INELASTIC SEISMIC PERFORMANCE OF CONCRETE PRECAST 3D PANEL SYSTEM WITH DISCONTINUOUS SHEAR WALLS SUPPORTED ON RC FRAMES

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ABSTRACT :

In the present investigation, numerical dynamic analysis of a four-story building by scale factor of 1:2.35 is performed by ANSYS soft ware. The purpose of this article is to study the static and dynamical performance of the compound system which is consisted of pre-cast 3D panel system in upper floors and concrete frame in the ground floor. Through the analysis, dynamic properties of the structure, including natural frequencies and dynamic responses such as story displacement, story drift, story shear force and energy absorbed by each story under different seismic motion, in linear and nonlinear region are investigated. In addition, the distribution of tensile and compressive stress in concrete and tensile stress in reinforcement of various partition and also quality of generated cracks are estimated.The results of this study would be used in predicting the behavior of the structure under real earthquakes.

KEYWORDS: Precast 3D Panel, Soft story, Concrete Frame, Dynamic Analysis, Non-linear

1. INTRODUCTION

With regard to the growing tend of making light weight structure and the application of precast member, one of the building construction methods is to use precast 3D panel. Most of the investigations, which have been done in precast 3D panel, were about determination of their behavior in the form of single member. Eiena et al. in 1995 suggested mathematical solution of semi-composite panels by developing differential equations and compared the analytical solution with numerical finite element analysis [1]. Salmon et al. in 1997 presented the results of full-scale test of prototype sandwich panel under transverse loading in a vertical position [2]. Nijhawan in 1998 measured experimentally the interface shear forces [3]. Bush and Wu in 1998 presented mathematical solution and finite element model for bending analysis of pre-stressed sandwich panels with truss diagonal shear members [4]. In an experimental research concerning the response of reinforced concrete panels subjected to in-plane cyclic loading, the behavior of structure reinforced with welded wire mesh fabric was investigated by Paolo and Franchi in 2001 [5]. Kabir and Hasheminasab in 2001 tested flexural, shear loading on 3D bearing wall and floor slab, and showed the load-deflection curves and failure mechanism [6]. In an experimental investigation, Benayoune et al. in 2005 studied the ultimate strength behavior of pre-cast concrete sandwich panels (PCSP) under eccentric axial loading [7]. In addition, in 2005 a comprehensive experimental research in order to better understanding of mechanical characteristics of such hybrid system is conducted by the first author. The compressive strength of sprayed concrete in the form of small cores is measured as a factor of standard cylindrical specimens by Kabir and Rahbar in 2005[8].

Concerning the insufficient parking spaces in metropolises it is necessary to use the ground floor to provide such a space. Thus the need for extensive opening to provide appropriate space is vital. The existence of such extensive opening would result in an unexpected change of lateral stiffness in the ground floor and would create soft story at this level, consequently the need for another lateral resistant system such as concrete frame in order to provide the required lateral stiffness. Thus, studying the static and dynamical performance of the compound system which is consisted of pre-cast 3D panel system in upper floors and concrete frame in the ground floor becomes necessary.

2. DESCRIPTION OF THE MODEL

In this study, the numerical dynamic analysis of a four story using 3D wall system in three upper floors and using concrete frame system at the parking level by scale factor of 1:2.35, was performed by ANSYS soft ware. In 3D wall and roof system two types of panel specimens were cast; wall and slab panels. Wall panel is consisted of two 17 mm shotcrete wythes with a central polystyrene core, 26 mm thick. The slab panel is thicker, 25 mm upper concrete and 17 mm lower shotcrete wythe. The central core in slab is 43 mm to provide more flexural stiffness. The welded wire mesh, which is spread on both sides and mounted in shotcrete, has 1 mm steel wires with mesh size of 18 mm. The panel's geometrical dimensions are also presented in Table 1.

