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Investigation of Improvement Methods in Existing Defective Building Stocks

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Abstract

The use of nonlinear analysis methods in strengthening structures has become increasingly important in recent years. In the first part of this thesis, major strengthening methods are outlined without going into detail. Furthermore, a model building with fire damage was modeled with ETABS and a method was proposed. In addition, the building was analyzed in detail using the finite element model using SAP2000. Shear walls used for reinforcement with modal analysis and push analysis were designed. Approaches are needed to increase structural safety of structural members after fire. Fiber reinforced polymer (FRP) is increasinglyused in civil engineering applications regarding its excellent strength/weight ratio and anti-corrosion ability. Existing experimental research on the performance of fire-damaged RC elements repaired with concrete coating, steel coating and fiber-reinforced polymer (FRP) coating has been reviewed.

Keywords: *Pushover Analysis, Modal Analysis, Shear Wall, Reinforcement, Seismic Performance.*

1. Introduction

Construction in Afghanistan is very different from construction in the US and offers many challenges to maintain good quality. The availability (or lack) of materials and equipment determines most construction methods. Outside major cities, building designs rarely had any impact on earthquake forces historically, but old methods were followed without any modern building code effect. There are no special building codes for Afghanistan, foreign laws and standards are used for the construction of buildings, roads, highways and bridges. This prevented the private sector from investing in buildings in Afghanistan without any official code. This policy will lay the groundwork for the future development of rules, regulations and building codes in the city of Kabul.

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1.1. General Finite Element Analysis of The Building

In the last decade, the computer application for structural analysis has been greatly developed. The Finite element method has been proved to be very accurate for static and dynamic analysis for the different types of structural modeling. Two Finite element and matrix models have been used for the Building analysis. In these models the structural components of Building are considered as three-dimensional frame element. The Building system is idealized and modeled using three-dimensional frame element with 6 degrees of freedom for each node for quick check of main displacement and stress control. The geometric characteristics of the Building and its various elementsare modeled and determined on the basis of the planned geometric layout. The geometrical and inertial properties foreach member of the Building system are computed as a whole and analysis were done. In this study SAP2000 and ETABS is used as finite element program and matrix. The elements that used in modeling the Building are three-dimensional Frame Elements which represents column and beam elements together with shell element that representthe floor slab and shear walls. Detailed descriptions of these elements are discussed in the following sections. Three-dimensional frame element is a two-node prismatic linear element based on three-dimensional beam-column formulations. It has a twelve-degree of freedoms and it is capable of resisting axial forces, shear forces and bending moments about the two principal axes in the plane of its cross section, and twisting moments about its longitudinal axis. The material properties of the beam-column element are the elastic modulus E, and shear modulus G. The geometrical properties of the element are defined locally and included cross sectional area A_x, shear Areas A_y, A_z, moments of inertia I_{yy}, I_{zz} and torsional moment of inertia J_r. The local coordinate system used for frame element is presented in Figures.

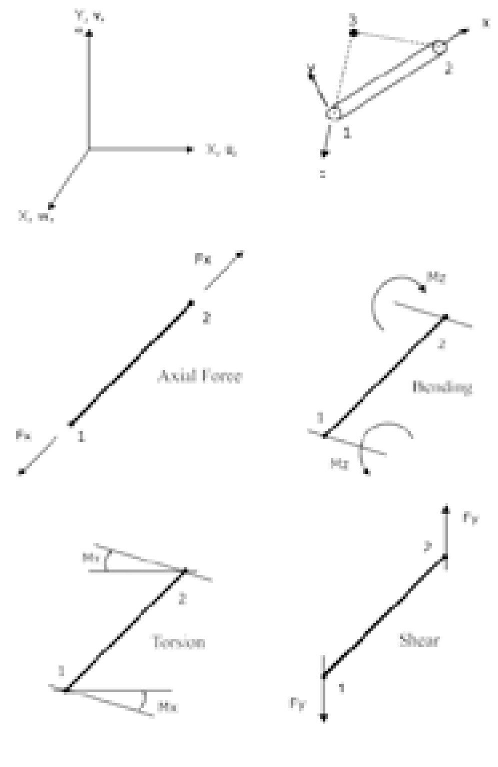


Figure 1: Three dimensional frame element with 6 degrees of freedom

Table 1: The element local forces in the local coordinates.

