Chapter 5
Sub-structure Strengthening

It is less customary to strengthen the foundations of structures compared to the superstructure. This could be due to the high safety factors imposed by design codes when designing foundations, in addition to that, in most of the soil types, the bearing capacity at the foundation level increases with time because of the long-term induced loads on the soil by the structure weight. The need to strengthen the footings may be for the reason of increased loads or concrete deterioration after failure of the insulation material protecting the footings. Concrete deterioration is aggravated by the increase of surface water or increase in the underground water level, especially when the soil is contaminated with chlorides, sulfates or other harmful chemicals. It should be noted that the concrete cover of the footings plays an important role in protecting the steel reinforcement from corrosion and as long as the concrete cover is in good condition and well protected, the structural condition of the foundations shall remain as per its first condition.

Inspection of the top surface of the footings requires removal of the slab-on-ground and the soil fill above the footings; however, it is very challenging to inspect the bottom concrete surface of the foundation despite it is more stressed than the top surface. Research on integrating infrared thermography or ground penetrating radar involving ultrasonic signals is still ongoing, especially for the footings bottom concrete surface. Once the foundation is exposed, visual inspection as well as conventional techniques of concrete evaluation is used to assess the structural condition of the top concrete surface. In case of observing any concrete distress, repair work should be carried out to reinstate the structural condition of the footings.

There are symptoms of soil movement, which may appear in the structure in the form of either rigid body rotation, settlement, translation or inclined cracks in the non-structural or structural elements. If any of those symptoms was observed, investigation of the soil beneath the structure shall be of a prime importance to evaluate its characteristics before problem treatment. The investigation shall reveal that the soil is capable to support the weight and the external loads imposed on the structure without excessive movement; otherwise, soil and/or sub-structure strengthening should be implemented to ensure enough capacity/behavior of the foundations. There

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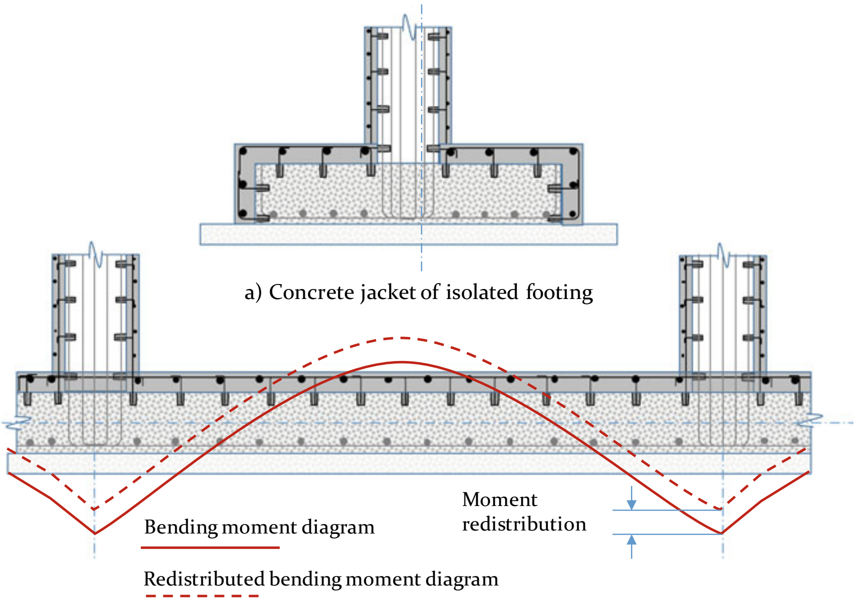
are several methods for the soil treatment including grout injection, water submerge or soil confinement be means of piling around the structure. After ensuring that the soil treatment has succeeded to stop the movement of the structure, which can be done by continuous monitoring of the structure, foundations shall be strengthened by any of the means hereafter.

5.1 Strengthening of Shallow Foundations

Shallow foundations including isolated and mat foundations may be strengthened in case of distress provided that the soil underneath can safely resist the induced loads without excessive settlement. The typical strengthening procedure of isolated footings is by increasing the depth and may be the width of the concrete footing using concrete jacket, as shown in Fig. 5.1a. Increasing the depth of the foundations shall enhance both the flexural and shear capacity of the footing. The increase of flexural resistance is due to increase of the lever arm between the compression in the concrete and tension in the bottom steel reinforcement; however, the area of the bottom steel reinforcement cannot be increased. In this case, the designer should ensure that the minimum steel ratio bound by design codes, after section increase in depth, is not violated; otherwise, the designer should treat the footings as if it is made of plain concrete. The increase in one-way and two-way (punching), shear strength is due to increase of the overall area resisting shear forces. Increasing the width of the isolated footings shall reduce the bearing stresses on the existing foundations resulting from the additional loads on the building not existing at the time of strengthening.

