Multi-record Incremental Dynamic Analysis of an IBC-Designed Wood Frame Building

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Abstract:

The current International Building Code (IBC) allows wood frame (light-frame wood) construction to be three stories high or four stories if sprinklers are included in the design. Several jurisdictions within the U.S. have opted to allow four stories and five if sprinklers are present. This paper presents a comprehensive numerical analysis of a six-story wood frame building designed based on the 2006 IBC methodology. Incremental Dynamic Analysis is used to numerically investigate the performance of the building based on three criteria: Criteria 1 is the inter-story drift at each level recorded as the geometric center of the story level; Criteria 2 is the drift for the controlling wall in the building; and Criteria 3 is the peak drift at roof level. The building was found to perform very well even without the consideration of gypsum wall board (GWB) at low to moderate levels of excitation up to the design-basis earthquake. However, excessive drifts in all three cases were identified at the Maximum Credible Earthquake (MCE) level when GWB was not included in the analysis. When GWB was included in the analysis the performance of the structure was very good until a spectral acceleration of 1.2g was reached.

KEYWORDS: performance assessment, IDA, IBC 2006, wood structure, seismic design

1. INTRODUCTION

The NEESWood project is a five-university collaboration between Colorado State University, Texas A & M University, the University at Buffalo, Rensselaer Polytechnic Institute, and the University of Delaware, whose objective is to enable economic and safe construction of mid-rise wood frame buildings through the development and demonstration of a performance-based seismic design (PBSD) philosophy for mid-rise wood frame buildings. The culmination of that project is full-scale 3-D testing of a six-story 1,400 m^2 23-unit condominium on the world's largest earthquake shake table in Miki City, Japan.

The performance-based seismic design of the building consists of 3 major steps: **Step 1:** Design of the building based on the 2006 IBC Methodology. This design was led by K. Cobeen of Cobeen and Associates in collaboration with researchers at Colorado State University. **Step 2:** Analysis of the building using a software package developed for time domain analysis of wood frame buildings to uni-axial or bi-axial ground motion. Incremental Dynamic Analysis (IDA) will serve as the investigative tool to assess the performance in terms of inter-story drifts at each floor level. **Step 3:** Application of the PBSD philosophy being developed within the NEESWood project as a re-design of the building. This paper focuses on Step 1 and Step 2 of this process keeping the focus on force-based design and the SAPwood (numerical) analysis.

2. Architectural Overview

Six-story buildings typically have more than 1,000 m^2 per story. However, the shake table in Japan is 15 m x 20 m and, although the largest in the world, it places a limitation on the building footprint. Figure 1 shows the 1st floor plan view as an example. The basic design parameters for the building are presented in Table 1. The building was sized to provide a perimeter walkway on the shake table of one meter. The elevation views are presented in Figure 2. A tapered joist roof with a 1-meter parapet with service equipment was including in lieu of a metal plated truss roof since this would be more typical in an urban setting.

