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JAN 04 1984

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VICE-PRESIDENT
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Mr. A. Schwencer, Chief
Licensing Branch No. 2
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Docket Nos.: 50-352
50-353

Subject: Limerick Generating Station, Units 1 and 2
Structural Steel Fire Resistance

References: (1) Limerick Safety Evaluation Report (NUREG-0991)
(2) NRC Chemical Engineering Branch Meeting, Held December 13, 1983
(3) L. S. Kintner to Applicant (PECO) Letter Dated December 23, 1983

File: GOVT 1-1 (NRC)

Dear Mr. Schwencer:

Reference (1) requires that the applicant provide fire protection for exposed structural steel supporting the floors of safe shutdown areas.

The Reference (2) meeting was held to describe the means for assuring equivalent protection for structural steel in lieu of strict application of protective coating.

The attachments provide the information requested in Reference (2) and documented in Reference (3), to support the program we presented to close out SER Open Item #14, Structural Steel Protection.

I. The following information was requested by Mr. C. P. Tan at the Reference (2) meeting:

A. Mechanical Properties of Steel at Elevated Temperatures

The curves contained in Attachment I were published by the U. S. Steel Corporation and illustrate the effect of temperature on yield strength, tensile strength and modulus of elasticity. Similar curves published by the American Iron and Steel Institute (AISI) are contained in Attachment II. The structural steel used at Limerick conforms to ASTM Specification A36.

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The steel behavior predicted by these curves is based on tension tests of small uniformly heated test specimens. While the curves provide an indication of the effect of temperature on the properties of steel, they are not considered to accurately represent the totally different stress condition existing in flexural members that are both end restrained and constructed compositely with the concrete slab. In the reference material submitted with the structural steel methodology, the behavior of structural members subjected to elevated temperatures is discussed in greater detail.

B. Typical Construction Details

Floor framing systems are typically composite construction utilizing steel studs to resist shear between the steel beam and the concrete slab. In many cases, the top flange of the beam is encased in concrete with the steel deck being supported by ledge angles welded to the web of the beam. In all cases the deck is welded to the supporting member, a type of construction that has been verified by testing to provide a degree of composite action.

In accordance with standard structural theory, the effect of constructing the steel beam integrally with the concrete slab is to increase the section modulus of the assembly and thus increase its load-carrying capacity. When subjected to elevated temperatures, a composite beam/slab assembly can withstand higher temperatures than a non-composite member of the same size prior to reaching the point at which the assembly can no longer support the applied loading. Loading considered in the design of the steel framing is contained in Section 3.0 of the Limerick FSAR.

The primary concern is with floor support beams since exposed steel columns occur in few fire areas. Column members are typically W14 rolled sections or built-up members consisting of a W14 section with flange plates. Exposed columns in identified problem areas will be evaluated similarly to the beam/slab assemblies. Typical framing details are contained in Attachment III.

C. Acceptance Criteria

The structural steel acceptance temperatures presented in the analysis methodology are specified by ASTM as end-point criteria for determining a fire endurance rating. Therefore, if the actual steel temperatures do not exceed the criteria during a defined fire in the area, the steel is acceptable as a fire barrier having a fire rating equal to the fire resistance required of the barrier.

The conservatism inherent in the ASTM end-point temperatures for beams is that these acceptance temperatures are applied uniformly to beams that are:

1. both end-restrained and free to move

2. individually acting versus one member of a multi-member floor system.

It is stated by AISI in Attachment II that test experience and research clearly show a significant improvement in fire endurance when the ends of a test specimen are restrained against expansion and that even minimal restraint will improve fire endurance. It is therefore conservative to apply end-point temperature criteria for an unrestrained beam to a restrained beam. Similarly, a beam that is constructed so as to act integrally with the concrete slab has greater load carrying capacity than an individual, non-composite beam of the same size. In addition to greater load carrying capacity, composite beams have continuous lateral support of the top flange which, according to Appendix X4 of ASTM E119, provides substantial restraint against thermal expansion. This is neglected in the end-point temperature criteria.

Finally, the steel temperature of 1100°F can be correlated to the temperature vs. strength curves given in Attachments I and II. It is commonly accepted that, at 1100°F, the yield stress of steel has decreased to approximately 60% of the room temperature value which is the normal design stress level. This criteria is reiterated in the NRC Generic Letter 83-33 dated October 19, 1983. As previously discussed, the application of this criteria to restrained, composite beams is, if anything, conservative. Therefore, by adhering to the ASTM criteria, the structural integrity of the structural steel is assured.

