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FINITE ELEMENT ANALYSIS OF MOVABLE, DEPLOYABLE ROOFS AND BRIDGES

By

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Abstract

Deployable space frames consisting of straight members that can be stored in a compact bundle and can be deployed into large-span, load-bearing structures by simple articulation are investigated. Besides their ease of storage and their fast erection procedure, they offer the additional advantage of reusability. Therefore, they can constitute a promising alternative for movable, small and medium scale roofs and lightly loaded bridges.

First, the concept of deployable structures is reviewed and their potential applications are outlined. Then, a special type of structures that are self-standing and stress-free in both the deployed and the folded configuration is introduced, and the geometric constraints required to achieve this behavior are explained. The main part of the paper deals with modeling issues that had to be resolved in order to carry out the geometrically nonlinear finite element analysis of the structure during deployment, which includes an elastic snap-through behavior associated with large displacement bending and buckling. Several levels of refinement of the model are described, until a model is obtained that is considered to provide sufficient accuracy. Finally, the importance of the nonlinear deployment analysis for the design of deployable structures is emphasized.

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1 Introduction

The deployable structures analyzed in this paper are prefabricated space frames that consist of straight bars linked to each other in the factory and packaged together in a compact, condensed bundle. Their erection results from simply articulating this bundle, thus deploying very rapidly large-span, load-bearing structures. The procedure is illustrated in Figure 1, which shows the folded, an intermediate, and the deployed configuration of a deployable arch model. Deployment of this structure was accomplished in seconds.

More specifically, self-locking deployable structures that are stress-free and self-standing in both the deployed and the folded configuration are investigated here. These structures offer considerable advantages in comparison to previous alternatives [7, 13, 14, 16, 17, 21]. Their design, however, is more complicated, because of the strict geometric constraints they have to satisfy, and because of their nonlinear behavior during deployment.

The interest in deployable structures is due to their promising applications and the advantages they offer when compared to conventional, non-deployable structures for certain types of applications. Their main advantage is the ease and speed of erection, which are crucial in emergencies, bad weather or situations with time constraints. Equally important is the ease of transportation and storage due to the compact shape in the undeployed form. Other advantages are reusability, minimal skill requirements for erection, dismantling and relocation, and competitive overall cost.

The most promising applications of deployable structures are related to recreation, the con-

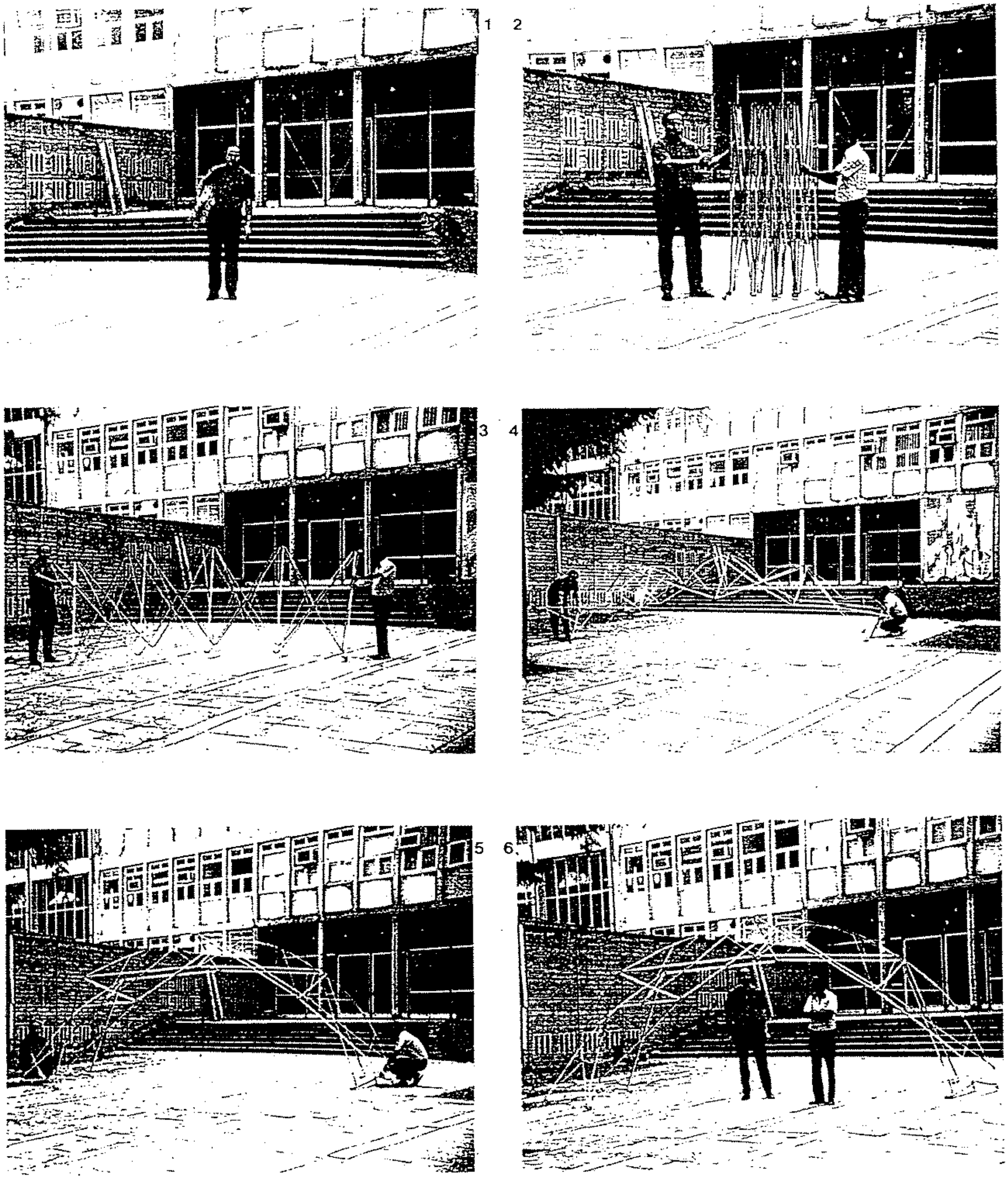


Figure 1: The Deployment Process of a Circular Arch

struction industry, temporary facilities, exhibitions, or structures in space. Some examples of possible applications and potential users are emergency shelters or bridges after earthquakes and other natural disasters, temporary buildings and shelters in remote construction sites, temporary protective covers during construction, surveying measurements, or cold weather concreting, domes for sport facilities, scaffolds, forms, skeletons or reinforcement for permanent structures, camping tents, exhibition structures, and traveling theaters. Deployable structures are of even greater interest in the aerospace industry, where air-inflated structures would not survive, and severe constraints apply to both payload capacity of space ships and to building time in space.