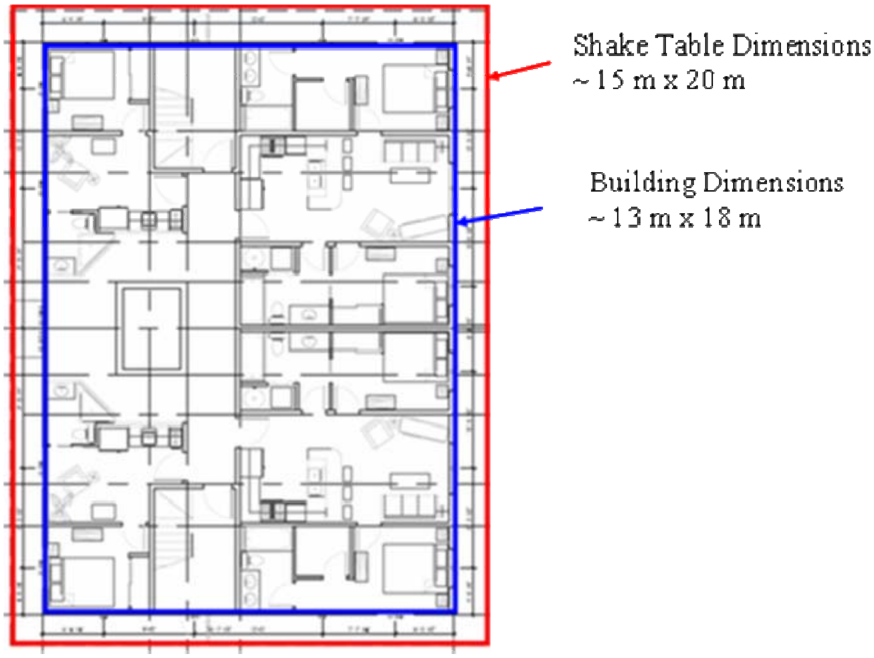


Figure 1: 1st floor level of plan view of six-story building

Figure 2: Elevation view of six-story building

3. Design Using the 2006 International Building Code

The Capstone 6-story wood-frame building is comprised of 23 single family dwelling units. The first through the fifth levels have (2) two bedroom units and (2) one bedroom units, and the sixth level has (2) one bedroom units and (1) two bedroom penthouse unit. There is a single main corridor in the center of the structure which connects (2) stair cores for means of egress and an elevator shaft which services all 6 levels.

The forced based design procedure was based on the 2006 International Building Code (IBC) and the 2005 American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7). The building codes provide engineering recommendations and requirements that the structural design must meet for both the vertical and lateral force resisting systems.

According to Section 310 of the IBC the multi-unit structure was categorized as Residential Group R-2, which describes the building use and occupancy as containing more than two dwelling units where the occupants are primarily permanent in nature. Section 602 of the IBC classified the wood framed construction as Type VA which requires exterior and interior bearing walls, floor and roof assemblies to have a fire-resistance rating of 1 hour. Section 504 of the IBC provides building height limits and per Table 503 the allowable height for Group R-2 occupancy and Type VA construction is limited to 15.3 meters or 3 stories. Section 504.2 allows for the maximum number of stories to be increased by 1, and the maximum height to be increase by 6.1 meters, but shall not to exceed 18.3 meters or 4 stories if the building is equipped with an approved automatic sprinkler system which conforms to code requirements. The Capstone building was based on the seismic methodology for design, but did not adhere to the fire requirements related to height. The Capstone building was designed based on having an approved sprinkler system, although it will not be tested with the sprinkler system in place. Also, the design used six stories and a building height of approximately 18 meters which was above and beyond the code approved number of stories but not the overall building height.

The Capstone structure was modeled to represent a typical wood frame structure in the San Francisco or Los Angeles area of California in the U.S. The location is assumed to be in a high seismic hazard area but not necessarily a near fault region. Because of the high seismic design requirements for the lateral resisting system, a cursory wind lateral analysis was performed and it was determined that the lateral resisting system would be governed by seismic activity and therefore further wind requirements were not considered herein.

According to the IBC Section 1604.5 the multi-family dwelling structure would be assigned an occupancy category according to Table 1604.5 as Category II, with a designated Nature of Occupancy of Buildings and other structures except those listed in Occupancy Categories I, III and IV. Based on an Occupancy Category II, the associated Importance Factor, I, was determined to be 1.0. Building load combinations for dead, live and seismic loads were determined from IBC Section 1605.3, Load combinations using allowable stress design.

4. Vertical Design

The vertical or gravity design was based on Sections 1606 and 1607 of the IBC which describe the dead and live load requirements for a structure with multiple-family dwellings. The dead load of the structure was determined by approximating the material weights used to construct the structure. Table 2 summarizes the building weight assumptions that were made.

Table 1607.1 of the IBC specifies the minimum uniformly distributed live loads used in the gravity analysis of the structure. Based on occupancy of residential use consisting of multiple-family dwellings, the IBC requires a minimum live load at private rooms and corridors serving them to be 1,915 N/m^2, balconies – 4,788 N/m^2, stairs and exits – 4,788 N/m^2 and ordinary flat roofs – 957.6 N/m^2. Live load reductions were not used in the analysis procedure. Due to the Southern California location, there were no snow load requirements for the structure.

Table 2: Approximate Building Dead Loads

Building Assembly	Estimated Dead Load (N/m^2)
Roof/Ceiling (1 Hour Rated)	885.78
Floor (1st - 5th Levels – 1 Hour Rated)	1173.06
Floor (6th Level – 1 Hour Rated)	1268.82
Balcony	1197
Exterior Wall w/ WSP Bracing (1 Hour from Interior Only)	526.68
Interior Wall w/o WSP Bracing(Unrated)	383.04
Interior Wall w/ WSP[1] Bracing(1 Hour Rated Both Sides)	526.68-622.44

[1]WSP – Wood Structural Panel

The above dead and live loads were used to determine the required roof and floor joist sizes and spacing, window and door headers and typical stud size and spacing. These initial member sizes will be confirmed and/or modified after the lateral analysis is performed.

