Fundamentals of the design of bamboo structures

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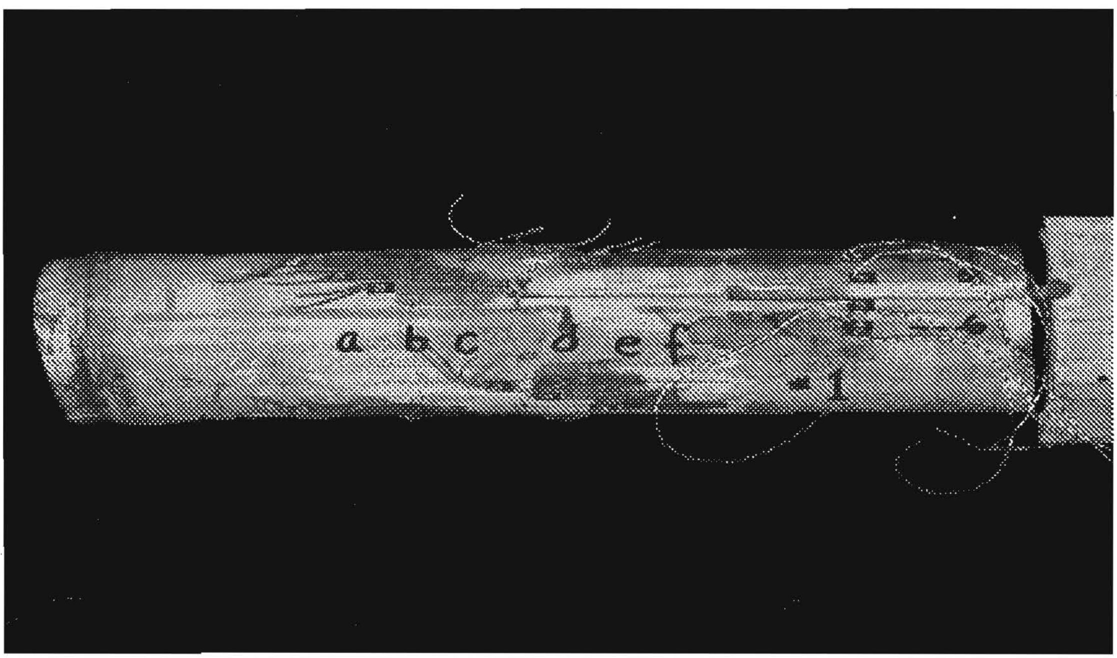
FUNDAMENTALS OF THE DESIGN OF BAMBOO STRUCTURES

IR. O.A. ARCE-VILLALOBOS



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FUNDAMENTALS OF THE DESIGN OF BAMBOO STRUCTURES

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to obtain the degree of doctor at the Eindhoven University of Technology by the authority of the Rector Magnificus, prof.dr.J.H. van Lint, to be defended in public in the presence of a committee nominated by the Council of Deans on Tuesday 21st September 1993 at 16.00 hrs.

by

Oscar Antonio Arce-Villalobos

born in San José, Costa Rica.

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Cover photo: Bamboo culm connected by gluing to a wood fitting, after a bending test.

FUNDAMENTALS OF THE DESIGN OF BAMBOO STRUCTURES

PROEFSCHRIFT

ter verkrijging van de graad van doctor aan de Technische Universiteit Eindhoven, op gezag van de Rector Magnificus, prof.dr.J.H. van Lint, voor een commissie aangewezen door het College van Dekanen in het openbaar te verdedigen op dinsdag 21 september 1993 om 16.00 uur

door

Oscar Antonio Arce-Villalobos

geboren te San José, Costa Rica.

Dit proefschrift is goedgekeurd door de promotoren:

ir. W.R. de Sitter
prof dr. J. Gutiérrez

Copromotor:

Dr.ir.J.J.A.Janssen

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Copromoter:

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To Mylenne, Carlos-Andrés and Mauricio,
who deserve all credit for all these years of sacrifice,
continuous support and confidence.
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The author was honoured by the acceptance of membership by the distinguished members of the Thesis Comission. Gratitude is expressed to them, for their great contribution to the quality of this thesis, as well as their generous comments and teachings.

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Table of Contents

Summary

This thesis presents the results of the theoretical and experimental analysis of bamboo culms, subjected to tensile and compressive loads, or acting as beam columns in frameworks. Glueability of bamboo wood phases is included as well. The connection of bamboo culms is also analyzed and a proposal based on the use of glue is made.

Evaluation and synthesis is made by presenting some general considerations for the structural design and analysis of bamboo frameworks.

Keywords: Bamboo / mechanical properties / structural design / compression capacity / tensile capacity / bamboo connections / gluing / structural analysis / bamboo research / bamboo construction / bamboo testing / buckling /

Cover photo: Test specimen of a bamboo wood glued connection

Samenvatting

Dit proefschrift geeft de resultaten weer van de theoretische en experimentele analyse van bamboe stammen, onderworpen aan trek en drukbelasting, of gebruikt in raamwerken.

De mogelijkheid van het lijmen van bamboe aan hout is onderzocht. De verbinding van bamboe stammen is geanalyseerd en een voorstel gebaseerd op het gebruik van lijm is gemaakt.

Evaluatie en synthese is verkregen door het weergeven van enkele algemene overwegingen met betrekking tot het constructief ontwerpen en analyseren van bamboe raamwerken.

Trefwoorden: Bamboe/ mechanische eigenschappen / konstruktief ontwerp / drukcapaciteit / trekcapaciteit / bamboe verbindingen / lijmen / constructive analyse / bamboe onderzoek / bamboe bouw / bamboe proeven / knik

Foto voorpagina: Proefstuk van een lijmverbinding tussen bamboe en hout

Chapter 1: Introduction

1.1.-Introductory remarks

This thesis reports the results of a four year research project on bamboo as a structural material carried out at the Eindhoven University of Technology in The Netherlands. This is of course not the first time such an enterprise has been undertaken. On the contrary, as shall be seen from some of the bibliographic references accompanying the thesis, the subject of structural properties of bamboo has been a research matter since the beginning of this century. Bamboo has attracted people's attention for centuries, and many publications have been devoted to the description of the hundreds of applications that mankind has given to this *Gramminae*[1].

Applications in the field of civil engineering are neither rare nor new. Bridges, panels for roofing and walling, full or split culms as earth supporting structures, full culms as beams or columns, as well as elements for the construction of frames and trusses are among the many applications described in the literature.

Bamboo grows very easily and yields high productivity levels. Moreover, its cultivation does not harm the environment, because among other reasons culms can be taken from the plant from time to time under a rotational scheme. In this way, the plant itself does not die. Instead, careful harvesting revitalises its growing. It can be cultivated in relatively poor soils, helping to control erosion by water and wind. It is therefore a renewable resource.

Nevertheless, people have systematically decreased the use of bamboo for construction purposes. When the author of this thesis asked a Taiwanese expert[2], ten years ago, why less and less bamboo was used in construction in south-east Asia, he answered that it had already been considered to be a poor man's material for some time. Indeed, one sees how the peasants of Thailand have practically abandoned the practice of building with bamboo though the material is available and handy as well, in spite of the fact that this material used to be a primary construction material all over south-east Asia and Japan. Now, one can observe that

[1]The grass family.

[2]Dr. Wei-chin Lin, head of the Taiwanese mission for the capacitation of bamboo craftmen and craftwomen in Limón, Costa Rica.

as soon as rural people have access to wood or other materials, they change to them. Some social factors are often put forward as the cause for this shift in attitude, among which is the fact that other materials give a sort of status to their users, while bamboo emphasizes their position as the poorest members of society.

Social factors are important and they have to be accounted for in the promotion of a product or material, but it is equally important to look at the causes of human reactions in the materials or technologies themselves. What can be observed in the rural areas of south-east Asia is that people still only use bamboo as a last resource, on a "better than nothing" basis. Is there any objective reason for this change of attitude ? When people are asked about the properties of the material for construction purposes, the first problem that is identified by them is the durability of the culms. Indeed, some field observations and the testimony of people lead to the conclusion that bamboo rots under some conditions in less than one year. When complexity of the construction and availability allow, this is not a serious problem, because replacement is cheap and straightforward. In terms of large-scale housing projects, however, this creates a long term problem of maintenance, unless effective preservation is put in place. So far no preservation process is readily available to the rural people, at a reasonable cost and with a sufficient level of efficiency and security. **A very important underlying assumption in this project is that this problem is being attended to by other researchers in different parts of the world, and that a solution for this problem will be soon available**[3].

One other major reason for the social neglect of bamboo is the lack of acceptance by professionals in the construction field. Engineers and architects prefer to work with the determinacy of a well-known system or material, supported by solid knowledge of its properties, backed by the existence of a minimum of code specifications on which they can base their judgement and design decisions. Besides these elements, most engineers and architects of the third world have a bias in favour of materials that are traditional ones in the North, the ones they are taught about. Actually, professionals in the Third world very frequently suffer from the same problem of social status observed among the rural people of

[3]Research has been undertaken in several places. Construction oriented research is under way in the Costa Rican Bamboo National Project, see for example, Gonzâlez (1992).

Thailand. In other words, they find it a bit degrading to work with such a 'rough' material like bamboo.

There is another group too, as observed in Latin America, for whom tradition is proof enough, that tries to promote the use of bamboo based more on 'beliefs' than on technical and rigorous knowledge, some times producing a vicious circle of repeated mistakes.

A similar problem of 'acceptance' is encountered among researchers in so-called developing countries, who endlessly repeat the subjects colleagues from the North tackle in their research projects, refusing to put their talent and intelligence to the solution of their own nation's problems. In the first place this occurs because in doing so, they get far more recognition among the public and the scientific community.

Another social issue around bamboo is the sort of target population associated with it.

In that respect, Prof. Krijgsman[4], a member of the Committee of this thesis, once said to the author that if one wants to promote the use of a certain system or material among the needy, one should first demonstrate that it is also acceptable to other [rich] members of a society. The author very much agrees with that. Poor people are poor because of economic problems, not because they lack the incentive to be better. So long as a material is regarded as a 'poor man's material' it is condemned to be refused, or to be used only on a very temporary basis. It is the author's belief that to some extent there is a connection between social and technical problems relating to the use of bamboo in the field of civil engineering, building and architecture. Or even more generally, in relation to overall construction purposes. The link is the lack of a sufficient understanding of the material among professionals and the general public. As a matter of fact, it is indeed a real difficulty to find consistent, ready-to-use and complete information in this respect.

One objective fact is a clear reality, resources are becoming more and more scarce, needs are growing faster than ever, and bamboo is available, breaking the soil and reaching towards the sky, as vital as it has always been, producing a substantial amount of biomass that calls for our attention. But perhaps more important, bamboo calls for our creativity and scientific

[4]Prof.K.J. Krijgsman, Faculty of Building and Architecture, Delft University of Technology, The Netherlands.

curiosity and ingenuity.

There is information, perhaps not as abundant and well codified as for other materials, that tells us of the wonderful properties of this material, and in one aspect it is consistent enough to spark imagination and admiration. The tensile capacity of bamboo is claimed to be as high as that of reinforcing steel. Besides that, certain species grow up to 30 m high and gain an external diameter of 200 mm in one year, with no need for special cultivation care or the like. The promise of good properties and availability is a powerful reason to undertake continued research into this material.

In this research project, the emphasis is not just on the exploration of the basic mechanical properties, but an attempt is made to correlate these to the design feasibilities of the material. **The final goal of structural engineering is the design of effective, safe and socially accepted structures. The final goal of the research reported here is to contribute to the design of bamboo structures in that same spirit.**

Research results are presented in this thesis in a manner hopefully consistent with the above stated aim. To explain the way in which the research is presented, the reader is referred to figure 1.1. The process of **design** itself, at the bottom of the figure, is founded on a number of previous steps collectively named the **justification process** (Addis, 1990). This can be divided into three different levels in which information about structural systems is gathered and processed first, as indicated. It is interesting to note that at these three levels the 'design codes' (which most of the time could better have been called 'calculation codes') are started and developed, and specifications are given.

The intensity of the design effort during each of the steps, strongly depends on the level of development and availability of information and the extent to which it can be regarded as being consolidated. The researcher, on the other hand, is compelled to start at the level where information needs are assessed as **not** being fulfilled. The more developed a structural system or material is, the more attention can immediately be paid to the final phase of the design process.

As stated at the International Bamboo Workshop in Cochin, Kerala, India, 1988 (Anonymous, 1988), in reference to the lack of information in certain areas, attention should be concentrated

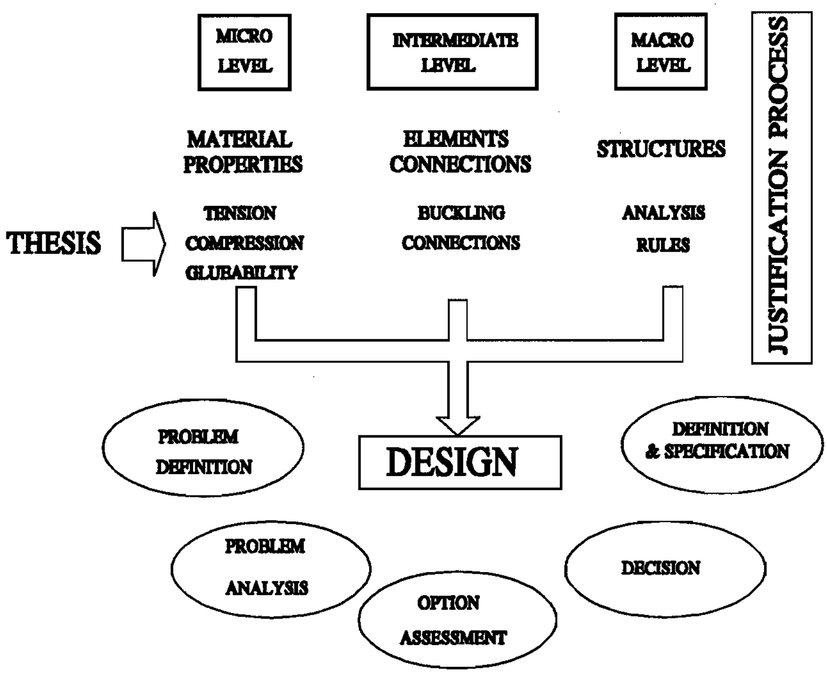

Figure 1.1: Steps/phases in the structural design process as employed in this research.

on " joints with bamboo to facilitate construction; strength properties as affected by specific applications; development of a design code for bamboo; establishment of an engineering data base to facilitate the use of bamboo in the construction and building industry..". All of the points addressed in this statement of purposes fall within the justification process of design, in clear recognition of the lack of information at this stage.

In view of this, the major effort, both in terms of time and attention during the development of the research reported in this thesis, has been put into the investigation of important points related to the structural nature of bamboo. Design proposals are based on the analysis of the results of this research.

1.2-Thesis organization.

The thesis follows two different semantic approaches. In the body of the thesis itself, the major emphasis has been placed upon the description of phenomena and results in qualitative terms, dealing mainly with those concepts that have direct consequences for the design process. Quantitative analysis, both theoretical or experimental, is dealt with in the appendixes. This set up is only diverged from when necessary for clarity's sake.

Chapters 2, 3 and 4 describe activities at the micro level of material properties, dealing with tensile and compressive strains and stresses, and the glueability of bamboo, as seen in figure1.2. Related appendixes are A, B, and C[5].

At the micro level problems are investigated to help find an explanation for the way bamboo is able to sustain strains and stresses. Tension both perpendicular and parallel to the fibres are included. The roles of the matrix and of the fibres are discussed and analyzed. The analysis of bamboo culms under compressive loading is presented in chapter three and appendix B.

Chapter 4 describes the work carried out on the glueability of bamboo wood phases, and the possibilities of using this approach as a solution to the problem of bamboo connections. Experimental results on gluing are summarized in appendix C.

A synthesis about design is done at two different instances. Firstly, some considerations are made at the material element level. Chapter 5 presents a design methodology / proposal for the connection of bamboo structural components based on gluing. A full description of the design process is included. The analysis of a glued connection is presented in appendix D.

Secondly, the design process is further considered at the level of the structure as a whole. In order to understand the important features of bamboo structures, the results of the investigation of a few features are presented as follows.

Buckling and the calculation of critical loads are included in appendix E, the analysis of bamboo trusses and frames is described in appendix F. The content of these appendixes

[5]Shear has not been included as a research subject in this thesis. The reader is referred to Janssen, (1981) for a good discussion on this subject.

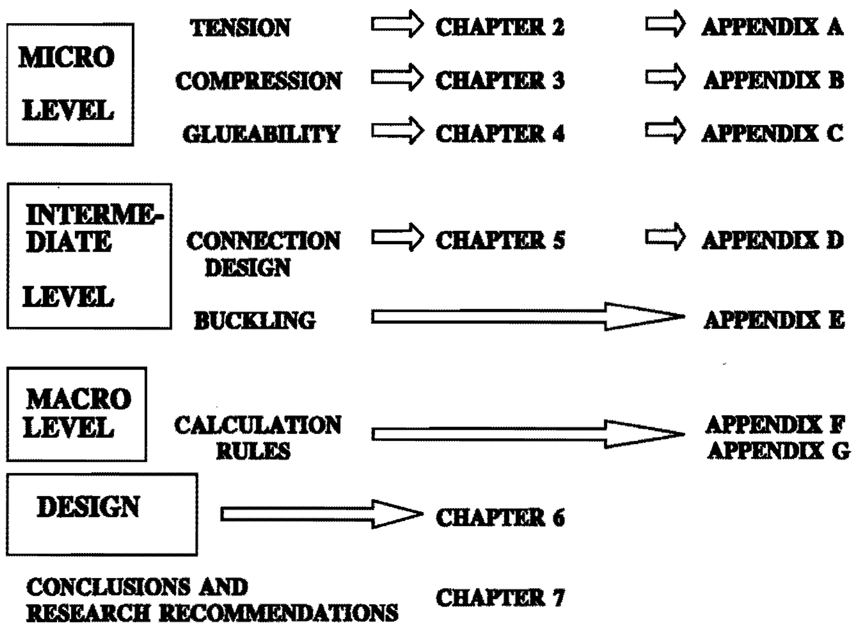

Figure 1.2: Thesis content description

supports the design considerations in chapter 6. Appendix G contains a description of statistical analysis of relevant geometric parameters for bamboo culms. Finally, chapter 7 summarizes the main conclusions, and discusses ideas for further research.

1.3-General remark

Evidently bamboo is a very complex material, and the culms have a complex geometry. In this work an effort has been made to account for these complexities in an inclusive manner, but at the same time to keep modelling and descriptions as simple as possible. Parameters are included in the resulting proposed formulae so that their effect can be studied, and the proposed models can be adapted to specific conditions after observation of experiments. The latter were insufficient in our case due to the lack of sufficient experimental material (only two species could be explored).

In this sense the proposed models are meant to indicate possible lines of thinking, rather than to be final and conclusive explanations about complex relations between parameters in a complex material. Models are proposed with the clear intention of motivating extensive discussions among bamboo scientists and structural designers. It is the author's hope that such a broad discussion would allow clarification of the way these results can be used in every specific circumstance, and that it would encourage the gathering of fundamental data needed to validate some of the results described here. In turn it is hoped that in this way, some code specifications will be agreed upon for bamboo, based on a better understanding of the structural properties of the material.

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2 Anonymous; 1988. Proceedings of the International Bamboo Workshop. Kerala, India, in the conclusions.

3 González, G.; 1992.Técnicas de preservación de culmos y esterillas de bambú. Seminario Centroamericano usos del Bambú. Pocora, Costa Rica.

4 Janssen, J.J.A.; 1981. Bamboo in building structures. Ph.D. Thesis, University of Technology of Eindhoven, The Netherlands.

Chapter 2: Tensile strength of bamboo

2.1.- Introduction

2.1.1-Bibliographic background

One of the more abundant pieces of information about the mechanical properties of bamboo is related to its tensile strength, although often very little account is given on the procedure followed to measure this quality, nor enough data included in reports or textbooks on the matter.

In the following section literature is referred to, to show the way the approach to the study of the tensile capacity of bamboo has evolved over the years. Results are quoted with the same purpose. Much of the carried out research deals with biological aspects, which are covered in many good papers. A complete and thorough review of this research is beyond the scope of this thesis, though reference is made to some of the most comprehensive and authoritative studies to support the here reported research , but readers with a specific interest in this field are strongly encouraged to review the existing literature.

Meyer (1923) reports a first set of results, coming from the testing of several concrete beams reinforced with bamboo. It is not clear in the paper what the appearance of the reinforcement was like, or whether full bamboo stems or strips were employed. The tensile strength of bamboo was calculated by means of formulae similar to those used for steel reinforced concrete, and a maximum value of **100 N/mm^2** is reported for the tensile strength of bamboo. The botanical species is not mentioned in the paper.

The same author reports that in a visit to the Upper Yangtze river, the working stress of the twisted and plaited bamboo ropes used for towing the junks, was calculated to be about **70 N/mm^2** but it could be doubled in some cases. These ropes were made of the outer layer of bamboo culms.

Duff (1941) carried out some tests on the different basic mechanical parameters of bamboo on *Phyllostachys pubescens*. It seems that he tested strips taken from the culms, and that were reinforced at the extremes to avoid damage by the grips. Reinforcing was enlarging the areas in the support region gluing some extra material. The exact shape of the specimens is not reported. He studied the variation of the tensile strength according to both the position along the culm and the cross-section in the radial direction. The author reports a maximum tensile strength in the outer layer of **342 N/mm^2** and a minimum in the inner layer of **54 N/mm^2**. He also found a consistent variation for the elastic modulus. In relation with the position along the culm, this author found that the strength does not vary much from the base to about half the full height, but that it increases toward the top where the strength was found to be 30% to 40% higher. **The tensile strength at the nodes was found to be only 80% of the strength at the internodes.**

Ota (1954) reports what seems to be a first attempt to relate mechanical properties to physical properties or biological factors. He studied the relation between tension at failure and the sample moisture content on *Phyllostachys edulis* and *Phyllostachys pubescens*. He studied bamboos from different localities, finding a linear experimental formula to relate tensile strength to moisture content. This same author summarized as well the values of average density and tensile strength for all the specimens tested, which may allow for comparisons with other results. Nevertheless, careful judgement is necessary since no detail is given about the test procedure.

Karamchandani (1959) examined the suitability of bamboo as a construction material from a more systematic approach, accounting for a series of engineering factors. In his paper he reports that the tensile strength for the outer layers varies from **100** to **335 N/mm^2** and from **150** to **160 N/mm^2** for the inner layers. This is in very poor agreement with the result reported by Duff (opus cit). The species tested was *Dendrocalamus strictus.*

A more comprehensive study on this matter was undertaken by Cox (1969) on *Arundinaria tecta*. From his literature review he reports that tensile strength of bamboo depends on age, physiological variation of individual culms, habitat, liquid content of soil in the habitat, and "external physical forces". In the same report it is established that the tensile strength of individual culms increases from the first node to the middle node and then decreases towards the top, and that specimens from the longest internodes usually had the largest tensile strength as well. In most cases the node was the weakest section. It is also reported that the Bamboo Research Committee for the Bureau of Public Highways from Manila stated that tensile strength increases with age of the specimen and from the basal to the distal section of individual culms. Besides that, it is reported that in studies both in the United States and in India it was found that peak values for tensile strength occurred for specimens three to four year old. The author performed a set of tests on full culms, with special grips to avoid failure by crushing in the supports. Rate of loading was controlled to a constant of 7 N/mm²/minute, and the results were **110 N/mm²** on average for the maximum stress and **18670 N/mm²** for the modulus of elasticity.

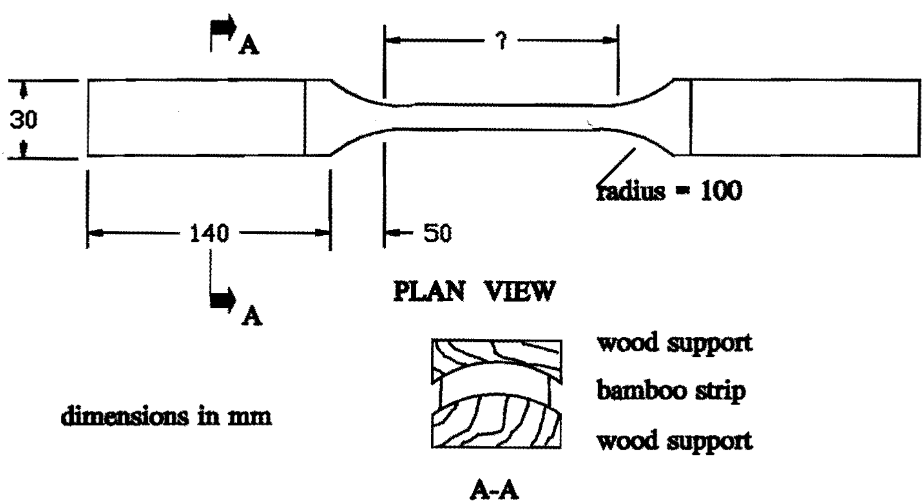
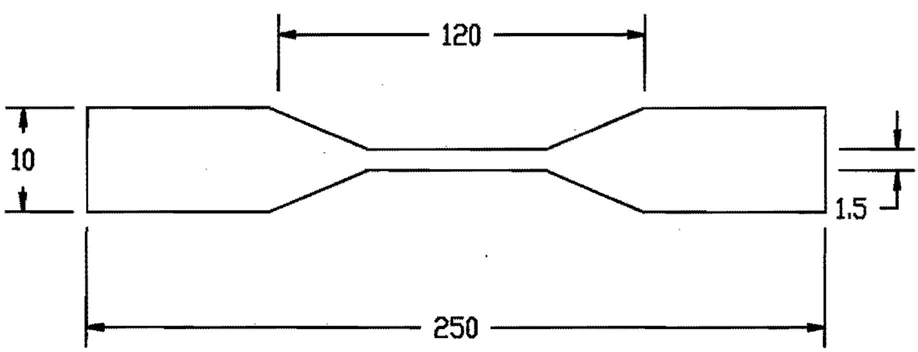


Figure 2.1: Specimen by Atrops.

A detailed description of a specimen for the tensile test is given by Atrops (1969), as shown in figure 2.1.

The loading rate was set at **20.27 N/mm²/min**, and the test was carried out for both the outer skin and the inner part of the culm, with results of **290 N/mm²** and **153 N/mm²** respectively. No details are given on the studied species .

More recently, McLaughlin (1978) accepted the at the time well-established principle according to which the mechanical properties of cellulose based materials correlate very well with density, and he proved that this was indeed a good approach for bamboo as well by studying the relation between density and tensile strength for very thin samples of bamboo tissues. This experimental work was performed on *Bambusa vulgaris.*



dimensions in mm

Figure 2.2: Specimen by Xiu.

Figure 2.2. shows the specimen used by Xiu-Xin (1985) to study the mechanical properties of *Phylostachys glauca* from four different locations in China. His report includes data on tensile strength, moisture and density, and regressions of tensile stress and age are calculated. Widjaja and Risyad (1985) studied the relation between anatomical and mechanical properties for some Indonesian bamboos.

Testing procedures were as recommended in ASTM D143-52, 72, though the type of modification and specific conditions of the tests are not described.

Soeprayitno et al (1988) carried out an analysis of certain physical and mechanical features for *Gigantochloa pseudoarundinacea* from two different localities. Results are summarized in the following table.

origin	av.strength [N/mm^2]	av.E modulus [N/mm^2]	av.density [kg/m^3] node	i.nod
slope hill	177.9	27631	770	609
valley bottom	149.4	19643	693	565

Table 2.1 Results from Soeprayitno (opus cit).

Prawirohatmodjo (1988) also studied some Indonesian bamboos, finding average tensile strengths of **297 N/mm^2** and **315 N/mm^2** for green and dry bamboo respectively. There is no complete description of test procedures in the paper. Species were *Bambusa arundinacea, Bambusa vulgaris, Dendrocalamus asper, Gigantochloa apus, Gigantochloa ater,* and *Gigantochloa verticilliata.*

Sharma (1990) reports on an experiment with the objective of investigating the possibilities of bamboo as reinforcement for concrete T-beams. He studied *Bambusa vulgaris* and found parallel tension strengths of **145 N/mm^2** for specimens with nodes, and **200 N/mm^2** for internode samples. No details are given on the test procedure or other mechanical aspects.

No reference was found on the **tangential strain or stress**[1] capacity of bamboo canes, though many researchers and users mention the fact that bamboo is cleavage prone, being one of its main advantages when used for handicraft purposes.

This current chapter is dedicated to the description of the tensile characteristics of bamboo. Experimental observations are described as an appendix (A), and some resulting data are

[1]in the direction perpendicular to the fibres.

introduced, to establish some reference points of further use in the following chapters.

2.1.2-Anatomical and physico-chemical background :

The properties of the culm are mainly determined by its anatomical structure. Basically the composition at any transverse section is determined by the shape, arrangement, size and number of vascular bundles.

Vascular bundles consist of the xylem[2] with one or two smaller protoxylem elements[3] and two large metaxylem vessels[4] and the phloem[5] . It consists of sieve elements and companion cells and/or parenchyma cells, often with fibres or sclereids (short sclerenchyma cell) with thin walled, unlignified sieve tubes connected to companion cells. Four major types of vascular bundles can be differentiated according to Liese (Liese, 1985).

The density of vascular bundles increases with height (Grosser; Liese; 1971).

Fibres, that constitute the sclerenchymatous tissue, occur as caps of vascular bundles, but in some species also as isolated strands. They contribute 40-50% to the total culm tissue (Liese, opus cit) and **60-70% to the weight**. They have a polylamellate wall structure, especially the outer skin, leading to very high tensile strength. This does not exist in other cell walls of fibres, nor the tracheids of wood from dicotyledons, which gives bamboo a very high tensile strength in the direction of the fibres.

From the point of view of the chemistry of bamboo, it can be said that the nodes contain less water soluble extractives, pentosans ash, and lignin but more cellulose than the internodes (Liese, opus cit).

[2]Vascular tissue with the primary function of water transport and that consists of vessels and tracheids and associated parenchyma and fibres.

[3]The first-formed primary xylem.

[4]The last-formed primary xylem, that matures after the organ has ceased to elongate and has reticulated thickened or pitted walls.

[5]Tissue with the major function of transporting metabolites, especially sugars, from sources to sinks.

According to the same author, the ash content is 1-5%, the silica content varies .5-4% on average increasing from bottom to top. Most silica is deposited in the epidermis.
The cellulose amounts to more than 50% of the chemical constituents. The lignin is the second most abundant constituent.
Wood normally contains 40-50% cellulose and 20-30% lignin, 10-30% hemicelluloses [6]. Cellulose is thoroughly mixed and sometimes covalently linked with lignin.
From the point of view of the physics of the material, it can be said that young one-year old shoots have a high relative moisture content of about 120-130%[7]. The nodes show a lower value than the internodes.
The above-mentioned author explains that 'shrinkage begins right at the beginning of seasoning as observed in different studies. It affects both the size and shape of the cross section. From green condition to about 20% of moisture content shrinkage goes from 4 to 14% in thickness and 3 to 12% in the diameter, but no shrinkage is noticed between 70 to 40%'. One wonders about the amount of prestressing that this process creates, which would be important in correlating small specimen testing to full culm testing, particulary in bending.
The fibres in bamboo culms are confined to the fibre strands and sclerenchyma sheaths of the vascular bundles.
Thick-walled mature bamboo is specially liable to crack. Collapse is a most serious seasoning defect. Liese (opus cit) argues that this is due to the fact that the outer fibre bundles are pressed together while the inner ones are stretched. A possible explanation may be that because tangential and longitudinal stresses are developed during drying, the different anatomical composition widthwise the thickness locates the resultant force towards the outer skin, creating inward bending moments along the circumference, thus producing tension perpendicular to the fibre in the inner skin, which then cracks, producing collapse.
This fact may play a significant role in dealing with the concentration of stresses at the

[6]Plant cell component that is soluble in dilute alkali or hot dilute mineral acids with the formation of simple sugars.

[7]There are reports of 195% for *Guadua s.p.* in Costa Rica.

bearings, or at load application points, where it is therefore possible to find less than the expected capacity.

Liese (opus cit) says that a close correlation exists between specific gravity and maximum crushing strength, a claim that is widely supported in the literature. Ghavami (1988) also maintains that the fibres are the main source of strength. But fibre content and density are rather variable. Liese (opus cit) says that besides marked variations from bottom to top within one culm, there are differences between individual culms from the same stand. This has to be taken into account in attempts to classify this material for engineering purposes.

It has been argued that bamboo is much harder than wood. In actual fact, it is a real problem for saws and sawing. Probably the major difference between bamboo and wood in this respect is not hardness, but the characteristics of its fibres[8]. First of all, fibres are all aligned to the axis, which means a perpendicular encounter with the faces of the saw. Second, because of the concentration of fibres at the cutting planes, the abrasiveness of bamboo is much higher than that of wood. Tests on this were performed in Malaysia (Latiff et al; 1988). Differences in wearing capacity between bamboo and rubber wood were as much as five-fold were found. Friction is a relevant factor in saw-ability, thus bamboo saws should be fabricated bearing this in mind.

Some ideas can be advanced in relation to the mechanism of failure when strains grow in the direction perpendicular to the fibres.

It has been found (Jeronimides, 1976) that, for wood, the work of fracture for cracks propagating across the grain is about 100 times greater than those propagating in the grain direction. On the other hand, in fibre-reinforced materials, fracture working perpendicular to the fibre is mainly due to the frictional work dissipated in pulling out fibres from the matrix. It is argued that in the case of wood this mechanism is not available or it is very small. In any case, since bamboo fibres are longer than wood fibres, and exhibit large diameters as well, and on top of it all they fully contribute to the tensile capacity because of their

[8]Some people believe that the difficulties for sawing are caused by the high silica content of the skin of bamboo, but laboratory observations of the author during this research indicate that difficulties remain, even when the skin has been removed.

orientation, it is logical to assume that this friction mechanism is far more important in the case of bamboo. In the radial and tangential directions this mechanism is not at all available, so therefore the amount of crack energy is much less. In other words, in the transversal direction only the matrix plays a direct role in the strength.

2.2.-Tangential tension capacity of bamboo

2.2.1.-Test specimen

The experimental study of this parameter is complicated because of the round shape of bamboo, thus making it difficult to perform a test keeping control of eccentricities within the thickness of the specimen.

A test specimen as shown in figure 2.3 was used for such purposes.

Forces are applied to the specimen by means of steel bars as indicated. In our case a constant rate of deformation was preferred as a controlling method, and 0.6 mm/min was maintained all along the different experiments. Dimensions are indicated in the figure.

The test is actually a three-point bending test, carried out on an element that probably behaves more like a deep beam, and that has two comparatively large holes where forces are applied. The analysis of stresses in this type of specimen is not straightforward, because the proposed shape creates complicated distributions.

So that this type of problem could be studied, a finite element model of the specimen was set up, and the calculations were done using the Diana Finite Element Code (Anonymous,1991). Axisymmetric quadratic elements were employed, and orthotropy was simulated by assuming different values for the elastic moduli in the directions parallel and perpendicular to the fibres. Completely proportional behaviour was assumed, as most of the experimental evidence suggests is the case. Though fibres can be regarded as reinforcement, they were not modelled, under the assumption that a brittle failure would occur in the matrix before any important bending deformation could occur in the specimen, allowing no change for the 'reinforcement' to act (it has to be noted that bamboo is a unidirectionally reinforced composite, so that reinforcement would be effective only when stresses make an angle different from 90 degrees with the fibres).

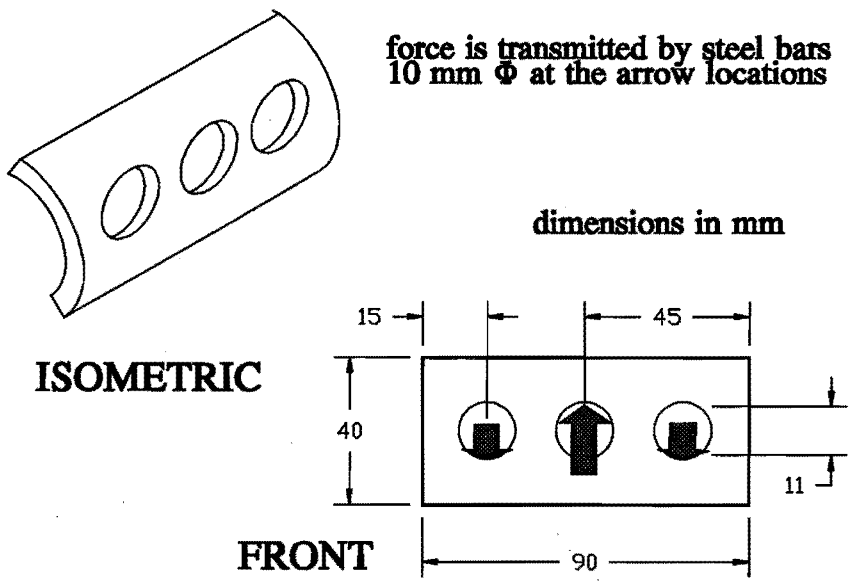


Figure 2.3: Specimen for tangential tension [mm].

The first element of concern was to know whether it was possible to find an appropriate place in the specimen, where strains perpendicular to the fibres could be measured. In other words, a place where these strains might occur without major disturbances from other effects.
Figure 2.4 shows a typical distribution of tangential strains, and figure 2.6 shows the same for shear stresses in the specimen. It should be noted that there is an area at the mid-point between the holes where conditions are good, since stresses there can be considered as being purely normal to the fibres.
At the beginning of the analysis, a value for the tangential E modulus (perpendicular to the fibres) had to be guessed. It could be assumed that the figure is close to that of lignin and, according to Janssen (1981), it has been taken between 1000 and 2000 N/mm^2 in this research. The specimen was analyzed under these two extreme conditions, and it was found that the relation between peak vertical stresses and average ones was equal to 2.97 on average. Now, the peak value occurs, as indicated in figure 2.5, as a compressive or contact

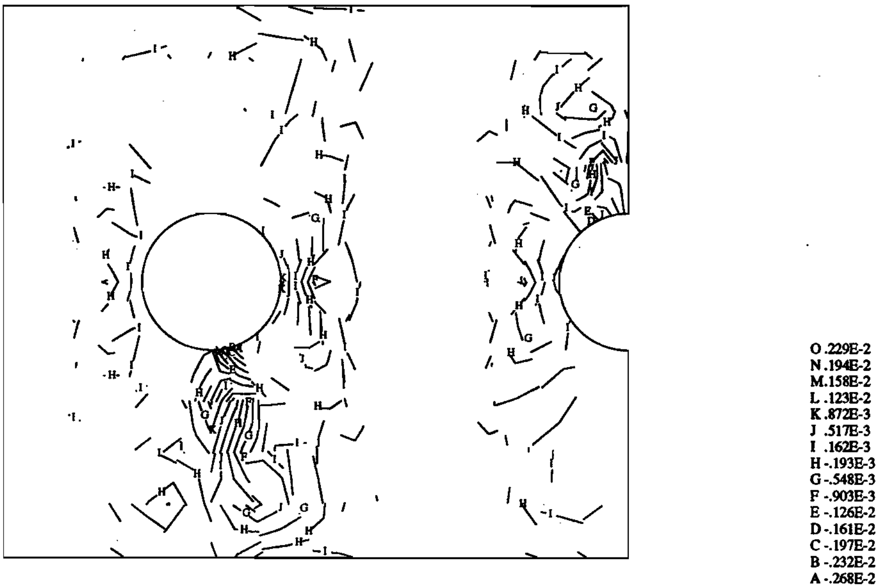


Figure 2.4: Tangential strain distribution in the specimen (mm /mm).

stress (see the bottom of the left circle, where concentration of lines can be observed). On the other hand, peak shear values occur in the region indicated in figure 2.6 by a concentration of lines at the bottom right of the left circle. It was later observed that, most of the time, the crack tends to start right at that point.

For either of the two extreme values of the tangential E modulus, stresses along the horizontal

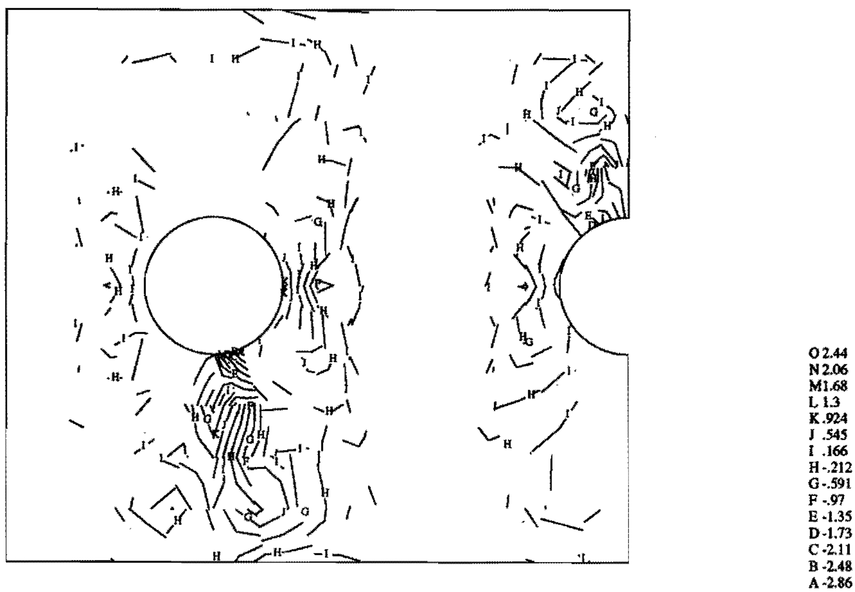


Figure 2.5: Tangential stress distribution in the specimen (N/mm^2).

mid plane were found to be very small for deformations like the ones observed later in the laboratory, close to the moment of failure. During the experimental phase, it was decided to calculate average tangential stresses in the plane of failure, so that the tendency of this figure could be compared to that of density and moisture, or any other physical property , under the assumption that if such a relation existed, it would be observed even by checking this gross

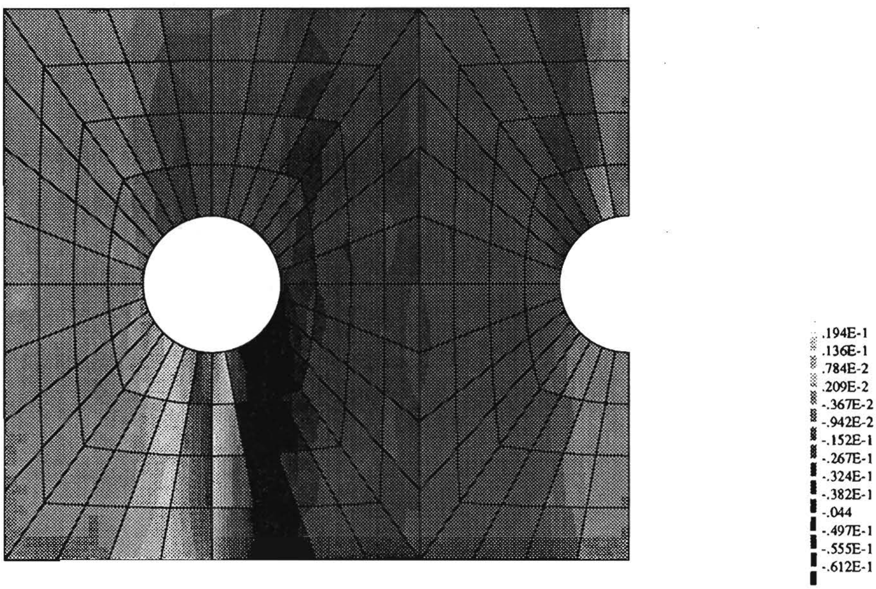


Figure 2.6:Shear stress distribution.

result (because the true values were just scaled). So in what follows it must be borne in mind that the values of strengths are only nominal ones.

2.2.2.-Experimental observations:

In the results reported here density has been measured by immersion of the whole specimen so that a better average could be achieved, but evidence suggests that the average of two small samples from both sides of the specimen would equally do.

Table 2.2 presents a summary of statistics of two samples of bamboo with different origins, *Guadua s.p.* from Costa Rica, and *Gigantochloa scortechinii* Gamble[9] from Malaysia.

The sample of *Gigantochloa scortechinii* Gamble was brought to Eindhoven only for the sake of having some indication of the possible stability of the results over the maximum strain found for *Guadua s.p.*, as will be explained later, but it is acknowledged here that the sample is small (12 specimens picked at random from a stock in the Forest Research Institute, Malaysia) and that more systematic tests must be undertaken before being conclusive on characteristic parameters for this or any other species.

In respect to the results for *Guadua s.p.*, experiments were exhaustive, many samples were carefully taken on different occasions, and resulting data proved to be consistent enough to be considered reliable (in total 50 tests were done for specimens with different positions in the culm and of different diameter, these properties being random ones).

The first striking figures in the table are the non-existent correlation between density and strength and maximum strain. It is important to mention that maximum strain is an unmistakable feature, since failure occurs in an absolutely brittle way (in the case of non brittle failure there is always room for discussion about this figure), and density was carefully determined in this investigation.

[9] Gratitude is expressed to Mr. Latiff Mohamad from the Forest Research Institute Malaysia, who supplied us with a small sample.

	Guadua s.p.	*Gigantochloa scortechinii*
av.strength	2.608	2.445
E modulus	2122.8	2444.7
maximum strain	0.001386	0.00111
av. density	795.7	788.68
r^2 density-strength	0.3568	-0.5134
r^2 density-strain	-0.1362	0.0035

Table 2.2: Summary of results. [N/mm^2],[Kg/m^3].
Average stresses are taken.

It is convenient at this time to take a look at the possible mechanism of failure, before discussing other results.

Figure 2.7 shows a close-up of the radial plane of failure for a specimen of *Gigantochloa scortechinii* Gamble and figure 2.8 does the same for a specimen of *Guadua s.p.*. Cracks develop joining weak points within the matrix, normally being situated around the intervascular bundles. When the direction of growth of the crack crosses through a bundle, it follows it surface or that of a fibre strand along the interface with the parenchyma cells and other weak structures.

It seems clear that the more fibres there are present, the more likely it is that the crack is longer. If, on the other hand, an extreme situation of only fibres and no matrix is taken, then probably the strength would be very low as well. In other words, like in any other composite material, there has to be an optimum combination of the number of fibres and the amount of matrix.

But yet another factor that plays a significant role in the strength is the density of the matrix itself, since the presence of voids would imply less energy needed to split the element.

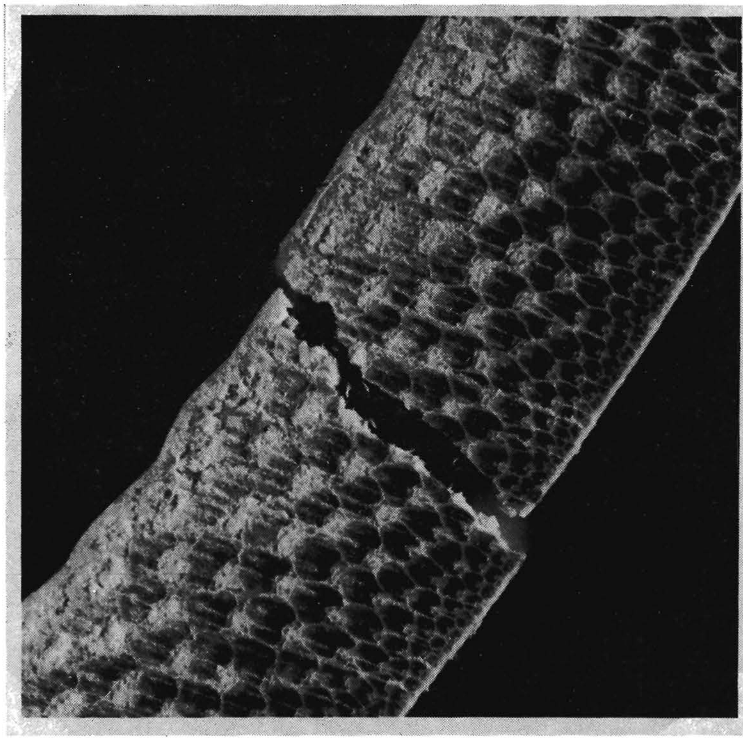


Figure 2.7: Radial crack in *Gigantochloa scortechinii.*

The above argumentation suggests that a sort of ceiling exists in relation to density and strength, that is to say, it is logical to think that density would increase the capacity, but only to a certain maximum, beyond which it may even have a negative effect. In the other direction, about the same can be said, because there can be a combination of few fibres and a dense matrix (and therefore very straight cracks), which would then produce another peak capacity, but the latter would decrease downwards from there if the density of the matrix decreases (giving rise to an *imperfect* matrix).

The above argument is put forward more with the intention of motivating specialists in the

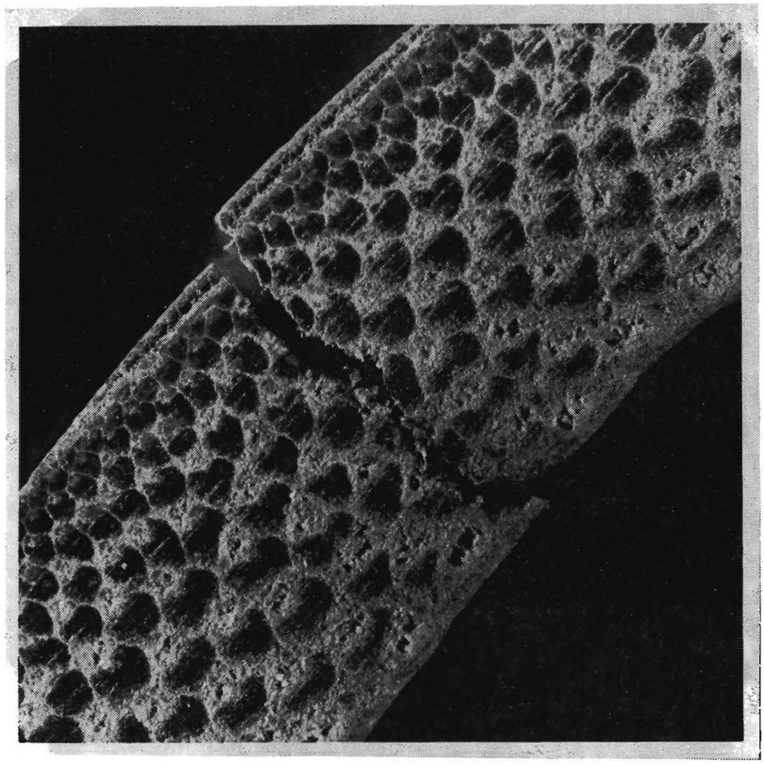


Figure 2.8: Radial crack in *Guadua s.p.*.

field because the current research project can not be exhaustive on this matter. Laboratory observations shed some light on the fracture fashion of bamboo, enabling to indicate that the above remarks have some objective basis.

