Structural Engineering and Mechanics, Vol. 17, No. 2 (2004) 167-186

DOI: http://dx.doi.org/10.12989/sem.2004.17.2.167

Design load-carrying capacity estimates and an improved wooden shore setup

Y.L. Huang†, Y.C. Lin†, C.F. Lee‡, H.J. Chen‡† and T. Yen‡‡

Department of Civil Engineering, Chung Hsing University, Taichung, 40227 Taiwan, R.O.C.

(Received January 29, 2003, Accepted October 16, 2003)

Abstract. The design load-carrying capacities of wooden shores depend on factors, such as the wood species and properties, and construction methods. This paper focuses on the construction methods, including an upright single shore, group of upright shores, group of inclined shores, butt connections and lap connections. This paper reports experiments to obtain critical loads and then developed an empirical equation based on Euler's formula for the critical loads and design load-carrying capacities. The test results show that the critical loads for an upright single wooden shore are greater than the average values for a group of upright shores, and the latter are greater than the average values for a group of inclined shores. Test results also show that the critical loads become smaller when butt or lap connections are used, butt connections possessing greater critical loads than lap connections. Groups of inclined shores are very popular at work sites because they have some practical advantages even though they actually possess inferior critical loads. This paper presents an improved setup for constructing groups of inclined shores. With this method, the inclined shores have larger critical loads than upright shores. The design load-carrying capacities were obtained by multiplying the average critical loads by a resistance factor (or strength reduction factor, ϕ) that were all smaller than 1. This article preliminarily suggests ϕ factors based on the test results for the reference of engineers or specification committees.

Key words: wooden shores; load-carrying capacity; construction.

1. Introduction

Wooden shores are popularly used at work sites all over the world. They are used either as permanent shores in a wooden structure or as temporary supports in a falsework to resist construction loads on concrete-slab/beam formworks that are composed of fresh concrete loads, equipment and workers. ACI 347R-88 defines those applied loads and gives a number of guidelines for safety and serviceability. Based on these guidelines, a number of charts and tables were developed for wooden shore designs for concrete formworks (Hurd 1987, Sommers 1984). However, in these charts and tables there is no information about the effects of upright group shores, inclined group shores and lap/butt connections in shores. A typical structural system for all-

† Professor
‡ Graduate Student
‡† Associate Professor
‡‡ Professor and the President of the University

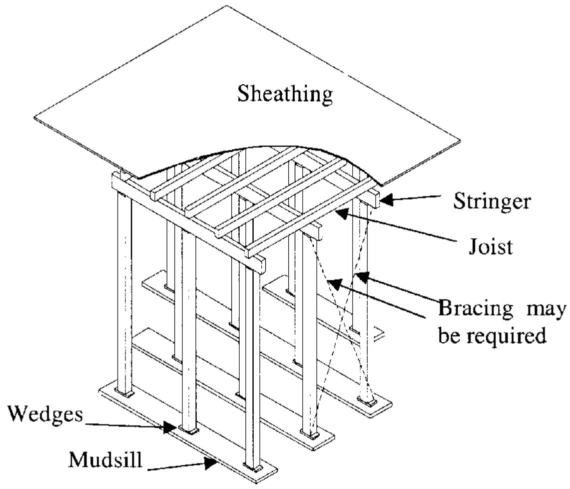


Fig. 1 A typical structural system for all-wood slab formwork (Spiegel and Limbrunner 1991)

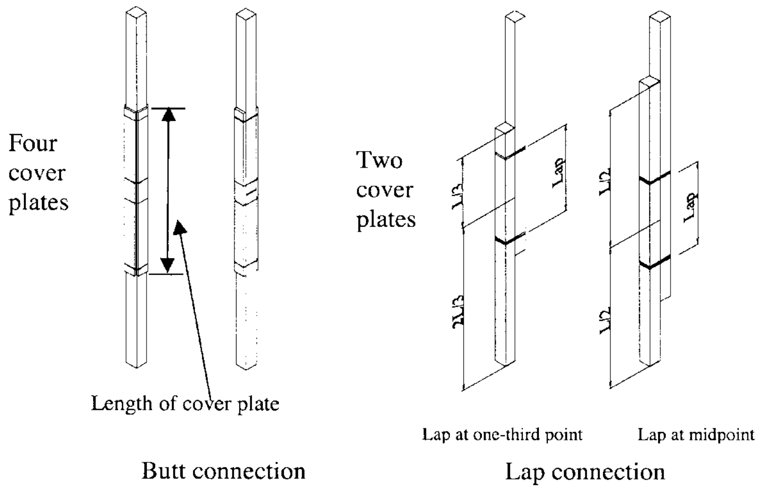


Fig. 2 Butt and lap connections

wood slab formworks is shown in Fig. 1 (Spiegel and Limbrunner 1991). As can be seen in this figure, all of the wooden shores are upright and do not have any butt or lap connections shown in Fig. 2.

In permanent use cases, the bolt connection is usually used to extend the wooden column length. Many studies have researched on this type of connection, e.g. Heine & Dolan (2001) and Daudeville *et al.* (1999). However, in temporary construction cases, the butt or lap connections are mainly used to extend the shore length. According to the surveys made at work sites, the lap connection is more popular than butt connections because the former is easier to form on sites. The survey also shows that the lengths of lap connections are from 70 cm and 90 cm using two iron wires to tie both ends of the lap connections. The effect of the lap and butt connections on the design load-carrying capacities, the load-carrying capacities for design purpose, is one of the main topics of this paper. The lap and butt connection behavior is also different from the bolt connection behavior studied by Mclain *et al*. (1993).

If the wooden shores are longer than the specified construction height, most contractors will not cut their wooden shores to fit the specified height. Instead, they often set up the shores obliquely. Therefore, in a temporary construction upright shores are rare, and almost all are inclined shores. According to the present test results, the critical load of an upright single shore is greater than the average critical load of a group of upright shores and far greater than the average critical load of a group of inclined shores. The load-carrying capacities will reduce due to the inclination of the shores. To continue using inclined group shores without decreasing the critical load, this study developed an improved setup for forming inclined group shores in which the average critical load of a group of inclined shores is greater than that for a group of upright shores. The stability of the group shores on sites was also suggested to consider the leaning column concept (Chen 1987, Peng 2000).