II. Structural Steel Exposure to the E-119 Time Temperature Curve

In Attachment IV, we have shown the results of exposing the beams (indicated as having a "local effects" problem) in the Control Enclosure, El. 217- Switchgear Area to the extreme fire condition depicted by the E-119 time-temperature furnace condition. They indicate that:

1. The time period for these beams reaching critical temperatures range from twenty to forty minutes, whereas the furnace temperature reached 1100°F in seven minutes.

2. There is a significant lagging effect of the structural steel temperature to the average compartment gas temperature.

III. In Attachment V, we have provided a comparison between the results of the NRC sponsored tests performed at Underwriters Laboratories as published in NUREG/CR-3192 and the results of Limerick structural steel survivability calculational methodology as applied to the Underwriters Laboratories fire tests.

Our efforts have been limited to evaluating the results of Experiment #3 and Test #1 because of the limited data in the NUREG. The data required to complete the comparisons for the remaining experiments and tests was received from Sandia National Laboratories on January 4, 1984. We expect to submit the additional comparison calculations by January 13, 1984.

IV. Structural Steel Analysis

In Attachment VI, we have included the results of our structural steel analysis calculations for an additional thirty-four plant areas which contain safe shutdown equipment and exposed structural steel members. Three areas; the Unit 1 Refueling Floor, the Spray Pond Pump Structure, and a Unit 1 Reactor Enclosure pipe chase remain to be completed. The calculations for these areas will be available for submittal by the end of January, 1984.

