

6. DEFINITION OF LOADS AND SHELL ANALYSIS

6.1 Introduction and Background Information

Criteria and rationale for the definition of loads acting at the time of the collapse are developed in sections 6.1 through 6.8 for use in the analytical investigations of the shell presented in section 6.9. A total of five basic sources of loading (load cases) and the manner of their distribution on the shell are identified for this analysis as follows:

- 1) weight of the tower - continuous
- 2) weight of the scaffolding - 96 ribs
- 3) live loads on the scaffolds - 96 ribs
- 4) weight of cathead assemblies - 12 ribs
- 5) hoisting loads - 2 ribs

Loads attributed to the weight of the tower are generated internally by the shell analysis programs used in this investigation from the tower geometry and unit weight of concrete prescribed in the input. The procedure for evaluating loads from the other sources cited above is discussed in sections 6.4 through 6.7. Assumptions made with regard to dynamic loading in the derivation of loads are discussed in section 6.8. Geometric considerations for the definition of loads and the mechanism for their conversion into forces directly applied to the shell are discussed in section 6.2 and 6.3, respectively.

The various steps used in the conversion of externally applied loads into equivalent forces acting directly on the shell structure may not be readily obvious to those having no prior familiarity with the construction scheme used. Therefore, some background information for the material contained in subsequent sections is in order.

The scaled line drawing shown in figure 6.1 is a key sketch that should assist the reader in understanding the construction scheme used at the Willow Island site. It represents a typical sectional profile of the scaffolding, the cathead assembly, the static and hoist lines, and the concrete bucket. The position of the bucket shown at the top is that assumed at the time concrete is unloaded into Georgia buggies (not shown) located on the top deck of the inside scaffold. The sketch of the bucket shown by the dotted lines represents an intermediate position during delivery.

The hoist line rides over two sheaves suspended from the cathead beam as indicated. On the outside, it continues on down to the ground level (fig. 2.17) where it is wound around a hoist drum which controls its movement. The hoisted loads (concrete bucket, bundled bars, construction hardware, etc.) are attached to the inside terminal of the hoist line, and, in addition, by means of a pulley mechanism (point K), ride on the static line GKB supported at points G and B. The static line provides stability against lateral movement during delivery of materials. The hoisting loads exert a transverse pull and thereby induce a tensile force in the static line. As hoisting of materials proceeds up from the ground level, the tension in the static line increases to a maximum somewhere close to but below the halfway mark, gradually diminishes thereafter, and vanishes entirely when the hoist line becomes vertical. At that point the static line is referred to as becoming slack (i.e., tensionless, or stress-free) but without play.

To determine the hoisting loads acting at the time of the collapse, a brief note of explanation is needed about the situation just prior to

the collapse. All reinforcing bars and various construction-related equipment were delivered to the top. The first bucket of concrete was delivered to cathead no. 4. Worker statements indicated that when the first bucket was delivered to the top, adjustments in the length of the static line were made at the grip-hoist below (point I in fig. 6.2 - sec. 6.2) to remove excessive slack, if any, with the assistance of the worker who unloads the bucket at the top. Therefore, if the first delivery of concrete was used as a trial to adjust (tighten) the static line for subsequent runs, the force in that line would be higher during the delivery of the second bucket of concrete to cathead no. 4. The collapse, in fact, did occur during the second delivery. In the meantime, the first bucket of concrete was delivered to cathead no. 5 and unloaded into Georgia buggies. This bucket had not commenced its descent at the time of the collapse, a situation which was established on the basis of the field data (fig. 6.3 - sec. 6.2). No other hoist lines were in operation at the time of the collapse so that hoisting loads (load case 5) occurred only at the two ribs flanking cathead no. 4.

In section 6.7, the hoisting loads at cathead no. 4 are calculated based on the assumption that the static line becomes slack when the bucket reaches its unloading position as shown in figure 6.1, so that its stress-free length is represented by the solid line GKB. This assumption is further verified by direct measurements of the length of the static line obtained at the site after the collapse, and used in the subsequent investigation of hoisting loads.

Referring to figure 6.1, it is noted that most of the externally applied loads (load cases 2 to 5) are transmitted to the pair of opposite

jacking frames present at each of the 96 rib locations (the exceptions are discussed in sec. 6.3). The jacking frames transmit these loads to the jumpform beams which in turn transmit them to the concrete shell through a series of interconnecting anchor bolts. The cathead forces, including the weight of the cathead assembly (load case 4), and the forces exerted by the static and hoist lines (load case 5), are transmitted to the shell through the cathead leg support brackets attached to the jacking frames, and through the counterstatic line which bears against a wide flange beam and is attached to the lower diagonal as shown.

Before the analysis of construction loads could be carried out, it was necessary to gather a substantial amount of information about the construction scheme, including sizes, lengths and material compositions of the various components comprising the scaffolding, hoisting and cathead assemblies. All of this information was assembled through numerous site investigations and laboratory testing of components recovered from the wreckage, supplemented and corroborated by information from project drawings and worker statements. The source and nature of this information is cited at the appropriate places in subsequent discussions.

6.2 Geometry

To evaluate the hoisting loads on the tower, it is first necessary to define the geometric configuration of the hoisting cables used for the delivery of construction materials to the top of the shell at the cathead locations (fig. 2.18). The cables of particular interest are those serving catheads no. 4 and no. 5 where concrete was being hoisted at the time of the collapse. The required information was developed on the basis of data obtained from the site following the collapse.

Part of the field survey data is presented in figures 6.1 through 6.3. Figure 6.1 is a sectional profile (through a vertical plane of revolution) of the scaffolding and hoisting systems at a typical cathead location. The scaffolding, jacking frames and jumpform beams occur at each of the 96 ribs while the six catheads, including the static, counter-static and hoist lines, the skid board and the wide flange beam, occur at equal intervals (every 16 ribs) along the circumference of the shell. Each cathead is located between two adjacent ribs and transmits its loads to these ribs through the counterstatic line (fig. 6.1) and two pairs of inclined legs at the interior and exterior of the shell, respectively.