Smith and Foliente (2002) suggested LRFD (load and resistance factor design) concept for timber joints. Skaggs *et al.* (1994) suggested safety factors for design. The concept of the resistance factor is used in the article. An empirical equation was developed to estimate the critical loads and design load-carrying capacities based on the test results and Euler's formula. The design load-carrying capacities were obtained by multiplying the critical loads by resistance factors (ϕ factors) that are suggested by this article and are less than 1.

2. Test programs and test results

2.1 Material properties

The species of wood used in this study were Kapur, Apitong and Lauan. Their ultimate compressive strength σ_u and modulus of elasticity E were tested following the CNS 453 (Chinese National Standards) specifications. All compression tests were performed in the along-grain direction. The test results are shown in Tables 1 and 2. A compression member can fail due to either material-strength failure or buckling failure. The ultimate strength σ_u is used to distinguish the failure modes and the modulus of elasticity is used to develop an empirical buckling equation.

Table 1 Test results for the along-grain ultimate compressive strength

Wood Species	Kapur	Apitong	Lauan
Ultimate Compressive Strength (Mpa)	47.2	54.4	41.2
	43.7	49.6	49.6
	38.0	55.6	37.8
	47.4	49.6	40.4
	52.7	50.5	38.6
	51.2	52.1	38.0
Average	46.7	52.0	39.4
Coefficient of Variation	11.3%	4.9%	3.4%

Table 2 Test results for the modulus of the along-grain elasticity

Wood Species	Kapur	Apitong	Lauan
Modulus of Elasticity (Gpa)	9.34 13.9 11.6 12.3 12.2 13.0 11.6 15.2 11.6	14.1 12.3 16.4 13.9	9.3 9.8 10.9 10.3 13.3 10.9 10.4
Average	12.3	14.2	10.7
Coefficient of Variation	12.6%	10.3%	11.1%

Table 3 Test results for the critical loads and stresses for the single wooden shores

Wood Species		Kapur	Apitong	Red Lauan
2 m-long single shores	Critical load and stress kN (Mpa)	55.4 (15.9) 49.3 (15.7) 56.6 (17.2) 57.6 (18.3)	69.6 (19.0) 65.9 (18.6) 58.9 (16.6) 53.0 (15.0) 42.6 (12.0)	38.6 (11.7) 40.7 (12.1) 34.5 (11.0) 44.6 (13.7)
	Average	54.7 (16.8)	57.4 (16.4)	39.6 (12.1)
	Coefficient of Variation (%)	5.9 (6.3)	30.2 (15.4)	9.2 (8.1)
3 m-long single shores	Critical load and stress kN (Mpa)	19.8 (6.2) 25.6 (7.6) 20.5 (6.0) 25.2 (7.4)	35.2 (9.8) 28.4 (7.8) 21.2 (6.0) 30.7 (8.5) 30.7 (8.3)	13.6 (3.8) 20.7 (5.7) 23.7 (6.6) 20.4 (6.6) 15.3 (4.9)
	Average	22.8 (6.8)	29.2 (8.1)	18.7 (5.5)
	Coefficient of Variation (%)	11.6 (6.3)	15.7 (15.3)	19.9 (19.6)
3.6 m-long single shores	Critical load and stress kN (Mpa)	22.1 (6.4) 16.4 (5.1) 20.9 (6.3) 16.7 (4.9) 17.5 (5.0)	16.2 (4.6) 14.3 (4.3) 20.0 (5.6) 21.9 (5.8) 19.9 (5.4)	12.2 (3.6) 12.8 (3.6) 12.7 (4.2) 9.6 (3.2) 9.3 (2.9)
	Average	18.7 (5.6)	18.5 (5.2)	11.3 (3.5)
	Coefficient of Variation (%)	12.5 (11.8)	15.1 (11.0)	13.7 (12.1)

2.2 Design load-carrying capacities of single wooden shores

Following the wood step stool compression tests performed by Burdette and Core (1996), the compression tests for single wooden shores for three wooden species of Kapur, Apitong and Lauan

Photo 1 Failure mode of an upright single shore

with three kinds of lengths 2 m, 3 m and 3.6 m were conducted. All of the specimens had the same 6 cm × 6 cm square section. During these tests, a wood plank was placed between the top of the shore specimen and the loading head of the testing equipment to avoid local failure on the top of the shores. The bottoms of the specimens were simply placed on the rigid concrete floor. This arrangement for both shore ends was very similar to the actual situation at the work sites. The test results are shown in Table 3. The failure mode is shown in Photo 1. According to the failure mode and failure stresses, which are much smaller than the ultimate compressive strength σ_u, shown in Table 1, all of the specimens were regarded as having failed in the buckling mode.

Since all of the shores failed in the buckling mode, this article constructed an empirical equation based on both the Euler buckling equation and the present test results. The Euler equation for the critical stress of a compressive member is

$$\sigma_{cr} = \frac{\pi^2 E}{(KL/r)^2} \tag{1}$$

where L is the length of the member, r is the radius of gyration and K is the effective length factor dependent on the boundary conditions of both ends. For the boundary condition of the present experiments, the K value should be between 0.5 and 1.0 because K is, in theory, equal to 0.5 for both fixed ends and 1.0 for both hinged ends. Substituting the tested failure stresses in Table 3, the lengths of the wooden shores and the modulus of the elasticity shown in Table 2 into Eq. (1) to determine the K values, the average K value is 0.8. This value is then suggested by this article for this empirical equation. Therefore, the empirical equation for estimating the critical stress of a single wooden shore becomes

Table 4 Comparisons of the estimated critical stresses from Eq. (2) to the tested critical stresses

