REGULATORY INFORMATION DISTRIBUTION SYSTEM (RIDS)

ACCESSION NBR:8311220173 DOC.DATE: 83/11/15 NOTARIZED: NO DOCKET #
FACIL:50-397 WPPSS Nuclear Project, Unit 2, Washington Public Powe 05000397
AUTH.NAME AUTHOR AFFILIATION
SORENSEN,G.C. Washington Public Power Supply System
RECIP.NAME RECIPIENT AFFILIATION
SCHWENCER,A. Licensing Branch 2

SUBJECT: Forwards addendum to 831031 response to violations noted in IE Insp Rept 50-397/83-38 re evaluation of concrete & reinforcing steel,in response to 831108 telcon w/NRC.

DISTRIBUTION CODE: IE01S COPIES RECEIVED:LTR 1 ENCL 1 SIZE: 39
TITLE: General (50 Dkt)-Insp Rept/Notice of Violation Response

NOTES:

	RECIPIENT ID CODE/NAME	COPIES LTTR	COPIES ENCL	RECIPIENT ID CODE/NAME	COPIES LTTR	COPIES ENCL
	NRR LB2 BC	1	1	AULUCK,R.	1	1
INTERNAL:	AEOD	1	1	ELD/HDS2	1	1
	IE ENF STAFF	1	1	IE FILE ~~01~~	1	1
	IE/DQASIP/ORPB	1	1	IE/ES FILE	1	1
	NRR/DSI/RAB	1	1			
EXTERNAL:	ACRS	2	2	LPDR	1	1
	NRC PDR	1	1	NSIC	1	1
	NTIS	1	1			

Add: Reg File 1 1

TOTAL NUMBER OF COPIES REQUIRED: LTTR ~~15~~ 16 ENCL ~~15~~ 16

Washington Public Power Supply System
P.O. Box 968 3000 George Washington Way Richland, Washington 99352 (509) 372-5000

November 15, 1983
G02-83-1057

Docket No. 50-397

Director of Nuclear Reactor Regulation
Attention: Mr. A. Schwencer
Licensing Branch No. 2
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Schwencer:

Subject: NUCLEAR PROJECT NO. 2
INSPECTION REPORT 83-38, NOTICE
OF VIOLATION - CONCRETE

Reference: Letter, G02-83-996, C. S. Carlisle (SS) to J. B. Martin (NRC), same subject, dated October 31, 1983

As requested by a phone conversation on November 8, 1983, between Messrs. R. Auluck and K. C. Leu (NRC) and P. Powell and H. Crisp (SS), the attached documents (3) are provided in clarification and support of the reference.

Should you have any additional questions, please contact Mr. P. L. Powell, Manager, WNP-2 Licensing.

Very truly yours,

GC Sorensen

G. C. Sorensen, Manager
Regulatory Programs

PLP/tmh
Attachments

cc: R Auluck - NRC
WS Chin - BPA
KC Leu - NRC
AD Toth - NRC Site

IE01 1/1 Add: Reg File Ltr Encl 1 1

ATTACHMENT I

WASHINGTON PUBLIC POWER SUPPLY SYSTEM
NUCLEAR PROJECT NO. 2
DOCKET NO. 50-397
LICENSE NO. CPPR-93

ADDENDUM TO RESPONSE TO INSPECTION REPORT 83-38
NOTICE OF VIOLATION

EVALUATION OF CONCRETE AND REINFORCING STEEL
FOR
WASHINGTON PUBLIC POWER SUPPLY SYSTEM UNIT 2

This attachment restates each question recorded by the Supply System at the October 14, 1983, meeting with the NRC and references the part of the Supply System response, G02-83-996 dated October 31, 1983, which addressed the question.

1. What was the disposition of NCRs written for questions raised during visual reinspections of beam bioshield wall connections?

   Response: Last paragraph, first bullet, of Attachment I to G02-83-996 dated October 31, 1983, states:

   "Visual reinspections were made of the beam-bioshield wall connections for 37 beams (framing into the bioshield wall), which represent 56% of the principal beams in the Reactor Building. Eight minor questions recorded were dispositioned 'accept as is' by the engineer."

2. Explain the background of the sampling plan.

   Response: The first bullet of the attachment to G02-83-996 dated October 31, 1983, addressed this question.

   "Concrete Sampling Program

   Beams 2B3, 2B11 and 2B25 were the subject of nonconformance report (NCR) 6426-1851. This NCR identified honeycombing on each beam, which was subsequently repaired. When these patches were reinspected and sounded (by tapping) in 1983, it appeared patches were deficient and it was decided to perform destructive examination of these beams. Removal of these patches showed that there were reinforcing steel placement deviations and honeycombing/voids existed in areas of congested rebars. Concerns were expressed by the Construction Appraisal Team (CAT) that similar conditions might exist elsewhere and additional destructive examination of concrete structural components was undertaken.

A total of 17 members with 23 excavation locations known to have congestion were selected for evaluation. These structures (with congested rebars) are the ones most likely to exhibit misplaced rebars and honeycombing. The sample was thus biased in the direction of those components and locations most susceptible to be affected by the deviations mentioned above.

Reinforced concrete design drawings were reviewed and it was established that the areas most likely to have a similar problem would be the beam-bioshield wall intersections where the main rebars are spliced with dowels. There are 66 such beam--bioshield intersections; six of these intersections were excavated. The sample was further biased by selecting 2B11 and 2B25, which represent all of the beams with three layers of bottom reinforcement at the beam-bioshield intersections. Also excavated were beams 2B3 and 2B5, with two layers of bottom reinforcement, and beams 3B18 and 6B9, with single layers of bottom reinforcement. In addition to beams framing into the bioshield wall, two beams (3B10 and 4B30) framing into column/exterior walls were also excavated. The total sample excavated is representative of the reinforced concrete in the remainder of the plant.

Reinforced concrete design drawings were also reviewed in order to include other type structures in the sample program. Again, congested areas were selected in order to obtain representative samples of columns, walls, slabs and mats.

The pour records were examined for each structure included in the sample to insure that RFIs and NCRs, which might have been issued on the structure, were considered in the analysis.

The selection and extent of each excavation also included evaluation of each member for the excavated condition to assure that the excavation did not weaken the member.

The sample selected for investigation was not based on a random nor a statistical approach. Rather it was selected to provide a conservative biased sample of representative types of construction. It included beams, columns, walls, slabs and mats. Each excavation was selected at an area most susceptible to construction problems where rebar congestion might lead to honeycombing, voids, rebar spacing deviation or misplacement of rebar."

3. Provide calculation to substantiate disposition of spent fuel pool wall.

<u>Response</u>: This calculation was transmitted initially to Mr. Auluck during the week of October 24, 1983. A second copy is attached herewith.

4. Speak to why no other areas of congested reinforcing bar exist in the plant.

Response: The second, third, and sixth paragraphs of the first bullet of the attachment to G02-83-996 dated October 31, 1983, addressed this question.

> "A total of 17 members with 23 excavation locations known to have congestion were selected for evaluation. These structures (with congested rebars) are the ones most likely to exhibit misplaced rebars and honeycombing. The sample was thus biased in the direction of those components and locations most susceptible to be affected by the deviations mentioned above.
>
> Reinforced concrete design drawings were reviewed and it was established that the areas most likely to have a similar problem would be the beam-bioshield wall intersections where the main rebars are spliced with dowels. There are 66 such beam-bioshield intersections; six of these intersections were excavated. The sample was further biased by selecting 2B11 and 2B25, which represent all of the beams with three layers of bottom reinforcement at the beam-bioshield intersections. Also excavated were beams 2B3 and 2B5, with two layers of bottom reinforcement, and beams 3B18 and 6B9, with single layers of bottom reinforcement. In addition to beams framing into the bioshield wall, two beams (3B10 and 4B30) framing into column/exterior walls were also excavated. The total sample excavated is representative of the reinforced concrete in the remainder of the plant."
>
> "The sample selected for investigation was not based on a random nor a statistical approach. Rather it was selected to provide a conservative biased sample of representative types of construction. It included beams, columns, walls, slabs and mats. Each excavation was selected at an area most susceptible to construction problems where rebar congestion might lead to honeycombing, voids, rebar spacing deviation or misplacement of rebar."

The last bullet also addressed this question.

> "Conclusion
>
> The Supply System has confirmed the adequacy of concrete construction at WNP-2 by performing a detailed investigation of selected as-built structural members. The investigation included 23 excavations in 17 structural members at locations of congested rebar in representative beams, columns, walls and

slabs. Such locations are difficult to construct and therefore provide a conservative sample of the plant structures. The investigation included three structural beams (2B3, 2B11 and 2B25) where the congestion had been so severe that honeycombing and voids had been identified during construction and had been repaired in accordance with approved construction procedures.

Results of the evaluations are summarized in Table 1. The excavations demonstrated acceptable construction quality in columns, walls, slabs and mats. The excavations in beams indicated a significant number of locations where the spacing of rebar was less than that specified in the code. This occurred primarily in areas where the main reinforcement was lap spliced. The code requirement on clear spacing between reinforcement is primarily imposed to assure good concrete consolidation. With the exception of the three beams where honeycombing and voids had already been identified during construction, all excavation locations showed good consolidation of the concrete, thereby demonstrating the adequacy of concrete construction in these locations.

All discrepancies from the design requirements were evaluated. In every case the structures were found to be adequate. The results show that for two cases some bond was not available on all the bars. This inadequacy occurred in beams at dowel splice locations where honeycomb and voids were most likely. Both of these beams have already experienced construction loads in excess of those specified during plant operation, and performed well. Of all the cases studied, only one dowel that should have been located in the excavation was not uncovered. This beam was conservatively evaluated assuming the dowel was missing and not just misplaced, and was found to be adequate. In summary, all structural members excavated during the investigation were demonstrated to be adequate for all specified loads.

The investigation demonstrated that structural members specifically selected for their difficulty in construction met the intent of the code for all design conditions. This biased sample provides confidence that the conclusion may be extended to all Category I concrete structures of WNP-2 since they were designed and constructed to the same quality procedures as those included in this investigation."

5. What was the statistical basis for the number of beams selected?

Response: Paragraphs two, three, and seven of the first bullet of Attachment I to G02-83-996 dated October 31, 1983, addressed this

question.

"A total of 17 members with 23 excavation locations known to have congestion were selected for evaluation. These structures (with congested rebars) are the ones most likely to exhibit misplaced rebars and honeycombing. The sample was thus biased in the direction of those components and locations most susceptible to be affected by the deviations mentioned above.

Reinforced concrete design drawings were reviewed and it was established that the areas most likely to have a similar problem would be the beam-bioshield wall intersections where the main rebars are spliced with dowels. There are 66 such beam-bioshield intersections; six of these intersections were excavated. The sample was further biased by selecting 2B11 and 2B25, which represent all of the beams with three layers of bottom reinforcement at the beam-bioshield intersections. Also excavated were beams 2B3 and 2B5, with two layers of bottom reinforcement, and beams 3B18 and 6B9, with single layers of bottom reinforcement. In addition to beams framing into the bioshield wall, two beams (3B10 and 4B30) framing into column/exterior walls were also excavated. The total sample excavated is representative of the reinforced concrete in the remainder of the plant."

"The sample selected for investigation was not based on a random nor a statistical approach. Rather it was selected to provide a conservative biased sample of representative types of construction. It included beams, columns, walls, slabs and mats. Each excavation was selected at an area most susceptible to construction problems where rebar congestion might lead to honeycombing, voids, rebar spacing deviation or misplacement of rebar."

6. In Sketch 17, clarify that splice is equivalent to a contact splice; i.e., that no greater than six inches separates the two bars.

Response: The second bullet of Attachment I to G02-83-996 dated October 31, 1983, addressed this question.

"Clarification of Lap Splice (SK-17 of Reference 1)

ACI Code 318-71, per paragraph 7.5.4, allows the use of non-contact lap splices, provided the bars to be spliced are not spaced transversely farther apart than one-fifth the required length of lap nor six inches. At column line M on the East Exterior Wall, the wall thickness is 3'-0" on the north side of the pilaster and 2'-6" on the south side of the pilaster.

The horizontal bars for 3'-0" and 2'-6" walls were terminated within the pilaster with C-1 splice, which is in full conformance with code requirements, because the difference in wall thickness is six inches and the horizontal bars are spaced transversely only six inches apart."

7. Certify that RFIs were considered in the re-evaluation of the concrete beams.

   Response: The fifth paragraph of the first bullet of Attachment I to G02-83-996 dated October 31, 1983, contains this certification.

   "The pour records were examined for each structure included in the sample to insure that RFIs and NCRs, which might have been issued on the structure, were considered in the analysis."

8. Summarize the findings on mix substitution. How does it affect other locations? Assess possible impact on other structures.

   Response: The fifth bullet of Attachment I to G02-83-996 dated October 31, 1983, addresses these questions.

   "Mix Substitution

   Contract drawing S749, note #2, specified concrete mixes. The mix for use in beams cast integral with floor slabs is based on the slab thickness. For beams 4B30 and 6B9 the required mix was 4SA-P (maximum size of aggregate equal to 3/4") based on the adjacent slab thickness of 12 inches. The beams and slab were constructed using mix 4MA-P (maximum size aggregate equal to 1-1/2"). The mix substitution was approved by the Burns and Roe Field Engineer prior to concrete placement.

   This substitution in no way affected the structural integrity of the beams because of the following:

   (a) Both classes of concrete (i.e., 4MA-P and 4SA-P) have the same required minimum 28-days strength of fc'=4000 psi. (The actual 28-day strength of these pours was over 5000 psi.)

   (b) Concrete bond and consolidation in the excavations made in these two beams was excellent, without honeycomb and voids.

   (c) Beams 4B30 (3'-0" x 3'-0") has seven bottom bars and beam 6B9 (2'-0" x 3'-0") has four bottom bars, which provides a minimum average clear space of four inches between the rebars. This spacing meets the requirements of paragraph 3.3.2 of the ACI Code 318-71."

9. Enhance credibility of Westinghouse as "independent" reviewer. Provide original charter if necessary.

Response: The sixth bullet of Attachment I to G02-83-996 dated October 31, 1983, addressed this question.

"Westinghouse Charter

The Westinghouse Corporation was given direction under a Basic Ordering Agreement with the Supply System to provide an independent overview of the Supply System effort to resolve the questions raised by the NRC CAT. Westinghouse reported their assessments through the Director of Technology to the Managing Director; however, there was daily contact between project people and the Westinghouse Team. Westinghouse was specifically asked to provide a third-party review of the concrete issue. Initially, this review was to have been a broad overview of the facts and conclusion reached by the Supply System. Direction to Westinghouse was expanded, however, after the inspection conducted by Messrs. Albert and Herring on July 25-27, 1983, to provide for a more detailed review of the facts being developed by the excavations related to the concrete issue. The detailed scope of the Westinghouse review is stated in their September 15, 1983, letter to the Supply System (Appendix D to reference (1))."

