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# BLIND PREDICTION OF THE SEISMIC RESPONSE OF THE NEESWOOD CAPSTONE BUILDING

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**ABSTRACT:** The NEESWood Project is a multi-year US research project that involves analysis, testing, and societal risk assessment with the intent of safely increasing the height of light-frame wood buildings to six stories in regions of moderate to high seismicity. Within this project a full-scale seven-storey, 12.1 m x 18.1 m, condominium building (one storey steel frame and 6 storey wood frame construction) has been tested during July 2009 on the world's largest earthquake shake table in Miki, Hyogo, Japan.

As part of the NEESWood Project the international engineering community was invited to blind predict the inelastic seismic response of the Capstone Building. In this paper results of the blind prediction using the commercially available DRAIN 3-D structural analysis program are presented. The model for the test structure was composed of essentially rigid straight members connected to semi-rigid rotational springs in the vertical plane to represent the shear walls, while floor and roof diaphragms were assumed as rigid. The semi-rigid spring elements were incorporated into the DRAIN-3D program using a proprietary subroutine simulating the hysteretic behaviour of wood mechanical connections. Properties of the hold-down rods were also included in the model. The required hysteretic parameters for each spring element were obtained by the data package provided by NEESWood researchers for this benchmark study. The results were then compared in terms of time-history responses, maximum base shear, maximum average displacements, inter-storey drifts and hold-down tension forces experienced at each storey.

**KEYWORDS:** NEESWood capstone building, blind prediction, DRAIN-3D, seismic analysis, base shear, inter-storey drift, hold-down forces.

## 1 INTRODUCTION

The NEESWood Project is a five-university project led by Professor John van de Lindt at Colorado State University that involves analysis, testing, and societal risk assessment with the intent of safely increasing the height of platform wood frame buildings to six stories in regions of moderate to high seismicity. Under the NEESWood Project, shake table tests were conducted on two full-scale platform wood frame buildings. The first building, which is a two-storey wood frame townhouse, was tested at the State University of New York at Buffalo's Structural Engineering and Earthquake Simulation Laboratory (SESL) in 2006 to benchmark the performance of current engineered wood frame construction. The second building, a six-storey wood frame condominium building on one-storey steel frame structure (also called NEESWood Capstone building), was tested on the world's largest earthquake shake table in Miki, Hyogo, Japan during the summer of 2009. The main objective of the NEESWood Project is to provide experimental results which will be used to confirm that a representative mid-rise wood-frame building designed according to the performance-based seismic design (PBSD) philosophy satisfies the performance objectives, as pre-defined during the design process [1]. In addition, they will provide a general understanding of the behaviour of a mid-rise wood frame structure in regions of moderate to high seismicity.

Moreover the results of the shaking table tests would also be used to validate available nonlinear models for seismic analysis of wood frame structures, which build the platform upon which the PBSD philosophy is developed. That is why, as part of the NEESWood Project the international engineering community was invited to blind predict the inelastic seismic response of the Capstone Building.

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# 2 SHAKING TABLE TESTS OF NEESWOOD CAPSTONE BUILDING

## 2.1 Description of Test Specimen

The tested building had a plan dimension of approximately 18 m × 12 m and was about 17m tall. As can be seen from the elevation views in Figure 1, the building showed a significant amount of openings on the 4 sides.


***Figure 1:*** *Elevation views of the six-storey test specimen*

The floor plans for storeys 1 to 6 are shown in Figure 2, along with locations and denomination of the shear walls. The 1st storey consisted of 2 small one-bedroom (Unit C and D) units and 2 two-bedroom units. The floor plans for 2nd to 5th storey are similar to the 1st storey plan, except for a small change for units C and D since no entrance door was required. The floor plan for the top storey (storey 6) was changed since a large two-bedroom unit (unit A) instead of the 2 two-bedroom units of the below storeys was foreseen, thus meaning a change in the shear wall schedule with some of the shear walls of storey 5 not extending to storey 6.

At each storey a seismic mass made of steel plates fastened to each floor and roof was added in order to account for dead loads due to construction materials used for the floor construction, including insulation, plumbing and floor finishing. The total weight at each storey, including the self weight of the structure, is summarized in Table 1.

***Table 1:*** *Total weight on the test specimen at each floor*

| Storey | Weight (kN) |
|---|---:|
| 1st | 471.7 |
| 2nd | 451.3 |
| 3rd | 445.3 |
| 4th | 448.8 |
| 5th | 482.3 |
| 6th | 288.6 |
| Total | 2587.9 |

As a design option for light commercial space for the wood-frame building a steel moment frame was added under the building. The frame also played an important role in order to move the building and to lift it over the shaking table, and, after the building was installed, has been connected and tightened to the shaking table. The steel frame, rigidly connected to the table, behaved during the tests like and extension of the shaking table. The test specimen over the shaking table is shown in Figure 3.

