

2002 World Cup Soccer Stadium (Jeonju)

Analysis and Design of a long span cable structure

In this technical note, the structural analysis and design of the roof of the Jeonju World Cup Soccer Stadium will be introduced. It summarizes the design concept for the stadium developed by C.S Structural Engineers in association with MIDAS IT. Its main contents are as follows:

1. Basic structural planning and outline of the structural system of the roof structure
2. Analysis of the cable stayed structure and the procedure of analysis reflecting the erection sequence
- 3 Detail connection analyses for checking the strength of the pipe truss joints



<Fig. 1> Perspective of 2002 world cup Jeonju stadium

The architecture symbolizes the tradition of the city, which structurally transforms into 4 roof segments supported by the stand frames and cables emanating from the 4 main masts at 4 corners.

■ Structural Plan

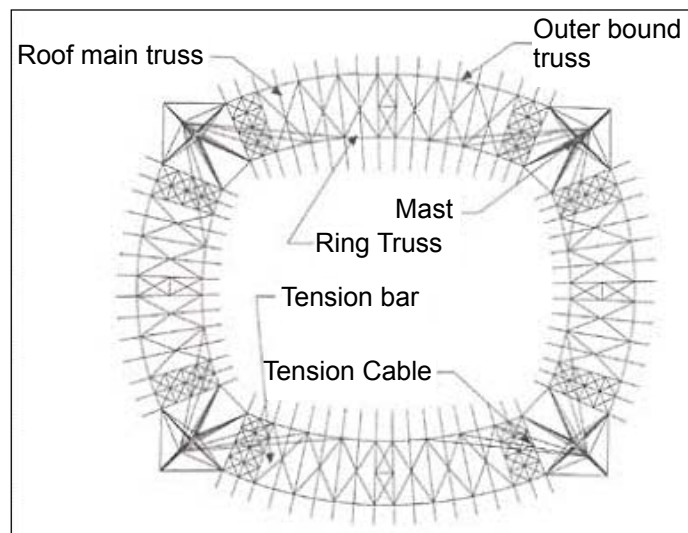
The roof of the stadium was planned as a truss structure with four separate fan-shaped roofs. Each roof structure is supported by a suspension system along the inner (cantilever) edge, and the direct supporting system is situated at the outer edge of the roof. The suspension system, which is a cable stayed system, consists of the cables, which emanate from the 4 masts located at the 4 corners and support the inner

edges of the roofs.

A Ball type Hinge system was adopted for the bases of the masts to minimize moments transferred to the foundation and for the ease of construction. The inner edge of each roof retains a ring truss in the form of an arch with a gentle gradient, which induces the behavior of a dome structure. The arch action of the ring truss in turn reduces the gravity forces that the cables need to resist.

The Direct Supporting System includes a series of diagonally shaped supporting structures, which connect the Outer Bound Trusses and Stand. The forces transferred to the Stand are then directly transmitted to the foundation.

In order to provide an appropriate level of diaphragm stiffness to the roof planes for the purpose of maintaining stability against lateral forces, prestressed tension rods are installed between the main roof trusses. The supporting structures under the outer bound trusses resist the design lateral forces. This load path simplifies the system of lateral load transfer to the foundation, and it in turn optimizes the member sizes by imposing only the gravity loads to the suspension system, which consists of cables and masts.



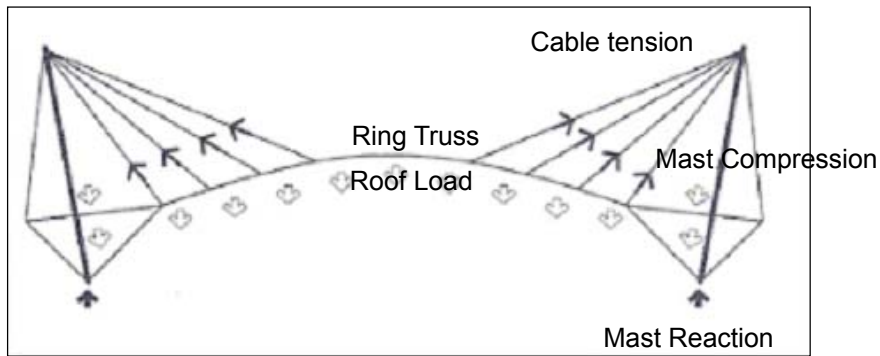
<Fig. 2> Structural system of the roof structure

