Research Report Load & Resistance Factor Design

File: LRFD

Fatigue

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Charles S. Nolan and Pedro Albrecht

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AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.

The Wrigley Building / 400 North Michigan Avenue / Chicago, Illinois 60611-4185 / 312 • 670-2400

August 8, 1983

Prof. Pedro Albrecht
Dept of Civil Engineering
University of Maryland
College Park, MD 20742

Dear Professor Albrecht:

Thank you for your letter of July 7, 1983, to W. A. Milek, transmitting a copy of the report entitled "Load and Resistance Factor Design of Steel Structures for Fatigue". Mr. Milek has retired from AISC and I have assumed his position. As you may know, the LRFD Tentative Specification will be made available to the profession for a one year period of review and trial use beginning September 1, 1983. During this period your suggestions regarding the fatigue provision will be considered by the Specification Committee.

In the interim, I have quickly reviewed your report and offer the following preliminary comments. First, I believe that we should attempt to harmonize the AISC Specification with international recommendations. We are participating in developing the recommendation of the European Convention for Constructional Steelwork (ECCS). The current draft of ECCS adopts a slope of $m = 3$ for presenting fatigue data in lieu of $m = 3.2$ derived in your study. The basic fatigue strength curves of ECCS are based on a survival probability of 97.3% (mean minus two standard deviations). The detail classifications are indentified by the fatigue strength at 2 million cycles in lieu of the letter designation in U.S. specification. The above survival probability corresponds to a safety index $\beta = 2.0$. When different safety indices are desired, the following resistance factors γ_m are recommended. (ECCS defines resistance factors for design strength as the inverse of our factors)

β	2.0	2.5	3.0	3.5
γ_{mn}	1	1.32	1.73	2.3
$\gamma_{m\sigma}$	1	1.10	1.20	1.30

γ_{mn} = resistance factor applied to fatigue life.

$\gamma_{m\sigma}$ = resistance factor applied to fatigue design strength.

For high safety requirements ECCS recommends improving fabrication inspection and monitoring service performance instead of further increasing the resistance factors.

We would appreciate your views on the merits of the ECCS approach. I believe it is very important that future specifications make the design criteria transparent to the designer. Instead of using a load factor of 1.6 for live load, which implies safety for static loading conditions, fatigue design should be based on service loads. Fatigue service loads should preferably be based on the expected load spectrum. Required life should be expressed in years based on anticipated frequency of load applications. Clearly, the Specification Committee will have to give some serious thought to the philosphy to be used in developing an improved approach to fatigue design that will help designers in recognizing the effects these important parameters.

Sincerely,

G Haaijer

G. Haaijer
Vice President

GH/cd

cc: John A. Edinger
A. P. Arndt

11ENG-Q1-3

ABSTRACT

Fatigue design provisions, based on LRFD, are proposed in a form that could be directly incorporated in the AISC "Tentative Specification for Load and Resistance Factor Design, Fabrication and Erection of Structural Steel for Buildings." They would replace the fatigue design provisions of the latter which still employ an allowable stress method. The target reliability for fatigue design, $\beta = 4.0$, was determined by calibration. The resistance factor, $\gamma = 1.6$, and load factor, $\Phi = 0.66$, account for the variabilities in fatigue test data, fabrication, modeling and load. In addition to having an LRFD format, the fatigue specifications proposed herein features continuous analytical definition of resistance and more realistic variable amplitude loading. In contrast, the present AISC version employs a tabular step-wise definition of allowable stress range ill suited for computer aided design, is limited to constant amplitude loading, and accounts only for the variability in fatigue test data. The proposed LRFD specifications are recommended for adoption by AISC. They could also be used, without changes, for highway bridges.

ACKNOWLEDGEMENT

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CHAPTER 1

INTRODUCTION

1.1 General Statement

Traditionally, design specifications have been based on deterministic approaches utilizing allowable stresses and safety factors. Recently, however, there has been a shift toward the use of probability based design.

In 1976, Galambos proposed criteria for Load and Resistance Factor Design (LRFD) of steel structures.[14] This method is based upon first-order probability theory and utilizes a limit states approach to design. In addition, the Committee on LRFD, American Society of Civil Engineers (ASCE), published eight papers in the Journal of the Structural Division that presented data in support of the previous work.[16] In these studies, the only major area that eluded LRFD treatment was fatigue design. Galambos stated that "the provisions of Section 1.7 in the AISC Specification shall apply for fatigue."[14,15]

The American Institute of Steel Construction (AISC) formed a LRFD Committee for the purpose of developing a new AISC LRFD Specification for steel buildings.[9] This proposed specification now exists in draft form and is intended as an alternate to the current AISC Specification.[15] The provisions for fatigue in this draft are taken directly from the present Specification with only a few revisions. They are not based on LRFD concepts.

1.2 Present Fatigue Specifications

The present AISC fatigue specification establishes a design S-N line to be the mean line minus two standard deviations.[15] The equation is given by:

$$\log N_d = (b - 2s) - m \log F_{sr} \qquad (1.1)$$

where

F_{sr} = allowable stress range (ksi)

N_d = number of design cycles

b = intercept of mean line

m = slope of mean line

s = standard deviation of log of cycle life

This method is a simplified probabilistic approach based solely on the variation of fatigue test data. Other sources of variation, such as fabrication and workmanship, errors in the fatigue model, use of Miner's linear damage rule, and errors in the stress analysis procedure, are not considered. Also, load is considered to be deterministic, meaning that it is assumed not to vary. The specification also utilizes a step function approach when defining allowable stresses. Because of this step function approach, the intended reliabilities are only achieved at 100,000, 500,000, and 2,000,000 cycles. Between any two such points, the reliability increases as one moves away from the design point. Fig. 1.1 illustrates the effect of the step function approach on Category A, C and E details. The shaded areas are the differences between the continuous design lines and the stepped design lines. For Category A, the AISC step function extends below the continuous Category B design line. This is typical for all design categories.

The failure to include additional sources of variation in resistance and the assumption that there is no variation in load are non-conservative. On the other hand, details are designed for the actual number of cycles and the maximum stress range, as if every load cycle produces the maximum stress range. This is overly conservative. These two facts, coupled with the effects of the step function approach, result in extreme variations in the reliability index, β.

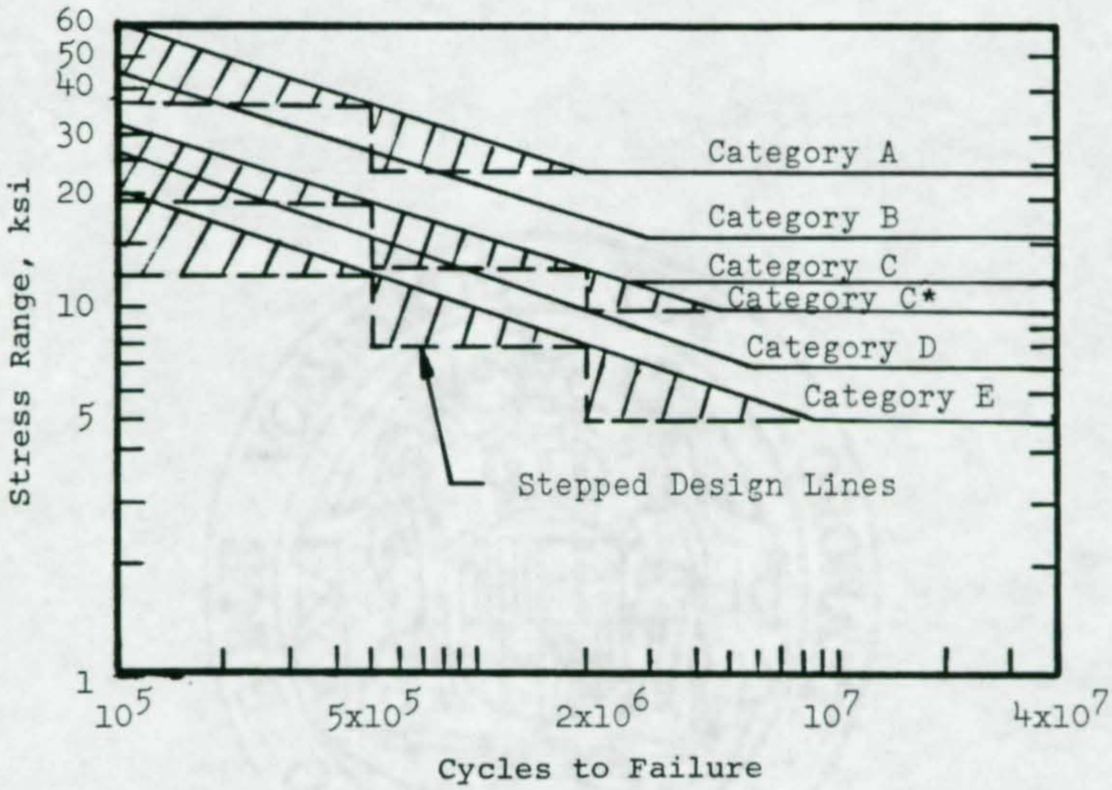

Fig. 1.1 Effect of Step Function Approach on AISC Categories A, C and E Design Lines

1.3 Objective

The objective of this work is to develop a fatigue specification that is based on uniform reliability and is compatible with the AISC LRFD Specification. The AISC LRFD Specification utilizes a live load factor of 1.6 following the recommendations of the American National Standard Institute's A58.1-1982 Standard.[13] The same is done herein.

The proposed fatigue specification accounts for the effects of all sources of variation, not just the variation in fatigue test data. The current step functions are replaced with continuous design lines. In addition, the design is based on an equivalent stress range, rather than the maximum stress range, so that variable amplitude load histories can be considered.

Emphasis is placed on developing a fatigue specification that can be easily and efficiently used in the design process.

CHAPTER 2

BASIC CONCEPTS OF LOAD AND RESISTANCE FACTOR DESIGN

2.1 Basic Concepts

The general load and resistance factor design equation is given by:

$$\Phi R \geq \Sigma\ \gamma_k\ Q_k \tag{2.1}$$

In this equation, R = resistance and Φ = resistance factor, always less than unity. On the right side of the equation, Q_k = load effect due to a particular type of loading, i.e., wind, live or dead loads. The term, γ_k, is the corresponding load factor, usually greater than unity. The load and resistance factors represent the uncertainties associated with Q and R, respectively.

2.2 LRFD Format for Fatigue

The LRFD format must be able to handle the large variances that are found in fatigue design. Wirsching has developed such a format, and a brief account of his derivation follows.[17]

Assuming that R and Q have lognormal distributions, the probability of failure is given by

$$P_f = P(R < Q) = P[\ln\ (R/Q) < 0] \tag{2.2}$$

The reliability index, β, is defined as

$$\beta = \frac{\ln\ (\tilde{R}/\tilde{Q})}{\sqrt{\ln\ ((1 + V_R^2)(1 + V_Q^2))}} \tag{2.3}$$

In Eq. 2.3, V_R and V_Q are the coefficients of variation of resistance and load, respectively. Also, $\tilde{R}$ is the median value of resistance, and $\tilde{Q}$ is the median value of load. The median values are used to account for the possibility of large variances. For random variables having large variances, the median is generally considered to be a more meaningful measure of central tendency than the mean.

Eq. 2.3, expressed in terms of $\tilde{R}/\tilde{Q}$, becomes

$$\tilde{R}/\tilde{Q} = \exp\ [\beta \sqrt{\ln\ ((1 + V_R^2)(1 + V_Q^2))}] \tag{2.4}$$

If the effects of load and resistance in Eq. 2.4 were separable, the resistance factors could be determined independently of the loading, and vice versa. To achieve this separation, one expresses the radical in the form

$$\sqrt{\ln\ (1 + V_R^2)(1 + V_Q^2)} \approx \alpha\ (V_R + V_Q) \tag{2.5}$$

The parameter α in Eq. 2.5 is called the splitting factor. Values of α for coefficients of variations, V_R and V_Q, typical of those encountered in fatigue are shown in Fig. 5.1

After α has been estimated, one can express Eq. 2.4 as

$$\tilde{R}/\tilde{Q} = \exp\ (\alpha\beta V_R)\ \exp\ (\alpha\beta V_Q) \tag{2.6}$$

The final form of the LRFD equation then becomes

$$\exp\ (-\alpha\beta V_R)\ \tilde{R} = \exp\ (\alpha\beta V_R)\ R = \exp\ (\alpha\beta V_Q)\ \tilde{Q} \tag{2.7}$$

By comparing Eqs. 2.1 and 2.7 one can see that the load and resistance factors are, respectively:

$$\gamma = \exp\ (\alpha\beta V_Q) \tag{2.8}$$

$$\Phi = \exp\ (-\alpha\beta V_R) \tag{2.9}$$

The general LRFD design criterion for fatigue is, therefore

$$\Phi\tilde{R} \geq \gamma\ \tilde{Q} \tag{2.10}$$

CHAPTER 3

RESISTANCE DATA

3.1 Analysis Procedures

The mean regression line for each set of fatigue test data was calculated by the least-square method of analysis. The line that best fits the data on a log-log plot of the stress range, f_r, versus the number of cycles to failure, N, is given by [2,3]

$$\log N = b - m \log f_r \tag{3.1}$$

in which the regression coefficients, b and m, determine the intercept and slope, respectively. As part of the analysis, the standard deviation, σ, of the logarithm of cycles to failure was also computed.

To ensure a consistent analysis of the 70 sets of data from 2,502 fatigue tests examined herein, the following guidelines were followed:

1. All runouts were excluded from the regression analysis.
2. The stress ranges used in the analysis were taken at the points in the specimens where the fatigue cracks initiated.
3. If the data tended to "run out" at the low levels of stress range, all data points at that stress range level were excluded from the regression analysis. The runout trend reflects the nonlinear effect of the fatigue limit and would bias the log-log linear regression model given by Eq. 3.1.
4. All other data points were included in the regression analysis. Any test specimens that exhibited unusually long or short cycle lives, but showed no signs of damage or inconsistencies in the testing procedure, were included in the analysis. This was done to ensure that

the best estimate for the variation of test data was obtained.

One typical S-N plot of fatigue test data is shown in Fig. 3.1. All others are included in Appendix A. Each S-N plot contains the data points, the mean regression line and, in most cases, the upper and lower confidence limits at two standard deviations from the mean. The present AISC fatigue specifications are based on the lower confidence limits of the applicable test data.(2,3)

3.2 Equations

The least-squares line is given by:

$$Y = B + m\ X \tag{3.2}$$

and

$$\hat{m} = \sum_{i=1}^{n} (X_i - \overline{X})(Y_i - \overline{Y}) / \sum_{i=1}^{n} (X_i - \overline{X})^2 \tag{3.3}$$

$$\hat{b} = \overline{Y} - m\ \overline{X} \tag{3.4}$$

$$s^2 = \frac{1}{n-2} \sum_{i=1}^{n} [Y_i - (b + m\ X_i)]^2 \tag{3.5}$$

where $\hat{b}$, $\hat{m}$ and s are the least square estimates of the intercept, b, the slope, m, and the standard deviation, σ.

Other useful equations involve the properties of logarithms and the lognormal probability distribution. The following equations are utilized throughout this report.(17)

Base-e lognormal:

$$Y_o = \ln X \tag{3.6}$$

$$\overline{Y}_o = \ln \tilde{X} \tag{3.7}$$

$$\sigma^2_{y_o} = \ln\ (1+V^2_x) \tag{3.8}$$

$$V_x = \ (\exp \sigma^2_{y_o}) - 1 \tag{3.9}$$

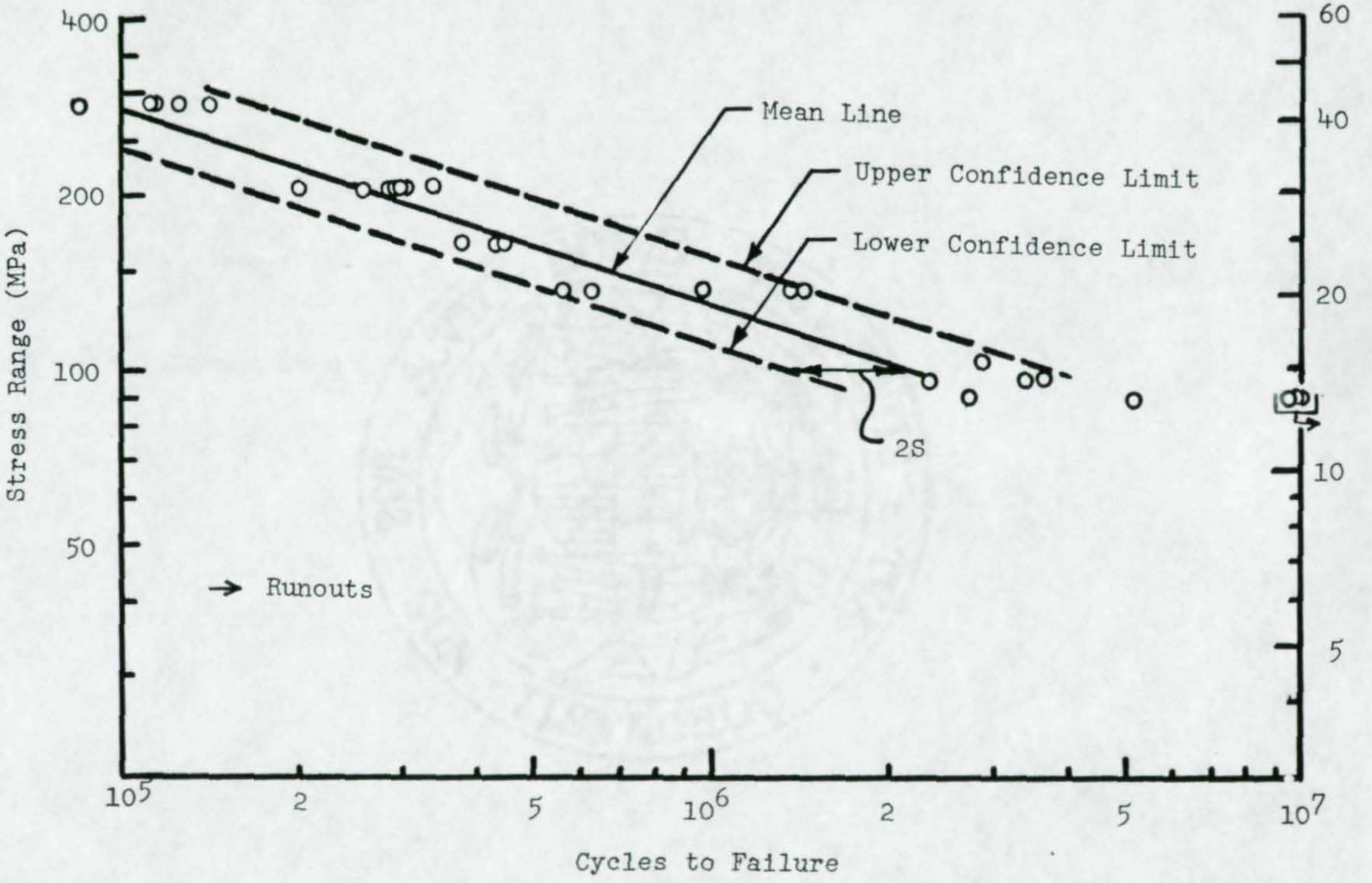

Fig. 3.1 A588 Steel Specimens with Transverse Stiffeners

Base-10 lognormal:

$$Y = \log X = cY_o = 0.434 \ln X \tag{3.10}$$

$$\overline{Y} = \log \tilde{X} \tag{3.11}$$

$$\sigma_y = c\sigma_{y_o} = 0.434\ \sigma_{y_o} \tag{3.12}$$

$$\sigma_y^2 = 0.434 \log (1 + V_x^2) \tag{3.13}$$

$$V_x = (10^{\sigma_y^2/.434} - 1) \tag{3.14}$$

where σ is the standard deviation and V_x is the coefficient of variation of X. Also, the tilde indicates the median, and the bar indicates the mean value.

3.3 Fatigue Notch Factor

The fatigue notch factor, or fatigue strength reduction factor, is defined as the ratio of the fatigue strength of a specimen with no stress concentration to the fatigue strength of a specimen with a stress concentration.[8] To apply this concept to the analysis of fatigue test data, a reference line must be chosen for a detail that has no stress concentration. It seems plausible to select the mean fatigue strength of the Category A plain rolled beams as the reference line for which the fatigue notch factor is defined as unity, $K_f = 1.0$. The fatigue notch factor for any other Category X detail, at a fixed number of cycles, is then given by the vertical distance between the solid and open circular symbols in Fig. 3.2.

$$K_f = \frac{f_{r,A}}{f_{r,X}} \tag{3.15}$$

The values of $f_{r,A}$ and $f_{r,X}$ are calculated by substituting into Eq. 3.1 the fixed number of cycles and the regression coefficients, b and m, given in Tables 3.1 and A.1.

A larger fatigue notch factor indicates a more severe, or critical detail.

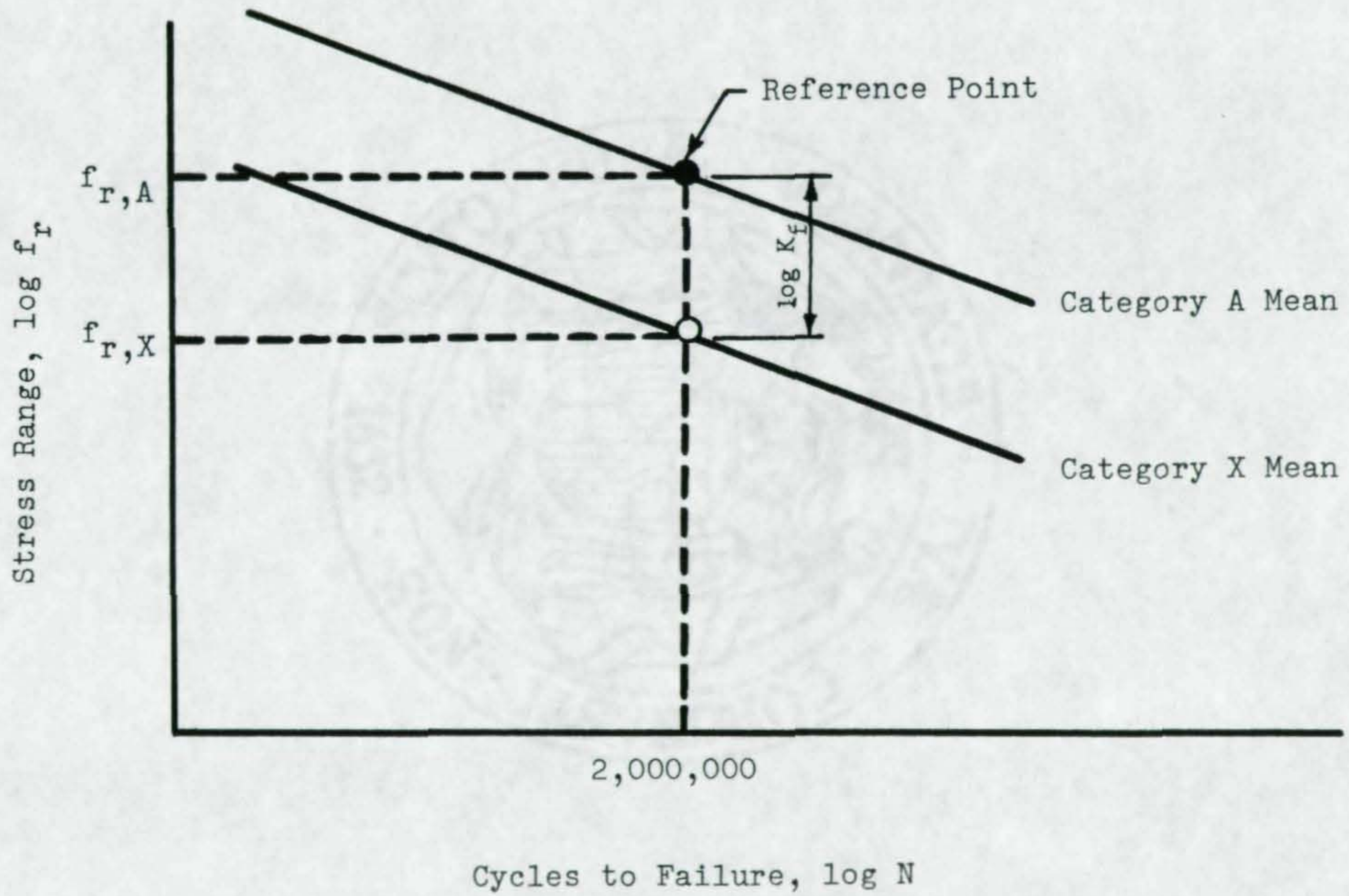

Fig. 3.2 Definition of Fatigue Notch Factor, K_f, at 2,000,000 Cycles

Indeed, as shown in Table 3.1, the fatigue notch factors for the data applicable to the AISC design categories increase from $K_f = 1.0$ for rolled beams to $K_f \leqq 4.19$ for cover plates on thick flanges.

As was stated previously, K_f is calculated at a fixed number of cycles. Its value changes slightly, depending on which number of cycles is used, when the slopes of the two mean regression lines are not equal. The effect of slope on the fatigue notch factor is shown in Table 3.2. For values of slope less than $m \leqq 3.18$ (slope of Category A line), the fatigue notch factor is larger at 2,000,000 cycles than at 500,000 cycles. For values of slope greater than 3.18, K_f is smaller at 2,000,000 cycles than at 500,000 cycles.

For the data base of the AISC fatigue specifications, listed in Table 3.1, K_f changes less than ±3 % at 2,000,000 cycles as compared to the value at 500,000 cycles. The latter, calculated near the center of gravity of the data, is more accurate. The former is used herein because past American studies and the new European specifications give values of fatigue strength at 2,000,000 cycles.

The values of K_f at 500,000 and 2,000,000 cycles are listed in Table A.2 of the Appendix for all 70 sets of fatigue data, and in Table 3.1 for the Category A through E' data.

3.4 Summary of Data

The results of the regression analyses are summarized in Table A.1 of Appendix A which lists the regression coefficients and standard deviation for each of the 70 data sets. The accompanying S-N plots are given in Plots 1-1 to 7-22 of Appendix A.

3.5 Uncertainty of Resistance - Coefficient of Variation

Reliable determination of the uncertainty in resistance is important in

TABLE 3.1 Regression Coefficients and Fatigue Notch Factors for Stress Categories

Category	Type of Detail	Intercept b	Slope m	Standard Deviation S	K_f at 500,000 cycles	K_f at 2,000,000 cycles
A	Rolled beam	11.121	3.178	0.221	1.00	1.00
B	Welded beam	10.870	3.372	0.147	1.49	1.45
C*	Stiffener	10.085	3.097	0.158	1.95	1.97
C	2-in. attachment	10.0384	3.25	0.0628	2.35	2.33
D	4-in. attachment	9.603	3.071	0.108	2.72	2.76
E	Cover plate end	9.2916	3.095	0.1006	3.51	3.55
E' (or G)	Cover plate end t>0.8 in.	9.1664	3.2	0.1943	4.19	4.18

TABLE 3.2 Effect of Slope on Percent Change in K_f at 2,000,000 Cycles as Compared with K_f at 500,000 Cycles

Slope	Change in Fatigue Notch Factor (Percent)
2.00	29
2.25	20
2.50	12
2.75	7
3.00	3
3.18	0
3.25	-1
3.50	-4
3.75	-6
4.00	-9
4.25	-11

LRFD, because the resistance factor, Φ, is a direct function of this uncertainty.

Several factors may contribute to the total uncertainty in fatigue life, such as:

1. Fabrication and workmanship, including assembly and installation procedures.
2. Use of Miner's linear damage rule.
3. Errors in the assumed fatigue model.
4. Scatter of fatigue life data.
5. Errors in the stress analysis, including impact effects.

Ang and Munse have developed a means for combining the variations resulting from these factors using a first order analysis.[10,11] The total coefficient of variation of fatigue life, V_{RN}, is given by Eq. 3.16. The notation has been changed to ensure consistency.

$$V_{RN}^2 = V_N^2 + V_F^2 + V_c^2 + (mV_s)^2 \tag{3.16}$$

where

V_{RN} = total uncertainty in fatigue life, i.e., the coefficient of variation of resistance in terms of cycle life.

V_N = coefficient of variation of fatigue data life about the S-N regression line.

V_F = variation due to errors in the fatigue model and use of Miner's rule.

V_c = variation due to uncertainty in mean intercept of the regression line; includes effects of fabrication and workmanship; also includes the effect of the uncertainty in slope.

V_s = variation due to uncertainty in mean stress range; includes effects of error in stress analysis and choice of impact factor.

m = slope of S-N regression line

In Eq. 3.16, V_s is a variation on stress range and, therefore, must be converted to a variation on life. The conversion, mV_s, is an approximate conversion. The actual conversion makes use of Eqs. 3.13 and 3.14 and the relationship between the standard deviation on log of cycle life and the standard deviation on log of stress range.

$$\sigma_{\log N} = m \, \sigma_{\log f_r} \tag{3.17}$$

The difference between the approximate and actual conversion is, in most cases, about 3 to 5 percent. Therefore, the approximation is accepted because of its simplicity.

To compute the total coefficient of variation of resistance, one must know the values of the terms in Eq. 3.16. The term, V_N, can be found directly from the regression analysis by converting the standard deviation of log of cycle life to coefficient of variation of fatigue life using Eq. 3.14. Ang and Munse proposed values of 0.15, 0.40 and 0.10 for V_F, V_c and V_s, respectively. These numbers were found to be quite reasonable and will be used herein.

The total coefficient of variation, V_{RN}, was computed for all sets of test data. The results are given in Appendix A, Table A.2.

3.6 Resistance Curve

One must establish a means for determining the coefficient of variation of resistance to be used in design. As can be seen from Table A.2, V_{RN} varies for each set of test data. It was thought that a relationship may exist between fatigue strength and coefficient of variation.

A plot of coefficient of variation, V_{RN}, versus fatigue notch factor at 2,000,000 cycles is given in Fig. 3.3 for all 70 data sets. The numbers beside the points identify the corresponding data set in Table A.2.

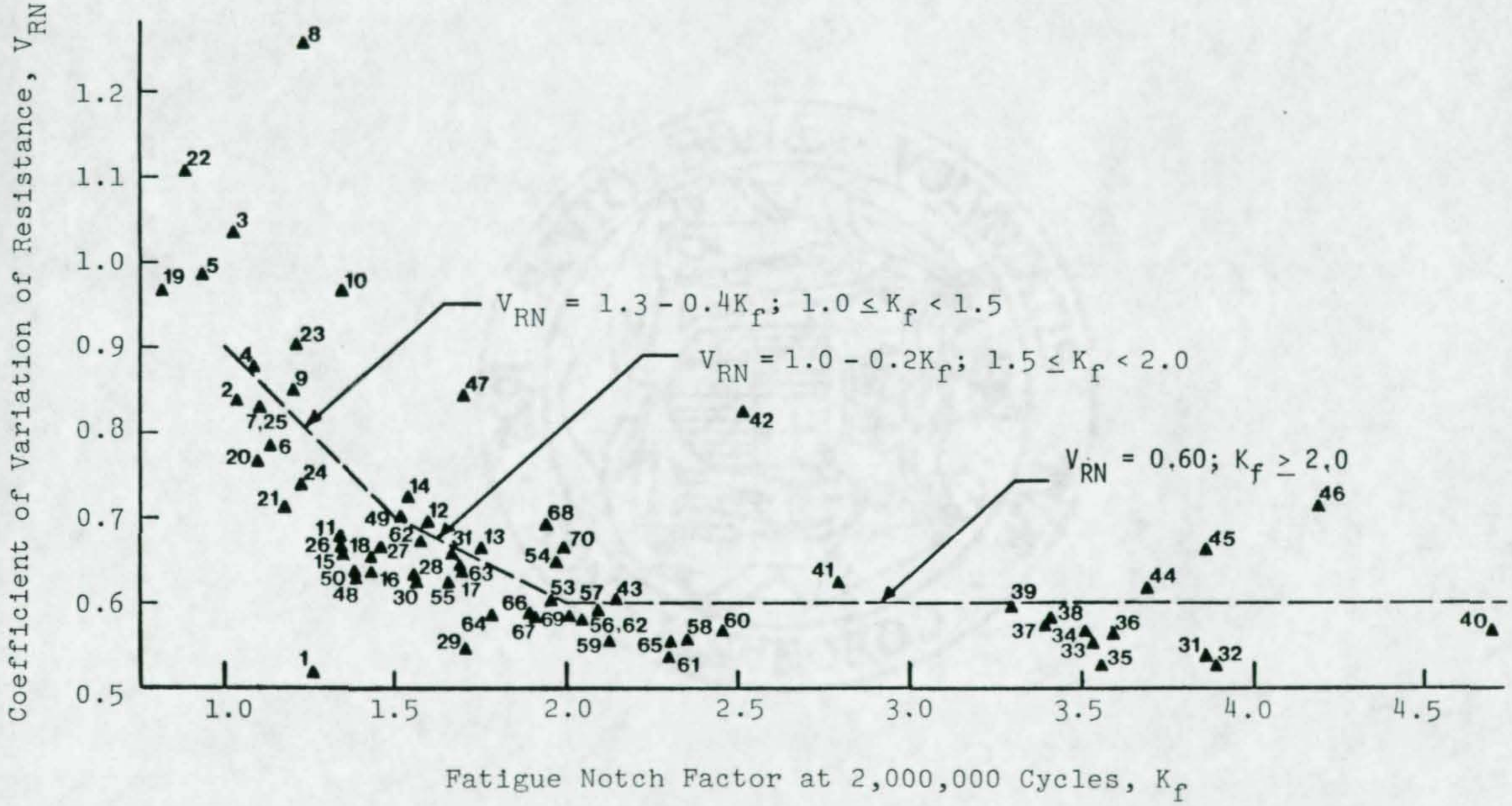

Fig. 3.3 Relationship Between Coefficient of Variation of Resistance on Life and Fatigue Notch Factor

It can be seen from this plot that V_{RN} drops as K_f increases to 2.0 and remains about constant for $K_f > 2.0$. A curve must now be established that gives a good representation of all the data points. It should be noted that the points labeled 5 through 10 and 19 through 26 include data from different fabricators. Therefore, the scatter of the fatigue life data already included some effects of fabrication. Consequently, these points may have been twice penalized for variations due to fabrication techniques. The points labeled 5, 8, 10, 19, 22 and 23 are probably artificially high due to this fact. Points 42 and 47, which also fall high in Fig. 3.3, represent details altered in an attempt to increase their fatigue strengths. The mean fatigue life was increased for both cases (lower K_f), but so did the corresponding coefficient of variation. Taking these factors into account, it was decided to assume about average values of V_{RN} and to tolerate the high points. The choice of three linear curves in Fig. 3.3, as opposed to one nonlinear curve, was made for the sake of simplicity. The equations defining V_{RN} are given by:

$$V_{RN} = 1.3 - 0.4\ K_f\ ;\ 1.0 \leq K_f < 1.5 \tag{3.18a}$$

$$V_{RN} = 1.0 - 0.2\ K_f\ ;\ 1.5 \leq K_f < 2.0 \tag{3.18b}$$

$$V_{RN} = 0.60\ ;\ K_f \geq 2.0 \tag{3.18c}$$

Chapter 4

LOAD DATA

4.1 Presentation of Data

The purpose for examining load data is to determine the load history and the variations that can be expected for different service conditions. The establishment of representative coefficients of variation of load, V_Q, is critical to the development of LRFD specifications. The most common type of fatigue critical structures designed in accordance with the AISC fatigue provisions are cranes and their supports. The lack of available load data for cranes makes the selection of average loads and coefficients of variation of load difficult. However, plausible values can be estimated from load data for other types of structures and from descriptions of crane usage.

For example, much load data are available for highway bridges. Albrecht and Duerling[18] analyzed 106 load histograms from 29 bridges located in 8 states. They determined that the equivalent stress ranges for the histograms, calculated as the root-mean-cube stress range with m = 3,

$$f_{re} = \left[\frac{\Sigma\, n_i\, f_{r_i}^{m}}{\Sigma\, n_i} \right]^{1/m} \qquad (4.1)$$

were lognormally distributed. This reinforces the assumption, made in Chapter 2, that load is lognormally distributed. The corresponding coefficient of variation was found to be 12 percent on stress range.

For light-usage cranes that may lift, on average, an equivalent load of 40 percent of capacity, one can expect larger variations in equivalent load from one plant to another. Heavy-usage cranes mostly lifting an equivalent load of 90 percent of capacity are likely to have smaller variations in load.

A closer look at crane structures and their usages will help to select plausible coefficients of variation.

4.2 Crane Service Classifications

The Crane Manufacturers Association of America (CMAA) has developed service classifications for cranes.[19] These classifications indicate the number of load applications as well as the types of loads to be expected. They are used herein to determine the coefficients of variation for crane structures and the equivalent stress ranges for fatigue design. The service classes are as follows:

Class A1 (Standby Service): This service class covers cranes used in installations such as: power houses, public utilities, turbine rooms, nuclear reactor buildings, motor rooms, nuclear fuel handling, and transformer stations where precise handling of valuable machinery at slow speeds with long idle periods between lifts is required. Rated loads may be handled for initial installation of machinery and for infrequent maintenance.

Class A2 (Infrequent Use): These cranes are used in installations such as: small maintenance shops, pump rooms, testing laboratories, and similar operations where the loads are relatively light, the speeds are slow, and a low degree of control accuracy is required. The loads may vary anywhere from no load to full rated load with a frequency of a few lifts per day or month.

Class B (Light Service): This service covers cranes used in repair shops, light assembly operations, service buildings, light warehousing, etc., where service requirements are light and the speed is slow. The loads may vary from no load to full rated load with an average load of 50 % of rated load, with 2 to 5 lifts per hour averaging 15 feet; not over 50 % of the lifts are at the rated load.

Class C (Moderate Service): This service covers cranes such as those used in machine shops, paper mill machine rooms, etc., where the service requirements are moderate. The cranes handle loads which average 50 % of the rated load, with 5 to 10 lifts per hour averaging 15 feet; not over 50 % of the lifts are at the rated load.

Class D (Heavy Duty): This service covers cranes, usually cab operated, that are used in heavy machine shops, foundries, fabricating plants, steel warehouses, lumber mills, etc. It also covers standard duty bucket and magnet operation where heavy duty production is required, but with no specific cycle of operation. Loads approaching 50 % of the rated load are handled constantly during the working period. High speeds are desirable for this type of service, with 10 to 20 lifts per hour averaging 15 feet; not over 65 % of the lifts are at the rated load.

Class E (Severe Duty Cycle Service): This type of service requires a heavy duty crane capable of continuously handling the rated load at high speed, in repetition throughout a stated period per day, and in a predetermined cycle of operation. Applications include magnet, bucket, magnet-bucket combinations of cranes for scrap yards, cement mills, lumber mills, fertilizer plants, etc., with 20 or more lifts per hour at rated load. The complete cycle of operation should be specified.

Class F (Steel Mill AISE Specification): Cranes in this class are covered by the current issue of the Association of Iron and Steel Engineers' Standard No. 6-1969, Specification for Electric Overhead Traveling Cranes for Steel Mill Service.

4.3 Coefficient of Variation of Load

The coefficient of variation is taken on the equivalent load or, since loads can be assumed to be proportional to stresses, on the equivalent stress range. It indicates the variability of the average (equivalent) load within a class of structures with similar usages. It is not the variability of all loads applied to one specific structure.

It can be seen from the crane service classifications that the loading is better defined the heavier the usage. Classes A1 and A2 cranes for stand-by or infrequent use may have large plant-to-plant load variabilities and co-efficients of variation. But, because the number of cycles expected at the A1 and A2 levels are small (<20,000 cycles), these cranes are not normally fatigue critical. The design would normally be governed by maximum stress, not stress range.

Classes B to F cranes, however, experience enough cycles of load to make fatigue govern the design of the supporting structure. For Class B cranes, the cycle of operation is not specified but the average load and the range of loading are given. Since the range is wide--from no load to full rated load with a 50 % average--one can expect large plant-to-plant variations with type of usage. The choice of high 15-percent coefficient of variation of load seems appropriate. It means that, in 95 % of the cases, the average load lifted by such cranes would roughly vary, from one plant to another, between 35 % and 65 % of the rated load.

At the other extreme, Class E cranes for severe duty service are expected to lift the rated load during each cycle of operation. Since the average load lifted by such cranes is already close to or at maximum, a much smaller coef-ficient of variation, say, 6 % can be assumed.

In summary, Table 4.1 summarizes the recommended coefficients of varia-tion of load, V_Q, for each crane service class. They decrease with increasing severity of service usage, from 0.18 for Class A to 0.06 for Classes E and F.

This range of V_Q values is probably typical of the coefficient of load variation for most structures of interest. They will be used to derive load factors.

4.4 Equivalent Stress Range

The present AISC fatigue specifications conservatively assume that each cycle of loading induces the maximum stress range. The variable amplitude nature of the load history is not considered. In LRFD, which already accounts for most likely sources of variation, it would be inordinately conservative to assume maximum loading all the time. The designer must, instead, estimate the average (equivalent) load based on a careful analysis of the expected loading history. Uncertainties in the estimate of average load can be accounted for by an appropriate choice of coefficient of load variation. The selection of average load and variation is the designer's responsibility. AISC does not specify loads.

The following example illustrates, for crane runways, some of the factors that need to be considered in determining the average load. The equivalent stress range can be calculated with the following equation:

$$f_{re} = T\ L\ f_{r,max} \tag{4.2}$$

The term $f_{r,max}$ is the maximum stress range due to the rated load and impact. The trolley is positioned against the bumper of the crane bridge. The bridge wheels are located along the runway at the critical influence points for the runway detail being designed.

In reality, the trolley is seldom positioned against the bumper. It is, on average, near the center of bridge. One needs to allow, however, for plants with layouts that mostly require load lifting and moving along one half of the bridge. In this case, the trolley would be positioned, on average, at the quarter-point of the bridge. The corresponding trolley position factor for simple span bridges is therefore T = 0.75.

Finally, the lift factor, L, in Eq. 4.2 accounts for the average load handled by the crane, expressed as a percentage of the rated load. It varies from L = 0.50 for Class B to L = 1.0 for Class F, in accordance with the crane service classifications described in Section 4.2.

Table 4.1 summarizes the suggested equivalent stress ranges and the coefficients of variation of load for the CMAA crane service classifications. The values range from $f_{re} = 0.38\ f_{r,max}$ and $V_Q = 0.15$ for light service Class B cranes to $f_{re} = 0.75\ f_{r,max}$ and $V_Q = 0.06$ for heavy duty Class F cranes.

The equivalent stress range for other types of structures should be determined in accordance with the expected loading pattern. For example, highway bridges are, in fact, designed for fatigue assuming single trucks weighing 50 % of the HS20 design truck.[20]

TABLE 4.1 Suggested Equivalent Stress Ranges and Coefficients of Variation of Load for CMAA Crane Service Classifications

CMAA Service Class	Equivalent Stress Range $f_{re} = T\ L\ f_{r,max}$	Coefficient of Variation of Load V_Q
A1, A2	-	0.18
B	$(.75)(.50)f_{r,max} = 0.38\ f_{r,max}$	0.15
C	$(.75)(.50)f_{r,max} = 0.38\ f_{r,max}$	0.12
D	$(.75)(.83)f_{r,max} = 0.62\ f_{r,max}$	0.09
E	$(.75)(1.0)f_{r,max} = 0.75\ f_{r,max}$	0.06
F	$(.75)(1.0)f_{r,max} = 0.75\ f_{r,max}$	0.06

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CHAPTER 5

PROPOSED LOAD AND RESISTANCE FACTORS

5.1 Choice of Splitting Factor

To determine the load and resistance factors, one must select a value of the splitting factor, α, defined by Eq. 2.5. Both γ and Φ are functions of α. See Eqs. 2.8 and 2.9.

Fig. 5.1 shows how the splitting factor, α, varies with the coefficients of variation of resistance, V_R, and load, V_Q. Assuming all coefficients of variation are taken on stress range, the expected range of the coefficient of variation of resistance is approximately 0.15 to 0.25. For the coefficient of variation of load, V_Q, the likely range is 0.06 to 0.18. Within those limits, and to avoid having to calculate a different splitting factor for each combination of the coefficients of variation, V_Q and V_R, a single, slightly conservative value of α should be chosen. A larger value of α gives more conservative load and resistance factors as is apparent from Eqs. 2.8 and 2.9. This study assumes a constant splitting factor, $\alpha = 0.75$, shown with a dashed line in Fig. 5.1.

