Amr Abdelrahman

Strengthening of Concrete Structures

Unified Design Approach, Numerical Examples and Case Studies

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About This Book

This book presents unified design approach for strengthening concrete members with different techniques such as concrete, steel and FRP jacketing. Preference between the various techniques is explained in light of strengthening limits, procedures, and application for each case. Examples for evaluation of existing structures and design equations for strengthening concrete elements are presented based on the ACI design codes and standards. Numerical examples are also given for different strengthening techniques with illustrations for the construction methodology and detailing for each case.

Several case studies are explained starting from problem diagnosis, structure analysis, proposal for different strengthening methodologies and implementation procedure for the selected scheme. The case studies include structures subjected to gravity loads, lateral loads, elevated temperature and differential settlement of the foundations. It addresses design and construction errors, environmental impact and soil movement. The case studies include conventional reinforced concrete, post-tensioned and precast concrete members.

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About the Author

Dr. Amr Abdelrahman is Professor of Concrete Structures and former chairman of the Structural Engineering Department (2014–2020), Ain Shams University, Cairo, Egypt. He is a member in the following Egyptian Codes; Design and Construction of Concrete Structures, Design and Application of Fiber Reinforced Polymers in Construction (Vice Chair), Strengthening of Concrete Structures, and Planning, Design and Construction of Bridges and Highways. He is a Fellow of the Egyptian Society of Engineers and a member in the standing committee of the Arab Code for Design of Bridges.

The author has several technical publications and supervised graduate students. He is also a consultant engineer and the founder of "AACE Consultant Engineers Office", who participated in the design of several projects in the Middle East. He has more than 36 years of experience working in the field of design of reinforced and prestressed concrete structures and strengthening of concrete structures.

Chapter 1
Evaluation of Concrete Structures

Structures are built to serve a certain function efficiently without deviation on the designed performance throughout its lifetime. While the structure is at service, it should perform well structurally, aesthetically and environmentally as well as it should fulfill all the requirements imposed by the design codes, standards and regulatory authorities. At any observed deviation of the structure performance, measures should be taken to protect, mitigate or improve the current condition, while the decision shall be based on technical assessment given by expert engineers addressing the need for immediate action, methodology of remedial action in addition to the cost. The action should consider the sequences of interruption of the structure use during implementation since it is a major element in the decision.

Rehabilitation of concrete structures is employed, where repair and strengthening of concrete members aim at improving the structural capacity or behavior to recover the originally designed performance, (repair), or to upgrade the concrete member to satisfy higher requirements, (strengthening). The rehabilitation may be also required to satisfy higher seismic performance, (retrofit). The rehabilitation starts by evaluation of concrete structures to understand the deficiencies and most important the reasons for these deficiencies. The evaluation aims also at determining the requirements for upgrade whether to increase the current strength or to improve the serviceability, or obviously to provide enough safety factor for the structure stability. Selecting the right material for rehabilitation is the success key for long-lasting satisfactory performance, accompanied by proper design and application of the strengthening scheme. Protection of concrete members intends to provide a second line of defense against the severe exposure to environmental conditions and/or misuse. In some cases, monitoring of the rehabilitated members is followed to provide information about any distress or deterioration that is unlikely to happen after the engineered implementation of rehabilitation. Maintenance of concrete structures, not only for the rehabilitated members but also for other structural members, is scheduled after periodic inspection to maintain the good performance and detect any shortcomings that may affect the structure lifespan.

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In general, satisfactory performance of the structure during its lifetime is an acceptable indicator of adequate safety in case that the structure has been subjected to known loads and environmental conditions.

1.1 Properties of Strengthening Materials

Different materials are nominated to be used in strengthening concrete members, while material selection should be based on many factors including compatibility with concrete, strength, constructability, resistance to environmental conditions, aesthetics and cost. Among several materials, concrete, steel and fiber reinforced polymers, (FRP), are the commonly used materials in strengthening existing concrete structures. Different strengthening techniques may be applied; yet, unified design approach still can be adopted while understanding the differences between materials in the mechanical properties and resistance to environmental conditions. The following sections describe the characteristics of each material, which has a direct impact on the strengthening design.

1.1.1 Concrete Characteristics

Concrete is the most used construction material worldwide due to its strength, durability and availability with different properties that can be tailored to suit a certain application. Normal and lightweight concrete, self-consolidated concrete, high-performance concrete, fiber reinforced concrete and shotcrete are available with large spectrum of strength, stiffness and properties while fresh or hardened. Figure 1.1 shows the stress–strain behavior of different types of concrete under uniaxial state of compression stress. In general, ductility of the fiber concrete is larger than that of the normal weight and self-consolidated concrete (Naaman 2017).

The concrete compressive stress is increased when subjected to triaxial state of compression stresses, where it can be presented with the model given in Eq. (1.1) (Park and Paulay 1975).

$$f'_{cc} = f'_c + k\, f_l \tag{1.1}$$

where "f'_{cc}" is the concrete compressive strength under triaxial state of stress, "k" is a factor that may range between 4.1 and 10, and "f_l" is the lateral compressive stress.

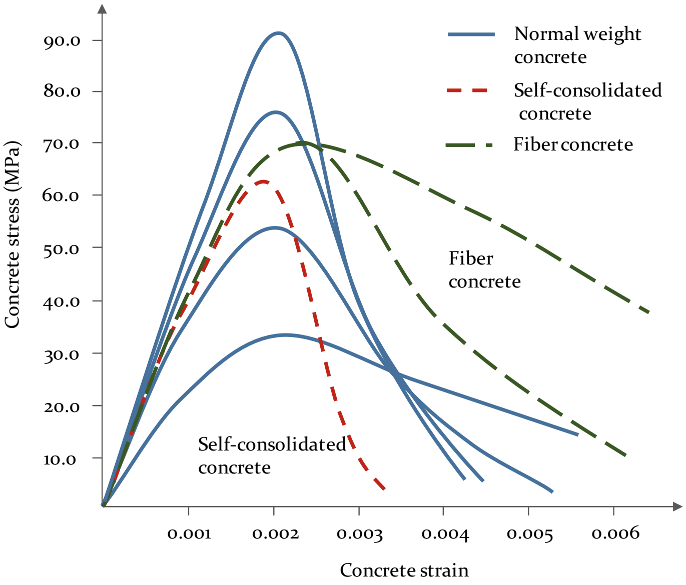

Fig. 1.1 Stress–strain of concrete in compression

1.1.1.1 Time Dependent Behavior of Concrete

The time dependent behavior of concrete is dominated by the dimensional variations due to creep and shrinkage. Creep of concrete occurs due to sustained loading, as the compressive strain increases with time while the compressive stress producing the strain is constant. Creep is usually considered in the design by modifying the elastic modulus using a creep coefficient, "ϕ", which depends on age at loading, size of the member and ambient conditions, in particular relative humidity. A typical behavior is that the concrete elastic modulus is reduced by time due to creep as expressed in Eq. (1.2).

$$E_c(t, t_o) = \frac{E_c(t_o)}{1 + \phi(t, t_o)} \tag{1.2}$$

where ϕ (t, t_0) is a dimensionless creep coefficient, "t_o" is the age at loading, "t" is the age at which the strain is calculated, E_c (t_o) and E_c (t, t_o) are the concrete elastic moduli at time "t_o" and "t".

It should be noted that the coefficient "ϕ" represents the ratio of creep to the instantaneous strain, as shown in Fig. 1.2; its value increases with the decrease of age at loading "t_o" and the increase of the length of the period $(t - t_o)$ during which the stress is sustained. When, for example, "t_o" is one month and "t" infinity, the creep coefficient may be between 2 and 4 depending on the quality of concrete, the ambient temperature and humidity as well as the dimensions of the element considered.

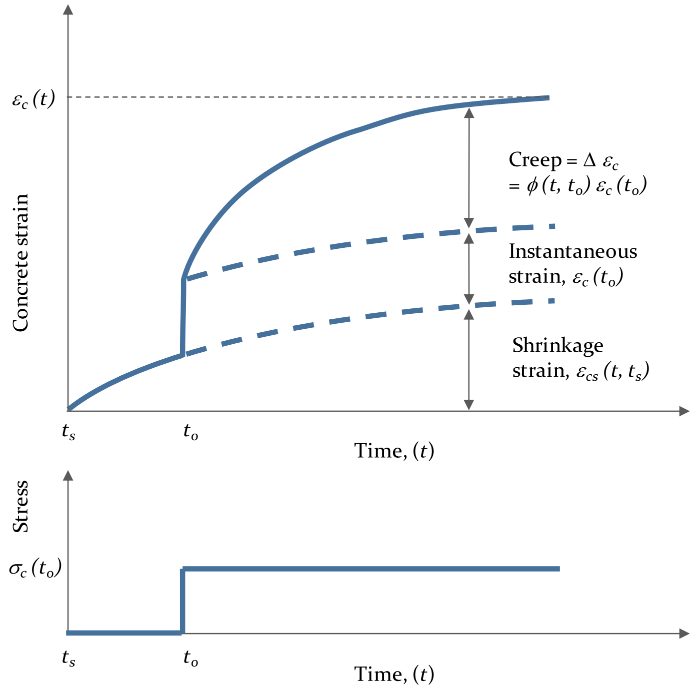

Fig. 1.2 Creep and shrinkage of concrete

Drying of concrete in air results in shrinkage, and if the change in volume is restrained, stresses develop. In reinforced concrete structures, the restraint may be caused by the reinforcing steel, by the supports or by the difference in volume change of various parts of the structure. Shrinkage starts to develop at time "t_s" when moist curing stops, as shown in Fig. 1.2. The strain that develops due to free shrinkage between "t_s" and a later instant "t" may be expressed as in Eq. (1.3).

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cs0}\beta_s(t - t_s) \tag{1.3}$$

where "ε_{cs}" is the free (unrestrained) strain due to shrinkage, "ε_{cs0}" is the total shrinkage that occurs after concrete hardening up to time infinity, where it depends upon the quality of concrete and the ambient air humidity. The function $\beta_s\ (t - t_s)$ depends upon the size and shape of the element considered. The free shrinkage, ε_{cs} $(t_2,\ t_1)$ occurring between any two instants "t_1" and "t_2" can be determined as the difference between the two values obtained by Eq. (1.3), substituting "t_2" and "t_1" for "t".

Stresses caused by shrinkage are generally reduced by the effect of creep of concrete (Ghali and Favre 1986); thus, the effects of these two phenomena must be considered in the analysis. Creep and shrinkage effects may be greater in prestressed than in non-prestressed concrete structures because of the prestressing forces and because prestressed structures typically have less bonded reinforcement.

Volume variation also occurs under the effect of change in the ambient temperature, where concrete expands and contracts with the temperature rise and fall. Temperature changes may be also caused by cement hydration at early age of concrete. Concrete coefficient of thermal expansion ranges from (8–12) $\times$ 10^{-6}/°C with an average value of 10 $\times$ 10^{-6}/°C. This coefficient varies primarily with aggregate type (limestone, siliceous gravel, granite, etc.); other factors also affect the coefficient of thermal expansions such as cementitious material content, water-to-cement ratio, concrete age and relative humidity.

1.1.2 Reinforcement Characteristics

Different types of reinforcements are used to strengthen concrete members, where it carries axial forces induced either from external loads or due to volume change of concrete. The most commonly used reinforcements are prestressed and non-prestressed steel reinforcement as well as carbon- and glass-fiber reinforced polymers, (CFRP and GFRP). Besides that it does not corrode, FRP reinforcement is characterized by its high strength-to-weight ratio, where its strength is up to 1.5 times the strength of the prestressing steel and its weight is about 20–25% that of steel, which makes its installation easy and fast. FRP reinforcement is a composite material made of fibers bound together with resin such as epoxy or vinylester. Other materials such as aramid and basalt FRP, micro-composite multi-structural formable steel, (MMFX) and stainless steel can be also used as reinforcement provided that its properties, bond characteristics and durability are well known. Understanding the reinforcement behavior is essential to properly design the strengthening scheme of concrete members.

The stress–strain behavior of commonly used reinforcements is given in Fig. 1.3. The prestressed and non-prestressed steel reinforcement have the same elastic modulus (195–210 GPa); however, the ultimate strain is much less for the prestressing steel, where it ranges between 3.5 and 4% compared to more than 10% for the non-prestressed steel. The ultimate tensile strength of the seven-wire prestressing steel ranges between 1770 and 1860 MPa.

Major difference exists between the tensile behavior of steel and FRP, where yield plateau exists for steel while linear brittle behavior exists for the FRP material. The tensile strength of the CFRP and GFRP is higher than that of the non-prestressed steel. The elastic modulus of the CFRP strips and high-modulus CFRP sheets are close to that of the steel (165–230 GPa), while it is much lower for GFRP. Normal-modulus CFRP sheets (using laminate properties) have much lower elastic modulus compared to steel (around 60–65 GPa). Unlike steel, FRP reinforcement has no universal standards in terms of shape, where it may be produced in circular bars with varying diameters, or laminates as rectangular strips (1.0–1.4 mm) or thin sheets (0.1 mm and more), as shown in Fig. 1.4. Nominal dimensions and surface roughness

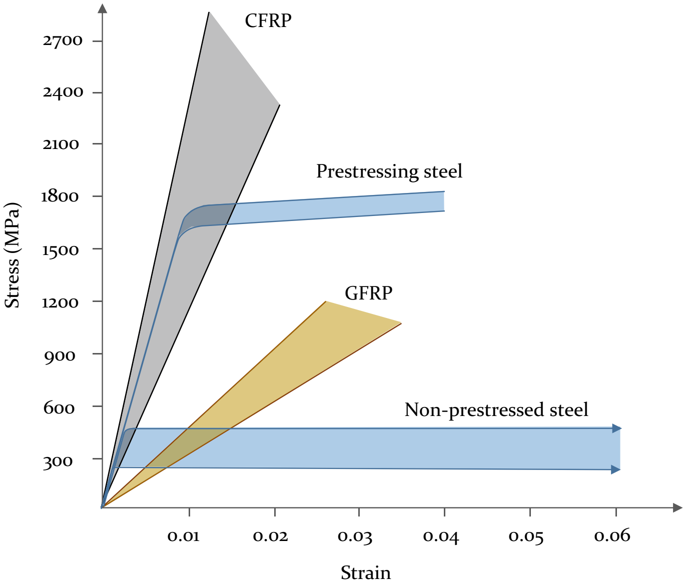

Fig. 1.3 Idealized tensile stress–strain behavior of different reinforcements

are also not unified, where smooth, indented and sand-coated surface are used for the FRP bar reinforcement. Therefore, bond characteristics of FRP bar reinforcements should be identified for each product individually. The rectangular strips are made of unidirectional fibers, while the sheets may be made of uni- or multi-directional fibers. FRP sheets are normally used as externally bonded reinforcement for concrete elements, where epoxy is usually used to adhere the laminates to the concrete surface.

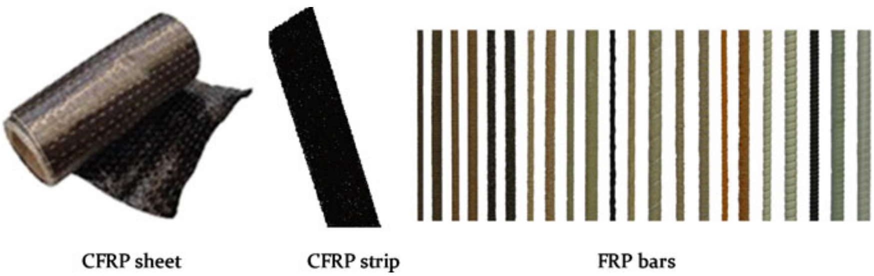

Fig. 1.4 Different shapes of FRP reinforcement

1.1.2.1 Time Dependent Behavior of Reinforcement

Tension force in the prestressing steel is reduced by time due to relaxation, which is defined as the loss in axial stress under constant strain. The time-dependent losses change with the type of prestressing steel and increase with the increase of the initial stress in the tendon. Unlike prestressing steel, FRP may experience creep rupture by time. When FRP materials are subjected to a sustained load, it can suddenly fail after a time period referred to as the endurance time. As the ratio of the sustained tensile stress to the short-term strength of the FRP reinforcement increases, endurance time decreases. The endurance time also decreases under adverse environmental conditions, such as high temperature, UV radiation exposure, high alkalinity, wet and dry cycles, or freezing-and-thawing cycles. In general, carbon fibers are the least susceptible to creep rupture, aramid fibers are moderately susceptible, and glass fibers are most susceptible.

To avoid creep rupture of the FRP reinforcement under sustained stresses or failure due to cyclic stresses and fatigue of the FRP reinforcement, the stress in the FRP reinforcement under these stress conditions should be limited. The stress in the FRP reinforcement can be computed using elastic analysis and an applied moment due to all sustained loads (dead loads and the sustained portion of the live load) plus the maximum moment induced in a fatigue loading cycle. The sustained stress should be limited as expressed in Table 1.1.

Unlike steel reinforcement, FRP is a non-isotropic material; i.e., its mechanical and thermal properties are not the same in all directions. The axial coefficient of thermal expansion for CFRP is -1.0×10^{-6} to 0/°C, while the transverse coefficient of thermal expansion is much higher and may reach 50×10^{-6}/°C, since it depends on the resin properties. The axial coefficient of thermal expansion for GFRP ranges from 6×10^{-6} to 10×10^{-6} to 0/°C, while the transverse coefficient of thermal expansion is much higher and may reach 23×10^{-6}/°C. Steel reinforcement has a coefficient of thermal expansion of 12×10^{-6}/°C, which is similar to that of concrete. Therefore, steel reinforcement is compatible with concrete compared to FRP reinforcement since there will not be induced stresses at the reinforcement interface when the concrete member is subjected to elevated temperature.

1.2 Concrete Evaluation

Concrete evaluation is accomplished using different techniques starting from visual inspection, concrete testing, which includes chemical composition, core test, rebound

Table 1.1 Stress limit in FRP reinforcement under sustained and cyclic loading (ACI 440.2R-17)

Stress type	Fiber type	
	CFRP	GFRP
Sustained plus cyclic stress limit	$0.55 f_{fu}$	$0.20 f_{fu}$

hammer, acoustic and infrared scanning, pull-out, and testing of steel bars to measure its tensile strength, modulus and bond with concrete. Description of the material testing can be found elsewhere (ACI 228.1 R-19 & ACI 228.2R-13) and is out of this book's scope. Yet, interpretation of the test results of concrete cores is an important step that has a great effect on the entire evaluation process and shall be discussed later. Concrete evaluation should give a complete understanding of the physical properties of the materials, which shall be used in the analysis of the structure. Concrete integrity should be also assessed to identify the level of cracking for better modeling of the relative stiffness between members. After endeavor of concrete evaluation, understanding of the structure behavior still relies on understanding of the expert engineer of how to model the different elements and connections to calculate the straining actions induced from loads and restraints.

1.3 Calculation of In-Situ Concrete Compressive Strength

Core testing is the most reliable method to measure the in-situ compressive strength of a concrete member since it involves direct compression test on samples extracted from the members under investigation. Core testing is endorsed either for suspect concrete in new structure or to evaluate existing structures based on the actual in-place concrete strength. The ACI318-19 specifies that "*Concrete in an area represented by core tests shall be structurally adequate if the average of three cores is equal to at least 85% of* f'_c *with no single core less than 75% of* f'_c". This criterion is considered only for new structures as an acceptance limit, which is quite different from that recommended for the calculation of concrete strength in existing structures.

Usually, there is a large scatter between the results of core tests of the same concrete structure, which should be carefully interpreted and statistically evaluated. In an early study, variation of 11% from the average concrete strength of core test results was recorded for a single batch of concrete (Bartlett and MacGregor 1996). The large variation in concrete strength could be due to many reasons including existence of micro-cracks, prestrain in the concrete, handling of specimens, curing of concrete, systematic and random variation in concrete strength of one or several batches in addition to effects of size, aspect ratio, direction of extracting the specimen and moisture condition. Accordingly, the engineer may use different concrete strengths for different zones in the structure depending on the concrete quality for better representation of the in-situ compressive strength. These zones may be categorized after visual inspection, review of the concrete casting history or performing rebound hammer tests on several concrete elements and grouping their results to characterize the concrete strength in the different zones. The categorized zones may be for different floors, or type of members, or concrete batches depending on the level of variation in concrete strength and confidence in the quality of construction.

1.3.1 *Interpretation of Core Test Results*

Statistical analysis of core strength data is complicated by the large scatter of the test results and is done only for the properly conducted tests of cores taken for each zone with homogeneous material in the structure. In the meantime, tests with considerably low strength should not be overlooked and the reasons for low strength should be identified. In case that the low strength is due to defects in the in-situ concrete and not due to error in the test procedure, measures should be taken for those members with low compressive strength and repair of the defected member is necessary.

For evaluation of concrete structures, ACI214.4-R-10 2016 introduces two methods for the calculation of the in-situ concrete compressive strength out of the core test results. The calculated equivalent design strength is the lower tenth percentile of the in-place strength and is consistent with the statistical description of the specified strength of concrete, f'_c as per the ACI 318–19 2019.

1.3.1.1 Method 1: Tolerance Factor Approach

The mean in-place strength is obtained from Eq. (1.4), while the sample standard deviation of the in-place strength, "s_c" is obtained from Eq. (1.5).

$$\overline{f}_c = \frac{1}{n}\sum_{i=1}^{n} f_{ci} \tag{1.4}$$

$$s_c = \sqrt{\sum_{i=1}^{n} \frac{(f_{ci} - \overline{f}_c)^2}{(n-1)}} \tag{1.5}$$

where n is the number of cores, and f_{ci} is the equivalent in-place strength of an individual core specimen.

The standard deviation of the in-place strength due to the empirical nature of the strength correction factors shall be obtained from Eq. (1.6), while the equivalent in-place compressive strength of concrete, $\overline{f}_{c,eq}$, , is calculated from Eq. (1.7).

$$s_a = \overline{f}_c\sqrt{V_{l/d}^2 + V_{dia}^2 + V_{mc}^2 + V_d^2} \tag{1.6}$$

$$\overline{f}_{c,eq} = \overline{f}_c - \sqrt{(Ks_c)^2 + (Zs_a)^2} \tag{1.7}$$

where $V_{l/d}$, V_{dia}, V_{mc}, V_d are coefficients, depending on height-to-diameter ratio, core diameter, moisture content and damage due to drilling, respectively. $V_{l/d}$ and V_{dia} can be estimated from Figs. 1.5 and 1.6 depending on the *l/d* ratio and core diameter, respectively. V_{mc} and V_d can be taken as 0.0025, as defined in ACI 214.4 R-10.

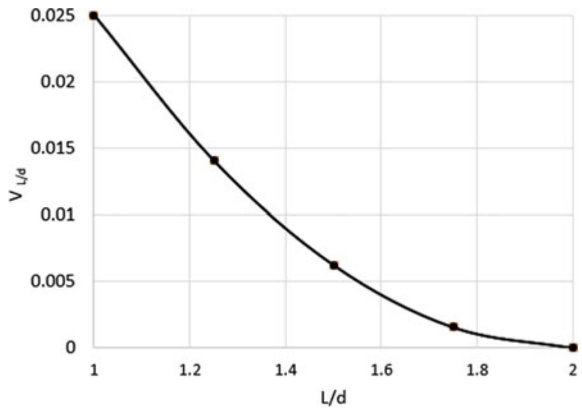

Fig. 1.5 Variation of $V_{l/d}$ with *l/d* ratio

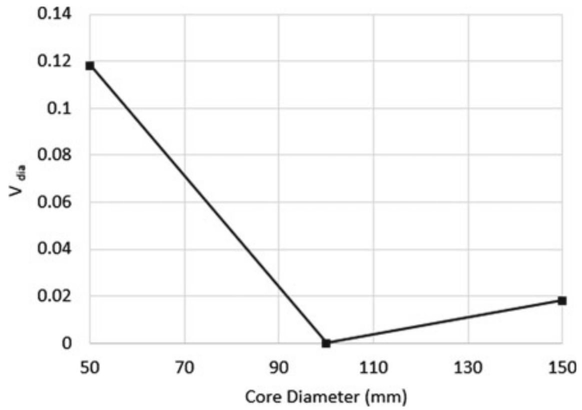

Fig. 1.6 Variation of V_{dia} with core diameter

K and *Z* are factors depending on the required confidence level as depicted in Fig. 1.7. Normally, a confidence level of 75% level is adopted for ordinary structures, 90% for important buildings and 95% for crucial components in nuclear power plants (ACI 214.4 R-10).

1.3.1.2 Method 2: Alternate Approach

The tolerance factor approach may be conservative in practice since core tests tend to overestimate the true variability of the in-place strengths. In the alternate approach by ACI 214.4R-10, the equivalent specified strength is estimated using a two-step calculation. First, a lower-bound estimate on the average in-place strength is determined from the core data. Then the 10% fractile of the in-place strength, which is equivalent to the specified strength, is obtained. The lower-bound estimate of the mean in-place strength, $(\overline{f}_c)_{CL}$, and the equivalent in-place compressive strength of concrete, $\overline{f}_{c,eq}$, , are calculated using Eqs. (1.8) and (1.9).

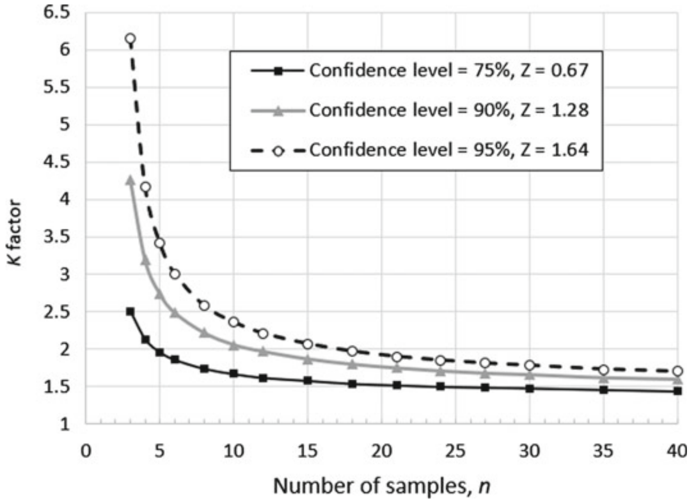

Fig. 1.7 *K* and *Z* factors depending on the required confidence level

$$(\overline{f}_c)_{CL} = \overline{f}_c - \sqrt{\frac{(Ts_c)^2}{n} + (Zs_a)^2} \tag{1.8}$$

$$\overline{f}_{c,eq} = C(\overline{f}_c)_{CL} \tag{1.9}$$

The first term under the square root of Eq. (1.5) represents the effect of the sample size on the uncertainty of the mean in-place strength. The factor "*T*" depends on the confidence level, as shown in Fig. 1.8.

"C" factor depends on the number of batches, number of members and type of construction as shown in Fig. 1.9

In case material properties cannot be evaluated from drawings, specifications or other documents, ACI 562 R-16 provides historical data that can be used in the USA as a guide to evaluate concrete properties. It shall be noted that the minimum concrete reported strength can be as low as 7 MPa, as shown in Fig. 1.10.

It should be noted that for the same concrete quality, results of the core tests may vary between the different members or within the same member. For example, Khoury et al. 2014 concluded that for the same concrete batch, results of the core tests extracted from the lower parts of the columns are 10–20% higher than those extracted from upper part depending on water-to-cement ratio (w/c). Figure 1.11 reveals a change in the concrete strength measured from core tests along the column's height.

Example 1.1 Table 1.2 shows core test results, measured for the in-place strength of concrete samples extracted from the ground floor in building "*A*". It is required to calculate the equivalent in-place compressive strength of concrete, $\overline{f}_{c,eq}$.

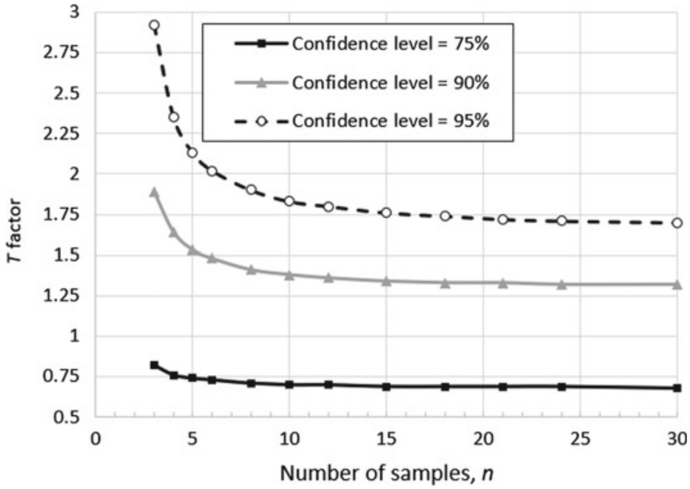

Fig. 1.8 "*T*" factor depending on the required confidence level

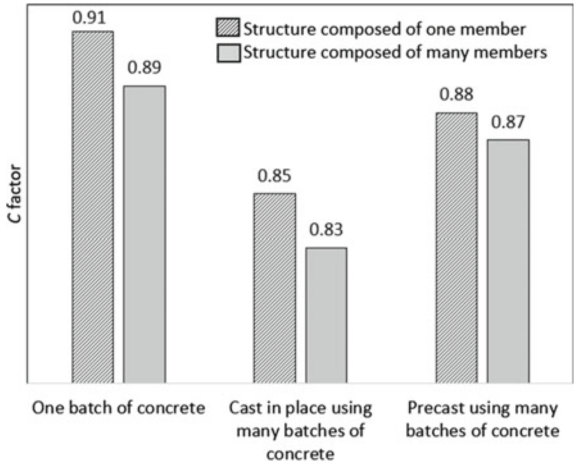

Fig. 1.9 "*C*" factor as per ACI 214.4 R-10

Answer

Figure 1.12 shows a plot of the core test results with a large scatter of data that ranges from 19 to 57 MPa, which is not recommended to be used as one set of data to calculate one value for the design concrete strength of all the structural members in the ground floor. The core test results shall be divided into two groups; where the first group shall be for the columns and walls and the second group shall be for the slab. The core test results for the slab-on-ground and grade beams shall not be included in the calculation of the design strength of the previous two groups. The two approaches described in the previous section were used to estimate the in-place

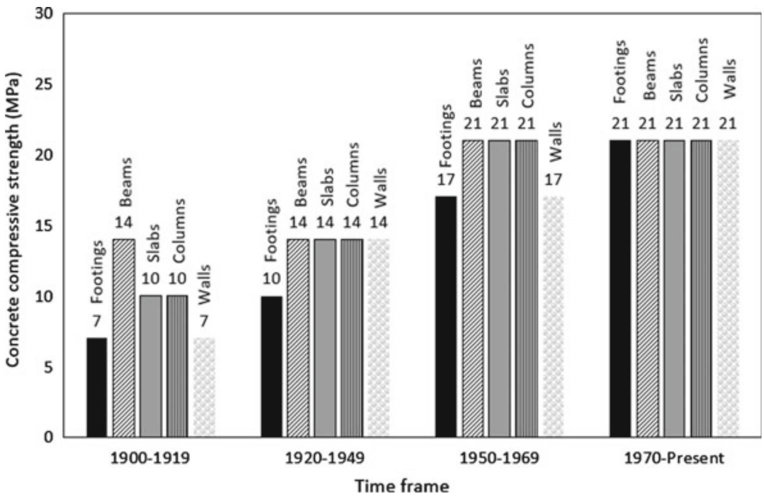

Fig. 1.10 Default concrete compressive strength for structural concrete as per ACI 562 R-16

Fig. 1.11 Strength variation through column's height (Khoury et al. 2014)

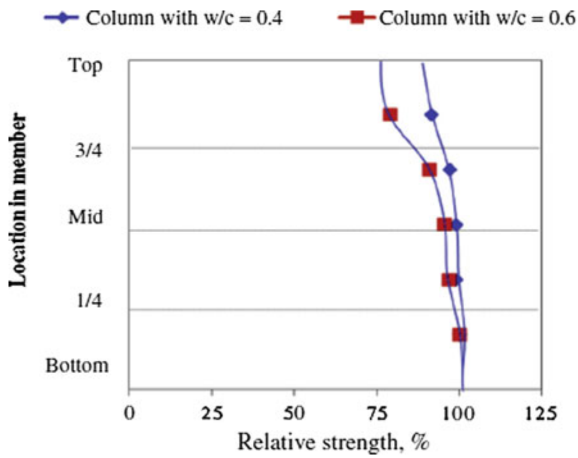

Table 1.2 Core test results of building A

Structural element	Results (MPa)
Columns at ground floor	34.4, 56.8, 44.6, 41.9, 45.9, 35.8
Walls at ground floor	32.9, 32.1, 31.7
Ground floor ceiling slab	29.5, 26.6, 19, 21.4, 23.8
Slab-on-ground	39.0, 33.8
Grade beams	42.8, 32.5, 33.0

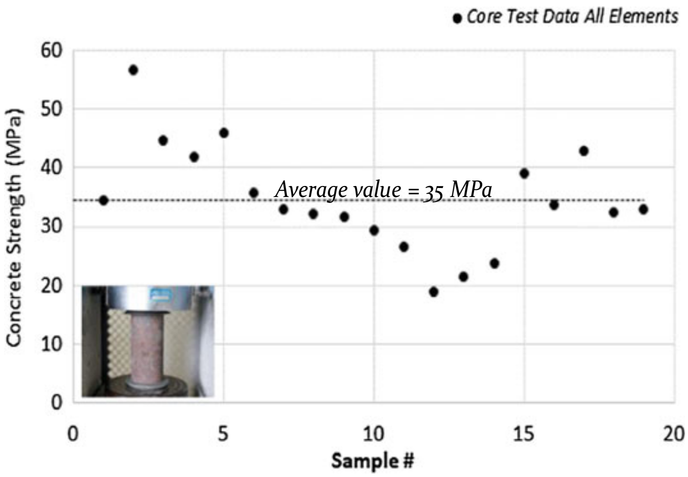

Fig. 1.12 Core test results of building "*A*"

concrete compressive strength for vertical elements and concrete slab using 90% confidence level. Calculations and results of the analyses are plotted in Table 1.3 and in Figs. 1.13 and 1.14.

The lower bound of the equivalent design strength is 21 and 13 MPa for the columns and slab, respectively. The calculated upper bound of the equivalent in-place cylinder strength is 28 MPa for columns and walls and 19 MPa for the ceiling slab of the ground floor, which is recommended to be the concrete strength used in the design check of the elements.

It shall be noted that all data has to be checked for outliers as per ASTM E178 based on the calculated standard deviation.

1.4 Structure Modeling

Three-dimensional finite element analysis of concrete structures is now customary, especially with the availability of many easy-to-use software programs using complicated and accurate techniques. Yet, hand calculations are essential in order to estimate the straining actions and preliminary dimensions of the structural strengthening. The load path should be clearly identified, especially when any changes in the structure supporting elements are made due to architectural requirements or for the purpose of relieving the overstressed members since it may affect the strength of other members in the structure. For example, adding inclined members to reduce the axial load in columns or to reduce the span of the concrete beam, as shown in Fig. 1.15, shall result in axial tension, which shall affect the shear strength of the beam. The new inclined member shall also change the magnitude and probably the direction of bending moments in the beam. In this regards, the engineer should identify the load path and

Table 1.3 Analysis of core test results

Structural element	Results (Mpa)	n	L/d	V_l/d	V_d	V_{mc}	V_d	S_a	S_c	z	AVG	k	(kS_c)^2	$(Z_{sa})^2$	T	$(TS_c)^{2/n}$	C	f'_c CL	f'_c 0.1	f'_c Tolerance approach	f'_c Alternate approach
Columns	34.4	9	1.81	0.000865	0.059	0.025	0.025	2.721666	8.447	1.28	**39.6**	2.14	326.78	12.136	1.395	15.429	0.83	34.32	21.5	**21**	**28**
	56.8		1.92	0.000168	0.059	0.025	0.025														
	44.6		1.60	0.004002	0.059	0.025	0.025														
	41.9		1.93	0.00013	0.059	0.025	0.025														
	45.9		1.91	0.000226	0.059	0.025	0.025														
	35.8		1.89	0.000286	0.059	0.025	0.025														
Walls	32.9		1.77	0.001288	0.059	0.025	0.025														
	32.1		1.94	8.12E-05	0.059	0.025	0.025														
	31.7		1.94	9.92E-05	0.059	0.025	0.025														
Slabs	29.5	5	1.74	0.001742	0.059	0.025	0.025	1.654927	4.147	1.28	**24.1**	2.74	129.12	4.487	1.53	8.052	0.91	20.52	12.7	**13**	**19**
	26.6		2.01	9.00E-07	0.059	0.025	0.025														
	19		2.05	6.50E-05	0.059	0.025	0.025														
	21.4		2.01	2.50E-06	0.059	0.025	0.025														
	23.8		2.06	0.000119	0.059	0.025	0.025														

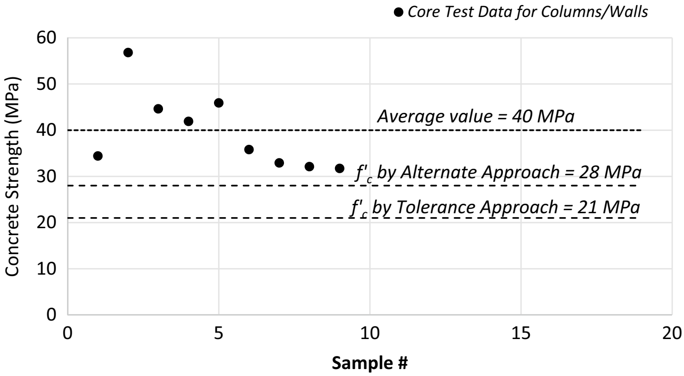

Fig. 1.13 Core test results of columns and walls

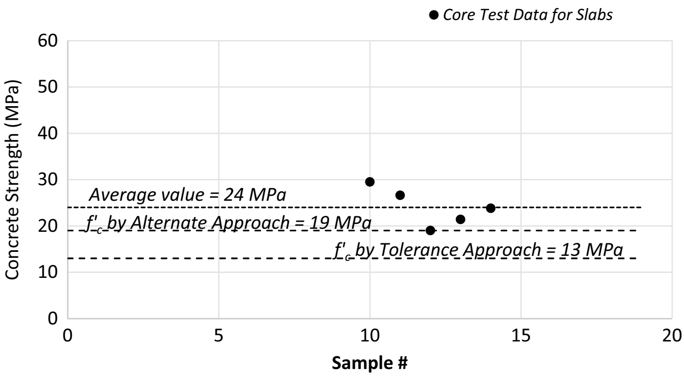

Fig. 1.14 Core test results of slab ceiling

validate all the changes in the straining actions as a result of addition or omission of any member in the structure. Another example is for buildings where vertical elements are under-designed and as a result and due to concrete creep, vertical loads were shared by existing blockwork, which was not originally designed as structural members. In this case, removal of non-structural walls may cause risk in the structure since the walls changes to be structural with time.

In general, challenges facing the structural engineers when modeling concrete structures either for evaluation purpose or for design of the strengthening scheme are (1) to assume the relative stiffness between members, (2) include the construction and loading sequence, (3) model the concrete members subjected to volume change, (4) modeling of the connections and (5) uncertainty of the physical properties of concrete members.

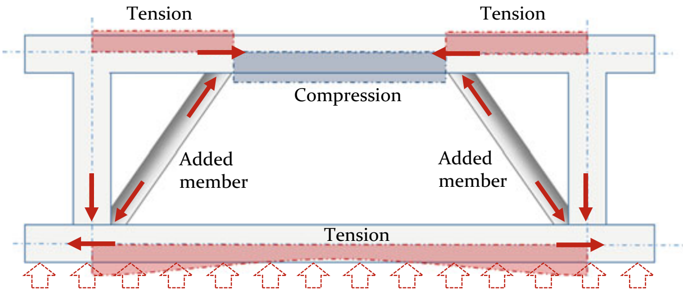

Fig. 1.15 Load path in strengthened structure

1.4.1 Relative Stiffness Between Members

Design parameters of concrete elements used in the model should represent the actual condition of the existing structure to reasonably estimate the relative stiffness between members and calculate the straining actions in the structure. Concrete elastic modulus and effective area and inertia of each section are the most effective parameters that have a direct impact on the distribution of straining actions between members. Modeling of the connections, whether it is pinned, fixed, partially fixed or allowing movement or rotation in one or more than one direction is also an important parameter in the modeling.

Distribution of the straining actions due to gravity loads shall differ based on the relative stiffness between elements; yet, the total actions shall remain the same regardless of the stiffness reduction. This is not true for the straining actions resulting from volume change, where the induced straining actions in the members and the total actions shall change with the change of the relative stiffness between members. The ACI318-19 recommends reduction in the moment of inertia to account for concrete cracking and cross-sectional area for concrete members subjected to gravity and lateral loads. ACI 224.2 R-92 provides recommendations for the effective concrete area when subjected to axial tension forces. Table 1.4 recommends design values for the inertia and cross-sectional areas to be used in analysis and design of structures. The recommended design values for axial stiffness of slabs and beams are mainly when those members are subjected to axial tension that may result from deformational restraint.

1.4.2 Construction Sequence of Rehabilitation Works

Strengthening of structural members by addition of other elements attached to the original concrete section may be applied using passive or active techniques. In the passive technique, new element is attached to the original section, while it is already stressed under the effect of its self-weight and other gravity loads. The original

Table 1.4 Recommended design values for moment of inertia and sectional area of concrete members

Member and condition	Moment of inertia	Cross-sectional area for axial deformation	Cross-sectional area for shear deformation
Columns	0.7 I_g	0.7 A_g	$b_w\ h$
Uncracked walls Cracked walls	0.7 I_g 0.35 I_g	0.7 A_g 0.6 A_g	
PT Beams	0.7 I_g	0.7 A_g	
Beams	0.35 I_g	0.5 A_g	
PT slabs	0.5 I_g	0.5 A_g	
RC slabs	0.25 I_g	0.25 A_g	

section shall also carry alone the weight of the new element used in strengthening. The new composite member shall participate in carrying the additional loads on the structure added after completion of the strengthening works. Algebraic superposition of the stresses shall be made to calculate the final stresses in the original and new section, as shown in Fig. 1.16. By time, the distribution of stresses shall change in the composite section since part of the increased strains in the original section due to creep shall be transmitted to the new material.

In active strengthening, either upward jacking of the structure or external prestressing shall be applied to relieve the stresses from the original section; then, the new composite section shall be resisting the full applied loads including self-weight of the members. In both passive and active strengthening, bond between the original and added section should be guaranteed to transmit the full horizontal shear at the interface efficiently without slip and to assume full composite behavior with the shown stress distribution in Fig. 1.16.

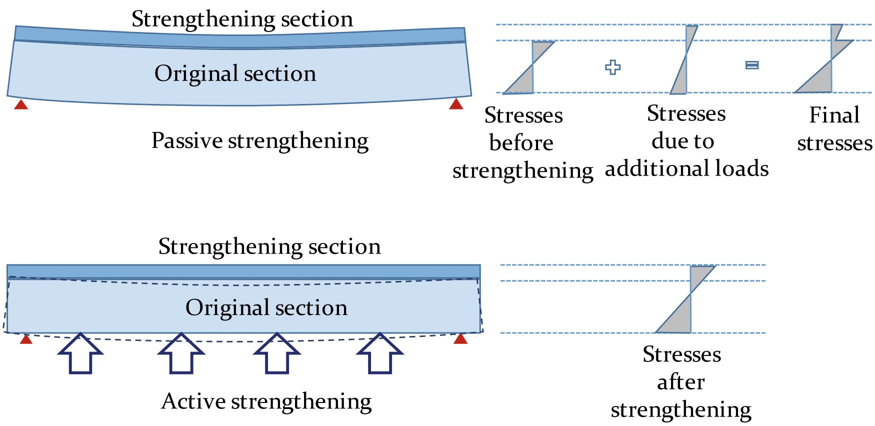

Fig. 1.16 Stresses in beams with passive and active strengthening

Modeling concrete structure to account for this construction sequence should be made to accurately calculate the stresses in the new sections. This may be done by creating more than one structural model and manually superimpose the stresses of each section.

1.4.3 Concrete Members Subjected to Volume Change

Concrete members are affected by volume change resulting from temperature variation, shrinkage, creep or prestressing, especially with the increase of the restraint provided by shear walls, cores or columns with large stiffness. Restraint of concrete members arises also by the internal steel reinforcement and with the increase of the structure dimension between joints. Despite that there is no general agreement on the design parameters of concrete sections subjected to this type of loading, attention should be given to the design assumptions since it will lead to a large deviation in the resulting straining actions. In most cases, the volume change shall result in axial forces in the slabs and bending moments in the vertical members, where considering the tension forces in the slab is vital to avoid large cracks and due to its negative impact on the shear strength of concrete members. Ignoring the tension forces induced from volume changes also risks the concrete slabs of initiating cracks affecting the overall durability of the structure.

One- and two-dimensional analyses of concrete members with pinned or fixed supports subjected to volume changes shall result in upper-bound straining actions in the slabs and floor beams. Three-dimensional analysis of structures, where the vertical members are represented in the model, shall result in more realistic straining actions, especially if the soil subgrade reactions are included in the three directions. Reinforced concrete members subjected to axial tension cracks once the concrete reaches its tensile strength leading to change in the axial stiffness; yet, the stiffness still higher than that of the bare steel bar due to tension stiffening, as shown in Fig. 1.17. Tension stiffening refers to the contribution of concrete between cracks at service loads. Prior to ultimate load, tension stiffening is completely lost and the full tension force is resisted by the steel reinforcement and the bare steel bar stiffness governs the behavior.

