Recent Research on Spatial Structures

Motohiko Yamada*

1. Philosophy of IASS

In the month of October 2001, the IASS International Sysmposium was successfully held at Nagoya, Japan. IASS, the abbreviation designation for International Association for Shell and Spatial Structures, was founded in 1950 by researchers, engineers and designers of the whole world gathered in sympathy with the E. Torroja's appeal. The new president of IASS Mamoru Kawaguchi who was also the chairman of the organization committee for the above symposium writes as the philosophy of IASS in the recently publicshed symposium's summary reports[1] as follows;

(1) to pursue the rationality in the transmission of force inherent in shells and spatial stuctures, and
(2) to have a great interest in the connection of the mechanical rationality with the form of such a structure.

And further, he emphasizes in particular the fact that the each member of IASS is regarded as a individual irrespective of his or her nationality, belief or thought and this thoroughgoing individualism impels IASS to function smoothly and vividly.

The present author believe that these ideas mentioned by M. Kawaguchi is essentially common to the foundation of any kind of association and its development, and so, the present author wants to give them to the newborn Korean Association for Spatial Structures and to all of its members with his best wishes.

2. Recent Trends in Analytical Research Subjects on Spatial Structures shown in the Previous Symposium

The accepted papers for the previous symposium counted 199 covering full categories of general remarks, analyses, designs and realized projects of shells, membrane structures, cable structures, hybrid tension structures, spatial frame structures, formations, optimum formation technics, load evaluations, vibrations, response control methods, structure-ground interaction analyses, morphology, morphological concepts, tensegric structures, retractable structures, new design concepts, new methods of construction, and technical innovations.

---

* New Industry Creation Hatchery Center
Tohoku University, Professor

The number of papers concerning the analyses of spatial frame structures are about 39, and these can be classified into the following 6 terms with some noteworthy subjects. The final numbers in brackets show the number of papers concerned[1].

① numerical computation methods and new finite elements(6)
  improvement of finite elements for thin open section members, and finite elements for members taken its local buckling into consideration,
② buckling analyses(17)
  buckling characteristics of spatial frame structures prestressed with tensile rods, evaluation of buckling strength, and buckling analysis using detailed finite element models,
③ vibrations and wave propagation(4)
  propagation of longitudinal impulsive waves
④ seismic responses and earthquake-resistant designs(7)
  evaluation of equivalent static seismic loads, amplification or attenuation in seismic responses caused by structure-ground interactions, earthquake-resisting properties, and applications of vibration control devices or base isolation systems
⑤ wind responses and wind-resistant design(2)
  evaluation of equivalent static wind loads based on dynamic response analyses of domes subjected to fluctuating wind pressure obtained the wind tunnel testing
⑥ maintenance and reinforcement technics(2)
⑦ damping properties(1)
  useful suggestions based one the analysis of many vibration measurements for real structures.

The studies in the field of ④ increased in the last few years and are especially paid attention to in Japan. Maintenance and reinforcement technics of ⑥ are inevitable subjects depending on some natural or artificial surroundings or the aging, and the further development of studies in this field is expected int the near future.

The number of papers given by classifying the above papers into structural method or construction are, in order of amount, 22 for single-layer roofs, 6 for double-layer roofs. 5 for truss beams, 2 for double-layer walls, and others. The 16 papers ofr single-layer roofs are included in ②, and the 4 papers belong to ④. The 2 papers for double-layer walls are both related to the earthquake-resisting properties in ④.

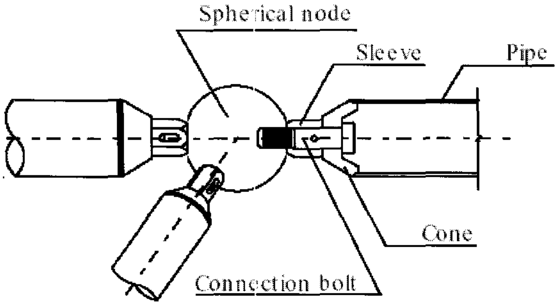
3. Introduction of Recent Subjects on Spatial Structures Studied in the Author's Laboratory

3.1 Evaluation of Strength of Ball-Joints subjected to Axial force and Bending Moment.

Recently, in Japan, A huge double-layer structure has been realized against such a precedent, in which the members and the joint parts except for the connection bolts are all of aluminum alloy and their dimensions are naturally big compared to those of steel. And so, putting such structural properties to use, the outside cord members have been planed to sustain not only the self-weights but also snow loads, wind loads and seismic forces, that is, both axial force and bending moment act at each joint as well as in the member.

On the other hand, the author and his co-worker, Akihiko Obata, have found out that even in the usual truss system both axial force and bending moment act at each joint inevitably because of lever-action effect. This lever-action effect in the joint can be characterized by bending of the connection bolt, which is caused by eccentric tensile force acting at the bolt-head according to rotation of the corn.

