THE SEVERE ACCIDENT ANALYSIS OF THE PRESTRESSED CONCRETE CONTAINMENT FOR TIANWAN NUCLEAR POWER PLANT

Pentti Varpasuo

FORTUM ENGINEERING, Vantaa, Finland

ABSTRACT

The design pressure for the containment building was 5 bars of absolute pressure. As the safeguard against higher pressures because of direct containment heating in severe accidents the ultimate load carrying capacity of containment had to be evaluated.

The analysis consisted of the determination of the average cracking pattern in the containment shell for 7 bars absolute pressure load. The approach for the analysis was to use the same global 3D model for the containment shell as the one used in the design. The extension of the analysis was that all material non-linearities were taken into account excluding direct analysis of concrete cracking. The concrete shell elements were modeled using orthotropic elasto-plastic material properties having different stiffness in compression and tension. In compression the element stiffness was the stiffness of concrete evaluated from Young's modulus, Poisson ratio and shell thickness. In tension the element responded with stiffness that consisted of the stiffness of reinforcement in hoop and vertical directions. The tensile strength of concrete was neglected. The tendons were depicted with elasto-plastic material properties that were evaluated from the measured stress-strain properties of tendons.

The results of the ultimate strength analysis were the strain plots for the containment shell for various pressure values. On the basis of calculated strains the average crack widths and crack distances were calculated and the overall containment leak amounts were evaluated.

INTRODUCTION

Because of the far-reaching consequences of core-melt accidents, these accident scenarios are an integrated part safety analysis for reactors in their design stage ever since the Three Mile Island Unit 2 (TMI-2) accident. The first priority is to prevent the core melt accident from happening. However, should the accident progress into a severe condition, then the next priority shall be to mitigate further development of the accident by utilizing all means of accident management available at the site. The purpose of the reactor containment building is to protect the environment against consequences of the loss-of-coolant accident. Further, containment is also used to prevent the leakages and minimize the vents in case of severe accident condition. The task in this assignment was to assess the behavior of a dry prestressed concrete containment structure for a pressurized water reactor for certain beyond design basis conditions. The structure under consideration is a double containment. The inner containment is a prestressed concrete structure consisting of a flat base mat, a cylindrical shell, and a hemispherical dome. The outer containment is a reinforced concrete structure consisting of a cylindrical shell with a flat top dome.

If the containment integrity is maintained during a severe accident, the radiological consequences will be negligible. If the containment function does fail, the timing of containment failure is very important. The longer the containment remains intact, relative to the time of core melting and fission product release from the reactor coolant system, the more time is available for the removal of radioactive material from the containment atmosphere by engineered safety features or natural deposition processes.

Early failure of containment is defined as the containment failure at or just after reactor vessel fails, and late failure from two hours after the vessel failure to three days after accident initiation.

The task in this assignment is to investigate the state of Tianwan plant containment in 7 bar absolute internal pressure condition and to establish the deformations and average cracking at containment shell.

CONTAINMENT SHELL CONFIGURATION

The containment under consideration is a pre-stressed concrete shell structure that consists of a cylindrical part and a hemispherical dome. The inner surface of the containment is covered with a 6 mm thick carbon steel plate to secure the tightness. The inside diameter of the cylinder is 44.0 m. The height of the cylinder part is 41.6 m and the top of the dome is at level +71.60. The thickness of the cylinder and dome are 1.2 m and 1.0 m respectively. The prestressing of the containment will be performed by means of the post-tensioning system. The tendons are divided into two horizontal and two vertical sets. The horizontal tendons in the cylinder and dome will be going around the whole 360 degrees so, that the anchorage is in turn on the opposite sides of the containment. Two buttresses at 120 and 300 degrees are used for the anchorage of the horizontal tendons. The vertical tendons are inverted U-shaped tendons and they are divided into two groups of tendons at 90 degrees to each other. The base slab is utilized for the anchorage of the vertical tendons.

The containment shell has a large number of penetrations. The majority of these, with a diameter between 20 cm and 50 cm, allow for the passage of electrical circuits and piping through the containment wall. The penetrations for the main process pipes, ventilation and electrical circuits have a diameter between 50 cm and 120 cm. The largest penetrations for the equipment hatch (diameter 742 cm), personnel air lock (diameter 273 cm), and auxiliary personnel air lock (diameter 273 cm) provide access for personnel and equipment. Other attached structures and equipment of the containment shell are the liner plate and the supporting structures of the polar crane and accumulator tanks of the emergency core cooling system.