The correlation values in table 2.2 thus indicate that we are in the presence of samples which are very different in nature from the point of view of the way they resist cracking, though they do have something in common. In both cases, the maximum strain is about the same. In fact, a hypothesis test was carried out comparing the two samples, only to prove that there is no difference between their maximum strains. A similar conclusion was reached when these two samples were compared to another one of *Bambusa blumeana* from the Philippines.

Though more studies are needed on more species, some evidence has been found to support the idea that bamboo is able to carry an amount of strain that is rather invariable, or at least would be so within a small range.

As a matter of fact, it was this finding what finally made that focus of attention was concentrated into the examination of critical strains, as an interesting design criterion.

Janssen (opus cit) found that the value of Poisson modulus is very stable for bamboo, and with this it is possible to relate longitudinal strains and tangential ones, closing a circle that allows one to establish a design criteria for the material.

Therefore, it is proposed here that a criteria of maximum admissible tangential strain be eventually adopted as a limiting criteria for design purposes. This would greatly simplify design procedures, because, as has been previously observed by many authors, and as is plainly validated in this research report, bamboo behaves very proportionally. Thus the calculation of strains would be an easy matter and could function as an easy check of ultimate capacity. This will be the subject of more discussion in the following chapters, since validation of this must be found in different stress conditions.

The tangential maximum strains for *Guadua s.p.*, from several samples from different experiments, were pooled together so that a better appreciation could be achieved, especially over the variability of the figures. Results are summarized in table 2.3.

item	figure
average	1.1057×10^{-3}
standard deviation	1.6170×10^{-4}
standard error	5.7175×10^{-5}

Table 2.3: Descriptive statistics for *Guadua s.p.* from Costa Rica
Ultimate tangential strains

It is recommended here, that similar studies be carried out for culms from other species and origins, since a lot more data are needed to be conclusive on the generality of these findings.

It is possible that different critical values may be found for different bamboo types, but in any case, specific design values can be calculated for each characteristic condition.

2.3.- Parallel tension:

As reported by Janssen (opus cit), a large amount of problems arise when dealing with testing the tensile strength of bamboo, mainly due to the very weak transversal direction of the material (as shown in the previous section). When pressure from a grip is applied, the most common result is a crushing effect in the area of the supports, generating very high shear stresses in the interfibre planes.

Most specifications for the tensile test of other materials, including wood (see for example, ISO 3345), recommend that the cross section of the specimen be reduced (gauge length), so that the area or section of failure can be kept under control and therefore final stresses can be accurately calculated.

For the purpose of structural capacity, the most relevant datum is the average tensile strength for the whole cross-section, and from this point of view it was decided that the full thickness had to be taken into account in the specimen (as opposed to examining the variation depending on the position across the thickness).

In figure 2.9 the transition zone from the gauge and the supporting area on a normal specimen is shown. Two small sections of *dy* high are taken, one on each side of the limiting section a-a in the figure and there is an increment of

$$x=\frac{t_A}{2}+my \qquad \text{eq.2.(1)}$$

on both sides of strip B, for every increment y. Then, once the tensile load F is applied to the specimen, deformations in the y axis will occur. If perfectly proportional behaviour is assumed then the deformations in each one of the elementary strips are

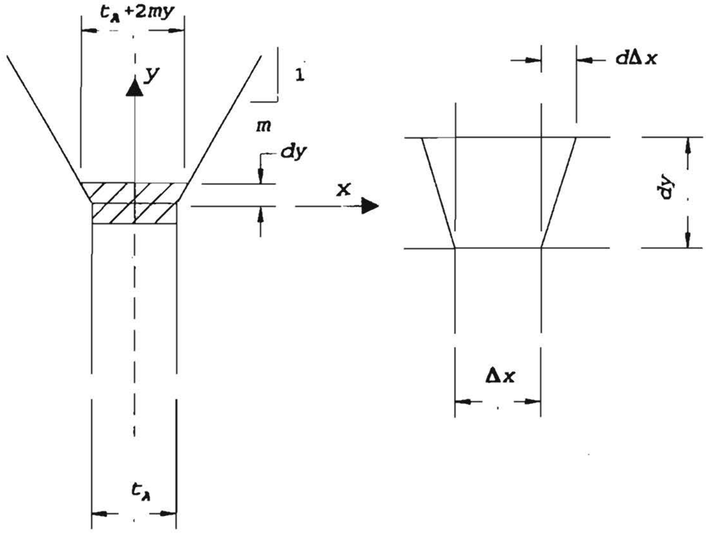


Figure 2.10: Transition zone between the gauge and the supporting section.

$$(\epsilon_y)_A = \frac{\frac{N_y}{t_A t}}{E_y} \qquad \text{eq.2.(2)}$$

$$(\epsilon_y)_B = \frac{\frac{N_y}{(t_A+2my)t}}{E_y} \qquad \text{eq.2.(3)}$$

Due to the Poisson effect, each section of the specimen will also deform in the lateral or, in this case, x direction. For strips A and B respectively this deformation is

$$(\epsilon_x)_A = \nu \frac{\frac{N_y}{t_A t}}{E_y} \qquad \text{eq.2.(4)}$$

and

$$(\epsilon_x)_B = \nu \frac{\frac{N_y}{(t_A + 2my)t}}{E_y} \qquad \text{eq.2.(5)}$$

From equations 2.(4) and 2.(5) it can be seen that there is a difference between the unit lateral deformations in each one of the successive elementary strips. This difference leads to the appearance of shear deformations which have to be mainly taken by the matrix of the material, because of the type of structural array in bamboo. From figure 2.10 it can be seen that the shear angle due to the difference in deformation is

$$d\gamma = \frac{\frac{d\Delta\epsilon_x}{2}}{dy} \qquad \text{eq.2.(6)}$$

and the corresponding shear stress is

$$d\tau = \frac{E_x}{2(1+\nu)} d\gamma \qquad \text{eq.2.(7)}$$

From equations 2.(4) and 2.(5)

$$\frac{\Delta\epsilon}{2}=\frac{N_y\ \nu}{2\ t\ E_y}\left(\frac{1}{t_A}-\frac{1}{t_A+2my}\right) \qquad \text{eq.2.(8)}$$

Back-substitution in 2.(7) gives

$$d\tau=\frac{E_x}{2(1+\nu)}\frac{d}{dy}\left[\frac{N_y\ \nu}{2tE_y}\left(\frac{1}{t_A}-\frac{1}{t_A+2my}\right)\right] \qquad \text{eq.2.(9)}$$

which after integration turns out to be

$$\tau=-\frac{E_xN_y\ \nu m}{2(1+\nu)tE_y}\int\frac{dy}{(t_A+2my)^2} \qquad \text{eq.2.(10)}$$

Integration gives

$$\tau=\frac{E_xN_y\nu}{4(1+\nu)(t_A+2my)E_yt}+\tau_i \qquad \textbf{(11)}$$

Since it can be assumed that no change occurs in the section below line a-a, then

$$\tau_i=0 \qquad \text{eq.2.(12)}$$

In order to know the value of shear stresses at the root of the gauge section, then a limit value of eq. 2.(12) has to be calculated, so

$$\lim_{y\to 0}\tau=\frac{E_x\nu N_y}{4(1+\nu)E_yt\ t_A} \qquad \text{eq.2.(13)}$$

which for more generality can also be written as

$$\tau = \frac{E_x \nu \ \sigma_t}{4(1+\nu)E_y}$$ eq.2.(14)

For average values of material parameters[10] the above equation gives

$$\tau \approx 2.2 \quad [N/mm^2]$$ eq.2.(15)

From data in this chapter it can be seen that this is too close to the maximum stress that bamboo can carry in the interfibre planes.

That is to say, the shape proposed in ISO 3345 for wood might not be suitable for a material like bamboo, since there will be a tendency to produce a failure of the specimen at the root of the gauge zone.

It seems, therefore, that a straight specimen may be more promising in the case of bamboo, or at least one with only a very slight change in section.

An additional conclusion that can be reached from the above result, is that materials with a structure like that of bamboo behave very poorly when initial cracks or sudden changes in section occur, especially in areas with a high concentration of tensile stresses. An initial crack may be regarded as a sudden change in section and therefore a similar phenomenon to the one just described will occur.

In choosing a way to measure parallel tension, another problem has to be taken into account, since large compressive stresses in the transversal direction are developed by the grips. What is more, the difference in toughness between the steel grips in the testing machine and bamboo is so important, that it is advisable to place a transition material between the grips and the specimen to avoid damage by contact.

Based on the above and after some trials, the specimen of figure 2.11 was used.

During the development of the specimen, it was found that hard wood is necessary for the

[10] say tension capacity is 270 N/mm², Young's modulus 2000 and 14000 in x and y respectively.

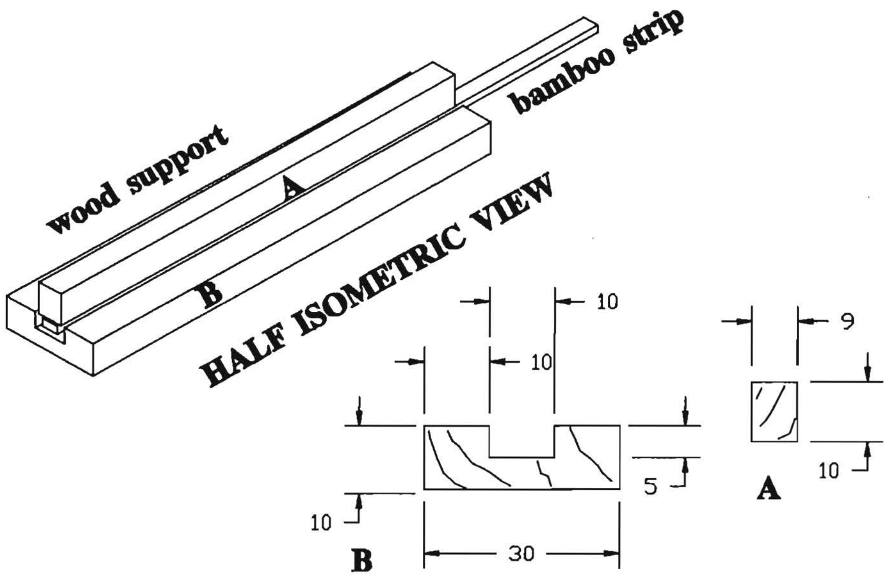


Figure 2.11: Parallel tension specimen for bamboo [mm].

making of the supports, since soft woods tend to split and slip under the application of loads. Of special importance is the choice of gluing length. This very much depends on the type of glue and of course on the amount of force to be expected[11]. Therefore, some trials are always advisable to get an insight into this problems, and calculations are needed to finally determine the total length of support.

2.3.-Some notes related to the modelling of bamboo elements

Appendix A includes the results of some experiments carried out at the Eindhoven University of Technology, to observe some of the structural features of bamboo in parallel tension. Figure 2.12 shows a typical stress strain curve for tension that supports the fact that bamboo behaves **proportionally**, and fails in a brittle manner under tension.

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[11]See appendix C for a way of determining glue capacity in the lab.

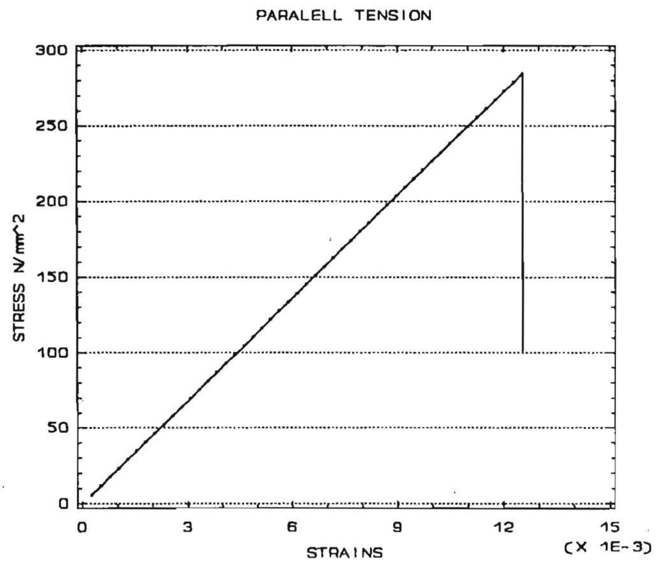

Figure 2.12: Stress strain curve from a parallel tension test.

A problem that will later prove to be of great importance is the determination of mechanical and geometric parameters for the modelling of the nodes. It has on occasion been argued that the thickness of the node is an x number of times the thickness of the internode, a fact that should be precisely defined to be taken into the analysis.

Figure 2.13 shows a longitudinal section across the node region of a sample of *Guadua s.p.*. It can be observed that the real shape of the node is complex in nature, ranging from a well defined disk to a sort of hilly plate. Thickness is small compared to that of the internode, and the transition region between internode and node occurs in a small region. In this transition region fibres bend towards the centre of the diaphragm, or they just stop, or, in the case of the outermost group, they point outwardly.

All this discontinuity and complexity in the shape of the node region calls for a more indirect way of determining its properties.

To calculate the elastic modulus of the nodes as reported in this chapter, strain gauges of 30 mm were located as shown in figure 2.14, so that the readings averaged the effects there. In this way the node Young's modulus refers to that average, and whenever used for calculation purposes, a node length of 30 mm should be used. The problem of the determination of the thickness can be overcome since for the length of 30 mm thickness can be considered constant, because whatever value it takes in the diaphragm, its weight within the 30 mm length can not be relevant.

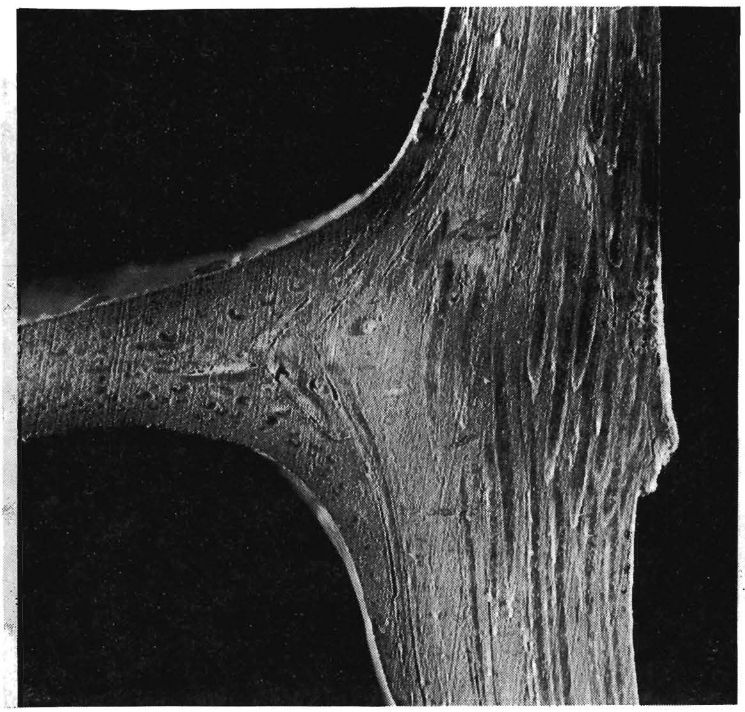

Figure 2.13: Longitudinal section along a node.

Another important reason for following the above recommendation is that the fibres in the transition zone can not play an important role in the longitudinal stiffness of the element, because before they can take any direct tension energy they have to be straightened, and for that purpose it is expected that very little energy is needed.

As presented in appendix A, nodes are the weakest and the softest components of the culm. Average capacity is only about 30% of that of the internode, and the average elastic modulus is only 40%.

Since they are the weakest link, their properties are of paramount importance in the definition of design capacities. Their influence in the elasticity of the members becomes very

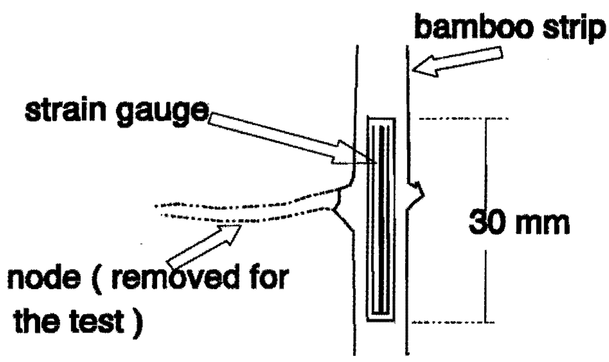

Figure 2.14: Location of strain gauges at the node region.

important as well. The relevance of this effect is a matter of discussion in appendix F of this thesis.

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Chapter 3: Compression Behaviour of Bamboo Culms

3.1- Introduction:

Meyer and Ekelund (1923) tested a total of 11 specimens under compressive stress, reporting an average strength of **39.27 N/mm²**, for specimens of 120 mm height and about the same in diameter. These authors report that two types of support conditions were employed: a lead cushion in between the specimen and the testing apparatus, or a support in which the specimen was placed in direct contact with the steel plates of the machine. **The type of failure was described as a set of vertical cracks which opened wider and wider with increments of the load.** When lead was used, the outer layer of the culm penetrated the lead cushion, showing that this layer is much stronger than the inner part of the culm. The authors report that when no cushion was used, the outer layer came apart from the inner section by shear, which was immediately followed by break-down.

Espinosa (1930) carried out compression tests using specimens 1200 mm and 350 mm in height, with a speed of deformation of 0.02 mm/s, on *Bambusa spinosa* from the Philippines. The author says that to resemble "actual" construction conditions, he U-shaped one of the ends, while the other was kept flat at an angle of 90 degrees. The results ranged from **27** to **32 N/mm²** for the 1200 mm specimens and from **51.8** to **82.8 N/mm²** for the shorter ones. He also tested small pieces of a cross-section (9 by 15 mm and 25 mm in height), getting a mean ultimate compressive stress of **53.5 N/mm²**.

In a summary report Roudakoff (1947) presents the average compressive values for bamboo from four different localities. No further details are given about the species, physical properties or method of testing.

Ota (1950) studied the relationship between the percentage of structural elements and the density and compressive strength of bamboo specimens. For that purpose specimens were taken from the outer, the inner and the full section of the wall thickness, with the approximate shape of a very small cube, and compression tests were carried out at a speed of 0.04 mm/s. For moisture contents between 10 and 14.7% Ota found correlations between the mass per volume and the ultimate compression stress of the outer skin, the inner skin and the middle part of the cross-section, for *Phyllostachis edulis* and *Phyllostachis reticulata*. The mean values obtained for pieces containing the whole section were **81.6** and **83.1 N/mm²** respectively for both species, for related masses of **965** and **866 kg/m³**. He found that compressive strength decreases towards the node.

Limaye (1952) carried out a matricial analysis of different factors presumably related to the mechanical properties of bamboo, namely moisture content, presence of node in the specimen, position along the culm, and age.

Some comments will be made to summarize the author's conclusions.

In regard to the position of the specimen along the culm and its influence, the author states that " the main effect between the three positions is significant in all cases. But the interactions between ages and positions are not significant for compression and the interactions between the position of nodes and positions along the culm are not significant for modulus of elasticity and compression". A conclusion in this research is that the modulus of Young is much higher for the bottom position than for both the middle and the top position in bamboo culms, a fact that the researcher associates with " the greater wall thickness of the culm near the ground level" (judgement on this affirmation is left to the reader !). Seasoning was found to be an important factor, increasing the strength of green bamboo by more than 40%, and it was also found that there is an increase in the strength with age up to three years, though no further increase in age was studied. The species was *Dendrocalamus strictus*.

Sekhar and Rawat (1956) made a first attempt to establish a standard test for compression of bamboo specimens, calling for a ratio of 10 or less for length to wall thickness. This comes from the review of the work by Limaye (opus cit) and others and the comparison with specifications for wood. They also suggested that two specimens should be taken, one in the green condition and the other being kiln dried to 12% moisture content, both originating from the internode section of the culm.

Atrops (1969) studied the compression and elastic characteristics of bamboo by performing a total of 108 tests using three types of specimens : a) internode b) nodes at the extremes of the specimen and c) node at the middle of the specimen . Tests were done at a loading rate of .33 N/mm²/s and a moisture content of 18.1% on average. The size of the specimens was chosen according to a relation of diameter to height of 1 to 4. Maximum and minimum strengths found for specimens type a, b and c were, respectively, 522 - 402, 537 - 407, and 528 - 433 N/mm^2.

Janssen (1981) performed a very extensive literature review and a comprehensive set of tests on the compression capacity of bamboo culms on *Bambusa blumeana*. Some aspects deserve special attention and will be summarized in the following.

The height of the specimens was 50, 100 and 200 mm, the diameter varied from 70 to 90 mm and the wall thickness from 5 to 9 mm. The following variables were taken into account, besides the height of the specimen:

-Moisture content

-position along the culm

-node and internode

The loading speed was set in the range of 2.7 to 3.2 kN/s or about 1.5 N/mm^2, for an approximate speed of deformation of 0.02 mm/s.

This author reports that **failure was either splitting or crushing and that no relation was found between this and the moisture content.** Results between **60** and **176 N/mm^2** are reported. It was found that the parameters **height of the specimen and node were not significant and a recommendation was given to search for an explanation for this effect.** The tests showed that there is no difference between the compressive strength of split bamboos and full culms, at least for the 5 % lower boundary, and a recommendation was made to carry out compression tests for split bamboos only, using a smaller machine.

A relation between compression strength and density was found as:

$$f_c = 0.094\ \rho \qquad \text{eq.3.(1)}$$

for 12% moisture content, and

$$f_c = .0075\rho \qquad \text{eq.3.(2)}$$

for green condition

A partial summary of the conclusions is:

-The moisture content is significant

-Node or internode in the specimen is not significant

-Height of the specimen within the area 1 to 3 times the diameter is not significant

-Instead of a full bamboo, a small piece of split bamboo may be used for the prediction of the 5 percent lower boundary.

Xiu-Xin et al (1985) carried out a set of experiments to study the relation between compression strength and the geographical conditions of four localities, for *Phyllostachys glauca*. To account for the fact that the culms are not perfectly circular in cross-section, a correction factor was introduced to the formula for the calculation of stresses, so that nominal stresses were divided by a factor of

$$\alpha = 1 + 158.5\Phi_e^{2.393} \qquad \text{eq.3.(3)}$$

A polynomial was found to calculate the compressive strength as a function of the age of the culm at the time of cutting as

$$f_c = 588.4 + 53.9t - 4.75t^2 \qquad \text{eq.3.(4)}$$

$$s = \pm 3.72 \frac{N}{cm^2} \qquad \text{eq.3.(5)}$$

$$t_{max} = 4.97 \ years \qquad \text{eq.3.(6)}$$

Espiloy (1985) carried out a very complete statistical analysis of the physical and basic mechanical properties of *Bambusa blumeana* and *Gigantochloa levis* from the Phillipines, following the recommendations of ASTM[1] for small specimens of wood. Variables included fibre length, fibre diameter, lumen diameter, cell wall thickness, vessel dimensions related to moisture content, relative density, shrinkage, compression strength, modulus of rupture and modulus of elasticity, and in turn these mechanic variables were statistically related to outside diameter, length-to-span ratio and culm wall thickness.

Prawirohatmodjo (1988) performed tests on both full culm 100 mm height and split bamboo 30 x 10 mm x thickness of the culm. The author reports that in his experiment **the presence of a node in the specimen does not seem to affect the strength**, and that moisture content is a significant factor. No data are included in the paper about the relation of compressive strength and density.

By request of the Costa Rican Bamboo National Project (Chaves and Gutiérrez, 1988), Sotela (1990) produced a handbook for 'physical and mechanical testing of bamboo'. It includes recommendations for the determination of density, contractions, bending, compression and shear capacity. In general, the specifications for bending shear and compression follow Janssen (opus cit), except that details are given on the number of specimens, the preparation of specimens prior to testing, and the protocol for reporting results. One important feature is the use of strain gauges on compression specimens though in a paper by the same author (Sotela, 1992) he maintains that 'it is difficult to calculate strains from compression tests, even with electronic devices'. In this paper the physical and mechanical properties of *Guadua s.p.* from two different localities in Costa Rica are compared. This author recommends that the elastic modulus of bamboo is calculated from bending tests only according to the recommendations of the above mentioned handbook. A comparison is made between bamboo and some hardwoods from Costa Rica, as summarized in the following table.

[1]American Society of Testing Materials.

	density [kg/m^3]	E modulus [kg/mm^2]	f_c [N/mm^2]
Cupressus lusitonicus	460.0	9800	41.1
Hieronyma achoneoides	790.0	14300	
Playmiscium pinnatum	860.0	16700	81.3
Terminalia lucida	770.0	13000	
Guadua s.p.	790.0	27700	42.0

Table 3.1: Bamboo and some Costa Rican wood types, after Sotela (opus cit).

The moisture content of each of these wood species was 12%, and it was 15% for the bamboo sample.

3.2 Structural behaviour:

3.2.1.-Failure Modes hypothesis:

As described in chapter 2, the anatomical structure of bamboo culms consists of fibres and vessels running parallel to each other and to the axis of the culm, with no radial structural elements as in the case of wood. The culm is made of a composite material which is the result of the combination of very strong fibres and a relatively weak matrix of lignin.

The examination of short elements from compressive tests points in the direction of at least four possible ways failure can occur; these are the so-called **failure modes**.

In the first place, if a specimen is set in a testing machine such that friction is negligible, and therefore, no horizontal reactions are possible, the state of stresses can be pictured as shown in figure 3.1. The top left of the figure shows the full specimen. It will be assumed in this case that the length of the specimen is comparable to the diameter, so that eccentricities due to tapering are negligible.

Detail A shows the fact that the axial stiffness varies along the thickness of the culm, increasing towards the outside. This means that axial stresses will be mainly taken by the outermost rings or layers of the culm, creating an uneven distribution of stresses, and inducing relative tensions in the innermost rings. The overall effect of this will be the generation of radial bending moments as shown in detail B.

The resulting compression in the outermost layers will generate, expansion of the rings in the tangential direction, due to the Poisson phenomena, as pictured in detail C, and it will create contraction in the innermost rings. The expansion of the outermost rings will eventually lead to longitudinal cracking as shown in figure B.2. The relative difference between the displacements in the outermost and the innermost rings also creates some sort of annular cracking, indicated by the radial arrows in detail C, as was observed in the laboratory.

In this way bamboo in compression will fail when a critical value of tangential stresses is reached in the outer skin, initiating a longitudinal crack. This mode of failure has been reported by, for example, Meyer and Ekelund (opus cit), and by Janssen (opus cit).

This failure mode leads to the conclusion that **fibre density** could contribute negatively to the strength of the specimen, since tangential tensile strength seems to be the more logical mechanism, and strength can be better related to the matrix in this case (see chapter 2, section 2.2).

Secondly, as mentioned in appendix B, it is possible that due to particular initial conditions of the geometry of the specimen, with an especially large influence from tapering in combination with the presence of friction, radial inward-bending moments occur, as illustrated in figure 3.2.

Thirdly, as explained in appendix B, when the specimen is short enough, it is possible to have failure by contact stresses, something close to a compression failure in a cylindrical or cubic specimen as observed in other types of materials. This mode of failure, which might be of interest from the point of view of the material, does not describe the way bamboo

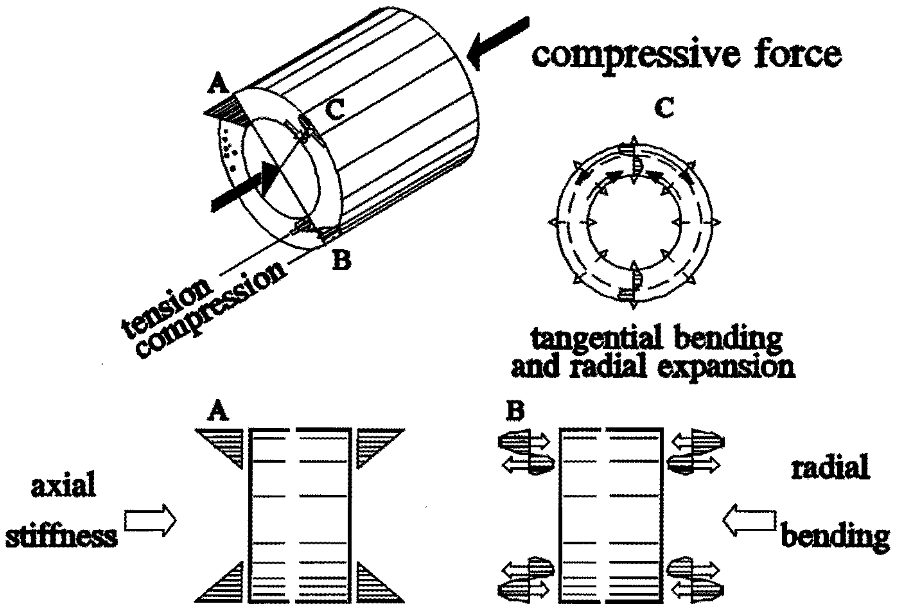

Figure 3.1:State of stresses of a specimen under frictionless compression

culms are likely to react.

Fourthly, figure 3.3 shows what may happen when the slenderness is high and friction is generated between the plates and the specimen. As the quantification of this phenomenon might be of some interest in determining the role of the different parameters involved, some detailed attention is given to its analysis in the following lines.

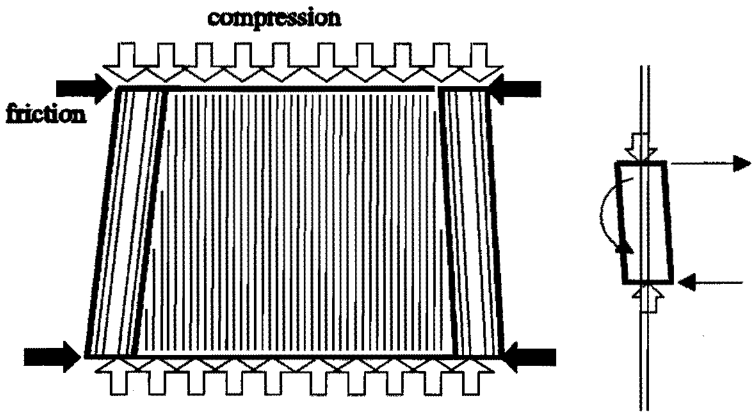

Figure 3.2:Longitudinal section of a tapered specimen under compression.

3.2.2- Lateral buckling analysis:

For the analysis of the element shown in figure 3.4 the following premises are assumed to hold :

1- The cross-section of the culm is a circular ring with a constant diameter and wall thickness.

2- E_x is constant along the element.

3- E_θ is constant

4- Deformations are small

5- Material is perfectly elastic.

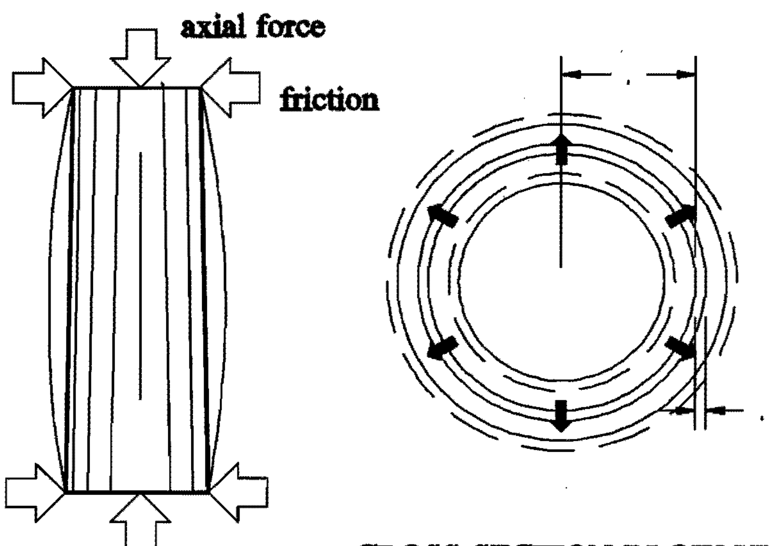

Figure 3.3:Lateral buckling

As shown in figure 3.4, two lateral displacements can be considered, e due to the lateral expansion of the specimen, and w_r due to the bending deformation of the wall of the culm. It can be shown that the relation between tangential strains and radial expansion is given by

$$\epsilon_\theta = \frac{w_r}{r} \qquad \text{eq.3.(7)}$$

or

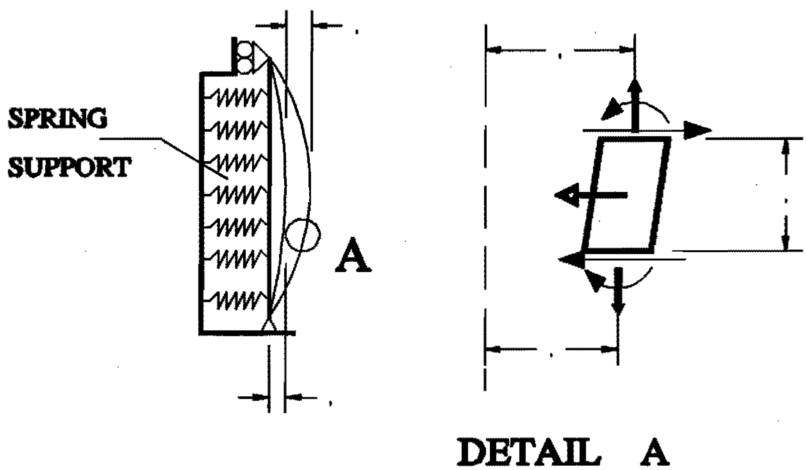

Figure 3.4:Beam on elastic foundation model

$$w_r = \frac{\nu \sigma_x r}{E_x}$$ eq.3.(8)

Tangential stresses are given by

$$n_{\theta\theta} = \sigma_\theta \, t$$

or

$$n_{\theta\theta} = E_\theta t \, \frac{w_r}{r}$$ eq.3.(10)

Equilibrium of radial forces on an infinitesimal sector gives

$$w_r k_r r d\theta = n_{\theta\theta} d\theta \qquad \text{eq.3.(11)}$$

from which we gain, after simplification and substitution

$$k_r = \frac{E_\theta t}{r^2} \qquad \text{eq.3.(12)}$$

which is the value of the reaction coefficient in the radial direction.

The sum of moments gives

$$\frac{\partial m_{xx}}{\partial x} - v_r + n_x \frac{\partial (w_r + e)}{\partial x} = 0 \qquad \text{eq.3.(13)}$$

which after derivation becomes

$$\frac{\partial^2 m_{xx}}{\partial x^2} - \frac{\partial v_r}{\partial x} + \frac{n_x \partial^2 (w_r + e)}{\partial x^2} = 0 \qquad \text{eq.3.(14)}$$

Equilibrium of forces in the radial direction implies that

$$\frac{\partial v_r}{\partial x} - k_r\, w_r = 0 \qquad \text{eq.3.(15)}$$

Substitution of this value in eq.3.(14) and the fact that

$$m_{xx} = -D_x \frac{\partial^2 w_r}{\partial x^2} \qquad \text{eq.3.(16)}$$

gives

$$D_x \frac{\partial^4 w_r}{\partial x^4} + k_r w_r - n_x \frac{\partial^2 (w_r + e)}{\partial x^2} = 0 \qquad \text{eq.3.(17)}$$

A solution of the above equation is

$$w_r + e = (\hat{e} + \hat{w}_r)\sin\frac{\pi x}{l} \qquad \text{eq.3.(18)}$$

thus eq.3.17 becomes

$$\hat{w}_r \left(\frac{\pi}{l}\right)^4 D_x + k_r \hat{w}_r - \sigma_x t \left(\frac{\pi}{l}\right)^2 (\hat{e} + \hat{w}_r) = 0 \qquad \text{eq.3.(19)}$$

from where it can be seen that

$$\hat{w}_r = \frac{\sigma_x}{\left(\frac{\pi}{l}\right)^2 \frac{D_x}{t} + \frac{k_r}{t\left(\frac{\pi}{l}\right)^2} - \sigma_x}\,\hat{e} \qquad \text{eq.3.(20)}$$

Taking into account that

$$\hat{w}_r = \infty$$

then, after the proper substitutions

$$\sigma_{cr} = \frac{\pi^2}{\left(\frac{l}{t}\right)^2}\,\frac{E_x}{12(1-\nu^2)} + \frac{E_\theta}{\left(\frac{r}{l}\right)^2 \pi^2} \qquad \text{eq.3.(22)}$$

The first component of the right hand side part of this equation can be recognized as the buckling load of the culm when initial cracking exists, or when the tangential capacity of the culm is negligible. The second component is, therefore, the contribution of the lateral stiffness of the specimen. To get some idea of the participation of these terms in the whole phenomenon, an example can be examined.

Suppose that a specimen with the following characteristics is taken

$$r = 40mm \quad t = 6mm \quad E_x = 16000\frac{N}{mm^2} \quad E_\theta = 2000\frac{N}{mm^2} \quad l = 60mm$$

For this case

$$\sigma_{cr} = 144 + 456 \quad \frac{N}{mm^2}$$

This means that the normal recommendation, to take the length as ten times the thickness of the specimen to prevent buckling, is far too conservative for bamboo, since the first component of the critical load (cracked specimen) already gives a value that is almost three times the average compressive capacity of bamboo.

It can be concluded that lateral buckling of the walls of the specimen is not a critical mode in compression tests. It has to be taken into account that the model of eq.3.(17) is even more conservative, because it does not consider the effect of radial reacting bending moments in the surface of contact between the specimen and the plates of the machine. Experience indicates that the height of the specimen should be equal to the diameter at most, in order to avoid the influence of tapering.

Finally, some comments about the influence of the nodes on the compression capacity. Janssen (opus cit), Limaye (opus cit), and Sekhar and Rawat (opus cit), among others, reported that no influence of the nodes on the compression capacity could be found. In compression tests the node impedes the lateral expansion of the specimen working in tension. If the low tensile capacity of the nodes is recalled, on the one hand, and on the other, the fact that nodes are not flat disks but curved surfaces then it is expected that indeed their contribution to the lateral stiffness of the walls is small. Even though this is the case, a simplification can be made by assuming that they are flat disks. The complete analysis is not presented here, but it can be shown that under these conditions, at the maximum compressive capacity, average tensile stress in the nodes are equal to about 30 N/mm^2, about ten times their expected capacity, which would explain the experimental findings.

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Chapter 4: Glueability of bamboo wood phases

4.1- Introduction :

No indication of previous studies on the gluing characteristics of bamboo phases could be found in the literature. This is not a surprising fact, since the use of bamboo in furniture making, for example, follows the tradition of joints being finely adapted pieces made to fit each other. Elements are connected by insertion through bored holes, or binding, or shaping by heating and so forth. The grade of structural redundancy in these pieces of furniture, on the one hand, and the refined details on the other, make it possible to avoid fittings other than those made of bamboo itself, and so far the possibility of gluing has not been seen as a necessary improvement in the process. This perhaps explains why little or no attention has been paid to the gluing properties of bamboo.

Resins have been used in experiments for the fabrication of bamboo composites (Jindal, 1988). Urea formaldehyde and Cardanol-Phenol-Formaldehyde have been in use in the production of ply-bamboo in India as well (Zoolagud, 1988), but no report was found on the properties of glued phases.

As has been mentioned in previous chapters, the mechanism of failure in bamboo is splitting, when strains reach a maximum sustainable value. From this point of view gluing seems an attractive solution to a number of problems, like the reparation of cracks, or, more importantly, the connection of elements. Glue may not only be useful for force transmission, but to keep the fibres together at points of load concentration as well.

In this chapter the structural possibilities of gluing are discussed. A more detailed description of testing procedures and quantitative results can be found in appendix C.

4.2.-Structural performance of bamboo wood glued phases:

4.2.1-Description of some factors and their influence on gluing capacity:

As is explained in appendix C, forces and deformations are perfectly proportional in bamboo wood phases (referred to as 'glued phases' in the remaining part of the thesis for the sake of simplicity). A description of the test specimen employed during this research can also be found in the same appendix.

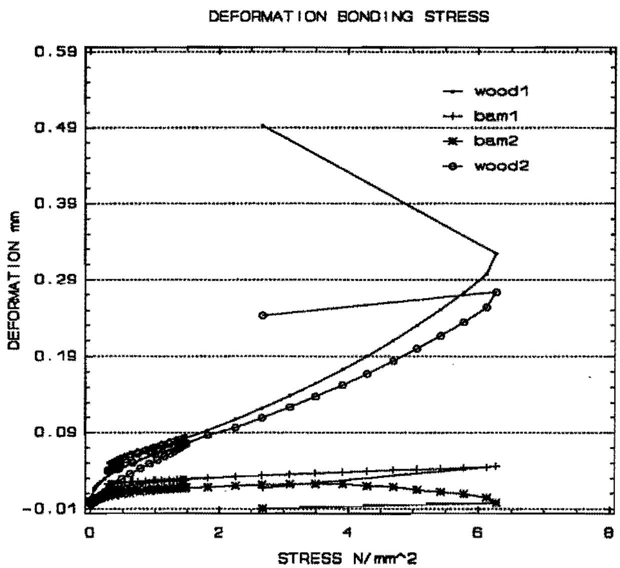

Figure 4.1: Bonding test showing all registered deformations. Glue type: PVAc (white wood glue).

Figure 4.1 shows results of a bonding test. Each one of the curves is codified to identify the corresponding point of measurement. The codification corresponds to pins, in such a way that those beginning with 'wood' are related to pins on the surface of wood, and those beginning with 'bam' to pins located on bamboo. The depth difference between the layer where bamboo pins and wood pins were located was about 1 mm (see figure C.2). It can be seen that at the depth where the bam pins are located very little deformation occurs, compared to that of the wood pins. This is an indication of the fact that only the innermost layer of bamboo is able to take stresses from the glue , due to the low shear capacity of the matrix. Glued phases actually fail because of shear in the interfibre surfaces.

It can be expected that the shear behaviour of bamboo differs from point to point because of the different concentration of fibres. Different crack lengths may occur as a consequence.

Despite of this fact attention is paid only to the gluing capacity of the inner layer of the culm in this research report, because of a predetermined interest in transmitting forces to or from the inside of the culms

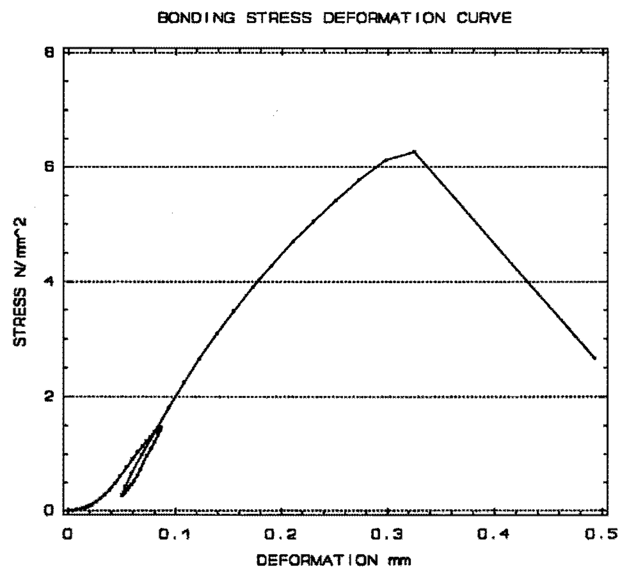

Figure 4.2: Stress deformation curve.

In figure 4.2 it can be seen that the relation between stresses and deformations is very smooth, almost directly proportional for most of the range. Failure always occurs in a rather sudden way, and it looks as if the preloading loop does not seriously affect the characteristics of the curve.

An experiment was set to examine the influence of certain parameters on the bonding capacity of the phases. Considered factors were :

- Bamboo density (represented as the average along the thickness).
- Bamboo thickness.
- Bamboo initial diameter.

- Type of wood either hard or soft[1] .
- Type of glue, either PVAc or Araldite (in what follows called 'resin').[2]

An analysis of variance showed that none of the above factors on its own, nor in combination to the other ones, significantly affects the results of bonding capacity as shown in table 4.2 below.

In relation to the type of glue, improvements on its quality do not alter bonding capacity unless penetration patterns change, because of the low shear capacity of the inner layers of bamboo. The experiments showed that glued phases always fail within the first mm of the internal part of the culm, bamboo is thus the weakest link in the glued phase.

Table 4.1 shows the results of an analysis of variance for all the studied factors.

Although changes in the considered factors do not by themselves influence the bonding capacity, it is of interest to obtain the 95% confidence limits for the means after an analysis of variance. Such a result is shown in table 4.2 below.

Such separate analysis helps to visualise the effects of the diverse variables, to confirm the previously stated result.

The figures show that in spite of the fact that no significant difference exists between the two types of glue, the resin seems to be less stable, yielding more scattered results than PVAc glue.

The experiments showed that the strength distribution was different for the two types of glue examined, so though the average value for the resin was slightly higher than wood glue (6.33 against 5.78 N/mm^2), a material factor calculated according to equation C.1, appendix C, produces hardly any difference in the design capacity.

Experimental results suggest that glued phases made with resin would tend to be stiffer than those made with wood glue, but the analysis of the correspondent values at the design level

---

[1]soft wood represented by an European conifer of 420 kgr/m^3, hard wood represented by a tropical hardwood of 780 kgr/m^3.

[2]PVAc following the German Institute of Normalization (DIN) norm 68602B2, Araldite AW 106 N(3-dimethylaminopropyl)1-3 propylenediamine.

of stresses shows an average deformation of about 0.1 mm, without a significant difference between the two types of glue.

For all practical purposes this deformation is little enough to produce a very stiff connection, though this fact has to be examined in more detail for every specific application.

Analysis of Variance for STRENGTH

Source of variation	Sum of squares	d.f.	Mean square	F-ratio	Sig. level
COVARIATES	.9626674	3	.3208891	.246	.8617
density	.0010824	1	.0010824	.001	.9780
diameter	.0043521	1	.0043521	.003	.9559
thickness	.7708084	1	.7708084	.592	.4717
MAIN EFFECTS	.9739144	2	.4869572	.374	.6994
woodtype	.0757670	1	.0757670	.058	.8179
gluetype	.7096751	1	.7096751	.545	.4891
2-INTERACTIONS	.2622303	1	.2622303	.201	.6702
GLUE-WOOD	.2622303	1	.2622303	.201	.6702
RESIDUAL	10.416414	8	1.3020517		
TOTAL (CORR.)	12.615226	14			

d.f.:degrees of freedom
F-ratio: Fisher's coefficient.

Table 4.1: Analysis of variance for the whole factors.

	Average	Std. Error internal	Std. Error pooled	95% Confidence Level for mean
WOOD TYPE				
h	6.20106	.407181	.4312857	5.20623-7.19588
s	5.80141	.300461	.4034309	4.87084-6.73199
GLUE TYPE				
r	6.21919	.401665	.4312857	5.22436-7.21401
w	5.78555	.302878	.4034309	4.85498-6.71612
INTERACTION WOOD-GLUE				
h r	6.34300	.732939	.5705374	5.02697-7.65903
h w	6.01180	.234167	.6587998	4.49218-7.53142
s r	6.05410	.176344	.6587998	4.53448-7.57372
s w	5.64980	.477621	.5103042	4.47271-6.82689
TOTAL	5.98791	.294624	.2946242	5.30832-6.66751

Table 4.2 : 95% confidence limits for the mean STRENGTHS after an analysis of variance.

4.2.2- Cyclic performance of glued bamboo wood phases :

A study was undertaken to observe the performance of glued phases under prescribed cyclic loadings.

A first experiment was set to determine whether the application of a certain number of cycles under the rate of loading specified in ISO 6891-1983 (E) would weaken the static bonding capacity.

Initially, it was supposed that a top figure of 20 cycles would be enough and satisfactory for the comparison, but actually as many as 90 cycles were applied to the specimens, with an amplitude of between 10 and 100% of the design load. Following this specimens were unloaded and final loading undertaken till failure.

Final stress values were then compared to those of a sample subjected only to steadily increasing load, where no significant difference between the two samples was found. Final deformation proved not to be affected either. A typical stress deformation plot from this experiment is shown in figure 4.3.

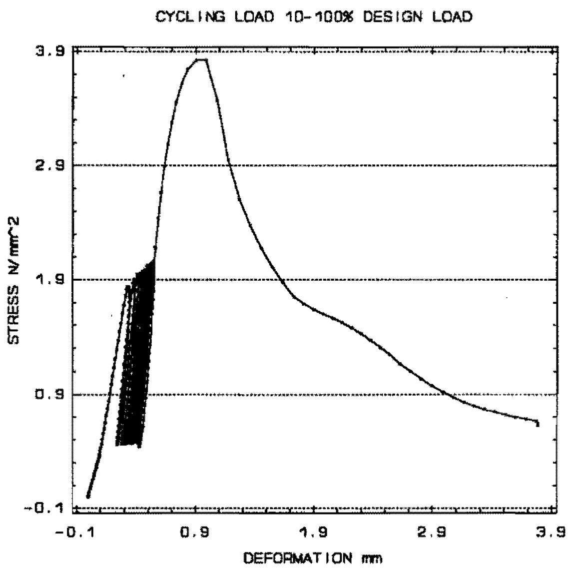

Figure 4.3:Cyclic load result.

The experiment was then repeated on another sample for a larger amplitude of the load, maintaining the same loading speed. The chosen range was between 10 and 80% of the expected ultimate stress level. Figure 4.4 shows that deterioration occurs at this level of stress and rigidity quickly deteriorates leading to the failure of the specimen.

An obvious question arises here. What is the level of stresses for which the behaviour changes from that reported in figure 4.3 and that of figure 4.4 ?

Because of time limitations, it was not possible to answer this question, but this matter is worthy of further research.

Another important aspect considered was load reversion. Structures under cyclic loadings frequently sustain some degree of reversion of the loading as well. This is especially true of light structures, because in those cases dead load is small compared to live and other types of load.

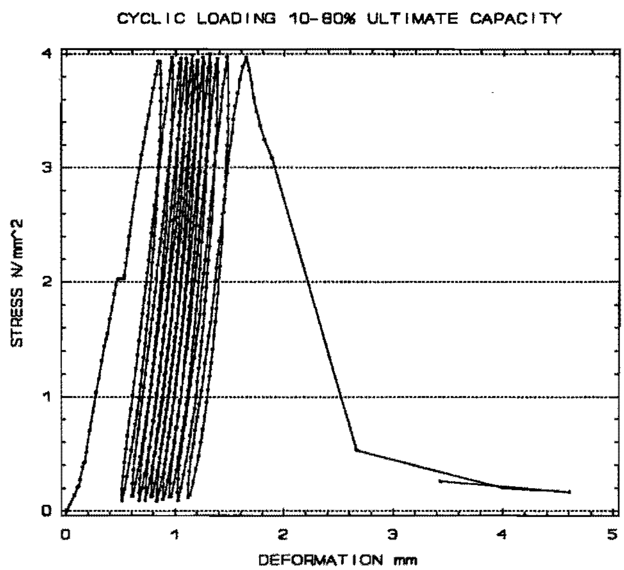

Figure 4.4: Cyclic load 10-80% of ultimate capacity.

