645-3601

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January 27, 1975

University of Minnesota
Engineering & Construction Division
Physical Planning Office
26 Folwell Hall
Minneapolis, MN 55455

Attn: Mr Paul E Kopietz
Assistant Director of Planning

Dear Mr Kopietz

Subj: Load Test of Spandrel Beam
Oak Street Parking Ramp

Attached is our report concerning the load test recently performed on a typical spandrel beam from the referenced project. Load testing was performed in our laboratory in accordance with the criteria set forth by the American Concrete Institute.

The spandrel beam will be retained at our laboratory until further notice.

Respectfully

Brian Pashina
Civil Engineer

Brian R Dobie, P.E.
Coordinating Engineer

BRD/jb
cc: 1) Johnston & Sahlman Inc
Attn: Dr Paul Anderson

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TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

ENGINEERS AND CHEMISTS

662 Cromwell Avenue - St. Paul, Minn. 55114

REPORT OF: LOAD TEST OF SPANDREL BEAM

PROJECT: OAK STREET PARKING RAMP
UNIVERSITY OF MINNESOTA
MINNEAPOLIS, MINNESOTA

DATE: January 27, 1975

REPORTED TO: University of Minnesota
Engineering & Construction Division
Physical Planning Office
26 Folwell Hall
Minneapolis, MN 55455 Attn: Mr Paul E Kopietz

FURNISHED BY:

COPIES TO: 1) Johnston & Sahlman Inc
Attn: Dr Paul Andersen

LABORATORY No. 6-13852

SUMMARY

In the fall of 1973, diagonal cracks became evident in the spandrel beams of the then partially completed parking ramp at the Minneapolis campus of the University of Minnesota. Investigation of these cracks by the project structural engineer and others indicated there was a deficiency in the steel reinforcement of the members. Our firm was retained by the project architects and directed to load test a typical spandrel beam in our laboratory. This was performed and a report issued in mid-December, 1973.

The load test performed in 1973 assisted in illustrating the problems inherent in the spandrel beam and verified that positive corrective procedures would be required. It is our understanding that recommendations for corrections were then developed and employed.

Our firm was recently directed by the owner of the structure to perform a second load test on a typical spandrel beam. This test was to incorporate the corrective procedures and would constitute a portion of a comprehensive evaluation of the spandrel beam correction work.

Test Objective

Our firm was directed by Mr Paul Kopietz of the University of Minnesota to perform a second test on a typical spandrel beam. One beam of approximately 12 extra beams for the north half of the structure was sent to our laboratory. Nondestructive investigation of the specimen by use of an electro-magnetic Pachometer verified that this member was of the original steel reinforcement design.

We were requested to simulate in our laboratory the bearing conditions for the spandrel beams at the project site. Loads of the same type and magnitude as present at the project (full service dead load) were to be applied to the test specimen. After typical spalling and cracking was achieved in the spandrel test beam, we were directed to perform a standard load test in accordance with Building Code Requirements for Reinforced Concrete (American Concrete Institute 318-71), Chapter 20, Strength Evaluation of Existing Structures. In accordance with Provision 20.1, the strength investigation of the member was to be conducted by a combination of structural analysis by Johnson and Sahlman Inc and of load testing performed by our firm.

In addition to our performing the load test , our firm was directed to review the results of the materials testing and provide the owner with an opinion concerning suitability of the typical correction procedures. Also, we were to provide recommendations for any additional investigation deemed necessary.

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SUMMARY - Cont.

Test Procedure

Concrete columns simulating the column bearing areas at the project site were cast in our laboratory, together with footing pads and other test framework. The spandrel beam was then positioned on the designated bearing areas in a configuration similar to that at the project. Steel wide flange beams were employed to simulate the legs of the concrete double-tee floor panels at the structure. The steel beams were simply supported but were prevented from moving outward as the spandrel beam rotated outward. In addition, the top surface of each of the Neoprene bearing pads was securely fixed to each respective steel beam, requiring that the bearing pad either deform or slip on the concrete surface with any movement in the spandrel beam.

During testing of the spandrel beam, dial indicators were employed to determine movement at various locations in the beam. In addition, strain gauges were utilized to monitor strain at the surface of several locations on the beam.

The full service dead load was held on the spandrel beam for a period of time longer than the minimum time required by the American Concrete Institute load test procedures in order to allow the spalling and cracking of the beam to more fully simulate the actual field conditions. Then, the designed correction haunches were grouted in identical to actual field conditions established recently at the project. The full ACI test load was achieved in more than four approximately equal increments by means of calibrated hydraulic jacks. Deflection and strain data were obtained at each load increment. Return to dead load and rebound data were also obtained, although deflection under full test load did not require rebound in the test. After performance of the load tests, nondestructive testing procedures were employed to evaluate the member, and concrete cores were obtained in order to obtain additional material strength data.

Test Results

The American Concrete Institute load test procedures are concerned primarily with deflection data. The deflection information and discussion are contained in a following appendix. However, it should be noted that the total mid-span movement downward and outward was approximately 0.067 in. and 0.069 in., respectively. In both cases, the residual downward and outward deflections after 24 hours of rebound were each 0.037 in. The mid-span movement of the beam was less than the total deflection allowed by American Concrete Institute test criteria (0.082 in. in the vertical direction and 0.121 in. in the horizontal direction). Therefore, a 75 per cent recovery of deflection in the member was not required. The reader will recall that neither the deflection nor the rebound criteria were met in the first test performed on the spandrel beam in 1973.

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SUMMARY - Cont.

Of significant importance during this test was that additional end bearing spalling and appreciable outward movement of the end bearing regions of the beam did not occur after the corrective procedures were incorporated in the test beam construction.

We should note that a pair of diagonal cracks did develop in the vertical portion of the spandrel beam under the full ACI test load. In addition, what appeared to be the first signs of horizontal crack development in the vertical leg were also noted during the final hours of the full ACI 24 hour test load. Nondestructive testing of the vertical portion of the end bearing regions of the spandrel beam indicated that no internal splitting of the member in the direction of the axis of the beam had occurred. This test information and all concrete core data are included in an appendix of this report.

Recommendations

Based on the load test data, it is our opinion that the performance of the structural member now satisfies the test requirements set forth by the American Concrete Institute. The deflection of the member falls within the prescribed guidelines of the test evaluation criteria. Although we did observe the diagonal and other cracking previously mentioned above, we would not consider the magnitude of this cracking to be "visible evidence of failure" as referenced in Paragraph 20.4.5 of the ACI 318-71 Building Code.

The strain gauge data and visual observations during the testing both attest to the fact that the concrete materials comprising this spandrel beam definitely are working in tension. This would be in contrast to typical cracked-section design of flexural non-prestressed concrete members. Project Drawing Number $S6_B$, dated June 10, 1974, Note Number 11 makes reference to the spandrel beam cracks and states "apply sealant bead along cracks...". Since the concrete is in tension and it has now been demonstrated by test that the haunches should permit the beam to function in a reasonably elastic manner under service live loads, it is our opinion that all diagonal cracking and all other cracking in the vicinity of the ends of the beams should be pressure-grouted with an epoxy type material having a minimum bond strength of 1,000 psi. We advise that a full-depth pressure grout of prescribed bond strength be used in lieu of a surface bead of sealant because the load test demonstrates that tensile strength in the concrete and therefore integrity of the concrete member is important in the performance of the beam. A load test is a short-term method of determining the safe capacity of a structural member. However, cracking in a concrete structural member is also time-related and this aspect relates to long-term performance and ultimately the useful life of the construction.

Normally pressure grouting of a member under tension simply results in the development of another crack in the concrete construction parallel to and very near the original repaired crack. However, the very inclusion of the beam modifications presupposes that the haunch assemblies will return the concrete materials to a

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SUMMARY - Cont.

workable range of tension. Pursuing this approach, development of companion parallel cracking would then be less likely.

In addition to restoring the integrity of the concrete construction, the epoxy pressure grouting would attempt to provide as much protection for the steel reinforcement as originally had been intended. Even with these pressure grouting modifications, we hold as doubtful that any predetermined useful life of the structure could now be fully realized.

The end cracking of the spandrel beams at the project site does present a problem with regard to pressure grouting. Therefore, we recommend that additional field investigation be performed to determine extent of splitting in the end regions of the beams and to develop the most feasible method of repair. We should note that both this investigative work and any epoxy pressure grouting can be performed while the structure is in service, since neither of these items affects the immediate safety of the construction but only the long-term performance and safety of the structure.

Our testing indicated that the vertical bolt extending through the haunches and into the beam is both in shear and in tension. The test also demonstrates to us the importance that is being placed on this one item in the modifications. Since the bolt is fully embedded in the grout applied on the haunch and this grout extends completely to the column, we are concerned about the performance of the entire construction over anticipated full temperature cycles. Thermal expansion and contraction of the spandrel beams could conceivably cause concern for the long-term performance of the beam-bolt-grout-haunch-column assembly, unless some other provision has been made in the construction, of which we are not presently aware.

TEST INFORMATION

Information concerning the test beams and bearing pads is similar to that reported in the initial test program. The beams for both the initial and the present load tests had been cast at the same time. Therefore, one significant difference between the test beams was that the present beam had a field cure of approximately 13 months longer than the initial test beam. In addition, this beam received the service dead load after considerably longer cure than the beams in the structure. However, in terms of application of the haunch modification and performance of the ACI load test on the modified spandrel beam setup, the age of the test beam vs. the age of the project beams is approximately the same.

Test Setup

Please refer to Appendix A for information concerning the test setup. Also, photographs illustrating the test setup are included in the last appendix.

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TEST INFORMATION - Cont.

There are several aspects worth noting concerning the recent load test . The concrete columns and pads were heavily reinforced and utilized a minimum 7000 psi compressive strength concrete material in order to achieve end reactions which would exhibit negligible deflection or movement under the full test load. Furthermore, these pads were securely fastened to one another in order to assure against spreading or other movement under full load. The bearing pockets and other end bearing conditions on the test columns precisely duplicated those at the project. As has been previously mentioned, the Neoprene bearing pads for the individual double-tee legs were securely fastened to the steel beams simulating the double-tee legs. These steel beams were simply supported but were not permitted to undergo any lateral movement with outward deflection of the spandrel beam. This was achieved by means of end-ties on the beams and steel blocking back to the columns and laboratory loading frame. By virtue of the simply supported condition and a preselected stiffness in the steel wide flange members, a vertical oriented load was established at each of the double-tee bearing locations during the load test .