NON-LINEAR CHARACTERISTICS OF MATERIALS

The isotropic material of the concrete of grade B45 is described using the stress-strain relationship presented in Figure 1 and Table 1. The reinforcement has been taking into account by giving the concrete correponding amount of tension strength.

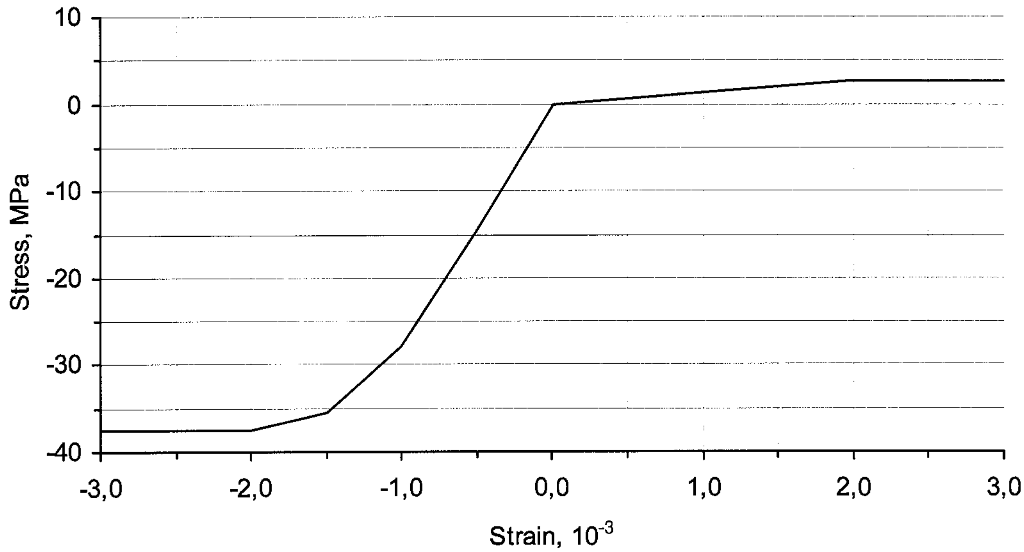

Figure 1 Non-linear stress-strain relationship of the concrete of grade B45

Table 1 Non-linear stress-strain relationship of the concrete of grade B45 in tabular form

Strain, 10^{-3}	-1000	-2.0	-1.5	-1.0	-0.5	0.0	1.95	1000
Stress, MPa	-37.5	-37.5	-35.5	-28.0	-14.5	0.0	2.613	2.613

In choosing the material properties for the nonlinear analysis of the containment shell the specifications given in references [1] and [2] were used. To carry out the physically non-linear stress analysis of the containment shell the options available in the references [3], [4] and [5] were used.

TENDON LAYOUT AND PENETRATIONS ARRANGEMENT

General view of the post-tensioning tendon system is given in Figure 1and Figure 2. As can seen from Figures 1 and 2 the containment shell penetrations are arranged in two main bands. The first band is situated in the lower part of the shell and the largest penetrations in this band are containment sump drain pipes and auxiliary personnel airlock. The second band is situated on the upper part of the containment shell above the main operation deck and biggest penetrations and openings in this band are the material air lock, personnel air lock . In between these two horizontal bands are the main steam and main feed water penetrations. The bands form clear weak zones in the containment shell, where the pre-stress losses concentrate and there the requirements for mild reinforcement amounts are the highest. These are also the areas, where the high values of deformations and pronounced cracking are to be expected in the beyond design basis investigations of the containments shell.

