# OYSTER CREEK NUCLEAR GENERATING STATION
# CONTAINMENT SPRAY SYSTEM ASSESSMENT
# ASSOCIATED WITH THE POSTULATED COLLAPSE
# OF REACTOR BUILDING SOUTHEAST
# STAIRWELL MASONRY WALLS

Submitted to

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# 1.0 INTRODUCTION

As part of an overall assessment of masonry walls for Oyster Creek Nuclear Generating Station, GPU had determined that without modification, Wall No. 29, which forms part of a stairwell in the southeast corner of the Reactor Building, could fail under seismic loading and potentially impact portions of the Containment Spray System located in the vicinity. A failure of that pressure boundary could result in a loss of fluid and possible draining of the pressure suppression pool (torus).

As a result of this, GPU requested Impell Corporation to conduct a study to assess the potential consequences of the impact of this wall on the Containment Spray System below. Part 1 of the report documents this study and its conclusions.

Part 2 of the report documents a related study of the effects of the masonry walls from el. 23'-6" to el. 119'-3" falling on the stairs. In the worst case scenario, the stair supports might fail under the impact load of the falling blocks. Consequently, stairs and blocks would drop from great heights onto the stairs below el. 23'-6" and potentially rupture the segment of the containment spray piping which runs through the stairwell at el. 11'-6". Based on this evaluation, additional vertical load carrying capacity has been provided for selected stair supports. Construction sketches for the support modifications are provided.

Part 2 also addresses the consequences of Wall 45 (the northwall of the elevator shaft between el. 23'-6" and 51'-3") falling across the elevator shaft and impacting the north wall of the stairwell (Wall 32).

# PART I - CONTAINMENT SPRAY SYSTEM ASSESSMENT

## 2.0 DESCRIPTION OF AFFECTED AREA

The affected area is in a triangular shaped room at floor elevation -19'-6" formed by the south and east walls of the Reactor Building and an interior concrete wall which divides this area from the torus area (Figure 1). All walls are continuous to the next floor elevation (23'-6"). A block wall stairwell is in the southeast corner of this room. Wall No. 29 is the north wall of this stairwell. Its dimensions are 16'-6" wide by 36'-10" high by 8" thick. The wall is assembled in a stack bond pattern with 18" x 8" x 8" nominal size blocks. The structural drawings indicate that the wall is reinforced with #5 bar running through each block in the vertical direction and extra heavy "dur-o-wall" placed in every course in the horizontal direction. The wall has a 2' x 4' opening between elevations 10'-6" and 12'-6" through which a section of the containment spray piping passes.

Since it is reasonable to assume that the potential damage is contained within the room, the affected Containment Spray System pressure boundary includes:

1. A horizontal run of 12" dia. pipe (section "A" in Figure 1) at elevation -18'-0" from the suction side of Containment Spray Pump 1-3 to a wall sleeve at the wall separating this corner room from the torus area. The pipe is anchored at the torus suction header on the other side of the wall. The pipe run includes a motor operated valve V-21-1. It is supported by one rigid vertical restraint and is connected to the pump with a flanged connection.

## 2.0 DESCRIPTION OF AFFECTED AREA

2. A horizontal run of 12" dia. pipe (section "C" in Figure 1) at elevation -18'-0" from the suction side of Containment Spray Pump 1-4 to a wall sleeve of the same wall described above and anchored to the same torus suction header. This run also includes a motor operated valve V-21-3. It is supported by two rigid vertical restraints and a flanged connection at the pump. In addition, a 4" dia. branch line for torus cleaning will be installed near the end closest to the pump.

3. Two runs of 10" dia. piping (section "B" in Figure 1) which come off the discharge sides of the containment spray pumps, run vertically to elevation -10'-0" and are connected by a horizontal run of pipe which runs normal to Wall No. 29. This segment forms a frame structure. The vertical segments include check valves V-21-2 and V-21-4. The piping is supported by spring hangers near the pumps and flanged connections to the pumps. Through reducers and tees, the piping is connected to a horizontal run of 14" dia. piping at elevation -10'-0" which runs parallel to Wall No. 29.

4. The 14" dia. run which runs horizontal and parallel to Wall No. 29 at elevation -10'-0". From the tee connection described above, the west side of the pipe goes beyond the block wall, turns south and continues to the discharge side of the system. The other side of the run turns upward near the east end of the block wall, runs vertically up to the opening in the wall and then goes south across the stairwell and through the south wall of the Reactor Building to a blind flange. The pipe is securely anchored in the three foot (3') thick concrete wall of the Reactor Building. The vertical section includes a manually operated valve V-21-6 and is supported by a spring hanger.

