Department of Civil and Environmental Engineering

SSI modeling for collapse simulations of a four story steel moment frame building

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Author:
Somayajulu Dhulipala

Affiliation:
Structural Engineering

Faculty:
Dr. Madeleine Flint

Affiliation:
Structural Engineering

Faculty:
Dr. Guney Olgun

Affiliation:
Geo-technical Engineering

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Contents

1 Introduction

Performance assessment of structural systems under earthquake loading is crucial for their safety and serviceability. In particular, collapse capacity assessment of buildings has received much attention in the recent past because of its socio-economic and environmental impacts (see for example [1]). Given the random nature of earthquake occurrence in terms of magnitude and ground motion amplitude etc., collapse capacity assessment of buildings is made probabilistically. The probability of collapse of a building given a scalar earthquake Intensity Measure (IM) is termed as the collapse fragility [1]. The commonly used scalar IM is the spectral acceleration at fundamental mode period ($Sa(T1)$). Collapse fragility of a building is evaluated by conducting Incremental Dynamic Analysis (IDA). In an IDA, a model of a building is subjected to an earthquake acceleration time series multiplied by a scale factor which just causes onset structural instability and hence collapse. The collapse scale factor for a particular earthquake record is determined through trial and error by scaling up the record. This collapse scale factor multiplied by $Sa(T1)$ for this acceleration time history gives the collapse capacity of the building under consideration. The numerical codes to perform collapse assessment declare the building to be collapsed when a pre-defined global deformation criterion are met. By utilizing a suite of ground motion records and conducting multiple IDA, a distribution of collapse capacities is obtained. In general, the natural logarithm of collapse capacities are assumed to be normally distributed. And the collapse fragility is obtained using equation (1). In this equation, Φ represents the normal cumulative density, $\mu_{ln(Sa(T1))}$ represents the log-median collapse capacity and $\sigma_{ln(Sa(T1))}$ represents log-standard deviation in collapse capacity.

$$Pr.(Collapse \mid Sa(T1) = x) = 1 - \Phi\Big(\frac{ln(x) - \mu_{ln(Sa(T1))}}{\sigma_{ln(Sa(T1))}}\Big) \tag{1}$$

Soil-Structure-Interaction (SSI) is a field in the intersection between Structural and Geo-technical engineering which deals with the interplay amid structural and foundation systems. Many previous studies concerning the impact of SSI on structural response reported its reduction as compared to the case without considering SSI (i.e. fixed base) (see for example [2]). In all the previous studies for collapse assessment of buildings,however, the effects of Soil-Structure-Interaction (SSI) were ignored. So, an investigation on the influence of SSI on structural collapse capacity is an open question. Finding the collapse capacity of structures can be described as a problem in chaos theory. In a typical chaos problem, small perturbations to the input variables cause large deviations in the final results. In the context of SSI, as compared to the case with fixed base, if SSI causes small differences to structural response at low levels of seismic demands, then it might cause large deviations to collapse capacities at high seismic demands. So, investigation of SSI's influence on collapse capacity is beneficial given the much emphasis being placed on collapse of structures in contemporary earthquake engineering research. And because of the same reason, collapse capacity being a chaos problem, it is crucial to model both the structural and foundation systems adequately in order to predict collapse capacity with reasonable accuracy.

In this report, the modeling and validation details are presented for simulating seismic structural collapse considering SSI. This investigation is a part of the ongoing RSB project. In section 2, the soil model adopted and its validation at Virginia Tech. are described. In section 3, the structure, problems with its two-dimensional idealization while considering SSI and a proposed solution are presented. Finally, a procedure to validate the proposed approach is discussed in section 4.

2 Foundation model and its validation

A classical way to model the soil behavior under the footings is to use linear vertical springs having an elastic stiffness. Such springs are termed as Winkler springs. For collapse simulations, however, soil non-linearity has to be taken into account to capture realistic behavior of structure and foundations. So, the Beam on Nonlinear Winkler Foundations (BNWF) model proposed by Raychowdhury and Hutchinson (2009) has been adopted. More detailed description of this model is given in section 2.1.

