HORIZONTAL - LOADING TEST OF STEEL BAR-TIMBER COMPOSITE COLUMN FOR MID - RISE BUILDING

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ABSTRACT: Recently, timber buildings are desired from a viewpoint of global warming, and moreover, in severe earthquake prone zones, such as Japan, they are more desired on the grounds of light weight of timber members. We have been developing a frame system formed by timber members strengthened with steel deformed bar (rebar) using epoxy resin adhesive. Its columns could produce, in mechanical properties, better performance than those of reinforced concrete structure (RC). This paper reports a horizontal loading experiment of two column specimens, its experimental results and calculation estimation of those behavior. The specimen was modelled for 5-7 storey buildings, i.e., mid-rise buildings. Performance of the specimen indicated to be better than that of RC columns with its same size except concrete.

KEYWORDS: Hybrid timber, Column, Deformed steel bar, Moment-resisting connection, Re-centering

1 INTRODUCTION

Recently, from a viewpoint of Global environment, timber, i.e. one of nature-cycle materials, has been attempted to be utilized as structural members of large timber buildings in world-wide. A representative timber member is Cross-laminated timber (CLT), however, CLT structural system very often restricts planning of building owing to CLT being plate member. High-stiffness-strength-timber slender beam, column, and timber structural system are significantly desired.

S. Shioya, i.e. one of authors, proposed the structural system and construction for buildings, adopting Hybrid Glulam Timber members using Steel bars (HGTSB, nicknamed “Samurai” in Japan) [1-3]. An 11-storey building adapted two HGTSB beams for a trial, with a refractory coating authorized as two-hour fireproof timber for HGTSB, constructed in Tokyo, Japan, February in 2020.

We now have been developing more refined and more commercial competitive structural system for buildings adopting HGTSB and its structural design methodology, as view of the followings: i) column producing moment-resisting and re-centring ability in big earthquake for low-rise and mid-rise buildings ii) beam-column connection producing the re-centering ability, iii) calculation methods for stiffness and strength of full-scale beam, iv) heat resistance of beam and column at fire, and v) creep property of beam under long-term loading, including effects of Mechano-sorptive creep and higher temperature. This paper reports an experimental loading test of column planned to yield by bending for mid-rise buildings of 5-7 storeys or less.

2 BACKGROUND

The study on hybrid steel bar-timber beam to improve bending stiffness and strength of timber beam was initialized by Granholm H. [4]. Also, after then, studies were and are conducted by using other materials beside steel bar, but most of studies focused on only bending stiffness and strength of the hybrid beam under short-term or long-term loading; no studies have been reported on the connection methods between column and beam to enhance the composite members, such as moment-resisting connection for rigid frame. S. Shioya has developed a technique for the rigid connection between rebars, inside of the hybrid beam, using carbon fiber plastic sleeve (CFS) and epoxy resin adhesive, as well as work process of the glued-in-rod, and reported the potential of adopting adopted the technique to the column, producing high stiffness, high strength, and abundant-energy dissipation through loading test of a column specimen with + shaped cross-section [2].

3 EXPERIMENT

3.1 SPECIMEN

Figure 1 shows specimen configuration and cross section. The number of specimens was two, named as No.1 and No.2. Force-applied height ‘h’ was made to be two types of 1300mm (No.1) and 1050mm (No.2). By lowering the