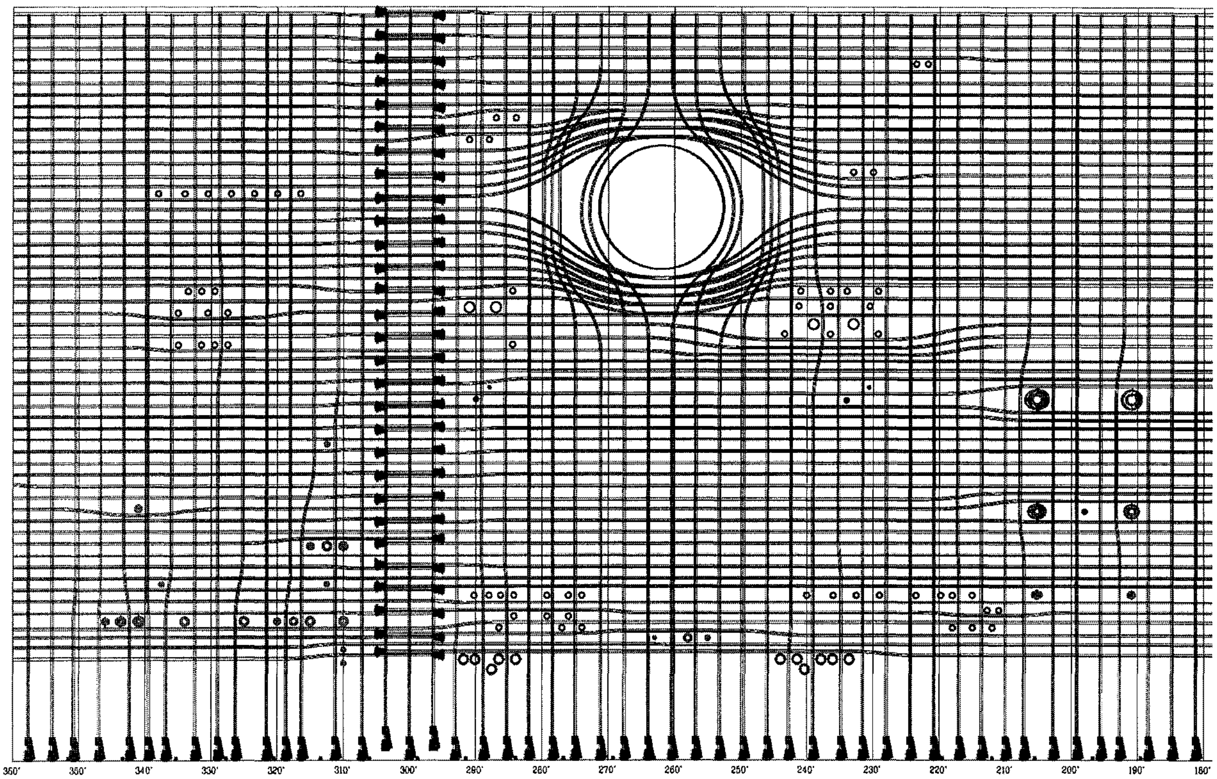

Figure 2 First half of the spread plot depicting the post-tensioning system in cylinder wall