Table 1 Panel Specification(Dimensions Are in mm)

Type	Total thickness	Core thickness	Upper wythe thickness	Bottom wythe thickness	Welded wire diameter
Wall	60	26	17	17	1
Slab	85	43	25	17	1

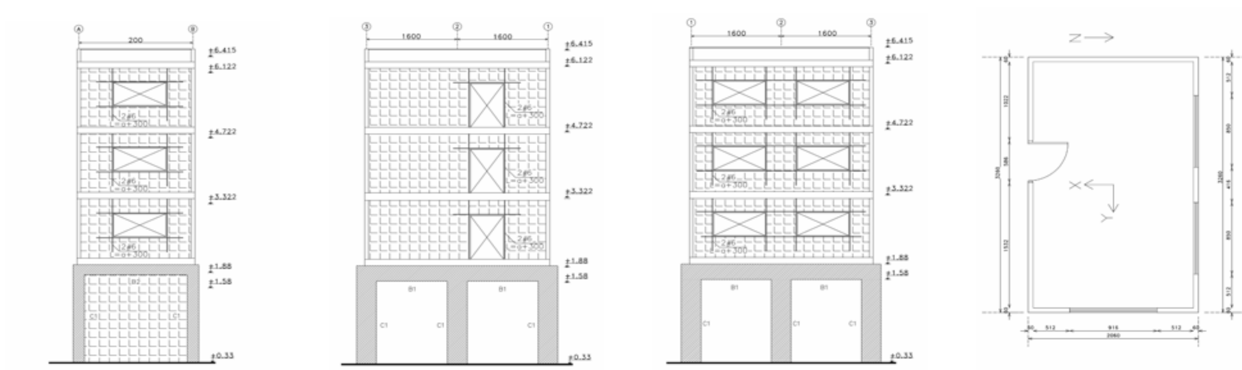
Table 2 Concrete Properties Used in Analysis

Type	Poisson ratio	Specific gravity(T/m^3)	Young modulus(GPa)	Tensile strength(MPa)	Compressive strength(MPa)
Concrete used in ground floor frame	0.2	2.5	20	2	20
Shotcrete used in 3d panel system	0.15	2.17	7	1	9

Table 3 Steel Properties Used in Analysis

Type	poisson ratio	Young modulus(GPa)	Yield stress(MPa)	Ultimate stress(MPa)
Steel bars (Steel mesh of panel Φ1)	0.29	190	600	650
Steel bars (Additional used in panel Φ4.3)	0.29	190	200	280
Steel bars (Additional used in panel Φ6)	0.29	190	280	360
Steel bars (used in ground floor frame)	0.3	200	400	630

Table 2 and Table 3 present the concrete properties and steel properties respectively which were used in analysis. The test model is a four-storey building with asymmetrical distributed stiffness in X and Y and Z directions. The plan and elevation of model are shown in Figure 1.



(a) Eastern elevation (b) Southern elevation (c) Northern elevation (d) Plan of model

Figure 1 Building plan and elevation

3. NUMERICAL ANALYSIS

3.1. Element Properties Used in Analysis

In the three-dimensional analysis, developed in ANSYS 9 environment, the numerical model was subdivided into shell elements (shell 91) describing 3D panel system in three upper floor and solid elements (solid 65) to model frame system used in ground floor. The reinforcements which were used in concrete frame of ground floor were modeled by LINK8. Figure 2 shows the specimen modeling by FEM.

3.2. Modal Analysis

Energy dissipation and the shear absorption capacity of structures are related to the dynamic characteristics of the systems. One of them is natural frequency. It is very difficult and somehow impossible to determine the natural frequencies of this complicated system theoretically therefore investigation about dynamic properties such as frequencies and mass participation factors can be useful. The modal analysis of specimen was carried out for eight primary modes in order to recognize the dynamic sensitivity of such system. The frequency and percent of mass sharing in each mode is also calculated in numerical procedure and is tabulated in Table 4. Table 4 shows that the 1^{st} mode (Freq=7.278Hz) is transmissive in X direction and rotational in Z direction and the 2nd mode (Freq=9.058Hz) is transmissive in Y direction and rotational in Z direction. Comparison between 1st and 2nd mode shows that the stiffness of the structure in Y direction is more than in X direction. The 7th mode (Freq=18.107Hz) is the other main mode that is rotational in Z direction. Figure 3 shows that the dominant deformation is formed in two primary modes and 7^{th} mode, figures 3-a, 3-b and 3-g and the rest modes are reflecting local and component resonance of system.