5. Lateral Design

The lateral seismic analysis of the 6 story structure was based on Section 1613 - Earthquake Loads of the IBC. Seismic ground motion was determined based on a structure location on Southern California. The 0.2 and 1 second spectral response accelerations, S_S and S_1, were 1.5g and 0.6g, respectively. The site was assumed to have a soil site class of D, which is the code required class when properties of the soil are not available or they are not know in sufficient detail to determine the site class.

The spectral response accelerations are adjusted by the site coefficients which take into account the soil site classifications and the maximum considered earthquake acceleration is determined. Table 1613.5.3(1) indicates that the site coefficient, F_a is 1.0 and the site coefficient, F_v is 1.5. The maximum considered earthquake spectral response acceleration corresponding to a short period, S_{MS}, was determined to be 1.5g and for the 1 second period, S_{M1}, was 0.9g.

Section 1613.5.4 of the IBC indicates a 5% damped structural response shall be assumed for the design spectral response which results in $^2/_3$ of the maximum considered response. The 5% damped design spectral response acceleration for short period, S_{DS} was 1.0g and for 1 second period, S_{D1} was 0.6g. The seismic design category was determined from Tables 1613.5.6(1) and (2). The design category was determined to be D with building occupancy category II with $S_{DS} \geq 0.50$ and $S_{D1} \geq 0.20$g.

Per Section 12.2 of ASCE 7 – Structural System Selection, the basic lateral and vertical force-resisting system must conform to type listed in Table 12.2.1 – Design Coefficients and Factors for Seismic Force-Resisting Systems. The Capstone Buildings' seismic force-resisting system was designed based on a bearing wall system of light-framed walls sheathed with wood structural panels rated for shear resistance. According to the table, the Response Modification Coefficient, R, was determined to be 6½, the System Overstrength Factor, Ω_0, was 3, the Deflection Amplification Factor, C_d, was 4 and based on a Seismic Design Category of D, the Structural System Limitations and Building Height Limit was 19.8 meters.

Each floor diaphragm was assumed to be rigid without any configuration irregularities. The Redundancy Factor, r, for Seismic Design Category D was determined to be 1.0 based on Section 12.2.3.4.2 part a or b of ASCE 7.

Section 12.6 of ASCE 7 describes the structural analysis procedure that is permitted to be used for design based on Seismic design Category. Per Table 12.6-1 – Permitted Analytical Procedures, Seismic Design Category of D and structural Characteristic description of Regular structures with T<3.5Ts and all structures of light frame construction, the Equivalent Lateral Force Analysis procedure was permitted to be used. Therefore the Seismic Base Shear, V, of the structure was determined by:

$$V = C_s * W \tag{1}$$

Where: C_s = the seismic response coefficient
W = the effective seismic weight of the structure, which includes the total dead load of the structure and other loads per Section 12.7.2 of ASCE 7 (kN)

The value of C_s was based on the design spectral response acceleration for short period, SDS, and the Response Modification factor, R which was based on the lateral resisting system and the Occupancy Importance factor, I. Maximum and minimum values of C_s are limited according to Section 12.8.1.1 of ACSE 7, which are based on SD1 and T, the fundamental period of the structure as determined by Section 12.8.2 of the code. The fundamental period of the structure was determined to be 0.453 sec and C_s was 0.154, which results in a Seismic Base Shear, V = 0.154* W. Substituting the base shear into the Allowable Stress Design (ASD) load combinations (0.7*E) results in an ASD Base Shear of 0.108* W.