The great challenge in designing deployable structures is because erection and dismantling are an integral part of the functionality of the product and effect the feasibility of the design. The goal for the designer of a deployable structure should be to design the members for regular service loads, and obtain deployability without adding weight to the structure or decreasing significantly its load bearing capacity. This paper focuses on the structural behavior of deployable structures during the critical phase of deployment and addresses modeling and analysis issues during that phase. This task is of special interest for the type of self-locking deployable structures investigated here since they exhibit a highly nonlinear behavior during deployment.

2 Self-Locking Deployable Structures

The problem of compact and transportable structures is not new. Such structures have been investigated, designed, and constructed by many engineers. Prior efforts however, suffer from some important drawbacks. Only recently have modern concepts for sophisticated deployable structures been introduced and neither their geometric configuration nor their structural behavior during and after deployment have been investigated to a satisfactory extent.

Some previous designs [9, 10, 11, 12, 18, 19, 20] consist of structures that are stress-free before, during and after deployment, and behave as mechanisms during the deployment process. Therefore, stability can only be achieved through the use of external locking devices. In general, locking consists of adding intermediate members between nodes of the structure after it has been deployed to the desired final configuration. This process requires temporary support in the deployed position, experienced workers, and scaffolding for large structures. This costs in time and money and is therefore undesirable. A second category [18, 19, 22] includes structures that are self-supported in the erected form, and do not require any type of external locking mechanism. However, the existence of bent elements in the final deployed configuration of these structures makes them more susceptible to buckling under service load, which reduces their load bearing capacity.

Needs and perceived opportunities for improvement led to the development of a new version of deployable structures that are investigated here [7, 13, 14, 16, 17, 21]. The idea is to

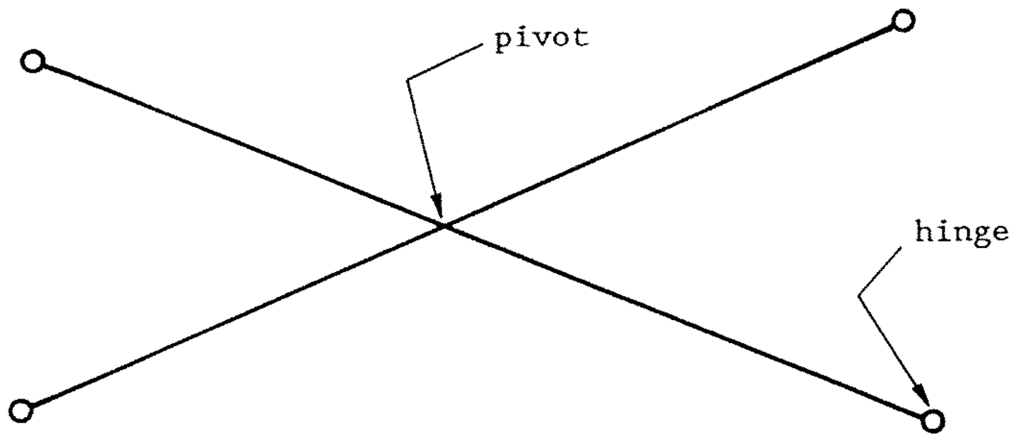


Figure 2: A Typical Scissor Like Element

use a geometric configuration for which the structure is stable and free of stresses in both its compact, folded form and its final, deployed form. Only during the deployment process do some of the members bend in order to maintain compatibility. While the structure is deployed forces in the members increase gradually and, after reaching a peak value, drop and return to zero at full deployment. The strain energy that had built up in the members during deployment is released by a snap-through 'clicking' into the self-sustained, stable form of a load-bearing structure, with no residual internal stresses.

The basic structural unit of these structures is called Scissor Like Element (SLE). It consists of two rods connected at an intermediate point through a pivotal connection and hinged at the four end points to other end nodes of other SLEs [Figure 2]. The SLEs are assembled in such a way that they form structural units with a planview of a polygon. Each side of the polygon is a scissor like element, and additional scissors connect the corners to an internal point. All regular polygons work, as do some non-regular shapes, such as a symmetric trapezoid. By combining several of these basic units we can create structures of various flat and curved geometric configurations.

Research on this type of deployable structures has been going on at the Civil Engineering Department of M.I.T. for the last three years. Detailed information about the geometric configuration of these structures is included in [13, 17]. The experimental work done in parallel at the Technion in Haifa, Israel is described in [7]. The present paper will focus on modeling issues for the nonlinear finite element analysis required to simulate the deployment process.

3 Finite Element Modeling for Deployment Analysis

The structural analysis of deployable structures involves two phases. The analysis in the deployed configuration under service loads is quite straightforward, since the behavior is largely linear. Analyzing the structure during deployment is a much more complicated problem. Large displacements occur, therefore, second order theory must be used. Because of the difficulties associated with such nonlinear analyses, earlier investigations concerning deployable structures were mainly qualitative and very little analytical or numerical work was used.

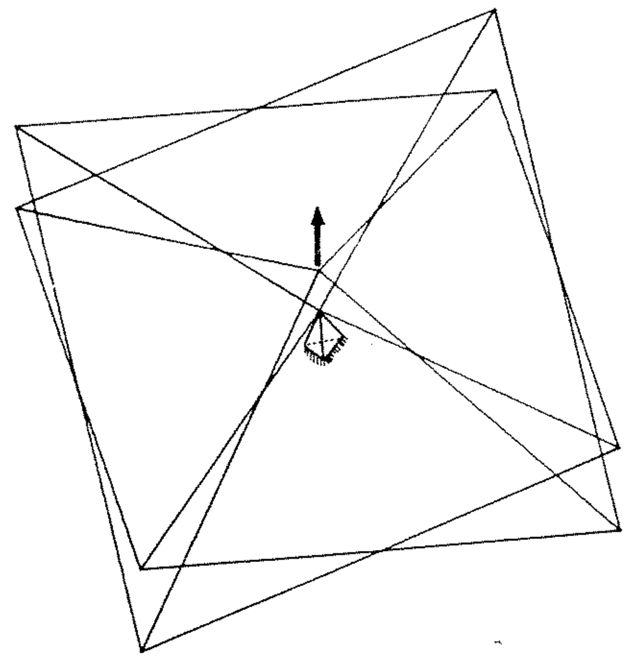


Figure 3: Method of Deployment

Experimental observations showed that the stresses occurring during deployment are very sensitive to changes in the geometry or member properties, and can become comparable or even much larger than stresses due to service loads. This may result not only in inefficient solutions, but also in making the feasibility of deployable structures questionable due to either very stiff response during deployment or reduced load bearing capacity under service loads. Therefore, both a qualitative understanding of the behavior and a quantitative evaluation of stresses occurring during deployment constitute an integral part of the design of deployable structures.

A relatively simple unit with a square planview, consisting of eight SLEs has been analyzed (Figure 3). As illustrated in Figure 3, the simplest method of deployment was applied to this single polygonal unit. The lower center node of the unit is considered fully supported, while the upper center node is free to move vertically only and is subjected to a vertical concentrated load. All other nodes are free. This deployment procedure offers the important advantage of symmetry, and therefore simplifies the analysis considerably since only part of the structure has to be analyzed.

