A Comparison of the Structural Performance of Low-Rise Timber Framed Buildings and Masonry Buildings in a Developing Country (Indonesia).

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Abstract— **This paper is concerned with the seismic resistance of low-rise building structures in Indonesia. Many of the low-rise buildings are of traditional or vernacular construction, not designed or constructed using modern engineering principles and regulations. Timber framed buildings and masonry buildings have been investigated in this study. Numerical simulations have shown knee-braces in timber structures connected using carpentry joints to be effective in providing good resistance to lateral forces. This helps explain their good seismic performance. Low-rise masonry buildings are built either with or without a confining frame of timber or reinforced concrete. The quality of the masonry is not very good: laboratory tests carried out as part of this study on Bali-Indonesia clay brick units have shown their mechanical properties to be poor in comparison with what would normally be required in a seismically active area. Numerical analysis results have indicated that the incorporation of knee-braces into masonry buildings can significantly improve lateral stiffness and strength and thus, seismic resistance.**

Keywords; timber building; masonry building; pushover analysis, failure mode; brittle material.

I. INTRODUCTION

This paper reports on numerical studies of the lateral stiffness of low-rise buildings in Indonesia as part of a research project into their seismic resistance. The resistance of Indonesian timber frame structures in earthquakes has generally been found to be better than that of unframed masonry buildings. Numerical modelling using ABAQUS & SAP 2000 has been carried out in order to simulate resistance capacity of structures for both timber-framed and masonry structures.

More than 65% of low-rise buildings in Indonesia are non-engineered structures *(Kusumastuti 2008)* including; vernacular buildings, housing, public and government facilities. Certain kinds of non-engineered structures are particularly vulnerable during earthquakes. They were not designed explicitly to withstand earthquakes of high magnitude and can only resist a low level of seismic force. Materials used are masonry, timber, mixed masonry-timber and low strength reinforced concrete frames.

II. TRADITIONAL TIMBER FRAME BUILDINGS

A. *Traditional Timber Structures Resistance to Collapse During Earthquakes*

During the Nias, North Sumatra, Indonesia earthquake of 28 March 2005 (8.7 on Richter scale), traditional timber dwellings locally called "Omo Hada" survived without damage. Detailed numerical study of this type of building under seismic loading was carried out by Pudjisuryadi at the University of Petra-Surabaya *(P. Pudjisuryadi 2005).*

Figure 1. Buildings in Padang, Indonesia after the September 30, 2009 earthquake (7.6 on Richter scale). A two-storey timber framed residential building providing safe shelter from rain after the earthquake. [Photo by Kevin Frayer, reproduced by permission *(Frayer 2009)*]

The two storey "Omo Hada" dwellings have braced timber columns to an open-sided lower storey, and timber cladding to a second storey beneath a thatched roof. The column bases simply rest on top of pad stones on the ground, without any mechanical connection. The numerical study by *(P. Pudjisuryadi 2005)* showed that this support system created base isolation which dissipates earthquake energy in shearing and sliding, reducing stresses in the superstructure. The study also reported that the dwelling foundation performed very well by maintaining stability to peak ground acceleration up to 0.38 g. Stresses in all structural members were fewer than 94% of the allowable value.

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In the present study, the structures are vernacular/traditional (Balinese-Javanese, Indonesia) structures in which sawn timber-frames are used with traditional joint arrangements. The buildings represent a form of construction dating from the 15th century, *(Runa 2000)*. Most Balinese traditional timber buildings used a unique measurement system called "Asta Kosala Kosali", where multiples of the lengths of the master builder's own body parts are the key dimension of the structure. A significant number of these buildings are still in use for temples, pavilions, and housing. Some modern modifications have been subsequently introduced in these buildings, however, the principal load carrying mechanisms remains unaltered.

Figure 2. Typical barn "Jineng" in Bali-Indonesia. (a). Front view of timber-framed barn. (b). Close up of the platform of the barn.

These traditional frames are typically in the form of tied portals. A lower horizontal member acts as a tie between the feet of a pair of columns connected between their tops by a horizontal beam. Diagonal and horizontal braces are connected to the column and beam at the corner. Joints in traditional timber frames are mainly of the tenon and mortice type. The stiffness and strengths of the joints are provided by dowel and wedge / peg connectors. The use of dowels/pegs is intended to provide stability during construction and to strengthen the joint at the final stage.

