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INCREMENTAL DYNAMIC ANALYSIS OF A FIVE–STORY LAMINATED GUADUA BAMBOO FRAMED BUILDING

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Abstract

The increasing interest in finding renewable materials for construction purposes has led to consider bamboo as an important alternative to wood and wood–based products, since it has a fast growing rate and high strength–to–weight ratio, besides outstanding environmental advantages. Furthermore, laminated bamboo products have emerged as a solution to the inherent difficulties of constructing with round pole bamboo. This study presents a preliminary evaluation of the seismic performance of a five–story laminated bamboo frame building, by means of an Incremental Dynamic Analysis (IDA). The building archetype is based on typical construction practice in Colombia for low– and mid–rise residential buildings, while the structural system is based on a light–framed shear wall system similar to conventional wood construction practice. Vertical and horizontal diaphragms are comprised of laminated sheathing panels made from *Guadua angustifolia* Kunth bamboo, which is widely available in Colombia, over wood framing elements. Using a simplified nonlinear model of the general cyclic response of shear walls (SAWS), the structure was subjected to various ground motions representative of local seismic demand conditions, and performance parameters were estimated for assessing its behaviour. Results indicate that the five–story laminated *Guadua* bamboo framed building has an adequate performance under seismic demands, and could be considered as a potential alternative material for the construction of low– and mid–rise buildings in Colombia.

Keywords: Laminated Guadua bamboo, Incremental Dynamic Analysis, light–frame system, bamboo shear walls.

1. Introduction

Bamboo is a highly renewable resource, with outstanding environmental advantages and high strength–to–weight ratio, and has been used extensively throughout America and Asia for the construction of diverse types of structures. However, due to its natural round pole shape and the inherent irregularities along its length, it poses difficulties for construction using straight elements, as well as standardizing connections. Therefore, laminated bamboo products have been developed as a solution for these geometric difficulties, and previous studies have shown that mechanical properties may be comparable to those of structural wood and wood–based products [1].

Guadua angustifolia Kunth is a giant species of bamboo that grows in many South and Central American countries. Research around laminated *G. angustifolia* Kunth products has indicated that its physical and mechanical properties are as good as wood species used for construction purposes. Recently, the Research Center on Materials and Civil Infrastructure (CIMOC) from Universidad de los Andes in Bogotá, Colombia, conducted a study for the validation of the structural behaviour of a system built with laminated *G. angustifolia* elements [2, 3]. This study intended to give insight and compliment the technical knowledge around *G. angustifolia* construction, since it is a widely available natural resource in Colombia that may be used as a structural material. Specifically, *G. angustifolia* may be used for the construction of low–income housing in both urban and rural areas, therefore contributing to decreasing the existing housing deficit of the country.

As part of this research project, the use of laminated *Guadua* panels as structural sheathing in shear walls was studied, indicating that these may be a suitable replacement for wood based panels traditionally used in light–frame systems. However, the fabrication process of these panels was expensive when compared to traditional wood based panels, due to its reduced production scale and limited industrialization level; hence it was not entirely competitive. As a result, a new type of laminated Guadua panel was developed, comprised of pressed–and–glued–together *Guadua* mats in a similar fashion as plywood, thus implying fewer stages in its manufacturing process, and therefore reducing its cost [4]. This new Laminated *Guadua* Mats (LGM) may be used as structural sheathing in shear walls or diaphragms, as wood based panels have been successfully used in light–frame construction in the United States, Canada and Europe.

As main components of the lateral force resisting system in light–frame structures, shear walls are to maintain an adequate performance under lateral forces as induced by earthquakes. Sheathing–to–framing connections are commonly assumed to govern the cyclic response of shear walls, since these are responsible for most of the energy dissipation. Therefore, numerical models have been developed to estimate the load–displacement response of shear walls based on the general cyclic behaviour of sheathing–to–framing connections. In order to study the potential use of LGM panels as structural sheathing in light–frame systems, this study presents a preliminary numerical evaluation of the dynamic behaviour of a shear wall structure based on the cyclic behaviour model of the sheathing–to–framing connection. The numerical model was implemented using the software CASHEW v. 1.0 [5] and SAWS v. 1.0 [6], developed during the *CUREE–Caltech Woodframe Project*. Connection models were calibrated from experimental tests conducted on LGM sheathing over wood framing elements, following the results obtained from previous work using LGM framing [7]. The adequate calibration of such models is intended to be used as framework for multiple purposes in advanced stages of the research project, such as technical viability of multi–story light–frame structures using LGM panels, determining characteristic seismic response coefficient (R) values for this structural system, and providing accurate design procedures via simplified numerical models for this particular constructive system.

Experimental tests conducted on light–frame shear walls with LGM sheathing panels, under both monotonic and cyclic loading conditions, were carried out as part of the larger research project [8], although are out of the scope of this paper. These shear wall tests allowed the evaluation of the lateral behaviour of the proposed system, as well as the calibration and validation of numerical models developed from sheathing–to–framing connections for predicting the global response of individual walls. Furthermore, the experimental phase of the project concluded that the maximum shear strength, ductility capacity and energy dissipation increase with the decrease of edge nail spacing, although limited to spacing values over 75 mm; for lower spacing a different failure mode governed by shear in the panel and limited nail yielding is achieved, and therefore adequate shear and ductility capacity are not reached. Therefore, these limitations were considered for the numerical study presented herein.

