Abstract— Numerous damages have been evidenced during Earthquakes (EQ) due to incomprehension of soil strata and structure interaction effects in the design of structure and foundation system. These damages are due to several reasons of changes in soil properties. Among these, Liquefaction of soils is more hazardous especially in Earthquake (EQ) prone zones. This paper deals with the Seismic analysis of RCC Frame structure founded on soil subjected to liquefaction and compares the behaviour of the superstructure and pile head deflection before and after Liquefaction. 20% of external wall area are provided with shear walls and analyzed. Seismic analysis is done by Response Spectrum method using STAAD software. Pile foundation which are usually suggested for Liquefied soil is also modelled and analyzed. Available field data of earthquake and liquefied prone zone near Muzaffarnagar, Uttar Pradesh, were collected and soil properties were extracted. Based on the stratigraphy and general trend of Standard Penetration Test (SPT) values, the pile capacities were calculated. The performance of Pile foundation in Liquefied earth and Non-liquefied earth were studied depending on various superimposed loads due to Dead load, live load, wind forces and seismic forces. Various comparisons and alternative studies on the performance of Super-structure for different pile capacities are compared and contemplated.

Keywords—Lateral load resisting frames, Liquefaction, Pile capacities, STAAD

I. INTRODUCTION

The performance of any superstructure depends on the type of substructure and nature of soil strata lying beneath. In case of hard or medium soil, the deflection and drift in beams and columns depends wholly on the superimposed loads. This can be controlled by addition of various types of lateral resisting systems. But this is not case of buildings resting over liquefied strata, here behaviour of the structure purely depends on the response of soil against external loadings. Piles are usually recommended for weak and loose soil strata and the drift in columns is directly proportional to deflection in pile head which in turn depends on the magnitude of lateral forces. The analysis of pile foundation under axial and lateral loading depends upon the movement of piles which in turn is dependent on reaction of soil. Under various lateral forces like wind and seismic forces horizontal pressures are developed against the piles which effect the overall behaviour of the superstructure. These results in increased drift of the frame structure and the structure may sink or move laterally without causing any major structural failure. The resulting drift in columns can be controlled by introducing shear walls in the frame structure but the deflection in the pile cap head cannot be reduced as the addition of shear walls in the frames may induce high shear stresses on the pile cap.

II. RELATED WORK

Shubamoy Bhattacharya, (2016) [1] This paper investigates the changes in modal parameters like damping and frequencies of pile supported structures during soil liquefaction. Experiment was carried using shaking table by means of 4 material models, consisting of 2 single piles and a group of 2 piles where the soil surrounding the pile experienced liquefaction due to EQ shaking. The experimental outcome showed that the natural frequency of pile-supported structures may reduce due
to thrashing of lateral/horizontal support presented by the soil to the pile and the damping ratio may amplify to about 20%.

B. Ganesh (IJETT), (2017) [2] In this paper a group of 9 piles united to a common pile cap was analyzed using L-pile and STAAD software. Analysis using software’s such as L-piles and STAAD gives more conservative outputs but has an advantage over the manual analysis. Soil-pile behaviour such as pile bending moment, soil reactions, p-y curves can be studied within short time with less effort. The STAAD software which is generally applicable for structural analysis and design can be used to analyze pile head deflections. Its dependence on soil spring constants and ultimately on the correctness in the inference of elastic modulus of soil which depends on soil properties and field data shall be well understood before analyzing in software.

Shin-Tower Wang, (2018) [3] This paper investigates the behaviour of pile groups in deposits of Liquefied soil which is moving laterally, but contains little information on quantifying the magnitude of lateral spreading against piles in a group. The analysis of a pile foundation under vertical and lateral loading is intricate by the reality that the soil reaction is reliant on the pile movement, and the pile movement, on the other hand, is dependent on the soil reaction. This is known as soil structure interaction. The problem is nonlinear because soil response is a nonlinear function of pile displacement.

Pallavi Badry and Neelam Satyam, (2014) [4] This paper investigates various methods in estimating the interaction methods in estimating the interaction effects between piles and the supporting soil strata during EQ. These are analyzed by direct and substructure method. During EQ analysis, the total interaction response is the summation of kinematic and inertial interaction. Kinematic interaction refers to the distortion of soil mass due to vibration when seismic waves pass through the soil. Once the excitation waves passes through the strata, the structure experiences vibrations which exerts additional dynamic force due to soil mass and this is known as inertia interaction which depends on the inertia force produced by the structure.

