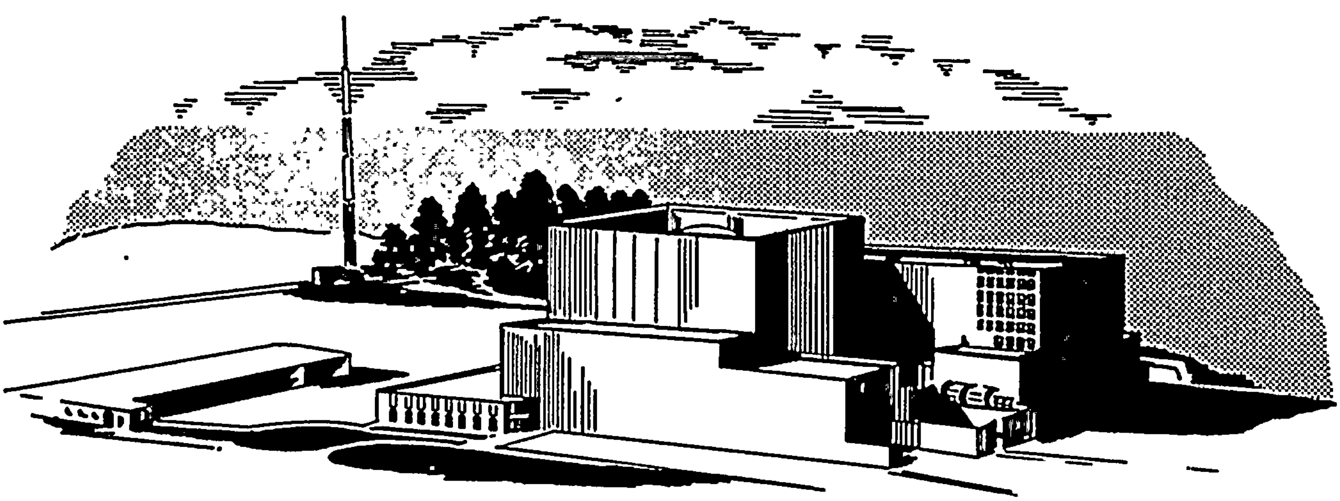
STRUCTURAL REANALYSIS PROGRAM

for the

ROBERT E. GINNA NUCLEAR POWER PLANT.



ROCHESTER GAS AND ELECTRIC CORPORATION

1.0 INTRODUCTION

Since construction of the R. E. Ginna Nuclear Power Plant, changes have been made to the codes, regulations and licensing criteria used in the design and construction of nuclear power plants. In an effort to compare nuclear power plant construction for 11 of the older nuclear plants, including Ginna, to present day criteria, the United States Nuclear Regulatory Commission established the Systematic Evaluation Program (SEP). One hundred thirty-seven (137) topics were addressed in the SEP. Of these, 21 were found to be not applicable to Ginna Station, and 24 were deleted from the review because they were being reviewed generically under either the Unresolved Safety Issues (USI) program or the TMI Action Plan. Of the 92 topics addressed in the Ginna Plant review, 58 were found to meet current NRC criteria or to be acceptable on an equivalent basis. Seven (7) topics were later added to this category as a result of modifications made or committed to by RG&E during the SEP review. The Ginna Station was found not to meet current criteria for all or part of the remaining 27 SEP topics. This report includes Rochester Gas and Electric Corporation's recommendations resulting from studies, calculations, and evaluations that were performed for all areas outside Containment as required by the NRC Staff for the assessment of the following five (5) SEP topics:

- II-2.A - SEVERE WEATHER PHENOMENA
- III-2 - WIND AND TORNADO LOADINGS
- III-4.A - TORNADO MISSILES
- III-7.B - DESIGN CODES, DESIGN CRITERIA, AND LOAD COMBINATIONS
- III-6 - SEISMIC DESIGN CONSIDERATIONS

2.0 SEP TOPIC SUMMARIES

2.1 SEP Topic II-2.A - Severe Weather Phenomena

Data for extreme meteorological conditions and severe weather phenomena were applied at the Ginna site to determine if safety-related structures, systems, and components were capable of functioning under severe weather conditions. This topic contains only that load combination which includes extreme snow loads. The value of the extreme snow load has been defined by the NRC in Reference 1 as the maximum design snow fall plus the 48-hour probable maximum winter precipitation. This results in a value of 50 lbs/ft^2 (snow) plus 50 lbs/ft^2 (accumulation) = 100 lbs/ft^2.

2.2 SEP Topic III-2 - Wind and Tornado Loadings

Based on a study of actual tornado occurrences in the site region, site-specific tornados and associated probabilities of recurrence were calculated by the NRC in Reference 2 and verified in Reference 3. For the Ginna site, winds of 132 mph, 188 mph and 250 mph were found to have associated probabilities of recurrence of 10^{-5}, 10^{-6}, and 10^{-7}, respectively. These tornado wind speeds are based on the upper 95 percentile value which provides conservatism to compensate for uncertainties in the analysis. The NRC Safety Evaluation Report on this topic indicates that the plant design parameters will be compared to the probability of recurrence of the tornado wind and the comparison will be used to evaluate the necessity of changes in the plant.

2.3 SEP Topic III-4.A - Tornado Missiles

Parts 2 and 4 of the General Design Criteria Appendix A, 10CFR Part 50, require that structures, systems, and components important to safety be designed or protected from missiles originating from events and conditions outside the plant. Missiles consist of any object or debris blown before tornado winds. In the event of a tornado, the plant structures must provide sufficient protection from the hazards of missiles in order to assure:

- o the integrity of the reactor coolant pressure boundary,
- o the capability to shut the reactor down and maintain it in a safe shutdown condition, and
- o the capability to prevent accidents which could result in offsite exposures in excess of the dose guidelines of 10CFR Part 100.

In Reference 4, the NRC recommended that one "train" of safe shutdown equipment be protected from damage due to the occurrence of two postulated missiles: a 35 foot, 1490 pound utility pole, and a 1 inch diameter, 8 pound steel rod.

2.4 SEP Topic III-7.B - Design Codes, Design Criteria, and Load Combinations

SEP plants are those generally designed from the late 1950's to late 1960's. These plants were designed according to criteria and codes which differ from those currently used in the design of new power plants. A general review of the present day design codes versus the design codes employed in the design of Ginna Station was done by the Franklin Research Center, on contract

to the NRC. This was done on a code versus code basis without specifically investigating how the codes were applied to the design of Ginna. After reviewing the drawings of the structures at Ginna, the NRC concluded that certain types of structures to which the codes referred were not included in the Ginna design. A number of other code changes were considered to potentially impact the margins of safety of certain plant structures due to the following:

- o new codes have imposed stricter limitations than old,
- o new codes have included sections governing design of certain types of structures which were not included in the older codes,
- o design loads specified in current codes were not included in previous codes, and
- o certain load combinations currently judged to be significant were not included in previous codes.

The NRC requested (Reference 5) that these code changes be addressed by RG&E.

A review of the design and fabrication drawings for Ginna Station was made in order to determine if the new code changes potentially impact the margins of safety of certain plant structures. Based on this review, certain construction details of the structures at Ginna Station did not meet current design code requirements. These areas were included with the Wind and Tornado Loading portion of the evaluation, and checked to see if they provided sufficient margins of safety.

2.5 SEP Topic III-6 - Seismic Considerations

An evaluation of Ginna Station was made by Lawrence Livermore Laboratories (Reference 6) on contract to the NRC in order to check the adequacy of the plant structures to resist the forces from a design basis earthquake. These structures must be capable of withstanding such forces and still provide protection of systems and components necessary to achieve and maintain the plant in a safe shutdown condition.

Based on this evaluation, the NRC concluded that the structures at Ginna Station were found to provide a sufficient margin of safety under the design basis earthquake except for two relatively small modifications. These modifications have been committed to by Rochester Gas and Electric Corporation, in Reference 7 and 8, and

will be incorporated into the plant design as part of any resultant modifications from the Wind and Tornado Loadings section of the analysis. No further seismic considerations are included in this evaluation.

3.0 EVALUATION

3.1 General Discussion

The Standard Review Plan (SRP) Sections 3.3.1 and 3.3.2 and Regulatory Guides 1.76 and 1.117 include guidance relative to the need for nuclear power plants to withstand the effects of natural phenomena such as wind and tornadoes. At the time of design and construction of Ginna Station, the design criteria for nuclear power plants did not include tornadoes and other phenomena such as extreme snow and tornado missiles to the extent currently required. Consequently, the existing design and construction of some structures important to safety may not meet current licensing criteria but are, nonetheless, capable of resisting loads to some level between current criteria and those specified in the FSAR.

The purpose of this evaluation is to determine the level of protection (tornado windspeed characteristics) which should be used as an appropriate backfitting basis for Ginna. In order to make this judgment, RG&E used a three-step process:

1. Determine the capability of the present Ginna structures, systems, and components to withstand tornado effects.

2. Determine the costs associated with backfitting tornado protection, at several windspeeds up to that specified in current criteria.

3. Define a reasonable level of tornado protection, based both on the costs associated with a range of tornado windspeed protection levels, and on the range of probabilities of these tornado windspeeds.

3.1.1 Reanalysis Approach

RG&E has consolidated all four SEP Topics (II-2.A, III-2, III-4.A and III-7.B) into this Structural Reanalysis Program. This program, to the extent practical, has been coordinated with other SEP topics such as III-6, "Seismic Design Considerations", III-5.B, "Pipe Break Outside Containment", as well as modifications contemplated to respond to Appendix R to 10CFR Part 50.

In the initial phase of this project, it was necessary to develop a logical approach in which this evaluation

would proceed. The following steps provided the basis for the initial evaluation:

1. define loads, load combinations, and initial acceptance criteria,

2. define assumptions,

3. evaluate the effects on the structure,

4. compare these effects to the original assumptions,

5. assess these effects as they pertain to plant shutdown,

6. estimate the costs associated with the repairs, and

7. based on the cost and effects, recommend final input and acceptance criteria and the recommended degree of repair.

The evaluation was performed in two parts. First, a structural evaluation was performed to determine the capabilities of all plant structures to resist wind, snow, and tornado wind and pressures. Second, a determination was made of the minimum set of plant equipment required to bring the plant to a safe shutdown condition and the impact of postulated tornado missiles on that capability. Backfit costs were estimated in both evaluations and were then combined in a consistent fashion to provide a uniform level of protection for all phenomena.

3.1.2 Structural Evaluation Approach

In order to perform a structural evaluation of this complexity, a complete evaluation of the main plant structures was made. This evaluation examined the interactions of the structures in the Auxiliary, Intermediate, Turbine/Diesel Generator and Control Buildings and the Facade Structure in order to distribute the loads throughout the entire structure in a manner that best simulates the actual field conditions. A separate evaluation was performed for the Screen House. (See Figure 3-1 for a general arrangement of the Ginna facility.) The computer program GTSTRUDL was used for the structural evaluation.

