International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:9, No:1, 2015

# Seismic Assessment of Old Existing RC Buildings on Madinah with Masonry Infilled Using Ambient Vibration Measurements

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***Abstract*—**Early pre-code reinforced concrete structures present undetermined resistance to earthquakes. This situation is particularly unacceptable in the case of essential structures, such as healthcare structures and pilgrims' houses. Amongst these, an existing old RC building in Madinah city (KSA) is seismically evaluated with and without infill wall and their dynamic characteristics are compared with measured values in the field using ambient vibration measurements (AVM). After updating the mathematical models for this building with the experimental results, three dimensional pushover analysis (Nonlinear static analysis) was carried out using commercial structural analysis software incorporating inelastic material properties for concrete, infill and steel. The purpose of this analysis is to evaluate the expected performance of structural systems by estimating, strength and deformation demands in design, and comparing these demands to available capacities at the performance levels of interest. The results summarized and discussed.

***Keywords*—**Seismic Assessment, Pushover Analysis, Ambient vibration, Modal update.

## I. Introduction

THE Western region of Saudi Arabia lies in low to moderate seismicity regions and seismic events of magnitude 5.7 were recorded in 2009 in areas near the holy city of Madinah city (KSA) [1]-[3]. Majority of the structures built in Saudi Arabia in the seismically active western region are designed primarily for combination of gravity and wind loads with no consideration of seismic loading. Non-ductile detailing practice employed in these structures makes them prone to potential damage and failure during earthquake. Therefore, analysis of such buildings is required to gain insight of these seismic performances.

A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces, which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads, various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic nonlinear force displacement relationship can be determined.

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Nonlinear static procedure is simply based on the assumption that the response of a structure can be related to the response of an equivalent single degree of freedom system.

This implies that the response is controlled by a single mode, or contributions of multiple modes, and that the shape of the selected modes remains constant throughout the time history response. Another simplification is the ‘pushover’ or ‘capacity’ curves, which form the foundation of the nonlinear static procedure. They are generated by subjecting a detailed structural model to one or more lateral load patterns (vectors) and then increasing the magnitude of the total load to generate a nonlinear inelastic force deformation relationship for the structure at a global level. In the Coefficient Method of FEMA356 [4], the global parameters are normally base shear and roof displacement. In the Capacity-Spectrum Method of ATC40 [5], base shear and roof displacement are transformed to spectral acceleration and spectral displacement.

To estimate seismic demands in the design and evaluation of buildings, the nonlinear static procedures using the lateral force distributions recommended in ATC-40 and the FEMA-356 documents are now standard in engineering practice. The nonlinear static procedure in these documents is based on the capacity spectrum method (ATC-40) and displacement coefficient method (FEMA-356), and assumes that the lateral force distribution for the pushover analysis and the conversion of the results to the capacity diagram are based on the fundamental vibration mode of the elastic structure. Consequently, these nonlinear static procedures based on invariant load patterns provide accurate seismic demand estimates only for low- and medium-rise moment-frame buildings where the contributions of higher `modes' response are not significant and inadequate to predict inelastic seismic demands in buildings when the higher `modes' contribute to the response [6], [7]. To overcome these drawbacks, an improved pushover procedure, called modal pushover analysis (MPA), was proposed by [8] to include the contributions of higher modes. The MPA procedure has been demonstrated to increase the accuracy of seismic demand estimation in taller moment-frame buildings compared to the conventional pushover analysis [9]. MPA procedure considering for the first few (two or three) modes contribution are typically sufficient.

With the increase in the number of alternative pushover analysis procedure proposed in recent years, it is useful to assess the accuracy and classify the potential limitations of these methods. Chopra and Chintanapakdee [10] investigated an assessment on accuracy of MPA and FEMA pushover analyses for moment resisting frame buildings. Then, an investigation on the accuracy of improved nonlinear static procedures in FEMA-440 [11] was carried out by [12]. Meanwhile, the ability of FEMA-356, MPA and AMC in estimating seismic demands of a set of existing steel and reinforced concrete buildings was examined by [13]. More recently, an investigation into the effects of nonlinear static analysis procedures which are the Displacement Coefficient Method recommended in FEMA 356 and the Capacity Spectrum Method recommended in ATC 40 to performance evaluation on low-rise RC buildings was carried out by [14].

Modal identification of existing buildings through the analysis of in-situ vibration measurements became a classic procedure for providing modal characteristics of a building, for studying the seismic response of buildings and even for damage detection. Modal characteristics are often identified from ambient vibration measurements (AVM) and from seismic records. Ambient vibration measurements are generally preferred to non-destructive forced vibration measurement techniques for obtaining the modal parameters of large structures for many reasons. A structure can be adequately excited by wind, traffic, and human activities and the resulting motions can be readily measured with highly sensitive instruments. Expensive and cumbersome devices to excite the structure are therefore not needed. Consequently, the overall cost of the measurements conducted on a large structure is reduced.