5.2 Choice of Reliability Index

Ellingwood, et al., investigated the selection of an acceptable reliability index, β, for various structural elements.[12] In their analysis, values of β were calculated based on the current AISC specifications and the available load data. Table 5.1 gives some examples of representative β values for different types of members and load combinations. From these values, target reliabilities were selected for use in LRFD specifications. This process of using present specifications as a basis for establishing acceptable reliabilities is known as calibration. For example, the target reliability

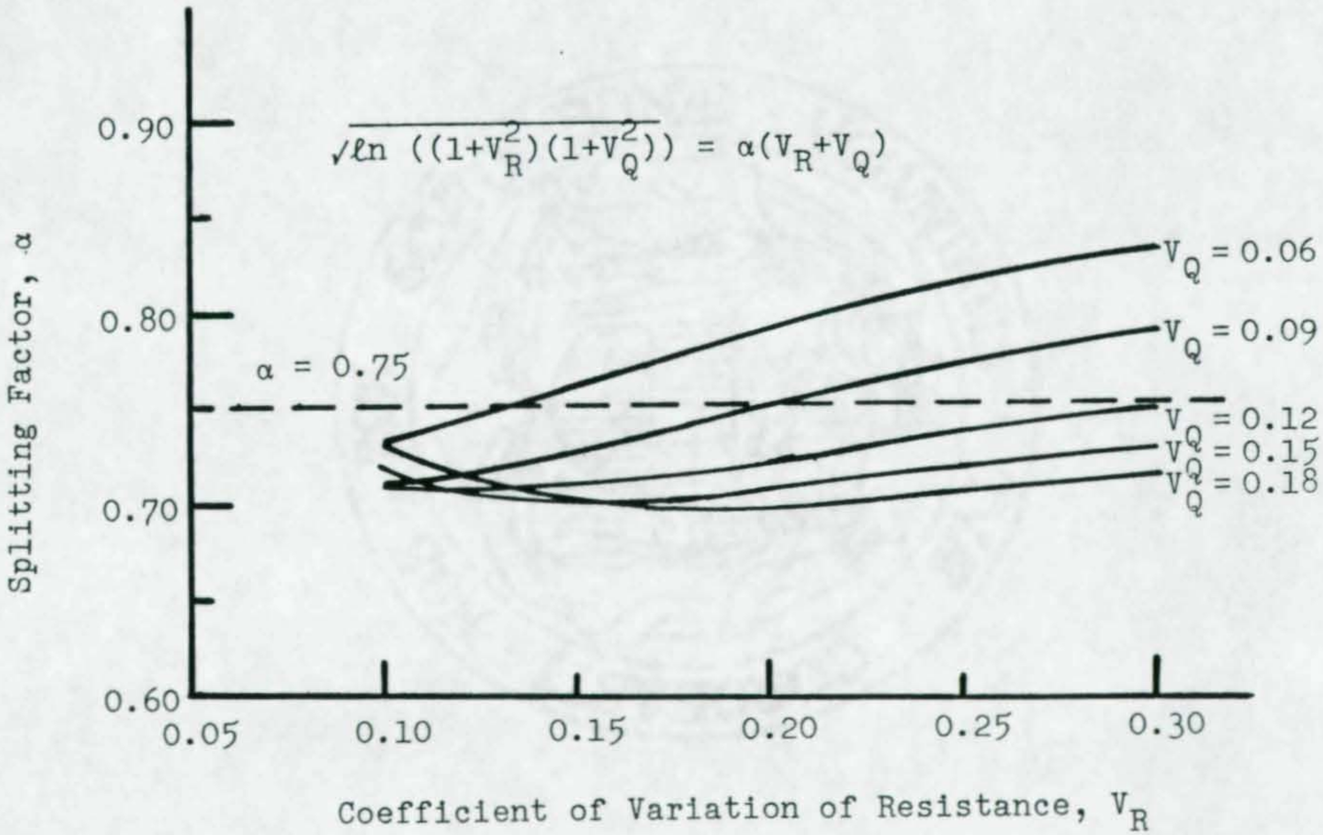

Fig. 5.1 Splitting Factor α as a Function of Coefficients of Variation, V_R and V_Q

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TABLE 5.1 Representative Values of Reliability Index Based on Current Design Specifications (Adapted from Table C.8 of Reference 12)

Member or Element	L/D	S/D	W/D	β
Tension Member, Yield	2	0	0	2.5
Tension Member, Ultimate	2	0	0	3.4
Compact Simple Beam, A_T=1000 ft^2	2	0	0	3.1
Compact Simple Beam, A_T=1000 ft^2	0	2	0	2.8
Compact Simple Beam, A_T=1000 ft^2	0.5	0	2	2.4
Column, A_T=2500 ft^2, λ=0.5	1	0	0	3.1
Column, A_T=2500 ft^2, λ=0.7	1	0	0	2.8
Column, A_T=2500 ft^2, λ=0.7	1	0	1	2.2
Fillet Welds	2	0	0	3.9
A325 Bolts, Tension	2	0	0	4.0
A325 Bolts, Shear	2	0	0	4.4

is $\beta = 3.0$ for members under gravity loading, and $\beta = 3.9$ to 4.4 for connectors. The AISC has also endorsed these target reliabilities for use in their LRFD specifications.

Similarly, one can match the target reliability for fatigue design with the value of the reliability index inherent in designs to the current AISC specification requirements. This will be done in the following for the case of structures supporting cranes. Past satisfactory performance of these structures suggests that the so found target reliability would be adequate.

The reliability index, β, given by Eq. 2.3, is a function of the resistance, the load, and the coefficient of variation of both. In the current AISC specification, the allowable S-N lines for maximum stress range are located two standard deviations on log of cycle life, s, to the left of the mean regression line (resistance curve). The allowable S-N lines for equivalent stress range are shifted downwards by an additional factor on stress range of $TL = f_{re}/f_{r,max}$ (load curve). The mean resistance-to-load ratio, in terms of stress range is, therefore:

$$\frac{\tilde{R}}{\tilde{Q}} = \frac{10^{2s/m}}{f_{re}/f_{r,max}} = \frac{10^{2s/m}}{TL} \tag{5.1}$$

in which s and m are the standard deviation and the slope of the mean regression lines for the stress categories, given in Table 3.1; and TL are the trolley position and load factors for crane classes, given in Table 4.1.

The coefficients of variation of resistance on cycle life are given for all categories by Eq. 3.18 and plotted in Fig. 3.3. Entering these equations with the corresponding fatigue notch factors at 2,000,000 cycles yields V_{RN} for each category. Since the calculations are being performed in terms of stress range, by choice and without changing the results, the coefficient of variation on cycle life must be converted to the coefficient of variation of resistance on stress range, denoted herein V_R. The following calculations will accomplish this conversion:

$$\sigma_{RN} = \sqrt{0.434 \log (1 + V_{RN}^2)} \tag{5.2}$$

$$\sigma_R = \sigma_{RN}/m \tag{5.3}$$

$$V_R = \sqrt{10^{(\sigma_R^2/.434)} - 1} \tag{5.4}$$

In the above equations, the subscript RN denotes a value for resistance on cycle life, N, while the subscript R indicates a value on stress range. In addition, m is taken as the average slope of all the data sets presented in the Appendix, m = 3.19. For example, substituting $K_f = 1.0$ for Category A in Eq. 3.18 gives $V_{RN} = 0.90$. Proceeding to Eqs. 5.2 to 5.4 one obtains $\sigma_{RN} = 0.3344$, $\sigma_R = 0.1048$, and $V_R = 0.2450$. The values of V_R for all categories, calculated in like manner, are summarized in Table 5.2.

The coefficients of variation of load (equivalent stress range), V_Q, are listed in Table 4.1 for class of crane.

Substituting the values of $\tilde{R}/\tilde{Q}$, V_R and V_Q in Eq. 2.3 yields the reliability indicies, β, for all combinations of crane classes and AISC design categories. The results, plotted in Fig. 5.2, show that β varies from 2.15 to 5.64, with a mean of about 4.0. Furthermore, designs for Class E and F crane loadings, which are potentially the most fatigue critical, have the lowest β values. This is just the opposite of what one would like to have, when β is not constant.

Structures designed to the AISC fatigue specification requirements have performed well in the past. Therefore, it seems reasonable to choose a target reliability index of 4.0 for use in calculations of load and resistance factors for fatigue.

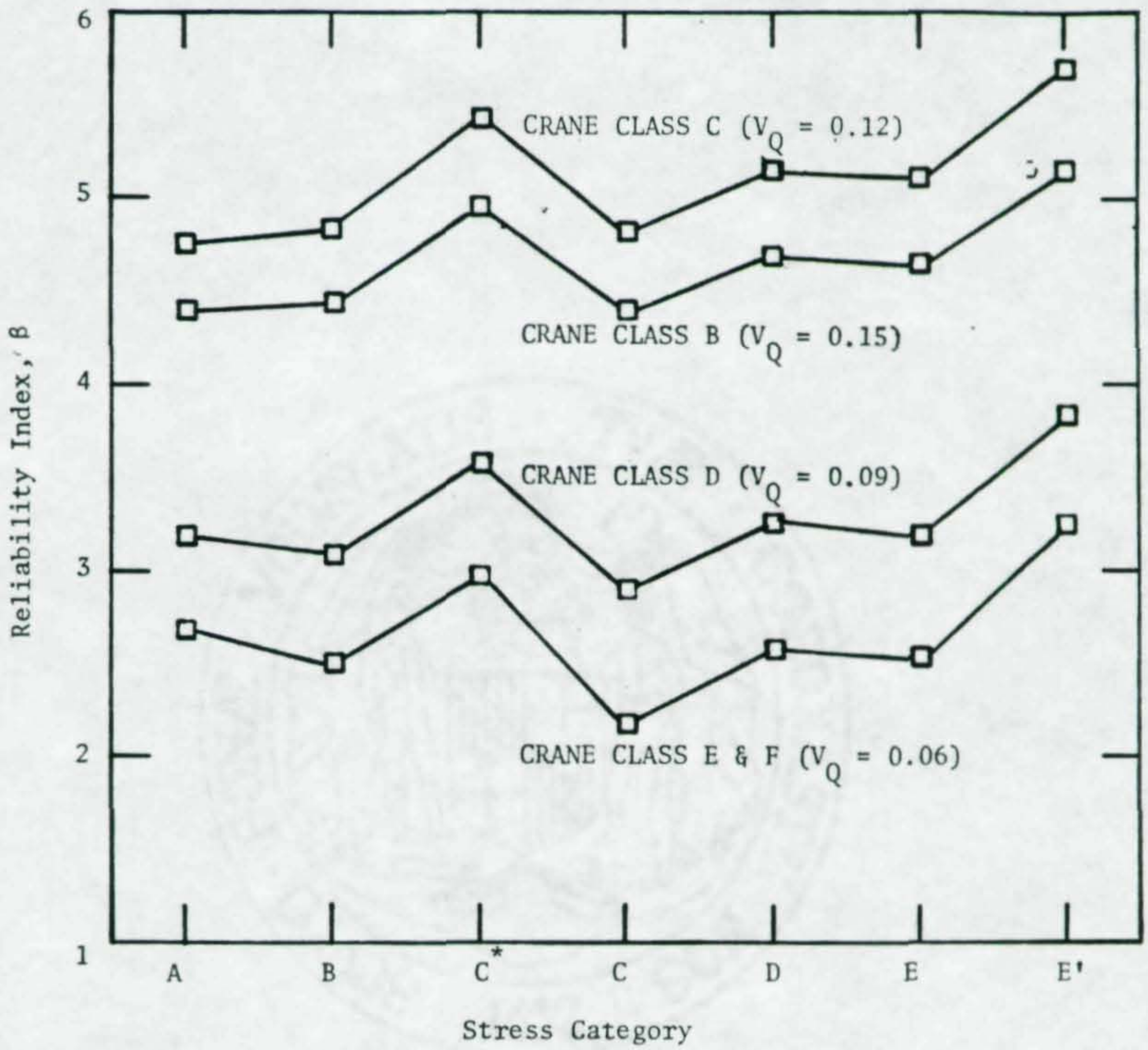


Fig. 5.2 Reliability Index for present AISC Fatigue Design Categories and CMAA Crane Classes

TABLE 5.2 Resistance Factors for AISC Design Categories

Category	Fatigue Notch Factor at 2,000,000 Cycles (Eq. 3.15)	V_{RN} (Eq. 3.18)	V_R (Eq. 5.4)	ϕ (Eq. 2.9)
A	1.00	0.90	0.245	0.48
B	1.45	0.72	0.205	0.54
C*	1.97	0.61	0.178	0.59
C	2.33	0.60	0.175	0.59
D	2.76	0.60	0.175	0.59
E	3.55	0.60	0.175	0.59
E'	4.18	0.60	0.175	0.59

Table 5.3 Load Factors for the Expected Range of V_Q

Coefficient of Variation of Load V_Q	Load Factor γ (Eq. 2.8)
0.06	1.20
0.09	1.31
0.12	1.43
0.15	1.57
0.18	1.72

5.3 Approach No. 1: Actual Load and Resistance Factors

The direct approach to calculating the actual load and resistance factors is to substitute in Eqs. 2.8 and 2.9 the following values: $\alpha = 0.75$, $\beta = 4.0$, V_Q from Table 5.3 and V_R from Table 5.2. The resulting load and resistance factors are listed in the last columns of Tables 5.3 and 5.2, respectively. The former is a constant value for each of five assumed coefficients of variation of load. The latter is a constant for each of the three coefficients of variation of resistance. The splitting factor is kept constant, although it varies slightly with V_Q and V_R.

A fatigue specification that is based on Approach No. 1 would ensure an uniform reliability index, $\beta = 4.0$, for all classes of usage and design categories, as shown in Fig. 5.3. But, it would require a table of 15 pairs of load and resistance factors for the 5 classes of usage (V_Q) and the three groups of categories (V_R). Table 5.4 lists the values of the design paramemeters that would have to be specified. Table 5.7 summarizes the number of design parameters needed in Approach No. 1. This approach is too cumbersome to be incorporated in a design specification.

5.4 Approach No. 2: Constant Load Factor

One possible simplification would be to set one of the factors equal to a constant. Choosing a constant load factor, γ, follows the ANSI and AISC recommended practice. Both of these code writing bodies have chosen a live load factor of $\gamma = 1.6$ for gravity loadings. A single value of γ still allows "material specification writing groups some leeway to adjust Φ for different quality control procedures, minor changes in target reliability, etc."[12] It seems, therefore, advisable to also assume $\gamma = 1.6$ for fatigue loading.

To achieve an uniform load factor, the actual load factors listed in

TABLE 5.4 Approach No. 1 - Actual Load and Resistance Factors

V_Q	Stress Category A	B	C*,C,D,E,E'
0.18	ρ =1.72 Φ =0.48	1.72 0.54	1.72 0.59
0.15	ρ =1.57 Φ =0.48	1.57 0.54	1.57 0.59
0.12	ρ =1.43 Φ =0.48	1.43 0.54	1.43 0.59
0.09	ρ =1.31 Φ =0.48	1.31 0.54	1.31 0.59
0.06	ρ =1.20 Φ =0.48	1.20 0.54	1.20 0.59

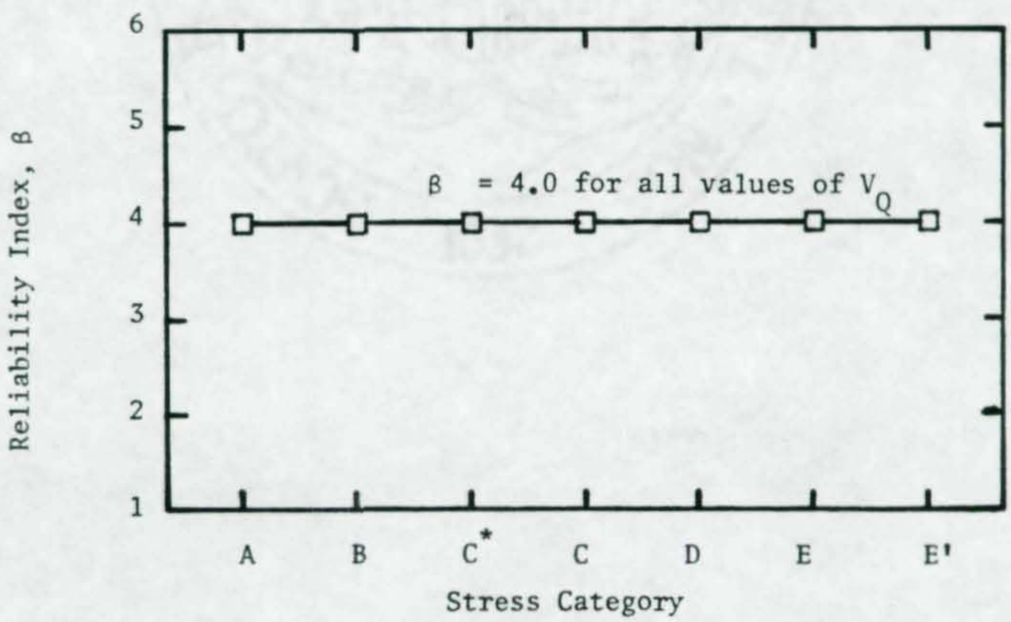


Fig. 5.3 Reliability Index for Approach No. 1 - Actual Load and Resistance Factors

Table 5.4 must be multiplied by the ratio $(1.6/\gamma_{actual})$. The reliability index can be kept constant if the resistance factors are accordingly multiplied by the same ratio $(1.6/\gamma_{actual})$. The results are given in Table 5.5.

A fatigue specification that is based on Approach No. 2 would still ensure an uniform reliability index, $\beta = 4.0$, for all classes of usage and all design categories, as shown in Fig. 5.3. While the number of load factors is reduced to one, there are now 15 resistance factors for the five classes of usage (V_Q) and the three groups of categories (V_R). The value of Φ now depends on both V_R and V_Q. Table 5.5 for Approach No. 2 is not significantly simpler than Table 5.4 for Approach No. 1. The designer would still be faced with 15 pairs of γ and Φ values.

5.5 Approach No. 3: Constant Load and Resistance Factors

From a design point of view the simplest approach would be to have only one pair of load and resistance factors for all classes of usage and design categories. One must pay two prices for such a sweeping simplification. First, all classes of usage have a single coefficient of variation of load, V_Q. Secondly, the reliability index is no longer uniform.

After several trials, the following values were chosen: $\alpha = 0.75$, for reasons explained previously; $\gamma = 1.6$, as recommended by ANSI and ASCE; an average usage class with $V_Q = 0.12$; and a target reliability, $\beta = 4.0$, for the average usage class and design categories C to E', that is, $V_R = 0.175$. See the horizontal line at $\beta = 4.0$ in Fig. 5.5.

Substituting the values of α, β and V_Q in Eqs. 2.8 and 2.9 gives $\gamma = 1.43$ and $\Phi = 0.59$. Multiplying both by the ratio (1.60/1.43) yields the desired single pair of load and resistance factors, $\gamma = 1.6$ and $\Phi = 0.66$. The reliability indices for all other combinations of classes of usage and

TABLE 5.5 Approach No. 2 - Constant Load Factor

V_Q	Stress Category		
	A	B	C*,C,D,E,E'
0.18	ρ =1.6 Φ =0.45	1.6 0.50	1.6 0.55
0.15	ρ =1.6 Φ =0.49	1.6 0.55	1.6 0.60
0.12	ρ =1.6 Φ =0.54	1.6 0.60	1.6 0.66
0.09	ρ =1.6 Φ =0.59	1.6 0.66	1.6 0.72
0.06	ρ =1.6 Φ =0.64	1.6 0.72	1.6 0.79

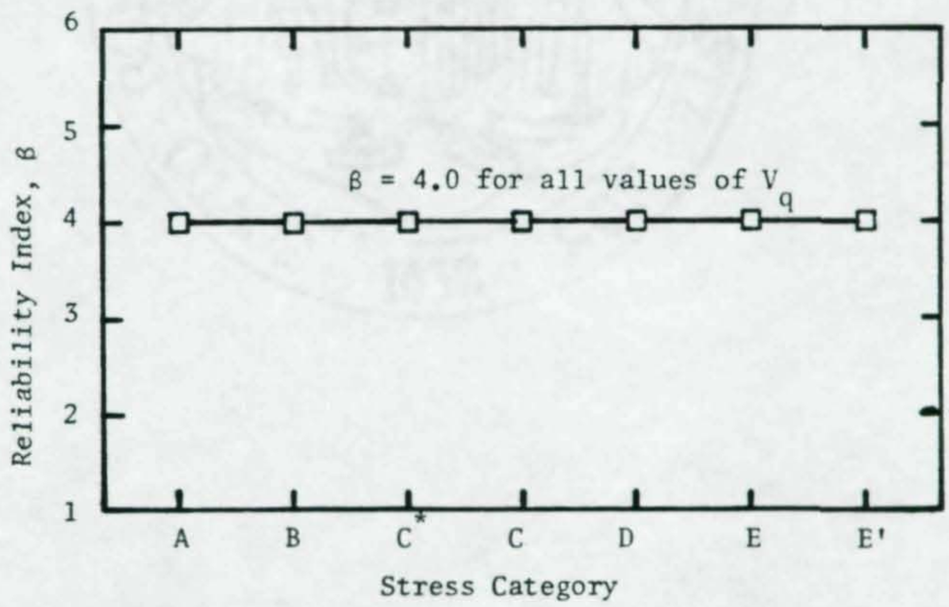


Fig 5.4 Reliability Index for Approach No. 2 - Constant Load Factor

. TABLE 5.6 Approach No 3 - Constant Load and Resistance Factors

V_Q	Stress Category		
	A	B	C^*,C,D,E,E'
0.18			
0.15			
0.12	γ = 1.6 and Φ = 0.66 for all combinations		
0.09			
0.06			

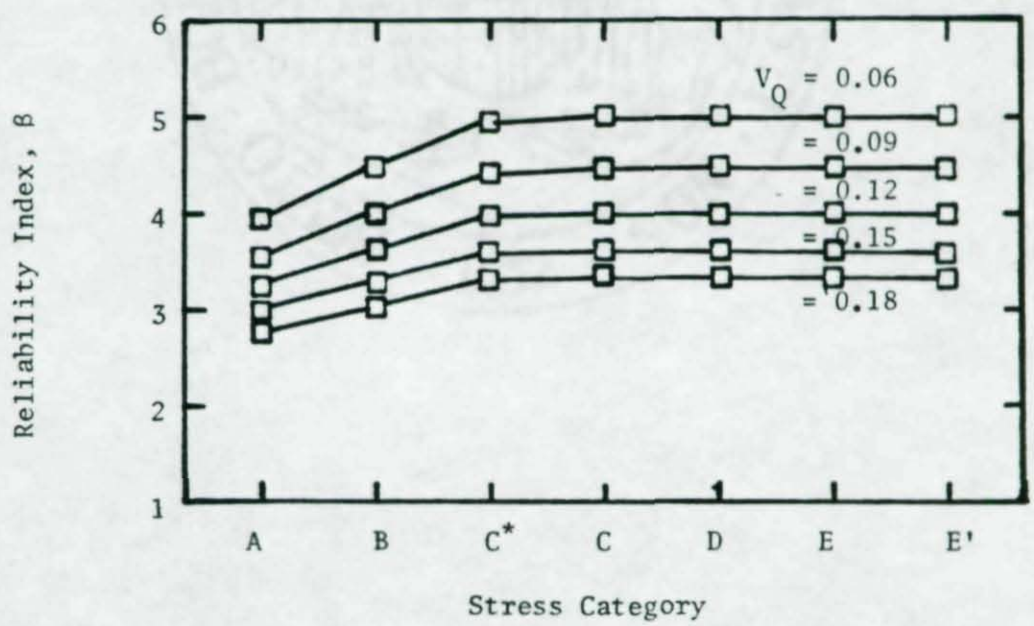


Fig.5.5 Reliability Index for Approach No. 3 - Constant Load and Resistance Factors

design categories can be calculated by rewriting Eq. 2.3 in the form

$$\beta = \sqrt{\frac{\ln(\gamma/\phi)}{\alpha(V_R+V_Q)}} \tag{5.5}$$

Substituting $\gamma/\Phi = 1.6/0.66$, $\alpha = 0.75$, and the applicable values of V_R and V_Q gives the reliability indices plotted in Fig. 5.5. The values are no longer constant. They vary from a minimum of $\beta = 2.8$ for light usage structures with Category A details to $\beta = 5.0$ for heavy usage structures with Category C to E' details. The mean value of β remains near the target reliability, $\beta = 4.0$. The fluctuations are smaller than those reported in Section 5.2 for crane supports designed to the current AISC specifications ($2.15 \leq \beta \leq 5.64$). Furthermore, in contrast to Fig. 5.2, the reliability indices in Approach No. 3 are higher for structures with heavy usage ($V_Q = 0.06$) than for structures with light usage ($V_Q = 0.18$). This is what one would like to have, when β is allowed to vary because of other considerations.

A fatigue specification that is based on Approach No. 3 would require only one pair of load and resistance factors. The achieved simplicity is well worth the moderate loss in uniformity of reliability. Table 5.6 for Approach No. 3 is much simpler than Tables No. 5.4 and 5.5 for Approaches No. 1 and 2, respectively. Table 5.7 summarizes the main features of the three approaches.

It is recommended that Approach No. 3 be chosen for a LRFD fatigue specification.

TABLE 5.7 Summary of Design Parameters

for Approaches No. 1,2 and 3

Design Approach No.	No. of Load Factors	No. of Resistance Factors	No. of Coefficients of Variation	No. of Pairs of γ and Φ Values	Reliability Index
1	5	3	5	15	4.0
2	1	15	5	15	4.0
3	1	1	1	1	varies (2.8 - 5.0)

CHAPTER 6

RECOMMENDED FATIGUE SPECIFICATIONS

As discussed in Chapter 1, the present AISC fatigue specifications utilize a step function approach to define allowable stress ranges. It would be more advantageous to use continuous S-N lines in a LRFD specification. In addition, the assumption of a constant slope for all design categories, and the classification of categories by fatigue notch factors would greatly simplify the specification.

6.1 Constant Slope Design Categories

The average value of slope for the present AISC mean lines, shown in Table 3.1 is approximately 3.2. This value was also nearly equal to the $m = 3.19$ mean value of all the data sets presented in Appendix A. Therefore, $m = 3.2$ will be used as the constant slope value.

The assumption of a constant slope is not enough to define the adjusted regression lines. Therefore, it is additionally assumed that the adjusted regression lines pass through the same point as the actual regression lines at 2,000,000 cycles. These assumptions are conservative and do not alter the fatigue notch factors at 2,000,000 cycles. The regression coefficients for the adjusted mean lines were calculated, and the results are given in Table 6.1.

6.2 Fatigue Notch Factor Approach

The combined use of the fatigue notch factor and the constant slope lines, allows one to derive a single design equation for all categories.

Substituting the new regression coefficients for Category A (Table 6.1) and solving Eq. 3.1 for the stress range gives

TABLE 6.1 Regression Coefficients for Adjusted Constant Slope Mean Lines

Category	Intercept	Slope	Fatigue Notch Factor
A	11.154	3.2	1.00
B	10.637	3.2	1.45
C*	10.211	3.2	1.97
C	9.981	3.2	2.326
D	9.742	3.2	2.76
E	9.393	3.2	3.55
E' (or G)	9.166	3.2	4.18

$$f_{r,A} = (10^{11.154}/N)^{1/3.2} \qquad (6.1)$$

After substituting $f_{r,A}$ into Eq. 3.15 and solving for $f_{r,X}$ one obtains a single resistance curve for any Category X with fatigue notch factor, K_f.

$$f_{r,X} = \frac{3060}{K_f \; N^{(1/3.2)}} \qquad (6.2)$$

The term $f_{r,X}$ is the nominal fatigue strength.

Eq. 6.2 can now be incorporated into a design specification, thus eliminating the previous step function approach.

6.3 Proposed LRFD Specifications for Fatigue

It is recommended that "Appendix G Fatigue" of AISC's Tentative Specifications for Load and Resistance Factor Design, Fabrication and Erection of Structural Steel for Buildings[9] be replaced by Appendix G of the present study.

6.4 Description of Design Procedure

Given below is a step-by-step procedure for fatigue design to the proposed LRFD fatigue specifications listed in Section 6.3.

1. Based on the expected usage of the structure, develop a histogram of nominal stress ranges versus the number of occurrence of each stress range.
2. Determine the applicable stress category from Table G-2.
3. If the maximum nominal stress range in histogram is less than or equal to the fatigue limit listed in Table G-1, no fatigue design is necessary.
4. Calculate the equivalent nominal stress range for the histogram

$$f_{re} = \left[\frac{\Sigma \; n_i \; f_{ri}^m}{N}\right]^{1/m}$$

where f_{re} = nominal equivalent stress range

f_{ri} = nominal stress range for i-th

n_i = number of nominal stress ranges f_{ri}

$N = \Sigma n_i$

$m = 3.2$

5. Calculate the factored equivalent stress range, γf_{re}, where:

   $\gamma = 1.6$ live load factor

6. Obtain the fatigue notch factor, K_f, from Table G-1.

7. Calculate the nominal fatigue strength.

$$f_r = \frac{3060}{K_f N^{1/3.2}}$$

   where

   f_r = nominal fatigue strength

   K_f = fatigue notch factor

   N = number of stress cycles

8. Calculate the factored fatigue strength, $\Phi_f f_r$, where:

   $\Phi_f = 0.66$, fatigue resistance factor

9. If the factored equivalent stress range is less than or equal to the factored fatigue strength, the fatigue design is satisfactory. Otherwise, the member must be redesigned.

CHAPTER 7

CONCLUSIONS

Both, the current AISC specifications[15] and the tentative AISC specifications for load and resistance factor design[9] of structural steel for buildings have fatigue provisions that are based on allowable stress design. An LRFD formulation has so far eluded specification writers. The present study closes this gap.

Section 6.3 outlines a proposed LRFD specification for fatigue in a form that could be directly incorporated in Appendix G of Ref. 9. The taget reliability, $\beta = 4.0$, was determined by calibration against current specifications. This seems reasonable since crane supporting structures designed for fatigue to the provisions of the current specifications have performed well in the past. A single equation gives the continuous resistance curves for all stress categories. The load factor is $\gamma = 1.6$, and the resistance factor is $\Phi = 0.66$ for all loadings and stress categories. They account for the variabilities in fatigue test data, fabrication, modeling and load.

In addition to having an LRFD format, the specification proposed herein features continuous analytical definition of resistance and more realistic variable amplitude loading. In contrast, the present and tentative specifications employ a tabular step-wise definition of allowable stress range ill suited for computer aided design, are based on constant amplitude loading, and account only for the variability in fatigue test data. The provisions for the design of bolts were not changed herein.

It is recommended that the AISC consider the proposed LRFD fatigue specification for adoption in Ref. 9. The proposed specifications could also be used, without changes, for the design of highway steel bridges.

APPENDIX A. ANALYSIS AND PLOTS OF FATIGUE TEST DATA

Table A.1 summarizes the results of the regression analysis, in SI units, of all 70 data sets. To convert the intercept to U.S. customary units subtract (slope) x (log 6.8947) from the intercept. Table A.2 gives the coefficient of variation of resistance and the fatigue notch factors which are used in Fig. 3.3. The tables are followed by the S-N plots for all 70 data sets.

TABLE A.1 Summary of Results from Regression Analysis

S-N Plot No.	Ref.	Type of Detail	Type of Steel	No. of Data Points		Regression Coefficient		Standard Deviation	Coefficient of Correlation
				Included	Excluded	Intercept	Slope		
Base Metal-Plain Rolled and Plain Plate									
2-5	2	Rolled Beam	A36	7	3	10.6521	1.930	.0941	.756
2-6	2	Rolled Beam	A441	12	0	12.8893	2.816	.2613	.503
3-1	3	Rolled Beam	A514	23	5	11.7225	2.313	.3382	.574
3-5	2&3	Rolled Beam	A36,A441, A514	42	8	11.3111	2.158	.2842	.570
7-1	7	Plain Plate Weathered 0-Yrs	SMA50 SMA58	54	19	13.8677	3.176	.3141	-
7-1	7	Plain Plate Weathered 2-Yrs	SMA50 SMA58	49	17	15.1199	3.833	.2167	-
7-3	7	Plain Plate Weathered 4-Yrs	SMA50 SMA58	61	22	15.2587	3.874	.2385	-
7-4	7	Plain Plate Weathered 0-Yrs	SM50 SM58	44	18	10.9809	2.067	.4029	-
7-4	7	Plain Plate Weathered 2-Yrs	SM50 SM58	57	12	14.7022	3.693	.2523	-
7-5	7	Plain Plate Weathered 4-Yrs	SM50 SM58	62	23	13.1205	3.063	.3083	-

TABLE A.1 Summary of Results from Regression Analysis (Continued)

S-N Plot No.	Ref.	Type of Detail	Type of Steel	No. of Data Points		Regression Coefficient		Standard Deviation	Coefficient of Correlation
				Included	Excluded	Intercept	Slope		
Butt Weld/Flange Splice									
2-8	2	Splice-Straight Transition	A36	17	9	13.5380	3.248	.1721	.845
2-9	2	Splice Straight Transition	A441	23	2	11.6766	2.498	.2010	.672
2-10	2	Splice-Straight Transition	A514	24	3	11.6329	2.525	.1825	.791
2-11	2	Splice-Straight Transition	A36,A441, A514	64	14	12.3964	2.811	.2093	.743
2-12	2	Splice-Radiused Transition	A36	19	8	12.4520	2.764	.1734	.788
2-13	2	Splice-Radiused Transition	A441	24	3	12.4768	2.809	.1585	.812
2-14	2	Splice-Radiused Transition	A514	24	3	10.5061	1.979	.1782	.624
2-15	2	Splice-Radiused Transition	A36,A441 A514	67	14	12.0490	2.621	.1741	.752
7-6	2	Butt Weld Weathered 0-Yrs	SMA50 SMA58	45	19	17.4371	4.551	.2860	-
7-6	2	Butt Weld Weathered 2-Yrs	SMA50 SMA58	58	12	16.2795	4.311	.1915	-
7-7	2	Butt Weld Weathered 4-Yrs	SMA50 SMA58	63	19	15.0050	3.808	.1752	-

TABLE A.1 Summary of Results from Regression Analysis (Continued)

S-N Plot No.	Ref.	Type of Detail	Type of Steel	No. of Data Points		Regression Coefficient		Standard Deviation	Coefficient of Correlation
				Included	Excluded	Intercept	Slope		
Butt Weld/Flange Splice (Cont'd)									
7-8	7	Butt Weld Weathered 0-Yrs	SM50 SM58	36	14	16.1164	4.071	.3451	-
7-8	7	Butt Weld Weathered 2-Yrs	SM50 SM58	56	15	14.6795	3.689	.2756	-
7-9	7	Butt Weld Weathered 4-Yrs	SM50 SM58	69	19	15.1260	3.893	.1903	-
7-10	7	Butt Weld Weathered 0-Yrs	SM	30	11	15.3963	3.933	.2373	-
7-10	7	Butt Weld Weathered 3-Yrs	SM	7	4	12.4359	2.754	.1803	-
Plain Welded Beams									
2-1	2	Welded Beam	A36	15	1	15.5871	4.239	.1207	.967
2-2	2	Welded Beam	A441	20	0	13.3538	3.260	.1396	.931
2-3	2	Welded Beam	A514	20	0	11.7066	2.547	.0992	.937
2-4	2	Welded Beam	A36, A441, A514	55	1	13.3706	3.271	.1351	.929
Cover Plate Ends									
2-16	2	End Welded	A36	34	0	11.3876	2.877	.0682	.985
2-17	2	End Welded	A441	34	0	11.3257	2.846	.0563	.991
2-18	2	End Welded	A514	35	0	11.3700	2.804	.0904	.973

TABLE A.1 Summary of Results from Regression Analysis
(Continued)

S-N Plot No.	Ref.	Type of Detail	Type of Steel	No. of Data Points		Regression Coefficient		Standard Deviation	Coefficient of Correlation
				Included	Excluded	Intercept	Slope		
Cover Plate Ends (Cont'd)									
2-19	2	End Unwelded	A36	34	0	11.3801	2.807	.1057	.963
2-20	2	End Unwelded	A441	31	2	11.1062	2.663	.0659	.987
2-21	2	End Unwelded	A514	34	1	10.5543	2.362	.1204	.936
2-22	2	Multiple C.P. End Welded	A36	30	0	12.3073	2.292	.0839	.988
2-23	2	Thick C.P. End Welded	A36	30	0	12.4426	3.369	.0896	.982
2-24	2	Wide C.P. End Welded	A36	30	0	12.7446	3.508	.0963	.981
2-25	2	Wide C.P. End Unwelded	A36	30	0	10.8962	2.735	.1041	.964
4-1	4	Ground Endweld	A36	8	0	14.0188	4.041	.0888	.99
4-2	4	Shot-Peened Endweld	A36	16	0	14.2443	4.064	.2340	.90
4-3	4	Remelted Endweld	A36	16	0	13.1277	3.374	.1129	.96
4-4	4	Ends Welded and Unwelded Constant Amplitude	A36 A514	27	0	11.6368	2.983	.1400	.98
4-5	4	Ends Welded and Unwelded Variable Amplitude	A36 A514	39	6	10.6526	2.460	.1826	.94

TABLE A.1 Summary of Results from Regression Analysis (Continued)

S-N Plot No.	Ref.	Type of Detail	Type of Steel	No. of Data Points		Regression Coefficient		Standard Deviation	Coefficient of Correlation
				Included	Excluded	Intercept	Slope		
Cover Plate Ends (Cont'd)									
4-6	4	Full Size C.P.	A36,A514,A588	18	20	11.8496	3.200	.1943	-
5-1	4	1:3 Tapered Transition	A36	11	7	15.9126	4.523	.230	.841
1-1	1	Retrofitted C.P.	A588	14	0	13.5216	3.261	.1393	.911
1-2	1	Retrofitted C.P. Plate Failures	A588	24	0	15.8328	4.384	.1456	.929
1-3	1	Retrofitted C.P. Flange Failures	A588	15	1	15.7334	4.259	.0838	.978
Stiffeners									
3-6	3	Type 1 Stiffeners	A441,A514	46	6	13.7406	3.490	.1492	.89
3-7	3	Type 2 Stiffeners	A441,A514	22	0	13.1063	3.330	.0874	.95
3-8	3	Type 3 Stiffeners	A441,A514	41	3	13.5342	3.5053	.1024	.95
3-9	3	Type 1,2, and 3 Stiffeners	A441,A514	109	8	12.6821	3.097	.1581	.86
6-1	6	Automatically Welded Stiffeners	A588	12	1	13.1998	3.23	.1360	-
6-2	6	Manually Welded Stiffeners	A588	21	4	12.5159	3.04	.1073	-
7-11	7	Stiffeners Weathered 0-Yrs	A588	24	5	12.4209	3.007	.1172	.97
7-12	7	Weathered 2-Yrs Continuously	A588	15	5	11.6900	2.717	.1008	.97

TABLE A.1 Summary of Results from Regression Analysis (Continued)

S-N Plot No.	Ref.	Type of Detail	Type of Steel	No. of Data Points Included	No. of Data Points Excluded	Regression Coefficient Intercept	Regression Coefficient Slope	Standard Deviation	Coefficient of Correlation
Stiffeners (Continued)									
7-13	7	Weathered 2-Yrs Alternately	A588	12	4	11.9734	2.794	.0885	.98
7-14	7	Weathered 4-Yrs Continuously	A588	15	5	11.2695	2.529	.1172	.95
7-15	7	Weather 4-Yrs Alternately	A588	12	4	11.8110	2.762	.0737	.98
7-16	7	Weathered 0-Yrs	A588	12	0	13.2698	3.230	.1693	-
7-17	7	Weathered 3-Yrs	A588	16	0	14.6166	3.918	.1211	-
Attachments									
3-2	3	9132 in. Attach.	A441	14	5	13.8619	3.593	.0805	.976
3-3	3	2 in. Attach.	A441	14	0	12.7626	3.246	.0627	.989
7-18	7	Attachments Weathered 0-Yrs	A588	17	7	13.3987	3.416	.0939	.98
7-19	7	Weathered 2-Yrs Continuously	A588	15	0	13.3419	3.398	.0905	.98
7-20	7	Weathered 2-Yrs Alternately	A588	8	0	11.6714	2.597	.1962	.74
7-21	7	Weathered 4-Yrs Continuously	A588	20	0	12.9336	3.234	.0976	.99
7-22	7	Weathered 4-Yrs Alternately	A588	8	0	12.4304	2.982	.1725	.82

TABLE A.2 Fatigue Notch Factor and Coefficient of Variation of Data Presented in Table A.1

S-N Plot No.	Symbol in Fig.3.3	Standard Deviation	Coefficient of Variation of Test Data (Eq. 3.14)	mV_s	Total Coefficient of Variation V_{RN} (Eq. 3.16)	Fatigue Notch Factor at 500,000	Fatigue Notch Factor at 2,000,000
2-5	1	.0941	.2194	.1930	.5176	0.95	1.26
2-6	2	.2613	.6608	.2816	.8357	0.98	1.04
3-1	3	.3382	.9136	.2313	1.0347	0.87	1.03
3-5	4	.2842	.7313	.2158	.8740	1.08	1.08
7-1	5	.3141	.8293	.3176	.9854	0.94	0.94
7-2	6	.2167	.5319	.3833	.7825	1.22	1.13
7-3	7	.2385	.5935	.3874	.8275	1.19	1.10
7-4	8	.4029	1.16	.2067	1.253	0.98	1.23
7-4	9	.2523	.6338	.3693	.8489	1.28	1.20
7-5	10	.3083	.8098	.3063	.9654	1.32	1.35
2-8	11	.1721	.4122	.3248	.6767	1.35	1.34
2-9	12	.2010	.4886	.2498	.6954	1.42	1.60
2-10	13	.1825	.4394	.2525	.6628	1.57	1.75
2-11	14	.2093	.5114	.2811	.7232	1.45	1.54
2-12	15	.1734	.4158	.2764	.6571	1.26	1.35
2-13	16	.1585	.3775	.2809	.6355	1.36	1.43
2-14	17	.1782	.4283	.1979	.6365	1.30	1.70
2-15	18	.1741	.4177	.2621	.6524	1.32	1.43
7-6	19	.2860	.7371	.4551	.9659	0.92	0.81
7-6	20	.1915	.4634	.4311	.7636	1.23	1.10
7-7	21	.1752	.4206	.3808	.7102	1.19	1.18
7-8	22	.3451	.9384	.4071	1.1088	0.97	0.88
7-8	23	.2756	.7045	.3689	.9027	1.29	1.21

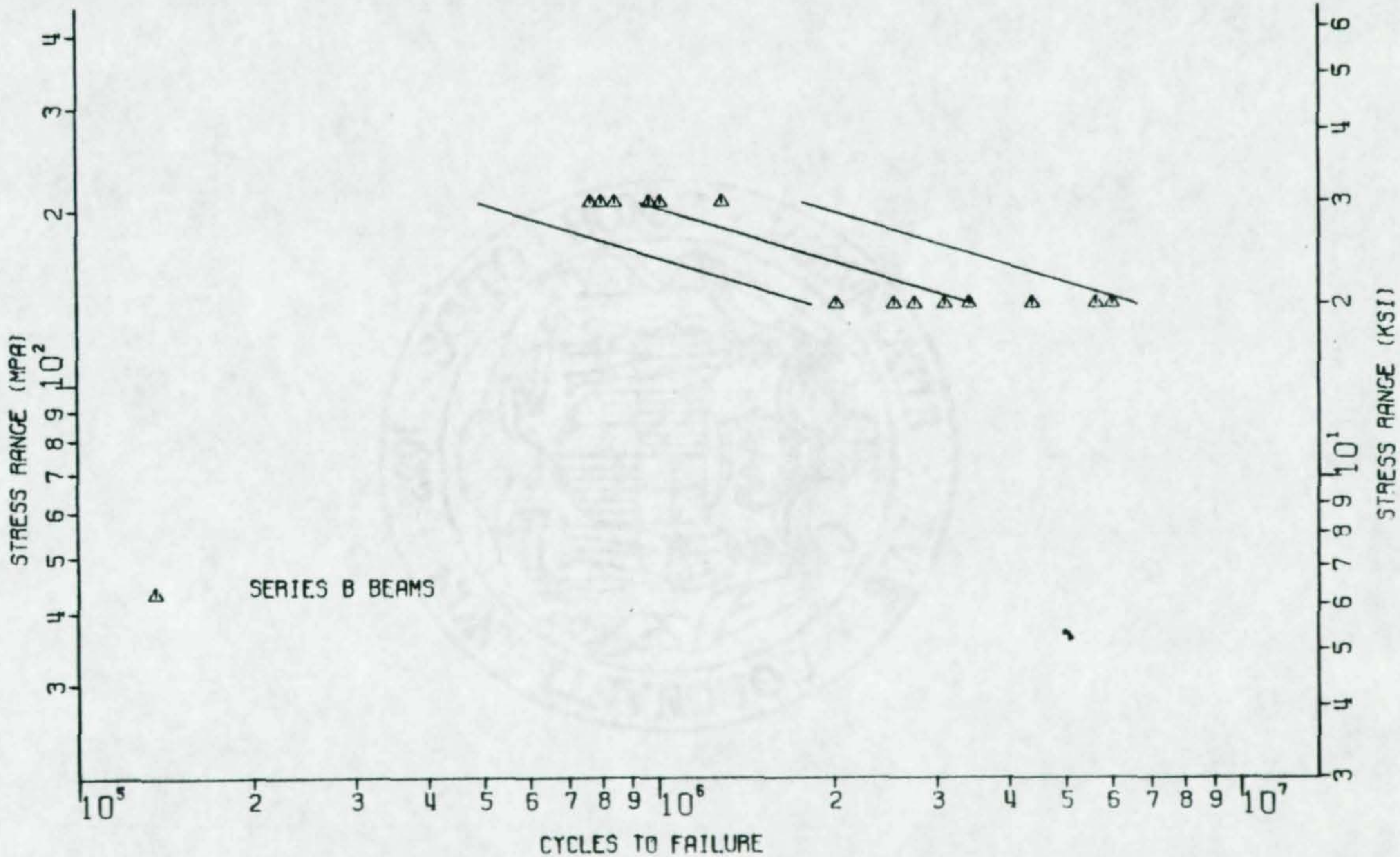
TABLE A.2 Fatigue Notch Factor and Coefficient of Variation of Data Presented in Table A.1 (Continued)