A 6 in. wide flange beam was employed to simulate the concrete topping in the ramp construction. This beam was secured to the concrete columns and blocked at mid-point to the laboratory loading frame. At the prescribed time during the load test, a 1 in. space between the steel beam and spandrel beam was grouted utilizing a minimum 4000 psi compressive strength sand-Portland cement grout. The location of the bottom edge of the grout corresponded to the bottom surface of the designated concrete topping which was placed on top of the double-tee concrete panels. The thickness of the simulated topping in the load test was 6 in. (the full width of the beam flange), in contrast with approximately 8 in. thick topping at the project. In addition, the six 3/4 in. coil rod inserts behind the topping in the test beam did not receive threaded steel rod, whereas we understand the beams at the project utilized threaded rod, the rod extending approximately 4 ft into the topping from the insert locations. Based on our judgment that the spandrel beam applied a compressive load to the topping at this location, that the coil rod inserts received no shear forces at the topping-spandrel beam interface, and that the coil rod inserts were located in the spandrel beam such that they could not develop any significant couple, we deleted these from the test setup. In our judgment, also, the variation in topping thickness does not play a significant role in determining adequacy of the spandrel member.

Instead of drilling holes in the concrete columns and positioning the haunches in a procedure similar to the project, we cast the bolts into the column and placed the haunches over the pre-positioned bolts. Using this procedure, it was necessary to enlarge the bolt holes which had been predrilled in the back plate. Once the haunches were set in the exact locations prescribed for the project beam , we secured the haunch assemblies to the columns. Since additional cutting had been done on the haunch back plates and no filling was performed, we machine-drilled 1/2 in. thick steel plates to the precise bolt tolerance, fully welded these washers to the back plates of the haunches, hand torqued the nuts in a manner similar to the project repair work, and then fully welded the nuts to the steel washers. In addi-

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TEST INFORMATION - Cont.

tion, we then provided solid blocking between the bottoms of the haunches and the laboratory footing pads by means of forming and placing a solid, rapid setting, 5000 psi minimum compressive strength commercial grout. The installation of the haunches is illustrated in the accompanying photographs of the last appendix of this report. Since our firm was directed to investigate only the performance of the spandrel beam, it was our judgment that the modification work incorporated into the haunch application work would not affect the performance of the haunches or bolts and would neither assist or hinder the spandrel beam in the performance of the load test.

Drilling was performed on the bottom of each end of the spandrel beam at the prescribed haunch slot locations. A 1 in. diameter by 6 in. long Qwik-Bolt was inserted into each of the two drilled holes. At the prescribed time during the test, these bolts were tightened and a Five Star grout with a minimum 5000 psi compressive strength was placed in the appropriate location noted by project drawing Number S6B, dated June 10, 1974. Two slotted pieces of thin sheet metal were drawn around each of the Qwik-Bolts so as to just cover the slot in the top plate of each of the haunches and thus prevent the grout from intruding into this slotted area. The grout was allowed to fully cure, after which time a 48 hour dead load hold period was maintained prior to commencement of the ACI load test.

Two systems (of three hydraulic rams connected to one pressure manifold and calibrated gauge and pump) were utilized in the load test. The six rams were positioned, one on each of the six simulated double tee legs. The end reactions calculated from the known span lengths and applied concentrated hydraulic loads comprised the test increments. Please refer to an attached appendix for chronology information, together with our observations during the entire test schedule. Please note that the calculated dead load was maintained on the spandrel beam for a period of approximately 11 days before the actual ACI load tests commenced.

Deflection Measurements

Dial extensometers accurate to at least 0.001 in. were mounted in order to record movement at 22 different locations on the spandrel beam. During dead load application (Stages 0 through 4) we recorded that compression of the end bearing Vosco pads did occur, together with a slight movement outward of the end reactions. This rotation noted by the dial indicators caused end spalling of the concrete materials in the spandrel beam, similar to the first load test performed in 1973 and similar to the construction conditions. The graphical representation of movement at each of the dial indicator positions is included in one of the attached appendixes. The mid-span movement of the beam in the horizontal and vertical directions, referred to earlier in this report, was derived from deflection information at Dials 14 and 15, respectively. These dial readings were corrected by average end movement in each of the two directions.

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TEST INFORMATION - Cont.

Strain Gauges

Several strain gauges were mounted on the spandrel beam at various locations noted in one of the attached appendixes. Two types of gauges were used: 2 in. long foil-type Micro-Measurement electrical resistance gauges having a resistance of 120 ohms utilizing a gauge factor of 2.11 nominal and 5 in. long wire type BLH electrical resistance gauges having a resistance of 120 ohms and utilizing a gauge factor of 2.09 nominal. Prior to commencement of the ACI load tests, three 2 in. long gauges were mounted in the vicinity of a 5 in. long gauge to check average strain across the longer gauge.

It should be noted that the "0 strain" condition of the strain gauges was set when the spandrel beam was in a simply supported condition on its designated concrete bearing pockets in the columns. At this time, no double tee load had yet been applied.

As will be seen later in this report, the modulus of elasticity for the concrete in tension was found to be approximately equal to the modulus of elasticity of the same concrete in compression for the lower stress ranges. Additional testing of the concrete is still in progress. It is apparent that the spandrel beam concrete materials failed in tension when the strain exceeded approximately 600 in. per in. Therefore, it is apparent that the concrete behaved elastically only in the lower range of tensile stress and the modulus changes as the material approaches failure.

Nondestructive Testing

Upon completion of the ACI load tests, we removed the concrete spandrel beam from the column bearing pockets and examined the end regions of the beam with ultrasonic testing equipment. The ultrasonic meter is equipped with transmitting and receiving low frequency sonic transducers, togather with monitoring instrumentation. Internal fracturing and discontinuities in the test item are revealed by higher required sonic transmission time. The test results from the sonic scanning are shown in one of the attached appendixes. The sonic transmission of both ends of the test beam compared with the 8 in. thickness of the test member indicate that no discontinuities (splitting or other fracturing) had developed in the spandrel beam during the load test.

Concrete Coring and Testing

Subsequent to load testing of the spandrel beam, additional mapping of steel reinforcement in the structural member was performed and eight 4 in. diameter cores obtained. Since the areas of concrete cored had been precisely located and this recent load test was terminated at load stage 10 instead of the appreciably higher load stages of the previous 1973 test, it is our opinion that no microcracking had occurred in the areas sampled and the concrete materials can therefore be considered sound and representative. Information concerning various tests performed on the concrete core specimens

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TEST INFORMATION - Cont.

is included in one of the following appendixes.

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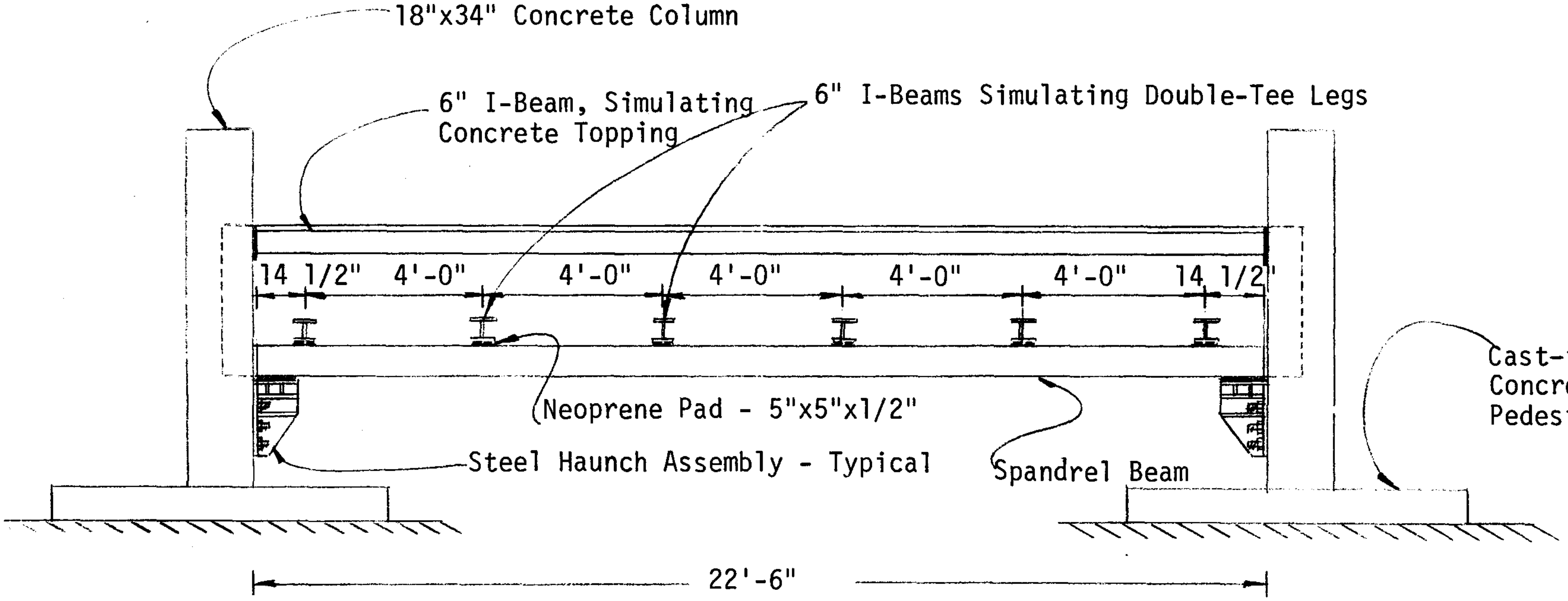
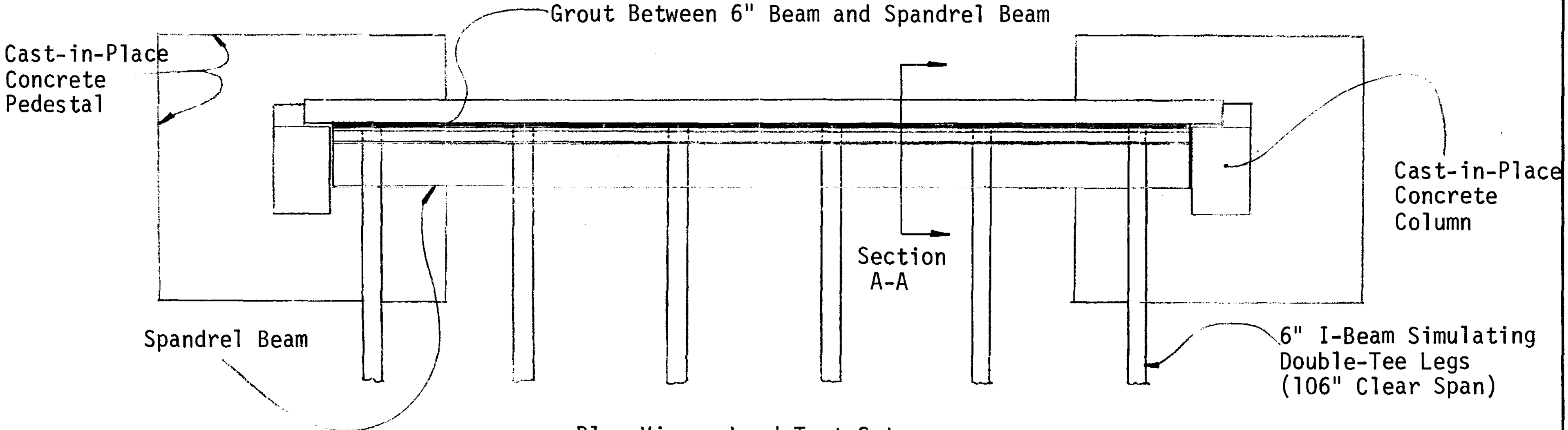
ACI LOAD TEST SETUP



