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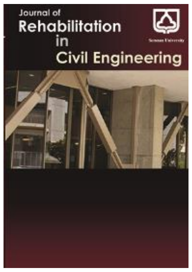

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Seismic Retrofit of a Historical Building in Tehran University Museum Using FRP Technology and Steel Jacketing

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ABSTRACT

In this paper evaluation of seismic capacity of a historical building is carried out. It had been used as a national library about 40 years ago. Also, before that, it had been constructed as a part of a Royal complex. This building has been constructed by traditional methods using bricks and mortars. The ceilings system of the structure consists of jack arches. Also, the roof of the building has been constructed with wood trusses. The building does not have any designed lateral bearing system. Lateral resistant of the building is due to masonry walls and due to the brittle and non-ductile performance of it. In secondary cycles of earthquake vibration, it will loose its stiffness and strengthening radically. Also, the building does not have any reliable about integrity. Load bearing systems are not reliable too. In this paper, the evaluation of the resistant capacity of the building is established, and the weak points of the system are distinguished. For some masonry columns some traditional method of steel jacketing is designed, and for increasing the reliability of the columns, FRP sheets are applied. In order to retrofits the walls, FRP sheets are installed on its surfaces; the externally bonded FRP sheets are strengthening the wall against in-plane and out of plane applied horizontal forces. The combination of the steel jacketing and externally bonded FRP sheets is prescribed to achieve an integrated system of the elements.

1. Introduction

In recent years, traditional methods for building design based on designing procedure for new building to obtain earthquake resistant structure has been reviewed and re-investigated.

Major differences in this review have been change the method of "strength" to method of "performance". Before, strength and performance had been used in same definition, but recently, it has been proved that increases of strength don't increase the safety of the structure and don't reduce damages.

The concepts and terminology of performance-based design are new and should be carefully studied and discussed with building owners before use. The terminology used for target Building Performance Levels is intended to represent goals of design. The actual ground motion will seldom be comparable to that specified in the Rehabilitation Objective, so in most events, designs targeted at various damage states may only determine relative performance. Even given a ground motion similar to that specified in the Rehabilitation Objective and used in design, variations from stated performance objectives should be expected and compliance should not be considered a guarantee of performance. Variations in actual performance could be associated with unknown geometry and member sizes in existing buildings, deterioration of materials, incomplete site data, variation of ground motion that can occur within a small area, and incomplete knowledge and simplifications related to modeling and analysis. Information on the expected reliability of achieving various target Building Performance Levels when the requirements are followed can be found in standards [1-4].

It is expected that most buildings rehabilitated in accordance with performance based design would perform within the desired levels when subjected to the design earthquakes. The practice of earthquake engineering is rapidly evolving, and both our understanding of the behavior of buildings subjected to strong earthquakes and our ability to predict this behavior are advancing. In the future, new knowledge and technology will provide more reliable methods of accomplishing these goals. New building codes are primarily intended to regulate the design and construction of new buildings; as such, they include many provisions that encourage the development of designs with features important for good seismic performance, including regular configuration, structural continuity, ductile detailing, and materials of appropriate quality. Many existing buildings were designed and constructed without these features and contain characteristics such as unfavorable configuration and poor detailing that preclude application of building code provisions for their seismic rehabilitation. Historically, criteria for evaluation have been set lower than those for design to minimize the need to strengthen buildings that would otherwise have only modest deficiencies. The expertise of the design professional in earthquake engineering is an important prerequisite for the appropriate use of retrofit standards in assisting a building owner to select voluntary seismic criteria or to design and analyze seismic rehabilitation projects [5,6].

Since the box section in Concrete Filed Tube(CFT) column Increase the confinement of concrete and final resistance of system, steel jackets used in this study caused confinement of masonry columns[7,8]. In CFT columns confined concrete in addition to increase the compressive strength f concrete, increases the ductility of column(See Figure1). Due to the rapid drop in the strength of SRC specimen, is the failure of the surrounding concrete. Also figure2 shows the increase the strength and ductility of the confined concrete in CFT column than the unconfined column.

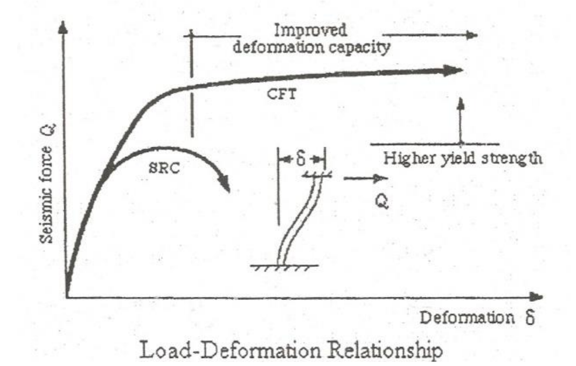


Figure1. Compare the CFT column with SRC column in seismic loading[9]

