Comparison of Fixed Base and Base Isolated Hotel Building Located in Kufri

A Thesis

Submitted in Partial Fulfillment of Requirements for the Degree of

MASTERS OF TECHNOLOGY
IN
CIVIL ENGINEERING

With specialization in
Structural Engineering

Under the Supervision of

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To

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HIMACHAL PRADESH, INDIA
JUNE, 2016

CERTIFICATE

This is to certify that the work which is being presented in the project title **"Comparison of Fixed Base and Base Isolated Hotel Building Located in Kufri"** in fulfillment of the requirements for the award of the degree of Master of technology and submitted to Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Ishant Kukreja** during a period from August 2015 to May 2016 under the supervision of **Mrs Poonam Dhiman** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of my knowledge.

Date: 6th June, 2016

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ACKNOWLEDGEMENT

Foremost, I would like to express my sincere gratitude to my advisor **Mrs. Poonam Dhiman** for the continuous support of my seminar study, for her patience, motivation, enthusiasm, and immense knowledge. Her guidance has helped me in all the time of this study and writing of this report.

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I also take the opportunity to thank all my friends who have directly or indirectly helped me in my project work.

Last but not the least I would like to thank my parents, who taught me the value of hard work by their own example.

Date: 6^{th} June, 2016 **ISHANT KUKREJA**

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ABSTRACT

Base isolation technology works by separating or greatly reducing the lateral movement of a building’s superstructure from the movement of the ground/foundation during a seismic event. The ultimate purpose of a base isolation system is to reduce the seismic forces exerted onto a reducing the superstructure’s spectral accelerations. These accelerations are reduced both by increasing the effective fundamental period of the isolated structure and through damping caused by energy dissipated within the isolation bearing.

Here, 4 story commercial building was analyzed which is located in Kufri it is a small hill station in Shimla district of Himachal Pradesh state in India. It is located 13 km from the state capital Shimla on the National Highway No.22 Seismic Zone IV. Kufri is located at 31.10°N 77.25°E. It has an average elevation of 2,290 meters (7,510 feet). Strata is hard rocky with bearing capacity around 220kn/m2.

The main focus of this study is to compare response of fixed-base and base-isolated model on the basis of Member Forces, Story Drifts, Time Period. From the work done, it was analyzed that there was large reduction in member forces. Member forces were reduced to about 40% by implementing base isolation technique. Time period of the structure was increased as compared to fixed base with results in decreasing acceleration and further there was decrease in story drifts of the structure.

Base Isolation technique requires high engineering skills of the engineer and also skilled labour which is not easily available. Further it has economic limitations which hampers the use of base isolation technique.

CHAPTER 1
INTRODUCTION

1.1 Earthquake

Buildings behavior in earthquakes depends on various uncertainty factors. These uncertainties originate from different sources, earthquake nature, components behavior, and the analytical methods. Therefore, the response of the building is dependent on ground motions and an assembly of individual responses of structural and nonstructural components in a fully probabilistic framework. Experience in past earthquakes has demonstrated that many common buildings and typical methods of construction lack basic resistance to earthquake forces. In most cases this resistance can be achieved by following simple, inexpensive principles of good building construction practice. Adherence to these simple rules will not prevent all damage in moderate or large earthquakes, but life threatening collapses should be prevented, and damage limited to repairable proportions. These principles fall into several broad categories:

i. Planning and layout of the building involving consideration of the location of rooms and walls openings such as doors and windows, the number of stories, etc. At this stage, site and foundation aspects should also be considered.

ii. Lay out and general design of the structural framing system with special attention to furnishing lateral resistance.

iii. Consideration of highly loaded and critical sections with provision of reinforcement as required.

Studies have provided a good overview of structural action, mechanism of damage and modes of failure of buildings. From these studies, certain general principles have emerged:

i. Structures should not be brittle or collapse suddenly. Rather, they should be tough, able to deflect or deform a considerable amount.

ii. Resisting elements, such as bracing or shear walls, must be provided evenly throughout the building, in both directions side-to-side, as well as top to bottom.

iii. All elements, such as walls and the roof, should be tied together so as to act as an integrated unit during earthquake shaking, transferring forces across connections and preventing separation.

iv. The building must be well connected to a good foundation and the earth. Wet, soft soils should be avoided, and the foundation must be well tied together, as well as tied to the wall.

v. Care must be taken that all materials used are of good quality, and are protected from rain, sun, insects and other weakening actions, so that their strength lasts.

vi. Unreinforced earth and masonry have no reliable strength in tension, and are brittle in compression. Generally, they must be suitably reinforced by steel or wood.

1.2 Categories of Buildings

For categorizing the buildings with the purpose of achieving seismic resistance at economical cost, three parameters turn out to be significant:

i. Seismic intensity zone where the building is located,

ii. How important the building is, and

iii. How stiff is the foundation soil.

A combination of these parameters will determine the extent of appropriate seismic strengthening of the building.

1.3 Seismic Zones in India

In most countries, the macro level seismic zones are defined on the basis of Seismic Intensity Scales. In this guide, we shall refer to seismic zones as defined with reference to MSK Intensity Scale as described:

Zone II: Risk of Minor Damage.

Zone III: Risk of Damage.

Zone IV: Risk of Collapse and Heavy Damage.

Zone V: Risk of Widespread Collapse and Destruction.

Table 1 Zone Factor (Z)

Seismic Zone	II	III	IV	V
Seismic Intensity (Z)	0.10	0.16	0.24	0.36

1.4 Bearing Capacity of Foundation Soil

Three soil types are considered here:

i. **Type I:** Rocky or hard soil these soils which have an allowable bearing capacity of more than 120kn/m2.

ii. **Type II:** Medium soil these soils, which have allowable bearing capacity less than or equal to 120kn/m2.

iii. **Type III:** Soft soil these soils, which are liable to large differential settlement or liquefaction during an earthquake.

Buildings can be constructed on firm and soft soils but it will be dangerous to build them on weak soils. Hence appropriate soil investigations should be carried out to establish the allowable bearing capacity and nature of soil. Weak soils must be avoided or compacted to improve them so as to qualify as firm or soft.

1.5 Assessment

The assessment type is based on quantifying the consequences of buildings response to earthquake. The performance measures must be meaningful and representative of parameters important to decision makers. In this methodology performance measures are probable future earthquake impacts expressed as follows:

i. **Casualties:** the number of deaths and injuries of a severity requiring hospitalization;

ii. **Repair cost:** including the cost of repairing or replacing damaged buildings and their contents;

iii. **Repair time:** the period of time necessary to conduct repairs or replace damaged contents, building components or entire buildings; and

iv. **Unsafe Placards:** the probability that a building will be deemed unsafe for post-earthquake occupancy.

Performance assessment is a complicated procedure, which requires considering all uncertainties involved. At this stage, using the information provided for each uncertain factor on the median, dispersion and types of distribution, a simulation is carried out by combining them using Monte Carlo technique. This simulation is repeated a large number 4 of times until an estimation of performance measures is obtained. Depending on computing power, this procedure may take a few hours to several days or to complete.

CHAPTER 2
PROBLEM DEFINITION

2.1 Introduction

The primary goal of the engineering effort is to benefit the society in terms of human life safety in extreme events like an earthquake. Aside from the human safety, reducing the environmental and economic impact of a disaster like a big earthquake is desirable. The present seismic design principles do not provide any clear recommendations for the selection of an optimal structural system solution, among the various alternatives. Previous performance-based design methodologies provide guidance and recommendations for various structural systems to satisfy the requirements of a selected performance objective. Such recommendations are made independent of the fact that of how different structural designs are compared in terms of the costs. On the other hand, the general inception of the engineering community on the cost consequences of high performance structural systems including base isolation has limited their use. To address this problem, high performance base isolation systems require a complete reevaluation considering initial and long term seismic costs. PEER performance assessment methodology is able to provide a powerful means for estimation of long term consequences of different design alternatives. The methodology is a big step forward in the performance-base design evolution path; but the applicability is restricted to due to its high analysis costs and time.

2.2 Building Data

Building is located in Kufri, it is a small hill station in Shimla district of Himachal Pradesh state in India. It is located 13 km from the state capital Shimla on the National Highway No.22. Kufri is located at 31.10°N 77.25°E. It has an average elevation of 2,290 meters (7,510 feet).

a) **Soil Type:** Hard Rocky Soil, bearing Capacity around 220kn/m2.

b) **Size of Members:**

Column: 300X600 mm

Primary Beam: 230X500 mm

Secondary Beam: 230X350 mm

Slab Thickness: 125mm

c) **Seismic Zone:** Zone IV with Z (zone factor) equal to 0.24.

In this specified building number of columns are 12, number of beams are 16. The load applied is according to IS 875 and IS 1893 for earthquake loads. Story height is 3m for each successive floor.

2.3 Objectives

For the good of the society, the future consequences of todays' decisions are required to be accounted for in a sustainable design. The effectiveness of base-isolation in reducing the impact due to large earthquake is evident. This fact serves both toward decreasing the social and environmental impact in a sustainable development. The main objective of this research is to compare the long term consequences of the high performance and the fixed-base structural systems in including repair costs, repair time, business interruption costs, fatalities and injuries. This research mainly focuses on:

i. Clarifying the potential of base isolation systems in providing an economical yet reliable and safe design alternative. The focus is to assess performance of base isolated and non-isolated designs considering initial costs and future losses during the useful life span of the building to help owners and designers on making decisions.

ii. The effect of different design seismic loads, for example seismic demands associated with different risk categories in International Building Code (2012), on the total costs.

iii. Perform a comparative analysis on how the fixed-base nonlinearly performing structural systems are compared with the corresponding linearly performing isolated systems in terms of the seismic demand forces.

iv. Compare the performance of fixed-based and isolated models based on a simplified response index.

2.4 Fundamentals of Base Isolation

Base isolation technology works by separating or greatly reducing the lateral movement of a building's superstructure from the movement of the ground/foundation during a seismic event.

To allow for this difference in lateral movement while still supporting the weight of the superstructure, base isolation bearings are designed to be very flexible laterally while being stiff vertically. This base condition is in contrast to a typical fixed-base structure, in which the connections between the superstructure and its base/foundation are rigid and translation of the superstructure is resisted in all directions. The difference between these two base conditions is illustrated in Figure below. The ultimate purpose of a base isolation system is to reduce the seismic forces exerted onto a reducing the superstructure's spectral accelerations. These accelerations are reduced both by increasing the effective fundamental period of the isolated structure and through damping caused by energy dissipated within the isolation bearing.

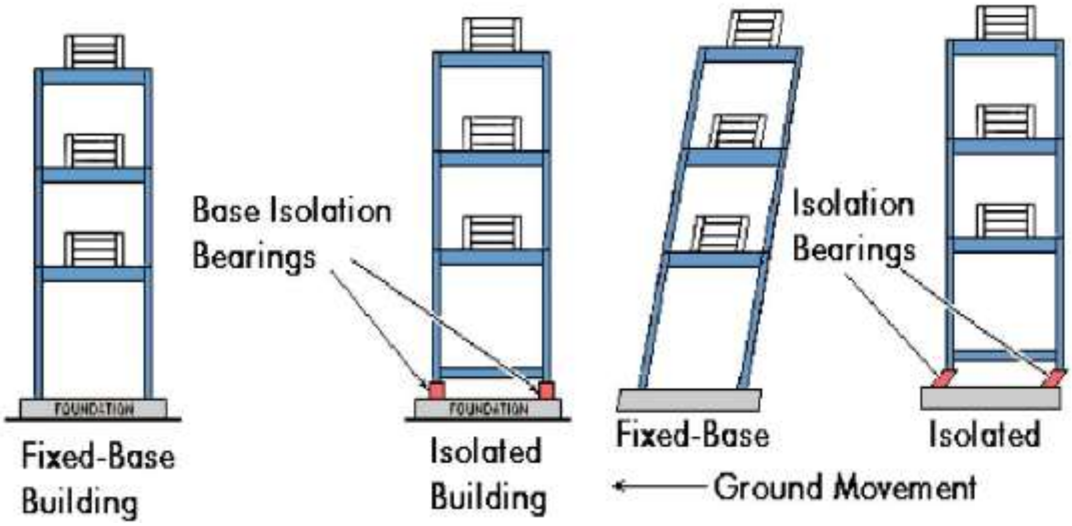


Figure 1 Fixed base and Isolated base

2.5 Comparison between Fixed and Isolated Base

Base isolation (BI) is a mechanism that provides earthquake resistance to the new structure. The BI system decouple the building from the horizontal ground motion induced by earthquake, and offer a very stiff vertical components to the base level of the superstructure in connection to substructure (foundation). It shifts the fundamental lateral period, dissipates the energy in damping, and reduces the amount of the lateral forces that transferred to the inter-story drift, and the floor acceleration. The structural bearing criteria include vertical and horizontal loads, lateral motion, and lateral rotation that transferred from the superstructure into the bearing and from the bearing to the substructure. Bearing allows for stress-free support of the structure in terms of (1) they can rotate in all directions, (2) they deform in all directions, (3) they take horizontal forces (wind, earthquake). In this study lead rubber bearings are used as the base isolation system.

CHAPTER 3
ANALYSIS AND DESIGN OF FIXED BASE

Introduction

In this chapter analysis and design of hotel building specified in previous chapter is done. Method of analysis is linear static analysis, after fixed base analysis and response of isolated base is carried out then comparison of the two is done on the basis of story drift, member forces etc.

3.1 Load Combinations

Load Combinations are taken as per **IS 1893** and are as follows:

In the limit state design of reinforced and pre-stressed concrete structures, the following load combinations shall be accounted for:

I. ***1.5(DL+LL)***

II. ***1.2(DL+ZL+EL)***

III. ***1.2(DL+ZL-EL)***

IV. ***1.5(DL+EL)***

V. ***1.5 DL-EL)***

VI. ***0.9DL+ 1.5EL***

VII. ***0.9DL- 1.5EL***

3.2 Combination for Two or Three Component Motion

When responses from the three earthquake components are to be considered, the responses due to each component may be combined using the assumption that when the maximum response from one component occurs, the responses from the other two components are 30 percent of their maximum. All possible combinations of the three components (*ELx, ELy and ELz*) including variations in sign (plus or minus) shall be considered, Thus, the response due earthquake force (EL) is the maximum of the following three cases:

I. ***±ELx±0.3ELy±0.3ELz***

II. ***±ELy±0.3ELx±0.3ELz***

III. ***±ELz±0.3ELx±0.3Ely***

Where x and y are two orthogonal directions and z is vertical direction.

3.3 Method of Analysis

Here we have used linear static analysis to analyze the structure. In linear static analysis displacements, strains, stresses, and reaction forces under the effect of applied loads are calculated.

A series of assumptions are made with respect to a linear static analysis:

a) Small Deflections

Determine whether the deflections obtained or predicted are small relative to the size of the structure. For thin structures, a deflection that is less than the thickness would be considered a small deflection. The deflection between two supports should be only a small percent of the distance between supports. This is especially true if the deflection causes a differential stiffness effect such as mid-plane stretching of a clamped plate.

b) Small Rotations

In linear codes all rotations are assumed to be small. Any angle measured in radians should be small enough that the tangent is approximately equal to the angle. Using this assumption, a ten-degree angle introduces an error of approximately one percent in all related calculations. A thirty-degree angle results in approximately a 10 percent error in deflection due to rotations assumed linear.

c) Material Properties

Linear solvers assume that all material behaves in a linear elastic manner. Some materials have a non-linear elastic behavior, and although they do not necessarily yield, they still result in non-linear structural behavior and require non-linear codes for solution. If a structure is to be loaded beyond its yield point, non-linear analysis would also be required. See the figure below for a comparison of material behavior. Some materials have a non-linear elastic behavior, and although they do not necessarily yield, they still result in non-linear structural behavior and require non-linear codes for solution

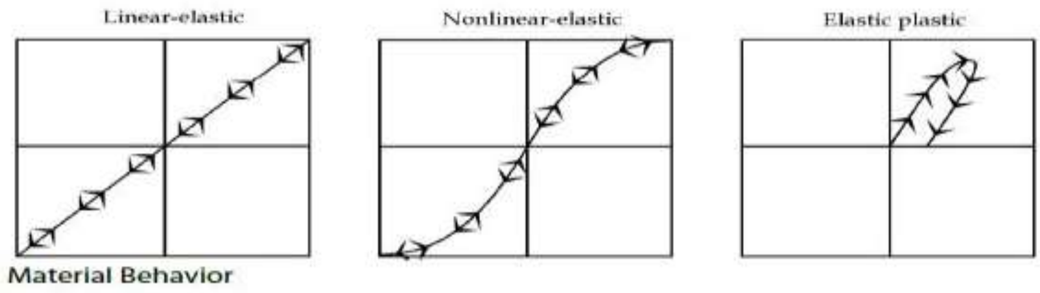


Figure 2 Material Behavior

d) Constant Boundary Conditions

In order to correctly use a linear finite element program, the boundary conditions must not be dependent on the load application. The figure below illustrates an example where this is not true. A structure placed on an elastic foundation might tend to physically separate under the load, resulting in the formation of a gap. This gap is dependent on the load and therefore behaves as non-linear.