Ambient vibration measurements of many buildings have been recorded across the world in the past to determine their dynamic properties, in particular, to ascertain the properties of the fundamental modes of vibration, [15]-[18]. It is also recognized that the experimental data from one region may not be used in another owing to the differences in the construction methods and materials. Crawford and Ward [19] and Trifunac [20] showed that ambient vibration-based techniques were as accurate as active methods for determining vibration modes and much easier to implement for a large set of buildings. A seismic vulnerability assessment of the old buildings has been studied, as well as of the major non-structural components. The structural seismic vulnerability assessment stages comprised the development of linear dynamic and nonlinear static numerical models for an old building used as pilgrims' houses is carried out. Taking the effect of infill walls as it is an exciting building, two models is build and compared. The first model considers the primary lateral-resisting system of the structure as well as flooring slabs. The second model adds infill walls as strut models. Structure model is updated using field measurement of building's dynamic properties by using ambient vibration techniques. The static nonlinear analysis for two models is carried out. The results are summarized and discussed.

## II. Features of the Building

The structure is an existing five-story reinforced concrete moment frame building in Madinah. The building is used as a hotel. The location of the building and plan of a typical story above basement are shown in Figs. 1 to 4. Fig. 5 shows plan and elevation for building dimensions.


Fig. 1 Position of building in Madinah city from Google

Fig. 2 Elevation of the case study building in Madinah

Fig. 3 Side view of the case study building in Madinah

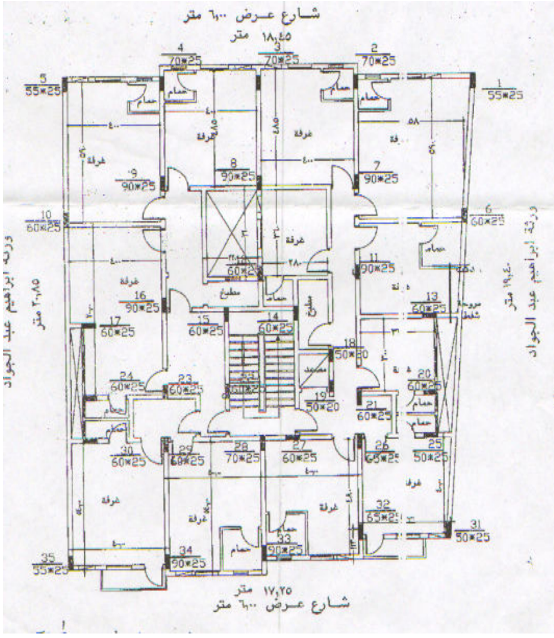

Fig. 4 Typical floor plan of the case study building in Madinah

TABLE I
MATERIAL PROPERTIES FOR BUILDING

| | | |
|---|---|---|
| concrete strength* | 20000 kN/m² | F'c |
| rebar yield strength | 243700 kN/m² | Fy |
| modulus of elasticity of concrete | 20000000 kN/m² | Ec |
| modulus of elasticity of rebar | 2.0E+8 kN/m² | Es |
| Shear modulus | 10356491 kN/m² | G |
| Poisson's ratio | 0.2 | Y |

*There properties were obtained from test on drilled concrete core specimens.

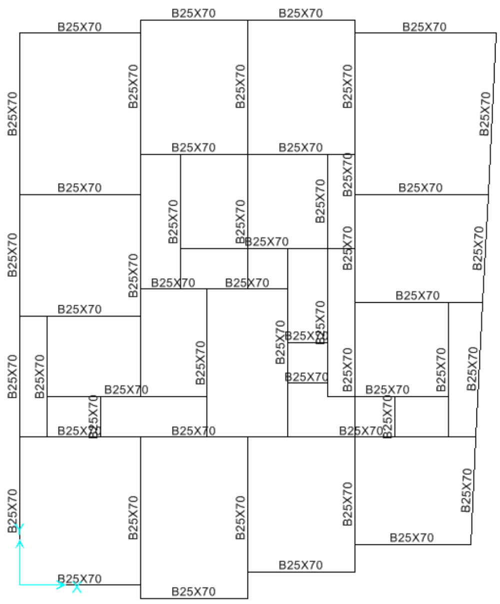


Fig. 5 (a) Typical Plan

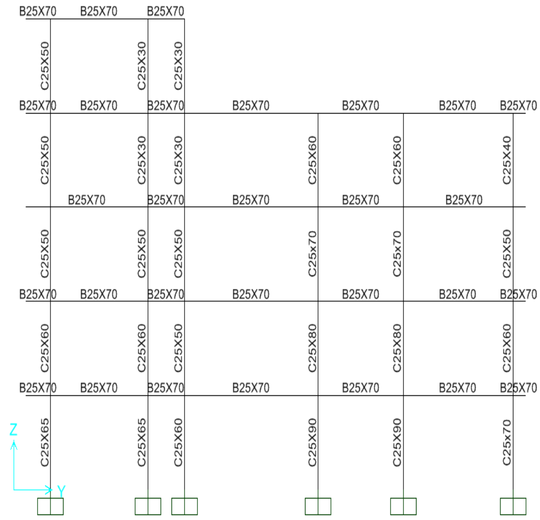


Fig. 5 (b) Elevation

Fig. 5 Reinforced concrete building

This 5-storey R C building is representative of old building types constructed in Madinah City more than 30 years ago. These buildings are consisting of reinforced concrete skeleton i.e. columns, beams and solid slab. The thickness of brick walls are almost equal 0.12 m and the storey height is about 3.00 m.