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height, it was expected that horizontal force capacity at bending fracture will increase by 24%; maximum shear stress of web will become 120% of standard shear strength of wood; the shear fracture will be occurred. The columns were fabricated in a square cross section by secondary adhesion. The rebar in column were arranged such as double array rebar in peripheral portion in the column cross section, and its total rebar ratio \( p_r \) was 4.09% over column section, and its tensile rebar ratio \( p_t \) and compressive rebar ratio \( p_c \) of the double array bar was 1.54%. These rebar ratios are very high amounts that cannot be arranged in RC column because bond-splitting failure around rebar may occur at the amount. The column bottom was connected with a reinforced concrete stub. After shaping the column, heat-resistant epoxy adhesive, shown in Table 1, was filled in insertion holes in the column bottom for joint rebar, and then the joint rebar, 32 bars of D13 for this case, was inserted in the holes each to bond the built-in carbon fiber plastic sleeve (CFS) with the joint rebar. After the adhesive hardened, the column bottom and the RC stub were connected by inserting the joint rebar into voids in the RC stub, for anchorage and then filling non-shrinkage grout into the column bottom-stub gap and the voids simultaneously. Fresh concrete of nominal strength Fc42 was casted for concrete of the RC stub. Two steel plates plastered with epoxy adhesive were screwed with on the column top portion to prevent the portion from locally fracturing by the horizontal force. Glulam timber was prepared according to E65-F255 in Japanese Agriculture Standard, and resorcinol-based resin adhesive was used for the glulam timber. Table 2 lists results of Young’s modulus and strengths of laminas of the specimen’s timber of which section size was 25×25mm\(^2\); the number was 10; loading was tested as four-point bending. Strain was measured by foil gages. The moisture content of the test pieces was 14.8% and the density was 0.45g/cm\(^3\) in average. Table 3 lists mechanical properties of the rebar by testing.

### 3.2 CARBON FIBER PLASTIC SLEEVE (CFS)

Figure 2 shows shape and dimension of CFS used, which is specified for joint rebar/deformed steel bar D13. Each rebar is inserted through holes at both ends of the CFS; CFS is filled with epoxy adhesive shown in Table 1 to join both one-side of rebars in rigid. Insertion length of each rebar into one-side of the CFS was specified to be 9.6 times of rebar diameter \( d \). As for carbon fiber, fiber sheet A1: carbon fiber weight, 400g/mm\(^2\); fiber length, 250mm; sheet width, 200mm; the sheet A1 was wound 200mm in the circumferential direction with the fiber direction aligned in the axial direction of the sleeve, and then from above it, fiber sheet A2: carbon fiber weight, 200g/mm\(^2\); fiber length, 220mm; sheet width, 250mm; the sheet A2 was wound 220 mm in the circumferential direction with the fiber direction aligned in the circumferential direction orthogonal to the axial direction of the sleeve. The CFS of No.1 was formed by an epoxy adhesive dedicated to the carbon fiber sheet, and that of No.2 was formed by the heat-resistant epoxy adhesive of Table 1. Inner diameter of CFS is 16mm and the outer diameter is 22mm. Figure 3 shows stress-strain relationship curves of test pieces of CFS joint for D13 by tensile test, where are two joint test pieces (CFS-1, CFS-2). The stress was calculated by dividing the tensile force by the nominal cross-sectional area of rebar D13. The strain was calculated by dividing

| Table 1: Epoxy adhesive property (Unit: N/mm\(^2\)) |
|-------------------------|--------|--------|
| E \( (2700) \) | Fc \( (110) \) | Fcs \( (60) \) |

**E:** Young’s modulus, \( Fc \): Compressive strength, \( Fcs \): Compressive shear strength

| Table 2: Lamina property (Unit: N/mm\(^2\)) |
|-------------------|--------|--------|
| Grade | Bending test | Compressive test |
| \( L70 \) | 9805 | 78.78 |
| \( L75 \) | 9735 | 42.92 |

**E:** Young’s modulus, \( F \): Compressive strength

| Table 3: Rebar tensile property (Unit: N/mm\(^2\)) |
|-----------------------------|--------|--------|
| Rebar | \( E \) | \( \sigma_y \) | \( \sigma_u \) |
| D13 | 1.09x10^5 | 361 | 522 |