GTSTRUDL is a computer aided structural engineering software system developed, maintained and continuously researched at the GTICES Systems Laboratory, School of Civil Engineering, Georgia Institute of Technology. Section 2.0 of Attachment A provides a detailed description of this program and its use in this evaluation.

3.2 Input and Acceptance Criteria Used in Determining the Plant Capabilities

3.2.1 General

The initial input and acceptance criteria used during the analysis of the Ginna structures were taken from the various SEP topic Safety Evaluation Reports, which reference appropriate "current criteria" as defined in the regulations, Regulatory Guides, Standard Review Plan and ANSI standards.

3.2.2 Input Load Criteria

Before the actual evaluation could be made, structural layout and load data were compiled. Plan and elevation drawings of only the primary members and cross-bracing were made. These drawings were reviewed in the field and checked to confirm that the member configuration and location on the drawings agreed with the field conditions. Member sizes were checked randomly to verify that the member sizes in the field conform to what was shown on the drawings.

The plant drawings were reviewed to determine the service and live loads on each floor. A field verification was done for the whole plant, whereby typical floor bays were examined, the equipment on these floor bays were located and an estimated service load calculated. The estimated service loads also included the weights of pipes, cable trays and conduits which are attached to the floors.

Dead loads were assumed to be the weights of the structure, fixed equipment, an allowance for permanently attached system components (e.g. pipe, duct and cable trays), and an allowance for thermal effects and pipe reactions. Live loads were assumed to be as specified in the FSAR and on the drawings, minus whatever was allowed for permanently attached system components. Dead, live, thermal effects and pipe reaction loads were applied as equivalent uniform loads where applicable through the slabs or decking into the main framing.

A 75 mph wind speed and 40 psf ground snow load were used as the severe environmental loading condition. These values are specified in the SER[2] for Topic III-2 and are stated to correspond to a 10^{-2} design wind load and a 10^{-2} design snow load.

An extreme snow load of 100 psf was used as a basis for this evaluation. This value is specified in the SER for SEP Topic II-2.A.

The effects of the two NRC design basis tornado missiles on equipment required for safe shutdown were also examined. The missiles, a 35 foot utility pole and a one inch diameter steel rod, were examined to determine the effect a missile strike would have on the equipment required to safely shutdown the plant. The two missiles (pole and rod) were assumed to travel at a speed of 0.4 and 0.6 times the tornado wind speed, respectively.

A spectrum of tornado wind speeds were chosen from the "Tornado and Straight Wind Hazard Probability" report prepared by Texas Tech. University (Reference 2). Wind speeds of 250 mph, 188 mph, and 132 mph were used. These wind speeds coincide with the Texas Tech. estimates for a probability of recurrence of 1×10^{-7}, 1×10^{-6}, and 1×10^{-5} per year, respectively, at an upper 95% confidence level and have been confirmed in Reference 3.

The wind speeds were converted into design pressures by utilizing the ANSI 58.1-1982 equation:

$$p = q\ G_h\ Cp$$

where $q = 0.00256\ K_z\ (IV)^2$

K_z = velocity pressure coefficient

I = importance factor

V = fastest - mile wind speed

G_h = gust response factor

Cp = external pressure coefficient

Differential pressures were calculated by using $q = 0.00512\ V^2$ where V represents the translational wind speed. Wind loads were applied uniformly to the plant structures as specified in the SRP.

3.2.3 General Assumptions

Once the three tornado wind speeds were converted to design pressures, the following assumptions were made prior to applying these pressures to the structures:

- metal siding and roof decking remain intact and attached to the main steel frame for all load conditions,
- all external block walls remain intact for all load conditions,

- plant windows, louvers and doors remain intact for all load conditions.

These assumptions maximize the loads transferred into the structures. From these assumptions, the wind and snow load combinations were then applied to the structures as uniform loads. Their influence was transferred to the main steel framing through the siding or decking.

In the evaluation, the columns were input with their orientation corresponding with the field condition. The columns were assumed to be braced against lateral buckling by floor beams or struts which are framed into the column centerlines. Columns on the building perimeter that have girts attached to their flanges were assumed not to be laterally braced by the girts against buckling on the columns subjected to axial loads. The effective lengths were usually considered to be the distance between floors in the plant for both the strong and weak axis under column buckling and lateral buckling due to beam action. Column bases were typically modeled as "pinned" connections (non-moment resisting). Floor beams were assumed to be laterally braced for bending by the floor slabs and beam to column connections were generally modeled as "simple" pin type connections.

Girts and purlins were considered to be secondary members in this evaluation. For positive wind pressure, the outside flange of the girt is in compression. Under this condition, the siding was assumed to provide full lateral support along the compression flange. However, negative wind or differential pressures reverse the compression flange to the inside of the girt or purlin. For this type of loading, the unbraced length of the compression flange was assumed to be the distance between supports.

The typical connection at the Ginna Station is a bolted connection. Beam to column connections, in general, consist of angles welded to the beam and bolted to the column. Connections in the trusses or cross bracing consist of members bolted to gusset plates. The connection evaluation was done in accordance with the guidelines of AISC and using basic statics and engineering mechanics.

Column anchorages were evaluated for basic shear and/or tension loads within the guidelines of ACI 349 Appendix B. (Reference 9)

3.2.4 Load Combinations and Acceptance Criteria

Load combinations for severe, extreme and tornado loadings were evaluated, consistent with the NRC Standard

Review Plan [Reference 11]. The following load combinations were considered in this evaluation:

1. D + L + Sn + W

2. D + L + S'n

3. D + L + W_t

where:

D = Dead Load

L = Live Load

Sn = 100 year recurrence snow = 34 psf roof load for the power block, and 27 psf for the screen house

W = 100 year recurrence wind = 75 mph for all structures

S'n = Extreme snow = 100 psf

W_t = Tornado wind loads as defined below, and corresponding to 250, 188 and 132 mph tornado wind speeds.

Wt = Ww or,

Wt = Wp or,

Wt = Ww + .5 Wp

Ww = tornado wind load

Wp = tornado differential pressure load

These load combinations have been broken into three categories. Load combination 1 is referred to as severe, load combination 2 is referred to as extreme while load combination 3 is referred to as tornado.

Since a probability of occurrence for all these load combinations is considered to be very low, a 1.6 S (1.6 multiplied by the allowable stress limit of the steel) acceptance criteria was used for the initial analysis.

3.3 Structural Evaluation

The structural evaluation combined the use of GTSTRUDL with hand calculations in order to accurately analyze the structural capacities of the primary members, secondary members, connections and anchorages, and building shell. The main structural framework was analyzed using the GTSTRUDL computer program in order to determine the forces and moments in the members for

each load combination. GTSTRUDL was also used to calculate the structural adequacy of the secondary members under the same loading conditions used in the primary member evaluation, only on a representative sampling basis. The end reactions found in the primary member evaluations were used to evaluate the connections and anchorages in the plant using a statistical sampling technique. [Refer to Attachment A for an explanation of the statistical approach used].

3.3.1 Primary Member Evaluation

The analysis was performed using the computer program GTSTRUDL. Two three-dimensional structural computer models of the plant were developed. One model addressed only the screen house, which is separate from the main plant, while the other model consisted of the Auxiliary, Turbine/Diesel Generator, Intermediate, Control and Facade Structures. The models were developed by establishing a global coordinate system whereby only the main steel structures were described.

The models consist of columns, beams, cross-bracing, roof trusses, and other framing components of the structure that contribute to the horizontal strength of the plant. Main interior floor framing, adjacent buildings, and secondary components had their load influence input, but were not discretely addressed. Concrete floor and roof slabs (or decking) were assumed to be plate elements in the horizontal plane, and were not developed in detail.

The plant structures were then analyzed for the load cases previously mentioned.

A software feature of the GTSTRUDL program is a means by which the resultant loads can be changed into stresses and checked to the American Institute of Steel Construction (AISC) Code. This procedure is done by assigning a number to each member in the computer model and inputting their respective properties (area, section modulus, radius of gyration, etc.). In the analysis, each primary member was checked in accordance with the Eighth Edition of the AISC code. The members which passed or failed the code check were listed, as well as a listing of the load combination which resulted in the overstressed condition.

3.3.2 Secondary Member Evaluation

Secondary members are those members whose purpose is to transfer the load from the intermediate areas of the roof and walls to the primary framing. These members consist of roof purlins and girts. The analysis was

performed using GTSTRUDL in a similar manner as done in the primary members evaluation; however, a representative sample of the girts and purlins was investigated instead of inputting each individual member. A sample size of 70 purlins and girts were checked with the AISC code. This representative sample addresses 95% of all the roof purlins and girts in the plant. The percentage of failures discovered in this evaluation was extrapolated to provide the number of failures expected for the 1100 actual purlins and girts.

3.3.3 Connections and Anchorages

The results of the primary and secondary member analyses were used to check the adequacy of the beam to beam, column to beam, column to base plate, and anchor bolts to base plate connectors, or simply, connections and anchorages. Since the plant contains approximately 6000 connections and 220 anchorages, a statistical approach was chosen in the review of these elements as described in Attachment A.

A statistical sample of sixty different connections was chosen and their associated axial and/or horizontal loads were applied and analyzed. Hand calculations and computer programs were used to check the strength of the bolts, welds and clip angles for the applied loads. The resultant stresses were checked with the allowable stresses specified in the Eighth Edition of the AISC code. For those load conditions not addressed in the code (horizontal and axial load occurring simultaneously), engineering mechanics were used to determine the adequacy of the connections. The results of this evaluation provided a percentage of overstressed connections which could be expected at a 95% confidence level. By multiplying this percentage by the actual number of connections in the plant, an expected number of the connections that would not satisfy the acceptance criteria was determined.

A statistical sample of fifty-three anchorages in the plant was also chosen and evaluated using their associated loadings. A percentage of expected overstressed anchorages was found and multiplied by the total number of anchorages in the plant to determine the expected number of overstressed anchorages.

3.3.4 Exterior Shell

3.3.4.1 Siding

Throughout the reanalysis program it was assumed that the siding would remain intact for all wind speeds. By making such an assumption, the load distribution was transferred evenly across all the steel framework, thus

maximizing the load on the framework, while removing the effects of the wind pressure directly on the internal walls and equipment. To verify this assumption Pittsburg Testing Laboratory (PTL) performed pressure tests on three types of siding at Ginna Station. These three types of siding, as manufactured by Elwin G. Smith Corporation, are:

1. ribwall
2. shadowall, and
3. "B" panel system.

The ribwall panel system is located on the middle portion of the four sides of the facade structure while the corners of the facade consist of the shadowall panels. The rest of the plant is covered by the "B" panel system. A total of six tests were performed on each panel system. The six tests consisted of three positive and three negative pressure loadings. The positive tests represented a wind load from the outside of the structure while the negative tests represented pressure from the inside of the structure or a suction from the outside. The tests checked the failure load of the panels and the fasteners. Failure was defined as loss of function resulting from tearing of the siding or failure of any or all the panel connectors. Once the siding pressure capacities were determined, calculations were done to determine the corresponding wind speed for various areas of the buildings.