When designing the concrete jacket of the isolated footings, the inherent stresses in the original section due to all applied loads at the time of strengthening shall be accounted for when calculating the capacity of the strengthened footing. The concrete jacket should be well connected to the original section by calculated number of shear dowels to ensure no slip occurs at the interface between the old and new concrete. Those dowels should be designed based on the interfacial shear force divided by the shear capacity of the dowels, as per Chap. 3. The composite section shall be resisting the additional loads applied on the structure after construction completion of the concrete jacket. Algebraic superposition should be made for the stresses in the original section including the inherent stresses and additional stresses after jacket construction.

Strengthening of the mat foundation is different from that of the isolated footings since the indeterminacy of the concrete member provides an additional safety factor, which can be realized by plastic analysis of the structural member. This can be implemented by moment redistribution between the positive and negative moments at the column location and at the mid-span, as shown in Fig. 5.1b. The bending moment at the column location may be redistributed on the account of the moment at the mid-span section, where its capacity can be increased by strengthening the top surface of the raft using any of the prescribed techniques, i.e., concrete jacket or by bonding steel or FRP reinforcement. Limits of the moment redistribution,



b) Concrete jacket of mat foundation

Fig. 5.1 Strengthening of isolated and mat foundations

as per the ACI318-19, are up to 20% provided that the net tensile strain in the steel reinforcement at flexural capacity is more than 0.0075, as shown in Fig. 1.20. Design of the external reinforcement used for strengthening, whether its steel plates or FRP strips, is as per Chap. 3. In all cases, the external reinforcement should be well protected from any aggravation that may be encountered from the surrounding soil.

5.1.1 Conversion of Shallow to Deep Foundations

Soil investigations could show that the soil beneath the shallow concrete footings cannot sustain the loads induced on the structure. In those cases, shallow foundations may be converted to deep foundations to reach lower soil stratum with better performance, and hence, the soil capacity at the new foundation level is increased. This conversion may be done by underpinning the existing footings with small diameter piles or adding concrete caissons. In both cases, the new structural elements should be well connected to the existing foundations to ensure that all loads are transferred to the lower soil strata.

Time is an important factor in the transfer of the structure loads to the good lower soil strata since most of the dead loads are in effect while the new structural elements (piles or caissons), are constructed. The dead load of the structure shall be carried only by the existing footings, while the new loads (live and lateral loads) shall be shared between the old and new footings. The new elements shall only participate in carrying those loads when excessive settlement of the soil beneath the existing footings occur.

5.1.2 Foundations Underpinning

Foundations underpinning is carried out by adding special type of small diameter bored piles known as needle-piles, or micro-piles with diameter less than 300 mm (normally between 120 and 250 mm). Those piles can carry high axial capacity because of the special method of construction during which concrete or mortar is forced into the soil. Due to its small diameter, micro-piles can sustain axial loads by skin friction at their interface with the surrounding soil layers. Those piles have much smaller lateral resistance due to its small diameter; however, it may be constructed with an angle to participate in resisting the lateral loads on the structures.

Figure 5.2 shows typical construction of the micro-piles, where drilling the concrete foundation is executed to insert the steel pipe used as pile casing. Large diameter steel bars are placed in the casing and grout is pumped under pressure to fill the casing and the toe of the pile. Finally, anchor plate is welded to the steel casing to work as an anchorage and provide the required bearing of the pile. Special reinforcement should be added at the vicinity of the steel anchor plate to resist the localized stresses induced by end bearing.

5.1.3 Use of Concrete Caissons

In case that a structure is suffering from soil movement and the good soil stratum is few meters away from the foundation level, strengthening may be done so that the loads of the structure are transmitted to the deeper level using caissons. Caissons, sometimes called "piers", are made by auguring a deep hole into the ground, and then filling it with either plain or reinforced concrete. Caissons with relatively small height and large diameter are constructed in the free areas between the existing foundations. Reinforced concrete mat is then cast at the level of the existing foundation, while connected with both the existing foundations and the concrete columns using steel dowels. The mat foundation is then supported on the caissons, as shown in Fig. 5.3.