**E:** Young’s modulus, \( \sigma_y \): Yielding strength, \( \sigma_u \): Braking strength

| Table 4: CFS tensile property (Unit: N/mm\(^2\)) |
|-------------------|--------|--------|
| Specimen | \( E \) | \( \sigma_y \) | \( \sigma_u \) |
| CFS-1 | 1.61x10^5 | 371 | 530 |
| CFS-2 | 1.85x10^5 | 367 | 532 |

**E:** Young’s modulus, \( \sigma_y \): Yielding strength, \( \sigma_u \): Braking strength

![Figure 1: Specimen configuration and cross section of column](image1)

![Figure 2: Carbon fiber sleeve (CFS) and connection of rebars](image2)

![Figure 3: Tensile stress-strain curve of CFS](image3)
elongational deformation along axis measured in the section (450 mm) containing the CFS by the length of the measured section. The test pieces did not fail at the joint but rebar portion outside of the CFS ruptured. Thus, the tensile strength of the joint can be judged to be equal to or greater than the strength of the rebar. The relationship of rebar (D13) is also shown in Figure 3; its measurement length was the same as the CFS test piece. Basically, the CFS is specified to be such that the CFS joint does not fracture and the rebar ruptures after its yielding. Table 4 shows mechanical properties of the CFS by the test. The axial stiffness of CFS, calculated in the same way as Young’s modulus, was 1.61×10⁵N/mm² and 1.85×10⁵N/mm², and their average was approximately 90% of Young’s modulus of the rebar.

3.3 YIELD SECTION OF REBAR

As shown in Figure 4(a), the joint rebar is devised to yield at the section between the RC stubs and the CFS, resulting in plastic elongation and contraction only over the section. Wood portion around the rebar is devised to be such that even if the rebar yields, the portion will not crack.

3.4 MECHANISM OF MINIMIZING RESIDUAL DEFORMATION OF COLUMN

As shown in Figure 5(a), yield bending moment ‘My’ of a column, which yields in bending by the joint rebar yielding in tensile, consists of moment component ‘sMy’ borne by the tensile rebar and moment component ‘Mn’ borne by the axial force of column. The yielded joint rebar will generate plastic elongation, resulting in plastic rotation angle in column bottom. After unloading, residual deformation occurs in the column due to its plastic rotation angle. In order to reduce the residual deformation, the tensile-yielded rebar needs to yield in compression, and the yield can be achieved by the moment Mn by axial force. Mn is defined as restoring moment, and ratio of ‘Mn/sMy’ is here named as restoring moment ratio ‘γ’. When γ is larger than 1.0, the residual deformation decreases more. Moreover, when vibration after maximum response deformation of building during big earthquakes is taken into account, the plastic rotation will decrease even if γ is smaller than 1.0.

3.5 LOADING

Figure 6 in the next page shows set-up for loading. The vertical load ‘Fv’ was applied to the column head; horizontal deformation of the horizontal force was gradually increased; the horizontal force was applied positively and negatively repeatedly, increasing the horizontal deformation. Fv divided by axial force capacity of column ‘Nu’ is defined as ‘η’ (Fv/Nu); the axial force capacity Nu is the product of compressive strength of glulam timber Fc (=20.6N/mm²) and cross-sectional area of the column (=Fc・b・D), ‘b’ is column width and ‘D’ is column depth. The target deformation angle ‘R’ was assumed to be one by dividing the horizontal deformation by height h: h=1300 mm for No.1 and 1050 mm for No.2. Figure 7 shows target displacement protocol for the horizontal loading. At first, the axial force of column was maintained to be constant at an axial force ratio 10%, and the target angle was gradually increased to 2.0×10⁻² rad. This is named as the first stage. After this, the horizontal displacement was returned to zero and the axial force ratio was changed to 6%, 15%, 25%, and 10%, and the horizontal force was applied in the same way as in the first stage. The axial force ratio of 6% was used for the second stage, 15% for the third stage, 25% for the fourth stage, and 10% for the fifth stage. After the first stage, the horizontal displacement of column was returned to zero; the target angle was gradually increased to 2.0×10⁻² rad, by keeping the axial force of the column constant at the axial force ratio of 6% (the second stage). Similarly, by
returning the angle to zero again, the axial force was to be constant at the ratio of 15%: the target angle was gradually increased to \(2.0 \times 10^{-2}\text{rad.}(\text{the third stage})\). In addition, the angle was returned to zero; the axial force was to be constant at the ratio of 25%: the target angle was gradually increased to \(2.0 \times 10^{-2}\text{rad.}(\text{the fourth stage})\). After that, the horizontal displacement was returned to zero, and the axial force was changed again to the ratio of 10%, and the target angle was gradually increased to \(2.0\times10^{-2}\text{rad.}\) in the constant axial force (the fifth stage). Finally, the target angle was progressively increased with the constant axial force until destruction of the column (the sixth stage).

