Effects of Internal Embedded Composite Steel Frame on Reinforced Concrete Walls Behavior Seismically Excited out-of-Plane, Experimental Study

F. Y. Al-Ghalibi, and Laith K. Al-Hadithy

Abstract— In the modern era, especially after middle of 19th century, the utility of concrete walls became widespread as a type of structural engineering applications. Various wall types and many construction methods involve building construction. Generally, structural walls can be classified according to load sources and directions. Building behavior can be improved with many engineering advantages that achieved using reinforced concrete walls as structural elements. Reinforced concrete walls increase the building stiffness, strength, ductility, seismic energy dissipation, plastic collapse resistance, and minimize seismic risk by changing the building failure mode. The current investigation deals, experimentally, with the nonlinear dynamic response of reinforced concrete walls including opening effects and role of internal embedded steel frame.

Eight flanged-section framing wall prototypes was experimentally subjected to seismic excitations to prepare a comparative study of composite embedded steel frame influence and opening effects. Experimental results showed that using internal embedded steel frame significantly improved the performance of reinforced concrete walls, for example the displacements of composite walls decreased by an average of 15.85% as compared with non-composite reinforced concrete walls, the velocities decreased by an average of 32.87%, the acceleration dropped by an average of 56.09%, and the concrete strain dwindled by an average of 50.88%. Also, the study introduced the effects of wall openings by percent increasing of the displacement of 52.44% if the wall contains an opening in the first story, while the percent increase of 60.92% if the wall contains an opening in the second story, and the percent increase will be 62.63% if the wall contains an opening in the first and the second story.

Key words— Embedded steel frame, seismic excitation, reinforced concrete walls, out-of-plane excitation, opening effects.

1 INTRODUCTION

Every year, building structures around the world are frequently hit by earthquakes causing several damages which may be loaded to collapse. The dynamic load can be defined as a load that its value varies with time. Therefore, the loads, displacements, and many other parameters can be represented mathematically as functions with time. The study of structural responses resulting from these dynamic forces is called structure dynamics. When the loads are applied in a very slow manner, the forces are considered static and independent on time. The loads are considered cyclic when applied with high frequency, whereas they are considered repeated loads when applied with low frequency.

The earthquake releases an energy which is generated by a sudden and random movement of earth segments (plate tectonics). The energy is released because of ground vibration whose amplitude is reduced with rupture distance. In addition, the earth vibration generates large random inertia forces that should be carried by the structural components safely.

Generally, the collapsed seismic force level depends on the nature of the region where the construction is to be built.

- *F.Y. Al-Ghalibi is currently pursuing doctor degree program in civil engineering in Al-NahrainUniversity, Iraq, PH-+9647819877878.furat.alghalibi@uokufa.edu.iq*
- *Laith K. Al-Hadithy is currently staff member in civil engineering dept.in Al-Nahrain University, Iraq, PH-+9647705362710.*

Structure geographic location plays a major role in seismic analysis and design of structures since the global seismicity is influenced by the earthquake hypocenter and plate tectonics nature.

An earthquake occurs if earth tectonic plate shifts and the mass of earth materials move with plates undergo interface stresses. The aim of the seismic design is to ensure that the structural elements are adequate to resist the released dynamic forces and to keep the structure to certain damage near the collapse. Depending on such failure criteria, the structure seismic designer can keep the people who occupy the damaged structure in a safe state. The engineering solution to reduce the lateral vibration is by providing viscous dampers installed under the structural elements. Such devices absorb ground vibrations and minimize the earthquake-released energy. Another active way to absorb earthquake energy is to attach tuned mass dampers which disperse the released energy direction away from the structure energy, and the effects of damper mass motion will render structure risk motion a vanity.

The composite walls can be constructed using several methods which can be listed as follows:

1- Composite walls with fully embedded steel sections.
2- Composite walls with partially embedded steel sections.
3- Composite walls with internal encased bracings.
4- Composite walls using external steel plates.
5- Composite walls using FRP sheets.

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In the present study, the composite walls are considered as fully embedded steel sections and the nonlinear behavior due to external seismic excitation is investigated including wall opening effects. The study is intended to deal with the nonlinear behaviour of reinforced concrete walls with and without fully embedded composite steel frames under the effects of out-of-plane seismic excitation. The purpose of the internal embedded steel frame is to improve reinforced concrete wall behaviour and to increase wall ductility, stiffness, strength, and wall lateral resistance. Also, as a function and architectural requirements, effects of an opening in the wall have been studied. The study demonstrates behavior comparisons between reinforced concrete walls containing or not containing embedded steel frames, also studies improvements of the reinforced concrete walls behaviour when composed encased steel frames are used. Furthermore, it develops the reinforced concrete walls dynamic response behaviour to be able to absorb the released internal forces of the selected seismic signal.

Generally, researchers, who are interested in wall type selection, studies on important parameters influencing wall responses such as stiffness, drift, force distribution in the structural elements, etc. As a benchmark of wall analysis, Fischinger, M., and Isakovic, T., (2000) discussed the seismic response behavior of five-story rectangular cross-section concrete wall with low reinforcement ratio affected by three earthquake types with different intensities. They modelled the wall using a multi-vertical line element (MVLE). Parametric study and results showed that the response was affected by many factors such as wall initial stiffness, three-test sequence, damping ratio, and strain hardening. The used model gave good accuracy for both stiffness and strength prediction. Also, the model gave closely failure prediction. They drew many important conclusions such as the effects of using viscous damping necessary to reduce excitation level. Their study depended on 5% of damping ratio while the appropriate value was 2%. Therefore, 5% gives safe and underestimated response. The time-history response affected by initial damages resulted from the sequence of previous tests. The response was affected by reinforcement pull out in the inelastic stage, and inelastic shear deformations. It is important to consider the unstable response after yielding of wall reinforcement.

Waugh, J., et-al (2008) explained and investigated in a report the nonlinear structural analysis of rectangular and T-shaped concrete walls. They introduced the methods of analyses and listed some literature review of the previous researcher works in wall analysis field. They also described in detail many concrete models. Experimental analysis, test set-up, and observations with the loading protocol and results comparison were illustrated for pre and post-test. Numerical analysis depending on OpenSees program was implemented. Compared results showed 5-10 % difference between numerical and experimental analysis. Generally, results showed that the fibre-based modelling was more adequate to simulate concrete walls numerically.

Xiaolei, H., et al. (2008) used a cyclic load test with various reinforcement amounts and construction details. Their study presented shear wall numerical analysis using a shear wall nonlinear analysis program (SWNA). It is a developed copy of OpenSees computer program using multi-vertical line element method (MVLEM). Good accuracy between analytical and experimental results was obtained. They concluded that (MVLEM) is an adequate method to simulate the nonlinear behavior of shear wall. The powerful of (MVLEM) is to use fewer DOFs. Therefore, the use of (MVLEM) is to reduce the required analysis time for modelling nonlinear inelastic analysis of shear walls. The shear wall transformation to micro-scale model using (MVLEM) is more practical by changing the problem from material to geometrical nonlinearity.

Energy dissipated in economical walls by increasing shear wall ductility using slit wall with shear connectors was studied by Baetu, S., and Ciongrad (2011) by conducting numerical simulations of nonlinear 3D finite element analysis using ANSYS12 software. Their idea of increasing wall energy dissipation was by achieving the yield point of shear connectors. They proposed that after yielding, the wall behaves as passive control resulting in damping system. The damping system of the global structure with slit wall increased and, accordingly, the global structure ductility increases after the yielding of the shear connector. They stated that for high-intensity seismic zones, slit walls were more adequate since the shear connectors yield point. As a comparison between walls with and without slits, they found that the structure collapsed and the wall base plastic hinge was prevented under extreme seismic load because the high ductility provided to the structure after yielding of shear connectors. They obtained good accuracy between experimental and numerical analysis. Dae-Han, J., (2014) studied reinforced concrete shear wall fibre element (mesoscale) to model the nonlinear flexural response. He, compared the analytical model with the experimental results of a high axial compression force rectangular cross-section shear wall. In fibre element, wall cross-section and reinforcement details have remained as its original case. Shear deformations were taken into account by including nonlinear shear spring element. In the fibre element model, the author divided the wall cross-section into vertical fibre slices that represented reinforcement and concrete by considering stress-strain relationship for each slice. Wall behavior for axial forces and bending moments were modelled by fibre slices. Depending on the stress-strain relationship of each slice, the stress at each fibre slice can be evaluated and the bending moment can be calculated using section equilibrium. Section equilibrium is conducted by making externally moments (from applied load) and internally moments (from fibre slices) be equal at the wall centre. Shear behavior of the wall was expressed by the shown three springs. The central spring represents the wall in-plane shear stiffness. The other two edge springs represent the wall out-of-plane shear stiffnesses. The validity of the analysis results showed good agreement and acceptable accuracy between fibre element model and test results.

The effects of different wall-end configurations were studied by El-Azizy, O., et-al. (2015) they presented experimentally rectangular, flanged, and boundary element type walls bent in a plane about their major axis under quasi-static

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cyclic loading. The study was conducted with different vertical and horizontal reinforcement ratios and various reinforcement details. Six half-scale reinforced concrete walls with different cross-section shapes. Wall ductility capacities, crack patterns, and the plastic hinge area near wall base were all investigated. They concluded that Canadian (CSA-A23.3-14) code provisions give an excellent approaching to the experimental results. Furthermore, their results showed that the ductile capacity of flanged and boundary section walls was approximately 33-40 % higher than a rectangular cross-section wall.

Shear walls can be divided into two classes: squat or short shear walls with aspect ratios of 2 or less, and slender or flexural shear walls with aspect ratios greater than 2 Martinelli, P. (2007).

