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Fundamental Study on a New Evaluation Method of The Safety Prefabricated Scaffolds

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Abstract: When a new member of a scaffold is developed, it is necessary to follow the standard. Therefore, all scaffolds will assume the same structure. The aim of this study was to establish a new method for evaluating scaffold performance. In the present study, a buckling analysis of prefabricated scaffolds was executed, using the shear rigidity of the vertical and the horizontal frames as parameters. From the results, an equation is proposed for evaluating the strength of prefabricated scaffolds.

Key words: scaffolds, vertical frame, horizontal frame, buckling strength, shear rigidity

1. Introduction

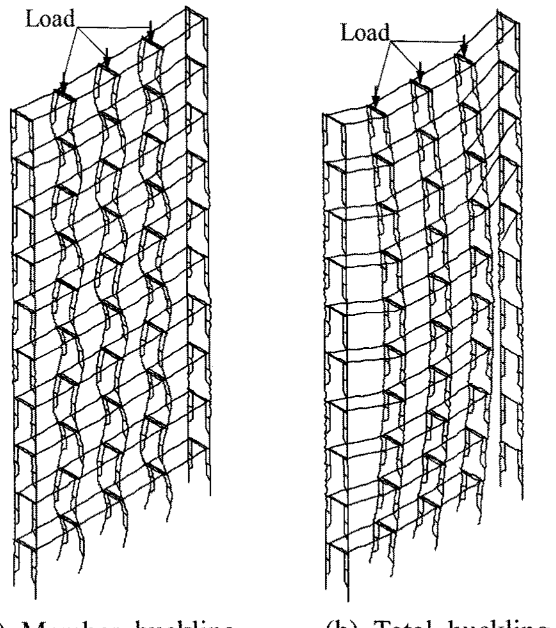
The vertical load on prefabricated scaffolds holds 'live weight', i.e. the people and materials on them. Expressly, standard prefabricated scaffolds might be used as the concrete support. Therefore, excessive vertical load is also likely to act on prefabricated scaffolds. The buckling modes of scaffolds are illustrated in Fig. 1. They include member buckling, when each story of the scaffold curves, and total buckling, when the entire side of the scaffold curves [1]. The buckling load for total buckling is smaller than that for member buckling because the buckling length in total buckling is greater than that in member buckling. Therefore, the member buckling is stronger than the total buckling.

Recent studies confirm that buckling comes about mainly when the stiffening member in the vertical frame is shorting [2]. Therefore, total buckling happens when the shear rigidity in the vertical frame is deficient.

On the other hand, the horizontal frame spans the vertical frame. Shear rigidity of the vertical frame is the influencing factor when scaffolds are buckling. Therefore, it is thought that the shear rigidity of the horizontal frame also influences the strength of scaffolds.

In recent studies by authors [3-4], the buckling strength of scaffolds was confirmed using analytical models of 10-story and 4-bay scaffolds, with parameters being the shear rigidity values for the vertical and horizontal frames. From the analytical results, the boundary between the two buckling modes, member buckling and total buckling, was clarified. However, a design method based on this new knowledge was not examined.

In this study a buckling analysis of standard prefabricated scaffolds was executed, using the shear rigidity of the vertical and the horizontal frames as parameters, to inves-



(a) Member buckling (b) Total buckling

Fig. 1. Scaffolds buckling modes.

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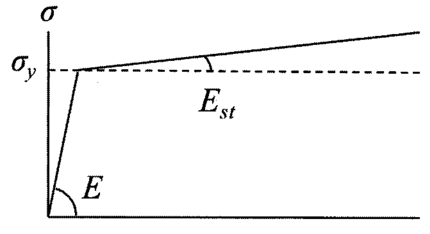


Fig. 2. Relationship between stress σ and strain ε in analysis.

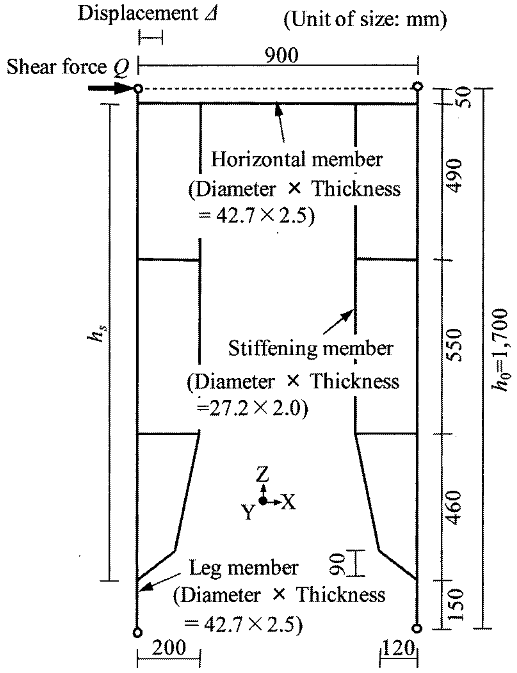


Fig. 3. Vertical frame in analysis.