■ Overview of the structural system

➔ Gravity System

- Suspension

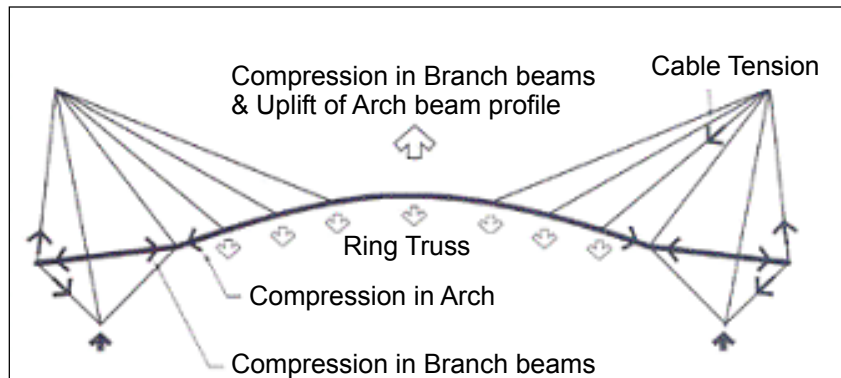
40% of the gravity loads carried by the roof is transferred to the foundation through the cables and masts.



<Fig. 3> Makeup of suspension system

- **Arch action**

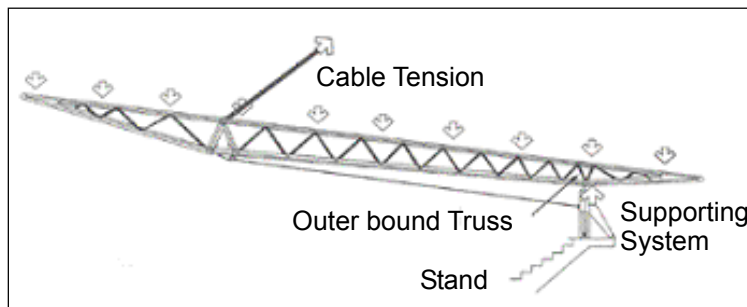
By the suspension action, the cable tension induces the masts to rotate about the base points, and this process leads to compression forces in the ring trusses and branch beams. The ring trusses tend to deflect upward due to the arch action. The upward deflection in turn controls the vertical deflection of the roofs.



<Fig. 4> Structural concept of dome system

- **Direct support**

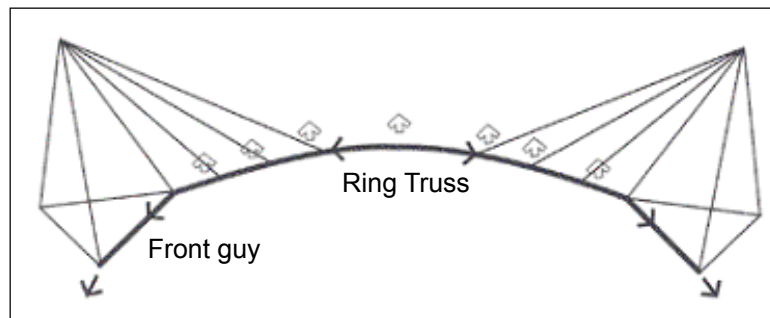
60% of the gravity loads is transferred to the stand through the supporting system beneath the outer bound truss.



