FINAL REPORT

ON

SHEAR STUDS

FOR

SUSQUEHANNA STEAM ELECTRIC STATION

UNITS 1 AND 2

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(P-85a)

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1.0 PURPOSE

The purpose of this report is to provide final data and information as required by 10CFR50.55 (e) (3) subsequent to the notification of a reportable deficiency. The subject deficiency is associated with the installation and inspection of steel shear connectors in the reinforced concrete composite floors.

2.0 SHEAR CONNECTORS

Shear connectors, used on this project, are round, headed steel studs, commercially manufactured. After the erection of floor beams and the placement of the metal decking, studs are attached to the top flange of structural steel floor beams, by resistance welding using a semi-automatic process. The studs are then embedded in subsequently placed concrete and provide a shear connection between the concrete slabs and structural steel framing to develop a composite floor system.

Materials, installation, welding, inspection and testing of the studs is in accordance with Project Specification 8856-C-19, "Installation of Shear Connectors," and American Welding Society Code AWS D1.1-75. The specification requires a bend test to be performed on the first two studs welded to each structural steel member. After the completion of stud installation on any beam, the weld between the stud and

structural steel is required to be inspected visually and tested by selectively bending the studs to a minimum angle of 30 degrees from the vertical. Such bending does not affect the functioning of the stud as a shear anchor.

Composite construction has been used in the following structures:

Category I

1. Reactor Building Units 1 and 2
2. Control Building
3. Diesel Generator Building

Non-Category I

1. Turbine Building Units 1 and 2
2. Radwaste Building
3. Circulating Water Pumphouse

Inspection of studs in all Category I structures is the responsibility of Quality Control (QC) personnel and the Quality Control program provides the technical directions and means of documentation of inspection and testing activities. For Non-Category I structures, this function is performed by Field Engineering; however, documentation is not a requirement.

3.0 BACKGROUND

Subsequent to QC final pre-concrete inspection and acceptance on May 21, 1977 for concrete placement 183-S-02 (Area 33 at Elevation 719'-1" in the Reactor Building Unit 2) Pennsylvania Power & Light Company Quality Assurance (PLNQA) personnel found some studs, which did not meet specification requirements. It was also observed that the inspection requirements were not completely met. Two other areas were in progress at this time (Placement 714-S-03, Area 21, Elevation 771'-0" in the Control Building and 201-S-02, Area 28, Elevation 749'-1" in the Reactor Building Unit 1). QC performed another inspection of all studs for these placements. On completion of the required repair/rework, QC accepted these placement areas on May 26, 1977. Subsequently, on the same date, PLNQA again found a few more nonconforming studs for these placements.

A stop work report was issued on May 27, 1977 precluding any concrete placement in the above noted areas.

4.0 DESCRIPTION OF DEFICIENCIES

4.1 Construction personnel failed to repair, test or replace the defective studs as required by the specification.

4.2 QC personnel failed to inspect and carry out the assigned responsibilities as defined in the quality control instructions (QCI) for stud weld inspection. The following specifics are cited:

a. Responsible QC engineering personnel in the welding discipline signed inspection records

signifying that 100% inspection had been performed. However, the inspections as defined by the program were not completely performed.

b. Responsible QC supervision personnel at the jobsite failed to provide adequate, definitive directions to the responsible QC engineering personnel in the welding discipline and failed to detect the lack of acceptable performance of the QC engineering personnel.

5.0 IMMEDIATE CORRECTIVE ACTION

5.1 Placements Identified in MCAR-1.18

Nonconformance reports (NCR's) were issued against the studs found to be in noncompliance with specified requirements for concrete placements 183-S-02, 201-S-02 and 714-S-03. These NCR's were evaluated and disposition provided to either "rework" or "use as is" depending upon engineering evaluation. In addition, Quality Assurance issued a Management Corrective Action Report (MCAR-1.18) on May 26, 1977 and a Stop Work Report on May 27, 1977. These reports precluded further embedment of shear studs pending complete reinspection of studs in these placements to assure conformance to specification and design drawing requirements. A complete reinspection of the three concrete placement areas within the scope of the MCAR was carried out. The reinspection was accomplished in accordance with a specially prepared program, containing several provisions to maximize the effectiveness of the inspection and to virtually eliminate any inspection error. The special provisions included the following:

a. A detailed training program specifically addressing the unique aspects of the special inspection and the fundamental requirements for stud inspection was conducted. Special emphasis was placed on the recent problems related to the studs.

b. Each stud to be inspected was uniquely identified by number, providing traceability to the inspection record for the particular stud.

c. As-built drawings were made identifying the location of every stud by providing the direction sequence of the stud numbers.

d. A separate check list was completed and signed for each particular stud.

e. Each individual stud received a "general soundness test," consisting of striking the stud using a heavy hammer. Studs failing the soundness test were replaced with new studs.

f. Each inspection for each individual stud was documented, and the resulting inspection records were independently reviewed for completeness and acceptability.

g. NCR's were written identifying nonconforming conditions and were dispositioned providing alternates of repair and retest or replacement thereby allowing the field engineer participating in the reinspection to provide direction for immediate replacement or repair as necessary. Each occurrence was documented.

All required repair was accomplished with acceptable results. Results of the above inspection activities have been properly recorded and documented.

5.2 <u>Field Test Data</u>

5.2.1 During this period, stud installation in progress in other areas, was also stopped. These areas included:

a. Reactor Building:
Placement 202-S-01, area 27; 199-S-01, area 25; 202-S-02, area 29, all at Elevation 749'-1" in Unit 1.

Placement 182-S-01, area 32; 184-S-01, area 34 at Elevation 719'-1" in Unit 2.

b. Control Building

Placement 714-S-03, area 21

c. There were also some studs exposed in a construction opening in a previously poured slab in the Diesel Generator Building.

All studs in the above areas were thoroughly inspected by QC using the same inspection criteria as described in Section 5.1.

5.2.2 Field Engineering also performed a thorough inspection of all exposed studs installed prior to May 1977 in the Turbine Building and Circulating Water Pumphouse.

5.2.3 For the Radwaste Building, civil construction was completed prior to May 1977. Thus, no exposed studs were available for inspection.

5.3 Above inspection results of Section 5.2 identified as field test data in the following sections, are the basis for statistical evaluation.

It must be noted here that for the three areas noted in Section 5.1,

1. Some studs were installed after the bottom reinforcing steel was placed, thus making the stud installation difficult.

2. Some studs were welded directly through decking. Thus, the stud installation in these areas cannot be considered as representative. Additionally, the studs in these areas were subjected to many inspections, therefore, the inspection results cannot be used as a reliable sample data. Based on these considerations, this data was excluded in the statistical analysis.

6.0 ANALYSIS OF SAFETY IMPLICATIONS

The stud installation is grouped into various categories noted below to provide a base for analyzing the safety implications and performing technical evaluation.

6.1 Studs embedded in the concrete prior to May 1977.

6.1.1 As these studs, are embedded, they are not accessible to determine the quality of the stud installation.

Until the discovery of the problem, there had been no major change either in the inspection and testing criteria or in the method of stud installation. Thus the field test data, obtained as described in section 5.0, can be considered as truly representative of the past work. At certain locations, the data indicates abnormally high stud failure rates, which deserve special attention.

6.1.2 A statistical evaluation of the field test data has been performed for the purpose of establishing the failure rate and projecting at 90% confidence level the number of reliable studs that are considered effective in the existing, installed beams. The statistical projection of the number of reliable studs, together with the calculated minimum number of studs required for each beam, are the basis for verifying the adequacy of the composite structural system.

6.1.3 Based on the foregoing general criteria the following two categories are established:

6.1.3.1 For areas which exhibit acceptable stud failure rates, the test data on welded studs indicates that either one of the following conditions is met:

a) Stud failure rates fall within acceptable industry practice so as not to jeopardize the structural requirements.

b) The projected number of reliable studs exceeds the actual minimum required according to structural design calculation.

Consequently, in these areas the structural integrity has not been compromised, and the structural system is in full conformance with the basic design criteria and the bases of the Safety Analysis Report.

The Turbine Building, Unit 1 and 2, Control Building, Circulating Water Pumphouse, Radwaste Building and Diesel Generator Building belong to this category.

6.1.3.2. In areas associated with high failure rates, there are some beams for which the projected number of reliable studs is insufficient with respect to the minimum required by structural design. This condition has the following implications: The design requirements stated in the Safety Analysis Report are not met completely due to the potential stud

deficiency. Repair work must be undertaken to correct the defective installations and assure that there are no structural systems which do not meet the design bases.

The Reactor Building Unit 1 and 2 fall in this category.

6.2 Studs Not Embedded in Concrete at the Time of the Reported Problem.

In these areas, deficient studs are traceable to specific construction and/or inspection practices, which have been positively identified. The studs in these areas have been inspected under strict enforcement of the revised inspection procedures and repaired or replaced as required. New studs were also inspected to the full inspection requirements. This provides adequate assurance regarding the quality of the stud installation in these areas.

7.0 TECHNICAL EVALUATION OF DEFICIENCIES

7.1 General

Impact of the above noted deficiencies renders the structural adequacy of the studs installed indeterminate in the absence of technical evaluation. Remedial measures taken and to be taken to prevent the recurrence are described in section 5.0 and 8.0.

Therefore, the technical evaluation in this section is limited to the studs embedded in the concrete slabs prior to May 1977.

The approach used for this evaluation is as follows:

a. Evaluate the design criteria and theoretical considerations, assumptions, associated research and testing, which are the basis for the design requirements in the AISC specification.

   Based upon this evaluation, reassess and/or revise the original design and compute the number of studs required, which not only satisfy strength requirements but also meet the specification requirements.

b. Analyze the field test data statistically to arrive at a success rate at a certain confidence level for each building.

   Based upon this analysis compute the number of reliable studs on every beam.

c. Design shear connectors.

d. Identify those beams where the number of studs required is larger than the reliable studs.

7.2 Design Criteria and Structural Design of Composite Construction

7.2.1 General

A common approach in the design of structural floor systems is to develop composite action between the steel framing beams and the reinforced concrete slabs. The composite action affords a flexural system superior to the beam or slab action alone and generally results in cost savings in the overall design. Composite action is achieved by providing shear connectors welded to the top side of the beam and embedded in the concrete. These shear connectors can also be used to improve the anchorage of steel framing into concrete slabs to permit the transfer of horizontal loads from the framing to the slab diaphragm and to incorporate the slab in resisting heavy loads suspended from the beams.

7.2.2 Design Criteria and Theoretical Considerations

Section 1.11 of 'Specification for Design Fabrication and Erection of Steel for Buildings' (Sixth Edition) adopted by American Institute of Steel Construction in 1969 and subsequent three supplements are the bases for structural design. The new revision of the specification is due for publication in early 1978. Revised section

1.11 to be incorporated in the forthcoming edition is published in "Inryco Composite Beam Design Manual, 21-12" by Inryco Inc. in July 1977. This revision is essentially based upon the paper "Composite Beams with Formed Steel Deck," by Grant, Fisher and Slutter, in AISC Engineering Journal, Volume 14, First Quarter 1977.

From the review of the development of this section, it is evident that the design criteria is still in the developmental stage, and is being modified continuously to reflect the latest state of the art.

The majority of the research and testing done to date pertains to composite beams with thin slabs. In the associated theoretical considerations, the ultimate moment capacity of the concrete section is disregarded. Thus, the contribution of the internal couple produced by shear connection becomes very significant in computing the ultimate structural capacity and the factor of safety. For reinforced thick slabs, however, the ultimate moment capacity of the concrete section becomes so dominant that the significance of the shear connection is greatly reduced. Thus, the design based upon the specification results in a high reserve capacity for composite beams with thick slabs. The AISC specification, however, has not recognized this phenomenon.

The AISC Specification and its supplements define the allowable horizontal shear loads for studs and also prescribe analytical procedures for evaluating incomplete composite action by equation (1.11-1) as follows:

$$S_{eff} = S_s + \frac{V'_h}{V_h} (S_{tr} - S_s)$$

Where:

V_h = the lesser of the horizontal shear associated with either the concrete or the steel section

V'_h = the shear value permitted by the number of connectors provided, relevant for incomplete composite action

S_s = section modulus of the steel beam referred to its bottom flange

S_{tr} = section modulus of the transformed composite section (full) referred to its bottom flange

S_{eff} = effective section modulus of the incomplete composite section

The equation is based on early research, and it represents a linear variation of S_{eff} with respect to V'_h.

Recent research recognized by the AISC indicates that the functional relationship described above is more accurately expressed by introducing a square root expression for the shear ratio in equation (1.11-1). This modification represents a refinement on the analytical technique for the evaluation of incomplete composite action, and it results in a substantially higher capacity than that allowed by the previous, extremely conservative linear expression. This proposed expression offers a liberalized analysis reflecting the current thinking, but it prudently affords some conservatism with respect to the research findings.

The specification also prescribes a minimum of 25% of complete shear connection to be developed by the studs. This lower limit, however, is arbitrary and is not necessarily based upon the theory. In fact, test results described in the above referenced paper indicate that the test beams with wide slabs and less than 25% of complete shear connection performed satisfactorily with an adequate factor of safety. Thus, the test proves that the percentage shear connection is not necessarily a function of the capacity of the composite beam or its factor of safety.

Detailed discussion on this subject can be found in the above noted paper by Grant, Fisher and Slutter and also in Appendix "E".

As a summary it is concluded that:

1. The analytical approach per the present AISC specification, although reasonable for beams with thin slabs, is a very conservative method for the composite beams with thick slabs.

2. The design based upon the specification using revised 1.11-1 equation and assuming 25% complete shear connection will still provide adequate margin of safety and conservatism.

7.2.3 Structural Design

In the current structural design, the welded studs were provided in the majority of the beams to develop complete action, and the steel beam sections were designed according to the arbitrary overall floor loads prescribed for the various areas. However, in view of the potential problem with the welded studs, the structural design was reassessed with the intention of relieving the stud requirements without violating the basic design criteria.

The first step in the reassessment was to review the loading associated with each of the floor beams. This was achieved by considering actual load distributions obtained from the equipment and floor occupancies which at this date have been established more definitely than at the time of initial design. Another aspect of the load refinement consisted of a more detailed analysis of the tributary areas for each beam by recognizing actual load distributions derived from the one-way and two-way flexural action of the corresponding concrete slabs.

The second step in the reassessment was to refine the design by computing the effective section modulus according to the latest analytical criteria, i.e., the AISC approved expression

with the square root. This analytical refinement allowed for a revised higher capacity for sections in which the projected number of reliable studs did not permit complete composite action. The above analytical features were used prudently, and the minimum number of studs required per beam was judiciously selected by the criteria described in Section 7.4.

7.3 <u>Outline of Statistical Analysis and Evaluation</u>:

This section provides a brief description of the statistical approach used in the projection of the reliability of studs installed to date. A more detailed coverage of the statistical analysis used for this report is provided in Appendix A. Another statistical analysis using different method was performed independently, which gave essentially same basic results (Refer Appendix F).

The initial phase of the statistical analysis was to segregate the field test data into homogeneous groups judged to be statistically compatible. This judgement was based on Chi-square test on similarities of the stud failure rates and their distribution patterns. The first level of segregation established was according to the various buildings within the plant. Each structure was thus recognized as a separate group with its own characteristic sampling and corresponding statistical projections.

The second phase of the statistical evaluation consisted of determining the reliable studs for each of the established groups. These projections are based on the failure rates derived from field test data. Their development takes into account the number of studs tested with respect to the total number installed, and recognizes that the reliability of the studs must not be on an individual basis, but with due regard to stud groupings derived from the required number of studs per beam. The analytical bases of the statistical projections are derived from the required number of studs per beam and are based on the hyperbinominal distributions, without resorting to empirical idealizations. The fundamental assumption is that the field samples are unbiased and applicable to the balance of the corresponding stud group. This assumption is justified since the exposed areas where the sampling was obtained came into existence randomly, and due to reasons which are unrelated to the stud welding and QC inspection. The quality of the stud welding in these exposed areas were not influenced by and are independent of the location of these areas.

The confidence level of the statistical projection of reliable studs was set at 90%. This level of confidence is consistent with the critieria used by governing organizations involved in the preparation of codes of practice. Additionally, based upon engineering judgement, the probability of exceeding the design live load is extremely low.

7.4 Design of Shear Connectors

7.4.1 General

The shear connectors used in all instances were welded headed studs, and are designed to be installed by using a semi-automatic welding process.

7.4.2 Design Criteria

a. As discussed in Section 7.2.2, partial composite action (V'_h) was limited to 25%.

b. The latest expression (square root) was used for computing the effective section modulus under incomplete composite action and the corresponding stud requirement.

c. Present AISC code does not address the effect of grouping of studs in a rib. Latest research and proposed revision to the specification requires that if there are more than three studs in a rib, the cumulative allowable capacity must be computed by using the reduction factor (Equations 1.11-8 and 1.11-9). The stud requirement, which is more stringent based upon the new code, has been used.

7.4.3 Following the above design criteria, the number of studs dictated by the revised structural design calculations, based on reassessed loading analysis, were computed.

7.5 Conservative Features Not Resorted to in the Design

This is a commentary on some features that would increase the margin of safety of the design.

1. Based on engineering judgement, the allowable loads studs could be increased in proportion to the square root of the concrete compressive strength f'_c. In the current design, the allowable stud loads based on $f'_c = 4000$ psi, according to the AISC Specification have been used without taking credit for the actual f'_c which is close to 5000 psi.

2. In the basic design criteria, live loads are assumed to be acting over the entire floor area. However, under actual operating conditions, this is highly unlikely to occur. Thus, the reduction that may be achieved by considering actual live loads is not used in the revised design.

3. For computing N_2, (Equation 1.11-7), the underlying assumption is that the horizontal shear is resisted by only those studs within the shear span. In reality, because of the longitudinal bottom reinforcing steel, the horizontal shear will be transferred to adjoining studs, although this phenomenon is not recognized by AISC. Thus, the computed N_2 based upon present design will result in an even higher factor of safety.

7.6 Discussion on Radwaste Building

The Radwaste Building was completed prior to May 1977. As no studs were exposed at the time the problem was discovered, actual test data could not be obtained on the same basis as it was collected for other structures. For the slab at 715'-0" elevation, there is some record available on the visual inspection and testing activities performed by Field Engineering collectively on area basis instead of individual beam basis. Additionally, there are no soundness test results available for these areas. The record including bend test results on the studs failing visual examination is shown in the following Table.

TABLE I

Area No.	No. of beams	Total studs	Studs failing visual examination	Studs failing bend test
1	4	272	32	0
2	35	2,490	184	8
3	16	941	103	0
4	15	881	77	0
5	13	757	61	0
6	14	1,095	85	5
7	12	729	59	4
8	12	801	59	4
9	9	759	51	0

Interviews with the responsible Field Engineer and the welder provided following information.

1. Studs failing visual or bend test were not in a single cluster but were spread over the entire area without any definite pattern.

2. The welder who did the majority of the stud welding on this building, worked previously on the Circulating Water Pumphouse, and is presently working on the Diesel Generator Building from the very beginning. It is noted that the field test data for the above two building indicate 0% failure rate, which is a reflection on the workmanship of the welder.

3. As a matter of routine, it has been the policy of the welder to replace the stud, when it would give unsatisfactory sound of the shot.

4. Additionally, although not required by the specification, the welder has been bend testing the last two studs on every beam.

Based upon the engineering judgement and the evaluation of above record and information, the potential failure rate on the existing stud installation would be extremely low. In addition, present structural design is based upon complete composite action; therefore, the additional factor of safety is inherently built into the design. Thus, with adequate assurance, it is concluded that the present stud installation meets the design criteria.

7.7 Conclusions

7.7.1 The design of composite beams with thick slabs per present AISC specification is extremely conservative.

7.7.2 All existing beams when designed based upon the basic theory and computed number of reliable studs, have adequate margin of safety without performing any repair or modification. This design, however, does not satisfy the requirement of the specification for all beams.

7.7.3 In order to meet the specification requirements as noted in the Safety Analysis Report, those beams where the number of studs required per revised design is smaller than the number of computed reliable studs, will be repaired.

7.7.4 Using the above criteria, it is observed that a few beams in the Reactor Building require repair. These beams are identified, and the associated repair methods are described in Appendix D.

8.0 CORRECTIVE ACTION

Corrective action are grouped in three categories. Each category and corresponding actions are described below.

8.1 Category I

This category describes those studs already embedded in concrete prior to discovery of this problem in May 1977.