Wood species		Kapur	Apitong	Red Lauan
Modulus of elasticity (Gpa)		12.3	14.2	10.7
2 m-long single shores (Mpa)	σ_{cr} from Eq. (2)	14.2	16.4	12.4
	σ_{cr} from tests	16.8	16.4	12.1
	Deviation	−18.3%	0	+1.87%
3 m-long single shores (Mpa)	σ_{cr} from Eq. (2)	6.3	7.3	5.5
	σ_{cr} from tests	6.8	8.1	5.5
	Deviation	−7.9%	−10.9%	0
3.6 m-long single shores (Mpa)	σ_{cr} from Eq. (2)		5.1	3.8
	σ_{cr} from tests		5.2	3.5
	Deviation		−2.2%	+8.3%

$$\sigma_{c.r.} = \frac{\pi^2 E}{(0.8L/r)^2} = \frac{15.38E}{(L/r)^2} \approx \frac{15E}{(L/r)^2} \tag{2}$$

and the critical load of a single wooden shore is

$$(P_{c.r.})_{\text{single}} = \frac{15.38EA}{(L/r)^2} \approx \frac{15EA}{(L/r)^2} \tag{3}$$

The estimated critical stresses from Eq. (2) and the tested critical stresses are listed in Table 4. This table shows that the maximum deviation of the critical stresses between Eq. (2) and the test results is 18.3%. Therefore, a conservative value 0.8 is suggested to cover all these deviations and the design load-carrying capacities are then obtained by multiplying the critical loads by the ϕ values. This article suggests ϕ = 0.8 based on the test results for the reference of engineers or specification makers. The empirical equation for the design load-carrying capacities of single wooden shores $(P_{all})_{single}$ becomes

$$(P_{all})_{\text{single}} = \phi \times (P_{c.r.})_{\text{single}} = 0.8 \times \frac{15.38EA}{(L/r)^2} = 12.3\frac{EA}{(L/r)^2} \approx 12\frac{EA}{(L/r)^2} \tag{4}$$

2.3 Butt and lap connection effects on the design load-carrying capacities

The butt and lap connections were also tested and studied in this article. At the work sites, the lap connection is more popular than the butt connection because it is easier to build. The effects of these two types of connections on the design load-carrying capacities were studied as follows.

2.3.1 Effects of butt connections

A typical butt connection is shown in Fig. 2, in which two shores are connected end to end, wrapped with two or four wooden plates and tied up using several iron wires. Because the wooden shores were placed end to end, the connection would not cause a slip when the compressive axial loads were applied. The quantity and length of the cover plates are the two major topics studied in this article.

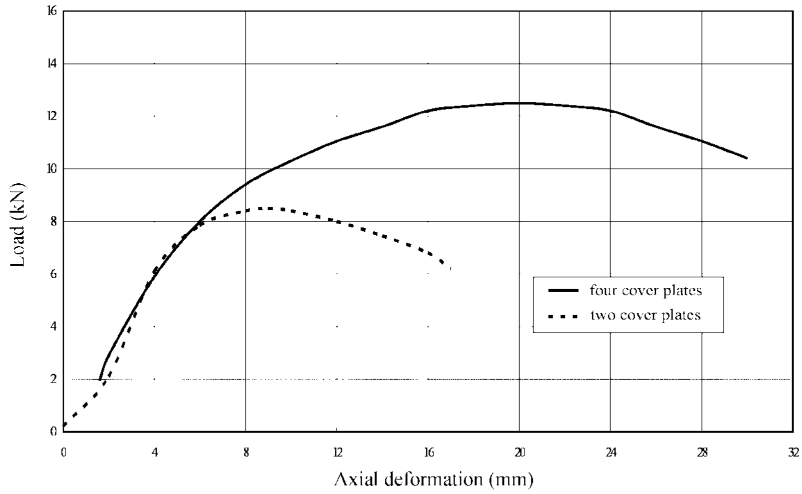


Fig. 3 Critical loads for the four-cover-plate and two-cover-plate butt-connected shores (cover plate length = 1.4 m)

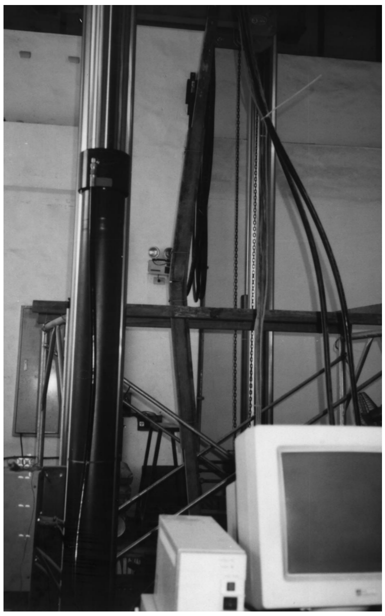


Photo 2 Failure mode for a butt-connected shore with two cover plates

Fig. 3 shows that at the same cover-plate length, the critical loads for the four-cover-plate shores are much larger than those for the two-cover-plate shores. This was because the latter had cover plates on only the two sides and caused a weak direction on the other two sides. Test results show that all of the two-cover-plate samples failed in the weak direction, shown in Photo 2.

In regard to the cover plate lengths, 1.8 m and 1.4 m were the two most popular lengths at the work sites according to the survey. These two lengths were therefore tested in this article. The test

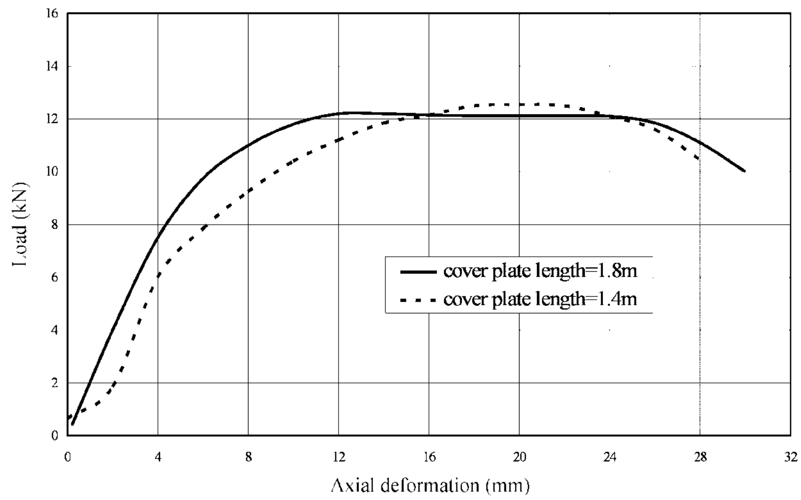


Fig. 4 Load and displacement relation for the four-cover-plate butt-connected shores