ASCE 7 Section 12.8.3 describes how to distribute the seismic base vertically to each story depending on story height and the weight associated with that story. The lateral seismic force applied at any level was determined by:

$$F_x = C_{vx} * V \tag{2}$$

and

$$C_{vx} = \frac{w_x h_x^{\ k}}{\sum_{i=1}^{n} w_i h_i^{\ k}}$$

Where: C_{vx} = vertical distribution factor
V = seismic base shear (kN)
w_i and w_x = the portion of the total effective seismic weight of the structure, W, located at Level i or x
h_i and h_x = the height (m) from the base to Level i or x
k = an exponent related to the structure period = 1, for structures having a period of 0.5 sec or less

Table 3 shows the vertical distribution calculations and the corresponding seismic lateral load imposed on each floor for the 6 story Capstone building. The final lateral load at each floor has been converted to earthquake loads that would be used with the ASD load combinations per IBC Section 1605.3.

Table 3: Vertical Seismic Load Distribution

Level x	h_x [m]	$h_x^{\ k}$ [m]	w_x [kN]	$\Sigma w_{x(total)}$ [kN]	$w_x h_x^{\ k}$ [kN-m]	$C_{vx} = \frac{w_x h_x^{\ k}}{\Sigma w_i h_i^{\ k}}$	$F_{x\ ASD} = C_{vx}V$ [kN]	$\Sigma F_{x(total)}$ [kN]
Roof	19.51	19.51	353.37	353.37	6894.25	0.21	67.61	67.61
6th Floor	15.85	15.85	566.30	919.67	8975.86	0.28	88.03	155.64
5th Floor	12.80	12.80	501.18	1420.85	6415.10	0.20	62.94	218.58
4th Floor	9.75	9.75	501.18	1922.03	4886.51	0.15	47.95	266.53
3rd Floor	6.71	6.71	501.18	2423.21	3362.92	0.10	32.96	299.49
2nd Floor	3.66	3.66	501.18	2924.39	1834.32	0.06	19.13	318.62
Σ =			2924.39		32368.96	1.00	318.62	

Each horizontal load distribution was applied at the center of mass of the corresponding story and each shear wall supporting that level was analyzed. A series of segmented shear walls were used in the short and long orthogonal directions along with perforated shear walls along the exterior walls of the short direction of the building. A 5% accidental torsional eccentricity was also analyzed included according to ASCE 7 Section 12.8.4.2.

The final lateral design of 1st story is shown in Figures 3 and the corresponding wood shear wall schedule is shown in Table 4.