To evaluate the impact of the deficiencies on the adequacy of the structural members, field data was obtained, analyzed and evaluated. Based upon this evaluation, the number of projected reliable studs was computed for each beam and compared with the number of studs required based upon reassessment of the design criteria. Wherever the revised stud requirement is found to be greater than the projected reliable studs, these beams will be repaired, as described in Appendix 'D' "Repair Procedures". On completion of the required repair, the existing structural members, will satisfy the design requirements.

8.2 Category II

This category describes the studs in eight placements in Control and Reactor Buildings, when the problem was discovered (See Section 3.0 and 5.0).

Studs in these placements have been extensively inspected, examined and tested as described in Section 5.0, thus providing adequate assurance that these studs will perform satisfactorily under design loads. Therefore, no further corrective action is deemed necessary.

8.3 Category III

This category belongs to present stud installation since the discovery of the problem. Since completion of above noted eight placements the following specific corrective actions have been instituted at the site.

8.3.1 Corrective Actions by Quality Control.

a. The QC welding discipline has been relieved of the responsibility for inspection of the studs, except those installed during prefabrication of embeds. The QC civil discipline has been directed to assume this responsibility. This action results in the following upgrading of the inspection program:

   i. The inspection of studs is now more closely integrated with other related preplacement inspections, such as embeds, reinforcing steel, conduit, etc.

   ii. Addition of the 'General Soundness Test'

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iii. The amount of QC engineering manpower which may be focused upon stud inspection is now increased.

iv. Inspection may now more often be carried out while stud installation is being performed, and while craft personnel are present to perform immediate rework or repair if necessary.

v. Stud inspection may now normally be completed before the studs are visually obscured by other installed items, such as curtains of reinforcing steel.

b. The inspection plan for stud inspection has been reviewed and strengthened in the following specific areas:

i. Marking to physically identify both acceptable and unacceptable studs has been clearly defined in the inspection plan.

ii. Verification of proper stud welding cable length (i.e., less than 100 feet) has been added.

8.3.2 <u>Corrective Actions by Field Engineering</u>.

a. A special training session on stud installation dated June 10, 1977 was conducted

at the jobsite for QC, Engineering and Supervision to guarantee improved quality of installation.

b. In future placements, installation of reinforcing steel or other materials which would interfere with installation or inspection of shear studs will be withheld until the shear stud installation in the area is completed.

c. A training session was held on June 26, 1977 for all ironworkers involved with stud installation. Emphasis was placed on the craftsman's primary responsibility for correct installation of shear studs. The complete installation sequence of studs was also reviewed in depth.

d. A vendor representative for the welding equipment was brought on site June 22, 1977. During this visit equipment settings, maintenance and trouble shooting were reviewed with the ironworkers and superintendents.

e. Equipment maintenance program has been revised and re-organized including a larger inventory of spare parts being maintained on site.

f. All rectifiers in the field are returned to the manufacturer on a rotational basis to ensure they are performing correctly.

9.0 CONCLUSION

9.1 In most of the areas, the projected number reliable studs are not only sufficient to perform structural function but also meet the specification.

9.2 Although all projected reliable studs are adequate to satisfy the structural requirement, there are some beams at a few elevations in the Reactor Building which do not conform to specification requirements in its entirety. Thus, these deficiencies will be corrected by repairs performed on the existing installation.

9.3 On completion of the required repair, the structural analysis and design will satisfy strength and code requirements and will also assure that the existing installation will conform to the design criteria and bases of Safety Analysis Report.

APPENDIX A

TO

FINAL REPORT ON SHEAR STUDS

STATISTICAL ANALYSIS

AND

EVALUATION OF FIELD TEST DATA

(P-74b)

STATISTICAL ANALYSIS AND EVALUATION OF FIELD TEST DATA

1.0 OBJECTIVE

To analyze the test data in each beam completed prior to May 1977 and to determine the statistical basis for estimating the total number of good studs that can be relied upon.

2.0 FIELD TEST DATA

2.1 General

In the fourth week of May 1977, when the problem was discovered, there were many areas where the stud installation was completed and also the studs were accessible. These studs were subjected to a thorough inspection and testing as shown below in the flow chart. In addition to visual examination and selective bend testing as per the specification requirement every stud received 'general soundness test'. Complete field test data and the reduced field test data used for statistical analysis is provided in Appendix B and C respectively.

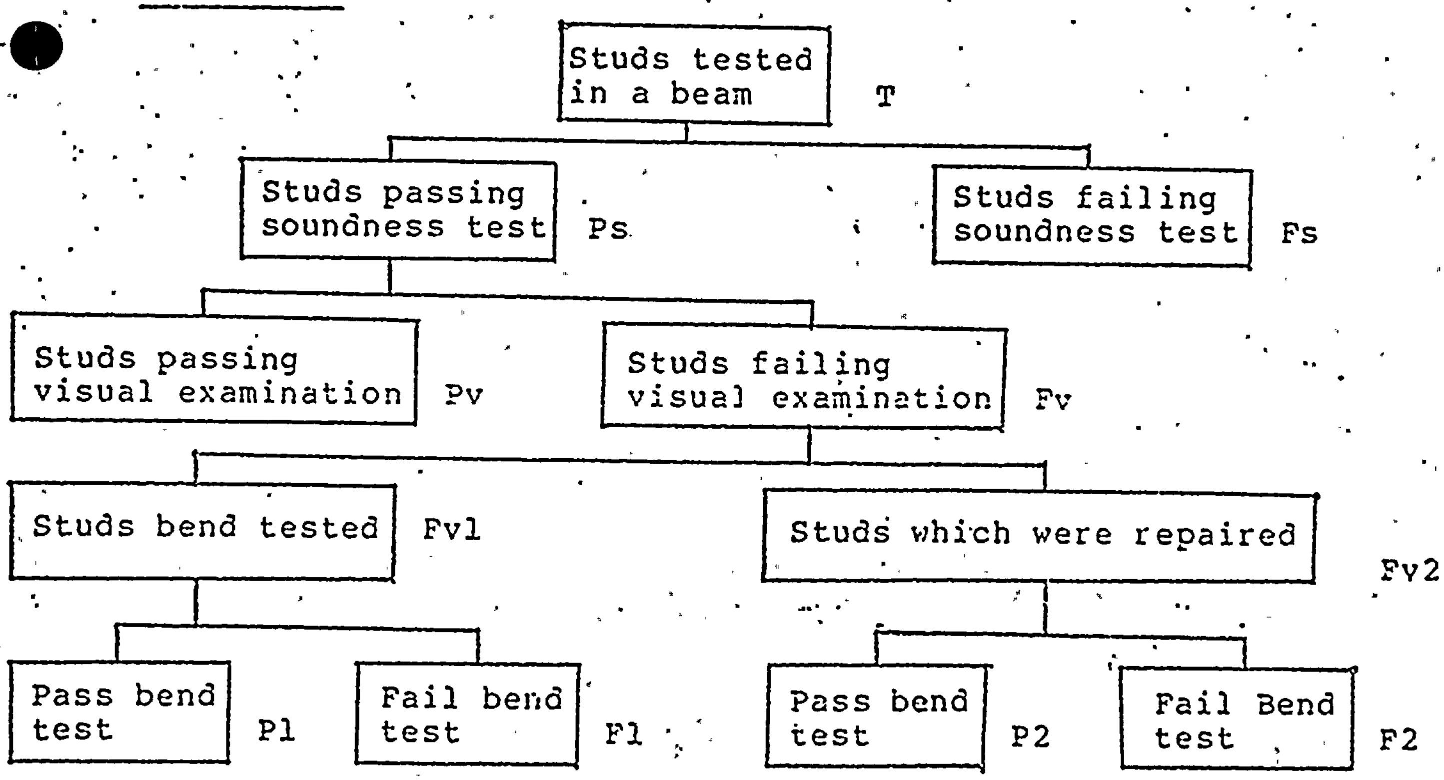
2.2 DEFINITIONS:

1. Soundness Test: On completion of stud welding, the stud is struck with a heavy hammer. If it gives a clean ringing sound, the stud is considered acceptable. Otherwise it is replaced with a new stud.

2. Visual Examination: After completion of the soundness test, each stud is examined visually to insure that there is fillet weld all around the periphery of the stud. If there are no voids, the stud is considered passing the visual examination.

3. Bend Test: Studs failing visual examination are bent 15° away from the void in the weld with respect to the vertical axis. If the stud does not develop cracks at the root or separates from the beams, it is considered acceptable. This is the most severe and reliable test.

2.3 FLOW CHART



Note: P2 and F2 are assumed numbers. See section 2.6.3.3 for clarification.

2.4 <u>Notations:</u>

X^2	=	Chi-square
N	=	Number of beams tested in each building.
T	=	Total studs tested in a beam.
Ps	=	Studs passing soundness test.
Fs	=	Studs failing soundness test.
Pv	=	Studs passing visual examination.
Fv	=	Studs failing visual examination.
Fv1	=	Studs failing visual examination, which were bend tested.
Fv2	=	Studs failing visual examination, which were repaired prior to bend test.
P1	=	Studs (Fv1) passing bend test.
F1	=	Studs (Fv1) failing bend test.
P2	=	Studs (Fv2) passing bend test (assumed).
F2	=	Studs (Fv2) failing bend test (assumed).
P	=	Good studs
	=	Pv + P1 + P2
F	=	Bad studs
	=	Fs + F1 + F2

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2.5 Summary of Field Test Data

Table 1

Structure	Number of beams	Total studs tested/examined
Reactor Building	111	11309
Control Building	11	1764
Turbine Building	17	831
Circulating Water Pumphouse	2	107
Diesel Generator Building	1	44

2.6 Discussion on Field Test Data

2.6.1 Studs failing soundness test (Fs)

The soundness test indicates the quality of the weld between a stud and structural steel but it may not be foolproof. That is, it is very likely that some of the studs failing this test may be good from a structural strength point of view. Since the exact reliability of the soundness test is not known, all studs failing the soundness test are considered to be bad studs, to insure conservative estimates.

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2.6.2 Studs passing visual examination. (Pv)

Stud manufacturers have indicated that irrespective of the method of testing, the overall failure rate is observed to be about 2% under normal working conditions. Based upon this fact, in a given population of studs (T), if the studs failing visual and soundness test (Fs + Fv) are removed, the success rate for the remaining sample (Pv) can reasonably be considered to be 100%. A recent bend test conducted on randomly picked population of 543 studs, which had passed both visual and soundness test gave 100% success rate. Thus, these results also reinforce the validity of the above assumption.

2.6.3 Studs failing visual examination (Fv)

For this category, the specification provides an option to the field either to perform a bend test or to repair. Field test indicates that all studs were not necessarily subjected to bend test. The test was performed on (Case 1) all, (Case 2) one, (Case 3) some or (Case 4) none of ths studs on a beam. Reasons for either including or excluding the studs to be subjected to bend test was based upon any one

of the following: construction schedule, accessibility, inadequate room for replacement in case of a failure and arbitrary decision by the field. Thus, for case 2, 3 and 4 to include the studs repaired (FV2) for statistical analysis, following criteria has been used.

2.6.3.1 Case 1: $Fv = FV1$

$FV2 = 0$

As the bend test is performed on all studs failing visual (Fv), the test data is used 'as is'.

2.6.3.2 Case 2: $Fv1 = 1$

$Fv2 = Fv - 1$

In this case, only one stud was subjected to bend test, thus its results can not be applied in a meaningful way to other studs. Therefore, beam samples containing this combination are omitted from the total sample.

2.6.3.3 Case 3 : $Fv1 > 1$

$Fv2 = FV - FV1$

For the reasons stated above, selection of the studs to be bend tested was arbitrary therefore the failure rate as observed for FV1 can reasonably be assumed to be same for FV2.

2.6.3.4 Case 4: Fv1 = 0

Fv = Fv2

As no bend test data is available for Fv1, beam samples containing this combination were excluded from the total sample.

2.7 Based upon the above criteria, failure rate for each beam is calculated as noted below.

$$\text{Failure rate} = \frac{Fs + F1 + F2}{\text{Total studs (T)}}$$

where Good studs = Pv + P1 + P2

and Bad studs = Fs + F1 + F2

3.0 <u>ANALYSIS OF FIELD TEST DATA</u>

3.1 Although the Field test data is available for five buildings, the data for only three buildings with higher failure rates is considered here for statistical analysis. The reason for this is, the failure rate for Circulating Water Pumphouse and Diesel Generator Building is 0%.

For the Reactor, Control and Turbine buildings, in a total sample of 72 beams, 7967 studs were tested. Following the criteria described in sections 2.6.3 and

2.7, 7427 passed and 540 failed for an overall success rate of 93.22%. It would be attractive to treat this data as a single aggregate sample since that would yield the greatest precision of the estimate of the success rate parameter p. However, different failure rates have been observed in different buildings so that failure parameters may differ from building to building. Statistical tests were used to determine whether this in fact did occur.

3.2 Construction of various buildings is done on the area concept, i.e. a separate group of Field Engineers, Superintendents and workers are assigned to and responsible for the construction of that particular building. Thus, even though the governing specification is the same for all buildings, workmanship and quality may vary within reasonable limits from building to building.

Test results for the above three buildings are summarized as below.

Table 2

Building	Studs passed	Studs failed	%Failure rate
Reactor	4970	402	7.48
Control	1633	131	7.42
Turbine	824	7	0.84
Total	7427	540	6.78

From the above table there is a noticeable amount of variation in the failure rate. The primary question is if these are variations to be observed in any random process (e.g., 10 tosses of the same fair coin may yield 7 heads in one sequence and 4 in the other). It must be emphasized here that all known parameters affecting the failure rate are the same for the entire stud welding operation in any building. If the different rates can be shown to lie within the realm of probabilistic 'noise,' then all individual tests may be pooled together into an aggregate sample and 6.78% as the failure rate. However, if this can not be shown, then the data must be regarded as separate subsamples and an allowance made for the lower precision which results. The subsequent section on the hyperbinomial distribution describes how the final recommendations incorporate this loss in precision to assure a rigorous and conservative analysis.

The key analytic question is whether or not the underlying pass/fail probability is the same for above three buildings. The principal statistical tool to be used is the X^2 test of homogeneity.

If the studs in all three buildings had a common failure rate of 6.78%, (i.e. if homogeneity is null hypothesis), the expected number of "passes" in the Reactor Building would have been 5008 with 1644 and 775 expected in the Control and Turbine Buildings respectively. Similarly, the expected number of failures would have been 364,120 and 56.

The X^2 test statistic is based upon the differences between all 6 observed and expected values.

$$X^2 \text{ test} = \frac{(4970-5008)^2}{5008} + \frac{(1633-1644)^2}{1644} + \frac{(824-775)^2}{775}$$

$$+ \frac{(402-364)^2}{364} + \frac{(131-120)^2}{120} + \frac{(7-56)^2}{56}$$

$$= 51.31*$$

This test statistic is approximately distributed as an X^2 random variable with 2 degrees of freedom [1] for which there is only 0.5% chance of exceeding 10.6. Since the test statistic is so much greater than this value, the conclusion is that the sample under consideration is non-homogeneous. Thus, each building must be considered as an individual subsample.

3.3 Even after the need to analyze the data building by building is established, the major concern is the adequacy of collection of studs on each individual beam or girder, for determining effectiveness of composite action. Therefore, it is necessary to consider the field data for each beam as an individual sample.

*This value differs from the exact X^2 value. The apparent difference is due to rounding off the expected values to integers for narrative purpose. The exact values were used in reaching all data clustering decisions.

[1] A. M. Mood and F. A. Graybill, Introduction to Theory of Statistics. McGraw Hill (1963) p. 318.

3.4 Based upon above discussion and criteria, the beam data for each building is analyzed.

3.4.1 Reactor Building Units 1 and 2

Although the following discussion pertains to the Reactor Building, it is also applicable to other buildings except as noted otherwise.

For a sample of 44 beams, the data can be grouped as follows:

Number of beams	Failure rate
5	20 to 38%
4	15 to 20%
6	10 to 15%
9	5 to 10%
20	0 to 5%

It is evident from the above grouping, that for the majority of the beams, the failure rate ranges from 0 to 10%. When the X^2 test was performed on the sample of 44 beams, the sample was found to be non-homogeneous. Notwithstanding that the method of stud installation, the governing specification, workmanship, construction sequence, and all other known variables were same, the wide variation in the failure rate can not be explained. Despite testing the sample with various permutations and combinations, no reason was found which could be attributed for this occurrence.

In light of this situation, it was decided to test the truncated sample i.e. disregarding the beam samples starting with the lowest failure rates, for establishing homogeneity. After several iterations, a sample of 6 beams with failure rate ranging from 19.05% to 38.36% was found to be homogeneous. This truncated sample with 390 'passes' and 146 'failures' gave overall failure rate of 27.2%. With the above discussion, it must be emphasized here that using this higher failure rate is indeed an extremely conservative assumption, and can be applied, with a high confidence level, in projecting 'good' studs in the areas where the studs have already been embedded in the concrete.

3.4.2 Control Building

The data is available for 11 beams with 1764 studs tested. The failure rate for the beams ranged from 3.53 to 25.93%. It was also observed that only one beam has unusually high failure rate. When the total sample was tested for homogeneity, the sample was found to be non-homogeneous. However, the sample excluding the beam with the highest failure rate was found to be homogeneous. In light of this fact, it can be concluded that the data for this particular beam with the highest failure rate is a stray sample. However, for computing the overall failure rate, this beam is included.

3.4.3 Turbine Building

Available data is for 17 beams with 831 studs tested. Out of this total, 824 passed and 7 failed giving average failure rate of 0.84%. It is observed that 15 beams out of 17 beams, have 0% failure rate. The sample consisting of remaining two beams was found to be homogeneous. Thus the failure rate of 4.14% for these two beams has been used for all the beams in Turbine Building which again is a conservative approach.

3.4.4 Circulating Water Pumphouse

At the time, when the problem was discovered, only two beams with a total of 107 studs were exposed. Out of this total, only one stud failed visual examination but the stud passed the subsequent bend test. Thus, the observed failure rate is 0%.

3.4.5 Diesel Generator Building

Forty-four studs on a beam in a construction opening were exposed. All the studs were tested with no failure, thus giving a failure rate of 0%.

3.5 Summary

Building	Studs Passed	Studs failed	Failure rate
Reactor	390	146	27.2%
Control	1642	121	6.85%
Turbine	162	7	4.14%

Above information was used as inputs into the hyperbinomial distribution to establish probabilistic characteristics of beams and girders for each building as described in the subsequent section.

4.0 HYPERBINOMIAL DISTRIBUTION

The results of the above analysis establishes the appropriate homogeneous groupings of test data for quality characteristics of individual studs.

This analysis proceeds by recalling the hyperbinomial distribution.[2] The motivation is as follows. First, if the success parameter, p, were known precisely then the total number of good studs (k) in a collection of h would vary according to a binomial distribution:

$$P[k \text{ of } h|p] = \binom{h}{k} p^k (1-p)^{h-k}$$

For example, if p = 6 and h = 5, then the numerical values of the resulting mass function would be:

---

[2] H. Raiffa and R. Schlaifer, Applied Statistical Decision Theory Harvard University Press (1961). p. 237

No. Good Studs = k	p k of 5; p = .6
0	.010
1	.077
2	.230
3	.346
4	.259
5	.078
	1.000

However, if p is not known but must be estimated, then such a binomial distribution assumes more precision than actually exists and makes things appear better than they are. For example, if n studs have been tested and only r passed, then the parameter p itself has a probability distribution,

$$f(p) = \frac{(n+1)!}{r!\ (n-r)!} p^r (1-p)^{n-r} \qquad \text{for } 0 \leq p \leq 1$$

the familiar beta distribution (3). Thus, while the expected value of p is r/n, other values of p between 0 and 1 may also have generated the sample, and these cannot be ignored in any subsequent inferences.

To obtain the probability of k good studs in a beam of h when r of n similar studs have passed the strike test, the unconditional distribution mav be found by:

$$P\ [k \text{ of } h;\ r \text{ of } n] = \int_0^1 P\ [k \text{ of } h|p] \cdot f\ (p;\ r,\ n)\ dp$$

$$= \int_0^1 \frac{h!}{k!(h-k)!} p^k (1-p)^{h-k} \cdot \frac{(n+1)!}{r!(n-r)!} p^r (1-p)^{n-r}\ dp$$

(3) A. M. Mood and F. A. Graybill, Introduction to the Theory of Statistics, McGraw-Hill (1963) p. 129 ff.