<Fig. 5> Structural concept of the direct supporting system

- **Uplift System**

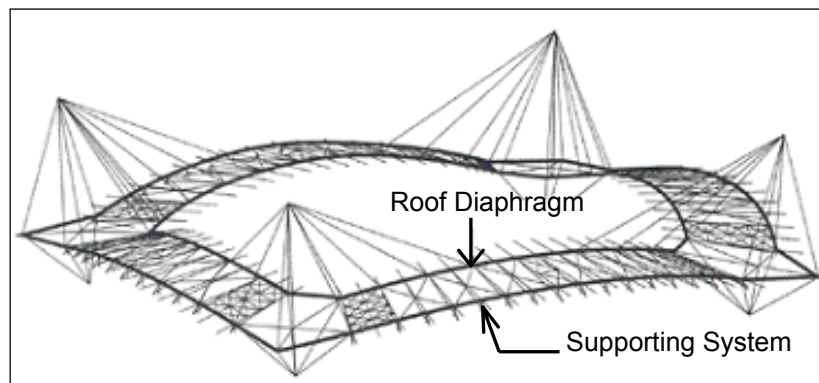
About 55% of the upward wind load is transferred to the tension system connected to the front guy and ring truss. The remaining 45% of the upward wind load is resisted by the direct supporting structure beneath the outer bound truss.



<Fig. 6> Resisting System for Uplift Wind Load

- ➔ **Lateral System**

Lateral wind and earthquake loads are transferred to the supporting structures beneath the outer bound trusses through a roof diaphragm system provided by the roof trusses, tension bars and branch beams.



<Fig. 7> Structural concept of lateral system

- **Structural Model**

The entire structure was included in the analysis model to capture the correct interaction between the roof and the base stand structure. The stand structure was modeled using plane stress and plate elements, and the masts were modeled as equivalent non-prismatic (tapered) beam elements to truly reflect the varying

cross sections along the lengths.

To increase the efficiency of dynamic analysis, a simplified equivalent model was used to prevent high frequency and local modes by substituting all truss members with beam elements of equivalent stiffness.

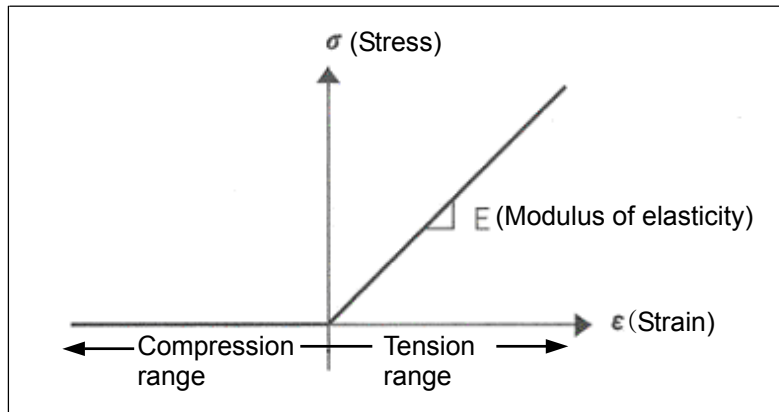
■ Cable Model

The analytical approach for cable structures differs from case to case depending on the conditions of cable connections. In the cases of suspension cable bridges or cable net structures, where only cables are connected at nodes, the behavior of large displacement may exist. Geometric nonlinear analysis is warranted in such cases, as the loading directions also change with large displacements.

Cable stayed structures in building applications are generally used to stabilize the structures consisted of general structural members, and as such cables are connected to the structural members rather than other cables.

