STABILITY ANALYSIS OF A TALL BUILDING WITH MAT FOUNDATION OVER SANDY SOIL WITH SOIL-STRUCTURE INTERACTION APPROACH

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ABSTRACT: This paper presents the stability analysis of a tall building with mat foundation founded on liquefaction susceptible soil during design earthquake. The stability of a structural foundation and structure itself is mainly depends on the bearing capacity of underlying soil during and after earthquake. Since modulus of subgrade reaction or stiffness of the soil is a conceptual relation between the soil pressure and deflection, it can be changed with fluctuation of net allowable bearing capacity of underlying soil. Bearing capacity of the soil can also be changed according with the value of liquefaction potential. The loss of spring stiffness occurred more or less depending on degree of liquefactions. Modification of localized spring stiffness of the foundation is carried out by numerical method. For a typical 2 unit 10 storey building, the stress states were chosen at a depth of 12 ft. It belongs to seismic zone 2A. ETABS software is used as a design aid for performance of numerical completion for the analysis of model. The focus of this paper is to study the influence of liquefaction potential on modifying spring stiffness of the soil under the building and potential failure modes of the building to be considered in soil structure interaction.

Keywords: Mat foundation, liquefaction, bearing capacity, spring stiffness, soil-structure interaction

1. INTRODUCTION

In the design of reinforced concrete structure, the structural engineer is to design the structure safely, efficiently and economically. Although design codes are utilized, engineering judgments are needed to provide adequate resistance of various structural actions. The main aim of this thesis is to study the influence of liquefaction potential on modifying spring stiffness of the soil under the building, hence modifying building performance. Difference in potential failure modes and stress states of the building are to be considered between soil-structure interaction approach and fixed support condition for mat foundation.

2. METHODOLOGY

In the analysis of soil-foundation interaction problem it is generally assume that the soil medium can be adequately represented by an elastic medium occupying a half-space region. In practice, of course, the foundation is located at some depth below ground surface. The surface of the soil medium is assumed to form the soil-foundation interface. The linear elastic idealization of the supporting soil medium is usually by a mechanical or mathematical model which exhibits the particular characteristic of soil behavior. The simplest model of linear elastic behavior of supporting soil medium is generally assumed that the surface displacement of the soil medium at every point is directly proportionally to the stress applied to it at that point and completely independent of stresses or displacements of other, even immediately neighboring points of the soil-foundation interface. Proposed building is two units, ten storey reinforced concrete building with one storey basement. It is located in down town area of Yangon (Zone 2A). Maximum length and width of building are 54 ft and 52 ft and overall height of building is 109 ft. Occupancy type of building is residential and dual system is used as framing system.

Study program of this paper is presented with flow chart as shown in Figure 1.

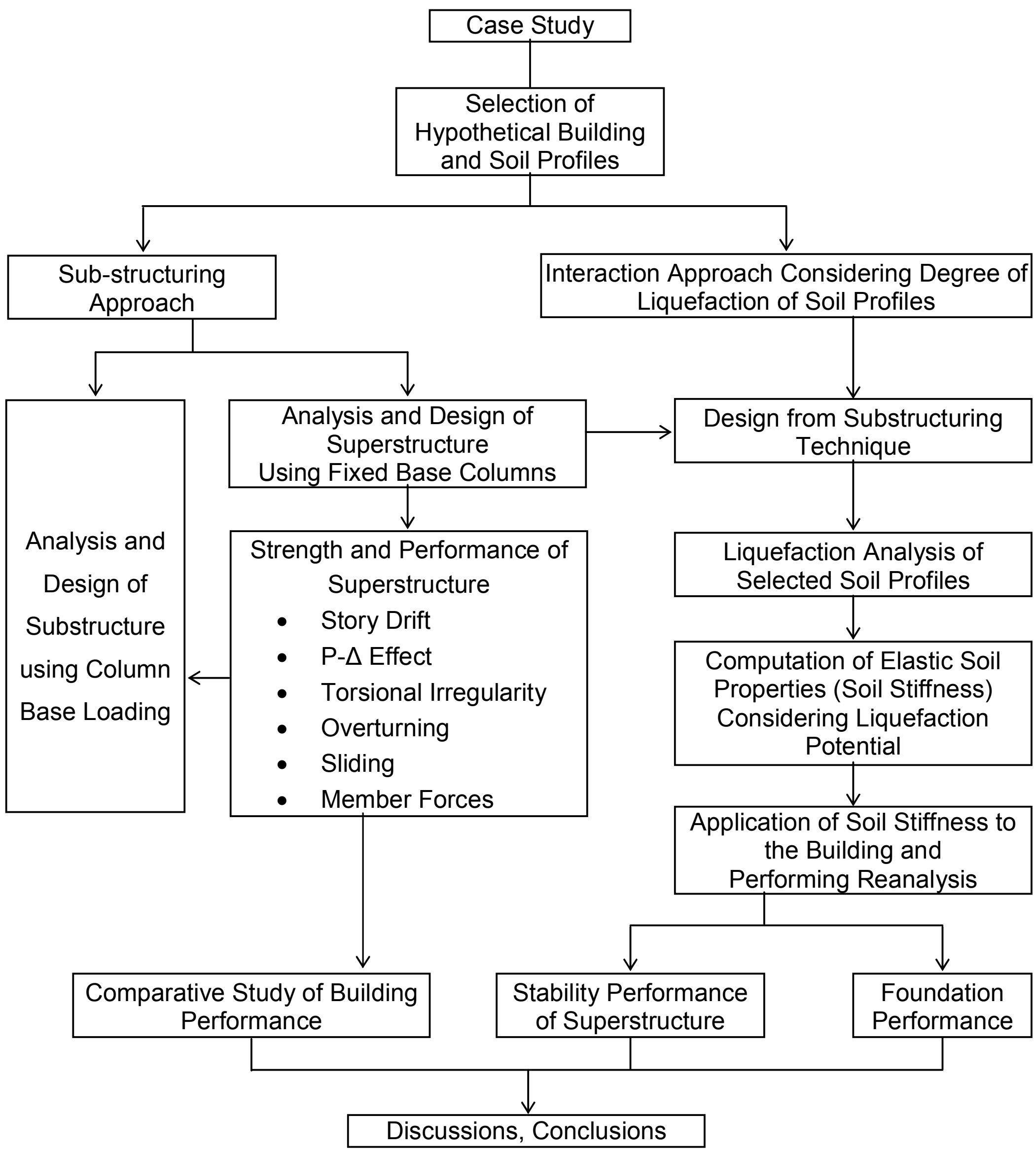

Figure 1. Flow Chart of the Study program

3. EVALUATION OF SUBGRADE REACTIONS CONDSIDERING EFFECT OF LIQUEFACTION

3.1. Prediction of Liquefaction

Shear strength of soil will lose during the design earthquake, so prediction of liquefaction potential is an important role in seismic deign of structure. The occurrence of liquefaction is affected by various geotechnical factors, which are classified into three categories.

1. Soil properties
2. Geological conditions and
3. Ground motion characteristics.

Both soil properties and geological condition are called ground characteristics, which control the resistance of the ground against liquefaction. Ground motion characteristics control the loading condition caused by earthquakes. Among these factors, three factors shown are important.

1. The ground is loose sandy deposit
2. The ground water table is shallow and the ground is saturated and
3. The earthquake intensity is sufficiently high and the duration of earthquake shaking is sufficiently long.

3.2. Method of Liquefaction Prediction

The following are four main steps in the prediction and it consequences.

A. Estimation of stress state before the earthquake and liquefaction resistance.

B. Estimation of seismic shear stress caused by the earthquake

C. Evaluation of liquefaction susceptibility, excess pore water pressure, and deformation of ground, and

D. Evaluation of consequences of liquefaction on soil structure systems.

Among these methods C is estimated using only ground characteristics.

3.3. Simplify Procedure Using SPT-N Value

In the simplified method, the liquefaction strength is estimated either from in-situ test or laboratory tests and the factor of safety against liquefaction is estimated by comparing the liquefaction strength with cyclic shear stress ratio developed in the deposit during an earthquake. Figure 2 shows the flowchart of the method.