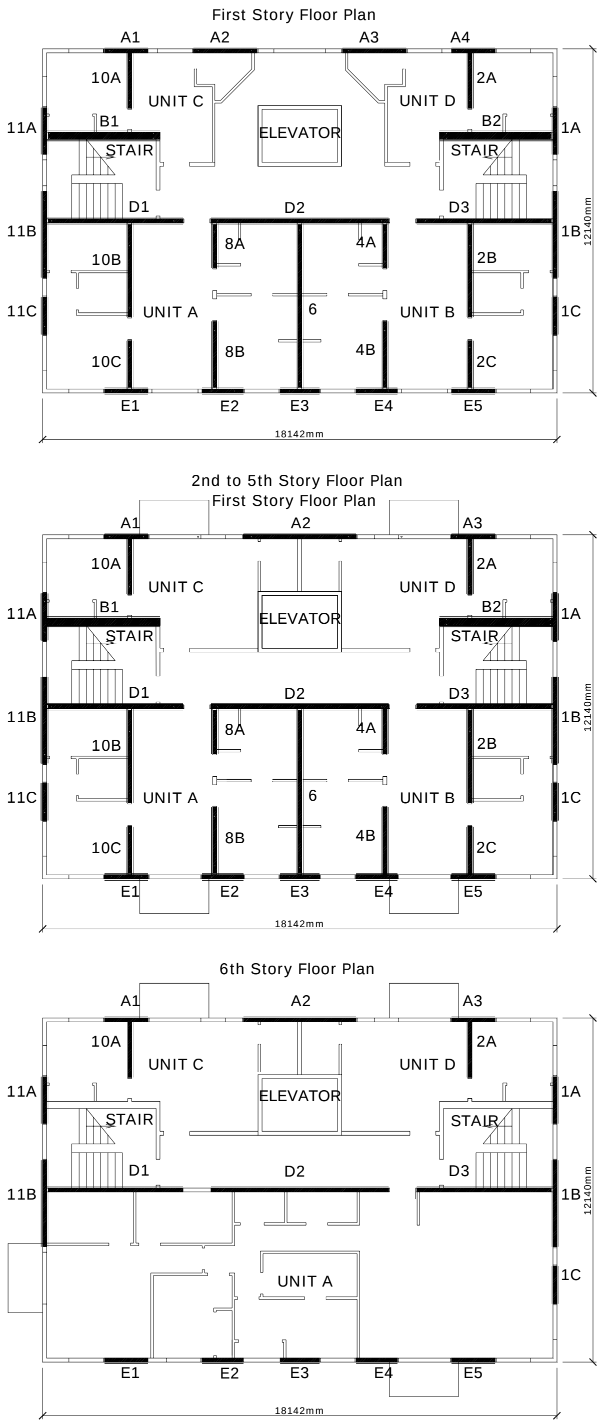


***Figure 2:*** *Floor plans for storey 1, 2-5 and 6*

Except walls B1 and B2, the shear walls were built with 38 mm × 140 mm (2 in. × 6 in.) D. Fir (in the lower 3 storeys) and S.P.F (in the upper 3 storeys) dimension lumber with studs spaced at 406 mm on center. The 10d common nails (3.76 mm in diameter) were used to fasten the 12mm (15/32 inch) OSB panels to the framing members. Continuous steel rods with mechanical shrinkage compensating devices were used at each end of shear wall to prevent overturning. Details of the shear wall information can be found in [2]. Due to the high lateral loads exerted on the walls B1 and B2, a high capacity wall system, midply wall, was used. Details of the midply wall used in the NEESWood Capstone building can be found in [3].

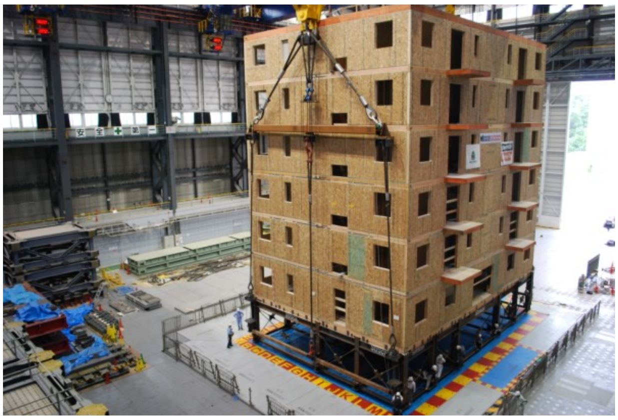

***Figure 3:*** *Test building over the shaking table (courtesy of J.W. De Lindt)*

### 2.2 Seismic test program

The six-storey wood frame building was subjected to three, tri-axial earthquake motions having probabilities of exceedance of 50%, 10%, and 2% in 50 years, which represents earthquakes with 72 year return period, 475 year return period and 2,500 year return period, respectively. All earthquakes used in the testing were scaled motions of the Northridge Earthquake that occurred in California in 1994, recorded at Canoga Park scaled to peak ground accelerations of 0.22g, 0.5g, and 0.8g respectively (Table 2). Figure 4 shows the spectral accelerations in the X, Y, and Z directions of the un-scaled Canoga Park record, with the Y-component (which has a higher PGA value) applied in the long direction of the building.

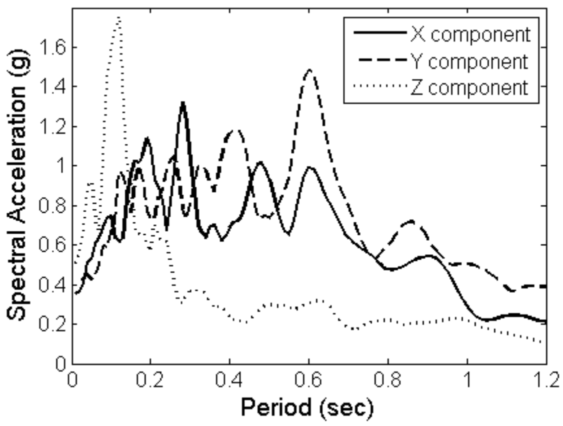


***Figure 4:*** *Unscaled acceleration response spectra for the Canoga Park recording of the 1994 Northridge earthquake (5% damping)*

On June 30th and July 6th, the building was subjected to 60% (50% in 50 year) and 120% (10% in 50 year) of the original ground motion recorded at Canoga Park during the Northridge Earthquake. The building tested on June 30th had dampers installed in the first storey steel frame. The same tests were repeated on July 6th on the building with the first storey steel frame locked down to minimize the influence of the first storey on the response of the building. On July 14th, the building, with the first storey steel frame locked down, was subjected to 180% (2% in 50 year) of the original ground motion recorded at Canoga Park during the Northridge Earthquake. The scaled earthquake record represents the Maximum Credible Earthquake given in the building code for design in California.

