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Seismic Vulnerability Assessment of Rizgary Hospital Building In Erbil City, the Capital City of KR of Iraq

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ABSTRACT

Collapsing building structures during recent earthquakes, especially in Northern and Eastern Kurdistan, including the 2003 earthquake in Cewlig; the 2011 earthquake in Van; and the 2017 earthquake near Halabja province, has raised several concerns about the safety of pre-seismic code buildings and emergency facilities in Erbil city. The seismic vulnerability assessment of the hospital buildings as emergency facilities is one of the necessities which have a critical role in the recovery period following earthquakes. This research aims to study in detail and to extend the present knowledge about the seismic vulnerability of the Rizgary public hospital building in Erbil city, which was constructed before releasing the seismic provisions in the region. ETABS software is employed to conduct Eigenvalue analyses, nonlinear static analyses, and about 120 incremental dynamic analyses; furthermore, the actual response of the hospital building is evaluated by considering possible irregularities in both directions and the effect of seismic pounding. The outcomes of the research indicate that the hospital building is in poor performance under anticipated earthquakes. In addition, the existing combination of irregularities and seismic pounding in the model increases the vulnerability under the seismic load. A suitable strengthening strategy is also recommended.

Keywords: IDA, Pushover, Rizgary hospital, Vulnerability Assessment

تقييم الضعف الزلزالي لمبنى مستشفى رزكاري في مدينة أربيل، عاصمة إقليم كردستان العراق

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الخلاصة

انهيار هياكل المباني خلال الزلازل الأخيرة ، وخاصة في شمال وشرق كردستان ، والتي تشمل زلزال 2003 في بنجول ؛ زلزال 2011 في وان ؛ وزلزال 2017 بالقرب من محافظة حلبجة ، أثار العديد من المخاوف بشأن سلامة المباني القديمة ومرافق

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الطوارئ في مدينة أربيل. يعد تقييم الضعف الزلزالي لمباني المستشفى كمرافق للطوارئ أحد الضروريات التي لها دور حاسم في فترة التعافي بعد الزلازل. يهدف هذا البحث إلى الدراسة التفصيلية وتوسيع نطاق المعرفة الحالية حول الضعف الزلزالي لمبنى مستشفى Rizgary العام في مدينة أربيل ، والذي تم إنشاؤه بدون اتخاذ احتياطات التصميم لمقاومة الزلازل. يتم استخدام برنامج ETABS لإجراء تحليلات Eigenvalue والتحليلات الاستاتيكية الغير مرنة وحوالي 120 تحليلًا الديناميكية متزايدة الشدة ، بالاضافة يتم تقييم الاستجابة الفعلية لمبنى المستشفى من خلال النظر في المخالفات المحتملة في كلا الاتجاهين وكذلك تأثير القصف الزلزالي. تشير نتائج البحث إلى أن أداء مبنى المستشفى ضعيف في ظل الزلازل المتوقعة ، بالإضافة إلى أن المزيج الحالي من المخالفات والقصف الزلزالي في النموذج يزيد من الضعف تحت الحمل الزلزالي. يوصى باستخدام استراتيجية تقوية مناسبة.

الكلمات الرئيسية :الديناميكية متزايدة الشدة، الاستاتيكية الغير مرنة، مستشفى رزكاري_اربيل، تقيم تأثير الزلازل على المباني.

1. INTRODUCTION

The province of Erbil has experienced minor to moderate earthquakes since it is situated at the northern corner of the Arabian plate. Recent studies have asserted that the Peak Ground Acceleration (PGA) in Erbil city has been updated, particularly after the last cyclic earthquakes in the Arabian plate **(Abduljaleel and Taha, 2019)**. Furthermore, the collapse of RC construction systems, particularly in northern and eastern Kurdistan, after the latest earthquakes (e.g., 2003 earthquakes in Cewlig; 2011 earthquakes in Van; and 2017 Iran-Iraq country's border earthquake near Halabja province) has raised several concerns about the safety of pre-seismic code buildings and emergency facilities in Erbil city. Buildings built before the seismic code can experience a significant risk of damage, so their vulnerability should be carefully evaluated. Since pre-code buildings are constructed before releasing seismic provisions, they typically have low levels of strength and ductility **(ASCE/SEI 41-13, 2014; ATC, 1996; FEMA-356, 2000)**. In addition, the infrastructure of emergency services must operate properly in the aftermath of a major earthquake in order to enable emergency response. Emergency facilities have received a lot of attention around the world. For example, the United States has taken action in this direction by assessing the vulnerability of emergency facilities and then retrofitting them in order to be completely operational in the event of an earthquake **(Bruneau and Reinhorn, 2007)**. Moreover, the United Arab Emirates has also placed a high priority on emergency facilities to ensure their preparation and sustained service in the event of an earthquake, although being built to modern seismic design provisions **(Issa and Mwafy, 2014)**. Recent researches have highlighted the importance of evaluating the vulnerability of pre-seismic code buildings and emergency facilities and the key need to reduce their seismic losses **(Bruneau and Reinhorn, 2007; Ray-Chaudhuri and Shinozuka, 2010; Bhuiyan, Pal and Mazumder, 2015; Issa and Mwafy, 2014; Bilgin, 2016; Bilgin and Frangu, 2017)**. Therefore, this study focuses on evaluating the seismic vulnerability of Rizgary hospital or four hundred bedded general hospital; as shown in **Fig 1**, that is the oldest and largest public hospital in Erbil city, where a similar project was built in the different regions of Iraq in the 1980s before issuing Iraqi seismic code in 1988.

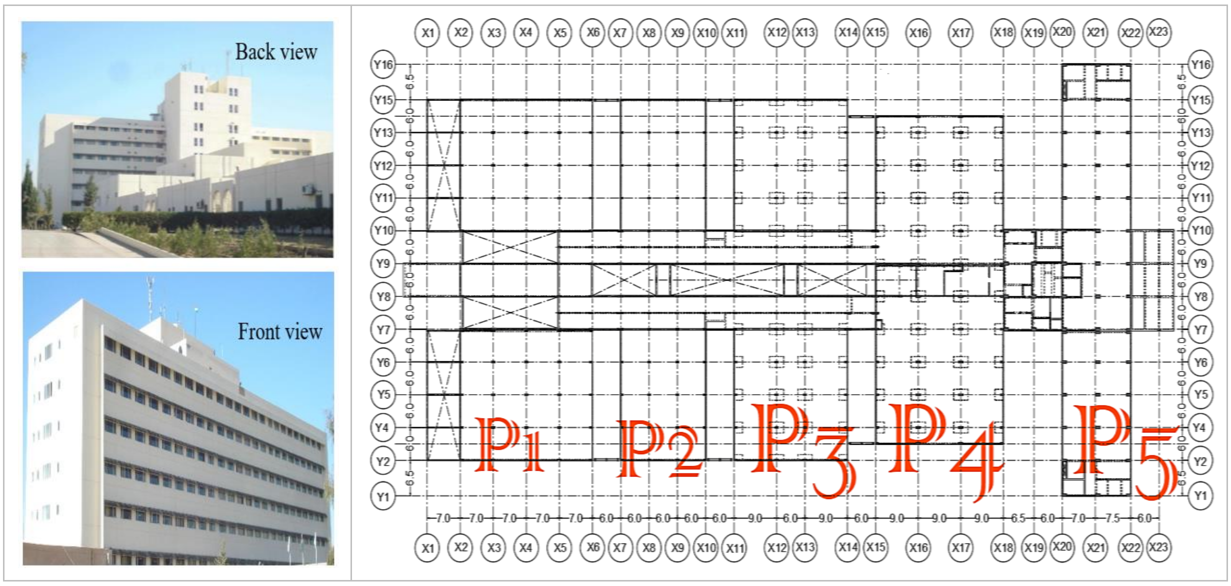


Figure 1. Photography (left) and layout (right) of Rizgary hospital in Erbil city.

The main objectives of the current study are determining the seismic vulnerability of the hospital building by performing inelastic pushover analysis and incremental dynamic analysis, in addition, evaluating the actual response of the hospital building by considering possible irregularities in both directions as well as the effect of seismic pounding. The proper recommendations can be provided in order to sustain the hospital building during expected earthquakes. For this purpose, the following information and data are required; original member sizes and reinforcement at the structural plan of the hospital building, reviewing the seismic characteristics of the region and selection set of ground-motion, and modeling finite-based model and performing analyses.

2. LITERATURE REVIEW

In the aftermath of a severe earthquake, emergency facilities such as hospitals, fire stations, police stations, and schools are essential. Critical facilities must keep functioning in a severe earthquake to manage the emergency evacuation and provide urgent medical attention to injured people. For critical facilities, the structural and non-structural components (such as for hospitals include elevators, stairs, water systems, electric power systems, etc.) must remain functional. A number of studies have been conducted to seismically upgrade and retrofit the structural and non-structural components in critical facilities. A summary review of previous studies related to the performance assessment of hospitals will be introduced below.