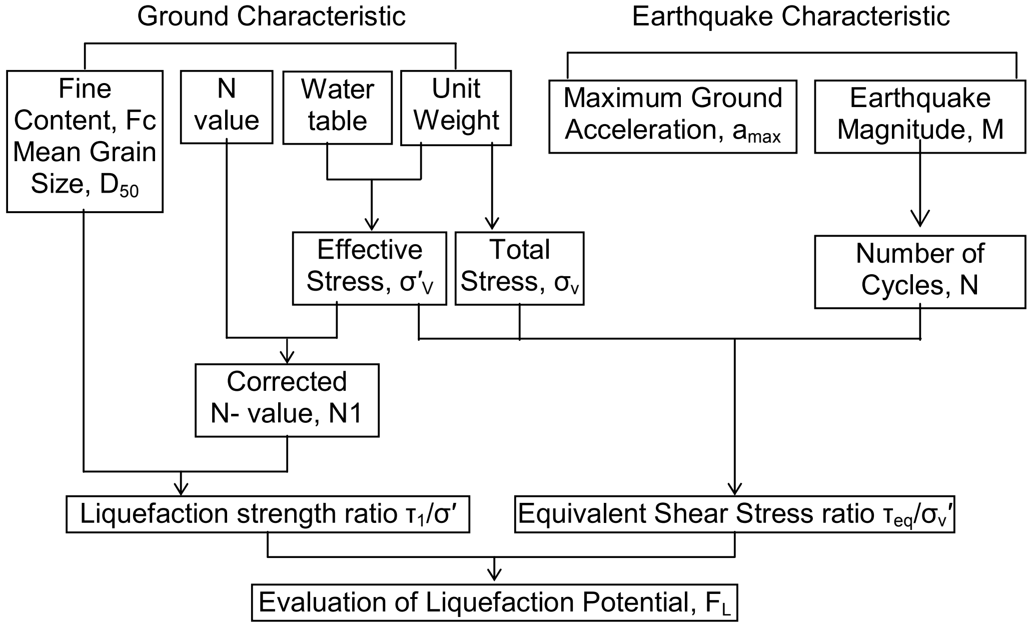

Figure 2. Flowchart of the Simplified Method Using SPT N –Value

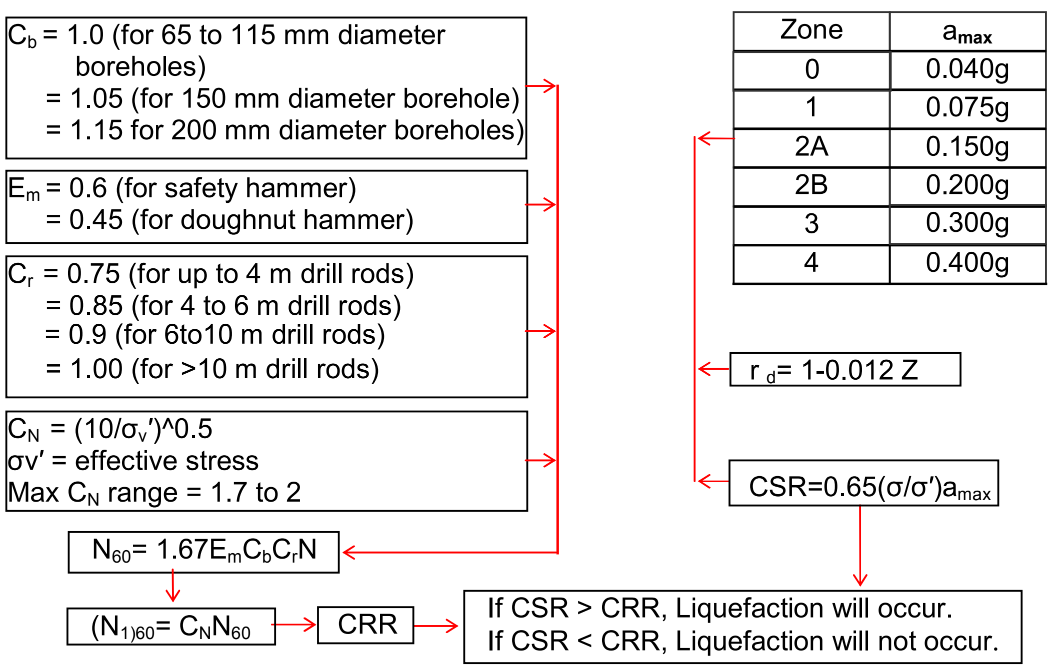

Figure 3. Procedure for Liquefaction Analysis Using SPT N –Value

Even if soil liquefaction may not occur under that condition, the resulting damage to structure is expected to be minor. Such a threshold *N*- value may be called the critical *N*- value. With this critical *N*- value, together with the soil classification, the possibility of liquefaction may be examined without considering the intensity of shaking.

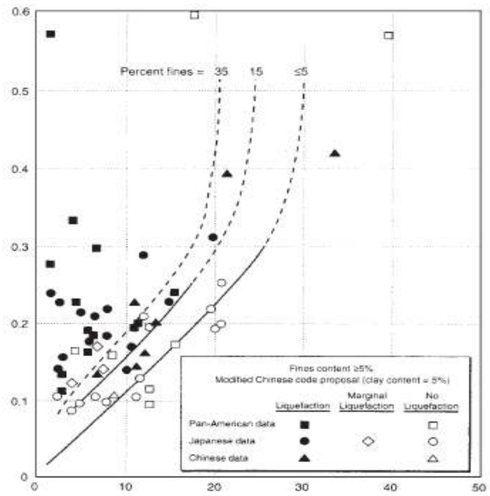

Figure 4. Plot used to determine the Cyclic Resistance Ratio for Clean and Silty Sand for M = 7.5 earthquakes

Table 1. Magnitude Scaling Factors

Anticipated earthquake magnitude	Magnitude scaling factor(MSF)
8.5	0.89
7.5	1.00
6.75	1.13
6	1.32
5.25	1.50

3.4. Soil Profiles

The soil samples were collected from corner of 31st street and Mahabandola road, Pabedan Township, Yangon. Soil samples are collected at the depth of 6, 12, 18, 24, 30, 40, 50, 60, 70, 80, 90 and 100 ft by using split spoon sampler. The water tables are found at a depth of 2.5 ft from existing ground. From boreholes 1 and 2 soil profiles data required for analysis of liquefaction potential are listed in Table 2. The prediction of liquefaction potential for soil profiles are calculated in Table 3.

3.5. Bearing Capacity

Any soil type is weakened during the earthquake due to liquefaction. In granular soil that does liquefy, but it below ground water table and have a factor of safety against liquefaction greater than 1.0 but less than 2.0, there is a reduction in shear strength due to an increased in pore water pressure. If the factor of safety against liquefaction greater than 2.0, the earthquake induce pore water will typically be small enough that their effect can be neglected. The final result of ultimate bearing capacity of strip foundation in earthquake time is

$$q_u = \frac{1}{2}(1 - r_u)\gamma_b B N_\gamma$$

Where, r_u = pore water pressure ratio
γ_b = buoyant unit weight of soil below foundation
B = width of foundation
N_γ = bearing capacity factor

The ultimate bearing capacity of soil profiles are tabulated in Table 4.

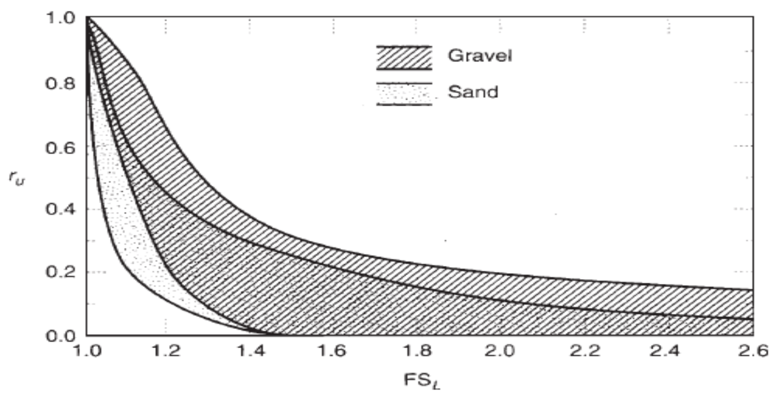

Figure 5. Factor of Safety against Liquefaction F_{SL} versus the Pore Water Pressure Ratio r_u for Gravel and Sand

From the liquefaction potential calculation, factor of safety against liquefaction for borehole 1 and 2 are 1.42 and 1.24 respectively. For borehole 1,

r_{umin} = 0.00, r_{umax} = 0.30, r_{uavg} = 0.15

q_{u1} in earthquake time = (1-0.15) 13.83 = 11.76 kip/ft^2

For borehole 2,

r_{umin} = 0.17, r_{umax} = 0.41, r_{uavg} = 0.29

q_{u2} in earthquake time = (1-0.29) 11.43 = 8.11 kip/ft^2

3.6. Subgrade Reaction

In the performance of the analysis for the structural design of soil-foundation interaction approach, it is required to know the principle of evaluating the coefficient of subgrade reaction, k. If a foundation of width B is subjected to a load per unit area of q, it will undergo a settlement, Δ. The coefficient of subgrade modulus, k, can be defined as