NAME	*EXPLANATION*	*UNIT*
Fx :	Axial Force (kN)	(kN)
Fy :	Shear Force in y-direction (kN)	(kN)
Fz :	Shear Force in z-direction (kN)	(kN)
Mx :	Torsion (kN.m)	(kN.m)
My :	Moment in xy-plane (kN.m)	(kN.m)
Mz :	Moment in yz- plane (kN.m)	(kN.m)

Sign convention for the frame element used in the analysis can be summarized as:

Table 2: Sign convention of beam element

Sign convention of beam element		
Axial Force	(positive)	Axial Tension
	(negative	Axial compression
Bending moment	(positive)	Hogging moment
	(negative)	Sagging moment
Torsion	(positive)	Anti-clockwise rotation (1st node), clockwiserotation (3rd node)
	(negative)	Clockwise rotation (1st node), anti-clockwiserotation (3rd node)

In the full Building model all elements are modeled as frame elements. All Building elements are made of concrete which its related properties are given in the following tables.

Table 3: Elastic Material Properties

Building Part	**Elasticity Modulus, E, kN/m²**	**Unit Weight, ρ (Ton/m3**	**Poisson Ratio, ν**
All Rebars	$210*10^6$	8.0	0.3
Concrete 116	$25*10^6$	2.4	0.2
Concere 250	$25*10^6$	2.4	0.2

To achieve more accurate results, the structural components of the Building have been modeled as beam elements. This model is used for the static, dynamic, natural frequency analysis. The geometric characteristics of the Building and its various elements are modeled and determined on the basis of the planned geometric layout. The support endsof the Building are assumed to be simple supported. They are generally located as contact face of wheels. The finite element discretization includes 2000 nodes, 3000 frame and 3000 shell elements.

1.2. Definitions of The Applied Loads on The Building System

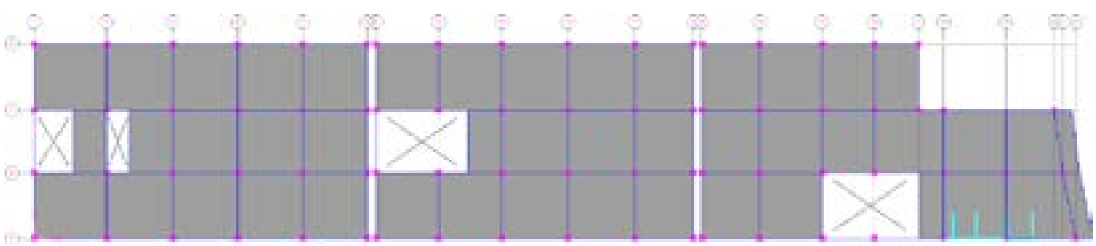
The Building system is exposed to different type of applied external and internal loads. The loads acting on the Building in its operating and non-operating conditions include the loads due to the dead weight, the wind load, and the dynamic loads caused by earthquake. At the end of this section the general views of each load cases are given.

The dead load is calculated by program automatically by using the cross sectional properties as area multiplied by length and the density of concrete and adde load as the 170 Kg/m^2. The live load is computed by assuming total loadsof slabs and the Live load is also as taken of 500 kg/m^2. Buildings and other structures, including the Main Wind-

Force Resisting System (MWFRS) and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein. The wind load to be used in the design of the MWFRS for an enclosed or partiallyenclosed building or other structure shall not be less than 10 lb/ft2 (0.48 kN/m^2) multiplied by the area of the buildingor structure projected onto a vertical plane normal to the assumed wind direction [1]. The design wind force for open buildings and other structures shall be not less than 10 lb/ft^2 (0.48 kN/m^2) multiplied by the area Af.