The results of the tests are discussed in Section 3.4 and are explained in more detail in Attachment A and the PTL test report in Attachment D.

3.3.4.2 Concrete Masonry Block Walls

The Auxiliary, Intermediate, Control and Turbine Buildings contain concrete masonry block walls. The interior block walls (building partitions) were assumed to contribute only their dead weight to the structure in the evaluation. No structural stiffness was considered.

For the purposes of this analysis, the exterior block walls were assumed to remain intact, contributing only their dead load, and were assumed to transfer the tornado wind loads into the steel structure. Based on this assumption, the interior walls would not be subject to tornado winds.

All of the exterior block walls at Ginna are constructed of unreinforced concrete masonry units. As part of the Reanalysis Program, these walls were investigated to determine their capacities at various wind speeds.

For the analysis, it was assumed that each wall panel had adequate boundary supports to resist lateral translation. The acceptance criteria used were "inspected" values of ACI-531-79 with allowable overstresses consistent with an extreme environmental event. In addition, further investigations were made into state-of-the-art analysis and methodology for evaluating unreinforced masonry walls. Test results and engineering mechanics considerations were used to develop a "stability criteria." "Stability criteria" can be explained as the capacity to resist further levels of loading after cracking has occurred. Factors that influence the stability of walls were incorporated into this analytical technique, i.e., height to thickness ratio, self weight, overburden weight and support stiffness.

Each exterior wall was classified by its physical dimensions and thickness into a group. These group classifications were then evaluated to see what direct wind speed they would resist using conventional design and evaluation criteria for unreinforced masonry walls based on allowable tensile stresses. Each wall was also evaluated to see whether or not "stability criteria" would be applicable for that particular wall.

The results of these analyses are provided in Section 3.4.4.3 of this report.

3.3.4.3 Architectural Items

Architectural items include doors, windows, and louvers. These items will be upgraded to withstand the effects of the design basis tornado windspeed, if required, to be consistent with the other components of the exterior shell.

3.4 Results of the Structural Evaluation

This section presents a summary of the results of the analysis and discusses overstresses and failures in terms of number of members, general failure mode and failure location for the various components of the structures. In terms of this report, "failure" does not mean collapse of a member or a mechanism but instead will mean, the inability of such a component to meet the recommended acceptance criteria. The results are presented based on the five load combinations listed below as compared to the acceptance criteria discussed in Section 3.2.

1. Severe environmental (D+L+Sn+W)
2. Extreme snow (D+L+S'n)
3. Tornado Winds (132 mph)
4. Tornado Winds (188 mph)
5. Tornado Winds (250 mph)

3.4.1 Primary Members

The evaluation of results of the various loading conditions on primary members was based upon the number of computer members in models 1 and 2 rather than actual structural members. The number of failures shown are generally higher than the actual number of member failures. This is especially true for columns where one structural member may be represented by several computer members, depending on the location of the bracing and struts. Model 1 contained 3500 computer members while Model 2 contained 766 computer members. In the discussion below, the turbine building also includes the control building and the diesel generator rooms. Table 3-1 provides a summary of the primary member failures for each building as well as a description of the failures. The numbers shown are accumulative and indicate the total number of failures for all load cases considered rather than an incremental amount of failures for each specific load case.

For severe environmental conditions, 168 primary members failed the acceptance criteria. Approximately 50% of all the failures are in the turbine building. The majority of the rest are about equally spread between the intermediate/facade building and auxiliary building with only about 5% in the screenhouse. For this loading case about one quarter of the failures are beams overstressed in bending from snow loads combined with axial wind loads. The remaining failures are about equally spread between column and bracing elements. Many of these failures, particularly for bracing, are not due to overstress but due to excessive kl/r values for compression members as allowed by codes.

One hundred and forty-one members failed the extreme snow load condition. Ninety-eight of these also failed severe loads, resulting in an additional 43 or a total of 211 failed members. About 50% of the additional failures occurred in the turbine building and about 25% each in the auxiliary and intermediate/facade area. Most of the additional failures were roof bracing members and roof truss members.

A total of 258 members failed the acceptance criteria for a 132 mph tornado including differential pressure effects. One hundred and seventy of these members had failed the severe and/or extreme environmental effects. An additional 88 failed members due to tornado wind only. Seventy percent of the additional members were in the turbine building and consisted primarily of cross bracing elements and various chord members of the roof trusses. Minor failures (about 15%) occur in the beams and bracing of the screenhouse at 132 mph. The

remaining 15% are miscellaneous additional members in the auxiliary and intermediate/facade building. Approximately 36% of the 88 members failed were the direct result of the differential pressure loadings.

Of the 299 members that failed load combinations 1 through 3, slightly more than 54% are in the turbine building, about 21% are in the auxiliary building, 18% are in the intermediate building/facade, and 7% are in the screen house.

A total of 332 members failed the acceptance criteria for a 188 mph tornado. This number included differential pressure failures which were projected using the 132 mph results. Similar to the 132 mph tornado, 177 of these members failed the severe and/or extreme environmental effects resulting in an additional 155 failed members caused by the 188 mph tornado alone. The percentage of the 155 failed members were distributed as follows: 20% for the combined auxiliary, intermediate, and facade structure; 55% turbine building and 25% screenhouse. Differentiating between a 132 mph tornado and a 188 mph tornado (67 additional members fail from 132 mph to 188 mph) the increased failures in the turbine building were 60% bracing, 40% columns; in the screenhouse, 75% roof trusses, and 25% bracing. The 20% of member failures located in all other buildings were found distributed evenly as beams and trusses. Approximately 38% of the 155 members failed were the direct result of the differential pressure loadings.

Of the 366 total members that failed the load combinations 1 through 4, slightly less than 52% are in the turbine building, about 18% are in the auxiliary building, 18% are in the intermediate building/facade, and 12% are in the screenhouse.

A total of 658 primary members failed the acceptance criteria, including differential pressure failures, for the 250 mph tornado. As in the two previous tornado wind applications, 178 of these members failed the severe and/or extreme environmental effects. Thus, for the 250 mph tornado wind, 480 failures were due to tornado wind alone. Of these 480 failures, 325 failures occur as a result of the 250 mph tornado loading over and above those found due to the 188 mph tornado results. Of the 325 additional failures, 22% were in the turbine building, 38% in the screenhouse, 16% in the facade structure, 15% in the auxiliary and 8% in the intermediate building. The majority of failures were bracing members, 32% of the 325, with 28% of the total being columns. The screenhouse roof truss system contributed 25% of the total by itself and the remaining 15% were composed of beams and other truss members. Approximately 21% of

the 480 members failed were the direct result of the differential pressure loadings.

Of the 691 total members that failed the load combinations 1 through 5, 38% are in the turbine building, 17% are in the auxiliary building, about 21% are in the intermediate building/facade, and 24% are in the screenhouse. A tabular breakdown by building and member type for failures caused by load combinations 1 through 5 is shown in Table 3-1.

3.4.2 Secondary Members

For the extreme snow load of 100 psf a few (21) isolated roof purlins became overstressed.

At a 132 mph tornado loading approximately 60% of the total girts and purlins did not meet the acceptance criteria. These members were not considered to detach themselves from the main frame but experience high stress levels and possible permanent deformations. The problems experienced by these members are due to tornado loads that create suction effects, and loads due to the differential pressure. For these load conditions, the bending stress allowables are low because of the large unbraced length of the compression flange.

When subjected to a 188 mph tornado, 77% of all secondary members experience overload.

At a 250 mph tornado loading, 94% of the secondary members are overloaded and they would fail by bending or by failure of their connections to the main frame.

3.4.3 Connections and Anchorages

As described in Section 3.3.3, the connections and anchorages were statistically sampled and then evaluated for the various load combinations.

The results for the connection analysis showed that 11% to 13% of the connections failed the acceptance criteria for the severe environmental, extreme snow and the 132 mph tornado loading conditions. As the tornado wind speeds increased the total percentages of failed connections went to 23% and 39% for the 188 mph and the 250 mph tornado loadings, respectively.

No anchorages failed under the extreme snow loading based on the downward loading direction. For anchorages under the severe environmental loading and the 132 mph tornado loading 18% failed in one of the three conditions checked for anchorage capacity: anchor bolts, welds to

base plates, or concrete capacity. This number increased to 50% and 75% for the increased tornado loadings of 188 mph and 250 mph respectively.

3.4.4 Exterior Shell

3.4.4.1 Metal Siding

The results of the siding tests determined the ultimate failure loadings. These results were then correlated to locations on the various buildings at Ginna. It was determined that with minor modifications all the exterior siding would perform its function under a 132 mph tornado loading. As the tornado loading increased to 188 mph all of the screen house siding failed. Twenty-eight percent of the total siding in the auxiliary building and the intermediate building failed. Twenty-three percent of the turbine building siding failed and approximately 50% of facade siding failed. When the 250 mph tornado loading results were calculated 100% of the siding failed.

The above results considered various regions of the plant as controlling failure areas due to the application of ANSI A58.1 pressure coefficients. It can be seen in Attachment A that the main plant siding itself, the "B panel system" has wind capacities of 221 mph and 176 mph for positive and negative pressure loadings respectively.

3.4.4.2 Roof Decking

The roof decking is acceptable for the extreme snow condition except for a few isolated spans. For a 132 mph tornado the theoretical calculations show that the roof decking itself is capable of supporting loads associated with this tornado. However, the decking to purlin connection might not be able to resist the uplift loads. As the tornado wind speeds are increased to 188 mph and 250 mph the portions of roof decking predicted to fail are 41% and 100%, respectively.

3.4.4.3 Block Walls

Of the 66 exterior block walls, 11 are capable of resisting a 132 mph tornado. An additional 22 potentially qualify for 132 mph tornado with the inclusion of static stability factors such as overburden and support stiffness. The remaining one half of the walls are not capable of resisting a 132 mph tornado without modifications and/or removal. As the tornado wind loadings are increased to 188 mph and 250 mph, 75% and 100% of the exterior block wall will have failed.

3.5 Tornado Missiles and Safe Shutdown Approach

3.5.1 Background

In the NRC's April 16, 1982 Safety Evaluation Report relative to this SEP topic, it was determined that the majority of plant structures, systems, and components required to ensure:

1. the integrity of the reactor coolant pressure boundary,

2. the capability to shutdown the reactor and maintain it in a safe shutdown condition, and

3. the capability to prevent accidents which could result in unacceptable offsite exposures,

were suitably protected from postulated tornado-generated missiles.

Several items were identified, however, which required additional evaluation with respect to tornado missile protection. An evaluation of these "open" issues, as identified in the Ginna Integrated Plant Safety Assessment Report, NUREG-0821, May 1982 (draft) and December 1982 (final), as well as a number of other items identified during RG&E's subsequent reviews, are provided in Section 3.5.3 below.

The two missiles required in the SER to be evaluated were a steel rod, 1-inch diameter and 3-feet long, weighing 8 pounds, and a wooden utility pole, 13.5 inch diameter and 35-feet long, weighing 1490 pounds. The velocity of the steel rod was assumed to be 60% of the tornado wind speed; the velocity of the wooden utility pole 40%.

