

ENHANCING SEISMIC RESILIENCE OF RC BUILDING THROUGH FOUNDATION ROCKING

A Thesis Submitted

in Partial Fulfilment of the Requirements

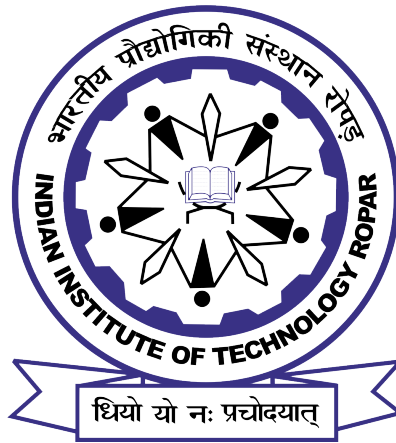
for the Degree of

DOCTOR OF PHILOSOPHY

by

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June, 2024

Manoj Kannan R: *Enhancing Seismic Resilience of RC Building Through Foundation Rocking*

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Dedicated to Sarav and Amma

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I hereby declare that the work which is being presented in the thesis entitled **Enhancing Seismic Resilience of RC Building Through Foundation Rocking** has been solely authored by me. It presents the result of my own independent investigation/research conducted during the time period from August, 2017 to May, 2024 under the supervision of Dr. Naveen James, Associate Professor, Department of Civil Engineering, Indian Institute of Technology Ropar and Dr. Putul Haldar, Assistant Professor, Department of Civil Engineering, Indian Institute of Technology Ropar. To the best of my knowledge, it is an original work, both in terms of research content and narrative, and has not been submitted or accepted elsewhere, in part or in full, for the award of any degree, diploma, fellowship, associateship, or similar title of any university or institution. Further, due credit has been attributed to the relevant state-of-the-art and collaborations (if any) with appropriate citations and acknowledgments, in line with established ethical norms and practices. I also declare that any idea/data/fact/source stated in my thesis has not been fabricated/ falsified/ misrepresented. All the principles of academic honesty and integrity have been followed. I fully understand that if the thesis is found to be unoriginal, fabricated, or plagiarized, the Institute reserves the right to withdraw the thesis from its archive and revoke the associated Degree conferred. Additionally, the Institute also reserves the right to appraise all concerned sections of society of the matter for their information and necessary action (if any). If accepted, I hereby consent for my thesis to be available online in the Institute's Open Access repository, inter-library loan, and the title & abstract to be made available to outside organizations.

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Acknowledgement

Acknowledgements are an opportunity to express heartfelt gratitude for peoples those who have directly or indirectly supported throughout the duration of Doctoral journey. I am profoundly grateful to my supervisors Dr. Naveen James and Dr. Putul Haldar for providing me with unwavering support and guidance. Your support has been the cornerstone of my journey, and your guidance has not only contributed to my journey of doctoral thesis alone but also had a direct influence on my knowledge gain as a research scholar. I am thankful for your encouragement, for believing in my abilities and for providing freedom to carry out my research works peacefully.

I am undoubtedly thankful to my doctoral committee members Prof. Deepak Kashyap, Dr. Reet Kamal Tiwari, Dr. Sagar Rohidas Chavan, Dr. Amar Nath Roy Chowdhury, Dr. Aditya Singh Rajput and Dr. Samir Chandra Roy for your insights and feedback throughout this process. Your collective experience and expertise have enriched my thesis work.

I extend my sincere thanks to the faculty members Dr. Resmi Sebastian, Dr. Mitesh Surana, Dr. Rahul T M, Dr. Raheena, Dr. Ravi Ganesh and Dr. Ratan Sarma for their intellectual contributions and support.

I would also like to thank junior lab assistants Mr. Vipin Kumar, Mr. Arpit Gupta and project staffs Mr. Ashish Kumar for their valuable support during essential times during the journey. I am thankful to the peoples of my research family Dr. Veena, Dr. Panna Lal Kurmi, Dr. Ankit Tyagi, Mr. Kaipa Kiran Kumar Reddy, Mr. Sachchidanand Kushwaha, Mr. Zimmy Singh Naorem, Mr. Onkar Mishra, Mr. Aravind K Suresh, Mrs. Aarthi Shylu, Mr. Hibretu Kaske Kasa and Mr. Shubham Sharma for their collaboration and engagement along with countless coffee breaks, late night conversations, moments of laughter. Your diverse perspectives and constructive feedback have contributed a lot to my research ideas.

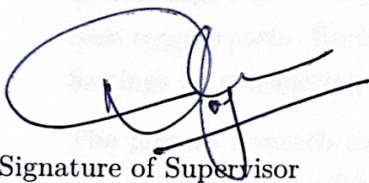
I am indebted to the friends Dr. Supratim Guha, Mr. Manoj Kumar, Mr. Jaswant Singh, Dr. Aditya Bhammidipati, Mr. Niraj Janardhan Sahare, Dr. Neha Gupta, Ms. Sakshi Rohilla, Mr. Amit Kumar, Ms. Deepali Gaikwad, Mr. Dharmil Baldev, Mr. K S Prasath for their companion and for lifting my spirits when the path seemed long.

The journey of research may not be possible without the immense support of my family of friends at long distance Mr. C Saravanakumar, Mr. C Karthikeyan, Dr. Peddireddy Srikanthreddy, Mr. D Karthikeyan, Mr. Bharanidharan and Dr. D. Nigitha.

To my beloved family Mr. Rajamanickam, Mrs. Nagavalli, Mr. Gowri Shankar, Mrs. Sridevi Gowri Shankar, Mrs. Saranyadevi, Mrs. Thilagavathi. I also extend sincere gratitude to my other family members and friends who have always supported and encouraged me throughout this journey.

Certificate

This is to certify that the thesis entitled **Enhancing Seismic Resilience of RC Building Through Foundation Rocking**, submitted by **Manoj Kannan R (2017CEZ0002)** for the award of the degree of **Doctor of Philosophy** of Indian Institute of Technology Ropar, is a record of bonafide research work carried out under our guidance and supervision. To the best of my knowledge and belief, the work presented in this thesis is original and has not been submitted, either in part or full, for the award of any other degree, diploma, fellowship, associateship or similar title of any university or institution. In our opinion, the thesis has reached the standard fulfilling the requirements of the regulations relating to the Degree.



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Abstract

The incorporation of rocking foundation is remarkably effective construction alternative for safeguarding the superstructure from significant damage caused by severe lateral forces during intense earthquakes. Even though, the mechanism of rocking foundations and its beneficial effects are widely documented, generalized design guidelines and its applicability on the Reinforced Concrete (RC) framed buildings are yet to be explored. Hence, The scope of this thesis work aims to to quantify the seismic force and displacement demands for RC frame buildings on the explicit and combined effects of the rocking foundation and superstructure behaviour with respect to key parameters addressing the seismic force and displacement demands. The objective of this study is to demonstrate the advantageous impact of using rocking foundations on the seismic performance of Reinforced Concrete (RC) framed building. This will be accomplished by comparing the performance of buildings that have conventionally designed foundations, rocking foundations and fixed base counterparts. Rocking at the foundation level is achieved by under proportioning the footings by considering the reduced earthquake loads for footing design.

The present research comprises of two distinct objectives, where for RC framed building without shear wall solely the supporting foundations are allowed for rocking. However, for RC building with shear wall solely the foundation supporting shear wall is allowed to rock. Within the OpenSees framework superstructural elements are model as fiber-based modelling with distributed plasticity whereas substructural elements and soil are modelled using Beam on Nonlinear Winkler Foundation (BNWF) modelling. The observations made from eigen analysis indicates the period lengthening for the both structural configurations considered. For RC framed buildings without shear wall, nonlinear static pushover assessments showed that permitting the foundation rocking increases yield and peak displacement by about 9% to 34% without substantial reduction in the strength. Also, the plastic displacement capacity increases as the rocking effect increases. This shows that rocking the foundations in a structure is advantageous for its overall seismic performance.

According to the nonlinear dynamic time history analyses, seismic moment transferred from the column to the foundation decreases by 20% to 50%. Due to reduction in the peak roof acceleration and increasing settlement at the base of the foundation with increasing effect of rocking, reduced seismic moment is noticed at the base of the structural members. Similar responses are noticed for the buildings where only the shear wall foundation is allowed to rock. It is found that the foundation of a shear wall can be designed by taking into account 40% of the earthquake loads for zone V design level and 60% of the loads for zone II design level without encountering excessive settlements beyond permissible limits as per Indian standards. From the hysteric responses for the shear wall foundation rocking, it is evident for very strong impact seismic motion, conventionally designed footings tends to experience higher flexural displacement along with higher seismic force demands and settlement demands too. This suggests that an oversized footing may not always be beneficial for the superstructure. From the fragility

assessment it is observed that the probability of exceeding 25mm settlement increases with increase in foundation rocking regardless of soil type. However, the probability at the collapse prevention level of 60mm is not considerably influenced for the foundation proportions while transitioning from conventional footings to moderate rocking footings. This implies that reducing the dimensions of the foundation may not necessarily result in reasonable settlement limitations being exceeded. The most favourable conditions for foundation rocking is observed to be dense and very dense sand than medium dense sand.

Keywords: rocking foundation; seismic performance; Beam on Nonlinear Winkler Foundation; shear wall; nonlinear seismic analysis; fragility assessment;

List of Publications

Journal

1. **Kannan, R. M.**, James, N., & Haldar, P. (2024). Seismic Response of Rocking Shallow Foundation in RC Framed Structure: A Parametric Study. *Journal of Vibration Engineering & Technologies*, 1-14.
2. **Kannan, R. M.**, Haldar, P., & James, N. (2024). Isolated shallow rocking foundation on different soils with varying embedment depth. *Disaster Prevention and Resilience*, vol. 3, no.1: 1.
3. **Kannan, R. M.**, Haldar, P., & James, N. (under review). Assessment of Shallow Rocking Foundation on RC Shear Wall Frame Structures: A Numerical Study. *Bulletin of Earthquake Engineering*. (Submitted on September 12, 2023)
4. **Kannan, R. M.**, Haldar, P., & James, N. (under review). Impact of Isolated Shallow Rocking Foundation on Fragility of RC Shear Wall Structure. *Structures*. (Submitted on August 14, 2023)
5. Reddy, K.K.K., **Kannan, R. M.**, James, N., & Haldar, P. (Under Preparation). Efficacy of Foundation Rocking on Seismic Performance of RC Shear Wall Building. *Engineering structures*.
6. **Kannan, R. M.**, James, N., & Haldar, P. (Under Preparation). Effect on Seismic Design Levels on Rocking Shallow Foundation Supporting RC Moment Resisting Frames. *Journal of Building engineering*

Book chapter

1. **Kannan, R. M.**, James, N., & Haldar, P. (2024). Seismic Response of RC-Framed Structures on Shallow Rocking Foundation. In: Sassi, S., Biswas, P., Naprstek, J., (eds) *Proceedings of the 15th International Conference on Vibration Problems*. ICOVP 2023. Lecture Notes in Mechanical Engineering. Springer, Singapore.

Conference Proceeding

1. **Kannan, R. M.**, James, N., & Haldar, P. (2024). Influence of foundation rocking and site conditions on seismic performance of RC shear wall frame structures. 6th *Eurasian Conference on OpenSees*, 24-25 July 2024, Beijing, China.
2. James, N., **Kannan, R. M.**, & Haldar, P. (2022). Effect of Soil Structure Interaction on Collapse Behavior of RC Buildings with Shear Wall. *Two days' symposium on SocioTechnological Aspects of Seismic Disaster and Mitigation (STASDM)*, June 23-24, 2022, IIT Guwahati, India.

3. **Kannan, R. M.**, Haldar, P., & James, N. (2021). Effect of soil structure Interaction on RC buildings with varying shear wall placement. *Proceedings of the 6th International Conference on Earthquake Engineering and Seismology (6ICEES)*, 13-15 October 2021, Turkey.
4. **Kannan, R. M.**, James, N., & Haldar, P. (2020). Influence of Soil Types on Seismic Behaviour of RC Framed Building with Shear Wall on Rocking Foundation. *17th World Conference on Earthquake Engineering*, September 13- 18, 2020, Sendai, Japan. 4c-002.
5. **Kannan, R. M.**, Haldar, P., & James, N. (2020). Behaviour of Mid-Rise Buildings with Shear Wall Rocking Foundation System on Medium Dense Site. *Proceedings of the 2nd ASCE India Conference on “Challenges of Resilient and Sustainable Infrastructure Development in Emerging Economies (CRSIDE2020)*, 2-4 March, 2020. Pg 1366-1371.
6. **Kannan, R. M.**, James, N., & Haldar, P. (Accepted). Effect of Shallow Rocking Foundation on Seismic Energy Dissipation Capacity and Self-Centering Ability of RC Framed Buildings. *8th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, 11-14 December 2024, IIT Guwahati, India.

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List of Symbols

α_m	Modal mass coefficient
β_{Ds_a}	Demand uncertainty
β_{DS}	Standard deviation for damage state
β_M	Modelling uncertainty
β_{RTR}	Uncertainty in record-to-record variability
β_T	Total uncertainty
δ	Angle of friction between footing element and supporting soil
Δ_{roof}	Roof displacement
ϵ_{50u}	Post peak strain at 50%
ϵ_{cr}	Average tensile strain of concrete in tension
ϵ_c	Strain in concrete
$\frac{\lambda_D}{S_a}$	Natural logarithm of calculated median displacement demand for average spectral acceleration
Γ	Modal mass participation factor
γ	Unit weight of the soil
λ_c	Natural logarithm of median displacement capacity for various limit states considered in this study
$\overline{S_{d,DS}}$	Median spectral displacement for the damage state
ϕ_i	Modal shape coefficient at the i^{th} floor level
ρ_s	Ratio of volume of transverse reinforcement to volume of concrete core
A_f	Surface area of footing
b''	Width of the concrete core measured outside the hoops
c	Parameters that controls the shape of the post yielding portion in the backbone curve
C_r	Parameter which controls the range of elastic portion
D_f	Embedment depth of footing
E_c	Young's modulus of concrete

E_h	Post yielding stiffness of reinforcing steel
E_s	Young's modulus of reinforcing steel
E_t	Young's modulus of concrete in tension
f'_c	Cylinder strength of concrete
f_{cr}	Tensile cracking strength of concrete
f_c	Compressive stress in concrete
f_{yh}	Yield strength of transverse reinforcement steel
h	Total height of the structure measured from ground level
K	Scalar factor to increase the unconfined concrete strength
k_{in}	Initial tangent stiffness
K_p	Passive earth pressure coefficient
n	Parameters that controls the shape of the post yielding portion in the backbone curve
p_{ult}	Passive earth pressure per unit length of footing
q	Load applied from foundation to soil
q_0	Load at yield point
q_{ult}	Ultimate load that the soil can carry
r	parameters controlling the shape of stress strain curve
S_a	Spectral acceleration
S_{du}	Spectral displacement at ultimate
S_{dy}	Spectral displacement at yield
S_d	Spectral displacement
s_h	Spacing of the hoop reinforcement
T_a	Natural fundamental period for fixed base structure as per (IS1893, 2016)
t_{ult}	Frictional resistance per unit area
V_b	Base shear of the structure from pushover analysis
W	Seismic weight of the structure
W_g	Weight acting from the super structure to the foundation

W_i	Storey weight at the floor i^{th} level
z	Instantaneous displacement due to vertically applied load
z_0	Displacement at yield load
z_{50}	Mobilized load at 50

1.1 Background

In India nearly 31.6% of the total population is living in the urban areas and it is expected to increase up to 40% by 2030 ([Ramancharla et al., 2019](#)). People from rural areas are migrating in large numbers to urban areas because it offers numerous employment opportunities and most importantly, enhanced infrastructure facilities like hospitals, educational institutions etc., are easily accessible. As a result of this rapid urbanization, availability of land for horizontal expansion is limited for occupancy. Hence, more multi storey structures are coming into existence in India. On the other hand, most of the urban areas are seismically vulnerable, since 56% of the area in India are vulnerable to moderate to severe earthquakes ([Ramancharla et al., 2019](#)). India has also witnessed numerous devastating earthquakes like Uttarkashi (1991), Khillari (1993), Jabalpur (1997), Chamoli (1991), Bhuj (2001), Sumatra (2004), Kashmir (2005), and Sikkim (2011) during the past three decades. These major earthquakes have resulted more than 40,000 casualties and numerous structural collapse.

Devastating consequences of earthquakes in terms of structural collapse and increased number of casualties are also noticed across the borders of India. The prominent spatial distribution of seismic events includes Northridge (1994) ([Trifunac et al., 1998](#)), Kobe (1995) ([Nakamura, 1996](#)), kocaeli (1999) ([Barka, 1999](#)), Bolu (1999) ([Dönmez and Pujol, 2005](#)), L'aquilam (2009) ([Indirli, 2010](#)), Christchurch (2010) ([Kam and Pampanin, 2011](#)), and Chile (2010) ([Wallace et al., 2012](#)). It is highlighted from the catastrophic events that the earthquakes do not threaten human; rather, structural failure increases the number of casualties. These seismic tragedies experienced across the globe accelerate the importance of earthquake resistant design of structures to resist the seismic induced lateral forces.

Extensive research on earthquake-resistant design of superstructure has been developed over the last few decades to protect the superstructure from failure and to ensure human life safety. Reinforced Concrete (RC) buildings with Moment Resisting Frames (MRF) are widely used for construction practices to resist the seismic forces ([Yön et al., 2017](#)). In addition to the moment resisting frames, shear walls are frequently incorporated as additional lateral force resisting member in RC framed buildings to enhance structural rigidity and resist lateral loads imparted by seismic motions ([Çavdar et al., 2018](#); [Wallace, 1994](#)). In order to control the seismic induced damages and structural collapse, super structural elements are typically designed to respond to earthquake-induced lateral loads which may lead to the formation of plastic hinge at the base of the super structural elements i/e., column, beam and wall elements. As a result, significant damage is expected to occur at the plastic hinge region while the other portions remains reasonably undamaged ([Kam and](#)

Pampanin, 2011; Wallace et al., 2012; Barka, 1999; Trifunac et al., 1998; Nakamura, 1996; Dönmez and Pujol, 2005; Indirli, 2010). Concrete crushing, longitudinal reinforcement yielding, and concrete cracking are acknowledged as common mechanisms for energy dissipation documented from previous earthquake occurrences as shown in Figure 1.1. Due to these characteristics and considering the rigidity of the frames and shear wall, there is an increased risk of damage and associated increase in retrofitting complexity post seismic events. Furthermore, if the shear wall or column fails during an earthquake event, the additional lateral forces will be unevenly distributed to surrounding frame members which may cause substantial damage to the superstructure.



(a)



(b)



(c)



(d)

Figure 1.1: RC super structural failure during various earthquake events; (a) compression zone failure of shear wall buildings damaged during the 2011 Christchurch earthquake (Kam and Pampanin, 2011); (b) shear wall damage in the 2010 Chile earthquake (Wallace et al., 2012); (c) column failure during 1999 Izmit earthquake (Barka, 1999); (d) structural collapse during Northridge earthquake (Trifunac et al., 1998)

During the conventional structural design approach as mentioned above the foundation is generally assumed to be fixed, while all the super structural elements are designed

to carry the earthquake induced forces and moments. As a consequence of this; 1) the foundations are oversized to comprehend the fixed base behaviour and 2) plastic hinging will occur at the super structural elements during severe seismic attack (Tomeo et al., 2018). In other words, non-linear responses of the soil and foundation is not permitted. However, techniques like base isolation and providing dampers exist, to decouple the foundation response and structural response and to absorb the induced seismic energy, respectively. (Barbat and Bozzo, 1997; Vratsikidis and Pitilakis, 2023; Yenidogan, 2021; Hamidi et al., 2003; Hamidi and El Naggar, 2007). These methods acts as a fuse mechanism and enhances self centring ability of the super structural elements with reduced ductility demands. However these methods are found to have lots of limitations and are not cost-effective in terms of construction and maintenance. Several researchers (Gelagoti et al., 2012; Gajan and Kutter, 2008a; Gavras et al., 2020) discovered that the soil foundation system has the capability to mitigate the seismic force demands imparted to the structure via foundation rocking and sliding (by geometric nonlinearity) and bearing capacity mobilization (via material nonlinearity). To achieve the nonlinearity in soil footing interface, footings are often under proportioned by reducing its bearing area to achieve the rocking phenomenon is thus the target area of this research work.

1.2 Mechanism and attributes of rocking foundation

The mechanism of rocking foundation that is represented in Figure 1.2 illustrates that there is a possibility of uplift on one side of the footing during seismic shaking (Gelagoti et al., 2012; Gajan and Kutter, 2008a; Gavras et al., 2020). This uplift on one side of the footing transmits larger loads on the soil medium on the opposite side of the foundation. As a direct result of this, soil yielding may take place, which would result in the footing becoming plastically settled. On the other hand, foundation uplift causes a larger dissipation of energy and prevents the super structural elements from yielding when there is a seismic force applied to the structure. The ideas of soil yielding and footing uplifting have made way for a novel approach to improve the overall seismic performance of the structure.

Milne (1886), was one of the earliest researchers who developed the overturning of rectangular column for measuring the intensity of ground motion. Later Housner (1963) developed the mechanics for modern studies dealing with rocking mechanisms. The beneficial effects of the rocking motion are first observed by Housner (1963) on tall slender structures like monuments, water tankss etc., survived during Arvin-Tehachapi, 1952 and Chilean, 1960 earthquakes, where freely rocking response at the foundation level are observed. Many researchers (Gelagoti et al., 2012; Gajan and Kutter, 2008a; Gavras et al., 2020; Burnwal and Raychowdhury, 2023; Liu and Hutchinson, 2018; Raychowdhury and Hutchinson, 2009; Raychowdhury, 2011; Deng et al., 2012a; Antonellis and Panagiotou, 2014; Sharma and Deng, 2019, 2020, 2021; Sharma et al., 2022), inspired by Housner's work, used this concept in various diverse structures typologies of practical significance, and highlighted that rocking shallow foundations possess following beneficial attributes:

Seismic resilience: Rocking foundations are generally allowed to uplift during severe earthquakes, this uplift offers separation between the foundation and the supporting soil. This separation acts as an isolation mechanism results in reduced ductility demands for superstructure.

Enhanced energy dissipation: Rocking of footing enhances the nonlinear behaviour in the foundation soil interface as a direct result of this nonlinear foundation moment rotation capacity will be noticed due to rocking and results in an enhanced energy dissipation capacity at the foundation soil interface.

Re-centering ability: Rocking will be noticed at the edge of footing, which is away from the center of gravity, as result of this gravitational restoration will takes place due to the self weight of the super structural members.

Cost efficient: Due to the reduced foundation sizes, rocking foundations reduces the construction and maintenance cost. In addition to that rocking foundation neither require any additional component to dissipate the energy induced nor to render re-centering behaviour.

Preserving the strength: In case of conventionally designed footings, superstructure experiences higher seismic forces and moments which lead to significant strength reduction. On the other hand, the strength of the superstructure will be preserved for the rocking foundation resting on competent soil.

1.3 Motivation

Earthquakes are one of the most commonly encountered natural hazard across the world which triggers extensive structural damages. As a consequence of this, seismic behaviour of

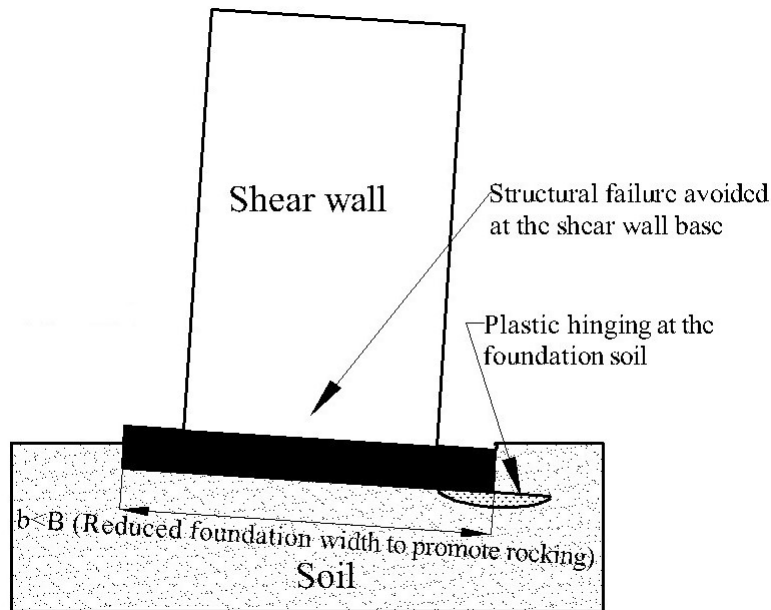


Figure 1.2: Mechanism of rocking footings

RC building members and shear wall for resisting the lateral forces during seismic events are well established from the past seismic events. Therefore, well defined seismic resistant design procedures are available across the world for super structural design. However, failure of such designed buildings are documented from the catastrophic seismic events. In contrast, soil is anisotropic in nature and its properties changes abruptly for different locations. Hence, the conventional design procedures for foundation design ignores this complexity by incorporating numerous over strength factors. Despite the fact that understanding the complex behaviour of soil has evolved over past few decades, the over strength factor for foundation design still persists among the codal regulations and practising engineers.

Incorporating the positive attributes of rocking foundation provides a simplest method to improve the overall seismic performance of the structure. Provision of rocking foundation also stimulates the optimization in the super structural and sub structural design as both are prone to significant non linear responses. Although mechanism of rocking foundation is well established, generalized design guidelines is yet to be developed. Hence, the motivation of this thesis is to offer new perspective on the rocking foundation and their effects when incorporated in multi-storey RC structures with and without shear wall, so that the concepts of rocking foundation are understood deeply and can be implemented as a part of earthquake resistant design.

1.4 Organization of thesis

The purpose of this thesis is to evaluate the advantages of a rocking foundation with different levels of rocking over conventionally designed footings for RC framed buildings with and without shear walls. The scope entails methodically adjusting the footing dimensions which is one of the crucial factor that controls the seismic performance of the structure. The focus of this thesis study is to specifically quantify the demands of seismic force and displacement on the explicit and combined effects of the behavior of the rocking foundation and superstructure with regard to important factors. Finding the beneficial effects of rocking foundation system for generic symmetrical structures under generalized soil conditions is the major goal of this study. In this context the thesis work has been organized as mentioned below:

Chapter 1 provides the background, motivation and mechanism of the rocking foundation. Along with that generalized scope is also provided.

Chapter 2 provides the discussion about the evolution of seismic resistant design and applicability of rocking foundation as one of the effective solution to improve the overall seismic performance of the structure through past studies performed. At the end of this chapter potential research gaps, scope and objectives of this present thesis work is provided.

Chapter 3 discusses about the different sets of RC frame buildings considered for this present work and their design parameters in detail. Subsequently, modelling techniques adopted for various structural members, soil types and foundation are also discussed.

In addition to that the developed numerical model is validated with the help of the experimental results from the past available literature.

Chapter 4 highlights the beneficial effects of foundation rocking by performing the parametric studies RC bare framed structures. The seismic performance parameters such strength, displacement at yield and peak, plastic displacement capacity and ductility are studied. Even though the above mentioned parameters provides the efficacy of foundation rocking it cannot able to capture the complexities due to the dynamic nature of earthquakes. Hence, seismic responses for rocking foundation in RC bare framed structure in terms of moment capacity, settlement are quantified. For Chapter 4 concentrates on the different height of the structure, soil types and foundation types resting at constant embedment depth.

Chapter 5 concentrates on the effect of embedment depth on the seismic response of rocking shallow foundations in RC framed building without shear walls and their seismic performances are compared in terms of seismic force and displacement demands.

Chapter 6 highlights the importance of incorporating the rocking foundation for the shear wall in RC frame buildings. To achieve this objective, parametric studies are conducted for different structural heights designed for various seismicity resting on soil having varying angle of internal friction. Seismic responses are quantified in terms of seismic force and displacement demands on both superstructure and substructure.

Chapter 7 aims to develop the fragility curves considering various damage states for rocking foundations for the buildings considered in above chapters i.e., RC frame buildings with and without shear wall. In addition to that element level responses on seismic response of rocking shallow foundations in RC framed building with shear wall are captured using the moment-curvature, moment-foundation rotation, foundation rotation-foundation settlement and shear-foundation sliding relationships.

Chapter 8 Summarizes the results obtained and also highlights the necessity for future work.

2.1 Overview

With the rapid development of the economy and the population explosion in metropolitan and cosmopolitan cities led to the gradual rise in the construction of multistorey structures due to the lack of space as mentioned in section 1.1. These cities need additional structures to meet the futuristic demands to accommodate the rising population growth. In most of the cases structures are generally supported by shallow foundation because it is cost effective and easy to construct than the deep foundations. Also, the structures are built close to each other. Earthquakes often impart additional challenges in the design of civil engineering structures. The historical development in the seismic-resistant design of structures includes incorporating high-strength materials for construction practices and high-performance structural elements (Goel et al., 2010). Generally, the structures are intentionally designed to dissipate the seismic energy through the nonlinear behavior of structural elements by adopting the capacity design concept. On the hand, soil-foundation has the potential to dissipate the induced seismic energy through foundation uplifting also termed as foundation rocking. The detailed literature review on the experimental and numerical studies on the rocking foundation are presented in this chapter.

2.2 Evolution of earthquake resistant design

Even though the historical earthquakes such as 1755 great Lisbon earthquake, 1862 Naples earthquake and 1897 Assam earthquake were reported as per Lawson and Reid (1908), San Francisco earthquake (Reid, 1910) was well documented and the records of deficiencies in structural system is reported extensively. Since then, continuous efforts are made towards the development of seismic resistant design or earthquake resistant design. 20th century marks the establishment for modern day seismic resistant designs. The basic philosophy behind the seismic resistant design lies on the capacity of the structure and the demand created by the seismic actions to the structure, where, the capacity of designed structures should not exceed the demand (MacRae et al., 2011). In general code provisions defines the capacity in terms of forces and stresses for e.g., applied lateral force due to the seismic activity should not exceed the lateral force capacity of the structure. In addition to that the design should satisfy both ultimate and serviceable design states as per (Code, 1997) and (Diebold et al., 2008). This philosophy of design is termed as capacity based design. The capacity based design methodologies are framed from the past experiences and it changes throughout the world. Hence, the capacity based design is subjective. However damages are still observed and sometimes structural failure also noticed.

After the 1994 Northridge earthquake, Structural Engineers Association of California (SEAOC) (Diebold et al., 2008) came with a new procedure to counteract the drawbacks of the capacity based design. Consequently this led a way for developing a new design concept based on the performance of the structure in conjunction with the capacity based design for achieving better sustainability. In performance based design framework each structures comes with specific performance objectives. The performance objectives are derived by stating four performance levels namely: fully operational, operational, life safety, collapse prevention. These objectives and levels are vbased on the loss of occupancy and cost of repair. These performance levels are based on the inter-storey drifts and the limits are found based on the deterministic approach. The Pacific Earthquake Engineering Research (PEER) centre developed an integral hazard level approach considering the shortcomings of the previous procedures. The PEER performance-based earthquake engineering (PBEE) methodology has been summarised in the literature (Cordova et al., 2000; Günay and Mosalam, 2013). The PEER PBEE methodology allows explicit calculation of system performance measures in terms of structural downtime, monetary losses, casualties etc. The key difference from the present PBEE methodology to the earlier procedures is the evaluation of seismic performance in a probabilistic framework. The uncertainties in the intensity of an earthquake, ground motion parameters, structural behaviour, physical damage and losses are considered explicitly in the PEER PBEE framework (Lee and Mosalam, 2006). The schematic representation of the PBSO procedure is depicted in Figure 2.1.

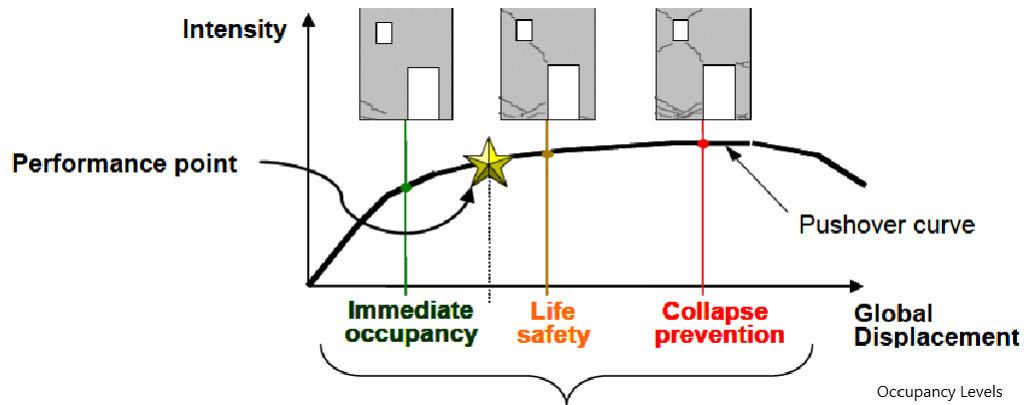


Figure 2.1: Performance levels as per FEMA356 (2000)

In the recent decades performance design is gaining popularity among the geotechnical engineers as well, since, it provides better design optimization. But performance based design requires requires a realistic modelling approach and large amount data concentrated on different structural and sub structural combinations. Hence it is important to conduct numerous deterministic and probabilistic analysis on various structural combination before adopting it to design guidelines. Rocking foundation can be considered as one of the suitable performance based design technique that can be adopted in the structures to balance the design between the superstructure and substructure.

In addition to that, Damage Avoidance Design (DAD) philosophy (Hamid and Mander,

2014) also emphasizes the similar objectives like PBEE philosophy (Cordova et al., 2000; Günay and Mosalam, 2013). The objectives of these design approaches are to minimize damage, economic loss, prevent collapse, and ensure life safety of the structures under Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) (Hamid and Mander, 2014; Khanmohammadi and Heydari, 2015; Hamid and Mander, 2012). To achieve these objectives, seismic energy dissipation devices and structural fuses are usually integrated into the structural elements. All the methods as mentioned earlier, majorly concentrates on the superstructure. On the other hand, plastic deformation in the shallow foundation element and underlying soil is not allowed. As opposed to the conventional design philosophy followed for shallow foundation, relatively new approach termed as rocking foundation which allows the foundation to uplift back and forth has been studied extensively. In this regard, foundations are purposefully under designed, typically by reducing the bearing area of the footing so that they can take advantageous effects of rocking and protect the super structural elements from failing.

Based on the aim of sustainable development during urban regeneration, we are increasingly concerned about whether a structure can be used effectively and sustainably. In this regards rocking foundation can be considered as a viable method to reduce the environmental impact. Although extensive research has been made on the rocking foundation, yet practical guidelines and implications are not well established. In this chapter, some of the important technical evolutions happened over the decades were discussed along with the future scope in the field of shallow rocking foundation.

2.3 Background

Milne (1886) was one of the earliest researchers who utilized the overturning of rectangular column for measuring the intensity of ground motion. Milne's study laid the foundation for the rocking science mechanism. However, Housner (1963) observed from the catastrophic earthquake events that the tall slender structures are more stable during the earthquake motion. Housner also observed the ability of these tall and slender structures to rock on their footings, which enhanced energy dissipation capacity, and Beck and Skinner (1973) employed this concept of rocking on the bridge pier structure. In order to achieve the rocking phenomenon, they employed an energy dissipating device which offers a vertical separation between the pier and the supporting foundation. This system was successfully applied for the construction of railway bridge pier in New Zealand. Later Priestley et al. (1978) analysed the shear wall framed structures by allowing the core to uplift freely at footing level with the provision of dampers and observed that allowing uplift greatly reduces the base shear and moments during seismic excitations. However, the studies mentioned above concentrates on the altering the formation of plastic hinges at the base of the structural members. The concept of uplift, yield and rocking at the foundation level due to moment rotation behaviour was first initiated by Taylor and Williams (1979) in early 1980s. Laboratory tests have been conducted on the model footing on clay and sand.

They observed that the seismic response of the foundation depends on the overturning moment. If the overturning moment due to seismic action is relatively small then foundation will be completely in contact with the soil. With the increase in overturning moment the foundation may uplift and loses its contact progressively to utilize the nonlinear soil responses. Their studies have shown that foundation can be intentionally under-designed to make use of the uplift as energy dissipating mechanism due to overturning moment during high-intensity earthquakes.

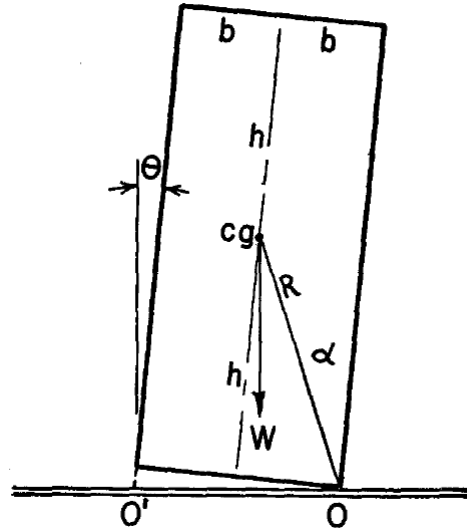


Figure 2.2: Rigid block considered by [Housner \(1963\)](#)

The utilization of rocking behaviour, either in the structural components of a structure or its foundations, has been identified as a promising design approach to mitigate damage caused by earthquakes. So far, researchers around the world have conducted many experiments and computational analyses to study how rocking affects the ability of a system to withstand earthquake forces. Fuse action may occur between the superstructure and on rigid base termed as rocking structures, or between the foundation and the superstructure of a system termed as foundation rocking structures, if the aforementioned categories are combined.

2.4 Experimental studies on rocking shallow foundation

The very first dynamic shaking table test was carried out by [Taylor and Crewe \(1996\)](#) on a concrete rocking base. At the shake table, a large flexible shear stack was created to imitate soil boundary conditions and provide genuine inelastic soil reaction. This was done in order to produce the desired results. This constructed stack was filled with sand, and a shallow foundation measuring 0.4 m by 0.4 m was built inside of it. The embedment was found to be 0.1 m. At the bottom of the shake table, an earthquake motion was applied, and the peak ground acceleration (PGA) was measured to be 1.23 g. The findings of the tests indicate that this foundation has experienced a large and permanent settlement.

Within the scope of the TRISEE project, one of the initial large-scale shaking table experiments was carried out to examine the rocking characteristics of shallow square footings (m^2) subjected to under lateral loading was conducted by Negro et al. (2000). In plan view, the foundation was measured to be 1 meter by 1 meter, and it was supported by Ticino sand that was saturated and spread uniformly (Figure 2.3). Both loose and dense sand, with relative densities (D_r) of 45% and 85%, respectively, were taken into consideration in this program. Both types of soil conditions were distinct from one another. The top of the foundation was subjected to a cyclic horizontal force in order to simulate the inertial force that was transferred from the superstructure when the structure was submitted to seismic loading. According to the findings of the experiment, the rocking foundation exhibits a moment-rotation hysteretic reaction that is rather broad when it is resting on low D_r sand. However, by the time the test program is over, it has accumulated a large permanent settlement and rotation. Observations of much smaller peak and permanent settlements are made by the foundation that was established on a high D_r sand.