This argument also applies to bamboo structures, and thus it is of interest to know what is the behaviour of glued phases under the action of reversing loadings.
An experiment was undertaken for such a purpose. Specimens were loaded with a signal of amplitude -100,+100% of the design load. An example of the results is shown in figure 4.5. After 20 cycles specimens were completely unloaded, and then reloaded till failure, and final stresses were again compared to those of the steadily increasing load specimens, resulting in no significant difference between the two sets of results. Final deformation was also the same, indicating no deterioration of the stiffness. It can be seen that the amplitude of the loops

remained very stable too, at this level of stresses. Even though no attempt was made to study the real hysteresis of the glued phases **evidence as shown in figure 4.4 suggests not much energy dissipation is available and therefore stress design levels should be kept in the elastic range. Structures in areas prone to seismic activity should be designed for the maximum earthquake response without any allowance for plastic deformation, that is, without considering any energy dissipation mechanism**, unless other dissipating points are included, of course. Because of the lightness of bamboo, peak loads are expected to be low in any case.

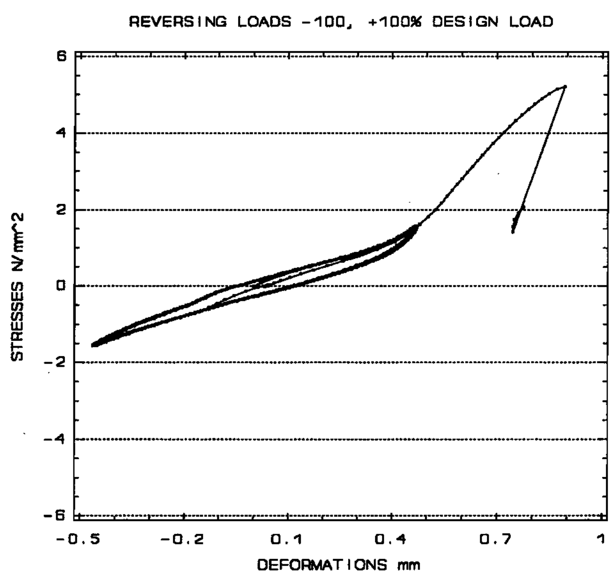

Figure 4.5: Completely reversing load.

It was considered important to observe the sensitivity of the above results to a change in the speed of the application of the load, because the speed recommended by ISO is very slow as compared to a real earthquake signal. In the case of the signal of figure 4.3, frequency is 0.008 hertz, while it is considered that the possible range of responses for bamboo structures are between 0.5 to 20 hertz, depending on a number of facts (mass distribution, stiffness, support and soil conditions, etc.). In order to observe the effect of this type of load on the glued phase, the signal of figure 4.6 was chosen. The period of the different trains was decided after examination of some earthquake registers (Arce,O.A., 1992). Different locations under different soil conditions, including extremes of stiff and very soft soil were looked into, so that the experimental input covered those extreme conditions. The basic idea was to imitate the initial, the intermediate and the final frequency trains that may typically occur in a real earthquake, for different combinations of amplitudes and frequencies. Because of limitations on the equipment, digitalization of the signals was not possible, so no real earthquakeinput was employed. The random character of peak loads that are typical of earthquake signals could not be replicated either because of the same limitations

It was also found that the equipment was only sensitive to frequencies of or below 0.5 hertz, after which there was no further control. Therefore it was not possible to go beyond this level. Further, it was not possible even at this frequency, to record deformations, because the data acquisition speed was found to be very slow for this signal, so the experiment was only checked with a continuous plot of force against time.

Again it was found that the final load of the specimens was not affected by the imposition of the dynamic signal. What is more, three specimens were subjected to more than fifty cycles of complete reversal between -100 and +100% of the design load at 0.5 hertz, with no apparent damage. Specimens were reloaded again up to failure, and the average result showed no difference with that of the reference sample.

Thoughthese results are only indicative, and more research is needed to be conclusive about the dynamic properties of glued bamboo wood phases, results so far are encouraging and can be taken as a confirmation of the degree of safety of the recommended design level.

The final choice of glue on a given design of course depends on a number of aspects. So far, evidence suggests that strength is not the most important issue, because, as explained before, failure occurs inside bamboo.

Three factors must be carefully examined though. First, durability, especially in relation to weathering, is a problem that requires close examination. In relation to this it has to be

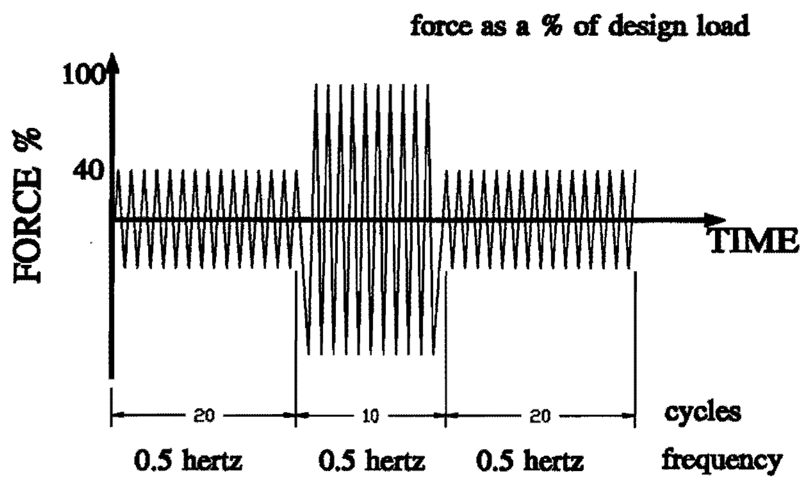

Figure 4.6:Design signal for high frequency tests.

remembered that bamboo requires preservation, and, as a complete definition of this problem is yet to be gained, preservants may react with glue, or may impede correct bonding. Second, glues are normally expensive, and sometimes unavailable in certain regions of the third world. So a cost effective choice is always a priority. Third, workability may be important, depending on the specific design application. It can be, for example, that a filling glue type is desirable for some reason. Among other solutions, a mixture of resin and portland cement[3] may turn out te be attractive, because in this case the resin volume is decreased. As a result there are lower costs with the advantages on the durability side. In addition, the need for

[3]Work done by Dr. Jurek Jassienko, a staff member of the University of Technology of Wroclaw, Poland, during a sabbatical in the University of Technology of Eindhoven, showed the potentials of this application in the case of wood restoration work.

pressure during the setting period is eliminated, and surface irregularities or are compensated for.

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Chapter 5: Design of bamboo connections

5.1.-Introduction

In the course of time a number of joints for bamboo construction have been developed, and some of the solutions in use can be traced back thousands of years. As in the case of wood initial trials consisted of two or more elements tied together with rope or string. Initially, organic fibres like bamboo itself were used, and more recently steel wires. Warner (?) describes some of the lashing type joints normally found in the field, and some of them are depicted in figure 5.1.

The lashing technique is popular among users of bamboo. The Chinese people are able to join together elements of bamboo to build structures as high as forty storeys, as in scaffolding (Anonymous, 1979). In Thailand this system is still common practice in the construction industry. Plastic bands are employed there to connect the culms, and a small piece of wood, bamboo or a metallic pipe is utilized to twist and fasten the lashing.

In more formal construction though, lashing does not produce enough stiffness in the connection and better options are needed.

An alternative has been to clamp the culms between pieces of wood or plywood connected with bolts, as shown in figure 5.2. Rigidity can largely be improved making use of the in plane moment of inertia of the pieces of plywood or the like. There is no need for much precision because the presence of plates relaxes this requisite, allowing for a certain amount of flexibility in the dimensions of the culms.

A different proposal is described by Sonti (1959) in which bamboo elements are lashed to a steel plate. The author reports that a full-scale dome was constructed in India using this joint with good results. The advantage of the design is that advantage is taken of the rigidity of the plate, but that damage to the culms is avoided. No report was made on the rigidity of the joint itself, but it is very likely that this characteristic largely depends on the quality of the lashing.

A detailed description of common lashing and lashing-like joints is given by Janssen in his Ph.D. dissertation (Janssen,1981), followed by an analysis of the way the examined joints react to the imposition of loads. The author carried out a set of tests on full-scale joints to

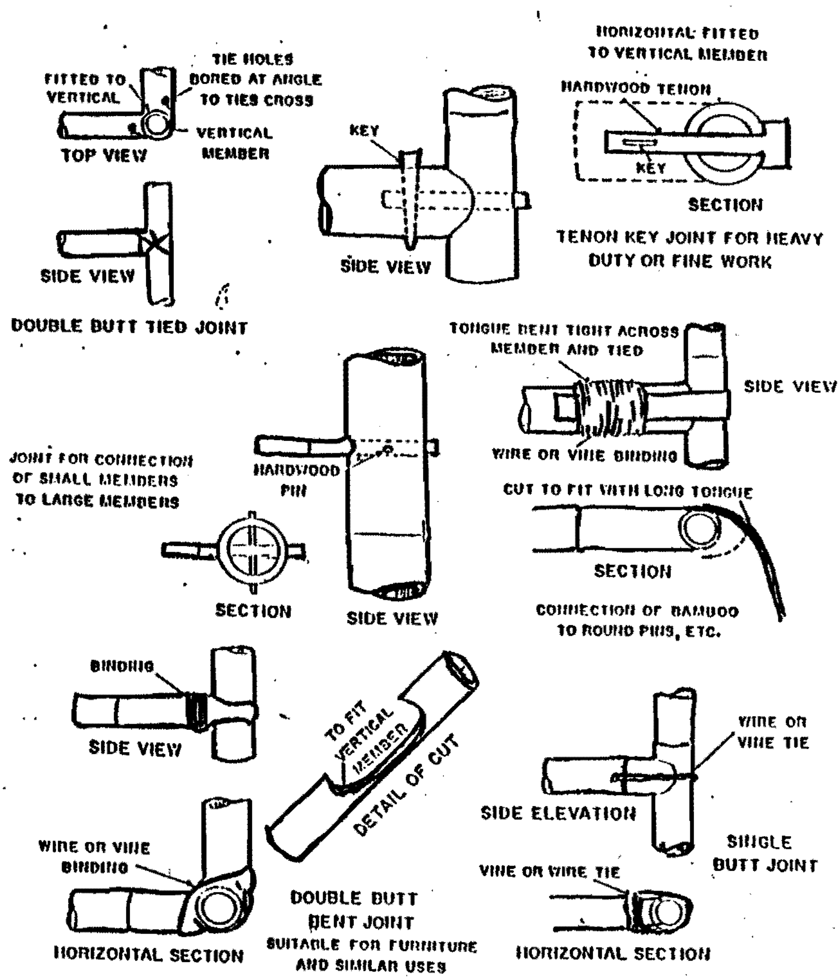

Figure 5.1:Some joints as reported by Warner, reference 13.

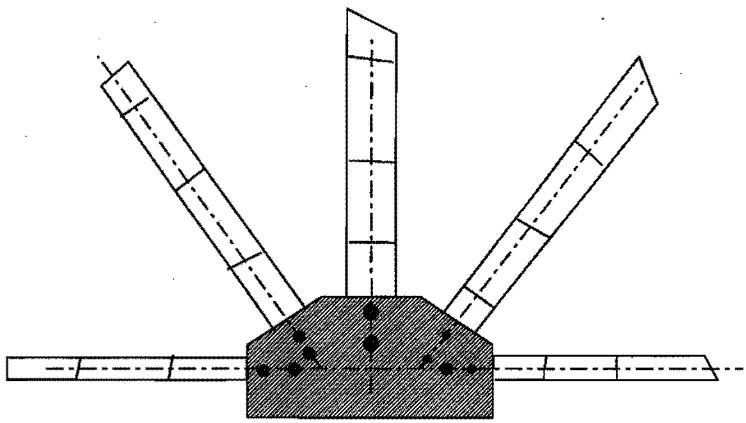

Figure 5.2:Plated joint.

simulate the under-loading behaviour of a king-post type truss.

The joint in figure 5.3 was reported by DANIDA[1] (Anonymous, 1985).

It is claimed to be especially serviceable in the case of axial forces, for which a set of design rules are described in the said document. The basic concept is to use external ties to take tension forces transmitted by pins through bamboo. Splitting of the culms is avoided by binding the zones of stress concentration with wires, though the authors report that during the test of full-scale trusses this type of reinforcement **did not prove to be effective**. Similar conclusions in this respect are reported by Janssen (opus cit), who compared the bending behaviour of bamboo culms with that of a sample that was reinforced by a wire wound around the elements.

The use of precast steel or aluminium fittings was suggested by Duff (1941). A detail is

[1]The Danish agency for development.

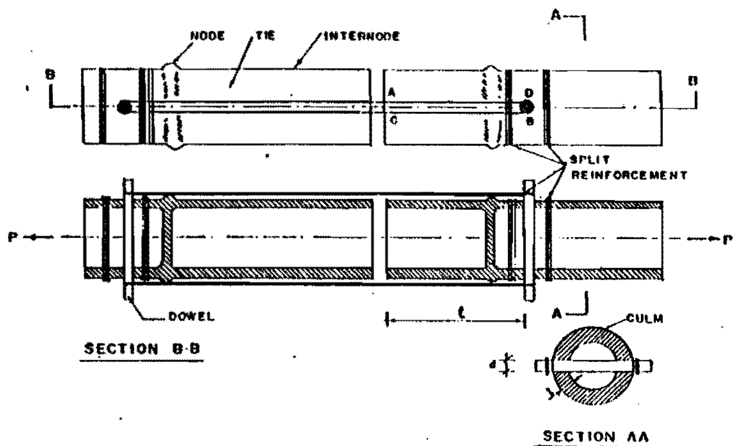

Figure 5.3:Connecting system proposed by DANIDA, from reference 2.

shown in figure 5.4.

More recently, Spoer (1982) tested some space structures in bamboo where the joint of figure 5.5 was employed. The author reports that the connection performed very well during the tests.

A system has been developed by the Costa Rican Bamboo National Project (see reference 2, chapter 3 for more information on this project) in which a steel bar runs inside the culm. The bar is welded at the extremes to steel plates. The extreme internodes are filled with mortar to prevent insects from entering and to keep the bar in position. The steel bar has the purpose of carrying tensile forces, whereas the culm takes on compressive forces. The element-to-element connection is achieved by welding or by machining the ends of the steel bar, so as to be able to screw it to some type of receptacle.

In the present chapter the description of a systematic approach to the design of a feasible

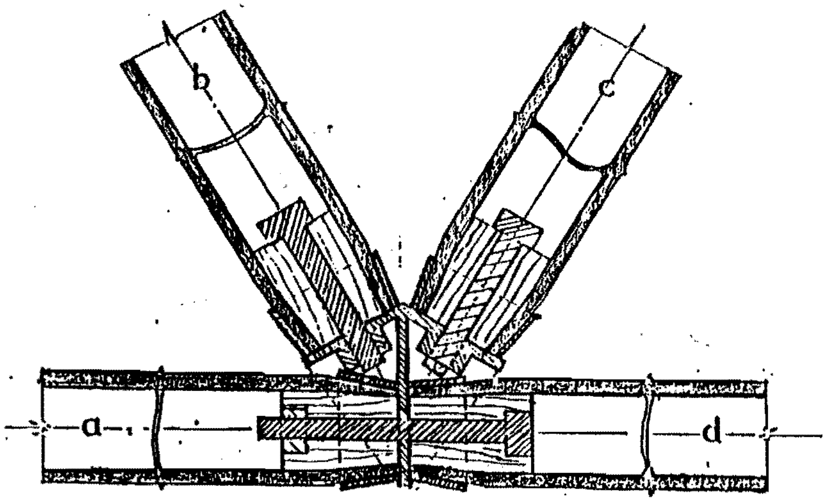

Figure 5.4:Steel fitting proposed by Duff, from reference 3.

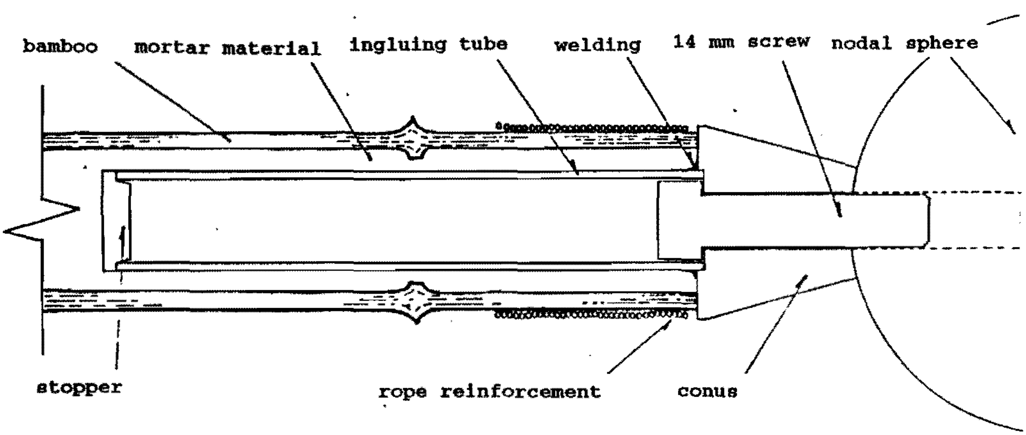

Figure 5.5: Proposal from Spoer, reference 12.

connection system for bamboo elements is undertaken. The experience summarized in this introduction is employed as background information in the process.

5.2-Design Methodology

A systematic design approach is now given according to the description in figure 5.6. Basically, the design problem is first defined by means of certain objectives to be accomplished by the resulting object, system or technique. In this case structural performance is the chosen semantic level, since the object to be produced is fundamentally a structural object. Next, constraints of the design are clearly stated. Basically, these first two elements should completely define the design problem, to which the resulting design should be a plausible solution. An instrument for the evaluation of the resulting design has to be developed, so that decisions can be made over a number of options. The actual application of the approach is described below.

5.3-Objective function

" To achieve structural continuity between elements "

In modern construction technology, the concept of connection involves the search for mechanisms such that deformations can be kept largely under control, and that predictions are also possible. Structural continuity also conveys the idea of force transmission according to a certain prescribed and desirable manner.

Perhaps the main role of the connection is indeed to safely transmit loads in a prescribed manner from element to element, and finally to the ground.

5.4- Constraints.

5.4.1-Internal Constraints.

5.4.1.1-Material properties.

In order to define material constraints all relevant features of the material are described and a synthesis is made, based on the descriptions made in previous sections of the thesis, and on other sources where necessary.

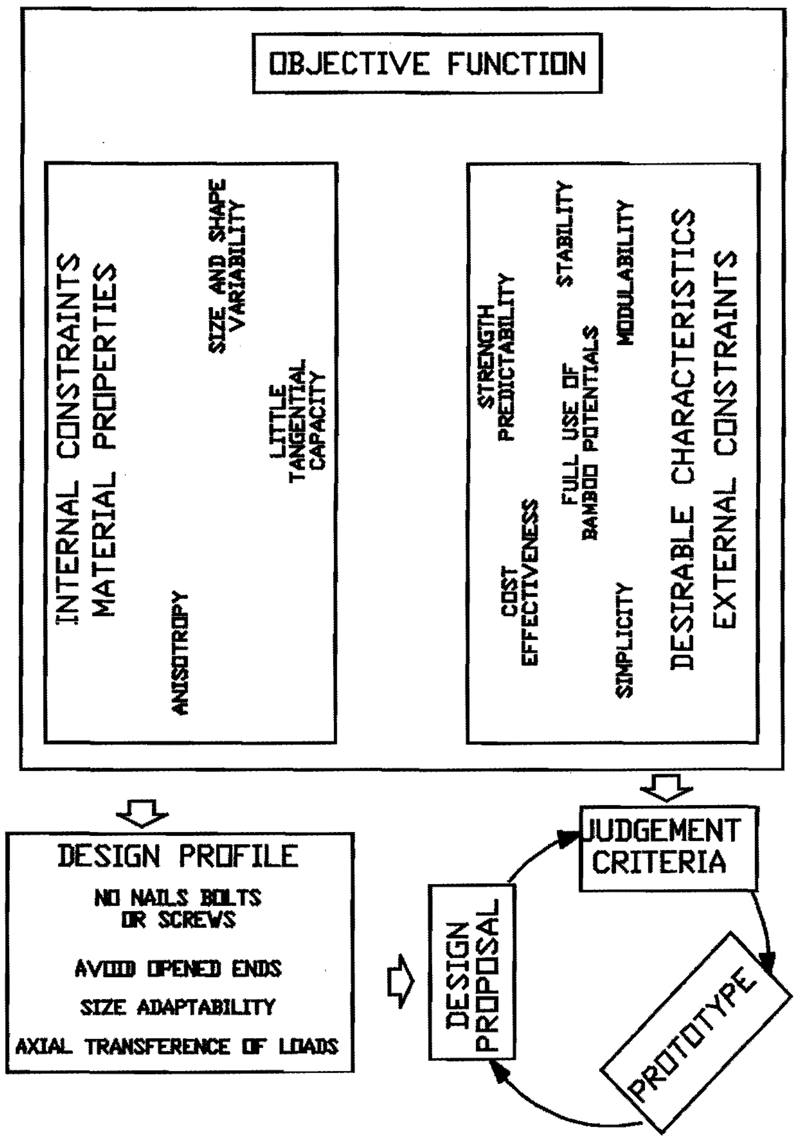

Figure 5.6:Design approach.

Bamboo is an anisotropic material which shows its best mechanical properties in the direction parallel to the grain, coincident with the longitudinal axis within very small margins of variation. Loaded in tension fibres face stresses with comparatively little shear disruption of the weak lignin in between them.

Under compression, lignin does play a significant role and failure occurs because of the presence of tangential expansive forces in the specimen leading to critical tangential strains.

This mixture of high-strength fibres running almost perfectly parallel (but certainly without crossing), and a weak lignin matrix in between, produces a material with little tensile and shear capacity in the direction parallel to the grain. **Bamboo is a unidirectionally reinforced composite with comparatively little tangential capacity.**

These facts are well-known to people in the villages and they actually use weaknesses of bamboo to their own advantage. They split bamboo longitudinally by wedging a knife in between the fibres, with very little effort, to produce the raw material they need for baskets and furniture.

5.4.1.2-Shape

Some of the most promising species are hollow and nearly round. This means there are a number of problems for the design of connections. Besides that, bamboo canes are tapered and thickness variation is important.

Forces generate high tangential stresses due to the round and hollow shape. As mentioned above, the absence of tangential or radial fibres creates a set of natural failure planes along the longitudinal direction and this stops bamboo from taking advantage of the dome effect, that could have been expected as a response to forces perpendicular to the culm. Open ends are then very easily crushed and this is no doubt the reason why common practice recommends the location of transversal loads on the nodes, to take advantage of their diaphragm effect. As node distribution along the cane is rather variable (Espiloy,1985), this is not always possible and in any case it works out to be restrictive and rigid from a constructional point of view.

Another inconvenience associated with the round shape of bamboo is the difficulty of aligning elements to produce a neat joint. This has led to element overlapping, but this produces a very massive joint, creates serious problems to the modulation and prefabrication of elements and what is equally important, prevents an easy solution for the problem of interaction between bamboo frames and other elements of the building. Besides that, overlapping leads to the creation of eccentricities, increasing bending moments and secondary effects in the construction.

But the main drawback of bamboo's shape need not be the stresses involved, but the difficulties that arise on the geometric side of the connection. Since sizes occur in a rather random fashion, each connection presents a specific case and produces specific cutting problems. So the connection of elements is a very labour intensive task, that requires highly qualified workers. The amount of skill needed is such that the furniture industry workers are real artists, and their wages follow suit. In Costa Rica, for example, this phenomenon is such that bamboo furniture is a luxurious product, in spite of the low cost of the material, far beyond the buying capacity of the low income levels of general society (Manger,1991).

Thus bamboo has a high variability in size, thickness and shape. Hollowness creates little crushing strength.

5.4.2-External constraints.

External constraints are set up as a list of desirable characteristics to be fulfilled by the design proposal.

5.4.2.1-Maximization of the use of bamboo.

Like any other material bamboo has a combination of good and bad material properties. Joints should take full advantage of the material and should support the use of bamboo to its full structural potential.

5.4.2.2-Simplicity:

Bamboo construction is fundamentally aimed at the solution of infrastructure problems in areas where sophisticated equipment and technical capacity are not readily available or are expensive. Therefore, good design must be simple in terms of the amount of skill and equipment involved in its production. These conditions are very important because solutions to building problems in developing countries normally call for the participation of untrained volunteers or users in self-construction projects.

Field tests would be necessary to fully understand and qualify the grade of simplicity of a design. Factors like those explained by the Theory of Learning and Learning Curves can then be quantified and accounted for, and, in the same way, the interaction between component and total costs can be better understood.

5.4.2.3- Stability:

Joints should be stable in relation to time. Design must take durability into account, in terms compatible with the expected life of the entire structure.

5.4.2.4- Adaptability of dimensions to a modular system:

So far the need for modulation on the basis of individual construction appears to be an unnecessary sophistification. But it has to be realised that without this important tool it is very difficult to achieve the goal of coping with the increasing problem of housing in the third world, which urgently needs the introduction of modern management to speed up construction, to improve quality and to lower costs. Besides, modulation has potentials to initiate systematic approaches to production problems, which may help users to improve their own solutions, may enable the participation of unskilled workmanship, may concentrate technical problems in areas of easy control like specialized shops, and may also allow the formation of production units. On top of this, modulation is a principle in its own right, with a wide range of applications in production systems, worthy of being tried out and learned with benefits beyond the particular solution of a building problem.

5.4.2.5- Strength predictability:

From an engineering point of view what is perhaps the greatest problem found in the literature on joints, is the absence of design rules to allow extrapolation, strength prediction, and safety estimation. Not even rules of thumb seem to be available. Plausible mechanical models and/or sound experimental data should be the natural companions of proposals for joints.

5.4.2.6-Cost effectiveness:

Taking the interests of bamboo users into account, any submission or proposition must fulfil requirements of cost-effectiveness.

In themselves joints are one of the highest components of structural costs in a building. But it is common knowledge that joints affect the overall structural costs as well, because they change the amount of structural material needed, construction time, labour needs, architectonic design possibilities, and so forth. So, the real impact of joints on the total cost is not a simple matter to determine, and it certainly would be a mistake to simply compare joint to joint by the cost of, for example, only the materials involved.

5.5-Design Profile

The analysis of internal constraints indicates certain elements that should be present in a 'Design Profile'. This can be summarized as a set of specifications for the designer and in this specific case works out to be :

-Avoid penetration by nails, screws or bolts.

-Avoid open ends.

-Solve the problems of size adaptability.

-Transfer forces by axially distributing them to the fibre system of the culm.

5.6.-Joint evaluation

In order to compare different design options or joints already available, it is necessary to produce an instrument, so that the decision on the quality of the different possibilities can be made in the most objective way.

At the end of this chapter an assessment is made of the design proposal, by summarizing its most important features, and comparing them with the objectives and constraints of the objective function.

5.7-Proposal for a Connecting System for Bamboo Elements

As indicated before, the use of pins, nails or screws entails some drawbacks and disadvantages for the purpose of making joints for bamboo structures.

The above argument explains why it has been of common practice to use natural or artificial materials to rope the elements together. No damage is done to the culm in this way, but some restrictions to the transfer of loadings become apparent.

Some of the natural fibres used for this purpose are not strong enough to generate the necessary tensile force, and besides that, there is very little friction with bamboo surfaces, which are rarely sand papered or cleaned. Another source of softening in this type of joint is the use of green bamboo, which shrinks and slips out of the bond made by the rope-like materials.

Improvements can be made by using devices like the Delft wire-lacing tool (Huybers, 1988-1,2, 1989, 1990). Basically, this tool winds steel wires around round pieces of wood, and has been suggested for the making of bamboo connections as well.

Some trials at the University of Technology of Eindhoven showed though that the amount of tying is not sufficient to avoid slipping of the culms in the case of bamboo, and sometimes the wires cause early cracking by crushing of the ends. The major problem would be the calculation of the safety level of such connections, especially because uniformity in quality is difficult to achieve.

The use of steel or aluminium fittings has given good results according to several authors (Duff, Spoer, opus cit). Indeed, it is possible to provide the right shape to avoid all, or most, of the problems mentioned problems above. However, those fittings are usually costly and unfeasible in the types of applications bamboo is aimed at.

A feature or characteristic is common among proposals like the ones made by Duff and Spoer. The problem of a low crushing capacity of open ends is eliminated, because the inner chamber is filled, in one case by wood and in the other by mortar. Both solutions required some external reinforcement, Duff's proposal in the way of a complex steel fitting, and

Spoer's by using a tightened wire that is wound around the culm until it is completely covered in spite of the author's claim of its dubious effectiveness. So, in both solutions an attempt is made to take care of the problem of the low tangential capacity of bamboo.

Experience shows that taking forces out from inside the culm solves a number of problems. For example, the alignment of elements, distribution of crushing stresses, etc. So far, the problem remains how to transmit forces to other elements without using expensive fittings, on the one hand, and how to protect bamboo in the tangential direction on the other.

Filling the culm seems to be a first logical step. To transmit forces from the culm to the filling, and at the same time to reinforce bamboo to sustain tangential strains, some sort of adhesive material is needed. Better still, a piece of wood can be used and glue can be employed to stick it to the inner surface of bamboo. As shown in chapter four and appendix C, any normal glue provides a capacity far larger than that of bamboo in the tangential direction. The piece of wood can be regarded as a wood fitting. This component of the connection can be extended outside the culm to meet the outcoming piece of wood from other elements, and the design of an appropriate way of joining them together can be achieved, as it is normally done in wood construction. In more demanding structural conditions, some other type of solution can be developed.

Figure 5.7 shows a proposal. The main principle behind it is, as explained before, that the system takes forces out from inside the culm, distributing contact stresses over as much area as needed. In order to achieve this a cylindrical piece of wood is glued to the culm internally. The first gain comes from the fact that the presence of the piece of wood changes the internal distribution of shear stresses, because of the enlargement of the net second moment of area in the region of the connection. Obviously, the cross-section is also enlarged. This means that bending stresses are diminished too, compared to those in the vicinity of the connection, where bamboo acts alone. Normally the thickness of bamboo is small compared to the diameter of the full combined section. This then produces a low demand on bonding stresses, as shown in appendix D, where an analysis of stresses is presented for the derivation of rules for calculation.

Two slots are needed to control cracking during the insertion of the wood cylinder into the culm, as shown in the figure, because, as said before, bamboo is rarely perfectly round. Besides this fact, the inner surface must be carefully cleaned prior to gluing. The preparation of the inner surface can be done using a hand-held drilling machine with sand paper attached to the drill. Up to 5 mm can be easily removed this way, making it possible to use a fixed diameter size of the wood cylinder in a variety of bamboo sizes.

It has been observed in the laboratory that up to 10 mm of difference between diameters can be dealt with normally by this means.

The way in which forces are transmitted from one structural member to another very much depends on the type of structure and the predominance of one force type upon another. Thus, a discussion about the actual connection of elements is presented in chapter 6, once some criteria for the structural possibilities of bamboo have been gathered.

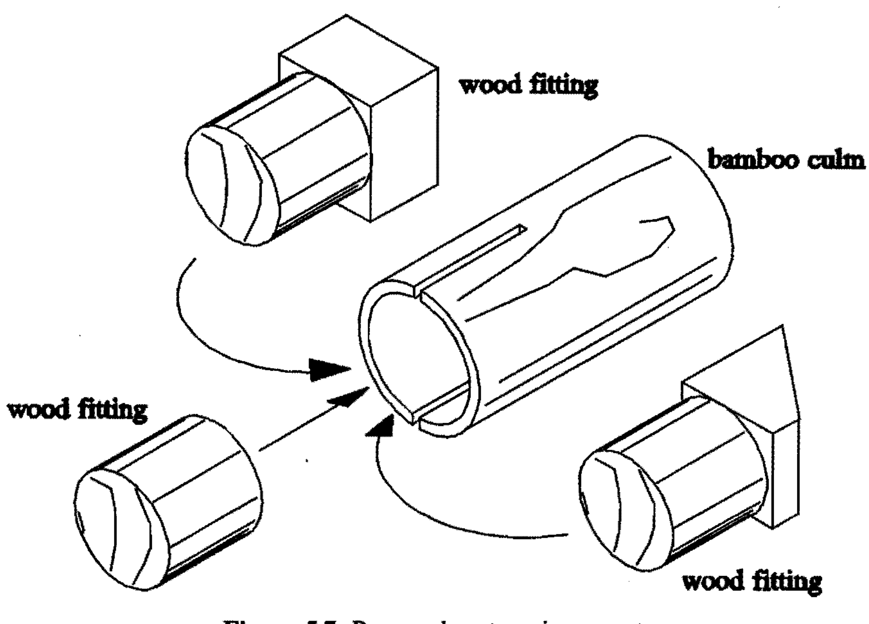

Figure 5.7: Proposed system, inner part.

5.8-Evaluation

5.8.1-Objectives

As shown in appendix D, it is possible to predict the level of deformations in the connecting system, and it is also possible to calculate material and geometric requirements. On the other hand, the study of glued phases in chapter 4 and appendix C shows that structural continuity is also achieved with the system.

5.8.2-Design profile

All the specifications of the design profile are met by the proposed system. Though size adaptability is not completely solved, some degree of achievement can be claimed in this respect as well. However, no complete evaluation of this matter can be made until some classification system is put in place, because otherwise the designer would always face an unlimited number of sizes and shapes.

5.8.3-External constraints

The making of the wood cylinders may present some problems in the field, but they can be made in small workshops to the specifications of the designer. This is economically important also. The construction industry generates a significant contribution to the economies of developed countries, mainly because of the amount of economic activities that it is able to generate. In the same way, if bamboo construction proves to be technically feasible, it should significantly contribute to the development of the economies of developing countries, by creating business and job opportunities, as well as forward and backward economic linkages. Once the wood fittings are shop tailored and made, the construction process in the field is greatly **simplified**. There is no need for skilled labour. Bamboo elements need but little preparation, sand-papering and cleaning of the interior surface in the region of the connection. Cutting requires no precision at all, since differences can be taken on by the connection itself. Angle cutting on bamboo is completely eliminated, since this can be done at the extreme end of the piece of wood.

The cost of wood can also be significantly reduced, since pieces of the required diameters are often the result of plantation thinning in tropical forests. The amount of glue involved is very small and good quality can be achieved with normally available types. So, though this can only be evaluated under real site conditions, it is believed that the **cost** of the proposal is low and makes it feasible from an economic point of view.

The system **makes the best use of bamboo** by taking advantage of its good properties, but also helps to protect and improve conditions in the region of the connection, not only from the point of view of stresses, but also from that of protection. No open ends means no place for insects to hide, neither places of difficult maintenance.

The system allows for the design of **modular systems**, concentrating the problem of dimensions in the connection itself, so bamboo becomes sort of neutral in the system, which is a real relief, since, because of the wide range of shapes and sizes, putting bamboo elements together would otherwise require very skilled workmanship.

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Chapter 6: Design of bamboo structures

6.1-Introduction

In the previous chapters bamboo was dealt with from the point of view of the material (chapters 2, 3 and 4 and appendixes A, B and C), and from the point of view of the properties of the culms (chapter 5 and appendixes D, E, F, anf G).

In this chapter some ideas related to the design of bamboo structures are based upon the basic concepts described there. The approach in this chapter is basically descriptive, remarks are made based on the physical relation of variables, that has been introduced in previous sections of the thesis, on the observations of different phenomena in the laboratory, and on some limited field observations. It is acknowledged, though, that probably the most important source of input to the development of structural design, namely, engineering experience, is very limited as a source in this case, due to circumstances which will be explained in the following. As an eventual 'fully engineering' material, bamboo is still at the very beginning of its development. Moreover, the crude application of 'common' engineering knowledge may be misleading, because of substantial differences between bamboo and other more normally used (and more normalized) materials.

Though basic mechanical properties have been dealt with by many authors and were also dealt with during this research project, field work should still be undertaken from the point of view of the practical applications of the findings. In wood, for example, and to talk about a material that may resemble bamboo, properties are related to species, broadly speaking, but in the case of bamboo it looks as if things are not so straightforward, because of the very random nature of the material. Basic valuable design data for tropical wood is still in the process of being developed (see Nakai, 1992, only to mention one example of this effort), so it would be too much of a simplification to believe that the bamboo data base is complete. The way bamboo should be classified still has to be developed, and only then will it be possible to determine, in a cyclic process, design levels for the different parameters, and the relation of these design levels to species, location, and so forth.

So far, though, one interesting possibility pointed out in this thesis (chapter 2) is that it may be possible to control the maximum tangential strain in the culms as a way of fixing desirable levels of safety. For that purpose all one needs to do is to check the maximum tangential capacity for a certain batch or species, and to determine the elasticity modulus in an appropriate and reliable way. This can not be done without serious statistical studies and careful laboratory investigations, but in this way the work of the designer can be greatly simplificated and grow in meaning.

In the following lines, therefore, consequences of the properties of the material and of the culms are connected to design possibilities from several points of view, covering matters like loads, shapes, connections and others that may have some relevance in the determination of the design procedure and the final product: the bamboo structure.

There might be some discussion upon the fact that perhaps practice should lead the way in the development of bamboo use in construction, as some maintain is the case in engineering. That criticism may be plainly justified, especially because in general terms a theory in engineering is not (really) engineering till it is put to work, that is, until there is some concrete engineering evidence of it, otherwise it just remains a theory.

In the case of the general methodology of this thesis, theory precedes practice to some extent (some people will not be surprised about this in any case, they probably even applaud the approach, especially many in favour of theory first and practice later). But practice in bamboo construction[1] is practically nonexistent, so there was no choice but to try to simulate a broad reality by modelling it, accepting that models may be incomplete, but that at least they may give some idea of the extreme situations that may arise from extreme values of the parameters involved. The design consequences that are referred to in the next sections contain this methodological bias, so the reader should keep this in mind.

6.2-Load systems

Loads can be classified in many ways, and it is not the intention of the author to propose criteria in that sense, or to extensively discuss ideas currently available on this matter. It is important to acknowledge, though, that there must be a connection between the properties of

[1]Meaning 'engineered'construction.

the structural elements that are to be used in a construction, and the type of loads that those particular elements may carry in an efficient way.

Perhaps this point is not so obvious in the case of shapeable materials (those that are susceptible to forming or shaping, the final shape of which is not an intrinsic property), because those materials are precisely shaped to meet certain structural and architectonic requirements, to mention just two demanding conditions.

From the perspective of the initial and basic scope of this research project, bamboo is considered unshapeable[2], that is to say, the accepted objective is to use the culms as they are produced by nature, as tubular tapered elements. This shape and the mechanical properties of the material should in the first place determine the type of loads that a particular structure may carry.

For simplicity's sake forces can be grouped according to the effect that they produce in the element, such as axial forces, transversal forces, bending moments, and twisting moments. Axial forces can produce compression or tension states in the element.

The final effect on the element depends on a number of conditions, like the shape of the structure, the position and direction of the forces, the conditions at the supports, initial imperfections, creep, time dependency of the loads, etc.

According to the arguments in chapters 2 and 3, bamboo is an elastic brittle material, with a relatively low modulus of elasticity[3]. Though the culms have nodes that sometimes are thought of as strong links, it has been shown that actually nodes are the weakest component of the culms, and the points where attention should be mostly concentrated. As a general rule, the more nodes a culm has, the less effective it is carrying loads, both from the point of view of strength and of deformation.

Mostly, it is therefore desirable that culms carry axial loads only, or at least that transversal loads and bending moments can be kept to a minimum. Bending moments will be present anyway, because of initial imperfections, even in visually straight culms, and because of some

[2]Though, in a broader sense, this is not true of course.

[3]That is, the overall phenomena in the full culm, as explained in appendix F.

time dependency of the shape of the culms, namely creep, that has been observed as an important factor. Though Janssen (1981) has shown that creep in bamboo is less than in wood, evidence suggests that the initial conditions of storage in the process of drying, play an important role in this matter, because culms supported at two extremes during this period, usually show important sagging effects. Moreover, bending and shear is introduced by the connections, which can never behave as perfect hinges.

As said before, nodes also generate bending moments, normally accompanied by twisting. In the experiment of appendix E, though, it was found that the angle of twisting was not considerable (but culms were selected on the criteria of straightness). In terms of twisting, in any case, the major obstacle remains the fact that research in this area is practically non-existing. This is of course limiting for the design and calculation of space trusses, especially those where, because of asymmetric loading or shapes, torsion may be important.

Though this research project did not cover the study of shear in an explicit manner, we explained in chapters 2 and 3 how tangential strains could be the single most critical parameter in the definition of the type of failure in bamboo culms. Transversal shears can be associated to tangential strains (perpendicular to the fibres) , of course, and therefore they should be avoided. It was pointed out by Janssen (opus cit), that shear calculated according to bending theory related poorly to the measured capacity of a sample of culms tested in bending, the measured shear capacity being about five times as large. Oddly enough, the figure found in bending calculations is much closer to the maximum capacity that has been found in this research project for tension perpendicular to the grain (chapter 2), confirming how critical this can be. From the point of view of its composition, bamboo is not naturally reinforced for shear, because, compared to a reinforced concrete beam, 'stirrups' are located on the longitudinal instead of the transversal direction in a bamboo culm.

So it is important to consider shapes that produce axial loads in the elements. In addition, account has to be given to bending moments due to initial eccentricities and those generated at the connections.

The discussion on whether to prefer compressive or tensile forces is a bit more difficult. The design value for tension and compression works out to be quite similar, but compression capacity is rather irrelevant from a structural point of view, because the culms are usually slender. Therefore, elastic instability may play a very important role, bringing down the maximum admissible stresses, depending on a number of parameters as indicated in appendix E.

A comparison of the values given by eq.E.(74) and the maximum compressive capacity, typically found for bamboo, indicates that the critical value of average slenderness at which the compression changes to buckling is $\lambda_{av} = 50$. It means that for the more common species the maximum length for a double hinged strut should be kept under 250 mm, because under this condition compression would govern the maximum capacity of the element.

So it is clear that elements under compression loads are bound to be much shorter (or less slender, in any case) than those under tension loads, and that the overall shape of the structure should reflect this fact.

One possibility that can be examined in relation to the need for augmentation of the second moment of area concerning elements under compression, would be the use of arrays of two or more culms.

The left-hand side of figure 6.1 shows an array of four culms of 80 mm in external diameter. This is perhaps not the most feasible shape for constructional purposes, but it is proposed here only for the sake of the current argumentation, though it will be shown in the next section that some constructional possibilities do exist.

When the second moment of area of the left side array is compared to that of a single culm of approximately the same size, as shown on the right hand side of the figure, we find that they are very similar to each other. On the other hand, it is always much simpler to build using single elements, especially if we consider the complexities arising in the connections. So, bamboo structures should tend to be formed by using single elements instead of groups of smaller ones.

It can be concluded, therefore, that **bamboo structures deliver their best structural performance under axial loads.**

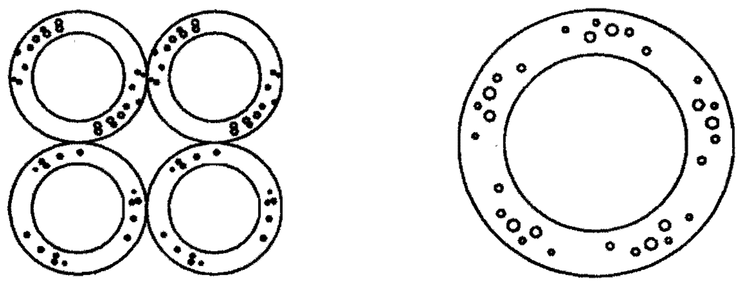

Figure 6.1: Four culm and one single culm arrays

6.2-Structural shapes and connections

In terms of the very limited recent experience, different approaches to the use of bamboo have been on trial, mostly at the laboratory level, but also in actual construction, particularly in housing projects. There is on the one hand a sort of a traditional approach, where bamboo is used for walling and partitions, and where some level of transformation is given to the culms. Woven strips, culm frames filled with these woven strips, and sometimes plastered faces, are examples of these types of applications. Part of this approach is the use of the culms to produce trusses and frame trusses as structural shapes, where the elements are connected to each other using natural fibres, crossing the elements over each other. This is the solution one typically comes across in traditional construction in South-East Asia.

More recently the literature reports the studies of Hidalgo (1981), Spoer (1982), Ghavamy (1988), to mention just the best documented cases, where connections follow the shapes used in the 'West' in steel and wood construction. Structural shapes also resemble the ones used in wood and steel.

Perhaps the most complete collection of design ideas can be found in " Bamboos" (Dunkelberg, 1985), a book published by the Institute for Lightweight Structures, University of Stuttgart, under the direction of Frei Otto.

The observations of these authors, and the examples of appendix F, seem to suggest that one of the most promising applications may be the building of trusses, frame trusses and spatial structures. Bamboo frames are definitely not a good option, leading to very flexible and weak structures, with very low degrees of redundancy, as is discussed in the last section of appendix F.

In accordance with the results of chapter three and the discussions of appendix E, structural shapes should be such that the length of compression elements is kept short to avoid buckling, and the connections should be as stiff as possible for the same reason.

In this sense the choice of shapes has been limited in the past by the type of connections available. Spöer (opus cit) tried the use of steel fittings bolted to a spherical receptacle, that allows the building of lamellae and grid-like structures. Ghavamy (opus cit) reports works in the same direction, with positive results.

The use of steel or metal to transfer loads from element to element seems to be a promising approach. As discussed in appendix F, wood-to-wood connections add a great deal of flexibility to bamboo structures, because the second moment of area that is available, due to the size of the incoming elements, is very low, though the effective elasticity modulus of wood is normally much higher than that of bamboo culms.

So, before going ahead with the discussion on feasible structural shapes, it is necessary to develop some ideas about the possible joining systems, departing from the fact that the basic problem of taking forces from the culm can be solved according to the proposal made in chapter 5.

A simple way of dealing with the problem is depicted in figure 6.2.

The steel plate C is introduced in the slot of the wood cylinder and glued to it with a mixture of epoxy resin and Portland cement. The plate is projected, so that its outer extreme can be adapted for different applications, as indicated in the details D and E in the figure.

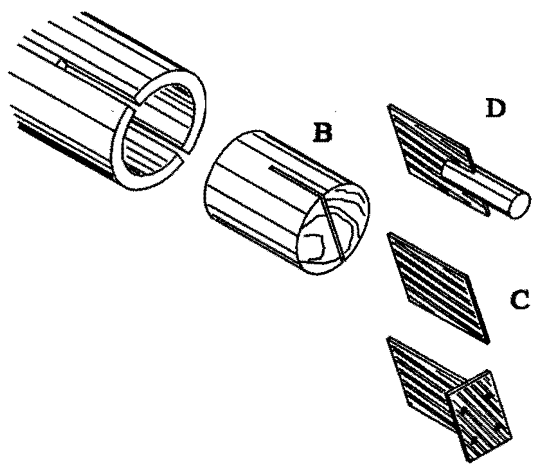

Figure 6.2: Connecting system.

In plane trusses, the plates from two or more incoming elements can be pre-welded to each other and then the rest of the connection can be assembled.

The specific way by which plates or steel tips are joined together is left to the creativety of the designer, but an idea is depicted in figure 6.3 just to show the kinds of possibilities that can be made available.

Figure 6.3 shows a connection in which a small box is made of steel plates, so that the faces are perpendicular to the axis of the incoming elements. The steel tips are then welded directly to those surfaces. Welding is thought of here because it is cheaper than machining of the tips, but in some cases this can be achieved as well.

Figure 6.4 shows a different solution, where a piece of steel pipe is used as a central element so that incoming plates, in this case, can be welded to the required angles. It can be seen that this type of generic solution gives great flexibility to the designer because there are few restrictions from the perspective of the structural shapes that it may support. The amount of

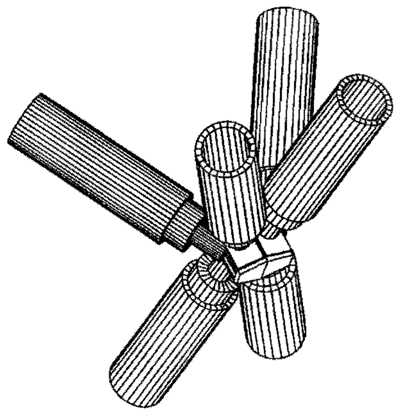

Figure 6.3: Connection for space truss, with centre steel box element.

steel that is needed is very small and the fabrication of the components is simple compared to precast steel fittings. A rough calculation of the thickness of the plate shows that a thickness of only 2.0 mm is needed to match strength and stiffness needs in the connection, for the expected capacity of bamboo culms between 70 and 120 mm diameters. Experimental evidence on the use of the mixture of resin and Portland cement at the laboratory of the Department of Structural Design of the University of Technology of Eindhoven[4] very much supports this idea both from the point of view of strength and stiffness. Though the cost has to be evaluated in a field situation, the amount of resin involved suggests that this would not be a major problem.

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[4]From the work of Dr. Jerzy Jasienko, invited researcher, staff member of the University of Technology of Wroclaw, Poland, who has studied this possibility for the reinforcement of wood beams with embedded steel bars.

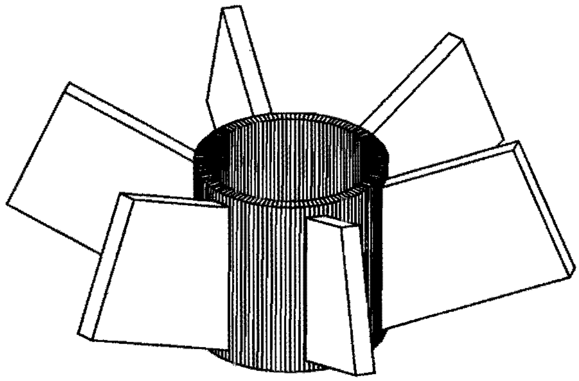

Figure 6.4:Connection made with a piece of steel pipe and plates.

Several problems are dealt with by means of this proposal.

In the first place, all the advantages described in chapter five are kept intact, both those related to the properties of the material and those related to dimensioning, detailing, and building.

In the second place, the proposal is more of a concept than a definitive rigid fitting or system, because it allows the designer to have choices in relation to the final specific shape of the connection. In that way it allows the interconnection of bamboo culms, but also their connection to elements of other materials, like a concrete beam or a foundation system, as shown in figure 6.5.

In this way the structural shape is not restrained or defined by the connecting system. So, what should the shape of bamboo structures be then?

