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STUDY ON JOINT DUCTILITY ASSURANCE DESIGN OF GLUED LAMINATED TIMBER FRAME WITH TENSILE BOLT TYPE JOINT

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ABSTRACT: This study proposed a concept on ductility assurance design of joints for a glued laminated timber frame using the upper 5% limit of tensile bolt unit stress when tensile bolt strain is 5%. In addition, the suitability of this concept for ductility assurance design of joints was verified by conducting a structural experiment on three tensile bolt type joints.

Key Words: Mid-rise timber building, Glued laminated timber frame, Tensile-bolt type joint, Ductility assurance design, Joint test

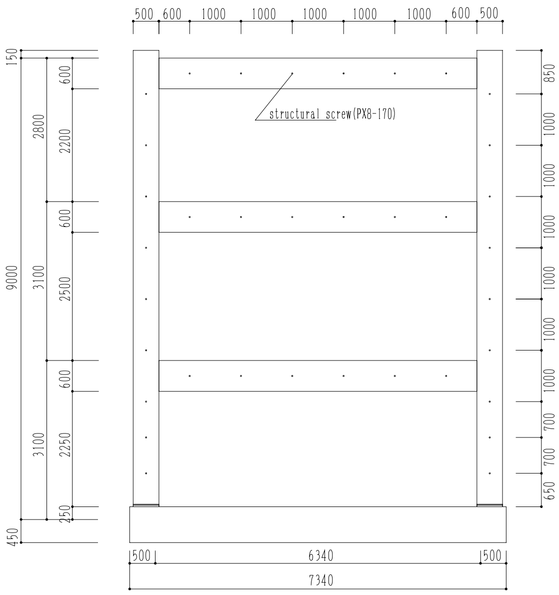
1. INTRODUCTION

For the purpose to promote and popularize mid-rise timber buildings, a three-story quasi-fireproof structure using glued laminated timber frames with tensile bolt type joints was created as a trial design, and the specification for joints and components that will satisfy the required fireproof and structural performances were studied. When lateral load-carrying capacity was studied in addition to allowable stress calculation, there was a need to estimate the collapse mechanism accordingly in order to compute the lateral load-carrying capacity. However, ductility assurance design of joints for a glued laminated timber frame has to date never been studied adequately. This study therefore will propose a concept on ductility assurance design of tensile bolt type joints. In addition, the suitability of this concept for ductility assurance design of joints will be verified by conducting a structural experiment on tensile bolt type joints.

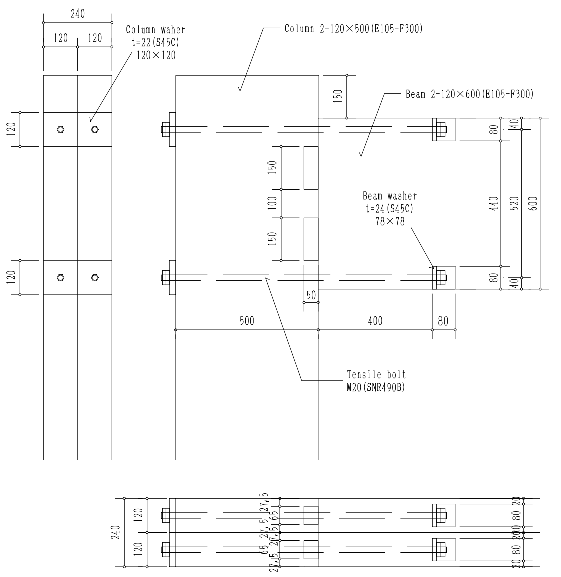
2. DUCTILITY ASSURANCE DESIGN FOR JOINT PERFORMANCES IN AN TRIAL DESIGNED BUILDING

2.1 Building summary

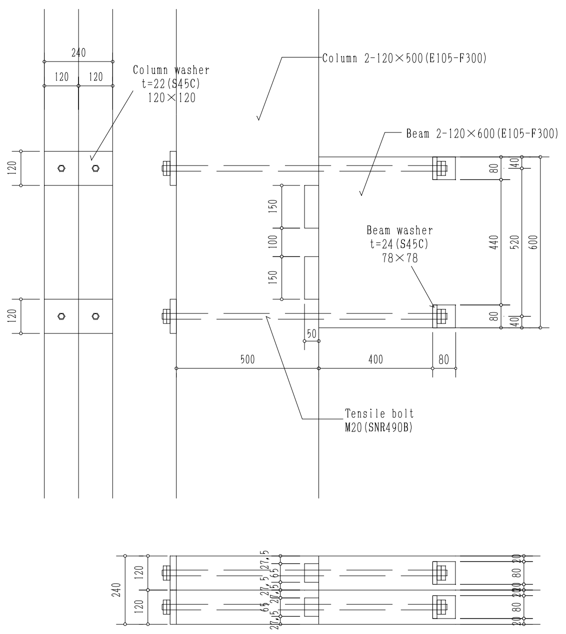
Fig. 1 illustrates the specification for each joint of each section and for the short side direction of a framing elevation of a three-story quasi-fireproof structure building made of glued laminated timber frame with tensile bolt type joints, which we created as a trial design. In terms of building scale,