Some basic modeling assumptions and initial conclusions have already been outlined in [14]. They are summarized also here for completeness. The strains and stresses that develop in the members of the structure during deployment result from compatibility requirements between the members of inner and outer SLEs. Hence, second order effects have to be taken into account, and a 'large displacements - small strains' formulation is appropriate [3].

In the collapsed configuration all nodes of the structure lie theoretically on a straight line. Furthermore, a small deformation has to take place before the structure can carry any loads. Therefore, the deployed configuration was used as initial state for the analysis, i.e. dismantling was simulated instead of deployment. Nonlinear beam elements [1] were used initially to model inner SLEs, while outer SLEs, subjected only to axial stresses, were represented by truss elements. After introducing auxiliary coordinate systems, the master node / slave node technique was used to model the pivotal connections.

Since the type of response of deployable structures was unknown at the beginning of this work, the automatic step incrementation method was initially used [2, 5, 6]. After a better understanding of the behavior was acquired, the more economical BFGS method was employed [3]. In some cases line search was required in order to achieve convergence.

Curves that describe the variation of the required external load and corresponding internal member forces, as the structure deforms from the deployed configuration to the collapsed one, are presented in [14]. The load-displacement curve indicates a snap-through type of behavior for the structure. The above simplified analysis was useful in order to gain initial insight into the structural behavior, and the results agreed qualitatively with the observed behavior of physical models. However, the actual values of the required deployment load were smaller for the numerical model than for an experimental one. This indicated that more sophisticated modeling was required for design purposes.

Initial results reported in [14] were based on a very simple finite element mesh with one Hermitian beam element for each member of radial SLEs, and one truss element for each member of circumferential SLEs. The modeling of the outer members is adequate because they are stressed in tension only throughout deployment. Refinement however, was necessary for the radial members that are subjected to both bending and compression. The Hermitian beam formulation used neglects the degradation of bending stiffness due to axial forces [1, 8]. To compensate for that, a finer finite element mesh has been used that consists of 4 elements per member, hence, 16 elements per scissor. The new mesh is shown in Figure 4. Symmetric deployment conditions are still assumed, therefore only one fourth of the structure is analyzed. The influence of the mesh refinement is illustrated in Figure 5.

The next step was to switch from Hermitian to isoparametric beam elements [3, 4]. When used for nonlinear analyses, the Hermitian beam element requires a high order of numerical

