Third International Conference on Sustainable Construction Materials and Technologies http://www.claisse.info/Proceedings.htm

# A Green New Deal Building System with Prestressed Glue-Laminated Timber Slabs

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## ABSTRACT

Prestressed glue laminated timber T-beams were experimentally studied to make super light timber slabs so that they replace concrete slabs ordinarily used for many types of concrete or steel buildings. The proposed timber slab system leads to light-weight buildings which have tremendous advantages under seismic events. By employing prestressing technology, it is possible to build long span timber slabs which can be used for large scale office or residential high-rise buildings. If timber slabs replace concrete slabs, good impacts on the environment are also expected by reducing the use of cement and steel product. The paper describes mechanical properties of the proposed T-beams for monotonic loading test.

**Keywords.** Glue laminated timber T-beam, timber slab system, prestressing, specific strength

## INTRODUCTION

It is necessary to make high-rise buildings in major cities when a large number of people live in a limited space. High-rise buildings usually have concrete slabs regardless of their structural system since concrete slabs have high stiffness and good control over vibration and sound disorder. However, concrete slabs increase the weight of buildings resulting in the larger seismic lateral force, and adverse effects are expected under seismic events. In addition, the production process of portland cement creates large amount of carbon dioxide.

The authors proposed a prestressed glue laminated timber slab system which is much lighter but equally functional to ordinary concrete slabs. Applying prestressing force to glue laminated timbers was first proposed by Bohannan in 1962 (Bohannan 1962). In Japan, Hasebe et al. (Hasebe et al. 2002) reported the experimental study on bending behavior of prestressed laminated timber beams in 2002. However, the application of prestressing to T-beams made of glue laminated timbers for slab usage has not been conducted. Structural performance and fire protection performance need to be upgraded for the practical use in large scale multi-story office and residential buildings. As the first step, this paper describes the structural performance of prestressed glue laminated timber T-beams under vertical monotonic loading. Measures for fire protection are explained elsewhere.

Advantages of using prestressed glue laminated timber T-section as a slab system is summarized.

- Structural advantages:
  - Weight of buildings decrease dramatically if concrete slabs are replaced by timber slabs. As a result, the seismic performance is greatly enhanced.
  - The specific strength of timbers is high and long span members can be made. Combined with prestressing technology, the long span members are easier to build.
  - Timbers fail in brittle manner when flexural failure is triggered by the fracture of tension fibers. Prestressing tendons, however, sustain the tensile force and prevent brittle failure.
  - If the proposed slab system is proved to have high fire protection capability, the slab system can be widely used in low - to high-rise buildings.
- Environmental advantages:
  - It decreases the use of concrete and steel products.
  - Timbers are adiabatically effective and the use of air-conditioning can be decreased.

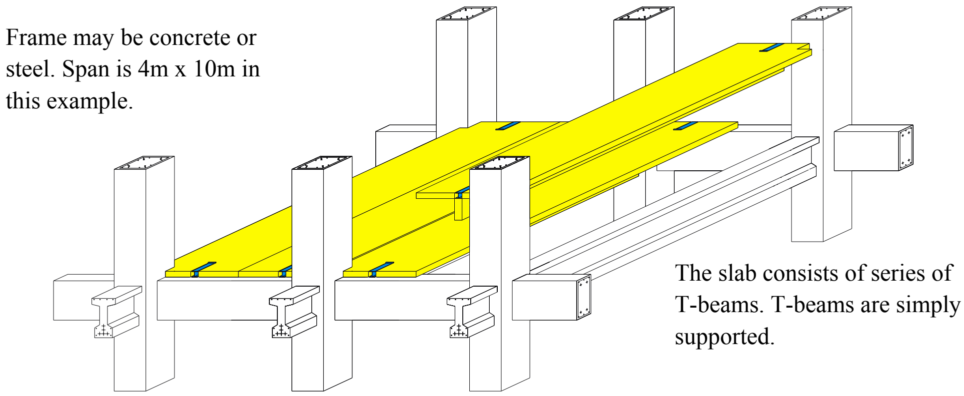



Figure 1 Proposed slab system with unbonded prestressed glue laminated timber T-beams placed on a concrete frame

A new timber slab system was developed to reduce the self weight of slab system and it can be used in concrete or steel high-rise buildings. The paper describes the mechanical performance of the proposed prestressed glue laminated timber T-beam slab system.