(Ray-Chaudhuri and Shinozuka, 2010) generated a strategy for identifying essential components in critical facilities. The sensitive components were identified using a sensitivity analysis that was needed to consider the complicated system of water and electric power systems for the hospital building. The study also found that substantial improvements at the component level are expected to reduce the system of failure in the hospitals in California.

(Bhuiyan, Pal and Mazumder, 2015) assessed the performance of an existing hospital building in Bangladesh, the hospital building was modeled in Sap software, and incremental dynamic analysis was conducted. Analysis outcomes showed that the selected building was vulnerable to different damage states as per the revised code. The author adopted the proper retrofitting strategy to reduce vulnerability and keep the selected hospital building operational during and after expected earthquakes.

(Issa and Mwafy, 2014) derived the vulnerability functions to assess the seismic response of an emergency facility such as a fire station, typical private hospital, police station, and an international school in a highly populated and seismically active area in the United Arab Emirate. Three limit states, IO, LS, and CP, were selected based on inelastic analysis results and the values recommended in previous studies and code provisions. It was concluded that the results indicate the important need for the seismic retrofit for certain emergency facilities to assure their continued service, proportionate improvements were observed in the police station and school buildings with the Fiber Reinforced Polymer wrapping of the internal column, the fragility of 6-stories hospital was decreased by RC jacketing of internal columns.

(Bilgin, 2016) investigated the seismic response of three existing hospital buildings with differing heights 3 to 5 stories. Pushover and time history analyses are implemented to generate the fragility curve by Sap software. The author confirmed the vulnerability of the selected hospital buildings to the severe scenario earthquake expected in earthquake-prone regions of Turkey.

(Bilgin and Frangu, 2017) presented a methodology to predict the seismic performance of 5 stories RC health care facility in Albania. Non-linear static and dynamic time history analysis was carried out by using modeling in the Zeus NL computer program. The structural capacity values and inter-story drift values were chosen to meet the performance levels as specified in FEMA-356. It was concluded that the values of inter-story drifts exceeded the limit in most of the cases, and the hospital building exhibits ineffectual seismic performance under different seismic excitations.

It has been observed that none of the studies have been performed on the seismic vulnerability of existing hospital buildings in the region. Therefore, the current study considered evaluating the seismic vulnerability of the pre-seismic code hospital building in the Kurdistan Region of Iraq, where a similar project of the hospital building was constructed by Taisei and Marubeni Corporation all over Iraq in the 1980s.

3. DESCRIPTION OF THE HOSPITAL BUILDING

Rizgary hospital building consists of five parts, as shown in **Fig 1**, with a total floor area of 22,384.7 m^2. It was constructed with an expansion joint (30 mm) in the center of each part and between parts. Part 4 and Part 5 are selected for the current study. Part 5 is the main part. It has one basement, six superstructures, a roof floor, two penthouses, and a penthouse roof floor, while part4 has one basement, three superstructures, and a roof floor, as explained in **Table 1**. According to the structural detailing of the members, the structural elements necessary for structural modeling, for example, columns, beams, slabs, and walls, are obtained from the original structural drawing plans of the selected hospital building. **Fig. 2** illustrates the basement, ground, first, second, typical, penthouse, and roof floor framing plan. At the same time, **Fig. 3** depicts vertical element schedules, for example, columns, walls, and tie schedules for the selected parts (P4) and (P5) for the hospital building. **Table 2** epitomizes the schedule of girders and beams for the ground floor to the sixth floor, while **Table 3** epitomizes the schedule of girders and beams for the penthouse and roof floor.

Table 1. Summary of Rizgary hospital.

	Part	No. of stories	Story height (m)							Total height (m)
			B	GF	1F & 2F	TF	RF	PF: P1&P2	PF: PR	
Hospital	Part 5	10	3.35	0.65	4	3.6	3.6	5.5	4.5	36.65
	Part 4	4	3.35	0.65	4	4	4.5	___	___	17.15

B: Basement, GF: Ground Floor, 1F& 2F: First & Second Floor, TF: Typical Floor, PF: penthouses Floor, P1and P2: First and Second Penthouses Floor, PR: Penthouses Roof Floor, RF: Roof Floor

4. STRUCTURAL MODELING

ETABS finite-based software (www.csiamerica.com/products/etabs) is applied for the current analysis to develop a 3D model for the hospital building structure idealization to carry out the essential analysis. For this reason, two models are developed for the hospital building, including the hospital model considering P4+P5 and the hospital model only for P5. Therefore, the members and the sections are created in the following manner:

1) Modeling of frame: Members of beams and columns are designed as frame elements; section designer is applied to produce the desired dimensions and reinforcements.

2) Modeling of slabs : Slabs are specified as area elements that represent the characteristic of shell elements with the desired thickness; slabs are designed as rigid diaphragm elements.

3) Modeling of shear/basement walls: As shell elements, shear walls, and basement walls are designed, section designers are applied to generate the desired dimensions and reinforcements.

4) Gap element (nonlinear link element):

For impact force between parts, the gap element is utilized to connect between parts since the hospital building was constructed with an expansion joint. This leads to seismic pounding. Pounding is a phenomenon in which two buildings strike as a result of their lateral movements promoted by lateral forces during an earthquake. Structural pounding happens because of the swaying of adjacent buildings with different mode shapes and periods under seismic loads. The experience in past earthquakes has shown that the pounding of adjacent structures increases the damage of the structural components since it induces higher floor acceleration in the form of large magnitude short duration pulses; for example, the Mexico city earthquakes were explained by Rosenblueth and Meli **(1986)**.

Table 2. Girders and beams schedule at the ground floor to six floors for the selected parts.

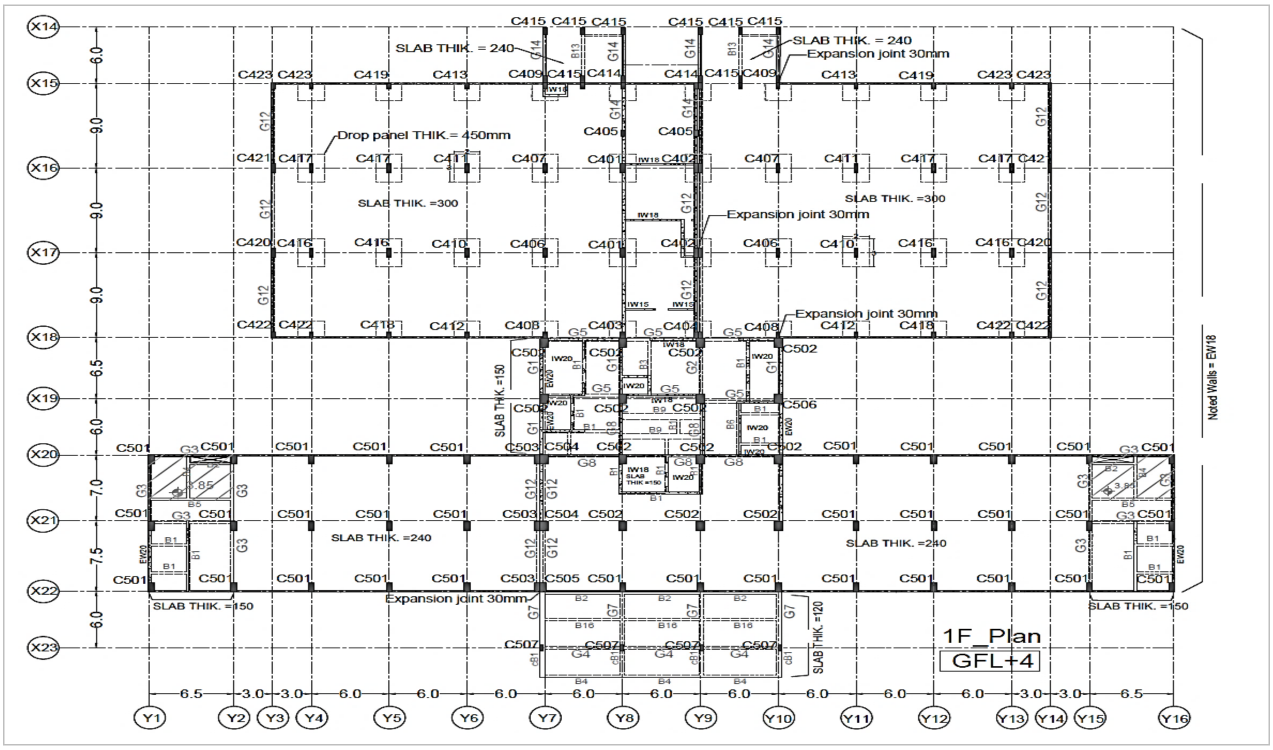
GF~6F Girder				BF~6F Beam			
Girder	*Size (bxh)*	*Top bar*	*Bottom bar*	*Beam*	*Size (bxh)*	*Top bar*	*Bottom bar*
G1[a]	250x600[1]	2 - Ø25	2 - Ø25	B1	250x400[1]	3 - Ø19	3 - Ø19
G2[a]	250x600[1]	3 - Ø25	3 - Ø25	B2	250x500[1]	3 - Ø19	4 - Ø19
G3[a]	250x800[2]	6 - Ø25	5 - Ø25	B3[a]	250x600[1]	3 - Ø19	3 - Ø19
G4[a]	250x700[1]	3 - Ø25	3 - Ø25	B4[a]	250x700[1]	3 - Ø25	3 - Ø19
G5[a]	300x600[1]	3 - Ø25	3 - Ø25	B5[a]	250x700[3]	3 - Ø25	5 - Ø25
G6[a]	250x800[2]	6 - Ø25	5 - Ø25	B6	350x400[3]	3 - Ø25	6 - Ø25
G7[a]	350x800[3]	9 Ø 25	4 - Ø25	B7[a]	350x700[2]	7-Ø25	4 - Ø25
G8	300x400[2]	3 - Ø25	3 - Ø25	B8	400x400[3]	3 - Ø25	5 - Ø25
G10[b]	300x1200[3]	4 - Ø25	4 - Ø25	B9	600x400[2]	4 - Ø25	10 - Ø25
G11[b]	300x1200[3]	6 - Ø25	4 - Ø25	B11[b]	300x1000[3]	3 - Ø25	6 - Ø25
G12[a]	250x450[4]	2 - Ø25	2 - Ø25	B13[a]	250x880[1]	2 - Ø25	3 - Ø25
G13[c]	180x1800[5]	2 - Ø19	2 - Ø19	B14[a]	350x600[3]	6 - Ø25	6 - Ø10
G14[a]	250x800[3]	2 - Ø25	2 - Ø25	B15	450x400[3]	3 - Ø25	3 - Ø25
G15[a]	250x800[3]	4 - Ø25	4 - Ø25	B16	250x600[1]	4 - Ø19	3 - Ø19
G20[b]	300x910[2]	8 - Ø25	6 - Ø25	cB1	350X800[3]	9-Ø25	4 - Ø25