## 3.0 DEFINITION OF POTENTIAL WALL COLLAPSE SCENARIOS

A review of the dimensions of the room and the block wall indicates that it would be impossible for the wall to fall as a complete unit directly upon the containment spray system below. The wall has a height of 36'-10". In the direction normal to the wall the floor length varies between 10' at the short end to 25' at the long end. If the wall were to fall unobstructed, pivoting about the floor, it would hit the concrete wall on the opposite side. However, the 14" dia. pipe which runs parallel to the block wall would be contacted first. There is only a 7" gap between the block wall and this pipe. If the wall were to fall as unit, it would first hit the vertical riser at the elevation of the 2' x 4' opening (10'-6"). If the pipe has sufficient strength to support at least a segment of the wall at the east end, the remainder of the wall could split along a vertical seam and continue to fall until it contacted the horizontal run of the same pipe at elevation -10'-0". Assuming sufficient strength in this section, the wall could continue falling if it yields along the pipe axis, which would then become its new pivot point. Even in this scenario, the room dimensions preclude a direct impact onto the remainder of the containment spray system. In order to contact the system, the wall would have to yield and break apart further, losing a great deal of energy prior to final impact.

A review of the Ebasco stress analysis of Wall No. 29 indicates that the highest stresses occur in the direction parallel to the bed joints near the top of the wall. These stresses are an order of magnitude higher than the stresses perpendicular to the bed joints. This implies that the anticipated failure mode would be cracks along vertical seams at the center of the wall. Considering this fact, as well as the greater strength of the wall in the vertical direction, the stack bond assembly and the weak support at the top of the wall, it is reasonable to anticipate a failure mode in which single or multiple

## 3.0 DEFINITION OF POTENTIAL WALL COLLAPSE SCENARIOS

columns of blocks fall pivoting about their base and then about the pipe. Even in this failure mode, the columns of blocks would strike the opposite wall and would have to break apart further losing significant energy before reaching the piping at the lower elevations. Under worst conditions, the columns would have to break in such a way that the length of the broken segment would equal the length of the room.

Another case to consider is individual blocks falling from a high elevation. This failure mode is considered unlikely due to the vertical reinforcement through each block column. However, even if we assume that the vertical bars do not reach the highest block run and individual blocks fall from the highest elevation, the kinetic energy of a single block was shown to be less than the kinetic energy of column of blocks. This is, therefore, not a controlling case.

For the reasons presented above, individual block column impacts would be the controlling load cases. Additional impacts from block sections rebounding off the concrete wall are considered secondary effects since they would not occur simultaneously with the first impact and the rebounding sections would lose significant energy. The selection of lengths of the columns will be discussed in the analysis section.

## 4.0 SELECTION OF LOAD CASES

Failure of the Containment Spray System is defined as significant violation of the pressure boundary. The system consists of piping, pumps and valves. Since operability is not a requirement, pump motors and valve operators were not included in the evaluation. Since pumps and valve bodies are designed to be stronger than pipes, the most critical areas were judged to be the pipes and their connections. Potential failure modes investigated included beam mode deformation of the pipes, failure of the flange bolts and perforation of the pipes.

These three sections of piping indicated as "A", "B" and "C" in Figure 1 have the potential for experiencing significant impact loads based on their low elevation and their location in the room. Of the three, "A" and "B" were selected for impact analysis. Section "C" was judged to be in an area protected from impact due to the 14" dia. vertical riser's ability to support that area of the block wall.

Additional calculations were performed to insure that the vertical 14" dia. pipe has the capacity to hold up the east section of the wall and that the horizontal section has the strength to form a pivot point for the remainder of the wall. A perforation check of the piping was performed to insure that a hard missile, such as a falling reinforcing bar protruding from a block, could not penetrate the pipe.

# 5.0 DESCRIPTION OF METHODOLOGY

Impact response of the piping was determined in accordance with energy balance techniques as described in references 1, 2 and 3. The method involved the following steps:

1. Determination of a resistance-displacement function of the target pipe. This involved a static piping analysis using the SUPERPIPE computer program to determine the load vs. deflection properties. It also involved the calculation of collapse load which was conservatively assumed to be the load required to form the first plastic hinge in the system. Assuming elastic-perfectly plastic material properties, the resistance curve was assumed to level off at that point.

2. The real system was then idealized to an equivalent single degree of freedom system using transformation factors to determine equivalent mass, load and resistance-displacement function. The transformation factors were established by assuming a deflected shape for the real structure $\phi(X)$. The equivalent mass is defined as:

   $$Me = \int m[\phi(x)]^2 \, dx$$

   where m = mass per unit length

   The equivalent load is defined as:

   $$Fe = \int p(x) \, \phi(x) \, dx$$

   where p(x) = distributed load on system.

   The equivalent resistance-displacement function is obtained by multiplying the real structure's resistance-displacement function by the ratio of equivalent load to actual load.

# 5.0 DESCRIPTION OF METHODOLOGY

3. Definition of a wall collapse scenario in accordance with the guidelines previously discussed. This included the selection of wall section size and height of drop. The kinetic energy at impact was then calculated.

4. Determining the maximum displacement of the equivalent system based on momentum and energy considerations. A plastic impact was assumed in order to determine the velocity and energy of the equivalent "target" and "missile" immediately after impact. The maximum displacement was determined by equating that energy to the strain energy of the equivalent system.

5. Calculation of the ductility ratio which is the ratio of maximum deflection to the deflection at "effective yield" for the system. The "effective yield" point is found by extending the elastic deformation line to its intersection with the collapse load line.

