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Study of Failure of Pile Foundation Due To Earthquake & Its Remedial Measures

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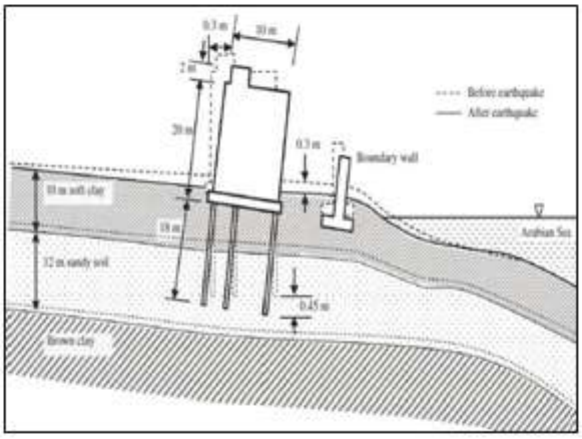
ABSTRACT :*The collapse of pile foundations has been observed in the majority of recent strong earthquakes. This paper reviews the current understanding of pile failure mechanism, its causes explained by many investigators. A hypothetical model of six floor building is analyzed using STAAD-PRO Software twice. Firstly building is analyzed considering no earthquake, & then it was analyzed considering earthquake effects for various earthquake parameters. From both cases reactions at column bases was obtained from STAAD result. Then two hypothetical cases of foundation soil condition were taken. First soil condition was that soil is Clayey soil (Black Cotton soil) up to greater depth, second soil condition was that soil strata comprises of 6m thick clay layer over a deep liquefiable sandy soil layer. For each soil condition & loading coming on column bases RCC design of bored cast in situ pile has been done as per IS 456 : 2000, IS 2911 (Part 1/ Sec 2) : 2010, IS 13920 : 1993 & IS 1893 (Part 1) : 2002.Ultimate vertical load bearing capacity of pile analysis is done by static analysis based on c-ϕ values in which bearing capacity factors suggested by IS 2911, Hansen & Terazaghi has been used. Conventional analysis of a single pile or pile group without considering the mat foundation along with piles result in severe tilting or settlement of the structure eventually leading to complete collapse of structure. It has been concluded that the foundation mat over the non- liquefied crust shares a considerable amount of load of the super structure & hence resist the complete collapse of structure.*
KEYWORDS*- STAAD- PRO Analysis, Pile Foundation, Earthquake Effect, RCC seismic design of Pile foundation,. Liquefaction.*

I. INTRODUCTION

Pile foundations are commonly used to transfer axial loads from a superstructure to the ground in cases where: (a) the structural loads are very high; (b) where the surface soil or soils at shallow depths cannot carry the imposed loads. Also, piles are used to support structures in areas of seismic risk especially where the soils can liquefy due to the seismic shaking. Following a moderate to strong earthquake in liquefiable areas, it has been observed that piled foundation suffer tilting along with settlement. Figure 1(a & b) shows such a case, In which the piles supporting the building was founded on liquefiable soil layer.

(a)



(b)

Figure 1 (a) Tilting of Customs Tower House in the 2001 Bhuj EQ; (b) Schematic diagram of failure of 1(a).

This paper investigates some aspects of failure of piles during earthquake & gives idea about how RCC design of pile can be done. If liquefiable soil layer lies in deep strata, how design of pile can be done in such situations & what are general problems has been assessed. Use of code of practice IS 2911 (part 1/sec 2) for analysis & design of bored cast in situ piles gives better understanding.

When the soil at or near the ground surface is not capable of supporting a structure, deep foundations are required to transfer the load to deeper strata. The most common types of the foundations are piles, piers & caissons. A deep foundation is generally much more expansive than a shallow foundation. It should be adopted only when shallow foundation is not feasible. Concrete piles are generally used for major construction work. Piles are generally classified based on method of installation as follows- 1) Driven piles & 2) Bored & cast in situ piles. Piles transfer the load in to soil in two ways. Firstly, through the tip in compression, termed as “end bearing” or “point bearing”; secondly, by shear along the surface termed as “skin friction”.