2.1 Beam on Non-linear Winkler Foundation (BNWF) model

A foundation, in general, has three modes of resistances. These are: resistance against lateral deformation, resistance against vertical deformation and resistance against rotation. In a BNWF model, the mentioned resistances are simulated using nonlinear springs. The lateral resistance offered by the foundation is divided into two components. The passive resistance and the sliding resistance. The passive resistance is modeled through a single horizontal nonlinear spring. The sliding resistance is modeled in a similar fashion. The vertical resistance, on the other hand, is modeled via several vertical nonlinear springs. The vertical springs at the two ends of the footing are made more stiffer in comparison to the footing mid-zone springs to account for rotational resistance. Each of the nonlinear springs in the BNWF model has a nonlinear backbone curve defined by an initial stiffness, ultimate strength and deformation at fifty percent of ultimate strength; and a hysteresis defined by experimentally calibrated parameters. A schematic of the BNWF model and hystereses plots for various springs is shown in Figure 1.

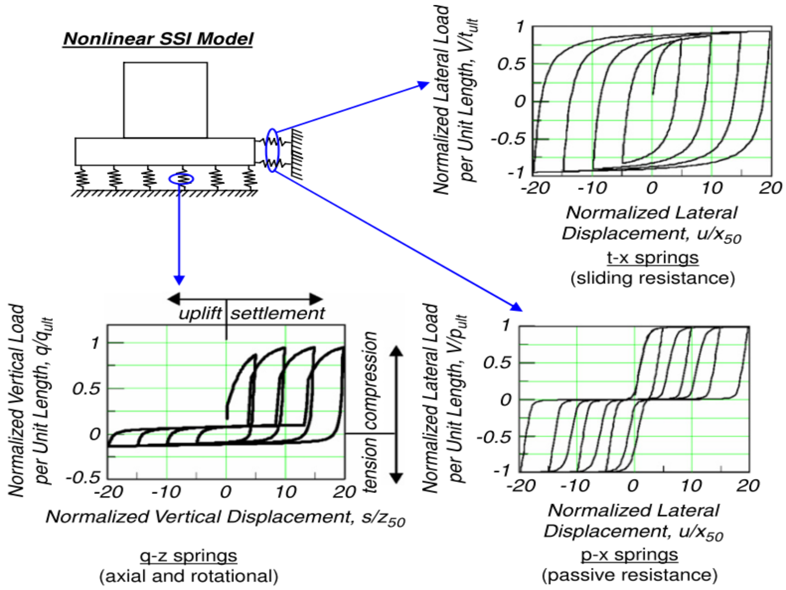

Figure 1: Schematic of the BNWF model showing vertical, passive and sliding springs and corresponding hysteresis (adapted from [4]).

For defining the backbone curve, the initial stiffness is calculated using standard formulas such as those proposed by Gazetas 1992 cite. The equations for initial stiffness of Gazetas 1992 take into account both the footing geometrical properties and soil properties. The ultimate resistance is calculated using theoretical formulas such as those proposed by Terzaghi or Coloumb etc., depending upon the orientation and type of spring. For example, if the spring is vertical, then either Terzaghi's theory or Meyerhoff's theory is used to calculate the ultimate resistance. Or if the spring is lateral-passive, then either Coloumb's or Rankine's theory is used. The final parameter to define the backbone curve is the deformation at fifty percent ultimate resistance (denoted as z_{50}). This parameter is mathematically defined in equation (2). In this equation, Q_{Ult} is the spring ultimate resistance, K_{init} is the spring initial stiffness, and F is an experimentally calibrated parameter.

$$z_{50} = F \, \frac{Q_{Ult}}{K_{init}} \tag{2}$$

2.2 Inputs for the BNWF model in OpenSees

The BNWF model has been implemented in OpenSees and Table 1 lists the inputs required.

Table 1: Inputs for the BNWF model

Property	Value	Notes
Soil related properties		
Specific gravity of solids	G	–
Unit weight	γ	–
Friction angle	ϕ	–
Poisson's ratio	ν	–
Low strain shear modulus	G_{max}	Seed and Idriss (1970)
Shear wave velocity	Vs	$Vs = \sqrt{\frac{g\ G_{max}}{\gamma}}$
Soil-footing interface friction co-efficient	f	–
Footing related properties		
Footing size	$L\ B\ H$	–
Embedment depth	D	–
Stiffness in vertical direction	K_z	Gazetas' 1991
Stiffness in horizontal direction	K_x or K_y	Gazetas' 1991
Radiation damping ratio	C_{rad}	Gazetas' 1991
Load on the footing	W_g	–
Ultimate vertical bearing capacity	Q_{Ult}	–
Bearing capacity reduction factor	Rd	To account for eccentricities in loading
Ultimate passive resistance	P_{Ult}	–
Ultimate sliding resistance	T_{Ult}	–
Modeling related parameters		
End length ratio	L_e	Fraction of footing end length with stiffer springs
End stiffness ratio	K_e	Stiffness increment factor for end springs
Vertical spring spacing	s	–