***Table 2:*** *PGA and hazard levels for the three tests [4]*

| Northridge Canoga Park | | Seismic Test | | |
|---|---|---|---|---|
| | | Level 1 | Level 2 | Level 3 |
| Hazard level | | 50% 50 years | 10% 50 years | 2% 50 years |
| Scaling factor | | 0.53 | 1.20 | 1.80 |
| PGA (g) | X | 0.19 | 0.43 | 0.64 |
| | Y | 0.22 | 0.50 | 0.76 |
| | Z | 0.26 | 0.59 | 0.88 |

### 2.3 Shaking table test results

The building performed extremely well during the shaking and survived the strongest quake (Level 3) without any significant structural damage. As reported in [5] the maximum average displacement obtained at the roof level relative to the shaking table in the long direction (Y) was 60, 140 and 211 mm for Level 1, 2 and 3, respectively. During the MCE (Level 3) test the building showed clearly a torsional response. Measured accelerations on the shaking table showed some significant differences with the input records, especially for the Level 3 test, as shown in Table 3.

***Table 3:*** *Input and measured PGA for the three level test*

| Northridge Canoga Park | | Seismic Test | | |
|---|---|---|---|---|
| | | Level 1 | Level 2 | Level 3 |
| | | PGA (g) | PGA (g) | PGA (g) |
| Input | X | 0.19 | 0.43 | 0.64 |
| | Y | 0.22 | 0.50 | 0.76 |
| | Z | 0.26 | 0.59 | 0.88 |
| Measured | X | 0.19 | 0.47 | 0.72 |
| | Y | 0.31 | 0.85 | 1.43 |
| | Z | 0.22 | 0.61 | 0.88 |

The measured natural period of the building was 0.41s, even if it is not specified in which direction [5]. The maximum base shear and the maximum average absolute displacements, measured at the top storey, are reported in Table 4.

***Table 4:*** *Maximum base shear and average maximum top storey displacements for the three level test [5]*

| Shake table tests | Base shear (kN) | | Maximum average top storey displacements (mm) | |
|---|---|---|---|---|
| | X | Y | X | Y |
| Canoga Level 1 | 477 | 716 | 40 | 60 |
| Canoga Level 2 | 1033 | 1445 | 90 | 140 |
| Canoga Level 3 | 1384 | 1824 | 142 | 211 |

In Table 5 the maximum averaged inter-storey drifts for each storey, expressed in percentage relative to the storey height, are reported.

***Table 5:*** *Peak average inter-storey drift for three earthquake levels at each storey [5]*

| Shake table tests | | Peak average inter-storey drift (%) | | | | | |
|---|---|---|---|---|---|---|---|
| | | St1 | St2 | St3 | St4 | St5 | St6 |
| Canoga Level 1 | X | 0.26 | 0.35 | 0.29 | 0.30 | 0.36 | 0.40 |
| | Y | 0.44 | 0.42 | 0.54 | 0.44 | 0.46 | 0.21 |
| Canoga Level 2 | X | 0.49 | 0.63 | 0.64 | 0.77 | 0.64 | 0.88 |
| | Y | 0.77 | 1.05 | 1.02 | 1.22 | 1.14 | 0.58 |
| Canoga Level 3 | X | 0.84 | 0.97 | 0.89 | 1.10 | 1.00 | 1.35 |
| | Y | 1.12 | 1.46 | 1.64 | 1.48 | 1.88 | 1.11 |

The maximum inter-storey drifts occurred in the long (Y) direction of the building, with the maximum values occurred at the 3rd storey for the Level 1 test, 4th storey for Level 2 test and 5th storey for Level 3 test.

In Table 6, the maximum steel rod tension forces recorded during the Level 3 test are reported (the reported value is the maximum between the two ends). For reference the location of each wall is shown in Figure 5.

***Table 6:*** *Maximum steel rod tension forces.*

| Wall | Storey | Force (kN) |
|---|---|---|
| D2 | 1 | 252.3 |
| B1 | 1 | 767.6 |
| A2 | 1 | 174.5 |
| 6 | 1 | 272.4 |
| 11B | 1 | 229.9 |
| B1 | 2 | 602.2 |
| 6 | 2 | 235.3 |
| B1 | 3 | 360.0 |
| 6 | 3 | 147.0 |
| B1 | 4 | 198.5 |
| 6 | 4 | 49.1 |
| B1 | 5 | 70.0 |
| 6 | 5 | 22.7 |

