# Strength Variability of Conventional Slab Formwork Systems

By Saeed Karshenas[1]

**Abstract:** Wooden formwork design is currently based on the criteria developed for the design of wood in new condition. To develop formwork design criteria that include the effect of material reuse and formwork installation workmanship, data on actual formwork member resistances and installation inaccuracies are required. In this paper, wooden slab formwork installation inaccuracies and strength variabilities are investigated. The formwork structural properties presented are based on laboratory experiments on formwork members collected from eight construction sites located in Chicago, Illinois, Milwaukee, Wisconsin, and Minneapolis-St. Paul, Minnesota. From each construction site, samples of shores, stringers, and joists were collected. In addition to formwork member sample collection, during site visits, installed shore out-of-plumb angles and stringer, joist, and sheathing spans were also measured. In this paper the observed formwork member physical conditions, installation inaccuracies, shore compressive strengths, and bending and shear strengths of the stringers and joists are presented. The observed structural properties are summarized with appropriate probability distributions.



## INTRODUCTION

The specifications that are currently used for the design of temporary structures are mostly based on experience and engineering judgement. Development of a consistent and rational framework of load and resistance factors for design during construction requires a knowledge of the variability of loads applied to structures during construction and the uncertainties in resistances of temporary or incomplete structures that support the construction loads.

In an effort to investigate the safety of current concrete construction practices and to develop probability-based criteria for formwork design, the writer initiated a comprehensive study of loads and resistances of slab formwork structures used in multistory concrete building construction. Some of the results of the load analysis are presented in Karshenas and Ayoub (1994), Ayoub and Karshenas (1994), and Karshenas and Heinrich (1994). The load investigation involved a survey of material, equipment, and worker loads in over 22 multistory concrete construction projects located in 12 cities around the country. To model the resistance of slab formwork systems, eight concrete building projects were visited with the objective to obtain samples of formwork members for laboratory experiments. A number of different slab formwork systems are used in concrete building construction. These include conventional wooden or metal formwork systems, flying forms, and column-supported forms. This study is limited to conventional wood slab formwork systems. This type of formwork is very popular in concrete building construction and has a higher strength variability than flying forms. This paper presents the observed formwork installation workmanship, physical conditions of the formwork samples collected from the construction sites, results of laboratory experiments conducted on the samples, and probability models that best represent the observed results. For a detailed discussion of testing apparatuses used for the experiments, sample loading methods, and formulas used for determining structural properties of the collected samples, refer to Karshenas and Walsh (1995), Karshenas and Mizian (1996), and Karshenas and Montes Rivera (1996).

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Note. Discussion open until February 1, 1998. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on November 13, 1995. This paper is part of the ***Journal of Construction Engineering and Management***, Vol. 123, No. 3, September, 1997. ©ASCE, ISSN 0733-9364/97/0003-0324–0330/ \$4.00 + \$.50 per page. Paper No. 12029.

## CONVENTIONAL WOODEN SLAB FORMWORK SYSTEMS

In projects where conventional wooden slab formwork is used, the formwork system components usually include 13–19 mm (0.5–0.75 in.) thick plywood sheathing, 100 × 100 mm (4 × 4 in.) joists, 100 × 150 mm (4 × 6 in.) stringers, and adjustable wooden shores known as Ellis shores. Fig. 1 shows a typical wooden slab formwork. In this type of formwork, support against lateral loads is provided by X-bracings made of 50 × 100 mm (2 × 4 in.) or 25 × 150 mm (1 × 6 in.) lumber nailed to the shores.

Considering only the material cost, 50 mm (2 in.) thick joists and stringers would provide the required strength at lower cost than the customary 100 mm (4 in.) thick lumber. However, the installation cost of 100 mm (4 in.) thick joists and stringers is substantially lower than that for 50 × 200 mm (2 × 8 in.) or 50 × 250 mm (2 × 10 in.) members. Formwork is a temporary structure that is erected and dismantled several times during construction of a building. Therefore, its design must facilitate the erection and dismantling processes. In conventional wooden slab formwork systems, 100 × 150 mm (4 × 6 in.) stringers are easily installed on top of shores using metal U-heads or wire heads. Joists are dropped in place by workers; 100 × 100 mm (4 × 4 in.) joists are equally stable when they are supported on either side and are not connected to stringers for fast erection and dismantling. Using 50 mm (2 in.) thick lumber for joists or stringers would require special connections and cross bracing for lateral stability of joists and