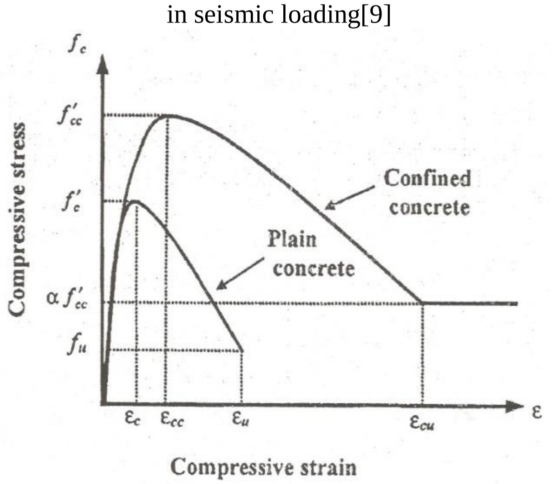


Figure2. The stress-strain curves for confined and unconfined concrete [9]

It follows from the above two curves the use of composite columns and create confinement for concrete by steel causes an increase in stress-strain properties of concrete and gives manifold increase The ultimate strain and ultimate displacement of concrete. In addition to increasing the strength and ductility properties too.

In Eunsoo Choi et. als. study considered the failure of the four tested columns and analyzed their later force-displacement behavior. Additionally, the effective stiffnesses of the forced is placement curves were evaluated. The GFRP wire winding jacket prevented splitting of the lap-spliced reinforcement in the lapspliced column and delayed buckling of the longitudinal reinforcement. The jacket protected the continuous reinforcement column against steel buckling and concrete spalling off and, thus, induced shear failure in the column. The GFRP wire winding jacket increased the failure drifts of both jacketed columns compared with those of the references[10]. Many researchers investigated how external confinement for reinforced concrete (RC) columns at plastic zone enhanced the flexural strength and ductility, and they showed that the external jackets protected the columns from severe seismic attacks [11,12]. Thus, new and effective jacketing materials or methods have been suggested continuously. During the last decade, a few seismic external jacketing methods for RC columns have been newly suggested. These include shape memory alloy (SMA) wire winding jackets [13,14], steel wrapping jackets [15,16], and FRP (fiber reinforced polymer) wire winding jackets [17], which have distinct characteristics compared with conventional jackets such as steel or FRP sheet jackets [18,19]. Their basic distinction is that there is no bond behavior between the jacketing materials and concrete. Conventional steel jackets need grout [20,21], and FRP sheet jackets requireapplying adhesive [22,23]; these attach them to the concrete surface.

In this paper evaluation of seismic capacity of a historical building is carried out. It had been used as a national library about 40 years ago. Also, before that, it had been constructed as a part of a Royal complex. In this paper the evaluation of resistant capacity of the building is established and the weak points of the system are distinguished.

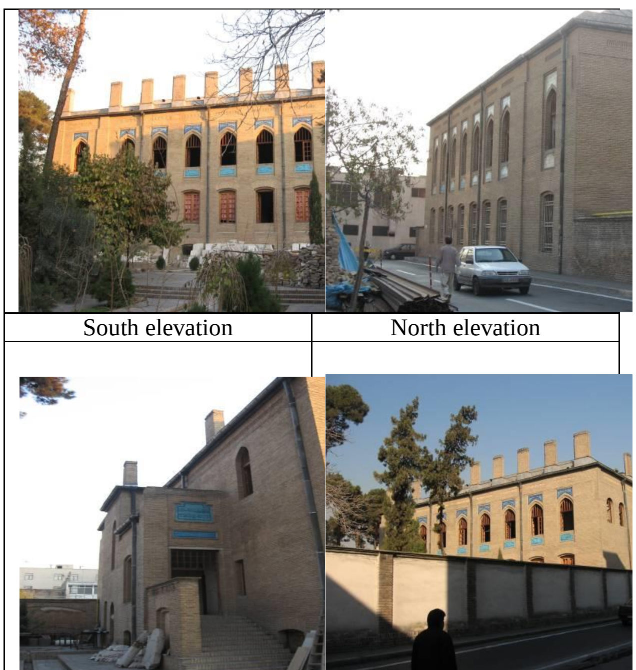
2. BUILDING DESCRIPTION

This building has been constructed by traditional methods using bricks and mortars. The ceilings system of the structure consists of jack arches. Also, the roof of the building has been constructed with wood trusses. The building does not have any designed lateral bearing system. Lateral resistant of the building is due to masonry walls and due to brittle and non-ductile performance of it. In secondary cycles of earthquake vibration, it will lose its stiffness and strengthening radically. Also the building does not have any reliable about integrity. Load bearing systems are not reliable too.

This building had been designed and constructed in 1928. All of the gravitas weight loads are carried out by bricks and mortars. Floors weights have been supported by jack arches. The structure of building roof is wood trusses that are covered by galvanized steel plates.