In plan, the hoisting layout for catheads no. 4 and no. 5 is shown in figure 6.2. In elevation, the layout for cathead no. 4 (others are similar) is shown in figure 6.3. These figures are helpful in visualizing the mechanism for the delivery of materials to the top of the partially-completed tower shell where the casting operations for lift 29 were being carried out. The lifting of materials at cathead no. 4 is controlled by the hoist drum operator stationed at U. Starting from the drum, the hoist line proceeds toward and around an interior sheave attached to sheave block T on the ground, toward and around exterior sheave Q, up and around the two sheaves suspended from the cathead beam and, before hoisting commences, all the way down to a concrete hopper at B near center A of the tower. A stationary static line is attached to point G on the cathead beam at the top. At the lower end, it passes through a clevis secured to a concrete hopper at B, and terminates at grip hoist I which in turn is secured to the ground.

The space coordinates of, and distances between, points given in figure 6.2 were established by double triangulation for independent verification of results. Note that the cathead is radially aligned (along AQ) but the horizontal projection BG (shown dotted) of the chord of the static line is not. Even though this introduces a horizontal force component of the static line acting on the cathead beam in the direction perpendicular to the beam axis, its effect is not significant and has been ignored in subsequent calculations of hoisting loads (sec. 6.7). In figure 6.3, dimensions such as those along the cathead beam, the diameter and pivotal distances of the sheaves, the lengths of the cathead legs, the location of their bases relative to the shell, and the height of lift 28, were obtained by direct measurement in the field and were verified against the drawings. The lengths of the hoist lines for catheads no. 4 and no. 5 shown in figure 6.3 were likewise obtained by measurement in the field. They represent the length from the hoist drum U to the point of attachment of the concrete bucket (point K in the figure). The cathead leg dimensions shown are the projections of the actual lengths of the cathead legs which are inclined with respect to the plane of the figure. Likewise, line BG in figure 6.3 is the projection of the chord length of the static line which is inclined relative to the plane of the figure (i.e., point B lies outside this radial plane).

The results shown in figure 6.3 were obtained from the foregoing data assuming the cathead beam to be level (see sec. 6.6 for explanation). They define the coordinates of the top support points for the static lines for catheads no. 4 and no. 5 as well as their chord lengths and corresponding horizontal and vertical projections. They also indicate

B_1 at the top, a force which may be resolved into components normal and parallel to the shell. The normal component is transmitted to the jumpform beam while the jacking frame receives the parallel component through a pinned extension. Similarly, the lower diagonal to which the counterstatic line is attached exerts a force through its lower connection F_1 . The normal and parallel components of this force are transmitted respectively to the jumpform beam and the jacking frame. In addition, the connection at this point develops a horizontal pull normal to the plane of the figure due to the inclination of the counterstatic line with respect to that plane.

The jumpform beams receive forces through two sets of end rollers and a pawl attached to each jacking frame (see fig. 5, appendix A). The pawl is mechanically engaged to one of the lugs on the outstanding flange of the jumpform beam. The rollers are free to slide along the flange but are constrained against movement in the normal direction. The forces on the jumpform beam were calculated by treating the jacking frame as a two-span continuous beam on two exterior roller supports and an interior pin support (at the pawl) as indicated in figure 6.4a.

Figure 6.5 identifies forces and couples acting on the outstanding flanges of opposite jumpform beams at a rib. Points A through F (also appearing in fig. 6.4a) define the location of the jacking frame supports relative to the jumpform beams. The two exterior diagonals noted above are connected to points B_1 and F_1 . Forces F_x , F_y , F_z and circumferential moment M_y are identified by reference to the local coordinate axes (x , y , z) as shown. Check marks and zeros shown in tabular form indicate respectively those actions (force component or moment) that can or cannot develop at each point. Actions F_z and M_y occur in opposite pairs as

noted in the figure, and develop as a result of the inclination of the cathode legs and counterstatic line relative to the plane of the figure.

The final stage in the conversion of external loads (load cases 2 to 5) into equivalent actions applied directly to the shell leads to the results tabulated in figure 6.6. The points of application of these actions, designated by the letters I, J, K and L (also appearing in figure 6.5), are located on the middle surface of the shell. Actions occurring at E and F (refer to fig. 6.5) are converted into their statically equivalent counterparts and placed at K. Actions at F_1 are converted in a similar manner and placed at L. The resultants of F_y forces and M_y moments acting on the cantilever portion of the jumpform beams (points A, B, C, D and B_1) are distributed equally to the two anchor bolts at I and J. The F_x forces at I and J induced by the forces acting on the cantilever portion of the jumpform beams are calculated by treating the anchors at these junctions as simple supports.

The preceding steps involve certain assumptions regarding the distribution of forces to the anchor bolts. For instance, the equal distribution of F_y forces to anchor bolts at I and J implies that the jumpform beams are axially rigid and no friction can develop at the shell interface. The other extreme situation would occur if the axial rigidity of the jumpform beams were negligible relative to that of the shell, in which case, nearly all of the F_y forces from the cantilever portion would be transferred to the top anchor bolt. In reality, the axial stiffness of the shell is many times that of the jumpform beams and therefore, the actual distribution will be somewhere between the two extremes with the greater portion of the force going to anchor bolt I. However, the

net effect of the two extreme assumptions on the results of the shell analysis is not significant. In fact, after examining the combined effect of all the assumptions made in the conversion of loads from the jumpform beams to the shell, it was determined that additional refinements will not alter the conclusions of this investigation.