## EXPERIMENTAL WORK

**Setup.** The proposed timber slab system consists of series of unbonded prestressed glue laminated timber (prestressed GLT hereafter) T-beams. Since these T-beams are simply supported, vertical load due to dead and live loads acts but no additional load is expected under seismic event except the vertical inertia force. Hence, prestressed GLT T-beams with different section properties were tested to see flexural and shear behavior under monotonic vertical loads.

Specimens were designed by checking stress level and deflection. The stress level at external fiber does not exceed the material tensile strength, $F_b$, at two cases: the introduction of prestressing and the action of maximum live load. The deflection does not exceed 1/1000 of the supported span length at the introduction of prestressing, and does not exceed 1/750 of the supported span length at the action of maximum live load. The shear capacity was made higher than the flexural capacity for flexure dominant specimens and vice versa for shear dominant specimens.

Specimens with test variables are shown in Table 1. Main variables were section configurations (Figure 2) , shear span (3.5m and 1.5m), and PS bar size ($\phi$13, $\phi$19, $\phi$21). The variables were combined to observe flexure or shear failure modes. Specimens were made of E95-F315 (JAS Notifications 111 and 235)(Japan Agricultural Standards, 2003) and prestressing steel bars were Type B SBPR 95/110 (JIS G 3109)(Japanese Industrial Standards, 2008). The diameter of holes for PS bars was five mille meters larger than that of PS bars. The hole was straight and not grouted. About 75% of yield force was introduced to PS bars as shown in Table 1. A set of anchorage elements used ordinary washers and nuts for concrete members. The size of anchor plates, however, was increased to 150mmx350mm in order to prevent splitting of timber due to the bearing force. No damage was observed at the anchorage regions during the test.

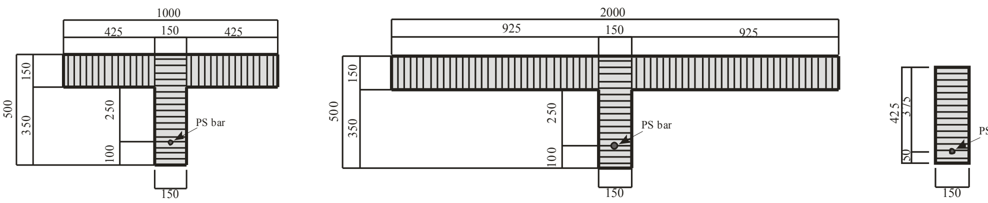
Specimens were produced at a medium-size timber mill factory in Japan so that similar products may be reproduced in future. Two flange elements and web element were made first by pressing with resorcinol resin adhesive and three elements were assembled by pressing again with the same resin. This production process made laminar directions of flange and web perpendicular to each other as shown in Figure 2, but any abnormality due to this was not observed in the test.

A loading system is shown in Figure 3. The specimens were simply supported and loaded monotonically with equal vertical loads through two transverse loading beams. The twisting rotation of specimen was controlled by applying the identical vertical displacements with two hydraulic jacks on both sides. Loading was basically terminated right after the peak load dropped (R10-5-B13-M, R10-5-B19-M, T10-5-B13-M, T10-5-B19-M, T20-5-B21-M) but the additional deflection was enforced to other specimens (T10-5-B13-F, T10-5-B13-S, T10-5-B19-S, T20-5-B21-S) to see the post-failure ductile behavior.