$$p = q(GC_f q_i(GC_{pi})\ (\mathrm{lb/ft^2})\ (\mathrm{N/m^2}) \tag{1}$$

Design and wind pressures for all buildings with h > 60 ft (18.3 m) will be determined from the equation above. According to the seismic map of a report from USGS and USAID, “preliminary earthquake hazard map of Afghanistan”, Kabul has a 2% chance in 50 years of exceeding a peak ground acceleration of 50% gravity respectively, and a 10% chance in 50 years of exceeding a peak ground acceleration of 27% gravity respectively. The designers considered 0.3g for base ground acceleration because this construction is not included in special constructions for %2chance in 50 years. Response spectrum analysis seeks to estimate the maximum response of the structure under earthquake excitation without recourse to direct integration of the model over the complete duration of the earthquakeand scaled to 1g. Response spectrum analysis assumes that the structure behaves linearly. The aim of the investigation conducted herein is to find out the initiation of maximum stress zones in the Building.



(a)

Mode	Period (sec)	Ux	Uy
1	1.93	0.0033	39.8455
2	1.77	39.3769	0.0088

Maximum Diaphragm CM Displacement & Rotation	Value
Ux (cm)	69.9
Uy (cm)	79.8
Rz (Radian)	0.012

Drift	Value
X direction	0.03
Y direction	0.04

(b)