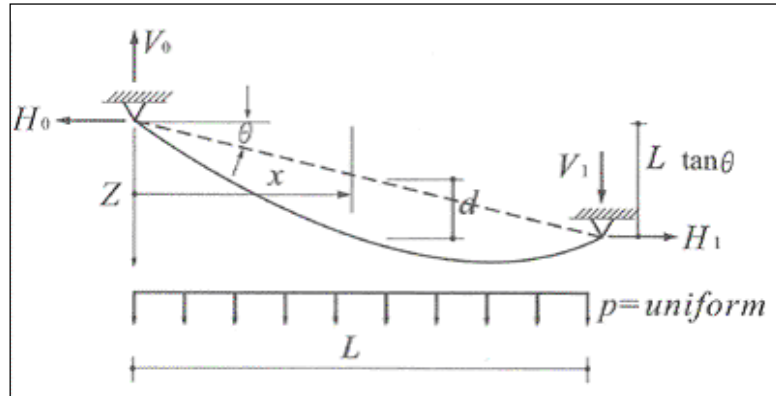
When cables are directly connected to the structural members, the design of the structural members is premised on the small strain condition in order for the members to be designed within the permissible strength. The corresponding connection nodes also meet the small strain condition, and the large displacement analysis is thus not required. Nevertheless, even if the structure stays within the small strain condition, the analysis must take into account the characteristics of cables, namely, sag and inability to resist compression. Although geometric nonlinear analysis is required to account for the change of stiffness due to the sagging of cables by self-weight, linear analysis may be desired in practice for simplicity based on the following reasons:

Since cables do not resist compression and their stiffness can be realized only under tension as indicated in <Fig. 8>, the cables can be modeled as geometrically nonlinear elements such as tension-only truss elements. Compression in a cable of a cable stayed structure may jeopardize the stability of the entire structure. Accordingly, cables are appropriately pretensioned so that they are subjected to tension under all possible load cases and combinations that can be expected during the life of the structure. The fact that tension is maintained in the cables at all times, the cables contract and expand under the tension condition. This enables us to simulate the structural behavior through only linear analysis.



<Fig. 8> Stress-strain diagram of a cable neglecting sag

In addition, appropriate cable sizes and pre-tensions are searched through trial and error in order to safely design the strength and stiffness for the erection and in-place conditions. Caution should be exercised at this point, as the effective stiffness of the cables change with sagging due to the self-weights, and the sagging is determined by the cable tensions. The magnitude of pre-tension should be controlled so that 90% of the regular stiffness can be maintained, which is a level that maintains the cable tension within the allowable strength while maintaining the required stability.



<Fig. 9> Tension and sag in a cable subjected to a uniformly distributed load

The equation below expresses the correlation of the self-weight of the cable, the magnitude of sag, cable tension and the effective stiffness of the cable.

$$T = \frac{PL}{8f} \sqrt{1 + \left[\tan \theta + 4f \frac{(1-2\pi)}{L} \right]^2} \quad \text{-----(1)}$$

where, P : Self weight of cable or a uniformly distributed load exerting on the cable

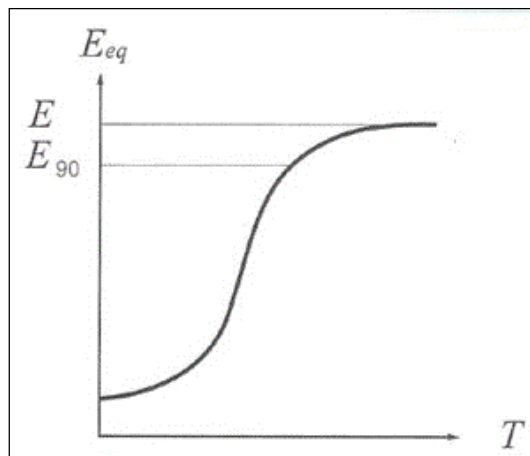
| | |
|------------|---|
| L : | Projected length of cable |
| f : | Sag Ratio (the ratio of the sag at the midpoint to the projected length of cable) |
| θ : | Angle of inclination |
| π : | The distance in the direction of the horizontal projected length between the end of the cable and any point along the cable |

in the above equation, if we assume $\theta=0$, and sag is small,

$$T = \frac{PL}{8f} \text{ -----(2)}$$

And the cable tension required to maintain 90% of the regular stiffness is obtained from the following equation:

| | |
|--------|---|
| | $E_{eq} = \frac{E}{[1 + \frac{(wL)^2 AE}{12T^3}]} \text{ -----(3)}$ |
| where, | E_{eq} : Effective modulus of elasticity of cable |
| | E : Regular modulus of elasticity of cable |
| | w : Self weight of cable per unit length |
| | L : Projected length of cable |
| | A : Effective area of cable |
| | T : Tension force in cable |



<Fig. 10> Effective modulus of elasticity with change in cable tension

We can see from the above equations (2) and (3) that there exist direct relationships between the effective stiffness of cable and tension and between the self weight and sag.