S-N Plot No.	Symbol in Fig.3.3	Standard Deviation	Coefficient of Variation of Test Data (Eq. 3.14)	mV_s	Total Coefficient of Variation V_{RN} (Eq. 3.16)	Fatigue Notch Factor at 500,000	2,000,000
7-9	24	.1903	.4603	.3893	.7389	1.33	1.23
7-10	25	.2373	.5901	.3933	.8279	0.88	1.10
7-10	26	.1803	.4339	.2754	.6683	1.25	1.34
2-1	27	.1207	.2833	.4239	.6652	1.63	1.46
2-2	28	.1396	.3299	.3260	.6306	1.57	1.56
2-3	29	.0992	.2315	.2547	.5486	1.53	1.71
2-4	30	.1351	.3186	.3271	.6253	1.58	1.56
2-16	31	.0682	.1580	.2877	.5387	3.69	3.86
2-17	32	.0563	.1302	.2846	.5295	3.69	3.89
2-18	33	.0904	.2104	.2804	.5526	3.33	3.53
2-19	34	.1057	.2470	.2807	.5677	3.32	3.51
2-20	35	.0659	.1526	.2663	.5260	3.27	3.55
2-21	36	.1204	.2826	.2362	.5642	3.08	3.59
2-22	37	.0839	.1950	.3292	.5735	3.45	3.39
2-23	38	.0896	.2086	.3369	.5826	3.49	3.40
2-24	39	.0963	.2245	.3508	.5966	3.44	3.30
2-25	40	.1041	.2433	.2735	.5626	4.41	4.73
4-1	41	.0888	.2067	.4041	.6233	3.06	2.79
4-2	42	.2340	.5806	.4064	.8275	2.77	2.52
4-3	43	.1129	.2645	.3374	.6052	2.20	2.15
4-4	44	.1400	.3310	.2983	.6173	3.58	3.69
4-5	45	.1826	.4399	.2460	.6607	3.40	3.86
4-6	46	.1943	.4709	.320	.7118	4.19	4.18

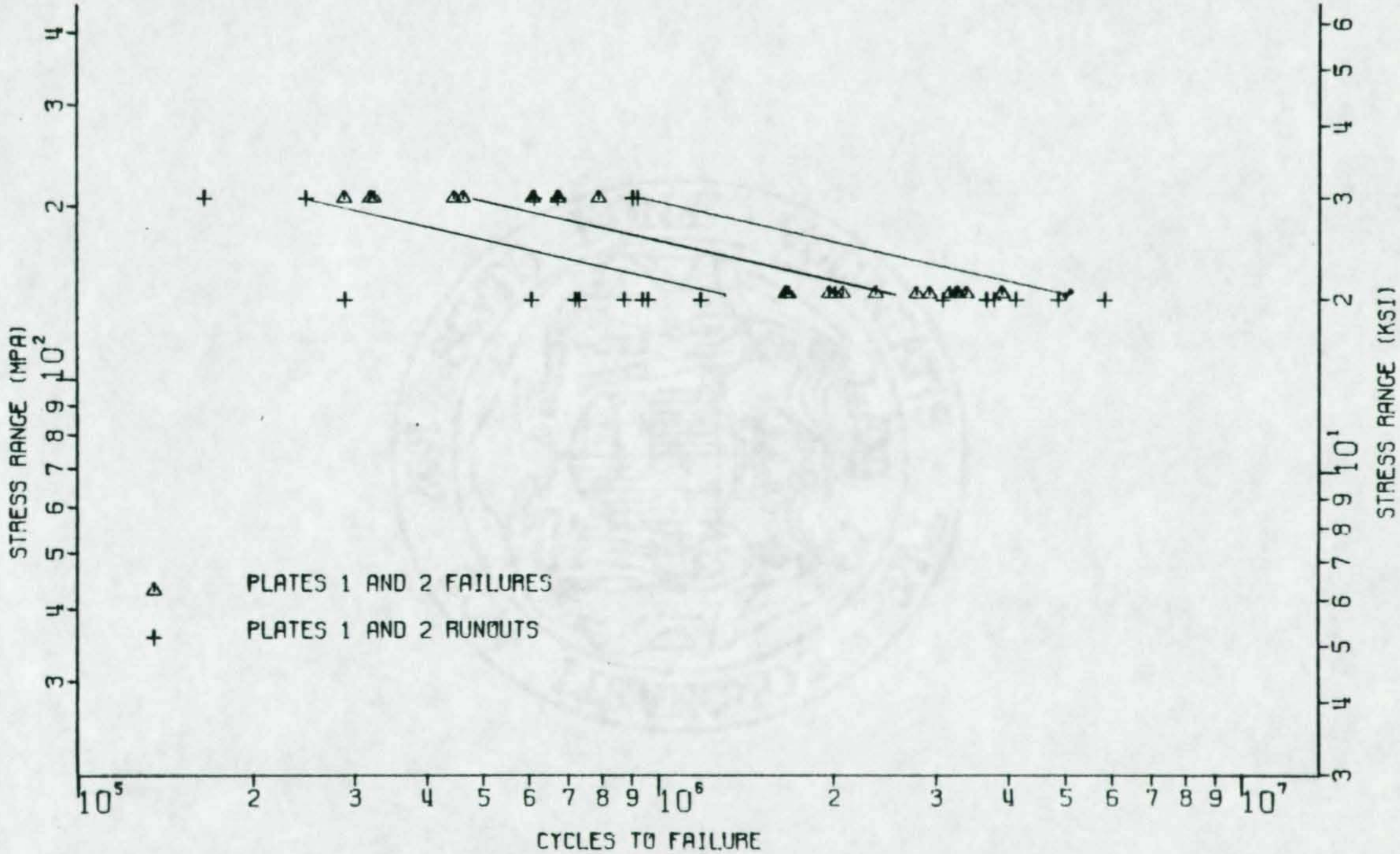
TABLE A.2 Fatigue Notch Factor and Coefficient of Variation of Data Presented in Table A.1 (Continued)

S-N Plot No.	Symbol in Fig.3.3	Standard Deviation	Coefficient of Variation of Test Data (Eq. 3.14)	mV_s	Total Coefficient of Variation V_{RN} (Eq. 3.16)	Fatigue Notch Factor at 500,000	2,000,000
5-1	47	.230	.5692	.4523	.8432	1.93	1.70
1-1	48	.1393	.3292	.3261	.6302	1.40	1.38
1-2	49	.1456	.3449	.4384	.7026	1.71	1.52
1-3	50	.0838	.1948	.4259	.6339	1.54	1.38
3-6	51	.1492	.3541	.3490	.6555	1.74	1.67
3-7	52	.0874	.2034	.3330	.5786	2.09	2.05
3-8	53	.1024	.2392	.3505	.6022	2.04	1.96
3-9	54	.1581	.3766	.3097	.6483	1.95	1.97
6-1	55	.1360	.3211	.323	.6244	1.67	1.66
6-2	56	.1073	.2510	.304	.5813	2.00	2.05
7-11	57	.1172	.2749	.3007	.5903	2.04	2.09
7-12	58	.1008	.2353	.2717	.5583	2.18	2.35
7-13	59	.0885	.2060	.2797	.5506	2.00	2.12
7-14	60	.1172	.2749	.2529	.5675	2.19	2.46
7-15	61	.0737	.1710	.2762	.5367	2.15	2.29
7-16	62	.1693	.4053	.3230	.6746	1.59	1.58
7-17	63	.1211	.2345	.3918	.6457	1.85	1.71
3-2	64	.0865	.1671	.3593	.5887	1.87	1.78
3-3	65	.0627	.1451	.3246	.5558	2.34	2.32
7-18	66	.0939	.2188	.3416	.5891	1.95	1.89
7-19	67	.0965	.2107	.3398	.5851	1.98	1.92
7-20	68	.1962	.4760	.2597	.6903	1.75	1.94
7-21	69	.0976	.2277	.3234	.5822	2.03	2.02
7-22	70	.1725	.4135	.2982	.6651	1.94	1.99

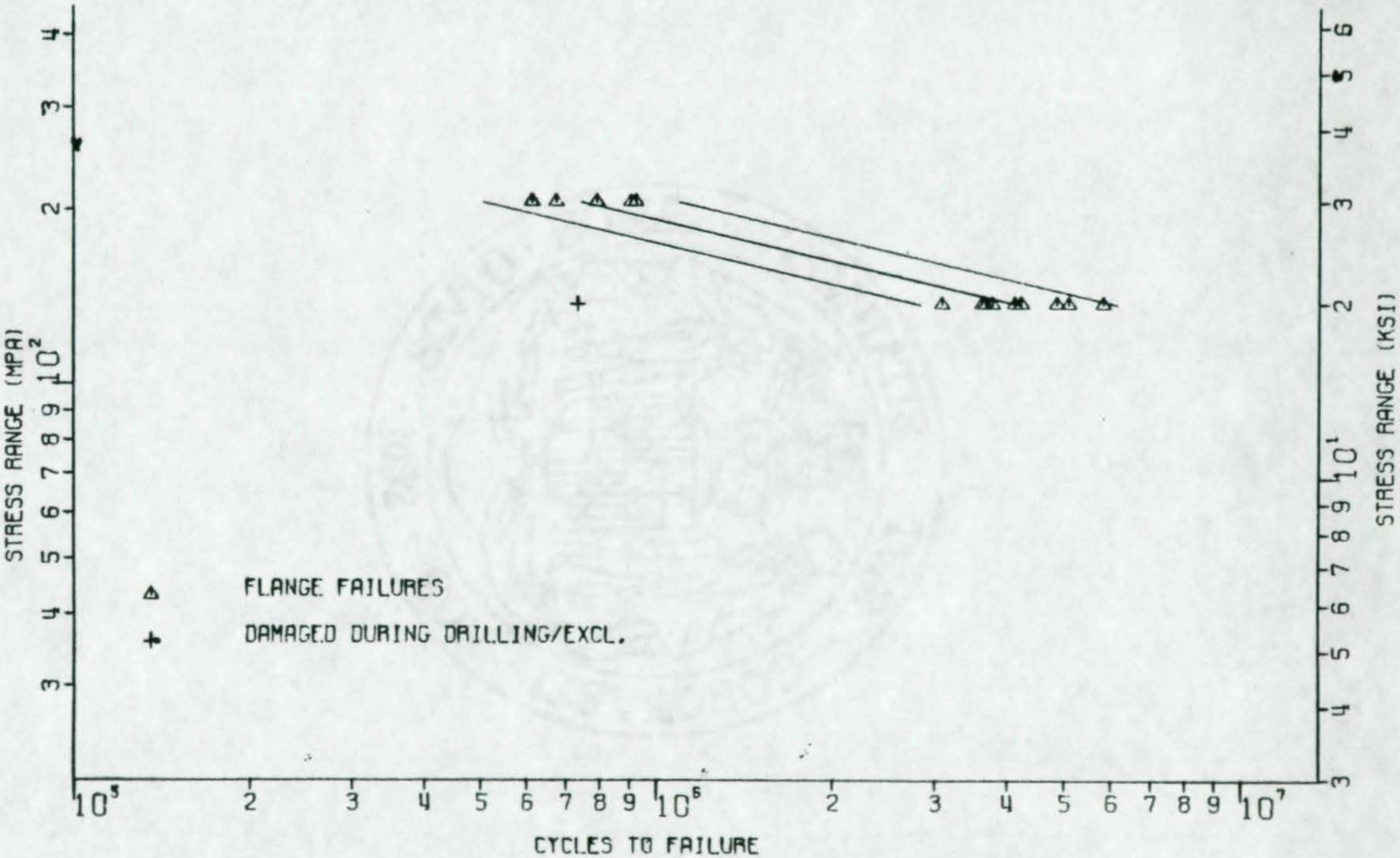
The 70 S-N plots No. 1-1
to 7-22 are available upon request.



PLOT 1-1 FATIGUE TEST DATA FOR SERIES B BEAMS-RETROFITTED BEFORE CYCLING. DATA FROM REF. 1.

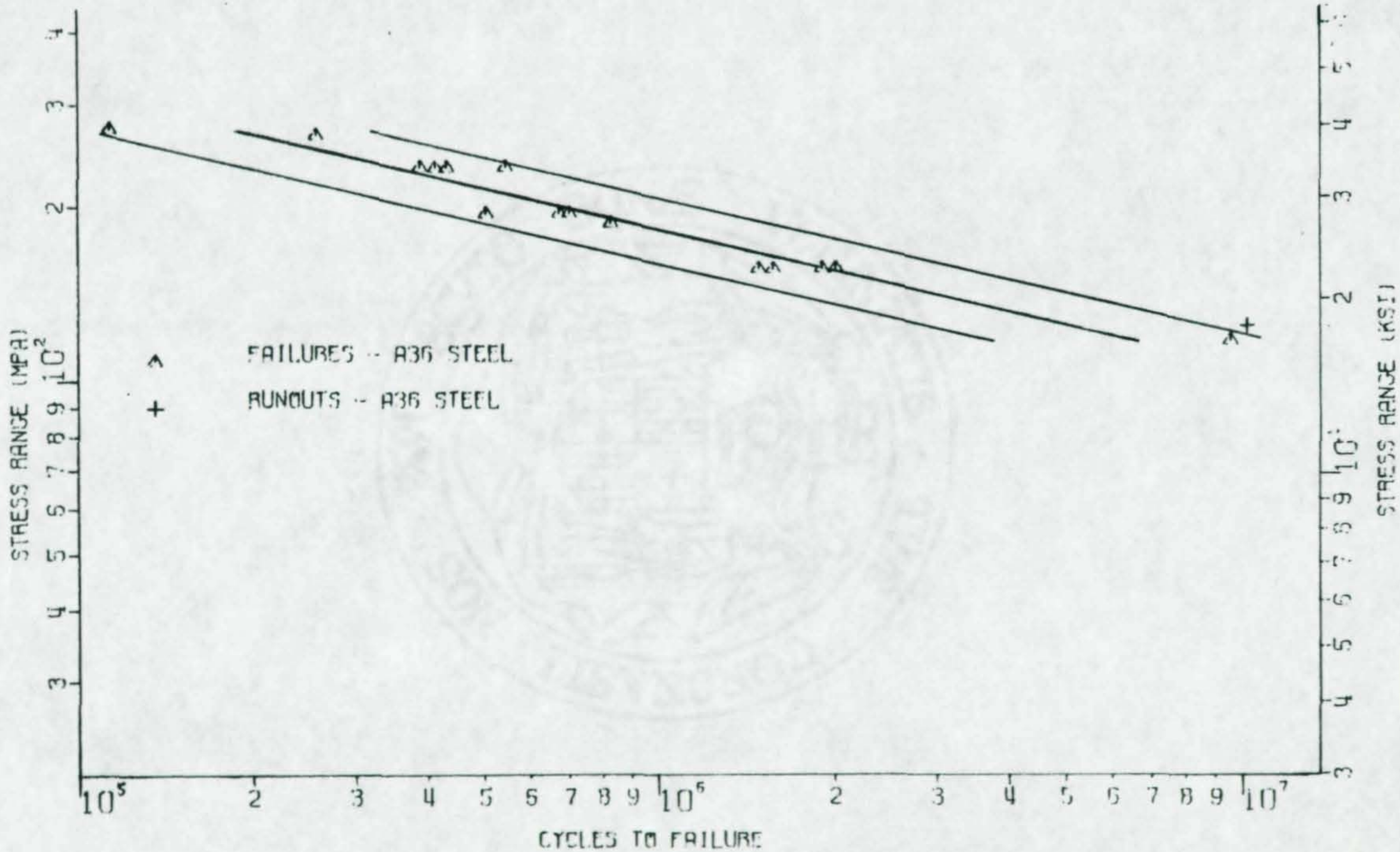


PLOT 1-2 FATIGUE TEST DATA OF PLATE FAILURES FOR SERIES A BEAMS WITH TENSION FLANGE HALF OF FULLY CRACKED AT TIME OF RETROFITTING. DATA FROM REF. 1.

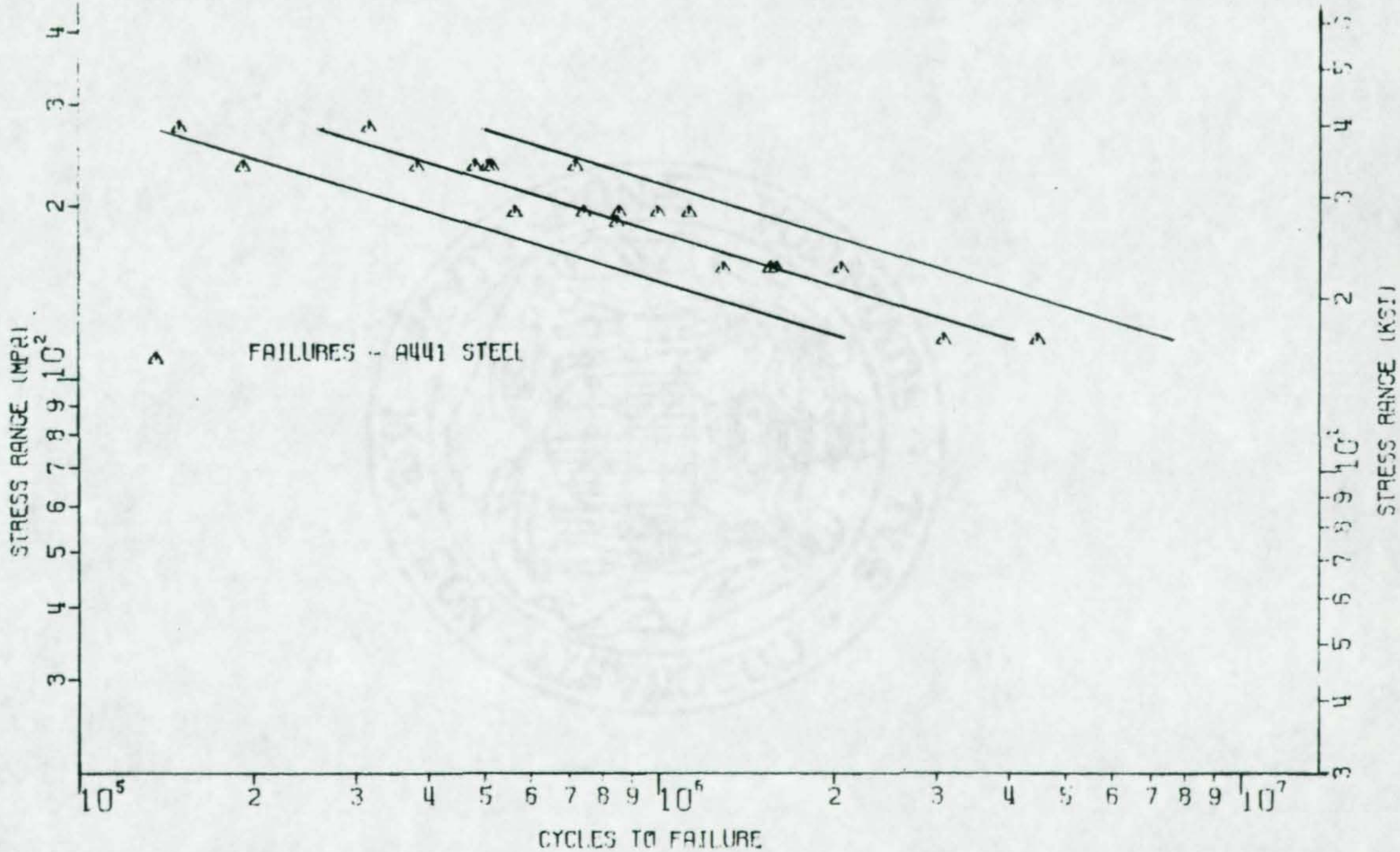


PLOT 1-3 FATIGUE TEST DATA OF FLANGE FAILURES FOR SERIES A SPECIMENS WITH FLANGE HALF OR FULLY CRACKED AT TIME OF RETROFITTING. DATA FROM REF. 1.

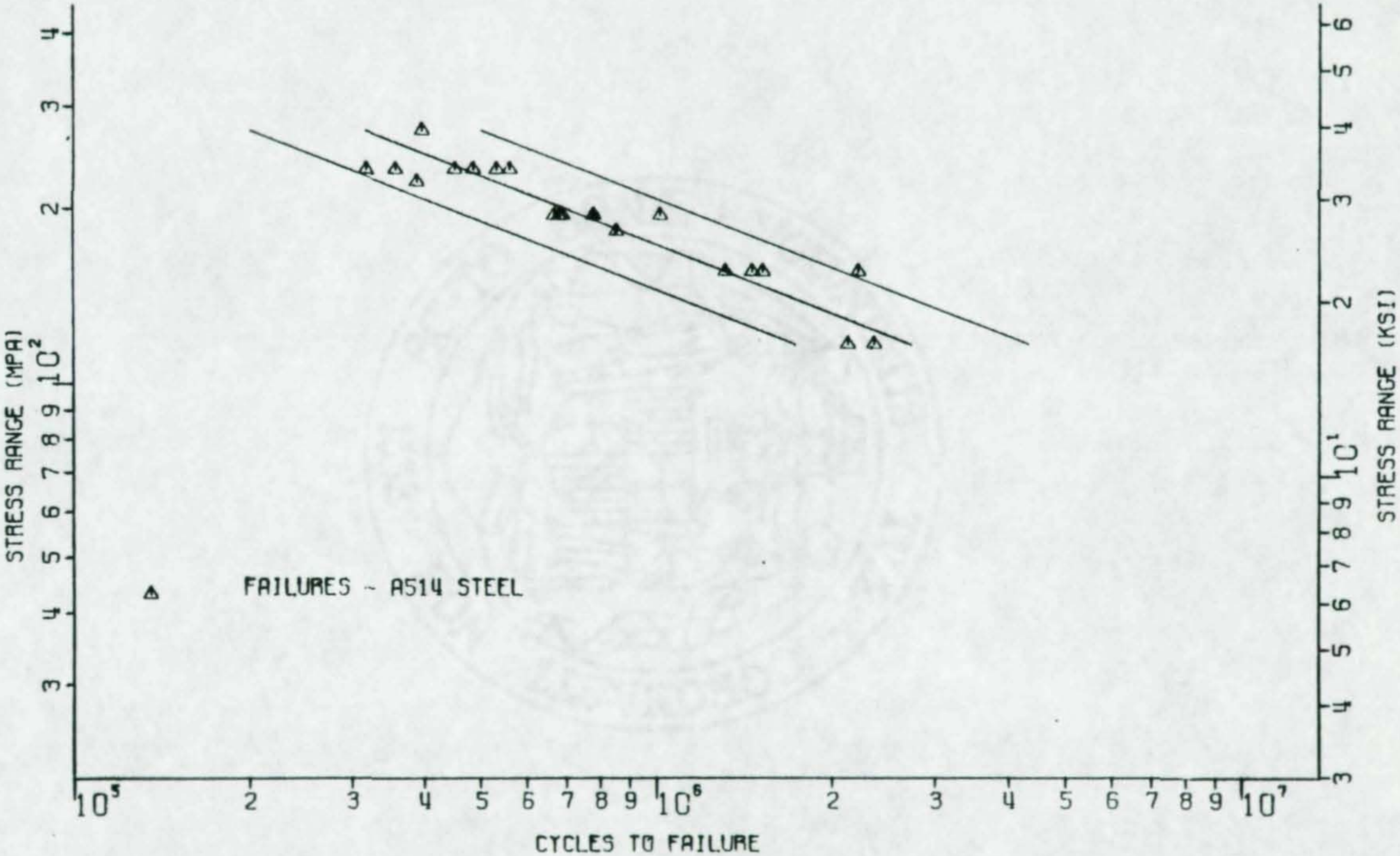
00782



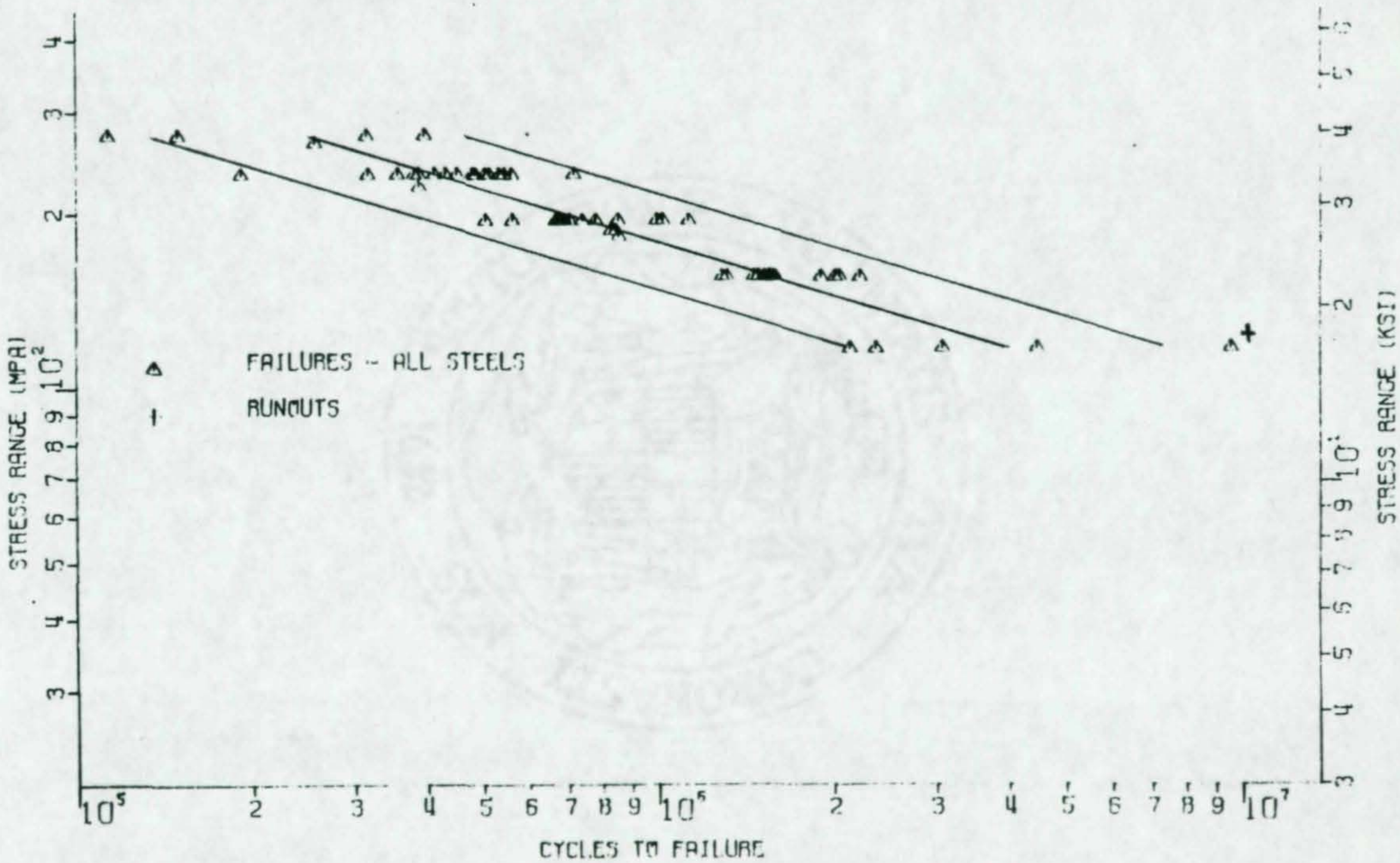
PLOT 2-1 FATIGUE TEST DATA FOR PLAIN WELDED BEAMS. A36 STEEL. WITH STRESS RANGE REDUCED TO THE INSIDE FIBER OF FLANGE. DATA FROM REF. 2 (NCHRP 102).



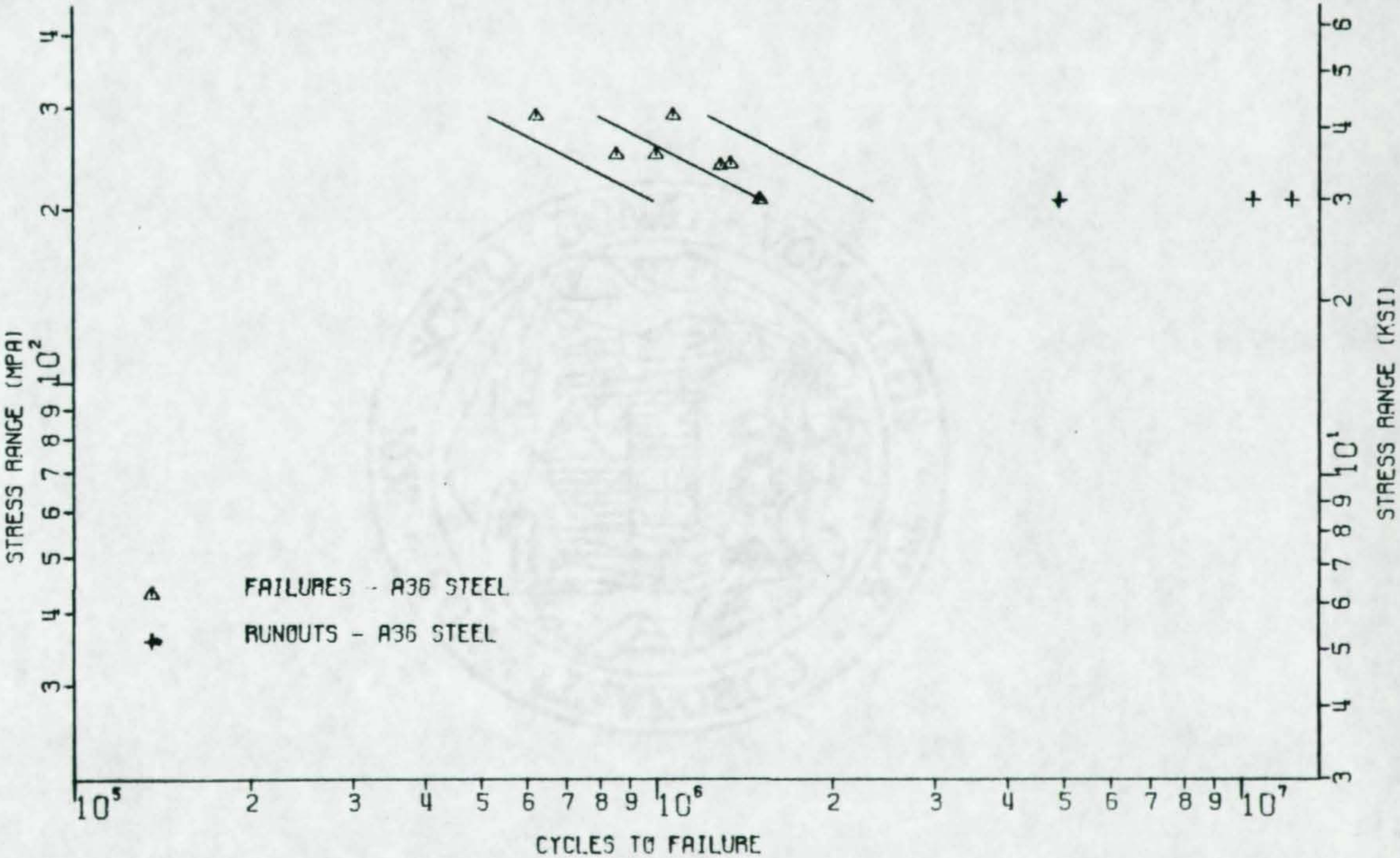
PLOT 2-2 FATIGUE TEST DATA FOR PLAIN WELDED BEAMS, A441 STEEL, WITH STRESS RANGE REDUCED TO INSIDE FIBER OF FLANGE. DATA FROM REF. 2 (NCHRP 102).



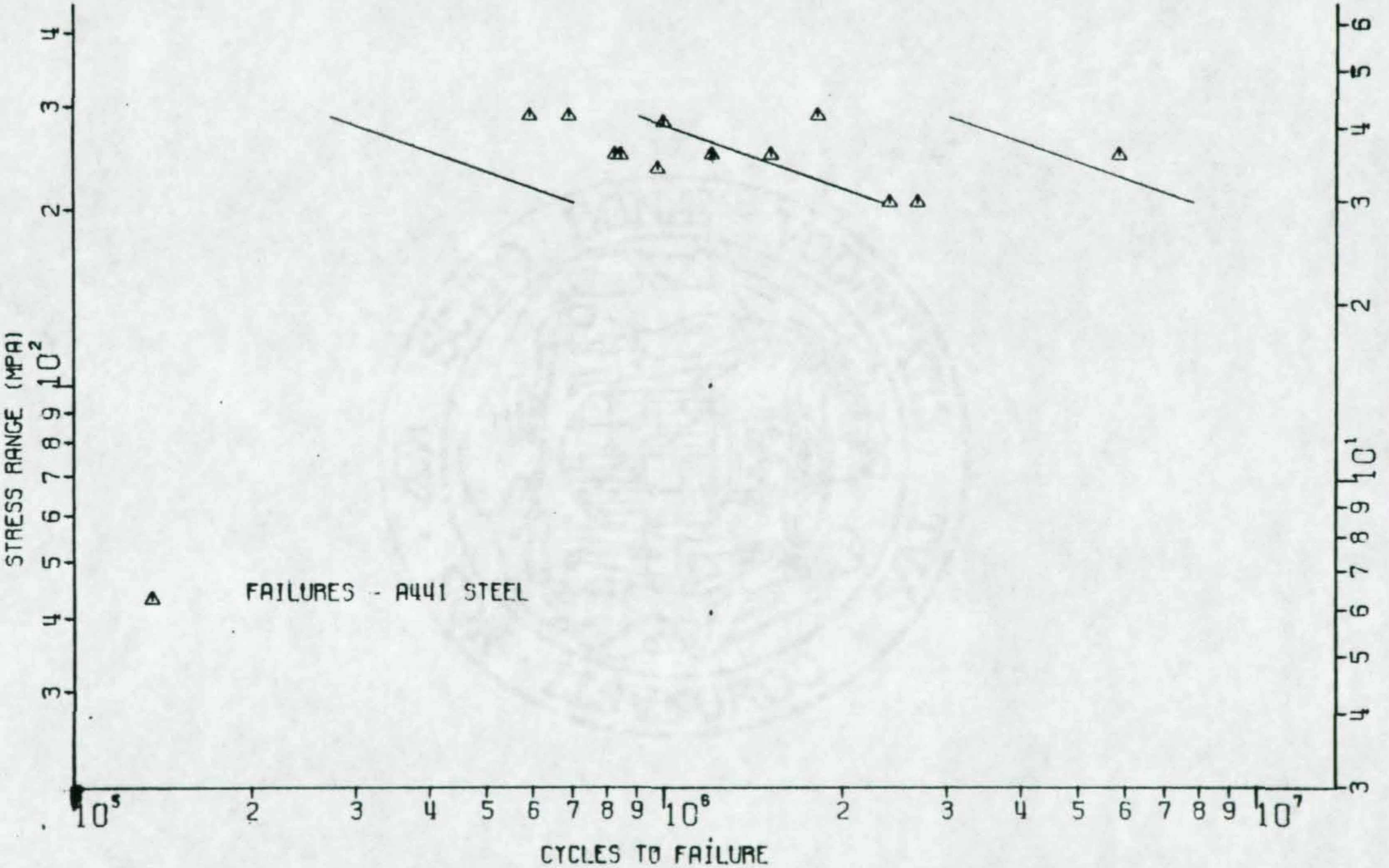
PLOT 2-3 FATIGUE TEST DATA FOR PLAIN WELDED BEAMS. A514 STEEL, WITH STRESS RANGE REDUCED TO INSIDE FIBER OF FLANGE. DATA FROM REF. 2 (NCHRP 102).



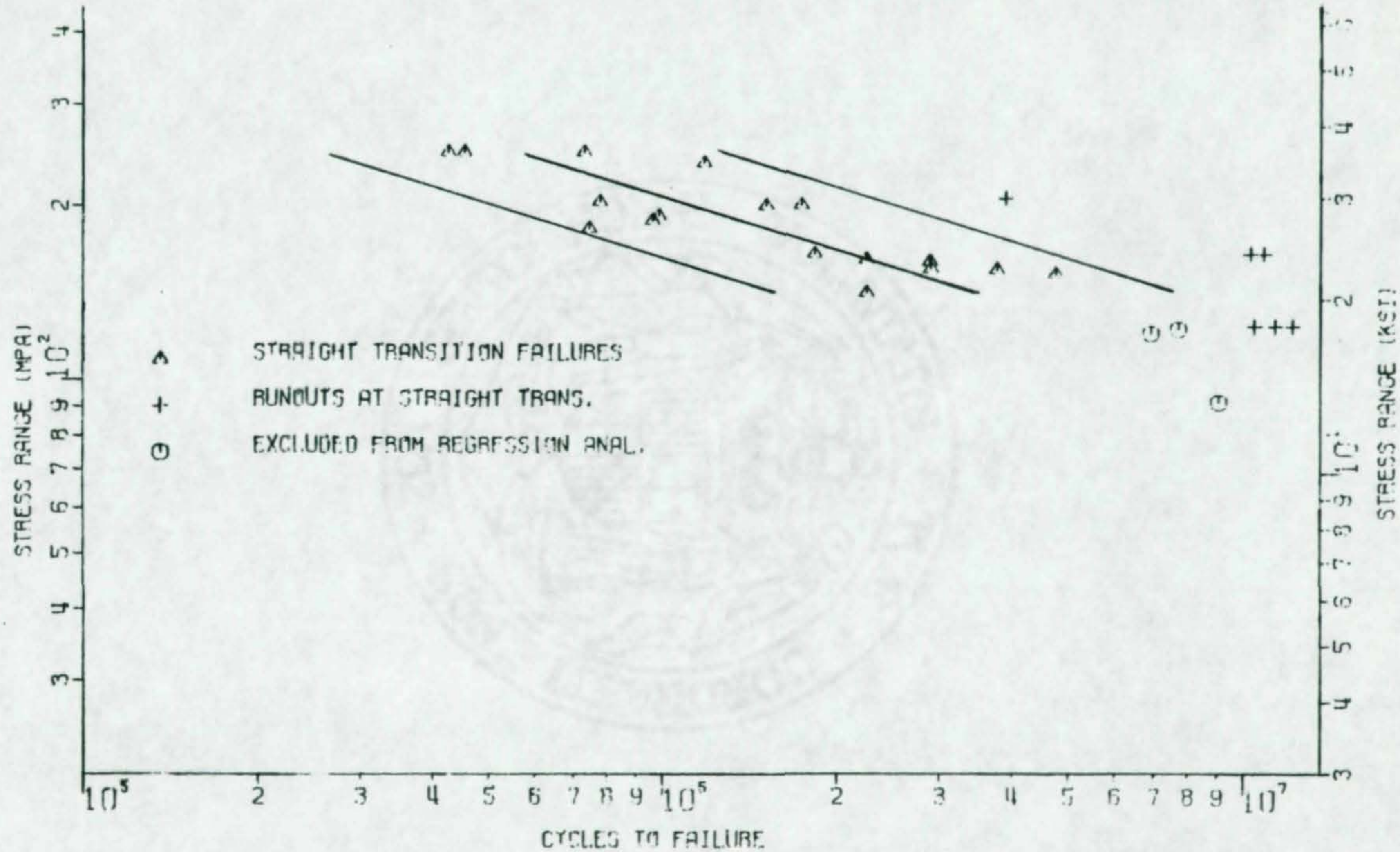
PLOT 2-4 FATIGUE TEST DATA FOR PLAIN WELDED BEAMS. ALL STEELS. WITH STRESS RANGE REDUCED TO INSIDE FIBER OF FLANGE. DATA FROM REF. 2 (NCHRP 102).



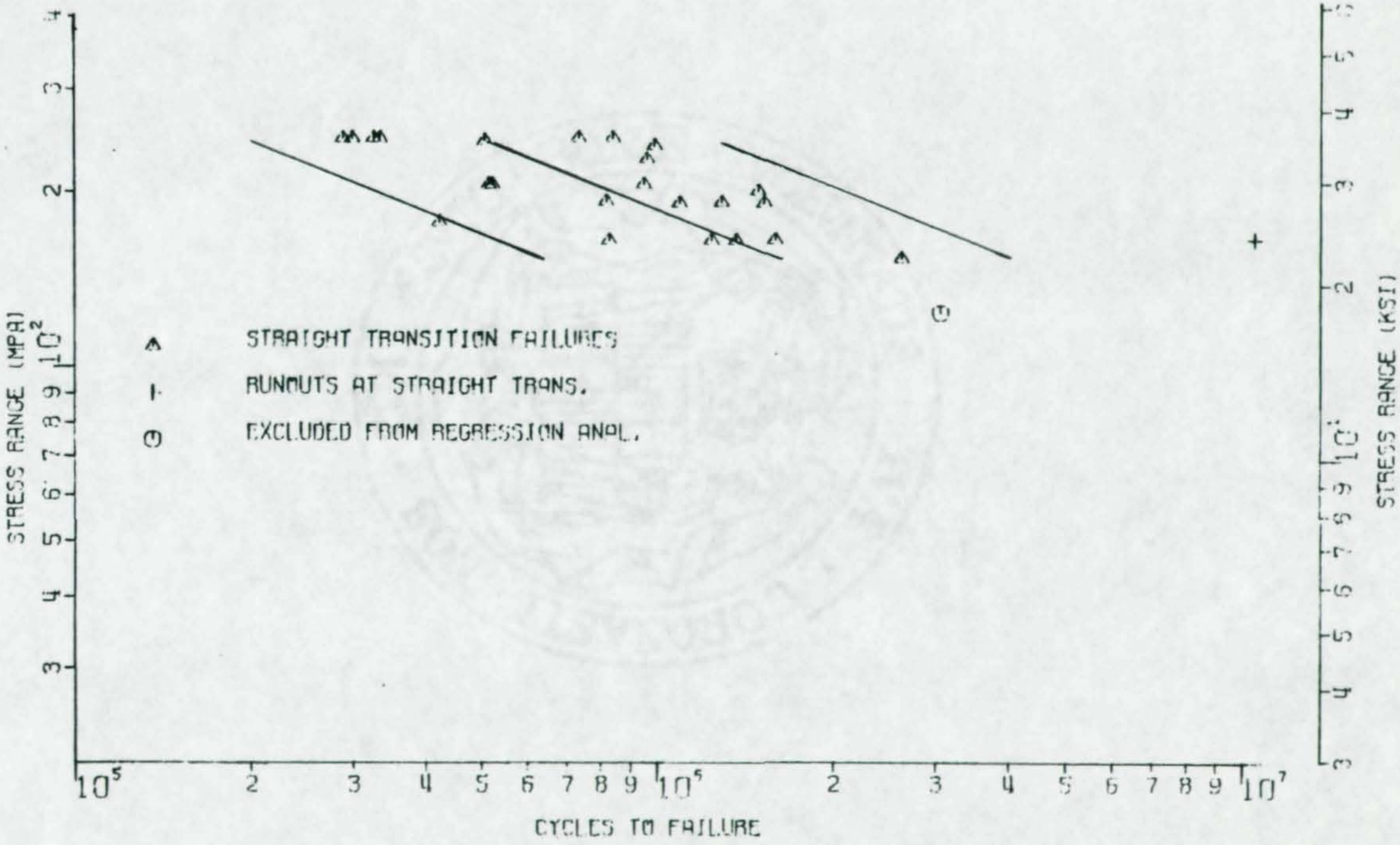
PLOT 2-5 FATIGUE TEST DATA FOR PLAIN ROLLED BEAMS. A36 STEEL(INITIAL TEST ONLY).
DATA FROM REF. 2 (NCHRP 102).