It has been shown how the use of the glued wood cylinder plug proposed in chapter five opens possibilities for the designer, as have been shown in the previous paragraphs. This fact

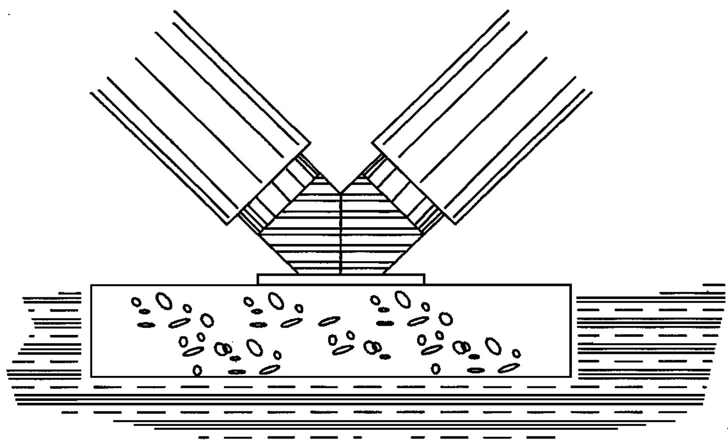

Figure 6.5: Connection to a foundation or concrete element.

combined with the recommendations of section 6.2 leaves the designer free to respond to the specific circumstances of the problem to be solved.

In spite of that, some other recommendations can be derived from the statements in previous chapters.

Though it was argued that the use of a single element, instead of a combination of several ones, is more efficient from the point of view of the stiffness, there may be other reasons why it is desirable to proceed in that manner.

In figures 6.6 and 6.7 we can see that the proposed system can be adapted to cope with this type of demand. There, a combination is shown of three and four culms, which together form a chord, or a diagonal element. The culms probably need to be kept together at midspan, and for this purpose the figures show the use of thin steel bands.

The connection is achieved by projecting steel tips out of the wood cylinder, so that these tips can be welded to a plated or any other central component. As shown in the figures, stiffening

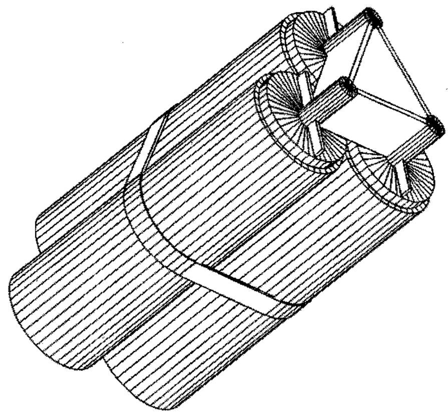

Figure 6.6:Structural element composed of three culms.

plates can be welded to the tips, to fix the relative position of the culms.

6.3-Bamboo's major assets

As explained in chapter one, bamboo is a renewable resource. So far, its 'silviculture' has been rarely defined in terms of construction purposes, except in the case of the Costa Rican Bamboo National Project (a bamboo housing project supported by the Government of Costa Rica and the Government of The Netherlands, see for example, Chaves and Gutiérrez, 1988). Bamboo supplies can easily be sustained in a process that is environmentally friendly, and this one of bamboo's major assets.

The second major advantage of bamboo is its relative lightness, a property that comes from the hollowness of the culms. Designers can take great advantage of this characteristic. It seems that the major field of applications is in structures for roofs and sheds, where the demands of stiffness of connections are not that high. This is an important factor to take into

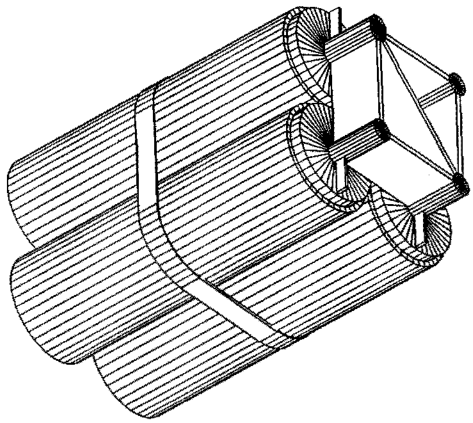

Figure 6.7: Structural element composed of four culms

account, because if the weight of the connections becomes an important factor in the total weight of the structure (which may happen, due to the lightness of bamboo), then little capacity may be left to carry the rest of the loads in the structure. An appropriate design of the shape may allow it to carry quite important loads with only a small amount of bamboo mass in the structure. Therefore, designer should pay most attention to the relation between bamboo and total weight so that an optimum can be reached. This fact may in some cases define the viability of the use of bamboo for a predetermined purpose.

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Chapter 7: Conclusions and further research recommendations

7.1-The structure of bamboo culms

7.1.1-Most bamboo species have hollow culms, though some solid stems can be found. The hollow internodes are connected to each other by nodes, or transversal walls (see section 2.1.2 and related references).

7.1.2-Bamboo culms are composed by a matrix that holds fibre strands together. These fibres run parallel to the axis of the culm, being more concentrated at the top of the culm and towards the outer layer of the cross-section. The fibres are confined to the fibre strands and to sclerenchyma sheaths (see section 2.1.2).

7.1.3-Fibre strands are not continuous along the length of the culm, instead they bend towards the inside of the nodes (see section 2.3).

7.1.4-No radial fibre strands exist in the case of bamboo (see section 2.1.2).

7.1.5-Culms are tapered, the wall thickness decreases from bottom to top, and the internode length varies along the culm.

7.2-Tangential tension capacity of bamboo

7.2.1-The elasticity modulus in the tangential direction is about one-eighth the elasticity modulus in the longitudinal direction (see chapter 2).

7.2.2-No clear correlation could be found between tangential tension capacity and density of the specimen (see section 2.2.2).

7.2.3-A maximum tangential strain was observed for different samples of *Guadua s.p.*, *Bambusa blumeana*, and *Gigantochloa scortechinii*, indicating the possible existence of a characteristic value for bamboo. More research is needed on more species and samples to verify this aspect. This is a matter of importance not only for constructional and structural purposes but also for handicraft (see section 2.2.2).

7.2.4-Fibres contribute indirectly to the tangential strength, because they change the 'texture' of the crack path. The more fibres the crack trajectory encounters, the more energy is needed to fracture the material. Research on this matter should be undertaken to determine the relation between fibre content, density of the non-sclerenchymatose components, and tangential tension capacity (see sections 2.2.2 and 5.4.1.1).

7.2.5-A specimen for the determination of maximum strain at failure in tangential tension was developed. Research is needed to develop a specimen which is also suitable for a simple determination of strength (see section 2.2.1).

7.2.6-Bamboo exhibits an elastic behaviour in tangential tension, with a brittle mode of failure (see section 2.2.2).

7.3-Parallel tension

7.3.1-A strong correlation between parallel tension strength and density was found (see appendix A, eq.A.(1)).

7.3.2-A specimen for the determination of tensile capacity of bamboo strips was developed (see section 2.3).

7.3.3-Nodes are the weakest components. The capacity of the nodes for a sample of *Bambusa blumeana* was found to be only 40% of that of the internode section (see section A.3).

7.3.4-Nodes are the more flexible part of the culm. The average elastic modulus in the region of the node (measured over a length of 30 mm, centred at the node position) was found to be 40% of that of the internode section for a sample of *Bambusa blumeana.*

7.3.5-No study was undertaken on the structural behaviour of the full culm under tension. Research should be done on this matter, taking the role of the nodes into account.

7.3.6-The capacity of strips from the internode section is very high, found to be *270* N/mm^2 for a sample of *Bambusa blumeana.* This fact should stimulate research into the use of the internode sections of bamboo culms in such a way that full advantage is taken of this capacity. In the use of full culms this capacity can not be used because the nodes are the limiting factors (see section A.3).

7.3.7-Further research into the utilization of the fibres of bamboo should include the way different factors affect fibre content and density.

7.3.8-Bamboo presents a perfectly elastic behaviour in parallel tension, with a clearly brittle mode of failure.

7.4-Compressive capacity of bamboo

7.4.1-The conditions of the test set-up in compression are a critical point to be taken care of. Friction and stiffness of the loading plates contribute to the modification of the mode of failure of the specimens (see section 3.1, 3.2 and B.1).

7.4.2-A distinction should be made between the compressive capacity of bamboo as a material and the compressive capacity of the culms. In the latter case, capacity is affected by the geometry of the specimen. Measured parameters may show a strong influence of imperfections in the specimen and the conditions of the test (see sections 3.2 and B.1).

7.4.3-When the full culms are intended to be used for structural purposes compressive capacity should be measured on rings with a ratio of 1:1 for the height and the diameter, and the test should be run in such a way that friction is limited to the minimum possible (see sections 3.2 and B.1).

7.4.4-The relation between longitudinal and tangential strains is about 0.3. The maximum tangential strain on the external surface of bamboo prior to failure is about 0.0012 (see sections 2.2.2 and B.1).

7.4.5-It is possible to predict the stress level at failure when the elastic modulus is known, by using 0.0012 as a limiting figure for the maximum tangential strain (see section B.1).

7.4.6-The secant value between 10 and 80 % of the maximum deformation is a good estimator of the elastic modulus, being more stable and less affected by interactions between the specimen and the plates of the testing machine (see section B.2).

7.5-Critical load of bamboo columns

7.5.1-The major source of lateral deformation in a bamboo column is the initial lateral deformation (see sections E.2 and E.5). This has to be taken into consideration in laboratory experiments on buckling.

7.5.2-It is possible to measure the critical load of bamboo culms by using a Southwell Plot procedure in the laboratory (see sections E.2 and E.5).

7.5.3- The critical load of bamboo culms is affected by a combination of the change in the cross-section and the change of the elasticity modulus along the length (see eq.E.36 and E.37).

7.5.4- Ignoring the influence of the nodes, the calculation of the critical load as a function of average properties of the culm gives a conservative estimate (see eq. E.36 and E.37).

7.5.5-Since the nodes are more flexible than the internodal sections, they decrease the value of the critical load of bamboo columns (see chapter 2).

7.5.6-Nodes are a very random feature, so their average influence should be studied for specific conditions. The results of the Southwell plot procedure can be statistically adjusted to take the influence of the nodes into consideration, and to simplify the calculation. The study of the influence of the nodes on the stiffness of culms indicates that the critical load is overestimated when the nodes are not taken into account (see section E.7 and appendix F).

7.6-Glueability of bamboo

7.6.1-The bonding capacity of bamboo glued phases is not affected by bamboo density, nor by the type of wood, the thickness or the diameter of the culm. A normal wood type of glue is enough to produce a high-quality connection (see section 4.2.1).

7.6.2-Bamboo glued phases can resist cyclic loadings below the design level with no damage or deterioration of the final capacity (see section 4.2.2).

7.6.3-Failure in bamboo glued phases always occurs inside bamboo, therefore bamboo is the most critical component of the phase (see section 4.2.2).

7.6.4-Failure of bamboo glued phases is brittle (see section 4.2.2).

7.7-Bamboo glued connections

7.7.1-Glued wood fittings are a feasible and effective way of taking or transmitting forces to bamboo beam columns, taking into account the weaknesses of the culms and making use of their full capacity (see section 5.1, 5.4, 5.5 and 5.6).

7.8-Connections for bamboo structures

7.8.1-The use of nails, screws or bolts to connect bamboo culms entails a number of disadvantages, because this type of solution concentrates stresses in regions and directions where bamboo is very weak (see section 5.4).

7.8.2-The insertion and gluing of a wooden cylindrical connector in the extremes of bamboo culms not only allows us to take and transmit forces, but also to reinforce the culm in those regions (see section C.4).

7.8.3-It is necessary to use steel fittings as central elements in the connections of bamboo structures. Simple nailing of the wood fittings leads to low capacity and high rotation of the connections (see section F.6).

7.8.4-Simple cheap steel fittings can be designed and developed to connect the wood fittings glued to the inner side of the culms. Some examples can be seen in section 6.2. Research should be concentrated in developing a complete construction system, that is based on the utilization of the possibilities of the glued wood fitting, allowing for a speedy and efficient utilization of bamboo in structures.

7.9-The Design of bamboo structures

7.9.1-When the problem of the connections has well been taken care of, it will be possible to produce bamboo structures that are light, stiff and strong, as shown in section F.6.

7.9.2-In general, the variation of the diameter and that of the modulus of elasticity do not compensate each other in terms of the final stiffness of the culm (see section G.2).

7.9.3-The influence of the variation of the cross-section and of the elasticity modulus produces a reduction of no more than 15% in the bending and the axial stiffness of bamboo, as compared to the figure resulting from using average properties of the culms.

7.9.4-Nodes largely affect the bending and the axial stiffness of bamboo culms. Therefore, it is better from this point of view to use culms with long internodes (see section G.4).

7.9.5-In general, it can be said that the combined effect of the variation of cross section and the presence of nodes results in a reduction up to 50% of the bending stiffness calculated using average properties[1], and up to 80% of the axial stiffness (see section G.4).

7.9.6-Both from the point of view of capacity and deformations, trusses and frame trusses are a much better application of bamboo than frames (see section 6.1 and 6.2).

7.9.7-Structural bamboo components in compression should be kept under a slenderness of 50. In this way buckling is controlled, but also bending moments arising from initial imperfections are kept to a minimum (see section E.7).

7.9.8-The crude application of 'commom' engineering knowledge to the structural design of bamboo may be misleading, because of substantial differences between bamboo and other more commonly used materials.

7.9.10-As a general rule, the more nodes a culm has, the less effective it is for carrying loads, both from the point of view of strength and of deformation.

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[1]the average of the properties at the extremes of the culms

7.9.11-The structural behaviour of bamboo culms under twisting loads is an area of research that is yet to be unexplored.

7.10-General remark regarding bamboo as a structural material

Structural materials have strengths and weaknesses, and it is up to the structural engineer and architect to use them in the best possible way for the benefit not just of a specific client or idea, but for that of mankind as a whole.

The facts found during this research show that bamboo can be given some restricted structural use. It is the hope of the author that some elements of relevance are put forward in this thesis, so that new structural shapes and applications can be devised. The gathering of controlled field experience should be the major focus of attention in the future, and would be the best way to promote the use of bamboo.

The results of the testing of trusses in the laboratory are encouraging, especially because the comparison of the weight of the structure with its capacity and stiffness gives a clear vote of confidence as regards the potential of the material.

During this research effort, bamboo proved to be a difficult material, sometimes it was even evasive and impenetrable. Laboratory work on the preparation of specimens of all types has shown how unworkable it can be, perhaps due to the fact that no tools, at the construction level, are yet available for bamboo.

During the very last phase of the research project, though, the making of a few trusses demonstrated that it is possible to build attractive, light, strong and well-engineered structures from this material. As a final statement the author wishes to share his enthusiam in this sense, and to encourage engineers and architects, especially those colleagues who have bamboo available, to give it a chance by using all their inventiveness. Bamboo will pay back.

7.11-General remark on future research

The attention of bamboo research should move from the specifics of the properties of bamboo as a material, to the applications in the construction industry.

It must be borne in mind that the major reason some institutions and individuals are paying attention to this materials is its abundance and easy reproduction, and the possibilities that it may hold for people in the third world. But the results of research activities will never reach these populations as long as they are kept in the sometimes ambiguous terrain of testing compression and bending capacity in the laboratory only. Communication between field engineers and researchers is needed in order to focus our attention on the real issues concerning the utilization of bamboo in construction.

The current efforts in looking for a solution for the problem of preservation and treatment should be strongly encouraged and supported.

Appendix A: Experimental analysis of parallel tension

A.1-Influence of the speed of loading on the tensile capacity of bamboo

Table A.1 shows the descriptive statistics for the results of three treatments (speeds of deformation).

Variable:	0.2 mm/min	0.6 mm/min	1.0 mm/min
Sample size	11	11	11
Average	272.135	294.144	309.983
Median	300.82	291.96	328.0
Mode	282.48	261.0	284.0
Geometric mean	264.718	288.377	307.832
Variance	3733.93	3910.7	1468.22
Standard deviation	61.1059	62.5356	38.3173
Standard error	18.4241	18.8552	11.5531
Minimum	143.68	195.81	263.44
Maximum	345.76	419.88	374.58
Range	202.08	224.07	111.14
Lower quartile	224.0	259.68	272.25
Upper quartile	314.64	317.0	339.6
Interquartile range	90.64	57.32	67.35

Table A.1: Results from three loading speeds.

These results show that strengths tend to be higher for higher rates of loading, although it is clear that more studies are needed to completely understand this phenomenon.

The goal of the study regarding the influence of the loading rate was to be able to gain some flexibility in the eventual specifications of the test. This is because most of the laboratories, where tensile tests may eventually be carried out, may not have conditions good enough to assure a permanent and non variable rate of loading. So, the question is to what extent some variation in the rate of loading may lead to variation in the tensile strength measured this way.

Two-Sample Analysis Results

Sample Statistics:	0.2 mm/min	1.0 mm/min	Pooled
Number of Obs.	11	11	22
Average	272.135	309.983	291.059
Variance	3733.93	1468.22	2601.08
Std. Deviation	61.1059	38.3173	51.0007
Median	300.82	328.	301.24

Difference between Means = -37.8482

Conf. Interval For Diff. in Means: 95 Percent

(Equal Vars.) Sample 1 - Sample 2 -83.2222 7.5258 20 D.F.

Ratio of Variances = 2.54317

Conf. Interval for Ratio of Variances: 0 Percent

Sample 1 ÷ Sample 2

Hypothesis Test for H0: Diff = 0 Computed t statistic = -1.7404

vs Alt: NE Sig. Level = 0.0971534

at Alpha = 0.05 so do not reject H0.

Table A.2: Comparison of two loading speeds.

Table A.2 reports the test results of the hypothesis that the difference in means between the two extreme speeds tested is zero. It can be seen that for a confidence interval of 95% this hypothesis is correct, in spite of the difference in loading rates. Therefore, at least in a preliminary step, it may be concluded that the proposed test could be done in a certain range of loading speeds without affecting the results thus obtained in a significant way.

A.2-Influence of density

It was found that when moisture and density were taken together in the analysis, moisture accounted for only 20,29 % of the variance. In our case, moisture ranged between 12.80 and 17.71 % as it can be seen in table A.3. It can be observed that specimens were not specifically conditioned from the perspective of constant moisture, nor was a wide enough range selected for this variable, since as we established before, our main interest was to see the influence of density under some flexibility regarding the values of other parameters[1]. No attempt has been made in this experiment to find a correcting factor for moisture, but its influence is analyzed anyway, of course, as explained in the following lines. Density and moisture were measured according to ISO 3131-1975(E) and ISO 3130-1975(E) respectively, except that the shape of the specimens was changed to small strips of 3 mm x 50 mm x thickness [2]. Table 2.4 gives us some appreciation of the high tensile strength of bamboo specimens from the internode section of the culm.

Two problems related to moisture and density become apparent after analyzing table A.3. First, both density and moisture tend to distribute in the way of a long right tail, but this factor is even more pronounced in the case of moisture, as revealed by the values of the skewness coefficients. Second, the distribution of density tends to be flat with short tails, but this feature is more pronounced in the case of moisture, as shown by the values of the Kurtosis coefficients. What is more important, the standard values of these coefficients also show that the normality assumption is very good for density but poor for moisture, facts that have to be taken into consideration in the analysis.

In spite of the above statement the influence of moisture was also checked by calculating a regression of tensile strength vs. density and moisture, as shown in table A.4.

Analysis of the significance levels and the t-factors leads to the conclusion that the regression

[1]The relation between density and tensile capacity was also studied for conditioned specimens at 12% moisture content, but, again, it was of interest to see if some deviation from this value would significantly affect the result, which was not found to be the case.

[2]Thus density refers to the average along the thickness.

is not a good one, since the possibility of failing to predict is very high. Perhaps this was to be expected due to the combination of two variables with two very different distributions as was shown before.

This result supports the possibility of looking for a simple relation between tensile strength and density, instead of complicating the equation by including moisture.

Table A.5 below shows the results of calculating a linear regression of tensile strength and density. The improvement in the significance level can be seen. It can therefore be concluded that density can indeed be a good estimator of the tensile strength of this batch of bamboo. An analysis of variances confirmed the accuracy of the model and, therefore, it could be concluded that

$$f_t = 0.4\ \rho \qquad \text{eq.A.(1)}$$

Variable :	density	moisture	strength
Sample size	11	11	11
Average	686.56	13.63	275.16
Median	666.	13.27	300.82
Mode	634.	13.24	300.82
Geometric Mean	682.17	13.57	269.79
Variance	6869.42	1.89	2978.84
Standard Deviation	82.88	1.37	54.58
Standard Error	24.98	.41	16.45
Minimum	585.4	12.8	177.0
Maximum	827.3	17.71	345.76
Range	241.9	4.91	168.76
Lower Quartile	622.	13.	224.
Upper Quartile	783.8	13.57	314.64
Interquartile Range	161.8	.57	90.64
Skewness	.66	3.11	-.57
Standarized skewness	.90	4.21	-.77
Kurtosis	-1.01	10.02	-1.01
Standardized Kurtosis	-.68	6.78	-.69

Units : density = kg/m^3 moisture = % tensile strength = N/mm^2

Table A.3: Descriptive statistics for a sample of *B.blumeana* from the Philippines.

Model fitting results for:TENSILE STRENGTH

Independent variable	coefficient	std. error	t-value	sig.level
DENSITY	0.200101	0.210589	0.9502	0.3668
Moisture	10.03717	10.62858	0.9444	0.3696
$r^2 = 0.9623$				

Table A.4: Density and moisture related to strength.

Model fitting results for: TENSILE STRENGTH

Independent variable	coefficient	std. error	t-value	sig.level
DENSITY	0.397578	0.024755	16.0606	0.0000
$r^2 = 0.9627$				

Table A.5: Regression model of density strength.

A.3- Influence of nodes.

To examine the effect of the nodes, samples were checked in such a way that internode and node samples were taken from neighbouring positions along the circumference , and tests ran under the same circumstances.

The results in table A.6 show that the tensile strength of the node region is only about 30% that of the internode.

Table A.7 shows that there is also a significant difference in the value of the elastic modulus. Node elastic modulus is about 40% the internode's.

Variable:	TENSION INTERNODE	TENSION NODE
Sample size	11	11.
Average	270.586	79.8936
Median	287.09	80.71
Mode	273.13	80.7
Geometric mean	263.66	78.6725
Variance	3597.13	201.64
Standard deviation	59.9761	14.2019
Standard error	18.0835	4.28204
Minimum	154.76	52.48
Maximum	334.27	101.89
Range	179.51	49.41
Lower quartile	224.37	69.4
Upper quartile	326.4	88.67
Interquartile range	102.03	19.27
Skewness	-0.76207	-0.29442
Standardized ..		
skewness	-1.03185	-0.398646
Kurtosis	-0.446007	0.172994

Table A.6: Strengths from node-internode tests.

Variable:	INTERNODE MODULE	NODE MODULE
Sample size	11	11
Average	18857.8	7452.62
Median	19444.7	7730.
Mode	17600.	6752.86
Geometric mean	18021.4	7317.02
Variance	2.99612×10^7	2.2288×10^6
Standard deviation	5473.68	1492.92
Standard error	1650.38	450.131
Minimum	8928.0	5511.0
Maximum	26794.9	9794.0
Range	17866.9	4283.0
Lower quartile	15376.0	6082.0
Upper quartile	22271.0	8392.0
Interquartile range	6895.0	2310.0
Skewness	-0.403314	0.216891
Standardized skewness	-0.546089	0.293671
Kurtosis	-0.380636	-1.28877
Standardized kurtosis	-0.257692	-0.872502

Table A.7: Elastic Moduli from node-internode tests.

Appendix B: Experimental analysis of short elements

B.1- Introduction

Several problems need to be taken into consideration in dealing with compression tests of bamboo elements.

Perhaps the first aspect that is usually noted by the experimenter is the fact that the cross-section of bamboo elements is not a perfect circle, but rather an irregular shape. Since it is customary to calculate stresses as simply being the ratio of the normal force to the area of an assumed cylinder of constant thickness, it is relevant to find out what influence this simplification has on the values measured in this way. As proposed by Xiu (ref. 11, chapter 3) a factor can be derived, like that of equation 3.3, to correct the cross sectional area of the specimen.

From a different stand-point when the cross-section resembles an elliptical ring, its area can be related to that of a circular ring by the equation

$$\frac{A_e}{A} = \frac{1+r_d-2r_t}{2-2r_t} \qquad \text{eq.B.(1)}$$

A sample taken from the stock of *Bambusa blumeana* and *Guadua s.p.* in the laboratory at Eindhoven University of Technology, showed that the range of values for the parameters r_d and r_t ranged, respectively, from 1.0 to 1.3 and from 0.05 to 0.15. Evaluation of equation B.(1) for the resulting extreme values shows that the amount of error in assuming a constant circular section ranges between 0 and 16 %. Therefore, a directly correspondent range also occurs in the estimation of stresses. The important point here is that the deviation of the cross-section from a circular one should be observed and its importance never underestimated, and that the actual conditions of the sample in this respect should be added to the report, to make comparisons possible.

A second problem, very much related to the above-mentioned irregularity of the cross section is that the thickness of the specimen is not constant, and neither is the density on the surface of the cross-section. This means that some concentration of stresses may occur during testing and this produces very different failure patterns, some of them merely due to higher contact stresses at the points of larger stiffness.

A third problem is that the steel plates of the apparatus act as a lateral reinforcement, leading to possible overestimations of the mechanic properties, as found by Meyer (ref. 7 chapter 3). Tests done in machines, which are very stiff in the movement of the heads, are likely to be influenced by the rigidity of the plates. In some cases the plates are supported by a ball-like connection, but even then some other measures should be taken to ensure that the test is a fair approximation to the envisaged conditions.

Figure B.1 shows the results of a test using a machine in which the upper plate was hinged, but no special measures were taken in relation to friction. Results are stresses vs. vertical strains calculated using the vertical displacement of the plates, as has traditionally been in the case of bamboo.

The result can be misleading in many ways. Firstly, the initial section of the curve registers the 'consolidation' of the plates. In other words, all the movement that takes place till all the pieces fit each other, including the specimen in relation to the plates. So, this is not a plot that gives information about the material only, but about the whole mechanism present in the test set-up.

Secondly, some recovery is observed after failure. In some cases even higher loads than the first peak can be recorded, but this is due to the fact that, because of the high normal stresses, friction keeps the specimen trapped between the plates, so that it can sustain some extra force. Besides the above-mentioned factors, and purely from a technical point of view, the making of the specimen is a very laborious and difficult task, due to the tapering and to the fact that the cross-section is not circular. Most of the time this leads to specimens in which the surfaces in contact with the plates are not parallel to each other, thus inducing accidental eccentricities during the test. What is more, the problem is worsened by the fact that an increase in the height of the specimen implies increments on the eccentricity of the load as well, inducing radial bending moments towards the inside of the specimen resulting as in figure B.2.

Control of the conditions during a compression test is therefore a factor of the major relevance.

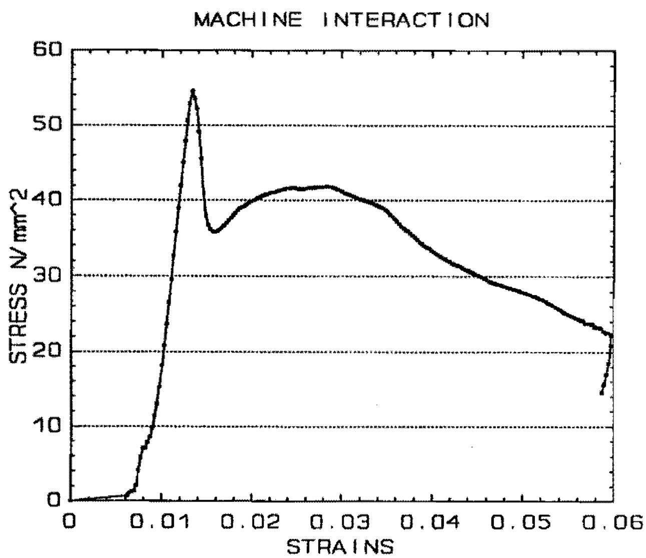

Figure B.1: Compression test stress strain curve.

Figure B.3 shows a test specimen tested using the special support shown in figures B.4, B.5 and B.6. It can be seen that in these cases the culm could expand freely in the lateral direction, displaying longitudinal cracking as a main feature.

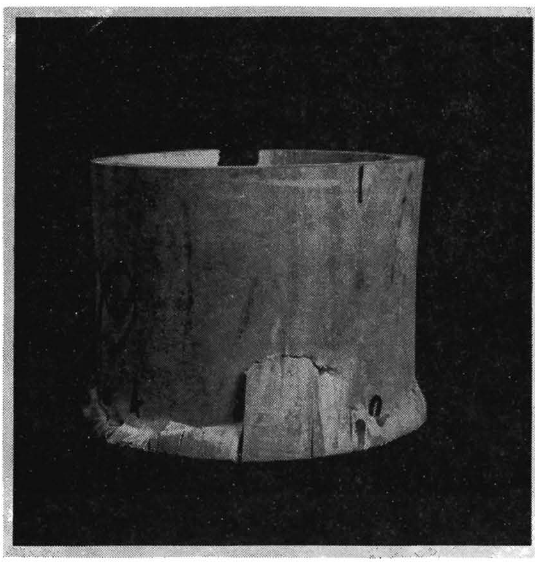

Figure B.2: Bamboo specimen after a compression test with no special consideration for the machine interaction problem.

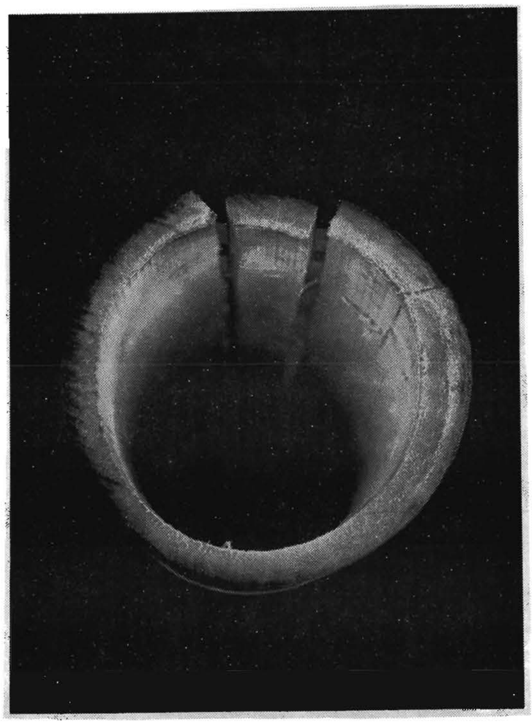

Figure B.3: Bamboo specimen after a compression test with special supports.

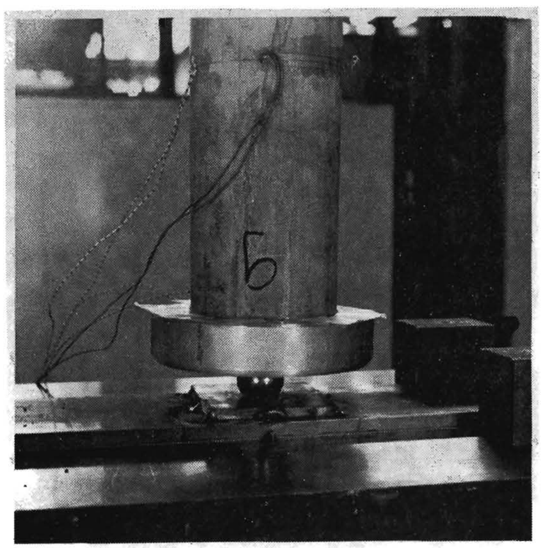

Figure B.4: Special hinged support.

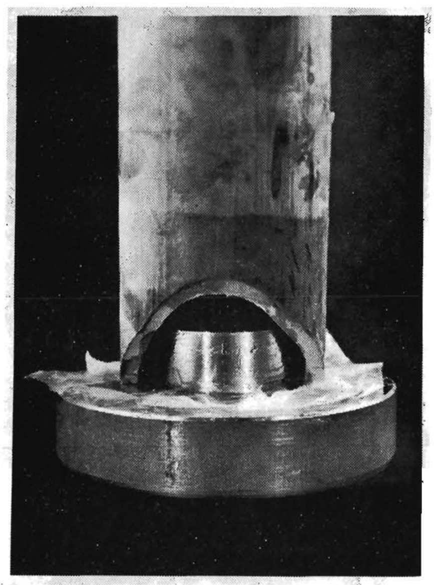

Figure B.5: Hinged support showing thin teflon layers and greased surfaces.

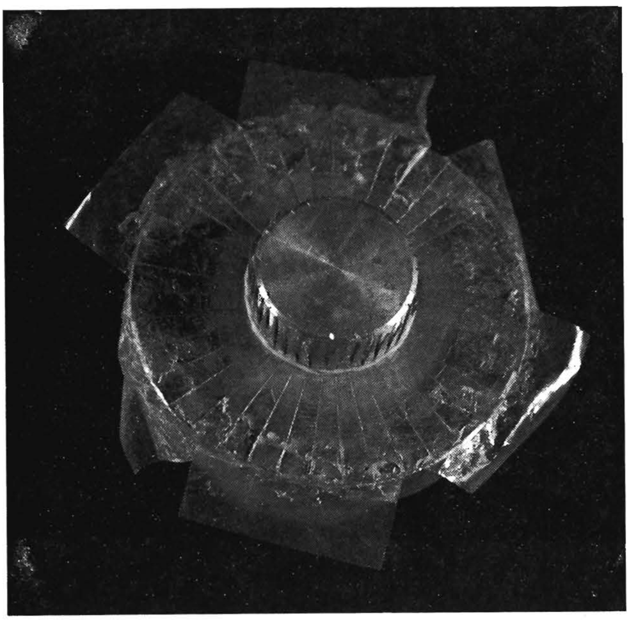

Figure B.6: Hinged support showing radial sliding elements made of steel.

The radial elements one can observe in figure B.6, displace radially with the lateral expansion of the specimen, as a result of remaining friction. They are made of pieces of galvanized steel 1 mm thick, that are independent from each other, so that they completely fill the surface of the supporting fitting. A layer of grease and teflon separates the galvanized steel radial sheets from the support.

B.2-Experimental results

Table B.1 shows the descriptive statistics for the maximum strains, both longitudinal and transversal, for a sample of *Bambusa blumeana.*

Variable:	long.strains	tang.strains	E modulus
Average	0.00317	0.001149	18784.7
Variance	$7.32x10^{-7}$	$2.68x10^{-7}$	$2.196x10^{7}$
Standard dev.	$8.56x10^{-4}$	$5.18x10^{-4}$	4686.7
Standard error	$2.58x10^{-4}$	$1.56x10^{-4}$	1413.09

Table B.1: Descriptive statistics for *Bambusa blumeana* in compression. [N/mm^2]

	One-Sample Analysis Results			
Sample Statistics:	Number of Obs.	25		
	Average	$1.22804x10^{-3}$		
	Variance	$1.07117x10^{-7}$		
	Std. Deviation	$3.27288x10^{-4}$		
	Median	$1.28x10^{-3}$		
Confidence Interval for Mean:		95 Percent		
		$1.09291x10^{-3}$	$1.36317x10^{-3}$	24 D.F.

Table B.2: Analysis of peak tangential strains for *Guadua s.p.*

It was found that at a 95% confidence level, the peak value of tangential strains found for *Bambusa blumeana* in compression is equal to the average value found for the peak tangential strain from tangential tension tests. This suggests that it is possible to estimate the maximum compression capacity by using the values of strains from perpendicular tension tests, if average values of the E modulus are known for a given batch.

The results on *Guadua s.p.* from Costa Rica show a highly similar maximum tangential strain as can be seen in table B.2. The 95% confidence limit for the mean is also included in the table. The similarity of the results is interesting, and it indicates the importance of exploring these parameters for other species. It is fairly likely that a critical value for tangential strain can be found for each species, or perhaps groups of species, allowing for a simplification in the calculation of the capacity of the culms.

Further, no correlation could be found between peak strains and the elastic modulus. Figures B.7 and B.8. illustrate these last two results. In the first figure one can see that, culms tend to fail at a commom maximum strain, as mentioned before, in spite of the wide range of observed strengths in the sample.

No reliable correlation could be found between the peak stress of compression tests and density, though, as indicated in section 3.1, several authors report that such a correlation exists. A possible explanation for this is that density does correlate with the amount of friction that can be generated between the specimen and the plates of the testing machine[1], on the one hand, and on the other, that the reinforcing effect that the interaction machine-specimen produces does increase this phenomenon. Figure B.9 presents data from *Guadua s.p.* concerning the above. Figure B.10 further illustrates the relation between densities and strengths found.

Descriptive statistics are included in table B.3 below. Moisture content was set at 12%.

[1]which would obviously depend on the number of fibres

Variable:	dry density	strength	E
Sample size	31	31	31
Average	691.121	50.4958	18464.6
Median	718	50.85	18098
Mode	713.91	49.97	18097
Geometric mean	684.869	49.8506	18248.5
Variance	7918.63	66.2256	8.50777×10^6
Standard deviation	88.9867	8.13791	2916.81
Standard error	15.9825	1.46161	523.874
Minimum	448	36.5	14062
Maximum	843.5	67.15	25233
Range	395.5	30.65	11171
Lower quartile	644.61	42.16	15808
Upper quartile	742.63	57.69	20619
Interquartile range	98.02	15.53	4811
Skewness	-1.13173	0.038994	0.520005
Standardized skewness	-2.57245	0.0886346	1.18199
Kurtosis	1.45706	-0.810767	-0.485961
Standardized kurtosis	1.65597	-0.921449	-0.552302
Coeff. of variation	12.8757	16.116	15.7967

Table B.3: Descriptive statistics for the sample of *Guadua s.p.*[kg/m^3, N/mm^2]

The skewness and the Kurtosis coefficients of the distributions give further evidence on the differences between the sample in relation to density, on the one hand, and strength and elastic modulus on the other.

Appendix B: Experimental analysis of short elements

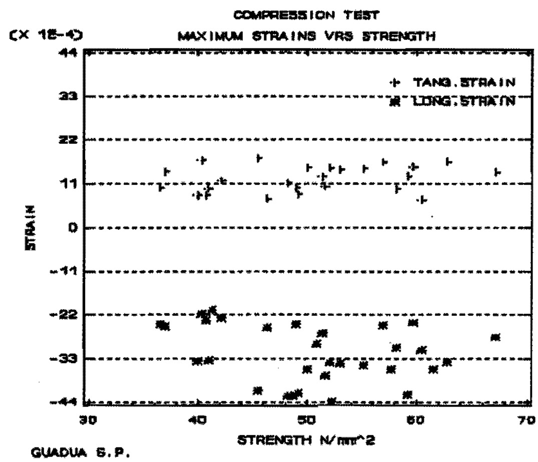

Figure B.7: Strains and stresses compared

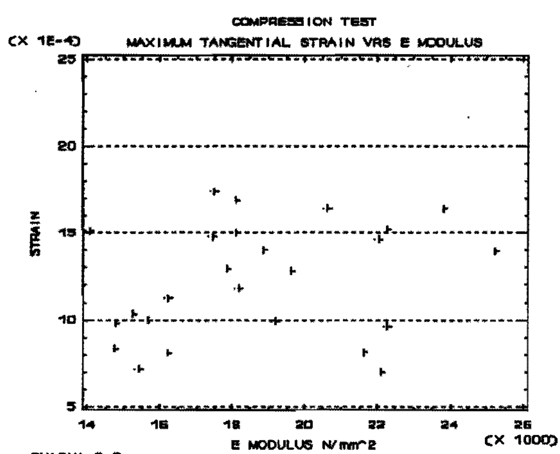

Figure B.8:Tangential strains vs. E modulus

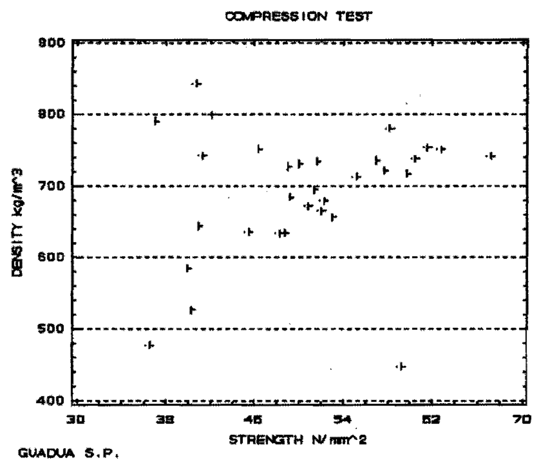

Figure B.9:Density vs. strength for a sample of *Guadua s.p.*

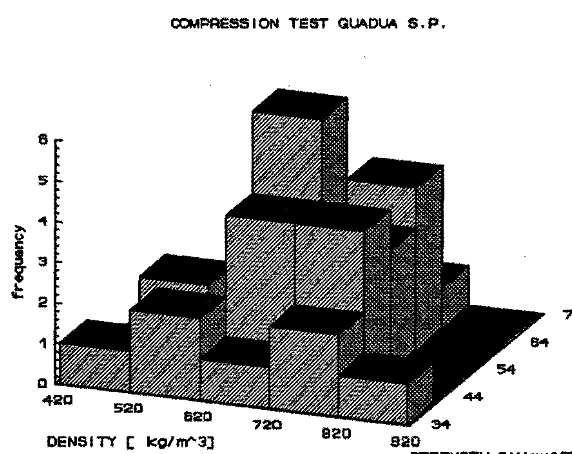

Figure B.10:3-D histogram for density and strength, *Guadua s.p.*

It is important to add some comments on the way the elasticity modulus was calculated, because, though it is customary to report its value from compression tests, the way it is calculated is rarely stated with clarity. Most of the time the elasticity modulus is calculated as the secant value between zero and the peak load, though sometimes the 'limit of proportionality' is taken for the calculation.

Figure B.11 shows the resulting plots of strains and stresses for four specimens of *Guadua s.p.*, from tests done using the frictionless set-up. As explained before, friction is eliminated sometimes so that the failure is absolutly brittle, but, as is shown in the examples, it may be that the cracked specimen is trapped between the supports, giving the impression of some plasticity, which is not the case. The point is that under these conditions, which can be ambiguous, it is difficult to establish what the 'limit of proportionality' is, which probably leads to results that are difficult to compare. As can be appreciated from the comparison of figures B.11 and B.1, it is even more difficult to avoid ambiguity where the test is run in a traditional way.

The results presented in figure B.11 show that there is also some ambiguity at the beginning of the loading path, which makes it seem as if the specimen was stiffer.

Taking into account the purpose of using the elasticity modulus in the prediction of deformations, and the need to make results comparable and self-consistent, it would perhaps be advisable to calculate the elastic modulus as the secant value, between ranges of 10 and 80% of the strength, or in any case, in a region where the curve strain stress is less influenced by extreme conditions. During this research project the values calculated in this way were compared to the secant values between 0 and 100%, and between 0 and 50%. It was found that there can be very big differences between one criterion and another, and therefore it is of the major importance to clearly state the way the modulus is calculated in each report.

Finally, gratitude is expressed to the staff members of the Costa Rican Bamboo National Project, who kindly supplied the studied sample of *Guadua s.p.*.

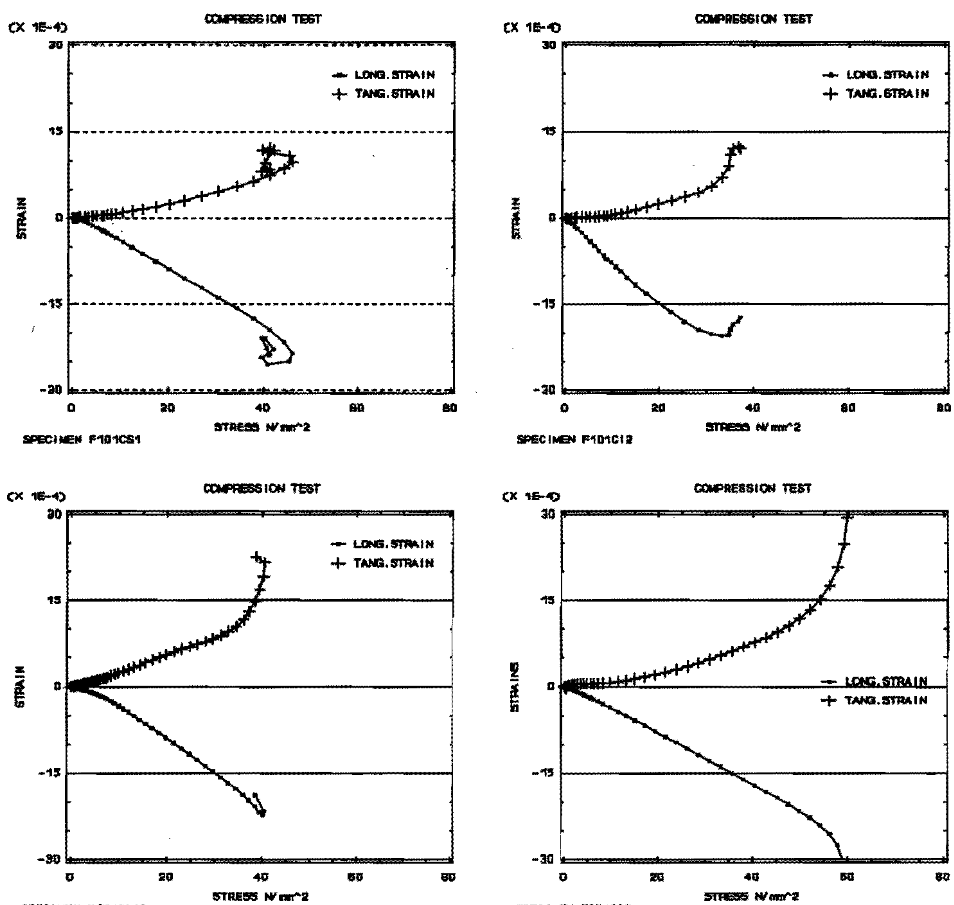

Figure B.11:Strain stress plots for four specimens of *Guadua s.p.*

Appendix C : Experimental analysis of bamboo wood glued phases

C.1-Bonding specimen.

Small samples of bamboo were first taken out of the culm, their inner surface flattened, and specimens as the one shown in figure C.1 were prepared.

Several problems were found in practice. For example, the removal of part of the inner surface (in order to have a flat face) introduces variations in the surface characteristics of the specimens. This is provoked by the fact that the number of fibres present varies with the radius, so there are more fibres at the borders of the samples because there the remaining thickness is smaller. It is very difficult to keep this fact under control in a manner that allows for the comparison and repetition of tests.

On the other hand, the preparation of the specimens proved to be rather cumbersome, because it is very difficult to keep dimensions and relative positions of the components under control. Because of particular interests in the gluing possibilities of the inner surface of the culms, it was considered best to try to use sections of the culms kept as intact possible. So, the specimen shown in figure C.2 was considered to be a better option.

As shown in the figure, two glue strips 10 mm wide connect the two pieces of bamboo to the inner core of wood. While setting, pieces are kept together by clamps, or most preferably by steel screw rings. The surfaces of both wood and bamboo are carefully cleaned and freed from wax, dust or whatever other substance or condition that may be deleterious to gluing. The bottom surface of bamboo, which is in contact with the supporting plate in the testing machine, is carefully levelled and made parallel to the upper surface of the wood cylinder to avoid eccentricities during loading. Grease and teflon are placed between bamboo and the steel support to avoid friction resistance, though this fact is not so important where the upper surface of the wood cylinder is concerned. The specimen is located between two freely rotating plates.

Deformation is measured by controlling the movement of wood. This is done by means of pins located at opposite sites half way down the length of glue. Initially two pins were located on bamboo as well, at about 1 mm from the inner surface, and two were located through holes on the wood cylinder. This was done in order to observe slipping between the two

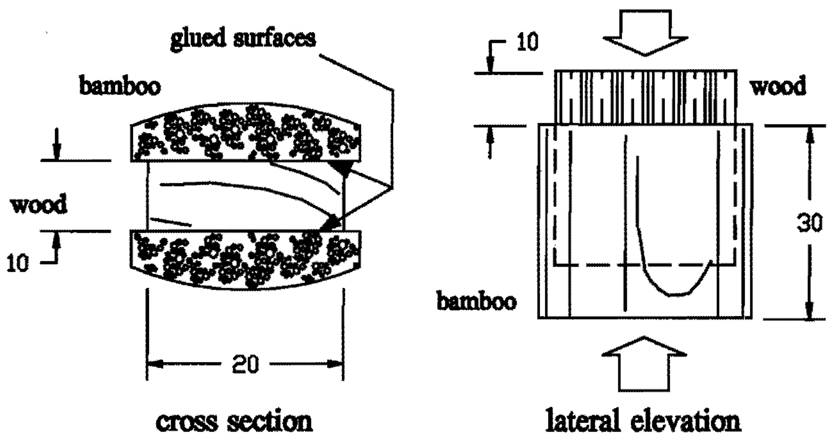

Figure C.1:Specimen for determination of gluing capacity. Dimensions in [mm].

materials and to be able to rate the quality of the specimen. For normal measurements, only the pins on wood are considered necessary. It has to be stated though that the readings from such pins correspond to the movement of a particular point on the skin of the wood cylinder, but not to the actual strains at the gluing surface, though the method does give an indication of the shape of the load deformation path for these glued phases. The area of glue is kept as small as possible, so that the wood cylinder does not significantly deform during loading. The speed of loading was set according to ISO 6891-1983 (E)(Anonymous, 1983).

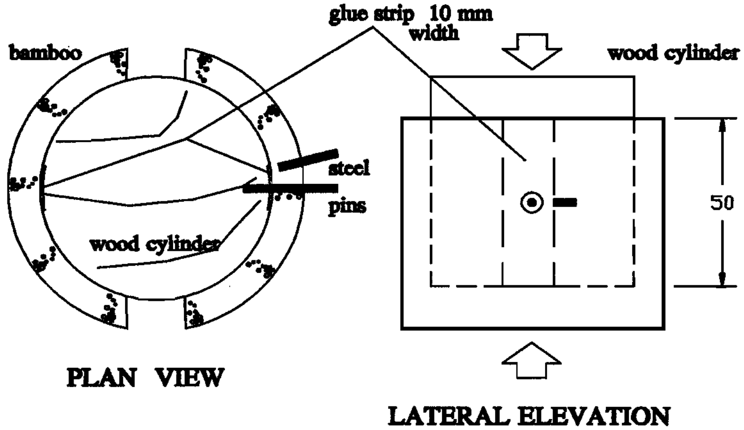

Figure C.2: Specimen used for bonding test. Dimensions in [mm].

C.2- Load deformation relations and influencing factors

An example of a stress deformation curve is given in figure C.3.Most of the observed results showed a very smooth and continuous relation between stresses and deformations, as seen in the figure. Failure occurs in a brittle manner, and was found within the first mm of the layer of bamboo for all cases studied, as will be explained in the following lines. Before going ahead with the description of results on capacity, deformations and other features, it is necessary to explain the results of some experiments, to determine the degree of influence of certain factors on the qualities of the phases.

Studied factors were :

- Bamboo density.
- Bamboo thickness.
- Bamboo initial diameter.