The types of wood most used are Jack-fruit (Artocarpus heterophyllus), Teak wood (Tectona gandis L.) Jati timber, Bingkarai/Balau (Dryobalanops) *(J K Grace 2005)*, Meranti (Shorea Sp., Burck), Kamper (Cinnamomum camphora) and Ulin (Eusideroxylon zwageri)/iron wood. These species of timber have high natural durability. In recent years, the use of Kamper and Bingkarai has increased significantly, because of their low cost and wide availability. These local timbers are hardwoods of strength class D30-D70 [E=9500-21000 MPa, Balau/Bingkarai (E=20900Mpa)] *(British Standard 2002)*. The species above are classified in Indonesian Standard (SNI)-in strength class of E10-E26 (9000-25000 Mpa)*(SNI 2002)*.

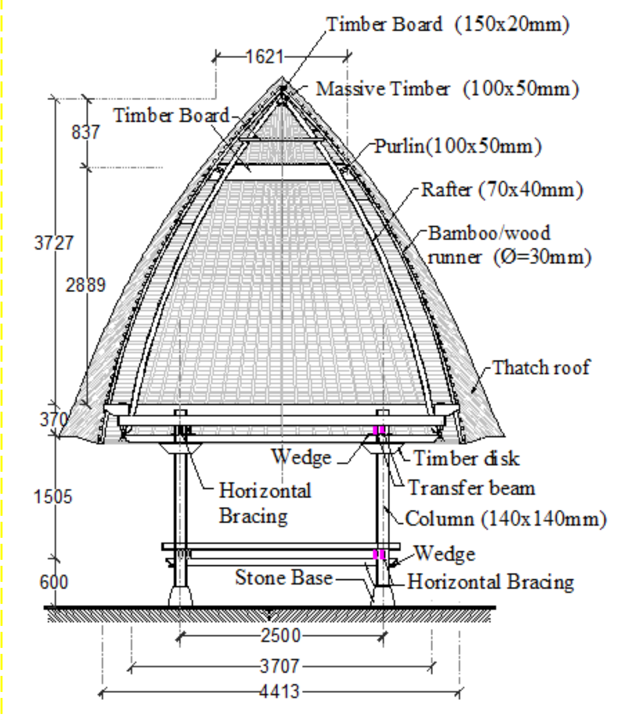

Figure 3. Typical section of timber framed of the "Jineng" Balinese barn

Advanced research has been conducted in developed countries to deal with effective connection methods for timber structures,*(Helmut G.L Prion 1999)*. In contrast, research has been very limited in developing countries like Indonesia *(Runa 2000)*.

B. Geometry of Traditional Frame and Loads Adopted

The structural geometry that was selected for the present study represents vernacular structures for a Granary (Jineng) and a pavilion (Bale) as shown in Figs. 2-3 and Figs. 4-5 respectively. The timber-frame combines saka/pillar/pile/column with horizontal brace (sunduk) or diagonal/knee (canggah wang) as shown in Figs. 2-5.

The roof materials for the barn are thatched with rice straw, grass, or even tiles for modern barns. The structure is two-storey; located at about 500-800mm above the ground is the lower platform. The main upper floor level is used for storage of grain or rice, of density 600-800 kg per cubic metres, for 3 to 6 months. The horizontal bracing or transfer beams are located on the top of the column to support the second platform (main level of storage)

"Jineng" = Traditional Balinese Barn

Figure 4. Scaled timber framed pavilion "Bale" type structure for Gazebo.

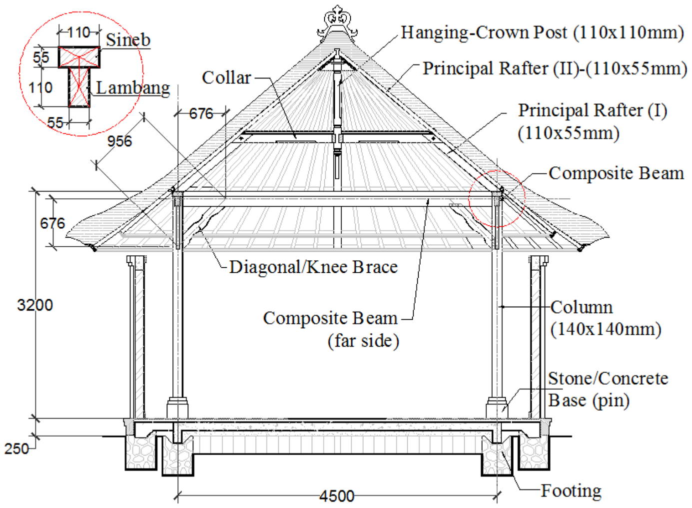

Figure 5. Section of a timber framed adopted for residential uses.

and similarly to split-level where relatively above some distance from the column base.