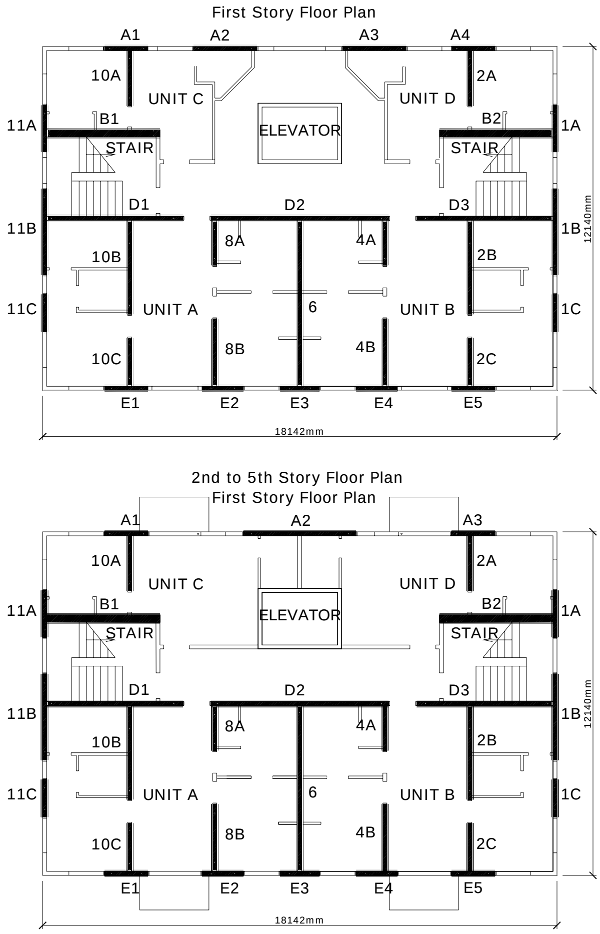


***Figure 5:*** *Denomination and location of shear wall at storey 1 and 2-5*

# 3 NUMERICAL MODEL AND PREDICTIONS

## 3.1 Description of the numerical model

The analytical model was a 3D space frame model constructed using the commercially available DRAIN-3D structural analysis program, in which, a hysteretic model with pinching behaviour developed at the University of Florence [7], [8] was implemented. The space frame is composed of rigid straight members connected to semirigid rotational springs to represent the shear walls, and a double simple connection element with an elastic-perfectly plastic behaviour in tension and a linear elastic behaviour in compression to represent hold-downs. Masses have been uniformly distributed on each floor and lumped on model nodes. Floor and roof diaphragms were considered infinitely rigid in their plane and schematized by means of equivalent rigid cross bracing elements. The Steel Moment Frame was not modelled. The layout of the 3D model with the denomination of each wall is shown in Figure 6.

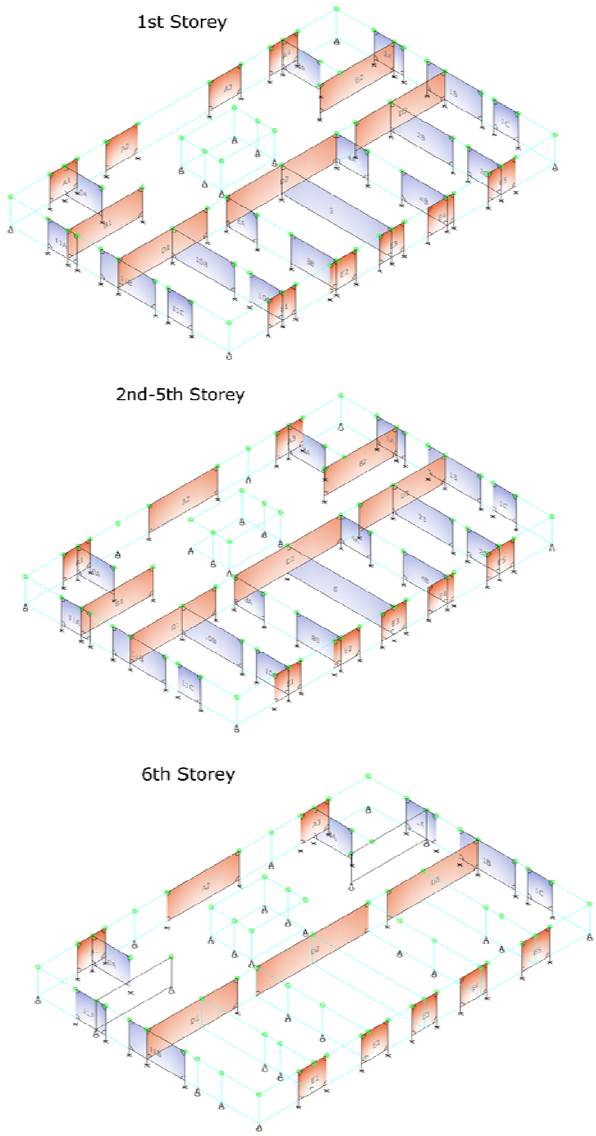


***Figure 6:*** *3D space frame model of the test structure*

Each wall is composed of four rigid straight beam and column elements and four semi-rigid elements at the corners (Figure 7). The masses were lumped in the two upper joints of the panel. The walls were connected to the foundation and to the storey below by means of two hold-downs placed at the bottom nodes.