- g is the acceleration due to gravity

2.3 Validation of the BNWF model at Virginia Tech.

The BNWF model proposed by [3] has been validated at Virginia Tech. against experiments conducted at UC Davis. The validation has been done for two types of tests: Static and Dynamic. For both the tests, the structure resting on the foundations is a shear wall. The stick model of the foundation and shear wall is shown in Figure 2. In the static test, the shear wall is subjected to a slow displacement time history at an eccentricity from the footing. In the dynamic test, both the shear wall and foundation are subjected to dynamic base shaking. The validation results for the two types of tests presented here are: SSG02 03 (static test) and SSG03 07 (dynamic test).

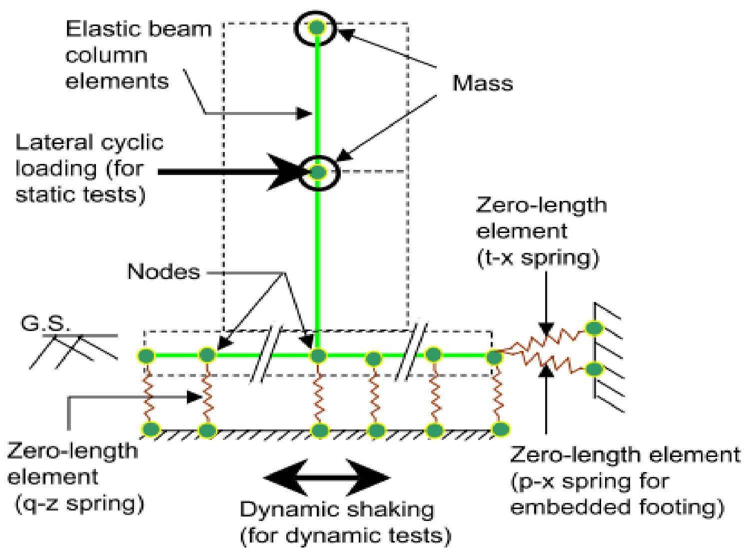

Figure 2: Schematic of the stick model used for BNWF model validation (adapted from [4]).

2.3.1 SSG02 03 (static test) validation

In this test, the foundation system is subjected to slow cyclic displacement at an eccentricity of 4.9 meters. The peak input displacement is around 360 mm. The foundation is resting on dense sand having a relative density of 85 percent. The detailed properties of soil, footing and modeling are summarized in appendix.

A comparison is made between the various force and displacement quantities recorded during centrifuge test at UC Davis and BNWF simulations at UCSD on one hand

and BNWF simulations at Virginia Tech. on the other hand. In particular, the recorded forces are base shear force and base moment; and the recorded displacements are foundation settlement, sliding and rotation. Figure 3 compares these force-displacement quantities. It can be observed that the simulations conducted at Virginia Tech. match closely with both the experiments and simulations at UC Davis and UCSD respectively.