6.4 Weight of Scaffolding - Load Case 2

The scaffolding assembly, including formwork and steel joist supports (fig. 2.5) is axisymmetric so that it transmits equal forces to the 96 peripheral ribs of the shell (fig. 3, appendix A). The procedure for evaluating the forces at a rib due to the weight of a repetitive scaffolding section (see fig. 6.7) is straightforward and involves no major assumptions. The weights of the individual components, evaluated from data on sizes and material composition obtained from the site and the drawings, were placed at their respective centroids and are indicated by vertical arrows in figure 6.7 (arrows with notation are for other load cases). These include planks used for decking and supporting brackets, diagonals and straps, formwork and joists, railings and posts, safety nets, hydraulic actuators, lines and jacks, stiffbacks, jacking frames, jumpform beams and miscellaneous other hardware. In the analysis, the junctions of the lower deck brackets and suspended outer straps (fig. 6.7) were assumed pinned so that with the exception of the diagonally braced bracket at the exterior of the shell (second deck from the top), the system was rendered statically determinate. This bracket, together with the two diagonals was treated as a pinned truss with one degree of redundancy. The final forces on the shell resulting from case 2 loading are tabulated in figure 6.6.

6.5 Live Loads - Load Case 3

Live loading consisted of the weight of 51 workers, reinforcing bars, Georgia buggies, electrical and welding equipment, gas cylinders, water drums, tools, grouting materials, fresh concrete, miscellaneous hoses, wires, cables and other hardware. The live loads were applied equally to the top decks of the interior and exterior scaffolds (designated by P_L in fig. 6.7) and were assumed to be uniformly distributed around the periphery of the shell. After examining all the evidence at hand (field observations at the Willow Island, W. Va. and Berwick, Pa. sites, plus worker statements), it was concluded that live load distributions other than those assumed cannot be reasonably justified so that large concentration of such loading occurring at any one location, including in the vicinity of catheads no. 4 and no. 5, were ruled out. Furthermore, it is noted by reference to figure 6.6 that the contribution of live loads to the total load on the shell is relatively small so that variations in live load distribution will not significantly affect the shell analysis results. The procedure for evaluating shell forces induced by live loads (fig. 6.6) is the same as load case 2 discussed in the foregoing section.

6.6 Weight of Cathead Assembly - Load Case 4

The free-body diagram of a cathead in the radial plane is shown in figure 6.8. Without both the static and counterstatic lines acting (at G and C, respectively), the cathead assembly is rotationally unstable. If, for any reason, the static line were absent, the cathead could still retain its stability provided the resultant overturning moment due to gravity loads about fictitious point O (point of intersection of cathead legs a and b) is counterclockwise so that the counterstatic line is in

tension. Using the values for the gravity load components W_1 , W_2 , W_3 and W and their locations specified in figure 6.8, it can be readily shown that the corresponding overturning moment is, in fact, counterclockwise if the cathead beam is assumed to be in horizontal alignment. It can also be shown that it does not take much backward tilting from this level position (and this can be manipulated by means of the chain hoist - fig. 6.7) to cause a rotational collapse of the cathead in the clockwise direction (outward) in the absence of the static line.

Referring to figure 6.1, it is noted that, with the cathead beam level, the proximity of the bucket to the top deck of the inner scaffold is sufficient to permit unloading of the concrete into on-deck Georgia buggies by means of a chute pivoted to the bottom of the bucket (see figure 6.9 for chute detail). It should be noted that the position of the bucket relative to the deck can be controlled, in addition to the chain hoist, by the special bolt on the slide plate assembly to which the static line and the inside sheave of the hoist line are attached (fig. 6.7). The position of this plate was established by measurement of dimensions of the component parts found among the wreckage at the site.

The assumptions that the cathead beam was level at the time of the collapse is based on the foregoing arguments and is further corroborated by the alignments observed at the Berwick, Pa. site.

The gravity loads which are transmitted to the six pairs of adjacent ribs at the six cathead locations around the shell are designated in figure 6.7 (shown in parentheses) and figure 6.8. They consist of the weights of the cathead legs W_1 , W_2 , W_3 , the weight of the cathead beam assembly W , which includes the weights of the sheaves, the weight of the

hoist line W_H but not the hoisting loads, the weight of the steel skidboard P_{SB} , the weight of the wide flange beam P_{WF} , and tension T_0 in the static line due to its own weight. It should be pointed out that tension T_0 depends on the stress-free length of the static line assumed. The criterion used in determining this length as well as the corresponding cable forces T , F_D and F_0 due to hoisting loads (load case 5) are discussed in section 6.7.

Having established the cathead position and component forces of the assembly, the corresponding leg reaction components at supports A and B (fig. 6.8) and the forces F_C , P_C and Q_C (figs. 6.4 and 6.8) transmitted by the counterstatic line were determined by statics. The final forces on the shell corresponding to load case 4 are tabulated in figure 6.6.

6.7 Hoisting Loads - Load Case 5

Once the geometry of the cable profiles are defined (sec. 6.2), cable forces and displacements induced by the hoisting loads can be determined. To minimize the human computational effort involved, a computer program was developed for that purpose. The program is based on the representation of the static line by the simple cable model depicted in figure 6.10.

The model assumes the static and hoist lines to be weightless (the weight of these lines are considered elsewhere - see sec. 6.6). The hoist line is very nearly parallel to segment KG of the static line when the bucket is about midway to the top (location for maximum tension in the static line) and the program assumes it to be parallel so that force F_D acts in the same direction as KG for all locations of point K. The input parameters are as defined in figure 6.10. The fixed input parameters (the values of which are indicated in the figure) are chord length L ,

coordinates X_G and Y_G of support point G relative to support point B, and the cross-sectional area A and modulus of elasticity E of the static line. The variable input parameters are, position K of the bucket along the static line as defined by chord length segments L_1 and L_2 , the normal distance of that point from the chord as defined by parameter D, and the hoisting load W_B . Two of the values used for W_B were 2900 lb (1314 kg) and 5800 lb (2627 kg). They represent, respectively, the weight of the bucket assembly and concrete (calculations shown in fig. 6.9), and twice that weight to assess the effect of a 100 percent dynamic amplification of hoisting loads on cable forces.