As shown in Fig. 1, each end of members, which is connected to a systematized ball joint with a single bolt, called here a spherical node, has been considered as a pin-joint as usual, and all of members have been designed so as only to transmit axial forces. Such a systematized joint is usually composed of the four parts, a spherical node, a sleeve, a connection bolt and a cone, though the names of these parts are not always common to all systems.



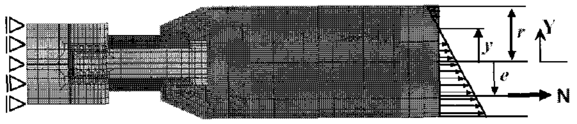
〈Fig. 1〉 Component of a systemized ball joint

The newest report concerning this subject is shown in Ref. 6 though it has been written in Japanese, in which a new proposal for the evaluation method of joint strength is shown. It is compared to the other two simple methods previously proposed by the author[5]. Fig. 2 shows an example of the finite element models used in the present detailed elastic-plastic numerical analyses.

Fig. 3 shows the three models of the assumed stress distributions.

As shown in Fig. 3-1, Model 1 has the simplest stress distribution, in which the sleeve does not transmit tensile stress and the connection bolt does not transmit compression stress. Their sections have a reference line shown by a broken line in common with each other. Both sections are in full plastic states under the above assumptions.

As shown in Fig. 3-2, Model 2 has the second simplicity, in which the sleeve does not transmit tensile stress but the connection bolt transmits both tensile and compression stresses. Their sections, as same as the Model 1, have a reference line shown by a broken line in common with each other. Both the sections are in full plastic states under the above assumptions.



〈Fig. 2〉 An example of finite element model

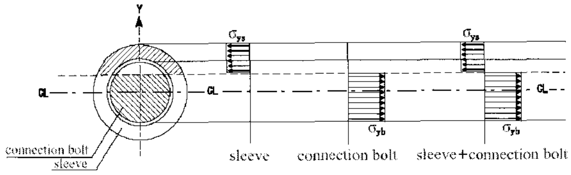


Fig. 3-1 Model 1

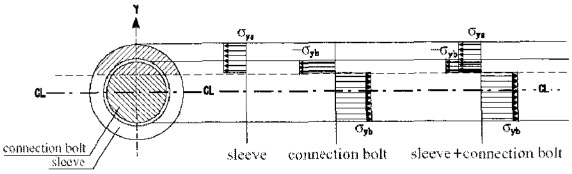


Fig. 3-2 Model 2

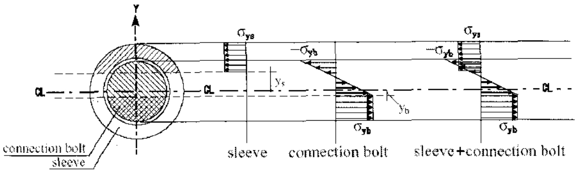


Fig. 3-3 Model 3

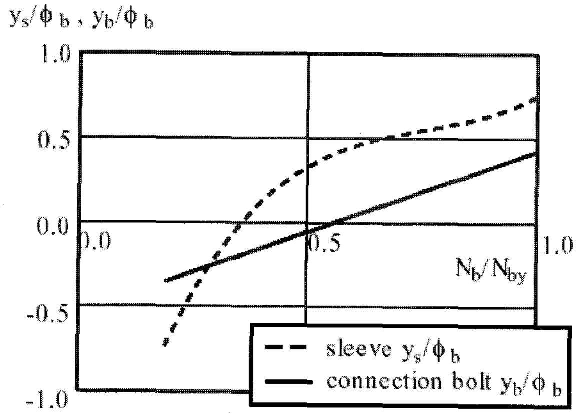
〈Fig. 3〉 Three models of assumed stress distributions

As shown in Fig. 3-3, Model 3, the new model, has a rather complex elastic-plastic stress distribution compared to the above two models. The sleeve does not transmit tensile stresses and the connection bolt transmit both tensile and compression stresses as same as Model 2. However, the reference lines in the sections of the sleeve and the connection bolt shown in the two broken liens do not always coincide with each other, where the reference line for the connection bolt is defined as the line at the compressive elastic limit. Under these assumptions, the sleeve's section is in a full plastic state, but the connection bolt's section is in a elastic-plastic state whose compressive fiber stress is in the elastic limit.

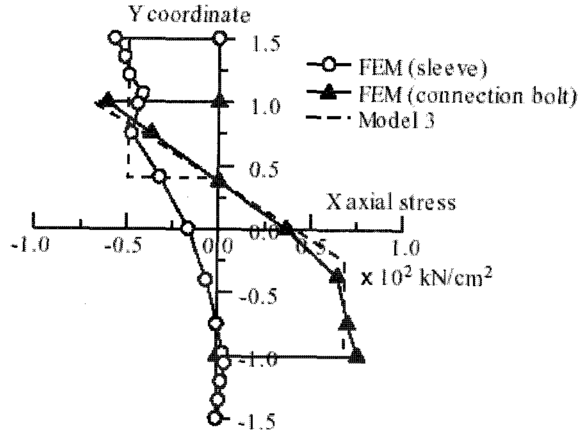
Model 3 has an indeterminate condition of the relation between the above two reference lines. This relation has been determined by the use of the stress distribution at the 1/200 offset strength along the curve of the joint rotation versus the bending moment which were obtained by a detailed finite element analysis.