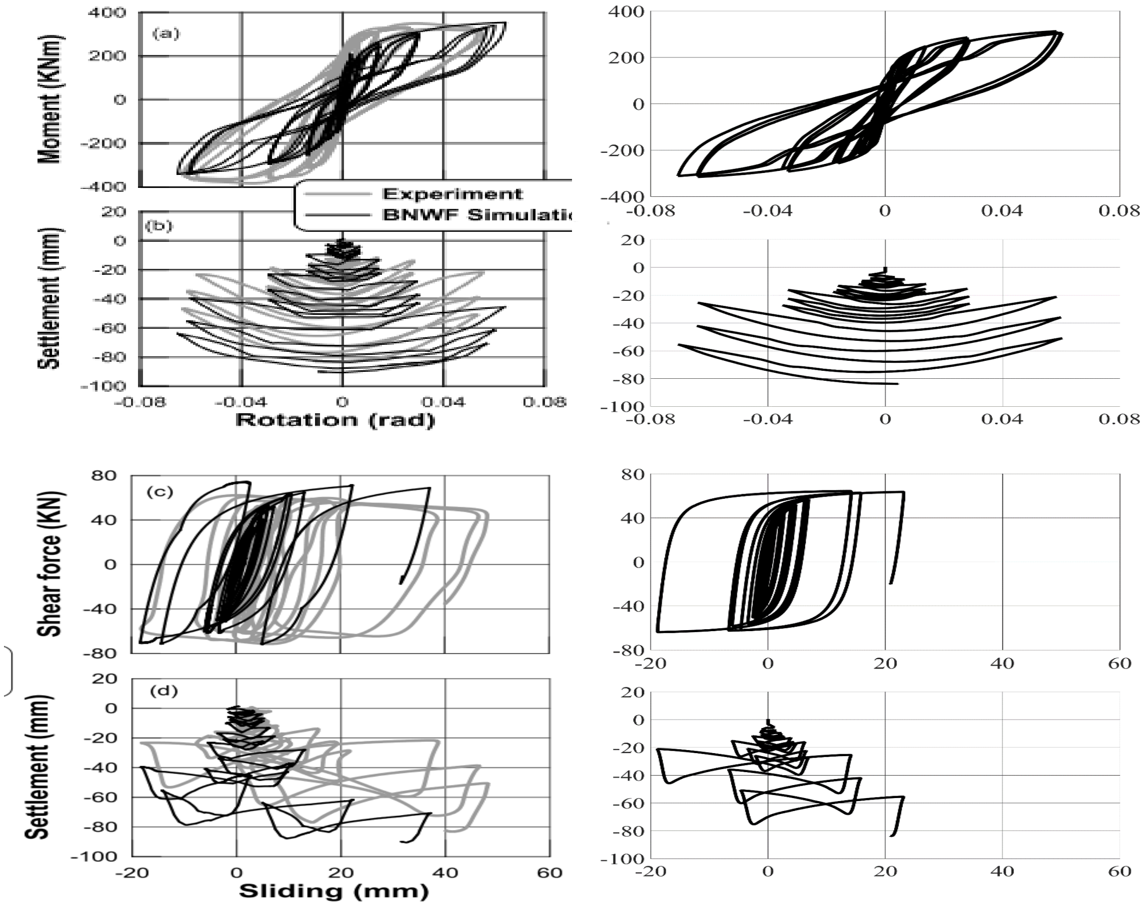

Figure 3: SSG02 03 (static test) validation: Left- Experiments at UC Davis and BNWF simulations at UCSD; Right- BNWF simulations at Virginia Tech.

2.3.2 SSG04 10 (dynamic test) validation

In this test, the foundation system and shear wall are subjected to dynamic base shaking. During simulations the mass of the shear wall has been concentrated at an eccentricity of 5.04 meters. The peak input base acceleration is around 0.6g. The foundation is resting on dense sand having a relative density of 85 percent. The detailed properties of soil, footing and modeling are summarized in appendix.

A comparison is made between the various force and displacement quantities recorded during centrifuge test at UC Davis and BNWF simulations at UCSD on one hand and BNWF simulations at Virginia Tech. on the other hand. In particular, the recorded forces are base shear force and base moment; and the recorded displacements are foundation settlement, sliding and rotation. Figure 4 compares these force-displacement quantities. It can be observed that the simulations conducted at Virginia Tech. match closely with both the experiments and simulations at UC Davis and UCSD respectively.

