BULETINUL INSTITUTULUI POLITEHNIC DIN IAŞI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iaşi
Tomul LV (LIX), Fasc. 1, 2009
Secţia
CONSTRUCŢII. ARHITECTURĂ

SEISMIC BEHAVIOUR OF AN EXPERIMENTAL MODEL MADE OF THIN-WALLED COLD FORMED STEEL PROFILES – HARDELL STRUCTURES

BY

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Abstract. The experimental results of the shaking table test of a Hardell structure made of thin-walled cold formed steel profiles are presented. The structure has the in-plane dimensions of 4×4 m and a height of 6 m, being classified as a P+1E type of structure. The dynamic characteristics of the structure were determined during the first stage of the experiment. Afterwards, the structure was subjected to different types of dynamic loadings such as sine-sweep functions and to seismic actions simulating the El Centro and Vrancea earthquakes. The damages induced by the seismic excitations consisted of local buckling of the steel profiles and breaking of two anchorage bolts holding the structure to the shaking table. Based on the experimental results it can be concluded that the structure can safely withstand seismic loads up to certain intensity, provided that some requirements are met.

Key Words: Seismic Behaviour; Shaking Table Tests; Hardell Structures.

1. Introduction

The significant damages and losses of lives caused by earthquakes during the past two decades (1990 Vrancea, Romania; 1994 Northridge, USA; 1995 Kobe, Japan; 1999 Chi-Chi, Taiwan; 2003 Bam, Iran; 2004 Niigata, Japan; 2008 Sichuan, China), with major impact on densely populated urban areas led to a gradual change in the way the public ascertains the structural safety to natural hazards. A notable increase in the number of research works related to the behaviour of different types of structures under seismic excitation could be observed as a consequence of the growing awareness of the general public. This major research topic was treated by means of several approaches starting from the seismic experimental tests on structural elements [1], . . . , [3], tests on scaled models of structures [4], . . . , [6] and even full scale tests on structures [7], [8]. The experimental results were later on confirmed and improved by numerical simulations [9], . . . , [11]. Also, a very important contribution in the field of earthquake engineering was brought by theoretical research works which tried to propose better design procedures and improved theoretical models with respect to the behaviour of structures under seismic excitation [12].

Romania is located in a highly active seismic area. Because of this, the structures have to be designed in such a way that they can withstand earthquakes [13], [14] and ensure the safety of the occupants should such an event occur.

2. Experimental Setup

2.1. The Structural Model

The present paper presents the experimental results of a shaking table test of a Hardell type of structure made of thin-walled cold formed steel profiles, (Fig. 1). The structure has the in-plane dimensions of 4×4 metres and the height of 6 metres, being of the type P+1E.

Fig. 1. – Hardell structure on the shaking table.

The structure is made out of CW 147/52/1.5 steel profiles (channel shape profile with the height of 147 mm, the width of 53 mm and the thickness of 1.5 mm) both for the vertical elements of the wall frame and for the beams in the floors, (Fig. 2). The steel profiles located on the rim of both the wall frame and the floor frame, (Fig. 3), are of the type UW 150/40/1.5 (channel shape profile with the height of 150 mm, the width of 40 mm and the thickness of 1.5 mm). MP 275 steel elements were used for the braces, (Fig. 2). The claddings consisted of wood panels.

In order to ensure a gravitational load corresponding to the live load of a real structure, the model was equipped with additional weights up to 1 kN/m^2, (Fig. 4).

The weights were uniformly distributed on the first and second floor, respectively.

Fig. 2. – CW 147/52/1.5 and MP 275 profiles used for wall and floor frames.

Fig. 3. – UW 150/40/1.5 profiles for the rims of the wall and floor frames.

2.2. The Shaking Table

The shaking table used in this study is an ANCO R3123 tri-axial shaking table with a 43×3 m^2 surface. The three directions along which the table can be controlled are two in the horizontal plane (X and Y) and a vertical direction, Z. The maximum payload of the shaking table is 16 t. The shaking table is able to induce dynamic excitations with a maximum horizontal displacement, at the table

Fig. 4. – Additional weights for loading the structure.

surface, of ±15 cm. The working frequency domain is 0.5 . . . 50.0 Hz, reaching a maximum acceleration (with a 10 t payload) of ±3g and a peak velocity of ±0.8 m/s.

The shaking table is moved by three actuators which are controlled by a fully automated system.