In most of the seismic design codes, pile foundations are designed solely against the inertial force. Analyzing pile foundations for seismic loading considers the inertial load that develops from pile and soil energy, also considering the interpretation of kinematic interactions which develops from the shaking of the surrounding soil and the pile. Corresponding soil-pile interaction also accounts the rigidness deterioration that develops due to seismic loading. The reckoning of kinematic curving that develops due to the sideward movements and displacements that are established on the pile due to ground movement and the inertial forces acts on the cap mass. Due to the effects of earth pressures on the foundation and pile integrated in, pseudo static analysis is carried out to evaluate the maximum moment distribution in pile. The earthquake response of pile foundations is quite a complicated process which involves inertial interaction between pile foundation and structure, kinematic interaction amidst soil and pile, induced seismicity of pore-water pressures (PWP) and the varying reaction of soils to dynamic seismic vibrations. The upper structure collaborates with its foundation and the soil surrounding it, generating extra soil deformities, which sums up to those developed from the movement of seismic waves, so as the movement in the proximity of the foundation can be different extensively from that of the free-field.

Generally the pile foundation is analyzed for earthquake loads considering the superstructure as a lumped mass. Nevertheless, it is imperative to predict the response of pile foundation under seismic loads considering the effect of superstructure flexibility. There are many parameters affecting the dynamic response of structures, such as; the type of structure, type of foundation, soil characteristics etc. Observations from the earthquake damaged sites show that the local soil properties and the foundation geometry have great influence on the dynamic behavior of the structures. The local soil conditions and the interaction between soil and foundation affect the dynamic behavior of a structure. Structures always interact with its surrounding soil and respond quite differently depending upon its own properties and that of the supporting soil. However, the seismic analysis of structures is often based on the assumption that the foundation soil is a rigid block.

The major problem concerning the seismic resistant design of pile foundations is the presence of liquefiable soils in the foundation region. Liquefiable soil layers alter the pile capacity and also can cause large lateral loads on pile foundations. Piles driven through a weak, potentially liquefiable, soil layer to a stronger layer not only have to carry vertical loads from the superstructure, but must also be able to resist horizontal loads and bending moments induced by lateral movements if the weak layer liquefies. Thus, it is very essential to investigate the liquefaction susceptibility of sub surface soil layers before proceed for the seismic design. Semi empirical method recommended by Idriss and Boulanger (2004) is being followed to evaluating the liquefaction potential.

Many observations of pile failure in past earthquakes showed that the buildings in liquefiable soils on sloping ground tilts towards the ground slope. These failures are believed to be mainly bending induced by the lateral spreading of liquefied soil. The qualitative surface investigation supports the above hypothesis. However, looking at the foundation failure pattern, this hypothesis cannot adequately explain some cases of pile failure observed. Some recent researches also signify the importance of buckling failure as a major failure mechanism for piles liquefiable soils. Dash (2010) summarizes the probable mechanisms of pile failure in liquefiable soils as bending, buckling, shear, settlement or dynamic amplification, and the piles may fail either due to any of the mechanisms or a combination of some or all of them. Similar kind of surface observation might be possible for different pile failure types. For example, bending, buckling, shear failure of the piles and uneven settlement of the pile group may lead to tilting of the superstructure. All the three buildings in liquefied soils studied in this paper titled after the earthquake, however the governing mechanism of their pile failure are very different.

II. METHODOLOGY

A six floor RCC frame building, with brick infill panels at all beams are taken as superstructure model. Fig. 2(a) & 2(b) shows the plan & STAAD Model of the building, Following are the dimensions of building:

No. of floors- 6, Height of each floor- 3.4 m, Ht. of building above ground- 6x3.4= 20.4m,

Size of Columns: $C_{2,3,5,6,8,9,11,12}$ -0.45m X 0.45m, $C_{1,4,7,10}$- 0.25m X 0.25m, Size of beams- 0.25m X 0.45m,

Thickness of Wall : At each floor- 0.25m, Parapet at roof- 0.15m, Thickness of Roof/ Slabs- 0.15m,