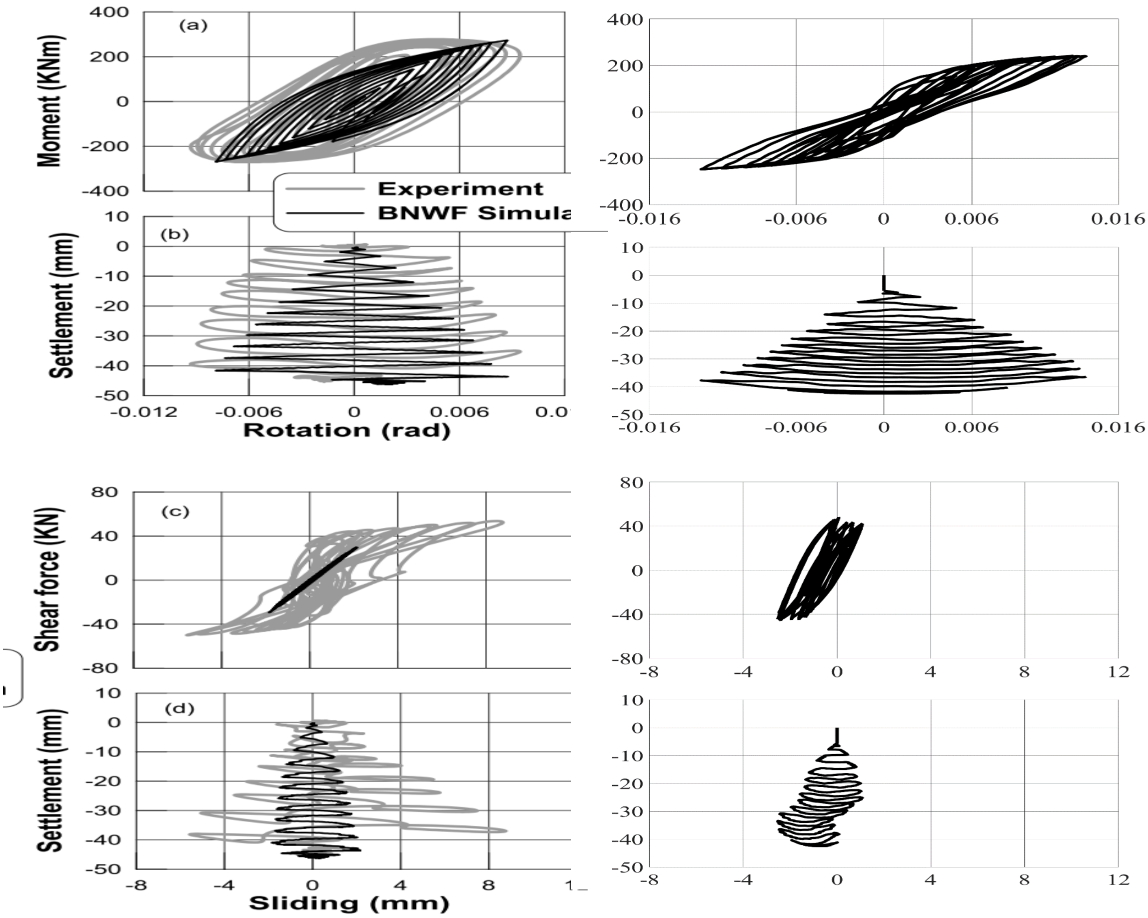

Figure 4: SSG04 10 (static test) validation: Left- Experiments at UC Davis and BNWF simulations at UCSD; Right- BNWF simulations at Virginia Tech.

3 Proposed modifications to structural/foundation model for collapse simulations considering SSI

For conducting collapse simulations considering SSI a four-story steel moment frame is chosen. The collapse simulations will be conducted on a two-dimensional, two bay frame which part of the four-story building. A foundation system consisting of a strip footing resting on low to medium dense sand has be designed. This section first describes the structural model and foundation/soil properties in detail. Then, problems with the two-dimensional idealization of the structure/foundation are discussed. In short, there is excessive lateral sliding of the foundations in the two-dimensional model when subjected to non-scaled earthquake ground motions. To rectify this, a proposed solution which accounts for non-tributary lateral foundation resistance is also discussed.

3.1 Description of the structural model

As described earlier, the structure is a four-story steel moment frame building. This building has been designed for seismic loads in Los Angeles, California. The building has a design base shear co-efficient of $V/W = 0.082$. The plan of this building is shown in Figure 5.

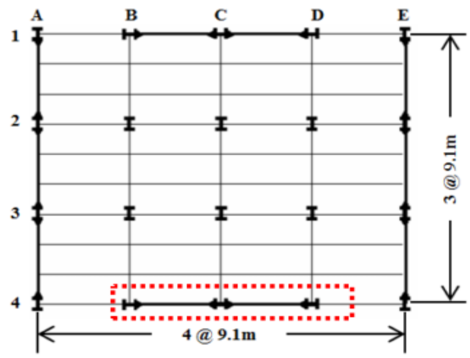

Figure 5: Plan of the four-story steel moment frame building (adapted from [1])

A two-dimensional model of a moment frame which is part of the described building (highlighted in red in Figure 5) has been modeled in OpenSees. Material non-linearity has been accounted for in the model through a lumped plasticity approach. In particular, a bi-linear Ibarra-Krawinkler model has been used with both strength and stiffness deterioration. The model accounts geometric non-linearity and a leaning column has been used to simulate $P - \Delta$ effects on the two-dimensional frame due to non-tributary gravity loads. The elevation of this frame is shown in Figure 6.