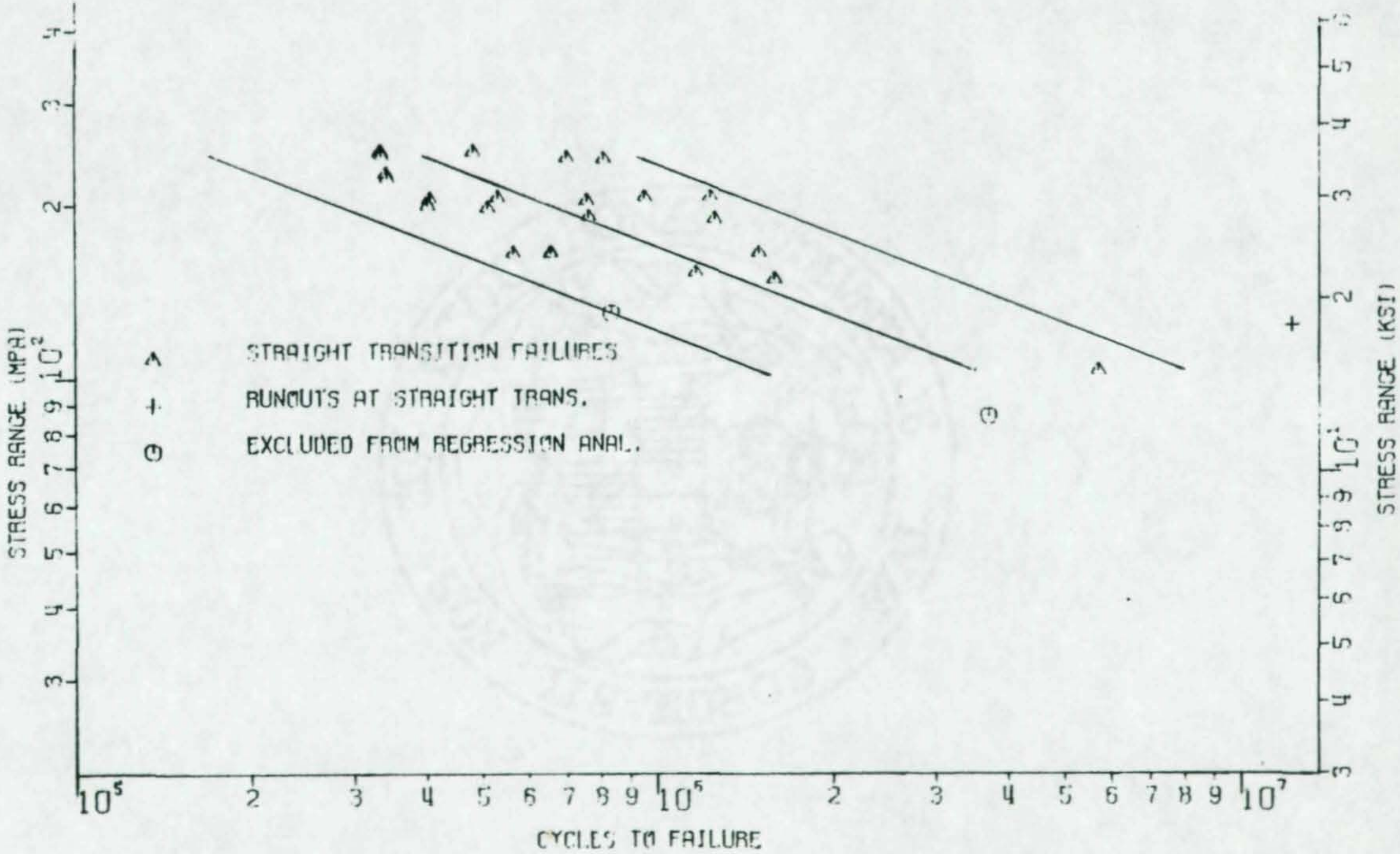
PLOT 2-6 FATIGUE TEST DATA FOR PLAIN ROLLED BEAMS. A441 STEEL(INITIAL TEST ONLY) DATA FROM REF. 2 (NCHRP 102). LONG LIFE DATA POINTS INCLUDED.



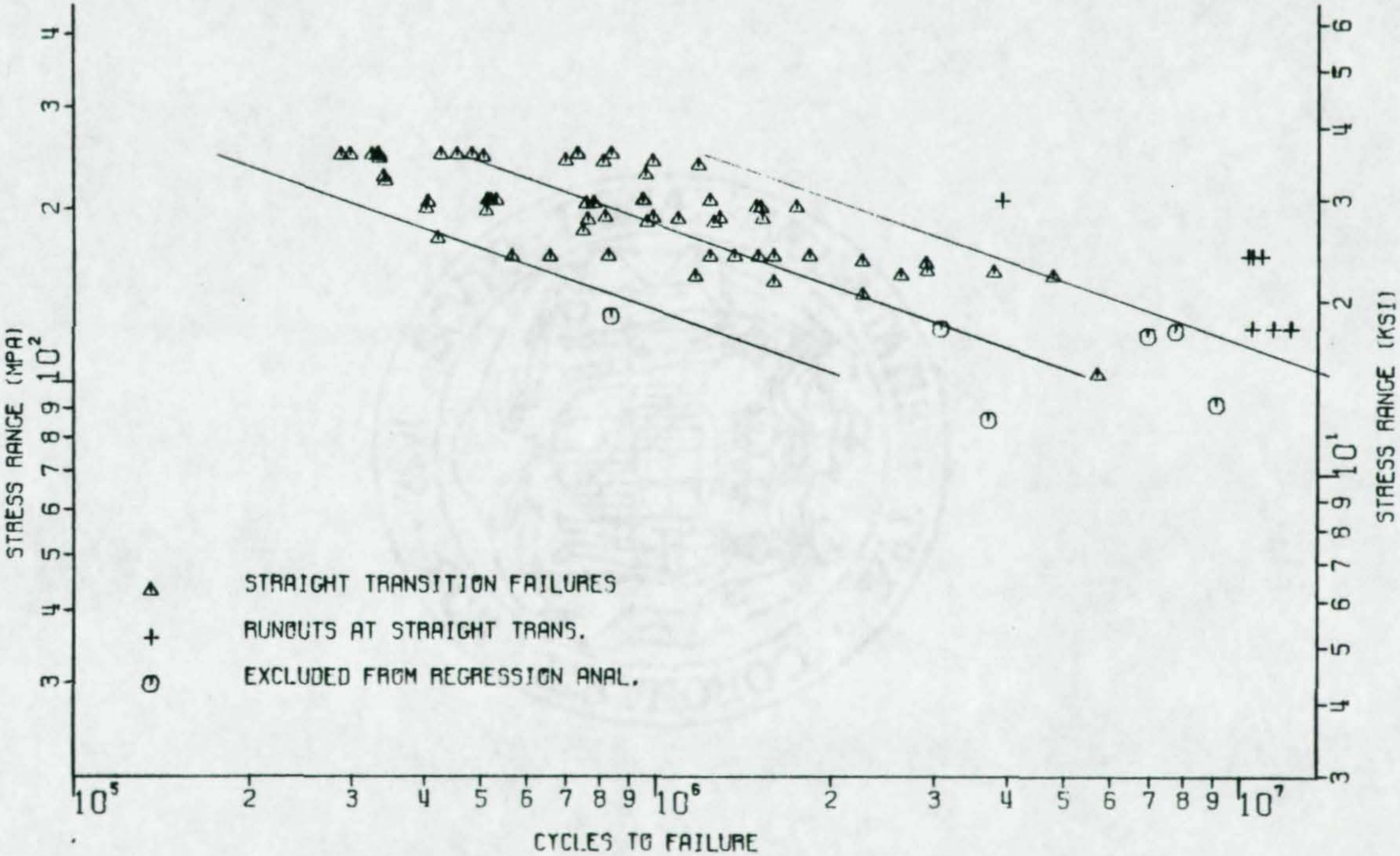
PLOT 2-8 FLANGE SPLICE DATA FOR FAILURES AT THE STRAIGHT TRANSITION FOR A36 STEEL ONLY (NCHRP 102).



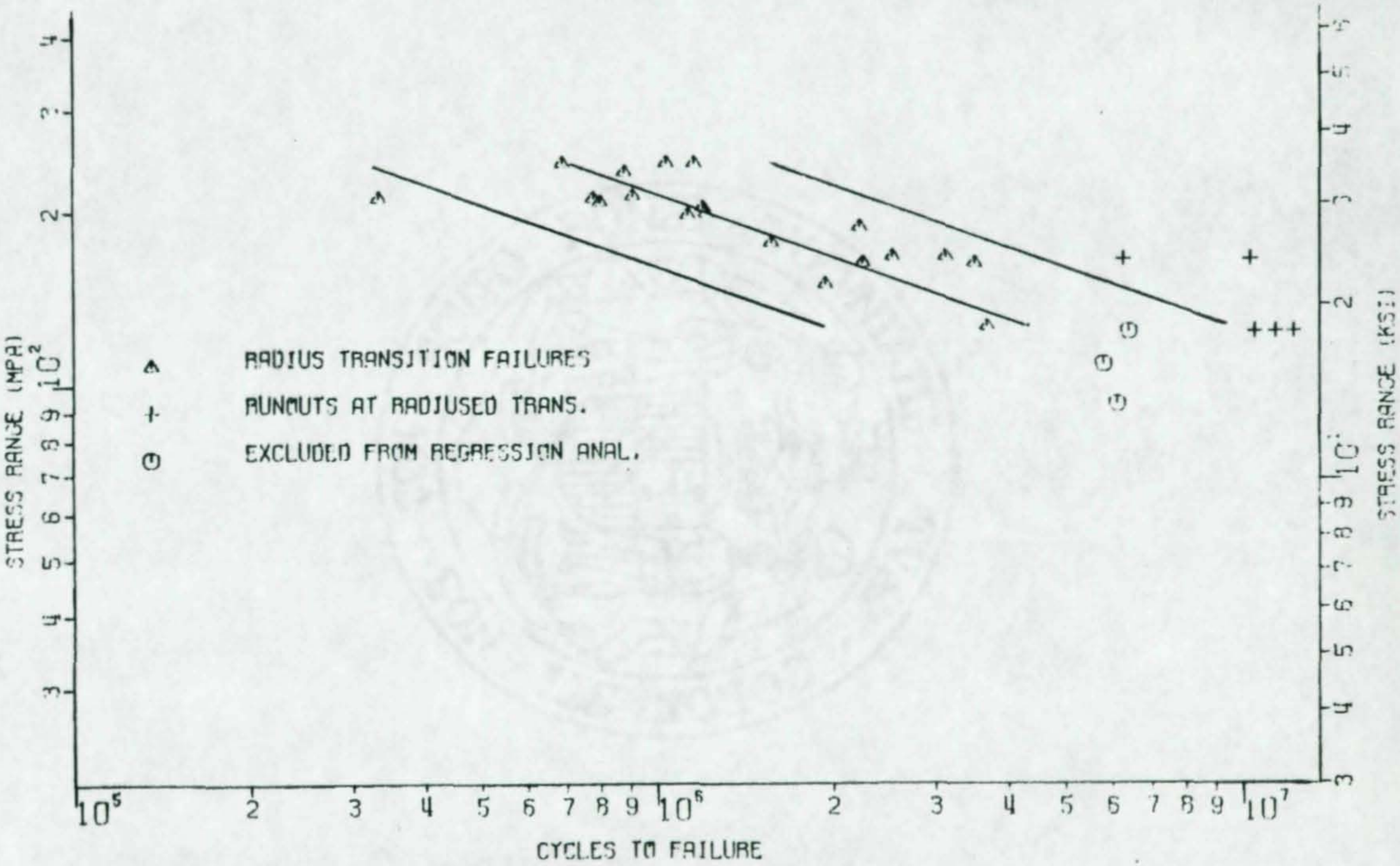
PLOT 2-9 FLANGE SPLICE DATA FOR FAILURES AT THE STRAIGHT TRANSITION FOR A441 STEEL ONLY (NCHRP 102).



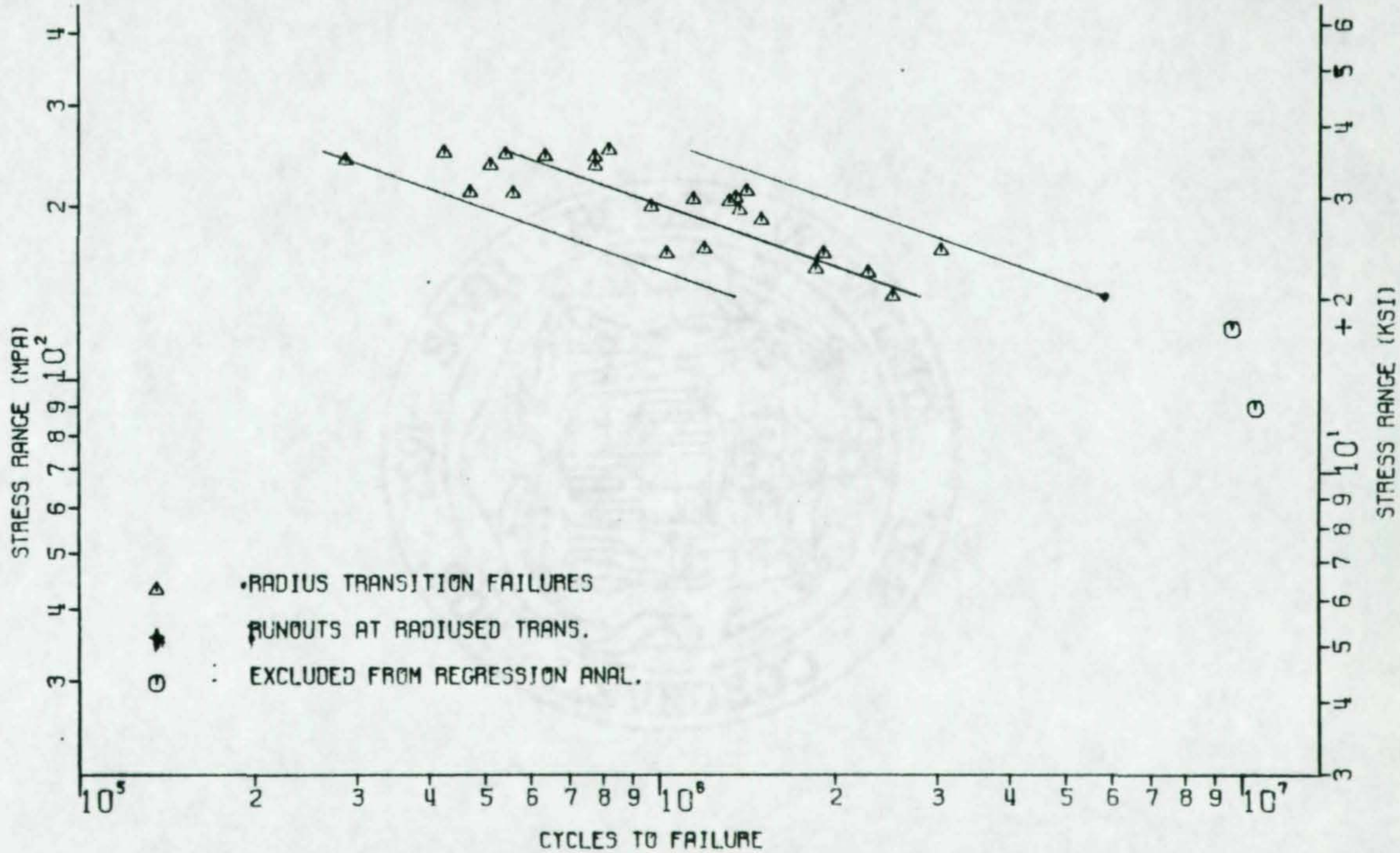
PLOT 2-10 FLANGE SPLICE DATA FOR FAILURES AT THE STRAIGHT TRANSITION FOR A514 STEEL ONLY (NCHRP-102).



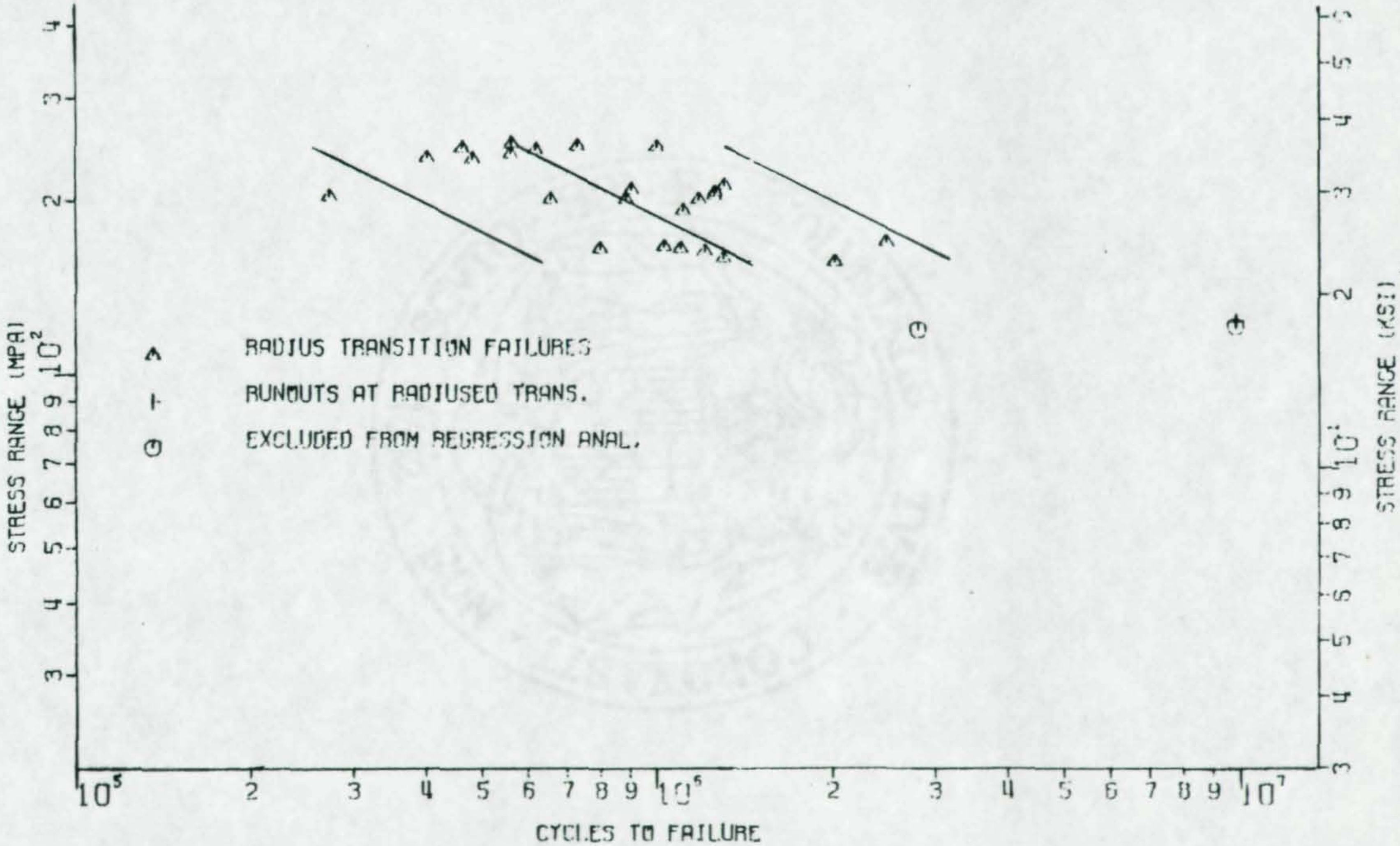
PLOT 2-11 FLANGE SPLICE DATA FOR FAILURES AT THE STRAIGHT TRANSITION FOR ALL STEELS–A36,A441,A514 (NCHRP 102).



PLOT 2-12 FLANGE SPLICE DATA FOR FAILURES AT THE RADIUSED TRANSITION FOR A36 STEEL ONLY (NCHRP 102).

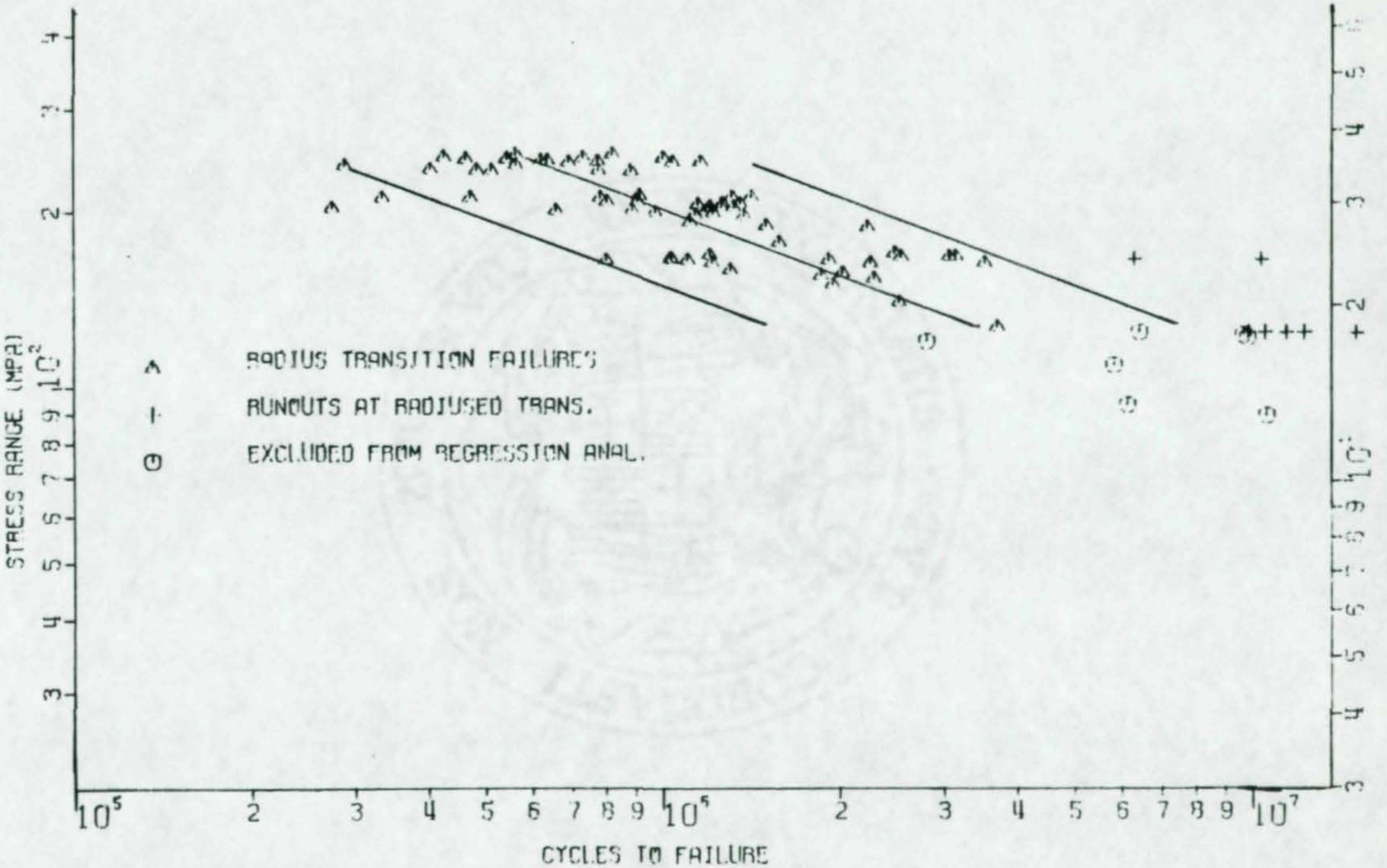


PLOT 2-13 FLANGE SPLICE DATA FOR FAILURES AT THE RADIUSED TRANSITION FOR A441 STEEL ONLY (NCHRP 102).

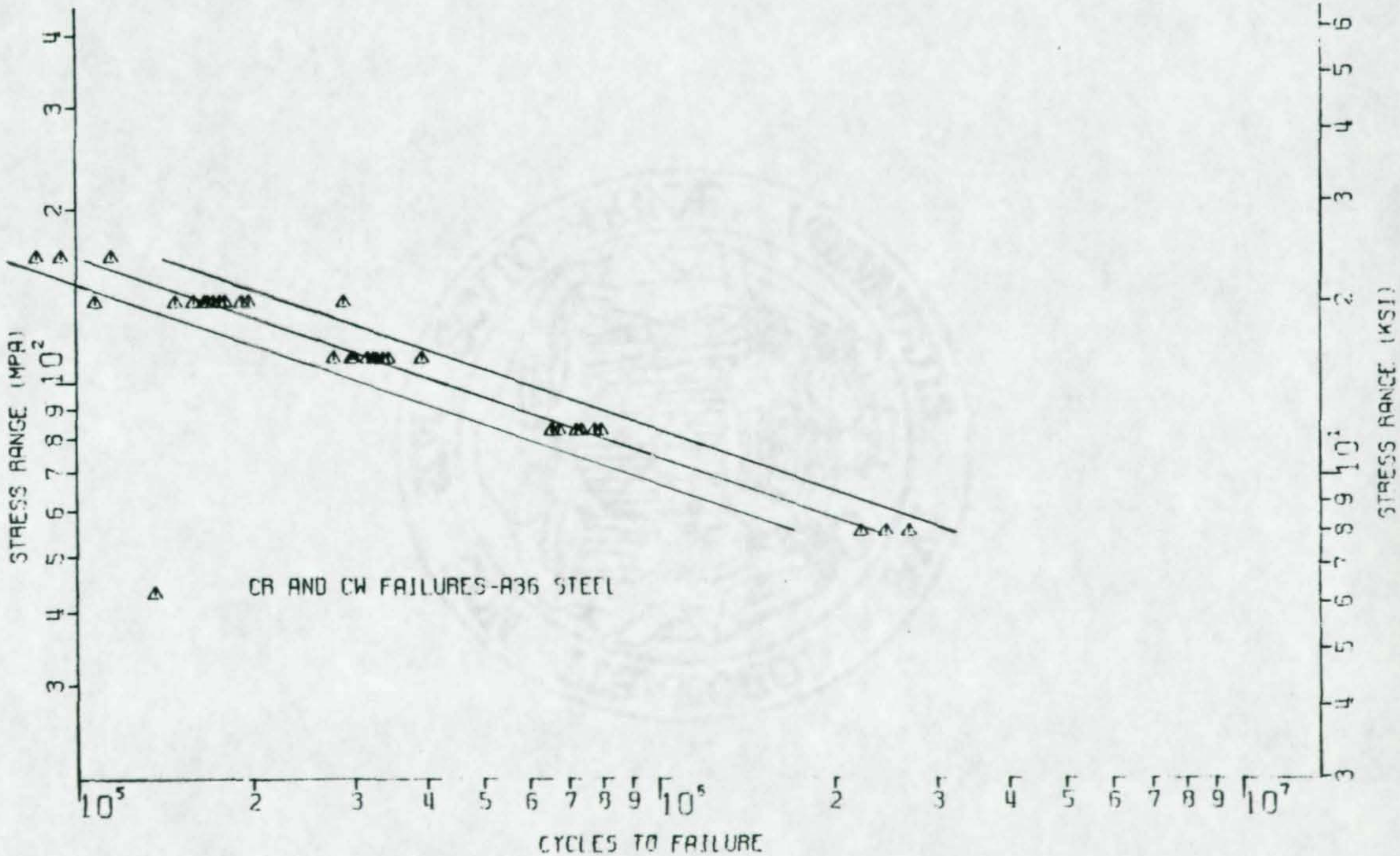


PLOT 2-14 FLANGE SPLICE DATA FOR FAILURES AT THE RADIUSED TRANSITION FOR A514 STEEL ONLY (NCHRP 102).

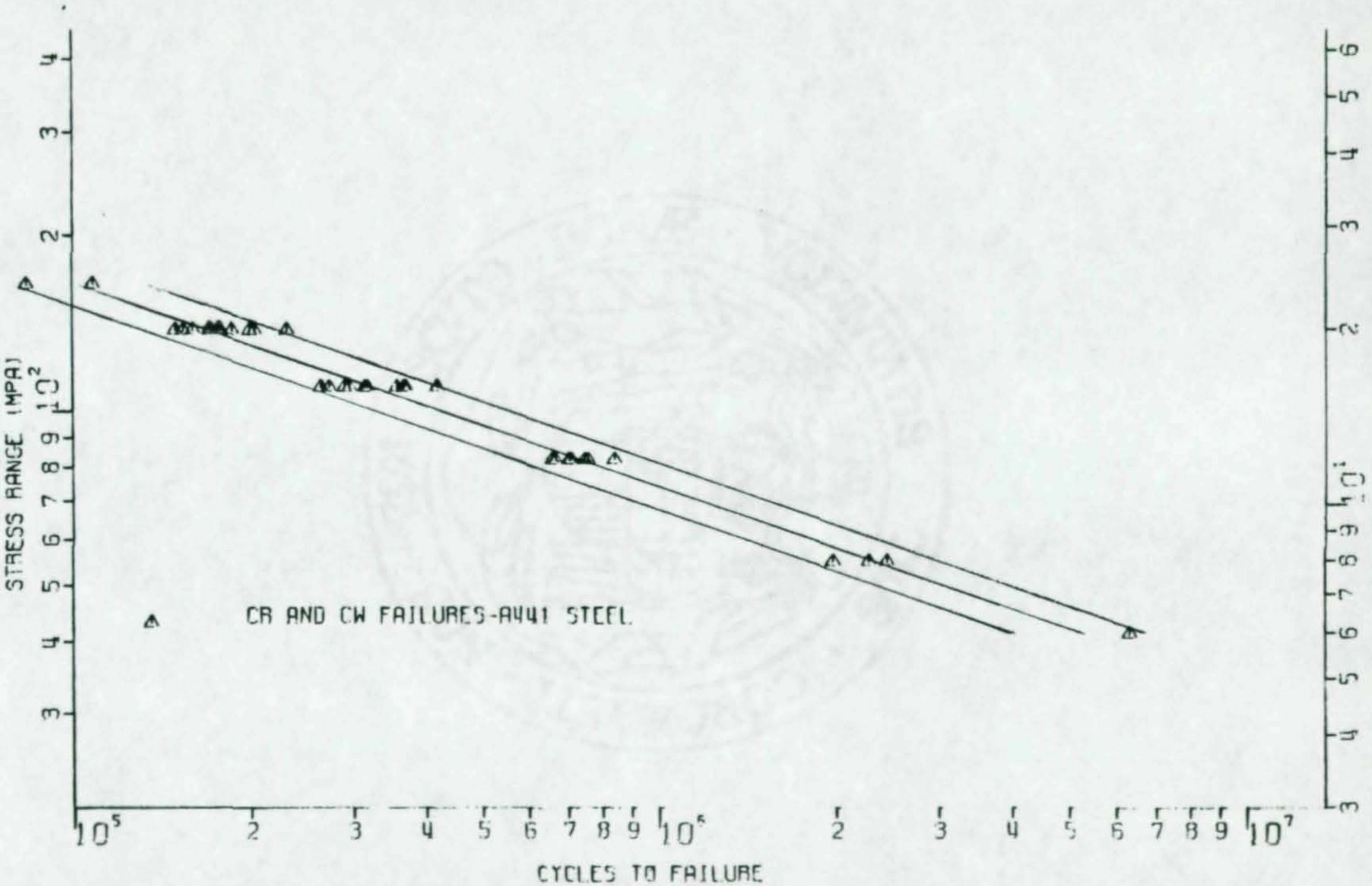
00795



PLOT 2-15 FLANGE SPLICE DATA FOR FAILURES AT THE RADIUSED TRANSITION FOR ALL STEELS-A36,A441,A514 (NCHRP 102).



PLOT 2-16 CR AND CW COVER PLATED BEAM DATA FOR A36 STEEL-WELDED END.
DATA FROM NCHRP 102.



PLOT 2-17 CR AND CW COVER PLATED BEAM DATA FOR A441 STEEL-WELDED END.
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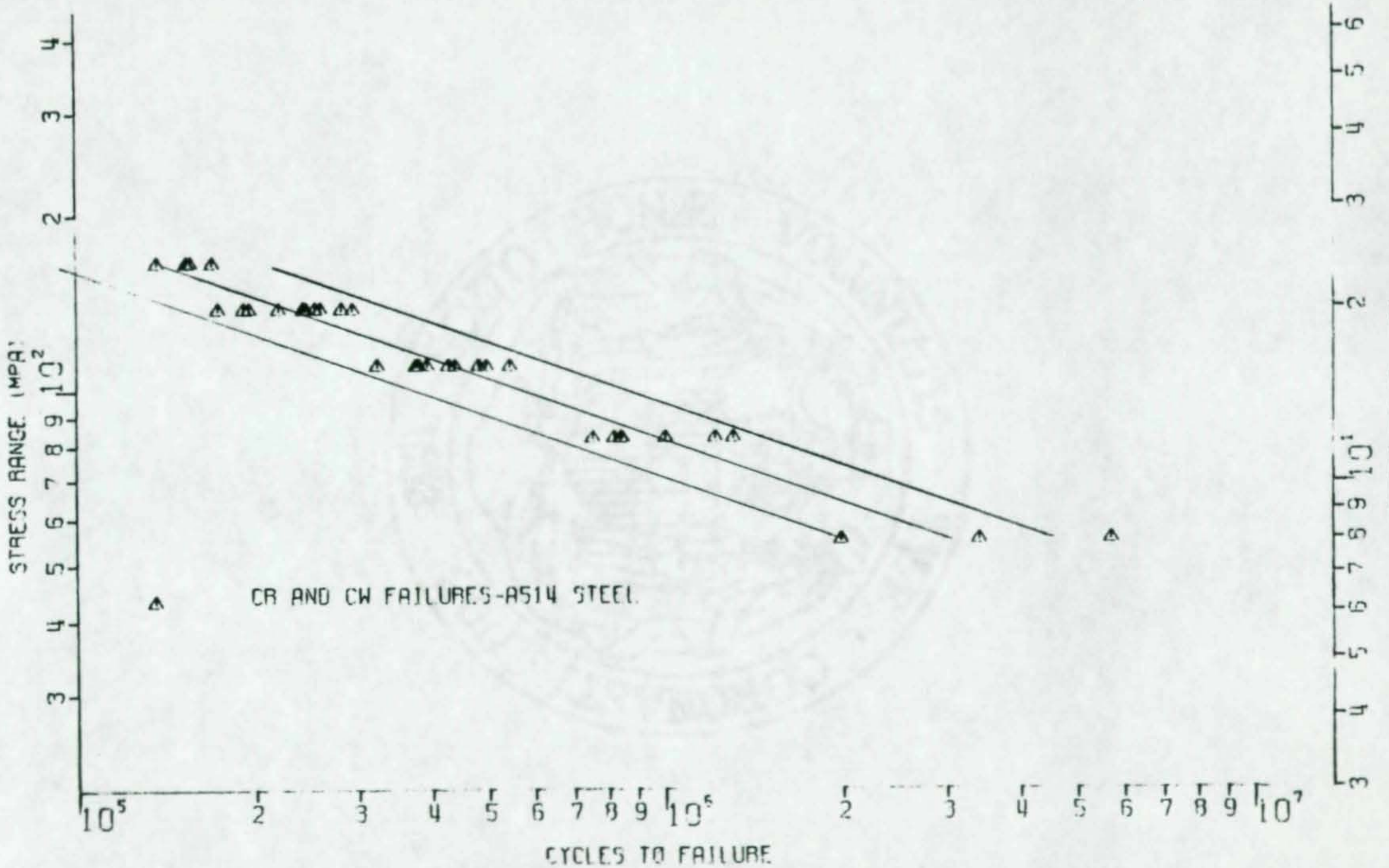
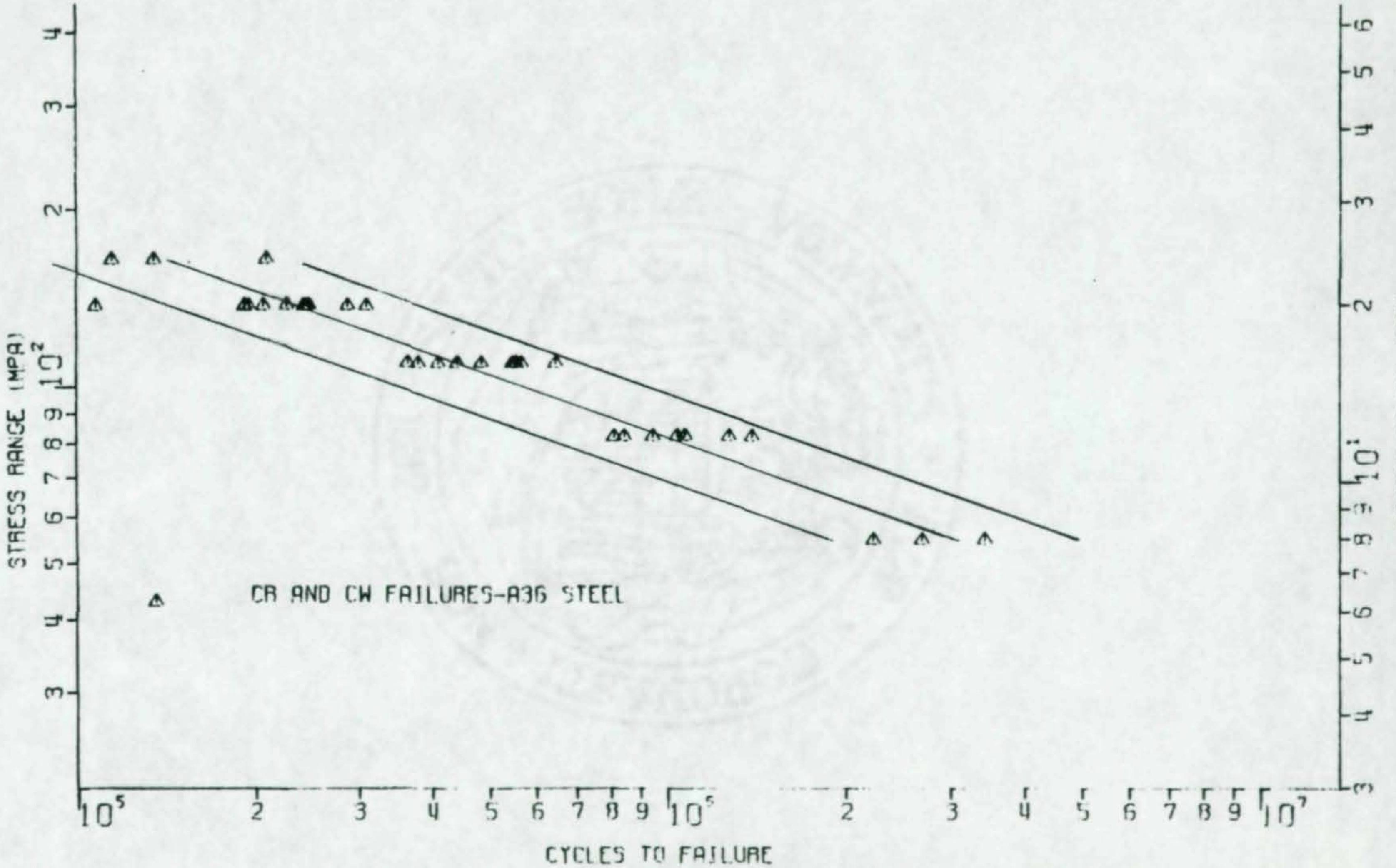
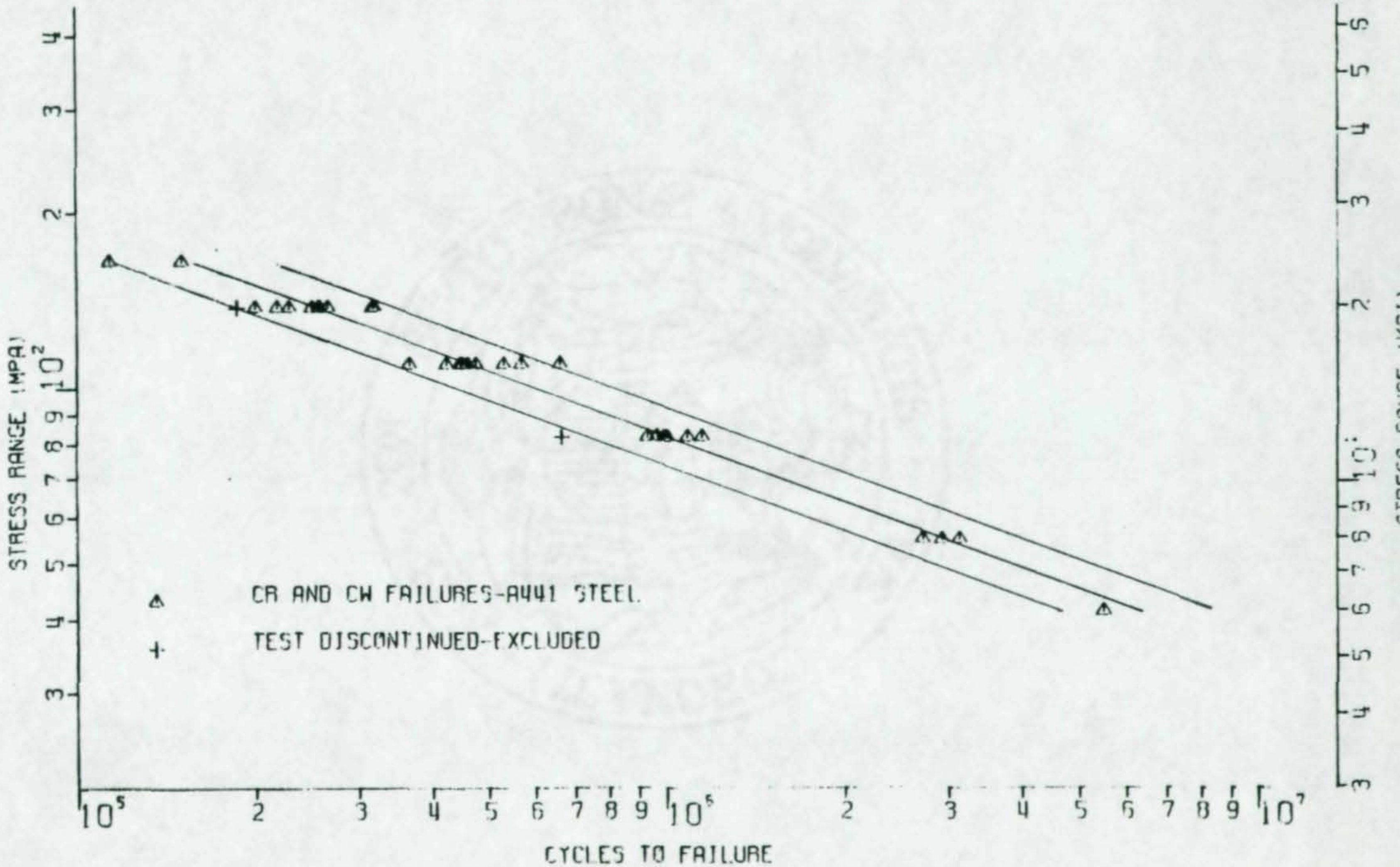


PLOT 2-18 CR AND CW COVER PLATED BEAM DATA FOR A514 STEEL-WELDED END.
DATA FROM NCHRP 102.