2. Laminated Guadua bamboo framed system

2.1. System overview

Light–frame systems are commonly used in North America and Europe for the construction of residential and commercial buildings. The system is usually comprised by a wood frame structure sheathed with plywood or OSB panels. The lateral load resisting system consists of a combination of vertical and horizontal diaphragms (shear walls and floor diaphragms, respectively). Horizontal diaphragms are comprised by supporting beams and sheathing panels connected by nails or screws, and are intended to receive lateral loads from each story and transfer these loads to shear walls. The load transfer mechanism starts from the top story or roof diaphragm connected by shear bolts to the top of the shear walls that are aligned with the direction of loading. These walls transfer the load to the walls in the story immediately below, by the same system of shear bolts and hold–down anchors, and which also receive the lateral load transferred from the diaphragm at this level. This load transfer mechanism repeats until reaching the foundation level.

Horizontal and vertical diaphragms work essentially as an I–shaped beam, where the shear force due to lateral load is resisted by the web (sheathing panels), whereas bending forces are resisted as tension and compression forces acting on the flanges (chords in floor diaphragms, and end studs in shear walls).

Connections between the framing elements are generally assumed to be pin–connected, since typical detailing consists on nails that provide almost negligible lateral stiffness of the framing. Therefore, wall stiffness is provided mainly by the high rigidity of the sheathing panel. Fig. 1 presents a scheme of the typical components of a floor and wall diaphragm. Particularly in the case of shear walls, framing elements are called studs (vertical elements) and plates (horizontal elements). The sheathing panel is connected to the framing elements by dowel fasteners along its edges, typically nails, and usually spaced by 50 to 150 mm. This sheathing–to–framing connections are considered the main component of the lateral loading behaviour of the wall. Nailing in the field of the panel is considered only for constructive purposes. Hold–down anchors are the elements intended to provide the tensile (uplift) strength in the bottom of the wall due to the action of lateral loading in the top of the wall, thus resisting the overturning moment. These anchors are usually prefabricated metallic plates connected to the end studs, with bolts or steel rods that are connected to the wall immediately below or fixed to the foundation element. Walls with this type of hold–downs are referred as fully anchored. Finally, shear bolts in the bottom (sill) plate, provide the capacity to withstand horizontal sliding and transfer of shear forces.

In this study, typical plywood and OSB panels are replaced by Laminated *Guadua* Mat (LGM) panels. These panels are pressed–and–glued–together *Guadua* mats in a similar fashion as plywood. The fabrication process of these panels is described in detail in [7]. Wood elements (Chilean Radiata pine) were used as framing, as opposed to the LGM framing elements presented in previous work [7], since technical and economical evaluations of this system conducted as part of the larger research project yielded better results for wood with LGM sheathing [8].

Several reasons support the selection of LGM panels as an alternative to wood based products traditionally used in other countries. First, laminated *G. angustifolia* products for construction uses may boost a new industrial sector in Colombia around the exploitation and manufacturing of products developed from this available, renewable resource, which is currently almost limited to minor craftsmanship. Second, previous studies have shown that laminated *G. angustifolia* products have adequate physical and mechanical properties for structural use, and may be comparable to wood–based products typically used for construction. But most importantly, accelerated times of construction as achieved for light–frame systems, and the relative simplicity of its construction, are major advantages for considering a wide implementation in low–income areas, thus contributing to decreasing the existing housing deficit of the country.