Table 1. Building specifications

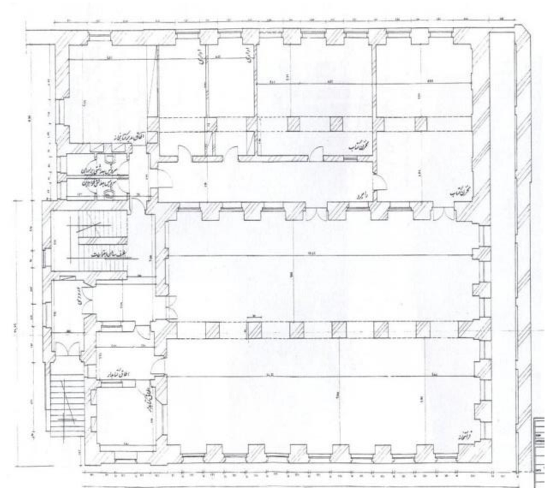
Structure	brick walls and clay mortar	
Plan Dimensions	27.75x22.75	m
Story number	3	m
Total plan surface	1895	m2
application	Library and amphitheaters	
Base level	-0.7	m
Story levels	5.15, 4.25, 3.55	m
Roof level	13.75	m
Foundation	Unknown	
Floors	Jack Arches + wood trusses	
Soil type	II	



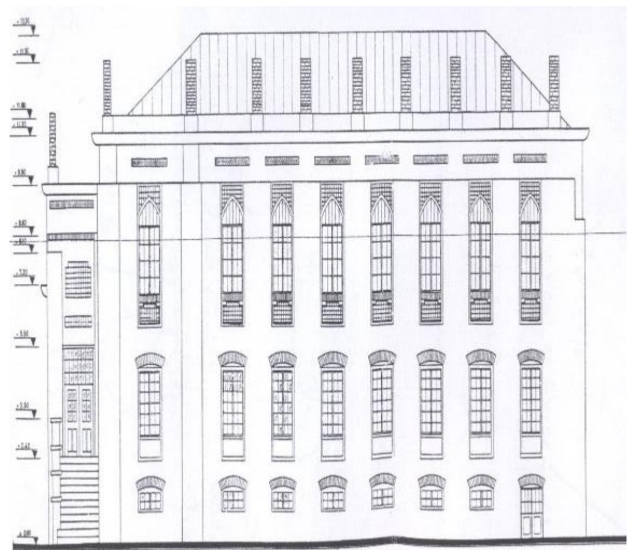
South elevation | North elevation

West elevation | East elevation

Figure 3. Building pictures



(a) Plan



(a) Evaluation

Figure 4. Plan and elevation of the building

3. DEFINITION OF REHABILITATION OBJECTIVE

The Rehabilitation Objective selected as a basis for design will determine, to a great extent, the cost and feasibility of any rehabilitation project, as well as the benefit to be obtained in terms of improved safety, reduction in property damage, and interruption of use in the event of future

earthquakes. Figure 3 indicates the range of Rehabilitation Objectives that may be used.
Operational Performance, Immediate Occupancy Performance, Life Safety Performance, Collapse Prevention Performance are levels of rehabilitation. For this building the Operational (1-A) with properties: Backup utility services maintain functions; very little damage, is selected because of its high important and historical.

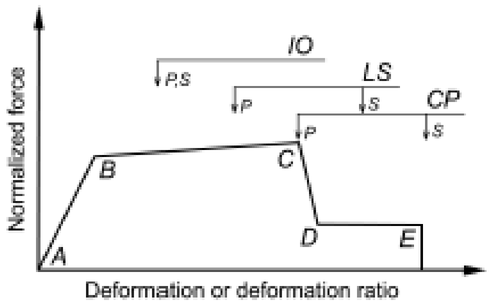


Figure 5. Force versus deformation ratio and rehabilitation levels

Risk analysis of earthquake hazards have been performed for local position of building and the results shows different values for maximum estimated Peak Ground Accelerations. The PGA for different return periods are given in Table 2. Spectral values for acceleration, displacement and velocity, F, are in two last columns of Table 2. It has been given by Eq. 1, in which, S_a, is Spectral Acceleration:

$$S_a = Fa.A \tag{1}$$

Spectral velocity and displacement can be obtained in same approach.

Table 2. Earthquake hazards- maximum acceleration, velocity and displacement due to earthquake

Return period (years)	A (m/s^2)	A (g)	V (cm/s)	D (cm)	F (d=2%)	F (d=5%)
225	2/57	0/262	0/241	0/136	2.739	2.116
475	3/49	0/356	0/327	0/184	2.026	1.650
2475	5/50	0/560	0/600	0/336	1.633	1.385

4. Definition of risk level

Level risk is equal to the percentage probability of an earthquake with a possible annual event in a time range(useful life of structure). According to the above definition, the relationship between the annual probability of an earthquake (p), return period TR and probability of earthquakes (q) over the life of the structure or n-year is calculated using the following formulas.

$$T_R = \frac{1}{1-(1-q)^{1/n}} \tag{2}$$

$$p = (1-q)^{1/n} \tag{3}$$

5. ANALYSIS PROCESS

All of Steels that use in this building are ST37, by minimum yielding stress $2400 kg/cm^2$ and $3700 kg/cm^2$ ultimate stress. Compressive strength of bricks is 50 kg/cm2 based on experimental tests and shear resistant of mortars have been obtained by averaging between 9 numbers of tests in each story. These values are displayed in Table 3. Slab details of each floor are given in Table 4. Density and weight of each component is measured and calculated. Summation of gravity loading of each square meter of floors is 610 kg.