S Adhikari, (2015) [5] This paper describes the dynamic volatility of the structure on liquefied soil during EQ with three different cases of before and after soil liquefaction and the transient phase between these two conditions. The analysis was done via Model tests and analytical works. The experiments shows that the overall frequencies of the building starts reducing with onset of soil liquefaction and completely decreases when soil is fully liquefied.

Neelima Satyam, (2014) [6] In this paper various insitu methods of testing are carried out to assess the magnitude of liquefaction. The tests proved that the structural failure was minimum but settled and tilted as rigid bodies due to the considerable loss of bearing capacity.

Zaheer Ahmed Almani, (2012) [7] In this paper, the settlement in buildings due to liquefaction are analyzed using various numerical modelling. The system was modelled using 2D dynamic modeling code referred as FLAC (Fast Lagrangian Analysis of continua). A comparison study was made with shallow foundation resting on untreated ground surface and on ground treated with jet grouted columns.
This paper investigates the performance of building over liquefied ground in Adapazari, during Kocaeli EQ. The literature explains different probable mechanism of foundation failure. It also correlates the foundation settlement with the vertical loads and width of foundation.

III. LIQUEFACTION

Soil liquefaction generally occurs in loose, saturated, cohesionless soil and this is triggered during earthquake. The shear strength between soil particles is reduced and the pore water pressure increases which make the soil to behave as liquid. This phenomenon is known as soil liquefaction. Liquefaction is reliant on degree of saturation, distribution of grain size, virtual density of the soil, intensity and duration of EQ. Its effects are most commonly observed in low lying areas near water bodies such as river, lakes, oceans etc, as the soils in such areas will be completely saturated. The potential effects include foundation bearing failure, excessive settlement, differential settlement etc.

IV. SOIL - PILE INTERACTION

The performance of the pile foundation primarily depends on the interaction between the surrounding soil and the pile. Based on many historical studies the soil behaves as a series of elastic springs connected to pile which undergoes constant compression and tension depending on the loading condition. Hence the deflection of the piles takes the same form against the elastic springs. This is known as soil-pile interaction effects and these factors are expressed in terms of sub-grade reactions which depend on pile deflection (y) and a sub-grade co-efficient (kh), P = kh x y. For sands and normally consolidated clays under long term loading on piles, horizontal subgrade reaction is expressed as \( kh = nh \times \frac{z}{d} \). Where \( nh \) = Co-efficient of subgrade reaction, \( z \) = depth from ground surface and \( d \) = Pile diameter.

V. PILE CAPACITY

The ultimate bearing capacity of piles depends either on the friction offered by the surrounding soil or resistance offered at pile tip or both. Tip resistance is achieved if hard strata are encountered at the point of pile termination otherwise the pile capacity will be purely due to soil friction. Based on these factors the piles are classified as point bearing, friction and compaction piles. The lengths of friction piles depend on the shear strength of the soil, the applied load, and the pile size. Under certain conditions, piles are driven in granular soils to attain suitable compaction of soil secure to the ground surface. These piles are called compaction piles. The lengths of compaction piles depend on factors such as the virtual density of the soil before and after compaction and the required depth of compaction. The capacity of individual pile must be checked against axial load, lateral load from externally applied forces like dead loads, live loads, wind loads and seismic loads. The pull out capacity of piles which is also referred as tensile capacity should also be assessed. The axial capacity of piles is the sum of friction capacity (Qs) and point bearing capacity (Qp), \( Q = Qs + Qp \). If Qp is zero then \( Q = Qs \). These capacities can be determined either experimentally or analytically. Many numerical methods like Meyerhof’s method, Vesic’s method, Coyle & Castello method, Briaud’s method can be used to assess the friction and point bearing capacity of an individual pile. Lateral capacity of pile can be calculated as mentioned in IS 2911 depending on whether the pile head is free or fixed. The tensile capacity of individual pile is always the sum of self weight and the friction capacity of pile. The pile capacities depends on various factors like bearing capacity factors, atmospheric pressure, angle of internal friction of soil, perimeter of pile section, length of embedment, general SPT value ‘N’ etc.. The derived pile capacities assist us to decide the number of piles in pile group, Diameter of pile, length of embedment and size of pile cap.
### Equations for Estimating Pile Capacity in sand

#### Point Bearing Capacity (Qp) = qp * Ap

- **Meyerhof’s Method**
  
  \[ Q_p = A_p \cdot q' \cdot N_q \leq A_p \cdot (0.5 \cdot p_a \cdot N_q \cdot \tan \theta) \]