5.2 Retaining Walls

Deformations of reinforced concrete retaining walls (RW), are likely happening from soil movement, which either happening due to nonconformity in the soil properties

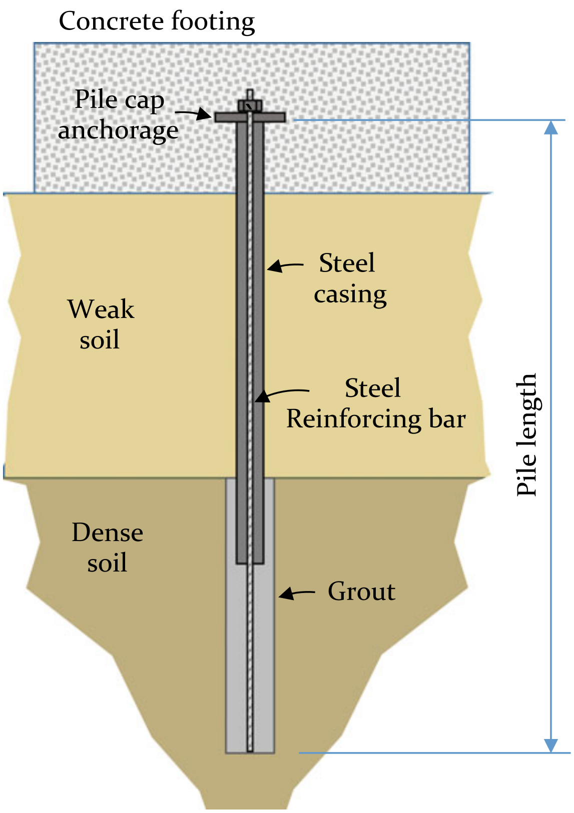


Fig. 5.2 Strengthening of foundations using micro-piles

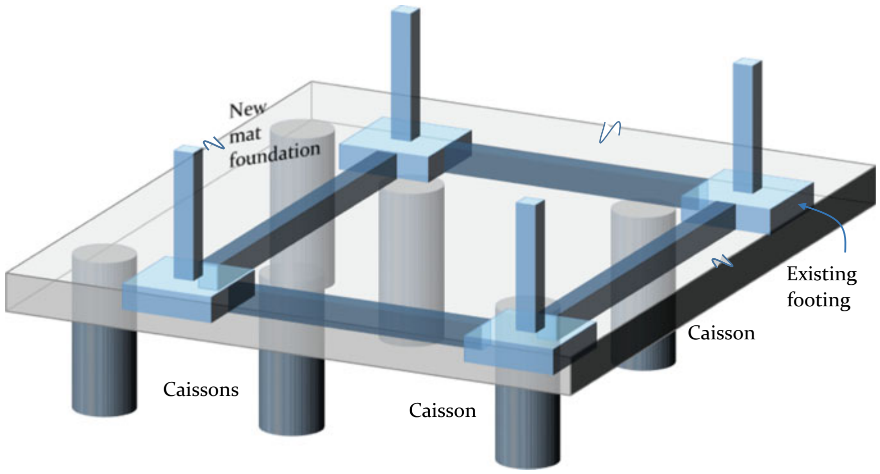


Fig. 5.3 New mat foundation supported on newly cast plain concrete caissons

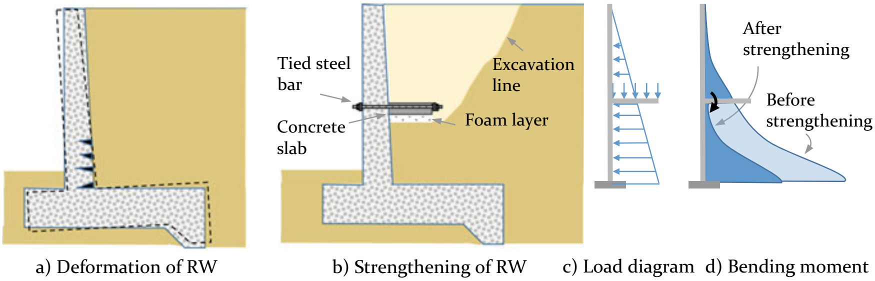


Fig. 5.4 Strengthening of retaining wall

or non-uniform pressure on the soil induced from neighboring structures, as shown in Fig. 5.4a. It is then logical to think how to lessen the pressure on the retaining walls or to reduce the resulting straining actions to minimize its deformations. One of the techniques to reduce the overall turning moment on a wall is to add counterweight causing rotation in an opposite direction to that resulting from the earth pressure. This counterweight is created by adding a reinforced concrete slab, called “shelf slab”, at the mid-height of the wall, as shown in Fig. 5.4b.

The shelf slab is constructed after excavating the soil behind the wall and inserting large diameter reinforcing bars at constant intervals along the wall to act as top reinforcement of the new cantilever slab. The bar is anchored with nuts from both ends to develop the tensile force in the bar. The counterweight is created by the weight of the soil carried by the new slab. It is important to provide compressible material beneath the concrete slab, such as polystyrene foam, to allow for the slab vertical deformations when loaded with soil fill above. The cantilever slab shall induce bending moment on the wall that will reduce the overall bending moment resulting from earth pressure, as shown in Fig. 5.4c, d.

