FOUNDATIONS

INTRODUCTION:

In the following paragraphs the failure of foundation is briefly explained with respect to earthquake. The failures do occur even for gravity forces but due care has to be considered in the planning, conceptual stages so that the foundation can be safely designed.

There are some practical solutions are given for the liquefaction, pad footings, pile foundation and raft when there is a problem due to seismic forces.

Further reading of geotechnical books is recommended to get in depth details.

NOTES ON SOIL FAILURE:

Failure of spread foundations is usually the result of failure of the supporting soil, which is often associated with liquefaction (in which loose, saturated, granular soils effectively turn to quicksand under earthquake shaking, and lose their shear strength). Often, these failures result in gross settlements, but the failing soil is unable to transmit strong shaking to the structures which survive.

Eurocode 8 Part 5 lists the following instances where SSI (Soil-Structure Interaction) should be allowed for.

(a) Structures where P–∆ effects play a significant role.

(b) Structures with massive or deep-seated foundations, such as bridge piers, caissons and silos.

(c) Tall and slender structures such as towers and chimneys.

(d) Structures supported on very soft soils.

(e) The effect of the interaction between piles and the surrounding soils during earthquakes needs to be considered when the piles pass through interfaces between very soft soils and much stiffer soils.

Some factors that should be considered in connection with seismic resistance are as follows.

(a) Where the superstructure is designed to achieve a high level of ductility, the foundation must be able to develop the superstructure’s yield capacity.
It is no use having a perfectly detailed ductile superstructure supported by a foundation which suffers brittle failure before that ductility is achieved.

(b) Superstructure systems that involve large uplift forces (e.g. shear walls with a high height-to-width ratio) are only suitable if foundations can be built economically to resist these tension forces.

(c) Piles have loads imposed upon them due to lateral deflection of the upper layers of softer soil during earthquakes. Small driven piles of less than 0.5m diameter are generally sufficiently flexible to accept this movement without suffering large bending stresses. Large-diameter piles, however, may experience significant lateral forces as they are relatively stiff compared with the soil.

(d) Raking piles are generally to be avoided because they add greatly to the lateral stiffness of the pile group. Their stiffness means that they will not be able to conform to the deformations of the soft soil strata, but will receive very large lateral loads, arising from the mass of the soft soils attempting to move past the stiffened pile group. Raking piles have been found to be prone to failure during earthquakes.

(e) Piling through potentially liquefiable layers needs careful consideration, since the piles would have to transmit the lateral forces from both the superstructure and adjacent non-liquefied soil through the liquefied strata. The piles would be effectively unsupported laterally in this region and so may be subject to large bending and shear stresses which would be difficult to resist.

(f) Raft foundation support via a basement may be an alternative solution when founding on potentially liquefiable layers.

The main features to consider in the seismic design of foundations are as follows.

(a) A primary design requirement is that the soil–foundation system must be able to maintain the overall vertical and horizontal stability of the superstructure in the event of the largest credible earthquake.

(b) The foundation should be able to transmit the static and dynamic forces developed between the superstructure and soils during the design earthquake without inducing excessive movement.

(c) The possibility of soil strength being reduced during an earthquake needs to be considered.

(d) It is not sensible to design a perfectly detailed ductile superstructure
supported by a foundation which fails before the superstructure can develop its yield capacity.

(e) Just as design of the superstructure should minimize irregularity, so irregular features in foundations need to be avoided. These include mixed foundation types under different parts of the structure and founding at different levels or on to strata of differing characteristics.

(f) Special measures are needed if liquefaction is a possibility.

(g) Special considerations apply to piled foundations.

**FAILURE:**

**Pad and strip foundations:** Failure modes

In addition to transferring vertical loads safely into the soil, shallow foundations in the form of pads or strips must also transfer the horizontal forces and overturning moments arising during an earthquake.

(a) **Sliding failure:**

Resistance to sliding in shallow footings will usually be mobilized from the shear strength of the soil interfacing with the footing. In granular materials, the minimum vertical load which could occur concurrently with the maximum horizontal force must be considered, since this condition will minimize shear resistance. The maximum seismic uplift should be assessed as the sum of components due to overturning and vertical seismic accelerations, combined by the SRSS method.

(b) **Bearing capacity failure:**

Static bearing capacity can be determined from formulae which allow for the inclination and eccentricity of the applied load.

(c) **Rotational failure (overturning):**

Where the soil is strong, the foundation may start to rotate before a bearing capacity failure occurs, particularly if the vertical load is small. In the case of pad foundations supporting a moment-resisting frame, such a rotation
may be acceptable, since a frame with pinned column bases still retains lateral stability.

However, the associated redistribution of moments would lead to increased moments at the top of the lower lift of columns, which would need to be designed for.

In contrast, an isolated cantilever shear wall is not statically stable with a pinned base. Rocking should, therefore, be prevented under design forces in most circumstances.