10. Discuss the misalignment of reinforcing bars in layers. Summarize industry studies with references.

Response: Bullets three and four of Attachment I to G02-83-996 dated October 31, 1983, address these questions. Paper No. 3047 of the 1960 ASCE Transactions was transmitted to Mr. Auluck during the week of October 24, 1983; a second copy is attached herewith.

"Rebar Alignment

Paragraph 7.4.1 of ACI 318-71 stipulates that where parallel reinforcement is placed in two or more layers, the bars in the upper layers shall be placed directly above those in the bottom layer. The commentary to ACI 318-71 code further clarifies that these spacing limits were developed from successful practice to permit concrete to flow readily into spaces between bars and between bars and forms without honeycomb. Rebar placement with some misalignment in layers (bars in different layers not directly above each other) meets the intent of the code and is acceptable, as long as rebars are placed to allow concrete to flow readily through the spaces without honeycombing. The excavation of beam 2B5 (SK-2 of Reference 1), which originated this concern, showed fully--consolidated concrete tightly bonded to the rebar with no

honeycomb."

"<u>Rebar Spacing</u>

The spacing of rebar has received considerable attention among structural engineers and constructors for decades. The ASCE has a committee to study the problems related to nuclear power plants. The various codes (ACI 301, ACI 318, ACI 349, and ACI 359) specify clearances that are desirable. It is desirable that the maximum size aggregate pass between the rebar. This is rarely practical on all parts of the plant. Sometimes the spacing between bars is actually designed to be zero. This 'bundling' has been studied and tested.

Some results of tests have been published in the 1960 ASCE Transactions in Paper No. 3047, "Concrete Beams and Columns with Bundled Reinforcement", by N. W. Hanson and Hans Reiffenstuhl. As the title states, both beams and beam--columns were tested. Bars were placed with zero clearance, both vertically and horizontally, so that 'bundling' occurred in part of the cross section of the member. In all cases, 'no significant difference in behavior or ultimate strength was found for bundled as compared to spaced reinforcement'.

The tests showed that 'there was no systematic difference in ultimate bond stress developed between spaced and bundled bars'. Zero spacing in bundled areas was determined to be satisfactory for both tension and compressive areas. Examination of the structures after testing indicated mortar had penetrated into and filled the cavity between the bars of the bundle.

Thus, it is concluded that the spacing of rebar can be less than specified if adequate concrete consolidation (without honeycomb) is obtained.

In examining the various structural members, there were only two cases where lack of bond was experienced. These were in the compressive areas on beams 2B11 and 2B25. The detailed structural evaluation showed that construction loads dominated and the beams had been 'tested' by the construction loads, which indicated that the systems met the design requirements."

11. Mix substitution 1-1/2" vs. 3/4" aggregate size. What effect does aggregate size have on strength of beam?

    <u>Response</u>: This question is the same as question 8.

12. Discuss proof tests of beams with construction loads.

<u>Response</u>: The third paragraph of the final bullet of Attachment I to G02-83-996 dated October 31, 1983, addressed this question.

"All discrepancies from the design requirements were evaluated. In every case the structures were found to be adequate. The results show that for two cases some bond was not available on all the bars. This inadequacy occurred in beams at dowel splice locations where honeycomb and voids were most likely. <u>Both of these beams have already experienced construction loads in excess of those specified during plant operation, and performed well</u>. Of all the cases studied, only one dowel that should have been located in the excavation was not uncovered. This beam was conservatively evaluated assuming the dowel was missing and not just misplaced, and was found to be adequate. In summary, all structural members excavated during the investigation were demonstrated to be adequate for all specified loads." (Emphasis added)

13. Did sample excavations weaken structures?

<u>Response</u>: The sixth paragraph of the first bullet of Attachment I to G02-83-996 dated October 31, 1983, addressed this question.

"The selection and extent of each excavation also included evaluation of each member for the excavated condition to assure that the excavation did not weaken the member."

14. Discuss the statistical basis for the selection of the sample.

<u>Response</u>: The question is the same as question 2.

15. Discuss the safety margins in the beams; use worst case assumptions.

<u>Response</u>: The fourth paragraph of the last bullet of Attachment I and Table I of Attachment I to G02-83-996 dated October 31, 1983, addressed this question.

"All discrepancies from the design requirements were evaluated. In every case the structures were found to be adequate. The results show that for two cases some bond was not available on all the bars. This inadequacy occurred in beams at dowel splice locations where honeycomb and voids were most likely. Both of these beams have already experienced construction loads in excess of those specified during plant operation, and performed well. Of all the cases studied, only one dowel that should have been located in the excavation was not uncovered.

This beam was conservatively evaluated assuming the dowel was missing and not just misplaced, and was found to be adequate. In summary, all structural members excavated during the investigation were demonstrated to be adequate for all specified loads."

16. Characterize the sample (conservative, representative) with rationale. Considering the deviations recorded, did the construction exposed by excavation meet the intent of the code?

Response: The last bullet of Attachment I and Table I of Attachment I to G02-83-996 dated October 31, 1983, addressed this question.

"Conclusion

The Supply System has confirmed the adequacy of concrete construction at WNP-2 by performing a detailed investigation of selected as-built structural members. The investigation included 23 excavations in 17 structural members at locations of congested rebar in representative beams, columns, walls and slabs. Such locations are difficult to construct and therefore provide a conservative sample of the plant structures. The investigation included three structural beams (2B3, 2B11 and 2B25) where the congestion had been so severe that honeycombing and voids had been identified during construction and had been repaired in accordance with approved construction procedures.

Results of the evaluations are summarized in Table 1. The excavations demonstrated acceptable construction quality in columns, walls, slabs and mats. The excavations in beams indicated a significant number of locations where the spacing of rebar was less than that specified in the code. This occurred primarily in areas where the main reinforcement was lap spliced. The code requirement on clear spacing between reinforcement is primarily imposed to assure good concrete consolidation. With the exception of the three beams where honeycombing and voids had already been identified during construction, all excavation locations showed good consolidation of the concrete, thereby demonstrating the adequacy of concrete construction in these locations.

All discrepancies from the design requirements were evaluated. In every case the structures were found to be adequate. The results show that for two cases some bond was not available on all the bars. This inadequacy occurred in beams at dowel splice locations where honeycomb and voids were most likely. Both of these beams have already experienced construction loads in excess of those specified during plant operation, and performed well. Of all the cases studied, only one dowel that

should have been located in the excavation was not uncovered. <u>This beam was conservatively evaluated</u> assuming the dowel was missing and not just misplaced, and was found to be adequate. In summary, all structural members excavated during the investigation were demonstrated to be adequate for all specified loads.

<u>The investigation demonstrated that structural members specifically selected for their difficulty in construction met the intent of the code for all design conditions</u>. This biased sample provides confidence that the conclusion may be extended to all Category I concrete structures of WNP-2 since they were designed and constructed to the same quality procedures as those included in this investigation." (Emphasis added)

17. Provide calculation to substantiate acceptability of East Wall.

    <u>Response</u>: This calculation was transmitted initially to Mr. Auluck during the week of October 24, 1983. A second copy is attached herewith.

18. Discuss the rebar spacing problem.

    <u>Response</u>: Response to this question was included with the response to question 10.

SUMMARY OF STRUCTURAL MEMBERS EVALUATED

SK.#	Member	Design Margin(See Footnote)			Observed Discrepancies			Remarks	Conclusions
		Maximum +ive Moment +M	Maximum -ive Moment -M	Shear	Rebar Spacing	Rebar/ Dowel Missing/ Misplaced	Honey-combing		
SK-1	2B3	2.1	1.4	1.5	Yes	None	None	Concrete consolidation is excellent.	Meets the intent of the code.
SK-2	2B5	3.5	1.5	1.2	None	None	None	Rebars were placed in 3 layers instead of two layers.	Meets the intent of the code.
SK-3	2B11	3.6	1,9	1.3	Yes	None	Yes	Honeycombing and rebar spacing deviations were found in congested area where main bars were spliced with dowels.	Meets the intent of the code.
SK-4	2B25	3.0	1.5	1.1	Yes	None	Yes	Honeycombing and rebar spacing deviations were found in congested area where main bars were spliced with dowels.	Meets the intent of the code.
SK-5	3B10	2.5	1.4	1.5	Yes	None	None	Concrete consolidation is excellent.	Meets the code requirement.
SK-6	3B18	2.5	1.0	1.6	Yes	Yes	None	One dowel not found. Dowel not needed per code. Consolidation is excellent.	Meets the intent of the code.
SK-7	4B30	4.3	2.4	2.1	None	None	None		Meets the code requirement.

Footnote to Table: Design margin as used herein is the capacity provided above that of the original design requirements. (A design margin of 1.0 signifies compliance with ACI 318 code requirements and licensing commitments.)

ATTACHMENT 2

SUMMARY OF STRUCTURAL MEMBERS EVALUATED

SK.#	Member	Design Margin(See Footnote)			Observed Discrepancies			Remarks	Conclusions
		Maximum +ive Moment +M	Maximum -ive Moment -M	Shear	Rebar Spacing	Rebar/ Dowel Missing/ Misplaced	Honey-combing		
SK-8	6B9	4.8	4.2	2.1	Yes	None	None	Concrete consolidation is excellent.	Meets the intent of the code.
SK-9	Pilaster	Not Calcu-lated	Not Calcu-lated	Not Calcu-lated	None	None	None	None.	Meets the code requirement.
SK-10	West Exterior Wall	Not Calcu-lated	Not Calcu-lated	Not Calcu-lated	None	None	None	None	Meets the code requirement.
SK-11	Dryer Separa-tor Pool	Not Calcu-lated	Not Calcu-lated	Not Calcu-lated	None	None	None	None	Meets the code requirement.
SK-12	Fuel Pool Wall(N.) El. 588-2½	Not Appli-cable	5.6	Not Appli-cable	Yes	None	None	Construction aid rebars at El. 588'-2½" were not placed per drawings.	Meets the code requirement for operating conditions
SK-13	Mat at El. 422'-0"	Not Calcu-lated	Not Calcu-lated	Not Calcu-lated	Yes	None	None	Trim additional rebar deviate spacing requirements. Concrete consolidation excellent.	Meets the code requirements.
SK-14	Mat at El. 422'-0"	Not Calcu-lated	Not Calcu-lated	Not Calcu-lated	None	None	None	None	Meets the code requirements.

Footnote to Table: Design margin as used herein in the capacity provided above that of the original design requirements. (A design margin of 1.0 signifies compliance with ACI 318 code requirements and licensing commitments.)

SUMMARY OF STRUCTURAL MEMBERS EVALUATED

K.#	Member	Design Margin(See Footnote)			Observed Discrepancies			Remarks	Conclusions
		Maximum +ive Moment +M	Maximum -ive Moment -M	Shear	Rebar Spacing	Rebar/ Dowel Missing/ Misplaced	Honey-combing		
K-15	Mat at El. 422'-0"	Not Calculated	Not Calculated	Not Calculated	Yes	None	None	Additional rebars deviate spacing requirements. Concrete consolidation excellent.	Meets the code requirements.
K-16	Slab at El. 471'	Not Calculated	Not Calculated	Not Calculated	None	None	None		Meets the code requirements.
K-17	East Ext. Wall	Not Calculated	Not Calculated	2.2	None	None	None		Meets the code requirements.

Footnote to Table: Design Margin as used herein in the capacity provided above that of the original design requirements. (A design margin of 1.0 signifies compliance with ACI 318 code requirements and licensing commitments.)

When the solutions to Eq. 44 are extended over those presented in Table 3 and the results plotted, curves of the type given in Fig. 11 are obtained which show strikingly the tendency of the ship movement and restoring force to become infinite when the natural period of vibration (when $A\sigma = 0$) is approached. Of course, no such thing can occur due to the "fuze" in the system in the form of the mooring lines which tend to break and thereby ruin what elegance there is in this problem.

Fig. 11 shows the relationship between period and amplitude of a moored ship (and standing wave) oscillation in surge with standing wave amplitude as a parameter. Note that both negative as well as positive displacements are plotted where this rather unconventional presentation is made to emphasize those situations where the oscillation (x_0) is 180° out of phase with the excitation ($A\ \sigma$). Usually this phase relation is considered of slight interest in comparison with the amplitudes. However, at the precise point of phase switching many ships could receive a jolt at a level high enough to rouse even the sleepiest seaman and, even worse, to break the ropes. Therefore, the negative signs are usually disregarded so a presentation is made entirely in the first quadrant.

The writers have obtained a record of such a shift correlated with changes in mooring forces, by a landing ship tank (LST) as spread-moored in the open Gulf of Mexico. This ship shifted the phase of its pitching motion by 180° as the period of the incident wave changed in a very short time from 4 to 7 1/2 sec where the point of shift is computed as about 6 sec.

The system, depending on its period of excitation will be subject to stable-motions, fro example, branch 1, 3, 4, and 5 in Fig. 11, and unstable-motions, branch 3 and 4-b. Some damping, however slight, must be present in order to permit the ship to cross from in-phase oscillation, periods greater than free period, to out-of-phase oscillation across the zone of transition. (from 4-a to 2 in Fig. 11, for example).

It would appear that the free period of oscillation, line designated A = 0 in Fig. 11, of the ship-line system is one of the dominant design parameters where care should be exercised toward avoiding period coincidence between this period and that of the excitation. A likely operational period of oscillation which is less than rather than greater than the free period would seem desirable.

A number of investigators, including Abramson and Wilson,[33] have discussed surge oscillation of a ship moored at the node of a standing wave, although none appear to have stretched the mechanical analogy as far as the writers herein. Other modes are not at all well covered. Another examination of the problem was made by Wilson.[34]

The writers hope that this closure has provided in some measure answers to and amplification of the questions raised by Mr. Wilson in his much appreciated discussion of their paper.

---

33 "A Further Analysis of the Longitudinal Response of Moored Vessels to Sea Oscillation," by H. N. Abramson, and B. W. Wilson, Proceedings, Joint Mid-West Conf., Solid and Fluid Mechanics, Purdue Univ., September, 1955.

34 "The Energy Problem in the Mooring of Ships Exposed to Waves," by B. W. Wilson, Proc. of Princeton Conference on Berthing and Cargo Handling in Exposed Locations, October, 1958, pp. 1-67.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS — 1960

Paper No. 3047

CONCRETE BEAMS AND COLUMNS WITH BUNDLED REINFORCEMENT

By Norman W. Hanson,[1] M. ASCE and Hans Reiffenstuhl[2]

With Discussion by Messr. Homer M. Hadley

SYNOPSIS

This paper reports on tests of pairs of large beams with conventionally spaced and with bundled longitudinal reinforcement. The bundles of reinforcement used comprised groups of four No. 6, four No. 8, or three No. 9 touching bars. Pairs of beams were compared with respect to width of flexural cracks, steel stress distribution, deflection, and ultimate strength. No significant difference in behavior or ultimate strength was found for bundled as compared to spaced reinforcement.