2.3. Data Acquisition

The acquisition of the primary data (the data that controls the shaking parameters of the table) and the data that describes the seismic behaviour of the tested specimen is performed by a digital acquisition system. The Hardell structure was equipped with Dytran 3202A1 LIVM high accuracy accelerometers (which can record data in the range of ±10g, Fig. 5) and PT5AV displacement transducers able to measure displacements up to ±0.5 m, (Fig. 6).

The displacements during the seismic tests were recorded by means of six displacement transducers located at the level of the shaking table (two transducers), the level of the first floor (two transducers) and the upper part of the structure (two transducers), Fig. 7. The accelerometers were located at the level of both floors, along the two in-plane directions, as shown in Fig. 7.

2.4. Loading patterns

The Hardell structure was subjected to the following seismic loads:

Sine-sweep load with constant amplitude in the frequency range from 2 Hz to 10 Hz and a 0.5 Hz/s loading step, (Fig. 8);

Seismic load by means of the time history function of the El Centro (1940) earthquake, east-west component, (Fig. 9);

Seismic load by means of the time history function of the Vrancea (1986) earthquake, east-west component recorded for the city of Bucharest, (Fig. 10).

Fig. 5. – Dytran 3202A1 accelerometer.

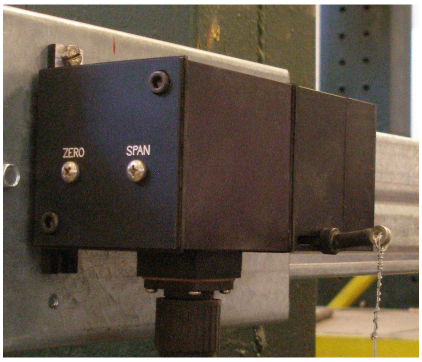

Fig. 6. – PT5AV displacement transducer.

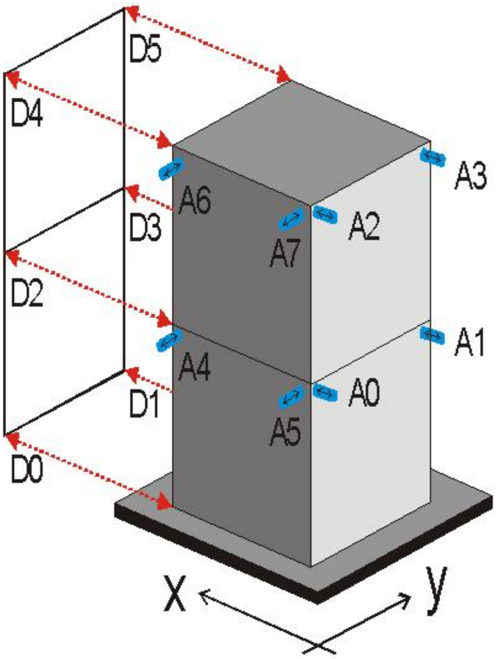

Fig. 7. – Location of the displacement transducers and the accelerometers.

The seismic loads applied to the structure were scaled to different magnitudes of the acceleration in order to study the behaviour of the structure under various earthquake actions.

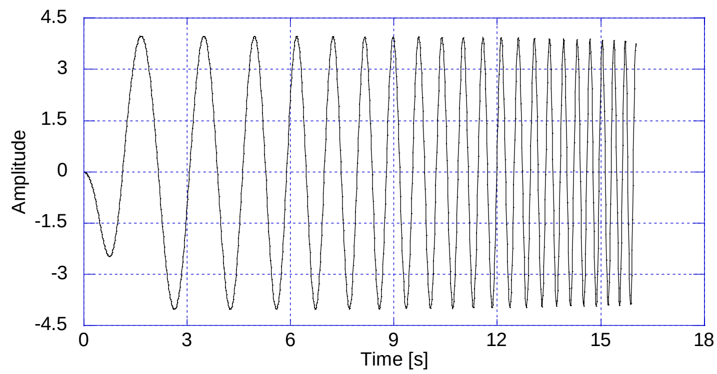

Fig. 8. – Sine-sweep function with constant amplitude.