Table 5 Test results for the critical loads and the maximum axial deformations for the butt-connected shores

Length	Types of cover plates	Critical loads (kN)		Axial deformations (mm)	
3 m-long shores	Two 1.8 m-long cover plates	14.5 20.1	17.3	8.7 9.9	9.3
	Two 1.4 m-long cover plates	14.0 16.0	15.0	9.0 8.9	8.9
	Four 1.8 m-long cover plates	23.0 16.0	19.5	12.5 10.0	11.3
	Four 1.4 m-long cover plates	25.0 21.0	23.0	13.0 10.1	11.5
3.6 m-long shores	Two 1.8 m-long cover plates	9.5 10.5	10.0	9.0 10.4	9.7
	Two 1.4 m-long cover plates	7.3 9.5	8.4	6.2 8.1	7.2
	Four 1.8 m-long cover plates	15.5 14.0	14.85	11.5 10.2	10.9
	Four 1.4 m-long cover plates	18.0 14.5	17.72	10.2 16.0	13.1

results shown in Fig. 4 indicate that if the number of cover plates is the same, the stiffness (the slope of the curve) of the shore with the 1.8 m-long cover plates was greater than that for the shore with the 1.4 m-long cover plates. However, both critical loads were almost the same. This phenomenon reveals that the length of the cover plates does not have an apparent effect on the critical loads if the length of the cover plates is over 1.4 m. All of the tested critical loads and the maximum axial deformations of the butt-connected shores are listed in Table 5. Based on this table,

Table 6 Suggested reduction rates for the critical loads due to butt connections

	Without connection (kN)		With a butt connection (kN)							
			Two cover plates				Four cover plates			
	Loads	Avg.	Loads	Avg.	Loss	Suggested reduction rates	Loads	Avg.	Loss	Suggested reduction rates
3 m-long shores	19.8 25.6 20.5 25.2	22.8	14.5 20.1 14.0 16.0	16.2	28.9%	30%	23.0 16.0 25.0 21.0	21.3	6.6%	10%
3.6 m-long shores	22.1 16.4 20.9 16.7 17.5	18.7	9.5 10.5 7.3 9.5	9.2	50.8%	50%	15.5 14.0 18.0 14.5	15.5	17.1%	20%

the reduction rates for the critical loads due to butt connections are suggested in Table 6. In Table 6, the loss of the critical loads for two-cover-plate shores were about 29% for 3 m-long shores and 50% for 3.6 m-long shores, and the loss of the critical loads for four-cover-plate shores were about 7% for 3 m-long shores and 17% for 3.6 m-long shores. These test results indicate that the loss of critical loads due to butt connections becomes larger when the total shore length gets longer and the four-cover-plate shores are much better than the two-cover-plate shores. If the total length of the butt-connected shores is less than 3.6 m, according to the test results, the rate for the critical load reduction is suggested 20% for the four-cover-plate connections and 50% for the two-cover-plate connections. In other words, the modification factor m_{butt} is 0.8 for the four-cover-plate connections and 0.5 for the two-cover-plate connections, respectively. Therefore, the design load-carrying capacity for a single butt-connected shore, $(P_{all})_{butt,\ single}$ becomes

$$(P_{all})_{butt,single} = m_{butt}(p_{all})_{single} = m_{butt}\phi(P_{c.r.})_{single} = m_{butt}\times 0.8\times(P_{c.r.}) = m_{butt}\times 12\frac{EA}{(L/r)^2} \quad (5)$$

m_{butt} : 0.8 for the four-cover-plate butt connection
m_{butt} : 0.5 for the two-cover-plate butt connection

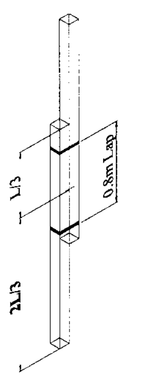
Table 6 shows that in both two-cover-plate and four-cover-plate cases the reduction rates almost double when the length of shores changes from 3 m to 3.6 m. The influence of the shore length on the load-carrying capacities for butt-connected shores is very large. The reasons for this large influence are that on one hand the length originally affects the capacities; on the other hand the longer shore causes a larger moment at the butt-connected area.

2.3.2 Effects of lap connections on design load-carrying capacities

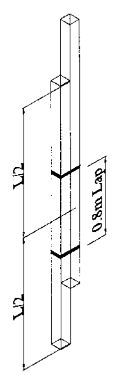
A typical lap connection for the wooden shores is shown in Fig. 2, in which the overlapping portion of the shores is tied up using iron wires. The critical load for the lap-connected shores depends only on the friction on the contact surface between these two shores. Unlike the butt connection, the lap connection will induce an apparent slip when axial loads are applied. Normally, the tighter the iron wires, the greater the critical loads for the lap-connected shores. According to the survey done at the work sites, the laborers used mostly #10 iron wires to make the lap connections and tied up the wires as tight as possible. Therefore, #10 iron wires were used in these experiments. There are several systems, SWG, B.W.G., BG and JDP systems, for regulating iron wire numbers. In all of these systems, the larger the wire number, the smaller the diameter. This article used SWG system in which the diameter of a #10 iron wire is 3.251 mm. The test results for the breaking loads and the axial deformations for the lap-connected shores are shown in Table 7. In

Table 7 Test results for the breaking loads and the axial deformations for the lap-connected shores