6. Determination of acceptability by comparing ductility ratio to an allowable ductility ratio. A minimum allowable ductility ratio of 10 is considered acceptable by the NRC (Reference 4).

   Additional calculations were performed to insure that the pipe connections had the strength to sustain the collapse load of the pipe. This was done by showing that the flange strength exceeded the pipe strength.

   Potential perforation of the piping was checked by applying the Stanford equation for perforation of missiles through steel (Reference 1).

## 6.0 DISCUSSION OF ANALYSES AND RESULTS

Load Case 1 - 12" dia. Piping Connected to Pump 1-3 (Section "A")

The piping section was modeled from the pump to the suction header using the SUPERPIPE computer program. The model was uniformly loaded from the pump to the wall sleeve to determine its elastic stiffness. Collapse load was determined by a review of the loads and section properties throughout the system. The bending or torsional moment required to produce a plastic hinge at the weakest point set the value of collapse load. In calculating this value a dynamic increase factor of 1.2 was applied to the yield strength as is common practice (Reference 1). For the ASTM-106 Grade B material, this resulted in a dynamic yield strength of 42,000 psi. Based on this a collapse load of 4950 lb/in was computed with a corresponding "effective yield" deflection of .168 inches. Based on an assumed hinged beam deflection shape, the equivalent single degree of freedom consisted of an equivalent mass equal to one third the total piping section mass and an equivalent collapse load equal to one half of the total distributed collapse load.

The wall collapse scenario assumed that a nine foot high section of wall two blocks wide collapsed as a unit pivoting about the 14" dia. pipe. The wall is conservatively assumed to fall flat such that it would apply a uniform load to the pipe. The 9' height is the longest column length that can fit in that floor area. A double column was assumed as an additional conservatism. In reality, a double column would split into two sections under the impact force and not impart its full energy to the pipe. Also in reality, the wall column would hit the valve operator first and split apart again not imparting its full energy to the pipe. A uniformly applied load was judged to be the most conservative case in which the maximum energy must be absorbed by the system.

## 6.0 DISCUSSION OF ANALYSES AND RESULTS

The energy balance equations gave a maximum deflection of .88" which corresponds to a ductility ratio of 5.24. This is well below the allowable ductibility ratio of 10. However, a review of the loads at the rigid vertical restraint indicated extremely high support reaction forces. The support details show that the pipe is on a pipe saddle which rests on a wide flange beam. In such an impact, the pipe saddle can be expected to crush and the support would no longer act as a "rigid" restraint. Rather than attempting a more accurate analysis of the system with a more flexible support, a bounding approach was pursued. The analysis was redone without the support. The results indicated a collapse load of 1440 lb/in with an effective yield deflection of .785 inches. The energy balance gave a maximum deflection of 3.19 inches with a ductility ratio of 4.06 (well below allowable). The two cases bound the real system.

The pump flange connection was also analyzed and shown to be stronger than the pipe, verifying that formation of a plastic hinge in the pipe is the anticipated failure mode.

From the above results, it is concluded that a block wall impact on this section of pipe will not result in failure of the pipe.

<u>Load Case 2</u> - <u>Frame Structure of 10" dia. piping (Section "B")</u>

This piping system was modeled using SUPERPIPE from each of the two pumps to the tee connection with the 14" dia. pipe. A uniform load was applied vertically. Using the same methods discussed in the first example, a collapse load of 810 lb/in was determined with a corresponding yield deflection of .074". The wall collapse scenario assumed a single block column, 10'3" long pivoting about the horizontal 14" dia. pipe striking the top pipe of the frame structure flat. Although a 17' section of block column could fit in that floor area, it was judged unreasonable to have a 6'-10" long section of piping absorb the total

## 6.0 DISCUSSION OF ANALYSES AND RESULTS

kinetic energy of such a long length. Upon impact such a section would either continue its rotational motion pivoting about the end of the frame or break off at the end of the frame. In either case, the full kinetic energy would not be absorbed by the frame. A 10'-3" long section which represents 150% of the length of the frame structure was judged to be a reasonable compromise between the 6'-10" section of piping and the 17' length of floor. Using the same equivalent single degree of freedom and energy balance methodology described in the first example, a maximum deflection of .627 inches was computed corresponding to a ductility ratio of 8.48. The flange was also shown to be stronger than the pipe. It was concluded that this section of piping can also withstand the block wall impact.

Additional Calculations

1. Horizontal Run of 14" dia. Pipe and Frame Structure of 10" dia. Piping Loaded Horizontally

   In the horizontal direction normal to the wall the 10" dia. pipe (Section "B") provides the primary support for the 14" dia. pipe which parallels the wall. As a first cut to the problem, it was assumed that the 14" dia. pipe transfers the impact load of the full wall to the frame. This frame was analyzed using the same types of models and techniques and making the conservative assumption that the wall remains intact after impact. This analysis indicated a collapse load of 31800 lbs. with an equivalent yield deflection of 1.16". The energy balance indicted a maximum deflection of 1.12", which means that the system remains elastic. Based on the conservatism of the model and the favorable results, it was judged that a more detailed analysis is not required to prove that pipe has the capacity to stop the wall and form a second pivot point for the collapsing wall.