Material properties and reinforced Concrete Member Sizes and Reinforcement for the building are illustrated in Table I and Fig. 5 respectively. Stress-strain curves for concrete, steel bares and brick wall are illustrated in Fig. 6.

## III. LOADING ASSUMPTIONS

1) Total Dead Load (D) is equal to DL+SDL+CL
2) Dead Load (DL) is equal to the self-weight of the members and slabs.
3) Super-imposed Dead Load (SDL) is equal to 3.0 kN/m². SDL includes partitions, ceiling weight, and mechanical loads.
4) Cladding Load (CL) is applied only on perimeter beams.
5) Live Load (L) is equal to 2.0 kN/m².

Table II shows the total static loads for RC building due to EQ and Wind load cases according to Saudi Code for Loads and Forces - (SBC 301) (2008). The results in this table show that the EQ loads are the dominant in design.

TABLE II
TOTAL STATIC LOADS FOR RC BUILDING DUE TO EQ AND WIND LOAD CASES

| Case | load (kN) | factored load (kN) |
|---|---|---|
| EQX | 873 | 873 |
| EQY | 873 | 873 |
| Wind x | 257 | 411 |
| Wind y | 208 | 332 |

Factor loads for EQ=1.0 and for W=1.6 according to Saudi code (SBC301-2008).

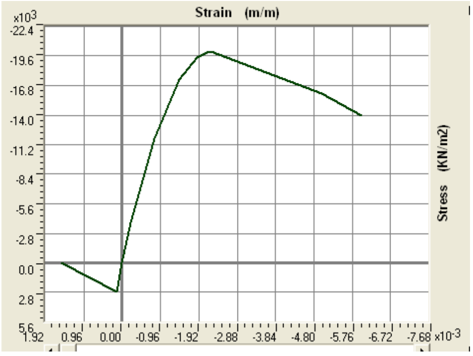


Fig. 6 (a) Stress-strain curve for concrete

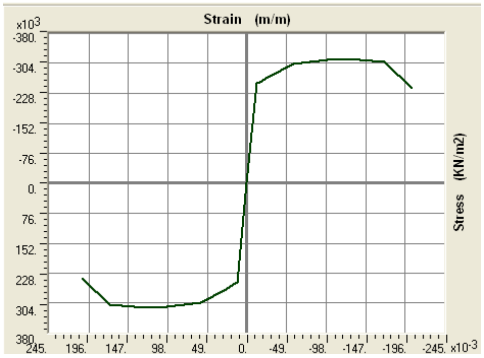


Fig. 6 (b) Stress-strain curve for steel bare

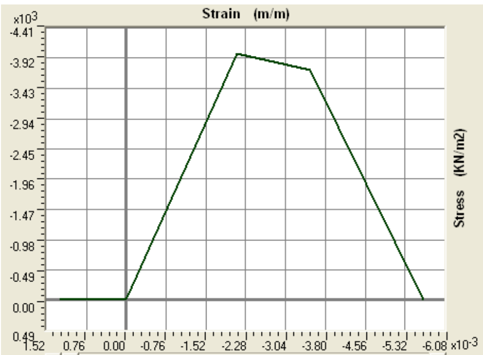


Fig. 6 (c) Stress-strain curve for clad brick

Fig. 6 Stress-strain curves introduced in SAP2000

## IV. Mathematical Models

For the five stories building, two mathematical models, Model I and Model II, were created using SAP2000 [21]. These two models are shown in Figs. 7 and 8 respectively.

## V. Results and Discussions

### *A. Experimental and Theoretical Frequencies as well as Mode Shapes*

A validation of the proposed structural numerical models for this 5-storey RC building can be achieved by comparing the experimentally measured and the analytically estimated natural frequencies.

Experimentally, eight server-type accelerometers with relevant signal conditioners were used for ambient response measurement. The measurements were performed at the four corners of plan on the top floor of the building and sufficient response signal were obtained. From the measured signal records and their normalized power spectra, the fundamental frequencies and the corresponding mode shapes in transverse, longitudinal and tensional directions were determined according to ambient vibration measurements procedure explained by [22].

Theoretically, a study has been conducted to assess fundamental transverse, longitudinal and tensional periods of the 5storey R C building and to determine the effect of considering non-structural elements (infill walls) in structural model. Modal analysis has been carried out for two different models of the building using SAP2000 program. These models are a) Model I (frame elements without infill wall), b) Model II (frame elements with infill walls).

Table III summarizes the first three natural periods measured for the building i.e. 0.32 sec, 0.27 sec and 0.24sec. The corresponding transverse, longitudinal and tensional mode shapes are illustrated in Fig. 9. Figs. 10-12 show that the corresponding mode shapes in transverse, longitudinal and coupled directions are similar for Model II. The corresponding mode shapes for Model I are completely different. Table IV summarizes the first six natural periods calculated for the two models of the building.