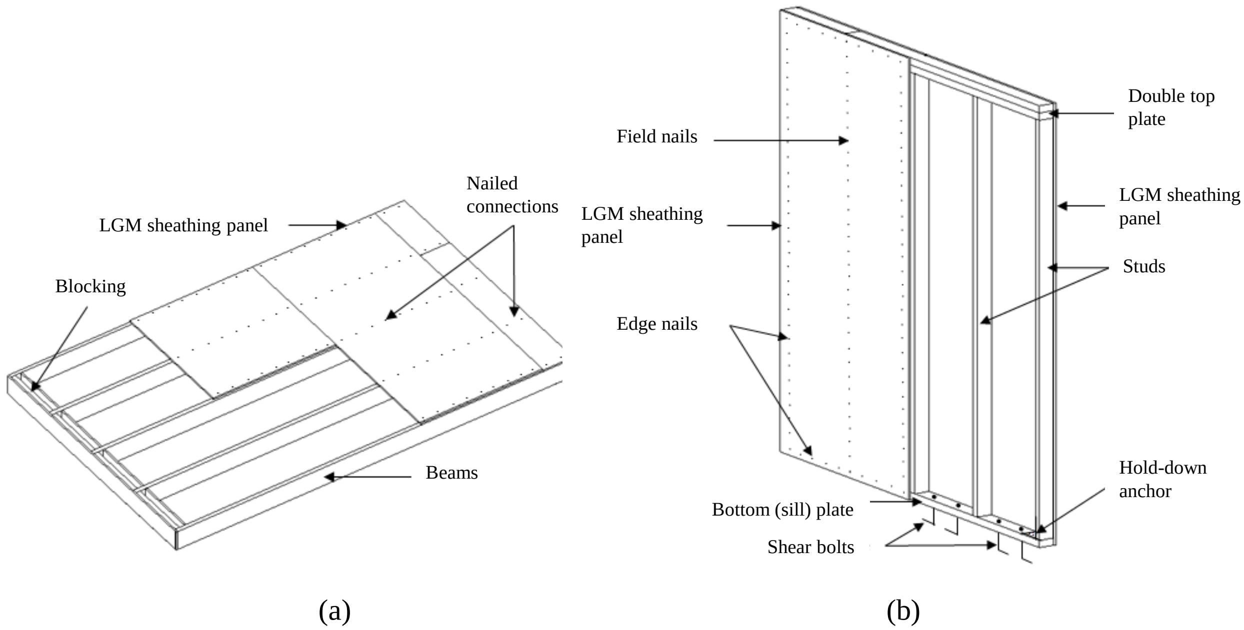


Fig. 1 – Main components of light–frame systems: (a) floor diaphragms; (b) shear walls

2.2. Case study structure

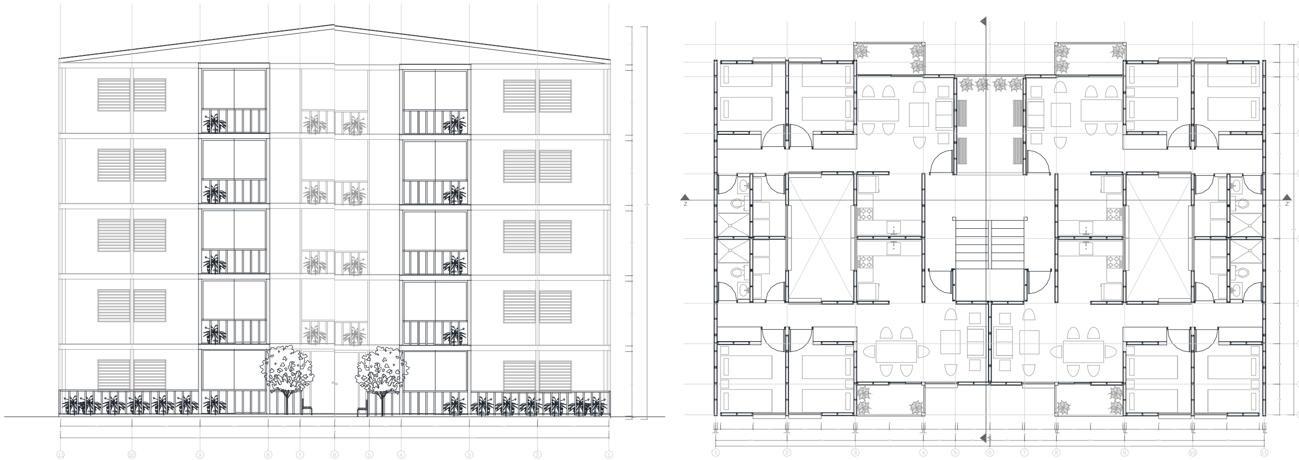
The structure for analysis in this study is a five–story residential building (Fig. 2), designed as part of the research project entitled "Structural Behaviour of Modular Housing with Laminated Guadua Panels" conducted by the Research Center on Materials and Civil Infrastructure (CIMOC) from Universidad de los Andes, at Bogotá, Colombia. The building was conceived as a typical low–cost housing solution, based on available information from similar projects, and the architectural design adopted parameters of modular and sustainable construction. The building has a floor plan area of approximately 240 m^2, and plan dimensions of 20.38 m by 13.08 m. Each story has four 50 m^2 apartments, and a story height of 2.60 m.

As part of the research project, all elements of the vertical and lateral load resisting systems were designed following the provisions of the Colombian Building Code NSR–10 [9], considering the building was located at a major Colombian city in a high seismic hazard zone (Cali). Although current provisions in the NSR–10 do not allow the design of light–frame structures over two stories (6 m) high, the evaluation of this building was considered a fundamental starting point for the research project, in order to study the expected performance of larger buildings using this system. Therefore, the SDPWS–2008 [10] was followed as reference, for complementing the provisions in the Colombian Building Code.

Cyclic behaviour model

3.1. Sheathing–to–framing connections

Sheathing–to–framing connections in light–frame shear walls exhibit a nonlinear behaviour under monotonic loading, and additionally have a hysteretic response with pinched cycles and stiffness degradation. A load–displacement response model was proposed by Folz & Filiatrault [11], which is based on a series of loading and unloading paths that reproduce the connection response, as shown in Fig. 3. Monotonic response is defined by six physically identifiable parameters: initial tangential stiffness (K_0), loading and unloading stiffness multiplication coefficients (r_1 and r_2), asymptotic load–intercept (F_0), displacement at maximum load (δ_u), and final displacement (δ_f) at 80% of the maximum load. These parameters are obtained from experimental data fitting procedures from individual connection tests. The response model captures the crushing under the nail shank in both the framing element and the sheathing panel, as well as the yielding of the nail due to bending.