For concrete slab with 1% steel reinforcement ratio, the gross sectional area resisting tension before cracking is "A_c", where A_c is the concrete sectional area. At ultimate and after losing the tension stiffening, the equivalent concrete area resisting tension force is the area of steel reinforcement "A_s" multiplied by the modular ratio "n", where n equals (E_s/E_c). Assuming that the modular ratio is equal to "10", the equivalent area resisting tension at ultimate is approximately equal to "0.1 A_c". This means that the stiffness of concrete section subjected to tension forces drops to only 10% of the uncracked stiffness at ultimate. At service loads, it is reasonable to assume that the equivalent area of the concrete section is not less than 50% of the gross concrete cross section in order to limit the crack width. In addition, the concrete elastic modulus may be reduced under sustained load due to creep, as per

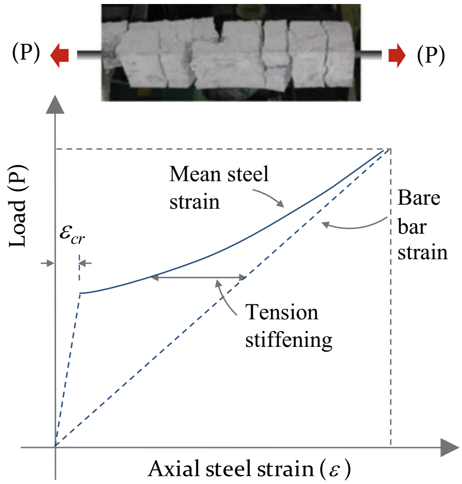

Fig. 1.17 Tension stiffening in concrete

Eq. (1.10).

$$E_{c(t)} = \frac{E_{ci}}{1 + \phi} \tag{1.10}$$

where E_{ci} and $E_{\mathrm{c}(t)}$ are the concrete elastic modulus at 28 days and at time (t) and ϕ is the creep factor. It is conservative to assume that the concrete elastic modulus is only 50% of the initial concrete modulus for members subjected to volumetric changes. Therefore, the axial tension stiffness of concrete section may be assumed as per Eq. (1.11).

$$(\mathrm{EA})_{\mathrm{T}} = (0.5E_{\mathrm{ci}})x(0.5A_{\mathrm{c}}) = 0.25E_{\mathrm{ci}}A_{\mathrm{c}} \tag{1.11}$$

1.4.4 Connections Modeling

Analysis of the connections in existing concrete structures is probably one of the difficult tasks since in-situ investigation of the reinforcement details and connections' stiffness is questionable. Structures made of precast concrete components are full of connections, which affects the overall stability and distribution of forces between the different elements. Assumptions of the connection properties based on visual inspection, common practice and laboratory testing of the used materials should be employed in the structural model. Strut-and-tie method is one of the best techniques to analyze connections since it is based on truss analogy, which can be applied at the discontinuity regions. Analysis using strut-and-tie model, (STM), requires the following:

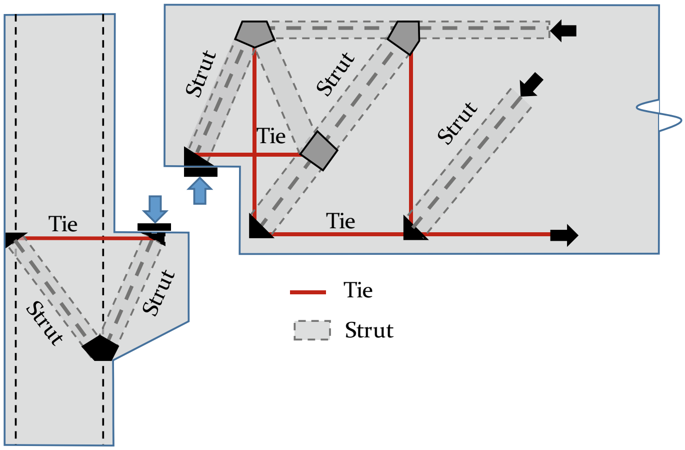

Fig. 1.18 Typical STM for short cantilever and dapped end beam

1. Define the "*B*-" and "*D*-" regions, i.e., beam and discontinuity regions.
2. Develop a STM; a truss system representing the stress flow in member and calculate the forces in the truss model.
3. Design the members of the STM—Design the compression struts and steel in the ties to resist the design forces as well as the stresses in the nodes.
4. Iterate the modeling for member optimization and to minimize strain energy.

Typical STM of short cantilever and dapped end beam are shown in Fig. 1.18. It should be noted that crack control reinforcement should be added independent from the strut-and-tie analysis in addition to anchorage of the tie reinforcement.

1.4.5 Uncertainty of the Physical Properties of Concrete Members

Evaluation of existing concrete structures requires that the engineer shall deal with the uncertainty in the design parameters inherent in the physical and material properties of concrete elements so that a safe and serviceable structure should be secured. This is in addition to the uncertainties in load values, load combinations and safety factors, which are also valid for new structures. The engineer should take all measures in order to build confidence in the design parameters, which include the following:

1. Use of available data including drawings, documents, material and soil testing results as well as available information given by the designer, contractor, authorities, neighbors, tenants and owner.
2. Observe symptoms of distress, cracking, deformation or any other deterioration of the concrete members.

3. Compare the material properties of the structure with commonly used properties in other structures built in the same era.
4. Conduct statistical analysis to calculate the lower and upper bound of parameters affecting the design. It is then the engineer decision to select a value in which he is confident in or perform sensitivity analysis for critical members. For example, prestressing forces in flexural members may be hard to measure after occurrence of long-term losses; yet, minimum and maximum values may be calculated and used in the evaluation.
5. Dynamic evaluation of the design parameters during construction, since new information may be encountered during application of repair or retrofit scheme.
6. Conduct load test of concrete floors (if needed); while it is not practical to perform in-situ testing of vertical members.

1.4.6 Archaic Structural Systems

Uncommon structural systems that have been developed at a certain era or in a certain place, may be utilized in an existing building that is under evaluation. Application of engineered procedure to understand the structural system requires first to understand both the architectural and structural historical background of the used construction technique. Once this system is acknowledged, it is then customary to find this technique recurrent with the same materials and dimensions for structures built in the same time frame. For example, floors in some buildings built in Europe and the Middle East in the beginning of the twentieth century are made of steel beams, which may be rail rods, supporting either plain concrete or brickwork arches, which in turn carries fill with variable thickness and wood floor, as shown in Fig. 1.19. The arch is mainly used to develop compression forces since the plain concrete cannot carry tension forces.

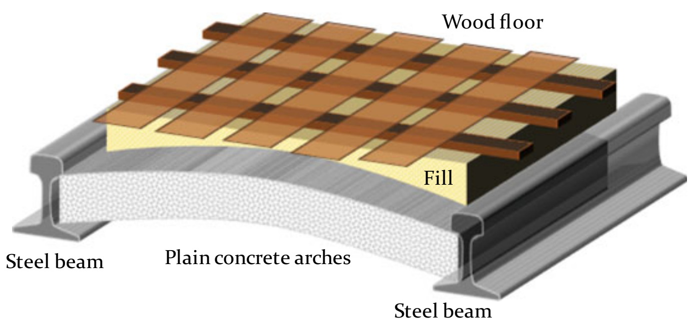

Fig. 1.19 Floor structural system

1.5 Design Guidelines

Design codes require that the demand-to-capacity ratio should be less than 1.00 for each member in order to have a safe structure. Selection of the design building code, based on which evaluation and/or rehabilitation of the structure shall be followed, is critical. The Code Requirements for Assessment, Repair and Rehabilitation of Existing Concrete Structures ACI562-16 specifies the following:

> Evaluations of damage and the limit for damage to be repaired using the original building code. The definition of "original building code" is the building code in effect at the time of original permitted construction. If this limit is exceeded or if the licensed design, professional judges the structural safety to be unacceptable based on rational engineering principles, rehabilitation is necessary in accordance with the requirements of the current building code, which is defined by the code which establishes the design and construction regulations for new construction.

If the demand-to-capacity ratio is less than 1.5 and there is a reason to question the structural safety due to structural defects or material deterioration, it shall be assessed using the original design building code with nominal loads and capacities to determine if the ratio exceeds 1.0. In this case, repair work shall be legitimate to restore the defected member/structure to its safe condition based on the materials of the original construction. Yet, if this ratio for a given member exceeds 1.5, the current design building code shall be the design basis criteria since no marginal of safety against failure exists. If the demand-to-capacity ratio does not exceed 1.0, no strengthening shall be required. It should be noted that mixing of load factors or load combinations from one code with strength reduction factors from a different code shall result in an inconsistent level of reliability.

In general, assessment should address if the demand or capacity of the original member or structure is considerably inconsistent with current codes and results in unacceptable structural safety. The structural engineer should check if there is a major change in the design criteria between the original and current building codes that lead to an unacceptable difference. Use of original building code is questioned in the assessment procedure of existing structures when differences with the current code incurred increase of applied loads, change in load factors, strength reduction factors or load combinations, modification of analytical procedures or major changes in the calculated capacity or in the cost savings.

During assessment of any "unsafe" structure, the design engineer should decide if it is for the benefit of the structure to include moment redistribution of flexural members. The ACI318-19 permits 7.5–20% moment redistribution provided that the net tensile strain in the steel reinforcement at flexural capacity is more than 0.0075, as shown in Fig. 1.20. Redundancy of the structural members, alternate load path, collapse mechanism, redistribution of superimposed dead and live loads and/or any restraint of the structure may be also included in the evaluation.

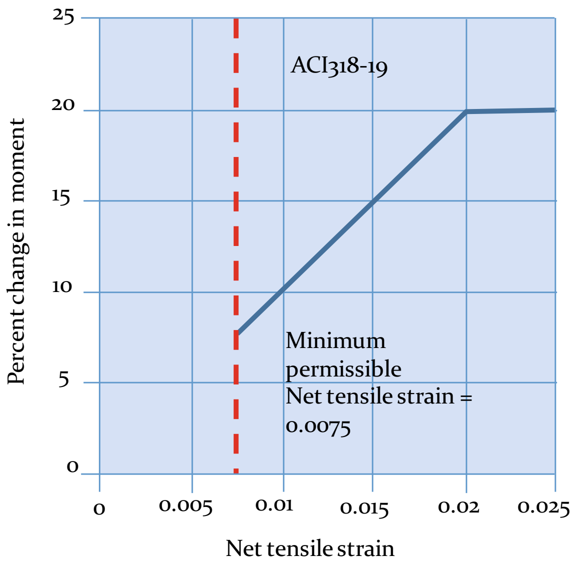

Fig. 1.20 Moment redistribution in flexural concrete members

1.5.1 Safety Factors

For evaluation of existing structures, the safety factors may be reduced using higher strength reduction factors (ϕ) provided that the as-built dimensions, location of steel reinforcement and material properties are verified at the critical sections on site. It should be noted that the reduced strength reduction factor shall not be used for the rehabilitation design. The ACI318-19 recommends the reduction of ϕ factors, as per Table 1.5.

Table 1.5 Reduced strength reduction factors for structure evaluation (ACI318-19)

Action		ϕ factor for new design	ϕ factor for structure evaluation
Axial compression, Flexure or combined action	Compression controlled members	0.65 for tied columns 0.75 for spiral columns	0.8 for tied columns 0.9 for spiral columns
	Tension controlled[a] members	0.9	1.0
Shear and torsion		0.75	0.8
Bearing		0.75	0.8

[a] For transition zone between compression and tension-controlled failures, see Fig. 1.21

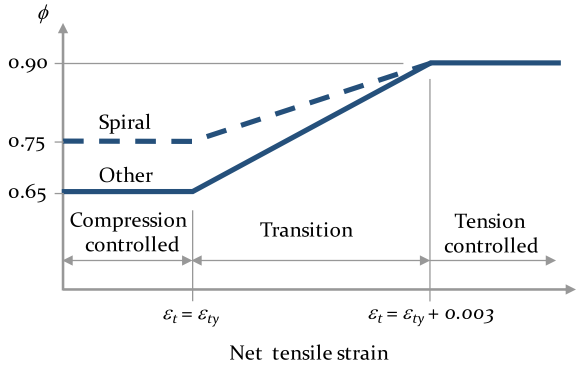

Fig. 1.21 Strength reduction factor (ϕ) with net tensile strain

For members strengthened with external reinforcing systems, such as external prestressing, externally bonded fiber reinforced polymer, (FRP) laminates, or externally bonded steel plates, the unstrengthened member should be checked to satisfy a minimum strength without the participation of the external reinforcing system, as per Eq. (1.12). This condition aims at minimizing the risk of overload or damage to the unstrengthened member during its service life due to loss of the external reinforcement as a result of fire, vandalism or collision.

$$\phi R_n \geq 1.1\,\mathrm{D} + 0.75\,\mathrm{L} + 0.2\,\mathrm{S} \tag{1.12}$$

where the live load factor may be increased to 1.0 instead of 0.75 for cases with live load higher than 4.8 kPa.

It should be noted that members subjected to only axial compression are considered to be compression-controlled and members subjected to only axial tension are considered to be tension-controlled. If the net tensile strain in the extreme tension reinforcement, ε_t, at nominal strength of the flexural member is sufficiently large ($\geq \varepsilon_{ty} + 0.003$), the section is defined as tension-controlled, where ε_{ty} is the yield strain of steel reinforcement but not more than 0.002. This relationship sets the reduction factor at 0.90 for ductile sections and 0.65 for brittle sections where the steel does not yield and provides a linear transition for the reduction factor between these two extremes. For tension-controlled members, warning of failure by excessive deflection and cracking and sufficient ductility for most applications may be expected. Compression-controlled members have less ductility and are more sensitive to variations in concrete strength.

1.5.2 Maximum Strengthening Ratio

Maximum strengthening limits should be imposed if failure of the strengthened concrete element is suspected to happen due to any reason under the effect of fire, vandalism, damage under certain environmental conditions, or other causes. Applying the maximum strengthening ratio ensures that the original member without strengthening shall not collapse in case that the strengthening scheme is completely lost. Using concrete jacket is the most reliable strengthening technique to guard against collapse of the strengthened member when subjected to any of those causes; hence, there are no strengthening limits in this case.

Physical and material properties of steel members are changed when subjected to fire, where reduction in the tensile strength starts at around 400 °C and the stiffness is reduced when temperature exceeds 150 °C. Due to the property changes, deformation, local buckling and twisting of the steel member can also occur. Strength of the steel connections and bolts is severely affected by fire and may result in catastrophic failure. In this respect, protection of the steel members against fire is mandatory to maintain the structure fire rating. Various approaches are available for fireproofing steel members such as encasement in concrete, which may be undesirable since it adds weight to the structure, a lath and plaster ceiling or spray-on materials such as mineral fibers, perlite, vermiculite or gypsum. This protection is usually enough not to impose limits on concrete members strengthened with steel sections. The decision shall then be made based on the constructability and cost–benefit of utilizing this technique in strengthening.

Corrosion protection of the steel section is also required, where painting is a practical and cost-effective way. Use of coating materials such as epoxies, or other mineral and polymeric compounds can be considered. Use of corrosion resistance steel such as weathering or galvanized steel is another alternative.

The most affected strengthening material with fire and vulnerable to damage under elevated temperature that can lead to loss of the strengthening scheme is FRP. One of the physical properties that should be carefully addressed when using FRP in strengthening is the glass transition temperature, T_g, which is defined as the temperature at or above which the molecular structure exhibits macromolecular mobility. At the glass transition temperature, the amorphous regions experience transition from rigid state to more flexible state making the material at the border of the solid state to rubbery state, as shown in Fig. 1.22. The glass transition temperature of the commercially available resins cured under ambient temperature ranges from 60 to 82 °C and may be increased to 160 °C when cured under elevated temperature. Those values are well below the temperature generated when fire occurs.

In this case, the maximum strengthening ratio is determined so that the unstrengthened structural member should have sufficient strength to resist a certain load level. The existing strength of the element should be sufficient to resist a level of load as described by Eqs. (1.13) and (1.14); therefore, the maximum strengthening ratio for members strengthened with FRP is around 40% more than that of the original section. More precisely Eq. (1.13) is to limit the strengthening ratio in case of loss of FRP due to bond failure, damage, vandalism, or other cause, while Eq. (1.14) is to safeguard the section from collapse in case of fire.

$$(\phi R_n)_{\text{existing}} \geq (1.1\text{SDL} + 0.75 \text{ to } 1.0\text{SLL})_{\text{New}} \tag{1.13}$$

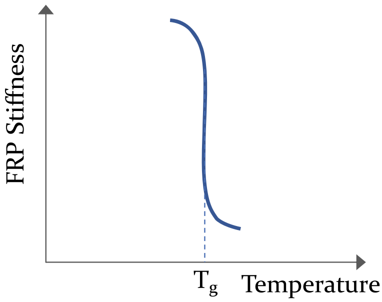

Fig. 1.22 Glass transition temperature

$$(R_{n\theta}) \geq (1.0\text{SDL} + 1.0\text{SLL})_{\text{New}} \tag{1.14}$$

where "R_n" is the nominal resistance of the original section, "$R_{n\theta}$" is the nominal resistance of the member at an elevated temperature, and SDL and SLL are the dead and live loads, respectively, calculated for the strengthened member. In Eq. (1.13), the live load multiplier is taken 0.75, unless the live load on the member has a high likelihood of being present for a sustained period of time; then, the live load multiplier should be taken equal to 1.0.

In case of FRP strengthening by more than 40% of the original section capacity is required, fire protection shall be provided using suitable materials. The materials shall prove adequate performance under fire by tests conducted at certified laboratories on full-scale models loaded under fire. The fireproofing materials should ensure that the temperature at the FRP surface shall not exceed the glass transition temperature of the matrix used in the FRP composite during fire, as shown in Fig. 1.23. Fire endurance of 1.0–1.5 h can be achieved and should be indicated on the design drawings for strengthening with a clear method of statement for testing. The structure should also have redundancy in transmitting the applied loads in case of allowing fireproofing materials to be used; i.e., exceeding the 40% strengthening limit is not recommended for cantilevers where redundancy is not achieved.

1.6 Case Study 1.1

This case study is for a building consisting of basement, ground, six typical floors and a roof. The basement is constructed on the full footprint of the land, while the building footprint is much smaller, as shown in Fig. 1.24. The basement ceiling is a post-tensioned concrete slab resting on post-tensioned wide beams from one side which in turn supported on a series of columns, and fixed to reinforced concrete retaining wall from the other side. The wall retains the earth from one side and is divided by expansion joints. The figure shows the building envelope, (in Blue), where the top surface of the basement slab is exposed to ambient temperature, while the underside of the slab and the inner side of the walls are not exposed.

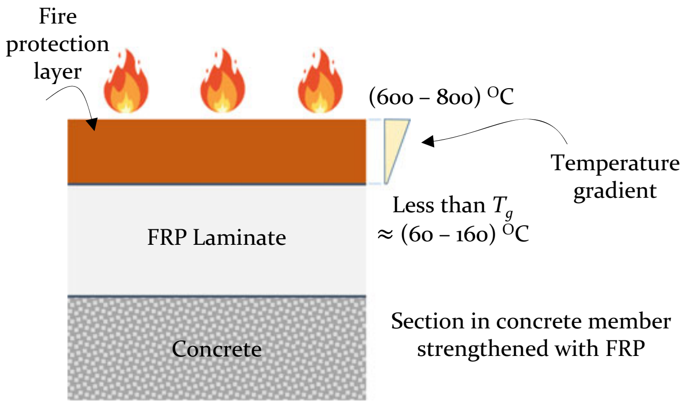

Fig. 1.23 Temperature gradient in FRP fireproofing material

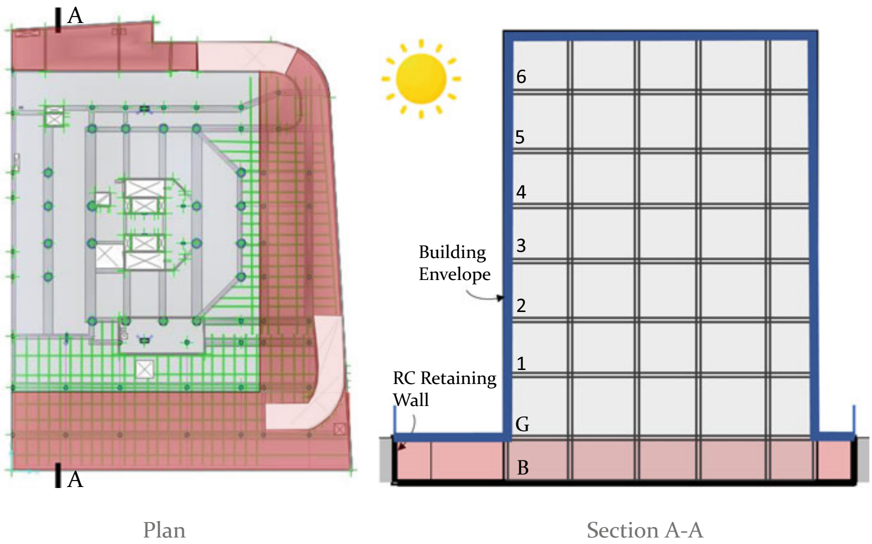

Fig. 1.24 Plan view and section of building (Case study 1.1)

1.6.1 Problem Statement

Concrete cracking in the basement ceiling exposed to ambient temperature was observed in the direction perpendicular to the reinforced concrete wall with a typical pattern along the slab, as shown in Fig. 1.25.

Analysis showed that the strength of PT slab can sustain safely the gravity loads resulting from dead and live loads. However, the concrete slab was also subjected to a large temperature variation. Since the concrete slab is fixed to the wall, which

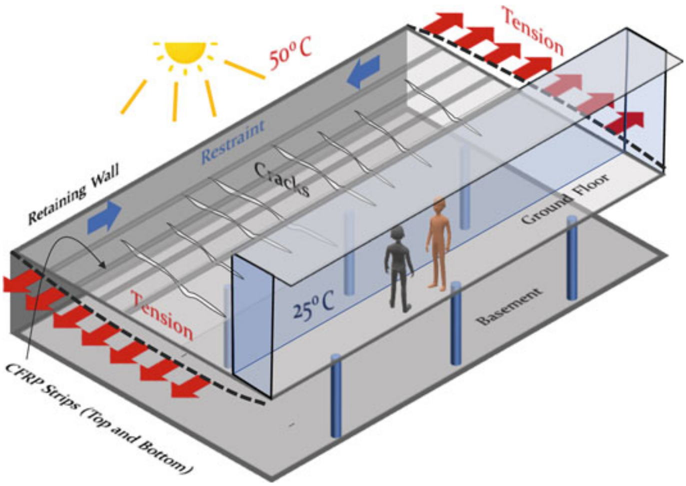

Fig. 1.25 Cracking in the PT slab

encounters high stiffness restraining the slab movement induced from the temperature variation, a large axial tension resulted from this restraint, as shown in the Figure. This tension in the slab led to patterned concrete cracking perpendicular to the retaining wall.

1.6.2 Strengthening Calculations

Three-dimensional model of the structure including the walls, columns and basement ceiling was used to calculate the additional straining actions in the slab due to temperature variation. It is not advisable to model concrete members subjected to tension forces using the gross concrete section properties since this will result in huge tension forces, which does not exist after occurrence of minor cracking in the slab, which in turn will reduce the member stiffness. It is also unwise to use the stiffness of the steel reinforcement only when modeling the concrete member subjected to tension since this will result in large concrete crack width. Therefore, the value of 50% of the gross member area was used combined with 50% of the short-term concrete elastic modulus, as per Eq. (1.8). The proposed strengthening scheme aimed at resisting the resulting tension forces in the concrete induced from temperature variation.

1.6.3 PT Slab Remedy

Strengthening the slab was carried out, first by injecting the cracks with low-viscous epoxy and adhering carbon fiber reinforced polymer, (CFRP), strips to the top and bottom surfaces of the concrete slab. The strips were bonded to the slab in the direction parallel to the wall and perpendicular to the existing cracks. This additional reinforcement stopped the development of further cracks and controlled the existing cracks in the slab.

1.7 Case Study 1.2

This case study is for a building consisting of two basements, ground and 20 typical floors with a footprint area of 2500 m^2. The structural system of the building consists of post-tensioned concrete flat slabs with 250 mm thickness supported on vertical concrete members, which are supported on concrete mat foundation. The lateral resisting structural elements consist of shear walls, cores and a series of columns. Several columns and walls in the building are planted and carried on adjacent columns through series of transfer walls, as shown in Fig. 1.26.

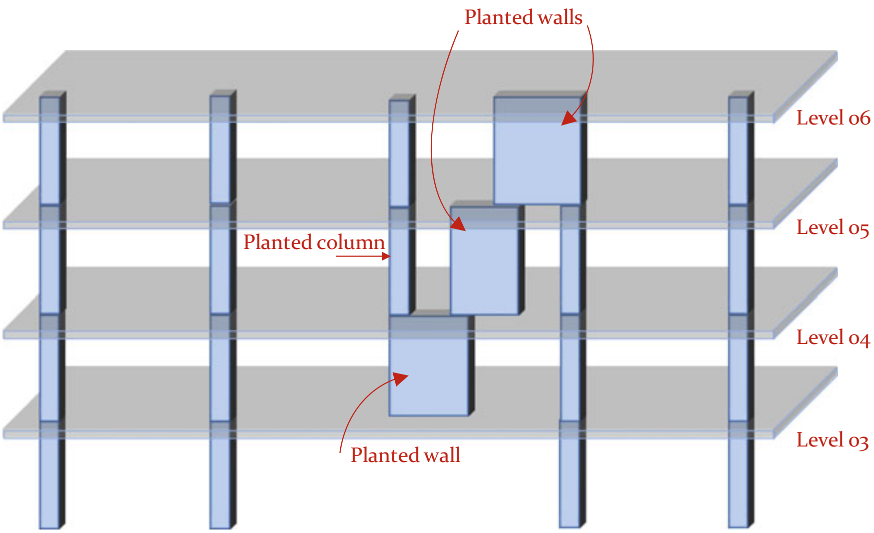

Fig. 1.26 Section elevation of the building (Case study 1.2)

1.7.1 Problem Diagnosis

Structural problems were recognized in the building as cracks were observed in the post–tensioned, (PT), concrete flat slab at different floors at the location of the flag/transfer wall, as shown in Fig. 1.27. Those cracks were typical at every floor, where planted columns/ walls exist. Added to the problem that no top steel mesh was used in the slab and only few steel bars were placed at the negative moment region, since the slab was post-tensioned, as shown in the Figure. Due to the absence of the top steel mesh, the observed cracks were as wide as 2.00 mm, as shown in Fig. 1.28. Cracks became wider as they extended in the middle of the slab away of the few top steel reinforcements placed at the column zone.

It was essential to study the overall behavior of the structure when checking the safety of each structural element in the building. Using 2D analysis is misleading since it overlooks the vertical deformations of the slab at the planted vertical elements. In order to consider the slab deformation at the column locations in the slab design check, 3D finite element model of the structure was used. The structural analysis of the building using the 3D model showed large deformation in the slabs at the location of the planted walls, as shown in Fig. 1.29. This is despite the fact that a tie existed to arrest the deformation of the slab as shown in the figure. This deformation was not considered in the first design of the PT slab, where 2D analysis was used.

Moreover, sequential construction was also considered in the analysis to account for the deformations of the vertical elements carrying lower floors, which already

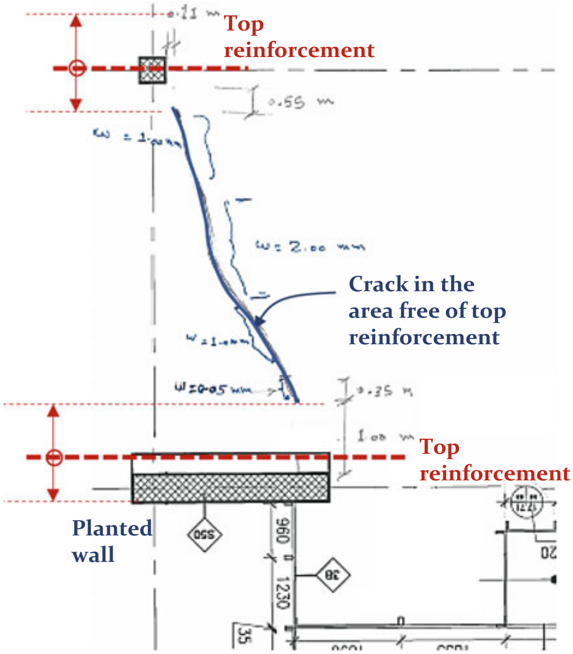

Fig. 1.27 Plan showing typical crack

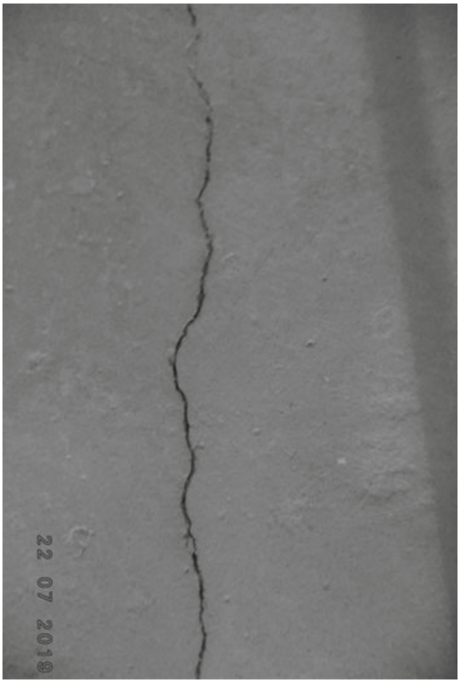

Fig. 1.28 Crack in the PT slab

occurred before casting the upper floors. The long-term deformations of the slab due to concrete creep and reduction of the flexural stiffness were also considered in the analysis using the modified concrete elastic modulus, as per Eq. (1.7) and 70% of the gross PT slab moment of inertia considering that the slab is designed not to crack under service loads.

1.7.2 Concrete Slab Repair

Since the cracks in the PT slab occurred as a result of the induced slab deformations after deflection of the planted columns, and due to the fact that there was no top reinforcement in the slab, crack widths were relatively large ranging from 1.0 to 2.0 mm. Despite that some design codes do not enforce adding top steel reinforcement in the PT slabs, providing this reinforcement has an important role to arrest any potential cracks that may be induced from unforeseen deformations resulting from unequal loads distribution, concrete shrinkage or temperature variations. Large cracks width will definitely affect the durability and overall integrity of the slab. It should be noted that some design codes recommend using minimum non-prestressed steel area in the PT slabs regardless of the stress level under service loading conditions (ECP 203–2020).

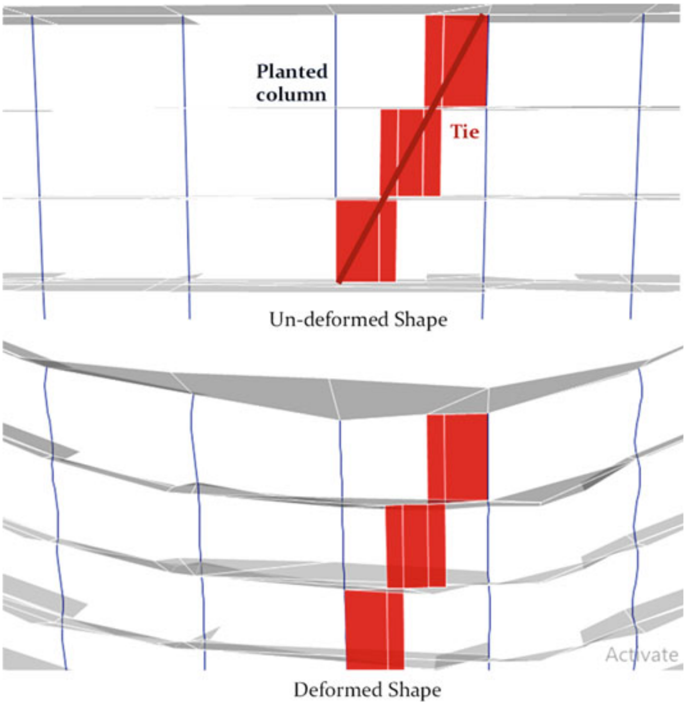

Fig. 1.29 Frame analysis of building zone with planted walls/columns

The procedure of repair started with monitoring of those cracks in order to investigate if the cracks are still live. After confirming that the cracks are stagnant and no propagation of the cracks was observed, injecting the cracks with low-viscous epoxy was recommended to seal the crack opening in order to regain its structural integrity. Strengthening of the slab was further implemented using externally bonded carbon fiber reinforced polymer, (CFRP), laminates on the top concrete surface at the crack location in order to arrest the cracks and preserve its durability aspects. Area of CFRP reinforcement was calculated so that it compensates for the tensile force generated in the slab after considering the induced deformations and secondary effects from concrete shrinkage and creep. It is important to consider the reduced section stiffness considering cracking and long-term concrete modulus when calculating the induced tensile forces in the concrete.

Chapter 2
Strengthening of Concrete Columns

Columns subjected to uniaxial compression fail due to lateral tensile stresses caused by Poisson effect resulting in lateral expansion, as shown in Fig. 2.1a. Poisson effect describes the expansion or contraction of a material in directions perpendicular to the direction of loading. If lateral expansion can develop freely, then the member is in uniaxial stress state. The resulting cracks are splitting tensile cracks parallel to the direction of loadings. This type of failure is usually observed for uniaxially loaded columns with low or normal compressive strength. Another mode of failure may be also observed, where inclined cracks occur, which was may be observed for columns with high concrete compressive strength, as shown in Fig. 2.1b.

2.1 Column Jacketing

Columns jacketing may be applied using concrete, steel or fiber reinforced polymer laminates. The jacketing aims at reducing the lateral expansion of repaired columns and as the stiffness of the jacket in the transverse direction increases, the lateral deformation of the column decreases. If the lateral expansion is completely restrained, i.e., no lateral deformation develops, then the member is in uniaxial deformation state. This is impractical since the stiffness of the jacket can never be infinite. Therefore, jacketed columns are always under triaxial state of stress and deformation. As the stiffness of the jacket increases, high tensile stresses are induced in the jacket and vice-versa, high confining pressure is induced on the original column section and the resulting lateral deformation becomes less. It should be noted that the concrete Poisson ratio increases with the increase of concrete cracking, and as such higher level of confinement is achieved before failure.

The jacket of square columns, shown in Fig. 2.2a, is loaded with internal pressure, which generates tension in the transverse direction of the jacket. The distribution of the confining stresses is not uniform; as different deformations are observed along the jacket-wall, as shown in Fig. 2.2b. Elongation of the jacket at the corners is generated

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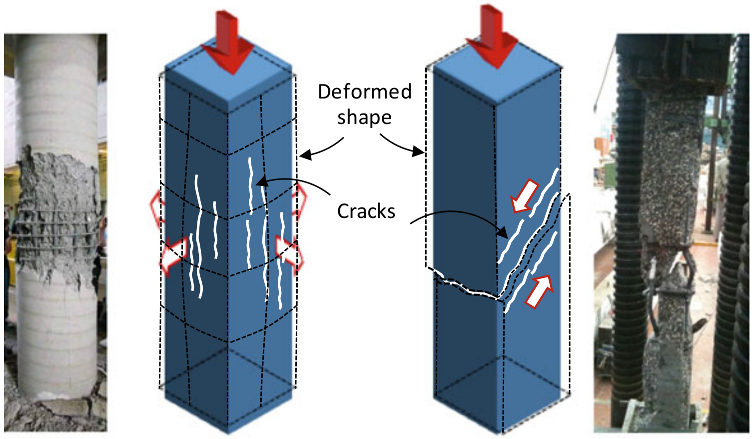


a) Failure mode 1: Splitting tensile cracks b) Failure mode 2: Inclined shear cracks

Fig. 2.1 Concrete columns under uniaxial load

by the tensile force; however, at the mid-wall width, additional flexural deformation is also observed (δ_{square} in Fig. 2.2b); therefore, the confining effect is reduced in the mid-wall zones. As, the dimensions of the square column increase, less confinement of the column is achieved at the mid-wall width and the overall confined area of the column is reduced.

For circular columns, the jacket is loaded with uniform internal pressure, which generates uniform tension and equal lateral deformation in the jacket-wall. In this case, the original concrete section is uniformly confined, as shown in Fig. 2.2c, and the utmost benefit of the jacket is achieved since the deformation of the jacket is minimal (δ_{circle} in Fig. 2.2c). Confinement of the rectangular columns by the jacket is much reduced, as shown in Fig. 2.2d, especially for columns with large aspect ratio. This is mainly due to the large outward deformation of the long side of the column ($\delta_{rectangle}$ in Fig. 2.2d), induced by expansion of the original column section.

2.1.1 Concrete Jacket

Concrete jackets are made to columns by increasing its dimensions and both its longitudinal and transverse steel reinforcement. The composite action between the old and the new concrete is the main key element for the success of the repair work. The purpose of using concrete jackets is to increase the axial-moment capacity, shear capacity or the columns stiffness. Increasing the columns stiffness will result in enhancement of the lateral deformation of the building; yet, it will attract higher

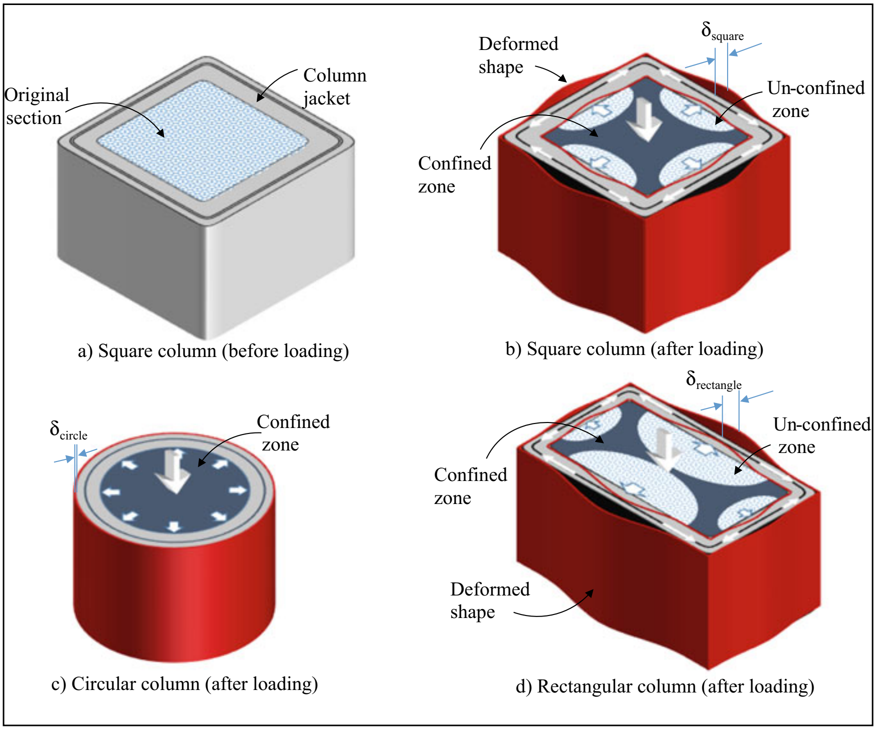


Fig. 2.2 Confinement of the columns induced by the jacket

straining actions due to increase of the lateral load demand. In this case, it is a tradeoff and the design engineer should calculate and does not overestimate the jacket thickness. Concrete jackets can also solve buckling problems of slender columns.

Despite that the concrete jacket is used at a large scale in repair of concrete columns, there are some disadvantages inherent with its application. For example, the size of final concrete section after repair is increased and free space in the building becomes less, which makes it unfavorable to architects and building owners. Care should be also given during the application of the jacket since it requires lots of drilling in the original column section, which might be already weak, and in the slabs/beams as well as footings. Application of concrete jackets is not a speedy process and casting the concrete monolithically requires precision, especially for the top parts of the jackets.

The strength of the repaired member after adding the concrete jacket mainly depends on the strength of the jacket and the added strength of the original member after the provided confinement of the jacket. Therefore, it is for the best to have the jacket from the four sides of the column; yet, this is not applicable in many cases, especially for the columns at the edge or corners of the buildings. Confinement of the original concrete section is much recognized for the small size columns compared to those of the large size columns.

2.1.1.1 Construction of Concrete Jacket

Application of the concrete jackets to columns should be done on steps, as shown in Figs. 2.3 and 2.4 and as follows:

1. Designed propping of the slabs surrounding the column and the slabs at other floors is provided to create another load path to the foundations since the column capacity is significantly reduced during application of the jacket.
2. Removal of the concrete cover is followed until sound concrete in the original section is reached in case no steel corrosion exists. Sandblasting is recommended to roughen the concrete surface since pneumatic hammering may induce micro-cracking of the substrate. In case that reinforcement of the original section is corroded, concrete cover should be fully removed until the longitudinal steel bars are exposed. The steel bars are cleaned and painted with anti-corrosion paint or replaced with new steel bars if the level of corrosion is high.

3. Steel dowels are inserted along the column all around the cross-section to mechanically bond the new to the old concrete section. The depth of the planted bars and the material used for planting the bars should be selected so that at least the yield force in the bars is developed. The longitudinal steel bars of the jacket shall be planted in the foundation and through holes in the slabs are made to allow for the longitudinal bars to continue across the floor slabs. The designed longitudinal

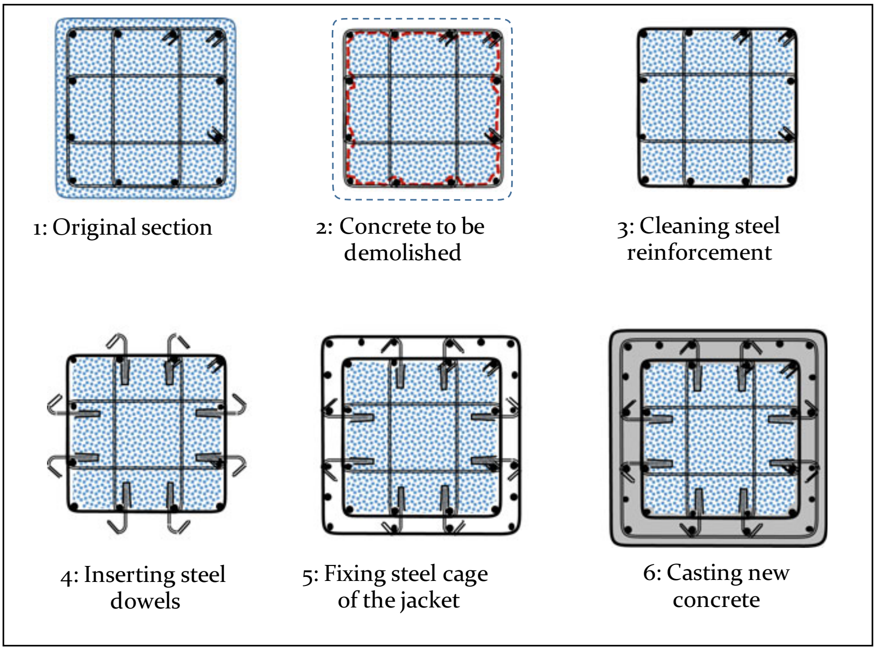


Fig. 2.3 Construction steps of column's concrete jacket

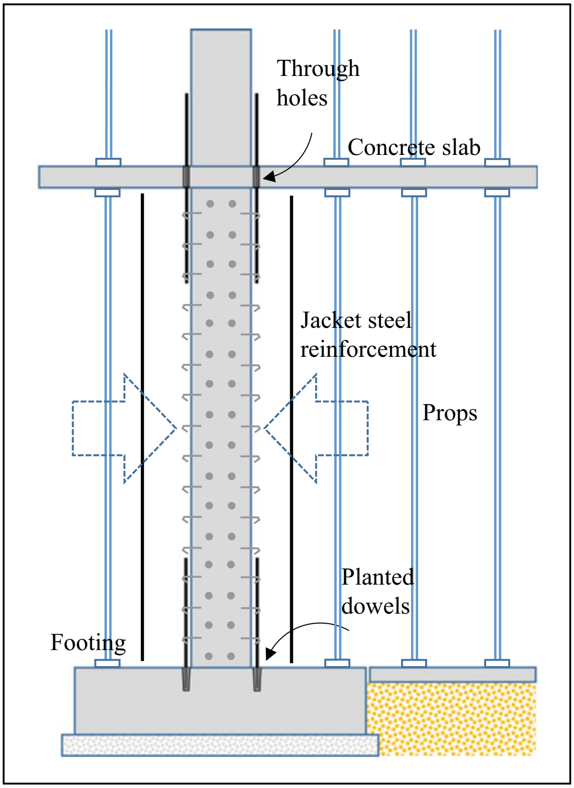


Fig. 2.4 Concrete jacket during construction

and transverse steel reinforcement of the jacket are placed in position, while it is recommended to hook the planted steel dowels to the longitudinal bars. The hooks will prevent early buckling of the longitudinal steel bars and enhance the confinement level of the column cross-section.

4. The jacket should be cast with material compatible with the material of the original concrete section. Shrinkage of the jacket material should be minimized to reduce the stresses on the steel dowels and reduce the cracking in the new concrete. Specific recommendations for the material used in the jacket are given in the following section. Monolithic behavior can be assumed if both dowels and surface roughening of the original column are provided; i.e., no separation exists at the interface between the old and new concrete.

2.1.1.2 Material Properties of the Jacket

Concrete used for the jacket is cast using design mix with physical properties similar to and/or compatible to that of the concrete of the original section. Those properties include the compressive strength, elastic modulus, coefficient of thermal expansion and creep. The aggregate type used in the mix should be similar to that of the old concrete unless it is proved by testing that similar physical characteristics are achieved. Vertical separation between the original section and the jacket is expected with time in case that the properties of the two materials are different. In this respect, grout or mortar with long-term behavior, which is different from that of the concrete should not be used in the jacket. Fibers or shrinkage compensating material may be added to the mix so that the tensile properties of the jacket are enhanced.

Bonding agent may be applied on the old concrete before casting, while the material instructions provided by the manufacturer are strictly followed. It is for the best to use form-and-pump technique in concreting, as shown in Fig. 2.5, so that no honeycombs or voids exist in the jacket. This technique shall also ensure that the top part of the jacket has no voids and completely filled with concrete. The form-and-pump technique relies on totally sealed form work and pumping concrete in the form with adequate pressure. If this technique is not available, holes of 100 mm minimum diameter may be opened in the slab above to be used for placement of the jacket concrete. Any voids in the newly cast concrete should be repaired with cementitious or epoxy grout.

2.1.2 Steel Jacket

Strength and deformation control of columns can be also enhanced using steel jackets composed of steel angles placed along the column corners and battens placed at the sides, as shown in Fig. 2.6. An interface layer of cementitious mortar may be used to improve the effectiveness of the confinement action. However, mechanical connection is advised using steel anchors connected to the batten plates to avoid drilling at the corners of the original column section. The mechanical anchors shall be designed to ensure vertical load transfer from the original section to the jacket. Eventually, the contribution of the steel jacket to the axial strength of the column shall be due to the axial compression of the added steel and to the increase of the concrete strength due to the provided confinement.

The steel angles are generally terminated at the top and bottom of the clear height of the column, and no continuation of the steel jacket is provided across the floor slabs. This is due to the difficulty to realize such kind of connections in practice. Alternatively, the steel angles are terminated at the columns ends by adding steel batten plates connected to the floor slab by anchor bolts. In this case, both the axial capacity of the steel angles and the increase in the axial capacity of the original column provided by the confinement of the steel jacket shall contribute to the overall capacity of the repaired column. Therefore, checking the structural safety of the beam-column joint may be required, in case of having significant increase in the column axial capacity using steel jacket, to ensure that the joint can safely transfer the increased axial load in the column. It should be mentioned that the ACI318-19 allows using concrete strength for the beam-column joint not less than 70% of the column

Fig. 2.5 Form-and-pump concreting

concrete compressive strength without additional requirements for the calculation of the transfer of column axial force through the floor system. Accordingly, design check of the beam-column joint must be provided in case that the axial capacity of the steel-jacketed column is more than 40%.

When columns are subjected to eccentric load that produces tension on one or more than one side of the column, steel plates should be continuous across the concrete slab in order to carry the induced tension. In this case, slots should be made in the slab to pass the reinforcing plates through the concrete floor, as per Fig. 2.7.