Bhatt, G., et al (2017) conducted an analytical study on six different levels (heights) up to 10, 9, 8, 7, 6, and 5 stories of shear walls in medium rise structure (Ground + 9 stories). Indian Standards (IS) and response spectrum analysis (RSA) were used in the analysis. Their study discussed the optimum height or level of the shear wall. Results showed that the required height of the shear wall is up to mid-height of mid-rise or tall buildings. Therefore, it is not necessary to raise shear wall up to total (entire) building height.

Tso, W. K., and Biswas, J. K., (1972) proposed a simplified approximate method for estimating seismic analysis of coupled shear walls. First, the method approximately obtained the dynamic properties of the structure. Then Rayleigh principle was used to obtain the structural natural frequencies. The adopted structural mode shapes were calculated using vibrating cantilever modes. Response spectrum method was used to obtain an equivalent static load. The wall base shear and moments and connecting beams shear forces were calculated using continuum method of coupled shear wall. They found that a 5% error in the seismic analysis between proposed approximate method and those of actual dynamic results of coupled shear walls. Also, results showed that the approximate frequencies presented were less than 4% of the exact values. They concluded that the proposed study was adequate for practical and design use.

Coull, A., and Mukherjee, P., (1973) presented an approximate method to calculate dynamic modes and natural frequency of coupled shear walls. The dynamic differential equation was derived, and then written as integro-differential form. The solution adopted approximate numerical Galerkin method which was dependent on continuous connection solution approach. The wall was modelled by replacing its continuous medium to equivalent bending stiffness. Depending on such simplification, the system was transformed to single D.E., and the deformation modes required the first two natural frequencies presented in simple form. Also, few result terms are required to calculate the bending moments and shear forces with acceptable accuracy. Therefore, the proposed method was useful for hand calculation, practical use, preliminary calculations, and for rough checking with computer results.

Mukherjee, P., and Coull, A., (1973) studied free vibration of the coupled shear wall using an extended continuous connection method. Galerkin solution technique was adopted in the theoretical analysis for predicting the natural modes and frequencies. The study compared the theoretical and experimental results of model structures. The proposed theoretical model adopted two vertical cantilever shear walls connected by beam elements. Such a model was used to study the effects of openings perforated for doors and windows. Results indicated that the numerical solution of natural frequencies were more close to published experimental models.

Rutenberg, A., (1975) used Dunkerley's formula to predict good accuracy of the approximate natural frequency of coupled shear walls. A continuous medium assumption was adopted and the structure-deflected shape was calculated as the sum of two components. The first one was due to flexural cantilever action, and the second is due to cantilever shear-flexure action. The two systems were then combined using Dunkerley's formula. In addition, the study extracted an approximate formula with high accuracy even for high modes of natural frequencies of coupled shear walls.

Response spectrum seismic design charts of the frame-wall system were prepared by Basu, A., et al. (1984). The considered walls had constant and uniform properties along their high. The building wall modelled to be continuum with one shear beam. The proposed charts illustrated moments and shear forces along the building height using non-dimensional parameters. The charts were verified with a 15-story building example. They concluded that the resulting charts could be dependent as a simple and practical method of calculating design bending moments and shear forces resulting from earthquake loading for tall buildings comprising of solid walls and frames only. The 15-story building example gave more close and accurate results as compared with the proposed charts procedure. Aktan, A., and Bertero, V., (1984) presented an analytical and experimental studies of a 15-story coupled frame-wall structure. The structure was designed according to the Uniform Building Code (UBC-1973), (UBC-1979), and Applied Technical Council (ATC 3-06) provisions. Results showed that UBS-1973 provisions provided very good energy dissipation and inelastic deformations with a lower amount of steel reinforcement ratios. The study can be considered as a guideline of coupled shear walls component design and optimum structure design required to select the coupled shear wall girders stiffness and strength. A modification was required for shear design provisions and shear strength evaluation procedure for both UBC and ATC 3-06 codes. Coupling girders stiffness calculations should be included to account for the effects of fix-end rotations.

Moehle, J., (1984) conducted a seismic analysis of reinforced concrete frame-wall structures with small-scale buildings used as test specimens. The study included strong base motions of four irregular multistory buildings. Two types of inelastic response spectra were used to extract building inelastic response analysis results. Results indicated that good agreement between the analytical response spectrum method and the experimental results of maximum responses if initial stiffness was estimated accurately.

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In the present study, eight walls prototypes with two stories, each were tested using displacement-time seismic excitation signal shown in Fig. (11), the signal data was tabulated and considered as an input seismic data to the LabVIEW computer software and the shacking table moved accordingly. Two sets of walls were chosen and tested under the given time-displacement signal. The first set included four reference (control) prototypes which were without internal embedded composite section but with and without opening. The second set was as same as the first set but with internal embedded composite steel frame. The sensors chosen were motion sensors (ADXL 335 accelerometers), concrete strain gauges type (PFLC-20-11), and steel strain gauges type (KFGS-5-120-C1-11L5M2R) with data output sampling rate 1000 read per second.

2 Structural Walls

As a useful method, structural walls efficiently resist earthquake lateral loads in seismically active zones, reduce drifts, minimize structure damage, and increase structural strength and stability. Before and during the 19th-century, structural walls were of load-bearing type, but after that, structural stability provided by other common building frame elements were used as non-loaded bearing walls. Later, several wall types for different purposes (basement walls, retaining walls, firewalls, and partition walls) were used as structural members. Generally, walls can be classified according to load type and direction into three type. With reference to Plates (1) to (5), the main three types of wall are as follows:

a- Non-loaded bearing walls support their self-weight only as shown in Plate (1).

b- Loaded bearing walls, used in buildings to resist gravity loads with wind and earthquake forces as shown in Plate (1). Such walls support vertical loads with or without lateral moments about wall minor (weak) axis (out-of-plan forces). For this loading condition, column design criteria can be used to design this wall type.

c- Shear walls, Plate (2): In tall buildings, this type of walls is used to provide adequate lateral stiffness to resist gravity, wind, and earthquake forces. High stresses, side-sway, displacements, and vibrations would take place if shear walls are not provided for tall buildings. Deep cantilever beam criteria can be used to design such wall types because the lateral force resulted in a moment about wall major (strong) axis (in-plan forces). Shear wall provides excellent resistance by increasing the horizontal stiffness of building which reduces lateral displacement.

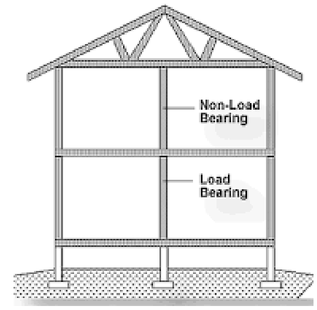

Plate (1) Non-loaded bearing and loaded bearing walls.

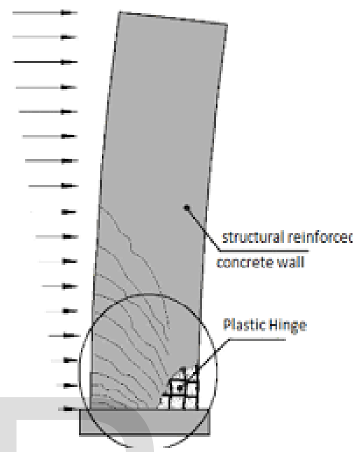

Plate (2) Shear wall (Moment about its major axis).

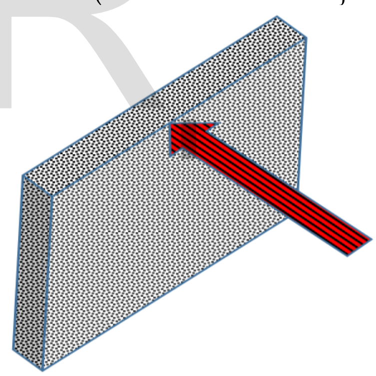

Plate (3) Wall bent about its minor axis (Out-of-plane bending).

Plate (4) Loaded bearing walls building.

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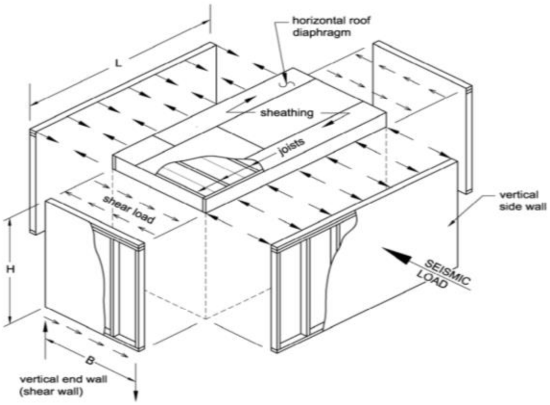

Plate (5) Perspective view of walls building illustrating load paths

TABLE (1)
DETAILS OF TESTED WALLS

Group name	Wall designation	Presence of embedded frame	Presence of opening in the 1st story	Presence of opening in the 2nd story
A	CONT1	without	without	without
	CONT2	without	included	without
	CONT3	without	without	included
	CONT4	without	included	included
B	COMP1	included	without	without
	COMP2	included	included	without
	COMP3	included	without	included
	COMP4	included	included	included

3 SCHEME OF THE EXPERIMENTAL PROGRAM

All eight tested walls had the same dimensions and flexural steel reinforcement. Each prototype had an overall height of 2280 mm, the clear height of each of the first and second stories was 1000 mm, the foundation depth was 200 mm and the slab portion thickness was 40 mm. Thicknesses of the walls in the first and the second stories were 50 mm and 40 mm, respectively. All prototypes had the same length which was 1000 mm and reinforced by BRC (50 x 50 x 3) mm. The test variables were the effect of using an embedded steel composite frame and the presence of opening. The walls were divided into two groups classified based on using composite embedded steel frames. The first group consisted of four control walls that were without internal embedded composite steel frames but with or without openings, the second group was as same as the first group but with internal embedded composite steel frame. A brief schedule for the eight prototypes constitution and geometries is given by Table (1). Meanwhile, front-views of the test prototypes showing locations of the accelerometers and concrete strain gauges are presented in Figs. (1) through (4), whereas profile of the steel frame embedded in each of the walls of group B prototypes is shown in Fig. (5). Moreover, vertical sections in typical profile of the two wall prototype groups are illustrated in Figs. (6) and (7), with their top views shown in Figs. (8) and (9). Finally, locations of the steel strain gauges on the steel frame of a typical wall-prototype classified is group B is presented in Fig. (10).