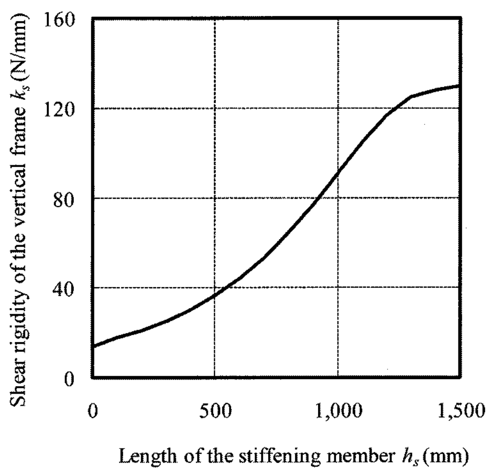


Fig. 4. Relationship between shear rigidity of the vertical frame k_s and length of the stiffening member h_s.

tigate the strength performance of prefabricated scaffolds.

2. Outline of Numerical Analysis

In the analysis, ANSYS 11.0, a general-purpose Finite Element Method, was used. The material was assumed to be the isotropic elastic-plastic model, and the yield criterion was assumed to be the von Mises yield. Fig. 2 shows the relationship between the stress σ and the strain ε used for the analysis, and the material properties in the analysis are shown in Table 1. In the table, v shows Poisson's ratio. Referring to the actual material of the vertical frame, the horizontal member and leg member in the vertical frame were STK500, and the stiffening member in the vertical frame was STK400. The horizontal frame was assumed to be SS400, a typical structural material.

3. Shear Rigidity of Vertical Frame

The vertical frame in the analysis is illustrated in Fig. 3; it represents the type of frame generally used on construction sites. The length of the stiffening member h_s shown in Fig. 3 is a typical length for those used in actual vertical frames. The beam elements with two nodes were used as a finite element. To examine the strength performance of the vertical frame, the length of the stiffening member h_s was adjusted, and the corresponding shear rigidity of the vertical frame k_s was examined in the analysis. The boundary condition at the bottom of the leg member in the vertical frame was assumed to be the pin node as the most risky case.

Fig. 4 shows the analytical results. When the length of the stiffening member h_s was 0 mm, the shear rigidity of the vertical frame, k_s, was 0.013 kN/mm. The value of k_s

Table 1. Material property in analysis

Member (Material)	v	E (N/mm^2)	E_{st} (N/mm^2)	σ_y (N/mm^2)
The leg member and the horizontal member of the vertical frame	0.3	205000	2050	355
The stiffening member of the vertical frame (STK400)	0.3	205000	2050	235
The horizontal frame (SS400)	0.3	205000	2050	235

increased as the length of the stiffening member, h_s, increased.

4. Buckling Load of Vertical Frame

4.1 Buckling load

To examine the strength performance of the vertical frame, the length of the stiffening member h_s was adjusted, and the corresponding buckling load of the vertical frame P_s was examined in the analysis. The boundary condition in this case, at the bottom of the leg member in the vertical frame was assumed to be the pin node as the most risky case. In setting the boundary condition, the upper and lower edge of the leg member in the vertical frame was taken as the pin node. The horizontal movement of the upper and lower edge of the leg member was held as shown in Fig. 5. The vertical load was set from the upper part of the vertical frame.

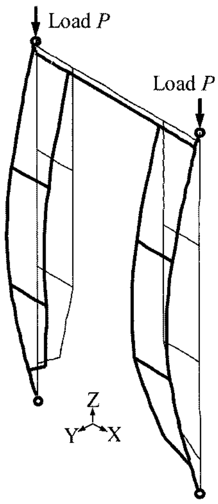


Fig. 5. Buckling of the Vertical frame.

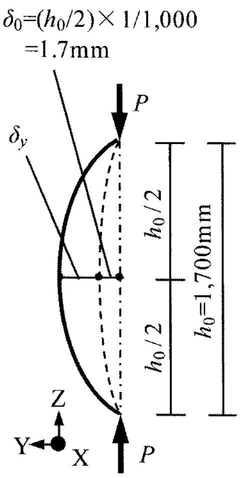


Fig. 6. Lateral deflection of the vertical frame with initial crookedness.