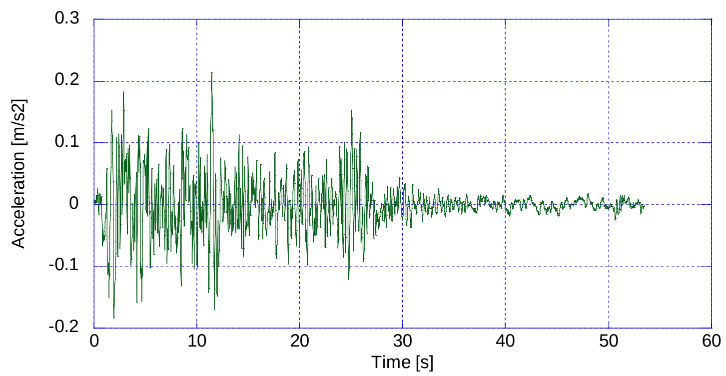

Fig. 9. – Time history for the El Centro (1940) earthquake, E-W component.

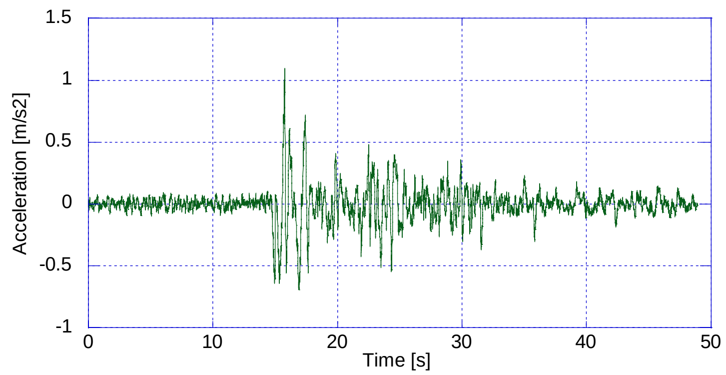

Fig. 10. – Time history for the Vrancea (1986) earthquake, E-W component.

3. Results and Discussions

As mentioned above, the experimental procedure followed three main steps: determine the dynamic characteristics of the model, run the seismic tests, determine the dynamic characteristics of the model after the seismic tests were run in order to theoretically assess the degree of the damage of the structure.

3.1. Dynamic Characteristics of The Structure

The dynamic characteristics of the model (fundamental period of vibration, fundamental frequency of vibration) were determined by means of a sine-sweep loading applied to the structure along the X direction, with a magnitude of 0.1g. Based on the recorded data, the fundamental period of vibration was determined to be $T_{1i} = 0.309$ s ($f_{1i} = 3.248$ Hz), (Fig. 11).

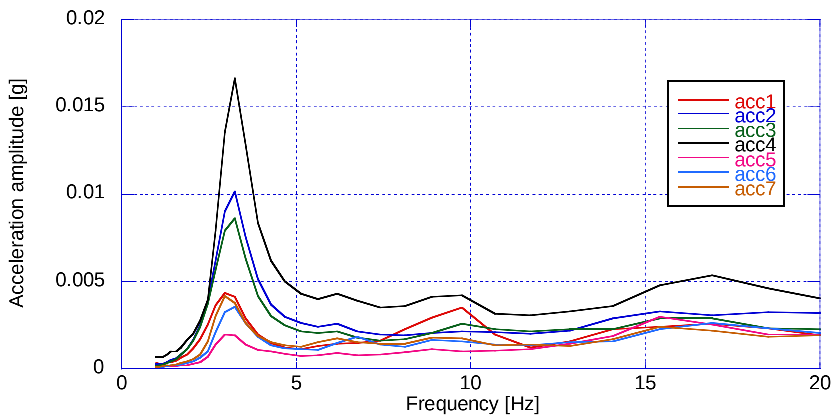

Fig. 11. – Fundamental frequency of vibration of the structure.

After this stage was completed, the model was further subjected to sine-sweep loading patterns with different amplitudes from 0.2g to 0.4g. For each loading stage, the recorded data was thoroughly analyzed in order to determine whether the structure exhibited any damages or not. Based on the analysed data an increase in the fundamental period of vibration of the model could be observed. Thus, for an amplitude of the loading of 0.2g the period of vibration was $T_{1ss02g} = 0.337$ s ($f_{1ss02g} = 2.963$ Hz), for 0.3g it was $T_{1ss03g} = 0.369$ s ($f_{1ss03g} = 2.704$ Hz) and for 0.4g the value further increased to $T_{1ss04g} = 0.399$ s ($f_{1ss04g} = 2.501$ Hz).