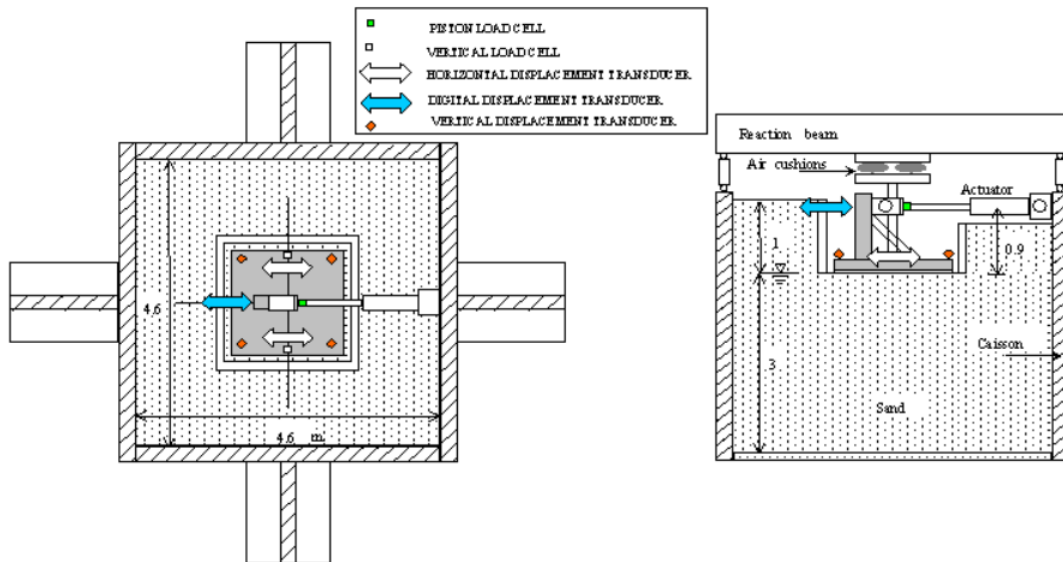


Figure 2.3: Experimental set up for large shake table test conducted by Negro et al. (2000)

Since 2000, the University of California, Davis (UCD) has had an experiment program using centrifuge testing to look into how the soil foundation capacity can be mobilized. This lets the footing rock, slide, and settle in nonlinear ways (Martin et al., 2002; Gajan et al., 2005; Gajan and Kutter, 2008b). These experiments looked at a number of shear wall-footing models, usually at an acceleration of 20 g. For each model, the footing dimensions, depth of embedment, and starting static FSv were all different. Also, different types of soil were looked at, such as cohesive (clayey) and cohesionless (medium to dense sandy soil) geotechnical variations. Usually, two types of loading were used on the model specimens: a slow cyclic load on the side and dynamic base excitement. Gajan and Kutter (2008b) carefully looked at and summed up how the foundations responded to moment-rotation and settlement-rotation in these test programs. Gajan and Kutter (2008b) defined FSv as A/A_c , where A is the footing area and A_c is the minimum area needed to support the

vertical load. One important thing that these tests showed was that footings with a high FSv show a small permanent settlement. The previous re-centering potential of the shallow rocking foundation was also reliably proven by these tests. This is because the axial load helps close the gap between the footing and soil when the load is removed. In addition, all rocking footings show a good amount of energy loss due to increased moment-rotation responses at the foundation level.

[Chang et al. \(2007\)](#) created and examined two reduced sizes of building models, specifically a one-bay and two-bay model. These models were evaluated in a centrifuge environment with a force of 20-g. The models were supported by shallow footings placed on top of dry, compact sand. The building models incorporated tailored ductile fuse components, strategically positioned at the extremities of the beam elements, to accurately simulate the nonlinear inelastic characteristics of the actual structure. During dynamic shaking and displacement-controlled cycle loading, it was continuously found that the rocking footings dissipate more than 65% of the total energy, even when subjected to large drift demands above 2%.

A series of shaking table tests were performed by [Antonellis et al. \(2014\)](#) on a virtually full-scale bridge pier supported by a $1.5m^2$ rocking shallow foundation. This initiative was supported by the California Department of Transportation (Caltrans) in order to establish design guidelines for bridges supported by piers whose foundations experience lateral movement. Shaking table tests revealed that extremely under-designed rocking foundations with minimal residual drift ratios and settlement without structural damage withstood extremely powerful seismic trembling. The incremental drift ratios during the design earthquake (DE) and maximum considered earthquake (MCE) levels of excitation were 0.1% and 0.3%, respectively.

[Anastasopoulos et al. \(2015\)](#) examined the seismic behavior of a pre-existing three-storey structure that was retrofitted by adding shear walls by undertaking a series of reduced scale shaking table experiments. Although the rocking-isolated system performs quite well, residual foundation rotations are not always significant. [Tsatsis and Anastasopoulos \(2015\)](#) and [Chiou et al. \(2018\)](#) performed shaking table experiments on a model consisting of a column and footing that is mostly affected by rocking. The purpose was to examine how the rocking phenomenon influences the dynamic response of a structure subjected to seismic forces. It was discovered that the rocking reaction of the footing can mitigate the dynamic amplification impact of the model.

[Hung et al. \(2014\)](#) conducted pseudo-dynamic tests and slow cyclic testing on many bridge piers. The purpose was to investigate the impact of foundation rocking on the ductility demand placed on the bridge column. The pier was evaluated under two alternative base conditions: a fixed base and a rocking base. The fixed-base condition was established by securing the foundation to the inflexible floor using four tie-down rods, while the rocking-base condition was simulated by placing a neoprene pad layer beneath the foundation. The experimental results show that the rocking-base bridge exhibits

significantly greater deck drift compared to the fixed-base bridge. However, permitting the foundation to move back and forth creates a system of isolation, thus reducing the force exerted on the column base and ultimately safeguarding the column from collapse.

[Algie et al. \(2010\)](#), [Pender et al. \(2013\)](#) and [Phipps \(2013\)](#) conducted a series of field experiments on shallow foundations that were inserted into cohesive soil layers. The experimental setup employed in [Pender et al. \(2013\)](#) involved the stimulation of a structure using a vibrating device with an off-center mass. This device was installed on the top of the frame, which was supported by a shallow foundation designed to allow rocking motion. [Phipps \(2013\)](#) examined the behavior of shallow footings under three distinct loading conditions: free vibration, quasi-static cyclic loading, and dynamic forced vibration testing. However, the maximum rotation of the footing was significantly small, measuring less than 3%. Additionally, the initial factor of safety (FSv) was exceptionally high, reaching up to 5.4, as reported by [Algie et al. \(2010\)](#). [Phipps \(2013\)](#) conducted multiple experiments at a single FSv to investigate bearing failure. However, [Algie et al. \(2010\)](#) and [Phipps \(2013\)](#) did not analyze or describe the important measures of performance, such as the time intervals, ratio of recentering, amount of remaining settlement of the footing, and alterations in soil qualities. The findings from these studies indicate that rocking foundations exhibit a significantly nonlinear moment-rotation relationship and possess a clearly defined moment capacity. A minute fraction of the soil exhibited yielding as a result of the substantial factor of safety. The static equation for the moment capacity shown a strong correlation with the experimental results. The settlement that occurred during the intense seismic activity was negligible, which is promising for the future development of rocking foundations.

[Deng et al. \(2014\)](#); [Deng and Kutter \(2012\)](#) performed centrifuge tests on framed structure and bridge systems on rocking foundation to account for interactions between soil, footing and superstructure systems. The rocking foundation's characterisation summarises the rocking foundation's mechanism as 1) sand falling into gaps due to the uplift 2) Observed dilatancy beneath the footing 3) bearing failure on the contact areas. Series of test conducted on the bridge system implies that an adequately designed rocking foundation is less prone to overturn. Lesser settlements were noticed on the good soil conditions. Later they selected six different two-story two bare frame wall system (Figure 2.4) to study the response of rocking footing. Slow cyclic centrifuge tests have been performed. From the tests, it has been clearly shown that the load carrying capacity of the system did not descend even for the roof drift ratio of 3.5%. Their Companion paper presents the dynamic tests for the same set of structural systems. From the dynamic tests rocking dominated structures absorbs around 75% of energy during moderate and high intensity ground motions. Also, they suggested that transient drift demands due to foundation rocking can be recoverable.

Similar kind of studies by [Liu et al. \(2013, 2015\)](#) from 2013 to 2015 Suggests the following considerations: 1) structural fuses can be introduced at the superstructure level. Still, its capacity should not be greater than the rocking foundation to reduce structural demands.

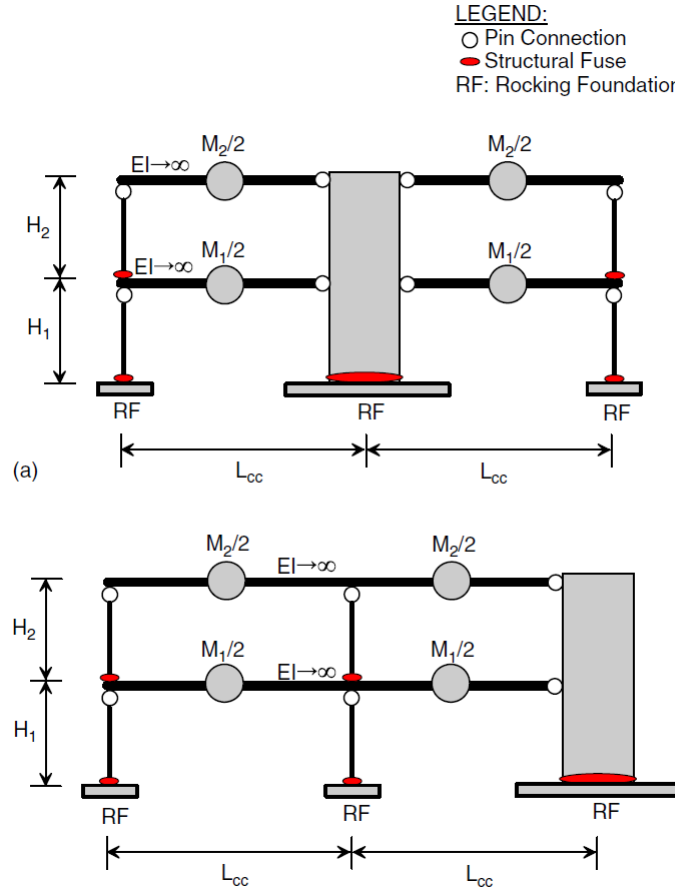


Figure 2.4: Model framed shear walled structure with Rocking foundation considered by [Deng et al. \(2014\)](#)

2) When the strength of structure and foundation are nearly equal, then the hysteresis energy is reasonably distributed between both components.

The studies on clayey soil ([Hakhamaneshi et al., 2012](#); [Hakhamaneshi and Kutter, 2016](#); [Hakhamaneshi et al., 2016](#)) shows that settlement is lesser for rocking foundations in clays. They carried out few centrifuge tests to investigate the mechanism involved in embedded footing system. Their centrifuge tests show that the uplift mechanism depends on sliding, dilation etc., than the settlement and bearing failure mechanisms as depth of foundation embedment increases. Also, their study mentions that footing being an integral part of structure during uplift creates additional demands on beams due to rocking nature of column elements.

[Heron et al. \(2014\)](#) used model Centrifuge tests and high-speed photography to track the rocking foundation movements precisely along with parametric analysis. Loose sand experienced higher settlements and lower rotations when compared with dense sand. Also, the rotation of footings is not linearly dependent on the acceleration of the input motion. There exists some threshold value above which the footing starts rocking below that value, no rocking was observed. The mechanism of failure for embedded shallow foundation is given by [Arabpanahan et al. \(2019\)](#), can be seen in Figure 2.5. Field studies on rocking

foundations are carried to demonstrate the beneficial effects and also to identify the failure modes (Sharma and Deng, 2019, 2020, 2021; Sharma et al., 2022).

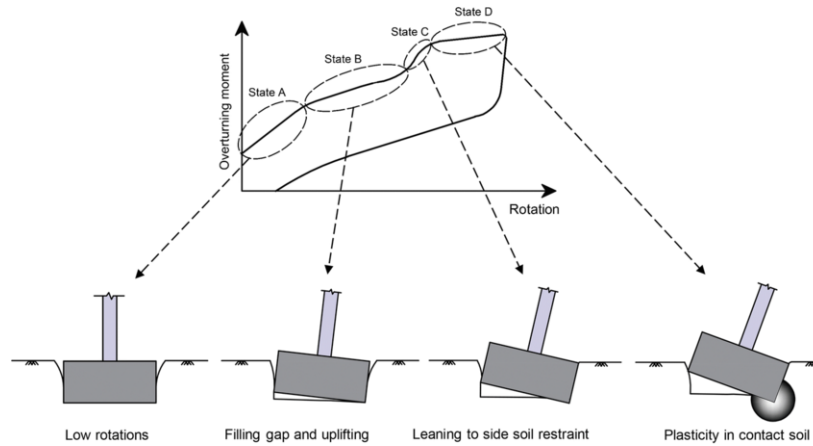


Figure 2.5: Mechanism of embedded shallow rocking foundation provided by Arabpanahan et al. (2019)

2.5 Numerical studies on rocking shallow foundation

Psycharis and Jennings (1983) considered soil structure interaction instead of having rigid base for analysing the response of rigid rocking blocks. He considered two spring models at both extreme ends of rectangular block and a distributed Spring model. His primary conclusion was that the two-spring model provides better response results and computationally simple. Succeeding that Yim and Chopra (1984a); Yim et al. (1980); Yim and Chopra (1984b) similarly had two different studies for evaluating single degree of freedom (SDOF) and multi degree of freedom (MDOF) oscillator response considering spring and dashpot systems (refer Figure 2.6). Their primary conclusions rely on the fact that there is a reduction in base shear during uplift for short period structures. In contrast, this reduction is more significant in case of long period structures. However, they have certain limitations in applying the same concept for the actual buildings due to the variation in embedment depth, excitation frequency, and foundation flexibility. Nakaki and Hart (1987) compared the response of fixed base and flexible system and noticed more extended period for flexible base systems.

Psycharis (1991, 1990); Psycharis et al. (2000) investigated the rocking response of SDOF System by considering various parameters such as amplitude of excitation, period of excitation slenderness ratio and damping. From his studies, he concluded that uplift is not necessarily beneficial to the structure, even it may be detrimental also. On the other hand, in some cases, the effect of uplift may not be significant. The impact of uplift majorly depends on the amplitude and period of excitation. However, his conclusions also shown that the uplift is more beneficial since it offers significantly higher damping. These conclusions opened a comprehensive gateway for exploring new dimensions in the area of

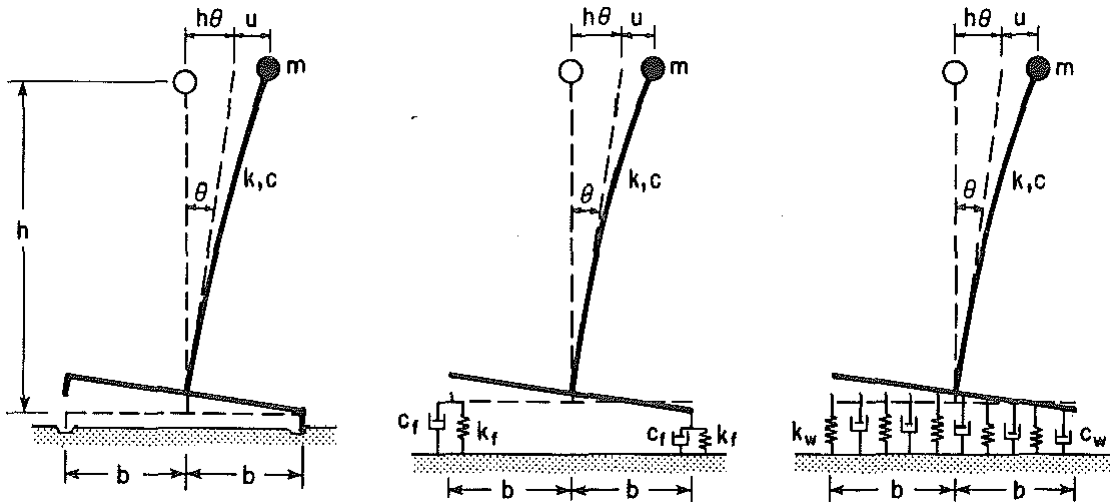


Figure 2.6: Rocking of columns having various flexibility at the foundation level [Yim and Chopra \(1984a\)](#)

rocking foundation studies. This study insists that there is a necessity to determine where uplift will be beneficial and where it has to be avoided.

[Filiatrault et al. \(1992\)](#) compared the effect of weak Foundation on the core wall type buildings with the fixed base type. They incorporated the nonlinear behaviour of footing, soil and structure. From his results, it has been clear that the weak foundation does not significantly influence the buildings' performance. Later, [Borja et al. \(1994\)](#) studied the response of one dimensional (1D) and three dimensional (3D) nonlinear Finite Element Analysis (FEA) on the rigid foundations by considering lateral, rocking and translational vibration modes. His conclusions highlighted the importance of considering the nonlinear behaviour of soil in dynamic responses. Several other studies ([Aslam et al., 1980](#); [Spanos and Koh, 1984](#); [Wong and Tso, 1989](#); [Andreus and Casini, 1999](#)) from the rocking rigid blocks states the block may not oscillate at zero mean amplitude continuously and coefficient of restitution is sensitive for different bases.

Later, [Xu and Spyarakos \(1996\)](#) selected the towers with foundation uplift. Their parameters mainly depend on the height of the tower and stiffness of supporting soil. Allowing foundation uplift reduces the seismic force response in hard soil and slender tower, whereas the response may substantially increase in hard soil and short towers. Their studies agree with the [Psycharis \(1991\)](#) studies in the view that uplift may not be beneficial in all cases. Also, this study has brought the new dimension that soil stiffness is one of the primary parameters to be considered its rocking foundation design.

In 2003 three different researchers parallelly brought various aspects of rocking foundation ([Allotey and El Naggar, 2003](#); [Anderson, 2003](#); [Gazetas et al., 2003](#)). [Allotey and El Naggar \(2003\)](#) concentrated on moment rotation response during foundation uplift. In contrast, [Anderson \(2003\)](#) studied the effects of force reduction factor which can be implemented on the foundation to have sustainable rocking. [Gazetas et al. \(2003\)](#) came up with case studies of Adapazari earthquake. From the studies of Anderson, the forces can be reduced

to the factor of 2 in shear wall structures to safely withstand the loads in case of static and dynamic conditions. However, Anderson studied mainly on the soils with higher bearing capacity and he suggested that foundation on weaker soil should be designed carefully. Gazetas summarizes his findings as 1) soft soils are more prone to have bearing capacity failure during foundation uplift 2) In dynamic conditions both the sides of footing deform for a shorter duration resulting in partial cancellation of deformation on opposite and hence there are many possibilities of survival. Allotey and Naggar developed analytical moment rotation curves shown in Figure 2.7 for rigid foundation based on Winkler model. Their curves show the relationship between uplift and yield of footing and foundation soil. Also, Shear modulus has significant effect on the calculated response.

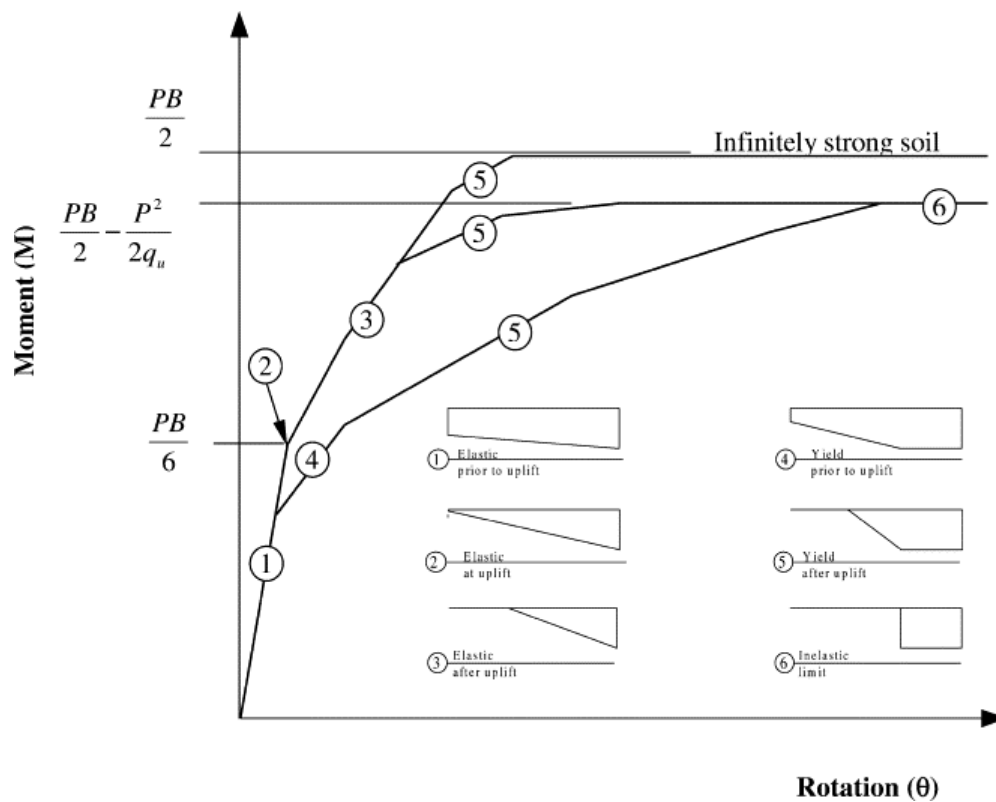


Figure 2.7: Mechanism of embedded shallow rocking foundation provided by Allotey and El Naggar (2003)

Soon after, Mergos and Kawashima (2005) concluded that rocking of footing acts as isolation and the effect of isolation increases under biaxial excitation. Also, residual settlements can be noticed due to soil yielding. Later, Allotey and El Naggar (2008a,b) explored the strengths and limitations of the Federal Emergency Management Agency's FEMA (FEMA356, 2000) nonlinear Winkler modelling approach when used to predict the cyclic response of various recent Soil Foundation Structure Interaction (SFSI) experiments. The study results show that the model can satisfactorily predict all the different cyclic response quantities apart from permanent horizontal displacements. This should therefore be noted when using the approach in design.

Similarly, Harden and Hutchinson (2009) developed a new model called BNWF (Beam

on Non-linear Winkler Foundation) model for modelling the rocking dominated shallow foundation. The modelling techniques of BNWF have been discussed in detail in the subsequent section of soil idealization. Adjoin studies from [Raychowdhury \(2008\)](#); [Raychowdhury and Hutchinson \(2009\)](#); [Raychowdhury \(2011\)](#) analysed the shear wall supported on rocking foundation. From the results, they concluded that for low rise structures, energy dissipation is mainly through the sliding mode. Whereas for high rise structures, the dissipation is dominated by rocking mode. For low to medium-rise, it is the combination of both modes. Also, the maximum settlements were observed to be well within the permissible limits prescribed by the design standards.

[Chatzigogos et al. \(2009\)](#) also present a new macroelement model for shallow foundations. They introduced several second-order effects due to uplift like soil rounding effect. They describe two phenomena: one is due to the irreversible behaviour of soil and another part that is reversible due to foundation uplift and yielding of foundation soil. [Anastasopoulos et al. \(2010\)](#) have assessed the seismic performance of the conventional design and an under-designed rocking foundation as a design alternative for the bridge structure subjected to around 29 accelerograms with various acceleration and velocity profiles. Their research shows that during moderate intensities of earthquake, both the design performs in a similar manner with minor variations. The larger intensity earthquakes under the designed rocking foundation offer better performance than the conventionally designed one by avoiding the collapse. However, it has to be noted there exists residual rotation and excessive settlement exists after seismic action, which has to be retrofitted later. [ElGawady et al. \(2011\)](#), varied the interface material for rocking rigid block and summarizes that rubber base can enhance the rocking. [Gelagoti et al. \(2012\)](#) considered the effects of geometric nonlinearities such as uplifting and p-delta effects along with material nonlinearities for soil and structure. Similarly, [Pender et al. \(2013\)](#) provides a simple approach to design the shallow foundation that accounts for rocking along with an example calculation. He showed the variation of vertical load and moment due to uplifting along with deformation response. Conclusions drawn by them were similar to that of previous researches. But the major difference is that this time two-dimensional (2D) frame has been selected. Apart from that, various researches ([Chiou et al., 2018](#); [Apostolou et al., 2007](#); [Cheng, 2007](#); [Palmeri and Makris, 2008](#); [Zhang and Tang, 2009](#); [Masaeli et al., 2014](#)) on rocking foundation shows that allowing the foundation to uplift is beneficial.

2.6 Soil idealization

[Gazetas \(1991\)](#) presented an uncoupled elastic stiffness for footing by idealising the soil and foundation as spring elements. His recommendations have been widely used in the guidelines of [Council \(1996\)](#) , [FEMA356 \(2000\)](#) and [ASCE41-17 \(2017\)](#). Gazetas presented global stiffness for surface foundation along with an embedment depth factor for embedded foundations. The idealization technique followed by Gazetas is shown in Figure 2.9.

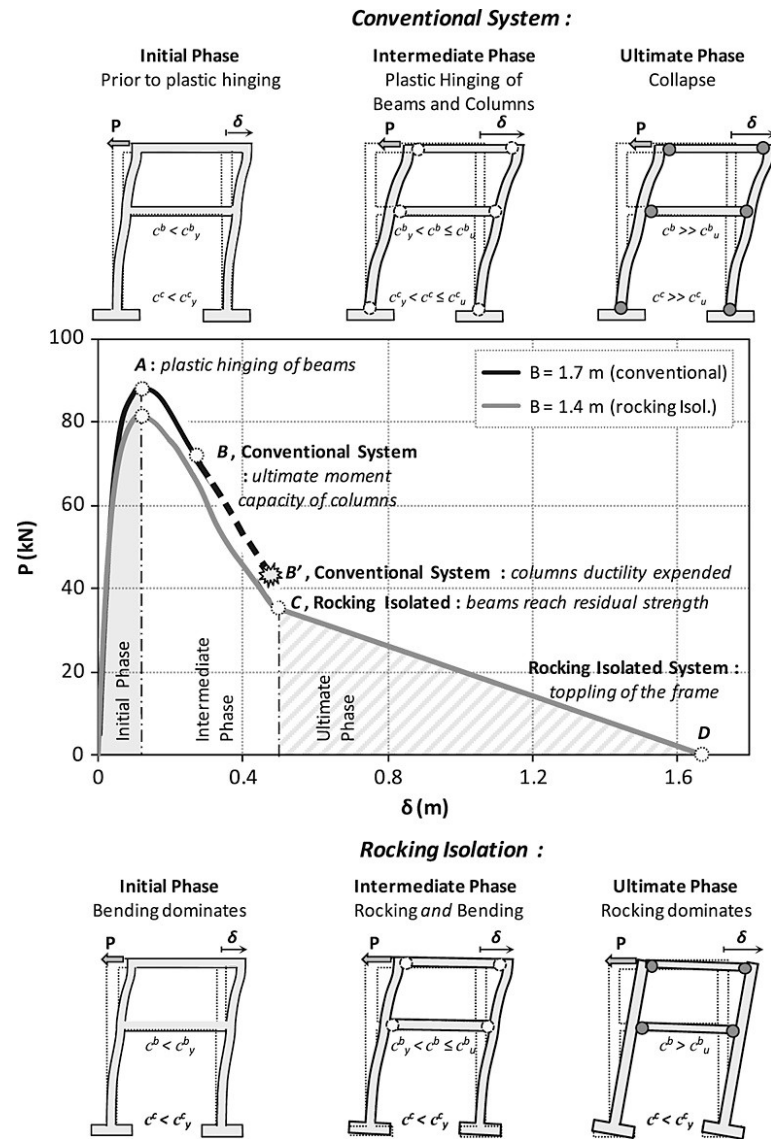


Figure 2.8: Mechanism of failure in RC frame supported by shallow rocking foundation and conventional foundation (Gelagoti et al., 2012)

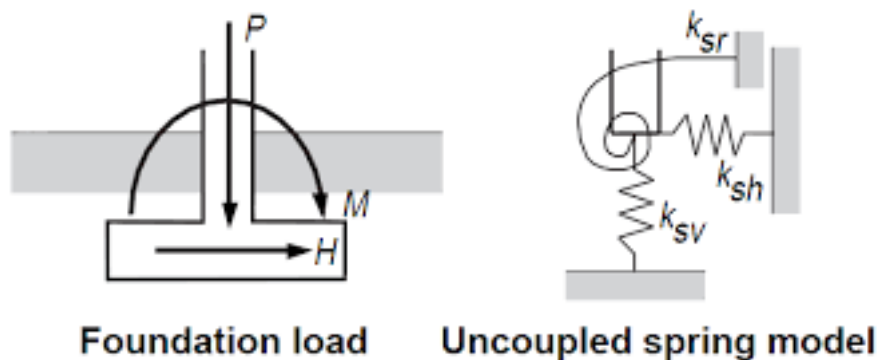


Figure 2.9: Uncoupled springs for idealizing the foundation and soil provided by Gazetas (1991)

Council (1996) also recommends the following idealizations that can be used for modelling the frame and shear wall components. One of the other alternatives is to provide higher Stiffness on the end zones than the middle zones. The reason behind this recommendation is that during uplift end zone experiences more loads than the central zones. Beam on Nonlinear Winkler Foundation (BNWF) proposed by Harden and Hutchinson (2009) idealized the foundation as a flexible elastic beam supported by a series of discrete nonlinear vertical Winkler Springs as shown in Figure 2.10. Also, two springs representing the behaviour due to sliding and rotation have been added in horizontal direction. All the springs are considered as 1D Zero-length elements. BNWF can incorporate both material & geometric nonlinearities to capture pure sliding, rotation and settlements. The concentration of vertical springs can be varied as per the requirements easily. However, this model is limited to C (Cohesion) and ϕ (angle of internal friction) soils majorly.

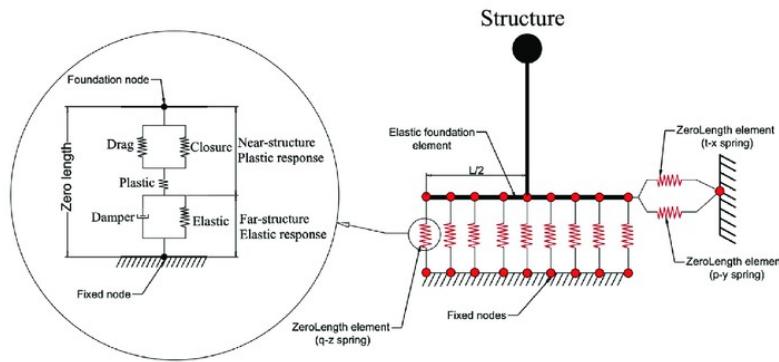


Figure 2.10: Beam on non linear Winkler foundation model provided by Raychowdhury (2008)

Contact Interface Modelling (CIM) by Gajan (2006) idealizes the footing as rigid and the soil beneath the foundation of the influence zone as a single macro element Figure 2.11. This model can predict the moment- rotation-settlement and shear-settlement-sliding relationships. This model is simple to use and also it requires only six parameters to be defined. This model assumes the footing as rigid and applicable only for rectangular footing with lateral loading along its length direction.

FEA is one of the most widely used numerical techniques to solve problems related to a wide range of SSI problems. The general procedure for conducting the finite element analysis is to discretise the soil and structure into smaller domains called finite elements (Figure 2.12). All of these formulated elements will be identified as nodes, which possess certain degrees of freedom. Solution for the problem will be first achieved at the local level, then assembling a discretised domain will give stresses and strains at the local level. Finite element method can represent the rocking foundation problem for all types of geometries, including pore water pressure in the soil medium. The interface between the soil and foundation to be carefully represented for better results on rocking foundation problems (Gelagoti et al., 2012; Anastasopoulos et al., 2014). Due to the wide range of capabilities, it can capture realistic responses of rocking foundation. However, they are computationally expensive

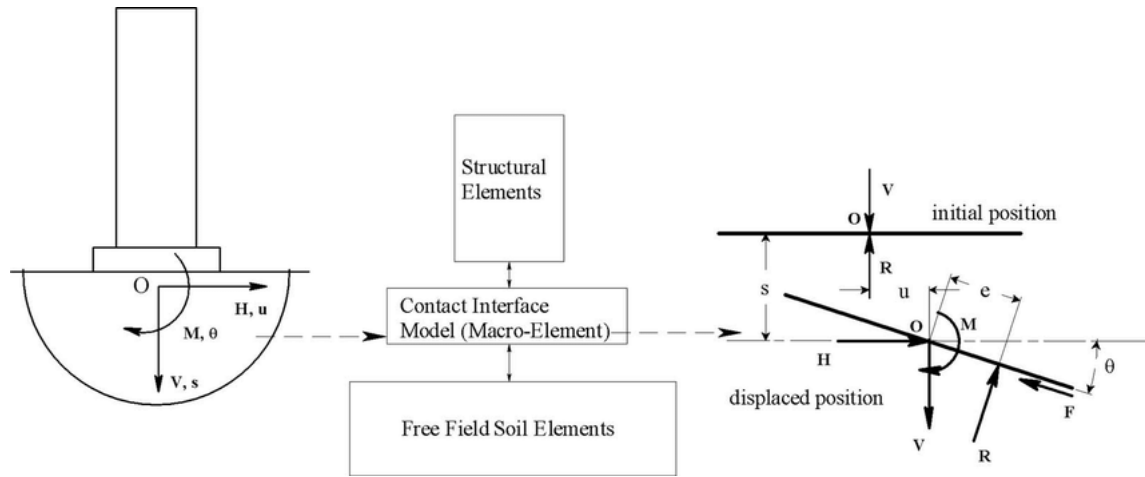


Figure 2.11: Contact interface modelling recommended by Gajan (2006)

than other modelling mentioned above.

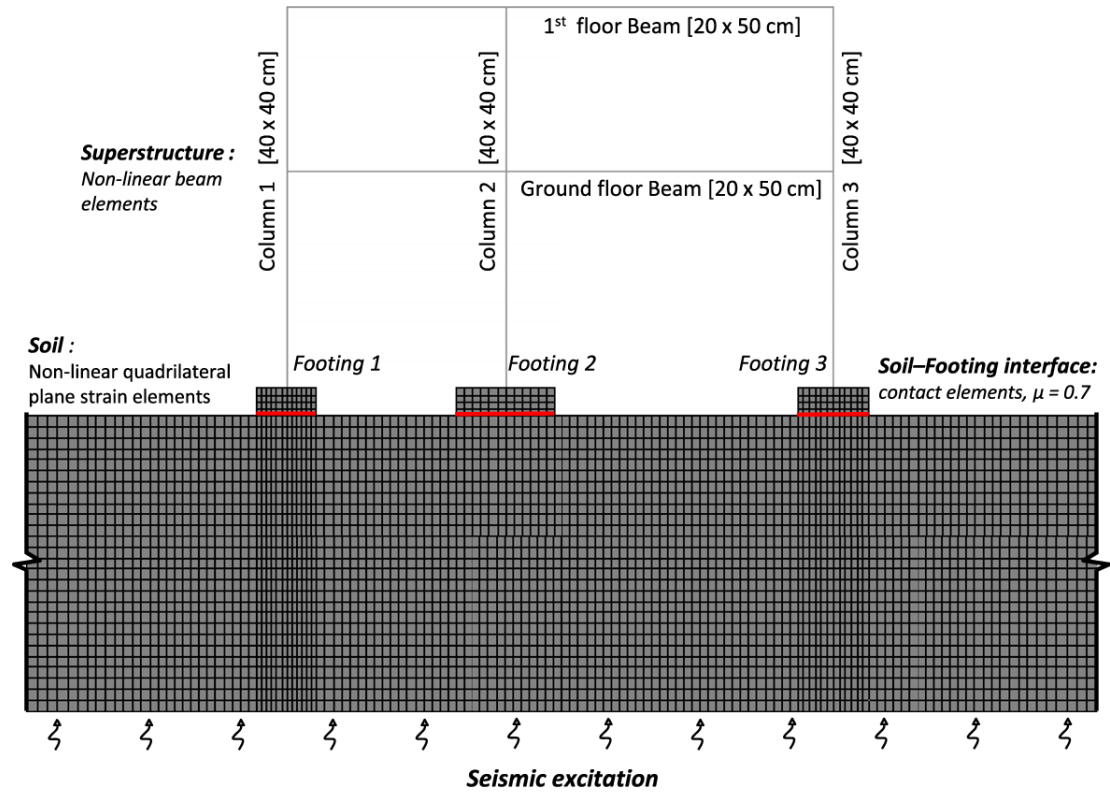


Figure 2.12: Finite element modelling for foundation rocking (Anastasopoulos et al., 2014)

2.7 Prevailing research gaps

From the past studies it is observed that allowing foundation rocking does not necessarily causes overturning of structures, instead it may cause excessive residual settlement. On the other hand conventionally designed structures are prone to have severe damages or even collapse under severe earthquake events. From the previous sections

it is also clear that allowing foundation rocking enhances the overall seismic performance of the structure. There are variety of numerical methods available to simulate the explicit and combined effects of foundation rocking and super structural responses. Conventional foundation design methods lead to a highly flexible structure with rigid foundation parts, resulting in increased damage during intense seismic activity. It is not cost-effective to repair such damages in such instances, which means that the usage of rocking foundation rather than conventional stiff foundation can be considered for further investigation. A lot of research has been done on rocking foundations for different types of structures, like bridge piers (Hung et al., 2014; Cheng, 2007; Loli et al., 2014), storage tanks (Haroun and Ellaithy, 1985; Veletsos and Tang, 1987), and single-bay framed buildings (Gelagoti et al., 2012; Anastasopoulos et al., 2015). However, there is a lack in establishing the general design guidelines for foundation proportioning to allow for an adequate amount of rocking, and there is less information on the damage risks of a multistorey RC building on a rocking foundation that is located on different types of soil. Therefore, this thesis work concentrates on the lagging part of multistorey RC buildings to find out optimum size reduction to improve the overall seismic performance of the combined superstructure and foundation.

2.8 Scope and objectives of this study

This present thesis work aims to compare the effectiveness of rocking foundation with varying degree of rocking over conventionally designed footings for RC framed structures with and without shear wall. The scope typically includes systematically varying the key parameter that governs the seismic performance of the structures. In particular the scope of this thesis work aims to to quantify the seismic force and displacement demands on the explicit and combined effects of the rocking foundation and superstructure behaviour with respect to key parameters. The main idea behind the scope is to identify the most beneficial rocking foundation system for the generic symmetrical structures on generalized soil conditions. Whereas, unsymmetrical structures, liquefiable soils and foundation of irregular shapes are beyond the scope of this present thesis work.