Tied columns were tested by concentric loading to compare spaced and bundled longitudinal reinforcement consisting of twelve No. 6 or twelve No. 8 bars. Comparison with respect to ultimate strength indicated that bundling is a safe detailing procedure when adequate ties are provided. This true even for 6.6% longitudinal reinforcement. Splicing of bundled reinforcement in columns was explored and found to be feasible.

INTRODUCTION

Bundled reinforcement in structural concrete refers to reinforcement placed in groups of touching bars. As compared to the minimum bar spacings commonly used in beams, for instance those given by the 1965 American Concrete

---

Note.—Published, essentially as printed here, in October, 1958, in the Journal of the Structural Division, as Proceedings Paper 1818. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

1 Assoc. Development Engr., Research and Development Div., Portland Cement Assn., Chicago, Ill.

2 Visiting Engr., Research and Development Div., Portland Cement Assn., Chicago, Ill.

Institute (ACI) Building Code Requirements for Reinforced Concrete (ACI 318-56 Section 505(a)), bundling permits the necessary bars to be placed in much narrower sections. As a result, bundling permits construction of lighter, more graceful and more economical beams of box, channel or T-B6, section. In beams of normal width, the clear distance between bundles will be considerably greater than the distance between individual evenly spaced bars. Bundling greatly facilitates concrete placement and insertion of spud vibrators, particularly when heavy negative moment reinforcement must be embedded in the top of beams. In columns, bundled reinforcement permits a reduced concrete cross section, which may be an important advantage in the lower stories of tall buildings. Bundling also permits interior ties to be omitted, so that concrete placement is facilitated. Finally, bundled bars in beams and columns may be a satisfactory alternate to large sizes of specially rolled reinforcing bars that are occasionally used in very large structures.

Practical use of bundled reinforcement in beams has been pioneered, and several structures with bundled reinforcement have been built,[2,3,4,5,6] for which good service records have been reported.

Laboratory tests that have been reported[7,8] concern principally bundles of four $\frac{1}{2}$-in. square bars in beams, and there may be some question regarding the performance of bundles of larger bar sizes. No tests of bundled reinforcement in columns have been reported. An experimental investigation was therefore carried out in the Research and Development Laboratories of the Portland Cement Association during 1955-57 to investigate the performance of large deformed bars placed in bundles as longitudinal beam and column reinforcement.

Notation.—The letter symbols adapted for use in this paper are defined where they first appear, in the text or in the illustrations, and are arranged alphabetically, for convenience of reference, in the Appendix.

TEST BEAM ARRANGEMENT

As compared to spaced bars, bundling may be questioned primarily with respect to the bond integrity of beams. The most serious conditions may then be expected for bars placed as negative reinforcement near the top of deep and short beams. Previous tests[9] have clearly indicated that, due to adverse effects of settlement, the bond resistance of top bars is less than that of bottom bars. They have also indicated that a short beam span leads to high bond stress before the flexural ultimate strength is developed. Therefore, the test specimens for this investigation were short, deep beams, with the tension steel at the top as cast.

In the beam designations to follow, the first number shows the number of bars and the second number their size; the letter S indicates spaced bars and B indicates bundled bars; H indicates high-strength steel.

The test beams 8-6S, 8-6SH, 8-8SH and 6-9S, with spaced reinforcement, shown in Fig. 1, were designed by first determining the minimum beam width for a chosen group of bars. By the ACI code previously mentioned, this width is governed by a minimum protective cover of 1-$\frac{1}{2}$ in. and, for 1-$\frac{1}{2}$ in. maximum size aggregate, a clear distance of 2 in. between parallel bars. The beam depth was chosen so that the ratio of reinforcement was 1.5%. Finally, the distance from the face of a centrally located column stub to the beam support was chosen as twice the effective beam depth. Thus, the test span L is 85 in. for the beams with No. 6 bars, 134.5 in. for the beams with No. 8 bars, and 151.8 in. for those with No. 9 bars. The beams and all bars were extended 6 in. beyond the supports. The gross concrete dimensions of beams 8-6S, 8-8S and 6-9S, excluding the column stubs, were 13 in. by 21 in. by 97 in., 14.5 in by 33.5 in. by 146.5 in., and 11.5 in. by 38.8 in. by 163.8 in., respectively.

The beams with bundled reinforcement, beams 8-6B, 8-8B and 6-9B were identical to the corresponding beams with spaced bars except for the bar arrangement. The effect of decreasing the beam width for bundled reinforcement was investigated through beams 8-6BH and 8-8 BH. For these two beams, and their companions with spaced reinforcement, high-strength reinforcement was used to delay flexural failure and develop very high bond stress.

The column stubs of all beams were reinforced with four bars of the same size as the longitudinal beam reinforcement, and these bars were extended through the beam. Vertical stirrup reinforcement was provided to prevent diagonal tension and shear failures. The stirrups also served the function of preventing horizontal splitting that might otherwise have been caused by high bond stress. Beams 8-6SH and 8-6BH had two No. 6 bars placed as compression reinforcement to prevent flexural compression failure. Beams 8-8SH and 8-8BH had two No. 8 bars placed as compression reinforcement.

Properties of Bundles.—The external perimeter for a bundle of four bars as shown for beam 8-6B and 8-8B in Fig. 1 is $3\pi D$, that is, 25% less than for the same bars spaced in the usual manner. On the other hand, a single large bar with the same cross-section area as a bundle of four bars with diameter D would have a diameter of 2D and a perimeter of $2\pi D$. Accordingly, the bundle of four bars has an exposed perimeter 50% greater than that of the single large bar. The bundle of three bars used for beam 6-9B similarly has an exposed perimeter 16.7% less than that for the same spaced bars, and 55% greater than that of a larger bar of the same area.

These geometric properties indicate that bundling leads to only a moderate increase in bond stress as compared to spaced bars. Replacing a single large bar by a bundle with the same area, leads to a reduced bond stress. It should be noted, however, that the deformation of lug height, as defined by ASTM A-305-53T, would be greater for a single large bar than for a bundle of bars, and bond resistance for top bars is known[9] to increase with increasing lug height.

Materials.—A laboratory blend of Type I cements was used. Sand and gravel aggregates were combined to gradations within the limits given by ASTM C-33-55T for 1-$\frac{1}{2}$ in. maximum size. The concretes were mixed in 6 cu ft

2 "Unusual Concrete Roof of Hollow Girders and Precast Slabs," by H. M. Hadley, Journal, A.C.I., Proceedings Vol. 37, February, 1941, pp. 453-460.

3 "Brazil's Wonder Hotel and Casino," by A. J. Boase, Engineering News-Record, Vol. 136, January, 1946, pp. 112-116.

4 "Bundle Reinforcing Saves Materials," Engineering News-Record, Vol. 140, April, 1948, pp. 509-510.

5 "Bridge with 'Bundled' Reinforcement," by H. M. Hadley, Western Construction, Vol. 26, June, 1951, pp. 89-90.

6 "'Bundled' Reinforcement," by H. M. Hadley, Journal, A.C.I., Proceedings Vol. 49, October, 1952, pp. 157-159.

7 "Precast Box Beams for High Strength," by H. M. Hadley, Engineering News-Record, Vol. 125, Dec., 1940, pp. 383-839.

8 "Tests of Beams Reinforced with 'Bundle Bars'," by H. M. Hadley, Civil Engineering, Vol. 11, February, 1941, pp. 90-93.

9 "An Investigation of Bond, Anchorage and Related Factors in Reinforced Concrete Beams," by C. A. Menzel and W. M. Woods, Bulletin 42, Research Dept., Portland Cement Assn., November, 1952, p. 114.

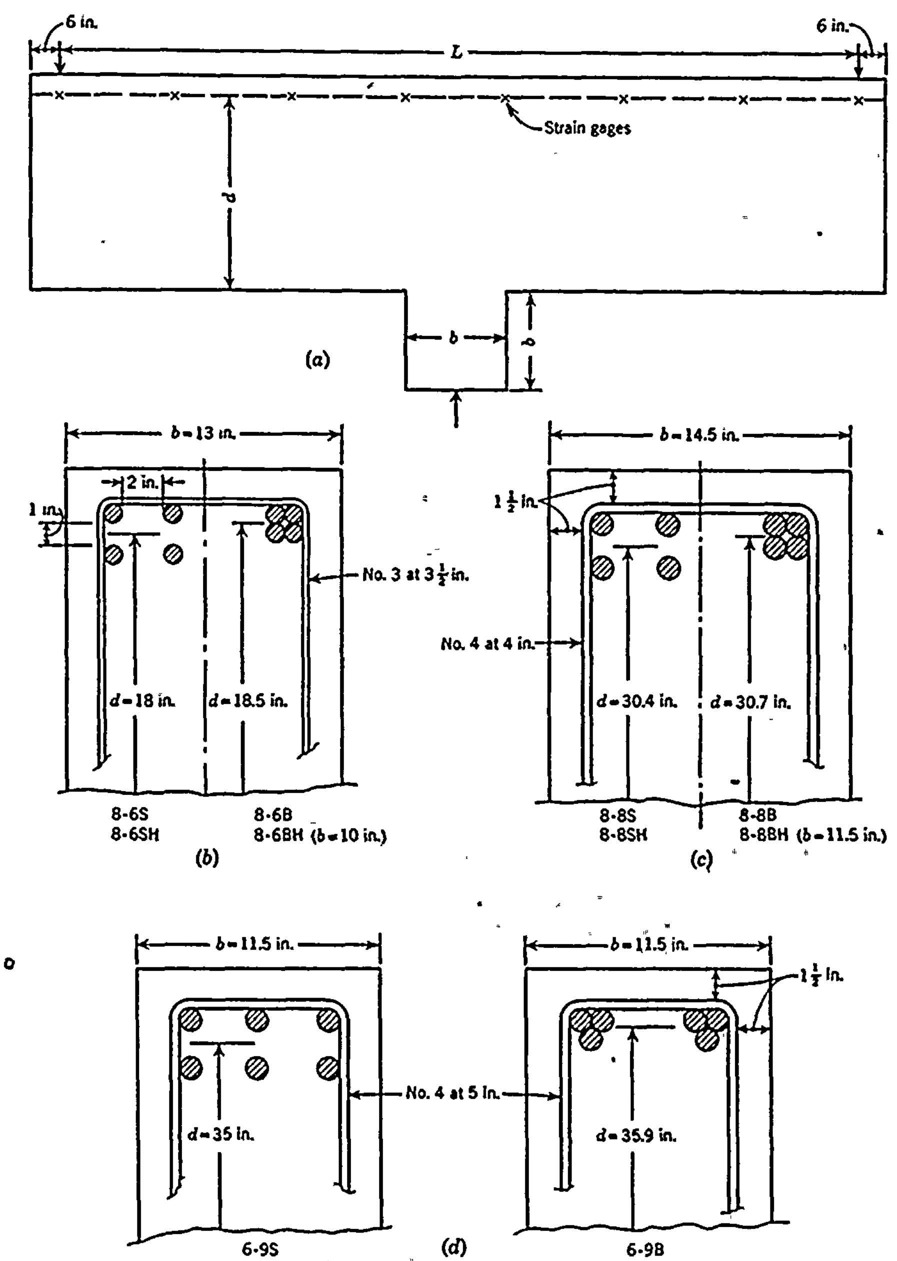


FIG. 1.—TEST BEAMS

batches, and had a slump of 2-1/2 to 3 in. The cement content was 4.0 to 4.2 bags per cu yd, and the water-to-cement ratio, by weight, was 0.62 to 0.70. The compressive strengths reported with the beam test data in Table 1 are averages of twelve to sixteen 6-in.-by-12-in. cylinders made, cured, and tested with each beam.

All reinforcing steel used conformed to ASTM A-305-53T for deformations and had the yield points given in Table 1. For No. 6, 8, and 9 bars, respectively, lug height was 0.048, 0.055, 0.067 in. and lug spacing was 0.35, 0.48, 0.50 in. The steel was purchased in such lengths that each bar yielded two beam lengths and a 3 ft length for tension test. By using one of these lengths in each beam of a pair, variation in steel properties between pairs was minimized. The No. 3 stirrups used in beams with eight No. 6 longitudinal bars had a yield point of 52,100 psi, whereas the No. 4 stirrups used in the remaining beams

TABLE 1.—BEAM STRENGTH

Beam Designation	Main Steel (f_y), in pounds per square inch	Cylinder Strength f'_c, in pounds per square inch	Test Age, in Days	Yield Load P_{yield}, in kips	Ultimate Load P_{test}, in kips	Calculated Flexural Ultimate Load P_{calc}, in kips	$\frac{P_{test}}{P_{calc}}$	Calculated Bond Stress at Ultimate Load, in pounds per square inch
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
8-6S	47,000	2935	15	137.5	150	142	1.05	252
8-6B	47,000	3080	16	137.5	161	148	1.09	351
8-8S	48,300	4375	26	280	314	281	1.12	235
8-8B	48,300	4240	28	260	349	283	1.23	345
6-9S	46,800	4480	14	240	308.5	255	1.21	238
6-9B	46,800	4600	15	240	281	256	1.10	253
8-6SH	84,200	4960	14	-----	310	262	1.18	520
8-6BH	84,200	3805	16	-----	235	253	---	513
8-8SH	80,000	4870	14	-----	450	462	---	337
8-8BH	80,000	3840	14	-----	396	443	---	391

had a yield point of 48,700 psi. All reinforcement for each beam was tied together without welding.

Casting.—All beams were cast in plastic coated plywood forms with the column stub down so that in the same manner as the longitudinal reinforcement was embedded near the top of the beam, because this reinforcement is near the columns in continuous beams. Concrete was placed through the use of spud vibrators in the test beams and their companion test cylinders. It was found that bundling of the bars eased the placing operation substantially. After testing the beams it was found that mortar had penetrated into and filled the cavity between the bars of all bundles. The test beams and their companion cylinders were cured one week under moist burlap. The beams were then turned upside down and stored in the laboratory until they were tested at the ages given in Table 1.

Test Method.—All beam specimens were tested as shown in Fig. 2, supported on a roller at one end and on a rocker at the other, with a 6-in. overhang at each end. The testing machine load was applied in increments to the column

stub through a 2-in. steel plate. The total duration of each beam test was approximately two hours.

Deflection dial gages were mounted directly below the two faces of the column stub and mid-way between these points and the supports. The widths of all cracks were measured by a graduated microscope at the level of the centroid of the longitudinal reinforcement.