The "Bale" pavilion is a partially open building with a roof to provide shelter from the rain and other weather effects. Some have masonry walls, which are structurally separate from the timber frame. The pavilions vary in the number of columns (saka) used, being up to four bays in length. *(Runa 2000)*. Typically compound T beams are used (lambing-sineb), spaning between the columns and the diagonal bracing. The top beam accommodates the load from the rafters (pemade) as Fig. 5. This roof arrangement acts as a bracing diaphragm across all the rafters to the perimeter beam. *(Suardana 2004; E. C. Ozelton 2006; KBM 2011)*.

Wind loading analysis to the British Standard (BS EN 1991-1-4:2005+A1:2010) has been carried out in order to know the response of the traditional timber frame structure. The pavilion is categorized as an empty duo-pitch canopy (no blockage or cladding) with a roof angle of more than 20o. The relevant uplift pressure coefficient is in the range of negative 1.2 to 1.4. Basic velocity is taken for the highest wind velocity of 35m/s in offshore area. However, numerical result shown that no significant displacement occurred on the structure. From those remark, lateral load relevant to the seismic activity has turned into account as the most vulnerable load force into the structure. The typical pushover-lateral load is used to present dynamic lateral load by taking lumped-mass procedure to identify seismic action. This lateral load is proportionally generated by the self-weight of the structural elements, the roofing and the ceiling as dead load. The dynamic load is generated by combination between type of gravity load and effect of the increasing displacement, which is generated monotonically. Further study of seismic load is relevant for an actual case, which is, depends upon the local (Indonesian) design response spectrum. The peak ground acceleration of local soil is significantly important to be considered as part of design criteria for further structure to be built in these area.

C. Timber framed with knee brace and tenon-mortice connections with types of dowel (peg) fastener

Mortice and tenon connections are traditional across the world including in the Balinese timber-frame and are still used in modern sawn timber structures. In modern frames, metal is generally used for the peg or dowel connectors, in place of traditional wood. The mortice and tenon connection typically comprises a projecting member (the tenon) which is slid into a slotted hole (the mortice) in the receiving member. Pegs or dowels are inserted through both the mortice and the tenon to secure the connection. A "blind" tenon connection is created when the tenon does not penetrate through to the opposite face of the mortice, as seen in Figs. 9(a). Most "sunduk" or horizontal bracing in traditional Balinese frames use penetrating or through tenons secured with wedges beyond the mortice as shown in Fig. 2.b. (inset). Similar type of connection is found in Japanese traditional semi-rigid joint "Kusabi-Nageshi" (beam-wedge). It had been studied by sustaining the capability of effective responses of the joint for future seismic impact. *(Takeshi Shiratori 2008)*

A number of experimental and theoretical studies have been carried out into the performance of traditional joints, as their performance is not readily predictable by modern design codes. (Berridge 2005; J. D. Shanks 2005). Experiments on timber mortice and tenon joints were carried out in the UK to provide design data for the reconstruction of a medieval barn (Brohn 1988). Braced frames subjected to racking loads have been investigated experimentally by (J. D. Shanks 2006). The stiffness response and strength of a timber frame were found to be significantly improved by knee-braces. Shanks found pullout failure at the joint at the top corner post-beam in which the post was tenoned into the beam and secured by a single peg. Shanks identified that the frame was losing lateral resistance due to sudden pullout failure with complete withdrawal of post-beam connection at 90mm sway deflection. For a timber frame without braces, Shanks found post-beam joint failure governed by failure of the peg connector associated with excessive rotation at the joints. Failure of peg connections

"Bale" = Pavilion, temple shelter, gazebo and traditional Balinese housings.

of oak and steel in traditional timber frame joints has also been investigated (L. Bogue Sandberg 2000) (Rohana Hassan 2010). Investigation of the failure modes of dowel connectors in single or double shear has led to the development of what is known as the European yield model for timber connections (Richard J. Schmidt 1999). This model has been employed in the present study of Bali-Indonesian timber frames. A graphical representation of the failure pattern as determined by numerical analysis is shown in Fig.9 (a).

An investigation by full scale testing of the tensile capacity of mortice and tenon joints looked at cases of braces fixed at different angles to the frame member *(Carson R.Walker 2008)*. Varied-angle between braces and frame has been tested by arranging angle of 90, 67.5 and 45 degrees, in which typical mortice and tenon joints are used. Three primary failure modes were identified as mortice splitting, tenon plug shear, and peg bending and shear.

Figure 6. Buildings in Padang, Indonesia after earthqauke September 30, 2009 (7.6 on Richter scale). Damages to an unconfined single story school building.