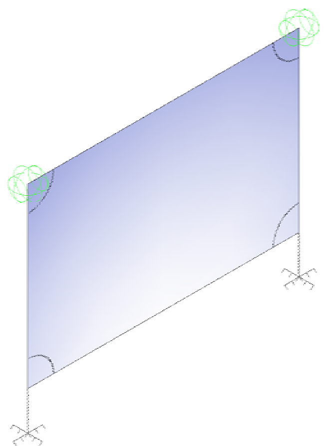

***Figure 7:*** *Walls schematization*

The wall behaviour is simulated by the four semi-rigid rotational springs which include the hysteretic model. The algorithm used is a piecewise tri-linear fitting of the cycles obtained from test data with six different inclinations, as shown in Figure 8. The algorithm doesn't take into account any impairment of strength.

The required hysteretic parameters for each spring element were obtained by the shear wall data package provided by the Colorado State University (CSU) for this benchmark study. Each shear wall, which was sheathed with OSB on one side and gypsum wall board on the other side, was modeled by applying both, the hysteresis parameters of OSB and gypsum wallboard (GWB) sheathing. This was obtained by modeling the wall as two walls one that has the hysteretic parameters for the wall with OSB sheathing and the other for wall with GWB sheathing (Figure 9).

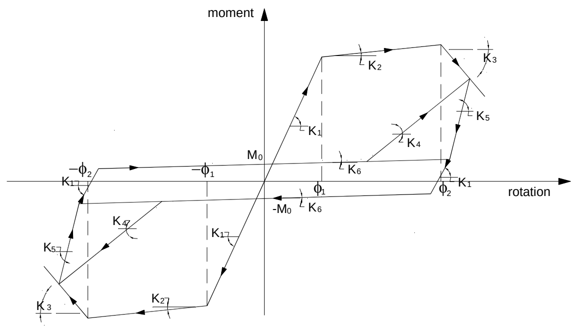


***Figure 8:****Pinching hysteresis model with six inclinations used to model the shear wall behaviour*


***Figure 9:*** *Schematization for a double sheathed shear walls.*

Hold-downs and bearing of top and bottom plates at end studs have been represented using a simple connection element (Element type 04) already included in DRAIN-3D. Each hold-down was represented by two elements working in parallel: the first one working only in tension with an elastic-perfectly plastic behaviour represents hold-downs at the ends of a shear wall and the second one working only in compression with a linear elastic behaviour represents bearing of top and bottom plates at end studs of a shear wall. The stiffness parameters for hold-downs elements working in tension were obtained from the producer of the tie rods (Simpson Strong Tie) , while for the compression element the stiffness value was calculated taking into account the perpendicular to the grain MOE of top and bottom plates. Hysteresis model for the two simple connection elements working in parallel used to model the hold-downs and bearing of top and bottom plates is shown in Figure 10.

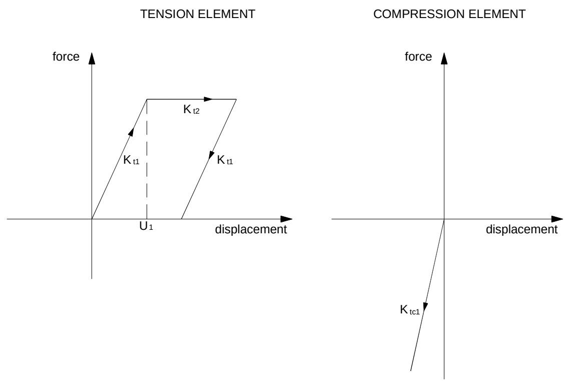


*Figure 10:* *Hysteresis model for the two simple connection elements working in parallel used to model the hold-downs and bearing of top and bottom plates at end studs of a shear wall*

### 3.2 Non linear analysis

The non-linear time-history analyses were performed giving simultaneously the three acceleration records in the three directions, X, Y and Z for the three different levels. The records used in the analysis were not the input records but the acceleration records measured on the table during the test, which, as explained before, showed different values of PGA from the input records. The results of the modal analysis are shown in Table 7 where the percentage of participating mass as a fraction of total mass is also indicated for each mode.

***Table 7:*** *Results of modal analysis*

| Mode | Period (s) | Effective modal mass as a fraction of total mass (%) | | |
|---|---|---|---|---|
| | | X | Y | Z |
| 1 | 0.46 | 76.0 | 1.7 | 0.0 |
| 2 | 0.45 | 5.6 | 44.7 | 0.0 |
| 3 | 0.42 | 0.7 | 31.7 | 0.0 |
| 4 | 0.18 | 12.1 | 0.0 | 0.0 |
| 5 | 0.16 | 0.0 | 13.3 | 0.0 |
| 6 | 0.15 | 0.4 | 0.7 | 0.0 |

A 2% stiffness proportional damping was applied to the model, defined on the 1$^{st}$ mode, to account for the contribution given by the partition walls. The results of the modal analysis show a good agreement with the measured natural period from the test, with a difference of 12%.

## 4 RESULTS OF ANALYSIS AND COMPARISON WITH TEST RESULTS

In this section the results of the non-linear analysis performed with DRAIN 3D are reported with the difference in percentage with the test results.

### 4.1 Maximum displacement and base shear

The maximum base shear and the results of the average maximum displacements measured at the top storey are presented in Table 8, with the percentage difference with test results given in parenthesis.

***Table 8:*** *Maximum base shear and average maximum top storey displacements for the three level tests and from the numerical analysis*

| Shake table tests | Base shear (kN) | | Maximum average top storey displacement (mm) | |
|---|---|---|---|---|
| | X | Y | X | Y |
| Canoga Level 1 | 729 (53%) | 697 (-3%) | 52.8 (32%) | 52.4 (-13%) |
| Canoga Level 2 | 1232 (19%) | 1560 (8%) | 111.8 (24%) | 183.3 (31%) |
| Canoga Level 3 | 1663 (20%) | 1727 (-5%) | 167.9 (18%) | 249.2 (18%) |

The results showed reasonable agreement.