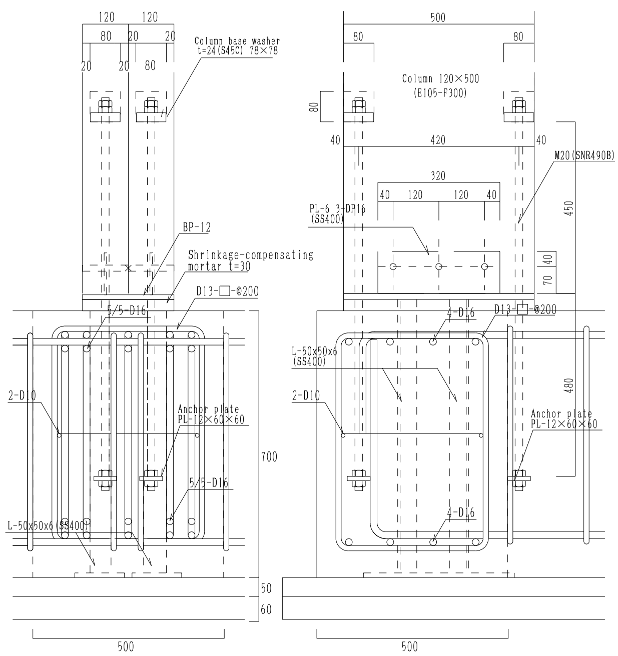
(1) Short side direction of a framing elevation



(2)Beam-column joint(L-shape) of 3rd floor



(3)Beam-column joint(T-shape) of 1st and 2nd floor



(4)Column base joint

Fig. 1 Trial design building for a three-story quasi-fireproofed structure standard floor plan is 7.2m x 14.4m, building height is 9m (floors 1~2: 3.1m; floor 3: 2.8m), and weight of each floor is approximately 500kN for floors 1 and 2 and approximately 380kN for floor 3.

Earthquake resistant elements on the short side direction is a frame where glued laminated timber frame with tensile bolt type joint that two materials are combined in the width direction (binding using structural screws (PX8-170) at 650mm ~ 1100mm intervals, avoiding panel zones), and the long side direction is a bearing wall structure of structural plywood. The footing beam has a width by height of 500 mm x 700 mm, and the building floor plan has a long side direction of 4-D16 for both the upper reinforcement bar and lower reinforcement bar, stirrup of D13-□-@200, and web reinforcement of 2-D10, and a building floor plan short side direction of 5-D16 for both the upper reinforcement bar and lower reinforcement bar, tie hoop of D13-□-@200, and web reinforcement of 2-D10, and the building floor plan center path of 5/5-D16 for both the upper reinforcement bar and lower reinforcement bar, tie hoop of D13-□-@200, and web reinforcement of 2-D10. Column base offers resistance to tensile force at the foundation anchor plate via anchor bolts, and offers resistance to shearing force at a L-shape angle (L50 x 50 x 6) that is welded to a base plate (See Fig. 1(4)).

2.2 Ductility assurance design for joints[1), 2)]

Assurance design is prepared to give ductility to tensile bolt type joints sections during failure mode. For example, to ensure ductility for a beam-to-column joints, out of the five anticipated failure modes ((1) tensile failure of tensile bolt; (2) shearing force failure from washer to beam's butt end; (3) wood yield due to compression in a beam washer's fiber direction (→ ultimately the failure mode of (2) has occurred)); (4) flexural failure of a column with possible defect in cross section; (5) flexural failure of a beam with possible defect in cross section), it must be guided towards (1). However, as embedding perpendicular to the grain in wood and column washer to a column face of a tensile bolt type joint has a high deformation performance, failure in this part was presumed to be not pre-existing and excluded. Equation (1) must be satisfied to guide the failure mode to (1).

$$M_{ut} \leq \min(M_{uk}, \quad M_{us}, \quad M_{uc}, \quad M_{ub}) \tag{1}$$

In which M_{ut}: Ultimate moment decided by tensile design strength of tensile bolt (=$A_t \cdot \sigma_u \cdot j$), A_t: Effective cross sectional area of bolt (mm^2), σ_u: Tensile strength of tensile bolt (N/mm^2), j: Distance between stress centers (mm) , M_{uk}: Moment decided by compression yielding strength in the wood fiber direction by a beam washer (=$N_{yb} \cdot j$), N_{yb}: Compression yielding strength in the wood fiber direction by a beam washer, M_{us}: Moment by shear failure from washer to beam's butt end (=$A_s \cdot F_s \cdot j$), F_s: Design shear strength of beam (N/mm^2), A_j: Area of shearing force from beam's seat boring surface to beam's butt end, M_{uc}: Flexural Failure moment of column (=$Z_c \cdot F_{bc}$), F_{bc}: Design bending strength of column (N/mm^2), Z_c: Cross sectional coefficient of column (mm^3), M_{ub}: Flexural Failure moment of beam (=$Z_b \cdot F_{bb}$), F_{bb}: Design bending strength of beam (N/mm^2) and Z_b: Cross sectional coefficient of beam (mm^3).