2.1.3 FRP Jacket

Columns are subjected to straining actions that produce compressive stresses on the entire section; yet columns in the upper floors of concrete structures or columns in concrete frames may be also subjected to partial tensile stresses. The strengthening technique is different for those columns since FRP laminates have no compression

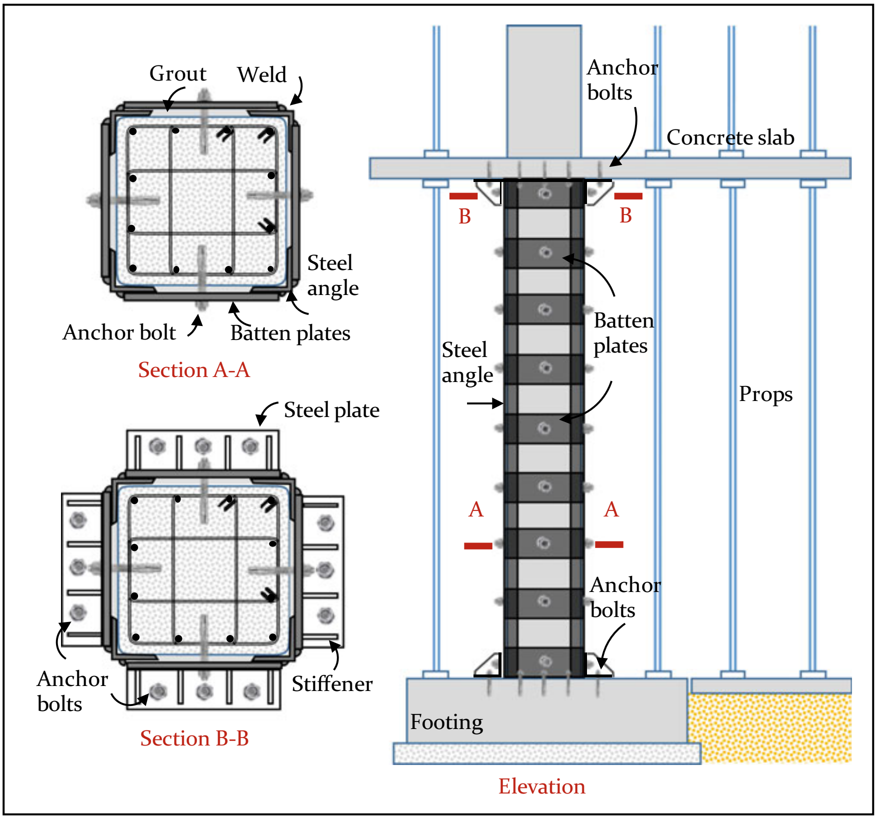


Fig. 2.6 Steel jacket for concrete columns

or flexural stiffness, but it can only carry tension forces in the fiber direction. Tensile stresses are induced in the transverse direction of columns subjected to compressive stresses due to Poisson's ratio, while the tensile stresses exist in the longitudinal direction, when flexural moments are dominating the failure mode of concrete sections. Therefore, FRP is applied with the fiber orientation in the transverse direction for columns with compression control failure and in the longitudinal direction with tension control failure.

2.1.3.1 Enhancement of Column Axial Capacity with FRP

Wrapping columns with FRP increases its axial capacity and ductility because of the confinement induced by the lateral compression provoked by the tensile stiffness of the laminates. The fibers are hence oriented in the transverse direction of the column in order to utilize its full tensile stiffness. Carbon fiber laminates in the form of sheets are the preferred type of laminates used in strengthening since it has the

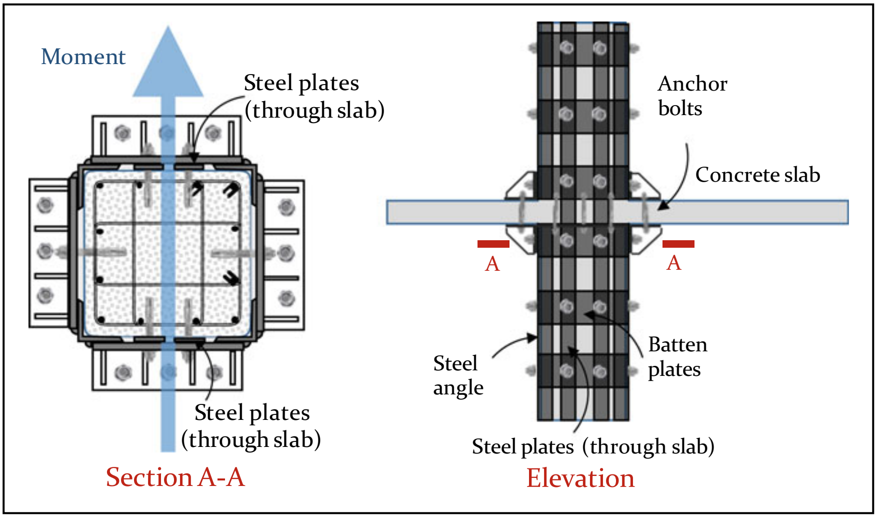


Fig. 2.7 Strengthening of column subjected to eccentric load

highest tensile strength and modulus among all types of fibers. The sheets have typical thickness ranging from 0.1 to 0.4 mm for the dry fibers. They are externally bonded to the concrete surface after cleaning the concrete from any debris, loose material and cleaning the steel reinforcement from corrosion. The concrete surface of the columns is roughened, and the corners are rounded to avoid any stress concentration in the wrapped FRP laminates, as shown in Figs. 2.8 and 2.9. FRP laminates are bonded to the concrete surface with thermosetting resin such as epoxy or vinylester using procured system or wet layup system (ACI 440.2-R17).

Wrapping circular columns with carbon fiber reinforced polymer, (CFRP), laminates were proven to be a very effective method to increase both the axial capacity and ductility of columns. However, it is less effective for square columns and much less effective for rectangular columns with aspect ratio exceeding 2.0. This is attributed to less confinement of the columns due to the out-of-plane deformation of the laminates, induced by the axial loads on columns.

Wrapping FRP laminates to enhance both the axial capacity and ductility of concrete columns is a contact-critical application. This means that FRP jacket is only activated when the original column expands laterally and pushes the jacket out-of-plane and only then tensile stresses are initiated in the laminates and in turns it induces confining pressure on the original column. There is no need to use mechanical anchorage to the column section since FRP jacket does not contribute in the axial stiffness of the repaired column. This makes the application of the FRP jacket easy and fast compared to concrete or steel jackets, which require installing time-consuming mechanical anchorages. However, it is expected that the overall efficiency of the FRP jackets is less compared to other types of jackets, especially for columns with large dimensions. It is preferable to have the FRP laminates spaced, as shown

Fig. 2.8 Surface preparation of column

in Fig. 2.10, so that the concrete column is exposed at certain intervals to avoid any water or dampness contaminated behind the laminates and give room for monitoring the column in case if concrete deterioration or cracking occurs.

2.1.3.2 FRP Strengthening of Columns with Partial Tensile Stresses

FRP strips or sheets may be also used to strengthen columns subjected to flexural moment either due to gravity loads or reversed moment due to lateral loads. In this case, FRP should be placed with the fiber in the longitudinal direction of the column. Developing the FRP laminates at the section subjected to the largest bending moment may be secured by casting concrete encasement with steel dowels, as per the detail given in Fig. 2.11. Steel encasement may be also used to develop the tensile forces in the terminated FRP laminates either at the footings or at the concrete floor slabs.

Fig. 2.9 Rounding column corners

Planting FRP bars in the concrete substrate may be also applied after experimental investigation of the proper bonding material used for fixation and required embedment length of the bars. Planting the reinforcement should be done so that the full strength of the bars is developed prior to bond failure.

Frames may be also strengthened at the joint sections to increase the sections capacity of the columns and the girder, as shown in Fig. 2.12. FRP sheets are used since it can be shaped at the corners. Rounding the concrete corners at the joints with minimum radius of 50 mm is essential to reduce the stress concentration. Relatively big radius is required to shape FRP strips, and hence, the strips are not recommended for strengthening frame joints. Tests had shown the effectiveness of the FRP laminates adhered at the joint in increasing the overall frame capacity (Elkarmouty 2004).

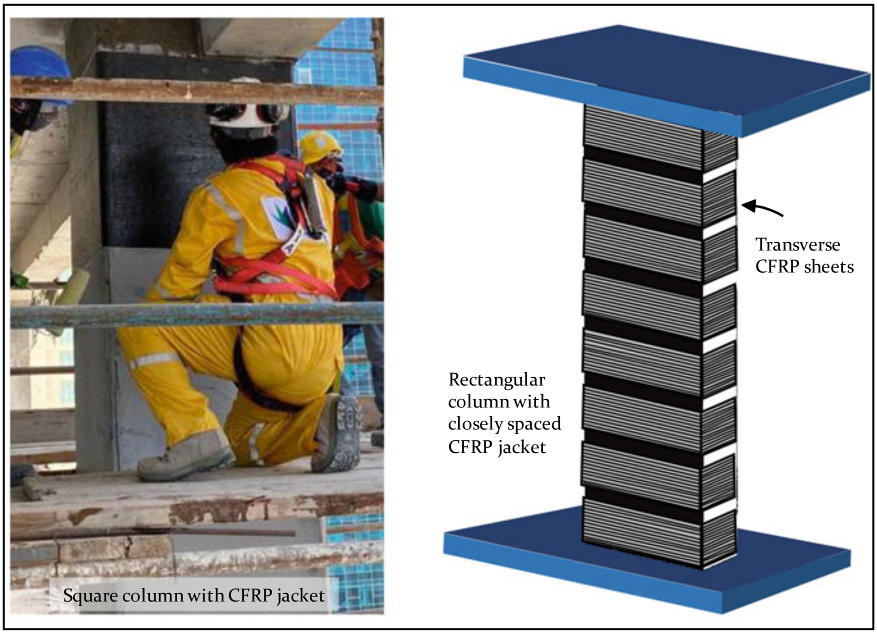


Fig. 2.10 CFRP wrapping of concrete columns

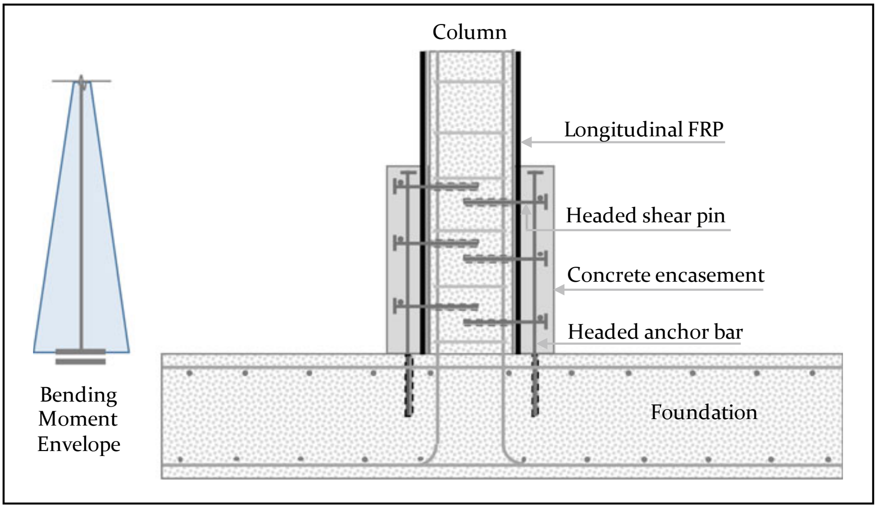


Fig. 2.11 Bond of longitudinal FRP

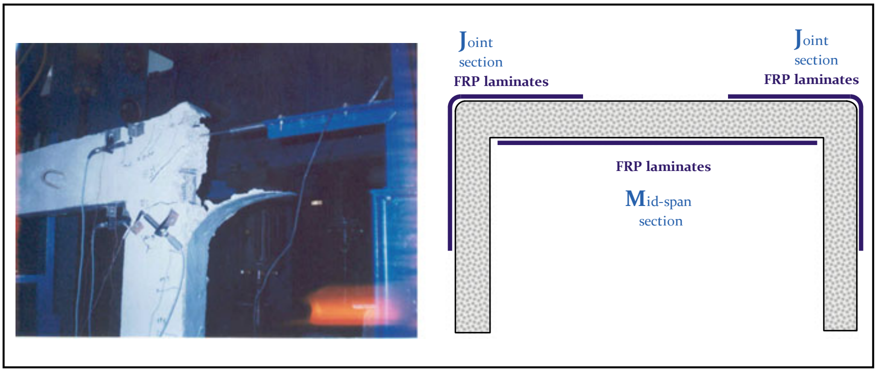


Fig. 2.12 Strengthening of concrete frames (Elkarmouty 2004)

2.2 Design of Jacketed Column Capacity

Calculation of the capacity of jacketed columns starts with measuring the in-situ concrete compressive strength, steel yield stress and percent of area loss of the longitudinal bars due to corrosion (if any). The measured material properties are used to calculate the member response under the effect of external loads. The capacity of strengthened concrete columns shall be calculated accounting for the increase in the compressive strength of the original section due to the confinement of the circular and four-sided jacket. For the three-, two- and one-sided jackets, no or very little confinement shall be induced on the original section, and therefore, the unconfined compressive strength of the original column is considered in the column capacity, as shown in Fig. 2.13. Confinement of the concrete provided by the internal stirrups of the original section shall be ignored, and the only confinement considered in the calculations shall be that provided by the external jacket. Furthermore, the confinement provided by the concrete jacket of rectangular sections may be also ignored due to its limited strength in the transverse direction.

2.2.1 Columns Subjected to Axial Loads

The axial strength of jacketed columns is provided by the strength of the jacket and that of the original section when concrete or steel jackets are provided. In case of using FRP jacket, the axial strength of the jacketed column shall be provided by the strength of the original section only, while the axial strength of the jacket shall be equal to zero. The general formula for the calculation of the axial capacity of jacketed columns is given in Eq. (2.1a).

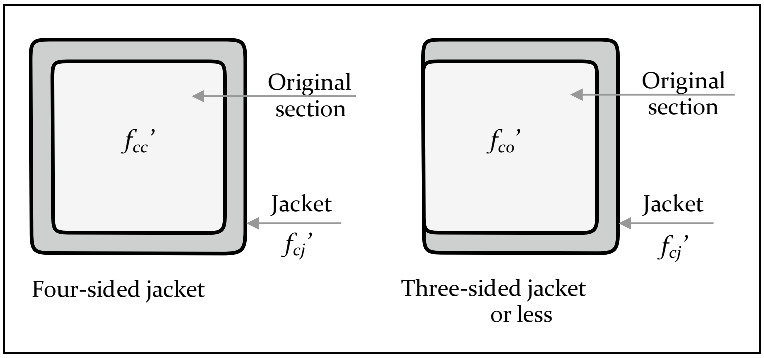


Fig. 2.13 Cross-section of Jacketed columns

$$P_u \leq \phi P_n = \phi \chi \left[\begin{array}{l} \left(0.85 f'_{cc}\left(A_{go} - A_{sto}\right) + f_{yo} A_{sto}\right) \\ +\left(0.85 f'_{cj}\left(A_{gj} - A_{stj}\right) + f_{yj} A_{stj}\right) \end{array} \right] \tag{2.1a}$$

where

ϕ	is equal to 0.75 for compression members with spiral reinforcement and 0.65 for other compression members,
χ	is equal to 0.8 for tied column and 0.85 for spiral columns,
A_{go} and A_{sto}	are the concrete and steel areas of the original section,
f_{yo}	is the yield stress of the longitudinal steel of the original section,
A_{gj}	is the concrete area of the jacket (equal zero in case of steel or FRP jackets),
A_{stj}	is steel area of the longitudinal reinforcing bars in case of concrete jacket, longitudinal steel sections in case of steel jacket and equal zero in case of FRP jacket,
f_{yj}	is the yield stress of the longitudinal steel of the jacket (if any),
f_{cc}'	is the confined concrete compressive stress of the original section and is calculated in the following section, and is equal to the unconfined concrete compressive stress f_{co}' of the original section in case of using one-, two- or three-sided jackets and,
f_{cj}'	is the unconfined concrete compressive stress of the jacket.

In case of ignoring the confining effect of the jacket, Eq. (2.1a) should be modified, as per Eq. (2.1b)

$$P_u \leq \phi P_n = \phi \chi \left[\begin{array}{l} \left(0.85 f'_{co}\left(A_{go} - A_{sto}\right) + f_{yo} A_{sto}\right) \\ +\left(0.85 f'_{cj}\left(A_{gj} - A_{stj}\right) + f_{yj} A_{stj}\right) \end{array} \right] \tag{2.1b}$$

where
f_{co}' is the unconfined concrete compressive stress of the original section.

Equation (2.1)a, b can be further detailed for the three types of jackets as shown in Tables 2.1, 2.2 and 2.3.

Table 2.1 Design equations for different structural concrete jackets

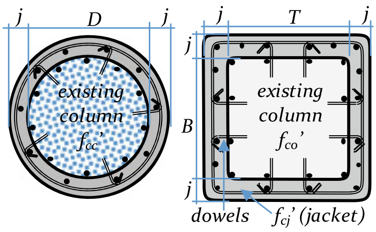
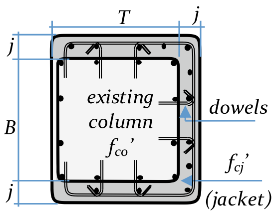
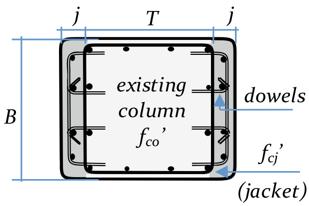
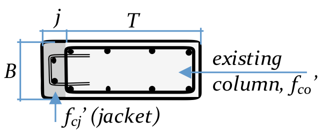
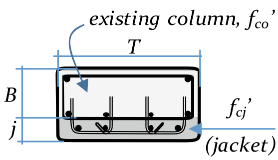
Type of Jacket	Sketch		Design Equation
Concrete Jacket	Four-Sided Jacket—Confined concrete		Equation (2.1a) (for circular sections) and Eq. (2.1b) (for rectangular sections)
	Three-Sided Jacket		Equation (2.1b)
	Two-Sided Jacket		Equation (2.1b)
	One-Sided Jacket		Not accepted
			Accepted depending on design requirements and column dimensions Equation (2.1b)

Table 2.2 Design equations for different structural steel jackets

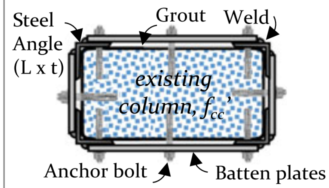
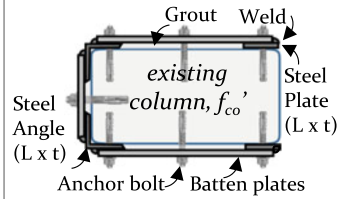
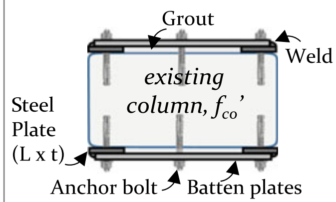
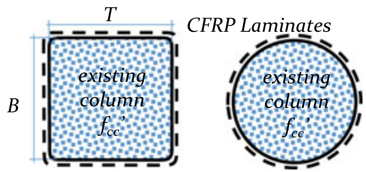
Type of Jacket	Sketch		Design Equation Equation (2.1) a or b, $f'_{cj} = 0$
Structural Steel Jacket	Four-Sided Jacket—Confined concrete		$P_u \leq \phi P_n =$ $\phi\chi\left[\left(0.85\ f'_{cc}\left(A_{go} - A_{sto}\right) + f_{yo}A_{sto}\right) + f_{yj}A_{stj}\right]$ A_{stj}: Area of the four steel angles f_{yj}: Yield stress of the structural steel angles
	Three-Sided Jacket		$P_u \leq \phi P_n =$ $\phi\chi\left[\left(0.85\ f'_{co}\left(A_{go} - A_{sto}\right) + f_{yo}A_{sto}\right) + f_{yj}A_{stj}\right]$ A_{stj}: Area of the two steel angles and two steel plates f_{yj}: Yield stress of the structural steel angles and plates
	Two-Sided Jacket		$P_u \leq \phi P_n =$ $\phi\chi\left[\left(0.85\ f'_{co}\left(A_{go} - A_{sto}\right) + f_{yo}A_{sto}\right) + f_{yj}A_{stj}\right]$ A_{stj}: Area of the four steel plates f_{yj}: Yield stress of the structural steel plates
	One-Sided Jacket		Refer to the accepted shapes in Table 2.1

Table 2.3 Design equations for different structural FRP jackets

Type of Jacket	Sketch		Design equation Equation (2.1a) with $f'_{cj} = 0$ and $A_{stj} = 0$
Structural FRP Jacket	Four-Sided Jacket—Confined concrete	 	$P_u \leq \phi P_n =$ $\phi \chi \left[\left(0.85\, f'_{cc} \left(A_{go} - A_{sto} \right) + f_{yo} A_{sto} \right) \right]$
	Three-Sided Jacket, Two-Sided Jacket or One-Sided Jacket		Not allowed

2.2.2 Columns Subjected to Axial Load and Bending Moments

Concrete sections subjected to axial force and bending moments can be designed using an equivalent concrete compressive strength representing the strength of the RC jacket and the confined strength of the original section when using circular or four-sided concrete jacket. It is the equivalent concrete compressive strength to be used in checking the capacity of the section under different straining actions can be calculated as per Eq. (2.2a). The confinement effect provided by the jacket may be ignored when using four-sided concrete jacket due to its limited transverse strength and in this case the equivalent concrete strength shall be calculated using Eq. (2.2b). When using four-sided steel jacket or FRP jacket, the confined concrete compressive strength, f'_{cc}, should be used in the design; while the uniaxial concrete compressive strength, f'_{co}, is used for one-, two- or three-sided steel jackets. Moment interaction diagrams are then used to design/check the strengthened concrete sections using the assigned concrete strength, as per Table 2.4.

$$f'_{c\,(\mathrm{equivalent})} = \frac{\text{Area of the original section} \times f'_{cc} + \text{Area of the concrete jacket} \times f'_{cj}}{\text{Total area of the enlarged concrete section}} \tag{2.2a}$$

$$f'_{c\,(\mathrm{equivalent})} = \frac{\text{Area of the original section} \times f'_{co} + \text{Area of the concrete jacket} \times f'_{cj}}{\text{Total area of the enlarged concrete section}} \tag{2.2b}$$

2.2.3 Calculation of Confined Concrete Compressive Stress

Concrete confinement of the original concrete section is provided by the jacket and is calculated based on the transverse stiffness of the jacket. As the jacket stiffness in the transverse direction increases, confinement of the concrete area increases. This confinement results in an increase in both the concrete compressive strength and ultimate strain. The concrete area affected by jacket confinement is the largest for circular sections, followed by square sections and the smallest for rectangular sections. Confinement of rectangular concrete cross-sections is further reduced with the increase of its aspect ratio or the increase of the long dimension of the column section, as shown in Fig. 2.15. The confined concrete areas are initiated at the corners of the column cross-section, as represented in the figure by the shaded areas with dark gray. The figure shows the confinement provided by concrete jacket; however, the same concept applies for steel and CFRP jackets.

Table 2.4 Recommended concrete strength for strengthened sections design

Type of Jacket	Concrete Compressive Strength
Circular concrete	$f_c{}'$ *(equivalent) (Eq. 2.2a)*
Four-sided concrete	$f_c{}'$ *(equivalent) (Eq. 2.2b)*
One-, Two- or Three-sided concrete	$f_{co}{}'$ for the original column, and $f_{cj}{}'$ for the jacket[a]
Four-sided steel jacket and FRP jacket	$f_{cc}{}'$
One-, Two- or Three-sided steel jacket	$f_{co}{}'$

[a] Analysis of concrete section using layers of concrete with different properties is recommended since the direction of the bending moment shall result in compression on concrete parts with different grades, as shown in Fig. 2.14a for sections subjected to single bending moment and Fig. 2.14b for sections subjected to double bending moments. Layered analysis of concrete sections can be found in many textbooks (Park and Paulay 1975) and can be easily implemented in the finite element analysis

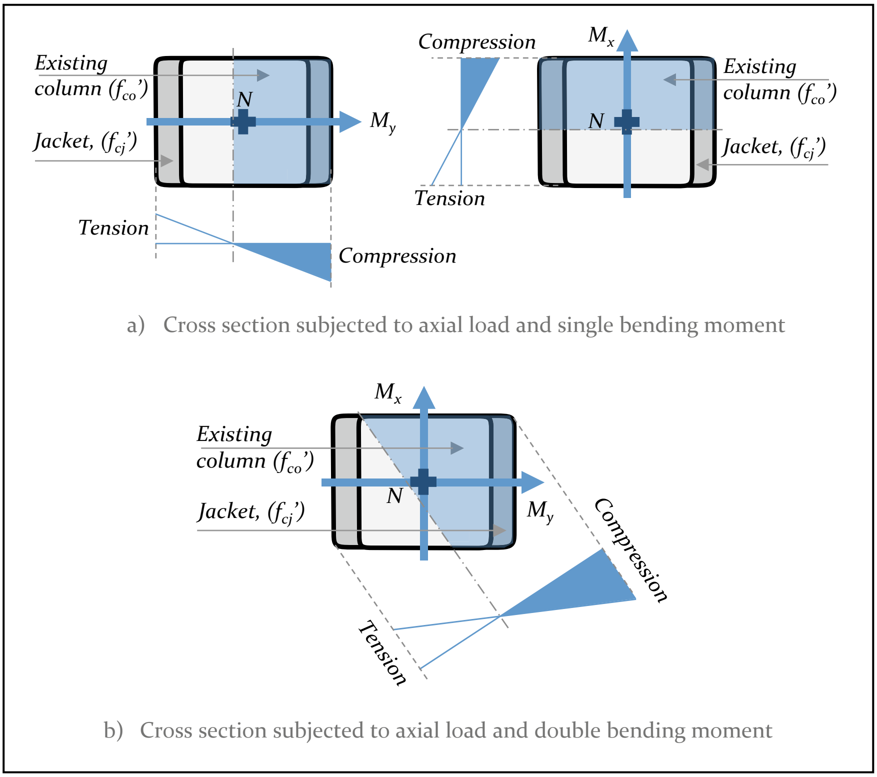


Fig. 2.14 Strengthened sections with concrete jacket subjected to axial load and or bending moment in one or two directions

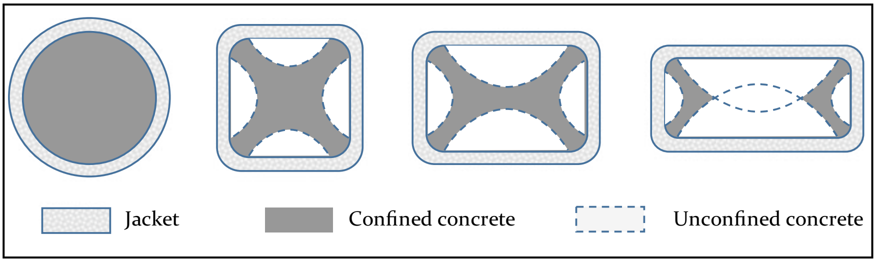


Fig. 2.15 Confinement of different jacketed concrete sections

Concrete confinement is not uniform along the height of the jacketed columns. The provided confinement by the RC jacket is mainly due to the tensile stress in the closed outer steel stirrups in the jacket, while the confinement provided by the tensile strength of the concrete in the jacket is ignored. Therefore, the confinement provided by the RC jacket is reduced between the steel stirrups of the concrete jacket along the column height.

Likewise, the confinement provided by either CFRP or steel jackets is reduced between the CFRP laminates or steel plates, as shown in Fig. 2.16. This further reduction in the concrete confinement should be considered in the calculations of the confined concrete strength of the original section. For steel and CFRP jackets and in case that the CFRP laminates or steel plates are not intermittent and the entire column height is covered, this further reduction in the confined concrete strength does not exist.

The stress–strain behavior of confined concrete is given in Fig. 2.17 by Mander et al. (1988) for sections confined with steel reinforcement. In this case, the confined compressive strength, f'_{cc}, is calculated by Eq. (2.3).

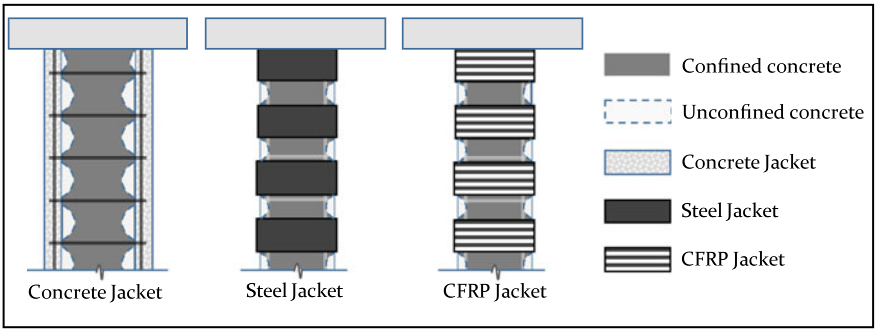


Fig. 2.16 Confinement along the jacketed column height

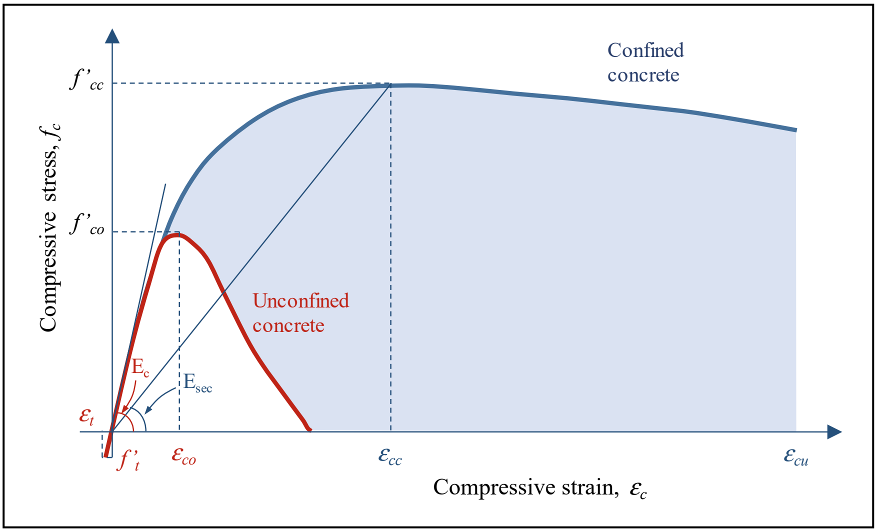


Fig. 2.17 Stress–strain behavior of confined concrete (Mander et al. 1988)

$$f'_{\mathrm{cc}} = f'_{\mathrm{co}}\left[-1.254 + 2.254\sqrt{1 + \frac{7.94 f'_{\mathrm{l}}}{f'_{\mathrm{co}}}} - 2\frac{f'_{\mathrm{l}}}{f'_{\mathrm{co}}}\right] \tag{2.3}$$

where f'_{co} is the unconfined concrete compressive strength in MPa and f'_{l} is the effective lateral confining pressure given by Eq. (2.4).

$$f'_{\mathrm{l}} = f_{\mathrm{l}} k_{\mathrm{e}} \tag{2.4}$$

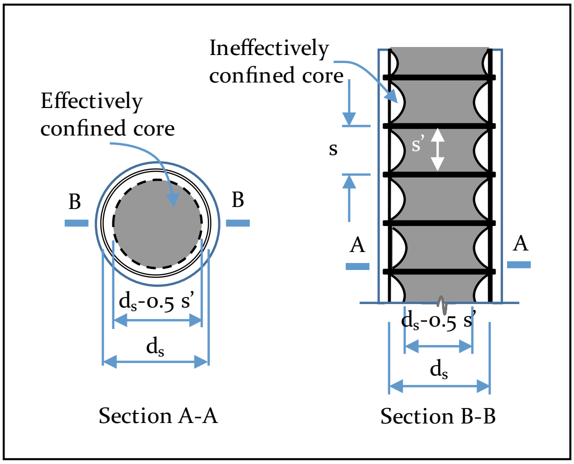
$$k_{\mathrm{e}} = \frac{A_{\mathrm{e}}}{A_{\mathrm{cc}}} \tag{2.5}$$

$$A_{\mathrm{cc}} = A_{\mathrm{c}}(1 - \rho_{\mathrm{cc}}) \tag{2.6}$$

f'_{l} is the lateral pressure from the transverse reinforcement assumed to be uniformly distributed over the surface of the concrete core; k_{e} is the confinement effectiveness coefficient; A_{e} is the area of effectively confined concrete core; A_{c} is the area of the core section enclosed by center line of the perimeter hoop or spiral; ρ_{cc} is the ratio of longitudinal reinforcement to area of the core of section. Figure 2.18 shows the arching action that is assumed to occur between the levels of transverse circular reinforcement. The area of ineffectively confined concrete is illustrated in the figure, midway between the levels of the transverse reinforcement.

Different expressions exist for the calculation of the confined concrete compressive strength. The ACI 440.2-R17 recommends the formula given by Eq. (2.7) to calculate the confined concrete stress under lateral pressure for sections confined with FRP.

Fig. 2.18 Effectively confined concrete core for circular hoop reinforcement



$$f'_{cc} = f'_{co} + 3.3\upsilon f'_l \tag{2.7}$$

where υ is a reduction factor depending on the material used for confinement. υ equals to 1.0 for concrete or steel jackets and is equal to 0.95 for FRP jackets as will be explained later.

2.3 Unified Design Method for Columns' Jackets

Despite that the properties of concrete, steel and FRP are quite different as steel is an elasto-plastic and ductile material, while FRP is perfectly elastic and brittle material, unified approach can be used for the design of strengthened concrete columns subjected to axial load with or without uni- or biaxial bending moment. The general formula for design of those sections is given in Eqs. (2.1) and (2.2).

In order to unify the calculation of confined concrete strength for columns strengthened with concrete, steel or FRP jackets, Eq. (2.3) by Mander et al., 1988 is compared to that of the ACI 440.2 R-17 and given by Eq. (2.7). The key variables considered in the study included spacing between stirrups (s), unconfined concrete stress (f_{co}), longitudinal steel reinforcement ratio and diameter of concrete section (d). Results of the comparison are shown in Fig. 2.19a–d.

The analysis indicates that the ACI 440.2 R-17 expression is slightly conservative compared to Mander et al. 1988 approach. The confined concrete stress calculated by the ACI 440.2 R-17 is 10–15% lower than that calculated by the expression given by Mander et al. (1988). It is therefore recommended to Eq. (2.7) in the calculation of the confined concrete compressive strength whether these sections are confined with reinforced concrete jackets, steel collars or FRP sheets. The expression in Eq. (2.7)

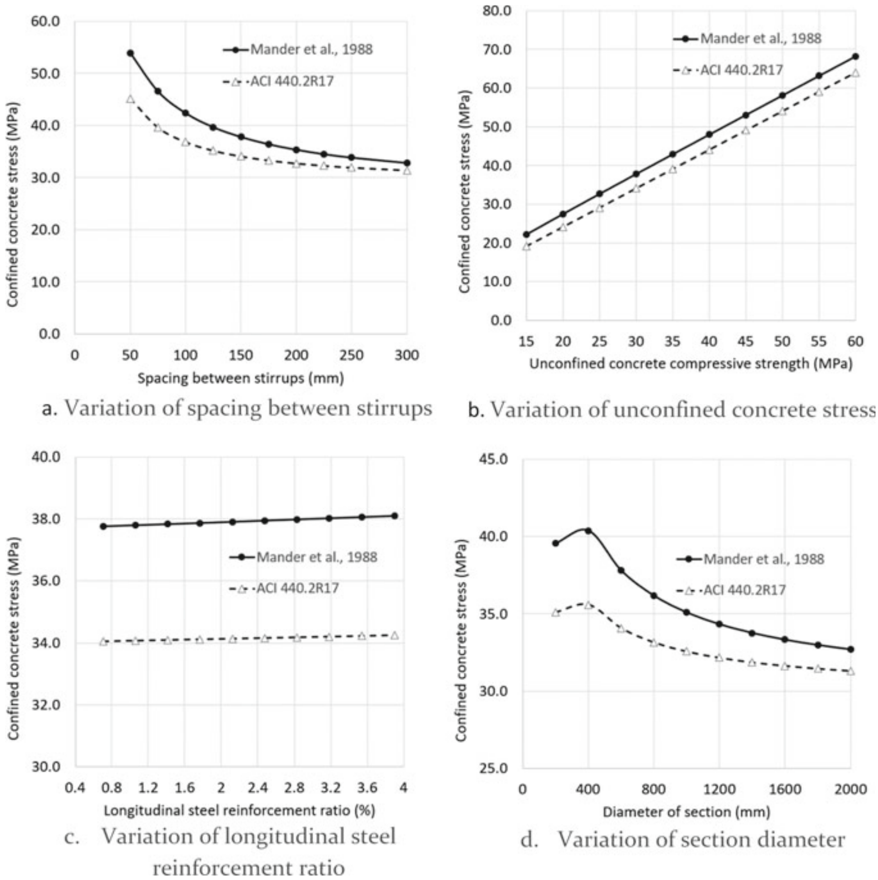


Fig. 2.19 Confined concrete stress predicted using Mander et al. (1988) and ACI 440.2R-17

can be applied for circular or rectangular sections as shown in the following sections, noting that "υ" equals 1.0 for steel and concrete jackets.

2.3.1 Circular Sections

Concrete confinement of the existing column section is provided by the hoop or spiral steel stirrups provided in the concrete jacket, while it is provided by the external steel plates or FRP wrapping for the other types of jackets. Concrete confinement provided by the steel stirrups of the original section may be ignored due to the uncertainties inherited in the amount and distribution of the transverse reinforcement inside the column.

For circular sections, with transverse reinforcement in the concrete jacket, the area of an effectively confined concrete core at midway between the levels of transverse reinforcement can be expressed by Eq. (2.8). Equations (2.9), (2.10) and (2.11) show the concrete area enclosed by the center line of the circular hoops or spiral and the effective lateral confining pressure for circular columns.

$$A_{\mathrm{e}} = \frac{\pi}{4}\left(d_{\mathrm{j}} - \frac{s'}{2}\right)^2 = \frac{\pi}{4}d_{\mathrm{j}}^2\left(1 - \frac{s'}{2d_{\mathrm{j}}}\right)^2 \tag{2.8}$$

$$A_{\mathrm{cc}} = \frac{\pi}{4}d_{\mathrm{j}}^2(1 - \rho_{\mathrm{cc}}) \tag{2.9}$$

$$f_{\mathrm{l}}' = 2k_{\mathrm{e}}\frac{A_{\mathrm{jt}}f_{\mathrm{jt}}}{d_{\mathrm{s}}s} = \frac{1}{2}k_{\mathrm{e}}\rho_{\mathrm{j}}f_{\mathrm{jt}} \tag{2.10}$$

$$\text{where for hoops :} \quad k_{\mathrm{e}} = \frac{\left(1 - \frac{s'}{2d_{\mathrm{s}}}\right)^2}{(1 - \rho_{\mathrm{cc}})} \tag{2.11}$$

where d_{j} is the diameter of the transverse reinforcement inside the jacket, s' is the clear spacing between transverse reinforcement, A_{jt} is the area of the transverse reinforcement enclosed by the concrete jacket, f_{jt} is the yield stress of the transverse reinforcement, s is the center-to-center spacing of circular hoop and ρ_{j} is the ratio of the volume of transverse confining steel to the volume of confined concrete core.

For concrete columns strengthened with steel collars, the confining pressure, f_{l}', can be estimated by Eq. (2.10), while A_{jt} is the area of the steel collars, f_{jt} is the yield stress of the steel collars, s is the center-to-center spacing between collars, s' is the clear spacing between collars and d_{s} is taken equal to d, the diameter of circular section.

For concrete columns wrapped with FRP laminates, the confining pressure, f_{l}', can be estimated by Eq. (2.10), while A_{jt} is the area of the FRP sheets/mm (A_{f}) = $n\ t_{\mathrm{f}}$, where n is the number of FRP plies and t_{f} is the thickness of each ply, f_{jt} is the design ultimate tensile strength of the FRP noted as f_{fu} in the ACI 440.2 R-17, k_{e} is 0.55, s' is equal to 1.0 for continuous wrapping and d_{s} is taken equal to d, the diameter of circular section. Therefore, Eq. (2.10) is expressed as per Eq. (2.12) for sections with FRP wrapping.

$$f_{\mathrm{l}}' = 2k_{\mathrm{e}}\frac{A_{\mathrm{f}}f_{\mathrm{fu}}}{d} \quad \text{for continuous FRP wrapping} \tag{2.12}$$

Example 2.1 A concrete circular column of diameter 600 mm is subjected to an ultimate normal force of 3500 kN and an ultimate moment of 300 kN m. The unconfined concrete compressive strength of the column is 20 MPa based on statistical analysis using core tests as per ACI 214.4-R10. The column is reinforced with 10 longitudinal bars of 20 mm diameter with yield stress of 420 MPa, as shown in Fig. 2.20. It is

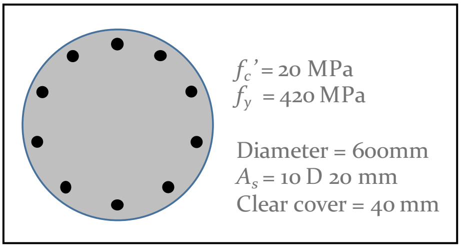


Fig. 2.20 Example of circular column

required to calculate if the column requires strengthening and investigate different strengthening schemes using:

(1) concrete jacket, (2) steel collars and (3) externally bonded CFRP sheets.

Answer

First, it is required to construct the axial load–bending moment interaction diagram of the column cross-section. For the purpose of evaluating the column strength, the reduction factors proposed for evaluation of concrete elements given in the ACI318-19 and presented in Table 1.5 are used. Accordingly, the interaction diagram of the concrete section is given in Fig. 2.21. The outer dotted line in the figure represents the ultimate capacity of the section using reduction factors equal to 1.0, while the inner solid line represents the safety limit of the section. The applied load on the column is shown in the figure with a red circle, where it is concluded that the column is inadequate to resist the applied loads and strengthening of the column is required.

(1) Strengthening of the column using 70 mm concrete jacket with $f'_c = 45$ MPa and stirrups of T12-every 100 mm with yield stress of 420 MPa and concrete cover of 40 mm.

Using Equations (2.10) and (2.11) and the section properties given in Figure 2.22, the following results can be obtained.

$A_{jt} =$	$\frac{\pi}{4}(12)^2 = 113$	mm^2	
$d_s =$	$(600 + 2 \times 70) - (2 \times 40 + 12) = 648$	mm	
$s' =$	$100 - 12 = 88$	mm	
$k_e =$	0.88		(Eq. 2.11)
$f_l' =$	1.29	MPa	(Eq. 2.10)
$f'_{cc} =$	24.25	MPa	(Eq. 2.7)
f_c'equivalent $=$	$\frac{282743x24.25+147340x45}{430084} = 31$	MPa	(Eq. 2.2)

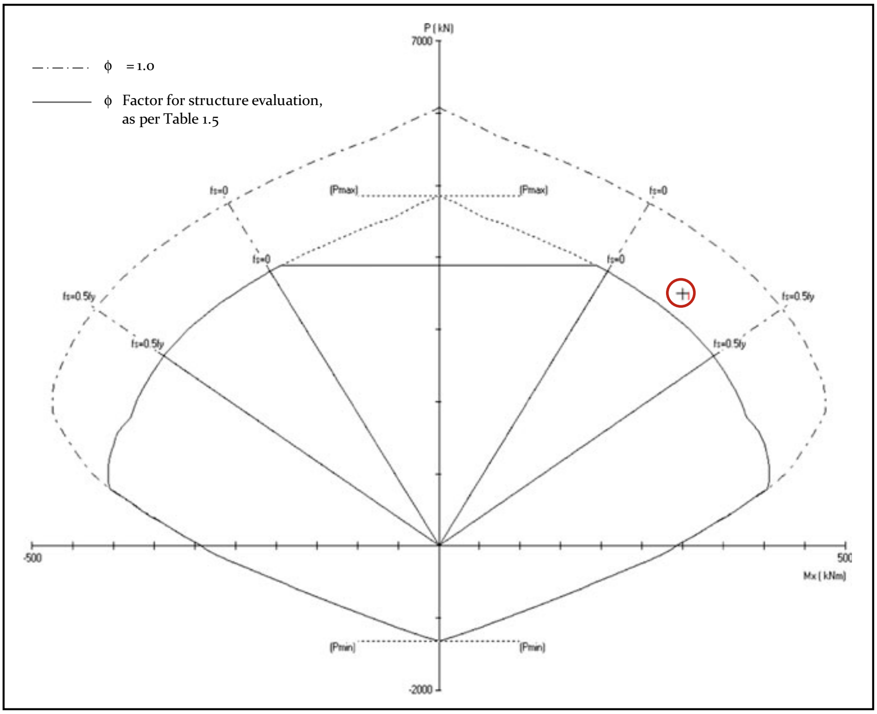


Fig. 2.21 Interaction diagram of the 600 mm circular column

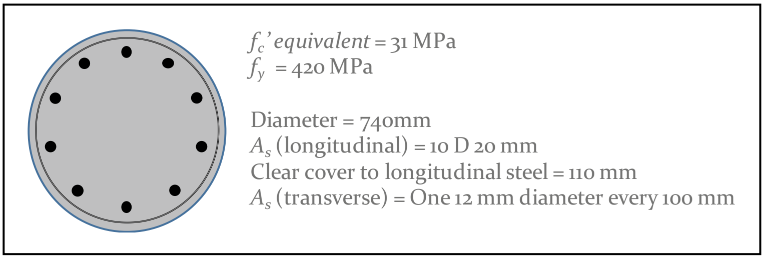


Fig. 2.22 Analyzed strengthened section

Using the equivalent concrete compressive strength of 31 MPa and the low reduction factors depicted for new design, as per the ACI318-19 and given in Table 1.5, the section with concrete jacket is analyzed and the axial load–bending moment interaction diagram is drawn in Figure 2.23.

It can be seen that the column is adequate to resist the applied loads, where the red circle representing the applied loads is within the safety limit of the section. It should be noted that in this case, the interaction diagram neglects the longitudinal steel reinforcement of the jacket.

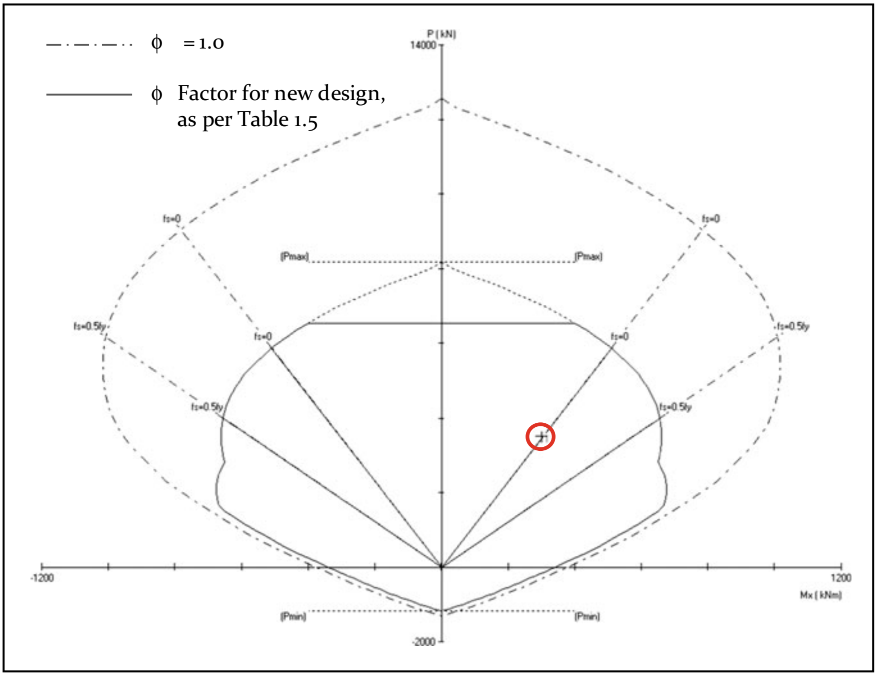


Fig. 2.23 Interaction diagram of the circular column strengthened with concrete jacket

It should be noted that the enlarged column should be remodeled in the 3D finite element model of the building under consideration using the enlarged section dimensions (740 mm diameter) and elastic modulus of concrete of f'_c = 31 MPa. The resulting straining actions acting on the column should be checked accordingly since it is expected to change after the increase of the column stiffness.

(2) Strengthening of the column using 150 × 10 mm steel collars spaced at 400 mm, with f_{yj} = 345 MPa. Using Eqs. (2.10) and (2.11), the following results can be obtained.