3.4 Staad Input File

Below is the Staad GUI file for the hotel building

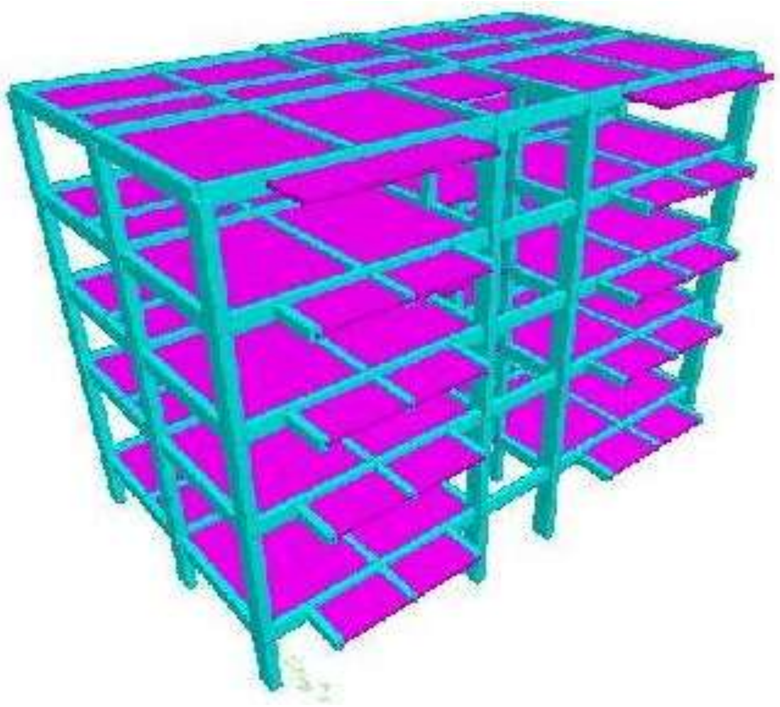


Figure 3

3.5 Analysis of Structure

As discussed method of analysis used is linear static method of analysis.

3.5.1 Support Reactions:

Table 2

Node Number	*Support Reactions (kN)*
19	2010
20	1756
21	1690
22	1763
23	2140
24	2145
25	392
26	395
27	2010
28	1685
29	913
30	916
31	1360
32	1400
33	1825
34	1825

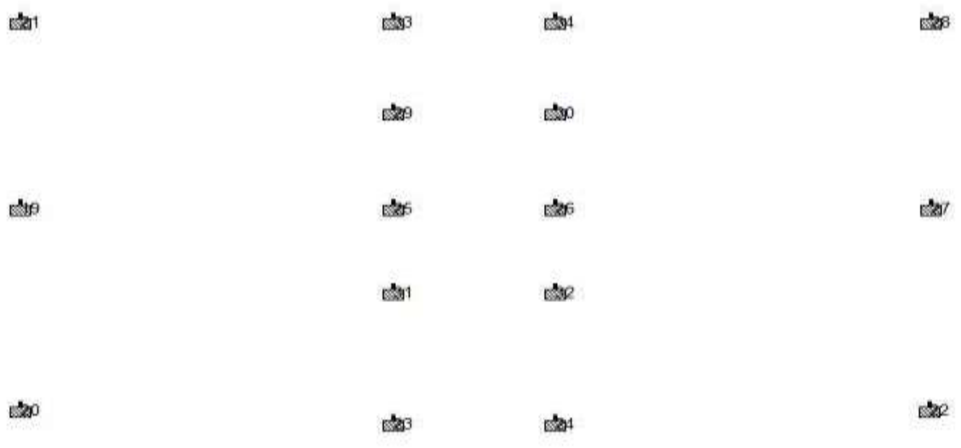


Figure 4 Node Number

3.6 Plan of the Structure

3.6.1 First of Plan: Autocad file for all floors plan.

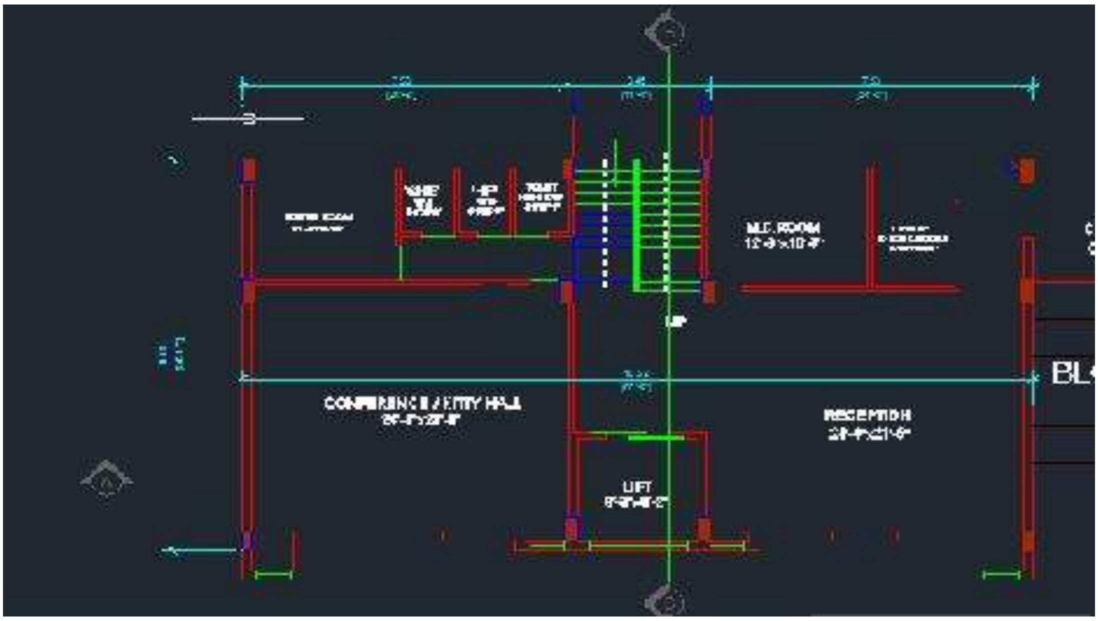


Figure 5

3.6.2 Second Floor Plan

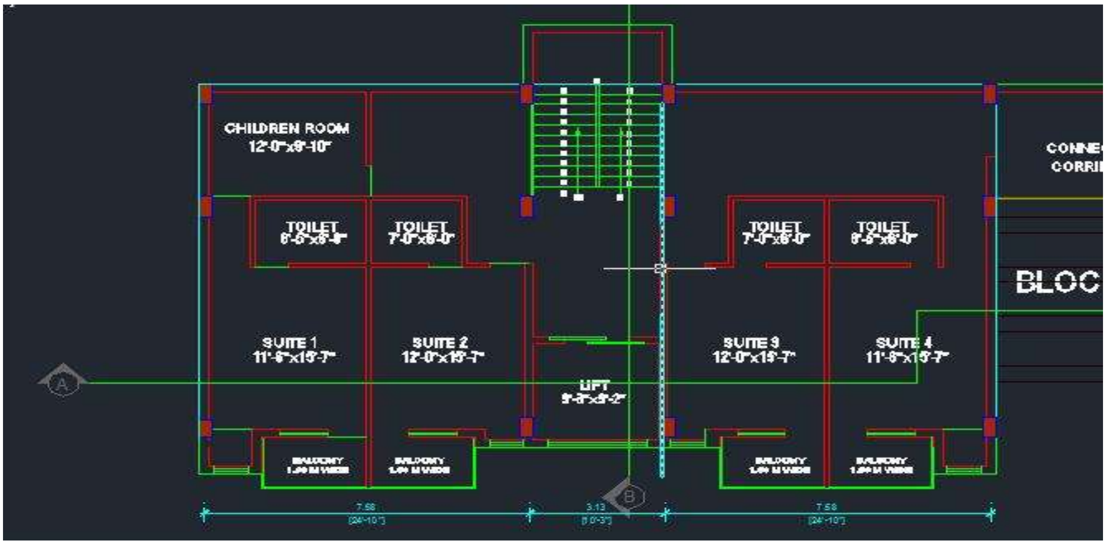


Figure 6

3.6.3 Third Floor Plan

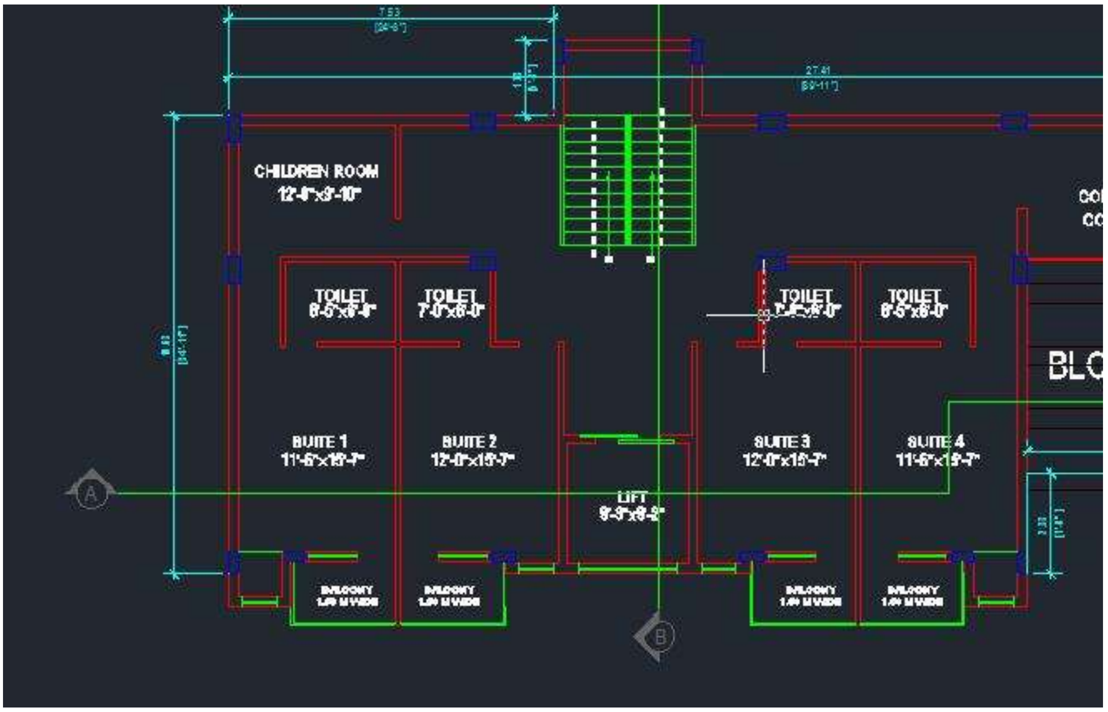


Figure 7

3.6.4 Fourth Floor Plan

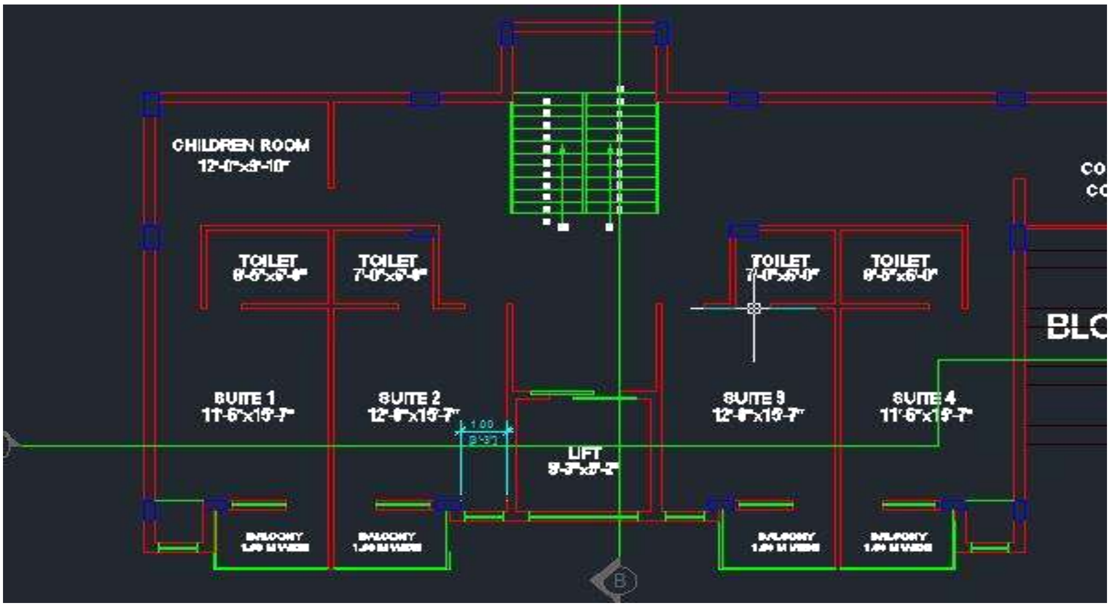


Figure 8

3.7 RCC Design of Structure

3.7.1 Foundation Design

3.7.1.1 Design of Isolated Footing: A spread footing (or isolated or pad) footing is provided to support an individual column. A spread footing is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped or hunched to spread the load over a large area. In this structure isolated footings are provided for the outer columns on the either side of the building. For rectangular footing Spread sheet is given below

Foundation for col.--A1

					0.045		
	P=1200	KN.		55	8		
		KN/m^					
	BC=150	2					
	A=	8.80					
		Provide	2.3X		3.40 FOUNDATIONS		
COL.							
SIZE		0.300	0.60		300	600	
	p=	153.45					
	Mx=	153.45	2.3	1.4	1.4	345.88	
	My=	153.45	3.4	1	1	260.875.7	
							mm
	d	790.15mm.	PROVIDE 600mm.		600	deff.=550	.
						Cm^	
	Mu/bd^2=	5.72%Ast.=		0.749	Ast.=	12.362	15.74
						Cm^	
	Mu/bd^2=	2.16%Ast.=		0.477	Ast.=	15.742	20.05

Check for Shear:

Critical Section for Shear will be at a distance of effective depth from face of col.

Max. Shear=	234.78 KN
Effective depth of Footing At This section=	330Mm
Effective width of Footing At This section=	1700Mm
Shear Stress=	0.627 8
Effective depth of Footing At This section=	264.2 9Mm
Effective width of Footing At This section=	1400Mm
Shear Stress=	0.951 8y

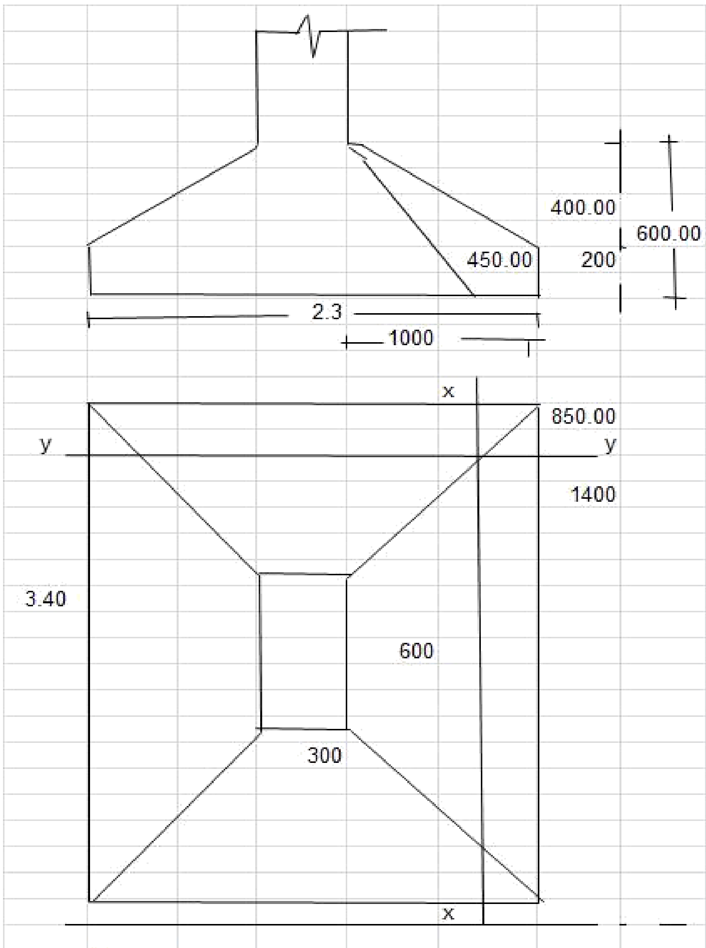
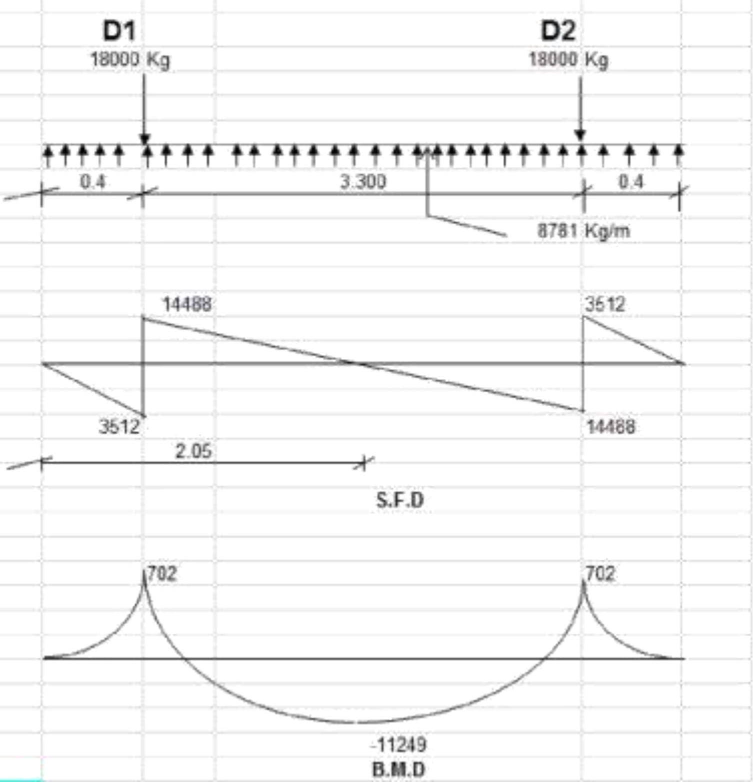


Figure 9 Isolated Footing

3.7.1.2 Combined Footing Design: A combined footing supports two columns. It issued when the two columns are so close to each other that their individual footings would overlap. A combined footing is also provided when the property line is so close to one column that a spread footing would be eccentrically loaded when kept entirely within the property line. By combining it with that of an interior column, the load is evenly distributed. A combined footing may be rectangular or trapezoidal in plan. Here, distance between two columns is less so combined footing has been provided. For combined footing spread sheet is given below. Combined footing has been used for enter columns.