III. MASONRY (UNCONFINED & CONFINED) STRUCTURE

Most non-engineered low-rise building structures in Indonesia are of masonry construction. Some are of unconfined masonry construction consist of masonry walls without an enclosing frame. This form of construction is particularly vulnerable in earthquakes, as shown in Fig. 6. The majority of buildings are of 'confined masonry' construction, with a frame of reinforced concrete braced by masonry walls that are built between the frame members. There are also a few buildings in which the masonry is confined by a timber frame. Typical confined masonry building construction has not been designed by an architect or engineer and has been carried out by unskilled or semi skilled builders for housing. In more isolated and rural parts of Indonesia construction quality is lower than elsewhere in the country, and the control over building is less (Basoenondo 2008).

Confining frames of reinforced concrete generally use small size elements; concrete columns and tie or ring beams of section size 110mm x 110mm. Plain round mild steel reinforcing bars are generally used of 6 mm to 12 mm in diameter with yield strength f_y = 240N/mm^2. Guidelines for construction details and specifications for materials and workmanship for housing have been published. These guidelines have been approved by the United Nations as the ideal model for common housing in Indonesia under seismic loading. *(Boen 2009) (Boen and Pribadi 2003).*

A. *Masonry Structures Likely to be Vulnerable to Collapse During Earthquakes*

Even though the UN approved guidelines have been disseminated across the country, continued poor workmanship and the poor quality of materials used leaves structures still vulnerable and prone to collapse during an earthquake. Possibly the builders believe that with the use of reinforced concrete framing, the quality of the masonry units and mortar is not important. The masonry used does not generally meet the requirements for masonry construction in seismic prone areas. *(Boen 2008)*

IV. EXPERIMENTAL AND NUMERICAL WORK

Experimental work was undertaken to assess the characteristics and mechanical properties of masonry as supplied by clay brick factories in the east (Gianyar), north (Buleleng) west regions (Tabanan/Negara) of Bali-Indonesia. Testing was carried out at the Civil engineering Department, Engineering Faculty, University of Udayana-Bali-Indonesia. A full report of the work will be published.

British and European Standards specify minimum values of compressive strength and modulus of elasticity for masonry in seismic regions (characteristic unit strength f_k = 2.5 N/mm^2 and Young's Modulus, E_m= [1000f_k]= 2500 N/mm^{2}) *(British Standard 2005) section 3.7.2)*. The properties of Bali-Indonesian clay brick masonry fell short of these minimum standards by around 40%.

Simple analysis of a typical timber pavilion frame is shown in Fig. 9. Braces are found to be important for improving performance under lateral load. Table I shows that difference values of deflection between right hand side (RHS) and left hand side (LHS) could be (?software wises?) in which material degradation is considered. The sway deflection for the frame without the knee brace (Case 1) is much greater than that for the knee-braced frames (Cases 2 and 3). The 0.707m long brace of Case 3$^{(a)}$ reduces the deflection by 26 %$^{(a)}$ and 45 %$^{(a)}$ of deflection for single brace in case 2$^{(a)}$ and for non-brace in case 1 respectively. The 1.2 m long brace of Case 3$^{(b)}$ reduces the deflection by 48 %$^{(b)}$ and 73 %$^{(b)}$ of deflection for single brace in case 2$^{(b)}$ and for non-brace in case 1 respectively. According to (E. C. Ozelton 2006), maximum allowable deflection for timber frame is equal to [0.003 x height] for sway frame structure, in this particular case, acceptable deflection is allowed up to 6 mm. Even though the result shows that the use of knee brace is important, problems are remaining in which inelastic analysis is considered (this is vague). Relevant study is needed to investigate plastic material model (vague again).

In Sap2000, combined vertical and lateral load are also taken proportionally into account on the top of the structure, a significant improvement is expected by the advent of diagonal braces. Both braces (left and right side) responded in compression as expected due to vertical load alone is applied or initial

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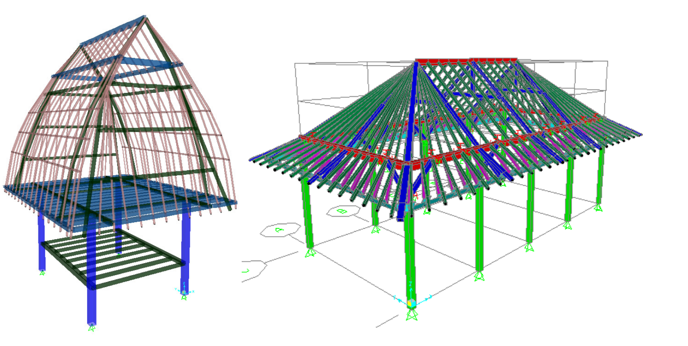
low intensity of lateral load is acting on the structure. Once massive horizontal load (seismic force) is imposed to the structure, the bracing will acts differently, one is being in tension and compression to the other side.