Fig. 4 shows the values of the two reference lines for the sleeve and the connection bolt, which are given with the ratio to the connection bolt's diameter. Fig. 5 shows an example of comparison between the stress distributions given by the numerical computation and Model 3.

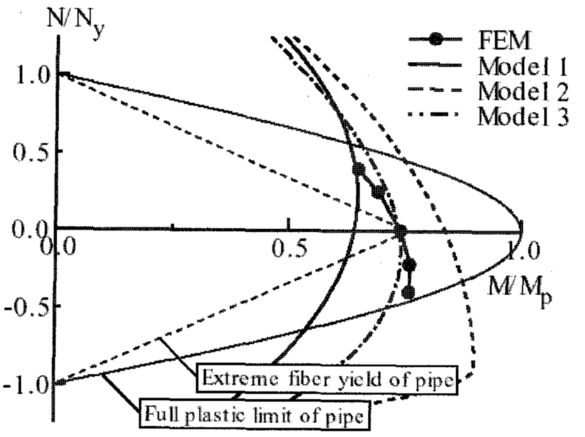
Fig. 6 shows an example of the N/Ny-M/Mp correlation curve of joint strength obtained for the model shown in Fig. 2. Ny and Mp are the pipe's yield axial force and full plastic bending moment, respectively. The thin dashed line is the pipe's elastic limit, and the thin solid line is the pipe's plastic limit, respectively. The curves of solid line with ●-marks is the 1/200 offset strength obtained by the FEM analysis. The three curves of bold solid, bold dashed and bold chain lines are the joint strength curves evaluated by Models 1, 2 and 3. Model 1 underestimates the joint strength in the first place. Model 2 is an idealized model but practically overestimates the joint strength because such a stress state will come after an extensively large deformation. Finally, theevaluation technic by the use of Model 3, the results of which agree very well with the FEM analysis as shown in Fig. 6, can be appled in practice.



〈Fig. 4〉 The values of the two reference lines for the sleeve and the connection bolt



〈Fig. 5〉 An example of comparison between the stress distributions



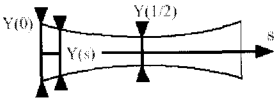
〈Fig. 6〉 An example of the N/Ny-M/Mp correlation curve of joint strength

3.2 Equivalent Static Seismic Force for the Earthquake-Resistance Design of Arches and Arch-like Structures

The earthquake-resistant designs of almost all spatial structures in Japan such as spherical domes, cylindrical roofs, arches and arch-like structures have been conducted in accordance with the Building Standards Act, its enforcement ordinance and the notices of the regulations in Japan, which means that the applied equivalent seismic forces are the story shear forces which are provided for usual buildings composed of columns, girders, floors and walls or bracings. But the actual seismic response properties of these spatial structures are essentially different form those of the usual buildings, that is, in the case of these spatial structures, not only the horizontal component but the vertical component of deflection can significantly be excited by the horizontal seismic motion, and further, the vertical seismic response can easily be amplified by the vertical seismic motion compared with the usual buildings.

In view of the above mentioned circumstances, the present author and his co-workers, Chanwoo Jung and Yohsuke Shinohara, proposed the equivalent seismic forces for circular arches in a form as simple as possible based on the numerous seismic response analyses[2][3][4]. This evaluation method is based on the seismic coefficient method, and the equivalent seismic forces are given by multiplying the distributions of seismic coefficient by the long term weight distribution, and the input acceleration intensity. The seismic coefficients are prepared individually for the horizontal and vertical response components against the horizontal and vertical seismic motions.

This study was intended for the clamped uniform arches at first, and then for such clamped non-uniform arches as they have smaller sections around the apexes[2][3]. Fig. 7 shows the applied section's varying rule. A further study was made for pinned uniform arches[4].