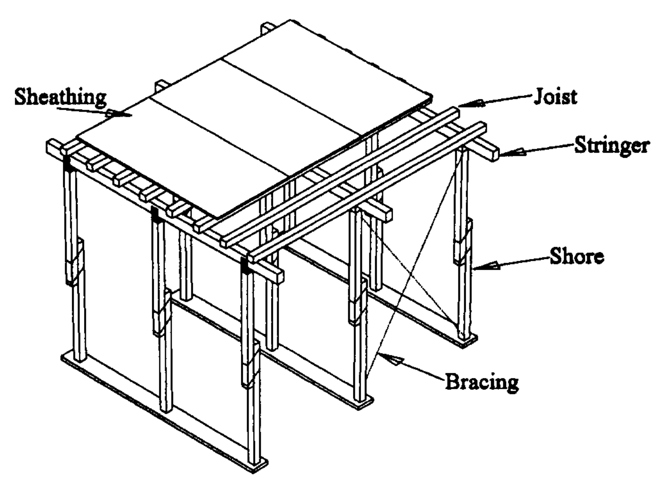


FIG. 1. Typical Wood Slab Formwork



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stringers that substantially increases erection and dismantling costs.

The species of lumber usually used in formwork construction is Douglas fir or Southern Pine (Hurd 1985). Formwork materials are usually reused until they are badly damaged and are not usable anymore. The decision to use or discard a formwork member is subjectively made by the formwork contractor.

Currently, wooden formwork system design is based on the criteria developed for structural design of wood in new condition. Although ACI 347R-88 ("Recommended" 1988) cautions formwork designers to take reuse effects into consideration, it is difficult to do this without specific guidelines. The strength of conventional formwork structures is also affected by their installation workmanship. Resistance variability of conventional formwork systems is much higher than that of a permanent wooden structure because of installation inaccuracies and repeated reuse of material. An example of installation inaccuracies is shown in Fig. 2. In this figure, some of the shores are installed out of plumb. Fig. 3 shows examples of careless joist placement in a formwork structure and that formwork members are not always spaced as specified by the formwork designer.

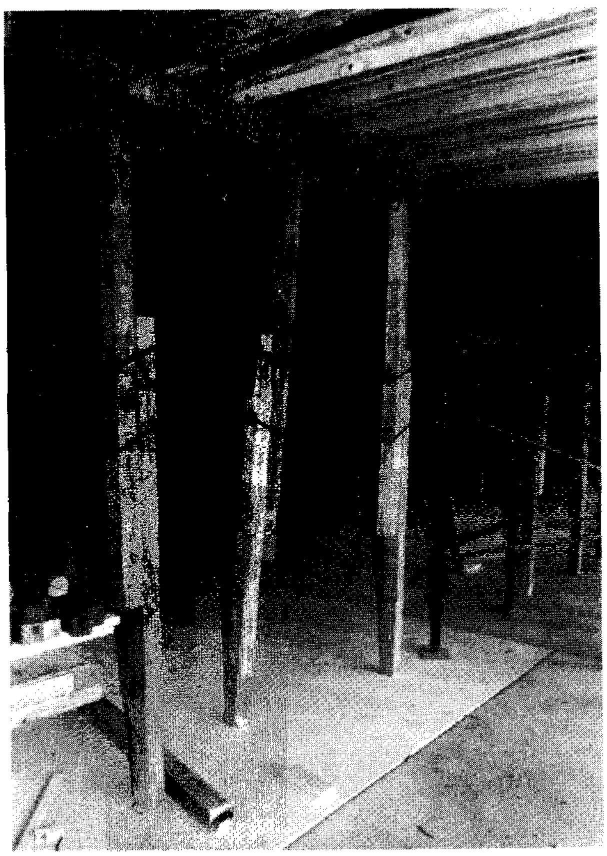

**FIG. 2. Out-of-Plumb Shore Installation in Formwork**

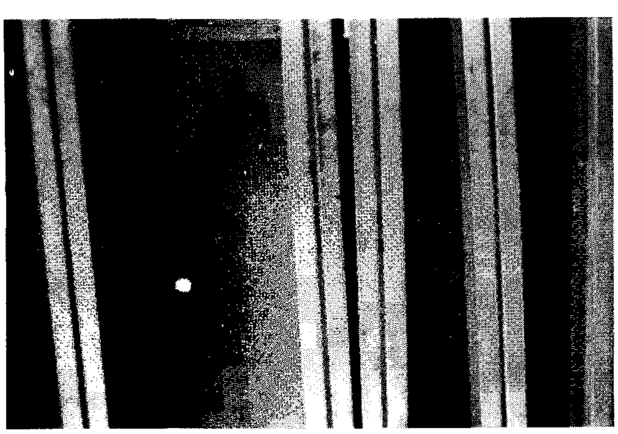

**FIG. 3. Out-of-Place Joist Installation in Formwork**

The foregoing discussion shows that design criteria developed for permanent wooden structures are not always adequate for formwork design and must be adjusted to take into consideration the workmanship used in formwork construction and the effect of material reuse on strength. Currently there is no information on actual formwork installation workmanship and material properties to allow such adjustments.