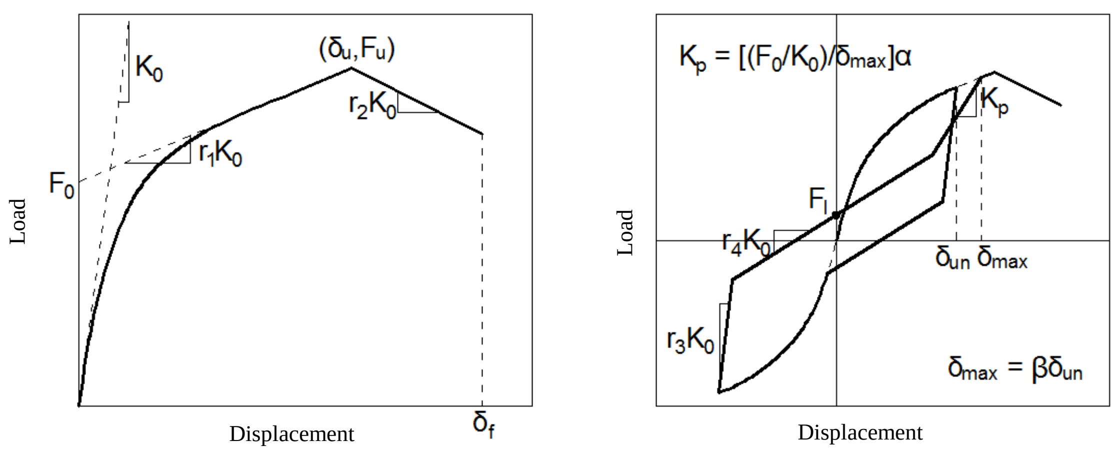


(a) (b)

Fig. 2 – Case study structure: (a) elevation; (b) plan view

The general cyclic response may be characterized by the same monotonic curve as the cyclic backbone curve, besides five additional parameters (F_I, r_3, r_4, α, β), that describe the unloading and reloading behaviour, stiffness degradation and pinching, based on linear load–displacement relations. These additional parameters are also fitted from experimental data, from individual cyclic connection tests. A scheme of the general cyclic behaviour model is presented in Fig. 3 (b).

To estimate the parameter values for the LGM sheathing–to–framing connection used in this study, individual connection tests were conducted as part of the larger research project, using 10d common nails as fasteners. The parameters adjusted from experimental data for the sheathing–to–framing connection model are presented in Table 1, while Fig. 4 presents the comparison between connection tests and the fitted model.



(a) (b)

Fig. 3 – General load–displacement behaviour model: (a) monotonic model; (b) cyclic model

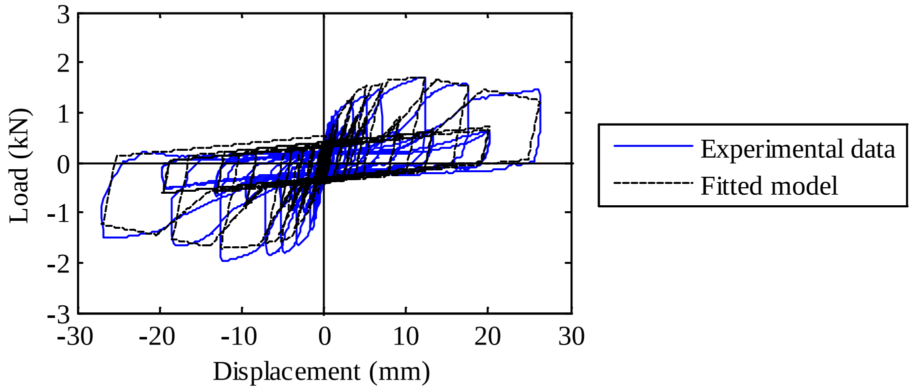


Fig. 4 – Comparison of experimental results and fitted model of sheathing–to–framing connection

3.2. Shear walls

In order to estimate the capacity and general behaviour of LGM shear walls under cyclic loading, the software CASHEW v. 1.0 [5] was used, which allows the numerical prediction of lateral loading behaviour of a fully anchored light–frame shear wall. The model is based solely in the model parameters of the sheathing–to–framing connection, according to the spatial distribution of fasteners in the wall. Equilibrium equations that describe the lateral loading response of the wall are obtained by applying the virtual work principle, and solving by means of an incremental iterative procedure over the displacement. The software, however, is limited to the following: (1) framing elements are pin–connected, and do not contribute to the lateral stiffness; (2) top and bottom plates remain horizontal and parallel at all times (pure shear behaviour); (3) sheathing panel remains elastic, and only contributes to the shear stiffness, but it has no effect on the loading or displacement capacities; (4) the cyclic response of the wall follows the same loading and unloading paths as the sheathing–to–framing connection model.