3.5.2 Shutdown Methodology

RG&E has developed methods to achieve and maintain safe shutdown conditions, following the postulated tornado strike. Certain assumptions of plant status and system unavailability are made.

3.5.2.1 Assumptions

a. Offsite a.c. power is lost.

b. All equipment not protected from tornado effects is considered inoperable unless detailed in 3.5.3 below. Also, if protection is not specifically provided, it is assumed that inadvertent operation due to ground or phase faults could occur.

c. Architectural details such as the building shell components and secondary members, are not considered capable of withstanding tornado windspeeds; however, the failure mode of these items is such that they will not become damaging missiles.

3.5.2.2 Shutdown Details

One train of safeguards equipment, which will serve to provide and maintain safe hot shutdown, will be protected. Due to the nature and methodology of the shutdown systems being protected, cold shutdown can also be achieved.

The safe shutdown function will be performed as follows:

a. The reactor will automatically trip as a result of the loss of the non-protected 4 Kv buses or other trip signal.

b. The turbine would trip, with resultant closure of the turbine stop valves. The operator would also close the MSIVs from the control room, if they did not automatically close.

c. The diesel generators would automatically start, and pick up the required loads. For purposes of this shutdown method, it is assumed that diesel generator 1B will be tornado protected. This would allow operation of all safeguards equipment associated with bus 16 (train "B").

   Since Service Water is not protected, the diesel might not have this source of cooling water. Modifications have been made to the diesel cooling system to permit alternate water supplies to be used. Several sources of water as listed in 3.5.3g are presently being evaluated to provide this cooling water.

d. The Standby AFW system would provide cooling to the steam generator(s). By use of one of the main steam safety valves, a safe hot shutdown condition would be established. A 10,000 gallon tank is available in the SBAFW Building which is used for SBAFW pump testing. The tank normally is maintained nearly full of water. Following use of the contents of that tank, additional AFW could be provided from the yard fire loop, a portable gasoline-driven pump, or a fire truck pumper which could be called on-site.

e. Charging flow for inventory makeup to provide for RCS heat sink would be available via the charging system. This function is presently tornado-protected.

f. In order to cool down, use of the atmospheric dump valves on the main steam header would be required. If the air or the backup nitrogen systems that control these valves could not be made operable, because they are not tornado-protected, these valves could be locally controlled.

g. To effect final cold shutdown, RG&E could make use of the steam generators as water-to-water heat exchangers. Using established procedures, the operators would fill up the steam generators, and in an orderly manner, achieve a cold shutdown condition to less than 200°F. It is contemplated that this cooldown would occur over several days.

3.5.3 Required Components

The structures, systems, and components required to be tornado-missile protected are those required to achieve and maintain a safe shutdown condition. Also, other systems considered for protection are the surface of the spent fuel pool, such that missiles and other large items would be prevented from causing unacceptable damage to the fuel assemblies; the reactor coolant pressure boundary and main steam and feedwater lines, to prevent major primary and secondary system breaks; and items which failure could cause unacceptable inadvertent operation or failure of safety-related equipment.

RG&E's proposed resolution of these items is detailed below.

a. <u>Refueling Water Storage Tank</u>

An analysis of the ability of the RWST to withstand tornado missiles has been completed. Based on calculations using both the Ballistic Research Laboratory and Stanford Research Institute equations for missile penetration, it was concluded that the steel rod would not perforate the RWST shell for tornado speeds up to 166 mph. A calculation was also performed of the effect of the utility pole impact on the tank, at tornado wind speeds of 132 and 150 miles per hour. The maximum structural response was found to be within the tank material's allowable ductility. Thus, no buckling would occur. RG&E concludes that for the analyzed tornado missile speeds, no flooding of safety-related equipment due to tank failure would be expected.

The Reactor Makeup Water Tank is also located on the operating floor of the Auxiliary Building. Identical calculations were made for this tank, and identical conclusions were reached.

b. Electrical Buses 14, 17, and 18

Bus 14 is located on the operating floor of the Auxiliary Building and could be subject to damage from tornado missiles. Originally, as part of the Fire Protection modifications being considered by RG&E, a protected bus 14A was to be installed. This latter modification is no longer planned. However, safety-related bus 16, located on the intermediate level of the auxiliary building, is protected from tornado missiles, and would be available in the event of a tornado.

Buses 17 and 18 are located in the screen house. The operating floor of the screen house is not protected from the effects of tornadoes, including missiles. However, RG&E has made modifications which will eliminate dependence on the Service Water System to achieve and maintain safe plant shutdown. Thus, no protection for buses 17 and 18 is required.

RG&E has also investigated the potential for damage to buses 17 or 18 causing failure of required electrical equipment, such as a diesel generator. In order to eliminate the potential damage from fault currents, RG&E is planning to install a protective breaker for diesel generator 1B, to be located in a tornado missile protected area.

c. Main Steam Lines A and B, and Main Feedwater Lines A and B

An analysis of the effects of the tornado missiles on the steam lines, feedwater lines, supports, and attached piping and valves has been completed.

Results - steel rod: The main steam line, main feedwater line, as well as attached piping and valves, are all thick-walled items, and would not be perforated by the steel rod impact. Damage to valve operators could prevent subsequent operation; however, no loss of pressure integrity would result. Thus, secondary system integrity would be maintained. The effect of damage to piping supports was also investigated. It was determined that damage could occur causing possible loss of support. However, damage to one support member would not

result in a loss of overall support to the piping system. Thus, the main steam and feed lines would not be expected to lose support function to the point of failure.

In order to maintain safe shutdown, decay heat removal via one safety or relief valve would be required. Although no guarantee is available that the safety or relief valves would be operable following a steel rod strike, RG&E does not believe it would be credible to postulate simultaneous failure of all ten safety and relief valves. Thus, RG&E is confident that decay heat removal capability would exist following a tornado.

Results - utility pole: An analysis has also been made of the potential effects of the utility pole on the steam and feedwater lines, and related piping and valves. Although it has been determined that the wall thickness is sufficient to resist perforation of the steam and feedwater lines, the integrity of the support system, and piping and valve attachments, cannot be ensured.

RG&E contracted with Dr. Larry Twisdale of Applied Research Associates, an expert in the field of tornado study, to determine the potential for windspeeds in the range of 130-180 miles per hour to cause utility poles to be become airborne. Dr. Twisdale's study, "Utility Pole Tornado Missile Trajectory Analysis," concludes that windspeeds lower than approximately 150 mph cannot provide the necessary aerodynamic lift required for a utility pole to cause potential damage to the steam and feedwater supports and attachments. Above 150 mph, the utility pole is considered a credible missile. Since the steam and feedwater piping is, for the most part, located more than 25 feet above grade, RG&E does not consider that the utility pole is a credible missile for consideration of damaging effects at tornado windspeeds below 150 mph.

Another evaluation presently underway, but not yet completed, involves determining the potential effects of damage to the above-noted steam and feedwater system components, due to failure of block walls. The block walls are located at the entire level in the intermediate building where the steam and feedwater lines are located. Based on the tornado missile evaluation, RG&E does not expect the majority of the systems to require protection; however, some local protection, such as for an atmospheric dump valve, may be anticipated.

d. Surface of the Spent Fuel Pool

An analysis has been performed for RG&E by Pickard, Lowe, and Garrick, Inc. entitled "Criticality Analysis for the Spent Fuel Storage Racks." It has been calculated that, even if the utility pole caused displacement of a fuel storage box, such that several fuel storage boxes were adjacent, a K_∞ of significantly less than 0.8894 would result, with borated water of 2000 ppm in the pool (such is the case). RG&E considers this issue resolved.

Present Ginna Technical Specifications restrict placement of "recently-discharged fuel assemblies", as shown in Figure 3.11-1 of the Technical Specifications. RG&E is considering deletion of this spacing requirement in the spent fuel pool. Thus, an analysis is being conducted concerning the potential consequences of a utility pole tornado missile entering the spent fuel pool, and impacting the fuel assemblies. Based on the damping effects of 30 feet of water above the top of the fuel assemblies, no major damage would be expected. This damage analysis is expected to be completed this summer.

Even if no additional damping can be assumed for water above the fuel assemblies, there would still be no safety concern. The new spacing being contemplated by RG&E is such that the design basis utility pole could, at most, impact two "recently-discharged fuel assemblies". RG&E has already performed a radiological analysis of the consequences of a fuel handling accident, in accordance with Regulatory Guide 1.25, as documented in SEP Topic XV-20. The results of this very conservative analysis show that the offsite radiological consequences associated with the failure of all rods in one fuel assembly 100 hours after shutdown is less than 100 rem dose to the thyroid. Thus, the radiological consequences of the failure of all rods in two fuel assemblies would still be substantially less than the guideline exposures of 10CFR Part 100.

Based on the fact that no severe physical damage to the fuel assemblies would be expected as the result of a utility pole tornado missile entering the spent fuel pool, and that no unacceptable consequences would be expected, even if such damage did occur, RG&E concludes that the plan to modify fuel assembly spacing requirements is acceptable with respect to the phenomena evaluated.

e. Diesel Generators and Their Fuel Supply

RG&E has determined that additional protection is required for the doors and roof of a diesel generator room. Based on that analysis, RG&E plans to provide protection from tornado missiles for the "B" diesel generator, and its required auxiliaries.

f. Relay Room

The east wall of the relay room is steel siding and some block wall. Although it is anticipated that the relay room wall could withstand tornado wind speeds, it is not anticipated that it could withstand the rod or the utility pole. RG&E thus plans to provide missile protection for that wall.

g. Service Water System

RG&E has performed an evaluation of alternative shutdown methods, which do not require use of the Service Water System, to achieve and maintain safe shutdown. The methods include use fire hose connections to the diesel generator and Standby Auxiliary Feedwater System from the yard loop, onsite portable pumps, or fire truck pumper which could be called on to the site. Thus, RG&E does not intend to provide tornado protection for the Service Water System.

h. Standby Auxiliary Feedwater System

Although the SAFW system is protected by the SAFW Building, the discharge piping is routed through the auxiliary building. All of the discharge piping for the "C" pump, associated with bus 14, is located in the intermediate level of the auxiliary building, and thus protected from tornado missiles, except for a small elbow section. This small section of piping is protected by concrete walls on the south and east sides, and by the RMWT on the north and west. Since the "C" pump is associated with the power supply and distribution equipment not tornado-protected, RG&E plans to make the necessary changes to exchange the power supply and distribution for the "C" pump and associated valves, from bus 14 to bus 16, and for the "D" pump and associated valves from bus 16 to bus 14 or modify the discharge piping arrangement. Thus, both the power supply and piping associated with an auxiliary feedwater pump will be missile protected.

i. Instrumentation

RG&E anticipates that some primary and secondary instrumentation may require rerouting from non-protected areas in the intermediate building to the intermediate floor of the auxiliary building. Sufficient instrumentation will be provided for the operator to monitor safe shutdown conditions.

j. Cable Tunnel

An opening exists between the cable tunnel and the operating level of the intermediate building. This opening which is 7' x 7', begins 6 feet above floor level, and extends to just below the ceiling. The opening is shielded from tornado missiles on the south, east, and west directions by virtue of being below grade. From the north, major equipment in the turbine building, such as the condenser, will block virtually any missile. Based on the size of the cable tunnel opening, and the shielding now in place, RG&E does not believe any additional protection is warranted.