Table 3. Experimental results of mortars test

Story	Shear strength of mortars(kg/cm^2)
Under ground	2.4
Ground	1.9
First	1.5

Table 4. Slab details

materials	thickness	density	weight
finishing	0.02	$2100\ Kg/M^3$	$0.02*2100=41\ Kg/M^2$
mortars	0.02	$2100\ Kg/M^3$	$0.02*2100=41\ Kg/M^2$
jack arch	0.23	$1850\ Kg/M^3$	$0.23*1850=425\ Kg/M^2$
beams	-	-	$100\ Kg/M^2$

Eq. 4 describes pseudo static method to calculate the base shear in terms of building parameters. This method is used in codes based on building weight.

V=C.W (4)

In which, V: base shear, W: building weight, C=f(A, B, I, R), A: intensity of acceleration, B: response factor, I: important factor, R: Behavior factor, these factors are different in various CODES, but C factor almost obtained the same value. Some of building codes assume that:

$$C = \frac{ABI}{R} \quad (5)$$

Where:

$$B = 2.5(\frac{T_0}{T})^{\frac{2}{3}} \quad (6)$$

$$T = 0.07H^{\frac{3}{4}} \quad (7)$$

$$T_0 = 0.5 \quad (8)$$

A=0.35 (9)

$$F_i = (V - F_t)\frac{W_i h_i}{\sum_{j=1}^{n} w_j h_j}, F_t = 0.07\,TV \quad (10)$$

Seismic specification of building due to existing codes is calculated in table 5. Period of structure is 0.35 seconds, and the base shear of building is 640 ton. Each story shear can be calculated by normal distributing model as a triangular distributing is presented in Table 6. Based on shear resistant of mortars and bricks, shear resistant of each walls of building can be calculated and after summation in each directions is presented in Table 7.

Table 5. Seismic specification

A	B	I	R	Te	T_0
0.35	2.5	1.2	4	0.35	0.1
T_S	C	W_{DL}	W_{LL}	We	V (ton)
0.5	0.26	2025	725	2460	640

Table 6. Base shear and story shears

Story	Dead load	Live load	surface	Wall Length	Dead W.	Live W.	Wall w.	Fi	Vi
First	300	150	631	156	189.3	94.65	288.6	249	249
Ground	610	500	631	210	384.91	315.5	388.5	269	518
Under G.	610	500	631	210	384.91	315.5	388.5	122	640
Sum.					959.12	725.65	1065.6	640	

Table 7. Shear resistant of walls

Story	South-North Wall length	East-West Wall length	Wall Width	Shear strength of mortars (kg/cm2)	S-N-wall Shear Resistant (ton)	E-W-wall Shear Resistant (ton)	Earth-quake Shear (ton)
First	75	81	1	1.5	1125	1215	249
Ground	75	135	1	1.9	1425	2565	518
Under G.	75	135	1	2.4	1800	3240	640

Based on FEMA codes the base shear, V, is calculated by Eq. 11:

$$V = 0.33AIW \quad (11)$$

Story shears, V_i, can be obtained by driving Vi and A_i, total section surface of story walls(with thickness over 200 mm), the sections should be considered without opening length and only walls by length greater than 1/6 summation length of openings or 2/3 height of smaller opening [24,25].

$$A_i = \frac{V_i}{V_a} \quad (12)$$

Allowable shear stress, V_a, can be calculated by Eq.13, in which maximum V_1 is 6 kg per square:

$$V_a = 0.1V_1 + 0.15\sigma_C \quad (13)$$

Based on this calculation each wall has a special specification that is presented in

Table 8. Base shear and story shear based on FEMA 356 is displayed in Table 9.

Table 8. Seismic specifications of walls

A	I	We	W_{DL}	W_{LL}	We	V(ton)
0.35	1.2	2460	2025	725	2460	341

Table 9. Base shear and story shears based on FEMA356

Story	Dead load	Live load	surface	Wall Length	Dead W.	Live W.	Wall w.	F_i	V_i
First	300	150	631	156	189	95	289	133	133
Ground	610	500	631	210	385	316	389	143	521
Under G.	610	500	631	210	385	316	389	65	341

Relative walls calculation shows in Table 10 indicates that building have a good ratio of this value and is safe about this criteria. Shear stress of walls is presented in Table 11 and the values of stress are less than allowable stress obtained by result of test.