## FORMWORK INSTALLATION WORKMANSHIP

Data were collected on shore out-of-plumb angles as well as errors in shore, stringer, and joist placement. In each project visited, the angles of inclination of the shores of the floor under construction were measured in both east-west and north-south directions. The angles were measured using a digital protractor that can measure the angle of a surface relative to the vertical direction with an accuracy of one-hundredth of a degree. The observed out-of-plumb angles ranged from 0 to 2.7°. The mean of the observed out-of-plumb angles was 0.76 degrees.

Sheathing, stringer, and joist spans in each project were also measured. The ratios of the observed to designer specified sheathing spans (joist spacings) had a mean value of 1 with a coefficient of variation (COV) of 0.23. The ratios of the observed to design spans of joists had a mean and a COV equal to 0.98 and 0.07, respectively; for stringers these ratios had a mean value of 0.95 and a COV of 0.15.

## SAMPLE COLLECTION PROCEDURE

Of the eight projects that were visited for sample collection, three were located in Chicago, Illinois, two in Milwaukee, Wisconsin, and 3 in Minneapolis-St. Paul, Minnesota. These projects were multistory concrete buildings that used conventional wood slab formwork. In each project, 15 randomly selected samples of each formwork component were marked with spray paint. The contractors agreed to set aside as many marked samples as they could find at the end of the project for shipping to the laboratory for testing. Eight-eight joists, 100 stringers, and 104 Ellis shores were shipped to the laboratory.

## OBSERVED PHYSICAL CONDITIONS OF SHORES

All 104 collected shore samples were adjustable wooden shores also known as Ellis shores. An Ellis shore consists of two wood posts attached together with two metal clamps. The wood posts are 100 × 100 mm (4 × 4 in.) nominal size lumber; metal clamps, known as Ellis clamps, are manufactured by Ellis Construction Specialties, Ltd., Oklahoma City, Oklahoma (*Ellis* 1994). One side of each clamp is permanently attached to the bottom post. The top post can be moved up or down inside the clamps for adjusting the length of the shore. After installation, clamps are tightened using a hammer. To reduce shore deflection under load, two nails are driven into the top post immediately above the clamp plates.

Among factors that can affect the load bearing capacity of an Ellis shore are the physical conditions of the clamps and wood posts, species and grade of lumber used, and angle of inclination of the shore after installation. Repeated reuse and mishandling of shores during installation, removal, and transportation can result in slight twisting of clamp collars or cracking of clamp plates. Damaged clamps do not tighten adequately during shore installation, which results in excessive deflection and failure of the shore. The clamps of most of the





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collected samples were in good condition. Eight samples had deformed or twisted clamps that did not allow proper alignment of the top and bottom shore members. One of the shore samples also had a cracked clamp plate.

Ellis shore posts must be in good physical condition. Decayed for cracked members must be discarded. Among the samples, one shore had a decayed area. The wood in the decayed area was soft and could be removed with a fingernail. Shore lumber must not have large knots, knotholes, or knot clusters. Fig. 4 shows a sample with a large knothole. The lumber used in shore construction must be No. 1 or better. The grade marks of most members were not legible due to weathering. Among the collected samples, 23 had legible grade marks; two were Douglas fir No. 2 and the rest were Douglas fir No. 1 or better. Shore posts must also be straight. A bow in the top or bottom member of a shore can prevent adequate tightening of the clamps and, therefore, excessive shore deflection under load. ASTM Specification D9-87 defines a bow in a piece of lumber as a deviation in a direction perpendicular to the flat face from a straight line from end to end of the piece. The bows recorded for the posts of the collected shore samples ranged from 0 to 19 mm (0.75 in.) with an average value of 2.5 mm (0.1 in.).

Another factor that can result in premature failure of a shore is damaged or improperly prepared shore ends. Fig. 5 shows the deteriorated and nonflat end of a highly reused shore. If the ends of a shore are not flat and at right angles to the longitudinal axis of the shore, after installation the shore ends will not be in full contact with the supports, which can result in premature crushing or splitting of the shore ends. Fig. 6 shows premature crushing of an imperfect shore end.

**FIG. 4. Shore Sample with Large Knot Hole**

**FIG. 5. Deteriorated Shore with Nonflat End**

## SHORE STRENGTH

For design purposes, the Ellis clamp manufacturer provides tables that specify safe shore spacings for various slab thicknesses and joist and stringer sizes. Table 1 shows the recommended member spacings for a formwork with Ellis shores up to 3.6 m (12 ft) long, 100 × 150 mm (4 × 6 in.) stringers, 100 × 100 mm (4 × 4 in.) joists, and 19 mm (0.75 in.) thick plywood. Assuming normal-weight concrete for slab, 480 Pa (10 psf) dead weight for formwork, and the maximum slab live load of 3.6 kPa (75 psf) specified by ACI 347R-88 (''Recommended'' 1988), shore loads corresponding to the shore tributary areas given in Table 1 range from 18 to 21 kN (4,050 to 4,700 lb). In other words, the maximum design load recommended by the shore manufacturer for shores up to 3.6 m (12 ft) long is 21 kN (4,700 lb).