COMBINED FOOTING DESIGN				
Given Load on Left column W1 (Kg)	18000		**Bearing Capacity Of Soil (T/m^2)**	**15**
Load on Right column W2 (Kg)	18000		Grade of Concrete(Mpa)	20
C/C distance between columns (m)	3.300		Grade of Steel (Mpa)	415
Area of Footing Required (m^2)	**2.64**		Net Upword Preessure on Footing (Kg/M^2)	5488
Trial Width of Footing (m)	1.6		C.G of Loads x (m)	1.65
Gives Length of Footing (m)	**1.65**		Projections p1(m)	0.4
Adopt Length of Footing (m)	4.1		p2(m)	0.4
Area of Footing Provided (m^2)	**6.56**	OK		



Design of Rib beam						
			Dia of bar (mm)	16		
Assume Width of Rib beam (mm)	650		No of Bars	5		
Required depth of Rib Beam deff mm)	432		Dia of bar (mm)	0		
Provide D(mm)	650		No of Bars	0		
d eff (mm)	615	OK	Ast Provided at **Bottom** (mm^2)	1005	OK	
			Dia of bar (mm)	20		
Ast Required at Bottom (mm^2)	53		No of Bars	5		
Minimum Ast requred at bottom(mm^2) **0.2%**	999		Dia of bar (mm)	0		
Ast Required at Top (mm^2)	857		No of Bars	0		
Minimum Ast required at Top(mm^2)**0.2%**	999		Ast Provided at TOP (mm^2)	1571	OK	
Check for Shear						
Maximum Shear Force(N)	144880			5		**20 mm Dia**
Shear Stress (N/mm^2)	0.36	Safe		0		**0 mm Dia**
%age of Steel provided p	0.25					
If %age p	0.25	tc	0.22	650		
%age p	0.5	tc	0.3	5		**16 mm Dia**
For p	0.25	tc	0.22	0		**0 mm Dia**
Shear Strength of Concrete(N)	87945		650			
Shear Force to be resisted by Shear Rein.	56936					
Provide Shear Stirrups	4	Legged	Spacing of Shear Stirrups(sv)	781	mm c/c	
Dia. Of Stirrup	10	mm				
Design of Footing						
Transverse BM at Face of Rib Beam(Kgm)	619.115		Shear force at adistance d eff. From beam	444.53	Kg	
deff (mm)	82		Shear stress	0.0110647	Mpa	
Provide D(mm)	**450**		Permissible shear stress without shear steel	0.22	Mpa	
Gives deff (mm)	394	OK	Shear Stress for %age of steel			
Ast Required (mm^2)	74		0.25	0.22		From IS :456
Minimum Ast required (mm^2) **0.25%**	985		0.5	0.3		
%age of steel provided	0.25		0.25	0.22	SAFE	

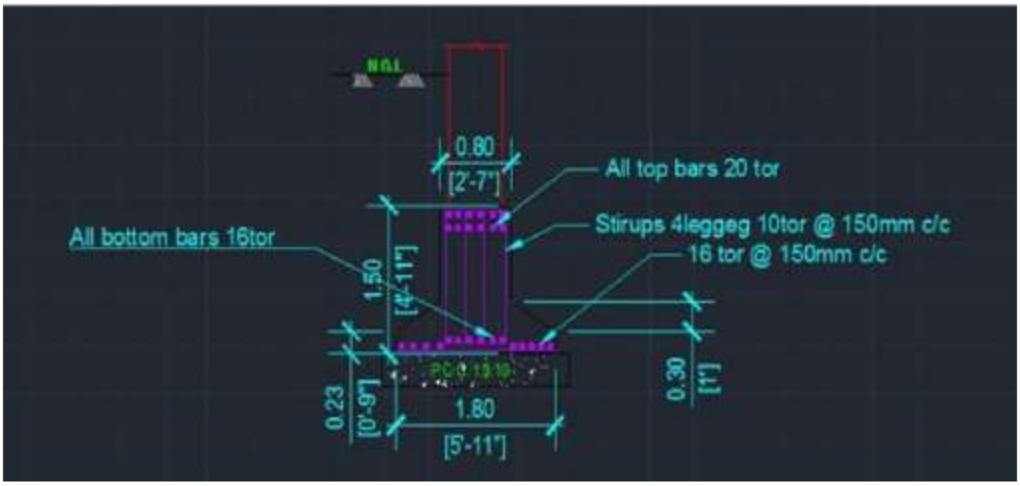


Figure 10 Detailing of Footing

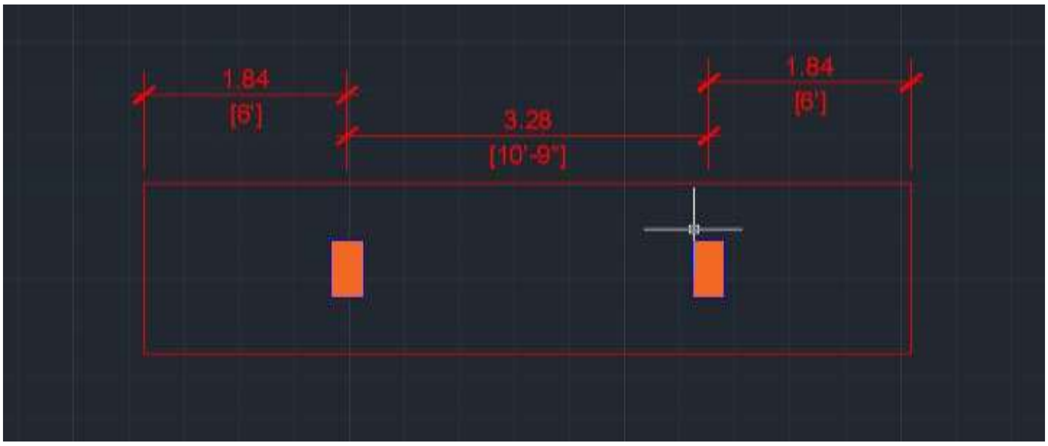


Figure 11 Plan for footing

3.7.2 Design of Column's: A column or pillar in architecture and structural engineering is a structural element that transmits, through compression, the weight of the structure above to other structural elements below. Spread sheet is given below.

length of column (mm)	5000	
width (mm) (x axis)	500	
depth (mm) (y axis)	500	
compressive strength of concrete (fck)	25	
Grade of steel (Fy)	415	
effective length (mm)	3250	
leff/width	6.5	
leff/depth	6.5	
eccentricity along width	23.16666667	
eccentricity along depth	23.16666667	
min. eccentricity along width	25	>value in b9
min. eccentricity along depth	25	>value in b10
percentage reinforcement(%)	0.8	
percentage reinforcement/fck	0.032	
Assumed bar dia	16	
clear cover (mm)	40	
d'	48	
Uniaxial Moment carrying capacity(x)		
d'/ width	0.096	
Pu (Newtons)	458000	
Pu/fckbd	0.07328	
Get Mu/fckbd from chart(Sp 16)	0.08	
Mux1 (N-mtr)	250000000	
Uniaxial Moment carrying capacity(y)		
d'/ depth	0.096	
Get Mu/fckbd from chart(Sp 16)	0.08	
Muy1 (N-mtr)	250000000	

Here we have used uniaxial columns.

Puz Calculation	
Gross Area (sq.mm)	250000
Get Puz/Ag from sp16	13.8
Puz (Newtons)	3450000
Pu/Puz	0.132753623
Mux (N-mtr)	51000000
Muy (N-mtr)	133000000
Mux/Mux1	0.204
Muy/Muy1	0.532
Mux/Mux1 + Muy/ Muy1	0.736
Area Of steel required (sq mm)	2000
Area of steel provided (sq mm)	1206

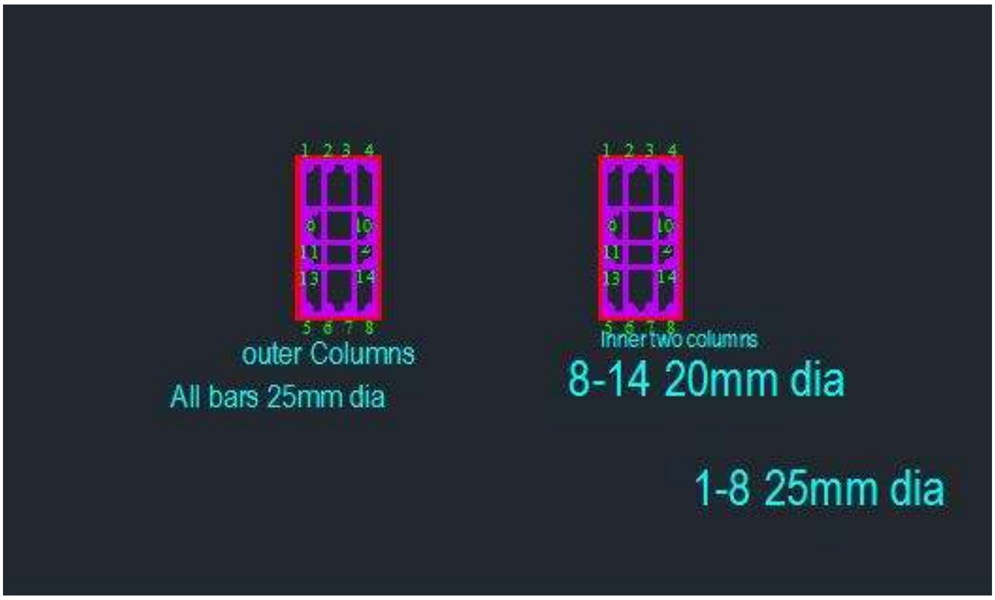


Figure 12 Detailing of columns

3.7.3 Design of Beams: A continuous beam is a statically indeterminate multi span beam on hinged support. The end spans may be cantilever, may be freely supported or fixed supported. At least one of the supports of a continuous beam must be able to develop a reaction along the beam axis.

Here we have used doubly reinforced beams, *rectangular beams with tension and compression reinforcement.* If a beam cross section is limited because of architectural or other considerations, it may happen that the concrete cannot develop the compression force required to resist the given bending moment

DESIGN OF BEAM

Inputs		
b=	400	mm
D=	500	mm
Dia of bar=	16	mm
Clear Cover=	30	mm
f_{ck}=	20	N/mm^2
f_y=	415	N/mm^2
Moment=	225	KN-m
Shear Force=	145	KN

Calculating Limiting Moment of Resistance=$M_{u\text{-}lim}$= $R_{u\text{-}max}.bd^2$

$M_{u\text{-}lim}$= 0.138 $f_{ck}bd^2$

Effective depth,d= 462 mm

$M_{u\text{-}lim}$= 235642176 N-mm

235.642 KN-m

Area of Steel, A_{st}= 1658.35 mm^2

Area of 1 Bar= 201.06 mm^2

No. of Bars Required= 8.25

9

Actual area of steel provided= 1809.54 mm^2

Moment of resistance is given by- M_{ur}= $0.87f_y.A_{st}.d(1-A_{st}.f_y/b.d.f_{ck})$

M_{ur}= 240512072.4 N-mm

240.512 KN-m

Needs Compreesion reinf

Provide	9	Nos. of	16	dia bars

Calculating Compression reinforcement, A_{sc}

Calculating, M_u/bd^2= 2.635

Corrosponding, P_c=	0.045	%	(Table-6.2, P-119, A.K. Jain or SP-16)
Hence, A_{sc}= P_c.b.d/100		mm^2	
A_{sc}=	83.16	mm^2	

Dia of bar to be provided as Compression Reinforcement= 12 mm

Area of bar=	113.1	mm^2
No. of Bars Required=	0.11	
	1	

$R_{u\text{-}max}$=

$0.148f_{ck}$	Fe-250
$0.138f_{ck}$	Fe-415
$0.133f_{ck}$	Fe-500

Check for Shear-				
Maximum Shear force, V_u=		145	KN	
Shear Stress, τ_v=		V_u/b.d		
	τv=	0.78	N/mm^2	
Also , P_t=	100 A_{st}/b.d			
	0.98	%		
Corrosponding value of τc=		0.52	N/mm^2	(Design Shear Strength of Concrete, Table-19, P-73, IS-456)
		Shear Reinf.Required		
Strength of Shear Reinforcement, V_{us}= V_u-τ_c.b.d				
	V_{us}=	48904	N	
Dia of stirrup bar=		8	mm	
	f_y=	415	N/mm^2	
Legged=		2		
	Asv=	100.53	mm^2	
	Spacing, S_v=	0.87.f_y.A_{sv}.d/V_{us}		
		342.89	mm	

3.7.4 Design of Slab

3.7.4.1 One way slab: One-way slabs are those slabs with an aspect ratio in plan of 2:1 or greater, in which bending is primarily about the long axis. In heavily loaded slabs, the thickness is often governed by shear or flexure, while in lightly-loaded slabs, the thickness is generally chosen based on deflection limitations.

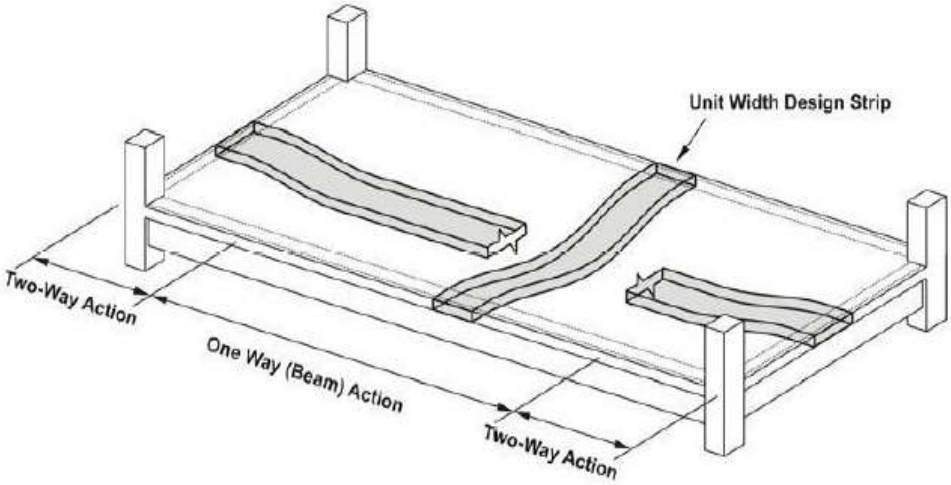


Figure 13 One Way Slab on beams

One-way slabs are designed for shear by assuming that they act as a series of adjacent beams spanning in one direction, it is reasonable to conclude that the size effect that governs the shear behavior of thick beams will apply. In one-way slabs supported on stiff supports along only two sides no redistribution will be possible, and the full width of the slab may be called upon to resist the full shear.