Table 1 Test variables

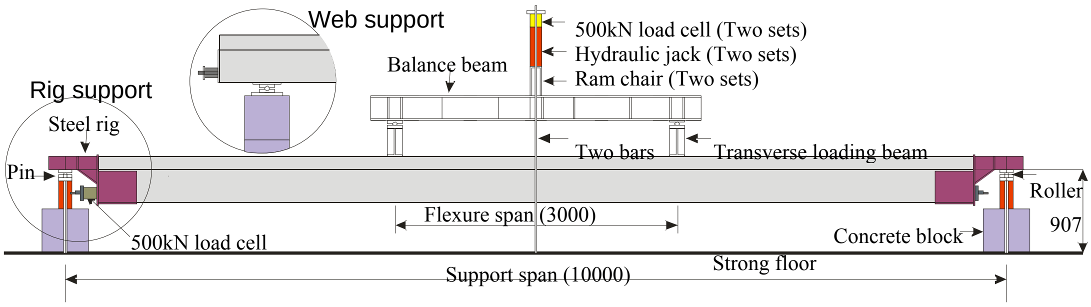
| Specimen | Section shape | Support span/ Flexure span(m) | Shear span (m) | End support | Section geometry (mm) | PS bar diameter (mm) | Prestressing force Npsi in kN (Npsi/Ny) |
|---|---|---|---|---|---|---|---|
| R10-5-B13-M | Rectangle | 10/3 | 3.5 | Web | 150×425 | 13 | 89 (0.72) |
| R10-5-B19-M | | | | | | 19 | 224 (0.85) |
| T10-5-B13-M | T-shape | | | | 1000×500×150×150 | 13 | 89 (0.72) |
| T10-5-B13-F | | | | Steel rig | | 13 | |
| T10-5-B19-M | | | | Web | | 19 | 190 (0.72) |
| T20-5-B21-M | | | | | 2000×500×150×150 | 21 | 249 (0.77) |
| T10-5-B13-S | T-shape | 4/1 | 1.5 | Steel rig | 1000×500×150×150 | 13 | 89 (0.72) |
| T10-5-B19-S | | | | | | 19 | 190 (0.72) |
| T20-5-B21-S | | | | | 2000×500×150×150 | 21 | 249 (0.77) |

Type of glue laminated timber: E95-F315, Prestressing steel bars: Type B SBPR 95/110 (JIS G 3109)

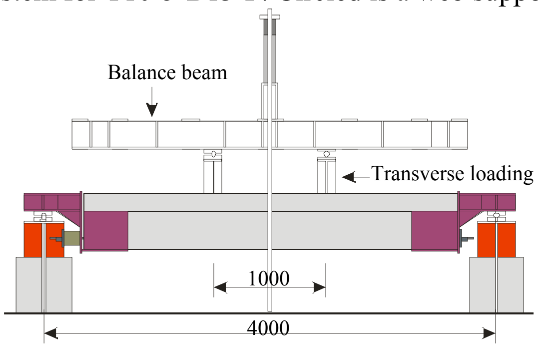



(a) 1000mm wide T-beam (b) 2000mm wide T-beam (c) Rectangular beam

Figure 2 Section configuration with laminar directions (Unit: mm)




(a) Loading system for T10-5-B13-F. Circled is a web support. (Unit: mm)

