RELIABILITY-BASED DESIGN FOR JAPANESE TIMBER STRUCTURES USING CANADIAN S-P-F DIMENSION LUMBER

By

MASATOSHI TOMOI
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ABSTRACT

Reliability levels of Japanese 2x4 wood frame structures were evaluated using lumber property data derived from evaluation of Canadian Spruce-Pine-Fir dimension lumber. The evaluations were made using the "Standard for Limit States Design of Steel Structures (Draft)", which was newly published by the LRFD Subcommittee of Architectural Institute of Japan, and In-Grade Data obtained by a Canadian Wood Council research project. These analyses were implemented using the computer program "RELAN" developed by Dr. R.O. Foschi at UBC and Monte Carlo simulations. Reliability levels of current Japanese 2x4 wood frame structures were also evaluated. Recommendations were made to encourage the application of limit states design into existing Japanese design methods.

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LIST OF ABBREVIATIONS

AIJ	Architectural Institute of Japan
CMHC	Canada Mortgage and Housing Corporation
COV	Coefficient of Variation
FFPRI	Forestry and Forest Products Research Institute
GHLC	Government Housing Loan Corporation
JAS	Japanese Agricultural Standard
LRFD	Load and Resistance Factored Design
LSD	Limit States Design
MOAFF	Ministry of Agricultural, Forestry and Fisheries
MOC	Ministry of Construction
NLGA	National Lumber Grades Authority
RBD	Reliability-Based Design
S-P-F	Spruce-Pine-Fir
SS	Select Structural
WSD	Working Stress Design

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1. INTRODUCTION

Limit States Design (LSD) codes incorporating safety assessments based on modern Reliability-Based Design (RBD) principles are rapidly replacing the traditional Working Stress Design (WSD) philosophy in structural materials design codes throughout the world. The transformation to the LSD or the Load and Resistance Factor Design (LRFD) format has largely been led by the steel, concrete and other non-wood materials groups. Interest in developing LSD timber design codes is expanding rapidly in order to keep timber structures design on a basis compatible with the other major structural materials.

The Canadian Code for Engineering Design in Wood [1] was the first timber code to be converted to the LSD format. The 1984 revision was largely a soft conversion of the WSD code with the exception that new sawn lumber materials property data was incorporated into the LSD revision. The 1989 revision of the Canadian Code [2] incorporates new design equations calibrated to provide minimum target safety levels derived using formal reliability assessment procedures. Applying reliability assessment procedures requires knowledge of actual loads (e.g., occupancy, snow, wind, earthquake), member structural behavior models and appropriate materials strength data as well as the corresponding member design equations for the strength and serviceability limit states. This information is required for the analysis of the member reliability and the relationship between member safety and the performance factor chosen for each design equation. The performance factor in the design equations determines the level of safety associated with the design. The reliability assessment framework and implementation methodology (Foschi [3]) developed for the Canadian LSD Code can be applied in other jurisdictions once the design equations are specified and when the appropriate load data and materials properties information are available.

The Canadian forest products industry is a major exporter of structural wood products. The continued acceptance of these products on an equitable basis in the United States, Europe, Japan, Australia and other markets is a major concern to the forest industry. The more or less common international WSD philosophy was a very significant benefit in helping the Canadian industry expand the use of Canadian timber products internationally. The WSD methodology was traditionally based on design properties developed from small clear specimens. The test methods, procedures for data analysis to convert test properties to working stresses were consistent in most countries with the exception of the choice of safety factors. The general consistency in design property development allowed sharing of test data. The common design property development concepts were readily understood within the technical community.

LSD timber codes are being investigated in several of Canada's important timber markets including the United States and the European Community. As a major exporting country Canada has a particular interest in working with the research community and codes committees in order to achieve as much uniformity in LSD code and support standards development as possible.

LSD philosophy provides a rational framework for specifying member safety which allows all structural materials to be compared on an equitable basis. This presents a significant challenge to the timber community to demonstrate that the traditional solid sawn visually graded timber structural members, systems and structures have adequate safety in relation to the competing non-wood and emerging engineered wood based products. Since the safety assessment system directly recognizes effects of material variability, products with lower variability will have an inherent advantage over higher variability materials. Initial concerns, within the timber community, about the long-term competitive position of wood-based structural materials being affected by the adoption of the LSD code framework still continue. Recently, with the expanding development of LSD timber codes, it is become evident that lack of consistent approaches in development of codes and standards could significantly impact lumber exporting countries such as Canada. New full-size material test methods, data interpretation procedures, development of new member resistance models coupled with attempts to simplify design codes has created a need for the Canadian dimension lumber industry to mount additional efforts to support development of consistent internationally accepted standards. With lack of attention to these developments the industry will risk significant losses in structural efficiency and product value for Canadian structural wood products. The traditional products such as visually graded nominal 2-inch dimension lumber are under particularly intense scrutiny in many market areas.

The Canadian forest products industry has been successful in promoting the use of the North American 2x4 platform construction system in Japan. The 2x4 wood frame system was officially adopted in Japan in 1974 when the Ministry of Construction (MOC) published the "2x4 Building Code". At the same time, the Ministry of Agriculture, Forestry and Fisheries (MOAFF) established the Japanese Agricultural Standard (JAS) for approval of Canadian dimension lumber for the 2x4 wood frame system. The JAS standard for dimension lumber closely parallels the National Lumber Grades Authority (NLGA) dimension lumber grading rules used by the Canadian 2-inch dimension lumber producers.

Japanese 2x4 wood frame structures are currently designed using WSD principles. For the convenience of architects, engineers and builders the Government Housing Loan Corporation (GHLC) publishes span tables [4] and a design specification manual for the 2x4 wood frame system. Since large amounts of Canadian dimension lumber are used in these structures, it is important to begin to understand how a code transformation from WSD to LSD will impact the use of dimension lumber in the Japanese market.

The Architectural Institute of Japan (AIJ) has studied the reliability of steel structures. The Load and Resistance Factor Design (LRFD) Subcommittee of the AIJ issued the "Standard for Limit States Design of Steel Structures (Draft)" [5] in February 1990. The draft provides LRFD design equations with associated target reliability levels for strength and serviceability limit states. Publication of the draft LSD Steel Standard provided the opportunity to investigate reliability of Japanese timber structures using the LRFD design philosophy proposed for steel structures.

The reliability of the Japanese 2x4 wood frame construction system can be studied using reliability assessment and implementation methodologies developed in Canada (Foschi [3]). While most Japanese wooden structures are built using the traditional post and beam system the material property information required for reliability studies of these structures is currently lacking. However, the Japanese 2x4 wood frame system is designed using Canadian nominal 2-inch dimension lumber for which material property data is available. Therefore this study focuses on (1) evaluating relationships between safety levels and member performance factors for single members designed according to the LSD code philosophy and achieving the same target safety levels proposed for the draft LSD Steel Standard and (2) assessing the safety levels associated with members designed according to the current WSD code for 2x4 wood frame structures. The results of the study will provide the first indication of the potential impact of adopting an LSD code philosophy for timber structures in Japan.

Reliability studies undertaken in this study are exclusively based on material property information for two grades (Select Structural (SS) and No.2) and three sizes (2x4, 2x8 and 2x10) of Spruce-Pine-Fir (S-P-F) nominal 2-inch dimension lumber. Since this species group is the most widely used in 2x4 wood frame structures in Japan these results will yield the preliminary information on which to base an analysis of the potential impact of LSD on 2x4 wood frame housing design in Japan.

2. OBJECTIVES

The general objectives of this study are to apply RBD principles to assess safety levels for Japanese 2x4 wood frame construction systems, designed using criteria from the draft LSD Steel Standard [5], and to assess the reliability levels associated with current 2x4 wood frame construction. Specific objectives of the study are as follows:

1) To review the Japanese building code and the structural calculation system as it applies to 2x4 wood frame construction.

2) To develop and present material property data for Canadian S-P-F dimension lumber on the basis required by Japanese building codes.

3) To derive load models for dead, occupancy and snow loads appropriate for analysis of 2x4 structures in Japan.

4) To study the reliability levels for bending, tension and compression members using design criteria taken from the Japanese draft LSD Steel Standard.

5) To derive performance factors for 2x4 wood frame construction to yield the target safety levels chosen for the draft LSD Steel Standard.

6) To derive duration of load adjustment factors for 2x4 wood frame construction using Japanese load models.

7) To assess the reliability of current Japanese 2x4 wood frame structures using S-P-F material property data.

3. CURRENT STRUCTURAL CALCULATION SYSTEM FOR JAPANESE TIMBER STRUCTURES

3.1 BUILDING CODES AND STANDARDS

With the exception of the strength data, the entire study was based on the Japanese Building Codes and Standards. The Canadian Code requirements are different from those of the Japanese. The following discussion highlights the major elements of the Japanese building code system related to 2x4 wood frame structures.

3.1.1 BUILDING STANDARD LAW

This law mainly provides fundamental requirements for buildings in general [6]. The contents of this law are similar to the National Building Code of Canada [7]. However, the National Building Code of Canada is valid only after acceptance by local authorities. Whereas in Japan, generally speaking, the Building Standard Law is valid throughout Japan at all times.

3.1.2 BUILDING STANDARD LAW ENFORCEMENT ORDER

Building Standard Law Enforcement Orders [8] address more specific details in buildings such as strength properties and design requirements. These Orders provide more complete guidelines for enforcement authorities and designers.

3.1.3 BUILDING NOTIFICATION

Building notifications are supplements for the Building Codes. There are two types of Building Notifications, 1) Building Notification for the Public and 2) Building Notification for Special Administrative Agency. The former Building Notification is a supplement to the Building Standard Law and Building Standard Law Enforcement Order. There are many Building Notifications for the Public. Each notification deals with one specific topic. For example, the so-called "2x4 Building Code" [9] is one of many such notifications. The latter Building Notification is a Notification from the Director of the Ministry of Construction (MOC) to the local Special Administrative Agency which has appointed building officials take charge of the affairs concerning building confirmation. For example, after new methods or new materials are approved by the Minister of Construction, or if there is a Building Code revision, a Building Notification is issued. For the 2x4 wood frame construction, the Building Notification for Special Administrative Agency [10] requires either simplified structural member checking using GHLC's span tables or detailed member structural calculations.

3.1.4 STANDARD FOR STRUCTURAL CALCULATION OF TIMBER STRUCTURES

Current structural design of timber structures is based on WSD. This standard published by AIJ [11] is similar to the Canadian Code for Engineering Design in Wood (Working Stress Design)[12]. Detailed design equations and strength properties for lumber and glue-laminated lumber are specified in this standard. The Japanese Building Standard Law was revised in 1987 to allow construction of higher and larger wooden structures. The Standard for Structural Calculation of Timber Structures was revised in 1988.

3.1.5 GHLC SPAN TABLE

GHLC is similar to the Canada Mortgage and Housing Corporation (CMHC). GHLC publishes specifications and span tables [4] for 2x4 wood frame construction, based on WSD criteria [11]. Design properties for lumber for 2x4 wood frame construction, load data and design criteria required for calculation of member spans include:

Allowable Unit Stresses for 2x4 Lumber and Structural Glue-Laminated Lumber;

Deflection Limits;

Unit Shear Resistance for Common Nail;

Nominal Dead Load;

Nominal Occupancy Load for Residential Building; and

Nominal Snow Load

which are given in the GHLC's span tables.

Span tables are based on simply supported beam analysis. No composite action between the framing members and sheathing is considered except for glued floors.

3.2 PRINCIPLES OF STRUCTURAL CALCULATION FOR TIMBER STRUCTURES

3.2.1 DESIGN REQUIREMENTS

Structural design requirements specified in Japanese Building Codes vary depending on the type, size and height of the structure. Generally speaking, timber structures are classified into one of the following five categories:

General Traditional Wooden Structures (mainly housing)

2x4 Wood Frame Structures (mainly housing)

Log Construction

Heavy Timber Structures

Special Structures.

Japanese Building Codes have requirements for the design of individual members and for the analysis and design of the complete structure. Depending upon the type, size and height of the structure, wooden structures shall comply with some or all of the requirements of the items which are shown in Table 1. Generally speaking, for the common residential 2x4 wood frame structures of less than three stories, individual members are designed by using GHLC's span table or alternatively by structural calculation according to the Standard for Timber Design. The entire structure must satisfy the effective wall length requirement explained in 3.2.5.

3.2.2 DESIGN LOADS

Loads and external forces are specified in the Building Standard

Law Enforcement Order. The load cases to be considered in design are shown in Table 2. Unlike the WSD methods of Canada and the US which recognized different duration of load factors for the various load cases, the Japanese system has only two load duration categories: sustained (normal duration) loading and temporary (short-term duration) loading. When temporary loads are considered, allowable unit stresses of lumber and structural glue-laminated timber become double those of sustained loads.

3.2.3 ALLOWABLE UNIT STRESS

Allowable unit stresses for lumber and structural glue-laminated timber are specified in the Building Standard Law Enforcement Order, Building Notifications and/or AIJ's "Standard for Structural Calculation of Timber Structures".

Since this study primarily focuses on 2x4 wood frame structures, the allowable unit stresses of lumber for 2x4 wood frame construction specified in Building Notification for Special Administrative Agency [13] are listed in Appendix 1.

3.2.4 WORKING STRESS DESIGN CRITERIA

Generally speaking, structural calculations to determine the individual size of structural wooden members shall be based on the following equations:

3.2.4.1 BENDING

$$\sigma_b = \frac{M}{Z_e} \leq C_f \cdot f_b \qquad (3.1)$$

where

σ_b : bending stress;

M : bending moment;

Z_e : effective section modulus;

f_b : allowable bending stress;

C_f : size factor for glued-laminated timber determined by the following formula but not less than 1.0. The factor is not applied for solid lumber, therefore C_f = 1.0 for solid lumber,

$$C_f = \left(\frac{30}{d}\right)^{\frac{1}{9}} \qquad (3.2)$$

where d is the depth of the member in cm.

3.2.4.2 SHEAR

$$\tau = \alpha \cdot Q \;/\; A_e \leq f_s \qquad (3.3)$$

where

τ : shear stress;

f_s : allowable shear stress;

α : shape factor (rectangular cross section, α = 1.5);

Q : shear force;

A_e : effective area of cross section.

3.2.4.3 TENSION

1) Tension Parallel to the Grain

$$\sigma_t = \frac{T}{A_e} \leq f_t \qquad (3.4)$$

where

σ_t : tensile stress;

f_t : allowable tensile stress;

T : axial tension force parallel to grain;

A_e : effective area of cross section;

2) Tension Perpendicular to the Grain

In areas where a tension perpendicular to the grain is generated, an appropriate reinforcement should be given in order to avoid an excessive stress in this direction. The allowable tensile stress perpendicular to the grain is assumed to be 1/3 of the allowable shear stress.

3.2.4.4 COMPRESSION

1) Compression Parallel to the Grain

$$\sigma_c = \frac{N}{A} \leq f_k \qquad (3.5)$$

where

σ_c : compression stress;

N : axial compression force parallel to the grain;

A : area of cross section;

f_k : allowable compression stress obtained as follows:

if $\lambda \leq 30$: $f_k = f_c$ (3. 6)

if $30 < \lambda \leq 100$: $f_k = f_c\ (1.3 - 0.01\ \lambda)$ (3. 7)

if $100 < \lambda$: $f_k = 0.3\ f_c\ /\ (\lambda/100)^2$ (3. 8)

where

f_c : allowable short-member compression stress;

λ : slenderness ratio of the compression member $\lambda = \frac{L_e}{i_e}$;

where

L_e : effective length;

i_e : radius of gyration of the column with respect to the axis of buckling $i_e = \sqrt{\frac{I}{A}}$

where

I : moment of inertia.

2) Compression Perpendicular to Grain

$$\sigma_c = \frac{N}{A} \leq f_{c\perp} \quad \text{or} \quad f'_{c\perp} \qquad (3.9)$$

where

$f_{c\perp}$: allowable compression stress perpendicular to the grain;

$f'_{c\perp}$: allowable bearing stress;

A : support (bearing) area;

N : compression force perpendicular to grain.

3.2.5 LATERAL RESISTANCE FOR RESIDENTIAL WOODEN STRUCTURES

The "Effective Wall Length Methods" are applied for the structural calculation of lateral loads for common residential wooden structures. This method has two components: "Required Ratio of Effective Wall length p" and "Resistance Factor of Bearing wall q". The parameter p has two values, i.e. p_e for seismic and p_w for wind force analysis. Both p and q for the 2x4 wood frame structures are specified in the Building Notification [9].

The following requirement should be satisfied for lateral resistance for common residential wooden structures:

$$pA = \sum q_i L_i \tag{3. 10}$$

where

pA : larger of either $p_e A_f$ or $p_w A_p$;

p_e : required ratio of effective wall length for seismic force;

A_f : floor area;

p_w : required ratio of effective wall length for wind force;

A_p : plumb measure area;

L_i : real length of the bearing wall with resistance factor q_i.

Appendix 2 shows measurement of the plumb measure sizes (vertically projected area) for the span or ridge direction of a bearing wall.

The required ratios of effective wall length p_e and p_w, and resistance factors for bearing wall q are given in the Building

Standard Law Enforcement Order for traditional post and beam structures and Building Notification for 2x4 wood frame structures. The parameters p_e and p_w were determined from the analysis of common residential buildings subjected to seismic and wind force. Resistance factors for bearing walls q, based on racking tests for bearing walls with different sheathing materials, are given in the Enforcement Order and Building Notification. Generally speaking, the load at shear strain versus resistance of 1/120 radian for traditional post and beam structures and 1/300 radian for 2x4 wood frame structures correspond to their allowable strengths.

It is important to note that although there are requirements to check structural safety against lateral loads such as wind and earthquake, most structural member sizes are determined by the gravity loads for residential wooden structures including 2x4 wood frame structures. Therefore only gravity loads were considered in the analysis of structural member in this study.

4. JAPANESE LIMIT STATES DESIGN

There are two organizations in Japan that are similar to the Canadian Civil Engineering Society. The Architectural Institute of Japan (AIJ), and the Japan Society of Civil Engineering (JSCE). The JSCE deals with major construction projects such dams, bridges, and highways. The AIJ deals with buildings, from local housing to large skyscrapers. Currently, the AIJ has been considering adoption of the LSD approach for steel and reinforced concrete structures in Japan. An LSD standard for reinforced concrete structures has already been published by the JSCE in 1986 however it does not apply to the regular buildings.

4.1 NEW JAPANESE STANDARD FOR LIMIT STATE DESIGN OF STEEL STRUCTURE (DRAFT)

The LRFD Subcommittee of AIJ published the "Standard for Limit State Design of Steel Structures (Draft)" [5] in February 1990. Prior to the draft standard, the Steel Structures Subcommittee of AIJ issued the "Load and Resistance Factor Design for Steel Structures (Proposal)" [14] in March 1986 to examine acceptability of RBD in steel structures. Although the draft LSD Steel Standard has not officially been accepted, the content would be the guideline for the Japanese RBD criteria. The following requirements adopted from the draft LSD Steel Standard are used and referred to in this study.

4.2 PROPOSED LOAD COMBINATION AND LOAD FACTORS

The LSD equation generally consists of the load effects multiplied by load factors and the resistance to load expressed as a product of some chosen specified strength (conventionally, the lower 5th percentile for lumber) and a performance factor ϕ. Typical design criteria is expressed as:

$$\text{Factored Resistance} \geq \text{Effect of factored load} \qquad (4.1)$$

In the general case, the total load effect is a linear combination of individual load effects which are related to the factored resistance according to,

$$\phi R_n \geq \sum_{i=1}^{n} \gamma_i Q_i \qquad (4.2)$$

where

ϕ : performance factor;

R_n : specified strength;

γ_i : individual load factor;

Q_i : individual load.

The proposed effect of factored loads and load combinations for strength limit states for steel structures with the associated target reliability index at performance factor $\phi = 0.9$ are given as:

$$1.3D_n \qquad \beta = 2.5 \qquad (4.3)$$

$$1.1D_n + 1.6L_n \qquad \beta = 2.5 \qquad (4.4)$$

$$1.1D_n + 1.6S_n + 0.6L_n \qquad \beta = 2.0 \qquad (4.5)$$

$$1.1D_n + 2.0E_n + 0.4L_n \qquad \beta = (1.5) \qquad (4.6)$$

$$1.1D_n + 1.6W_n + 0.6L_n \qquad \beta = 2.0 \qquad (4.7)$$

$$0.9D_n - 1.6W_n \qquad \beta = 2.0 \qquad (4.8)$$

Following load combinations shall be also considered in heavy snow areas,

$$1.1D_n + 1.5W_n + 0.5S_n + 0.4L_n \qquad \beta = 2.0 \qquad (4.9)$$

$$1.1D_n + 1.7E_n + 0.4S_n + 0.4L_n \qquad \beta = (1.75) \qquad (4.10)$$

Effect of factored loads and load combination for serviceability limit states with the associated target reliability index at performance factor $\phi = 0.9$ shall be taken as:

$$1.0D_n + 1.0L_n \qquad \beta = 1.0 \qquad (4.11)$$

$$1.0D_n + 0.9S_n + 0.6L_n \qquad \beta = 1.0 \qquad (4.12)$$

$$1.0D_n + 0.4E_n + 0.4L_n \qquad \beta = 1.0 \qquad (4.13)$$

$$1.0D_n + 0.9W_n + 0.6L_n \qquad \beta = (-0.35) \qquad (4.14)$$

$$1.0D_n - 0.9W_n \qquad \beta = 1.0 \qquad (4.15)$$

Following loads combinations shall also be considered in heavy snow areas,

$$1.0D_n + 0.9W_n + 0.5S_n + 0.4L_n \qquad \beta = 1.0 \qquad (4.16)$$

$$1.0D_n + 0.4E_n + 0.4S_n + 0.4L_n \qquad \beta = (0.4) \qquad (4.17)$$

where

D_n : nominal dead load;

L_n : nominal live (occupancy) load;

S_n : nominal snow load;

W_n : nominal wind load;

E_n : nominal earthquake load.

Basic statistical data (means and coefficients of variation of lognormal distribution) assumed for the normalized load variables and the normalized material resistance in the draft LSD Steel Standard are shown in Table 3. The β - ϕ relationships of four combinations in steel design were derived using the basic statistical data obtained from Table 3 for two ratios of nominal dead to nominal live load (γ = 2.0 and 0.25) and two coefficients of variation of material strength (cov = 0.15 and 0.2) as shown in Figure 1.

5. JAPANESE FULL SIZE TEST PROGRAM FOR STRENGTH OF LUMBER

Japanese strength properties used for structural design of wood are derived from small clear wood specimens. Only a few full size test results are available. Among them, the following two full size test reports review results of the Japanese full size lumber test programs:

1) Forestry Agency, Study on the Stress Grading in Structural Lumber, Report No.25, 1985. [15]

2) Strength of Timber and Wood Based Structural Group, Japan Wood Research Society, Structural Lumber - Collection and Analysis of Strength Data, 1988. [16]

The first report [15] summarizes results of bending tests of square sections proposed by the Forest Agency and Forestry and the Forest Products Research Institute (FFPRI) which were carried out by the nine prefectural Research Institutes throughout Japan.

All tests were performed using the same testing procedure. The size of specimen was 10.5 x 10.5 x 300 cm. The third point load was applied with 270 cm test span. Deflection at mid-span was measured when a specified load was applied at the lumber yard in green moisture condition. After air drying, a bending test was conducted on the testing machine to obtain bending strength (MOR) and modulus of elasticity (MOE).

The second report [16], the Strength of Timber and Wood Based Structural Group of the Japan Wood Research Society compiled and analyzed the full size lumber test data which had been carried out by the 21 Research organizations. Since a full size test procedure is not standardized, these research organizations did the tests differently. The collected data was adjusted as described in the following sections. Although there is no standard for full size tests for lumber, procedures described in the following sections are considered to be reasonable in this study.

5.1 BENDING

A one-third point loading system shall be used. The maximum strength affecting defect shall be randomly placed on the tension side between the test span.

5.2 MOE

MOE values shall be adjusted with a span depth ratio of 21 to 1 under an assumed uniform load as described in ASTM D2915 [17].

5.3 MOISTURE CONTENT

MOE and MOR values shall be adjusted at target moisture content (MC) of 15%, as described in ASTM D 2915. The adjustment should not be applied where the difference of moisture content is larger than five percentage points from the chosen value. The ASTM D 2915 moisture adjustments are made:

$$P_2 = P_1 \ (\ \alpha \ - \ \beta \cdot M_2 \ ) \ / \ (\ \alpha \ - \ \beta \cdot M_1 \ ) \qquad (5.1)$$

where

P_1 : original strength property at moisture content M_1;

P_2 : target strength property at moisture content M_2;

α and β are coefficients given in ASTM D 2915.

5.4 MATERIAL STRENGTH

Building Standard Law Enforcement Order specifies material strengths of lumber and glued-laminated lumber. None of the publications mention that material strength values are based on lower 5th percentile values of the material property distribution. However AIJ's Standard for Timber Design explains that the limited Japanese in-grade test results showed the material strengths are usually less than or approximately equivalent to the 5th percentile of available data sets. Generally speaking, allowable unit stresses for timber are derived by simply dividing material strength by 3.0 for sustained load and 1.5 for temporary load.

5.5 DURATION OF LOAD ADJUSTMENT

The test machine shall be adjusted so that failure occurs several minutes after loading starts. Since allowable unit stresses are correlated with either sustained load or temporary load, data need not be adjusted using the safety and duration of load factors provided in ASTM D2555 [18].

5.6 TENSION

There is no specific full size tension testing procedure available in Japan.

5.7 COMPRESSION

There is no specific full size compression testing procedure available in Japan.

6. CANADIAN FULL SIZE TEST PROGRAMS

Traditionally, strength properties of Canadian visually graded lumber have also been determined by testing small clear wood specimens. In the late 70's, large scale in-grade tests were conducted to find the mechanical properties of full-size, on-grade Canadian visually stress-graded lumber sampled from production. The tests conducted were mostly in bending with limited tension parallel to the grain evaluations. More than 55,000 full size samples were tested. At this time, the proof loading concept was introduced to estimate lower 5th-percentile values for a range of size/grade and species combinations without breaking the entire test samples. The bending and tension results were used to derive new design properties, which were included in the CAN3-086-M84 version [1 and 12].

Although the aforementioned in-grade tests were adequate for characterizing the traditional strength properties of lumber, i.e., the average bending modulus of elasticity, and the lower 5th-percentile exclusion values of strength, a more detailed second phase of a major lumber research program was undertaken from 1983 to 1985. The major reason for further testing was to provide information required for the probabilistic LSD format.

The major species groups of Douglas Fir-Larch, Hem-Fir and S-P-F with three sizes 2x4, 2x8 and 2x10 and nine minor species with three sizes of 2x4, 2x6 and 2x8 were tested.

The in-grade tests were conducted to establish bending strength, bending modulus of elasticity, tension parallel to the grain and compression parallel to the grain strength according to ASTM D 4761 [19].

The third-point load was applied on the bending specimen with span to depth ratio of 17: 1. The maximum strength-reducing defect was randomly located within the span for the bending test. The gauge lengths of 2462mm for 2x4, 3683mm for 2x8 and 3683mm for 2x10 were selected for the tension test. The gauge lengths of 2438mm for 2x4, 3658mm for 2x8 and 4267mm for 2x10 were selected for the compression test. Compression specimens were laterally restrained so that the test results provide short column strength properties.

The results of these tests were included in CAN/CSA-086.1-M89 version [2 and 20].

7. ADJUSTMENTS OF CANADIAN IN-GRADE DATA TO JAPANESE BASIS

In order to use Canadian test results in this study, appropriate adjustments were necessary to compensate for the differences between Canadian and Japanese testing methods and data analyzing procedures.

7.1 MOR

The Canadian bending strength test data were obtained using the span to depth ratio of 17 to 1 and adjusted to 15% moisture content. The parameters for Normal, Lognormal, 2 Parameter Weibull and 3 Parameter Weibull distributions were developed for the strength data. Each data set was then truncated at the 15th percentile, and these lower tail data were fitted with the same four distribution types. Parameters shown in Appendix 3 and 4 were obtained using 100% data and using lower 15% data.

7.2 MOE

Test MOE values were measured using the displacement of the loading cross-head and a span to depth ratio of 17 : 1. These data were subsequently adjusted to yield MOE values which would be derived using a full span yoke to measure midspan deflections in accordance with ASTM D 2915 procedures. For Japanese code requirements, these data were further adjusted to a span to depth ratio of 21 : 1. The obtained data were already adjusted to the target moisture content of 15 percent and MOE stroke to MOE yoke with span to depth ratio of

17: 1. The following formulae [21] were developed to adjust to span to depth ratio of 17 : 1 to that of 21 : 1 for this study.

$$\frac{1}{E_{17}} = \frac{1}{E_{TLP}} - 0.00591\ H \times 10^{-6} \qquad (7.1)$$

$$\frac{1}{E_{21}} = \frac{1}{E_{TLP}} - 0.00298\ H \times 10^{-6} \qquad (7.2)$$

where

E_{17} : MOE_{yoke} at span to depth ratio of 17 : 1;

E_{TLP} : $\mathrm{MOE}_{loading-head}$ at span to depth ratio of 17 : 1;

E_{21} : MOE_{yoke} at span to depth ratio of 21 : 1;

H : nominal depth of the specimen.

The 2 Parameter Weibull distribution parameters of the adjusted MOE distribution are shown in Appendix 5.

7.3 TENSION

Since there is no Japanese test standards for full size tension tests of lumber, no adjustment was applied for the CWC tension data. The data for tension strength was taken from Reliability-Based Design of Wood Structures Structural Research Series [3]. Parameters for 2P Weibull with lower 15% fits are shown in Appendix 6.

7.4 COMPRESSION

No adjustments were applied to the compression in-grade test results for the same reason as the tension. The data for compression strength was also taken from Reliability-Based Design of Wood Structures Structural Research Series. Parameters for 2P Weibull distribution with lower 25% fits are shown in Appendix 7.