$$k = \frac{q}{\omega}$$

Where,

q = bearing capacity of supporting soil

ω = settlement

Table 2. Evaluation of Liquefaction Potential for Borehole 1and 2

			Cyclic stress ratio (CSR)							N value correction						
Borehole	Depth		γ_{sat}	σ_v	σ_v'	r_d	CSR	N	C_r	N_{60}	C_N	$(N_1)_{60}$	FC(%)	CRR	FS(0.15)	
	ft	m	pcf	psf	psf										M=7.5	M=6
1	6	1.83	129.81	778.86	515.34	0.98	0.14	9	0.75	6.76	2.00	13.53	20	0.16	1.11	1.47
	12	3.66	128.13	1537.56	903.84	0.96	0.16	11	0.85	9.37	1.52	14.24	24	0.17	1.07	1.42
	18	5.49	126.47	2276.46	1272.49	0.93	0.16	13	0.95	12.37	1.28	15.85	22	0.19	1.17	1.54
	24	7.32	132.29	3174.96	1782.04	0.91	0.16	17	0.95	16.18	1.08	17.52	12	0.15	0.95	1.25
	30	9.14	129.60	3888.00	2127.40	0.89	0.16	20	1.00	20.04	0.99	19.86	16	0.19	1.20	1.58
	40	12.19	130.04	5201.60	2815.90	0.85	0.15	18	1.00	18.04	0.86	15.53	17	0.17	1.11	1.46
	50	15.24	130.23	6511.50	3501.33	0.82	0.15	21	1.00	21.04	0.77	16.25	19	0.19	1.28	1.69
	60	18.29	127.92	7675.20	4046.80	0.78	0.14	22	1.00	22.04	0.72	15.84	16	0.21	1.45	1.92
	70	21.34	127.69	8938.30	4686.48	0.74	0.14	24	1.00	24.05	0.67	16.05	36	0.27	1.95	2.58
	80	24.38	120.75	9660.00	4801.53	0.71	0.14	30	1.00	30.06	0.66	19.83	23	0.27	1.95	2.57
	90	27.43	118.94	10704.60	5226.65	0.67	0.13	38	1.00	38.08	0.63	24.07	22	0.32	2.39	3.15
	100	30.48	121.54	12154.00	6045.55	0.63	0.12	50	1.00	50.10	0.59	29.45	34	0.60	4.83	6.37
2	6	1.83	129.49	776.94	509.80	0.98	0.15	11	0.75	8.27	2.00	16.53	20	0.13	0.89	1.18
	12	3.66	126.43	1517.16	883.27	0.96	0.16	12	0.85	10.22	1.54	15.72	22	0.15	0.94	1.24
	18	5.49	126.01	2268.18	1260.94	0.93	0.16	12	0.95	11.42	1.29	14.70	23	0.16	0.98	1.29
	24	7.32	132.25	3174.00	1776.76	0.91	0.16	21	0.95	19.99	1.08	21.67	15	0.15	0.94	1.25
	30	9.14	130.36	3910.80	2143.88	0.89	0.16	19	1.00	19.04	0.99	18.79	14	0.15	0.95	1.25
	40	12.19	133.49	5339.60	2940.86	0.85	0.15	20	1.00	20.04	0.84	16.89	14	0.17	1.12	1.48
	50	15.24	128.34	6417.00	3407.13	0.82	0.15	23	1.00	23.05	0.78	18.04	24	0.22	1.47	1.94
	60	18.29	128.63	7717.80	4083.21	0.78	0.14	22	1.00	22.04	0.72	15.77	20	0.24	1.67	2.20
	70	21.34	123.81	8666.70	4420.16	0.74	0.14	30	1.00	30.06	0.69	20.66	48	0.24	1.69	2.23
	80	24.38	119.99	9599.20	4738.21	0.71	0.14	45	1.00	45.09	0.66	29.94	12	0.22	1.57	2.08
	90	27.43	119.68	10771.20	5286.98	0.67	0.13	67	1.00	67.13	0.63	42.20	10	0.23	1.73	2.28
	100	30.48	121.42	12142.00	6029.43	0.63	0.12	53	1.00	53.11	0.59	31.26	2	0.21	1.69	2.23

Table 3. Soil Properties from Borehole 1and 2

Borehole	DEPTH FT	MOISTURE CONTENT (%)	DENSITY (lb/cu.ft)		Shear Characteristics		FC (%)	U.C.S TEST Strength (lb/sq-ft)	NUMBER OF BLOW PER FOOT
			Wet	Dry	Cohesion (lb/sq-ft) C	Angle of Friction (Φ)			
1	6	18.11	129.81	109.91	360.0	29.0	20	502.75	9
	12	18.47	128.13	108.15	0.0	32.0	24	0.00	11
	18	17.87	126.47	107.30	432.0	31.0	22	633.10	13
	24	13.61	132.29	116.44	576.0	29.0	12	875.16	17
	30	14.68	129.60	113.01	144.0	34.5	16	382.33	20
	40	14.65	130.04	113.42	0.0	35.0	17	0.00	18
	50	21.84	130.23	106.89	360.0	39.0	19	488.85	21
	60	16.23	127.92	110.06	302.4	35.0	16	341.10	22
	70	19.70	127.69	106.68	259.2	19.0	36	1244.77	24
	80	20.12	120.75	100.52	360.0	24.0	23	486.72	30
	90	21.55	118.94	97.85	576.0	18.5	22	407.40	38
	100	20.55	121.54	99.99	0.0	27.0	34	0.00	50
2	6	18.11	129.49	112.54	72.0	25.0	20	502.75	11
	12	18.47	126.43	104.76	0.0	30.5	22	0.00	12
	18	17.87	126.01	106.27	350.5	31.0	23	633.10	12
	24	13.61	132.25	114.68	288.0	29.0	15	0.00	21
	30	14.68	130.36	144.38	144.0	34.5	14	382.33	19
	40	14.65	133.49	116.67	0.0	36.0	14	528.12	20
	50	21.84	128.34	112.76	144.0	37.0	24	0.00	23
	60	16.23	128.63	112.40	144.0	28.0	20	341.10	22
	70	19.70	123.81	100.15	43.2	25.5	48	1244.77	30
	80	20.12	119.99	97.96	72.0	21.5	12	486.72	45
	90	21.55	119.68	97.36	259.2	24.0	10	0.00	67
	100	20.55	121.42	101.80	0.0	39.0	2	231.84	53

Table 4. Evaluation of Bearing Pressure for Borehole 1and 2

Borehole	Depth		γ_{sat}	c	N	N_{60}	$(N_1)_{60}$	ϕ	ϕ′	N'_q	N'_c	N'_γ	$q_{ult(loc)}$
	ft	m	pcf	psf				deg	deg				ksf
1	6	1.83	129.81	360.0	9	6.8	13.5	29.0	20.3	6.60	15.14	5.62	13.86
	12	3.66	128.13	0.0	11	9.4	14.2	32.0	22.6	8.35	17.64	7.80	13.83
	18	5.49	126.47	432.0	13	12.4	15.9	31.0	21.8	7.71	16.74	6.98	16.90
	24	7.32	132.29	576.0	17	16.2	17.5	29.0	20.3	6.60	15.14	5.62	16.42
	30	9.14	129.60	144.0	20	20.0	19.9	34.5	24.6	10.27	20.23	10.34	20.70
	40	12.19	130.04	0.0	18	18.0	15.5	35.0	25.0	10.72	20.82	10.95	20.00
	50	15.24	130.23	360.0	21	21.0	16.3	39.0	28.4	15.38	26.61	17.69	38.79
	60	18.29	127.92	302.4	22	22.0	15.8	35.0	25.0	10.72	20.82	10.95	23.57
	70	21.34	127.69	259.2	24	24.0	16.1	19.0	12.9	3.25	9.79	1.95	5.13
	80	24.38	120.75	360.0	30	30.1	19.8	24.0	16.5	4.57	12.03	3.31	8.10
	90	27.43	118.94	576.0	38	38.1	24.1	18.5	12.6	3.14	9.61	1.85	6.51
	100	30.48	121.54	0.0	50	50.1	29.4	27.0	18.8	5.68	13.77	4.54	7.25
2	6	1.83	129.49	72.0	11	8.3	16.5	25.0	17.3	4.91	12.57	3.68	7.26
	12	3.66	126.43	0.0	12	10.2	15.7	30.5	21.4	7.41	16.32	6.61	11.43
	18	5.49	126.01	350.5	12	11.4	14.7	31.0	21.8	7.71	16.74	6.98	15.90
	24	7.32	132.25	288.0	21	20.0	21.7	29.0	20.3	6.60	15.14	5.62	13.50
	30	9.14	130.36	144.0	19	19.0	18.8	34.5	24.6	10.27	20.23	10.34	20.91
	40	12.19	133.49	0.0	20	20.0	16.9	36.0	25.8	11.70	22.07	12.31	23.62
	50	15.24	128.34	144.0	23	23.0	18.0	37.0	26.7	12.79	23.45	13.86	26.92
	60	18.29	128.63	144.0	22	22.0	15.8	28.0	19.5	6.12	14.43	5.05	10.41
	70	21.34	123.81	43.2	30	30.1	20.7	25.5	17.6	5.09	12.85	3.87	6.79
	80	24.38	119.99	72.0	45	45.1	29.9	21.5	14.7	3.84	10.83	2.55	4.48
	90	27.43	119.68	259.2	67	67.1	42.2	24.0	16.5	4.57	12.03	3.31	7.20
	100	30.48	121.42	0.0	53	53.1	31.3	39.0	28.4	15.38	26.61	17.69	28.19

For 1 in settlement,

$$k = 12\,S.Fq_{all}$$

Where,

q_{all} = allowable bearing capacity of supporting soil

S.F = safety factor

Horizontal coefficient of subgrade reactions at mat node points are

$$k_x = k_y = \frac{32(1 - v_{sr})G_r a}{7 - 8v_{sr}}$$

Where,

G_y = soil shear modulus beneath the mat

υ_{rs} = Poisson' ratio of soil near the mat

a = equivalent radius of the mat element

The stiffness values are force required to applied 2 ft x 2 ft mat element to translate 1.0 *in* in respective direction. Assign spring stiffness in each node points are listed in Table 5.