Table 10. Relative walls calculations

Story	South-North Wall length	East-West Wall length	Wall Width	Story surface(m2)	S-N relative walls(m)	W-E relative walls(m)
First	75	81	1	631	0.12	0.13
Ground	75	135	1	631	0.12	0.21
Under G.	75	135	1	631	0.12	0.21

Table 11. Shear stress of walls (Kg/Cm^2)

Story	South-North Wall length	East-West Wall length	Wall Width	Vi(ton)	S-N Wall Shear Stress	W-E Wall Shear Stress
First	75	81	1	133	0.18	0.16
Ground	75	135	1	521	0.69	0.39
Under G.	75	135	1	341	0.45	0.25

Nonlinear static analysis has been performed to analyzing the structure of the system and the target displacement obtained by Eq. 14 as below [25]:

$$\delta_t = C_0\, C_1\, C_2\, C_3\, S_a \frac{T_e^2}{4\pi^2} g \tag{14}$$

Coefficients are presented in FEMA356 Codes [26-28]. Building performance is a combination of the performance of both structural and nonstructural components. On average, the expected damage would be less. For comparative purposes, the estimated performance of a new building subjected to the 2475 years return period level of shaking is indicated.

The acceptability of force and deformation actions shall be evaluated for each component in accordance with the requirements of Section 3.4 of FEMA. Each action shall be classified as deformation-controlled (ductile) or force-controlled (non-ductile. The rehabilitated building shall be provided with at least one continuous load path to transfer seismic forces, induced by ground motion in any direction, from the point of application to the final point of resistance.

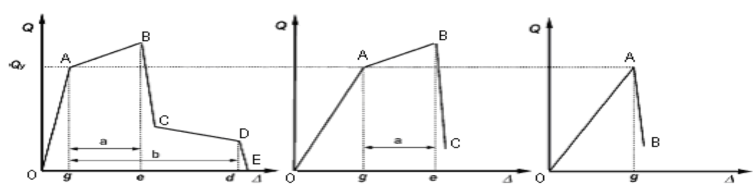


Figure 6. Acceptance criteria for various structures

	
Arches and walls	Brick foundation

Figure 7. Foundation and walls detail

Figure 8. Rehabilitated slabs (left hand), walls and columns with FRP (green lines in right hand)