Elevation - Load Test Setup



Plan View - Load Test Setup

Scale: 1/4"=1'-0"

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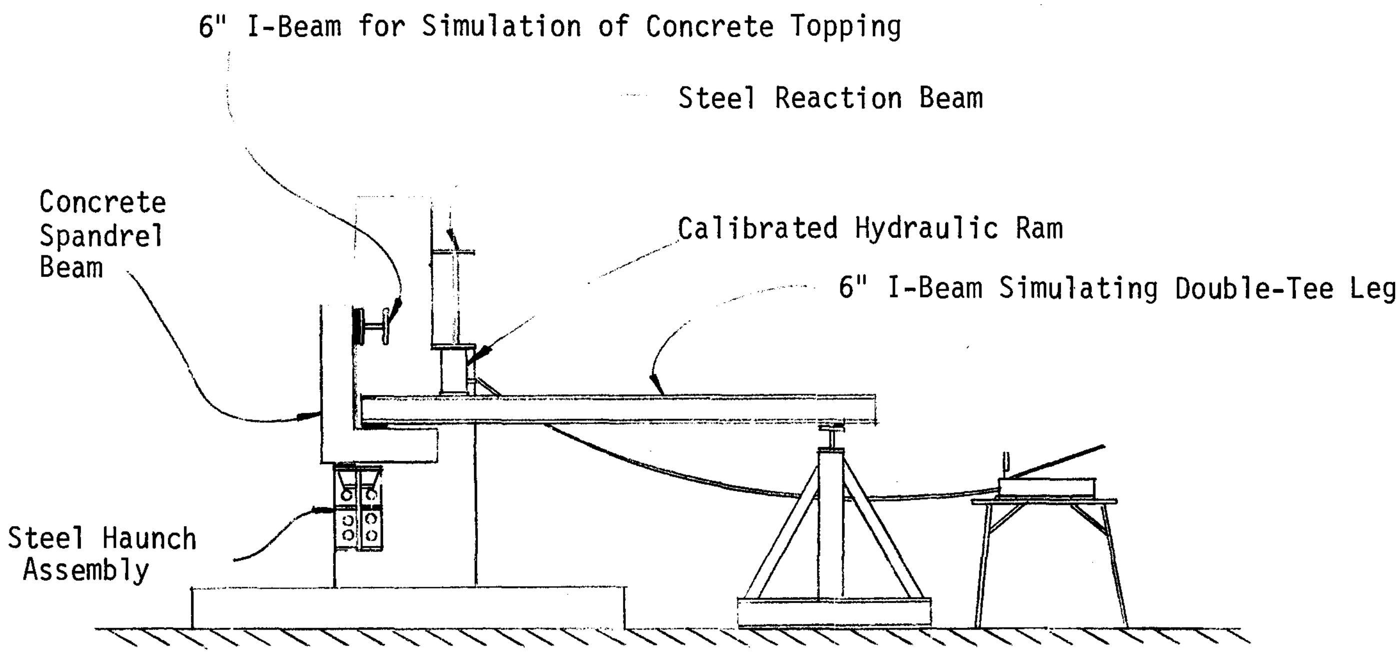
ACI LOAD TEST SETUP (Continued)



Section A-A - Load Test Setup

Scale: 1/4"=1'-0" Laboratory Number 6-13852

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LOADING PROCEDURE

Load Increment	P-# (Load Applied)	P-# (Cumulative Load Applied)	R_V-# (Calculated End Vertical Reaction	R_H-# (Calculated End Horizontal Reaction	Remarks
#1	2,500#	2,550#	13,570#	1,660#	
2	2,550	5,100	21,220	2,890	Weight of double tees
3	2,390	7,490	28,400	4,030	
4	2,390	9,880	35,570	5,170	Weight of conc. topping, record immed., wait 48 hr, record again
5	1,740	11,620	40,790		
6	1,740	13,360	46,010		Working live load = 30 psf
7	950	14,310	48,860		
8	950	15,260	51,710		
9	950	16,210	54,560		
10	950	17,160	57,410		ACI test load = 63 psf live load, record immed., wait 24 hr, record again; remove loads 5-10, record immed., wait 24 hr, record again.

Lab. No. 6-13852

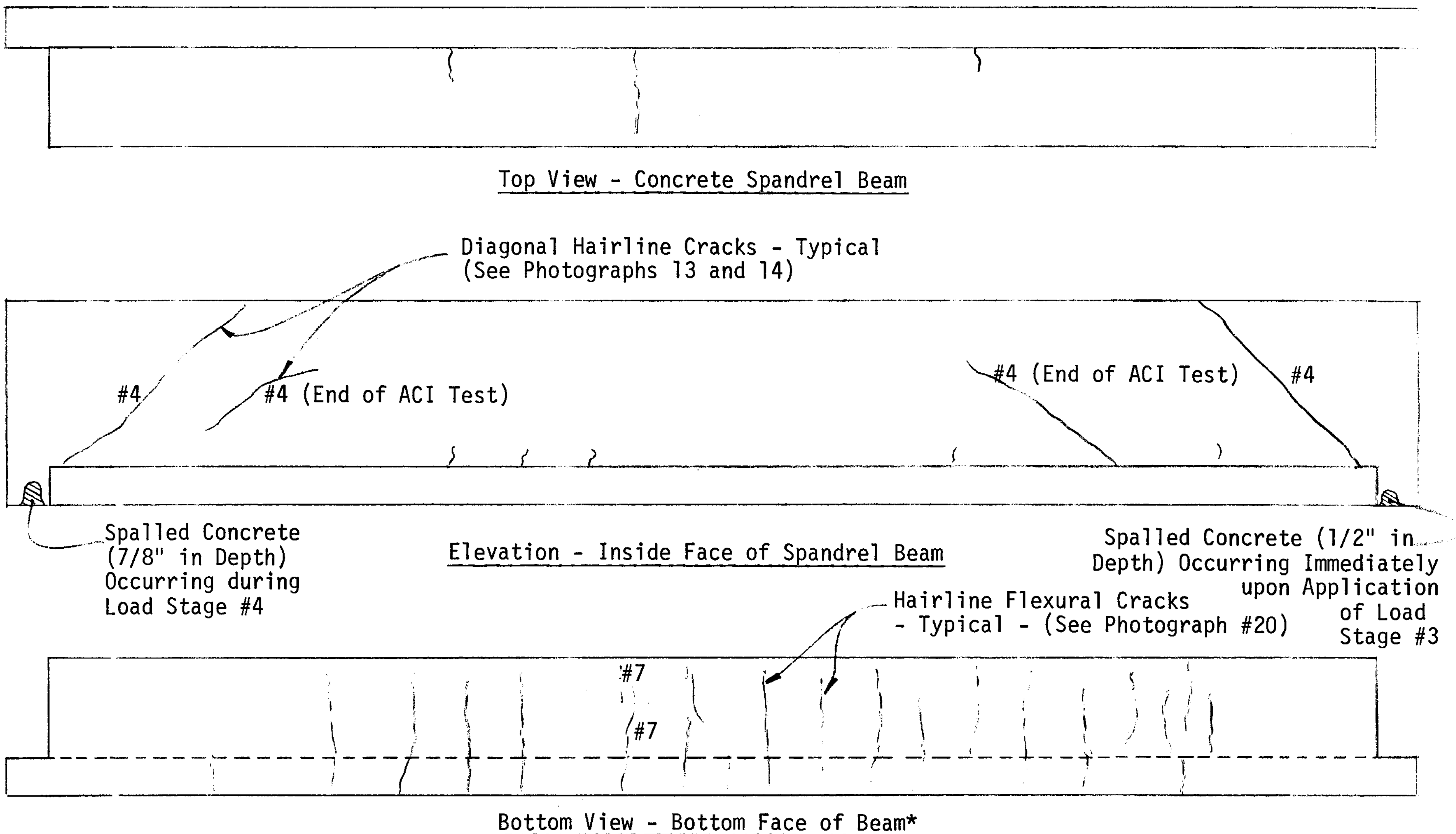
CHRONOLOGY OF SPANDREL BEAM LOAD TEST

DAY	DATE	TIME	STAGE	REMARKS
Wednesday	12-11-74	10:00		Beam delivered to the laboratory and positioned on concrete pedestal
Monday	12-16-74			Beam positioned on concrete, cast-in-place, columns.
	12-24, 26, 27-74			Strain gauges mounted on beam.
Friday	12-27-74	9:00 a.m.		Zero readings recorded on all strain gauges and dial indicators
Friday	12-27-74	9:05 a.m.	0	START OF LOAD TEST
		9:15 a.m.	1	
		10:15 a.m.	2	No visual damage of beam observed.
		11:15 a.m.	3	Spalling observed at the south end reaction immediately after application of load stage #3. No other damage noted in beam.
Monday	12-30-74	10:30 a.m.	4	
		12:00 (Midnight)		Diagonal hairline cracks (on the inside face of the stem) observed at both end reactions.
Tuesday	12-31-74	4:15 p.m.		Grout placed between concrete beam and 6" steel I-beam, to simulate the concrete topping.
				Diagonal cracks observed to have increased in separation (See Photograph #14)
Monday	1-6-75			Spalling observed at the north end reaction.
		9:00 a.m.		Additional grout also placed between haunch and concrete slab at each end reaction.
Tuesday	1-7-75	1:30 p.m.		Grout placed between steel haunch assembly and spandrel beam.
Friday	1-10-75	10:15 a.m.	5	START OF ACI LOAD TEST
				Hairline cracking observed in grout between steel haunch assembly and spandrel beam (Both end reactions)
				(See Photograph #22)

DAY	DATE	TIME	STAGE	REMARKS
Friday	1-10-75	11:30 a.m.	6	Hairline flexural cracks observed on underside of beam (See Photograph #20)
		1:45 p.m.	7	Hairline cracking observed on both the shelf and underside of beam.
		2:45 p.m.	8	
		3:00 p.m.	9	
		3:45 p.m.	10	No new cracking observed. ACI test load in place for 24 hr.
Saturday	1-11-75	4:45 p.m.	8	
		5:15 p.m.	6	
		5:45 p.m.	4	
Sunday	1-12-75	5:45 p.m.	4	END OF ACI LOAD TEST (END OF 24 HR REBOUND). Fine hairline diagonal cracks observed at each end reaction. Also, several small areas of horizontal micro-cracking observed above shelf on stem.