Length	Types of lap connection	Loads to cause 10 mm axial deformation (kN)		Breaking loads (kN)		Axial deformations at breaking loads (mm)	
3 m-long shores	1 m-long lap connection at midpoint	6.8 7.8	(Avg.) 7.3	14.5 14.0	14.25	68.0 36.5	52.3
	0.8 m-long lap connection at midpoint	4.7 5.5	5.1	14.0 12.1	13.05	93.1 71.2	82.2
	1 m-long lap connection at one-third position	6.0 5.8	5.9	13.5 15.5	14.5	75.1 77.0	76.1
	0.8 m-long lap connection at one-third position	8.0 6.8	7.4	16.5 14.5	15.5	83.7 82.1	82.9
3.6 m-long shores	1 m-long lap connection at midpoint	5.0 6.0	5.5	11.0 12.0	11.5	73.0 92.7	82.9
	0.8 m-long lap connection at midpoint	7.2 5.8	6.5	13.1 14.5	13.8	58.7 80.1	69.4
	1 m-long lap connection at one-third position	5.7 5.8	5.75	15.5 15.0	15.25	47.2 83.0	65.1
	0.8 m-long lap connection at one-third position	6.8 6.3	6.55	14.6 15.5	15.05	76.1 67.0	72.6



0.8m-long lap connection at one-third position



0.8m-long lap connection at midpoint

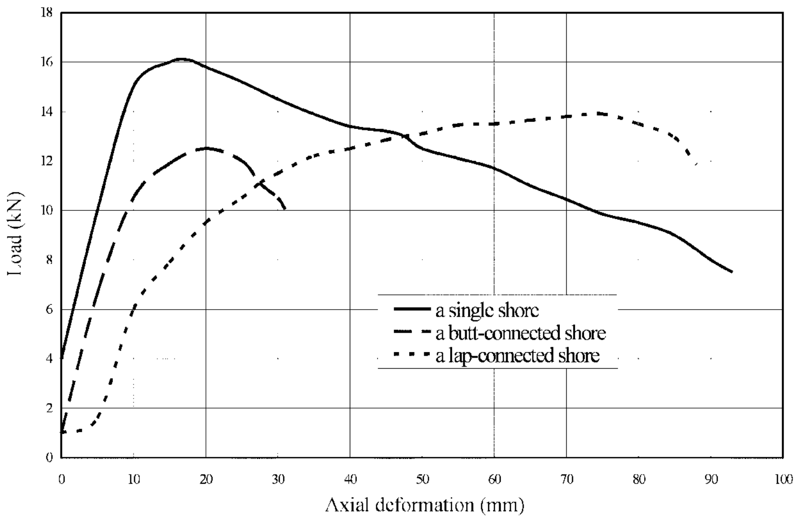


Fig. 5 Load-deformation relationships for a single shore, a single butt-connected shore and a single lap-connected shore

this table, the overlap lengths of the 1 m and 0.8 m did not have an apparent effect on the critical loads. The lapping locations at the midpoint or the one-third position did not have apparent effects, either. The test results also show that the iron wires failed first in all of the lap-connected specimens. The load-deformation relationships for a single shore, a single butt-connected shore and a single lap-connected shore are plotted together in Fig. 5. This figure shows that the axial deformation of the lap-connected shore at its breaking load is much larger than the other two types of shores. The axial deformation of the lap-connected shores is so large that the failure definition for the lap-connected shores needs to change to be defined by an amount of axial deformation rather than the axial load. According to Fig. 5, the yielding deformation of 3 m-long single shore and butt-connected shore are about 10 mm. If a wooden shoring system is combined with shores without connection, shores with butt connections and shores with lap connections, when the overall axial deformation reaches 10 mm, all of the shores except for the lap-connected shores will yield. Under this circumstance, the shoring system can be considered yielding although the lap-connected shores do not yield. Moreover, even if a wooden shoring system is made up of purely lap-connected shores, too large a deformation is not allowed in any construction. For these two reasons, this article suggests that the maximum allowable axial deformation for the lap connection be 10 mm. According to Table 8, if the critical load is the breaking load, the loss of critical load for a lap connection will be about 40%. But, if the critical load is defined by the axial deformation of 10 mm, the loss will be about 69%. It is recommended that the reduction rate for the critical loads due to the lap connection be decided by the axial deformation and be 0.7. In other words, the modification factor for the lap connections, m_{lap}, is 0.3.

Test results show that the butt connection is much better than the lap connection, especially the butt connection with the four cover plates. Since the loss of the critical load in the lap connections is so large, this article recommends that lap connections NOT BE USED at the work sites. However, if the lap connection is inevitable, the design load-carrying capacities, $(P_{all})_{lap}$, are suggested as follows

Table 8 Suggested reduction rates for the critical loads due to lap connections

	Without connection (kN)		With a lap connection (kN)							
			Loads to cause 10 mm axial deformation				Breaking loads			
	Loads	Avg.	Loads	Avg.	Loss	Suggested reduction rates	Loads	Avg.	Loss	Suggested reduction rates
3 m-long shores	19.8 25.6 20.5 25.2	22.8	7.3 5.1 5.9 7.4	6.43	71.8%	70%	14.25 13.05 14.5 15.5	14.33	37.2%	-
3.6 m-long shores	22.1 16.4 20.9 16.7 17.5	18.7	5.5 6.5 5.75 6.55	6.08	67.5%	70%	11.5 13.8 15.25 15.05	13.9	25.7%	-

$$(P_{all})_{lap,\,single} = m_{lap}(P_{all})_{single} = m_{lap} \times 12.3\frac{EA}{(L/r)^2} = 3.69\frac{EA}{(L/r)^2} \approx 3.5\frac{EA}{(L/r)^2} \tag{6}$$

2.4 Design load-carrying capacities for group shores

Wooden shores are used mostly in a manner for group shores at the work sites. The behavior of group shores is somehow different from a single shore. Because a single shore has only one shore, it must be stable in any direction from side sways. According to the test results described previously, the average tested value for K is 0.8, which means there is partial moment restraint when the top of the shore is kept aligned by friction. But for group shores (see Fig. 6) on work sites, some shores are braced from side sways and the other shores have little lateral stability and