In order to induce the final tension cable forces for the in-place condition extracted from the above

process, we gradually increase the pretension forces in all the cables by similar ratios for each erection step. Tensioning and adjusting operations are repeated through a central monitoring system. The pretensioning operations stop when the finally required tensions are obtained in all the cables.

Accordingly, nonlinear analysis has no meaning at the preliminary design stage because the pretensions are adjusted for erection stages until we reach the final tensions required to attain an appropriate level of stiffness and stability for the in-place condition. However, nonlinear analysis can be effectively used to calculate the reference tensions required at every tensioning stage to reduce the number of trial-and-errors during erection.

Since the supporting system underneath the outer bound trusses entirely resists the lateral forces, the cable tensions do not vary more than 5% from the maximum and minimum even under the extreme condition. Nonlinear analysis is thus not warranted.

■ Overview of erection

The sequence of construction for the roof structure is as follows:

1. Roof truss erection stage
2. Cable installation stage
3. Cable pretensioning stage

The roof trusses are erected on a falsework system, which incorporates a jacking system that can lower the trusses incrementally to suit the overall sequence.

No camber was adopted for the main roof trusses as we found the vertical displacement was relatively small under the dead load condition including pretensioning.

The mast cables are installed after erecting the roof trusses and the mast/branch beam assembly. Lower parts of the mast cables are attached with tensioning devices and turn buckles.

■ Erection sequence of roof structure

| | |
|--------|--|
| step 1 | Temporary shores are installed under the ring trusses, and the supporting system is installed over the top of the stand to support the outer ends of the main roof trusses at the same time. |
|--------|--|

| | |
|--------|--|
| | Jack-down devices are installed on the top of the shoring system. All subsequent assembling of the roof structure takes place relative to the tops of the jacking devices. |
| step 2 | The main trusses are pre-assembled on site and installed between the temporary shores and the supporting system of the outer bound. The assembly of the roof trusses is completed through the installation of the ring trusses and outer bound trusses (including the assembly of main/branch beams). |
| step 3 | Install the mast cables (excluding the pretensioning operations). All cables should be fully plant-stretched two or three times prior to cable tensioning. The length for each should be adjusted and cut to the distance between the pin holes considering the lengths of the end sockets and prestressing device and the lengthening of the cable under the final tension load for the in-place condition. |
| step 4 | Commence the equipment installation work (speakers, lighting devices, machines, electric facilities, etc) including the finishing materials on the roof. |
| step 5 | Pretension the mast cables (using hydraulic jacks of 30 to 100ton capacity). The pretension procedure is divided into several stages with a regular interval between each stage of stressing. The examination of excess and deficiency of inducted pretension is undertaken at each individual stage, and the displacements of the masts are measured and compared to the analysis values simultaneously. |
| step 6 | Install the diagonal steel rods for roof diaphragms and pretension them. |
| step 7 | Jack down the jacking devices installed on the tops of the temporary shores gradually. Jacking down operation is executed through the integrated control of the central control system. Jacking down operation is terminated when the roof loading acting on the jacks are completely transferred to the final structure. |
| step 8 | Install and pretension the front guy cables. |
| step 9 | Disassemble the jacking devices and temporary supports, and complete the erection of the roof structure. |

■ The procedure of structural analysis considering the erection steps

| | |
|--------|--|
| step 1 | Estimate the necessary pretension to reach the 90% (E90) of regular modulus of elasticity in the mast cables. |
| | 1.1 Generate a 3D model. |
| | 1.2 Estimate the tension to reach the effective elastic modulus (E90) of the cables. |
| | 1.3 Assign the support condition representing the equivalent stiffness at the bottom of the front cable reflecting the temporary support condition. |
| | 1.4 Estimate the initial pretensions that can manifest stiffness in excess of the effective |

elastic modulus (E_{90}) for all the cables.

step. 2 Execute the analysis with the pretensions intended to maintain the effective modulus of elasticity of the mast cables.