## 6.0 DISCUSSION OF ANALYSES AND RESULTS

2. Vertical Run of 14" Dia. Piping Along Block Wall

The 14" dia. riser section of piping, which penetrates the block wall at el. 11'-6" and is anchored in the south wall of the Reactor Building, was analyzed using the same energy balance techniques. As a first cut, a simple hand calculation was performed instead of a SUPERPIPE model. The full wall was assumed to pivot about its base and hit the pipe at elevation 10'-6" (bottom of opening in wall). Assuming that the wall remains intact, the pipe was shown to have sufficient strength to remain elastic. Based on the conservatism of the model and the results, a more refined analysis was not required to prove that the pipe has the strength to hold the eastern section of the wall.

3. Perforation Analysis

A perforation analysis was performed using the Stanford equation. It was assumed that a #5 rebar protruding from a block strikes a pipe after falling from the top wall elevation. The results showed a perforation thickness of .02 inches, which is an order of magnitude less than the thickness of any pipe under consideration. Therefore, it is concluded that perforation is not a problem.

# 7.0 CONCLUSION

The load cases discussed in this report have demonstrated that in the unlikely event of failure of Block Wall No. 29:

1. A section of the east end of the wall would be supported by the 14" dia. pipe riser thus preventing block impact in the eastern section of the room, which includes the horizontal run of 12" dia. pipe attached to Containment Spray Pump 1-4 and the attached 4" dia. torus clean-up line.

2. If the remaining section of wall breaks off, it will hit the 14" dia. pipe which runs horizontally at floor elevation -10'-0". The momentum of the wall would cause the pivot point to shift from the floor to the pipe, but the impact will not damage this pipe.

3. The wall may break up into columns which will impact the concrete wall on the opposite side of the room. These columns may further break up and impact the remaining portion of the containment spray system. The critical pipe areas are the 10" dia. piping which is attached to the discharge sides of the pumps and forms a frame structure normal to the wall, and the 12" dia. piping which is attached to the suction side of pump 1-3 and goes to the torus suction header. Based on what were judged to be the worst potential impact load cases, analysis has shown that both pipe sections would experience permanent deformation but not pipe rupture.

4. In the event that a steel bar from the wall falls directly from the highest elevation and hits any part of the system, it would not have sufficient energy to perforate the system.

## 8.0 DEFINITION OF STAIRWAY PROBLEM

The stairs in the southeast corner of the Reactor Building go from el. -19'-6" up to el. 119'-3". They are completely enclosed by a stairwell. The north and west walls of the stairwell are vertically reinforced masonry walls, which span between floors at el. -19'-6", 23'-6", 51'-3", 75'-3", 95'-3", and 119'-3". Inspection by utility personnel indicated that the vertical reinforcement stops short of the top two courses of block to allow for closure of the masonry walls. All of the stairwell masonry walls are of the same construction; the details were previously described in Section 2.0 of this report. The stair details above el. 23'-6" are shown in Drawing 4065-2 (2 sheets); the details below el. 23'-6" are shown in Drawing 4064-2.

The only concern associated with failure of these masonry walls in the direction of the stairs is the potential for unacceptable damage to the section of the containment spray system piping which passes through the stairwell at el. 11'-6".

One way to insure that this will not happen is to demonstrate that each section of stairs between floors can withstand the failure of the blockwalls spanning between these floors. Thereby, each stair section will absorb the impact of the adjacent falling walls and prevent a running failure from the top of the stairs at el. 119'-3" down to el. 11'-6", where the pipe is located.

The section of stairs between el. 23'-6" and 51'-3" was judged to be the most susceptible to failure, based on a review of the stair structure details and the geometric dimensions of the stairwell. Consequently, this section of stairs served as the structural model for this evaluation. The results and conclusions are also directly applicable to the three (3) stair sections above el. 51'-3".

# 8.0 DEFINITION OF STAIRWAY PROBLEM

The stair section between el. -19'-6" and 23'-6" is significantly different from the other stair sections, with respect to stair support details and geometric dimensions of the stairwell. This section was evaluated separately.

# 9.0 DEFINITION OF STAIRWELL COLLAPSE SCENARIO

Based on a review of the stair and wall structural details, the stairwell dimensions, and available EBASCO masonry wall calculations, the following wall collapse scenario has been postulated:

1. A wall between floors fails by breaking up into a series of vertical columns (excluding the top two courses of blocks) of one (1) block width; these columns are held together by the vertical reinforcing bars.

2. The falling columns pivot about their base and fall along a circular arc.

3. The columns impact the sides of the stairs and remain intact, in an inclined position against the stairs.

4. The top two courses of blocks, below each floor, become free-falling bodies, which impact the steps and landings immediately below.

5. The north and west masonry walls of the stairwell do not simultaneously impact the stairs: since an actual earthquake has directionality, simultaneous failure of both walls in the direction of the stairs is highly unlikely.