To minimize the amount of bar surface area isolated from bond by the waterproofing of the electric strain gages, SR-4 Type A-12 gages were placed in the intermediate grade bars in milled slots 3/32 in. wide, 3/8 in. deep, and approximately 6 in. long. The high-strength bars could not be milled. Thus, the location of the strain gages in Fig. 1(a) does not refer to the beams using high-strength steel rods. A gage was cemented to the side of each slot, lead

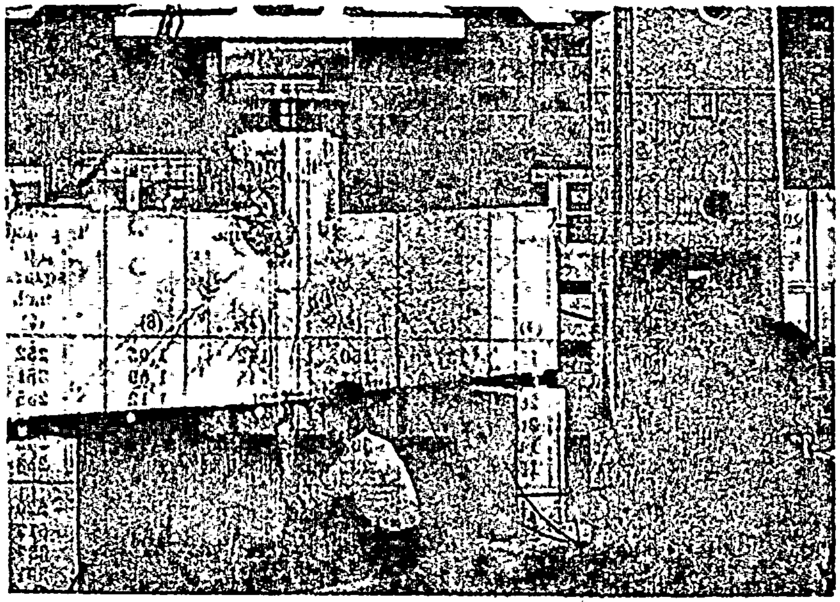


FIG. 2.—TESTING ARRANGEMENT FOR BEAMS

wires were attached, and the slot was filled to the bar surface with wax waterproofing. Tension tests indicated that gages so placed yielded measurements in close accord with mechanical strain measurements over the same reduced section. The stress at any bar load was 5% to 9% higher in the slotted section than in the full bar section. Eight strain gages for each beam were located as shown in Fig. 1(a). Two gages were placed on each of four bars symmetrically about mid-span. The measured strain without correction was assumed to represent the average strain in all bars at the location of the gage. Therefore, measured strain is reported herein as stress obtained by multiplying the average strain in the two half spans by the modulus of elasticity for the full section of the various bar sizes as obtained in tension tests.

BEAM TEST RESULTS

All beams with intermediate-grade steel, beams 8-6S through 6-9B in Table 1, failed by yielding of the longitudinal reinforcement, followed by large de-

flections and final crushing of the concrete compression zone at the column face. As shown in Fig. 2, both flexural and diagonal cracks tended to extend upward toward the corner at the column stub so that it was hardly possible to differentiate between flexural and diagonal cracks. No indication of bond failure was found in any of these beams, and no visual difference in behavior was noted for beams with bundled as compared to spaced bars.

Three beams with high strength reinforcement failed in bond as indicated by large amounts of bar slip at the beam ends. For beam 8-6SH the tension steel yielded following bar slip at the beam ends.

Steel Stress and Deflection.—Measured steel stress and deflection at various load levels for the three beam pairs with intermediate grade steel are shown in Figs. 3(a),(b)and(c). Both steel stress and deflection are given as an average of two measurements symmetrical about midspan for each beam.

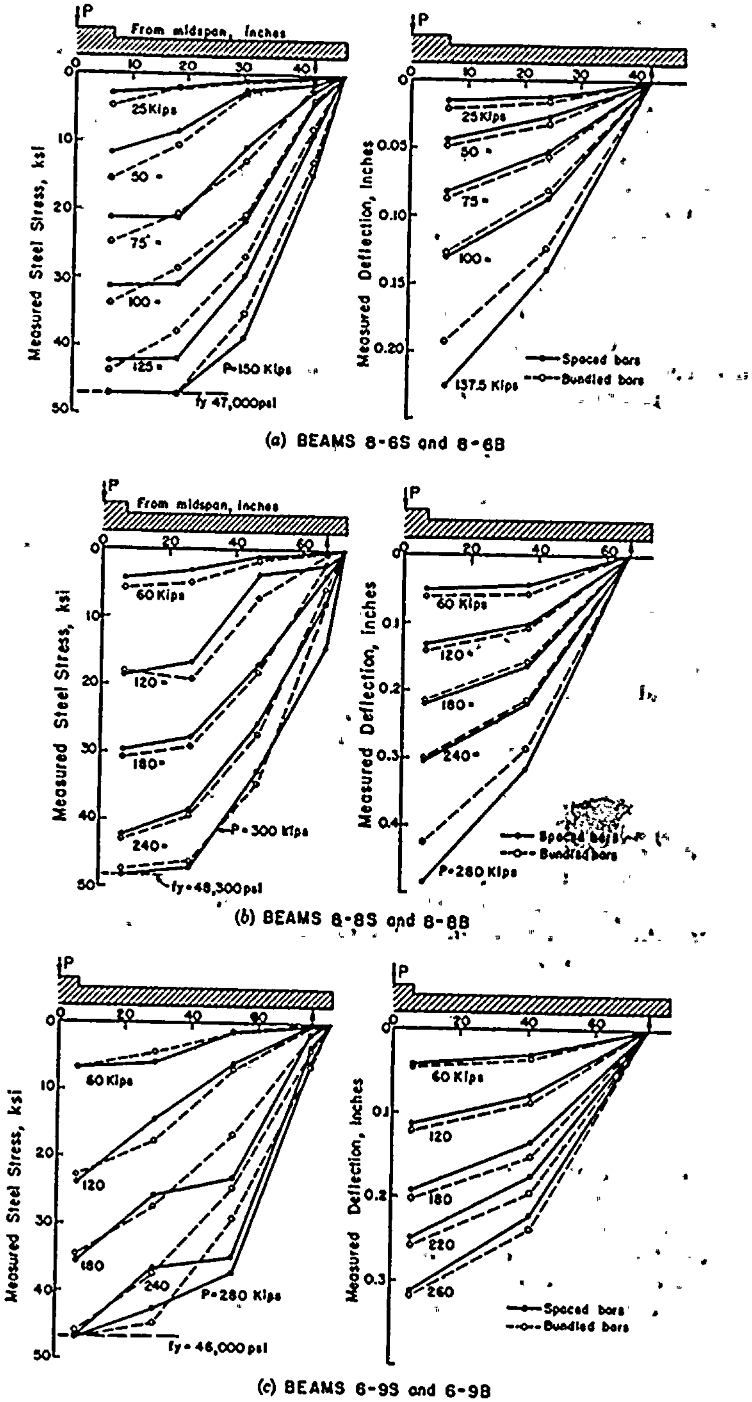
Distribution of measured steel stress along a longitudinal reinforcing bar, in a beam specimen, may be expected to reflect bond distress. Preceding a final destruction of bond, an abnormal rise of steel stress should take pl toward the beam ends. Fig. 3 shows that the distribution of steel stress very similar for the two members of each pair of test beams even at high steel-stress. It may be noted, on the other hand, that the steel-stress for all beams was practically uniform at high loads in the middle third of the span. This was certainly caused by a stress redistribution resulting from the deep-beam type of crack pattern seen in Fig. 2. It is also seen that the overhangs contributed to the bonding action because the steel-stress of all beams is not zero over the supports at high loads. It is felt that this behavior is related to local stress disturbances in the support region where heavy reaction forces entered the abnormally short beams.

The deflection curves are also similar for all beam pairs. Accordingly, both steel stress and deflection measurements indicate that there was no significant difference in behavior between bundled and spaced reinforcement.

Crack Width.—Crack patterns were closely similar within pairs for all tests. Bond distress may be expected to open up a few wide cracks near the beam ends rather than to increase the width of all cracks. Crack widths are therefore given in Fig. 4, as the average width of the three widest cracks in the beam. Steel stress is given in the figure as values computed from applied moment at the column face section, taking the internal moment arm as 7/8 times the effective depth. It is seen that there is no systematic difference between crack widths for bundled and for spaced reinforcement. Furthermore, no cr opened suddenly before yielding of the reinforcement was in progress. This indicates that even the high local bond stress, which acts near cracks, resulted only in the normal minor bond slip for bundled as well as for spaced reinforcement.

For the four beams with high strength reinforcement, a similar lack of systematic difference was observed between crack widths for bundled and spaced reinforcement. However, for beams 8-6BH, 8-8SH, and 8-8BH, as the ultimate load was reached, a few cracks near the beam ends became very wide shortly before final bond failure took place.

Flexural Strength, Beams with Intermediate Grade Steel.—It is seen in Table 1 that some of the beams with intermediate grade steel carried loads considerably above their yield loads. These yield loads are listed as detected by strain



(a) BEAMS 8-6S and 8-6B

(b) BEAMS 8-8S and 8-8B

(c) BEAMS 6-9S and 6-9B

FIG. 3.—MEASURED STEEL STRESS AND DEFLECTIONS

and crack width measurements. Considering the external moment at the face of the column the computed flexural ultimate loads, P_{calc}, were obtained by the equation for ultimate internal moment

$$M_u = b\, d^2\, f_c'\, q(1-0.59\, q) \quad (1)$$

in which f_c' is the concrete cylinder strength, and q is the factor pf_y/f_c', in which p equals A_s (the effective cross-sectional area of reinforcement) divided by bd, and f_y is the yield point of reinforcement. This equation is given in ACI 318-56, A605(b). The ratio of measured to computed ultimate load exceeds one (1) for all beams. The average ratio for the three beams with spaced bars is 1.13, and the average ratio for the beams with bundled bars is also 1.13. This indicates that there was no systematic difference in ultimate flexural strength developed by spaced and by bundled bars except that the beams with bundled bars were slightly stronger by virtue of the slight increase in effective beam depth.

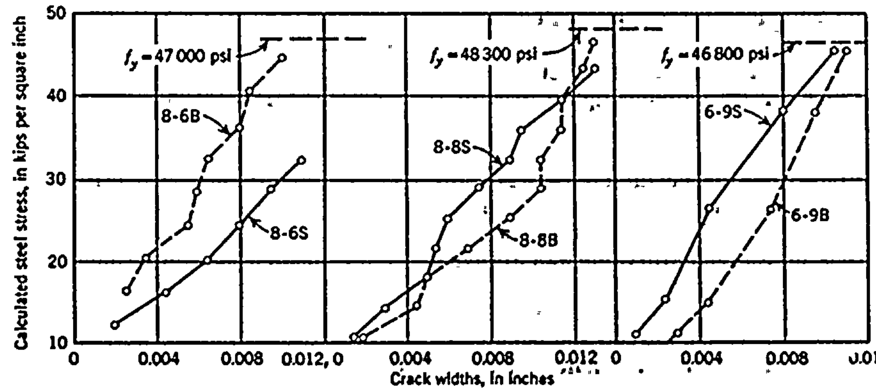


FIG. 4.—CRACK WIDTH MEASUREMENTS

The average ratio of 1.13 also confirms previous findings that the equation for ultimate moment, which was developed essentially by tests of small beams, is also applicable to the large beams of this investigation. It is believed that the excess of measured ultimate loads over the computed load resulted principally from strain hardening of the reinforcement. A biaxial state of stress at the column stub appeared to delay crushing of the compression zone so that large steel strains were developed locally at the major flexural cracks.

Bond stresses are given in Table 1 as computed at ultimate load, by dividing the shearing force by the external perimeter of the bars times 7d/8. These bond stresses, for the beams with intermediate grade steel, were sustained without any indication of bond failure. They are in no way to be regarded as ultimate bond stresses. To develop higher bond stresses with intermediate-grade steel it would have been necessary to make special test beams with part of the tension zone removed, or to make the beams so short that they would act as walls rather than beams. Both of these cases were thought not to represent practical conditions under which bundled bars may be used. High-strength steel was therefore used to study ultimate load stress.

Bond Strength, Beams with High-Strength Steel.—Table 1 shows that the beams with high-strength reinforcement failed at ultimate loads close to the flexural strengths computed by ACI 318-56, A606(a),

$$M_u = (A_s - A_s') f_y d \left[1 - \frac{0.59 (p - p') f_y}{f_c'}\right] + A_s' f_y (d - d') \quad \ldots . (2)$$

in which A_s is the area of tensile reinforcement, A_s' is the area of compressive reinforcement, d equals the distance from the extreme compressive fiber to the centroid of tensile reinforcement, whereas d' is the distance from this fiber to the centroid of compressive reinforcement and p' is the factor A_s'/bd. Beam 8-6SH failed in flexure after bond slip had been observed at the beam ends. The remaining three beams failed at loads below the computed ultimate flexural strength. Failure was in bond, as indicated by large amounts of bar slip observed by dial gages as a relative movement between bar ends and the concrete surface at the beam ends.

Bar slip is plotted as a function of computed bond stress in Fig. 5. Bond stresses at ultimate strength, calculated by dividing shearing force by external-bar-perimeter times 7d/8, are also shown. It is seen that bond slip was in progress when beam 8-6SH failed in flexure at a bond stress of 520 psi (Table 1). By comparison with the slip records for beams 8-8SH and 8-8BH in Fig. 5, both of which failed in bond, it must be expected that beam 8-6SH would have failed in bond at a stress only slightly greater than 520 psi if flexural failure had been prevented by a higher yield point for the steel. Hence, the ultimate bond stress for spaced No. 6 bars must be expected to exceed only slightly the value of 513 psi observed for bundled bars. For No. 8 bars, the ultimate bond stress for spaced bars was 337 psi, which is slightly less than the stress of 391 psi observed for bundled bars. However, it should be noted from Fig. 5 that bond stress for a given slip value was always lower for bundled than for spaced bars.

It can be concluded that, when only external bar perimeter was used to calculate bond stress, there was no systematic difference in ultimate bond stress developed between spaced and bundled bars.

Thus, the beam tests indicated that bundling of tension reinforcement is a satisfactory detailing procedure.

TEST COLUMN ARRANGEMENT

A series of ten tied columns was designed to study bundled compression reinforcement. Concentric loading was chosen. An outline of the test program is shown in Fig. 6. All columns were 12-in.-by-12-in. with a height of 6 ft. Two amounts of longitudinal reinforcement were used. These were 6.58% and 3.67%, made up of 12 No. 8 and 12 No. 6 bars, respectively. The 1/4-in. tie-diameter used is the minimum permitted by ACI 318-56, 1104(c). The corresponding maximum tie spacing of forty-eight tie diameters in 12 in., which is also the maximum spacing as governed by the 12-in. column size and by sixteen times the diameter of the No. 6 bars.