Relationship between the right and left sides of Equation (1) can be illustrated as shown by Fig. 2. The lower limit of 5% of a 75% confidence level assuming a normal distribution was used for the design bending strength of wood materials F_s, F_{bc}, and F_{bb} when M_{us}, M_{uc}, and M_{ub} were calculated. However, when JIS B 1220 ABR anchor bolts (hereinafter, ABR Anchor Bolts) were used as tensile bolts, the upper limit on the top side corresponds to a value of approximately 7σ for the average value according to statistical figures shown in Table 1[3)], when the average value of tensile strength of JIS G3138 (rolled round steel bar for building structure) B type SNR product (hereinafter, SNRB Product) is similar to the tensile strength of steel for type B building structure (hereinafter, SNB Product). Therefore, when the upper limit of tensile strength is used to calculate M_{ut}, this figure will be considerably on the safe side as compared with when design strength is used for strength of the wood material. Subsequently, an upper limit of 5% was also used for tensile bolts as was used for wood products. However, unit stress will also change by the deformation performance that is required by the

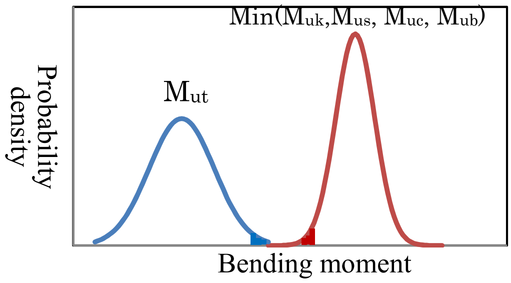


Fig. 2 Relationship between M_{ut} and (M_{us}, M_{uc}, M_{ub})

Table 1 Statistics of mechanical characteristics of SNB products
(Plate thickness: 9 mm $\leqq$ t $\leqq$ 22 mm)[3)]

		SN400B(Number of data：2187)			SN490B(Number of data：11302)		
		Nominal value /Upper limit value	Average	Standard deviation	Nominal value /Upper limit value	Average	Standard deviation
Yielding stress	σ_y (N/mm^2)	235/355	300	14.9	325/445	385	17.1
Tensile strength	σ_u (N/mm^2)	400/510	441	9.8	490/610	531	9.5

tensile bolt, when an upper limit of 5% is considered. The deformation performance of tensile bolts was subsequently studied, which is needed for a tensile bolt type frame in the frame shown in Fig. 3, the assumption is that bolt strain will be the largest when it is assumed that embedding in wood will be ignored because members are rigid and that there will be no destruction of splitting in the beam or column by bending deformation to the bolts. In this instance, the relationship between ultimate deformation angle θ of the frame and bolt strain ε is represented by the relationships of Equations (2) and Equations (3) (See Table 2 for the representation of letters in the equation).

$$\text{Column-beam joint}\quad \varepsilon = (d_b \cdot \theta_b) / (L_{1b}+D_c) \tag{2}$$
$$\text{Column-base joint}\quad \varepsilon = (d_b \cdot \theta_c) / (L_{1c}+ d_t) \tag{3}$$

In Equation (3), with reference to Literature[4)], bolt tightening length in the denominator was set to a length that includes fixing length (d_t). Table 2 is the results of calculating tensile bolt strain using equations (2) and (3) when ultimate deformation angle of the frame is 1/20 rad. Based on Fig. 1, distance between beam compression edge and tensile bolt "d_b" is 560 mm for the column-beam joint of the top floor and floors other than the top floor. In addition, fixing length of a tensile bolt for column-base d_t is 480 mm, and distance between column compression edge and tensile bolt distance "d_c" is 460 mm. Based on Table 2, deformation performance of about 5% strain for a tensile bolt is agreeable for a column-beam cross section of an expected range.

Table 3 indicates the upper 5% limit of tensile bolt unit stress when tensile bolt strain is 5% and 10%. For tensile unit stress when tensile bolt strain is 5%, tensile test results (3 sets of each)[5)] for

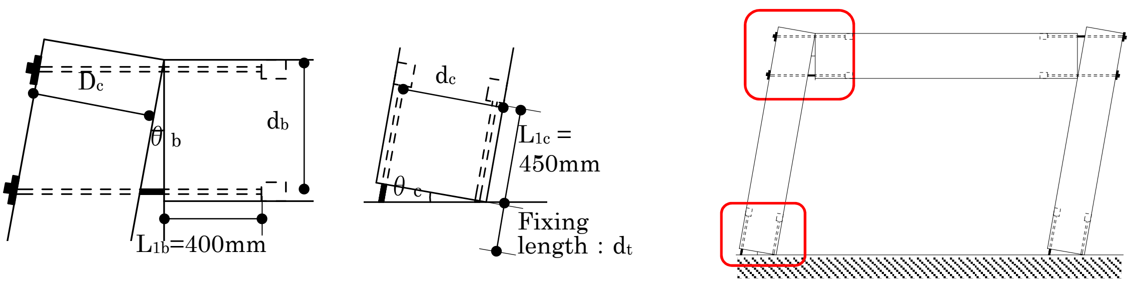


Fig. 3 Relationship of a tensile bolt strain to a deformation angle for the frame

Table 2 Relationship of a tensile bolt strain to a deformation angle for the frame

(1) Column-beam joint (distance from a beam's butt end to a seat boring position: L_{1b} = 400 mm)