$$Y(s)=\frac{3Y_0}{1+2\alpha}\{4(1-\alpha)(s-1/2)^2+\alpha\}$$

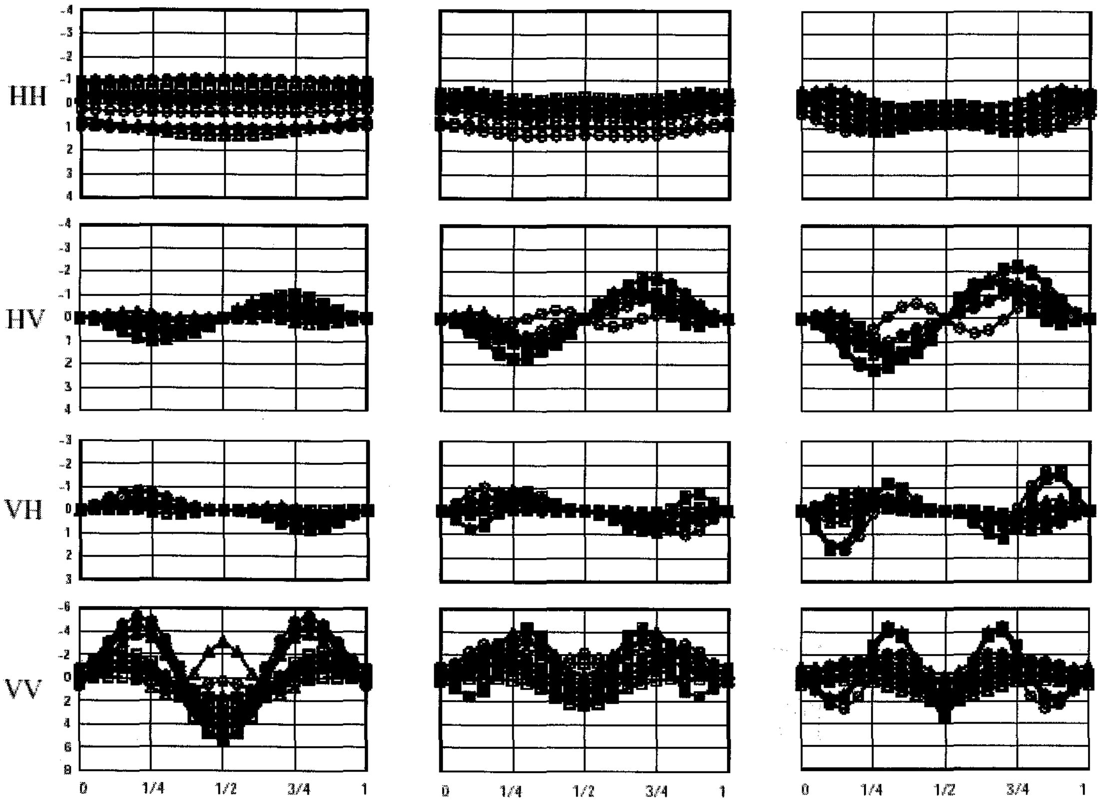
$$\alpha=\frac{Y(1/2)}{Y(0)}$$

⟨Fig. 7⟩ The varying rule of variables

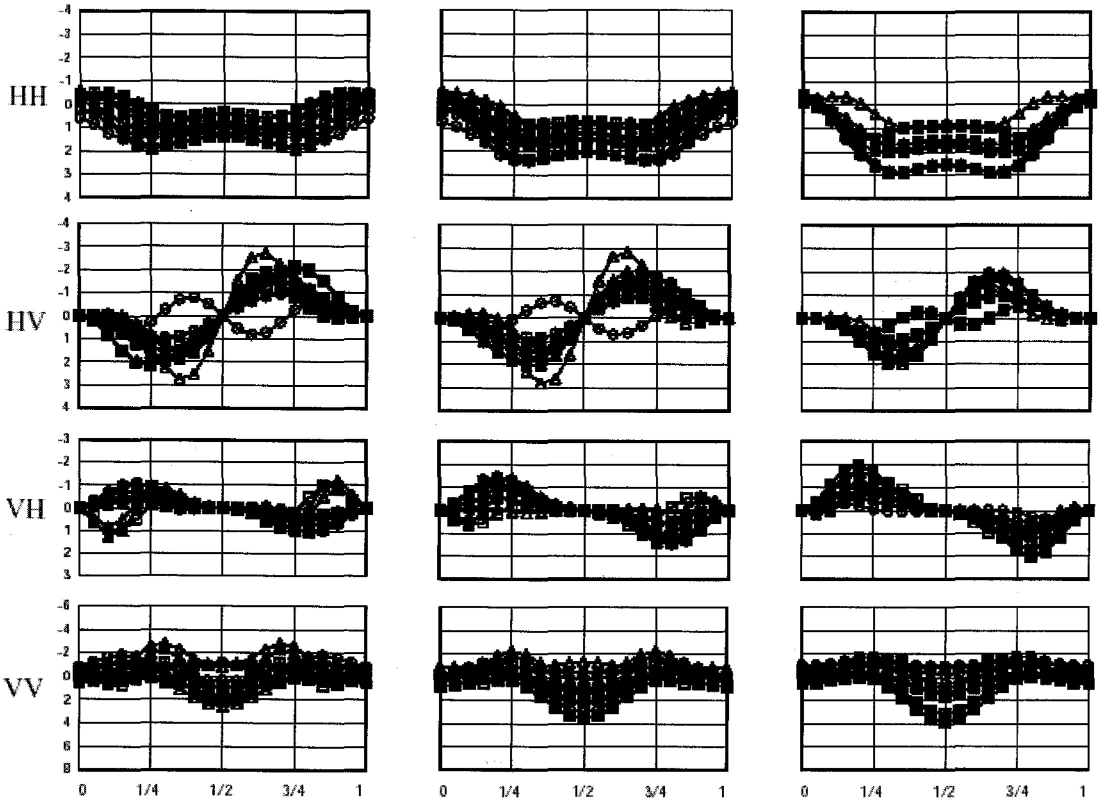
Figs. 8, 9 and 10 orderly show examples of above three cases. The model's first natural frequency is limited only to 2.0 Hz here. The non-uniform arch applied here is characterized by the change in the second moment of inertia along the neutral axis. The horizontal axis is the location along the arch normalized by the open angle. The vertical axis is the instantaneous acceleration coefficient obtained for several recorded seismic motions under the estimation conditions with respect to the maximum response values in the acceleration response, the fiber stress response, the reaction force and the deflection. HH, HV, VH and VV means the horizontal or vertical seismic motion in the their first letter and the horizontal or vertical response in their last letter, that is, VH means the vertical acceleration response coefficient excited by the horizontal seismic motion for example.