8. DEVELOPMENT OF LOAD MODEL AND LOAD PARAMETERS FOR JAPANESE BUILDINGS

8.1 DEAD LOAD

Because the same values of the nominal load were used in the LSD and WSD for steel, they too were utilized in this study for the 2x4 wood frame construction.

The design dead load is based on the average weight of materials. The detailed information of dead load is available in the AIJ's "Recommendations for Building Design, Load" [22].

Since this study was intended to evaluate the reliability levels for Japanese 2x4 wood frame structures, the design load values specified in the GHLC's span table used for this study are listed below.

for Floor Joists:	
Tatami Mat	18 kgf/m^2 (177 N/m^2)
Plywood Sheathing(15mm)	10 kgf/m^2 (98 N/m^2)
Gypsum Board	15 kgf/m^2 (147 N/m^2)
for Rafters:	
Plywood Sheathing(9mm)	6 kgf/m^2 (59 N/m^2)
Plywood Sheathing(12mm)	8 kgf/m^2 (78 N/m^2)
Light Roofing Material	20 kgf/m^2 (196 N/m^2)
Clay tile	60 kgf/m^2 (588 N/m^2)

for Lumber:

2 x 4	4 kgf/m (39 N/m)
2 x 6	5 kgf/m (49 N/m)
2 x 8	6 kgf/m (59 N/m)
2 x 10	8 kgf/m (78 N/m)
2 x 12	9 kgf/m (88 N/m)

For this study, the normalized dead load random variable $d = D/D_n$, where D is dead load (random variable) and D_n is the design dead load, was assumed Normally distributed with a mean of 1.0 and standard deviation of 0.1.

8.2 OCCUPANCY LOAD

Occupancy loads are assumed to be the superposition of two live load processes: sustained and extraordinary as shown in Figure 2. The magnitude of both the sustained and extraordinary components were assumed distributed according to a Gamma distribution. The period between changes are assumed as Poisson processes. The following load statistics, taken from the draft LSD Steel Standard, were used to model occupancy loads for this study.

Sustained Load:

mean	65 kgf/m^2 (637.4 N/m^2)
cov	0.40
mean return	8 years

Extraordinary Load:

mean	45 kgf/m^2 (441.3 N/m^2)
cov	0.55
mean return	1 year
duration of loading	impulse

For the sustained load, the load magnitude is modeled using the Gamma distribution:

$$f(x) = \frac{\lambda(\lambda x)^{k-1}}{\Gamma(k)} e^{-\lambda x} \qquad (8.1)$$

where:

$$\text{mean} = \frac{k}{\lambda} \; ; \qquad (8.2)$$

$$\text{standard deviation} = \sqrt{\frac{k}{\lambda^2}} \qquad (8.3)$$

The parameters of k and λ can be calculated as:

$$k = \frac{\text{mean}^2}{(\text{standard deviation})^2} = \frac{65^2}{(65\times 0.42)^2} = 6.25 \qquad (8.4)$$

$$\lambda = \frac{\text{mean}}{(\text{standard deviation})^2} = \frac{65^2}{(65\times 0.42)^2} = 9.615\times 10^{-2} \qquad (8.5)$$

and the duration of the load is modeled by the Exponential distribution as:

$$f(t_s) = \lambda e^{-\lambda t} \qquad (8.6)$$

where

$$\overline{t_s} = \frac{1}{\lambda} \; ; \qquad (8.7)$$

$$\lambda = \frac{1}{\overline{t_s}} = \frac{1}{70080} = 1.427\times 10^{-5} \qquad (8.8)$$

(8 years = 70080 hours).

Similarly, the magnitude of extraordinary live load can be modeled by the Gamma distribution in Eq.8.1 and the parameters k and γ are given by:

$$k = \frac{45}{(45 \times 0.55)^2} = 3.306 \qquad (8.9)$$

$$\lambda = \frac{45}{(45 \times 0.55)^2} = 7.346 \times 10^{-2} \qquad (8.10)$$

and Exponential distribution in Eq.8.6 for its time between events as:

$$\lambda = \frac{1}{8760} = 1.142 \times 10^{-4} \qquad (8.11)$$

Maximum occupancy loads were determined for a 50 year load simulation. Five thousand realizations of maximum sustained load plus extraordinary load for the 50-year period were generated by the Monte Carlo simulation. The upper 10% of the maximum load data were fitted (Figure 3) using Extreme Type I (Gumbel) distribution model:

$$Q_{50} = B + \frac{(-\ln(-\ln p))}{A} \qquad (8.12)$$

The parameter B for the 50-year model was adjusted for 8-year return periods according to:

$$Q_8 = B_8 + \frac{(-\ln(-\ln p))}{A} \qquad (8.13)$$

where

$$B_8 = \left(B - \frac{\ln 50}{A} + \frac{\ln 8}{A}\right) \qquad (8.14)$$

The parameters for 50-year return were used in reliability analysis for strength limit states and the parameters for 8-year return were used for reliability analysis for serviceability limit states.

These parameters were used to derive the normalized random variable $q = Q/Q_n$, where Q is either Q_{50} or Q_8 and Q_n is the design live load of 180 kgf/m^2 (1.765 KN/m^2) for residential building given in Building Standard Law Enforcement Order. Therefore, $q = Q/Q_n = Q/180$ can be expressed as:

$$q = B^* + \frac{(\ -\ln\ (-\ln\ p)\ )}{A^*} \qquad (8.15)$$

where

B^* : B / 180

A^* : $A \times 180$

where B is given by Eq.8.12 or Eq.8.13 for the 50 year and 8 year return load models respectively.

These parameters were calculated for 50 year and 8 year return loads which are shown in Table 4.

8.3 SNOW LOAD

In Japan, geographical locations are designated either light (general) snow areas or heavy snow areas. The light snow area is defined as an area with 50-year return snow height less than 100 cm. Locations with 50-year return snow heights greater than or equal to 100 cm are defined as heavy snow area.

The annual maximum snow height and annual average snow duration in Sapporo, Niigata, Tokyo and Osaka were obtained from the local meteorological observatories. Sapporo and Niigata belong to the heavy snow areas, whereas Tokyo and Osaka belong to the general snow areas. Figure 4 and 5 are samples of the snow data showing a snow height and an average annual snow duration with the same scale in Sapporo and Tokyo for six successive years.

Design snow load S_n, on a roof is expressed in the AIJ's "Recommendations for Building Design, Snow Load" [23] and draft LSD Steel Standard as the product of a series of factors:

$$S_n = \rho \cdot Z_s \cdot E_s \cdot C_r \qquad (8.16)$$

where

ρ : unit weight of snow

2.1 kg/m^2/cm (20.6 N/m^2/cm) for heavy snow area

2.0 kg/m^2/cm (19.6 N/m^2/cm) for light snow area

Z_s : 50-year return height of snow accumulation (cm);

E_s : environment factor;

C_r : roof shape factor.

The snow load distribution considered corresponds to those for the maximum in a period of 50 years.

The annual maximum ground snow height is represented by an Extreme Type I (Gumbel) distribution:

$$G = B - \frac{(-\ln(-\ln p))}{A} \qquad (8.17)$$

where A and B are model parameters.

A corresponding distribution of maximum snow height in N years can be expressed as:

$$G = B - \frac{\ln N - \ln(-\ln p)}{A} \qquad (8.18)$$

where p is a probability of non-exceedance and A and B are parameters of the Type I distribution.

The 50-year return snow height G_{50}, corresponding to a probability of non-exceedance of 49/50, can be obtained using Eq. 8.17. Also using Eq.8.17 and Eq.8.18, the normalized $g = G/G_{50}$ can be expressed as:

$$g = B^* + \frac{(-\ln(-\ln p))}{A^*} \qquad (8.19)$$

where

$$B^* = \frac{AB + \ln N}{AB + 3.9019} \qquad (8.20)$$

$$A^* = AB + 3.9019 \qquad (8.21)$$

These parameters were calculated for 50 year and 8 year return load. The parameters, the average annual snow duration and the design snow loads are shown in Table 5. The location, annual snow duration and design snow load for this study are shown in Figure 6. 50-year return value was assumed in reliability analysis for strength limit states and 8 year-return value was assumed in reliability analysis for the serviceability limit states, the same as occupancy load.

The variability of the environment, roof shape and snow density factors in Eq.8.16 should be considered in the calculation of the snow load from annual maximum snow height. However, this study assumed that those factors were constant because of lack of available information. Therefore the normalized snow load s = S/S_n is defined as:

$$s = \frac{S}{S_n} = \frac{G}{G_{50}} \qquad (8.22)$$

where

S	:	snow load;
S_n	:	design snow load;
G	:	maximum snow height;
G_{50}	:	50-year return snow height.

9. PERFORMANCE FACTORS FOR JAPANESE 2x4 WOOD FRAME STRUCTURAL MEMBERS

In general, the reliability level β and corresponding different performance factors ϕ, can be generated using the RELAN program [24] given the appropriate performance function G and required statistical data. The performance functions G are formulated using a specific design equation. Therefore the performance factor ϕ at the given target reliability β_T is obtained from the results of a β - ϕ analysis.

In this chapter, design equations and target reliability levels β_T from the draft LSD Steel Standard, strength data from CWC's research project and the aforementioned statistical data for loads are used to derive the performance factors at the given target reliability β_T for typical S-P-F members used in the 2x4 wood frame structures.

Since the gravity load cases i.e, dead, occupancy and snow load, govern in many practical design situations and are considered to be of fundamental importance in the calibration work, the dead load and occupancy load case for bending, and the dead load and snow load cases for bending, tension and compression were considered in this study.

9.1 STRENGTH LIMIT STATES

Effects of factored loads and the specific load combination for evaluation of the β - ϕ relationship for strength limit states are obtained from the draft LSD Steel Standard. Analyses are performed for floor and flat roof member designs.

For floor joists, effects of dead and occupancy load will be compared with the factored resistance using the design equation

$$1.1\ D_n + 1.6\ L_n \leq \phi\ R_{0.05} \qquad (9.1)$$

For rafters, effects of dead load and snow load will be evaluated using:

$$1.1\ D_n + 1.6\ S_n \leq \phi\ R_{0.05} \qquad (9.2)$$

where

D_n : effect of design dead load;

L_n : effect of design live load;

S_n : effect of design snow load;

ϕ : performance factor;

$R_{0.05}$: specified strength.

The relation of β - ϕ can be calculated using the performance function G. For strength limit states, the performance function G for joists and rafters can be formulated as:

$$G = R - (D + L) \qquad (9.3)$$

for floor joist and

$$G = R - (D + S) \qquad (9.4)$$

for rafters in flat roof,

where

R : strength (a random variable);

D : effect of the dead load (a random variable);

L : effect of the occupancy load (a random variable);

S : effect of the snow load (a random variable).

By substitution of the appropriate design equation, the performance functions can be expressed as:

$$G = R - \frac{\phi\ R_{0.05}}{1.1\gamma + 1.6}\ (d\gamma + l\ ) \qquad (\ 9.\ 5\ )$$

for joist and

$$G = R - \frac{\phi\ R_{0.05}}{1.1\gamma + 1.6}\ (d\gamma + s\ ) \qquad (\ 9.\ 6\ )$$

for rafter.

where

γ	:	D_n/L_n for joists,
		D_n/S_n for rafters;
d	:	D/D_n;
l	:	L/L_n;
s	:	S/S_n.

9.1.1 EFFECT OF LOAD RATIO γ

The deterministic value $\gamma = D_n/L_n$ or $\gamma = D_n/S_n$ was required as a load related input for the computation of β - ϕ relations using Eq.9.5 or Eq.9.6.

A ratio γ = 0.25 was chosen for initial safety studies in this project. In order to validate the choice, actual γ_a values were calculated using GHLC's span table requirements. The ratio γ = 0.25 was appropriate in the heavy snow area and for occupancy load. However in the light snow area γ was higher than 0.25 as shown in Table 6 and Table 6a. The effect of the choice of γ on safety was studied for ϕ =

0.8, 0.9 and 1.0 in Sapporo and the same ϕ-values in Tokyo for 2x8 No.2 grade. Results in Figure 7 show that increasing the ratio $\gamma = D_n/S_n$ tends to reduce safety levels, if strength distribution was assumed as a Weibull distribution; therefore subsequent analysis of the actual values of γ for heavy roofing γ_a is also studied.

9.1.2 RESISTANCE DISTRIBUTION MODEL

The choice of the resistance distribution model affects the β - ϕ relationship. Figure 8 shows β - ϕ relations derived using four different resistance distribution models fitted to the complete data sets. Figure 8 shows the influence of the resistance distribution model on the β - ϕ relations when the distribution models are fit to the complete data sets. Foschi et al. showed that the influence of the distribution model was significantly reduced when the models were fitted to lower tail resistance data [3]. Figure 9 compares the 2 Parameter Weibull distribution fits to the lower 15% of the data and entire data sets. Figure 10 compares the fitted distribution to the data for cumulative probabilities less than 0.3. The model fitted to entire data range does not fit the data well at the lower percentiles. In order to avoid these problems, the resistance distribution models were fitted to the lower 15 percent of the data (15% truncation). Figure 11 shows the variation in the β - ϕ relationship for four distribution types when the distribution parameters are determined by fitting to the lower 15% of the data. All four distribution models were used to analyze β - ϕ relationships for each size and grade combination. In most cases, the 2 Parameter Weibull distribution tended to give results following the average trend for the four distributions. Thus, the β - ϕ results obtained using 2 Parameter Weibull distribution and 15% truncation were used in subsequent analysis.

Results in Figure 12 and 13 show that the data fitting strategy also affects the reliability ranking of data sets in a β - ϕ analysis. From Figure 12 using lower 15% data, the visual lumber grade No.2 has a higher reliability index than SS, while Figure 13 showed the opposite result when using the entire data set to calculate distribution parameters.

9.1.3 BENDING PERFORMANCE FACTOR

Performance factors are tabulated for four snow loads and the occupancy load case using $\gamma = 0.25$ and the actual γ_a for the load case taken from Tables 6 and 6a. Table 7 summarizes ϕ-values corresponding to three target β-values ($\beta_T = 3.0$, 2.5 and 2.0) for the selected size/grade combinations for S-P-F. The results show the lower β-values corresponding to given ϕ-values in Tokyo and Osaka which have significant differences between $\gamma = 0.25$ and the actual γ_a values.

Figure 14 shows the average β - ϕ trends for five load cases at $\gamma = 0.25$. Figure 15 shows the average β - ϕ trends for five load cases at the actual γ_a. Table 8 gives the approximate mean β-values corresponding to specific ϕ values at $\gamma = 0.25$ and the actual γ_a.

As explained in draft LSD Steel Standard, $\beta = 2.5$ is targeted for a load combination of dead load plus occupancy load (Eq.4.4) at ϕ =

0.9 and β = 2.0 for a load combination of dead plus snow plus live load in (Eq.4.5) at the same ϕ = 0.9. In this analysis, live load was assumed 0 for rafters of the residential building. If Eq.4.5 from the draft LSD Steel Standard is used for S-P-F lumber, Table 8 shows the resulting ϕ-value at β_T = 2.0 would be larger than 1.0. Alternatively if we adopt Eq.9.2 which requires β_T = 2.5 then ϕ = 0.95 for γ = 0.25 and ϕ = 0.9 for actual γ_a values. Therefore the design equation (Eq.9.2) with ϕ = 0.9 (β_T = 2.5), has been adopted for the dead plus snow load combination for rafter design. For this case Fig. 1 shows that 2x4 wood frame structures have comparable ϕ - β relationships to steel structures.

Although a current Japanese design methods do not apply size effect adjustments to design strength properties of lumber (size effects are applied to glued-laminated lumber for depth more than 30 cm), size effect adjustments are required to simplify the presentation of the design strength properties for dimension lumber. The size effect adjustment equation relates member strength to member dimensions according to

$$\frac{\sigma_1}{\sigma_2} = \left(\frac{H_2 \, L_2}{H_1 \, L_1} \right)^{\frac{1}{k}} \qquad (9.7)$$

where σ_1 and σ_2 are the strength corresponding to the lengths L_1 and L_2 and to the depths H_1 and H_2 and k is the parameter determining the magnitude of the size effect.

Using this Eq.9.7, then Eq.9.1 can be expressed as

$$1.1\ D_n + 1.6\ L_n = \phi_0\ R_0 \left(\frac{H_0 L_0}{H\ L}\right)^{\frac{1}{k}} \qquad (9.8)$$

also Eq.9.2 can be expressed as

$$1.1\ D_n + 1.6\ S_n = \phi_0\ R_0 \left(\frac{H_0 L_0}{H\ L}\right)^{\frac{1}{k}} \qquad (9.9)$$

where ϕ_0 is the performance factor associated with R_0 which is the characteristic bending strength for a standardized 2x8 beam having a depth H_0 = 184 mm and a length L_0 = 3000 mm; H and L are the actual depth and length of the member being evaluated; and k is the size factor parameter.

The performance factor ϕ_0, the strength R_0 and parameter k are obtained using least squares techniques to minimize the function:

$$F = \sum_i \sum_j \sum_l \left\{ \phi_{ijl}\ R_{(0.05)ij} - \phi_0\ R_{0i} \left(\frac{H_0\ L_0}{H_j\ L_j} \right)^{\frac{1}{k}} \right\}^2 \qquad (9.10)$$

where

i = 1,2 (one species, two grades);

j = 1,..,3 (three sizes);

l = 1,..,5 (five loading conditions).

The minimization was carried out using the actual ϕ values for γ = 0.25 and actual γ_a at the β = 2.5 and adopting ϕ_0 = 0.9 as shown in Table 7. The test spans for bending (span/depth ratio of 17:1) are 1511 mm for 2x4, 3131 mm for 2x8 and 3994 mm for 2x10. The bending size effect parameter determined in the minimization was approximately 4.5 in both γ cases. Adjusted characteristic strength values R_0, are shown in Table 9 with the corresponding non-parametric 5th percentiles for 2x8's.

The safety index β values associated with the design Eqns. 9.8 and 9.9 were evaluated for the case $\gamma = 0.25$ and the actual γ_a, with $\phi_0 = 0.9$ using the R_0 and size parameters k given in Table 9. The average β calculated for the locations and size grade combinations in Table 7 was approximately 2.5 for $\gamma = 0.25$ but did not achieve the target $\beta_T = 2.5$ for the actual γ_a. The safety assessment was repeated with $\phi_0 = 0.85$ for the case $\gamma = 0.25$ and $\phi_0 = 0.85$ and 0.8 for the actual γ_a. Table 10a shows the results of this analysis for $\phi_0 = 0.9$ and 0.85 when $\gamma = 0.25$. Results obtained when the actual γ_a is used are given in Table 10b for $\phi_0 = 0.85$ and 0.80.

According to the Table 10a, for $\phi_0 = 0.9$ and $\gamma = 0.25$ the mean β is 2.49 with a range from 2.31 to 2.69. For $\phi_0 = 0.85$ and using the actual γ_a the mean β is 2.48 with a range from 2.13 to 2.80, and the mean β is 2.59 with a range from 2.25 to 2.92 at $\phi_0 = 0.8$ for the actual γ_a in Table 10b.

For bending strength limit states Eq.9.8 is recommended for dead plus occupancy load and Eq.9.9 is recommended as the design checking relationship for dead plus snow load. The specified strength referenced to the 2x8 size are given in Table 9. Using $k = 4.5$, $\phi_0 = 0.85$ and the actual γ_a yields average safety indices β of approximately $\beta = 2.5$. If the choice of $\gamma = 0.25$ and the same $\phi_0 = 0.85$ are adopted for calibration then the average safety indices β can be increased.

9.1.4 TENSION

Reliability levels for tension members were calculated using the procedures previously described for bending members. However only one load case dead load plus snow load was considered. The reliability levels for the four snow locations were studied using the material resistance distribution derived by fitting the 2 Parameter Weibull to the lower 15% of the test data.

Table 11 summarizes ϕ-values corresponding to three target β-values (β_T = 3.0, 2.5 and 2.0) for selected size/grade combinations for four snow loads cases at γ = 0.25 and the actual γ_a for the location. The calculated performance factors are not significantly affected by the choice of γ. However ϕ-values in Tokyo and Osaka are slightly lower where the actual γ_a values are greater than 1.

Figure 16 shows the average β - ϕ trends for four load cases at γ = 0.25. Table 12 gives the approximate mean β-values corresponding to specific ϕ values for γ = 0.25 and the actual γ_a at each location. Average β-values were not affected by the choice of γ.

The minimization was carried out assuming a standardized 2x8 tension member having a depth H_0 = 184 mm and a length L_0 = 3000 mm; using results shown in Table 11 for β_T = 2.5 with ϕ_0 = 0.9 for γ = 0.25 and the actual γ_a. The gauge lengths of 2462mm for 2x4, 3683mm for 2x8 and 3683mm for 2x10 were used for the tension strength tests. The resulting tension size effect parameters were 8.9 for γ = 0.25 and 9.5 for the actual γ_a. Adjusted characteristic strength values R_0 for the lumber grades SS and No.2 are shown in Table 13 with the corresponding non-parametric 5th percentiles for 2x8's.

Eq.9.9 is adopted for design checking for the tension strength limit state. Actual β's for the four load cases shown in Table 14a and Table 14b are derived using ϕ_0 = 0.90 and 0.85 for γ = 0.25 and the actual γ_a with the corresponding specified strengths R_0 from Table 13.

According to the Table 14a, the mean β is 2.50 (range from 2.25 to 2.72) at ϕ_0 = 0.9 and the mean β is 2.61 (range from 2.39 to 2.83) at ϕ_0 = 0.85 for γ = 0.25. The mean β is 2.50 (range from 2.21 to 2.77) at ϕ_0 = 0.9 for the actual γ_a and the mean β is 2.61 with a range from 2.34 to 2.88 at ϕ_0 = 0.85 for the actual γ_a from Table 14b.

Eq.9.9 is recommended for design checking for the tension strength limit state with k = 9.5, ϕ_0 = 0.90 and the adjusted 2x8 strength R_0 for the actual γ_a from Table 13. When γ = 0.25, then k = 8.9, ϕ_0 = 0.90 when used with the adjusted 2x8 strength R_0 for γ = 0.25 from Table 13. In all cases this design equation would lead to reliability levels comparable to steel structure.

9.1.5 COMPRESSION

The approach adopted tension reliability studies was applied for reliability level analysis for compression members. Analyses were performed for the four load cases (dead load plus snow load) used for tension studies. Compression members are considered to be fully restrained against buckling (i.e. short columns). Resistance parameters were obtained by fitting a 2 Parameter Weibull to the lower 25% of the test data.

Table 15 summarizes ϕ-values corresponding to three target β-values (β_T = 3.0, 2.5 and 2.0) for selected size/grade combinations for four snow loads γ = 0.25 and the actual γ_a. The results do not show significant differences with the choice of γ.

Figure 17 shows the average β - ϕ trends for four load cases at γ = 0.25. Table 16 gives the approximate mean β-values corresponding to specific ϕ values at γ = 0.25 and the actual γ_a.

The minimization was carried out assuming for a standardized 2x8 compression members having a depth H_0 = 184 mm and a length L_0 = 3000 mm; using results shown in Table 15 for β = 2.5 with ϕ = 0.9 at γ = 0.25 and the actual γ_a. The gauge lengths of 2438mm for 2x4, 3658mm for 2x8 and 4267mm for 2x10 were used for the compression test. The resulting compression size effect parameters were approximately 8.3 in γ = 0.25 and 8.7 in the actual γ_a, and adjusted characteristic strength values R_0 are shown in Table 17 with the corresponding non-parametric 5th percentiles for 2x8's.

Using Eq.9.9 for design with ϕ_0 = 0.9 and 0.85 for γ = 0.25 and the actual γ_a, and the adjusted characteristic values R_0, the safety index β are shown in Table 18a and Table 18b for size, grade and load combination.

According to the Table 18a and Tale 18b, the mean β is 2.51 with a range from 2.15 to 3.11 at ϕ_0 = 0.9 and the mean β is 2.65 with a range from 2.30 to 3.25 at ϕ_0 = 0.85 for γ = 0.25, the mean β = 2.51 with a range from 2.09 to 3.15 at ϕ_0 = 0.9 and the mean β = 2.66 with a range from 2.23 to 3.29 at ϕ_0 = 0.85 for the actual γ_a.

Based on this analysis, it is recommended that k = 8.7, ϕ_0 = 0.9 with the adjusted 2x8 strength R_0 for the actual γ_a, or k = 8.3, ϕ_0 = 0.9 with the adjusted 2x8 strength R_0 for γ = 0.25 would be comparable with steel structures.

9.2 SERVICEABILITY LIMIT STATES

Effects of factored load and load combination for serviceability limit states are obtained from the draft LSD Steel Standard in Section 3.2.

To follow this standard, the deflection of the beam is controlled in terms of either a proportion of the span or specified deflection limit expressed as:

$$\Delta_{max} \leq \Delta_{allow} = \frac{L}{K} \text{ or } d_{allow} \tag{9. 11}$$

where

Δ_{max} : maximum deflection;

Δ_{allow} : allowable deflection;

L : beam span;

K : limiting deflection factor;

d_{allow} : specified deflection limit.

The maximum deflection can be calculated for a single joist under uniformly distributed load as:

$$\Delta_{max} = \frac{5 \cdot (\ D_n + Q_n\ ) \cdot s \cdot L^4}{384 \cdot \phi \cdot \overline{E} \cdot I} \leq \Delta_{allow} \tag{9. 12}$$

where

Δ_{max} : maximum deflection corresponding to $\overline{E}$;

Δ_{allow} : allowable deflection;

D_n : design uniformly distributed dead load;

Q_n : design uniformly distributed occupancy load;

s : spacing between members;

$\overline{E}$: mean modulus of elasticity for the population of lumber;

I : moment of inertia of the member cross-section,

whereas for rafters:

$$\Delta_{max} = \frac{5 \cdot (\ D_n + 0.9 \cdot S_n\ ) \cdot \mathrm{s} \cdot L^4}{384 \cdot \phi \cdot \overline{E} \cdot I} \leq \Delta_{allow} \qquad (\ 9.\ 13\ )$$

where

S_n : design uniformly distributed snow load.

A performance function for deflection limit state of a joist can be formulated as:

$$G = \Delta_{allow} - \frac{5 \cdot (\ D + Q\ ) \cdot \mathrm{s} \cdot L^4}{384 \cdot E \cdot I} \qquad (\ 9.\ 14\ )$$

where

D : dead load (a random variable);

Q : occupancy load (a random variable);

E : modulus of elasticity of the member (a random variable),

whereas for the rafter:

$$G = \Delta_{allow} - \frac{5 \cdot (D + S) \cdot s \cdot L^4}{384 \cdot E \cdot I} \qquad (9.15)$$

where

S : snow load (a random variable).

By substitution, the performance function and design equation can be expressed for joist as:

$$G = 1 - \frac{(d\gamma + q) \cdot \phi \cdot \overline{E}}{(\gamma + 1) \cdot E} \qquad (9.16)$$

where

d : D / D_n;

γ : D_n / Q_n;

q : Q / Q_n,

and for the rafter, the performance function is given by

$$G = 1 - \frac{(d\gamma + s) \cdot \phi \cdot \overline{E}}{(\gamma + 0.9) \cdot E} \qquad (9.17)$$

where

γ : D_n / S_n;

s : S / S_n.

The parameters for random variables Q and S are assumed 8-year return loads for the serviceability limit states.

The performance functions of Eq. 9.16 and Eq. 9.17 were evaluated using the RELAN program.

Table 19 summarizes ϕ-values corresponding to three target β-values (β_T = 2.0, 1.5 and 1.0) for selected size/grade combination for four snow loads γ = 0.25 and the actual γ_a. The results do not show significant differences with the choice of γ.

The results did not meet the satisfactory reliability of β_T = 1.0 at ϕ = 0.9 as was recommended by the draft LSD Steel Standard. In order to get β_T = 1.0, a performance factor ϕ = 0.85 should be used for the deflection serviceability limit states to yield safety levels comparable with steel structures.

10. DURATION OF LOAD EFFECTS

Strength properties of lumber depend on the duration of the load. Members subjected to short-term duration load have a higher strength than obtained for longer duration loads. In order to account for these characteristics, the current Japanese standard regards snow load as the temporary (short-term) load in the light snow area, and sustained load (long-term) in the heavy snow area. The allowable unit stresses for temporary load are two times the unit stresses for sustained load. In this chapter, a newly developed damage accumulation model by Foschi et al. [3 and 25] was used to evaluate Japanese practice with respect to duration of load adjustments. Monte Carlo simulation was applied to evaluate the effect of selected load combination on the duration of load adjustments for two qualities of S-P-F dimension lumber (SPF Q1 and SPF Q2). S-P-F Q1 is a high quality grade with a coefficient of variation of approximately 20 percent. S-P-F Q2 is a low quality grade with coefficient of variation of approximately 28 percent.

The objective of the duration of load analysis is to develop duration of load adjustment factors K_D such that the member reliability under short-term load is maintained when duration of load effects are considered.

Since data of an average snow duration can be obtained from the local meteorological observatories, simple snow load models were applied in this analysis. Snow load was assumed constant for the average duration of the ground snow load in each year in four locations.

Duration of load response of two qualities of S-P-F lumber will be evaluated in this study.

10.1 DAMAGE MODEL

A nonlinear damage accumulation model expressed by the following differential equation was developed by Foschi et al. [3 and 25]:

$$\frac{d\alpha}{dt} = a\,[\tau(t) - \sigma_0\tau_s]^b + c\,[\tau(t) - \sigma_0\tau_s]^n\,\alpha \qquad (10.1)$$

where

- α : damage state variable ($\alpha = 0$ in the initial state and $\alpha = 1$ at failure);
- a, b, c, n : model parameters;
- σ_0 : threshold stress ratio;
- $\tau(t)$: stress history;
- τ_s : standard short- term bending strength of the member obtained in a ramp test of one minute duration.