Column depth D_c (mm)	Beam compression edge - tensile bolt distance d_b(mm)	Ultimate deformation angle θ_b (rad)	Tensile bolt tightening length $L=D_c + L_{1b}$ (mm)	Strain ε(%)
500	560	0.05	900	3.11

(2) Column-base joint (distance from a column's butt end to a seat boring position: L_{1c} = 450 mm)

Fixing length of tensile bolt d_t(mm)	Column compression edge – tensile bolt distance d_c(mm)	Ultimate deformation angle θ_c (rad)	Tensile bolt tightening length $L=d_t+ L_{1c}$ (mm)	Strain ε(%)
480	460	0.05	930	2.71

SNR490B with a diameter of 6 mm as shown in Fig. 4 was used. However, the diameter of the tensile bolt that is studied in this research is about 20 mm, and effect of bolt size is a future research topic. Furthermore, in view of safety, standard deviation for yield stress in Table 1 was used for standard deviation, and for calculation of the upper 5% limit, value of K = 3.152 (specimens: 3) in Literature[6] was used. For tensile unit stress when strain is 10%, because, based on Fig. 4, this is closer to the tensile strength, tensile strength and standard deviation in Table 1 were used. For calculation of the upper 5% limit, K = 1.66 (SN490B: specimens 11,302) in Literature[6] was used.

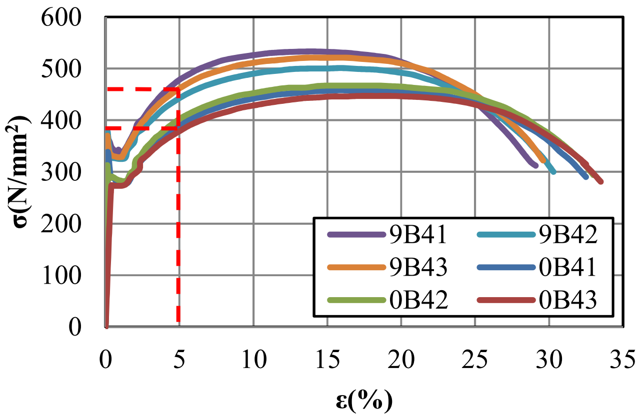


Fig. 4 σ- ε relationship of SNB products (9B:490B, 0B:400B)[2]

Table 3 5% upper limit value for tension and load levels of tensile bolts (SNRB product) (unit: N/mm^2)

		SNR400B			SNR490B		
		Average	Standard deviation	5% upper limit value	Average	Standard deviation	5% upper limit value
Stress intensity at 5% strain	$\sigma_{5\%\varepsilon}$	390	14.9	437(n=3)	461	17.1	515(n=3)
Stress intensity at 10% strain	$\sigma_{10\%\varepsilon}$	441	9.8	458(n=2187)	531	9.5	546(n=11302)

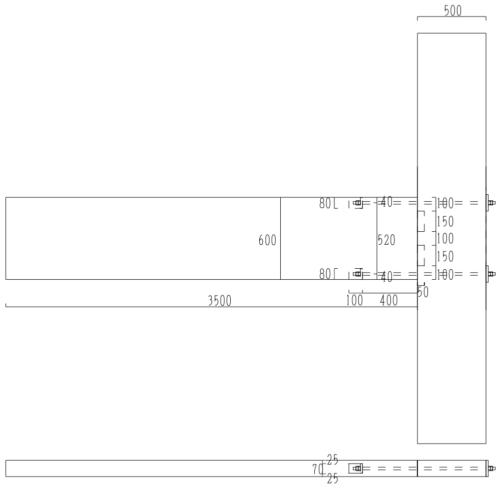
3. EXPERIMENT ON SUITABILITY VALIDATION OF A JOINT DUCTILITY ASSUARANCE DESIGN

3.1 Objective

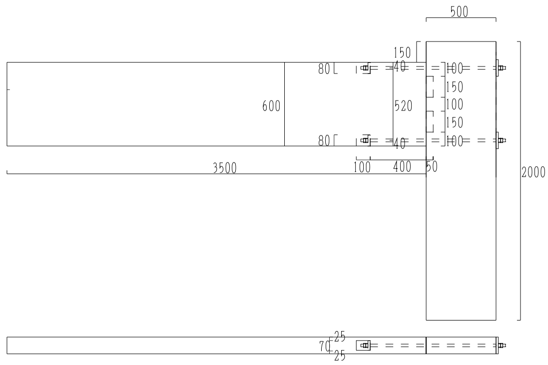
For tensile bolt type joint, a method of ductility assurance design for joints was proposed which uses the upper 5% limit of a tensile unit stress for tensile bolt strain (5% and 10% in the previous section), where an assumption was made on strain in line with the frame's ultimate deformation angle (this angle was 1/20 rad in the previous section). Suitability of this design will be verified by experiment.