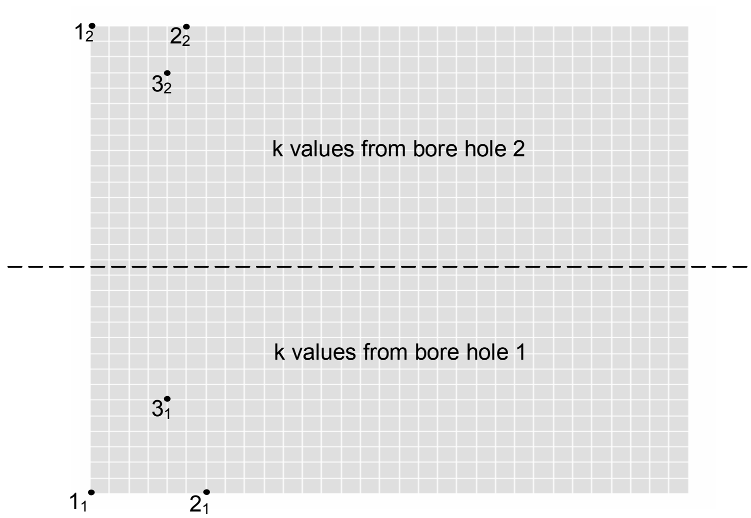

Figure 6. Mat Nodes Points

Table 5. Soil Stiffness on Mat Nodes

Location	Node pt	kx	ky	kz	Node pt	kx	ky	kz
Corner	1_1	250	250	294	1_2	250	250	203
Periphery	2_1	500	500	588	2_2	500	500	406
Interior	3_1	1000	1000	1176	3_3	1000	1000	812

4. **STABILTY ANALYSIS OF PROPOSED BUILDING**

4.1. Profile of Structures

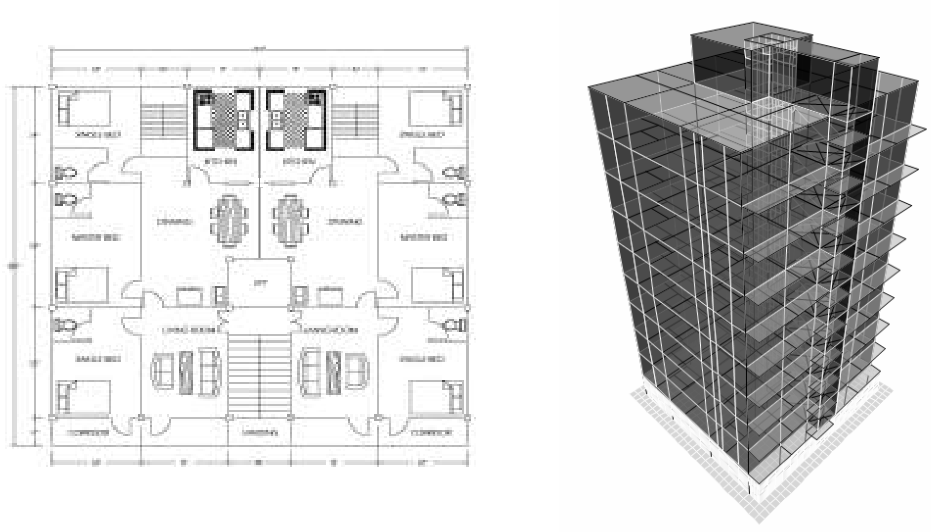

Figure 7. Typical Floor Plan and Three-Dimensional View of Proposed Building

4.2. Structural Stability Consideration

Storey drift, P-Δ effect, torsional irregularity, overturning moment and sliding should be checked for the structural stability. The discussion on stability is, therefore concerned with the whole structure, or the whole stories of the structure, rather than with individual members.

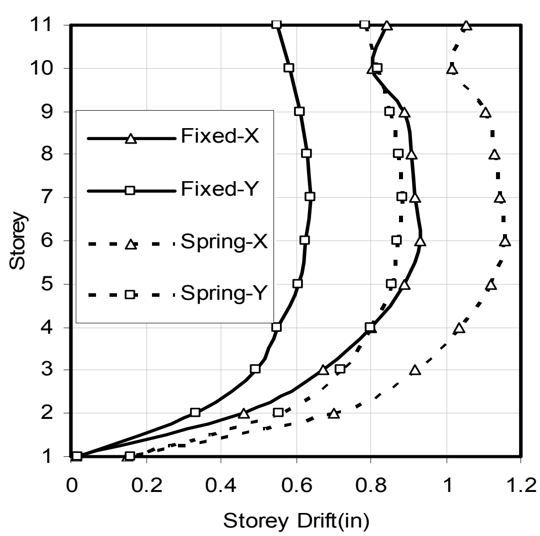

Figure 8. Comparison of Maximum Storey Drifts due to EQX and EQY

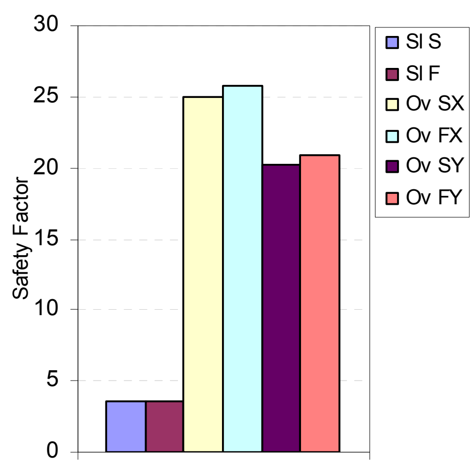

Figure 9. Comparison of Safety Factor Against Sliding and Overturning

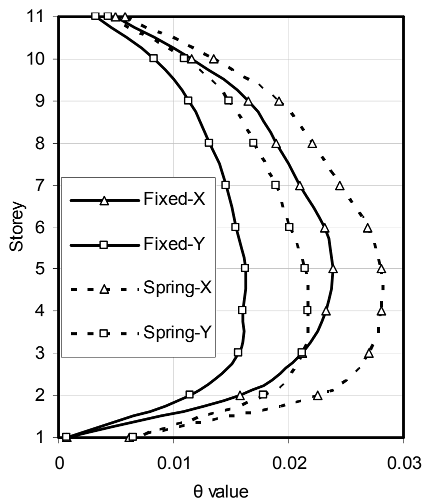

Figure 10. Comparison of P-Δ Effect due to EQX and EQY

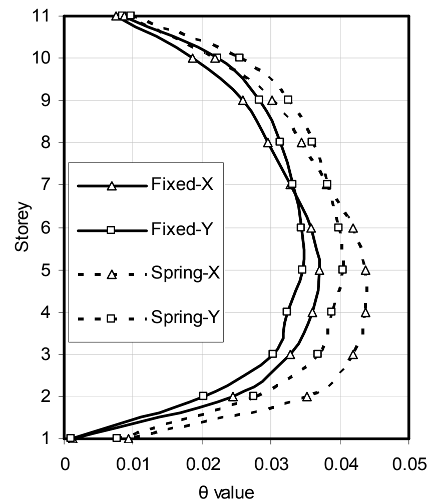

Figure 11. Comparison of P-Δ Effect due to DL+LL+EQX and DL+LL+EQY

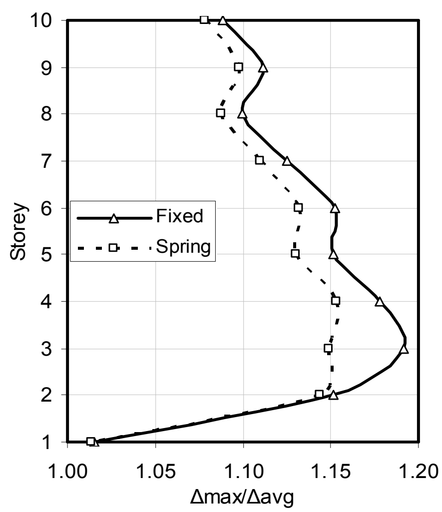

Figure 12. Comparison of Torsional Irregularity of Building in X direction due to EQX

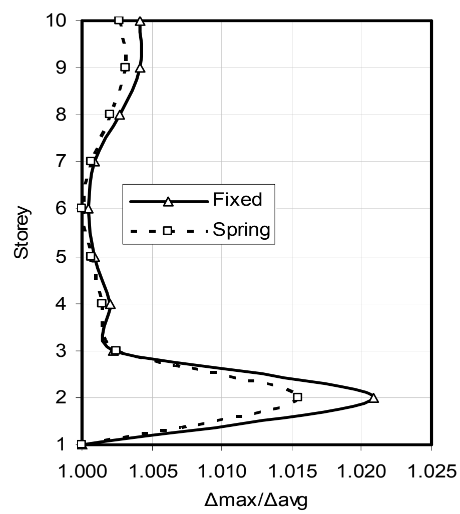

Figure 13. Comparison of Torsional Irregularity of Building in Y direction due to EQY

4.3. Comparison of the Critical Forces for Beams

The forces result from the various load combination and maximum values of critical forces of beams in fixed base and spring base foundation are compared. To compare the forces, beams are groped into the following:

(1) Exterior beam
(2) Interior beam and
(3) Shear wall end beam.

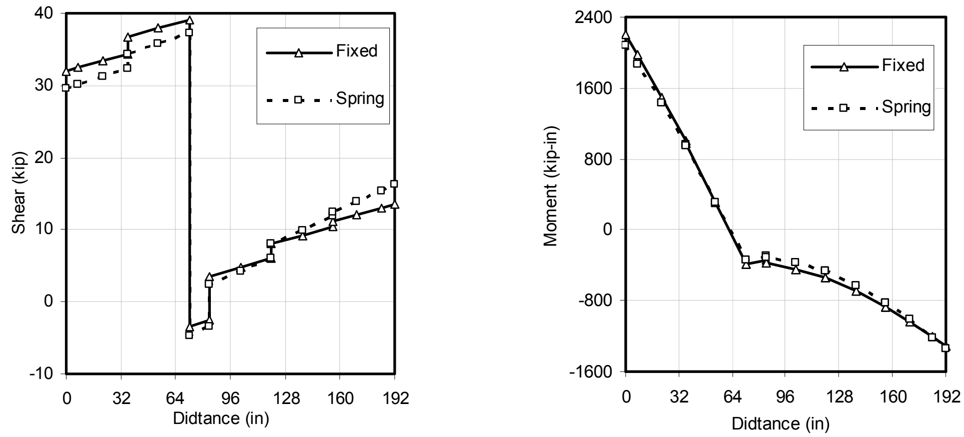

Figure 14. Beam Shear and Moment Diagrams (Shear Wall End Beam)