Example 5.1 Reinforced concrete mat foundation has a total thickness of 800 mm and a depth from the compression face of the mat to the tensile reinforcement of 750 mm. The mat is reinforced with steel mesh of eight bars of 22 mm diameter bars each direction at the bottom face and eight bars of 18 mm diameter bars each direction at the top face, as shown in Fig. 5.5.

The concrete compressive strength is 35 MPa, while the yield stress of the steel is 420 MPa. It is required to propose a strengthening procedure for the concrete mat so that it can carry the following unfactored bending moments (see Fig. 5.5):

At the column sections (Point A), MDL = 500 kNm per meter, MLL = 250 kNm per meter.

At the mid-span section (Point B), MDL = 300 kNm per meter, MLL = 150 kNm per meter.

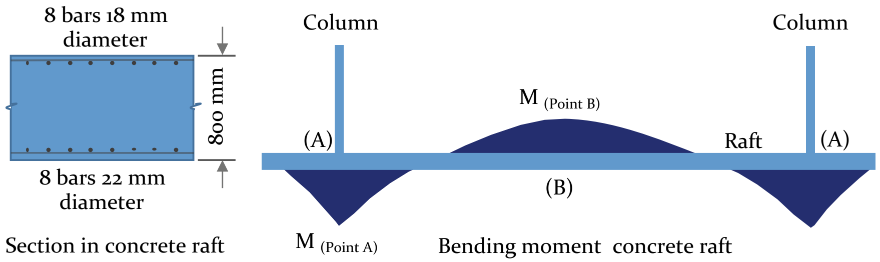


Fig. 5.5 Concrete raft (Example 5.1)

Answer

The factored moment is calculated using the equation "$M_u = 1.2$ MDL + 1.6 MLL", therefore, at Point "A": $M_u = 1000$ kNm per meter and at Point "B": $M_u = 600$ kNm per meter.

As per Chap. 3,

Point "A", the section capacity, $M_r = 837.5$ kNm per meter < 1000 kNm, "unsafe". Steel tensile strain at section capacity, $\varepsilon_t = 0.0389$.

Point "B", $M_r = 566$ kNm per meter < 600 kNm, "unsafe", Steel tensile strain, $\varepsilon_t = 0.0596$.

It is recommended to redistribute the bending moment so that sections at the columns, "A", shall have a bending moment equal to their capacities "837.5 kNm", while the bending moment at section "B" shall be accordingly increased, as shown in Fig. 5.6. This is mainly since increasing the steel reinforcement at the bottom of the raft is not possible.

As per Fig. 1.20, the maximum percentage of moment redistribution is 20% at net tensile strain of 0.02,

Percent of redistributed moment at section "A" $= \frac{1000-837.5}{1000} = 16.25\% < 20\%$ (ok).

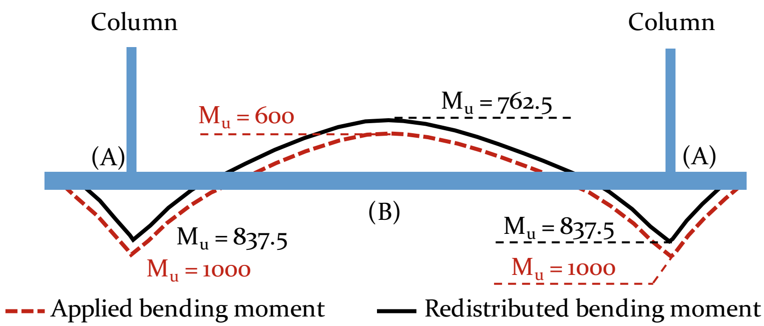


Fig. 5.6 Moment redistribution of the raft

The increased bending moment at section “B” = 1000 + 600 − 837.5 = 762.5 kNm per meter.

The top surface of the concrete raft may be strengthened using reinforced concrete jacket, externally fixed steel plates or externally bonded CFRP laminates to increase its flexural capacity from 566 to 762.5 kNm per meter.

5.3 Case Study 5.1

This case study is for a high-rise building, which consists of two basements, ground floor, 30^{+} typical floors and roof. The structural system of the floors consists of reinforced concrete flat slabs supported on concrete columns and shear walls. The vertical elements in the building are supported on concrete raft with varying thickness, which in turns supported on piles. The built up area of the basements is larger than that of the typical floor (see plan in Fig. 5.7). The building is subjected to uplift forces due to high ground water table.