Figure 6.11 shows in notation form a partial listing of the equations used for calculating the desired output parameters which are indicated by asterisks. Not shown are the equations for calculating the cathead leg reaction components A_p , B_p , A_z , B_z and tension F_c in the counterstatic line (fig. 6.8), and a refinement introduced in the program to account for changes in the output parameters resulting from the displacement of support point G due to the elastic deformation of the cathead assembly.

Note that s_0 , the stress-free length of the static line, is an output value. The program was used in an iterative fashion by adjusting the input parameter D to produce the value of s_0 that matched the true stress-free length of the line. The true stress-free length is discussed below.

The cathead leg reactions and the force in the counterstatic line are determined using the free body diagram of the cathead (fig. 6.8) under the action of the static line force T and hoist line forces F_D and F_O . The cathead sheaves are assumed to be frictionless so that

$F_O = F_D$. By reference to figure 6.8, note that cathead leg reactions A_Z and B_Z are normal to the plane of the figure and occur in opposite pairs; reactions A_P and B_P act along projections a and b of the respective inside and outside cathead legs in the plane of the figure; and reactions A_N and B_N are zero because no intermediate loads (such as, W_1 , W_2 and W_3) are acting in this case (refer to sec. 6.6). Thus, the resultant cathead leg reactions are axial. Figures 6.12 and 6.13 should further assist in visualization of the spatial configuration of the cathead legs, the counterstatic line, and reaction components (also see fig. 9, appendix A).

For a given location of point K (fig. 6.10), the program outputs two sets of results. The first set corresponds to the assumption that point G is fixed in space. The second set is obtained by an iterative process in which successive trials are based on the adjusted position of G corresponding to the elastic deformation of the cathead assembly under cable forces from the preceding trial.

Figures 6.13 through 6.15 show the analytical formulations for calculating the movement of point G attributed to the elastic deformation of the cathead assembly under the action of the static and hoist line forces. The expression for the displacement Δ_C of the counterstatic line under tension F_C is derived in figure 6.13. In figure 6.14, small rotational displacement geometry is used to develop the expression for (X'_G, Y'_G) , which represent the movement of point G, due to the elongation of the counterstatic line. The axial deformations of the cathead legs are smaller than the elongation of the counterstatic line by about one order of magnitude, and therefore, their contribution to the movement of

point G is negligible. Figure 6.15 shows the formulation for displacements (X_G'' , Y_G'') attributed to the flexural deformation of the cathead beam. As in the case of the legs, the effect of the axial deformation of the beam is neglected so that X_G'' is assumed to be zero. The total displacement (X_G , Y_G) then is obtained by adding the displacements due to the elongation Δ_c of the counterstatic line at the top and flexural deformation of the cathead beam.

In this case, superposition of tension T_0 in the static line due to self weight and tension T caused by loads W_B gives results that are not appreciably different from the tension in the static line (at support G on the cathead beam) that would be obtained from a solution based on both loads acting simultaneously. A more refined analysis was performed for a few cases to verify this fact. The refined analysis considers simultaneously the weight of the cable and the hoisting loads using an approach similar to that advanced by Harrison [6.1]. The reasons for the small difference between the two analyses are obvious. The distributed weight of the static line (100 lb or 45.3 kg) is only about 3 percent of the weight of the bucket (2900 lb or 1314 kg). In addition, with sufficient slack in the static line (this was the case to enable unloading of concrete at the top of the tower), differences in the results (tension in the static line as well as tension F_D in the hoist line - fig. 6.8) from the two alternate procedures tend to disappear.

A glance at the results tabulated in figure 6.6 will indicate that by far the major portion of the forces acting on the shell is produced by the hoisting loads (load case 5). In particular, tension T in the static line is the major source of the bending moments in the shell (hoist

line forces F_D and $F_O = F_D$ remain nearly constant as T varies and balance out so that they contribute mostly to axial loads F_y but not to normal forces F_x - figs. 6.5 and 6.8). Tension T in turn is quite sensitive to variations in the tautness (or, conversely, in the amount of slack) in the static line. Thus, the criterion for estimating the actual stress-free length of the static line at the time of the collapse would affect significantly the results of the analysis.

As mentioned in section 3.2, the length of the static line of cathead no. 4 (the line was recovered from the wreckage) was measured on two separate occasions in the field. The measured lengths were within 2 in (51 mm) of each other. The average measured length, after deducting the length of the portion from grip hoist I to clevis B (fig. 6.2), was 219.17 ft (66.85 m). With the bucket at the unloading position shown in figure 6.1 and the static line assumed tensionless but without play for that position, the stress-free length s_0 is represented by the solid line GKB as shown. This length can be calculated from the space coordinates of points B and G (figs. 6.2 and 6.3) with the known position of point K (the bucket is hanging vertically down). The computed length is 219.19 ft (67.46 m), which is almost identical to the length measured in the field.

If a certain amount of play in the static line were allowed when the bucket is at the unloading position, it would permit workers on the top deck to manipulate the bucket or pull it in, if need be, without encountering resistance from the static line. However, even without such resistance, a worker will have to exert a pull of about 400 lb (1780 N) to bring the bucket in a distance of 1. ft (0.31 m). From the

layout shown in figure 6.1, with the bucket at the unloading position, it is apparent that concrete can be discharged without difficulty into Georgia buggies on the deck by means of the chute projecting inward above the top rail. Thus, there is no reason to believe that such manipulation with the bucket was necessary. Furthermore, if there were any play in the static line at the discharge level, the line would become slack below that level and the possibility of the bucket swaying and impacting the scaffold while being pulled up would increase accordingly. Conversely, with no play at the discharge position, the static line will develop tension at lower levels and will pull the bucket away from the scaffold thus diminishing the likelihood of severe impact.