TABLE III
Measured Modes for the Building

| Mode | Modal parameters MDOF | |
|---|---|---|
| | **Type** | **T (sec.)** |
| **1** | Translation X | 0.32 |
| **2** | Translation Y | 0.27 |
| **3** | Coupled | 0.24 |

TABLE IV
Theoretical Modes for the Building

| Mode number | Eigen values from modal analysis T (sec.) | |
|---|---|---|
| | Model I (frame elements without infill wall) | Model II (frame elements with infill wall) |
| 1 | 0.950<br>Not Pure Coupled | 0.323<br>First Translation X |
| 2 | 0.902<br>Not Pure coupled | 0.268<br>First Translation Y |
| 3 | 0.637<br>First Translation Y | 0.246<br>First Coupled |
| 4 | 0.346<br>Second Coupled | 0.117<br>2-Trans X+ Coupled |
| 5 | 0.334<br>Second Coupled | 0.102<br>2-Trans Y +Coupled |
| 6 | 0.239 | 0.098 |

Fig. 10 shows that the first period is 0.950 sec, 0.323 sec and 0.325 sec for Model I, and Model II respectively. Fig. 11 shows that the second period is 0.902 sec and 0.268 for Model I, Model II respectively. Similarly, Fig. 12 shows that the third period is 0.637 sec and 0.246 sec Model I, and Model II respectively.

From the analysis investigations presented in Figs. 9-12, the following remarks can be seen:

- A good agreement was found between the experimentally measured periods and the numerically calculated periods with the infill wall "Model II". The corresponding mode shapes in transverse, longitudinal and tensional directions are similar.
- Modeling the building without infill wall, Model I, gives different results for both period values and corresponding mode shapes. The first and second periods i.e. 0.950 sec and 0.902 sec are torsion modes while the third period i. e. 0.637 sec is transverse mode in Y direction.
- Modeling the building with infill wall, Model II shows the importance of contribution of infill walls in changing dynamic characteristic of the building. The existing infill walls have been adjusted to give results similar to those obtained in field.
- By considering the above facts, the main results of the study is that the contribution of infill walls should be carefully judged by considering the importance of them in changing dynamic response and collapse status of existing RC structures.

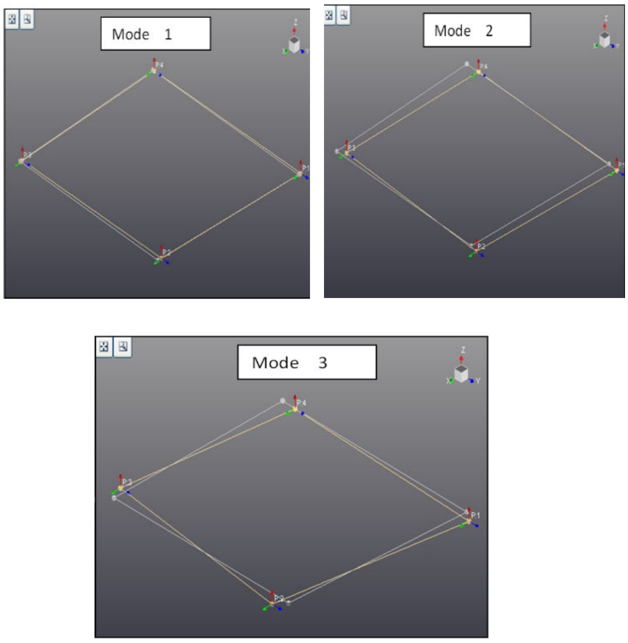


Fig. 9 Experimental mode shapes for the building

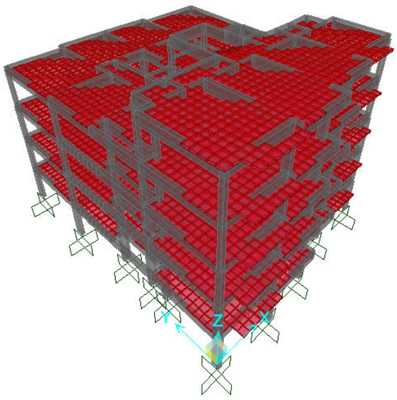

Fig. 7 Model I for the building (frame element + slab)

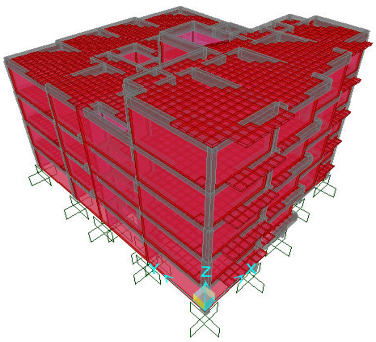

Fig. 8 Model II for the building (frame element + slab + clad)