For this study, shear wall models were developed for 1.20 m by 2.40 m wall units, with sheathing panels on one side. Two distinct edge nail spacing values (150 mm and 75 mm) were considered, since with these two types of spacing, the shear force demands over walls were satisfied during the structural design process. For modeling sheathing panels on both sides of the wall unit, the parameters of the model would be doubled in terms of loading capacity, but not displacement. The same may be extended to multiple wall units working on a same loading line, and therefore, multiple wall configurations may be considered with only these two behaviour models. Fig. 5 presents the results of the cyclic behaviour of the two types of wall models considered. Furthermore, CASHEW allows the estimation of the generalized model parameters for the wall unit, in order to obtain a simplified SDOF element model that represents the wall behaviour. The set of adjusted parameters for each wall type are presented in Table 1. According to the behaviour model observed, it may be noticed that a decrease in nail spacing results in a considerable increment of loading capacity and stiffness, but no difference is observed in terms of displacement capacity. The latter is consistent with typical building code provisions.

Table 1 – Generalized behaviour model parameters

Model	K_0 (kN/mm)	r_1 –	r_2 –	r_3 –	r_4 –	F_0 (kN)	F_I (kN)	δ_u (mm)	α –	β –	F_U (kN)
10d nail connection	0.98	0.051	–0.024	0.92	0.050	1.27	0.37	16.99	0.80	1.10	2.12
Shear wall with edge nail spacing 150 mm	2.65	0.071	–0.048	1.66	0.075	11.50	2.88	35.84	0.78	1.40	17.56
Shear wall with edge nail spacing 75 mm	4.75	0.077	–0.061	1.44	0.075	22.31	5.78	38.01	0.80	1.39	36.36

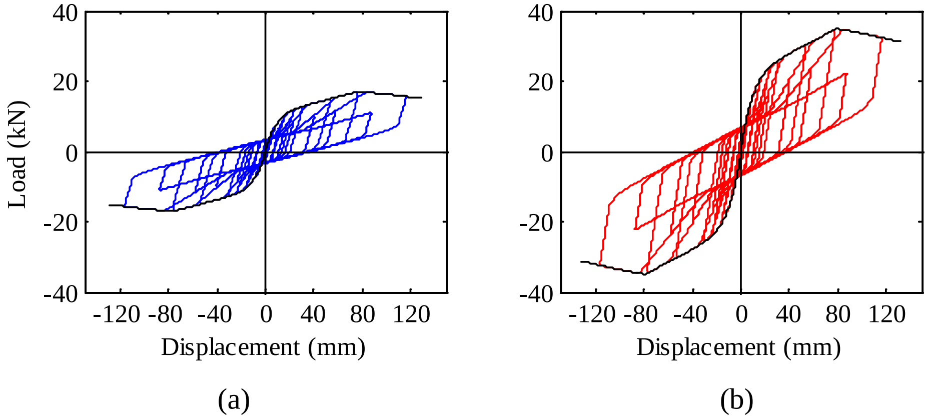


Fig. 5 – Cyclic response of shear wall models: (a) edge nail spacing 150 mm; (b) edge nail spacing 75 mm

4. Analysis model

The general seismic response of the case study structure was estimated using SAWS v. 1.0 [6]. The software analyses a simplified 2D model of the actual 3D structure, by comprising a stiffness matrix with multiple cero–height SDOF elements that represent each line of walls in each story, and joining the degrees of freedom from all stories. All floor diaphragms are assumed as rigid, in order to reduce the number of degrees of freedom to three per story: two translational and one rotational DOF. Each of the SDOF elements is defined by the general behaviour model parameters of shear walls. As stated previously, multiple walls on a single line were modeled as one SDOF element with the lateral loading capacity of the sum of walls, but maintaining the displacement capacity.

4.1. Model geometry definition

The SAWS model is limited for regular plans, and therefore a simplified plan of the building was considered (Fig. 6). Nevertheless, all structural walls were considered, and only relocated in this simplified plan. The numbering of resisting wall lines, number of walls per line, and nailing schedule for the five different levels, are presented in Table 2.

Table 2 – Distribution and nailing schedule for the case study model

Line	No. of walls	Nailing schedule (edge nail spacing, mm)				
		Story 1	Story 2	Story 3	Story 4	Story 5
A	8	75*	75	75	75	150
C	4	75*	75*	75*	75	150
D	6	75*	75	75	75	150
E	8	75*	75	75	75	150
F	4	75*	75*	75	75	150
G	4	75*	75*	75*	75*	150
I	6	75*	75	75	75	150
1, 11	8	75*	75*	75*	75	150
2, 10	4	75*	75*	75*	75	150
3, 9	6	75*	75*	75*	75	150
4, 8	4	75*	75*	75*	75	150
5, 7	3	75*	75*	75*	75	150
6	2	75*	75*	75*	75	150