Only when $\tau(t) \geq \sigma_0\tau_s$, will there be damage accumulation. Since this differential equation model is difficult to evaluate, the damage accumulation was calculated step by step as shown in the following procedures:

$$\alpha_i = \alpha_{i-1}\,K_i + L_i \qquad (10.2)$$

where

$$K_i = \exp \left[c \left(\tau_i - \sigma_0 \tau_s \right)^n \Delta t \right] \qquad (10.3)$$

$$L_i = \frac{a}{c} \left(\tau_i - \sigma_0 \tau_s \right)^{b - n} \left(K_i - 1 \right) \qquad (10.4)$$

$$a \simeq \frac{K_s \left(b + 1 \right)}{\left(\tau_s - \sigma_0 \tau_s \right)^{b+1}} \qquad (10.5)$$

where K_s = median of τ_s ×60 (MPa/Hour) and the damage α can be obtained at any time by the recurrence relationship of Eq.10.2.

The parameters b, c, n, σ_0 were assumed lognormally distributed and the distribution of τ_s is assumed lognormally distributed and known from the analysis of short-term tests. The parameters obtained for the random values b, c, n, σ_0 and τ_s for the S-P-F duration of loads studies for quality level 1 (SPF Q1) and quality level 2 material (SPF Q2) were taken from Reliability-Based Design of Wood Structures Structural Research Series [3]. Those parameters are listed in Table 20.

Damage is defined as a state variable taking the values $\alpha = 0$ in the initial state and $\alpha = 1$ at failure. The damage α is a function of the stress history. For 50 years of service life, the performance function is expressed as:

$$G = 1.0 - \alpha(50) \qquad (10.6)$$

10.2 LOAD CASE

Duration of load effects were evaluated for the dead load plus snow load combination. The dead load model assumed was as explained in Sec. 8.1. The annual snow load record was assumed as one rectangular distribution in time with a duration Δt, equal to the annual average snow period as shown in Figure 18. Parameters for annual snow height distribution used in this analysis are explained in Sec. 8.3.

The total load is the superposition of the dead load and the snow load. A load sequence for 50 years was considered.

At any time t, actual stress $\sigma(t)$ is expressed as:

$$\sigma(t) = (D + S(t)) F \qquad (10.7)$$

where

F : factor to convert from load to stress.

Since the design equation for combination of dead load and snow load is expressed as:

$$(1.1 D_n + 1.6 S_n) F = \phi R_{0.05} \qquad (10.8)$$

Eq. 10.7 can be rewritten as:

$$\sigma(t) = \frac{\phi R_{0.05}}{1.1\gamma + 1.6} \left(d\gamma + \frac{S(t)}{S_n} \right) \qquad (10.9)$$

10.3 SIMULATION

Monte Carlo simulation was applied to obtain duration of load factors as follows:

1) A value of performance function ϕ and a value of the ratio of the design dead load to the design live load γ were chosen.

2) A load sequence of 50 segments was created.

a) A Uniformly distributed random number was chosen to calculate the dead load d, which was taken as a constant for the 50 years.

b) A Gumbel-distributed random number for annual snow was chosen, which was taken as a constant for the whole year. If the selected Uniformly distributed random number was smaller than $p_0 = \exp\{-\exp(AB)\}$, there was no snow in the year.

c) The dead load and live load were combined as Eq.10.9.

3) The damage accumulated the 50 years for each sample was computed.

a) Five Uniformly distributed random numbers were chosen to obtain the values of b, c, n, σ_0 and τ_s from their Lognormal distributions from Table 20.

b) Using the recurrence relationship in Eq.10.2, the accumulated damage α was calculated for 50 cycles.

c) The performance function

$$G = 1.0 - \alpha \qquad (10.10)$$

was then evaluated. If $G \geq 0$, the sample survived, and if G $\leq$ 0, the sample failed.

4) Step 2 and 3 above were then repeated for 10,000 replications.

5) The number of failures occurring in 50 years was used to compute the probability of failure

$$P_f = \frac{\text{number of failures}}{\text{number of replications (10000)}} \qquad (10.11)$$

6) The associated reliability index β was obtained from

$$\beta = -\Phi^{-1}(P_f)$$

7) The above process, starting at step 1, was then repeated for different values of ϕ.

10.4 DURATION OF LOAD RESULTS

The reliability results for SPF Q1 and Q2 are listed in Table 21 and 22 which shows ϕ-values corresponding to different target β-values for S-P-F Q1 and S-P-F Q2 material with and without DOL effect. The duration of load adjustment factor K_D is calculated for the four locations, where K_D is defined as $\frac{\phi_{with\ DOL}}{\phi_{without\ DOL}}$. These ϕ and K_D values were obtained from the analysis using three combinations of γ = 0.25 with an average annual snow duration in each location, an actual γ_a with an average annual snow in each location and γ = 0.25 with an annual snow duration of five months (155 days)(DOL-5) in order to compare the effects of those parameters.

The trends in ϕ-values were consistent across different β levels. But, ϕ and K_D values were influenced by the location, annual snow duration, and γ-values.

The differences between ϕ-values obtained using the average annual snow duration and the assumed five months snow duration are greater in the light snow areas but less in the heavy snow areas because heavy snow areas have a longer snow duration shown as Figure 19 and 20. For example, the annual snow duration in Osaka is two days and that in Sapporo is 136 days. Although the Canadian snow model was quite different from the rectangular model, the snow duration of five months was considered for the Canadian study of duration of load effect. It is too conservative to only use a snow duration of five months for the rectangular model.

Foschi et al. states that the choice of γ's has a significant affect on the calculated duration of load factor K_D [3].

Differences between ϕ-values at γ = 0.25 and at the actual γ_a are larger in the light snow areas but smaller in the heavy snow areas. The actual γ_a in Sapporo was 0.25, whereas that in Osaka was 4.4.

From Table 21 and 22, the results between SPF Q1 and Q2 were quite different. However, the ϕ-values in both cases were closer in Osaka with γ = 0.25 but were different with using the actual γ_a.

Generally speaking, results of SPF Q1 are closer to the results obtained for Hem-Fir as explained in Reliability-Based Design of Wood Structures Structural Research Series, but quite different from those of SPF Q2. Since SPF Q2 is a low quality grade and the K_D factors for SPF Q1 are similar to those of obtained for Hem-Fir the K_D values derived for SPF Q1 are recommended for S-P-F lumber.

The results of the SPF Q1 were plotted in Figure 21. It can be seen that they correspond closely to the Canadian analysis. Where K_D = 0.8 when $0 \leq \gamma \leq 1$; $K_D = 0.8 - 0.43\log(\gamma)$ when $1 < \gamma \leq 5$; and K_D = 0.5 when $\gamma > 5.0$. Because only limited data were feasible, it seems reasonable to use the Canadian analysis for the remainder of this study, given the fact that the present data generally have the same trend as the Canadian analysis.

11. ASSESSMENT OF RELIABILITY LEVELS ASSOCIATED WITH CURRENT WORKING STRESS DESIGN

WSD is currently used for the Japanese timber design. The reliability assessment procedures used to study β - ϕ relationships for the LSD code format can also be used to study the reliability levels in the WSD codes. The current reliability levels for 2x4 wood frame construction are evaluated in this chapter. Typical floor and roof systems were studied.

11.1 LOAD

Since this chapter is intended to evaluate the reliability level for current Japanese 2x4 wood frame construction, the nominal load values specified in the GHLC's span tables were used for this evaluation. The random variable $d = D/D_n$ was assumed to be Normally distributed with mean of 1.0 and standard deviation of 0.1. An actual ratio $\gamma = D_n/L_n$ or $\gamma = D_n/S_n$ was calculated at each location using data obtained from GHLC's span table.

The nominal occupancy load for residential structures of 180 kgf/m^2 (1.765 KN/m^2) is used for the span table calculations. The normalized occupancy load is explained in Sec.8.2. The 50-year return occupancy load was used the bending analysis. The annual maximum value was used for deflection analysis.

For the snow load, the random variables $s = S/S_n$ in each location are explained in Sec.8.3. The 50-year return snow load was used as the nominal load for the bending analysis. The annual maximum value was used for deflection analysis.

11.2 ALLOWABLE UNIT STRESS

The allowable unit stresses for lumber for 2x4 wood frame construction are specified in the Building Notification for Special Administrative Agency [13], explained in Sec.3.2.3, and tabulated in Appendix 1.

11.3 RELIABILITY LEVELS IN BENDING (SHORT-TERM BASIS)

WSD procedures taken from the Standard for Timber Design are used to develop span tables.

For floor joists:

$$D_n + L_n \leq Ra \qquad (11.1)$$

For rafters:

$$D_n \leq Ra. \qquad \text{(in light snow area)} \qquad (11.2)$$

$$D_n + S_n \leq 2\,Ra \qquad \text{(in light snow area)} \qquad (11.3)$$

$$D_n + S_n \leq Ra \qquad \text{(in heavy snow area)} \qquad (11.4)$$

where

D_n : design dead load effect;

L_n : design live load effect;

S_n : design snow load effect;

Ra : allowable unit stress for sustained load.

The evaluation of the reliability index β, using the RELAN program requires a performance function G. The safety levels for floor joists under short-term loading are evaluated by determining the probability that the performance function $G < 0$.

For the joist, the performance function is:

$$G = R - (D + L) \qquad (11.5)$$

where

R : strength (a random variable);

D : effect of the dead load (a random variable);

L : effect of the live load (a random variable).

Introducing the design equation in Eq.11.1 into the performance function in Eq.11.5 yields:

$$G = \mathrm{R} - \frac{Ra \cdot (d\gamma + l)}{(\gamma+1)} \qquad (11.6)$$

where

γ : D_n/L_n;

d : D/D_n;

l : L/L_n.

In a light snow area, snow load is regarded as the temporary load so that the allowable unit stress of a temporary load is used, i.e. two times its sustained load. The factor 2 takes account of a duration of load effect. However the structural member should also be checked for the dead load only. Therefore two performance functions must be considered in the light snow area.

If $D_n \geq S_n$, the governing design equation is $D_n \leq Ra$.

For rafters the performance function G can be expressed as :

$$G = R - (D + S) \qquad (11.7)$$

Introducing the design equation in Eq.11.2, the performance function in Eq.11.7 can be expressed as:

$$G = R - \frac{Ra \cdot (d\gamma + s)}{\gamma} \qquad (11.8)$$

If $S_n \geq D_n$, the governing design equation is $D_n + S_n \leq 2\ Ra$
By substitution, the performance function in Eq.11.7 and the design equation in Eq.11.3 can be also expressed as:

$$G = R - \frac{2 \cdot Ra \cdot (d\gamma + s)}{(\gamma+1)} \qquad (11.9)$$

For heavy snow areas, the snow load is regarded as a sustained load. Therefore, the allowable stress for sustained load is used.
By substitution, the performance function Eq.11.7 and the design equation Eq.11.4 can be expressed as:

$$G = R - \frac{Ra \cdot (d\gamma + s)}{(\gamma+1)} \qquad (11.10)$$

The short-term reliability of current construction was evaluated for bending for floor joist and rafter systems. The ratio's γ used for the analysis were derived using the material weights and nominal occupancy and snow load specified in the GHLC span tables. Results of these analysis are summarized in Table 23.

11.4 RELIABILITY LEVELS IN DEFLECTION

In the Standard for Timber Design, the allowable deflections for structural members are specified. GHLC's span table was applied to the requirements from the standard. The current serviceability limit states analysis was performed for the short-term deflection of a single member flat roof and floor.

Following GHLC's requirement, the deflection limit as specified as a proportion of the span or a maximum allowable deflection expressed as:

$$\Delta_{max} \leq \frac{L}{K} \qquad (11.11)$$

or

$$\Delta_{max} \leq \text{allowable deflection} \qquad (11.12)$$

The deflection limits for structural members for 2x4 wood frame structure are summarized in Table 24.

The maximum deflection can be calculated for single lumber member under uniformly distributed load as:

$$\Delta_{max} = \frac{5 \cdot (\ D_n\ +\ Q_n\ ) \cdot s \cdot L^4}{384 \cdot \overline{E} \cdot I} \leq \Delta_{allow} \qquad (11.13)$$

where

Δ_{max} : maximum deflection;

Δ_{allow} : allowable deflection;

D_n : design uniformly distributed dead load;

Q_n : design uniformly distributed live load;

s : spacing between members;

$\overline{E}$: mean modulus of elasticity for the population of lumber;

I : moment of inertia of the member cross-section.

A performance function for deflection limit state can be formulated as :

$$G = \Delta_{allow} - \frac{5 \cdot (D + Q) \cdot s \cdot L^4}{384 \cdot E \cdot I} \qquad (11.14)$$

where

D : dead load (a random variable);

Q : live load (a random variable);

E : modulus of elasticity of the member (a random variable).

Introducing Eq.11.13 the performance function can be expressed as :

$$G = 1 - \frac{(d\gamma + q) \cdot \overline{E}}{(\gamma + 1) \cdot E} \qquad (11.15)$$

where

d : D/D_n;

γ : D_n/Q_n;

q : Q/Q_n.

The performance function of Eq.11.15 was evaluated using the RELAN program and reliability levels at the serviceability limit state for current construction are shown in Table 25.

11.5 RELIABILITY OF CURRENT RAFTER CONSTRUCTIONS UNDER LONG-TERM LOADING

The actual reliability levels associated with current construction will vary depending on the design criteria controlling the member design. The reliability levels for bending members subjected to short-term loads have been evaluated for the bending strength limit state (Sec.11.3) and for the deflection serviceability limit state (Sec.11.4). The reliability levels are calculated for each limit state separately. The purpose of this section is to evaluate the actual reliability levels associated with current constructions for the bending strength limit state using the actual maximum spans permitted in the GHLC span tables.

The first step in this analysis is to calculate the allowable spans as governed by the bending, shear and deflection design requirements. The analysis is undertaken for five typical rafter systems for the four locations.

In the light snow area, two times the allowable unit stress, $2 \cdot R_a$, must be greater than or equal to the sum of the effects of the uniformly distributed dead and snow load, when the uniformly snow load is greater than the uniformly dead load. Thus, $(D_n+S_n) \leq 2R_a$, when

$S_n \geq D_n$. When the uniformly distributed snow load is less than the uniformly distributed dead load, the allowable unit stress, R_a, must be greater than the effect of distributed dead load. $D_n \leq R_a$, when $S_n \leq D_n$. For deflection considerations of the rafters, both $\frac{l}{200}$ and 2 cm must not be exceeded. The governing span l is the minimum value determined using the following equations.

Deflection:

$$l_1 = \sqrt[3]{\frac{384 \cdot E \cdot I}{200 \cdot 5 \cdot w}} \quad \text{when } \frac{l_1}{200} \leq 2\text{cm} \qquad (11.16)$$

$$l_1' = \sqrt[4]{\frac{2 \cdot 384 \cdot E \cdot I}{5 \cdot w}} \quad \text{when } \frac{l_1}{200} > 2\text{cm} \qquad (11.17)$$

Bending:

$$l_2 = \sqrt{\frac{8 \cdot Z \cdot f_b}{w}} \qquad (11.18)$$

Shear:

$$l_3 = \frac{4 \cdot A \cdot f_s}{3 \cdot w} \qquad (11.19)$$

Where

l_1, l_1' : span governed by deflection;

l_2 : span governed by bending;

l_3 : span governed by shear;

E : modulus of elasticity;

f_b : allowable bending stress;

f_s : allowable shear stress;

I : moment of inertia;

Z : section modulus;

A : cross section area;

w : uniformly distributed load without safety factor as with the LSD.

Consider the following example:

given: Tokyo, light roof, spacing = 455mm, S-P-F, SS, 2x8

Dead load was calculated from Table 6

D_n = (20 + 6) × .455 + 6 = 17.83 kg/m

Snow load was obtained from Table 5

S_n = 35.88 × 2 × .455 = 32.65 kg/m

Since uniformly distributed snow load is greater than uniformly distributed dead load, therefore the design equation $(D_n+S_n) \leq 2R_a$ must be satisfied. In order to use from Eq.11.16 to Eq.11.19 with R_a (not $2 \cdot R_a$), nominal uniformly distributed dead plus nominal uniformly distributed snow load was divided by two to get w (convert short-term to long-term) so that w can also be used for the deflection checking equations on a long-term basis.

$$w = (D_n+S_n)/2 = 0.252 \text{ kg/cm} \qquad (11.20)$$

Introducing w to the Eq.11.16 to Eq.11.19

$$l_1' = \sqrt[4]{\frac{2 \cdot 384 \cdot 85000 \cdot 1972}{5 \cdot 0.252}} = 565.1\text{cm} \qquad (11.21)$$

$$l_2 = \sqrt{\frac{8 \cdot 214.4 \cdot 110}{0.252}} = 864.6\text{cm} \qquad (11.22)$$

$$l_3 = \frac{4 \cdot 69.9 \cdot 6}{3 \cdot 0.252} = 2215.5\text{cm} \qquad (11.23)$$

The allowable span for this case was 5.56m, the span governed by deflection.

An analysis of rafter spans as governed by deflection, bending and shear is summarized in Table 26. Generally speaking, bending requirements governs in the heavy snow cases and deflection governs in the light snow cases. When deflection governs, the allowable span will be shorter than permitted by bending requirements. In these cases the actual reliability levels for the bending strength limit state will be greater than predicted versus when the bending strength limit state is examined independently (Sec. 11.3).

The actual reliability levels for the bending strength limit state are evaluated in a two stage process. (1) evaluate the short-term reliability of the bending member and then (2) adjust the short-term reliability for duration of load effects to get the actual reliability under long-term loading.

The relationship between span and corresponding β for bending can be obtained from the following performance function:

$$G = R - (D+S) \cdot p \cdot \frac{6L^2}{8BH^2} \qquad (11.24)$$

$$= R - S_n(d\gamma+s) \cdot p \cdot \frac{6L^2}{8BH^2} \qquad (11.25)$$

where

D : dead load;

S : snow load;

p : spacing between members;

L : span;

B : member width;

H : member depth;

S_n : nominal snow load;

d : normalized dead load;

s : normalized snow load;

γ : D_n/S_n.

The duration of load factor for snow load cases, K_D = 0.8 for Sapporo, Niigata and Tokyo and K_D = 0.52 for Osaka were applied for the results from the short-term reliability analysis to obtain the long-term basis. Since the duration of load factor is defined as $K_D = \left(\frac{L_{long-term}}{L_{short-term}}\right)^2$ for the bending analysis, an adjustment span from the short-term reliability to the long-term reliability can be obtained as $L_{long-term} = L_{short-term} \times \sqrt{K_D}$ at the same reliability level. Figure 22 shows the relationship between member span and corresponding β values for a light roof case and Tokyo snow load for the short-term (without DOL) and long-term (with DOL) cases. In this case, the maximum calculated span due to short-term bending was 8.64 m (Table 26) with a β-value of 1.566. This same β-value was obtained in the analysis of the reliability of current construction summarized in Table 23. Because the governing factor in the example was deflection, the governing span was then reduced to 5.65. At this span the β-value is found to be : β = 3.01 for the short-term basis and β = 2.68 for the long-term basis. Similarly, different member sizes, grades and spacing; snow loads of different geographic locations; and different roofing material commonly used in the selected regions were incorporated to generate Figures 23, 24, 25, 26, and 27. Since the strength distribution parameters of 2x6 were not available, those of 2x8 were tentatively used for this analysis.

11.6 DISCUSSION

The short-term reliability levels for floor joists in bending range are from 2.68 to 3.03. However the safety levels for rafters range from 1.40 to 3.63 depending on location, γ, nominal dead load and nominal snow load ratio. In heavy snow area ie., Sapporo and Niigata, they range from 2.67 to 3.63. In light snow areas ie., Tokyo and Osaka, if rafter spans are determined by D_n, when $D_n \geq S_n$ or $\gamma \geq 1.0$, then reliability indices range from 1.63 to 3.35. However if rafters are determined by $D_n + S_n$, when $S_n \geq D_n$ or $\gamma < 1.0$, reliability levels drop to range from 1.38 to 2.40. This is because allowable stresses for temporary load (twice as much as the allowable unit stress for sustained load) are used for the snow load in light snow areas.

The reliability levels in bending for rafter design in the light snow area are very low, if they were considered individually. Since rafter sizes are determined not only by bending strength but shear strength and stiffness of the lumber, the reliability levels of actual roof member in long-term bending were evaluated assuming K_D = 0.8 for five cases. From the five cases, the minimum and mean-β values were 2.35 and 2.78 respectively, which is quite acceptable for the bending strength limit states. The values for 2x6 were not investigated.

12. DISCUSSION AND CONCLUSION

Most timber structures in Japan are of the so-called traditional post and beam construction and utilize various traditional sizes of sawn lumber. Introduced into Japan in 1974, 2x4 wood frame construction makes up approximately 3% of total housing units constructed in 1989. The current structural design method for all types of wood structures is based on WSD. It is highly recommended that the design of 2x4 wood frame system be converted to the LSD format as a test case for development of LSD codes for all timber structures. It would be ideal for Japanese building codes to be converted from the WSD to the LSD for all types of structures.

Although intensive in-grade test results from CWC's research project are available for the dimension lumber used for 2x4 wood frame structures, currently there is no standard for evaluating full-size lumber and very little data is available for the lumber used for traditional post and beam structures in Japan. The analysis and results obtained in this study were entirely based on an analysis of lumber used for the 2x4 wood frame structures. However, due to the limited availability of relevant strength data for other wood structural systems, it is important to develop a standard test method to evaluate full-size lumber for the reliability analysis. Therefore, the development of LSD codes for all timber structures requires a tremendous amount of collaborative work among designers and engineers in the future.

In this study, reliability levels were evaluated mainly in bending. The species selected throughout this study was S-P-F which constitutes more than two thirds of total lumber used for 2x4 wood frame construction in Japan. The four locations of snow load chosen for the analysis represent typical Japanese snow conditions.

The performance factors for LSD equations have been calibrated for 2x4 wood frame structures using draft LSD Steel Standard requirements for reliability levels. The outcome from this study shows that the 2x4 wood frame system yields performance factors ϕ of 0.85 for bending and 0.9 for tension and compression at the target reliability of β_T = 2.5 which is compatible with steel structures.

Ratio of nominal dead to live load γ, of 0.25 and the actual dead to live load ratios γ_a were used in this analysis. The γ = 0.25 was chosen for reliability studies of wooden structures in Canada. The actual γ_a value resulted in lower reliability levels for bending design in certain areas of Japan where snow accumulations are not as severe. However the reliability levels for tension and compression did not show significant differences with the choice of γ. Therefore, results indicate that the actual dead to live load ratio γ_a should be used for the calibration studies.

Size effects are currently not considered in non-glulam timber structures in Japan. To achieve a better strength prediction of the non-glulam timber, size effects should be incorporated into design procedure for 2x4 wood frame construction.

The recommended ϕ-values and the corresponding average β and range of β were evaluated taking into account size effects are summarized in table 27.

Table 27. Recommended ϕ and β-values

	γ	ϕ	Min. β	Mean β	Max. β
Bending	$\gamma = 0.25$	0.9	2.31	2.49	2.69
	Actual γ_a	0.85	2.13	2.48	2.80
Tension	$\gamma = 0.25$	0.9	2.25	2.50	2.72
	Actual γ_a	0.9	2.21	2.50	2.77
Compression	$\gamma = 0.25$	0.9	2.15	2.51	3.11
	Actual γ_a	0.9	2.09	2.51	3.15

Based on the study of duration of load effects, the results from SPF Q1 were close to those obtained from Hem-Fir for the Canadian study. The duration of load effect varies with the choice of dead to live load ratio. In the Canadian case, regardless of the live load a factor of K_D = 0.80 can be used where γ is less than or equal to 1.0. Where γ is greater than 5.0, a factor of K_D = 0.50 can be applied. In this study, a simple snow load model was used for S-P-F duration of load analysis. Results were relatively close that obtained in the more detailed Canadian studies. The factors K_D at γ = 0.25 and the actual γ_a are summarized in Table 28 for four locations in Japan.

The bending reliability levels of current 2x4 wood frame for rafter design were found to be quite comparable with steel structures. The reliability levels for bending strength in the light snow area are very low. In order to calculate the actual reliability levels in

Table 28. Duration of Load Factor K_D for SPF Q1 at $\beta_T = 2.5$

	$K_D(\gamma = 0.25)$	Actual γ_a	$K_D(Actual\ \gamma_a)$
Sapporo	0.70	$\gamma_a = 0.25$	0.70
Niigata	0.78	$\gamma_a = 0.41$	0.77
Tokyo	0.87	$\gamma_a = 1.22$	0.81
Osaka	0.90	$\gamma_a = 4.40$	0.57

rafters, we take the reliability level as a function of span. The rafter span is determined by bending, shear and deflection requirements. Generally speaking, the rafter spans in the light snow area are governed by deflection. Therefore, the calculated bending spans are reduced to the governing deflection spans and the reliability levels for the strength limit states are increased. The duration of load adjustment factor K_D = 0.8 for Sapporo, Niigata and Tokyo and K_D = 0.52 for Osaka were applied to assess reliability under long-term loading. The actual minimum reliability levels for the strength limit states in long-term bending and short-term bending were summarized in Table 29.

Table 30 compares rafter spans using the new LSD equations vs. the current design procedures and shows the corresponding long-term reliability levels. Five different conditions and three size combinations were used in the comparison. The results from the new LSD equations (using ϕ_0 = 0.85 and the actual γ_a) show consistent reliability levels above 2.5. The span values from the new LSD equations do not deviate too much from the current design spans. Only when the reliability level goes below 2.5 in the current design

Table 29. Reliability Level β in Current 2x4 Wood Frame Structure

	Short-Term Bending		Final Long-Term Bending
	Min. β	Max. β	Min. β
Sapporo	2.72	3.63	2.63
Niigata	2.67	3.59	2.60
Tokyo	1.38	2.59	2.35
Osaka	2.12	3.35	2.20

method, will we witness a decrease in span. The table shows the LSD method provides more consistent reliability levels than the current design method where the range of β-values varies considerably.

This study is carried out mainly in bending with the load combination of dead plus occupancy load and dead plus snow load using S-P-F strength data. Therefore further studies will be required in areas such as load combinations with wind and earthquake load and for other species and other strength limit states in order to develop a complete LSD code. For example, Japanese structural design is based on single members and design properties are not adjusted by system modification factors. Although system modification factors were not considered in this study, and probably not applicable in the traditional post and beam structures (due to the extensively long spacing between members), they should be considered when calculating the reliability levels in structures made up of repetitive single members such as the 2x4 wood frame system. By incorporating these modification factors the safety levels β, would represent more closely actual behavior and allow for more efficient use of materials [3].

Table 30. Rafter Span Comparison Using Current and New LSD Design Equation

Tokyo, Light Roofing, Spacing=455mm

Grade		2x4 Current	2x4 New	2x8 Current	2x8 New	2x10 Current	2x10 New (unit: m)
SS	D-Span	3.10	3.18	5.65	5.75	6.72	6.85
	B-Span	4.26	3.19	8.64	5.71	10.82	6.94
	Span	3.10	3.18	5.65	5.71	6.72	6.85
	β	3.14	3.05	2.68	2.63	2.74	2.69

Sapporo, Light Roofing, Spacing=303mm

Grade		2x4 Current	2x4 New	2x8 Current	2x8 New	2x10 Current	2x10 New (unit: m)
No.2	D-Span	1.75	1.84	3.61	3.83	4.43	4.59
	B-Span	1.59	1.81	3.26	3.24	4.14	3.95
	Span	1.59	1.81	3.26	3.24	4.14	3.95
	β	3.09	2.66	2.65	2.66	2.63	2.81

Niigata, Heavy Roofing, Spacing=455mm

Grade		2x4 Current	2x4 New	2x8 Current	2x8 New	2x10 Current	2x10 New (unit: m)
No.2	D-Span	1.68	1.73	3.47	3.61	4.30	4.38
	B-Span	1.50	1.73	3.08	3.10	3.90	3.77
	Span	1.50	1.73	3.08	3.10	3.90	3.77
	β	3.08	2.59	2.63	2.61	2.60	2.72

Tokyo, Light Roofing, Spacing=455

Grade		2x4 Current	2x4 New	2x8 Current	2x8 New	2x10 Current	2x10 New (unit: m)
No.2	D-Span	2.98	3.06	5.47	5.66	6.51	6.70
	B-Span	3.52	2.76	7.13	4.94	8.94	6.00
	Span	2.98	2.76	5.47	4.94	6.51	6.00
	β	2.46	2.71	2.35	2.68	2.45	2.77

Osaka, Heavy Roofing, Spacing=455mm

Grade		2x4 Current	2x4 New	2x8 Current	2x8 New	2x10 Current	2x10 New (unit: m)
No.2	D-Span	2.63	3.03	4.97	5.63	5.90	6.66
	B-Span	2.93	2.49	5.90	4.46	7.34	5.42
	Span	2.63	2.49	4.97	4.46	5.90	5.42
	β	2.42	2.61	2.20	2.56	2.29	2.64

where

D-Span ; Span Calculated by Deflection Requirement
B-Span ; Span Calculated by Bending Requirement
Span ; Minimum of Above Two Calculated Span
β ; Bending (Long-Term) Reliability Level at Allowable Span

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Table 1. Design Requirements of the Japanese Building Codes

<table>
<tr><th colspan="2"></th><th>Allowable Unit Stress</th><th>Relative Story Displacement Angle</th><th>Rigidity and Eccentricity</th><th>Retained Horizontal Strength</th><th>Remarks</th></tr>
<tr><td rowspan="4">Traditional Post and Beam Structure</td><td>Stories of 2 or less, Total Floor Area of 500 m^2 or less, Building Height of 13 m or less, and Eaves Height of 9 m or less</td><td>N/A</td><td>N/A</td><td>N/A</td><td>N/A</td><td rowspan="3">Effective Wall Length Building Standard Law Enforcement Order Article 46</td></tr>
<tr><td>Stories of 3 or more</td><td>*</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td>Total Floor Area of more than 500 m^2</td><td>*</td><td>N/A</td><td>N/A</td><td>N/A</td></tr>
<tr><td>Building Height of more than 13 m or Eaves Height of More than 9 m</td><td colspan="4">Prohibited</td><td>Building Standard Law Enforcement Order Article 129</td></tr>
<tr><td rowspan="2">2x4 Wood Frame Structure</td><td>Stories of 2 or less and Total Floor Area of 500 m^2 or less</td><td>1)</td><td>N/A</td><td>N/A</td><td>N/A</td><td>Effective Wall Length Notification No.56</td></tr>
<tr><td>3 Stories or more or Total Floor Area of more than 500 m^2</td><td>*</td><td>N/A</td><td>N/A</td><td>N/A</td><td>Effective Wall Length Notification No.56</td></tr>
<tr><td>Log House</td><td>Maximum 2 Stories with Maximun Height of 8.5 m and Total Floor Area of 300 m^2</td><td>N/A</td><td>N/A</td><td>N/A</td><td>N/A</td><td>Notification No.1126</td></tr>
<tr><td rowspan="6">Heavy Timber Structure</td><td>Stories of 2 or less, Total Floor Area of 500 m^2 or less, Building Height of 13 m or less, and Eaves Height of 9 m or less</td><td>*</td><td>*</td><td>N/A</td><td>N/A</td><td></td></tr>
<tr><td>Stories of 3 or more</td><td>*</td><td>*</td><td>N/A</td><td>N/A</td><td rowspan="2">Building Height of 13 m or less and Eaves Height of 9 m or less</td></tr>
<tr><td>Total Floor Area of more than 500 m^2</td><td>*</td><td>*</td><td>*</td><td>N/A</td></tr>
<tr><td>Total Floor Area of more than 500 m^2</td><td>*</td><td>*</td><td>*</td><td>*</td><td rowspan="2">Building Height of 31 m or less</td></tr>
<tr><td>Building Height of more than 13 m or Eaves Height of more than 9 m</td><td>*</td><td>*</td><td>*</td><td>*</td></tr>
<tr><td>Building Height of more than 31 m</td><td>*</td><td>*</td><td>*</td><td>*</td><td></td></tr>
<tr><td>Special Structure</td><td colspan="5">N/A</td><td>Building Standard Law Article 38</td></tr>
</table>

N/A ; Not Applicable

* ; Must Check Conditions

1) ; Notification for Special Administrative Agency Requires Simplified Structural Member Check

Table 2. Load Combinations for Working Stress Design

Kinds of stress	Possible conditions regarding loads and external forces	General cases	In heavy snow areas designated by the Special Administrative Agency under the provision to Article 86 Paragraph 2	Remarks
Stress due to sustained loads	Normal time	D + L	D + L + S	
Stress due to temporary loads	Snow season	D + L + S	D + L + S	
	Storm	D + L + W	D + L + W	Safety in case of overturning of buildings or pulling out of columns. L shall be a value obtained by reducing the live load according to the actual conditions of the building concerned
			D + L + S + W	
	Earthquake	D + L + E	D + L + S + E	

where
- D ; Stresses due to dead load
- L ; Stresses due to live (occupancy) load
- S ; Stresses due to snow load
- W ; Stresses due to wind pressure
- E ; Stresses due to seismic force

Table 3. Basic Statistical Data for Safety Analysis of Steel Structures

Load	γ	μ	$\overline{X}/X_n$	V_X	Remarks
D	-	-	1.0	0.10	
L_S	1/8	8	0.36	0.40	sustained load
L_E	1	-	0.25	0.55	extraordinary load
S	1	1/3	0.45	0.48	heavy snow area
W	1	1	0.42	0.47	annual maxima
E	1/3	-	0.42	0.80	
*R*esistance			1.10	$0.15 \sim 0.20$	

where

D : dead load effect;

L : maximum values of live load effect in 50-Year return period;

L_S : sustained live load effect;

L_E : extraordinary live load;

S : maximum values of snow load effect in 50-Year return period;

W : maximum values of wind load effect in 50-Year return period;

E : maximum values of earthquake load effect in 50-Year return period;

γ : the time between events (year^{-1});

μ : duration of tenancy (year);

$\overline{X}$: mean in variable X;

X_n : nominal value in variable X;

V_X : coefficient of variation in variable X;

*R*esistance : material strength.