Five columns with 12 No. 8 bars were tested. Column 12-8S contained bars spaced in the normal manner and surrounded by a square tie. The interior bars were held firmly by two interior rectangular ties. All ties of this column were spaced at 12 in. Column 12-8B-1 contained bars bundled at the corners. Interior ties were omitted, and the exterior tie spacing was maintained at 12 in. For column 12-8B-2, the exterior tie spacing was decreased to 6 in. A splice was provided at mid-height of column 12-8B-3 as shown in Fig. 6. The spliced bars were cut by a saw, and each bar was touching its longitudinal extension. The tie spacing in both columns 12-8 B3 and 12-8 B4 was 6 ins. The

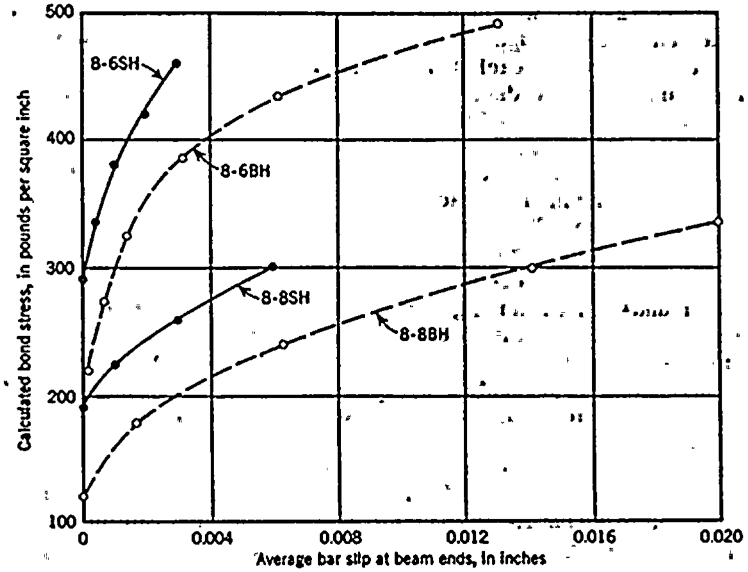


FIG. 5.—BAR SLIP MEASUREMENTS

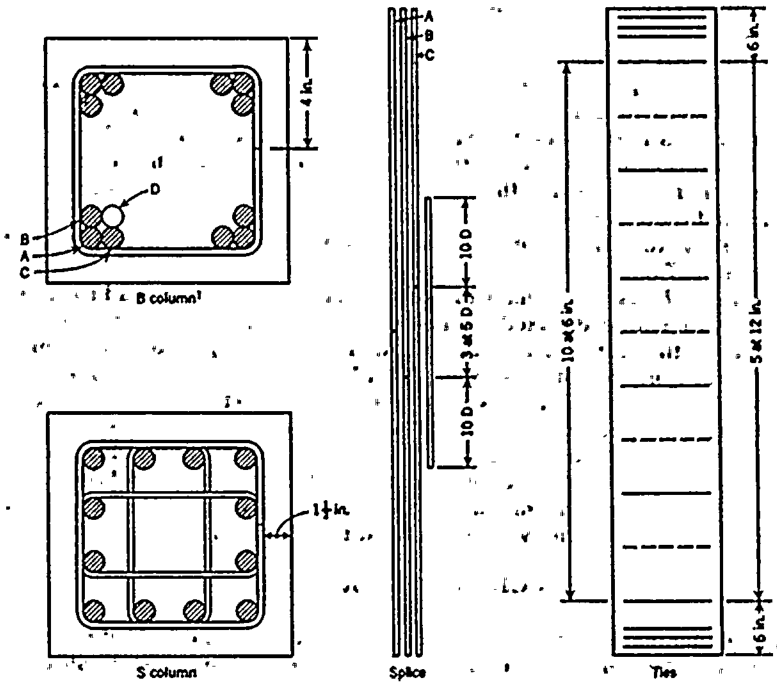


FIG. 6.—TEST COLUMNS

bars of column 12-8B-4 were cut at the splice with a hydraulic bar cutter so that the bar-ends were wedge-shaped. A clear spacing of 1/4 in. was provided between the two parts of each longitudinal bar. It should be noted that the splice lap is only ten bar-diameters as compared to the minimum amount of twenty diameters given by ACI 318-56, 1103(c). A similar group of five columns with 12 No. 6 bars was tested.

Materials.—A laboratory blend of Type I cements was used. Sand and gravel aggregates were combined to gradations within the ASTM C-33-55T limits for 3/4-in. maximum size. The mix ratio of cement to sand to gravel was 1 to 3.58 to 2.38 by weight, and the water-cement ratio was from 0.64 to 0.68 by weight.

Two concrete batches were used for each column. In previous column tests it has been found[10] that failure generally takes place near the top of vertically cast columns. To explore this phenomenon, the bottom batch of some columns was made with a slightly higher water-cement ratio than the top batch. Compressive strengths, representing averages of three to four 6-in.-by-12-in. cylinders for each batch, made and cured with the corresponding columns, are given in Table 2.

All reinforcement was intermediate-grade steel and was tied into cages without welding. The ties were 1/4-in. plain bars. The longitudinal reinforcement conformed to ASTM A-305-53T for deformations and had the yield points reported in Table 2. All reinforcing bars were cut by a saw to a length tolerance of 1/32-in. Bearing plates, 3/4-in. thick, were placed touching the bars at the top and bottom of the columns. The lower plate was placed in the form before casting, the upper plate was set in a thin layer of high-strength plaster after the concrete was cured. The heavy tie reinforcement shown in Fig. 6 prevented failure at the column ends by possible local non-uniform stress conditions.

Casting.—All columns were cast in a vertical position, in plywood forms protected by an epoxy resin paint. Concrete was placed in columns and companion cylinders with the aid of spud vibrators. It was noted that the absence of interior ties in the columns with bundled reinforcement eased the concrete placing operation substantially. By inspection after testing the columns, it was found that mortar had filled the cavity between the bars of all bundles. The test columns and their companion cylinders were cured four to five days under wet burlap. They were then stored in the laboratory until they were tested at the ages given in Table 2.

Test Method.—All columns were tested under concentric loading as shown in Fig. 7, with both ends fixed against rotation. Spherical bearings permitted rotation at both column ends until a load of 20 k was applied, after which the bearings were blocked by steel wedges. Electric strain gages applied at mid-height of all four column faces were monitored by a continuous strain recorder. Even at ultimate strength, the spread between the four gage readings was less than 15%, which indicates that a closely concentric loading was obtained for all columns. In addition to electric strain measurements, the total shortening over the entire column-height was measured by a dial gage. A continuous loading speed of 160 k per min was maintained for all columns.

COLUMN TEST RESULTS

The column test results were evaluated essentially in terms of the equation for ultimate strength of concentrically loaded tied columns established during

10 "A Study of Combined Bending and Axial Load in Reinforced Concrete Members," by E. Hognestad, Bulletin No. 399, Engrg. Experiment Sta., Univ. of Illinois, 1951.

the ACI column investigations in the 1930's. The equation used is

$$P_o = 0.85 f'_c \left(A_g - A_{ST}\right) + A_{ST} f_y \ldots\ldots\ldots\ldots \quad (3)$$

in which A_g is the gross area of the section and A_{ST} is the total area of longitudinal reinforcement.

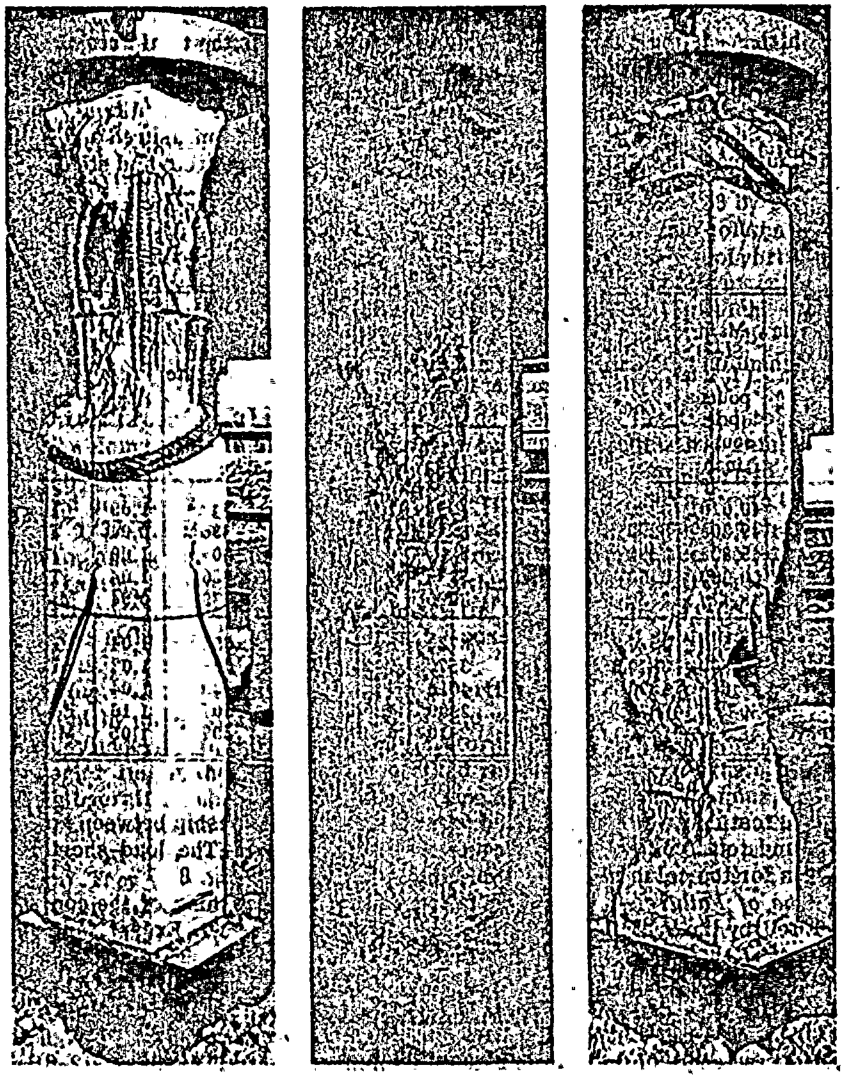
This equation has been confirmed by several recent investigations[10] and is used in Section A608(b) of ACI 318-56. A comparison of measured and calculated ultimate loads is given in Table 2 together with concrete and steel properties.

TABLE 2.—COLUMN STRENGTH

Column Designation	Main Steel, (f_y), in pounds per square inch	Cylinder Strength, f'_c, in pounds per square inch			Test Age, in Days	Measured Ultimate Load, (P_{test}) in kips	Calculated Ultimate Load (P_{calc}) in kips	$\frac{P_{test}}{P_{calc}}$	Location of Failure
		Top	Bottom	Average					
12-8S	49,610	3220	3290	3250	5	915	842	1.09	Top
12-8B-1	49,500	3290	3150	3220	8	783	836	0.94	Top
12-8B-2	49,800	3930	3680	3800	6	909	906	1.00	Top
12-8B-3	50,000	3550	3150	3350	6	889	856	1.04	Top
12-8B-4	48,470	3280	3360	3320	7	789	839	0.94	Middle
12-6S	48,510	3970	3470	3720	8	726	695	1.04	Top
12-6B-1	49,300	3840	3310	3570	7	702	681	1.03	Top
12-6B-2	48,800	4200	3820	4010	7	758	730	1.04	Top
12-6B-3	50,200	3270	2540	2900	4	702	607	1.16	Bottom
12-6B-4	48,230	2960	2860	2910	6	626	597	1.05	Middle

The test data were also studied in terms of the relationship between applied load and total column shortening expressed as strain. The load-shortening curves for the columns with 12 No. 8 bars are given in Fig. 8.

Type of Failure.—All columns failed through the crushing of the concrete followed by the buckling of the longitudinal reinforcement. Except for three columns, failure took place in the upper half of the columns. The typical nature of such a failure is shown in Fig. 7(a). The strength of the concrete placed in the lower half of some columns was reduced to explore the phenomenon of top failure, which had been observed[10] in numerous previous tests. Though the cylinder strength of the bottom batch for columns 12-8B-1, 12-8B-2, 12-8B-3, 12-6S, 12-6B-1 and 12-6B-2, was from 4% to 14% less than that of the top batch, failure took place in the upper half of the columns. For column 12-6B-3, cylinder strength of the bottom batch was 22% below that of the top batch, and in this case failure took place in the lower half of the column as shown in Fig. 9(c). Even so, the measured ultimate load exceeded by 24% the value calculated on the basis of the cylinder strength for the bottom batch.



No. 12-8B-1 (a) No. 12-8B-4 (b) No. 12-6B-3 (c)

FIG. 7.—TYPICAL COLUMN FAILURES

These findings confirm the previously reported observation that the column strength of the concrete placed in the lower half of columns is increased, probably by the improved compaction afforded when the upper half is cast. Similarly, the column strength of the concrete placed in the upper half may decrease somewhat by water gain from below. To evaluate effects of bundling, therefore, measured ultimate loads were compared to calculated values based on the average cylinder strength for the top and bottom batches for each column.

The two spliced columns 12-8B-4 and 12-6B-4, which had 1/4 in. clear between bars at the splice, failed in the splice region at mid-height as shown in Fig. 7(b).

Effect of Bundling.—The ratios between measured and calculated ultimate load given in Table 2 exceed one (1) for all except two columns. This confirms previous findings that led to the ACI column investigation equation.

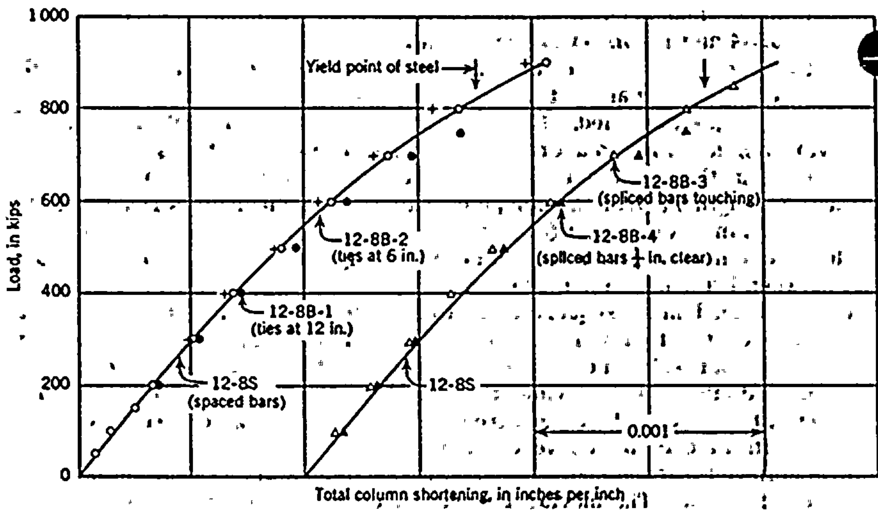


FIG. 8.—LOAD-SHORTENING CURVES FOR COLUMNS

The load ratios of columns 12-6B-1 and 12-6B-2 were 1.03 and 1.04 respectively as compared to a value of 1.04 for column 12-6S that had conventionally spaced bars. For 3.67% longitudinal reinforcement, therefore, no detrimental effect of bundling was found regardless of tie spacing.