3.2 Specimen specification

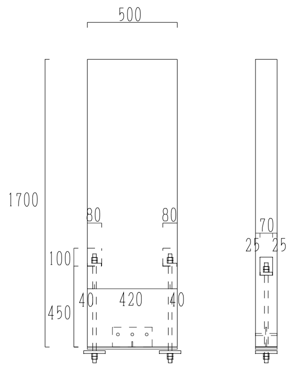
Specimen diagram is shown in Fig. 5 and the specification of each section is shown in Table 4. T-shaped and L-shaped specimens correspond to the column-beam joint of the common floors and top floor of a three story quasi-fireproof building structure, and I-shaped specimen corresponds to the column base of a three story quasi-fireproof building structure. The shear force between column and beam was transmitted by the short tenon of a beam's butt end in the T-shaped and L-shaped specimen and by the anchor bolt on the compression side in the I-shaped specimen. However, because the bolt length of a column-base joint anchor bolt is shorter than it was in the trial design, it is possible that the rotational rigidity by tensile bolt will be larger than what it was in the trial design; however, because ultimate moment is decided by effective cross sectional area of a bolt, a plan was made to verify that yield and shearing of a tensile bolt occur first. The number of specimens of each specification is one.



(1) Column-beam joint specimen（T-shaped）



(2) Column-beam joint specimen（L-shaped）



(3) Column-base joint specimen（I-shaped）

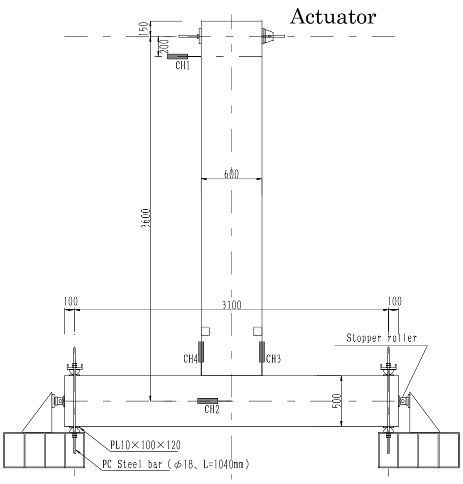
Fig. 5 Specimen diagram

3.3 Load control method

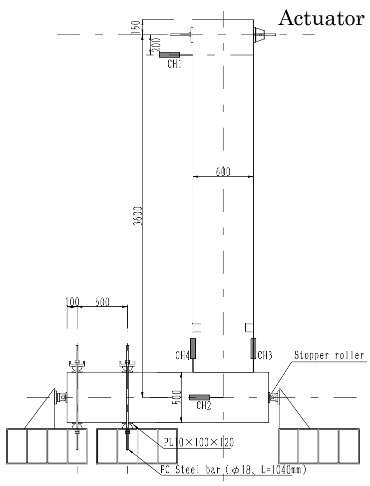
Specimen was fixed on a jig, and an actuator with 1000 mm stroke and ± 100 kN capacity was used to apply stress. Photo 1 shows an example of how to set up a loading device and specimen, and Fig. 6 shows a measurement position, stressor position, and how to fix each specimen. Column-beam specimen was fixed at two locations on the column side using pivoting pins. In addition, for the column-base specimen, a base plate was fixed onto the jig. Loading method has a displacement control, and stressing schedule was set to repeat the positive negative alternator three times at 1/450 rad ~ 1/30 rad. After repeated at 1/30 rad, load was applied in one direction to verify ultimate failure characteristics. Load was measured by a load meter onto which an actuator was attached, and rotational displacement of the joint was measured by a displacement meter. Furthermore, a strain gauge was put on the tensile bolt to measure strain in the tensile bolt.

Table 4　Specification of each section of specimen

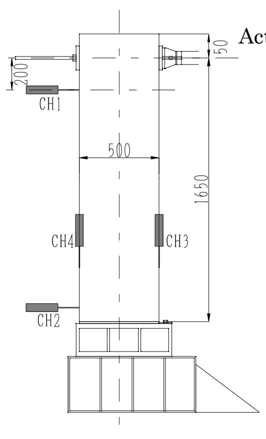
	Specification	Strength	Size
Column-beam joint specimen (T)	Japanese larch symmetric mixed-grade composition glulam	E105-F300	Column:120mm×500mm×3300mm Beam:120mm×600mm×3500mm
	Tensile bolt	SNR490B	M20, L=1020mm
	Beam washer（78×78）	S45C	t=24mm
	Column washer(120×120)	S45C	t=22mm
Column-beam joint specimen (L)	Japanese larch symmetric mixed-grade composition glulam	E105-F300	Column:120mm×500mm×2000mm Beam:120mm×600mm×3500mm
	Tensile bolt	SNR490B	M20, L=1020mm
	Beam washer（78×78）	S45C	t=24mm
	Column washer(120×120)	S45C	t=22mm
Column-base joint specimen (I)	Japanese larch symmetric mixed-grade composition glulam	E105-F300	Column:120mm×500mm×1800mm
	Tensile bolt(Anchor bolt)	SNR490B	M20, L=521mm
	Column washer(78×78)	S45C	t=24mm



(1) Column-beam joint specimen(T-shaped)



(2) Column-beam joint specimen(L-shaped)



(3) Column-base joint specimen(I-shaped)