2.1

Prepare an analysis model without considering the temporary support condition.

2.2

Execute the analysis after inputting the dead load of the structure and the initial pretensions calculated from step 1.

2.3

Confirm whether or not the effective elastic modulus is maintained through examining the tensions generated in the cables.

2.4

Examine the necessity of introducing cambers after reviewing the displacements under the condition of the dead load and the initial pretensions.

step 3 Estimate the pretensions of the mast cables such that the cables remain in tension under all the load combinations.

3.1

Generate all possible load combinations.

3.2

Review the results of the analysis subjected to all the load combinations with the initial pretensions obtained from step 1.4 whether any of the cables is in compression.

3.3

If compression occurs, estimate the second stage pretension such that the corresponding cables will remain in tension under all the load combinations.

step 4 Review the effect of cable failure.

4.1

Ensure the redundancy of major cables through cable failure analyses (non-single cables for major cables).

4.2

Investigate the effect on the adjacent cables and prepare alternate load paths in the event of any major cable failure.

4.3

Establish the third stage pretension to maintain tension in the cables prone to develop compression under the failure conditions.

step 5 Review for in-place condition

5.1

Estimate the pretensions to be applied to the roof diagonal steel rods and maintain the tension state under all the load combinations.

5.2

Estimate the magnitude of the 4th pretension to be applied to the front guy cables, which will be necessary to maintain an effective elastic modulus (E_{90}).

5.3

Perform the analysis after inputting all the pretensions calculated to this point and other load conditions.

5.4

Examine the levels of safety on the roof structural components under all the load combinations.

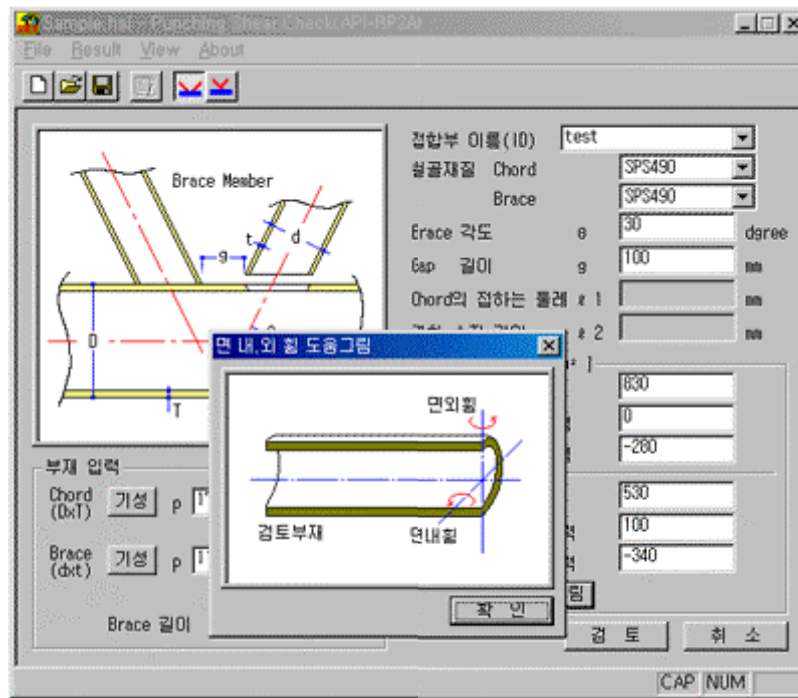
■ Design of roof truss joints

Joint design is especially important in the case of a truss system consisting of circular steel pipes. In fact, it is more common to have failures in a pipe truss by local deformation of the joints rather than by local buckling or excessive tensile stress in the members.