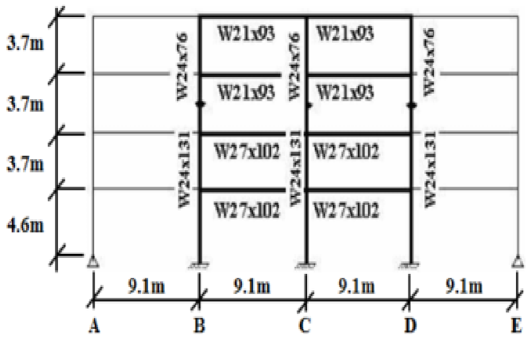

Figure 6: Elevation of the 2D OpenSees model (adapted from [1])

3.2 Foundation design and input model parameters

The soil adopted for collapse simulations with SSI for the 2D frame previously described is a low-medium density sand. Also, it can be seen from Figure 6 that the model has two bays with three column lines. So, a strip footing has been designed to avoid problems with differential settlement during seismic excitation. The detailed properties of soil and strip footing are summarized in Table 2.

Table 2: Soil-footing properties for collapse simulations

Property	Value	Notes
Low to medium dense sand properties		
Unified classification	Poorly graded sand (SP)	–
G	2.65	–
Voids ratio (e)	0.75	–
γ_{Sat}	19 KN/m^3	–
γ	16.2 KN/m^3	–
ϕ	35^o	–
ν	0.3	–
G_{Max}	99 MPa	Seed and Idriss 1970
V_{S30}	228 m/s	–
Interface friction co-efficient	0.67	–
Strip footing properties		
$L\ B\ H$	$20-2-0.5\ m$	–
D	1 m	–
K_Z	3390.32 MN/m	Gazetas' 1991
K_X	5370 MN/m	Gazetas' 1991
C_{rad}	0.05	–
Footing and surcharge weight	824 KN	–
Load from super-structure	2430 KN	–
Q_{Ult}	47258.96 KN	–
FOS (vertical static)	14.5	–
R_d	0.15	PEER report by Harden *et al.*
Input dynamic bearing capacity	7089 KN	–
P_{Ult}	121.5 KN	Coloumb's theory with $K_p = 10$
T_{Ult}	1411.5 KN	–
Design base shear (AISC 2005)	200 KN	–
T_{Ult}	1411.5 KN	–
FOS (lateral static)	7.6	Couldn't reduce this any further
Modeling details		
L_e	0.05	PEER report by Harden *et al.*
K_e	1.5	PEER report by Harden *et al.*
s	0.02 L	–

3.3 Problems with foundation lateral resistance in a 2D building model

The 2D building model with SSI in the form of vertical and lateral springs is shown in Figure 7. In this figure the 2D frame which is part of the four-story building and strip footing are shown in thick lines. The leaning column, rigid links connecting leaning column and frame are shown in thin lines. Also, the leaning column is placed on a roller to allow lateral sliding of the foundation. As mentioned earlier, the foundation vertical load carrying capacity is lumped on several vertical springs. And the lateral load carrying capacity divided into passive and sliding resistances is lumped on one spring each. The properties of soil and footing were summarized in Table 2.

Figure 7: 2D building model with SSI. The 2D frame and strip footing are shown in thick lines. Elements of the leaning column are shown in thin lines.

The 2D frame model of Figure 7 has been subjected to earthquake ground motions in the FEMA P695 record set. This is set of forty-four ground motion records (both components) with magnitudes ranging from 6.5-7.5 and distances ranging from 7.1 Km-26 Km. During the simulations different structural and foundation response quantities such as roof drift, foundation settlement and sliding etc. have been recorded. Figure 8 shows the histogram of foundation lateral displacement when subjected to the FEMA P695 record set. It can be observed that the lateral displacements at foundation are not small. This problem of excessive foundation sliding will be worsened when these earthquake records are scaled during collapse simulations. For example, when the 2D model has been subjected to the one of the Chi-chi earthquake recordings scaled twice, the foundation sliding was around 20″ and there was about fifty percent reduction in roof drift and complete reduction residual drift of the building as compared to the fixed base case. This excessive sliding will have an impact on the collapse analysis results of the building. This also raises the question if the 2D frame/foundation modeling is lacking some important resistance components from the non-tributary part of the building.