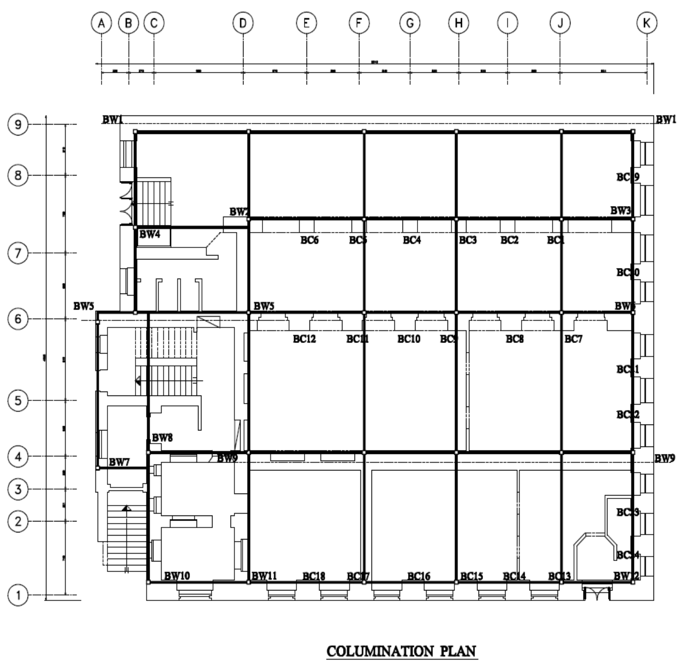


Figure 9. Vertical and horizontal tie beams

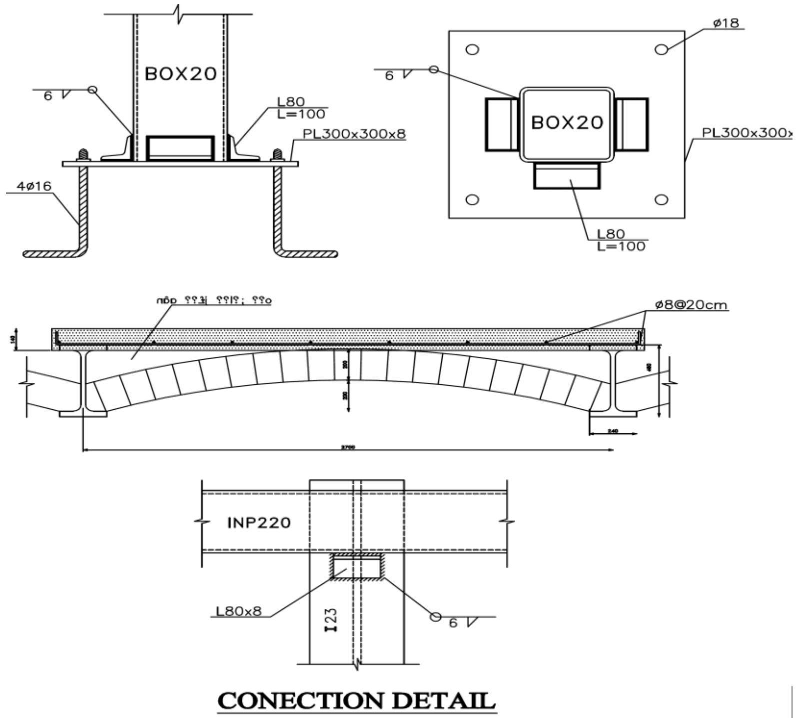


Figure 10. Connection detail of ties

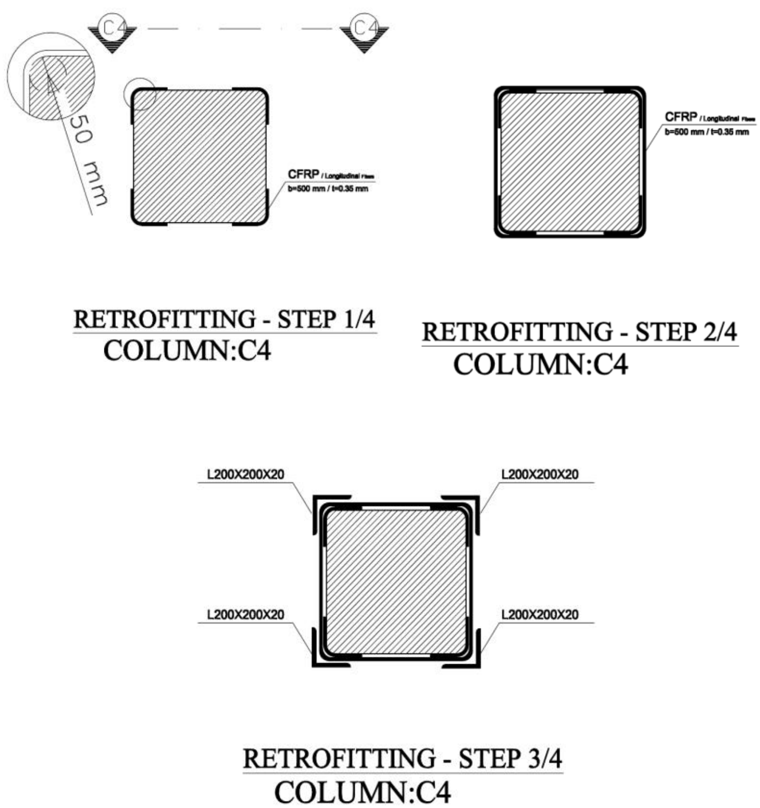


Figure 11. Detail for retrofit of columns

6.REHABILITATION STRATEGY

This building has some problems that are listed below and can be seen in Figure 5. Structural Irregularity and component Consistency of the building has poor. The structural system needs more consistency system to make a good resistant and integrative structure. The building has not any Secondary system to be safe in shakes. Foundation of system is very poor and don't has tie beams between foots.

Height of walls in this building is very tall and it needs to be tied at semi level of height. Also, free length of walls is too long and needs to be established supporting by tie columns as be designed. Distance between opening and wall edges is too small; it should be retrofitted by appropriate approach. Tie beams over walls need to give integrity between walls and roofs. Floors slabs and its stiffness are not complete and it should be braced in horizontal plane. For improving this properties it should be considered that the path of loads will be completed are better material quality. Using better quality of materials could be helping the building to be safe in resistant of loads. Retrofit strategy for this historical building should be special because the façade of building should be kept as well as existing. To obtain these goals this approach has been designed:

a- Retrofit of foundation
b- Steel jacketing of columns(Figure 12) after FRP confined
c- FRP confinement of walls and columns (Figure 8)
d- Tie beams, vertically and horizontally (Figure 9)
e- Retrofit of opening by FRP sheets in around
f- Make a rigid slab with horizontal bracing and added concrete slab (Figure 8).
g- Bracing the short towers on the roof.
h- Change the roof system by steel truss.