A 29% increase in the value of the fundamental period of vibration of the structure was observed at the end of the sine-sweep loading stage. It was concluded that the structure suffered some damages and a visual inspection of the model was deemed necessary. It was observed that some of the bolts fixing the model to the shaking table were lose.

After tightening the bolts, a new sine-sweep type of loading with an amplitude of 0.3g was applied to the structure to check whether the 29% increase in the value of the fundamental period of vibration was caused by the lose bolts or not. The new obtained value was $T_{1ss03g2} = 0.404$ s ($f_{1ss03g2} = 2.474$ Hz). It was therefore concluded that the lose bolts were not the cause of the increase but the structure exhibited damages at the level of the wooden panels of the cladding and that some of the joints of the wall steel frame were weakened.

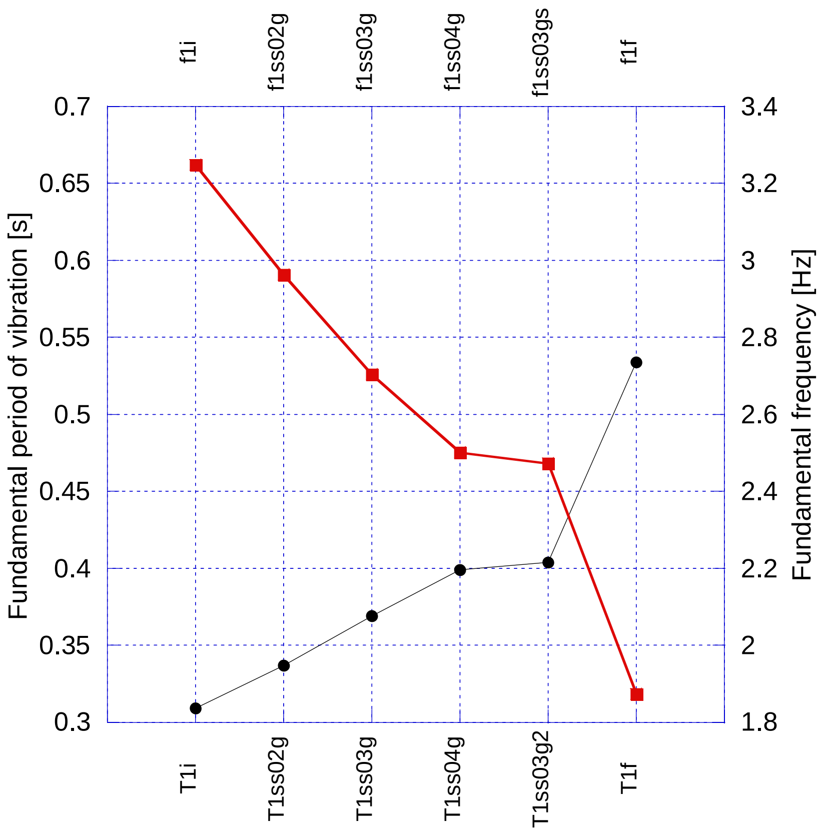

Fig. 12. – Variation of the fundamental period of vibration, T, and fundamental frequency of vibration, f.

Fig. 12 presents the increasing trend of the fundamental period of vibration. Conversely, a decrease in the values of the fundamental frequency of vibration was observed, also shown in Fig. 12.

At the end of the experimental program, the dynamic characteristics of the model were measured once again. The data was recorded only with two accelerometers located at positions *1* and *4*, (Fig. 7). The value of the fundamental period of vibration was computed as $T_{1f} = 0.534$ s ($f_{1f} = 1.874$ Hz). A 73%

increase in the value of T leads to the conclusion that the model exhibited extensive damage at the structural level, as it was later on confirmed by visual inspection.

3.2. Seismic Tests

The model was also subjected to earthquake loads based on the time histories recorded for the El Centro (1940) and Vrancea (1986) earthquakes. The amplitudes of the input ground accelerations for the two earthquakes were scaled to different magnitudes.

For the El Centro earthquake the peak values of the accelerations were between 2.747…3.335 m/s^2. These values exceeded the values given in the Romanian code of seismic design [15]. The largest displacement of 40 mm was recorded when the model was subjected to the maximum acceleration.

In case of Vrancea earthquake, the peak values of the accelerations were between 2.619…6.332 m/s^2. The extreme values exceeded more than two times the maximum values prescribed by the seismic design code in Romania [15]. Even for such high values of the acceleration, the maximum recorded displacement was less than 1% of the structure height (60 mm).