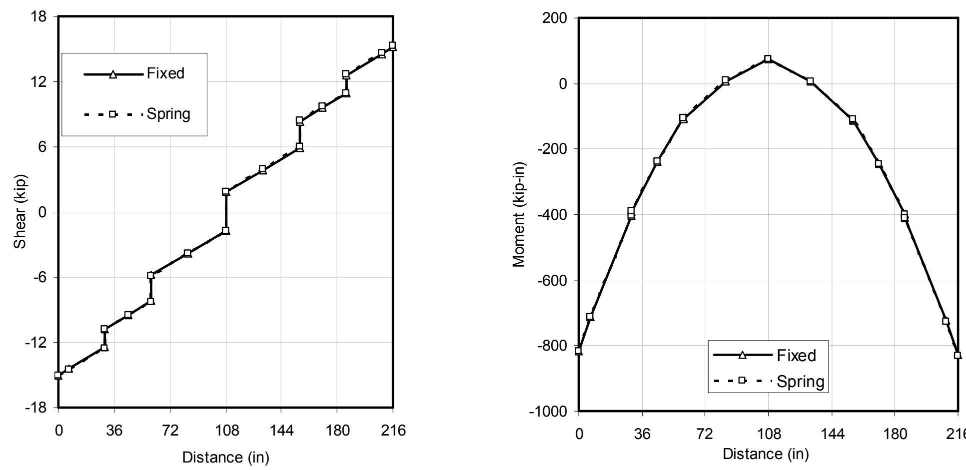

Figure 15. Beam Shear and Moment Diagrams (Interior Beam)

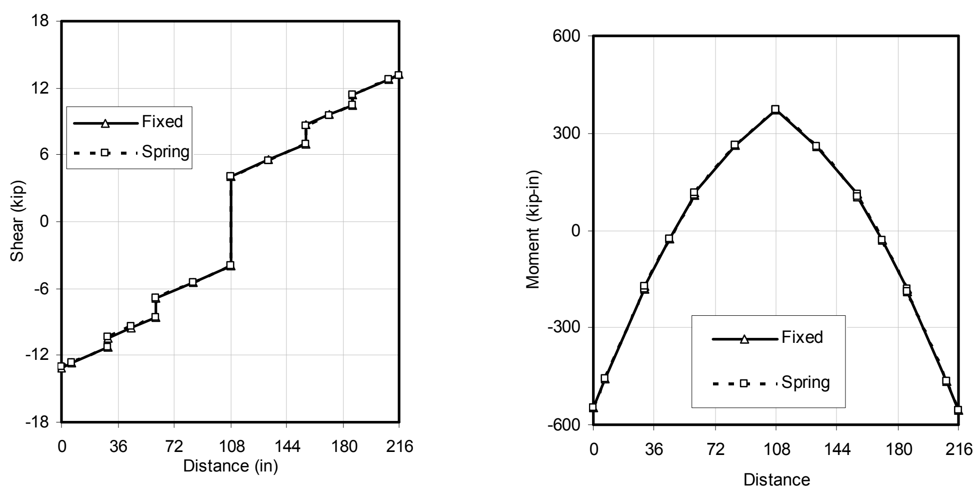

Figure 16. Beam Shear and Moment Diagrams (Exterior Beam)

4.4. Comparison of Critical Forces for Columns

The forces result from the various load combinations and maximum values of critical forces of columns in fixed base and spring base foundation are compared. To compare the forces, columns are grouped into the following:

Location	Section Label	Coordinate	
Corner column	C1	0,	0
Interior column	C15	36,	34
Exterior column	C13	0,	34

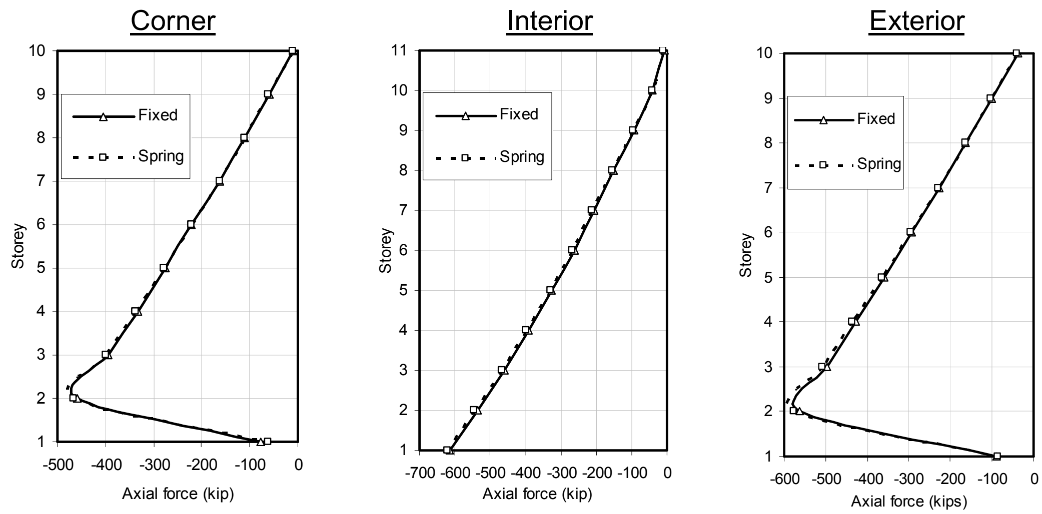

Figure 17. Axial Force of Corner, Interior and Exterior Columns

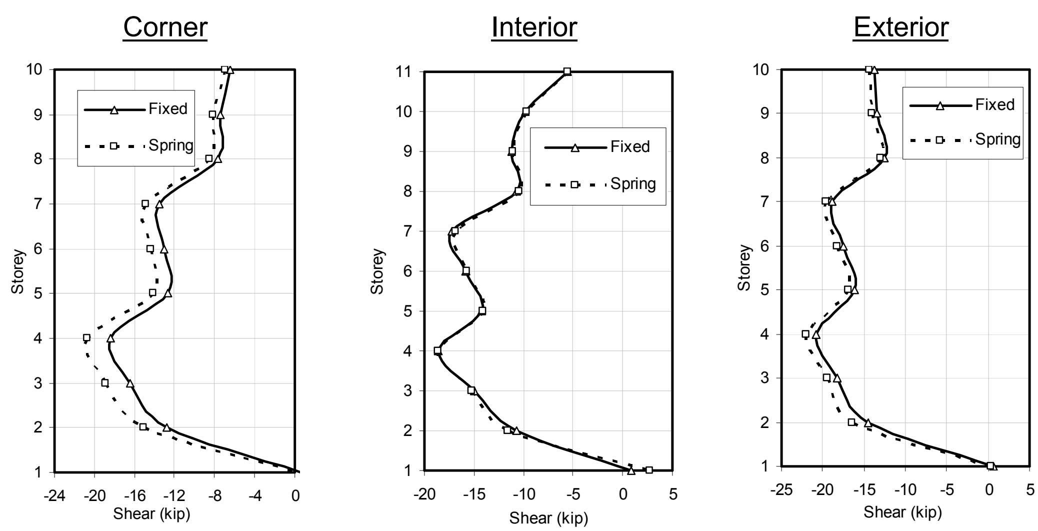

Figure 18. Shear Force of Corner, Interior and Exterior Columns

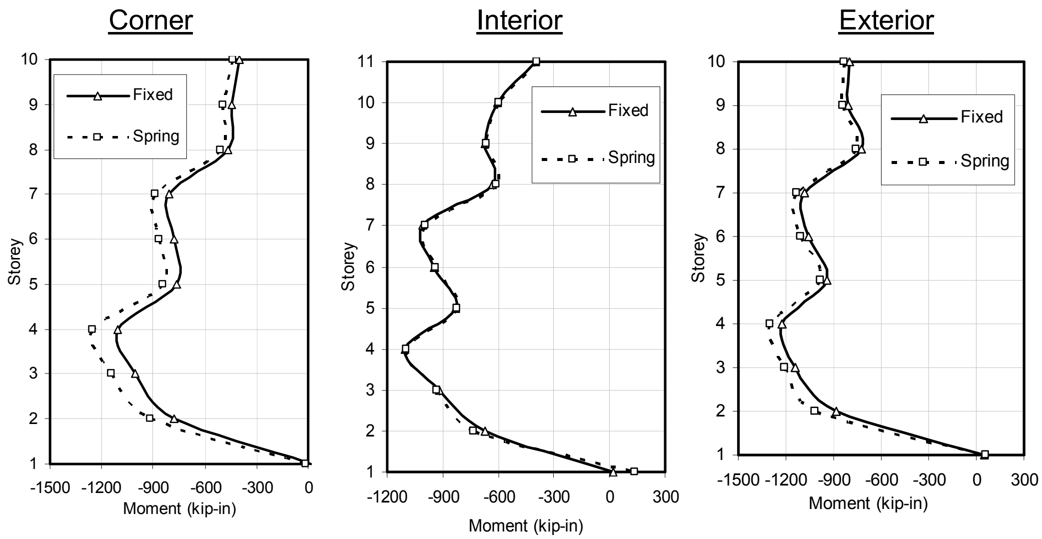

Figure 19. Bending Moment of Corner, Interior and Exterior Columns

Table 6. Settlement of Points under Columns and Shear Walls

Col.No.	Point No	Coordinate		Load combination				Max
		x	y	11	12	13	14	
1	99	0	0	0.30	0.57	0.28	0.59	0.59
2	107	23	0	0.33	0.61	0.44	0.50	0.61
3	111	31	0	0.33	0.61	0.49	0.44	0.61
4	119	54	0	0.31	0.58	0.59	0.29	0.59
5	243	0	16	0.40	0.47	0.30	0.58	0.58
6	256	18	16	0.51	0.45	0.40	0.55	0.55
7	261	23	16	0.55	0.46	0.46	0.55	0.55
8	265	31	16	0.55	0.46	0.55	0.46	0.55
9	270	36	16	0.51	0.45	0.56	0.40	0.56
10	283	54	16	0.41	0.48	0.58	0.30	0.58
11	338	23	23	0.62	0.41	0.48	0.56	0.62
12	342	31	23	0.63	0.41	0.56	0.47	0.63
13	413	0	34	0.52	0.39	0.31	0.59	0.59
14	421	18	34	0.59	0.37	0.42	0.53	0.59
15	429	36	34	0.59	0.37	0.53	0.42	0.59
16	437	54	34	0.52	0.39	0.59	0.32	0.59
17	549	0	48	0.60	0.32	0.32	0.60	0.60
18	557	18	48	0.60	0.33	0.42	0.51	0.60
19	575	36	48	0.61	0.33	0.51	0.42	0.61
20	583	54	48	0.61	0.32	0.61	0.33	0.61
21	103	12	0	0.31	0.58	0.35	0.53	0.58
22	115	42	0	0.31	0.58	0.53	0.36	0.58
SW 1	298	23	19.5	0.60	0.43	0.47	0.56	0.60
SW 2	299	31	31	0.60	0.44	0.56	0.47	0.60
SW 3	340	27	23	0.64	0.41	0.52	0.52	0.64
SW 4	775	26	16	0.56	0.46	0.50	0.52	0.56
SW 5	776	28	16	0.56	0.46	0.53	0.50	0.56