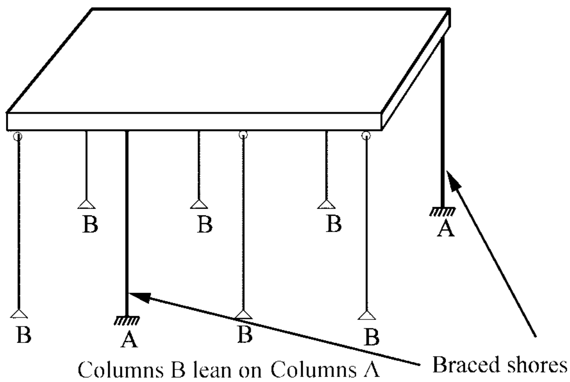


Fig. 6 A leaning column system

have to lean on the braced shores. Such a system in which some shores cannot resist side sways and lean on the other shores is called a leaning column system (Chen and Lui 1987). In this system, the effective length factor K of the shores providing the side-sway resistance is modified and the critical loads will become smaller. Le Messurier (1977) suggested a formula for this modification.

$$K_i = \sqrt{\frac{\pi^2 EI}{L^2 P_i}\left(\frac{\Sigma P}{\Sigma P_{ek}}\right)} \tag{7}$$

where

- K_i : the modified effective length factor of the column providing the side-sway resistance
- P_i : the axial force in the column providing the side-sway resistance
- ΣP : the axial loads on all columns in a storey
- ΣP_{ek} : the Euler loads for all columns in a storey providing the side-sway resistance for the frame evaluated using the effective length obtained from the nomograph (Chen and Lui 1987)

This research performed experiments to determine the critical loads for group shores. The tested samples included upright group shores, inclined group shores and the improved inclined group shores developed by this research. All of the tested samples were 3 m long.

2.4.1 Design load-carrying capacities for upright group shores

The design load-carrying capacities for the upright group shores containing 2, 4, 8 and 12 wooden shores were tested and the test results are shown in Table 9. The setups for a group of 8 upright wooden shores and their failure modes are shown in Fig. 7 and Photo 3. In Table 9, the average critical loads for a group of upright shores was smaller than that for an upright single shore, but the reduction rate becomes stable when the quantity of shores are equal or more than four. This stable reduction rate was approximately 35%. In other words, the modification factor m_{group} for upright group shores is 0.65. According to the test results, the design load-carrying capacities for the upright group shores, $(P_{all})_{group}$, were suggested as follows

$$(P_{all})_{group} = m_{group} N (P_{all})_{single} = 0.65 \times N \times 12.3 \frac{EA}{(L/r)^2} = 8.0 \frac{NEA}{(L/r)^2} \tag{8}$$

in which N is the number of shores.

Table 9 Test results for the critical loads for the upright single and group shores (kN)

	Upright single shore	2 shores	4 shores	8 shores	12 shores
Total loads	22.8	38.56	61.84	116.48	175.00
Average	22.8	19.28	15.46	14.56	14.58
Reduction rates compared to an upright single shore	-	–15.3%	–32.1%	–36.1%	–36.1%
Suggested reduction rates compared to an upright single shore	35%				

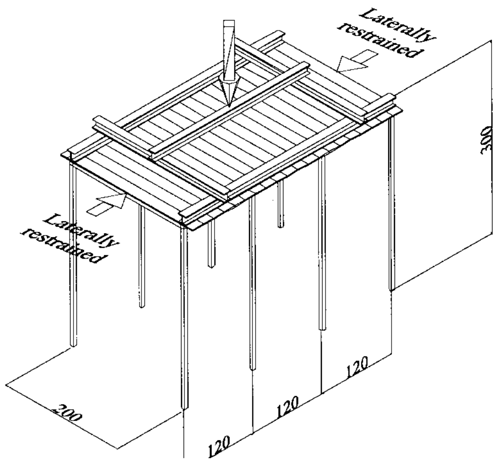


Fig. 7 Setup for a group of 8 upright wooden shores

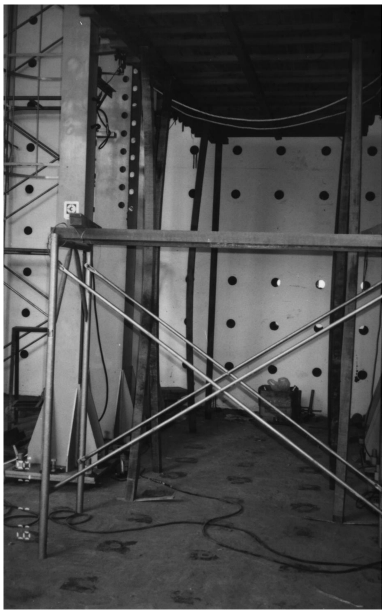


Photo 3 Failure mode for a group of 8 upright shores

The test setup of group shores in the laboratory can be considered the structure form shown in Fig. 8. This figure shows that all the shores are set up the same way and do not have side-sway resistance. Therefore, it is needed to have the side-sway resistance from the outside. Under this circumstance, each wooden shore became a stable pin-pin column. The effective length factor for each wooden shore should, theoretically, be K = 1. Then the critical loads and design load-carrying capacities for the upright group shores become

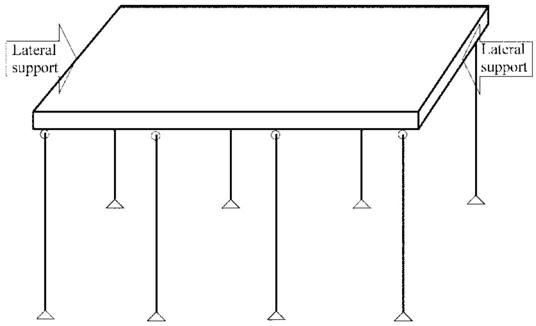


Fig. 8 A schematic structural form for tested group shores

$$(\sigma_{c.r.})_{upright,\,group} = \frac{\pi^2 E}{(Kl/r)^2} = \frac{\pi^2 E}{(l/r)^2} = \frac{9.87E}{(l/r)^2} \tag{9}$$

$$(P_{c.r.})_{upright,\,group} = \frac{9.87NEA}{(l/r)^2} \tag{10}$$

$$(P_{all})_{upright,\,group} = \phi(P_{c.r.})_{upright,\,group} = 0.8 \times \frac{9.87NEA}{(l/r)^2} = \frac{7.90NEA}{(l/r)^2} \tag{11}$$

which met the test results for Eq. (8) very well.