The foregoing arguments led to the conclusion that the field measured length was the actual length being used at the time of the collapse and, therefore, was adopted as the basis for the stress-free length used in the calculations of hoisting loads (load case 5) and the static line tension due to self weight (load case 4).

The analysis of forces induced by hoisting loads leads to the results tabulated in table 6.1a. The results obtained by considering the elastic deformation of the cathead were used to develop the forces for load case 5 tabulated in figure 6.6. For purposes of comparison only, table 6.1a also shows computer results obtained by assuming support G to be fixed in space. It should be noted that the forces in this table correspond to the location of the bucket which produces maximum tension in the static line. This is about 123 ft (37.5 m) below the cathead beam which is at variance with the 60-ft (18.3 m) position estimated from the field data (fig. 6.3). From manufacturer's specification and worker statements the speed at which

the hoist line was being pulled in was estimated to be about 10 to 12 ft/sec (3.1 to 3.7 m/s). Hence, it appears reasonable to assume that failure was initiated when the tension in the static line was approaching or at, its maximum, and during the next several seconds while collapse was in progress, it ended up at the 60-ft (18.3-m) mark below the carhead beam.

Table 6.1b tabulates forces produced by twice the weight of the bucket which is equivalent to 100 percent impact on the hoist load (or 100 percent dynamic amplification of the hoist loads). These values are only used to demonstrate that distress conditions would not have occurred in the mechanical system even if there had been reason to believe that 100 percent impact conditions were present (see chap. 5).

6.8 Other Effects

The posture adopted in the foregoing derivations was to use lower bound values for loads in situations where they were not known precisely. Hoisting loads, for instance, were treated as though they were stationary because not enough information was available to assess the inherent dynamic effects of the hoisting system used. The reasoning behind this approach was that should such loads lead to structural distress conditions (which turns out to be the case - see sec. 7), the distinct possibility of higher loads having occurred at the time of the collapse will not change the final conclusions of this investigation. The following paragraphs discuss the assumptions made with regard to dynamic loading.

Dynamic amplification of loads occurs as a result of construction activities on the scaffolds (load case 2) and during hoisting of materials (load case 5). Dynamic amplification of live loads (load case 2) is negligible because, for the most part, they are stationary and well-dispersed (see sec. 6.5).

Dynamic amplification of hoisting loads occurs principally as a result of the initial acceleration of the hoist line to bring the bucket up to speed from at-rest position on the ground, fluctuations in the speed with which the bucket is lifted as the speed of the drum hoist varied, uneven winding of the hoist line on the drum, or any other changes in speed made by the hoist drum operator for whatever reasons during the bucket's journey to the top. An increase in the speed of the bucket will cause a rise in tension in both the static and hoist lines while a drop in speed will have the opposite effect.

The dynamic loading condition induced by start-up acceleration depends on how fast the bucket is brought up to speed and the damping characteristics of the hoisting system. This and the other factors noted above could possibly produce substantial amplification of hoisting loads. However, not enough information is available to make a quantitative assessment of these effects with sufficient accuracy to justify their use as part of the basis for arriving at the conclusion drawn.

It should be pointed out that the path of the bucket is curvilinear and therefore, even when moving at a constant speed, the bucket will exert a transverse dynamic force (outward normal to the path) on the static line. The path will be elliptic with foci at supports B and G (fig. 6.3)

if the cable is inextensible and nearly so if it is elastic. Placing the bucket at midspan, assuming a speed of 10 ft/sec or 3.1 m/sec (see sec. 6.7) and using the equation of the ellipse yields an incremental tension of about 60 lb (267 N) in the static line, which is negligible compared with the 4772-lb (21235-N) tension (table 6.1) used in this investigation.

6.9 Analysis of Shell

An analysis of the cooling tower shell subjected to the construction loads described in the preceding sections was made with SHORE-III [6.2], a finite element program developed for the static and dynamic analysis of axisymmetric shells. The cooling tower shell was discretized by a series of curved ring elements, starting at the top of lift 28. Modeling was consistent with the shell structure described in Chapter 2 of this report. Details of the SHORE-III analysis and a verification analysis using SAP IV [6.3] are presented in Appendix B. Construction loads were applied to the shell model and the resultant maximum forces in the shell were determined along with their location.

The finite element model used in the cooling tower analysis is shown along with an expanded view of the model for the top 3 lifts, in figure 6.16. The tower structure is discretized by a series of nineteen closed ring elements and one open type element at the bottom which models the column supports. In order to obtain better stress distributions and properly load the tower, the top two lifts (28 and 27) were subdivided into eleven elements. The other element divisions were generally chosen to account for either changes in shell thickness or changes in the modulus of elasticity of the shell material. Points for load application were

located at the elevations in lifts 28 and 27 where the form system was bolted to the tower. Figure 6.16 shows the elevations, radii, thickness, and elastic moduli used for each element in the model. While the elastic modulus remains constant for an element, the thickness may vary linearly from the top to the bottom of the element as dictated by the shell geometry. Material properties (elastic moduli) used in the analysis were obtained from the test data presented in chapter 4 of this report using the maturity of the concrete in the various lifts on the day of collapse.

The construction loads presented in figure 6.6 are applied to the shell model at the load points indicated as distributed line loads by means of a Fourier Series. This distributed line loading technique is depicted in figure 6.17 for the normal, meridional, and tangential forces, F_x , F_y and F_z , respectively, and for the meridional moment M_y . The x, y, z coordinate system shown applies to the construction loads as described in section 6.3 while the corresponding u, v, w system is the element coordinate system used in SHORE-III.

The loading function used in the SHORE-III analysis is developed by first distributing the concentrated loads about the centerline of the jumpform beam over 0.358 degree circumferentially (10 in or 254 mm at lift 28) for load cases 2 and 3 and 1.875 degrees circumferentially (4.3 ft or 1.31 m at lift 28) for load cases 4 and 5. The 10 in (254 mm) distribution width is the surface contact length between the shell and jumpform beam. This distributed load is then expanded in a Fourier Series which applies the load at the required points around the circumference of the tower for the particular construction load case. The larger distribution angle used in load cases 4 and 5 was chosen because it reduced the number of Fourier series harmonics required to adequately define the loads.