TABLE I. STRUCTURAL IDEALIZATION FOR TIMBER-FRAMED MODEL(1)

Case	Lateral load (H = 2500 N) for column height h=2m & beam span L = 3m, brace [(a)] = 0.707m & brace [(b)] = 1.2m, maximum allowable deflection: Δ = 0.003h =6 mm				
	Deflection (mm)		*Max Bending Moment (kN.m)*	*Section / Element (mm)*	*I / E (cm^4) / (MPa)*
	LHS	*RHS*			
1	8.15	8.14	2.5	T – Beam (2x55/110)	3084 / 20900
2	6.01 [(a)] *4.21* [(b)]	6.01 [(a)] *4.20* [(b)]	2.15 [(a)] *1.86* [(b)]	Brace (55/110)	601 / 20900
3	4.46 [(a)] *2.18* [(b)]	4.44 [(a)] *2.17* [(b)]	1.88 [(a)] *1.45* [(b)]	Column (140/140)	3201 / 20900

In addition, the use of braces is provides advantage in order to prevent excessive buckling on the beam since the braces is taken axial force proportionally relevant to the ratio around 5 to 1 in which comparison made between knee brace and beam. Axial compression load has taken by the knee/diagonal bracing partially 5 times greater than the beam. The braces is considerably reduced the effective length of the beam so that it can improve the performance of the frame in general.



(a) (b)

Figure 7. Elastic model in SAP 2000 & Abaqus for (a) "Jineng" & (b) "Bale".

A. *Pushover Analyis Procedure for Traditional Timber Frame*

Simulations are also conducted relevant to frame of 3m height, which is associated with Fig. 9. In this particular case, linear static pushover procedure is adopted by using Abaqus simulation. Vertical load is taken into account for typical load of traditional roof. Dead weight of typical roof on traditional timber is taken as total mass applied into the timber frame model. The self-weight of timber is generated by density of Belau timber of 1250 kg/m^3 and dead load due to roof cover is used of 40 kg/m^2 for typical wooden roof ["iron wood" or "sirap"]. Finally, line load of 1.433 kN/m is applied into the beam. Deflection control is relevant to maximum allowable deflection for timber frame is equal to 9 mm for sway frame structure is also taken into account.

An obvious problem faces for the braces in which one will take axial tension, timber pegs is suffering significantly compare to the one under axial compression. However, in this particular case, assumption has also sufficiently been adopted that weaker connections compare to the frame member is necessary. This assumption has been found to be valid in which stiffer and stronger frame member compare to the relatively flexible of connection in simulation. In Abaqus simulation found that highest stress distribution is concentrated dominantly on the peg and mortice-tenon joint, shown in Fig.9 (a). In this figure, ffailure mode of the pegs has also discovered relevant to typical failure orientation. The failure is recognized as European failure mode pattern of V_d is also found in this numerical work.

To predict collapse load of frame in lateral direction, Abaqus provides typical energy approach in which external work is equal to the energy dissipation during the collapses of the frame. This method can be relevant to the General method and modified Riks method in which geometrically nonlinear static problems procedure is adopted for collapse behaviour of frame model. This method is also suitable for predicting large deformation in which effect geometric non-linearity is involved *(Abaqus 2012)*. The result can be seen in Fig. 11(d).

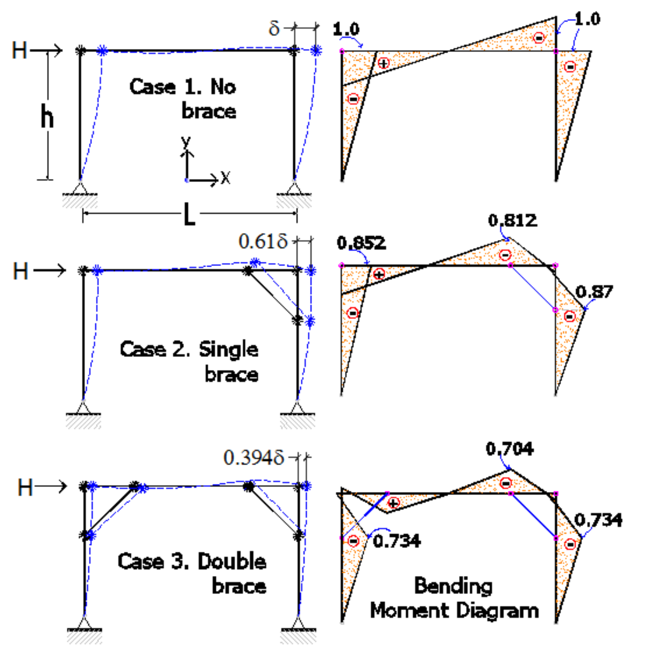


Figure 8. Numerical and structural analysis resulting averaged moment-capacity and deflection ratio among three different cases for single bay, single storey timber-framed structure; case 1 (no brace), case 2 (single brace) and case 3 (double brace – both side)