It appears reasonable to assume that one-way slabs will exhibit a size effect in one-way shear. Because one-way slabs are often dimensioned to avoid the use of stirrups, it would appear that they would therefore be particularly vulnerable to the size effect.It appears reasonable to assume that one-way slabs will exhibit a size effect in one-way shear. Because one-way slabs are often dimensioned to avoid the use of stirrups, it would appear that they would therefore be particularly vulnerable to the size effect. Spread sheet is given below

Here the aspect ratio Ly/Lx is greater than 2 hence the slab is designed as a one way slab.

One Short Edge Discontinuous:

Basic dimensions of slab		=Lx	Ly
		1.2	3.6
Basic Ly/Lx ratio		=3.000	>2
		Hence designed as an one way slab	
Clear cover to reinforcement	d'	=	25mm
Provided overall depth	D	=	175.00mm
Effective depth	d	=	145.00mm
Diameter of bar	ϕ	=	10mm
Select Grade of Concrete	fck	=	20N/mm²
Select Grade of Steel	fy	=	415N/mm²

Load calculation :

Dead load of the slab	DL	=	4.375kN/m²
Floor finish(Roof finish)	FF	=	0kN/m²
Live load	LL	=	10kN/m²
Total load	TL	=	14.375kN/m²

Moment and Area of Steel calculations:

Mu kN.m	Mu/bd² N/mm²	Pt %	Ast reqd mm²	Min Ast mm²	Dia of bar mm	Spacing mm	Ast pro mm²	
3.88	0.18	0.05%	74.94	174	10	250	314.16	safe
	0.06							
	0.05							
	0.03							

Check for Deflection

The effective depth provided = 145.000mm

From figure 3 of I.S 456:1978 modification factor is

Modification factor = 1.59

Required depth under deflection consideration = 45.87mm

HENCE SAFE

○ Simply suppurted

◉ continuous

check for shear

Factored Load	sheartouv force		Pt	touc
25.875	15.525	0.10707	0.21666	0.338

20

415

3.7.4.2 Two way Slab: A rectangular slab supported on four edge supports, which bends in two orthogonal directions and deflects in the form of dish or a saucer is called two way slabs. For a two way slab the ratio of ly/lx shall be greater than 2.

Since, the slab rest freely on all sides, due to transverse load the corners tend to curl up and lift up. The slab losess the contact over some region. This is known as lifting of corner. These slabs are called two way simply supported slabs. If the slabs are cast monolithic with the beams, the corners of the slab are restrained from lifting. These slabs are called restrained slabs. At corner, the rotation occurs in both the direction and causes the corners to lift. If the corners of slab are restrained from lifting, downward reaction results at corner & the end strips gets restrained against rotation. However, when the ends are restrained and the rotation of central strip still occurs and causing rotation at corner and the end strip is subjected to torsion.

Below is the figure showing behavior of two way slab.

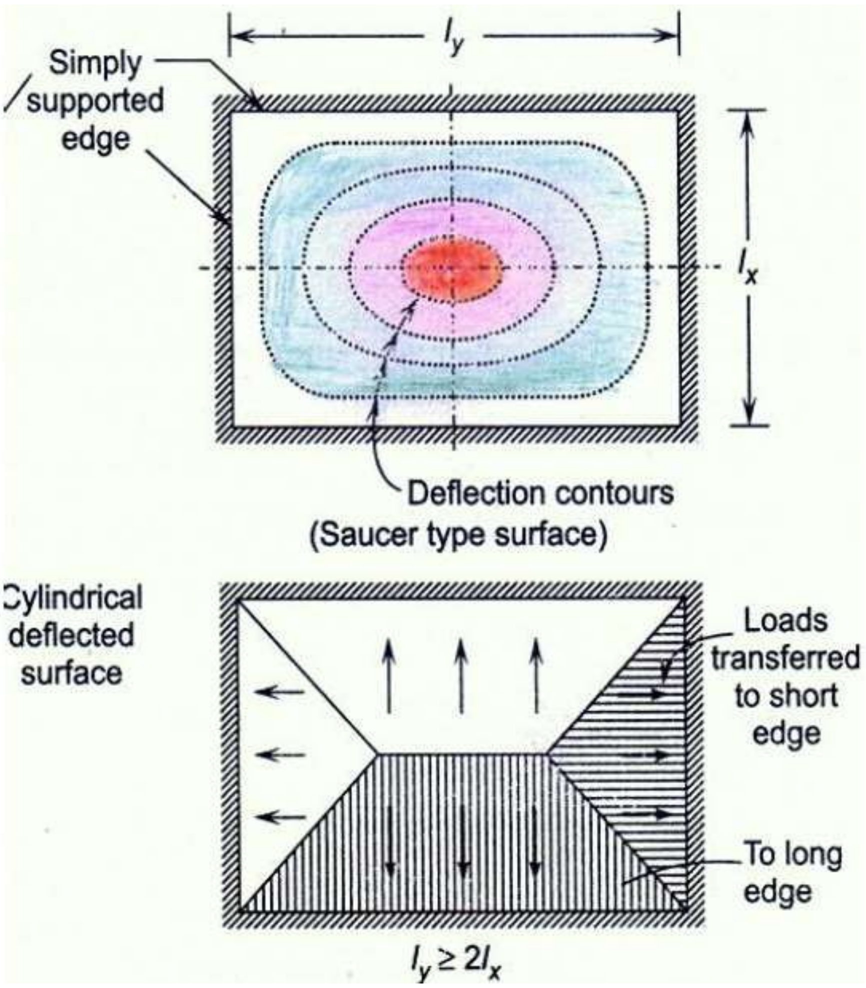


Figure 14 Behaviour of two way slab

These slabs are called restrained slabs. At corner, the rotation occurs in both the direction and causes the corners to lift. If the corners of slab are restrained from lifting, downward reaction results at corner & the end strips gets restrained against rotation. The slab loses the contact over some region. This is known as lifting of corner. These slabs are called two way simply supported slabs. If the slabs are cast monolithic with the beams, the corners of the slab are restrained from lifting.

Here the aspect ratio Ly/Lx is less than 2 hence the slab is designed as a two way slab.

Interior panel:

Basic dimensions of slab		=	Lx: 3.7	Ly: 4.8
Basic Ly/Lx ratio		=	1.297 <2	
			Hence designed as two way slab	
Clear cover to reinforcement	d'	=	20	mm
Provided overall depth	D	=	130.00	mm
Effective depth	d	=	104.00	mm
Diameter of bar	ϕ	=	12	mm
Select Grade of Concrete	fck	=	20 ▾ 20	N/mm²
Select Grade of Steel	fy	=	415 ▾ 415	N/mm²

Load calculation :

Dead load of the slab	DL	=	3.250	kN/m²
Floor finish(Roof finish)	FF	=	1.5	kN/m²
Live load	LL	=	2	kN/m²
Total load	TL	=	6.750	kN/m²

Moment and Area of Steel calculations:

Span	Moment Coefficient		Mu kN.m	Mu/bd² N/mm²	Pt %	Ast reqd mm²	Min Ast mm²	Dia of bar mm	Spacing mm	Ast pro mm²	
shorter	$\alpha_{x\ neg}$	0.046	6.39	0.59	0.17%	176.39	124.8	12	200	565.49	safe
	$\alpha_{x\ pos}$	0.035	4.79	0.44	0.13%	131.06	124.8	12	200	565.49	safe
longer	$\alpha_{y\ neg}$	0.032	4.44	0.41	0.12%	121.05	124.8	12	200	565.49	safe
	$\alpha_{y\ pos}$	0.024	3.33	0.31	0.09%	90.22	124.8	12	200	565.49	safe

Check for Deflection

The effective depth provided	=	104.000	mm
From figure 3 of I.S 456:1978 modification factor is			
Modification factor	=	1.24	
Required depth under deflection consideration	=	98.63	mm
		HENCE SAFE	

○ Simply suppurted

◉ continuous

3.7.5 Design of Stair Case

DESIGN OF DOG-LEGGED STAIR

Name of work :- pkn

1	Stair hall measure		2.50	x		5.00	m
2	Available verical space between floor		3.00	m		3000	mm
3	Horizontal Span of stair case		1.20	mtr		1200	mm
4	Risers		0.15	mtr		150	mm
5	Treads		0.25	mtr		250	mm
6	Conrete	M -	20	wt. of concrete		25000	N/m^3
		σ_{cbc}	7	N/mm^2	m	13.33	
7	Steel	fy	415		σ_{st}	230	N/mm^2
8	Nominal cover		25	Effective cover		30	mm
	Reinforcement						
	Main Bottom slab		10	mm Φ bars		100	mm c/c
	Anchor bars (Bottom)		8	mm Φ bars		2	Nos.
	Strirrups		8	mm Φ bars		270	mm c/c

DESIGN OF DOG-LEGGED STAIR

Name of work :- pkn						
Stair hall measure		2.50 x 5.00				
Available verical space between floor		3.00	m			
Horizontal Span of stair case		1.20	m		1200	mm
Risers		0.15	m		150	mm
Treads		0.25	m		250	mm
Concrete	M-	20		wt. of concrete	25000	N/mm^2
	σ_{cbc}	7		m	13.33	
Steel	fy-	415	N/mm^2	σ_{st}	230	N/mm^2
Nominal cover		25	mm	Effective cover	30	mm

1 ***Genral arrngment:-***

Fig. shows plan of stair hall.

Height of 1^{st} flight.	=	3.00	/	2	=	1.50	m	minimum 1.8 m which is heigher
No. of risers required	=	1.80	/	0.15	=	12		No. in 1^{st} flight.
No. of treads required	=	12	-	1	=	11		No. in 1st flight.
Spce occupied by treads	=	11	x	0.25	=	2.75	m	
Keep width of landing equal to					=	1.20	m	
∴ Space left for passage					=	1.05	m	
Height of 1^{st} flight.	=	1.20	m					
No. of risers required	=	1.20	/	0.15	=	8		No. in 2^{nd} flight.
No. of treads required	=	8	-	1	=	7		No. in 2nd flight.
Spce occupied by treads	=	7	x	0.25	=	1.75	m	
Keep width of top landing					=		m	

2 ***Design Constants:-*** For HYSD Bars Cocrete M = 20

σst = 230 N/mm^2 wt. of concrete = 25000 N/mm^2

σcbc = 7 N/mm^3 m = 13.33

$$k = \frac{m*c}{m*c+\sigma st} = \frac{13.33 \times 7}{13.33 \times 7 + 230} = \mathbf{0.289}$$

$$j = 1 - k/3 = 1 - 0.289 / 3 = \mathbf{0.904}$$

$$R = 1/2 xc \times j \times k = 0.5 \times 7 \times 0.904 \times 0.289 = \mathbf{0.913}$$

3 ***Loading Each Flight :-***

The landing slab is assume to span in the same direction as stair, and is considered as acting together to form a single slab. Let the bearing of landing slab in wall be = 160 mm

The effective span = 2.75 + 1.20 +(0.16 / 2)= 4.03 m say = 4.00 m

Let the thickness of waist slab '= 5.00 x 40 = 200 mm

$\therefore$ Weight of slab w' on slope = 0.2 x 1 x 1 x 25000 = 5000 N/m^2

Dead weight of horizontal area $w_1 = w' \times \frac{\sqrt{R^2+T^2}}{T} = 5000 \times \frac{\sqrt{150^2 + 250^2}}{250} = 5831$ N/m^2

Dead weight of step is $w^2 = \frac{1}{2} \times \frac{150}{1000} \times 25000 = 1875$ N/m

$\therefore$ Total Dead weight per meter run = 7706 N

Weight of fiishing etc. = 100 N

Live load = 2500 N

Total weight = 10306 N

Note. The load w on the landing portion will be 10306 - 1875 = 8431 will not come on it. However, a uniform value of w has been adopted here.

4 ***Design of waist slab :-***

$$\text{B.M.} = \frac{wl2}{8} = \frac{10306 \times 4.00^2}{8} = 20612 \text{ N-m}$$

$$\text{Effective depth required} = \frac{\sqrt{BM}}{Rxb} = \sqrt{\frac{20612000}{0.913 \times 1000}} = 150 \text{ mm}$$

But available = 150 + 2x cover = 25 = 175 mm say = **180** mm

5 ***Reinforcement:-***

$$\text{Ast} = \frac{\text{BM} \times 100}{\sigma st \times j \times D} = \frac{20612000}{230 \times 0.904 \times 150} = 659.91 \text{ mm}^2$$

using 10 mm Φ bars $A = \frac{3.14 \times dia^2}{4 \times 100} = \frac{3.14 \times 10 \times 10}{4 \times 100} =$ **79** mm^2

$$\text{Nomber of Bars} = \frac{660 \times 1.20}{78.50} = 11 \text{ No}$$

Spacing = 1200 / 11 = 109 mm c/c

Hence used **10** mm Φ bars = **100** mm c/c

$$\text{Distribution steel} = \frac{0.12 \times 150 \times 1000}{100} = 180 \text{ mm}^2$$

using 8 mm Φ bars $A = \frac{3.14 \times dia^2}{4 \times 100} = \frac{3.14 \times 8 \times 8}{4 \times 100} =$ **50** mm^2

$$\text{Spacing} = \frac{50 \times 1000}{180.31} = 279 \text{ mm c/c}$$

Hence used **8** mm Φ bars = **270** mm c/c

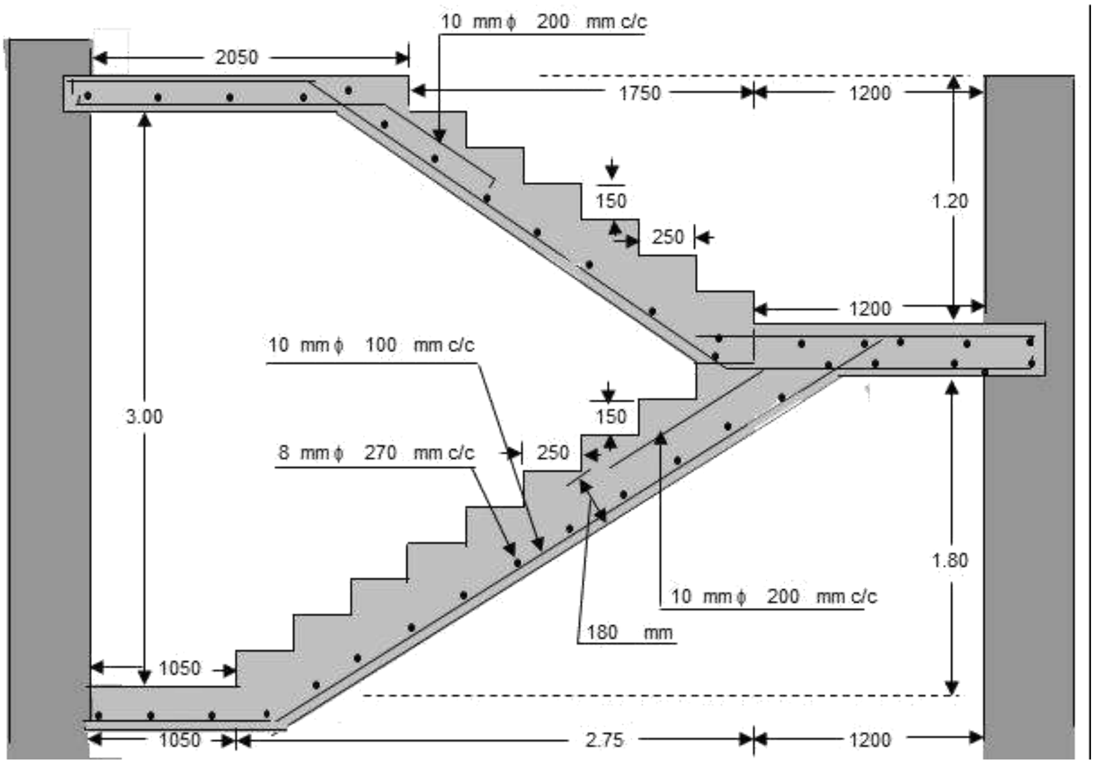


Figure 15 Detailing of Stair case

CHAPTER 4
LITERATURE REVIEW

4.1 Introduction

Use of base isolation devices in the foundation of critical structures as a mean of a seismic design has attracted considerable attention in the recent years. A variety of designs for base isolation ranging from rubber bearings, roller bearings and frictional types have been developed. At present there exists several leading base isolation systems that have been implemented in the construction of medium to large size structures. All of these systems have certain features in common, the most important of which are the horizontal flexibility and energy dissipative capacity. Recent years have seen a number of catastrophic structural failures due to severe, impulsive, seismic events. Some researchers e.g., Hall et al. 1995; Heaton et al. 1995. Have raised concerns as to the efficacy of seismic isolation during such events. The present work investigates the performance of a smart base isolation system and shows that it can reduce base drifts without the accompanying acceleration increases seen with passive strategies. Active and semi active strategies may be able to provide the reduced base drifts without the increase in superstructure motion seen for passive devices.