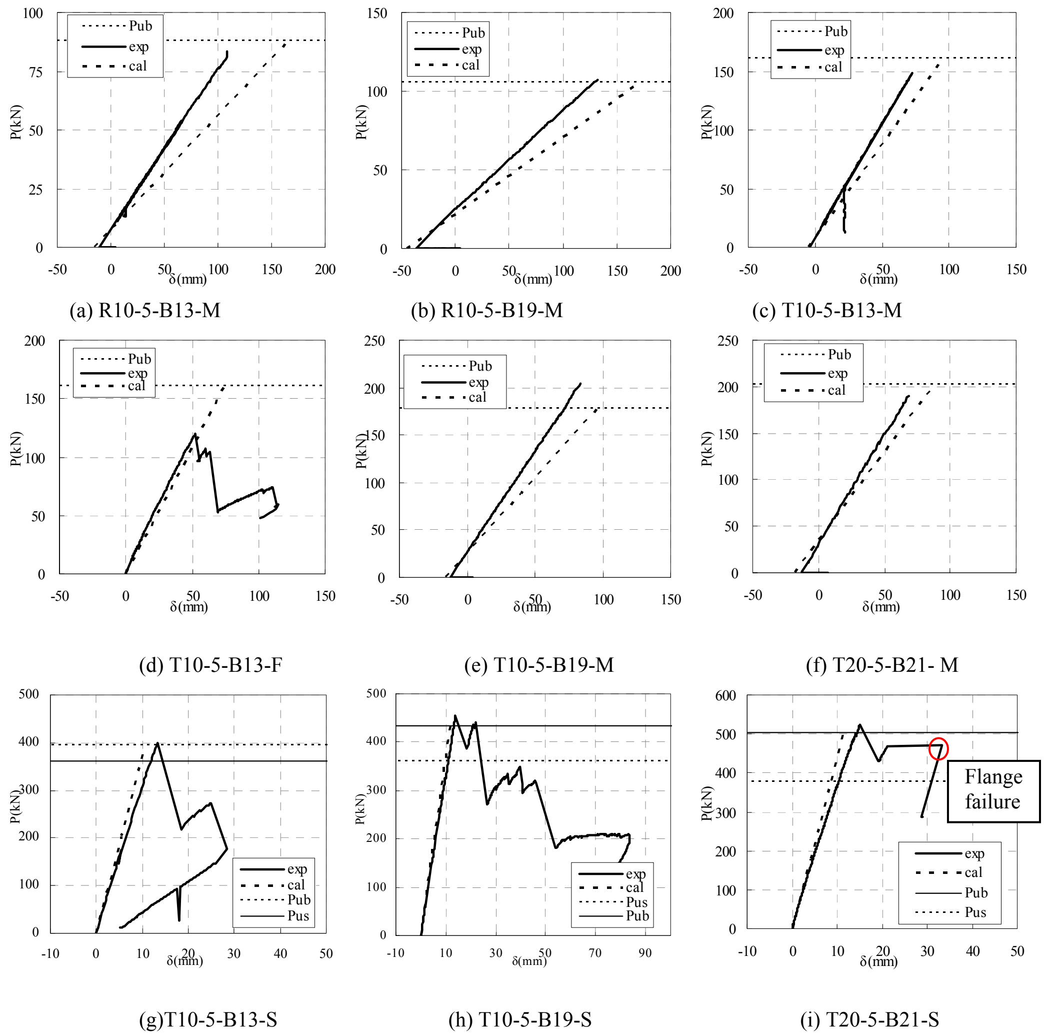



(b) Loading system for T10-5-B13-S.

(b) Overview of loading system

Figure 3 Loading setup

**Test results.** Figure 4 shows vertical load (P) - deflection (δ) relations. They are linear up to the peak load and load drop was sudden after the peak. As explained in the previous section, some specimens were loaded further to see the post failure behavior. Ordinary timber normally lose the load carrying capacity completely after the failure but prestressed T-beam specimens still had at least 50% of load carrying capacity after the first peak as Figure 4(d) or (g). Tendon bars prevented complete failure by carrying part of the tensile force which was carried by the failed tension laminae. For further deflection, specimens regained the load carrying capacity until the more tension laminas failed. As long as the tendon bar remains elastic, specimens with flexural failure did not collapse completely. T10-5-B19-S failed in shear at the web but also showed relatively ductile behavior at the post-peak region. On the other hand, T20-5-B21-S reached the peak load due to the splitting crack at the flange base. After some 20% loss of the capacity, the deflection increased with constant load. One side of the flanges suddenly bent down from the base with sudden drop of load. This is probably caused since the bending stiffness of the loading steel beam was not enough and the free end of flange was pushed down as a point load is acting. Hence T20-5-B21-S is eliminated from the following discussions.