Fig. 6 Specimen fixation method and measurement position

Table 5　Ductility assurance design for joints

	T-shaped	L-shaped	I-shaped
Distance from bolt shaft center to beam or column compression edge d_b or d_c (mm)	560	560	460
Distance from position of embedding resultant force by triangle displacement of beam or column butt end to compression-end X_a(mm)	21.83	58.95	1.86
Stress center distance $j = d_b - X_a$ (mm)	538.17	501.05	458.14
Design bearing strength in fiber direction of beam or column F_e (N/mm^2)	25.4	25.4	25.4
Depth of beam or column washer　X_b (mm)	78	78	78
Width of beam or column washer　Y_b (mm)	78	78	78
Compression yield resistance in fiber direction by fixed plate　$N_{yb} = X_b \cdot Y_b \cdot F_e$ (kN)	154.5	154.5	154.5
(4)Moment by compression yield resistance in fiber direction by fixed plate　: $M_{uk}=N_{yb} \cdot j$	83.17	77.43	71.08
Distance from beam or column butt end　L_1 (mm)	400	400	450
Design shear strength　F_s (N/mm^2)	3.6	3.6	3.6
Shear area from bolt fixing point to beam or column butt end　$A_s=(2X_b+Y_b)\times L_1/1.5$ (mm^2)	62,400	62,400	70,200
Shear fracture load from fixing plate to beam or column butt end　$N_{us} = A_s \cdot F_s$ (kN)	149.8	149.8	168.5
(5)Moment by shear fracture load : $M_{us}=N_{us} \cdot j$ (kNm)	80.6	75.0	77.5
Tensile bolt specification	SNR490B	SNR490B	SNR490B
Bolt nominal diameter	M20	M20	M20
Cross-section area of the bolt shaft A_t(mm^2)	260	260	260
Design tensile unit strength of tensile bolt (Nominal value) F_{tuL} (N/mm^2)	490	490	490
Tensile unit strength of tensile bolt (5% upper limit vale at 5% strain) F_{tuU} (N/mm^2)	515	515	515
Tensile strength of tensile bolt (5% upper limit value)　$T_{uU} = A_t \cdot F_{tuU}$　(kN)	133.8	133.8	133.8
(6)Flexural moment at tensile fracture of the bolt (5% upper limit value) $M_{utL}=T_{uU} \cdot j$	67.03	71.99	61.29
Column width　Y_c (mm)	120	120	-
Column depth　D_c (mm)	500	500	-
Diameter of bolt hole Φ(mm)	24	24	-
Design bending unit strength of column or beam F_{bc} or F_{bb} (N/mm^2)	25.4	25.4	25.4
Column section modulus considering section defect of bolt hole　$Z_c=(Y_c-\Phi) \cdot D_c^2/6$　(mm^3)	4.00×10^6	4.00×10^6	-
(7)Column flexural strength : $M_{cu}=Z_c \cdot F_b$　(kNm)	111.60	111.60	-
Beam or column depth D_b (D_c) (mm)	600	600	500
Seat boring hole width of beam or column Y_z (mm)	80	80	80
Seat boring hole depth of beam or column X_z (mm)	80	80	80
Beam or column width Y_p (mm)	120	120	120
Column or beam section modulus considering section defect of boring for washer. (Column : D= D_c, Beam : D= D_b) $Z_b=(Y_z \cdot (D-2X_z)^3+(Y_p-Y_z) \cdot D^3)/12 \cdot (D/2)$ (mm^3)	4.29×10^6	4.29×10^6	2.71×10^6
(8)Beam or column flexural strength : $M_{bu}=Z_b \cdot F_b$ (kNm)	119.77	119.77	75.74
(9)Ultimate moment at 5% upper limit tensile strength of tensile bolt : min((4),(5),(6),(7),(8))	67.90	63.22	58.04

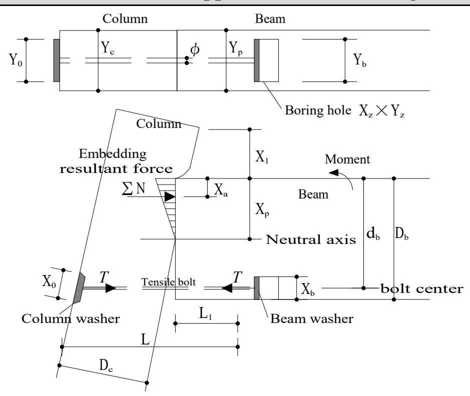


Fig. 7 Column-beam joint mechanical model[1)]

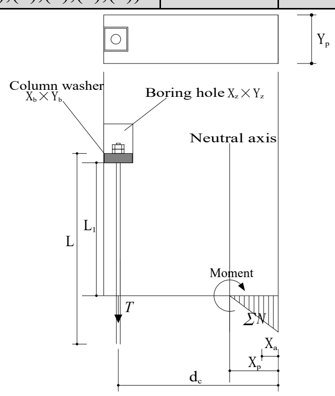


Fig. 8 Column-base joint mechanical model[2)]

3.4 Ductility assurance design for the joint[1)]

In line with the method that was proposed in section two, the results of a ductility assurance design for the joint are shown in Table 5. Some of the symbols in the table are as shown in Fig. 7 and Fig. 8. (9) in this table was set using member cross sections, seat boring positions, steel type, washer dimension, and others, to allow the tensile strength of a tensile bolt to be set at the upper 5% limit. However, "(5) Load at failure when shearing (from column or beam washer to column or beam's butt end)" was calculated by multiplying 1/1.5 to the shearing area that is calculated based on a seat boring position, with Literature[7)] as reference. Stress center distance was approximately 538 mm for the T-shaped specimen, approximately 501 mm for the L-shaped specimen, and approximately 458 mm for the column-base specimen, whereas distance d_b and d_c which are distances from compression edge to tension side tensile bolt center were 560 mm for the T-shaped and L-shaped specimens and 460 mm for the column-base specimen.