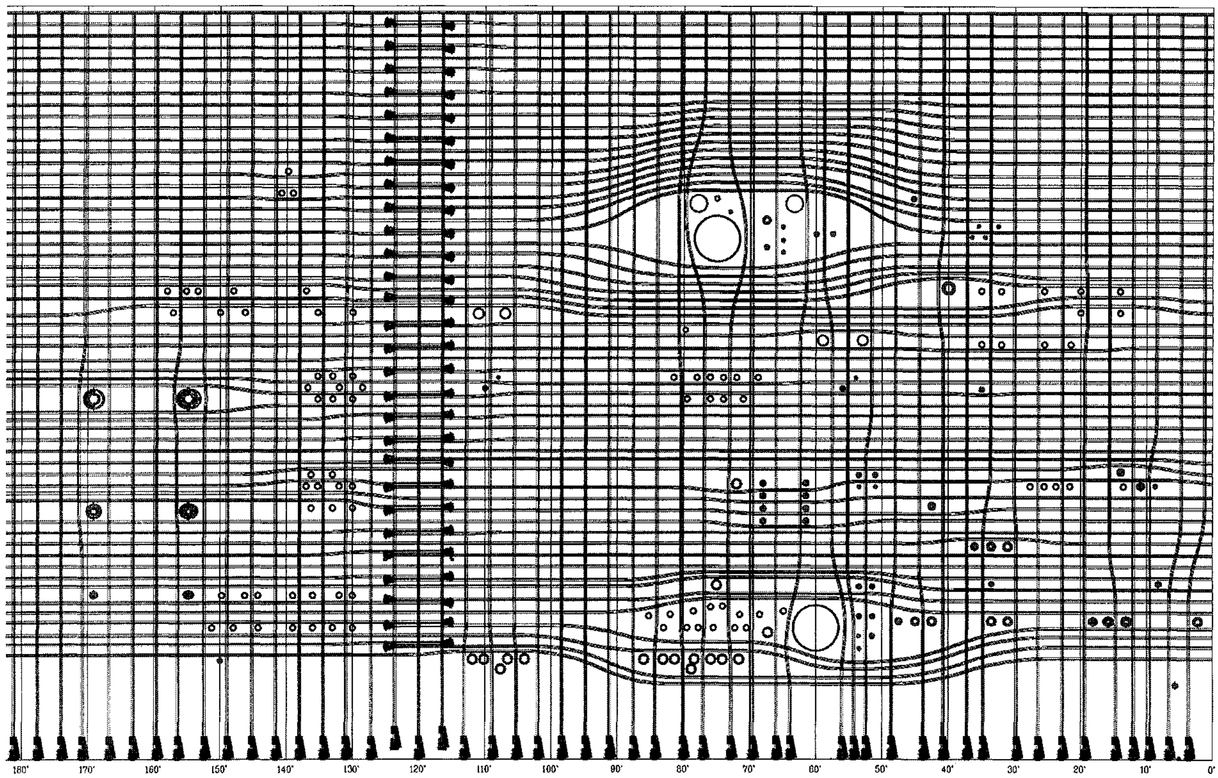

Figure 3 Second half of the spread plot depicting the post-tensioning system in cylinder wall

LOADS AND LOAD COMBINATIONS

The dead load of the reinforced and pre-stressed concrete structures was calculated by multiplying the volume of the concrete with the weight density of 25 kN/m^3. The influence of the additional surface concrete of thickness 60 mm was added using equivalent uniformly distributed load 1.5 kN/m^2 on each intermediate concrete floor. On the roof the additional uniformly distributed load is 2.5 kN/m^2. The dead load of the pre-stressing tendons and the steel structures was calculated using the weight density of 77 kN/m^3.

The pre-stressing forces of the tendons were calculated by taking into account all losses in tendon forces. Two separate cases were analysed: pre-stress immediately after pre-stressing work and pre-stress at the end of the design lifetime i.e. after 40 years. Stresses are presented separately for four groups of tendons described in Table 2. The extreme values of these stresses are collected in Table 3. In this analysis only the pre-stress after 40 years was used.

Table 2 Group names of the tendons

Group name	Description
Horizontal tendons group 1	Horizontal tendons anchored in buttress at 120^o
Horizontal tendons group 2	Horizontal tendons anchored in buttress at 300^o
Vertical tendons group 1	Vertical tendons in direction of 30^o
Vertical tendons group 2	Vertical tendons in direction of 120^o

Table 3 Extreme values of axial stresses of the tendons without elastic losses

	Horizontal tendons, MPa		Vertical tendons, Mpa	
	Group 1	Group 2	Group 1	Group 2
After prestressing, maximum	1452	1452	1413	1413
After prestressing, minimum	603	649	740	672
After 40 years, maximum	1347	1347	1315	1315
After 40 years, minimum	540	583	674	610

The design pressure load generated by the design basis accident (DBA) is 0.40 MPa overpressure inside the inner containment. In this analysis the internal overpressure of 0.60 MPa was used.

LOAD COMBINATIONS

Two static analyses were made using the aforementioned loads. In the linear analysis all the loads were applied at the same time. In the non-linear analysis the internal pressure was increased in small steps, however, the total load is the same as in the linear analysis, see Table 4. For choosing the load combinations utilized in this beyond the design basis stress analysis the specifications given in the references [6] and [7] were used.

Table 4 Analysed load combinations

Category		Description	Linear analysis	Non-linear
D		Dead load	1.0	1.0
F		Prestress after 40 years	1.0	1.0
P		Internal pressure	1.0	1.0

RESULTS OF THE ANALYSIS

Maximum and minimum principal strains at inner and outer surface of the inner containment are collected in Table 5. The strains were evaluated in the centre point of each shell element.

Common values of the strains are less than half of the extreme values. Distribution of the linear and non-linear strains is quite similar in compression zones but different in tension zones, which is a direct consequence of the different stress-strain curves.

