740 ft <sup>2</sup>		
	$P_{UD} =$	1769 kips		
	$q_c =$	10.5 ksf		
	$A_c =$	168 ft <sup>2</sup>		
	$A_f =$	740 ft <sup>2</sup>		
	$A_{rect} =$	740 ft <sup>2</sup>		
	$A_{rect} - A_f / A_{rect} =$	0.00		
	$A_c / A_f =$	0.23		
	$L_c = A_c / b =$	16.05 ft		
	$b / L_c =$	0.65		
	allow. rotation =	0.0306 radians, 41-17 Table 8-4		

**Figure B-79 Allowable footing rotation utilizing entire footing length and width.**

### B.10.6.2 ENTIRE RETROFIT FOOTING LENGTH AND EFFECTIVE WIDTH

This method idealizes the footing as rectangular. An effective width is calculated by dividing the footing area of the retrofit footing by the length. This effective width is used to calculate the footing acceptance criteria.



b=	8.7 ft		
L =	70.5 ft		
A <sub>f</sub> =	613 ft <sup>2</sup>		
P <sub>UD</sub> =	1769 kips		
q <sub>c</sub> =	10.5 ksf		
A <sub>c</sub> =	168 ft <sup>2</sup>		
A <sub>f</sub> =	613 ft <sup>2</sup>		
A <sub>rect</sub> =	613 ft <sup>2</sup>		
A <sub>rect</sub> - A <sub>f</sub> /A <sub>rect</sub> =	0.00		
A <sub>c</sub> /A <sub>f</sub> =	0.27		
L <sub>c</sub> = A <sub>c</sub> /b =	19.37 ft		
b/L <sub>c</sub> =	0.45		
allow. rotation =	0.0224	radians, 41-17 Table 8-4	

**Figure B-80** Allowable footing rotation utilizing entire footing length and effective width.

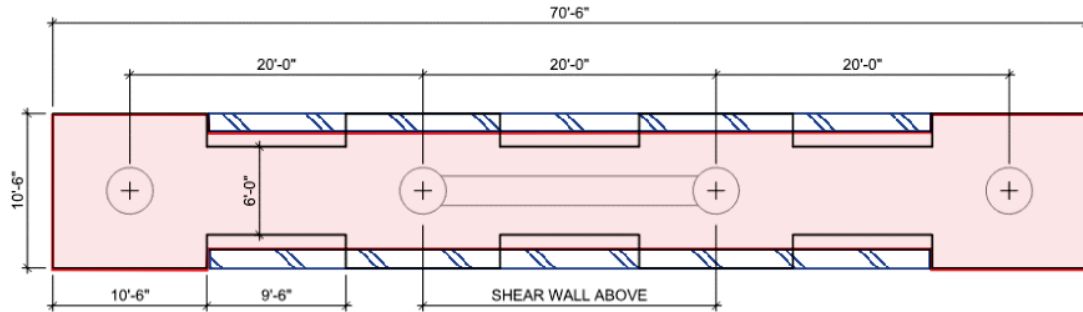
### B.10.6.3 EQUIVALENT I-SHAPED FOOTING

Equivalent I-shaped footing dimensions are calculated to provide an equivalent moment of inertia as the actual footing configuration. The moment of inertia calculation is provided in Figure B-81.

Moment of Inertia - Retrofit Footing			
Contribution of interior existing footings			
b =	10.5 ft		
h =	10.5 ft		
A =	110.25 ft <sup>2</sup>		
d =	10 ft	middle interior footings	
$I = bh^3/12 + ad^2 =$	24,076 ft <sup>4</sup>		
Contribution of web (between existing footings)			
b =	6 ft		
h =	9.5 ft		
A =	57 ft <sup>2</sup>		
d <sub>1</sub> =	0 ft	middle web	
d <sub>2</sub> =	20 ft	exterior web	
$I = bh^3/12 + ad^2 =$	46,886 ft <sup>4</sup>		
Total I =	70,962 ft <sup>4</sup>		
Moment of Inertia of web - determine added width for transformed section			
h =	49.5 ft		
I =	70961.9 ft <sup>4</sup>		
$b = 12I/h^3 =$	7.02 ft		
b =	7.02 ft, width used for transformed footing		

**Figure B-81 Equivalent I-shaped footing moment of inertia.**

The calculated  $b/L_c$  ratio for the equivalent I-shaped footing does not fall within the range specified in ASCE/SEI 41-17 (1 to 10). Allowable rotation varies based on use of rectangular versus I-shaped footing acceptance criteria values provided in Table 8-4. Figure B-82 provides both results for comparison.



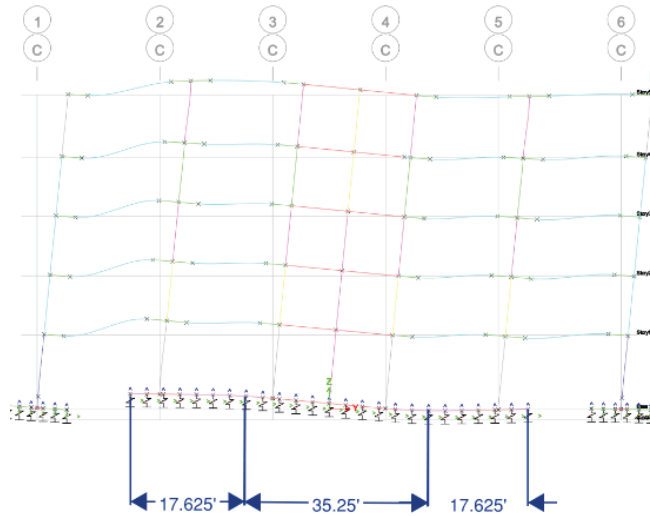
3) I-transform: convert retrofit footing to transformed section