From Abaqus modeling (Fig.9.), there is no pullout failure occurred at the top corner of post-beam connection. It found to be better than previous study (J. D. Shanks 2006), it may be due to typical locking system of traditional carpentry joint is providing better restrain rotation at the joint. Interlocked system provided by typical fork and tongue joint with extended beam out of the central joint between the post and beam. However, splitting failure is found at typical composite beam if no mechanical connectors are used. In addition, varied lateral load is intro-

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duced in order to examine the lateral load capacity generated by using scale factor of typical ground peak acceleration (g). Maximum capacity of lateral load for timber framed is found of around 50 kN with maximum displacement of up around 200 mm under lateral load of 0.25g, 0.5g and 1g.

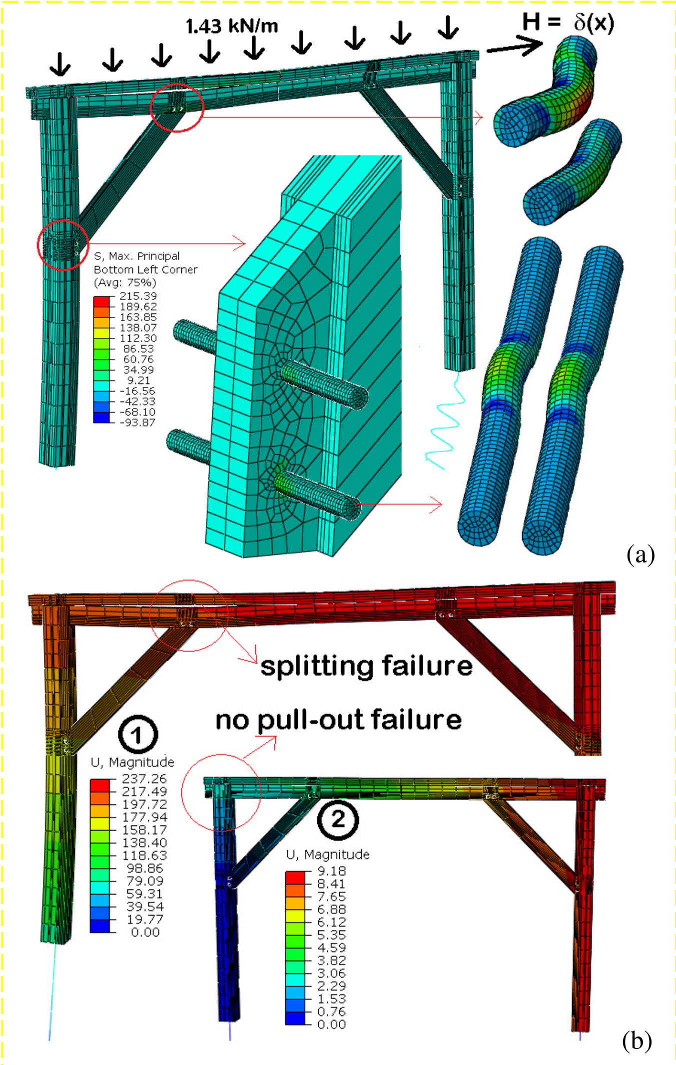

Figure 9. Full model and coupling-kinematic model used on traditional framed-timber relevant to Table II : (a) Stress distribution on frame members, the peg connector and spliting failure at the beam and typical failure mode on peg (European Model) (b) Under pushover- monotonic load: (1) Deflection under max loading, (2) At max allowable deflection (Δ=0.003h=9mm): Lateral load= 7.33k N

In one part, the advantages of knee brace is adopted as a sample method for retrofitting approach or it may be used for design structures of confined masonry structure in the future, shown in Fig. 10. On the other hand, traditional joint seems complicated in behaviour so that investigation is required such as the use of plastic material for modelling. It can tough the detail study out for another times.

B. Pushover Analyis Procedure for Confined Masonry

Un-confined masonry (UMR) is the most vulnerable structure during an earthquake event. The resistances used to be better for the gravity load only rather than horizontal racking load, which is the most catastrophic external force, used to brings unstable conditions for the standing wall. Most of masonry wall does not act in a ductile manner so as brittle failure occurs when the stress state is exceeding the strength wall. The uses of steel reinforcement bar on concrete frame is intended to provide more ductile structure and to provide resistance to lateral load by producing ductility and strengthen the masonry wall from total collapse.

The linear and non-linear pushover analysis procedures *(SAP2000 2011)* are carried out as one of the prominence procedures in performance based design. The static linear and non-linear procedure in which applied load is incrementally increased in accordance with the loading pattern such as monotonic loading in order to find the failure mode of the structure. To predict peak responses of structural members, the effects of lateral loading and behaviour are estimated by using typical incremented lateral force and deformation control criteria.