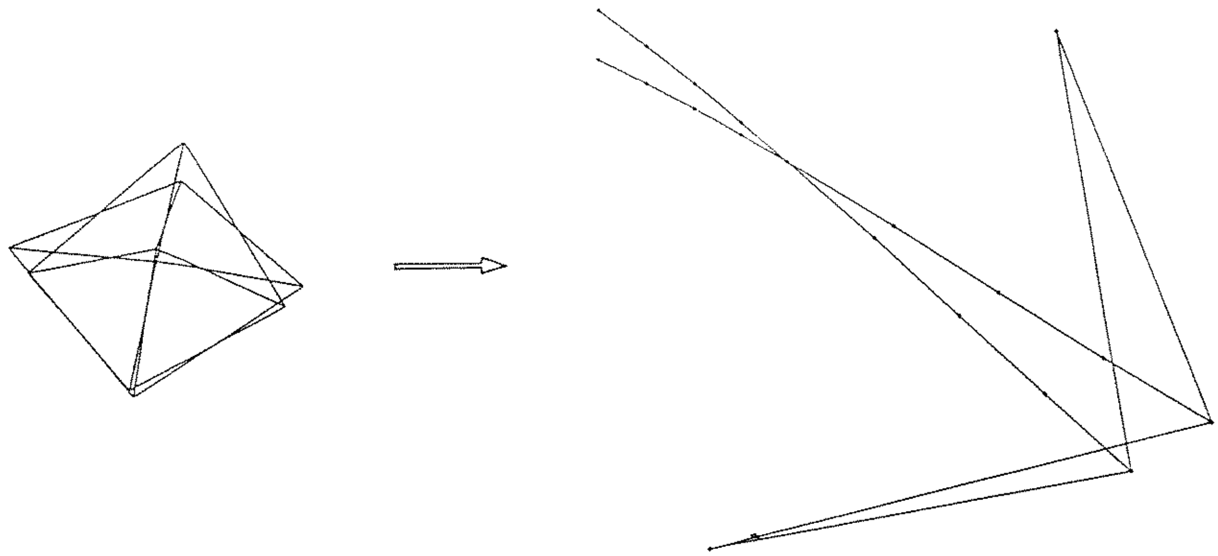


Figure 4: Symmetry Considerations and the Finite Element Mesh

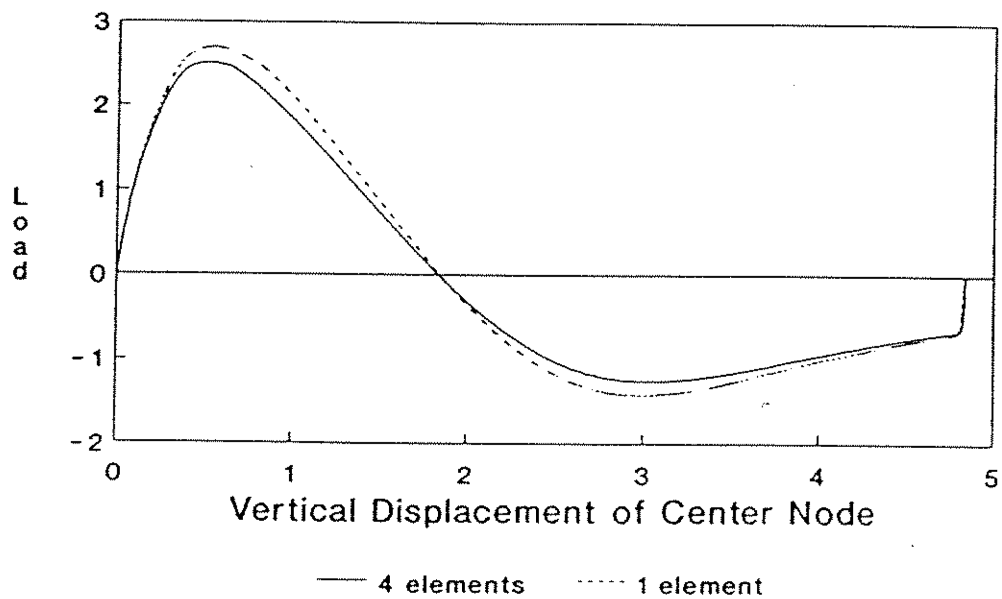


Figure 5: Influence of Mesh Refinement

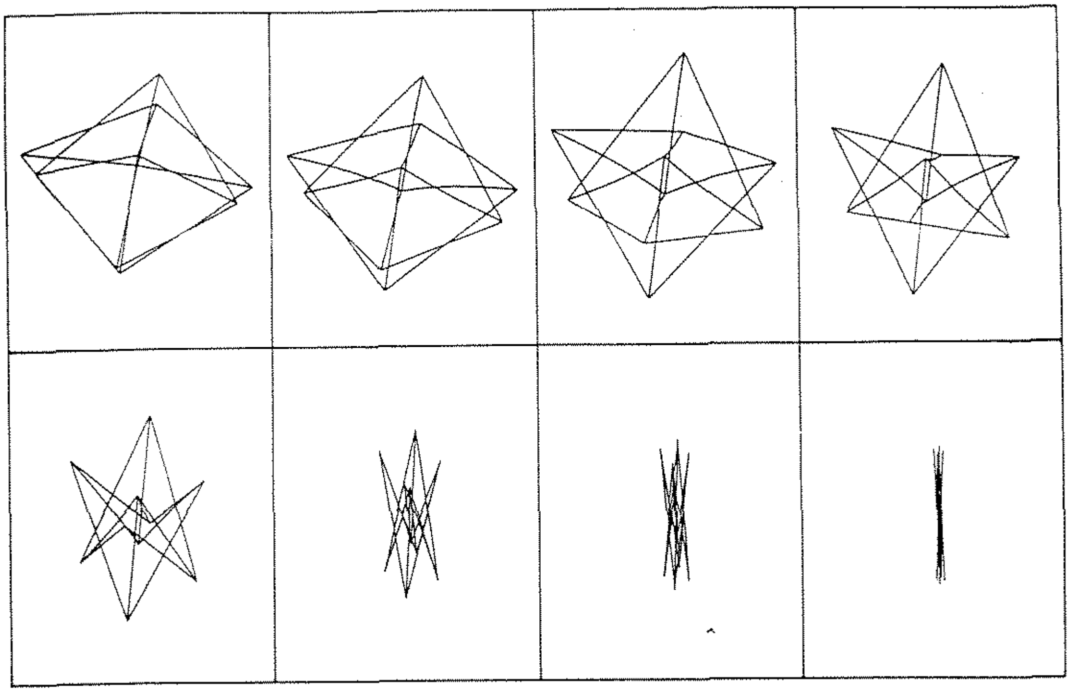


Figure 6: Successive Deformed Configurations from Deployment to Collapse

integration, and is therefore very expensive. By using 2-node isobeam elements we reduced significantly the cost of the analysis. Numerical integration of first order, which is equivalent to mixed interpolation for the transverse displacements and the shear deformation, has been used along the longitudinal axis of the 2-node isoparametric beam elements to avoid shear locking [3]. Figure 6 shows plots of successive deformed configurations of the structure during the dismantling process. In Figure 7 the load-displacement curve is illustrated. The type of response is qualitatively the same as the one obtained with the simpler model. A snap-through type of behavior can be observed. The almost vertical slope of the curve in the collapsed configuration corresponds to the sum of the axial stiffnesses of the members, that by then lie practically on a straight line.

The model developed for the analysis of single units for flat structures has been extended to

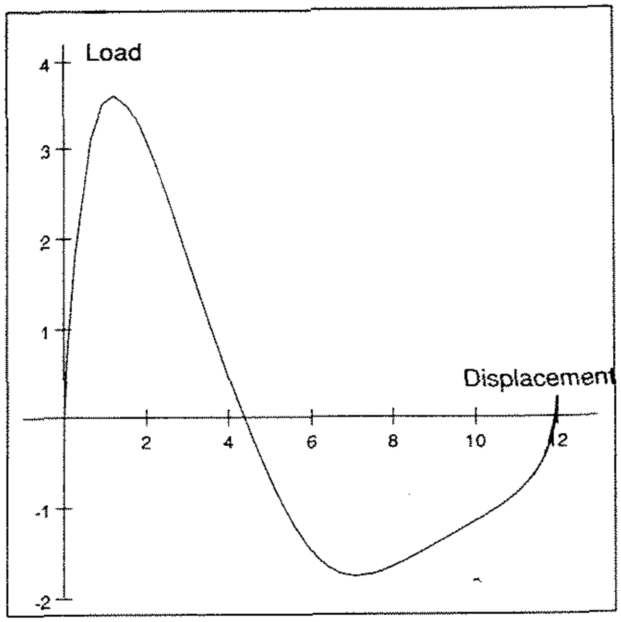


Figure 7: The Load-Displacement Curve

include curved spherical structures as well. The only thing that changes in the model are the nodal coordinates, which should be such that lines connecting corresponding lower and upper nodes meet at the same point, the center of the sphere.

By observing the deformation of the members of the experimental models one can identify the behavior as a combination of in-plane bending and out-of-plane buckling. This behavior could not be captured with the 'perfect' geometry used for our initial finite element models. During the analysis of these 'perfect' models the members remain in their plane and are subjected to in-plane bending and axial compression. As a result, the response is stiffer than in reality.

In order to model the real mode of behavior of the structure during deployment an initial

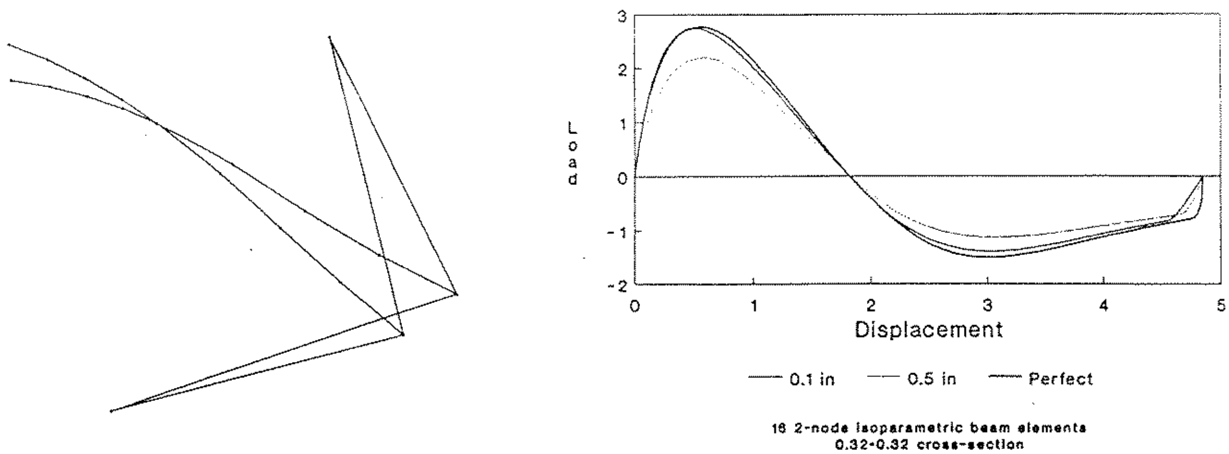


Figure 8: Influence of Imperfections on the Structural Response

imperfection has been imposed on the members of radial scissors, in the form of distorted out-of-plane initial node coordinates. This causes the radial scissor to deform both in-plane and out-of-plane during the deployment process, and to respond in a more flexible manner. The distorted initial mesh as well as the influence of this type of imperfections on the load-displacement path is illustrated in Figure 8.