(All data are assumed Lognormal distribution)

Table 4. Occupancy Load (Extreme Type I Distribution)

	Q		q	
	A	B	A*	B*
	$(kg/m^2)^{-1}$	(kg/m^2)		
50-Y Return	0.0578493	196.272	10.41287	1.09040
8-Y Return	0.0578493	164.594	10.41287	0.91441
Annual	0.0578493	128.468	10.41287	0.71471

Table 5. Japanese Snow Data

Location		Sapporo	Niigata	Tokyo	Osaka
Annual Snow Duration	(days)	132	61	5	2
Annual Snow Height	A (cm^{-1})	0.04677	0.05534	0.13291	0.38919
Gumbel Distribution	B (cm)	83.92	32.66	6.52	0.023
50 Years Return Snow Height	(cm)	167	103	36	10
Design Snow Load	(kg/m^2)	351	216	72	20
Normalized 50 Years Return Snow Height	A	7.8267	5.7091	4.7686	3.9111
Distribution Gumbel Distribution	B	1.0013	1.0018	1.0021	1.0026
Normalized 8 Years Return Snow Height	A	7.8267	5.7091	4.7686	3.9111
Distribution Gumbel Distribution	B	0.7671	0.6808	0.6178	0.5340

Table 6. Nominal Design Dead, Occupancy and Snow Load

Dead Load			
Floor Joists	Tatami Mat	18	kgf/m^2
	Plywood Sheathing (12 mm)	8	kgf/m^2
	Gypsum Board	15	kgf/m^2
	Joist (2x8 455mm spacing)	13	kgf/m^2
	Total	54	kgf/m^2
Rafters			
Light Roofing Material			
	Light Roofing Material	20	kgf/m^2
	Plywood Sheathing (9 mm)	6	kgf/m^2
	Rafter (2x8 455mm spacing)	13	kgf/m^2
	Total	39	kgf/m^2
Heavy Roofing Material			
	Heavy Roofing Material (Clay Tile)	60	kgf/m^2
	Plywood Sheathing (9 mm)	8	kgf/m^2
	Rafter (2x8 303mm spacing)	20	kgf/m^2
	Total	88	kgf/m^2

Design Occupancy Load			
	Residential Type	180	kgf/m^2

Design Snow Load			
	Sapporo	350	kgf/m^2
	Niigata	216	kgf/m^2
	Tokyo	72	kgf/m^2
	Osaka	20	kgf/m^2

Table 6a. Nominal Dead to Live Design Load Ratio

γ			
$\gamma = D_n / L_n$		0.30	
$\gamma = D_n / S_n$		Light Roofing	Heavy Roofing (γ_a)
	Sapporo	0.11	0.25
	Niigata	0.18	0.41
	Tokyo	0.54	1.22
	Osaka	1.95	4.40

Table 7. Bending Performance Factors for S-P-F

(ϕ-values Corresponding Target β-values)

Sapporo		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ_a=0.25	γ=0.25	γ_a=0.25	γ=0.25	γ_a=0.25
2x4	SS	0.77	-	1.01	-	1.28	-
	No2	0.72	-	0.98	-	1.28	-
2x8	SS	0.65	-	0.90	-	1.20	-
	No2	0.67	-	0.94	-	1.24	-
2x10	SS	0.68	-	0.94	-	1.24	-
	No2	0.81	-	1.04	-	1.30	-
Average		0.72	-	0.97	-	1.26	-

Niigata		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ_a=0.41	γ=0.25	γ_a=0.41	γ=0.25	γ_a=0.41
2x4	SS	0.76	0.74	1.00	0.98	1.26	1.23
	No2	0.71	0.70	0.97	0.95	1.26	1.23
2x8	SS	0.64	0.62	0.89	0.87	1.19	1.16
	No2	0.67	0.65	0.93	0.90	1.23	1.19
2x10	SS	0.67	0.65	0.93	0.90	1.22	1.19
	No2	0.80	0.78	1.03	1.00	1.28	1.25
Average		0.71	0.69	0.96	0.93	1.24	1.21

Tokyo		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ_a=1.22	γ=0.25	γ_a=1.22	γ=0.25	γ_a=1.22
2x4	SS	0.76	0.69	0.99	0.90	1.25	1.14
	No2	0.71	0.64	0.96	0.87	1.25	1.13
2x8	SS	0.63	0.57	0.88	0.80	1.17	1.07
	No2	0.66	0.59	0.92	0.83	1.21	1.10
2x10	SS	0.66	0.60	0.92	0.83	1.21	1.10
	No2	0.79	0.72	1.02	0.93	1.27	1.15
Average		0.70	0.64	0.95	0.86	1.23	1.12

Osaka		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ_a=4.40	γ=0.25	γ_a=4.40	γ=0.25	γ_a=4.40
2x4	SS	0.75	0.63	0.98	0.82	1.24	1.04
	No2	0.70	0.58	0.95	0.80	1.24	1.04
2x8	SS	0.62	0.53	0.87	0.73	1.16	0.97
	No2	0.65	0.55	0.90	0.76	1.20	1.01
2x10	SS	0.65	0.55	0.90	0.76	1.19	1.00
	No2	0.78	0.66	1.01	0.85	1.25	1.05
Average		0.69	0.58	0.93	0.79	1.21	1.02

Occupancy		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ_a=0.28	γ=0.25	γ_a=0.28	γ=0.25	γ_a=0.28
2x4	SS	0.73	0.73	0.96	0.95	1.21	1.20
	No2	0.69	0.69	0.93	0.92	1.21	1.20
2x8	SS	0.61	0.61	0.85	0.85	1.13	1.13
	No2	0.63	0.63	0.88	0.88	1.17	1.17
2x10	SS	0.64	0.64	0.88	0.88	1.17	1.16
	No2	0.76	0.76	0.98	0.98	1.22	1.22
Average		0.68	0.68	0.91	0.91	1.19	1.18
Total Average		0.70	0.66	0.94	0.89	1.22	1.16

Table 8. Mean β-values Corresponding to Given ϕ in Bending

Performance Factor ϕ	Mean β γ=0.25	Mean β Actual γ_a
0.6	3.2	3.1
0.7	3.0	2.9
0.8	2.8	2.7
0.9	2.6	2.5
1.0	2.4	2.3

Table 9. Size Factor and Standard Strength R_0 for Bending

$\beta = 2.5$ at $\phi_0 = 0.9$ unit : MPa

γ	Size Factor	Grade	R_0	$R_{0.05}$
0.25	4.495	SS	24.358	23.050
		No.2	17.666	16.890
Actual[1)]	4.483	SS	22.744	23.050
		No.2	16.491	16.890

1) see Table 6a

Table 10a. Modified β-values in Bending at ϕ_0=0.9 and ϕ_0=0.85 (γ=0.25)

Size	Grade	Sapporo		Niigata		Tokyo		Osaka		Occupancy	
	ϕ_0	0.9	0.85	0.9	0.85	0.9	0.85	0.9	0.85	0.9	0.85
2x4	SS	2.69	2.80	2.67	2.77	2.65	2.75	2.62	2.73	2.58	2.69
	No.2	2.42	2.52	2.39	2.49	2.37	2.48	2.35	2.45	2.31	2.42
2x8	SS	2.43	2.52	2.41	2.50	2.39	2.48	2.37	2.46	2.33	2.43
	No.2	2.50	2.59	2.48	2.57	2.47	2.56	2.44	2.54	2.41	2.50
2x10	SS	2.49	2.58	2.47	2.56	2.45	2.54	2.43	2.52	2.39	2.49
	No.2	2.69	2.80	2.66	2.77	2.64	2.75	2.61	2.73	2.57	2.69
	ϕ_0=0.9		Maximum=2.69			Minimum=2.31			Average=2.49		
	ϕ_0=0.85		Maximum=2.80			Minimum=2.42			Average=2.59		

Table 10b. Modified β-values in Bending at ϕ_0=0.85 and ϕ_0=0.80 (Actual γ_a)

Size	Grade	Sapporo		Niigata		Tokyo		Osaka		Occupancy	
	ϕ_0	0.85	0.8	0.85	0.8	0.85	0.8	0.85	0.8	0.85	0.8
2x4	SS	2.80	2.90	2.73	2.84	2.57	2.69	2.40	2.52	2.68	2.79
	No.2	2.52	2.62	2.45	2.55	2.30	2.41	2.13	2.25	2.40	2.51
2x8	SS	2.52	2.62	2.46	2.55	2.32	2.42	2.17	2.27	2.42	2.52
	No.2	2.59	2.69	2.53	2.63	2.40	2.50	2.25	2.35	2.50	2.59
2x10	SS	2.58	2.68	2.52	2.62	2.38	2.48	2.23	2.33	2.48	2.58
	No.2	2.80	2.92	2.72	2.82	2.56	2.68	2.37	2.50	2.68	2.80
	ϕ_0=0.85		Maximum=2.80			Minimum=2.13			Average=2.48		
	ϕ_0=0.80		Maximum=2.92			Minimum=2.25			Average=2.59		

Table 11. Performance Factors in Tension for S-P-F

(ϕ-values Corresponding to Target β-values)

Sapporo		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=0.25	γ=0.25	γ=0.25	γ=0.25	γ=0.25
2x4	SS	0.64	-	0.91	-	1.16	-
	No2	0.64	-	0.90	-	1.20	-
2x8	SS	0.81	-	1.02	-	1.25	-
	No2	0.80	-	1.03	-	1.28	-
2x10	SS	0.80	-	1.02	-	1.26	-
	No2	0.71	-	0.95	-	1.23	-
Average		0.73	-	0.97	-	1.23	-

Niigata		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=0.41	γ=0.25	γ=0.41	γ=0.25	γ=0.41
2x4	SS	0.65	0.64	0.87	0.86	1.12	1.10
	No2	0.61	0.64	0.87	0.85	1.16	1.14
2x8	SS	0.77	0.76	0.97	0.96	1.19	1.17
	No2	0.76	0.75	0.98	0.97	1.22	1.21
2x10	SS	0.76	0.75	0.97	0.96	1.20	1.19
	No2	0.68	0.68	0.92	0.90	1.18	1.16
Average		0.71	0.70	0.93	0.92	1.18	1.16

Tokyo		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=1.22	γ=0.25	γ=1.22	γ=0.25	γ=1.22
2x4	SS	0.63	0.60	0.84	0.80	1.08	1.03
	No2	0.59	0.57	0.84	0.80	1.13	1.07
2x8	SS	0.73	0.72	0.93	0.91	1.14	1.10
	No2	0.73	0.71	0.94	0.91	1.18	1.13
2x10	SS	0.73	0.71	0.93	0.90	1.15	1.11
	No2	0.66	0.63	0.88	0.85	1.14	1.09
Average		0.68	0.66	0.89	0.86	1.14	1.09

Osaka		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=4.40	γ=0.25	γ=4.40	γ=0.25	γ=4.40
2x4	SS	0.59	0.56	0.80	0.75	1.03	0.96
	No2	0.57	0.53	0.80	0.74	1.08	0.99
2x8	SS	0.69	0.67	0.87	0.84	1.08	1.03
	No2	0.69	0.66	0.89	0.85	1.12	1.05
2x10	SS	0.69	0.66	0.88	0.84	1.09	1.04
	No2	0.62	0.59	0.84	0.78	1.09	1.01
Average		0.64	0.61	0.85	0.80	1.08	1.01
Total Average		0.69	0.68	0.91	0.89	1.16	1.12

Table 12. Mean β-values Corresponding to Given ϕ in Tension

Performance Factor ϕ	Mean β γ=0.25	Mean β Actual γ_a
0.6	3.2	3.2
0.7	3.0	3.0
0.8	2.7	2.7
0.9	2.5	2.5
1.0	2.3	2.3

Table 13. Size Factor and Standard Strength R_0 for Tension

$\beta = 2.5$ at $\phi_0 = 0.9$ unit : MPa

γ	Size Factor	Grade	R_0	$R_{0.05}$
0.25	8.864	SS	13.628	12.27
		No.2	8.8125	8.320
Actual [1)]	9.543	SS	13.317	12.27
		No.2	8.5487	8.320

1) see Table 6a

Table 14a. Modified β-values in Tension at ϕ_0=0.9 and ϕ_0=0.85 (γ=0.25)

Size	Grade	Sapporo		Niigata		Tokyo		Osaka	
	ϕ_0	0.9	0.85	0.9	0.85	0.9	0.85	0.9	0.85
2x4	SS	2.69	2.79	2.61	2.71	2.55	2.65	2.45	2.56
	No.2	2.51	2.60	2.45	2.54	2.40	2.49	2.33	2.42
2x8	SS	2.61	2.74	2.49	2.62	2.39	2.52	2.25	2.39
	No.2	2.72	2.83	2.62	2.73	2.55	2.62	2.41	2.53
2x10	SS	2.62	2.74	2.51	2.63	2.42	2.54	2.29	2.41
	No.2	2.63	2.73	2.55	2.65	2.49	2.59	2.40	2.50
ϕ_0=0.9		Maximum=2.72			Minimum=2.25			Average=2.50	
ϕ_0=0.85		Maximum=2.83			Minimum=2.39			Average=2.61	

Table 14b. Modified β-values in Tension at ϕ_0=0.9 and ϕ_0=0.85(Actual γ_a)

Size	Grade	Sapporo		Niigata		Tokyo		Osaka	
	ϕ_0	0.9	0.85	0.9	0.85	0.9	0.85	0.9	0.85
2x4	SS	2.74	2.84	2.64	2.74	2.53	2.63	2.38	2.49
	No.2	2.57	2.66	2.49	2.58	2.38	2.47	2.25	2.34
2x8	SS	2.66	2.78	2.52	2.65	2.38	2.52	2.21	2.35
	No.2	2.77	2.88	2.65	2.77	2.53	2.65	2.37	2.50
2x10	SS	2.66	2.78	2.53	2.65	2.39	2.52	2.22	2.36
	No.2	2.68	2.77	2.58	2.68	2.46	2.56	2.32	2.43
ϕ_0=0.9		Maximum=2.77			Minimum=2.21			Average=2.50	
ϕ_0=0.85		Maximum=2.88			Minimum=2.34			Average=2.61	

Table 15. Performance Factors in Compression for S-P-F

(ϕ-values Corresponding to Target β-values)

Sapporo		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=0.25	γ=0.25	γ=0.25	γ=0.25	γ=0.25
2x4	SS	0.90	-	1.08	-	1.26	-
	No2	0.72	-	0.94	-	1.17	-
2x8	SS	0.86	-	1.05	-	1.24	-
	No2	0.90	-	1.05	-	1.23	-
2x10	SS	0.90	-	1.06	-	1.23	-
	No2	0.79	-	1.01	-	1.24	-
Average		0.84	-	1.03	-	1.23	-

Niigata		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=0.41	γ=0.25	γ=0.41	γ=0.25	γ=0.41
2x4	SS	0.84	0.84	1.01	1.00	1.18	1.18
	No2	0.69	0.68	0.89	0.88	1.12	1.10
2x8	SS	0.81	0.80	0.98	0.98	1.17	1.16
	No2	0.81	0.81	0.98	0.98	1.16	1.15
2x10	SS	0.83	0.84	0.99	0.99	1.15	1.15
	No2	0.75	0.75	0.96	0.95	1.18	1.17
Average		0.79	0.79	0.97	0.96	1.16	1.15

Tokyo		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=1.22	γ=0.25	γ=1.22	γ=0.25	γ=1.22
2x4	SS	0.79	0.80	0.95	0.96	1.13	1.11
	No2	0.66	0.64	0.86	0.83	1.08	1.03
2x8	SS	0.77	0.76	0.94	0.93	1.12	1.10
	No2	0.77	0.77	0.93	0.93	1.10	1.09
2x10	SS	0.78	0.80	0.93	0.95	1.09	1.09
	No2	0.72	0.70	0.92	0.89	1.14	1.10
Average		0.75	0.75	0.92	0.92	1.11	1.09

Osaka		β=3.0		β=2.5		β=2.0	
		γ=0.25	γ=4.40	γ=0.25	γ=4.40	γ=0.25	γ=4.40
2x4	SS	0.73	0.76	0.74	0.90	0.91	1.05
	No2	0.62	0.60	0.71	0.77	0.88	0.96
2x8	SS	0.71	0.72	0.75	0.87	0.91	1.03
	No2	0.71	0.73	0.73	0.87	0.89	1.02
2x10	SS	0.72	0.76	0.75	0.89	0.91	1.03
	No2	0.68	0.66	0.72	0.83	0.89	1.02
Average		0.70	0.71	0.73	0.86	0.90	1.02
Total Average		0.77	0.77	0.95	0.94	1.14	1.12

Table 16. Mean β-values Corresponding to Given ϕ in Compression

Performance Factor ϕ	Mean β γ=0.25	Mean β Actual γ_a
0.6	3.5	3.6
0.7	3.2	3.2
0.8	2.9	2.9
0.9	2.6	2.6
1.0	2.4	2.3

Table 17. Size Factor and Standard Strength R_0 for Compression

$\beta = 2.5$ at $\phi_0 = 0.9$ unit : MPa

γ	Size Factor	Grade	R_0	$R_{0.05}$
0.25	8.312	SS	21.676	19.32
		No.2	17.232	18.20
Actual [1)]	8.675	SS	21.784	19.32
		No.2	16.967	18.20

1) see Table 6a

Table 18a. Modified β-values in Compression at ϕ_0=0.9 and ϕ_0=0.85 (γ=0.25)

Size	Grade	Sapporo		Niigata		Tokyo		Osaka	
	ϕ_0	0.9	0.85	0.9	0.85	0.9	0.85	0.9	0.85
2x4	SS	2.90	3.05	2.70	2.86	2.54	2.70	2.33	2.49
	No.2	2.49	2.61	2.39	2.51	2.30	2.43	2.18	2.31
2x8	SS	2.66	2.80	2.49	2.64	2.36	2.51	2.18	2.33
	No.2	3.11	3.25	2.94	3.08	2.80	2.94	2.60	2.75
2x10	SS	2.78	2.95	2.55	2.73	2.37	2.55	2.15	2.32
	No.2	2.51	2.64	2.40	2.53	2.30	2.43	2.17	2.30
ϕ_0=0.9		Maximum=3.11			Minimum=2.15			Average=2.51	
ϕ_0=0.85		Maximum=3.25			Minimum=2.30			Average=2.65	

Table 18b. Modified β-values in Compression at ϕ_0=0.9 and ϕ_0=0.85 (Actual γ_a)

Size	Grade	Sapporo		Niigata		Tokyo		Osaka	
	ϕ_0	0.9	0.85	0.9	0.85	0.9	0.85	0.9	0.85
2x4	SS	2.90	3.05	2.70	2.86	2.56	2.73	2.37	2.55
	No.2	2.53	2.65	2.41	2.53	2.27	2.40	2.10	2.24
2x8	SS	2.62	2.79	2.46	2.62	2.31	2.47	2.12	2.29
	No.2	3.15	3.29	2.97	3.12	2.85	3.00	2.68	2.84
2x10	SS	2.76	2.93	2.53	2.71	2.38	2.57	2.18	2.38
	No.2	2.54	2.66	2.40	2.53	2.26	2.40	2.09	2.23
ϕ_0=0.9		Maximum=3.15			Minimum=2.09			Average=2.51	
ϕ_0=0.85		Maximum=3.29			Minimum=2.23			Average=2.66	

Table 19. Performance Factors in Serviceability for S-P-F

(ϕ-values Corresponding to Target β-values)

Sapporo		β=2.0		β=1.5		β=1.0	
		γ=0.25	γ=0.25	γ=0.25	γ=0.25	γ=0.25	γ=0.25
2x4	SS	0.63	-	0.75	-	0.86	-
	No2	0.56	-	0.69	-	0.82	-
2x8	SS	0.64	-	0.75	-	0.87	-
	No2	0.59	-	0.71	-	0.84	-
2x10	SS	0.64	-	0.76	-	0.87	-
	No2	0.58	-	0.71	-	0.83	-
Average		0.61	-	0.73	-	0.85	-

Niigata		β=2.0		β=1.5		β=1.0	
		γ=0.25	γ=0.41	γ=0.25	γ=0.41	γ=0.25	γ=0.41
2x4	SS	0.63	0.64	0.75	0.76	0.88	0.88
	No2	0.57	0.57	0.70	0.71	0.85	0.85
2x8	SS	0.64	0.65	0.76	0.76	0.89	0.89
	No2	0.59	0.60	0.73	0.73	0.86	0.86
2x10	SS	0.64	0.65	0.76	0.77	0.89	0.89
	No2	0.58	0.59	0.72	0.72	0.86	0.86
Average		0.61	0.62	0.74	0.74	0.87	0.87

Tokyo		β=2.0		β=1.5		β=1.0	
		γ=0.25	γ=1.22	γ=0.25	γ=1.22	γ=0.25	γ=1.22
2x4	SS	0.62	0.65	0.75	0.77	0.89	0.88
	No2	0.56	0.58	0.71	0.71	0.86	0.84
2x8	SS	0.63	0.66	0.76	0.77	0.90	0.89
	No2	0.59	0.61	0.73	0.73	0.88	0.86
2x10	SS	0.63	0.66	0.76	0.78	0.90	0.89
	No2	0.58	0.60	0.72	0.72	0.87	0.85
Average		0.60	0.63	0.74	0.75	0.88	0.87

Osaka		β=2.0		β=1.5		β=1.0	
		γ=0.25	γ=4.40	γ=0.25	γ=4.40	γ=0.25	γ=4.40
2x4	SS	0.60	0.64	0.74	0.75	0.91	0.86
	No2	0.56	0.56	0.71	0.69	0.88	0.86
2x8	SS	0.61	0.65	0.75	0.76	0.91	0.86
	No2	0.58	0.59	0.73	0.71	0.89	0.83
2x10	SS	0.61	0.65	0.75	0.76	0.91	0.87
	No2	0.57	0.58	0.72	0.71	0.89	0.83
Average		0.59	0.61	0.73	0.73	0.90	0.85

Occupancy		β=2.0		β=1.5		β=1.0	
		γ=0.25	γ=0.28	γ=0.25	γ=0.28	γ=0.25	γ=0.28
2x4	SS	0.63	0.63	0.74	0.74	0.85	0.85
	No2	0.56	0.56	0.68	0.68	0.81	0.81
2x8	SS	0.64	0.64	0.75	0.75	0.86	0.86
	No2	0.59	0.59	0.71	0.71	0.83	0.83
2x10	SS	0.65	0.65	0.75	0.75	0.86	0.86
	No2	0.58	0.58	0.70	0.70	0.82	0.82
Average		0.61	0.61	0.72	0.72	0.84	0.84
Total Average		0.60	0.61	0.73	0.73	0.87	0.86

Table 20. Statistical Data for Analysis of Duration of Load Effect

Short Term Strength

	Mean (MPa)	S.D. (MPa)
SPF Q1	48.90	9.83
SPF Q2	25.77	7.09

Parameters of Damage Accumulation Model

	b		c		n		σ_0	
	Mean	COV	Mean	COV	Mean	COV	Mean	COV
SPF Q1	77.392	0.174	2.810×10^{-6}	0.057	1.162	0.231	0.420	0.038
SPF Q2	158.656	0.009	7.525×10^{-7}	0.042	1.285	0.170	0.365	0.562

Table 21. Duration of Load Effects for S-P-F Q1

Sapporo		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.96		1.10		1.25	
	DOL	0.69	0.72	0.77	0.70	0.87	0.70
	DOL-5	0.70	0.73	0.77	0.70	0.85	0.68

Niigata		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.87		1.01		1.16	
	DOL	0.70	0.80	0.79	0.78	0.91	0.78
	DOL-5	0.65	0.75	0.74	0.73	0.84	0.72
0.41	No-DOL	0.88		1.01		1.16	
	DOL	0.70	0.80	0.78	0.77	0.89	0.77

Tokyo		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.80		0.95		1.11	
	DOL	0.73	0.91	0.83	0.87	0.98	0.88
	DOL-5	0.62	0.77	0.72	0.76	0.82	0.74
1.22	No-DOL	0.86		0.98		1.11	
	DOL	0.72	0.84	0.79	0.81	0.90	0.81

Osaka		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.74		0.87		1.03	
	DOL	0.69	0.93	0.78	0.90	0.93	0.90
	DOL-5	0.56	0.76	0.66	0.76	0.77	0.75
4.40	No-DOL	0.84		0.94		1.05	
	DOL	0.48	0.57	0.54	0.57	0.60	0.57

Table 22. Duration of Load Effects for S-P-F Q2

Sapporo		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.91		1.07		1.24	
	DOL	0.51	0.56	0.58	0.54	0.68	0.55
	DOL-5	0.48	0.53	0.56	0.52	0.65	0.52

Niigata		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.84		0.99		1.17	
	DOL	0.63	0.75	0.73	0.74	0.85	0.73
	DOL-5	0.51	0.61	0.61	0.62	0.72	0.62
0.41	No-DOL	0.84		0.99		1.16	
	DOL	0.57	0.68	0.67	0.68	0.78	0.67

Tokyo		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.78		0.94		1.12	
	DOL	0.72	0.92	0.87	0.93	1.01	0.90
	DOL-5	0.54	0.69	0.64	0.68	0.75	0.67
1.22	No-DOL	0.81		0.95		1.10	
	DOL	0.39	0.48	0.46	0.48	0.54	0.49

Osaka		$\beta = 3.0$		$\beta = 2.5$		$\beta = 2.0$	
γ		ϕ	K_D	ϕ	K_D	ϕ	K_D
0.25	No-DOL	0.73		0.88		1.06	
	DOL	0.67	0.92	0.83	0.94	0.98	0.92
	DOL-5	0.54	0.74	0.65	0.74	0.77	0.73
4.40	No-DOL	0.78		0.90		1.03	
	DOL	0.25	0.32	0.30	0.33	0.36	0.35

Table 23. Bending Reliability Levels for Current 2x4 Wood Frame Structure (Short-Term Basis)

Floor Joist

Size	Grade	Spacing	γ	β
2x8	SS	455 mm	0.30	2.778
		303	0.34	2.782
	No2	405	0.30	2.938
		303	0.34	2.942
2x10	SS	455	0.33	2.680
		303	0.37	2.686
	No2	455	0.33	3.021
		303	0.37	3.028

Rafters (light weight roofing material)