Table 5 Extreme values of strains for the inner containment

Load combination	Strain component	Maximum principal strain, 10^{-6}		Minimum principal strain, 10^{-6}	
		Inner surface	Outer surface	Inner surface	Outer surface
Linear analysis	Maximum	290	502	75	130
	Minimum	-90	-107	-740	-372
Non-linear analysis	Maximum	1900	2220	501	850
	Minimum	-175	-165	-1070	-683

The distribution of principal strains in the outer surface of the containment shell is shown in Figure 3.

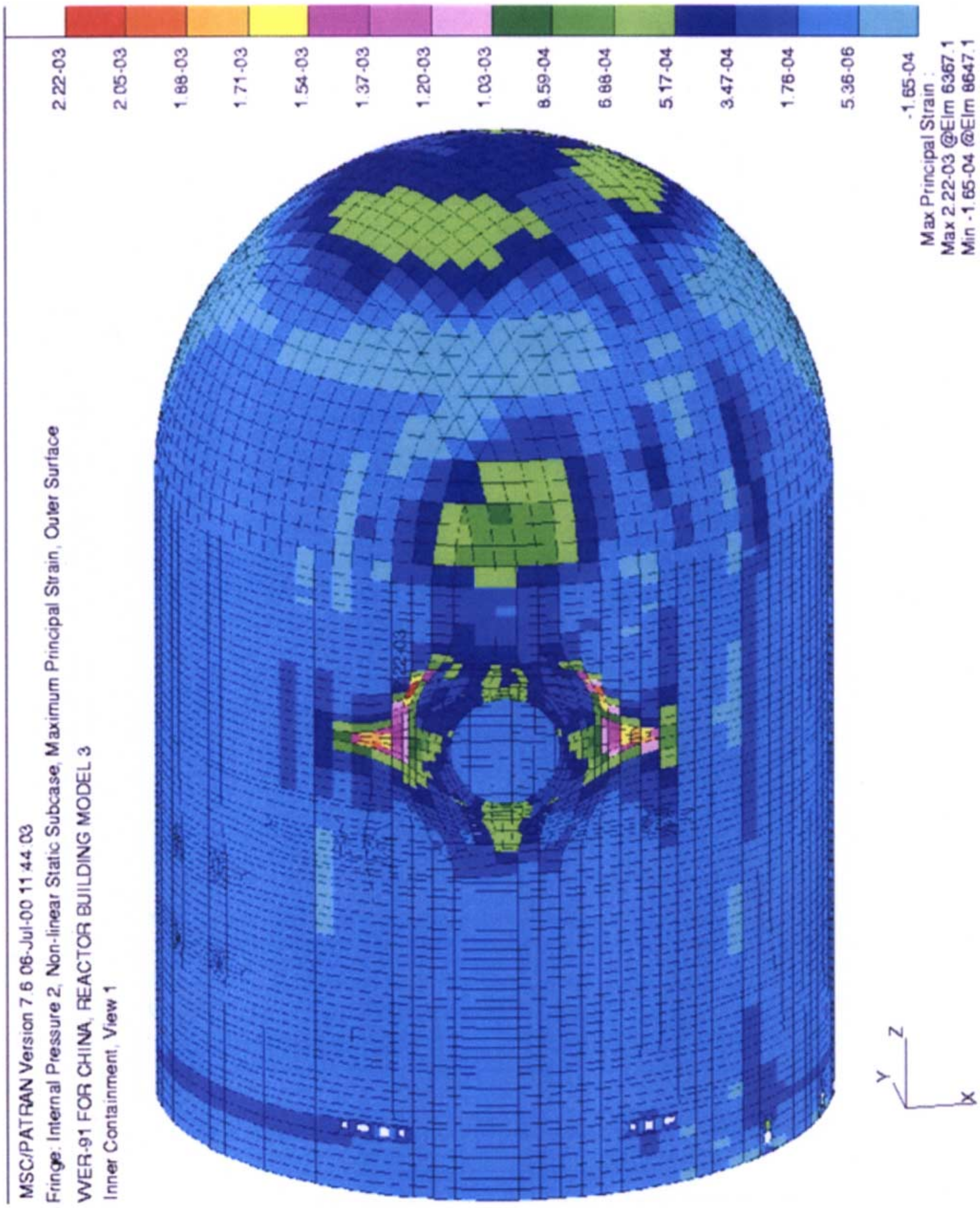

Figure 4 Distribution of the maximum principal strain in the outer surface of the containment shell for 7 bar absolute internal pressure

In the non-linear analysis the concrete structures are more compressed than in the linear analysis, which is a consequence of the different stress-strain curves. Common values of the stresses are less than half of the extreme values. Distribution of the linear and non-linear stresses is quite similar in compression zones but different in tension zones. The extreme values of concrete stresses are collected in Table 6. The stresses were evaluated in the centre point of each shell element.