Elastomeric bearings are one type of isolator consisting of alternating layers of rubber bonded to intermediate steel shim plates. Lead-rubber (LR) bearings are another type of seismic isolator similar in construction to elastomeric bearings but with an added lead plug typically inserted in the center of the bearing. The horizontal flexibility required to achieve the period shift translates into large lateral displacements across the isolation interface during earthquake ground shaking that must be accommodated by the individual bearing. Therefore, during earthquake ground shaking, some of the individual bearings, specifically those located around the perimeter and under braced frames, will be subjected to simultaneous large lateral displacements and axial compressive loads, caused by gravity plus overturning loads. An important consideration for the design of seismic isolation systems composed of elastomeric or LR bearings are that the individual bearings remain stable under this loading condition. Most elastomeric bearings consist of multiple layers of elastomer bonded to intermediate steel shim plates. After repeated cycles of mechanical loading, fatigue cracks will initiate typically at the edge of the laminate and propagate toward the interior of the bearing, potentially leading to significant changes in the stiffness properties of the bearings.

4.2 Previous Study on Base Isolation System

Lin Su et al. (1989) studied "*Comparative Study of Base Isolation Systems*". A comparative study of effectiveness of various base isolators is carried out. These include the laminated rubber bearing with and without lead plug and several frictional base isolation systems. Combining the desirable features of various systems, a new design for a friction base isolator is also developed and its performance is studied. It is shown that, under design conditions, all base isolators can significantly reduce the acceleration transmitted to the superstructure. It is shown that in general the base isolation systems protect the structure from the effects of high amplitude and high frequency oscillations that fall in the same range as the natural frequencies of the structure. For earthquakes with considerable energy at low frequencies the LRB and the NZ systems are not suitable. The presented results indicate that for such earthquakes, undesirable amplification of ground excitation may occur.

Ramallo et al. (2002) studied "*Smart Base Isolation Systems*". A smart base isolation strategy is proposed and shown to effectively protect structures against extreme earthquakes without sacrificing performance during the more frequent, moderate seismic events. The proposed smart base isolation system is composed of conventional low-damping elastomeric bearings and ‘‘smart’’ controllable semi active dampers, such as magneto rheological fluid dampers. Smart isolation is shown to achieve notable decreases in base drifts over comparable passive systems with no accompanying increase in base shears or in accelerations imparted to the superstructure. In contrast to passive lead-rubber bearing systems, the adaptable nature of the smart damper isolation system provides good protection to both the structure and its contents over a wide range of ground motions and magnitudes. A smart base isolation system, comprised of low-damping elastomeric bearings, and ‘‘smart’’ controllable semi active dampers, was shown to have superior performance compared to several passive base isolation designs using lead-rubber bearings.

Satish et al. (1993) studied "*Torsion In Base Isolated Structures with Elastomeric Isolation Systems*". Torsion in base-isolated structures with inelastic elastomeric isolation systems due to bidirectional lateral ground motion is studied. In a companion paper by the writers, torsional coupling in sliding base-isolated structures was investigated. In this paper, which is the second part of the sequence, torsional coupling in elastomeric base-isolated structures is investigated. Response to different ground motions is also studied. The results are used to explain: (1) The behavior of actual buildings; and (2) some inconsistencies in the conclusions of previous studies. The main source of torsional motions in etastomeric isolated structures is the isolation system eccentricity e_b/L. increasing isolation eccentricity e_b/L leads to increased torque amplification T_{amp}. It can be stated that, although the magnitude of shear and torque generated in an elastomeric isolated structure is less than that of the fixed-base structure, the torsional amplifications may not be negligible, and may lead to torques that cannot be ignored.

Vasant and Jangid (2008) studied "*Base Isolation for Seismic Retrofitting of Structures*". Analytical seismic responses of structures retrofitted using base isolation devices are investigated and the retrofit schemes are illustrated. The retrofitting of various important structures using seismic isolation technique by incorporation of the layers of isolators at suitable locations is studied. It is observed that the seismic response of the retrofitted structures reduces significantly in comparison with the conventional structures depicting effectiveness of the retrofitting done through the base isolation technique. This study distinctively presented modalities involved in the construction technique of seismic retrofitting using the base isolation strategy.

Weisman and Warn (2012) studied "*Stability of Elastomeric and Lead-Rubber Seismic Isolation Bearings*". Elastomeric and lead-rubber bearings are two commonly used types of seismic isolation devices. During seismic events, some of the bearings in an isolation system will be subjected to large axial compressive loads, caused by gravity plus overturning forces, accompanied by simultaneous large lateral displacements. However, the critical load capacity of elastomeric bearings has been shown to reduce with increasing lateral displacement.

Husam et al. (2007) studied "*Evaluation of Laminated Circular Elastomeric Bearings*". This paper evaluates the behavior and performance of laminated circular elastomeric bearings and compares them to those of square and rectangular bearings.

The study included an experimental evaluation and a nationwide survey of state Department of Transportation's on the use and performance of circular bearings and bearings in general in their states. The experimental investigation studied the bearings' behavior in compression, compression and rotation, and compression and shear. Results from this limited study showed that the three bearings have similar stress-strain behavior in compression and they are in agreement with the AASHTO LRFD guide stress–strain curves. In compression and rotation, the AASHTO LRFD substructure moments are slightly less than the measured values for circular bearings and rectangular bearings rotated about their strong axis for a compressive stress of 10.3 MPa and slightly higher than those of rectangular bearings rotated about their weak axis.

Deng and Warn (2016) studied "*Modeling the Compression Stiffness Degradation in Circular Elastomeric Bearings Due To Fatigue*". Laminated rubber, or elastomeric, bearings fatigue when subjected to repeated cycles of loading. Fatigue in these elements is characterized by the formation of cracks typically originating at the interface of the steel-rubber laminate at the outermost edge of the laminate then propagating at an inclination toward the center of the bearing under subsequent cycling. The presence of fatigue cracks alters the bulging surface of the rubber layers, thereby degrading the stiffness properties of the bearings.

4.3 Lessons Learned from Study

Besides the results that were intended to be investigated in this study, there were also many beneficial lessons that were learned, which are summarized below:

- Lowering the coefficients of friction of the TFP bearings is the most effective way to improve seismic performance (i.e. reduce the superstructure's response values, including floor accelerations and interstory drifts) when implementing base isolation in a tall, flexible building.
- Using TFP bearings with larger radii of curvature (R) leads to a more flexible (smaller lateral stiffness) isolation system and improves seismic performance, although larger bearing sizes are also more expensive.
- Most of the seismic damage occurred in the interior partitions and accessories (including workstations and desktop electronics). They were among the most fragile components in the building and had the largest impact on cost.

- The bearing displacements of the base-isolated structure were sensitive to damping during the nonlinear time history analyses. Therefore, it is important to use good judgment when assigning damping values in models of base-isolated structures. Since base-isolated structures allow the superstructure to remain essentially elastic, it is wise to use a smaller modal damping ratio for an isolated structure than the ratio used for a fixed-base structure (i.e., use 1% in lieu of 3%).
- When modeling base isolation systems, remember to assign rotationally rigid restraints to the isolation platform, which lies directly above the isolation bearings. The isolation platform must be designed and modeled to resist the large bending moments induced by the bearings during seismic events. Since isolation platforms are typically assigned as rigid diaphragms, and rigid diaphragms are often modeled with only translational restraints, it is easy to forget to include the rotationally rigid restraints for the isolation platform.
- The isolation platform should be designed to prevent uplift of the isolation bearings during a seismic event, especially since uplift of the bearings is more likely to occur for tall and flexible buildings. The use of a concrete mat was effective for preventing uplift of the 12-story steel office building in this study. Uplift of isolation bearings is more prone to occur at bearings located along braced frames than bearings located along moment frames, due to the larger aspect ratios (height-to-width) of braced frames.
- In order to reduce the prevalence of torsion in the structure's base-isolated modes, it was effective to design the outer base isolation bearings that run along the perimeter of the structure to have a lateral stiffness that was roughly a third greater in value than the lateral stiffness of the inner bearings.

CHAPTER 5
DESIGN OF ELASTOMERICBEARING

5.1 Introduction

Elastomeric bearings are widely used in civil, rail, and aerospace applications to accommodate movement and to suppress vibrations. These bearings are manufactured in a variety of different shapes and configurations for various loading conditions, e.g. axial, radial, multi axial. Most elastomeric bearings consist of multiple layers of elastomer bonded to intermediate steel shim plates. After repeated cycles of mechanical loading, fatigue cracks will initiate typically at the edge of the laminate and propagate toward the interior of the bearing, potentially leading to significant changes in the stiffness properties of the bearings.

Following are the types of Elastomeric bearing:

5.2 Elastomeric Based Systems

a) Low Damping Natural and Synthetic Rubber Bearings

b) Lead Plug Bearings

c) High Damping Natural Rubber System

5.3 Design of Elastomeric Bearings

Elastomeric and lead-rubber bearings are two commonly used types of seismic isolation devices. During seismic events, some of the bearings in an isolation system will be subjected to large axial compressive loads, caused by gravity plus overturning forces, accompanied by simultaneous large lateral displacements. However, the critical load capacity of elastomeric bearings has been shown to reduce with increasing lateral displacement. These bearings are manufactured in a variety of different shapes and configurations for various loading conditions, e.g. axial, radial, multi axial. Most elastomeric bearings consist of multiple layers of elastomer bonded to intermediate steel shim plates. After repeated cycles of mechanical loading, fatigue cracks will initiate typically at the edge of the laminate and propagate toward the interior of the bearing, potentially leading to significant changes in the stiffness properties of the bearings. However, the critical load capacity of elastomeric bearings has been shown to reduce with increasing lateral displacement. These bearings are manufactured in a variety of different shapes and configurations for various loading conditions, e.g. axial, radial, multi axial. Most elastomeric bearings consist of multiple layers of elastomer bonded to intermediate steel shim plates.

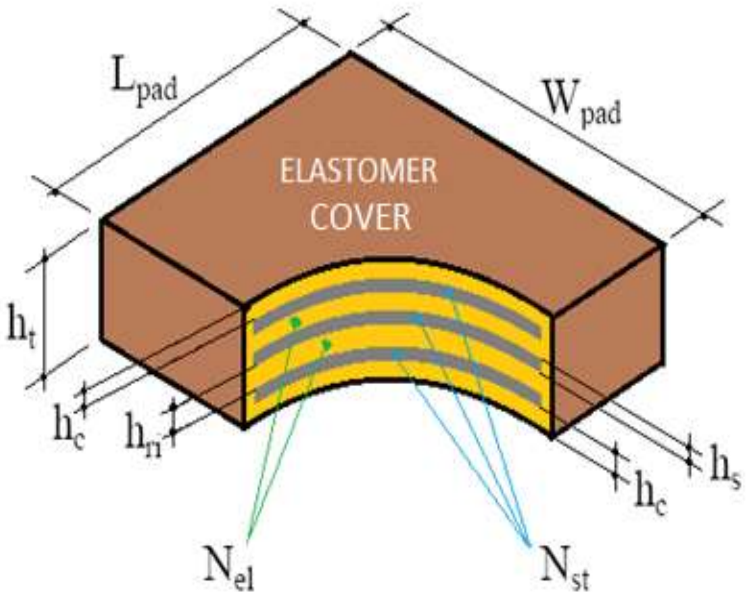


Figure 16 elastomeric bearing

System and material input data :

Expandable span length	Ls =	17300	mm
Constant amplitude fatigue threshold for Category A	□Ft =	165	MPa
Elastomer hardness:	Hshore =	50	
Shear modulus of elastomer { (0,68 - 0,93) SELECT }	G =	1	MPa
Steel reinforcement yield strength:	fy =	240	MPa
Pad length (bridge longitudinal direction):	Lpad =	250	mm
Pad width (bridge transverse direction):	Wpad =	250	mm
Elastomer cover thickness:	hc =	2.5	mm
Elastomer internal layer thickness:	hri =	8	mm
Number of steel reinforcement layers:	Nst =	5	
Steel reinforcement thickness:	hs =	3	mm

System and material output data :

Elastomer creep deflection at 25 years divided by the instantaneous deflection:	Cd =	0.25	
Number of elastomer internal layers	Nel =	4	
total elastomer thickness	hrt =	37	mm
Total steel plate heigth	hst =	15	mm
Total bearing heigth	ht =	52	mm
Bearing surface area	Area =	62500	mm^2

Check Nst :

Nst =	5	Nst> 2 ise;	
hc =	2.5		
0.70 hri =	5.6	hc ≤ 0.70 hri	OK.

Compute Shape Factor :

Sint =	7.8125	Si = L.W / (2.hri.(L+W))
Scov =	25	Si = L.W / (2.hc.(L+W))
S =	7.8125	S=min(Sint,Scov)

Check Compressive Stress:

DLs =	106	kN	DL reaction/girder
LLs =	111	kN	LL reaction /girder
σs =	3.472	MPa	σs = (DLs+LLs) /Area
σL =	1.776	MPa	σL = LLs / Area

Shear deformation? -YES- :

1.66 G.S =	12.96875	Mpa	σs ≤ 1,66 G.S	OK.
0.66 G.S =	5.15625	Mpa	σs ≤ 11	NO
			σL ≤ 0,66 G.S	OK.

Shear deformation? -NO- :

G.S =	7.8125	Mpa	σs ≤ 2 G.S	OK.
2G.S =	15.625	Mpa	σs ≤ 12	OK.
			σL ≤ G.S	OK.

Check Compressive Deflection :

εi =	0.030928	
δLi =	0.247427	mm
δLt =	1.144351	mm
δcr =	0.007732	mm
Σδ =	1.152083	mm
0.07 hri =	0.56	mm

durometer	50	60	70
C	0.01676	0.018156	0.013966
x	0.29805	0.27257	0.311109
εi = C□^x	0.030928	0.016969	0.012929

δLi ≤ 0.07 hri OK.

Check Shear Deformation:

α =	1.17E-05	/□C	
tset =	20	□C	
γTU =	1.2		
Δco =	4.0482	mm	$\Delta co = \alpha$. tset .Ls
Δs =	4.85784	mm	$\Delta s = \Delta co .\gamma TU$
$2.\Delta s$ =	9.71568	mm	$2.\ \Delta co \leq$ hrt — OK.

Check Rotation or Combined Compression and Rotation:

Construction Tolerance

Ls =	17300	mm	
θsx =	0.003	rad.	
θsz =	0.003	rad.	
σs =	3.472	MPa	
Nel =	4		
G.S =	7.8125	Mpa	
n =	5		
			Lch = 0.5 GS (Lpad/hri)^2 (θsx/n)
Lch =	2.165166	Mpa	Lch $\leq \sigma s$ — OK.
			Wch = 0.5 GS (Wpad/hri)^2 (θsz/n)
Wch =	2.165166	Mpa	Wch $\leq \sigma s$ — OK.

Check Stability:

ht =	52	mm	minLW=min(Lpad/3 , Wpad/3)
minLW =	83.33333	mm	ht $\leq$ minLW — OK.

Check Reinforcement:

hmax =	8	mm	
hsi =	0.3472	mm	hsi =3 hriσs / fy
			hsi $\leq$ hmax — OK.
hsii =	0.172218	mm	hsii =2 hriσL / Aft
			hsii $\leq$ hmax — OK.

5.4 Results

The above assumed bearing is safe.

Pad length (bridge longitudinal direction):	Lpad	=	250	mm
Pad width (bridge transverse direction):	Wpad	=	250	mm
Elastomer cover thickness:	hc	=	2.5	mm
Elastomer internal layer thickness:	hri	=	8	mm
Number of steel reinforcement layers:	Nst	=	5	
Steel reinforcement thickness:	hs	=	3	mm

5.5 Calculation of Spring Stiffness

Spread sheet for calculation of spring stiffness has been given in appendix.

CHAPTER 6
COMPARISON OF FIXED BASE AND ISOLATED BASE

Introduction

Base isolation technology is used primarily in critical facilities such as hospitals, museums, and emergency response centers, where the benefits of protecting the structure and its property from seismic damage far exceed the cost of implementing the system. The purpose of this thesis is to offer a relative understanding of the seismic performance enhancements that a typical 5-story concrete commercial building can achieve through the implementation of base isolation technology. To reach this understanding, the structures of a fixed-base and a base-isolated of similar size and layout are designed, their seismic performance is compared. To a greater extent, this study demonstrates the feasibility and cost effectiveness of implementing base isolation on tall, flexible, and non-critical structures. As a result of this thesis, building owners and construction industry professionals can recognize the benefits of implementing base isolation on a wider range of projects, thereby creating the potential for a significant increase in the technology’s use.