For the columns with 6.58% reinforcement, which exceeds the maximum value of 4% given by ACI 318-56, 1104(a), the 12-in. tie spacing of column 12-8B-1 led to a load ratio of 0.94 as compared to 1.09 for column 12-8S with spaced bars. By reducing the tie spacing to 6 in. for column 12-8B-2, the load ratio increased to 1.00. Furthermore, column 12-8B-3, which had a 6-in. tie-spacing and failed above the splice, had a load ratio of 1.04. Therefore, even for 6.58% reinforcement, no detrimental effect of bundling was found when the tie spacing for bundled bars was reduced to 6 in., which is equal to 24 tie-diameters, or one-half of the least dimension of the column.

Bundling did not significantly affect the relationship between applied load and column shortening. This is shown for the columns with No. 8 bars in Fig. 8.

The results for the columns with No. 6 bars indicated a similar lack of effect of bundling.

Splicing.—Bundled reinforcement placed in the corners of a column section may be spliced in the same manner as single corner bars. The bars from below may be offset to a position inside the bars above the splice, and a proper amount of lap may then be provided. The bars may also be butted and welded.

In these tests, a splice particularly suitable for bundled bars was explored. As shown in Fig. 6 the splices of the three bundled corner bars were staggered a distance of five bar-diameters, and a fourth splice-bar, 35 bar-diameters long, was added at each corner. For columns 12-8B-3 the bars were cut by a saw, and each bar was touching its longitudinal extension. The contact was not perfect and after testing, a mortar layer 1/32 in. to 1/16 in. thick was found between the bars. Both of these columns failed outside the splice, and the measured ultimate loads exceeded the computed values by 4% and 16%, respectively.

To simulate less accurate manufacture of reinforcement, the bars of columns 12-8B-4 and 12-6B-4 were cut at the splice by a hydraulic bar cutter with 60° cutting edges. The bar ends were wedge-shaped by the cutting to an angle of 90°, and a clear distance of 1/4 in. was provided between the bars when the reinforcing cages were tied. Both columns failed at mid-height in the splice. However, in spite of the unfavorable conditions for a direct stress transfer between bars in the longitudinal direction, the No. 6 bar column developed an ultimate strength 5% over the computed value. The ultimate strength of the No. 8 bar column was only 6% below the computed value. As shown for the No. 8 bar columns in Fig. 8, the splice did not significantly change the relationship between load and column shortening.

It was planned in subsequent tests to strengthen the splice by longitudinal welds between the four bars at each splice. Even without welds, however, three out of four spliced columns developed an ultimate strength in excess of the computed values. It is obvious that the strength of splices with longitudinal welds would exceed that of the three bars outside the weld. Therefore, no columns with welded splices were made.

It is believed that the short lap used in the splices did not suffice to transfer stress by bond. The mortar between meeting bar ends was probably subjected to a triaxial state of stress so that a compressive strength far in excess of the cylinder strength could be developed. To assure that the longitudinal bars do not buckle in the splice, the reduced tie-spacing used in the tests, twenty-four tie-diameters or one-half the column dimension, may be necessary. If a splice of the type studied is used in eccentrically loaded columns so that tensile stress may be developed in the longitudinal bars, the mortar between bars cannot be expected to transfer stress and welds, or a longer lap, are obviously necessary.

CONCLUDING REMARKS

The test results reported confirm the previous findings that the use of bundled reinforcement is a sound detailing procedure. It can be expected that bundling of tension reinforcement in beams will not lead to detrimental consequences as compared to spaced bars, for the following conditions: 1. There are not more than four touching bars in each bundle, 2. Bond stress computed on the basis of external bar perimeter is limited to the values now permitted for spaced bars, 3. Each bar in a bundle is a deformed bar and is individually well anchored, and 4. Stirrup reinforcement is provided in regions of high bond stress.

Bundling of compression reinforcement in tied columns can also be used even for high ratios of longitudinal reinforcement, if the provisions of ACI 318-56 regarding other details are strictly complied with. For large amounts of longitudinal bundled reinforcement, it is advisable to reduce the maximum tie-spacing to about one half of that given by the ACI Building Code.

Because bundling of reinforcement was found to be safe in tests involving the extreme cases of bending alone and compression alone, bundling should also be satisfactory for members subject to combined bending and axial load.

APPENDIX.—NOTATION

The following symbols, adapted for use in the paper and for the guidance of discussers, conform essentially with "American Standard Letter Symbols for Structural Analysis" (ASA A10.8-1949), prepared by a committee of the American Standards Association with Society representation, and approved by the Association in 1949:

A_g = Gross area of section;

A_s = Area of tensile reinforcement;

A_s' = Area of compressive reinforcement;

A_{ST} = Total area of longitudinal reinforcement;

d = Distance from extreme compressive fiber to centroid of tensile reinforcement;

d' = Distance from extreme compressive fiber to centroid of compressive reinforcement;

f_y = Yield point of reinforcement not to exceed 60,000 psi;

f_c' = Concrete cylinder strength of test specimen;

p = A_s/bd;

p' = A_s'/bd; and

q = $(pf_y)/f_c'$.

DISCUSSION

HOMER M. HADLEY,[11] F. ASCE.—The writer was pleased to read this paper on the testing of bundled reinforcement in both beams and columns. On

11 Cons. Engr., Seattle, Wash.

numerous occasions, he has found bundling highly advantageous in beams particularly in precast channel-shaped concrete sections for short-span bridge decks, for which A. C. I. or AASHO bar spacing is peculiarly ill-adapted. There are probably several hundred of such short spans - 16-ft-to-30-ft long and ten or fewer years old - installed and in service in various parts of the state of Washington. These have been made with four bundled bars in each leg of the channel. The bar size depends on the span length, with single-bar stirrups looped around the bundle at the bottom. The stems of the channel webs are usually given a 6-in. bottom thickness with a 7-in. top thickness at the underside of the slab. These over-all web thicknesses, except in the case of the outer curb units, are initially reduced to approximately 4 in. by notching with a 2-in. plank on their outer faces. The notch starts approximately 6 in. above the bottom of the stem. When the units are placed side by side, these notched spaces, and any additional spaces are filled with concrete and thereafter there is full cover of the bundle everywhere.

There have been a few such small bridges on Federal Aid projects. After quite a number of small county bridges had been successfully installed, permission was granted on a project having twenty-eight ft trestly spans to use the precast units with four bar bundled reinforcement in each web. This project was likewise successfully installed and to the best of the writer's knowledge has proven entirely satisfactory. Unfortunately, an engineer from Washington, D. C. visited the project during construction and voiced some misgivings about bundling. This brought on a local reaction of rejection to the practice and permission to bundle reinforcement was withdrawn for several years. It is the writer's understanding that currently three bars may be bundled on local Federal Aid projects but not four bars. He is unable to explain the rationale of this ruling. The California Highway Department has bundled four bars on Federal Aid projects since 1949 and continues to do so.

Not mentioned by the authors among the named advantages of bundling is the fact that it affords opportunity for having the quantity of beam reinforcement conform roughly with the moment diagram, by stopping unneeded steel areas somewhere near the points at which they become unneeded. In these days of high-priced reinforcement, such savings can total a considerable sum. The original use of 1/2 in. square bars in contrast with 1-in. square bars was in demonstration of this fact. In the beam with the single 1-in.-square bar, that bar had to run through from end to end of the beam, whereas with the bundled four 1/2-in. square bars only two of them carried through from end to end, and slightly offset longitudinally in the beam, provided as much effective bond area as the 1-in.-square bar offered.

The authors state in their conclusion that "bundling of tension reinforcement in beams will not lead to deterimental consequences as compared to spaced bars for the following conditions: 1. There must not be more than four touching bars in each bundle; 2. Bond stress computed on the basis of external bar perimeter must be limited to the values now permitted for spaced bars . . . "

Strictly limited to their test findings these statements are correct. However, attention should be drawn to the fact that they did not test five or six bars in a bundle and that there is nothing to be found in these tests to indicate or imply that a larger number of bars should not be bundled if that is desirable. The writer has used six No. 10 bars in a bundle, stacked 1-2-3 from top down in one bridge in beams of 9 in. width, and 2-2-2 from top down in a second bridge in beams of the same 9 in. width. No ill-effects have been observed. In the latter case the two vertical tiers were not in contact with one another but there was considerably less than orthodox spacing between the tiers.

In the writer's mind it has been the long-held and continued concept that if the bars in a large bundle are successively well anchored in the concrete at their ends, so that they can develop their designed stress at these ends, it then matters little how much or how little bond they have between these terminal zones. It is at these end zones that anchorage is indeed vital. The intermediate concrete is simply fireproofing or weatherproofing. With a dozen bars in a bundle, with good plastic concrete and with vibration, the fines of the mortar will penetrate and fill the interstitial spaces of the bundle and afford all needed protection. But the dozen bars must be well anchored at their several ends. About that necessity there must be no misunderstanding.

The writer is particularly pleased to see bundling applied to columns, where it will unquestionably effect marked improvement in economy and quality. The contrasting column cross-sections in Fig. 6 convincingly show this. The authors and whoever else participated in this development are to be congratulated upon its excellence.

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/18/83 Book No. SV-1022 Page No. ____
Drawing No. ____ Calc. No. 6.19-137 Sheet 1 of ____
By Hickey Checked LF Approved ____
Title WPPSS - Hanford No 2 - Reactor Bldg - Investigation of East Ext. Wall

Investigation of East Exterior Wall of Reactor Bldg

Reference: ① Bethlehem Placement Dwg. B&R File No 206-00-3153 (See Sheets 8 & 9)
② Field Sketch 17, showing exposed East Wall bars, (see sheet 10)

Concern: The possibility exists that in the horizontal rebar in the outside face of the East Exterior Wall of the Reactor Bldg, approximately 8'-6" North of Col. line N, Between Elev. 471'-0 & 501'-0" the "As Built" splices are not staggered as called for on Burns & Roe Drawing S-749

Approach to Problem Resolution

① The horizontal rebar for this wall is designed primarily to take the building shear forces in this wall acting as a shear wall.

② The steel requirement for this wall acting as a shear wall controls the wall design. Steel requirements for other forces (Perpendicular Seismic, Perpendicular Tornado, Missiles) are not controlling.

③ Actual Reinforcing used has a maximum computed stress of less than 0.5 fy.

④ As Per (ACI 318-71)
Section 7.6.3.2; In regions of low computed stress. Splices in regions where the maximum computed stress in the bar is always less than 0.5 fy shall meet the following requirements:
Section 7.6.3.2.2; If more than three quarters of the bars are lap spliced within a required lap length, splices shall meet the requirements for Class B splices (lap of 1.3 ld)

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/18/83 Book No. SV-1022 Page No. ____
Drawing No. ____ Calc. No. 6.19-137 Sheet 2 of ____
By Hickey Checked LS Approved ____
Title WPPSS - Hanford No 2 - Reactor Bldg - Investigation of East Ext. Wall

Investigation of East Exterior Wall

(5) Splices in question in the outside face of the wall are Class B splices (lap = 1.3 ld)

Conclusion: Since the maximum stress in the rebar in the outside face of the East Exterior Wall is less than 0.5 fy, the Class B (lap = 1.3 ld) "As Built" splice is acceptable as per ACI 318-71 Code, sections 7.6.3.2 & 7.6.3.2.2.

The following calculations substantiate that the maximum computed rebar stress is less than 0.5 fy.

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 1[illegible]/18/83 Book No. SV-1[illegible]2 Page No. ______
Drawing No. — Calc. No. 6.19-137 Sheet 3 of ____
By Hicks Checked 4F Approved ______
Title WPPSS - Hanford No 2 - Reactor Bldg - Investigation of East Exterior Wall

SHEAR STRESS CRITERIA FROM ORIGINAL CALCULATIONS

BURNS AND ROE, INC.
Oradell, N.J. • Hempstead, N.Y. • Los Angeles, Calif.

SHIPPED JUL 18 1973

W.O. No. ______ 9.14.72 Book No. SV-37 Page No. 44
Drawing No. 5737 Calc. No. 6.13.15 Sheet 12 of 27
By ______ Checked ______ Approved ______
Title WPPSS - HANFORD NO. 2 - REACTOR BLDG. EXTERIOR WALLS

SHIPPED AUG 17 1977.

NOMINAL SHEAR STRESS v_u ACI 318-71 - 11.2 AND 11.16

$$v_u = \frac{V_u}{\varphi b_w d} \qquad \varphi = 0.85$$

$$= \frac{V_u}{(0.85)\times(.8)A} \qquad d = 0.8\ell_w$$

$$= \frac{V_u}{0.68 \times A \times 144} \text{ PSI} \qquad A = \text{AREA IN FT}^2$$

SHEAR REINFORCEMENT IN WALLS

$$A_v = \frac{(v_u - v_c)\, b_w s}{f_y} \qquad v_c = 2\sqrt{f'_c} = 126.6 \text{ PSI}$$

$$(\text{ACI 318-71, 11.16.5})\ 10\sqrt{f'_c} = 633 \text{ PSI}$$

HORIZONTAL SHEAR REINFORCEMENT ACI 318-71 11.16.4.1

A_v AS COMPUTED BY $A_v = \dfrac{(v_u - v_c)\, b_w s}{f_y}$

$$\rho_h = \frac{\text{HORIZ SHEAR REINF AREA}}{\text{GROSS VERT. CONC. AREA}} \not< .0025$$

VERTICAL SHEAR REINF. ACI 318-71 11.16.4.2

$$\rho_n = \frac{\text{VERT. SHEAR REINF. AREA}}{\text{GROSS HORIZ. CONC. AREA}} = .0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_h - .0025)$$

$$\not> \rho_h$$

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/18/83 Book No. SV-1022 Page No. ____
Drawing No. ____ Calc. No. 6.14-137 Sheet 4 of ____
By Hinkey Checked LF Approved ____
Title WPPSS - Hanford No 2 - Reactor Bldg - Investigation of East Exterior Wall
East Exterior Wall

Determination of Allowable Shear Stress in Shear Wall

Reference: ACI Code 318-77 Section 11.16.2

Equation 11-32

$$V_c = 3.3\sqrt{f'_c}\,h d + \frac{N_u d}{4 l_w}$$

Equation 11.33

$$V_c = \left[0.6\sqrt{f'_c} + \frac{l_w\left(1.25\sqrt{f'_c} + \frac{0.2 N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}} \right] h d$$

see sheet 5 for determination of M_u & V_u

Form BR 8002-2

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/19/83 Book No. SV-1022 Page No. ____
Drawing No. ____ Calc. No. 6.19-137 Sheet 5 of ____
By Hickey Checked LF Approved ____
Title WPPSS - Hanford No 2 - Reactor Bldg. - Investigation of East Exterior Wall

Investigation of East Exterior Wall

Determination of Allowable Shear Stress

Determination of Moment @ Elev 471'-0

Reference Calc Book SV37 Page 43

Location	Shear (Wall 3 or 4)	×	Dist to EL471	Moment
Node 2	11,554	×	606.86 - 471 (135.9)	1,570,189
Node 3	16,396 - 11554 (4842)	×	572 - 471 (101)	489,042
Node 4	17,768 - 16,396 (1372)	×	548 - 471 (77)	105,644
Node 5	19,189 - 17,768 (1421)	×	522 - 471 (51)	72,471
Node 6	21,582 - 19,189 (2393)	×	501 - 471 (30)	71,790
				2,309,136 k-ft

Cumulative Shear @ El 471'-0 = 21,582k

Note: The shear and moment values above, used in the determination of allowable shear in equation 11-33 of sheet 4, are values obtained from Calc Book SV 37, page 43. Later shear values were determined in Calc Book SV-1004 & SV-1026 for use on page 7 in the determination of horizontal steel requirements. However the values used above in the determination of shear and Moment for use in Equation 11-33, are satisfactory because of the constant relationship between moment and shear. In addition, Equation 11-32 of sheet 4, not equation 11-33 governs. (see sheet 6).