To support the scope and to provide further insights, the objectives of thesis work is carried in out in three phases as follows:

Objective 1: Seismic performance of rocking foundation on RC bare framed building

- *First step* is to evaluate the seismic performance of rocking foundations supporting RC framed building under various parametric conditions, including type of soil, varying foundation sizes, embedment depth of foundation and height of the superstructure.
- *Second step* is to Compare the seismic performance of rocking foundations with those of conventionally designed footings to assess its effectiveness and limitations

Objective 2: Seismic performance of rocking foundation supporting shear wall in RC framed building

- *First step* is to evaluate the seismic performance of rocking foundations supporting RC shear wall in framed building under various parametric conditions, including type of soil, varying foundation sizes of foundation and height of the superstructure.
- *Second step* is to Compare the seismic performance of rocking foundations supporting shear walls with those of conventionally designed footings to assess its effectiveness and limitations.

Objective 3: Seismic fragility assessment of rocking foundation

- *First step* is to develop the seismic fragility curves for of rocking foundations on RC framed building by nonlinear static pushover analysis
- *Second step* is to develop the seismic fragility curves for of rocking foundations supporting shear wall in RC framed building through Incremental Dynamic Analysis (IDA)

2.9 Summary

This chapter provides the brief evolution of seismic resistant design practices followed by the review studies on rocking shallow foundation. The review included experimental, analytical and numerical studies. The beneficial effects of rocking foundations are clearly demonstrated by various researchers across decades are highlighted. In addition to theta numerical modelling tools available are also highlighted. From the research gaps are pointed out to carry the future research to incorporate the rocking foundation as potential energy dissipating mechanism. Finally the scope and objectives for this present thesis work is outlined.

Analytical modelling and validation

3.1 Overview

The main objective of this thesis is to assess the impact of foundation rocking on the behaviour of substructure and superstructure under nonlinear static and nonlinear dynamic analyses. Therefore, it is crucial to choose elements and materials that can effectively replicate the behavior of both superstructure and the substructure, since both are prone to experience substantial inelastic deformations when exposed to seismic forces. In this chapter, suitable material and element modelling approaches that are adopted from the existing studies are discussed to understand the nonlinear behavior of soil and RC superstructural elements. The entire analytical modelling is carried out with the help of the OpenSees ([Mazzoni et al., 2006](#)), an open-source finite element package. The analytical model for the superstructure is validated by comparing its results with those available from the previous experimental studies. Further, the analytical modelling developed in the present work is compared with the results obtained from the Extended Three-Dimensional Analysis of Building System [ETABSv.17 \(2017\)](#) package. The aforementioned modelling strategies are discussed in detail in the subsequent sections.

3.2 Analytical modelling for RC framed building without shear wall

For the accurate simulation of inelastic behaviour in RC framed building, it is crucial to estimate the sectional and reinforcement properties of structural members. To obtain these properties proper design and reinforcement detailing is required as per the compliant standards. Open source frame work ‘OpenSees’([Mazzoni et al., 2006](#)) is used to model and analyze the building investigated in this thesis work. This framework is capable to carry out large displacement analysis of framed building by taking into account of material nonlinearity and geometric nonlinearity for both superstructure and sub structural elements. In addition to that this framework also capable to carry out wide range analysis including eigen value analysis, linear and non-linear static cum dynamic analyses. The analysis results are further exported to MATLAB ([Inc, 2022](#)) for further post processing. A detailed discussion which is provided in the subsequent sections to describe the design parameters followed by the constitutive material modelling are outlined here.

3.2.1 Design parameters of superstructure

The parametric studies involves two sets of building viz. 8-storey, and 10-storey having a storey height of 3.3 m. A generic plan dimensions are selected from the pilot survey (Haldar et al., 2016; Kurmi and Haldar, 2024). As per the survey majority of the building having plan dimensions between 15m to 35m. Accordingly, the dimensions along the longitudinal direction is considered to be 25m and along transverse direction it is chosen to be 18m as shown in Figure 3.1.

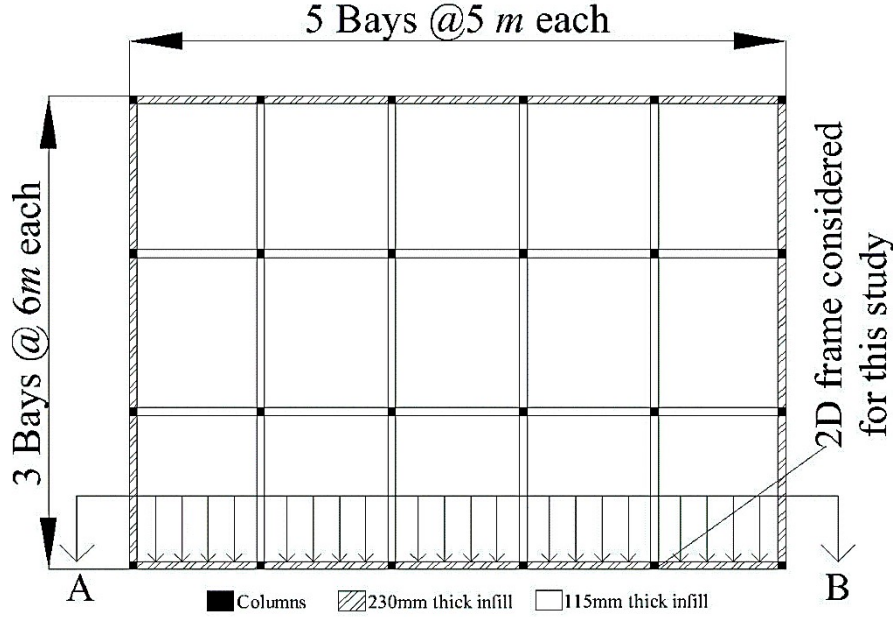


Figure 3.1: Plan of the considered RC structure without shear wall

Both 8-storey, and 10-storey building are assumed to be situated in seismic zone IV as per IS1893 (2016). M40 grade of concrete and Fe-550 grade of steel is used for the design. The dead loads and live loads are assigned in accordance with IS875 Part-I (2003) and IS875 Part-2 (2003) respectively. The frame members for the building i.e., beams and columns are designed as Special Moment Resisting Frames (SMRF) as per relevant Indian standards (IS1893, 2016; IS456, 2000; IS13920, 2016). The self-weight of infill i.e., 230 mm thick infill for exterior walls and 115 mm thick infill for interior walls, as shown in Figure 3.1, are considered to be acting on the beam as dead load without considering actions of infill. The detailed design parameters are provided in Table 3.1

3.2.2 Nonlinear structural modelling

To simulate the seismic responses within the superstructure, the mass of the superstructure is assumed to be acting at every floor level as shown in Figure 3.2. A representative diagram of the scheme adopted for the mass (dead load as well as live load) related to the various components of the structure from both longitudinal and transverse directions (Figure 3.2). It can also be seen from the figure that apart from the in-plane elements, the

Table 3.1: Design parameters for the considered bare framed structure

General	Design level	Bare frame with response reduction factor R=5
	Storey height	3.3m between each storeys
Materials used	Concrete	M40
	Steel	Fe500
Loading details	Dead load	Self weight of structural members
		Weight of slab along with floor finish
		Weight of infills
		Weight of parapet walls in roof
	Live load	3kN/ <i>m</i> ²
	Design load combinations	1.5(Dead load+live load)
		1.2(Deadload + live load <u>±</u> earthquake load)
		1.2(Deadload <u>±</u> earthquake load)
0.9 Deadload <u>±</u> 1.5 earthquake load)		
Structural modelling details for design	Software used	ETABS v17
	Model type	3D space frame
	Element models adopted	3D line elements for beams and columns
		Rigid diaphragms to simulate slab response
		Action of infills not considered
	Plasticity model	Lumped plasticity model
Geometric nonlinearity	P- δ effects is considered	

mass is lumped for the out-of-plane elements (for half-width of the bay in the out-of-plane direction) such as beams and slabs at the respective nodes for all the storey are also considered for the simulation. The selection of external frame is made because external frames plays a crucial role in resisting the lateral due to seismic actions as the frames having higher exposure than the internal frames.

The reliability of numerical results are heavily dependent on the analytical modelling techniques adopted and also on the analysis types. Since, structural members of RC building consists of non-homogeneous material sections and hence it is important to select suitable nonlinear structural modelling techniques to capture the nonlinear behaviour. Broadly two major modelling strategies called as lumped plasticity or distributed plasticity ([Adhikari and Pinho, 2010](#); [Antoniou and Pinho, 2018](#)) are adopted to formulate nonlinear behaviour in structural members as shown in Figure 3.3.

In the lumped plasticity approach, nonlinearity in the material is assumed to get concentrated mostly at the edges of the structural element whereas the remaining the portion of the structural portion is assumed to remain elastic. This behaviour can be generated by providing the plastic hinging zones or nonlinear springs with the specified

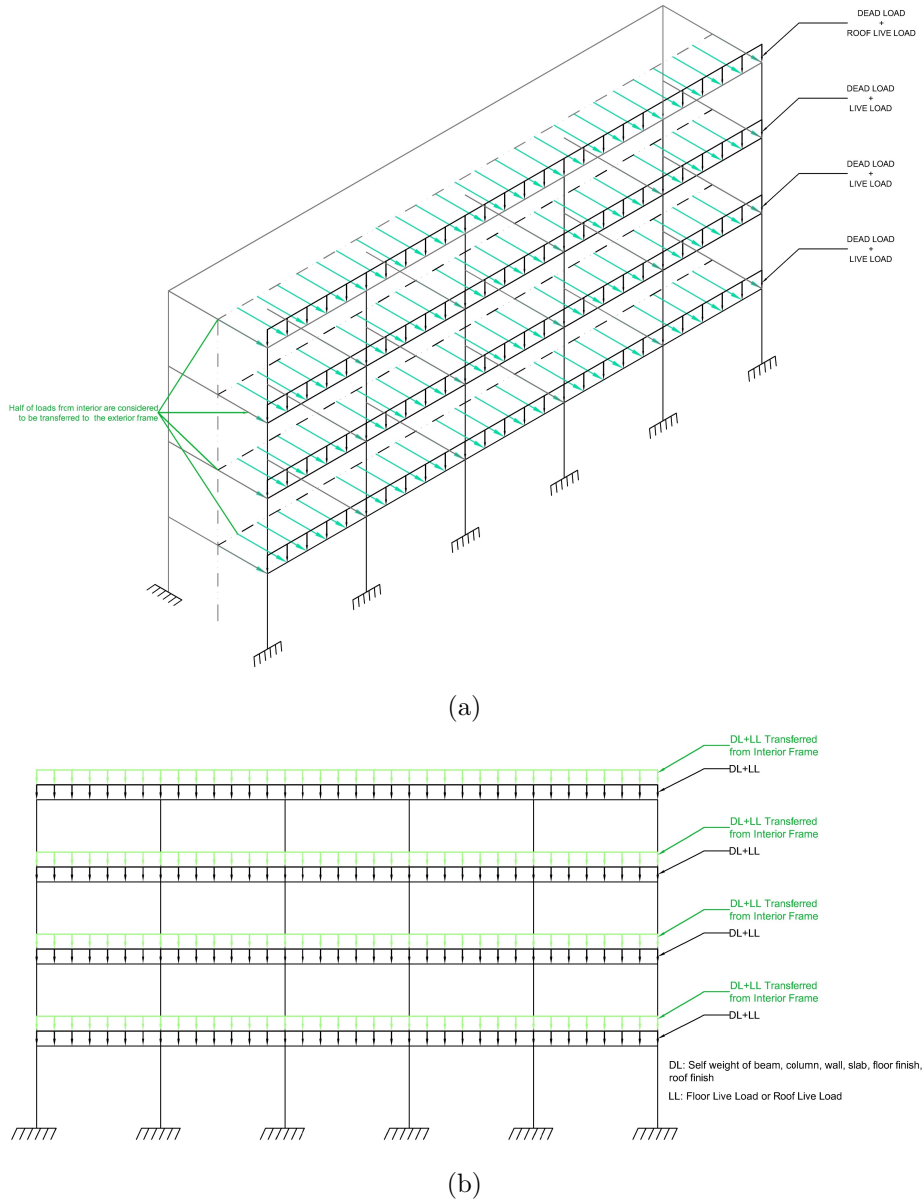


Figure 3.2: RC super structural idealization; (a) loads acting on the external frame under consideration ; (b) 2D model adopted for this thesis work

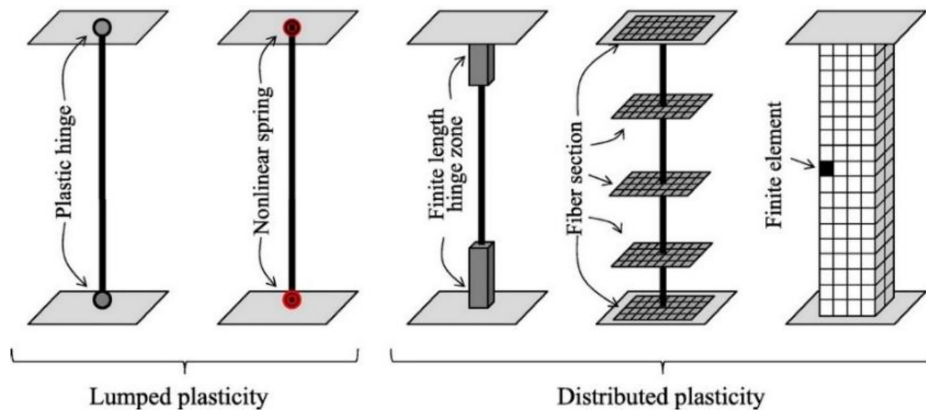


Figure 3.3: Various methods for plasticity formulation (Deierlein et al., 2010)

force deformation relationships. Contrary to the lumped plasticity approach distributed plasticity allows the plastic deformations throughout the length of the structural element. Distributed plasticity approach provides more realistic approach, however, computational efficiency and cost is higher than the lumped plasticity approach. Some of the major pros and cons of both the method is presented in Table 3.2.

Table 3.2: Pros and cons on types of plasticity formulation

Plasticity Formulation	Pros	Cons
Lumped plasticity approach	✓Can efficiently model the strength and stiffness degradation resulting from concrete crushing and rebar yielding during seismic loading	✗High expertise on structural modelling is needed to calibrate the requires relationships to form plastic hinges
	✓Reduced computational cost and storage	✗Provides accurate results only when assumed plastic hinges matches is appropriate ✗Difficult to capture the axial and flexural interaction behaviour
Distributed plasticity approach	✓High expertise on structural modelling and calibration is not requires	✗Increased computational and storage costs
	✓Axial and flexural interaction can be captured efficiently	✗Difficult to capture shear ddeformation effects
	✓ Can capture the gradual strength degradation along the span of structural member	

Owing to availability of computational facilities and also taking into consideration of advantages in distributed plasticity approach fiber based formulation with distributed plasticity is used in this present study to model the structural members such as beams and columns. Distributed plasticity can be accounted in two ways 1.through displacement based formulation ([Adhikari and Pinho, 2010](#); [Antoniou and Pinho, 2018](#)) and 2. force based formulation ([Adhikari and Pinho, 2010](#); [Spacone et al., 1996](#); [Neuenhofer and Filippou, 1997](#)). In the displacement formulation field variables are displacements whereas in the force based formulation the field variables are internal forces in the structural elements. The magnitude of the field variables in intermediate locations are evaluated based on the interpolation functions. In this present thesis work, the force based formulation is used to model the structural elements since it can accurately simulate the inelastic behaviour than the displacement based formulation in a simplified manner by providing sufficient number of integration points in a single element ([Adhikari and Pinho, 2010](#); [Antoniou and Pinho, 2018](#)).

The Gauss-Lobato method of weighted numerical integration is used to simulate the nonlinear behaviour in structural elements. Another criteria in selecting the number of integration points is critical since it decides the accuracy of the results. Higher number of integration points may lead to numerical instability whereas lower number may affect the accuracy of the analysis results. Hence five to eight integration points (Berry and Eberhard, 2006) for single structural element is considered to be adequate to represent the nonlinear responses as shown in Figure 3.4. Each integration points consists of section which consists of collection of nonlinear materials such as concrete and steel, generally termed as nonlinear fiber materials. Where each material is formed with different constitutive laws as per requirements. The collection of non-linear fiber materials includes unconfined cover concrete fiber, confined core concrete fiber and longitudinal steel reinforcement fibers (Figure 3.4). This modelling strategy is collectively called as fiber based modelling approach (Haselton et al., 2008). As mentioned earlier each fiber should be represented with appropriate material constitutive laws to depict the nonlinear responses. This present study uses three material constitutive laws, two for concrete and other one for steel.

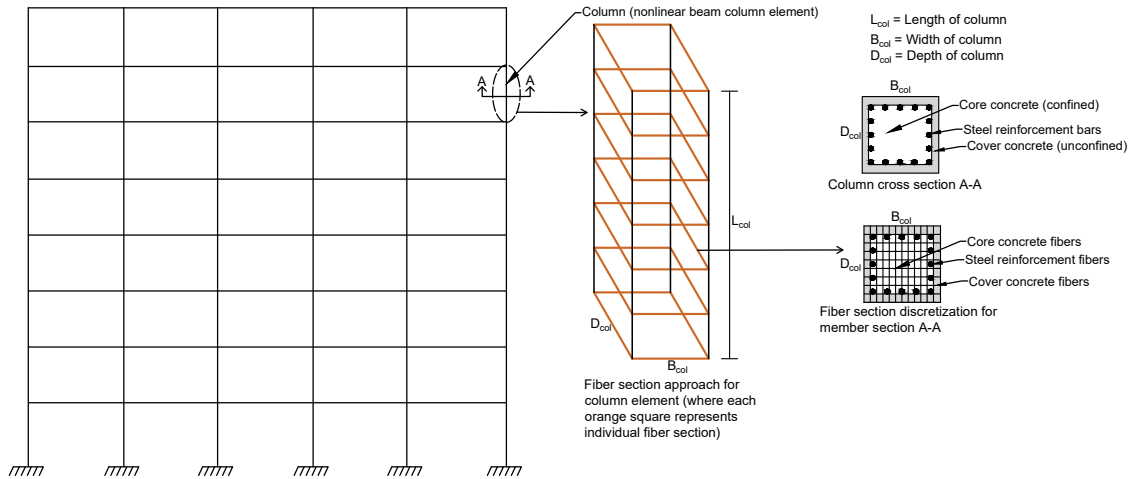


Figure 3.4: Fiber based modelling for RC structural elements

The constitutive laws for confined and unconfined concrete differs based on the transverse reinforcement bars (Laterza et al., 2017). Under compressive loads the transverse reinforcement bars imparts confining action to the core concrete due to which the strength and ductility of the section is increased. In case of absence of this effect the strength and ductility will be lesser and will be equal to the unconfined concrete i.e., cover concrete. Generally for building designed as SMRF the confining action will be noticed and hence it is important to account for it by providing appropriate stress strain relationships. In this thesis work the stress-strain behaviour of unconfined concrete fibers are modelled using Kent and Park (Kent and Park, 1971) model, and that of confined concrete fibers is modelled using the modified Kent and Park (Scott et al., 1982) model as shown in Figure 3.5.

In an unconfined concrete the stress strain relationship descent immediately

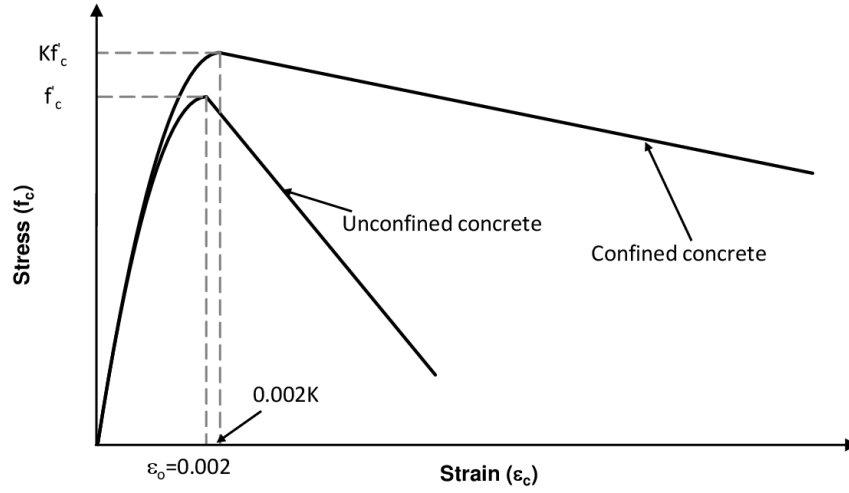


Figure 3.5: Stress strain relationship for concrete

after attaining the peak stress. The slope of the stress strain relationship after attaining the peak stress can be identified by calculating the strain at 50% (ϵ_{50u}) using the following relationship where the failure stress in unconfined concrete is defined as $0.2f'_c$.

$$f_c = f'_c \left[\frac{2\epsilon_c}{0.002} - \left(\frac{\epsilon_c}{0.002} \right)^2 \right] \quad (3.1)$$

$$\epsilon_{50u} = \frac{3 + 0.002f'_c}{f'_c - 1000} \quad (3.2)$$

where:

f_c denotes compressive stress in concrete

f'_c denotes cylinder strength of concrete

ϵ_c denotes strain in concrete

For confined concrete the strength and ductility of section increases with the provision of transverse rebar. To account for this action initially Kent and Park model is used to take care of increased ductility alone without considering the increase in strength due to confining action. For calculating the increase in strength, Scott modified the Kent and Park model where the effect of ductility remains same as before. The increased strength is given by Kf'_c where the concrete strength is obtained using Kent and Park model. The value of K is given by the following equation,

$$K = 1 + \frac{\rho_s f_{yh}}{f'_c} \quad (3.3)$$

where:

K denotes scalar factor to increase the unconfined concrete strength

ρ_s denotes ratio of volume of transverse reinforcement to volume of concrete core

f_{yh} denotes yield strength of transverse reinforcement steel

To find the corresponding strain for increased strength, the strain corresponding to the 50% reduction and post 50% reduction in peak stress is calculated using the Kent

above mentioned parameters are incorporated using Steel02 (Filippou et al., 1983) material in OpenSees (Figure 3.7).

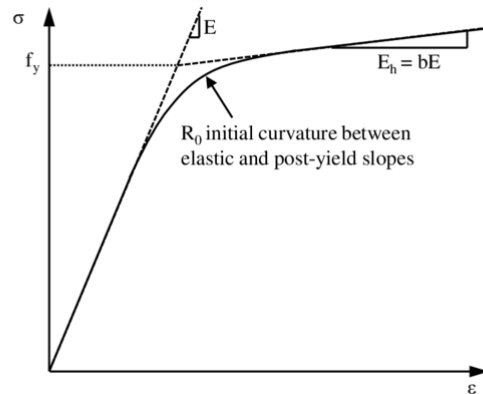


Figure 3.7: Pictorial representation of Steel02 material model

In addition to the material nonlinearity, geometric nonlinearity (McGuire et al., 2000) plays a crucial role in governing the overall seismic behaviour of the building due to deformations of frame members along with the expected higher displacements due to the uplifting nature of the foundation. The effect of geometric nonlinearity may be negligible for the fixed base building whereas it may increase with the increasing effect of rocking.

Analysis after incorporating geometric nonlinearity are also called as second order analysis in which the equilibrium and kinematic equations are formed with respect to the deformed configuration of the structure (Chen and Lui, 2005). In other words second order analysis effectively incorporate the effects of P- δ and P- Δ . The P- δ effects captures the effect of axial force on the elements of the frame on its flexural stiffness. On the other hand, P- Δ effect incorporates the effects of lateral displacement along with the flexural deformations of frame elements. In the RC building the effect of P- δ under compressive loading is known to cause significant flexural stiffness reduction (Devadas, 2003) and hence it is crucial to incorporate this effect especially during the nonlinear analyses, therefore its is incorporated in the all the analyses considered in this study using the geometric transformation command in OpenSees module.

3.2.3 Design parameters of substructure

Foundation is that part of the structure which is in direct contact with soil and effectively transfers the load from the superstructure to underlying soil medium. The present thesis evaluates explicit and combined seismic responses of both superstructure and substructure, which exhibits substantial inelastic behaviour during seismic excitation. The detailed modelling techniques for simulating inelastic behaviour of superstructure is presented in the above section whereas the following section highlights the design parameters and modelling techniques adopted for this thesis work for substructure and supporting soil.

Isolated shallow footings are proportioned for dead loads, live loads, and seismic loads in accordance with the capacity design concept as outlined in the Indian standards (IS1904, 1986; IS6403, 1981) by considering various soil condition (Bowles and Guo, 1996; DC, 1990; Peck et al., 1991) represented by its properties given in Table 3.3. Conventionally Designed Footings (CDF) have a greater moment resisting capacity than the super structural part they support. The eccentricities are kept to a minimum for CDF, typically no more than one-sixth the width of the footing. Relaxing the eccentricity requirement and reducing the earthquake load considered for the footing design results in an under designed footing width for the respective structural member i.e., the moment carrying capacity of footing is reduced. By means of the reducing the footing dimensions foundation flexibility can be increased which will result in the foundation uplift during seismic actions. In other words, the factor of safety against the vertical loading will be higher due to higher dimensions of footings when earthquake loads are considered. This factor of safety is reduced gradually till it reaches close to the factor of safety against the static loading. Beyond which the reduction is not considered since the serviceability of the structure may gets affected.

Table 3.3: Soil types considered for this study

Soil type	Angle of internal friction (°)	Unit weight (kN/m^3)	Modulus of elasticity (Mpa)	Poissons ratio
Medium dense sand	35	17	47.88	0.3
Dense sand	40	20	71.82	0.3
Very dense sand	45	22	95.76	0.4

3.2.4 Nonlinear substructure modelling

Rocking of foundation may trigger footing uplift on one side during the lateral load application, which causes yielding of soil beneath the foundation depending upon the intensity of lateral load and magnitude of uplift. Also, upon load reversal, permanent settlement may occur beneath the foundation as well. The supporting soil and foundation are modelled together as BNWF (Beam on Nonlinear Winkler Foundation) as per the guidelines provided by Raychowdhury (2008), considering its potential to capture the aforementioned characteristics of the rocking foundation. The BNWF model consists of nonlinear one-dimensional (1D) spring elements that connects the foundation element with the supporting soil is used to simulate the bearing, sliding and rotational resistances. Whereas the foundation element is modelled as 1D elastic beam with three degrees of freedom which incorporates the properties of the three dimensional (3D) shallow foundation element. The springs for the bearing resistance comprises of individual discrete non linear winkler springs that are considered as zero length 1D elements as shown in Figure 3.8.

Similarly, two discrete springs that are connected at the edge of the 1D elastic footing element to capture the sliding and lateral resistances (Figure 3.8).

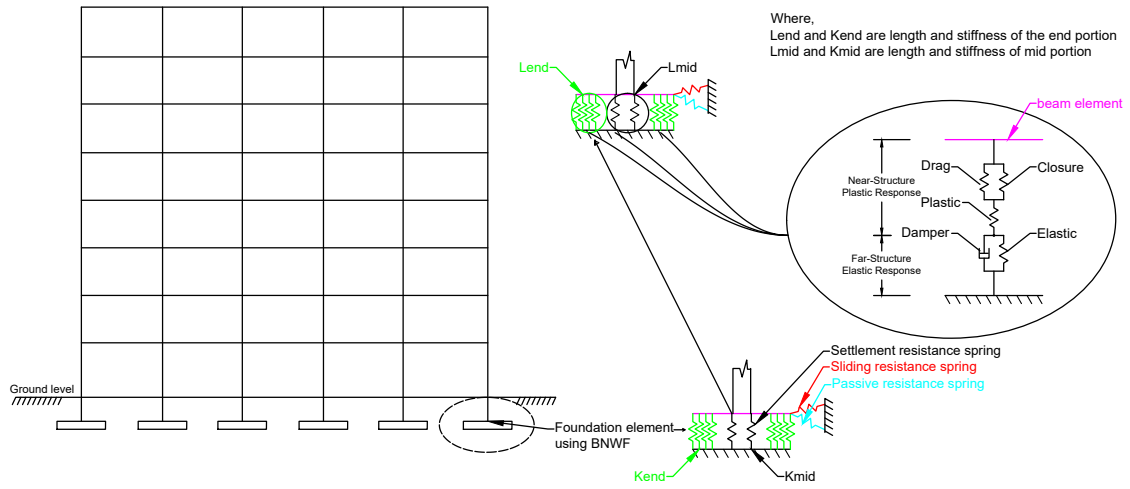


Figure 3.8: Pictorial representation BNWF model

Nonlinear behaviour of all the three types of springs mentioned above is represented using the modified Qzsimple1, Pysimple1 and Tzsimple1 material models which are originally developed by Boulanger (2000) for modelling the soil pile interaction. Later, modifications are proposed by Harden and Hutchinson (2009) and implemented further by Raychowdhury (2008) for simulating nonlinear shallow foundation responses. These modified elements can simulate the vertical load-settlement behaviour and moment-foundation rotation behaviour using the vertically distributed springs and simultaneously horizontal passive load behaviour along the footing side and shear-sliding response at the footing base using horizontally placed springs. The benefits of using BNWF are as follows

- The BNWF model can effectively capture the material and geometric nonlinearity of foundation and supporting soil. Along with the nonlinearity, inelastic deformations can also be effectively accounted. As a result of this BNWF can capture elastic and inelastic settlement, rotation and sliding displacements.
- Due to hysteretic material behaviour, BNWF can predict the energy dissipation capacities for all modes of deformation.

The unsymmetrical hysteresis response of Qzsimple1 is derived by ultimate vertical load on compression behaviour of soil and near zero tension to account for soil's weak behaviour in tension. Qzsimple1 material consists of three components to i.e., 1.elastic material to simulate the near field elastic behaviour; 2.plastic material component to simulate the far-field plastic behaviour and; 3. gap element that is connected in series between elastic and plastic component to account for uplifting behaviour of foundation element. The elastic portion of the Qzsimple1 material and its initial range is described using the following relationships

$$q = k_{in}z \quad (3.7)$$

$$q_0 = C_r q_{ult} \quad (3.8)$$

where:

q denotes load applied from foundation to soil

k_{in} denotes initial tangent stiffness

z denotes instantaneous displacement due to vertically applied load

q_0 denotes load at yield point

C_r denotes parameter which controls the range of elastic portion

q_{ult} denotes ultimate load that the soil can carry (q_{ult} can be calculated as per [Meyerhof \(1963\)](#) bearing capacity equation)

For the nonlinear portion i.e., after yielding the backbone curve can be represented by

$$q = q_{ult} - (q_{ult} - q_0) \left[\frac{cz_{50}}{cz_{50} + |z^p - z_0^p|} \right]^n \quad (3.9)$$

where:

z_{50} denotes mobilized load at 50%

z_0 denotes displacement at yield load

c and n denotes parameters that controls the shape of the post yielding portion in the backbone curve

$P_{ysimple1}$ material module characterizes the ultimate passive resisting force and it can be calculated using

$$p_{ult} = 0.5\gamma K_p D_f^2 \quad (3.10)$$

where:

p_{ult} denotes passive earth pressure per unit length of footing

γ denotes unit weight of the soil

K_p denotes passive earth pressure coefficient

D_f denotes embedment depth of footing

$T_{zsimple1}$ material module on the other hand accounts for the frictional resistance along the length of the footing and is depicted by

$$t_{ult} = W_g \tan \delta + c' A_f \quad (3.11)$$

where:

t_{ult} denotes frictional resistance per unit area

W_g denotes weight acting from the superstructure to the foundation

δ denotes angle of friction between footing element and supporting soil

A_f denotes surface area of footing

To follow the uniformity in analysis some of the parameters such as elastic range, post yield stiffness, unloading stiffness and curve controlling parameters are incorporated as

per the OpenSees hard coded parameters. It is also been noted that these parameters are not dependent on the soil type and footing type hence default hard coded parameters are adopted. Numerous studies (Deviprasad et al., 2022; Pelekis et al., 2021; Rahgozar et al., 2018) have been provided on the most effective method of applying the BNWF technique to modelling the rocking foundation and the parameters are well documented for depicting the nonlinear behaviour of rocking foundation. Therefore, validation of the foundation modelling is not considered for this study.

3.2.5 Validation and verification of proposed analytical modelling of RC building

It is necessary to validate with experimental data to ensure the accuracy of the analytical modelling techniques used in this study. The laboratory experiments on one bay two-storey RC frame by Vecchio and Emara (1992) is taken as a reference model. With a center-to-center span of 3.5m for beam and a story height of 2m, the total height of the structure is 4.6m. Each structural element such as beams and columns are proportioned as 0.3m in width and 0.4m in depth. Further information required about the experimental details and material properties are obtained from Vecchio and Emara (1992). Consistent axial load of 700kN is applied during the experiment to each column and the lateral load is monotonically increased until the maximum capacity of the structure is achieved. Seismic response of the simulated analytical model obtained from OpenSees is compared in Figure 3.9 with the observations during the experiment. A high degree of concurrence is observed between the experimental and OpenSees models, thereby validating the simulated model's dependability and being considered for further study.

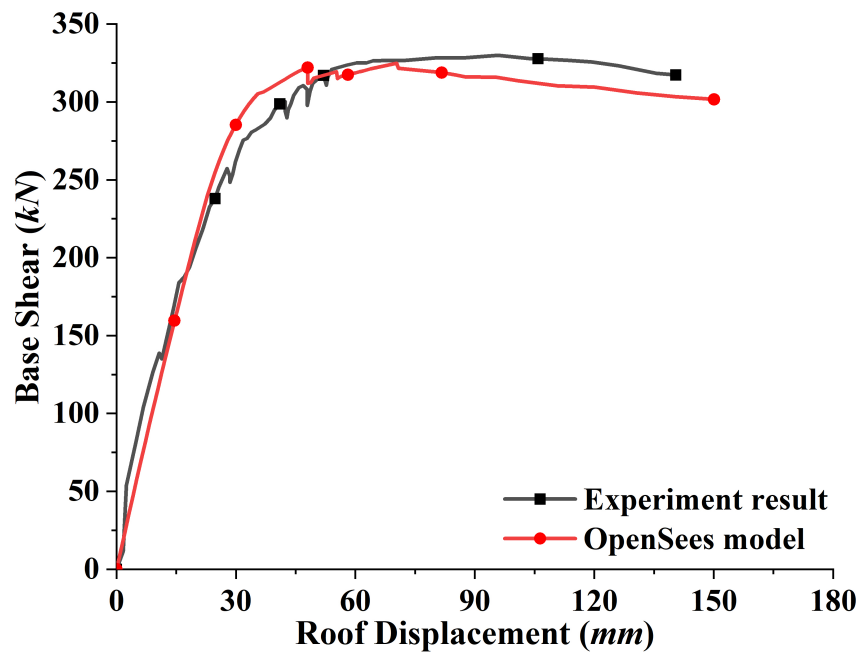


Figure 3.9: Validation of the simulated analytical model with experimental results obtained from Vecchio and Emara (1992)

In addition, six story three bay two dimensional frame model from the studies of [Sharma \(2020\)](#) is selected for the verification of the bare frame model. Figure 3.10 illustrates the roof drift to base shear resulting from nonlinear static analysis. In the linear region, the accuracy of modelling has been determined to be adequate based on the graph. Additionally, it is observed from the Figure 3.10 that the behaviour in the nonlinear regime was convincing. The numerical predictions from the study conducted by [Sharma \(2020\)](#) is found to be in close accordance with OpenSees simulated model. So, the above-mentioned numerical model can be used for a more rigorous non-linear analysis.

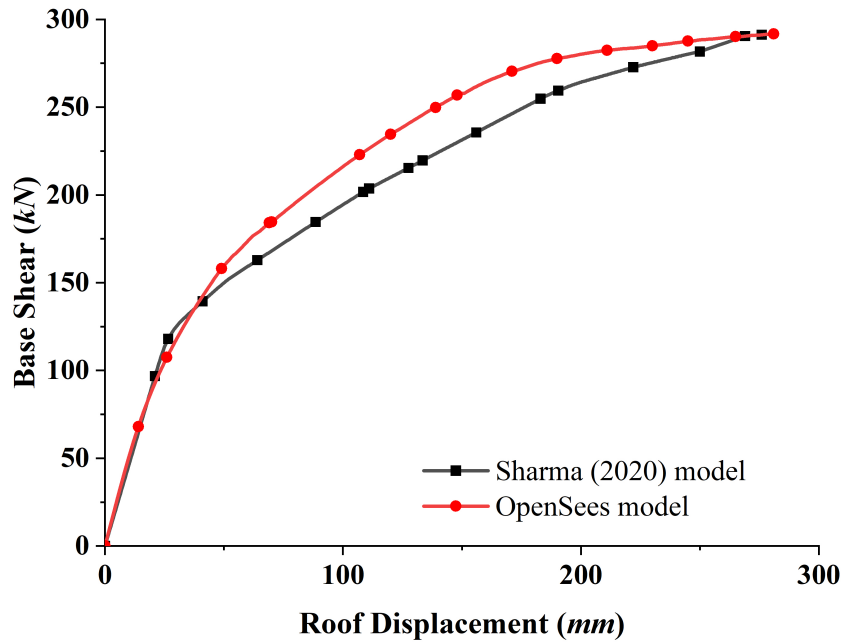


Figure 3.10: Verification of the simulated analytical model with results obtained from [Sharma \(2020\)](#)

3.3 Analytical modelling for RC framed building with shear wall

Similar to section 3.2, seismic responses of the rocking foundation and conventionally designed foundations supporting the super structural elements can be simulated with high accuracy by adopting suitable design and modelling procedures. To comprehend the nonlinear behaviour of soil and RC super structural elements with shear wall, the following material and element modelling methodologies are employed.

3.3.1 Design parameters of superstructure

Three sets of building viz. 4-storey, 6-storey and 8-storey having a storey height of 3.3 m is considered for the parametric studies. A generic plan as mentioned in subsection 3.2.1 is considered here also, but additional shear walls are provided as a lateral force resisting

member to counteract the seismic forces as shown in Figure 3.11. Seismic performance of RC shear wall frame building relies heavily on the location and of the shear wall.

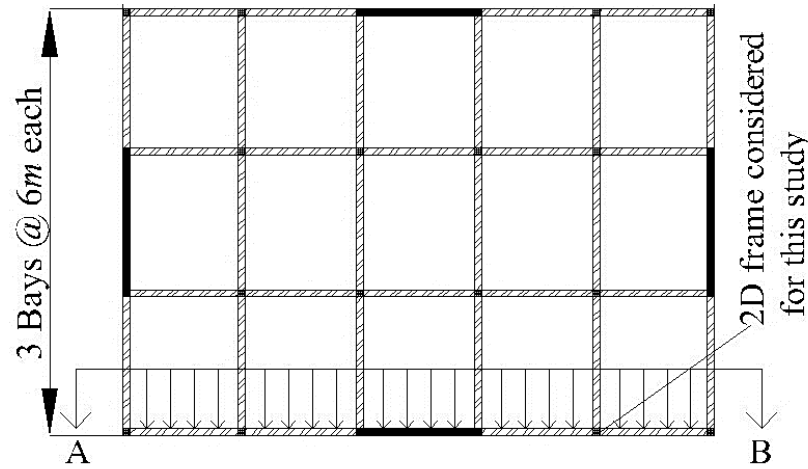


Figure 3.11: Plan of the considered RC structure with shear wall

Considering the complexity of foundation rocking and to study the explicit and compound effects of foundation rocking and super structural responses, a simplified symmetric plan is selected to minimize the effects of irregularity and torsion. The detailed design parameters are provided in Table 3.4.