2.4.2 Design load-carrying capacities for inclined group shores

During the tests, the inclined group shores were setup in pairs. In each pair, one shore was inclined to one direction and the other was inclined to another direction. All of the shores were inclined at an approximate angle of 15$^{\mathrm{o}}$ to the vertical line. The setup for a group of 8 inclined shores is shown in Fig. 9. In this study, experiments on various inclined group shores, containing 4, 8 and 12 wooden shores, were conducted. The test results are listed in Table 10. According to this

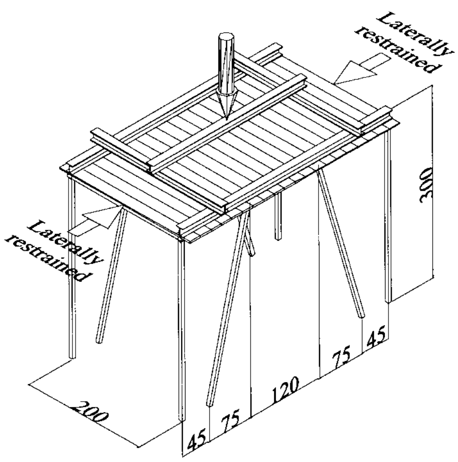


Fig. 9 Setup for a group of 8 inclined shores

Table 10 Test results for the critical loads for the inclined group shores (kN)

	Upright single shore	2 shores	4 shores	8 shores	12 shores
Total loads	22.8	-	46.56	87.44	151.04
Average	22.8	-	11.64	10.88	12.58
Reduction rates compared to an upright single shore	-	-	–48.9%	–52.3%	–44.8%
Suggested reduction rate compared to an upright single shore	50%				

table and Table 9, the critical loads for the inclined group shores were less than those for the upright group shores. From Table 10, the reduction rate for the inclined group shores to an upright single shore was approximately 50%. In other words, the modification factor $(m_{group})_{inclined}$ for the inclined group shores was 0.5. Then, the estimation for the design load-carrying capacities for the inclined group shores are suggested as follows

$$(P_{all})_{group,\ inclined} = (m_{group})_{inclined} \times N \times (P_{all})_{single}$$
$$= 0.5 \times N \times 12.3 \frac{EA}{(L/r)^2} = 6.15 \frac{NEA}{(L/r)^2} \approx 6 \frac{NEA}{(L/r)^2} \tag{12}$$

in which N is the number of inclined shores.

The failure mode for the inclined group shores is shown in Photo 4. Although the inclined group shores have a remarkable loss of critical loads (about 50%), they are still popularly used at the work sites because they have two advantages: (1) there is no need to cut the shores to fit the construction height, (2) they are easier to tear down when the construction is done.

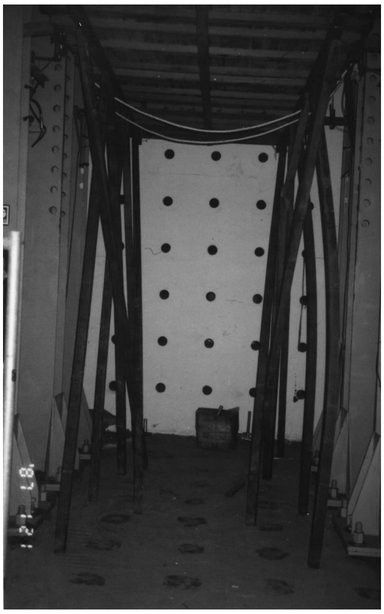


Photo 4 Failure mode for a group of 8 inclined shores

3. Improved setup to construct inclined group shores

An improved setup to construct inclined group shores was developed and suggested in this paper. It is schematically shown in Fig. 10. In this figure, the shores are set in pairs. Both shores in each pair are crosswise and tied up about in the middle location using #8 iron wires. Each pair of shores forms a basic unit. Since each shore is tied in the middle position, the effective length of the shore is shorter than the original and then possesses a much larger critical load.

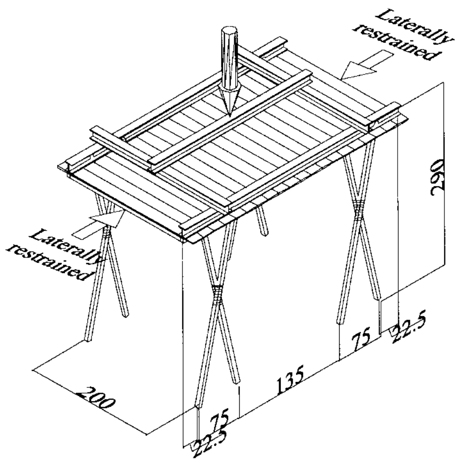


Fig. 10 Improved setup for a group of 8 inclined shores

Photo 5 Failure mode for an improved group of 8 inclined shores (both shores in a pair failed at the same time)

Table 11 Test results for the critical loads for improved group shores (kN)

	Upright single shore	2 shores	4 shores	8 shores	12 shores
Total critical loads	22.8	-	65.2	137.0	181.92
Average	22.8	-	16.3	17.13	15.16
Compared to the regular inclined group shores (Table 10)			11.64 (+40%)	10.88 (+57.4%)	12.58 (+20.5%)
Compared to upright group shores (Table 9)			15.46 (+5.4%)	14.56 (+17.7%)	14.58 (+4.0%)
Reduction rates compared to an upright single shore	-	-	22.8 (–28.4%)	22.8 (–24.8%)	22.8 (–33.4%)
Suggested reduction rates compared to an upright single shore			30%		