There exist many different ways of connecting hollow sections in a truss system – direct welding, ring plate reinforcement, penetration of braces into chords, etc. Whatever the method we choose, it must be safe, esthetic, economical and easy to fabricate. All in all, the direct welding method generally satisfies all the criteria.

The design for the pipe joints of the Jeonju stadium was accomplished by satisfying the guidelines suggested by API (American Petroleum Institute) and IIW (International Institute of Welding). The design guidelines of API and IIW for pipe joints use the equations based on experimental test results for local deformation and are known to be highly reliable. Also, local joint finite element analyses were performed by using Gen with the aid of the FEmodeler mesh generator for critical joints.

There are several failure modes for hollow section connections – propagation of flexural deformation locally developed on the wall of the member, simultaneous formation of local deformation and buckling at the joint between the braces in a K joint, or braces punching through chords. <Figure 11> and <Figure 12> are excerpts of the pipe joint strength check module in the MIDAS/Set-Building program, which will be released in mid 2003.



<Figure 11> Main menu of the pipe joint strength check module in MIDAS/Set-Building

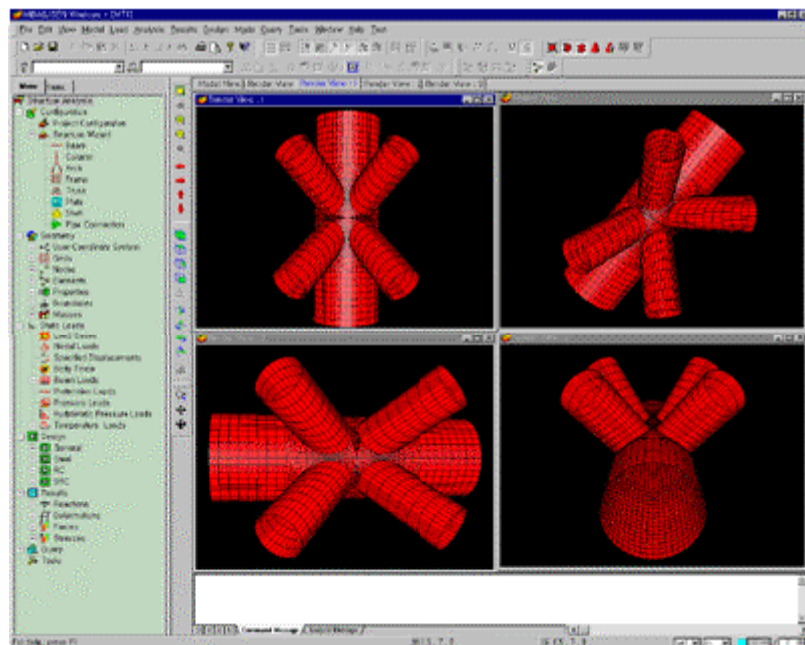
PUNCHING SHEAR CHECKING RESULT - API-RP2A

| | | | | | | |
|----------------|--|----------------------------------|-----------|---|--|--------------------|
| V_p | $f + \tau + \sin \theta$ | 속용력 2.54 면내휨 4.24 면외휨 6.78 | V_{pa} | $Q_a + Q_f + F_y / 0.6y$ | 속용력 216.37 면내휨 524.01 면외휨 387.25 | |
| Q_b | $\rho > 0.6$ $0.3/\rho$ $(1-0.833\rho)$ $\rho \leq 0.6$ 1.0 | 1.16 | Q_g | $v \leq 20$ $1.8-0.1g/T$ $v > 20$ $1.8-4g/D$ | 1.00 | |
| A | $\sqrt{(f_{ax}^2 + f_{pb}^2 + f_{opb}^2) / 0.6F_y}$ (chord stress) | 0.65 | λ | Brace Ax 0.030 | Brace IPB 0.045 | Brace OPB 0.021 |
| Q_c | * 사용자 지참서 참조 | 속용력 1.56 면내휨 4.53 면외휨 2.53 | Q_f | $1 - \lambda + y * A$ | 속용력 0.75 면내휨 0.62 면외휨 0.82 | |
| 평가값 ≤ 1.0 | | | 평가 | | | |
| 1차 평가 | $\left(\frac{V_{pIPB}}{V_{pIPB}}\right)^2 + \left(\frac{V_{pOPB}}{V_{pOPB}}\right)^2$ | 0.000 | 0.0 | 중복 | | |
| 2차 평가 | $\frac{V_{pAX}}{V_{pMAX}} + \frac{2}{\pi} \cdot \arcsin \left\{ \sqrt{\left(\frac{V_{pIPB}}{V_{pIPB}}\right)^2 + \left(\frac{V_{pOPB}}{V_{pOPB}}\right)^2} \right\}$ | 0.024 | 0.0 | 취소 인쇄 | | |