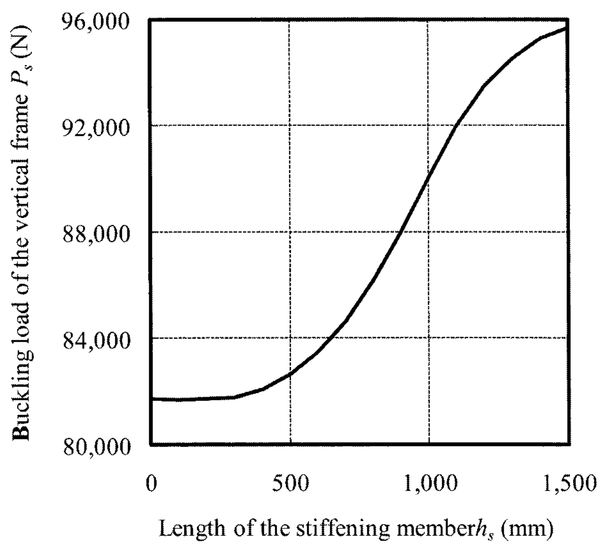


Fig. 7. Relationship between buckling load of the vertical frame P_s and length of the stiffening member h_s.

The buckling load of the one vertical frame P_s in this boundary condition more or less equaled the buckling load of prefabricated scaffolds, when the member buckling is occurred.

Moreover, an actual steel member usually bends a little at the beginning, and this is known as the 'initial crookedness'. The initial crookedness at the center of the member was assumed to be about 1/1,000 of length of the member from measurements of the actual member [5]. The initial crookedness was set by the sine wave referring to this value. In the Y direction of the vertical frame as shown in Fig. 6, the maximum displacement, due to the initial crookedness, was 1/1,000 (1.7 mm) of the height of the vertical frame.

Fig. 7 shows the analytical results. When the length of the stiffening member h_s was 0 mm, the buckling load of the vertical frame, P_s, was 81,700 N. P_s increased as the length of the stiffening member, h_s, increased.

4.2 Comparing analytical and theoretical values

To verify the consistency of the analysis in this study, the analytical results were compared with the theoretical value. Euler's load for member buckling of the vertical frame, P_{se}, is shown in the following equation:

$$P_{se} = \frac{\pi^2 E I_e}{h_0^2} \tag{1}$$

Where π is the ratio of the circumference of a circle to its diameter; E is Young's modulus; I_e is the equivalent geometrical moment of inertia of the scaffolds [1]; and h_0 is the height of the vertical frame.

I_e is calculated to consider the influence of the scaffold's stiffening member and is shown in the following equation:

$$I_e = 2\left(I_0 + I_s\frac{h_s}{h_0}\right) \tag{2}$$

Where, I_0 is the geometrical moment of inertia of the leg member in the vertical frame; I_s represents the stiffening member's geometrical moment of inertia; h_0 is the height of the vertical frame; and h_s is the height of the stiffening member.

Fig. 6 shows the displacement of the center of the member δ_y in the column, showing initial crookedness δ_0, when the load P was placed on the vertical frame. The relationship between δ_y and load P is shown as follows.

$$\delta_y = \frac{P_{se}}{P_{se}-p}\delta_0 sin\frac{\pi}{2} \tag{3}$$

Fig. 8 shows the results of comparing the analytical results of Fig. 7 with the theoretical values by means of equation (3). The vertical axis as shown in Fig. 8 is the ratio of load P to the theoretical value P_{se} in the equation (1). The horizontal axis, as shown in Fig. 8, is the ratio of the displacement of the center of the member $\delta_y + \delta_0$ to height of the vertical frame h_0. The black point as shown in Fig. 8 is the buckling load of the one vertical frame P_s. The curve is the theoretical value of the equation (3). The analytical value is shown by the length of the stiffening member of the vertical frame h_s = 0-1,500 mm at 100 mm intervals.

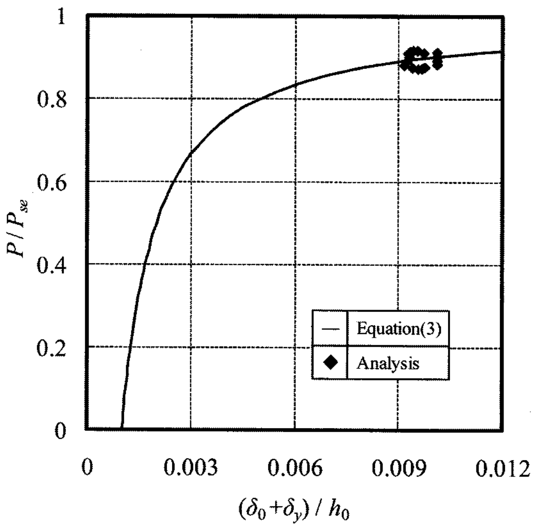


Fig. 8. Relationship between buckling load of the vertical frame and the lateral deflection.