Collecting constants:

$$= \frac{h!\ (n+1)!}{k!\ (h-k)!\ r!\ (n-r)!} \int_0^1 p^{k+r}\ (1-p)^{n+h-r-k}\ dp$$

performing the integration,

$$= \frac{h!\ (n+1)!}{k!\ (h-k)!\ r!\ (n-r)!} \cdot \frac{(k+r)!\ (n+h-r-k)!}{(n+h+1)!}$$

and rearranging terms in combinational notation yields the hyperbinomial distribution:

$$P\ [k \text{ of } h;\ r \text{ of } n] = \frac{\binom{r+k}{r}\binom{n+h-r-k}{h-k}}{\binom{n+h+1}{h}} \quad \text{for } k = 0, \ldots, h \text{ and } r \leq n$$

To gain a sense of the effect of this distribution, suppose that 15 studs have been tested and 9 have passed. The estimated value of p is 9/15 (i.e., still .6) as before. However, repeated evaluations of the above expression yields the following distribution:

No. Good Studs (k)	p k; 9 of 15
0	.023
1	.103
2	.227
3	.303
4	.246
5	.098
	1.000

Note that this distribution is more diffuse than the simple binomial; i.e. the tails of the distribution are "fatter" and less probability mass is concentrated around the central value. The import of this is that when inferences are made about the adequacy (or inadequacy) of studs on beams or girders, a more stringent, conservative set of standards are applied than would result from the simple (and inappropriate)

binomial distribution.

The values of n and r are on the order of 20 studs to several hundred in some instances. Thus, the evaluation of all the appropriate mass and cumulative distributions is a laborious and computationally demanding task. Accordingly, a computer program was developed to assist in these studies. The program listing accompanies this appendix. The program contains comments to make it self-documenting.

Statements 20, 30, and 40 are used to set the parameters of the distribution. The two key ideas are:

i) all probabilities are carried in logarithmic form until the final printout to guard against round-off error and assure the requisite level of accuracy.

ii) each value of the mass function is related to the previous one, so that once p(0 of h; r of n) is found, the other values may be calculated recursively. This reduces the number of factorial evaluations and aids the computational efficiency of the total program.

Execution of the computer program yields the density and the probability functions derived from a given set of field test data for a given total of studs grouped according to the number of studs per beam. Next this output is reduced to obtain the probability of exceeding the prescribed design criteria as a function of the number of reliable studs which exist or which are to be provided in a given beam. From this information, the projected number of reliable studs for a given beam is derived observing the stipulated 90% confidence level.

Acknowledgement:

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PROGRAM LISTING FOR THE
HYPERBINOMIAL PROBABILITY DISTRIBUTION

STUDS

```
10   DIM P[300]
20   H=5
30   R=9
40   N=15
45   REM......................................
50   REM       FIND P(0) FOR THE STARTING POINT
60   REM       SET THE NUMERATOR FACTORS
70   N[1]=N+H-R
90   N[2]=N+1
110  REM          SET THE DENOMINATOR FACTORS
140  D[1]=N-R
150  D[2]=N+H+1
160  N1=D1=0
170  FOR J=1 TO 2
180  F=N[J]
190  GOSUB 500
200  N1=N1+F1
210  NEXT J
220  FOR J=1 TO 2
230  F=D[J]
240  GOSUB 500
250  D1=D1+F1
260  NEXT J
270  P[1]=N1-D1
280  GOTO 600
500  REM...................................................
510  REM          SUBROUTINE TO GET F1=LOG(F!)
520  F1=0
530  IF F>1 THEN 550
540  RETURN
550  FOR Z=2 TO F
560  F1=F1+LOG(Z)
570  NEXT Z
590  RETURN
595  REM...................................................
600  REM          COMPUTE P(1),P(2),....,ETC.
610  FOR K=2 TO H+1
615  X=K-1
620  P[K]=P[K-1]+LOG(R+X)-LOG(N+H-R-X+1)
625  P[K]=P[K]-LOG(X)+LOG(H-X+1)
630  NEXT K
640  REM          CHANGE LOGS TO PROBABILITIES
650  FOR K=1 TO H+1
660  P[K]=EXP(P[K])
670  NEXT K
680  REM          PRINT THE RESULTS
690  C=0
700  FOR K=1 TO K+1
710  C=C+P[K]
720  PRINT K-1,P[K];C
730  NEXT K
9000  END
```

APPENDIX B

TO

FINAL REPORT ON SHEAR STUDS

FIELD TEST DATA

(P-74b)

Notes:

1. Inspection results noted as Field Test Data on the following pages, pertain to the exposed studs installed prior to May 1977

2. For the explanation of the terms and expressions used, refer to Appendix "A".

FIELD TEST DATA FOR REACTOR BLDG. #1

Placement: 202-S-01 Area: 29 Elev. 749'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
16	2	88	0	21	2	0	Case 1
17	3	86	0	27	8	1	Case 3
18	4	88	4	16	3	0	Case 1
19	5	86	0	34	7	0	Case 1
20	6	88	0	8	3	15	Case 3
21	7	86	1	1	0	13	Case 2
22	8	88	2	0	0	47	Case 4
23	9	86	0	0	0	11	Case 4
24	10	86	0	0	0	35	Case 4
25	11	83	1	0	0	30	Case 4
26	12	80	2	0	0	32	Case 4
27	13	213	0	37	1	2	Case 3
28	14	90	2	18	3	2	Case 3
29	15	132	0	8	1	10	Case 3

(P-86a)

FIELD TEST DATA FOR REACTOR BLDG. #1

Placement: 199-S-01 Area: 25 Elev. 749'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
1	1	450	2	0	0	188	Case 4
2	2	39	0	0	0	15	Case 4
3	3	51	0	0	0	21	Case 4
4	4	26	2	0	0	10	Case 4
5	5	50	3	0	0	16	Case 4
6	6	30	0	0	0	22	Case 4
7	7	48	1	0	0	31	Case 4
8	17	216	4	0	0	105	Case 4
9	18	76	8	0	0	12	Case 4
10	19	76	5	0	0	16	Case 4
11	20	76	3	0	0	15	Case 4
12	21	76	0	0	0	27	Case 4
13	22	76	6	0	0	0	Case 1
14	30	123	7	0	0	8	Case 4

(P-86a)

FIELD TEST DATA FOR REACTOR BLDG. #1

Placement: 199-S-01 Area: 25 Elev. 749'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results		Studs Failing Visual Exam. But Repaired	Remarks
				Total	Failing Bend Test		
		T	FS	FV1	F1	FV2	
15	31	165	2	0	0	29	Case 4

(P-86a)

FIELD TEST DATA FOR REACTOR BLDG. #1

Placement: 202-S-01 Area: 29 Elev. 749'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results — Total	Studs Failing Visual Exam. With Bend Test Results — Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
30	16	62	0	16	3	0	Case 1
31	17	32	0	0	0	20	Case 4
32	18	711	4	52	1	102	Case 3
33	19	177	4	62	9	0	Case 1
34	20	149	0	19	0	0	Case 1
35	21	86	1	14	3	0	Case 1
36	22	84	0	0	0	23	Case 4
37	23	96	1	16	5	0	Case 1
38	24	106	8	0	0	35	Case 4
39	25	27	0	0	0	22	Case 4
40	26	34	0	1	1	9	Case 2
41	27	25	1	0	0	17	Case 4
42	28	101	1	41	0	4	Case 3
43	29	105	0	0	0	18	Case 4

(P-86a)

FIELD TEST DATA FOR REACTOR BLDG. #1

Placement: 202-S-02 Area: 29 Elev. 749'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results		Studs Failing Visual Exam. But Repaired	Remarks
				Total	Failing Bend Test		
		T	FS	FV1	F1	FV2	
44	30	96	1	0	0	39	Case 4
45	31	88	0	7	0	0	Case 1
46	32	130	15	0	0	14	Case 4
47	33	130	3	24	4	24	Case 3

FIELD TEST DATA FOR REACTOR BLDG. #1

Placement: 202-S-01 Area: 27 Elev. 749'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
48	1	114	0	0	0	45	Case 4
49	2	13	0	0	0	8	Case 4
50	3	34	0	13	3	1	Case 3
51	4	10	0	0	0	0	Case 1
52	5	76	0	0	0	66	Case 4
53	6	157	0	16	5	2	Case 3
54	7	274	51	67	15	20	Case 3
55	8	57	0	18	6	1	Case 3
56	9	57	0	18	8	1	Case 3
57	10	44	2	30	9	0	Case 1
58	11	45	4	18	4	0	Case 1
59	12	48	2	14	0	6	Case 3
60	13	42	0	0	0	1	Case 4
61	14	21	1	6	3	0	Case 1

FIELD TEST DATA FOR REACTOR BLDG. #1

Placement: 202-S-01 Area: 27 Elev. 749'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
62	17	223	4	94	19	0	Case 1
63	19	38	1	22	12	0	Case 1

(P-86a)

FIELD TEST DATA FOR REACTOR BLDG. #2

Placement: 182-S-01 Area: 32 Elev. 719'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results		Studs Failing Visual Exam. But Repaired	Remarks
				Total	Failing Bend Test		
		T	FS	FV1	F1	FV2	
64	1	66	3	0	0	21	Case 4
65	2	70	1	1	0	23	Case 2
66	3	62	2	0	0	29	Case 4
67	4	62	0	0	0	36	Case 4
68	5	62	1	0	0	18	Case 4
69	6	122	2	0	0	7	Case 4
70	7	44	2	0	0	3	Case 4
71	8	41	3	0	0	16	Case 4
72	9	87	0	0	0	21	Case 4
73	10	50	0	0	0	19	Case 4
74	11	32	0	0	0	12	Case 4
75	12	241	1	1	1	31	Case 2
76	13	204	5	11	1	10	Case 3
77	14	198	3	0	0	53	Case 4

FIELD TEST DATA FOR REACTOR BLDG. #2

Placement: 182-S-01 Area: 32 Elev. 719'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
78	15	307	2	0	0	0	Case 1
79	20	36	1	0	0	19	Case 4
80	21	57	1	0	0	8	Case 4
81	22	68	5	0	0	9	Case 4
82	23	76	0	0	0	22	Case 4
83	29	38	0	0	0	15	Case 4

FIELD TEST DATA FOR REACTOR BLDG. #2

Placement: 184-S-01 Area: 34 Elev. 719'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
84	1	68	2	16	0	16	Case 3
85	2	68	1	1	0	19	Case 2
86	3	68	2	8	0	25	Case 3
87	4	68	0	2	0	31	Case 3
88	5	76	3	1	1	2	Case 2
89	6	76	0	0	0	20	Case 4
90	7	68	3	0	0	17	Case 4
91	8	72	1	1	0	23	Case 2
92	9	65	0	0	0	23	Case 4
93	11	266	11	4	0	113	Case 3
94	12	125	3	0	0	32	Case 4
95	13	166	3	42	8	0	Case 1
96	15	44	0	8	0	0	Case 1
97	16	56	0	0	0	26	Case 4

(P-86a)

FIELD TEST DATA FOR REACTOR BLDG. #2

Placement: 184-S-01 Area: 34 Elev. 719'-1"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
98	17	76	1	0	0	10	Case 4
99	18	153	15	0	0	64	Case 4
100	19	71	4	0	0	0	Case 1
101	20	70	4	7	3	7	Case 3
102	21	70	5	4	0	14	Case 3
103	22	72	9	1	1	45	Case 2
104	23	269	9	0	0	110	Case 4
105	24	70	2	1	0	20	Case 2
106	25	70	3	0	0	27	Case 4
107	26	69	7	0	0	8	Case 4
108	27	73	23	28	5	0	Case 1
109	28	256	37	13	1	105	Case 3
110	29	86	5	35	13	1	Case 3
111	31	245	12	0	0	89	Case 4

(P-86a)

FIELD TEST DATA FOR CONTROL BUILDING

Placement: 714-S-03 Area: 21

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
1	1	169	0	24	6	19	Case 3
2	2	174	7	15	4	5	Case 3
3	3	170	6	14	0	21	Case 3
4	4	167	4	22	5	15	Case 3
5	5	202	0	38	11	11	Case 3
6	5A	54	1	9	7	7	Case 3
7	6	204	1	34	7	20	Case 3
8	7	210	2	29	6	9	Case 3
9	8	141	0	13	4	13	Case 3
10	9	138	0	3	2	19	Case 3
11	10	135	5	9	8	0	Case 1

FIELD TEST DATA FOR TURBINE BLDG. #1

Placement: - Area: 16 Elev. 729'-0"

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
1	1	18	0	0	0	0	Case 1
2	2	64	0	8	0	0	Case 1
3	3	36	0	1	0	0	Case 1
4	4	32	0	1	0	0	Case 1
5	5	100	0	8	0	0	Case 1
6	6	24	0	1	0	0	Case 1
7	7	24	0	4	0	0	Case 1
8	8	124	5	10	1	0	Case 1
9	9	80	0	1	0	0	Case 1
10	10	46	0	0	0	0	Case 1
11	11	45	1	1	0	0	Case 1
12	12	48	0	0	0	0	Case 1
13	13	6	0	0	0	0	Case 1
14	14	6	0	0	0	0	Case 1
15	15	42	0	5	0	0	Case 1
16	16	40	0	4	0	0	Case 1
17	17	96	0	9	0	0	Case 1

(P-86b)

FIELD TEST DATA

Sample No.	Beam No.	Stud Installed	Studs Failing Soundness Test	Studs Failing Visual Exam. With Bend Test Results: Total	Studs Failing Visual Exam. With Bend Test Results: Failing Bend Test	Studs Failing Visual Exam. But Repaired	Remarks
		T	FS	FV1	F1	FV2	
Circulating Water Pumphouse							
1	1	53	0	0	0	0	Case 1
2	2	54	0	1	0	0	Case 1
Diesel Generator Building							
1	1	44	0	0	0	0	Case 1

(P-86b)

APPENDIX C

TO

FINAL REPORT ON SHEAR STUDS

REDUCED FIELD TEST DATA

(P-74b)

SUMMARY

OF

REDUCED FIELD DATA

Structure	Sample Nos.	Total Studs	Total Pass	Total Fail
Reactor Building Units 1 and 2	44	5372	4970	402
Turbine Building Units 1 and 2	17	831	824	7
Control Building	11	1764	1633	131
Circulating Water Pumphouse	2	107	107	0
Diesel Generator Building	1	44	44	0

Note: For the explanation of terms and expressions used on this and the following pages refer to Appendix "A".

REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Building : Reactor Building

Sample No.	Total Studs	Studs Fail-ing Soundness	Studs Passing Visual	Studs failing visual with bend test results: Total	Pass bend test	Fail bend test	Studs failing visual but repaired prior to bend test: Total	Assumed Pass	Assumed Fail	Pass (Pv+P1 +P2)	Fail (Fs+F1 +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
13	76	6	70	0	0	0	0	0	0	70	6	
16	88	0	67	21	19	2	0	0	0	86	2	
17	86	0	58	27	19	8	1	0	1	77	9	
18	88	4	68	16	13	3	0	0	0	81	7	
19	86	0	52	34	27	7	0	0	0	79	7	
20	88	0	65	8	5	3	15	9	6	79	9	
27	213	0	174	37	36	1	2	1	1	211	2	
28	90	2	68	18	15	3	2	1	1	84	6	
29	132	0	114	8	7	1	10	8	2	129	3	
30	62	0	46	16	13	3	0	0	0	59	3	

REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Building : Reactor Building

Sample No.	Total Studs	Studs Failing Soundness	Studs Passing Visual	Studs failing visual with bend test results: Total	Pass bend test	Fail bend test	Studs failing visual but repaired prior to bend test: Total	Assumed Pass	Assumed Fail	Pass (Pv+Pl +P2)	Fail (Fs+Fl +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
32	711	4	553	52	51	1	102	100	2	704	7	
33	177	4	111	62	53	9	0	0	0	164	13	
34	149	0	130	19	19	0	0	0	0	149	0	
35	86	1	71	14	11	3	0	0	0	82	4	
37	96	1	79	16	11	5	0	0	0	90	6	
42	101	1	55	41	41	0	4	4	0	100	1	
45	88	0	81	7	7	0	0	0	0	88	0	
47	130	3	79	24	20	4	24	20	4	119	11	
50	34	0	20	13	10	3	1	0	1	30	4	
51	10	0	10	0	0	0	0	0	0	10	0	
53	157	0	139	16	11	5	2	1	1	151	6	
54	274	51	136	67	52	15	20	15	5	203	71	

REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Building : Reactor Building

Sample No.	Total Studs	Studs Failing Soundness	Studs Passing Visual	Studs failing visual with bend test results: Total	Pass bend test	Fail bend test	Studs failing visual but repaired prior to bend test: Total	Assumed Pass	Assumed Fail	Pass (Pv+Pl +P2)	Fail (Fs+Fl +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
55	57	0	38	18	12	6	1	0	1	50	7	
56	57	0	38	18	10	8	1	0	1	48	9	
57	44	2	12	30	21	9	0	0	0	33	11	
58	45	4	23	18	14	4	0	0	0	37	8	
59	48	2	26	14	14	0	6	6	0	46	2	
61	21	1	14	6	3	3	0	0	0	17	4	
62	223	4	125	94	75	19	0	0	0	200	23	
63	38	1	15	22	10	12	0	0	0	25	13	
76	204	5	178	11	10	1	10	9	1	197	7	
78	307	2	305	0	0	0	0	0	0	305	2	
84	68	2	34	16	16	0	16	16	0	66	2	
86	68	2	33	8	8	0	25	25	0	66	2	

(P-86b)

REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Building : Reactor Building

				Studs failing visual with bend test results			Studs failing visual but repaired prior to bend test					
Sample No.	Total Studs	Studs Failing Soundness	Studs Passing Visual	Total	Pass bend test	Fail bend test	Total	Assumed Pass	Assumed Fail	Pass (Pv+P1 +P2)	Fail (Fs+F1 +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
87	68	0	35	2	2	0	31	31	0	68	0	
93	266	11	138	4	4	0	113	113	0	255	11	
95	166	3	121	42	34	8	0	0	0	155	11	
96	44	0	36	8	8	0	0	0	0	44	0	
100	71	4	67	0	0	0	0	0	0	67	4	
101	70	4	52	7	4	3	7	4	3	60	10	
102	70	5	47	4	4	0	14	14	0	65	5	
108	73	23	22	28	23	5	0	0	0	45	28	
109	256	37	101	13	12	1	105	96	9	209	47	
110	86	5	45	35	22	13	1	0	1	67	19	

REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Building : Turbine Building

Sample No.	Total Studs	Studs Fail-ing Soundness	Studs Passing Visual	Studs failing visual with bend test results: Total	Pass bend test	Fail bend test	Studs failing visual but repaired prior to bend test: Total	Assumed Pass	Assumed Fail	Pass (Pv+P1 +P2)	Fail (Fs+F1 +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
1	18	0	18	0	0	0	0	0	0	18	0	
2	64	0	56	8	8	0	0	0	0	64	0	
3	36	0	35	1	1	0	0	0	0	36	0	
4	32	0	31	1	1	0	0	0	0	32	0	
5	100	0	92	8	8	0	0	0	0	100	0	
6	24	0	23	1	1	0	0	0	0	24	0	
7	24	0	20	4	4	0	0	0	0	24	0	
8	124	5	109	10	9	1	0	0	0	118	6	
9	80	0	79	1	1	0	0	0	0	80	0	
10	46	0	46	0	0	0	0	0	0	46	0	
11	45	1	43	1	1	0	0	0	0	44	1	
12	48	0	48	0	0	0	0	0	0	48	0	

REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Building : Turbine Building

Sample No.	Total Studs	Studs Failing Soundness	Studs Passing Visual	Studs failing visual with bend test results: Total	Pass bend test	Fail bend test	Studs failing visual but repaired prior to bend test: Total	Assumed Pass	Assumed Fail	Pass (Pv+P1 +P2)	Fail (Fs+F1 +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
13	6	0	6	0	0	0	0	0	0	6	0	
14	6	0	6	0	0	0	0	0	0	6	0	
15	42	0	37	5	5	0	0	0	0	42	0	
16	40	0	36	4	4	0	0	0	0	40	0	
17	96	0	87	9	9	0	0	0	0	96	0	

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REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Building : Control Building

Sample No.	Total Studs	Studs Failing Soundness	Studs Passing Visual	Studs failing visual with bend test results: Total	Pass bend test	Fail bend test	Studs failing visual but repaired prior to bend test: Total	Assumed Pass	Assumed Fail	Pass (Pv+P1 +P2)	Fail (Fs+F1 +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
1	169	0	126	24	18	6	19	14	5	158	11	
2	174	7	147	15	11	4	5	3	2	161	13	
3	170	6	129	14	14	0	21	21	0	164	6	
4	167	4	126	22	17	5	15	11	4	154	13	
5	202	0	153	38	27	11	11	7	4	187	15	
6	54	1	37	9	2	7	7	1	6	40	14	
7	204	1	149	34	27	7	20	15	5	191	13	
8	210	2	170	29	23	6	9	7	2	200	10	
9	141	0	115	13	9	4	13	9	4	133	8	
10	138	0	116	3	1	2	19	6	13	123	15	
11	135	5	121	9	1	8	0	0	0	122	13	

(P-86b)

REDUCED FIELD DATA FOR STATISTICAL ANALYSIS

Sample No.	Total Studs	Studs Failing Soundness	Studs Passing Visual	Studs failing visual with bend test results: Total	Pass bend test	Fail bend test	Studs failing visual but repaired prior to bend test: Total	Assumed Pass	Assumed Fail	Pass (Pv+P1 +P2)	Fail (Fs+F1 +F2)	Remarks
	T	FS	PV	FV1	P1	F1	FV2	P2	F2	P	F	
Circulating Water Pumphouse												
1	53	0	53	0	0	0	0	0	0	53	0	
2	54	0	53	1	1	0	0	0	0	54	0	
Diesel Generator Building												
1	44	0	44	0	0	0	0	0	0	44	0	

APPENDIX D

TO

FINAL REPORT ON SHEAR STUDS

REPAIR PROCEDURES

(P-74b)

REPAIR PROCEDURES

1.0 General

As noted in section 7.6 of the final report, some beams in the Reactor Building have been identified, where some restitution of studs is necessary. These beams are marked on the plans (See figures 1 thru 5).