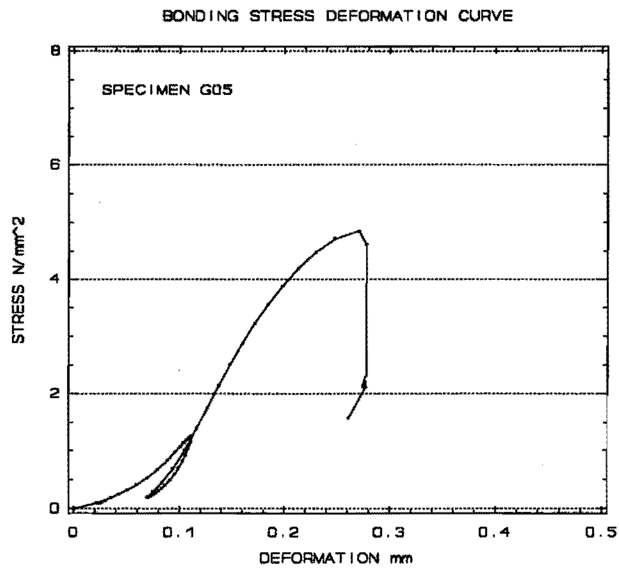

Figure C.3: Typical stress deformation curve.

- Type of wood either hard or soft.
- Type of glue, either PVAc or Araldite.

A total of 32 specimens were prepared, 16 for each type of glue. Half of each block was made using hard wood, and the other half using soft wood. The initial diameter, the thickness, and density were variables that were entered randomly into the experiment. The analysis of variance for the experiment is shown in table 4.1. The results have been already discussed in chapter 4, but a few other elements are added here for the sake of completeness.

Table C.1 shows the result of pooling each one of the glues together (that is, involving the two types of wood for each type of glue). This was done in order to have some indication of the variability of the data in this way, and to be able to calculate material factors for the two glues, so that design values could be compared.

The material factor in the table is worked out by using the expression

$$W = \frac{\left[1+\left(1-0{,}9375\left(1-6{,}25\,V_s^2\right)\right)\right]T_s}{1-6.25\,V_s^2} \qquad \text{eq.C.(1)}$$

(for a reference on this equation see for example, Huybers, 1990). Here V_s is the coefficient of variation and T_s is a time factor, normally taken as 0.8.

To add an extra element to the analysis, a statistical comparison of the two sets of results was undertaken, and the result of the test of hypothesis is given in table C.2.

Variable:	wood glue	resin
Sample size	16	16
Average	5.78555	6.32779
Median	6.0937	6.1795
Mode	5.548	5.905
Geometric mean	5.72079	6.25775
Variance	0.733879	1.0612
Standard deviation	0.856667	1.03015
Standard error	0.302878	0.364212
Minimum	3.929	4.85
Maximum	6.57	8.348
Range	2.641	3.498
Lower quartile	5.454	5.82015
Upper quartile	6.345	6.7125
Interquartile range	0.891	0.89235
Coefficient of variat.	0.1469	0.1627
Material factor	**2.985**	**3.1629**
Design value	**1.94**	**2.00**

Table C.1: Summary of statistics for strength [N/mm^2].

Two-Sample Analysis strengths

		wood glue	resin	Pooled
Sample Statistics:	Number of Obs.	16	16	32
	Average	6.32779	5.78555	6.05667
	Variance	1.0612	0.733879	0.897541
	Std. Dev.	1.03015	0.856667	0.947387
	Median	6.1795	6.0937	6.1387

Hypothesis Test for H0: Diff = 0 Computed t statistic = 1.1447
vs Alt: NE Sig. Level = 0.271526
at Alpha = 0.05 so do not reject H0.

Table C.2: Comparison of the two glue types.

As can be deduced from the table it can be stated that there is no difference between the strengths of the two types of glues. The difference is even less significant for the design values, as shown in the previous table.

One final element concerning the influence of the two glues is examined.

When dealing with structures, the stress level is not the only concern matter, deformations should be looked at too. In order to see the effect of the different factors on this parameter, an analysis of variance for the final deformations was undertaken. Table C.3 shows the results, and table C.4 shows the results of calculating the 95% confidence limit for the means. Certainly, it is clear that though the results are below significance, different glues give different end deformations. The same can be said of bamboo density, and the table also shows as that the type of wood is even less of an influence in this matter.

Analysis of Variance for DEFORMATION

Source of variation	Sum of squares	d.f.	Mean square	F-ratio	Sig. level
COVARIATES	.0066216	3	.0022072	.773	.5409
density	.0056442	1	.0056442	1.977	,1974
diameter	.0000204	1	.0000204	.007	.9356
thickness	.0000734	1	.0000734	.026	.8783
MAIN EFFECTS	.0056313	2	.0028157	.986	.4142
woodtype	.0001708	1	.0001708	.060	.8155
gluetype	.0055857	1	.0055857	1.956	.1995
2-INTERACTIONS	.0001179	1	.000118	.041	.8461
GLUE-WOOD	.0001179	1	.000118	.041	.8461
RESIDUAL	.0228419	8	.0028552		
TOTAL (CORR)	.0352127	14			

Table C.3: Influencing factors on final deformation.

The results seem to suggest that phases tend to fail at about the same level of deformation. At the level of the design stress, deformations are also similar.

The analysis of the corresponding values shows that the average deformation at the design level of stresses is about 0.09 mm, without a noticeably significant difference between the two types of glue, as found in a statistical comparison.

It is also of interest to see if there is any correlation between the maximum stress and the maximum deformation.

Such an analysis was undertaken for wood glue only. A correlation analysis for strength and deformations gives a coefficient of (-)0.3731, but the significance level of a Student's test is 0.1546.

Level	Average	Std. error (internal)	Std. error (pooled s)	95% Confidence for the mean	
woodtype					
h	0.27349	.012573	.0201963	0.22690	0.32007
s	0.26610	.022508	.018891	0.22252	0.30968
glue type					
r	0.25301	.010497	.0201963	0.20643	0.29960
w	0.28401	.021957	.0188919	0.24044	0.32759
INTERACTION WOOD-GLUE					
h r	0.25277	.014465	.026717	0.19115	0.31440
h w	0.30110	.004563	.030850	0.22994	0.37226
s r	0.25330	.018782	.030850	0.18217	0.32449
s w	0.27376	.035687	.023896	0.21864	0.32888
Total	0.26955	.013797	.0137967	0.23772	0.30137

Table C.4: Confidence limits for the mean maximum deformation.

In other words, evidence shows that no correlation exists between strength and final deformation.

In order to observe the way strains vary along the length of glue, a different test specimen was designed as shown in figure C.4. Small strain gauges were glued along the length of the specimens, and readings of the strains were taken constantly during loading. Data were later summarized so that strains at each one of the strain gauges could be plotted against the position of the strain gauges along the length, as seen in figure C.5. In this way it is possible to obtain a plot for specific levels of force. Two of these plots are given in the figure with an indication of the force level as a percentage of the maximum reached value. The graph gives an indication of the approximate linear variation of the strains from one end to the other. The angle of the linear variation increases with increments in load.

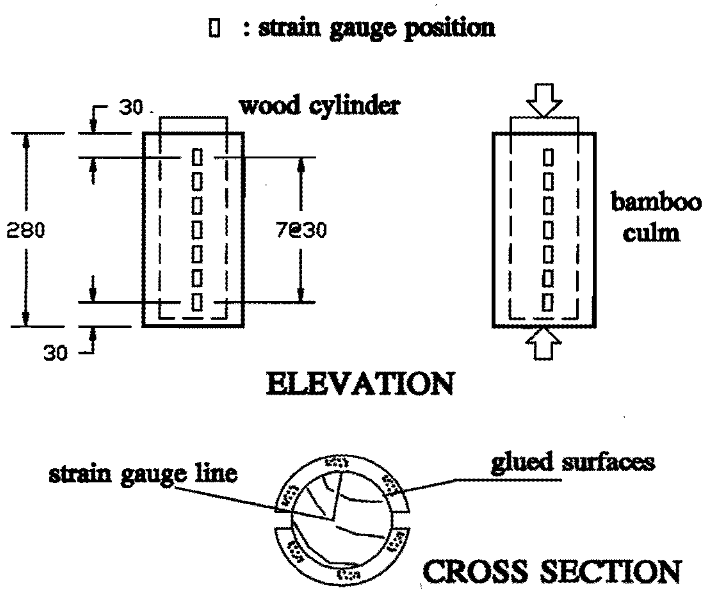

Figure C.4:Test specimen for the distribution of strains. Dimensions in [mm].

The plot helps to visualize to what extent one can be precise in assuming a certain distribution of strains along the length of glue. It can be seen that at the design level a

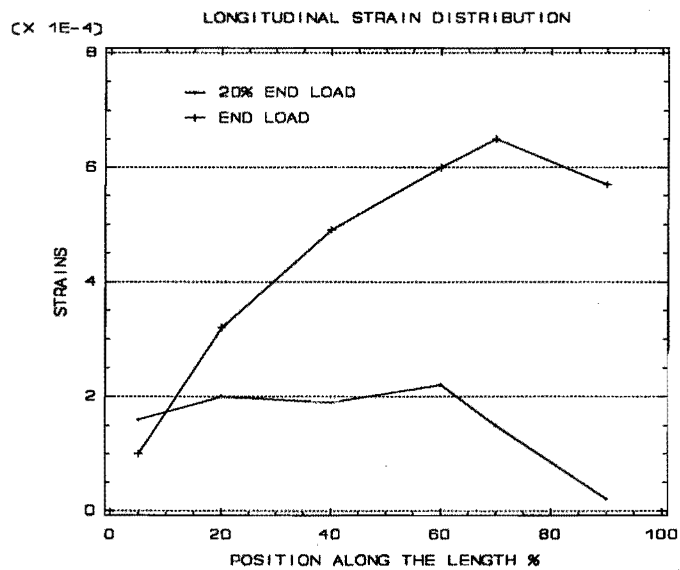

Figure C.5:Strain distribution along the length of glue.

constant distribution of strains (and therefore of stresses) can be justified, but for higher levels of load the distribution becomes more complex. According to the plot, strains tend to be larger towards the side where the wood cylinder projects beyond the connection. A possible explanation for this is that the culm is completely glued to the piece of wood on this side , so that it is constrained to move radially. At the other extreme, the culm expands more freely, due to compressive forces, so therefore less stresses can be transferred along the glued phase. The higher the load, the more important this mechanism becomes, and thus the more pronounced the effect shown in figure C.5 is.

C.3- Glued phases under the effect of dynamic loads

Four different types of experiment were set to study the effect of dynamic loads on glued phases, as is explained in the following.

Five groups of six specimens each were prepared using soft wood and PVAc glue, as explained in section C.1. The first group was set as a reference, each specimen was steadily loaded till failure, and relevant information was recorded.

Each one of the remaining groups was subjected to different load conditions, and the result of final capacity and deformation was compared to that of the reference group. In other words, each load condition was regarded as a different treatment from a statistical point of view. As will be seen, statistical comparisons were not always necessary, because of the clarity of the results.

C.3.1-Load type one.

At the speed of loading prescribed by ISO specimens were loaded up to 100% of the calculated design load[1], then unloaded to 10% of that same value, and finally the cycle was repeated 20 times. A result was presented in figure 4.3. After the 20th cycle, loading continued till failure. It can be observed that though, some degree of consolidation occurred, as was mentioned before, it looks like the curves returned to that of the reference group, after the last cycle. The statistical comparison of both ultimate load and ultimate deformation showed no significant difference between this group and the reference group, though final deformation was found to be slightly higher for specimens under the repeated load condition.

C.3.2-Load type two.

Another group of specimens was loaded following the same load pattern as that of load type one, except that the maximum load in the cycling phase was increased to about 250% of the design load, or about 80% of the ultimate capacity. Speed of loading was kept the same.

Figure C.6 below shows a typical result. As seen in the figure, the effect is clear enough in showing an accumulative deterioration. Plastic permanent deformations are reached from the first cycle, and the specimen fails after just a few loops. As in all other cases, failure occurs

[1]the ultimate load reduced by the material factor calculated following eq.C.(1).

within the first mm inside bamboo, however, in this case far more deterioration is observable in the matrix of lignin. It was thought that perhaps the total amount of deformation, and not necessarily the amount of stress was the source of this early failure, so the next experiment was set to observe if that was the case when the stresses were kept under the design value.

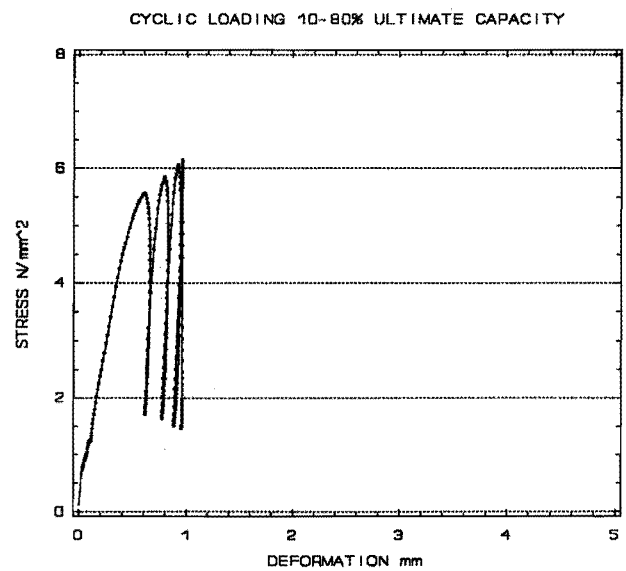

Figure C.6:Cyclic load for second load case.

C.3.3-Load type three

The third group of specimens was subjected to a completely reversing signal, keeping the same speed of loading, with amplitudes between -100% and +100% of the design value. Again, after 20 cycles results were compared to those of the reference group. No significant difference could be observed between the capacities or the ultimate deformations. A typical result can be seen in figure C.7.

Appendix C: Experimental analysis of bamboo glued phases

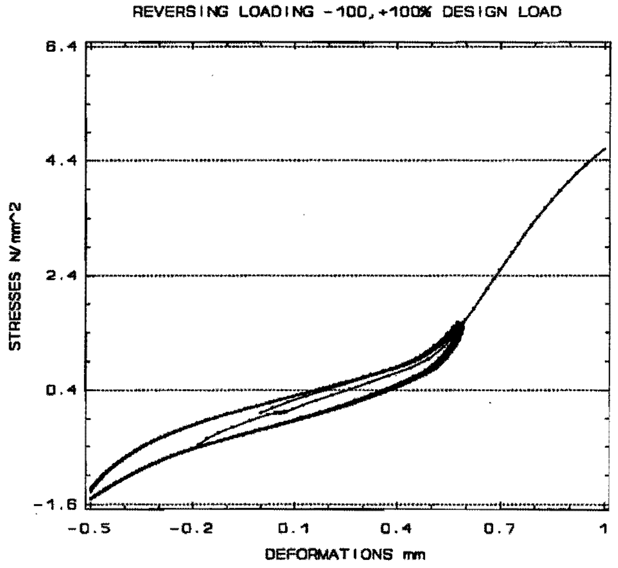

Figure C.7:Cyclic load for third load case.

C.3.4-Load type four.

As reported in reference 1 chapter four, certain load signals were chosen after the examination of several earthquake registers. In principle the idea was to try to replicate as closely as possible the type of frequency trains that may be typical of the initial phase of seismic signals, the intensive phase, and the final one.

It was determined that registers could be represented by the extreme conditions of frequency and amplitude indicated in figure C.8, but because of equipment limitations only the second signal could be implemented, with restrictions in the possibilities of data acquisition. Only time load records were possible, and only realtime plots of that were possible, so no data files were kept for post-processing of the results.

Actually, in the end, each specimen was subjected to a different load pattern in a random way, with some level of control over the peak amplitude of the signal only. In some cases the experiment was allowed to run up to 100 cycles in a kind of random combination of amplitudes between 40 and 100% of the design load, at approximately 0.5 hertz. In any case, since no damage was observable in the specimens, they were all loaded to failure afterwards, and the capacities and deformations were compared to those of the reference group. Again, no significant difference was found between the two groups.

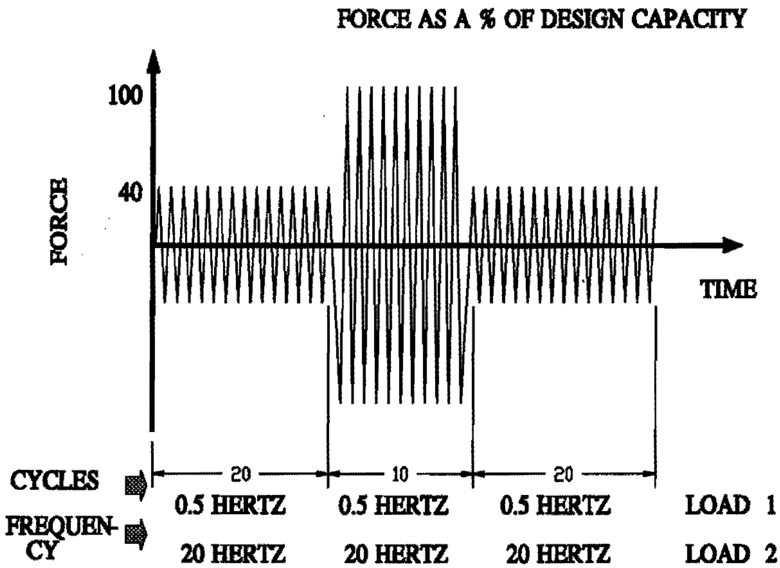

Figure C.8:High frequency signals.

C.4-General Remarks

A large number of observations were additionally made in the laboratory, related to the response of gluing phases to dynamic loads, beyond the initial goal of the experiments. **Results are, clearly, indicative, including the calculation of the design level of stresses. The major goal of this phase of the project was to study the feasibility of gluing in bamboo construction, and not the prescription of design parameters, because to reach**

that point more studies are necessary.

One more example of laboratory observations is included in figure C.9 below.

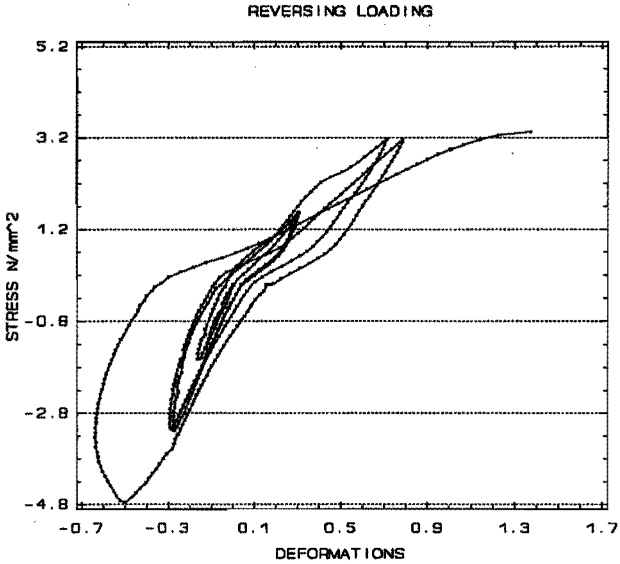

Figure C.9: Response of a glued phase to different amplitudes.

This specimen was subjected first to two cycles of -100,+100% of the design load, then two cycles of -200,+200% of the design load, and then loaded to failure. The plot shows how deterioration happens soon after stresses get closer to expected non-dynamic strengths.

But after the observation of the results of the different types of loads, evidence suggests that up to the design value glued phases are able to sustain cyclic and repetitive loading without significant loss of properties, though, again, somewhere above this limit deterioration happens after a very low number of cycles, even at very low speeds of loading. The same evidence suggests that the speed of loading is not a critical condition in the testing of this type of specimen. It can be concluded that gluing can be an interesting option for the solution of certain problems pertaining the use of bamboo for applications like furniture making, but in construction activities as well, though more facts are shown regarding this respect in appendix D.

Finally, some tests were run to observe the behaviour of full-scale connections made by gluing, following the design proposal described in chapter 5. Some traditional connections were also tested to have a reference.

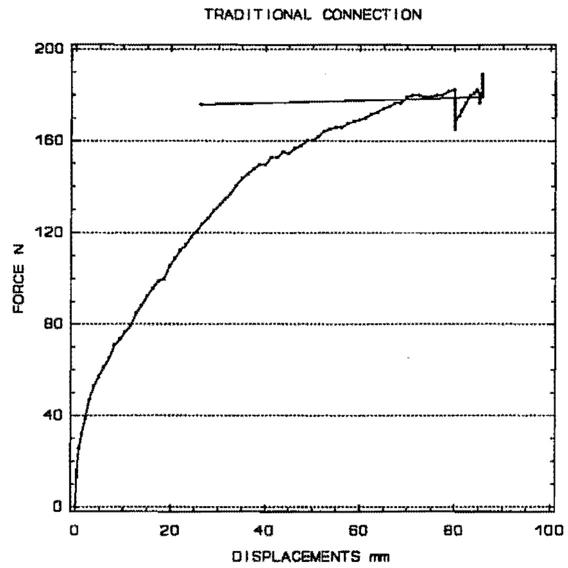

Figure C.10:Force deformation curve for a traditional connection.

All tests were done on cantilever specimens. Traditional connections were made using galvanized wire 3 mm in diameter, wound around the culm and a wooden support, as shown in figure C.12. The surfaces of contact between the culm and the support were made to perfectly fit each other, to avoid undesirable concentration of stresses. Figure C.10 shows an example of one of the resulting force deformation diagrams. It is noticeable that a large amount of rotation happened during the test. Actually, the culm became loose, being unable to sustain any reversion of loads, deformation at the ends of the culm were very large under a comparatively low load, as seen in the figure.

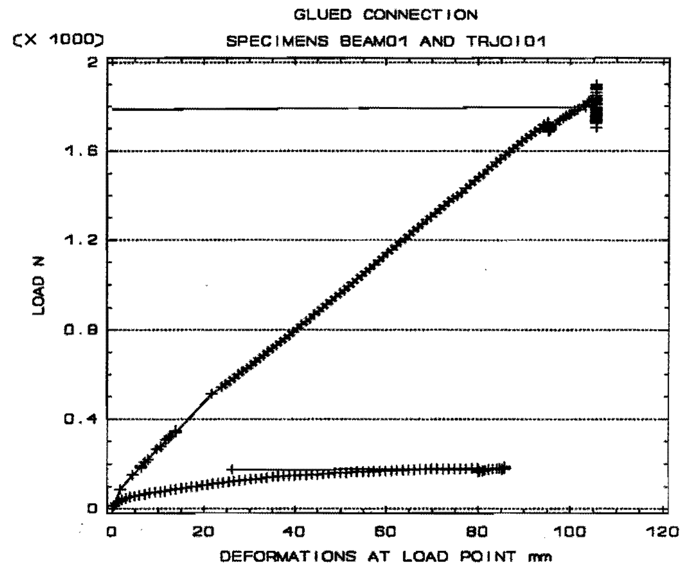

Figure C.11:Force deformation for glued (upper) and traditional (lower) supports.

Figure C.11 shows one example of a force deformation plot for a glued connection (as indicated in figure C.13), together with the previously shown result, that is included here to facilitate the comparison. The straightness of the glued connection plot indicates the small degree of or nonexistent rotation at the support up until the failure load was reached.

To confirm this, a plot of maximum strains at 20 mm from the support is presented in figure C.14. There, one can see that continuity is achieved to a good level, with no observable participation of rotations in the end deformation of the culm.

Figure C.15 shows maximum strains at sections located 20 mm from the end of the wooden support, one of them in the support itself, and the other on the free standing part of the culm. The distance between the two sections is comparatively small, so the difference between the two levels of strain is due to the change in the second moment of area, but it can be seen that plots remain straight, indicating little rotation in this other example. Tested beams had a gluing length of about two times the diameter, which, as will be shown in appendix D, should leave no room for failure in the connection, as happened in reality. Failure occurred by compression. Maximum strains were well within the same range of those observed during compression tests for bamboo, as indicated in chapter 3 and appendix B.

Table C.4 contains a description of the two beams presented in the previous plots .

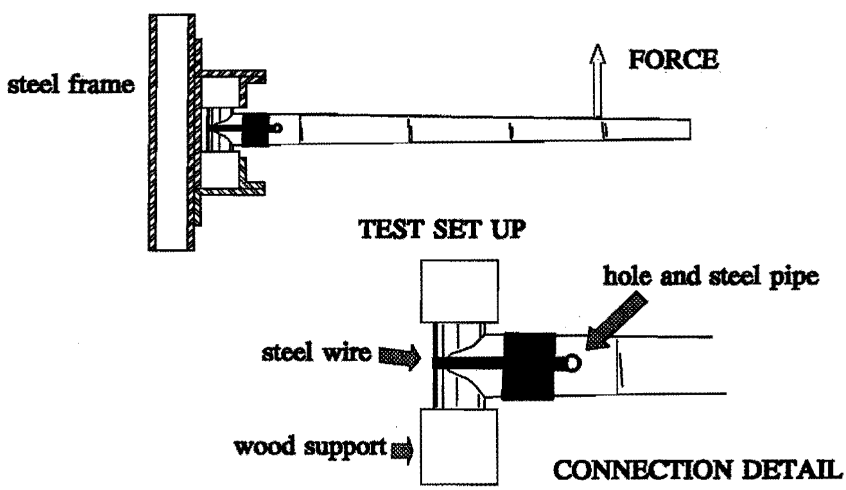

Figure C.12: Traditional connection test set-up.

specimen	External diameter [mm]	thickness [mm]	length [mm]	E [N/mm^2]
Traditional	86.68	6.98	1500	18627
Glued	84.44	7.60	1485	18303

Table C.4: Properties of the compared beams.

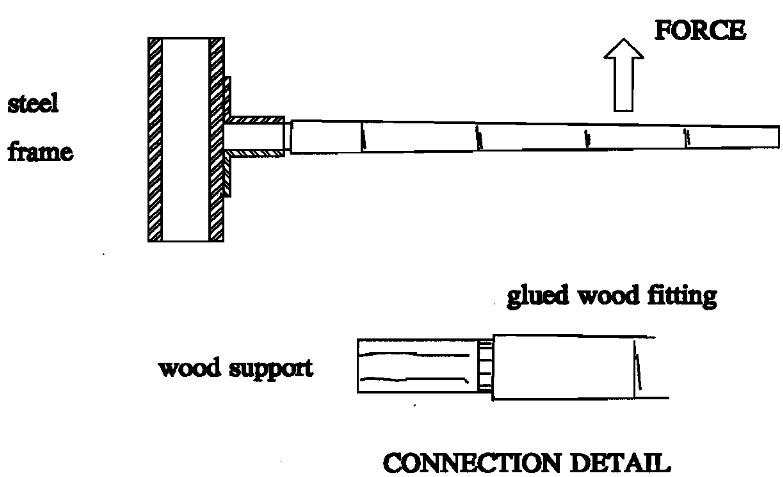

Figure C.13: Glued connection test.

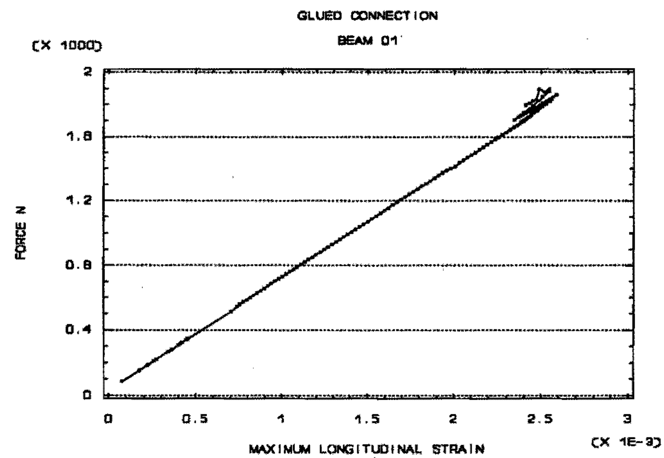

Figure C.14: Glued connection, beam 01, maximum strains at 20 mm from the support on the free standing culm.

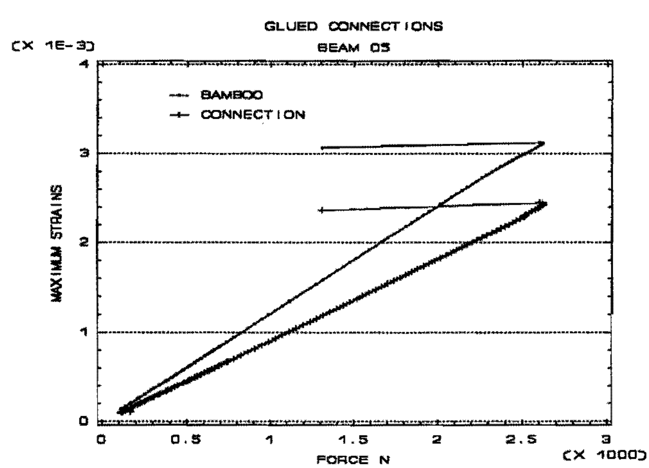

Figure C.15:Comparison of strains at both sides of the limit between the support region and the culm.

Appendix C: Experimental analysis of bamboo glued phases

REFERENCES

1 Huybers, P.; 1990. Thin Poles of roundwood for Structural Engineering Applications in Building. Structural Engineering Review,**2**, pp.169-182.

Appendix D:Glued connectors for bamboo.

D.1-Calculation of bonding stresses

D.1.1-Basic equations of elasticity

The distribution of bonding stresses, on the surface of contact between the two materials, is examined in the following sections, according to the reference system of figure D.1.

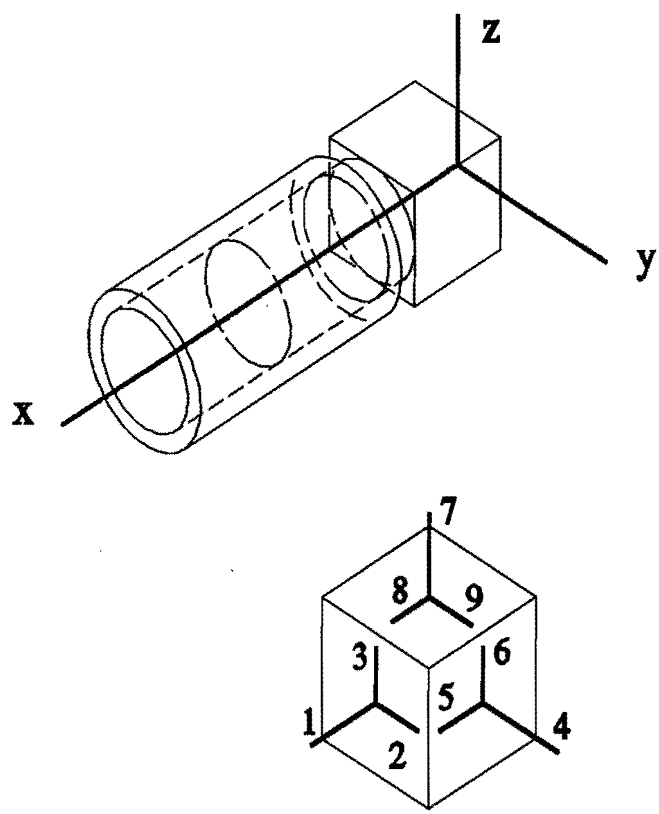

Figure D.1:Reference system.

The analysis is based on the following basic assumptions :

1- Bernoulli's assumptions hold true

2- Each material is homogenous and isotropic

3- There is no slipping in the interface between the two materials

4- Sections are perfectly circular

5- There are no discontinuities in the glued surfaces

6- The maximum strain capacity of bamboo is predictable

7- The three materials behave elastically in the range of analysis

Stresses on the surfaces of the infinitesimal element of figure D.1 (bottom) are

$$1:\ \sigma_x + \frac{\partial \sigma_x}{\partial x} \quad 2:\ \tau_{xy} + \frac{\partial \tau_{xy}}{\partial x} \quad 3:\ \tau_{xz} + \frac{\partial \tau_{xz}}{\partial x}$$

$$4:\ \sigma_y + \frac{\partial \sigma_y}{\partial y} \quad 5:\ \tau_{yx} + \frac{\partial \tau_{yx}}{\partial y} \quad 6:\ \tau_{yz} + \frac{\partial \tau_{yz}}{\partial y}$$

$$7:\ \sigma_z + \frac{\partial \sigma_z}{\partial z} \quad 8:\ \tau_{zx} + \frac{\partial \tau_{zx}}{\partial z} \quad 9:\ \tau_{zy} + \frac{\partial \tau_{zy}}{\partial z}$$

For an infinitesimal element in the body of the connection, as the one shown in figure D.1 (bottom), the following well known equations of elasticity are examined (see for example, Timoshenko,1984).

If no body forces are present then the state of equilibrium of the element is given by the relationships

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + \frac{\partial \tau_{zx}}{\partial z} = 0 \qquad \text{eq.D.(1)}$$

$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \tau_{zy}}{\partial z} = 0 \qquad \text{eq.D.(2)}$$

$$\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} = 0 \qquad \text{eq.D.(3)}$$

On the other hand, for a particular element on the external boundary of the connection it holds that

$$n_x = \sigma_x l + \tau_{yx} m + \tau_{zx} n \qquad \text{eq.D.(4)}$$

$$n_y = \sigma_y m + \tau_{zy} n + \tau_{xy} l \qquad \text{eq.D.(5)}$$

$$n_z = \sigma_z n + \tau_{xz} l + \tau_{yz} m \qquad \text{eq.D.(6)}$$

Compatibility of deformations is given by

$$\frac{\partial^2 \epsilon_x}{\partial y^2} + \frac{\partial^2 \epsilon_y}{\partial x^2} = \frac{\partial^2 \gamma_{yx}}{\partial y \partial x} \qquad \text{eq.D.(7)}$$

$$\frac{\partial^2 \epsilon_y}{\partial z^2} + \frac{\partial^2 \epsilon_z}{\partial y^2} = \frac{\partial^2 \gamma_{yz}}{\partial y \partial z} \qquad \text{eq.D.(8)}$$

$$\frac{\partial^2 \epsilon_z}{\partial x^2} + \frac{\partial^2 \epsilon_x}{\partial z^2} = \frac{\partial^2 \gamma_{xz}}{\partial x \partial z} \qquad \text{eq.D.(9)}$$

$$\frac{2\partial^2 \epsilon_x}{\partial y \partial z} = \frac{\partial}{\partial x}\left(-\frac{\partial \gamma_{yz}}{\partial x} + \frac{\partial \gamma_{xz}}{\partial y} + \frac{\partial \gamma_{xy}}{\partial z}\right) \qquad \text{eq.D.(10)}$$

$$2\frac{\partial^2 \epsilon_y}{\partial x \partial z} = \frac{\partial}{\partial y}\left(\frac{\partial \gamma_{yz}}{\partial x} - \frac{\partial \gamma_{xz}}{\partial y} + \frac{\partial \gamma_{xy}}{\partial z}\right) \qquad \text{eq.D.(11)}$$

$$2\frac{\partial^2 \epsilon_z}{\partial x \partial y} = \frac{\partial}{\partial z}\left(\frac{\partial \gamma_{yz}}{\partial x} + \frac{\partial \gamma_{xz}}{\partial y} - \frac{\partial \gamma_{xy}}{\partial z}\right) \qquad \text{eq.D.(12)}$$

Under a state of only axial stresses

Appendix D: Glued connectors for bamboo

$$\sigma_z = \sigma_y = \tau_{zy} = 0 \qquad \text{eq.D.(13)}$$

Bending stresses follow the rule

$$\sigma_x = \frac{M_y z}{I_{eq}} \qquad \text{eq.D.(14)}$$

I_{eq} is the equivalent second moment of area of the two material section, and will be dealt with later.

Substitution of the above values in equations D.1 to D.3 implies that

$$\frac{\partial \tau_{zx}}{\partial x} = 0 \qquad \text{eq.D.(15)}$$

([1])

$$\frac{\partial \tau_{yx}}{\partial x} = 0 \qquad \text{eq.D.(16)}$$

$$\frac{z}{I_{eq}} M_y{}' + \frac{\partial \tau_{zx}}{\partial z} + \frac{\partial \tau_{yx}}{\partial y} = 0 \qquad \text{eq.D.(17)}$$

Since forces are zero on the surface of the connection, then

$$\tau_{xz} l + \tau_{yz} m = 0 \qquad \text{eq.D.(18)}$$

Now, because $l = \frac{dy}{ds}$ $\quad m = \frac{-dx}{ds}$ then

[1]Note that this is true when the length of the connection is small compared to the length of the culm.

$$\tau_{xz}\frac{dy}{ds} - \tau_{yz}\frac{dx}{ds} = 0 \qquad \text{eq.D.(19)}$$

Using Hooke's law, the equations of compatibility can now be written as

$$(1+v)\nabla^2\sigma_z + \frac{\partial^2\Theta}{\partial z^2} = 0 \qquad \text{eq.D.(20)}$$

$$(1+v)\nabla^2\sigma_y + \frac{\partial^2\Theta}{\partial y^2} = 0 \qquad \text{eq.D.(21)}$$

$$(1+v)\nabla^2\sigma_x + \frac{\partial^2\Theta}{\partial x^2} = 0 \qquad \text{eq.D.(22)}$$

$$(1+v)\nabla^2\tau_{yx} + \frac{\partial^2\Theta}{\partial y\partial x} = 0 \qquad \text{eq.D.(23)}$$

$$(1+v)\nabla^2\tau_{zx} + \frac{\partial^2\Theta}{\partial z\partial x} = 0 \qquad \text{eq.D.(24)}$$

$$(1+v)\nabla^2\tau_{zy} + \frac{\partial^2\Theta}{\partial z\partial y} = 0 \qquad \text{eq.D.(25)}$$

$$\frac{\partial\Theta}{\partial z} = \frac{M_y'}{I_{eq}} \qquad \text{eq.D.(26)}$$

$$\frac{\partial^2\Theta}{\partial z\partial x} = \frac{1}{I_{eq}}M_y' \qquad \text{eq.D.(27)}$$

This set of equations reduces to

$$\nabla^2\tau_{yx} = 0 \qquad \text{eq.D.(28)}$$

and

$$\nabla^2\tau_{zx} = \frac{M_y'}{(\nu+1)I_{eq}} \qquad \text{eq.D.(29)}$$

It can be seen that the equations of equilibrium are satisfied if

$$\tau_{zx} = \frac{\partial\varphi}{\partial y} + \frac{M_y' z^2}{2I_{eq}} + f(y) \qquad \text{eq.D.(30)}$$

and

$$\tau_{yx} = -\frac{\partial\varphi}{\partial z} \qquad \text{eq.D.(31)}$$

φ is a stress function on y and z, and $f(y)$ depends on the boundary conditions.

D.1.2-Stress due to shear.

Transversal forces generate shear stresses that affect bonding stresses. This effect can be accounted for in the following way.

Back-substitution of equations D.(30) and D.(31) on equations D.28 and D.29 gives

$$\frac{\partial\varphi}{\partial y}\frac{dy}{ds} + \frac{\partial\varphi}{dz}\frac{dz}{ds} = \frac{\partial\varphi}{\partial s} \qquad \text{eq.D.(32)}$$

$$\frac{\partial\varphi}{\partial s} = \left[\frac{M_y' z^2}{2\, I_{eq}} - f(y)\right]\frac{dy}{ds} \qquad \text{eq.D.(33)}$$

Since φ must be constant along the boundary, then $f(y)$ must be such that the right side of equation D.33 becomes zero. If the attention is now concentrated on the glued region of the connection, then, for a circular section of external radius r_e, the external boundary is defined by

$$z^2 + y^2 = r_e^2 \qquad \text{eq.D.(34)}$$

Therefore

$$f(y) = \frac{M_y' \,(r_e^2 - y^2)}{2\, I_{eq}} \qquad \text{eq.D.(35)}$$

and thus

$$f'(y) = -\frac{M_y' y}{I_{eq}} \qquad \text{eq.D.(36)}$$

Appropriate substitutions show that

$$\frac{\partial^2\varphi}{\partial z^2} + \frac{\partial^2\varphi}{\partial y^2} = \frac{\nu}{1+\nu}\frac{M_y' y}{I_{eq}} + \frac{M_y' y}{I_{eq}} \qquad \text{eq.D.(37)}$$

or

$$\frac{\partial^2\varphi}{\partial z^2}+\frac{\partial^2\varphi}{\partial y^2}=\left(\frac{1+2\nu}{1+\nu}\right)\frac{M_y^{\prime}y}{I_{eq}} \qquad \text{eq.D.(38)}$$

A solution of this equation is

$$\varphi=\frac{1}{8}\left(\frac{1+2\nu}{1+\nu}\right)\frac{M_y^{\prime}}{I_{eq}}\left(z^2+y^2-r_e^2\right)y \qquad \text{eq.D.(39)}$$

In order to obtain stress values and taking the convenience of using know polar coordinates into account, this boundary function can be now back substituted in D.30 and D.31 to obtain

$$\tau_{zx}=\frac{(3+2\nu)}{8\ (1+\nu)}\ \frac{M_y^{\prime}}{I_{eq}}\left[r_e^2-r^2\cos^2\theta\frac{1-2\nu}{3+2\nu}\ r^2\sin^2\theta\right] \qquad \text{eq.D.(40)}$$

$$\tau_{yx}=-\frac{1+2\nu}{4(1+\nu)}\frac{M_y^{\prime}}{I_{eq}}r^2\cos\theta\ \sin\theta \qquad \text{eq.D.(41)}$$

Forces can now be calculated for an elemental sector such that

$$dA=r_i dr d\theta$$

If a unit arc is defined on the gluing surface, then $dA = dr$ and the force acting on such elementary sector is

$$dv_{zx}dx = \frac{3+2\nu}{8(1+\nu)}\frac{M_y'}{I_{eq}}\left[r_e^2 - r^2\cos^2\theta - \frac{1-2\nu}{3+2\nu}r^2\sin^2\theta\right]dr \qquad \text{eq.D.(42)}$$

$$dv_{yx}dx = -\frac{1+2\nu}{4(1+\nu)}\frac{M_y'}{I_{eq}}r^2\cos\theta\ \sin\theta dr \qquad \text{eq.D.(43)}$$

After double integration the above equations become

$$v_{zx} = \frac{3+2\nu}{8(1+\nu)}\frac{V_x}{I_{eq}}\left[r_e^2(r_i - r_e) - \frac{1}{3}\left(r_i^3 - r_e^3\right)\cos^2\theta - \frac{1}{3}\frac{1-2\nu}{3+2\nu}\left(r_i^3 - r_e^3\right)\sin^2\theta\right] \qquad \text{eq.D.(44)}$$

and

$$v_{yx} = -\frac{(1+2\nu)}{4(1+\nu)}\frac{V_x}{I_{eq}}\left(\frac{r_i^3 - r_e^3}{3}\right)\cos\theta\ \sin\theta \qquad \text{eq.D.(45)}$$

It is convenient to convert shear stresses into tangential and normal stresses on the surface of contact between the two materials, which can be done as

$$v_N = v_{zx}\cos\theta + v_{yx}\sin\theta \qquad \text{eq.D.(46)}$$

$$v_T = v_{zx}\sin\theta + v_{yx}\cos\theta \qquad \text{eq.D.(47)}$$

After proper substitutions, the above two equations become

$$v_N = \frac{\frac{3+2\nu}{8(1+\nu)}\frac{V_x}{I_{eq}}\left[r_e^2(r_i-r_e)\cos\theta-\frac{1}{3}(r_i^3-r_e^3)\left[\cos^3\theta + \left(\frac{1-2\nu}{3+2\nu}+2\frac{1+2\nu}{3+2\nu}\right)\cos\theta\sin^2\theta\right]\right]}{} \quad \text{eq.D.(48)}$$

$$v_T = \frac{\frac{3+2\nu}{8(1+\nu)}\frac{V_x}{I_{eq}}\left[r_e^2(r_i-r_e)\ \sin\theta-\frac{1}{3}(r_i^3-r_e^3)\right.}{\left(\cos^2\theta\ \sin\theta\left(1+2\frac{1+2\nu}{3+2\nu}\right)-\left(\frac{1-2\nu}{3+2\nu}\sin^3\theta\right)\right]} \quad \text{eq.D.(49)}$$

The maximum values of these shear forces occur at the value of θ, for which the first derivatives of the above equations are zero. It can be shown that the maximum normal force occurs at $\theta = 0$ and it takes the value of v_{zx}, and that the maximum value of the tangential force occurs at $\theta = \frac{\pi}{2}$, and that it takes the value of v_{zx}. Now, in order to see the significance of these stresses, a parametric analysis is undertaken in the next section.

D.1.3-Effect of bending on the value of bonding stresses

Under a condition of pure bending, stresses acting on an elementary sector will be like those indicated in figure D.2, where

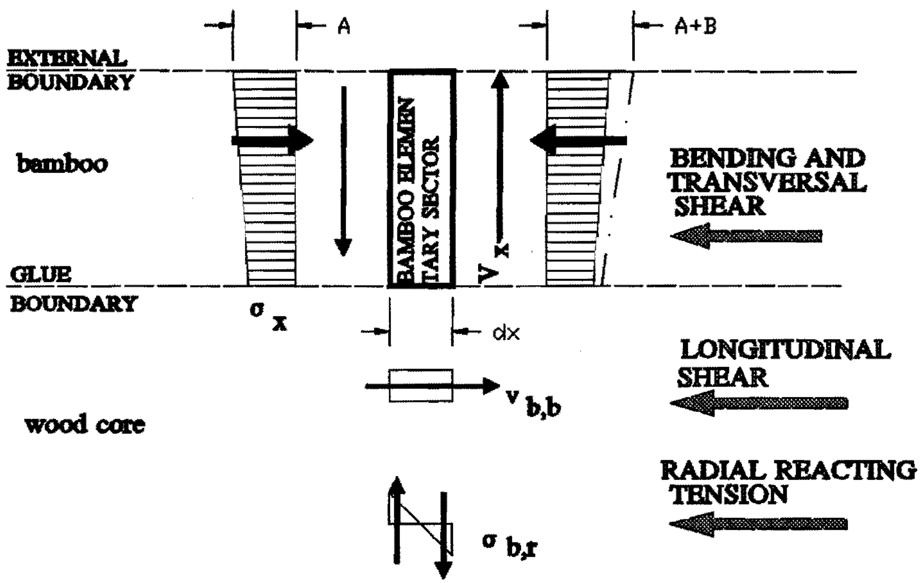

Figure D.2:Stresses on an elementary bamboo sector.

$$A = \epsilon_x \; E_m \qquad \text{eq.D.(50)}$$

$$E_m = [E_w \vee E_b] \qquad \text{eq.D.(51)}$$

$$B = \Delta\epsilon_x \; E_m \qquad \text{eq.D.(52)}$$

The total resistant moment of the section can be calculated by taking moments around the neutral axis, so that

$$dM_y = dr\,dS \frac{\epsilon_{max}\, r \sin\theta}{\frac{\phi}{2}} E_m r \sin\theta \qquad \text{eq.D.(53)}$$

Since

$$dS = r d\theta$$

then the total moment is given by

$$M_y = \int_0^{r_w}\int_0^{2\pi} \frac{\epsilon_m\, r^3 \sin^2\theta}{r_e} E_w dr d\theta + \int_{r_w}^{r_e}\int_0^{2\pi} \frac{\epsilon_m\, r^3 \sin^2\theta}{r_e} E_b dr d\theta \qquad \text{eq.D.(54)}$$

which after integration and grouping results in

$$M_y = \frac{\pi\epsilon_m}{r_e}\left[\frac{r_i^4}{4}E_w + \left(\frac{(r_i+t)^4}{4} - \frac{r_i^4}{4}\right)E_b\right] \qquad \text{eq.D.(55)}$$

The term in brackets is the equivalent stiffness of the composed section of two materials. Equation D.55 can be simplified as

$$M_y = \frac{\epsilon_m}{r_e} E_b\, I_{eq} \qquad \text{eq.D.(56)}$$

where

$$I_{eq} = \frac{I_w}{p} + I_b \qquad \text{eq.D.(57)}$$

and

Appendix D: Glued connectors for bamboo

$$p = \frac{E_b}{E_w}$$ eq.D.(58)

The maximum strain in the section is given by

$$\epsilon_m = \frac{M_y r_e}{E_b I_{eq}}$$ eq.D.(59)

and therefore

$$\sigma_m = \frac{M_y r_e}{I_{eq}}$$ eq.D.(60)

Axial stresses of a particular point on the section can thus be calculated as

$$\sigma_x = \frac{M_y r \cos\theta}{I_{eq}}$$ eq.D.(61)

The corresponding force, after integration works out to be

$$N_x = \frac{M_y}{2\, I_{eq}} \cos\theta \left(r_i^2 - r_e^2\right)$$ eq.D.(62)

(force per unit sector)

Along the length of glue, the bonding stress will be

$$v_{b,b} = \frac{\partial N_x}{\partial x}$$ eq.D.(63)

Therefore

$$v_{b,b} = \frac{\cos\theta\left(r_i^2 - r_e^2\right)}{2\ I_{eq}} V_x$$ eq.D.(64)

D.2-Parametric study

Since normal and tangential forces reach their maximum at $\theta = 0$ and $\theta = \frac{\pi}{2}$

respectively, then

$$v_N = \frac{3+2\nu}{8(1+\nu)} \frac{V_x}{I_{eq}} \left[r_e^2 (r_i - r_e) - \frac{1}{3}\left(r_i^3 - r_e^3\right) \right]$$ eq.D.(65)

and

$$v_T = \frac{3+2\nu}{8(1+\nu)} \frac{V_x}{I_{eq}} \left[r_e^2 (r_i - r_e) - \frac{1}{3} \frac{1-2\nu}{3+2\nu} \left(r_i^3 - r_e^3\right) \right]$$ eq.D.(66)

It can be assumed that $\nu = 0{,}3$. On the other hand, the author found that for *Bambusa Blumeana* from the Philippines, and for *Guadua s.p.* from Costa Rica, $t = 0{,}09\ \phi$ ([2]),or $r_i = 0{,}82\ r_e$. Substitution of these values gives

$$v_T = \frac{-0{,}04\, V_x}{\pi\, r_e \left[0{,}45 - \frac{p-1}{p} \right]} \qquad (\theta = 0)$$ eq.D.(67)

[2]See appendix G.

In the same way it can be shown that

$$v_T = \frac{-0{,}23\, V_x}{\pi\, r_e \left[0{,}45 - \frac{p-1}{p}\right]} \qquad \left(\theta = \frac{\pi}{2}\right) \qquad \text{eq.D.(68)}$$

and

$$v_{b,b} = \frac{-0{,}65\, V_x}{\pi\, r_e^2 \left[0{,}45 - \frac{p-1}{p}\right]} \qquad (\theta = 0) \qquad \text{eq.D.(69)}$$

Equilibrium indicates that

$$\sigma_b = v_{b,b} \qquad (\theta = 0) \qquad \text{eq.D.(70)}$$

and

$$\sigma_b \frac{l_g}{2} \frac{1}{2} = \frac{0{,}23\, V_x}{\pi\, r_e \left[0{,}45 - \frac{p-1}{p}\right]} \qquad \left(\theta = \frac{\pi}{2}\right) \qquad \text{eq.D.(71)}$$

This equation can be reordered as

$$\frac{l_g}{r_e} = \frac{0{,}2944}{\left[0{,}45 - \frac{p-1}{p}\right]} \frac{v_{adm}}{\sigma_{b_{adm}}} \qquad \text{eq.D.(72)}$$

The extreme right term of the equation is the ratio of maximum design values, for the

transversal shear in the culm, and bonding capacity in the glue, respectively.
It is convenient to see the effect of changes in the parameters involved, what can be done via a plot like shown in figure D.3.