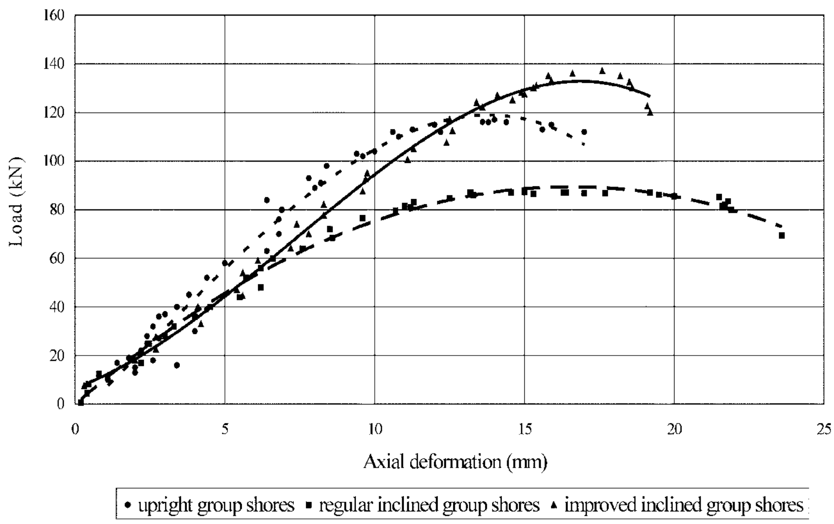


Fig. 11 Relation between loads and vertical deformations for upright group shores, regular inclined group shores and improved inclined group shores

The failure mode for the improved inclined group shores is shown in Photo 5. The photo shows that both shores in a pair fail at the same time. This phenomenon reveals that a pair of shores do form a basic unit and are subjected to the loads together. The test results for the improved group shores are listed in Table 11. The table indicates that the critical loads for the improved inclined group of shores are larger than those for the regular inclined group shores by approximately 40% and even larger than those for the upright group shores by around 5%. Unlike before, the labourers do not need to worry about the loss of critical loads using the improved inclined group shores. Conservatively, the improved inclined group shores are suggested to have the same design load-carrying capacities as the upright group shores.

Test results also show that the improved inclined group shores have better ductility. The relations between loads and vertical deformation for the upright group shores, regular inclined group shores and improved inclined group shores are plotted together in Fig. 11. This figure indicates that the improved inclined group shores have slightly larger critical loads and slightly better ductility than the upright group shores.

4. Conclusions

It was suggested that the critical stress and load for a single upright wooden shore in laboratory can be estimated using the Euler Equation with $K = 0.8$ which are

$$\sigma_{c.r.} = \frac{\pi^2 E}{(0.8L/r^2)} = \frac{15.38E}{(L/r)^2} \approx \frac{15E}{(L/r)^2}$$

$$(P_{c.r.})_{single} = \frac{15.38EA}{(L/r)^2} \approx \frac{15EA}{(L/r)^2}$$

To determine the design load-carrying capacity (P_{all}) for a single upright wooden shore, this article preliminarily suggests $\phi = 0.8$ for the reference of engineers or specification makers. Therefore,

$$(P_{all})_{single} = \phi \times (P_{c.r.})_{single} \approx 12\frac{EA}{(L/r)^2}$$

This article also suggests several modification factors, $m_{butt} = 0.8$ or 0.5, $m_{lap} = 0.3$, $m_{group} = 0.65$ and $(m_{group})_{inclined} = 0.5$, for modifying the design load-carrying capacities when the shores are butt-connected, lap-connected, a group of upright shores or a group of inclined shores.

Since the loss of the critical load in the lap connections is so large, this article recommends that the lap connections NOT BE USED at the work sites.

An improved method to setup a group of inclined shores was developed and suggested in this article. The design load-carrying capacities for this style of shores are larger than those for regular groups of inclined shores by approximately 40% and even larger than those for groups of upright shores by around 5%. The improved setup also have slightly better ductility than the regular setup.

References

ACI 347R-88 (1988), “Guide to formwork for concrete”, American Concrete Institute (ACI), Detroit, Mich.

Burdette, E.G. and Core, H.A. (1996), “Compression test of a wood step stool”, *Forest Products Journal*, **46**(7-8), 93-95.

Chen, W.F. and Lui, E.M. (1987), *Structural Stability Theory and Implementation*, PTR Prentice Hall, Englewood Cliffs, New Jersey.

CNS 453, “Method of test for compression wood”, Chinese national Standards (CNS), Taipei, Taiwan.

Daudeville, L., Davenne, L. and Yasumura, M. (1999), “Prediction of the load-carrying capacity of bolted timber joints”, *Wood Science and Technology*, **33**(1), 15-29.

Heine, C.P. and Dolan, J.D. (2001), “A new model to predict the load-slip relationship of bolted connections in timber”, *Wood and Fiber Science*, **33**(4), 534-549.

Hurd, M.K. (1987), “Formwork for concrete”, SP-4, 4th Ed, *American Concrete Institute (ACI),* Detroit, Mich.

Le Messuier, W.J. (1977), “A practical method of second order analysis, part 2 - rigid frames”, *Engineering*

Journal, AISC, **14**(2), 49-67.
Mclain, T.E., Soltis, L.A., Pollock, D.G. and Wilkinson, T.L. (1993), "Lrfd for engineered wood structures - connection behavioral equations", *J. Struct. Eng.*, ASCE, **119**(10), 3024-3038.
Peng, J.L. (2000), "Stability analyses and design recommendations for practical shoring systems during construction", *Journal of Construction Engineering and Management*, ASCE, **128**(6), 536-544.
Skaggs, T.D., Woeste, F.E., Dolan, J.D. and Loferski, J.R. (1994), "Safety factors for metal-plate-connected wood trusses - theoretical design versus test specifications", *Forest Products Journal*, **44**(9), 11-18.
Smith, I. and Foliente, G. (2002), "Load and resistance factor design of timber joints: International practice and future direction", *J. Struct. Eng*., ASCE, **128**(1), 48-59.
Sommers, P.H. (1984), "Charts aid in design of horizontal formwork", *Aberdeen's Concrete Constr.*, **29**(4), 648-651.
Spiegel, L. and Limbrunner, G.F. (1991), *Applied Statics and Strength of Materials,* Merill/MacMillan, Columbus, Ohio.