b=	10.5 ft
L =	70.5 ft
A <sub>f</sub> =	568 ft <sup>2</sup>
P <sub>UD</sub> =	1769 kips
q <sub>c</sub> =	10.5 ksf
A <sub>c</sub> =	168 ft <sup>2</sup>
A <sub>f</sub> =	568 ft <sup>2</sup>
A <sub>rect</sub> =	740 ft <sup>2</sup>
A <sub>rect</sub> - A <sub>f</sub> /A <sub>rect</sub> =	0.23
A <sub>c</sub> /A <sub>f</sub> =	0.30
L <sub>c</sub> = A <sub>c</sub> /b =	16.05 ft
b/L <sub>c</sub> =	0.65
allow. rotation =	0.0239 radians, rectangular ftg
	0.0289 I-shaped footing

**Figure B-82 Allowable footing rotation utilizing equivalent I-shape.**

#### B.10.6.4 INFLECTION POINTS

Acceptance criteria is calculated for the portion of the footing between the inflection points based on displaced shape of the footing. The effective footing width is used for this approach.

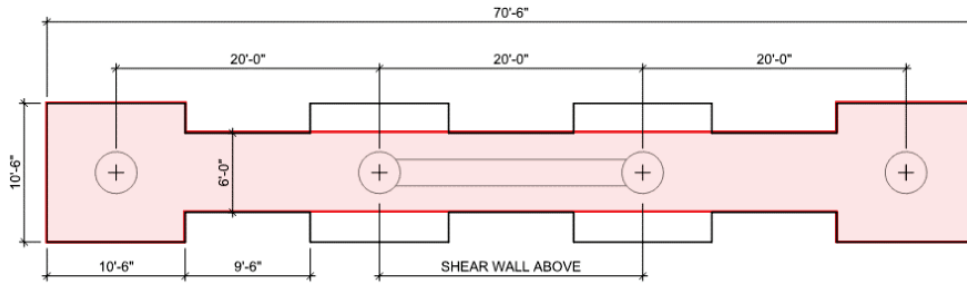


b=	8.7 ft
L =	35.3 ft
$A_f$ =	307 ft <sup>2</sup>
$P_{UD}$ =	756 kips
$q_c$ =	10.5 ksf
$A_c$ =	72 ft <sup>2</sup>
$A_f$ =	307 ft <sup>2</sup>
$A_{rect}$ =	307 ft <sup>2</sup>
$A_{rect} - A_f / A_{rect}$ =	0.00
$A_c / A_f$ =	0.23
$L_c = A_c / b$ =	8.27 ft
$b / L_c$ =	1.05
allow. rotation =	0.0374 radians, 41-17 Table 8-4

**Figure B-83 Allowable footing rotation utilizing inflection points.**

### B.10.6.5 I-SHAPED FOOTING

An I-shaped footing neglecting existing pad footings at the interior columns on either side of the retrofit shear wall is assumed. The calculated  $b/L_c$  ratio for the equivalent I-shaped footing does not fall within the range specified in ASCE/SEI 41-17 (1 to 10). Allowable rotation varies based on use of rectangular versus I-shaped footing acceptance criteria values provided in Table 8-4. Figure B-84 provides both results for comparison.



b=	10.5 ft
L =	70.5 ft
A <sub>f</sub> =	518 ft <sup>2</sup>
P <sub>UD</sub> =	1769 kips
q <sub>c</sub> =	10.5 ksf
A <sub>c</sub> =	168 ft <sup>2</sup>
A <sub>f</sub> =	518 ft <sup>2</sup>
A <sub>rect</sub> =	740 ft <sup>2</sup>
A <sub>rect</sub> - A <sub>f</sub> /A <sub>rect</sub> =	0.30
A <sub>c</sub> /A <sub>f</sub> =	0.33
L <sub>c</sub> = A <sub>c</sub> /b =	16.05 ft
b/L <sub>c</sub> =	0.65
allow. rotation =	0.0210 radians, rectangular ftg
	0.0251 I-shaped footing

**Figure B-84 Allowable footing rotation utilizing I-shaped footing.**

### B.10.6.6 SUMMARY

The following table compares the allowable footing rotation values using each of the methodologies described above.

**Table B-47 Summary of Results for Allowable Footing Rotation**

Approach	A <sub>c</sub> /A <sub>f</sub>	b/L <sub>c</sub>	Allowable Rotation (rad)
Entire footing length and width	0.23	0.65	0.0306
Entire footing length and effective width	0.27	0.45	0.0224
Equivalent I-shaped footing	0.30	0.65	0.0239
Inflection Points	0.27	0.45	0.0374
I-shaped footing neglecting interior existing footings	0.33	0.65	0.0210

## Conclusions

- The current provisions can be applied to non-rectangular or non-I-shaped footings by a number of methods.
- Guidance should be provided to the user for cases where I-shaped footings when  $b/L_c$  is not between 1 and 10.
- There are multiple approaches to determining allowable rotations for the atypical foundation configuration.
- I-shaped vs. rectangular footings provide numerically different allowable rotations.
- Rotation demand can be determined as rotation between end points of wall or between points of contraflexure.

## B.11 References

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