### 4.2 Maximum inter-storey drift

In Table 9 the maximum averaged inter-storey drifts for each storey, expressed in percentage relative to the storey height, are shown. The percentage difference with test results is given in the parenthesis.

***Table 9:*** *Peak average inter-storey drifts at each storey for the 3 earthquake levels obtained from the analytical study*

| Shake table tests | | Peak average inter-storey drift (%) | | | | | |
|---|---|---|---|---|---|---|---|
| | | St1 | St2 | St3 | St4 | St5 | St6 |
| Canoga Level 1 | X | 0.18 (-32%) | 0.25 (-27%) | 0.30 (3%) | 0.34 (13%) | 0.37 (2%) | 0.48 (19%) |
| | Y | 0.17 (-61%) | 0.29 (-30%) | 0.32 (-41%) | 0.41 (-7%) | 0.41 (-11%) | 0.29 (37%) |
| Canoga Level 2 | X | 0.33 (-33%) | 0.48 (-24%) | 0.61 (-4%) | 0.73 (-6%) | 0.82 (28%) | 1.06 (20%) |
| | Y | 0.42 (-46%) | 0.74 (-30%) | 0.97 (-5%) | 1.30 (7%) | 1.77 (56%) | 1.40 (142%) |
| Canoga Level 3 | X | 0.49 (-42%) | 0.70 (-28%) | 0.89 (0%) | 1.05 (-4%) | 1.23 (23%) | 1.64 (21%) |
| | Y | 0.45 (-60%) | 0.83 (-43%) | 1.13 (-31%) | 1.65 (12%) | 2.50 (33%) | 2.37 (113%) |

In general, the results show reasonable agreement in the short direction of the building (X direction) for all the tests, with predicted lower displacement for storeys 1 and 2 and higher displacement for storeys 5 and 6. Similar trend was also observed in the long direction of the building (Y direction), with largest differences occurred in storey 6 under Level 2 and 3 tests.

Based on the shear wall hysteresis loops provided by CSU, it is noticed that the initial stiffness for walls A1, A3, E1, E2, E3, E4 and E5 in the 6th storey is approximately 1/4 of the same stiffness at the 5th storey. This is a drastic reduction considering that the same stiffness does not vary that much in the lower stories. This may explain why large displacement in the 6th storey in the Y direction was obtained in the analysis for Level 2 and 3 tests.
Figures 10 to 18 show the time-histories recorded in the centre point of Storey 1, 3, 4 and 6 from the numerical analysis compared to the test results under Level 3 test in X and Y direction.

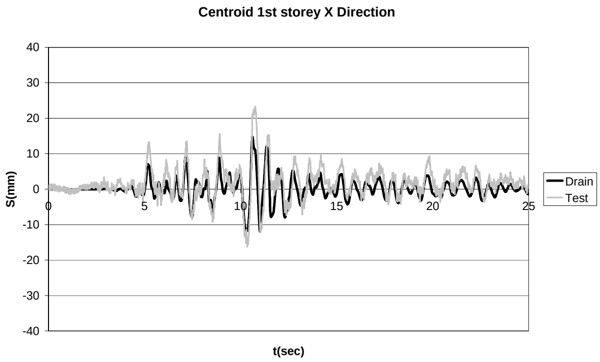


***Figure 11:*** *Time- history comparison between numerical analysis and test results - Storey 1 in X direction (short direction)*

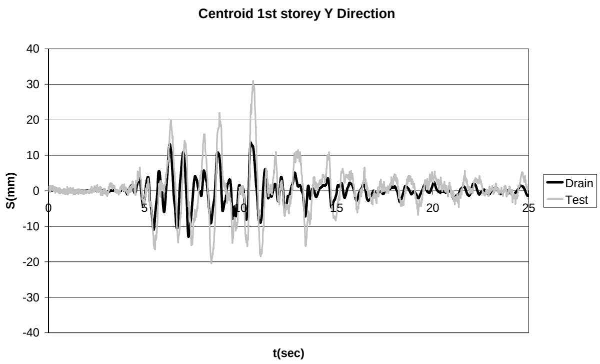


***Figure 12:*** *Time- history comparison between numerical analysis and test results at Storey 1 in Y direction (long direction)*

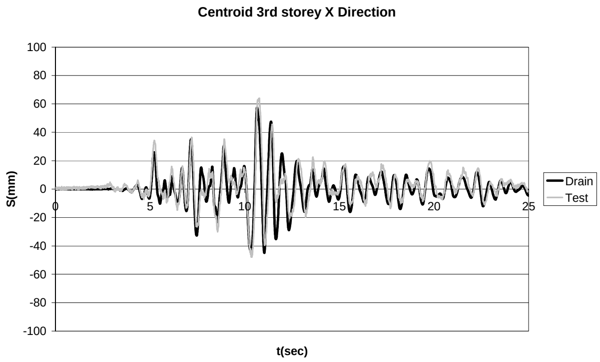


***Figure 13:*** *Time- history comparison between numerical analysis and test results - Storey 3 in X direction (short direction)*