During the visual inspection of the structure at the end of the seismic tests, there could be observed several areas where the damage was located. Most of the damages consisted of local buckling such as the main frame girder at the level of the first floor, (Fig. 13), the vertical steel profiles located at the corners of the structure, (Fig. 14) and at the left lower part of the door frame, (Fig. 15). More severe damage was observed at the interface between the structure and the shaking table: ruptured bolts and cracked corner steel pieces, (Fig. 16).

Fig. 13. – Local buckling of the frame girder at the level of the first floor.

Fig. 14. – Deformation of the vertical steel profile at the corner of the structure.

Fig. 15. – Local deformation of the lower left steel profile of the door frame.

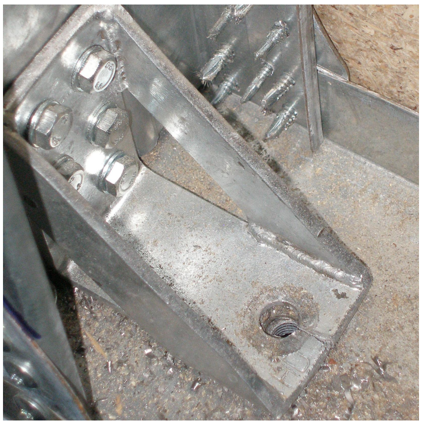

Fig. 16. – Breaking of the anchorage bolt and of the corner steel piece.

4. Conclusions

Based on the analysis of the recorded data it can be concluded that the structure did not exhibit any severe damage up to a value of the applied acceleration of 0.3g. At the end of the experimental program, a 73% increase in the fundamental period of vibration was observed.

The structural damage consisted in local buckling of the steel profiles making up the frame girder of the first floor, of the vertical steel profiles located at the corners of the model and the of the steel profiles of the door frame.

However, it can be concluded that the structural system can safely withstand earthquake loads provided that the following design requirements are met:

1. A shape factor $q = 1$ should be considered in the seismic design of this type of structures.

2. The entire load at the floor level should be resisted by the stiffenening frame (the frame with braces) corresponding to that particular floor.

3. the load carrying capacity of the joint between the stiffening frame and the infrastructure of the building should be at least 1.2 times greater that the carrying capacity of the frame itself.

4. The stiffening frames should have continuity along the height of the structure (the frames from the upper floor should be connected directly to the frames from the lower floor).

5. The stiffening frames should be placed in such a way so that to avoid the effect of torsion in the structure during an earthquake.

6. In case of large spans, the horizontal beams should have closed cross-section so that to ensure a 20% larger load carrying capacity in the elastic range than the one resulted from the seismic calculations.

7. The diagonal braces should be connected to all the vertical elements of the wall frame thus providing a better and more even distribution of the stresses in the stiffening frames.

Received, March, 25, 2009

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COMPORTAMENTUL SEISMIC AL UNUI MODEL EXPERIMENTAL DIN ELEMENTE ŞI SUBANSAMBLE DIN TABLĂ ZINCATĂ PROFILATE LA RECE – STRUCTURI TIP HARDELL

(Rezumat)

Se prezintă rezultatele experimentale obţinute în urma încercării pe platforma seismică a unei structuri de tip Hardell alcătuită din profile cu pereţi subţiri formate la rece. Structura are dimensiunea în plan de 4 x 4 m şi înălţimea de 6 m, fiind de tipul P+1E. În prima fază a programului experimental s-a procedat la determinarea caracteristicilor dinamice ale structurii. Ulterior, structura a fost supusă la acţiuni seismice de tip sinus glisant şi la acţiuni seismice corespunzătoare cutremurelor de pământ El Centro şi Vrancea. Degradările structurii după aplicarea succesivă a tuturor acţiunilor seismice sunt: flambajul local al profilelor metalice ce formează grinda de centura a pereţilor şi a planşeului de peste parter, deformări ale profilelor metalice montanţi la colţurile structurii, la partea inferioară a montantului de la golul de uşă, precum şi ruperi ale şuruburilor de ancoraj de la montantul marginal al profilului de rigidizare pentru pereţi. În urma încercărilor efectuate asupra modelului se constată că sistemul structural poate prelua încărcările din acţiunea seismică dacă sunt respectate anumite condiţii.