A_{jt} =	150 × 10 = 1500	mm^2	
d_s =	= d = 600	mm	
s' =	400 − 150 = 250	mm	
k_e =	0.63		(Eq. 2.11)
f_l' =	2.7	MPa	(Eq. 2.10)
f'_{cc} =	29	MPa	(Eq. 2.7)

Using the equivalent concrete compressive strength of 29 MPa and the reduction factors for new design, as per the ACI318-19 and given in Table 1.5, the axial load–bending moment interaction diagram is drawn in Figure 2.24. It can

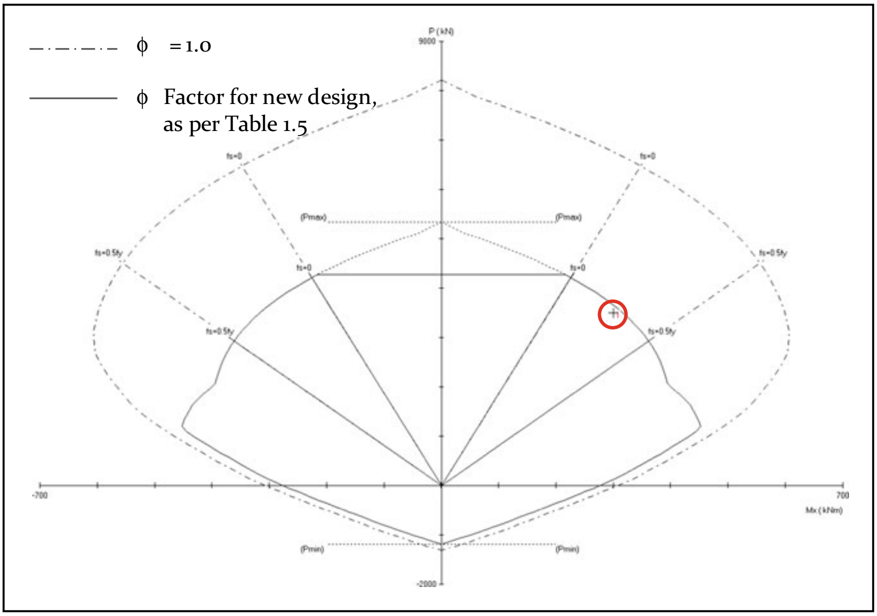


Fig. 2.24 Interaction diagram of the circular column strengthened with steel collars of 150 × 10 mm spaced at 400 mm

be seen that the column is adequate to resist the applied loads, where the red circle representing the applied loads is within the safety limit of the section. The analysis ignores any contribution of the steel collars to the axial column capacity since there are no longitudinal steel sections used in the jacket. The strengthened column section should be modeled in the 3D finite element model of the building under consideration using the elastic modulus of confined concrete of $f'_c = 29$ MPa. The resulting straining actions acting on the column should be checked accordingly.

(3) Strengthening of the column using three continuous plies of externally bonded CFRP sheets with $E_f = 65$ GPa, $f_{fu} = 700$ MPa and $t_f = 1.0$ mm. Using Eqs. (2.10) and (2.11), the following results can be obtained.

$A_{jt} = (A_f) =$	$3 \times 1 \times 1.0 = 3$	mm^2	
$d_s =$	$= d = 600$	mm	
$s' =$	1	mm	
$k_e =$	0.55		ACI 440.2 R-17
$f_1' =$	3.85	MPa	(Eq. 2.10)
$f'_{cc} =$	$20 + 3.3 \times 0.95 \times 3.85 = 32.0$	MPa	(Eq. 2.7)

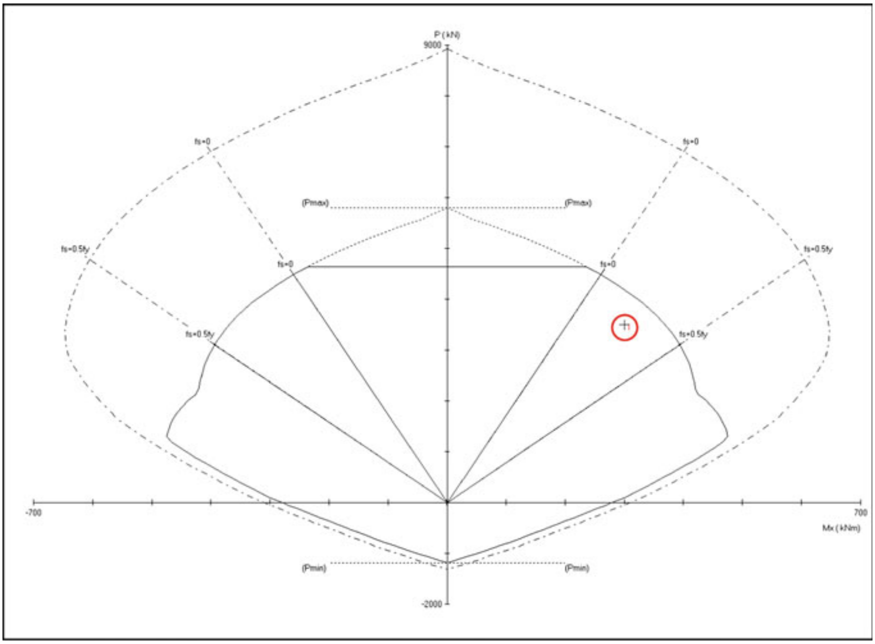


Fig. 2.25 Interaction diagram of the strengthened column using three plies of CFRP sheets

The column is adequate using an equivalent compressive strength of 32 MPa as depicted in the interaction diagram given in Figure 2.25. Again, the strengthened column section should be modeled in the 3D finite element model of the building under consideration using the elastic modulus of confined concrete of $f'_c = 32$ MPa. The resulting straining actions acting on the column should be checked accordingly. Different strengthening schemes for the column are illustrated in Figure 2.26.

2.3.2 Rectangular Sections

Confinement of rectangular concrete sections provided by transverse reinforcement is much more complicated and less effective than that of the circular sections. Figure 2.27 shows the arching action that is assumed to occur in rectangular columns at both the cross-section level and along the column height. The effectively confined area of concrete at stirrups level is found by subtracting the area of the parabolas containing the ineffectively confined concrete (Mander et al. 1988). The total area of effectively confined concrete at the level of stirrups where "*n*" is the number of longitudinal bars tied with stirrups is:

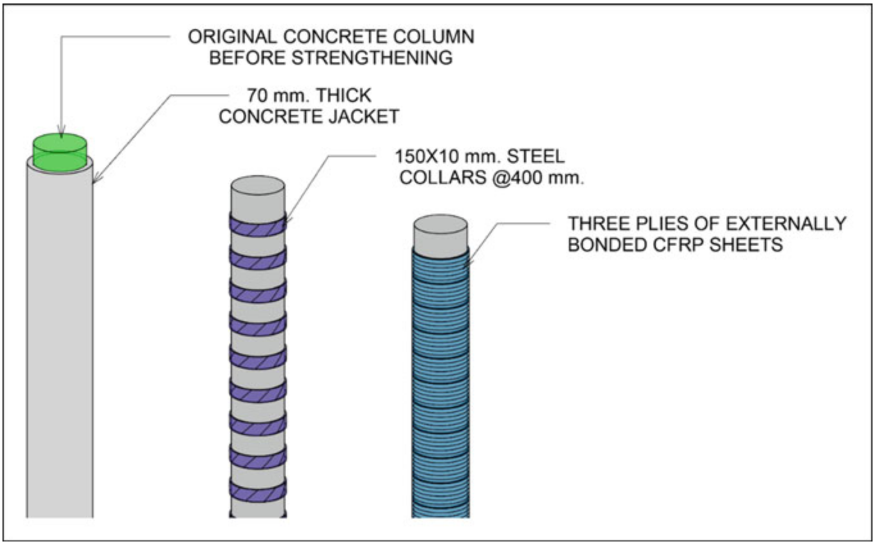


Fig. 2.26 Different strengthening schemes for the circular column (Example 2.1)

$$A_{\mathrm{e}} = \left(b_{\mathrm{c}}h_{\mathrm{c}} - \sum_{i=1}^{n}\frac{w_i^2}{6}\right)\left(1 - \frac{s'}{2b_{\mathrm{c}}}\right)\left(1 - \frac{s'}{2h_{\mathrm{c}}}\right) \tag{2.13}$$

where b_{c} and h_{c} are the core dimensions to centerline of perimeter stirrup in x- and y-directions, respectively.

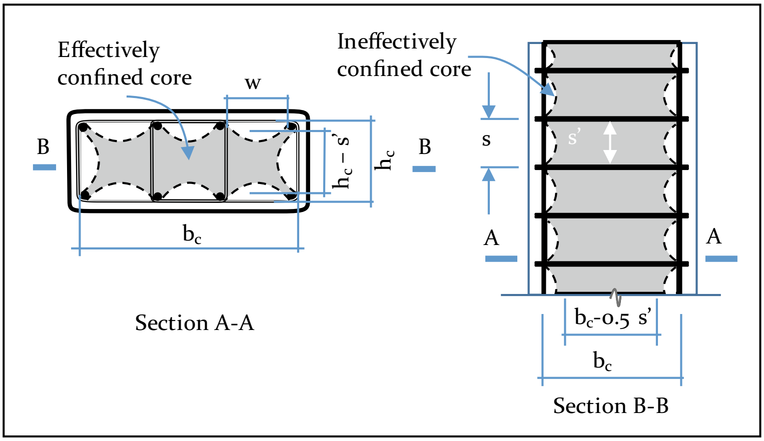


Fig. 2.27 Effectively confined core for rectangular sections

$$k_e = \left(1 - \sum_{i=1}^{n} \frac{w_i^2}{6b_c h_c}\right)\left(1 - \frac{s'}{2b_c}\right)\left(1 - \frac{s'}{2h_c}\right)\frac{1}{(1-\rho_{cc})} \tag{2.14}$$

$$\rho_x = \frac{A_{sx}}{sh_c} \tag{2.15}$$

$$\rho_y = \frac{A_{sy}}{sb_c} \tag{2.16}$$

where A_{sx} and A_{sy} are the total area of transverse bars running in x- and y-directions, respectively.

Accordingly, the effective lateral confining stress on the concrete can be expressed as:

$$f'_{lx} = k_e \frac{A_{sx}}{sd_c} f_{yh} \tag{2.17}$$

$$f'_{ly} = k_e \frac{A_{sy}}{sb_c} f_{yh} \tag{2.18}$$

Knowing f'_{lx} and f'_{ly}, the effective lateral pressure f'_l can be determined using Fig. 2.28 and the confined concrete compressive strength can be determined using the generalized equation of confinement, Eq. (2.7) where υ is a reduction factor and can be taken as 1.0.

The original column transverse reinforcement should not be considered in the calculation of the confined concrete strength. On the other hand, the transversal steel

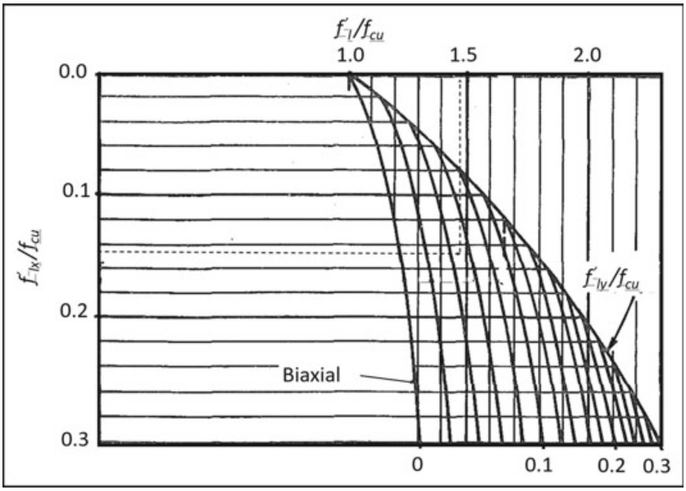


Fig. 2.28 Confined strength determination from lateral confining stresses for rectangular sections (Mander et al. 1988)

stirrups provided in the rectangular concrete jacket shall result in little confinement effect of the original concrete section. For steel- and FRP-strengthened concrete columns, the confining pressure, (f_1'), shall be calculated using the concept presented by Mander et al. (1988), which can be further explained in the next section.

It is worth mentioning that the confinement effect of the concrete jacketed columns may be ignored due to its small effect. In this case, the axial strength of the column should be calculated using Eq. (2.1b), while the equivalent concrete strength may be calculated using Eq. (2.2b) to check the column safety when subjected to axial load and bending moments. Modeling of the strengthened column using the enlarged section dimensions and f_1' (*equivalent*) has to be considered in the finite element analysis.

2.3.2.1 Calculation of the Confining Pressure in Rectangular Columns

For columns strengthened with steel jackets comprising of longitudinal steel angles at the corners and four batten plates welded to the angels forming closed section at the perimeter of the column, as shown in Fig. 2.29, the Eurocode 8 (CEN 2003) recommends to account for the concrete confinement effect provided by the steel batten plates and ignore the vertical contribution of the steel angles to the axial capacity of the strengthened column. In this case, the confined concrete compressive strength is used in the calculation of the axial column strength.

The Eurocode 8 (CEN 2003) specifies that the spacing between two successive steel battens should not exceed b/2, where b is the smaller section dimension of the column. For the case of rectangular columns with high aspect ratio, it needs to

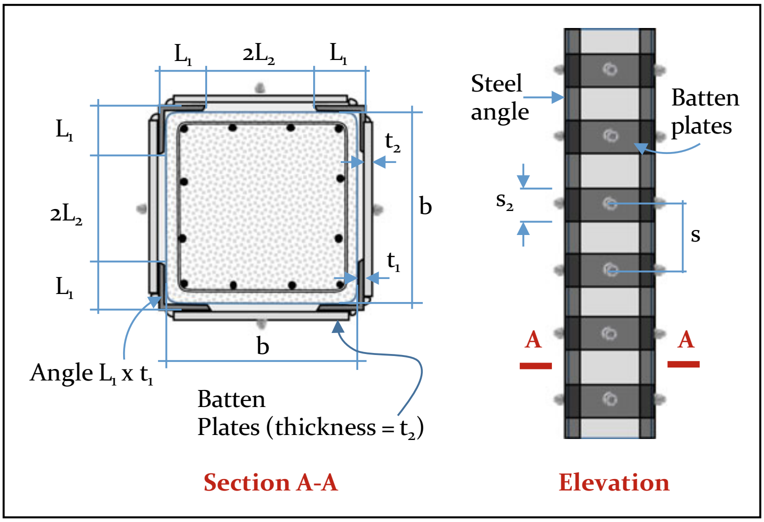


Fig. 2.29 Strengthening of rectangular columns using steel angles and battens

diminish the free length between the steel battens by means of steel bars crossing the structural member to tighten the two opposed battens. Eurocode 8 (CEN 2003) specifies that the minimum mechanical percentage of transverse steel battens, ω_s, in pitch s, necessary to achieve strength and ductility for low (L), medium (M) and high (H) ductility classes should be at least 0.07, 0.11 and 0.13, respectively. The mechanical ratio of steel battens ω_s is defined as follows:

$$w_s = \rho_s \frac{f_{ydb}}{f_{cd}} \tag{2.19}$$

$$\rho_s = \frac{4t_2 s_2}{\text{bs}} \tag{2.20}$$

$$B = 2L_1 + 2L_2 \tag{2.21}$$

where f_{cd} and f_{ydb} are the design strength of concrete and steel battens and can be taken as $0.8 f_{cu}$ and $f_y/1.15$, respectively.

In a recent research by Campione, 2013, an analytical model for the design of axially loaded strengthened RC columns with steel angles and battens is developed. The model considers the contribution in confinement pressures caused by transverse battens and steel angles and the contribution in terms of the load-carrying capacity of steel angles subjected to axial force and bending moments.

For square sections of side length, b, strengthened with batten plates and four steel angles at the corners, the lateral confining pressure can be estimated using Eq. (2.13) and rearranged as per Eq. (2.22).

$$f_l' = \left\{1 - \frac{2}{3}\left(\frac{(b - 2L_1)^2}{b^2}\right)\right\}\left(1 - \frac{s'}{2b}\right)^2 x2\frac{S_2 t_2 f_y}{bS} \tag{2.22}$$

where L_1 is the steel angle side length, S_2 and t_2 are the width and thickness of the batten plates, respectively, and f_y is the yield stress of the batten plates. The confined concrete compressive strength can be calculated using Eq. (2.7) and using $\upsilon = 1.0$.

For square columns strengthened with concrete jacket as shown in Fig. 2.30, the confinement effect will result from the stirrups in the concrete jacket. No contribution from the concrete jacket itself will be considered in increasing the confinement effect as the jacket might be cracked. The lateral confinement pressure f_1' can be expressed as per Eq. (2.23).

$$f'_1 = \left\{1 - \frac{2}{3}\left(\frac{(b_S - 2r)^2}{b_S^2}\right)\right\}\left(1 - \frac{s'}{2b_S}\right)^2 x2\frac{A_{str} f_y}{b_s S} \tag{2.23}$$

where b_s is the side length of the stirrup inside the jacket, s' is the clear spacing between stirrups of the jacket, S is the center line to center line spacing between the stirrups, r is the radius of chamfer at the column corners, A_{str} is the area of the

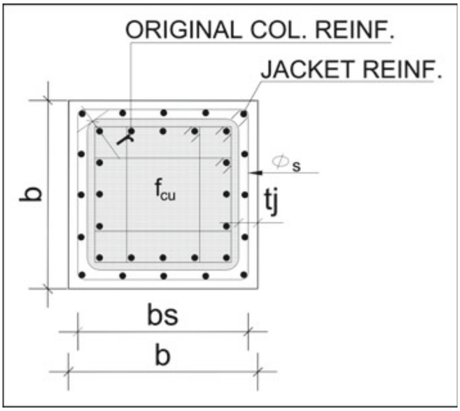


Fig. 2.30 Strengthening of square columns using RC Jacket

stirrups inside the concrete jacket and f_y is the yield strength of the stirrups inside the concrete jacket.

For square columns strengthened with fully wrapped FRP jackets ($s' = 0$), the same unified approach for confinement as presented before can be used as per Eqs. (2.24) and (2.25).

$$f_l' = \left\{1 - \frac{2}{3}\left(\frac{(b-2r)^2}{b^2}\right)\right\}\left(1 - \frac{s'}{2b}\right)^2 x2\frac{E_f n t_f S \in_{fe}}{bS} \tag{2.24}$$

$$f_l' = \left\{1 - \frac{2}{3}\left(\frac{(b-2r)^2}{b^2}\right)\right\} x2\frac{E_f n t_f \in_{fe}}{b} \tag{2.25}$$

where E_f is the elastic modulus of FRP, n is the number of plies, t_f is the thickness of each ply and ε_{fe} is the effective strain in FRP and is equal to 0.55 ε_{fu}

ACI 440.2-R17 provides conservative design guidelines for columns with aspect ratio less than 2.0 and column dimensions not exceeding 900 mm. An equivalent circular section of diameter, d, can be estimated as per Eq. (2.26).

$$d = \left(b^2 + h^2\right)^{0.5} \tag{2.26}$$

where b and h are the dimensions of the rectangular column section.

The confined concrete compressive strength, f_{cc}', is calculated using Eq. (2.7) and the effective lateral pressure, f_l', is calculated using Eq. (2.12) for continuous FRP wrapping. For FRP-strengthened rectangular or square sections, $k_e = 0.55$ and υ is equal to 0.95.

$$k_a = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^{0.5} \tag{2.27}$$

$$\frac{A_e}{A_c} = \frac{1 - \left\{ \frac{b/h(h-2r)^2 + h/b(b-2r)^2}{3A_g} \right\}}{1 - \rho_g} \tag{2.28}$$

$$f_l' = k_a x 2 \frac{E_f n t_f \in_{\text{fe}}}{d} \tag{2.29}$$

For square section, with $b = h$, Eq. (2.29) can be expressed in Eq. (2.30).

$$f_l' = \left\{ 1 - \frac{2}{3} \left(\frac{(b-2r)^2}{b^2} \right) \right\} x 2 \frac{E_f n t_f \in_{\text{fe}}}{d} \tag{2.30}$$

where r is the radius of chamfer at the column corners. Requirements of ACI 440.2 R-17 as far as ultimate concrete strain and serviceability requirements shall be strictly followed. It shall be noted that Eq. (2.30) takes the same format as Eq. (2.25) with the replacement of b in Eq. (2.25) by d in Eq. (2.30).

2.3.2.2 Parameters Affecting the Confining Pressure in Rectangular Columns

The effect of confinement was further investigated on a typical square column of 600 × 600 mm. Different key parameters are studied including:

a. Column dimensions, b (varied from 400 to 1200 mm)
b. Spacing of batten steel plates, S (varied from 200 to 600 mm)
c. Dimension of the corner angle, L_1 (varied from 50 to 150 mm).

Figures 2.31, 2.32 and 2.33 show the influence of different parameters on the confined concrete strength of square columns strengthened with steel plates. It was observed that increasing the column dimension, b, while keeping the same configuration of the batten plates reduces the lateral confined pressure, f_1', and hence reduces the confined concrete compressive strength. Doubling the column dimensions from 400 to 800 mm reduces the confined concrete strength by around 10% as shown in Fig. 2.31. Such an effect reduces by increasing the concrete original compressive strength.

On the other hand, reducing the spacing between batten plates has a considerable beneficial effect on the lateral confined pressure and the confined concrete compressive strength. For a 600 mm square column, reducing the spacing between batten plates from 600 to 300 mm, which is equivalent to $b/2$ as recommended by Eurocode 8 (CEN 2003) increases the confined concrete compressive strength by around 15% as illustrated in Fig. 2.32.

The dimensions of the corner angle, L_1, have a minor effect on the confinement of square columns as depicted in Fig. 2.33. Increasing L_1 from 50 to 150 mm increases the confined concrete compressive strength by less than 6%.

Figures 2.34, 2.35 and 2.36 show the influence of different parameters on the confined concrete strength of square columns strengthened with RC Jacket.

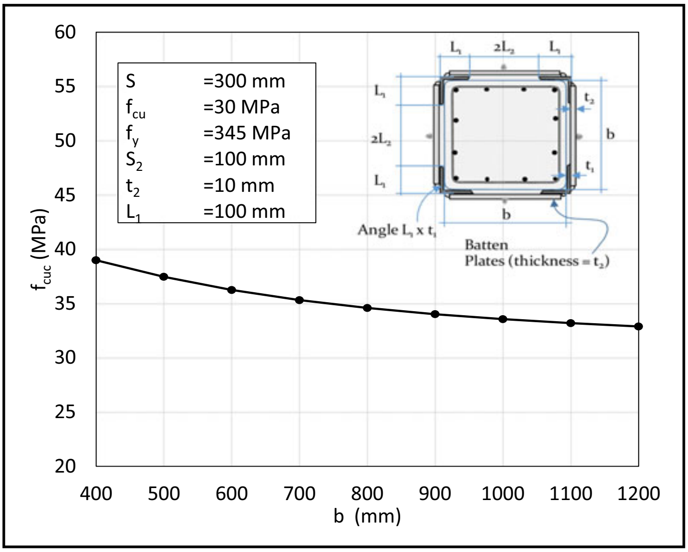


Fig. 2.31 Influence of column dimensions

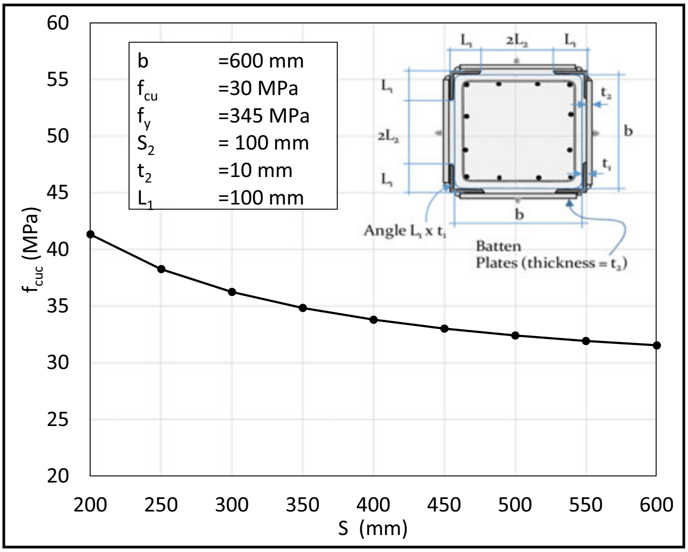


Fig. 2.32 Influence of batten plate

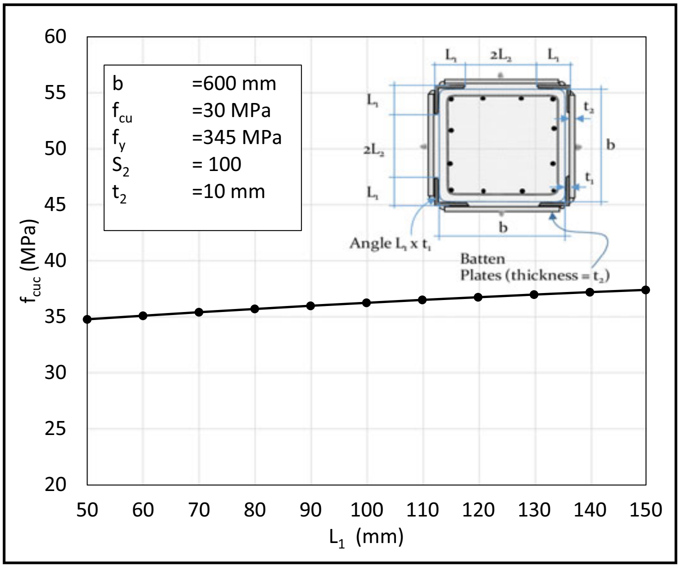


Fig. 2.33 Influence of angle length

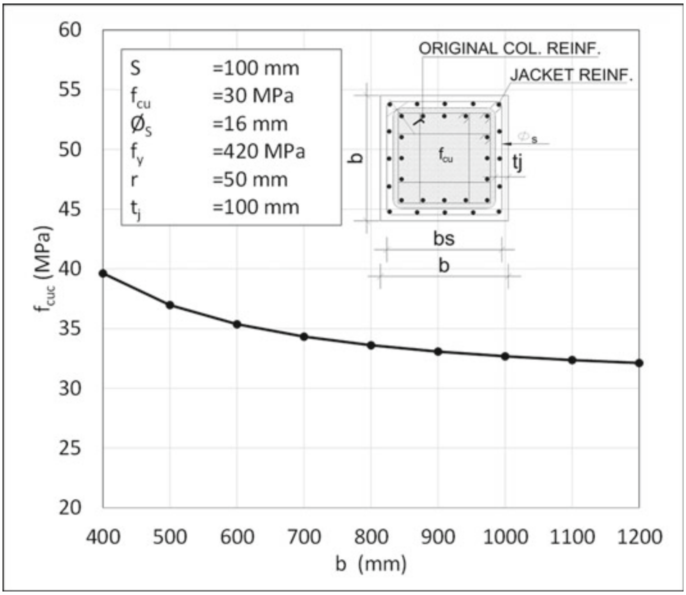


Fig. 2.34 Influence of column dimension

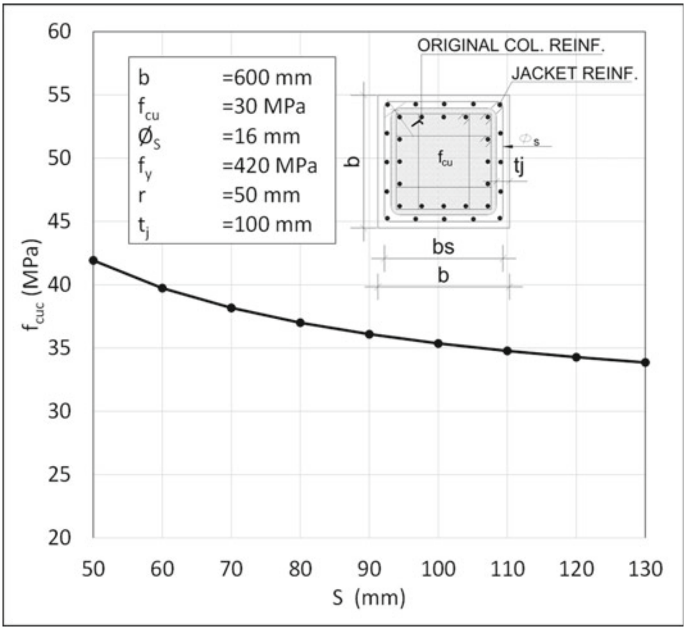


Fig. 2.35 Influence of jacket stirrup spacing

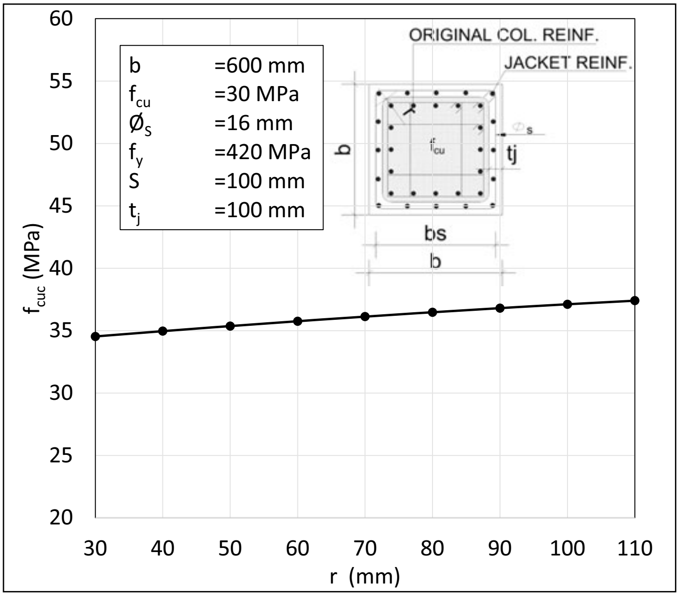


Fig. 2.36 Influence of chamfer radius, *r*

Three different parameters are investigated including:

a. Column dimensions, b (varied from 400 to 1200 mm)
b. Spacing between stirrups in the concrete jacket, S (varied from 50 to 130 mm)
c. Chamfer radius, r, (varied from 30 to 110 mm).

Column dimensions and spacing of the confining stirrups inside the concrete jacket are proven to be the most important parameters governing the confinement level in square concrete columns strengthened with RC jacket. It shall be noted that stirrups inside the concrete jacket are the only source providing confinement to square concrete columns. Confinement is highly pronounced in smaller concrete columns of dimensions less than 600 mm as shown in Fig. 2.34. Increasing the column dimensions will require closely spaced stirrups of large diameter bars to achieve an acceptable confinement level, which might be impractical in real applications. Therefore, it is recommended that confinement can be ignored for columns larger than 800 mm. Reducing the spacing between confining stirrups has a considerable beneficial effect on the lateral confined pressure and the confined concrete compressive strength. For a 600 mm square column, reducing the spacing between stirrups from 150 to 50 mm increases the confined concrete compressive strength by around 25% as illustrated in Fig. 2.35.

The chamfer radius, r, has a minor effect on the confinement of square columns as depicted in Fig. 2.36. Increasing r from 50 to 100 mm increases the confined concrete compressive strength by less than 5%. Such an effect diminishes by increasing the concrete original compressive strength.

This indicates that Mander et al. 1988 confinement model can be applied for calculating the confined concrete strength for square columns strengthened either with steel, concrete or FRP jackets. Such an effect results in conservative values for f_{cc}' predicted using ACI 440.2-R17 by around 13–20%.

Example 2.2 A square concrete column of dimensions 300 × 300 mm is subjected to the following ultimate straining actions:

- $P_u = 50$ tons
- $M_x = 10$ mt
- $M_y = 10$ mt.

Check the adequacy of the column if $f_{cu} = 25$ MPa using ACI 318–14.

Answer

The column is modeled on ETABS, and design of the section was conducted under the given straining actions using ACI 318–14. The column was found inadequate with Design-to-Capacity Ratio, D/C of 2.8.

The eccentricity divided by the column dimension in this example (*e/t*) is 0.667. Therefore, big eccentricity (tension failure) is dominant and hence, FRP strengthening will not be used. Two strengthening schemes will be investigated using either structural steel batten plates or concrete jacket.

Strengthening Scheme, I

The column is strengthened with structural steel batten plates of dimensions 80 × 6 mm spaced at 150 mm C.L. to C.L. Four angles measuring 100 × 100 × 10 mm are used at the four corners. The yield strength of the structural steel is 345 MPa. Equation (2.22) was used to evaluate the lateral confinement pressure, f_l' considering the following:

- $b = 300$ mm
- $t_2 = 6$ mm
- $S_2 = 80$ mm
- $S = 150$ mm
- $S' = 70$ mm
- $f_y = 345$ MPa.

The lateral confining pressure, f_l', was found to be 5.37 MPa.

Using Eq. (2.7), the confined concrete compressive strength was found to be 42.7 MPa.

The column cross-section was modeled on ETABS using section designer and accounting for the four angles at the corners. Design of the section was conducted under the given straining actions using ACI 318–14. The column was found adequate with Design-to-Capacity Ratio, D/C of 0.76.

Strengthening Scheme, II

The column is strengthened with RC jacket of 70 mm thick. The compressive strength of the jacket was set to 35 MPa. The jacket is reinforced with 12T12. Equation (2.23) was used to evaluate the lateral confinement pressure, f_l' considering the following:

- $\Phi_s = 10$ mm
- $t_j = 70$ mm
- $r = 50$ mm
- $S = 150$ mm
- $S' = 140$ mm
- $f_y = 420$ MPa.

The lateral confining pressure, $f_{l,}'$ was found to be 0.54 MPa. Using Eq. (2.7), the confined concrete compressive strength was found to be 26.8 MPa.

The column cross-section was modeled on ETABS using section designer, and design of the section was conducted under the given straining actions using ACI 318–14. The column was found adequate with Design-to-Capacity Ratio, D/C of 0.63.

Column cross-sections for different strengthening schemes are illustrated in Fig. 2.37, and the corresponding interaction diagrams are given in Fig. 2.38.

Example 2.3 A square concrete column of dimensions 300 × 300 mm is subjected to the following ultimate straining actions:

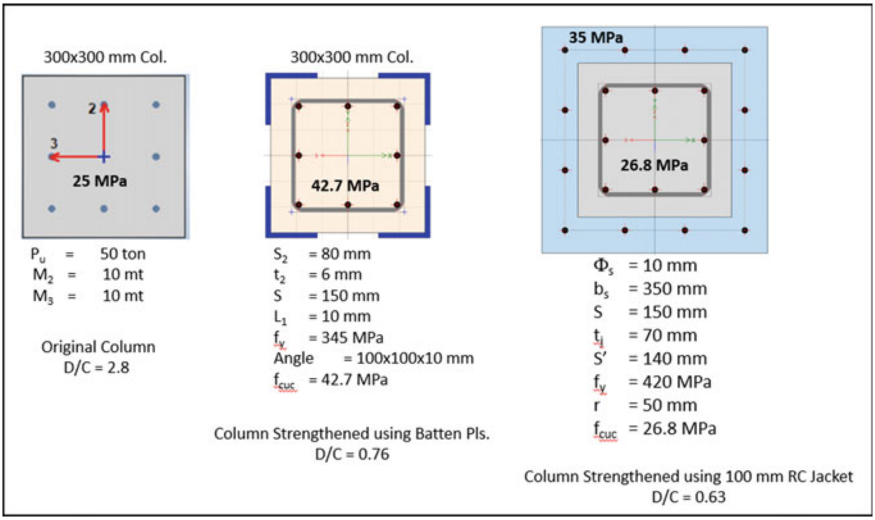


Fig. 2.37 Column cross-sections for different strengthening schemes (Example 2.2)

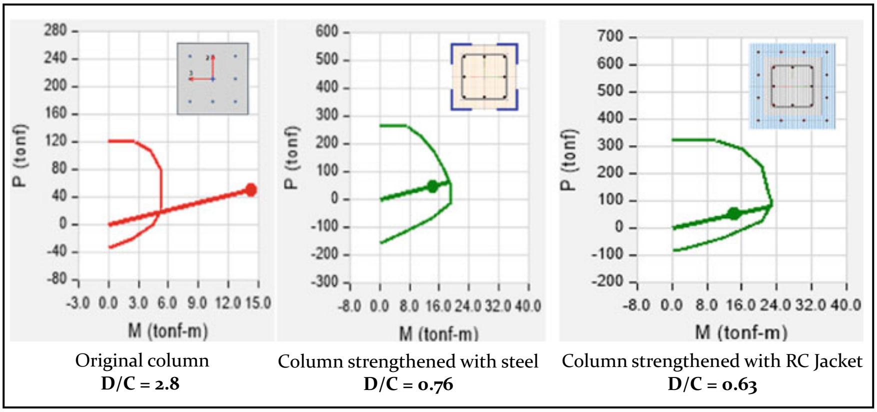


Fig. 2.38 Column interaction diagrams for different strengthening schemes

- $P_u = 200$ tons

  Check the adequacy of the column if $f_{cu} = 25$ MPa using ACI 318–14.

Answer

The column is modeled on ETABS, and design of the section was conducted under the given straining actions using ACI 318–14. The column was found inadequate with Design-to-Capacity Ratio, D/C of 1.7.

The eccentricity divided by the column dimension in this example (*e/t*) is zero. Therefore, small eccentricity (compression failure) is dominant, and hence, FRP strengthening will be investigated in addition to the previous two strengthening schemes described in the previous example.

Strengthening Scheme, I

The column is strengthened with structural steel batten plates of dimensions 80 × 6 mm spaced at 150 mm C.L. to C.L. Four angles measuring 100 × 100 × 10 mm are used at the four corners. The yield strength of the structural steel is 345 MPa. Equation (2.22) was used to evaluate the lateral confinement pressure, f_1' considering the following:

- $b = 300$ mm
- $t_2 = 6$ mm
- $S_2 = 80$ mm
- $S = 150$ mm
- $S' = 70$ mm
- $f_y = 345$ MPa

The lateral confining pressure, f_1', was found to be 5.37 MPa.

Using Eq. (2.7), the confined concrete compressive strength was found to be 42.7 MPa.

The column cross-section was modeled on ETABS using section designer and accounting for the four angles at the corners. Design of the section was conducted under the given straining actions using ACI 318-14. The column was found adequate with Design-to-Capacity Ratio, D/C of 0.77.

Strengthening Scheme, II

The column is strengthened with RC jacket of 70 mm thick. The compressive strength of the jacket was set to 35 MPa. The jacket is reinforced with 12T12. Equation (2.23) was used to evaluate the lateral confinement pressure, f_1' considering the following:

- $\Phi_s = 10$ mm
- $t_j = 70$ mm
- $r = 50$ mm
- $S = 150$ mm
- $S' = 140$ mm
- $f_y = 420$ MPa

The lateral confining pressure, f_1', was found to be 0.54 MPa.

Using Eq. (2.7), the confined concrete compressive strength was found to be 26.8 MPa.

The column cross-section was modeled on ETABS using section designer, and design of the section was conducted under the given straining actions using ACI 318-14. The column was found adequate with Design-to-Capacity Ratio, D/C of 0.63.

Strengthening Scheme, III

The column is strengthened with four plies of externally bonded CFRP sheets. Equation (2.12) was used to evaluate the lateral confinement pressure, f_l' considering the following:

- $\varepsilon_{fu} = 0.0152$
- $E_f = 210$ GPa
- $f_{fu} = 3200$ MPa
- $k_a = 0.7$ Eq. (2.24)
- $Ae/Ac = 0.7$ Eq. (2.25)
- $r = 50$ mm
- $t_f = 0.29$ mm
- $n = 4$

The lateral confining pressure, f_l', , using ACI approach (Eq. 2.30) is 6.72 MPa and using the unified approach (Eq. 2.25) is 9.5 MPa. Using Eq. (2.7), the confined concrete compressive strength if using ACI approach is 46 MPa while using the unified approach is 54.8 MPa, which is 19% higher than predicted using ACI 440.2-R17. The column cross-section was modeled on ETABS considering the column compressive strength of 54.8 MPa. Design of the section was conducted under the given straining actions using ACI 318–14. The column was found inadequate with Design-to-Capacity Ratio, D/C of 0.85.

Column cross-sections for different strengthening schemes are illustrated in Fig. 2.39, and the corresponding interaction diagrams are given in Fig. 2.40.

2.4 Stress Limits Design Recommendations

Calculations may show that increase in the concrete confining pressure after jacketing are increasingly high. This is true when the jacket stiffness is large compared to the stiffness of the original column. Accordingly, the calculated axial capacity of the original column after strengthening will be much higher than that before strengthening, and therefore, the participation of the original column in carrying the additional axial loads shall be also high. In this case, it is preferable to put a threshold of the axial capacity of the original column after strengthening and allow the axial strength of the jacket to participate in the overall column strength. This may be achieved by limiting the confined concrete compressive strength, f_{cc}' so that it does not exceed twice the uniaxial concrete compressive strength of the original column, as per Eq. (2.31). It is worth noting here that axial strength of the jacket can be included for columns strengthened with concrete or steel jackets only, while no axial strength exists for FRP jackets.

$$f_{cc}' \leq 2\, f_c' \tag{2.31}$$

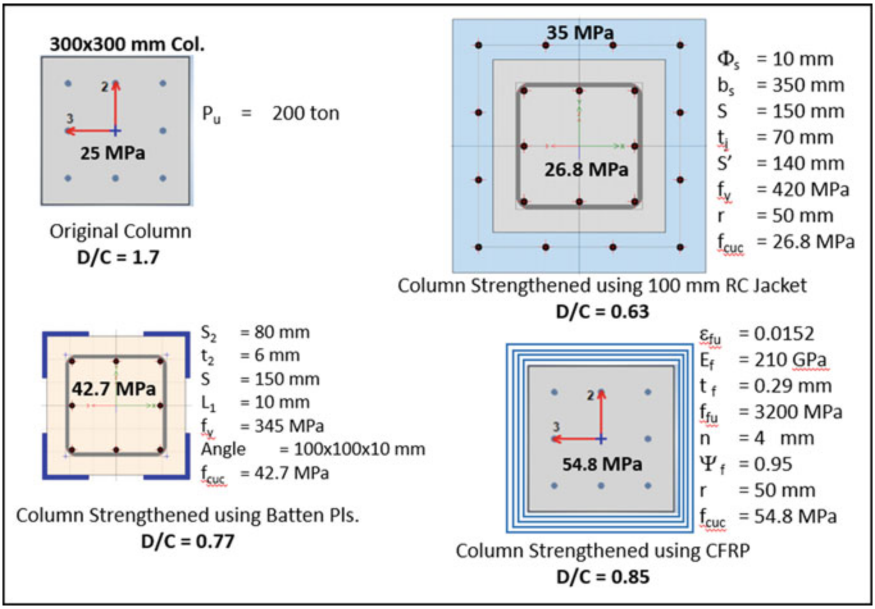


Fig. 2.39 Column cross-sections for different strengthening schemes (Example 2.3)

At service load, the ACI 440.2R-17 recommends limiting the concrete and steel stresses below levels that will ensure that no concrete cracking or plastic steel deformations under sustained or cyclic loading shall occur. Table 2.5 gives the limiting stresses in concrete columns under full service loads.

2.5 Columns Replacement

Concrete columns lacking significant capacity compared to the demand level may be replaced by adding new vertical members made of concrete or steel to carry the induced loads. This may be done for small number of columns, while it is not recommended to replace large number of columns in the structure. It is challenging to construct reliable connections between the new vertical members and the floors of the existing structures.

The new columns should be placed so that they can carry all the gravity shearing forces induced on the floor, as shown in Figs. 2.41 and 2.42 for interior and edge columns. The new columns should surround the original deficient column in order to have their share from the gravity loads on the slabs. Columns in this case may be modeled as pinned columns since fixity of the columns at its top and bottom will occupy large areas of the floor with impractical drilling in the slabs to provide the fixation. The new columns shall be designed to carry vertical loads and the

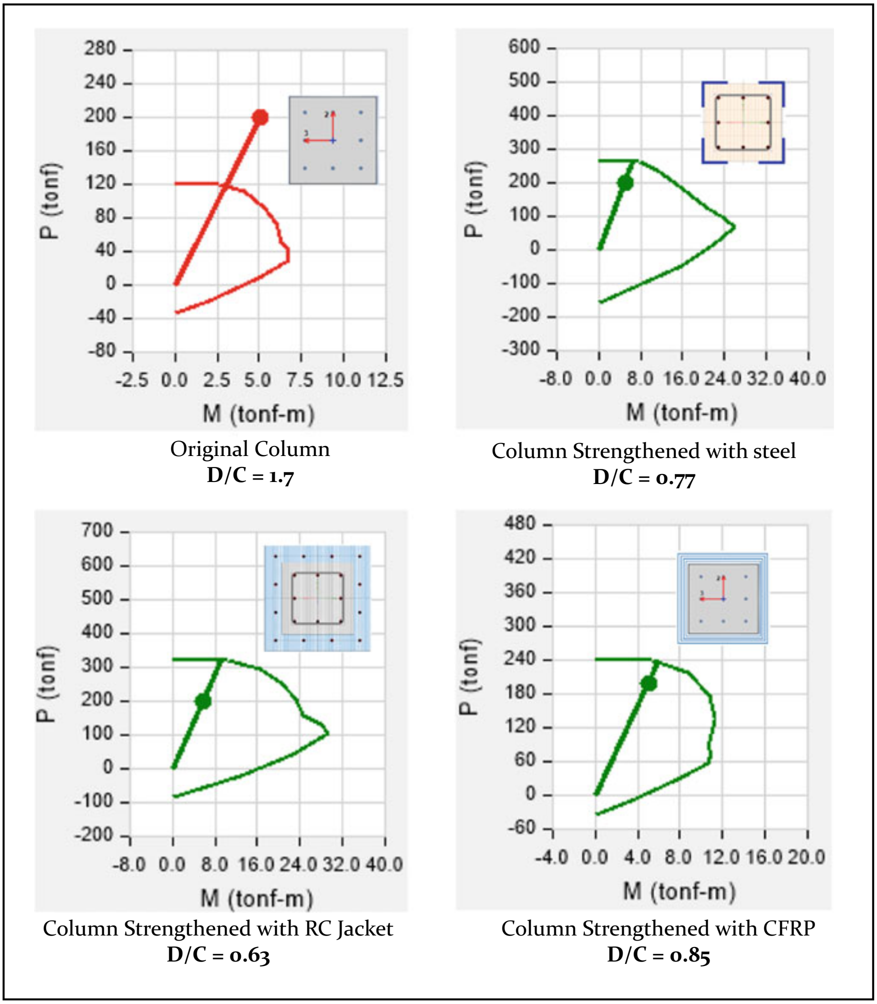


Fig. 2.40 Column interaction diagrams for different strengthening schemes

Table 2.5 Allowable service stresses in the original section

Member	Allowable stress in steel	Allowable stress in concrete
Concrete column	$f_{s,s} \leq 0.6 f_y$	$f_{c,s} \leq 0.65\ f'_c$

contribution of the original columns, which are replaced by the new columns, to the capacity of the structure shall be ignored. Meanwhile, the rest of the columns in the structure, which were not replaced by new ones, shall be checked to sustain both the vertical and horizontal loads on the structure.