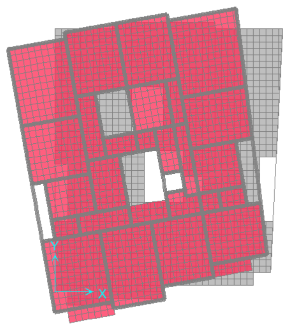

Fig. 10 (a) T1=0.950 sec (Model I) no infill wall

Fig. 10 (b) T1= 0.323 sec (Model II) infill wall

Fig. 10 Theoretical mode shape (1) for different models for the building

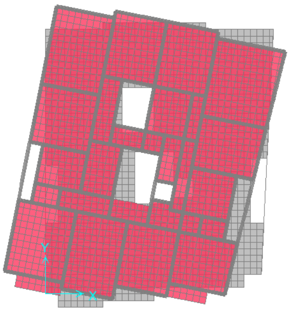

Fig. 11 (a) T2=0.902 sec (Model I) no infill wall

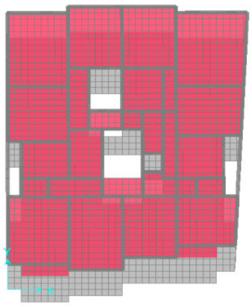

Fig. 11 (b) T2=0.268 sec (Model II) infill wall

Fig. 11 Theoretical mode shape (2) for different models for the building

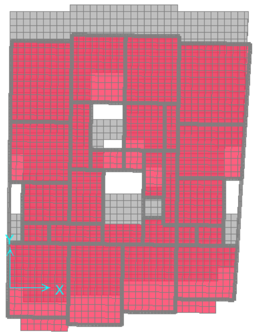

Fig. 12 (a) T3= 0.637 sec (Model I) no infill wall

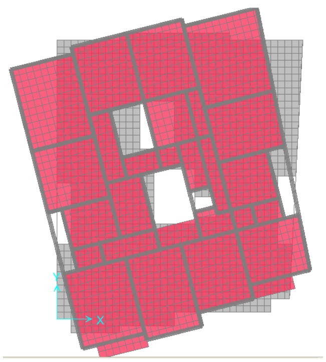

Fig. 12 (b) T3=0.246 sec (Model II) infill wall

Fig. 12 Theoretical mode shape (3) for different models for the building

## VI. Hinge Status at Target Displacement for Pushover Analysis of RC Building

The lateral load pattern in Madinah City corresponding to the Saudi Building Code - Structural requirements for Loads and Forces - (SBC 301-2008 [23]) is adopted and applied as auto lateral load pattern in SAP2000. The load pattern is calculated using DL+SDL+0.25LL for the EQ load case. The direction of monitoring the behavior of the building is same as the push direction. In case of columns, program defined auto PM2M3 interacting hinges are provided at both the ends according to FEMA 356, while in case of beams, M3 auto hinges are provided.

In this study, displacement-controlled pushover analyses were performed on the two models for 5storey RC building using SAP2000 program in order to determine the performance level and deformation capacity (capacity curve).

Columns isometric shapes for hinge status at target displacement for the two studied models are illustrated in Figs. 13 and 14 for XX and YY directions respectively. From these figures, it's observed that:

– In case bare frame Model I, Figs. 13 (a) and 14 (a), all columns are in IO-CP range (i.e. immediate occupancy to collapse prevention range) and plastic hinges are distributed at all stories and there will be sever damages during earthquake.
– In case of considering masonry wall, Model II, Figs. 13 (b) and 14 (b), most plastic hinges for columns are concentrated at lower stories and in B range (i.e. operational range) which is acceptable criteria for hinges

1. The lateral load resisting mechanism of the masonry infill frame is essentially different from that of the bare frame. The bare frame, Model I acts primarily as a moment resisting frame with the formation of plastic hinges at the joints under lateral loads. In contrast, the infill frame, Model II behaves like a braced frame resisted by a truss mechanism formed by the compression in the masonry infill panel and tension in the column.
2. The above results show that modeling building with infill walls has greater strength as compared to building without infill walls. The presence of the infill walls increases the lateral stiffness considerably. Due to the change in stiffness and mass of the structural system, the dynamic characteristics change as well. The total storey shear force increases considerably as the stiffness of the building increases in the presence of masonry infill. This is useful to understand the contribution of infill walls in formation of plastic hinges in beams and columns in multistory frame.

Figs. 15 and 16 show the building capacity response up to failure for the two studied models in X direction and in Y direction respectively. The strength and stiffness of the in-filled frame is significantly increased due to the presence of infill, but the displacement capacity decreases, which is evident from the displacement profiles in these figures.