END OF TEST

CONDITION OF SPANDREL BEAM AFTER COMPLETION OF ACI LOAD TEST



*Note: All Cracks Noted in the Above View were Observed to Occur at Load Stage #6 (With the Exception of the Two Noted)

Scale: None

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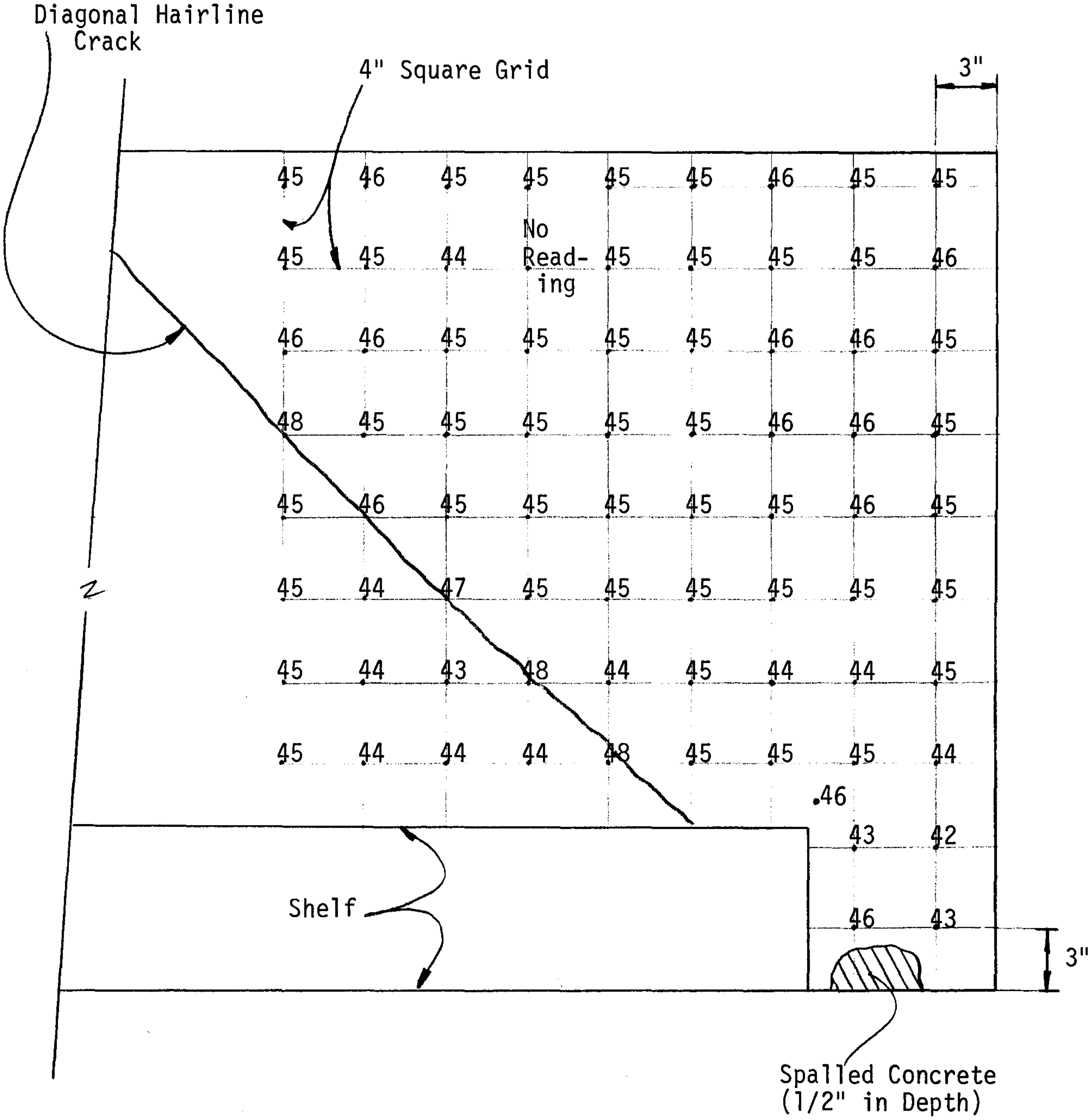
ULTRA-SONIC TEST RESULTS



Elevation - South End Reaction - Inside Face

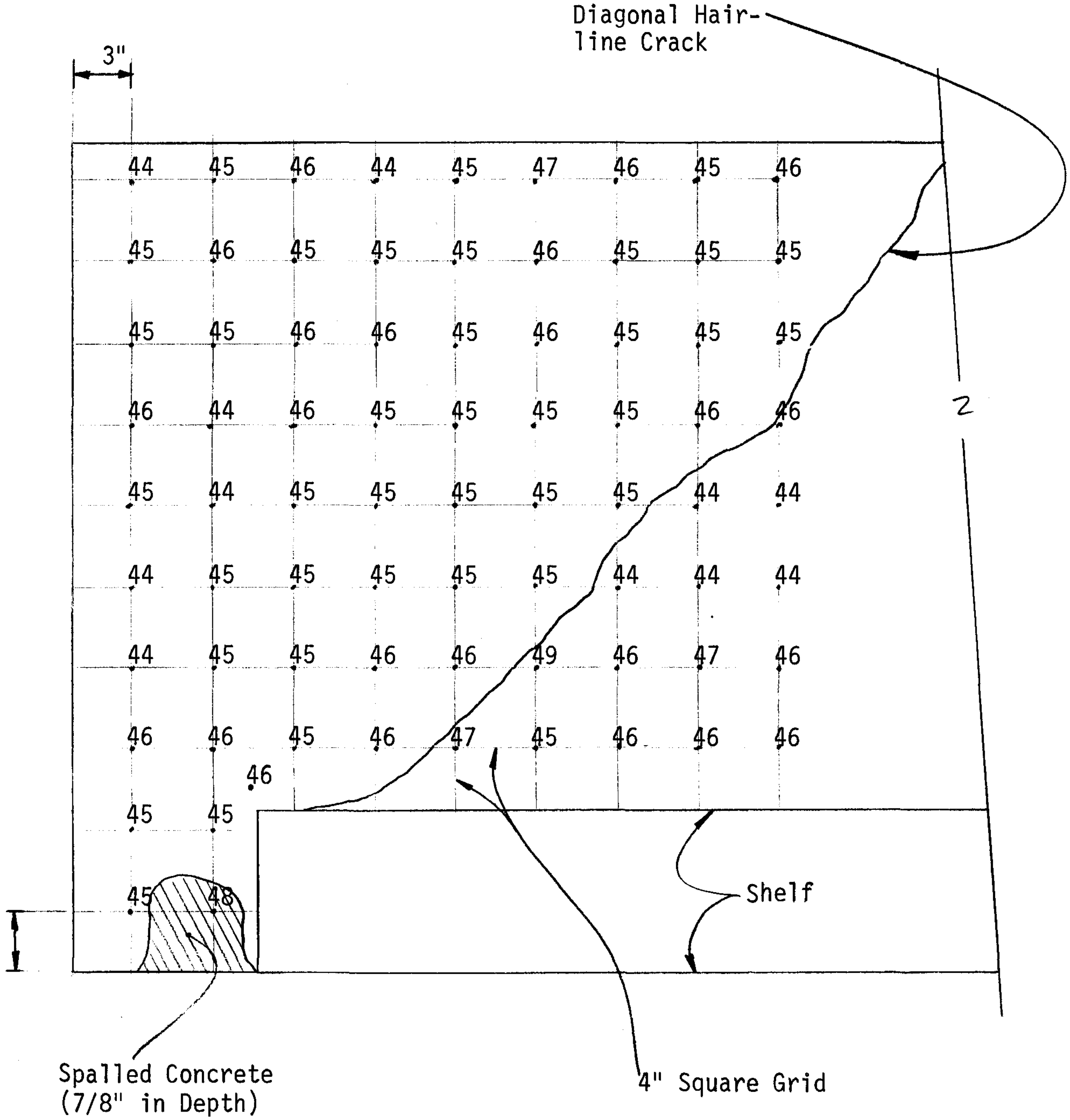
Legend

48 Ultra-Sonic Test Location and Corresponding Reading (in Micro-Sec)

Scale: 1 1/2" = 1'-0"

Laboratory Number 6-13852

ULTRA-SONIC TEST RESULTS



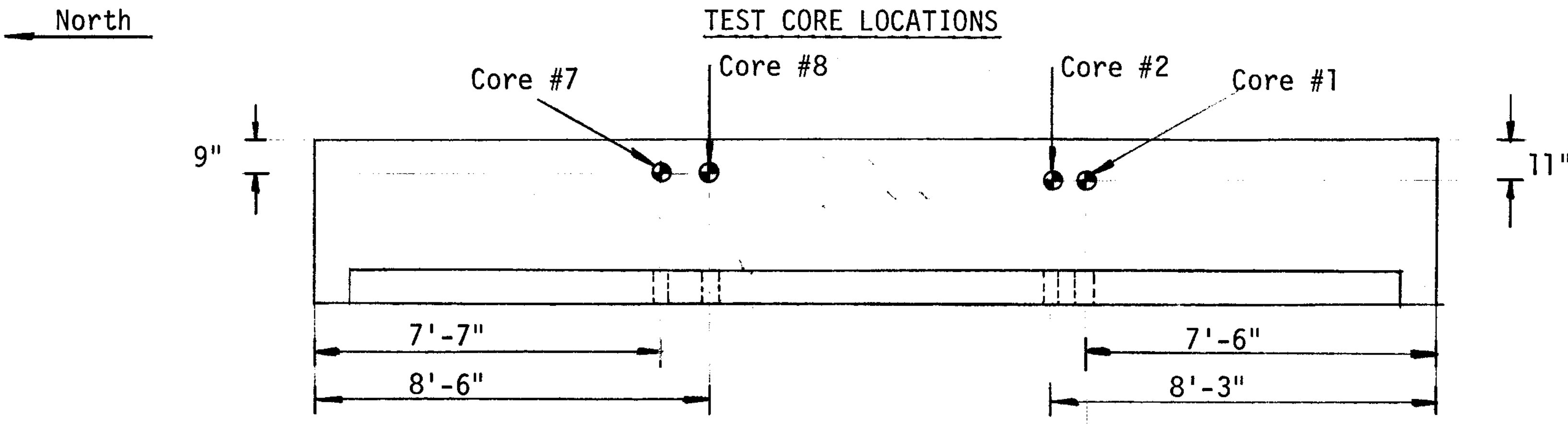
Elevation - North End Reaction - Inside Face

Legend

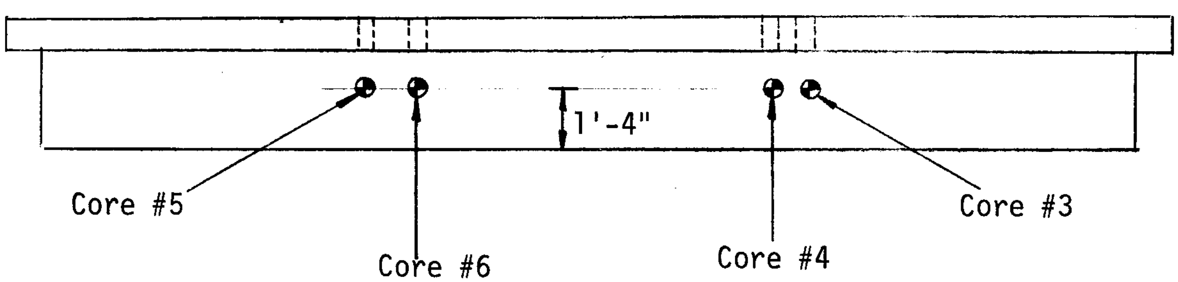
.46 Ultra-Sonic Test Location and Corresponding Reading (in Micro-Sec)