### 3.6 QUASI-STATIC LOADING FOR FREE-VIBRATION

In order to confirm that the residual deformation can be reduced after the maximum response deformation during big earthquakes, quasi-static loading was applied after several target angles were reached, assuming damped free vibration. For more information of the loading force, please refer to Literature 5. Figure 8 schematically shows a hysteresis loop obtained by the loading. Final residual deformation \(\delta_F\) was assumed to be the average of \(\delta_1\) and \(\delta_2\).

### 3.7 MEASUREMENT METHOD

Figure 9 shows set-up for displacement and strain. Displacement transducers, Disp.1 and Disp.2, were used to measure horizontal deformation between the stub and the horizontal force-applied point; Disp.3-6 were used to measure angle of rotation in hinge region of the column bottom; Disp.7 and Disp.8 were used to measure the shear slip deformation in the loading direction between the bottom and the stub surface. Axial strain of wood at both flange surfaces of timber and shear strain of wood of web were measured with strain gauges at the locations shown in Figure 9(b)(a) and (c). An axial strain of joint rebars were measured within the RC-stub side.

![Figure 8: Quasi-static loading for free-vibration and determination final displacement \(\delta_F\)](image)

![Figure 9: Set-up for displacement and strain](image)

### 4 EXPERIMENTAL RESULTS

Figures 10 and 11 show failures of the specimens; Figure 12 and 13 show horizontal force-displacement relationship of the column’s top. The displacement angle is the same as \(R_t\) described above. For the specimens, the loading finished when the target displacement reached limit of the loading jack.

#### 4.1 FAILURES

##### 4.1.1 FAILURES OF NO.1 AND NO.2 UP TO THE FIFTH STAGE

Regarding yielding of rebars in No.1, in positive loading, outermost joint rebar yielded in tension at \(0.90\times10^{-2}\text{rad.}\) of R10 (axial force ratio 10%), and in negative loading, second rebar from outside yielded in tension at \(-0.85\times10^{-2}\text{rad.}\). In No.2, at R10 (axial force ratio 10%), the joint rebar at the outermost yielded in tension at \(1.0\times10^{-2}\text{rad.}\) and the second rebar yielded in tension at \(-1.0\times10^{-2}\text{rad.}\). Regarding damage to the wood, it was observed only in No.2, where fine longitudinal cracks were observed at the first stage of \(0.5\times10^{-2}\text{rad.}\) as seen in Figure 11(a). No other damage was observed in the specimens until \(2.0\times10^{-2}\text{rad.}\) in the fifth stage. In RC-stubs, fine cracks on the top surface due to pull-out force of the joint rebar by bending were observed, and the largest width of the cracks was 0.35 mm on the top surface and 0.25 mm on the side surface of the sub for No. 1. The first crack occurred at \(+0.75\times10^{-2}\text{rad.}\) with an axial force ratio of 10%. In No. 2, the maximum crack width was 1.75 mm on the top surface. It can be said that the hybrid column can be 5 times subjected to cyclic-reversed loading to \(2.0\times10^{-2}\text{rad.}\) without conspicuous damage.