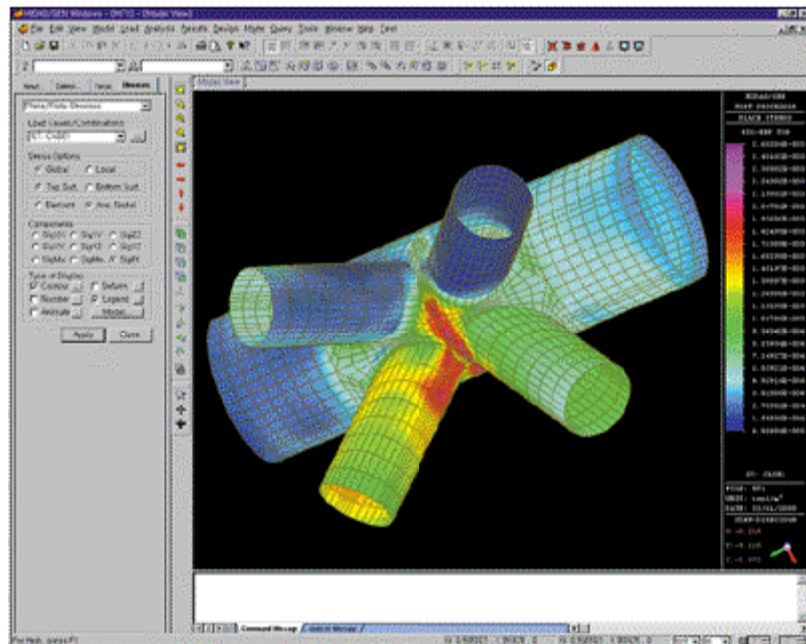
<Figure 12> Output of the pipe joint strength check module in MIDAS/Set-Building

Many pipe joints do not satisfy the required conditions to apply the equations classified in the above guidelines. For practical purposes, the guidelines are used to check the joints that meet or resemble the criteria. Those joints that exceed or are close to the allowable design criteria and the joints where high local stress concentration is anticipated were reanalyzed and designed by Gen aided by FEmodeler.

The captured screens below show von-Mises stress contour on the mesh model generated by FEmodeler using the Feature Based Modeling technique and the Auto-mesh Generation function. The results are obtained by applying the deformations of the total wire frame pertaining to the particular joint using the function of forced displacement analysis in MIDAS/Gen. The maximum distortion energy theory is applied to the verification of strength. The design is carried out so that the resulting von-Mises stresses do not exceed the yield stress after applying a safety factor.



<Fig. 13> Fine Mesh Model of a pipe joint generated automatically by the MIDAS/FEmodeler program



<Fig. 14> von-Mises Stress in the pipe joint

$$\sigma_{eff} = \sqrt{\sigma_1^2 - \sigma_1\sigma_2 + \sigma_2^2}$$

$$\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

$$\sigma_2 = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

σ_{eff} : Effective (von-Mises) Stress

σ_x : Axial Stress

σ_y : Biaxial Stress

τ_{xy} : Shearing Stress

σ_1 : Maximum Principal Stress

σ_2 : Maximum Principal Stress