4.0 MODIFICATION COST ESTIMATE

4.1 Introduction

Cost estimates were performed for the modifications identified for the structural elements and for the modifications required for tornado missile protection.

The cost estimates were then evaluated in Section 5 based on the level of protection required to ensure the capability to reach a safe hot shutdown.

4.2 Structural Upgrade

4.2.1 Assumptions

Upon completion of the previously discussed analysis, RG&E attempted to establish a conceptual cost for required structural modifications. Since the actual designs of the modifications have not been determined, the cost of the modifications was calculated using the following basis:

1.	Primary Member Modifications	$/pound
2.	Secondary Member Modifications	$/pound
3.	Steel Connections	$/connection
4.	Anchorages	$/anchorage
5.	Exterior Building Shell	$/sq. foot

A review of previous plant modification costs as well as cost estimates for modifications currently in the design and construction phase have been made in order to establish a base cost for the aforementioned items.

Since certain areas of every nuclear power plant are more congested or require additional personnel controls due to higher radiation exposure levels than other areas, an adjustment factor was applied to the base costs on a building by building basis. These adjustment factors have included considerations for accessibility, radiation levels, and interferences.

By multiplying the base costs by the adjustment factors, which ranged from 1.0 to nearly 3.0, the expected cost to install the modification was established on a per building basis for each of the components listed above.

In addition, a stress factor was applied to the adjusted base cost to account for the magnitude of increased overstress incurred by increasing the tornado windspeed over 132 mph. This stress factor accounts for the fact that the complexity of a modification increases as the windspeed increases. Multiplying the base costs by the appropriate adjustment factor or both the adjustment factor and stress factor resulted in an adjusted base cost. This adjusted base cost varied depending on the area of concern. The weight of the steel required in the conceptual modification was multiplied by the associated base cost to provide an approximate cost per modification.

4.2.2 Results

Figures 4-1 through 4-4 graphically show the results of the estimate for primary members, secondary members, connections and anchorages, and the exterior shell components. For the purpose of the estimate, the severe and extreme environmental load cases were combined as a base cost and the estimated cost to upgrade to the three tornado wind speeds was added to the base cost. Figure 4-5 presents a total of Figures 4-1 through 4-4 and is a total estimated cost of structural repairs assuming all components are equally upgraded to the defined loading conditions. As seen on Figure 4-5 the estimated base cost is \$10,600,000 and is increased to \$21,300,000; \$45,200,000; and \$70,800,000 respectively for the 10^{-5}, 10^{-6} and 10^{-7} tornado events. The total cost of the individual components is also summarized on Table 4-1.

4.3 Tornado Missile and Shutdown Protection Upgrade

In addition to the structural modifications, the items addressed in Section 3.5.3 require protection from tornado missiles to assure that one train of safe shutdown equipment is available. Table 4-2 shows the estimated cost of missile protection for the three tornado events considered.

4.4 Cost Conclusions

The combined total modification estimated costs are shown in Table 4-3. These costs assume all components are modified or protected to the same level and to the general acceptance criteria discussed earlier. As seen on Table 4-3 the total respective costs for the 10^{-5}, 10^{-6} and 10^{-7} events are $24,800,000; $56,000,000 and $84,700,000.

It should be noted that these cost estimates assumed that exterior walls are upgraded to resist winds and tornadoes but not to resist tornado missiles. Costs were then estimated based on not upgrading either the secondary members or the exterior shell to resist wind and tornadoes, thus providing a consistent level of protection for required safe shutdown equipment. As seen on Table 4-4, the total respective costs for the 10^{-5}, 10^{-6} and 10^{-7} events are $14,000,000, $27,300,000 and $40,900,000.

5.0 RECOMMENDATIONS AND CONCLUSIONS

In the previous two sections, RG&E presented the analytical findings concerning the present level of protection of the Ginna facility from tornado occurrences, and the estimated costs associated with protection for severe and extreme environmental effects, and a range of tornado windspeeds and associated missiles. In this section, the level of tornado protection considered to be appropriate for use as a "design basis tornado" for the Ginna facility is defined. The design basis windspeed is chosen, considering many factors, including the cost of providing capacity for increasingly severe tornado windspeeds and missile effects, and the potential safety benefit derived from the increasing capacities.

As can be seen from the analysis described in Sections 3 and 4, the extent and cost of modifications increases substantially, as the tornado windspeed is increased from a probability level of 10^{-5} to 10^{-6} to 10^{-7}. This is not unexpected, since the forces increase as the square of the windspeed. RG&E has also attempted to consider the added safety benefit which would be derived

by designing protection to increasingly severe windspeeds. It is true that some additional safety benefit would exist, as specific protection measures were increased. However, because of the substantial safety protection available for the most important plant structures and systems, such as the containment, control complex, and auxiliary feedwater, the incremental safety benefit, although not quantified, is expected to increase only slightly with increasing windspeeds. This is especially true when considering the additional materials capacity available in the plant structures not accounted for in the analysis, the lack of credit taken for safety system separation, and inherent wind and missile damage resistance. (A discussion of this "reserve capacity" is provided in more detail in Section 5.2).

Based on the following justifications, expected modification costs, and the safety level provided by the modifications to be implemented, RG&E recommends that protection be provided for a tornado of 10^{-5} (132 mph).

5.1 Safety Assessment

RG&E believes that the safety afforded by protection to a windspeed associated with a probability of 10^{-5} per year is adequate. This probability level selected is considered congruent with the protection levels associated with other severe natural phenomena, such as earthquakes and flooding, and with postulated events, such as pipe breaks. This level of protection is also compatible with the draft NRC secondary safety goal of a probability of 10^{-4} per year of core melt. RG&E believes that the tornado risk will be only a small fraction of the total core melt risk. It is important to note that there is conservatism even in the 10^{-5} value selected as the backfitting design basis for tornado protection at Ginna. First, the 10^{-5} windspeed is associated with the upper 95th percent confidence level, rather than the median. At a median level, the selected windspeed would be a probability on the order of 10^{-6} per year. Secondly, many of the structures, systems, and components required for safe shutdown, such as the containment, control building, and auxiliary feedwater system, will withstand windspeeds significantly higher than those associated with the 10^{-5} level. Finally, the method of analysis to determine current protection, and any subsequent modifications, are conservative. Tornadoes are postulated to strike the plant from all directions, and thus no credit for shadowing, or physical separation, is claimed. Tornadoes are postulated to strike with equal intensity throughout the plant, thus seemingly affecting all structures, systems and components with equal intensity concurrently, when actually, only a fraction of the plant would see the most intense

characteristics of the tornado. And residual strength is expected in the Ginna structural and equipment elements, beyond that assumed in the analysis. These conservatisms are described in more detail in Section 5.2 below.

Thus, RG&E is confident that backfitting to a tornado level associated with a 10^{-5} tornado windspeed, at the upper 95th percent confidence level, will provide a significant level of plant protection. Further conservatisms inherent in the selection of tornado characteristics, and the analysis process, provide confidence that the risk associated with a tornado strike of this magnitude would be only a small fraction of the overall risk associated with the operation of Ginna Station.

5.2 Modification Recommendations

Modifications to the structures and missile protection for equipment, are required to upgrade the facility to a level assuring capability to a 10^{-5} or 132 mph tornado event. However, based upon the shutdown methodology discussed in Section 3.5.2, not all systems and components need be modified to the same degree to assure this protection. The design phase of the Ginna Station Structural Upgrade Program will be conducted with the philosophy that the various systems and structural components, critical for plant shutdown, must maintain their integrity with "no loss of function." In the proposed upgrade, "no loss of function" will be interpreted so that, a failure of a structural component will not lead to a collapse of a building nor will a failure of a structural component or system negate the ability of the plant to be safely shutdown.

Modifications will be made only to those members and systems found overstressed after evaluating these components using all allowable plastic deformation and stress criteria specified in AISC (Reference 10), plus any applicable NUREG/CR-0098 Ductility Factor Criteria (Reference 11). The overstressed members and systems will be modified to the extent required to provide adequate margins of safety to resist the 132 mph tornado. The following discussion indicates the modifications RG&E believes are necessary to provide adequate protection, along with the assumptions which these proposed modifications will be based.

5.2.1 Primary Members, Connections and Anchorage

The turbine building and screen house are not required to bring the plant to a safe shutdown. Complete failure of the screenhouse would not prevent reaching shutdown

based on the methodology described in Section 3.5.2 and the turbine building must only be maintained to the extent necessary to assure that its failure will not impact the surrounding buildings. The integrity of the other building's primary framing systems must be maintained to assure they will not adversely affect required systems and will be maintained within allowable code levels unless higher allowables can be shown not to have any effect on the overall structural integrity. Since RG&E believes it is prudent to maintain the general integrity of the screen house and turbine building, modifications will be made, but the need will be based on an overall "no loss of function" basis allowing localized failures and consideration of plastic deformation and ductility ratios.

As an initial screening, the main analytical model will be checked using the Standard Review Plan (SRP 3.8.4) (Ref. 12) acceptance criteria of 1.6 S, where S is the required section strength based on elastic design methods and the allowable stresses defined in AISC (Ref. 10).

The members identified by the first level of screening as not meeting the 1.6 S acceptance criteria will be further evaluated to a second level of acceptance as defined below:

a. Compression:

Allowable compressive stress, on the gross section of axially loaded members shall be basic column buckling stress and shall be calculated as defined in AISC for column buckling.

b. Bending:

For beams with an I-shaped profile symmetrical about both axis, and the subject to strong axis bending, the allowable compressive stress due to bending shall be the theoretical elastic lateral buckling stress as specified in AISC.

Because of the low probabilities associated with the loads, conservatism in the methods of analysis and inherent reserve capacity in the structure a factor of safety of 1.0 for compression and bending will be adopted.

c. Tension:

Except for pin connected members, F_t will not exceed F_y on the gross area nor .8 F_u on the effective net area (where the terms are as defined in AISC).

d. All Others:

All other allowable stresses not covered will be 1.6 S where S is as defined in Section 5.2.1a of the AISC code.

e. Slenderness Ratios:

The slenderness ratio, KL/r, limitations of AISC (Ref. 10) will not be used as an acceptance criteria.

Compression members which do not meet the slenderness ratio requirements of AISC will be evaluated on a case by case basis. Basic buckling capacity of these members will be calculated as in Section 5.2.1a of the AISC code.

This is consistent with the AISC code as noted in AISC commentary page 5-127.

The members which do not meet the requirements outlined in this report will be examined on a case by case basis to determine the criticality of the member for overall structural stability and for the effect on the plant shutdown.