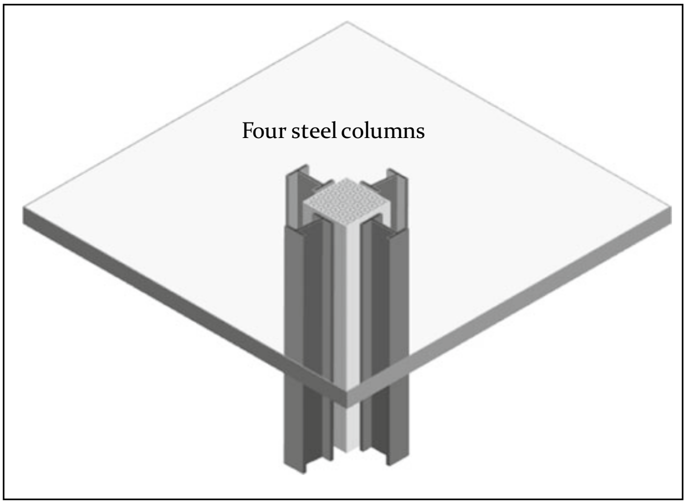


Fig. 2.41 Replacement of interior column with new steel columns

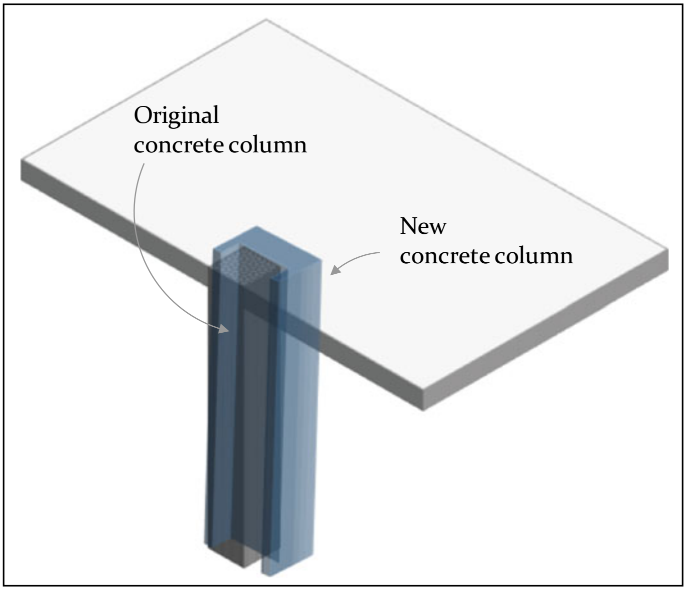


Fig. 2.42 Replacement of edge column with new concrete column

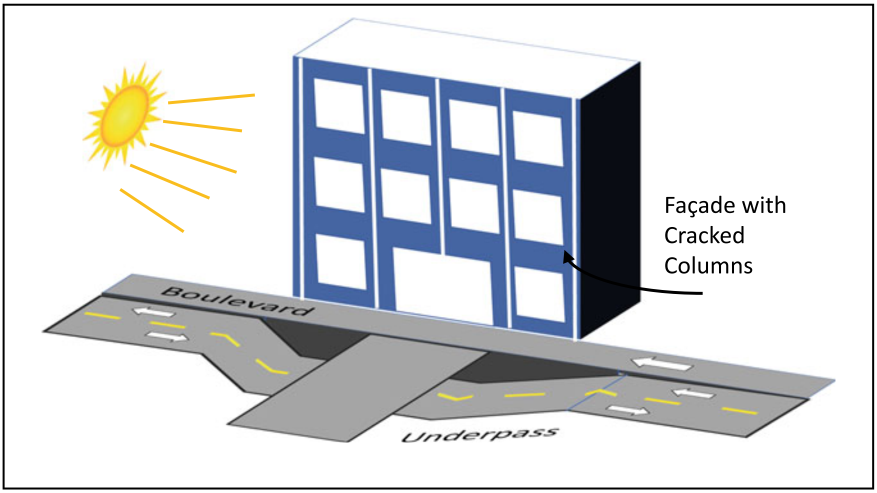


Fig. 2.43 Layout of building

2.6 Case Study 2.1

This case study is for a building, which consists of ground floor, Mezzanine and roof, in addition to upper roof deck level. The structural system of the floors consists of reinforced concrete ribbed slab made of RC ribs and hidden beams, in addition to drop beams, which is supported on columns with dimensions varying from 300 × 700, 200 × 600 to 200 × 400 mm. The spans between the internal RC columns are 8.0 m; however, the spans between the external columns are 4.0 m. The front façade of the building is facing south direction, and the building is located on a boulevard, where an underpass exists in front of the building, as shown in Fig. 2.43.

2.6.1 Problem Description

Cracks were observed at several edge columns during installation of a new stone cladding at the main façade of the building facing the boulevard. Those cracks appeared in the concrete and at the interface between edge columns and the block work (Fig. 2.44).

Material testing showed that the concrete compressive strength was in the range of 25 MPa. Analysis of the building showed that the columns are safe to tolerate the applied vertical and horizontal loads, which are induced from the total dead, live, wind and seismic loads. It was observed that parts of the edge columns located at the façade line are exposed outside the building, while other parts of the columns are inside the building (see Fig. 2.45).

Fig. 2.44 Cracks at the edge column

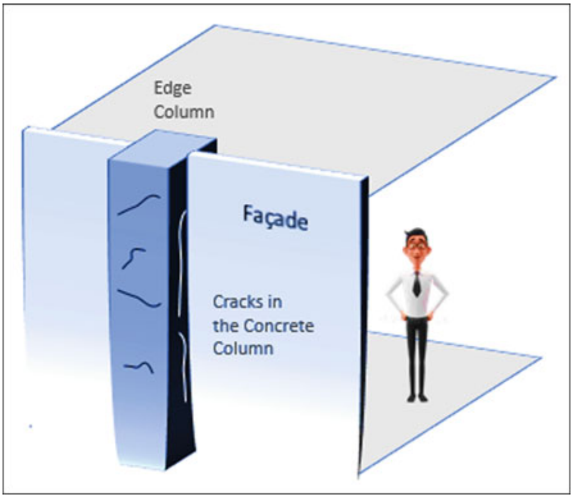


Fig. 2.45 Schematic of the façade

Since the structural strength of the columns is adequate to resist the induced loads, cracks in the columns are attributed to one or more of the following reasons:

1. The building is located on the boulevard with its south façade exposed to the sun all day long; accordingly, high stresses are induced from the temperature effect;

especially that part of the edge columns is exposed to the ambient temperature, while the other part is exposed to the conditioned air temperature inside the building. It is worth noting that the ambient temperature may reach 45 °C in the summer, while the air temperature inside the building may reach 20 °C. Figure 2.46 shows the temperature effect on the edge columns, which are directly exposed to weather. The effect of temperature leads to tension stresses in the vertical direction, which is restrained by compression in the inner part of the column, as shown in the figure. These stresses may result in cracks on the surface of the outer part of the column.

2. It was also observed that a tunnel was executed in the boulevard at the vicinity of the building façade. Excavation of the tunnel may have resulted in disruption of the soil beneath the foundations of the building, which could be the reason for movement of the south façade of the building. This outward movement of the façade shall result in tensile stresses/cracks in the edge columns.
3. The in-situ tests of the concrete columns showed that a concrete cover of more than 60 mm thick exists at different locations of the columns. This large concrete cover, when exposed to change in temperature, shall result in surface cracks separating the cover from the core of the columns.

2.6.2 Concrete Repair

Before commencement of the repair works of the edge concrete columns, enough data was collected to ensure that there is no movement of the building. This data was verified through continuous monitoring of the façade deformations. After ensuring that cracks' widths are not increasing, edge columns were repaired. The repair work was carried out by injecting the cracks using low-viscous epoxy. Concrete repair work of the columns to resist the additional stresses induced by temperature variations was applied using carbon fiber reinforced polymer, (CFRP), sheets, as per Fig. 2.47.

Two layers of the sheets were applied with the fibers in the transverse and longitudinal directions. The purpose of applying the fibers was to arrest the cracks and stop further propagation of cracks. Periodic inspection was made to check the structural condition of concrete columns at the façade, especially during summer, where temperature is at its peak.

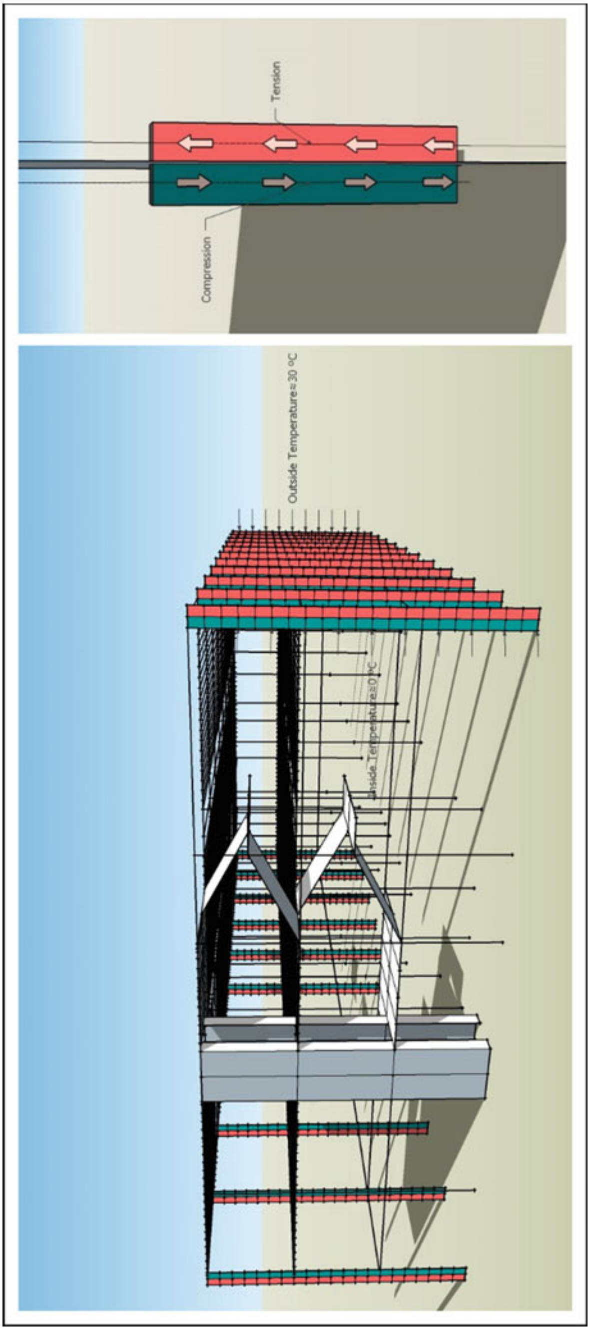


Fig. 2.46 Induced stresses in the edge columns due to temperature

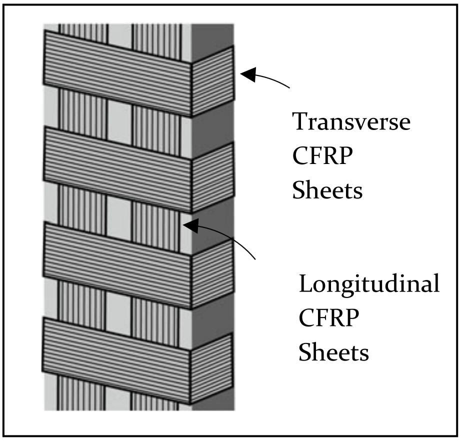


Fig. 2.47 Column strengthening with CFRP

Chapter 3
Beam Strengthening

Structural engineer should investigate the existing concrete members and collect all possible information to diagnose the safety and serviceability conditions before commencement of any repair or strengthening procedure. The amount of steel corrosion, member deformation, deflections, cracking, vibration, material defects and member capacity should be identified in order to choose the best rehabilitation scheme. The rehabilitation process should start with repair of the existing members including removal of steel corrosion, steel protection and crack injection, possibly rectifying member deformations/deflections, and maintain good quality concrete. The concrete substrate should be cleaned from any loose material or debris, and the jacket should be installed on rough and sound surface. The concrete roughness shall be reached using either sand blasting, water blasting or equipment producing low vibration. No sledge hammer or pneumatic hammer of large weight (more than 10 kg) should be used in order not to create surface/internal cracks in the concrete substrate.

Several strengthening methods can be used to improve the beam capacity, stiffness or serviceability, which include concrete, steel or FRP jackets. Concrete jackets may be reinforced or prestressed, which in turn could be post-tensioned or externally applied. Steel plates or sections may be used to strengthen the concrete beams, which may be chemically or preferably mechanically bonded to the concrete surface. Mechanical bond between the substrate and the jacket is provided using anchor bolts or steel dowels. FRP jackets may be applied using externally bonded laminates or near-surface-mounted reinforcement. FRP laminates or bars may be made of carbon, glass, aramid or basalt fibers, while chemically bonded to the concrete using adhesives such as epoxies or vinylester. The short-term and long-term characteristics of the jacket material should be compatible with the material of the substrate in order to avoid peeling, flaking, chipping or disintegration of any form due to change in temperature, humidity, ultraviolet radiation or other environmental conditions. In both cases, bond should be able to transfer the interfacial shear to utilize the full structural capacity of the jacket.

A. Abdelrahman, *Strengthening of Concrete Structures*,
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3.1 Concrete Jacket

It is preferable that concrete material of the jacket has the same type of aggregate and with compressive strength higher than or at least equal to that of the original concrete member. Concrete should have low shrinkage and preferably self-consolidated since the steel reinforcement ratio in the jacket is usually high. The substrate shall be roughened to at least 5 mm amplitude in order to increase the friction between the old and new concrete. Installing the steel dowels shall be carried out using epoxy adhesives in predrilled holes with diameter of 2–4 mm larger than that of the dowels including the bar deformations. The drilled holes should avoid the location of the existing steel bars and should be cleaned from dust or loose material. Quality control should be implemented by testing the installed anchors randomly and measure its pull-out force. The steel cage shall be fixed, and the jacket shall be cast in a formwork using either the form-and-pump technique, shotcrete or self-consolidated concrete with 9 mm maximum aggregate size.

3.1.1 Reinforced Concrete Jacket

Figure 3.1 shows typical details for flexural and shear beam strengthening using concrete jacket. The overall concrete section is increased in dimensions in addition to the internal steel reinforcement including the longitudinal bottom, top and transverse bars. Part of the bottom longitudinal steel reinforcement should be planted in the columns to comprehend the arching action behavior. In case that top reinforcement is needed to increase the moment capacity above the supporting columns, steel bars shall be placed in the top concrete overlay outside the beam section to avoid the column location, as shown in the figure.

For shear strengthening, the stirrups should be extended in the concrete compression zone in order to realize its full capacity. The stirrups shall penetrate through the existing concrete flange and be embedded in the newly cast overlay after bending each leg to achieve the required lap length of the steel bars. In case that no top concrete overlay is used, planting the stirrups in the top flange only shall not allow full utilization of the stirrup capacity. It is then recommended to anchor each leg of the stirrups using a nut, as shown in the figure. The nut should develop the full force in the stirrup leg. The exposed steel bars and the nuts should be protected from corrosion using anti-corrosion paint.

3.1.2 Prestressed Concrete Jacket

Flexural strengthening of concrete beams may be applied using prestressed concrete jacket. Prestressing shall enhance the serviceability limit states of the existing

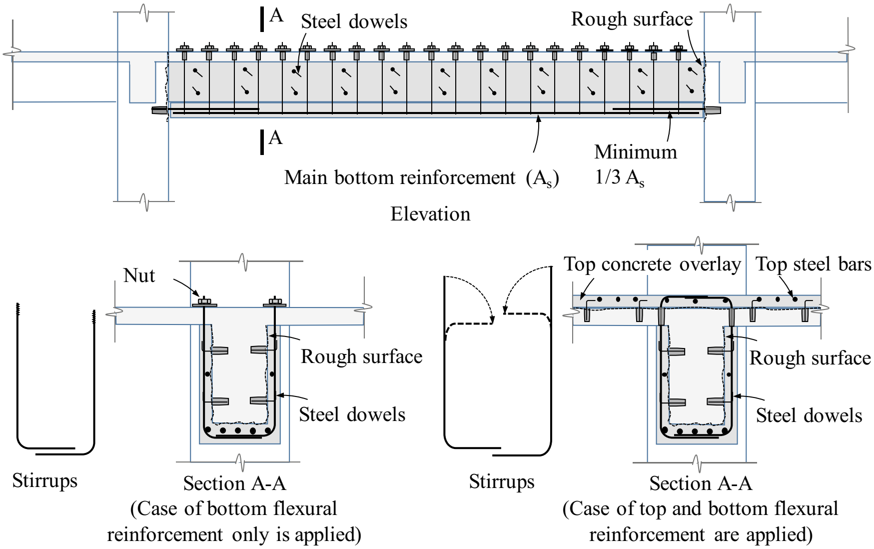


Fig. 3.1 Concrete jacket for RC beams

concrete member since it will close the existing cracks and reduce the existing deflection. This "active strengthening" procedure results in that the concrete jacket shall have its share in carrying the existing loads on the beams at the time of strengthening including the self-weight of the concrete member. This is unlike the "passive strengthening" concept when an RC jacket will share in carrying only the additional loads imposed on the beam after completing the strengthening works. It is advisable to use prestressed concrete jacket when the serviceability limit states are crucial or when the original concrete member is prestressed. It is not advised to use this type of strengthening when the concrete strength of the original member is low since the long-term losses of the prestressing force due to creep shall be high. Prestressing may be applied using post-tensioned concrete jacket or externally unbonded prestressing steel reinforcement.

Post-tensioned concrete jacket should have a minimum thickness that allows enough space for the diameter of the duct and the steel cage, which is normally larger than that of RC jacket. The thickness of the jacket is even increased at the ends to allow for the jacking and dead ends fixtures of the prestressing steel strands, which ranges from two to three times the diameter of the duct. Special steel reinforcement should be placed at the jacking and dead ends to resist the splitting tensile forces induced in the concrete at the time of applying the force to the jacket. The prestressing tendons may consist of circular ducts containing multi-strands or several flat ducts containing up to five strands each, as shown in Fig. 3.2. The flat ducts are commonly used for slabs, and it gives the benefit of reducing the jacket thickness as well as good distribution of the forces at both ends of the beam.

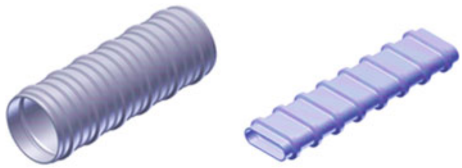


Fig. 3.2 Prestressing steel ducts

Profile of the prestressing steel strands should be as close as possible to the shape of the induced bending moment on the strengthened beam in order to minimize the final flexural stresses on the beam. The shape of the prestressing steel should be well secured in place before casting the jacket. Concrete properties used for the jacket should contain small aggregate size and flowable enough to fill the gaps around the ducts and the non-prestressed steel. Enough space behind the jacking ends should be left in order to allow for the jacks to apply the prestressing forces on the steel strands. The prestressing steel stands are only tensioned when the concrete of the jacket reaches the initial compressive strength, as per the design recommendations. The prestressing force is transmitted to the original concrete section through shear in the steel dowels. Extra care should be given to planting the shear dowels since the success of the jacket depends on prevention of any slip between the jacket and the original section. If slip occurs at the interface between the new and old concrete surfaces, the prestressing force shall not be transmitted to the concrete beam. After completing the stressing procedure and grouting the ducts, RC jackets are cast at the ends of the beam, which was left to allow room for prestressing the steel strands. This is only done if the shear stresses in this zone are critical and shear strengthening is required to accompany the flexural strengthening, as shown in Fig. 3.3.

Prestressing concrete beams may be also carried out using unbonded tendons externally fixed to the structural member, as shown in Fig. 3.4. In this case, application requires adding fixtures at both ends of the beam to be used as blocks for tensioning the tendons. These fixtures may be made of steel or concrete; yet, it has to be well anchored to the concrete member by steel anchors or bolts.

Profile of the external prestressing steel tendons may be draped at one or two points as shown in the figure. It is important to pay attention to the minimum radius of the prestressing steel strands at the deviator, which is 2.0 m for small ducts, as shown in Fig. 3.5, and to provide friction-reducing material between the deviator and the tendon. This is to minimize the effect of the fretting fatigue that will increase due to bending the cables at the deviator and due to vertical deformation of the beam under the effect of gravity loads. The designer should select the number of deviators along the beam and the eccentricity of the tendons so that each section along the beam satisfies the code requirements. The prestressing tendon is a mono- or multi-strand, well protected from corrosion by coating and plastic sheath, as shown in Fig. 3.6. Application of external prestressing system to increase the beam capacity and improve its serviceability is fast; however, maintenance of this system is essential to avoid corrosion or loss of the prestressing force with time. The exposed steel

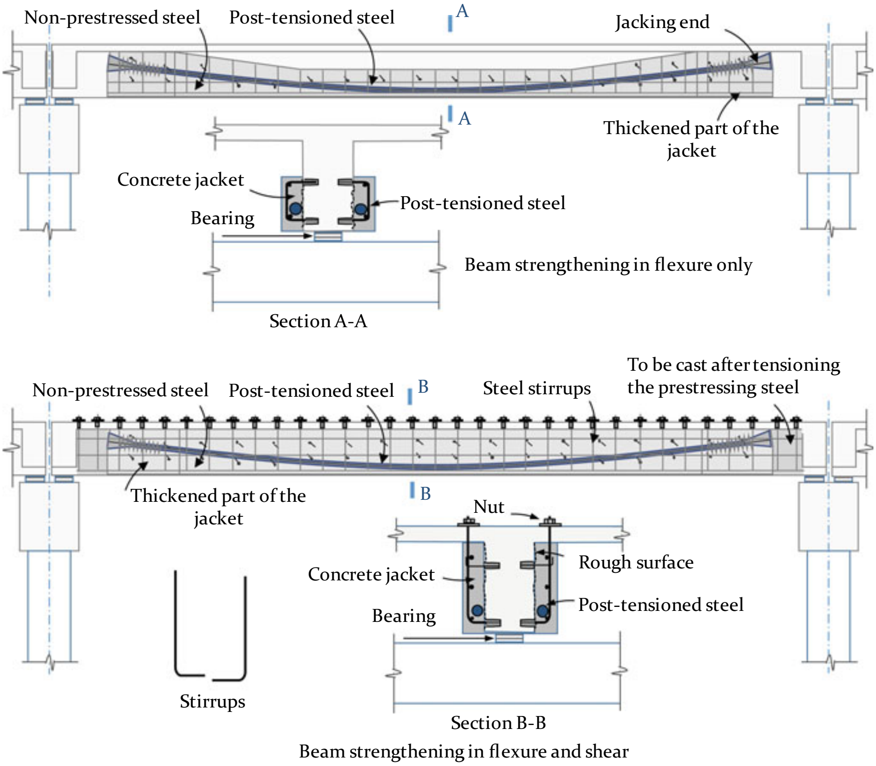


Fig. 3.3 Post-tensioned concrete jacket

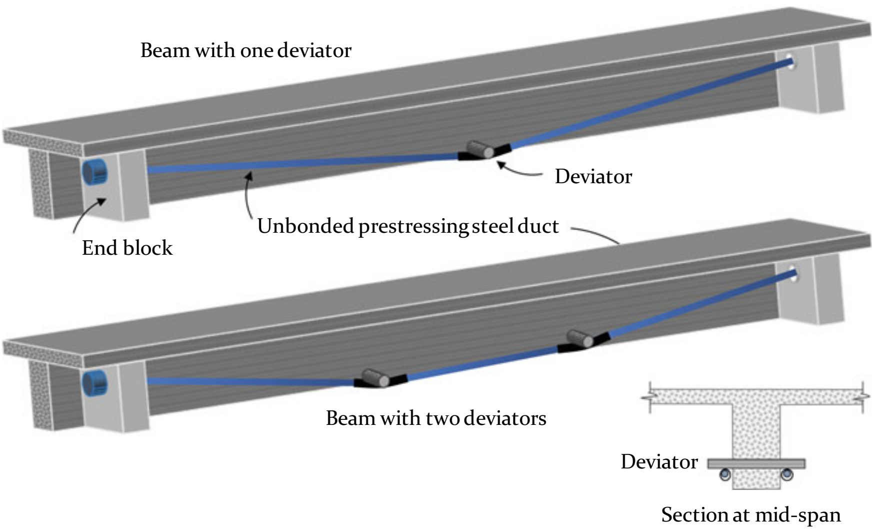


Fig. 3.4 Unbonded prestressing of concrete beams

Fig. 3.5 Min. radius of prestressing steel

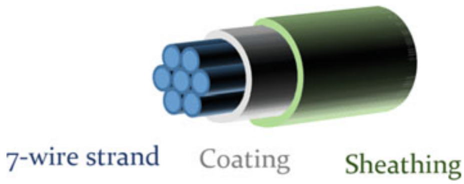


Fig. 3.6 External prestressing steel strand

tendons should be also protected against fire, either by enclosure or by fire-resistant paint.

Unbonded prestressing fiber reinforced polymer bars are also used for strengthening concrete beams (El-Hacha et al. 2001), which has the benefit of being non-corrosive compared to steel tendons. Tensioning the FRP bars should be carried out using special anchors compatible with the FRP bars to avoid premature failure at the anchorage zone. Analysis of the beams strengthened with unboned or external steel or FRP prestressing reinforcement depends on the overall deformation of the beam to calculate the stresses in the reinforcement, where the stresses in the unbonded reinforcement are constant all over its length. This is different from the analysis of concrete beams with bonded system, where strain compatibility is applied across the concrete section at service and ultimate load stages. Several formulae are given to calculate the stresses in the external prestressing reinforcement (Ghallab et al. 2004).

3.2 Steel Jacket

Steel plates or steel sections may be used as reinforcement to increase the capacity of the concrete section. Despite that chemical adhesives were previously used to bond the steel plates to concrete surface, it is much more effective to bond the steel sections to the concrete member using either chemical anchors or expansion steel bolts. Design of those anchors should address the embedment length of the anchor, spacing between anchors, edge distance and material properties including substrate concrete (strength, cracking state), steel and adhesives (if used).

Beam capacity strengthened to increase its flexural capacity is at its utmost if the steel sections are bonded to the soffit of the concrete section; however, this might not be practical since this zone is usually congested with internal steel bars. It is then advisable to fix the steel sections to the tension zone of the web, as shown in Fig. 3.7, to minimize the risk of cutting any of the longitudinal reinforcement, and the flexural capacity should be calculated accordingly. In case that shear strengthening

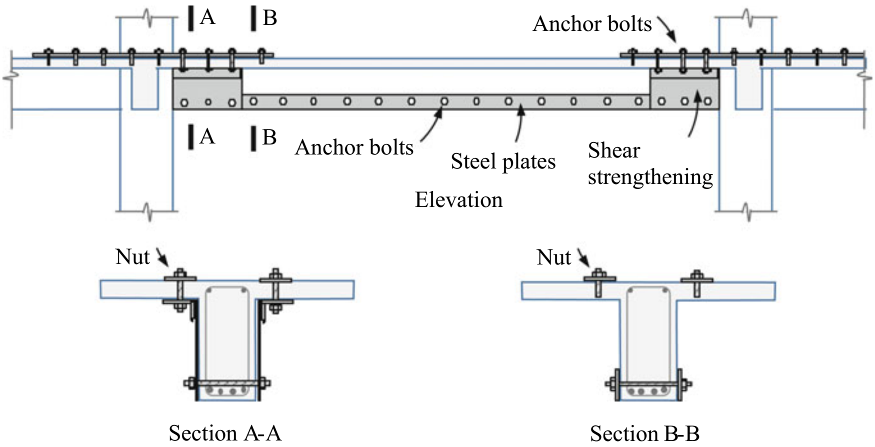


Fig. 3.7 Beam strengthening with steel plates

is required, attaching steel plate of the full web thickness is required, where it shall be anchored at the bottom of the web and to the top concrete flange.

Grout should be used between the steel sections and the concrete in order to prevent humidity or dampness to reach this zone. Welding the steel members should be avoided when epoxy or any other thermosetting resin is used in the strengthening scheme, since the heat produced by welding shall soften the resin, and hence, it will not be effective. The steel should be painted with anti-corrosion if not fire-rated paint, as per the requirements of the fire strategy of the building. The maximum strengthening limit given in Eqs. (1.10) and (1.11) may be imposed in cases when steel jackets are used with no fire protection.

3.3 FRP Jacket

Fiber reinforced polymer reinforcement is produced in the form of strips with thickness of 1.2 and 1.4 mm, or in the form of sheets, similar to that used in column strengthening and in the form of bars of 6, 8, 10, 12 and 16 mm diameter. FRP systems are made of fibers and resin, where its function is to bond the fibers together and to the concrete member. FRP is a laminated structure, where the fibers are oriented in one or multi-directions (for the sheets only). The commonly used fiber in structural strengthening is the carbon fiber since it has the highest strength and modulus, while glass, aramid and basalt fibers are less used in strengthening because of its low modulus of elasticity, which requires higher reinforcement ratio to satisfy the serviceability limit state requirements. FRP jackets are aesthetically appealing since it has limited thickness and it does not change the architectural appearance of the member. It has also the advantage of being corrosion-free, high strength, where it can reach five times that of the steel, and lightweight where its density is 20–25% that of steel. However, it has almost no resistance to fire and low resistance to elevated temperature unless special resins are used.

Unlike steel reinforcement, there are no universal standard for production of the FRP reinforcement. Therefore, the manufacturer data of the FRP material should be reviewed carefully and verified experimentally before application unless received by a certified producer, who applies quality control protocol on the manufactured materials. The sheets, strips and bars are commonly used for flexural strengthening, while only the sheets are used for shear strengthening since it can be wrapped around the beams. FRP laminates in the form of sheets or strips are externally bonded to the concrete surface, while FRP bars and strips are mounted in grooves near the concrete surface made in the concrete cover.

In all cases, FRP application for strengthening in flexure, shear, torsion or axial tension is bond-critical, unlike the FRP application to increase the axial compression capacity, where it is contact-critical. The bond-critical application requires good surface preparation and minimum requirements for the concrete strength, where a value of the concrete tensile strength should not be lower than 1.4 MPa. The surface humidity and evenness should be measured before application; in addition, the concrete surface should be free from cracks, voids or any loose material. Application of the FRP system should be done by certified applicators using resins and fibers produced by the same supplier since compatibility between the materials is essential. It should be noted that bond is the main design criterion that controls the system capacity. Two bond failure modes may occur for the FRP reinforcement: plate end debonding, where the end of the laminates fails by bond, or intermediate crack debonding, where the bond failure starts at the middle of the beam at a flexural crack location and extends toward the end of the beam. Both modes should be checked and included in the design.

Figure 3.8 shows the layout of concrete beam with externally bonded FRP in flexure and shear. The longitudinal FRP laminates are placed first on the concrete surface and covered with the transverse laminates. This will enhance the FRP bond strength since the laminates will act as anchorages in addition to being transverse reinforcement, as shown in Fig. 3.9. The increase in bond strength is about 30% compared to that of the laminates without transverse anchorages.

For near-surface-mounted (NSM), FRP systems, the minimum dimension of the grooves should be 1.5 times the diameter of FRP bar or 3.0 times the thickness of FRP strip. The minimum clear spacing between the grooves containing the NSM FRP bars should be twice the depth of the groove to avoid stress concentration. Furthermore, a clear edge distance of four times the depth of the groove should be provided to avoid debonding failure. Figure 3.10 shows the minimum dimensions of the grooves of the NSM FRP system. Hassan and Rizkalla (2003) concluded that NSM FRP reinforcement has better bond characteristics than the externally bonded FRP strips and consequently better performance.

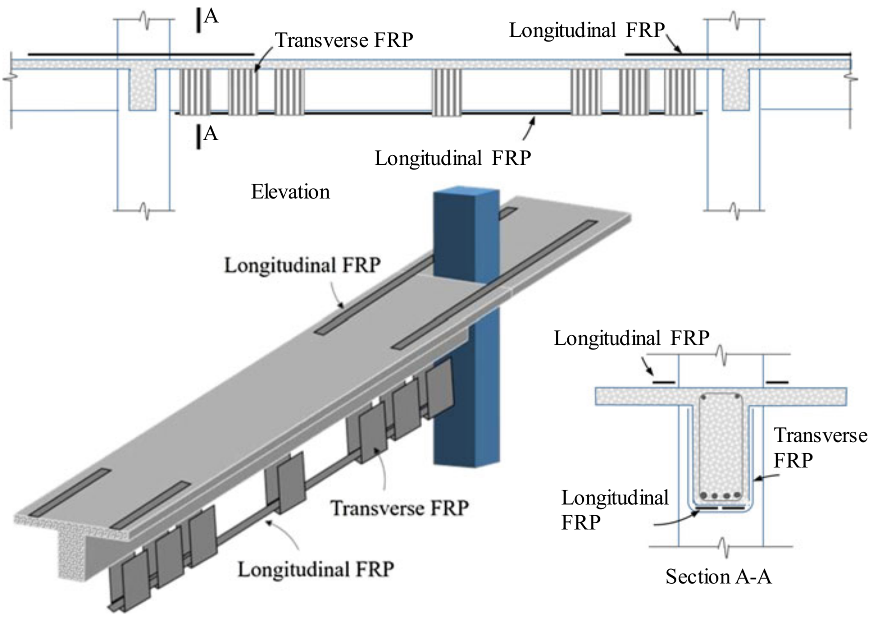


Fig. 3.8 Beam strengthening with externally bonded FRP

Fig. 3.9 Externally bonded FRP

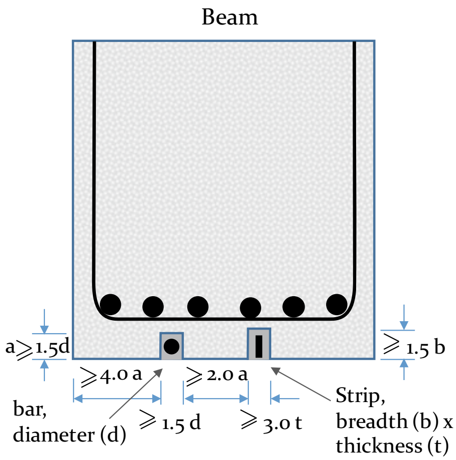


Fig. 3.10 NSM FRP reinforcement

3.4 Strength Capacity of Jacketed Beams

A unified design methodology is used to calculate the structural capacity of concrete beams with concrete, steel or FRP jacket. The strength is calculated taking into consideration the following parameters:

1. Existing strain in the member at the time of strengthening application.
2. Failure mode of the strengthened members.
3. Ductility of the strengthened members.
4. Maximum ratio between the strengthened and the unstrengthened sections.
5. Shear strength of the new section.
6. Bond stresses and development length of the jacket reinforcement.
7. Interfacial shear between the jacket and the original concrete section.

3.4.1 Flexural Strength of Strengthened Beams

Figure 3.11 shows the strain and force distribution of a rectangular concrete section with breadth "b", total thickness "h" and depth of steel reinforcement "d", strengthened with either concrete, steel or externally bonded or near-surface-mounted FRP. The section is subjected to flexural moment, which produces tension at the bottom soffit with linear strain distribution along the beam height. If the applied flexural moment at the time of strengthening is "M_s", the maximum compressive strain at the top soffit is "ε_{ci}" and the tensile strain at the level of the newly added reinforcement is "ε_{bi}". At flexural strength of the beam, "M_u", the total tensile strain at the bottom soffit is "ε_{ct}", while the compressive strain at the top soffit is "ε_{cu}" which is equal to 0.003. Curvature of the concrete section, "Φ", is calculated at service loads using Eq. (3.1).

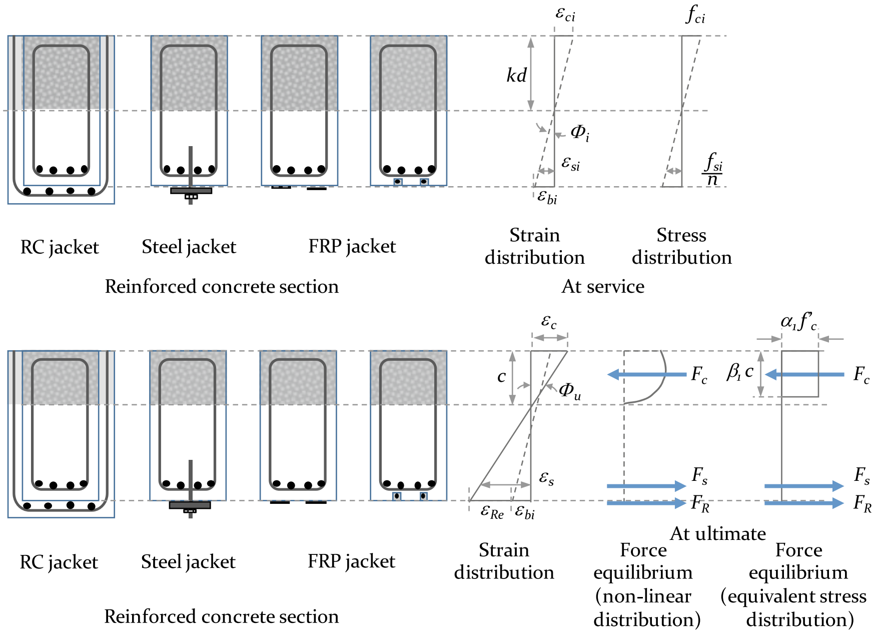


Fig. 3.11 Analysis of concrete section strengthened with concrete, steel or FRP jacket

$$\Phi_{\mathrm{i}} = \frac{\varepsilon_{\mathrm{ci}}}{kd} = \frac{\varepsilon_{\mathrm{bi}}}{h - kd} = \frac{M_{\mathrm{s}}}{E_{\mathrm{ci}}\, I_{\mathrm{cr}}} \tag{3.1}$$

where "kd" is the neutral axis depth at service loads, "M_s" is the flexural moment at time of strengthening, "E_{ci}" is the initial concrete modulus, and "I_{cr}" is the cracked moment of inertia.

The tensile strain of the added reinforcement for strengthening purpose, "ε_R" where "R" may refer to steel bars, steel plate or FRP, is calculated using Eq. (3.2).

$$\varepsilon_{\mathrm{R}} = \varepsilon_{\mathrm{cu}}\left(\frac{d_{\mathrm{R}} - c}{c}\right) - \varepsilon_{\mathrm{bi}} \tag{3.2}$$

where "d_R" is the depth of the newly added reinforcement and "c" is the neutral axis depth at ultimate moment, which can be calculated using iterative procedure of strain compatibility as per Eq. (3.2) and force equilibrium as per Eqs. (3.3) and (3.4). An initial value for "c" is first assumed, and the strains and stresses are calculated. A revised value for the depth of neutral axis is then calculated until the calculated and assumed values for "c" agree.

$$F_{\mathrm{c}} = F_{\mathrm{s}} + F_{\mathrm{R}} \tag{3.3}$$

$$\alpha_1 f_c^{'} \beta_1 cb = A_s f_s + A_R f_R \tag{3.4}$$

where "α_1" is equal 0.85, "β_1" is as per Fig. 3.12, "A_s" is the steel area in the original section, "A_R" is the area of the newly added reinforcement, "f_y" is the yield stress of the longitudinal steel in the original section, and "f_R" is the stress in the newly added reinforcement, which is calculated as per Fig. 3.13 and Eqs. (3.5)–(3.10).

$$\text{For steel bars and sections,}\quad f_R = E_s \varepsilon_R \le f_y \tag{3.5}$$

$$\text{For CFRP laminates and bars,}\quad f_R = E_f \varepsilon_R \le f_{fd} \tag{3.6}$$

where "E_s and E_f" are the steel and FRP elastic moduli, respectively. In order to consider both the environmental effect and bond between CFRP and concrete, the following equations are followed:

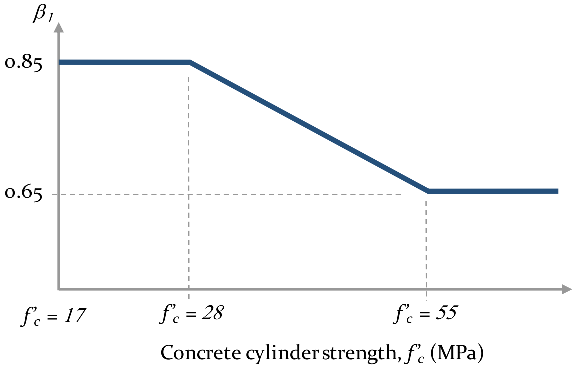


Fig. 3.12 Calculation of β_1

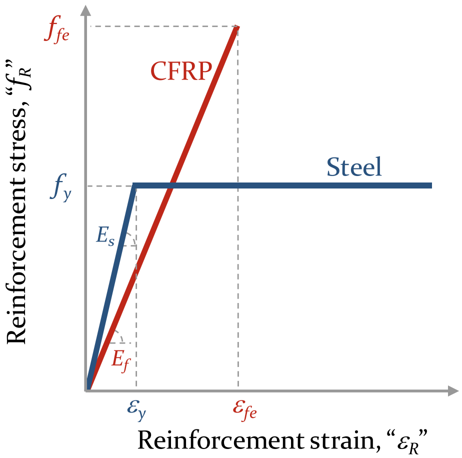


Fig. 3.13 Calculation of "f_R"

For externally bonded CFRP,

$$f_{fd} = \psi_f C_e f^*_{fu} \leq \psi_f \kappa_b \left(0.41 \sqrt{\frac{f'_c}{n E_f t_f}} \right) E_f \leq \psi_f (\varepsilon_R) E_f \tag{3.7}$$

For near-surface-mounted CFRP,

$$f_{fd} = \psi_f C_e f^*_{fu} \leq \psi_f (0.7 \varepsilon_{fu}) E_f \tag{3.8}$$

where

$$f^*_{fu} = \overline{f_{fu}} - 3\sigma \tag{3.9}$$

$$\varepsilon_{fu} = \frac{f^*_{fu}}{E_f} \tag{3.10}$$

And "f^*_{fu}" is the FRP ultimate tensile strength, "$\overline{f_{fu}}$" is the mean value of the FRP tensile strength, "σ" is the standard deviation of the strengths, "C_e" is the environmental reduction factor equal to 0.95 for interior exposure and 0.85 for exterior exposure, "ψ_f" is the FRP strength reduction factor equal 0.85, "κ_b" is a factor depending on the bond of the longitudinal FRP, equals 1.0 in case there is no anchorage and 1.3 if the FRP is anchored by transverse FRP, "n" is the number of externally bonded CFRP plies used on top of each other, and "t_f" is the thickness of single CFRP ply.

The above procedure is valid for the mode of failure when concrete crushing governs the design, i.e. ($\varepsilon_{cu} = 0.003$). If FRP rupture, cover delamination or FRP debonding occurs, the above procedure still gives reasonably accurate results (ACI 440.2R-17). After establishment of the forces and strain gradient in the cross-section, the resisting flexural capacity is calculated using Eq. (3.11).

$$M_r = \phi \left[A_s f_s \left(d - \frac{a}{2} \right) + A_R f_R \left(d_R - \frac{a}{2} \right) \right] \tag{3.11}$$

where "ϕ" is calculated according to Fig. 1.21 to account for the ductility of the flexural member.

Example 3.1 A simple beam has a rectangular cross-section with dimensions of 300 mm width (b), 600 mm total height (h), and clear span of 7.0 m. The beam is longitudinally reinforced with (4) steel bars of 20 mm diameter. Due to architectural changes, the beam shall be carrying additional loads so that the induced bending moments are as follows:

Moments due to dead loads (MDL), superimposed dead loads (MSID) and live load (MLL) = 100, 60 and 100 kNm, respectively.

If the concrete cylinder strength is 30 MPa and the yield stress of the steel is 420 MPa, check if the beam is safe and design the strengthening concrete jacket.

Answer

$A_s = 1257\ \text{mm}^2$, $E_{ci} = 4700\sqrt{f_c{}'} = 4700\sqrt{(30)} = 25{,}742$ MPa.

n (modular ratio) $= E_s/E_{ci} = 200{,}000/25{,}742 = 7.769$.

Using Fig. 3.12, $\beta_1 = 0.8357$.

To calculate the flexural capacity of the original concrete section (M_r).

Compression in the concrete ($0.85 f_c{}' ab$) = Tension in the steel ($A_s f_y$), therefore:

$a = 68.99$ mm, c (neutral axis depth at ultimate) $= a/\beta_1 = 82.55$ mm.

Resisting moment, $M_r = \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right] = 0.9\left[1257 \times 420\left(550 - \frac{68.99}{2}\right)\right] =$ 244.9 kNm.

Factored moment, $M_f = 1.2$ MDL + 1.2 MSID + 1.6 MLL = 352 kNm > M_r = 244.9 (unsafe—therefore, strengthening required).

Calculation of the concrete jacket:

Reference is made to Fig. 3.11, and in order to calculate the neutral axis depth at service load, "kd", the statical moment of area about the neutral axis is as follows: $\frac{b \times (kd)^2}{2} = nA_s(d - kd)$, therefore $kd = 159.4$ mm.

The cracked moment of inertia (I_{cr}) = 1,894,527,198 mm^4.

Using Eq. (3.1) to calculate the strain at the bottom soffit of the original concrete section at time of strengthening where the superimposed dead load and live load were not applied on the concrete beam,

$$\varepsilon_{bi} = \frac{M_s(h - kd)}{E_{ci} I_{cr}} = \frac{100 \times 10^6 \times (600 - 159.4)}{25{,}742 \times 1{,}894{,}527{,}198} = 0.000903$$

Assume the new cross-section is 400 × 700 mm, as shown in Fig. 3.14 with 4 bars 16 mm diameter at the tension side with concrete compressive strength = 30 MPa and steel yield stress of 420 MPa.

$A_s = 804\ \text{mm}^2$.

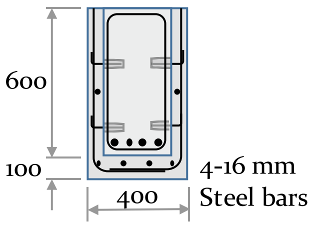
Compression in the concrete ($0.85 \times 30 \times a \times 400$) = Tension in the steel in the original section (1257×420) + Tension in the newly added steel (804×420).

Therefore: $a = 84.86$ mm, $c = 101.54$ mm.

Using Eq. (3.2), $\varepsilon_R = 0.003\left(\frac{650-101.54}{101.54}\right) - 0.000903 = 0.0153 > \varepsilon_y = 0.0021$.

Therefore, $f_R = 420$ MPa, and applying Eq. (3.11),

Fig. 3.14 Cross-section



$$M_\mathrm{r} = 0.9\left[1257 \times 420\left(550 - \frac{84.86}{2}\right) + 804 \times 420\left(650 - \frac{84.86}{2}\right)\right] = 425.8\,\mathrm{kNm}$$

$M_\mathrm{r} = 425.8 > M_\mathrm{f} = 352$ kNm (safe).

Example 3.2 For the beam given in Example 3.1, it is required to increase the flexural strength due to application of the following loads using CFRP strips:

Moments due to dead load (MDL), superimposed dead load (MSID) and live load (MLL) = 100, 40 and 80 kNm, respectively.

The properties of the CFRP strips are as follows: dimensions: 100 × 1.0 mm, CFRP ultimate tensile strength, "f_fu^*" = 2500 MPa and elastic modulus = 160 GPa.

Answer

Factored moment, M_f = 1.2 MDL + 1.2 MSID + 1.6 MLL = 296 kNm > M_r = 244.9 (unsafe—therefore, strengthening required).