3.3.2 Nonlinear structural modelling

It is crucial to model the nonlinear behaviour of shear wall, as it tends to attract more lateral forces than any other members in RC shear wall frame system. Various methodologies (Vulcano et al., 1988; Park and Eom, 2007; Lai et al., 1984; Giberson, 1967; MULAS, 2007) are adopted for simulating the nonlinear response of the shear wall based on the extensive researches. Some of the commonly adopted techniques are broadly grouped into micro models and macro models. In the micro models shear wall elements are discretized into finite number of small elements known as finite elements and each elements are provided with constitutive material models to simulate non-linear responses. Even though micro models provides higher accuracy in simulating the responses, it requires large system equations to solve the problems. As a result of this modelling is complex and requires higher numerical processing efforts and hence not cost efficient computationally. On the other hand macro models simplifies this process and allows fast integration based on the uniaxial material hypothesis and simulates the responses reasonably well. This thesis work primarily focuses the effect of foundation rocking on the response of shear wall rather than simulating failure mechanism of shear wall and hence macro modelling techniques are adopted. The macro level models concentrates on representing the overall behaviour of the shear wall in terms strength and deformation characteristics. There are several macro modelling techniques available which includes wide column method, multi spring method and macro scale multi component model. This thesis work concentrates on the utilization

Table 3.4: Design parameters for the considered structure with shear wall

General	Design level	Shear wall framed structure with response reduction factor $R=5$
	Storey height	3.3m between each storeys
Materials used	Concrete	M40
	Steel	Fe500
Loading details	Dead load	Self weight of structural members
		Weight of slab along with floor finish
		Weight of infills
		Weight of parapet walls in roof
	Live load	3kN/m ²
Structural modelling details for design	Design load combinations	1.5(Dead load+live load)
		1.2(Deadload + live load $\pm$ earthquake load)
		1.2(Deadload $\pm$ earthquake load)
		0.9 Deadload $\pm$ 1.5 earthquake load)
	Software used	ETABS v17
	Model type	3D space frame
	Element models adopted	3D line elements for beams and columns and shell elements for shear wall
		Rigid diaphragms to simulate slab response
		Action of infills not considered
	Plasticity model	Lumped plasticity model
	Geometric nonlinearity	P- δ effects is considered

of macro scale multi component model e.g, Shear Flexure Interaction Multiple Vertical Line Element Model (SFI-MVLEM), since it can capture the nonlinear responses of slender shear walls with better accuracy at global and local levels.

The base model for the SFI MVLEM is derived from the MVLEM proposed by (Vulcano et al., 1988; Orakcal et al., 2006) is a 2D fiber based model. MVLEM model can capture the nonlinear flexural response by providing series of uniaxial materials that consists of boundary elements and panel elements. On the top and bottom of the MVLEM elements rigid beams are provided to simulate the responses of walls at each storey level. The distribution of moment curvature is assumed to be constant for one panel of wall elements. Also the shear and flexural behaviour in MVLEM elements are simulated discretely without considering any coupled behaviour. To overcome the effects such as constant curvature and also to incorporate coupled interactions between shear and flexure SFI MVLEM are derived (Koložvari and Wallace, 2016; Koložvari et al., 2018, 2019). In SFI MVLEM model each uniaxial material in MVLEM element is replaced by macro element with a panel element. The usage of panel element provides the advantage of coupling the axial and flexural

behaviour at the panel level which can be extrapolated to simulate the same responses at the element level. The proposed SFI MVLEM element model assumes the distribution of shear strains are uniform throughout the cross section of wall. This assumption does not significantly affect the responses of slender RC walls and for squat walls this assumption alters the deformation mechanism. Therefore, SFI MVLEM model holds good for mid-rise and high-rise building having the aspect ratio greater than 1. For this proposed thesis work the limitation mentioned above is not applicable and therefore SFI MVLEM is used for modelling the RC shear wall elements.

The panel behaviour mentioned above is directly related to the uniaxial material behaviour of concrete and steel, appropriate material models should be considered. SFI MVLEM utilizes ConcreteCM and SteelMPF constitutive material models for simulating the behaviour of concrete and steel behaviours (Kolozvari et al., 2015). The uniaxial material model provided by Chang and Mander (1994) is utilized for modelling the nonlinear stress strain behaviour of concrete. Since this model is a detailed and non dimensional model which can simulate the hysteresis responses for both unconfined and confined concrete of any strength. This model also accounts for progressive stiffness degradation during unloading and reloading and also tension softening behaviour with a good accuracy. Major parameters are mentioned here in the equations below whereas further details of the model can be found in the referenced report (Chang and Mander, 1994).

$$E_c = 8200 (f'_c)^{\frac{3}{8}} \text{ Mpa} \quad (3.12)$$

$$\epsilon'_c = \frac{(f'_c)^{\frac{1}{4}}}{1150} \quad (3.13)$$

$$r = \frac{f'_c}{5.2} - 1.9 \quad (3.14)$$

where:

E_c denotes Young's modulus of concrete

f'_c denotes peak strength of concrete in compression

ϵ_c denotes peak strain of concrete in compression

Tension stiffness parameters are derived using the following relationships

$$E_t = 3875 (f'_c)^{\frac{1}{2}} \text{ Mpa} \quad (3.15)$$

$$\epsilon_{cr} = 0.00008 \quad (3.16)$$

$$f_{cr} = 0.31 (f'_c)^{\frac{1}{2}} \text{ Mpa} \quad (3.17)$$

where:

E_t denotes Young's modulus of concrete in tension

f_{cr} denotes tensile cracking strength of concrete

ϵ_{cr} denotes average tensile strain of concrete in tension

Considering the symmetry of the structure and as well to concentrate on the effect of foundation rocking only on the shear wall element two dimensional RC frame is considered for modelling as shown in Figure 3.12. The RC shear wall element is modelled using two equal length element for one storey level using SFI MVLEM elements. Whereas for horizontal discretization six elements is selected for all types of building to maintain the uniformity. Out of six elements the elements on outer most boundaries are considered as boundary elements and the remaining four elements are considered for plane section. The material models are chosen as per the material models mentioned above and further details of the material models and other modelling related parameters are utilized as per Kolozvari et al. (2015). RC frame elements i.e., beams and column elements are modelled as per the subsection 3.2.2.

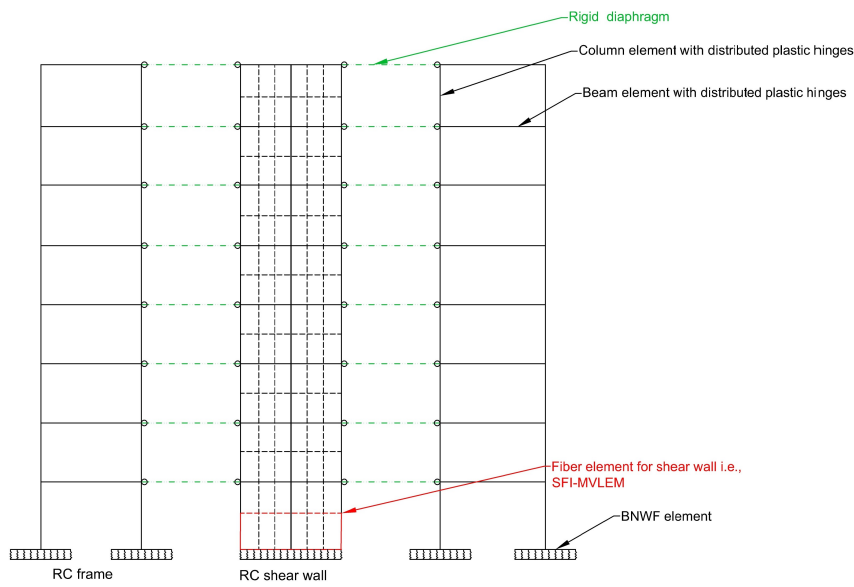


Figure 3.12: RC shear wall frame modelling representation

3.3.3 Design parameters and Nonlinear modelling of substructure

Isolated footings were proportioned for dead loads, live loads, and seismic loads in accordance with the capacity design concept subsection 3.2.4. The eccentricities are kept to a minimum for CDF, typically no more than one-sixth the width of the footing. Relaxing the eccentricity requirement results in an under designed footing width for the shear wall member.

For RC frame building with shear wall, CDF for column elements are designed as square footings and foundation for shear wall elements as rectangular footings by taking into account 100% of the earthquake loads, as well as dead and live loads coming from the superstructure denoted as EL-100. In these sets of analysis, the foundation for the column members is designed as CDF only, whereas only shear wall footing alone is under proportioned to utilize the advantageous effects of foundation rocking. It is imperatively decided to minimize the damages in the shear wall, as shear wall being rigid and slender member attracts more than 80% of the lateral forces generated during seismic motions. To

achieve the rocking of foundation the earthquake loads are reduced in the order of 20% till it reaches the 0% i.e., the foundation is designed only for dead loads and live loads coming on to the footing. The current study uses the fundamental notation of EL-C% (Earthquake Load-Considered for shear wall foundation design in %, where C varies from 100 to 0) to represent the foundation typologies. For instance, EL-0 means the magnitude of earthquake load considered for the design of shear wall foundation is 0%. The nonlinear modelling techniques for modelling the combined foundation and underlying soil medium are adopted as per subsection 3.2.4

3.4 Summary

This chapter primarily examines the analytical modeling techniques employed in this thesis work. The utilization of distributed plastic hinges in fiber-based modeling techniques is increasingly acknowledged as the most efficient approach for achieving reliable superstructure modeling. Besides, extra attention is directed on the potential impacts of integrating geometric and material nonlinearities. Furthermore, within the context of substructural modeling, the BNWF approach is selected and its capacity to accurately comprehend the nonlinear impacts arising from the interaction between soil building is examined. In addition, a validation study is undertaken through an intricate method of selecting an experimental model from the available literature. This investigation aims to demonstrate the effectiveness of the methodologies under consideration in predicting structural responses. The validation provided a favorable outcome, indicating the confidence to employ the procedures for more rigorous analysis. In the following chapters, any specific assumptions employed throughout the analysis will be specified, if relevant, in the respective chapter.

Parametric investigation on seismic response of rocking foundations supporting RC-framed building

4.1 Overview

This chapter aims to comprehensively examine the impact of a rocking foundation on the explicit and combined inelastic behavior of both the rocking foundation and a multistorey RC frame. This chapter also investigates the feasibility of utilizing a rocking foundation as a substitute for a conventional foundation. The seismic performance of an eight and ten storey reinforced concrete (RC) frame with foundation rocking is compared to its conventionally designed footings and fixed base counterpart. The comparison is based on seismic force and displacement demands obtained from nonlinear static analysis. In addition to that seismic moment transferred from column to foundation, resulting permanent settlement at the base of the footing are observed by conducting nonlinear dynamic time history analysis using suite of ground motions. The impact of independent variables, such as soil type, footing width, and structure height, on the overall seismic performance is also assessed for both type of analysis.

4.2 Description of design details

Two sets of building viz., 8-storey and 10-storey building are designed as per the design procedures outlined in subsection 3.2.2. The design column size for both the building are kept to be 450mm x 450mm throughout the height of the structure. Similarly, for beams the section sizes are taken as 400mm x 450mm irrespective of the structure type. Isolated footings are proportioned and modelled as per the procedures that are mention in subsection 3.2.3 and subsection 3.2.4. For RC bare framed building, CDF for column elements are designed as square footings to limit the foundation uplift by taking into account 100% of the earthquake loads, as well as dead and live loads coming from the superstructure denoted as EL-100. Both 8-storey and 10-storey building are designed with an increasing degree of foundation rocking by increasing foundation flexibility. Three sets of rocking foundations are considered by monotonically decreasing the size of the foundation, thereby increasing foundation flexibility. To incorporate the increasing rocking effect of footing, earthquake load considered for the footing design alone is reduced from 100% (denoted as EL-100) to 75% (denoted as EL-75) and 50% (denoted as EL-50). Dead load and live load from the superstructure are not reduced for footing design since the

serviceability of the superstructure and supporting footing should not be affected at any cost. It is observed that beyond 50% of reduction in earthquake load, the footing dimensions are insufficient to carry the dead loads and live loads from the superstructure. The details of the foundation sizes are mentioned in Table 4.1.

Table 4.1: Design parameters of substructure

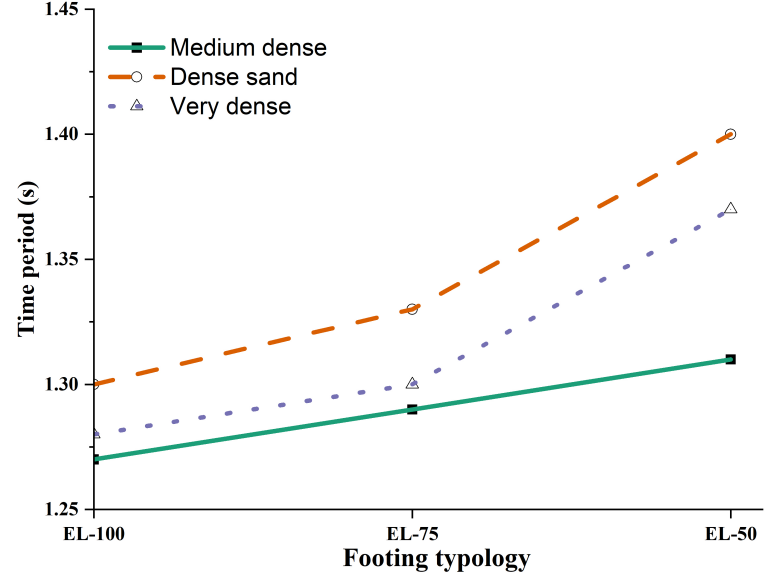
Soil type	Footing Dimensions (m)	Footing tag	Moment to shear ratio	Factor of safety against bearing
Medium dense sand	4.0m x 4.0m	CFD	4.19	9.3
	3.5m x 3.5m	SR	3.89	6.8
	3.0m x 3.0m	HR	3.16	4.7
Dense sand	2.5m x 2.5m	CFD	3.5	7.6
	2.0m x 2.0m	SR	3.3	4.6
	1.5m x 1.5m	HR	3.1	2.5
Very dense sand	2.5m x 2.5m	CFD	3.5	16.7
	2.0m x 2.0m	SR	3.3	10.6
	1.5m x 1.5m	HR	3.1	4.5

4.3 Influence of foundation rocking on fundamental natural period

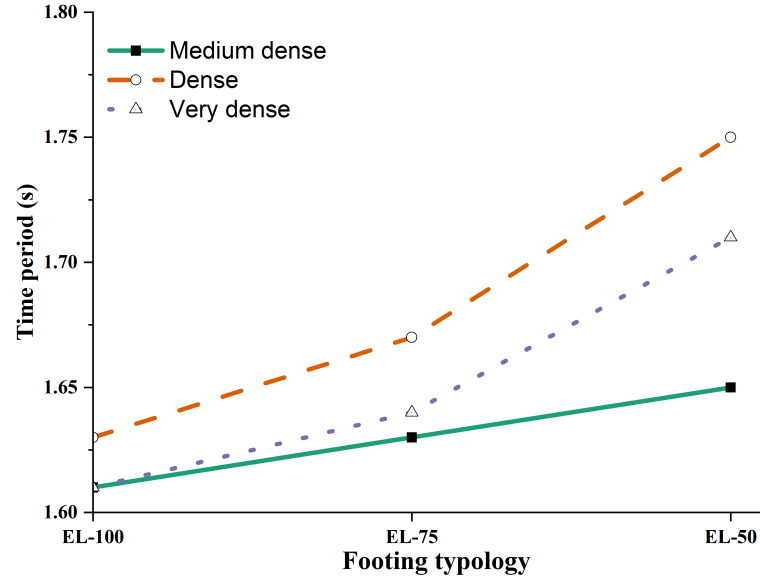
The natural fundamental period of a structure is a critical parameter that significantly influences seismic performance. The Indian seismic design standards (IS1893, 2016) provides an empirical equation, referred to as (Eq.1), which can be used to estimate the natural fundamental period (T_a) of building based on the building height (h) measured from the ground level.

$$T_a = 0.075h^{0.75} \quad (4.1)$$

By applying Equation 4.1, the fundamental natural period of 8-storey and 10-storey RC frame building, without accounting for the effects of infills, according to Indian seismic regulations, is determined to be 0.87s and 1.03s, respectively. Typically, the foundation is considered to be fixed for all design-related purposes. Nevertheless, the flexibility of the foundation and underlying soil has a considerable impact on the natural fundamental period of the structure. In order to investigate the impact of soil flexibility on the natural fundamental period, series of Eigen analyses are conducted. These analyses involve altering the degrees of foundation flexibility by varying foundation sizes and comparing the results with fixed-based counterparts. The comparison is illustrated in Figure 4.1a and Figure 4.1b.



(a)

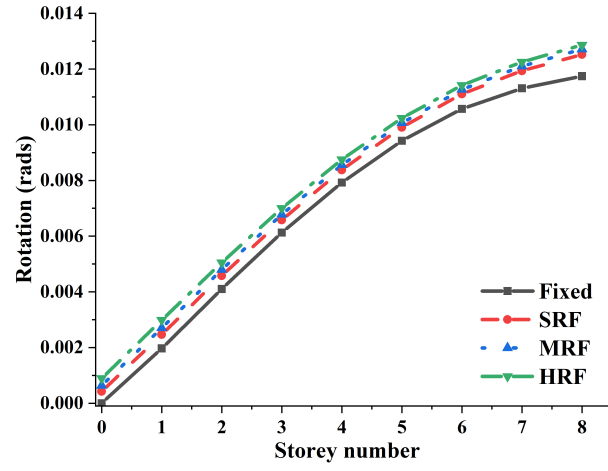


(b)

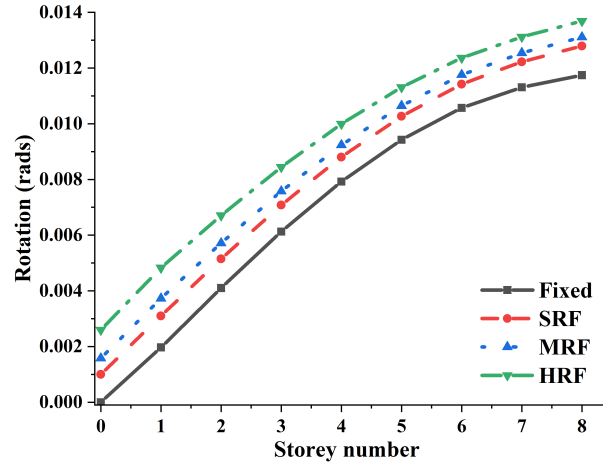
Figure 4.1: Influence of rocking foundation on the fundamental natural period of the (a) 8-storey and; (b) 10-storey building

The degree of period lengthening is quantified as a percentage and is determined by comparing the natural fundamental period with a rocking foundation to the conventional design period as base period. Figure 4.1 demonstrates that the natural fundamental period of the structure increases as the influence of foundation rocking becomes more prominent. This lengthening of period ranges from 45% (with EL-75 foundation) to 50% (with EL-50 foundation) for building on medium dense sand. In addition, for 8-storey building on dense sand and very dense sand increases by around 49% to 60% and 47% to 57%, respectively. For 10-storey constructions, the extent of period lengthening ranges from 56% to 60% for medium dense sand, 58% to 69% for dense sand, and 56% to 66% for very dense sand. The

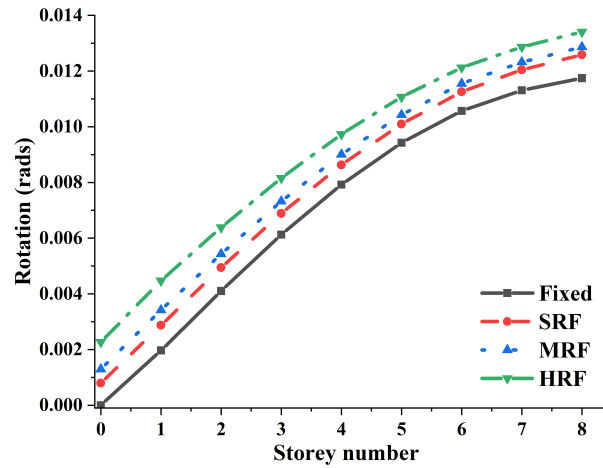
increase in the fundamental time period is accompanied with the decreased footing stiffness and increased rotation as shown in Figure 4.2



(a)



(b)



(c)

Figure 4.2: Influence of foundation rocking on the foundation rotation resting on (a) medium dense sand; (b) dense sand and;(c) very dense sand

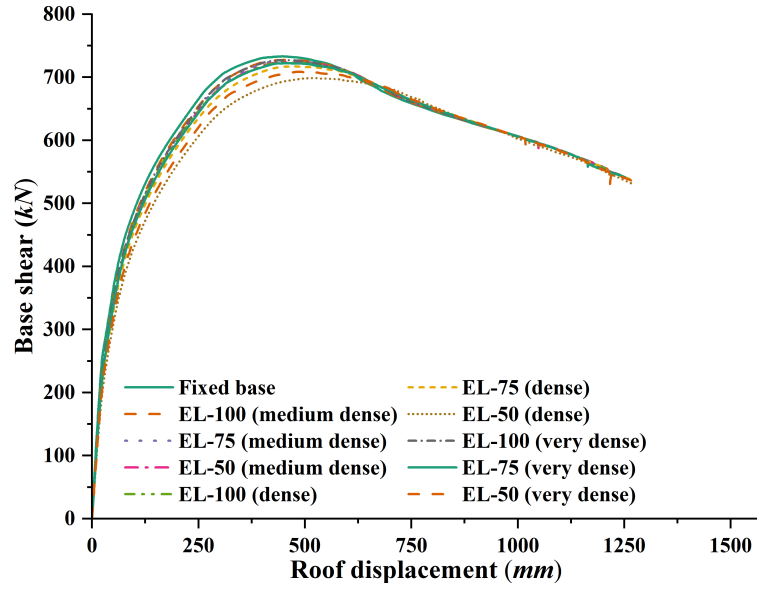
The variation of the natural fundamental period is determined by the relative stiffness of the foundation and soil. When the underlying soil is loosest state then the range of lengthening is lower when compared to dense and very dense sand. On the hand dense sand shows significantly higher period lengthening than the very dense sand eventhough the foundation size is same. This due to the fact the increased stiffness in very dense sand relatively makes structure more rigid than the dense sand.

4.4 Influence of foundation rocking on seismic performance

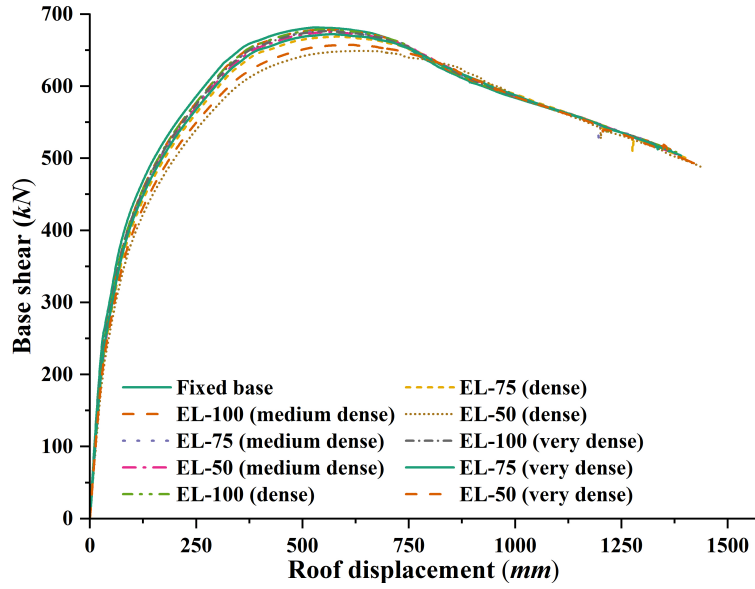
Nonlinear static pushover analyses are conducted on the building under consideration in order to evaluate their capacity, considering the lack of severity of higher mode effects FEMA356 (2000). A comparison is made between the outcomes in terms of capacity curves derived from the nonlinear static pushover analyses with respect to the strength, ductility, and plastic displacement capacity of the building. Figure 4.3a and Figure 4.3b illustrate the capacity curves of 8-storey and 10-storey RC frame building, respectively. The curves compare the performance of conventionally designed footings, rocking footings, and fixed base counterparts. From the capacity curves, it can be noticed that nearly all curves align in the elastic range until the roof displacement is below 100 mm. Like the initial pattern, the capacity curve reaches a convergence point after the roof displacement reaches 600 mm, regardless of the specific structural and foundation types. Nevertheless, there is a noticeable divergence in the roof displacement ranging from 100 mm to 600 mm. In order to gain a more comprehensive understanding of the behavior, a bilinear idealization is employed for each capacity curve to accurately calculate characteristics such as yield strength, yield displacement, ductility, and plastic displacement.

Figure 4.4a and Figure 4.4b displays the yield strength and displacement values for all the building under consideration. Figure 4.4a clearly shows a little decrease in the yield strength of both conventionally designed footings and rocking footings, in comparison to fixed base building. However, the yield displacement increases drastically as the influence of foundation rocking increases. The yield displacement of both 8-storey and 10-storey building, which have fixed base foundations in dense sand, increases from 12% to 34% as the foundation rocking increases from EL-100 to EL-50. Similarly, the amount of displacement experienced by the building increases from 10% to 23% for the very dense sand, regardless of whether they are 8-storey or 10-storey building. In addition, the yield displacement for the medium dense sand increases from 9% to 14% for both 8-storey and 10-storey constructions. In general, the foundation sizes for medium dense soil will be wider due to its reduced bearing ability compared to dense and extremely dense sand situations. Consequently, there is a decrease in the amount of increase in yield displacement.

Figure 4.5 presents a comparison of the peak strength and peak displacement for all the constructions under consideration. The peak strength is seen to decrease when the degree of foundation shaking increases due to greater flexibility. However, the rate of decline in maximum strength is also determined to be at a minimum (0.4% to 4%) for the



(a)

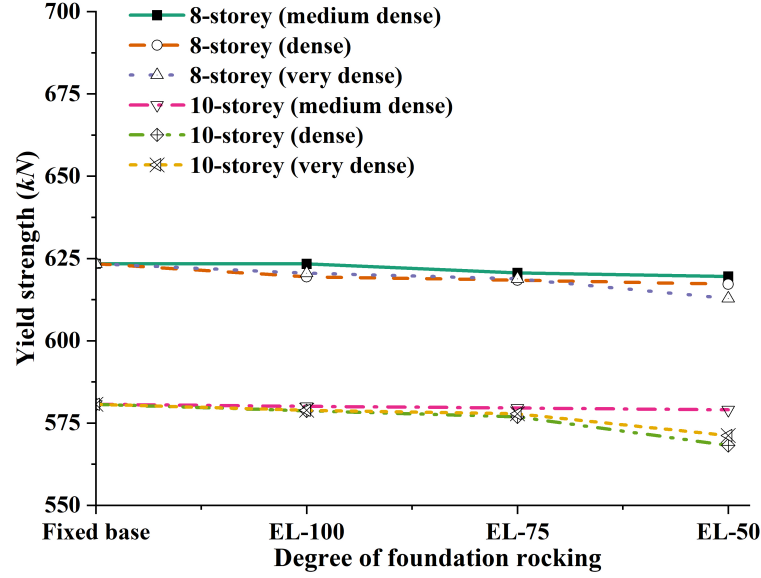


(b)

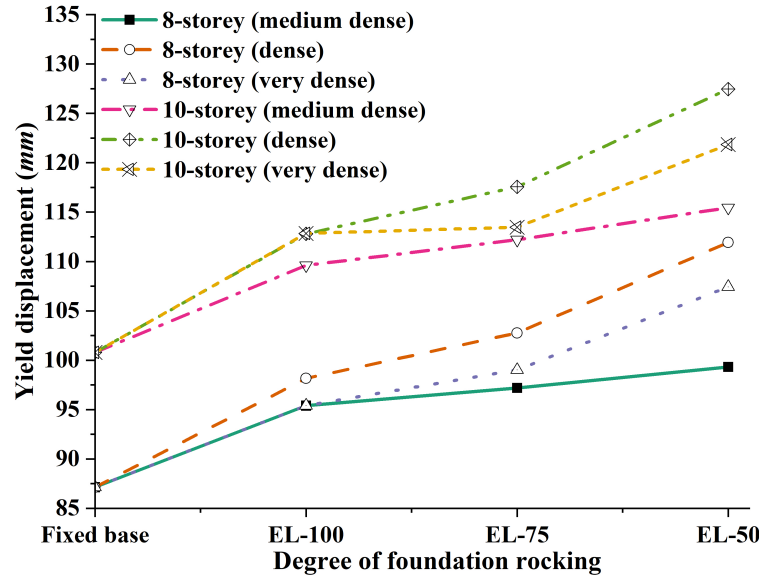
Figure 4.3: Effect of rocking foundation on capacity curves for (a) 8-storey and; (b) 10-storey building

rocking foundations compared to the conventional footings. In contrast, the magnitude of displacement at its peak point grows considerably as the foundation experiences more rocking. The peak displacement exhibits an increase ranging from 3% to 7% for medium dense sand, 3% to 18% for dense sand, and 2% to 10% for extremely dense sand, in both 8-storey and 10-storey structural configurations.

Ductility is a crucial metric that indicates the ability of a superstructure to undergo significant inelastic displacements before collapse. It is determined by calculating the ratio of the ultimate displacement to the yield displacement of the superstructure. This information is graphically depicted in Figure 4.6a. Figure 4.6a clearly demonstrates that



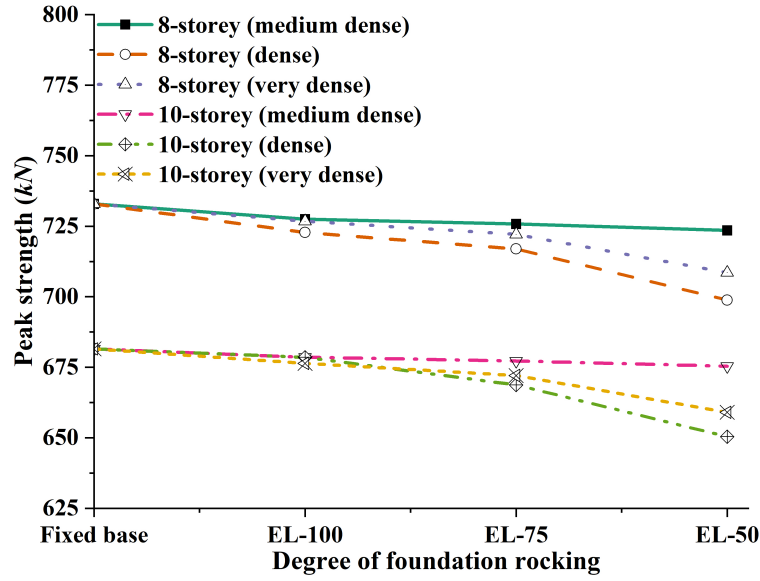
(a)



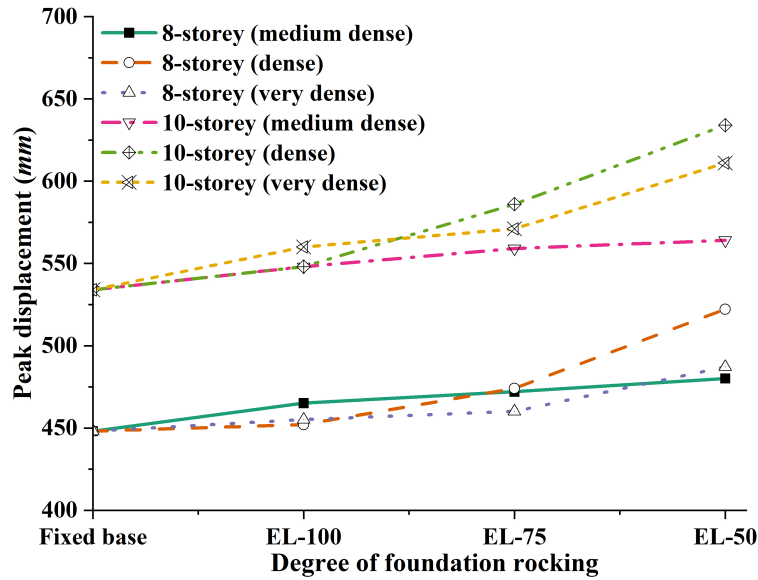
(b)

Figure 4.4: Effect of rocking foundation for 8-storey and 10-storey on (a) yield strength and; (b) yield displacement

the ductility of both 8 and 10 story building decreases as the degree of foundation rocking on any soil type increases. However, ductility alone can occasionally be deceptive and fails to offer an understanding of the structure's ability to undergo inelastic deformation. This is because ductility is quantified as the ratio between the ultimate displacement and the displacement at yield. Figure 4.6b illustrates the plastic displacement capacity of the building under consideration. The plastic deformation capacity is defined as the difference between the ultimate displacement and the yield displacement. The plastic deformation capacity constantly increases with the increased rocking impact, except for building located on medium dense sand. This is because the foundation sizes in medium dense sand are



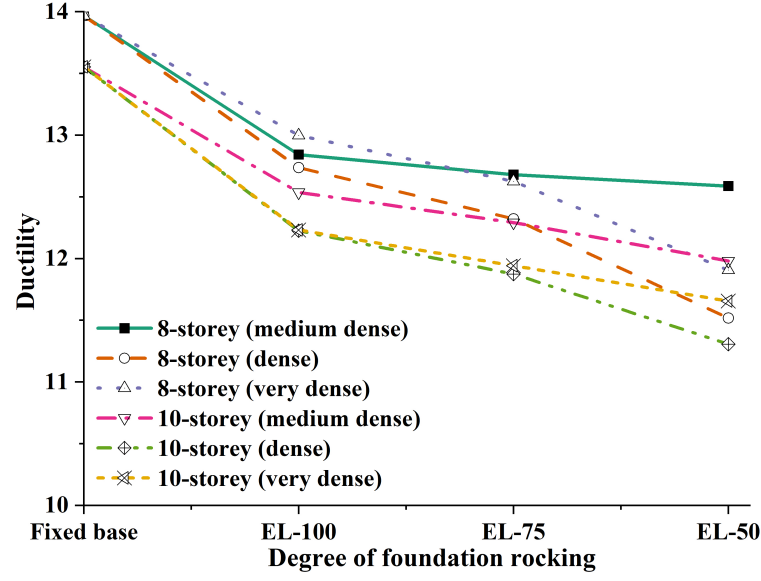
(a)



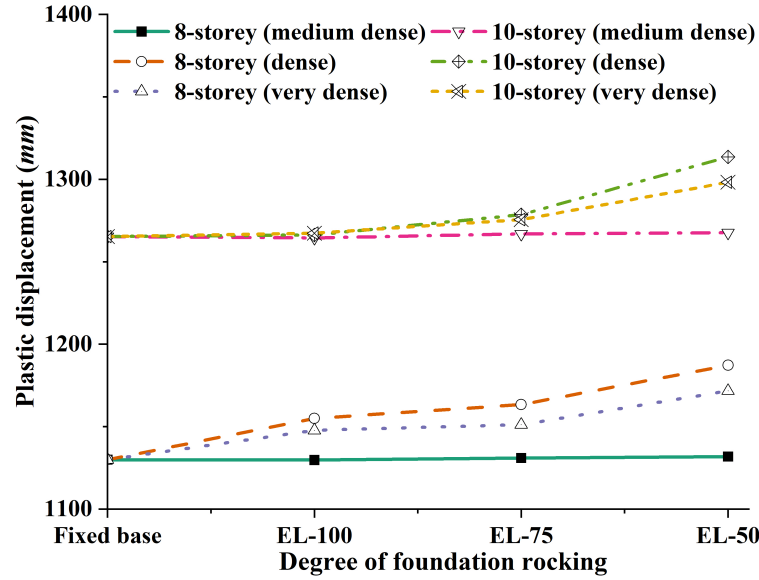
(b)

Figure 4.5: Effect of rocking foundation for 8-storey and 10-storey on (a) peak strength and; (b) peak displacement

relatively big due to the poor bearing capacity compared to dense and very dense sand. It is worth mentioning that while ductility declines as the foundation rocking increases, the yield displacement Figure 4.4b and plastic deformation capacity Figure 4.6b actually increase with the rocking effect. This suggests a desired increase in ductile behavior.



(a)



(b)

Figure 4.6: Effect of rocking foundation for 8-storey and 10-storey on (a) ductility and; (b) plastic displacement

4.5 Influence of foundation rocking under nonlinear seismic analysis

While nonlinear static pushover analysis offers understanding of the general nonlinear reactions of building, it is unable to take into account the degree of complexity that emerges from the dynamic characteristics of earthquakes. Consequently, a set of ground motions listed in Table 4.2 are considered to perform nonlinear dynamic time history analysis and its corresponding spectral acceleration is provided in Figure 4.7. Dynamic analyses are performed using sixteen ground motions that are chosen based on ground motion

parameters and the distance from the fault rupture to the site as per the recommendations provided by [Naeim \(1989\)](#).

Table 4.2: List of ground motion considered for parametric study

S.No	Event	Station	Year	Distance in km	PGA in g	Nature of event
1	Hector	Altadena	1999	166	0.023	Far-Field
2	Hector	Altadena	1999	166	0.037	Far-Field
3	Kobe	Mogirgawachi	1995	24.78	0.13	Far-Field
4	Kobe	Mogirgawachi	1995	24.78	0.21	Far-Field
5	Kocaeli	Arcelik	1999	13.5	0.21	Near-Field
6	Kocaeli	Arcelik	1999	13.5	0.22	Near-Field
7	Imperial Valley	Delta	1979	49.4	0.24	Far-Field
8	Landers	Coolwater	1992	19.74	0.28	Near-Field
9	Imperial Valley	Delta	1979	49.3	0.35	Far-Field
10	Loma Prieta	Gilroy-Array #3	1989	12.23	0.37	Near-Field
11	Landers	Coolwater	1992	19.74	0.42	Near-Field
12	Northridge	Beverly Hills	1994	18.36	0.45	Near-Field
13	Loma Prieta	Gilroy-Array #3	1989	12.23	0.56	Near-Field
14	Northridge	Beverly Hills	1994	18.36	0.62	Near-Field
12	Duzce Turkey	Bolu	1999	12.04	0.74	Near-Field
12	Duzce Turkey	Bolu	1999	12.04	0.81	Near-Field

The earthquake-induced peak acceleration is a crucial factor that influences the seismic performance of building and is highly reliant on the inertial forces encountered by the structural components ([Calvi and Sullivan, 2014](#); [Huang and Lu, 2021](#); [Wang et al., 2021](#)). The transfer of moment from structural components to the foundation under earthquake loading is highly dependent on the acceleration that the structure experiences. Moreover, it is observed that the roof acceleration requirements for building with fixed-base counterparts are approximately three to four times greater than the peak ground acceleration ([Calvi and Sullivan, 2014](#); [Huang and Lu, 2021](#); [Wang et al., 2021](#)). In order to understand the impact of foundation rocking on the maximum roof acceleration, the moment transmitted from the column to the foundation, and the permanent settlement of the foundation, a series of dynamic time history analyses are conducted. The figures, specifically from Figure 4.8 to Figure 4.10, display the peak roof acceleration, moment transferred from column to foundation, and permanent settlement of the foundation for both 8-storey and 10-storey building. These parameters are observed carefully with increasing rocking effect

and compared between conventionally designed footings and fixed base counterparts.