To determine the load bearing capacities of the shore samples, they were tested according to the guidelines recommended by Scaffolding, Shoring and Forming Institute (''Recommended'' 1986). These guidelines recommend that sample installation simulate field conditions. The samples were divided into four length groups: 2.4, 2.9. 3.5, and 4 m long groups. To study the effect of out-of-plumb installation on shore performance, each shore length group was randomly divided into three sets; a set was tested plumb, a set 1.5° out-of-plumb, and a set 3° out-of-plumb. Samples were installed out-of-plumb about both shore cross-section axes. All samples were loaded in compression until failure. Samples were tested at 8% moisture content.

Table 2 shows the statistics of the observed shore ultimate axial loads. The wide range of the observed axial load capacities shows that if Ellis shores are used properly they can resist loads over five times the manufacturer's recommended safe





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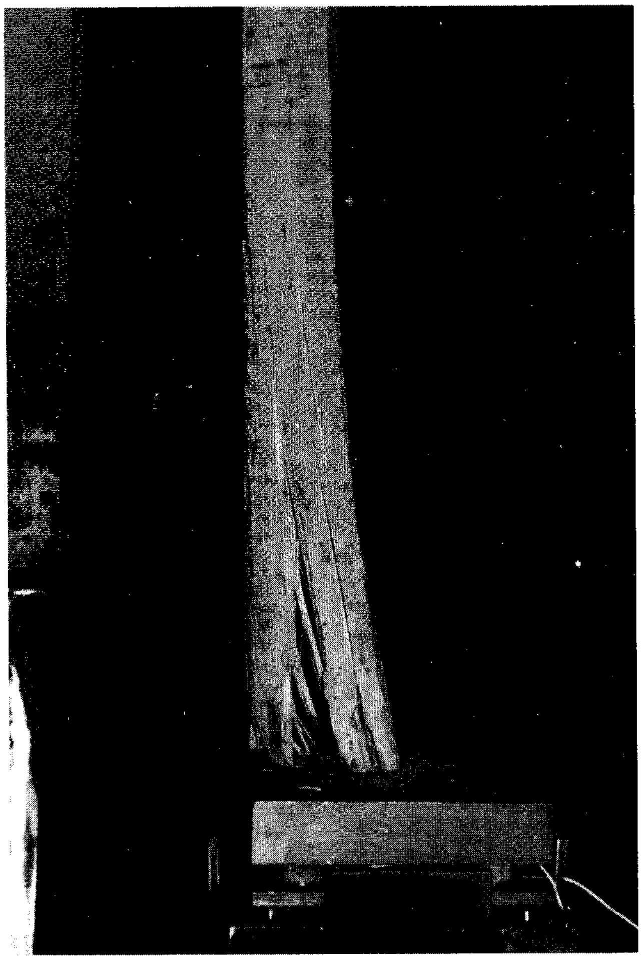

FIG. 6. Crushing of Defective Shore End

TABLE 1. Recommended Formwork Design (*Ellis* 1986)

| Slab thickness (mm) (1) | Formwork Member Spans[a] Stringer[b] (m) (2) | Joist[c] (m) (3) | Plywood[d] (m) (4) |
|---|---|---|---|
| ≤100 | 1.5 | 1.8 | 0.6 |
| 127–178 | 1.5 | 1.5 | 0.6 |
| 203–241 | 1.3 | 1.5 | 0.6 |
| 250–304 | 1.2 | 1.5 | 0.5 |
| 330–355 | 1.2 | 1.3 | 0.5 |

[a]Shores up to 3.6-m-tall and braced as required.
[b]100 × 150 mm Douglas fir construction grade.
[c]100 × 100 mm Douglas fir construction grade.
[d]19-mm-thick, placed with face grain parallel to span.

load. On the other hand, damaged or improperly installed shores cannot resist even half of the recommended design load. ACI 347R-88 (''Recommended'' 1988) requires that shores be installed plumb. A comparison of the mean shore capacities in Table 2 shows that, on average, the mean capacities of shores installed 1.5° out of plumb is 83% of those installed plumb. The average of the mean capacities of the shores installed 3° out of plumb is 63% of the average of the mean capacities of plumb shores. An investigation of the observed shore capacities for various length groups showed that normal distribution best represents the data.

Shores that failed prematurely had defects such as large knots, knotholes or clusters of smaller knots, badly deteriorated material, excessively large bows, nonflat and damaged ends, and cracked or twisted clamps. Excluding samples with visible defects and considering only shores tested plumb results in shore bearing capacity statistics shown in Table 3. This table shows a substantial performance improvement compared with the statistics given in Table 2. The minimum observed shore capacity in Table 3 is approximately three times the maximum recommended design load. Therefore, contractors who ensure plumb shore installation and continuously discard defective shores substantially improve their formwork safety.