5. **Discussion and Conclusion**

In this study, performance of proposed buildings located in seismic zone 2A is checked, in terms of stabilities and strengths. A tall building with mat foundation founded on two different soils under building is analyzed by mean to introduce liquefaction potential of foundation soil in spring stiffness calculation and modification for inertia interaction problem is presented. According to the finding from this study the prediction of liquefaction potential is required and useful in sandy soil even it is on the liquefaction of greater than 1.0. Liquefied soils loss its shearing strength and cannot bear any structural loads on it and the result can be excessive settlements and bearing capacity failures of the buildings, most of which are supported on shallow foundations. In practice, however, it also involves the estimation of seismic pore water pressures and displacement in soil which do not fully liquefy and their effect on structures. If borehole data are available for corresponding influenced zone specifically, compatible soil springs can be predicted as closely as the real soil performance under the effect of liquefaction potential. More reliable performance prediction can be obtained and precautions can be done. For the selected problem in this study, because of the selection of geometry of the building, it is founded that larger storey drift in upper stories, it may influence the spacing between near by building and also increase the second order effect although there is no serious result for overturning and strength deficiency. It may cause open up between near by building although the amount is within the tolerance. Although there is no serious in beam but it is serious for columns, especially which is located at building corner.

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SEISMIC PERFORMANCE OF INTEGRAL ABUTMENT BRIDGE

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ABSTRACT

In this study, proposed model of integral abutment bridge is considered in seismic zone 2A. The requirements for this model analysis and design specifications for structural elements according to Uniform Building Code (UBC 97) and AISC-LRFD 93 are accomplished in this study. The finite element program SAP 2000 is used to model the structure with the required input data for analysis and design.

In integral abutment bridges, the lateral loads, mainly thermal deck movements and seismic loads are accommodated by soil-structure interaction between the abutment backfill soil, supporting piles and surrounding strata. The effective temperature governs the overall longitudinal movement of the bridge superstructure. All the forces developed in the bridge will be transferred to the foundations through abutments and piles. The forces in the different soil behind the abutments and next to the foundation due to thermal variation and seismic loading are major concern in this study.

In this study, the superstructure, substructure and piles are modelled by using frame elements. The abutment is modelled as shell element. Rigid elements are used to connect the deck to the piles and springs will be used to model the soil reaction. The soil response next to the piles is modelled using link element.

The main objective of thesis is to study the behaviour of integral abutment bridge under temperature variations and seismic loading and investigate variation of stresses at critical locations of bridge and to compare the various analysis results on abutment and along the bridge's deck slab. It is hoped that this study will provide useful information on seismic performance of integral abutment bridges.

Keywords: Integral Abutment Bridge, Soil-Structure Interaction, Frame, Shell, Rigid, Spring, Temperature Variations, Seismic Loading

1. INTRODUCTION

Integral abutment bridges are simple or multiple span bridges and act as a rigid-frame structure or a single structural element. So, integral abutment bridges provide greater protection against translation and uplift than conventional bridges. The superstructure of integral abutment bridge is to provide smooth transition with the adjacent approach slabs.

In integral abutment bridges, the lateral loads, mainly thermal deck movements and seismic loads are accommodated by soil-structure interaction between the abutment back soil, supporting piles and surrounding strata. The lateral soil reaction is inherently nonlinear. So, integral abutment bridges, the soil-structure interaction (SSI) are the main. To solve these, the finite element program SAP2000 is used to model the structure.

1.1. Objective

The length of integral abutment bridges increase and decrease. Pushing the abutment against the approach fill and pulling it away. As a result, the bridge's superstructure, the abutment, the approach fill, the foundation piles and the foundation soil are all subjected to cyclic loading. The main objective is to study the behaviour of integral abutment bridge under temperature variations and seismic loading, and investigate variation of stresses at critical locations of bridge. At the same time, SAP2000 software will be used to achieve the integral abutment bridges which have acceptable performance under seismic loads and the possibility of using procedures for design efficiencies of integral abutment bridges.

1.2. **Scope**

To get the objectives as mentioned above, the scope of the study is defined as follows:

1. The bridge is designed to carry two lanes of AASHTO HS 20-44 traffic loading.
2. Uniform temperature changes from a reference temperature is considered for the whole bridge.
3. Equivalent static method is used for seismic loading.
4. Effect of backfill soil behind abutments is considered as loading on bridge.
5. No interaction between soil and abutment is considered in this study.
6. Interaction between supporting piles and surrounding soil is modelled using appropriate linear elastic springs.

2. **METHODOLOGY**

Bridges without any expansion joints and without any bearings are integral abutment bridges. Integral abutment bridges are simple or multiple span bridges in which the superstructure is cast. An integral abutment bridge is to physically and structurally connect the superstructure and abutments (substructure). So, integral abutment bridges act as a single structural element. Due to the elimination at the bridge deck expansion joints, which allow water to leak onto substructure elements and accelerate deterioration, construction and maintenance costs are reduced and few piles are required for foundation support of integral abutment bridge. Accident and vehicle damage caused by defective expansion joints raised safety concern. As it is monolithically simple structure modification, future widening or bridge replacement becomes easier. Integral abutment bridges should help to increase the serviceable bridge age and extend replacement cycles by two or more decades.

2.1. **Types of Integral Abutment Bridge**

Integral abutment bridges have also been called, integral abutment bridges, jointless bridges, rigid-frame bridges and U-frame bridges. The integral abutment bridges designs depend on structure materials, soil properties, types of foundation and climate condition.

Integral abutment bridges' designs are variables. Ranges of design criteria for Integral abutment bridges are shown in Table 1.

Table 1. Range of Design Criteria for IAB [1]

	Steel Girders	**Concrete**
Maximum span (ft)	65-300	60-200
Total length (ft)	150-650	150-1175
Maximum skew (degree)	15-70	15-70
Maximum curvature	0-10	0-10

2.2. **Integral Abutment Bridge**

The bridge superstructure can change in temperature and tend to change dimension in its longitudinal direction, because of natural, seasonal variations in air temperature in air temperature. To accommodate the seasonal relative movement between superstructure and abutments and prevent temperature-induced stresses from developing within the superstructure, the traditional solution has been to provide expansion joints and bearings at each end of the superstructure. Therefore, the concept was developed to physically and structurally connect the superstructure and abutments as known an integral abutment bridge. The integral abutment bridges are sensitive to daily and seasonal temperature variation. If this variation is less, lesser forces are induced in the structure. The areas where difference between maximum and minimum temperature is minimal, the integral bridge must be supported. The integral abutment bridges with vertical pile systems are suitable in seismic areas under moderate weather conditions.

2.3. **Model of Integral Abutment Bridge**

Seismic zone	- Zone II
Total length of bridge	- 100 ft
Number of span	- Single span
Roadway width	- 30 ft
Curb width	- each 3 ft
Type of bridge	- Reinforced concrete (IAB)

Girder	- 5 Nos
	- Spacing 6ft
Girder section	- W 44X335
Type of Abutment	- Concrete
Type of Soil	- Medium Consolidated Soil
Pile section	- HP 14X117
Pile	- 5 Nos
	- Spacing 6ft
Maximum permissible traffic load	
-Traffic	- HS- 20-44 two lane
	- H - 20-44 two lane

2.4. **Material Properties of Structure**

Unit weight of concrete,	γ_c	= 150 lb/ft^3
Yield strength of steel,	f_y	= 60000 lb/ft^2 for main frame members such as girders and piles
	f_y	= 50000 lb/in^2 for slab
Design strength of concrete, f'_c		= 3000lb/in^2

2.5. **Loads on Model**

A set of standard loading conditions are applied to the design model of the structure. The principle loading constraint which highway bridges are designed by is truck loading. The variety of trucks in use, it was determined that a stand set of design loading caused by truck traffic. Loads represent actions upon the structures, such as force, pressure, support displacement, thermal effects, ground acceleration, and others. Loads that need to vary independently, either for design purposes or because of how they are applied to the objects as part of that load case. The program automatically computes built-in ground acceleration loads.

This study, the loadings are applied to the computer structural model by using SAP2000 structural analysis software. The loadings applied to the structural model for the proposed bridge are follows:

(a) Dead load
(b) Live load
(c) Impact or dynamic effect of the live load
(d) Thermal force
(e) Pressure

(f) Seismic Loads

2.5.1. Dead Load

Dead loads are constant in magnitude and fixed in location throughout the lifetime of the structure. The major part of the dead load is the weight of the structure itself. Dead loads are defined as gravity loads that will be accelerated laterally with the structural frame under earthquake motion. They may include weight of the slab, girders and other members, wearing surfaces, sidewalks, tailing and an allowance is made for piping and other public utility services.