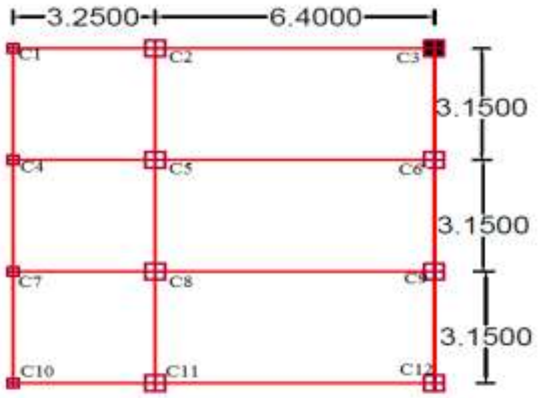


Fig 2 (a) Plan of building

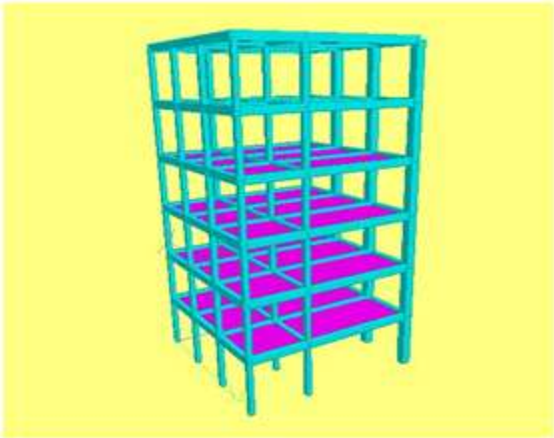


Fig 2(b) STAAD Model of building Frame

III. ANALYSIS OF STRUCTURE:

3.1 For Static STAAD Analysis of the building Following loads is given as input :

1. Dead load : Self weight of beams + columns + Floors (Auto generated in STAAD Pro
UDL due to Wall at floors- 16.68 KN/m, Parapet Wall- 2.85 KN/m

2. Live Load: On Floors- 3KN/m^2 , On Roof- 5KN/m^2

3. Types of supports: Fix support at each column base.

Out of various load combinations As per IS 875, It was found that 1.5x (DL+LL) gives Maximum support reactions at column Base, Which is represented in following table. These reactions are used to design Piles.

Column no.	C_1	C_2	C_3	C_4	C_5	C_6	C_7	C_8	C_9	C_{10}	C_{11}	C_{12}
Axial Force, F_y (KN)	767.7	1366.01	1232	1137.2	1826.9	1673.14	1137.2	1826.9	1673.1	767.7	1366	1232
Moment, $M_{x,z}$ (KNm)	9.4	12.6	16.6	9.8	12.8	12.8	9.8	12.8	12.9	9.4	12.6	16.6

3.2 For seismic Analysis of building Following inputs were given in the STAAD:

1. Dead load: Self weight of beams + columns + Floors (Auto generated in STAAD Pro
   UDL due to Wall at floors- 16.68 KN/m, Parapet Wall- 2.85 KN/m
2. Live load: On floors- 25% of LL of actual Load = 0.75 KN/m^2 , On roof- 0 KN/m^2
3. Types of supports: Fix support at each column base.
4. Soil type : Black cotton soil (SPT Value ϵ (10-15)) hence "soft soil"
5. Type of building : OMRF, All general building, 6. Location of building – Zone –V
7. Damping Ratio- 0**,** 8. For brick Infill, time period $T_{a,X} = 0.09h/\sqrt{d} = 0.591$ sec, $T_{a,Y} = 0.597$ sec.

Out of different load combinations as per IS 1893: 2002, those combinations were considered which gives maximum reaction at column bases. The maximum reactions have been tabulated in the following table.

Column no.	C_1	C_2	C_3	C_4	C_5	C_6	C_7	C_8	C_9	C_{10}	C_{11}	C_{12}
SF.(F_x or F_z) (KN)	48.9	287.5	225	52.65	340.22	247.4	49.6	340.13	247.4	98.92	287.6	225.0
Axial force = F_y (KN)	1304.4	3116.6	2276.4	1639.4	2348.9	2185.4	1639.4	2348.9	2185.4	1304.5	3116.7	2276.4
Moment , M_x (KNm)	83.98	582.7	13.7	90.4	664.2	0	90.14	662.2	472.9	83.99	8.4	13.7
M_z (KNm)	12.3	86.8	544.7	4.0	22.9	570.1	6.4	60.9	43.7	12.25	626.4	544.6

3.3 Analysis of load carrying capacity of Piles in different soil conditions:

3.3.1 Soil type -1 {Black Cotton Soil (clay) is extended beyond the pile depth}:

Soil properties are: Gravel- 2%, Sand- 25%, Fines- 73%, Cohesion C= 75 kn/m^2 & Ultimate load capacity of Pile is calculated by the formula given in IS 2911 (part 1/ sec 2) : 2010 as follows-

$Q_u = CN_C A_P + \boldsymbol{\alpha} CA_S$ …... [eqn. 1]

N_C= Bearing Capacity Factor. For Depth/Dia. ratio > 5, N_C= 9.0, A_p = bearing area of pile = Π/4 x (pile dia.)^2= Π/4x0.6^2, **α**= Adhesion factor =0.6, From α V_S C graph IS 2911 (part 1/ sec2), A_S = surface area of pile = Π x Dia. x Height= Πx0.6x9

Diameter of pile = 0.6m, Length of Pile = 9m

We get capacity of single pile= 827 KN, Two Piles = 1654 KN, Three piles = 2481 KN

No. of piles required in this type of soil for static & seismic analysis has been tabulated in the following table.