Web bar[a]: 2 Ø10; Web bar [b]: 6 Ø 10; Web bar [c]: 18 Ø10 and Stirrup[1]: Ø10-@200; Stirrup[2]: Ø13-@150; Stirrup[3]: Ø13-@200; Stirrup[4]: Ø10-@190; Stirrup[5]: Ø10-@250

Table 3. Girders and beams schedule at the roof and penthouse for the selected parts.

RF~PRF Girder				RF~PRF Beam			
Girder	*Size (bxh)*	*Top bar*	*Bottom bar*	*Beam*	*Size (bxh)*	*Top bar*	*Bottom bar*
G1	300x450[1]	6 - Ø25	4 - Ø25	B1	250x400[2]	2-Ø25	3-Ø19
G2	250x450[2]	2 - Ø25	2 - Ø25	B2	300x450[4]	3-Ø25	6-Ø25
G3	250x550[2]	2 - Ø25	2 - Ø25	B3	250x450[2]	3-Ø19	3-Ø19
G4[a]	250x600[2]	2 - Ø25	2 - Ø25	B4	250x550[2]	3-Ø19	3-Ø19
G5[a]	250x700[2]	2 - Ø25	2 - Ø25	B5	250x550[4]	5-Ø25	3-Ø25
G6[a]	300x550[2]	3 - Ø25	3 - Ø25	B6[a]	250x600[2]	3-Ø19	3-Ø19
G7[a]	300x600[2]	3 - Ø25	3 - Ø25	B7[a]	250x800[2]	3-Ø19	3-Ø19
G8[a]	300x700[3]	3 - Ø25	4 - Ø25	B8[c]	200x1100[2]	2-Ø25	4-Ø25
G9[a]	300x800[2]	3 - Ø25	3 - Ø25	B9[a]	300x600[2]	3-Ø19	3-Ø19
G10[b]	300x1040[2]	3 - Ø25	3 - Ø25	B10[a]	300x700[2]	3-Ø25	5-Ø25
G11[a]	350x700[3]	3 - Ø25	3 - Ø25	B11[a]	300x700[3]	3-Ø25	7-Ø25
G12	250x450[2]	2 - Ø25	2 - Ø25	B12[a]	300X800[3]	3-Ø25	6-Ø25
G13[a]	250x800[2]	2 - Ø25	2 - Ø25	B13[a]	400X700[2]	3-Ø25	3-Ø25
cG1	250x500[2]	3 - Ø19	2 - Ø19	B14	250X450[2]	2-Ø25	4-Ø25

Web bar [a]: 2 Ø10; Web bar[b]: 6 Ø10; Web bar[c]: 10 Ø10
Stirrup[1]: Ø13-@150; Stirrup[2]: Ø10-@200; Stirrup[3]: Ø13-@200; Stirrup[4]: Ø10-@150;



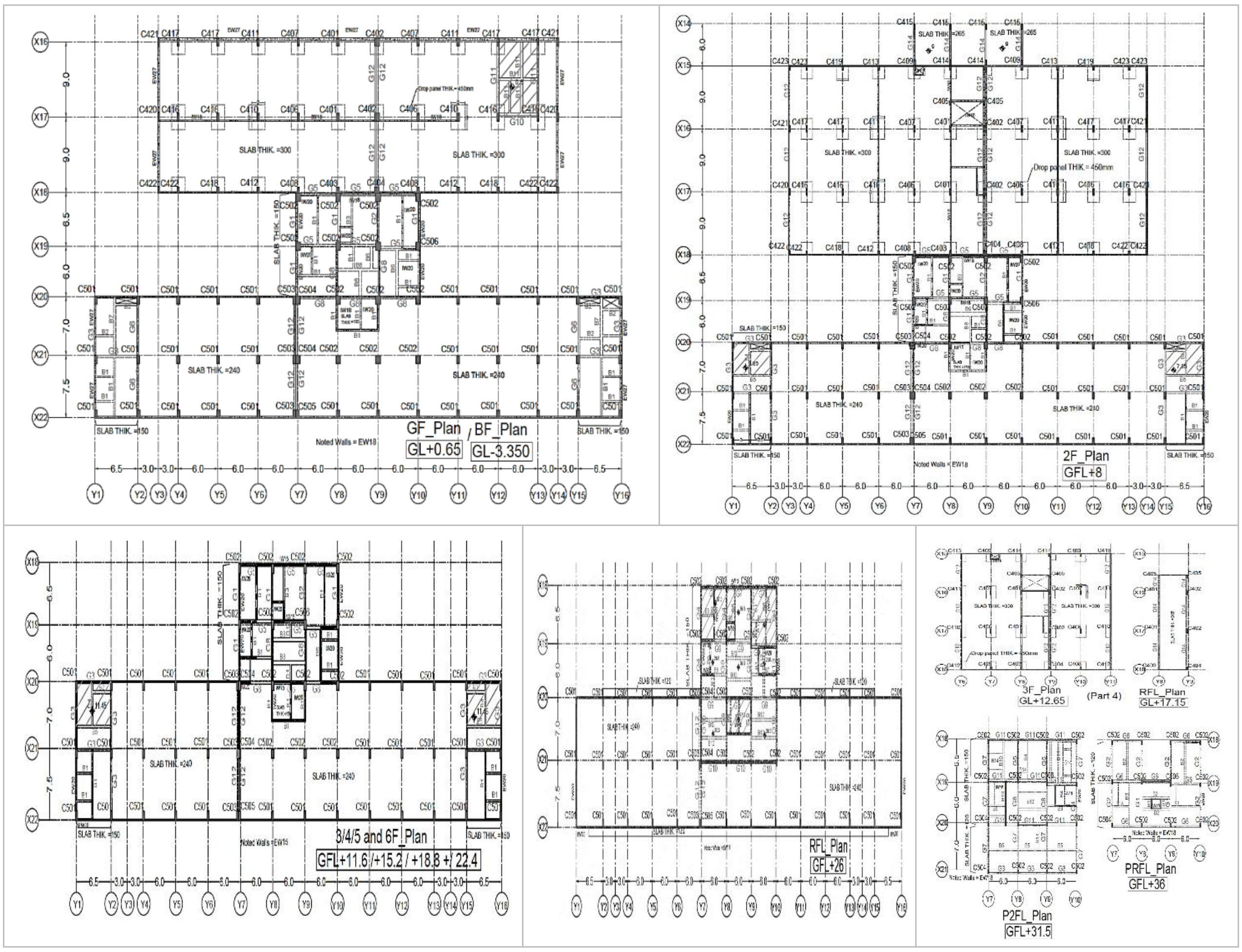
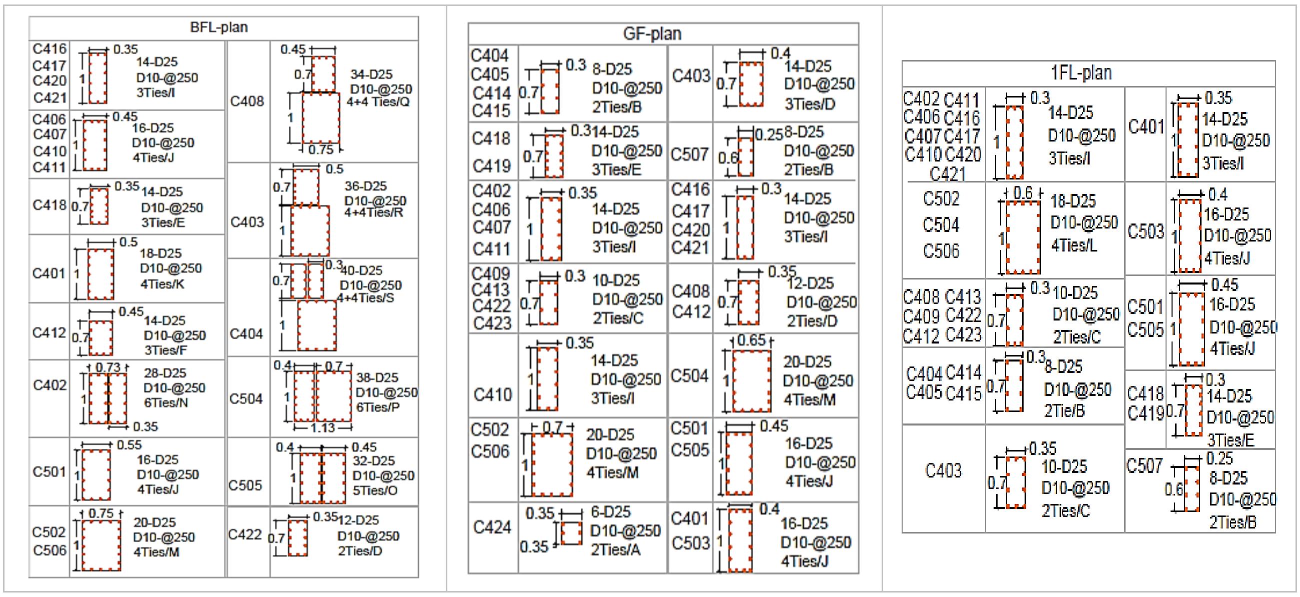