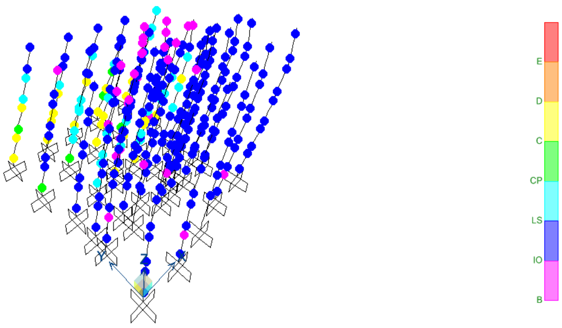


Fig. 13 (a) Model I (frame element +slab)

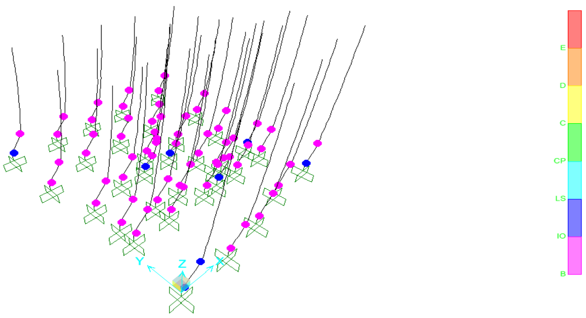

Fig. 13 (b) Model II (frame element +slab+ infill walls)

Fig. 13 Columns isometric shape for hinge status at target displacement, static nonlinear analysis XX

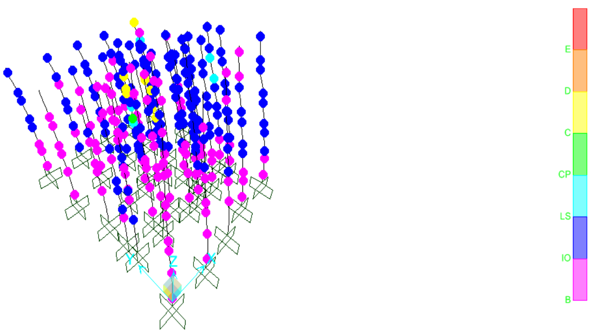


Fig. 14 (a) Model I (frame element +slab)

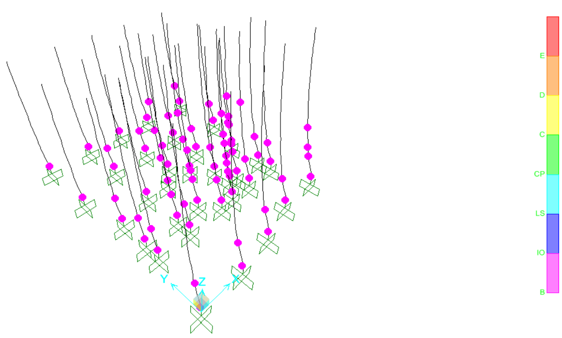

Fig. 14 (b) Model II (frame element +slab+ infill walls)

Fig. 14 Columns isometric shape for hinge status at target displacement, static nonlinear analysis YY

The maximum base shear ($V_B$) and target displacement ($\delta$) values for the two different models are summarized in Table V. Table VI shows that the ratio of base shear of Model II (with infill walls) to the corresponding value of base shear for Model I (without infill) are 1.44 and 1.34 in X and Y directions respectively.

TABLE V
BASE SHEAR AND TARGET DISPLACEMENT VALUES FOR THE TWO MODEL

| Case | Target Value | Model I (No clad) | Model II (infill walls) |
|---|---|---|---|
| Case x-x | $V_B$ (kN) | 7056 | 10178 |
| | $\delta$ (m) | 0.243 | 0.026 |
| Case y-y | $V_B$ (kN) | 11140 | 14954 |
| | $\delta$ (m) | 0.099 | 0.027 |

TABLE VI
THE RATIO OF BASE SHEAR FOR MODEL WITH INFILL TO MODEL WITHOUT INFILL IN X OR Y DIRECTIONS

| Case | Base shear for Model II / Base shear for Model I |
|---|---|
| Case x-x | 1.44 |
| Case y-y | 1.34 |

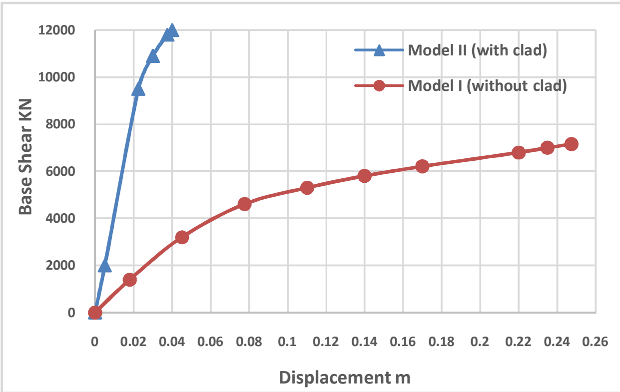


Fig. 15 Comparison of pushover curves for the two models, static nonlinear analysis X-X

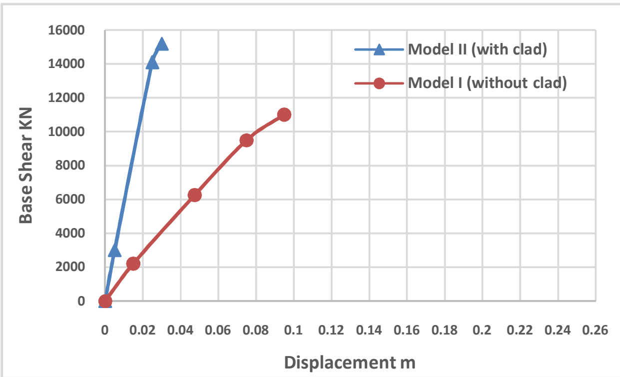


Fig. 16 Comparison of pushover curves for the two models, static nonlinear analysis Y-Y