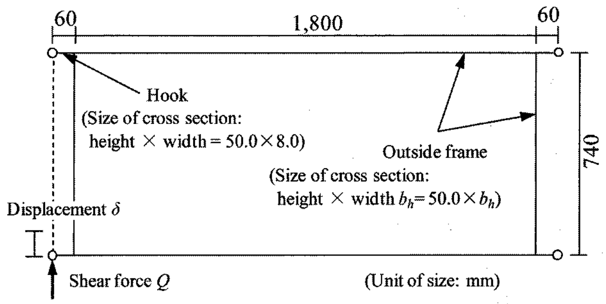


Fig. 9. Horizontal frame in analysis.

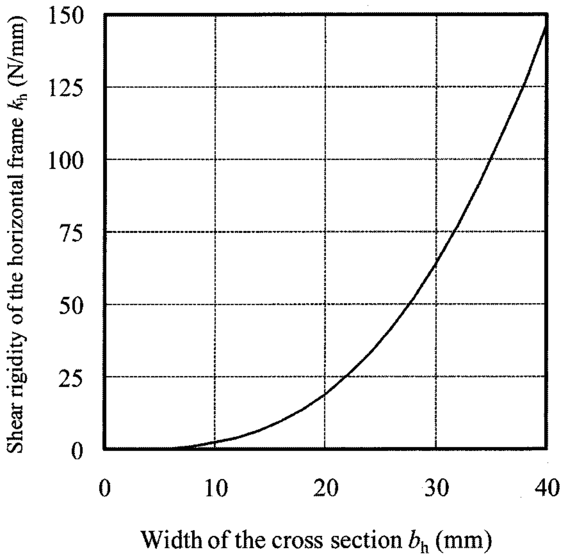


Fig. 10. Relationship between shear rigidity of horizontal frame k_h and width of cross-section of horizontal frame b_h.

The analytical and the theoretical values show a strong relationship, indicating that the analytical value is an appropriate value. Moreover, the relationship P/P_{se} of an analysis is about 0.9. The strength of the vertical frame in analysis decreases less than the theoretical values by about 10%; the initial crookedness is probably an influencing factor. We may therefore conclude that the vertical frame showing initial crookedness retains about 90% of the theoretical value for buckling load.

5. Analytical Model of Horizontal Frame

An analytical model of the horizontal frame was modeled simply because an actual horizontal frame has a complex shape. The horizontal model used in the analysis is illustrated in Fig. 9. Beam elements with two nodes were used as a finite element of the horizontal frame. The end point of the horizontal frame was assumed to be the pin joint because this generally forms the junction between the vertical frame and the horizontal frame in actual construction sites.

The shear force as shown in Fig. 9 was placed on the horizontal frame, and the frame's shear rigidity was tested. Then, a cross-section of the outside frame in the horizontal frame, b_h, was adjusted for width and the shear rigidity k_h was tested again.

Fig. 10 shows the analytical results. The shear rigidity of the horizontal frame, k_h, increased as the cross-section of the outside frame in the horizontal frame, b_h, increased.

6. Buckling Analysis of Scaffolds

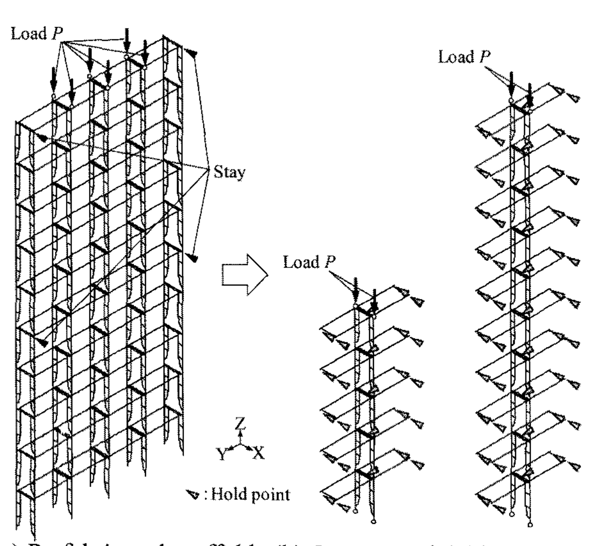
6.1 Analytical model and method

Stays must be fastened to the scaffold in the current regulations. The intervals between stays must be not more than 8,000 mm in the horizontal direction and 9,000 mm vertically. The prefabricated scaffolds in this study had stays placed at intervals appropriate for a 5-story (1,700 mm [length of vertical frame] × 5 = 8,500 mm) and 4-bay (1,800 mm [length of horizontal frame] × 4 = 7,200 mm) scaffold, as shown in Fig. 11(a). When a vertical load was set on top of the vertical frame with stays in position, it is thought that the stay shared the load. However, we then set the vertical load on the top of the frame without stays in place, to simulate a situation of greatest risk. To determine the influence of the number of scaffold stories, these were set to 5 and 10, representing the minimum value within regulations regarding the vertical direction of the stay and that twice 10. We then compared both models. In the tested models of prefabricated scaffolds, the row of the vertical frame containing the stay was assumed to have no movement in a horizontal direction.