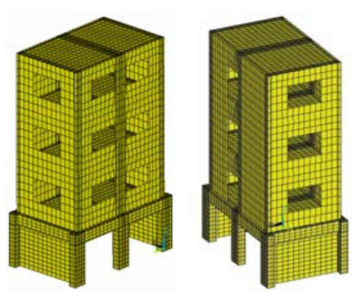


Figure 2 specimen modeling by FEM

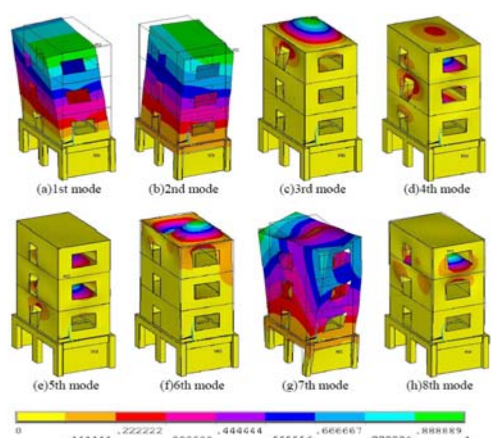


Figure 3 Deformations from modal analysis

Table 4 Frequencies and Mass Participation Factors

Mode no.	Freq(Hz)	Part. Mass. in X dir	Part. Mass. in Y dir	Part. Mass. in Z dir
1st	7.278	78.885	0.432	44.539
2nd	9.058	0.4879	78.226	19.315
3rd	10.23	0.0345	0.0662	0.0721
4th	11.28	0.0144	0.0124	0.0236
5th	11.61	0.0216	0.0226	0.0401
6th	17.407	0.0262	0.3962	0.3777
7th	18.107	0.1126	1.0263	22.876
8th	18.842	0.001	0.7171	1.2811

3.3. Dynamic Analysis

In order to study seismic behavior of the structure, the dynamic time history method was chosen for analysis and the specimen was analyzed by two records of earthquake. In dynamic analysis, the time step sizing in the analysis becomes important and to get more accurate results, Δt was chosen infinitesimal. In this study, this parameter was taken as identical for all cases and equal to 0.005. Elcentro (Imperial Valley station–1940) and Naghan (Iran–central Zagros station–1977) records were selected as two level of excitation. Table 5 lists the applied earthquake records specifications. Accelerations in time-history of the records for selected earthquake are shown in Figure 4.

Table 5 Applied Ground Motion Records

Name	Rec.	Dur (s)	X-direction			Y-direction		
			PGA (g)	PGV (mm/s)	PGD (mm)	PGA (g)	PGV (mm/s)	PGD (mm)
Elcentro-Imperial valley-1940	ELC	40.00	0.31	298.00	133.00	0.21	297.20	231.90
Naghan-Iran-Zagros-1977	NGH	22.50	0.53	374.30	35.20	0.71	459.20	61.00

(a) Acceleration time history of Elcentro-1940 record.

(b) Acceleration time history of Naghan-1977 record.
Figure 4 Selected applied records.

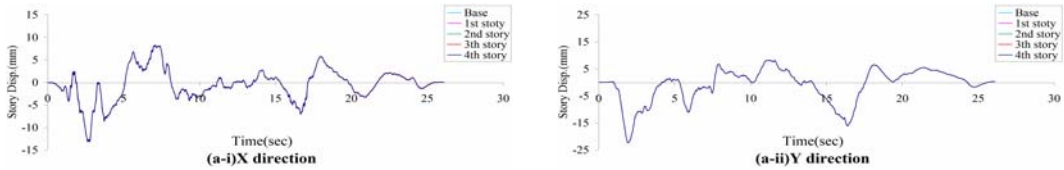
In order to evaluate the behavior of the system under different level of shaking the dynamic loading applied on the specimen in both E–W and N–S directions simultaneously by two level of excitation, respectively low-level and high level seismic excitations. The description of shaking levels is presented in Table 6.