##### 4.1.2 Fracture at the sixth stage (large deformation) of No.1

Just before the first \(-2.25\times10^{-2}\text{rad.}\) in the sixth stage, vertical cracks appeared in wood along the carbon fiber sleeve (CFS) on the right-side tensile surface as shown in Figure 10(b). This was the first damage to wood. At this time, a sound of CFS breaking was heard once. After this, a longitudinal crack progressed along the rebar with repeated force application. On the left side, longitudinal cracks appeared along CFS of the corner at \(-4.0\times10^{-2}\text{rad.}\). After that, wood portion covering over the rebar was buckled by compressive force and popped out of outside. It can be judged that the failure is due to compressive force in bending. The aforementioned breakage of CFS on the right side was visually confirmed when the target angle was \(-4.5\times10^{-2}\text{rad.}\). Horizontal shear misalignment deformation occurred within the CFS joint between the joint rebar at one outermost corner and a rebar within the column jointing with it. We have conducted many loading experiments for beams and columns adopting CFS and have not experienced any fracture of CFS. The reason of this fracture may be that cover thickness of the wood over the CFS was reduced compared to those. As only one CFS was fractured, the decrease in capacity of column strength was limited to 4-5%. After this, the capacity remained nearly constant until the final deformation.
4.1.3 Fracture at the sixth stage of No.2
Just before the peak of \(-5.0\times10^{-2}\) rad. in the sixth stage, vertical cracks appeared in the wood due to compression by bending, as shown in left side view of Figure 11(b). This was the first major damage to timber. After this, the cracks progressed along those near rebars as repeated force application. On the back side, vertical cracks with width of about 0.5 mm in wood reached the steel plate of column at the first \(-2.25\times10^{-2}\) rad. cycle. No damage to the CFS occurred in this specimen. The capacity at \(-5.0\times10^{-2}\) rad. was almost maintained until the final deformation. If CFS is prevented from rupturing, it can be said that it will exhibit extremely stable capacity. CFS was broken only in one within No.1, but it will be necessary to investigate to secure the cover thickness of wood against CFS to prevent the rapture. Concrete spalling occurred at the left corner of the bottom of the RC-stub. It is thought that there was a problem with the flatness of the bottom of the RC stub that contacts the reaction frame.

4.2 HORIZONTAL FORCE-DISPLACEMENT RELATIONSHIP AND LOOPS
Figure 12 and 13 show loops at each stage. In the second stages through the fourth stage, envelope curve of the stage just before each stage is shown as a red curve. The fifth stage shows the curve of the first stage. After the first stage, due to the Bauschinger effect, stiffness and yield capacity decreased compared to R10. Figure 12(h) and Figure 13(h) show loops of the quasi-static loading imitating a free-vibration from \(+2.0\times10^{-2}\) rad. in each stage. As the axial force increases, the maximum force capacity should increase, but after the second stage, the increase in the capacity was comparatively small. As force to pull out the joint rebar from the RC-stubs was generated repeatedly in the first stage, as a result, the stiffness of the fixing portion was reduced, and the increase in the capacity by the increase in axial force can be thought to have been cancelled out by the reduction of the stiffness. In No.2, the maximum force capacity was increased by shortening column length compared to No.1, and the shear force was larger than the design calculation shear capacity. However, no sign of shear failure occurred at all, and at R10, after bending yielding, stable capacity was maintained up to a deformation angle of \(7.14\times10^{-2}\) rad. In the other axial force stages, the horizontal loading was conducted to be up to \(2.0\times10^{-2}\) radian. Thus, it can be judged that the maximum capacity was almost maintained up to \(2.0\times10^{-2}\) rad. though there was a decrease in stiffness due to repeating loading, as seen in Figure 12 and 13. Except the reduction in stiffness due to the Bauschinger effect, the loop produced abundant energy dissipation even after repeated loading up to \(2.0\times10^{-2}\) rad. Figure 14 shows changes of the energy dissipation of a loop at each target angle. Because the energy dissipation will be derived from yielding of joint rebar in the column bottom, its amounts of each specimen obtained during the cycle of same target angle can be thought to be nearly the same.
without much influence of the axial force. The energy
dissipation of No.1 was larger than that of No.2 as seen in
Figure 14. This is due to the fact that the force-applied
height of No.1 is higher than that of No.2; the rotation
angle at the hinge of the column’s bottom, in No.1,
becomes larger than the target deformation angle $R_t$. The
experimental results in the next section will confirm this.