5.3.1 Problem Description

The concrete raft experienced through cracks at the vicinity of the piles and as a result the ground water found its way inside the lower basement. It was clear that the

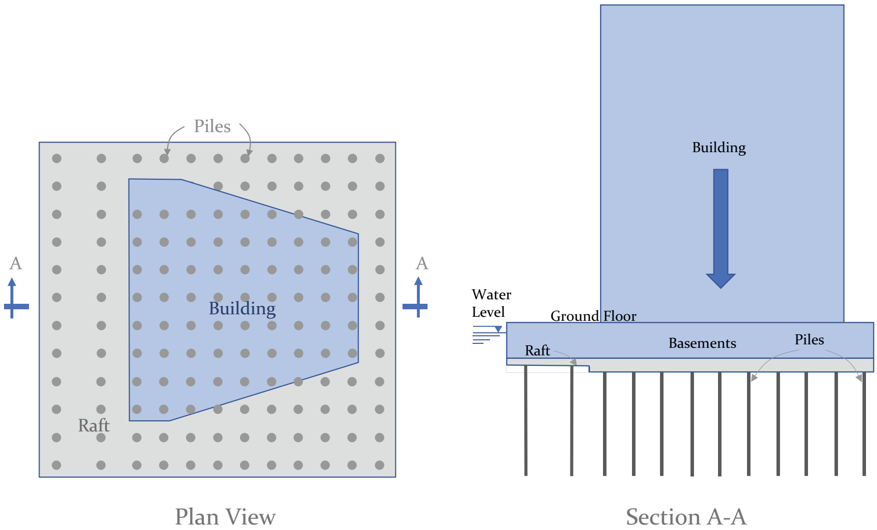


Fig. 5.7 Layout of the building

Fig. 5.8 Punching shear cracks in the raft

cracks were due to punching shear of the raft supporting the outer piles. Figure 5.8 shows picture of the punching shear cracks at the top surface of the raft.

Analysis of the raft foundations under the effect of gravity loads due to the weight of the building and uplift forces induced from the high water table resulted in compression on most of the piles and tension in other piles. The analysis was made using 3D-finite element model for the raft and the piles, where the raft was modeled using shell elements, divided into sections based on its thickness. The piles were modeled using frame elements with lateral joint springs every one meter along its depth in two perpendicular directions.

Figure 5.9 shows a schematic of the loads on the raft, displacement of the raft and induced forces on the piles. It is clear that in case of uplift induced from the high water table, outer piles in the area away from the footprint of the tower are subjected to tensile forces that caused pull-out of piles from the raft. The small thickness raft in that area helped in obtaining high punching shear stresses in the raft away from the tower vicinity.

5.3.2 Punching Shear Check

Punching shear failure was checked for the raft taking into consideration that the piles are subjected to tension. Figure 5.10 proposes two possible modes of failure that are used to check the resistance against pull-out of the piles. The first mode occurs with a failure plane outside the pile core, while the second mode occurs inside the pile core.

The checks against the possible failure modes show that some of the external piles are unsafe and would fail by pull-out or reverse punching shear failure.

The first mode may occur provided that proper confinement is provided for the pile reinforcement inside the raft. The second mode represents the other extreme where pile reinforcement is flared inside the raft.

The actual shear stresses in the raft are calculated by multiplying the area of the failure plane by the shear strength of the concrete raft, where the area of the failure