Excluding samples with visible defects and considering only shores with out-of-plumb angles up to 1.5° results in the shore bearing capacity statistics shown in Table 4. The minimum observed shore capacity in Table 4 is approximately twice the maximum recommended design load.

TABLE 2. Statistics of Observed Shore Capacities

| Shore: Length (m) (1) | Shore: Angle[a] (degree) (2) | Ultimate Axial Load: Minimum (kN) (3) | Maximum (kN) (4) | Mean (kN) (5) | COV (6) |
|---|---|---|---|---|---|
| 2.4 | 3 | 38 | 123 | 85 | 0.26 |
| | 1.5 | 33 | 119 | 68 | 0.30 |
| | 3 | 42 | 90 | 69 | 0.22 |
| 2.9 | 0 | 59 | 115 | 89 | 0.18 |
| | 1.5 | 53 | 78 | 65 | 0.11 |
| | 3 | 13 | 74 | 53 | 0.40 |
| 3.6 | 0 | 28 | 92 | 59 | 0.36 |
| | 1.5 | 33 | 77 | 54 | 0.24 |
| | 3 | 6 | 46 | 32 | 0.43 |
| 4 | 0 | 28 | 62 | 50 | 0.24 |
| | 1.5 | 28 | 64 | 45 | 0.31 |
| | 3 | 14 | 49 | 29 | 0.42 |

[a]Out-of-plumb angle.

TABLE 3. Statistics of Shore Capacities Excluding Defective Shores; All Shores Installed Plumb

| Length (m) (1) | Minimum (kN) (2) | Maximum (kN) (3) | Mean (kN) (4) | COV (5) |
|---|---|---|---|---|
| 2.4 | 63 | 123 | 90 | 0.19 |
| 2.9 | 59 | 115 | 88 | 0.18 |
| 3.6 | 60 | 92 | 71 | 0.16 |
| 4.0 | 38 | 62 | 53 | 0.17 |

TABLE 4. Statistics of Shore Capacities Excluding Defective Shores; Out-of-Plumb Angle ≤ 1.5°

| Length (m) (1) | Minimum (kN) (2) | Maximum (kN) (3) | Mean (kN) (4) | COV (5) |
|---|---|---|---|---|
| 2.4 | 43 | 123 | 80 | 0.26 |
| 2.9 | 53 | 115 | 80 | 0.22 |
| 3.6 | 44 | 92 | 64 | 0.20 |
| 4.0 | 28 | 64 | 48 | 0.27 |

## STRINGER PHYSICAL CONDITION

One hundred stringer samples were obtained for laboratory testing; 10–15 samples from each site visited. All samples were 100 × 150 mm (4 × 6 in.), 4.88 m (16 ft) long lumber. The grade marks of most samples were not legible due to reuse. Among 17 samples with legible grade marks, two samples were Douglas fir select structural, 14 were Douglas fir No. 1, and one was Douglas fir No. 2. These samples did not belong to a single project. There was not a considerable difference between physical conditions of the samples from various projects. No decayed areas or saw cuts were observed on the surfaces of the samples. Eight samples had 1–4 holes drilled through their depths. The diameter of these





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holes ranged from 10 to 20 mm (0.4 to 0.8 in.). The maximum observed surface check depth was 29.6 mm (1.16 in.) with a length of 0.28 m (11 in.). Ten samples had checks deeper than 15 mm (0.6 in.) with lengths ranging from 0.35 to 1.5 m (14 to 60 in.). The maximum check depths in 14 members were in the range of 5–15 mm (0.2–0.6 in.) with lengths ranging from 0.2 to 0.9 m (8 to 36 in.). In the middle half of the length, 19 samples had 1–2 knots larger than 38 mm (1.5 in.) and 63 samples had 1–6 knots smaller than 38 mm (1.5 in.). Oven dry specific gravity of 10 randomly selected samples ranged from 0.45 to 0.53, with a mean value equal to 0.476.

## STRINGER STRUCTURAL PROPERTIES

Laboratory experiments were conducted to determine stringer structural properties in bending and shear. All experiments were conducted according to the specifications and guidelines recommended by ASTM D198-84. Samples were tested at moisture contents ranging from 11 to 15%.

Among the flexural properties of the used stringers, moduli of rupture and moduli of elasticity were determined. The mean and COV of the observed moduli of rupture were 42.4 MPa (6.1 ksi) and 0.38, respectively. Some of the results of an in-grade testing program conducted by the Forest Products Laboratory (Evans and Green 1987) for 50 × 200 mm Douglas-Fir Larch lumber in new condition are given in Table 5. The observed stringer moduli of rupture statistics are very close to those shown in Table 5 for Douglas-Fir Larch No. 1 in new condition. This shows that reuse has not substantially affected the bending strength of stringers. An investigation of various probability models showed that among normal, lognormal, and Weibull distributions, normal distribution best represents the observed data. Fig. 7 shows some of the stringer samples after failure in bending.