Figure 2: (a) Existing structure (b) Existing structure analysis

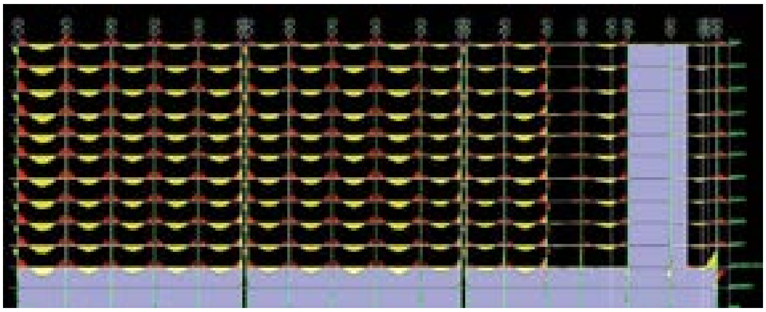


Figure 3: Moment 3-3 diagram

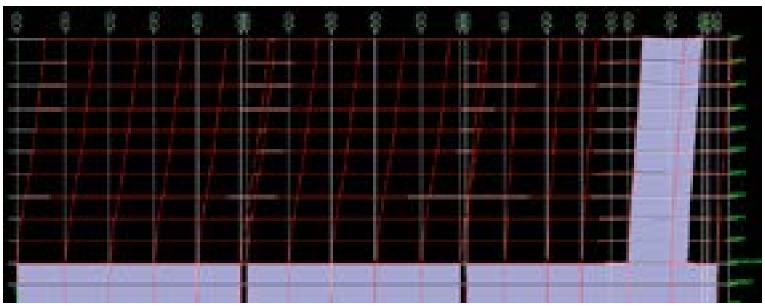


Figure 4: Deformed shape for spectral loading

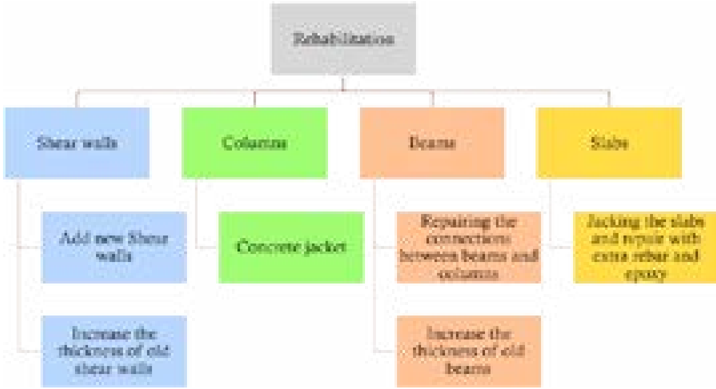


Figure 5: Rehabilitated

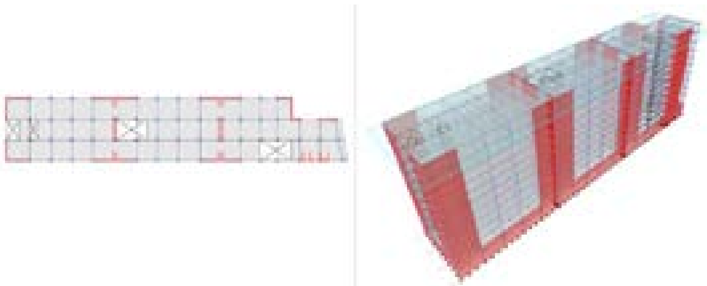
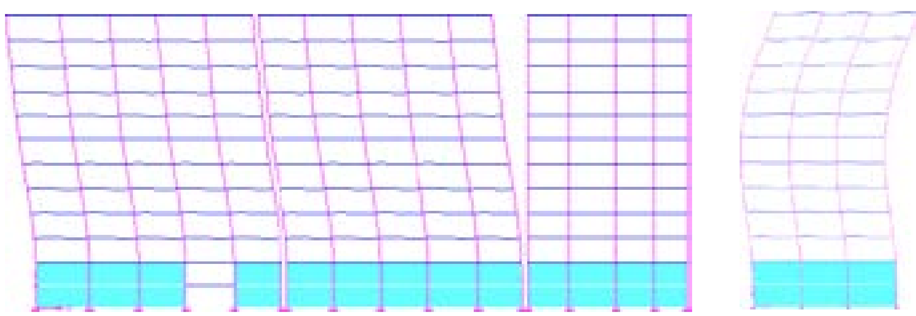


Figure 6: Rehabilitated structure

1.3. Modal Analysis of Building

For detailed consideration, a few mode shapes are shown in 3D and plan views and a little explanation is worth mentioning. Due to to the special design of this Building mode shapes are rather different than the ordinary building.



(a) (b)

Figure 7: (a) Mode 2, Period 1.78 sec (b) Mode 7, Period 0.7 sec

Figure 8: Center of mass displacement for diaphragm D3

1. Materials and Methods

For modeling and analysis of this construction, ETABS software was used. According to test of available construction, specified concrete compressive strength 11.6 Mpa and modulus of elasticity 2551461 has been modeled and all typesof shear walls and columns and beams drawn in the software. For this project we have 3 types of loading as: Dead load, Live load, Earthquake. According to ACI318 all of the load combinations have been known for software. Of course, earthquake loading modeled for X direction and Y direction and for both of them ±5% eccentricity, because this construction may be having torsion side force. Therefore, has been analysis.

Assignment

There are many cracks in the columns and beams, so according to ACI 318 crack must be considered for components in the property modifier moment of inertia around X and Y. It should be noted that the main structural frame is ordinarymoment frame, therefore according to FEMA 360 $R_u = 3$.

$$C = \frac{ABI}{R_u} = \frac{0.35 \times (N \times B_1) \times 1.2}{3} = \frac{0.35 \times 1.087 \times 1.73 \times 1.2}{3} = 0.2632$$

$$N = \frac{0.7}{4 \quad T_s}(T - T_s) + 1$$

Soil type is III: $T_0 = 0.15$, $T_s = 0.7$, $S = 1.75$, $S_0 = 1.1$, $T = 0.05 \times (H)^{0.9} = 1.1122$ Sec

$$B_1 = (S + 1)\left(\frac{T_s}{T}\right)$$

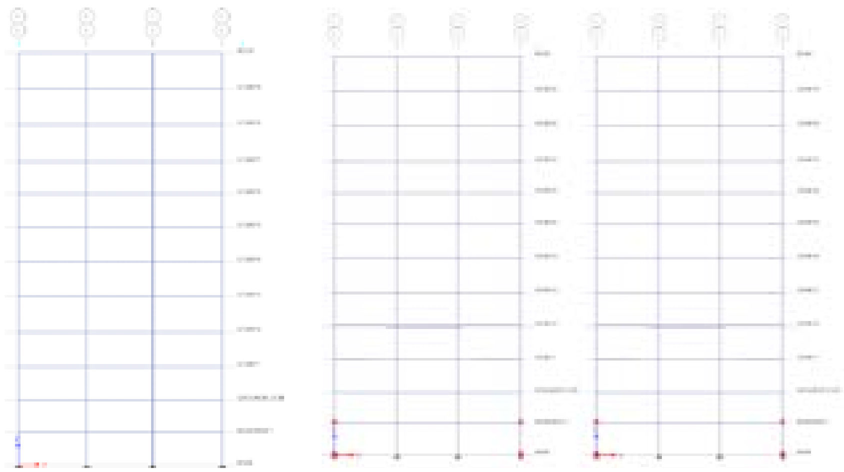
Dead load is 170 Kg/m^2 and Live load is 500 Kg/m^2.