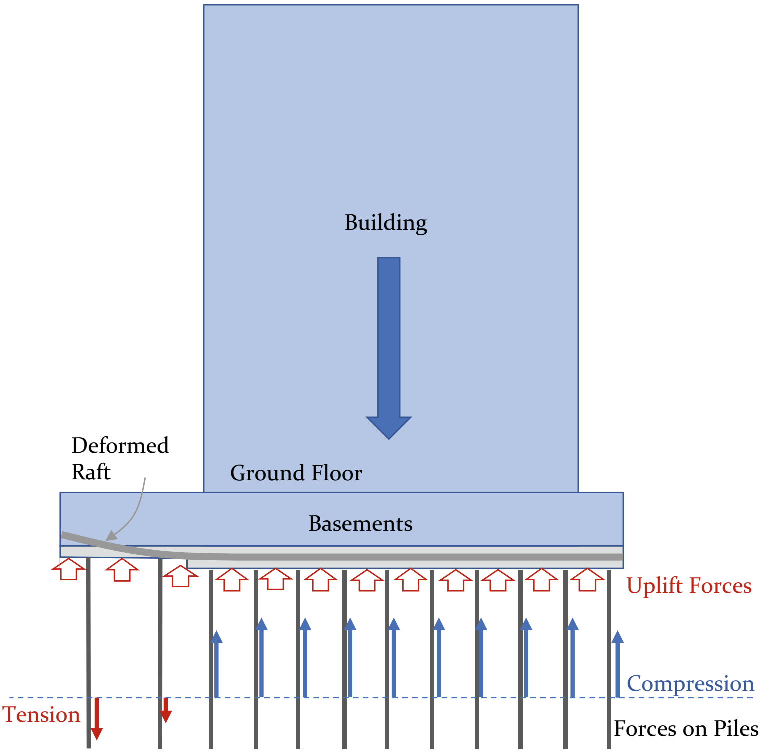


Fig. 5.9 Loads on raft and forces on piles

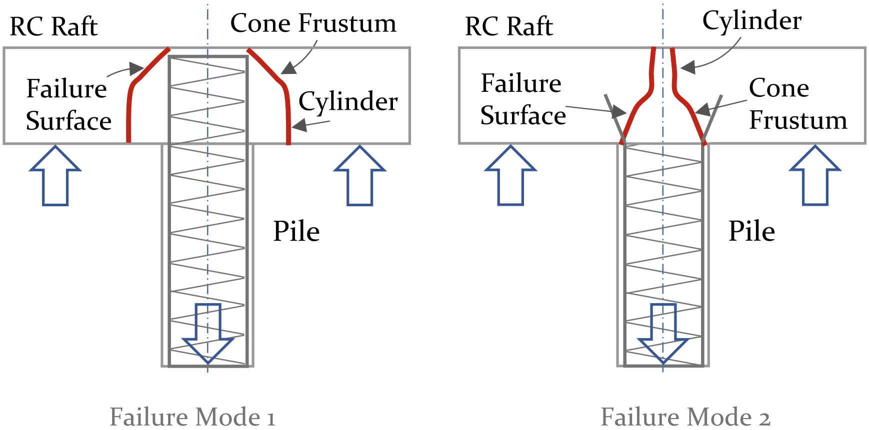


Fig. 5.10 Punching shear failure modes

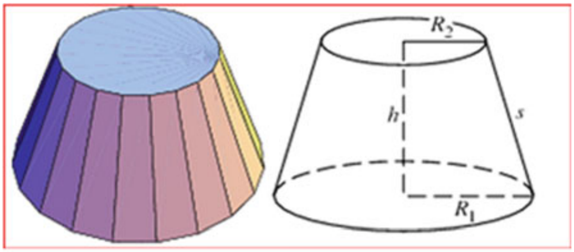


Fig. 5.11 Punching shear failure plane

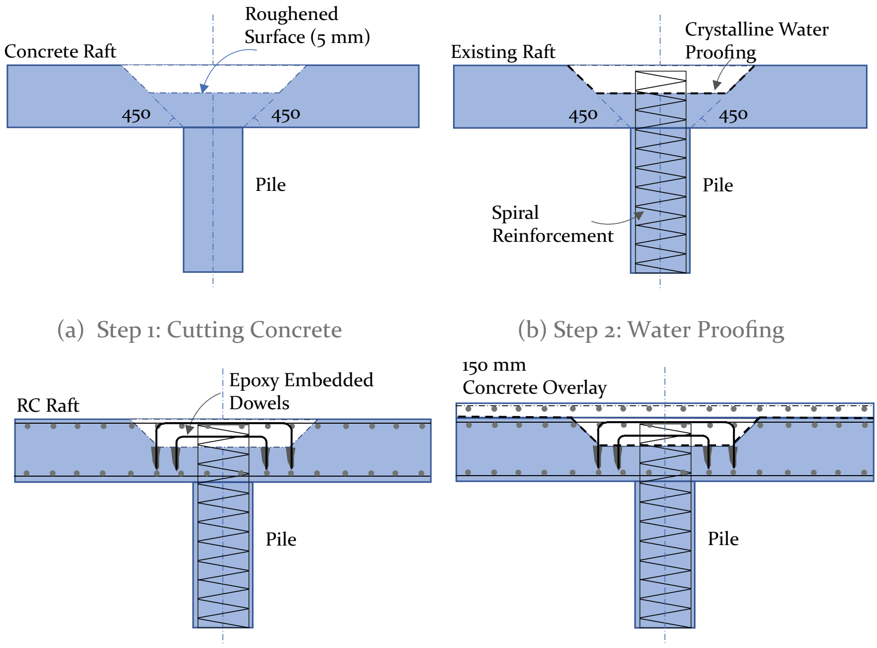
$$
\begin{aligned}
A &= \pi (R_1 + R_2) s \\
&= \pi (R_1 + R_2) \sqrt{(R_1 - R_2)^2 + h^2}
\end{aligned}
$$

plane is calculated using the formula given in Fig. 5.11. Punching shear failure occurs when the shear stress in the raft exceeds the concrete shear strength. Calculations showed that the second mode of failure is the governing mode.