The moduli of elasticity determined for the samples had a mean value of 11,472 MPa (1,664 ksi) and a COV of 0.26. The mean and COV determined in the forementioned in-grade testing program for 50 × 200 mm Douglas-Fir Larch lumber in new condition are given in Table 6. The mean value of the observed moduli of elasticity for used stringers is also comparable to that determined for Douglas-Fir Larch No. 1 in new condition. A statistical analysis of the observed moduli of elasticity showed that among normal, lognormal, and Weibull distributions, normal distribution best fits the data.

**TABLE 5. Modulus of Rupture for 50 × 200 mm Douglas-Fir Larch Lumber, 12% MC (*Ellis* 1986)**

| Grade (1) | Modulus of Rupture Mean (MPa) (2) | Modulus of Rupture COV (3) |
|---|---|---|
| Select structural | 58.5 | 0.30 |
| No. 1 | 42 | 0.38 |
| No. 2 | 43.1 | 0.43 |

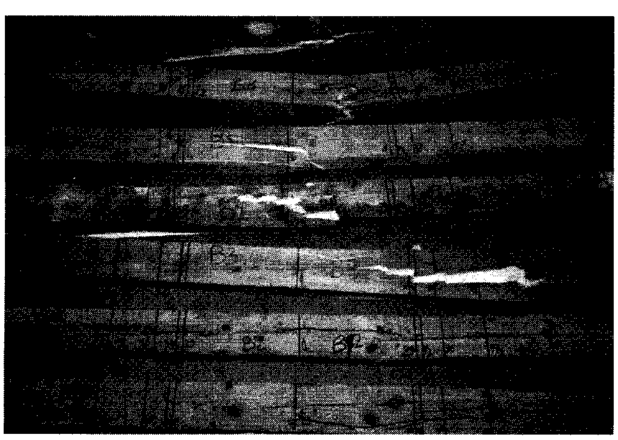

**FIG. 7. Stringer Samples after Failure in Bending**

**TABLE 6. Modulus of Elasticity for 50 × 200 mm Douglas-Fir Larch Lumber, 12% M.C. (*Ellis* 1986)**

| Grade (1) | Modulus of Elasticity Mean (MPa) (2) | Modulus of Elasticity COV (3) |
|---|---|---|
| Select structural | 13,160 | 0.21 |
| No. 1 | 11,078 | 0.20 |
| No. 2 | 11,189 | 0.25 |

For determining shear strengths of stringer samples, 0.56 m (22 in.) long sections of the samples were tested according to ASTM D198-84 guidelines. The mean and COV of the observed stringer shear strengths were 5,860 kPa (850 psi) and 0.2, respectively. An examination of the observed results showed that normal distribution also provides the best fit to the shear strength data.

## PHYSICAL CONDITION OF JOISTS

Eighty-eight joist samples were received from the construction sites that were visited. Eight samples were 4.88 m (16 ft) long and 80 samples were 2.44 m (8 ft) long. All samples were 100 × 100 mm (4 × 4 in.) nominal size lumber. Only nine of the samples had legible grade marks; one sample was Douglas fir select structural, six Douglas fir No. 1, and two Douglas fir No. 2. These samples did not belong to a single project. Some of the samples were in good physical condition and some were highly reused and damaged.

Among the samples collected, four had split ends 300–400 mm (12–16 in.) long. The deepest observed surface check was 23 mm (0.9 in.) with a length of 1.22 m (4 ft). The surface check depths were greater than 15 mm (0.6 in.) in nine samples, and less than 7 mm (0.3 in.) in 27 samples. The maximum knot size in the middle half of the joist lengths was equal to or greater than 38 mm (1–1.5 in.) in 28 samples and less than 25 mm (1 in.) in the rest of the samples.

Since plywood sheathing is nailed to the joists, each reuse adds a few nail holes to a joist face, In addition to nail holes, used joists may also have drilled holes, crushed faces, or broken edges due to the application of hammer and levers during installation and dismantling. Four of the samples had 1–3 drilled holes 12.5 mm (0.5 in.) in diameter. Two samples had decayed areas on their surfaces. Eight samples had saw marks; one saw mark was about 20 mm (0.8 in.) deep; the rest were 1–7 mm (0.05–0.3 in.) deep. The saw marks were along the width of the samples.

## STRUCTURAL PROPERTIES OF JOISTS

Strengths and stiffnesses of the samples in flexure and shear were determined. All tests were conducted according to ASTM D198-84 recommendations. Sample moisture contents at the time of tests varied from 11.5 to 14.5%.