2.0 Repair Methods and Design Criteria

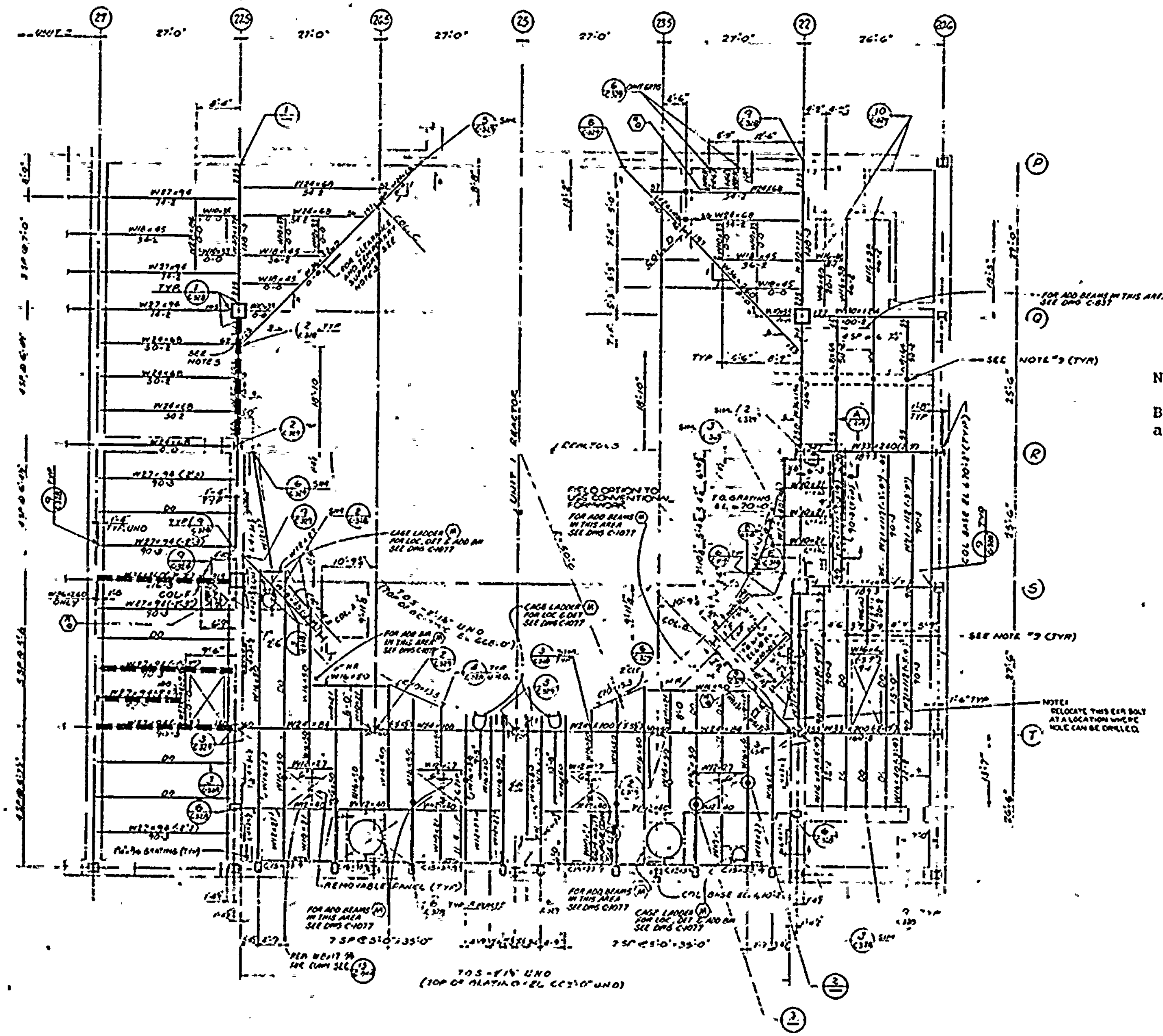
Following repair methods are proposed to achieve the required restitution.

2.1 The first method is to provide a horizontal shear key within the ridge when the metal deck is provided over and across the steel beams. The shear key is well anchored to the top flange by a friction type bolt. Positive engagement and the contact at the key-decking is attained by the bonding properties of the epoxy agent, and at the decking-slab interface is developed by the concrete engagement into the corrugation of the decking. See figure 6 for details.

2.2 The second approach is to provide a through-bolt where the decking corrugations are parallel to the steel beams. The basic concept here is to develop a friction type connection between beam and slab through the pre-tensioned, high strength bolt. The

grouting of the bolt in the drilled hole and the friction connection render the detail effective by minimizing the tendency of initial slip. See figure 7 for details.

2.3 In some instances, when the decking is parallel to the beam and the above method cannot be used because of embedded conduits in the slab, it is proposed to design the steel beam as a non-composite section and reinforce the existing beam to provide the required section modulus. The actual details of reinforcement will be designed on a case by case basis depending on the existing conditions at the time of repair.



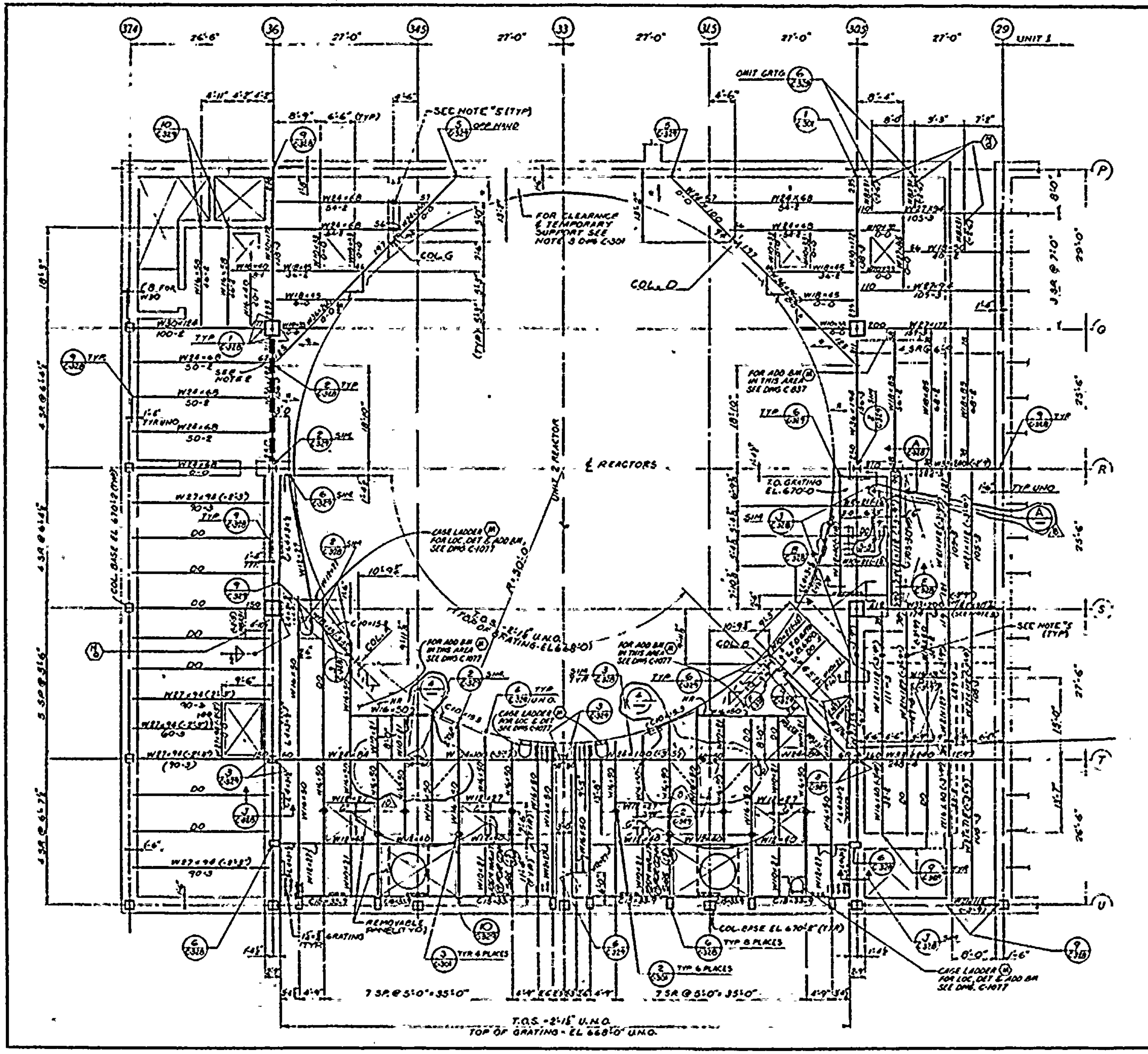
NOTE:

Beams to be repaired are marked thus: ▬ ▬ ▬ ▬ ▬

Structural Steel
Framing Plan El. 670'-0"
Reactor Building Unit 1

Figure 1

Appendix D



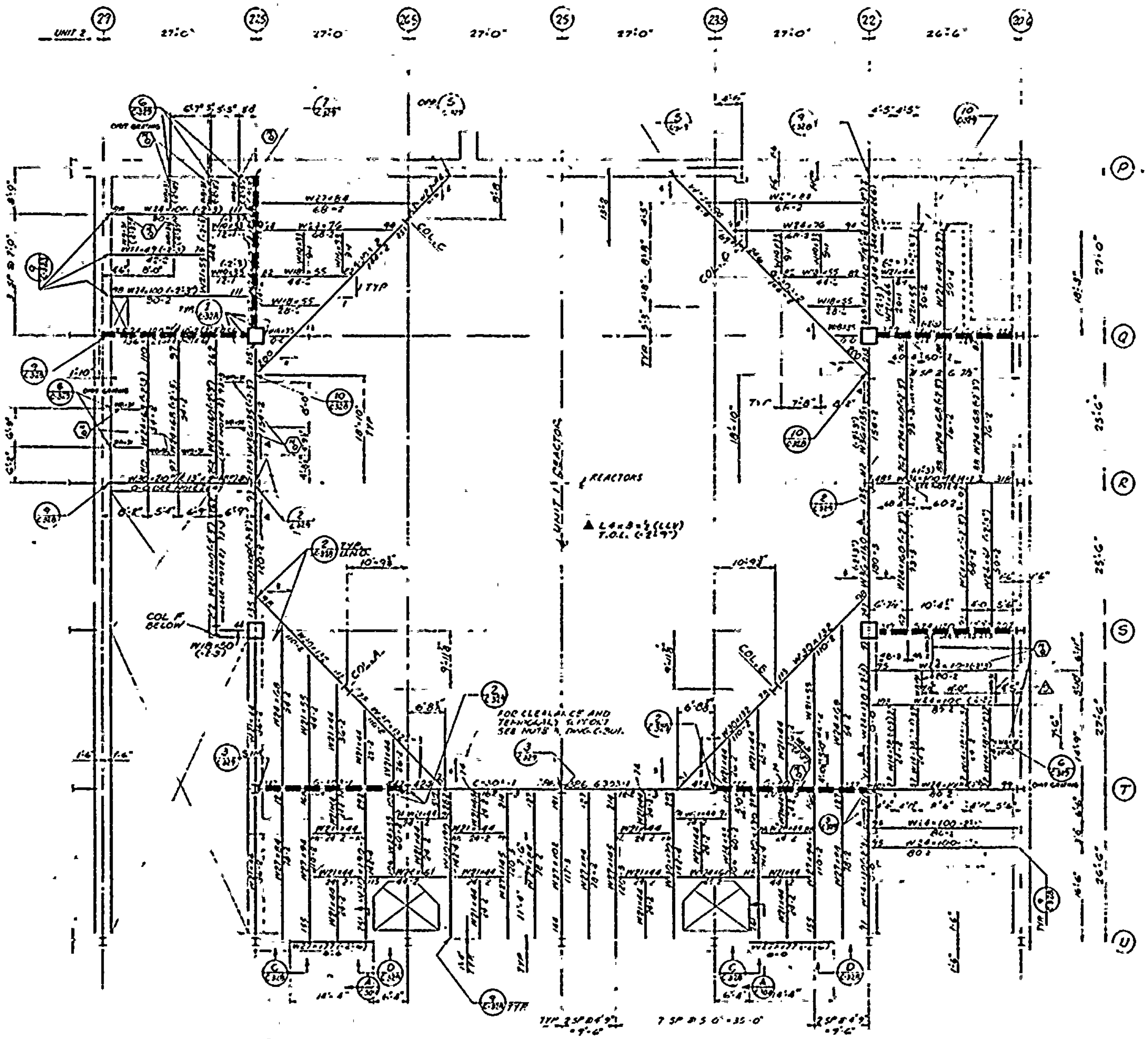
NOTE:

Beams to be repaired are marked thus: ▬ ▬ ▬ ▬ ▬ ▬ ▬

Structural Steel
Framing Plan at El. 670'-0"
Reactor Building Unit 2

Figure 2

Appendix D



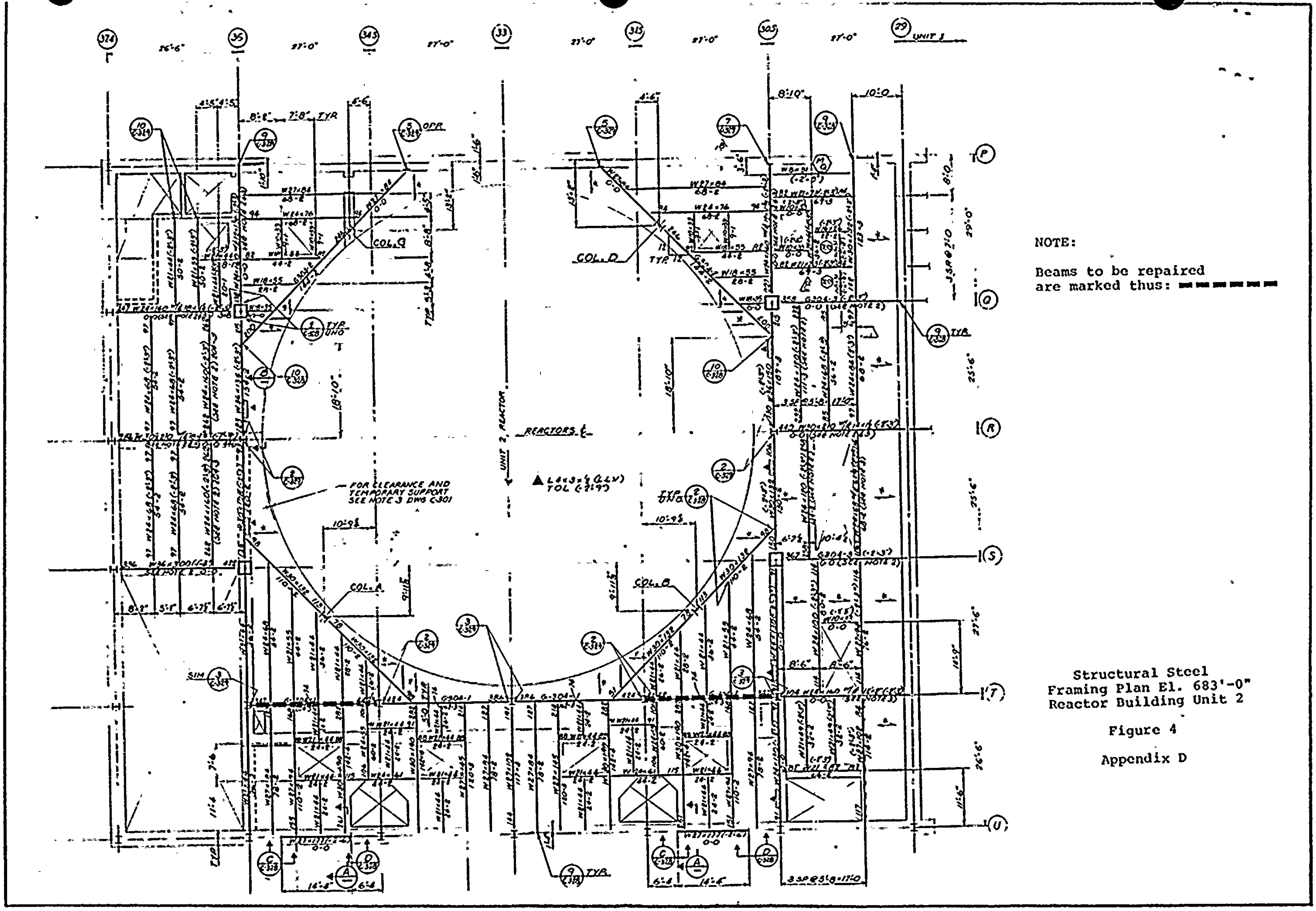
NOTE:

Beams to be repaired are marked thus:

Structural Steel
Framing Plan El. 683'-0"
Reactor Building Unit 1

Figure 3

Appendix D



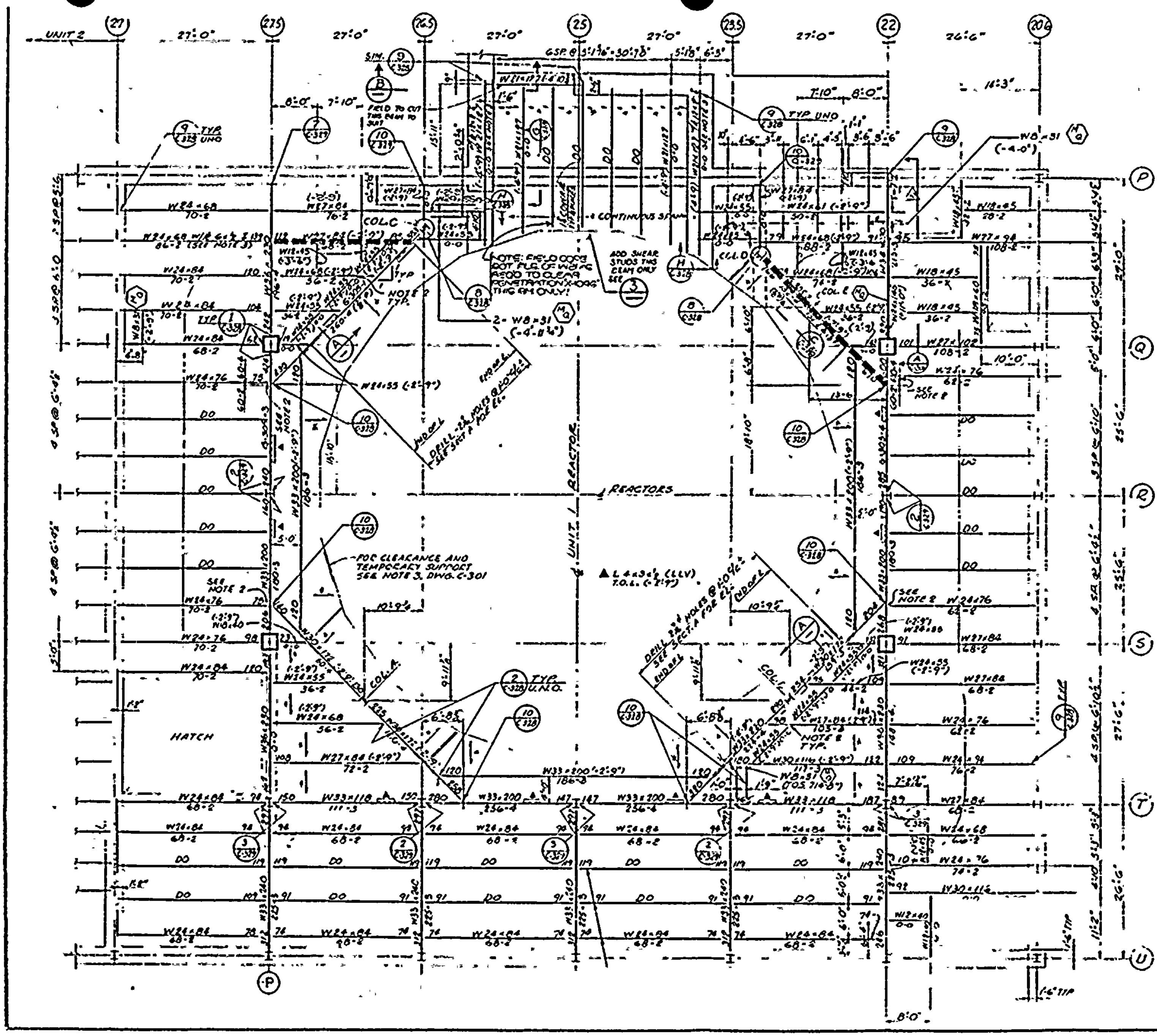
NOTE:

Beams to be repaired are marked thus: ▬ ▬ ▬ ▬ ▬ ▬

Structural Steel
Framing Plan El. 683'-0"
Reactor Building Unit 2

Figure 4

Appendix D



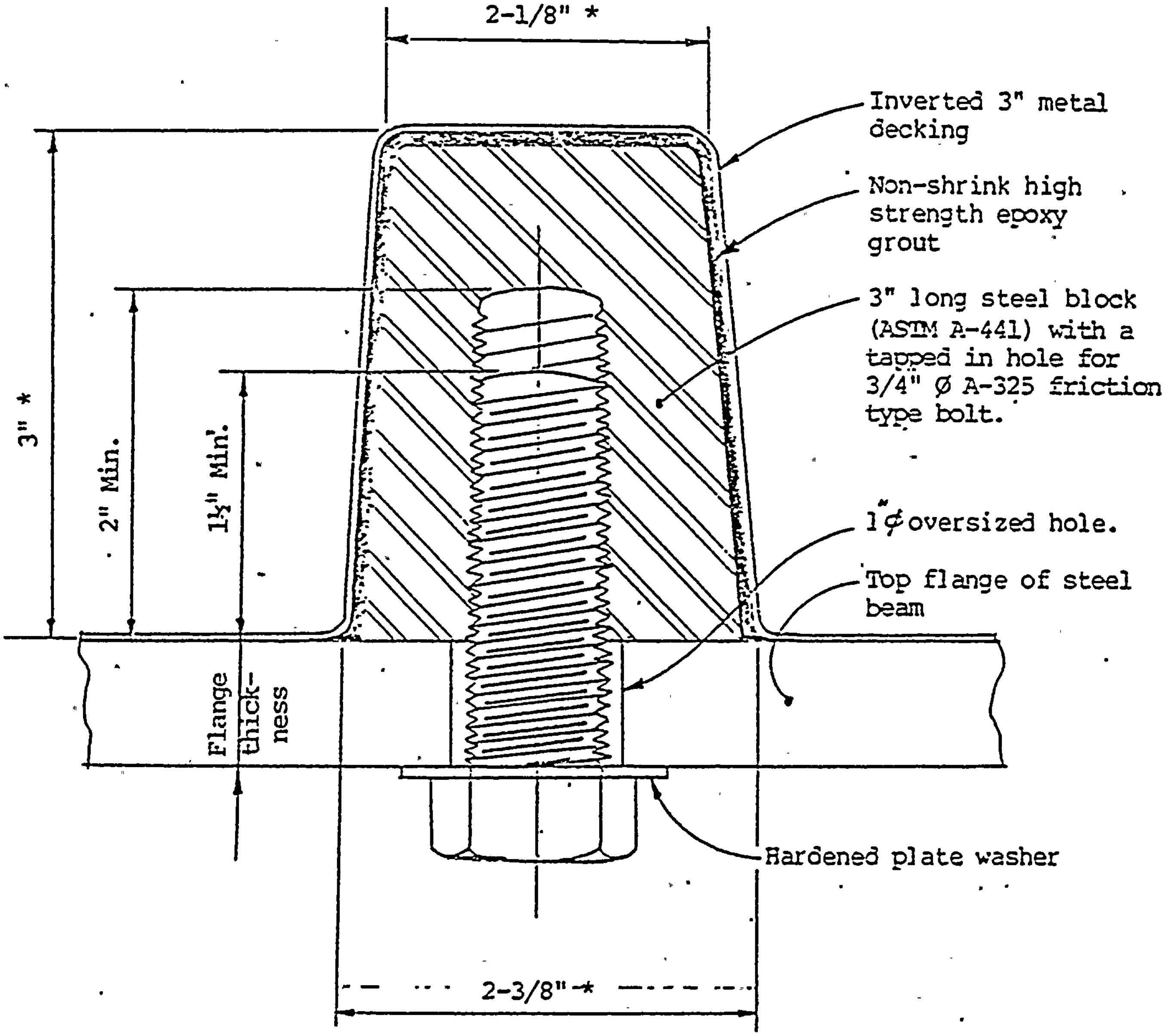
NOTE:

Beams to be repaired are marked thus: ▬ ▬ ▬ ▬ ▬

Structural Steel Framing Plan El. 719'-1"
Reactor Building Unit 1

Figure 5

Appendix D

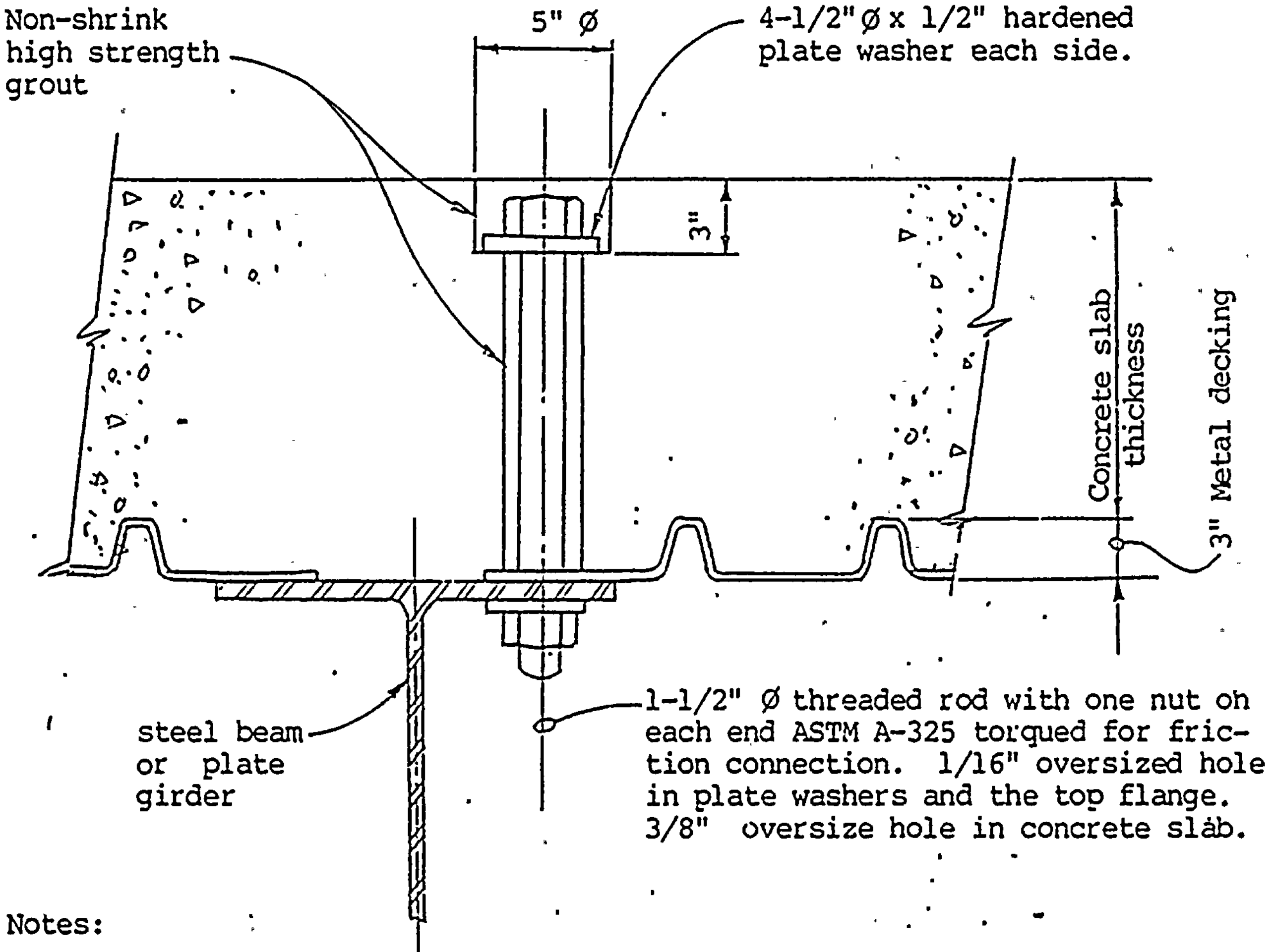


* Nominal decking dimensions per manufacturer's catalog

REPAIR PROCEDURE - METHOD '2'

FIGURE 6

APPENDIX 'D'



Notes:

1. Prior to drilling check hole location as follows:

   - with rebar detector, ascertain that top layer or reinforcement and any embeds are clear of hole.

2. Preferred location is at valley of decking corrugations. Do not locate thru sides of decking.

REPAIR PROCEDURE - METHOD '1'

FIGURE 7

APPENDIX 'D'

APPENDIX E

TO

FINAL REPORT ON SHEAR STUDS

BASIC THEORY OF COMPOSITE BEAM CONSTRUCTION

BY

ENGINEERING DECISION ANALYSIS COMPANY

(P-74b)

BASIC THEORY OF COMPOSITE BEAM CONSTRUCTION

SUSQUEHANNA STEAM ELECTRIC STATION

prepared for

BECHTEL POWER CORPORATION
San Francisco, California

21 December 1977

EDAC

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TABLE OF CONTENTS

SYNOPSIS

This report presents a general ultimate strength theory for composite beams that fits the type found in the Susquehanna Steam Electric Station (SSES) and more conventional construction. The construction of the SSES employs composite beams having heavy, thick reinforced concrete slabs poured on a formed steel deck which in turn is supported by the generally unshored steel beams. In contrast, the construction in ordinary buildings employs a thin lightweight floor slab with a formed steel deck supported on deep but light steel rolled sections.

An extensive study of the experimental data upon which the AISC specifications are based was made since the project beams are very different from those for which the AISC specifications are meant to apply. It is shown that the AISC specifications are grossly conservative. A valid ultimate strength procedure which fits the experimental data and the project beams is derived based on recognized concepts.

The study closes with recommendations for use in evaluating the project beams.

1. INTRODUCTION

This report is prepared in accordance with Bechtel Contract No. 7 PE-TSA-11 and in accordance with meetings between Bechtel Power Corporation and Engineering Decision Analysis Company, Inc. (EDAC). This report is concerned with a study of the basic theory of composite beam construction and the relationship to the specifications of the American Institute of Steel Construction. The focus is on the type of composite construction employed in the SSES.

Chapter 2 of this report is concerned with the general theory of composite beam construction and the verification of that theory. Chapter 3 focuses on the suitability of the AISC specifications for composite construction with beams of the type employed in the SSES design. The experimental data upon which the AISC specifications are based involve a thin concrete slab poured on a formed steel deck with shear studs connecting the concrete slab to a steel beam. In laboratory tests, there was sufficient slippage between the slab and the steel beam for all studs in the shear span to be developed, and failure was associated with concrete failure involving pull out of the studs from the slab and the development of a yield hinge in the steel beam. The bending strength of the slab by itself on the span of the steel beams was very small, so that the strength of the composite beam was the sum of the strength of the steel beam and the stud connection in terms of ultimate bending movement. In all cases, the dead load was very small compared to the ultimate load.

The beams employed in the project differ greatly from the test beams in that the slab thickness is of the same order as that of the steel beam. The slab is heavily reinforced. The dead load is not small compared to the live load and the steel beams are generally unshored when the slab is placed so that the steel beam supports all of the dead load while composite behavior is present under live load.

Analyses presented in Chapter 2 disclose that the AISC specifications must be modified to fit beams of the type of interest in this study. A general method of analysis and design is presented in Chapter 3 which fits the experimental data, is consistent with the literature, and provides a relationship between the AISC specifications and construction of the type employed in the project.

Finally, Chapter 4 presents recommendations and conclusions.

2. GENERAL THEORY OF COMPOSITE BEAM CONSTRUCTION AND VERIFICATION OF THE THEORY

This chapter is concerned with a development of a general strength theory and verification of that theory by comparison with experimental results of tests of composite beams employing a formed steel deck. The proven analytical methodology is then compared with the AISC specifications in Chapter 3. A methodology for analysis of the composite beams in the SSES is also presented in Chapter 3.

THEORY

The discussion that follows is based on the work of Grant, Fisher, and Slutter (Ref. 1). The methodology is based on the ultimate strength of the composite beam. Sufficient slippage is assumed to take place at the slab beam interface to assume that each shear stud in the shear span carries the same loading.

The AISC specifications assume that it is possible to relate the ultimate bending strength of the composite section in which the steel beam develops a yield hinge to an elastic stress analysis at the same section using transformed section techniques focused on the unit stress in the bottom tension flange of the steel beam. The assumption is also made that the effective section modulus of the composite section is a linear function of the ratio of the capacity of the shear studs in the shear span to the theoretical limit of this capacity.

Examination of the experimental data upon which the AISC specifications are based discloses that the composite beams that have been tested fit a particular type of building construction, that involving a thin concrete floor slab, and light but deep steel beams. The largest slab thickness in 74 tests was 9 in. with a 3 in rib height making a 6 in. net slab thickness. The beam span was 34.9 ft. More than half of the slabs were constructed of lightweight concrete. The bending strength of the slab was neglected in the analysis. The slab was effectively considered to be a purely compression member with the compressive force located at the center of gravity of the concrete section neglecting the rib concrete.

The single elastic deformation requirement is that the curvature of the net concrete slab be the same as that of the steel beams. If both slab and beam are elastic, the live load carried by the slab and beam is proportional to their stiffnesses (EI). The largest ratio of slab to beam stiffness in the experimental data is 0.15, that for the 17 Lehigh test ranges from 0.009 to 0.021, and Grant, Fisher, and Slutter say that this ratio is generally less than 0.05. With project beam 14, this ratio is 2.07.

Grant, Fisher, and Slutter (Ref. 1) state that the ratio of the section modulus of the transformed section to that of the steel beams is approximately 1.5 for composite beams commonly used in building construction. This ratio is 2.9 for project beam 14.

The general theory for ultimate strength of a composite beam is shown in Figure 2-1. The equilibrium condition is shown in Figure 2-1b and 2-1c. With the experimental beams, the slabs were very flexible compared to the steel section. In Figure 2-1c, a bending momemt is shown to exist at the slab to steel beam interface. This bending moment is large compared to that from load distribution in all experimental tests. With very thin slabs, it is reasonable to assume that the compressive force in the slab acts at the center of gravity of the net concrete section (see Grant, Fisher, and Slutter) (Fig. 2-1c). The tensile force on the steel section acts to reduce the plastic moment capacity (Fig. 2-1d). In the analysis of the experimental tests made in this study, it was assumed that the web and flanges of the steel rolled section were of constant thickness as given in AISC handbook.

With thick slabs it is necessary to modify the theory to account for the ultimate strength characteristics of the slab (Fig. 2-2). Equation 4 results and this relationship were checked by comparison with the experimental data. The analysis showed that the mean ratio of experimental to calculated strength was 1.000 (0.9997) with a standard deviation of 0.081 for the 74 test beams and the data had a range of 0.835 to 1.1884. The ratio of observed-to-calculated capacity is plotted in the histogram of Figure 2-3 and the same data are plotted on normal probability paper in Figure 2-4. The fit to a straight line is excellent so that the observed variability can be assumed to be the sum of random variations no one of which is dominant. The standard deviation is equal to the coefficient of variation with these data since the mean is unity. The coefficient of variation is of the same order as that found in the yield point of steel rolled sections of nominally identical material.

The analytical comparison is also shown in Figure 2-5 in which the ratio of experimental-to-calcuated strength is plotted against the ratio of shear stud capacity provided to maximum shear stud capacity. It appears reasonable to state that the reliability of the theory is not a function of the shear stud design level. That is, the design with a V'h/Vh of 0.25 is fully as reliable as that with a ratio of unity.