Consistent with the primary member evaluation, the connections and anchorages will be evaluated based on the methodology described. This approach requires all primary member connections and concrete anchorages must maintain their overall integrity based on the same "no loss of function" criteria. Modifications to the connections and anchorages will be made using appropriate plastic deformation and ductility ratios for the steel and/or bolts only if the component is found unacceptable. For those components, modifications will be made to the extent required to provide an adequate margin of safety against the 10^{-5} for 132 mph tornado event.

Hanger type connections will be modified in accordance with the method of analysis described in AISC. However, required factor of safety for bolts in tension, and tee flanges, or angle legs in bending will be 1.0 or greater.

For all other types of connections, the acceptance criteria will be 1.6 S.

At Ginna, the connections have been categorized as either beam to beam, beam to column, or truss type. These connections will be analyzed using "The Guide to Design Criteria for Bolted and Riveted Joints" (Reference 14), and all applicable AISC criteria.

For anchorages to concrete, the capacity and the acceptance criteria for the steel component of the anchorage will be in accordance with the ACI 349 Appendix B "Steel Embedments" (Ref. 13). Concrete capacity of the anchorage will be calculated in accordance with the ACI 349 Appendix B. If the concrete capacity is found to be less than that of the minimum ultimate strength of the steel component of the anchorage, the concrete capacity will be compared to the applied loads under study. In such cases, a factor of safety greater than 1.0 will be acceptable against concrete pull-out or busting. Due consideration will be given to the edge distance and embedment lengths in calculating concrete capacity.

5.2.2 Exterior Shell and Secondary Members

The extent of modification required for the building shell and secondary members is dependent on the degree of failure and the effects of failure. If the entire shell and secondary systems were to fail early in the event and become missiles themselves, they could affect safe shutdown capability and would expose internal components to large wind loads. However, as can be seen from the analysis, the various components of the shell and secondary system will generally only begin to show indication of failure in local cases near the actual proposed design windspeed. Further, the method for bringing the plant to a safe shutdown already assumes substantial plant damage from tornado missiles.

For a positive pressure loading, the siding, roofing and secondary members can withstand loads much greater than 132 mph before actual failure will occur. These components will begin to "fail" at a lower windspeed due to negative and differential pressure. In the case of siding, the "failures" are at eaves and cornices and are because of local pressure coefficients. The areas of concern are smaller than the actual panel size and failure will likely be the edge connection breaking while the siding remains in place. This will in essence result in self venting, in immediate relief of the structure of any differential pressure, and in an unloading of the primary structural frame.

The girt failures are generally due to overstress because of the large unsupported length of the compression flange under negative loads. It is not expected that they would fail and break loose from the structure but would simply twist and become permanently deformed.

Since localized failures will provide load relief on the primary framing system and should remain attached and since missile protection as described below is being provided, no modifications will be made to the exterior shell or secondary members for the design basis tornado.

Secondary members which do not meet the 1.6S acceptance criteria will be reevaluated using energy techniques. Secondary members not meeting the acceptance criteria will be reevaluated on a case by case basis to determine if their failure would damage any component such that safe shutdown could not be accomplished.

The steel roof decking will be assumed to provide no resistance if there is a net upward load primarily due to the puddle weld connection unless additional testing verifies the puddle weld capacity. AISC does not recognize puddle welds as tension resistent connections.

5.2.3 Tornado Missile Protection

Based on the tornado missile evaluation for safe shutdown equipment, and consistent with a design basis windspeed associated with a 10^{-5} tornado. RG&E expects that missile protection modifications will be required for the following items:

a) breaker between bus 18 and Diesel Generator 1B
b) steam line atmospheric dump valves
c) diesel generator building 1B roof and doors, and air discharge muffler
d) east wall of relay room
e) exchange of power and control cables for the SB AFW system
f) safe shutdown monitoring instrumentation
g) block walls above spent fuel pool.

5.2.4 Reserve Plant Capacity

An examination of the results of the evaluation was made in order to establish an approximate value of the reserve capacity of the plant framing after completion of the upgrade.

a) Theoretical physical properties of the materials that exist in the structure and those that are used for analysis are typically lower than the actual values. For example, A36 steel has a minimum yield strength of 36 ksi but typically the actual yield values are higher.

b) The upgrade will be done to assure that there are no actual failures in the primary structural framing. This means that the buildings generally will be upgraded based on elastic behavior, i.e. strains below the yield stress. In reality steel structures are capable of absorbing a large amount of energy above the yield strain of the material. For mild steel, the ratio of strain at rupture to strain at first yield is as much as 100 times the yield strain value. This ductility feature of steel implies that gross and sudden failures will not occur at design levels although permanent deformations may result.

c) The application of loads for analysis purposes is conservative. The live loads used in the analysis are those that are defined for design considerations. In reality, the full live load on all floors will not occur simultaneously. However, for the analysis and evaluation, the full live loads were applied. These loads are vertical and contribute to the total state of stress in the beams and columns.

d) The evaluation examined the plant for tornado winds applied in four directions (N, E, S, W). The number of overstressed members which were found as a result of the 132 mph windspeed is the total of all failures found in all four directions. The recommended upgrade will modify all these primary members regardless of the wind direction. The actual occurrence of a tornado would effect the plant from only one direction. Therefore, the upgraded plant will have inherent conservatism because the actual number of members experiencing high loads for a single direction tornado will be less than the total number that will be upgraded.

e) The analyses that were performed assume that the building response is completely elastic. In most steel structures, local plastic deformations will occur in conjunction with the elastic response of the main frame system. For bolted steel structures such as those at Ginna, some degree of slipping will occur in the connections when they are loaded with these extreme loads.

The combination of local deformations and slipping in the connections will absorb some of the total load that is applied to the structure and lessen the total stresses predicted by the elastic analysis.

f) The results of the evaluation have shown that of all the tornado windspeed components, the differential pressure had the most significant impact on the secondary members and exterior shell. At the design tornado windspeed of 132 mph, certain areas of the plant siding and secondary members experience large deflections and minor failures, primarily along the edges and corners of the roof. RG&E has proposed to allow the secondary members and siding to fail, since they will have no consequence on the overall plant integrity. However, the failures of these areas of the exterior shell will tend to relieve the differential pressure by providing additional venting of the structure along with the existing vent area in all the buildings. This vent area will reduce, if not eliminate, the loads created by the differential pressure. The result will be an immediate stress relief for all the plant structures.

5.2.5 System Reserve Capacity

In addition to the structural reserve capacity expected to be available, due to material specifications and analytical methods, substantial conservatisms were incorporated into the safety system analysis assumptions.

In terms of tornado wind and missile protection, RG&E has assumed that, unless specifically analyzed for or denoted otherwise, failure of a non-protected system or structure would occur. Generally, no credit has been taken for the protection inherent in the equipment itself to resist tornado winds. In fact, the majority of items would not experience the peak wind characteristics of the design basis tornado. Thus, realistically, separation of components, and equipment capability itself, would lessen the number of failures.

For tornado missiles, RG&E has assumed that all equipment not tornado-missile protected could be damaged. Actually, for the design basis windspeeds expected, only the lightest objects would be capable of experiencing the aerodynamic forces to actually become missiles. These lighter objects would not be expected to perform substantial damage. Also, shadowing of components would be expected to be highly effective in ameliorating missile damage.

Further, for tornado missiles, it is assumed there is an equal probability of damage to all non-protected equipment. On a probabilistic basis, this would not be expected to occur. The probability of a tornado missile striking small objects would be expected to be significantly lower than the probability of the tornado itself,

which is already considered a 10^{-5} to 10^{-6} per year event. Therefore, on a realistic basis additional safety margins exist for tornado missile protection.

5.3 Conclusions

Based on the recommended upgrade proposed by RG&E, Ginna Station will be modified to the following criteria:

1. all primary steel framing will be modified to resist a 132 mph tornado windspeed and 100 psf extreme snow load,

2. no exterior shell or secondary member modifications will be made except those previously mentioned and,

3. the required safe shutdown equipment will be protected from tornado missiles.