* Indicates sheathing panels on both sides of the wall

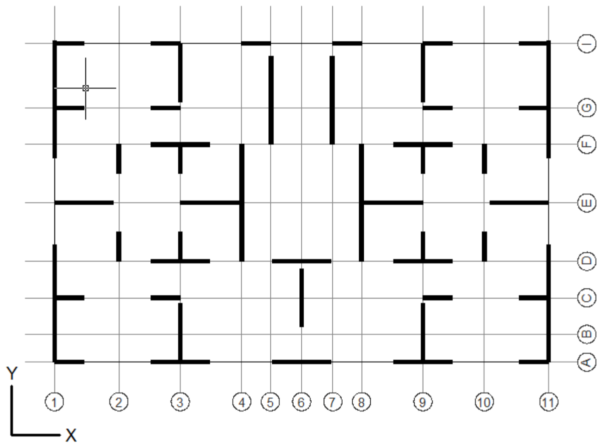


Fig. 6 – Simplified plan model and structural walls considered

Story heights were modeled as 2.60 m, except for the fifth story that was modelled at 3.10 m over the fourth level, considering a third of the height of the inclined roof structure. Dead weight over each story was estimated as 1.20 kN/m^2, while over the fifth story was 0.66 kN/m^2. As input, SAWS requires the definition of the damping coefficient for the first two vibration modes, and estimates internally the remaining damping coefficients for all modes. Folz & Filiatrault [6] recommend a value of 1%, since the hysteretic behaviour model of shear walls includes most of the system damping.

Dynamic properties (vibration frequencies, modal mass participation, and modal shapes), were estimated using SAWS, and are presented in Table 3. As observed from the numerical results, the first four modes (two in each plan direction), represent over 90% of the excited mass. Additionally, Fig. 7 shows a scheme of the modal shapes for these four modes. As a particular interest of the dynamic properties evaluation, it may be observed that the fundamental period estimated by means of the model is significantly lower than the approximate fundamental period based on building code provisions [9]. When applying the conventional $T_a = C_t h_n^x$ equation, the resulting period for these type of structures is 0.345 s, whereas the model yields a value of 0.189 s. This represents a distinct condition in terms of expected spectral accelerations, for example. These differences are due mainly to the assumption of a similar stiffness for all wall–based systems for the approximate fundamental period equation in building codes, since this particular LGM light–frame system has shown a major increase in lateral stiffness due to the panel thickness.

Table 3 – Dynamic properties of the case study structure model

Mode	Frequency	Period	Damping	X direction		Y direction	
	(rad/s)	(s)	(%)	Mass part.	Cum. mass	Mass part.	Cum. mass
1	5.282	0.189	1.00	88.4%	88.4%	0.0%	0.0%
2	7.195	0.139	1.00	0.0%	88.4%	88.6%	88.6%
3	7.521	0.133	1.01	0.2%	88.6%	0.0%	88.6%
4	13.89	0.072	1.56	8.5%	97.1%	0.0%	88.6%
5	17.19	0.058	1.69	0.0%	97.1%	8.5%	97.2%
6	17.79	0.056	1.86	0.0%	97.2%	0.0%	97.2%
7	19.81	0.050	2.26	2.2%	99.4%	0.0%	97.2%
8	25.65	0.039	2.48	0.0%	99.4%	2.2%	99.4%
9	26.24	0.038	2.76	0.0%	99.4%	0.0%	99.4%

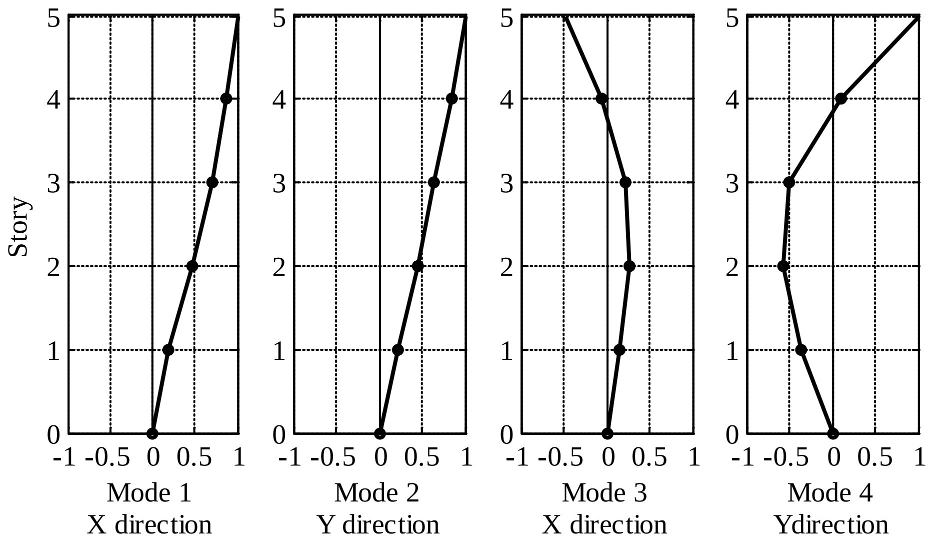


Fig. 7 – Modal shapes for the first two modes in each plan direction

4.2. Incremental Dynamic Analysis

In order to evaluate the nonlinear dynamic behaviour of the system, an Incremental Dynamic Analysis (IDA) was conducted over the case study model. IDA consists on submitting the model structure to multiple seismic records that represent increasing demand levels, and determining performance or behaviour parameters for each demand level [12]. It is recommended that this procedure should be conducted over a wide range of acceleration records from different seismic events, scaled at various demand levels. However, only 5 different records were considered in this study, and scaled at spectral acceleration levels ranging from 0.2 g to 4.5 g, as a preliminary approach to the nonlinear behaviour of the system. Table 3 summarizes the record set used for the analysis, and their corresponding response spectra are shown in Fig. 8.