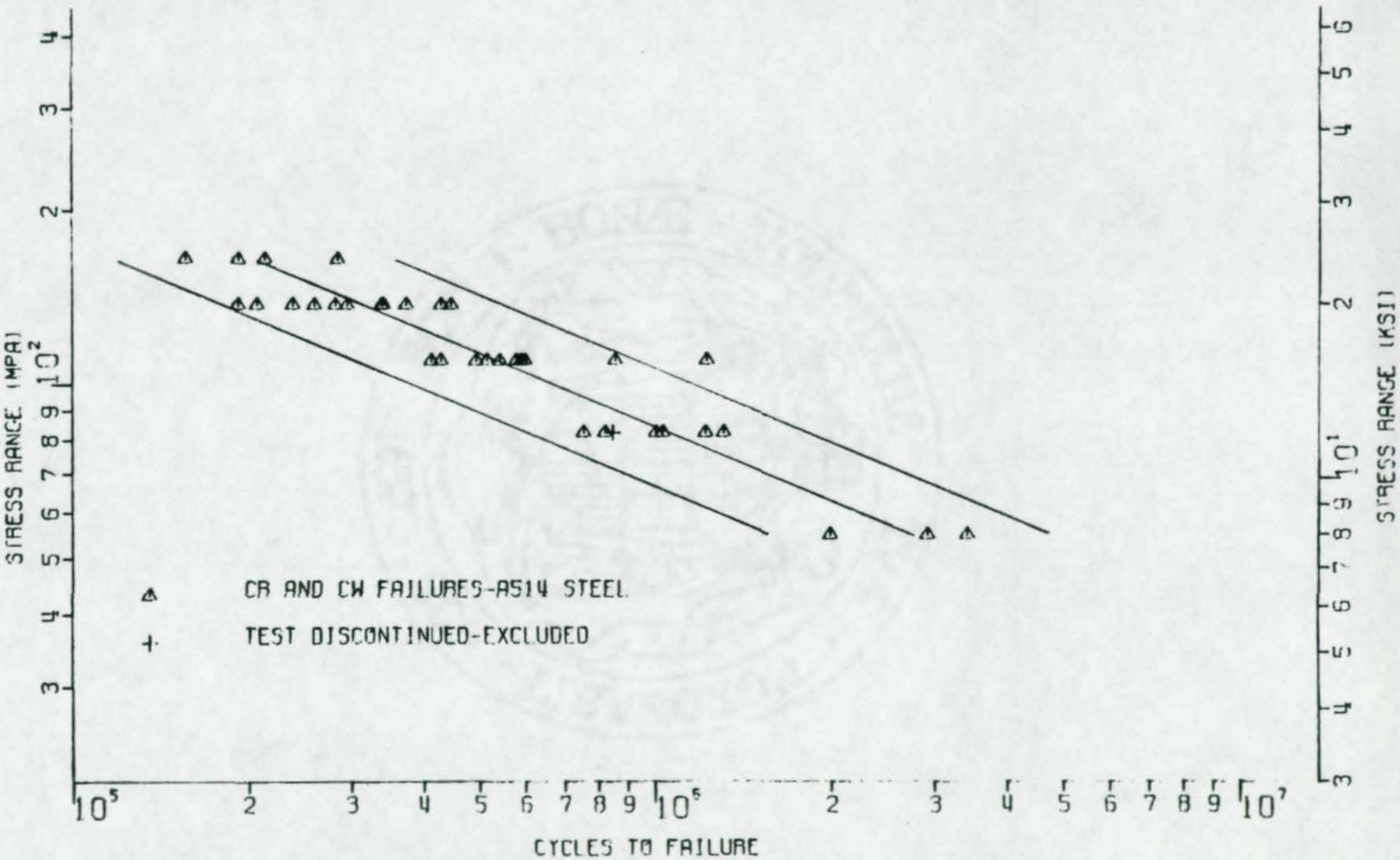
00799



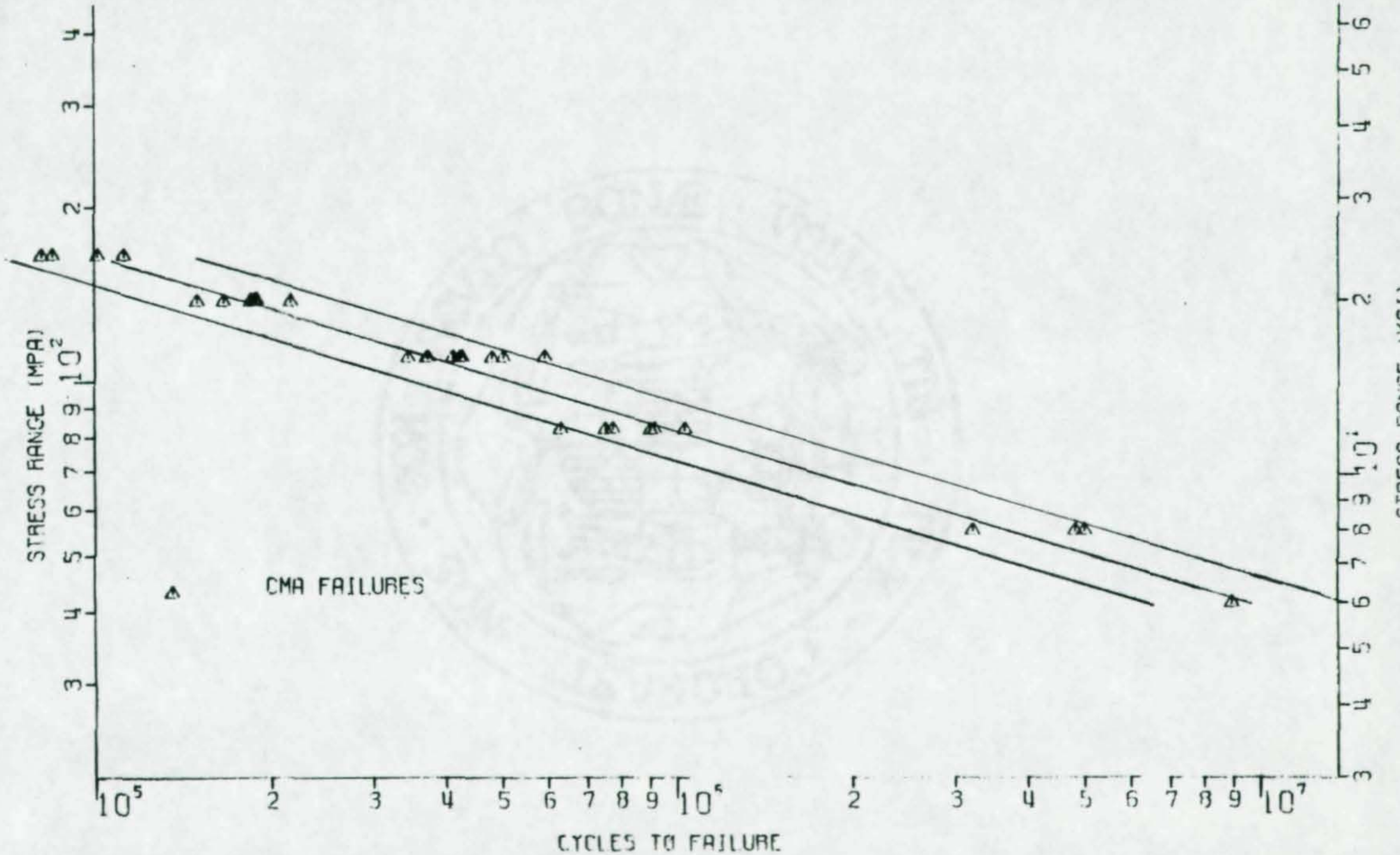
PLOT 2-19 CR AND CW COVER PLATED BEAM DATA FOR A36 STEEL-UNWELDED END.
DATA FROM NCHRP 102.



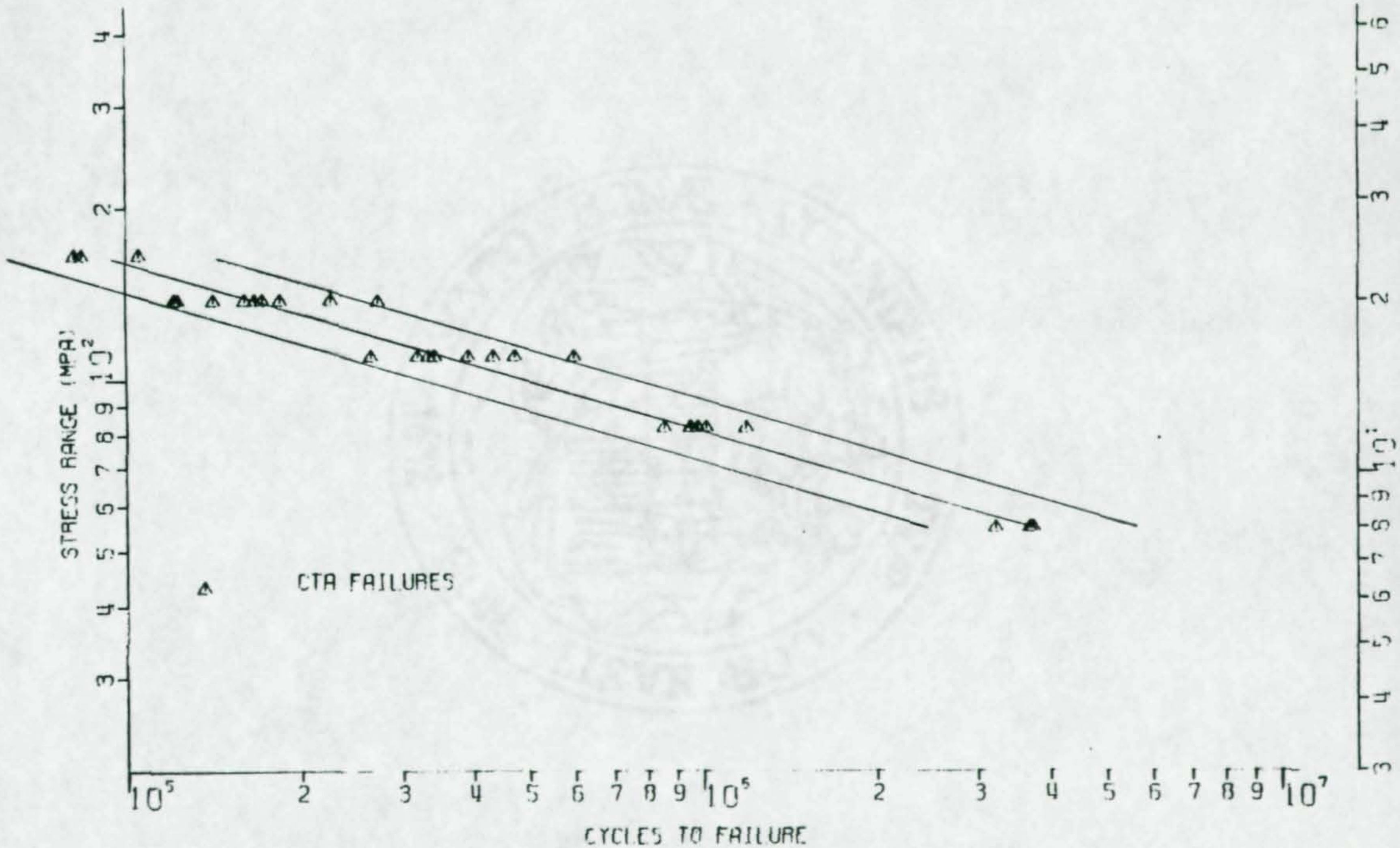
PLOT 2-20 CR AND CW COVER PLATED BEAM DATA FOR A441 STEEL-UNWELDED END. DATA FROM NCHRP 102.



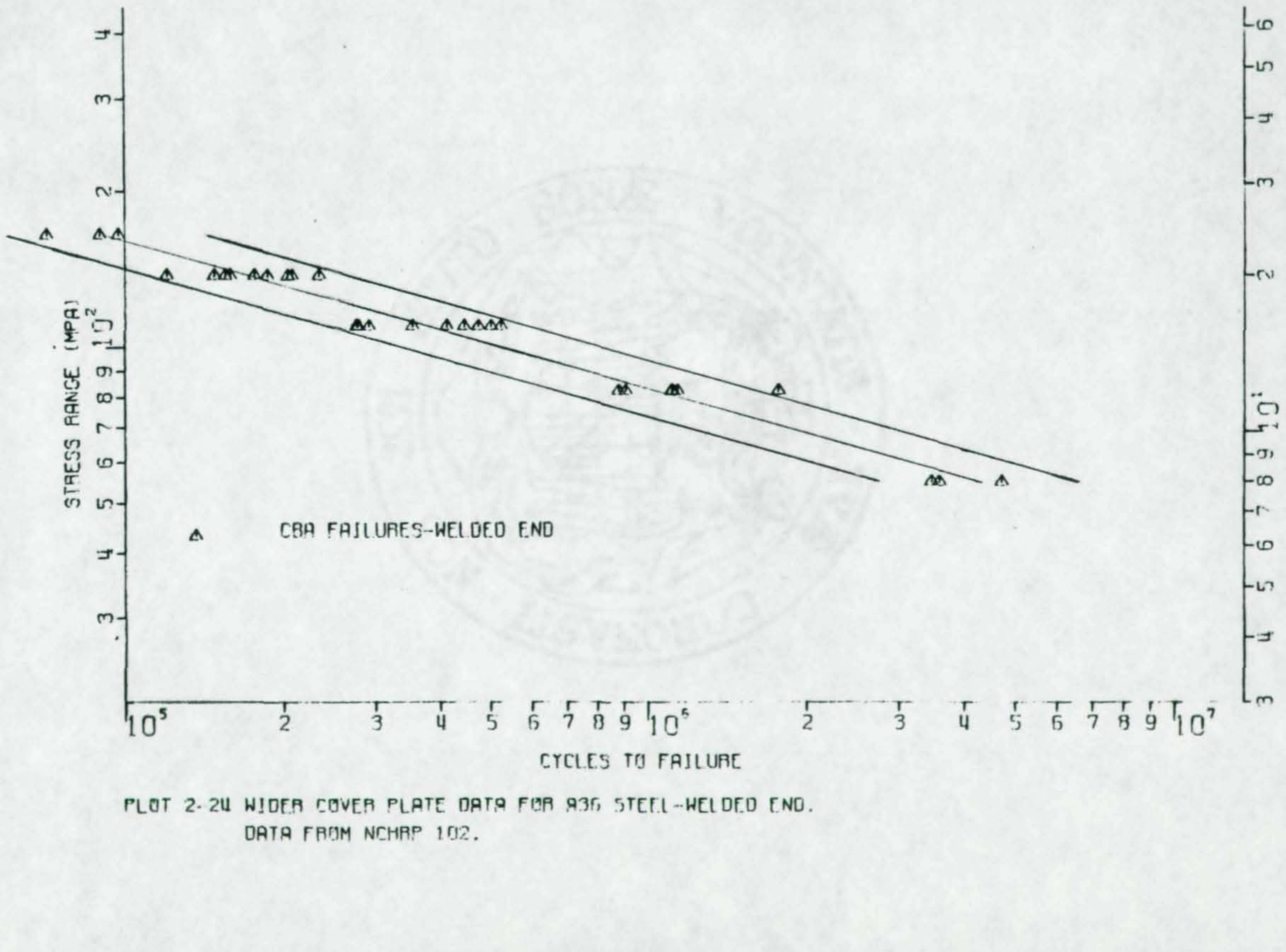
PLOT 2-21 CR AND CW COVER PLATED BEAM DATA FOR A514 STEEL-UNWELDED END.
DATA FROM NCHRP 102.



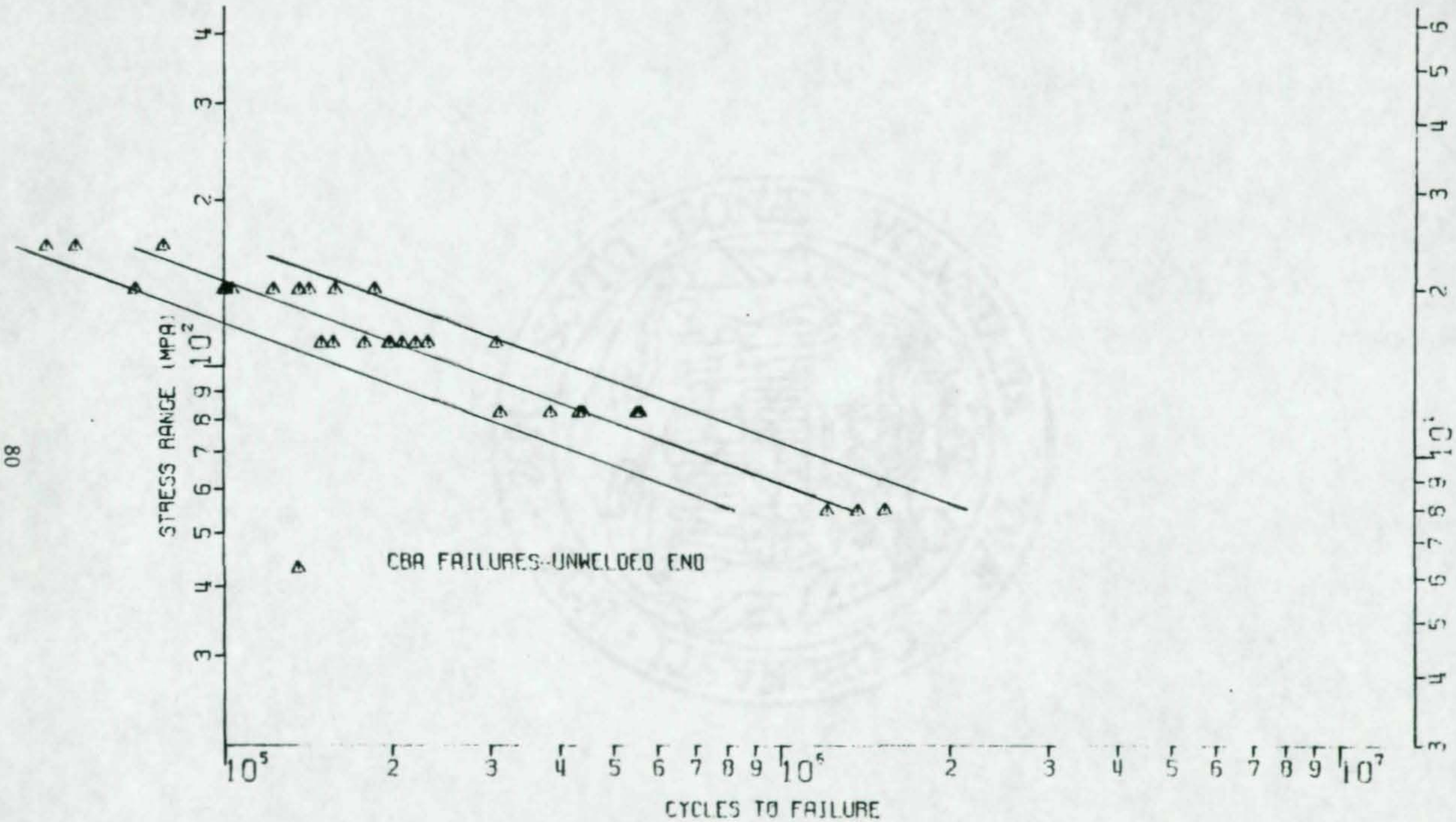
PLOT 2-22 MULTIPLE COVER PLATED BEAM DATA FOR A36 STEEL-WELDED END.
DATA FROM NCHRP 102.



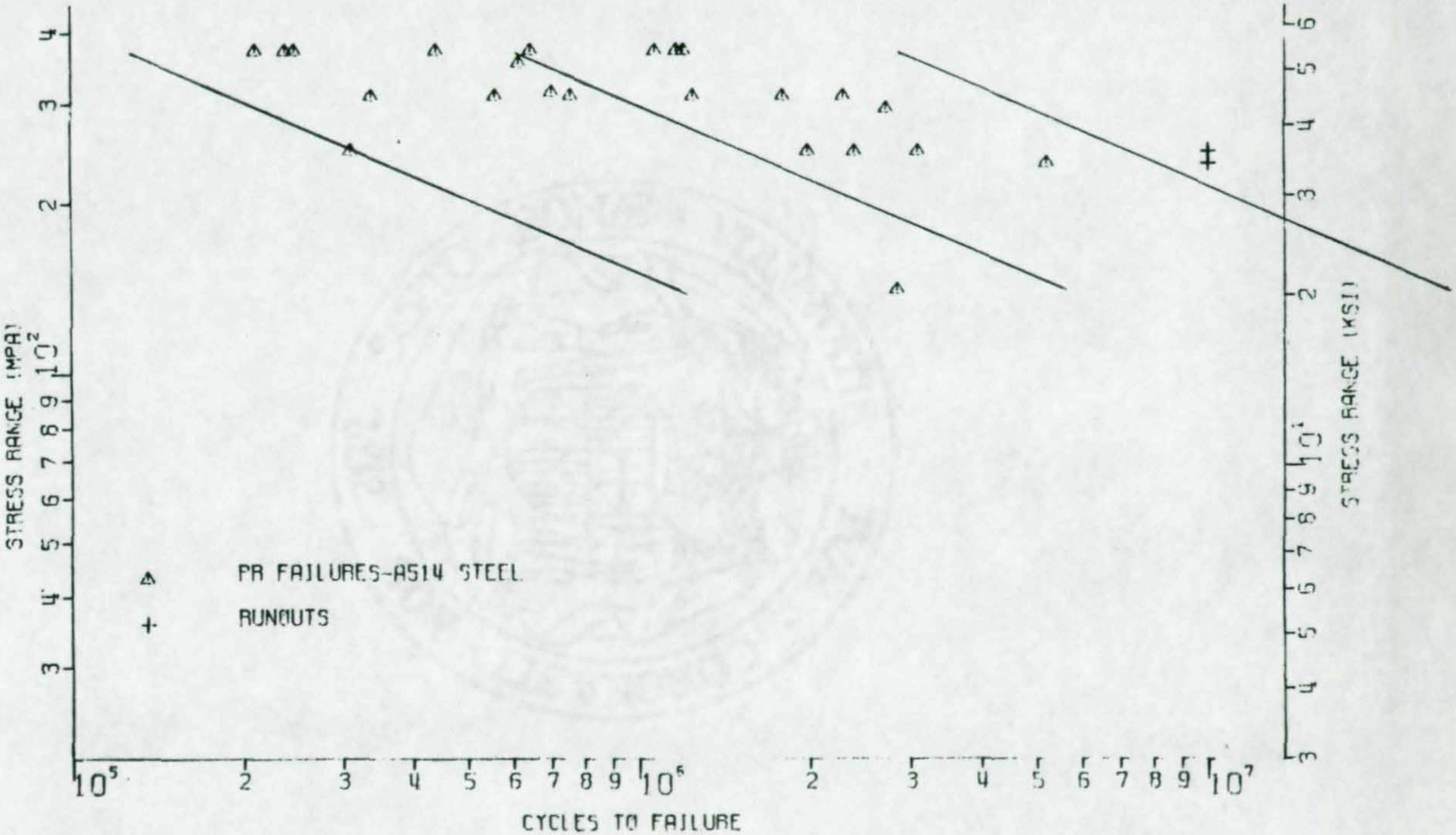
PLOT 2-23 THICK COVER PLATE DATA FOR A36 STEEL-WELDED END.
DATA FROM NCHRP 102.



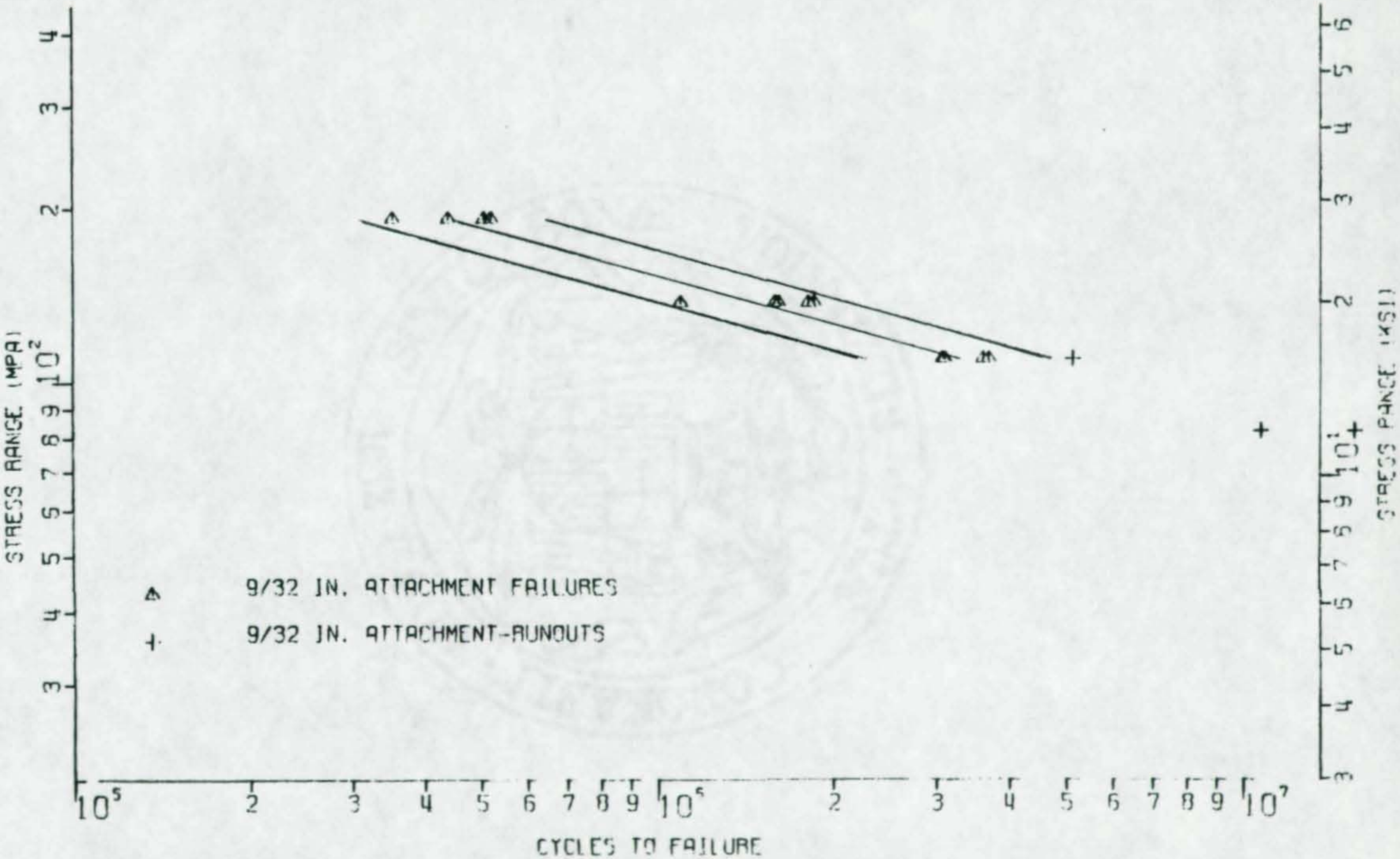
PLOT 2-24 WIDER COVER PLATE DATA FOR A36 STEEL-WELDED END.
DATA FROM NCHRP 102.



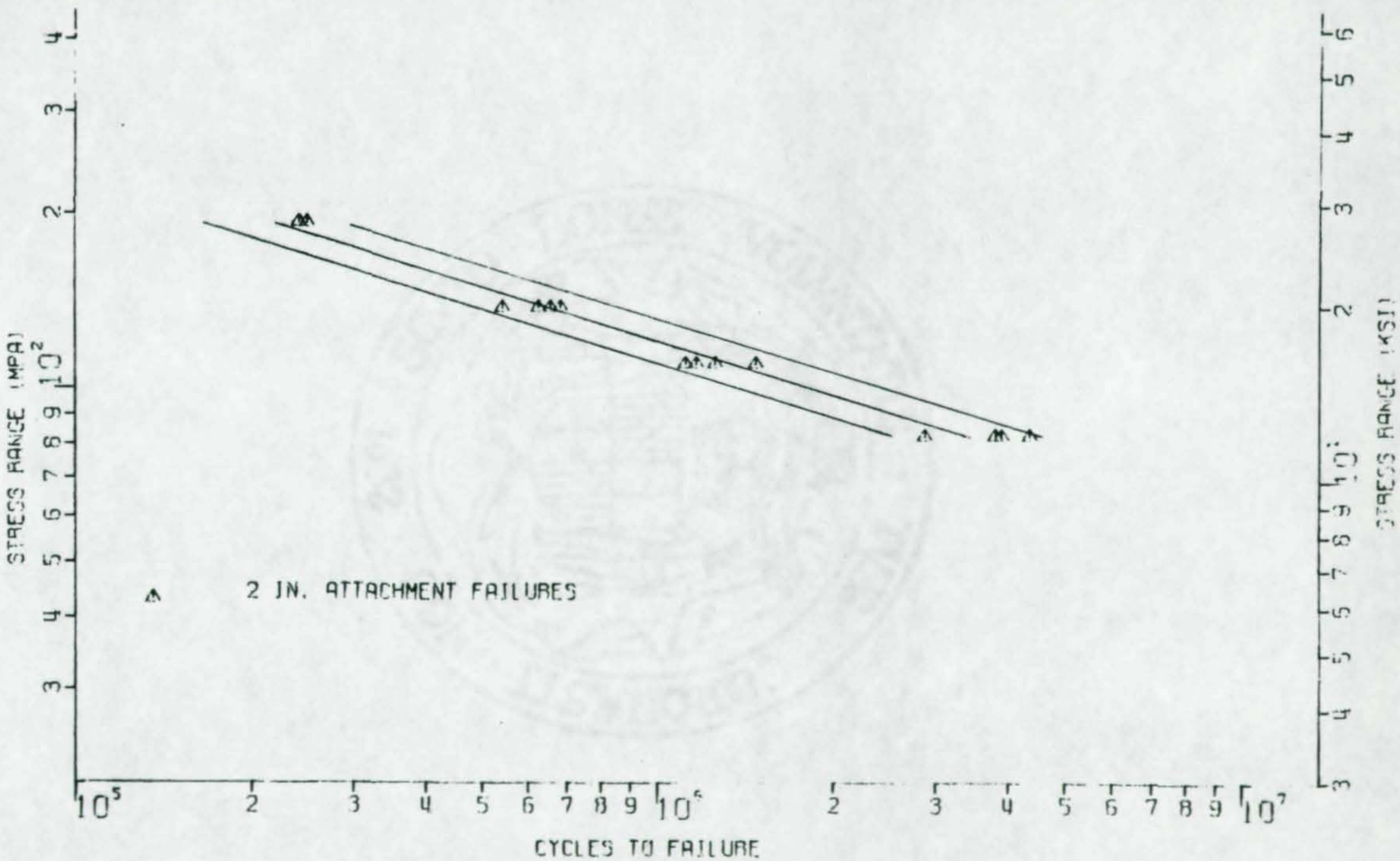
PLOT 2-25 WIDER COVER PLATE DATA FOR A36 STEEL-UNWELDED END.
DATA FROM NCHRP 102.



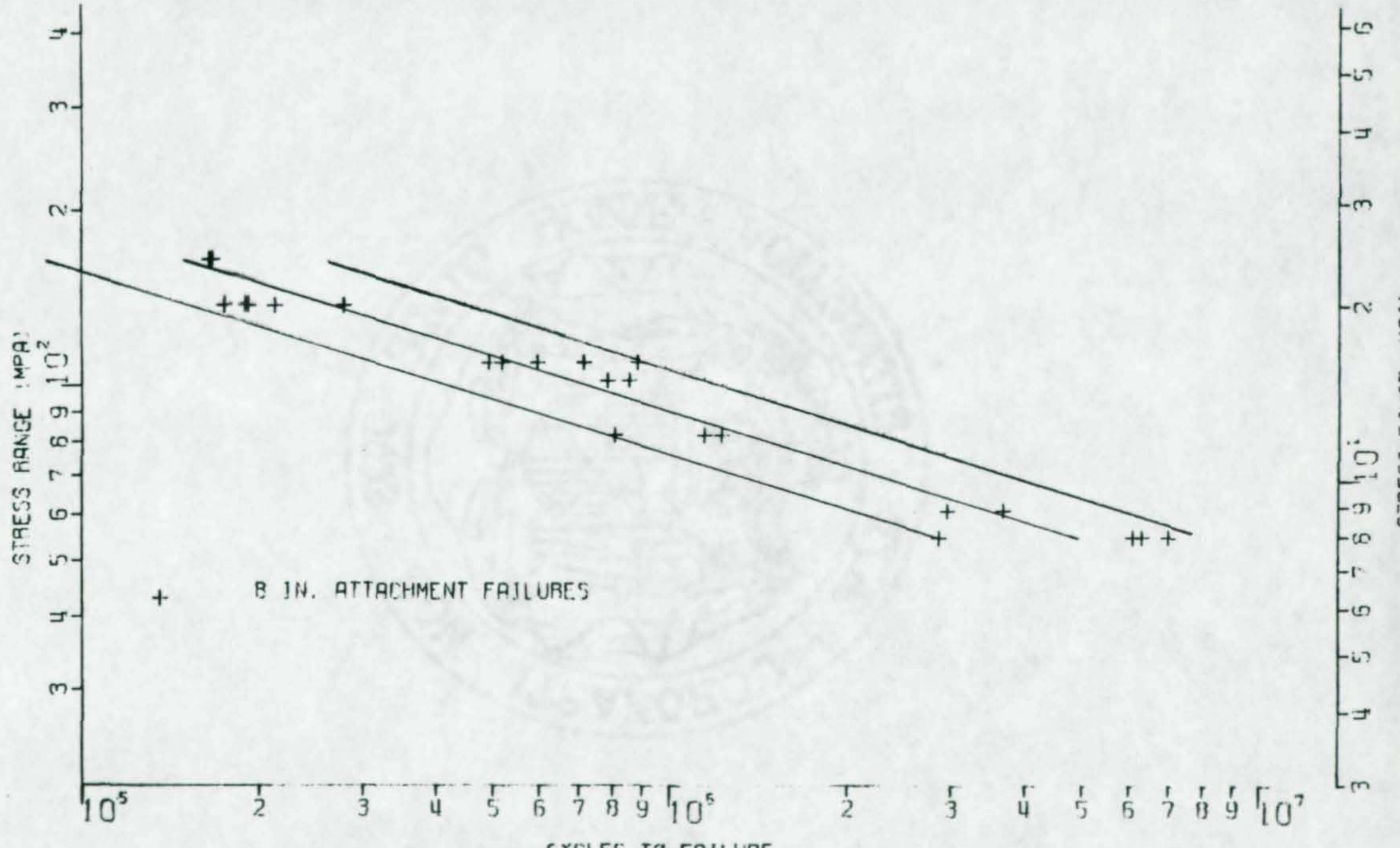
PLOT 3-1 PLAIN ROLLED BEAM DATA FOR A514 STEEL.
DATA FROM NCHRP 147.



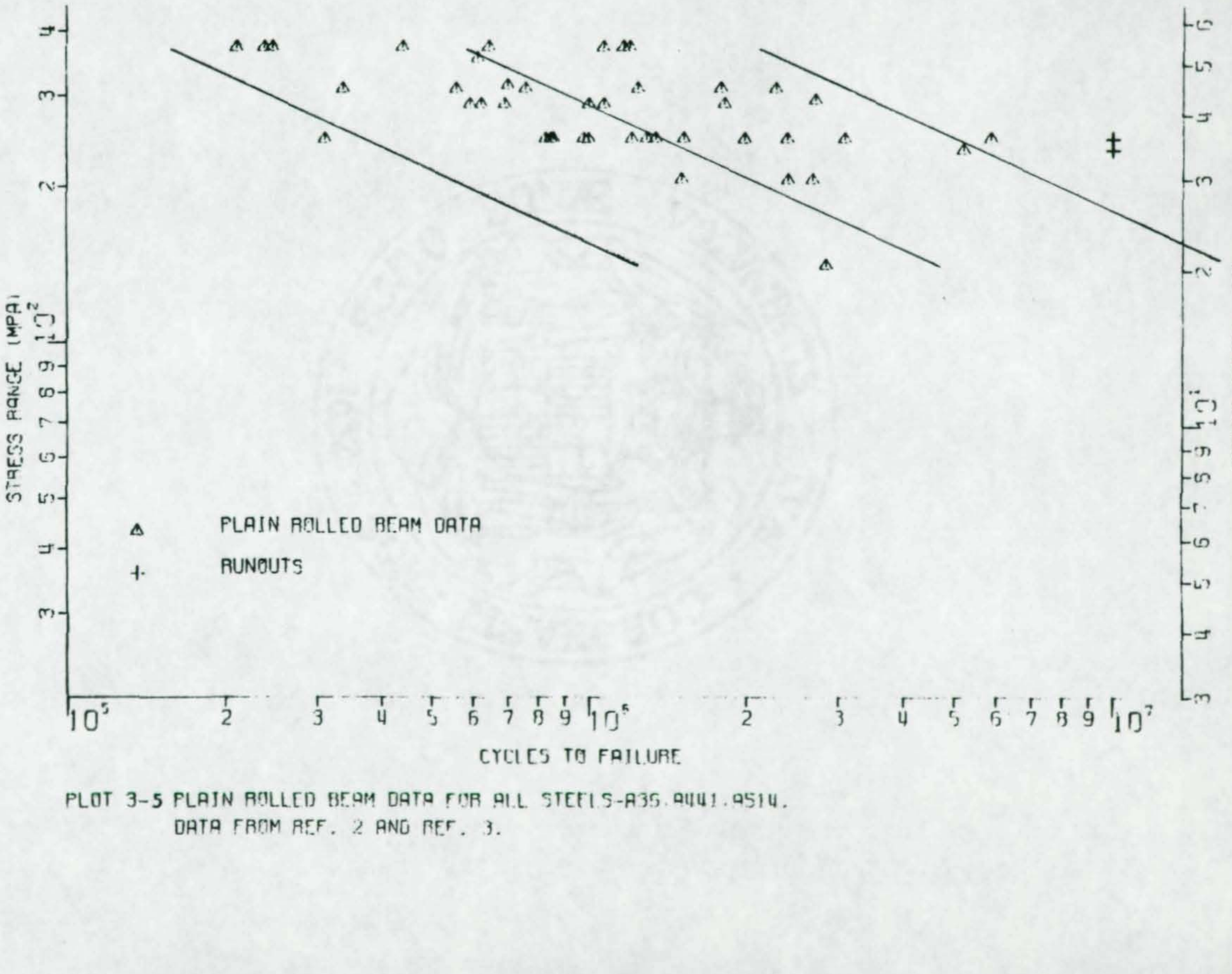
PLOT 3-2 WELDED BEAM WITH 9/32 IN. ATTACHMENTS.
DATA FROM NCHRP 147.



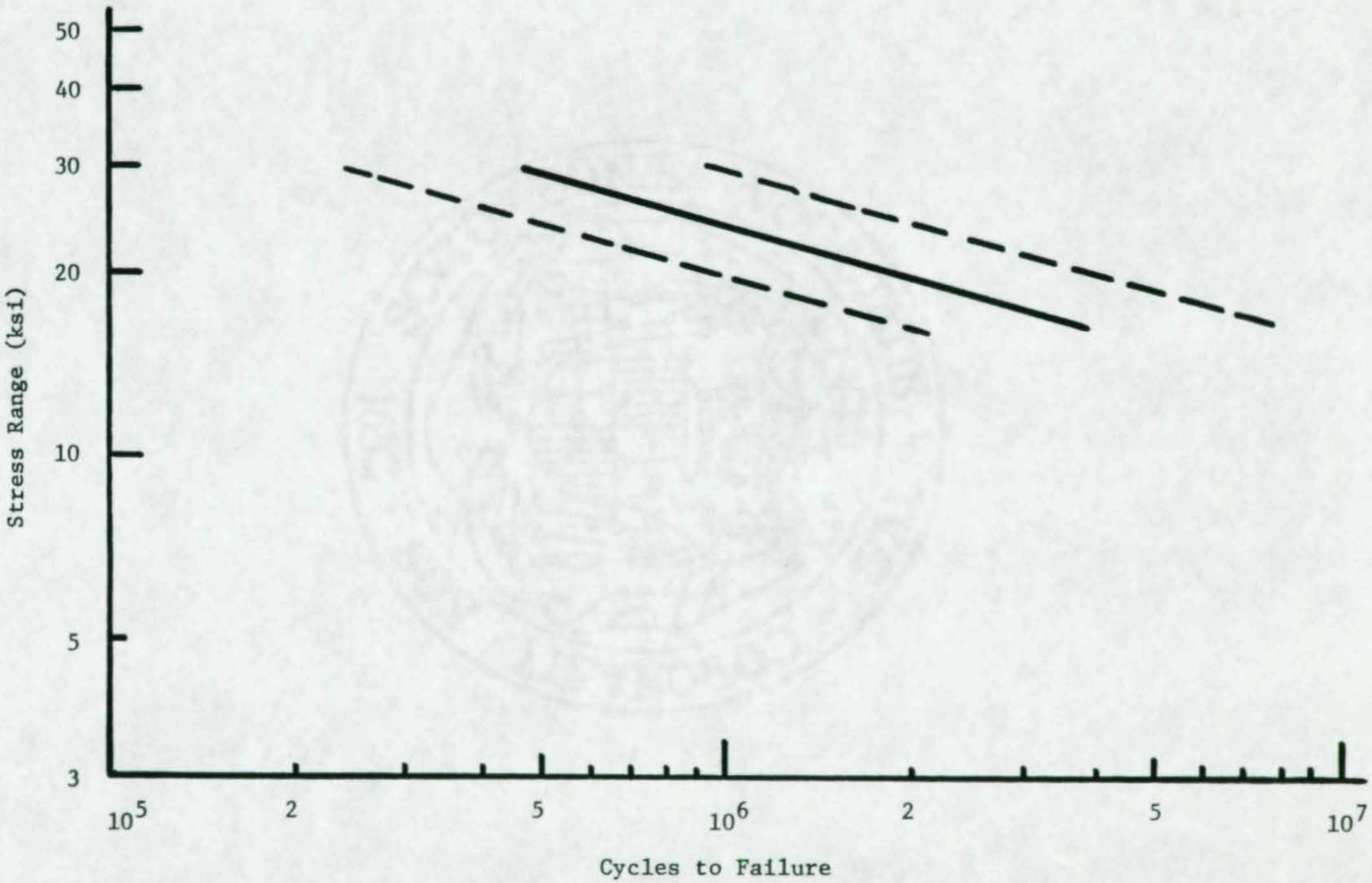
PLOT 3-3 WELDED BEAM WITH 2 IN. ATTACHMENTS.
DATA FROM NCHRP 147.



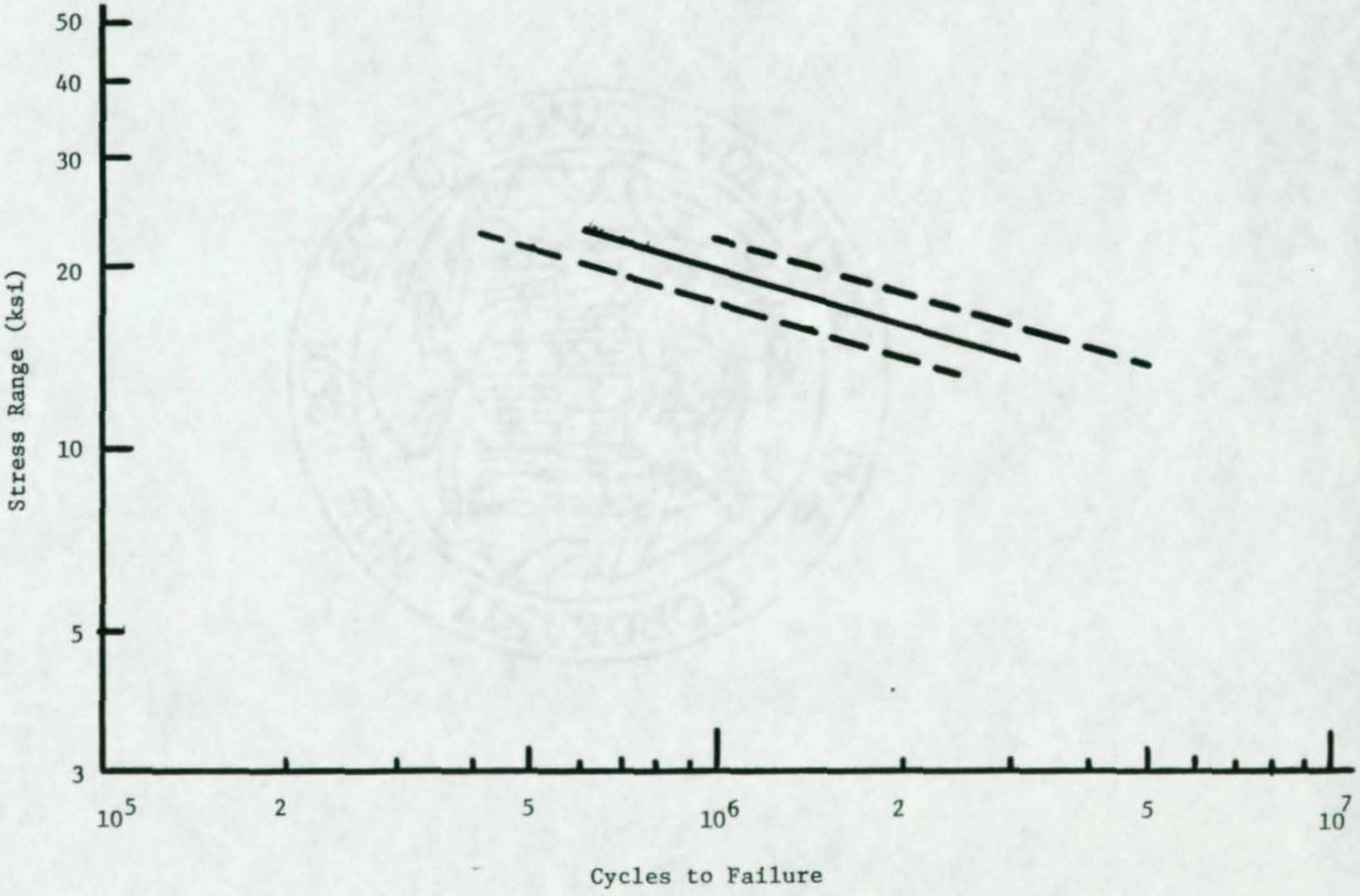
PLOT 3-4 WELDED BEAM WITH 8 IN. ATTACHMENTS.
DATA FROM NCHRP 147.