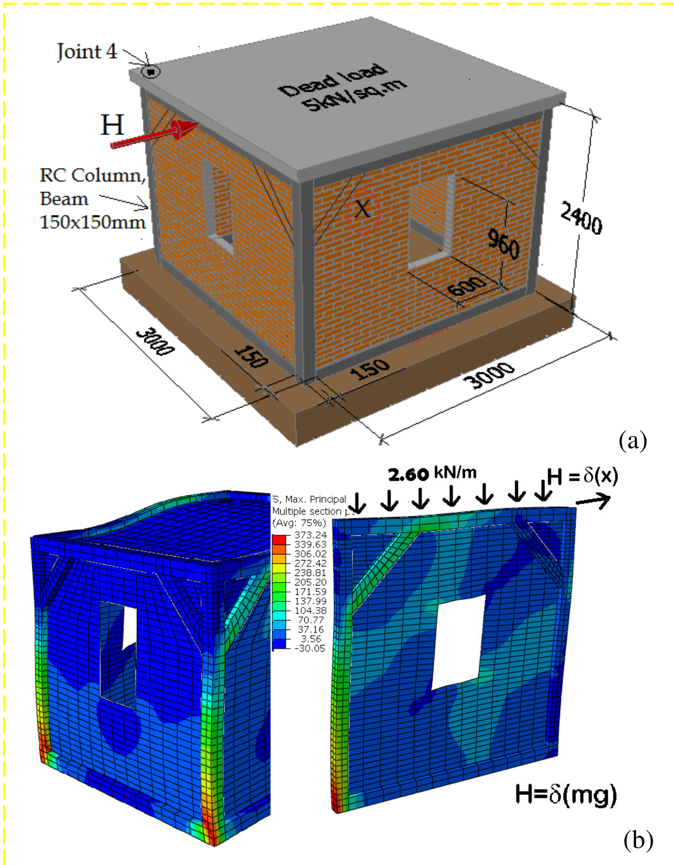

Figure 10. (a) An idealised single-bay, single-storey of a confined masonry building with knee brace (X indication). (b) In Abaqus, stresses distribution on conrete frame and masonry wall: the use local Bali-masonry for full and single plain model.

The procedure is adopted to analyze idealized infill masonry structure in Fig.10 with section properties provided in Table III. Typical concrete strength is used of fc' =18 N/mm2 and mild steel of f_y =240 N/mm^2 simulated by 3-D model using shell and beam element in Sap 2000 and Abaqus. In Abaqus, dead load of concrete roof slab (thick=110 mm) which is change into line load of 2.60 kN/m is applied at the top of beam as the initial load (starting point). For instance, by generating a scale factor of 0.1g acceleration in one direction (x) laterally, the linear or non-linear static pushover analysis is adopted. Similar to timber frame model, lateral load can also be applied monotonically at a rate velocity of 2 kN/sec or under rate load procedure such as 0.05N/mm^2/sec (BS EN 772-1:2000). In Sap2000, design dead load of 5kN/m^2 and self-weight are used. Displacement control is locates at the top of structure which is expected to deflect up to 500 mm. Point hinges are considerably placed offset from joint members so as damages can be predicted by hinges performance at the loading carrying capacity up to the maximum moment is reached.

TABLE I. IDEALIZATION FOR CONFINED MASONRY-STRUCTURAL MEMBERS

No	Section Schedule			
	Name	*Second moment of area (cm^2)*	*Section Area (cm^2)*	*E (MPa)*
1	Beam	4219	(15x15)=225	21526
2	Column	4219	(15x15)=225	21526
3	Brick Wall	225000	(10x300)=3000	1433.2

The magnitude of the loading is incrementally increased until the weakest link or the failure mode of RC framed are found. Level of failures are associated with performance point defined of force-deformation criteria for flexural hinges with acceptance criteria according to the ATC-40 and FEMA-356 documents and it is line up with the Eurocode-8:2004-3. Local non-linear effect (flexural hinges) is modeled as discrete hinges at several locations along the length of frame such as at the joints, column-tie beam, brace and beam joints. Result of pushed-over suggested that a collapse mechanism is developed for plastic deformation in the plastic zone length (offset to certain length from the joint).