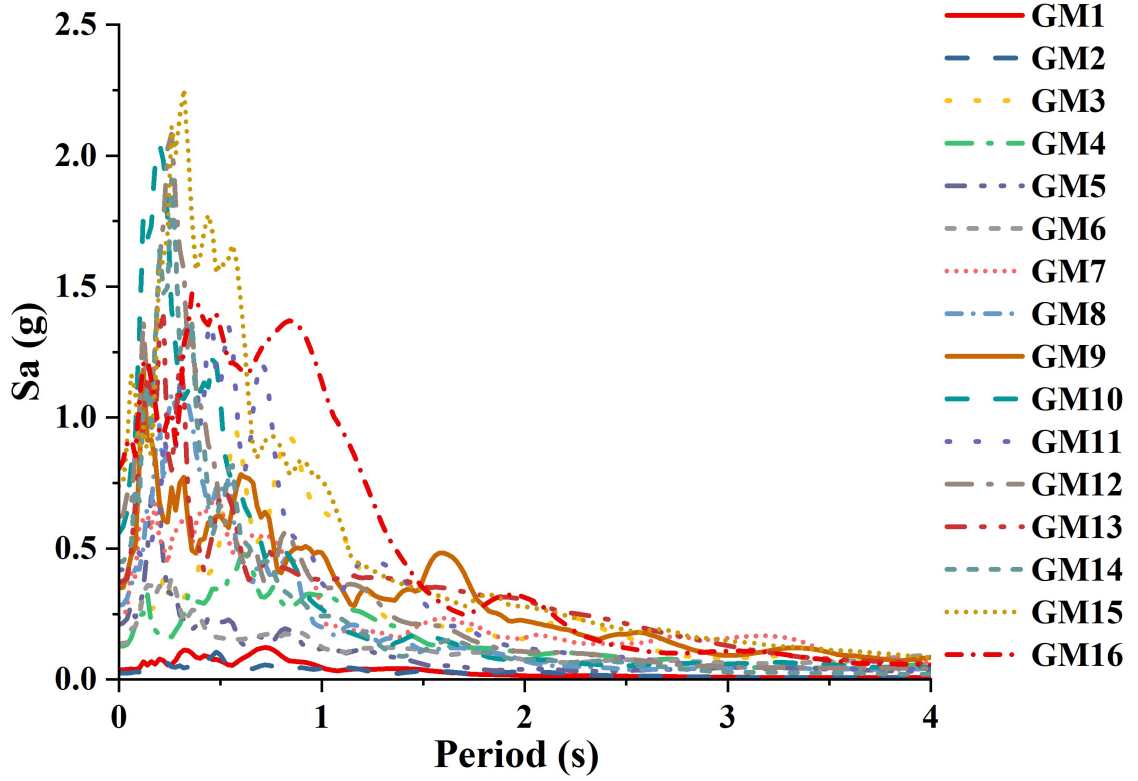
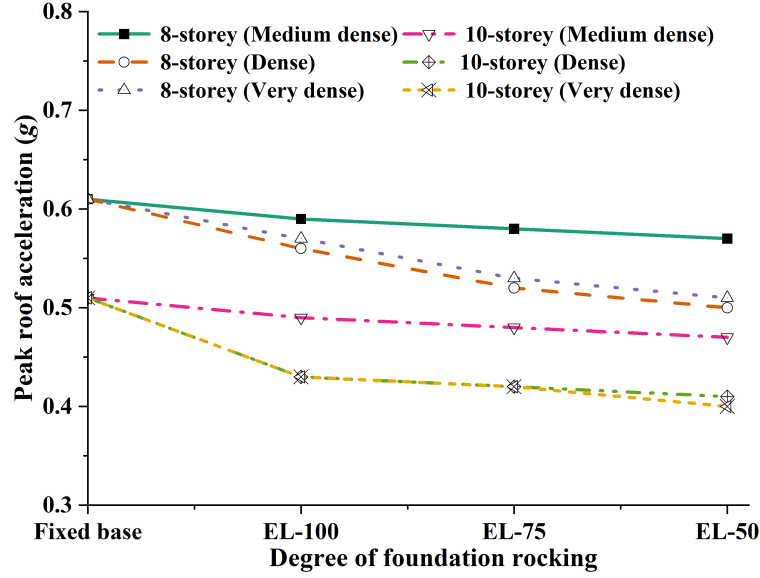


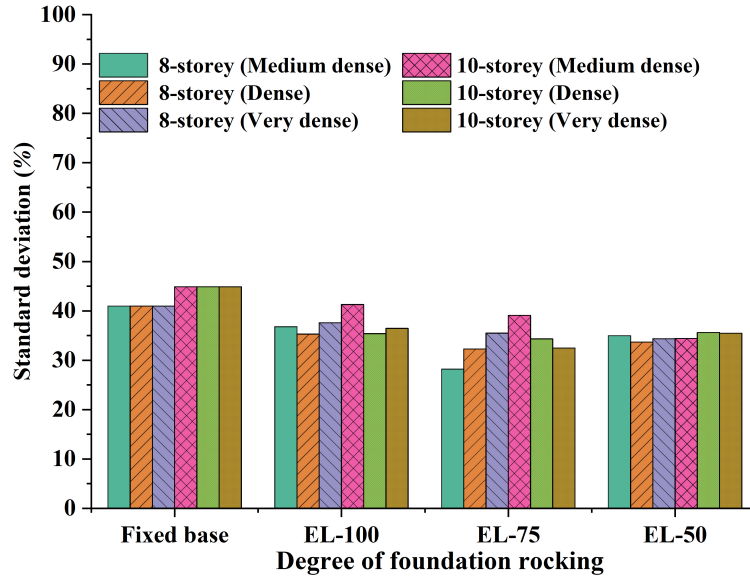
Figure 4.7: Acceleration spectra (S_a) for selected ground motion records for the nonlinear dynamic time history analysis

The data in Figure 4.8 clearly demonstrates that the peak roof acceleration reduces as the influence of foundation rocking increases, in comparison to fixed base counterparts. The peak roof acceleration can be decreased by 3% to 6% when the foundation rocking is increased from EL-100 to EL-50 for both 8-storey and 10-storey constructions on medium dense sand. Similarly, the maximum acceleration experienced at the roof level of building supported by rocking footings might drop by 8% to 20% when built on dense and very dense sand. Due to the uplift of the foundation and the mobilization of bearing capacity, the rocking effect at the foundation increases, resulting in a decrease in the amplification of the acceleration transferred to the superstructure.

The comparison of the peak moment transmitted from the base of the column to the foundation is shown in Figure 4.9. This clearly illustrates that the superstructure supported by a fixed base experiences higher forces compared to the building supported by rocking footings. The impact of foundation uplifting results in a decrease of up to 8% in the moment transferred to the foundation for the rocking footing on medium dense sand, and a decrease of up to 36% for the rocking footing intended for dense and very dense sand, for both 8-story and 10-story building. On the other hand, building with conventional foundations experience greater seismic force demands because these foundations are specifically designed to carry lateral forces caused by seismic activity solely through the superstructural elements. Rocking foundations, which are usually proportioned



(a)

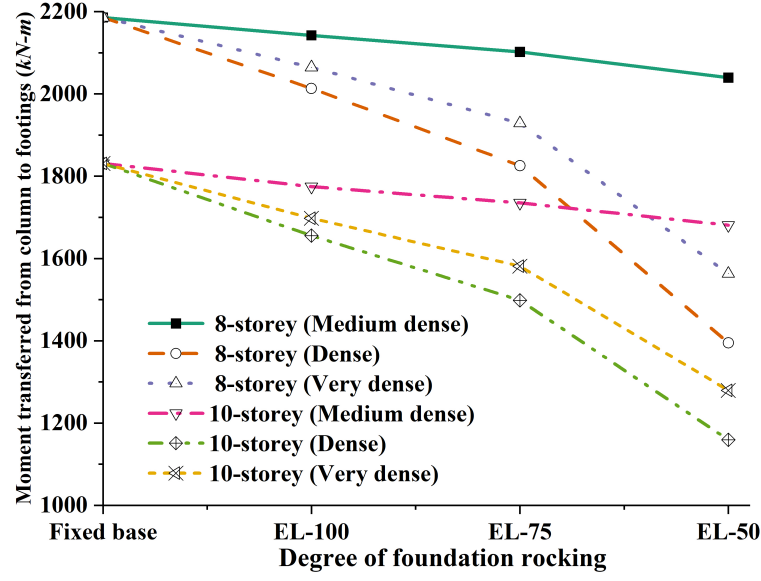


(b)

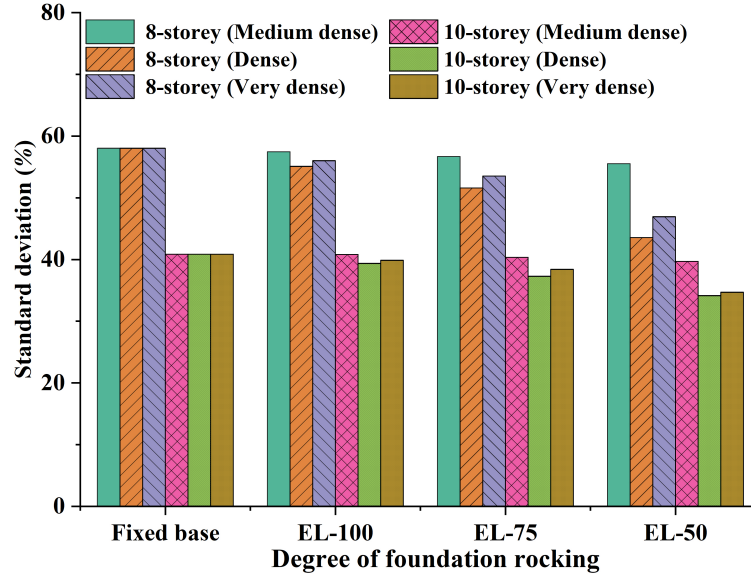
Figure 4.8: Effect of rocking foundation for 8-storey and 10-storey on (a) peak roof acceleration; (b) standard deviation

to have smaller contact area, disperse the seismic forces by utilizing the nonlinear behavior of both the substructural and superstructural parts.

Due to the uplifting of the foundation and the mobilization of bearing capacity, the foundation may experience significant and permanent settlement behaviours (El Naggar et al., 2021, 2022). Therefore, it is important to observe that the permanent settlements caused by rocking should not surpass the permissible thresholds specified by the Indian norms (IS1904, 1986). Figure 4.10 illustrates the comparison between permanent settlement at the foundation level and with increased effect of rocking. Conventionally designed footings, specifically EL-100 footings, settle less than expected due to their larger dimensions.



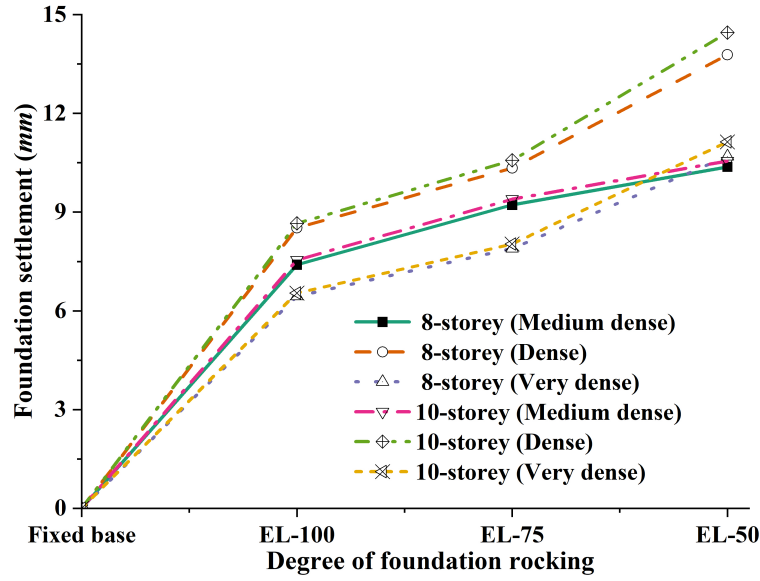
(a)



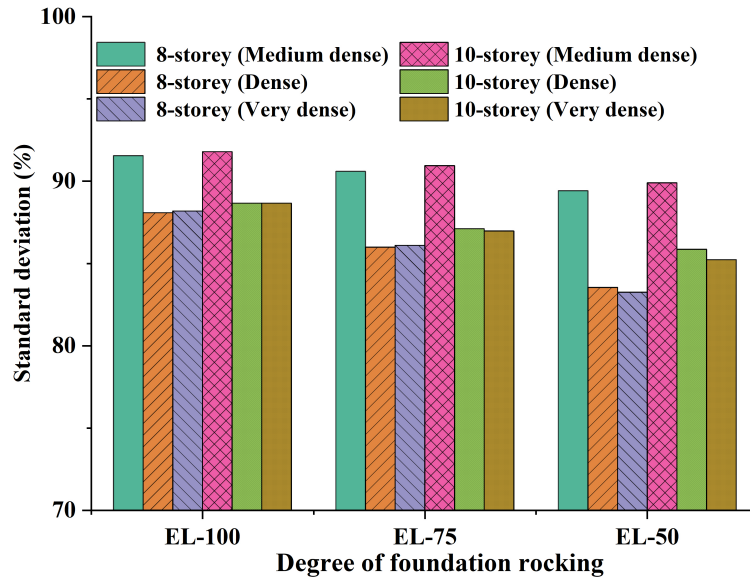
(b)

Figure 4.9: Effect of rocking foundation for 8-storey and 10-storey on (a) peak moment transferred from column to foundation; (b) standard deviation

This settlement is further increased by foundation rocking. The increase in settlement ranges from 17% to 25% for EL-75 footings and from 40% to 80% for EL-50 footings, regardless of the soil condition, for both 8-story and 10-story building. Furthermore, a greater degree of settlement increase is observed in the case of very dense sand, followed by dense sand and medium dense sand. Moreover, the settlement that has been seen is comfortably within the acceptable range of settlement levels as outlined by Indian guidelines (IS1904, 1986).



(a)



(b)

Figure 4.10: Effect of rocking foundation for 8-storey and 10-storey on (a) permanent foundation settlement; (b) standard deviation

4.6 Summary

In this chapter, a rigorous numerical modelling is developed for realistic assessment of the parameters affecting for the seismic performance of building with rocking foundation on different soil types and to compare the responses with the conventionally designed footings. The comparative assessment involves conducting both nonlinear static analysis and nonlinear dynamic time history analysis. Two sets of high rise building are considered for the parametric study. The capacity curves obtained from nonlinear static analyses shows that inelastic responses of superstructure vary with varying foundation rocking.

Increased foundation rocking increases roof displacement at yield and peak by 9% to 34%, with corresponding reduction in yield and peak strength by less than 4%. This suggests that foundation rocking prolongs yielding without reducing its strength. According to the ductility responses, foundation rocking reduces ductility. In contrast both yield and plastic displacement increase significantly with increasing effect rocking, indicating that foundation rocking improves the ductility and overall seismic performance of the structure. In addition, earthquake-induced peak rock acceleration, moment transferred from column to foundation, and permanent foundation settlement are examined using nonlinear dynamic time history analysis. The peak roof acceleration demands drops by 3% to 7% for building on medium dense sand and by 20% for building on dense and extremely dense sand as foundation rocking increases. With increasing foundation rocking, peak moments transferred to the foundation are lowered noticeably. The reduction ranges from 20% to 50% for EL-100 to EL-50 footings. Compared to conventional footings, foundation rocking raises peak permanent settlement by 80%. However, the highest magnitude of permanent settlements found are within tolerable limits as specified by Indian standards. The dense and very dense sand provide the most favorable conditions for rocking foundations in comparison to the medium dense sand.

Effect of embedment depth on seismic response of rocking foundation supporting RC framed structures

5.1 Overview

The aforementioned studies from chapter 2, indicate that the behavior of rocking foundations is influenced by several structural parameters, including aspect ratio, stiffness, and period of structure. Additionally, soil typologies such as soil type, relative density, and cohesion, as well as ground motion characteristics like excitation period, seismic action intensity, and seismic motion direction, also play a role in determining the response of rocking foundations. However, the majority of the aforementioned research primarily concentrate on shallow footings situated at ground level. A scarcity of research exists about the impact of embedment depth on shallow rocking footings. This chapter primarily focuses on evaluating the dynamic responses of rocking shallow foundations on RC framed building without shear wall. The assessment on the influence of rocking foundation involves the consideration of various distinct aspects, including soil type, footing width, and embedment depth, in conjunction with seismic characteristics. A comparative study is made based on the parameters such as the fundamental natural time period of the structure obtained from eigen analysis along with the roof displacement, induced seismic moment at the column base, and foundation settlement for conventionally designed footings and rocking footings obtained from nonlinear seismic analysis.

5.2 Description of design details

The design parameters provided in subsection 3.2.1 is adopted for this chapter and also the column, beam sectional dimensions and foundation proportions are also kept same as that of section 4.2. In contrast, the embedment depth of the footing is varied from minimum embedment depth to the maximum embedment depth to assess the effectiveness of rocking foundations with varying depths. The minimum depth of embedment is taken as 0.5m as per the compilant Indian standards ([IS1904, 1986](#)). Similarly maximum embedment should be less than the width of the footing for shallow foundation consideration. Hence 2.5m is chosen to be maximum embedment depth. Between 0.5m to 2.5m embedment depth is increased by 0.5m and its corresponding seismic responses are estimated for the considered superstructure. The moment to shear ratios are calculated as per [Gajan and Kutter \(2008a\)](#).

Table 5.1: Design parameters of substructure

Soil type	Footing Dimensions (m)	Footing tag	Moment to shear ratio	Factor of safety against bearing
Medium dense sand	4.0m x 4.0m	CFD	4.19	9.3
	3.5m x 3.5m	SR	3.89	6.8
	3.0m x 3.0m	HR	3.16	4.7
Dense sand	2.5m x 2.5m	CFD	3.5	7.6
	2.0m x 2.0m	SR	3.3	4.6
	1.5m x 1.5m	HR	3.1	2.5
Very dense sand	2.5m x 2.5m	CFD	3.5	16.7
	2.0m x 2.0m	SR	3.3	10.6
	1.5m x 1.5m	HR	3.1	4.5

5.3 Influence of foundation rocking on fundamental natural period

The distribution of base shear in the structural components relies on the fundamental natural period of the structure, which is a crucial dynamic property. According to Figure 5.1, the fundamental natural period of the structure is much lower when only the fixed base assumptions are considered, compared to, when the footing and underlying soil are taken into account. In medium dense and very dense sand, the fundamental natural period of CFD is comparable to that of a fixed base, with a slight increase of 5%. However, it is lower than the fundamental natural period of CFD in dense sand, which experiences a 7% increase. The increase is much less pronounced for medium dense sand, but for dense and very dense sand, the improvement is quite rapid in the case of the HR footing. Interestingly, the time period of the structure remains unaffected by changes in the embedment depths of the footing.

5.4 Influence of foundation rocking on seismic performance

The response of building to seismic activity is greatly influenced by the ground motions experienced during earthquakes. The chosen ground motions must be able to accurately represent the various intensity levels, frequency content, duration, and other factors that impact the seismic response of the structure. Therefore, this study considers 16 ground motions as mentioned in Table 4.2, which exhibit a range of amplitudes, frequency content, and duration parameters. The seismic responses for all the buildings have been determined through the application of nonlinear time history analysis, following the guidelines of FEMA356 48.

This study assesses the seismic responses, specifically roof drift and roof accelerations,

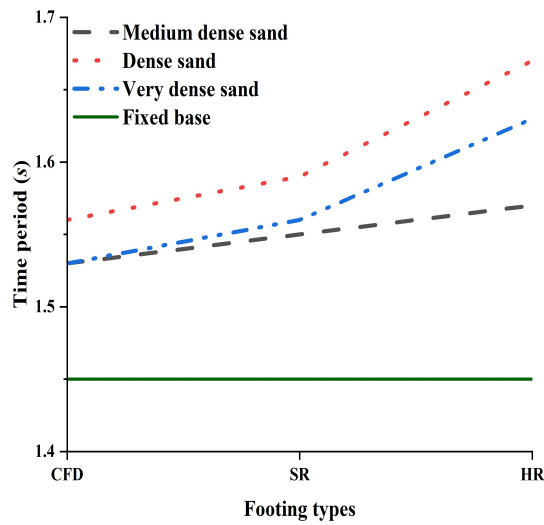


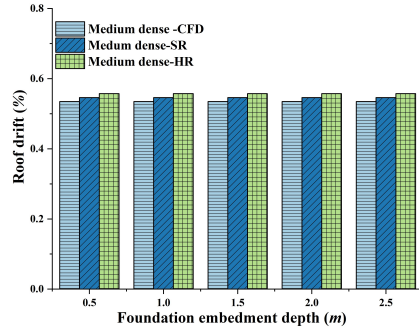
Figure 5.1: Influence of rocking foundation with varying embedment depths on natural fundamental period

of conventionally designed footings and rocking footings. The evaluation considers different embedment depths of footings intended for diverse soil types. Figure 5.2 illustrates the observed maximum roof drift for the ground motions under consideration, taking into account various rocking conditions of the foundation. Based on the data presented in Figure 5.2, it is apparent that the structural elements on the rocking footings exhibit greater peak roof displacements compared to the structural elements with conventional footings. This variation ranges from 0.4% to 0.6% when the foundation size changes from SR to HR.

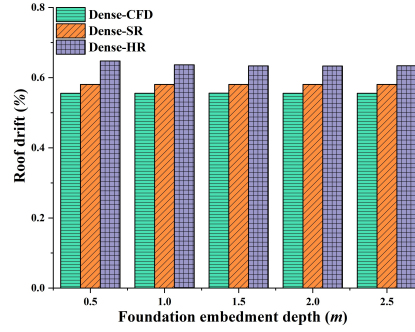
In order to assess the effectiveness of foundation rocking on the inertial response, specifically peak roof acceleration for RC building. Figure 5.3 illustrates the impact on various types of foundations that were taken into account. Figure 5.3 clearly demonstrates that the peak roof acceleration diminishes as the effectiveness of foundation rocking increases. The highest level of reduction is observed at approximately 20% to 25% for building on dense and very dense sand. Nevertheless, the reactions were consistent irrespective of the depth of footing embedment.

As a consequence of foundation uplift, roof displacement is found to increase with increasing rocking effect of foundation. Simultaneously, the roof acceleration transmitted into the superstructure is found to de-amplify, indicating the beneficial responses in the overall behaviour of the superstructure.

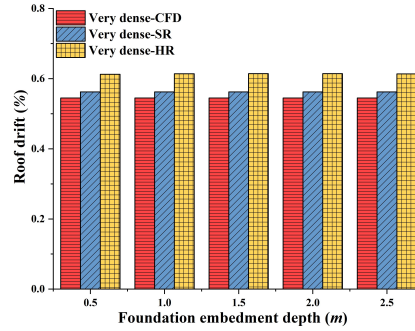
The median responses for the ground motions under consideration, as measured by peak moment imparted to the supporting footings and peak settlement at the foundation level, are illustrated in Figure 5.4 and Figure 5.5. HR footings on dense sand exhibit the most significant decrease in peak moment, with a drop of 35%. This is followed by HR footings on very dense sand, which see a reduction of 15%. In the other instances, namely



(a)



(b)

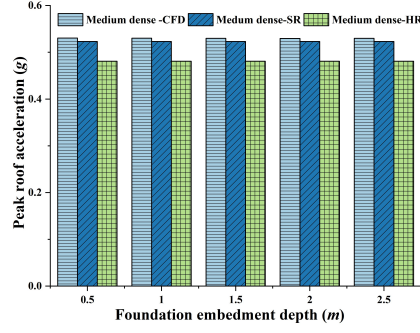


(c)

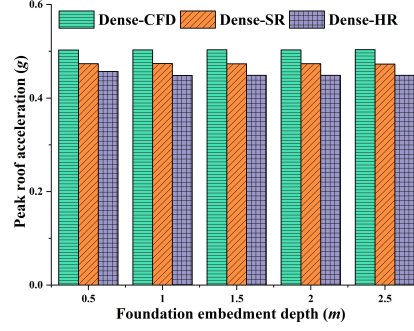
Figure 5.2: Influence of rocking foundation with varying embedment depth on peak roof drift under nonlinear seismic analysis for high-rise structure resting on (a) medium dense sand; (b) dense sand and; (c) very dense sand

for building constructed on medium dense sand with SR footings, the decrease in responses is noticed below 10%. It is also noted that the depth at which the foundation is embedded does not impact the seismic behavior of medium and very-dense sand.

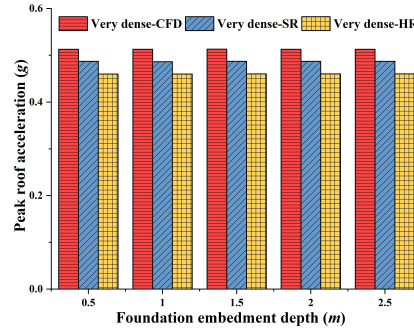
According to Figure 5.5, the settlement at the foundation increases significantly when the foundation rocking is more effective, regardless of the type of soil. Nevertheless, the settlements that have been observed fall within the acceptable range of 50 mm, as specified in [IS8009 \(1976\)](#). However, the settlement responses remained unaffected by the varying embedment depth of the foundation. The settlement at the foundation increases due to the narrower footings, which have lower bearing capacity and are more susceptible to excessive settlement compared to conventionally designed ones during seismic actions. The increase in settlement suggests that the seismic force applied to the RC building is significantly



(a)



(b)

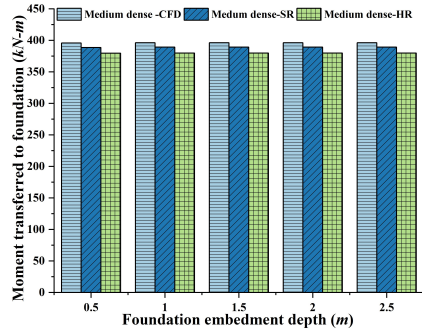


(c)

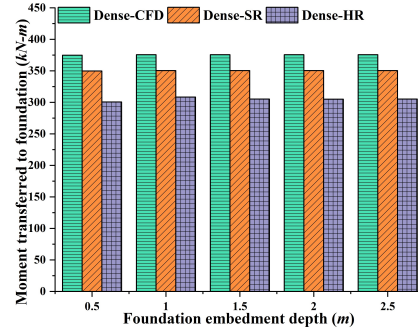
Figure 5.3: Influence of rocking foundation with varying embedment depth on peak roof acceleration under nonlinear seismic analysis for high-rise structure resting on (a) medium dense sand; (b) dense sand and; (c) very dense sand

redistributed to the interface with the soil foundation.

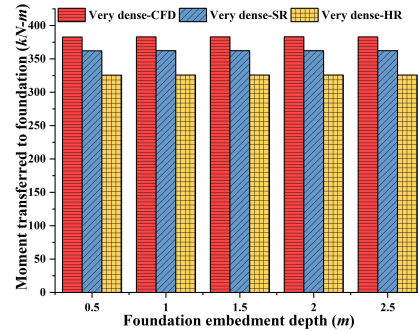
The variance in roof drift (Figure 5.2) and settlement (Figure 5.5) behavior is influenced by the relative stiffness between the foundation and soil. In comparison to dense and extremely dense sand conditions, medium dense soil often exhibits a lesser bearing ability, resulting in larger foundation sizes. It is important to acknowledge that the displacement of the roof is influenced by both the flexural displacement of the structural elements and the rocking displacement caused by the upward movement of the foundation. The utilization of superstructure inelasticity is more pronounced in medium dense sand compared to other soil types, as depicted in Figure 5.4 (peak moment transferred to the foundation from column base). Consequently, in the case of building situated on medium dense sand, the primary factor influencing roof drift is flexural drift, rather than the inherent rocking



(a)



(b)

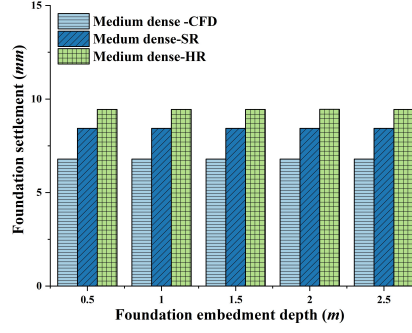


(c)

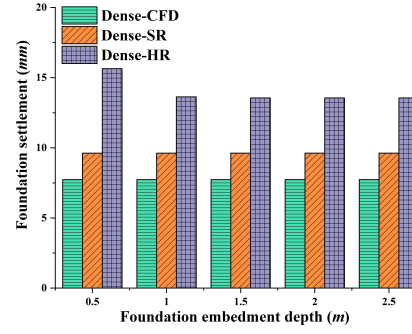
Figure 5.4: Influence of rocking foundation with varying embedment depth on peak moment transferred from column to footing under nonlinear seismic analysis for high-rise structure resting on (a) medium dense sand; (b) dense sand and; (c) very dense sand

characteristics. In contrast, when considering building built on dense sand, it is observed that the foundation size is narrower compared to those on medium dense sand. Additionally, the seismic force demands decrease as settlement increases, as depicted in Figure 5.4. This suggests that the inelasticity of both the superstructure and substructure is effectively utilized. Consequently, the uplifting nature of the footing leads to a significant increase in roof drift. In contrast, building constructed on very dense sand exhibit reduced utilization of substructure inelasticity compared to those constructed on medium dense sand, owing to the increased soil stiffness.

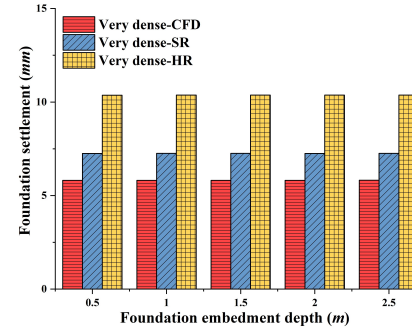
Furthermore, [Hakhamaneshi and Kutter \(2016\)](#) conducted centrifugal investigations on embedded rocking footings, which revealed that the uplift mechanism of the foundation is contingent upon the dilatation characteristics of the soil medium and soil ravelling



(a)



(b)

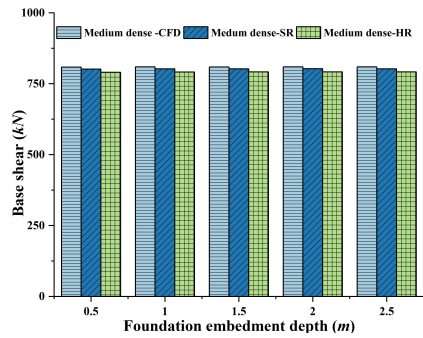


(c)

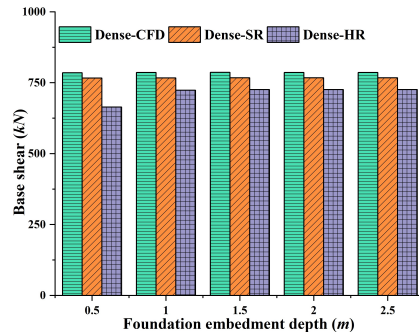
Figure 5.5: Influence of rocking foundation with varying embedment depth on peak settlement under nonlinear seismic analysis for high-rise structure resting on (a) medium dense sand; (b) dense sand and; (c) very dense sand

mechanisms. This work employs the use of BNWF, specifically parallel springs, to simulate the settling and rocking behavior of footings in soil. Therefore, the BNWF cannot able to capture the complex mechanism due to dilatancy and soil raveling (Deng et al., 2012b). Additionally, it is important to acknowledge that the prominence of the foundation rocking mechanism surpasses that of the sliding mechanism, primarily attributed to the higher moment to shear ratio, as indicated in Table 1. Therefore, this study does not examine intricate foundation rocking mechanisms caused by the depth of embedment and its influence on foundation rocking. Hence, it is imperative to conduct a comprehensive nonlinear finite element analysis in order to gain a thorough understanding of the failure mechanism resulting from the depth of foundation embedment, including phenomena such as dilatation and soil ravelling.

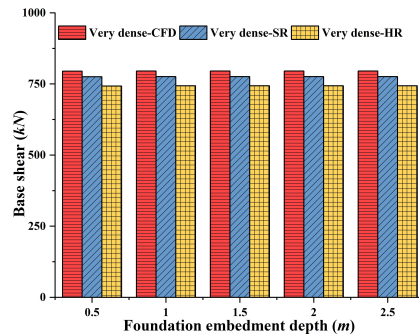
The overall base shear is determined by adding together the base shear observed at each base of all columns. Figure 5.6 displays the base shear for the RC building under consideration, using CFD, SR, and HR. The impact of increasing rocking efficacy and embedment depth on base shear in medium dense sand is not found to be substantial. This is due to the fact that larger foundation sizes are often required for medium dense sand soil types, since they tend to have lesser shear strength compared to dense and very dense sand. In contrast, it can be observed that the base shear decreases as the rocking efficacy increases for highly compacted sand. This again suggests that rocking foundations impose a lower seismic demand on RC building in comparison to CFD scenarios.



(a)



(b)

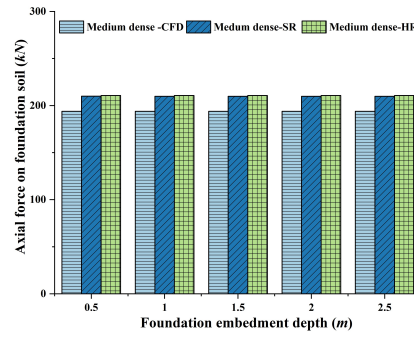


(c)

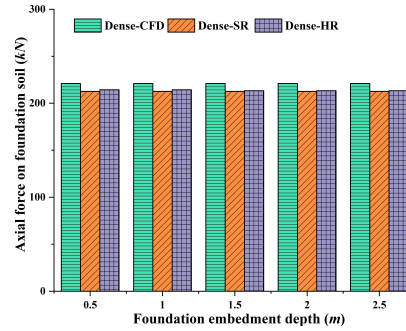
Figure 5.6: Influence of rocking foundation with varying embedment depth on peak base shear under nonlinear seismic analysis for high-rise structure resting on (a) medium dense sand; (b) dense sand and; (c) very dense sand

Figure 5.7 depicts the peak force exerted on the foundation soil, namely in the form of a spring force, near the edge of the footing for the central column. It has been observed that

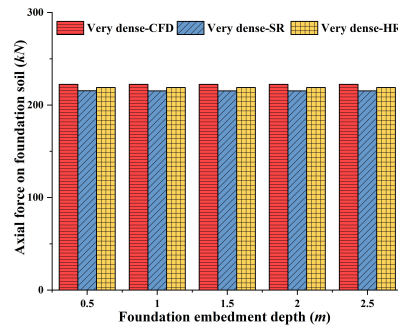
the spring force magnitude exhibits an upward trend when the size of the footing decreases in the case of medium dense sand. The end spring force experiences a 7.5% increase for SR footing and a 3.5% increase for HR footing type. Conversely, the amplitude of the spring force at the end decreases for sand that is dense and very dense. Nevertheless, there is no substantial influence on the reactions when considering different levels of embedment depth in the footing. The increase in rocking effect leads to a corresponding rise in foundation settlement as a result of the foundation uplift. Conversely, there is a notable drop in the maximum moment transferred to the foundation element and base shear at the foundation level, indicating that an increase in the rocking effect leads to a reduction in the overall seismic force requirements of the building.



(a)



(b)



(c)

Figure 5.7: Influence of rocking foundation with varying embedment depth on peak force experienced at soil under nonlinear seismic analysis for high-rise structure resting on (a) medium dense sand; (b) dense sand and; (c) very dense sand

5.5 Summary

The current study aimed to examine the impact of different soil types and embedment depths of footings on the rocking behavior of shallow foundations. The seismic responses indicate that the rocking of the footing leads to a decrease in roof acceleration with increasing displacement demands. This study demonstrates that by underproportioning the footings, the rocking efficacy of reinforced concrete (RC) buildings on dense and extremely dense sand can be increased. This leads to a reduction in seismic force demands and damages caused by lateral forces. There is no significant impact on overall reactions when considering the different embedment depth of the footing. Therefore, it is advisable to do nonlinear finite element modeling of the underlying soil medium in order to monitor the influence of integration depth. Due to the fact that finite element modelling is capable of capturing intricate failure mechanisms linked to the settlement problem, which BNWF modelling falls short of.

Parametric investigation of seismic response of rocking foundations supporting RC shear wall framed structures

6.1 Overview

The seismic response of two-dimensional (2D) RC shear wall framed building supported by a conventional foundation is conducted in this chapter, then the investigation delves deeper into the substitution of the shear wall footing with rocking foundation, in order to highlight the positive impacts of the rocking foundation. This chapter entails conducting analysis on three distinct building with varying heights viz., 4, 6 and 8-storey. To accurately assess the seismic behavior of the structural system, nonlinear dynamic time history analysis is conducted for all the building under consideration. These results are then compared to those obtained from conventional footing and an ideal condition as fixed base building.

6.2 Description of design details

Three sets of building viz., 4-storey, 6-storey and 8-storey building are designed as per the design procedures outlined in subsection 3.3.2. The design details for column, beam and shear wall is given in Table 6.1. The sectional size for beams and columns are kept uniform throughout to avoid the varying stiffness within the super structural elements. The CDF for shear wall foundation is designed to limit the foundation uplift by taking into account 100% of the earthquake loads, as well as dead and live loads coming from the super structure. Later, to achieve the rocking of foundation the earthquake loads are reduced in the order of 20% till it reaches the 0% i.e., the foundation is designed only for dead loads and live loads coming on to the footing. The current study uses the fundamental notation of EL-C% (Earthquake Load-Considered for shear wall foundation design in %, where C varies from 100 to 0) to represent the foundation typologies. For instance, EL-0 means the magnitude of earthquake load considered for the design of shear wall foundation is 0%. On the other hand, the foundation for the frame members is designed as CDF only. Whereas, conventional foundation sizes for shear wall footing designed for zone II is proportioned to be 7m x 3.5m and for zone V is proportioned to be 12m x 9m. For columns square footings are proportioned to behave as conventional footings only and it is found to be 2.75m x 2.75m.