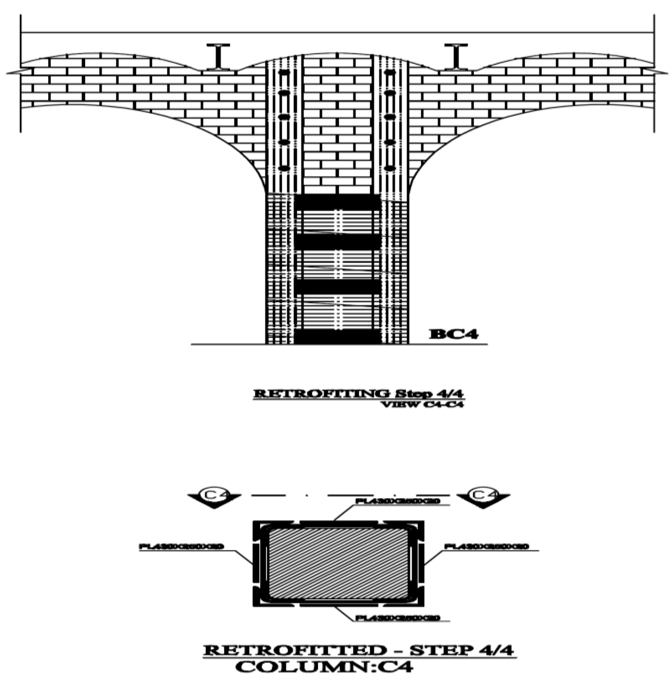


Figure 12. Elevation of columns and sections

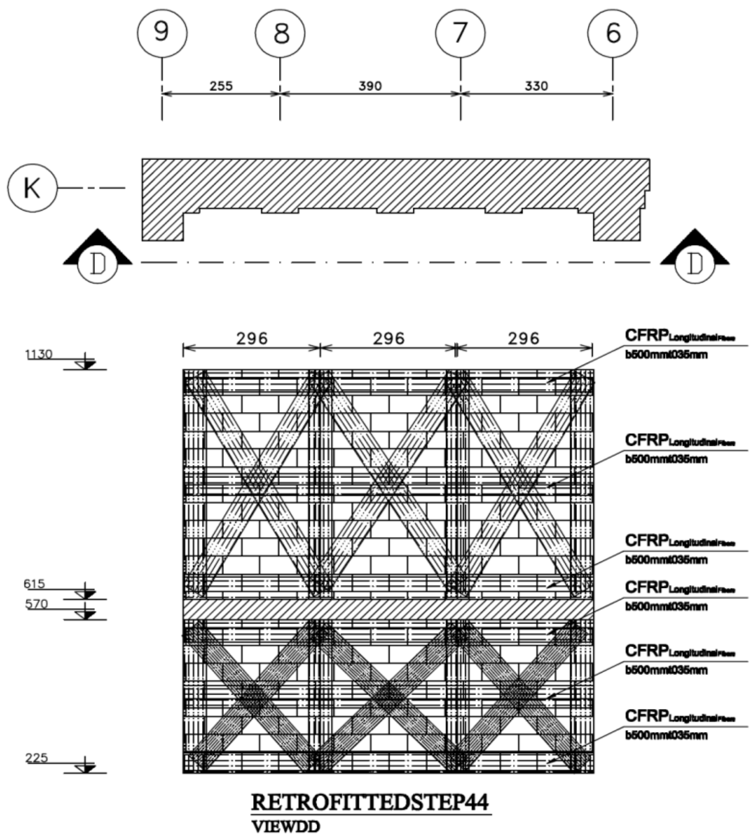
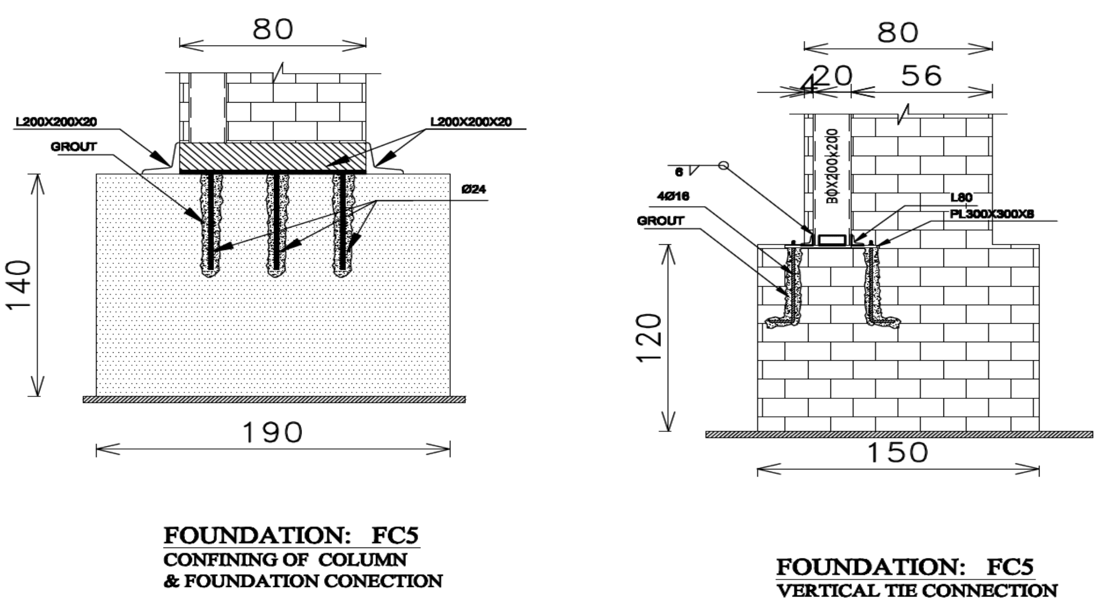