For each of the time history analyses, the behaviour parameter was defined as the maximum interstory drift, and the IDA curve was defined as this drift versus the spectral acceleration for the fundamental frequency of the structure and 5% damping ratio. This analysis was conducted for the two main plan directions, considering the difference in terms of stiffness due to the number of walls in each direction. Figure 9 presents IDA curves for the two plan directions.

Table 3 – Acceleration records selected for IDA

ID	Event name	Year	Station	NPTS	DT (s)	PGA (g)	S_a (T_1, 5%) (g)
1	Northridge	1994	Rinaldi Receiving Station 228	1991	0.01	0.87	1.50
2	Northridge	1994	Rinaldi Receiving Station 318	1991	0.01	0.47	1.12
3	Loma Prieta	1989	Corralitos – Eureka Canyon NS	2015	0.02	0.63	1.05
4	Imperial Valley	1940	El Centro Array #9 NS	2655	0.02	0.35	0.65
5	Kobe	1995	Tadoka 000	14000	0.01	0.25	1.26

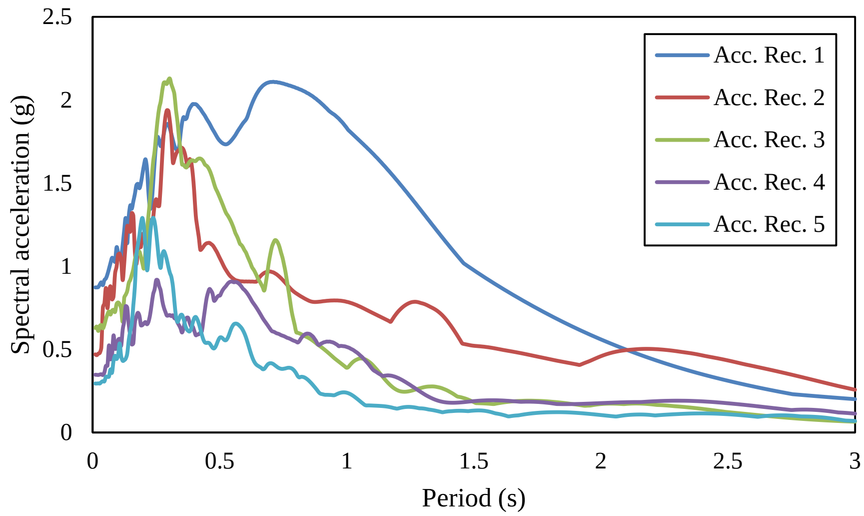


Fig. 8 – Response spectra of original seismic records selected for IDA

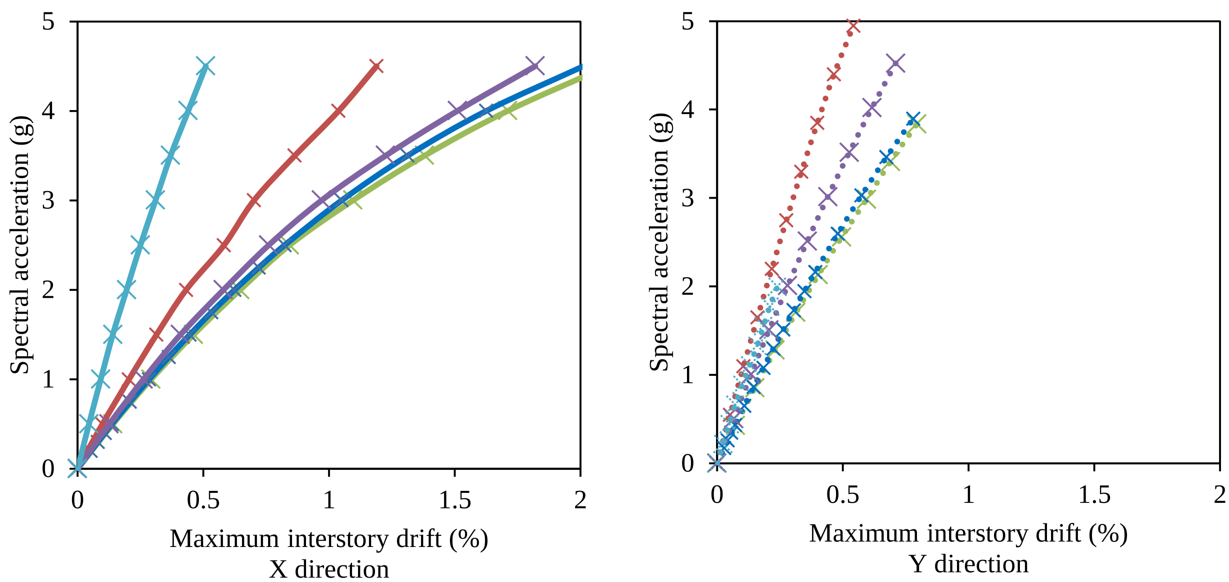


Fig. 9 – IDA curves of maximum interstory drift ration versus spectral acceleration for fundamental frequency

As observed from the IDA curves in Fig. 9, maximum interstory drift increases with respect to the spectral acceleration. However, only for some selected acceleration records, a slight tendency to an asymptotic spectral acceleration value may be observed. This asymptotic value would represent the collapse limit according to this analysis, where the interstory drift increases significantly for a negligible increase in spectral acceleration. Further drift–acceleration pairs could not be determined for the case study structure, due to the relatively low post–peak deformation capacity conserved by the generalized model. Numerical convergence problems arise when certain wall lines exceed their deformation capacity, since the behaviour model is not defined for displacements past the 80% of the maximum load. Additionally, differences between the resulting performance parameters for each plan direction are consistent with the increased stiffness in the Y direction.