The response modification factor (R) for the 5 story RC building is evaluated from capacity and demand spectra (ATC-40). The capacity diagram and the demand diagram are shown in Figs. 17 and 18 in X and Y directions for Model I and Model II respectively. The results indicate that:

For Model I: (Frame Elements without Infill Wall),

- The performance base shear V performance is 1085kN and 1538 kN in X and Y directions respectively.
- The lowest resultant response reduction factor R equals 2.04.

For Model II: (Frame Elements with Infill Wall),

– The performance base shear V performance is 2953kN and 3473kN in X and Y directions respectively.
– The lowest resultant response reduction factor R equals 4.55.

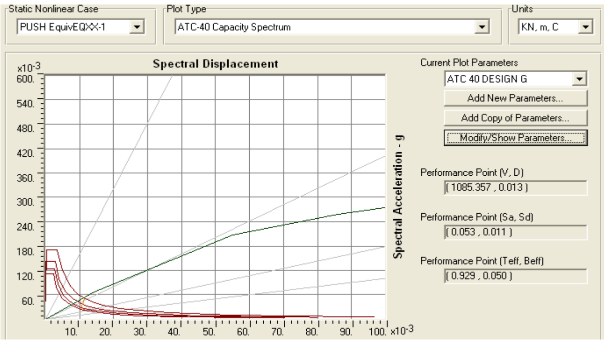


Fig. 17 (a) Model I (frame element +slab)

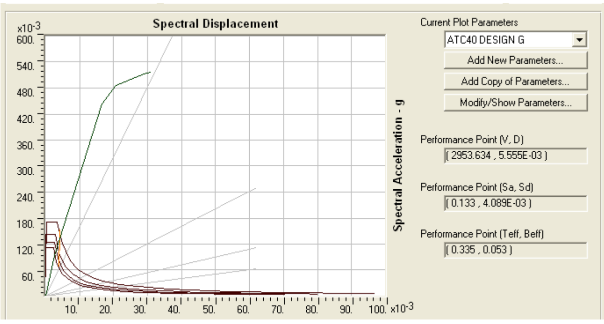


Fig. 17 (b) Model II (frame element +slab+ infill walls)

Fig. 17 ATC40 Capacity spectrum, EQX, design spectrum function in Madinah

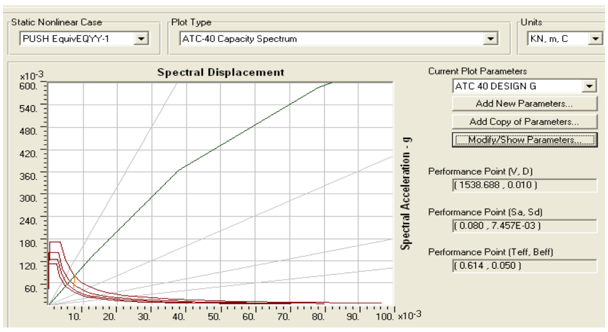


Fig. 18 (a) Model I (frame element +slab)

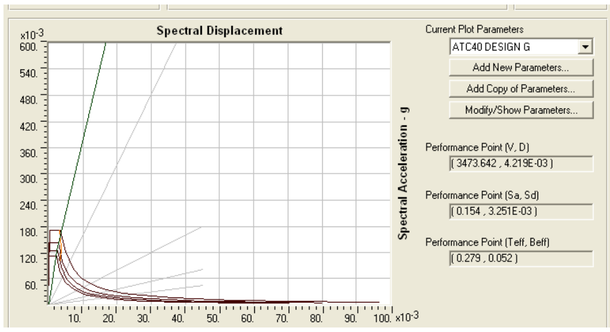


Fig 18 (b) Model II (frame element +slab+ infill walls)

Fig. 18 ATC40 Capacity spectrum, EQY, design spectrum function in Madinah

The following comments for the above results can be deduced:

1. The total shear force increases considerably as the stiffness of the building increases in the presence of masonry infill. The lateral load resisting mechanism of the masonry infill frame is essentially different from the bare frame. The bare frame acts primarily as a moment resisting frame. In contrast, the infill frame behaves like a braced frame resisted by a truss mechanism formed by the compression in the masonry infill panel and tension in the column.
2. The values of response modification factor R as per international standards (Saudi Building Code SBC 301-2008 and ASCE7-10 [24]) for ordinary reinforced concrete moment frame is 2.5. This means that:

– Model I (frame elements without infill wall) does not satisfy the code requirements for response modification factor R.
– Including infill wall in the analysis, Model II (frame elements with infill wall as strut elements), increase the stiffness of the building and give higher value of R satisfying the code requirements.

## VII. Conclusion

The ambient vibration measurements (AVM) on buildings have provided valuable data for the validation and updating of the detailed finite element models.