It can be seen that for average material properties a glue length of about 1.5 times the external diameter of the culm will guarantee that failure will not occur within the region of the connection due to bending alone.

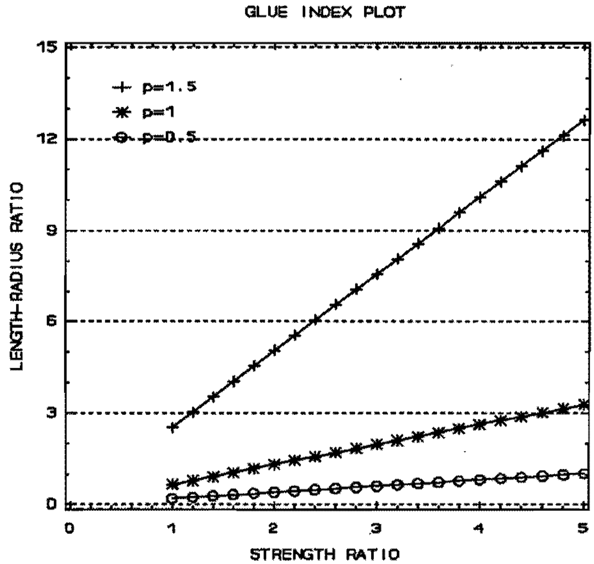

Figure D.3:Parametric analysis of equation D.72.
$p=E_t/E_w$.

D.3-Stresses due to pure tension or pure compression

If stresses are to be kept equal or below the design values found in appendix C, then it is reasonable to assume a linear distribution along the length of glue. In that case the calculation of stresses becomes very simple, because their average value can be obtained by dividing the total tensile force by the area of the connection.

REFERENCES :

1 Timoshenko,S.P.; Goodier,J.N., 1984. Theory of Elasticity. Third Edition. Mc. Graw Hill International Book Company.

Appendix E: Bamboo columns

E.1-Calculation of the critical load

E.1.1-Introduction

In what follows the critical load of bamboo columns is investigated. The role of the geometry and of the changing mechanical properties of the culm is analysed. The influence of the nodes is not studied in this chapter.

A direct link is established between the theoretical analysis and the experimental phase, since both are based upon an adaptation of the Southwell Plot procedure, which enables the experimenter to take care of the problem of initial deformations during testing. A detailed description is given in the following sections.

Since the measuring procedure is crucial for good results, a complete description is given of two options devised during the development of this study.

E.1.2-Influence of initial twisting deformations

As experience shows, bamboo culms are naturally twisted to some extent. Fibres run almost parallel to each other within the internode region, but they are not necessarily parallel from internode to internode, so that this initial condition leads to twisting under axial loading.

It is well-known that twisting and lateral deformation participate in an uncoupled manner in a column, as will be shown in the following lines.

The initial deformation can be described by a sine series of the form

$$\phi(x)=\sum_{n=1}^{\infty} C_n \sin(n\pi \frac{x}{l}) \qquad \text{eq.E.(1)}$$

The figure shows how fibres elongate under the twisting of the culm. It is assumed though that straight fibres before loading remain so afterwards.

It can be seen that

$$\frac{d\Delta x}{rd\phi(x)}=\frac{rd\phi(x)}{dx-d\Delta} \qquad \text{eq.E.(2)}$$

which after elimination of second order differentials becomes

$$\frac{d\Delta x}{dx}=\left(\frac{rd\phi(x)}{dx}\right)^2 \qquad \text{eq.E.(3)}$$

or by introducing the value of the derivative,

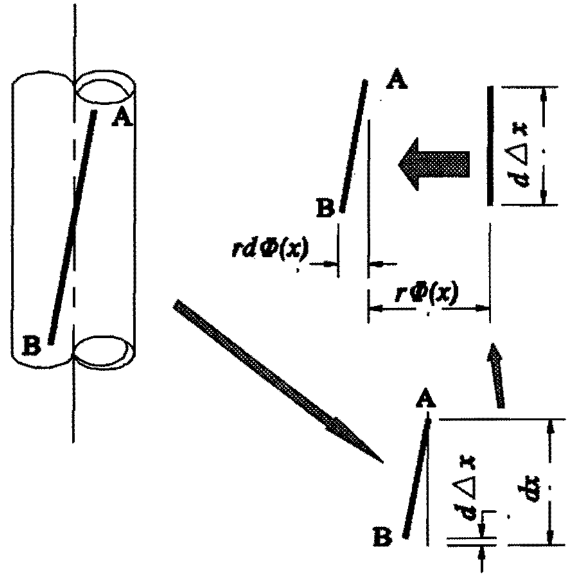

Figure E.1:Geometric considerations on twisting.

$$\frac{d\Delta x}{dx}=\left[r\frac{\pi}{l}\sum_{n=1}^{\infty}C_n\cos\left(n\pi\frac{x}{l}\right)\right]^2 \qquad \text{eq.E.(4)}$$

The work done by twisting is therefore

$$Td\phi = GI_p\phi(x)d\phi(x) \qquad \text{eq.E.(5)}$$

or

$$Td\phi = GI_p\sum_{n=1}^{\infty} C_n \sin\left(n\pi\frac{x}{l}\right)\frac{\pi}{l}\sum_{n=1}^{\infty} C_n \cos\left(n\pi\frac{x}{l}\right) \qquad \text{eq.E.(6)}$$

The bending moment at a distance x along the culm is

$$M = E_x I\frac{d^2(w-w_i)}{dx^2} \qquad \text{eq.E.(7)}$$

or simply

$$M = E_x\ I\ K \qquad \text{eq.E.(8)}$$

The work done by bending can be calculated as

$$dU_b = M\ d\theta \qquad \text{eq.E.(9)}$$

and since

$$d\theta = K\ dx \qquad \text{eq.E.(10)}$$

this results in

$$U_b = \frac{1}{2}E_x I\int_0^l \frac{d^2(w-w_i)}{dx^2}dx \qquad \text{eq.E.(11)}$$

On the other hand the work done by the axial load is, using eq.E.4

$$dU_x=\frac{N_x}{2}\left[\left(\frac{dw}{dx}\right)^2-\left(\frac{dw_i}{dx}\right)^2\right]dx+\left[r\frac{\pi}{l}\sum_{n=1}^{\infty}C_n\cos\left(n\pi\frac{x}{l}\right)\right]^2dx \qquad \text{eq.E.(12)}$$

since a stationary value is looked for then

$$0=E_xI\cdot\int_0^l\left[\frac{\pi^2}{l^2}\sum_{n=1}^{\infty}n^2b_n\sin\left(n\pi\frac{x}{l}\right)\right]\cdot\left[\frac{\pi^2n^2}{l^2}\sin\left(n\pi\frac{x}{l}\right)\right]dx$$

$$+\int_0^l GI_p\sum_{n=1}^{\infty}C_n^2\left[-\sin^2\left(n\pi\frac{x}{l}\right)+\cos^2\left(n\pi\frac{x}{l}\right)\right]\frac{\pi}{l}dx \qquad \text{eq.E.(13)}$$

$$-N_x\cdot\int_0^l\left[\frac{\pi}{l}\sum_{n=1}^{\infty}nb_n\cos\left(n\pi\frac{x}{l}\right)\right]\cdot\frac{\pi m}{l}\cos\left(n\pi\frac{x}{l}\right)dx$$

$$-N_x\int_0^l \frac{-2r\pi}{l}\sum_{n=1}^{\infty}C_nn\frac{\pi}{l}\sin\left(n\pi\frac{x}{l}\right)dx$$

Since the second integral vanishes only the last one remains associated with the initial twisting deformation of the culm. If a further assumption is taken, in the sense that it is always possible to express the angle of twisting using the first term of the series only, the contribution of this term only remains important depending on the values of deformations actually occurring during loading. It is assumed here for the time being that this value is random with respect to the force and that it is very small, but this point will be discussed later on in the section on experimental results.

With this in mind and due to the fact that

$$\frac{\partial U^2}{\partial x^2}<0 \qquad \text{eq.E.(14)}$$

in order to have a maximum value of the force, then

$$0=\frac{l}{2}\frac{E_x J\pi^4 n^4}{l^4}b_n-\frac{l}{2}\frac{N_x\pi^2 n^2}{l^2}b_n \qquad \text{eq.E.(15)}$$

or

$$N_x=\frac{E_x J\pi^2}{l^2} \qquad \text{eq.E.(16)}$$

which is the first Euler load, **unaffected by the presence of initial twisting of the culm.**

E.2-Buckling load of an initially crooked tapered bar with variable modulus of elasticity

Let us suppose that the bar shown in figure E.2 is initially deformed in a random way, that can be described by the series

$$w_i=\sum_{n=1}^{\infty} a_n \sin\left(n\pi\frac{x}{l}\right) \qquad \text{eq.E.(17)}$$

Let us also suppose that the solution of the differential equation for the pin-ended strut can be approximated by

$$w = \sum_{n=1}^{\infty} b_n \sin\left(n\pi\frac{x}{l}\right) \qquad \text{eq.E.(18)}$$

Two further assumptions will be made in relation to the elasticity of the element and its cross section. First, suppose that the Young's modulus varies from the smallest section following the rule

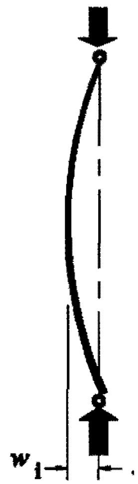

Figure E.2: Initially crooked bar.

$$E(x) = E_1 - \delta_E \frac{x}{l} \qquad \text{eq.E.(19)}$$

and second, that the moment of inertia of the section follows the rule[1]

$$I(x) = I_1 + \delta_I \, x^4 \qquad \text{eq.E.(20)}$$

it can be noticed that

[1]Actually a linear relationship would do as well, as explained in appendix G, though at the time this section of the research project was undertaken evidence for this was not available. An option in that sense is added at the end of this appendix.

$$E(l) - E(0) = -\delta_E \qquad \text{eq.E.(21)}$$

and that

$$\therefore \quad \delta_E = \Delta E \qquad \text{eq.E.(22)}$$

and in the same way

$$\delta_I \, l^4 = \Delta I \qquad \text{eq.E.(23)}$$

The critical load of the strut can be calculated by finding the maximum potential energy of the system.

The potential energy due to bending in the bar is equal to

$$U_b = -\frac{1}{2} \int_0^l E(x) I(x) \left[\frac{d^2(w-w_i)}{dx^2} \right]^2 dx \qquad \text{eq.E.(24)}$$

It also can be shown that the potential energy due to the shortening of the bar is given by

$$U_N = \frac{N_x}{2} \int_0^l \left[\left(\frac{dw}{dx} \right)^2 - \left(\frac{dw_i}{dx} \right)^2 \right] dx \qquad \text{eq.E.(25)}$$

Substitution of the respective values of the derivatives and products and the fact that the derivative of this function with respect to b_n has to be zero gives

$$0 = \int_0^l \left[\sum_{n=1}^{\infty} \frac{n^4\pi^4}{l^4} \left(E_1 I_1 \sin^2(n\pi\frac{x}{l}) + \delta_I E_1 x^4 \sin^2(n\pi\frac{x}{l}) - \delta_E I_1 \frac{x}{l} \sin^2(n\pi\frac{x}{l}) \right) (b_n - a_n) \right] dx$$

$$- \int_0^l \left[\sum_{n=1}^{\infty} \frac{n^4\pi^4}{l^4} \left(\delta_I \delta_E \frac{x^5}{l} \sin^2(n\pi\frac{x}{l}) \right) (b_n - a_n) \right] dx \qquad \text{eq.E.(26)}$$

$$- N_x \int_0^l \left[\frac{\pi}{l} \sum_{n=1}^{\infty} n b_n \cos n\pi\frac{x}{l} \right] \left[\pi\frac{n}{l} \cos m\pi\frac{x}{l} \right] dx$$

After integration and some further simplification eq.E.26 can be reduced to

$$0 = A\ (b_n - a_n) - b_n N_x B \qquad \text{eq.E.(27)}$$

where

$$\frac{A}{B} = \frac{\pi^2}{l^2}[E_1 I_1 + 0.4494(\Delta E) I_1 + 0.4786(\Delta I) E_1 - 2.616(\Delta I)(\Delta E)] \qquad \text{eq.E.(28)}$$

and

$$B = \frac{\pi^2 n^2}{2l} \qquad \text{eq.E.(29)}$$

therefore

$$b_n = \frac{A a_n}{A - N_x B} \qquad \text{eq.E.(30)}$$

The lateral deformation due to the application of the load is

$$w = b_n - a_n \qquad \text{eq.E.(31)}$$

$$\frac{w}{N_x} - \frac{a_n}{\frac{A}{B}} = \frac{w}{\frac{A}{B}} \qquad \text{eq.E.(32)}$$

which is the equation of a straight line with intersection at $\frac{A_n}{\frac{A}{B}}$ and slope $\frac{B}{A}$.

Eq.E.32 is a variation of the Southwell Plot (see for example, Gregory, 1967) procedure that accounts for variations in the second moment of area and the modulus of Young.
Further examination of the values of eq.E.27 can be done taking into account that in order to have a maximum value for the equation of energy

$$\frac{\partial U^2}{\partial x^2} < 0 \qquad \text{eq.E.(33)}$$

so that

$$0 = A - B\, N_x^{cr} \qquad \text{eq.E.(34)}$$

or

$$N_x^{cr} = \frac{A}{B} \qquad \text{eq.E.(35)}$$

Substitution of the corresponding values gives

$$N_x^{cr} = E_1 I_1 \frac{\pi^2}{l^2} + 0.4494(\Delta E) I_1 \frac{\pi^2}{l^2} + 0.4786(\Delta I) E_1 \frac{\pi^2}{l^2} - 2.616(\Delta I)(\Delta E)\frac{\pi^2}{l^2} \qquad \text{eq.E.(36)}$$

which is therefore the magnitude of the critical load of a perfectly straight bar with a tapered cross-section and variable Young's modulus, as a function of properties at the extreme of the elements and their variations.

E.3-Theoretical buckling load of bamboo culms

It can be noted that the first component of eq.E.36 corresponds to the buckling load of a bar with constant elastic properties along its length. The other three components can be regarded as corrections due to the presence of changes in these properties.

In the case of bamboo it is possible to plot the result of eq.E.36 as a sequence of graphs each representing the influence of each component for a family of values of the variations in these parameters for the ranges found in the field.

Such a set of graphs is presented in figures E.3 to E.6 in the following pages. In the figures units are N/mm^2, and

$$\lambda = \frac{1}{\sqrt{\frac{I_1}{A_1}}} \qquad \text{eq.E.(37)}$$

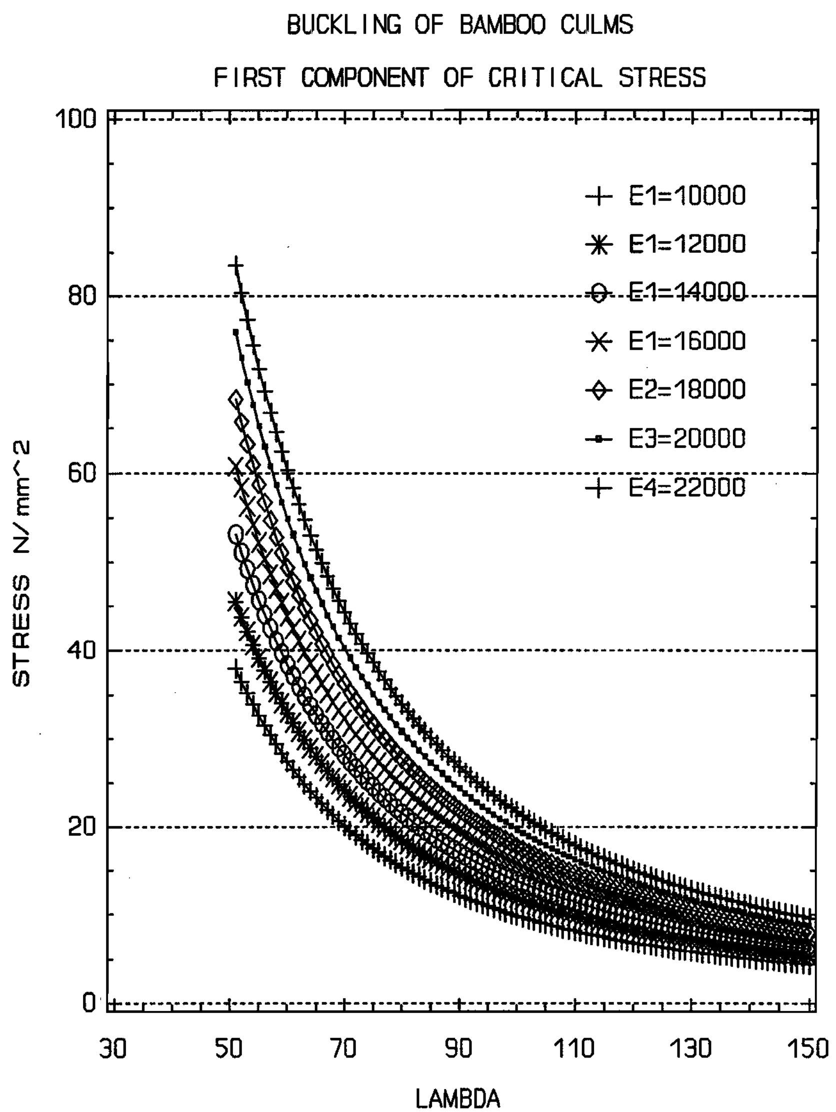

Figure E.3:First component of the critical stress for bamboo [N/mm²].

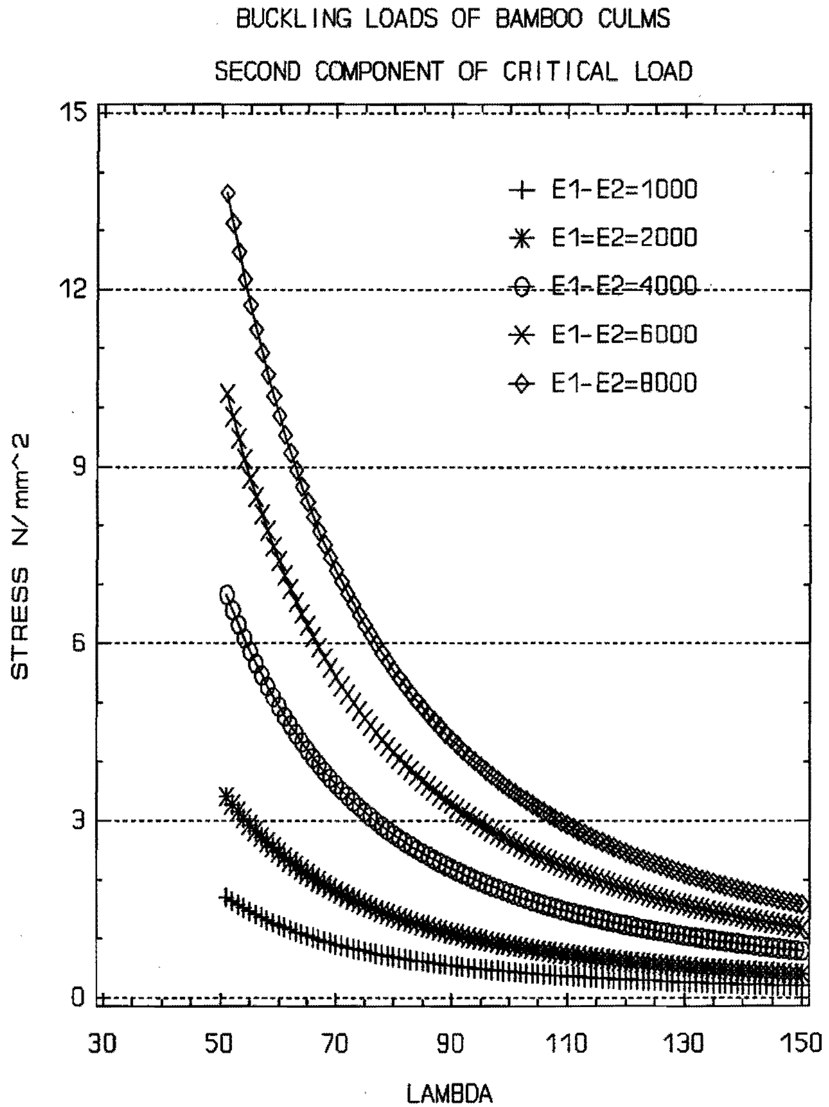

Figure E.4:Second component of critical stress[N/mm^2].

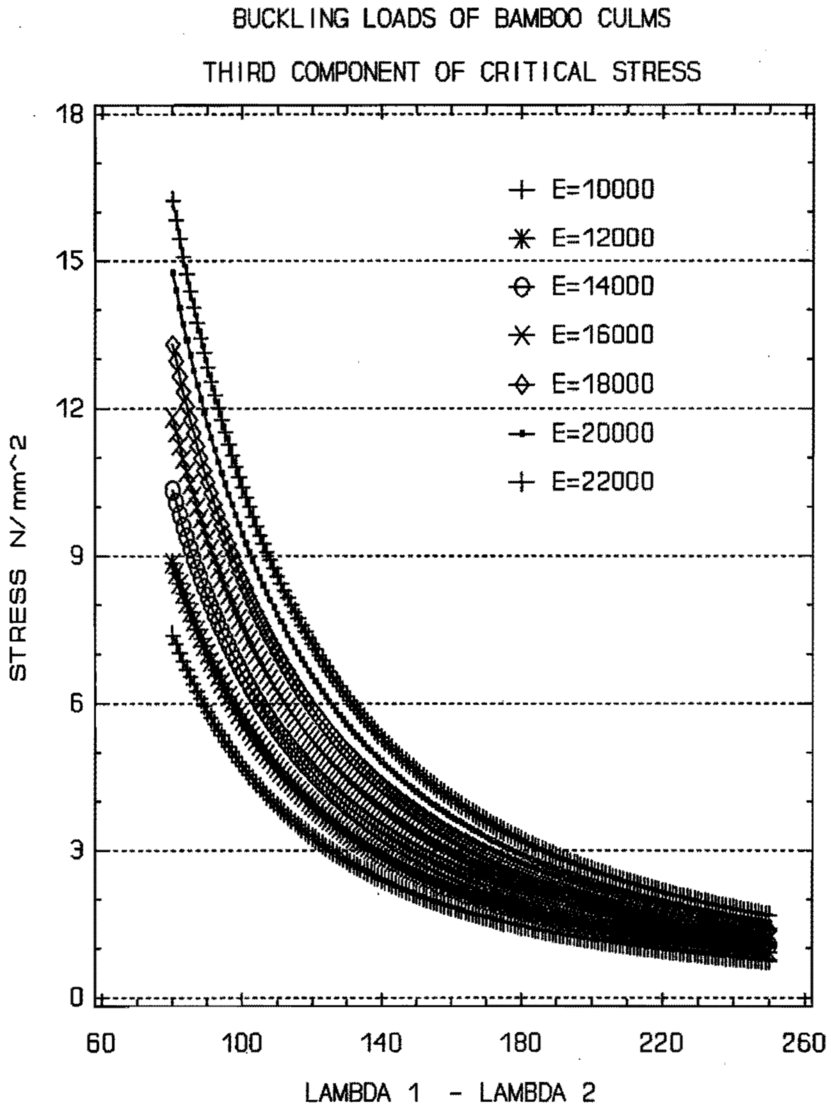

Figure E.5:Third component of critical stress [N/mm^2].

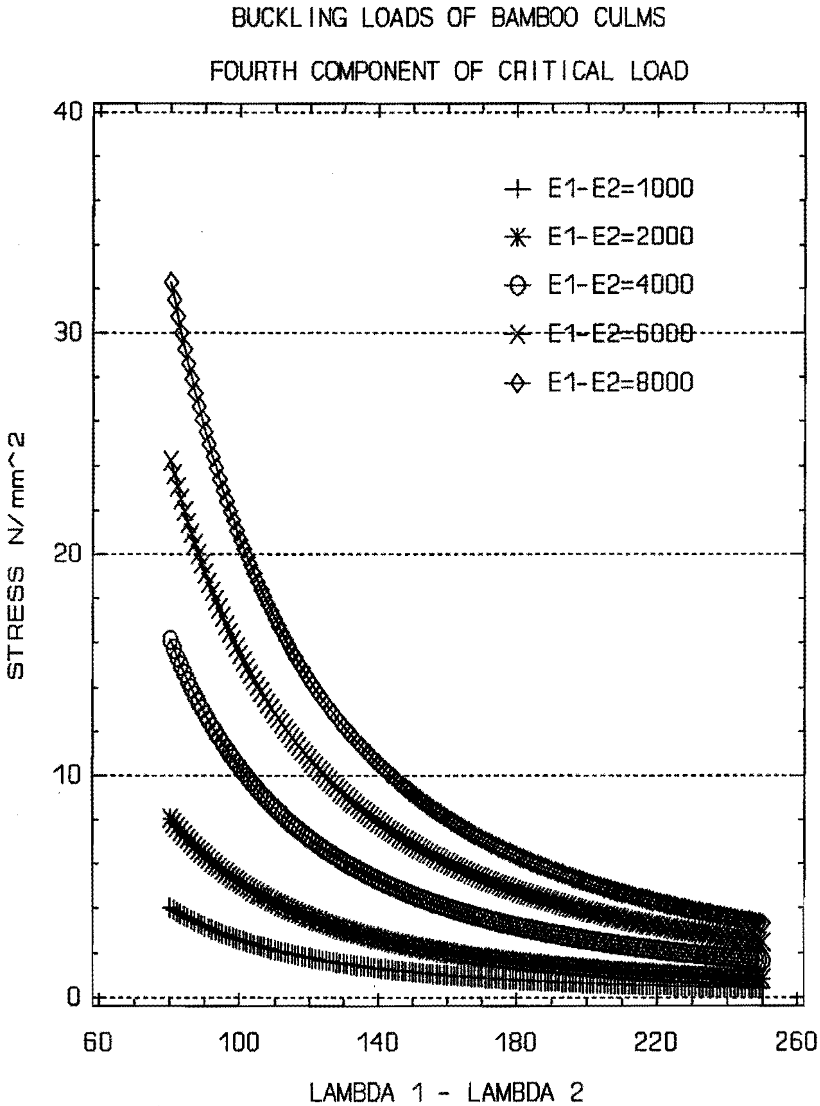

Figure E.6:Fourth component of critical stress [N/mm^2].

E.4- Buckling Experimental Analysis of Bamboo Columns

E.4.1-Theoretical basis

The value in eq.E.36 can be regarded as

$$N_x^{cr} = N_1 + N_2 + N_3 - N_4 \qquad \text{eq.E.(38)}$$

Each term represents the different contributions to the buckling load of the ideal strut.
In this way eq.E.32 can be written in a simpler way as

$$\frac{w}{N_x} - \frac{a}{N_x^{cr}} = \frac{w}{N_x^{cr}} \qquad \text{eq.E.(39)}$$

This is the equation of a so-called " Southwell Plot of Deformations", as shown in figure E.7.

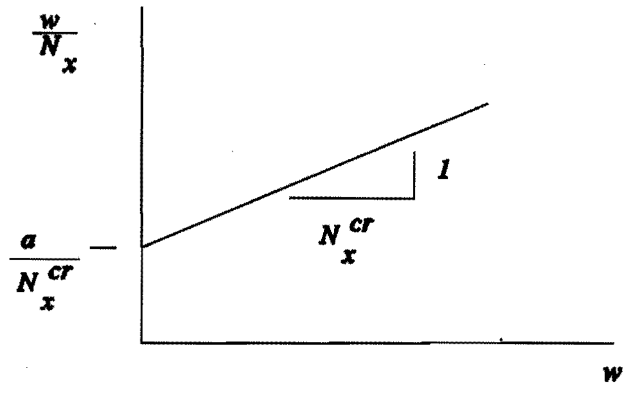

Figure E.7:Southwell Plot of deformations.

So it is possible to run an experiment on an initially crooked strut and get rid of the influence of initial deformations, by taking the inverse of the slope of the line shown in figure E.7. In the case of bamboo culms this may be a powerful instrument, because crookedness is more the normal case than the exception. In the following lines an experimental set-up for the determination of force deformation relations in the case of bamboo culms is presented.

E.4.2-Experiment set-up

Due to the fact that both rotations and displacements are expected to occur during a buckling test, it is not possible to read lateral displacements by simply attaching a device to a point on the culm skin. This is because the movement of the point of contact between the device and the culm will be affected by both the lateral movement of the piece and whatever rotation of its section has.

In the following lines a test set-up is proposed. It takes into account the above mentioned facts and therefore allows us to make a Southwell Plot for bamboo columns.

The general lay-out is shown in figure E.8. It is basically an arrangement of four rotating potentiometers, each attached to the extreme of a 6 mm diameter needle, which passes through the centre of the culm. The attachment points between the ropes and the needle are set equally apart from the centre of the culm. When the needle moves due to any sort of movement of the specimen, it makes the potentiometers rotate thus allowing for registrations of the angle of rotation of each potentiometer, together with the force producing the change, the longitudinal deformation of the culm and whatever other variable of interest.

From geometric relations as shown in figure E.8, it follows that

$$r_2^2 - a^2 = r_1^2 - (d_{12} - a)^2 \qquad \text{eq.E.(40)}$$

Therefore, the value of a is

$$a = \frac{r_2^2 - r_1^2 + d_{13}^2}{2d_{12}} \qquad \text{eq.E.(41)}$$

With this value it is now possible to calculate the coordinates of point number 1 on the needle as

$$x_1 = x_{pot\,2} + \sqrt{r_2^2 - a^2} \qquad \text{eq.E.(42)}$$

and

$$y_1 = y_{pot\,2} + a \qquad \text{eq.E.(43)}$$

From the triangle on the left it follows that

$$r_3^2 - b^2 = r_4^2 - (d_{34} - b)^2 \qquad \text{eq.E.(44)}$$

which after simplification becomes

$$b = \frac{r_3^2 - r_4^2 + d_{34}^2}{2d_{34}} \qquad \text{eq.E.(45)}$$

and this leads to the calculation of the coordinates of point 2 of the needle as

$$x_2 = x_{pot\,3} - \sqrt{r_3^2 - b^2} \qquad \text{eq.E.(46)}$$

and

$$y_2 = y_{pot\,3} + b \qquad \text{eq.E.(47)}$$

If the coordinates of the end-points on the needle are known, then it is easy to calculate the coordinates of the centre. They are

$$x_c = \frac{|x_1 - x_2|}{2} + x_1$$ eq.E.(48)

and

$$y_c = \frac{y_2 + y_1}{2}$$ eq.E.(49)

The initial angle between the needle and the horizontal is

$$\psi = \tan^{-1}\left[\frac{|y_{1_i} - y_{2_i}|}{|x_{1_i} - x_{2_i}|}\right]$$ eq.E.(50)

The lateral absolute displacement of the middle of the culm at any time is

$$w = \sqrt{(x_c - x_{ci})^2 + (y_c - y_{ci})^2}$$ eq.E.(51)

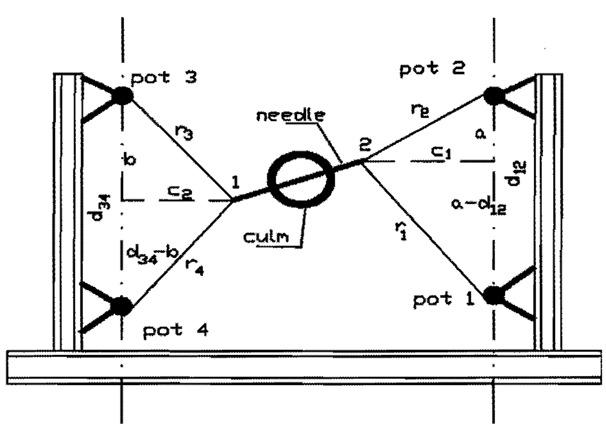

Figure E.8:Cross-section of the experimental lay-out.

and the total rotation of the cross-section is

$$\phi = \tan^{-1}\left[\frac{|y_1 - y_2|}{|x_1 - x_2|}\right] - \psi \qquad \text{eq.H.(52)}$$

E.4.2.1-Alternative procedure

The experimental lay-out of the previous section may present some limitations or may be found too cumbersome, and therefore it is convenient to have a second option. In order to find out whether this alternative does exist, it is first necessary to return to the basic formulation of the Southwell Plot.

E.4.2.1.1-Extension of the Southwell Plot to the Plot of Strains

If the corresponding value of the coordinates is substituted in the equation of deformations then the result is

$$w = \sum_{n=1}^{\infty} \frac{a_n}{1 - \frac{N_x}{n^2 N_x^{cr}}} \sin n\pi \frac{x}{l} \qquad \text{eq.E.(53)}$$

If the initial deformation of the bar can also be described by the series

$$w_i = a_1 \sin \pi \frac{x}{l} \qquad \text{eq.E.(54)}$$

then the lateral deformation can be expressed as

$$w = \frac{a_1}{1 - \frac{N_x}{N_x^{cr}}} \sin \pi \frac{x}{l} \qquad \text{eq.E.(55)}$$

and thus the maximum lateral deformation at half the length of the culm is

$$w_{\frac{l}{2}} = \frac{a_1}{1-\frac{N_x}{N_x^{cr}}}$$ eq.E.(56)

The bending moment associated with this deformation is therefore

$$M_{\frac{l}{2}} = \frac{N_x a_1}{1-\frac{N_x}{N_x^{cr}}}$$ eq.E.(57)

The maximum stress somewhere in the periphery of the culm at this level can easily be calculated by superposition of the bending and the compressive effects. This stress is

$$\sigma_x = \frac{N_x}{A} + \frac{N_x a_1}{1-\frac{N_x}{N_x^{cr}}}\frac{r_e}{I}$$ eq.E.(58)

The maximum strain corresponding to eq.E.58 is

$$\epsilon_x = \frac{N_x}{A\ E} + \frac{N_x a_1}{1-\frac{N_x}{N_x^{cr}}}\ \frac{r_e}{I\ E}$$ eq.E.(59)

Eq.E.59 can be rearranged to get

$$\frac{\epsilon_x - \frac{N_x}{E\ A}}{N_x} = \frac{1}{N_x^{cr}}\left[\epsilon_x - \frac{N_x}{A\ E}\right] + \frac{a_1 r_e}{E\ I}$$ eq.E.(60)

It should be noted that eq.E.60 is the equation of a straight line with intersection at $\frac{a_1 r_e}{E\ I}$

and slope $\frac{1}{N_x^{cr}}$, in a plot of $\frac{\epsilon_x - \frac{N_x}{E\,I}}{N_x}$ against $\epsilon_x - \frac{N_x}{A\,E}$ as indicated in figure E.9.

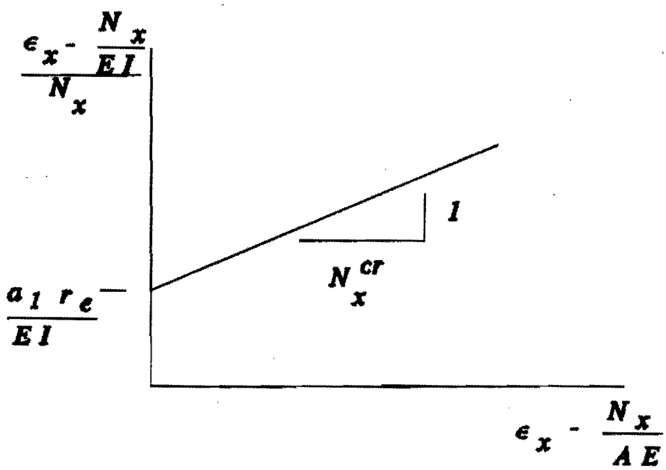

Figure E.9: Southwell plot of strains.

E.4.2.1.2 Experimental lay-out

The point of maximum strains can hardly be anticipated during a buckling test on a strut of bisymmetric cross-section. In the past some readings were approximated during bending tests, by using a large number of strain gauges located longitudinally around the periphery of the culm, si that the maximum strain could be guessed by observation after the test, and then some sort of interpolation could take place.

In order to produce a continuous plot of eq.E.60, it is necessary to find a simpler and more precise way of calculating the position of the point of maximum strains and of calculating this value too.

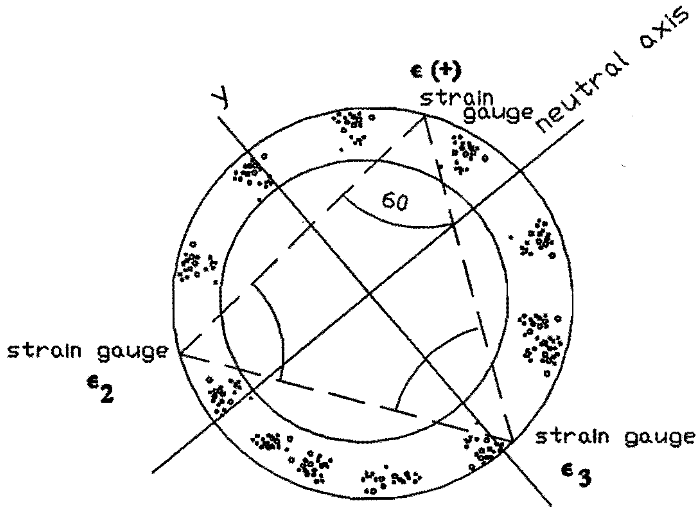

Figure E.10: Three-point strain gauge lay-out.

Figure E.10 shows an option to the solution of this problem. Three strain gauges are attached to the prepared surface of the culm. If great precision is to be achieved, then different sets can be glued at different levels of the culm near the point of maximum initial lateral deformation.

In the figure, the position of the y axis on which the points of maximum and minimum strain lay has been guessed.

It is a matter of simple geometric relations that, under the conditions shown, the value of the maximum strain can be calculated as

$$\epsilon_{max} = \frac{\epsilon(+)}{\cos(90-\xi)} \qquad \text{eq.E.(61)}$$

in this expression

$$\xi = \tan^{-1}\left[\frac{a\sin 30}{r_e - a\cos 30}\right] \qquad \text{eq.E.(62)}$$

and

$$a = 2\frac{r_e}{1+\dfrac{\epsilon_3}{\epsilon(+)}}\cos 30 \qquad \text{eq.E.(63)}$$

The value of $\epsilon(+)$ in the equations is chosen from any positive value of the readings of strains, and ϵ_3 is one of the remaining different sign strains (or the different sign strain if there is just one).

E.5-Experimental results

A sample of bamboo canes was selected from a larger random sample, so as to have a good description of the possible variation in the value of λ . Following the procedure of the above paragraphs, a Southwell Plot was generated as can be seen in figure E.11.

The value of the combined critical load N_x^{cr} was determined in each case by calculating a linear regression model for each one of the graphs, after which results were compared with those predicted by the theory.

The two sets of results were statistically compared, resulting in a correlation of r^2 = **.81** between predicted and measured critical load. As can be seen the amount of agreement between the theory and the experimental results is remarkably high, and therefore gives a good indication of the potential of the proposed model to explain the relation between slenderness and buckling load of bamboo culms.

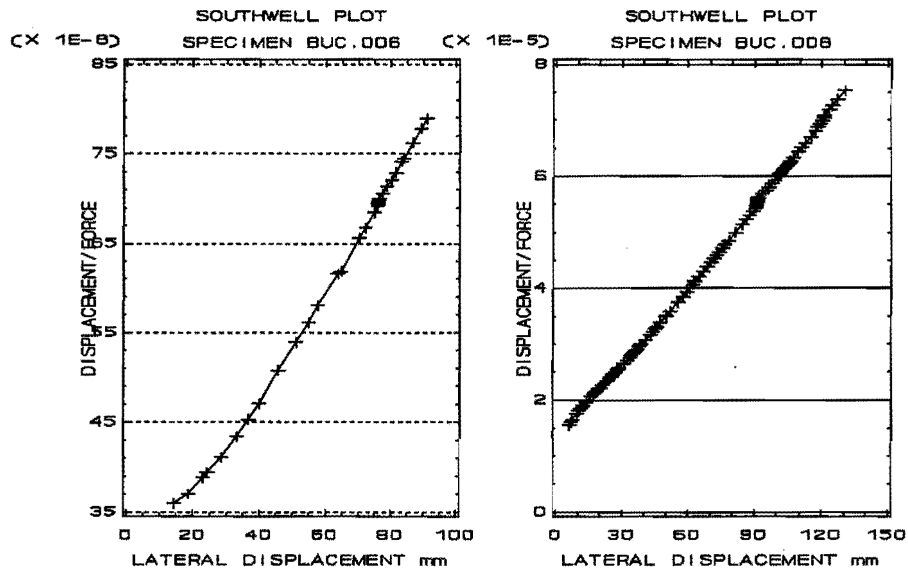

Figure E.11:Resulting Southwell plots of deformations.

A final experimental result of interest is the influence of twisting in the phenomena of buckling . It was already said that this effect would be of no interest if the amount of rotation of the transversal section was limited enough, so that the assumptions made in the first sections of the appendix hold true.

Figure E.12 illustrates the relation between twisting of the cross section and axial load. There, it can be seen that the amount of rotation registered during this test was very little, in spite of the fact that the culm was completely free to rotate at the ends. This fact lends support to the decision not to take the twisting energy into account in the general equation.

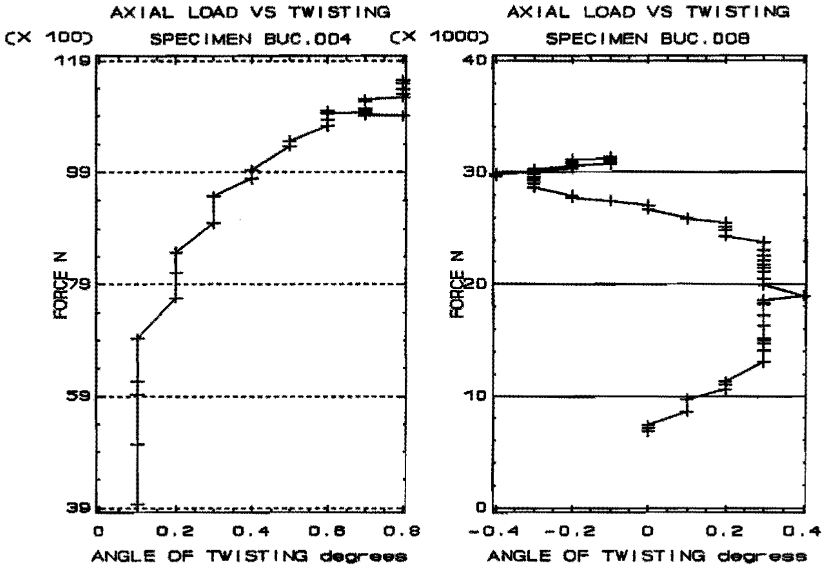

Figure E.12:Twisting of the cross-section.

E.6-Calculation of the critical load as a function of average properties

An alternative way of calculating the critical load of bamboo culms can be proposed if the simplifications suggested in appendix G are followed. If, besides that, the elastic modulus and the moment of area are written as a function of the average values such that

$$E(x) = E_{av}\left(A_E+\delta_E\,\frac{x}{l}\right)$$

$$I(x) = I_{av}\left(A_I-\delta_I\,\frac{x}{l}\right)$$

where

$$A_E = 1 - \frac{\delta_E}{2}$$

and

$$A_I = 1 - \frac{\delta_I}{2}$$

then the result of integrating eq.E.26, and further simplifying it as in eq.E.28, is that the critical load can be calculated as

$$N_x^{cr} = \frac{\pi^2 E_{av} I_{av}}{l^2}\left[1 + 0.0214\left(\delta_I - 2.36\delta_E\right) + 2.83\delta_I\delta_E\right] \qquad \text{eq.E.(64)}$$

In this equation the term in brackets represents a 'correction factor'. In other words, a factor by which the average critical load has to be corrected, to account for the variation in geometric and mechanical properties.

A sample of *Guadua s.p.* was taken to run a parametric study using this last equation, and see the effect of combinations of some real values on the critical load.

Figure E.13 shows the results of such study. In the figure the values of the right term of eq.E.(64) are plotted against the product $\delta_I\ \delta_E$ for each specimen in the sample.

As can be observed, for this particular batch, the calculation of the critical load using average properties gives conservative figures for most of the specimens, but this is not a general rule, since, as can be seen in the plot, there are culms for which the average values give underestimations of the critical load.

Eq.E.(64) can be used for the study of particular batches of bamboo, so that, with the right input, the range of values of the factor of figure E.13 can be estimated, and proper design factors can be agreed upon and recommended.

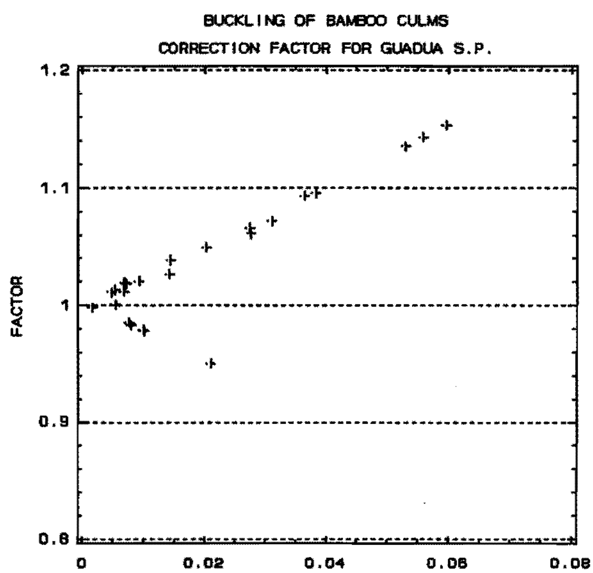

Figure E.13:Result of the estimation of the correction factor for *Guadua s.p.*

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1 Gregory, M.;1967. Elastic Instability: Analysis of Buckling Modes and Loads of Framed Structures. E.& F.N. Spon ltd., London.

Appendix F: Structural modelling and analysis of bamboo structures

F.1-Introduction

In this appendix the influence of bamboo's geometric and mechanical properties on the deformations of structures is evaluated.

Bamboo culms are **tapered**, the cross-section decreases from bottom to top, the elastic modulus increases from bottom to top, and they have nodes with complicated geometrical and mechanical properties, different from those of the neighbouring internodes. The spacing of these nodes is not a constant feature, it varies along the axis of the element and the pattern of variations is different in samples from the same origin.

The differentiation between node and internode properties has been explained by several researchers, for example, Grosser and Liese (Grosser, D.; Liese, W.; 1971; Liese; W.; 1985). These and other scientists have found significant anatomical differences between the nodes and the internodes. However, it was not possible to find any quantification of the effect of these differences in the literature, on the overall structural properties of the culms. As indicated in chapter 2, the nodes constitute the weakest link in bamboo culms.

In the next sections the intention is to explain and to quantify, on a theoretical basis, the essential role that conicity and nodes play in the structural performance of bamboo culms. The followed methodology is summarized in figure F.1. As seen there, the stiffness matrix for a tapered element with varying modulus of elasticity is calculated first of all, adding to it the effect of nodes so that bamboo can be studied. Parameters that show any relevancy in the definition of the matrix, are evaluated by studying real-case culms in a sensitivity analysis. This approach is chosen because there can be combinations of the parameters that just do not exist in real cases, therefore, a simple mathematical analysis may lead to unreal situations.

Secondly, the stiffness of some sample elements is calculated following two different approaches, using the generated matrix, and using average properties only (i.e. average section

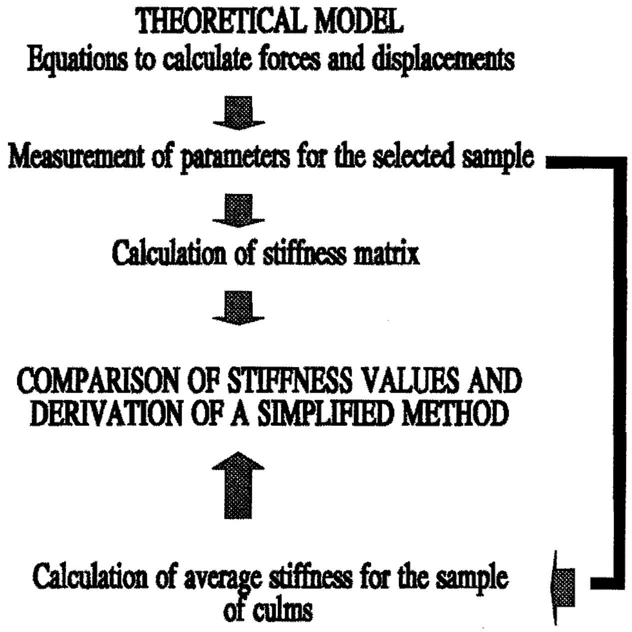


Figure F.1: Method

and average elastic modulus[1]).

Thirdly, the two sets of stiffnesses are compared to derive a simplified approach for the calculation of bamboo trusses and frames (and therefore to avoid the use of complicated equations).

From a methodological point of view, conicity and variation of the elastic modulus, on the one hand, and node influence, on the other, are treated in a separate way, as will be seen later in the chapter. The validity of this procedure is of course subject to doubt in the first instance, but on the pragmatic side, it simplifies the problem. In this way, a theoretical model is developed first to explain the combined effect of conicity and variation of the elasticity modulus. The model is then used to undertake a parametric and a sensitivity study. A second model is developed based on the results, to take account of the influence of nodes on the structural properties of bamboo culms.

From the perspective of the behaviour of bamboo structures, comparisons between three different structural types are presented in the last sections of the chapter, in accordance with laboratory observations of their performances, with the purpose of giving input for the decision about the best structural options for bamboo constructions.

F.2-Theoretical model

F.2.1-The effect of conicity

Several assumptions are made in order to set up a theoretical model.

-.The material is elastic.

-.Elements are perfectly straight.

-.The **conicity** can be described as in eq.F.(1-3).[2]

-.The variation of the **elastic modulus** can be described as in eq.F.(4)

---

[1]average properties are calculated by taking the average of the values at the extremes of the culm.

[2]see appendix G for more details on this matter.It is also possible to express the variation in terms of the average values, but it can be proved that that way does not contribute to the simplification of the final equations.