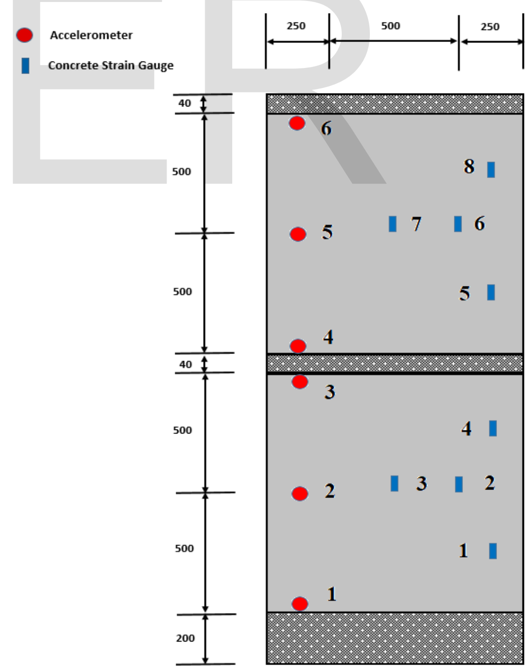

Figure (1) CONT1 and COMP1 walls with sensors numbers and locations.

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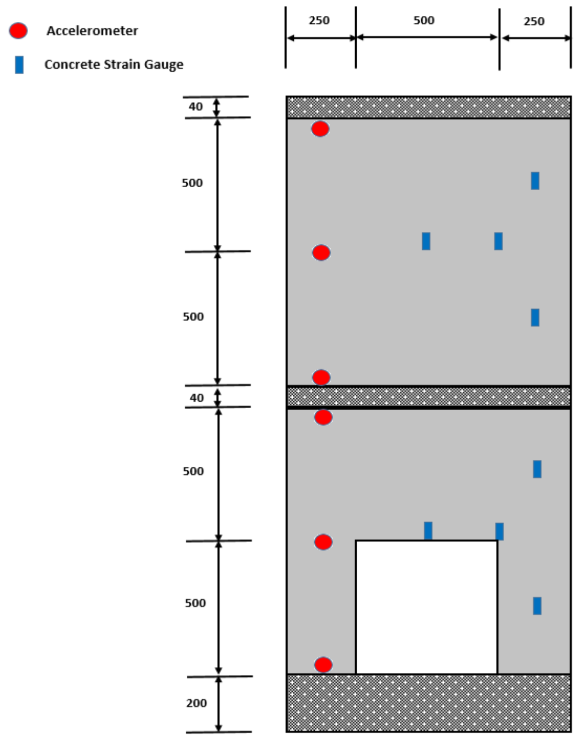

Figure (2) CONT2 and COMP2 walls with sensors locations.

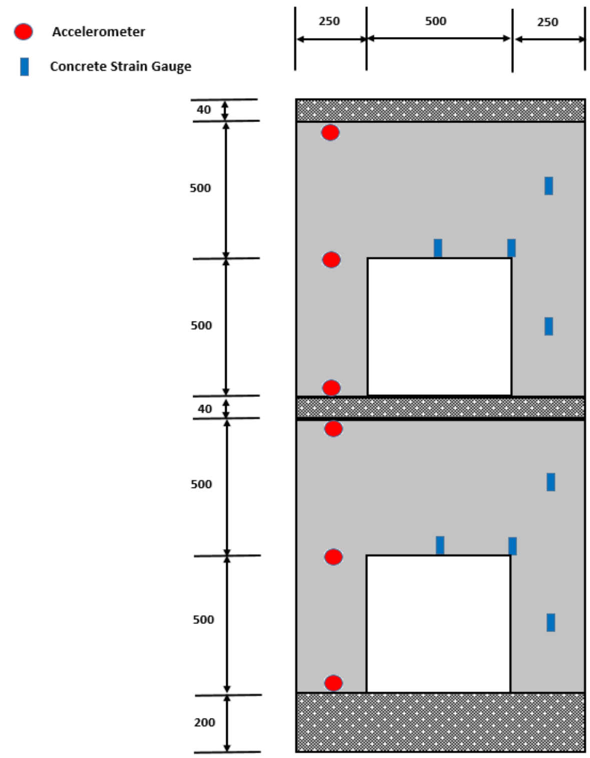

Figure (4) CONT4 and COMP4 walls with sensors locations.

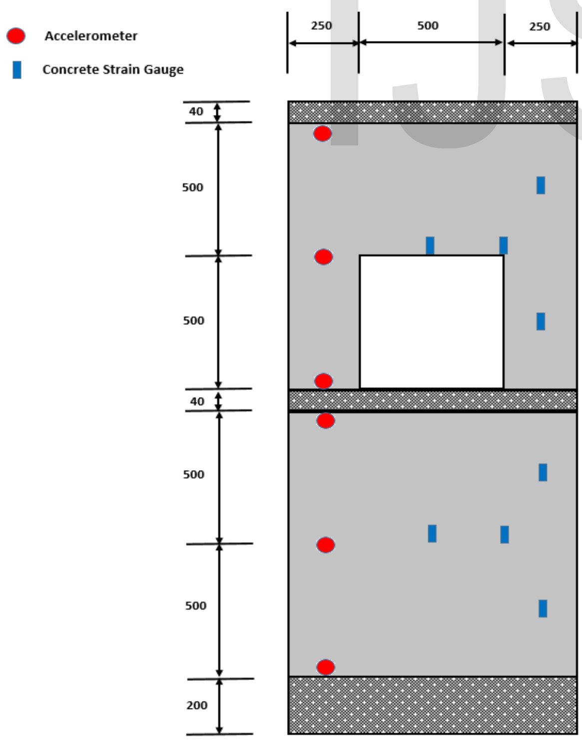

Figure (3) CONT3 and COMP3 walls with sensors locations.

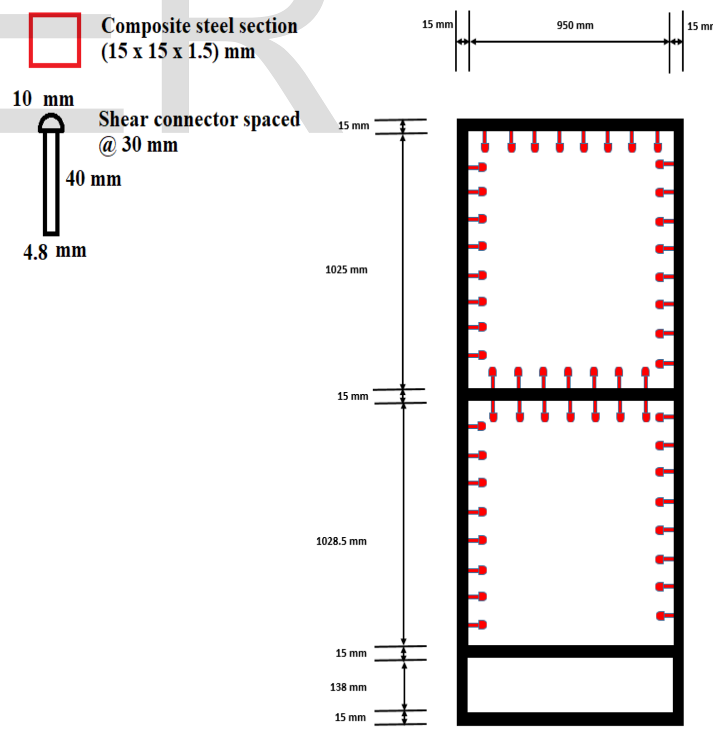

Figure (5) Typical steel frame embedded in each wall-prototype of group B.

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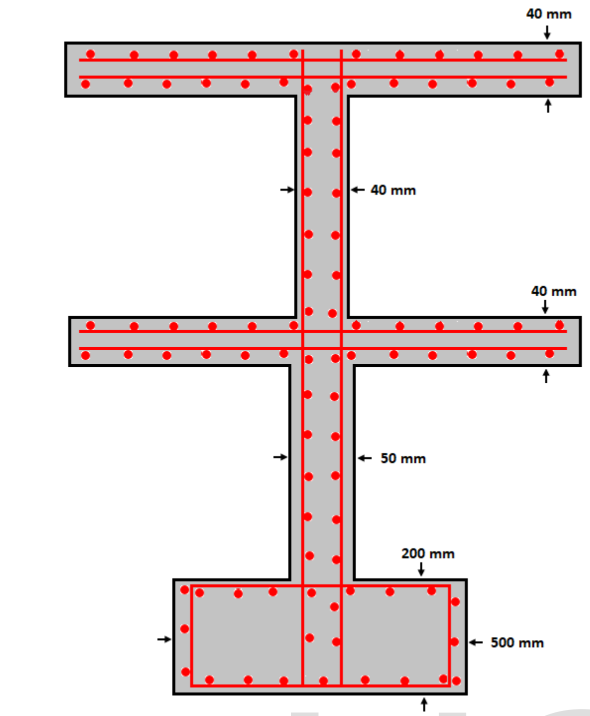

Figure (6) Vertical section in a typical wall-prototype of group A.