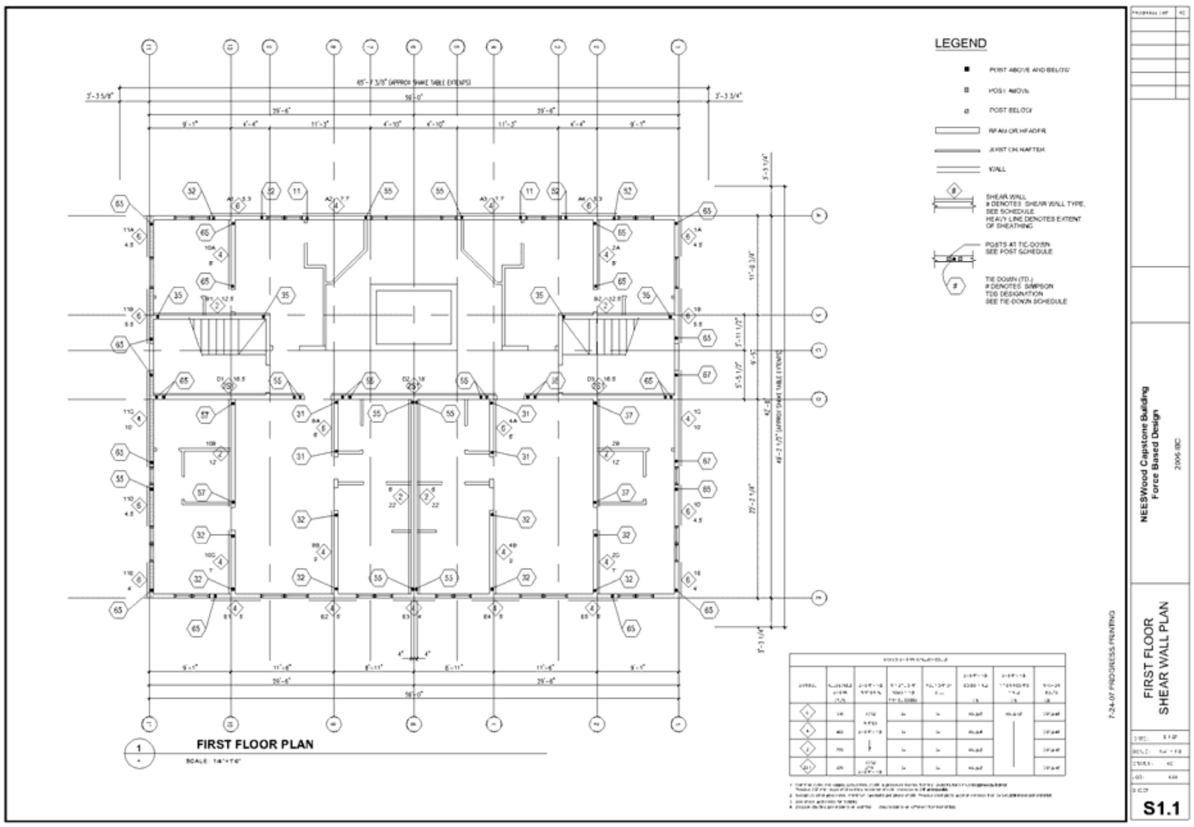


Figure 3: 1st Floor Shear wall Plan

Table 4: Wood Shear Wall Schedule

WOOD SHEAR WALL SCHEDULE							
SYMBOL	ALLOWABLE SHEAR (kN/m)	SHEATHING MATERIAL	MIN STUD AT ADJOINING PANEL EDGES	FOUNDATI ON SILL	SHEATHING EDGE NAILS (1)	SHEATHING INTERMEDIA TE NAILS (1)	ANCHOR BOLTS (2)
6	4.5	15/32 RATED SHEATHING	2x	3x	10d@152.4 mm	10d@304.8 mm	15.875 mm @1219.2 mm
4	6.7		3x	3x	10d@101.6 mm		15.875 mm @1219.2 mm
2	11.2		3x	3x	10d@50.8 mm		15.875 mm @384 mm
2S1	12.7	15/32 STR I SHEATHING	3x	3x	10d@50.8 mm		15.875 mm @384 mm

1. Common nails, hot-dipped galvanized at sill, & pressure treated framing. Substitutions must be approved by Engineer.
2. Except otherwise noted, minimum two bolts per piece of sill. Provide steel plate washer no less than 7.6x7.6x0.58 cm on each anchor bolt.
3. See shear wall notes for fastening.
4. Stagger abutting joints on each face (locate joints on different framing member)

As an example, tables 5 and 6 shows the number of Doug-Fir Larch compression studs and the diameter size of the holddown rod required at each end of the shear wall for 1st level. The Tie-Down number corresponds to the Tie-Downs indicated in Figures 3 of the shear wall plans.

Table 5: Compression Studs (DF-L) for Tie-Down Runs

Tie-Down #	11	31	32	35	52	55	57	65	67
1st Floor	4-2X6	2-2X6	4-2X6	6-2X6	4-2X6	9-2X6	8-2X6	8-2X6	6-2X6

Table 6: Tie-Down Rod Gross Diameter for Tie-Down Runs (mm)

Tie-Down #	11	31	32	35	52	55	57	65	67
1st Floor	12.7	15.875	0.750	1.250	0.750	1.125	0.875	1.125	0.875

6. Performance Assessment Using Time History Analysis

The SAPWood V1.0 software package decomposes each story into two translational and one rotational degrees-of-freedom as shown on an arbitrary building in Figure 4. Each shearwall is represented by a nonlinear hysteretic spring model and these are combined based on the geometry assuming an infinitely rigid floor diaphragm. In SAPWood, the spring can range from a simple linear elastic spring to an evolutionary parameter hysteretic model (Pang, W.C. et al., 2004) (EPHM). Any mix of spring types can be used within a single model. This provides flexibility when modeling, for example, a building whose gravity system is steel columns but whose lateral force resisting system is a system of wood shearwalls. The shearwalls would best be modeled with a hysteretic model than pinches and degrades in both stiffness and strength whereas the steel columns could be modeled reasonably well with a bilinear hysteretic oscillator.