  - \( A_p = \text{Area of Pile tip} \)
  - \( q' = \gamma \cdot L = \text{Effective vertical stress at pile tip} \)
  - \( \gamma = \text{Unit weight of Sand in KN/m}^3 \)
  - \( L = \text{Recommended depth of pile} \)
  - \( N_q = \text{Bearing capacity factor} \)
  - \( p_a = \text{Atmospheric pressure} = 100\text{KN/m}^2 \)
  - \( \theta = \text{Angle of Internal friction} \)

- **Vesic’s Method**
  
  \[ Q_p = A_p \cdot \sigma_o \cdot N_{\sigma} \]

  - \( \sigma_o = \frac{(1+2(1-\sin \theta))}{3} \cdot q' \)
  - \( N_{\sigma} = \text{Bearing capacity factor} \)

- **Coyle & Castello Method**
  
  \[ Q_p = A_p \cdot q' \cdot N_q \]

  - \( N_q = \text{Bearing capacity factor depends on Embedment ratio i.e L/D ratio} \)
  - \( L = \text{Recommended depth of pile} \)
  - \( D = \text{Diameter of Pile} \)

#### Friction Capacity of Pile (Qs) = qs * As

- **Meyerhof’s Method**
  
  \[ Q_s = p \cdot L \cdot f_{av} \cdot A_s \]

  - \( p = \text{perimeter of pile section} \)
  - \( L = \text{Recommended depth of pile} \)
  - \( p_a = \text{Atmospheric pressure} = 100\text{KN/m}^2 \)
  - \( N = \text{SPT value corresponding to depth} \)
  - \( A_s = p \cdot L \)

- **Briaud’s Method**
  
  \[ Q_s = p \cdot L \cdot f_{av} \cdot A_s \]

  - \( f_{av} = 0.224 \cdot p_a \cdot (N)^{0.29} \)

#### Lateral Capacity of Pile

- **For Free head Piles**
  
  \[ Q = \frac{(3 \cdot Y \cdot E \cdot I)}{(L_1 + L_f)} \times 3 \]

  - \( Y = \text{Deflection near pile head in cm} \)
  - \( Q = \text{Lateral capacity of Pile in Kg} \)
  - \( E = \text{Youngs modulus of pile material in Kg/cm}^2 \)
  - \( I = \text{Moment of Inertia of the pile cross section in cm}^4 \)
  - \( L_1 & L_f = \text{from fig -2 of IS -2911, part-4} \)

- **For Fixed Head Piles**
  
  \[ Q = \frac{(12 \cdot Y \cdot E \cdot I)}{(L_1 + L_f)} \times 3 \]

  - \( Y = \text{Deflection near pile head in cm} \)
  - \( Q = \text{Lateral capacity of Pile in Kg} \)
  - \( E = \text{Youngs modulus of pile material in Kg/cm}^2 \)
  - \( I = \text{Moment of Inertia of the pile cross section in cm}^4 \)
  - \( L_1 & L_f = \text{from fig -2 of IS -2911, part-4} \)

#### Tension Capacity of Pile = Self weight of pile (W) + Friction Capacity (Qs)

### V. SOIL - PILE INTERACTION

The performance of the pile foundation primarily depends on the interaction between the surrounding soil and the pile. Based on many historical studies the soil behaves as a series of elastic springs connected to pile which undergoes constant compression and tension depending on the loading condition. Hence the deflection of the piles takes the same form against the elastic springs. This is known as soil-pile interaction effects and these factors are expressed in terms of sub-grade reactions which depend on pile deflection (y) and a sub-grade co-efficient (kh).

\[ P = kh \times y \]
For sands and normally consolidated clays under long term loading on piles, horizontal subgrade reaction is expressed as

\[ kh = nh \times \left(\frac{z}{d}\right) \]

Where,

- \( nh \) = Co-efficient of subgrade reaction
- \( z \) = depth of from ground surface
- \( d \) = Pile diameter

As per IS-2911, part-2, table-1, the co-efficient of Horizontal sub-grade reaction varies depending on type and moisture content of soil.