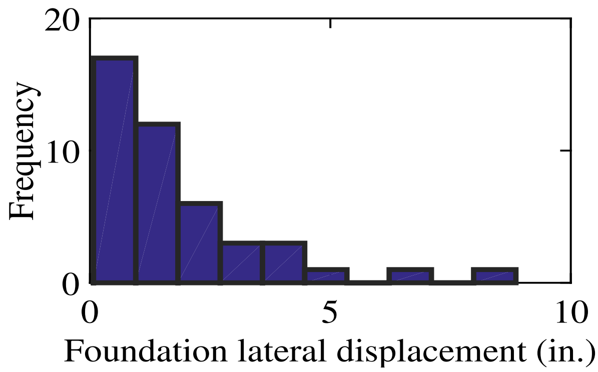

Figure 8: Foundation lateral displacements histogram when subjected to the forty-four ground motions in the FEMA P695 record set.

The reason for this excessive foundation sliding is, the 2D model of the foundation (Figure 7) does not take into account the lateral foundation resistance of the other non-tributary footings in the plan of the building as shown in Figure 5. This is why when the earthquake records are scaled and applied to the 2D model (Figure 7) there is unrealistic sliding of foundation which in turn impacts structural response such as roof drifts, residual drifts etc. The following section proposes a new methodology which accounts for non-tributary foundation resistances in the 2D model.

3.4 Proposed modifications which account for non-tributary foundation resistances

Because of the problems with excessive sliding noted in section 3.3, new additions to the 2D model of Figure 7 (will be referred to as old model) are proposed which account for non-tributary foundation resistances. The proposed model is shown in Figure 9 (will be referred to as new model). In the new model, instead of one Leaning Column (LC) there will be two LCs ($i = 1$ or 2) each carrying half the gravity loads on the single LC of the old model. Each LC is attached to an Imaginary Foundation (IF) system consisting of a sliding spring (S), a passive spring (P) and a vertical spring (V). These springs are defined by an Elastic Perfectly Plastic backbone curve with initial stiffness ($K_{IFi(P \text{ or } S \text{ or } V)}$) and ultimate strength ($Qult_{IFi(P \text{ or } S \text{ or } V)}$). Also it is noted that the Real Foundation (RF) passive, sliding and vertical springs are similar to that described in section 3.3. The calculation of ($K_{IFi(P \text{ or } S \text{ or } V)}$) and ($Qult_{IFi(P \text{ or } S \text{ or } V)}$) will be discussed.

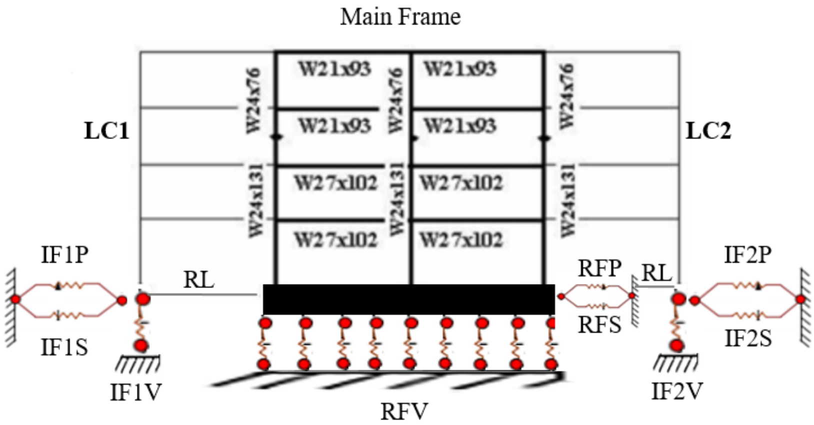

Figure 9: Proposed 2D model to account for non-tributary foundation resistances. (LCi: ith leaning column, IFi(P or S or V): ith imaginary foundation ($Passive$ or $Sliding$ or $Vertical$) spring, RF(P or S or V): Real foundation ($Passive$ or $Sliding$ or $Vertical$) spring(s), RL: Rigid link.)