Figure 2. Generic structural plan from the basement floor framing to roof floor framing. for the selected parts.



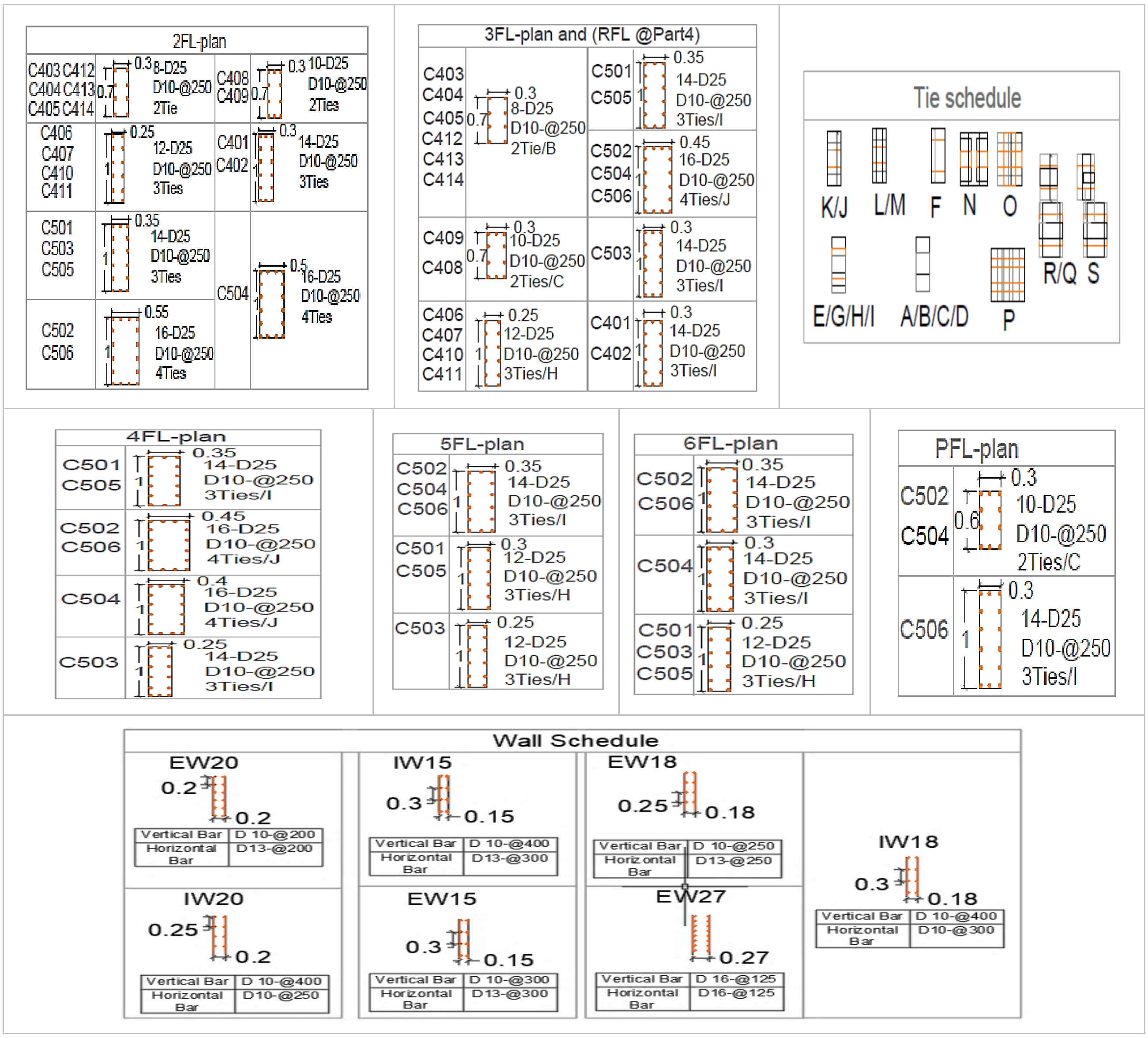


Figure 3. Vertical members schedule; column, tie, and wall for the selected parts.

More recently, **(Hameed et al., 2012)**, **(Jameel et al., 2013)**, **(Kumar and Karuna, 2015)**, **(Abhina et al., 2016)**, and **(Yaseen, 2017)** studied the effect of seismic pounding between adjacent RC buildings; it was found that interactions could be dangerous for both structures. Moreover, the recommended mitigation methods were applied. In ETABS software **(CSI Computers and Structures, 2017)**, the gap is defined by the nonlinear link element. A two-joint connecting link is specified by a link element. That has only compression properties. The stiffness of the gap element is commonly approved as 102 to 104 times the stiffness of the adjacent linked element.

5. MATERIAL MODELING

The elastic performance of frame and shell elements is dictated by the frame and shell sections assigned to the elements. For modeling, the compressive strength of concrete 20 MPa and reinforcing steel grade Gr 40 are used with Young's modulus E of 200,000 MPa and yield strength of 240 MPa. In ETABS **(CSI, 2017)**, the nonlinear analysis could be performed using hinges. At the location of the maximum potential of the force, hinges are assigned, for instance, the ends of beams and columns; the theoretical adopted distance of 0.1 and 0.9 lengths, as accepted by code of practice. Each hinge is modeled as a separate point hinge and reflects post-yield concentrated behavior. In nonlinear static and nonlinear time history analysis, hinges only affect the structure's behavior. In addition, for each degree of freedom, axial and shear specified by force-displacement behavior, while bending and torsion characterized by plastic moment rotation behavior. Obviously, in ETABS, three various hinge properties are defined: default hinge, nonlinear hinge properties that are user-defined, and automated hinge characteristics. Automated hinge characteristics are automatically determined according to ACI 318-14 criteria from the material of the frame element and the section properties **(ACI, 2014)**; plastic behavior of the hinges is simply expected from the assigned material properties of the frame and shell elements. Therefore, the following types of automatic hinges, as determined by ASCE41-17, can be obtained from ETABS:

1) It is possible to generate concrete beams in the flexure (M) hinges, using items from table 10-7
2) Concrete columns in flexure (P-M-M) hinges can be generated using items from table 10-8/9.
3) Concrete walls in flexure (P-M) fiber hinges can be generated.

The models of Takeda and Kinematic have certainly been used sequentially to reflect the hysteretic performance of concrete and steel materials **(CSI, 2017)**. In order to describe stress-strain curves of concrete and steel materials, respectively, Mander and Simple parametric descriptions were also used. For better understanding the response of the hospital building, three cases are considered: hospital model P4+P5 is analyzed without considering fiber hinges in the RC walls, whereas hospital model P 5 is analyzed with considering fiber hinges in the RC walls.