Table 6 Applied Ground Motion Records

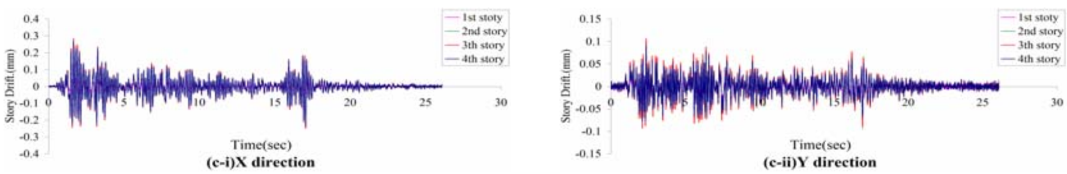
Level	Name	Percentage (%)	PGA (g)	PGV (mm/s)	PGD (mm/s)	Nickname
A	Elcentro-Imperial valley-1940	25	0.095	10.52	68.84	ELC025
B	Naghan-Iran-Zagros-1977	135	1.196	79.97	95.07	NGH135

3.3.1. Structural responses

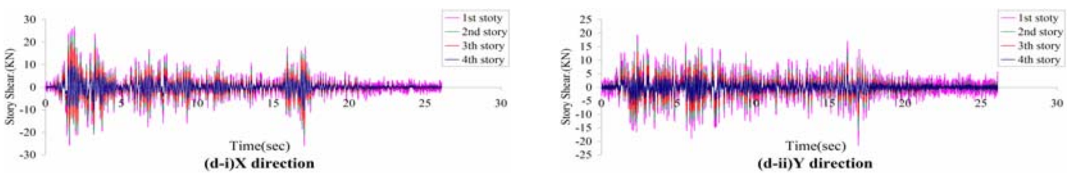
Figure 5 and Figure 6 show structural responses including stories displacement, stories drift and shear in stories for record ELC25 and 5 seconds of NGH135 record, respectively. Figure 5-a depicts that the stories displacements are approximately close to each other indicating high rigidity of the structure against of low frequency record and therefore the structure remains in the linear zone. However, Figure 6-a shows that in record NGH135, the structure is exposed to a stronger earthquake since the difference between the stories displacements are greater and the structure has presumably entered in the nonlinear zone.



a) Story displacement response

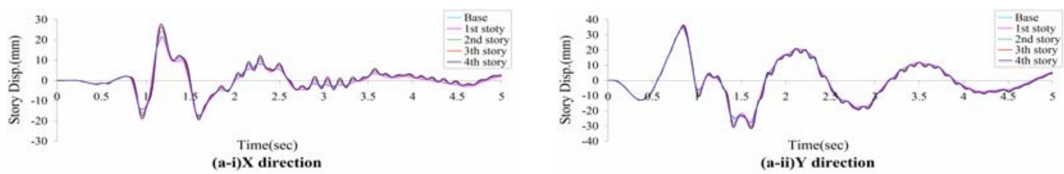


b) Story drift response

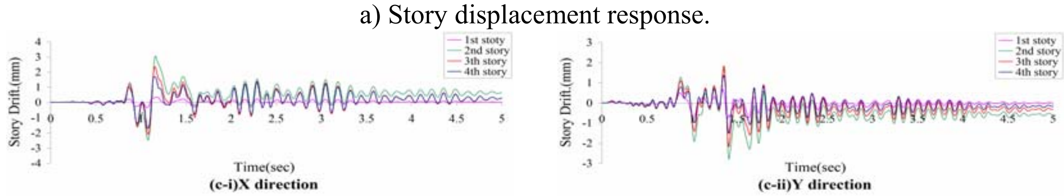


c) Story shear response

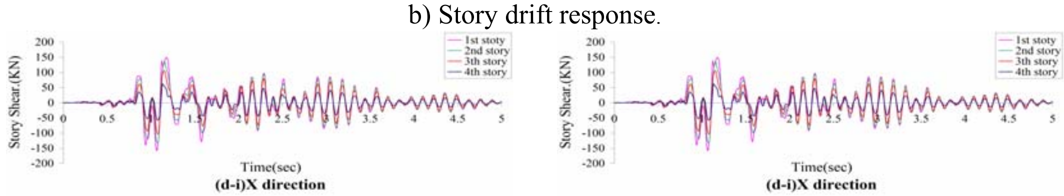
Figure 5 Building responses under ELC25.record



a) Story displacement response.



b) Story drift response.



c) Story shear response.

Figure 6 responses under NGH135.