These studies were performed using isoparametric beam elements with square cross-section. To verify the expectation that member imperfections would have a more significant effect on structures with weak out-of-plane members, an analysis was attempted where the ratio of width to height of the cross-section was 1:2. Convergence difficulties related to a stiffening in the response that made no physical sense were observed. These problems have finally been attributed to an insufficient (for this problem) formulation of warping that is used for the

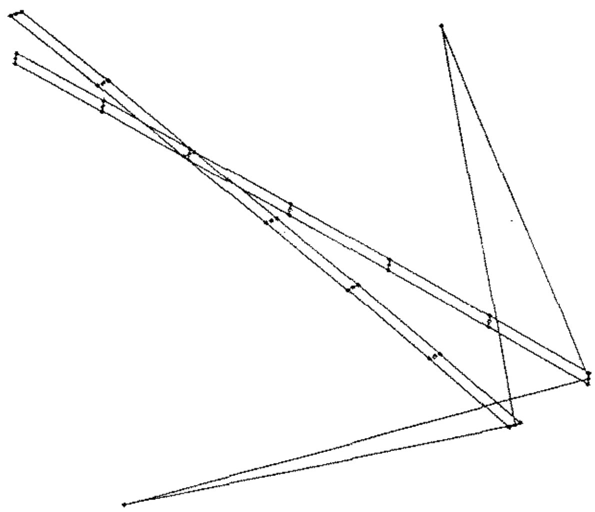


Figure 9: Finite Element Mesh with Shell Elements

isoparametric beam elements [4].

Thereafter, 9-node shell elements [3] have been successfully used to avoid this problem. Figures 9, 10, and 11 show respectively the finite element mesh used, a series of plots of successive deformed configurations from deployment to collapse, and the load-displacement curve that has been obtained. Figure 12 shows the influence of member imperfections that is indeed more important here than for the square cross-section.

The next development in our analysis efforts was the modeling of discrete size for the hinges. Figure 13 shows the type of hinge that has been used for the physical model. It also illustrates the idea of modeling this hinge as a stiff grid composed of short 2-node isoparametric beam elements. Figure 14 shows the actual finite element mesh used. In Figure 15, the analogous series of deformed configurations is presented.