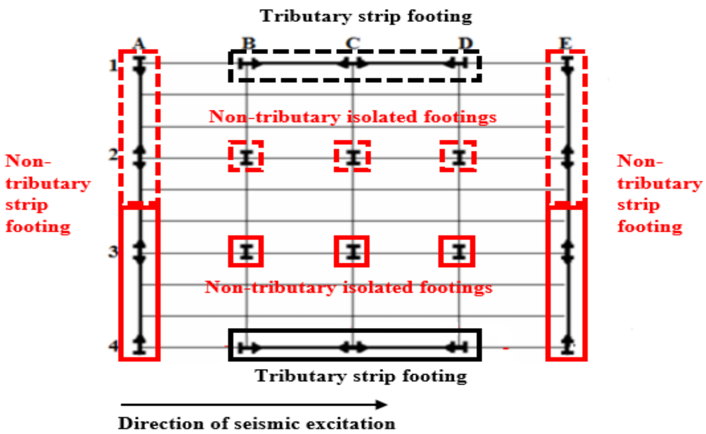

Figure 10: Plan of the building showing foundation layouts. (Solid lines: Foundation resistances contributing to the 2D main frame considered in the analysis. Dotted lines: Foundation resistances contributing to the 2D main frame not considered in the analysis. Black lines: Tributary foundations to the main frame. Red lines: Non-tributary foundations to the main frame.)

The plan of the four-story steel moment frame building with foundation layouts (and descriptions) is shown in Figure 10. The ($K_{IFi(P \text{ or } S)}$) is one half of the sum of (either sliding or passive) stiffnesses of one Non-Tributary Strip Footing ($NTSF$) and three

Non-Tributary Isolated Footings ($NTIF$). This is due to the series arrangement of either passive or sliding springs of one half of non-tributary footings. The ($\frac{1}{K_{IFiV}}$) is two times the sum of inverse vertical stiffnesses of one $NTSF$ and three $NTIF$s. This is due to the parallel arrangement of vertical springs of one half of non-tributary footings. The ($K_{IFi(P \text{ or } S)}$) and ($\frac{1}{K_{IFiV}}$) are mathematically defined in equations (3) and (4).

$$K_{IFi(P \text{ or } S)} = \frac{1}{2}\,(K^{NTSF}_{IFi(P \text{ or } S)} + 3\,K^{NTIF}_{IFi(P \text{ or } S)}) \tag{3}$$

$$\frac{1}{K_{IFiV}} = 2\,(\frac{1}{K^{NTSF}_{IFiV}} + \frac{3}{K^{NTIF}_{IFiV}}) \tag{4}$$

The ultimate resistances ($Qult_{IFi(P \text{ or } S \text{ or } V)}$) is one half of the sum of ultimate resistances of one $NTSF$ and three $NTIF$s. This is shown in equation (5).

$$Qult_{IFi(P \text{ or } S \text{ or } V)} = \frac{1}{2}\,(Qult^{NTSF}_{IFi(P \text{ or } S \text{ or } V)} + 3\,Qult^{NTIF}_{IFi(P \text{ or } S \text{ or } V)}) \tag{5}$$

4 A procedure to validate the proposed approach for collapse simulations considering SSI

The proposed procedure to account for non-tributary foundation resistances during seismic collapse simulations will be validated for a simple test building having two bays in both the principal directions and two stories. The validation will be done by first creating a 3D model of the simple test building with SSI. Then the seismic analysis results are compared to that of its (simple test building) 2D idealization. If the validation is successful, then the seismic collapse simulations using 2D model (Figure 9) of the four-story steel building will be done.

5 Conclusions

Collapse simulations for a four-story steel moment frame building considering SSI will be conducted. For this the Beam on Nonlinear Winkler Foundation (BNWF) model is adopted. The BNWF model's performance has been validated at Virginia Tech. against centrifuge tests at UC Davis. It was observed that direct utilization of the BNWF model for seismic collapse assessment of a 2D frame led to excessive sliding of the foundations, especially when the earthquake records are scaled. This problem arises because the non-tributary foundation resistances are not accounted for in the 2D model. To overcome this issue, a modeling methodology is proposed which takes account non-tributary foundation resistances. The proposed methodology will be validated for a simple 3D test building.

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