The prefabricated scaffold, as shown in Fig. 11(a), was simplified to form the 2-span models as shown in Fig. 11(b) and Fig. 11(c). Fig. 11(b) shows a 5-story model. Fig. 11(c) is a 10-story model. The X-axis represents the span direction of the vertical frame of one row and two horizontal frames; the Y-axis represents the longitudinal direction of the scaffold; and the Z-axis represents the height direction. In the both Fig. 11(b) and Fig. 11(c) the point where the end of the horizontal frame is not connected with the vertical frame was fixed in the X direction. The load required for member buckling in the models, Fig. 11(b) and Fig. 11(c), is the same as that required for the model shown in Fig. 11(a), because both models shown in Fig. 11(b) and Fig. 11(c) were modeled to represent one row of the whole vertical frame shown in Fig. 11(a).

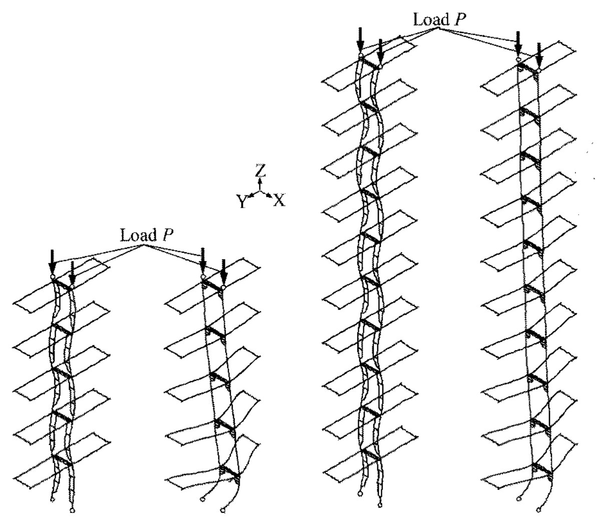
The Y (length) direction of the horizontal member in each story was fixed in terms of the effect of the brace, so the brace was not modeled as the element. The boundary of the bottom edge of the scaffolds was determined as the pin joint as the highest risk case. The joints of both vertical and horizontal frames were also determined as the pin joints, and the joint connecting sections of the vertical frame was determined as the rigid joint with reference to the actual construction sites. The vertical load was set from the top of the scaffolds on the assumption that standard prefabricated scaffolds are used with the concrete support.

Initial crookedness was set both in the X direction, where total buckling is usually caused, and in the Y



(a) Prefabricated scaffolds (b) 5 story model (c) 10 story model

Fig. 11. Prefabricated scaffolds in analysis.



5 story model 10 story model
(a)Member buck. (b)Total buck. (c)Member buck. (d) Total buck.

Fig. 12. Buckling mode.

direction, where member buckling is usually caused and was set according to the sine wave. In the X direction the maximum displacement due to initial crookedness, was 1/1,000 (17 mm) of the height of the scaffolds. In the Y direction the maximum displacement was 1/1,000 (1.7 mm) of the height of each story of the scaffolds.

The shear rigidity of the vertical and horizontal frames was adjusted and a buckling analysis was carried out on the models.

6.2 Analytical result for buckling mode and evaluation equation for scaffolds strength

Degree of deformation of the scaffolds at maximum load (buckling load) is illustrated in Fig. 12 by numerical analysis. Fig. 12(a) and 12(b) show the results for the 5-story model, and Fig. 12(c) and 12(d) show the results for the 10-story model. Fig. 12(a) and 12(c) also show the results of $k_s = 130$ N/mm ($h_s = 1{,}500$ mm), $k_h = 30$ N/mm. Fig. 12(c) and (d) show the results of $k_s = 25$ N/mm ($h_s = 300$ mm), $k_h = 1$ N/mm. The value for degree of deformation is expressed by the analysis to be ten times the actual deformation.