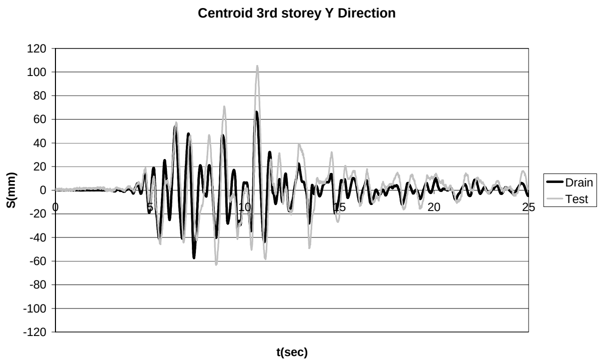


***Figure 14:*** *Time- history comparison between numerical analysis and test results at Storey 3 in Y direction (long direction)*

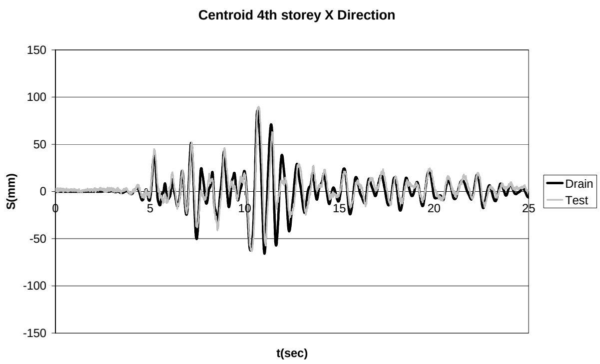


***Figure 15:*** *Time- history comparison between numerical analysis and test results - Storey 4 in X direction (short direction)*

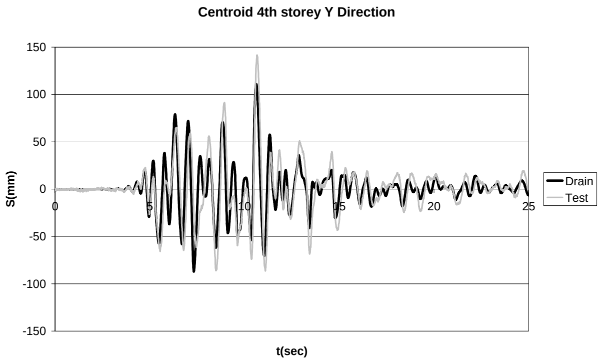


***Figure 16:*** *Time- history comparison between numerical analysis and test results at Storey 4 in Y direction (long direction)*

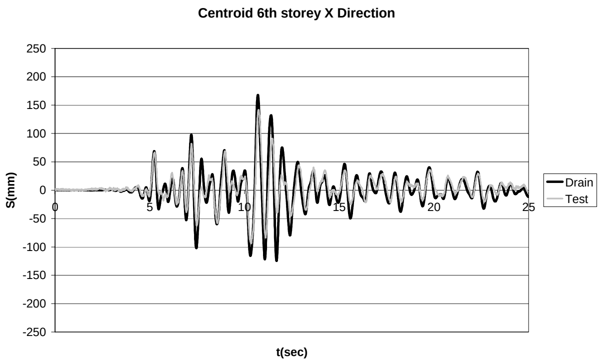


***Figure 17:*** *Time- history comparison between numerical analysis and test results at Storey 6 in X direction*

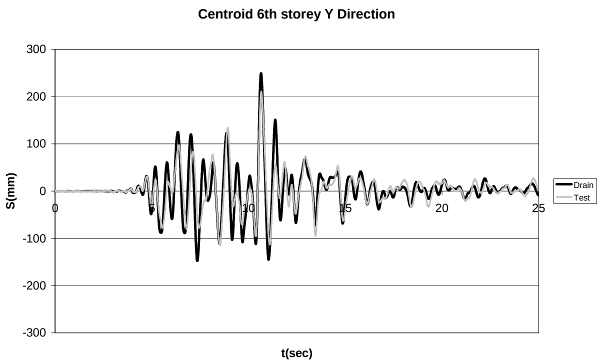


***Figure 18:*** *Time- history comparison between numerical analysis and test results at Storey 6 in Y direction*

Figures 19 to 22 show the deformed shape of the building at the time where the maximum displacement at the top of the building was reached under the Level 3 test.