Scale: 1 1/2" = 1'-0" Laboratory Number 6-13852

TEST CORE LOCATIONS



Side View - Inside Face of Concrete Spandrel Beam



Top View - Concrete Spandrel Beam

Scale: None

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TEST CORE DATA

SPLITTING TENSILE STRENGTH

(ASTM:C42-68 Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete)

Core Number	2	6	8
Diameter, in.	4.02	4.02	4.02
Length, in.	7.22	7.88	6.92
Load at Failure, lb	42,000	41,000	38,500
Tensile Stress, psi	920	825	880
Average Tensile Stress, psi		875	

COMPRESSIVE STRENGTH

(ASTM:C42-68 Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete)

Core Number	3	5	7
Diameter, in.	4.02	4.02	4.02
Length, in.	8.01	8.02	7.04
Density (saturated), pcf	147.3	147.3	147.3
L/D Ratio	2	2	1.75
Correction Factor	1.0	1.0	0.98
Load at Failure	118,000	109,250	-
Corrected Compressive Strength, psi	9300	8610	
Average, psi	8955		
Modulus of Elasticity, psi			5.3×10^6

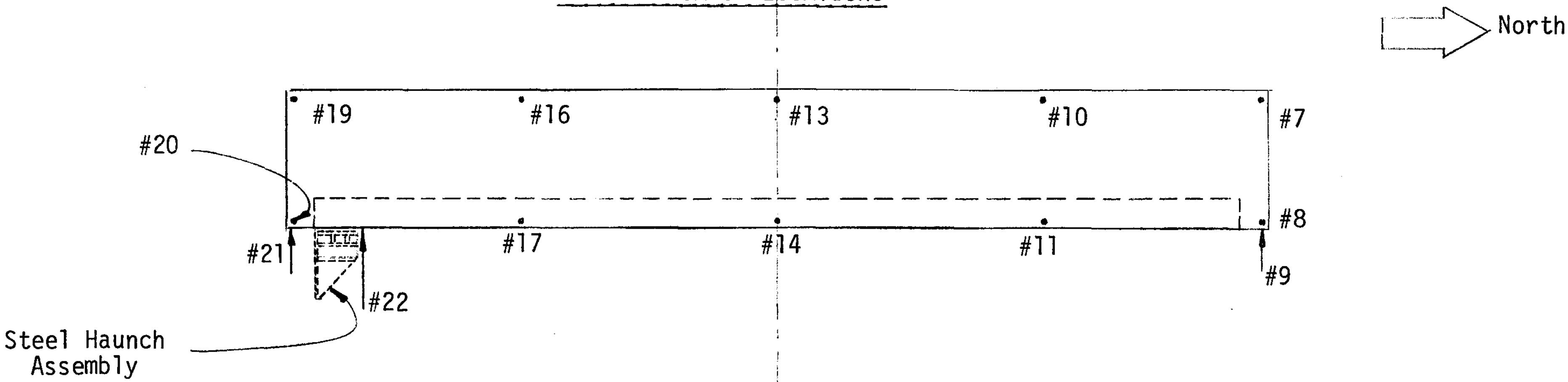
MODULUS OF ELASTICITY IN TENSION

Core Number	1	4
Diameter, in.	4.02	4.02
Length, in.	7.51	8.01
*Modulus of Elasticity, psi	*5.4×10^6	*5.4×10^6

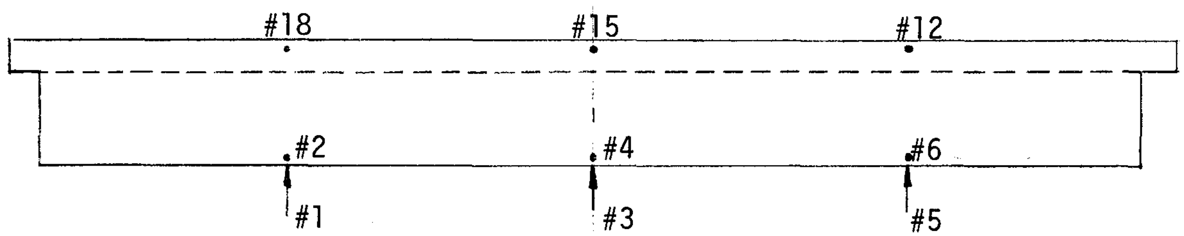
*Note: The above values for the modulus of elasticity in tension was determined in our laboratory up to a unit tensile stress of approximately 200 psi, and a total strain of 40 microinches/inch.

Lab. No. 6-13852

DIAL INDICATOR LOCATIONS



ELEVATION - EXPOSED AGGREGATE FACE OF SPANDREL BEAM

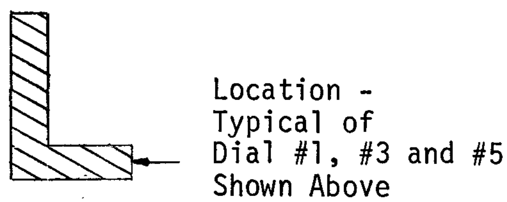


PLAN VIEW - BOTTOM FACE OF SPANDREL BEAM

Legend

• Location of

→ Dial Indicator - Typical



SECTION - CONCRETE SPANDREL BEAM

Laboratory Number 6-13852

Scale: None

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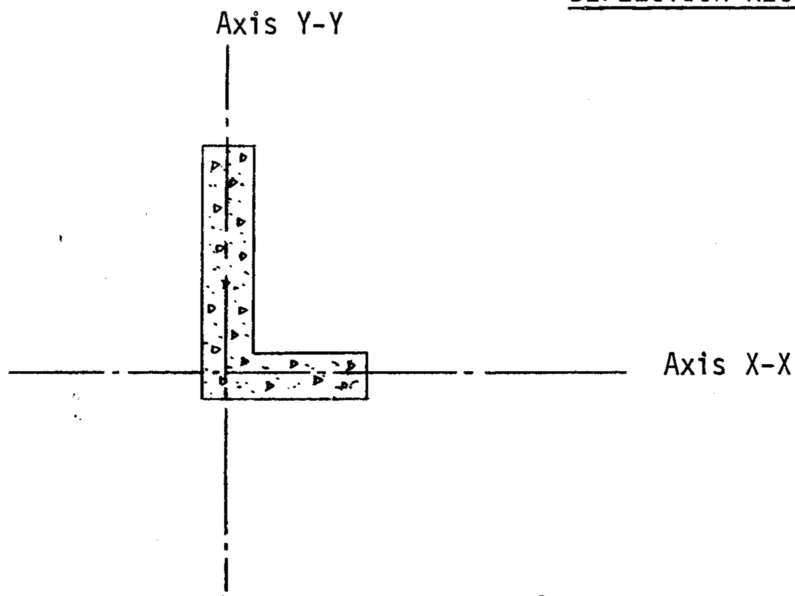


Section-Concrete Spandrel Beam

According to Paragraph 20.4.6(b) of the Building Code Requirements for Reinforced Concrete (ACI 318-71) the requirement for deflection recovery (for a flexural member) shall be waived, provided the maximum measured deflection is less than $L_t^2/20{,}000h$.

where L_t = span of member under load test, in.
h = overall thickness of member, in.

We were directed by the structural engineer (hired by the owner) to use the following values for the above:

L_t = center to center distance of the steel brackets, in.
= clear span - 12" = 22'-6"-12 = 21'-6"
= 258 in.
h_{X-X} = 27.5 in.
h_{Y-Y} = 40.5 in.

Our calculations, utilizing the above values are as follows:

(A) Analysis Along Axis_{X-X} (shown above)

$$L_t^2/20{,}000h_{X-X} = \frac{(258)^2}{20{,}000\,(27.5)} = 0.121"$$

Since the maximum deflection observed = 0.069 in. < 0.121 in. in Axis_{X-X}

** The Requirement for Recovery is Waived **

(B) Analysis Along Axis_{Y-Y} (shown above)

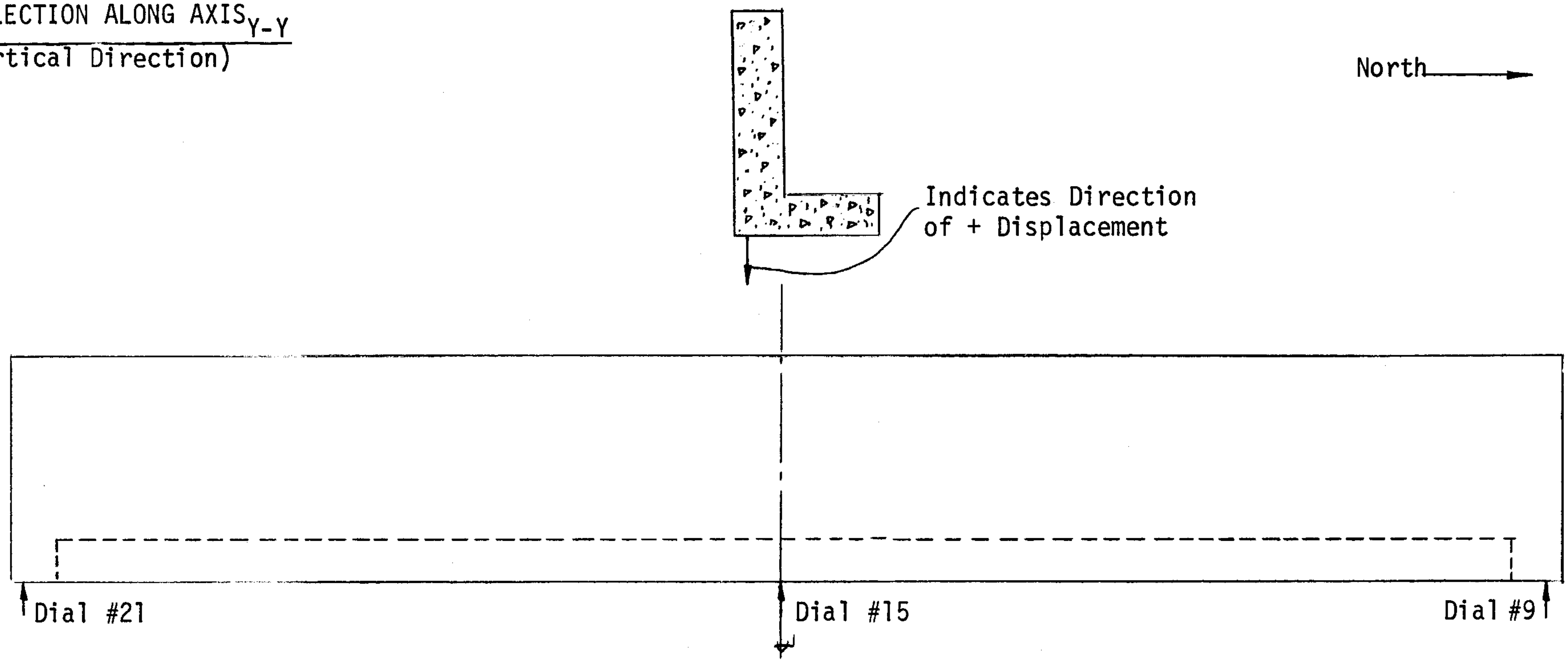
$$L_t^2/20{,}000h_{Y-Y} = \frac{(258)^2}{20{,}000\,(40.5)} = 0.082"$$