Sincerely,

John S. Kemper

DMO/GMM/GJR/DMG/gra/122983415

cc:	Judge Lawrence Brenner	(w/enclosure)
	Judge Peter A. Morris	(w/enclosure)
	Judge Richard F. Cole	(w/enclosure)
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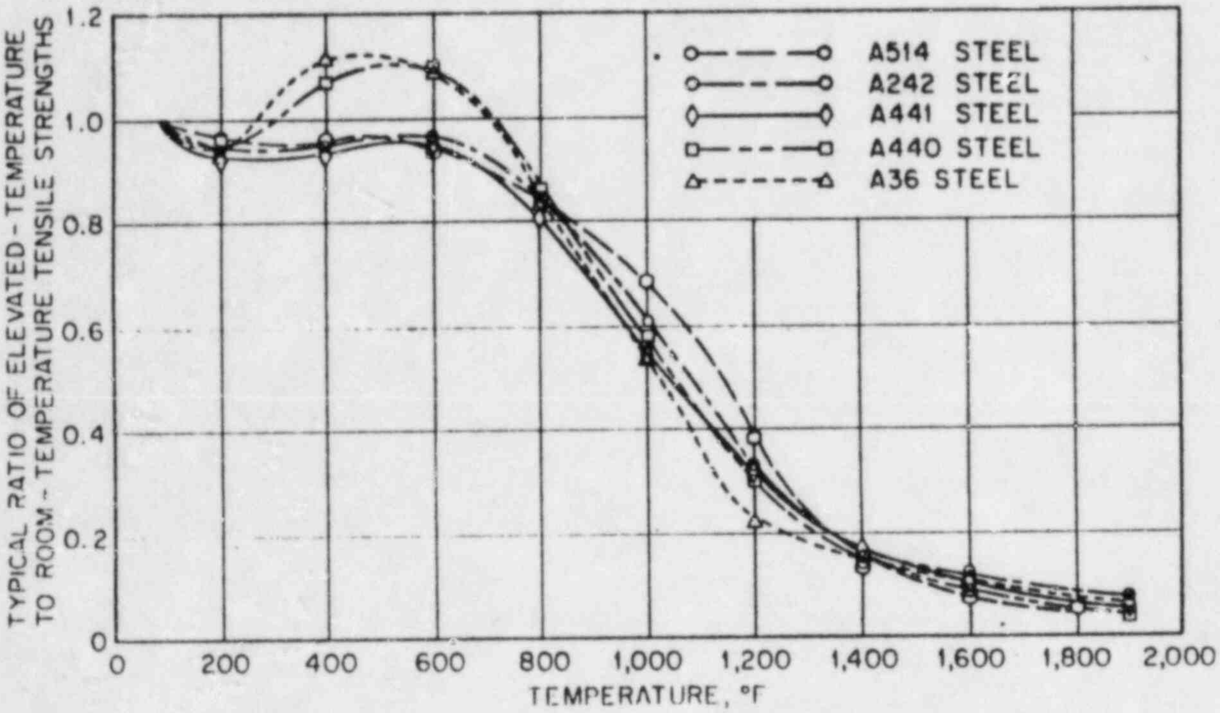
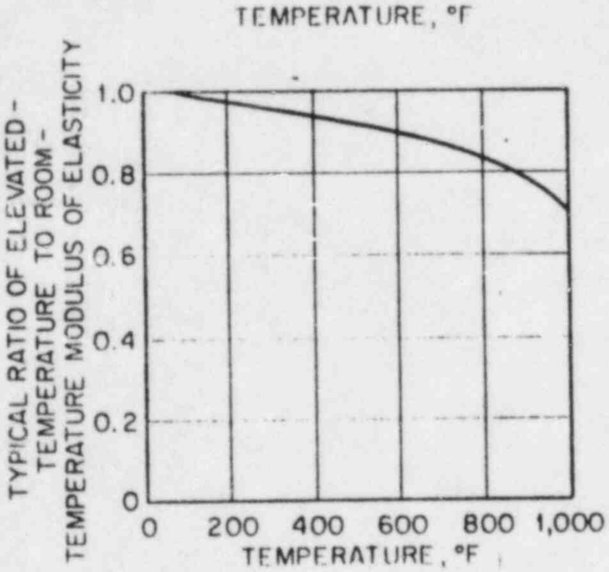
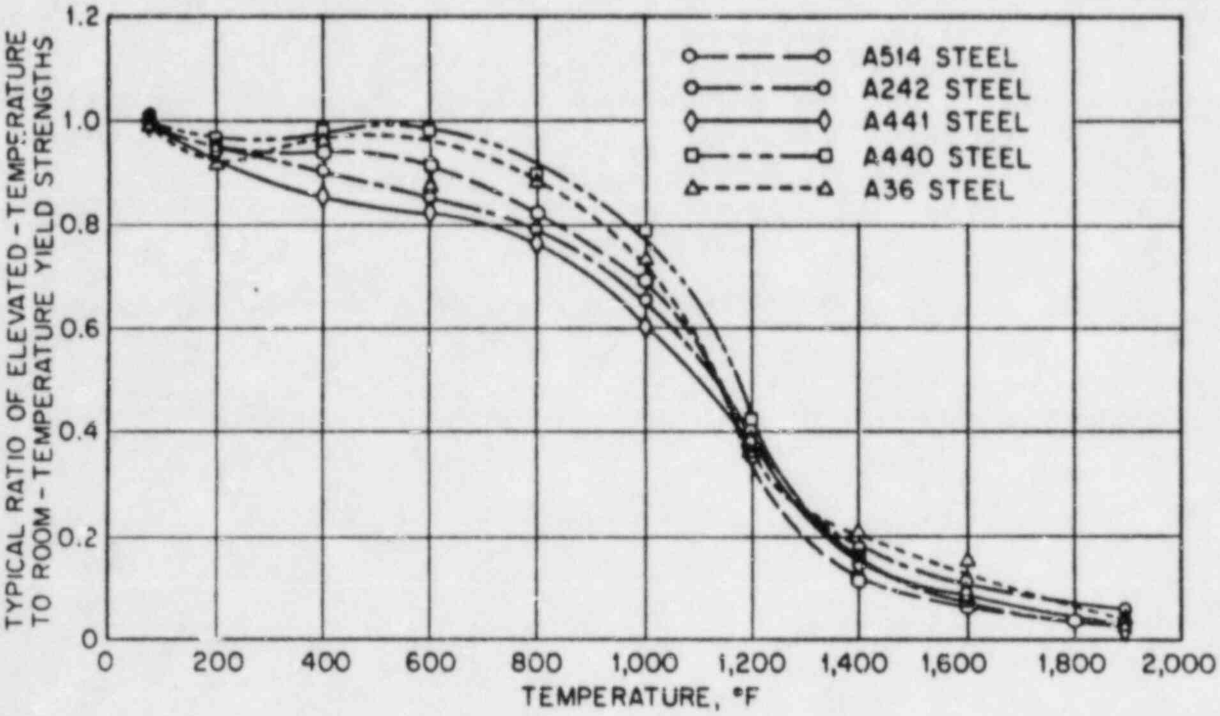




Reference

R. L. Brockenbrough and B. G. Johnston, "USS Steel Design Manual", U.S. Steel Corporation as reprinted in "Structural Steel Designers Handbook", 1972 Frederick S. Merritt, Editor.



TYPICAL RATIO OF ELEVATED-TEMPERATURE TO ROOM-TEMPERATURE YIELD STRENGTHS
1.2
1.0
0.8
0.6
0.4
0.2
0
A514 STEEL
A242 STEEL
A441 STEEL
A440 STEEL
A36 STEEL
0
200
400
600
800
1,000
1,200
1,400
1,600
1,800
2,000
TEMPERATURE, °F