Table 6 Extreme values of concrete stresses for the inner containment

Load combination	Stress component	Maximum principal stress, Mpa		Minimum principal stress, MPa	
		Inner surface	Outer surface	Inner surface	Outer surface
Linear analysis	Maximum	8.77	14.68	2.87	5.11
	Minimum	-4.01	-3.93	-21.67	-11.15
Non-linear analysis	Maximum	7.29	4.93	0.87	1.58
	Minimum	-7.13	-6.13	-29.33	-20.05

In the non-linear analysis the tendon stresses are a somewhat higher than in the linear analysis, because tension forces are partly transferred from concrete to the tendons. The tendon stresses are well below the yield strength of 1630 MPa and so the linear material model for the pre-stressing steel is adequate. The extreme values of the tendon stresses are collected in Table 7. The stresses were evaluated in the centre point of each bar element.

Table 7 Extreme values of axial stresses of the tendons

Load combination	Stress	Horizontal tendons, MPa		Vertical tendons, MPa	
		Group 1	Group 2	Group 1	Group 2
Linear analysis	Maximum	1414	1413	1331	1332
	Minimum	582	616	691	628
Non-linear analysis	Maximum	1427	1443	1393	1445
	Minimum	600	624	730	657

The distribution of tendon stresses in hoop and vertical tendons are calculated using the non-linear material definition are shown in Figures 4 and 5.

DISCUSSION

As can be seen from Tables 5, 6 and 7 the differences for 7 bar internal absolute pressure between linear and non-linear analysis are not very pronounced. For Group 2 vertical tendons the increase in the tensile stress is about 10% and for Group 1 vertical tendons the increase is even less. For Group 2 hoop tendons the increase in tensile stresses is less than 5 %. Four Group1 tendons that do not circumference the material air lock the differences are smaller than for Group 2.

For strains the increases are much bigger as can be easily seen from Table 5. This means that when the concrete cracks the maximum and minimum strains at the containment shell surfaces grow many fold compared to the obtained from linear analysis. The inner surface maximum principal strain increases from 290 microstrain up to 1900 microstrain, which over six fold increase. The outer surface maximum principal strain grows from 502 microstrain up to 2220 microstrain, which means over four fold increase. The increase in corresponding minimum values of maximum principal strain is about two fold in both cases. For minimum values of minimum principal strains the variations are also spectacular in largest absolute values, which for the inner surface is about 150 % and for the outer surface is about 200 % The maximum values of minimum principal strain for both inner and outer surface grow five fold, when the nonlinear material properties are taken into account in the non-linear analysis.

As for concrete compressive stresses the maximum compression in outer surface increases two fold in the value and for the inner surface the increase is about 50 %.

The conclusion of the assessment of the numbers in Tables 6 and 7 is that the extreme fiber strian variations can large, when non-linear material behavior of shell is taken into account. The extreme fiber stress stress variations are somewhat mitigated by the plastic flow phenomenon of the material. The changes in tendon strains and tendon stresses are the smallest because the tendons are located near the middle surface of the shell and the internal pressure load can not cause through cracking in the shell even at the value of 7 bar absolute pressure value.