4.3 MOMENT-ROTATION LOOPS OF PLASTIC HINGE

Figure 15 shows moment-rotation angle relationship of
column’s hinge. The angle is the angle measured by
displacement transducer 3-6 in Figure 9(a). From the third
stage on, the stiffness of No.1 is smaller than that of No.2
in the negative loading. The cause of that is now unknown.
The maximum of the rotation angle of each loop of No.1
in the dashed line is larger than that of No.2. That is effect
of the height of the horizontal loading as described in the
previous section. The maximum shear force of No.2 is
1.24 times higher than that of No.1 because of the
difference in the height of the loading between No.1 and
No.2. However, this difference in shear force is seen to
nearly never affect adversely to the moment-rotation angle relationship. In RC columns, if the amount of shear reinforcing rebar is small, bending shear cracking occurs in the hinge region, resulting in an inverted S-loop shape with low energy dissipation as the shear force increases. On the other hand, as those hybrid columns do not crack at web of glulam timber, bending characteristics of the
hinge are not likely to be degraded by the difference in shear force.

4.4 RESIDUAL DISPLACEMENT SUPPRESSION AND RESTORING MOMENT

Figure 12(g), 12(h), 13(g), and 13(h) show hysteresis loops by the quasi-static loading for free-vibration at the target deformation angles of 1.0×10^-3 radian and 2.0×10^-2 radian. Figure 16 shows variation of residual displacement angle with the target angle and compares that of each stage. ‘Rf’ is the deformation angle of δf in Figure 8(b), and ‘Rt’ is the deformation angle of δt in Figure 8(b). It can be confirmed that the final deformation angle Rf decreases from Rf due to the quasi-static loading for free vibration. Figure 16(a) and (b) show loop in the first stage and the fifth stage. The reduction of the residual deformation angle did not degrade much even after the loading of until 2.0×10^-2 rad. was applied 4 times. Figure 16(c) and (d) show comparisons of the second stage-the fourth stage. The residual displacement angle of No.2 is larger than that of No.1. It is thought to be an effect of the loading height. The horizontal single-dotted line in Figure 16 is the visual allowable limit of the residual displacement angle of 0.25×10^-2 radian. The angle is the upper limit of the angle at which the inclination of column cannot be visually recognized. The restoring moment ratio γ of each specimen was 0.16 for R10, 0.10 for R6, and 0.33 for R25. Even when γ was 0.1 and 2.0×10^-2 rad. is experienced, it was suppressed below the allowable limit. For RC column, γ is required to be at least 0.6, which means that the hybrid column have superior performance in suppressing residual deformation. This main reason is that the shear deformation component accounted for a large proportion of the deformation in HGBTB, and the shear deformation was close to that of elasticity. This detail will be mentioned in the next section.