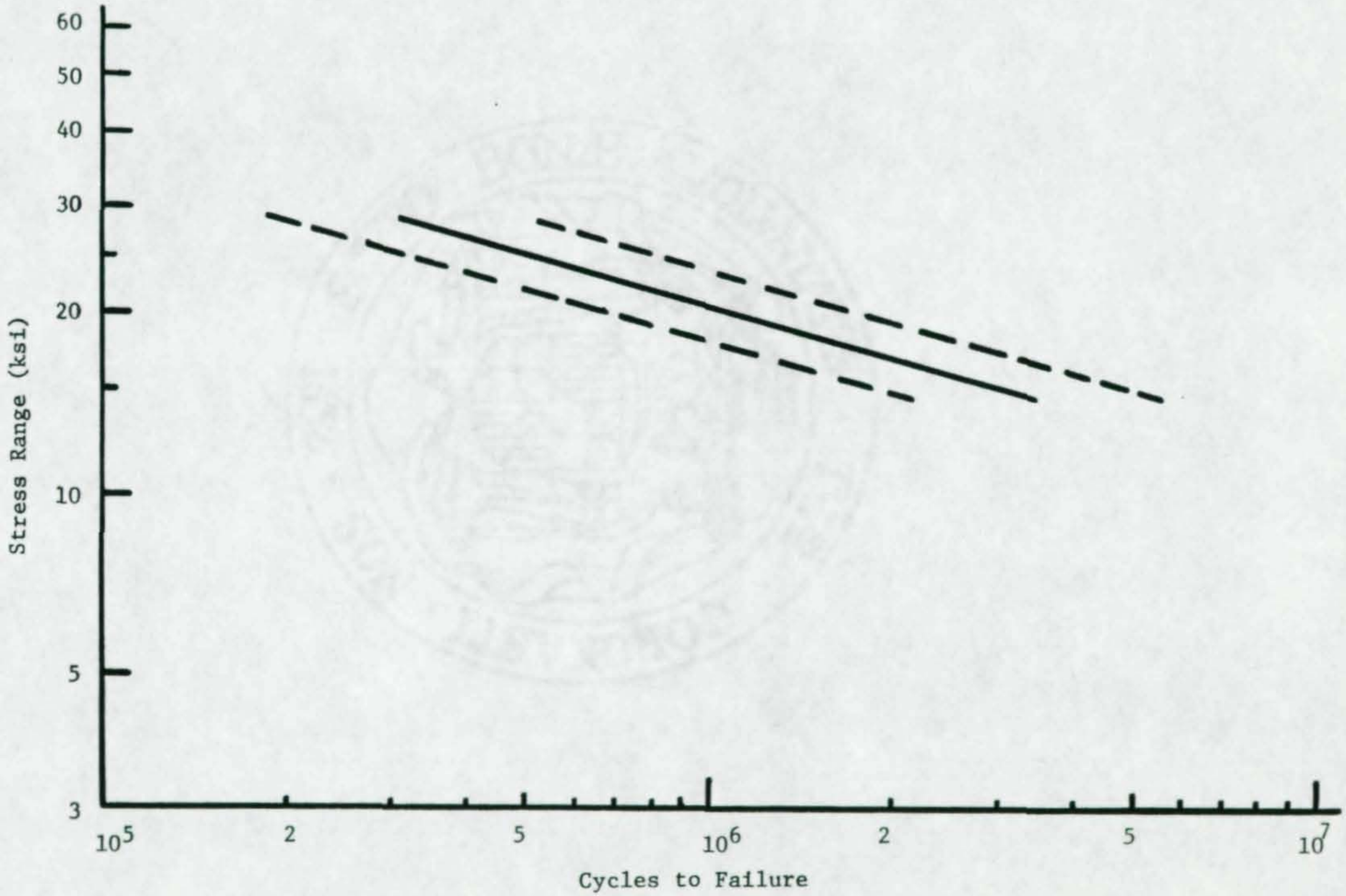
PLOT 3-5 PLAIN ROLLED BEAM DATA FOR ALL STEELS-A36.A441.A514.
DATA FROM REF. 2 AND REF. 3.



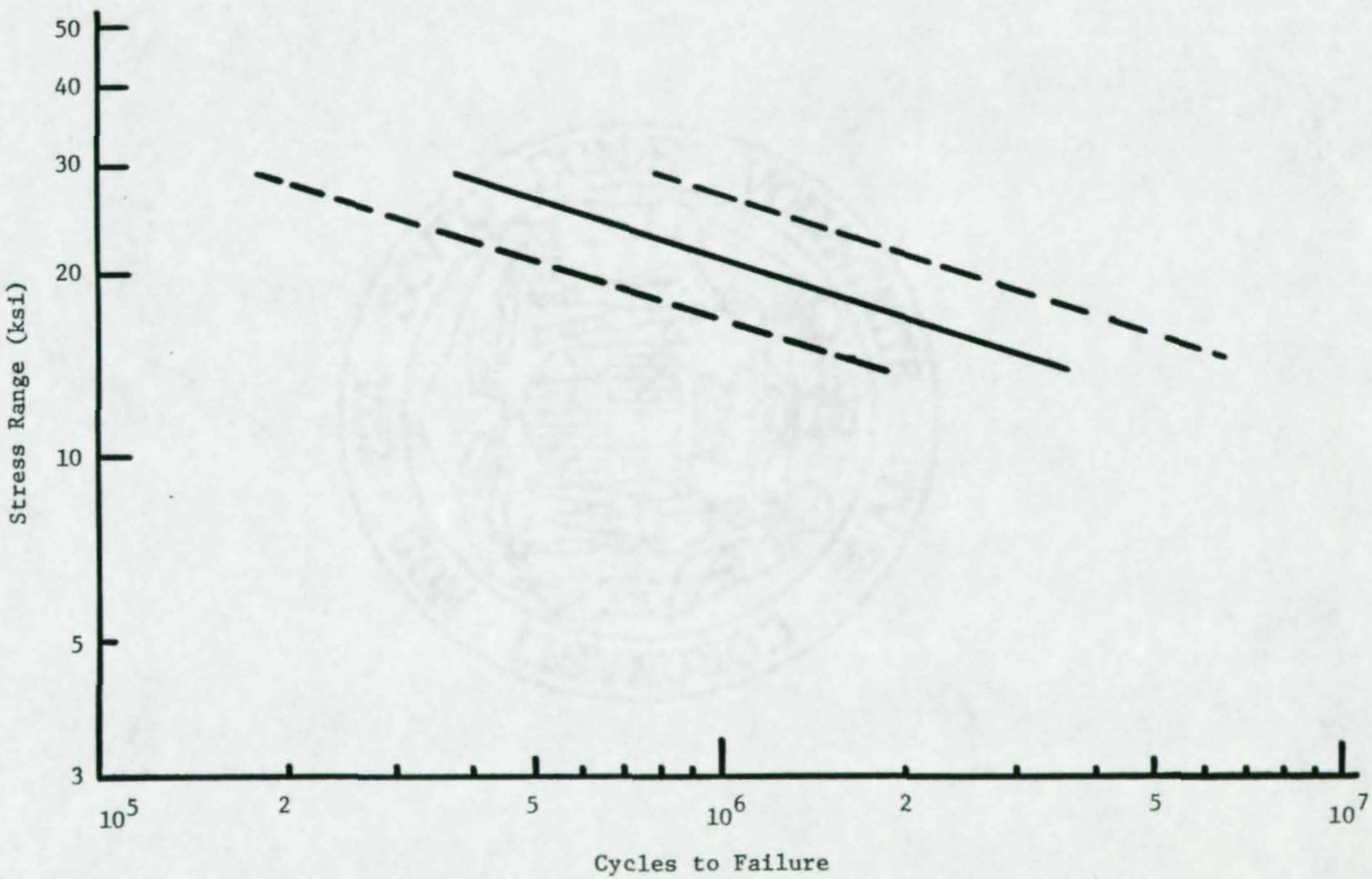
Plot 3-6 Type 1 Stiffeners - Welded to Web Alone in a Constant Moment Region
(Taken from Fig. 25 of Ref. 3)



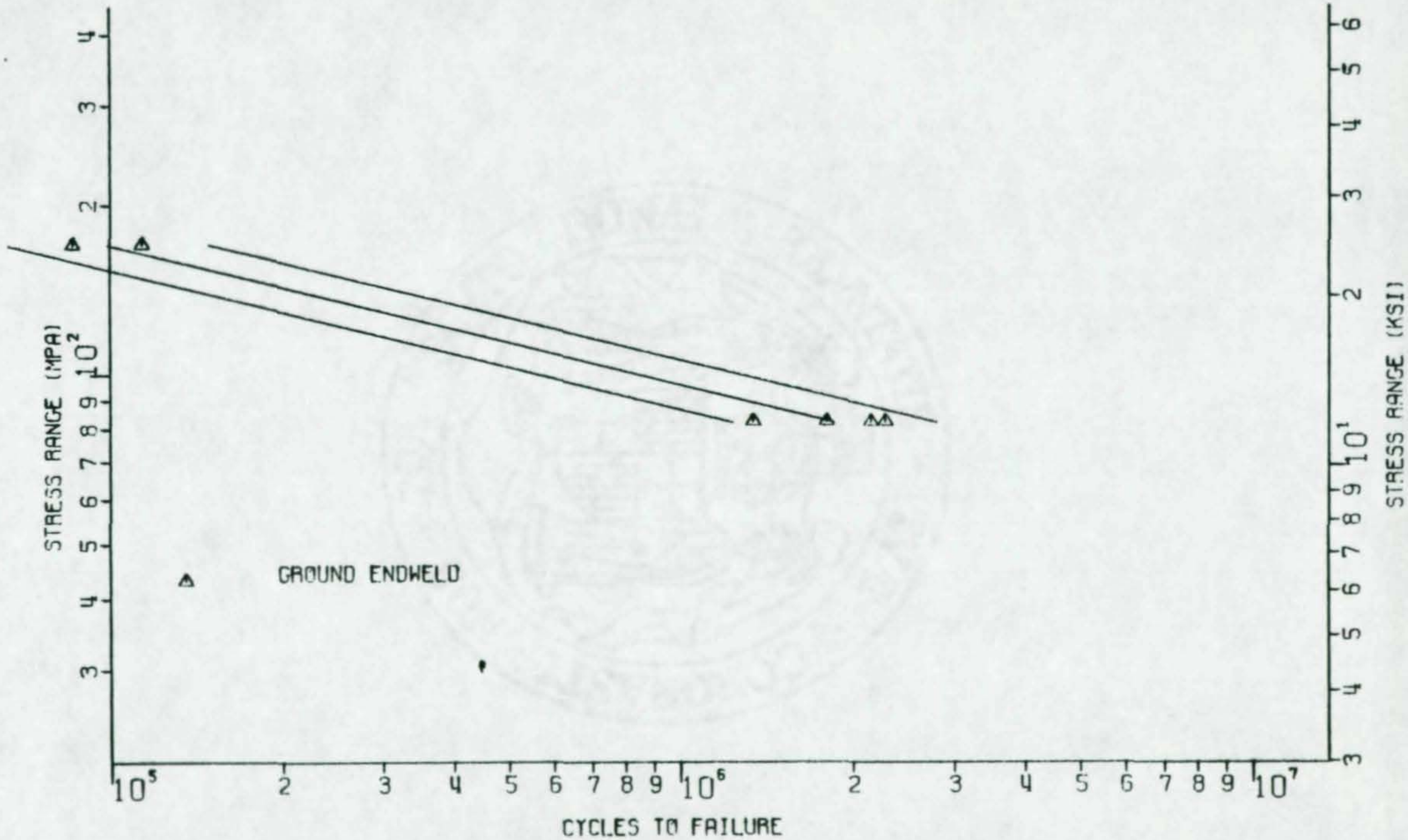
Plot 3-7 Type 2 Stiffeners - Welded to Web Alone in a Constant Moment Region
(Taken from Fig. 29 of Ref. 3)



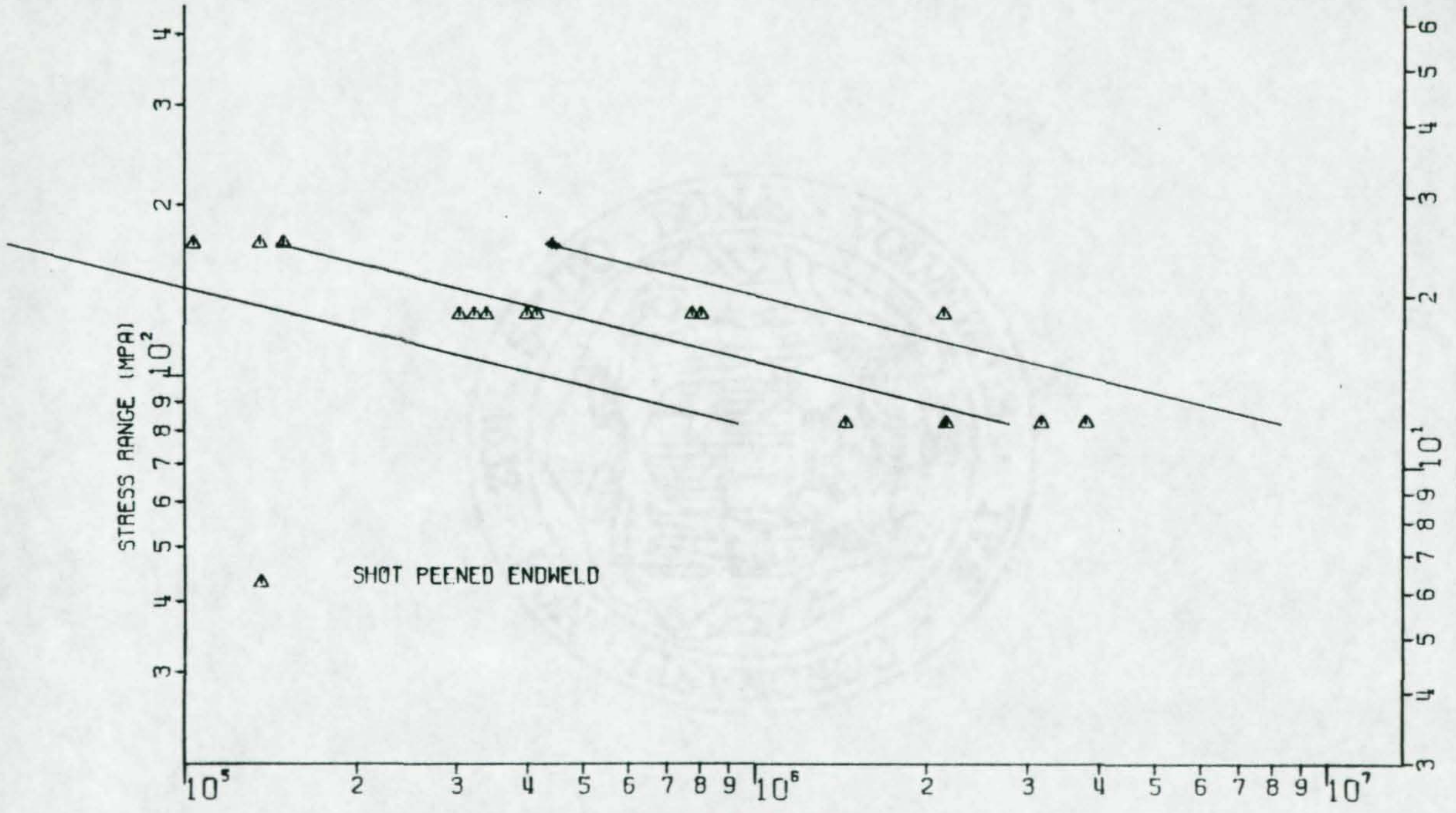
Plot 3-8 Type 3 Stiffeners - Welded to the Flange and Web (Taken from Fig. 29 of Ref. 3)



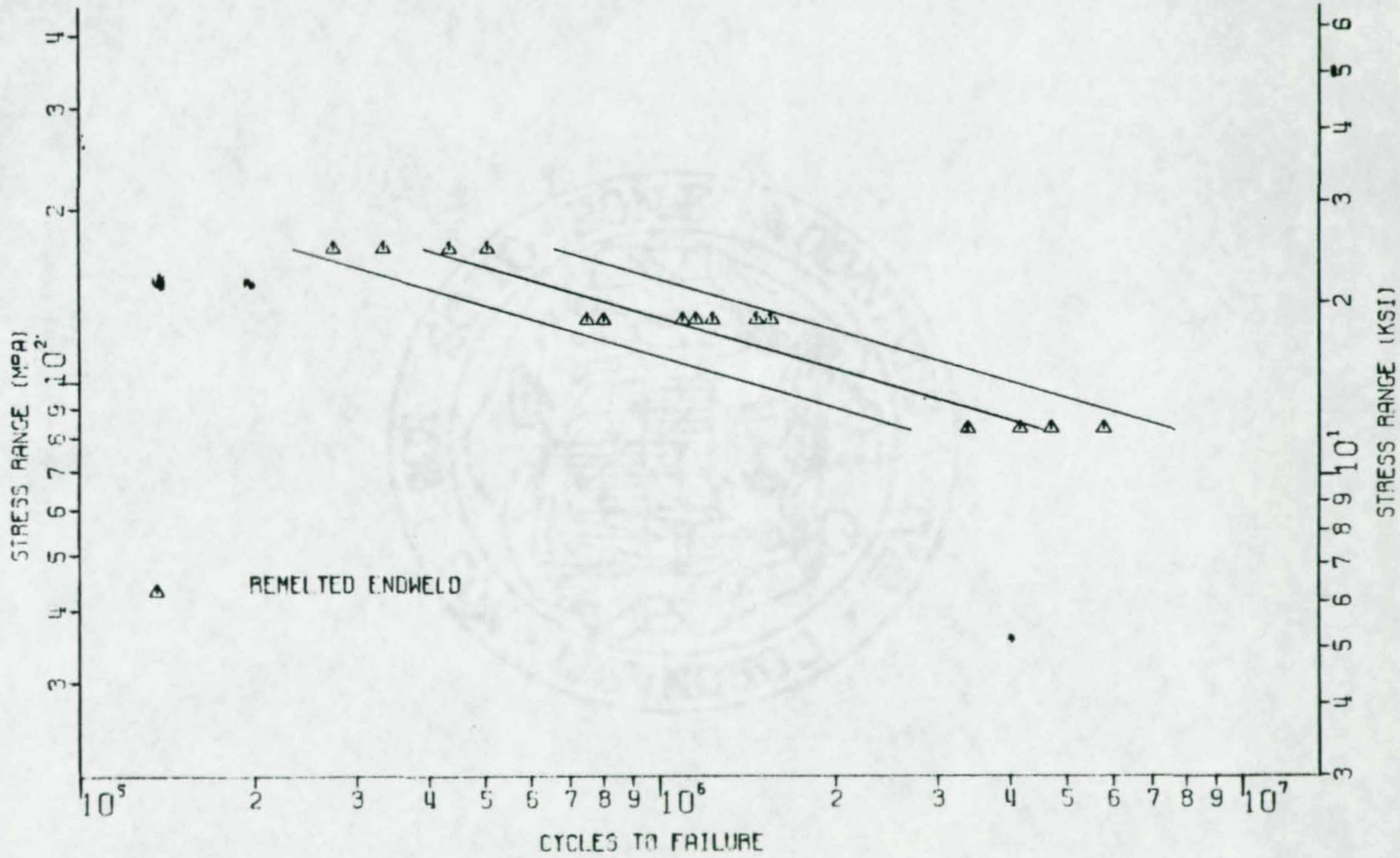
Plot 3-9 Type 1, 2 and 3 Stiffeners (Taken from Fig. 30 of Ref. 3)



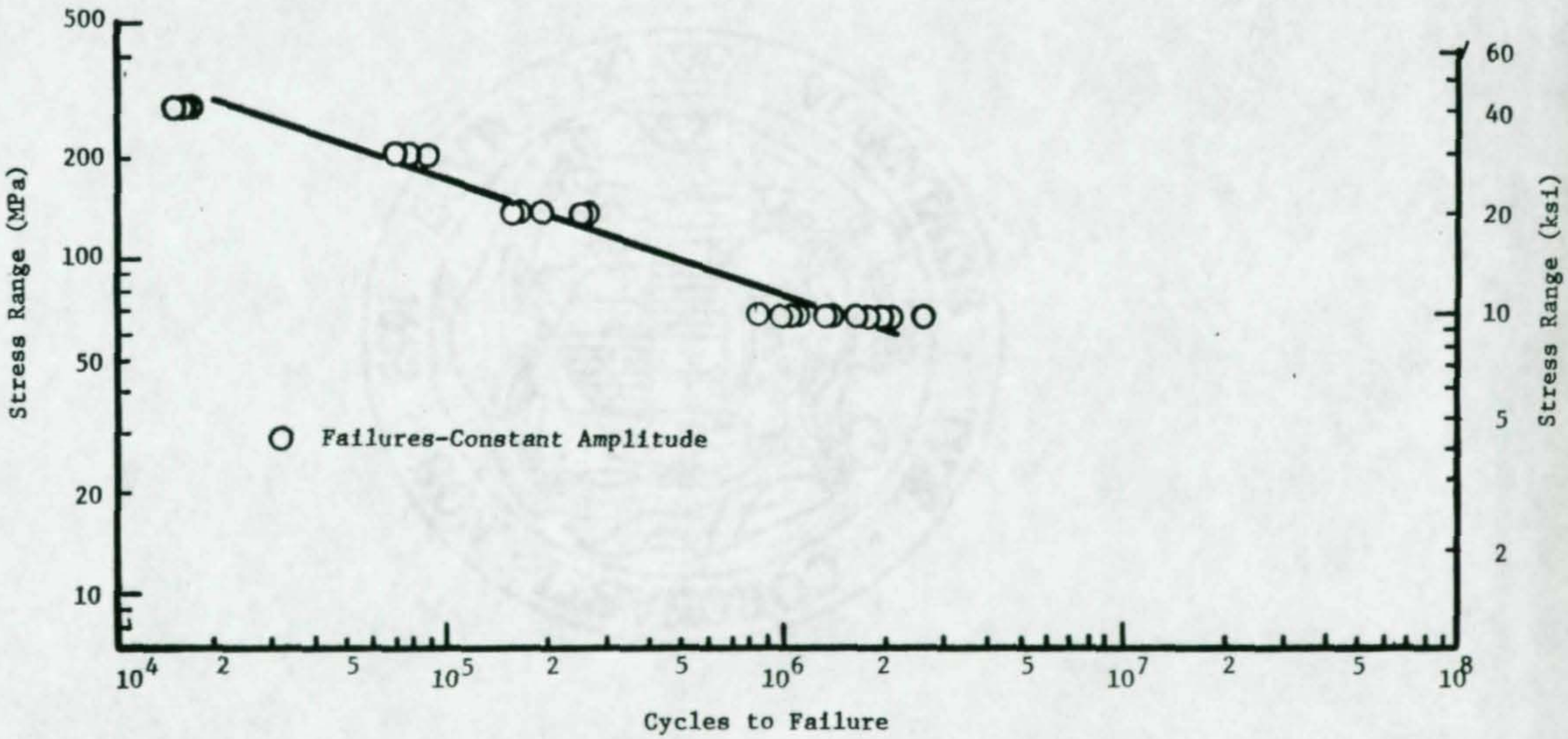
PLOT 4-1 COVER PLATE WITH GROUND ENDWELDS
(ADAPTED FROM REF. 4, FIG. 2.7).



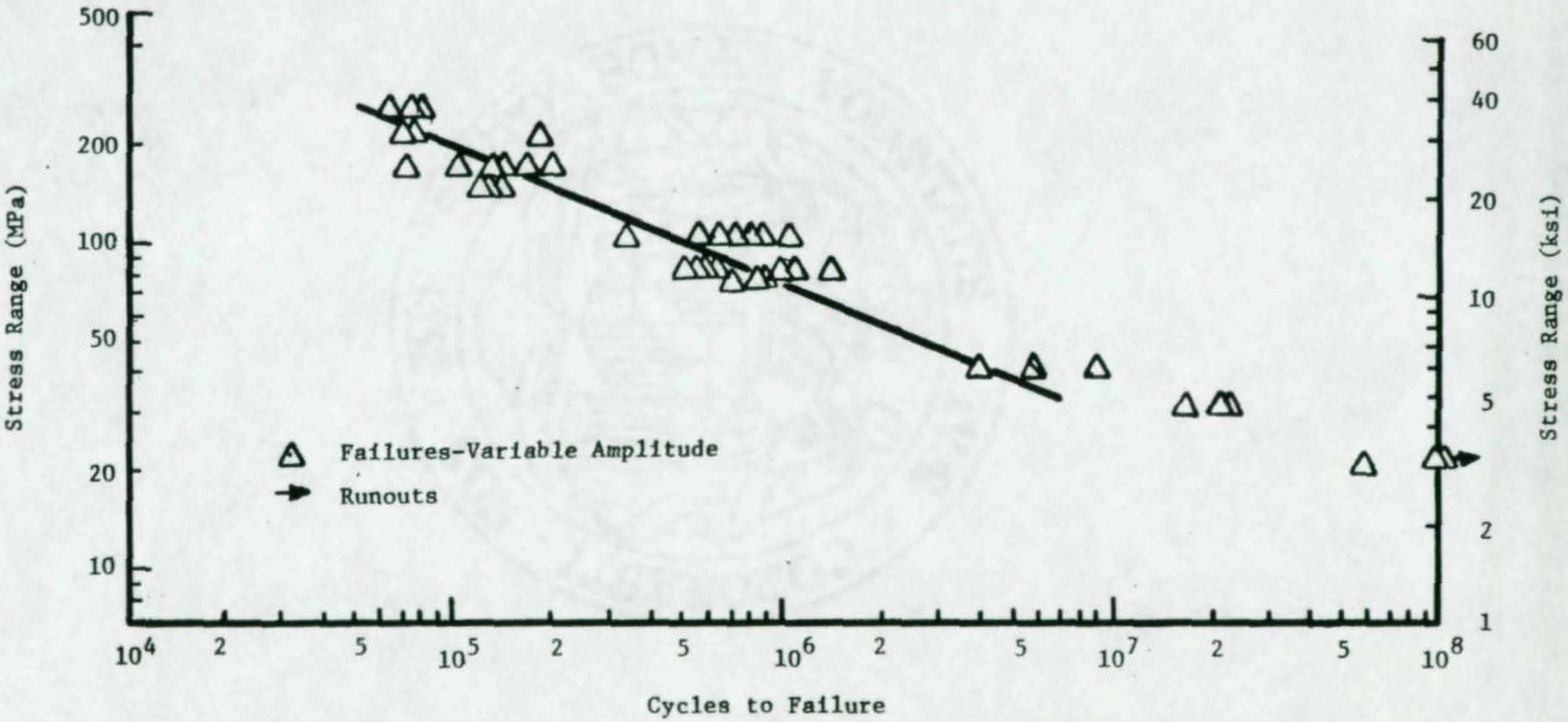
PLOT 4-2 COVER PLATE WITH SHOT-PEENED ENDWELDS
(ADAPTED FROM REF. 4, FIG. 2.7).



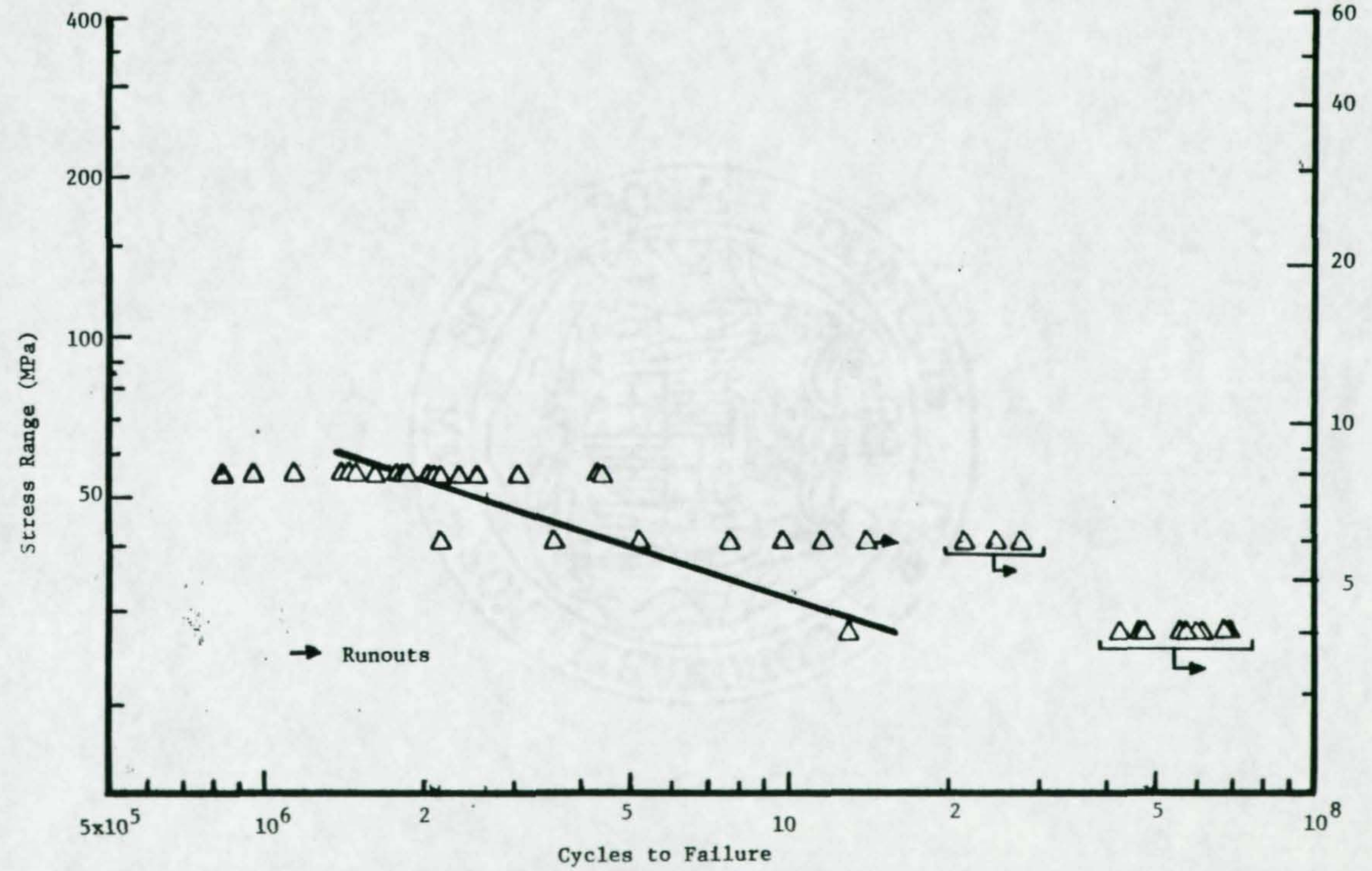
PLOT 4-3 COVER PLATE WITH REMELTED ENDWELDS
(ADAPTED FROM REF. 4, FIG. 2.7).



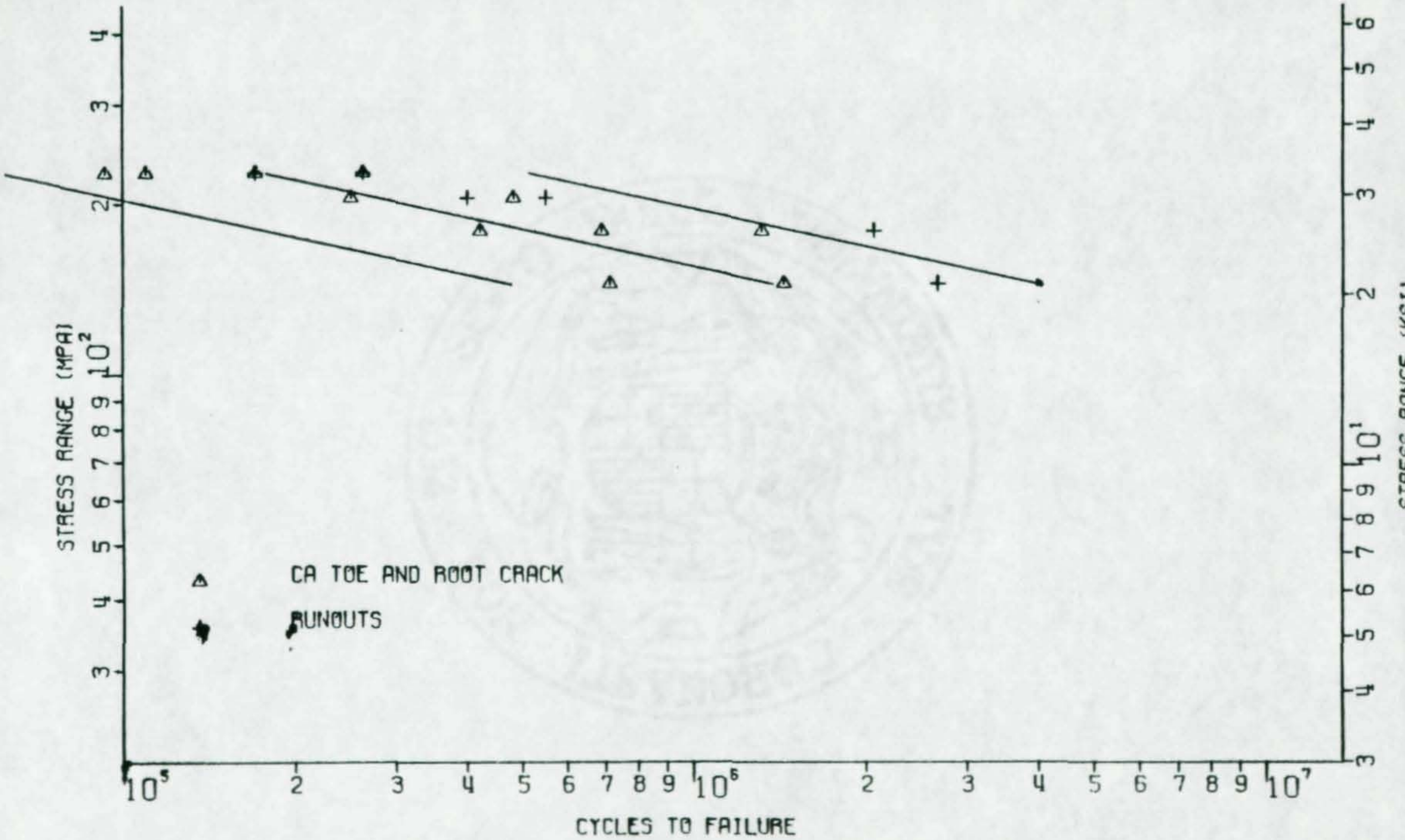
Plot 4-4 Cover Plates Series B and C, Constant Amplitude(Adapted from Ref. 4, Fig. 2.10)



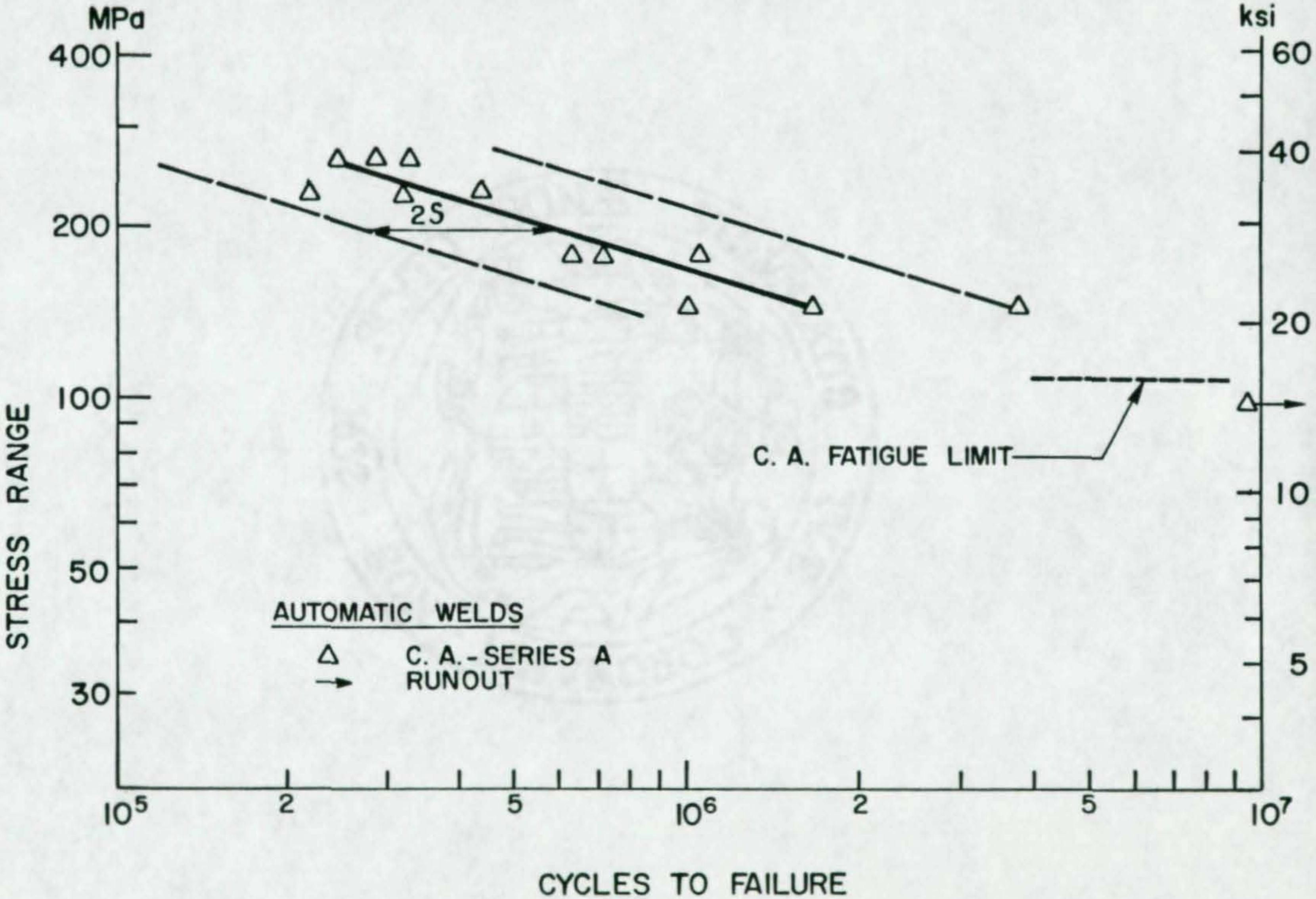
Plot 4-5 Cover Plates Series B and C, Variable Amplitude(Adapted from Ref. 4, Fig.2.10)



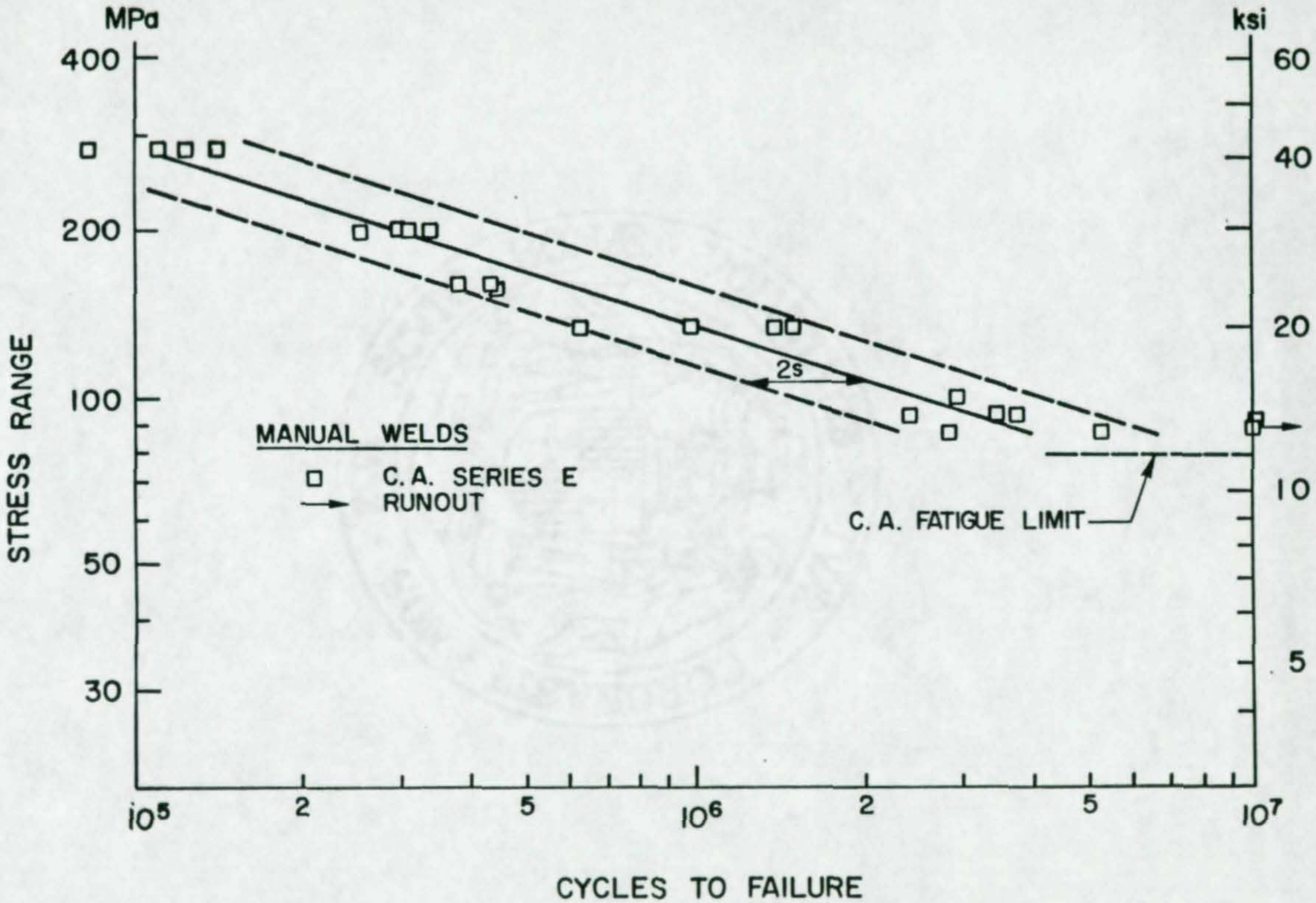
Plot 4-6 Full Size Cover Plated Beams(Adapted from Ref. 4, Fig. 2.13)



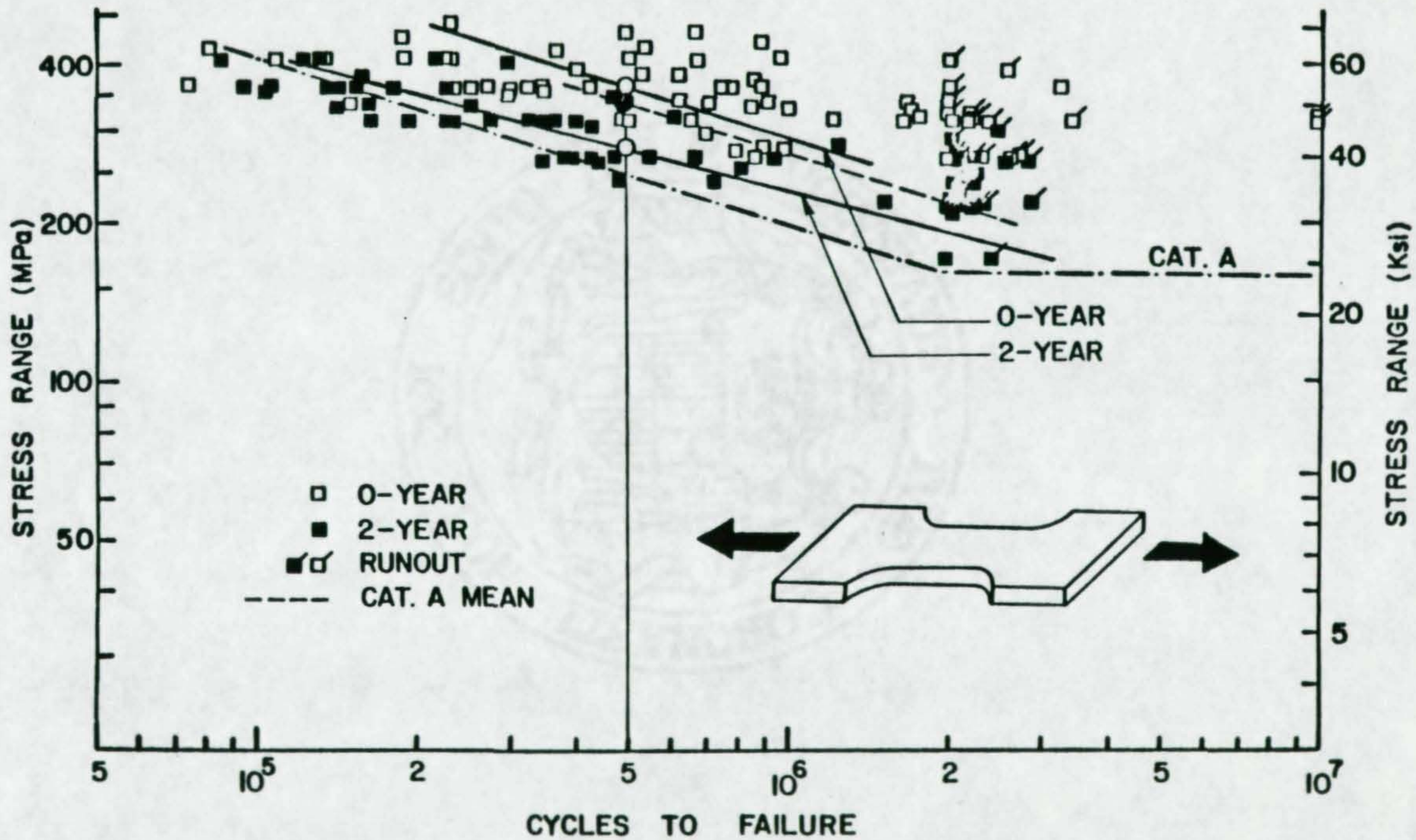
PLOT 5-1 COVER PLATE ENDWELD WITH 1:3 TAPERED TRANSITION (REF. 5).