Performance-based seismic vulnerability assessment of existing structures based on updating finite element models by using AVM can greatly benefit from possible modal identification stages that can show unexpected strength and stiffness contribution of secondary structural or non-structural components. In the particular case of early RC structures, strong masonry walls, either facade or partition walls, have a significant stiffening effect that greatly determines the early nonlinear stages and can, as was the case, lead to a sudden drop of strength (and stiffness).

## REFERENCES


[1] J. Roobol (2007) "Cenozoic faults in Western Saudi Arabia" In: 7th meeting of the Saudi society for geosciences, King Saudi University, Riyadh, Saudi Arabia.
[2] M. Al-Saud (2008) "Seismic characteristics and kinematic models of Makkah and central Red Sea regions" Arab J Geosci 1:49–61.
[3] S. Aldamegh, H. Moussa, S. Al-Arifi and M. Moustafa (2012)" Focal mechanism of Badr earthquake, Saudi Arabia of August 27, 2009" Arab J Geosci (2012) 5:599–606.
[4] Federal Emergency Federal Agency, FEMA-356, (2000), "Pre-standard and Commentary for Seismic Rehabilitation of Buildings", Washington DC.
[5] ATC-40 (1996) "Seismic Evaluation and Retrofit of Concrete Building "Report, Applied Technology Council. Redwood City, California.
[6] C. Chintanapakdee and A. K. Chopra (2003) "Evaluation of modal pushover analysis using generic frames" Earthquake Engineering and Structural Dynamics; 32(3): 417-42.
[7] S. K. Kunnath and E. Kalkan (2004) "Evaluation of seismic deformation demands using nonlinear procedures in multistory steel and concrete moment frames" ISET Journal of Earthquake Technology 41(1):159_82.
[8] A. K. Chopra, A.K. and R. K. Goel, (2002) "A modal pushover analysis procedure for estimating seismic demands for buildings" Earthquake Engineering and Structural Dynamics 31(3):561-82.
[9] A. K. Chopra, R. K. Goel and C. Chintanapakdee (2004) "Evaluation of a modified MPA procedure assuming higher modes as elastic to estimate seismic demands" Earthquake Spectra;20(3):757-78.
[10] A. K. Chopra and C. Chintanapakdee (2004) "Evaluation of modal and FEMA pushover analyses: vertically regular and irregular generic frames" Earthquake Spectra; 20(1):255-71.
[11] Federal Emergency Federal Agency, FEMA-440, "Improvement of nonlinear static seismic analysis procedures" Federal Emergency Management Agency, Washington, D.C., 2005.
[12] S. Akkar and A. Metin (2007) "Assessment of improved nonlinear static procedures in FEMA-440" Journal of Structural Engineering, ASCE; 133(9):1237_46.
[13] E. Kalkan and S. K. Kunnath (2007) "Assessment of current nonlinear static procedures for seismic evaluation of buildings". Engineering Structures; 29(3): 305-16.
[14] E. Irtem and U. Hasgul (2009) "Investigation of effects of nonlinear static analysis procedures to performance evaluation on low-rise RC buildings" Journal of Performance for Constructed Facilities, ASCE; 23(6):456_66.
[15] S. Midorikawa (1990) “Ambient vibration test of buildings in Santiago and Viña del Mar”, a report on the Chile-Japan joint study project on seismic design of structures, The Japan international cooperation agency.
[16] C. Negulescu, R. Radoi, and A. Aldea (2004) "Micro tremor measurement for evaluation of building vibration characteristics", Report, National Center for Seismic Risk Reduction, Bucharest, Part 1 - 23p. and Part 2, - 20p
[17] C. Michel, P. Guéguen and P. Y. Bard (2008)"Dynamic parameters of structures extracted from ambient vibration measurements", an aid for the seismic vulnerability assessment of existing buildings in moderate seismic hazard regions, Soil Dynamics and Earthquake Engineering; 28 (8), 593-604.
[18] S. Demetriu and A. Aldea (2012)"Modal parameters of RC frame structure identified from ambient vibration measurements" 15 WCEE, Lisbona.
[19] R. Crawford, and H. S. Ward (1964) “Determination of the Natural Period of Buildings,” Bulletin of the Seismological Society of America, Vol. 54, No. 6, pp. 1743-1756.
[20] M. D. Trifunac (1972) “Comparison between Ambient and Forced Vibration Experiments,” Earthquake Engineering and Structural Dynamics, Vol. 1, pp. 133-150.
[21] SAP2000 (2011) Integrated Software for Structural Analysis & Design. Computers & Structures, Inc., Berkeley, California, USA.
[22] T. M. Alguhane (2014) "Monitoring of Buildings Structures in Madinah", Ph.D., Ain Shams University Faculty of Engineering 2014.
[23] Saudi Building Code - Structural requirements for Loads and Forces - SBC 301 (2007).
[24] ASCE 7-10 (2010) "Minimum Design Loads for Buildings and Other Structures", Published by the American Society of Civil Engineers, ASCE/SEI 7-10.