Ideally, a 0.358 degree distribution angle should have been used for all cases. However, when the 0.358 degree distribution angle was chosen for load cases 4 and 5, 150 harmonics were required to produce a load function with an acceptable shape. Increasing the distribution angle to 0.859 degree made it possible to generate a load function with acceptable shape using only 56 harmonics; thereby, reducing computer time and cost by about two thirds. A comparison analysis using both the 0.358 and 0.859 degree distribution angles showed that the maximum stresses differed by only a few percent and the stress distributions were essentially the same. Consequently, the compromise between distribution angle and computer analysis time is justified. Figure 6.18 illustrates the loading function for a normal force applied to the shell by the jumpform beams at a cathead. This load would then be repeated at all six (6) cathead locations in load case 4 to produce a symmetric loading condition.

The development of a convergent Fourier Series with only a few harmonics was found to be a difficult task for load case 5 which is applied only at cathead no. 4. This is because as the number of application points decreases (two points or ribs for case 5) the number of harmonics required for a convergent series increases rapidly. In addition, the computer time required for solution of a load case is related to the number of harmonics in the series. An investigation of the stress distributions that occurred in load case 4 where the six cathead gantry loads are applied to the shell indicated that the internal shell forces of interest decayed rapidly to a small value at approximately 20° from the cathead. The SAP IV verification

analysis also exhibited this stress distribution (see appendix B). Thus, it was concluded that since the catheads are 60° apart, the loads applied at a cathead have little influence on the internal forces at the catheads on either side. Consequently, the loads for load case 5 were applied at all six cathead locations instead of just at cathead no. 4.

The loading functions were developed for each construction load (cases 2 to 5) and a separate analysis was made for each case, including case 1 which was internally generated by the program. Since SHORE-III is a linear elastic finite element program, the principle of superposition applies and the results for each load case may be combined algebraically to find the resultant stresses for any combination of the construction loads. The effect of using a line load with a distribution angle rather than a concentrated load is to predict stress magnitudes which are lower than those that occur in the shell.

Table 6.2 presents the stress resultants in lift 28 at cathead no. 4 for the sum of load cases 1, 2, 3, 4, and 5 at $\theta = 0^\circ$ and 2° , respectively. Figure 6.19 shows the sign conventions for the respective stress resultants. Values at $\theta = 0^\circ$ occur midway between the jumpform beams to which cathead no. 4 is attached while the values at $\theta = 1.875^\circ$ occur at the meridian along which the jumpform beams are attached to the shell.

Figures 6.20 through 6.23 exhibit the distribution of the meridian stress resultant, N_ϕ , the hoop stress resultant, N_θ , and the moments, M_ϕ , M_θ , with θ for selected elevations in lift 28 (refer to fig. 6.17) for the sum of load cases 1, 2, 3, 4, and 5. Because they are symmetric about the cathead, the distributions are shown for only one side of the cathead. It should be noted that the values of the meridional stress at $\theta = 1.875^\circ$, figure 6.20 and also table 6.2, increases significantly at

elevations of 165.721 and 162.125 ft (50.512 and 49.416 m respectively) due to the loads being applied at points I and J (fig. 6.6). The shape of the stress distribution for N_ϕ about the circumference of the shell, figure 6.20, can be understood by considering the way the loads are applied to the shell. The loads prescribed in cases 2 and 3 are applied at each rib and the resulting stress distribution is symmetric about each rib and, furthermore, is identical between all ribs. On the other hand, the loads prescribed in cases 4 and 5 are applied only at the cathead locations. The stress distributions illustrated in figure 6.20 reflected the superposition of these cases and the stresses due to load cases 4 and 5 control the shape of the distributions shown. The hoop stress, N_θ , as seen from figure 6.21 is highest at the top of lift 28 and decreases with decreasing elevation. Finally, the maximum values for N_ϕ and N_θ occur near the top bolt in lift 28 at the jumpform beams on either side of cathead 4. A relative maximum for N_ϕ occurs near the bottom bolt in lift 28 at the jumpform beams. The maximum moments M_ϕ and M_θ also occur at the location of the jumpform beams ($\theta = 1.875^\circ$) with M_θ a maximum near the top of lift 28 and M_ϕ a maximum near the bottom of lift 28 as shown in figs. 6.22 and 6.23.

7. INTERPRETATION OF SHELL ANALYSIS

7.1 Introduction

The overall safety and the reliability of the construction method used at the Willow Island site depend on the integrity of the partially completed shell structure to resist all construction loads. This demands that sufficient strength must be developed in the previously cast shell so that the structural resistance exceeds the construction load effects by a reasonable margin of safety. Obviously, the shell did not have adequate strength to resist the applied load. In order to determine to what extent the applied load effects exceeded the capacity of the shell, the results of the analysis of the shell at a number of critical locations are compared with the resistance values determined by applying existing strength theories. It should be pointed out that no dynamic amplification of the hoist load was included in the shell analysis. Dynamic amplification is known to exist in hoisting systems such as the one used at the Willow Island site, and inclusion of the dynamic effects would have increased the forces in the shell over those determined in this report.

7.2 Strength of the Shell Under Combined Axial Load and Bending Moment

For a given cross section and reinforcement, an interaction diagram such as the one shown in figure 7.1 can be constructed in terms of ultimate axial forces as ordinates and ultimate bending moments as abscissa. For various combinations of axial forces and bending moments, the interaction curve defines a failure envelope in that all points lying on and outside the interaction curve constitute failure. On the other hand, combinations which fall within the area bounded by the interaction curve

represent a safe condition. In this section a number of cross sections in the critical region of the shell are examined using the interaction diagrams. It was shown previously in the analysis of the shell that maximum forces in lift 28 would occur in the vicinity of the two ribs where the legs of cathead no. 4 were positioned. This critical region includes the section between the two ribs as well as the immediate vicinity along the ribs.