6. BUILDING PERFORMANCE LEVELS

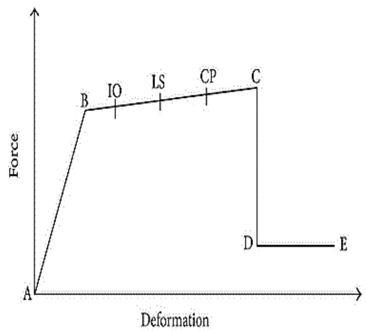
The seismic performance of building structures is characterized by the level of damage that is continuously taken into account by seismic provisions. The levels of performance that classified in codes and standards include the following items **(ASCE/SEI 41-13, 2014; ATC, 1996; FEMA-356, 2000)**:

1) Prevention of collapse (CP): Permits a minimum margin of safety against collapse during a severe earthquake.
2) Life Safety (LS): Specifies considerable damage to the lateral force-resistant system of the building but maintains a large collapse margin.
3) Immediate Occupancy (IO): The building may suffer relatively minor damage, and the lateral force-resistant components retain their initial strength and much of their original ductility.

Other levels of performance were also considered; operational performance (OP) was also taken into account. Inter-story drift ratio (IDR) is frequently classified as the primary performance criterion as it is linked to the level of performance **(ASCE/SEI 41-13, 2014)**; different studies have used IDR to evaluate the RC building wall system, confirmed values are shown in **Table 4**.

Table 4. Study of IDRs for various limit states and structural systems of RC walls.

Authors	Approach of selection	Inter-story drift (%)			Performance levels **(ASCE/SEI 41-13, 2014)**
		IO	L S	C P	
ASCE **(ASCE/SEI 41-13, 2014)**		0.5	1	2	
Ghobarah **(2004)**	Experimental studies	0.4	1.5	2.5	
Lehman et al. **(2013)**	Experimental studies	0.50	1.0	2.27	
Panagiotou et al. **(2011)**	Experimental studies	0.35	0.89	2.36	

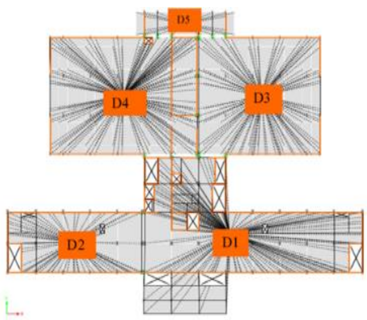
<table>
<tr><td>Beyer et al. (2008)</td><td>Experimental studies</td><td>0.30</td><td>------</td><td>2.39</td><td rowspan="6">
</td></tr>
<tr><td>Ramamorthy et.al.(2008)</td><td>Analysis of pushover</td><td>0.75</td><td>------</td><td>1.71</td></tr>
<tr><td>Kircher et al. (1997)</td><td>Dynamic incremental analysis</td><td>0.8</td><td>2.3</td><td>6</td></tr>
<tr><td>Liel et al. (2011)</td><td>Dynamic incremental analysis</td><td>------</td><td>------</td><td>4.17</td></tr>
<tr><td>Hassan and Mwafy (2014)</td><td>Dynamic incremental analysis, time history, and pushover</td><td>0.5</td><td>1.135</td><td>2.27</td></tr>
<tr><td>Issa and Mwafy (2014)</td><td>Dynamic incremental analysis, time history, and pushover</td><td>0.34</td><td>0.89</td><td>1.78</td></tr>
</table>

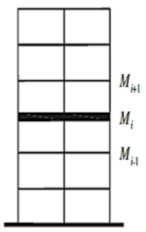
7. STRUCTURAL IRREGULARITIES EXAMINATION

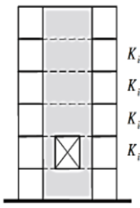
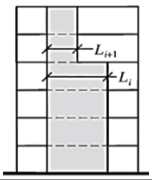
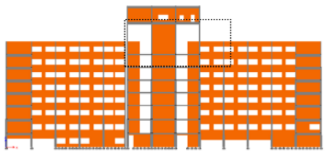
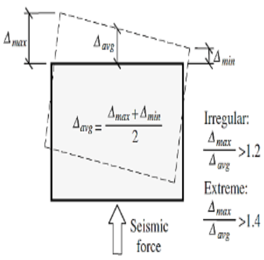
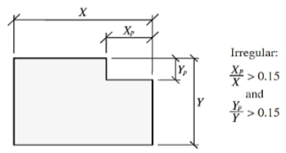
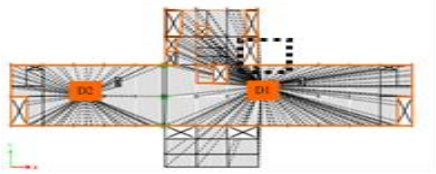
The structural irregularities are one of the main reasons for the building damage during the earthquake excitation. Irregular structures have certain physical discontinuities either in the plan or in elevation or both, which affect the performance of the structure that is subjected to lateral loads. Soni et al. **(2015)** and Teruna **(2017)** observed that mass irregularity and soft-story are more vulnerable during an earthquake, Abraham and SD **(2019)** noted that the combination of irregularities is also more vulnerable to seismic excitation, Varadharajan **(2015)** demonstrated that the vertical and the horizontal irregularities and their location are more effective. **(Khalifa and Mwafy, 2015)** confirmed that the pressing need for mitigation strategies to reduce the expected seismic losses of irregular high-rise buildings.

Consequently, possible irregularity of the hospital building was investigated using linear static analysis and according to the definitions of ASCE **(ASCE/SEI 7-16, 2017)**. It was observed that the hospital building experiences possible vertical irregularity, including mass, stiffness (soft-story), geometric irregularity, and horizontal irregularity, including torsional irregularity and reentrant corner irregularity, as explained in **Table 5**.

Table 5. Possible vertical and horizontal irregularities in the hospital building.

<table>
<tr><td colspan="6">Requirements for vertical and horizontal irregularity by ASCE (ASCE/SEI 7-16, 2017)</td><td rowspan="6">

Diaghram (Digh.) loading (D1 to D5)</td></tr>
<tr><td colspan="2">Irregularity in the vertical: weight (mass)</td><td>Dir.</td><td>FL</td><td>Mi/± Mi1</td><td>Digh.</td></tr>
<tr><td rowspan="4">
</td><td rowspan="4">$M_i > 150\% \ M_{i+1}$
or
$M_i > 150\% \ M_{i-1}$</td><td rowspan="4">X, Y-Dir.(loading)</td><td>RFL</td><td>1.84 (+)</td><td>D1</td></tr>
<tr><td>1FL</td><td>1.54 (+)</td><td>D3</td></tr>
<tr><td>2FL</td><td>2.0 (+)</td><td rowspan="2">D4</td></tr>
<tr><td>1FL</td><td>1.51 (-)</td></tr>
<tr><td colspan="7">Vertical irregularity: stiffness</td></tr>
<tr><td></td><td></td><td></td><td colspan="3">X-Dir. (loading)</td><td>Y-Dir. (loading)</td></tr>
</table>

<table>
<tr><td rowspan="5"></td><td rowspan="2">Stiffness /soft story</td><td rowspan="2">K_i < 70% K_{i+1}
K_i < 27% K_{avg}</td><td>FL</td><td>Ki/Ki+1</td><td>Ki /Kavg</td><td>FL</td><td>Ki/Ki+1</td><td>Ki /Kavg</td></tr>
<tr><td>RFL</td><td>0.4</td><td></td><td>RFL</td><td>0.11</td><td>0.25</td></tr>
<tr><td rowspan="3">Stiffness / extreme soft story</td><td rowspan="3">Ki < 60% Ki+1
Ki < 24% Kavg</td><td>2FL</td><td>0.31</td><td>0.43</td><td>6FL</td><td></td><td>0.32</td></tr>
<tr><td>1FL</td><td></td><td>0.68</td><td>5FL</td><td></td><td>0.41</td></tr>
<tr><td>BFL</td><td>0.2</td><td>0.56</td><td>BFL</td><td>0.36</td><td></td></tr>
<tr><td colspan="9">Vertical irregularity: vertical geometric</td></tr>
<tr><td></td><td colspan="2">Li > 130% Li+1</td><td colspan="3">Vertical geometric irregularity Li/Li+1 = 2.6 >1.3</td><td colspan="3"></td></tr>
<tr><td colspan="9">Horizontal irregularity: torsional irregularity</td></tr>
<tr><td colspan="2" rowspan="8"></td><td colspan="4">X-Dir (loading) "ratio: $\Delta_{max}/\Delta_{avg}$"</td><td colspan="3">Y-Dir. (loading) "ratio $\Delta_{max}/\Delta_{avg}$"</td></tr>
<tr><td>FL</td><td>Ratio</td><td colspan="2">Digh.</td><td>FL</td><td>Ratio</td><td>Digh.</td></tr>
<tr><td>PFL</td><td>1.56</td><td colspan="2" rowspan="2">D1</td><td>P1FL</td><td>1.3</td><td>D1</td></tr>
<tr><td>GFL</td><td>2.4</td><td>1FL</td><td>1.24</td><td>D1</td></tr>
<tr><td>1FL</td><td>1.3</td><td colspan="2" rowspan="2">D2</td><td>GFL</td><td>1.5</td><td>D1</td></tr>
<tr><td>GFL</td><td>2.53</td><td>GFL to RFL</td><td>1.6 to 1.7</td><td>D2</td></tr>
<tr><td>GFL to 3FL</td><td>1.4 to 2.01</td><td colspan="2">D3</td><td>GFL to 3FL</td><td>1.4 to 1.75</td><td>D3</td></tr>
<tr><td>GFL to 4FL except 1FL</td><td>1.3 to 1.70</td><td colspan="2">D4</td><td>GFL to 4FL except 2 and 3FL</td><td>1.3 to 1.5</td><td>D4</td></tr>
<tr><td colspan="9">Horizontal irregularity: reentrant corner</td></tr>
<tr><td colspan="3"></td><td colspan="3">Recurrent irregularity YP/Y = 0.4>0.15</td><td colspan="3"></td></tr>
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8. FREE VIBRATION (EIGENVALUE) ANALYSIS