Form BR 8002-2

BURNS AND ROE, INC.

W.O. No. 3900-03 Date [redacted]0/18/83 Book No. SV-[redacted]2 Page No.

Drawing No. — Calc. No. 5.19-137 Sheet 6 of —

By Archer Checked LF Approved

Title WPPSS-Hanford No 2 - Reactor Bldg - Investigation of East Exterior Wall

East Exterior Wall

Determination of Allowable Shear Stress in Shear Wall

Reference Sh 4

ignore this term (conservative since N_u is positive)

① $V_c = 3.3\sqrt{f'_c}\, hd + \left(\dfrac{N_u d}{4 l_w}\right)$

thus $v_c = 3.3\sqrt{f'_c} = 3.3\sqrt{4000} = 209 \text{ psi}$

Reference Sh. 4 & 5

ignore this term (conservative since N_u is positi

② $V_c = \left[0.6\sqrt{f'_c} + \dfrac{l_w\left(1.25\sqrt{f'_c} + \dfrac{0.2 N_u}{l_w h}\right)}{\left(\dfrac{M_u}{V_u} - \dfrac{l_w}{2}\right)}\right] hd$

$l_w = 133'$ (See Dwg S-709)

thus $v_c = \left[0.6\sqrt{4000} + \dfrac{133' \times 12\,(1.25\sqrt{4000})}{\left(\dfrac{2{,}309{,}136 \times 12}{21{,}582} - \dfrac{133 \times 12}{2}\right)}\right]$

$= 38 + \left(\dfrac{126{,}175}{1284 - 798}\right) = 38 + 260 = 298$

as per ACI 11.16.2, use lesser value of above

thus $v_c = 209 \text{ psi}$

Form BR 8002-2

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/18/83 Book No. SV-1022 Page No.
Drawing No. Calc. No. 6.19-137 Sheet 7 of
By Hickey Checked LF Approved
Title WPPSS-Hanford No 2-Reactor Bldg-Investigation of East Exterior Wall

Determination of Rebar Requirement in East Exterior Wall

Total Bldg Shear @ El. 471'0 = 23,776 k Reference:
{CalcBk SV-1026
{CalcBk SV-1004
Calc.No. 6.19-121

% of total building shear delivered to exterior walls = 64% (Reference: Calc Bk SV37, page 35)

Thus total factored shear to exterior walls
$= 0.64 \times 1.9 \times 23,776 = 28,912$ k

Shear delivered to East Wall $= \frac{28,912}{2} = 14,456$
Factored Torsional Shear $= \frac{1,705}{16,161}$ ← see CalcBk SV37 page 43

$v_u = \frac{16,161}{.68 \times 370 \times 144} = 446$ psi
see sh 3
(Reference: CalcBk SV37, page 43)

$v_c = 209$ psi (see Sh 5)

$A_v = \frac{(v_u - v_c) \times b \times s}{f_y} = \frac{(446 - 209) \times 30 \times 12}{60,000} = 1.42 \text{ in}^2$ (237)

Wall steel provided = #11@12 = $2 \times 1.56 = 3.12 \text{ in}^2$

Stress level in bars $= \frac{1.42}{3.12} = 0.46 < 0.5 f_y$

Form BR 8002-2

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/19/83 Book No. SV-1022 Page No. ____
Drawing No. S723 Calc. No. 6.19-137 Sheet 1 of ____
By Hickey Checked LF Approved ____
Title WPPSS Hanford No 2 - Reactor Bldg - Investigation - North Fuel Pool Wall

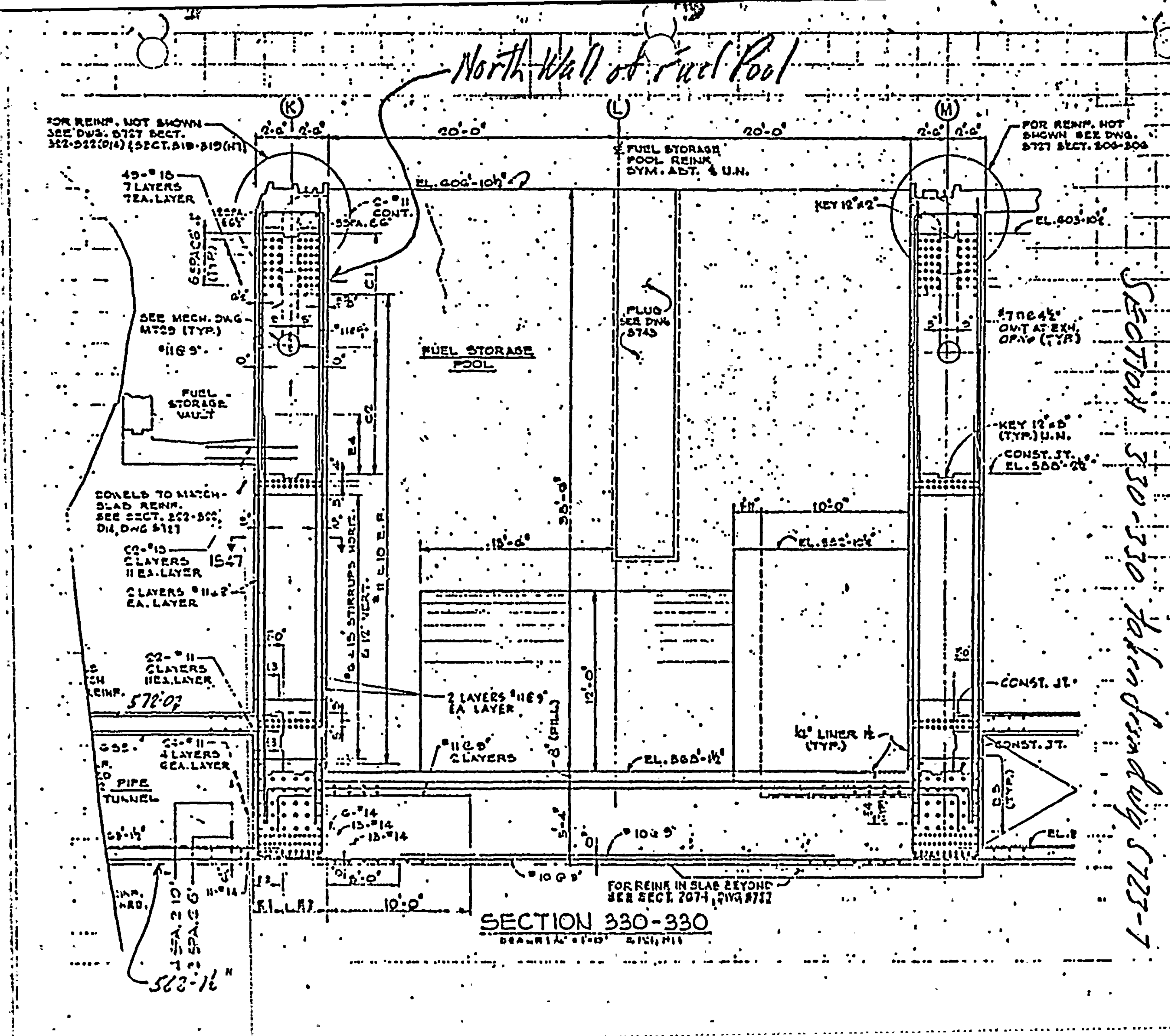
NORTH FUEL POOL WALL INVESTIGATION

Burns & Roe Dwg S-723, Section 330 shows the horizontal face reinforcing in the North Wall of the Reactor Bldg Fuel Pool Wall to be #11@10 each face, placed continuously from just above the construc joint at elevation 568'-1½" to the underside of the heavy horizontal wall beam reinforcing below elevation 603'-10½". Bethlehem Steel Reinforcing placement drawing (Burns & Roe File No 6809-00-0095) for this wall, shows the same reinforcing, at the same spacing, between the same elevations, but with an interruption in the spacing in the vicinity of elevation of 588'-2½" due to the presence of heavy horizontal Fuel Pool Wall Beam Reinforcing, placed primarily for Fuel Pool Wall Construction purposes.

It is the purpose of this calculation to show that the horizontal reinforcing in the wall in the vicinity of Elev 588'-2½, is adequate in spite of the interruption in the spacing described above.

This calculation shows that the wall beam reinforcing in the vicinity of elev 588'-2½, placed there for construction purposes, has sufficient reserve strength to replace the #11 @ 10 called for on the Burns & Roe Drawing S723, Section 330, but interrupted by the presence of this wall beam reinforcing as shown on the Bethlehem Steel Reinf placement drawing.

Form BR 8002-2



BURNS AND ROE, INC.

W.O. No. 3900-03 Date 1/19/83 Page No. ___

Drawing No. ___ Book No. S.V-102

By ___ Checked ___ Calc. No. 6.14-47 Sheet 2 of ___

Approved ___

Title WNP-1 Hanford No 2 Reactor Bldg - Investigation of North Wall

BURNS AND ROE, INC.

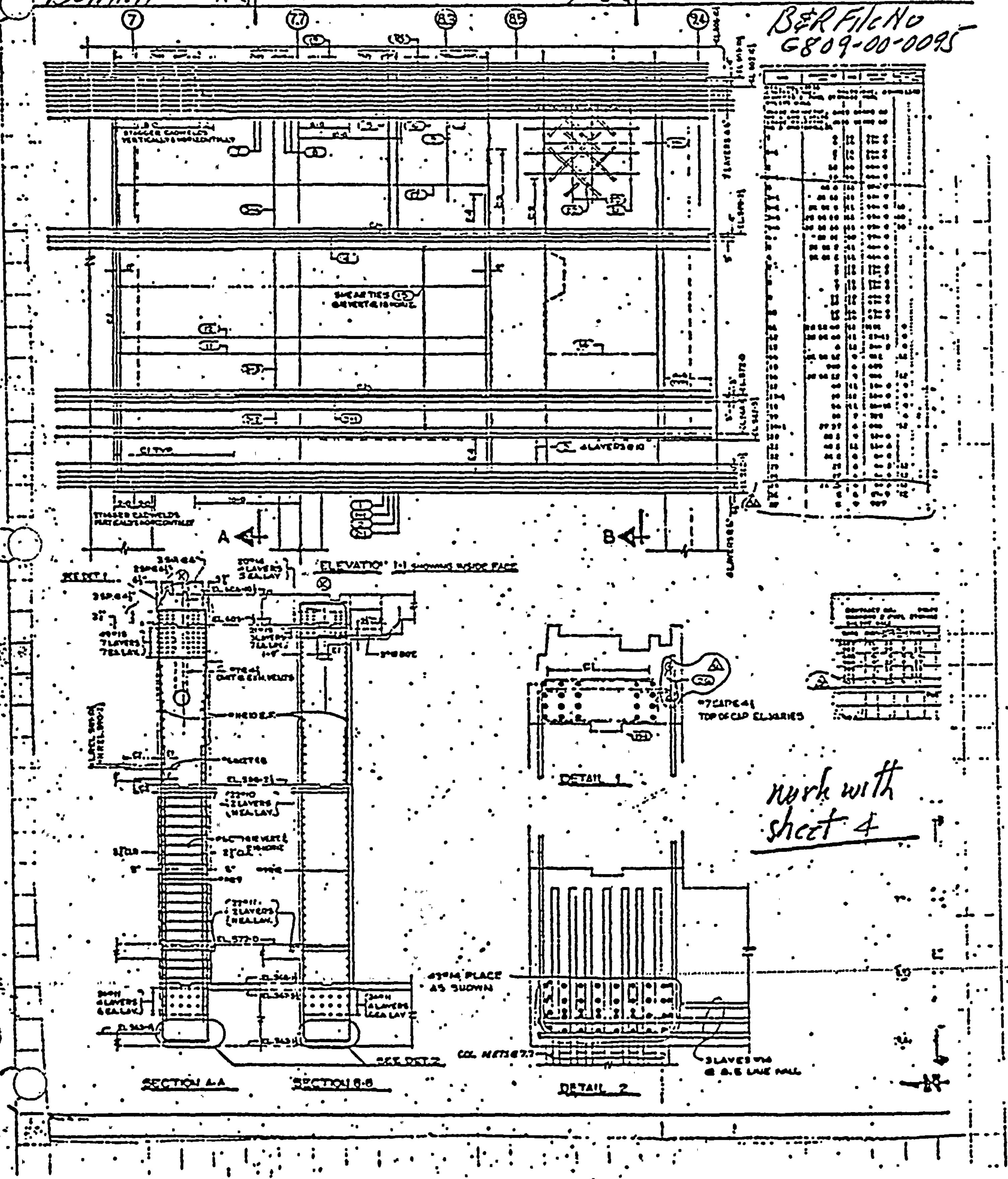
W.O. No. 3900-03 Date 10/[illegible]3/83 Book No. SV-102[illegible] Page No