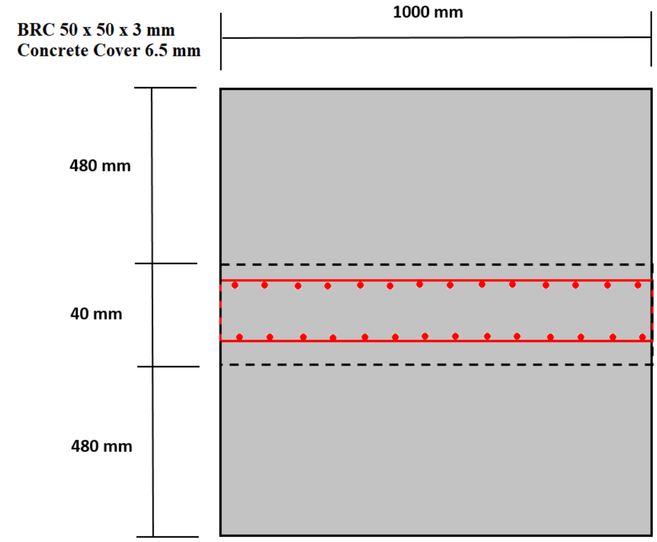

Figure (8) Top-view of a typical group A prototype showing the wall steel reinforcing bars.

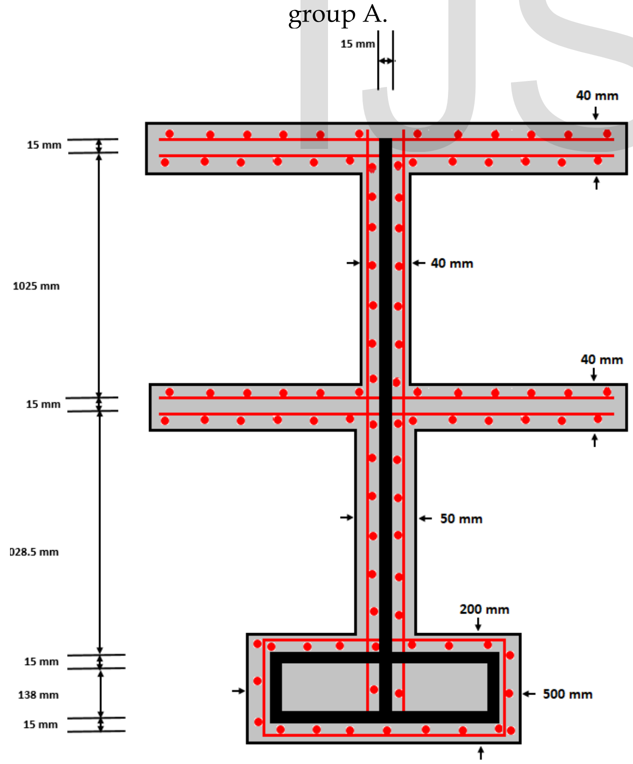

Figure (7) Vertical section in a typical wall-prototype of group B.

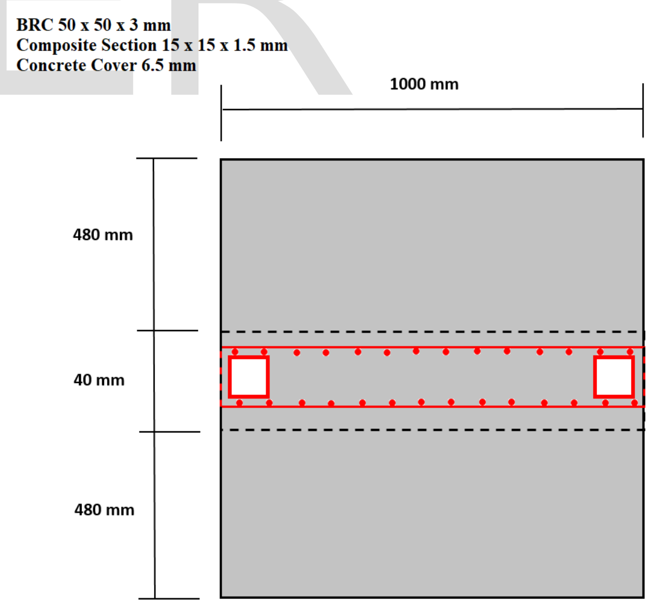

Figure (9) Top-view of a typical group B prototype showing the wall steel reinforcing bars and the embedded steel frame.

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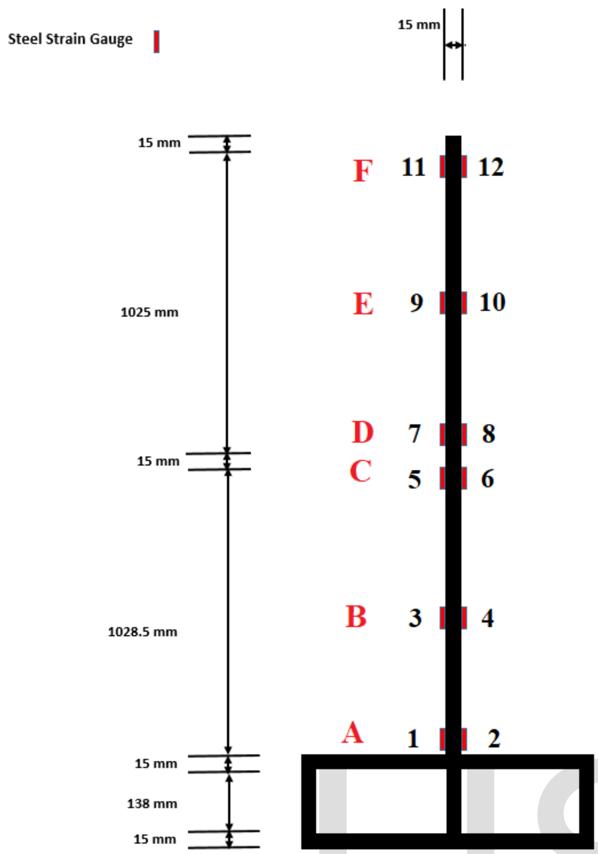

Figure (10) Locations of strain gauges on the embedded steel frame of a typical group B prototype.

4 Steel Reinforcing Bars and Structural Sections

A BRC φ 3 mm @ 50 mm smooth bars was used as longitudinal and transverse reinforcements in the eight wall prototypes. Also, a composite steel square hollow-section section of dimensions 15 x 15 x 1.5 mm was used. Tensile tests of the two types of reinforcement were performed according to ASTM C615-2005 [4]. A tensile test machine as shown in Plate (4-1) below in laboratories of the Consultant Engineering Bureau of Al-Kufa university was used. Test specimens for the square hollow steel sections were of dog-bone shape of 95 mm length and 15 x 1.5 mm mid-cross-section.. Numerical values of the mechanical properties for the two types of steel elements are given in Table(2).

Table (2)

The properties of steel reinforcement*.

Steel type	Fy (MPa)	Fu (MPa)	E (GPa)
BRC	280	408.8	184.964
Composite	240	460	190.325

5 Concrete

In the present work, same materials proportion of mixes were used for all wall specimens. Mixing proportions by weight was used throughout the present work, the designed mix proportions were 1 cement: 1.475 sand: 2.21 gravel, the water-to-cement (w/c) ratio was 0.33, and the dosage of GLENIUM 54 was 1 litre per 100 kg of cement added to mixing water. A 0.05 m3 volume tilting drum mixer was used with a mixing time range of 2-3 minutes. Concrete components were mixed according to B.S. 1881 [12], the ingredients were mixed by putting one-half of the gravel firstly, then inserting the sand, followed by the cement, and finally adding the remaining half of the gravel. The water and the GLENIUM 54 were mixed and then poured to the dry components.

To evaluate the concrete properties, six standard cubes of size 150 mm and four standard cylinders of size (150 x 300) mm were sampled for each casting group. Fresh concrete was poured in two equal layers into the cube moulds accompanied by compacting, for each of the two-layer, uses a standard laboratory rod. The cylinders were also filled by fresh concrete in three equal layers with compacting of each layer by a standard laboratory rod. Three cubes and two cylinders were cured under the same curing conditions of the walls, the other cubes and cylinders were cured as fully submerged in a water tank for 27 days.

The cubes and cylinders fully submerged in water were tested at age of 28 days, while the other cubes and cylinders were tested one day before testing the walls. Compressive strength test was carried out the resulting cylindrical compressive strength of approximately 40 MPa has been considered as normal strength concrete (since f'c is < 41 MPa).

6 Testing Procedure

After preparing the wall prototype, each one was placed on its decided position on the shacking table platform and fixed its base. All test instrumentation, which was used to detect the prototypes walls behavior at seismic excitation during the test, were fixed in their suitable locations.

Wall prototypes were tested seismically and all measurements data were recorded digitally. The shacking table received a time-displacement data that was transformed to electrical signals which ordered the motor to rotate accordingly. A trial non-composite wall prototype was tested under many time-displacement tabulated data till reaching the failure under the seismic excitation time-displacement response shown in Fig. (11) which was selected as seismic test response of all eight wall prototypes. The acceleration, velocity, displacement, and strain were recorded digitally at each selected point with a sampling rate of 1000 readings per second using NI DAC with a total capacity 1.25 million readings per second.