Dead loads used in the structural analysis are as follows:

Weight unit volume for dead load = 150 lb/ft^3

Superimposed dead load = 20 lb/ ft^2

2.5.2. Live Load

Gravity loads acting when the structure is in service, but varying in magnitude or location, are termed live loads. They may be fully or partially or not present at all, and may also change in location. Live loads are defined as gravity loads that do not accelerate at the same as the structural frame when the structure undergoes earthquake motion. The term live load of the bridge means a load that moves along the length of the span. The live load consists of truck loading, lane loading and sidewalk loading. In this model, the impact factor is 26.

2.5.3. Thermal Effect

The effective temperature governs the overall longitudinal movement of the bridge's superstructure. Change in effective bridge temperature causes the deck to expand and contract. Temperature variations causes repeated cycles of expansion and contraction over time and control the extreme displacements of the integral abutment bridges. Provision shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be figured from an assumed temperature at the time of erection. It is the most important effect governing the design of the integral abutment bridges.

Temperature variations are used for the rise and fall of the temperature. In this model, the referenced temperature is used 86° F. The rise range of temperature is 86° F to 113° F and the fall range of temperature is 86° F to 59° F.

2.5.2. Pressure (Lateral earth pressure)

Pressures due to earth or water are also considered permanent loads. When these loads primarily affect substructure elements, they have the potential of impacting superstructure as well at points of the abutment backwall.

For normally consolidated soil,

$$k_0 = 1 - \text{sine}\,\phi$$

For lateral earth pressure,

$$P = k_0 \gamma z$$

where, k_0 = Coefficient of at rest earth pressure

ϕ = Angle of soil friction

γ = Unit weight of soil

z = Depth of soil

p = Lateral earth pressure

2.5.4. Seismic Loads

The integral abutment bridge's design depends on the connection of the superstructure to the piles and the abutment for the transfer of horizontal forces. This connections design is to transform such forces without damage to the piles or the soil behind the abutment. Seismic loads effect can be significantly more than the thermal movements in regions of high seismicity. So, seismic loads will be transferred to the foundations through abutments and piers.

An earthquake consists of horizontal and vertical ground motions, with the vertical motion usually having the much smaller magnitude because the horizontal motion of the ground causes the most significant effect, it is that effect which is often thought of as earthquake load. In regions where earthquakes of significant intensity may occur, provision shall be made to accommodate lateral forces from these earthquakes. The structures shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site.

The magnitude of earthquake loads the response of the bridge to the ground motion. In this study, the UBC Method of analysis is used for earthquake analysis.

The following are used as data for UBC method of analysis to match local condition.

Zone factor	=0.15g
where, g	= gravity constant
Important factor	= 1
Soil profile type	= S_d
Ecc Ratio	= 0.05
Response modification factor, R	= 8.5

(Intermediated Moment Resisting Frame, IMRF)

4. **Analysis on Seismic Performance of Integral Abutment Bridge**

In current bridge design, it is common practice to include joints and bearings, the main practice to include joints and bearings. The main reason for their popularity is that these structures are simple to design and execute. The use of an integral abutment bridge eliminates the need for deck expansion joints and bearings. The absence of joints ad bearings significantly reduces costs during construction. More significantly, maintenance costs are also reduced since deck joints which allow water to leak onto substructure elements and accelerate deterioration, are totally eliminated. In addition, future widening or bridge replacement becomes easier, since the simple design of the integral abutment bridge lends itself to simple structural modification.

The model is considered to study the seismic performance of integral abutment bridge. A 3-D model for the integral abutment bridge had been developed using SAP 2000. The 3-D model was used to accurately represent the thermal response and seismic effect of the bridge. The analyses are developed for dead load, moving load, and temperature effect and seismic.

4.1. **Estimation of Lateral Earth Pressure for Abutment Backfill**

In this calculation, the unit weight of soil γ is assumed as 17 kN/m^3, the coefficient of friction ϕ is taken as 33° and the surcharge load is supposed as 200 psf.

4.2. **Estimation of Subgrade Reaction (Soil Spring) for Piles**

To analyze the structure of the integral abutment bridge with the help of SAP 2000 software, the bearing capacity of soil is assumed as q_u = 814 k/ft by Meyerhof. The subgrade reaction (k_s) along the piles is estimated. The unit weight and coefficient of friction ϕ in each depth is assumed and the results of k_s obtained by using Meyerhof's method are shown in Table 2.

Table 2. Subgrade Reaction (k_s) for Piles

Soil Depth (ft)	Coefficient of Friction (ϕ)degree	Soil Unit weight (kN/m^3)	Subgrade Reaction(k/ft^2)
10	33	17	364.93
15	33	17	672.33
20	35	18	1100.16
25	37	18.5	1707.16
30	39	19	2641.6
35	40	20	3703.39

4.3. **Hypothetical Model of Integral Abutment Bridge**

The hypothetical model includes superstructure, substructure and foundation. The concrete deck slab and the steel girder as superstructure, the abutments as substructure and piles as foundation are used in the hypothetical model structures. The profile of 3-D model for the integral abutment bridge is shown as Fig. 1.

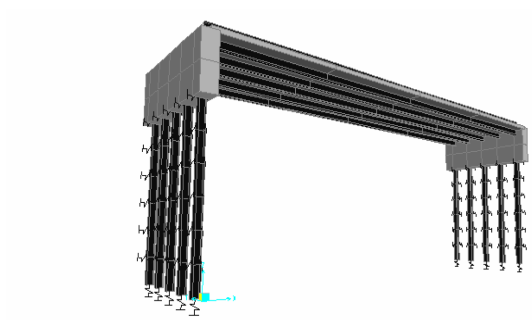

Figure 1. Hypothetical 3-D Model of Integral Abutment Bridge

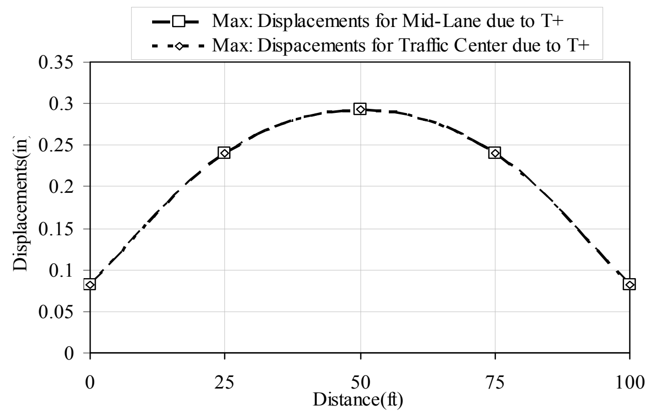

Figure 2. Comparisons of Maximum Vertical Displacements along Bridge's Slab due to Increasing Temperature

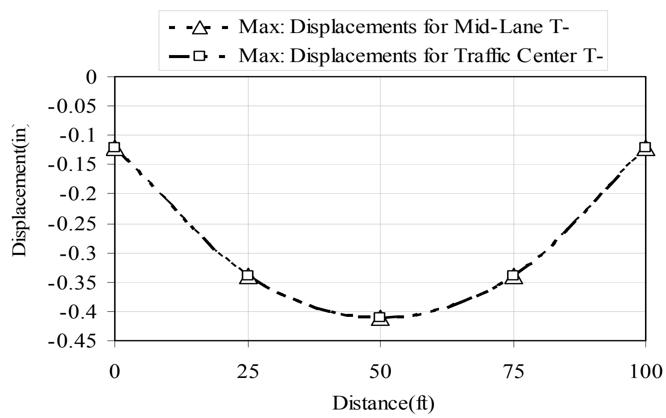

Figure 3. Comparisons of Vertical Displacements along Bridge's Slab due to Decreasing Temperature

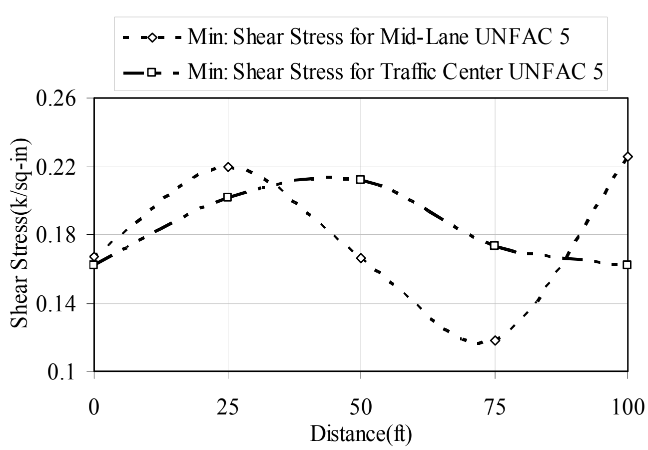

Figure 4. Comparisons of Maximum Shear Stress along Bridge's Slab due to Combination (1.25 DL+ 1.3 EPH+ MOVE+ T_+)