Fig. 12(a) and (c) became the member buckling, and Fig. 12(b) and (d) did not become the total buckling. In case of no member buckling, the displacement was caused in the X direction, and the lowest story of the scaffolds was greatly deformed. This displacement does not have been the total buckling, but it is thought that this displacement was caused due to a lack of shear rigidity in the vertical frame. Moreover, it is thought that also influence at the boundary condition between the scaffolds and ground.

The lowest story of the scaffolds is particular attention. Fig. 13(a) shows a model of the vertical frame at the lowest story. Fig. 13(b) is a simplified model of Fig. 13(a), and here the vertical frame is shown as one member. The value k as shown in Fig. 13(a) and 13(b) represents a spring constant occurring where the top point O of the leg member in the vertical frame is braced. k takes all the horizontal force in the X direction of the vertical frame. When the horizontal frame is assumed to be a horizontal bracing member for the vertical frame, k is added, as well as the value for shear rigidity k_h for the horizontal frame at the lowest story. Therefore, k is given as follows:

$$k = k_s + k_h \tag{4}$$

When the vertical frame with initial crookedness buckles as shown in Fig. 13(a) and 13(b), horizontal displacement δ is caused. Simultaneously, a spring reaction force $Q_k = k(\delta_0 + \delta)$ is caused. The balance of the moment at point O as shown in Fig. 13(b) then becomes as follows:

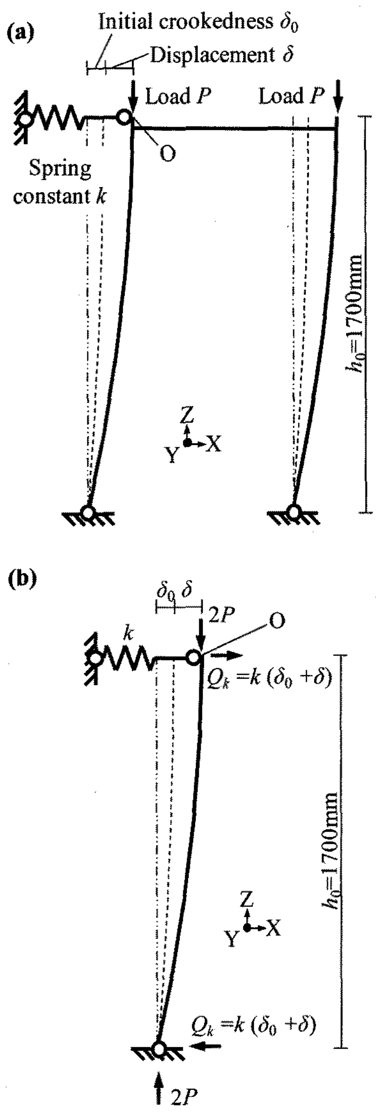


Fig. 13. (a) Model of the vertical frame at the scaffold's lowest story, (b) Simplified model of the vertical frame at the scaffold's lowest story.

$$2P(\delta_0 + \delta) = k(\delta_0 + \delta)h_0 = Q_k h_0 \tag{5}$$

Where, h_0 is the height of the vertical frame. When it is solved with the spring constant k, k is shown as follows.

$$k = \frac{2P}{h_0} \tag{6}$$

The buckling load $2P$ is proportional to the spring constant k. When k becomes infinity, the buckling load $2P$ becomes infinity also. When $2P$ is the same value as the buckling load of the vertical frame P_s, the prefabricated scaffolds experience member buckling. At this time, point O doesn't move. The buckling load always becomes P_s whatever the value of the spring constant k. When this P_s is substituted for $2P$ in the equation (6), the next equation is obtained:

$$k_{cr} = \frac{P_s}{h_0} \tag{7}$$

When the equation (6) is expressed by the equivalent geometrical moment of inertia I_e that derives from the influence of the stiffing member of the vertical frame, k_{cr} is shown in the following equation:

$$k_{cr} = \frac{\pi^2 EI_2}{h_0^3} = \frac{2\pi^2 E\left(I_0 + I_s \frac{h_s}{h_0}\right)}{h_0^3} \tag{8}$$

When the shear rigidity of the vertical frame or the horizontal frame is the same as or more as k_{cr} as shown in equation (8), the scaffolds experience member buckling. For this situation Fig. 14 shows a comparison between k_{cr} of the equation (8) and the shear rigidity of the vertical frame k_s.

The horizontal axis in Fig. 14 is the slenderness ratio of the vertical frame λ calculated by using the equivalent geometrical moment of inertia I_e. λ is shown as follows:

$$\lambda = \frac{h_0}{i_e} = \frac{h_0}{\sqrt{\frac{I_e}{2A_0}}} = \frac{h_0}{\sqrt{\frac{I_0 + I_s \frac{h_s}{h_0}}{A_0}}} \tag{9}$$

Where A_0 is a cross-section of the leg member, and i_e is the radius of gyration as influenced by the stiffening member of the vertical frame.