Appendix F: Structural modelling and analysis of bamboo structures

$$\phi_e(x) = \phi_{e_A}\left(1 - \frac{\delta_\phi}{l}\, x\right) \qquad \text{eq.F.(1)}$$

$$A(x) = A_A\left(1 - \frac{\delta_A}{l} x\right) \qquad \text{eq.F.(2)}$$

$$I(x) = I_A\left(1 - \frac{\delta_I'}{l} x\right) \qquad \text{eq.F.(3)}$$

$$E(x) = E_A\left(1 + \delta_E' \frac{x}{l}\right) \qquad \text{eq.F.(4)}$$

Figure F.2 Degrees of freedom of the element

The element stiffness depends upon a combination of its elasticity, Poisson ratio and geometry. The stiffness matrix of prismatic elements is readily available in many books (for

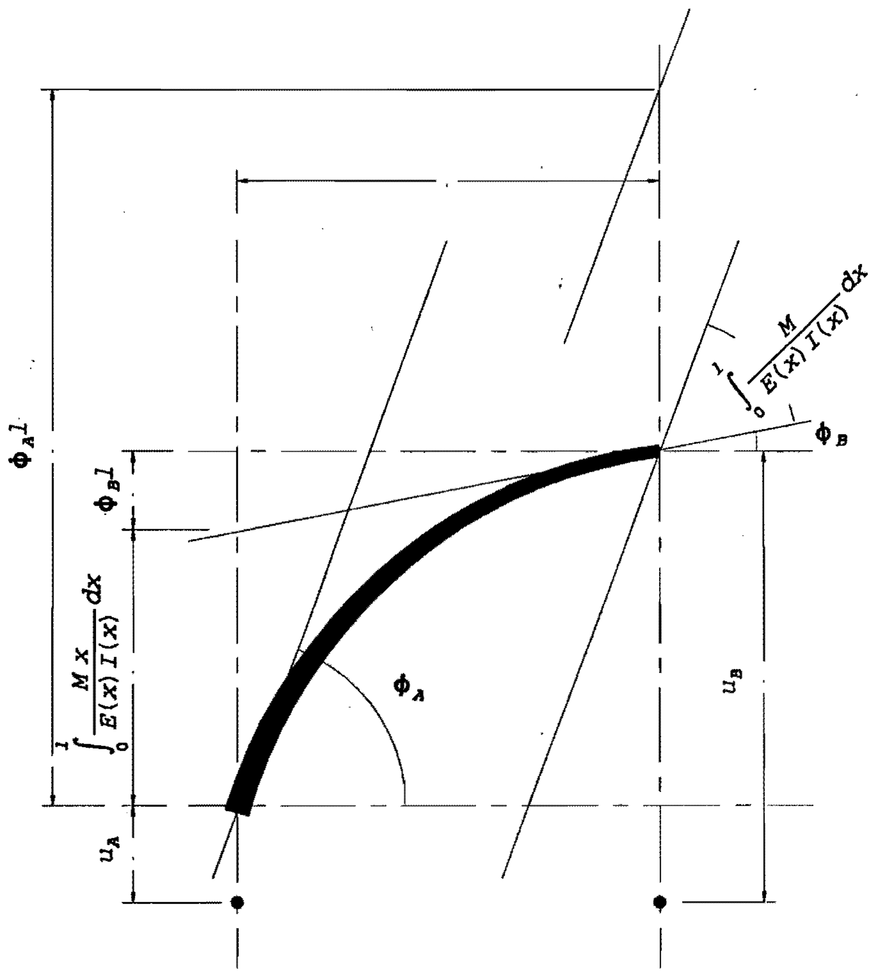


Figure F.3: Relations between displacements and rotations.

instance, Norris,Ch.; Wilbur, J.B.; 1960; Willems, N.; Lucas, W.M.; 1978) and it permits the full analysis of such elements and of structures made of those elements.

In what follows details of the calculation of the stiffness matrix for **non prismatic** elements are shown, according to the assumptions depicted by equations F.1 to F.4.

Suppose that the beam-column of figure F.2 is displaced and rotated at the extremes as shown.

Following the principles of the Reduced Area Moment method the equations of displacements and rotations for the beam-column can be set as

$$u_A = u_B - \left(\phi_A l - \int_0^l \frac{M}{E(x)I(X)}(l-x)dx \right) \qquad \text{eq.F.(5)}$$

or

$$u_A = u_B - \phi_B l - \int_0^l \frac{M\,x}{E(x)I(x)}dx \qquad \text{eq.F.(6)}$$

$$u_B = u_A + \left(\phi_A l - \int_0^l \frac{M}{E(x)I(X)}(l-x)dx \right) \qquad \text{eq.F.(7)}$$

or

$$u_B = u_A + \phi_A l - \int_0^l \frac{M}{E(x)I(x)}(l-x)dx \qquad \text{eq.F.(8)}$$

and the rotations are related such that

$$\phi_A = \phi_B + \int_0^l \frac{M}{E(x)I(x)}dx \qquad \text{eq.F.(9)}$$

$$\phi_B = \phi_A - \int_0^l \frac{M}{E(x)I(x)}dx \qquad \text{eq.F.(10)}$$

If the element is allowed to displace or to rotate at the extremes, in such a way that only one possible displacement or rotation is permitted at a time, then four cases can be distinguished as explained in the following lines.

F.2.1.1-Case one

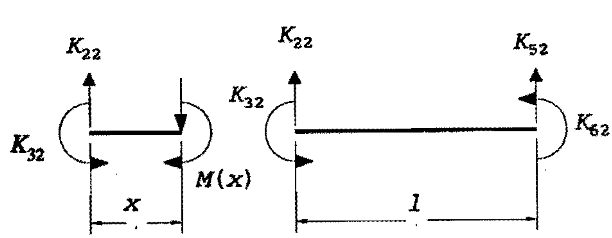


Figure F.4: Equilibrium conditions for case one.

The state of displacements and rotations is described by

$$\begin{aligned} u_A &= u_2 = 1 \\ u_B &= u_5 = 0 \\ \phi_A &= u_3 = 0 \\ \phi_B &= u_6 = 0 \\ u_1 &= 0 \; ; \; u_4 = 0 \end{aligned} \qquad \text{eq.F.(11)}$$

From the equilibrium of the beam-column

$$K_{62} = -K_{32} + K_{22} l$$ eq.F.(12)

$$K_{52} = -K_{22}$$ eq.F.(13)

$$M(x) = K_{32} - K_{22} x$$ eq.F.(14)

F.2.1.2-Case two

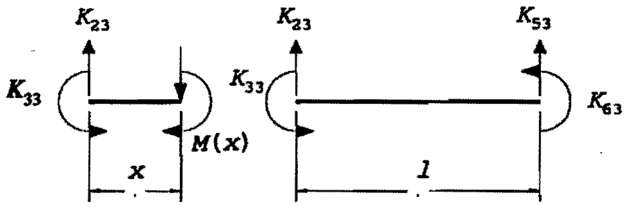


Figure F.5: Equilibrium of the beam column for case two.

Displacements and rotations are described by

$$\begin{aligned} u_A &= u_2 = 0 \\ u_B &= u_5 = 0 \\ \phi_A &= u_3 = 1 \\ \phi_B &= u_6 = 0 \\ u_1 &= 0 \; ; \; u_4 = 0 \end{aligned}$$ eq.F.(15)

From the equilibrium of the beam-column

$$K_{63} = -K_{33} + K_{23}l \qquad \text{eq.F.(16)}$$

$$K_{23} = -K_{53} \qquad \text{eq.F.(17)}$$

$$M(x) = K_{33} - K_{23}x \qquad \text{eq.F.(18)}$$

F.2.1.3-Case three

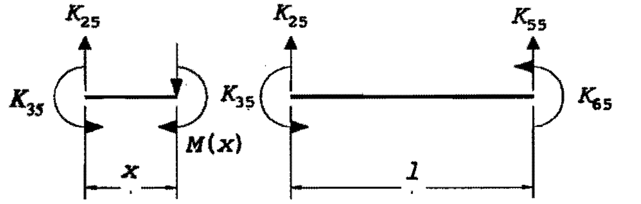


Figure F.6: Equilibrium of the beam column for case three.

The state of displacements and rotations is described by

$$u_A = u_2 = 0$$
$$u_B = u_5 = 1$$
$$\phi_A = u_3 = 0 \qquad \text{eq.F.(19)}$$
$$\phi_B = u_6 = 0$$
$$u_1 = 0\ ;\ u_4 = 0$$

From the equilibrium of the beam-column

$$K_{65} = -K_{35} + K_{25}\,l \qquad \text{eq.F.(20)}$$

$$K_{25} = -K_{55} \qquad \text{eq.F.(21)}$$

$$M(x) = K_{35} - K_{25}\,x \qquad \text{eq.F.(22)}$$

F.2.1.4-Case four

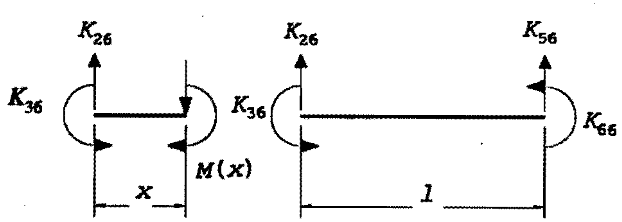


Figure F.7: Equilibrium of the beam column for case four.

Displacements and rotations are described by

$$\begin{aligned} u_A &= u_2 = 0 \\ u_B &= u_5 = 0 \\ \phi_A &= u_3 = 0 \\ \phi_B &= u_6 = 1 \\ u_1 &= 0\ ;\ u_4 = 0 \end{aligned} \qquad \text{eq.F.(23)}$$

From the equilibrium of the beam-columnn

$$K_{66} = -K_{36} + K_{26} l \qquad \text{eq.F.(24)}$$

$$K_{26} = -K_{56} \qquad \text{eq.F.(25)}$$

$$M(x) = K_{36} - K_{26} x \qquad \text{eq.F.(26)}$$

F.2.1.5-Solution of the equations of displacement and rotation

Substitution of the values of eq.F.11 into eq.F.5 to F.10 gives

$$0 = \int_0^l \frac{K_{32} - K_{22} x}{E_A I_A \left(1 + \delta_E' \frac{x}{l}\right)\left(1 - \delta_I' \frac{x}{l}\right)} dx \qquad \text{eq.F.(27)}$$

and

$$1 = -\int_0^l \frac{(K_{32} - K_{22} x) x}{E_A I_A \left(1 + \delta_E' \frac{x}{l}\right)\left(1 - \delta_I' \frac{x}{l}\right)} dx \qquad \text{eq.F.(28)}$$

For the sake of simplification the following notation will be kept throughout :

$$F_0 = E_A I_A \left(1 + \delta_E' \frac{x}{l}\right)\left(1 - \delta_I' \frac{x}{l}\right) \qquad \text{eq.F.(29)}$$

$$F_1 = \int_0^l \frac{dx}{F_0} \qquad \text{eq.F.(30)}$$

$$F_2 = \int_0^l \frac{x\,dx}{F_0} \qquad \text{eq.F.(31)}$$

$$F_3 = \int_0^l \frac{x^2 dx}{F_0} \qquad \text{eq.F.(32)}$$

If the values of equations F.15 to F.26 are substituted one case at a time into equations F.5 to F.10 then it is possible to write the following expressions:

From case one,

$$0 = K_{32}F_1 - K_{22}F_2 \qquad \text{eq.F.(33)}$$

$$1 = -K_{32}F_2 + K_{22}F_3 \qquad \text{eq.F.(34)}$$

From case two,

$$1 = K_{33}F_1 - K_{23}F_2 \qquad \text{eq.F.(35)}$$

$$0 = -K_{33}F_2 + K_{23}F_3 \qquad \text{eq.F.(36)}$$

From case three,

$$1 = K_{35}(-lF_1 + F_2) + K_{25}(lF_2 - F_3) \qquad \text{eq.F.(37)}$$

$$0 = -K_{35}F_1 + K_{25}F_2 \qquad \text{eq.F.(38)}$$

From case four

$$1 = -K_{36}F_1 + K_{26}F_2 \qquad \text{eq.F.(39)}$$

$$0 = K_{36}(-lF_1 + F_2) + K_{26}(-F_3 + lF_2) \qquad \text{eq.F.(40)}$$

The solution of the above equations can be expressed in terms of K_{32} and of the following relations

$$Z_1 = \frac{F_1}{F_2} \qquad \text{eq.F.(41)}$$

$$Z_2 = \frac{F_3}{F_2} \qquad \text{eq.F.(42)}$$

to find that

$$K_{32} = \frac{F_2}{F_1F_3 - F_2^2} \qquad \text{eq.F.(43)}$$

$$K_{22} = K_{32}\, Z_1 \qquad \text{eq.F.(44)}$$

$$K_{52} = -Z_1K_{32} \qquad \text{eq.F.(45)}$$

$$K_{62} = (lZ_1 - 1)K_{32} \qquad \text{eq.F.(46)}$$

$$K_{23} = K_{32}$$ eq.F.(47)

$$K_{33} = Z_2 K_{32}$$ eq.F.(48)

$$K_{53} = -K_{32}$$ eq.F.(49)

$$K_{63} = (l - Z_1)\, K_{32}$$ eq.F.(50)

$$K_{35} = -K_{32}$$ eq.F.(51)

$$K_{25} = -Z_1\, K_{32}$$ eq.F.(52)

$$K_{55} = Z_1\, K_{32}$$ eq.F.(53)

$$K_{65} = (1 - Z_1 l) K_{32}$$ eq.F.(54)

$$K_{36} = (l - Z_2)\, K_{32}$$ eq.F.(55)

$$K_{26} = (l Z_1 - 1)\, K_{32}$$ eq.F.(56)

$$K_{56} = (1 - Z_1\, l\,)\, K_{32}$$ eq.F.(57)

$$K_{66} = \left(Z_2 + l^2 Z_1 - 2l\right) K_{32}$$ eq.F.(58)

Integration of equations F.30, F.31, and F.32 allows to state that:

$$F_1 = \frac{1}{\left(\delta_E' + \delta_I'\right)} \ln\left(\frac{1+\delta_E'}{1-\delta_I'}\right) \frac{l}{E_A I_A} \qquad \text{eq.F.(59)}$$

$$F_2 = \frac{\ln\left(1+\delta_E'\right)\left(1-\delta_I'\right)}{\left(\delta_I' + \delta_E'\right)} \frac{l^2}{E_A I_A} \qquad \text{eq.F.(60)}$$

$$F_3 = \frac{1}{\delta_E' \, \delta_I'} \left[-1 + \frac{\frac{\delta_I'}{\delta_E'} \ln\left(1+\delta_E'\right) - \frac{\delta_E'}{\delta_I'} \ln\left(1-\delta_I'\right)}{\delta_E' + \delta_I'} \right] \frac{l^3}{E_A I_A} \qquad \text{eq.F.(61)}$$

The members K_{11} and K_{44} of the stiffness matrix can be found taking

$$u_1 = 1 \quad ; \quad u_4 = 1 \qquad \text{eq.F.(62)}$$

and all the other displacements and rotations equal to zero. Then

$$\frac{1}{K_{11}} = \int_0^l \frac{F_x}{E_A\left(1+\delta_E' \frac{x}{l}\right) A_A \left(1-\delta_A \frac{x}{l}\right)} dx \qquad \text{eq.F.(63)}$$

and therefore

$$K_{11} = \frac{\delta_E' + \delta_A'}{\ln\left(\frac{1-\delta_A'}{1+\delta_E'}\right)} \frac{E_A A_A}{l} \qquad \text{eq.F.(64)}$$

Equations F.43 to F.58 and F.64 describe the members of the stiffness matrix of a tapered beam-columnn element with linearly varying elastic modulus. It is possible to show that, for example,

$$\lim \begin{bmatrix} \delta_I{}' \\ & \to 0 \\ \delta_E{}' \end{bmatrix} K_{32} = \frac{6\ E_A I_A}{l^2}$$ eq.F.(65)

and

$$\lim \begin{bmatrix} \delta_A{}' \\ & \to 0 \\ \delta_E{}' \end{bmatrix} K_{11} = \frac{E_A\ A_A}{l}$$ eq.F.(66)

which are the corresponding members of the stiffness matrix of a prismatic beam-column with constant modulus of elasticity.

F.3-Parametric study

The major difficulty in the use of the stiffness matrix, to see the effect of changes in the various parameters, rises from the fact that valuable data to facilitate a practical exploration is not available. Data do exist, of course, but they have been gathered and produced under a different focus of attention.

Basically, the method followed consisted of changing the values of the different parameters involved, so that a more complete picture of their effect could be produced.

To set an adequate reference to the values that the different parameters may take, a sample out of a larger sample of 120 culms of *Guadua s.p.* from Costa Rica was studied.

A total of 24 pieces of different lengths were taken. The sample was not completely randomized, because part of the current stock at Eindhoven University has been attacked by borers, so all the damaged culms were regarded as unsuitable. Every one of the culms was closely examined and its relevant properties determined, namely the geometric and mechanical characteristics as described in previous chapters.

In the search for a way to easily carry out a parametric study, it was of initial interest to know whether there was a way of relating the stiffness calculated using the proposed **matrix**, and that calculated taking only **average** properties for every culm into account, because the formulae suggest there should be some straightforward relation between them. Should the formulae be close to reality, that type of relation would then come forward in an obvious way after the examination of a sample. Results are encouraging, as can be seen in the following paragraphs.

Some basic information about element properties is given to put results in the right perspective.

statistic	external diam.	E (10-80%)
average	83.01	18710.0
variance	25.32	4724710.0
standard dev.	5.03	2173.52
standard error	.726	443.67
minimum	71.82	14930.0
maximum	97.68	22235.0

Table F.1 : summary of element properties. Diameter [*mm*]. Average elasticity modulus [N/mm^2].

Axial deformation stiffness was examined first.

The values of K_{11} for each one of the elements were calculated according to equation F.64, as well as the corresponding average value. The results can be better visualized in a plot like the one shown in figure F.8.For this sample it was found that the ratio between axial stiffnesses according to the proposed formulae and that calculated based on average properties is .97. Thus, for the examined sample it looks as if the error in the estimation of the axial stiffness is an average of approximately -3% in relation to the axial stiffness based on average properties.

(For the sake of clarity, in what follows, stiffness calculated according to the proposed matrix, will be referred to as 'theoretical stiffness', though it is acknowledged that the average one is also theoretical. **Average properties are calculated by taking the values at the extremes A and B of the culm and averaging them**).

The same type of comparison can be made for other factors in the matrix. If for example K_{22} is considered then the result is like that shown in figure F.9.

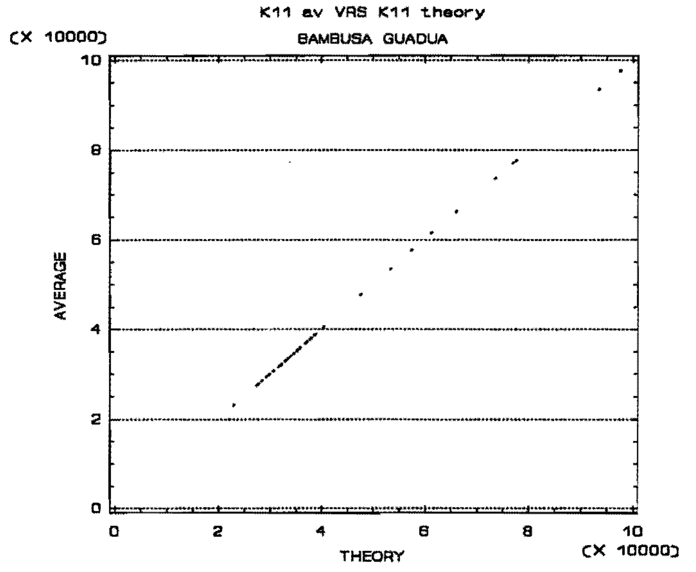


Figure F.8 Average and theoretical axial stiffness compared

Next, the most relevant elements of the stiffness matrix were compared to those from average values for all the elements in the sample, so that a more general picture could be gained. Figure F.10 shows the ratio between theoretical values and average ones, on the y axis and the identification label number of the element is located on the x axis.

There it can be seen that, at least for this sample, the range of variation of the coefficients for different stiffness elements is within -.92 and +1.05 times the average value. The plots show that in general, values tend to fall below 1.0, meaning that the average approach tends to overestimate stiffness.In order to broaden the picture on these effects, and to see how general this conclusion can be, a limited sensitivity analysis was carried out.

Culms were studied in such a way that the ratio $\frac{\delta_I'}{\delta_E'}$ changed while the remaining factors were kept constant. The results of the calculations of the ratios for different stiffness members are given in figures F.11 to F.14.

The studies of the author on samples of *Bambusa Blumeana* from the Philippines and *Guadua s.p.* from Costa Rica allow us to conclude that in these two cases the conicity rates spread over a wide range of values (though the conicity itself is small). More refined anatomical studies show that conicity rates change along the length. Therefore, equations F.1 to F.4 are only a simplification, of course, but as indicated in appendix G they are precise enough for structural calculations, **as long as the length of the element is bellow say one third of the total length of the original culm** , which seems to be the case in structural applications.

The figures show that in most of the cases the error, in the estimation of the stiffness by taking average properties, is about 15% and that **average values overestimate the stiffness of bamboo elements.**

Further calculations were undertaken in order to show the individual effects of different combinations of parameters, as shown in figures F.15 to F.32.

Hopefully this analysis covers all the extreme possibilities that can be found in structural applications of bamboo, though it is acknowledged that further data should be gathered to be conclusive about on these results. Nevertheless, some observations can be made.

-The sample of *Guadua s.p.* showed that the ratio $\frac{\delta_I'}{\delta_E'}$ has a large variation, in the range from 60 to about 400, with some concentration of values in the region of 350. According to the plots, it is possible to have overestimations as large as 25% on the bending stiffnesss, though a typical figure would tend to be closer to the 5% mentioned above, for all ranges of initial diameters and initial elasticity moduli examined in this study.

-So, for most of the cases of structural interest, it probably does not pay to go into any refinement in taking the influence of the variation of the diameter and the elasticity modulus in the bending stiffnesss of the culms into account. In relation to the axial stiffness, this characteristic seems less sensitive to the value of $\frac{\delta_A}{\delta_E^{\prime}}$.

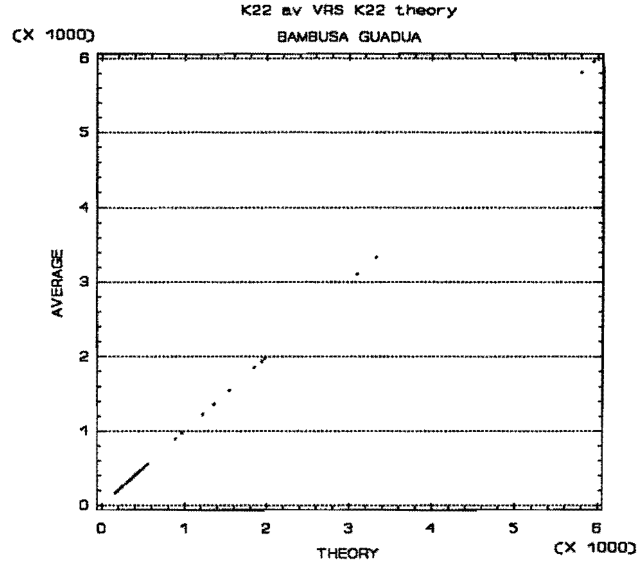


Figure F.9 Average and theoretical bending stiffness compared.

Appendix F: Structural modelling and analysis of bamboo structures

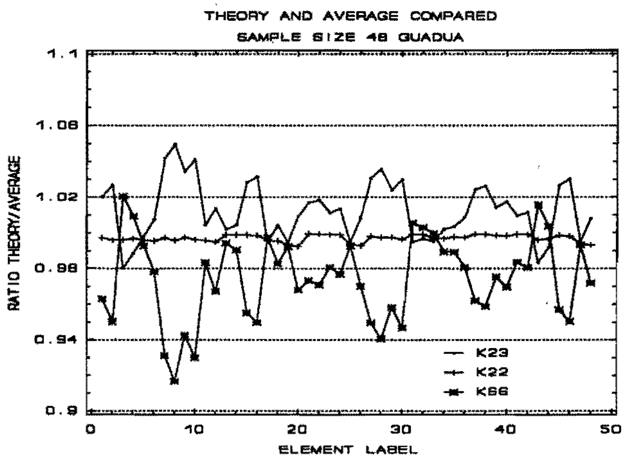
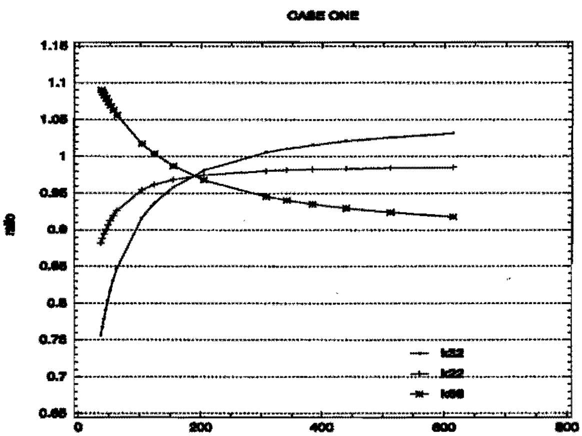


Figure F.10: Ratio of theoretical stiffness to average ones for different stiffness elements.



$\frac{\delta'_I}{\delta'_E}$

Figure F.11 *Case 1: l= 4500; Φ_i=110 mm; E_i=14000 N/mm^2.*

Appendix F: Structural modelling and analysis of bamboo structures

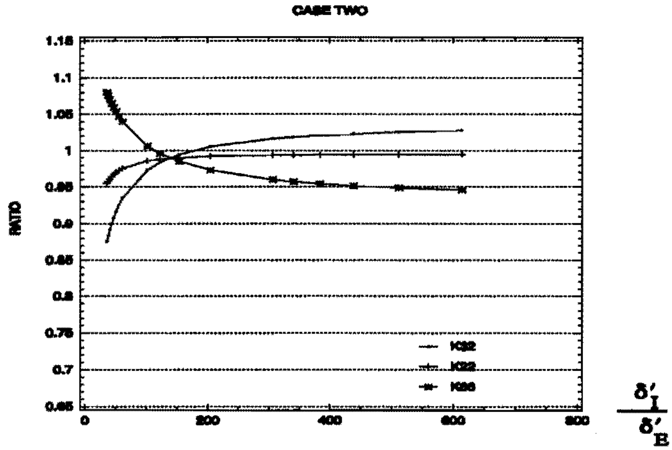


Figure F.12 *Case 2: l=3000 mm; Φ_i= 82 mm; E_i=22034 N/mm^2.*

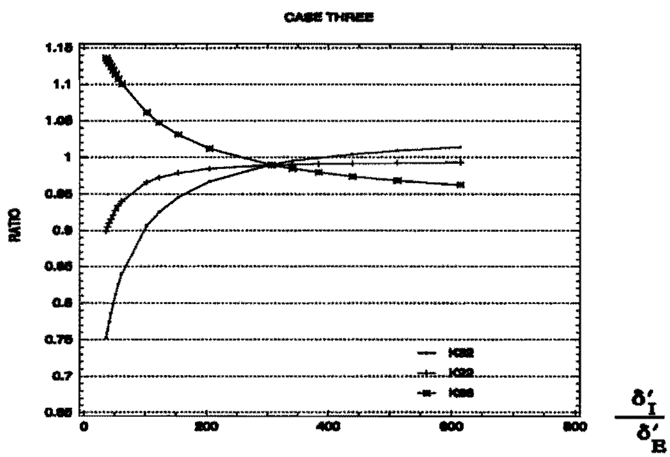


Figure F.13 Case 3: *l=2000 mm; Φ_i 110 mm; E_i=10000 N/mm^2.*

Appendix F: Structural modelling and analysis of bamboo structures

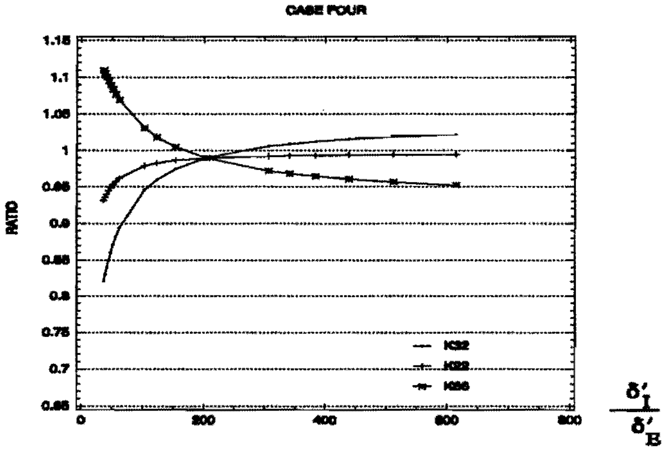


Figure F.14 Case 4: *l=3000; Φ_i=82 mm; E_i=15000 N/mm²* .

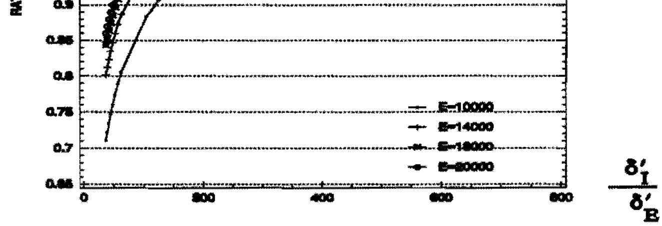


Figure F.15: Correction factors for average stiffness, conditions as indicated.

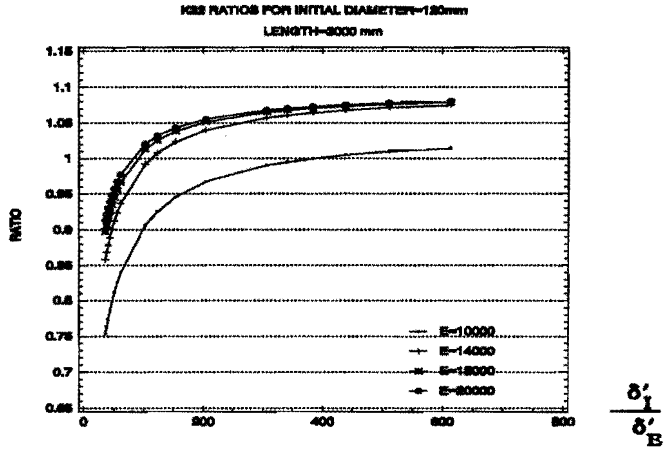


Figure F.16: Correction factors for average stiffness, conditions as indicated.

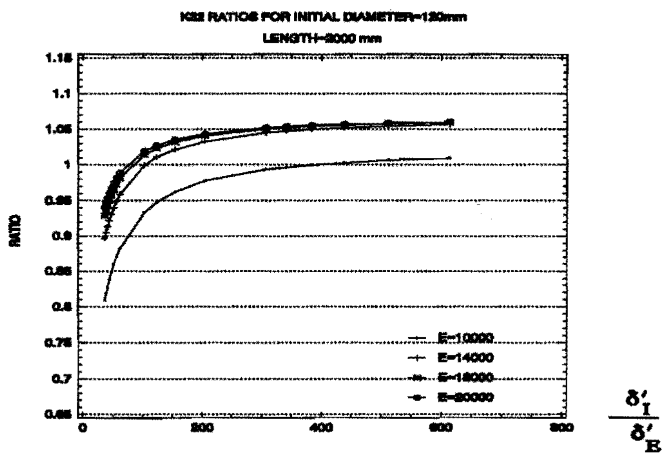


Figure F.17: Correction factors for average stiffness, conditions as indicated.

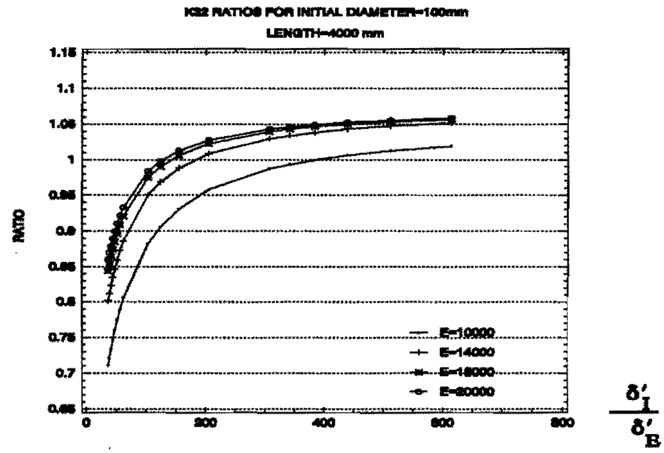


Figure F.18: Correction factors for average stiffness, conditions as indicated.

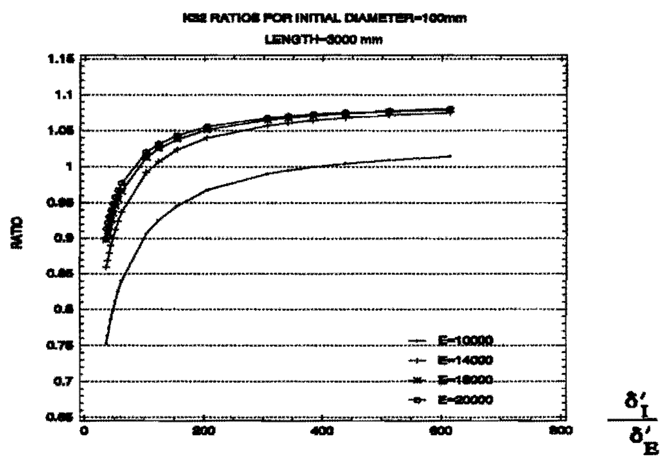
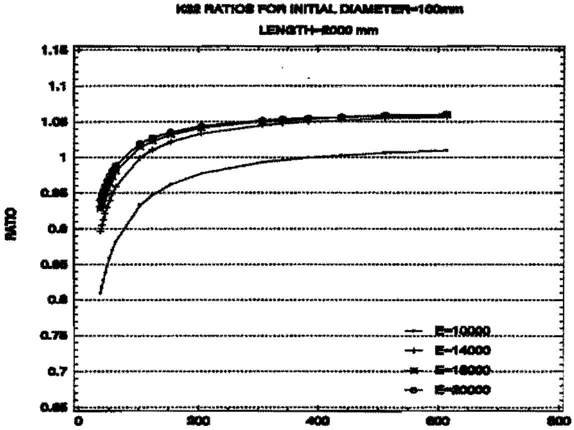


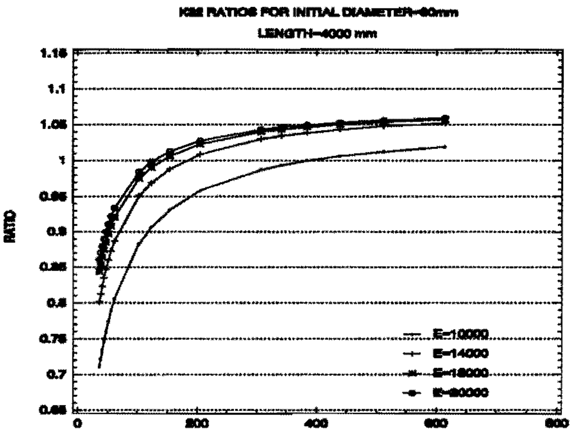
Figure F.19: Correction factors for average stiffness, conditions as indicated.

Appendix F: Structural modelling and analysis of bamboo structures



$$\frac{\delta'_I}{\delta'_E}$$

Figure F.20: Correction factors for average stiffness, conditions as indicated.



$$\frac{\delta'_I}{\delta'_E}$$

Figure F.21: Correction factors for average stiffness, conditions as indicated.

Appendix F: Structural modelling and analysis of bamboo structures

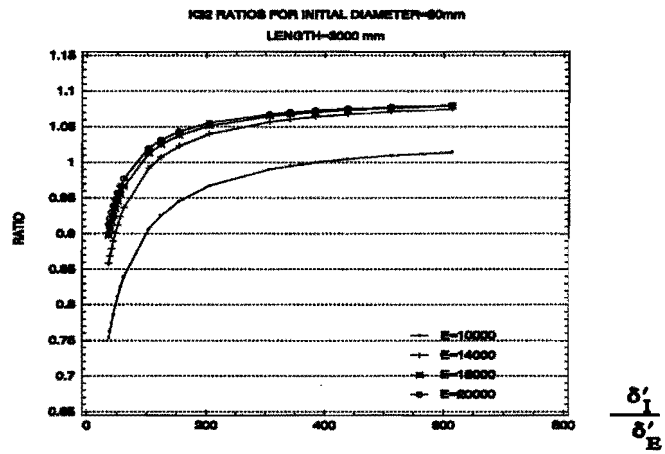


Figure F.22: Correction factors for average stiffness, conditions as indicated.

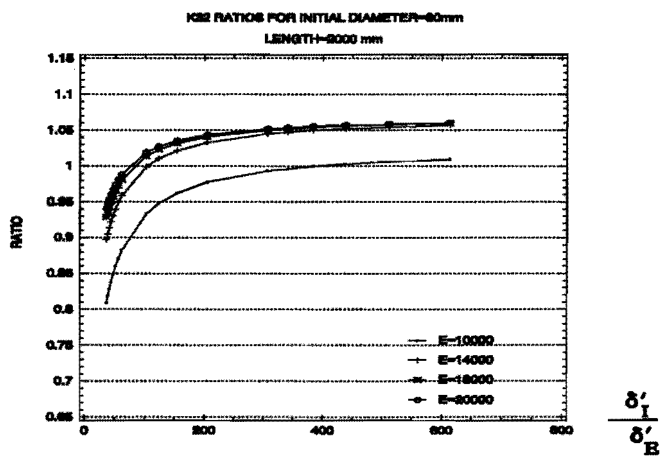


Figure F.23: Correction factors for average stiffness, conditions as indicated.

Appendix F: Structural modelling and analysis of bamboo structures

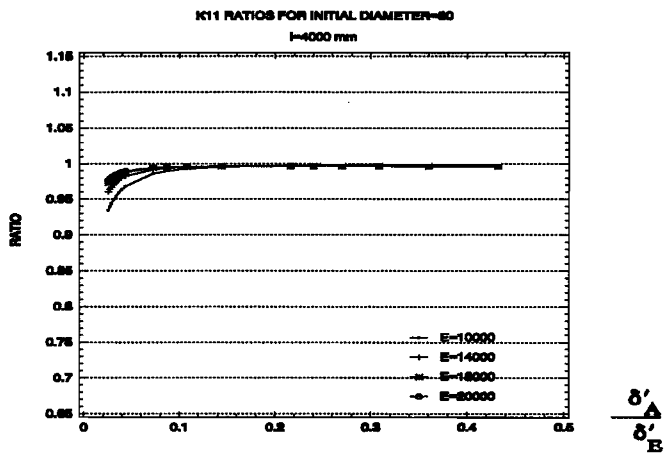


Figure F.24: Correction factors for average stiffness, conditions as indicated.

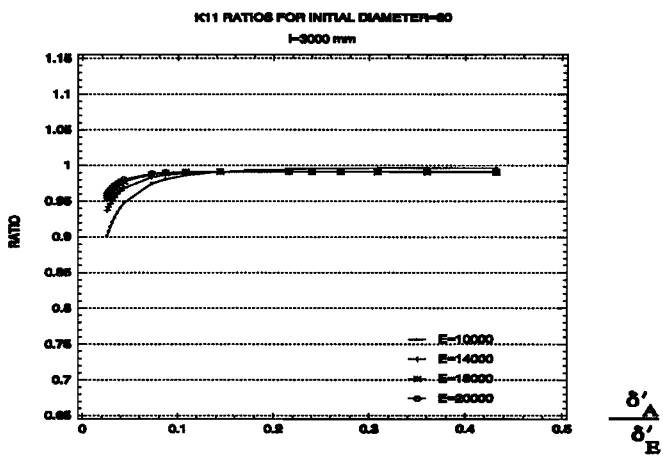


Figure F.25: Correction factors for average stiffness, conditions as indicated.

Appendix F: Structural modelling and analysis of bamboo structures

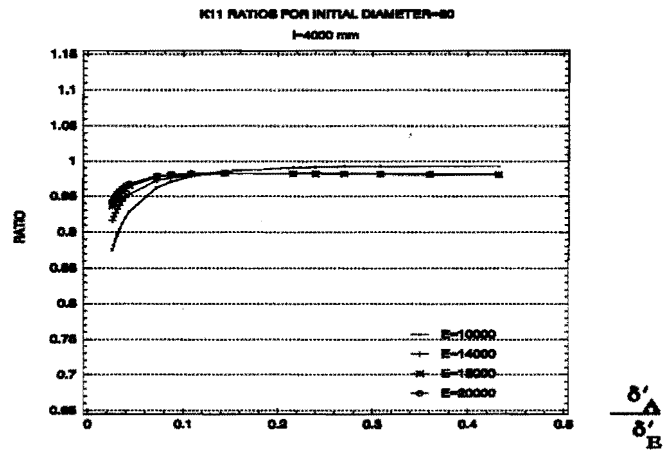


Figure F.26: Correction factors for average stiffness, conditions as indicated.

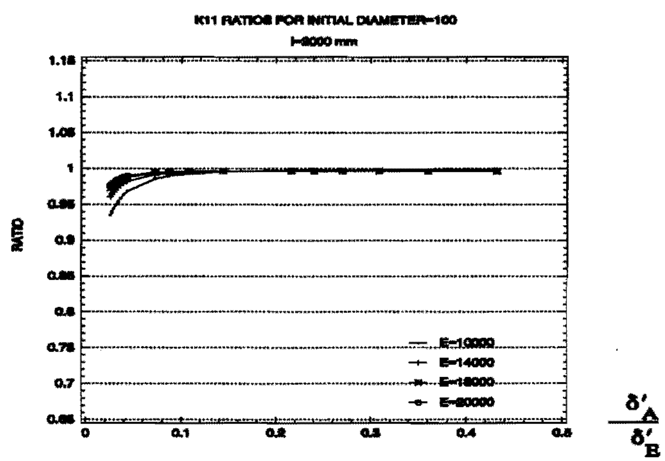


Figure F.27: Correction factors for average stiffness, conditions as indicated.

Appendix F: Structural modelling and analysis of bamboo structures

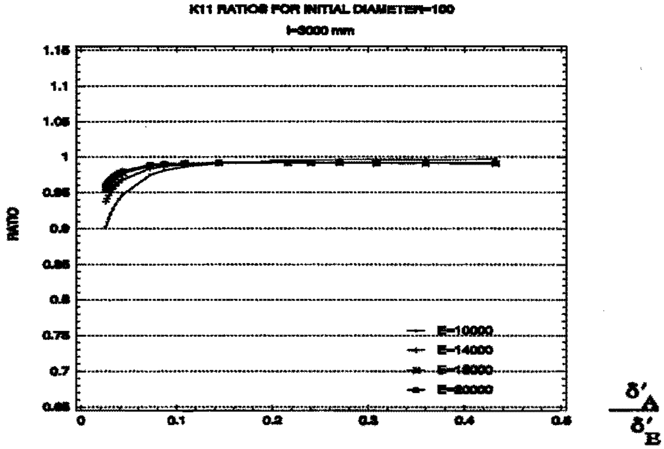


Figure F.28: Correction factors for average stiffness, conditions as indicated.

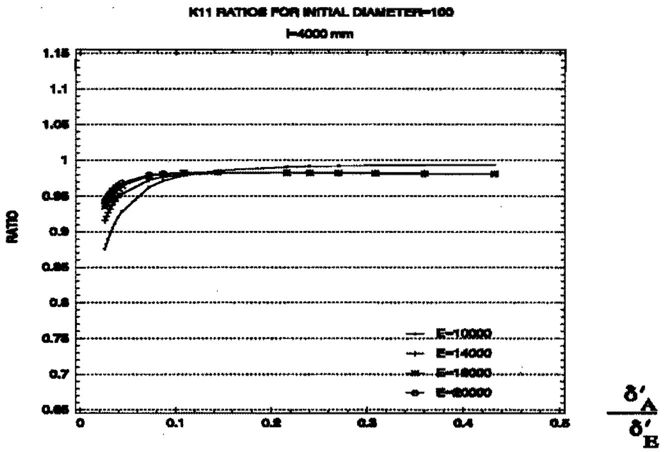


Figure F.29: Correction factors for average stiffness, conditions as indicated.

Appendix F: Structural modelling and analysis of bamboo structures

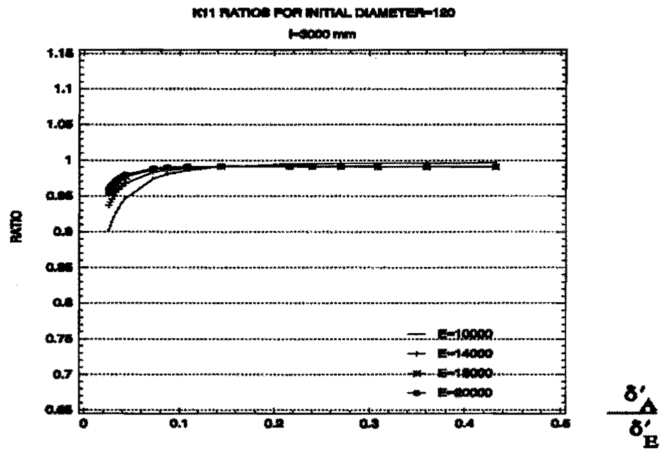


Figure F.30: Correction factors for average stiffness, conditions as indicated.

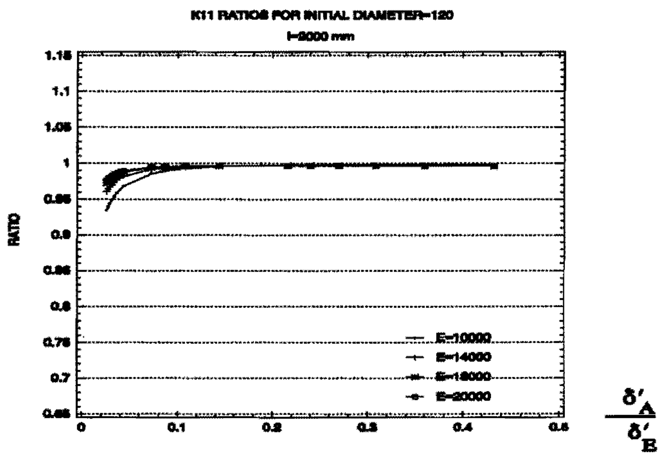


Figure F.31: Correction factors for average stiffness, conditions as indicated.

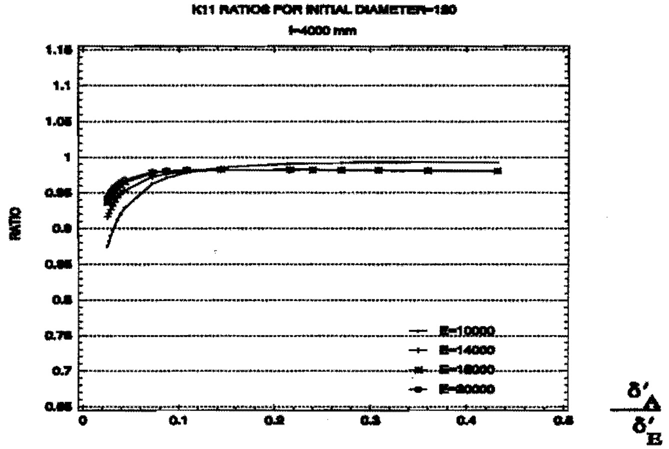


Figure F.32: Correction factors for average stiffness, conditions as indicated.

F.4-The effect of the nodes

As indicated in chapter 2, the nodes of bamboo culms are much softer than the internodes, so it is reasonable to expect that they decrease the general stiffness of the canes.

The amount of stiffness decrement depends upon the number of nodes and upon the stiffness relation between internodes and nodes. As said before, the statements in chapter 2 in this respect are based on the examination of a sample available in the University of Eindhoven, but the description of anatomical features of the nodes by Grosser (opus cit) leads one to believe that those remarks are widely valid. The specific relations for each species or group of species will have to be worked out to be used for structural purposes.

On this basis nodes can be incorporated in the model of figure F.2.

Equation F.27 can be rewritten so that

$$0 = \int_0^l \frac{K_{32}-K_{22}x}{E_I I_I}dx + \sum_{i=1}^{N}\left[\int_{is}^{is+a_n} \frac{K_{32}-K_{22}x}{E_N I_N}dx - \int_{is}^{is+a_n} \frac{K_{32}-K_{22}x}{E_I I_I}dx\right] \qquad \textbf{eq.F.(67)}$$

and equation F.28 now becomes

$$1 = \int_0^l \frac{(K_{32}-K_{22}x)x}{E_I I_I}dx + \sum_{i=1}^{N}\left[\int_{is}^{is+a_n} \frac{(K_{32}+K_{22}x)x}{E_N I_N}dx - \int_{is}^{is+a_n} \frac{(K_{32}+K_{22}x)x}{E_I I_I}dx\right] \qquad \textbf{eq.F.(68)}$$

Integration and some reorganization of terms permits us to write these equations as

$$0 = \left(-\frac{l}{E_I I_I}+\frac{F_4}{E_N I_N}-\frac{F_6}{E_I I_I}\right)K_{32} + \left(\frac{l^2}{2E_I I_I}-\frac{F_5}{E_N I_N}+\frac{F_7}{E_N I_N}\right)K_{22} \qquad \textbf{eq.F.(69)}$$

and

Appendix F: Structural modelling and analysis of bamboo structures

$$1 = \left(\frac{-l^2}{2E_I I_I} + \frac{F_5}{E_N I_N} - \frac{F_7}{E_I I_I}\right) K_{32} + \left(\frac{l^3}{3E_I I_I} - \frac{F_8}{E_N I_N} + \frac{F_9}{E_I I_I}\right) K_{22} \qquad \text{eq.F.(70)}$$

where

$$F_4 = F_6 = -30\ N \qquad \text{eq.F.(71)}$$

$$F_5 = F_7 = \sum_{i=1}^{N} \left(-a_n is - \frac{a_n^2}{2}\right) \qquad \text{eq.F.(72)}$$

$$F_8 = F_9 = \sum_{i=1}^{N} \left(\frac{(is)^3}{3} - \frac{(is+a_n)^3}{3}\right) \qquad \text{eq.F.(73)}$$

The solution of these equations is

$$K_{32} = -\frac{B}{B^2 + CA} \qquad \text{eq.F.(74)}$$

and

$$K_{22} = \frac{A}{B^2 + CA} \qquad \text{eq.F.(75)}$$

where

$$A = \frac{1}{E_I I_I} \left(-l + a_n N \left(1 - R_{EI}\right)\right) \ \wedge \ R_{EI} = \frac{\textit{internode stiffness}}{\textit{node stiffness}} \qquad \text{eq.F.(76)}$$

$$B = \frac{1}{E_I I_I}\left(\frac{l^2}{2} + (1-R_{EI})\sum_{i=1}^{N}\left(-a_n is - \frac{(a_n)^2}{2}\right)\right)$$ **eq.F.(77)**

$$C = \frac{1}{E_I I_I}\left(\frac{l^3}{3} + (1-R_{EI})\sum_{i=1}^{N}\left(\frac{(is)^3}{3} - \frac{(is+a_n)^3}{3}\right)\right)$$ **eq.F.(78)**

This solution has been written in terms of the average properties, so that comparisons can be done in a more direct way.