4.5 BENDING STRAIN DISTRIBUTION IN FLANGE AND SHEAR STRAIN DISTRIBUTION IN WEBS

Figure 17 shows distribution of axial strain both side of glulam timber by bending. The strain in compression by bending increases nearer the column bottom. On the other hand, in the tensile, the strain increases linearly downward from the force-applied height. However, it decreased from the height of about 300 mm and became zero at the column bottom. The section is one where the plane section holding cannot be assumed, i.e., hinge section. The length of the section was nearly the same as the column depth D. Figure 18 shows shear strain intensity distribution in the first stage with coloured lines and symbols. The strain was measured by the rosette gauge in Figure 9(b). Positions of the gage were at range where the assumption of the plane section holding can be assumed. Distributions of shear strain calculated assuming the plane section is also shown in Figure 18 as light black dotted and single-dotted lines. The dotted line indicates distribution ignoring the rebar (Cal.1) and the single-dotted line indicates the distribution considering the rebar (Cal.2). The calculated values in Cal.2 were made to be correspond with the experimental values of the middle of the column depth at 0.5×10^-2 rad. and 2.0×10^-2 rad. The shape of the distribution considering the rebar was more close to the experimental distribution shape than that of ignoring rebars. The calculation distribution suggests the fact that shear strain and shear stress can be estimated based on the assumption of the plane-holding.
5 EVALUATION OF SKELETON CURVE OF HORIZONTAL FORCE-DISPLACEMENT RELATIONSHIP

5.1 EVALUATION MODEL AND ASSUMED MATERIAL PROPERTIES

The hybrid timber column with yielding at the column bottom by bending shows a suitable behavior to estimate its deformation as mentioned in Section 4.5. Figure 19 shows a model to estimate that deformation, which is modelled on the basis of the bending strain distribution. The length of the hinge section ‘Lp’ is assumed to be one of column depth D; the hinge section is divided into two halves; the lower half portion resists in elasto-plastic against the moment, with only the rebar; the upper half portion is assumed to resist the bending in only elastic as average of the equivalent bending stiffness ‘EJs’ of the hybrid timber and the bending stiffness ‘EJ’ of the rebar in the lower part.

In addition, portion other than the hinge is assumed to resist in elastic on the basis of the plane section holding. Shear deformation component is assumed to be uniform in elastic over full length of the column. The column base is assumed to be completely fixed. Young’s modulus and strength of rebar and timber were taken as material test values. The effect of the weakness due to direction of glulam timber lamina assembly was ignored. For timber, Young’s modulus by bending was used. Shear deformation was calculated, assumed to be resisted only by glulam timber, ignoring rebar. The shape factor for shear deformation was chosen to 1.2. Shear elastic modulus was assumed to be 643N/mm², obtained from relationship between the shear strain and shear stress described above. The bending yield moment was assumed to be one obtained from Equation (1) considering multi-array rebar. Assuming that there is a uniform stress block in bending compressive zone, stress ‘σm’ of the middle rebar was calculated as 50% of yield strength. For ultimate bending moment, σm was chosen as the yield strength. Two-array rebar at the outer portion and the mid-depth portion of the column, were consolidated into a single bar at the center of gravity of cross-sectional area of the rebars each.

\[ cM_y = a_t \cdot \sigma_y \cdot j + 0.5 \cdot N_m \cdot D \cdot \left(1 - \frac{N_m}{F_{wc} \cdot B \cdot D}\right) \]  

(1)

where, \(N_m = N + a_m \cdot \sigma_y, 0 \leq N_m \leq 0.5 \cdot F_{wc} \cdot B \cdot D\)

\(a_t = \) Gross cross-sectional area of the outermost two-array rebar, \(\sigma_y = \) Yield strength of rebar, \(j = \) Distance between stress centers of both outermost two-array rebars, \(N = \) Axial force of column, \(F_{wc} = \) Compressive strength of wood, \(B = \) Column width, \(D = \) Column depth, \(a_m = \) Gross cross-sectional area of rebars in mid-depth of column. After yielding by bending, the middle rebars were assumed to still remain elastic. Bending stiffness of the lower section of the hinge was assumed to be one calculated as 0.3% of the Young’s modulus of rebar to account for strain hardening of the yielded outmost rebars. And then, strain hardening was also considered for the middle rebar after the hinge reached the ultimate bending moment.