Table 6.1: Categories of foundation proportions considered for this chapter

Design level	Column size	Beam size	Thickness of shear wall	Shear wall boundary element thickness
4-storey ZoneII	300 x 300	300 x 350	250	NA
6-storey ZoneII	300 x 300	300 x 350	250	NA
8-storey ZoneII	450 x 450	400 x 450	300	600
4-storey ZoneV	400 x 400	400 x 450	300	600
6-storey ZoneV	400 x 400	400 x 450	350	875
8-storey ZoneV	500 x 500	450 x 500	350	875

All the dimensions are in mm and NA=Not Applicable

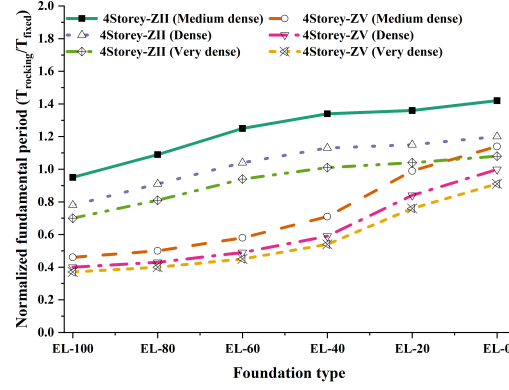
6.3 Influence of foundation rocking on fundamental natural period

Rocking foundations typically extend the fundamental natural period of the structure, and this issue has been handled in the simulated models. Figure 6.1 illustrates the impact of the soil type, seismic design factor, and number of storeys.

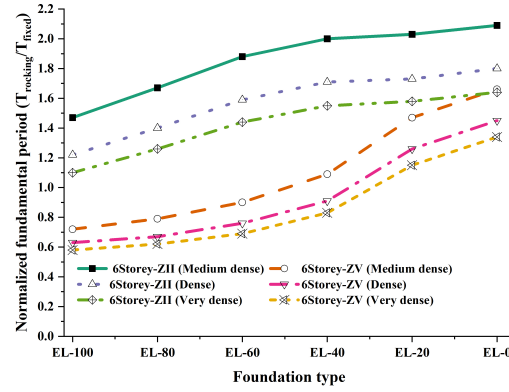
The plot reveals that the duration of the time period is mostly influenced by the stiffness of the soil and foundation. There exists an inverse relationship between the magnitude of the time period and the size of the foundation. Furthermore, it is shown (Figure 6.1) that the constructions intended for seismic zone II exhibit a notable increase in time period from EL-100 to EL-60, ranging from 25% to 40%, after the magnitude of the fluctuation is decreased to approximately 10%. In contrast to the reactions observed in seismic zone II building, zone V building exhibit notable fluctuations in the time period increase following the EL-60 footing scenario. The reason for this is because zone V building are typically proportioned to withstand greater seismic forces, leading to the need for larger footing dimensions compared to seismic zone II building.

6.4 Influence of foundation rocking on seismic performance

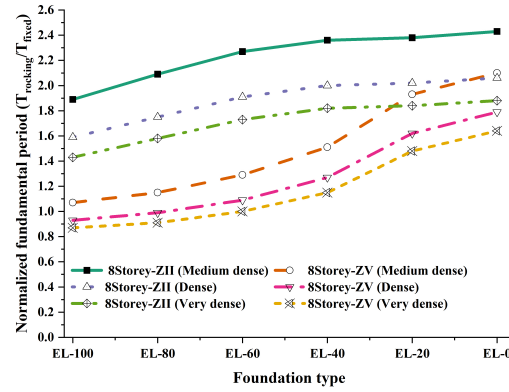
This study involves the random selection of 20 ground motions from the Pacific Earthquake Engineering Research (PEER) database. The selection process is made due to the absence of guidelines for ground motion selection in accordance with Indian norms (IS1893, 2016). The response of the structure is affected by nature of ground motion and is contingent upon the parameters of amplitude, frequency content, duration, and source to site distance. The 20 ground motions listed in Table 2 are categorized into four groups (Naeim, 1989) based on the distance (R) from the source to the site and magnitude (M). These categories are: 1. strong earthquakes in the far field ($R > 20\text{km}$ and $M > 6.9$), 2. major earthquakes in the far field ($R > 20\text{km}$ and $M > 7$), 3. strong earthquakes in the near field ($R < 20\text{km}$ and



(a)



(b)



(c)

Figure 6.1: Effect of foundation rocking on fundamental natural period for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

M>6.9), and 4. major earthquakes in the near field ($R < 20\text{km}$ and $M > 7$).

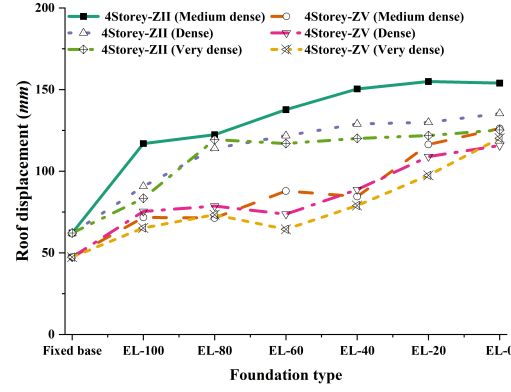
The study presents the median lateral peak roof displacement responses for all structural models evaluated, together with their respective standard deviations (see Figure 6.2). The data suggests that there is a progressive increase in the maximum displacement of the roof as the level of rocking in the foundation increases. For 4-storey and 6-storey building, the fixed base structure has the lowest peak roof displacement, ranging from 0.4% to 0.5% of H (where H represents the entire height of the structure). For 8-storey building, the peak roof displacement ranges from 0.3% to 0.35% of H . The

Table 6.2: Categories of ground motions considered for the study and its parameters

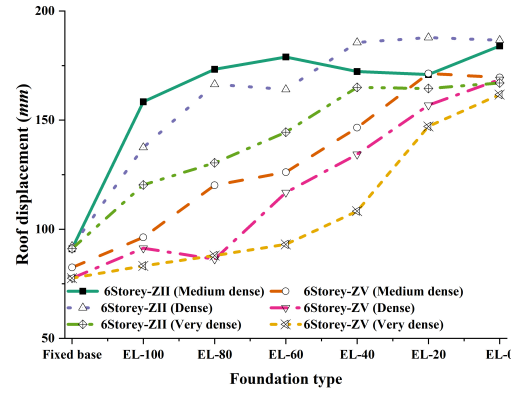
Ground motion category	Event name	Magnitude	Source to site distance R (km)	Peak ground acceleration (g)	Ground motion tag
Far-field strong earthquakes	Imperial Valley	6.5	22	0.24	GM1
	Imperial Valley	6.5	22	0.35	GM2
	Kobe	6.9	24.78	0.13	GM3
	Kobe	6.9	24.78	0.21	GM4
	Morgan Hill	6.19	39.08	0.14	GM5
Far-field major earthquakes	Hector	7.13	41.81	0.18	GM6
	Hector	7.13	41.81	0.15	GM7
	Tabas	7.35	89.76	0.10	GM8
	Kern Country	7.36	38.42	0.16	GM9
	Denali	7.9	42.99	0.11	GM10
Near-field strong earthquakes	Northridge	6.69	12.39	0.45	GM11
	Northridge	6.69	12.39	0.62	GM12
	Loma prieta	6.93	12.23	0.37	GM13
	Loma prieta	6.93	12.23	0.56	GM14
	Christchurch	6.2	12.91	0.15	GM15
Near-field major earthquakes	Duzce turkey	7.14	12.02	0.74	GM16
	Duzce turkey	7.14	12.02	0.81	GM17
	Landers	7.28	19.74	0.28	GM18
	Landers	7.28	19.74	0.42	GM19
	Kocaeli	7.51	10.56	0.21	GM20

structure with foundation designed for EL-0 demonstrates a peak roof displacement that is 2.5 to 4 times more than that of fixed base constructions. The rise in observed peak roof displacement can be ascribed to a reduction in the rigidity of the shear wall footing element. The direct relationship between the reduction in footing size and the loss in stiffness of the shear wall footing element has a direct impact on the flexibility of the structure, leading to an increase in peak roof displacement. building designed for seismic zone II in medium dense sand see a more pronounced rise in peak roof displacement compared to building planned for seismic zone V, for 4-storey and 6-storey building. Irrespective of the design zone factor, the rise in peak roof displacement for 8-storey building remains reasonably stable.

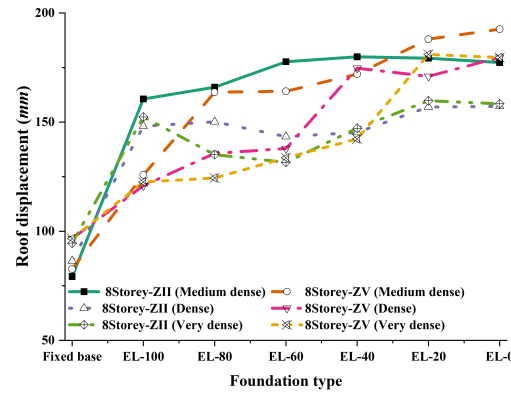
The plot in Figure 6.3 illustrates the median peak roof acceleration at the superstructure level for all the models being examined. The inclusion of the standard deviation allows for the estimation of the range of reactions, taking into account the variations in the seismic motion under consideration. The plot reveals that the fixed base constructions exhibit a



(a)



(b)

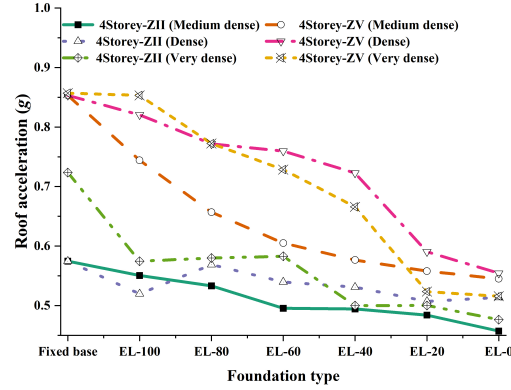


(c)

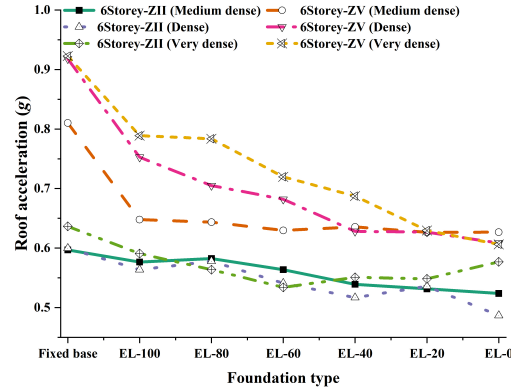
Figure 6.2: Effect of foundation rocking on peak roof displacement for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

peak roof acceleration ranging from 0.75g to 0.9g. In contrast, the rocking footing, which is designed to withstand only operational loads, demonstrates a lower peak roof acceleration of approximately 0.5g to 0.7g. The constructions intended for zone V have a greater extent of decrease in peak roof acceleration, varying between about 15% and 28%, irrespective of the height of the structure. However, this decrease is not significantly noticeable for the four-storey and six-storey buildings specifically designed for zone II. In contrast, the 8-storey building specifically designed for zone II demonstrate a notable decrease in their maximum roof acceleration. The reduction in the maximum acceleration experienced by

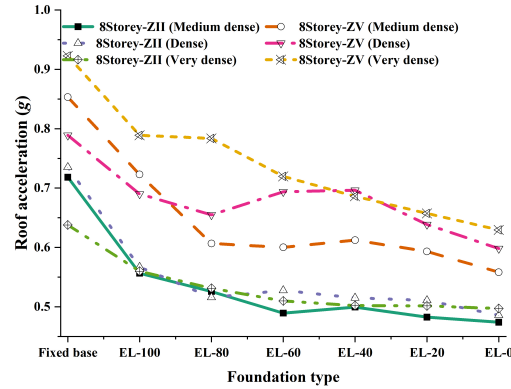
the roof is in line with the lengthening of the initial period due to the increased flexibility of the supporting footing element.



(a)



(b)



(c)

Figure 6.3: Effect of foundation rocking on peak roof acceleration for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

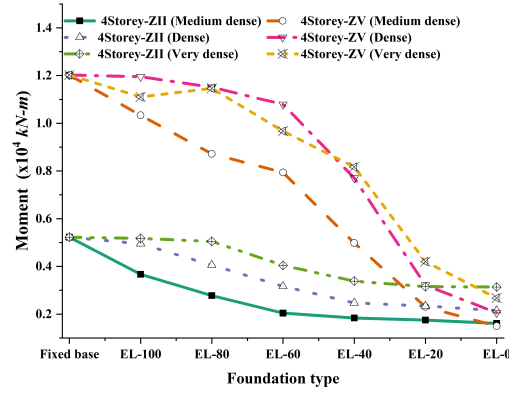
The determination of the seismic force demands of a structure relies on two crucial parameters: the peak base shear of the structure and the peak base moment experienced at the shear wall element. Figure 6.4 displays the median maximum moments exerted at the base of the shear wall, together with their corresponding standard deviation. It is important to acknowledge that the presence of fixed base building exhibits the highest magnitudes of moment at the base of shear walls, followed by conventionally designed foundations and

rocking foundations. Furthermore, the magnitude of the peak base moment is substantially influenced by the relative stiffness of the shear wall and footing. High stiffness in both the shear wall and footing results in the attraction of bigger moments. For example, it is noted that the shear wall foundations intended for seismic zone V leads to increased footing dimensions, which in turn draw more forces compared to foundations planned for EL-0. For 4-storey building supported by EL-0 rocking footing, the induced moment at the base of the shear wall for zone V design is reduced by 3 to 6 times. For 6-storey building, the reduction is about 9 times. In contrast, the degree of decrease for zone II design, when supported by EL-0 rocking footing, exhibits a range of 0.25 to 2.5 times for 4-storey building. Similarly, for 6-storey and 8-storey building, the reduction is roughly 2 to 9 times.

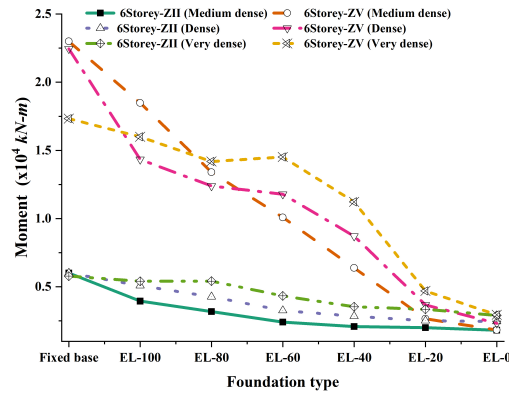
In order to determine the median value of the total base shear, it is necessary to aggregate the base shear values obtained from the base of the column and shear wall, as depicted in Figure 6.5. The implementation of a rocking foundation tends to decrease the maximum base shear experienced by the construction intended for zone V in medium dense sand and very dense sand. On the contrary, for zone II structures the variation is not significantly affected.

During seismic loading, the foundation may experience sliding, settling, and tilting, either alone or in combination. It is important to acknowledge that the topic of settlement and tilting, specifically differential settlement, is examined in this study, while the impact of sliding caused by seismic loading is determined to be insignificant. Figure 6.6 displays a summary of the median maximum settlement observed during seismic shaking, together with its standard deviation shown as a percentage. This information is used to anticipate the lower and upper boundaries. The shear wall on a rocking footing designed for zone V experiences a higher maximum settlement compared to a conventionally designed footing. This increase occurs when the considered earthquake loads for design decreases, resulting in a settlement of up to 20% higher for 4-storey building and 40% higher for 6-storey and 8-storey building. Subsequently, the maximum settlement either closely resembles that of a foundation designed according to conventional methods is negligible. The buildings on medium dense sand exhibit a maximum settlement of around 40mm, which is higher than that reported on dense and very dense sand. According to Indian norms for EL-40, EL-20, and EL-0, the settlement magnitude for the 4-storey building exceeds the allowable limit of 40mm with a factor of safety of 1.5. However, it was discovered that the maximum settlement for zone II planned building on medium dense sand is in close proximity to the permissible settlement for EL-100, EL-80, and EL-60. In contrast, the EL-40, EL-20, and EL-0 foundations exhibited higher settlements for 6 storey and 8 storey building, approaching the allowed limits.

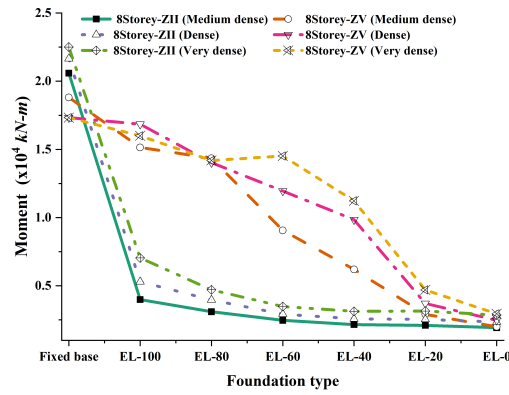
Typically, the foundations will be specifically constructed to accommodate uniform settlements. Differential settlements are more prone to develop in shear wall foundations due to their rocking nature and the uncertainty connected with soil conditions. Differential



(a)



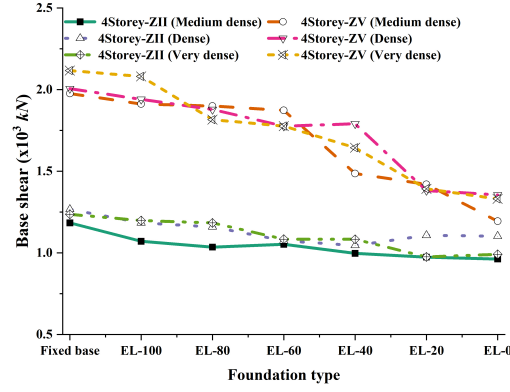
(b)



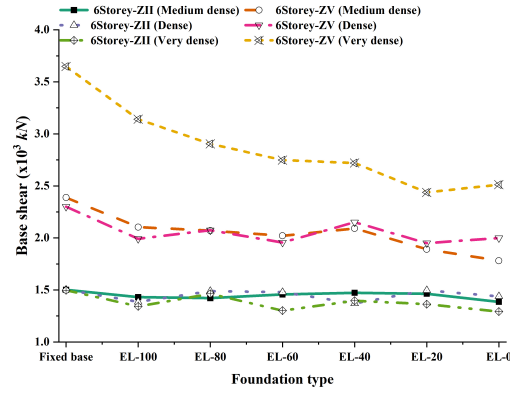
(c)

Figure 6.4: Effect of foundation rocking on moment transferred from shear wall to footing for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

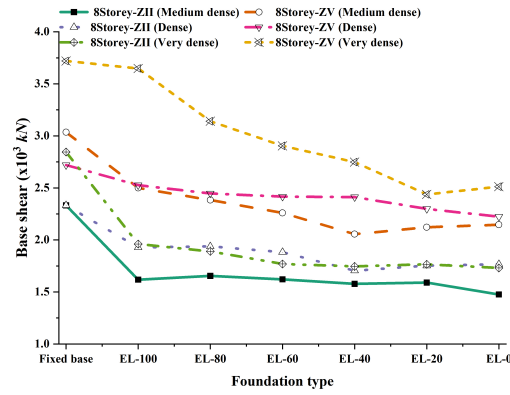
settlements are of greater significance compared to uniform settlements, since they have the potential to induce vertical misalignment, overburden structural members due to distortion, and cracking in walls and adjacent structural elements. Hence, addressing these problems is crucial in order to maintain the prescribed limits of differential settlements as outlined by the Indian norms (IS8009, 1976). The examination of the building intended for seismic zone V on dense and very dense sand reveals that the peak differential settlement recorded (Figure 6.7) suggests that the differential settlement values in the shear wall footing are significantly lower than the permissible limit of 0.002 times the length of the footing. The



(a)



(b)

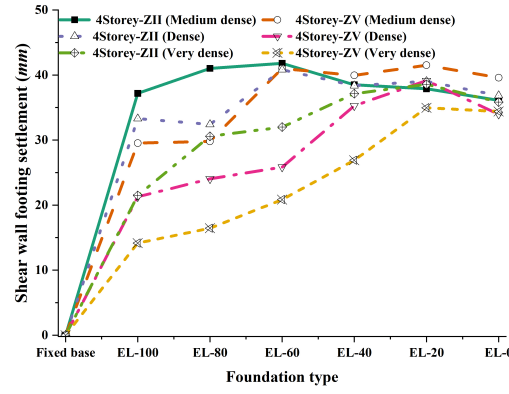


(c)

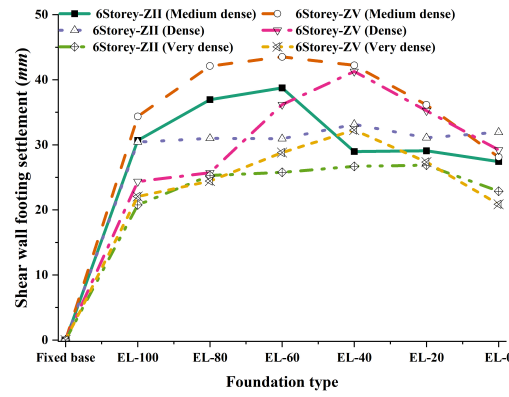
Figure 6.5: Effect of foundation rocking on base shear for (a) 4-storey; (c) 6-storey and; (c) 8-storey building

observed differential settlement values for the foundation of other building are in close proximity to the stipulated limitations, although they do not surpass the permissible limit. Additionally, it is shown that rocking footings exhibited greater differential settlement for shear wall footings compared to conventionally designed footings as expected.

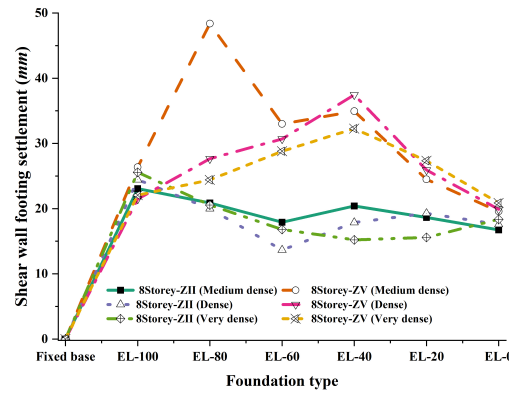
The base shear contribution ratios are determined by isolating the base shear experienced by the shear wall from the other column elements. The analysis of Figure 6.8 reveals that the shear wall exhibits a greater magnitude of base shear contribution. In conventionally proportioned foundations, shear walls (>80%) are primarily responsible for



(a)



(b)

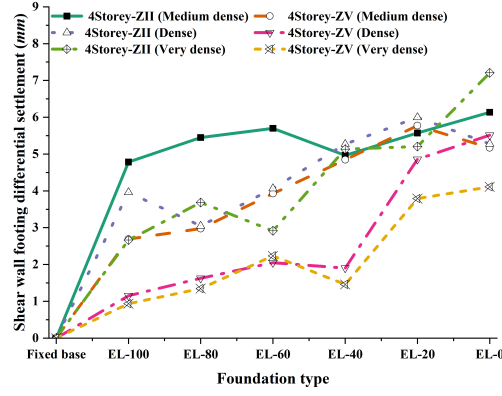


(c)

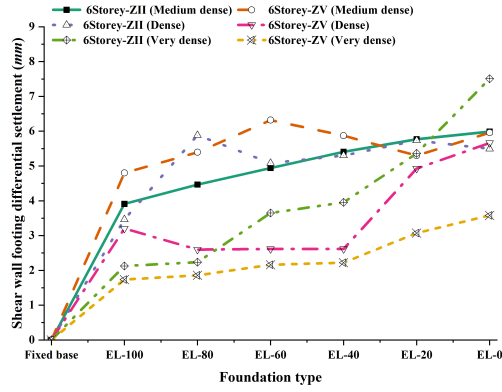
Figure 6.6: Effect of foundation rocking on foundation settlement at shear wall for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

supporting the base shear caused by seismic disturbances. This is because shear walls and footings have higher stiffness. A greater amplitude of base shear indicates that the shear wall holds the utmost importance as a structural component.

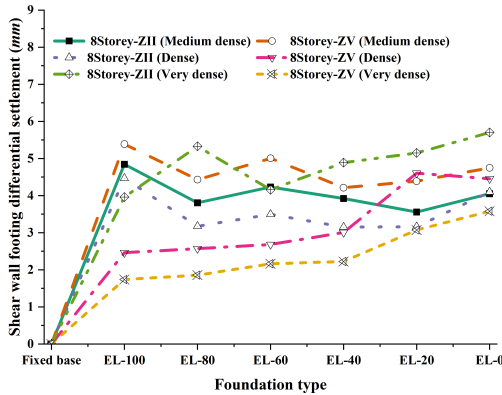
Conversely, a contradictory scenario is observed when considering the shear wall on a rocking base. The rocking motion of the shear wall in the rocking foundation results in a decrease in the base shear contribution of the shear wall, as well as a reduction in the overall base shear, as previously stated in Figure 6.5. For zone V building, the degree of reduction for EL-0 foundation is observed to decrease by 20% to 40%, whereas for zone



(a)



(b)

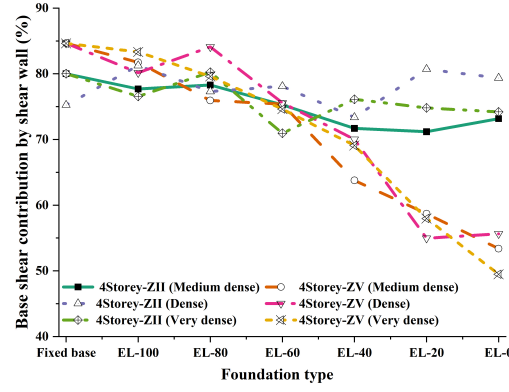


(c)

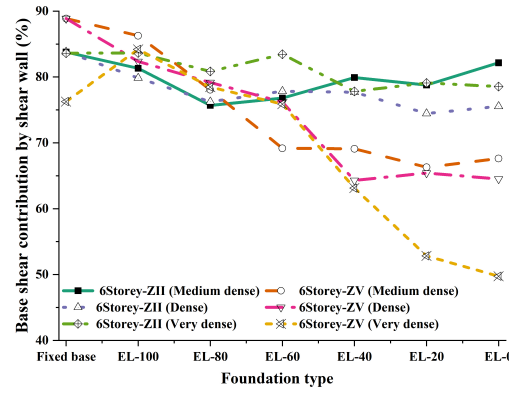
Figure 6.7: Effect of foundation rocking on differential settlement at shear wall footing for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

II designed building, the reduction ranges from 5% to 20%. The primary force acting on the shear wall is mostly redirected towards the supporting footing soil interface and some amount to the adjacent moment-resisting frames.

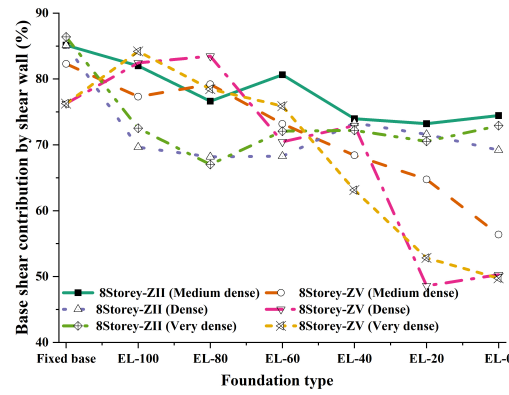
The median of maximum moments exerted at the base of the column near to the shear wall, together with their associated standard deviation is given in Figure 6.9. It is important to acknowledge that the moment values at the base of columns for fixed base building, followed by conventionally proportioned foundations, exhibit a range of 100kN-m to 200kN-m. When the shear wall footing is sufficiently large, the nearby column



(a)



(b)

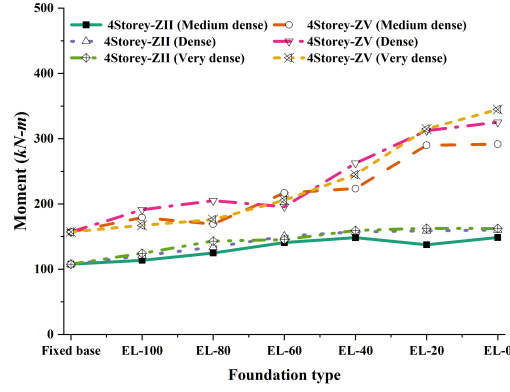


(c)

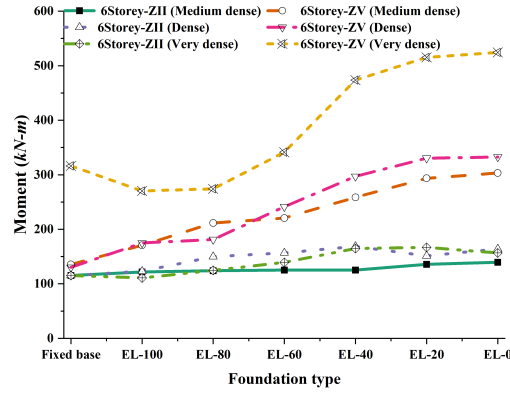
Figure 6.8: Effect of foundation rocking on base shear for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

experiences reduced moments. In zone V design supported by EL-0 rocking footing, the base moment at the column increases by a factor of 2 to 2.5 for 4 storey and 6 storey building, and by approximately 1.75 for 8 story building. Conversely, the extent of increase in zone II design, when supported by an EL-0 rocking footing, exhibits negligible fluctuations regardless of the height of the structure.

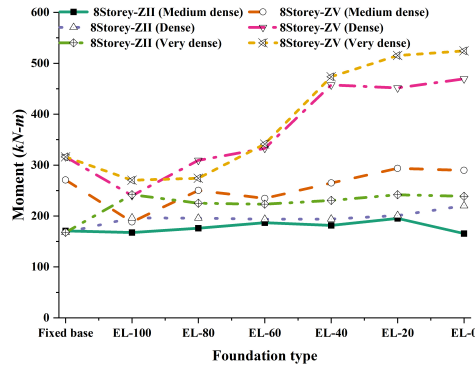
Both the substructure and superstructure exhibit inelastic responses, which serve to disperse seismic energy by means of foundation rocking. This successfully reduces the seismic demands on the entire structure. To quantify the total energy dissipated at



(a)



(b)



(c)

Figure 6.9: Effect of foundation rocking on moment experienced by column adjacent to shear wall for (a) 4-storey; (b) 6-storey and; (a) 8-storey building

the soil footing contact interface as a result of foundation rocking, the area under the moment rotation hysteresis curve is calculated. The calculation of the Energy Dissipation Ratio (EDR) involves the normalization of the total dissipated energy with the greatest potential hysteretic area (Liu, 2014). EDR ranges from 0 to 1, with a higher EDR value indicating a greater capacity of the rocking foundation system to dissipate energy (Liu, 2014). Figure 6.10 displays the median of the EDR for all substructure and superstructure layouts. The analysis results presented in Figure 6.10 reveals that the building intended for zone V in dense sand exhibit a greater degree of energy dissipation, followed by those

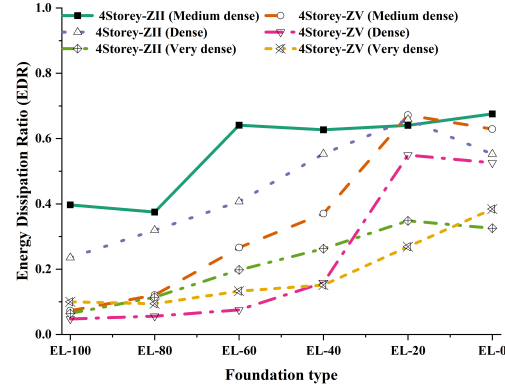
designed for very dense sand. Furthermore, there is a notable and substantial rise in EDR for foundations that are specifically designed to withstand earthquake loads below 60%. In comparison to building planned for seismic zone V, the rate of growth in EDR is minimal for building designed for zone II.

Rocking of shear wall footings, resulting from rocking and settlement, leads to a decrease in the maximum base moment and base shear encountered by the shear wall component. On the other hand, the moment frame experiences an elevation in peak base moment and base shear, but concurrently observes a reduction in the base shear contribution of the shear wall. The research findings about the behavior of shear walls indicate that the maximum capacity of the wall is not fully utilized when exposed to rocking. However, the dissipation of energy during seismic motion is achieved by utilizing the potential of the contact between the foundation soil and the moment resistant frame elements to a certain extent.

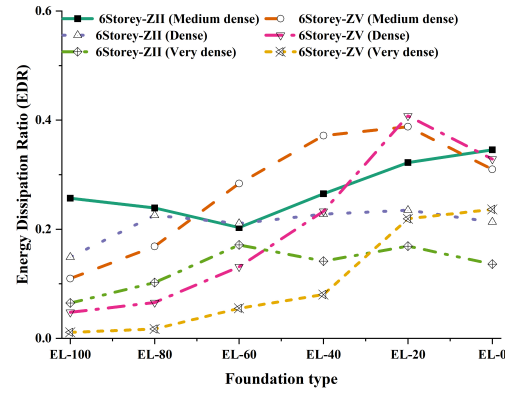
6.5 Summary

This chapter examines the seismic response of a rocking foundation in a RC shear wall framed structure under different ground motions with varying characteristics. The results are then compared to those obtained from a conventionally constructed foundation. To understand of the effect of rocking, the investigation entailed manipulating many parameters, such as foundation sizes, structure height, design parameters, and soil types. These parameters will have direct impact on the overall seismic performance of the structure and hence these parameters are selected. The shear wall footing alone is underproportioned to achieve the rocking effect whereas for the adjacent the footings are proportioned in a conventional manner. The findings from the modal analysis demonstrate a positive correlation between the reduction in footing sizes and augmenting fundamental natural period of the rocking building. Higher degree of period lengthening ranging from 25% to 40%, due to rocking was noticed for zone II structural design. Moreover, it is evident that medium dense sand demonstrates a higher degree of period elongation in comparison to dense and very dense sand. On the other hand, it is found that fixed base and conventional footings exhibited the largest peak roof acceleration compared to rocking footings. Furthermore, it is noted that the utilization of rocking foundations leads to a reduction in the maximum moment transferred from base of shear wall to its foundation due to seismic motion. Shear walls with rocking foundation can experience up to nine times greater reduction in peak moment compared to building with conventional footings i.e., EL-100, intended for seismic zone V and eight-storey building designed for seismic zone II. However, it is noted that the decrease in peak base moment for the 4-storey and 6-storey building intended for zone II was negligible and similar to that of the fixed base building.

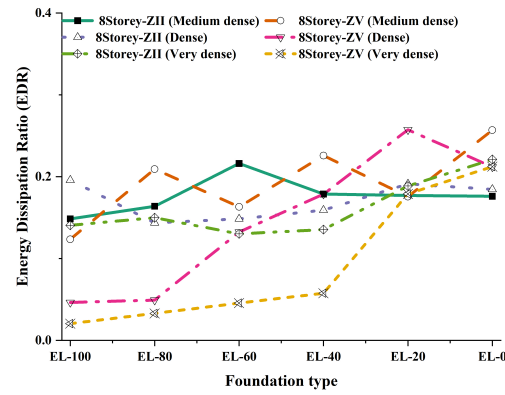
The maximum base shear responses are similar to the observed maximum moment responses. The findings indicate that the inclusion of a rocking allowed footing in a shear wall configuration results in a decrease in base shear by approximately 1.25 to 2 times



(a)



(b)



(c)

Figure 6.10: Effect of foundation rocking on energy dissipation capacity for (a) 4-storey; (b) 6-storey and; (c) 8-storey building

the magnitude recorded at the base of the footing. Furthermore, it is observed that the settlement of the shear wall increases and approaches acceptable thresholds for EL-60 foundation in the case of zone v built building. On the other hand, building designed in zone II have settlement values that are within the acceptable range after reducing the design earthquake load of up to 60%. In conclusion, it can be deduced that an augmentation in the fundamental period results in a decline in the acceleration prerequisites for rocking foundation, thus leading to a decrease in base shear and moments at the base of the shear wall. The numerical analysis findings indicate a significant increase in peak roof

displacement along the height of the structure when the shear wall rocking footing is provided. building designed for zone II exhibit a greater degree of increase in peak roof displacement compared to building designed for zone V. Furthermore, the degree of peak roof displacement diminishes as the height of the structure increases. The examination of peak shear wall settlement responses indicated that foundations developed according to conventional methods demonstrate lower levels of settlement compared to foundations designed using rocking techniques. There is a substantially lower rate of settlement observed in four-storey building, but the rate of settlement is relatively higher in eight-storey building. The current study concludes that the foundation for shear walls can be designed by taking into account a by reducing the earthquake loads for foundation design by maximum of 40% for zone V building and a maximum of 60% for zone II building. Excessive seismic displacement demands are observed over a certain threshold. Redistributing the seismic demands by means of rocking foundation enhances the overall seismic performance of the superstructure.

Effect of rocking foundation on seismic fragility

7.1 Overview

The behaviour of RC structural members, specifically the nonlinear behaviour depends upon geometry and properties of the materials. Properties of the construction materials, underlying soil medium, modelling assumptions for design methodologies and seismic motions are random in nature and hence uncertainty within the members of RC structure prevails. As a consequence of this, uncertainty exists in the predicted seismic strength and ductility demands. Several combinations of deterministic analyses mentioned from chapter 4 to chapter 6 provides the base for estimating the seismic response through probabilistic approach. The present chapter is an attempt to develop the fragility functions for RC structures with and without shear wall supported by rocking foundation under various soil conditions. The primary objectives of this chapter is to assess and compare the efficacy of conventionally designed footings and rocking footings in terms of their fragility functions as determined through a series of nonlinear time history analyses or nonlinear static analyses. In addition to that hysteretic responses for RC shear wall structures are also plotted to predict the dominant energy dissipation mechanism.

7.2 Description of analysis and design details

Estimation of capacity curve for considered RC framed structures with and without shear wall is a crucial step in the estimation fragility functions. For realistic estimations, it is preferred to carry out non linear dynamic time history analysis under several combinations of ground motion records to generate the statistical data and to perform the probabilistic analysis for estimation the seismic performance. However, conducting non linear dynamic time history analysis requires huge computational effort for several structural combinations. To counteract these difficulties empirical fragility curves are developed based on the post seismic surveys for various structural typologies. This can be achieved by performing non linear static analysis. In this present chapter fragility functions based on nonlinear static analysis is performed for RC framed building without shear wall for all the structural and foundation configuration. On the other hand non linear dynamic analyses based fragility functions are developed for one representative structural configuration for RC building with shear wall. Along with the fragility functions hysteretic responses are also studied for suite of ground motions.