Nevertheless, the IDA procedure reveals the expected nonlinear behaviour below accepted performance limits such as the 2% transient story drift, considered for the Life Safety (LS) level [13]. For this drift level, all the analyses performed indicate that the case study structure would maintain adequate capacity. According to this limit state criterion, the dynamic behaviour of the structure would be considered satisfactory.

Further analyses are to be conducted on the given case study structure, considering a wider range of seismic records, with different types of frequency content, that may impose particular dynamic conditions to the structure. More important, however, shall be the analysis of other types of structures, considering different story and building heights, number of walls, and especially, the development of model configurations for irregular plans. These further analyses are intended to provide a framework for multiple purposes in advanced stages of the research project, such as technical viability of multi–story light–frame structures using LGM panels, determining characteristic seismic response coefficient (R) values for this structural system, and providing accurate design procedures via simplified numerical models for this particular constructive system.

5. Concluding remarks

Base on the numerical analyses conducted, the following conclusions may be drawn:

1. The cyclic models developed from experimental tests on sheathing–to–framing connections, allow the preliminary evaluation of more complex structural systems by means of a simplified model. However, the larger research project considers the validation of these numerical models with full–scale experimental tests on shear walls.
2. High stiffness of the LGM shear wall models results in considerably lower fundamental periods than the typical approximate fundamental period in most building codes. This may represent considerable differences in terms of expected spectral accelerations, for example, which directly affect the robustness of the structural design, and further its construction costs.
3. Considering the Life Safety (LS) performance level, the behaviour of the five–story building comprised of LGM shear walls is satisfactory, since the structure would maintain adequate capacity at this level.
4. Further analyses are to be conducted on the given case study structure, considering a wider range of seismic records, with different types of frequency content, that may impose particular dynamic conditions to the structure. More important, however, shall be the analysis of other types of structures, considering different story and building heights, number of walls, and especially, the development of model configurations for irregular plans.

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5. References

[1] Mahdavi M, Clouston PL, Arwade SR (2011): Development of laminated bamboo lumber: review of processing, performance and economical considerations. *Journal of Materials in Civil Engineering*, **23** (7), 1036–1042.

[2] Varela S, Correal JF, Yamín LE, Ramírez F (2013): Cyclic performance of Glued Laminated Guadua bamboo–sheathed shear walls. *Journal of Structural Engineering*, **139** (11), 2028–2037.

[3] Correal JF, Echeverry JS, Ramírez F, Yamín LY (2014): Experimental evaluation of physical and mechanical properties of Glued Laminated *Guadua angustifolia* Kunth. *Construction and Building Materials*, **73**, 105–112.

[4] Correal JF, Echeverry JS (2015): Evaluation of selected mechanical properties of new Laminated Guadua Mats for structural use. *10th World Bamboo Congress WBC 2015*, Damyang, Korea.

[5] Folz B, Filiatrault A (2000): CASHEW – Version 1.0: A computer program for cyclic analysis of wood shear walls. *Report No. SSRP – 2000/10*, *CUREE–Caltech Woodframe Project*, Division of Structural Engineering, University of California, San Diego, La Jolla, CA.

[6] Folz B, Filiatrault A (2001): SAWS – Version 1.0: A computer program for seismic analysis of woodframe structures. *Report No. SSRP – 2001/09*, *CUREE–Caltech Woodframe Project*, Division of Structural Engineering, University of California, San Diego, La Jolla, CA.

[7] Echeverry JS, Correal JF (2015): Cyclic behaviour of Laminated Guadua Mat sheathing–to–framing connections. *Construction and Building Materials*, **98**, 69–79.

[8] Orozco AF (2016): *Experimental research on the effect of vertical load on the behaviour of laminated Guadua structural walls (in Spanish: Investigación experimental del efecto de la carga vertical en el comportamiento de los muros estructurales de Guadua laminada)*. Master's thesis, Department of Civil and Environmental Engineering, Universidad de los Andes, Bogotá, Colombia.

[9] AIS (2010): *Reglamento Colombiano de Construcción Sismo Resistente NSR–10*. Asociación Colombiana de Ingeniería Sísmica, Colombia.

[10] SDPWS–2008 (2008): *Special Design Provisions for Wind and Seismic standard with commentary*. American Forest & Paper Association, American Wood Council, United States.

[11] Folz B, Filiatrault A (2001): Cyclic analysis of wood shear walls. *Journal of Structural Engineering*, **127** (4), 433–441.

[12] Vamvatsikos D, Cornell CA (2002): Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics*, **31**, 491–514.

[13] FEMA 356 (2000): *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Federal Emergency Management Agency, United States.