The observed moduli of rupture for the joist samples had a mean value of 49.5 MPa (7.2 ksi) with a COV of 0.36. Fig. 8 shows some of the joist samples after failure in bending. The moduli of rupture determined for 50 × 100 mm Douglas-Fir Larch lumber in the in-grade testing program conducted by the Forest Products Laboratory (Evans and Green 1987) are shown in Table 7. The mean value of the observed moduli of rupture is comparable to that of standard grade Douglas-Fir Larch lumber in new condition. Considering that the samples with legible grade marks were No. 2 and better, repeated reuse





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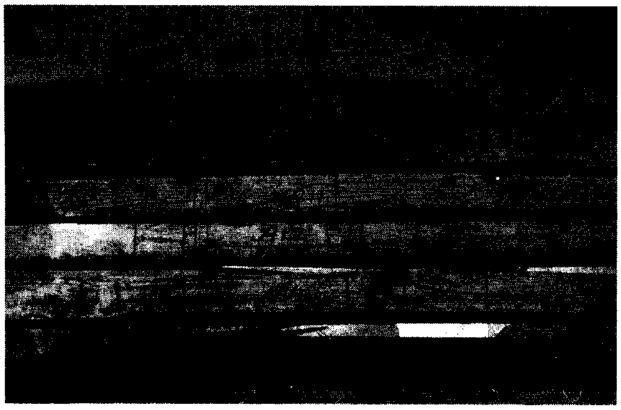

FIG. 8. Joist Samples after Failure in Bending

TABLE 7. Moduli of Rupture for 50 × 100 mm Douglas-Fir Larch Lumber 12% M.C. (*Ellis* 1986)

| Grade (1) | Modulus of Rupture: Mean (MPa) (2) | Modulus of Rupture: COV (3) |
|---|---|---|
| No. 2 | 58.7 | 0.375 |
| Construction | 57.1 | 0.370 |
| Standard | 52.0 | 0.450 |
| Utility | 43.1 | 0.530 |

TABLE 8. Moduli of Elasticity for 50 × 100 mm Douglas-Fir Larch Lumber, 12% M.C. (*Ellis* 1986)

| Grade (1) | Modulus of Elasticity: Mean (MPa) (2) | Modulus of Elasticity: COV (3) |
|---|---|---|
| No. 1 | 11,700 | 0.22 |
| No. 2 | 11,300 | 0.27 |
| Construction | 10,400 | 0.25 |
| Standard | 10,000 | 0.31 |

has deteriorated the bending strength of the joists. The low moduli of rupture values belong to badly deteriorated members and members with large knots, saw cuts, drilled holes, and broken or crushed faces. Normal, lognormal, and Weibull distributions were investigated for representing the observed data. Both quantitative and graphical methods showed that Weibull distribution best fits the data.

The mean and COV of the observed moduli of elasticity of joist samples are 11,270 MPa (1,630 ksi) and 0.31, respectively. The mean and COV determined in the aforementioned Forest Products Laboratory in-grade test program for 50 × 100 mm (2 × 4 in.) Douglas fir are shown in Table 8. The foregoing results show that the mean value of the observed joist moduli of elasticity is very close to that for Douglas-Fir Larch No. 2 in new condition. This shows that joist flexural stiffness is also reduced due to repeated reuse. The observed joist moduli of elasticity are best represented with a normal distribution.

The mean and COV of the observed shear strengths of the joist samples were 7.77 MPa (1.13 ksi) and 0.18, respectively. An investigation of the observed shear strength data showed that normal distribution best summarizes the data.

## SUMMARY AND CONCLUSIONS

Wooden formwork systems are currently designed based on the criteria developed for the structural design of wood in new condition. To adjust the current design criteria for the effect of material reuse and installation inaccuracies and development of a framework of probability-based design criteria for formwork, data on the actual formwork member, formwork resistances, and installation workmanship are required. In this paper, the results of a comprehensive investigation of wooden slab formwork structural properties and member installation inaccuracies were presented. The presented data are based on laboratory experiments on shore, stringer, and joist samples obtained from eight multistory concrete building projects located in Chicago, Illinois, Milwaukee, Wisconsin, and Minneapolis-St. Paul Minnesota.

Among the factors that affect formwork strength is its installation workmanship. The observed shore out-of-plumb angles in the eight project sites visited ranged from 0 to 2.7° with a mean value equal to 0.76°. The ratios of the observed to design spans had a mean and COV equal to 0.95 and 0.15 for stringers, 0.98 and 0.07 for joists, and 1.0 and 0.23 for plywood sheathing, respectively.