1.1. Pushover Analysis

When analyzing frame objects, material nonlinearity is assigned to discrete hinge locations where plastic rotation occurs according to FEMA-356 or another set of code-based or user-defined criteria [2]. Strength drop, displacementcontrol, and all other nonlinear software features, including link assignment, P-Delta effect, and staged construction,are available during static-pushover analysis. After adding shearwalls to structure and changing old profile to new according to below:

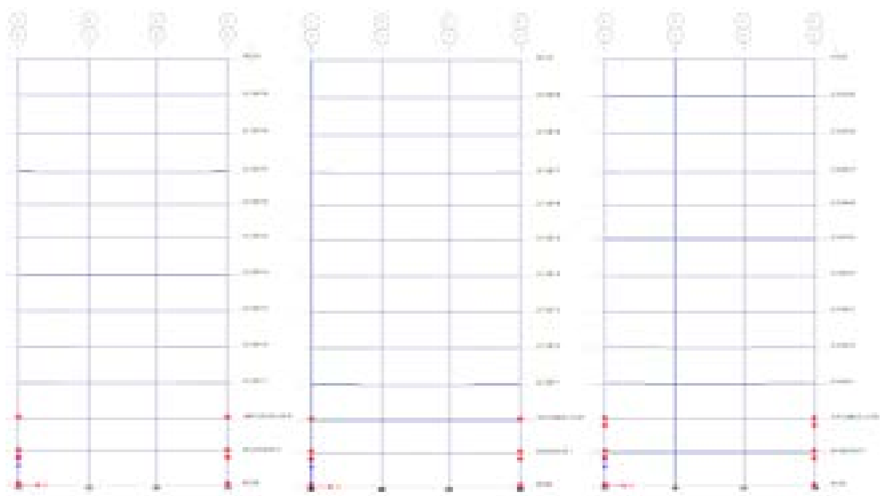
$$\delta_t = C_0 . C_1 . C_2 . C_3 . S_a . \frac{T_e^2}{4\pi^2} . g \quad (5)$$

δ_t = 0.2 m, Sample Column: IO = 0.005, LS = 0.015, CP = 0.02, Sample Beam: IO = 0.01, LS = 0.02, CP = 0.025



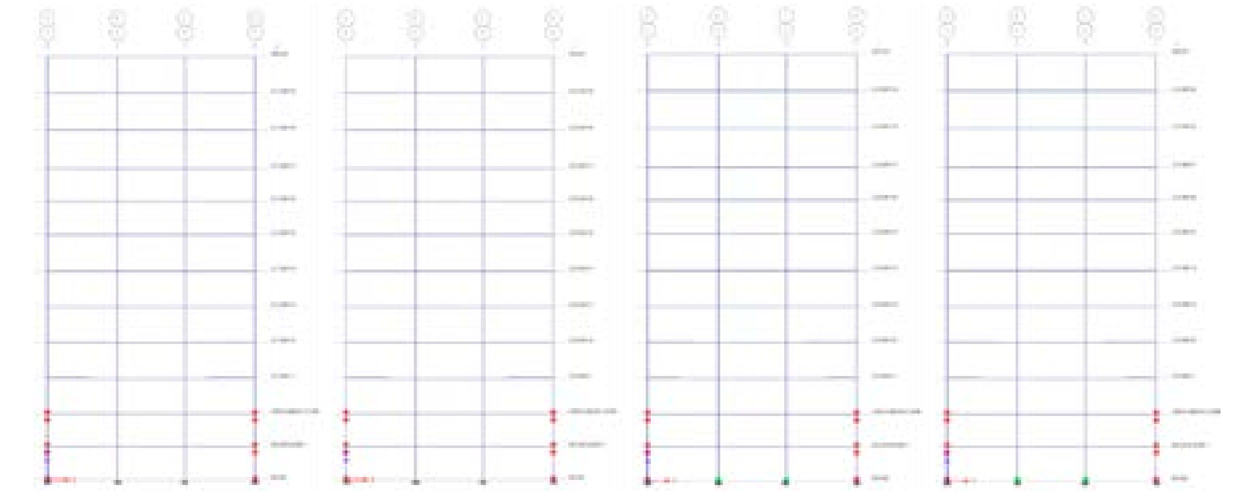
(a) (b) (c)