The nonlinear behavior of the structure under record NGH135 is more obviously seen in Figure 6-b which corresponds to the Stories drift since irreversible displacements occur at the second, third and fourth stories starting from 3.5 seconds after the earthquake began. However, in Figure 5-b it is seen that the structure behaves linearly and no irreversible displacements occur. The reason for the linear behavior of the first story and the nonlinear behavior in the second, third and fourth stories in record NGH135 is the difference in the materials used as well as the difference in the structural systems applied for the stories. also Figure 5-b shows that the greatest story drift in record ELC25 in both x and y directions is seen at the second story while Figure 6-b shows that the greatest story drift has occurred in record NGH135 at the second floor and after that in the third story. In addition Figure 6-b clearly shows that the most irreversible displacement created in record NGH135 in both directions is for the second story and it can be concluded that the greatest damage occurs in the same story as well. Figures 5-c and Figure 6-c correspond to the shear response of the stories under the two records discussed. Regarding 257KN weight of the structure, the greatest base shear in record ELC25 along x and y are 26.8KN and 21.5KN respectively that are 10.45 and 8.36 percent weight of the structure. The proportion of the first story in absorbing the total shear along x direction is 36% and 70% along y direction which shows a triangular distribution of the shear force along x and an approximately rectangular distribution along y direction. The greatest base shear in record NGH135 along x and y directions are 158.3KN and 222.8KN that are 65 and 87 percent weight of the structure respectively. The proportion of the first story in absorbing the total shear along x and y directions is 20 percent.

a) ELC25 b) NGH135

Figure 7 Structure hysteresis curves for selected record

Figure 7 shows hysteresis curves for the model in both x and y directions under the selected records. This figure which shows the base shear versus the upper end of the stories displacements indicates that the structure has a complete linear behavior in record ELC25 with a PGA of 0.095g.The stiffness of the structure can be estimated about 33KN/mm along x and 50KN/mm along y direction, while the structure enters its nonlinear behavior with a PGA of 1.2g in record NGH135 and by increasing the area below the hysteresis curves the amount of energy absorption is increased. In order to more accurately evaluate the nonlinear behavior of the model under record NGH135, the hysteresis curves for the stories are depicted in Figure 8. It is seen in this figure that the second story has the greatest nonlinearity, and after that, stands the third story. It's seen the first story has very slightly entered the nonlinear zone. Figure 9 shows the cumulative energy absorption in each story. For instance, energy absorption up to the fourth story equals the sum of energy absorptions in all the stories from the first to the fourth. This figure also shows that the greatest amount of energy absorption is for the second story and the least belongs to the first story.

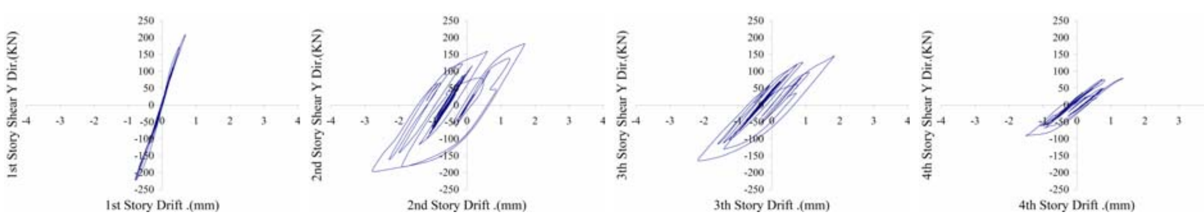
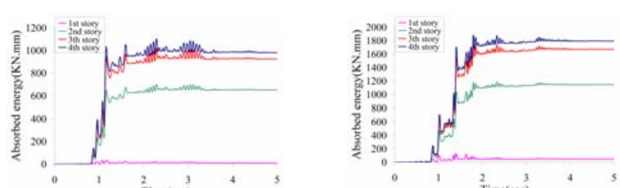


Figure 8 Story hysteresis curves for NGH135

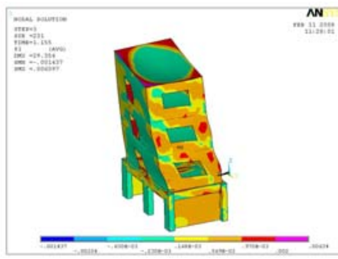


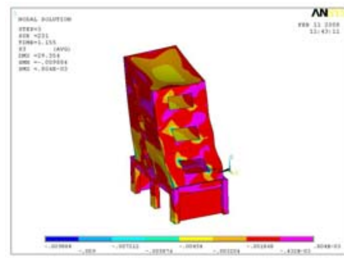
a) X Direction b) Y Direction

Figure 9 Total absorbed energy versus time for walls and whole specimen under NGH135 record.