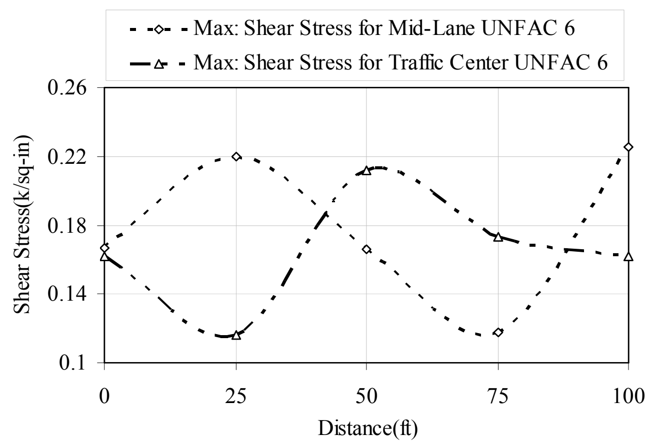

Figure 5. Comparisons of Maximum Shear Stress along Bridge's Slab due to Combination (1.25 DL+ 1.3 EPH+ MOVE+ T-)

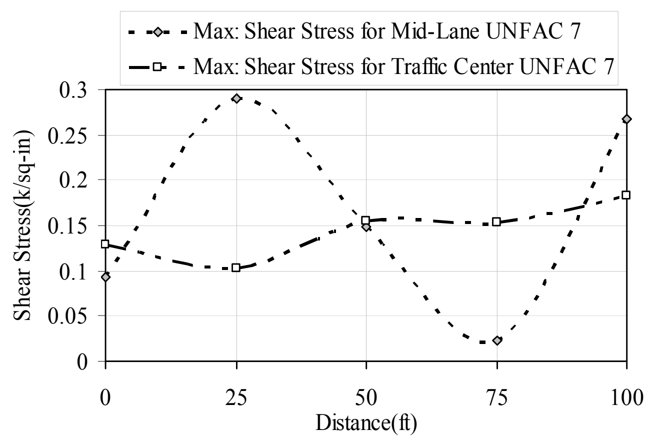

Figure 6. Comparisons of Maximum Shear Stress along Bridge's Slab due to Combination (1.25 DL+ 1.3 EPH+ EQX)

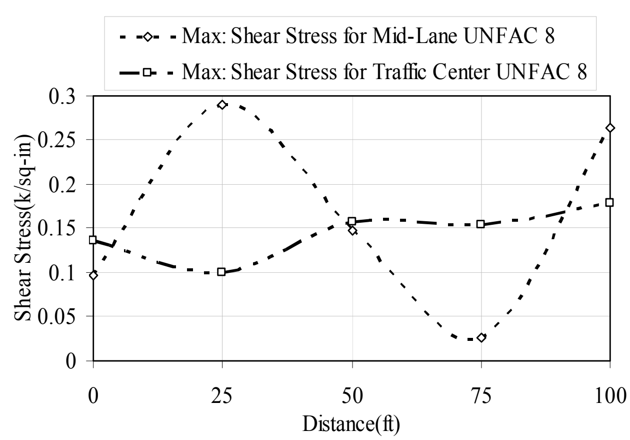

Figure 7. Comparisons of Maximum Shear Stress along Bridge's Slab due to Combination (1.25 DL+ 1.3 EPH+ EQY)

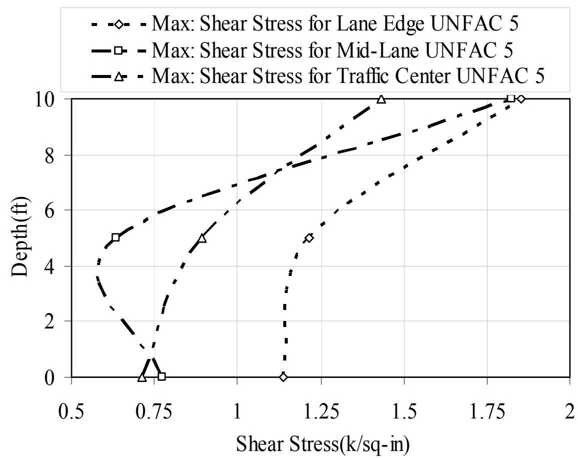

Figure 8. Comparisons of Maximum Shear Stress on Abutment due to Combination (1.25 DL+ 1.3 EPH+ MOVE+ T_+)

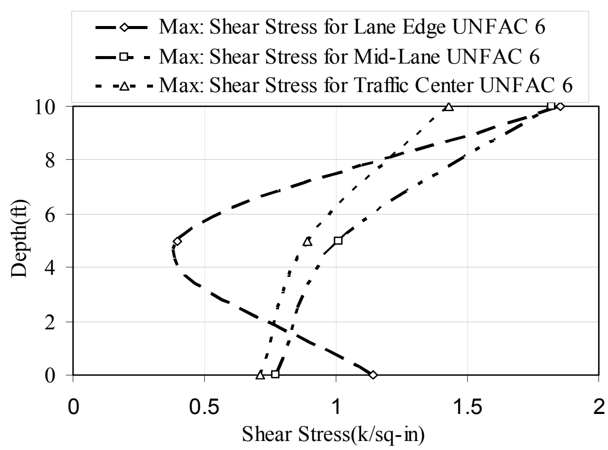

Figure 9. Comparisons of Maximum Shear Stress on Abutment due to Combination (1.25 DL+ 1.3 EPH+ MOVE+ T-)

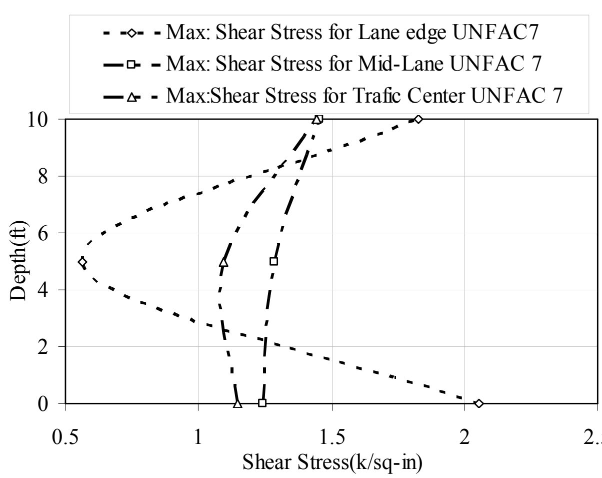

Figure 10. Comparisons of Maximum Shear Stress on Abutment due to

Combination (1.25 DL+ 1.3 EPH+ EQX)

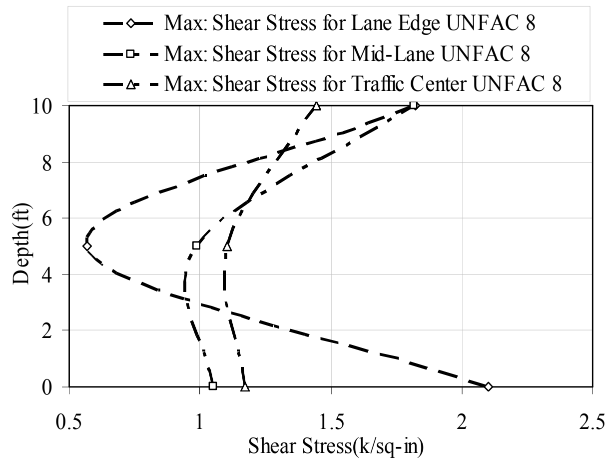

Figure 11. Comparisons of Maximum Shear Stress on Abutment due to Combination (1.25 DL+ 1.3 EPH+ EQY)

5. **Discussion**

The main aim of this study is to predict and analyze the performance of integral abutment bridge under temperature variation and seismic behaviour by using available design method and SAP 2000 structural analysis software. To analyze the combined structure; temperature, earth pressure, moving load and seismic load, the thermal variations, and seismic effects are considered. Elastic behaviour of soil under lateral load can simply be represented by lateral spring. Therefore, the developed analysis model is a vertical beam-column laterally supported by a series of springs located along the length of pile. The stiffness of the springs which represents the lateral soil restraint is estimated by using the approximation method is based upon the ultimate bearing capacity and allowable settlement. The structural design of integral abutment bridge is checked by using SAP 2000 software.

In this study, the allowable bearing capacity is calculated by using the method modified from the general bearing equations. The safety factor is taken as 3 to determine the allowable bearing capacity. The bearing capacity of soil is estimated by using Meyerhof’s method. Dead load, moving live load, surcharge load, lateral earth pressure, thermal effect and seismic effect are applied in structural analysis and sixteen load combinations are used in structural design. The analysis points are considered at mid-lane, traffic centre and abutment’s edge.

The comparisons of displacements along the bridge’s slab and the abutment are analyzed by the increasing and decreasing temperature, the longitudinal and transverse earthquake effects, and the combinations UNFAC 5, UNFAC 6, UNFAC7 and UNFAC 8 which have the increasing and decreasing temperature, moving load, earth pressure, longitudinal and transverse earthquake effects. In the comparisons of displacements along the bridge's slab, temperature variations are critical and vertical displacements are controlled due to temperature variations. In the comparisons of displacements on the abutment, longitudinal displacements are controlled in temperature variations. The longitudinal rotations of temperature variations are also controlled.

The comparison of shear stress along the bridge's slab and on the abutment is considered by combinations UNFAC 5, UNFAC 6, UNFAC 7, and UNFAC 8. The maximum and minimum shear stresses are critical in which considered earthquake effects. These shear stresses on abutment are more than along the bridge's slab.

6. **Conclusion**

1. In this study, the displacements between along the bridge's slab and on the abutment are nearly the same.
2. In the analysis results, the vertical displacements are controlled due to the temperature variations.
3. The maximum shear stress of mid-lane on the bridge's slab is more than on traffic centre in the considered combinations and the minimum shear stress is nearly the same.
4. In both maximum and minimum shear stresses on the abutments, the lane's edge are critical at the mid-depth of them.
5. Both the bridge's slab and the abutments due to seismic combinations are applied more variables.
6. Therefore, in the displacements, temperature variations are controlled and In the shear stresses, seismic effect is critical.

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