In developing the interaction diagrams for the critical cross sections, a shell section of unit width (1 ft) was treated as a rectangular column. In an actual shell element the forces and moments occur in adjacent cross sections parallel and perpendicular to the meridian. By treating a shell element as a column-like element, the forces and moments in each direction are dealt with separately. Any effect on the compressive strength of concrete due to biaxial compression is small [7.2]. Furthermore, the forces acting normal to the reinforcement have little, if any, effect on the stress that can be developed in the reinforcement. Thus, the treatment of a shell element as an uniaxial element is considered a reasonable and expedient approach.

The cross sectional dimensions of the shell sections used for determining interaction diagrams are shown in figure 7.2. The vertical section has two no. 4 bars and the horizontal section has two bars with an equivalent steel area for a 12-inch (305 mm) wide cross section. The actual shell cross section in lift 28 had #4 vertical bars spaced at 8.7 in on center (220 mm) on each face and #4 horizontal bars spaced at 12 in (305 mm) on center on each face (see fig. 2.2).

Because the strength-gain characteristics of the lift 28 concrete as shown by the results of cylinder tests were significantly different from that of the NBS specimens (fig. 4.3), it was decided to use the lift 28 test data to estimate the concrete strength. The strength of concrete at the time of the collapse, approximately 20 hr after placement of concrete (sec. 4.2), is estimated to be 220 psi (1.52 MPa) based on the results of 24 and 25 hr tests, 283 psi (1.95 MPa) and 299 psi (2.06 MPa), respectively (Table 4.5 a and b). The procedure followed to establish the concrete strength is shown in figure 7.3. It is seen that a linear interpolation is made between the time of set (6 hr) and the time when the field cured cylinders were tested. The time of set was established by the experimental results with cement mortar, with consideration given to the inclusion of flyash and water reducing admixture in the lift 28 concrete. Approximately the same value of the compressive strength can be obtained from the strength-maturity plot shown in figure 4.3 with a value of 43 degree F-days being the maturity at the time of collapse. It should be pointed out that the actual concrete strength at the top of lift 28 could have been lower than 220 psi (1.52 MPa) due to the temperature condition of lift 28 was different than that of the test cylinders which were cured on the ground level and due to the rise of free water to the top of the wall both of which tend to lower the concrete strength. If these factors were taken into consideration, the concrete strength could have been as low as 200 psi (1.38 MPa) rather than 220 psi (1.52 MPa). However, in the evaluation of cross-sectional strengths, 220 psi was used.

The reinforcing steel must be embedded adequately in the concrete to develop its yield strength. The length of embedment required to develop

yield is proportional to the square root of the strength of concrete. According to ACI 318-77 [4.4], at a strength level of 220 psi (1.52 MPa) the embedment length for a no. 4 bar in compression is 40.5 in (1029 mm). Because the ACI code requirement incorporates a 25 percent increase in embedment length over that required to develop yield [7.1], the value obtained from the ACI code should be reduced by 25 percent to reflect the actual needed embedment length. Thus, the corrected compression embedment length for a no. 4 bar becomes 32.4 in (823 mm).

The maximum amount of stress that can be developed in the steel was computed based on available length of embedment. The bars in the circumferential direction, which are in compression, could develop full yield strength of 60 ksi (414 MPa) except at lap splices. At the locations where lap splices occur the maximum stress that can be developed in the bar is limited by the length of splice. For the splice length of 22 in (559 mm), which was provided for no. 4 bars (fig. 2.2), the maximum compression stress in the steel is limited to $60 \text{ ksi} \times 22/32.4 = 40.7 \text{ ksi}$ (281 MPa). For the bars in the meridian direction the available length of embedment varies depending on the distance from the top edge of lift 28 to cross sections being considered. For a cross section located 32.4 in (823 mm) below the top of lift 28, the steel can develop to yield (60 ksi, 414 MPa). Under combined bending moments and axial forces, some bars were in tension. For these bars embedment length of 28.8 in (732 mm) was computed using the pullout test results. However, at all locations where the tension embedment length would be a governing factor, the magnitude of these moments and axial forces are relatively low and their combined effects fall well within the interaction diagram.

For the shell cross sections of unit width shown in figure 7.2 and using the limiting concrete and steel stresses described above, interaction diagrams for axial load and bending moment are obtained for critical locations in lift 28 (fig. 7.4). In developing the interaction diagram, it was assumed that the crushing strain of concrete is 0.003 in per in^{4/} and that the reinforcing steel has an elastic-perfectly plastic stress-strain relationship with a nominal yield strength of 60 ksi (414 MPa).

Figure 7.5 shows an interaction diagram for a cross section resisting meridional forces (hereafter referred to as the horizontal cross section) for all cross sections 32.4 in (823 mm) below the top of lift 28. Below this point a sufficient embedment length is available for the vertical bars to develop yield. In lift 28 the meridional force and bending moment are maximum at cross sections along a line through points BAB as shown in figure 7.4. Maximum force and moment combinations for cross sections along this line are plotted in figure 7.5 in which the point corresponding to location A is shown as a square and the point corresponding to location B as a circle. It is to be noted in this figure that at both locations the cross section is controlled predominantly by bending moment. Because the cathead gantry loads are supported at the rib locations, the bending moment is substantially greater at this location than at the center of the panel, exceeding the capacity of the shell cross section. This indicates that a compression failure of concrete would initiate at the inside of the shell

^{4/} In rectangular beam tests, strains 0.003 to 0.004 have been measured near maximum load carrying capacity. Many tests of beams and columns have shown that a satisfactorily accurate prediction of ultimate strength can be made using the crushing strain of 0.003 [4.4]. Limited test data are available for concrete strength of 220 psi (1.52 MPa). However, a more accurate determination of the concrete strain at maximum resistance was not made in this investigation because the effect on the interaction diagram is not large.

at the rib locations and propagate along the circumference at a distance 1 ft (0.3 m) above the bottom of the panel.