One aspect that can be noticed after examination of the above equations is that, as common sense dictates, the length of the nodes a_n plays an important role in the calculation, but the determination of that figure is a matter subject to great discussion, as has been already pointed out in chapter 2. In what follows, the assumptions made in this respect in that chapter also hold for the sake of the present arguments.

The recalculation of the longitudinal stiffness can be done in an easy way as well. It is possible to model the element as indicated in figure F.33.

Using basic principles the sum of the stiffness of the different components along the cane can be written as:

$$(K_{11})_N = \frac{A_I E_I}{l+(R_E-1)a_n N}$$ **eq.F.(79)**

or

$$(K_{11})_N = K_{11}\frac{l}{l+(R_E-1)\,a_n N} \wedge R_E = \frac{\text{axial stiffness internode}}{\text{axial stiffnes node}}$$ **eq.F.(80)**

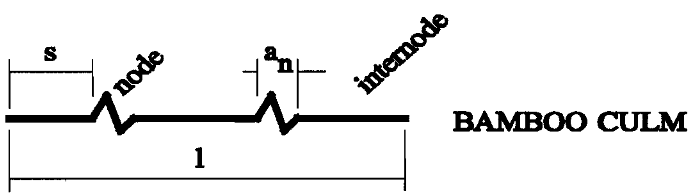


Figure F.33 Model for the calculation of axial stiffness

F.4.1-Parametric study on node influence on the stiffness.

Figures F.34 to F.39 show the results of a parametric study on the influence of nodes on the bending stiffness of bamboo elements.

As can be observed, nodes have a very important impact on the stiffness of culms, theireffect averaging out to a reduction of 40% on the bending stiffness, and of 12% on the axial stiffness, being the major source of flexibility.

A small word has to be said about the node width indicated in the plots. The value of a_n corresponds to the indications that are made in chapter 2, and therefore it is only a nominal figure. The fact that some of the curves are saw-like may be confusing, but it is only due to the discrete character of the number of nodes.

Appendix F: Structural modelling and analysis of bamboo structures

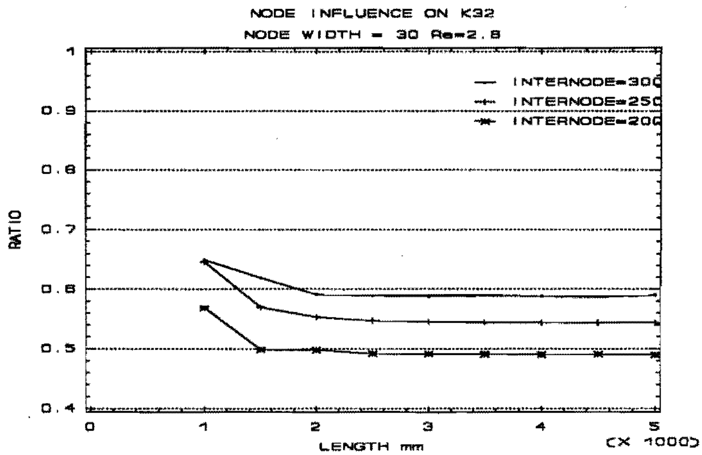


Figure F.34 Influence of node on bending stiffness. Internode lengths in mm. Conditions as indicated.

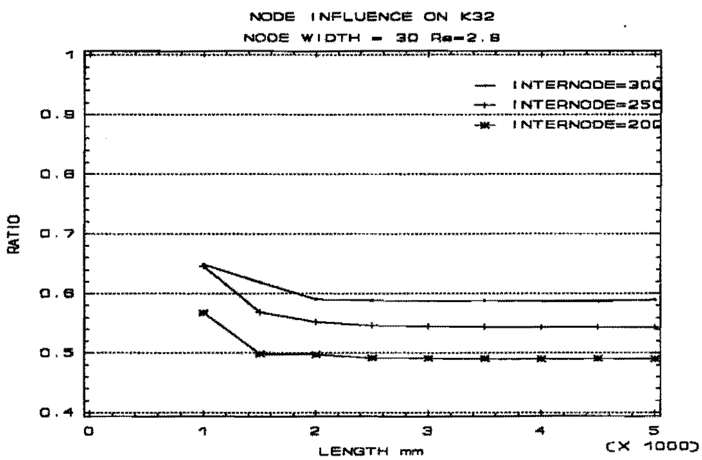


Figure F.35 Influence of node on bending stiffness. Internode lengths in mm. Conditions as indicated.

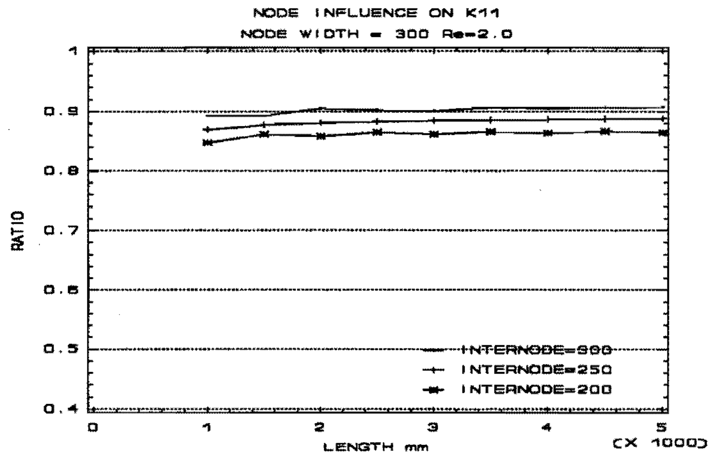


Figure F.36 Influence of node on bending stiffness. Internode lengths in mm. Conditions as indicated.

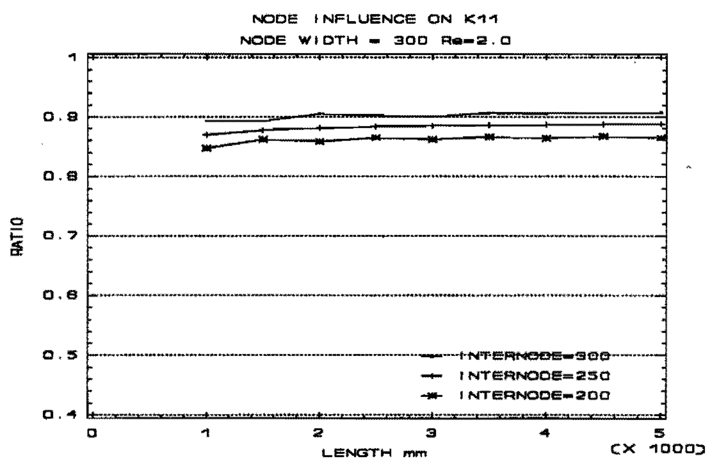


Figure F.37 Influence of node on axial stiffness. Internode lengths in mm. Conditions as indicated.

Appendix F: Structural modelling and analysis of bamboo structures

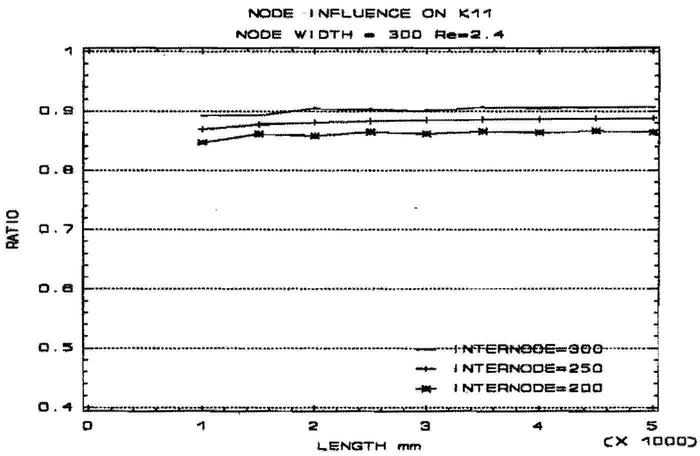


Figure F.38 Influence of node on axial stiffness. Internode lengths in mm. Conditions as indicated.

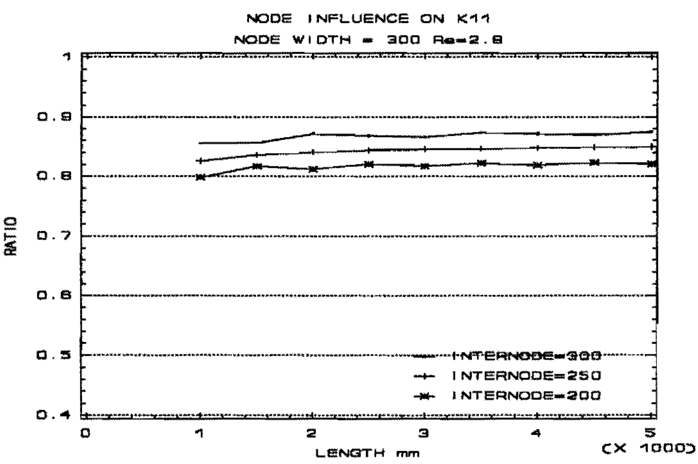


Figure F.39 Influence of node on axial stiffness. Internode lengths in mm. Conditions as indicated.

F.5-Simplified calculation of bamboo structures.

So far the question has in the first place been, to what extent conicity, variation in the elastic modulus and nodes should or should not be taken into account in the calculation of bamboo structures, and, in the second place, as a corollary, if using average properties would be precise enough for structural purposes.

Some theories have been adapted in this appendix to be able to simulate a wide range of possible situations, in relation to the combination of the different parameters involved.

In the first place it has been shown that in principleconicity accounts for an average reduction of 5% in the bending and axial stiffness.

On the other hand, a much more important role is played by the nodes, amounting to as much as a reduction of 40% in the bending stiffness and 12% in the axial one.

This suggests that within certain margins of error it is perhaps possible to reduce the average stiffness of bamboo elements by a certain factor, to account for the overall influences. It is not the intention of the author to state precisely what that factor should be. In the first place this is because such a proposal should come from a broad experimental validation, for which the data at the Eindhoven University are probably not representative enough, and second, because it is believed that such an approach is absolutely unfeasible without a correct connection to classification criteria, which does not exist yet for bamboo.

However, it is considered that light has been shed on the importance and relevance of the different parameters, and a general approach has been put forward, so that it can be further supported for any particular situation.

A general conclusion is that just taking the average values for the parameters of the culms, is indeed an over-simplification that may lead to underestimations of the deformations(though, not necessarily, to unsafe designs).

Just for the sake of a practical appreciation of what is said above, the result of figure F.40 is presented.

In this case the connections were made following the proposal of chapter 5, in such a way that wood was projected outside the culms, and then the joints were made by screwing and gluing them to steel plates, to sandwich the pieces of wood, so that the joint was as stiff as possible, as indicated in figure F.41.

It can be seen in figure F.40 that certainly the estimation of deformations based on average properties is worse than those made when the effects of the variations are taken into account. In this case, of course, culm properties were determined specifically for each of the elements of the truss, so this is strictly an academic exercise, because it would be unfeasible to do that in a real construction case. This does, however, give us an idea of the validity of the findings presented in this chapter.

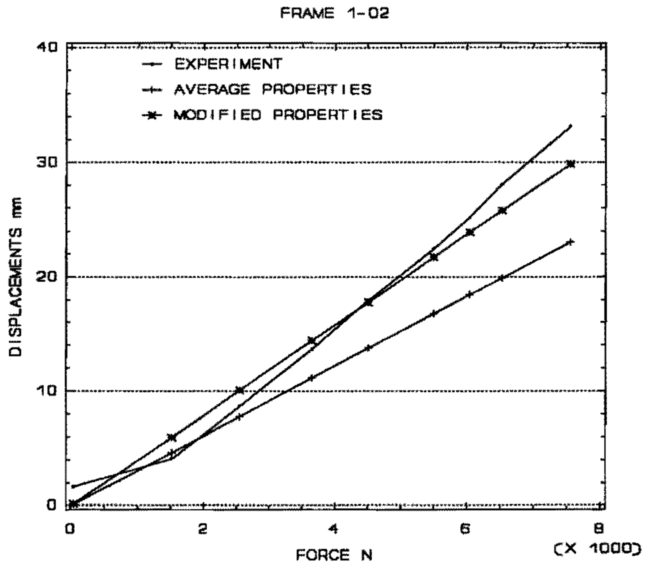


Figure F.40: Deformation load curves for a bamboo truss.

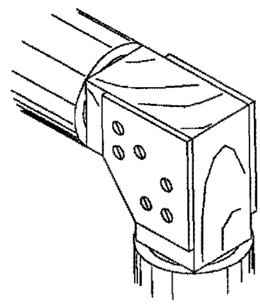


Figure F.41: Double-plated connection.

F.6-Experimental analysis of the behaviour of bamboo structures.

Figures F.42 to F.45show relations between displacements and forces for three different structural configurations.

The first of these figures shows the complete load deformation curve for a truss type of structure. The same type of connection as the one indicated in figure F.41 was used for all these cases, except that the steel plates were not glued to the wood fittings.

It can be seen that, but for some settlement at the beginning, the behaviour is very elastic, ending with a brittle type of failure. The structure is pretty stiff, and the reached capacity is very satisfactory in terms of the employ of the capacity of bamboo, but also in terms of the relation of the load carried and the weight of the structure. Failure occurred by longitudinal splitting due to tangential strain near the connection of element number 5 and joint 3. One interesting, though accidental observation, that could be made during this test, was that failure occurred in the first trial through buckling of the steel plates in the connection with the stiffly reacting steel frame, near the maximum capacity of the truss (because of an underestimation of the thickness of the plates, of course). This is interesting. Though the main advantage of bamboo in areas prone to seismic activity is its lightness, this observation suggests that in the

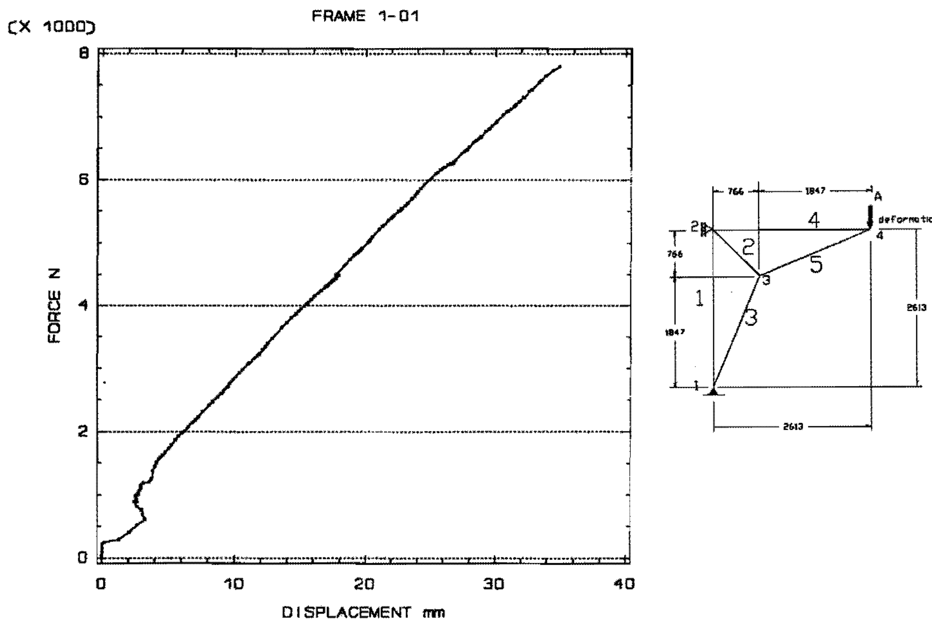


Figure F.42: Test result for a truss.

case energy dissipation is needed, this can be concentrated in the region of the steel connectors. (It is good to remember that such a possibility does not exist anywhere else in the structure of bamboo culms).

Figure F.43 presents the result on a frame-truss. In this case, and in relation to the elements, transversal forces are applied, increasing the amount of bending moments. Still, it can be seen that performance is very good. Capacity and stiffness are high. Failure happens in a brittle way here too, in the highly compressed diagonal element.

Finally, figures F.44 and F.45 show the result of testing a portal frame. In figure F.44 the scale is kept the same as in figure F.43 to facilitate the comparison, but the plot is repeated in figure F.45 to permit further observation of the behaviour. It is clear that this type of

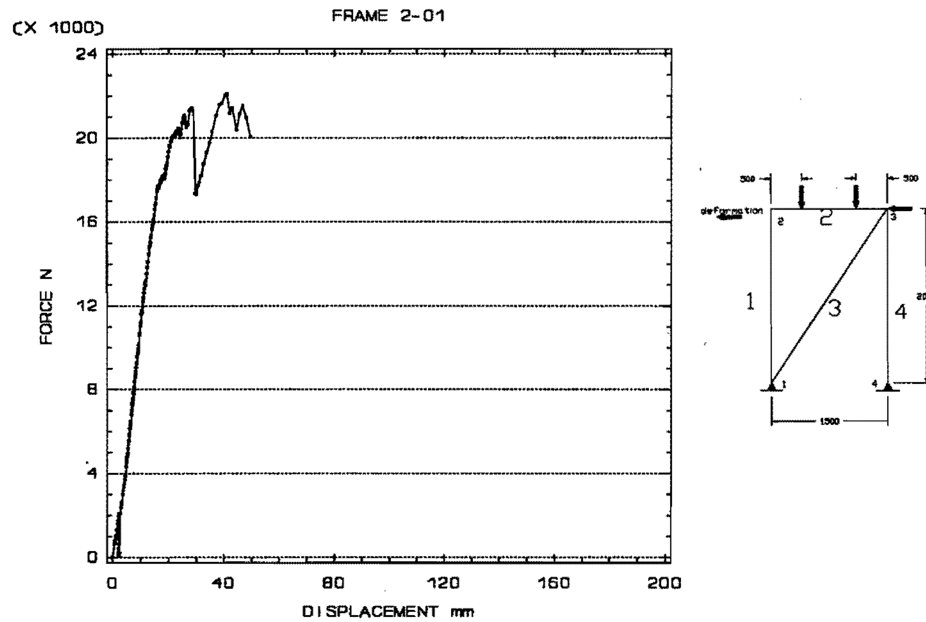


Figure F.43: Test result for a frame-frame.

application is not the most promising for bamboo culms. Deformations are very high for very low rates of loading. The induced rotations of the connections are also high, leading to early local failure.

One aspect that deserves attention is the fact that in all these tests, the amount of rotation of the connections was very high, though it always happened outside the region of the glued bamboo wood connection. This is a fact that has to be given special attention in the calculation of bamboo structures, though its treatment lies beyond the scope of this research project.

This observation led us to make the proposal in chapter 6, that a connecting system in which elements are put together using a steel fitting, helps to keep rotations under more control.

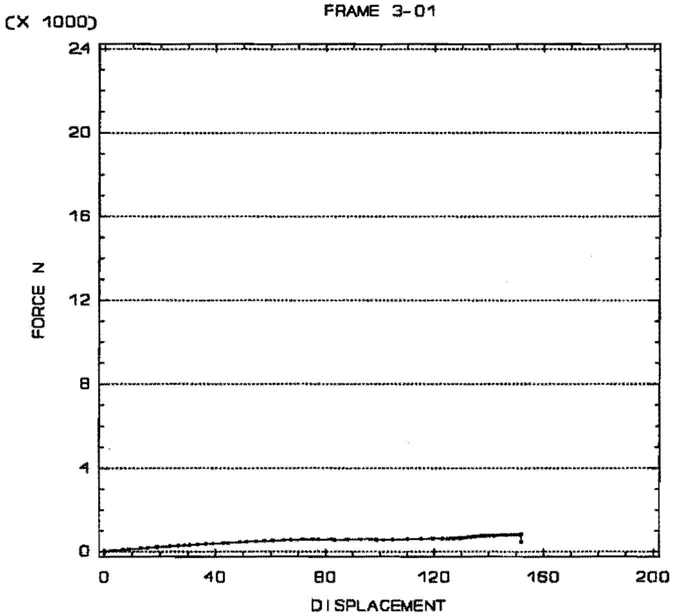


Figure F.44: Test result for a frame test.

Some limited laboratory experience suggests that even in the case of frames, this proposal might significantly improve their structural performance. In spite of that, cost may be the major limiting aspect in this type of application, thus the recommendation of using bamboo for trusses and frame-trusses is still valid.

From figure F.46 onwards, details of the tested frames are given. The respective indications are found at the bottom of each of the figures.

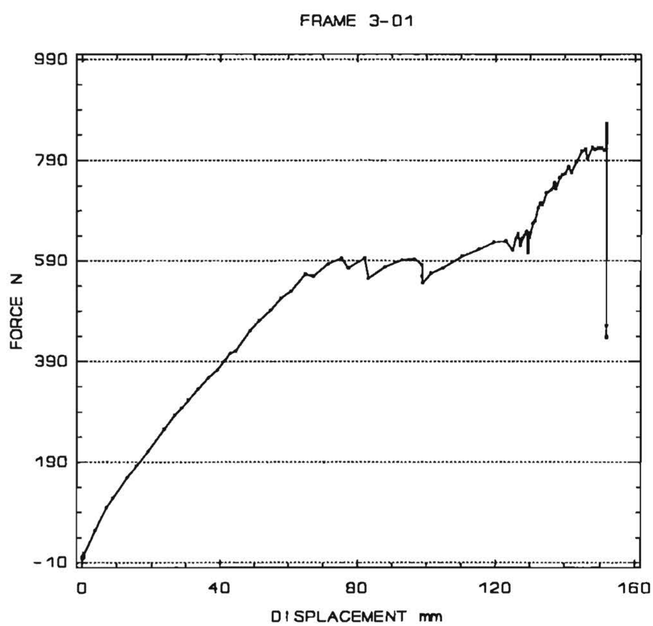


Figure F.45: Frame test, enlarged vertical scale.

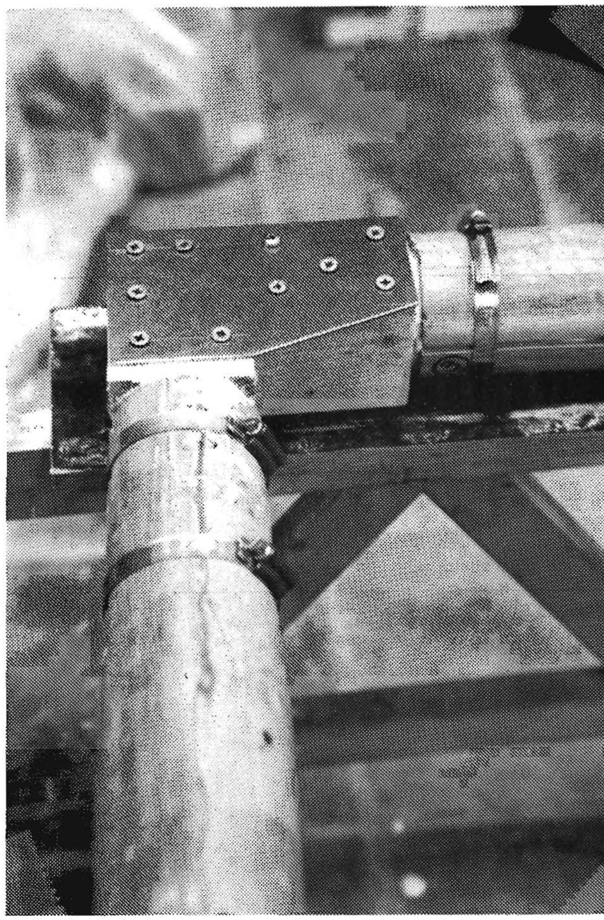


Figure F.46: Typical L-type connection employed.

The above figure shows the 1 mm thick steel plate used for joining the wood components of the connection. The connection was made in two steps. First, the wooden components were glued to the culms, and temporaly aligned to each other. Once the glue settled, the steel plates were screwed in position.

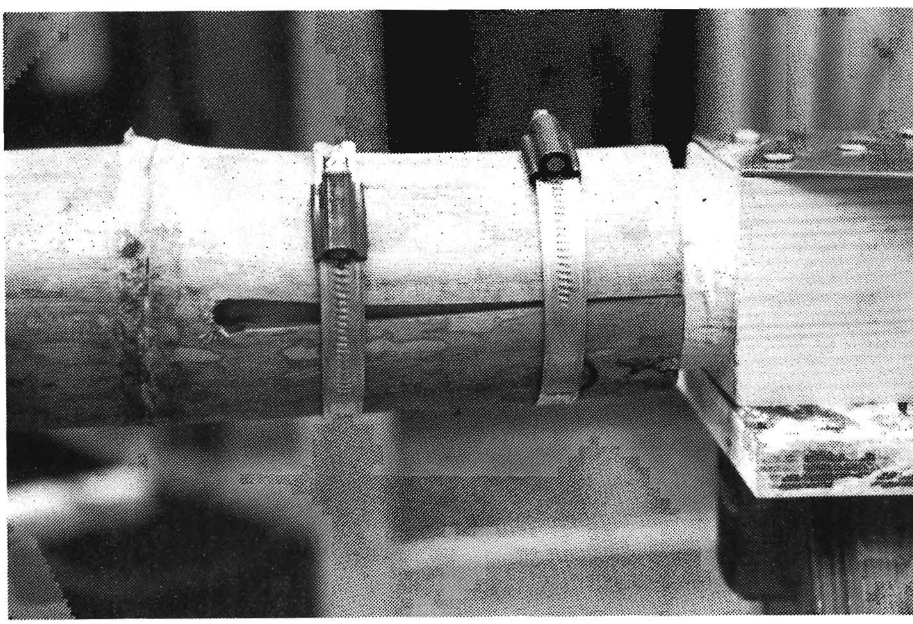


Figure F.47:Culm-to-support connection, showing diameter adjustment. The steel ring-cramp holds bamboo against the wood fitting until the glue is settled.

This figure shows how, within certain margins, size differences between the culm and the wood fitting, mean no major problem for the making of the connection.

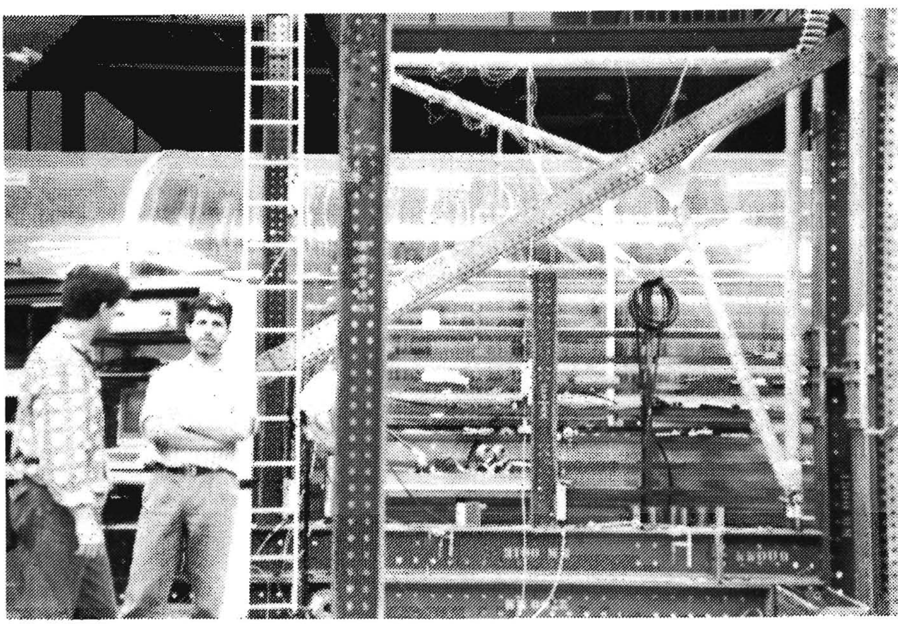


Figure F.48:Truss placed in the steel reacting frame, prior to testing.

The load was applied as indicated in figure F.42. In this particular case a maximum load of 9.5 kN was reached, with a deformation of about 35 mm. The total weight of the truss was 120 N.

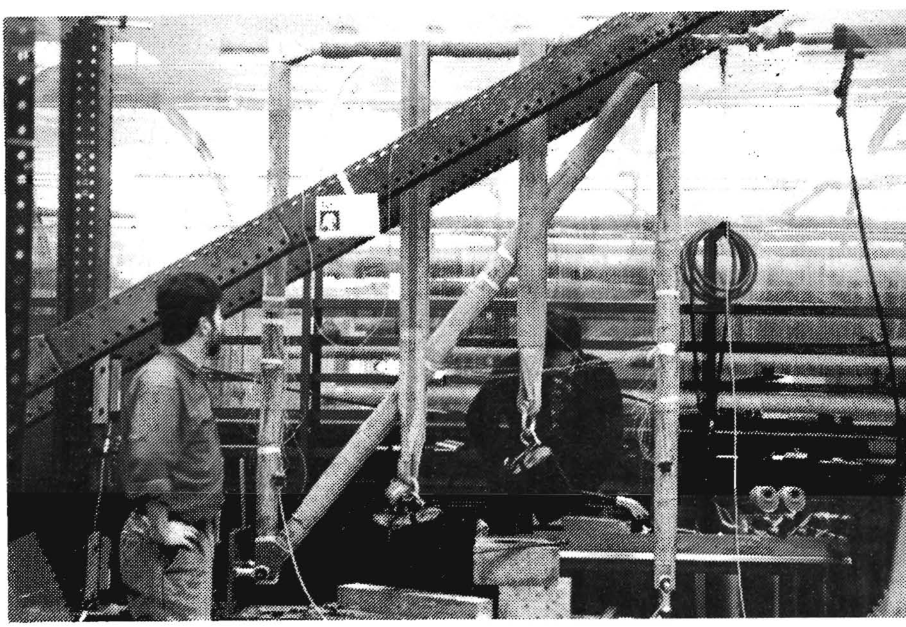


Figure F.49: Frame-truss in the process of testing.

As seen in the figure, vertical loads were hung using cottom towing bands, to keep their position on the beam. At the moment the photo was made, the test was about half way before failure.

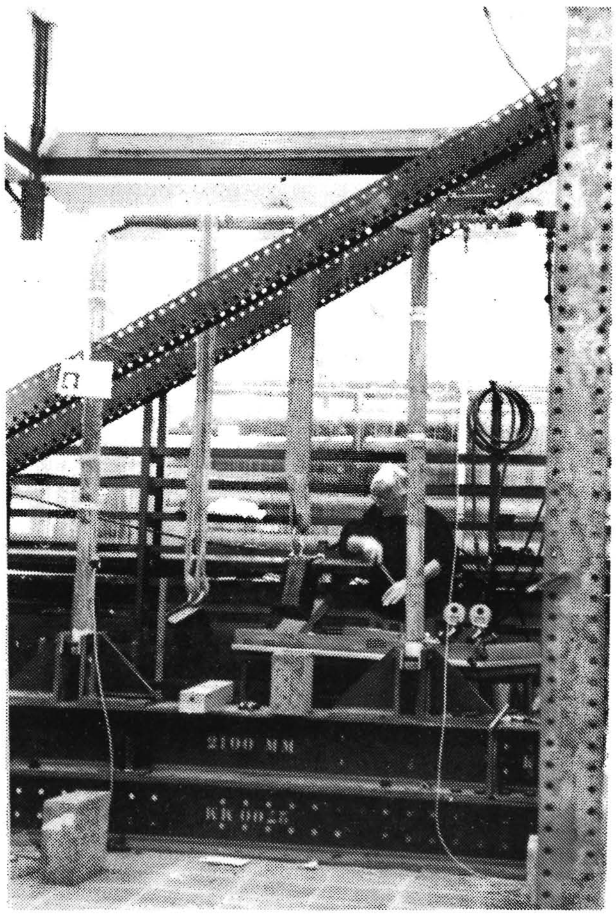


Figure F.50:Bamboo frame being tested.

The figure shows the beginning of the test of a bamboo frame. The structure proved to be very flexible, as will be shown in figure F.52.

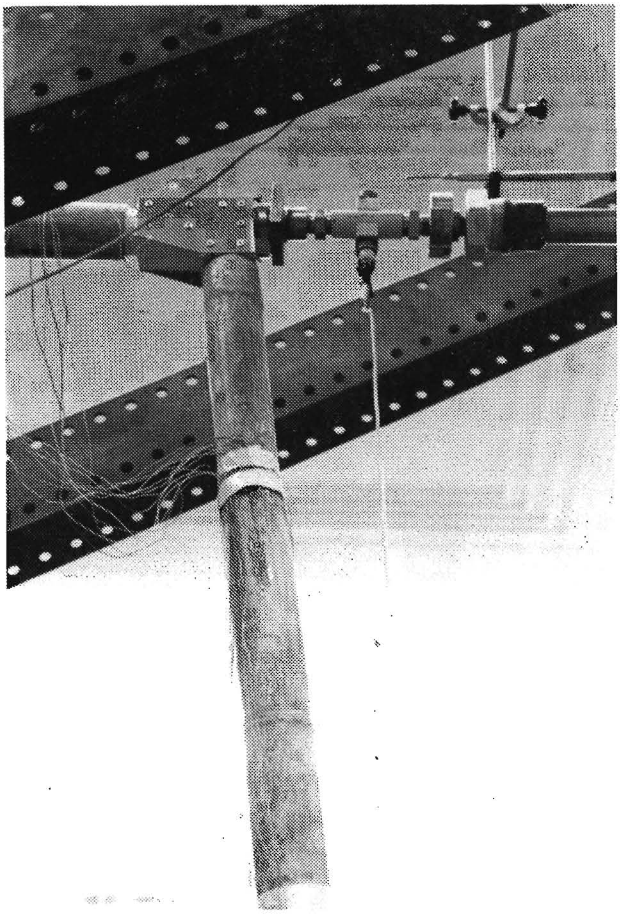


Figure F.51:Detail of the load application point in frame tests.

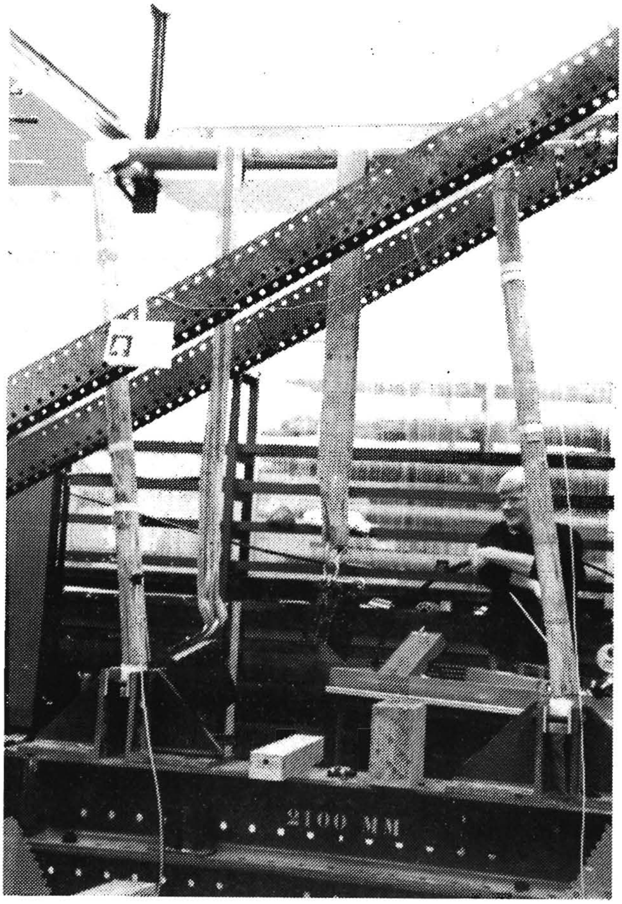


Figure F.52: Bamboo frame after failure.

The above figure shows a bamboo frame right after failure. Large plastic rotations of the top connections initiated early during the test. Final mechanism of failure included explosive splitting of the culms, in the bottom connections to the steel reacting frame.

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Appendix G: Geometric parameters of bamboo culms

G.1-Introduction

Nature supplies bamboo in all sorts of diameters, thicknesses and shapes. The exact way the external diameter changes along the length of the culm is very complex. In a strict sense, it is specific for every culm.

Though for structural purposes it is important to determine the way geometric parameters change along the length, an exact description of this may seem rather irrelevant.

A mixed sample of *Bambusa Arundinaria* and *Guadua s.p.* was studied at the Eindhoven University of Technology to determine a simple way to relate external diameter to thickness, and to see if the area of the cross-section and the second moment of area could be appropriately approximated by a simple relation to be used for structural purposes.

G.2-External diameter and thickness

Finding an easy relation between the thickness and the diameter may contribute significantly to the simplification of calculating formulae, but the level of inaccuracy must be kept under control so that this simplification does not lead lack of safety.

Table G.1 on the next page shows the result of calculating descriptive statistics for the relation between the thickness and the external diameter for a large sample, as indicated in the introduction.

As can be seen, the average ratio is 0.09. In other words, we can say that

$$t = 0.09\ \phi_e \qquad \text{eq.G.(1)}$$

To have a better idea of the level of accuracy in this assumption, an estimation of the 97% limits for the mean was undertaken, as shown in table G.2. Here, one can see that such a simplification can not lead to any significant change in calculation output, because the level of variation of the probable error is far below that related to other variables.

Appendix G: Geometric parameters of bamboo culms

Variable:	ratio thickness/diameter
Sample size	87
Average	**0.0919237**
Median	0.0849339
Mode	0.06875
Geometric mean	0.0897555
Variance	4.44906×10^{-4}
Standard deviation	0.0210928
Standard error	2.26138×10^{-3}
Minimum	0.0632911
Maximum	0.154762
Range	0.0914708
Lower quartile	0.0763889
Upper quartile	0.103659
Interquartile range	0.0272696
Skewness	0.981581
Standardized skewness	3.73775
Kurtosis	0.328252
Standardized kurtosis	0.624972

Table G.1:Descriptive statistics for ratio between thickness and diameter.

Sample Statistics: Number of Obs.:	87
Confidence Interval for Mean:	97%
Sample 1	0.0869324-0.0969149

Table G.2: 97% confidence limit for the mean ratio thickness/diameter.

Independent variable	coefficient	std. error	t-value	sig.level
diameter	0.0927350	.00226840	.8834	0.0000

$r^2 = 0.9511$

Table G.3: Regression thickness diameter.

A further look can be taken by calculating a regression of thickness and diameter. The result of such a calculation is shown in table G.3, where the resemblance between the regression coefficient and the average of the ratio in the previous tables turns out to be self-evident.

G.3-Area and second moment of area

It is possible to describe the variation of the external diameter with a linear rule like

$$\phi(x) = \phi_A - \delta_{\phi_e} x \qquad \text{eq.G.(2)}$$

In the same way the area and the second moment of area can be calculated respectively as

$$A(x) = A_A - \delta_A x^2 \qquad \text{eq.G.(3)}$$

$$I(x) = I_A - \delta_I x^4 \qquad \text{eq.G.(4)}$$

It can be proved that, from the above equations

$$\delta_A = \frac{0.1638\,\pi\,\phi_A\,\delta_{\phi_e}}{l} - 0.0819\,\pi\,\delta_{\phi_e}^{\,2} \qquad \text{eq.G.(5)}$$

and

$$\delta_I = \frac{-0.0086\pi}{l^4}\left[\left(\phi_A - \delta_{\phi_e}\, l\right)^2 - \phi_A^4\right] \qquad \text{eq.G.(6)}$$

Now, though these relationships may represent the variation of geometric parameters in bamboo in an appropriate way, it is worth seeing whether it is possible to say that, for example, both the area and the moment of area vary according to a linear relationship with the length of the culm.

A sample was studied with the purpose of estimating the possible error of such a simplification. The errors in the estimation of the middle area and the middle moment of area for the culms are statistically analyzed in table G.4.

The 97% confidence limits for the means are also calculated. It can be seen that a linear relation between moment of area and length is good enough for structural purposes, and the same can be said for the area, though the 97% confidence limit for the mean error is much wider, due to two outlying values in the sample. Without them, the results are equally good. It can in any case, be seen facts suggest that it is possible to assume linear relations between area and moments of area and the length, without significantly affecting the accuracy of structural calculations. In this simplified approach, of course no rational relation would exist between the expression for area and that for moment of area, and thus the respective parameters δ_I and δ_A must be statistically determined for every batch of bamboo according to pre-established classification criteria.

Variable:	moment area error %	area error %
Sample size	51	51
Average error	**1.52957**	**0.254269**
Median	0.426717	0.0711111
Mode	0.0	0.0
Variance	12.256	0.337543
Standard deviation	3.50085	0.580985
Standard error	0.490217	0.0813541
Minimum	0.0	0.0
Maximum	19.1415	3.17346
Range	19.1415	3.17346
Lower quartile	0.135776	0.0226286
Upper quartile	1.31713	0.219442
Interquartile range	1.18136	0.196813
Skewness	3.70161	3.69561
Standardized skewness	10.7919	10.7745
Kurtosis	14.4306	14.3744
Standardized kurtosis	21.036	20.9541

Table G.4: Descriptive statistics for the errors in the estimations of areas and moments of areas.

	moment of area
Sample Statistics: Number of Obs.	51
Confidence Interval for Mean:	97%
Sample 1	0.434294-2.62484 %

Table G.5: 97% confidence interval for the mean error in the estimation of the moment of area.

	area
Sample Statistics: Number of Obs.	51
Confidence Interval for Mean:	97%
Sample	10.0725023-0.436036 %

Table G.6: 97% confidence interval for the mean error in the estimation of the area.

SYMBOLS

α :correction factor for the calculation of cross sectional area

γ :shear deformation angle [radians]

Θ : $\sigma_x + \sigma_y + \sigma_z$

θ :coordinate angle [radians]

Δ :change

δ_E : $\dfrac{2\,E_1}{E_{av}} - 2$

δ_I : $\dfrac{\Delta I}{l^4}$, but in section E.6, where it is $\dfrac{2}{3} - \dfrac{2}{3}\dfrac{I_l}{I_{av}}$

δ_ϕ : $\dfrac{\phi_{e_A} - \phi_{e_B}}{\phi_{e_A}}$

δ_A : $\dfrac{A_A - A_B}{A_A}$

δ_I' : $\dfrac{I_A - I_B}{I_A}$

Symbols

δ'_E : $\dfrac{E_B - E_A}{E_A}$

ϵ_i :strain in direction indicated by i

$(\epsilon_i)_I$:strain in direction indicated by i , at the location I

ϵ_m :maximum strain

ν :Poisson's ratio

ρ :density [kg/m^3]

ϕ_e :external diameter [mm]

ϕ_i :internal diameter [mm]

$(\phi_e)_{max}$:maximum external diameter [mm]

$(\phi_e)_{min}$:minimum external diameter [mm]

$\phi(x)$:x function of the initial twisting deformation

λ :slenderness

σ_c :compressive stress [N/mm^2]

Symbols

σ_θ	:tangential stress [N/mm^2]
σ_x	:stress in the x direction [N/mm^2]
σ_{cr}	:Euler stress, or critical stress [N/mm^2]
σ_m	:maximum stress [N/mm^2]
σ_t	:tensile stress [N/mm^2]
τ	:shear stress [N/mm^2]
φ	:stress function
A	:area [mm^2]
A	:when used as an index, it refers to the culm extreme with the largest cross section
A_e	:area of an elliptical ring [mm^2]
a_n	:node length [mm]
B	:when used as an index, it refers to the culm extreme with the smallest cross section
D_x	: $\dfrac{E_x}{12(1-\nu^2)t^3}$

Symbols

E_{av}	:average elasticity modulus, calculated as the average of the correspondent values at the extremes of the culm
E_1	:elasticity modulus at the extreme with the smallest cross section
E_i	:elasticity modulus in the direction indicated by the subindex
E_b	:bamboo elasticity modulus [N/mm^2]
E_w	:wood elasticity modulus [N/mm^2]
e	:lateral expasion [mm]
$\hat{e}$	:lateral expansion coefficient [mm]
F_i	:as defined in eq.F.(59-61) and F.(71-73)
f_c	:compressive strength [N/mm^2]
G	:shearing modulus [N/mm^2]
h	:hard-wood code in table 4.2
HO	:null hypothesis
HE	:alternative hypothesis
I_{av}	:average second moment of area [mm^4]
I_{eq}	:equivalent second moment of area for a composed section [mm^4]

Symbols

I_p :polar moment of area [mm^4]

k_r :reaction coefficient in the radial direction [N/mm^3]

K_{ij} :stiffness coefficient

K :curvature [1/mm]

l :length [mm]

M_y :bending moment around the y axis [N-mm]

m_{xx} :bending moment per unit length [N-mm/mm]

m :slope

n_i :stress in the direction indicated by the subindex i

N :number of nodes

N_i :force in the direction of the subindex [N]

N_x^{cr} :critical load [N]

p : $\frac{E_b}{E_w}$

r :resin code in table 4.2

Symbols

r :middle radius [mm]

r_d : $\frac{\phi_{e_{max}}}{\phi_{i_{min}}}$

r_e :external radius [mm]

r_t : $\frac{t}{\phi_{e_{max}}}$

r^2 :square of the correlation coefficient

s :standard deviation; soft wood code in table 4.2

s :internode length [mm]

t :time [years]; thickness [mm]

t_g :time coefficient

T :torsional moment [N-mm]

u_i :displacement in the direction indicated by the subindex [mm]

U_b :bending potential energy [N mm^2]

U_N :axial load potential energy [N mm^2]

V_i :shear force in the direction indicated by the subindex [N]

Symbols

V_s	:coefficient of variation
$v_{b,b}$	:bonding stress due to bending [N/mm]
v_{ij}	:shear stress in the direction j and perpendicular to i [N/mm^2]
vrs	:versus, in figures and plots
vs	:versus, in statistical tables
W	:material factor in eq.C.(1)
w	:PVAc glue code in table 4.2
w_r	:radial bending displacement [mm]
$\hat{w}_r$	:radial bending displacement constant [mm]
Z_i	:as defined in eq.F.(41-42)
∇	: $\frac{\partial}{\partial x}+\frac{\partial}{\partial y}+\frac{\partial}{\partial z}$

Curriculum Vitae

The author graduated as a civil engineer from the University of Costa Rica. He obtained a Post-graduate Diploma in Advanced Structural Engineering from the University of Southampton in England, and he later undertook a Course on Seismic Engineering in the National Autonomous University of Mexico.

He was the coordinator of the Structural Design Section of the Bridge Department of the Ministry of Public Works of Costa Rica, Head of Construction of the Program of Rural Health Centres of the same Ministry, and Chief Engineer of the Civil Aviation Authority of Costa Rica. For many years he has been a private consultant on structural and seismic engineering. His professional work includes structural design and field supervision of the construction of buildings and bridges, and the structural design of especialized port facilities in Costa Rica. The evaluation of buildings after earthquake damage and the reinforcing of existing ones has been an area of professional concentration in the last years. This includes the reinforcing of hospitals and telecommunication centres.

He is Senior Lecturer on Concrete and Masonry Structures at the Department of Construction Engineering of the Costa Rican Institute of Technology. He has also served as Head of this Department, as Dean of the Faculty of Construction Engineering and Forestry, and as Vice-Rector for Research and Extension, in that University. He has contributed papers to international technical congresses on bamboo and masonry structures, having been the chairman of the First Central American Seminar on Masonry Design and Construction. He has been member of organizing committees of regional congresses on Science and Technology Policy Making.

He has worked as advisor for the Ministries of Housing and of Science and Technology of Costa Rica. With the support of the Organization of American States he carried out an evaluation mission of Housing Research Centres in Latin America. In 1988 he was invited by Appropriate Technology International, to be guest speaker of the Committee of Foreign Affairs of the Congress of the United States of America, when he spoke about Development and the Role of Universities.

Since 1989 he joined the Department of Structural Design of the Faculty of Architecture and Planning of the Eindhoven University of Technology, where he carried out the research project reported in this thesis.

Statements

pertaining to the thesis of O.A.Arce-Villalobos

Fundamentals of the Design of Bamboo Structures

1-Tests on the tangential tensile capacity and on compression for three different species show the possible existence of a common maximum for tangential strain in bamboo (see sections 2.2.1 and appendix B).

2-Bamboo culms do not fail in compression, in bending or shear, but do fail when a maximum tangential tensile stress is reached (see chapters 2, 3, and appendices B and D).

3-The majority of fittings based on some sort of penetration normally used in construction, are not suitable for bamboo because they create high tangential stresses (see section 2.2.2).

4-The use of glue and a wooden fitting has not only proven to be a way of taking and transmitting forces to the culms, but it actually reinforces them (see chapters 4 and 5, and appendices C and D).

5-Contrary to general belief not the fibres, but the matrix, is the major source of strength in bamboo culms (see sections 2.2.2, 5.1, 5.4 and 5.6).

6-The nodes contribute negatively to the axial and bending stiffness of the culm, and to the parallel tension capacity of bamboo (see sections 2.2.2 and G.4).

7-The calculation of bamboo structures can easily be made by using a factor to take the variation of the modulus of elasticity, and the cross-section along the culm into account (see appendices G and H, and chapter 7).

8-The results of tests on bamboo trusses are highly encouraging. The balance between lightness, on one hand, and capacity and stiffness, on the other, gives a vote of confidence to the possibilities of this material (see appendix G and section 7.10).

9-Bamboo culms can be appropriately employed in trusses, space trusses and frame trusses, provided connections are well taken care of. Bamboo culms are not adequate for frames nor for beams.(see appendix G and chapter 7).

10-It is necessary to pay more attention to practical design problems in future bamboo research activities (see chapter 7).

11-Some people would like to 'preserve third world cultures' like keeping monkeys in a cage. There has to be a balance between cultural preservation and the right of people to decide about their own future. This is only possible through education and knowledge. No lasting freedom is possible in the darkness of ignorance.

12-Through the power of science we have managed to develop our knowledge to unthinkable horizons. Our brain has reached formerly unimaginable frontiers. Now it is time we start developing our hearts, and to embrace ethics as the centre piece of our behaviour.

13-Our ego is so huge that we believe it is we who will destroy this planet with our careless treatment of the environment. The truth is that we will destroy ourselves first, not the earth, by making it uninhabitable, unless we seriously incorporate environmental considerations as a routine part of everything we undertake.

14-We no longer elect our leaders, we only 'buy' images. Like with other things in the market, we do not buy what is the best for our interests, but just what is sold in the best manner. No wonder actors become politicians. No wonder politicians are becoming actors.

15-On top of the unfair division of the world according to the level of one's possessions, or according to the colour of our skin, there is the unfair distinction between men's and women's chances of success and happiness. Even in the most developed countries women are still destined to secondary roles, violence of all kinds, even the right access to knowledge and education, being men's main instrument of domination. Men's appreciation of women is limited to Mother's day and to the sexist concession that **behind** every great man there is a great woman.

16-Some people blame science and technology for everything that is going wrong on our planet. But even if they are right, science and technology, in a proper moral context, are the most powerful instruments to make a better life possible.

17- My family and I have been favourably impressed by the Dutch people. We have found people who are warm, hospitable and generous. Everywhere we went, people made us feel well treated, respected. Part of our hearts will remain here, with our friends, for ever.

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