6.1 Performance Assessment

In the performance assessment phase, the member forces and interstory drifts Obtained from the response spectrum analyses in the analysis phase were used to assess the seismic performance of the structures. Both fixed base and isolated base are analyzed. Various charts are plotted for the member forces and drifts.

6.2 Assessment Procedure

An overview of the performance assessment phase procedure is shown below:

1. Enter building data i.e. its specification, member properties, loading conditions and support conditions (fixed and isolated).
2. Analyze the building on the basis of story drift and member forces for fixed support conditions.

3. Design of elastomeric bearing and find the stiffness of spring using codal provisions.

4. Analyze the building for spring support

5. Compare the results of the two conditions.

6.3 Analysis

This section discusses the results obtained from the analysis phase and performance assessment phase, including member forces, interstory drifts, structural seismic performance of each level. Implementing base isolation reduced the response of the fixed-base structure by roughly half for nearly all response parameters, directions, and ground motion records. The fixed-base response is shown in blue and the base-isolated response is shown in red in each figure. The implementation of base isolation was slightly more effective for reducing floor accelerations than for reducing interstory drifts. The interstory drifts in the direction of the braced frames were the most consistent parameters for all ground motions and buildings, due to the greater lateral stiffness of the braces compared to the moment frames.

Out of all the ground motions, a maximum acceleration was reached at the roof of the fixed-base structure, while the base-isolated structure had a maximum value less than fixed base structure.

Out of all the ground motions, a maximum interstory drift of 2.5cm was reached at the lower half of the fixed-base structure, while the base-isolated structure had a maximum value of only 1.1cm.

Out of all the ground motions, member forces were reduced nearly 60% in the base isolated structure as in fixed base structure.

The performance of these systems is also compared using a simplified response index. A more realistic performance assessment is carried out by using benchmark building alternatives designed for this purposes as explained in chapter 3. The performance model including initial cost analysis and fragility specifications of components are discussed. Reponses analysis and assumptions made for performance assessment is also discussed in detail. Repair costs and time, human injuries and fatalities, and collapse probabilities are estimated for all benchmark buildings and conclusions are made based on the net present value of all costs involved.

As a result, base isolation is currently utilized primarily for the continued operation of essential facilities or other types of buildings where a highly-effective and less intrusive property of the base-isolation is desirable such as in historical buildings.

6.4 Results

Comparison of Fixed base and Base isolated structure on the basis of member forces, interstory drifts and natural time periods are illustrated as below.

Various Graphs showing comparison between the two i.e. fixed base and isolated base.

6.4.1 Comparison of Forces in Columns

1. **Axial Forces:** For column number 104 below is the graph showing reduction in member forces by applying base isolation technique.

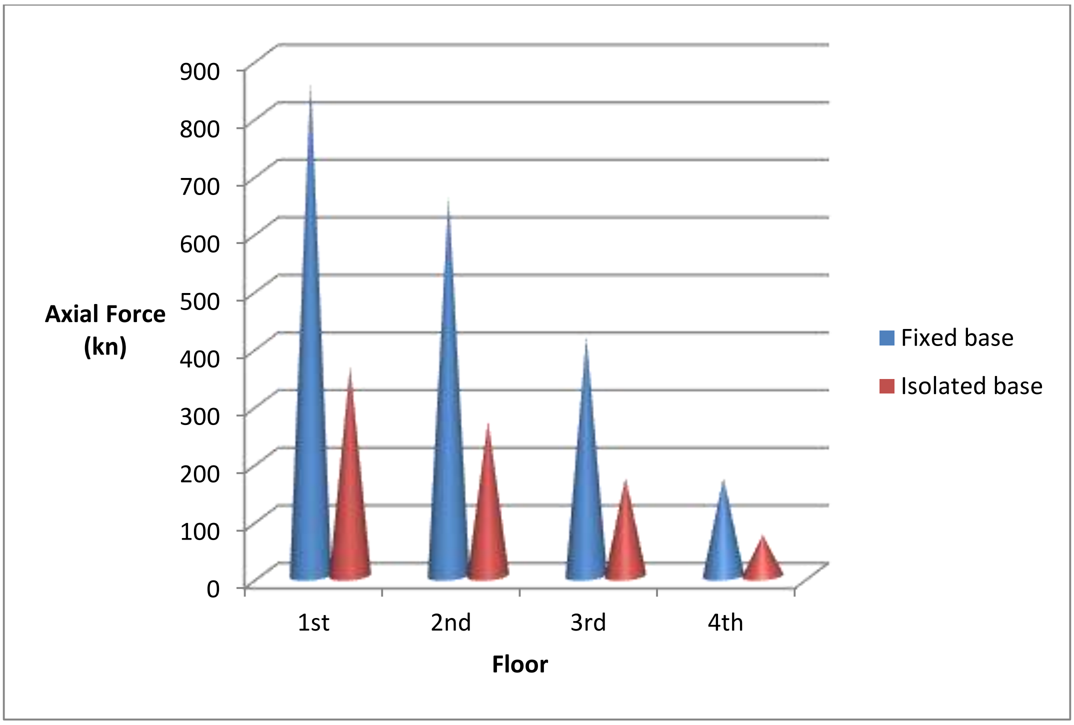


Figure 17

2. Average Axial Force

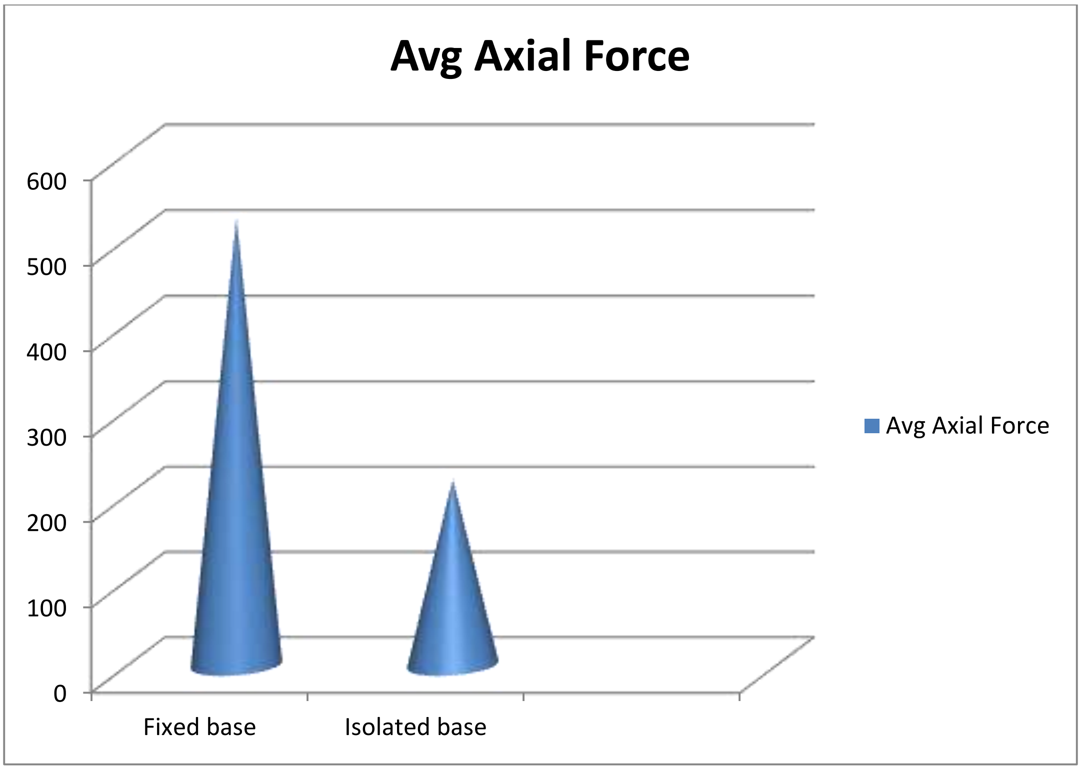


Figure 18

3. **Percentage Reduction:** Table 3 Shows percentage reduction in Axial forces

Table 3

	Average Axial force(kn)	% reduction
Fixed base	524.44	41.36
Isolated base	218.36	

4. **Moment in Y and Z direction:** The graph below shows the reduction in moment due to base isolation technique.

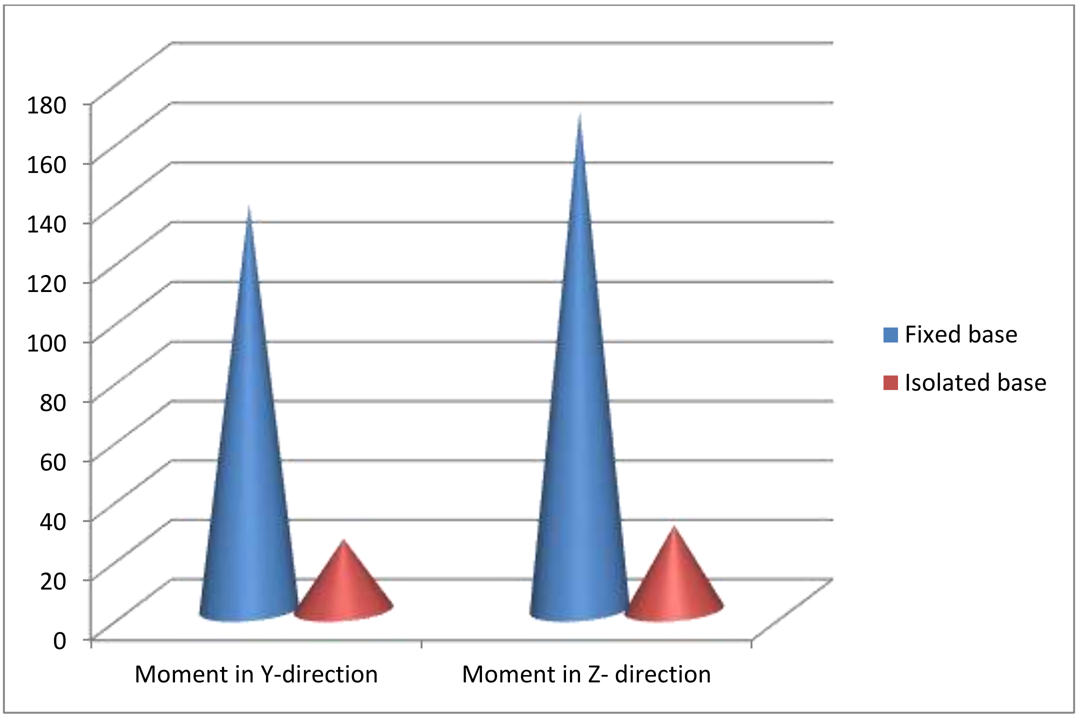


Figure 19

6.4.2 Comparison of Forces in beams:

1. **Shear Force:** For beam number 136 reduction in shear force.