Since the maximum deflection observed in Axis_{Y-Y} = 0.067 < 0.082

** The Requirement for Recovery is Waived **

Lab. No. 6-13852

DEFLECTION ALONG AXIS$_{Y-Y}$
(Vertical Direction)



Elevation-Exposed Aggregate Face of Test Beam

Deflection (relative to readings taken at Load Stage #4)-just prior to #5

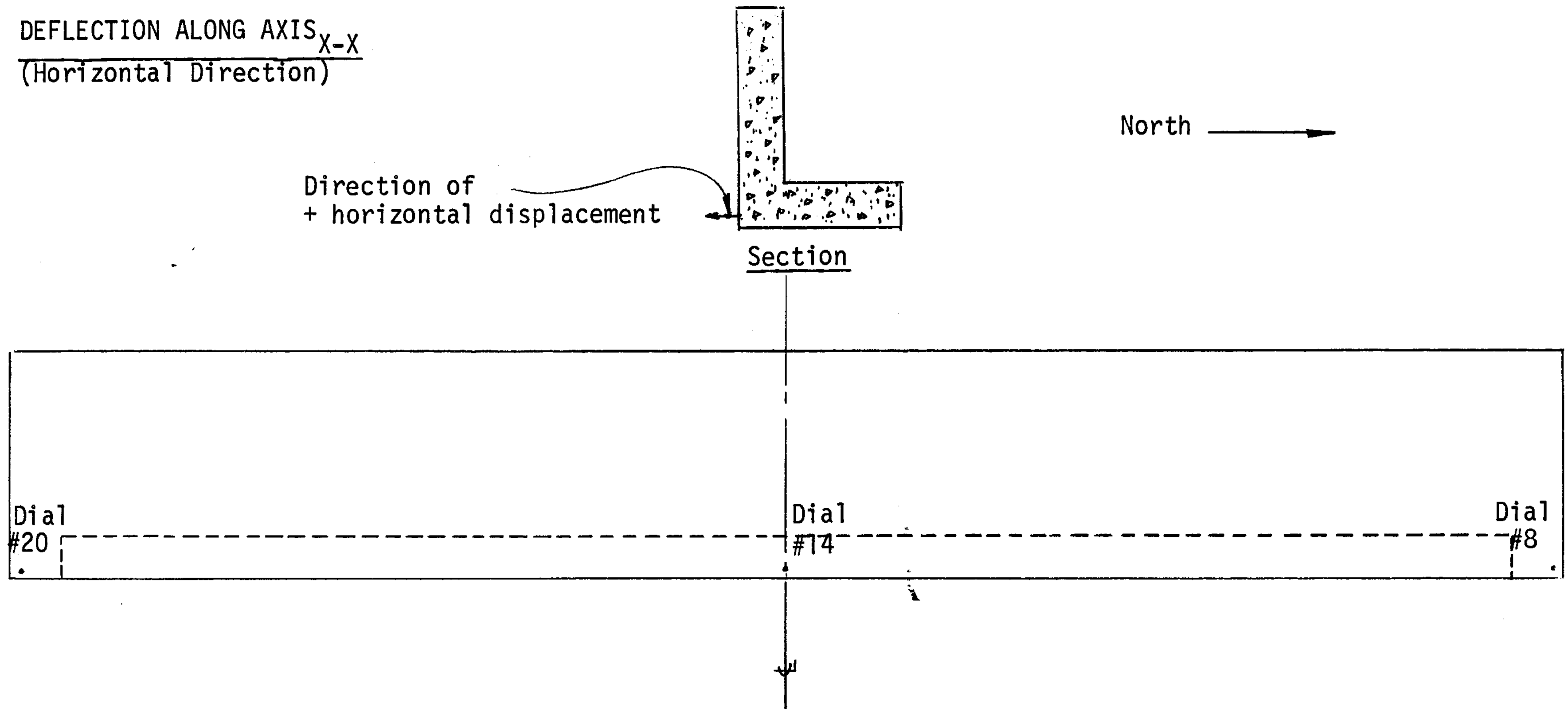
Loading Condition	Dial #21	Dial #15	Dial #9	Maximum Midspan*
Load Stage #4 (just prior to #5)	0	0	0	0
Load Stage #10 (end of 24 hr test period)	+0.002	+0.068	0	0.067
Load Stage #4 (end of 24 hr rebound)	0	+0.037	0	0.037

*Note: The maximum midspan deflection corrected to account for the deflection of the end bearing points.

Lab. No. 6-13852

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

DEFLECTION ALONG AXIS $_{X-X}$
(Horizontal Direction)



Elevation-Exposed Aggregate Face of Test Beam

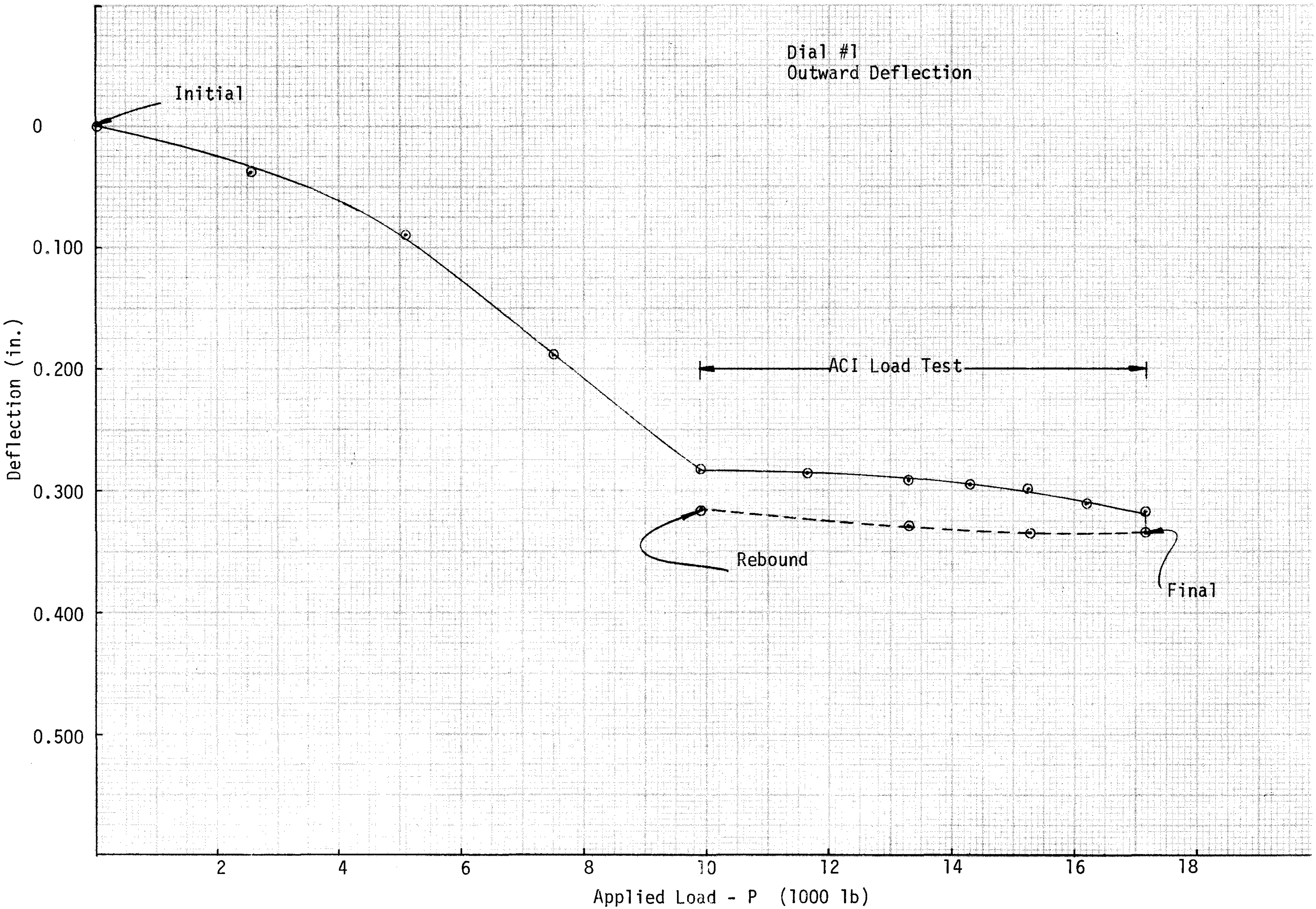
Deflection (relative to readings taken just prior to Load Stage #5)