Drawing No. — Calc. No. 6.19-137 Sheet 3 of

By Hicks Checked [illegible] Approved

Title WPPSS - Hanford #2 - Reactor Bldg. - Investigation - North Fuel Pool Wall

Bethlehem Steel Rebar Placement Dwg - North Wall Fuel Pool



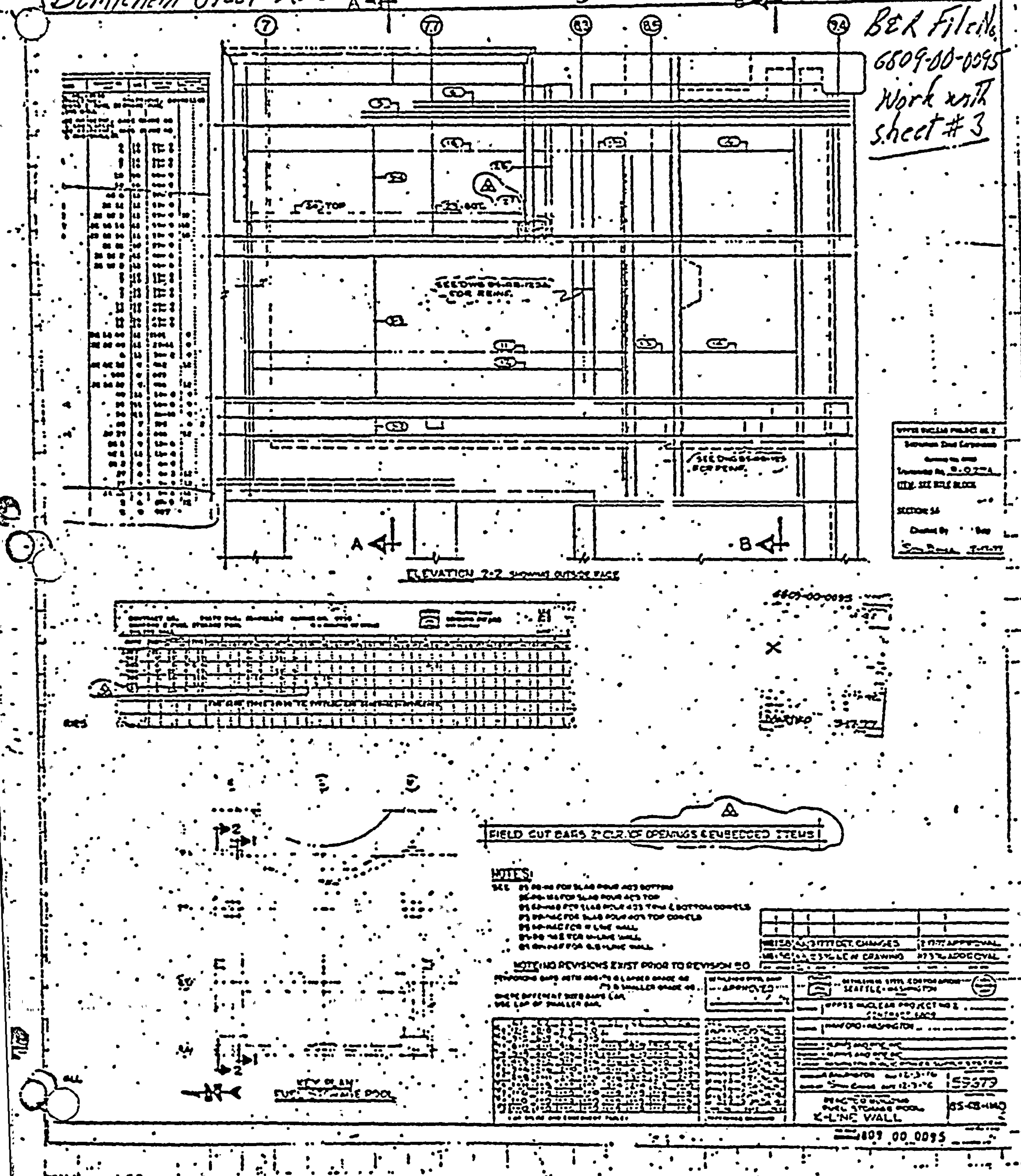
B&R File No G809-00-0095

work with sheet 4

W.O. No. 3900-03 Date 10/19/83 Book No. SV-1022 Page No.
Drawing No. Calc No. 6,14-137 Sheet 4 of
By Hickey Checked LF Approved
Title WPPSS-Hanford No 2-Reactor Bldg-Investigation-North Fuel Pool Wall

Bethlehem Steel Rebar Placement Dwg - North Wall of Fuel Pool

BER File No. 6809-00-0095
Work with sheet #3

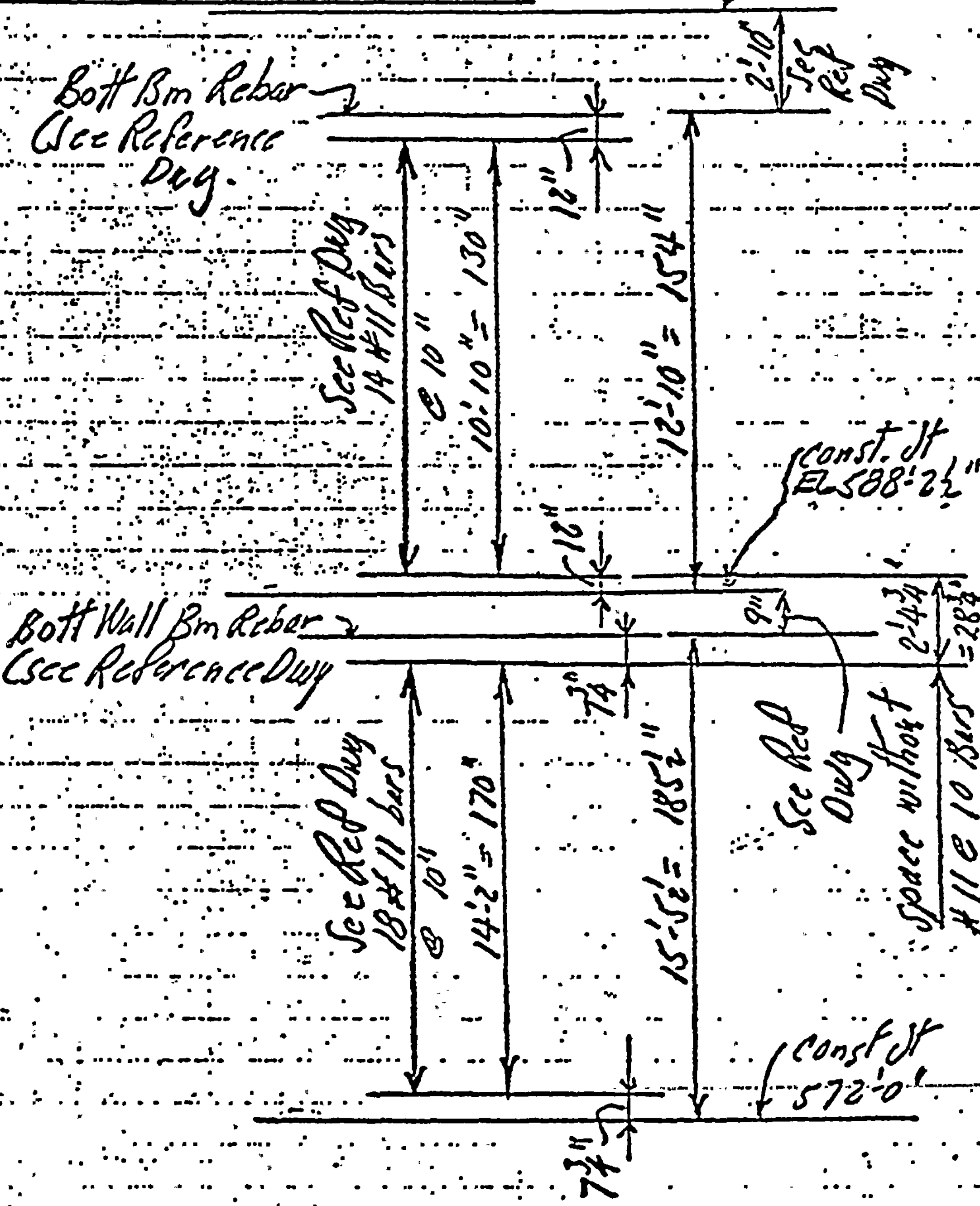


BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/19/83 Book No. SV-1022 Page No. ____
Drawing No. — Calc. No. 6.19-137 Sheet 5 of ____
By Hickey Checked LF Approved ____
Title WPPSS - Hanford No 2 - Reactor Bldg - Investigation - North Fuel Pool Wall

Investigation of Horizontal Wall Face bars in North Fuel Pool Wall in vicinity of Elev 588'-2½" Where Face Bars are replaced by Horizontal Wall Beam Steel

Reference: Bethlehem Steel Dwg: B&R File No 6809-00-0095 (Sheets 2&3)



number of #11 face bars required

$$= \frac{(28.75-10)}{10}$$

$= 1.875$ bars

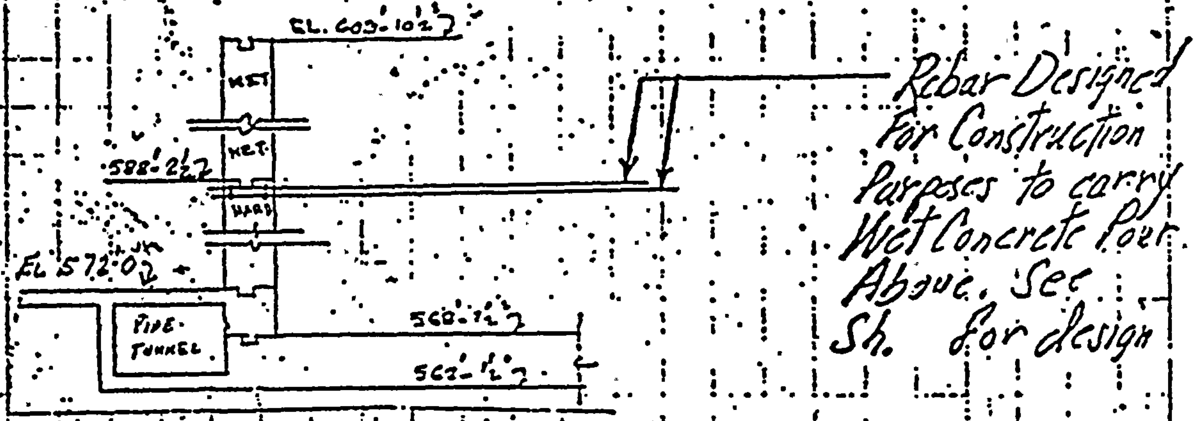
$= 1.875 \times 1.56$ in

$= 2.93 \text{ in}^2$

From above, in the vicinity of Elev 588'-2½ 2.93 in² of horizontal rebar is replaced by Horizontal Wall Beam Steel

Form BR 8002-2

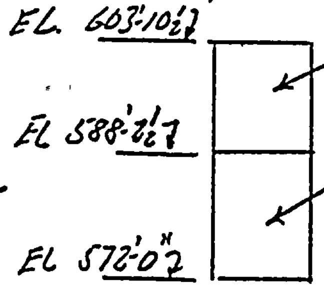
W.O. No. 3900-03 Date 10/19/83 Book No. SV-1022 Page No.
Drawing No. 2 Calc. No. 6-19-137 Sheet 6 of
By Hickey Checked LE Approved
Title WPPSS-Hanford No 2 Reactor Bldg Investigation - North Fuel Pool Wall



North Fuel Pool Wall - Construction Sequence

The above section shows the location of the wall beam steel in the vicinity of elev 588'-2½, and its functional use in the supporting of the wet concrete pour above elev 588'-2½. After this wet concrete hardened and cured, this primary function was served. The design of this reinforcing is found on sheet 7, and it is shown that there is sufficient reserve strength in this reinforcing to satisfy horizontal face wall reinforcing requirements during operating conditions. (See Sh. 8)

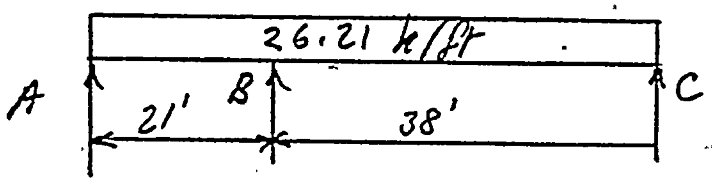
The following is a calculation for the design of a wall beam between elev 572'-0" & 588'-2½' for construction purposes to carry the wet concrete pour between elev 588'-2½' & Elev 603'-10½" (See Sh 6)



Wet Wt = .15 × 15.67 × 5 = 11.75

Cured dead Wt = .15 × 16.2 × 5 = 12.15

Factored Load = 0.9 × W + 1.3 C
(Ref FSAR, Table 3.8-15)
= 0.9 × 12.15 + 1.3 × 11.75
= 10.94 + 15.28
= 26.21 k/ft



	.64	.36	
+963	−963	+3154	−3154
−701	−1402	−789	−394
+262	−2365	+2365	−3548

Fixed End Moms

$FEM_{AB} = FEM_{BA} = 26.21 \times \frac{21^2}{12} = 963 \text{ kft}$

$FEM_{BC} = FEM_{CB} = 26.21 \times \frac{38^2}{12} = 3154 \text{ kft}$

Dist Fact

$Rel \frac{I}{L}_{(AB)} = \frac{38}{21} = 1.81$, $Rel \frac{I}{L}_{(BC)} = \frac{38}{38} = 1$

$Dist Fact_{AB} = \frac{1.81}{2.81} = .64$, $Dist Fact_{BC} = \frac{1}{2.81} = .36$

Design of Top Steel

$M = 3548 \text{ kft}$, $F = \frac{bd^2}{12{,}000} = \frac{60 \times (194.5 - 6.5)^2}{12{,}000} = 176.7$

$K = \frac{M}{F} = \frac{3548}{176.7} = 20$, for $f'_c = 4000$, $f_y = 60{,}000$; $a_u = 4.45$

$A_s = \frac{3548}{4.45 \times 188} = 4.24 \text{ in}^2$

BURNS AND ROE, INC.

W.O. No. 3900-03 Date 10/19/83 Book No. SV-1022 Page No. —
Drawing No. — Calc. No. 6.19-137 Sheet 8 of
By Hickey Checked LF Approved
Title WPPSS - Hanford No 2 - Reactor Bldg - Investigation - North Fuel Pool Wall

Investigation of North Fuel Pool Wall

The referenced calculation on sheet 7 for the design reinforcement requirement for construction conditions for the wall beam of the North fuel pool wall in the vicinity of Elev 588'-2½, is 4.24 in^2

Dwg S723, and the Bethlehem Steel placement drawing furnishes 22 #10
$= 22 \times 1.27 = 27.95$
used for min steel considera

Thus, of the 4 #10 Beam Bars nearest the face of the wall:

$\frac{4.24}{27.9} \times 4 \times 1.27 = 0.77\ in^2$ is required for strength for construction condition

4#10 → $5.08 - 0.77 = 4.31\ in^2$ is available for operating wall requirements

From sheet 5, 2.93 in^2 is required in the vicinity of 588-2½

Since 4.31 in^2 available > 2.93 required, wall design in the vicinity of El 588'-2½ is satisfactory.

Form BR 8002-2