The sub structural and super structural design details for RC framed building without shear wall is taken same as that mentioned in chapter 4. For the building with shear wall,

the design level is selected to be zone IV and number of storeys is chosen to be 8. Only one set of structure with varying soil types and foundation dimensions are preferred for this section, considering the computational cost for developing the fragility functions based on nonlinear seismic analysis. In order to assess the seismic performance of a rocking foundation, the dimensions of the shear wall footing are reduced in the order of 10% compared to the conventional footing size. The relaxation of the eccentricity criterion leads to a footing width for the shear wall member that is smaller than the intended design. In other words, to monitor the favourable responses of the rocking foundation, the moment carrying capacity of the footing is gradually reduced. Dimensions of footings have experienced a maximum reduction of 60% in size to achieve varying degree of rocking at footing level. However, above this threshold, the dimensions of the footings became inadequate to withstand the static loads of the superstructure system. The Table 7.1 provides an outline of the footing widths and the Design Moment Factor (DMF), which is defined as the ratio of the moment carrying capacity of the footing to the moment carrying capacity of the shear wall.

Table 7.1: Foundation dimensions for RC building with shear wall

Footing dimensions	Factor of safety against overturning	DMF	Footing tag
11m x 5.5m	3.09	1.19	CFD1
9.9m x 4.95m	2.62	1.07	CFD2
8.91m x 4.46m	2.25	0.96	SLR1
8.02m x 4.01m	1.94	0.87	SLR2
7.22m x 3.61m	1.69	0.78	MRF1
6.5m x 3.25m	1.52	0.70	MRF2
5.85m x 2.92m	1.3	0.63	HRF1
5.26m x 2.63m	1.14	0.57	HRF2

CFD=Conventional Footing Design, SRF= Slight Rocking Footing,

MRF= Medium Rocking Footing and HRF= Heavy Rocking Footing

7.3 Effect of foundation rocking on fragility of RC framed Structure

The examination of seismic fragility provides an assessment of the probability of a structure surpassing predetermined levels of damage. The fragility curves for the building under consideration are developed using the methods proposed by Hazus (2003), in accordance with the damage state definition proposed by Barbat and Pujades (Barbat et al., 2006), as indicated by Equation 7.1.

$$P \left[\frac{DS}{S_d} \right] = 1 - \varphi \left[\frac{1}{\beta_{DS}} \ln \left(\frac{S_d}{\overline{S_{d,DS}}} \right) \right] \quad (7.1)$$

where:

DS denotes damage state

S_d denotes spectral displacement

β_{DS} denotes standard deviation for damage state

$\overline{S_{d,DS}}$ denotes median spectral displacement for the damage state

In this context, the variable β_{ds} represents the overall uncertainty linked to the fragility curve and is commonly assumed to be 0.7, as stated by [Halder \(2013\)](#). In addition to that for developing the fragility functions, damage states needs to be defined to indicate the intensity of damage levels. [Barbat et al. \(2006\)](#) classified four damage states based on the spectral displacement at yield (S_{dy}) and spectral displacement at ultimate (S_{du}) as mentioned in Table 7.2.

Table 7.2: Damage state definition ([Barbat et al., 2006](#))

Damage state	Spectral displacement	Damage grade
DS1	$0.7S_{dy}$	Slight
DS2	S_{dy}	Moderate
DS3	$S_{dy} + 0.25(S_{du} - S_{dy})$	Extensive
DS4	S_{du}	Complete

To obtain the S_{dy} & S_{du} , pushover curve obtained from chapter 4 are tranformed from capacity curves to capacity spectra using the following relationships,

$$S_a = \frac{V_b}{W\alpha_m} \quad (7.2)$$

$$S_d = \frac{\Delta_{roof}}{\Gamma\phi_{roof}} \quad (7.3)$$

$$\alpha_m = \frac{\sum (W_i\phi_i)^2}{\sum W_i \sum W_i\phi_i^2} \quad (7.4)$$

$$\Gamma = \frac{\sum (W_i\phi_i)}{\sum W_i\phi_i^2} \quad (7.5)$$

where:

V_b denotes base shear of the structure from pushover analysis

W denotes seismic weight of the structure

α_m denotes modal mass coefficient

Δ_{roof} denotes roof displacement

Γ denotes modal mass participation factor

ϕ_{roof} denotes modal shape coefficient at roof level

W_i denotes storey weight at the i^{th} floor level

ϕ_i denotes modal shape coefficient at the i^{th} floor level

Figure 7.1 to Figure 7.3 illustrate the fragility curves for complete damage condition in 8-storey and 10-storey building with fixed base counterparts and with the increasing foundation rocking.

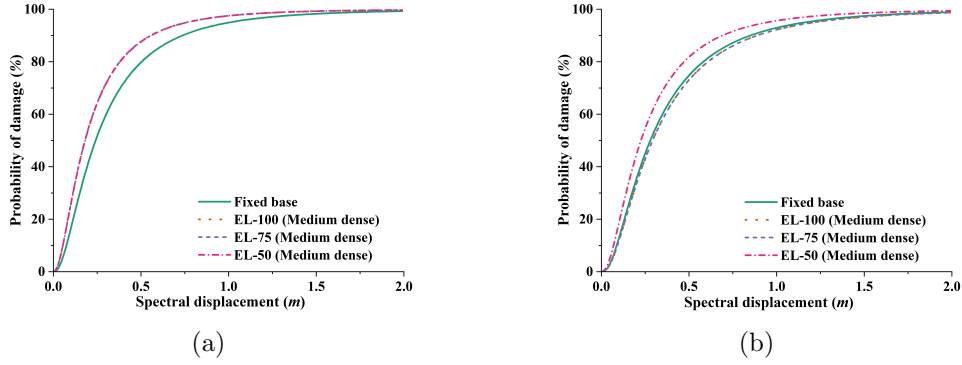


Figure 7.1: Effect of foundation rocking on the fragility functions for; (a) 8-storey and; (b) 10-storey storey on medium dense sand

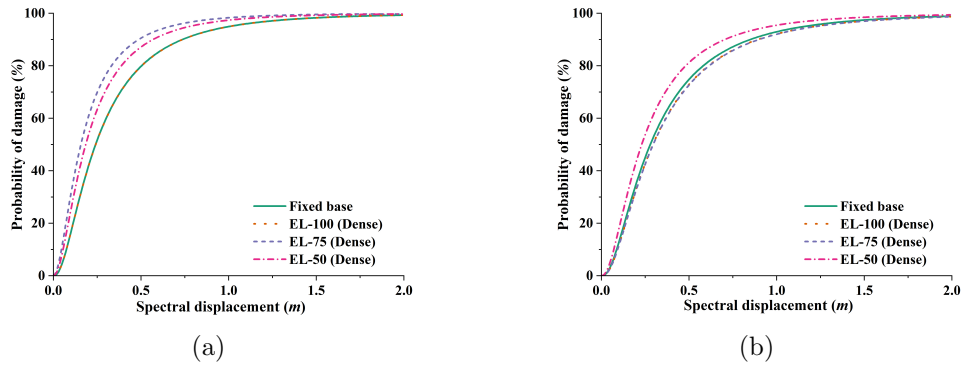


Figure 7.2: Effect of foundation rocking on the fragility functions for; (a) 8-storey and; (b) 10-storey storey on dense sand

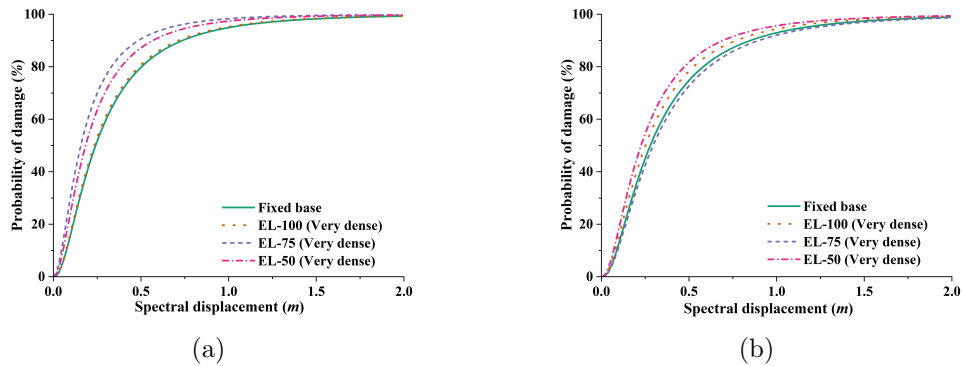


Figure 7.3: Effect of foundation rocking on the fragility functions for; (a) 8-storey and; (b) 10-storey storey on very dense sand

The fragility curves of 8-storey and 10-storey building on medium-dense sand remain comparable, regardless of the size of the foundation (see Figure 7.1). In contrast, the fragility curves depicted in Figure 7.2 and Figure 7.3 demonstrate that building situated on dense and very dense sand exhibit a decreased likelihood of collapse when subjected to escalating foundation rocking, as compared to fixed base building and conventional footings. Furthermore, it is been observed that there exists a maximum variation of 10% between constructions featuring a fixed base and those exhibiting a progressive increase in foundation rocking. As mentioned earlier in section 4.4, it is noticed that fixed base building and building supported by EL-100 footings exhibit a reduction in inelastic displacement when subjected to greater grades of damage. Nevertheless, the magnitude of inelastic displacement exhibits a positive correlation with the degree of foundation rocking, even when the damage level remains constant. This observation suggests a notable enhancement in the seismic resilience of the building.

7.4 Effect of ground motion intensity levels on foundation rocking

The seismic response of the rocking shear wall system is examined on an elemental level with the help of hysteretic curves for the suite of ground motions as mentioned in section 6.4. Three ground motions are selected to represent various levels of intensity ranging from moderate level to very strong level (see Figure 7.4) as follows: 1) moderate, where the seismic motion is equivalent to the design spectrum and as well as close to the median minus standard deviation (SD) of the selected ground motions; 2) strong, where the spectrum of the considered seismic motion is higher than the design spectrum and close to the median spectrum of the ground motions mentioned above; and 3) very strong, where the spectrum of the seismic motion is much higher the design spectrum and close to the median plus standard deviation of the ground motions considered in this study. The responses at the element level are illustrated using one ground motion from each category considered. The comparison of the shear wall with various foundation configurations is presented in the form of a) moment-curvature (M-C) response; b) moment induced at the foundation level to the foundation rotation (M-R); c) foundation settlement at the shear wall footing edge to the foundation rotation (V-R); d) foundation shear sliding response (B-H).

The Denali record is selected as a representative example of seismic motion with moderate impact. The M-C and M-R responses depicted in Figure 7.5 to Figure 7.7, indicates that the HRF2 under dense and very dense soil conditions exhibited a greater degree of foundation rotation, specifically 0.005 radians, compared to the CFD1 where foundation rotation is nearly negligible. Nevertheless, when the footing rotation and settlement increases, the MRF and HRF foundation exhibited a significant reduction in the peak moment transferred from shear wall to its supporting foundation, by near about twofold of its magnitude. In medium dense soil conditions, CFD1 exhibits a greater moment compared to rocking foundation designs that involve minimal foundation rotations. On the

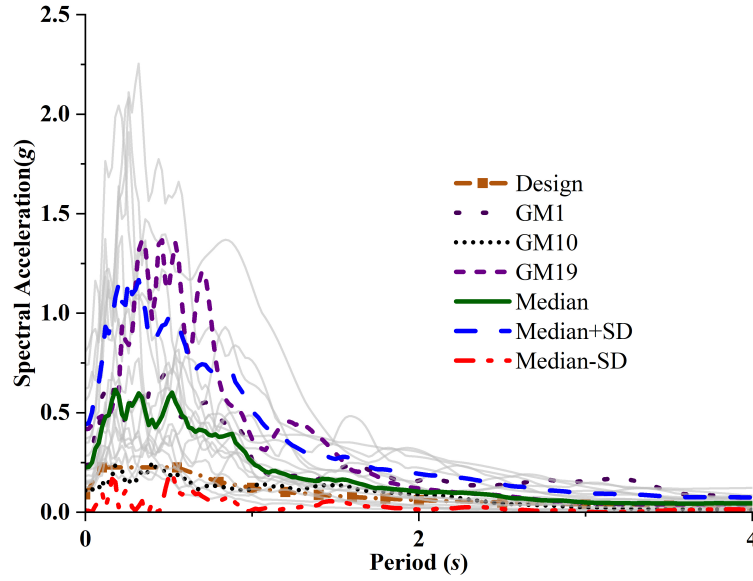


Figure 7.4: Acceleration spectra (Sa) for selected ground motion records for the nonlinear dynamic time history analysis

other hand, B-H reactions exhibit minimal sliding, suggesting that rocking is the dominant response.

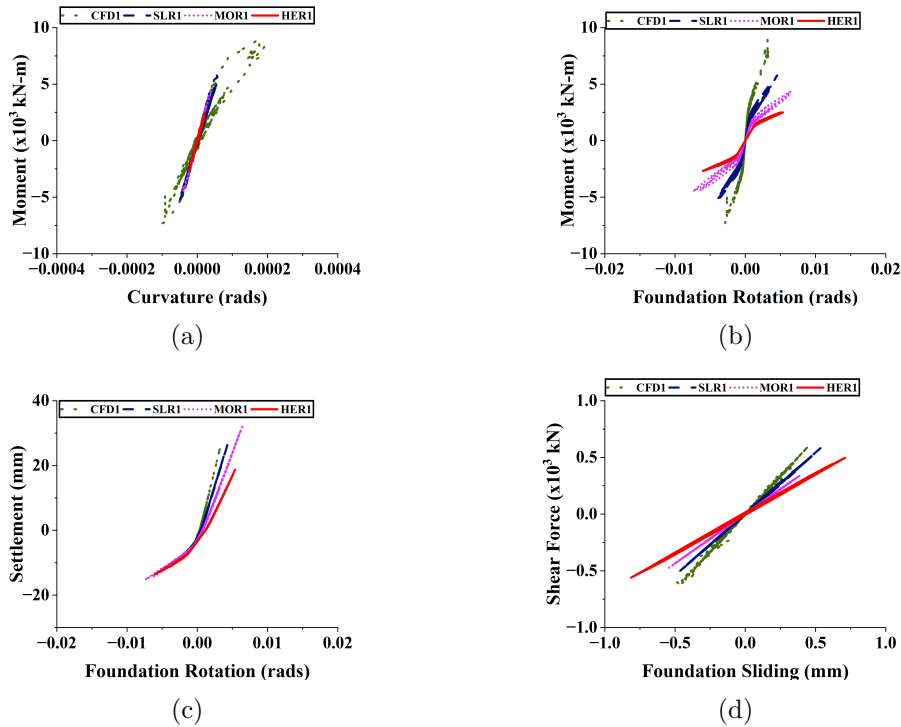


Figure 7.5: Hystretic response of shear wall with varying degree of rocking supported by medium dense sand subjected to moderate seismic motion (Denali GM10); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

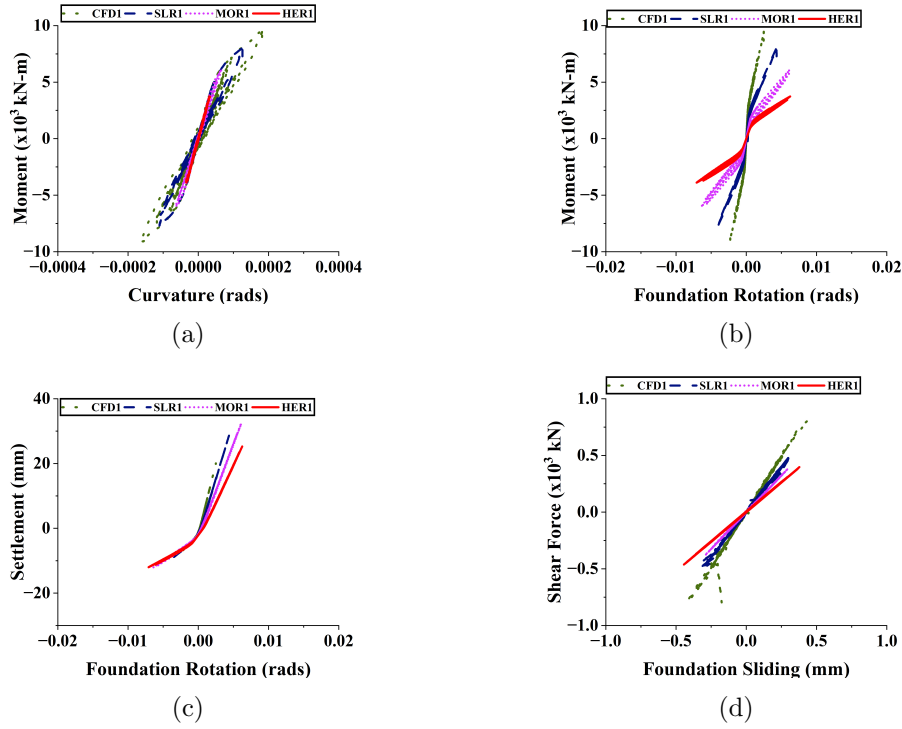


Figure 7.6: Hystretic response of shear wall with varying degree of rocking supported by dense sand subjected to moderate seismic motion (Denali GM10); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

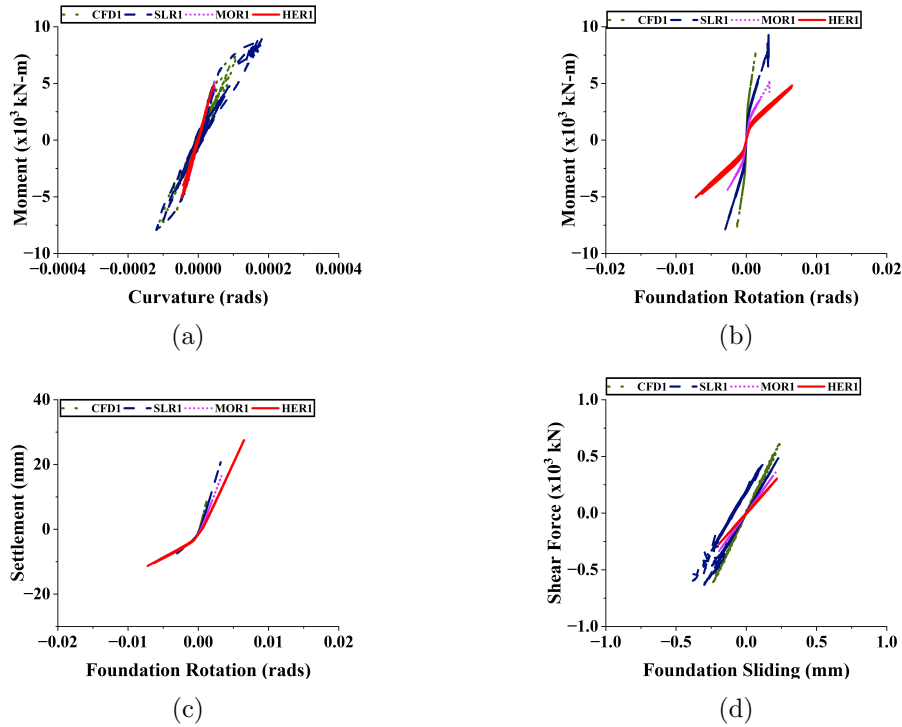


Figure 7.7: Hystretic response of shear wall with varying degree of rocking supported by very dense sand subjected to moderate seismic motion (Denali GM10); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

The record of Imperial Valley (GM1) is selected as a representative example of strong impact seismic activity. The M-C responses (Figure 7.8 to Figure 7.10) demonstrate that the CFD1 and SRF1 exhibit nearly identical behavior in both dense and very dense soil situations, instead with a somewhat larger foundation rotation of 0.005 radians. In contrast, it can be noticed that CFD1 exhibits a higher moment within the range of 10MN to 15MN, but SRF1 demonstrates the moment that is about half of the moment obtained from CFD1. Whereas, for HRF footing experiences nearly three times higher rotation than the SRF and CFD foundation cases with decrease in moment by about two times. The increase in settlement is noticed for each increase in the foundation rocking scenario. The V-R responses indicate that the rocking footings exhibit a threefold increase in footing rotation and settlement compared to the CFD1, regardless of the soil conditions. Based on the plot, it can be shown that the B-H responses demonstrate minimal sliding, suggesting that the reaction is primarily influenced by rocking in soil conditions characterized by medium dense and dense. On the other hand, it is shown that circumstances of very dense soil exhibited noticeable nonlinear shear sliding reactions.

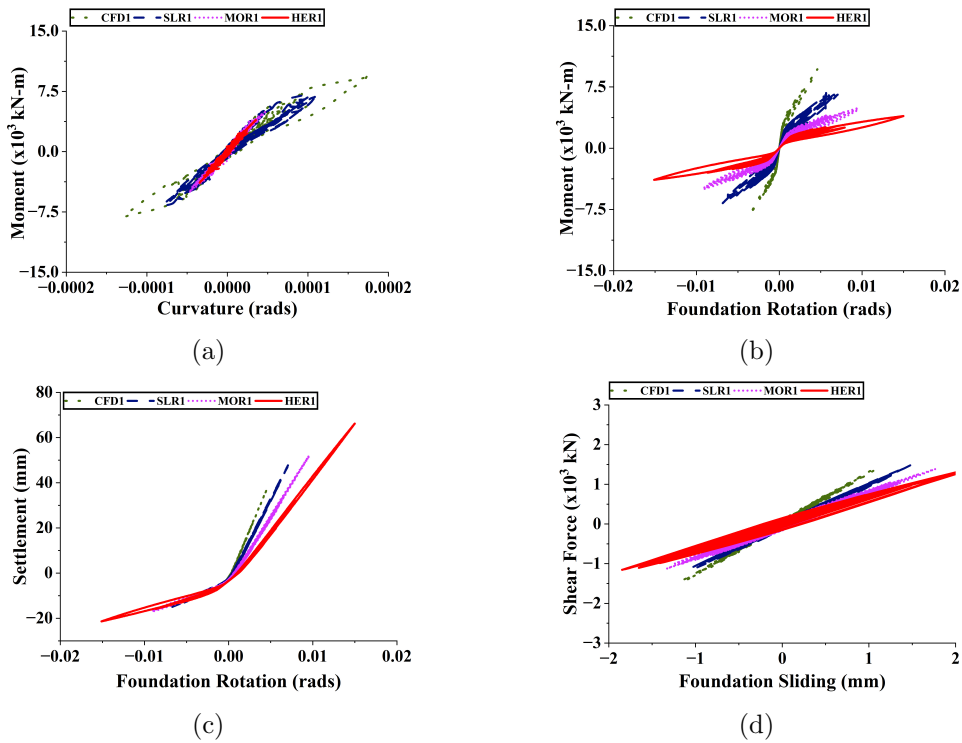


Figure 7.8: Hystretic response of shear wall with varying degree of rocking supported by medium dense sand subjected to moderate seismic motion (Imperial GM1); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

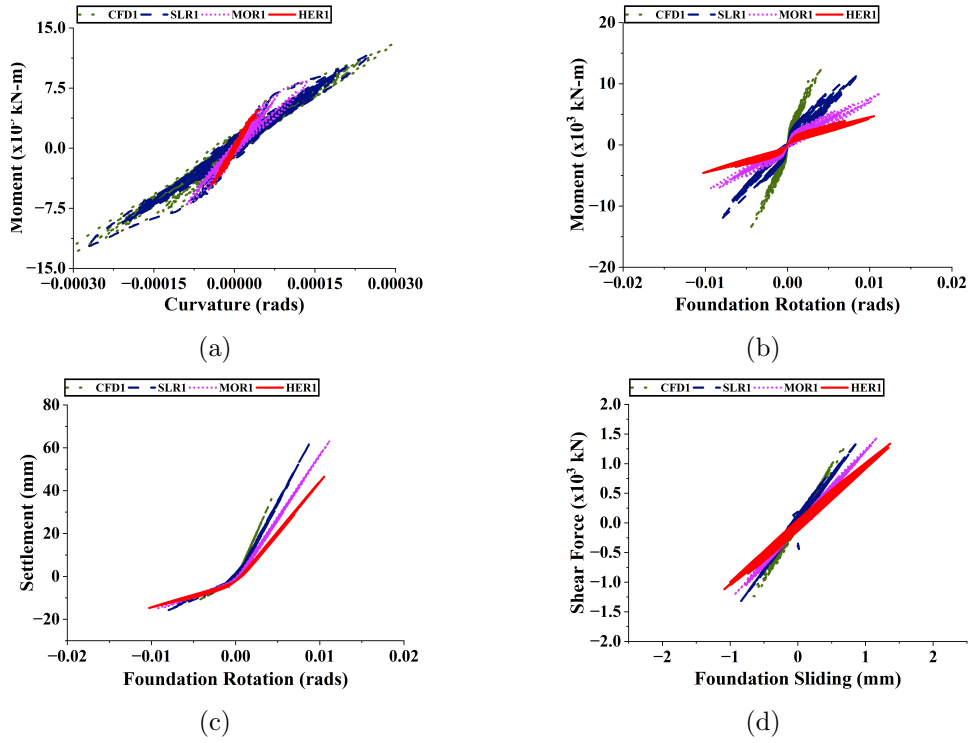


Figure 7.9: Hystretic response of shear wall with varying degree of rocking supported by dense sand subjected to moderate seismic motion (Imperial GM1); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

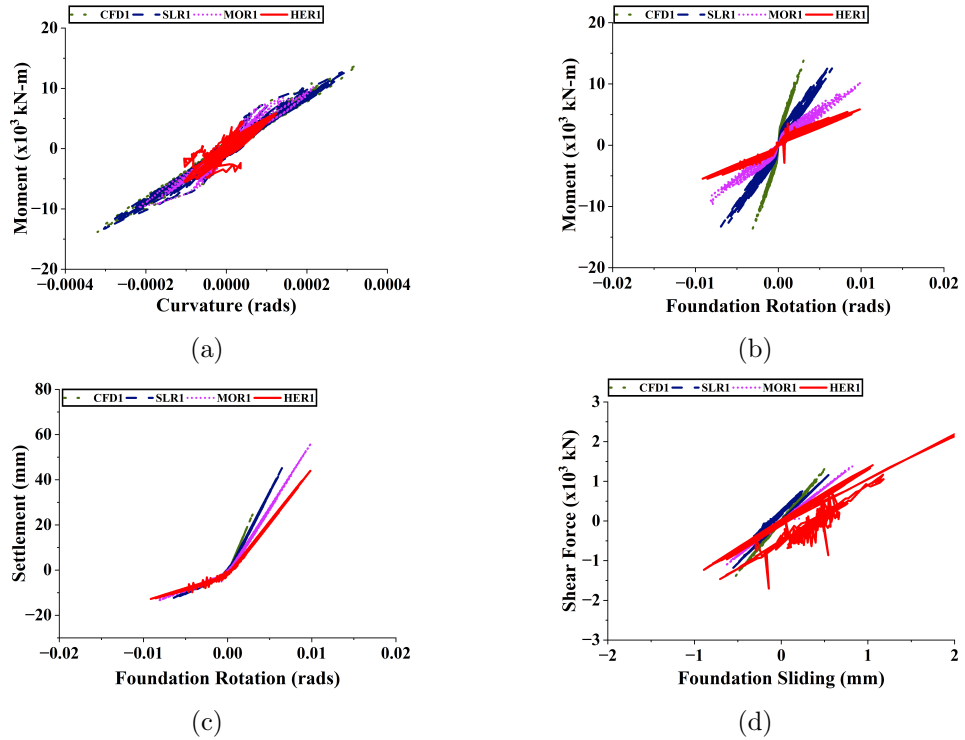


Figure 7.10: Hystretic response of shear wall with varying degree of rocking supported by very dense sand subjected to moderate seismic motion (Imperial GM1); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

The Landers record, GM19, is chosen to accurately depict highly intense movement. The M-C and M-R responses, as depicted in Figure 7.11 to Figure 7.13, indicate that CFD1 and CFD2 model exhibits a notably elevated foundation rotation of approximately 0.01 radians and a stronger bending moment of approximately 30MN compared to the previously stated ground motions. All of the CFD1 footing design scenarios exhibited higher settlements of approximately 60mm. In contrast, with respect to the V-R reactions, the behavior of CFD and SRF exhibits a high degree of similarity under medium dense and very dense soil conditions. Meanwhile, the B-H responses exhibit minimal sliding, suggesting that the reaction is mostly governed by rocking movement. The study revealed a significant reduction in shear wall reactions, approximately 3 to 5 times higher for CFD1 footing scenarios, when comparing the MRF and HRF foundation conditions. Based on the Landers record, it is clear that there is a potential drawback to overdesigning the footing.

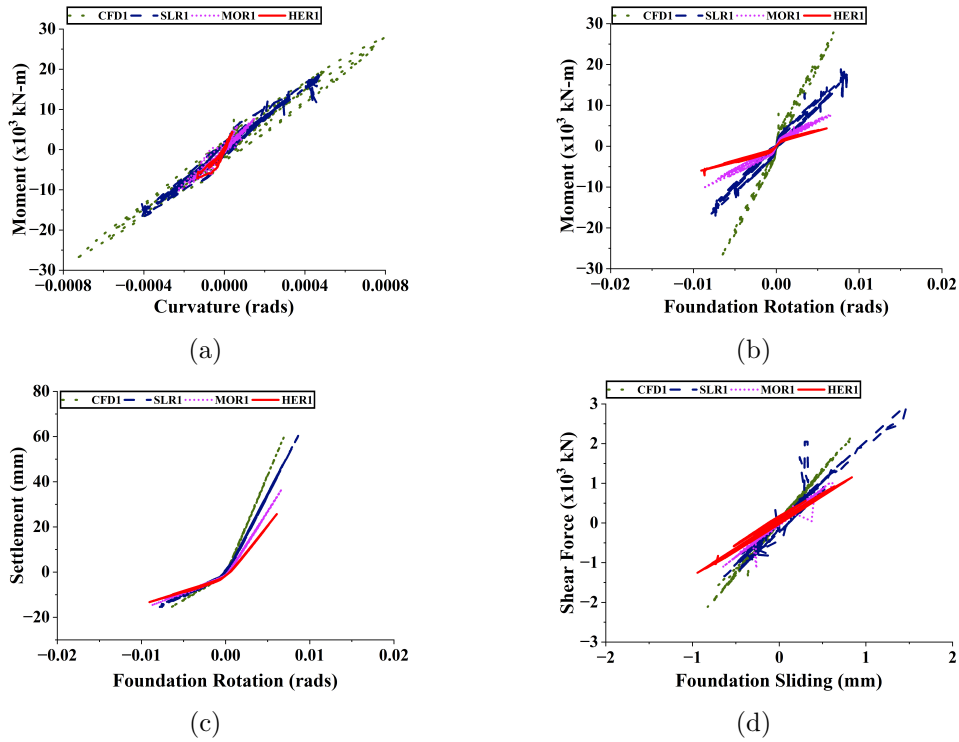


Figure 7.11: Hystretic response of shear wall with varying degree of rocking supported by medium dense sand subjected to moderate seismic motion (Landers GM19); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

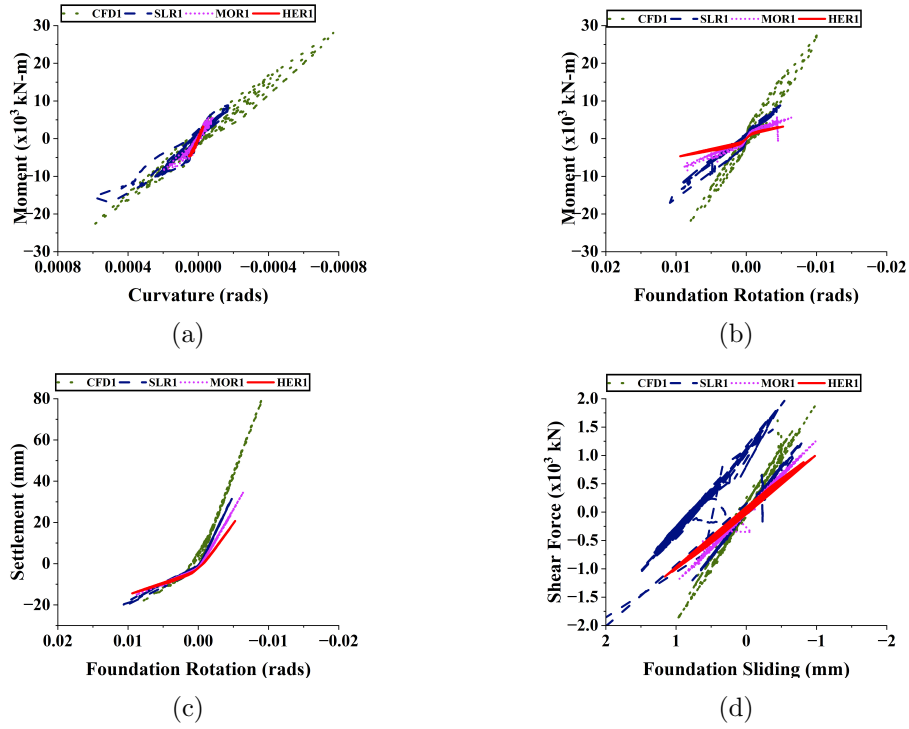


Figure 7.12: Hystretic response of shear wall with varying degree of rocking supported by dense sand subjected to moderate seismic motion (Landers GM19); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

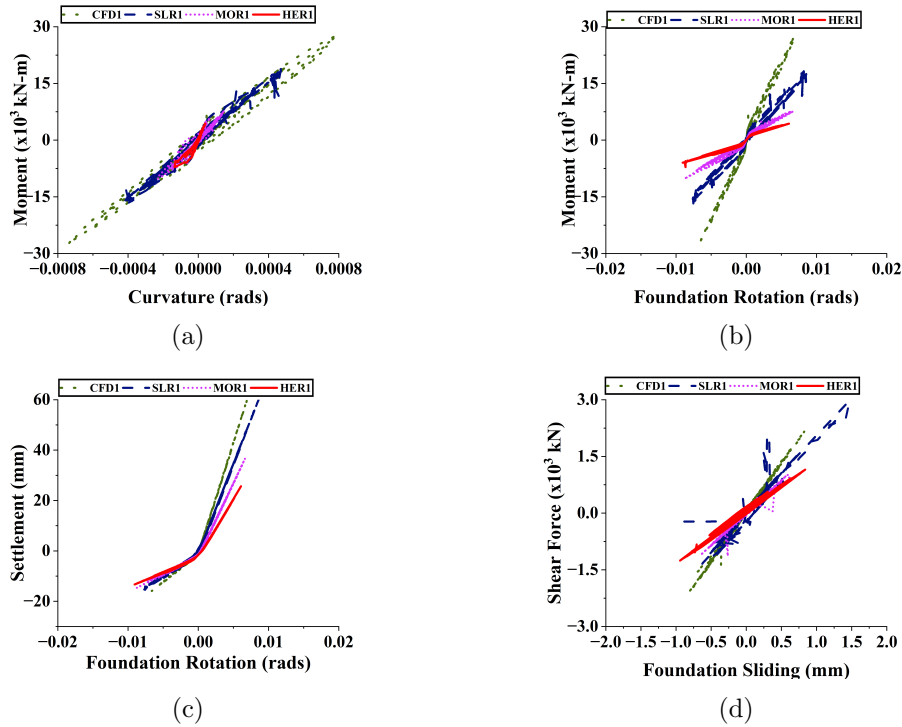


Figure 7.13: Hystretic response of shear wall with varying degree of rocking supported by very dense sand subjected to moderate seismic motion (Landers GM19); (a) M-C response; (b) M-R response; (c) V-R response; (d) B-H response

7.5 Effect of foundation rocking on seismic fragility of RC shear wall frame structure

Seismic fragility curves characterise the likelihood that a structure will sustain the damage levels. In particular, shear wall framed constructions' seismic fragility is evaluated using Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell, 2002; Kircher et al., 2009). Seismic fragility analysis, evaluated using the result obtained from IDA, is regarded as one of the most effective methods for assessing the earthquake-induced vulnerability of building. This section concentrates on developing the capacity curve based on IDA and to develop fragility functions based on IDA.

7.5.1 Incremental dynamic analysis

IDA aims to offer a comprehensive reaction of the structure in relation to the probable magnitude of ground motion. This study use the IDA methodology to establish a clear correlation between seismic capacity and demand. In the context of governing Engineering Demand Parameters (EDP), provides the ability to estimate principal response variables in relation to seismological Intensity Measures (IM) (Vamvatsikos and Cornell, 2002; Kircher et al., 2009). The IDA approach involves performing nonlinear dynamic analysis on a prototype structural system using a set of ground motion data. These records are scaled to various IM levels, with the aim of assessing the structure's reaction from elastic to ultimate global dynamic instability. The average spectral acceleration S_{avg} is selected as the IM, using the scaling method outlined by Eads et al. (2016). The EDP, on the other hand, is chosen to represent roof drift and shear wall footing settlement. Figure 7.14 displays the IDA results for representative ground motion record, including the median, median+Standard Deviation (SD), and average spectral acceleration S_{avg} versus roof drift ratio. For other combinations of foundations sizes and soil types, median ad standard deviation plots are provided to represent the benchmark limit states as mentioned below.

The median and standard deviation (SD) were computed for benchmark damage limit states, taking into account the variability seen across different foundation sizes and soil types, as mentioned in Section 7.2. The study utilized the damage states for roof drift ratio, as specified in Table 7.3, in accordance with the guidelines provided by Xue et al. (2008). Defining different damage states is crucial when assessing the settlement-based damage of isolated footings. Exceeding the defined limits may result in excessive settlements, which can negatively impact the intended structural performance.