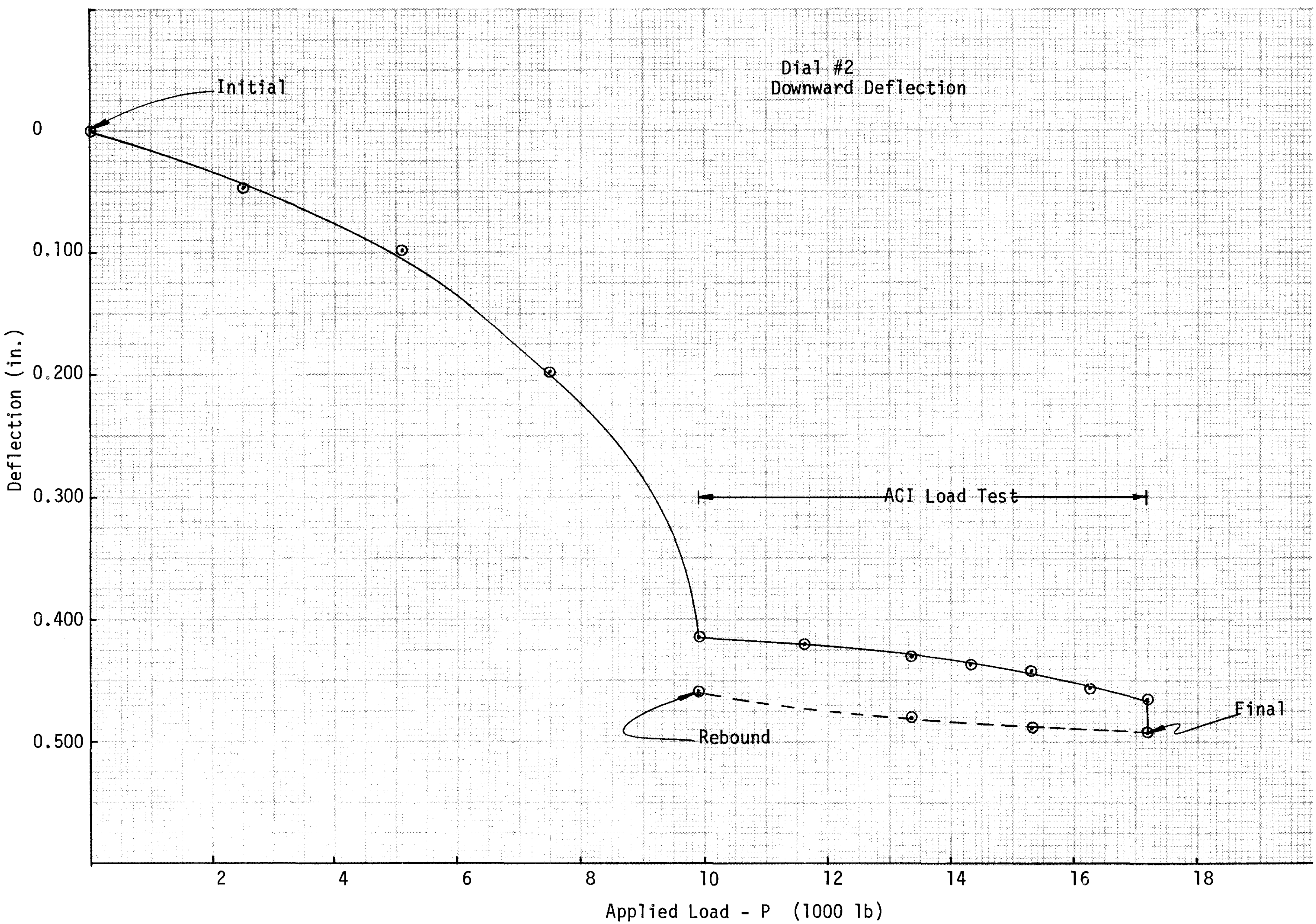
Loading Condition	Deflection, in. Dial #20	Dial #14	Dial #8	Maximum Midspan*
Load Stage #4 (just prior to #5)	0	0	0	0
Load Stage #10 (end of the 24 hr ACI test period)	+0.009	+0.075	+0.004	0.069
Load Stage #4 (end of 24 hr rebound)	+0.006	+0.041	+0.002	0.037

*Note: The maximum midspan deflection corrected to account for the deflection of the end bearing points.

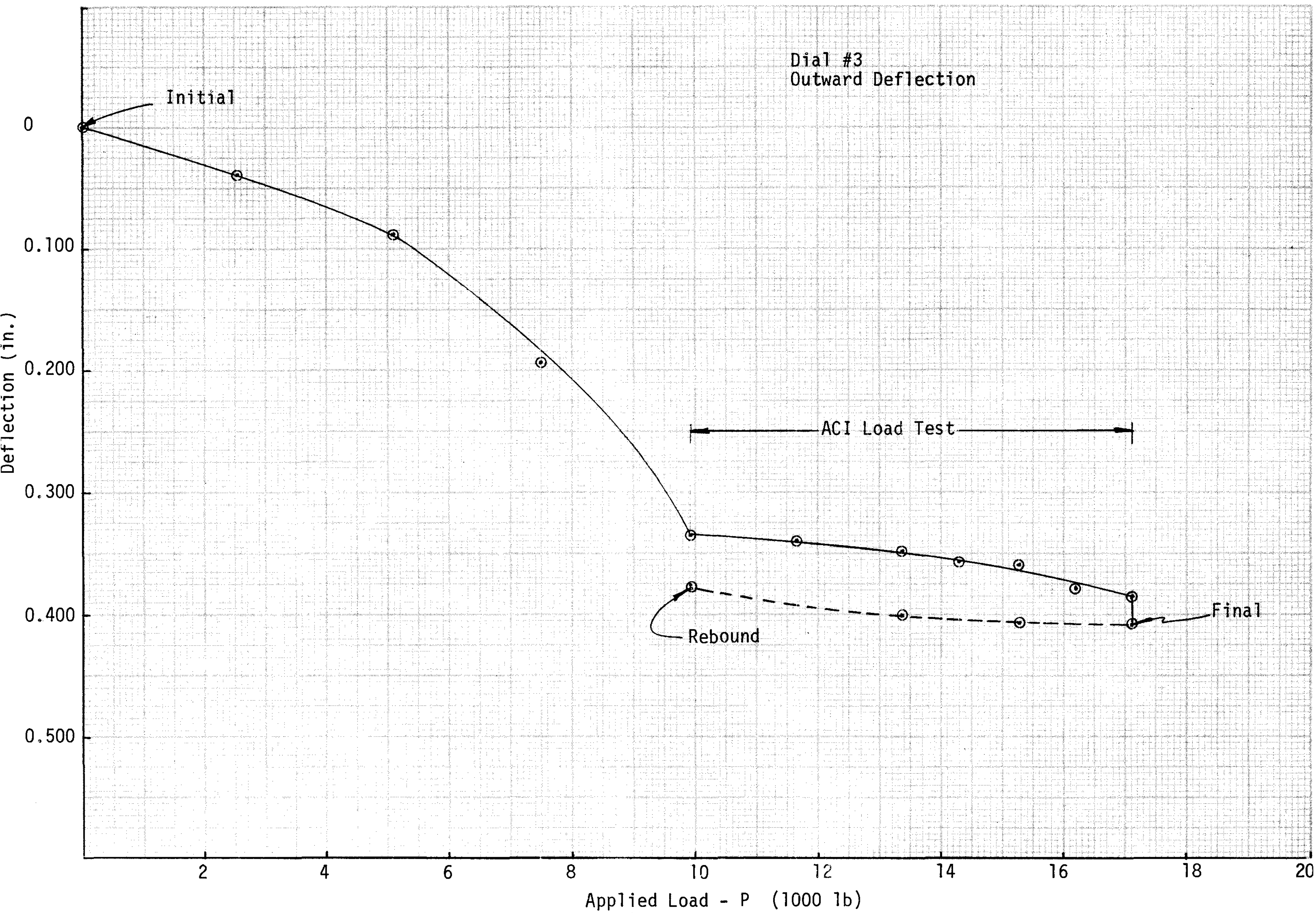
Lab. No. 6-13852

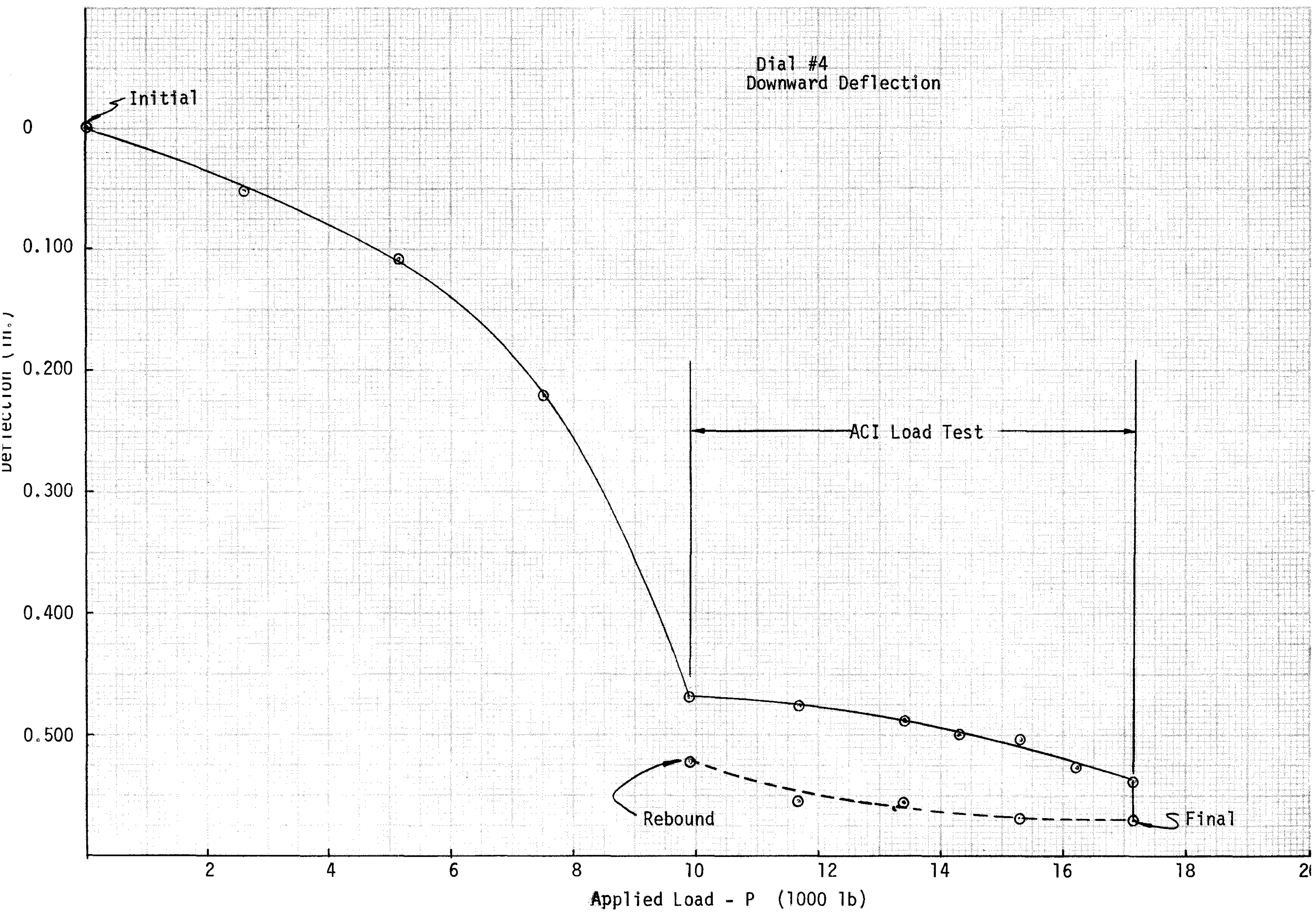
TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