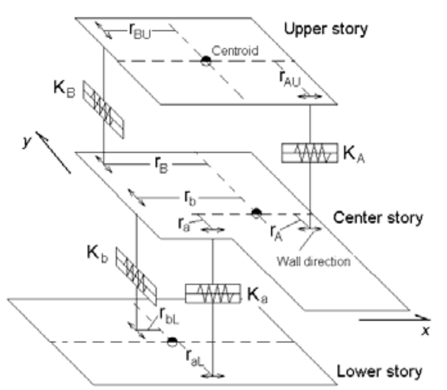


Figure 4: 3 Three degrees of freedom idealized building (Pei and van de Lindt, 2008)

Modeling a wood frame building in SAPWood can be done two ways. Test data for each wall can be used to fit the hysteretic springs and assembled into a full system model. While this is an accurate option it is unreasonable since test data for few, if any, of the walls in any particular building are available. Another option which is widely used in wood frame seismic research is to fit the sheathing connector hysteresis and then force a numerical wall assembly through a reversed-cyclic displacement protocol while computing the resisting spring force. Figure 5 shows the procedure use to go from a nail hysteresis to a six-story 1,400 m^2 building system model. It is important in all modeling but particularly with nonlinear models, to have a benchmark for comparison. This could be, for example test results from 2.43m x 2.44m (8 x8 ft) wood shearwalls with the same (or similar) fasteners and spacing. Although not exact, the estimate for a similar wall without openings should be proportional to length.

Determine Hysteresis for 10d common nail connecting 12mm OSB 5.08cm x 15.24cm → Develop fastener-level model for each unique shearwall in the building → Assemble the SAPwood building system model using the hysteretic springs from step 2 and building weights → Perform multi-record IDA to assess performance numerically

Figure 5: The procedure from nail to building system model

7. Wall Modeling

In order to analyze the building model, a nonlinear spring, a hysteresis, is developed using the SAPWood Nail Pattern (NP) model.

Figure 6 shows the final result of a target shearwall and the wall type parameters for an evolutionary parameter hysteretic model (EPHM) oscillator. In Figure 7, a screen capture of the program is shown for a simple 1.22 m wide wood shearwall within the building system model.