Table 7.3: Tolerable limits for various performance levels

Performance level	Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
Roof drift (%)	1.0	2.0	2.5
Settlement (mm)	25	40	60

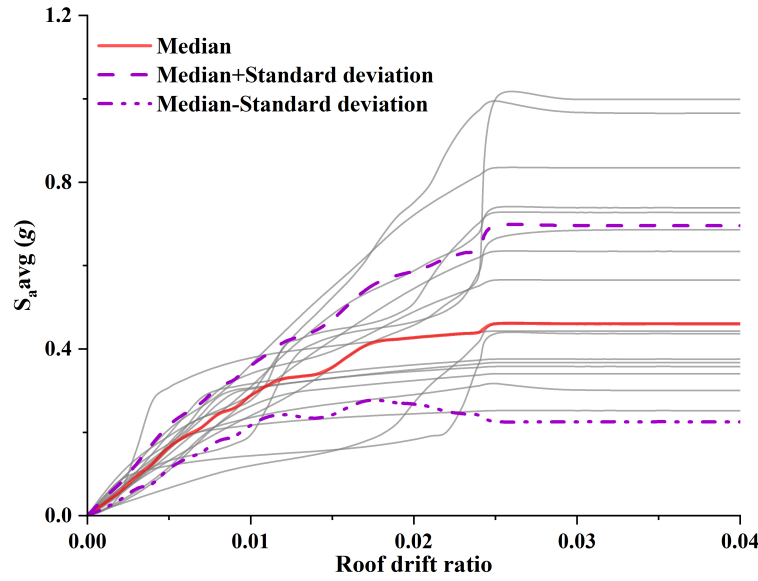
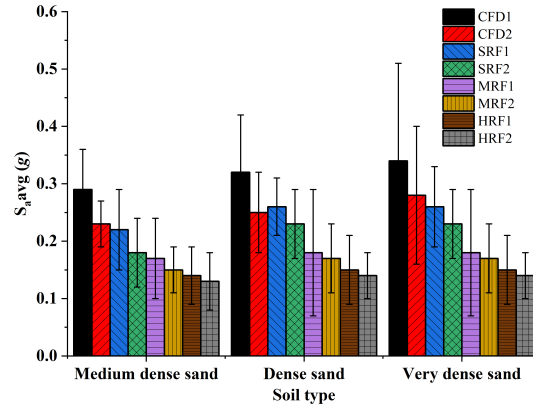


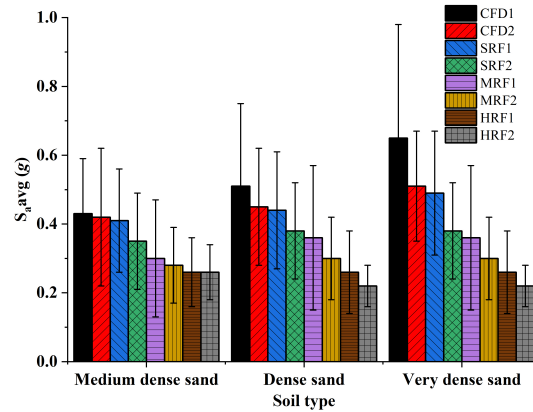
Figure 7.14: Dynamic capacity curve for CFD1 on medium dense sand

According to probabilistic studies conducted by Zhang and Ng (2005, 2007), the likelihood of surpassing the deterministic maximum settlement of 25mm, as suggested by Terzaghi et al. (1996), is below 5%. When the foundation reaches a maximum settlement of 25mm, it is unlikely to experience significant differential settlement. Additionally, the soil exhibits elastic behavior, hence reducing the likelihood of shear failure. Therefore, a limit state of 25mm is selected as the initial occupancy level for the foundation. Conversely, it is shown that the upper limit of acceptable foundation settlement for mitigating collapse to the superstructural elements in reinforced concrete frame building situated in sandy environments is approximately 60mm Zhang and Ng (2005, 2007); Terzaghi et al. (1996). Beyond this point, the foundation may experience significant differential settlements, leading to the mobilization of soil shear strength. Hence, a collapse prevention level of 60mm is deemed appropriate. In the context of life safety performance, a selection of 40mm is made, which is in close proximity to the median settlement values for immediate occupancy and collapse prevention. Table 7.3 provides a concise summary of the acceptable thresholds for different performance levels associated with the super-structure and sub-structure, specifically in relation to roof drift and foundation settlement. Figure 7.15 and Figure 7.16, illustrate the mean spectral acceleration of the superstructure in relation to roof drift and settlement behavior of the superstructure and substructure, respectively.

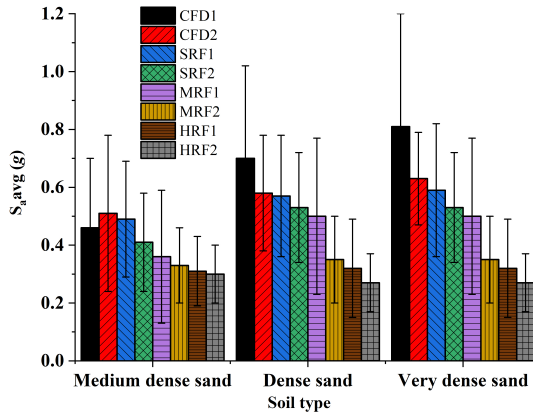
Figure 7.15 demonstrates that the structure will reach the IO-roof drift damage states when the S_{avg} is greater, ranging from 0.3g to 0.35g for CFD1. This damage decreases linearly as the foundation sizes drop, indicating a decrease in the stiffness of the supporting foundation and soil medium. The settlement responses of the sub-structure, as illustrated in Figure 7.16, indicate that the structure will reach the IO-settlement damage state at a 25% higher S_{avg} for CFD1. This may be attributed to the greater stiffness of the sub-structure, which results in the transmission of larger lateral forces to the superstructure.



(a)



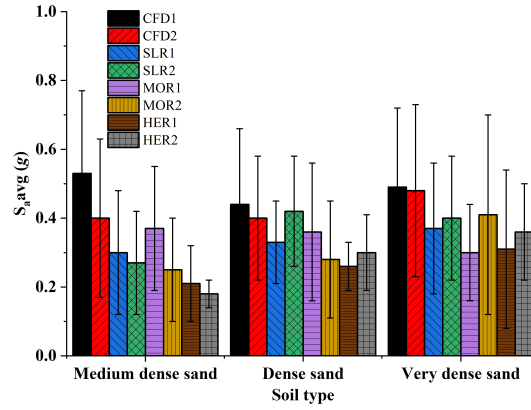
(b)



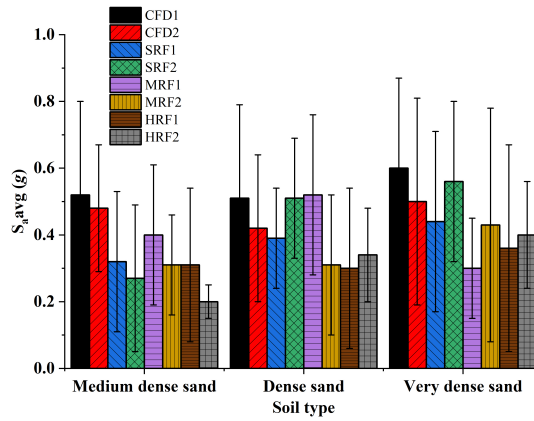
(c)

Figure 7.15: Average spectral acceleration of superstructure in terms of roof drift and its corresponding standard deviation conventional footing design and rocking footings for (a) limit state IO; (b) limit state LS and; (c) limit state CP

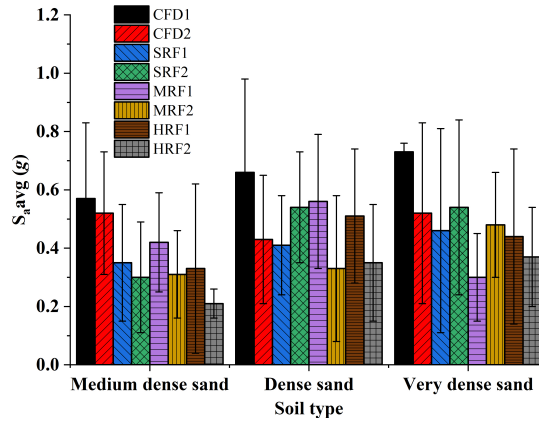
On the other hand, it is observed that heavy rocking footings in dense and very dense sand exhibit S_{aavg} values that are 2 to 3 times lower. This suggests that the induced seismic energy is partially absorbed as a result of foundation uplift. Nevertheless, it is observed that in the situation of medium dense sand, the SRF1 footing will experience CP-settlement damage at a greater average S_{aavg} compared to CFD1, and about equal



(a)



(b)



(c)

Figure 7.16: Average spectral acceleration of substructure in terms of settlement and its corresponding standard deviation conventional footing design and rocking footings for (a) limit state IO; (b) limit state LS and; (c) limit state CP

to that of CFD2. In contrast, it can be observed that shear walls supported by MRF2, HRF1, and HRF2 exhibit CP-settlement damage states at a lower average $S_{a,avg}$, than other types of footings.

7.5.2 Effect of foundation rocking on seismic fragility of RC Shear Wall building

Seismic fragility analysis entails the assessment of the probability of a structure undergoing varying degrees of damage when exposed to seismic forces. In this study, the lognormal distribution given by Liel et al. (2009) and Haselton et al. (2011) is utilized to generate fragility curves based on the IDA curves (refer Subsection 7.5.1). The following relationships are used to estimate the uncertainties associated with the fragility curve functions and its distribution;

$$P \left[\frac{LS}{S_a} \right] = 1 - \varphi \left[\frac{\lambda_c \frac{\lambda_D}{S_a}}{\beta_T} \right] \quad (7.6)$$

where:

S_a denotes spectral acceleration

λ_c denotes natural logarithm of median displacement capacity for various limit states considered in this study

$\frac{\lambda_D}{S_a}$ denotes natural logarithm of calculated median displacement demand for average spectral acceleration

β_T denotes Total uncertainty which is given by the following equation

$$\beta_T = \left\{ (\beta_{Ds_a})^2 + (\beta_{RTR})^2 + (\beta_M)^2 \right\}^{\frac{1}{2}} \quad (7.7)$$

where:

β_{Ds_a} denotes demand uncertainty

β_{RTR} denotes uncertainty in record-to-record variability

β_M denotes modelling uncertainty

Figure 7.17 to Figure 7.19 illustrate fragility curves for different foundation types in conjunction with various soil types. These curves are examined in order to investigate the influence of rocking on the seismic performance of reinforced concrete shear walled building. Given that settlement and roof displacement are the primary factors of vulnerability, Figure 7.17 illustrate the fragility curves for medium dense sand with respect to these two parameters. It is observed from Figure 7.17 that different degrees of footing dimensions can lead to significantly different probabilities of attaining different states of damage. Footings that are slightly overdesigned in CFD1 have a reduced likelihood of reaching a residual settlement of 25mm and a flattening of the curve for 40mm and 60mm for medium dense sand. Nevertheless, the fragility curve for roof drift indicates a higher likelihood of reaching the performance limits at an earlier stage for rocking footings compared to conventionally designed ones. The rise in displacements caused by footing uplift is attributed to the rocking of the footing. Conversely, the shear wall supported by CFD footings undergoes flexural displacement, as depicted in Section 7.4.

With an average spectral acceleration of 0.5g, it can be shown that nearly all of the footing types depicted in Figure 7.18 exhibit higher probabilities of attaining the IO

Table 7.4: Various uncertainties considered for the study and its observed values

Soil type	Foundation type	Record to record variability (roof drift)			Record to record variability (roof drift)			Demand uncertainty	
		IO	LS	CP	IO	LS	CP	Roof drift	Settlement
Medium dense sand	CFD1	0.07	0.16	0.24	0.33	0.29	0.30	0.01	0.11
	CFD2	0.04	0.20	0.27	0.31	0.30	0.32	0.01	0.07
	SRF1	0.06	0.13	0.17	0.14	0.17	0.20	0.11	0.10
	SRF2	0.05	0.13	0.16	0.15	0.20	0.20	0.02	0.20
	MRF1	0.05	0.09	0.12	0.19	0.25	0.38	0.03	0.30
	MRF2	0.04	0.10	0.14	0.18	0.19	0.18	0.01	0.33
	HRF1	0.05	0.09	0.12	0.09	0.15	0.12	0.01	0.35
	HRF2	0.04	0.07	0.11	0.10	0.11	0.12	0.04	0.22
Dense sand	CFD1	0.11	0.26	0.34	0.25	0.30	0.30	0.02	0.05
	CFD2	0.07	0.18	0.20	0.22	0.25	0.25	0.01	0.04
	SRF1	0.06	0.13	0.16	0.13	0.15	0.17	0.01	0.12
	SRF2	0.07	0.14	0.18	0.19	0.18	0.18	0.02	0.07
	MRF1	0.10	0.21	0.27	0.22	0.31	0.54	0.01	0.13
	MRF2	0.08	0.15	0.17	0.19	0.21	0.24	0.01	0.28
	HRF1	0.07	0.15	0.20	0.08	0.13	0.60	0.02	0.15
	HRF2	0.05	0.10	0.13	0.12	0.11	0.20	0.01	0.95
Very dense sand	CFD1	0.16	0.34	0.41	0.24	0.24	0.32	0.01	0.08
	CFD2	0.15	0.15	0.21	0.27	0.29	0.32	0.01	0.12
	SRF1	0.04	0.16	0.20	0.16	0.22	0.35	0.04	0.14
	SRF2	0.07	0.14	0.18	0.20	0.22	0.30	0.02	0.09
	MRF1	0.10	0.21	0.27	0.15	0.15	0.14	0.01	0.07
	MRF2	0.08	0.15	0.17	0.33	0.31	0.38	0.01	0.10
	HRF1	0.07	0.15	0.20	0.31	0.31	0.43	0.02	0.10
	HRF2	0.05	0.10	0.13	0.14	0.15	0.17	0.01	0.16

performance condition. Additionally, approximately 50% of these kinds demonstrate the ability to achieve a settlement of 25mm. The fragility curve for dense sand in footings with somewhat oversized CFD1 exhibited a flattening effect for settlements of 40mm and 60mm. At lower average spectral acceleration, there is a significant increase in the chance of residual settlement for heavy rocking footings. Nevertheless, the fragility curve pertaining to settlement exhibits nearly identical reactions in cases of SRF and MRF, regardless of the extent of settlement. The fragility curve pertaining to roof drift exhibits an upward trend in the likelihood of attaining the performance thresholds.

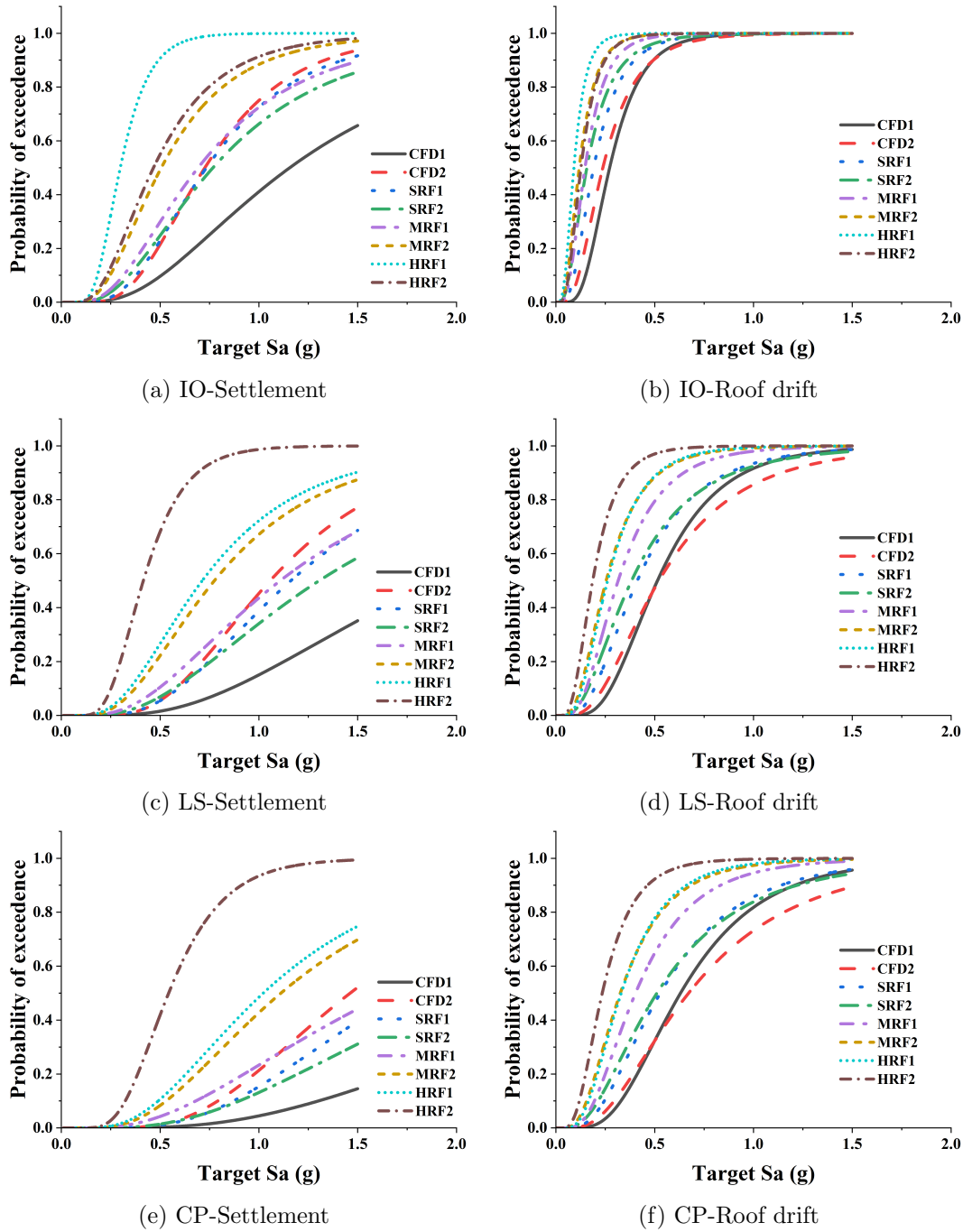


Figure 7.17: Effect of foundation rocking on the seismic fragility functions for RC shear wall framed building resting on medium dense sand

Figure 7.19 shows that nearly all types of footing have a higher chance of reaching the IO performance stage and about half of them have a chance of reaching the 25mm settlement at an average spectral acceleration of 0.5g. For 40mm and 60mm of dense sand, the fragility curve for slightly oversized footings CFD1 has flattened. At decreasing average spectral acceleration, the residual settling probability for HRF rose dramatically. Regardless of the level of settlement, the fragility curve for settlement displays nearly identical responses for cases of SRF and MRF. Roof drift fragility curves in dense sand,

like those in medium dense sand, show an increasing chance of reaching the performance limits.

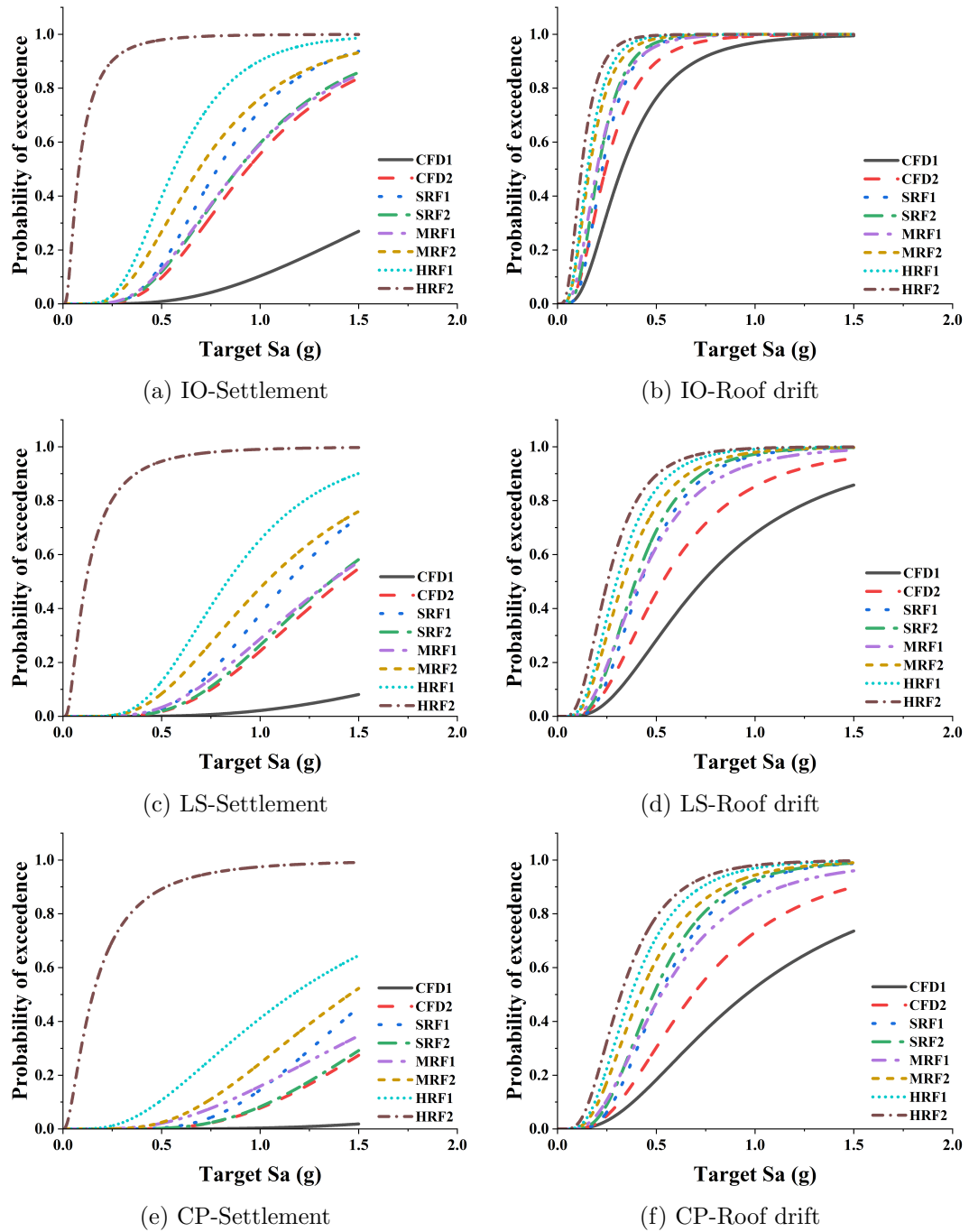


Figure 7.18: Effect of foundation rocking on the seismic fragility functions for RC shear wall framed building resting on dense sand

The likelihood of reaching the IO performance state is greater than 75% for all footing types at an average spectral acceleration of 0.5g (Figure 7.19), while the probability of reaching the 25mm settlement is less than 50%. In contrast, 40mm and 60mm settlements at 0.5g have not shown any signs of damage. The fragility curve for 40mm and 60mm conventionally oversized footings has flattened. At lower average spectral acceleration, heavy rocking footings are more likely to experience residual settlement and roof drift than moderate or slight rocking footings.

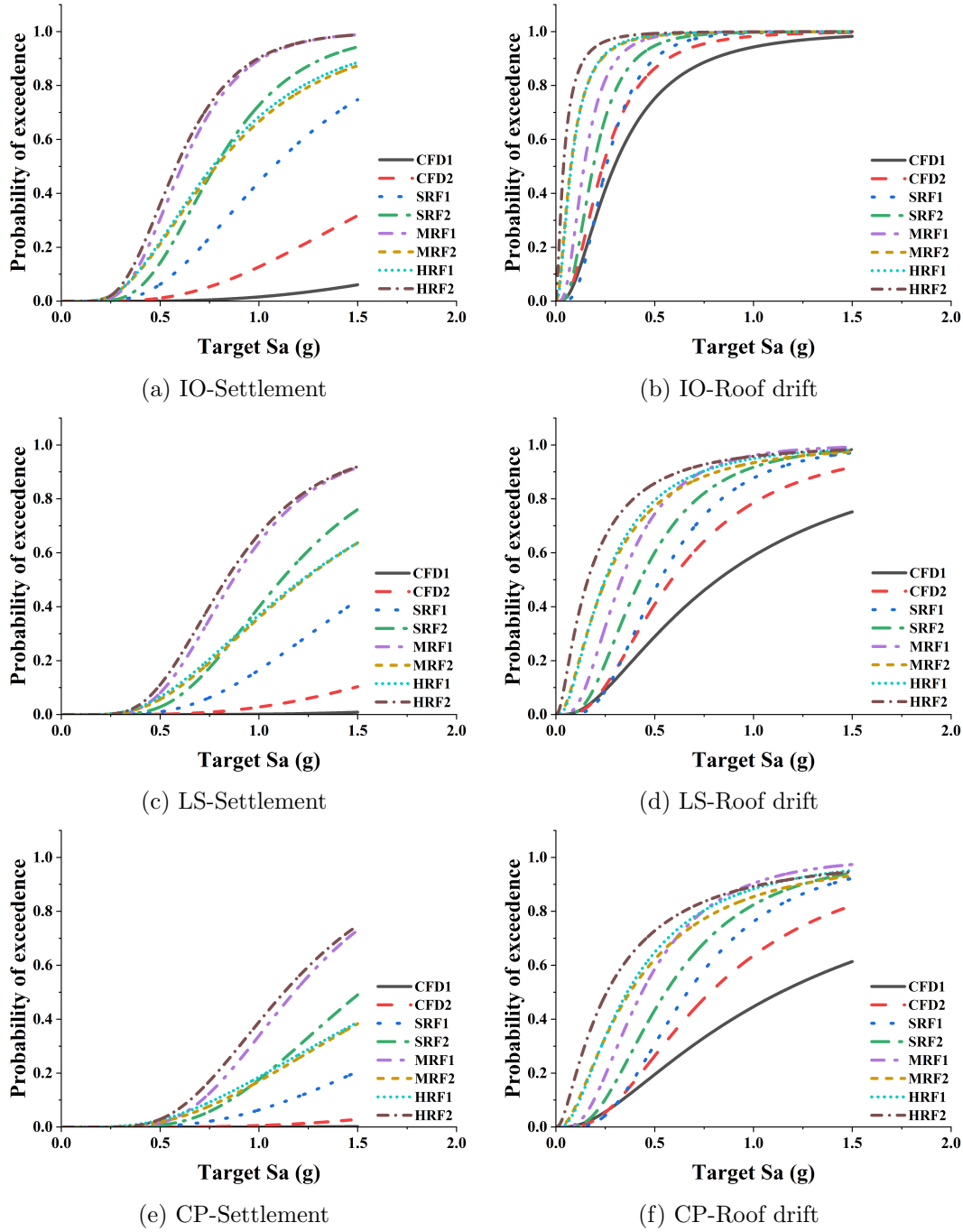


Figure 7.19: Effect of foundation rocking on the seismic fragility functions for RC shear wall framed building resting on very dense sand

In general, rocking footings exhibit a higher likelihood of surpassing the performance limit compared to CFD1 and CFD2. The probability of surpassing the maximum settlement suggests that the conventionally designed footing effectively transfers the lateral stresses due to seismic actions through the superstructural components. It is also found that the LS and CP level settlement for slight and moderate rocking footings in both dense and very dense sand has a reduced likelihood of surpassing the average spectral acceleration less than 0.5g. In most of the cases, the maximum roof drift is seen to surpass the range required for life safety and collapse prevention by over 60 percent, irrespective of the sizes of the foundations. The roof drift can be attributed to two primary factors: firstly, the flexural displacement of the structural part, and secondly, the rocking of the footing. In the first situation, as the flexural displacement increases, the probability of shear wall damage also increases. Conversely, the rocking displacement transforms the failure of the shear wall into uplifting movement and increased settlement of the underlying soil. The act of uplifting the foundation prevents the shear wall from experiencing failure and also eases the process of retrofitting.

7.6 Summary

While shear walls are typically included in earthquake-resistant designs, their failure during a large earthquake can lead to complete structural collapse. Foundation rocking is a highly effective method for mitigating earthquake-induced damages to shear walls. The current investigation offers a comprehensive understanding of the seismic response exhibited by RC shear wall framed building when allowed to rock at the foundation level. The seismic behavior of a structure is influenced by various factors, including the height of the superstructure, the soil types beneath it, and the level of contact between the soil and foundation. The rocking mechanism of the foundation has been implemented to optimize the utilization of the available strength, stiffness, and ductility of the combined substructure and superstructure. This is achieved by under proportioning the foundation to enable uplift without inducing overturning. The seismic response of RC shear wall framed building is found to exhibit an increase in the degree of foundation rocking as the Design Moment Factor (DMF) decreases from 1.17 to 0.57. Additionally, the induced moment resulting from seismic action in the shear wall is observed to decrease by more than twofold with an increase in foundation uplift for medium dense sand. Moreover, when the soil beneath the foundation becomes denser, the extent of decrease in generated moment diminishes. Conversely, the medium dense sand exhibited the highest settlements, followed by dense sand and very dense sand. In addition, the rotation of the foundation has a notably reduced effect, as evidenced by the moderate seismic motion and the response of moment curvature. Both designs exhibit a similar reaction, although with a slightly greater foundation rotation for ground motion that falls within the design spectrum. In the context of high impact strong ground motion, moderate rocking footings demonstrate enhanced foundation rotation, resulting in a reduction of the moment by over twofold compared to

conventionally designed footings. In comparison to rocking footings, conventional footings typically demonstrate greater moment and foundation rotation when subjected to very strong seismic action on dense and medium dense sand, respectively. This observation suggests that the over designed footing may not always yield advantageous outcomes for superstructures. The foundation sliding shear responses demonstrate that rocking is the primary factor influencing the seismic performance of all foundation types. In general, the findings of this study indicate that the reduction of DMF can reach values of 0.87 for medium dense sand, 0.78 for dense sand, and 0.70 for extremely dense sand. Exceeding the specified limitations of the DMF may result in increased permanent settlements, even during moderate and strong earthquakes. A DMF value exceeding 1 has the potential to induce damages to the superstructural elements.

Likewise, the likelihood of surpassing different performance thresholds is contingent upon the DMF. The observation of the likelihoods are made at an average spectral acceleration of 0.5g as per the Indian standards. It is observed that irrespective of the soil type and decreasing the foundation sizes, causes the likelihood of settlement surpassing 25mm is increased constantly. Over designed footing i.e., CFD1 experiences the lowest settlement than all other footing types irrespective of the soil type. On the other hand the extent of likelihood is not significantly affected for the foundation proportions from CFD2 to MRF1. This suggests that decreasing the foundation dimensions may not always leads to excessive tolerable limits of settlements. Instead it may enhance in reducing the seismic force demands of the superstructure member which they support i.e., shear wall in this case. The range of likelihood is less than 15% for all the settlement based limit states. Whereas for MRF2, HRF1 and HRF2 footing the likelihood of attaining 25mm settlement is higher than 40% irrespective of the soil type. In the similar manner the likelihood of attaining 40mm settlement is found to be around 20% to 25%. However, the probability of reaching the CP level of 60mm is found to lower than 10% for all the soil types considered. Contradiction to this, the likelihood of reaching the IO and LS level of drift is higher for all the foundation types considered. For reaching the CP level of roof drift the likelihood for all the footings except the over designed footing is higher than 40%. It is evident from the combined responses noted from the hysteretic behaviour and fragility functions that CFD1 and CFD2 experiences higher flexural deformations than the rocking deformations followed by SLR, MRF and HRF footings. Higher the flexural deformations higher will be the damage accumulated by the shear wall member, which may cause the retrofitting a difficult task. On the other hand slight rocking footings has higher flexural displacement and lower settlement based failure criteria. The magnitude of contribution to the flexural deformations of superstructure decreases with increasing foundation rocking scenario. However, HRF1 and HRF2 may experience higher settlement and may dissipate energy only through the foundation soil interface rather than utilizing the flexural deformation ability of the superstructure. Therefore it is preferred to have SRF type of footing for medium dense sand and MRF type of footing for dense and very dense sand.

Additionally for the RC framed structures without shear wall, the fragility evaluation reveals that the likelihood of collapse is greatly decreased for structures on dense sand and very dense sand when foundation rocking occurs. However, when it comes to medium dense sand, there is an analogous collapse probability for both rocking foundation rocking and fixed base counterparts. Furthermore, it should be noted that in the case of fixed base structures, total deterioration occurs as a result of the accumulation of greater seismic force requirements solely in the superstructure, without effectively utilizing the strength and stiffness of the foundation and underlying soil. likewise, when the foundation rocking increases, the efficient utilization of the strength and stiffness of the soil beneath the foundation is enhanced. Consequently, this investigation demonstrates that it is more advantageous to design the footings with reduced proportions, taking into account reduced earthquake loads (EL-75 and EL-50). These reduced loads are less likely to cause structural damage compared to the fixed base counterparts.

Conclusion and recommendations

8.1 Overview

The present thesis work concentrates on the optimum size reduction for the rocking foundation from the conventionally designed footings and its corresponding influence in the seismic performance RC framed building. This chapter briefly summarizes the entire study carried out for thesis work from Chapter 4 to Chapter 7, and outlines the major conclusions. Based on the observations, possible recommendations for size reduction is provided along with the limitations and possible future scope for further studies.

8.2 Summary and conclusions

Construction of multistorey RC building with or without shear walls is quite commonly adopted construction technique in the Indian urban environment. Generally it is preferred to have shallow foundation to support these multi storey building and also to safely transfer the loads from the superstructure to the underlying soil medium. The conventional footing design methodology neglects the effects of nonlinear behaviour in the supporting soil medium during seismic events and assumes to dissipate entire seismic energy through superstructural elements itself. However considering non linear behaviour of underlying soil medium provides additional flexibility which in turn improves the overall seismic performance of the building. This can be achieved with the provision of under proportioned rocking foundation. The present thesis work examines the effect of increasing degree of foundation rocking on the seismic behavior RC building and compares with those of the fixed base, conventionally designed footings resting on three different soil types

The seismic behaviour of the RC building with and without shear walls is investigated through an exhaustive analytical study with the help of OpenSees. The entire thesis work is subdivided into three parts and their major conclusions drawn are presented:

1. Influence of foundation rocking on the inelastic behaviour of RC framed building without shear wall

- The provision of rocking foundation lengthens the fundamental natural period of the building by about 50%, when compared with the conventionally designed footings.
- The capacity curves from nonlinear static analysis reveals that the inclusion of a rocking foundation prolong the yielding span of structural elements while

maintaining their respective strength levels. Furthermore, comparable responses are observed in both the elastic segment and the post-peak segment.

- Nonlinear seismic analysis demonstrates a substantial reduction in the seismic force demands of the superstructure as the rocking effects increase. There is an observed correlation between the reduction in seismic force demands and an accompanying increase in settlement demands at the foundation level. But still, the witnessed settlement demands are within the permissible limits defined by Indian codal regulations.
- The dense and very dense sand provide the most favorable conditions for rocking foundations in comparison to the medium dense sand.
- When considering various embedment depths of the footing, there is barely any impact on overall responses.

2. Influence of foundation rocking on the inelastic behaviour of RC framed building with shear wall

- It can be observed that an enhancement in the fundamental period results in a decreasing trend in the acceleration criteria for rocking foundation, thus leading to a decrease in base shear and moments at the base of the shear wall.
- The numerical analysis findings indicate a significant increase in peak roof displacement along the height of the structure when the shear wall footing is allowed to rock. Building designed for zone II exhibit a greater degree of increase in peak roof displacement compared to building designed for zone V. Furthermore, the degree of peak roof displacement reduces as the height of the structure increases.
- The examination of peak shear wall settlement responses indicated that foundations proposed according to conventional methods demonstrate lower levels of settlement compared to foundations designed using rocking techniques.
- The foundation for shear walls can be designed by taking into account by reducing the earthquake loads for foundation design by maximum of 40% for zone V building and a maximum of 60% for zone II building. Beyond which, excessive seismic displacement demands are observed.
- The most favourable conditions are observed for the seismic design level of zone V that the zone II design. Similarly dense and very dense sand conditions shows most favourable conditions than medium dense sand.

3. Influence of foundation rocking on the inelastic behaviour of RC framed building with and without shear wall

- Under the moderate and strong seismic action of the moment from the shear wall to its foundation is reduced by more than 2 times when the effect of rocking increases. This is accompanied with the increased rotation and settlement demands. The flexural deformation observed from moment curvature relationship at the

bottom of shear wall reveals that inelastic responses are greatly reduced due to the incorporation of rocking foundation. From the shear sliding responses, it is evident the rocking is dominated than the sliding responses.

- In the context of high impact strong ground motion, conventional footings typically demonstrate greater moment and foundation rotation in conjunction with flexural displacements. This observation suggests that the over designed footing may not always yield advantageous outcomes for superstructure.
- It is noted that seismic responses are primarily governed by the ratio of moment carrying capacities between foundation and supporting shear wall. When the moment carrying capacity of foundation is higher than the shear wall, the damages in superstructure will be higher. In contrast if the foundation moment carrying capacity is much lower than the shear wall, very large permanent settlements at the foundation level will be encountered. Therefore the threshold level for reduction in foundation moment carrying capacity is observed to be 0.87 for medium dense sand, 0.78 for dense sand and 0.7 for very dense sand.
- Similarly, the fragility curves the building are also largely dependent on the relative moment carrying capacities. Decreasing the moment carrying capacities causes the increasing likelihood of attaining the settlement of 25mm. However for attaining the 60mm of settlement the likelihood is not greatly affected. This suggests that under proportioning the footing may not leads to excessive settlements beyond tolerable limits. Ultimately, effective reduction seismic force demands can be achieved by relaxing the settlement criterion.
- On the other hand, the increase in the likelihood of roof drift occurs due to the rocking nature, whereas the flexural strength of the shear wall is greatly preserved with increasing foundation rocking.
- Additionally for the RC framed building without shear wall, the fragility evaluation reveals that the likelihood of collapse is greatly decreased for building on dense sand and very dense sand when foundation rocking occurs. However, when it comes to medium dense sand, there is an analogous collapse probability for both rocking foundation rocking and fixed base counterparts. Furthermore, it should be noted that in the case of fixed base building, total deterioration occurs as a result of the accumulation of greater seismic force requirements solely in the superstructure, without effectively utilizing the strength and stiffness of the foundation and underlying soil.

8.3 Limitations of the present study

- The present thesis work is carried out by considering the design and detailing procedures prescribed as per Bureau of Indian Standard (BIS). Hence, the findings of this study may not be applicable for other codal provisions. In addition to that the

findings of this study may not be applicable to important building such as nuclear power plants.

- The present work concentrates regular and symmetrical building and hence the findings of this study may not be suitable for irregular or unsymmetrical building.
- The present thesis work neglects the effect of ground water table during the bearing capacity estimations and hence this findings of this study may not be applicable to the coupled soil water systems.
- The present study assumes that the soil structure and structure lies on the uniform soil layer. In case of layered soil the seismic behaviour will vary from the findings that are mentioned in this study.
- This study does not investigate the intricate mechanics of foundation rocking caused by the depth of embedment and its influence on foundation rocking. Hence, a comprehensive examination of the failure mechanism resulting from foundation embedment depth, including dilatation and soil ravelling characteristics, necessitates the utilization of rigorous nonlinear finite element analysis.

8.4 Recommendations for the future study

To strengthen the understanding in the area of rocking foundation, additional efforts are needed in future. This may help in framing the proper design methodologies for considering the rocking foundation as viable alternative to counteract seismic forces. The future research strategies that can be helpful in the development of rocking foundation are as follows:

Various categories of soil may be taken into consideration: Examining the behavior of foundation rocking on liquefiable soil may offer varying perspectives on the seismic force and displacement demands. Also, soils having both cohesion and angle of internal friction can be taken into account. Also the study can be extended for layered soil in future.

Parametric studies on foundation typologies can be considered: The present study concentrates on the isolated footing of mostly square footings except for shear walls it is proportioned as rectangular footings. Hence, various foundation geometries and types can be carried out.

Parametric studies on structural typologies can be considered: The present study concentrates majorly on the regular building. Hence studies on the effect of irregularity in the super structural elements affecting the seismic performance can be conducted to identify the favourable and unfavourable conditions for rocking.

Parametric studies on 3D models: Only the 2D models are considered here, hence it can be considered to extend it for 3D structural and foundation modelling to identify the most favourable conditions for rocking foundation.

Effect of ground motion parameters can be performed: The effect of ground motion parameters on the impacts of the foundation rocking, correspondingly design methodologies can be framed based upon the observations.

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