# 10.0 DEFINITION OF REQUIRED ANALYSES

To support the collapse scenario and to preclude gross stair failure, it is necessary to demonstrate the following:

1. The stairs can withstand the horizontal impact of the vertical columns of block.

2. The vertical columns of block are sufficiently strong to remain intact under this same impact.

3. The impact of the free-falling blocks can be absorbed by the steps, landings, and stair stringers without gross failure.

4. The stair supports can withstand the vertical reaction loads resulting from the free-fall impact.

These requirements define the four (4) basic analyses which were conducted.

One additional analysis was conducted. A section of the north wall of the stairwell also serves as the southwall of an adjacent elevator shaft (see Drawing 4509-3). Between els. 23'-6" and 51'-3", this section of the north stairwell wall is designated as Wall 32. The opposite wall (north wall) of the elevator shaft is designated as Wall 45. Wall 45 is of the same construction as the stairwell walls. For an earthquake of strong north-south ground motion, both the stairwell north wall and the elevator shaft north wall are assumed to fail and to fall in the same direction. Therefore, if the stairwell north wall fell towards the stairs, then the elevator shaft north wall would fall across the elevator shaft against the stairwell north wall. This creates a secondary impact on the side of the stairs and potentially could cause the stairwell north wall to break apart. Breakup of the stairwell north wall would generate a secondary free-fall vertical impact on the stairs. An analysis was performed to determine if, under these conditions, Wall 45 would break apart and fall into the elevator shaft before Wall 32 would break apart and fall onto the stairs. Also, the secondary impact against the side of the stairs was evaluated.

# 11.0 METHODOLOGY FOR STAIR ASSESSMENT

The basic approach (Steps 1 - 6) described in Section 5.0 of this report was also applied to the determination of impact response for the stair assessment.

The specific analysis sequence was:

1. A reinforced vertical column of one (1) block width was evaluated to establish that it could absorb the worst case energy of impact against the side of the stairs at the initial point of contact, without gross failure. A maximum ductility ratio of 5 was set as the acceptance criterion for a reinforced concrete column. The analysis was conducted using a simple beam model. The impact energy was calculated from the loss in potential energy of the column, in falling from its initial vertical position to an inclined position against the side of the stairs.

2. From the analysis above, the maximum horizontal load from a single column pushing against the side of the stairs was determined. The stairs were then checked to assure that the total horizontal load from either the north wall or west wall could be safely supported.

3. The steps and landings directly below the top two (2) courses of blocks were evaluated to establish whether they could absorb the energy of impact of the free-falling blocks, without gross failure. The impact distribution for the north wall failure was based on a specific number of blocks striking the upper east landing, west landing, and connecting steps, respectively. The impact energy was calculated based on the free-fall drop heights to these levels. For the west wall failure, only the west landing was assumed to be impacted by free-falling blocks. A ductility ratio $\leq 10$ was considered acceptable. Simple beam models were utilized for this calculation.

## 11.0 METHODOLOGY FOR STAIR ASSESSMENT

4. From the analysis above, maximum vertical loads transmitted to the stair stringers were determined. An ANSYS model of the entire stair stringer assemblage between els. 23'-6" and 51'-3" was developed, to establish the capability of the stairs to carry these maximum loads and to calculate the vertical reaction forces at the stair support points. The reaction forces were utilized to evaluate the stair support details, which were judged to be the "weak link" in the ability of the stairs to survive failure of the masonry walls.

5. The additional analysis to evaluate the secondary impact of Wall 45 aginst Wall 32, was conducted in an analogous fashion to 1. and 2. described above.

# 12.0 DISCUSSION OF ANALYSES AND RESULTS

## 12.1 Evaluation of Reinforced Vertical Column Under Impact

The west stairwell masonry wall between els. 23'-6" and 51'-3" was chosen for a worst case analysis of the capability of a vertically reinforced column of blocks to remain intact after impact against the side of the stairs. This wall was selected because the clearance was large (2.5') and the height of the impact point (14') was significantly below the top of the column (24'). These characteristics lead to a high energy of impact. The impact energy was calculated to be 5,184 in-#. The strength and stiffness of the column was based on a rectangle of material composed of the concrete block material bordering the mortar-filled cell of the block, the mortar, and the #5 rebar located at the middle of th cell. A concrete strength of 2000 psi was assumed. The effective moment of inertia, based on E for concrete, is 25.5 in.$^4$, the maximum elastic moment is 32,600 in-#, and the maximum plastic moment is 60,000 in-#. The concrete controls the elastic moment, while the section is balanced for the plastic moment. The column was analyzed as a beam simply supported at its base and at the impact point, with a 10' overhang above the impact point. The load distribution was assumed maximum at the top and varied linearly to zero at the base. This simulates the distribution of inerital loads in the column, since the velocity is highest at the top and zero at the bottom. The maximum load carrying capacity of the column was defined as the load which induces a moment at the impact point equal to the maximum plastic moment. The linearly varying distributed load which induces this moment is w = 9.7 in-#; the total load is 1394#. The reaction force at the impact point is 1595#.