5.3.3 Proposed Retrofitting Scheme

The observed crack patterns for two piles in the raft (see Fig. 5.8) clearly draw a map of the pile location and its circumference, which indicates a mode of failure that lies in between the aforementioned failure modes. It is obvious that the failure planes are located along the pile circumference; therefore, any retrofitting scheme should have proper amount of shear reinforcement crossing that plane. Therefore, the concept of the foundation retrofit relied on enhancing its shear capacity by inserting steel reinforcement crossing the existing and anticipated cracks. The repair procedure is summarized as follows:

1. Demolish Cracked Pile Tops
   In this step, hydro-demolition was used to remove the cracked concrete all around the piles with the dimensions as shown in Fig. 5.12. All debris were cleared, and a surface roughness of a minimum of 5 mm should be achieved as well as 45° inclined surface should be formed, as shown Fig. 5.12a.
2. Water Proofing and Pile Confinement
   A proper crystalline water proofing material was applied in two perpendicular coats to the formed surface. Pile reinforcement was straightened out and confined with spiral reinforcement, as shown in Fig. 5.12b.
3. Installation of Horizontal Reinforcement
   Additional U-shape steel reinforcement was planted using epoxy in the reinforced concrete raft, as shown Fig. 5.12c. Two layers of the steel reinforcement were



(a) Step 1: Cutting Concrete (b) Step 2: Water Proofing

(c) Step 3: Planted Reinforcement (d) Step 4: Concrete Overlay

Fig. 5.12 Strengthening steps of the concrete raft

implemented; one at the level of the original raft top reinforcement and one-layer mid-way in the demolished thickness. The planted reinforcement crossed the potential cracks and added to the punching shear resistance of the raft.

4. Installation of Overlay Reinforcement

   A 150 mm concrete overlay was cast on the top of the existing raft along with the demolished parts around the piles. The entire surface of the raft was cleared of debris and roughened to 5 mm amplitude. A top mesh reinforcement layer was applied to the top of the overlay with a minimum cover of 35 mm, as shown Fig. 5.12d. Proper bonding agent was applied to the old substrate, and concrete with compressive strength of 40 MPa was used for the overlay with crystalline admixture.

5.4 Case Study 5.2

The Panorama building in one of the hotels in the city of Sharm-El-Shaikh, Egypt, is 11.0 m wide and 40.0 m long with a triangular shape in plan. The one-story building is located close to the edge of a hill, as shown in Fig. 5.13. The ground level of the building is divided and used for accommodation, while the roof was not used. It was

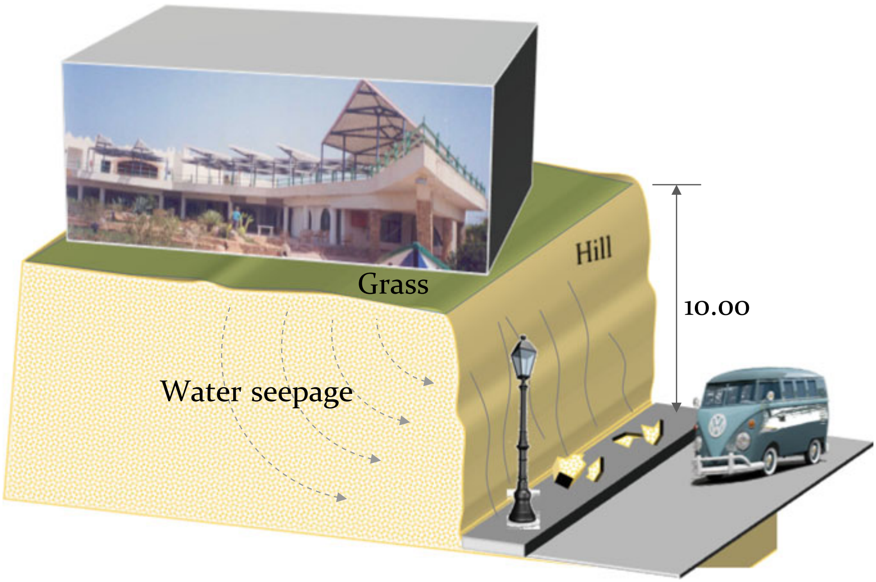


Fig. 5.13 Location of the Panorama building

the owner's desire to use the roof as an open-air café having a view of the city from the top of the hill. The building consists of reinforced concrete slabs supported on rectangular beams and reinforced concrete columns.

After two years of construction, both the slabs and beams were severely cracked, as shown in Fig. 5.14. The crack width was up to 1.0 mm at some locations. The cracks were observed on the entire width of the slab, at the fourth bay away from the hill, crossing all beams and walls. Flexural and shear cracks were also observed at some of the beams. The Parapet of the roof, which is made of brick, was severely cracked. The cracks were wide at the top of the parapet, reducing in width toward the bottom. The location of these cracks coincided with the cracks in the concrete slab. No cracks were observed in the columns.