It has been clarified that the effect of the first natural frequency on the magnitude of acceleration response coefficient is small if it is smaller than 3 Hz, and that the effect of the open angle on the mode in distribution of acceleration response coefficient is also small if it is greater than or equal to 90 degrees.

Fig. 11 shows the proposed equivalent seismic coefficient distributions; A_{HH}, A_{VH}, A_{HV}, A_{VV1}, and A_{VV2}, and the equivalent seismic force distributions; P_{HH}, P_{VH}, P_{HV}, P_{VV1} and P_{VV2}, for the open angle greater than or equal to 90 degrees.

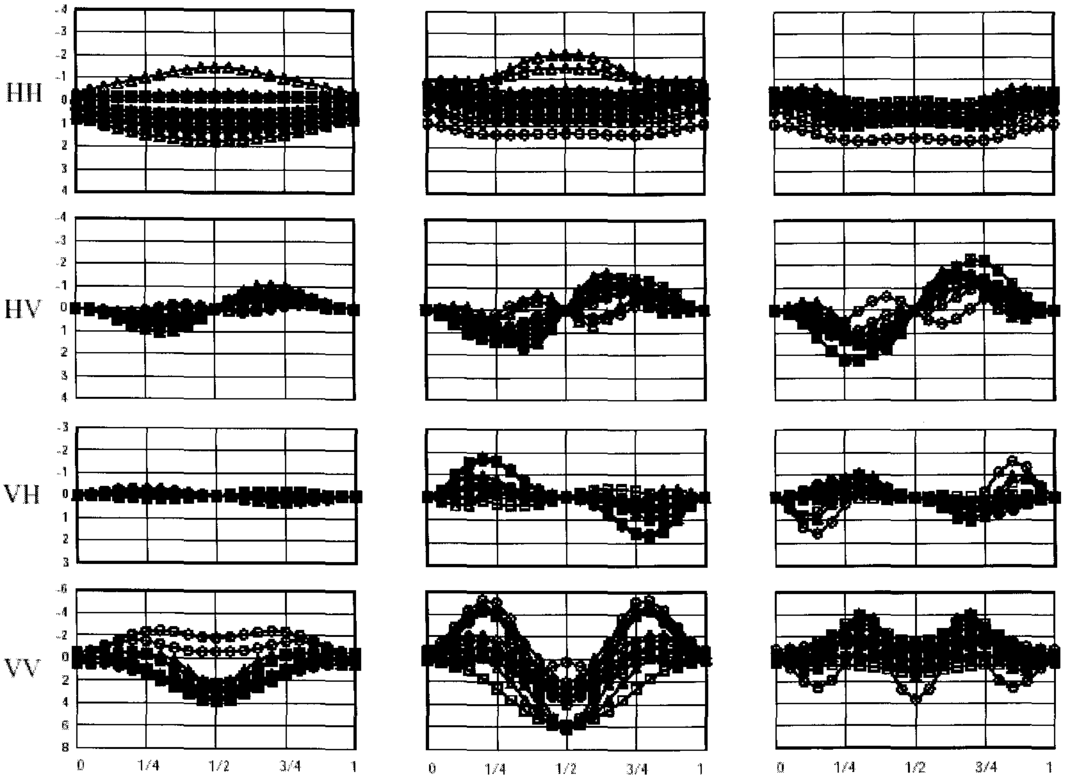


open angle 30°　　open angle 60°　　open angle 90°

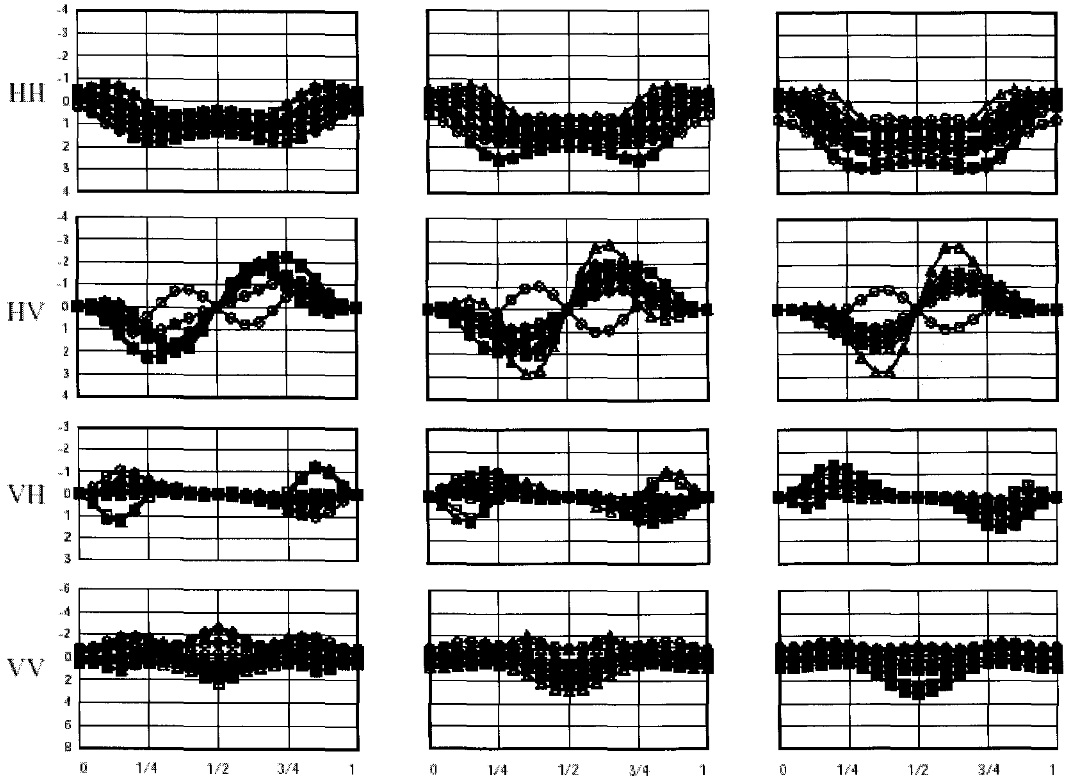


open angle 120°　　open angle 150°　　open angle 180°

Fig. 8 The instantaneous acceleration response distributions of the clamped uniform arch (f_1=2.0Hz)