Column No.		C_1	C_2	C_3	C_4	C_5	C_6	C_7	C_8	C_9	C_{10}	C_{11}	C_{12}
No. of piles	Static Analysis	1	3	2	2	3	2	2	3	2	1	3	2
	Seismic Analysis	2	4	3	2	3	3	2	3	3	2	4	3

3.3.2 Design of different piles in soil type-1 (see 3.3.1) for two types of loading (static loading & seismic loading):

RCC Design of piles is done as per IS 2911 (part 1/ sec2), IS 456: 2000 to resist Axial loads, Shear Force, & moments given in the tables under section 3.2.1 & 3.2.2 (Uniaxial moment in case of static loading & biaxial moment in seismic loading).

For non-seismic loading, all piles will be designed same & Provide 6# of 16mm ϕ as main reinforcement & provide 12 mm ϕ bar @ 100 mm pitch in the form of spiral. Along with this Stiffener rings made up of 16mm ϕ are provided to prevent inward buckling of r/f mesh @ 1.5 m c/c.

For seismic loading, two types of piles are designed. In piles designed for Column no. C_1, C_4, C_7, C_{10}, we have provided 6# of 16 mm ϕ bars as main reinforcement & 12 mm ϕ bar @ 100 mm pitch in the form of spiral. Along with this Stiffener rings made up of 16mm ϕ are provided to prevent inward buckling of r/f mesh @ 1.5 m c/c. In Piles designed for column no. C_2 C_3 C_5 C_6 C_8 C_9 C_{11} C_{12}, we have provided 9# of 25 mm ϕ bars as main reinforcement & 12 mm ϕ bar @ 100 mm pitch in the form of spiral. Along with this Stiffener rings made up of 16mm ϕ are provided to prevent inward buckling of r/f mesh @ 1.5 m c/c.

3.3.3 Soil type- 2 {Black cotton soil (clay) over liquefiable sandy soil layer}

Without considering seismic effect:

Thickness of clay soil layer is 6m deep from the ground level. Properties of soil of this layer are assumed same as that of soil type-1 in section 3.3.1. Next deeper soil layer is saturated sandy due to presence of ground water table. Properties of this soil layer are : $\boldsymbol{\gamma_{sat}}$ = 17 KN/m^2 , Angle of shearing resistance, **ϕ=** 33°. Diameter of pile= 0.6m & length of pile = 9m. Each pile passes through 6m upper clayey soil layer & 3m lower sandy soil layer & rest in sandy strata. Ultimate load bearing capacity of each pile is the sum of shearing resistance from clayey & sandy soil & end bearing resistance from sandy soil. Hence ultimate load carrying capacity of pile will be determined by the formula suggested by IS 2911 (part 1/ sec 2) as follows:

$Q_u = \{\boldsymbol{\alpha} CA_{S1}\} + 0.5A_P\{q_1.N\mathbf{q} + 0.5\,\gamma DN_\gamma\} + \{q_2.K.\tan\square\,.A_{S2}\}$ …..[eqn 2]

Here **α**, C, A_S, A_P holds the same meaning as in [eqn 1]. A_{s1}=Πx0.6x6, A_{S2}= Πx0.6x3, A_P= Π/4x0.6^2 , γ= 17 kn/m^2, D= dia of pile= 0.6m, K= earth pressure coefficient= 1-Sinϕ= 1-Sin33°, N_γ = bearing capacity factor =31.65 hansen's chart, N_q= bearing capacity factor=35 for for ϕ=33° according to IS 2911. $\mathbf{q_1}$ = overburden soil pressure at pile tip= $\boldsymbol{\gamma_{clay}}$ x depth of clay + $\boldsymbol{\gamma_{sand}}$ x depth of sand =18.8x6+17x3, this value is less than overburden pressure at critical depth (15 to 20 times pile dia). $\mathbf{Q_2}$ = overburden soil pressure at pile 7.5 m depth=18.8x6+17x1.5, this value is also less than overburden pressure at critical depth (15 to 20 times pile dia).

Also 2nd term of the equation 2 is multiplied with 0.5 to compensate with looseness in soil at pile end during excavation of bore hole.

We get capacity of single pile = 1508.1 KN, No. of piles required at each column for static loading reactions (section 3.2.1) is shown in table below

Column No.		C_1	C_2	C_3	C_4	C_5	C_6	C_7	C_8	C_9	C_{10}	C_{11}	C_{12}
No. of piles	Static Analysis	1	2	2	2	3	2	2	3	2	1	2	2

We see that design capacity of RCC piles provided to resist reactions for non-seismic loading (in 3.3.2) is sufficient to take reactions coming in above case also, so same design of piles will be followed here also.