The result from Sap 2000 suggested that distribution of the stresses on the wall (shell) were reduced up to 12.5 % as the result of introducing knee brace compare to the infill masonry without knee brace. Maximum deflections between infill frame masonry without and with knee brace from linear pushover analysis are found of 27.59 mm and 19.25 mm and for non-linear static analysis, maximum displacements have been found of 71.72 mm compare to 26.3 mm for frame without and with braces respectively. Ultimate capacity is reached with collapse criterion at coloum base joint for confined masonry without braces and no significant damage on infill frame structure with knee braces. In Abaqus simulation, by using lateral non-linear pushover, lateral load capacity for confined masonry is found maximum of 160kN and the displacement recorded up to 300 mm. For timber frame structure, lateral load capacity is found of 45-50 kN with maximum displacement of less than 200 mm. Both structures were given initial small (10-100mm) and large displacement controled relevant of 0.1g*s^2 =980mm, 0.25g*s^2 =2415mm, 0.5g*s^2 = 4903mm and 1g*s^2 =9806mm.

V. CONCLUSION AND DISCUSSION

The study used numerical to examine the behaviour of Balinese traditional timber frame and confined masonry structures, observing how their capability to response lateral load related to the influenced of knee braces. The braces on traditional timber frame reduces deflection approximately 40% compare to the one without braces. Results show that mechanism failure is occurs for the uses of pin joint stiffness. Therefore, this study suggests that semi-rigid or fix joint can be more relevant to use in the tenon-mortice joint of traditional timber frame. Pull-out failure is not occurred at the beam-column joint that is potential improvement because the braces and joints prevent large drifts to make low rotational stiffness of connection.

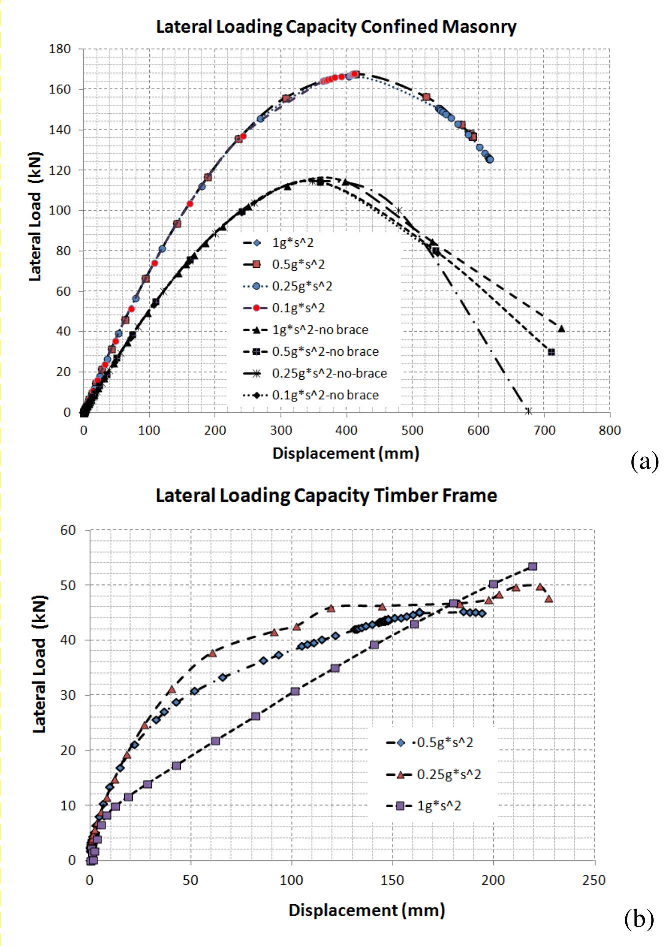

Figure 11. Lateral load capacity between confined masonry and traditional timber frame under monotonic loading: (a). By giving small displacement as a lateral load (b). Large deformation designed as lateral load, (c). Load equally given of 0.25g, 0.5g and 1g for timber frame structure under influences of geometric non-linearity along with General or Riks Method simulation.

The used of knee brace into the confined masonry is reduced deflection more than 50% compare to the one without knee brace. The braces are providing for better lateral stiffness and better level of responses under performance based design procedure. However, it has been found that is no significant effects on displacement, for which weaker clay brick is used in confined masonry. Therefore, the results suggested that UMR structures are not recommended in this region, unless advanced treatment is applied for the structures. In addition, it has been found that lateral load capacity of confined masonry is greater than timber frame by factor of four (4). Ratio of mass is count-

* = multiply ; s^2= second * second.

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ed by factor of 12 which total mass is recorder of 2423 and 210 kg for confined masonry and timber frame respectively. Centre of gravity is defined of 2.73 and 2.36 m away from the supports respectively.

This paper is clearly under developed, as the nature of preliminary works, more significant works in progress will be reported for the next publications. Further works will deal with plastic material model for UMR, confined masonry and framed-timber structures under dynamic load. In performance base design structures, crack failure on structural members could also mainly be concerned for further study.

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