<table>
<thead>
<tr>
<th>Calculation of Spring constant on Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal spring constant (kh) = nh * (z/d)</td>
</tr>
<tr>
<td>Vertical spring constant as frictional resistance (kv) = p * z * fav</td>
</tr>
<tr>
<td>nh = co-efficient subgrade reaction</td>
</tr>
<tr>
<td>z = depth of pile in m</td>
</tr>
<tr>
<td>d = diameter of pile</td>
</tr>
<tr>
<td>p = perimeter of pile section</td>
</tr>
<tr>
<td>fav = 0.224<em>pa</em>(N)^0.29</td>
</tr>
<tr>
<td>pa = atmospheric pressure = 100 KN/m^2</td>
</tr>
<tr>
<td>N = SPT value corresponding to depth</td>
</tr>
</tbody>
</table>

VI. STRUCTURAL MODELING AND ANALYSIS IN STAAD

The type of building considered for analysis is a multi-storey building with Stilt and G+3 floors. The structure is modelled as a Frame structure with beam and columns as member elements. Provision for lateral resisting system is also made in the form of shear walls. Size of building is about 22.0m x 16.0m. Size of columns and beams are taken as 0.35m x 0.35m and the section are also checked for ductile adequacy as per IS-13920. Total height of building is about 17.20m with stilt 3.20m and floor height of 3.50m. Thickness of external and internal wall is taken as 230mm and 115mm respectively. About 20% of external wall area is provided with 230mm thickness shear walls. Geotechnical investigation recommends RCC bored cast-in-situ pile foundation extending well below the liquefied zone and opines that there may be a potential for liquefaction in the sand strata encountered at the site to about 5.0m depth in the event of the design earthquake. Based on these field data and based on computed pile capacity, 4 numbers of piles in a group and 0.50m pile diameter is fixed. Horizontal and vertical spring constants are calculated based on the trend of SPT values and these constants are applied along the length of pile in the form of fixed but supports with minimum value near pile head and increasing to maximum at the pile tip.
Analysis are carried out for different cases as explained, **Case-1**: Seismic analysis of Frame on Pile foundation in zone IV with condition in which pile derives its resistance from the friction offered by the surrounding soil for its full length. This case will arise when hard strata is not met at the point where pile is terminated. **Case-2**: Seismic analysis of Frame on Pile foundation in zone IV with condition in which pile derives its resistance from both the friction offered by the surrounding soil for its full length and also due to tip resistance. This case will arise when hard strata is met at the point where pile is terminated. **Case-3**: Seismic analysis of Frame on Pile foundation in zone IV with condition in which pile derives its resistance from the friction offered by the surrounding soil for its partial length as some depth of soil undergoes complete Liquefaction. **Case-4**: Seismic analysis of Frame on Pile foundation in zone IV with condition in which pile derives its resistance from both the friction offered by the surrounding soil for its partial length and also due to tip resistance.
VIII. ANALYSIS AND RESULTS

Various loads on the superstructure like dead load due to walls and slabs, live loads classified as per IS – 875, part-2, wind load with a basic wind speed of 47 m/sec and the major seismic force with seismic zone co-efficient of 0.24 was applied on the superstructure and after assigning the supports to the piles, the model was analyzed and the results are compared and contemplated in the form of graphs shown below.
The maximum lateral deflection of pile should not exceed 1 to 2% of the pile diameter and the drift in columns should be less than 0.004 times the storey height. From the above graph it can be seen that prior to liquefaction the lateral deflection near the pile head and the drift in columns are within the allowable limits and as the soil liquefies the pile head deflection and the column drift increases and in case of only friction piles these values exceeds the maximum limits. This is due to the loss of resistance of the top layer of soil which is susceptible to liquefaction and thus resulting in high lateral deflection in piles and column drift. This results in the formation of an hinge at the junction of liquefied and non-liquefied soil and the piles may fail due to buckling. In case of friction bearing piles, the structure may experience heavy settlement.

IX. CONCLUSIONS

From the study, it can be concluded that the structure resting on piles essentially in liquefied zones should be analyzed along with substructure to properly assess the behaviour of the entire system. During liquefaction soil resistance decreases considerably resulting in large deflection in columns
and in pile head. Some amount of drift in columns can be reduced by addition of lateral resisting system like braced frame, shear walls etc, but the lateral deflection in pile head can only be controlled by ground improvement methods. Ground improvement techniques includes a) Densified crust of Natural soils, b) Cement stabilized crust, c) Using deep soil mix columns, d) Using Perimeter curtain wall etc. Finally it is of the opinie that, in liquefied prone area especially in seismic zone, construction of multi-storey buildings with deep foundation along with soil stabilization will reduce the risk factor but results in heavy construction cost. As such for ordinary dwellings number of storey’s must be avoided as far as possible except in case of important structures or any other important buildings where cost criteria are significantly ruled out as compared to safety.

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