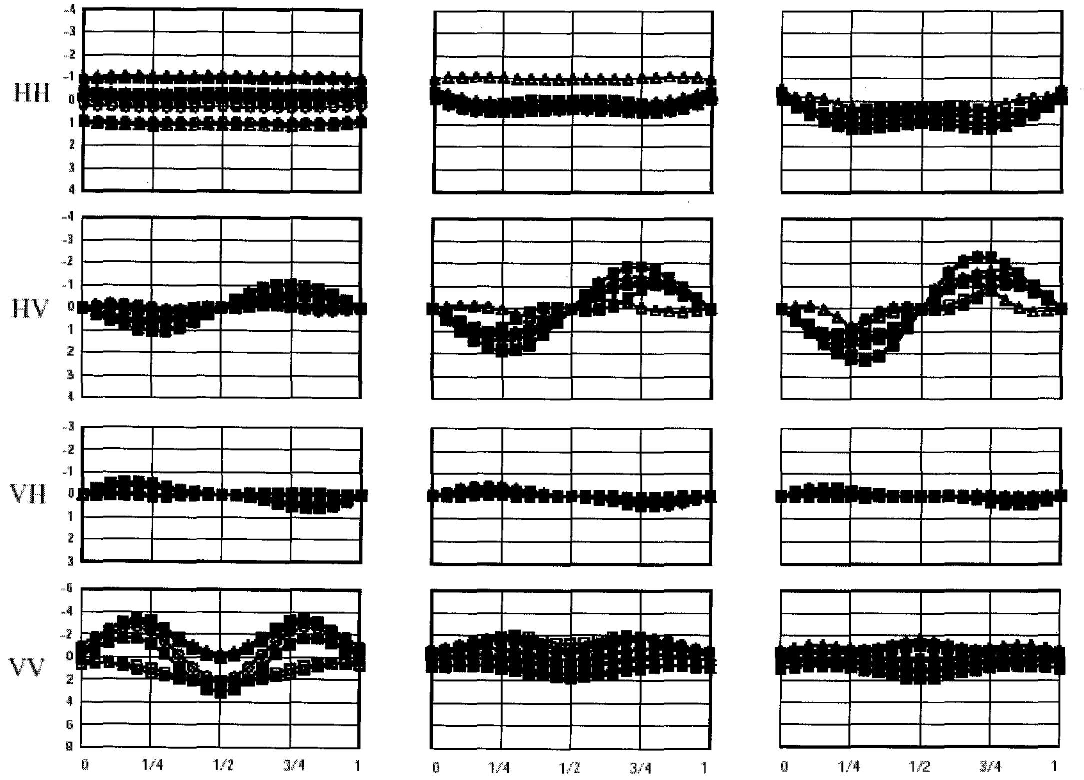


open angle 30°　　open angle 60°　　open angle 90°

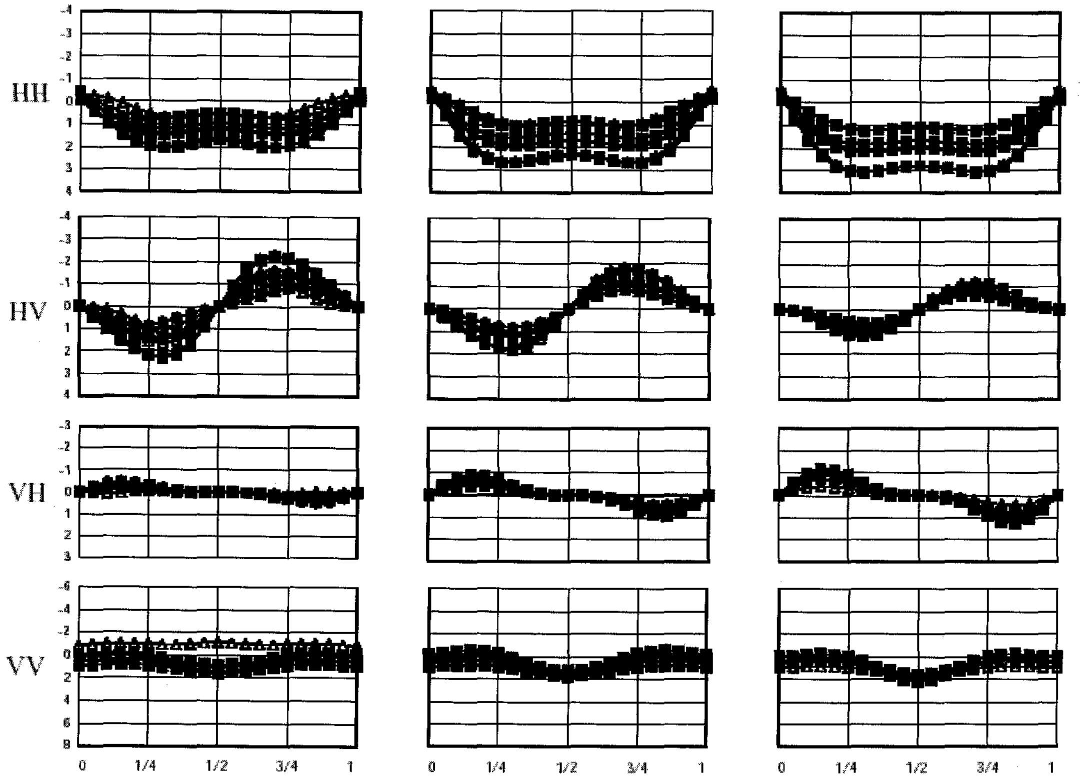


open angle 120°　　open angle 150°　　open angle 180°

Fig. 9 The instantaneous acceleration response distributions of the clamped non-uniform arch (f_1=2.0Hz, α-1/2)



open angle 30°　open angle 60°　open angle 90°



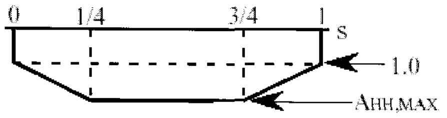
open angle 120°　open angle 150°　open angle 180°

Fig. 10 The instantaneous acceleration response distributions of the pinned uniform arch (f_1=2.0Hz)

The suffixes HH etc. of these symbols are the same as defined above. The two components of P_{VV1} and P_{VV2} do not need to be considered at the same time. In this figure, s is the location along the arch normalized by the open angle, ξ is the open angle normalized by 180 degrees, C_H and C_V are the horizontal and vertical input acceleration intensity and w is the weight per unit length, respectively. The coefficient α is the adjustment factor for the pinned arch.

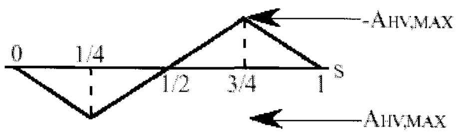
The non-uniform arch, if it has the same natural frequency and the same open angle as a uniform arch, has nearly the same seismic coefficient distribution as the uniform arch. This fact generally means that the proposed seismic coefficient distribution are available to clamped uniform and non-uniform arches.

The effect of the boundary condition on the magnitude of acceleration response coefficient can also be said small. The above seismic coefficient distribution for the clamped arch can also be applied to the pinned uniform arches, though the magnitudes of response slightly decrease. If necessary, as shown in Fig. 11, the seismic coefficient distributions for the pinned uniform arch are easily provided by multiplying the adjustment factor a in Fig. 11 by the seismic coefficient distributions for the clamped uniform arch.