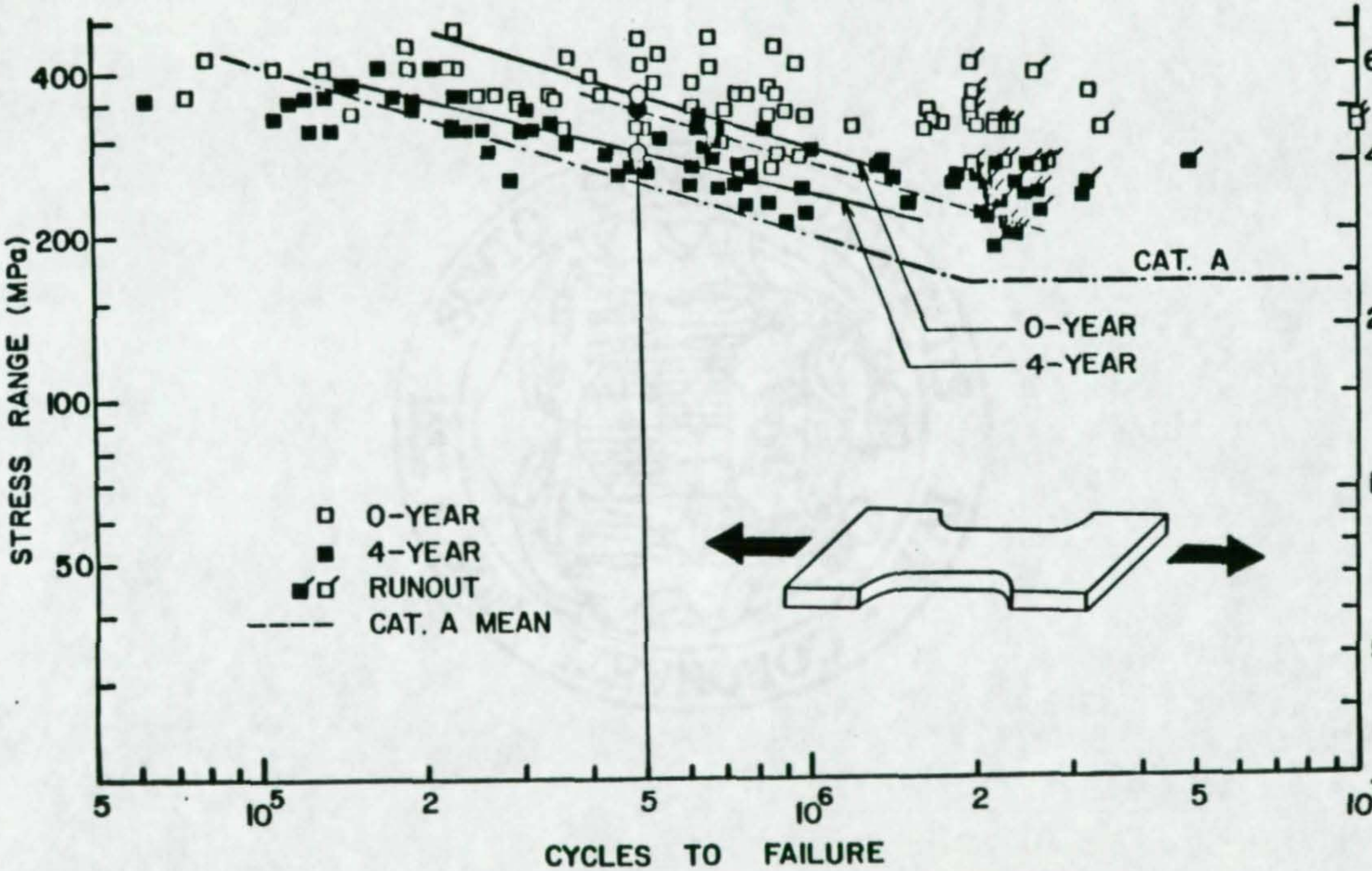
Plot 6-1 Automatically Welded Stiffeners (Taken from Fig. 5 of Ref. 6)



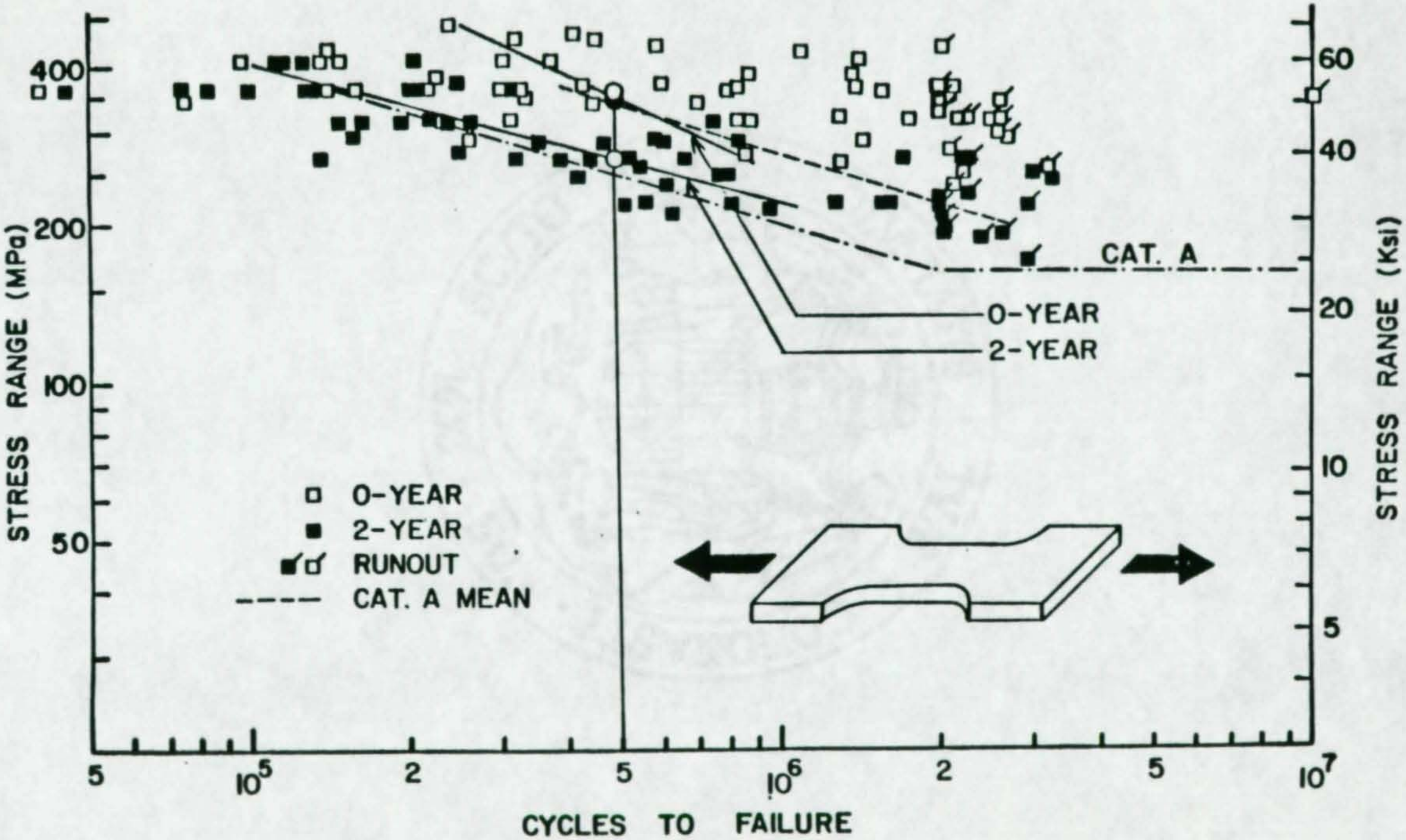
Plot 6-2 Manually Welded Stiffeners (taken from Fig. 9 of Ref. 6)



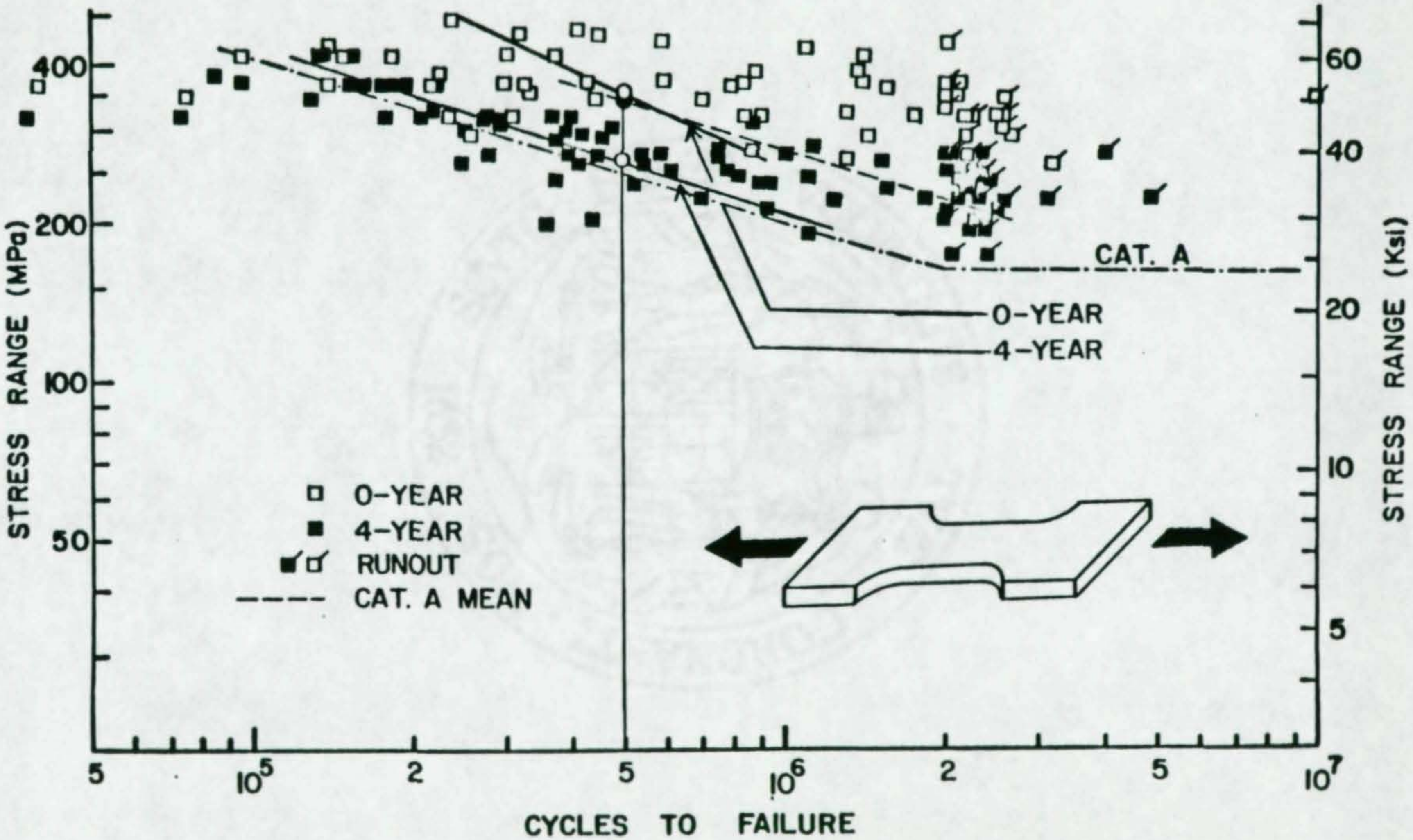
Plot 7-1 Plain Plate Specimens Weathered 0-Years and 2-Years, SMA 50 and SMA 58 Steels (Taken from Fig. 2 of Ref. 7)



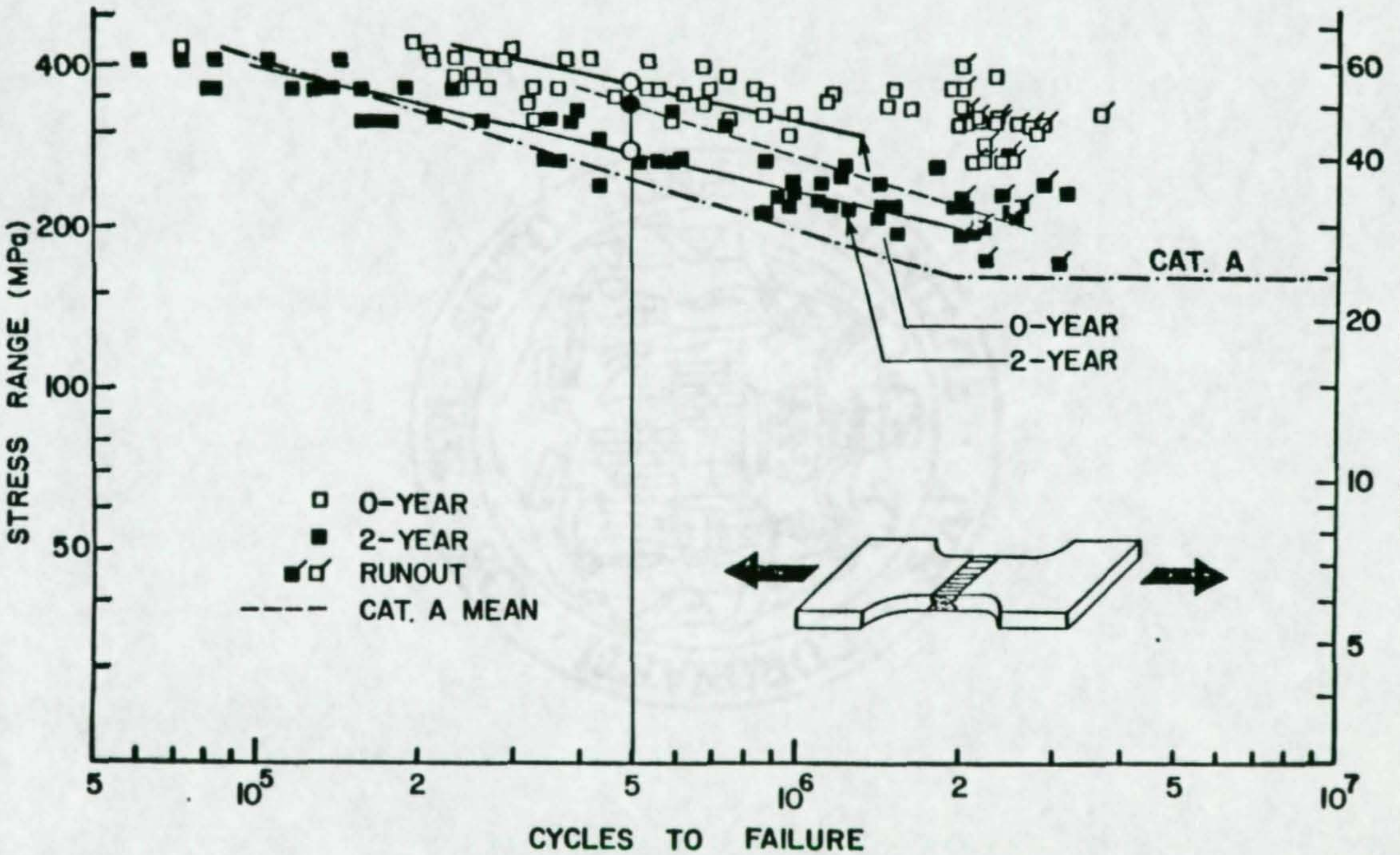
Plot 7-3 Plain Plate Specimens Weathered 4-Years, SMA 50 and SMA 58 Steels
(Taken from Fig. 3 of Ref. 7)



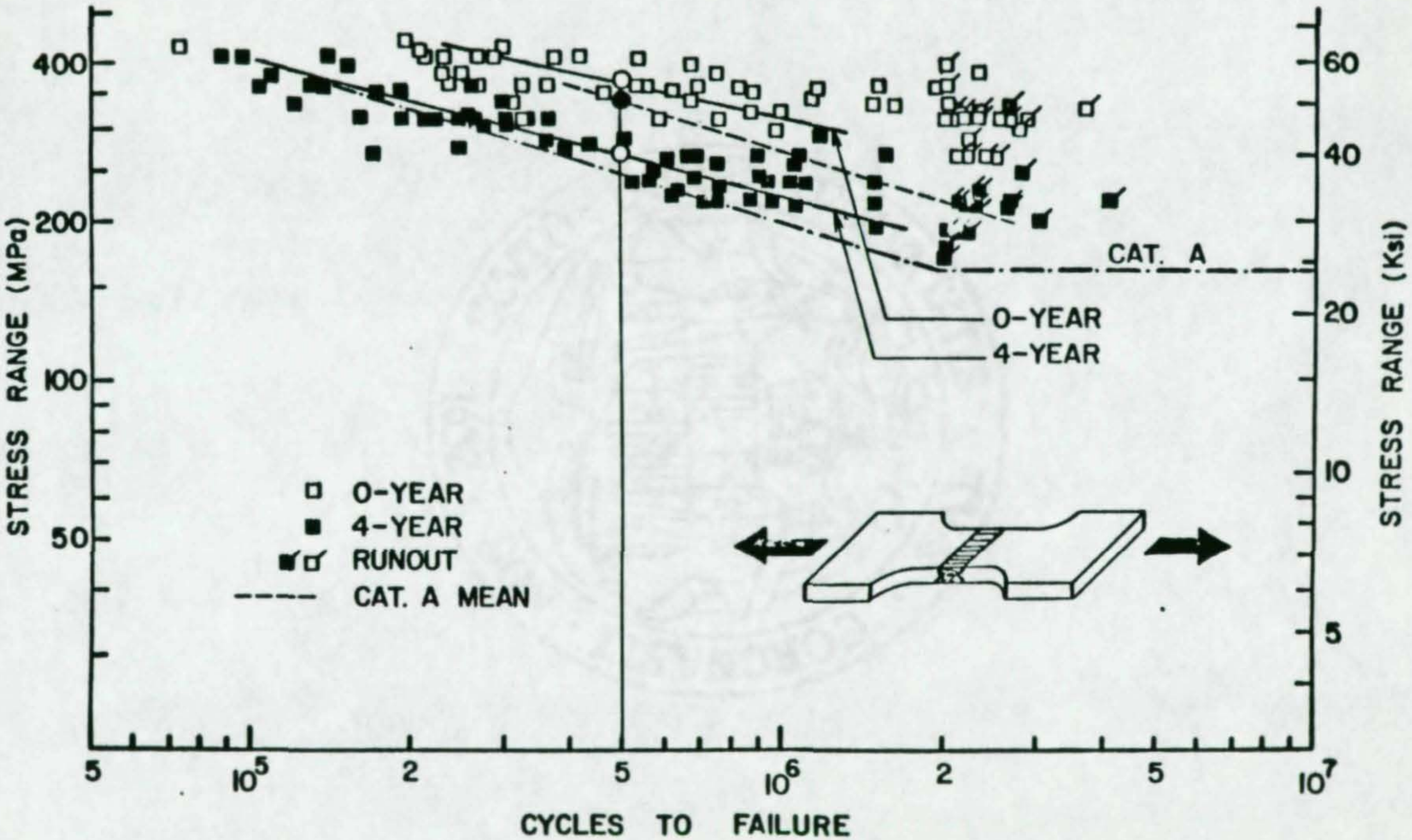
Plot 7-4 Plain Plate Specimens Weathered 0-Years and 2-Years, SM 50 and SM 58 Steels
(Taken from Fig. 7 of Ref. 7)



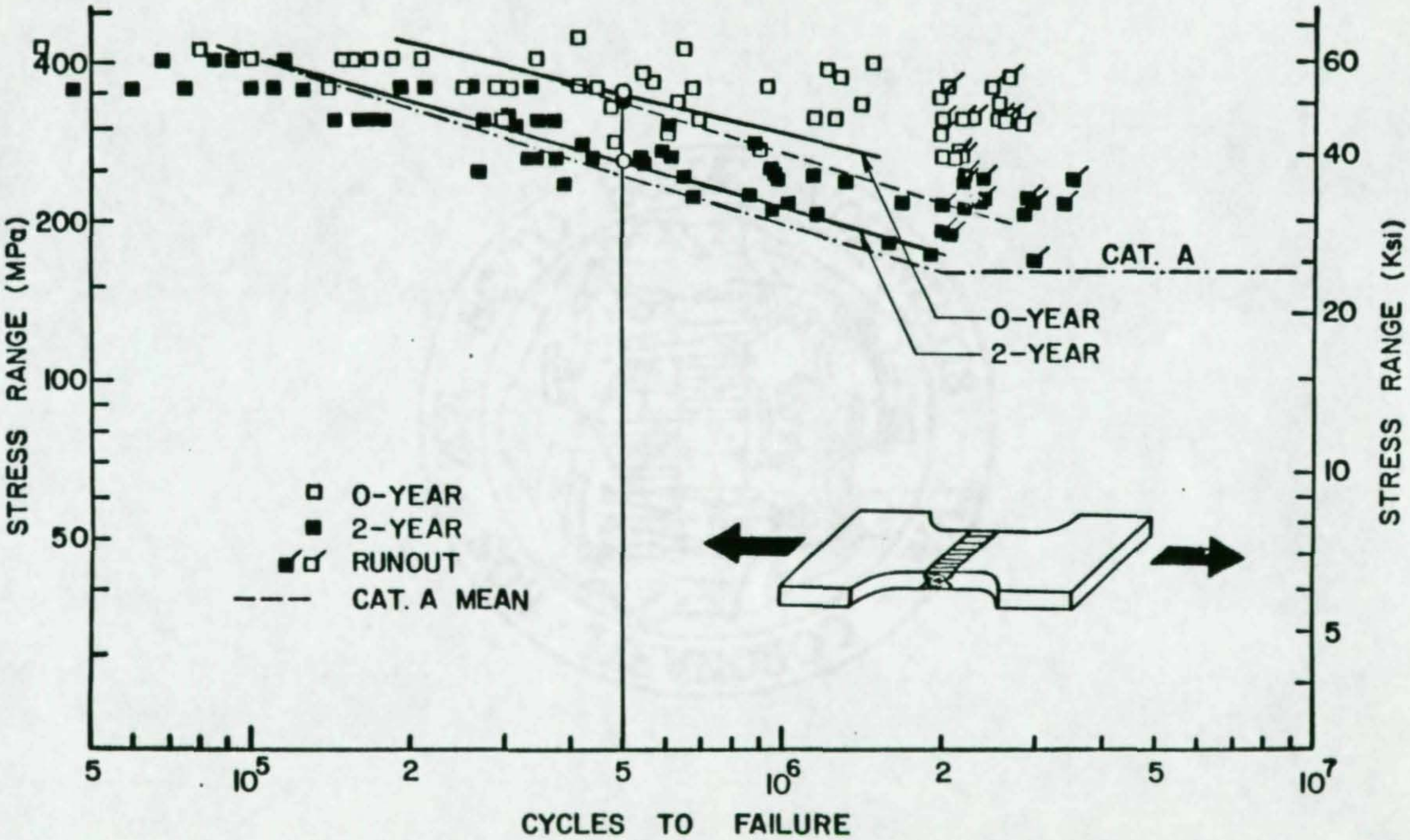
Plot 7-5 Plain Plate Specimens Weathered 4-Years, SM 50 and SM 58 Steels
(Taken from Fig. 8 of Ref. 7)



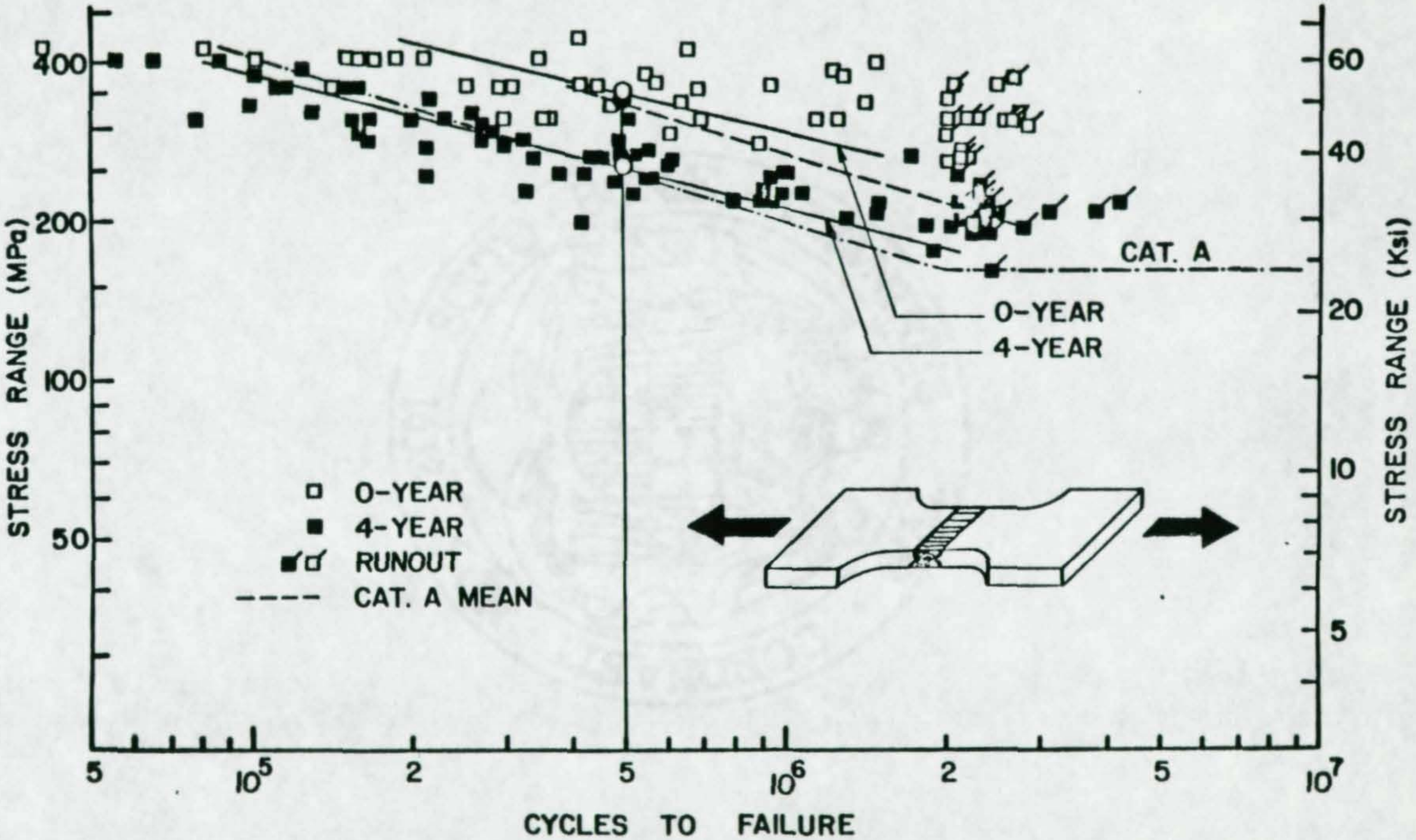
Plot 7-6 Butt Welded Specimens Weathered 0-Years and 2-Years, SMA 50 and SMA 58 Steels (Taken from Fig. 4 of Ref. 7)



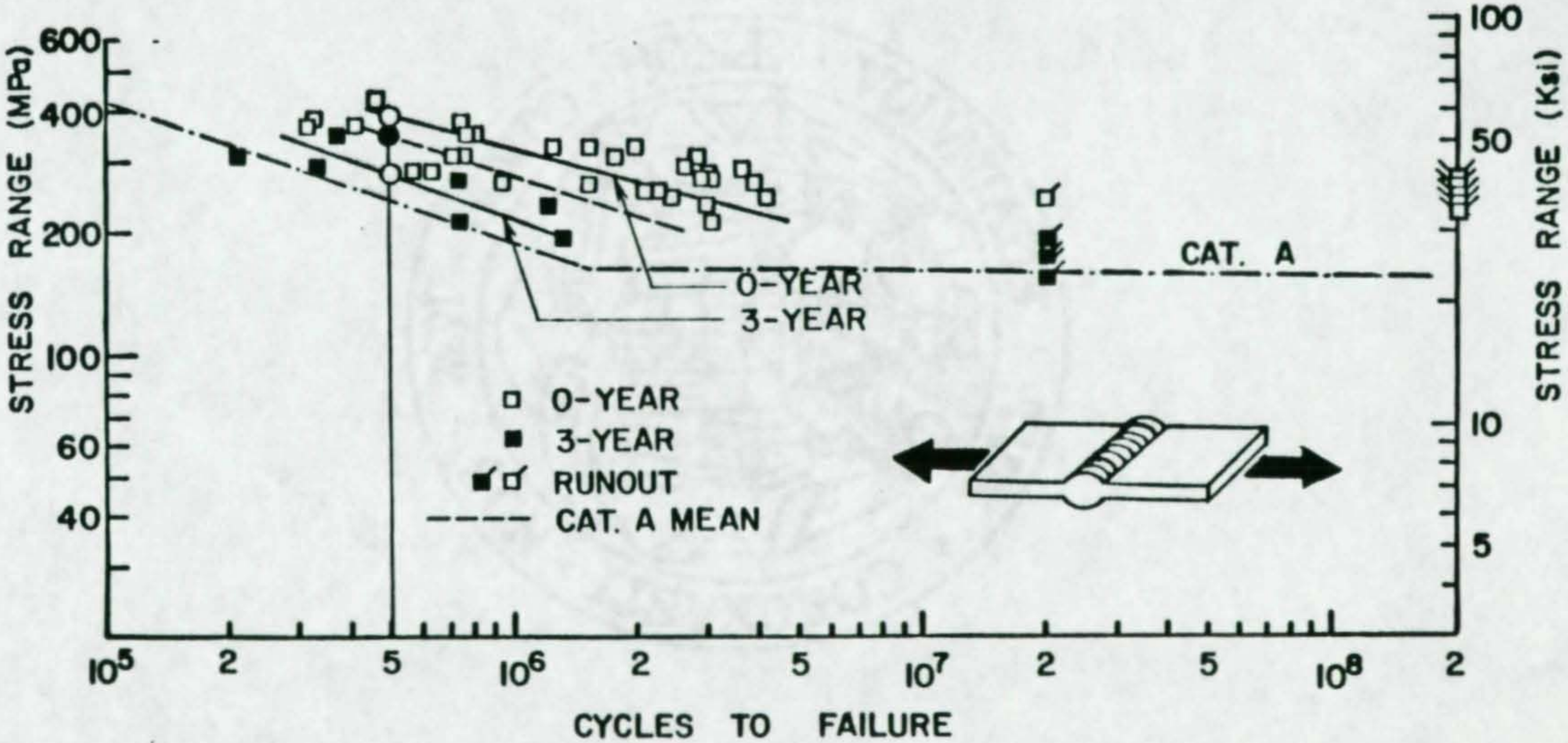
Plot 7-7 Butt Welded Specimens Weathered 4-Years, SMA 50 and SMA 58 Steels (Taken from Fig. 5 of Ref. 7)



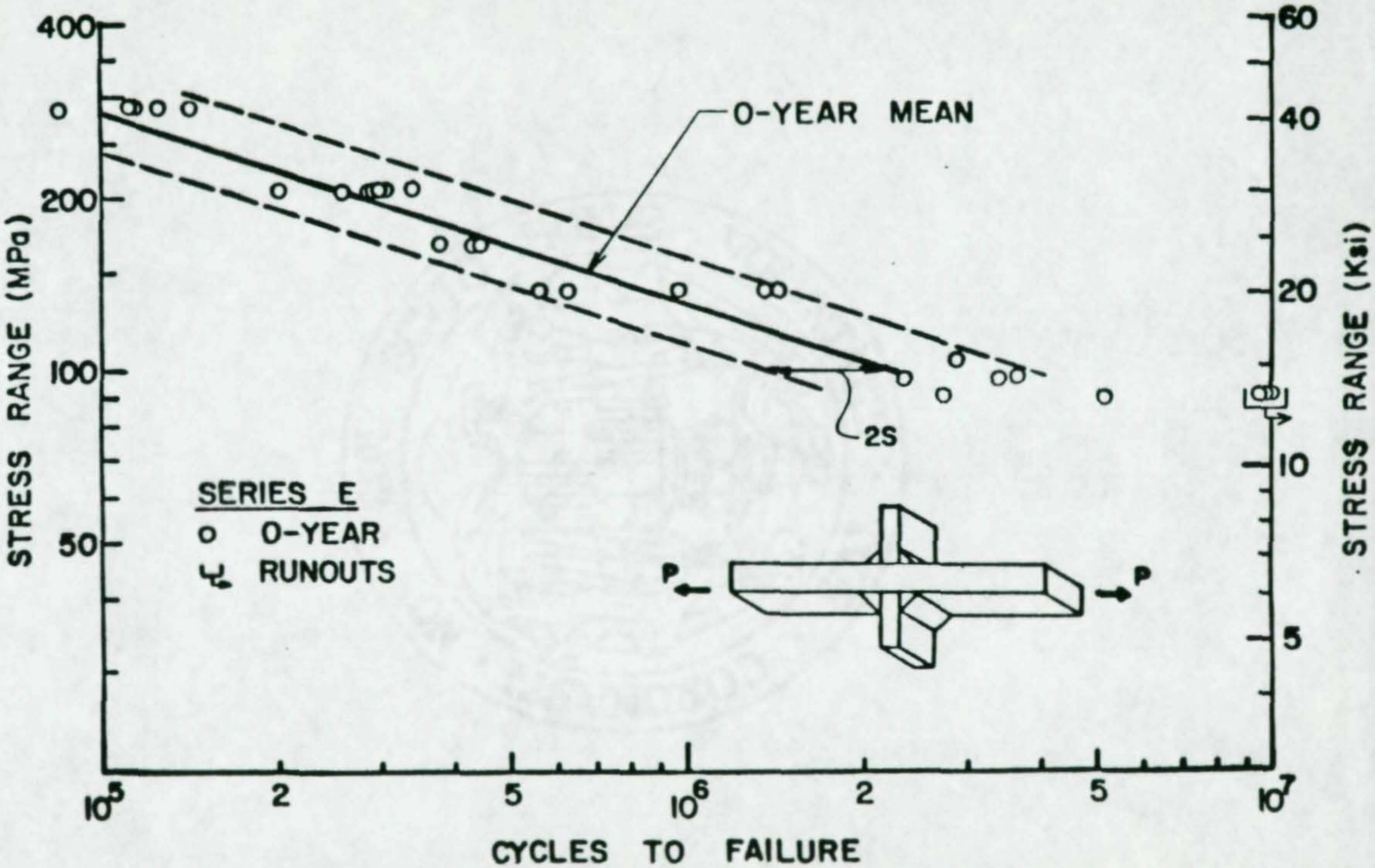
Plot 7-8 Butt Welded Specimens Weathered 0-Years and 2-Years, SM 50 and SM 58 Steels (Taken from Fig. 9 of Ref. 7)



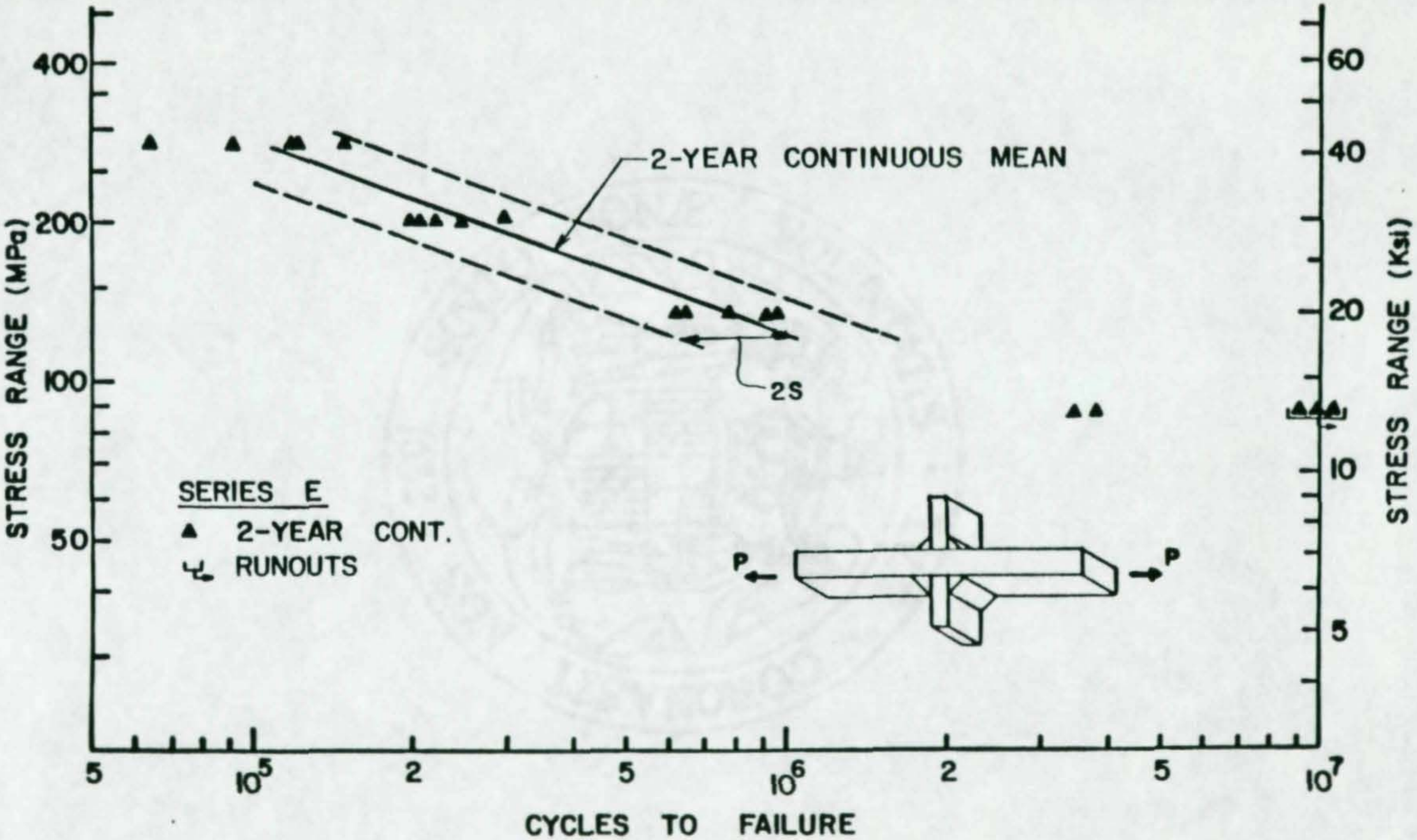
Plot 7-9 Butt Welded Specimens Weathered 4-Years, SM 50 and SM 58 steels (Taken from Fig. 10 of Ref. 7)



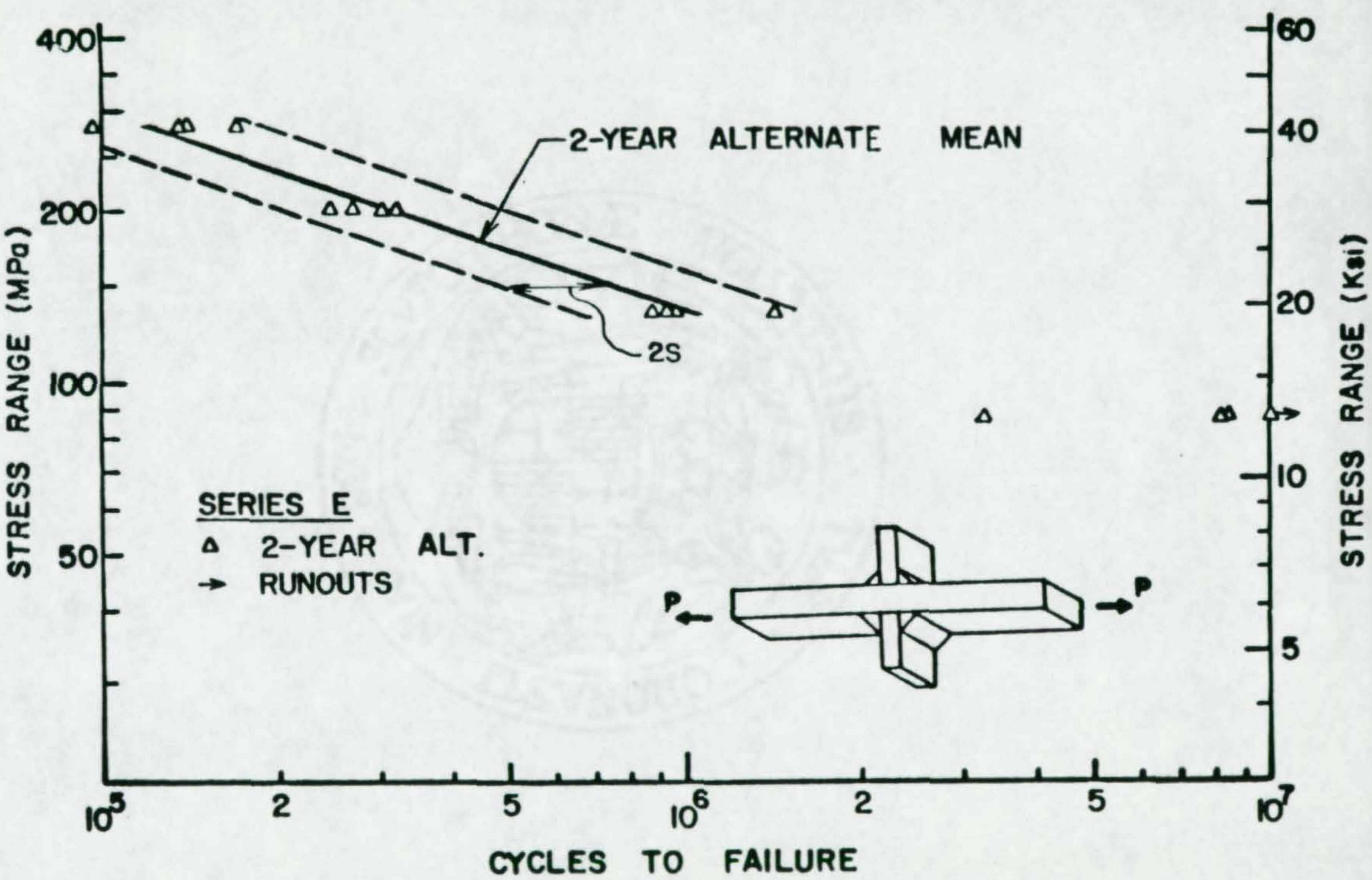
Plot 7-10 Butt Welded Specimens Weathered 0-Years and 3-Years, SM 50 A Steel
(Taken from Fig. 14 of Ref. 7)