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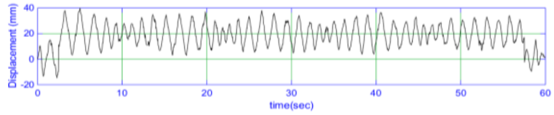

Figure (11) Displacement-control selected signal of seismic excitation.

7 Experimental Results

Eight walls prototypes with two stories, each were tested using displacement-time seismic excitation signal previously shown in Fig. (4-12), the signal data was tabulated and considered as an input seismic data to the LabVIEW computer software and the shacking table moved accordingly. Two sets of walls were chosen and tested under the given time-displacement signal. The first set included four reference (control) prototypes which were without internal embedded composite section but with and without opening. The second set was as same as the first set but with internal embedded composite steel frame. The sensors chosen were motion sensors (ADXL 335 accelerometers), concrete strain gauges type (PFLC-20-11), and steel strain gauges type (KFGS-5-120-C1-11L5M2R) with data output sampling rate 1000 read per second.

7.1 Comparison between Responses of Reference and Composite Walls

This main section is dedicated to wide comparative observation and assessment of the performance of the four reference wall prototypes walls and the four corresponding composite wall prototypes which contained internal embedded steel frames. The reduction percentages in values of displacements, velocities, accelerations, and strains are calculated to construct an idea about the role of embedding composite steel frame in reinforced concrete walls. Furthermore, effects of wall openings on the seismic response behavior is also studied as discussed below.

7.1.1 Test Results Comparisons between CONT1 and COMP1 Walls

In order to draw an idea about the influence of using embedded composite steel frames on the reinforced concrete walls behavior, the response of displacements, velocities, accelerations, and concrete strains at the decided positions of the indicated points are compared. The behavior comparisons of CONT1 and COMP1 walls are given in Figs. (12) through (15).

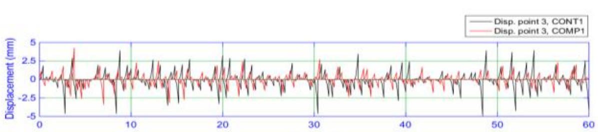

Figure (12) Displacement versus time response comparisons of the two-stories walls CONT1 and COMP1 at the indicated points.

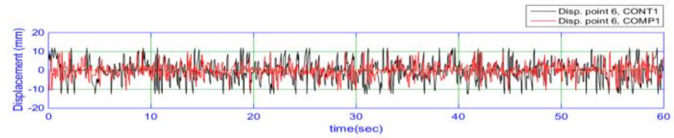

Figure (12) Continued

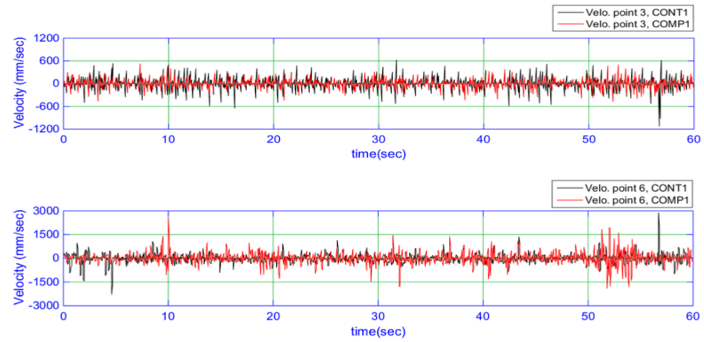

Figure (13) Velocity versus time response comparisons of the two-stories walls CONT1 and COMP1 at the indicated points.

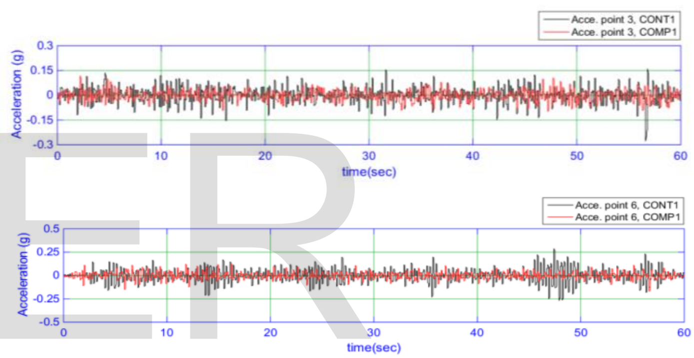

Figure (14) Acceleration versus time response comparisons of the two-stories walls CONT1 and COMP1 at the indicated points.

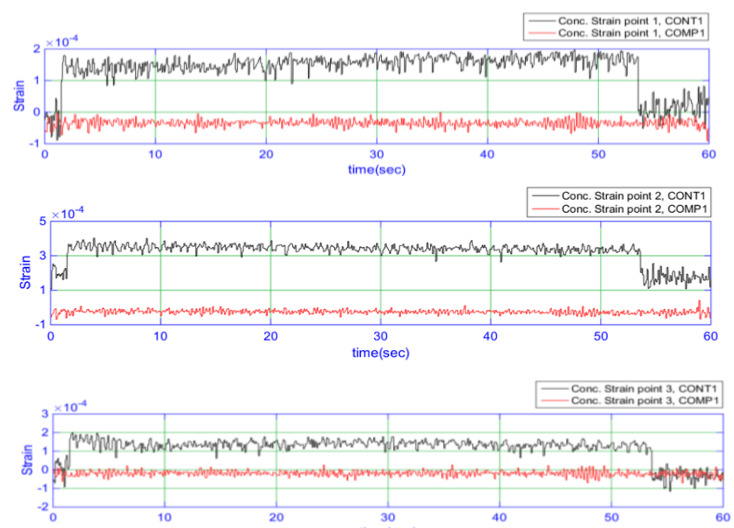

Figure (15) Concrete strain versus time response comparions of the two-stories walls CONT1 and COMP1 at the indicated points.

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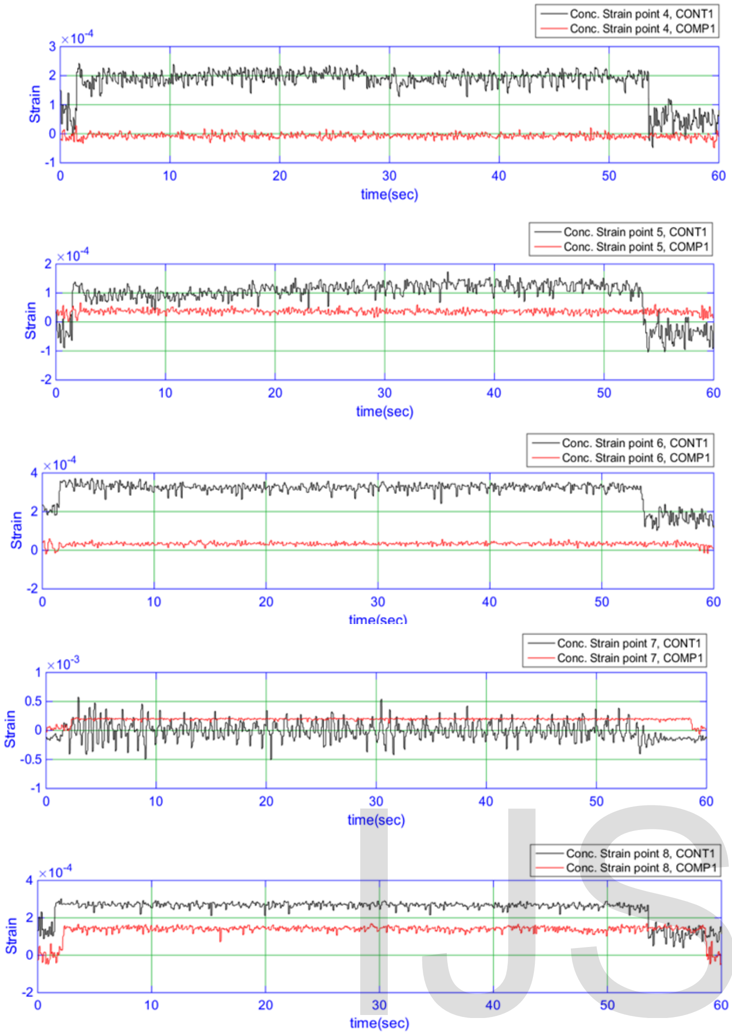

Figure (15) Continued.

7.1.2 Test Results Comparisons between CONT2 and COMP2 Walls

For the sake of a comprehensive overlook on the role of incorporating composite steel frames on performance of the reinforced concrete walls with single first-story opening, the responses of displacements, velocities, accelerations, and concrete strains in the specified directions of the indicated locations are compared. The behavior comparisons of CONT2 and COMP2 walls are shown in Figs. (16) through (19).

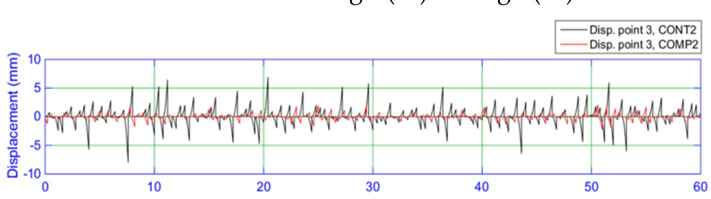

Figure (16) Displacement versus time response comparisons of the two-stories walls CONT2 and COMP2 at the indicated points.

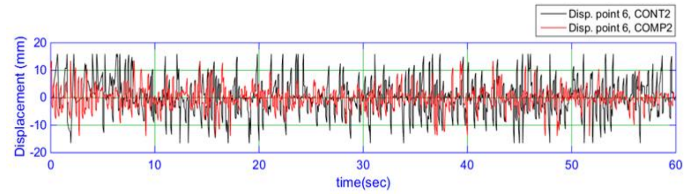

Figure (16) Continued.