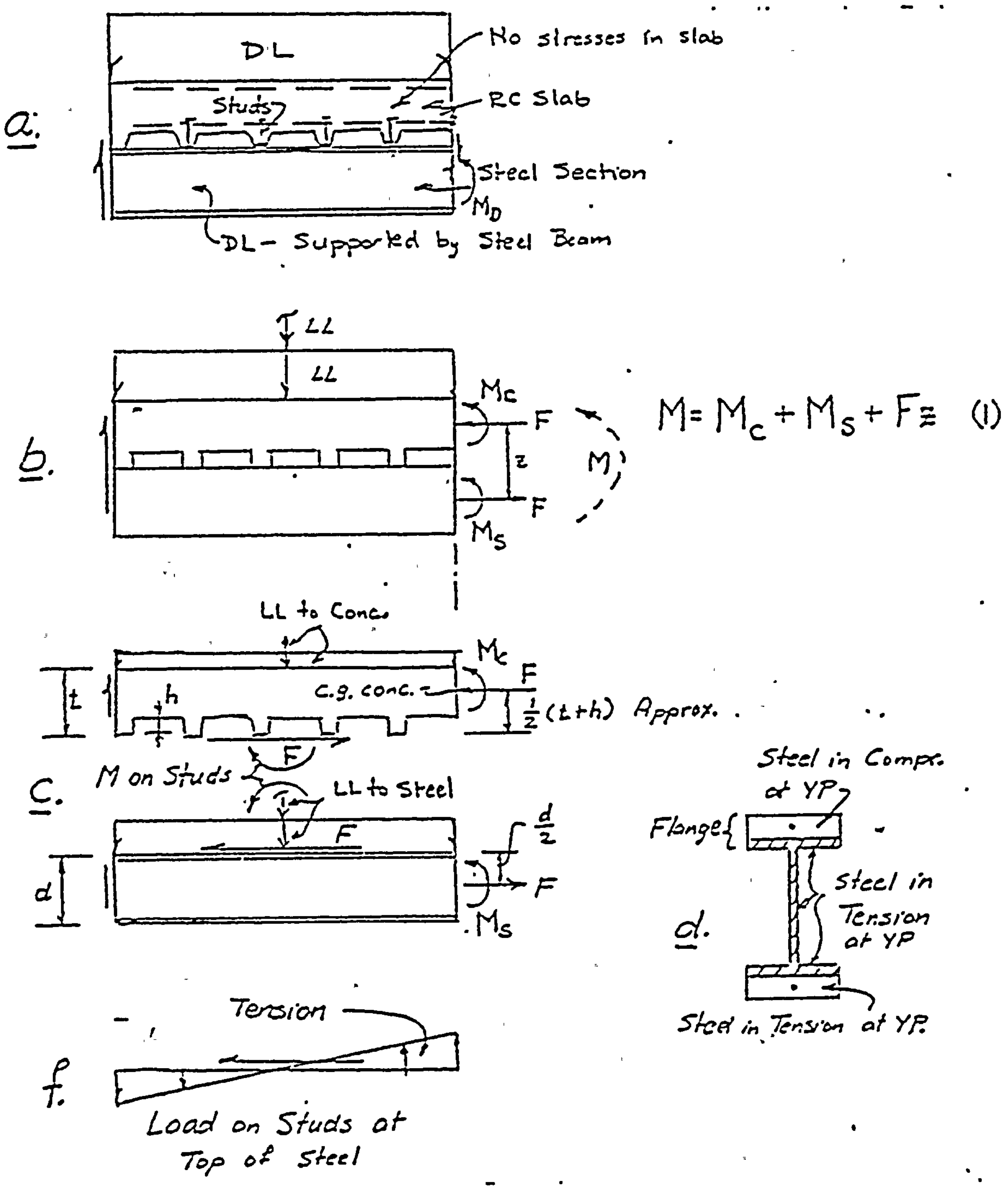
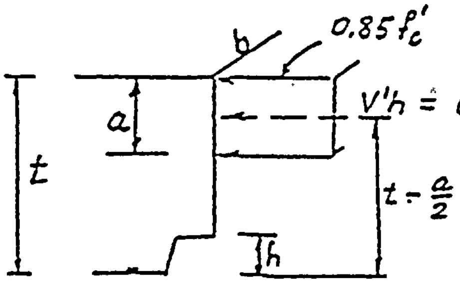
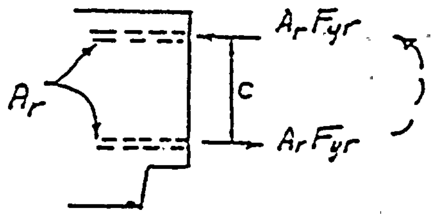


FIGURE 2-1 COMPOSITE BEAM RELATIONSHIPS



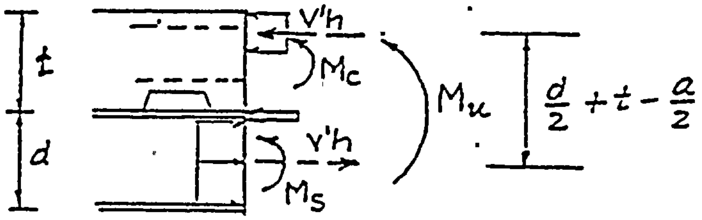
$$V'h = 0.85 f'_c a b$$

$$a = \frac{V'h}{0.85 f'_c b} \qquad (2)$$



$$M_c = A_r F_{yr} c$$

a. Slab Conditions



$$M_u = M_s + M_c + V'h\left(\frac{d}{2} + t - \frac{a}{2}\right) \qquad (3)$$

$$a = \frac{V'h}{0.85 f'_c b}$$

$$Vh_c = 0.85 f'_c b (t-h)$$

$$\underline{M_u = M_s + M_c + \frac{V'h}{2}\left[d + t + h + (t-h)\left(1 - \frac{V'h}{Vh_c}\right)\right]} \qquad (4)$$

$$\underline{S_T = \frac{M_u}{(1.7)(0.66 F_y)}} \qquad (5)$$

FIGURE 2-2 COMPOSITE BEAM ULTIMATE STRENGTH RELATIONSHIPS

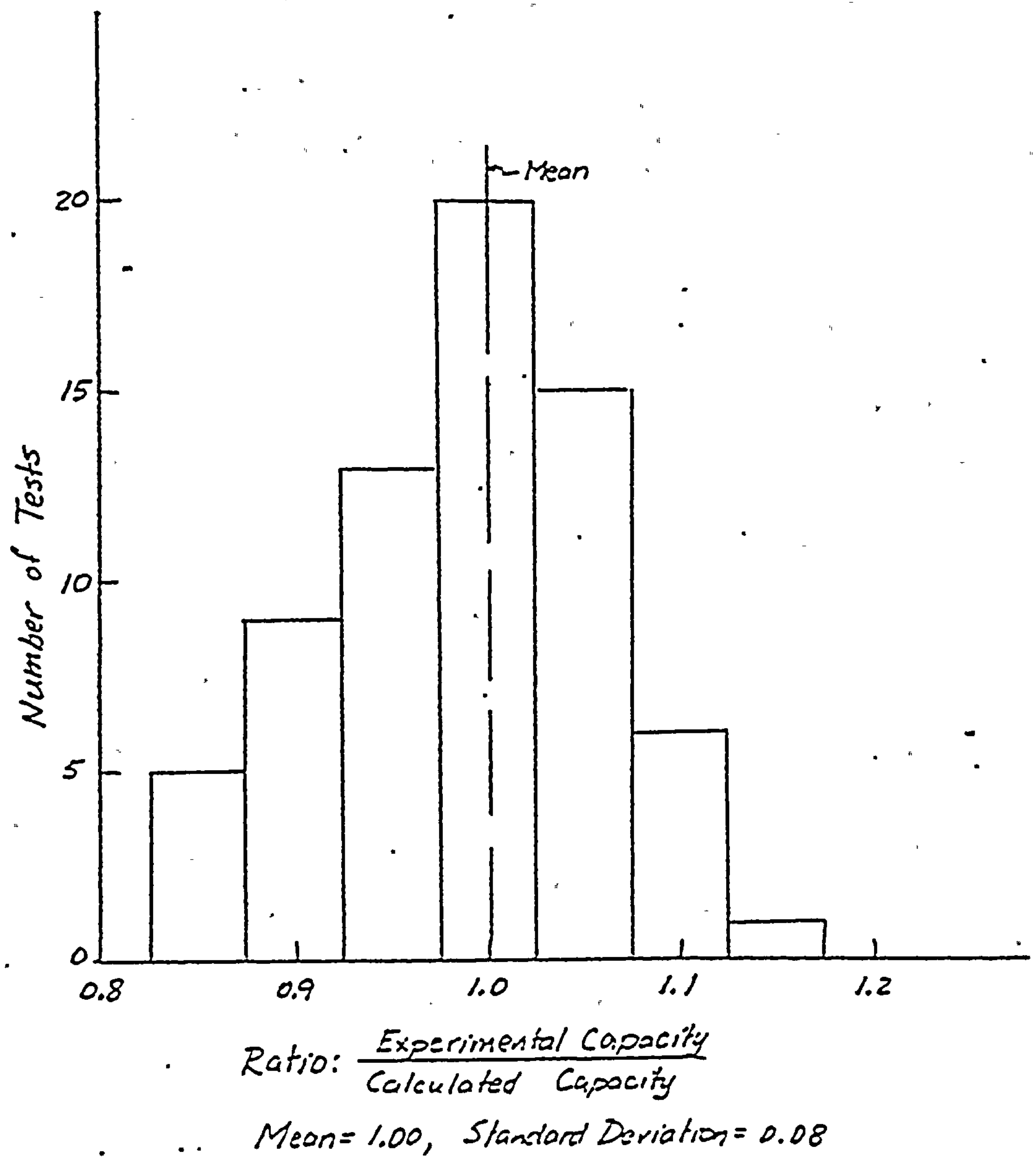


FIGURE 2-3 HISTOGRAM OF EXPERIMENTAL TO CALCULATED ULTIMATE STRENGTH

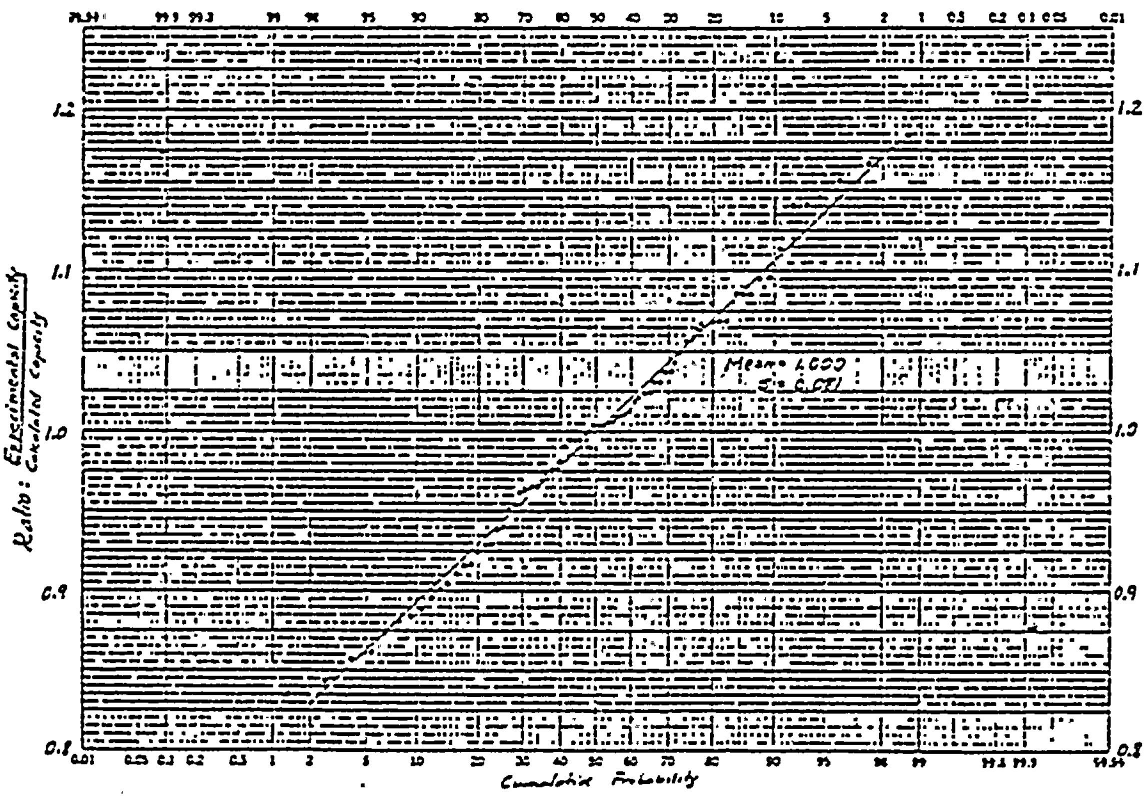


FIGURE 2-4 PLOT OF RATIO OF EXPERIMENTAL CAPACITY TO CALCULATED CAPACITY ON NORMAL PROBABILITY PAPER

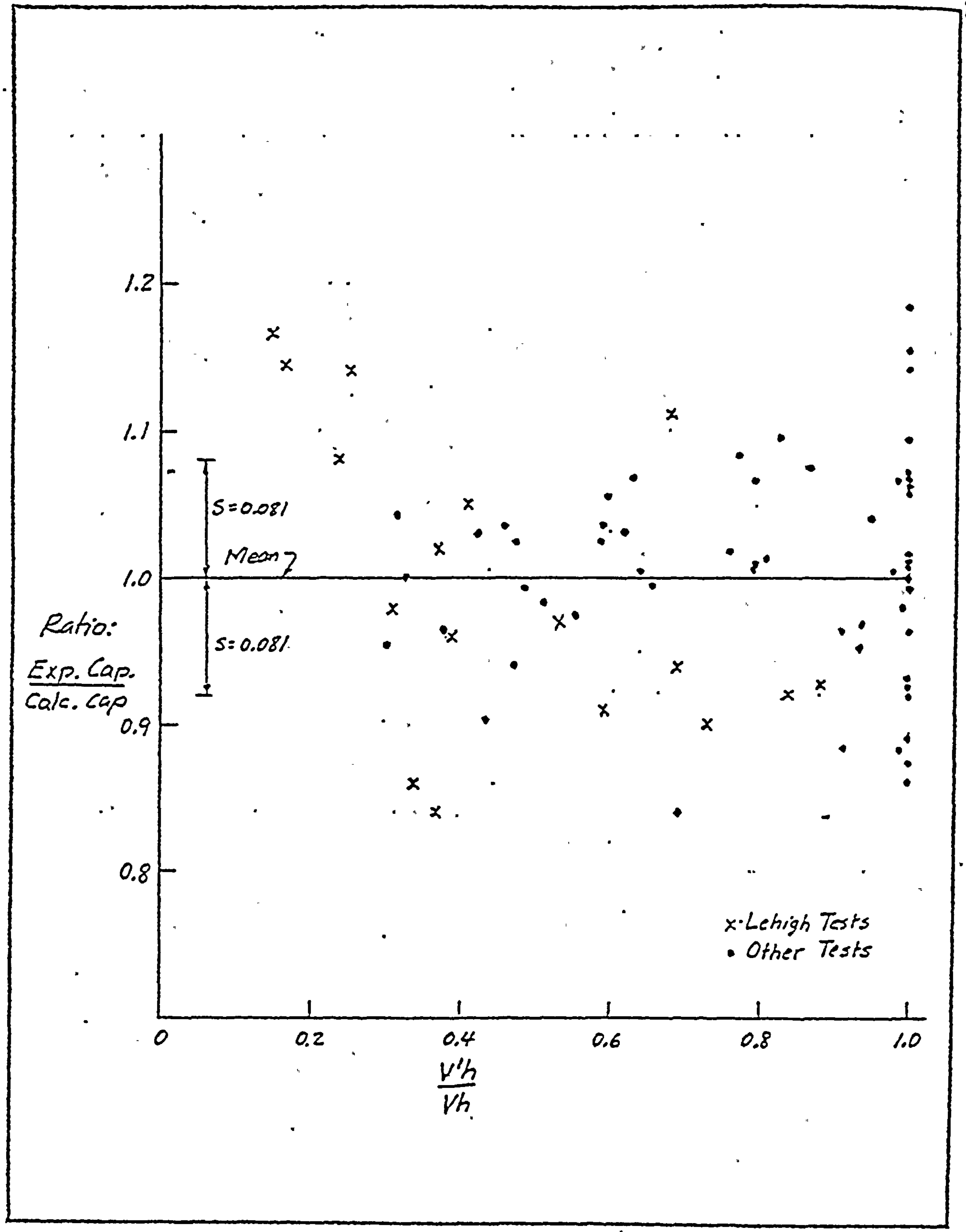


FIGURE 2-5 PLOT SHOWING ZERO CORRELATION OF STRENGTH RATIO WITH V'h/Vh

3. COMPARISON OF THEORY WITH AISC SPECIFICATIONS

The 1969 Edition of the AISC specifications employs the relationship shown in Figure 3-1 for elastic design based on ultimate strength properties. The criteria is the tensile stress in the bottom flange of the steel beam (0.66 Fy) and the effective section modulus for elastic design is equal to a simple linear function of the section modulus of the rolled steel section, the transformed section modulus referred to the bottom flange, and the ratio of actual shear stud capacity to the maximum shear stud capacity. The true effective section modulus for pseudo elastic design is given by Equation 5 (Fig. 1-2) in which the load factor is 1.7 and the allowable unit stress is 0.66 Fy.

The true section modulus for each of the experimental beasm using the calculated ultimate strength by Equation 4 of Chapter 2 is plotted in Figure 3-1 against the effective section modulus defined by the AISC specifications. The plot shows that the AISC relationship is conservatively biased by approximately 30 percent based on a mean value function. However, approximately 50 percent of the beams have capacities smaller than that defined by the mean value function. The variability of the data about the mean value function appears to be independent of the section modulus and independent of V'h/Vh. The AISC relationship approximates a lower bound on strength for section modulus up to approximately 50 to 100 $in.^3$.

The variability shown in Figure 3-1 is consistent with that of the plastic design methodology for structural steel beams so that it does not appear reasonable to require the conservatism for composite beams with a section modulus larger than approximately 100 in.^{3} The project beams of interest have very large section modulus, of the order of 1200 in.^{3} There is a strong trend for the shear stud connection to show a decrease in variabilty with increase in the number of studs owing to the low correlation between individual stud strengths.

No studies were made of the experimental data with respect to stud properties.

ANALYSIS OF COMPOSITE BEAMS

Strict elastic analysis of a composite beam cannot account for the undefined slippage on the slab to steel beam interface so that it is necessary to employ pseudo elastic procedures which fundamentally are based on ultimate strength properties. Thus this discussion will focus on the analysis based on ultimate strength, Figure 3-2.

Equation 4 of Chapter 2 defines the ultimate moment capacity of a composite section for combined dead and live load. At ultimate, the beam develops a yield hinge, the reinforced concrete slab is at its ultimate capacity, and the V'h force has its largest possible moment arm consistent with the strain conditions in the steel beam and the slab.

With three interrelated sources of strength, it is possible for any one source to develop the necessary capacity, any combination of two souces, or all three sources together. In general, the design will not be balanced so that at least one source need not be fully developed. The analysis that follows considers first the steel beam to its plastic limit, then adds the reinforced concrete slab to its ultimate, and then adds as many shear connectors as necessary to satisfy the loading criteria while accounting for the influence of the tension on the steel beam and for the compression in the slab.

From the standpoint of ultimate load, it makes no difference whether the steel beam is shored or unshored at the time the concrete for the slab is placed. This is true regardless of the stress condition in the steel beam under dead load alone as a consequence of redistribution of loading among the three resisting systems prior to ultimate. The ultimate strength is independent of the path employed to attain the ultimate strain conditions.

The same is not true with regard to deflections and rigidity. If both the steel beam and the slab deform elastically while slippage is allowed at the stud line, the requirement of identical curvature allows the calculation of the load carried by the slab and the steel beam. If no shear studs are provided, the deflection is that of the steel beam under the loading supported by the steel beam (with proper accounting for the dead load deflection). With shear studs, the elastic stress conditions are undefined since the slippage conditions at the shear studs are undefined. However, if the dead load (concrete slab and steel beam) unit stresses in the bottom flange of the steel beam reach the yield point under this loading, the composite beam will show degrading rigidity with the application of further loading although the ultimate capacity of the composite section is unchanged.

A pseudo elastic analysis of the composite section is shown in Figure 3-2. A wide variety of such empirical procedures are possible.

ANALYSIS OF PROJECT BEAM 14

Project beam 14 is analyzed in Figure 3-3 both on an ultimate strength and a pseudo elastic analysis concept. From the standpoint of ultimate strength, it is seen that the slab and steel beam without composite action can supply 93 percent of the required moment capacity. A trial stud capacity (in the shear.span) of 200 kips was assumed. The strength exceeded the required capacity with only nine studs needed when 46.5 are provided and 42 are effective at a normal 2 percent level. See EDAC Report 249.03, "Studies of Shear Stud Adequacy -- Susquehanna Steam Electric Station," for development of the equivalence relationship.

A pseudo elastic analysis of project beam 14 is also shown in Figure 3-3. The analysis begins by assuming that there are no shear studs and checks for design adequacy assuming that the steel beam supports all the dead load and its proportion of the live load. It is found that the stiff slab is not adequately reinforced to support its portion of the live load while the steel beam unit stresses are less than allowable. The elastic slab capacity plus the steel beam capacity is 92 percent of that needed (neglecting elastic strain requirements). A trial V'h of 200 kips (elastic) produced a satisfactory capacity with the steel section not used to capacity or a V'h of 100 kips was satisfactory with the steel at elastic capacity. The required number of studs was nine with 100 kip stud loads and 18 with 200 kip stud loads.

OTHER AISC PROVISIONS

The AISC specifications contain a limitation on the transformed section modulus which is a function of the ratio of dead to live load bending moment (Equ. 1.11-2) and stud layout relationship (11.1-6). There appears to be no justification for the equation involving the live to dead load bending moment ratio. From the standpoint of ultimate strength, the strain condition at ultimate strength is independent of the

ratio of live-to-dead load. Even if the unit stresses in the bottom flange of the steel beam are at full yield under the dead load (unshored), the ultimate moment capacity of the composite section is unchanged. The dead load is considered the same as the live load in the strength calculation. With unit stresses under dead load limited to 0.66 Fy, there appears to be no justification for the specification. It was not possible to determine the basis of the requirements.

The second requirement dealing with the layout of shear studs in the shear span problem cannot be justified on the basis of ultimate strength considerations. The Lehigh tests involved a four-point loading with one-quarter of the loading applied at a point 19 to 22 percent of the span from the end supports.