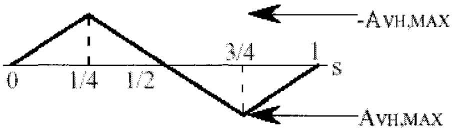
$A_{HH,MAX,clamped} = 1 + 2\xi$
$A_{HH,MAX,pinned} = \alpha A_{HH,MAX,clamped}$
$\alpha = 1.1$
$P_{HH} = wC_H A_{HH}$

(a) A_{HH} and P_{HH} for HH



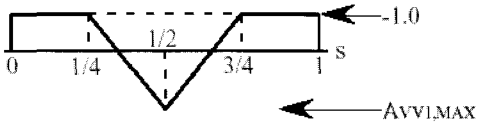
$A_{HV,MAX,clamped} = 2.5$
$A_{HV,MAX,pinned} = \alpha A_{HV,MAX,clamped}$
$\alpha = 1.1$
$P_{HV} = wC_H A_{HV}$

(b) A_{HV} and P_{HV} for HV



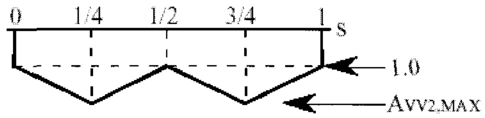
$A_{VH,MAX,clamped} = 2.0$
$A_{VH,MAX,pinned} = \alpha A_{VH,MAX,clamped}$
$\alpha = 0.9$ (for combination with VV1)
$\alpha = 0.75$ (for combination with VV2)
$P_{VH} = wC_V A_{VH}$

(c) A_{VH} and P_{VH} for VH



$A_{VV1,MAX,clamped} = 4\xi - 0.5$
$A_{VV1,MAX,pinned} = \alpha A_{VV1,MAX,clamped}$
$\alpha = 0.9$
$P_{VV1} = wC_V A_{VV1}$

(d) A_{VV1} and P_{VV1} for VV1



$A_{VV2,MAX,clamped} = 12(1-\xi)^2 + 1$
$A_{VV2,MAX,pinned} = \alpha A_{VV2,MAX,clamped}$
$\alpha = 0.75$
$P_{VV2} = wC_V A_{VV2}$

(e) A_{VV2} and P_{VV2} for VV2

〈Fig. 11〉 The seismic coefficient distribution functions

3.3 Evaluation Method for Design Wind Load on Spatial Structures

The author and his co-worker, Yasushi Uematsu, have been studied on the wind induced response characteristics and on the wind load evaluation and further on the fatigue-resistant design of roofing materials based on both the dynamic response analyses and the wind tunnel experiments[7][8].

On the large-span flat roofs with square plans or circualr plans, the author and his co-worker showed that the time-averaged wind pressure multiplied by a gust loading factor for the natural vibration mode corresponds to the design wind load, or in other words, the effective wind load. Empirical formulas for the gust loading factor are provided as the functions of both the geometric and the structural parameters of the roof and the turbulence intensity of the approach flow. The details of this result have been shown in the paper presented at the international conference held at Seoul, Korea in 2000[7].

The researchers of this study are further investigating the design wind load widely applicable to single-layer latticed domes[8].

4. Acknowledgments

The author expresses his gratitude to Professor Kwun, Taek-Jin, President of Korean Association for Spatial Structures, for inviting the author as a special speaker to the KASS 2002 Spring Academic Lecture Meeting. The author wishes the great success of KASS from the bottom of his heart.

References

1. Members of committees for IASS International Symposium, Nagoya, 2001 : The present and the future of the spatial structures of the world, THE STRUCTURAL TECHNOLOGY, Koukouzou Shuppan, vol.15, No.167, pp.23-33, 2002.4 (in Japanese)
2. M. Yamada : Buckling of Arches, Stability problems of steel structures 2001, Japan Institute of Architecture, Subcommittee of Buckling of Steel Structures, pp.127-139, 2001 (in Japanese)
3. C. W. Jung, Y. Shinohara, M. Yamada : A study on the static seismic force for the design of the arch, IASS International Symposium 2001, Nagoya, Japan, TP080, 2001
4. Y. Shinohara, C. W. Jung, M. Yamada : Annual Meeting of Japan Institute of Architecture 2002, Kanazawa, Japan, to be presented, 2002.8 (in Japanese)
5. M. Yamada, A. Obata, et al. : Three dimensional stress and deformation analysis of joint parts used in truss system, Proceedings of 6th Asian Pacific Conference on Shell and Spatial Structures, Seoul, Korea, pp.331-338, 2000.10
6. A. Obata, M. Yamada : Annual Meeting of Japan Institute of Architecture 2002, Kanazawa, Japan, to be presented, 2002.8 (in Japanese)
7. Y. Uematsu, M. Yamada : Design wind loads for structural frames of flat long-span roofs, Proceedings of 6th Asian Pacific Conference on Shell and Spatial Structures, Seoul, Korea, pp. 303-310, 2000.10
8. Y. Uematsu, M. Yamada, et al. : Wind-induced dynamic response of a single-layer latticed dome, IASS International Symposium 2001, Nagoya, Japan, TP084, 2001.