Calculation of the longitudinal CFRP strips required for flexural strengthening:

Using Eq. (3.7),

$$f_\mathrm{fd} = \text{Min. of}\left(\begin{array}{c} (0.85)(0.95)(2500) \\ (0.85)(1.3)\left(0.41\sqrt{\frac{(30)}{(1)(160{,}000)(1.0)}}\right)(160{,}000) \\ (0.85)(\varepsilon_\mathrm{R})(160{,}000) \end{array}\right)$$

$$= \left(\begin{array}{c} 2018 \\ 992 \\ \text{to be checked} \end{array}\right)\text{MPa}$$

Using f_fd = 992 MPa and κ_b = 1.3 (transverse anchorage shall be used for the longitudinal CFRP, as shown in Fig. 3.15), the preliminary estimation of the area of CFRP strips is as follows:

$$M_\mathrm{f} \approx \phi(F_\mathrm{s} + F_\mathrm{R})(0.8\text{ to }0.85d)$$

$$296 \times 10^6 \approx 0.9(1257 \times 420 + A_\mathrm{f} \times 992)(0.8\text{ to }0.85) \times 550$$

Hence, A_f = 177–221 mm^2.

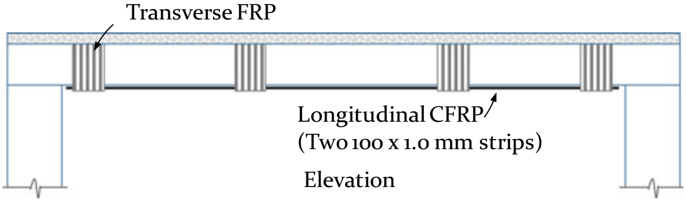


Fig. 3.15 Beam with CFRP strengthening

Try two strips 100 × 1.0 mm with area of 200 mm^2.

Applying force equilibrium, $C = F_s + F_R$,

0.85 (30) $a \times 300 = 1257 \times 420 + 200 \times 992$. Therefore, $a = 81.8$ mm and $c = a/\beta_1 = 98.1$ mm.

Applying Eq. (3.1),

$$\varepsilon_R = 0.003\left(\frac{600 - c}{c}\right) - \varepsilon_{bi} = 0.003\left(\frac{600 - 98.1}{98.1}\right) - 0.000903 = 0.0144$$

Therefore $f_{fd} = \text{Min. of}\begin{pmatrix} 2018 \\ 992 \\ 0.85 \times 0.0144 \times 160{,}000 \end{pmatrix} = \begin{pmatrix} 2018 \\ 992 \\ 1964 \end{pmatrix}\text{MPa} = 992\,\text{MPa}$.

Applying Eq. (3.11), $M_r = 0.9\left[1257 \times 420\left(550 - \frac{81.8}{2}\right) + 200 \times 992\left(600 - \frac{81.8}{2}\right)\right]$

$M_r = 341.7$ kNm $> M_f = 296.0$ kNm (safe).

3.5 Calculation of Shear Connectors

Full composite behavior of concrete beams or slabs strengthened with top concrete overlay shall not be guaranteed unless no slip occurs of the shear dowels connecting the original section with the overlay. The interfacial shear transfer between the top slab and the supporting beams (horizontal shear), shall affect the load carrying capacity and behavior of the beam if not considered in the design. Interfacial shear capacity is directly influenced by the distribution and the percentage of shear connectors. Research and design guidelines suggest two different approaches to calculate the required shear reinforcement at the interface, namely based on the shear force along the beam length or the actual longitudinal force in the newly cast element resulting from bending moment in the member.

In the first method, distribution of the shear dowels is calculated based on the shear force along the beam, where the dowels are intensified at the maximum shear zone and reduced at the mid-span, as shown in Fig. 3.16. The dowels are calculated using shear friction theory and based on the shear flow at the interface and the shear strength of the dowels, using Eqs. (3.12)–(3.16).

$$\phi V_{nh} \geq V_u \tag{3.12}$$

In case that $V_u < \phi\,(3.5 b_v d)$:

1. When concrete is cast against hardened concrete intentionally roughened to an amplitude of approximately 6 mm:

$$V_{nh} = \left(1.8 + 0.6\frac{A_v f_y}{b_v s}\right) b_v d \leq 3.5 b_v d \tag{3.13}$$

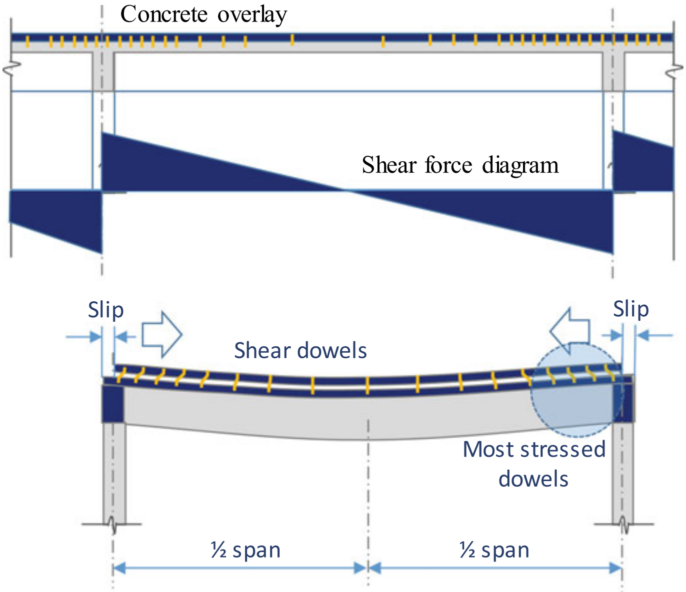


Fig. 3.16 Shear dowels calculated based on the shear force

2. When concrete is cast against hardened concrete not intentionally roughened:

$$V_{nh} = 0.55 b_v d \tag{3.14}$$

$$\text{where } A_{v(\min.)} \geq \text{The greater of} \begin{pmatrix} 0.062\sqrt{f'_c}\,\frac{b_w s}{f_y} \\ 0.35\frac{b_w s}{f_y} \end{pmatrix} \tag{3.15}$$

In case that $V_u > \phi\,(3.5 b_v d)$:

$$A_v = \frac{V_u}{\phi f_y \mu} \tag{3.16}$$

where V_{nh} is the horizontal shear strength, ϕ is taken as per Table 1.5, b_v is the width of the contact surface, A_v is the steel area resisting shear friction crossing the assumed shear plane, f_y is the yield stress of the steel shear reinforcement, and μ is the coefficient of friction equals: 1.0 for concrete cast against hardened concrete that is clean, free of laitance and intentionally roughened to a full amplitude of approximately 6.0 mm; 0.6 for concrete cast against hardened concrete that is clean, free of laitance and not intentionally roughened; and 0.7 for concrete placed against as-rolled structural steel that is clean, free of paint and with shear transferred across the contact surface by headed studs or by welded deformed bars or wires.

Alternatively, shear dowels may be calculated based on the change in flexural compressive or tensile force in the concrete member. Accordingly, the number of shear dowels is calculated based on the actual compression or tension force in the newly cast element and the shear capacity of the steel dowels placed to resist this force, as shown in Eq. (3.17). The steel dowels should be able to transfer that force as longitudinal shear to the supporting member, as shown in Fig. 3.17. In any case, the factored longitudinal shear force shall not exceed the longitudinal shear resistance. Spacing of the shear dowels along the contact surface should provide horizontal shear resistance distributed approximately the same as the distribution of shear stress since redistribution of the horizontal shear resistance is limited.

$$A_{\mathrm{v}} = \frac{C \text{ or } T}{\phi f_{\mathrm{y}} \mu} \tag{3.17}$$

where C or T is the actual compression or tension force in the newly added element.

It was proved by testing (Awry et al. 2013) that beams with non-uniform distribution of shear dowels shall result in the same capacity compared to that with uniform dowel distribution; yet, ductility of the beam with intensified distribution of dowels at the shear zone is higher, as shown in Fig. 3.18. The figure shows the capacity of two composite beam specimens with 6 mm diameter shear dowels connecting

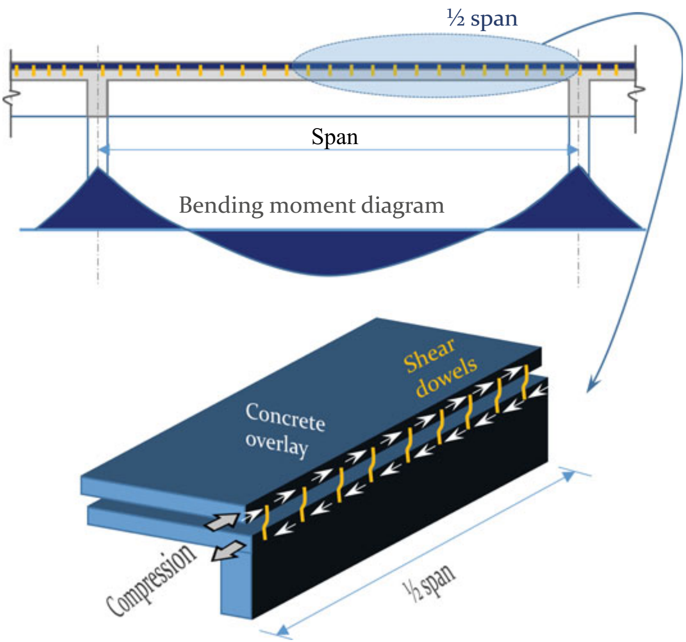


Fig. 3.17 Shear dowels calculated based on the change in flexural compressive or tensile force

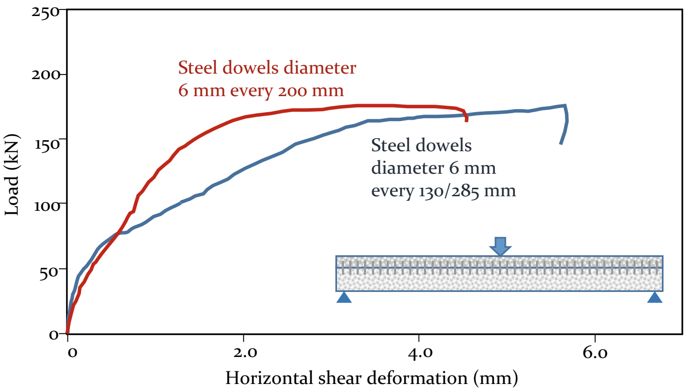


Fig. 3.18 Behavior of composite beams with variable distribution of shear dowels

the rectangular cross-section to the concrete slab on top. The two specimens have the same number of dowels; yet, the first one has uniform dowel distribution every 200 mm and the second one has varying distribution of 130 mm spacing at the shear zone and 285 mm at the mid-span. It is clear that the ductility of the beam with varying spacing at the shear zone is higher. Moreover, it should be noted that in strength computations of composite members, no distinction shall be made between members strengthened with passive or active strengthening; i.e., calculation of the required steel area of shear dowels shall not differ in case that the beam was carrying load at the time of strengthening.

Example 3.3 For the beam in Example 3.1, design the shear dowels connecting the original section to the new concrete jacket, knowing that the shear capacity of the 10 mm steel dowel in shear is 28 kN.

Answer

Reference is made to Fig. 3.14, and using the lower two anchor bolts to transfer the tension in the longitudinal steel reinforcement in the jacket, the required area of steel bolts in half the beam length is calculated according to the following:

$$\text{Number of steel shear dowels} = \frac{A_s f_y}{\phi V_a} = \frac{4 \times 201 \times 420}{0.75 \times 28{,}000} = 16.08$$

Use (9 rows $\times$ 2 = 18 steel dowels of 10 mm diameter) in half the beam length, therefore spacing between each row of dowels is 3500/9 = 380 mm, as shown in Fig. 3.19.

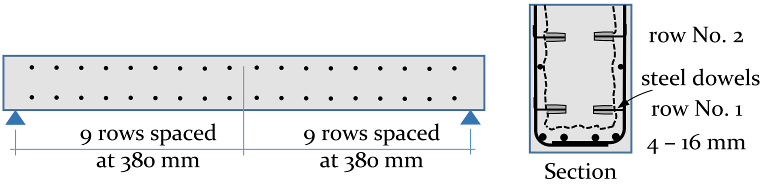


Fig. 3.19 Steel dowel distribution

3.6 Shear Strengthening of Beams

The shear capacity of concrete section strengthened with concrete, steel or FRP jacket shall be calculated so that the nominal shear strength multiplied by the strength reduction factor, as per Table 1.5, should be higher than the applied factored shear force, as per Eq. (3.18). The shear capacity of the strengthened section is calculated according to Eq. (3.19).

$$\phi V_n \geq V_u \tag{3.18}$$

$$\phi V_n = \phi(V_c + V_s + V_R) \tag{3.19}$$

where V_c, V_s and V_R, are the concrete, steel and strengthening reinforcement capacities, respectively, calculated as per the following equations.

$$V_c = \left(0.66\lambda_s(\rho_w)^{\frac{1}{3}}\sqrt{f'_c} + \frac{N_u}{6A_g}\right)b_w d \leq 0.42\sqrt{f'_c}b_w d \tag{3.20}$$

where $\lambda_s = \sqrt{\frac{2}{1+0.004d}} \leq 1.0$ and $= 1.0$ if $\frac{A_v}{s} \geq \left(0.062\sqrt{f'_c}\frac{b_w}{f_{yt}}\right)$ or $\left(0.35\frac{b_w}{f_{yt}}\right)$, ρ_w is the ratio of the longitudinal reinforcement of the beam ($A_s/b_w d$), and N_u is the axial force on the section, taken (+) for compression and (−) for tension.

$$V_s = \frac{A_v f_{yt} d}{s} \tag{3.21}$$

$$\text{For steel stirrups shear strengthening,}\quad V_R = \frac{A_{vR} f_{yt} d}{s} \tag{3.22}$$

$$\text{For FRP shear strengthening,}\quad V_R = \psi_f \frac{A_{fv} f_{fe} d_f}{s_f} \tag{3.23}$$

where $\psi_f = 0.85$ for two-sided and U-wrapped FRP laminates and 0.95 for fully wrapped FRP laminates, $A_{fv} = 2nt_f w_f, f_{fe} = E_f\,(0.004)$, and d_f, s_f and w_f are defined in Fig. 3.20.

The ultimate shear force (V_u), should not exceed the value given in Eq. (3.24).

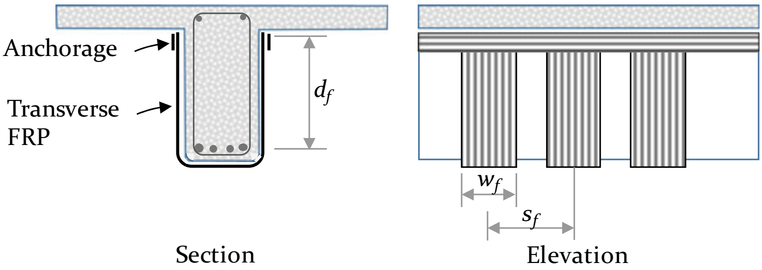


Fig. 3.20 Variables used in FRP shear strengthening

$$V_u \leq \phi\left(V_c + 0.66\sqrt{f_c'}b_w d\right) \tag{3.24}$$

Example 3.4 The beam given in Example 3.1 is subjected to shear forces due to dead, superimposed dead and live loads (VDL, VSID and VLL), of 90, 55 and 90 kN, respectively. It is required to calculate the following:

1. Shear capacity of the original section knowing that the existing steel stirrups in the original section are two branched 8 mm diameter bars every 250 mm with 240 MPa yield stress.
2. Steel stirrups of the jacket using 10 mm diameter bars of 420 MPa yield stress.

Answer

For the original section: $\frac{A_v}{s} = \frac{2\times 50.3}{250} = 0.402 < 0.062\sqrt{f_c'}\frac{b_w}{f_{yt}} = 0.062\sqrt{30}\frac{300}{240} = 0.4245$

$$\lambda_s = \sqrt{\frac{2}{1 + 0.004 \times 550}} = 0.79,\ \rho_w = \frac{1257}{300 \times 550} = 0.0076$$

$$V_c = 0.66 \times 0.79(0.0076)^{\frac{1}{3}}\sqrt{30} \times 300 \times 550 \times 10^{-3} = 92.9\,\text{kN}$$

$$\text{check } V_c < 0.42\sqrt{30}\,300 \times 550 \times 10^{-3} = 379.6\,\text{kN}$$

$$V_s = \frac{2 \times 50.3 \times 240 \times 550}{250 \times 1000} = 53.1\,\text{kN}$$

$$V_u = 1.2\,\text{VDL} + 1.2\,\text{VSID} + 1.6\,\text{VLL} = 318\,\text{kN}$$

Shear strength $= \phi(V_c + V_s) = 0.75\ (92.9 + 53.1) = 109.5$ kN.
Therefore, $V_u > \phi\ (V_c + V_s)$. Strengthening is needed.

For the jacket given in Example 3.1, the new dimensions of the beam are 400 width and 700 depth with four longitudinal bars of 16 mm diameter.

$$\lambda_s = 1.0,\ \rho_w = \frac{1257 + 804}{400 \times 650} = 0.0079$$

$$V_c = 0.66 \times 1.0(0.0079)^{\frac{1}{3}}\sqrt{30} \times 400 \times 650 \times 10^{-3} = 187.7\,\text{kN}$$

$$V_R\ (\text{Jacket}) = \frac{V_u}{\phi} - V_c - V_s = \frac{318}{0.75} - 187.7 - 53.1 = 183.2\,\text{kN}$$

V_R (stirrups of the jacket) $= 183{,}200 = \frac{2\times 78.5\times 420\times 650}{s}$, therefore spacing ($s$) $=$ 233 mm.

It is proposed to use two branches steel stirrups of 10 mm diameter spaced every 220 mm, therfore, $V_R = 194.9$ kN.

V_u (318) $< \phi$ ($V_c + V_s + V_R$) $= 0.75$ (187.7 + 53.1 + 194.9) $= 326.8$ kN (safe).

Check for the maximum shear force:

$$V_u(318\,\text{kN}) \le \phi\left(V_c + 0.66\sqrt{f_c'}b_w d\right) = (845\,\text{kN})\ (\text{OK}).$$

3.7 Bond and Development Length

Bond failure of tensile reinforcement is brittle and catastrophic and should be avoided by providing adequate development length. For simply supported beams, at least one third of the total required steel reinforcement should be extended into the support, while only one fourth of the longitudinal steel reinforcement is adequate. This is unless additional reinforcement is required to be extended in the support as tension or compression reinforcement resisting the induced flexural moments. Development of the FRP reinforcement is required with a minimum length of "l_{df}", calculated as per Eq. (3.25). In addition, FRP reinforcement should be extended after the point of zero moment in the flexural member with a minimum length of 150 mm.

$$l_{df} = \sqrt{\frac{nE_f t_f}{\sqrt{f_c'}}} \tag{3.25}$$

3.8 Serviceability of Strengthened Beams

Serviceability limit states of the strengthened members should be satisfied by limiting the deflection and crack width of concrete members to the allowable values. In addition to the aesthetics of the structure, deflection is controlled to protect the non-structural elements from damage and prevent ponding of the roof floors. Large crack width in concrete elements will increase the potential of steel corrosion, and hence, the concrete durability shall be risked. Serviceability limit states should be also checked by limiting the floor vibration to satisfy the comfort level of the users, while fatigue of the reinforcing steel should be endured to ensure the design lifetime of the structure.

The steel and concrete stresses should be also limited under the effect of service loads in order to avoid inelastic deformation in the member. Equations (3.26) and (3.27) show the maximum allowable service stresses in the concrete and internal steel of the original section under the effect of full service loads. For passive strengthening and in order to calculate the service stresses, superposition shall be made between the calculated stresses of the original section under the effect of the existing loads at the time of strengthening and the stresses of the strengthened section under the effect of additional loads. In case of jacking the beam to release it from the acting loads or in case of prestressing the beam, the stresses of the new section shall be calculated using the full loads including self-weight of the beam.

$$f_{c,s} \le 0.6 f_c\prime \tag{3.26}$$

$$f_{s,s} \le 0.8 f_y \tag{3.27}$$

Example 3.5 For the same beam in Example 3.1, it is required to calculate the stress in the steel of the original section and the stress in the newly added steel.

Answer

Following the sequence of loading, the stress in the steel of the original section shall be calculated on two steps; in the first step, the stress shall be calculated using the properties of the original section and the bending moment due to dead load only. In the second step, the steel stress shall be calculated using the strengthened section properties and the bending moment due to additional loads, which shall be induced on the beam after strengthening. The total steel stress shall be the summation of the steel stress resulting from the two steps, as shown in Fig. 3.21.

Steel strain due to dead load only,

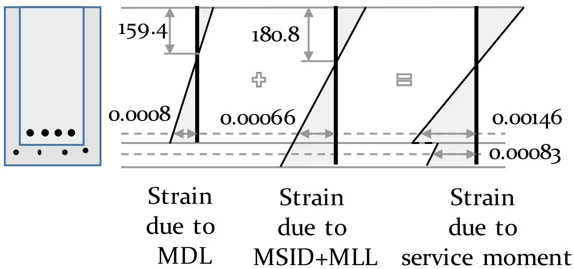


Fig. 3.21 Strain profile of strengthened beam

$$\varepsilon_{s(DL)} = \frac{M_{DL}(d - kd)}{E_{ci} I_{cr}} = \frac{100 \times 10^6 \times (550 - 159.4)}{25{,}742 \times 1{,}894{,}527{,}198} = 0.0008$$

Steel strain due to superimposed dead and live loads (properties of the strengthened section are: $kd = 180.8$ mm and $I_{cr} = 3{,}494{,}342{,}027$ mm^4),

$$\varepsilon_{s(SID+LL)} = \frac{(M_{SID} + M_{LL})(d - kd)}{E_{ci} I_{cr}} = \frac{160 \times 10^6 \times (550 - 180.8)}{25{,}742 \times 3{,}494{,}342{,}027} = 0.00066$$

$$\varepsilon_{s(Total)} = \varepsilon_{s(DL)} + \varepsilon_{s(SID+LL)} = 0.0008 + 0.00066 = 0.00146$$

$$f_s = 0.00146 \times 200{,}000 = 292\,\text{MPa}.$$

Steel stress of the jacket is calculated using the moments due to superimposed dead and live loads only,

$$\varepsilon_R = \frac{(100 + 60) \times 10^6 \times (650 - 180.8)}{25{,}742 \times 3{,}494{,}342{,}027} = 0.0008346$$

$$f_R = 0.0008346 \times 200{,}000 = 167\,\text{MPa}.$$

Example 3.6 For the same beam in Example 3.5, it is required to calculate the stress in the steel of the original section and the stress in the newly added steel in case that the beam was jacked to release the stresses in the beam before application of the strengthening jacket.

Answer

The new section of dimensions 400 × 700 and two levels of reinforcements shall be resisting the full loads including the dead, superimposed dead load and live loads

$$\varepsilon_{s(DL+SID+LL)} = \frac{(M_{DL} + M_{SID} + M_{LL})(d - kd)}{E_{ci} I_{cr}} = \frac{260 \times 10^6 \times (550 - 180.8)}{25{,}742 \times 3{,}494{,}342{,}027} = 0.00107$$

$$f_s = 0.00107 \times 2{,}000{,}000 = 213\,\text{MPa}$$

$$\varepsilon_{R(DL+SID+LL)} = \frac{260 \times 10^6 \times (650 - 180.8)}{25{,}742 \times 3{,}494{,}342{,}027} = 0.00136$$

$$f_s = 0.00136 \times 2{,}000{,}000 = 271\,\text{MPa}.$$

Table 3.1 shows the stress in the steel reinforcement of the original section and the strengthening jacket in case of passive and active strengthening. It is clear that

Table 3.1 Steel stress in the strengthened beam

Type of strengthening	Steel stress of original section (MPa)	Steel stress of jacket (MPa)
Passive (Example 3.5)	292	167
Active (Example 3.6)	213	271

the steel stress of the original section is higher in case of using passive strengthening (292 vs. 213 MPa); yet, it did not exceed the service limit of 80% of the steel yield stress, as per Eq. (3.27). In case that the steel stress exceeds the service limit, jacking the beam to release the loads from the beam before strengthening is recommended to reduce the stress in the steel of the original section under full service loads.

Meantime, the steel stress of the jacket is higher in case of using "active strengthening" since it participates in resisting more loads compared to "passive strengthening". This stress may be reduced by increasing the area of the used steel reinforcement in the jacket.

3.9 Corbel Strengthening

Concrete corbels are damaged either due to lack of steel reinforcement, which leads to tensile cracks in the corbel or due to bad detailing, where the bearing pads are placed close to the concrete edge. In the first case, where tensile cracks exist in the concrete corbel, conventional strengthening either by concrete jacket, steel plates or FRP laminates is applicable. The added tensile reinforcement shall be oriented in the same direction of main reinforcement and distributed along the height of the corbel. The success key of strengthening is to provide enough development length since the length of the corbel is usually less than the development length of tensile reinforcement. Mechanical anchorage may be used at the reinforcement end to develop the tension force and transmit it to the concrete. Figure 3.22a shows typical strengthening of concrete corbel using FRP laminates, where FRP anchorage is used at the end of the laminates. The anchorage is made of unidirectional carbon fibers placed in a predrilled hole in the concrete filled with epoxy resin. The exposed part of the carbon fibers is opened in a fan shape and bonded to the externally bonded laminates.

Concrete corbels may be also damaged if the bearings are placed close to the concrete edge while there is no special reinforcement provided to prevent the concrete cover from spalling as a result of the splitting force induced from high stresses beneath the bearing. Lifting the concrete beam and replacing the bearing may not be feasible since it requires to stop the use of the structure until completion of the repair works. A simple technique by fastening a steel pin to the concrete beam and the concrete corbel, as shown in Fig. 3.22b, may be adopted to transmit the vertical reaction to the

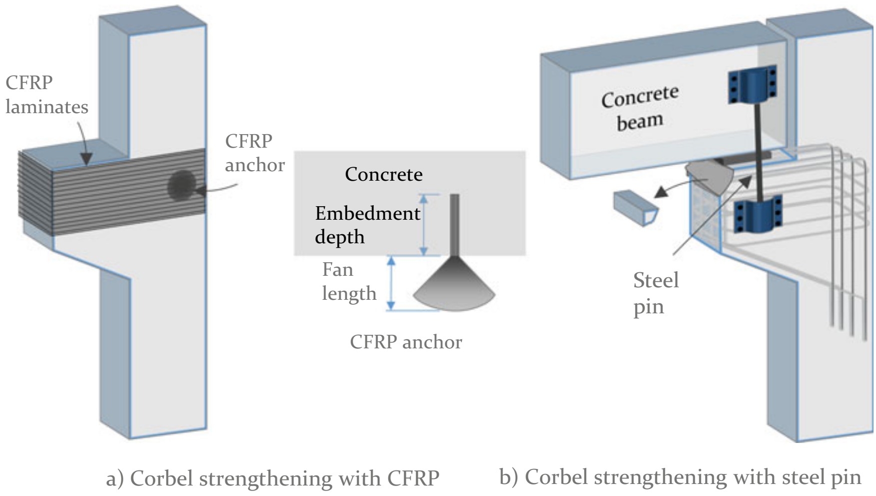


Fig. 3.22 Strengthening of RC corbels

undamaged concrete part of the corbel. The pin should be allowed to rotate freely to avoid any restraint and reinstate the initial condition at the time of construction.

In case that longitudinal beams exist at the edges of the corbel and the column, as shown in Fig. 3.23, external prestressing bar may be used to induce compression force at the tension zone in the corbel. Losses in the short prestressing bar are high and increase by time, which can be overcome by checking the nut and tightening it regularly with torque wrench. Protection of the steel bar against corrosion is a must since the rate of corrosion increases with the increase of the axial tension in the bar. This may be addressed by providing anti-corrosion paint, grease and plastic tube on the bar. The prestressing bars may be also galvanized or made of stainless steel.

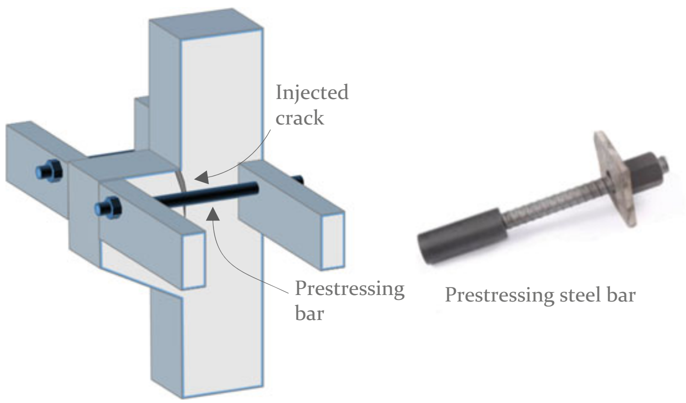


Fig. 3.23 Strengthening corbels with external prestressing

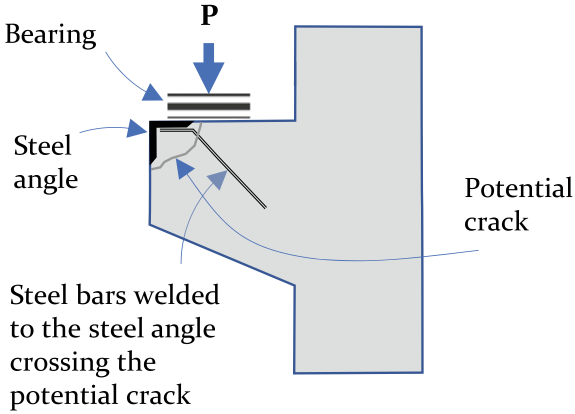


Fig. 3.24 Detailing of newly cast concrete corbels

It should be highlighted that in new construction, steel angle may be added at the corner of the corbel across its full width to protect the concrete cover of being chipped. In that case, steel bars should be welded to the steel angle and developed in the concrete in order to prevent the potential crack that may result in failure at the corbel edge, as shown in Fig. 3.24.

3.10 Case Study 3.1

This case study addresses a concrete bridge, which consists of 13 spans (29 m exterior spans and 35 m interior spans) supported on 12 frame piers. Each pier consists of arch girder with minimum cross-section of 1.20 × 1.30 m supported on columns with the same dimensions and two overhanging cantilevers. The frame supports box girder bridge deck resting on five elastomeric bearings. Cracking of the bridge piers is shown in Fig. 3.25, where a large number of cracks were observed in the top of the cantilevers and covering the zones of negative bending moments in the frame pier. The cracks were extended through the entire cross-section as shown in the figure, in addition to excessive deflection at the end of the cantilever.

Analysis of the bridge showed that the concrete frame was under-designed in flexure and shear and the amount of steel reinforcement in the frame does not satisfy the code requirements. Application of concrete jacket or external steel or FRP reinforcement at the top of the frame for flexure strengthening was not feasible because of the limited space between the frame and the bridge deck. Accordingly, the retrofitting scheme of the bridge was based on the concept of reducing the flexural moment on the cantilever to avoid flexural strengthening of the cantilever. The construction scheme included the following:

1. Existing bridge bearings were replaced with new ones, while adding two more bearings on top of the pier so that the total number of bearings supported by

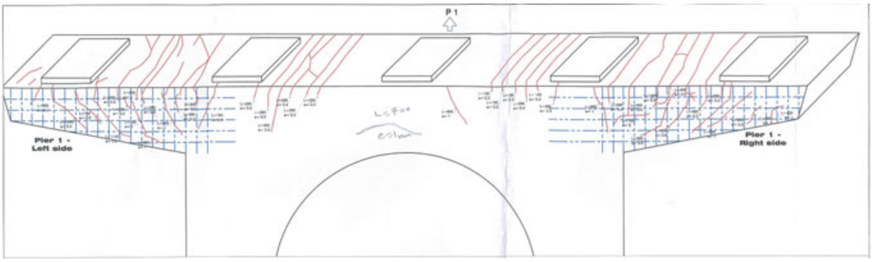


Fig. 3.25 Bridge pier supporting five elastomeric bearings

each pier became seven, as shown in Fig. 3.26. The two additional bearings were placed at the vicinity of the columns to relief some of the vertical reactions on the bearings mounted at the edges of the frame; consequently, the negative bending moment of the cantilever was reduced. The existing flexural reinforcement in the cantilever was adequate after the considerable reduction in the bending moment of the cantilever frame. Flat hydraulic jacks were used to lift the deck slab while resting on top of the concrete frame. The jacks were synchronized so that equal stroke of the jacks and even lifting of the deck slab is granted.

2. All the observed cracks in the piers were injected with low-viscous epoxy to retain the structural integrity of the member.
3. Externally bonded carbon FRP (CFRP), laminates were used to strengthen the cantilevers, where a number of U-Wraps were attached to the sides of the frame to enhance the shear strength of the cantilevers. It is worth noting that the shear strength of the cantilevers was still below the shear demand even after the reduction of the applied shear force on the cantilevers. The laminates were heat treated during curing in order to increase the glass transition temperature of the epoxy since the laminates were exposed to the environment. CFRP wraps were covered with cementitious material and painted to protect it from vandalism.

Fig. 3.26 Bridge pier after strengthening

Chapter 4
Slab Strengthening

Concrete slabs suffering from lack of structural capacity or material deterioration may be rehabilitated by any of the means used for beams strengthening, including concrete overlay, adding steel sections or FRP reinforcement. Combination of two or more of those strengthening techniques may be also used, where one technique is used at the top and another technique is used to strengthen the slab bottom soffit, depending on architectural requirements, functionality and ease of construction.

Slab strengthening may also include reduction of the straining actions by adding new structural members, whether concrete or steel beams or columns. Slab strengthening may be done for the enhancement of the flexural moment or punching shear resistance. Slabs may also be strengthened to enhance its serviceability limit states by increasing its stiffness to control the crack width, reduce its deflection or improve its vibrational behavior. Moreover, strengthening may be introduced for the improvement of the slab fire resistance.

4.1 Concrete Overlay for Slab Strengthening

Concrete overlays of slabs are constructed at either the top, bottom or both surfaces of the slab depending on the sections required to be strengthened. Despite that the concrete overlay at the top of the slab is easier to construct, the overlay at the bottom of the slab could be also done while concrete is cast using form-and-pump technique to ensure that no honeycomb or voids exist in the overlay. It might be more economic to use concrete overlay at the top, while using either CFRP reinforcement or steel plates to strengthen the positive moment section of the slab.

After propping the slab to carry its weight and the weight of the overlay, steel dowels are planted to transfer the interfacial shear forces between the old and new concrete. The total area of steel shear dowels planted in one quarter of the slab panel ($1/2\ l_x \times 1/2\ l_y$), as per Fig. 4.1, can be calculated based on the bending moments in the x and y directions integrated within half the slab length/width ($\overline{M_x}$, $\overline{M_y}$) as per

A. Abdelrahman, *Strengthening of Concrete Structures*,
https://doi.org/10.1007/978-981-19-8076-3_4

Eqs. (4.1) and (4.2). Both the positive and negative moments should be integrated and used for calculation of the shear dowels.

$$\overline{M_x} = \int_0^{\frac{1}{2}l_y} M_x \, \mathrm{d}l \tag{4.1}$$

$$\overline{M_y} = \int_0^{\frac{1}{2}l_x} M_y \, \mathrm{d}l \tag{4.2}$$

The induced forces on the shear dowels in the x and y directions ($\overline{F_x}$ and $\overline{F_y}$), can be calculated using Eqs. (4.3) and (4.4) for both the positive and negative bending moments independently. The total force on the shear dowels in one quarter of the slab panel ($\overline{F_s}$) is calculated using Eq. (4.5) after adding the forces in the both directions (x and y) resulting from the positive and the negative moments. Similar to Eq. (3.17), the area of steel shear dowels can be calculated using Eq. (4.6).

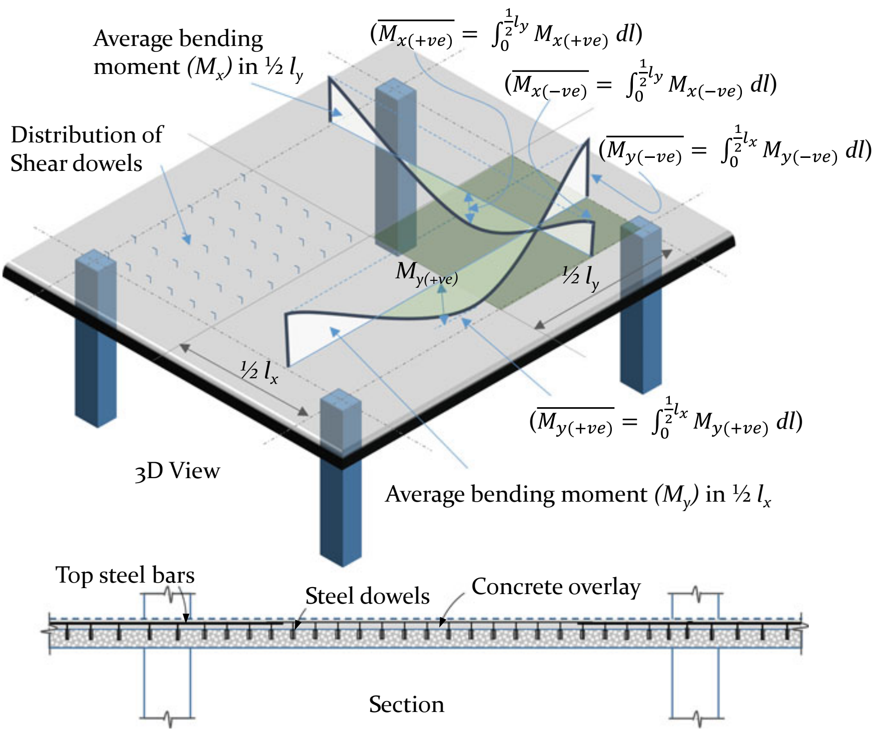


Fig. 4.1 Slab strengthening by concrete overlay

$$\overline{F_x} = \frac{\overline{M_x}}{d - \frac{a}{2}} \tag{4.3}$$

$$\overline{F_y} = \frac{\overline{M_y}}{d - \frac{a}{2}} \tag{4.4}$$

$$\overline{F_s} = \sqrt{\overline{F_x}^2 + \overline{F_y}^2} \tag{4.5}$$

$$A_v = \frac{F_s}{\phi f_y \mu} \tag{4.6}$$

4.2 External Reinforcement for Slab Strengthening

Concrete slabs may be strengthened with external steel plates, externally bonded FRP laminates or near-surface-mounted FRP reinforcement. Analysis and design of slabs strengthened with external reinforcement are similar to that of beams given in Chap. 3. The external reinforcement may be applied in one direction or two directions depending on the aspect ratio of the slab and the required strengthening of the slab, as shown in Fig. 4.2. Use of different strengthening schemes in the same slab may be applied so that steel plates are attached to the top of the slab and FRP strips are attached at the slab bottom soffit. This may be preferably applied since it is easier to install the steel strips and dowel insertion from the slab top compared to working overhead for the bottom surface. Calculation of the required area of steel dowels is carried out as per that given for slabs strengthened with concrete overlay.

In case that the external steel plates are applied in two directions, care should be given to fill the gap behind the steel plates with filling material such as grout or epoxy grout to protect the steel plates from corrosion, as shown in Fig. 4.2.

4.3 Adding Structural Member

Slabs can be also strengthened by adding structural members made of steel or concrete, while the new member may be column or beam. The structural system of the slab is changed by adding columns or beams aiming at dividing the slabs to smaller areas and increasing its stiffness. The straining actions and deformations of the slabs shall be reduced after adding the new member. Planting concrete beams to support existing slabs requires drilling in the slabs for the transverse steel stirrups and in the supporting elements for the longitudinal reinforcement, as shown in Fig. 4.3.

After assembling the steel cage, concrete with low shrinkage admixtures is cast to reduce the interfacial shear stresses between the old and new concrete induced

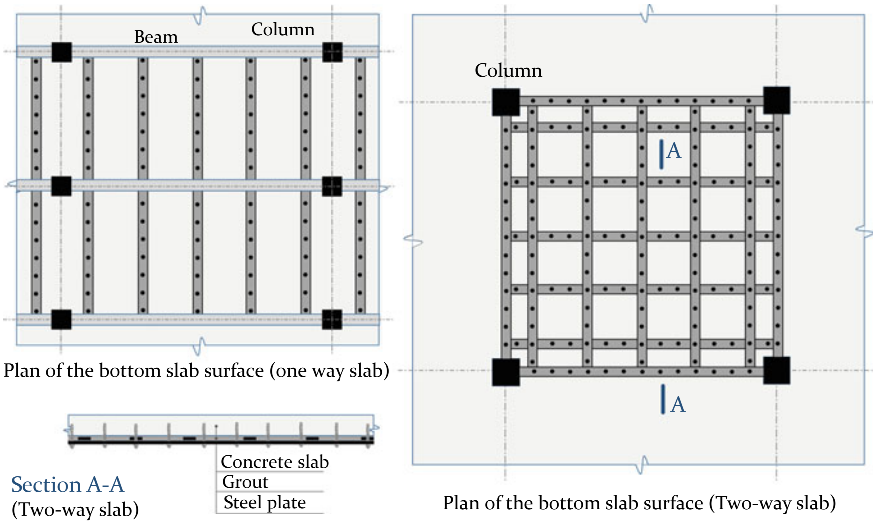


Fig. 4.2 Slab strengthening with steel plates

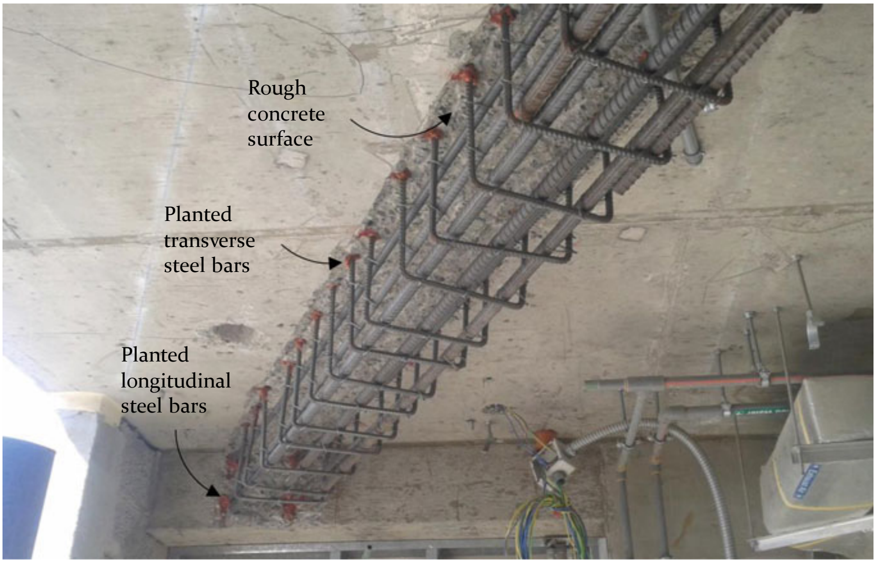


Fig. 4.3 Slab strengthening by adding concrete beam

by concrete shrinkage. Before concrete casting, the slab should be either propped so that the props support the concrete weight and the weights above or jacked up to release the load from the slab, as shown in Fig. 4.4. The figure shows detailing of the required steel reinforcement for the new beam. Releasing the loads from the

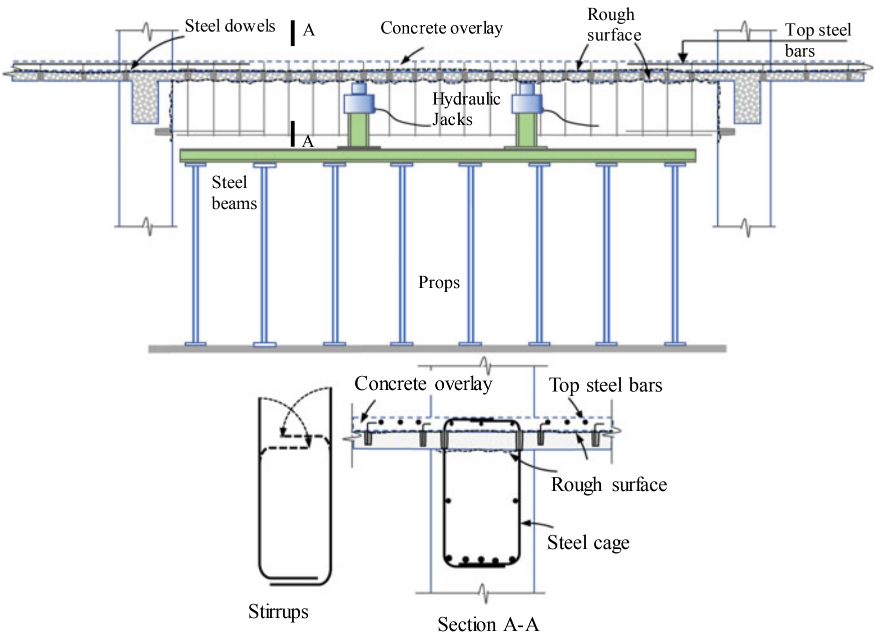


Fig. 4.4 Jacking the strengthened slab

slab at the time of casting the new supporting beam will reduce the overall stresses in the slab since the new beam will share in carrying the total weights with the slabs including the weight of the slab and the weights above.

4.3.1 Lifting Concrete Slabs Before Strengthening

Excessive deflection of concrete slabs may occur due to insufficient flexural steel reinforcement or due to construction errors such as early removal of formwork, movement of formwork during concrete casting or use of defective materials. In those cases, lifting the slab to reduce the in-situ deflection could be desirable to improve the aesthetics of the structure. Proper measures should be taken during lifting the slab in order to avoid inducing reversed straining actions on the slab, which the slab is not designed for.

Figure 4.5a shows typical load–deflection behavior of concrete slab subjected to repeated loading. It is clear that the inelastic deflection of the slab after removal of the applied load (Δ_r) is considerable compared to that of the elastic deflection of the slab (Δ_e). The elastic deflection can be recovered by lifting the slab, while the inelastic deflection cannot be recovered after load removal.

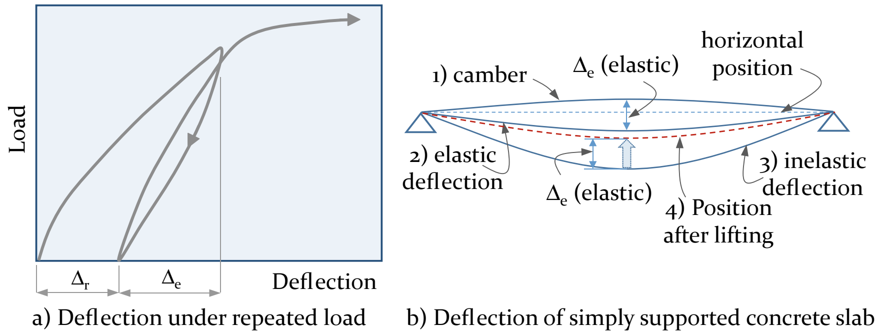


Fig. 4.5 Load–deflection of concrete slabs

The simply supported slab shown in Fig. 4.5b is cambered at position number (1). The applied loads on the slab result in instantaneous elastic deflection to position number (2) and further by time to position number (3) after occurrence of the inelastic deflection. The slab can only be lifted up to position number (4) by the amount of the elastic deflection leaving the slab deflected downward with irrecoverable deflection.

The inelastic slab deflection due to concrete creep may reach double the elastic deflection depending on the amount of tension and compression reinforcement, concrete strength and other environmental conditions. Lifting the slab to recover more than the elastic deflection (Δ_e), shall result in reversed straining action with values proportional to the values of the recovered deflection exceeding the elastic value. These straining actions may result in cracking of the slab or, eventually, local failure at one or more than one section. Accordingly, it should be ensured that jacking the slab is monitored so that it does not exceed the calculated elastic deflection resulting from the slab self-weight and weights carried by the slab at the time of jacking (if any).