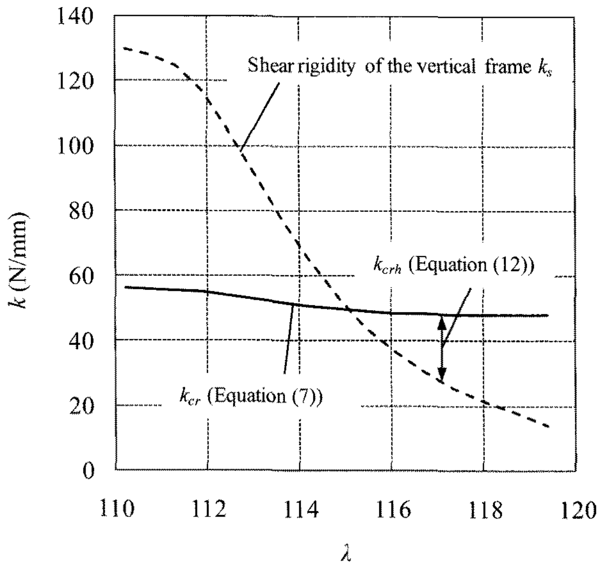


Fig. 14. Relationship between shear rigidity of the vertical and the horizontal frames and scaffolds strength.

Here, h_s = 1,500 mm is as about λ = 110.2, and h_s = 0 mm is as about λ = 119.4.

It is compare between k_{cr} and the shear rigidity of the vertical frame.

In the next equation, the prefabricated scaffolds become member buckling despite the shear rigidity of the horizontal frame.

$$k_{cr} < k_s \tag{10}$$

In the next equation we see that when shear rigidity is only found in the vertical frame, the prefabricated scaffolds do not become member buckling because k_s is insufficient.

$$k_{cr} > k_s \tag{11}$$

In case of the equation (11), when the value of k_h in equation (4) becomes greater than the value of k_{crh} in the next equation, the prefabricated scaffolds do become member buckling.

$$k_{crh} = k_{cr} - k_s \tag{12}$$

6.3 Result of buckling analysis and assessment of evaluation equation

Fig. 16 shows the analytical results. The vertical axis shown in Fig. 16 is the ratio of the buckling load Pm to the yield axial force P_y. The horizontal axis shown in Fig. 16 is the slenderness ratio of the vertical frame λ calculated by using the equivalent geometrical moment of inertia I_e. P_y is shown as follows:

$$P_y = 2A_0\sigma_{ys} \tag{13}$$

Where A_0 is a cross-section and σ_{ys} is the yield stress of the leg member.

The curve in Fig. 16 represents the buckling load of the vertical frame P_s. The white points show the analytical results for the 5-story model, and the black points show the analytical result for the 10-story model. At the time

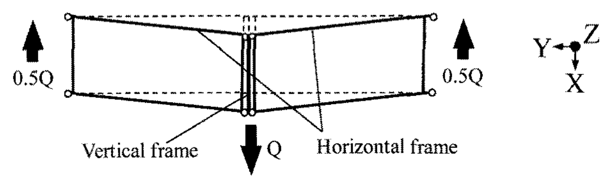


Fig. 15. Plane in the prefabricated scaffolds.

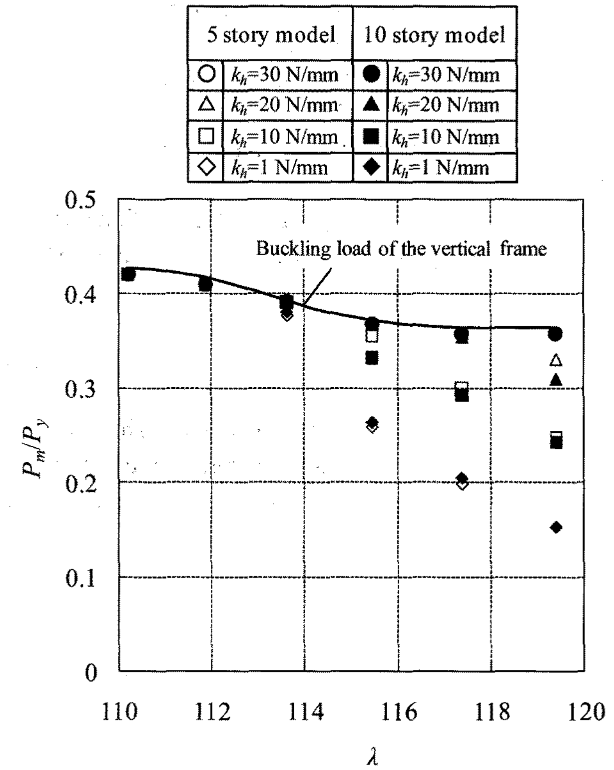
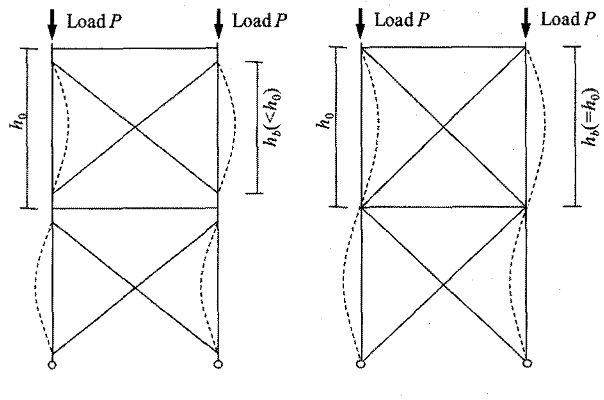