			Sapporo		Niigata		Tokyo		Osaka	
Size	Grade	Spacing	γ	β	γ	β	γ	β	γ	β
2x4	SS	455 mm	0.10	3.608	0.17	3.542	0.48	2.382	1.74	2.921
		303	0.12	3.610	0.19	3.547	0.54	2.396*	1.96	3.003
	No2	405	0.10	3.387	0.17	3.332	0.48	2.258	1.74	2.476
		303	0.12	3.389	0.19	3.337	0.54	2.269	1.96	2.821
2x8	SS	455	0.12	2.817	0.19	2.765	0.54	1.566	1.96	2.189
		303	0.14	2.820	0.22	2.771	0.64	1.580	2.29	2.286
	No2	455	0.12	2.976	0.19	2.925	0.54	1.780	1.96	2.373
		303	0.14	2.978	0.22	2.930	0.64	1.795	2.29	2.466
2x10	SS	455	0.13	2.720	0.21	2.666*	0.61	1.384*	2.18	2.121
		303	0.16	2.723	0.25	2.674	0.73	1.402	2.62	2.232
	No2	455	0.13	3.051	0.21	2.973	0.61	1.456	2.18	2.360
		303	0.16	3.056	0.25	2.985	0.73	1.482	2.62	2.493

Rafters (Heavy weight roofing material)

			Sapporo		Niigata		Tokyo		Osaka	
Size	Grade	Spacing	γ	β	γ	β	γ	β	γ	β
2x4	SS	455 mm	0.22	3.626	0.36	3.581	1.07	2.532	3.84	3.335
		303	0.23	3.628*	0.38	3.585	1.13	2.588	4.06	3.354*
	No2	405	0.22	3.402	0.36	3.365	1.07	2.387	3.84	3.127
		303	0.23	3.404	0.38	3.368	1.13	2.438	4.06	3.145
2x8	SS	455	0.23	2.832	0.38	2.796	1.13	1.753	4.06	2.547
		303	0.25	2.834	0.41	2.800	1.22	1.831	4.39	2.573
	No2	455	0.23	2.990	0.38	2.955	1.13	1.959	4.06	2.715
		303	0.25	2.992	0.41	2.959	1.22	2.033	4.39	2.740
2x10	SS	455	0.24	2.735	0.40	2.698	1.19	1.634*	4.28	2.450
		303	0.27	2.737	0.44	2.704	1.31	1.735	4.72	2.483
	No2	455	0.24	3.073	0.40	3.020	1.19	1.775	4.28	2.751
		303	0.27	3.077	0.44	3.028	1.31	1.899	4.72	2.790

Table 24. Deflection Limits

Member	Allowable Deflection (Long-Term)	Allowable Deflection (Short-Term)
Floor Joist Floor Beam	L/300 and 2.0 cm	N/A
Ceiling Joist Flat Roof Joist Rafter Rafter Beam	L/200 and 2.0 cm	L/100 and 4.0 cm
Header	L/300 and 1.0cm	L/150 and 2.0 cm

Table 25. Serviceability Reliability Levels for Current 2x4 Wood Frame Structures

Floor Joist

Size	Grade	Spacing	γ	β
2x8	SS	455 mm	0.30	1.143
		303	0.34	1.167
	No2	405	0.30	1.200
		303	0.34	1.219
2x10	SS	455	0.33	1.041
		303	0.37	1.073
	No2	455	0.33	0.995
		303	0.37	1.020

Rafters (light weight roofing material)

			Sapporo		Niigata		Tokyo		Osaka	
Size	Grade	Spacing	γ	β	γ	β	γ	β	γ	β
2x4	SS	455 mm	0.10	1.210	0.17	1.389	0.48	1.893	1.74	2.325
		303	0.12	1.228	0.19	1.423	0.54	1.974	1.96	2.329
	No2	405	0.10	1.011	0.17	1.165	0.48	1.619	1.74	2.167
		303	0.12	1.027	0.19	1.194	0.54	1.672	1.96	2.182
2x8	SS	455	0.12	1.085	0.19	1.296	0.54	1.857	1.96	2.255
		303	0.14	1.114	0.22	1.348	0.64	1.929	2.29	2.260
	No2	455	0.12	1.153	0.19	1.328	0.54	1.818	1.96	2.273
		303	0.14	1.177	0.22	1.372	0.64	1.886	2.29	2.283
2x10	SS	455	0.13	0.985	0.21	1.222	0.61	1.824	2.18	2.200
		303	0.16	1.024	0.25	1.291	0.73	1.907	2.62	2.205
	No2	455	0.13	0.951	0.21	1.147	0.61	1.681	2.18	2.155
		303	0.16	0.983	0.25	1.205	0.73	1.764	2.62	2.169

Rafters (Heavy weight roofing material)

			Sapporo		Niigata		Tokyo		Osaka	
Size	Grade	Spacing	γ	β	γ	β	γ	β	γ	β
2x4	SS	455 mm	0.22	1.363	0.36	1.651	1.07	2.197	3.84	2.334
		303	0.23	1.377	0.38	1.674	1.13	2.211	4.06	2.334
	No2	405	0.22	1.140	0.36	1.394	1.07	1.946	3.84	2.227
		303	0.23	1.153	0.38	1.414	1.13	1.964	4.06	2.228
2x8	SS	455	0.23	1.246	0.38	1.566	1.13	2.131	4.06	2.265
		303	0.25	1.270	0.41	1.601	1.22	2.150	4.39	2.265
	No2	455	0.23	1.286	0.38	1.557	1.13	2.098	4.06	2.300
		303	0.25	1.305	0.41	1.587	1.22	2.120	4.39	2.301
2x10	SS	455	0.24	1.149	0.40	1.493	1.19	2.072	4.28	2.212
		303	0.27	1.181	0.44	1.539	1.31	2.094	4.72	2.212
	No2	455	0.24	1.085	0.40	1.377	1.19	1.951	4.28	2.190
		303	0.27	1.111	0.44	1.418	1.31	1.979	4.72	2.192

Table 26. Typical Rafter Span

Tokyo, Light Roofing, Spacing=455mm

Grade		2x4	2x6	2x8	2x10	unit
SS	D-Span	310.81	462.75	565.13	672.46	cm
	B-Span	426.47	664.39	864.58	1082.9	cm
	S-Span	1115.5	1720.3	2215.5	2722.5	cm
	Span	3.10	4.62	5.65	6.72	m

Sapporo, Light Roofing, Spacing=303mm

Grade		2x4	2x6	2x8	2x10	unit
No.2	D-Span	175.72	275.74	361.35	443.51	cm
	B-Span	159.36	249.76	326.92	414.09	cm
	S-Span	228.45	356.55	464.59	583.83	cm
	Span	1.59	2.49	3.26	4.14	m

Niigata, Heavy Roofing, Spacing=455mm

Grade		2x4	2x6	2x8	2x10	unit
No.2	D-Span	168.79	264.97	347.32	430.73	cm
	B-Span	150.02	235.27	308.06	390.56	cm
	S-Span	202.47	316.38	412.54	519.38	cm
	Span	1.50	2.35	3.08	3.90	m

Tokyo, Light Roofing, Spacing=455

Grade		2x4	2x6	2x8	2x10	unit
No.2	D-Span	298.11	448.50	547.72	651.74	cm
	B-Span	352.14	548.60	713.90	894.20	cm
	S-Span	1115.5	1720.3	2215.5	2722.5	cm
	Span	2.98	4.48	5.47	6.51	m

Osaka, Heavy Roofing, Spacing=455mm

Grade		2x4	2x6	2x8	2x10	unit
No.2	D-Span	263.91	408.52	497.98	590.50	cm
	B-Span	293.31	455.16	590.11	734.04	cm
	S-Span	773.89	1184.2	1513.8	1834.6	cm
	Span	2.63	4.08	4.97	5.90	m

where

D-Span ; Span Calculated by Allowable Deflection
B-Span ; Span Calculated by Allowable Bending Unit Stress
S-Span ; Span Calculated by Allowable Shearing Unit Stress
Span ; Minimum of Above Three Calculated Span

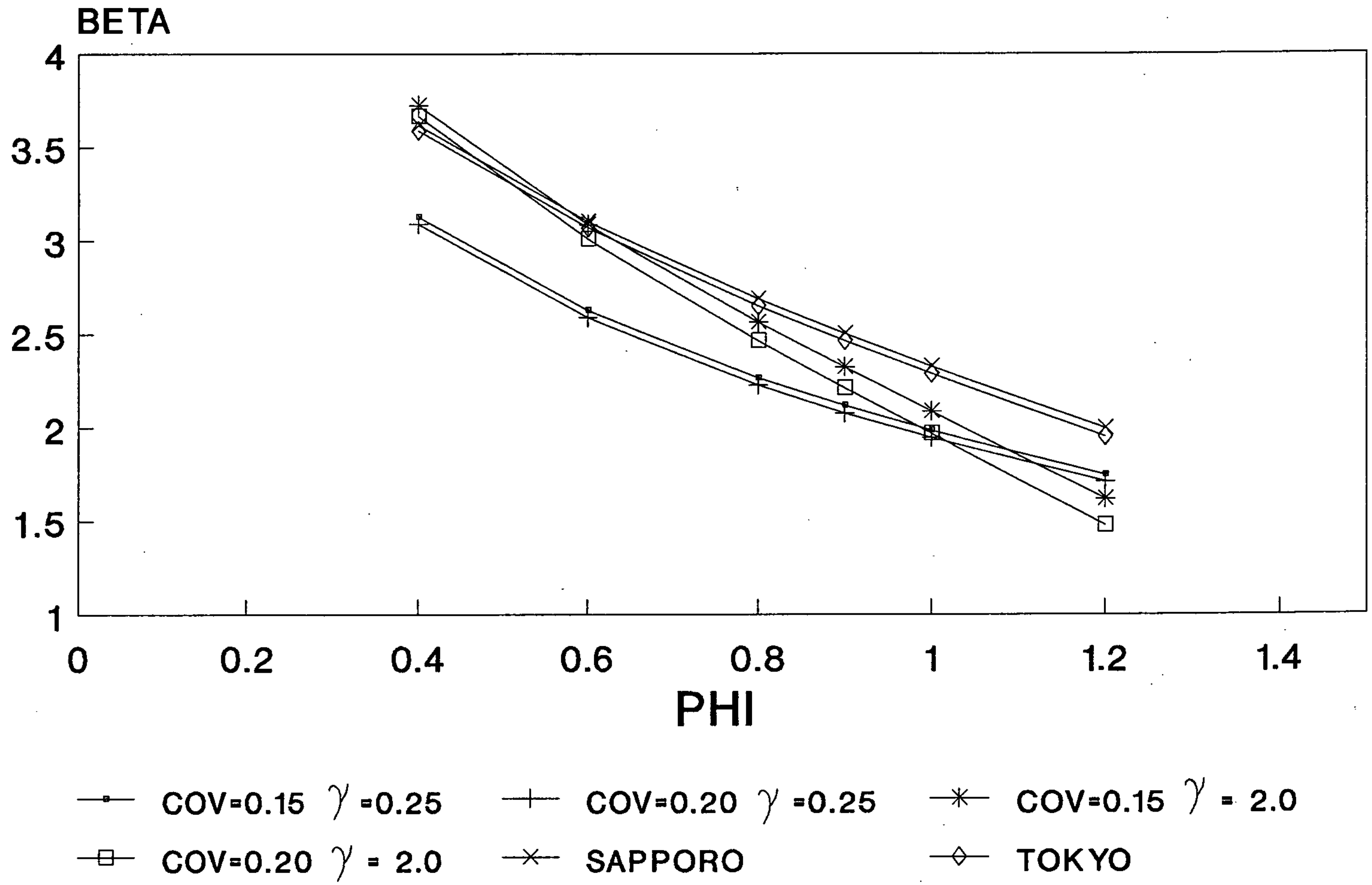


Figure 1. Beta vs Phi for Japanese Steel Code

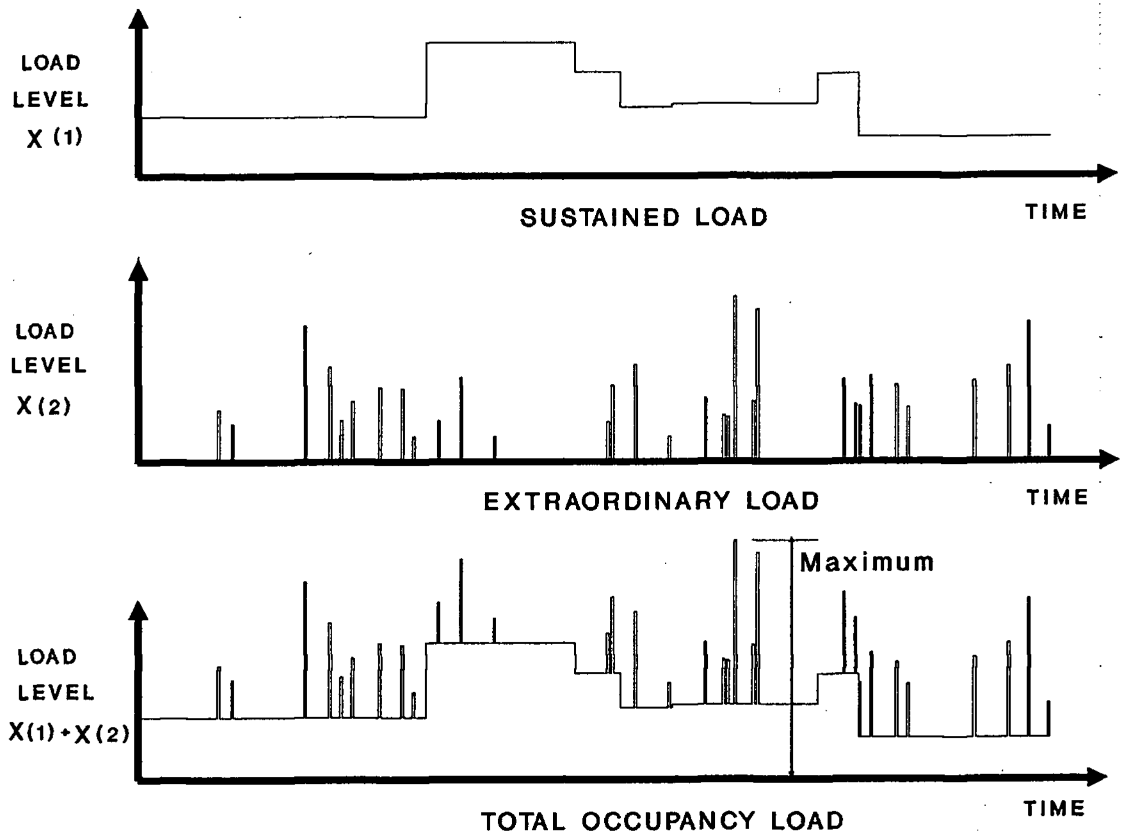


Figure 2. Occupancy Load Model

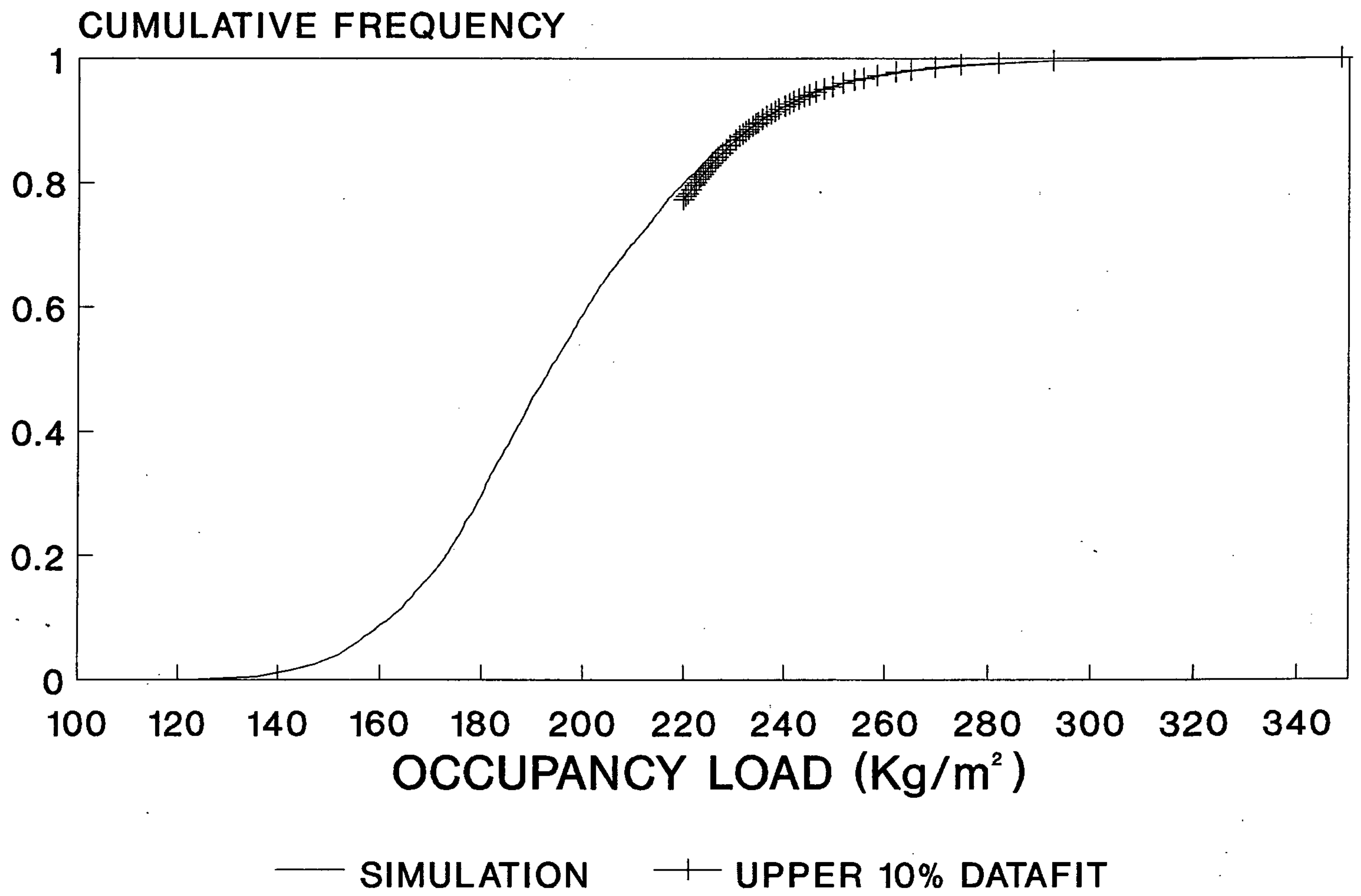


Figure 3. Japanese Occupancy Load (50-Year Return)

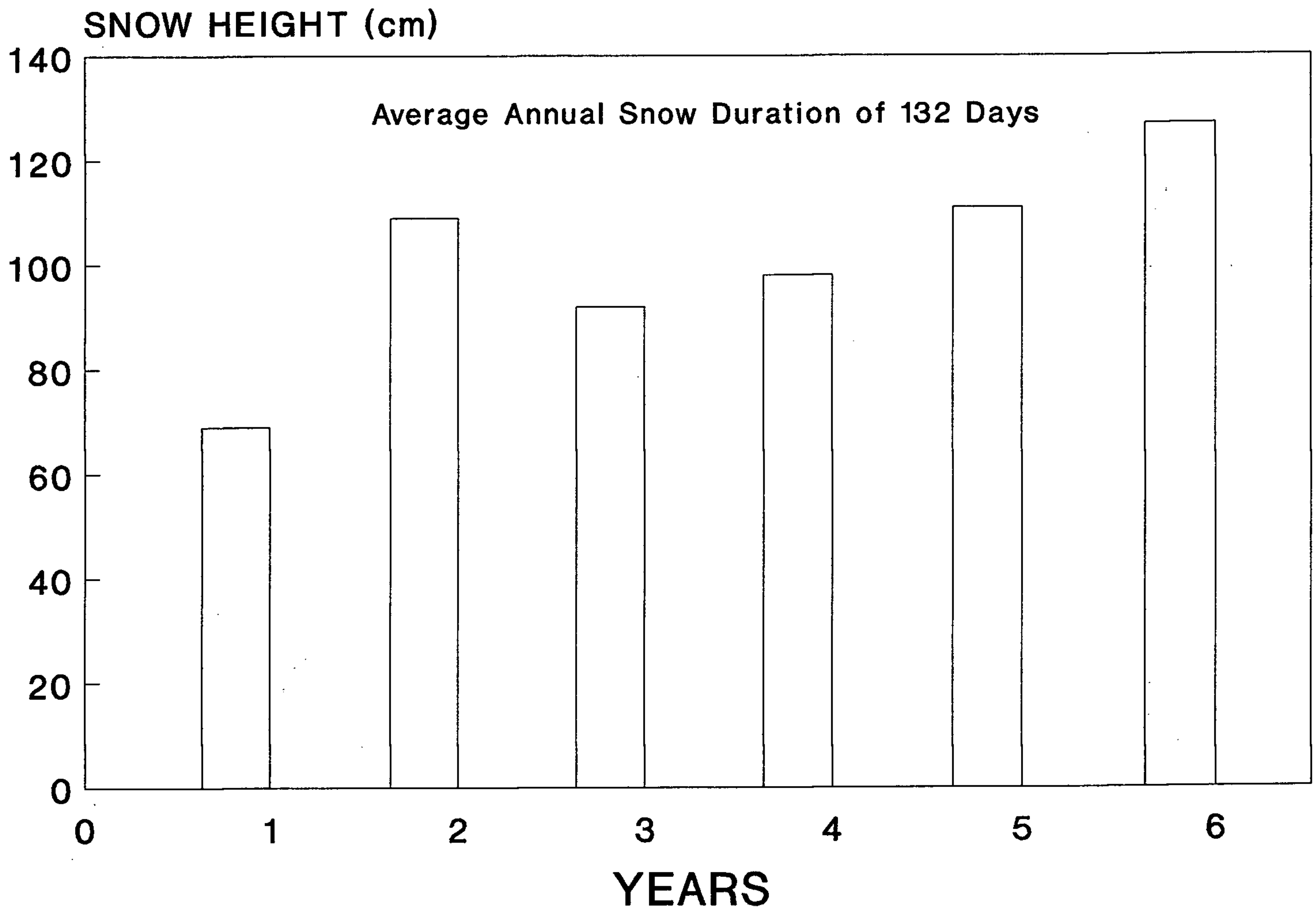


Figure 4. Snow Model in Sapporo

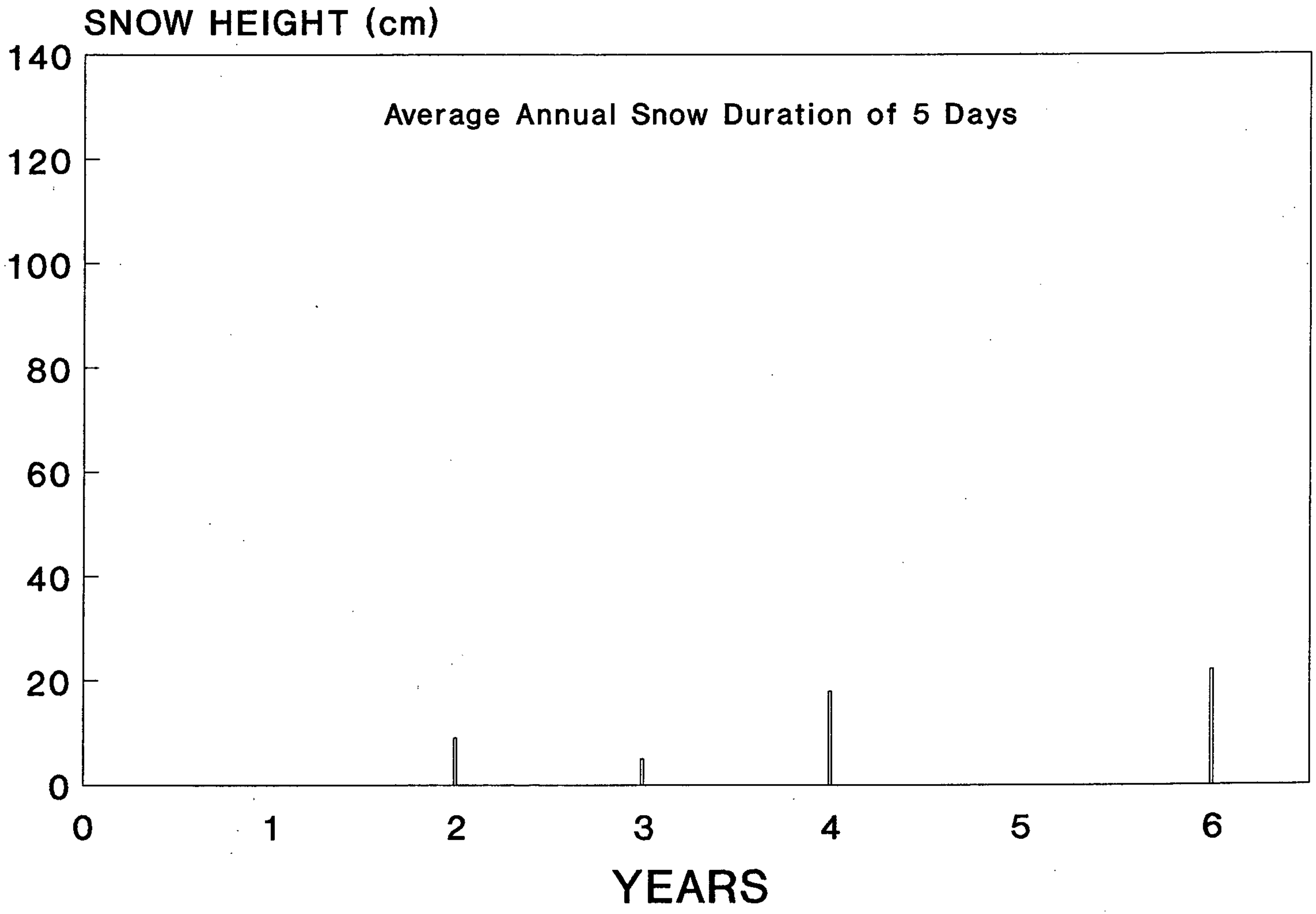
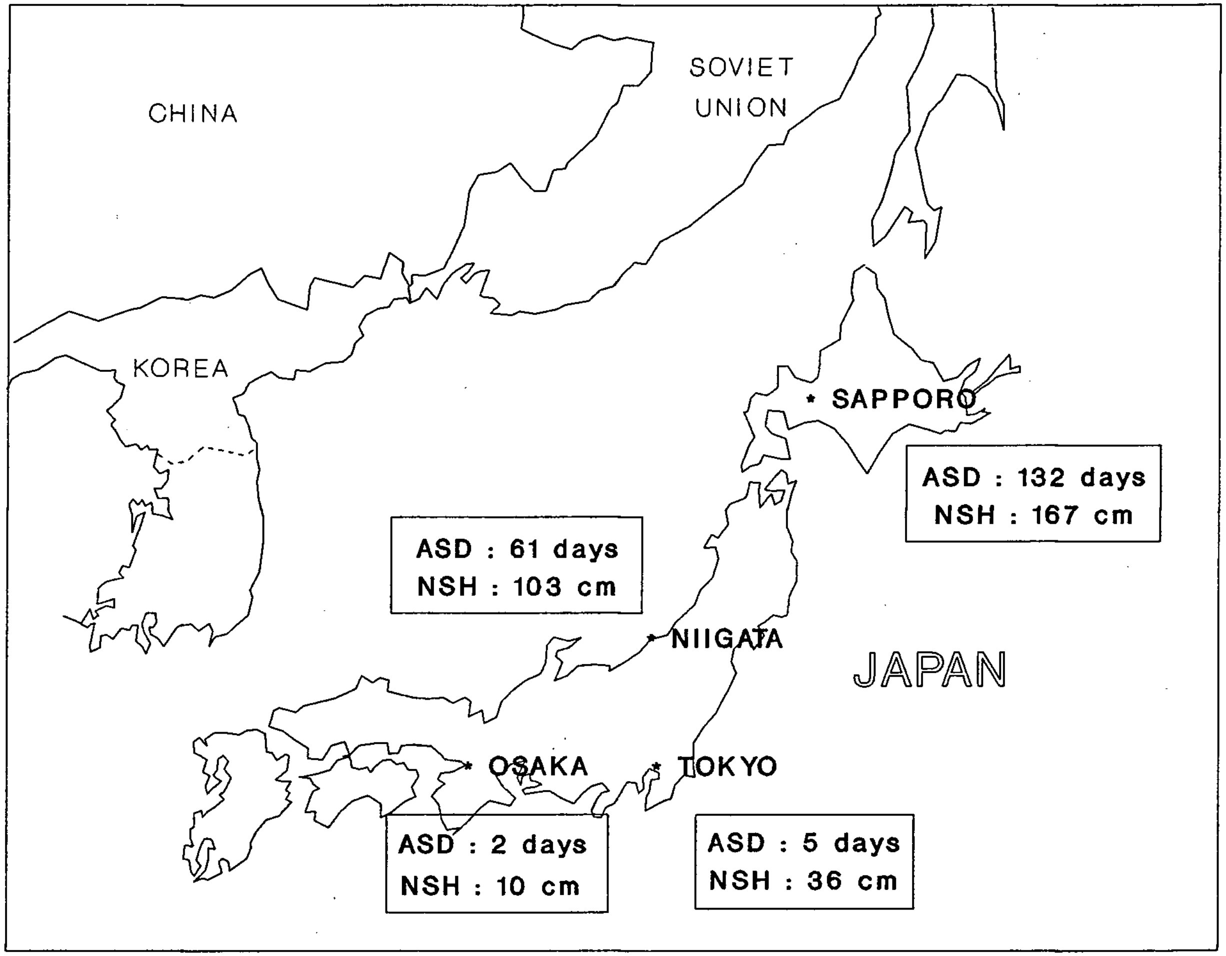