Synchronized jacking, in case of using more than one jack, should be used to lift the slab so that the stroke of each jack shall be the same. This can be achieved using the same type of jacks connected to one hydraulic pump, as shown in Fig. 4.6. The jacks should have locking nuts in order to maintain the force induced on the slab during the whole process of slab strengthening.

The jacks should be placed at axis-symmetric points at the slab bottom soffit as close to the mid-span of the slab to maximize the flexural moment-to-shear force ratio (M/V ratio). The jacks' vertical movement shall be recorded during lifting in order to ensure having the same force on the slab at each lifting point. In addition to measurement of the slab movement, the induced forces on the slab should be also recorded using calibrated pressure gauge connected to the pump. The maximum uplift forces induced on the slab should not exceed the calculated values based on the bending moment resulting from the slab self-weight and weights above (if any), while the measured movement of the slab shall be only used to make certain that balanced loads are exerted on the slab. The slab position after lifting should never control the process since it is expected that the slab upward movement shall be less

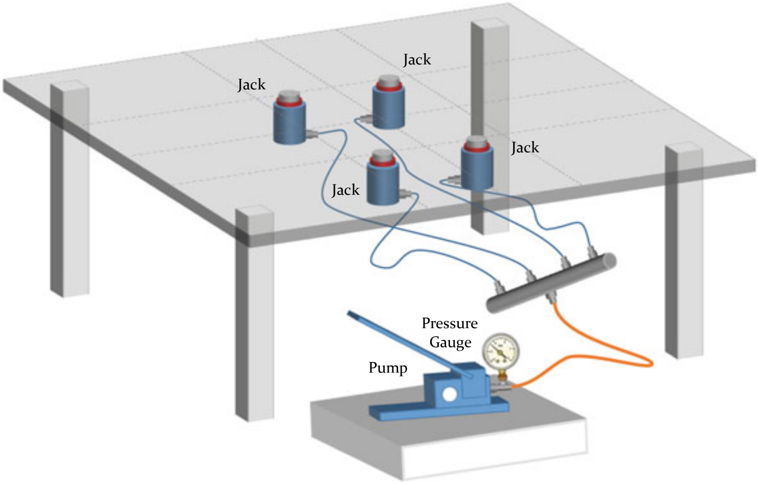


Fig. 4.6 Synchronized jacking used for slab lifting

than the existing slab downward deflection, and as a result, the overall slab deflection will be reduced but the slab will not be back to its original position.

4.4 Prestressing Flat Slabs

Post-tensioned (PT), concrete slabs are strengthened with externally prestressed steel tendons or carbon fiber reinforced polymer (CFRP) strips. When openings are needed to be made in PT slabs and internal prestressing steel strands are cut or when rehabilitation of PT slabs is required, addition of prestressing reinforcement is recommended either to compensate for the loss of the prestressing force or to improve the slab structural capacity and/or serviceability.

The prestressing reinforcement is externally attached to the bottom or top soffit of the concrete slab using steel plates and steel bolts. The prestressing steel shall be straight and unbonded with constant eccentricity outside the concrete section, as shown in Fig. 4.7. Steel strands should be well protected against corrosion with coating and sheathing. Maintenance of such system should be implemented through regular inspection to check the durability and integrity of the system through its lifetime.

In case of using prestressed CFRP strips, special anchorage system should be used to tension the strips. A comprehensive research was made to select the best system for anchoring the CFRP strips (Helmy et al. 2017). The proposed system composes of live- and dead-end anchorages as well as special fixation and additional anchor that can be reused for different strips, as shown in Fig. 4.8. After tensioning the strips, epoxy is used to bond the strips to the concrete surface and the live anchor

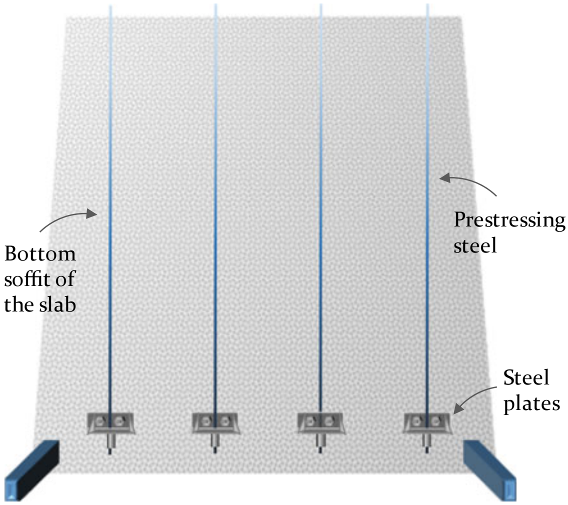


Fig. 4.7 External prestressing of slabs with steel

may be reused again. The system was tested on simple and two-span continuous slabs (El-Sefy 2016), and it was found out that the capacity of the continuous slabs increases by adding prestressed CFRP strips, and at the same time, the serviceability of the slabs improves.

Figure 4.9 shows the results of four slabs, a control slab without strengthening and three other CFRP-strengthened slabs at different locations and length of prestressed strips, as shown in the legend of the figure. Testing showed that there is no need to use individual prestressing strips at the bottom soffit for each span, where one continuous strip may be used efficiently to minimize the number of anchorages (see the curve in green). Moreover, the overall slab capacity is not much reduced when eliminating the

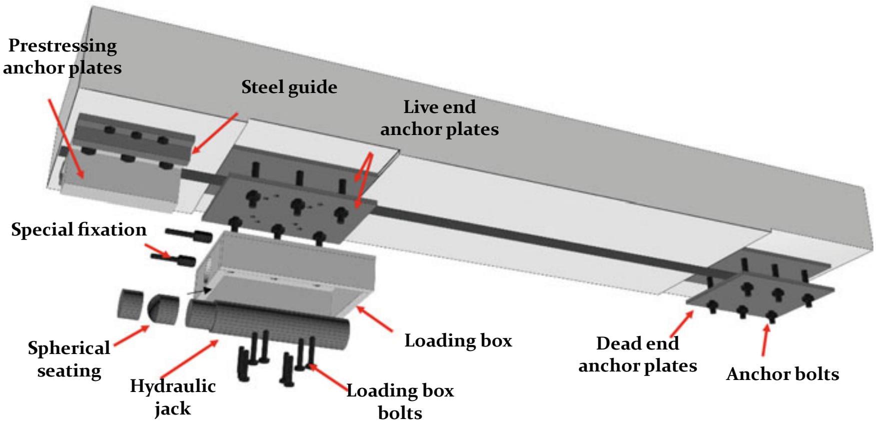


Fig. 4.8 Prestressing system for CFRP strips

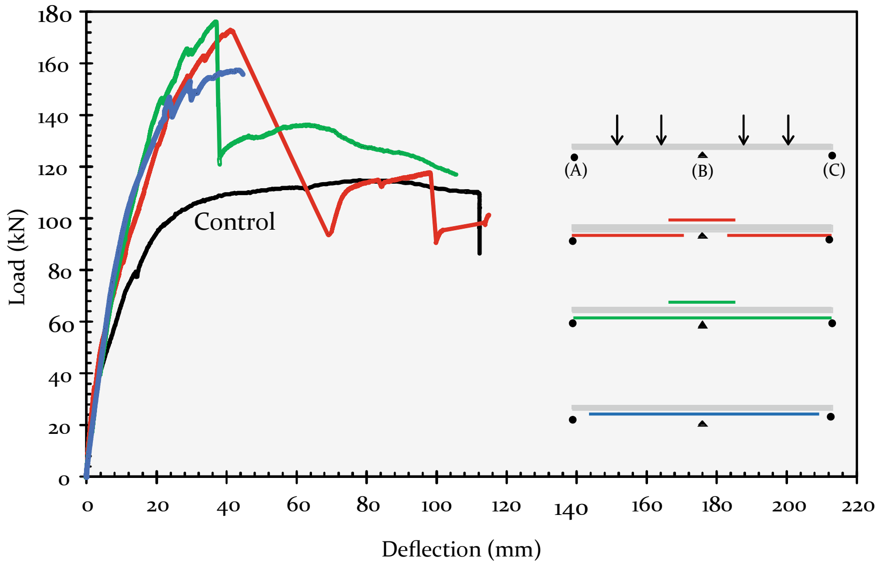


Fig. 4.9 Load–deflection of slabs prestressed with externally bonded CFRP

top prestressing strip and only strengthening at the bottom soffit is used (see the curve in blue). This phenomenon is explained by the secondary bending moment resulting from the prestressing reinforcement, which induces positive bending moment at the support section. Redistribution of the bending moment from the negative moment section to the positive moment sections is also one of the reasons to obtain large load capacity of the specimen strengthened only at the mid-span sections.

Figure 4.10 shows the primary, secondary and final bending moment resulting from one continuous strip placed at the bottom soffit of the slab ignoring the effect of the development length of the strips. While the primary bending moment is the resultant of the prestressing force multiplied by the cable eccentricity, the secondary bending moment (hyperstatic moment) is the moment resulting from the reaction induced from prestressing at the internal support(s). The final bending moment due to prestressing is the summation of the primary and the secondary moments. The final moment at the support section for the given prestressed strip profile is half the value of the prestressing force multiplied by the eccentricity of the strip and resulting in tension at the bottom soffit in a direction opposite to that resulting from gravity loads. This moment due to prestressing force should be linearly added to the calculated bending moment from gravity loads when calculating the service stresses in the concrete element.

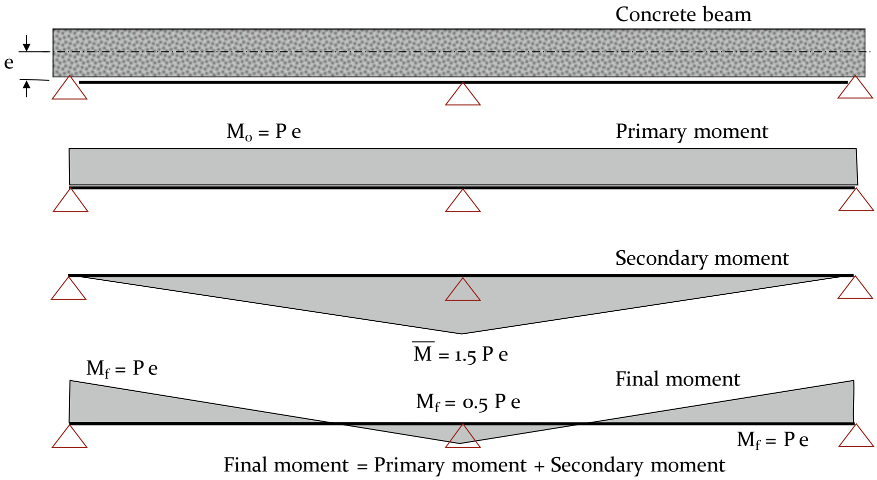


Fig. 4.10 Bending moments due to prestressing

4.5 Punching Shear

One of the most unfavorable failures of the flat slabs is due to punching shear since it is catastrophic and brittle. Therefore, design codes enforce high safety factors when designing for punching shear to ensure that this type of failure will not happen. Addition of steel reinforcement in the high shear zone, such as stirrups, shear heads or shear studs, provides some ductility, which is certainly required in design. Yet, several errors may be the reason for a reduced punching shear strength in the flat slabs such as casting concrete with compressive strength lower than the design value, misplacing the punching shear reinforcement or overlooking tension forces induced in the slab due to shrinkage restraint or temperature, which was proven to lessen the concrete shear strength. In this case, strengthening is required to restore the punching shear strength of the slab. This strengthening may be done by adding drop beams, thickening the slab at the column's vicinity using either steel or concrete caps or adding punching shear reinforcement.

4.5.1 Slab Strengthening by Columns Caps

Steel or concrete caps may be used at the top of the columns fixed to the slab so that the critical section of the slab in punching shear is shifted away from the column face and, hence, increasing the overall slab punching shear capacity. Typical steel cap used for slab strengthening is shown in Fig. 4.11. The steel cap is fixed to the column and the slab soffit using steel dowels. Steel stiffeners are used to increase the flexural stiffness of the steel plates fixed to the slab. Grout is used to fill the gap

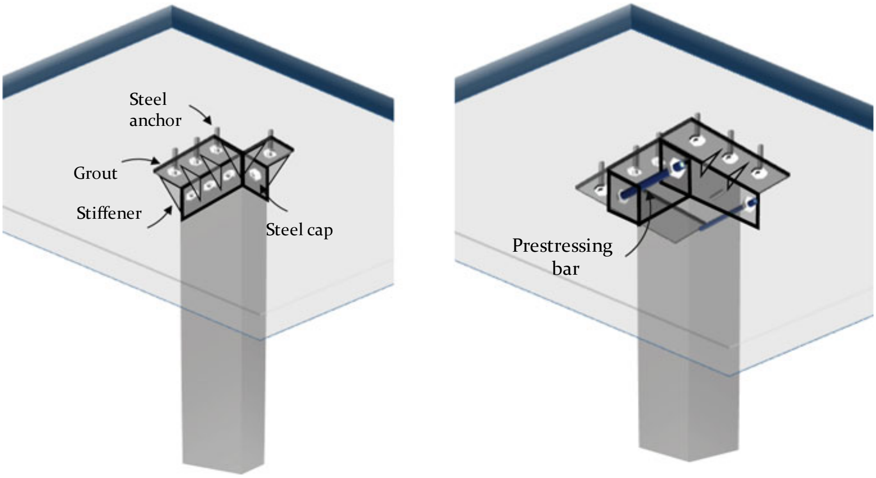


Fig. 4.11 Punching shear steel cap strengthening

between the steel cap and the bottom slab soffit to ensure good contact between the cap and the slab soffit. Strengthening efficiency shall be much reduced if crack or small gap exists between the slab and the cap since there are no expected vertical deformations of the slab in this zone under the effect of gravity loads. The shear strength of the concrete slab at a perimeter "b_o" located at "$d/2$" from the cap, where (d) is the slab depth, should be greater than the factored punching shear stress, v_u, as per Eqs. (4.7)–(4.14).

$$v_u \le \phi v_c \tag{4.7}$$

where

$$v_u = \frac{V_u}{b_o d} + \frac{\gamma_v M_{sc} C_{AB}}{J_c} \tag{4.8}$$

$$\gamma_v = 1 - \gamma_f, \tag{4.9}$$

and

$$\gamma_f = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{c_1+d}{c_2+d}}} \tag{4.10}$$

J_c is a property of the assumed critical section analogous to polar moment of inertia about the centroidal axis and calculated as per Eqs. (4.11) and (4.12) for the slab at intermediate and edge columns, respectively.

$$J_c = \frac{d(c_1+d)^3}{6} + \frac{(c_1+d)d^3}{6} + \frac{d(c_2+d)(c_1+d)^2}{2} \tag{4.11}$$

(Section at intermediate column)

$$J_c = d(c_2+d)(C_{AB})^2 + \frac{2}{3}d(C_{CD})^3 + \frac{2}{3}d(C_{AB})^3 + \frac{1}{6}(c_1+0.5d)d^3 \tag{4.12}$$

(Section at edge column)

M_{sc} is the moment transferred from the slab to the columns below and above, calculated at the centroidal axis of the critical section; the distances c_1, c_2 and C_{AB} are defined in Figs. 4.12 and 4.13. The figures also show the critical slab section in punching shear at intermediate and edge columns, respectively. It should be noted that at edge column, the punching shear load, V_{uv}, and moment resulting in punching shear stresses, M_{sc}, are calculated at the point (O'), as shown in the figure.

$$v_c = \text{The least of} \begin{pmatrix} 0.33\lambda_s\sqrt{f'_c} \\ \left(0.17 + \frac{0.33}{\beta}\right)\lambda_s\sqrt{f'_c} \\ \left(0.17 + \frac{0.083\alpha_s d}{b_o}\right)\lambda_s\sqrt{f'_c} \end{pmatrix} \tag{4.13}$$

$$\lambda_s \text{ (size effect)} = \sqrt{\frac{2}{1+0.004d}} \leq 1 \tag{4.14}$$

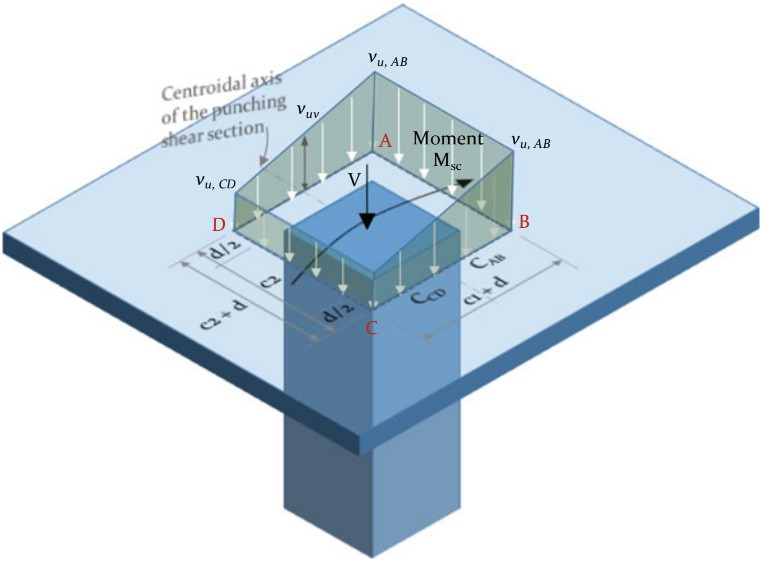


Fig. 4.12 Shear stresses at of the critical section at an intermediate column

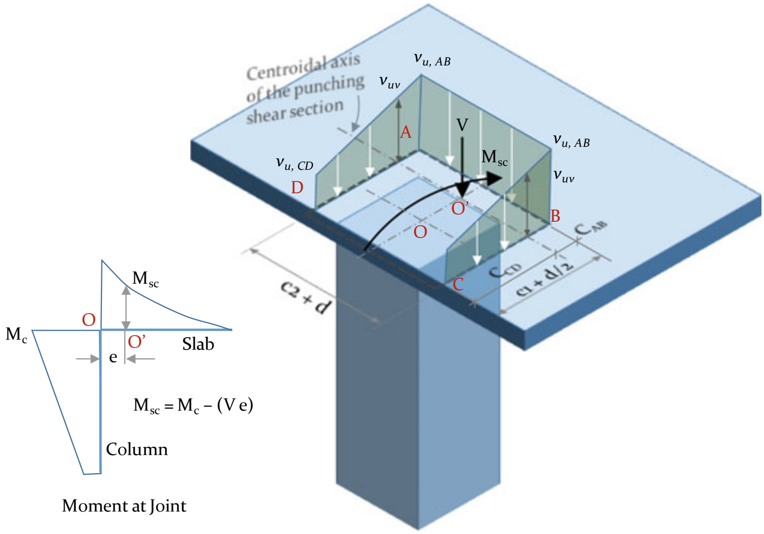


Fig. 4.13 Shear stresses at of the critical section at an edge column

where

ν_u applied shear stress
ν_c concrete shear strength
ϕ is the strength reduction factor = 0.75
β is the ratio of long-to-short sides of the column
α_s 40 for interior columns, 30 for edge columns and 20 for corner columns.

The value of $\sqrt{f'_c}$ shall not exceed 8.3 MPa.

4.5.2 Punching Shear Reinforcement

Adding punching shear steel reinforcement to increase the slab overall strength requires through drilling in the slab at the column's vicinity, which is usually congested with the flexural steel reinforcement. Cutting the steel rebars while drilling in the slab can be avoided using steel detector devices to locate the reinforcement. Once the rebars are located, holes with diameter larger than that of the stirrups are drilled and the steel shear bars are inserted through and tied from the top and bottom concrete soffits using steel nuts, as shown in Fig. 4.14. The holes must be injected with epoxy in order to protect the steel bars and prevent corrosion from happening. Number of shear bars are determined based on the design and the aspect ratio of the columns.

The total area of the steel bars at the critical section shall be designed according to Eqs. (4.15)–(4.19). Two critical sections shall be checked; the first section at

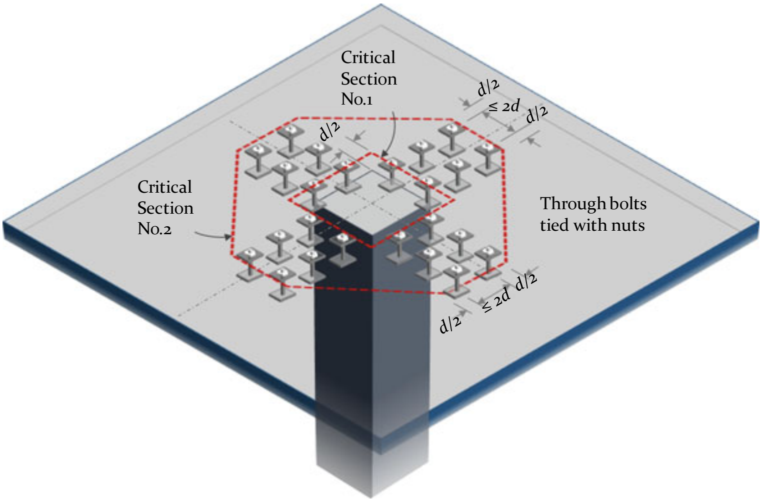


Fig. 4.14 Slab strengthening in punching shear

($d/2$) from the column face, where d is the depth of the concrete slab, is checked so that the total shear capacity shall be the summation for both the concrete and steel shear reinforcement capacities. The second section shall be checked so that only the concrete shear strength at the polygon perimeter beyond the shear reinforcement shall satisfy Eqs. (4.16) and (4.17). The spacing between shear bars should be less than $d/2$. For the case of rectangular columns and large square columns, additional steel reinforcement should be added so that the spacing between legs of the shear reinforcement does not exceed "2d" in the transverse direction.

$$v_u \leq \phi\,(v_c + v_s) \tag{4.15}$$

$$v_c = 0.17\lambda_s\sqrt{f'_c} \tag{4.16}$$

$$\lambda_s \text{ may be taken} = 1.0 \text{ if } \frac{A_v}{s} \geq 0.17\sqrt{f'_c}\,\frac{b_o}{f_y};\ \text{otherwise use Eq. 4.13} \tag{4.17}$$

$$v_s = \frac{A_v f_{yt}}{b_o s} \tag{4.18}$$

$$\text{Maximum } v_u = 0.66\phi\sqrt{f'_c} \tag{4.19}$$

where

v_s shear stress provided by steel reinforcement

A_v is the sum of the area of the shear reinforcement on one peripheral line that is geometrically similar to the column perimeter
s is the spacing between the peripheral lines in the direction perpendicular to the column face
f_y is the yield stress of the shear reinforcement $\leq$ 420 MPa.

Example 4.1 Concrete flat plate has a total thickness of 250 mm and depth to the top steel reinforcement of 210 mm. The plate is supported on an intermediate column with dimensions of 400 × 400 mm. It is required to check if the punching shear in the slab is safe according to the ACI318-19 code knowing that the concrete cylinder strength is 30 MPa and the applied loads are as follows:

VDL = 300 kN, VSID = 100 kN, VLL = 160 kN, and moment transferred to column (M_{sc}) = 70 kNm.

Answer

The critical section of punching shear is at distance equals $d/2 = 105$ mm from the column face. Therefore, $b_o = 4 \times (400 + 210) = 2440$ mm.

Considering Eqs. (4.9) and (4.10), $\gamma_f = 0.6$ and $\gamma_v = 0.4$.

Using Eq. (4.11), $J_c = 32{,}718{,}875{,}000\ \text{mm}^4$.

$$V_u = 1.2 \times (300 + 100) + 1.6 \times 160 = 736\,\text{kN}$$

$$v_u = \frac{736 \times 1000}{2440 \times 210} + \frac{0.4 \times 70 \times 10^6 \times \frac{620}{2}}{32{,}718{,}875{,}000} = 1.436 + 0.265 = 1.7\,\text{MPa}$$

To calculate v_c as per Eq. (4.13), $\lambda_s = 1.0$, $\beta = 1.0$, $\alpha_s = 40$,

$$v_c = \text{The least of} \begin{pmatrix} 1.81 \\ 2.74 \\ 2.5 \end{pmatrix} = 1.81\,\text{MPa}.$$

Therefore, $v_u = 1.7 > \phi\ v_c = 0.75 \times 1.81 = 1.36$ MPa (unsafe).

Example 4.2 It is required to strengthen the slab given in Example 4.1 using shear reinforcement with yield stress 0f 420 MPa.

Answer

Check if the maximum shear stresses are not exceeded as per Eq. (4.19) to avoid shear compression failure.

v_u (maximum) $= 0.66 \times 0.75\sqrt{30} = 2.71$ MPa $> v_u = 1.7$ MPa, therefore, steel reinforcement may be used to strengthen the slab in punching shear.

Check shear stresses at $d/2$ from the column face (section 1, see Fig. 4.15):

$$v_c = 0.17 \times 1.0 \times \sqrt{30} = 0.93\,\text{MPa}.$$

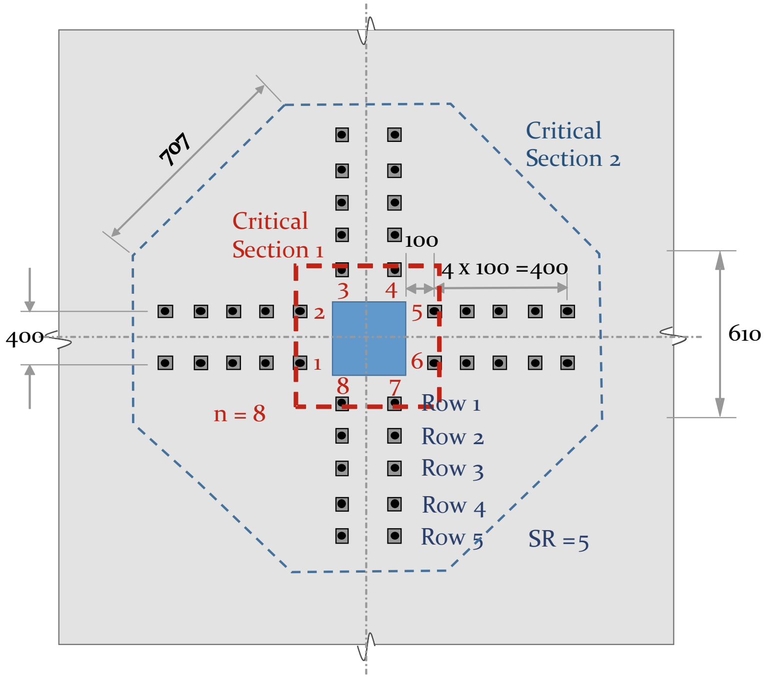


Fig. 4.15 Strengthening of slab in punching shear

Using eight 12 mm diameter steel shear bars ($n = 8$), per row every 100 mm:

$$A_v = 8 \times 113 = 904\,\text{mm}^2,\ b_o = 4 \times 610 = 2440\,\text{mm}$$

$$v_s = \frac{904 \times 420}{2440 \times 100} = 1.56\,\text{MPa}$$

Therefore, $v_u = 1.7 < \phi(v_c + v_s) = 0.75 \times (0.93 + 1.56) = 1.87$ MPa (safe).

Check shear stresses at section 2:

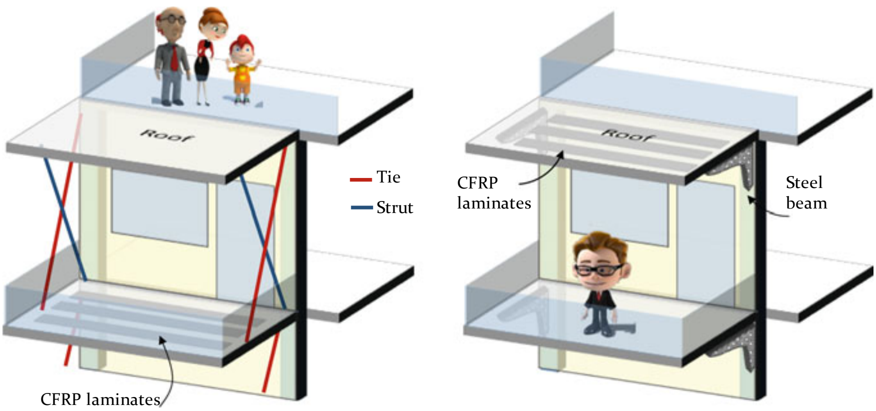
Use 5 rows of the shear reinforcement (SR = 5), therefore, $b_o = 4 \times 610 + 4 \times 707 = 5268$ mm.

$$v_u = \frac{736 \times 1000}{5268 \times 210} = 0.66\,\text{MPa} < \phi\, v_c = 0.75 \times 0.93 = 0.7\,\text{MPa} \quad \text{(safe)}$$

4.6 Balcony Strengthening

One of the common problems is the defects in cantilever balconies, which suffer from sagging and deterioration due to corrosion of steel reinforcement resulting from rain and bad maintenance. The cantilevers may be only made of concrete slab or slab resting on beams. It is sometimes required to do the strengthening works without disturbing the use of the building; therefore, adding concrete overlay or attaching steel or FRP reinforcement is not the best solution since it requires developing the reinforcement length inside the building. It is then recommended to do all the strengthening works avoiding disruption to the use of the internal space in the building. Adding inclined tension member (tie), or compression member (strut), to support the cantilever slab, as shown in Fig. 4.16a, is one of the techniques that fulfill the strengthening criteria. The inclined member shall be anchored at the slab-column connection, and as close to the tip of the cantilever to provide support to the slab edge.

Steel cantilever beams fixed to concrete columns and to the slab/beam bottom soffit may be also used, as shown in Fig. 4.16b. All the steel parts used in any of the systems should be well protected since it is subjected to rain and other environmental condition that may lead to steel corrosion. Strengthening the concrete balcony may be still required by adhering CFRP laminates at the slab bottom soffit in a direction parallel to the slab free edge to enhance the section capacity.



a) Use of tie and/or strut

b) Use of steel cantilever beam

Fig. 4.16 Strengthening of concrete balcony

4.7 Openings in Slabs

Openings in concrete floors are sometimes required after construction or in existing buildings to pass air conditioning or heating ducts, electric cables or sanitary pipes. Openings may be also needed for vertical movement between floors by adding stairs or elevators. In general, openings create localized straining actions and may change the load path in the slabs. The influence of openings on both the strength and deformational behavior of the slab should be considered.

In case that openings are made with dimensions less than that given in the plan shown on Fig. 4.17, externally bonded steel or FRP reinforcement at least equal to that interrupted by the opening shall be added on the sides of the opening. Adequate development length of the bonded reinforcement should be provided after the corners of the opening. There are zones where openings should be avoided as possible in the floors, such as in the punching shear zones around the columns, i.e., closer than four times the slab thickness, unless detailed calculations of the floor strengthening are provided.

For large openings, structural analysis of the floor and may be for the full structure shall be required to calculate the changes in the overall behavior and add new elements to restore both the capacity and serviceability of the elements. The new elements may be steel or concrete beams, external prestressing or punching shear reinforcement depending on the size and location of the opening.

Example 4.3 Concrete flat slab with 250 mm thickness is supported on columns spaced every 8.0 m in both directions. At the middle of an interior panel, the slab

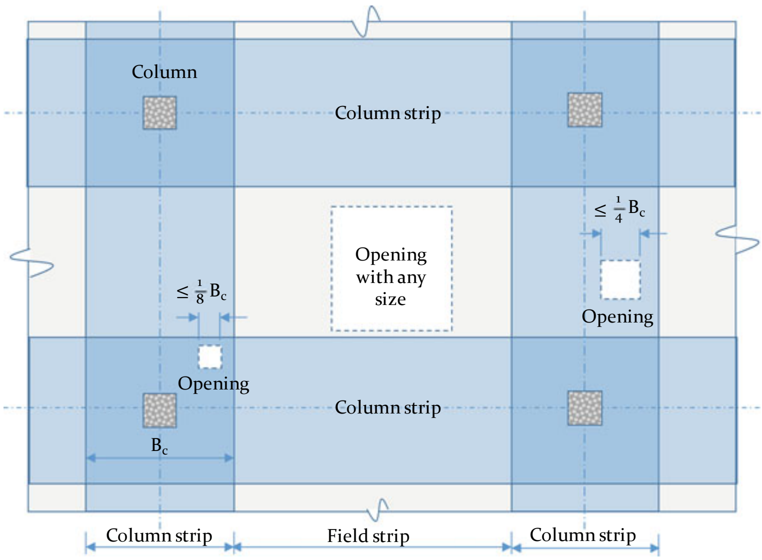


Fig. 4.17 Openings in different zones in concrete flat slab

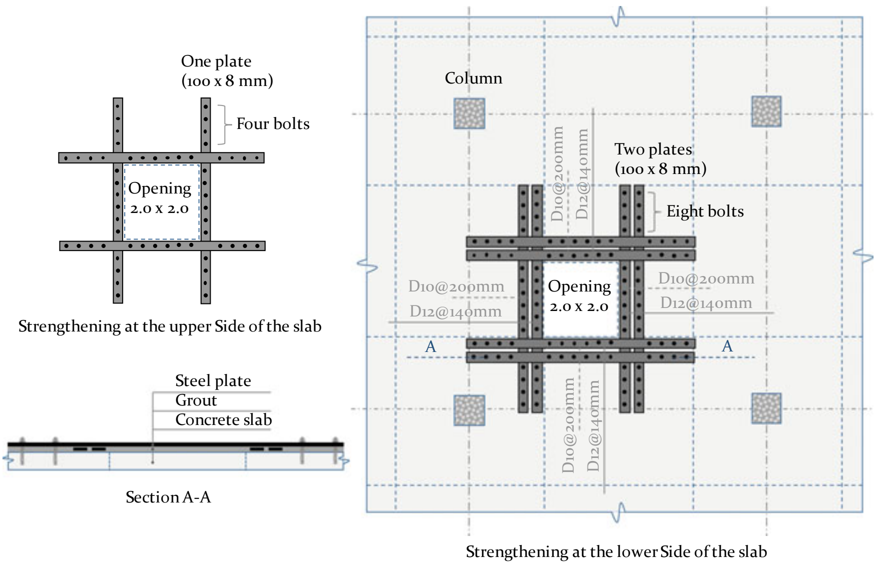


Fig. 4.18 Slab strengthening around opening

is reinforced with 12 mm diameter bar every 140 mm as main reinforcement in both directions and 10 mm diameter bar every 200 mm in both directions as top reinforcement. The yield stress of the reinforcing steel is 420 MPa. It is required to create an opening with dimensions of 2.0 × 2.0 m at the location shown in Fig. 4.18. It is required to design the slab strengthening around the opening using steel plates fixed to the slab with steel bolts. The yield stress of the steel plates and the bolts is 280 MPa.

Answer

Since the required opening is in the panel zone of intersection between two field strips, the slab strengthening is designed by compensation of the cut steel reinforcement at the top and bottom of the slab.

For the lower steel mesh,

The tensile force in the cut steel bars $= 2 \times 113 \times \frac{1000}{140} \times 420 \times \frac{1}{1000} = 678.0\,\text{kN}$.

Required area of steel plates on each side of the opening $= \frac{678 \times 1000}{2 \times 280} = 1210\,\text{mm}^2$.

Use two plates with dimensions 100 × 8 mm with area $= 2(100 - 20) \times 8 = 1280\,\text{mm}^2$.

Where holes of 20 mm diameter shall be made for the bolts.

Applying Eq. (3.6) to calculate the area of the steel bolts

$$A_\text{v} = \frac{C \text{ or } T}{\phi f_\text{y} \mu} = \frac{\frac{678 \times 1000}{2}}{0.75 \times 280 \times 1.0} = 1614\,\text{mm}^2$$

Use eight bars of 16 mm diameter, as shown in the figure. Strengthening of the slab due to cutting the top steel mesh is also shown in the figure.

4.8 Case Study 4.1

This case study is for a post-tensioned (PT), concrete floor under construction. The concrete slab is divided into three parts by two expansion joints, shown in red in Fig. 4.19. The two parts at the left and right are for twin towers, while the part in the middle represents the podium slab. The expansion joints are constructed so that the podium slab in the middle part (shaded in Blue), is resting on short cantilevers "corbels" protruding from circular columns located on both the left and right parts of the slab. Bearings were used to support the concrete slab on top of the corbels, as shown in Fig. 4.20, where high shear forces exist in this zone of the slab.

4.8.1 Problem Description

The concrete floor experienced punching shear failure at two locations in the slab close to the corbels. Figure 4.21 shows the failure in the slab, where cracking occurred, and vertical deformation of the slab was observed by 45 mm with respect to the supporting corbel after structural distress.

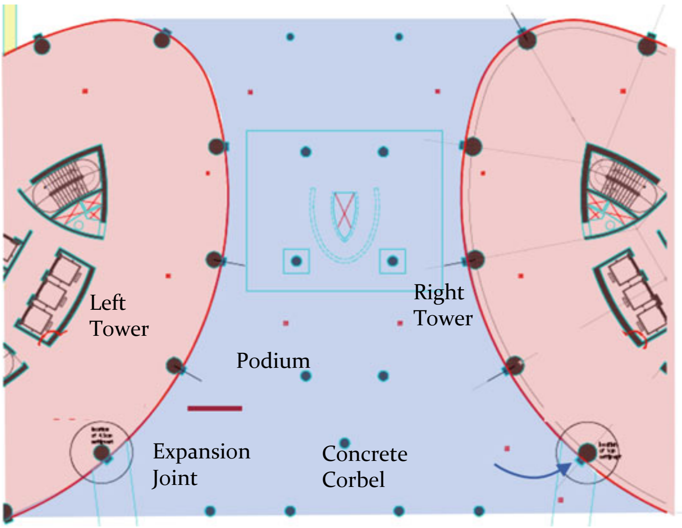


Fig. 4.19 Basement floor slab

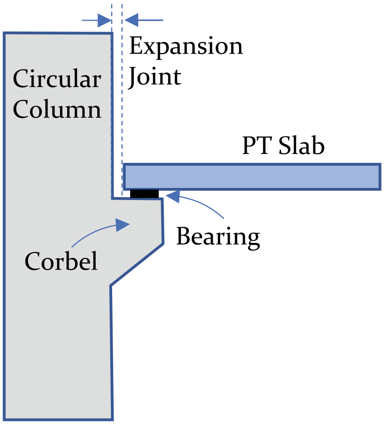


Fig. 4.20 Detail at the expansion joint

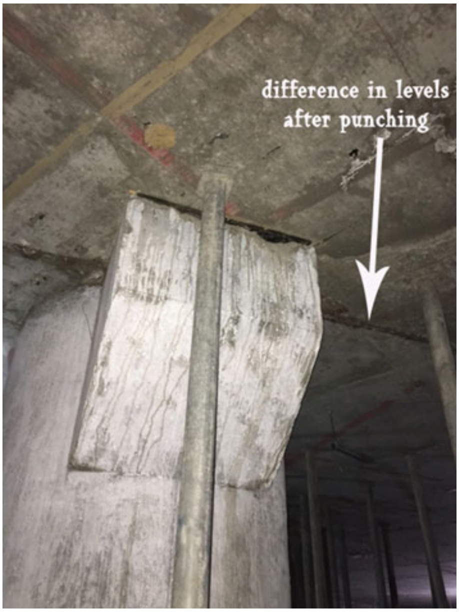


Fig. 4.21 Punching failure of the slab

Design and construction details of the prestressing steel reinforcement of the concrete slab indicated that the steel tendons were terminated in the slab away from the corbel vicinity and were not extended up to the support. Terminating the prestressing steel tendons before the corbel location reduced the punching shear capacity of the slab below the demand level and resulted in slab failure. This was aggravated by the developed tensile forces in the high shear zone due to the slab restraint at the time of stressing, as shown in Fig. 4.22. Despite the fact that the slab was placed on top of the bearings, friction still existed between the slab and the corbel and the bearings did not allow for free movement in the horizontal direction, and as a result, tensile stresses were developed in the shear zone. It is worth noting that tensile forces reduce intensely the shear capacity of the concrete section, as shown in Fig. 4.23.

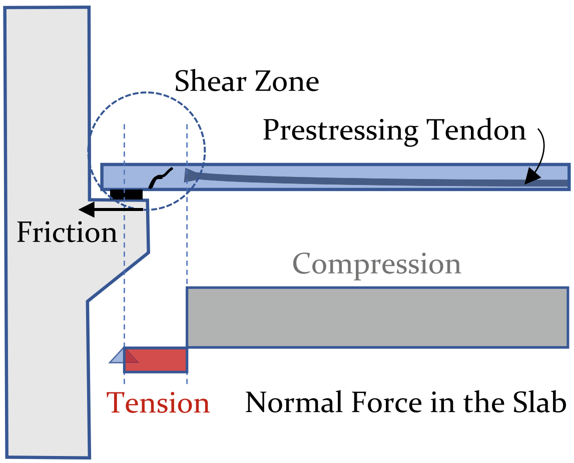


Fig. 4.22 Developed forces in the slab

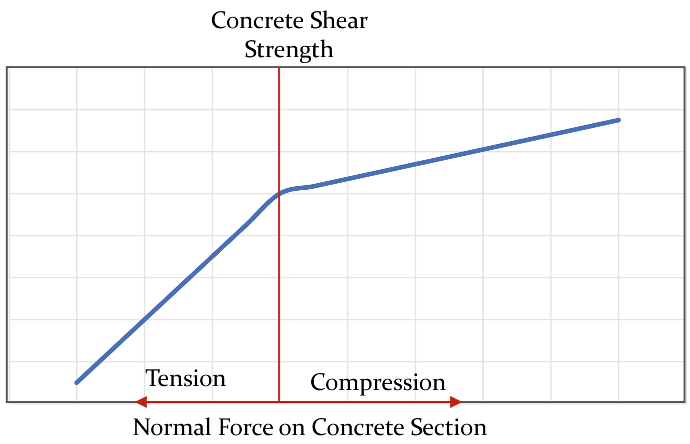


Fig. 4.23 Concrete shear strength

4.8.2 Retrofitting Proposal

In order to overcome the structural distress in the shear zone, two alternatives were proposed. The first alternative included removal of the concrete at the vicinity of the bearings and recast the slab with an enlarged thickness and punching shear reinforcement to satisfy the shear demand. This alternative required replacement of the bearing with adequate capacity to allow for free movement in the horizontal direction. This proposal has implications of stopping the use of the building during construction process, which was rejected by the client.

The second strengthening alternative relied on implementing steel truss, anchored to the circular column, to support the bottom of the slab, as shown in Fig. 4.24. The steel truss is composed of inclined member tied with another horizontal steel member, which are both connected to the column and the corbel by anchor bolts. Neoprene bearing was used on top of the inclined steel member to allow for free movement at the expansion joint. The damaged concrete was conventionally repaired after installing the truss system. This strengthening scheme aimed at reducing the shear demand as well as transferring the high shear zone away from the tension zone of the slab resulting from early termination of the prestressing steel strands. The strengthening scheme was successfully applied with minimum interruption of the construction time schedule of the project. A fireproof intumescent paint was applied on the steel system to protect it in the unlikely event of fire and keep the fire rating of the structure, as per the original design.

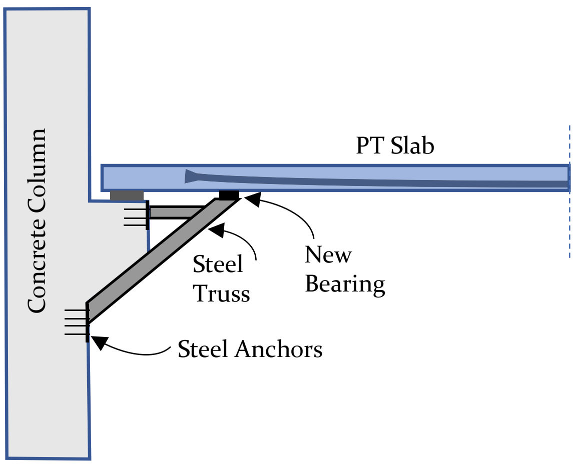


Fig. 4.24 Steel truss used to support the PT slab

4.9 Case Study 4.2

This case study is for an office building which after construction, the owner decided to change the use of the building to a commercial use. The "Plaza" building was constructed of two basements, ground and five typical floors. The concrete floors were constructed using post-tensioned (PT), flat plates with 220 mm thickness with drop panels to increase the slab thickness above the columns at the most stressed zones. The slabs were supported on square concrete columns with spans varying from 6.0 to 11.0 m.

The structural design modifications were challenged by several constraints including time limitations of construction, required clear head of each floor and removal of one of the interior columns supporting the post-tensioned concrete slab. After detailed discussions between the different parties of the project including the main contractor, specialized sub-contractor, consultants and owner, it was decided to implement different techniques in the strengthening design including concrete jacketing, steel jacketing and carbon fiber reinforced polymer laminates.

4.9.1 Strengthening Demand

Strengthening of the PT slabs was required to account for the increase in the design superimposed dead and live loads after the conversion of the building use from "offices" to "commercial". The new design included addition of large openings in the slabs for escalators and elevators to allow for vertical movement between floors. One of the openings was located above an interior column supporting a panel 11.0 × 8.28, as shown schematically in Fig. 4.25. The large openings had dimensions of 6.5 × 4.0 m, and repeated at three consecutive floors, as shown in Fig. 4.26. The challenge was to demolish one of the interior columns, shaded in red in Fig. 4.25, in the last floor which was located at the opening of the PT slab.

The slab was strengthened using a group of steel beams between the columns and carbon fiber reinforced polymer laminates and strips to increase the slab flexural capacity. The PT slab was propped and the opening was cut using diamond saw and core drills, as shown in Fig. 4.27. The slab was monitored during the entire process to ensure no excessive deflection of the slab. A new inclined column was cast, shaded in blue in Fig. 4.25, to support the slab. Figure 4.28 shows the opening after demolishing the supporting column and casting the newly added inclined column.