A variety of shear stud arrangements were examined in the Lehigh tests ranging from proportioning the layout in accord with the relative shear in the span to a uniform layout indepedent of the shear in the composite beam. Statistical analysis of the data relating the experimental to calculated strength (not considering stud layout) as a function of the studs in the region of maximum shear to the total number of studs showed that strength is uncorrelated with layout (Fig. 3-4). Unless other evidence exists to verify AISC Equation 1.11-6 (p. 5-35), the relationship is not valid. The result of the application of the equation is to increase the proportion of studs in the portion of the beam having the largest shear and more or less reflects analysis and design procedures based on an assumed elastic behavior of the studs.

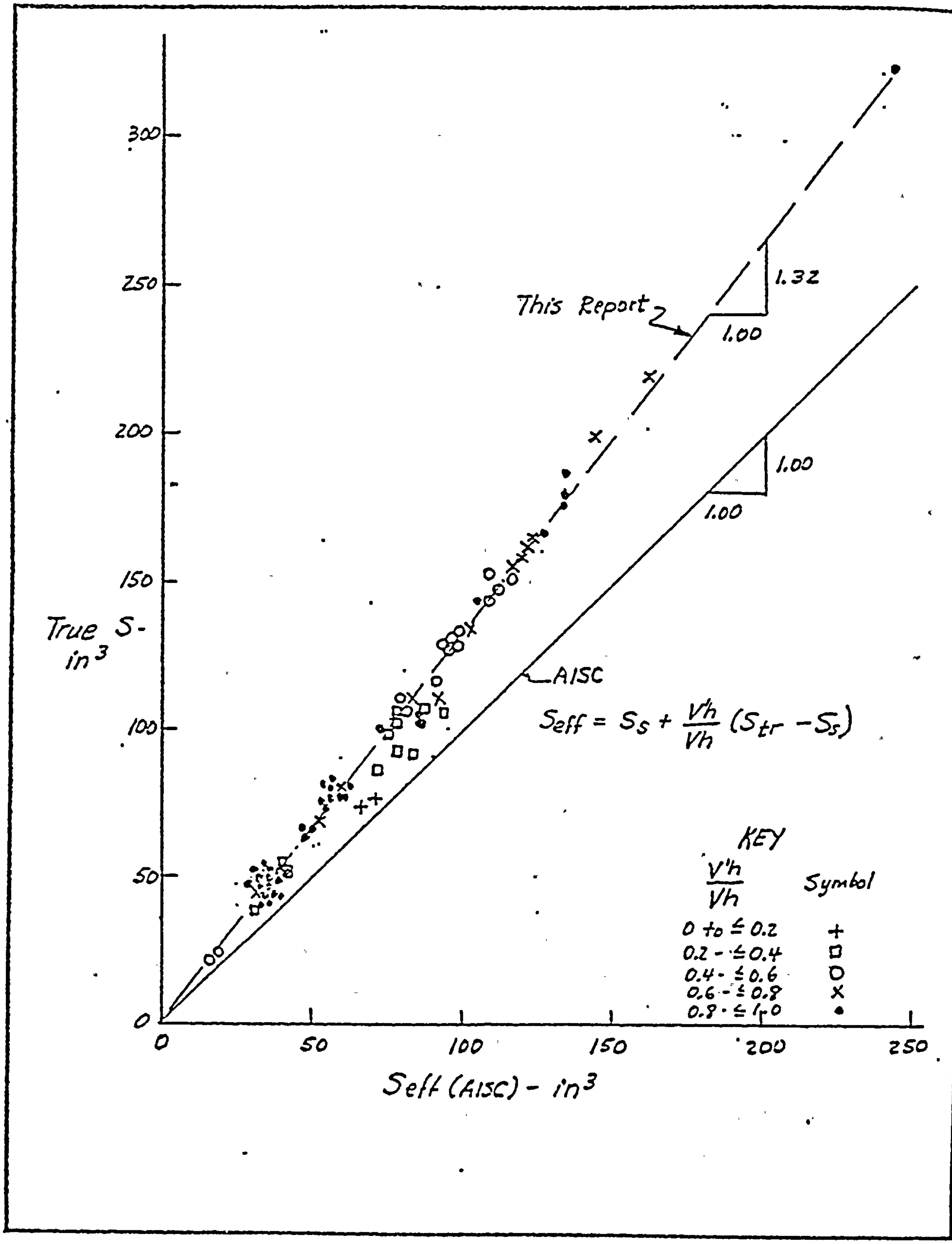


FIGURE 3-1 PLOT OF TRUE EFFECTIVE SECTION MODULUS TO THAT BY AISC SPECIFICATIONS

ANALYSIS: ULTIMATE STRENGTH

$$M_u = 1.7\,(M_D + M_L) = M_D + 0.7\,M_D + 1.7\,M_L$$

DL on steel alone:

$$0.7 M_D + 1.7 M_L \leq (M_S - M_D) + M_c + \frac{V'h}{2}\left[d + t + h + (t-h)\left(1 - \frac{V'h}{V'hc}\right)\right]$$

CASE I: $V'h = 0$

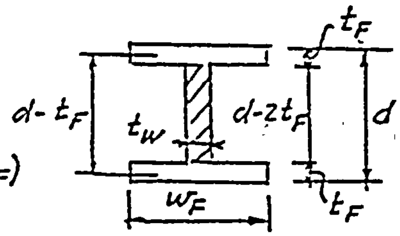
$$0.7 M_D + 1.7 M_L \leq (M_S - M_D) + M_c$$

or $M_u \leq M_S + M_c$

CASE II: $V'h \neq 0$, ($V'h$ to be a minimum)

Steel Beam Moment Capacity:

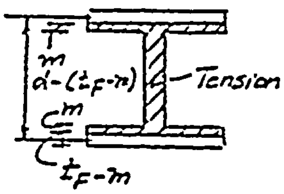
a. $V'h \leq t_w (d - 2t_F) F_y$



$$M_S = M_D + M_{SL} = w_F\, t_F\, F_y\, (d - t_F)$$

b. $V'h \geq t_w (d - 2t_F) F_y$

$$M_S = M_D + M_{SL} = w_F (t_F - m)(d - (t_F - m)) F_y$$



$$m = \frac{V'h - t_w (d - 2t_F) F_y}{w_F\, F_y}$$

Procedure: Case II

1. Calculate $M_u = 1.7(M_D + M_L)$ and $M_u - M_D$
   $M_D < 0.66 F_y S$
2. Check $M_u \leq F_y Z + M_c$ If so, $V'h = 0$
3. If $V'h$ is needed,
   a. Assume values of $V'h$
      (1). Compute M_{SL} (Note $M_{SL} \geq 0$ only)
      (2) Compute M_c
   b. Check: $0.7 M_D + 1.7 M_L \leq M_{SL} + M_c + \frac{V'h}{2}\left[d + t + h + (t-h)\left(1 - \frac{V'h}{V'hc}\right)\right]$
   or $M_u = M_S + M_c + \frac{V'h}{2}\left[d + t + h + (t-h)\left(1 - \frac{V'h}{V'hc}\right)\right]$

FIGURE 3-2 ULTIMATE STRENGTH ANALYSIS

ANALYSIS: ULTIMATE STRENGTH (Continued)

CASE III: $V'h \neq 0$

Live load can be proportioned according to EI.

ANALYSIS: ELASTIC (PSEUDO)

CASE I: $V'h = 0$

Live Load to slab and beam proportional to EI.

Dead Load to steel beam.

(Use M_{CE} of ultimate divided by 1.7, $M_{CE} = \frac{M_C}{1.7}$)

Check capacities:

Slab: LL (proportion only)

Steel = DL + LL (proportion)

CASE II: $V'h \neq 0$

DL to steel, M_D

LL to steel and concrete proportional to EI

a. Concrete: Check $M_{CL} \leq M_{CE}$ (Pseudo)

b. Steel: Check $M_D + M_{SL} \leq .0.66 F_y S$

c. Compute required $V'h$

(1) Concrete: $V'h = \dfrac{M_{CL} - M_{CE}}{t - \frac{a}{2}}$

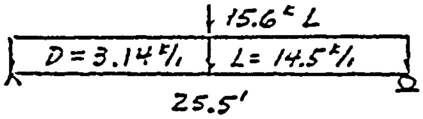
(2) Steel: Moment $= M_D + M_{SL}$

Capacity $= \frac{1}{1.7}$ (Case II Ultimate) (Pseudo)

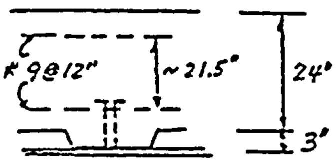
Compute $V'h$ or check capacity for given trial $V'h$.

FIGURE 3-2 (continued) ULTIMATE STRENGTH ANALYSIS

ANALYSIS PROJECT BEAM (14)



$f'_c = 4$ ksi
$b = 76.5''$
$F_y = 40$ ksi



93 - $\frac{3}{4}$ in. studs in 3 rows
31 Studs per row
46.5 studs in shear span

W 24×160
$F_y = 50$ ksi

$d = 24.72''$
$t_F = 1.135''$
$w_F = 14.091''$
$t_w = 0.656''$
$I = 5120''^4$
$S = 414''^3$
$Z = 465''^3$
$A = 47.1^{\square''}$

$M_D = 255.2^{k'}$ ($f_s = 7.4$ ksi)

$M_L = 1278.1^{k'}$ ($f_s = 37.0$ ksi)

$I_c = 88128''^4$

$E_c = 3.6\ ^k/_{\square''}$ (ACI)

$\frac{E_c I_c}{E_s I_s} = 2.09$

$Vh_s = 2355^k$ Steel
$Vh_c = 6242^k$ Conc

$Q_{us} = 1.106\, A_s f'^{\,0.3}_c E_c^{\,0.44}$

$Q_{us} = 27.34^k$ (AISC 3" 13.3^k)

$Q_u = \frac{0.85}{\sqrt{N}}\left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right) Q_{us} = 0.859\, Q_{us}$

$Q_u = 23.48\ ^k/\text{stud}$

ULTIMATE STRENGTH – Minimum number of studs needed.

$M_u = 1.7\,(M_D + M_L)$

$M_u = 2606.6^{k'}$

CASE I: If $V'h = 0$ (No Studs Needed)

$M_c = (40)(1)\left(\frac{21.5}{12}\right)\left(\frac{76.5}{12}\right) = 456^{k'}$

$M_{SP} = \frac{50}{12}(465) = 1937.5^{k'}$

Sum = $2393.5^{k'}$
Need $2606.6^{k'}$

CASE II:

Approx. Needed $V'h = \frac{2606.6 - 2393.5}{2} = 106.3^k$:

Web Tension Cap. $= t_w\,(d - 2t_F)\,F_y$
$= (0.656)(22.45)(50) = 736.4^k$

Try $V'h = 200^k$ web o'k

$M_s = w_F\, t_F\, F_y\,(d - t_F)$

$M_s = (14.091)(1.135)(50)(24.72 - 1.135)\left(\frac{1}{12}\right) = 1571.6^{k'}$

$\frac{V'h}{2}\left[d + t + h + (t-h)\left(1 - \frac{V'h}{Vhc}\right)\right] = \frac{200}{(2)(12)}\left[24.72 + 27 + 3 + 24\left(1 - \frac{200}{6242}\right)\right] = 650^{k'}$

$M_c + M_s + M_{V'h} = 456 + 1571.6 + 650 = 2677.6^{k'} > 2606.6^{k'}$ o'k

No. Studs needed $= \frac{200}{23.48} = 8.5$ (46.5 Supplied)
(42 effective at 2%)

FIGURE 3-3 EXAMPLE: ANALYSIS OF PROJECT BEAM 14

ANALYSIS PROJECT BEAM (14) (Continued)

ELASTIC : (Pseudo)

CASE I: $V'h = 0$

$M_D = 255.2^{k'}$ TO STEEL

$M_L = 1278.1^{k'}$

$M_{CL} = \frac{2.09}{1+2.09}(1278.1) = 864.5^{k'}$. $M_{CE} = \frac{456}{1.7} = 268^{k'}$ NG

$M_{SL} = \frac{1}{3.09}(1278.1) = 413.6^{k'}$

$M_{SL} + M_D = 413.6 + 255.2 = 668.8^{k'}$ $0.66 F_y S = 1138.5^{k}$ ok

$M_D + M_L = 1533.3^{k'}$ $\therefore$

$M_{CE} + 0.66 F_y S = 1406.5^{k'}$ NG

CASE II:

Approx $V'h \approx \frac{1533.3 - 1406.5}{2' \text{ to } 3'} \approx 40^k$ to 60^k overall

or $V'h \approx \frac{864.5 - 268}{3'} = 198.8^k$ Approx.

Try $V'h = 200^k$ Elastic ULT. Equiv. = $(2)(200) = 400^k$

$\frac{V'h}{2}\left[d + t + h + (t-h)\left(1 - \frac{V'h}{V'h_c}\right)\right] = 1286.4^{k'} = M_{V'h}(ULT)$

$M_{V'h}(EL) = \frac{1}{1.7} M_{V'h}(ULT) = 756.7^{k'}$

$M_{CE} + M_{V'h}(EL) + M_{SL} = 1438.3^{k'} > 1278.1^{k'}$ ok.

(Does not use steel to capacity)

If $V'h = 100^k$, $M_{V'h}(EL) = \frac{650}{1.7} = 382^{k'}$

$M_{CE} + 0.66 F_y S + M_{V'h}(EL) = 1788.5^{k}$ ok An Approximation

Studs ok Cap $= (0.859)(13.3) = 11.4^k$/stud

At 200^k 18 Needed, at 100^k 9 Needed ok

FIGURE 3-3 (continued) EXAMPLE: ANALYSIS OF PROJECT BEAM 14

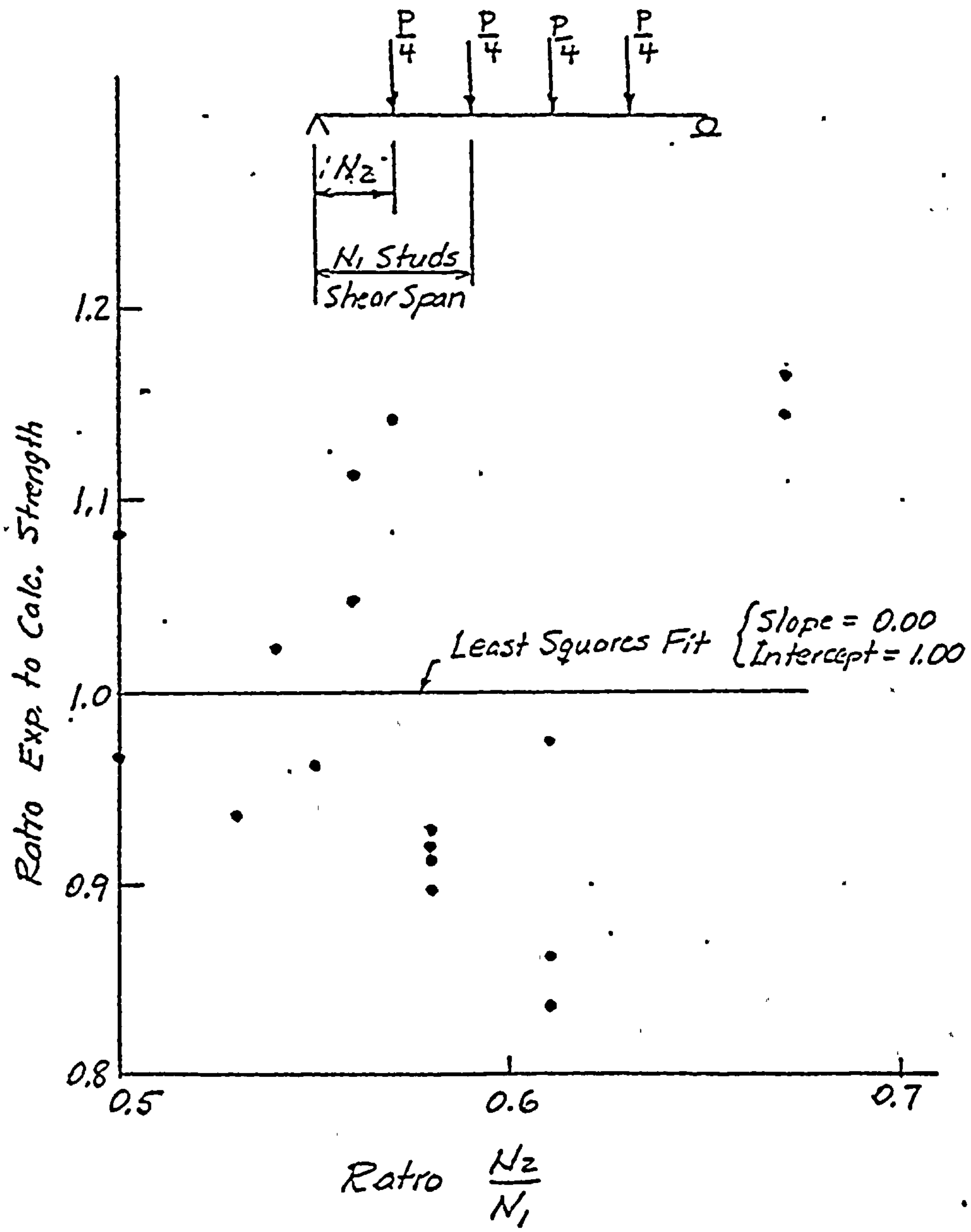


FIGURE 3-4 PLOT SHOWING LACK OF CORRELATION OF ULTIMATE STRENGTH WITH VARIATION IN STUD PLACEMENT PATTERN

4. RECOMMENDATIONS AND CONCLUSIONS

The two basic conclusions of the study are, first, an adequate ultimate strength theory exists for evaluating composite beams, and second, the AISC specifications for composite beams reflect a specific type of design rather than a general methodology and thus should only be applied to thin slabs combined with deep steel beams. It is shown in the report that thick-slab composite beams of the type employed in the project are approximately 30 percent stronger than the strength by AISC specifications. The influence of the formed steel deck appears to be adequately covered by existing relationships.

REFERENCES

1. Grant, J. A., Fisher, J. W., and Slutter, R. G., "Composite Beams with Formed Steel Deck," Engineering Journal AISC, First Quarter 1977.

2. "Manual of Steel Construction," AISC, Seventh Edition and Supplements

3. Benjamin, J. R. and Cornell, C. A., Probability, Statistics, and Decision for Civil Engineers, McGraw Hill Book Company, Inc., 1970.

APPENDIX F

TO

FINAL REPORT ON SHEAR STUDS

STUDIES OF SHEAR STUD ADEQUACY

BY

ENGINEERING DECISION ANALYSIS COMPANY

(P-74b)

EDAC-249.03

STUDIES OF SHEAR STUD ADEQUACY
SUSQUEHANNA STEAM ELECTRIC STATION

prepared for
BECHTEL POWER CORPORATION
San Francisco, California

21 December 1977

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SYNOPSIS

Upon inspection at the Susquehanna Steam Electric Station construction site, a higher proportion of improperly welded shear studs was observed than is considered normal in composite beam construction. It is normal for approximately 2 percent of the shear studs to be inadequately welded to the steel beam. Of the shear studs tested, approximately 9 percent failed to pass inspection on an average. A portion of the reinforced concrete floor slab was in place at the time of the inspection and the question is to determine whether or not measures should be taken to improve the shear connection between the steel rolled section and the concrete slab in that portion of the structure where the floor slab has been placed, since the shear stud connection is uncertain.

The construction at the power plant employs heavy, thick slabs on heavy steel rolled sections. In contrast, the common construction in ordinary buildings employs a thin lightweight floor slab with a formed steel deck (as slab forming) and the structural steel beam. A formed steel deck was employed in the project construction and the steel beams were generally not shored when the slab concrete was placed.

The statistical analysis of data on shear stud properties where they could be tested showed that the mean number of studs not passing inspection in any beam in Reactor Buildings 1 and 2 and the Control Building was 9.2, percent, and the standard deviation of this measure was 6.4 percent. The data for the three structures were so similar that they could be combined. In contrast, the mean percent of studs not passing inspection was 0.42 percent in the Turbine Building, so that two different conditions exist. No detailed analytical study appears to be necessary for the Turbine Building.

A total of 13,904 studs were examined in the field, 13,073 for Reactor Buildings 1 and 2 and the Control Building, and 831 in the Turbine Building. The mean failure rate of individual studs in the former group of structures is estimated to be 0.0842 and for the latter structure is estimated to be 0.0084. The reason for the need to estimate these rates arises from the fact that many studs were repaired upon failing to pass the visual test, while only approximately 18 percent of those failing the visual test actually failed the bending test.