The locations of lap splices of the horizontal bars were not known for the cross section resisting hoop forces (hereafter referred to as the vertical cross section). Hence, two interaction diagrams were prepared depending on the maximum stress that can be developed in the steel. If lap splices of the bars are a sufficient distance away from a section being considered, the steel can develop yield (60 ksi, 414 MPa). For cross sections where lap splices occurred, the maximum stress that can be developed is limited by the length of the splice (40.7 ksi, 281 MPa). The interaction diagrams obtained using these two steel stresses are shown in figure 7.6. These diagrams give the combinations of the hoop force and bending moment for failure of the cross section. In lift 28, the hoop force and associated bending moment are maximum at locations C and D. They are plotted on the interaction diagram for these sections in figure 7.6 in which the point corresponding to location C are shown in square and the corresponding to location D in circle. It is seen in this figure that with point D lying considerably outside of the interaction curve a compression failure of concrete at the top of the shell would take place at the rib locations, points D, due to combined effect of high moment and axial force with or without the presence of lap splices. At the center of the panel, the shell section is subjected predominately to axial force.

7.3 Strength of Shell Under Shear

In the presence of axial compression the shear capacity of a reinforced concrete section increases. This complex interaction between shear and axial compression is not fully understood. In the absence of a suitable means of determining the shear strength of a shell section, the

expressions in the ACI code [4.4] are used for insight into the strength of the shell in shear. Those expressions are:

$$V_c = 2 \left(1 + \frac{N_u}{2000 A_g} \right) \sqrt{f'_c} bd$$

and not greater than

$$V_c = 3.5 \sqrt{1 + \frac{N_u}{500 A_g}} \sqrt{f'_c} bd$$

where

V_c = nominal shear strength provided by concrete

N_u = axial normal load to cross section

A_g = gross area of section

f'_c = compressive strength of concrete in psi

b = width of section

d = distance from extreme compression fiber to centroid of tension reinforcement

Radial shear values in the vertical and horizontal cross sections of elements at the top of lift 28 are given in table 7.1 together with corresponding shear capacities computed in accordance with the above formulas. It may be seen in the table that the radial shears in the vertical and horizontal sections (Q_θ and Q_ϕ) are very high at the top of lift 28 along two ribs, while the radial shears at the center of the panel are either zero or very small. The radial shear in both the vertical and horizontal sections in the region near the top of lift 28 along the ribs, points D in figure 7.4, exceed those values computed by the formulas. Although the calculated shear values may be influenced by the distribution of concentrated loads along a line, as discussed in section 6.5, this clearly indicates that relatively high radial shear forces were present in this region of the shell.

7.4 Probable Mode of Failure

It was shown in the previous sections that maximum stresses occur in the region of the panel bounded by the two ribs where cathead gantry no. 4 was located. The analysis showed that a compression failure of concrete would have initiated in lift 28 along the circumference of the shell at a distance 1.0 ft (0.3 m) above the bottom of lift 28, the line through points BAB in figure 7.4. In addition, because both the large hoop forces and moments along the ribs, a band of the compression failure would also have spread along the rib of the panel. The presence of high radial shear in the panel could have further weakened the shell where combination of high axial loads and moments occurred and could have initiated the failure before the capacity in axial forces and moments was reached.

Therefore, it may be concluded that the failure of lift 28 resulted from inadequate strength of the shell section where cathead no. 4 was located to resist the applied construction loads. The failure was brought about by compressive crushing of the concrete due to combinations of axial forces and moments, and/or cracking due to the high radial shear.

8. SUMMARY AND CONCLUSIONS

As a result of the cooling tower construction collapse at Willow Island, West Virginia, the National Bureau of Standards (NBS) was requested by the Occupational Safety and Health Administration (OSHA) to carry out a detailed study aiming at the determination of the most probable cause of the collapse. In response to this request, NBS has undertaken a comprehensive field investigation, laboratory tests of construction assembly components and concrete specimens, and chemical analyses of concrete. In addition, mathematical models of the tower were prepared and analyzed by computer using shell analysis program. The findings presented in this report are based on the results of these field, laboratory and analytical studies with other information such as structural drawings of the tower and OSHA case records.

1. At the time of failure the concrete bucket was in transit from the base of the tower to cathead no. 4. The measured length of the hoist cable indicated that the bucket was about 60 ft (16.4 m) below the cathead beam. This also agrees with eyewitness accounts. Therefore, it is believed that the concrete bucket did not hit the cathead to cause it to fail.
2. Although the hoist cables for catheads no. 4 and 5 were broken, field observations and the laboratory test indicate that the breakage of the cables occurred after the onset of collapse of lift 28. Thus, the breakage of the cables did not trigger the failure.

3. Field and laboratory investigations revealed that the major components of the hoisting, scaffolding and formwork systems did not fail prior to the collapse. Thus, the collapse did not initiate due to any component failure of these systems.
4. Based on the results of field cured cylinder tests, the compressive strength of concrete of that part of lift 28 where cathead no. 4 was located was estimated to be about 220 psi (1.52 MPa) at the time of the collapse.
5. The analysis of the shell as well as eyewitness accounts indicated that the collapse initiated in lift 28 at the area where cathead no. 4 was located. The analysis showed that calculated stress resultants at several points in this area equaled or exceeded the strength of the shell in compression, bending and shear. Failure at any of these points would have propagated causing the collapse of lift 28.
6. These results of the analysis indicate that the most probable cause of the collapse was due to the imposition of construction loads on the shell before the concrete of lift 28 had gained adequate strength to support these loads.

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