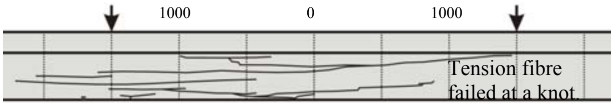



(a) R10-5-B13-M (b) R10-5-B19-M (c) T10-5-B13-M

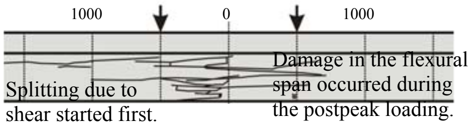
(d) T10-5-B13-F (e) T10-5-B19-M (f) T20-5-B21- M

(g)T10-5-B13-S (h) T10-5-B19-S (i) T20-5-B21-S

Figure 4 Vertical load (P) - deflection (δ) relations




(a) Flexural failure (T10-5-B13-F)




(b) Shear failure (T10-5-B19-S)

(c) Flange bent down and failed (T20-5-B21-S)

Figure 5 Elevated views of typical failure modes

Table 2 Test results

| Specimen | Failure mode (Starting point) | Max. Load Pe | Pfc (Pe/Pfc) | Psc (Pe/Psc) |
|---|---|---|---|---|
| R10-4-B13-M | Flexure(FJ) | 84 | 86 (0.98) | 335(-) |
| R10-4-B19-M | Flexure(Knot) | 107 | 104 (1.03) | 335(-) |
| T10-5-B13-M | Flexure(Knot) | 148 | 153 (0.97) | 361(-) |
| T10-5-B13-F | Flexure(Knot) | 119 | 153 (0.78) | 361(-) |
| T10-5-B19-M | Flexure(FJ) | 204 | 171 (1.20) | 361(-) |
| T20-5-B21-M | Flexure(Knot) | 190 | 186 (1.02) | 376(-) |
| T10-5-B13-S | Flexure(FJ) | 399 | 388 (1.03) | 361(1.11) |
| T10-5-B19-S | Shear (Web) | 456 | 429 (-) | 361 (1.26) |
| T20-5-B21-S | Shear (Flange) | 524 | 492 (-) | 376 (1.28) |

Starting point: FJ (Finger joint of laminar), Knot (Knot of laminar), Web (Delamination of web), Flange (Whole flange bent down), Pe, maximum vertical load, Pfc: computed load for flexural strength, Psc: computed load for shear strength

Test results are summarized in Table 2. Specimens with 10m span failed in flexure and the failures were triggered by knot or finger joint failure of the first or second lamina. Fracture of fibers and delamination propagated from the bottom of web to the upper region in the flexural span web as shown in Figure 5(a). On the other hand, specimens with 4m span failed in different manners. T10-5-B13-S failed in flexure unexpectedly, T10-5-B19-S in shear as designed, and T20-5-B21-S at flange base unexpectedly. The failure of T10-5-B19-S started at the web of the left shear span in Figure 5(b) and the load dropped to 85% of the peak load. Fracture of laminae and delamination spread from the bottom of the flexural span web after the peak when further deflection was enforced.

## NUMERICAL SIMULATION

**Procedures.** Vertical load - deflection relations were simulated with an elastic analysis. Young's modulus (9500MPa) and the tensile strength (31.5MPa) of E95-F315 specified in the Japanese Agricultural Standards (Japanese Agricultural Standards, 2003) were used for the analysis. Moment is considered to reach the flexural capacity, $M_u$, when the tensile fiber reaches the strength of 31.5MPa as Eq. (1). Full width of flange was assumed effective for the moment capacity. The stress increase in tendon was neglected in this paper although detailed study was conducted.

$$M_u = \left( \frac{N_P}{A} + \frac{M_P}{Z_b} + \frac{M_d}{Z_b} + K_Z \cdot F_b \right) \cdot Z_b \tag{1}$$

where $N_p$ is the prestressing force, $M_p$ moment due to $N_p$, $M_d$ moment due to dead load, $A$ section area, $Z_b$ section modulus for the area under the neutral axis, $F_b$ tensile strength (31.5MPa). $K_z$ is the coefficient to take into account the size effect for flexure and expressed as:

$$K_z = \left( \frac{h_0}{h} \right)^k \tag{2}$$

where $h$ is the total depth (500mm or 425mm), $h_0$ standard depth (300mm is used here), and $k$ constant (1/9). The vertical load capacity, $P_{fc}$, was computed by dividing $M_u$ by the shear span length.