IV. CONSIDERING SEISMIC EFFECT ON SOIL:

Considering seismic effect, it is assumed that sandy soil get liquefy & now gives no support to pile. Hence in this case ultimate load capacity of pile is calculated by following formula, assuming only support from clay layer.

$Q_u = \alpha C A_{S1}$ ……….. [eqn 3]

Here we get load carrying capacity of single pile= 509KN. No. of piles required at each column for seismic loading reactions (section 3.2.2) is shown in following table.

We see that no. of piles required at few columns are as much high up to 8, that it is practically impossible to place that much no piles due to very limited space & economy point of view also. Increasing depth of pile will lead to very much cost as compared to importance of building, so it is not also a good option.

So we provide under reams of diameter equal to two times the diameter of pile. Load carrying capacity of under reamed pile is determined by:

$Q_u = CN_C A_b + \alpha C A_S$ ………[eqn 4]

$A_b = \Pi/4 \times (1.2^2 - 0.6^2)$, $A_s = \Pi x 0.6 x 6$, C, N_c, α are same as used in earlier equations. Here we get load carrying capacity of each piles = 1081.35 kn. The no. of piles required at each column is also shown in following table.

Column No.		C_1	C_2	C_3	C_4	C_5	C_6	C_7	C_8	C_9	C_{10}	C_{11}	C_{12}
No. of piles after Seismic Analysis	Normal piles	5	11	8	6	8	8	6	8	8	5	11	8
	Under reamed piles	2	5	4	3	4	4	3	4	4	2	5	4

Here we see that still no of piles at few columns are as much high up to 5, that there is will be practically congestion in providing piles in ground. So another alternative which was suggested by many investigators in their research papers is providing a common mat Foundation along with piles.

V. CONCLUSION

A) Load carrying capacity of Piles due to presence of sandy layer estimates higher than that of presence of clayey soil throughout depth in static condition. But we see tremendous loss of load carrying capacity of piles in same stratified two layered soil, in seismic condition due to lack of support from liquefied sandy soil layer. This loss of support will result in excessive settlement (may be differential settlement also) if pile is designed considering support from liquefiable soil in seismic condition.

B) Piles passing through a deep non-liquefied crust and resting on liquefiable soil can suffer excessive settlement and tilting rendering it unusable or expensive to rehabilitate following the earthquake. This should be avoided in practice.

C) Use of a large foundation mat or a large pile cap has a number of advantages such as: (a) it reduces the risk of sudden and/or catastrophic collapse and it is difficult for the large raft to punch through the soil even if the top soil is liquefied and (b) it reduces the settlement of the foundation. During liquefaction, the piles will loose the shaft resistance in the liquefied zone which will lead to settlement of the supporting structure. However, due to the integrity of the pile foundation mat system, a part of the superstructure load will be transferred through the foundation mat and will reduce the possibility of the structure to sink into the soil.

D) Use of under reamed pile foundation instead of plain piles proves more stability in case of non-liquefiable soil layer of a considerable depth lies over liquefaction zone.

REFERENCES

[1]. Suresh R. Das, L. Govindaraju, Shubhamoy Bhattacharya, A case study of damages of the Kanla Port & Custom office tower supported on a mat – Pile Foundation in liquefied soils under the 2001 Bhuj earthquake. www.elsevier.com/locate/soildyn. Soil Dynamics and Earthquake Engineering 29 (2009) 333– 346.

[2]. V. Suneetha & Dr. D.S.V. Prasad, Design of Pile Foundation in Black Cotton Soil, IJIRST-International Journal for Innovative Research in Science & Technology| Volume 4 | Issue 1 | June 2017 ISSN (online): 2349-6010

[3]. Kanakeswararao Thadapaneni, Sarikonda Venkata sivaraju, Ravi teja Grandhi, Analysis of pile foundation- Simplified methods to analyse the pile foundation under lateral and vertical loads, 2017 IJEDR | Volume 5, Issue 3 | ISSN: 2321-9939

[4]. A. Murali Krishna, A. Phani Teja, S. Bhattacharya, Barnali Ghosh, Seismic Design of Pile Foundations for Different Ground Conditions, 15 WCEE LISBOA 2012.

[5]. K. R. Arora, Soil Mechanics & Foundation Engineering, 7th edition

[6]. Dr. BC Punmia, Ashok kr. Jain, Arun Kr. Jain., Soil Mechanics & Foundations, 16th edition.

[7]. IS 2911 (Part 1/Section 2): 2010, IS 456 2000, IS 1893 (Part 1): 2002, IS 13920 : 1993