Figure 5. Snow Model in Tokyo



ASD: Annual Snow Duration
NSH: Nominal Snow Height

Figure 6. Location and Snow Data

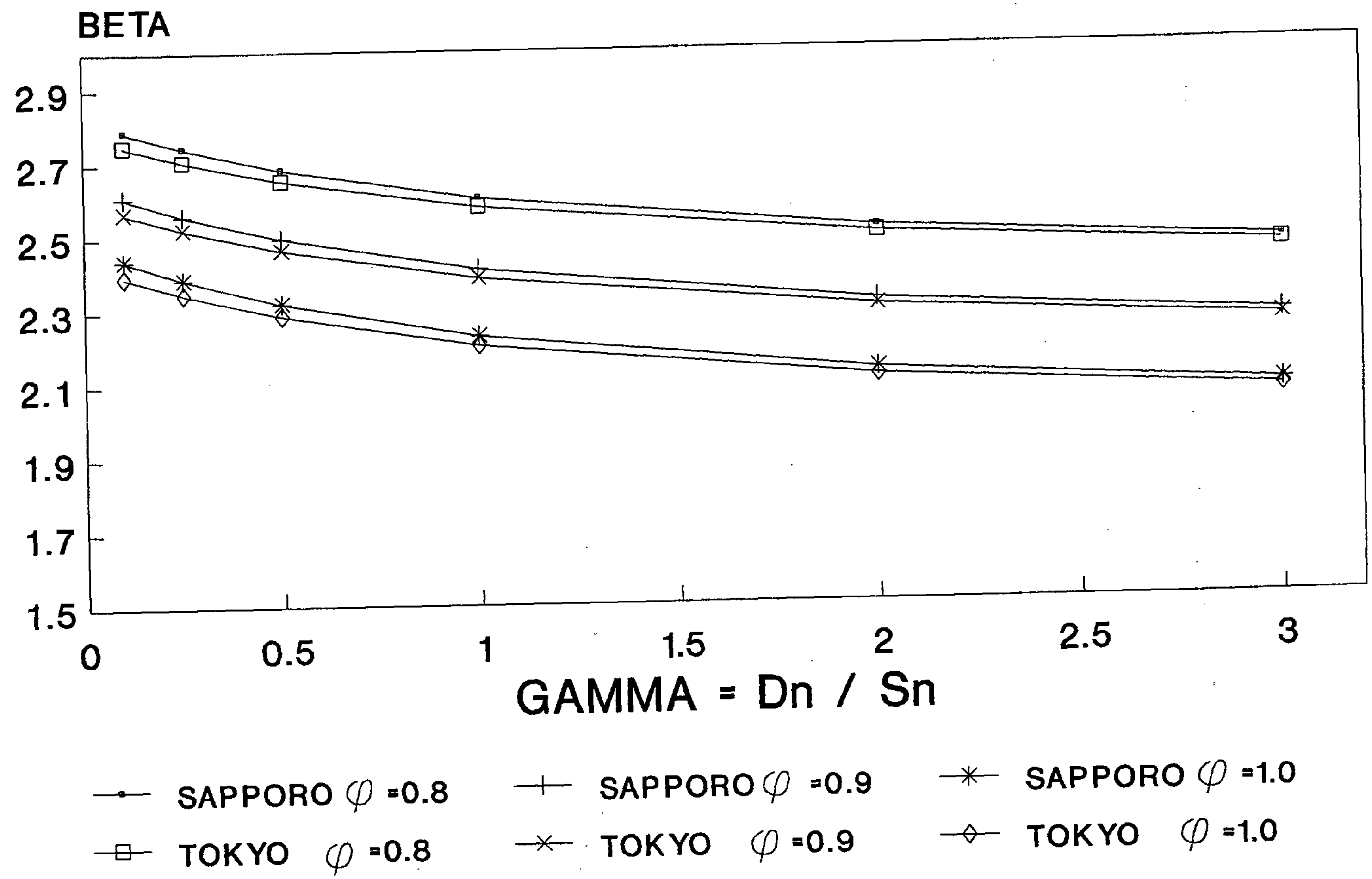


Figure 7. Beta vs Gamma (2x8, No.2)

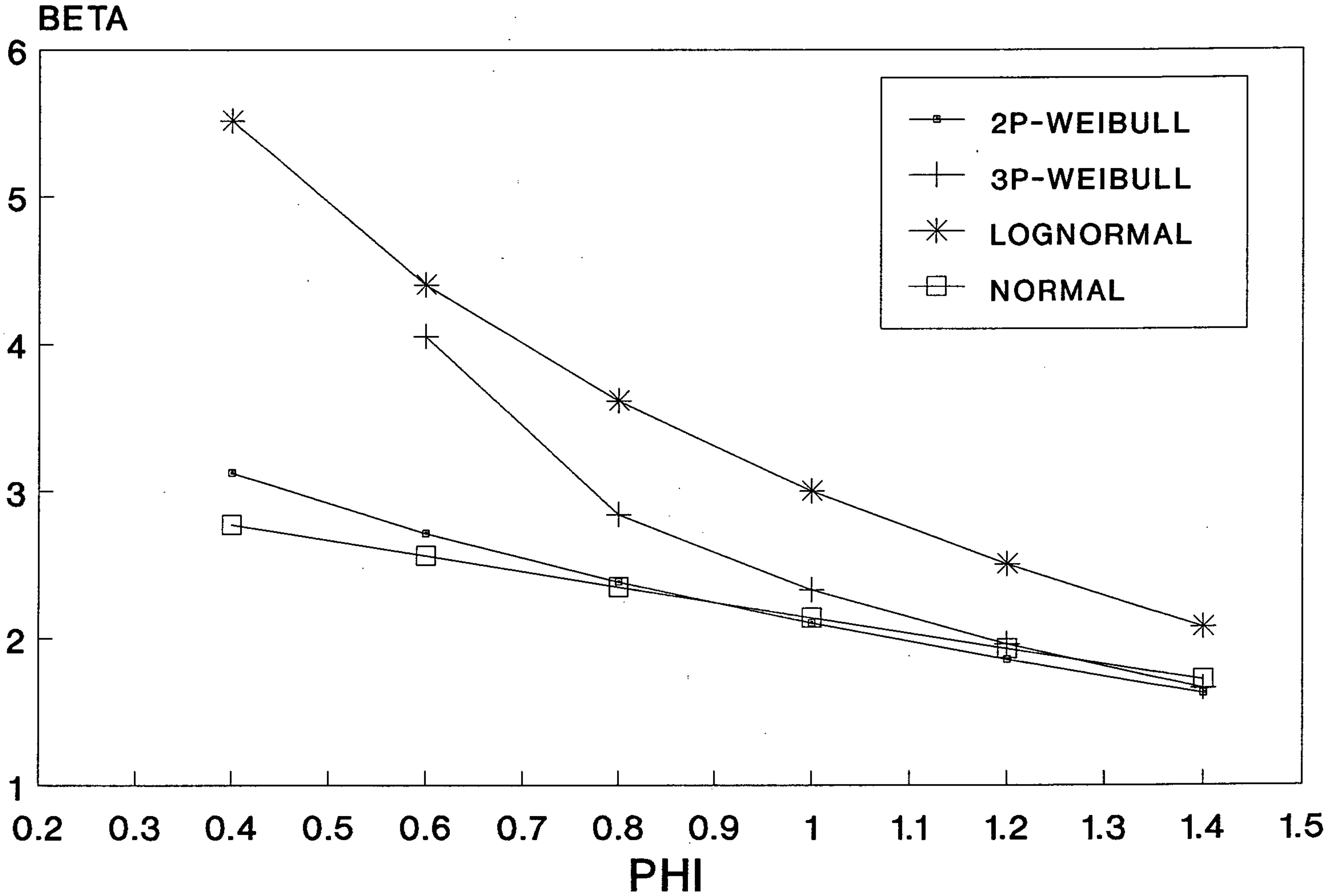


Figure 8. Bending Beta vs Phi in Tokyo (2x8, No.2, All Data)

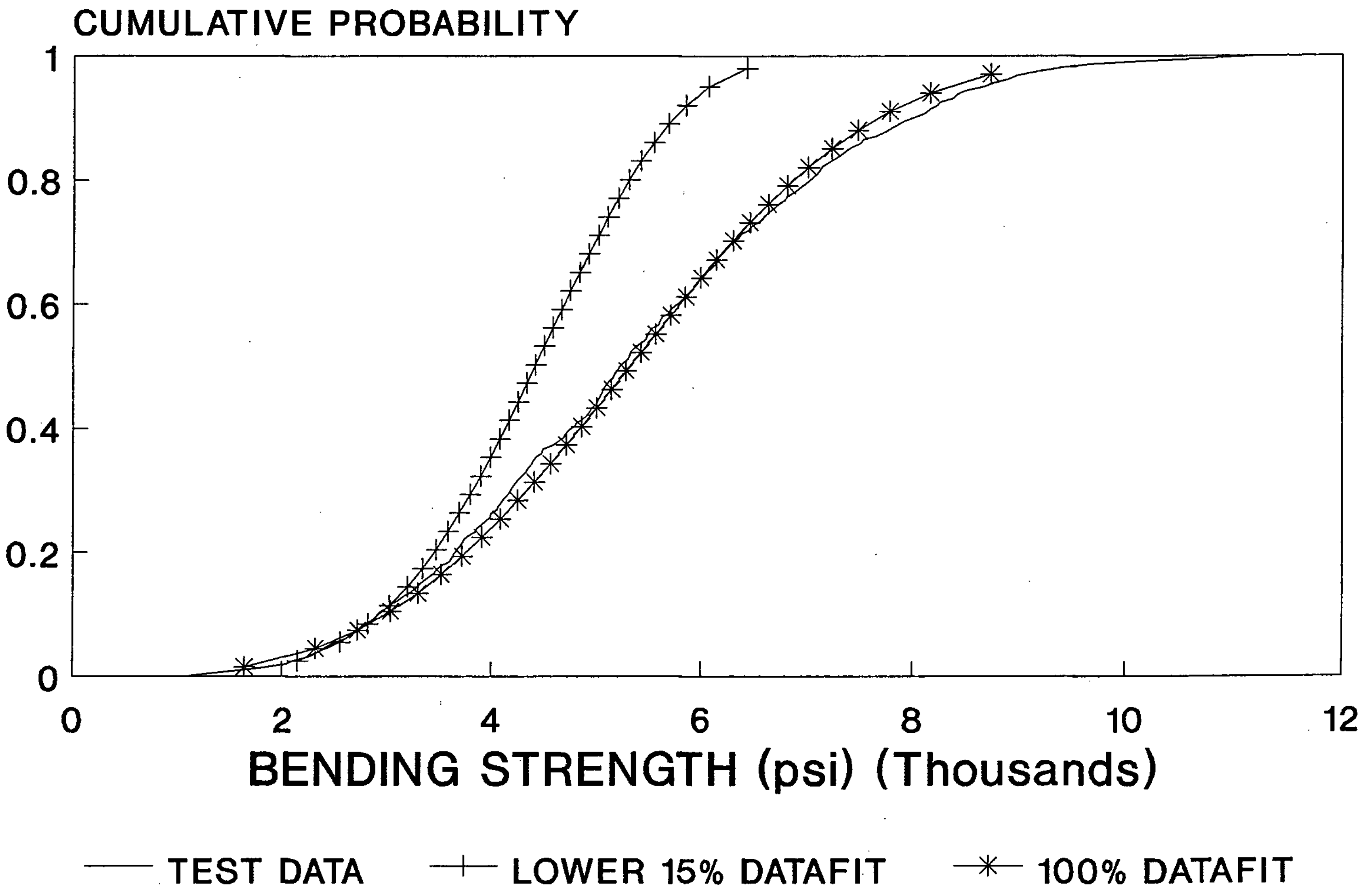


Figure 9. 2P Weibull Datafit (1) (2x8, No.2, CWC Test Data)

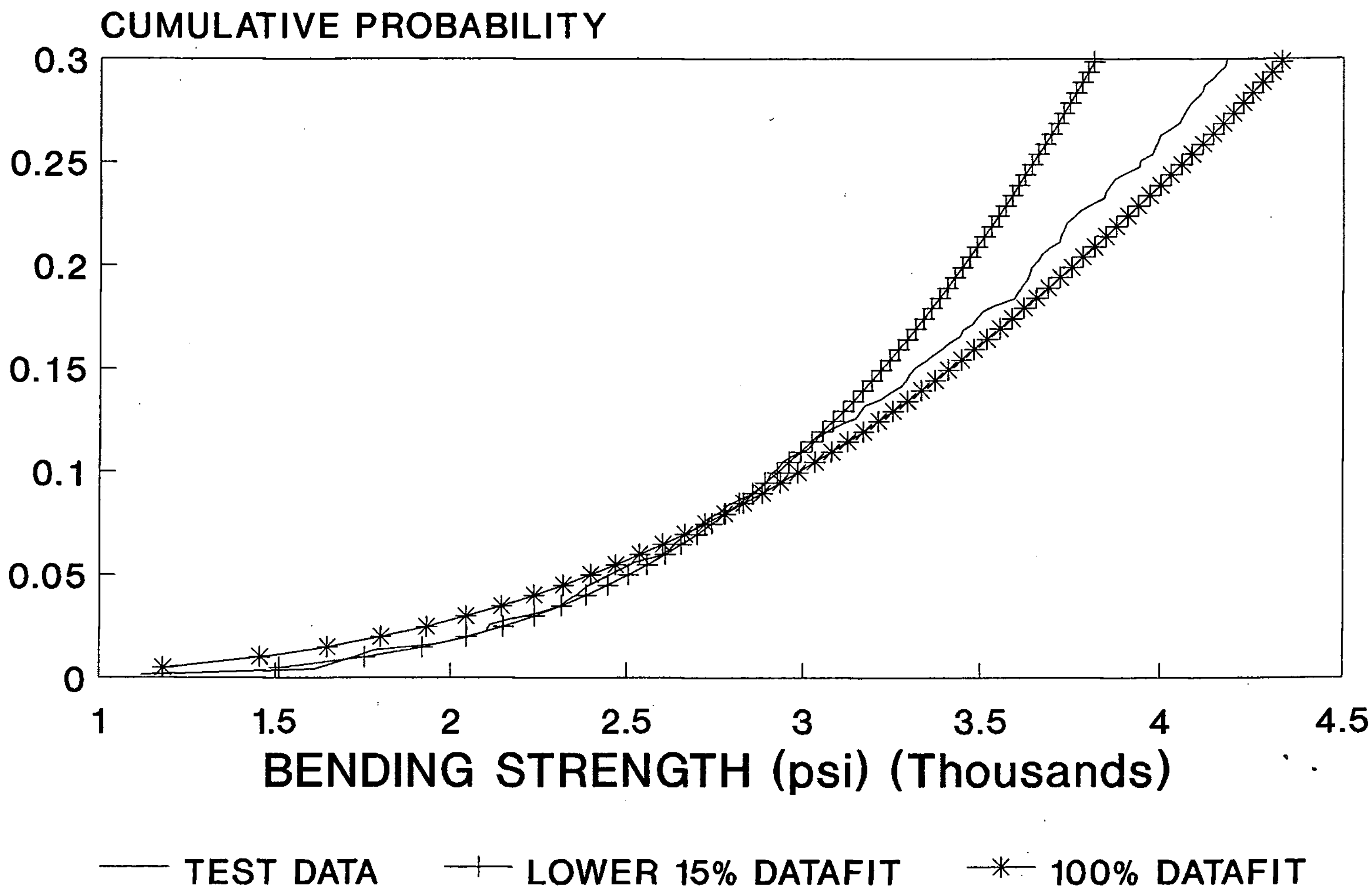


Figure 10. 2P Weibull Datafit (2) (2x8, No.2, CWC Test Data)

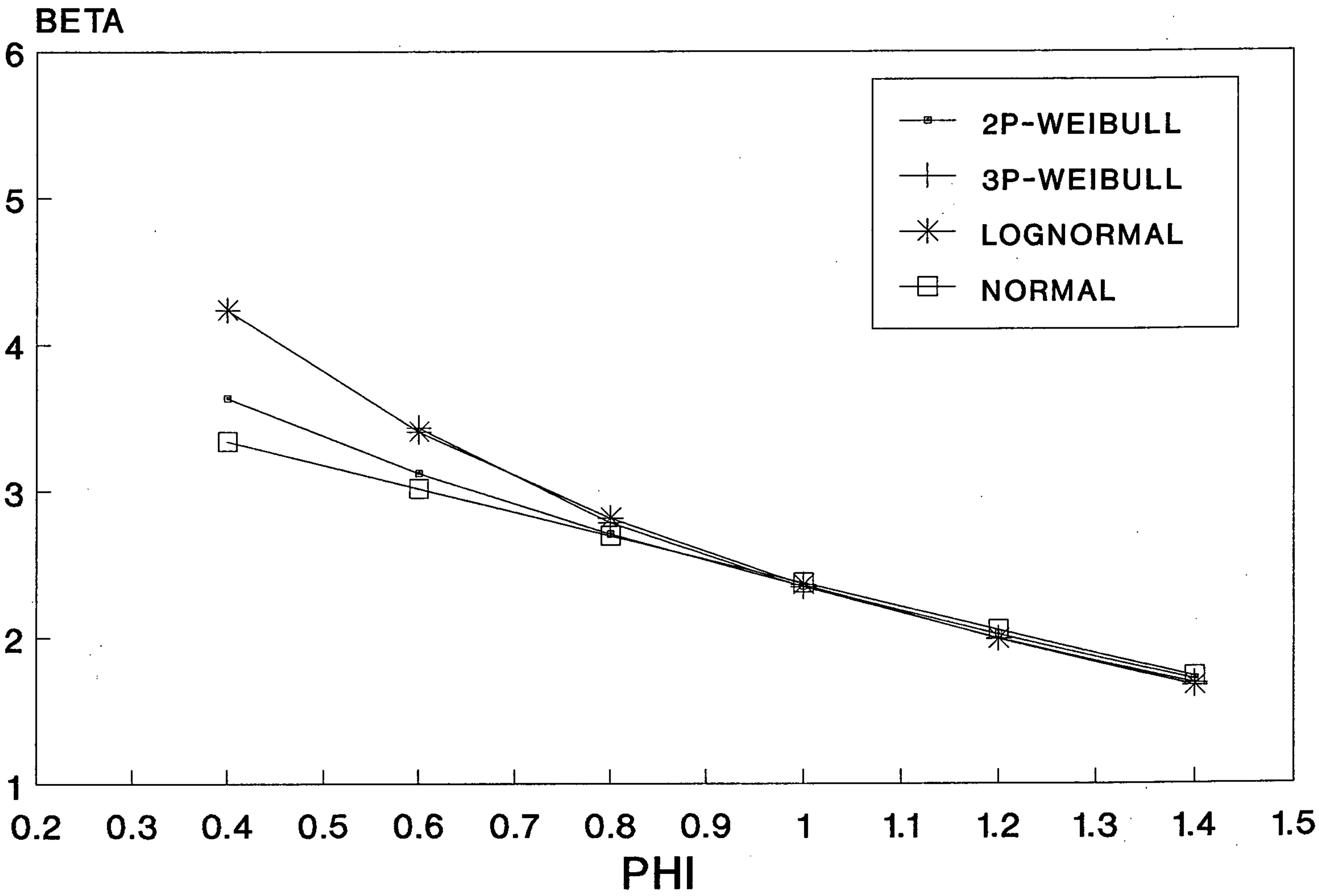


Figure 11. Bending Beta vs Phi in Tokyo (2x8, No.2, 15% Truncation)

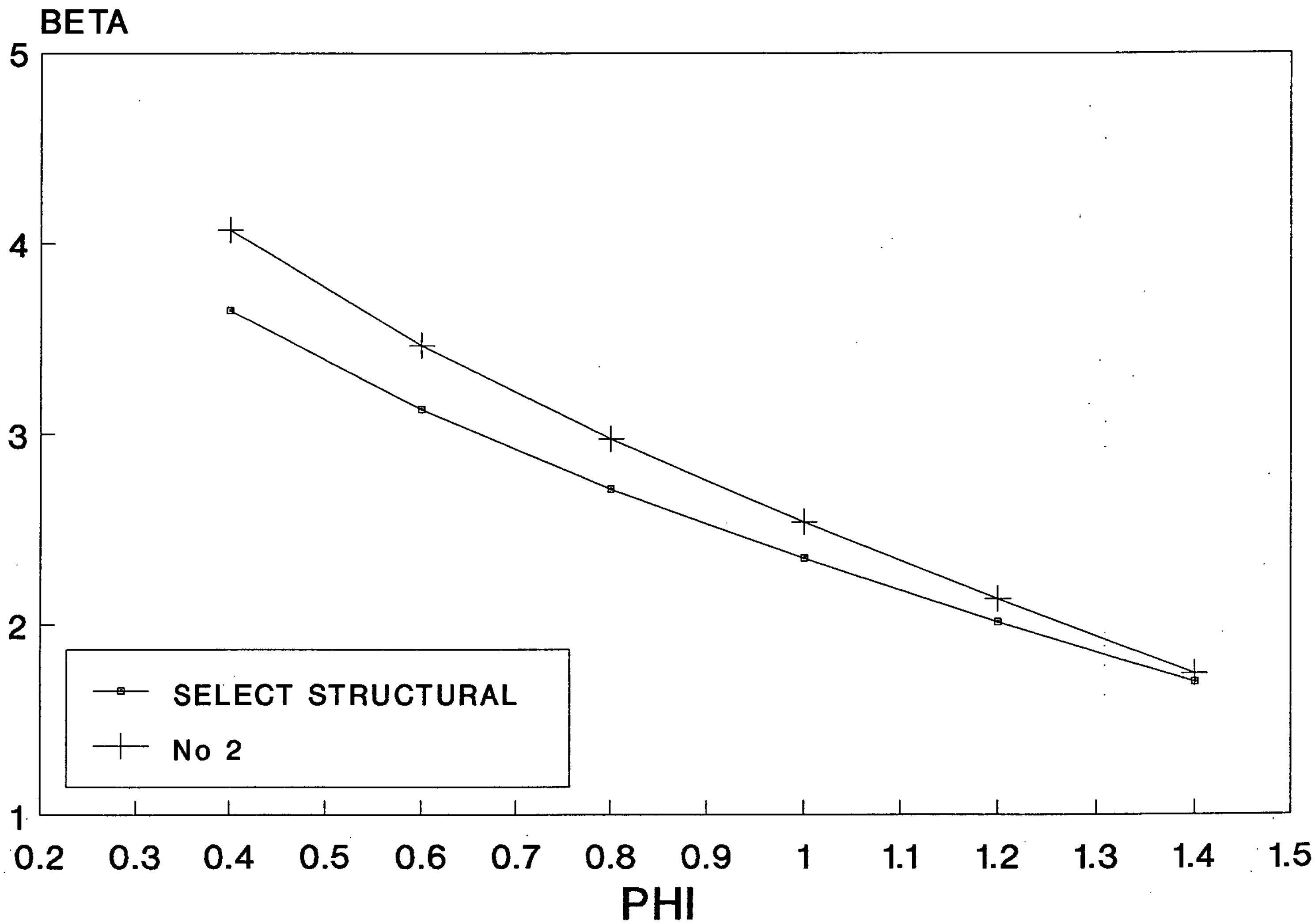


Figure 12. Bending Beta vs Phi (SS-No.2) (Tokyo, 2x10, 15% Truncation)

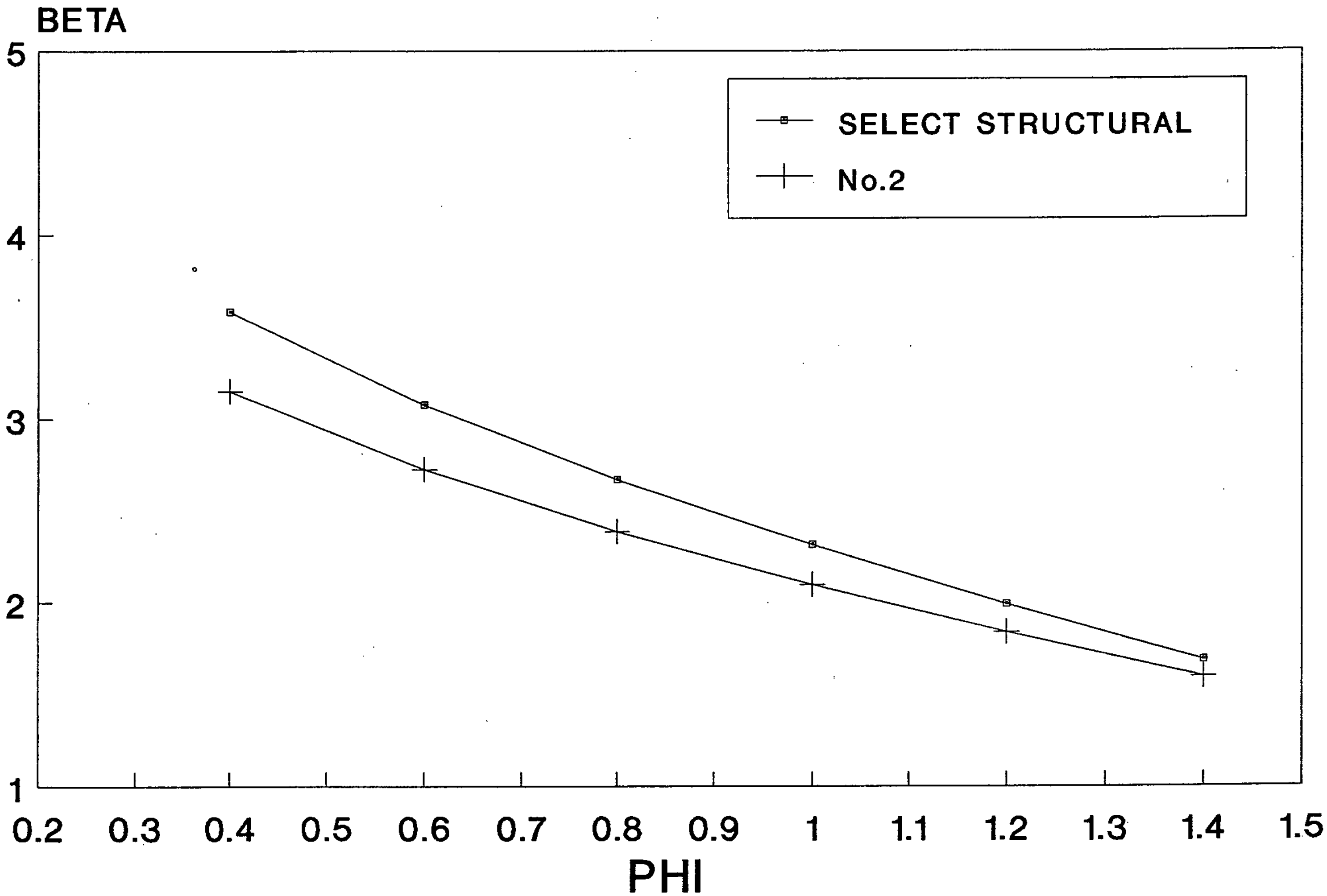


Figure 13. Bending Beta vs Phi (SS-No.2) (Tokyo, 2x10, All Data)

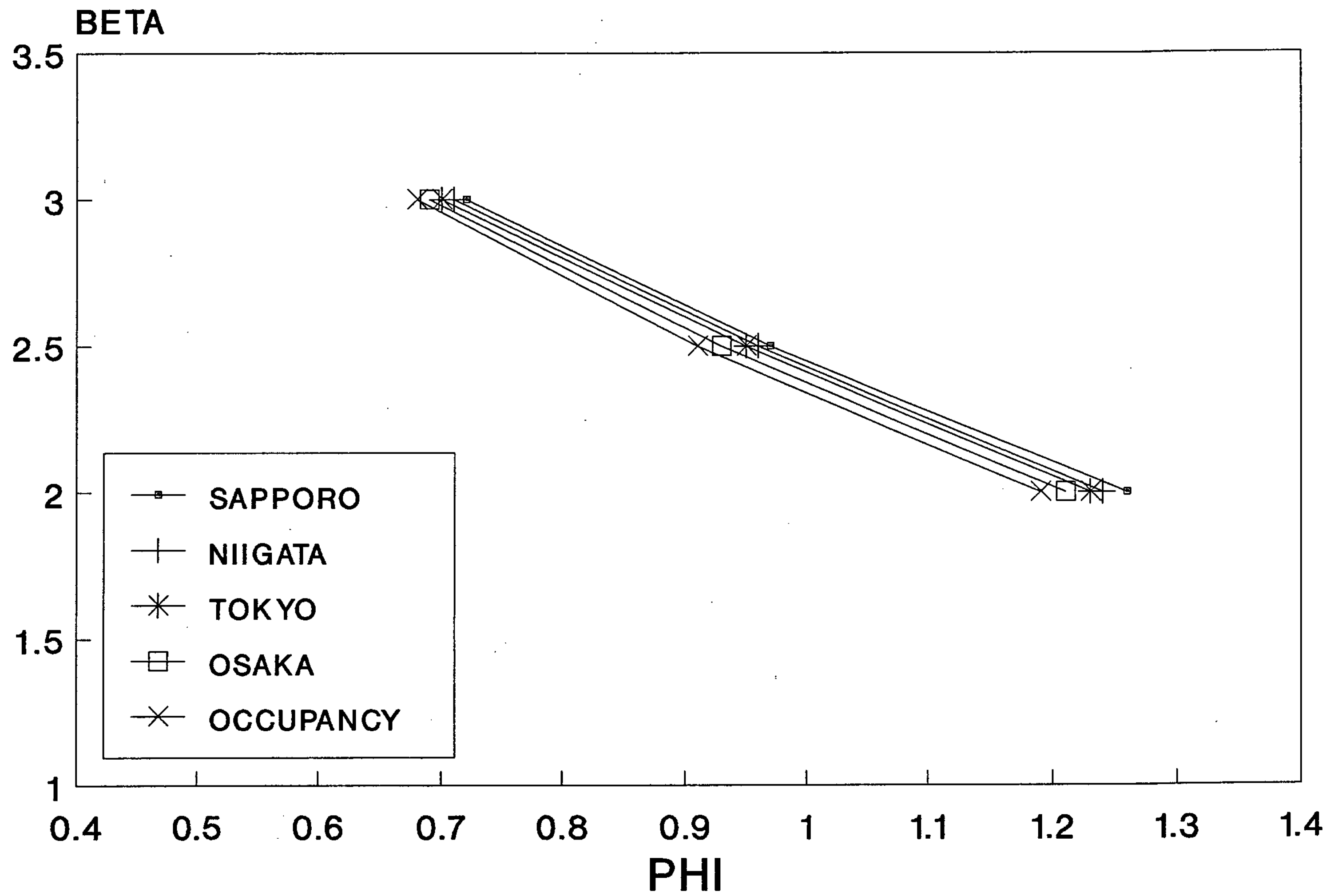


Figure 14. Bending Beta vs Phi (All Cases)