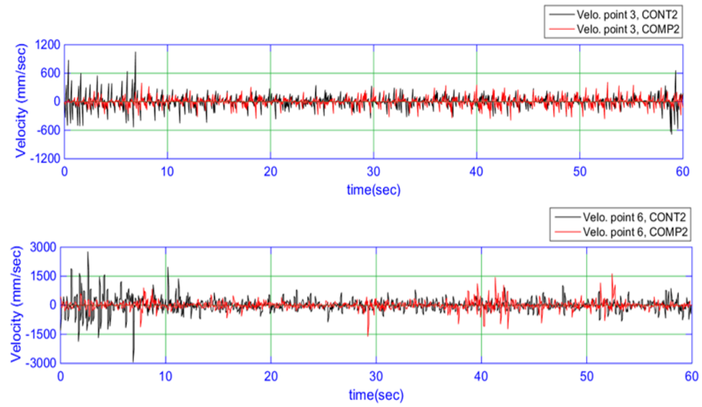

Figure (17) Velocity versus time response comparisons of the two-stories walls CONT2 and COMP2 at the indicated points.

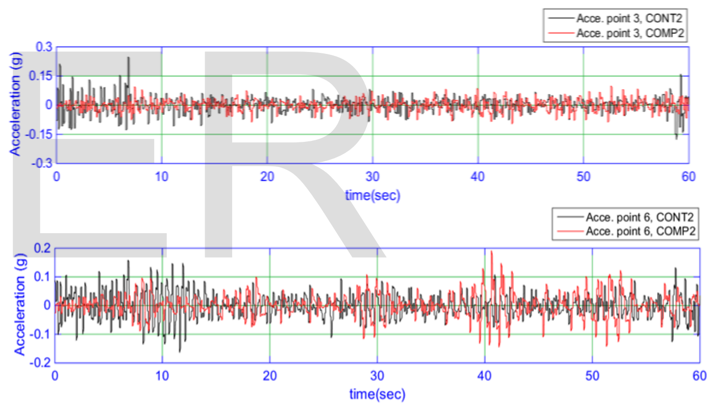

Figure (18) Acceleration versus time response comparisons of the two-stories walls CONT2 and COMP2 at the indicated points.

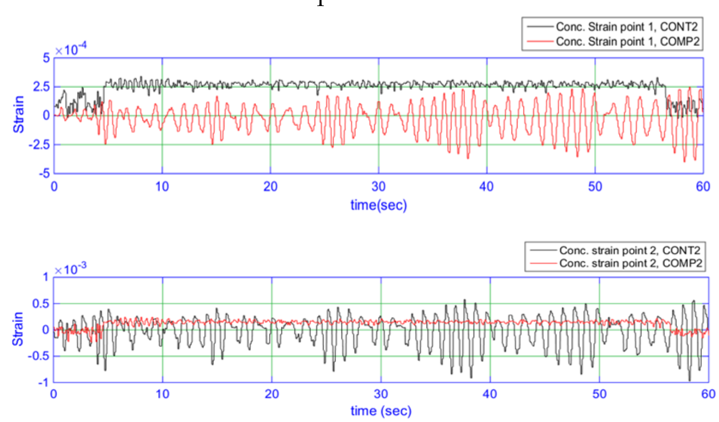

Figure (19) Concrete strain versus time response comparisons of the two-stories walls CONT2 and COMP2 at the indicated points.

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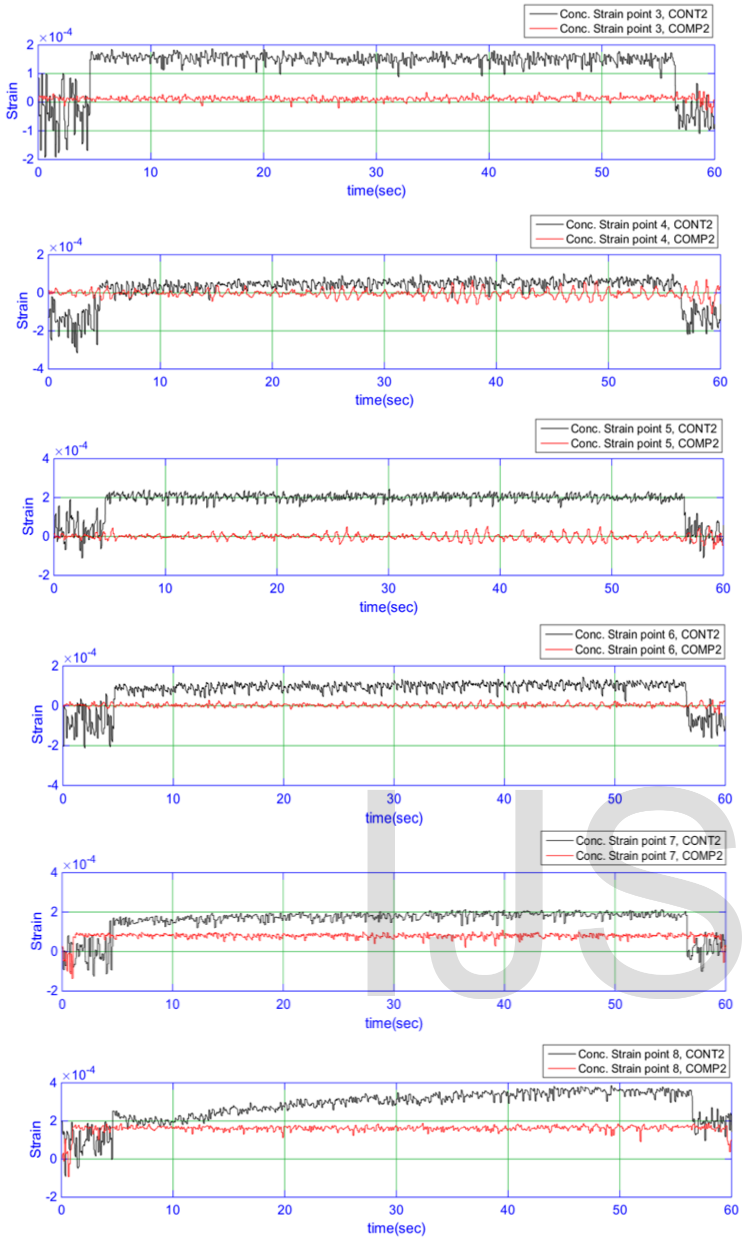

Figure (19) Continued.

7.1.3 Test Results Comparisons between CONT3 and COMP3 Walls

To the purpose of investigation the influence of using embedded composite steel frames on the reinforced concrete walls behavior with single second-story opening, the response of displacements, velocities, accelerations, and concrete strains at the decided positions of the indicated points are comparatively inspected for CONT3 and COMP3 walls through Figs. (20) to (23).

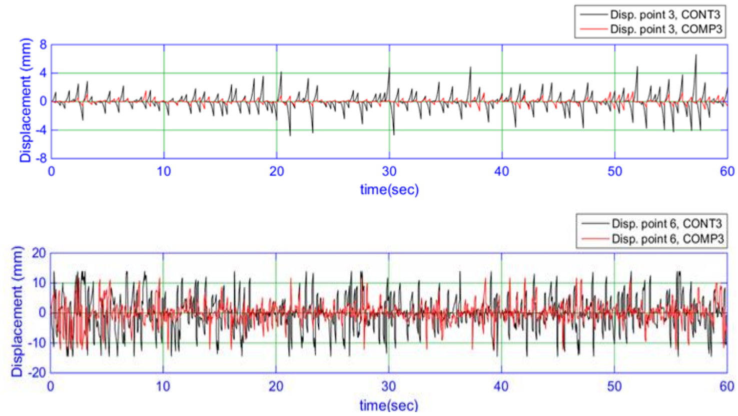

Figure (20) Displacement versus time response comparisons of the two-stories walls CONT3 and COMP3 at the indicated points.

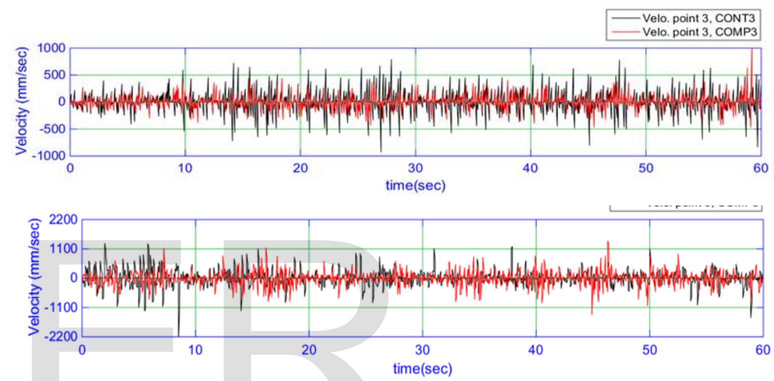

Figure (21) Velocity versus time response comparisons of the two-stories walls CONT3 and COMP3 at the indicated points.

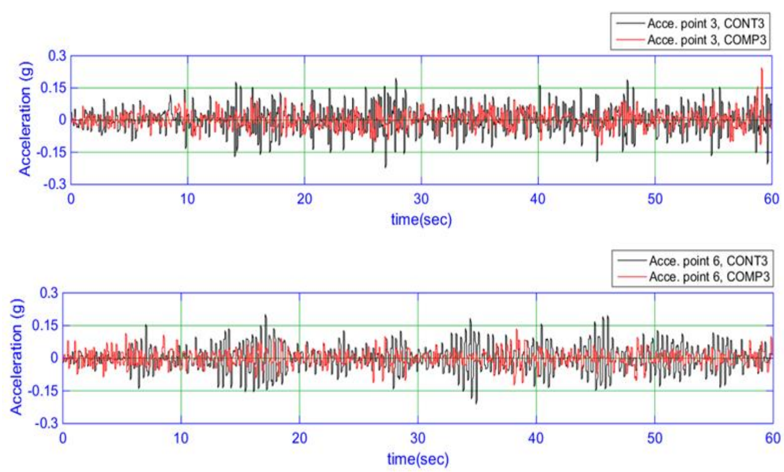

Figure (22) Acceleration versus time response comparisons of the two-stories walls CONT3 and COMP3 at the indicated points.