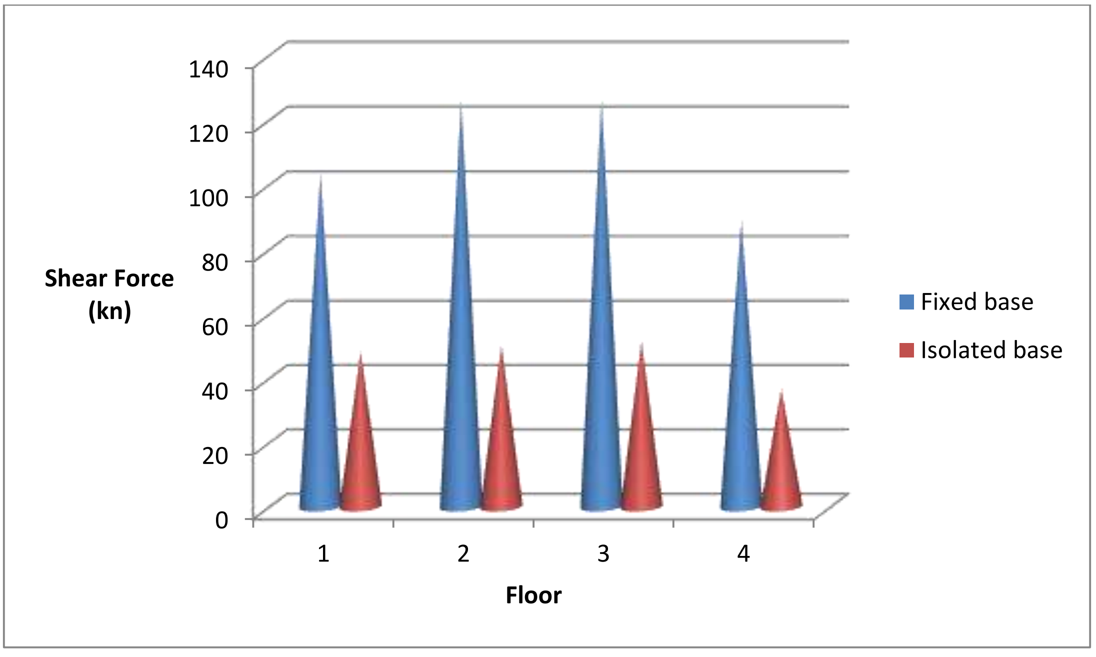


Figure 20

2. Average Shear force

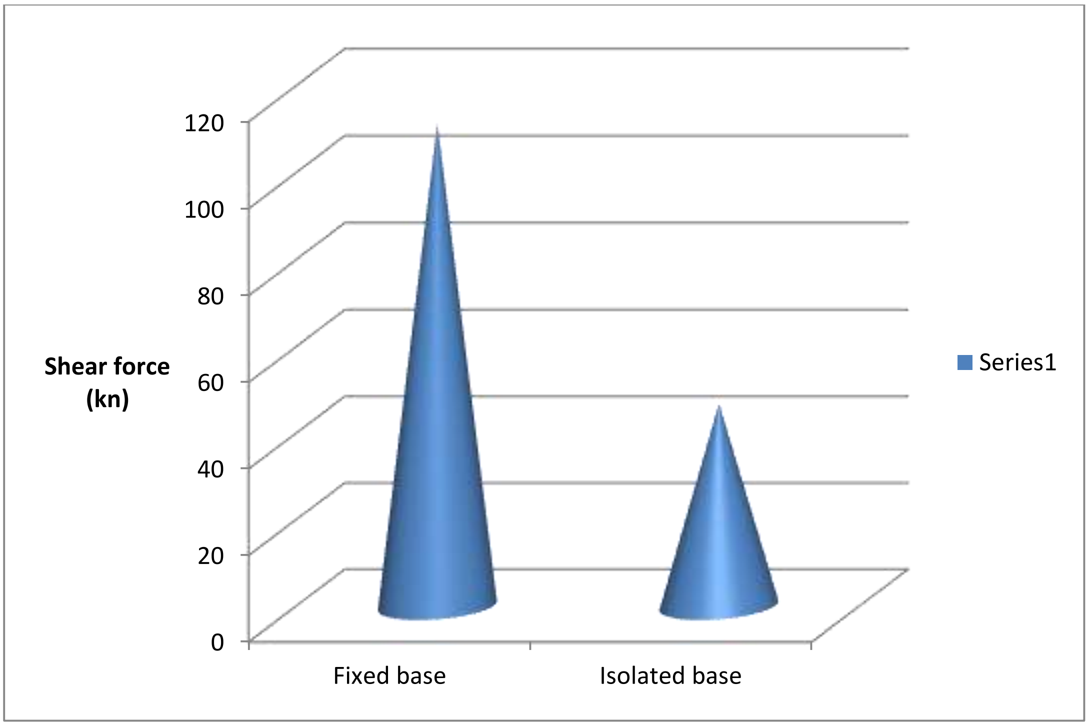


Figure 21

3. Percentage Reduction:

3. Percentage Reduction: Table 4 Shows percentage reduction in shear forces

Table 4

	Average Shear force	% Reduction
Fixed base	111.15	41.99
Isolated base	46.68	

4. Bending Moment

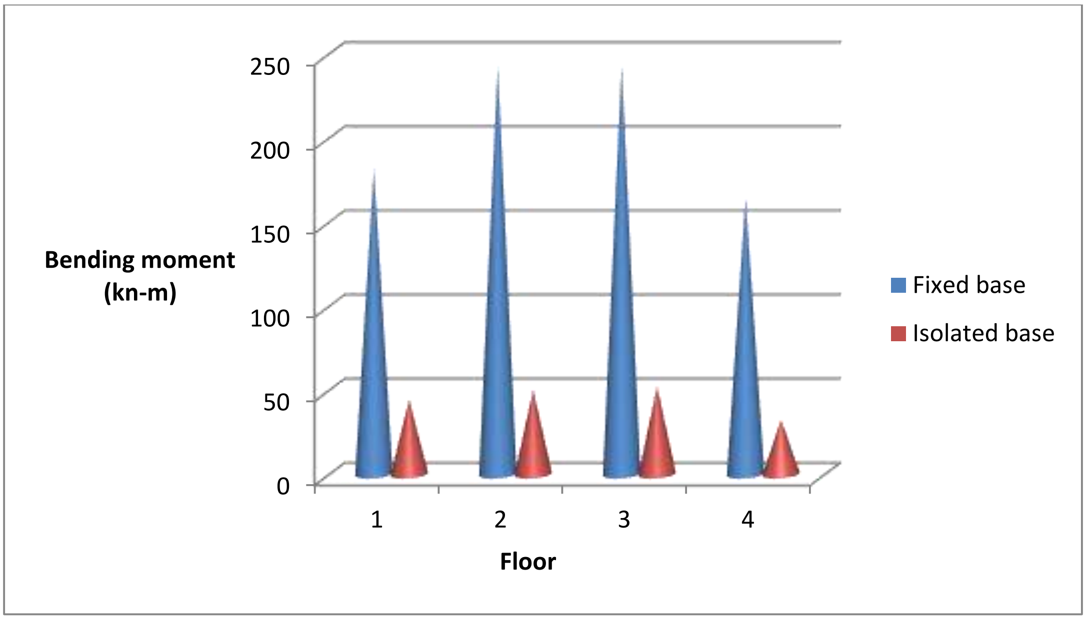


Figure 22

5. Average Bending Moment

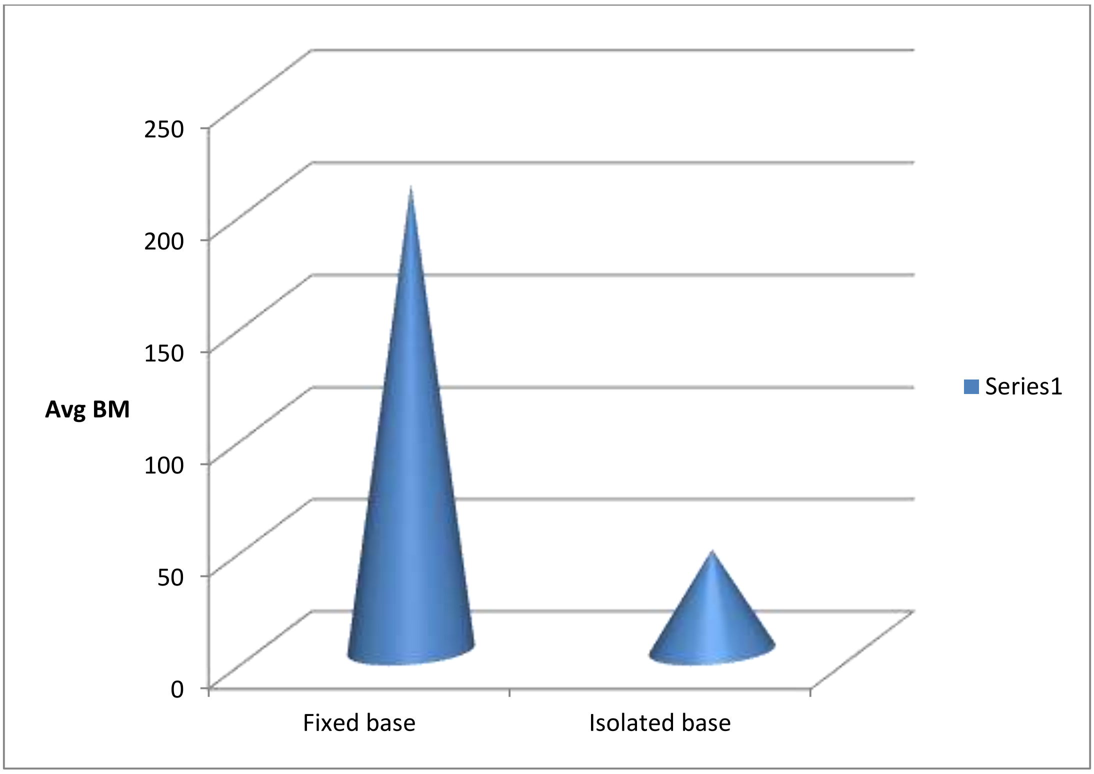


Figure 23

6. **Percentage Reduction:** Table 5 Shows percentage reduction in bending moement

Table 5

	Average bending moment	% reduction
Fixed base	208.02	21.57
Isolated base	44.87	

6.4.3 Comparison on the Basis of Drift

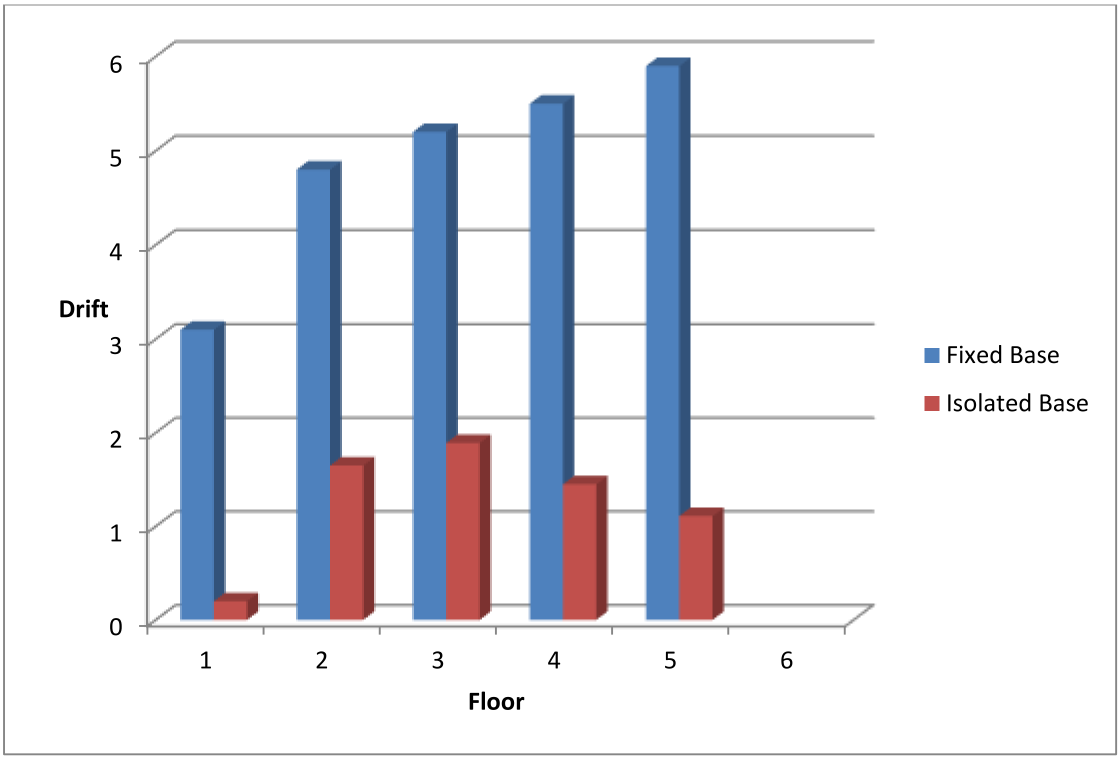


Figure 24

6.4.4 Natural Time Period

Fundamental natural period T is an inherent property of a building. Any alterations made to the building will change its T. Fundamental natural periods T of normal single storey to 20 storey buildings are usually in the range 0.05-2.00 sec.

As we can see there is decrease in time period of the structure by applying the technique of base isolation which is known as period shift effect and due to which acceleration of the structure decreases in the case to seismic motions.

Table 6

Fixed Base		
Mode	***Frequency (cycles/sec)***	***Period (sec)***
1	**0.661**	**1.512**
2	0.843	1.185
3	0.854	1.170
4	1.390	0.719
5	1.607	0.622
6	2.013	0.496

Table 7

Isolated Base		
Mode	***Frequency (cycles/sec)***	***Period (sec)***
1	**0.475**	**2.104**
2	0.534	1.873
3	0.572	1.749
4	1.198	0.834
5	1.496	0.668
6	1.775	0.563

The bearing displacements of the base-isolated structure were sensitive to damping during the nonlinear time history analyses. Therefore, it is important to use good judgment when assigning damping values in models of base-isolated structures. Since base-isolated structures allow the superstructure to remain essentially elastic, it is wise to use a smaller modal damping ratio for an isolated structure than the ratio used for a fixed-base structure. When modeling base isolation systems, remember to assign rotationally rigid restraints to the isolation platform, which lies directly above the isolation bearings. The isolation platform must be designed and modeled to resist the large bending moments induced by the bearings during seismic events. Since isolation platforms are typically assigned as rigid diaphragms, and rigid diaphragms are often modeled with only translational restraints, it is easy to forget to include the rotationally rigid restraints for the isolation platform.

CHAPTER 7
CONCLUSIONS

This section summarizes the conclusions that were reached as a result of this study.

7.1 Conclusions of the Study

The results obtained after implementing base isolation in the four-story concrete commercial building were clearly shown study:

i. Member Forces were considerably reduced for both beams and columns. There was about 40 to 45 % reduction in member forces for the beams and column considered.

ii. For example in beam number 102 shear forces for fixed base were 115.15kn and were reduced to 46.5kn by using base isolation technique which is about 41.99%.

iii. For example in column number 136 axial forces were 524.44kn for fixed base and were reduced to 218.36kn by using base isolation technique which is about 41.36%.

iv. Story Drifts were reduced considerably using base isolation technique. Average story drift for the structure earlier for base isolated structure was 3.6mm. After implementing base isolation technique it was reduced to about 1.8mm which is half of the fixed base.

v. Time Period of the structure was increased after implementing base isolation technique i.e. acceleration was reduced to about 35.6% for the seismic forces.

vi. Time Period for fixed base was 1.51sec and that for isolated base comes out to be 2.15sec for the structure.

For a service level event (SLE), the implementation of base isolation would likely minimize or negate seismic damage and related damage costs of the building, leaving the structure at a fully operational structural performance level. Conversely, if the fixed-base office building were subjected to SLE seismicity it could incur structural deformations to the Immediate Occupancy.

APPENDICES

A. DETERMINATION OF DESIGN CRITERIA

B. FIXED-BASE RESPONSE SPECTRUM ANALYSIS CALCULATIONS

C. BASE-ISOLATED RESPONSE SPECTRUM ANALYSIS CALCULATIONS

D. ROOF UNIT LOAD TAKE-OFF

E. FLOOR UNIT LOAD TAKE-OFF

F. CALCULATION OF MEMBER FORCES

G. CALCULATION OF STORY DRIFTS

H. COMPARISON OF BOTH THE STRUCTURES

I. RESULTS

8.1 Fixed Base Program

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 17-May-16

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 5 0 0; 3 10 0 0; 4 15 0 0; 5 20 0 0; 6 0 4.2 0; 7 5 4.2 0;

8 10 4.2 0; 9 15 4.2 0; 10 20 4.2 0; 11 0 7.4 0; 12 5 7.4 0; 13 10 7.4 0;

14 15 7.4 0; 15 20 7.4 0; 16 0 10.6 0; 17 5 10.6 0; 18 10 10.6 0; 19 15 10.6 0;

20 20 10.6 0; 21 0 13.8 0; 22 5 13.8 0; 23 10 13.8 0; 24 15 13.8 0;

25 20 13.8 0; 26 0 0 5; 27 5 0 5; 28 10 0 5; 29 15 0 5; 30 20 0 5; 31 0 4.2 5;

32 5 4.2 5; 33 10 4.2 5; 34 15 4.2 5; 35 20 4.2 5; 36 0 7.4 5; 37 5 7.4 5;

38 10 7.4 5; 39 15 7.4 5; 40 20 7.4 5; 41 0 10.6 5; 42 5 10.6 5; 43 10 10.6 5;

44 15 10.6 5; 45 20 10.6 5; 46 0 13.8 5; 47 5 13.8 5; 48 10 13.8 5;

49 15 13.8 5; 50 20 13.8 5; 51 0 0 10; 52 5 0 10; 53 10 0 10; 54 15 0 10;

55 20 0 10; 56 0 4.2 10; 57 5 4.2 10; 58 10 4.2 10; 59 15 4.2 10; 60 20 4.2 10;

61 0 7.4 10; 62 5 7.4 10; 63 10 7.4 10; 64 15 7.4 10; 65 20 7.4 10;

66 0 10.6 10; 67 5 10.6 10; 68 10 10.6 10; 69 15 10.6 10; 70 20 10.6 10;

71 0 13.8 10; 72 5 13.8 10; 73 10 13.8 10; 74 15 13.8 10; 75 20 13.8 10;

76 0 0 15; 77 5 0 15; 78 10 0 15; 79 15 0 15; 80 20 0 15; 81 0 4.2 15;

82 5 4.2 15; 83 10 4.2 15; 84 15 4.2 15; 85 20 4.2 15; 86 0 7.4 15;
87 5 7.4 15; 88 10 7.4 15; 89 15 7.4 15; 90 20 7.4 15; 91 0 10.6 15;
92 5 10.6 15; 93 10 10.6 15; 94 15 10.6 15; 95 20 10.6 15; 96 0 13.8 15;
97 5 13.8 15; 98 10 13.8 15; 99 15 13.8 15; 100 20 13.8 15;
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1 6 7; 2 7 8; 3 8 9; 4 9 10; 5 11 12; 6 12 13; 7 13 14; 8 14 15; 9 16 17;
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18 2 7; 19 3 8; 20 4 9; 21 5 10; 22 6 11; 23 7 12; 24 8 13; 25 9 14; 26 10 15;
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113 86 87; 114 87 88; 115 88 89; 116 89 90; 117 91 92; 118 92 93; 119 93 94;
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127 78 83; 128 79 84; 129 80 85; 130 81 86; 131 82 87; 132 83 88; 133 84 89;
134 85 90; 135 86 91; 136 87 92; 137 88 93; 138 89 94; 139 90 95; 140 91 96;
141 92 97; 142 93 98; 143 94 99; 144 95 100; 145 6 31; 146 7 32; 147 8 33;
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155 16 41; 156 17 42; 157 18 43; 158 19 44; 159 20 45; 160 21 46; 161 22 47;
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176 42 67; 177 43 68; 178 44 69; 179 45 70; 180 46 71; 181 47 72; 182 48 73;
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190 61 86; 191 62 87; 192 63 88; 193 64 89; 194 65 90; 195 66 91; 196 67 92;
197 68 93; 198 69 94; 199 70 95; 200 71 96; 201 72 97; 202 73 98; 203 74 99;
204 75 100;
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E 2.17185e+007
POISSON 0.17