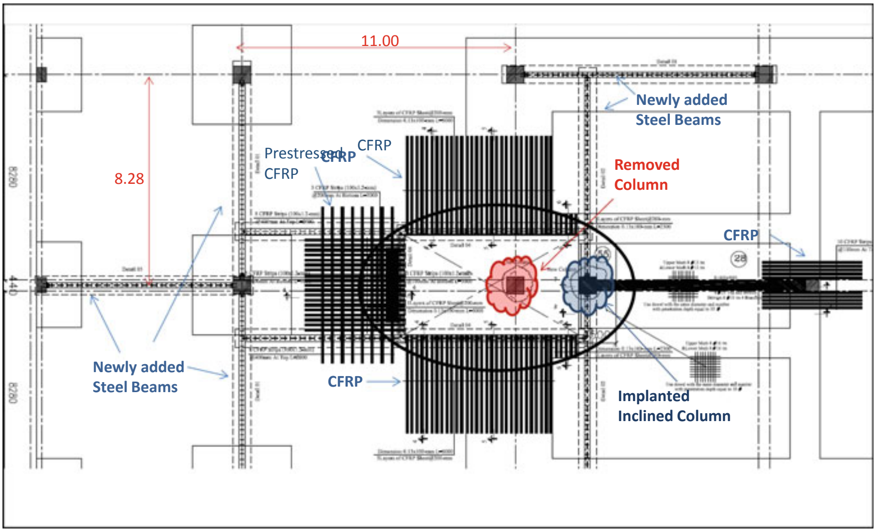


Fig. 4.25 Schematic of the slab strengthening due to large opening

Fig. 4.26 Three openings in three slabs

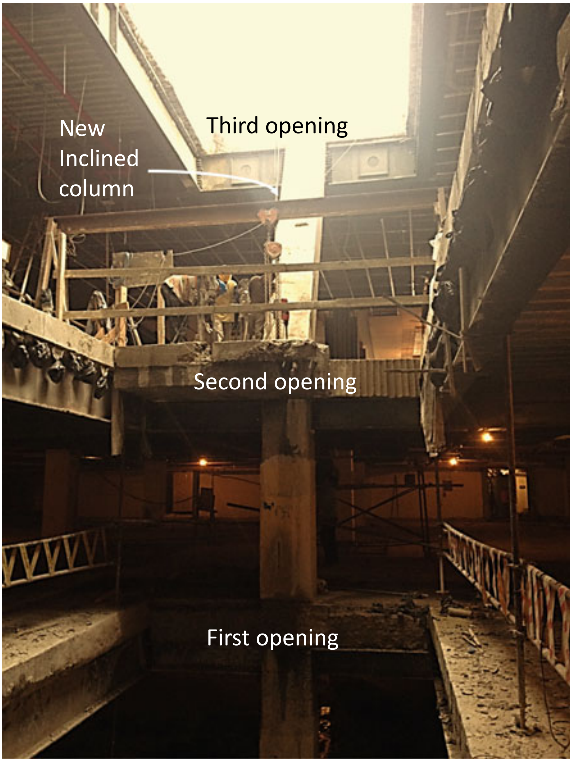


Fig. 4.27 Cutting the slab

After cutting several prestressing steel tendons, design showed that there is a need to compensate the loss of the precompression in the slab. It was decided to use externally bonded prestressed carbon fiber reinforced polymer (CFRP), strips, as shown in Fig. 4.29. CFRP offered a corrosion-free solution and in the same time did not alter the overall thickness of the concrete slab since its thickness was only 1.2 mm.

Fig. 4.28 Opening in the slab and new inclined column

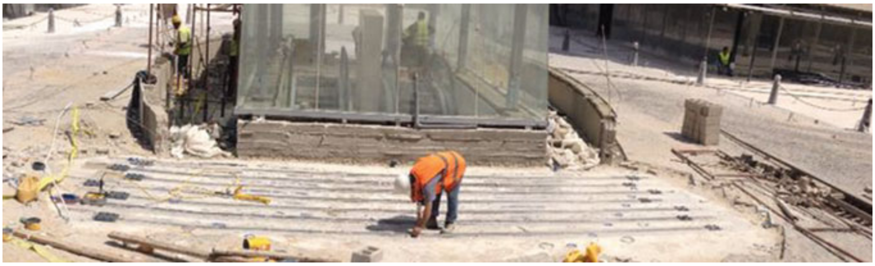


Fig. 4.29 Use of externally prestressed CFRP strips

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,

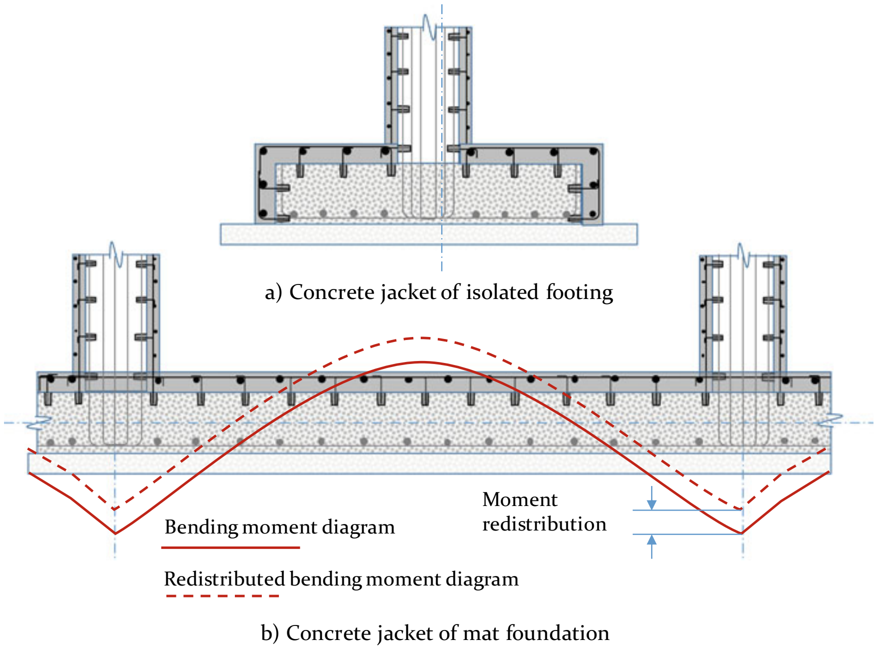


Fig. 5.1 Strengthening of isolated and mat foundations

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

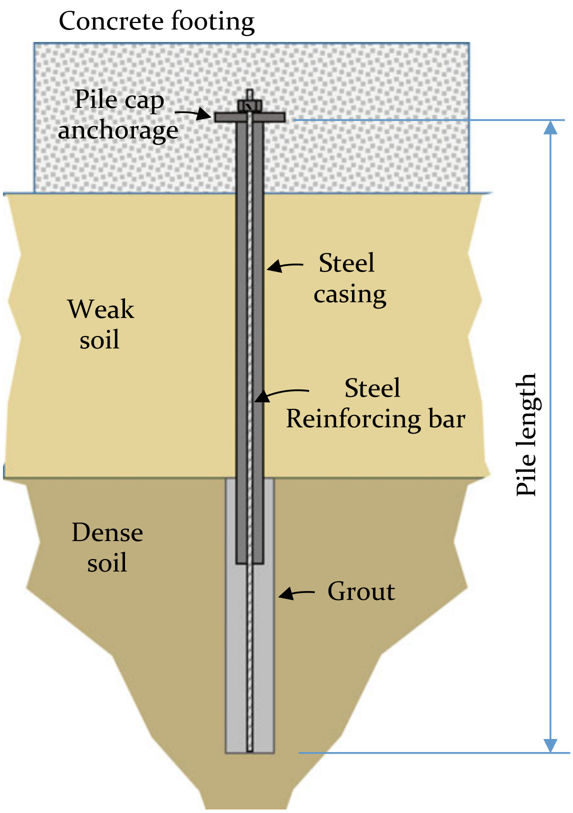


Fig. 5.2 Strengthening of foundations using micro-piles

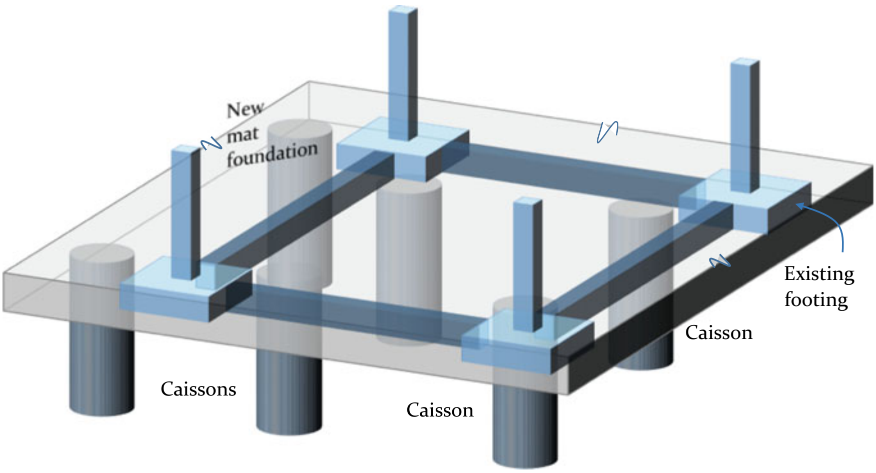


Fig. 5.3 New mat foundation supported on newly cast plain concrete caissons

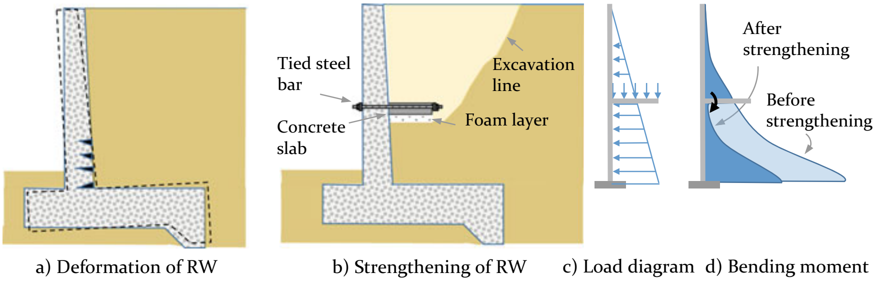


Fig. 5.4 Strengthening of retaining wall

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.

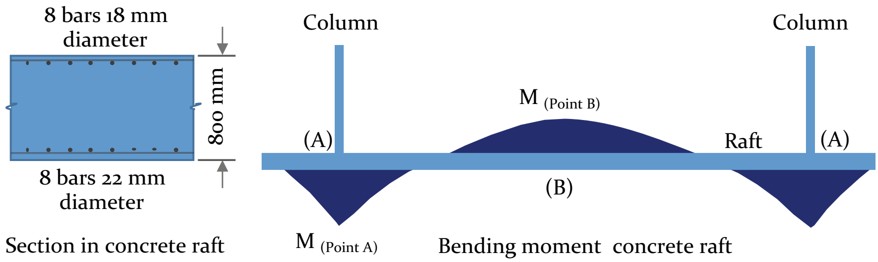


Fig. 5.5 Concrete raft (Example 5.1)

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).

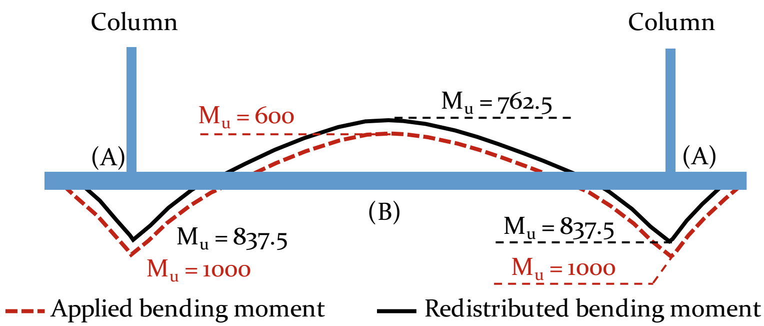


Fig. 5.6 Moment redistribution of the raft

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

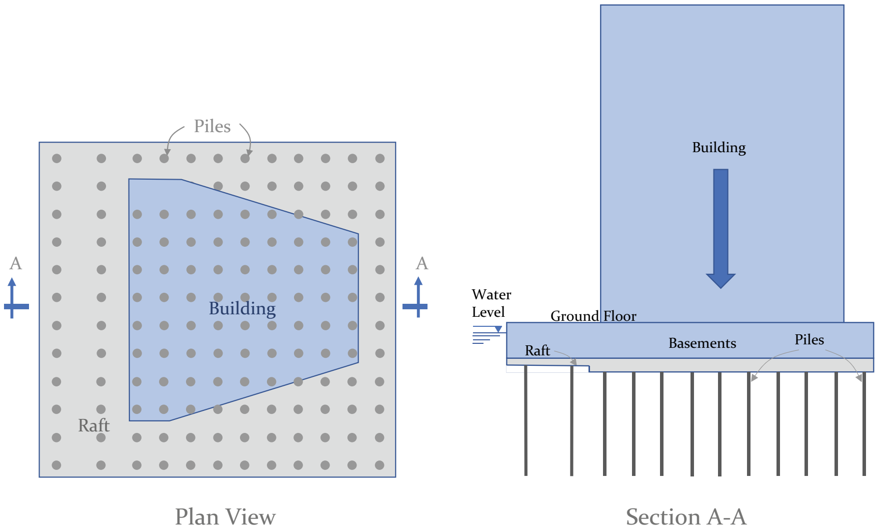


Fig. 5.7 Layout of the building

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

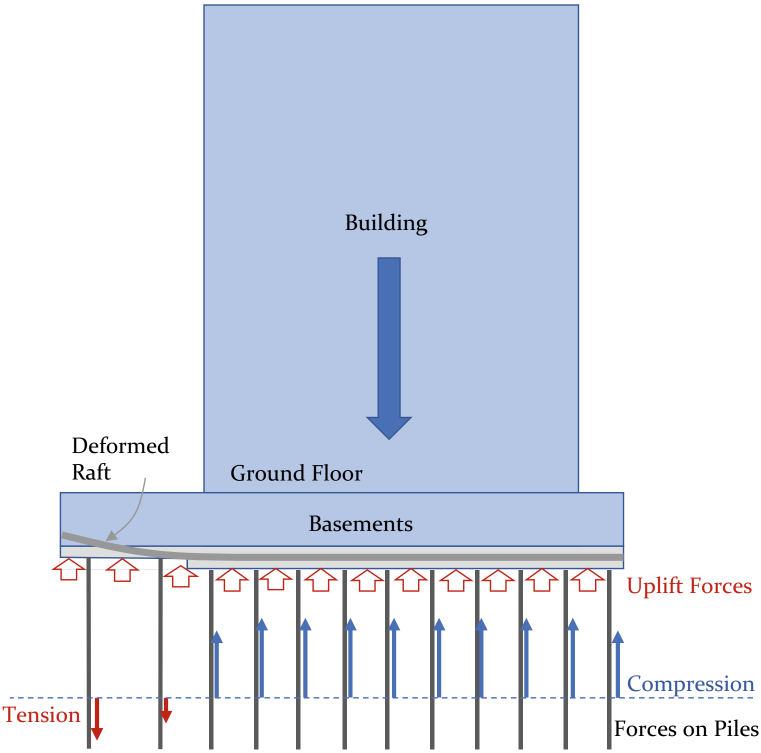


Fig. 5.9 Loads on raft and forces on piles

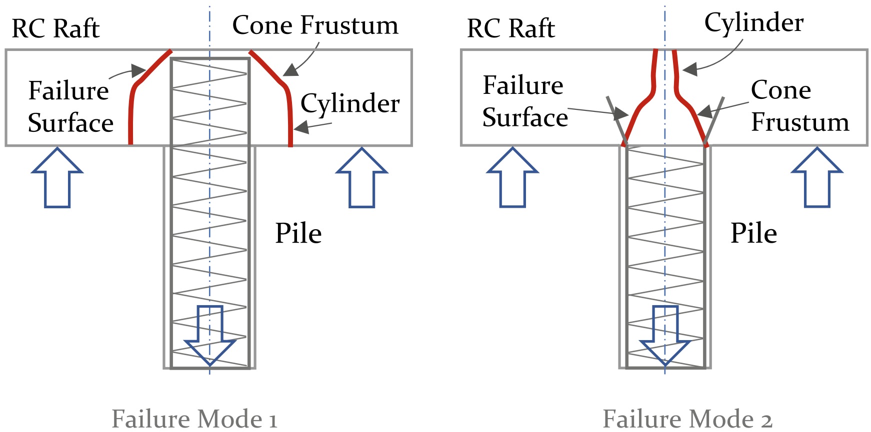
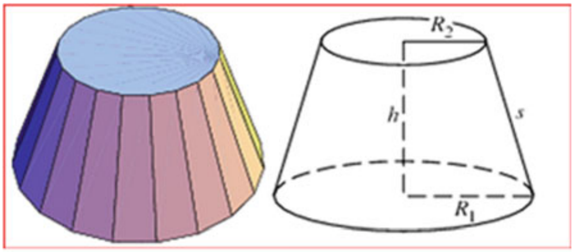


Fig. 5.10 Punching shear failure modes

Fig. 5.11 Punching shear failure plane



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

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

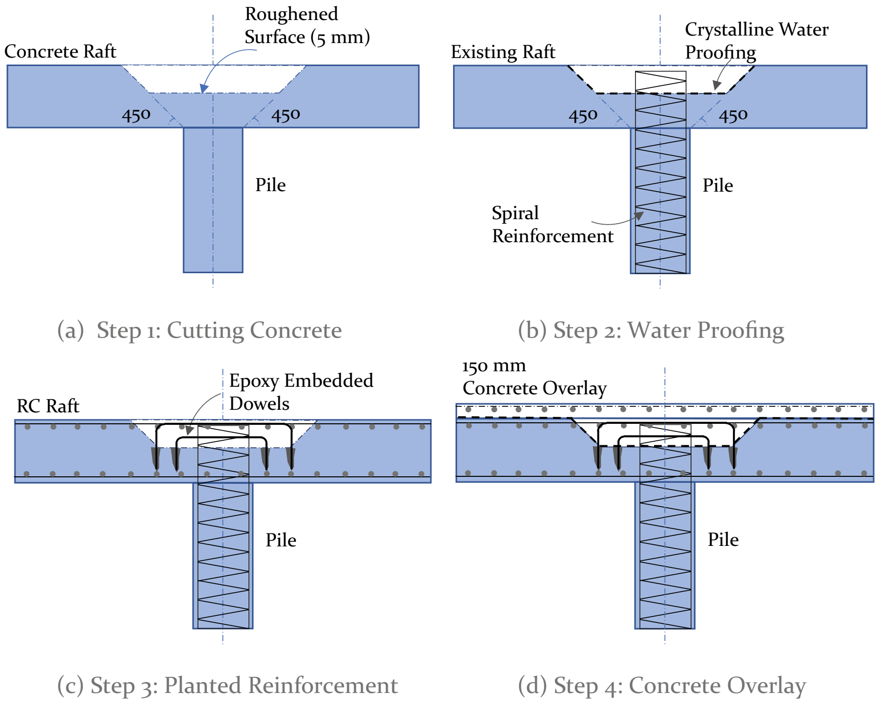


Fig. 5.12 Strengthening steps of the concrete raft

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

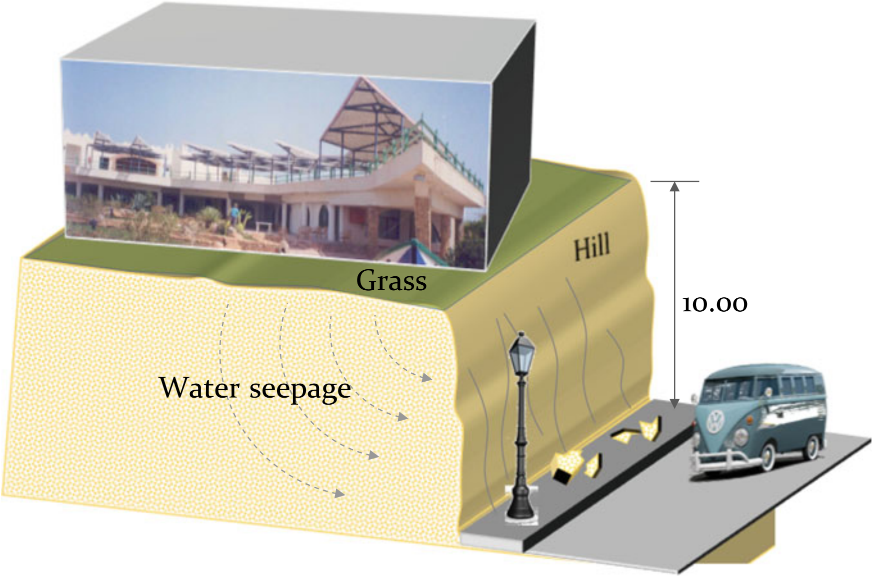


Fig. 5.13 Location of the Panorama building

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

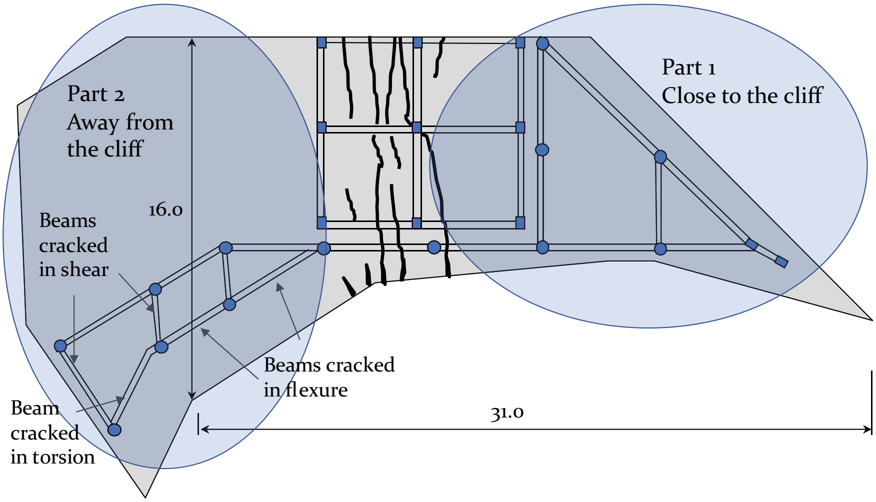


Fig. 5.14 Cracking in the slab of the Panorama building

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.

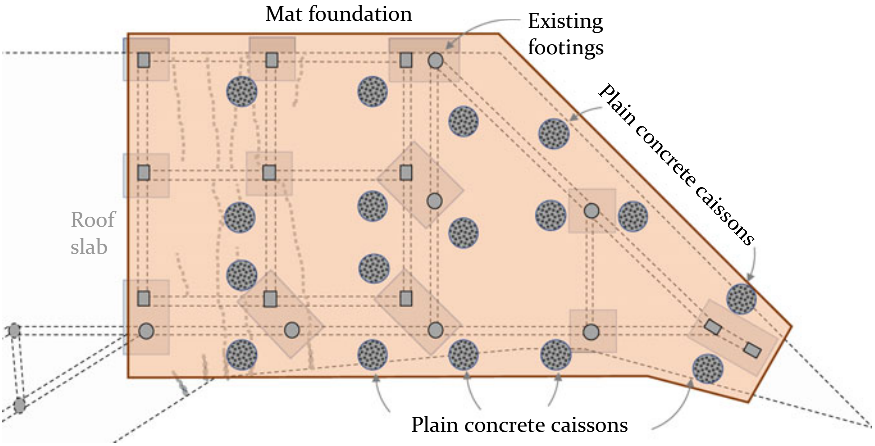


Fig. 5.15 Strengthening of the foundations

Fig. 5.16 Strengthening of the superstructure with CFRP

Chapter 6
Case Studies

This chapter introduces different case studies addressing special cases either for the loading type or structure type. Recognition of the problem and its causes are the success key to mitigate the distress of the structure. It is important to address different strengthening schemes after detailed analysis of the problem since there is no unique solution for the problem. The most efficient solution is the one allows for minimum disruption along with the functionality of the structure and cost of strengthening implementation. Usually, monitoring of the structure after mitigation is important especially when strengthening involves new techniques or confidence in the information of the existing structure is not high.

6.1 Case Study 6.1

This case study is for a cooling bed in an industrial plant for steel manufacturing, where hot steel elements are cooled down while being moved on an inclined plane on top of reinforced concrete footings. Figure 6.1 shows a part plan of the cooling bed foundations and typical view for one of the footings (2.0 × 2.0 × 3.7 m). As a result of the cooling process, temperature rises at the concrete surface of the footings with different values reaching up to 135 °C. The elevated temperature is a main design criterion for such structure, which may have not been considered in the original design.

6.1.1 Problem Statement

The visual inspection revealed that footings were affected by the high temperature and relatively wide cracks on all faces of the footings were recorded. The temperature

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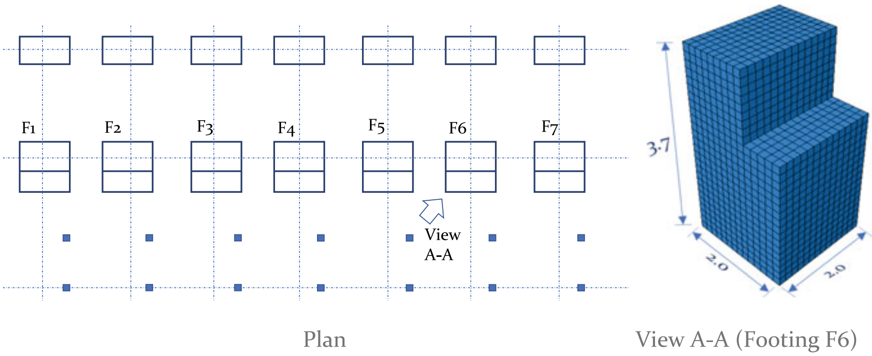

Fig. 6.1 Footings plan and view

readings report for the affected footings showed that the recorded surface temperatures on the concrete ranged from 50 to 135 °C. NDT reports showed that the vertical steel reinforcement in the footings is spaced from 220 to 280 mm across the footing, and the concrete cover ranges between 30 and 80 mm. As for the horizontal steel, the spacing was found to vary between 140 and 160 mm, and a concrete cover between 20 and 60 mm was measured. Furthermore, the concrete compressive strength was found to be at 34 MPa.

6.1.2 Finite Element Analysis

In order to investigate the effect of the high surface temperature of the concrete foundations, a three-dimensional finite element model was built up and analyzed. The model was created using elastic 20-node thermally coupled brick, tri-quadratic displacement, tri-linear temperature elements to model the concrete continuum and elastic 3-node 3D thermally coupled truss elements to model the steel reinforcement. Temperature loading was applied on the footing model as uniform surface temperature using average values of the recorded measurements at each face. The thermal properties of the used materials were taken as per Table 6.1.

Figure 6.2 shows a plot of the principal stresses contour on the front and back faces of the footing and the associated observed crack pattern. The contour shows the zones, where the principal tensile stresses are in excess of 3 MPa, which was assumed to be the tensile strength of the concrete. The results confirm, to high degree, the

Table 6.1 Assumed material properties in the foundations analytical model

Material	Thermal expansion (mm/mm °C)	Thermal conductivity (W/(mm °C))
Concrete	10.0^{-6}	3
Steel	13.5^{-6}	45

observed crack pattern shown in the figure, which may be visualized as a consequence of the thermal expansion of the concrete cover.

The depth of cracks inside the footings was also investigated by plotting the principal stresses contour in excess of 3.0 MPa on a section cut through the footing. Figure 6.3 shows that the tensile stresses that exceeded the concrete tensile strength go as deep as 150 mm from the surface. The depth of some cracks was measured on site to be extending up to 200 mm far from the surface, which further verifies the validity of the developed model. FE analysis also showed that the tensile stresses in the steel reinforcement of the footing were always less than the yield stress of the reinforcement.

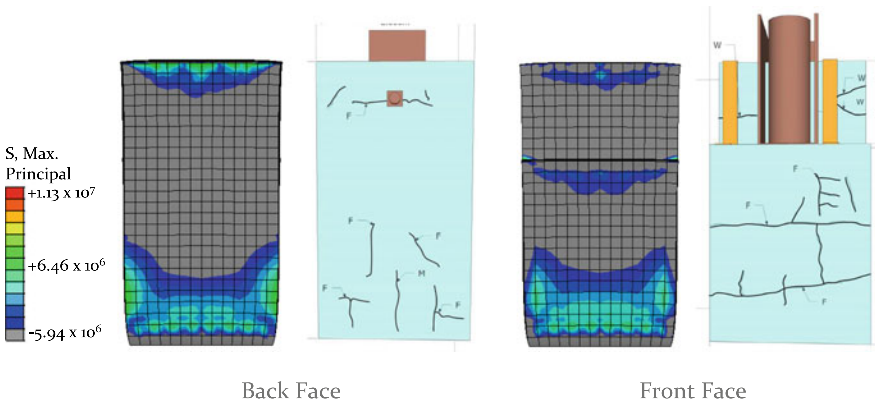

Fig. 6.2 Contour of maximum principal stresses versus crack pattern

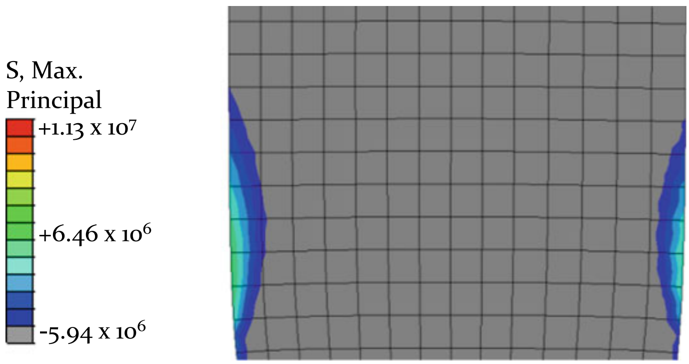

Fig. 6.3 Section cut of the contour of maximum principal stresses

6.1.3 Strengthening Scheme

The structural problems with the concrete foundations of the cooling bed may be viewed as a result of the weak tensile strength of the concrete cover with small and variable thickness. It is believed that cracking of the concrete footing does not affect its structural capacity, but rather its durability. In order to restore the footing original condition and to maintain the durability of the concrete footing and prevent any steel corrosion, the following conventional retrofitting scheme was implemented:

– Peeling of the entire concrete cover of the footing to the level of the steel reinforcement.
– Injection of any visible cracks with pressured low-viscous material.
– Insertion of steel dowels (M16 on 250 mm grid at each face).
– Installation of welded wire mesh all around the footing with adequate overlap and development length.
– Casting a new concrete cover of a minimum of 60 mm thickness with the steel mesh 20 mm away from the exterior face. The concrete had characteristic compressive strength of 60 MPa.

It should be noted that all used materials had high "glass transition temperature (T_g)" to withstand the high temperature at the concrete surface.

6.2 Case Study 6.2

Concrete dolphin close to the shore of the Mediterranean Sea was exposed to a ship hit resulted in various degrees of flexural damage in the top part of the concrete pile. The structure is 8.0 × 8.0 m supported on 4 inclined piles, as shown in Fig. 6.4. The piles were constructed with steel casing with its top level above the sea level and 600 mm below the bottom level of the pile cap. The horizontal component caused by the impact force of the ship hit resulted in structural damage in the part of the pile between the steel casing and the bottom surface of the pile caps, as shown in Figs. 6.5 and 6.6. Concerning the damaged piles, it was noted that two distinct levels of flexural damage occurred in the concrete piles, as follows:

1. High Level of Flexural Damage (Piles where Concrete Crushing Occurred): This category of flexural damage involved a few number of piles and was characterized by a large number of flexural cracks with large widths accompanied by concrete crushing in the opposite side. This clearly reveals the occurrence of flexural failures in these regions, involving the high tensile stresses in the internal steel reinforcing bars and the associated loss of the initial flexural capacity under horizontal loads.
2. Low Level of Flexural Damage (Piles where Only Concrete Cracking Occurred): This category of flexural damage involved the rest of the damaged piles and was characterized by a fewer number of flexural cracks with smaller widths. No

Fig. 6.4 Overview of the dolphins

Fig. 6.5 Concrete crushing

concrete crushing was observed in this category, hence, revealing that only a partial loss of the initial flexural capacity under horizontal loads is suspected in these regions.

6.2.1 Strengthening Methodology

Strengthening of the damaged piles aimed at retaining the flexural capacity and integrity of the top parts of the piles. The strengthening methodology included application of both glass and carbon fiber reinforced polymers (GFRP), and (CFRP) sheets as well as CFRP anchors, as shown schematically in Fig. 6.7 and as follows:

Fig. 6.6 Large widths flexural cracks

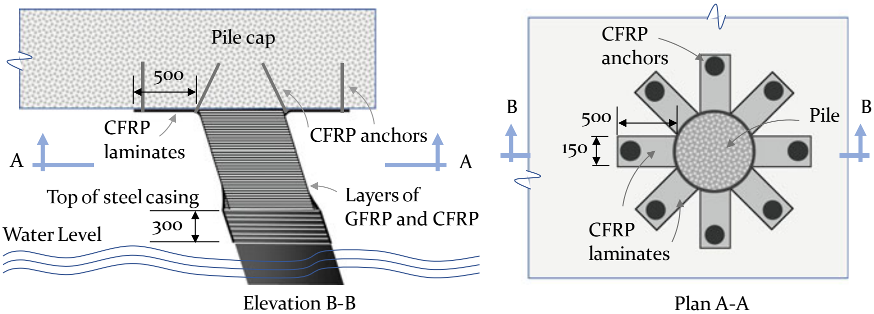

Fig. 6.7 FRP strengthening of the piled dolphin

1. The visible cracks in the piles were injected with low viscosity epoxy resin using adequate pressure in predrilled holes or fixed nipples. The exposed concrete part of the pile was repaired with epoxy mortar in order to be ready to attach the FRP on its surface.
2. The intersection between the pile and the pile cap was rounded using repair mortar with a minimum radius of 50 mm. This is to avoid any stress concentration in the FRP laminates at the edges of the concrete section.
3. One layer of GFRP sheets was applied on the top 300 mm of the steel casing in order to isolate the steel pipe from the CFRP sheets, which will be adhered on top of the GFRP sheets. The reason behind application of the GFRP sheets is to prevent galvanic corrosion, which is likely to occur if carbon and steel are in touch with each other. This corrosion will affect the structural integrity and capacity of the CFRP sheets.
4. One layer of CFRP sheets was applied, while the fibers oriented parallel to the centerline of the piles, to increase the flexural strength of the top part of the pile. The sheets were bonded to the top 300 mm of the steel casing and applied

partially on the repair mortar on top of the exposed concrete part of the pile. The sheets were extended to the bottom of the pile cap with a length of 500 mm and width of 150 mm, as shown in Fig. 6.7.

5. CFRP anchors were applied with a 6 mm equivalent diameter and uniformly spaced around the circumference of the pile at the intersection with the pile cap. Another CFRP anchors were applied and uniformly distributed around the circumference of the pile, as shown in Fig. 6.8. The anchors were applied 75 mm away from the end of each CFRP sheet. Figure 6.9 shows the steps of CFRP anchor application.
6. Two more layers of CFRP sheets were applied, while the fibers oriented parallel to the centerlines of the piles to add to the pile flexural capacity. The sheets covered 600 mm from the top of the piles and extended to the bottom of the pile cap with a length of 300 mm and width of 150 mm.

Fig. 6.8 Preparation of CFRP anchors application

Fig. 6.9 Application steps of CFRP anchor

7. Two final layers of CFRP sheets were applied, while the fibers oriented perpendicular to the centerlines of the piles, as shown in Fig. 6.10 to protect the internal CFRP sheets from the environmental conditions. Protection of the CFRP sheets was applied by applying special paint on top of the FRP, as shown in Fig. 6.11.

A load test was carried out after rehabilitation of the piles to verify the validity of the strengthening scheme. The test was conducted by having a ship with a specific weight traveling at a fixed low speed hitting the edge of the dolphin. No cracks were recognized, and the horizontal movement of the pile cap was measured and found to be acceptable.

Fig. 6.10 CFRP in the transverse direction

Fig. 6.11 Dolphin after rehabilitation

6.3 Case Study 6.3

A multi-story building consists of 20 floors used as hotel in the first five floors and for residential use in the remaining top floors. The sixth floor is a transfer slab, with 1.0-m thickness, where the columns distribution in plan is changed to accommodate the different functions of all floors. The transfer floor was used to accommodate the electromechanical equipment for the entire building. The columns distribution below and above the sixth floor level, shown in Fig. 6.12, indicates that there is a shift in the alignment of the vertical supporting elements, especially at the façade of the building.

It was the owner request to add three more floors on top of the building and add a heavy-weight marble cladding at the south façade, when the calculations showed that the transfer slab need strengthening at some locations to allow for the new loads. A three-dimensional finite element model showed that the transfer slab at the vicinity of four vertical elements (number 2, 14, 16 and 17, see Fig. 6.12) requires strengthening with a large percentage in flexure and shear. The conventional techniques using external steel or FRP reinforcement or concrete jacketing were studied and found to be impractical due to the large amount of strengthening, in addition to the owner request

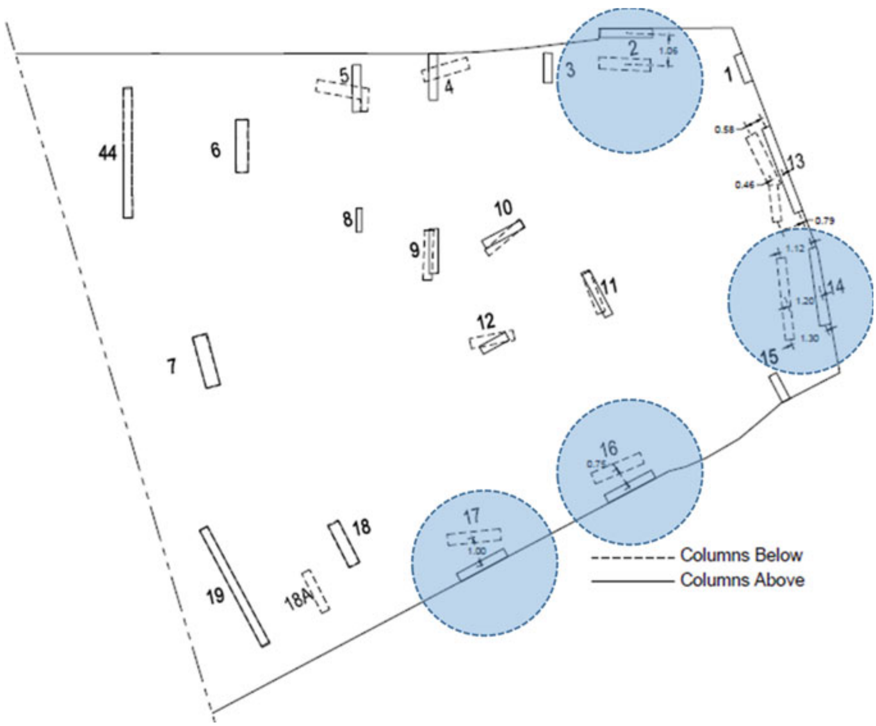

Fig. 6.12 Plan of the transfer slab

not to evacuate the sixth floor during construction. Accordingly, it was decided to follow a different approach in the slab strengthening.

6.3.1 *Strengthening Scheme*

The main idea of the proposed strengthening scheme relied on redirecting the vertical load of the top supported floors to the supporting columns and walls below the transfer slab. This was achieved by adding inclined struts connecting the shifted supporting element to the columns below, as shown in Fig. 6.13. Two steel struts were used at each location of the four vertical elements (2, 14, 16 and 17) to carry their share of the applied loads on the vertical elements. Simple truss analysis, as per shown in Fig. 6.13, was used to calculate the compression forces in the struts (F_1) and resulting tension force in the slab above (F_2). Table 6.2 summarizes the results of the analysis and the forces in the truss model.

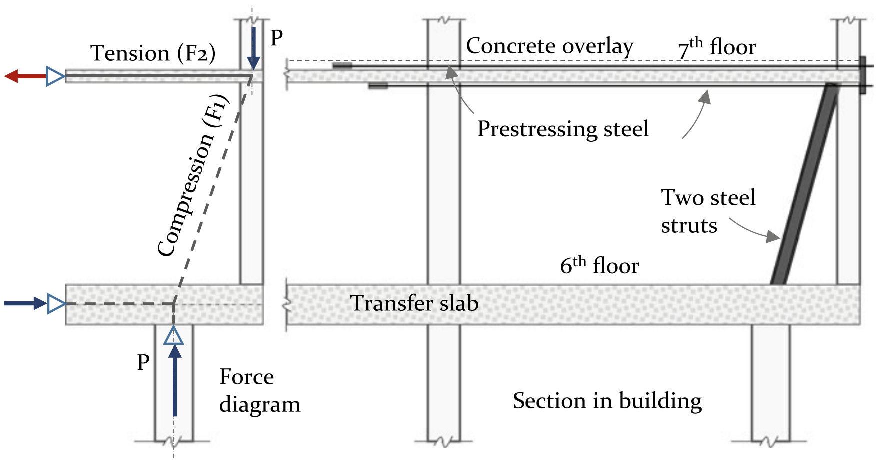

Fig. 6.13 Strengthening scheme of the transfer slab

Table 6.2 Truss model analysis summary

Element	Load (P) (kN)	Axis shift (m)	F_1 (kN)	F_2 (kN)	Number of struts
C2	2911	1.06	3073	985	2
C16	4200	0.75	4318	1006	2
C17	4220	1.00	4430	1348	2
W14	3965	1.20	4246	1520	2

It should be noted that the strengthening system will share in carrying the loads applied on the structure after construction completion of all the works. The self-weight of the existing structure and any other loads existed on the structure before applying the strengthening works were carried completely by the existing system before strengthening.

"H" steel members were used to carry the compression in the strut and fixed to the slabs at the top and bottom using steel plates and anchor bolts. The slab at the seventh floor was strengthened at its top and bottom using prestressing steel tendons to carry the induced tension component resulting from the inclined struts. The tendons at the top of the slab were embedded in 100-mm concrete overlay, while the externally applied prestressing steel tendons at the slab bottom surface were well protected against corrosion since they were left exposed. The used tendons applied on the top of the slab were anchored in flat ducts and the concrete overlay was connected to the existing 250 mm slab using epoxy installed shear dowels.

Applying this strengthening concept had multiple advantages such as reducing the deflections at the end of the transfer cantilever slab, which could have affected the façade of the top floors negatively. The scheme also minimized the quantities of strengthening steel elements when compared to conventional schemes relying on increasing the capacity of the transfer slab. Moreover, the system was implemented without interruption of the use of the electromechanical equipment placed in the sixth floor.

6.4 Case Study 6.4

A residential complex consists of several villas were under construction in a city situated in a high seismic zone, where the city was codified to have a design ground acceleration (a_g) of "0.25 g". The villas were one and two stories high made of concrete skeleton with a small footprint in plan. The structural design peer review found out that the earthquake loads were not considered in the design, while the contractor was half way in construction. Structural analysis of the villas showed that large concrete jackets are needed for retrofitting the columns in order to resist the earthquake loads, which was rejected by the architect since it has a negative impact on the interior design of the villas. In order to meet the time schedule of the project and to fulfill the architect requirements, an innovative solution was implemented to ensure that the structures of the villas are earthquake resistant.

6.4.1 Retrofitting Scheme

After review of the architectural plans of the villas, it was observed that infill walls were located at two exterior façades and around the interior stairs. The strengthening idea was based on converting some of the infill walls to structural walls resisting the

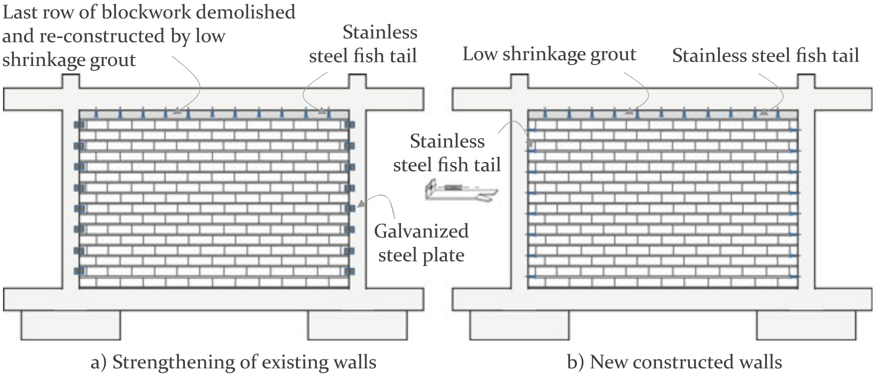

Fig. 6.14 Strengthening of existing walls and construction of new walls

earthquake loads. Consideration of the infill walls to resist the lateral loads will result in reduction of the straining actions in the vertical concrete elements; yet, it has a disadvantage of increasing the overall stiffness of the structure and hence increasing the earthquake load demand. The tradeoff between increasing the number of lateral-resistant walls and stiffness of the building was calculated to select the infill walls converted to structural walls.

After a number of the infill walls was selected to participate in the lateral load resistance, strengthening of those walls was applied. The strengthening included connecting the side boundaries of the walls to the concrete skeleton using galvanized steel plates anchored with steel bolts. At the top of the wall, the top row was dismantled and low shrinkage grout was cast, while connected to the beam on top with stainless steel fishtail anchors, as shown in Fig. 6.14a. Connection between the infill walls and the concrete skeleton confined the walls and provided shear transfer at the walls' edges.

For the newly constructed walls, which were selected to participate in the lateral load resistance of the structure, the walls were connected at its boundaries to the concrete skeleton. Stainless steel fishtail anchors were used every other course in the blockwork and anchored to the concrete columns, as shown in Fig. 6.14b. The last top course was cast with grout, where the anchors were connected to the top beam and embedded in the grout.

In a recent study to investigate the behavior of both infill and confined frames under lateral loads (Mantawy et al. 2017), failure of the infill frames, shown in Fig. 6.15a, occurred after separation between the wall and the concrete skeleton. The confined walls behaved differently at failure, where no separation was observed at the blockwork-concrete interface, as shown in Fig. 6.15b. Failure of the confined wall was observed experimentally due to a major inclined crack passing through the wall and the concrete element, as shown in Fig. 6.15c. The wall braced the concrete skeleton where a strut was formed in the inclined direction with large tension forces in the perpendicular direction causing one major crack and failure of the system.

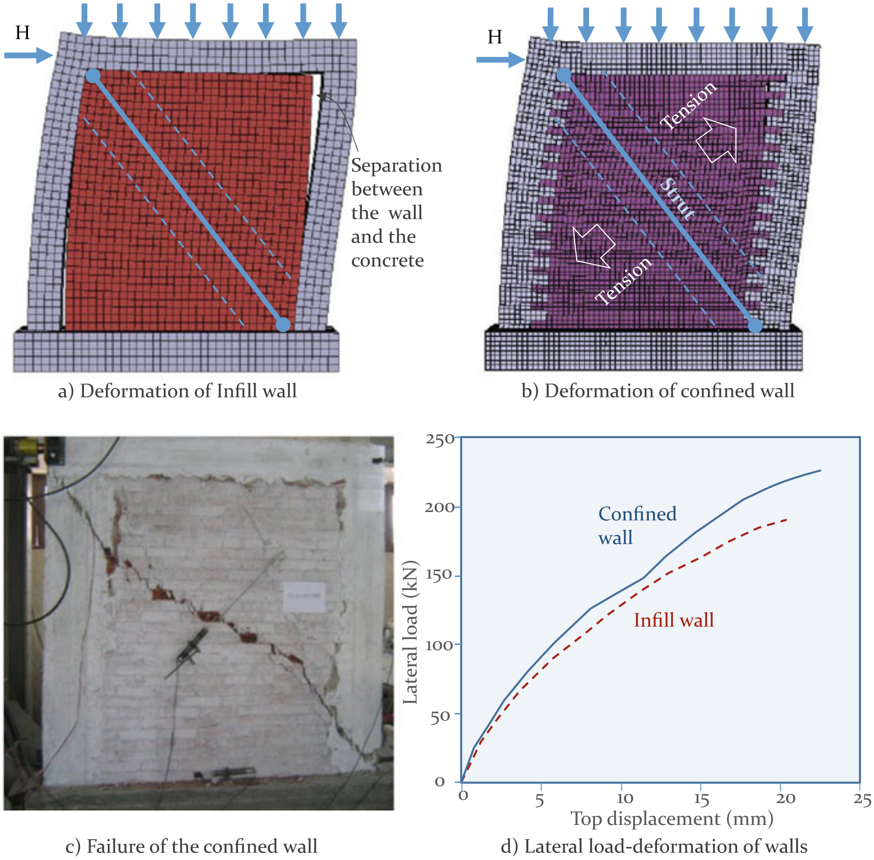

Fig. 6.15 Infill versus confined masonry walls (Mantawy et al. 2017)

In general, the deformational behavior and ultimate carrying capacity of the confined wall was better than that of the infill wall, as shown in Fig. 6.15d. It should be noted that the difference between the infill and confined walls is the type of connection between the wall and the concrete skeleton, where shear transfer all along the interface between the wall and the concrete elements should be granted.

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