5.4.1 Problem Diagnosis

The crack pattern of the structure indicated that differential settlement between the foundations of the building had occurred. The columns close to the edge of the hill settled more than the interior columns causing a rigid body movement of that part of the structure (part 1 in Fig. 5.14), as well as cracks across the entire building.

Soil pits were carried out to determine the properties of the soil. The investigations showed a layer of fine sand beneath the shallow foundations. It is believed that the fine sand was washed out of the hill with the water drained from spraying plants close to the building (see Fig. 5.13). The washed sand found its way out from the side of the

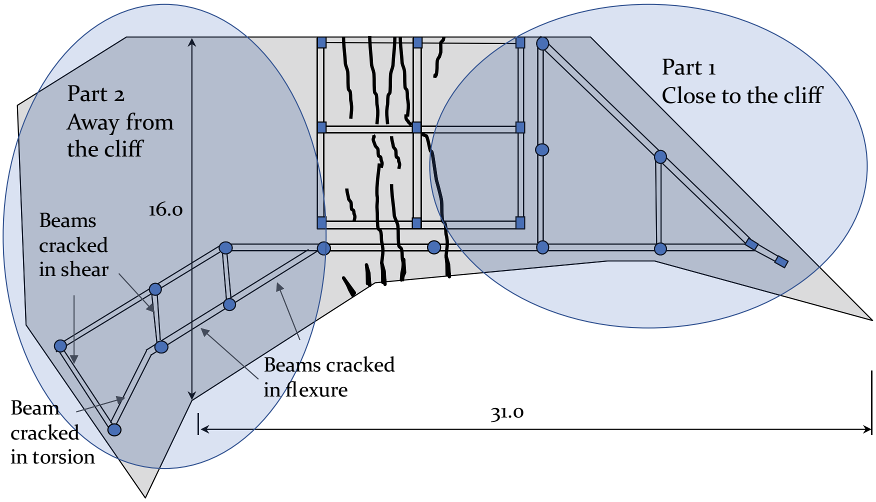


Fig. 5.14 Cracking in the slab of the Panorama building

hill causing large settlement of the footings adjacent to the hill. The investigations also showed a good soil stratum located 10.0 m below the ground level.

5.4.2 Proposed Strengthening Scheme

Before starting implementation of the strengthening works, it was decided to move the plants away from the building and eliminate any source of water close to the foundation of the building. In the first phase of the strengthening scheme, it was essential to eliminate the cause of the problem and stop the settlement of the foundations before remedy of the superstructure. A rigid reinforced concrete mat foundation supported on plain concrete caissons was cast to support the columns, as shown in Fig. 5.15. The caissons were 1.0-m diameter and 10 m high, bearing on the good soil stratum and at least one caisson was cast around each existing column. In order to ensure that the deformation of the structure was stopped after casting the new foundation, the cracks were monitored to record any changes. Several crack meters were installed crossing the existing cracks in the slabs and beams. It was ensured that no further development of the cracks was observed before commencement in strengthening of the superstructure.

The second phase of the strengthening scheme was to restore the building and increase the structural capacity of both the slabs and beams to resist higher live loads. CFRP strips and laminates were used to strengthen the slabs and beams in flexure and shear, as shown in Fig. 5.16. The measured crack width of the slabs and beams was used to estimate the tensile stress in the steel reinforcement. The required area of CFRP was calculated to allow for double the live load, accounting for the increase in the stress of the steel reinforcement.

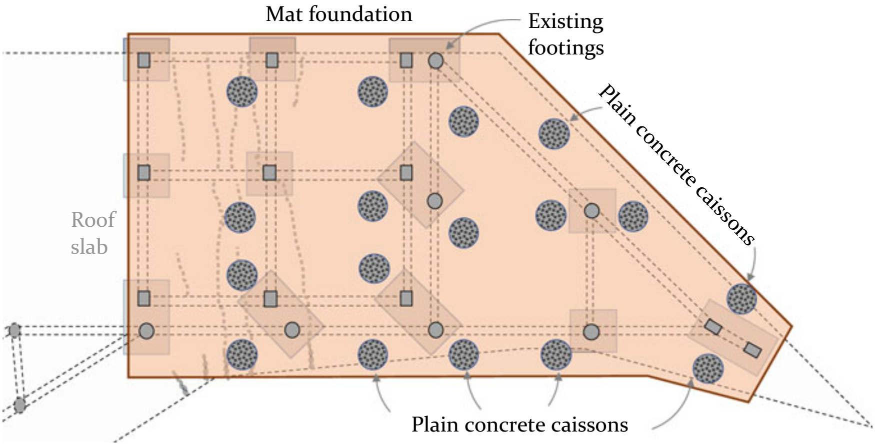


Fig. 5.15 Strengthening of the foundations

Fig. 5.16 Strengthening of the superstructure with CFRP