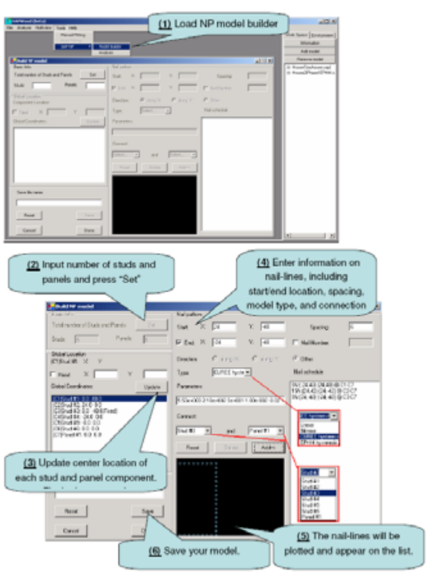


Figure 6: Dialog box for NP model Builder

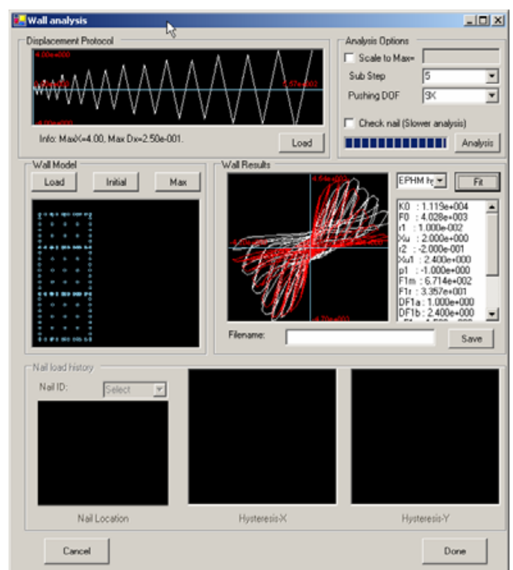


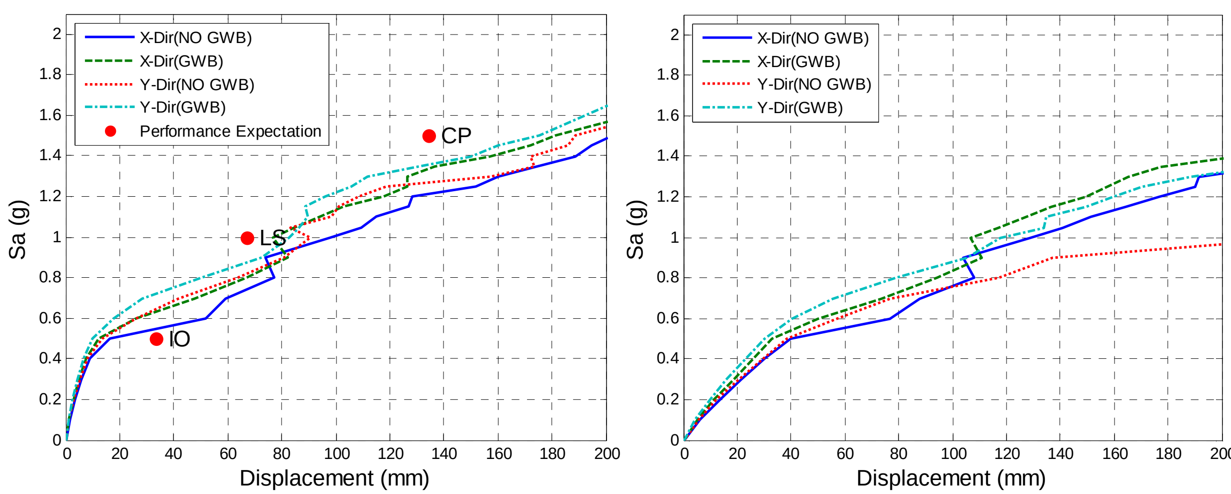
Figure 7: Results of wall analysis

8. Incremental Dynamic Analysis

The incremental dynamic analysis (IDA) approach is used as an investigative tool in order to determine the performance of the IBC-designed six-story building under increasing levels of seismic intensity. Incremental Dynamic Analysis (IDA) is a parametric analysis method that has recently emerged in several different forms to estimate seismic demand and capacity of structures under seismic loads. A single IDA generates one curve of structural response versus seismic intensity levels.

In this study, analysis using multi-record IDA (MIDA) for the IBC-designed six-story wood frame building was conducted. Figure 8 shows the responses for the structure with OSB and gypsum wall board (GWB). The MIDA analysis was conducted for an existing suite of 20 earthquakes and Figure 8 was plot using the average of the peak 20 earthquake responses. The performance levels currently being considered within the NEESWood project (see http://www.engr.colostate.edu/NEESWood/ for details) for performance-based seismic design (PBSD) are shown as bold red dots in the figures.

Figure 8 indicates the "immediate occupancy" (IO), "life safety" (LS), and "collapse prevention"(CP) performance expectations shown by the red dot with the mean IDA curves from the M-IDA. One can see that the IBC-designed building satisfies the IO performance requirements but slightly under performs at the LS and CP levels. Interestingly, the GWB make only a slight difference in the mean value for the IDA but does help to control responses at higher seismic intensities. At the CP level the response was approximately 8% inter-story drift at the first story when GWB was not included. With GWB included the maximum inters-story drift at story 1 was approximately 7%. The drift a roof level without drywall became excessive, i.e. almost the mean response a horizontal curve, as it approaches a 1g spectral acceleration. With GWB included in the model the mean response was able to be controlled, but not all the way to the CP seismic intensity level of 1.5g spectral acceleration



(a) 1st story (b) Total drift at Roof level

Figure 8: The plot of disp. versus Sa using mean value of 20 EQs response

9. Conclusion

The six-story wood frame building was designed using the 2006 IBC methodology. Incremental Dynamic Analysis was used to numerically investigate the performance of the building based on the inter-story drift at each level and the peak drift at roof level. From the results of the numerical analysis, the building was found to perform very well even without the consideration of gypsum wall board (GWB) at low to moderate levels of excitation until approaching seismic intensity levels consistent with the design-basis earthquake. When GWB was included in the analysis the performance of the structure was very good until a spectral acceleration of 1.2g was reached. The building drifts became excessive at CP level seismic intensities. Through the results of the M-IDA, there appeared to be no dominant earthquake for the entire building. It can be concluded that a six-story force building designed using the IBC methodology performs adequately, based on mean value peak responses to 20 earthquakes, up to the design basis level earthquake but does not perform adequately at the maximum credible level earthquake, i.e. corresponding to CP in this paper. Thus, an alternative design approach more consistent with performance-based design methods being developed is recommended to control these excessive drifts and allow the building to perform better at high seismic intensity levels.

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