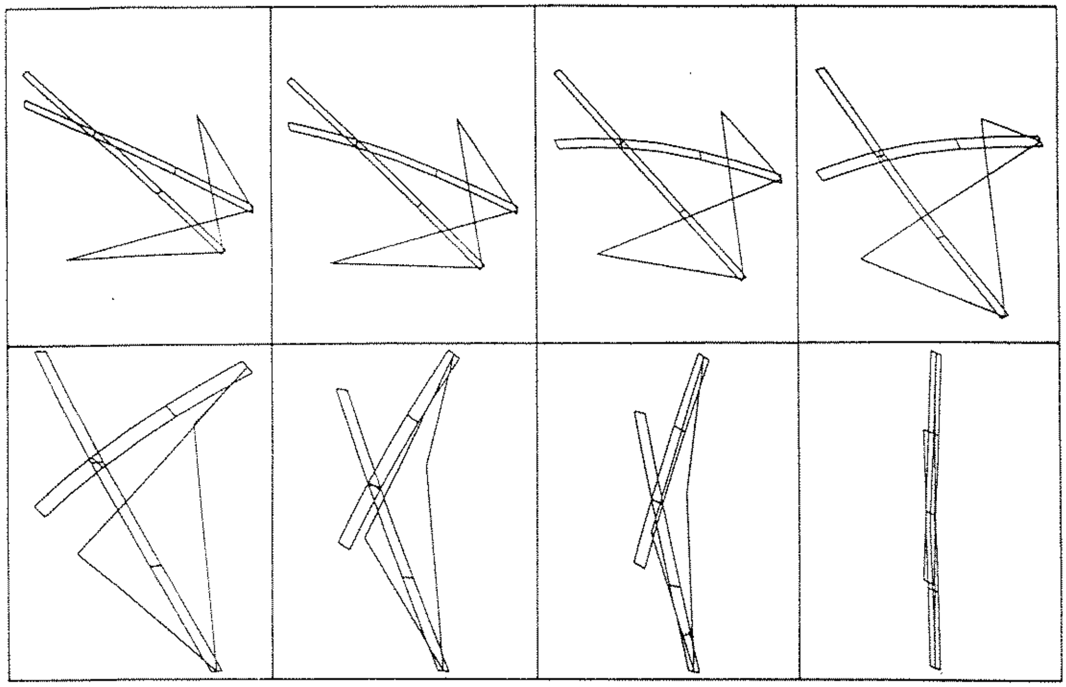


Figure 10: Successive Deformed Configurations of the Shell Mesh

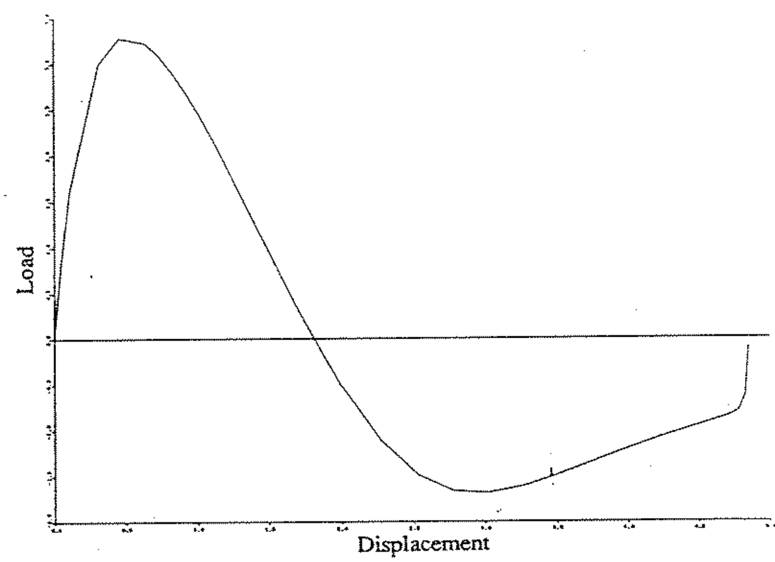


Figure 11: The Load-Displacement Curve for the Shell Mesh

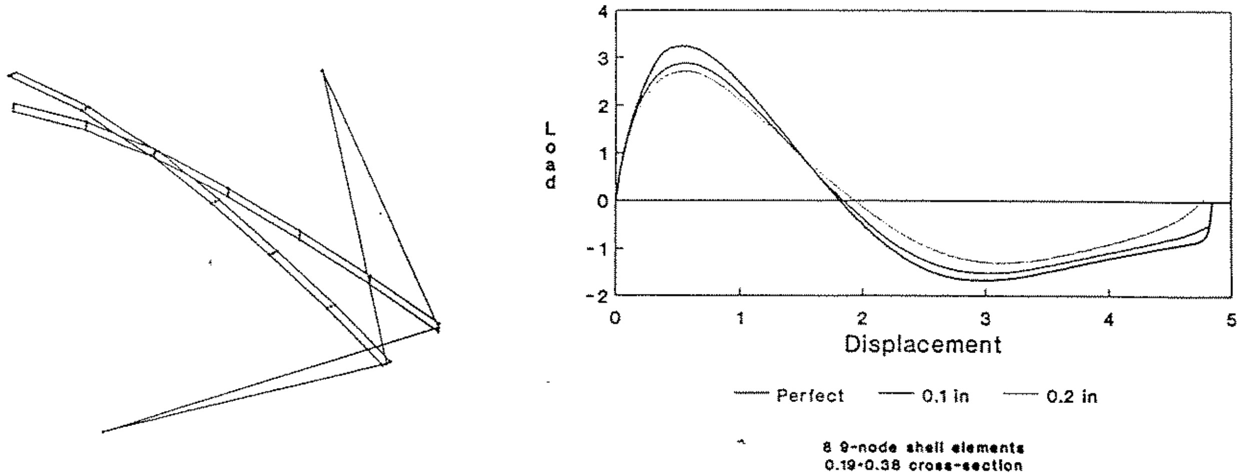


Figure 12: Influence of Imperfections for the Shell Mesh

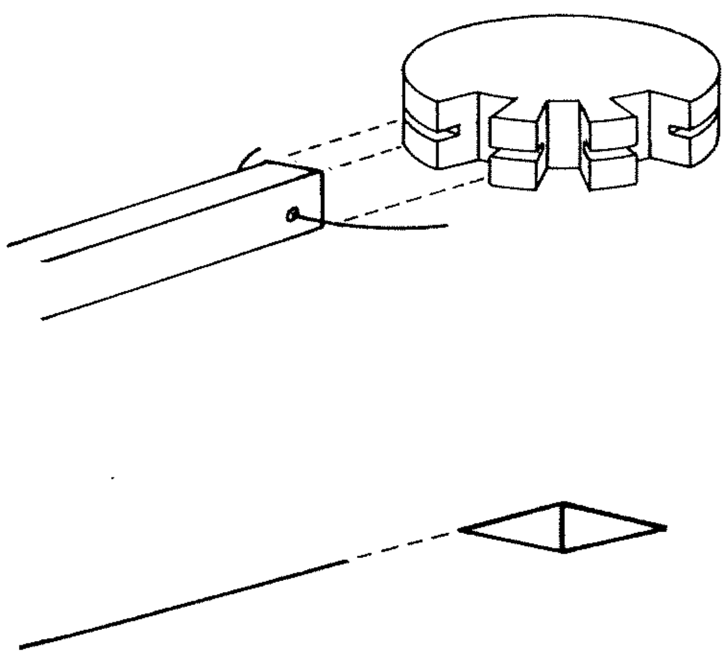


Figure 13: A Real Hinge and its Finite Element Model

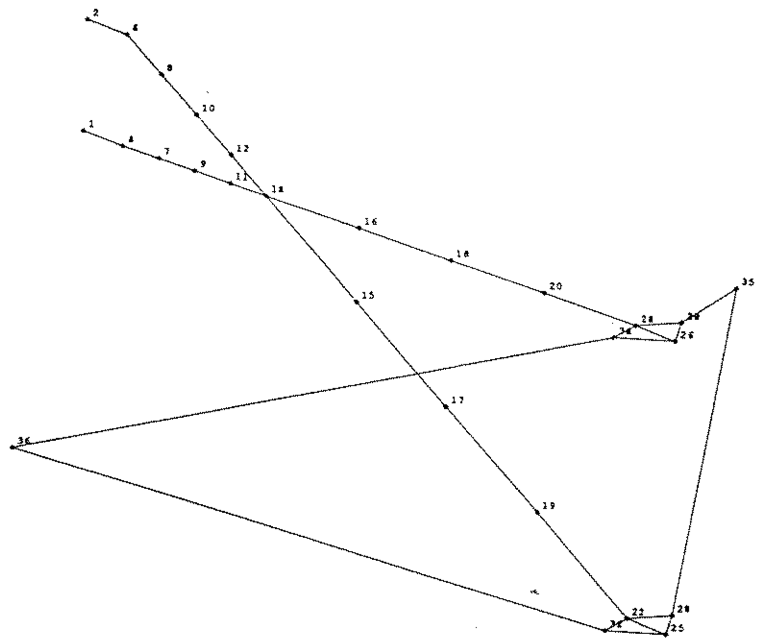


Figure 14: The Finite Element Mesh with Discrete Joint Dimensions

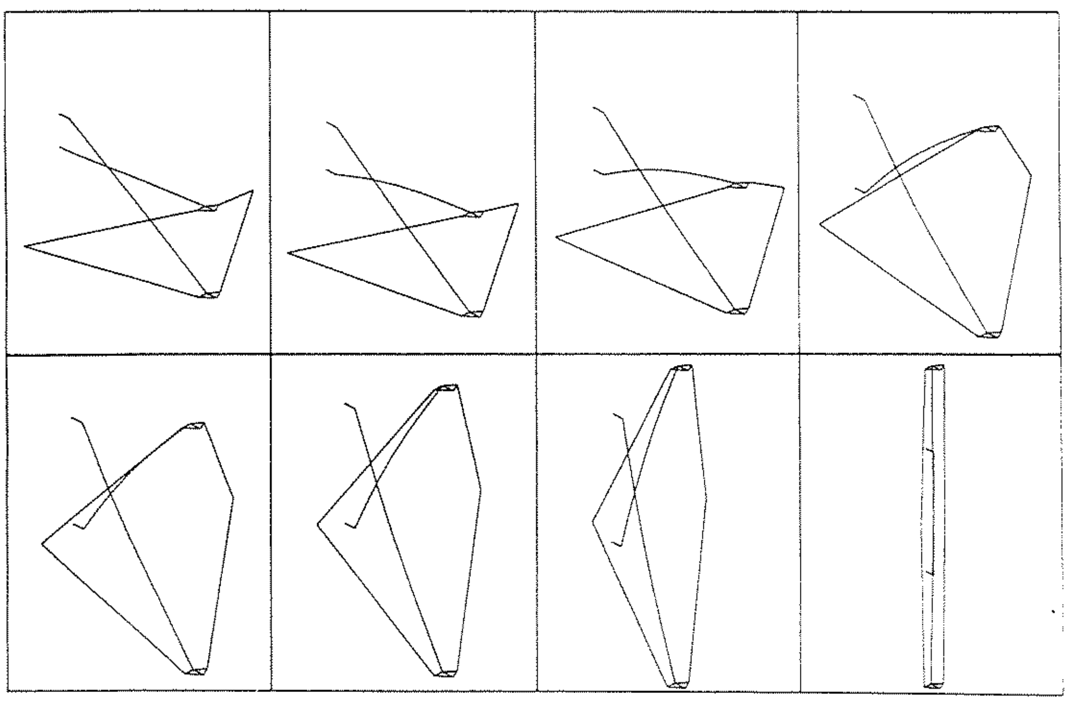
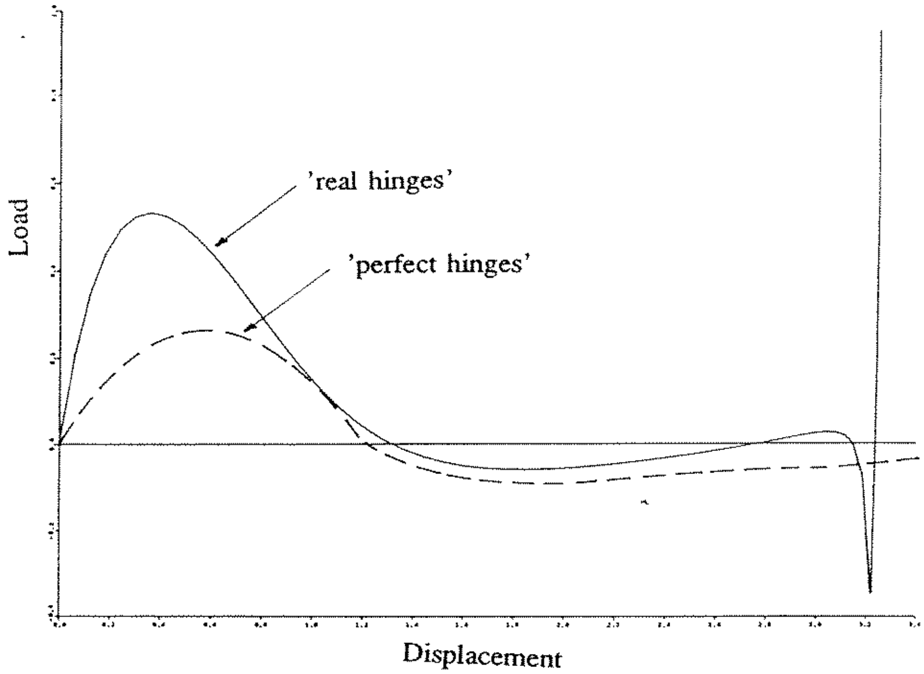