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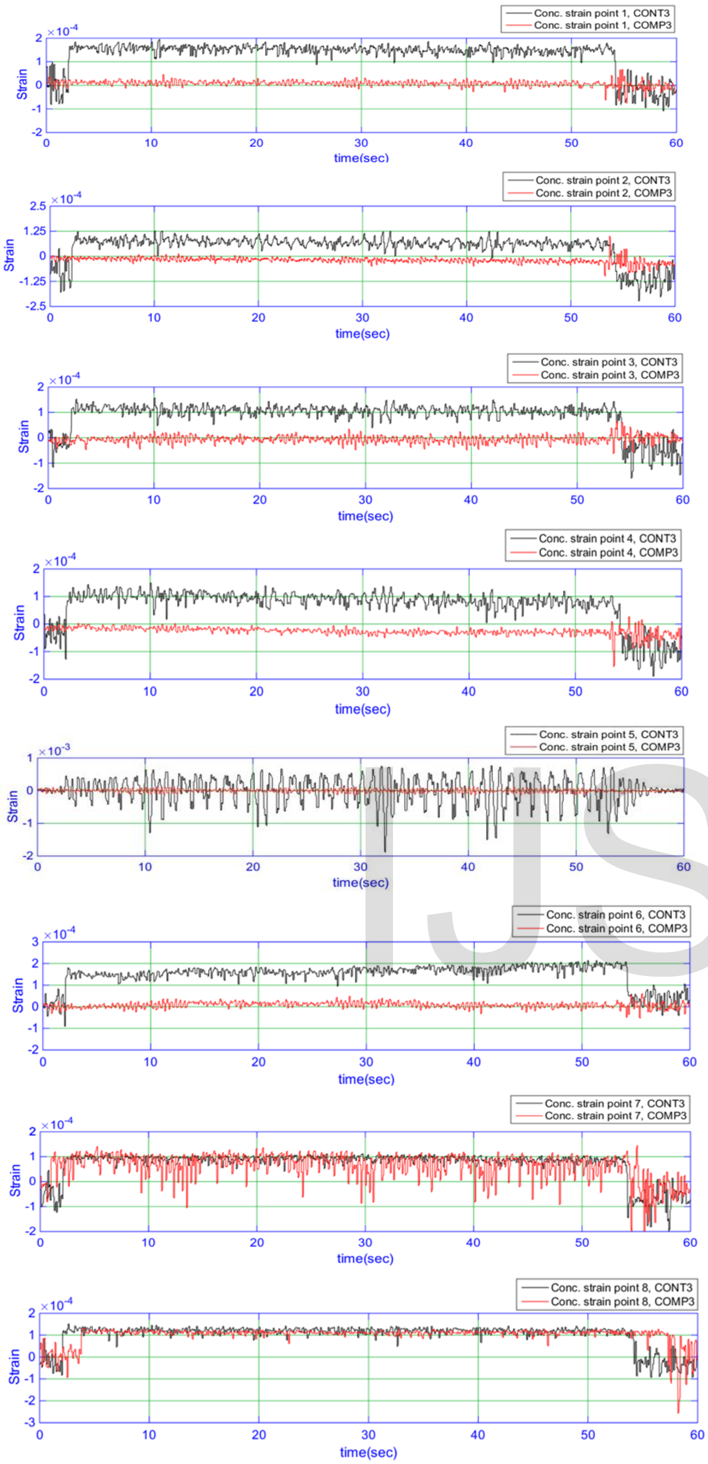

Figure (23) Concrete strain versus time response comparisons of the two-stories walls CONT3 and COMP3 at the indicated points.

7.1.4 Test Results Comparisons between CONT4 and COMP4 Walls.

For the reason of drawing a theme about the influence of using embedded composite steel frames on the reinforced concrete walls behavior with both first and second-story openings, the response of displacements, velocities, accelerations, and concrete strains in the specified directions of the indicated positions are compared. The comparative performances of CONT4 and COMP4 walls are shown in Figs. (24) to (27).

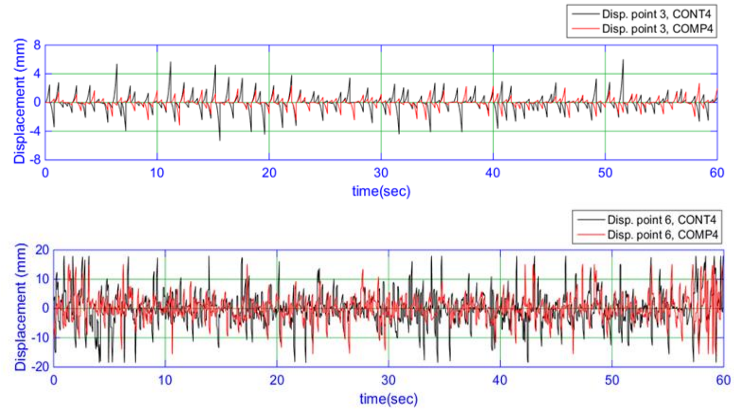

Figure (24) Displacement versus time response comparisons of the two-stories walls CONT4 and COMP4 at the indicated points.

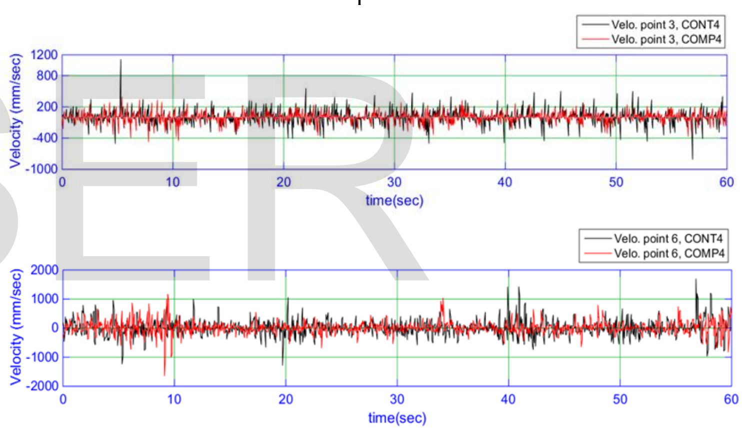

Figure (25) Velocity versus time response comparisons of the two-stories walls CONT4 and COMP4 at the indicated points.

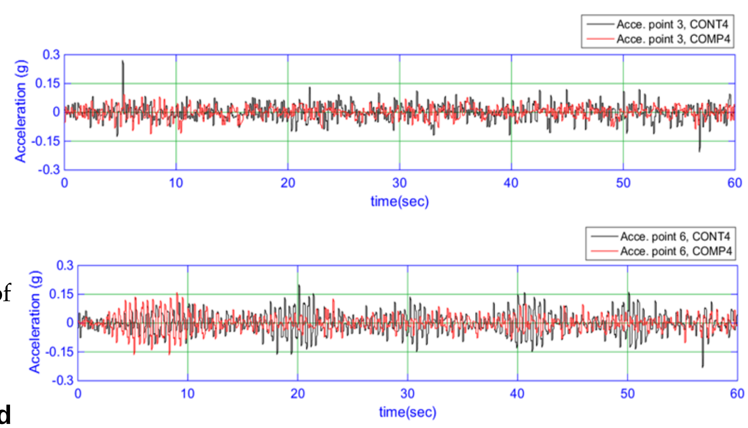

Figure (26) Acceleration versus time response comparisons of the two-stories walls CONT4 and COMP4 at the indicated points.