3.5 Experiment results

3.5.1 Failure characteristics

Photo 2 shows the failure characteristics for each specimen. For all types, tension failure of the tensile bolt was ultimate failure mode (Photo 2 (3) 2). Furthermore, it was verified that triangular embedding perpendicular to the column side of the beam's butt end and equal displacement embedding perpendicular to the column washer were in ultimate failure mode (Photos 2(1), (2)). In addition, it was verified that embedding perpendicular to the column side of a column washer and triangular embedding perpendicular to the column side of a beam's butt end are in ductility failure mode.

3.5.2 Relationship between moment and rotational angle

Fig. 9 shows a relationship between moment and rotational angle based on calculated values and experimental results. Moment was calculated based on a value that is the product of load measurement using a load cell and loading height which is shown in Fig. 6 (T-shaped and L-shaped specimens: 3,600 mm; column-base specimen: 1,650 mm), and rotational angle was calculated by a displacement meter that was attached to both ends (distance 500 mm) of the column of a column-base specimen and to both ends (distance 600 mm) of the beam of a T-shaped and L-shaped specimens.

Calculated and indicated values are calc1 which is when the Young modulus in the fiber's orthogonal direction is set to 1/50 of the fiber's direction, and calc2 which is when this figure is doubled and set to 1/25, both of which are proposed in Literature1). Because in the column-base specimen there is no embedding perpendicular to the fiber's orthogonal direction, there is only one calculated value. Further, because the bolts used are JIS compliant products, maximum moment for the calculated value was calculated as F value of a tensile bolt (325 N/mm2) x 1.1, in accordance with Construction Ministry Notice No 2464 No 3.1 of 2000. When the elasticity and rigidity of experimental values and calculated values were compared, calc2 was believed to approximate the elasticity and rigidity of experimental values better. Literature1) proposes "when the Young modulus in the fiber's orthogonal direction is set to 1/50 of the fiber's direction"; this is a figure when the Young modulus in the lumber fiber's orthogonal direction is evaluated on the safer side. However, for glued laminated timber such as what is used in this research, when the Young modulus in the fiber's orthogonal direction is set to approximately 1/25 of the fiber's direction, results suggested that the experimental results were approximated better.

3.5.3 Relationship between rotational angle and volume of bolt expansion

Fig. 10 shows the relationship between a joint's rotational angle and a tensile bolt strain. A tensile bolt strain was determined by calculating the amount of a bolt deformation as a product of stress center distance and the joint rotational angle, for the M-θ curve above, and dividing the bolt tightening length. In addition, deformations that were measured by a strain gauge (Tokyo Measurement FLA-2) were also shown in the figure. From this figure, based on data from the strain gauge, it can be verified that the joint's rotational angle was approximately 1/30 rad when the tensile bolt yielded. Furthermore, the volume of a bolt's expansion when the joint's rotational angle is 1/20 (= 0.05) rad was 2.5~3%; even when the joint's rotational angle was 1/15 (= 0.067) rad, it is still at about 3.2~5%. Based on this, when

1) Column washer embedding in the column 2) Beam’s butt end embedding in the column

(1) Column-beam joint specimen(T-shaped)

1) Beam’s butt end embedding in the column 2) Column washer embedding in the column

(2) Column-beam joint specimen(L-shaped)

1) Tension side anchor-bolt elongation 2) Tension side anchor-bolt elongation and rupture

(3) Column-base joint specimen(I-shaped)

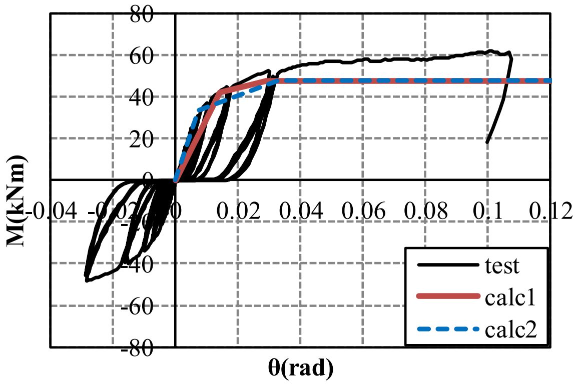
Photo 2 Failure characteristics

the ultimate deformation angle is 1/20 rad, the experiment confirmed the suitability of using “5% upper limit value of load when bolt strain is 5%” for the ductility assurance design of a joint.