FIGURE 4-1

ESTIMATED PRIMARY MEMBER MODIFICATION COSTS

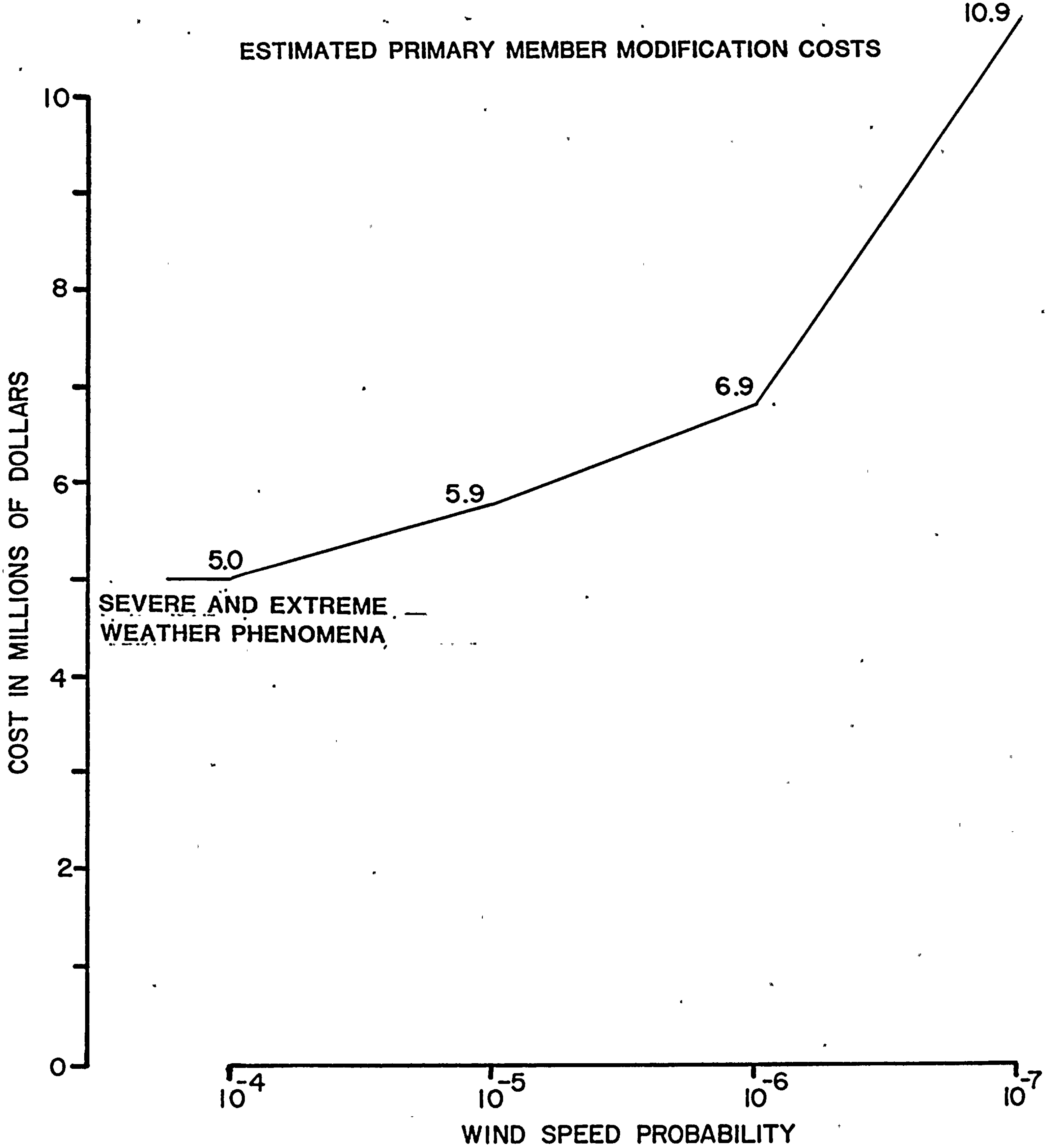
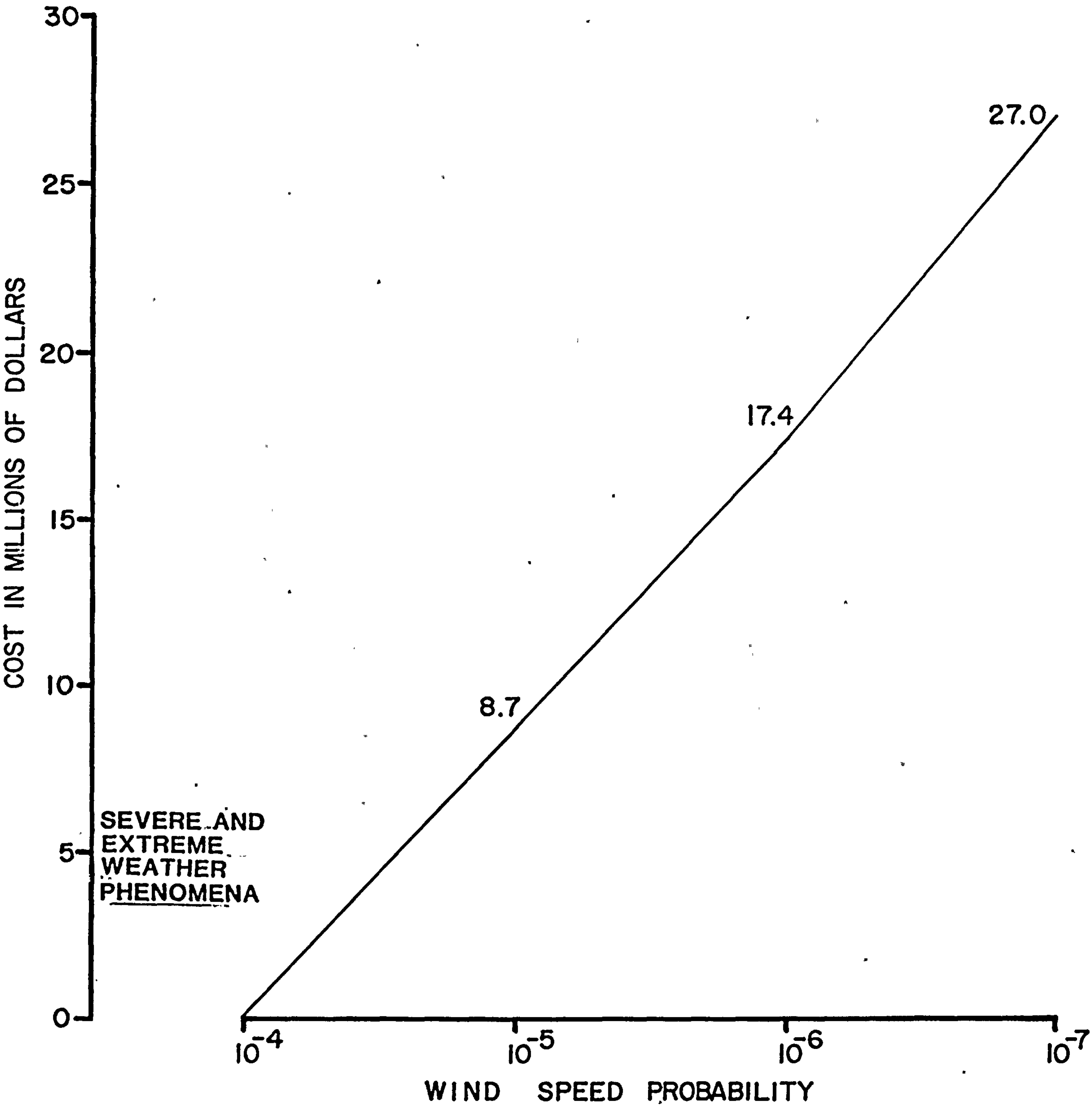


FIGURE 4-2

ESTIMATED SECONDARY MEMBER MODIFICATION COSTS



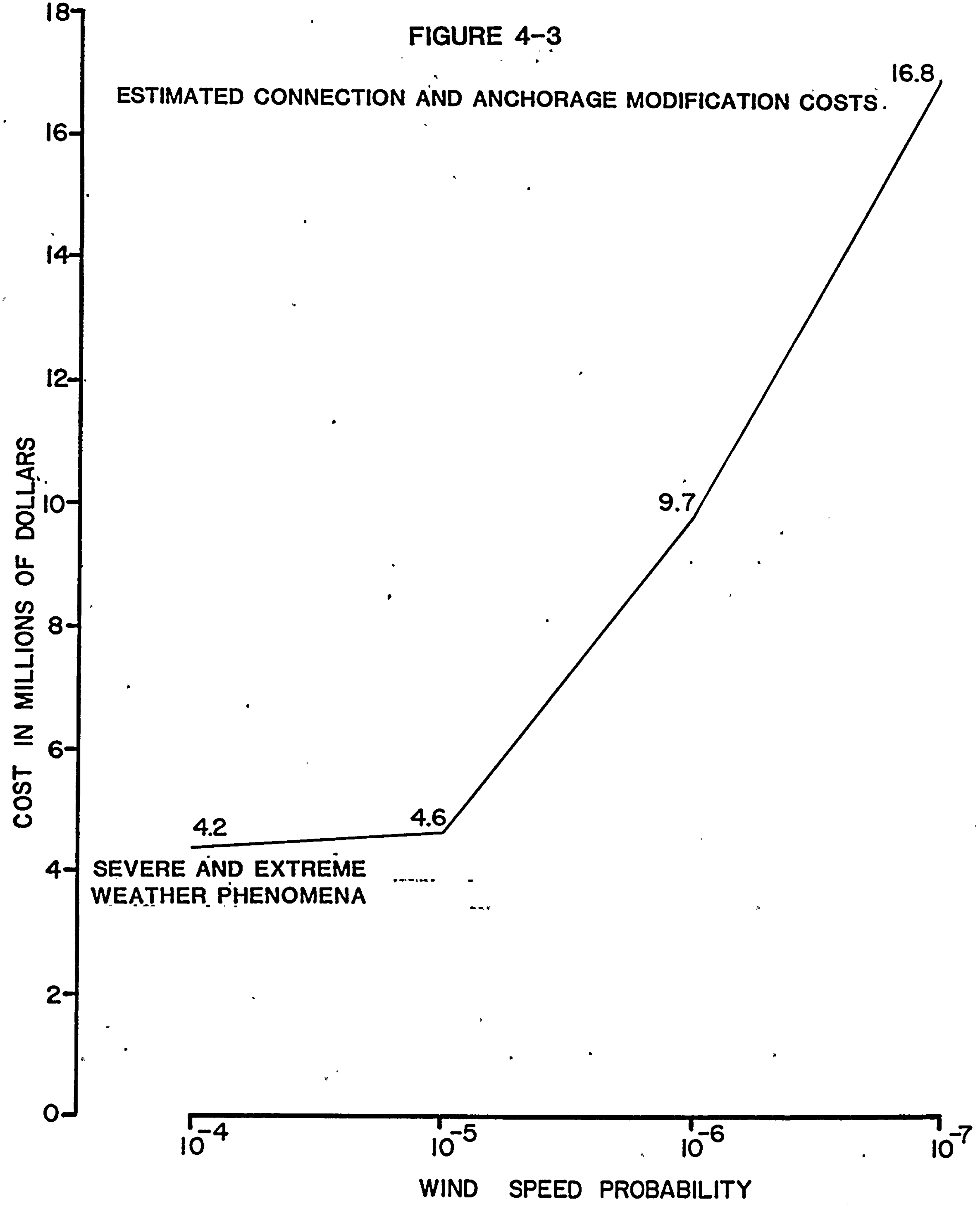


FIGURE 4-3
ESTIMATED CONNECTION AND ANCHORAGE MODIFICATION COSTS.
COST IN MILLIONS OF DOLLARS
0
2
4
6
8
10
12
14
16
18
4.2
4.6
9.7
16.8
SEVERE AND EXTREME WEATHER PHENOMENA
10^{-4}
10^{-5}
10^{-6}
10^{-7}
WIND SPEED PROBABILITY