When the building oscillates without any external dynamic excitation, it is under a free vibration state **(Chopra, 2012)**. Eigenvalue analysis was performed for the extraction of natural frequencies and mode shapes for the hospital building. Furthermore, taking into consideration the P-delta effect as studied by **(Abbas and Abdulhameed, 2019)**, and 90 percent participating modal mass as recommended by ASCE **(ASCE/SEI 7-16,2017)**. As a predecessor to dynamic analysis, this analysis is essential because information on the natural frequencies and mode shapes helps to understand the dynamic response. **Fig. 4** represents the first six mode shapes (out of the 150 mode shapes) for selected parts (P4+P5) and only P5 of the hospital building.

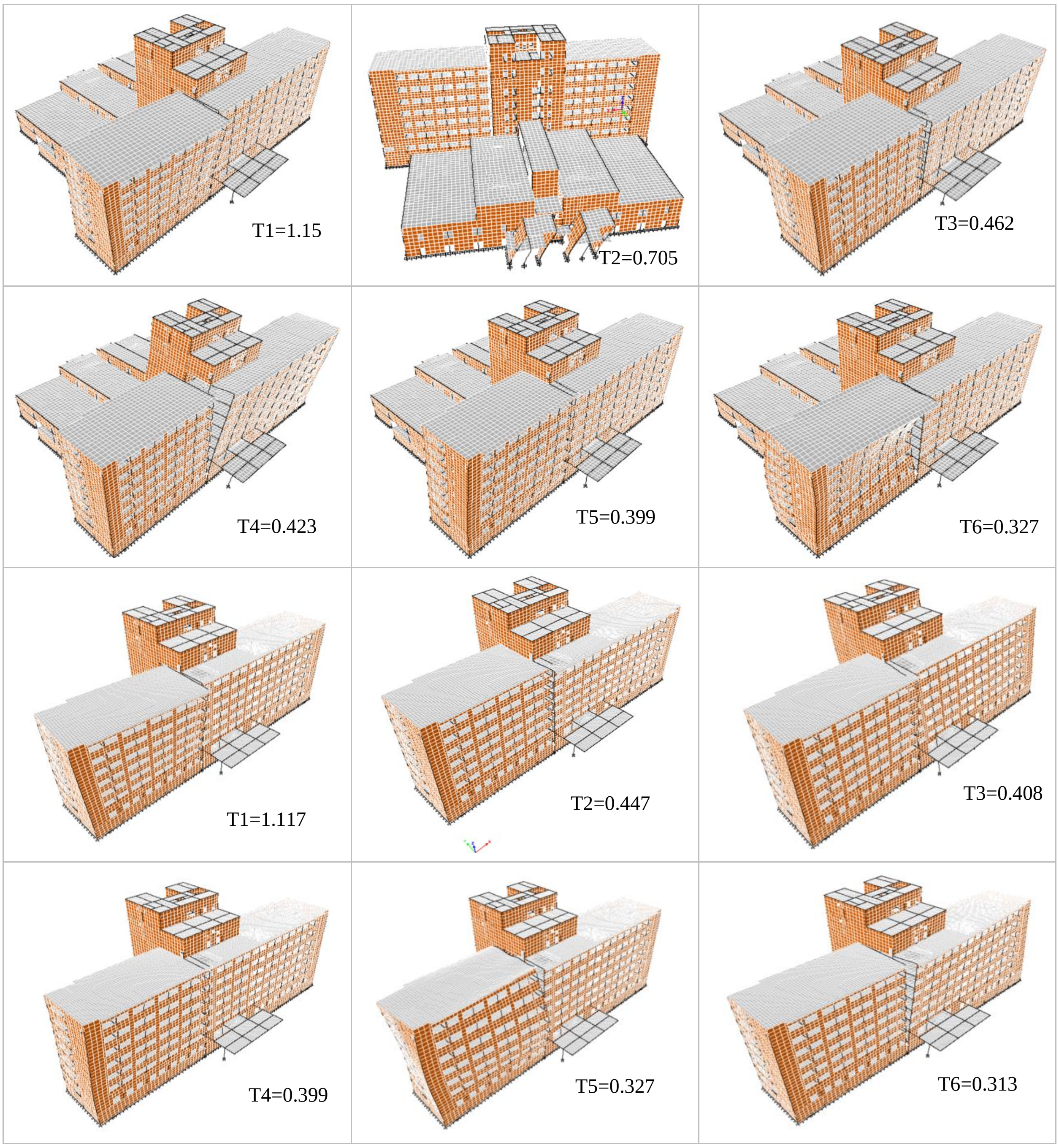


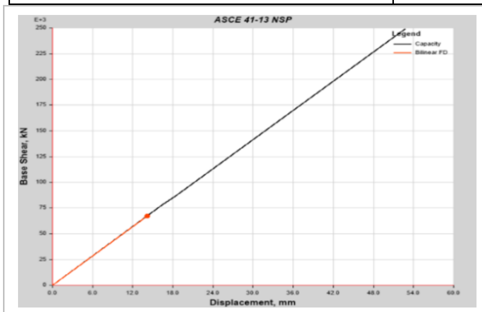
Figure 4. First six mode shapes and periods of the hospital building for p4+p5 (top) and only P5 (bottom).

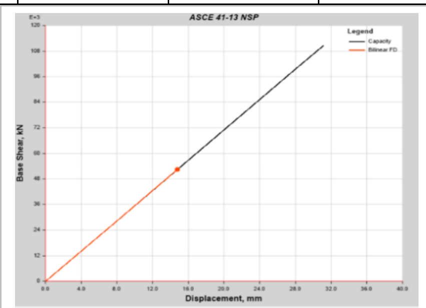
9. NONLINEAR STATIC ANALYSIS (NLSA)

The NLSA is employed to evaluate the structure's lateral capacity in the inelastic region. Under permanent vertical loads, this analysis covers incrementally increasing the lateral loads. The lateral load pattern, which is spread along with the height of the building, is increased proportionally until the structure reaches a certain limit state or a target displacement. NLSA enables difficulties to be avoided, even if precise, non-linear analyses of time history are accurate. Such an advantage is much more evident in the high difficulty associated with seismic input selection and scaling, the definition of evolutionary hysteretic models, the interpretation of the analysis results, and the time for analysis. Through various previous studies, the NLSA approach was developed and validated for example, **(Gupta and Kunnath, 2000)**, **(Mwafy and Elnashai, 2001), (Chopra, and Goel, 2002)**, **(Aydinoglua, 2003)**, **(Antoniou and Pinho**, **2004.)**, **(Hassan and Mwafy, 2014)**, **(Issa and Mwafy, 2014)**, and then **(Khalifa and Mwafy, 2015)**). Some of the previous studies have not recommended the NLSA for the seismic assessment of high-rise buildings **(TBI, 2010)**. Contrarily, some previous studies concluded that the NLSA precision was not significantly decreased even for irregular structures **(Chintanapakdee and Chopra, 2004)**. Additionally, other previous research concluded that the uniform lateral load pattern in NLSA can be conservatively used to estimate high-rise buildings' initial stiffness and lateral capacity **(Hassan and Mwafy, 2014; Issa and Mwafy, 2014; Khalifa and Mwafy, 2015)**. For the evaluation of the hospital building, the classical pushover capacity spectrum approach is applied by displacement controlled nonlinear analysis including predefined permanent vertical loads (100% dead load and 25% live load) then using acceleration load case at X and Y-direction by considering P-delta effects; following the ASCE document **(ASCE/SEI 41-13, 2014)**, which was established by Freeman **(1975)**. Consequently, by plotting the maximum base shear against top displacement and demand curves assessed by spectrum source acquired as specified by ASCE, the lateral capacity of the hospital building is assessed. The intersection of the capacity curve and the demand curve also shows performance points. The capacity curve, demand curve, performance point, hinge situation, and the NLSA derived profiles for drift are illustrated in **Fig. 5**. The detailed situation of the hinges with regard to the different performance level stages as explained in **Table 6**.