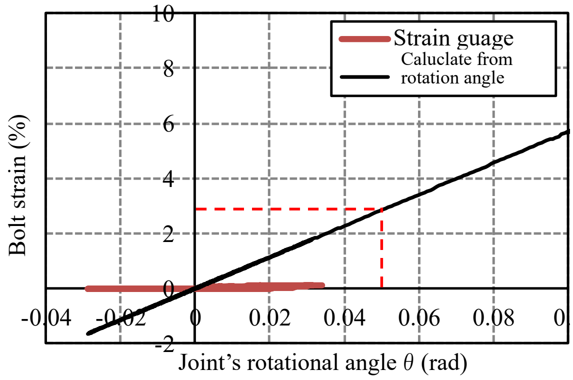
4. CONCLUSION

In order to estimate the collapse mechanism for lateral load-carrying capacity of mid-rise timber buildings accordingly, a ductility assurance design of tensile bolt type joints is proposed. From the study, it can be concluded that:

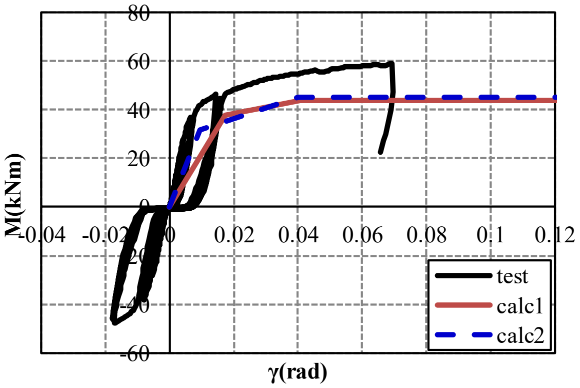
1) For the tensile bolt type joint, a method to control the collapse mechanism by tensile failure of the tensile bolt to guarantee the ductiliy of the joint is proposed. In addition, a method to use the upper limit 5% of tensile unit stress corresponding to the assumed limit deformation angle of the frame



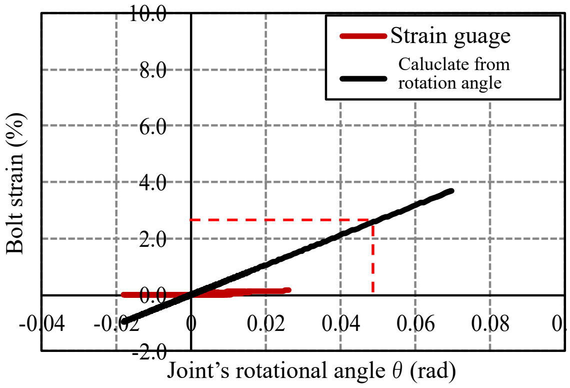
(1) Column-beam joint specimen(L-shaped)



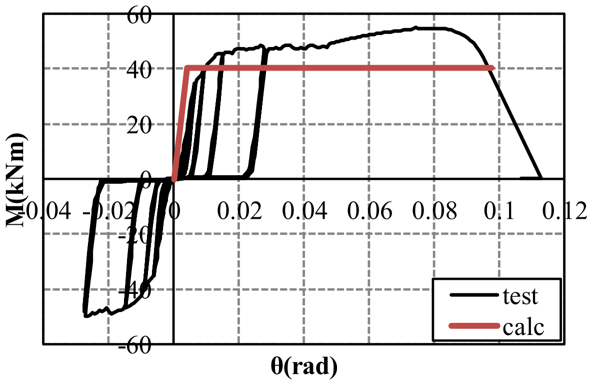
(1) Column-beam joint specimen(L-shaped)



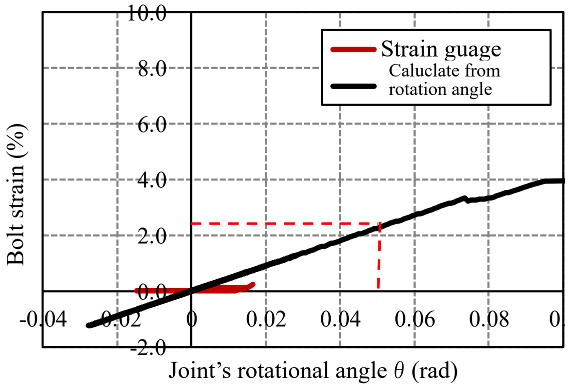
(2) Column-beam joint specimen(L-shaped)



(2) Column-beam joint specimen(L-shaped)



(3) Column-beam joint specimen(I-shaped)



(3) Column-beam joint specimen(I-shaped)

Fig. 9 Relationship between moment and rotational angle

Fig. 10 Relationship between joint's rotational angle and bolt strain

for the ductiliy assurance design of the tensile bolt joint is proposed.

2) Joint is designed using the upper limit 5% of tensile unit stress at 5% strain of the bolt assuming the limit deformation angle to be 1/20 rad, and the suitability of the proposed ductility assurance design of joints was verified by the experiment on three tensile bolt type joints.
3) As a result of experiments on T-shaped, L-shaped and I-shaped joint, load-deformation relationship can be controled by the proposed method. In addition, it was also confirmed that the tensile bolt strain is 5% or less when the joint rotation angle is 1/20 rad. From these points, it was confirmed that designing the joint using the upper limit 5% tensile bolt unit stress at 5% strain is effective when the limit deformation angle of the frame is 1/20 rad.

It should be noted that the diameter of the tensile bolt that is studied in this research is about

20 mm, and effect of bolt size is a future research topic.

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