Figure 13. FRP bracing on walls



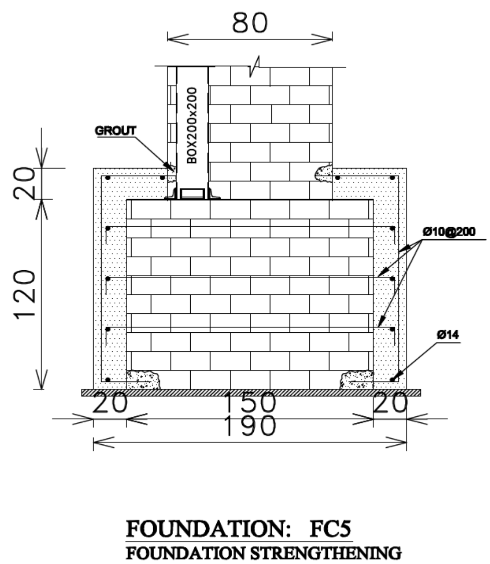


Figure 14. Foundation confinement

Table 15. Result of analysis

Wall-story	Wall shear (Ton)	Total Wall shear(Ton)	Wall length(m)	Wall thickness (m)	Shear stress (kg/cm^2)	Allowable Shear stress (kg/cm^2)
E-W-STORY 3	1.41	1.41	7.40	0.90	0.002	1.5
E-W-STORY 2	2.28	3.69	7.40	0.90	0.055	1.9
E-W-STORY 1	1.48	5.17	7.40	0.90	0.077	2.4
N-S- STORY 3	0.50	0.50	6.10	0.90	0.009	1.5
N-S- STORY 2	2.00	2.50	6.10	0.90	0.046	1.9
N-S- STORY 1	2.50	5.00	6.10	0.90	0.091	2.4

Resistant of brick-mortars walls will be reduced after first cycles of earthquake and after initial cracks, so the FRP sheets can be reduced the walls drifts and make a higher integrity after first cycles of shakes. This FRP sheets that are used in this sketched have below specifications:

F_y=3000 MPa , E= 600 GPa, thickness= 0.35 mm, Ultimate strain of FRP=0.04,
Maximum strain of FRP under loading=0.04*816.4/3000, Maximum allowable drift =0.01, Maximum-lateral drift(angle=60deg.)=0.005.
So the FRP sheets can cover all supporting forces to improving the wall specifications to reduce drifts and make a good safety up to don’t appears any cracks for complete the structure as a good special rehabilitated building under high level of earthquake vibration. Results in analysis show that stress after rehabilitation is reduced in walls, and integrity of the system leads it to be a complete system with excellent distributing of lateral forces. Figure 15 is modal deformation of structure that improved and gets higher value in compare with existing building. Figure 16 shows that stresses is reduced by this FRP bounding up to 50 percent, this strategy of rehabilitation constrains the drifts and cracks during shakes, and this purpose is very necessary for this historical building. FRP sheets and Steel jacketing make a more stiffened and ductile resistant system.

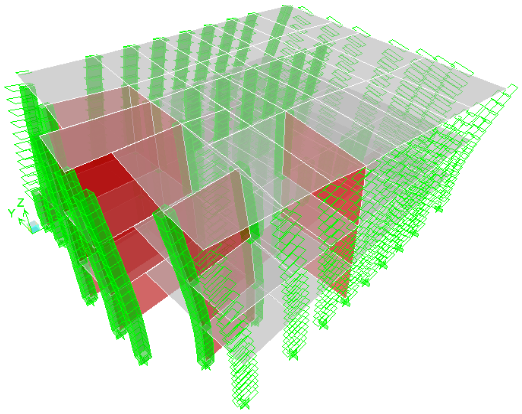


Figure 15. Modal deformation of structure

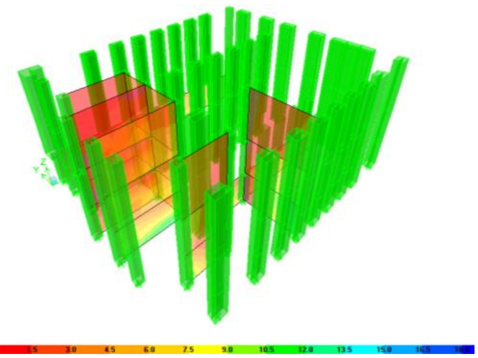


Figure 16. Stresses result of analysis under lateral loading

7. CONCLUSIONS

This building has a special approach for retrofitting because of its special important and the constraint of keep the façade of building. Due to its old structural system, only gravity loads could be supported by walls and system has not any lateral resistant system to earthquake shakes. Some advanced and custom method has been used together for make a consistent resistant system for the building. As it can be seen in the nonlinear analysis the structure of building that is retrofitted analyzed and has not any plastic hinges under 2475 years return period earthquake. So the building expected will be stable and without any structural cracks due to the earthquake with return period of 2475 years.

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