The deflection for the flexural capacity, $\delta_u$, is computed as follows.

$$\delta_u = \left( \frac{L_s \cdot \left(3 \cdot L^2 - 4 \cdot L_s{}^2\right)}{24 \cdot E_t \cdot I} + \frac{k_s \cdot L_s}{G \cdot A} \right) \frac{M_u}{L_s} + \delta_{dead} + \delta_{PS} \tag{3}$$

where $L_s$ is the shear span, $L$ the total span, $E_t$ Young's modulus (9500MPa), $I$ moment of inertia, $k_s$ shape coefficient, $G$ shear modulus (633MPa=$E_t$/15), $A_t$ section area. Deflection due to dead load, $\delta_{dead}$, and deflection due to prestressing, $\delta_{PS}$, can be computed from the fundamental elastic analysis.

Shear strength of beams, $V_u$, was computed from Eq. (4).

$$V_u = \frac{4 \cdot I \cdot F_s}{\left\{ (D - y_0)^2 - (y_0 - t)^2 \right\}} \tag{4}$$

where $I$ is the moment of inertia, $F_s$ the specified shear strength in JAS (Japanese Agricultural Standards, 2003), $D$ the overall depth, $y_o$ distance between the compression fiber to the neutral axis, $t$ thickness of the flange. In this equation, the neutral axis was assumed to stay in the flange, that is, $y_o$ is larger than $t$. The vertical load capacity, $P_{sc}$, was computed by multiplying 2 to $V_u$.

**Discussions.** In Figure 4, computed and experimental vertical load - deflection curves are compared. The experimental curves have slightly higher stiffness for 10m-span specimens and slightly lower stiffness for 4m-span specimens. The horizontal break lines represent the flexural strengths of Table 2 for Figure 4(a) through Figure 4(f), and the shear strengths for Figure 4(g) through Figure 4(i).

The experimental flexural capacity for T10-5-B13-F is only 78% of computed strength. This is the only specimen supported by steel rig for 10m-span specimens. Although no clear damage was observed at the supported region, steel rig support may be inferior to web support for a certain occasion. The reason is still under investigation. Other specimens failed in flexure exceeded or nearly reached the flexural capacity. Specimens T10-5-B19-S and

T20-5-B21-S exceeded the shear capacity although the ultimate failure mode of T20-5-B21-S is not shear.

Stiffness computed from the maximum vertical load and its displacement is slightly larger or nearly equal to the simulated value. In order to control deflections in design, the error is not large and safe side. It is interesting to see creep effects for long term loading on the deflection characteristics. The creep test is under way using the similar specimen to T10-5-B13-M and the results will be published elsewhere.

## CONCLUSIONS

Mechanical properties of prestressed glue laminated timber T-slab was experimentally studied under monotonic loadings. The behavior was linear up to the peak load and the flexural or shear failure occurred at the peak load. Although some load drop was observed, the prestressing tendons prevented the complete collapse of beams and helped regaining load carrying capacity for the further deflection. The flexural and shear strengths based on linear elastic behavior and specified materials strength specified in the Japanese Agricultural Standards simulated the flexural capacity well (except T10-5-B13-F) and shear capacity conservatively.

Although it is necessary to study the fire protection, the proposed timber slab system is worth the further study for the future practical use.


## ACKNOWLEGEMENTS

This study was financially funded by the Ministry of Land, Infrastructure, Transportation and Tourism (PI: Prof. Hitoshi Tanaka) between 2007 and 2010.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