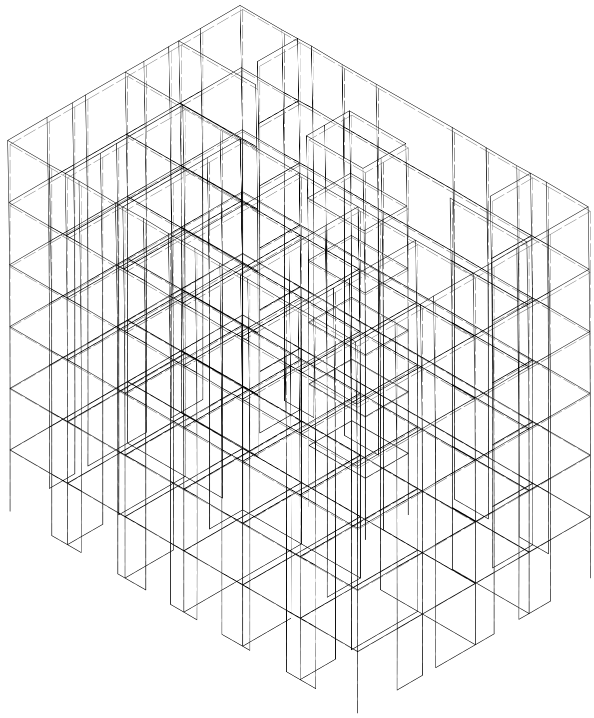

***Figure 19:*** *Axonometric view of the deformed shape of the model at the maximum displacement*

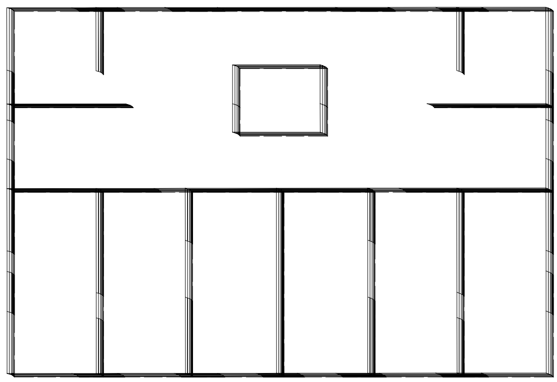

***Figure 20:*** *Top view of the deformed shape of the model at the maximum displacement*

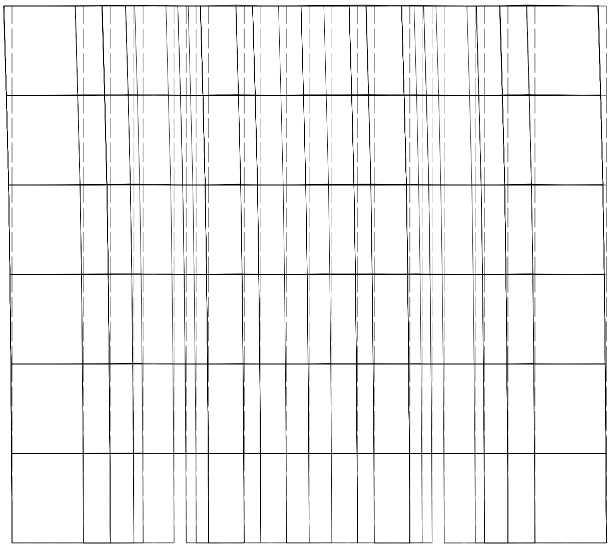

*Figure 21: Front view of the deformed shape of the model in the long (Y) direction at the maximum displacement*

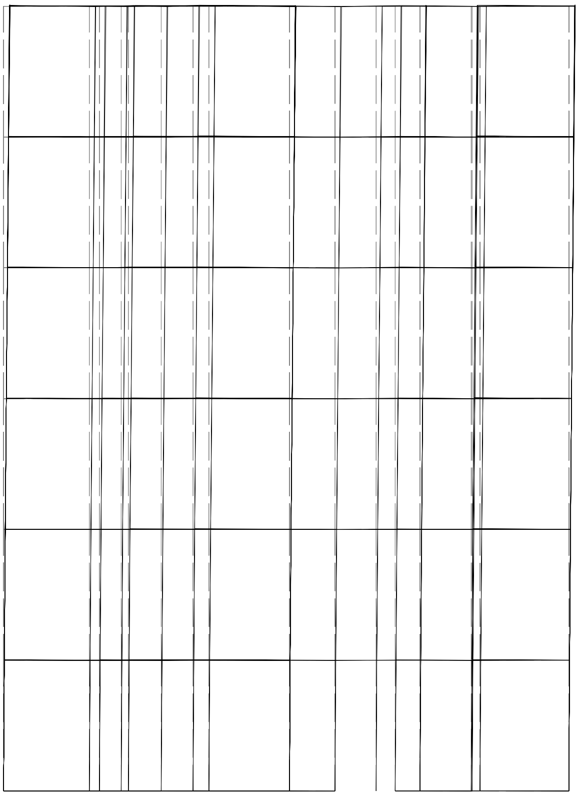

*Figure 22: Front view of the deformed shape of the model in the short (X) direction at the maximum displacement*

As it can be observed from the time-history comparison and from the deformed shape of the building the maximum inter-storey drift occurred in the 5$^{th}$ storey in the Y direction and a slight torsional deformation of the building can be observed, even if it is not so evident as it was during the test.

### 4.3 Maximum hold-down forces

In Table 10 the maximum steel rod tension forces from the numerical analysis are reported for the Level 3 test together with the percentage difference with the test results.

***Table 10:** Maximum hold-down tension forces from the numerical analysis*

| Wall | Storey | Force (kN) | Difference |
|---|---|---|---|
| D2 | 1 | 284.3 | 13% |
| B1 | 1 | 671.2 | -13% |
| A2 | 1 | 112.8 | -35% |
| 6 | 1 | 204.8 | -25% |
| 11B | 1 | 455.7 | 98% |
| B1 | 2 | 521.8 | -13% |
| 6 | 2 | 199.0 | -15% |
| B1 | 3 | 373.8 | 4% |
| 6 | 3 | 125.4 | -15% |
| B1 | 4 | 190.5 | -4% |
| 6 | 4 | 87.4 | 78% |
| B1 | 5 | 79.2 | 13% |
| 6 | 5 | 35.7 | 57% |

The results show a general good agreement except for wall 11B at storey 1 and wall 6 at storey 4 and 5.

## 5 CONCLUSIONS

Blind theoretical predictions for the three levels of NEESWood earthquake simulation test of a 6-storey wood-frame building were made. The predictions are, in general, in reasonable agreement with the measured natural period of the building, base shear, maximum average top storey displacements, inter-storey drift and hold-down tension forces as per determined from the tests.


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