Figure 9: (a) Step 0 (b) Step 1, Fx=620000 KN (c) Step 2, Fx=910000 KN



(a) (b (c)

Figure 10: (a) Step 3, Fx=1600000 KN (b) Step 4, Fx=2304000 KN (c) Step 5, Fx=3100000 KN



(a) (b) (c)

Figure 11: (a) Step 6, Fx=3962000 KN (b) Step 7, Fx=4888000 KN (c) Step 8, Fx=6000000 KN (d) Step 9,Fx=6206000 KN

2. Conclusion and Suggestions

In the control calculations, the characteristic concrete compressive strength is 8 MPa, and the characteristic yield strength 220 Mpa for longitudinal and transverse reinforcement is taken. Since it is located in the second degree earthquake zone, T_a = 0.15sec, T_b = 0.40 secs were used as ground characteristic periods for A_0 = 0.30 and local ground class Z3. The calculations are based on cracked section stiffnesses. The information level coefficient was taken as 0.90. The building in question was modeled in three dimensions with the existing material quality and ground data and subjected to linear performance analysis.

Comparison: When two plastic joints are created in a column, the column will fail. According to the last slides, the first column in the 3th step will fail, therefore we must compare the equivalent force at this step and maximum base reactionin X direction.

Step 3, Fx=1600000 KN & ELX load combination, ETABS: Fx=172881 KN, IDECAD: Fx=180004 KN

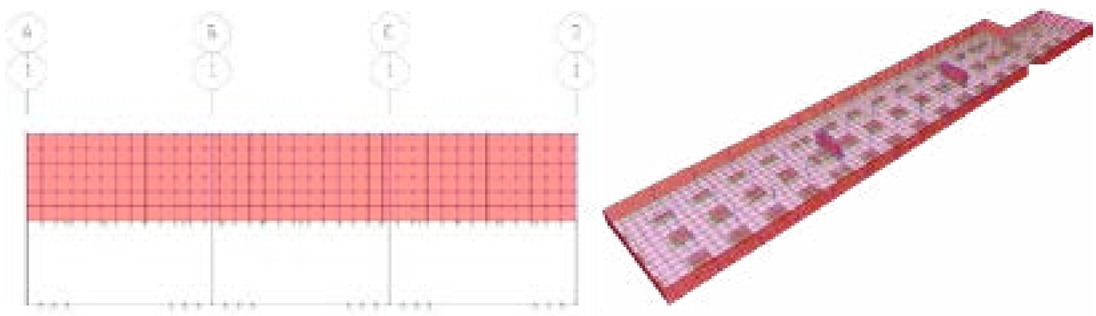


Figure 12: 3D model SAP2000 (Foundation)

As can be seen, some of the columns and beams are below the expected performance, and it is seen that there is a considerable insufficiency in terms of earthquake performance besides the axial force deficiencies in the structure. Various strengthening options have been tried to overcome this situation. According to the preliminary examination calculations, shear wall should be added in both directions in all 3 sections.

Considering the concrete quality, it will be more convenient to coat the columns with FRP. In addition, it should be reinforced with FRP in beams damaged by fire. Compared with steel jacketing and concrete section enlargement, the advantages of FRP jacketing are ease of use, lightness, anti-corrosion and high strength, stiffness / weight ratio. Meanwhile, compared to EBR jacketing, the advantages of FRP jacketing such as better fixation and ductile performance are more obvious [3, 4].

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