Table 6. Hinge situations at the performance level in the hospital building.

Hinges	hospital (P4+P5)		hospital P5 (Gap)		hospital P5 (Open)	
Push direction	X	Y	X	Y	X	Y
Overall hinge	5212		7265		7265	
Total steps	5	3	12	17	12	11
Performance point (located step)	3	3	5	9	4	11
Hing status						
A-IO	5170	5184	7240	7231	7248	7239
IO-LS	5	6	0	10	0	13
LS-CP	2	3	0	7	0	5
>CP	35	19	25	17	17	8
Hing states						
A-B	4996	5088	7184	7137	7185	7170
B-C	194	103	61	106	65	81
C-D	2	2	0	1	0	1
D-E	0	2	0	0	0	1
>E	20	17	20	21	15	12





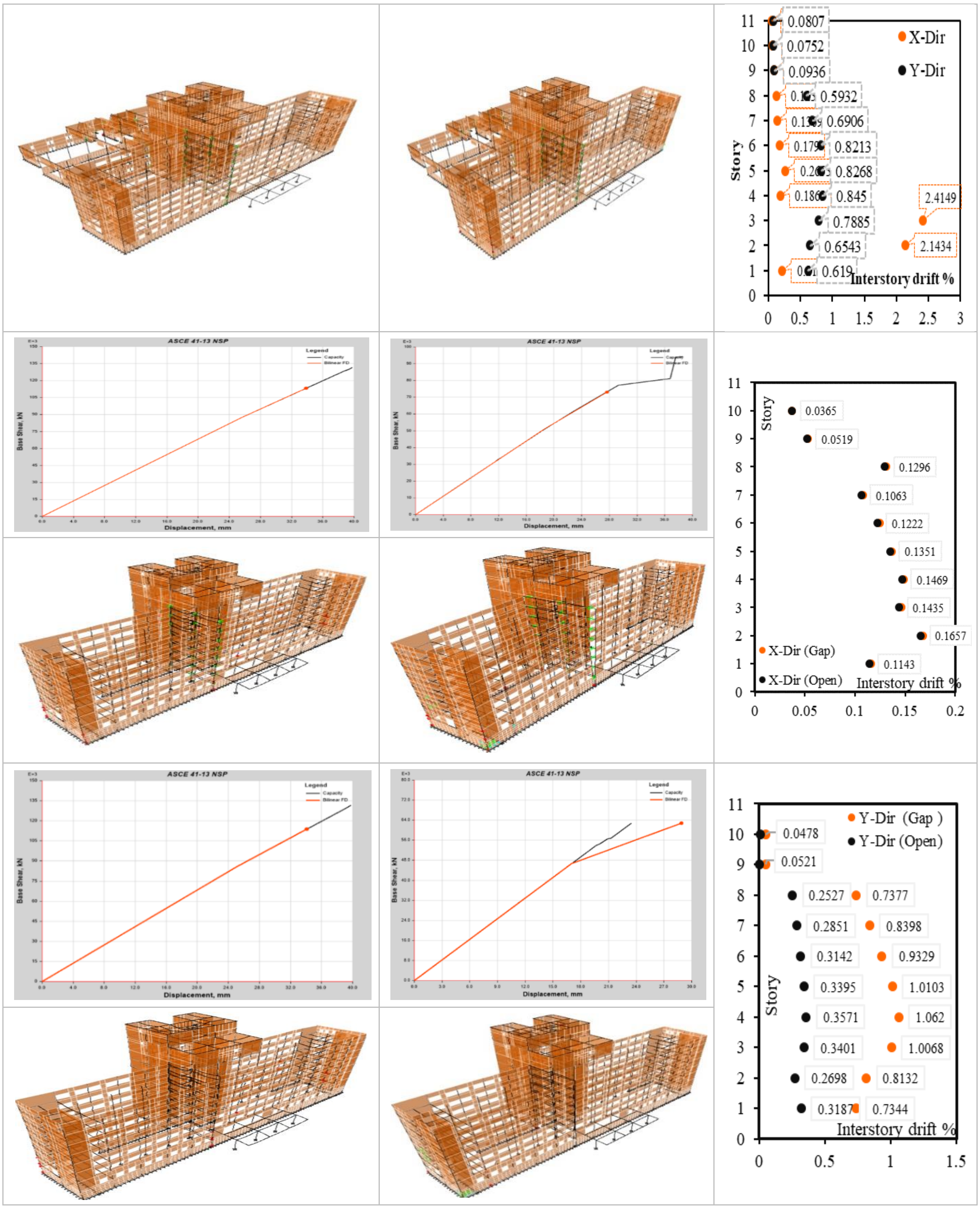


Figure 5. Capacity curves and plastic hinge distributions for selected p4+p5 (top); only P5. with a gap (middle); and only P5 without a gap (bottom) for load case Push X (left) and push Y (right), and drift profiles (end right)

Mapping of plastic hinges revealed that the level of plastic hinges attained the point of prevention of collapse (CP) with a small limit of safety, which demonstrated substantially weak performance, which was reflected back by a high number of plastic hinges, especially in the vertical components at the basement and ground story, resulting in a strong-beam weak column. The possible irregularities (**Table 5**) have been verified the accuracy of the results in terms of the inadequacy of the seismic performance. The seismic pounding also increases deficiency, as represented for P5 in **Fig 5**. It is clear from results IDR at CP is 2.41%, 1.06%, and 0.357% for the hospital (P4+P5), hospital (P5) with and without considering gap element, respectively. It is also clearly apparent from **Table 4** that the IDR at the CP ranges from 2% to 2.5% is widely used for the construction of RC walls.

10. INCREMENTAL DYNAMIC ANALYSIS (IDA)

IDA procedure was used to evaluate the inelastic seismic response of a structural system with an increasing severity under seismic loads. In conjunction with the probabilistic seismic risk analysis, potential damage to the concerned structure was evaluated **(Vamvatsikos and Cornell, 2002)**. IDA involves performing multiple inelastic time history analyses under a series of selected ground motions for a structural model, which was scaled to several seismic intensity levels, as verified by **(Mwafy, 2012)**, **(Hassan and Mwafy, 2014)**, **(Issa and Mwafy, 2014)**, **(Khalifa and Mwafy**, **2015)** and **(Mahmoud and Al-Baghdadi, 2018)**. For the current study, nonlinear analyses of dynamics are performed as Nonlinear Modal Time History: Fast Nonlinear Analysis utilizing modal Ritz by considering P-Δ effects and damping ratios equal to 5 percent for all modes.

10.1 Selection and Scaling Ground-Motions

The set of ten ground-motion records are selected for vulnerability assessment of hospital buildings, and the selected natural records are shown in Error! Reference source not found.. These ten ground-motion records were selected from the pacific earthquake engineering research center (PEER_NGA) for covering the seismic hazard assessment of Erbil city **(Abduljaleel and Taha, 2020)**. Based on their PGAs, the input ground motions used in the hospital building are scaled; therefore, seismic forces are directly linked to the acceleration of the inputs. This methodology of scaling is in accordance with the codes of design and has therefore been applied in various previous studies, e.g., **(Ji et al., 2007)**, **(Mwafy, 2012)**, **(Hassan and Mwafy, 2014)**, and **(Issa and Mwafy, 2014)**); authors were also proposed different scaling factors depending on the seismic scenarios and building characteristics. The scaling levels are thus selected to force the structure from the elastic to the inelastic range across the entire range of behavior and finally to collapse.

Table 7. Summary of ground-motion records.

N GA #	Recorded Earthquake	Component	Date	Mw	R_{rup} (km)	Dur. (Sec)	PGA (g)
126	"Gazli_ Ussr"	GAZ 000	1976	6.8	5.46	13.5	0.70
183	"Imperial Valley-06"	E 08140	1979	6.53	3.86	37.7	0.61
5827	"El Mayor-Cucapah_Mexico"	MDO 000	2010	7.2	15.91	100	0.54
8124	"Christchurch_New Zealand"	86 W	2011	6.2	17.87	22	0.29
6	"Imperial Valley-02"	ELC 180	1940	6.95	6.09	53.7	0.28
6893	"Darfield_New Zealand"	17 E	2010	7	11.86	150	0.47
1602	"Duzce_ Turkey"	BOL 000	1999	7.14	12.04	55.9	0.74
1082	"Northridge-01"	RO3 000	1994	6.69	10.05	30.28	0.28
761	"Loma Prieta"	FMS 090	1989	6.93	39.85	39.7	0.19
730	"Spitak_ Armenia"	GUK 000	1988	6.77	23.99	20	0.20

10.2 Incremental Dynamic Analysis Results

The hospital building response parameters are obtained from approximately 120 inelastic time history analyses, such as base shear and IDR. The most important IDA findings are performed in terms of the IDA curve, as shown in **Fig 6**. The IDA curve includes PGA plotting against IDR. As a result, the yield stage is considered to be reached when the slope of the IDA curve changes from linear to nonlinear. When the curve of the IDA becomes a significantly flat or nonlinear slope, less than 20 percent of the elastic slope is deemed to be achieving the collapse capacity of the structure. **(Vamvatsikos and Cornell, 2002)**. Acceleration of collapse (g) is obtained from the IDA curve, which is particularly used in various previous studies and current studies to explain the results of IDA. The collapse acceleration was estimated by **(Cosenza et al., 2000)** by adjusting one record of ground motion. At the same time, **(Bruno et al., 2000)** used optimum outcomes from three records to assess the conditions of primary collapse, while **(Tiwari and Kasnale, 2017)** and **(Rakshe and Kalwane, 2018)** assessed the structural vulnerability depending on the nine ground-motion records' median collapse acceleration, the authors also confirmed the insufficiency of the structural model for the case analysis that obtained less acceleration of collapse than the original unscaled ground-motion record. **Table 8** summarizes collapse acceleration for the selected parts of the hospital building.