Fig. 16. Relationship between the yield ratio of the axial force and the slenderness ratio.



(a) Case of h_b<h_0 (b) Case of h_b=h_0

Fig. 17. Member buckling of scaffolds.

when member buckling occurred, the value of P_m/P_y for the white points and the black points were the almost same as for the curve. When no member buckling was occurring, the value of P_m/P_y for the white points and the black points were less than the value of P_m/P_y for the curve. From Fig. 14 we see that when the analytical model is almost at $\lambda < 115$, the scaffolds experienced member buckling regardless of the value of shear rigidity of the horizontal frame k_h. Fig. 16 shows that when the value of P_m/P_y of the white and black points at $\lambda < 115$ were the almost same as the P_m/P_y value of the curve, regardless of the value of k_h, the scaffolds experienced member buckling. In the case of $\lambda > 115$, when λ is larger and k_h is smaller, the value of P_m/P_y for both white and black points was less than the P_m/P_y value of the curve.

When λ is larger, it is necessary to enlarge k_h to see member buckling occur. It is thought that the results of Fig. 16 correspond to the results of Fig. 14.

Therefore, it was proven that k_s and k_h can be calculated from equations (7)-(12), to ascertain when member buckling would occur. Moreover, when the value of λ and k_h for the white and black points is the same, the value of P_m/P_y in the white and black points is almost the same. Therefore, when the boundary condition between the scaffold and the ground is marked by a pin and the scaffold is between 2 stories and 10 stories in height, we can assume that the strength of the scaffold is decided by the shear rigidity of the vertical and horizontal frames of the lowest story of the scaffolds, regardless of the number of stories.

In this study, we assumed the position of the boundary at the bottom of the scaffold to be the pin joint as in a situation of highest risk. However, the boundary between the scaffold and the ground does not always get pinned; actual scaffolds use a jack base at their lowest point; in that situation it is probable that the number of stories does influence the strength of scaffolds.

The length of the brace h_b in a scaffold is usually shorter than the one story of the scaffold shown in Fig. 17(a). Therefore, if the scaffolds are member buckling, the length of the buckling in the scaffolds becomes shorter than one story of the scaffolds under the influence of braces. In this case, the buckling load in the scaffolds becomes higher than the length of the buckling for the one story of the scaffolds shown in Fig. 17(b). When the shear rigidity of the vertical frame or the horizontal frame is the same as, or more than, k_{crb}, as shown in the next equation, the scaffolds undergoes member buckling:

$$k_{crb} = \frac{2P}{h_b} \tag{14}$$

7. Conclusion

A buckling analysis of prefabricated scaffolds was conducted in our study, to provide a parameter in the study of shear rigidity of the vertical and horizontal frames. We also investigated the validity of a proposed evaluation equation. The results of this study can be summarized as follows:

(1) When the junction between the scaffold and the ground is pinned, as for a highest risk situation, and the scaffold is between 2 and 10 stories, we conclude that

that the strength of the scaffold is decided by the shear rigidity of the vertical and horizontal frames of the lowest story of the scaffold, regardless of the number of stories.

(2) When the shear rigidity of the vertical frame k_s is greater than k_{cr} of equation (5), i.e., $k_{cr} < k_s$, prefabricated scaffolds will undergo member buckling despite the shear rigidity of the horizontal frame k_h. Member buckling will also occur in the case of $k_{cr} > k_s$, when the value of k_h is $k_{cr} - k_s$ ($= k_{crh}$) or more.

(3) When the shear rigidity of the vertical frame or the horizontal frame is the same as, or more than k_{crb} under the influence of braces, the scaffolds undergo member buckling.

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