# 12.0 DISCUSSION OF ANALYSES AND RESULTS

Next, the ability of the column to absorb the impact energy was determined. Conservatively, only strain energy of the cantilevered section of the column from 14' to 24' was considered. The strain energy and average deflection, as a function of w, were calculated, assuming elastic behavior. From these results, a single degree of freedom (SDOF) system was defined, such that the area under its load-deflection curve (the strain energy) would match the strain energy in the cantilever beam at an average displacement equal to the SDOF displacement. The load-deflection curve was taken as bi-linear, rising along the elastic stiffness line until the load magnitude which produces the maximum plastic moment at the impact point is reached. Then the curve becomes horizontal, since no additional load can be carried. The horizontal line is extended to an SDOF deflection (average beam deflection) equal to five times the deflection at the onset of yield. This corresponds to about .3% compressive strain in the concrete, which is the normal accepted upper limit. The total area under this curve, which equates to the strain energy, is 5,283 in-#, compared to the impact energy of 5,184 in-#. Therefore, the column will remain intact under the worst case impact. Conservatisms in the analysis assure that this is the case, despite the apparently small difference between strain energy and impact energy.

## 12.2 Evaluation of Stairs for Side Impact by Masonry Walls

In the previous analysis, a worst case impact load, per column of blocks, was calculated. The load is 1595# per column. Analysis was performed to evaluate the capability of the stairs to withstand these horizontal loads. The 1595# is directly applicable to west wall impacts, since the value resulted from an evaluation of the west wall. West and north wall impacts are not considered to act simultaneously. Also, in terms of sequence of events, the free-fall impact of the top two courses of blocks onto the stairs is assumed to occur before the side impact of the columns.

The west wall impact was evaluated first. The west walls are all about 9' wide, which is 6 blocks in width, yielding a total impact load of 9570# against the west landing of the stairs. This load is reacted by (1) the west landing horizontal support to the south Reactor Building wall; (2) the upper east landing, through the connecting stair stringers, and (3) the lower east landing through the connecting stair stringer. Based on the geometry, about half of the load is reacted by the upper east landing and half by the west landing horizontal support. The 2-3/4 concrete anchors on the west landing support are loaded in shear. Together they can carry at least 5,000# in shear, based on minimum design allowables. The load going to the upper east landing through the stair stringer creates a significant moment at the top of the stairs. The greatest height rise between landings occurs between els. -19'-6" and 23-6" : 7.5'. A moment on the order of 420,000 in-# is generated. This moment can be safely carried by one 10 [15.3 stringer of A-36 or equivalent grade steel. The horizontal force is taken out through bearing aginst the east wall of the Reactor Building. The moment will be reacted by vertical loads on the landing supports and the stair stringers going up from the upper east landing to the next floor. A west wall impact will not cause gross stair failure.

The north wall impact poses a number of problems. First, these walls are much wider (up to 19') than the west walls. Secondly, the immediate impact is taken about the weak bending axis of the closest landing and stair stringers. It was determined that more accurate impact loads, representative of north wall impacts, should be calculated. Based on a review of all north walls between floors, the worst case was judged to be the east most 11' of the wall between els. 75'-3" and 95'-3". The impact energy is 826 #/in. For this wall, the clearance is 12", wall height is 17', and the height of the lowest impact point is 10'. Calculations similar to those described in Section 12.1 were made. However, in this case, the total strain energy of the column was calculated as a function of the magnitude of distributed load (assumed again to be linearly varying). By equating, the impact energy to the strain energy, the load magnitude was defined to be W = 12.3 #/in. About 90% of the impact energy is balanced by elastic strain energy. The associated impact load on the side of the stairs is 1,422 #/column. Below 23'-6", the clearance between wall and stairs is only 4", leading to a very low impact energy of 67 in-#. For the walls above 23'-6", only the first 11' from the east wall have a 12" clearance; the rest of each wall has a clearance on the order of 2-4". As a result, 11' of the wall was used in determining the total impact load of a north wall on the side of the stairs. The total impact load is then 10,427#. This is applicable above 23'-6". This impact load, though more accurate than one based on a west wall impact, is still conservative; this is because the height of the impact point for the columns is assumed constant at the lowest height, when in reality it increases linearly to the height of the upper east landing.

The north wall side impact load is taken out (1) by bearing at the west landing horizontal support to the south Reactor Building wall, and (2) by shear at the upper east landing supports to the east Reactor Building wall. Based on statics, the bearing load on the west landing support is 3349#, and the shear load on the upper east landing supports is 7078#. The bearing load is acceptable. The shear load will be carried by two supports, but not equally. A minimum safe working load is 5,000# for two 3/4" concrete anchors. A small load transfer to the other support should keep the anchors near their minimum safe working load.

The last concern is bending of the stair stringers about their weak axis. To resist the impact load, both stringers which support the steps will have to carry close to an equal share of the total load. Therefore, the steps must be capable of transmitting load from the outside stringer to the inside stringer, after the free-fall block impact. Because of predicted axial shortening of the step as a result of free-fall impact, the outside stringer would be acting alone until the gap is closed. This gap is estimated to be about .6". At yield, assuming A-36 or equivalent steel, the two stringers can support from 80-120% of the predicted impact load, depending on the degree of restraint at the ends of the stair stringers. Considering that the stringers can support some additional load beyond initial yielding and the known conservatism of the predicted impact load, a north wall impact will not cause gross stair failure.