Figure 15: Successive Deformed Configurations of Discrete Joint Mesh

The influence of the stiffness of the isobeam elements that constitute the grid has been investigated. This influence is insignificant for the major part of the load-displacement curve, but the last part of the curve is affected, and local disturbances occur (Figure 16), that have been attributed to geometric incompatibilities, and to the fact that the members that are connected to the hinge are not concurrent during the deployment process. A plot of the stress variation in the truss elements revealed that these disturbances are associated to a sudden increase in tension. This led us to increase slightly the length of these members, which resulted indeed in obtaining a smooth load-displacement graph (Figure 17). However, this change in length creates compression of the truss members and a change in sign of the load-displacement curve before the structure is completely dismantled. This detail is not of great importance for the final model that includes also the effect of friction between members and joints.

In both figures (16 and 17) the load-displacement curve for 'real' hinges is compared to the one for 'perfect' hinges. As expected, an upwards shift of the curve can be observed, due both to shorter members and to the more realistic geometric modeling.

The latest effort is the inclusion in the finite element model of frictional effects. The dominant mechanism of friction during deployment is the contact between the two bars that form an SLE because they are not idealized straight lines but have a finite width, and therefore a significant contact area (Figure 18). This forces them to deform out-of-plane and act upon each other with normal forces that induce friction during the relative rotation between the



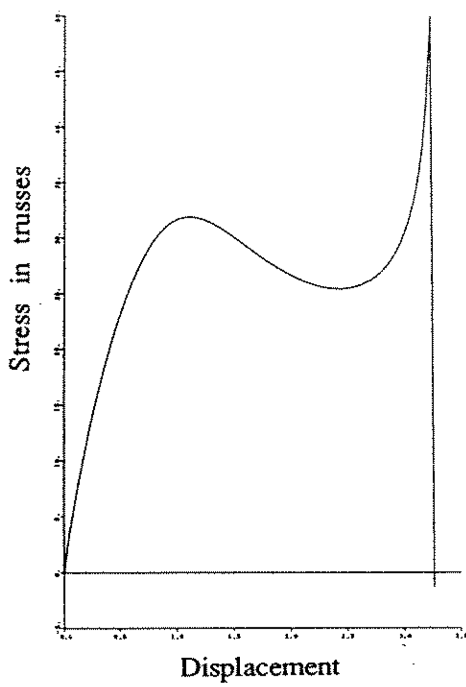
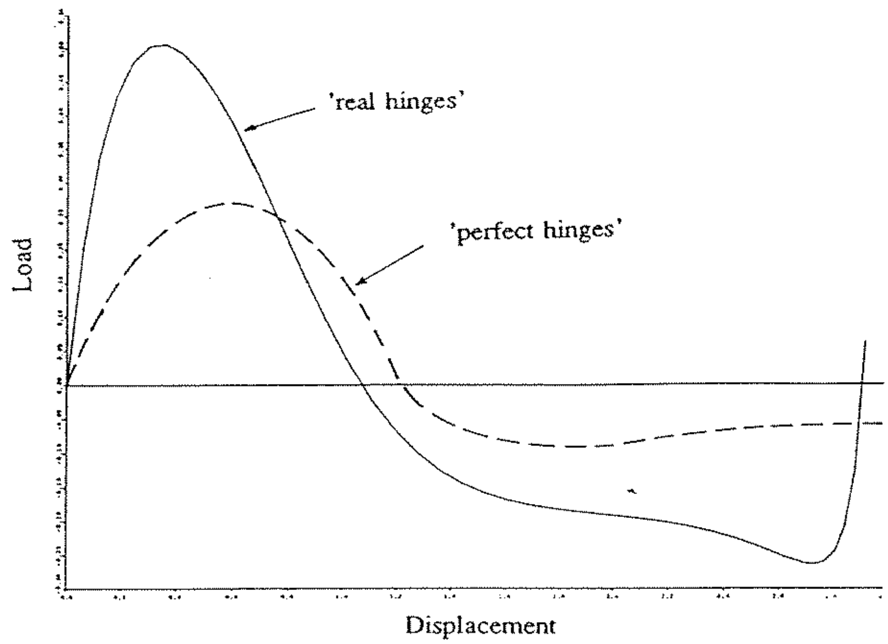


Figure 16: Results for Real and Perfect Hinges (Theoretical Member Lengths)



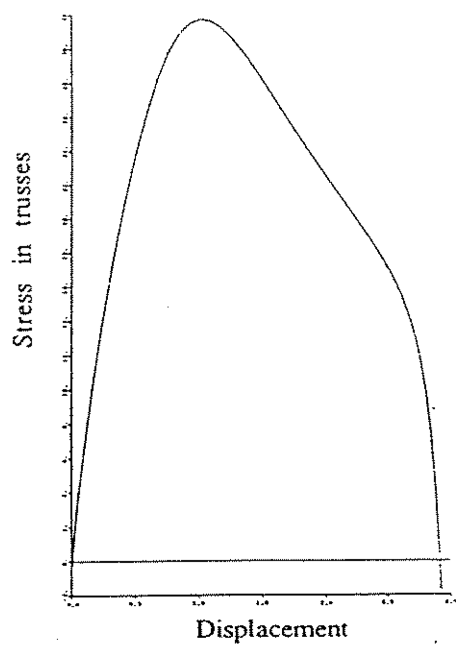


Figure 17: Results for Real and Perfect Hinges (Longer Truss Members)