The physical condition of the 104 shore samples that were collected varied from highly reused and deteriorated to shores in relatively new condition. Among the 23 shores with legible wood grade marks, two were Douglas fir No. 2 and the rest were Douglas fir No. 1 or better. To investigate the effects of shore length and out-of-plumb installation on shore strength, the samples were divided into four length groups. Each length group was also divided into three sets; a set was tested plumb, a set 1.5° out-of-plumb, and a set 3° out-of-plumb. The mean values of the ultimate loads observed for shores tested plumb were 85, 89, 59, and 50 kN for 2.4, 2.9, 3.6, and 4 m long shore groups, respectively. The mean value of the bearing capacities of shores tested 1.5° out-of-plumb was 83% of the mean bearing capacity of plumb shores. The mean bearing capacity of shores tested 3° out-of-plumb was 63% of the mean capacity of plumb shores.

The very low observed shore bearing capacities belonged to defective shores installed 3° out-of-plumb. The shore defects that substantially affected shore capacities included deteriorated material, low wood grade, defective clamps, large initial bow, and imperfect shore end conditions. Samples that were badly deteriorated due to repeated reuse, samples with decayed areas, and samples with large knots or knotholes failed prematurely, especially if they were installed out-of-plumb. Defective clamps do not allow proper alignment of the top and bottom parts and cannot be tightened adequately during installation, which results in early failure of the shore. Large initial bends in shores results in excessive deflection and premature failure of clamps. If the end of a shore is not flat and at a right angle to the longitudinal axis of the shore, after installation the shore end will not be in full contact with the support. This causes failure of the shore by crushing or splitting of its end.

The observed shore capacities ranged from 6 to 123 kN (1,288 to 27,650 lb). This wide range of shore bearing capacities shows that a defective and out-of-plumb shore can have a dangerously low capacity; on the other hand, shores in good condition when installed plumb can resist loads over 120 kN (27 kips). By excluding visibly defective samples, the set of shores that were tested plumb had a minimum strength three times the maximum Ellis shore design load. The minimum strength of samples without major visible defects, installed plumb or at most 1.5° out-of-plumb, was twice the maximum Ellis shore design load.

One hundred stringer samples were tested in bending and shear. Among the 17 samples with legible grade marks, two were Douglas fir select structural, 14 were Douglas fir No. 1, and one was Douglas fir No. 2. Samples were in relatively good condition; no decayed areas or raw cuts were observed on the surfaces of the samples. In the middle half of the sample



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length, 19 samples had one to two knots larger than 38 mm (1.5 in.). Ovendry specific gravity of 10 randomly selected samples had a mean value of 0.476.

The mean of COV of the observed stringer moduli of rupture were 42.4 MPa (6.1 ksi) and 0.38, respectively. The mean and COV determined for the observed moduli of elasticity were 11,472 MPa (1,664 ksi) and 0.26, respectively. The observed shear strength of the samples had a mean value equal to 5,860 kPa (850 psi) and a COV equal to 0.2. Normal probability distribution provided the best fit to the observed stringer bending and shear properties.

Eighty-eight joist samples were tested in bending and shear. Nine samples with legible grade marks were all Douglas fir; one sample was Select Structural, six were No. 1, and two were No. 2. Some of the samples were highly reused and in poor physical condition; two samples had decayed areas, four had spilt ends 300–400 mm (12–16 in.) long. Eight samples had saw marks on their surfaces ranging in depth from 1 to 20 mm (0.05 to 0.8 in.). The maximum knot size in the middle half of the sample length was equal to or greater than 38 mm (1.5 in.) in 25 samples, between 25 and 38 mm (1–1.5 in.) in 28 samples, and less than 25 mm (1 in.) in the rest of samples.

The mean and COV of the observed moduli of rupture for joist samples were 49.5 MPa (7.2 ksi) and 0.36, respectively. Sample moduli of elasticity had a mean and COV equal to 11,268 MPa (1,630 ksi) and 0.31, respectively. The shear strengths determined for the samples had a mean value equal to 7.8 MPa (1.13 ksi) and a COV of 0.18. The moduli of elasticity and shear strength data were best represented by normal distribution. Weibull distribution provided the best fit to the observed joist moduli of rupture.

## ACKNOWLEDGMENTS

This study was sponsored by the National Science Foundation under Grant MSM-9002571. This study would not have been possible without the cooperation of the construction companies that allowed access to their construction sites for sample collection. These companies include: Adjustable Joist Company, Minnetonka, Minnesota; Adolfson and Peterson, Inc., Minneapolis, Minnesota; The CECO Concrete Construction, Chicago, Illinois, and Minneapolis, Minneasota offices; J. H. Findorf and Sons, Inc., Madison, Wisconsin; Gateway Concrete Forming, Mequon, Wisconsin; Kraus Anderson, Minneapolis, Minnesota; M. A. Mortenson Company, Minneapolis, Minnesota; Morse Diesel, Chicago, Illinois; Pepper Construction Company, Chicago, Illinois; and C. G. Schmidt Construction Company, Milwaukee, Wisconsin.

## APPENDIX. REFERENCES

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