### 12.3 Evaluation of Blocks Falling Onto Stair Steps and Landings

The top two courses of blocks for any stairwell wall are assumed to impact the landings and steps immediately below. For north stairwell walls, six (6) blocks impact the upper east landing, four (4) blocks impact the west landing, and, in the worst case, twelve (12) blocks impact the steps going between these landings. For west stairwell walls, ten (10) blocks impact the west landing. The block impact loads are assumed to be evenly distributed over the area of the landing or step gratings. The impact energy was equated to the loss in potential energy of the blocks.

The approach taken was to demonstrate that grating cross-sectional strength is adequate to absorb the impact energy and keep the blocks from falling through to lower elevations.

The steps and landings were analyzed using simply-supported beam models, based on end support details. The load to produce a plastic hinge at the center was calculated to be 5280# per step and 42,240# per landing. These are considered the maximum loads which are transmitted to the stair stringers. An equivalent SDOF load-deflection model was developed to establish the magnitude of plastic deformation required to balance the impact energy. The highest ductility ratio calculated was 8.3, for a west wall drop onto the west landing. This is below the acceptance criterion of 10. Therefore, the steps and landings were judged acceptable to contain the free-falling blocks from either the north wall or the west wall.

One other concern was whether any steps, although capable of absorbing the impact energy, would be deformed sufficiently to fall through its supports. A check of the axial shortening due to transverse deflection was made. The maximum shortening was calculated to be about 0.6". This is not a problem since there is a 0.75" overlap of the step on its supporting angle at each end of the step, giving a total of 1.5" overlap. These supporting angles and the bolts holding the angles to the stringers are adequate to take the maximum load from the steps.

This evaluation covers stairs between all elevations.

### 12.4 Evaluation of Stairs for Maximum Vertical Impact Loads

The impact loads transmitted to the stairs due to vertical impact of the free-falling blocks onto the steps and landings were calculated in the analysis described above (Section 12.3). The north wall presents the worst case: loads of 42,240# on each of the two landings and 5,280# on each of the steps.

# 12.0 DISCUSSION OF ANALYSES AND RESULTS

The stair support details to the east and south Reactor Building walls were determined to be weaker for the stairs above el. 23'-6" than for the stairs below el. 23'-6". Therefore, the stair section between els. 23'-6" and 51'-3" was used as the model for this evaluation. An ANSYS beam finite element model of the stair stringers and supports was developed to evaluate the vertical impact load case. Separate static analyses of stair self weight, upper east landing impact, steps impact, and west landing impact, were performed. Both simply-supported and fixed boundary conditions at the support points were analyzed. The true condition is somewhere between these, probably closer to simply support due to clip angle flexibility and bolt hole tolerances.

The separate load cases were set up because the maximum effect of the three impacts is not experienced simultaneously. The drop height to the upper east landing is only about 50", while the drop height to the west landing is about 130". Along the steps, it varies between these two values. A more realistic, but still conservative, approach was used. The upper east landing impact and steps impact were assumed to occur simultaneously; then the steps impact and the west landing impact were assumed to occur simultaneously. The largst support reaction force at any support, from these two load cases, was used to evaluate the support details.

Based on this evaluation, certain stair supports are overloaded; support modifications to increase their vertical load capability were designed. These are included as Appendix A to the report. All other structural details are adequate to carry the impact loads without gross failure. Some local plastic deformation will occur. These support modifications are applicable to the four sets of stairs between els. 23'-6", 51'-3", 75'-3", 95'-3", and 119'-3".

## 12.0 DISCUSSION OF ANALYSES AND RESULTS

As mentioned, the supports for the stairs below el. 23'-6" are of a different design. It was determined that even though failure of selected supports might occur, no unacceptable loading of the containment spray system pipe at el. 11'-6" would result. Therefore, no support modifications were identified here.

### 12.5 Effect of Wall 45 Falling Against Wall 32

The secondary impact of Wall 45 against Wall 32 was evaluated using the same procedures as previously described in Sections 12.1 and 12.2. The results indicated that Wall 45 would fail before Wall 32, based on the maximum moment at any cross-section of the vertical reinforced column. Wall 32 can safely support the maximum impact load from Wall 45.

The stairs can support this secondary side impact. However, to preclude any accumulative loss of horizontal load carrying capacity (under the two side impacts) at the upper east landing supports, a bearing support has been added to reduce the horizontal loads on these supports. This support modification is included in the Appendix, and is applicable to all four sets of stairs between els. 23'-6" and 119'-3".

## 13.0 CONCLUSIONS

Based on the analyses performed, the following conclusions have been reached:

1. Vertical reinforced columns of block can withstand impact against the sides of the stairs.

2. The strength of the stairs and supports is adequate to withstand these side impacts.

3. The landings and steps can contain the free-fall of the top two courses of blocks.

4. With the specified support modifications, the stairs and supports are adequate to withstand the vertical impact of the free-falling blocks.