FIGURE 4-4

ESTIMATED EXTERIOR SHELL MODIFICATION COSTS

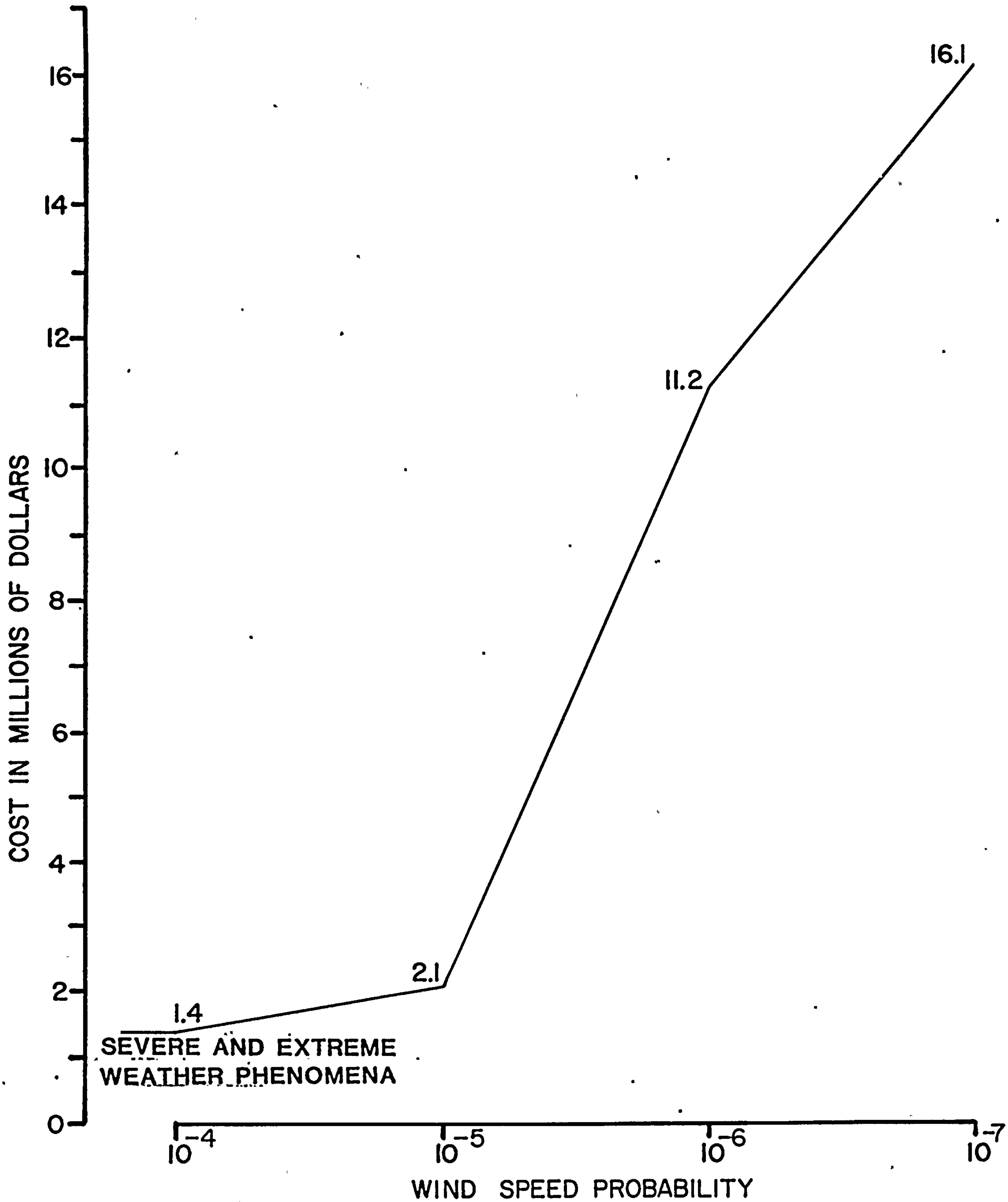


FIGURE 4-5

ESTIMATED TOTAL STRUCTURAL MODIFICATION COSTS

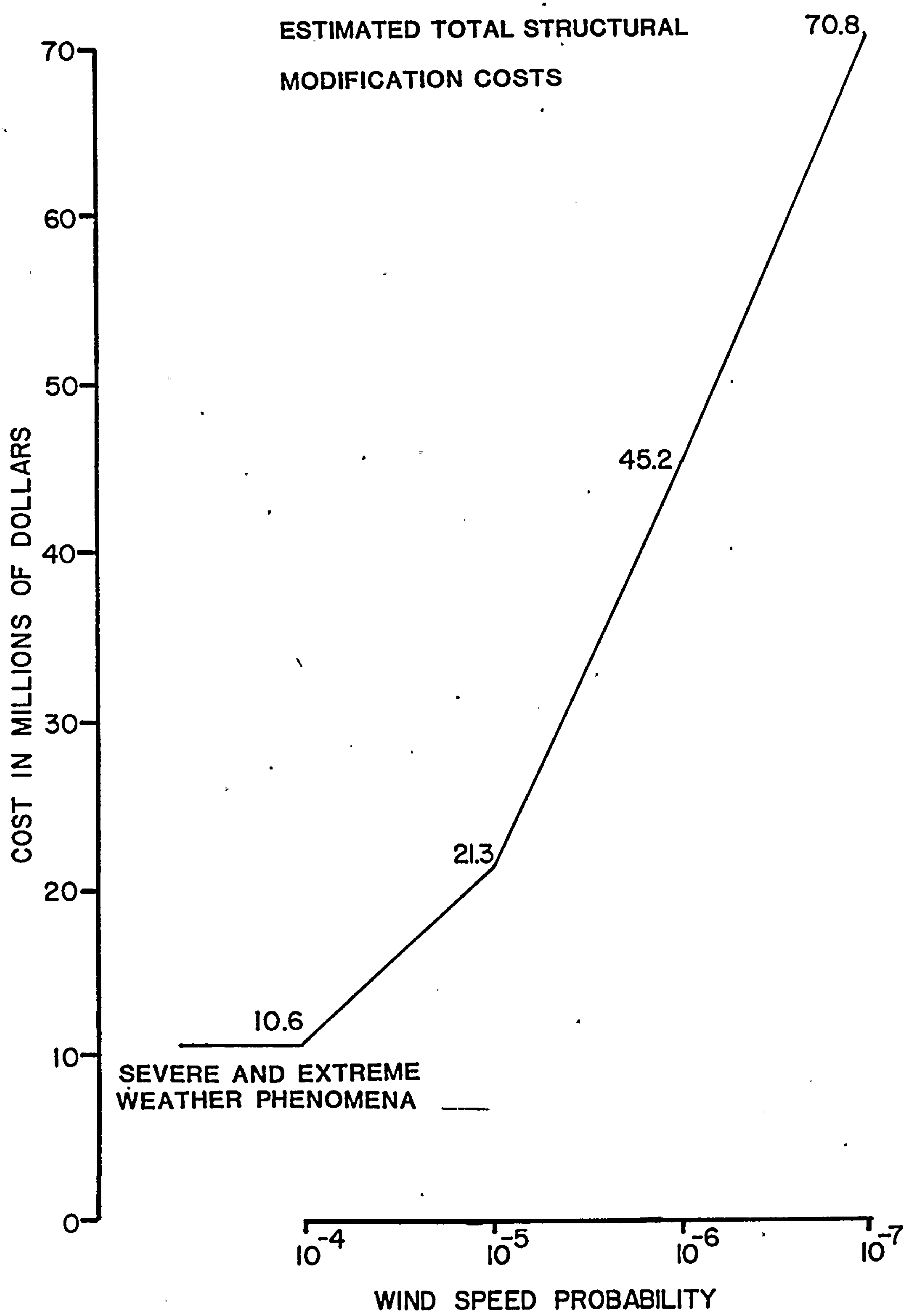


TABLE 3-1

PRIMARY MEMBER FAILURES PER LOADING COMBINATION

LOADING COMBINATION

BUILDING	MEMBER TYPE	SEVERE	SEVERE +SN'	SEVERE + SN' + WT(132)	SEVERE + SN' + WT(188)	SEVERE + SN' + WT(250)
Auxiliary	Columns	20	23	25	25	39
	Beams	21	22	24	26	38
	Bracing	6	14	14	15	36
	Truss	0	0	0	0	0
	Total	47	59	63	66	113
Inter/Facade	Columns	17	19	19	20	60
	Beams	10	12	13	18	22
	Bracing	2	7	12	12	37
	Truss	4	7	10	15	25
	Total	33	55	54	65	144
Turbine, Control, Diesel	Columns	16	17	34	44	51
	Beams	5	5	6	6	7
	Bracing	57	72	87	104	154
	Truss	2	5	34	35	52
	Total	80	99	161	189	264
Screen house	Columns	2	2	2	2	31
	Beams	4	4	4	4	10
	Bracing	2	2	6	12	20
	Truss	0	0	9	28	109
	Total	8	8	21	46	170
Total		168	211	299	366	691

TABLE 4-1

SUMMARY OF STRUCTURAL REPAIRS

$ x 1000

	Severe Weather Phenomena	Tornado Wind Speed Probability 10^{-5}	10^{-6}	10^{-7}
Primary Members	5,000	5,900	6,900	10,900
Secondary Members	0	8,700	17,400	27,000
Connections and Anchorages	4,200	4,600	9,700	16,800
Building Shell	1,400	2,100	11,200	16,100
TOTAL	10,600	21,300	45,200	70,800

TABLE 4-2

TORNADO MISSILE PROTECTION COST

$ x 1000

ITEM	WIND SPEED PROBABILITY 10^{-5}	10^{-6}	10^{-7}
RWST	---	1330	1750
Spent Fuel Pool Block Wall	700	980	1330
Move SBAFW Piping	1400	1400	1400
Miscellaneous Instrumentation Reroute	140	140	140
D/G Building Missile Protection	700	980	1340
Move D/G Breakers From Screen House	140	140	140
Missile Shield On East Wall - Relay Room	420	580	800
Harden Elev. 278' Int. Building	---	5250	7000
TOTALS	3500	10,800	13,900

TABLE 4-3

TOTAL COST FOR STRUCTURAL UPGRADE
INCLUDING TORNADO MISSILE PROTECTION

$ x 1000

	SEVERE WEATHER PHENOMENA	TORNADO 10^{-5}	WIND SPEED 10^{-6}	PROBABILITY 10^{-7}
Structural Upgrade	10,600	21,300	45,200	70,800
Missile Protection	0	3,500	10,800	13,900
TOTAL	10,600	24,800	56,000	84,700

TABLE 4-4

TOTAL COST FOR STRUCTURAL UPGRADE
INCLUDING TORNADO MISSILE PROTECTION
WITHOUT SECONDARY MEMBER OR EXTERIOR SHELL FIXES

$ x 1000

	Severe Weather Phenomena	Tornado Wind Speed 10^{-5}	10^{-6}	10^{-7}
Structural Upgrade	9,200	10,500	16,500	27,000
Missile Protection	0	3,500	10,800	13,900
TOTAL	9,200	14,000	27,300	40,900

REFERENCES

1. USNRC Final Evaluation of SEP Topic II-2.A, "Severe Weather Phenomena", November 3, 1981.

2. USNRC Final Evaluation of SEP Topic III-2, "Wind and Tornado Loadings", April 21, 1982.

3. USNRC, "Results of Peer Review and NRR Staff Review of the McDonald Methodology", L.L. Beratan to W.P. Gammill, December 13, 1982.

4. USNRC Final Evaluation of SEP Topic III-4.A, "Tornado Missiles", April 16, 1982.

5. USNRC Final Evaluation of SEP Topic III-7.B, "Design Codes, Design Criteria and Load Combinations", April 21, 1982.

6. Lawrence Livermore Laboratory, "Seismic Review of the R.E. Ginna Nuclear Power Plant as Part of the Systematic Evaluation Program", November 15, 1980.

7. RG&E Letter to Dennis M. Crutchfield, Chief Operating Reactors Branch No. 5, re: "SEP Topic III-6, Turbine Building Bracing", August 26, 1982.

8. RG&E Letter to Dennis M. Crutchfield, Chief Operating Reactor Branch No. 5, re: "SEP Topics III-6 and III-11, Seismic Design Considerations", October 28, 1981.

9. American Concrete Institute (ACI) - "Steel Embedments," ACI 349 Appendix B, 1980.

10. American Institute of Steel Construction (AISC) - "Manual of Steel Construction," Eighth Edition (AISC 1980).

11. USNRC, Systematic Evaluation Program Position, re: "Consideration of Inelastic Response Using the NUREG/CR-0098 Ductility Factor Approach".

12. USNRC Standard Review Plan, Section 3.8.4, "Other Seismic Category I Structures".

13. "Guide to Stability Design Criteria for Metal Structures," Structural Stability Research Center, John Wiley and Sons, 1976.

14. "Guide to Design Criteria for Bolted and Riveted Joints," Fisher, J. W., and Struik, J. N. A., John Wiley and Sons, 1974.