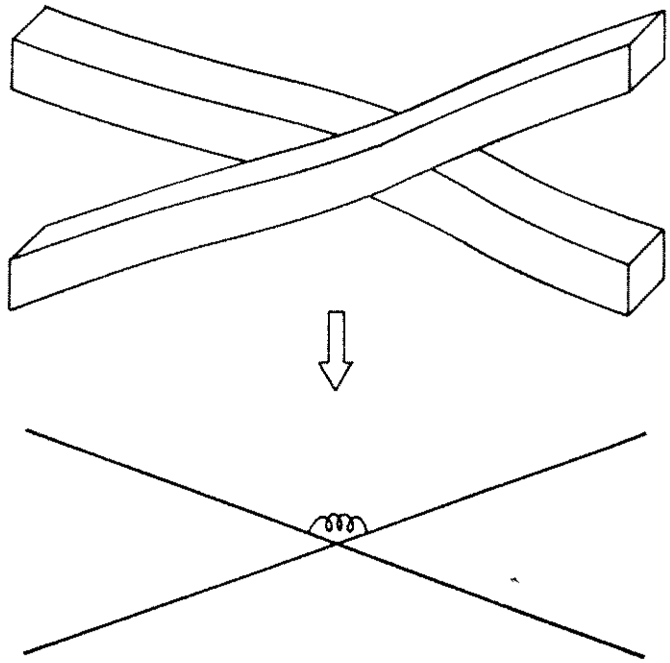


Figure 18: Friction Mechanism and Equivalent Model

two bars. This effect has been taken into account by calculating the resisting frictional moment as a function of the angle between the bars, and including in the model a nonlinear rotational spring that produces the same moment. More details about the frictional model will be included in a future paper currently under preparation. However, a comparison of numerical and experimental results for a curved pentagonal deployable unit, illustrated in Figure 19, indicates the accuracy of the model and its adequacy for design purposes.

4 Summary and Conclusions

A new type of deployable structures has been investigated in this paper. It consists of structures that are self-standing and stress-free in the folded and the deployed configuration,

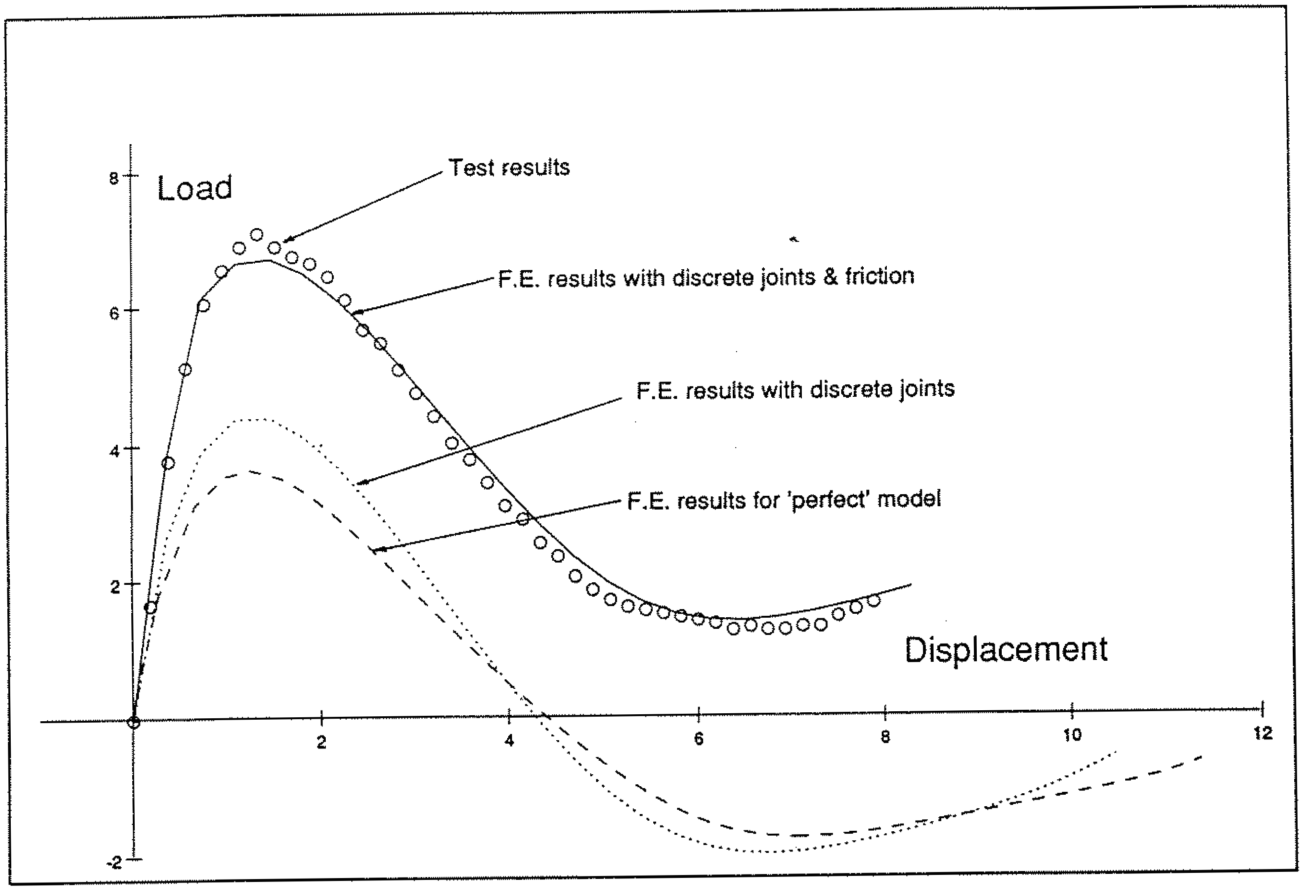


Figure 19: Load-Displacement Curves for a Curved Pentagonal Unit

but develop stresses during deployment due to geometric compatibility requirements.

This nonlinear response is of great importance for the design of such structures. The basic design philosophy is to design the structure for service loads in the deployed configuration, and achieve deployability without decreasing significantly this load bearing capacity. An optimum design is the one that finds the best trade-off between desired stiffness in the design configuration, and desired flexibility during deployment.

The importance of a reliable numerical tool for the nonlinear deployment analysis is therefore great. Several refinement levels of a finite element model are described here. The final model, including the effects of initial imperfections, joint size, and friction is found to show excellent agreement with experimental results.

Acknowledgment

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