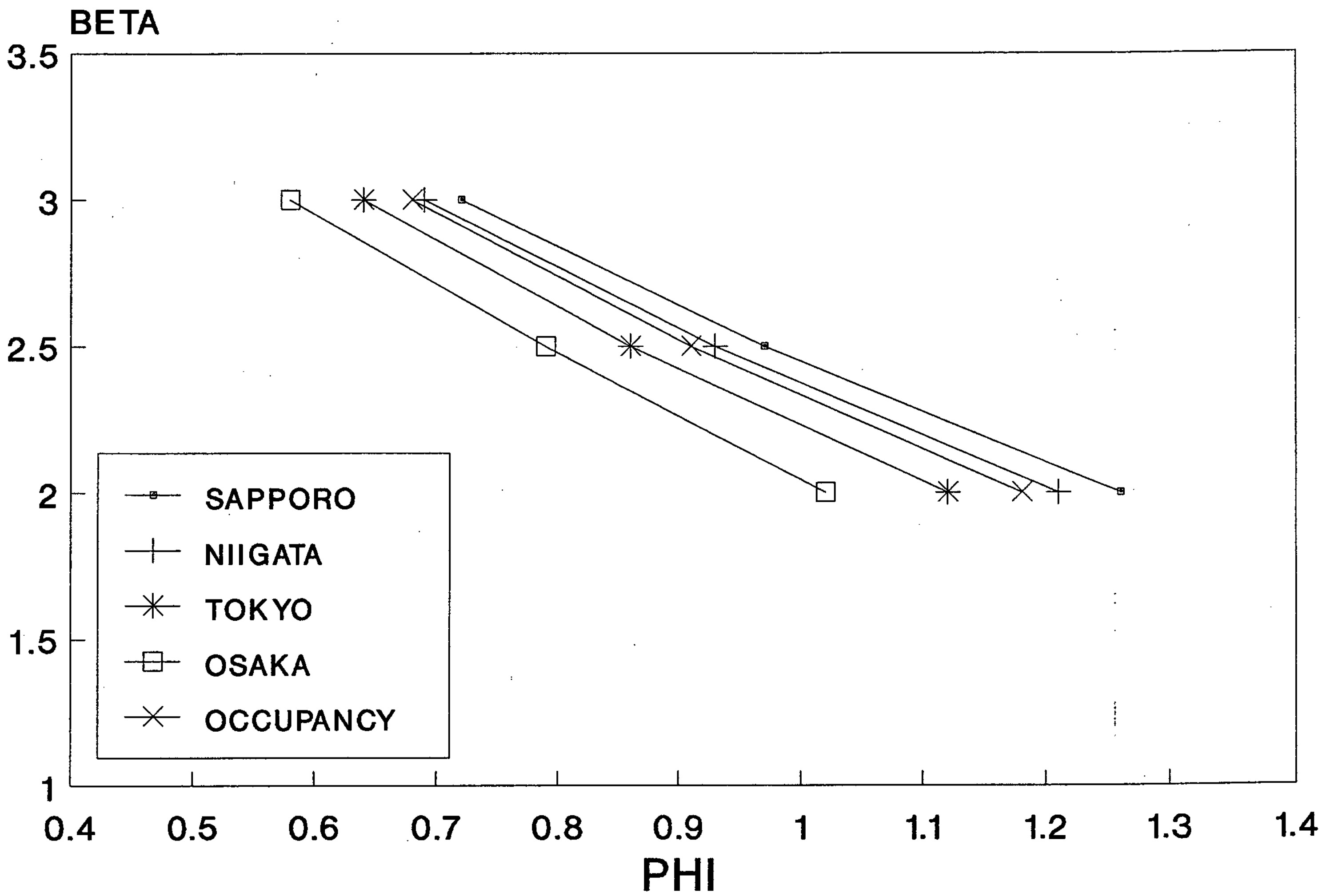


Figure 15. Bending Beta vs Phi (All Cases, Actual Gamma)

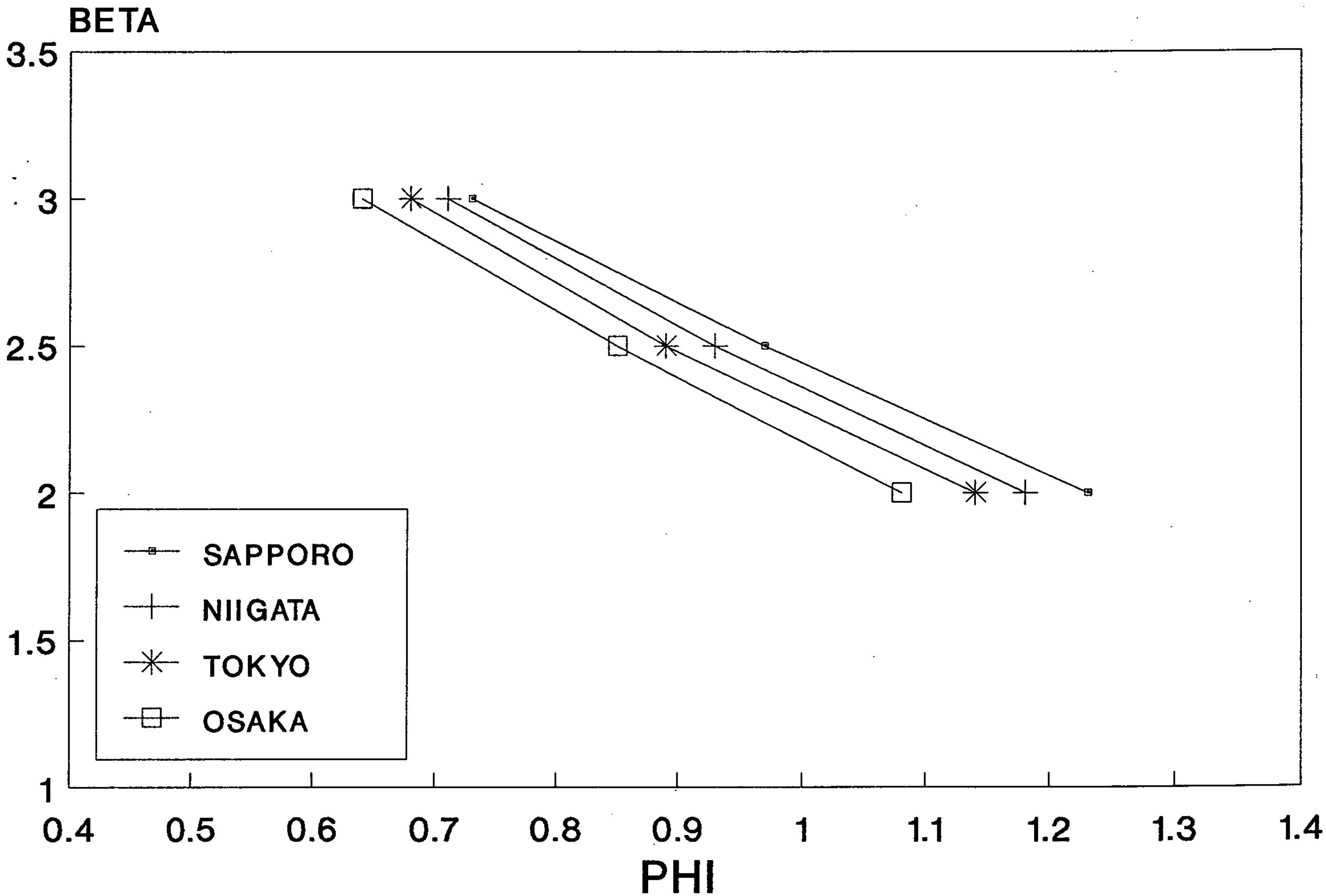


Figure 16. Beta vs Phi in Tension (All Cases)

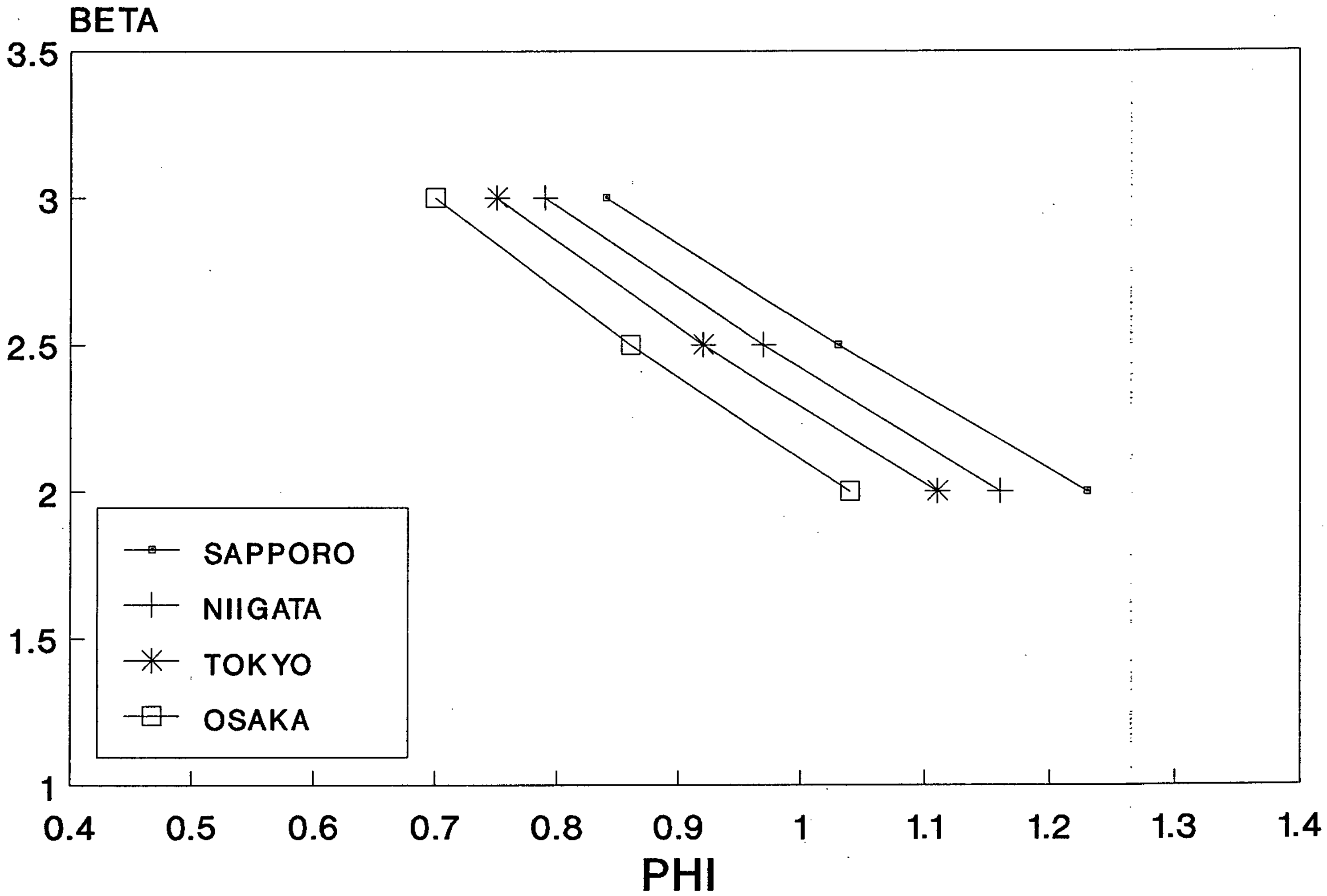


Figure 17. Beta vs Phi in Compression (All Cases)

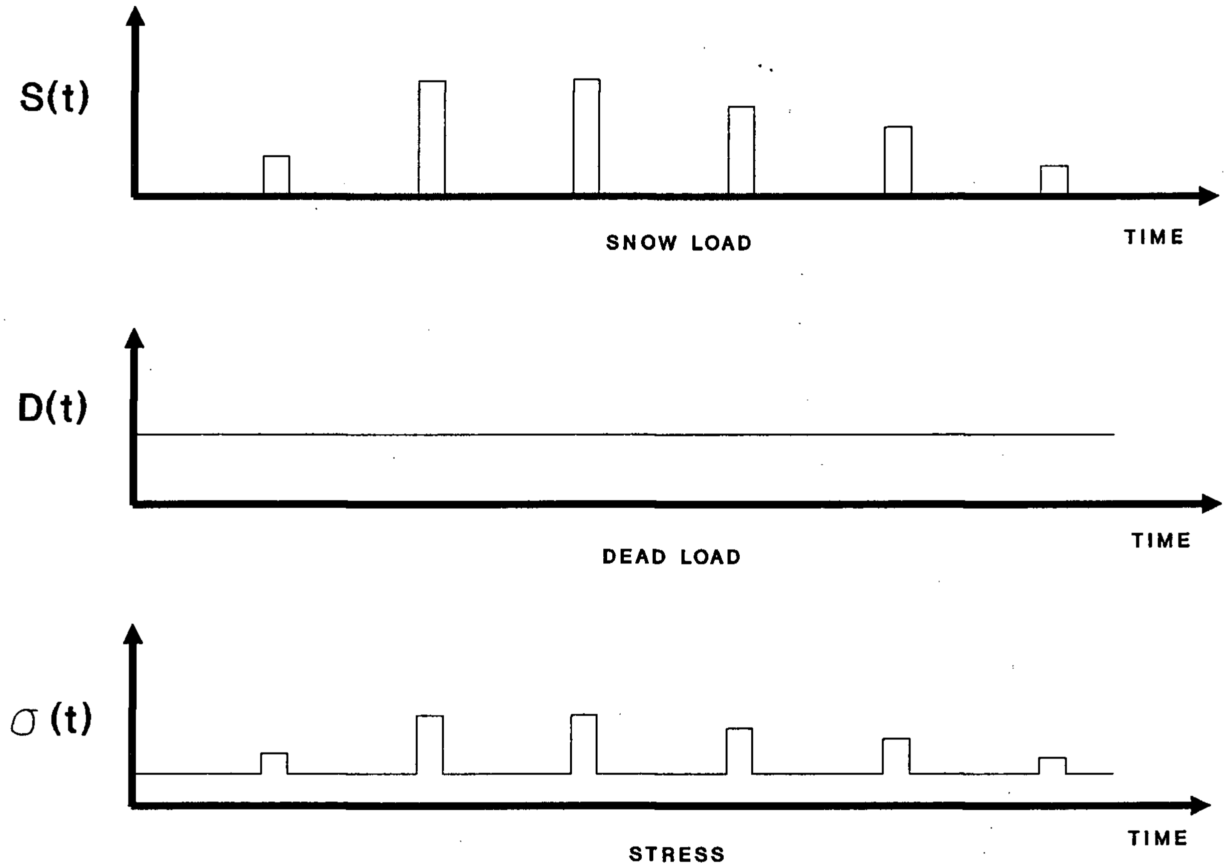


Figure 18. Load and Stress Model for DOL

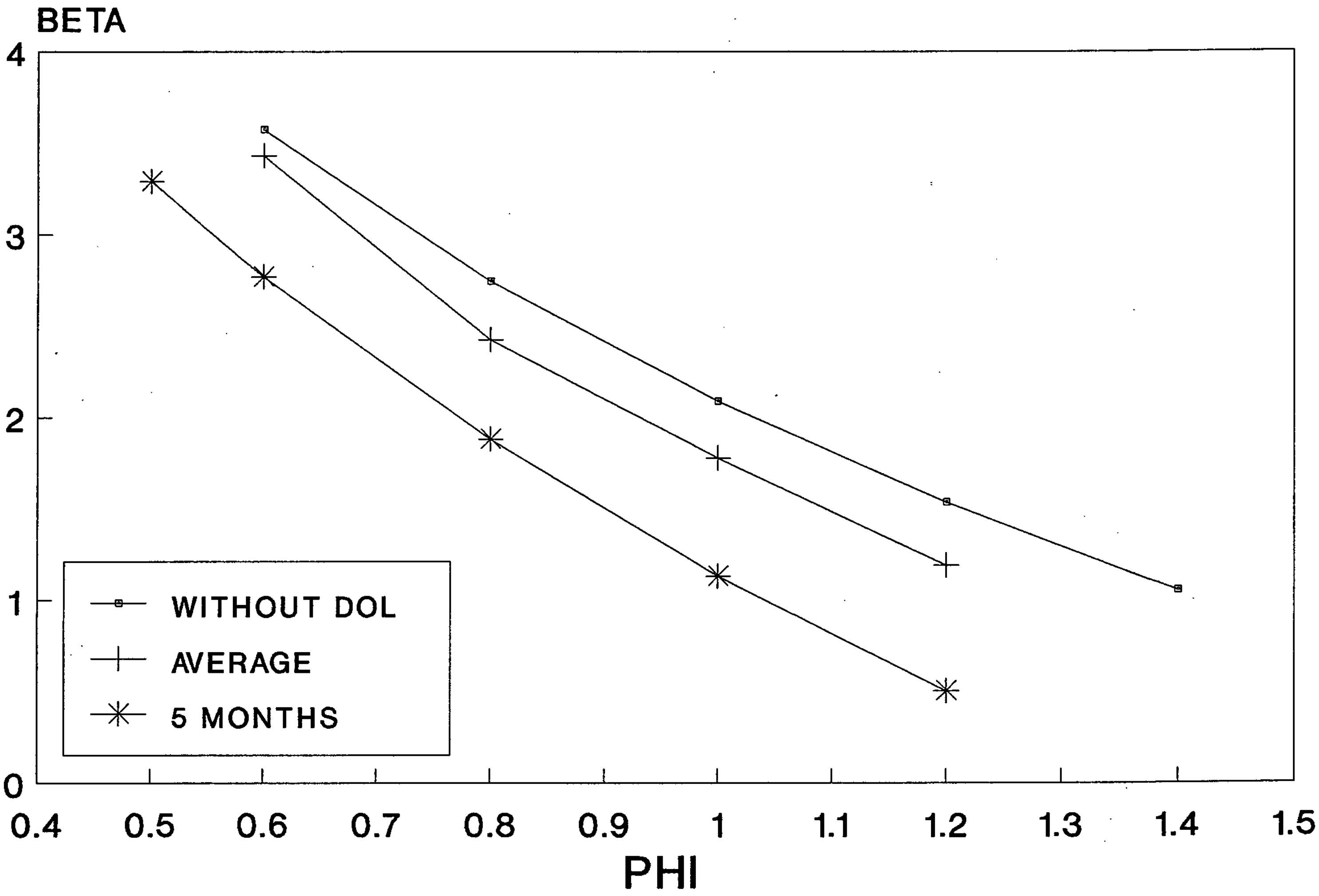


Figure 19. DOL in Osaka (SPF Q1, Gamma = 0.25)

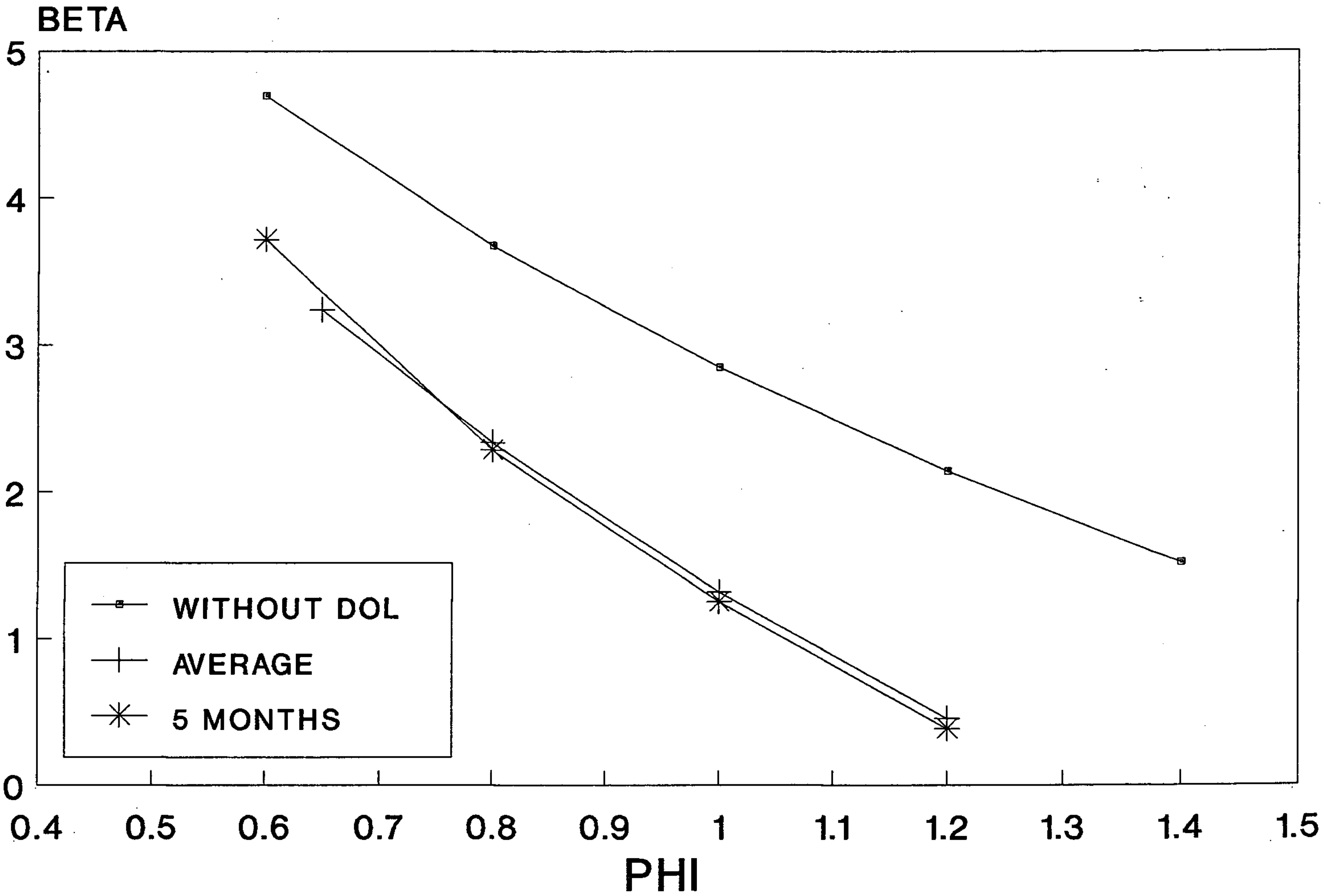


Figure 20. DOL in Sapporo (SPF Q1, Gamma = 0.25)

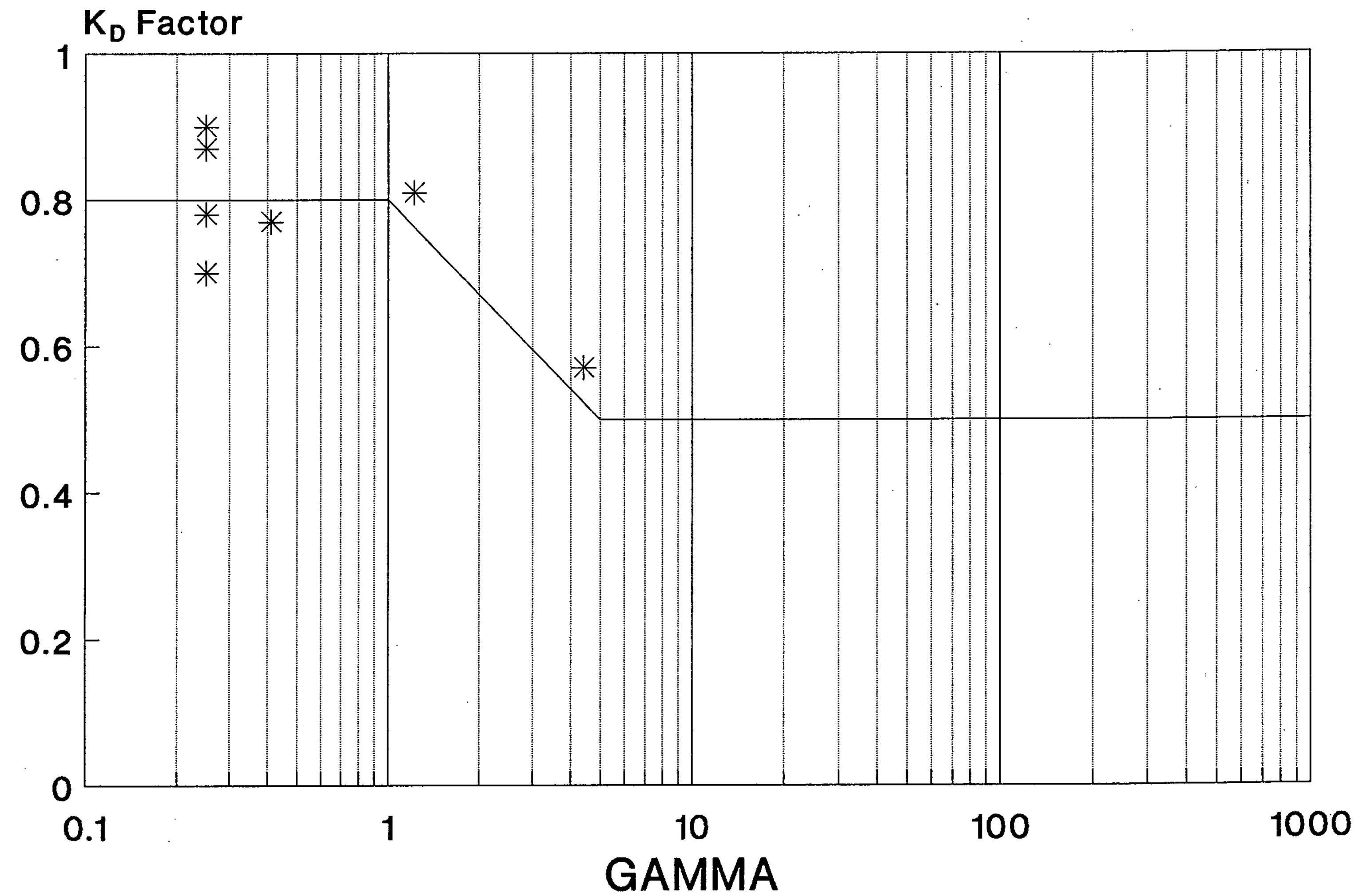


Figure 21. Recomended Relationship between GAMMA and K_D Factor

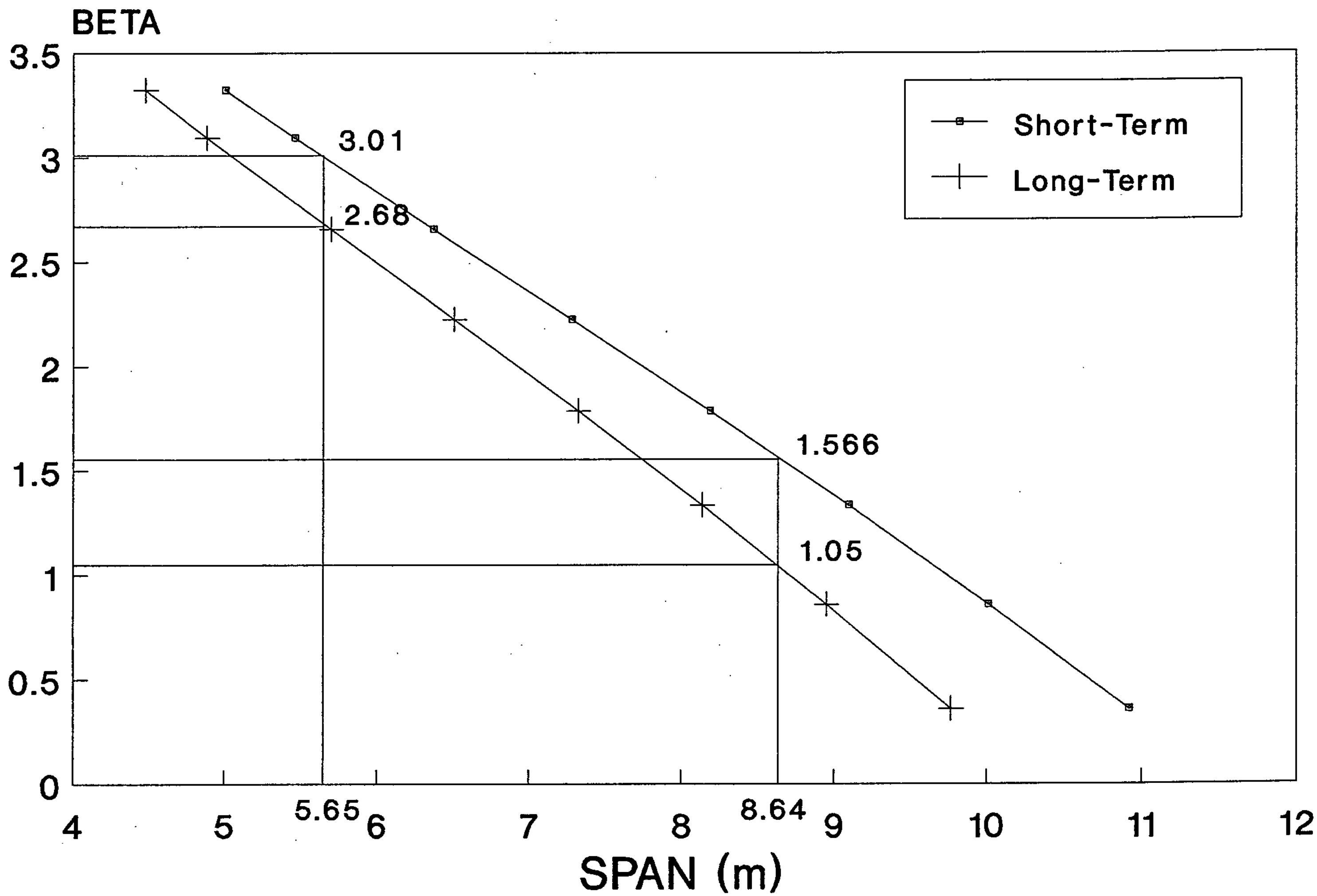


Figure 22. Bending Beta vs Span (Tokyo, 2x8, SS, Light Roof)