The sample size is adequate for estimation and forecasting.

The study closes with recommendations for use in evaluating the project beams.

1. INTRODUCTION

This report is prepared in accordance with Bechtel Contract No. 7 PE-TSA-11 and in accordance with meetings between Bechtel Power Corporation and Engineering Decision Analysis Company, Inc (EDAC). This report is concerned with a stastical study of shear stud adequacy and recommendations for handling the problems from the standpoint of design.

Reference is made to the Bechtel Power Corporation report (Ref. 1) of 17 June 1977 for a statement of the problem. In essence, a higher failure rate (soundness and bend test) of shear studs than expected has been observed in the construction of some of the composite beams in the Susquehanna Steam Electric Station construction. The question is whether or not those beams which had their slabs poured prior to this observation are adequate.

Stud failure data analysis and forecast procedures are discussed in Chapter 2 using two different types of analysis. The first type of analysis assumes that the occurrence of inadquate studs is by beams with independence between beams. This type of analysis produces a failure rate in terms of the percent of studs that are satisfactory and unsatisfactory in any given beam. The second type of analysis assumes that the occurrence of an inadequate stud is an independent chance event. No systematic phenomena appear to exist which makes failures tend to occur together on a particular beam or in areas of the structure. The two statistical procedures yield slightly different forecasts of the number of adequate studs in any beam. It was not found possible to consider partial strengths of studs in the study owing to a lack of data.

Finally, Chapter 3 presents recommendations and conclusions.

2. STATISTICAL ANAYSIS OF SHEAR STUD DATA

Two different analyses of the same data are presented in this chapter. In the first analysis, the data are considered in a beam-by-beam basis assuming independence between beams but not necessarily between the studs in any one beam. In contrast, the second type of analysis assumes that each individual stud is independent of all other studs. The chapter closes with an interpretation of the results in terms of equivalence of the portion of the construction of concern and normal conditions.

ANALYSIS BY BEAMS

The data fall into four sets, Reactor Buildings 1 and 2, Control Building, and Turbine Building. In each set, the total number of inadequate studs was taken as the sum of those that failed the soundness (hammer blow) test, plus those that failed the visual test and the bend test, plus a portion of those that failed the visual test and were repaired without further testing. The latter portion was assumed to have the same proportion of failures as those that failed the bending test after failing the visual test. The results of the analysis are given in Table 2-1. It is seen that all data except for the Turbine Building have similar properties so that the data on beams for Reactor Buildings 1 and 2 and Control Building were combined into the first data set (Fig. 2-1), with that from the Turbine Building being the second data set. No detailed analysis of the second data set was necessary owing to the low inadequacy rate.

The data of the first set were ordered and plotted on both normal and lognormal probability paper. The fit of the data to a straight line was fair on normal probability paper (Fig. 2-2) and fair on lognormal probability paper (Fig. 2-3). This result is reasonable considering the fact that some dependency is apparent in the data on an area basis that cannot be quantified statistically. The median of the lognormal distribution was 7.5 percent and the standard deviation was 0.626 (log).

ANALYSIS BY STUDS

If the same treatment of the data is employed on an individual stud basis, the failure rate is 0.0842 for Reactor Buildings 1 and 2, and Control Building. If each stud amounts to an independent trial, the probability of any combination of failures and successes can be readily calculated using the binominal probability model. Ample data exist to allow the point estimate of the failure rate to be used in the binomial distribution. Thus if a beam contains 100 studs, the mean number of unsatisfactory studs is (100)(0.0842) = 8.42 studs or the mean number of satisfactory studs is 100 - 8.42 = 91.58. Using the analysis by beams, the corresponding mean number of satisfactory studs is 90.82.

INTERPRETATION

The two different probability models yield slightly different results, with the lognormal model being more conservative than the binominal model. That is, the lognormal model produces a larger probability of high failure rates than with the binomial model.

From a practical standpoint, however, the two models yield very similar results. Figure 2-4 provides a useful interpretation of the statistical studies. The figure was constructed by assuming that a beam contained 100 studs, and inspection has shown that the proportion of studs which do not pass the bending test is 5, 8.42, or 10 percent (binomial by studs) or 9.18 percent by beam (lognormal). If the acceptable failure rate is 2 percent (ordinate), analysis can be based on the concept that 100 studs are placed when the design only needs 92.5 (8.42 percent curve) studs in order to achieve an effective mean failure rate of 2 percent.

Thus to achieve an effective mean failure rate of 2 percent (acceptable) when the actual rate is larger than this value, it is only necessary to place additional studs. With the binomial model, 100 studs in place at a failure rate of 8.42 percent becomes a 2 percent failure rate using 92.5 of the 100 in place studs. The beam (lognormal) analysis yields 91 of 100 studs in place associated with 2 percent failure rate. The two solutions are essentially that same with the lognormal (beam) analysis being very conservative. A gamma model was also investigated with results shown.

The concept of equivalence expressed in Figure 2-4 is useful in analysis and design since the curves relate 100 studs at a particular failure rate to a reduced number of studs at an acceptable or normal failure rate.

The above results agree with the study made by Bechtel Power Corporation (Ref. 5) (Appendix A).

TABLE 2-1

DATA PARAMETERS BY BEAMS

Source	Beams	Mean Percent	Standard Deviation Percent	Coefficient of Variation
RB1	63	9.26	6.55	0.71
RB2	48	9.38	6.69	0.71
Control	11	7.88	3.75	0.48
Composite Set	122	9.18	6.36	0.69
Turbine	17	0.42	1.26	Insufficient Data

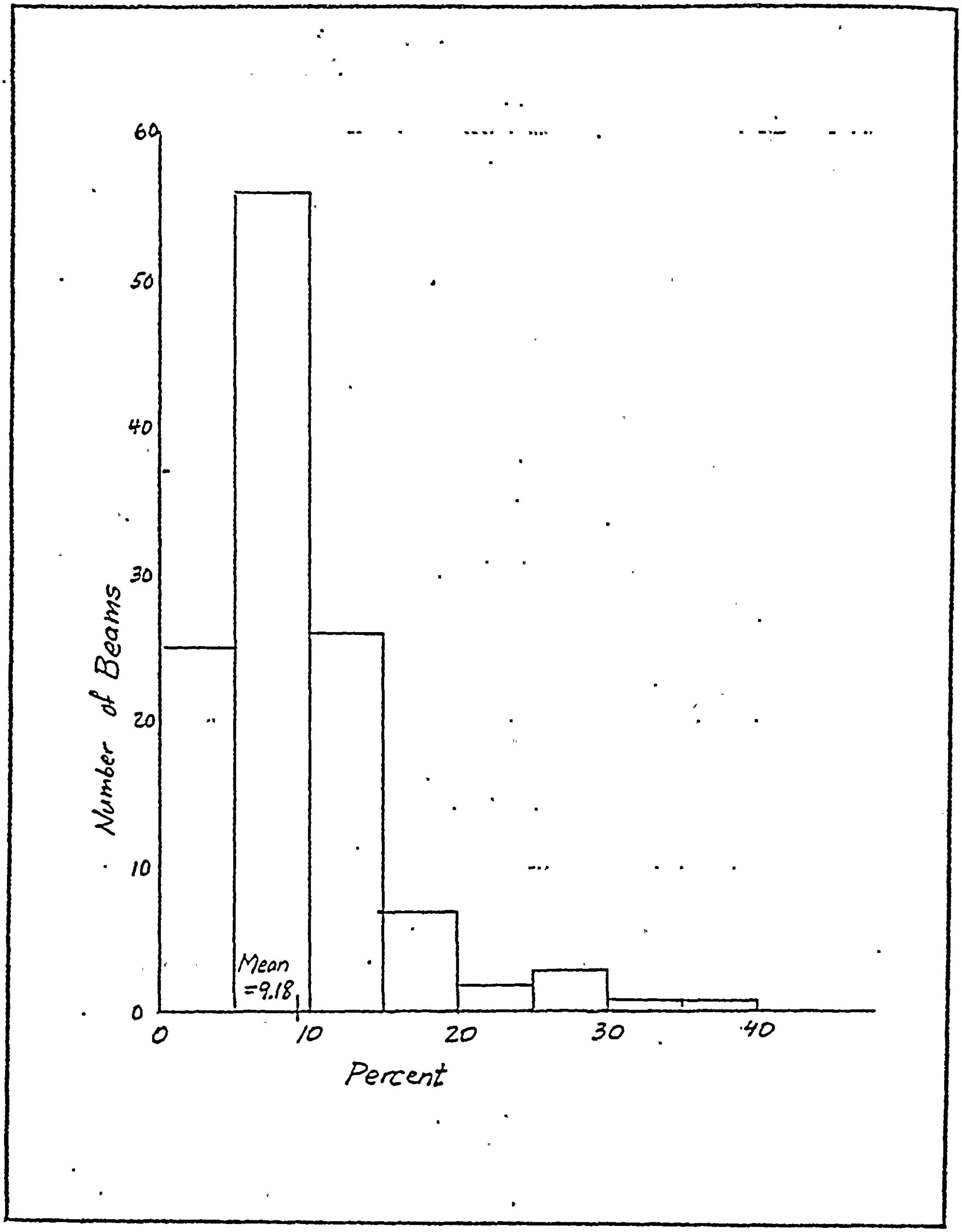

FIGURE 2-1 HISTOGRAM OF BEND TEST FAILURES IN PERCENT OF STUDS PROVIDED IN A BEAM

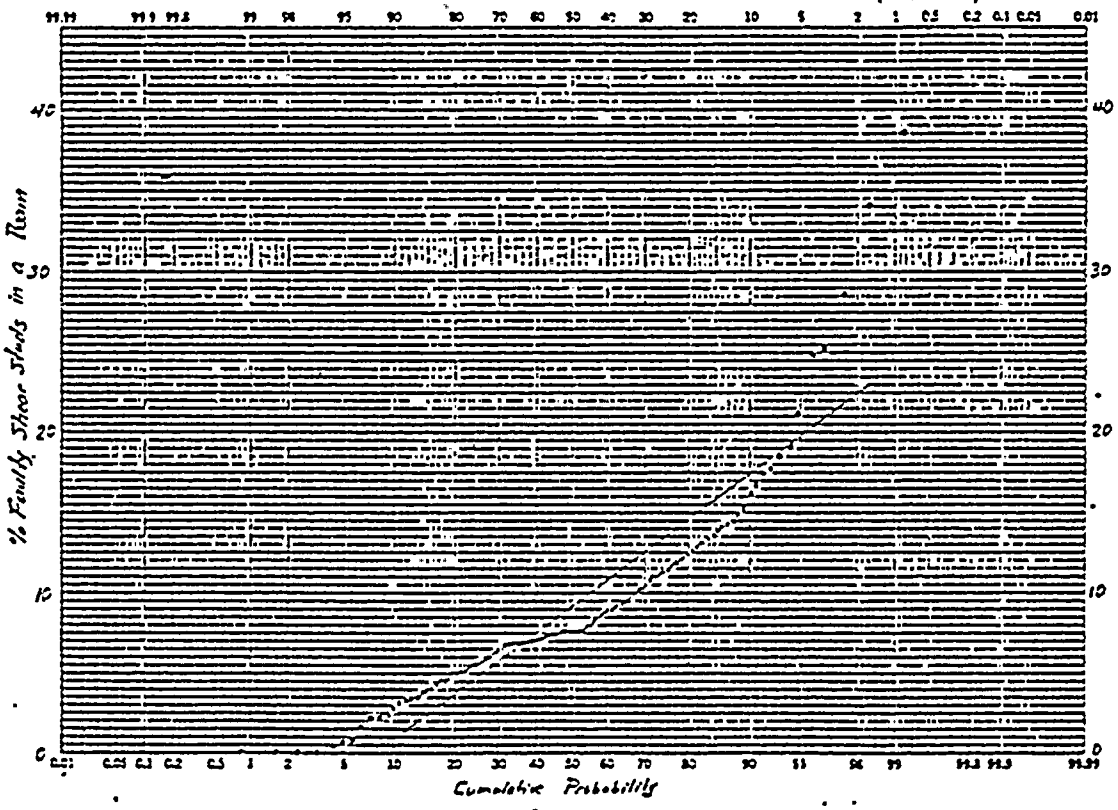

FIGURE 2-2 PLOT OF BEND TEST FAILURE RATE ON NORMAL PROBABILITY PAPER

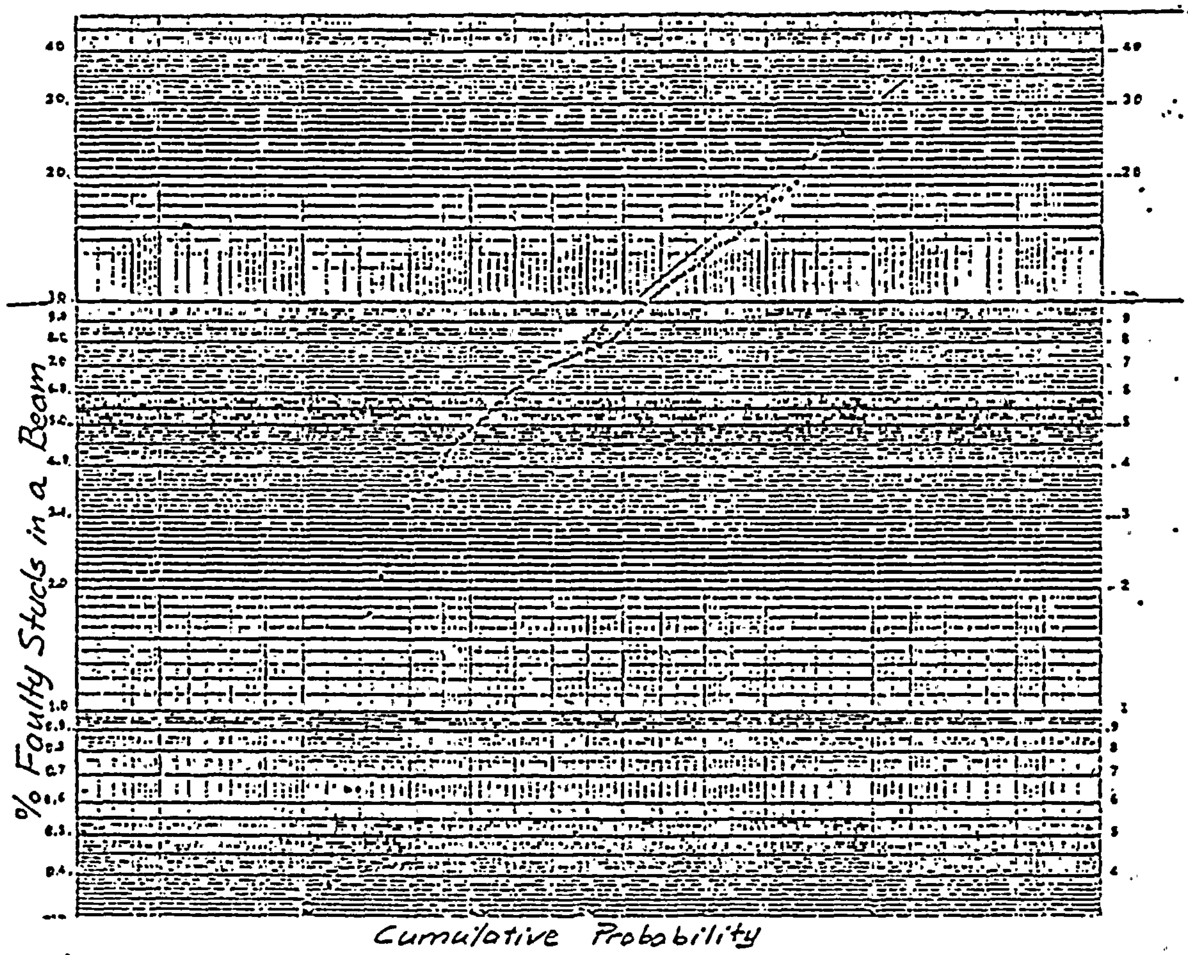

FIGURE 2-3 PLOT OF BEND TEST FAILURE RATE ON LOGNORMAL PROBABILITY PAPER

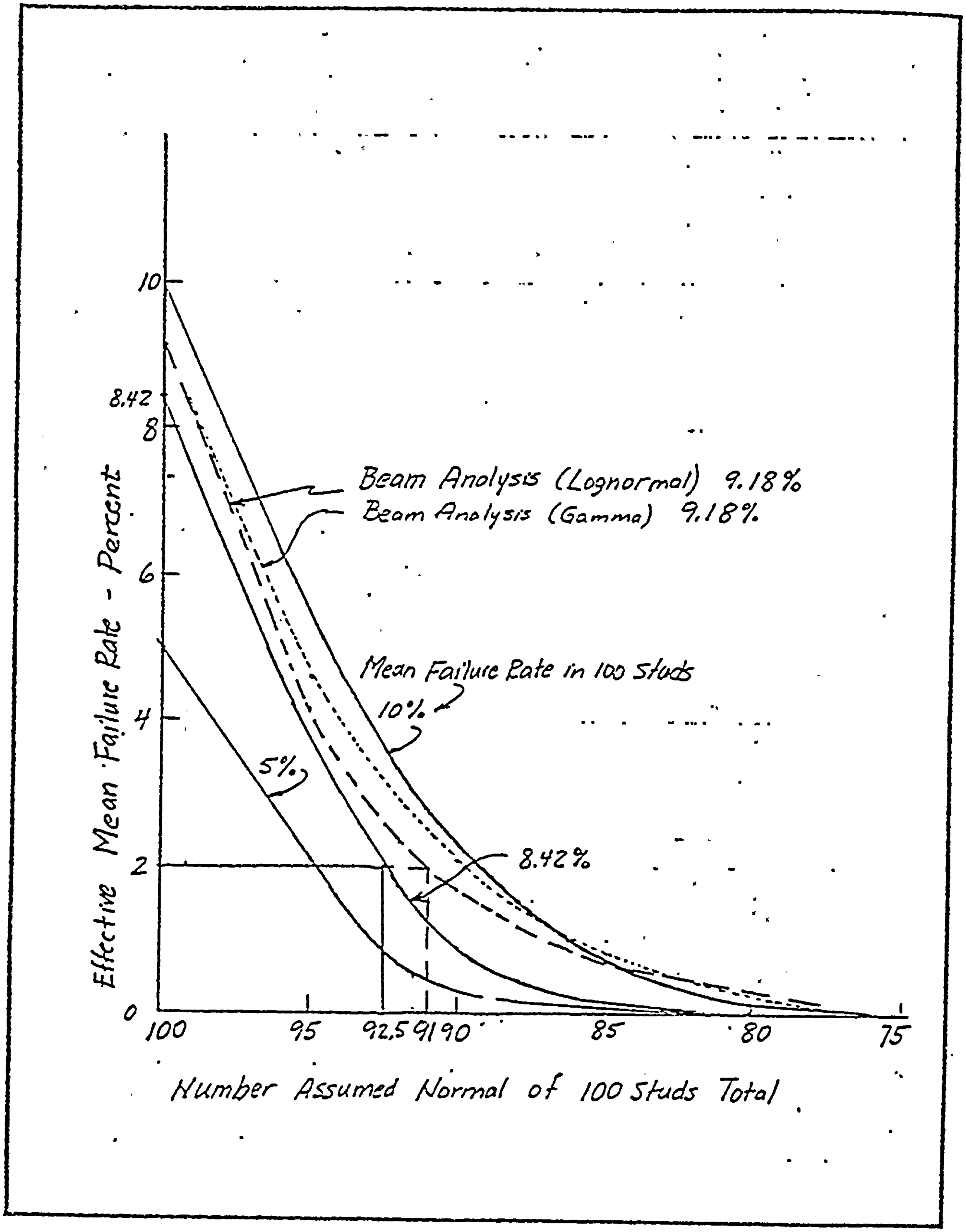

FIGURE 2-4 EQUIVALENCE DIAGRAM

3. RECOMMENDATIONS AND CONCLUSIONS

A detailed statistical analysis of shear stud adequacy disclosed that the occurrence of studs which fail to pass the soundness and bend test follows recognized probabilistic models. Detailed analyses provided a valid basis for forecasting stud adequacy on the basis of equivalence of those provided with those having a 2 percent inadequacy rate by the soundness and bend tests. A slightly different alternate technique was used by Bechtel Power Corporation (Ref. 5) with the same basic results.

REFERENCES

REFERENCES

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