5.2 COMPARISON OF CALCULATED AND EXPERIMENTAL RESULTS

Figure 20 shows comparisons of calculated skeleton curve (red broken line) to the experimental history loops (gray history loop). Symbol ○ is one in when the column base reaches the bending yield moment, and symbol ● is one in when it reaches the ultimate bending moment. The loops of the experiment is for an axial force ratio \(\eta\) of 10%; its loading was applied up to a large deformation range of 7.26×10⁻² radian. The skeleton curve by red line roughly estimated the curve of the experiment.

The calculated value of shear capacity ‘cQsu’ is shown in the Figure 20 as a pink horizontal single-dotted line. No. 2 (R10) exceeded the shear capacity but did not fracture in shear. In Figure 20, the relationship between shear force and elastic shear deformation in blue is shown. For No.1, the shear deformation at the time of the yielding accounted for 28% of the calculated deformation; for No.2, 36%. The amount of shear deformation component of the hybrid timber column is larger than that of conventional glulam timber column because the bending stiffness is extremely increased by the rebar. Therefore, if...
shear yielding does not occur even after the yielding, the shear deformation returns to be zero by unloading to zero. As a result, the residual deformation is extremely reduced for the hybrid timber column, as described in Section 4.4. The skeleton curve of a conventional glulam timber column with completely fixed column base is shown by the yellow broken line of which calculation factors other than rebar were assumed to be the same as in the test specimen. The horizontal stiffness of No.1 (R10) was 33% higher than that of the glulam timber column; No.2 was 22% higher. The horizontal loading capacity of No.1 (R10) was 78% higher than that of the glulam timber column; No.2 was 88% higher. It can be confirmed that the hybrid column exhibits extremely high horizontal stiffness and bending capacity.

6 CONCLUSION
Horizontal loading tests were conducted on square cross-sectional columns whose bottom is planned to yield by bending, assuming the first floor of a mid-rise building consisting of rigid connected frame using HGTSB, and the elasto-plastic behavior of the column was clarified. The results are summarized below.

1) The horizontal stiffness and the bending capacity of the column was higher than those of conventional glulam timber column with its same cross section assuming that column bottom was fixed end, and the increase ratios were at range of 122-133% and 178-188%, respectively. Extremely high horizontal stiffness and bending capacity were demonstrated of the column.

2) When the axial force ratio of column was less than 0.25, the columns were hardly damaged even after subjected 5 times, which were cyclic loading from 0.5×10⁻⁴ rad. to 2.0×10⁻⁴ rad. After joint rebar of the column bottom yielded, its loops were shaped to exhibit abundant energy dissipation. The only degrading property was the reduction in bending moment of the proportional limit, due to the Bauschinger effect that occurs after the rebar yield.

3) After 2.0×10⁻¹ radian on, the rectangular section column in Literature 5 failed in shear, while the present specimens did not fail in shear but finally those wood of compressive side by bending failed in compression. However, decrease of those bending capacity was small, and the bending capacity was stable up to 7.26×10⁻⁴ rad. One of the specimens exceeded a calculated shear capacity, but there was no sign of shear failure.

4) In the case of HGTSB column, the shear deformation component accounted for a larger proportion of the total deformation of the member, and the residual deformation after yielding in bending may be extremely reduced. It was confirmed that if the ratio of restoring moment due to axial force, γ, is 0.1 or more, the residual deformation angle of the column due to the force-applied to be free vibration after the maximum response could be reduced below the visual allowable limit of the residual displacement angle (0.25×10⁻¹ rad.) even if the column was subjected to several times of loadings up to 2.0×10⁻² radian.

5) The skeleton curve of horizontal force-displacement relationship of the column was evaluated accurately by the model proposed in this paper.

6) After 2.0×10⁻² rad., one carbon fiber sleeve for joining of the joint rebar and rebar within the column fractured. In order to prevent the fracture, it is necessary to increase cover thickness of outside wood over the carbon fiber sleeve.

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