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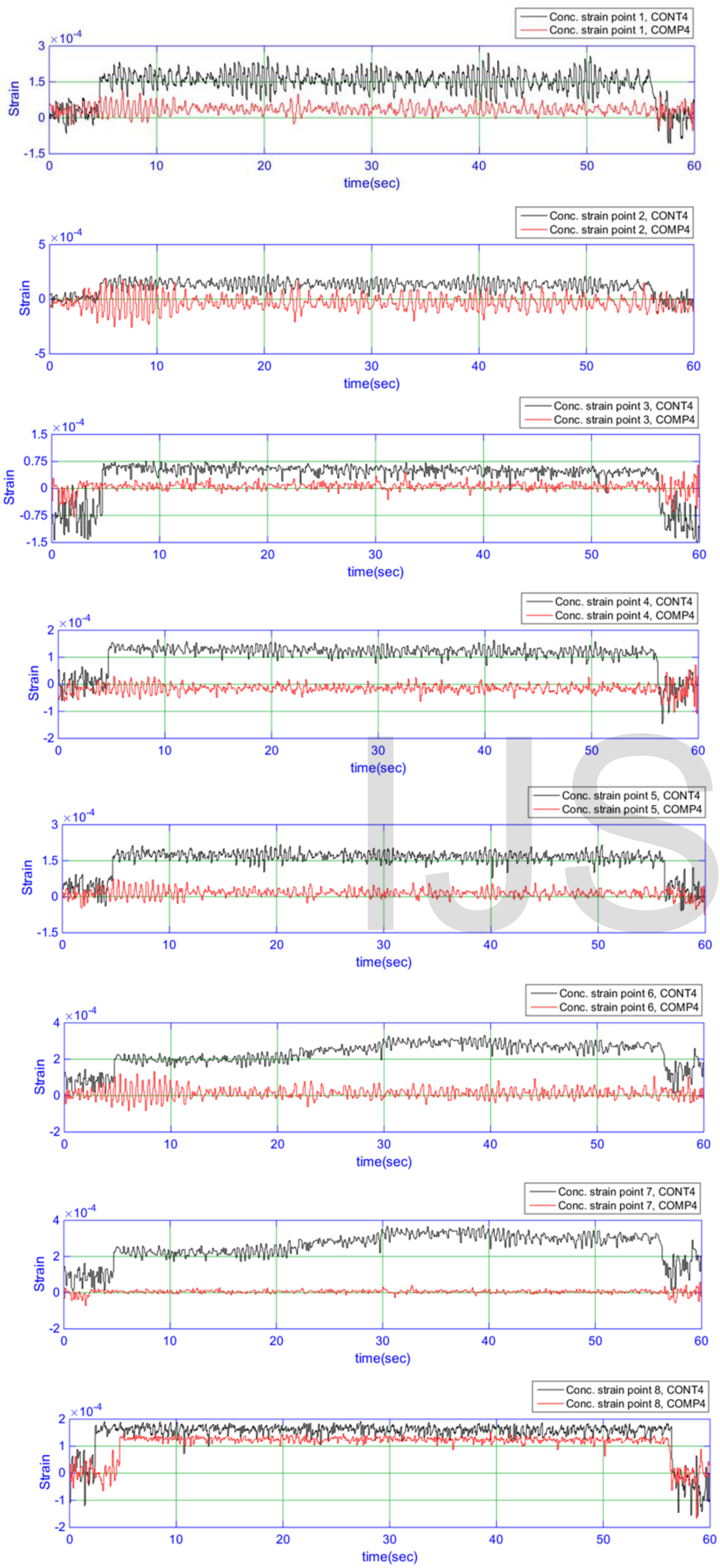
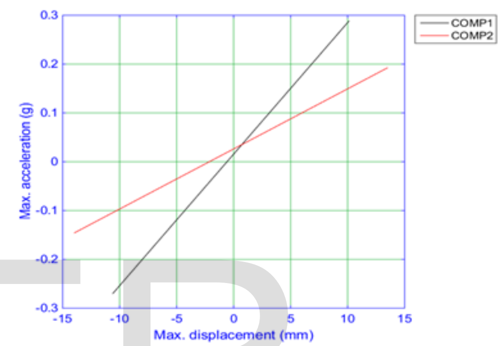


Figure (27) Concrete CONT4 and COMP4 at the indicated points.strain versus time response comparisons of the two-stories walls

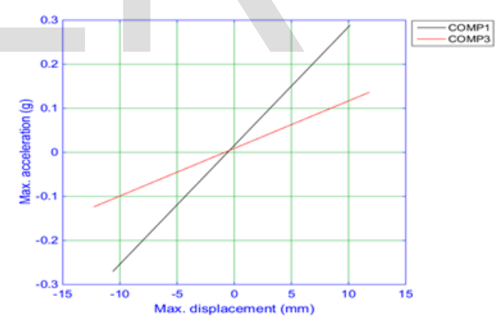
7.2 Effects of Opening Numbers and Locations on Displacement-Acceleration Relation of Composite Walls.

Figs. (28) a, b, and c illustrate the individual comparative of normal-to-plane accelerations with the corresponding displacements at the same locations for COMP1 wall with COMP2, COMP3, and COMP4 wall, respectively. Meanwhile, Fig. (28) d gives a comprehensive comparative spectate on the variations of the specified accelerations with their time-corresponding displacements at the same specified locations for the congregations of the four composite walls.

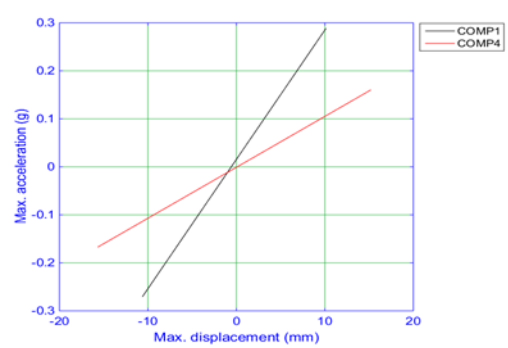
From these figures, it is observed that the effect of openings in those composite walls has significantly negative influence on the acceleration versus displacement relation. The locations and numbers of these opening have slight effects on this relation.



(a)



(b)



(c)

Figure (28) An overview on the effects of openings numbers and locations in the four composite walls.

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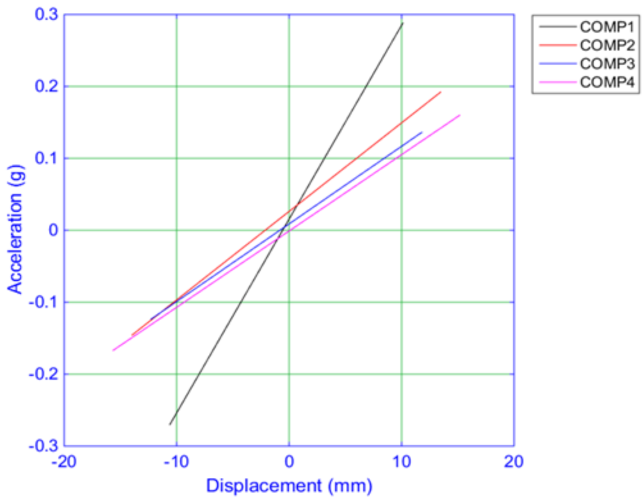

(d)

Figure (28) Continued.

8 Conclusions

1. Shaking table manufacturing using steeper electric operated motor with LVDT feedback provides an ability to change the motor steep accordingly, this disposal improved the method of shack table design using stepper motor instead of using an expensive servo electric operated motor of high cost.
2. The present work illustrates the possibility of using the accelerometers to evaluate velocities and displacements by considering the first and second integrations instead of using LVDTs. This technique was used since it is preferred when the DAC instrument contains the waveform property. Accuracies of the calculated velocities and displacements increased with increasing the accelerometer sensitivity.
3. As a results of comparisons between reinforced concrete walls without and with internal embedded steel frames, the reduction percentage of displacement was 15.75% for walls without openings. For walls with single opening in the first story the reduction was 16%, while for walls with single opening in the second story the reduction was 15.7%, and for walls with single opening in both the first and the second stories, the reduction was 15.95%. Therefore, for the tested walls, the maximum displacement of the composite walls with internal embedded steel frame was reduced by an average of 15.85% as compared with the non-composite reinforced concrete walls.
4. The results of this investigation showed that as a comparison between composite and not composite walls, the reduction percentage of the velocity was 16.59% for walls without openings. For walls with single opening in the first story the reduction was 45.03%, for walls with single opening in the second story the reduction was 38.8%, and for walls with single opening in both the first and the second stories was 31.06%. Therefore, for the tested walls, the maximum velocity of the composite walls with internal embedded steel frame was reduced by an average of 32.87% as compared with non-composite reinforced concrete walls.
5. Regarding the maximum acceleration comparison between composite and non-composite reinforced concrete walls, it's found that the reduction percentage of acceleration was 57.25% for walls without opening. For walls with single opening in the first story, the reduction was 59.87%, for walls with single opening in the second story the reduction of 41.36% has been obtained, while for walls with openings in the first and the second stories the reduction was 65.88%. Therefore, for the tested walls, the maximum acceleration of the composite walls with internal embedded steel frame was reduced by an average of 56.09% as compared with non-composite reinforced concrete walls.
6. It has been experimentally observed that the average maximum concrete strain of composite walls was extremely less than that of non-composite walls, where percent of reduction of average maximum concrete strain was 74.96% for walls without openings. For walls with single opening in the first story, the reduction was 12.63%, for walls with single opening in the second story the reduction was 61.37%, and for walls with openings in the first and the second stories, the reduction was 54.59%. Therefore, for the tested wall prototype, the maximum concrete strain of the composite walls with internal embedded steel frames was substantially reduced by an average of 50.88% as compared with non-composite reinforced concrete walls.
7. For the four control wall prototypes, its observed that crack patterns, propagation, and local plastic damages were located around wall opening region, and at wall end near the bottom wall joints. Meanwhile, the other four composite wall prototypes had adequate capacities to resist and dissipate such plastic damages.
8. Inspecting on the previous conclusions in marks 3, 4, 5, 6, and 7 it's observed that the utilization of embedded steel frame method provides a sufficient and practical technique of composite walls construction which dissipates earthquake energy, minimizes seismic risk and reduce global building damage, thus as structural elements, reinforced concrete composite walls reduce the probability of building fully plastic collapse or failure to keep people life in a safe mode. Furthermore, under the given displacement-time signal, the four control wall specimens reached significant cracks and local plastic failure while the other four composite wall specimens were adequate to

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resist the selected signal and provided an ability to avoid any damage.

9. Results showed that composite walls with openings have significantly influenced wall behavior. In general, for a given acceleration, the corresponding displacement will increase accordingly. The increasing percentage of the displacement will be 52.44% if the wall contains an opening in the first story, while the increasing percentage of 60.92% if the wall contains an opening in the second story, and the increasing percentage will be 62.63% if the wall contains openings in the first and the second stories.
10. From Figs. (28rat.alghalibi@uokufa.edu.iq) a, b, c, and d, it is observed that while the existence of openings in those composite walls have significant negative influence on the acceleration versus displacement relation, the locations and numbers of these opening have slight effects on this relation.

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