Uplift can be prevented by provision of additional weight or by piles or anchors to resist the transient vertical loads, or by a wider foundation.
Modes of failure in pad foundations: (a) sliding failure; (b) bearing capacity failure; (c) overturning; and (d) structural failures, where (i) shows shear failure in footing, (ii) shows shear failure in stub column, (iii) shows bending failure in footing, and (iv) shows bending failure in ground beam.

Structural failure in the foundation:

Sufficient strength must be provided to prevent brittle failure modes in the foundation structure, such as shear failure in footings or stub columns.

Ties between footings:

Some form of connection is usually needed at ground level to link isolated footings supporting a moment-resisting frame. The ties prevent excessive lateral deflection in individual footings, caused by locally soft material or local differences in seismic motion. Where the footings are founded on rock or very stiff soil, however, the tendency for relative movement is much less and the ties are generally not required.

The connection can take the form of a ground beam, which will also assist in providing additional fixity to the column bases and will help to resist overturning. Alternatively, the ground-floor slab can be specially reinforced to provide the Restraint.

Raft foundations:

All of the soil failure modes illustrated in Fig below may apply to raft foundations. The analysis would assume a uniform soil pressure distribution in equilibrium with the peak and moments within the raft near its edge, since the soil, being poorly restrained, has low bearing capacity there. More complex analysis would allow for soil nonlinearity and dynamic effects. applied loads. Figure below shows that this may lead to an underestimate of shears.
Pressure distribution near the edge of a raft under seismic loading the effect of the uplift on internal forces within the raft foundation and superstructure must be accounted for.

Piled foundations:

Vertical and horizontal effects:

Vertical loading on pile groups during an earthquake arises from gravity loads, seismic overturning moments and vertical seismic accelerations. Since the two latter effects are not correlated, they can be combined by the SRSS method, and added to the gravity load. The procedures are straightforward, and the design of end-bearing piles is similar to that for static vertical loads. Friction piles may be less effective under earthquake conditions and require special consideration. Flexible piles may be able to conform to the deflected soil profile without distress, but large-diameter piles are relatively much stiffer than the soil and large forces may be generated.
Inertial and kinematic loading on piles

Usually, locations of plastic hinges other than at the tops of the piles are not considered acceptable. Further considerations for detailing of concrete piles are given in the next section.

Particular regions where special detailing measures may be required are as follows.

(a) The junction between pile and pile cap is a highly stressed region where large curvatures may occur in the pile. Unless adequate confinement and good connection details are present, brittle failure may occur.

(b) Junctions between soft and hard soil strata may also impose large curvatures on piles; such junctions are likely to be potential points for formation of plastic hinges.

(c) Piling through soil which may liquefy can pose special problems. In this case the pile may have a large unsupported length through the liquefied soil and should be reinforced as though it were an unsupported column. A reliable ductile behaviour will also be necessary in this situation.

Detailing concrete piles:

Both Eurocode 8 and IBC (ICC 2003) require additional confinement steel in the form of hoops or spirals, both at the pile head and at junctions between
soft and stiff soils, since these are potential plastic hinge points. Eurocode 8 also provides for minimum anchorage requirements of the vertical steel into the pilecap where tension is expected to develop in the pile.

**Raking piles**

Raking piles pose a special problem, because they tend to attract not only the entire dynamic load from the superstructure, but also the horizontal load from the soil attempting to move past the piles (Fig. above), which is a particularly severe example of the kinematic interaction effect.

Raking piles are found to be particularly susceptible to failure in earthquakes.

They should therefore be used with care in seismic regions, with particular attention to the kinematic interaction effects

**Design in the presence of liquefiable soils:**

Two types of countermeasure are possible in the presence of liquefiable soils.

1. Either the structures can be modified to minimize the effects of liquefaction, or the soils can be modified to reduce the risk of their liquefying.

2. Foundations may also be designed which minimise the consequences of liquefaction.

**Possible options are as follows:**

(a) Provision of a deep basement, so that the bearing pressures due to vertical loads are greatly reduced. Essentially, the structure is designed to float in the liquefied soil. This may be less effective in countering soil pressures due to overturning forces, and so is likely to be an option confined to relatively squat structures.

(b) Provision of a raft with deep upstands. The structure is designed to sink if liquefaction occurs until vertical equilibrium is regained. The solution may imply large settlements and again is most applicable to relatively squat structures.

(c) Provision of end-bearing piles founded below the liquefiable layers. Although this will counter vertical settlements due to gravity loads and overturning moments, the piles may be subject to large horizontal
displacements occurring between top and bottom of the liquefying soil layer, and the piles must be designed and detailed to accommodate this.

The alternative strategy is to reduce the liquefaction potential of soils. A number of methods are possible and consist of three generic types, as follows:

1) Densification, for example by vibro compaction, which produces a more stable configuration of the soil particles. This may not be an option for existing structures, because of the settlements induced by the process.

(2) Soil stabilization or example by chemical grouting, which makes the soil less likely to generate rises in pore water pressure.

3) Provision of additional drainage, for example by provision of sand drains which tends to reduce the rise in pore water pressure. These tend to be expensive solutions but they can be effective.

T.RangaRajan.