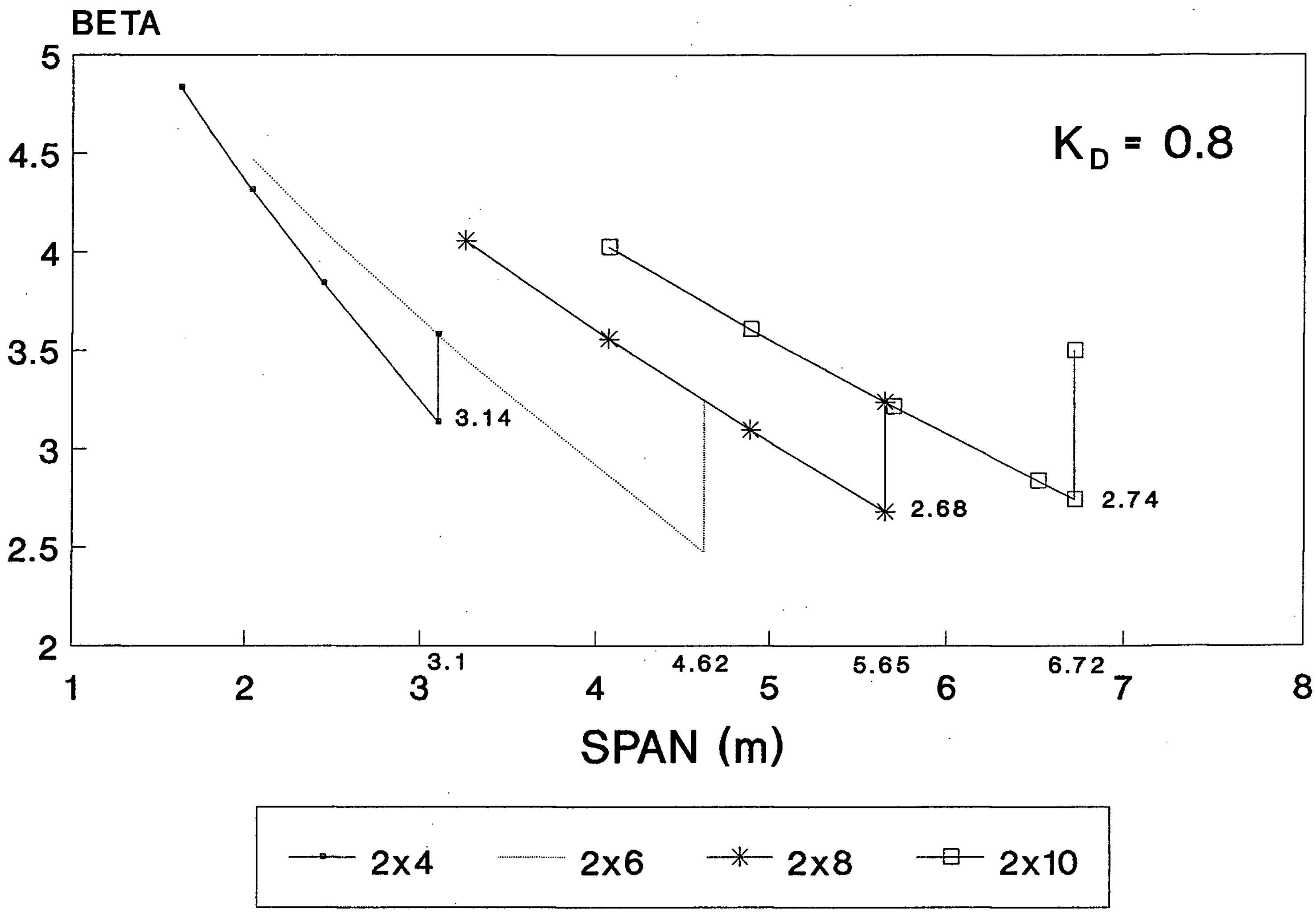


Figure 23. Long-Term Bending Beta vs Span (Tokyo 1, SS, 455mm, Light Roof)

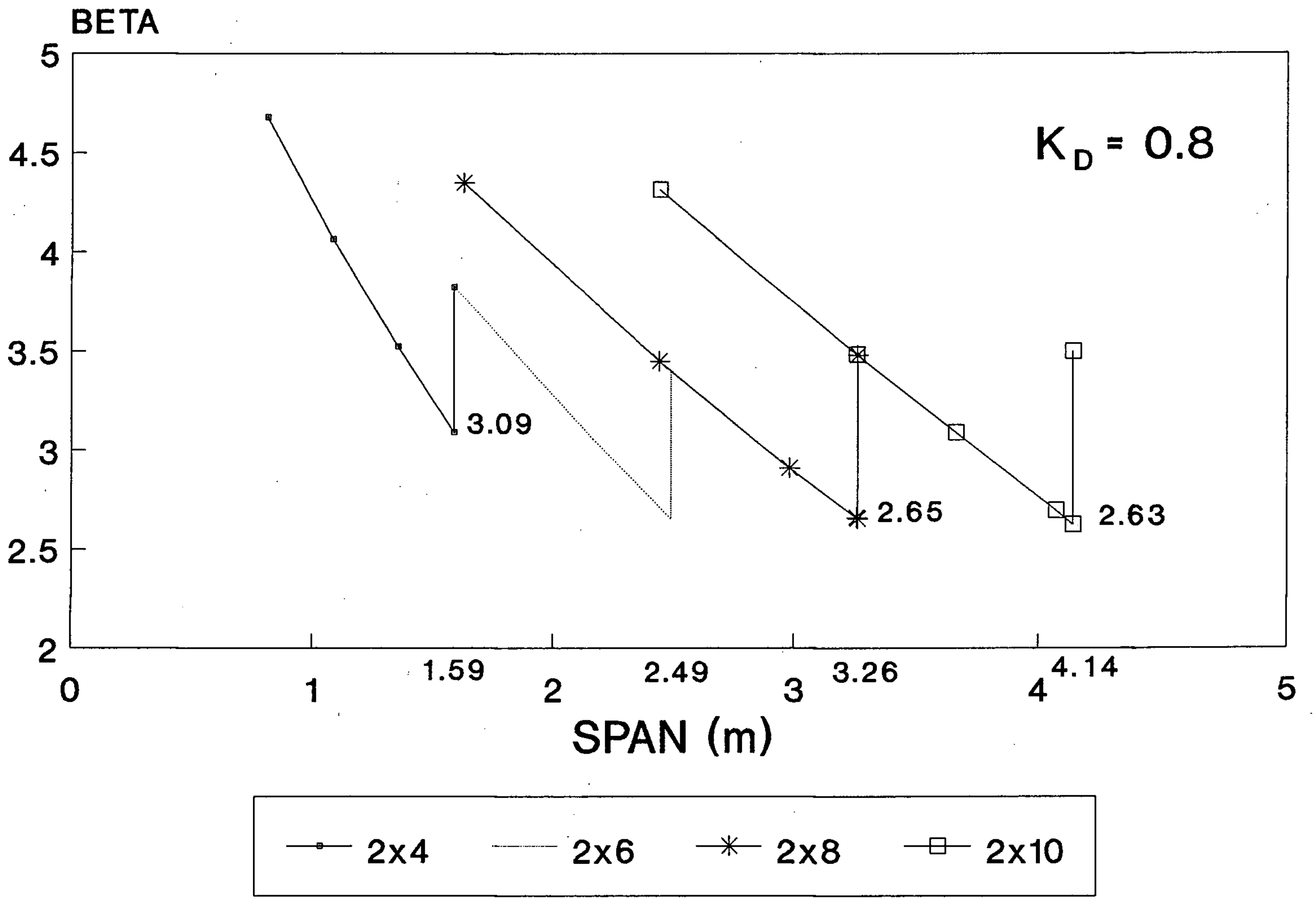


Figure 24. Long-Term Bending Beta vs Span (Sapporo, No.2, 303mm, Light Roof)

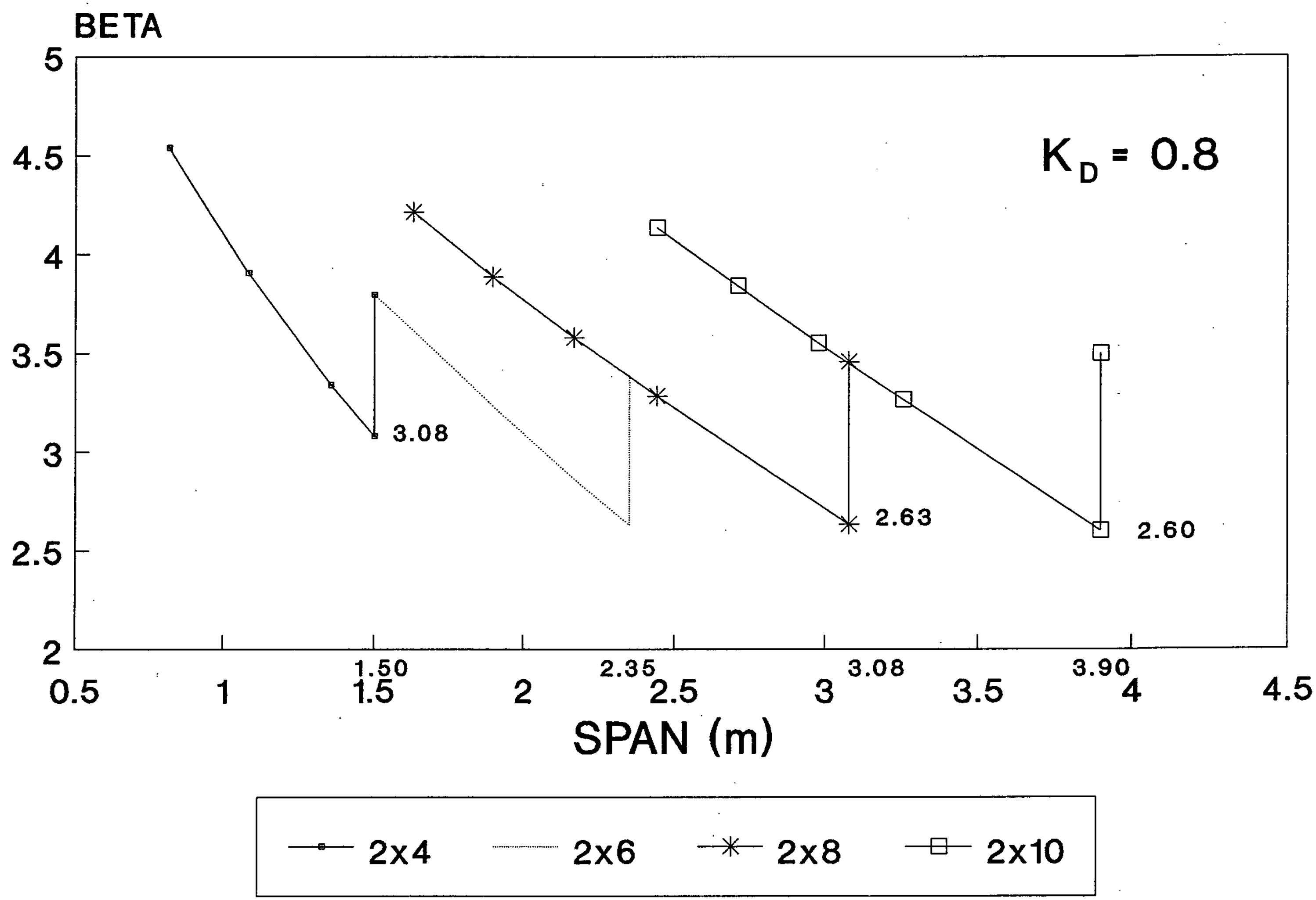


Figure 25. Long-Term Bending Beta vs Span (Niigata, No.2, 455mm, Heavy Roof)

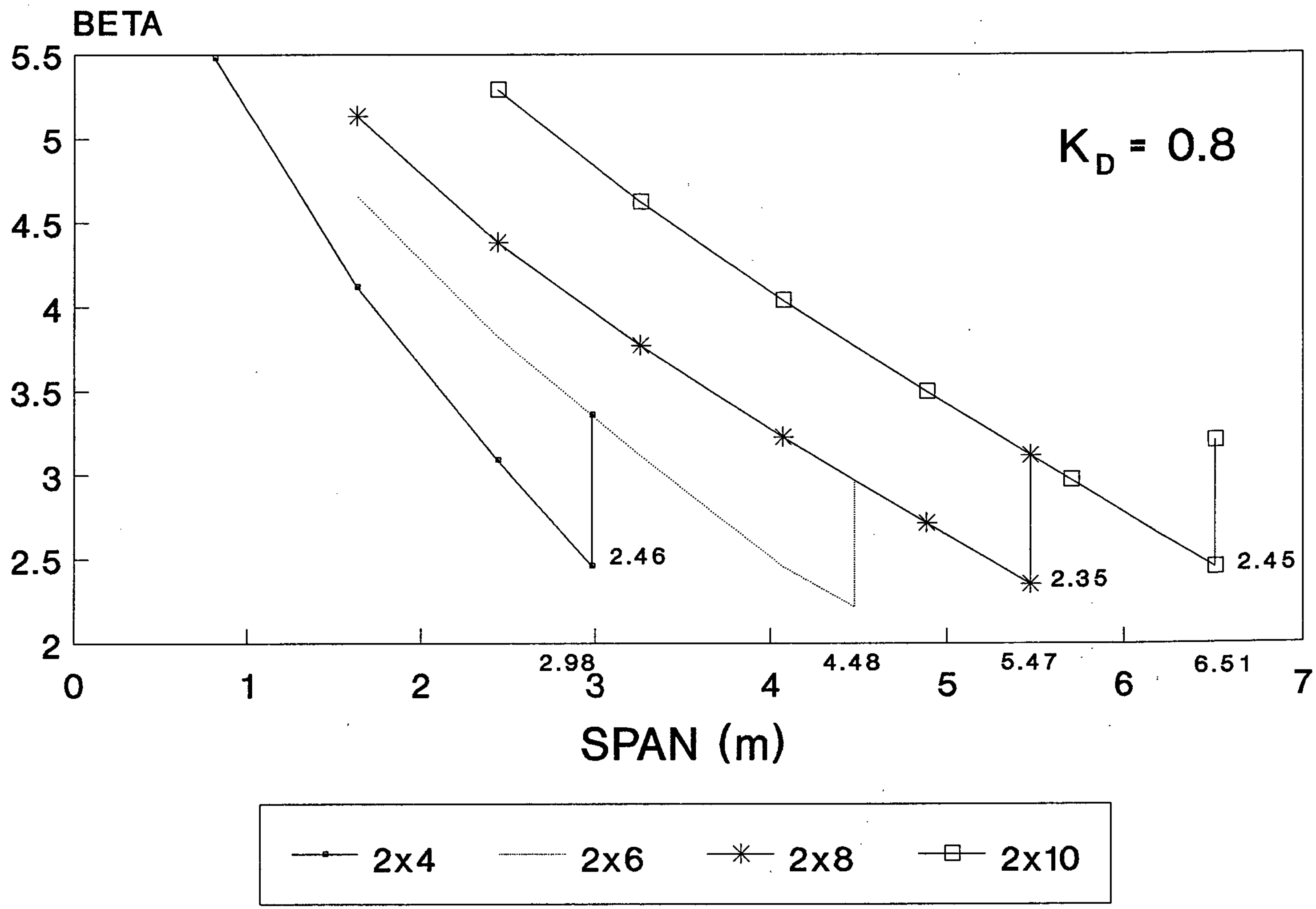


Figure 26. Long-Term Bending Beta vs Span (Tokyo 2, No.2, 455mm, Light Roof)

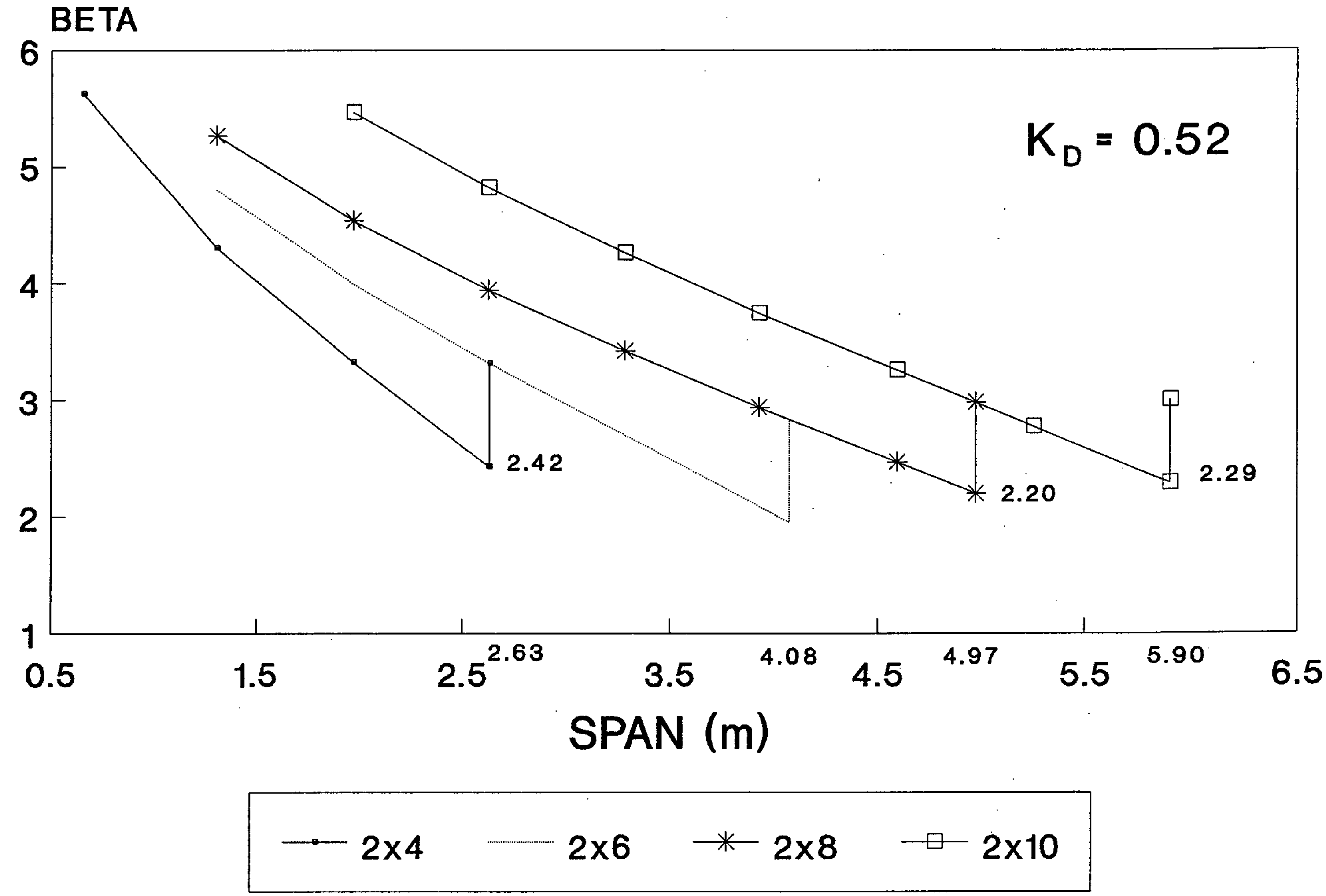
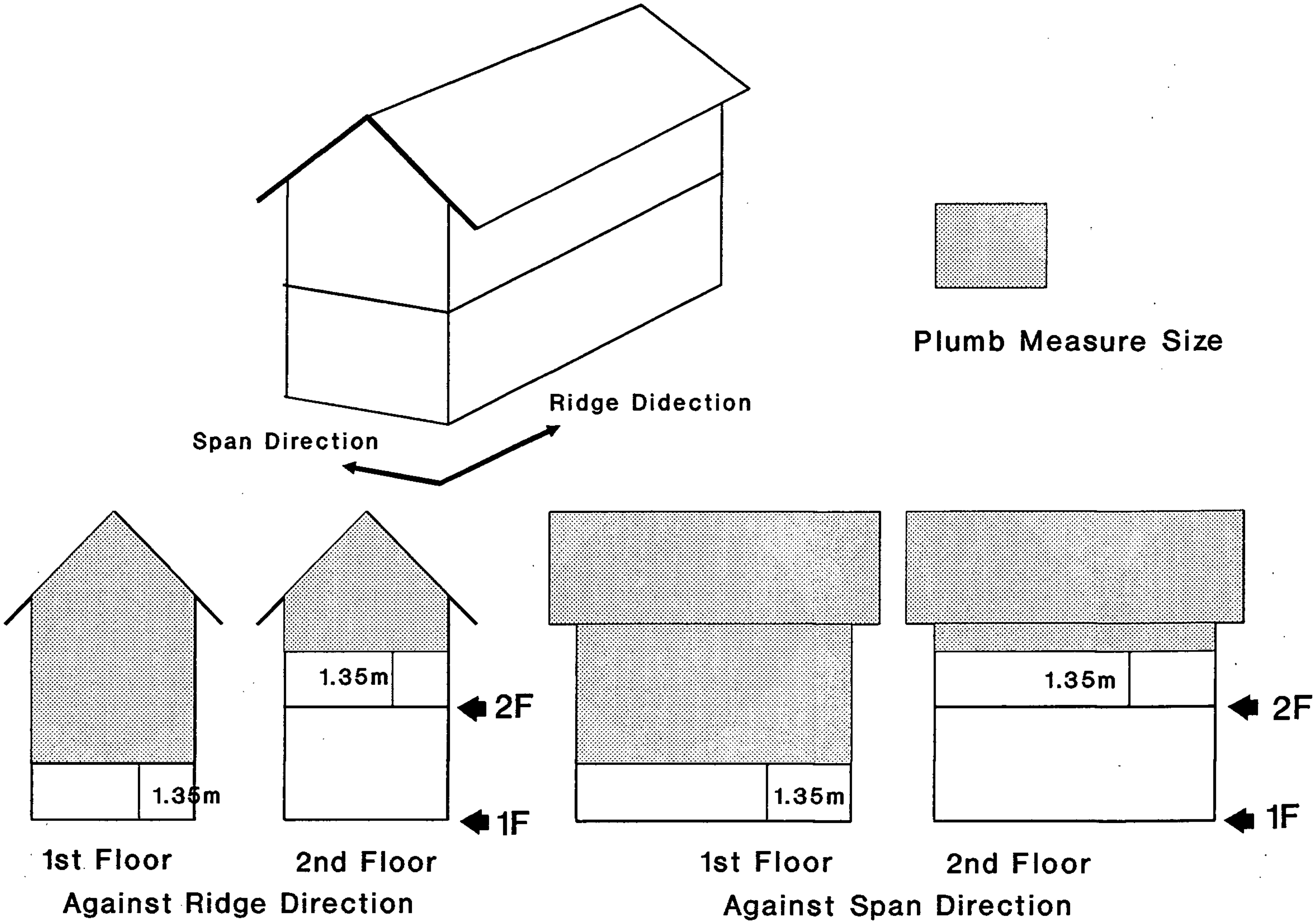


Figure 27. Long-Term Bending Beta vs Span (Osaka, No.2, 455mm, Heavy Roof)

Appendix 1. Allowable Unit Stress for Lumber for 2x4 Wood Frame Structure

Species Group	Grade	Allowable Unit Stress for Sustained Loads (unit: MPa)				Allowable Unit Stress for Temporary Loads (unit: MPa)				Modulus of Elasticity (unit: MPa)
		Compression	Tension	Bending	Shear	Compression	Tension	Bending	Shear	
DFir-L	S.S.	9.8	8.3	13.7	0.78	2 times the values of allowable unit stress for compression, tension, bending or shear for sustained loads, respectively				10800
	No.1	9.3	6.9	11.8	0.78					10800
	No.2	7.4	5.9	9.8	0.78					9800
	No.3	4.4	3.4	5.4	0.78					8800
	Construction	8.3	4.4	7.4	0.78					8800
	Standard	6.9	2.5	3.9	0.78					8800
	Utility	4.4	1.0	2.0	0.78					8800
Hem-Tam	S.S.	8.8	7.8	12.7	0.69					9800
	No.1	7.8	6.4	10.8	0.69					9800
	No.2	6.4	5.4	8.8	0.69					8800
	No.3	3.9	2.9	4.9	0.69					7800
	Construction	7.4	3.9	6.9	0.69					7800
	Standard	5.9	2.5	3.9	0.69					7800
	Utility	3.9	1.0	2.0	0.69					7800
Hem-Fir	S.S.	8.8	6.9	11.8	0.69					8800
	No.1	7.8	5.9	9.8	0.69					8800
	No.2	6.4	5.4	8.8	0.69					7800
	No.3	3.9	2.9	4.9	0.69					6900
	Construction	7.4	3.9	6.4	0.69					6900
	Standard	5.9	2.0	3.4	0.69					6900
	Utility	3.9	1.0	1.5	0.69					6900
S-P-F	S.S.	7.4	6.4	10.8	0.59					8300
	No.1	6.4	5.4	9.3	0.59					8300
	No.2	4.9	4.4	7.4	0.59					7400
	No.3	2.9	2.5	4.4	0.59					6900
	Construction	5.9	3.4	5.9	0.59					6900
	Standard	4.9	2.0	3.4	0.59					6900
	Utility	2.9	1.0	1.5	0.59					6900
W Cedar	S.S.	7.4	5.9	9.8	0.59					7800
	No.1	6.4	5.4	8.8	0.59					7800
	No.2	5.4	4.4	7.4	0.59					6900
	No.3	2.9	2.5	4.4	0.59					6400
	Construction	5.9	3.4	5.4	0.59					6400
	Standard	4.9	2.0	2.9	0.59					6400
	Utility	2.9	1.0	1.5	0.59					6400



Appendix 2. Measurement of Plumb Measure Size

Appendix 3. Characteristics of the Bending Strength (100% Data)

unit : MPa

	Size	2x4		2x8		2x10	
	Grade	SS	No2	SS	No2	SS	No2
Normal	Mean	55.43	45.78	41.71	36.32	37.15	30.68
	S.D.	13.50	14.37	11.27	11.38	9.61	9.52
Lognormal	Mean	55.41	46.05	41.53	38.16	37.01	30.88
	S.D.	14.70	17.22	12.38	14.37	10.52	11.27
2P Weibull	Scale	60.51	51.47	45.90	41.00	40.70	34.47
	Shape	4.765	3.335	4.215	3.272	4.481	3.382
3P Weibull	Location	14.09	8.94	3.35	6.72	1.35	7.26
	Scale	46.18	42.19	42.50	34.06	39.33	26.88
	Shape	3.303	2.491	3.792	2.460	4.278	2.368
Non-Parametric							
5th Percentile		32.77	21.17	23.05	16.89	20.71	14.86

Appendix 4. Characteristics of the Bending Strength (Lower 15% Datafit)

unit : MPa

	Size	2x4		2x8		2x10	
	Grade	SS	No2	SS	No2	SS	No2
Normal	Mean	54.05	36.99	39.68	29.61	36.39	23.85
	S.D.	12.44	9.01	10.15	7.43	9.26	5.22
Lognormal	Mean	68.58	50.82	56.00	42.69	50.46	29.75
	S.D.	28.24	24.29	28.79	22.16	25.35	11.45
2P Weibull	Scale	57.12	40.42	43.91	32.96	39.71	25.07
	Shape	5.578	4.912	4.548	4.598	4.674	5.979
3P Weibull	Location	3.66	12.62	3.68	4.86	0.00	10.04
	Scale	54.57	45.92	42.26	31.84	39.71	26.88
	Shape	4.912	1.792	3.730	3.102	4.674	1.713
Non-Parametric							
5th Percentile		32.77	21.17	23.05	16.89	20.71	14.86

Appendix 5. 2 Parameter Weibull Distribution Parameters for MOE for S-P-F

Size	Grade	Mean ($x10^4$ MPa)	COV	Scale m ($x10^4$ MPa)	Shape k
2x4	SS	1.029	0.167	1.100	7.056
	No.2	0.910	0.210	0.986	5.499
2x8	SS	0.984	0.161	1.049	7.326
	No.2	0.923	0.192	0.994	6.070
2x10	SS	0.954	0.160	1.017	7.403
	N0.2	0.872	0.198	0.941	5.868

Appendix 6. Parameters for Tension Strength (Lower 15% Datafit)

Size	Grade	Mean (MPa)	2-P Weibull Parameters COV	Scale m (MPa)	Shape k	Non-parametric 5-th Percentile (MPa)
2x4	SS	25.83	0.22	28.04	5.30	16.53
	No.2	17.73	0.25	19.43	4.49	9.80
2x8	SS	18.21	0.18	19.52	6.62	12.27
	No.2	13.19	0.19	14.21	6.09	8.32
2x10	SS	18.45	0.18	19.83	6.33	12.07
	No.2	14.11	0.22	15.33	5.23	8.50

Appendix 7. Parameters for Compression Strength (Lower 25% Datafit)

			2-P Weibull Parameters			Non-parametric
Size	Grade	Mean	COV	Scale m	Shape k	5-th Percentile
		(MPa)		(MPa)		(MPa)
2x4	SS	31.19	0.14	32.97	8.81	23.28
	No.2	27.01	0.19	29.12	5.97	18.48
2x8	SS	26.54	0.15	28.20	7.88	19.32
	No.2	24.15	0.14	25.57	8.46	18.20
2x10	SS	23.96	0.13	25.24	9.50	18.69
	No.2	21.69	0.18	23.31	6.36	14.40