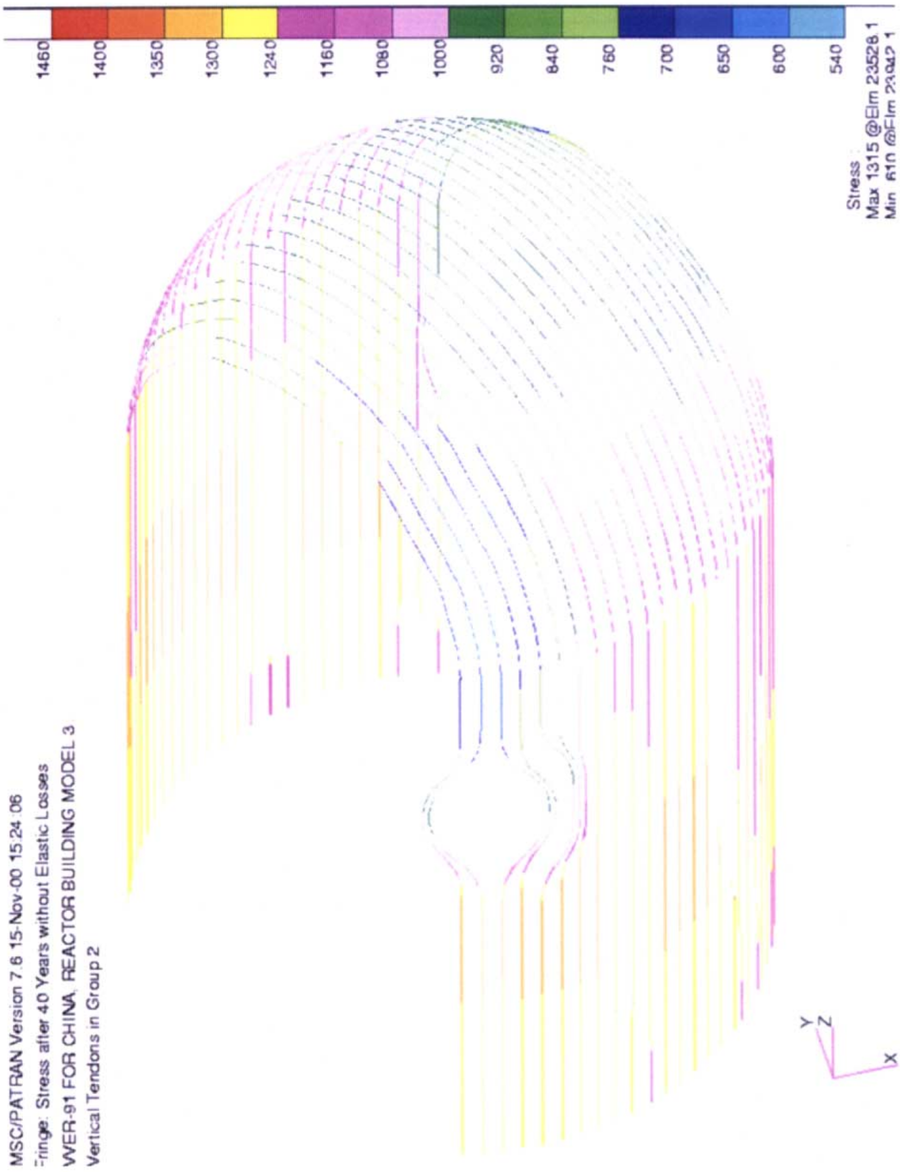

Figure 5 Tendon stress distribution in vertical Group 2 tendons for 7 bar absolute internal pressure

Figure 6 Tendon stress distribution in horizontal Group 2 tendons for 7 bar absolute internal pressure

CONCLUSION

The paper presented the first phase of the stress and strain analysis of the containemnt shell of the Tianwan nuclear power plant for the beyond design basis conditions. The chosen internal pressure value was 7 bars absolute pressure. The load combination ised for the investigation was internal pressure combined with dead load and pre-stress after the losses of 40 years operational life of the plant. The elasto-plastic behavior of the concrete material as well as pre-stressing tendons were taken into account in the analysis. The load was increased to final in ten equally large load steps.

The obtained results show large increase in the extreme fiber tensile strains in the containment wall because of non-linear behavior of the materials. The increases in tendon strains and tendon stresses were only minor because of the non-linear behavior of the materials.

This analysis was only the first step in the integrity assessment of the containment shell for the beyond design basis loading situations. In the second step of containment ultimate strength analysis the reinforced concrete model based on smeared crack concept has to be adopted. This step requires developing of the new element model for containment shell. The pre-stressing tendons and reinforcing bars have to be modeled on individual bar basis using elasto-plastic material properties for steel. Concrete should be modeled with the aid of measured stress-strain curve and crushing strength on the compression side and with the aid measured rupture strain in the tension side. The cracking status has to be monitored in each integration point during the loading sequence of the finite element model. The cracks can open and close but never heal. The open cracks should have shear resistance parallel to crack surface because of aggregate interlock effect. The aim of the second step of the beyond the the design basis strength analysis is to load containment until failure and the result is the ultimate failure pressure of containment.

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