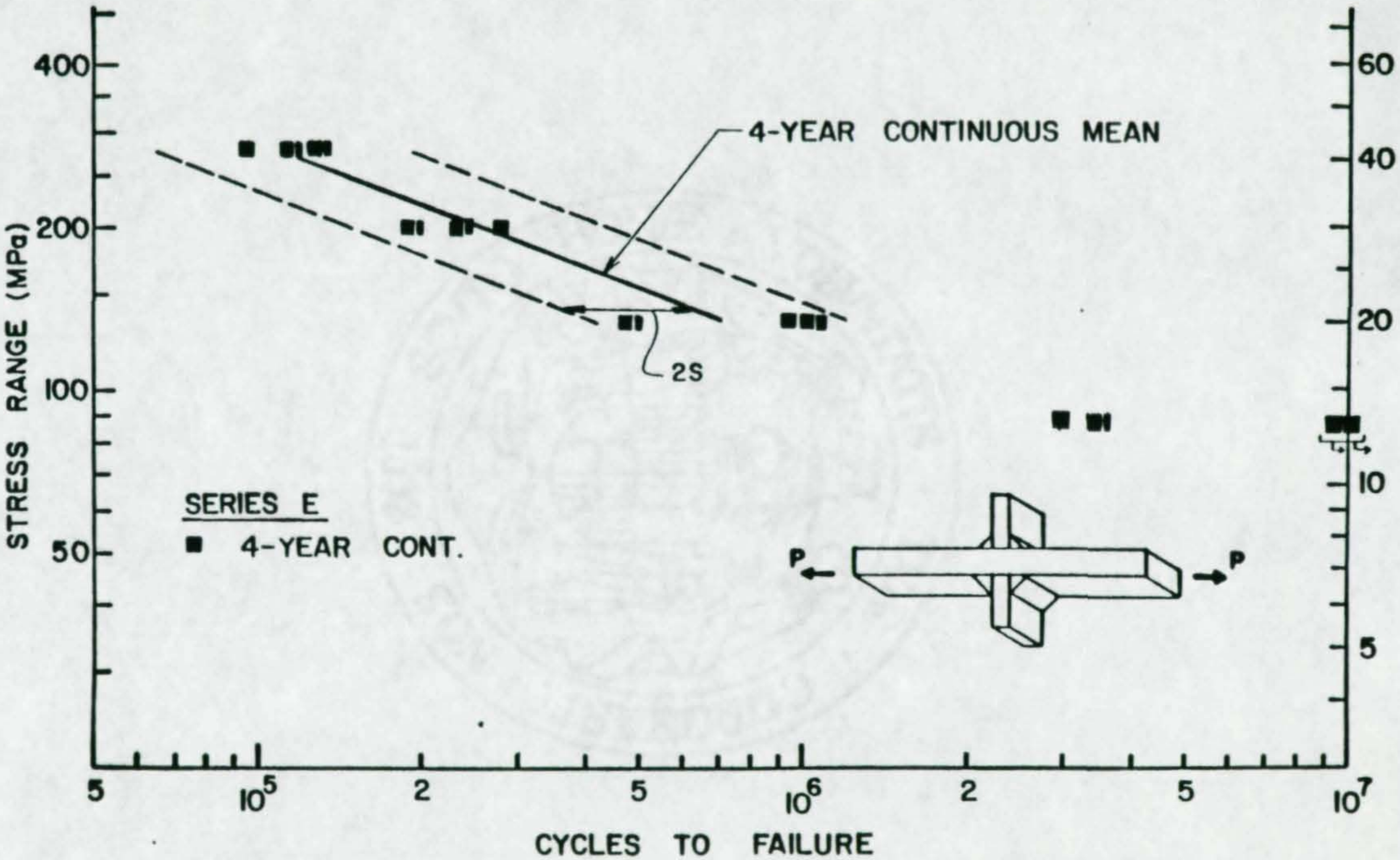
Plot 7-11 Transverse Stiffeners Weathered 0-Years, A588 Steel
(Taken from Fig. 26 of Ref. 7)



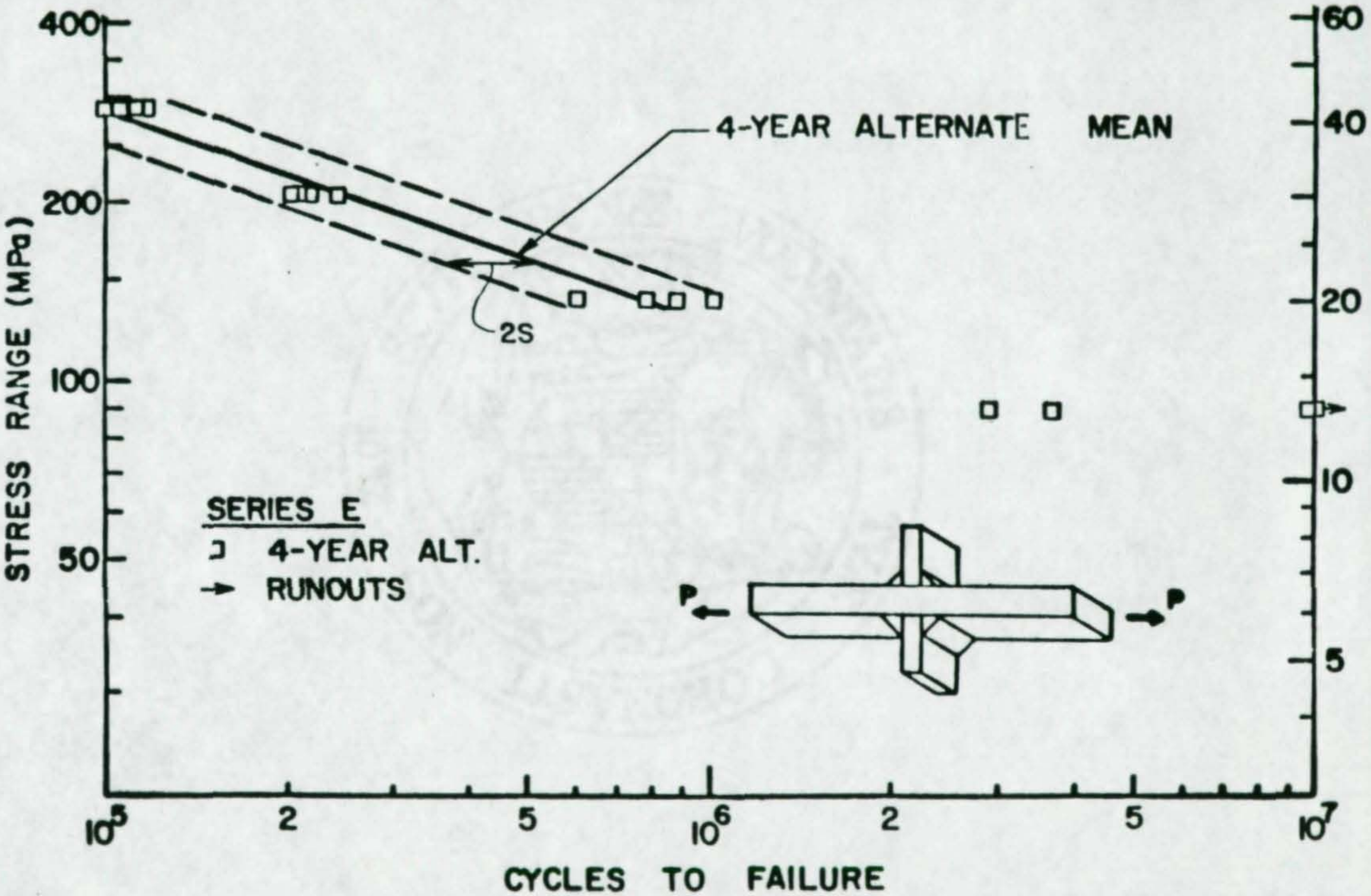
Plot 7-12 Transverse Stiffeners after 2-Year Continuous Weathering, A588 Steel
(Taken from Fig. 28 of Ref. 7)



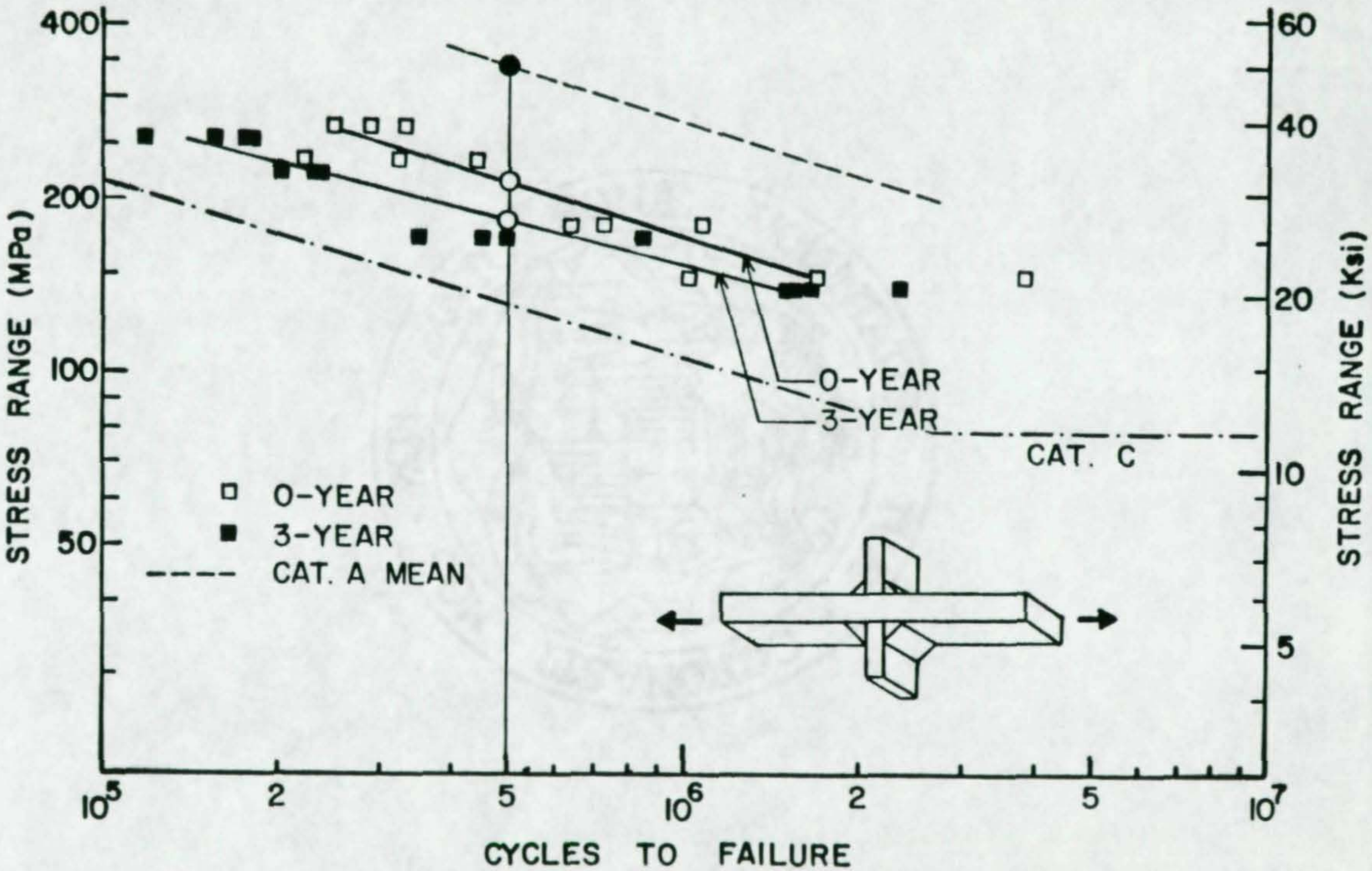
Plot 7-13 Transverse Stiffeners after 2-Year Alternate Weathering, A588 Steel
(Taken from Fig. 29 of Ref. 7)



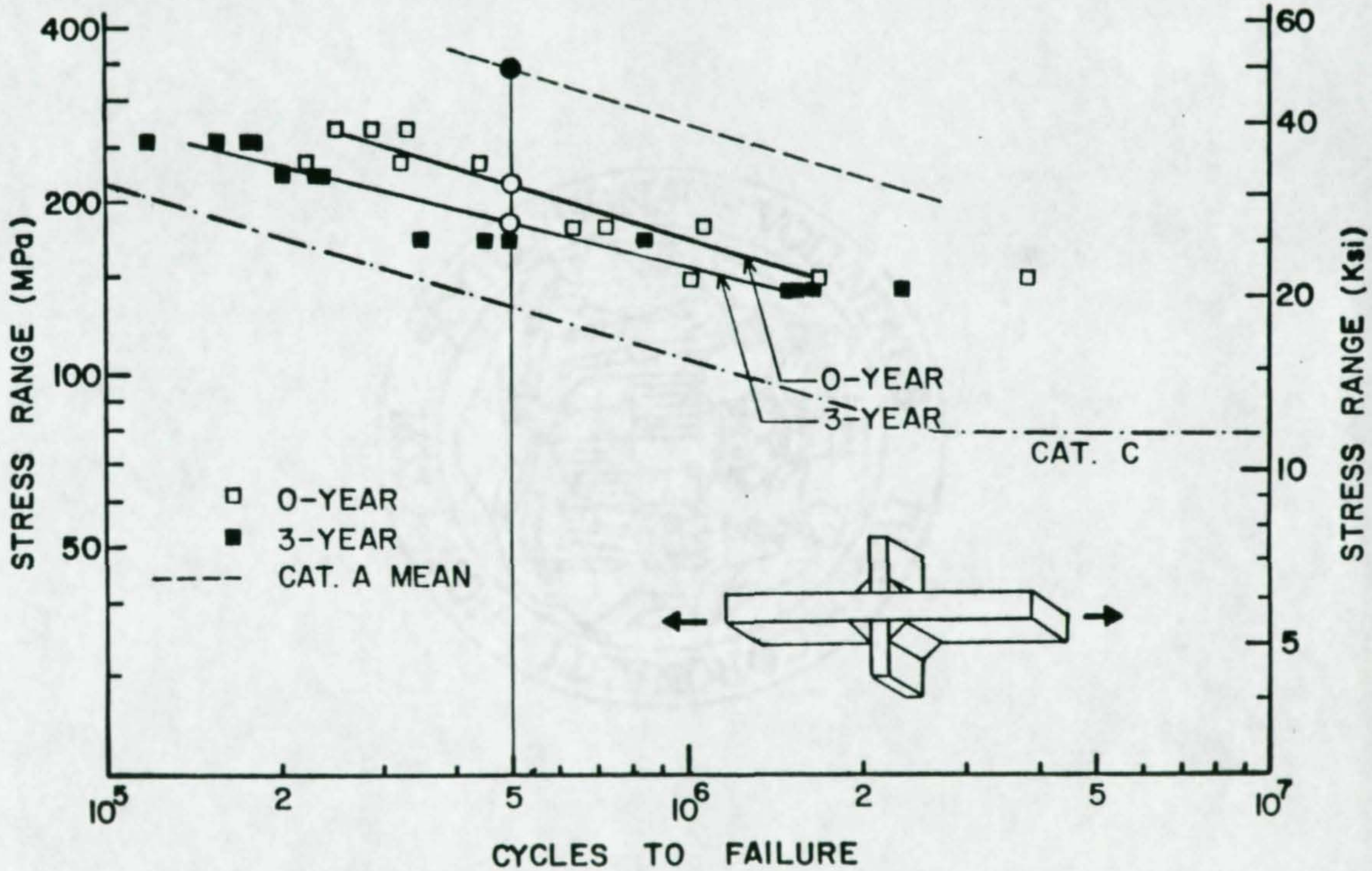
Plot 7-14 Transverse Stiffeners after 4-Year Continuous Weathering, A588 Steel (Taken from Fig. 30 of Ref. 7)



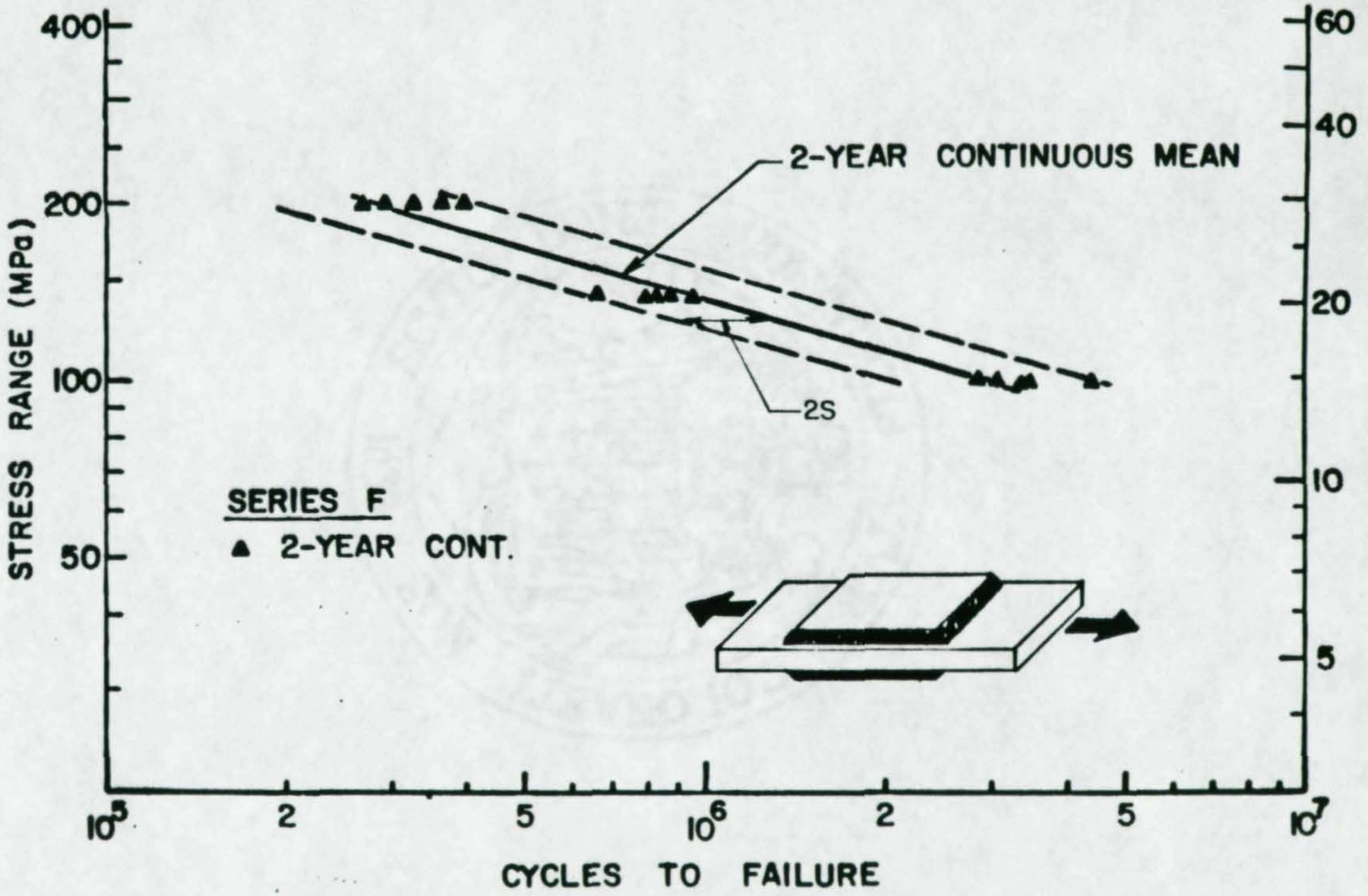
Plot 7-15 Transverse Stiffeners after 4-Year Alternate Weathering, A588 Steel (Taken from Fig. 31 of Ref. 7)



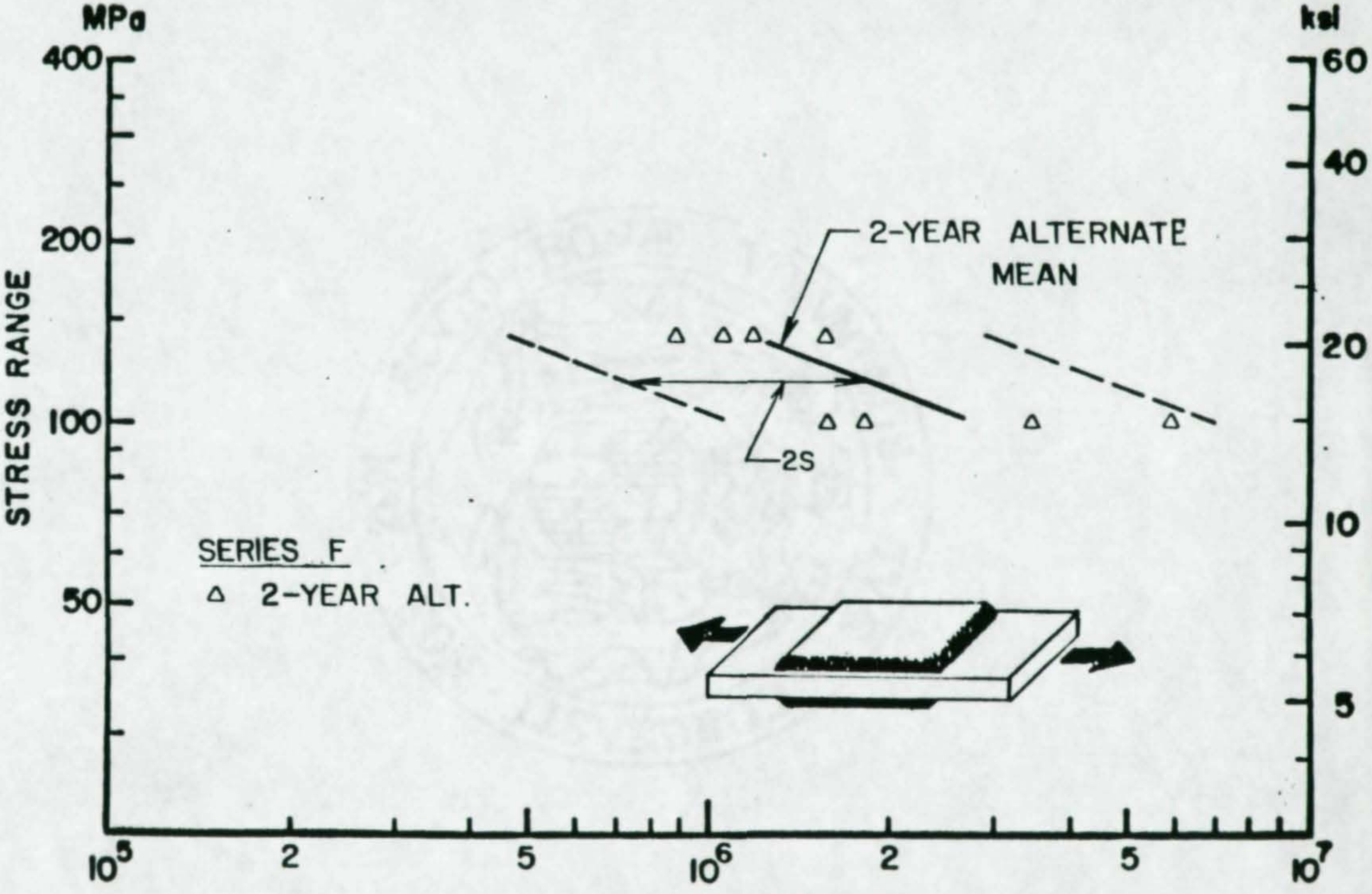
Plot 7-16 Transverse Stiffeners Weathered 0-Years, A588 Steel(Taken from Fig. 6 of Ref. 7)



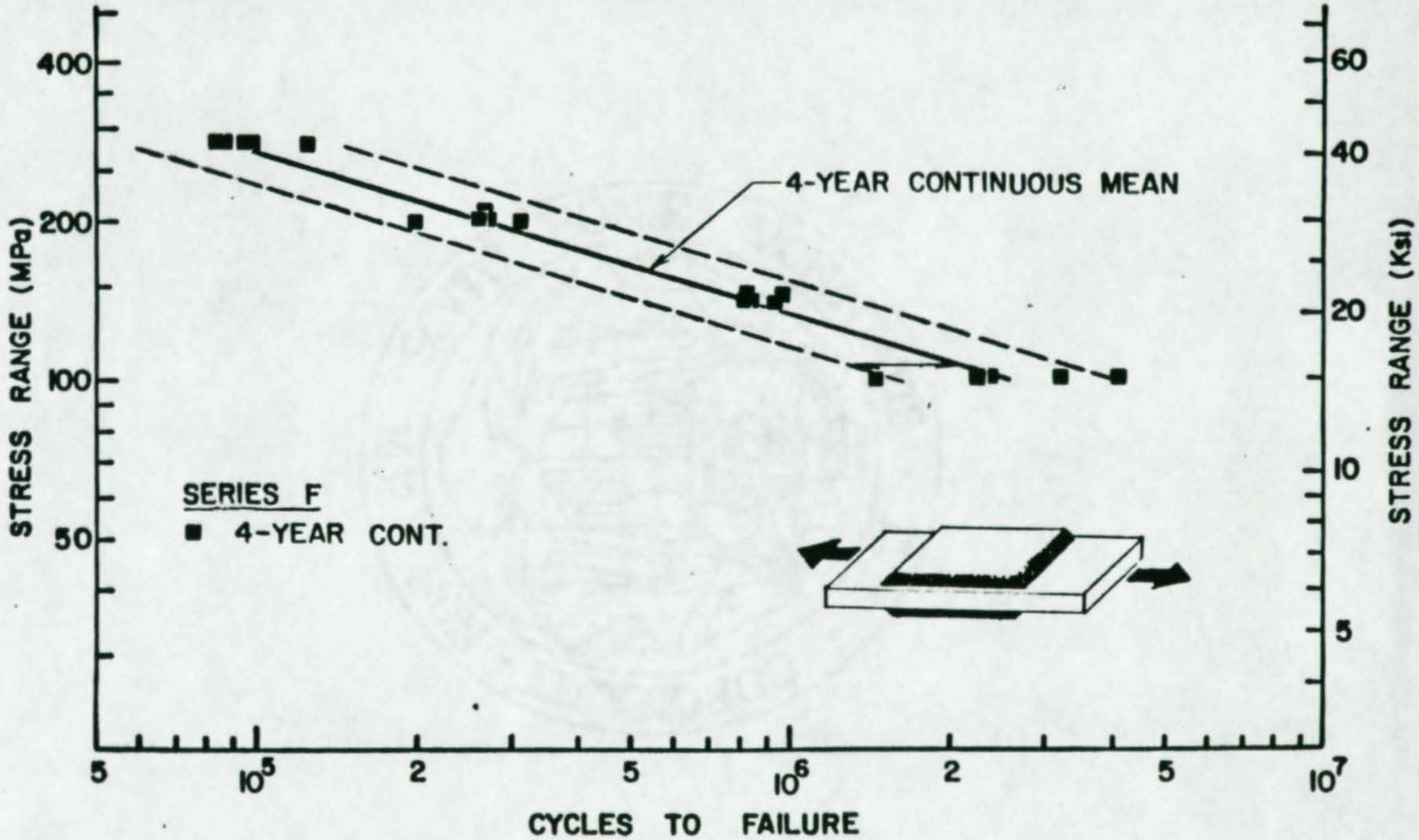
Plot 7-17 Transverse Stiffeners Weathered 3-Years, A588 Steel (Taken from Fig. 6 of Ref. 7)



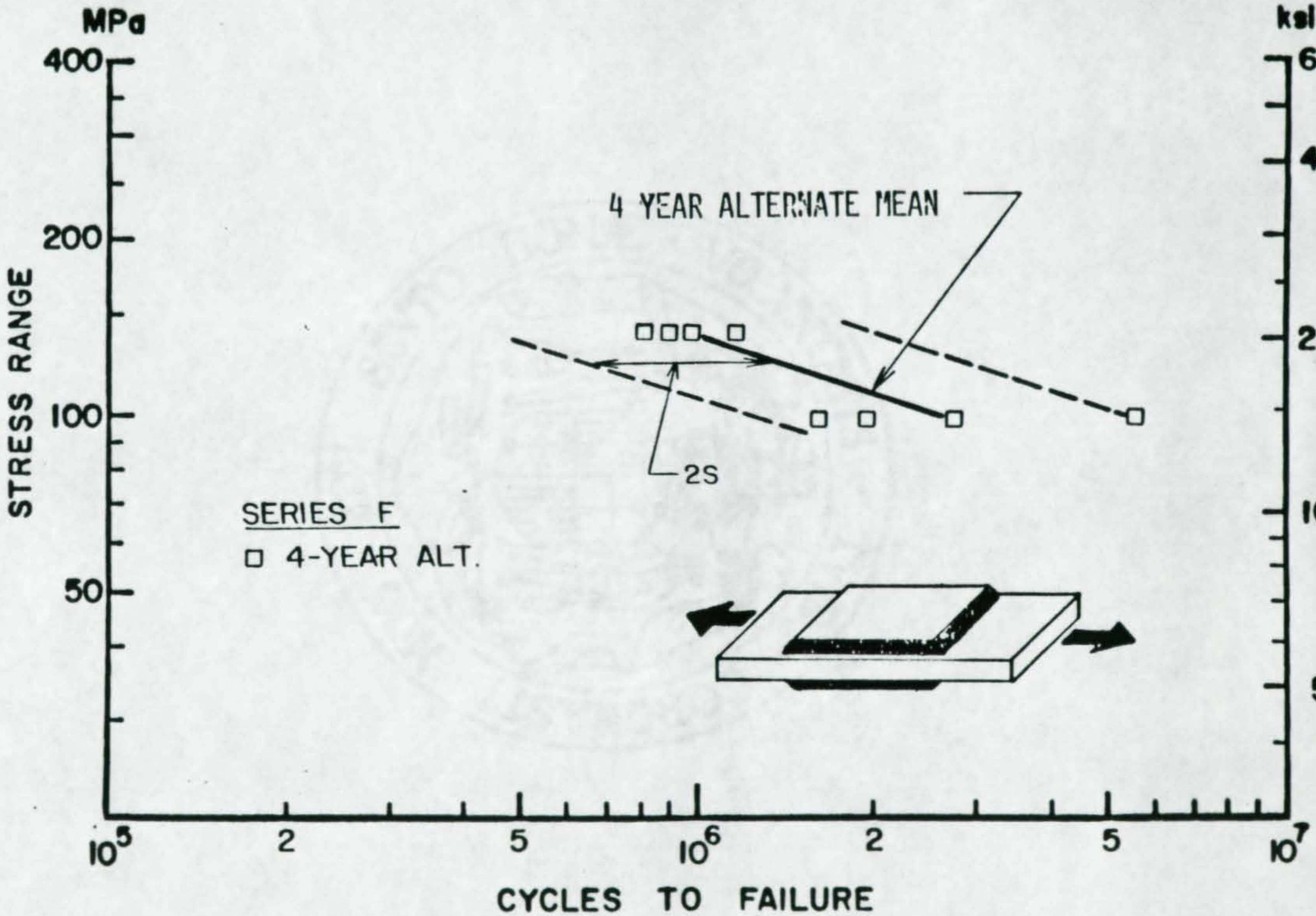
Plot 7-19 Specimens with Attachments after 2-Year Continuous Weathering, A588 Steel (Taken from Fig. 39 of Ref. 7)



Plot 7-20 Specimens with Attachments after 2-Year Alternate Weathering, A588 Steel
(Taken from Fig. 40 of Ref. 7)



Plot 7-21 Specimens with Attachments after 4-Year Continuous Weathering, A588 Steel (Taken from Fig. 41 of Ref. 7)



Plot 7-22 Specimens with Attachments after 4-Year Alternate Weathering, A588 Steel (Taken from Fig. 42 of Ref. 7)

APPENDIX G. FATIGUE

G1. Scope

Members and connections subjected to fatigue loading shall be proportioned in accordance with the provisions of this Appendix.

Fatigue is defined as the damage that may result in fracture after a sufficient number of fluctuations of stress. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the sum of maximum shearing stresses of opposite direction at a given point, resulting from differing arrangement of live load.

G2. Loading Conditions; Type and Location of Detail

In the design of members and connections subject to repeated variation of live load, consideration shall be given to the number of load cycles, the expected equivalent stress range, and type and location of member or detail.

The number of stress cycles shall be equal to the number of times the live load is applied during the service life of the structure.

The equivalent stress range for fatigue design shall be calculated from the expected load history with the equation

$$f_{re} = \left[\frac{\Sigma n_i \, f_{ri}^{3.2}}{\Sigma n_i} \right]^{1/3.2} \qquad \text{(G-1)}$$

where:

f_{re} = equivalent constant amplitude stress range

f_{ri} = stress range due to i-th loading

n_i = number of occurrence of i-th stress range

When reliable information on the load history is not available, the equivalent stress range shall be conservatively set equal to the maximum stress range produced by the most severe loading.

The type and location of material shall be categorized as in Table G-2.

G3. Nominal Fatigue Strength

The nominal fatigue strength of a member or connection shall be determined from

$$f_r = \frac{3060}{K_f\ N^{1/3.2}} \tag{G-2}$$

where:

f_r = nominal fatigue strength

K_f = fatigue notch factor for applicable stress category given in Table G-1

N = number of stress cycles

G4. Design Criteria

If the maximum stress range of the histogram is smaller or equal to the fatigue limit, F_{rl}, for the applicable category given in Table G-1, the detail need not be designed for fatigue.

If the maximum stress range is greater than the fatigue limit, the factored equivalent stress range shall be smaller or equal to the factored fatigue strength.

$$\Phi_f\ f_r \geq \gamma\ f_{re} \tag{G-3}$$

where:

Φ_f = 0.66, resistance factor for fatigue

γ = 1.6, live load factor

G5. Design Strength of Bolts

When subject to tensile fatigue loading, bolts shall be designed for the combined tensile design strength due to external and prying forces within the limits given in Table G-3.

TABLE G-1 Fatigue Notch Factors

Category	Fatigue Notch Factor K_f	Fatigue Limit F_{rl} (ksi)
A	1.00	24
B	1.45	16
C*	1.97	12
C	2.33	10
D	2.76	7
E	3.55	5
E' (or G)	4.18	2.5

TABLE G-3 Design Strength of Bolts

Number of Cycles	Design Strength
Not more than 20,000	As specified in Section 4.3.4.1
From 20,000 to 500,000	$0.30\ A_b\ F_u$ [a]
More than 500,000	$0.25\ A_b\ F_u$ [a]

[a] At service loads

Table G-2

Type and Location of Material

(Taken from AISC LRFD Draft-Ref. 9)

General Condition	Situation	Kind of Stress[a]	Stress Category. (See Table G-1)	Illustrative Example Nos. (See Fig. G-1)[b]
Plain material	Base metal with rolled or cleaned surfaces.	T or Rev.	A	1,2
Built-up members	Base metal and weld metal in members, without attachments, built up of plates or shapes connected by continuous full- or partial-penetration groove welds or continuous fillet welds parallel to the direction of applied stress.	T or Rev.	B	3,4,5,6
	Base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners.	T or Rev.	C	7
	Base metal at end of partial-length welded cover plates having square or tapered ends, with or without welds across the ends. Flange thickness $\leq$ 0.8 in. Flange thickness $\geq$ 0.8 in.	 T or Rev. T or Rev.	 E G	 5 5
Mechanically fastened connections	Base metal at gross section of high-strength-bolted friction-type connections, except connections subject to stress reversal and axially loaded joints which induce out-of-plane bending in connected material.	T or Rev.	B	8
	Base metal at net section of other mechanically fastened joints.	T or Rev.	D	8,9
	Base metal at net section of high-strength bolted bearing connections.	T or Rev.	B	8,9

a. "T" signifies range in tensile stress only; "Rev." signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear including shear stress reversal.

b. These examples are provided as guidelines and are not intended to exclude other reasonably similar situations.

Table G-2 (Continued)

General Condition	Situation	Kind of Stress[a]	Stress Category. (See Table G-1)	Illustrative Example Nos. (See Fig. G-1)[b]
Attachments (Cont'd)	Base metal at detail less than 1 inch thick attached by groove welds or fillet welds subject to longitudinal loading, with transition radius, if any, less than 2 inches :			
	2 in. < a ≤ 12b or 4 in.	T or Rev.	D	15
	a > 12b or 4 in.	T or Rev.	E	15,23,24, 25,26
	Base metal at detail greater than 1 inch and a > 4 inches where a - detail dimension parallel to the direction of stress b - detail dimension normal to the direction of stress and the surface of the base metal	T or Rev.	G	15,23,24, 25,26
	Base metal at a detail of any length attached by fillet welds or partial-penetration groove welds in the direction parallel to the stress, when the detail embodies a transition radius R, 2 inches or greater, with weld termination ground smooth :			
	R ≥ 24 in.	T or Rev.	B	19
	24 in. > R ≥ 6 in.	T or Rev.	C	19
	6 in. > R ≥ 2 in.	T or Rev.	D	19
	Base metal at a detail attached by groove welds or fillet welds, where the detail dimension parallel to the direction of stress a, is less than 2 in.	T or Rev.	C	23,24,25
	Base metal at a stud-type shear connector attached by fillet weld.	T or Rev.	C	22
	Shear stress on nominal area of stud-type shear connectors.	S	F	22

Table G-2 (Continued)

General Condition	Situation	Kind of Stress[a]	Stress Category (See Table G-1)	Illustrative Example Nos. (See Fig. G-1)[b]
Fillet welded connections	Base metal at intermittent fillet welds.	T or Rev.	E	
	Base metal at junction of axially loaded members with fillet welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses.	T or Rev.	E	17,18,20
	Weld metal of continuous or intermittent longitudinal or transverse fillet welds.	S	F	5,17,18,21
Groove welds	Base metal and weld metal at full-penetration groove welded splices of parts of similar cross section ground flush, with grinding in the direction of applied stress and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Table 9.25.3 of AWS D1.1-80.	T or Rev.	B	10
	Base metal and weld metal at full-penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2-1/2, with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of Table 9.25.3 of AWS D1.1-80.	T or Rev.	B	12,13

Table G-2 (Continued)

General Condition	Situation	Kind of Stress[a]	Stress Category (See Table G-1)	Illustrative Example Nos. (See Fig. G-1)[b]
Groove welds (Cont'd)	Base metal and weld metal at full-penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2 1/2, when reinforcement is not removed and/or weld soundness is not established by radiographic or ultrasonic inspection in accordance with the requirements of Table 9.25.3 of AWS D1.1-80.	T or Rev.	C	10,11,12,13
	Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds.	T or Rev.	F	16
Plug or Slot Welds	Base metal at plug or slot welds.	T or Rev.	E	27
	Shear on plug or slot welds.	S	F	27
Attachments	Base metal at detail of any length attached by groove welds subject to transverse and/or longitudinal loading, when the detail embodies a transition radius R, 2 inches or greater, with the weld termination ground smooth :			
	$R \geq 24$ in.	T or Rev.	B	14
	24 in. $> R \geq 6$ in.	T or Rev.	C	14
	6 in. $> R \geq 2$ in.	T or Rev.	D	14

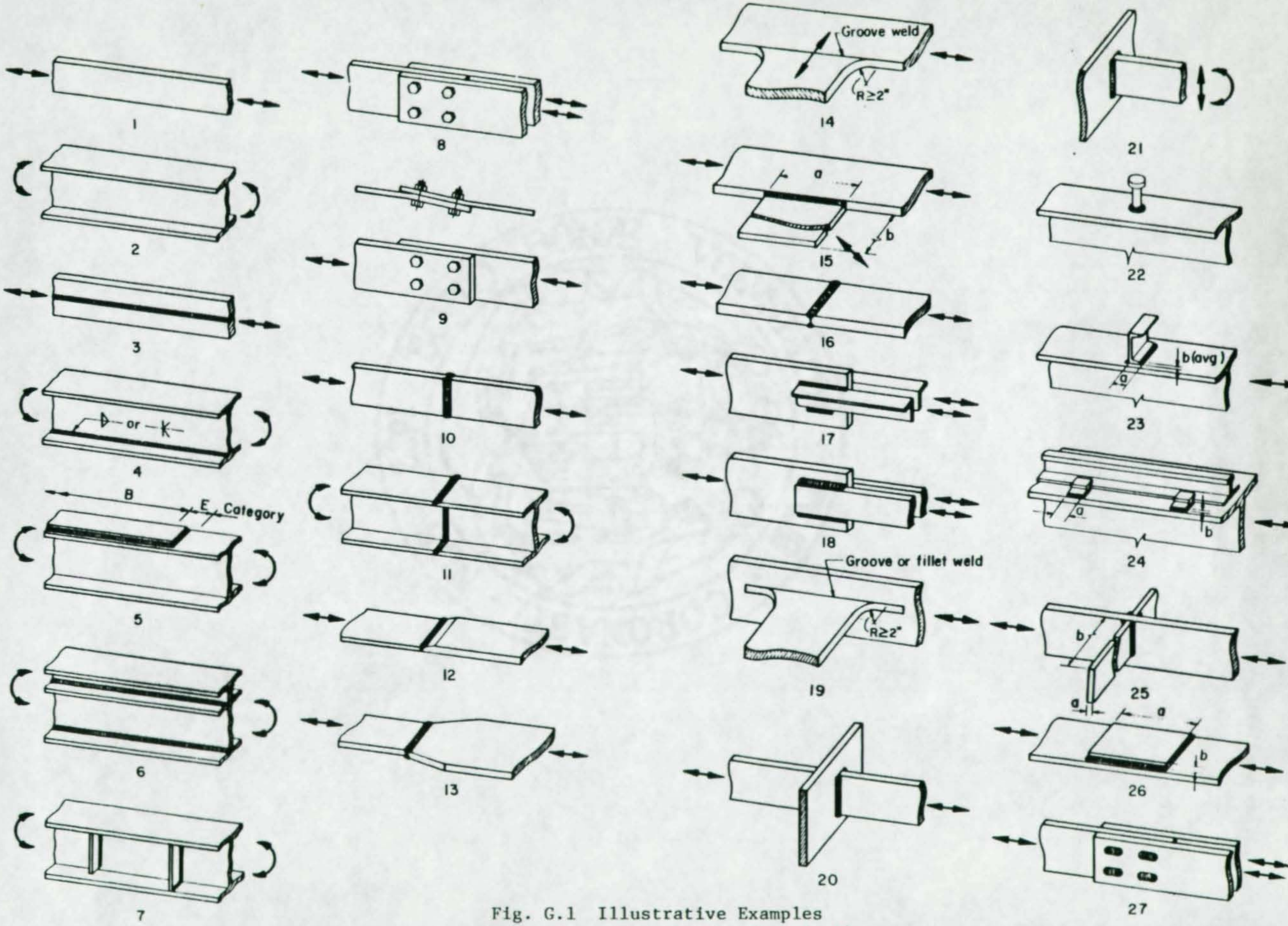


Fig. G.1 Illustrative Examples
(Taken from AISC LRFD Draft-Ref. 9)

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