It was observed that the hospital building could not sustain all the selected ground-motion records, the median value also evaluated by 0.13, 0.15, and 0.18 for the hospital (P4 + P5) and hospital (only part 5) with and without considering Gap element, respectively (**Table 8**). The poor performance is proved by the combination of vertical and horizontal irregularities, as indicated in **Table 5**. Pounding also affected the results and increased the inadequacy as verified with and without considering the Gap element in the hospital building (only Part 5).

Generally, pushover and IDA provided comparable results as possible failure mechanisms. For purposes of comparison, the results of the max IDR calculated by pushover analysis at the CP limit state are 2.41%, 1.06%, and 0.357% for hospital P4+P5, hospital P5 with and without considering the Gap element, respectively. However, the average IDR obtained from the collapse IDA curves have somehow a similar result and less than 1%, which is 0.45%, 0.39%, and 0.410% for hospital P4+P5, hospital P5 with and without considering the Gap element, respectively. This shows that compared to pushover analysis, the incremental dynamic analysis is more accurate.

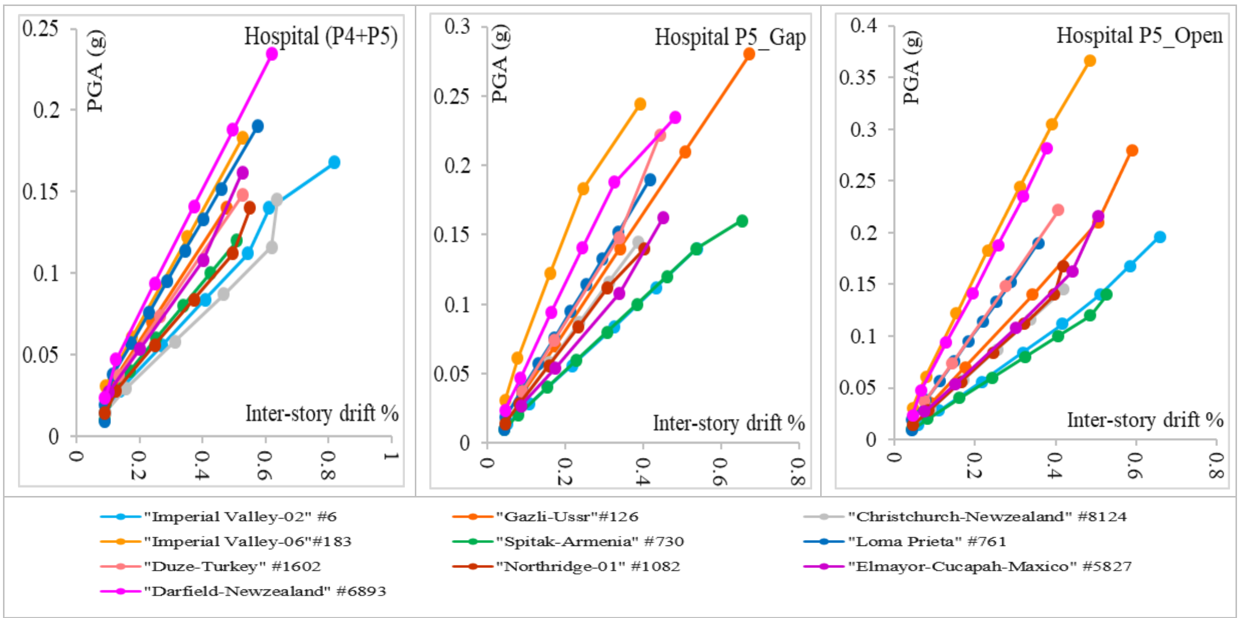


Figure 6. Incremental dynamic analysis curve for selected parts of the hospital building.

Table 8. Acceleration of collapse for selected parts of the hospital construction.

NGA Record Number	PGA (g)	Collapse acceleration (g)		
		hospital (P4+P5)	hospital P5	
			Gap	Open
"Imperial Valley-02" #6	0.28	0.08	0.14	0.14
"Gazli-Ussr"#126	0.70	0.14	0.14	0.21
"Imperial Valley-06"#183	0.61	0.18	0.18	0.24
"Spitak-Armenia" #730	0.20	0.10	0.10	0.10
"Loma Prieta" #761	0.19	0.13	0.17	0.17
"Northridge-01" #1082	0.28	0.08	0.13	0.17
"Duze-Turkey" #1602	0.74	0.15	0.15	0.22
"Elmayor-Cucapah-Maxico" #5827	0.54	0.16	0.16	0.19
"Darfield-Newzealand" #6893	0.47	0.14	0.24	0.28
"Christchurch-Newzealand" #8124	0.29	0.09	0.12	0.12
Median		0.13	0.15	0.18

11. CONCLUSIONS

Erbil city is prone to earthquakes caused by the Zagros Taurus belt. Obviously, after the last cyclic earthquakes and updating the acceleration map of the Arabian plate have been raised many questions regarding the safety of pre-seismic code buildings, especially hospital buildings as essential emergency facilities, which have a key role in the recovery period during and after earthquakes. As a result, the current study was concentrated on the seismic vulnerability of pre-seismic code hospital buildings, which is the most common public hospital structure in the region. Obviously, original member sizes and reinforcements were used to model the hospital building; moreover, for better understanding, the seismic response three models were developed; considering the main two parts, with and without considering the pounding effect for one main part. Irregularities in the structural model were also evaluated by using linear static analysis according to criteria specified in ASCE/SEI 7-16.

For this purpose, Eigenvalue analyses, nonlinear static analyses, and about 120 incremental dynamic analyses were performed to calculate the vulnerability of the hospital building under anticipated earthquake, as follows:

- Dynamic characteristics were calculated by Eigenvalue analysis; this analysis aids in the understanding of the actual dynamic response in terms of natural frequencies and mode shape of the structure.
- Nonlinear static pushover analysis in two main directions was used to determine the capacity curves of the hospital building, which was utilized for determining the load-deformation (pushover) curves. The ASCE/SEI 41-13 enhanced capacity spectrum method was used in the evaluation of the seismic vulnerability of the hospital building under the seismic hazard specified in Erbil city.
- Incremental dynamic analysis was employed to determine the collapse acceleration by plotting PGA against the IDR curve; by applying a suitable set of ground motion which is compatible with seismic characteristics in Erbil city.

The most significant observations and conclusions from the present study are summarized below:

1. The Eigenvalue results were found to confirm the numerical models and add value to the current study's findings.

2. The key findings of nonlinear static pushover analysis revealed that the demand curve intersects the capacity curve with a limited preserve of strength and deformation capacity, implying that the hospital building will perform poorly during an earthquake and will be more vulnerable to seismic excitation.
3. The notable achievements of the incremental dynamic analyses indicated that the hospital building's seismic performance is very poor, with collapse accelerations of less than 0.2g, whereas the expected PGA value was 0.4g for a return period of 2475 years.
4. It was confirmed that the incremental dynamic analysis is more accurate than the nonlinear static analysis, in spite of being time-consuming and requiring detailed information about seismic characteristics of the region.
5. It was remarked that the hospital building behaves such as weak column and strong beam system because the building constructed according to the conventional design approach and poor material qualities.
6. It was observed that the pounding effect increases the vulnerability of the structural model under seismic load.
7. It was noticed that the possible combination of irregularities in the hospital buildings has a significant impact on seismic performance and an increase in the deficiency during earthquakes. The hospital building generally suffers from the sudden change in mass, stiffness, geometric irregularity in a vertical direction, and horizontal irregularity, including torsional irregularity and reentrant corner irregularity, which leads to suffering damage in columns at basement and ground story.
8. The selected hospital building experienced major damage to the vertical components, particularly columns at the basement and ground story. However, the detailed failure mechanism regarding first yielding and crushing concrete was limited.
9. It was recommended a suitable strengthening and retrofitting strategy, especially for the columns at the basement and ground level in order to keep operational the hospital building aftermath expected earthquakes. For new construction emergency facilities such as hospitals in the city, the performance-based design and analysis according to updated seismic code or updated draft code (e.g., updated codes dealing with the northern corner of the Arabian plate) were adopted.

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