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DENSITY 23.5616
ALPHA 1e-005
DAMP 0.05
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CONSTANTS
MATERIAL CONCRETE ALL
SUPPORTS
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SELFWEIGHT 1
FLOOR WEIGHT
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YRANGE 11.5 15 FLOAD 10
YRANGE 0 11 FLOAD 2
LOAD 1 LOADTYPE None TITLE LOAD CASE 1
FLOOR LOAD
YRANGE 0 11 FLOAD -12 GY
YRANGE 0 11 FLOAD -4 GY
YRANGE 11.5 15 FLOAD -2 GY
YRANGE 11.5 15 FLOAD -10 GY
LOAD 2 LOADTYPE None TITLE LOAD CASE 2 RESPNSE
SELFWEIGHT X 1 LIST 1 TO 204
SELFWEIGHT Y 1 LIST 1 TO 204
SELFWEIGHT Z 1 LIST 1 TO 204
FLOOR LOAD
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YRANGE 0 11 FLOAD 12 GY
YRANGE 0 11 FLOAD 12 GZ
YRANGE 11.5 15 FLOAD 10 GX
YRANGE 11.5 15 FLOAD 10 GY
YRANGE 11.5 15 FLOAD 10 GZ
YRANGE 0 11 FLOAD 2 GX
YRANGE 0 11 FLOAD 2 GY
YRANGE 0 11 FLOAD 2 GZ
```

SPECTRUM SRSS 1893 X 0.036 ACC DAMP 0.05

SOIL TYPE 2

PERFORM ANALYSIS PRINT MODE SHAPES

PRINT STORY DRIFT

PRINT MEMBER FORCES LIST 1 TO 204

FINISH

8.2 Isolated Base Program

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 17-May-16

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 5 0 0; 3 10 0 0; 4 15 0 0; 5 20 0 0; 6 0 4.2 0; 7 5 4.2 0;

8 10 4.2 0; 9 15 4.2 0; 10 20 4.2 0; 11 0 7.4 0; 12 5 7.4 0; 13 10 7.4 0;

14 15 7.4 0; 15 20 7.4 0; 16 0 10.6 0; 17 5 10.6 0; 18 10 10.6 0; 19 15 10.6 0;

20 20 10.6 0; 21 0 13.8 0; 22 5 13.8 0; 23 10 13.8 0; 24 15 13.8 0;

25 20 13.8 0; 26 0 0 5; 27 5 0 5; 28 10 0 5; 29 15 0 5; 30 20 0 5; 31 0 4.2 5;

32 5 4.2 5; 33 10 4.2 5; 34 15 4.2 5; 35 20 4.2 5; 36 0 7.4 5; 37 5 7.4 5;

38 10 7.4 5; 39 15 7.4 5; 40 20 7.4 5; 41 0 10.6 5; 42 5 10.6 5; 43 10 10.6 5;

44 15 10.6 5; 45 20 10.6 5; 46 0 13.8 5; 47 5 13.8 5; 48 10 13.8 5;

49 15 13.8 5; 50 20 13.8 5; 51 0 0 10; 52 5 0 10; 53 10 0 10; 54 15 0 10;

55 20 0 10; 56 0 4.2 10; 57 5 4.2 10; 58 10 4.2 10; 59 15 4.2 10; 60 20 4.2 10;

61 0 7.4 10; 62 5 7.4 10; 63 10 7.4 10; 64 15 7.4 10; 65 20 7.4 10;

66 0 10.6 10; 67 5 10.6 10; 68 10 10.6 10; 69 15 10.6 10; 70 20 10.6 10;

71 0 13.8 10; 72 5 13.8 10; 73 10 13.8 10; 74 15 13.8 10; 75 20 13.8 10;

76 0 0 15; 77 5 0 15; 78 10 0 15; 79 15 0 15; 80 20 0 15; 81 0 4.2 15;

82 5 4.2 15; 83 10 4.2 15; 84 15 4.2 15; 85 20 4.2 15; 86 0 7.4 15;

87 5 7.4 15; 88 10 7.4 15; 89 15 7.4 15; 90 20 7.4 15; 91 0 10.6 15;

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MEMBER INCIDENCES

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DEFINE MATERIAL START

ISOTROPIC CONCRETE

E 2.17185e+007

POISSON 0.17

DENSITY 23.5616

ALPHA 1e-005

DAMP 0.05

END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN

1 TO 204 PRIS YD 0.45 ZD 0.3

CONSTANTS

MATERIAL CONCRETE ALL

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SUPPORTS
1 TO 5 26 TO 30 51 TO 55 76 TO 79 -
80 FIXED BUT KFX 200 KFY 1400 KFZ 200 KMX 200 KMY 200 KMZ 200
DEFINE 1893 LOAD
ZONE 0.36 RF 5 I 1 SS 1 ST 1 DM 0.05
SELFWEIGHT 1
FLOOR WEIGHT
YRANGE 0 11 FLOAD 12
YRANGE 11.5 15 FLOAD 10
YRANGE 0 11 FLOAD 2
LOAD 1 LOADTYPE None TITLE LOAD CASE 1
FLOOR LOAD
YRANGE 0 11 FLOAD -12 GY
YRANGE 0 11 FLOAD -4 GY
YRANGE 11.5 15 FLOAD -2 GY
YRANGE 11.5 15 FLOAD -10 GY
LOAD 2 LOADTYPE None TITLE LOAD CASE 2 RESPNSE
SELFWEIGHT X 1 LIST 1 TO 204
SELFWEIGHT Y 1 LIST 1 TO 204
SELFWEIGHT Z 1 LIST 1 TO 204
FLOOR LOAD
YRANGE 0 11 FLOAD 12 GX
YRANGE 0 11 FLOAD 12 GY
YRANGE 0 11 FLOAD 12 GZ
YRANGE 11.5 15 FLOAD 10 GX
YRANGE 11.5 15 FLOAD 10 GY
YRANGE 11.5 15 FLOAD 10 GZ
YRANGE 0 11 FLOAD 2 GX
YRANGE 0 11 FLOAD 2 GY
YRANGE 0 11 FLOAD 2 GZ
SPECTRUM SRSS 1893 X 0.036 ACC DAMP 0.05
SOIL TYPE 2
PERFORM ANALYSIS PRINT MODE SHAPES
PRINT STORY DRIFT
PRINT MEMBER FORCES LIST 1 TO 204
FINISH
```

8.3 Elastomeric Bearing Design

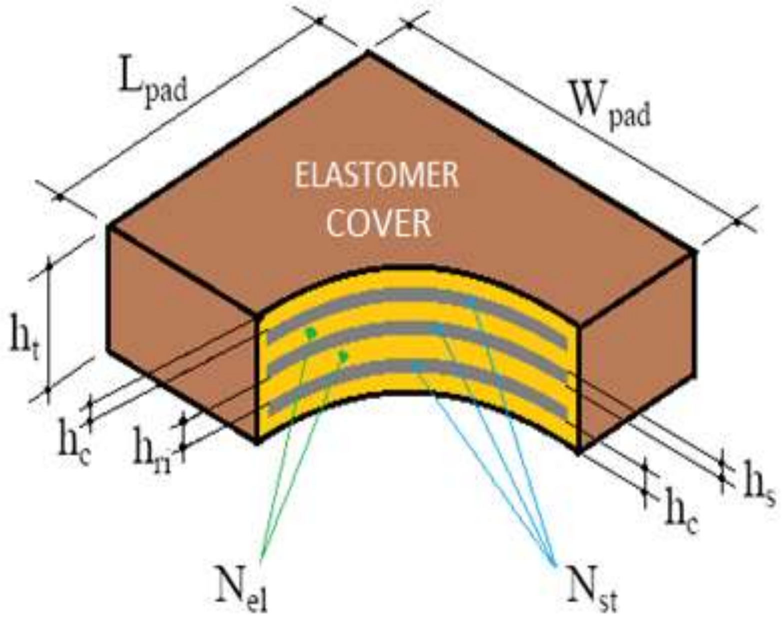


Figure 16 elastomeric bearing

System and material input data :			
Expandable span length	Ls =	17300	mm
Constant amplitude fatigue threshold for Category A	□Ft =	165	Mpa
Elastomer hardness:	Hshore =	50	
Shear modulus of elastomer { (0,68 - 0,93) SELECT }	G =	1	Mpa
Steel reinforcement yield strength:	fy =	240	Mpa
Pad length (bridge longitudinal direction):	Lpad =	250	mm
Pad width (bridge transverse direction):	Wpad =	250	mm
Elastomer cover thickness:	hc =	2.5	mm
Elastomer internal layer thickness:	hri =	8	mm
Number of steel reinforcement layers:	Nst =	5	
Steel reinforcement thickness:	hs =	3	mm
System and material output data :			
Elastomer creep deflection at 25 years divided by the instantaneous deflection:	Cd =	0.25	
Number of elastomer internal layers	Nel =	4	
total elastomer thickness	hrt =	37	mm
Total steel plate height	hst =	15	mm
Total bearing height	ht =	52	mm
Bearing surface area	Area =	62500	mm^2

Check Nst (14.7.6.1) :

Nst =	5		Nst> 2 ise;	
hc =	2.5			
0.70 hri =	5.6		hc ≤ 0.70 hri	OK.

Compute Shape Factor (14.7.5.1-1) :

Sint =	7.8125		Si = L.W / (2.hri.(L+W))
Scov =	25		Si = L.W / (2.hc.(L+W))
S =	7.8125		S=min(Sint,Scov)

Check Compressive Stress (14.7.5.3.2) :

DLs =	106	kN	DL reaction/girder
LLs =	111	kN	LL reaction /girder
σs =	3.472	MPa	σs = (DLs+LLs) /Area
σL =	1.776	MPa	σL = LLs / Area

Shear deformation? -YES- (14.7.5.3.2-2) :

1.66 G.S =	12.96875	Mpa	σs ≤ 1,66 G.S	OK.
0.66 G.S =	5.15625	Mpa	σs ≤ 11	NO
			σL ≤ 0,66 G.S	OK.

Shear deformation? -NO- (14.7.5.3.2-4) :

G.S =	7.8125	Mpa	σs ≤ 2 G.S	OK.
2G.S =	15.625	Mpa	σs ≤ 12	OK.
			σL ≤ G.S	OK.

Check Compressive Deflection (14.7.5.3.3) :

εi =	0.030928		durometer	50	60	70	
δLi =	0.247427	mm	C	0.01676	0.018156	0.013966	
δLt =	1.144351	mm	x	0.29805	0.27257	0.311109	
δcr =	0.007732	mm	εi = C□^x	0.030928	0.016969	0.012929	
Σδ =	1.152083	mm					
0.07 hri =	0.56	mm			δLi ≤ 0.07 hri		OK.

Check Shear Deformation (14.7.5.3.4) :

α =	1.17E-05	/□C		
tset =	20	□C		
γTU =	1.2			
Δco =	4.0482	mm	Δco = α . tset .Ls	
Δs =	4.85784	mm	Δs = Δco .γTU	
2.Δs =	9.71568	mm	2. Δco ≤ hrt	OK.

Check Rotation or Combined Compression and Rotation (14.7.5.3.5) :

Ls =	17300	mm		
θsx =	0.003	rad.	Construction Tolerance	
θsz =	0.003	rad.		
σs =	3.472	MPa		
Nel =	4			
G.S =	7.8125	Mpa		
n =	5			
			Lch = 0.5 GS (Lpad/hri)^2 (θsx/n)	
Lch =	2.165166	Mpa	Lch ≤ σs	OK.
			Wch = 0.5 GS (Wpad/hri)^2 (θsz/n)	
Wch =	2.165166	Mpa	Wch ≤ σs	OK.

Check Stability (14.7.6.3.6) :

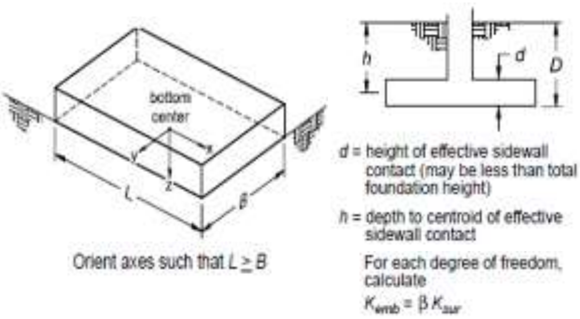
ht =	52	mm	minLW=min(Lpad/3 , Wpad/3)	
minLW =	83.33333	mm	ht ≤ minLW	OK.

Check Reinforcement (14.7.6.3.7) :

hmax =	8	mm		
hsi =	0.3472	mm	hsi =3 hriσs / fy	
			hsi ≤ hmax	OK.
hsii =	0.172218	mm	hsii =2 hriσL / Aft	

SOIL SPRING CONSTANTS FOR FOUNDATIONS

Input Data:



Foundation Length, L =	40.0000	ft.
Foundation Width, B =	25.0000	ft.
Foundation Thk., d =	2.5000	ft.
Soil Unit Weight, γ =	0.120	kcf
Embedment Depth, D =	5.000	ft.
Soil Shear Modulus, G =	5000	psi
Poisson's Ratio, μ =	0.35	

Results:

Nomenclature

Foundation Area and Effective Embedment:

Foundation Area, Af =	1000.00	ft.^2	Af = L*B
Eff. Embed. Depth, h =	3.750	ft.	h = D-d/2, for D >= d/2 (Note: for D < d/2, neglect embedment)

Correction Factors for Embedment:

Embedment Factor, βz =	1.133	βz = (1+1/21*D/B*(2+2.6*B/L))*(1+0.32*(d*(B+L)/(B*L))^(2/3))
Embedment Factor, βx =	1.422	βx = (1+0.21*SQRT(D/B))*(1+1.6*(h*d*(B+L)/(B*L^2))^(0.4))
Embedment Factor, βy =	1.463	βy = (1+0.21*SQRT(D/L))*(1+1.6*(h*d*(B+L)/(L*B^2))^(0.4))

Soil Spring Constants of Entire Foundation at Surface:

Stiff. @ Surface, Kz(sur) =	83217.2	k/ft.	Kz(sur) = G*B/(1-μ)*(1.55*(L/B)^(0.75)+0.8)
Stiff. @ Surface, Kx(sur) =	63434.6	k/ft.	Kx(sur) = G*B/(2-μ)*(3.4*(L/B)^(0.65)+1.2)
Stiff. @ Surface, Ky(sur) =	66052.8	k/ft.	Ky(sur) = G*B/(2-μ)*(3.4*(L/B)^(0.65)+0.4*L/B+0.8)

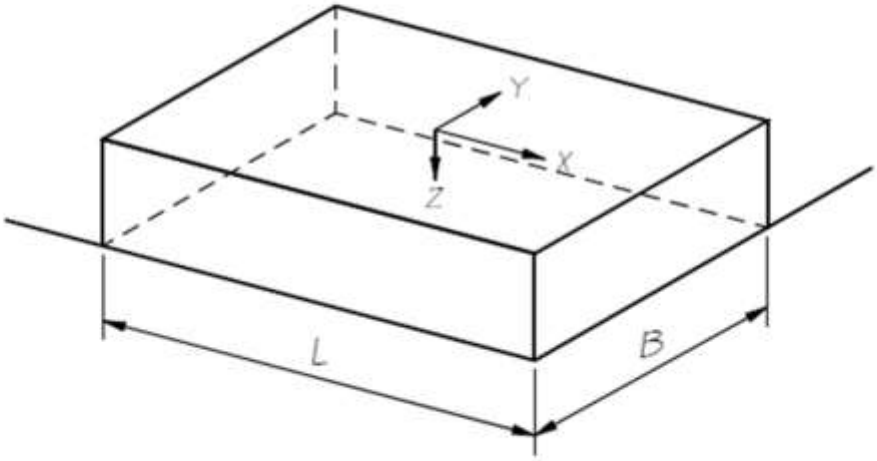
Soil Spring Constants of Entire Foundation Considering Embedment:

Stiff. using Emb., Kze =	94293.8	k/ft.	Kze = βz*Kz(sur)
Stiff. using Emb., Kxe =	90215.9	k/ft.	Kxe = βx*Kx(sur)
Stiff. using Emb., Kye =	96654.6	k/ft.	Kye = βy*Ky(sur)

Unit Values of Soil Spring Constants:

Z-dir. Spring Stiff., kz1 =	94.29	kcf	kz1 = Kze/Af (Z-dir. spring constant per square foot of foundation)
X-dir. Spring Stiff., kx1 =	90.22	kcf	kx1 = Kxe/Af (X-dir. spring constant per square foot of foundation)
Y-dir. Spring Stiff., ky1 =	96.65	kcf	ky1 = Kye/Af (Y-dir. spring constant per square foot of foundation)

From Reference 1:



Orient axes such that $L > B$.
If $L = B$ use x-axis equations for both x-axis and y-axis.
Figure 7.2.7-1

Where:
K = βK_{sur}
K = Translation or rotational spring
K_{sur} = Stiffness of foundation at surface, see Table 7.2.7-1
β = Correction factor for embedment, see Table 7.2.7-2

Degree of Freedom	K_{sur}
Translation along x-axis	$\frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+1.2\right]$
Translation along y-axis	$\frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+0.4\frac{L}{B}+0.8\right]$
Translation along z-axis	$\frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75}+0.8\right]$

Stiffness of Foundation at Surface
Table 7.2.7-1

Degree of Freedom	β
Translation along x-axis	$\left(1+0.21\sqrt{\frac{D}{B}}\right)\left[1+1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$
Translation along y-axis	$\left(1+0.21\sqrt{\frac{D}{L}}\right)\left[1+1.6\left(\frac{hd(B+L)}{LB^2}\right)^{0.4}\right]$
Translation along z-axis	$\left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\cdot\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{\frac{2}{3}}\right]$

Correction Factor for Embedment
Table 7.2.7-2

Typical Values for Soil Properties				
Description	Allow. Bearing (ksf)	Soil Weight (kcf)	Poisson's Ratio (μ)	Shear Modulus G (psi)
Granite	> 10	0.150 - 0.160	0.15 - 0.20	(4 - 6) x 10^6
Limestone	> 10	0.145 - 0.155	0.16 - 0.22	(2 - 5) x 10^6
Sandstone	> 10	0.145 - 0.155	0.17 - 0.24	(1 - 4) x 10^6
Dense Sand	7 - 10	0.115 - 0.140	0.28 - 0.34	(10 - 19) x 10^3
Medium Sand	5 - 7	0.110 - 0.130	0.30 - 0.36	(8 - 15) x 10^3
Loose Sand	3 - 5	0.095 - 0.125	0.32 - 0.38	(5 - 11) x 10^3
Hard Clay	4 - 6	0.125 - 0.145	0.38 - 0.41	(11 - 15) x 10^3
Medium Clay	2 - 4	0.115 - 0.135	0.41 - 0.44	(7 - 11) x 10^3
Soft Clay	1 - 2	0.100 - 0.125	0.44 - 0.47	(3 - 7) x 10^3

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