5. The fall of Wall 45 against Wall 32 (and comparable conditions at higher elevations) does not cause any unacceptable damage to Wall 32 or to the stairs. As insurance against horizontal shear failure of upper east landing support anchors, a bearing support has been added.

## 14.0 REFERENCES

1. ASCE Manuals and Reports on Engineering Practice No. 58. Structural Analysis and Design of Nuclear Plant Facilities, American Society of Civil Engineers, 1980.

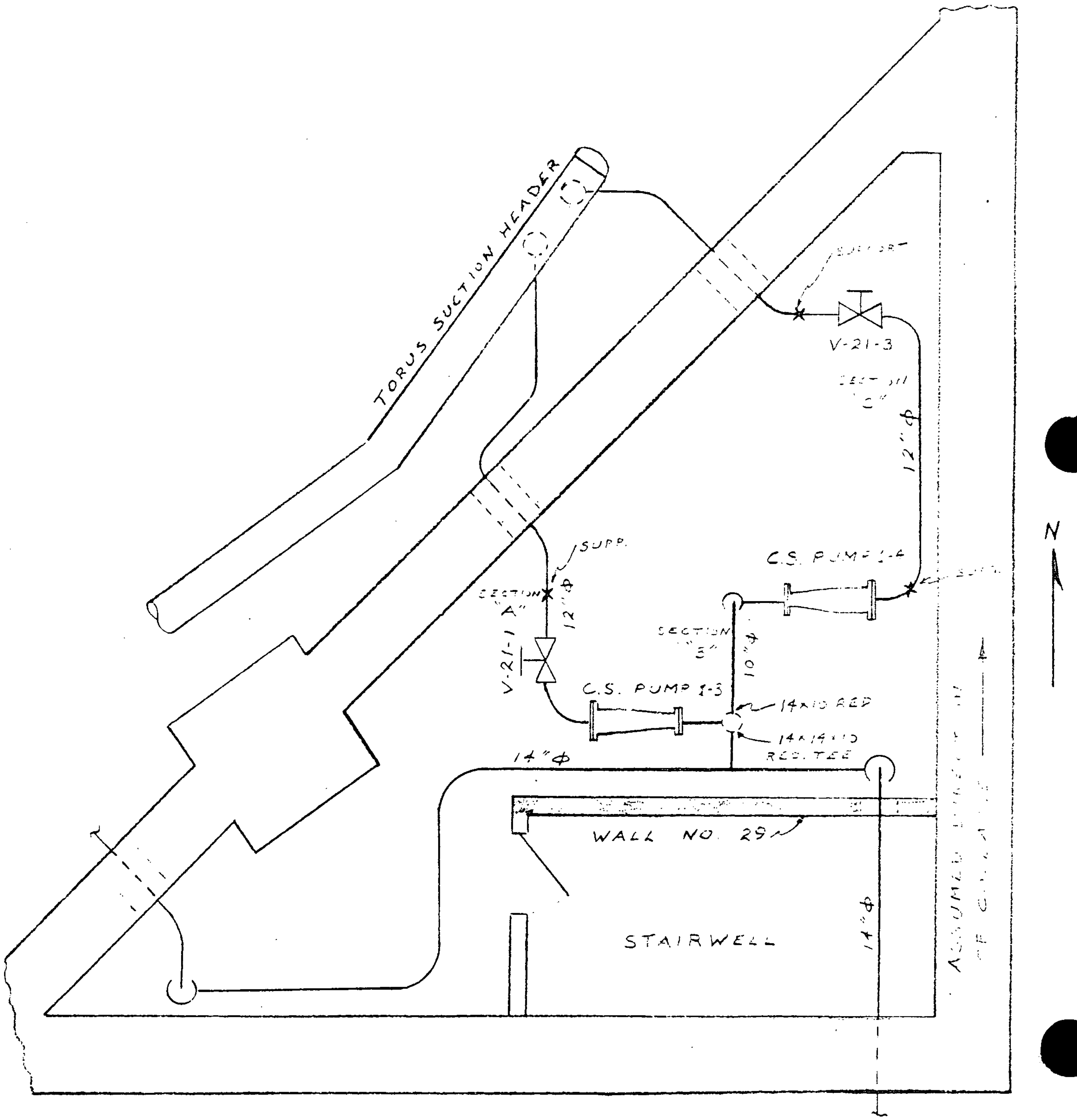
2. J. M. Biggs, "Introduction to Structural Dynamics", McGraw-Hill, 1964.

3. R. A. Williamson and R. R. Alvy, "Impact Effects of Fragments Striking Structural Elements", Holmes and Narver, Revised November, 1973.

4. U.S. Nuclear Regulatory Commission Standard Review Plan 3.5.3, "Barrier Design Procedures", NUREG-0800, Rev. 1, July 1981.

FIGURE 1

SOUTHEAST CORNER OF REACTOR BUILDING
FLOOR ELEVATION -19'-6"



# APPENDIX A

DETAILED CONSTRUCTION SKETCHES

STAIRWAY MODIFICATIONS