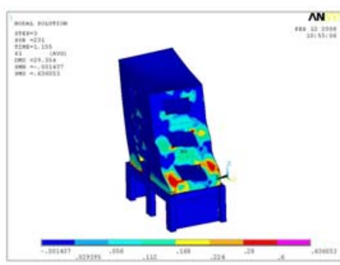
3.3.2. Stress distribution

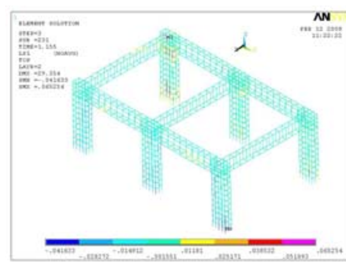
The principal stress distributions under NGH135 are sketched in Figure 10. It is seen that the maximum stress intensity occurs at 1.155 s of the response record. Figure 10-a shows that the maximum tensile stress which is occurred in concrete, is more than its strength in some parts of structure. These parts are places where probably cracks may happen such as zone between to windows of east wall.





a) Tensile stress in concrete　　b) compressive stress in concrete





c) Tensile stress in steel mesh of panels　　d) Tensile stress in reinforcement

Figure 10　Stress distribution under NGH135 record at 1.155 s (inkN/mm2)

Figure 10-b describes the compressive stress distribution in concrete. This figure clearly shows that the corner of the opening such as windows and door and also the connection zone where the three upper floors are connected to ground floor are places with maximum compressive stress distribution, but value of maximum compressive stress in some places as it mentioned such as corner of the west window in second floor is more than compressive strength of shotcrete and are the places, where crushing must be occurred. In other locations of 3D panel system and in every where of concrete frame of ground floor the maximum compressive stress is less than compressive strength of concrete and no crushing happen. Figure 10-c and Figure 10-d depict the tensile stress distribution in steel mesh of panels and reinforcement of concrete frame in ground floor. It is easy to see that the maximum value of the tensile stress in steel mesh of panels is located at the corners of door in second floor and windows in 2nd and 3rd floor that is less than its yield stress but is more than yield stress of additional bars which are used in corner of door and windows.

4. CONCLUSION

In this investigation, a 1:2.35 scaled four story building was analyzed by using numerical FEM modeling under transient dynamic loading. Two ground motions were selected to apply to the building, ELC25, NAGH135, in order to recognize dynamical properties and mechanical behavior of a new compound system which was consisted precast 3D panel system in upper floors and concrete frame in the ground floor. According to the modal analysis, Initial natural frequency of system in X direction was 7.278 Hz and 9.058 Hz in Y direction and It was concluded that the this system was sensitive to high frequency ground motion due to its natural high rigidity. Therefore, for the active seismic zones where have exercised high frequency earthquake, this system should be constructed with special design considerations. In motions with 0.095g PGA (ELC25 record), the specimen completely behaved linear and no crack was observed in all walls area and in frame of ground floor. The stiffness of the structure can be estimated about 33KN/mm along x and 50KN/mm along y direction. At the level of 1.2g PGA (NGH135 record), specimen behaved nonlinear. Most irreversible displacement created in this record in both directions was for the second story and it was concluded that the greatest damage occurred in the same story as well and the greatest amount of energy absorption was for the second story and the least belonged to the first story. Regarding the 257KN weight of the structure, the greatest base shear in record ELC25 along x and y were 10.45 and 8.36 percent weight of the structure, that proportion of the first story in absorbing the total shear along x direction was 36% and 70% along y direction which showed a triangular distribution of the shear force along x and an approximately rectangular distribution along y direction. The greatest base shear in record NGH135 along x and y directions were 65 and 87 percent weight of the structure respectively and proportion of the first story in absorbing the total shear along x and y directions was 20 percent. Maximum stresses were appeared at the corner of openings and at the connection zone where the three upper floors were connected to ground floor. Value of maximum compressive stress in some places as it mentioned such as corner of the west window in second floor was more than compressive strength of shotcrete and was the places where crushing must be occurred, but in every where of concrete frame of ground floor the maximum compressive stress was less than compressive strength of concrete and no crushing happened. Maximum value of the tensile stress in reinforcement of concrete frame was less than its yield stress in history of NGH135 record and it showed that frame of ground floor had a probable behavior and no serious damage happened.

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