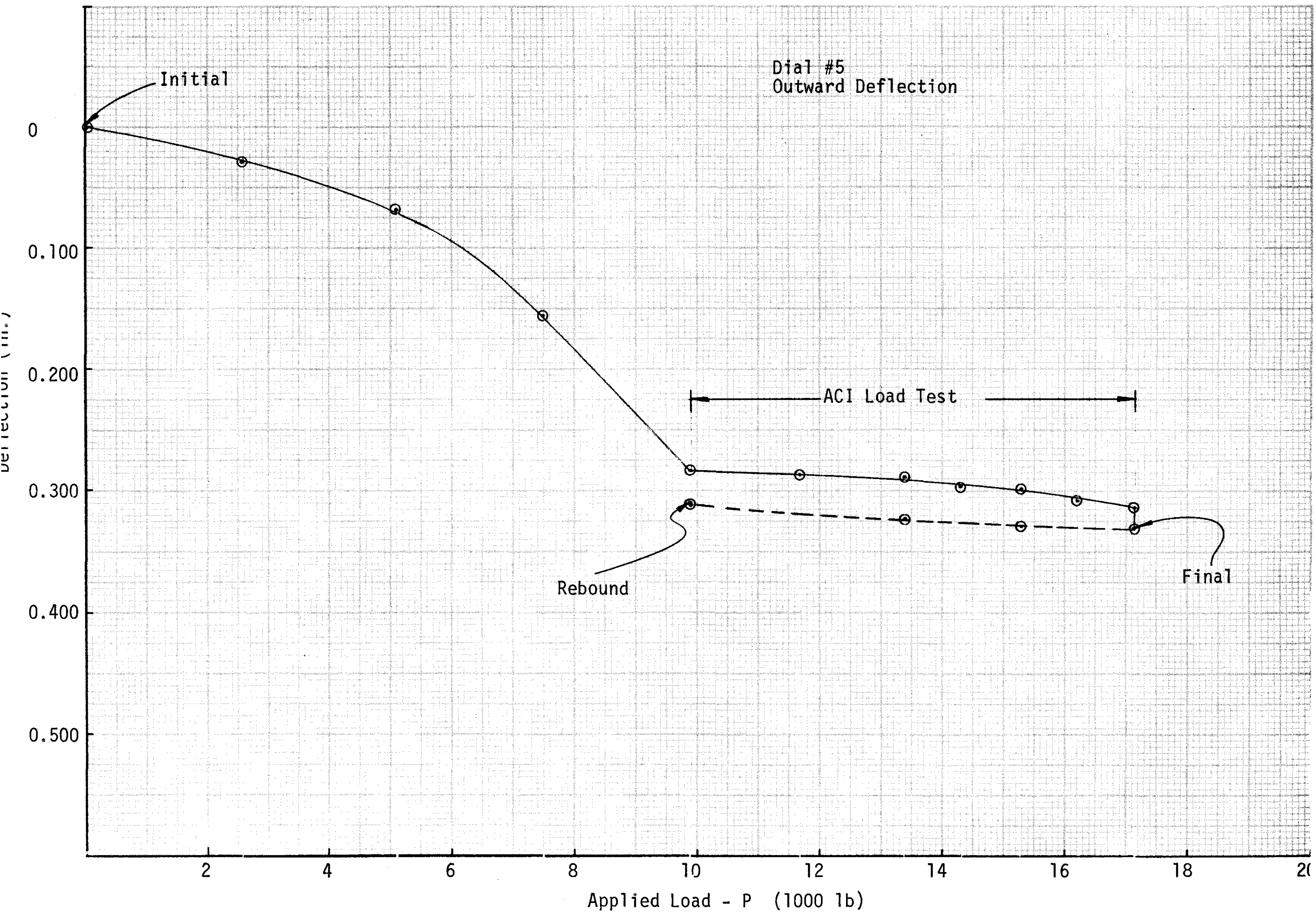

Dial #1
Outward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
Applied Load - P (1000 lb)



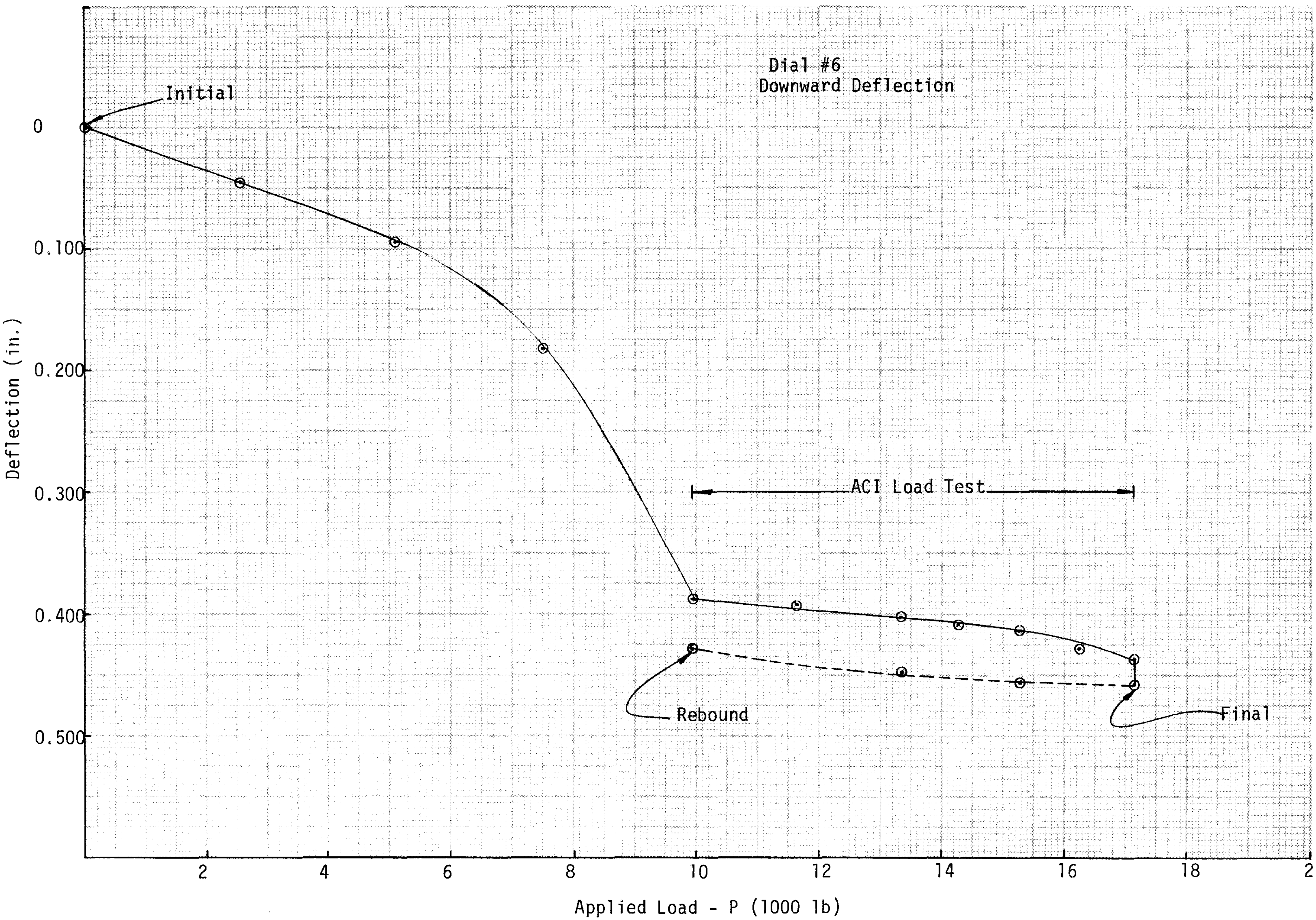
Dial #2
Downward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
Applied Load - P (1000 lb)

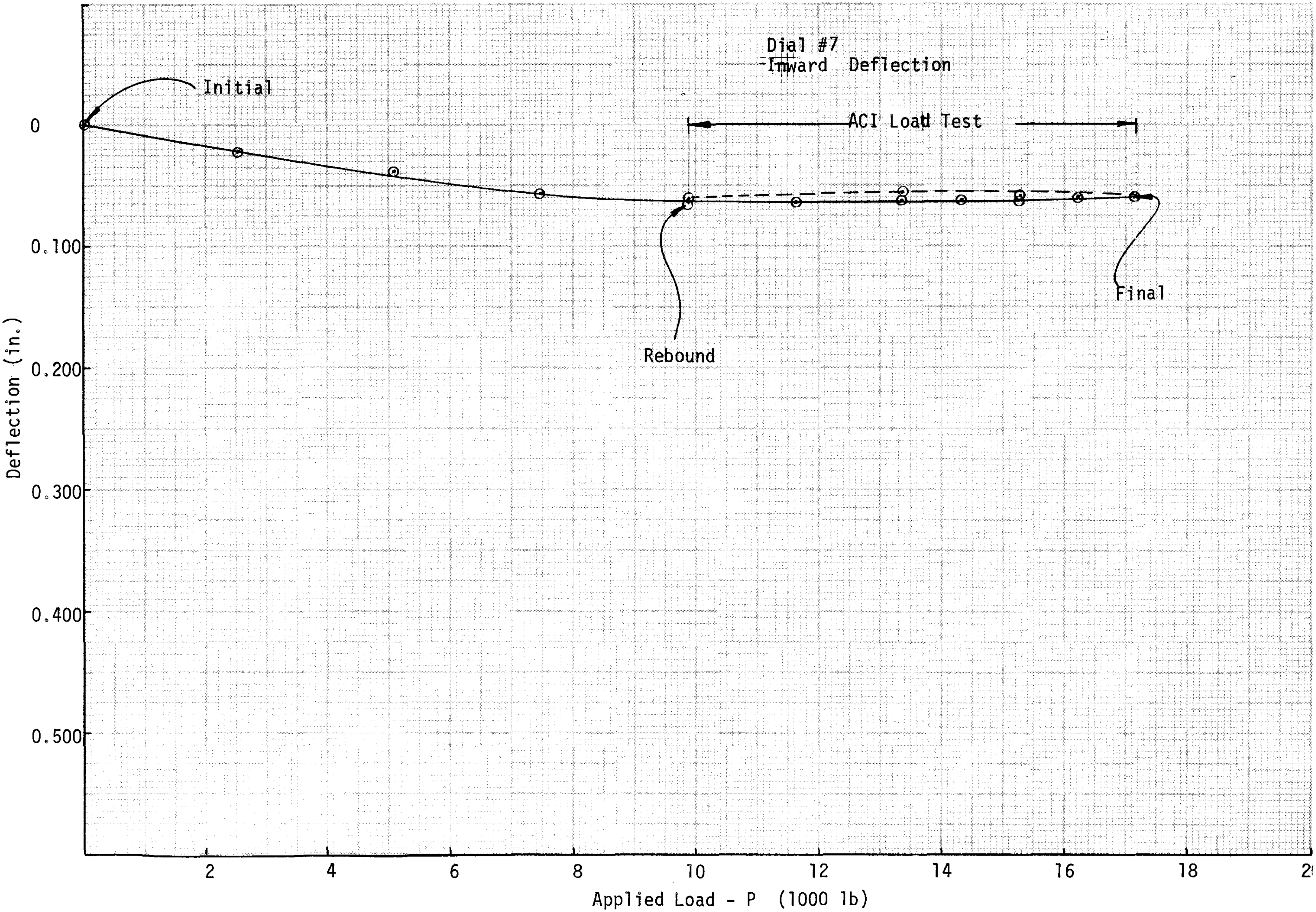

Dial #3
Outward Deflection
Initial
ACI Load Test
Rebound
Final
Deflection (in.)
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)


Dial #4
Downward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
(in.)
2
4
6
8
10
12
14
16
18
Applied Load - P (1000 lb)

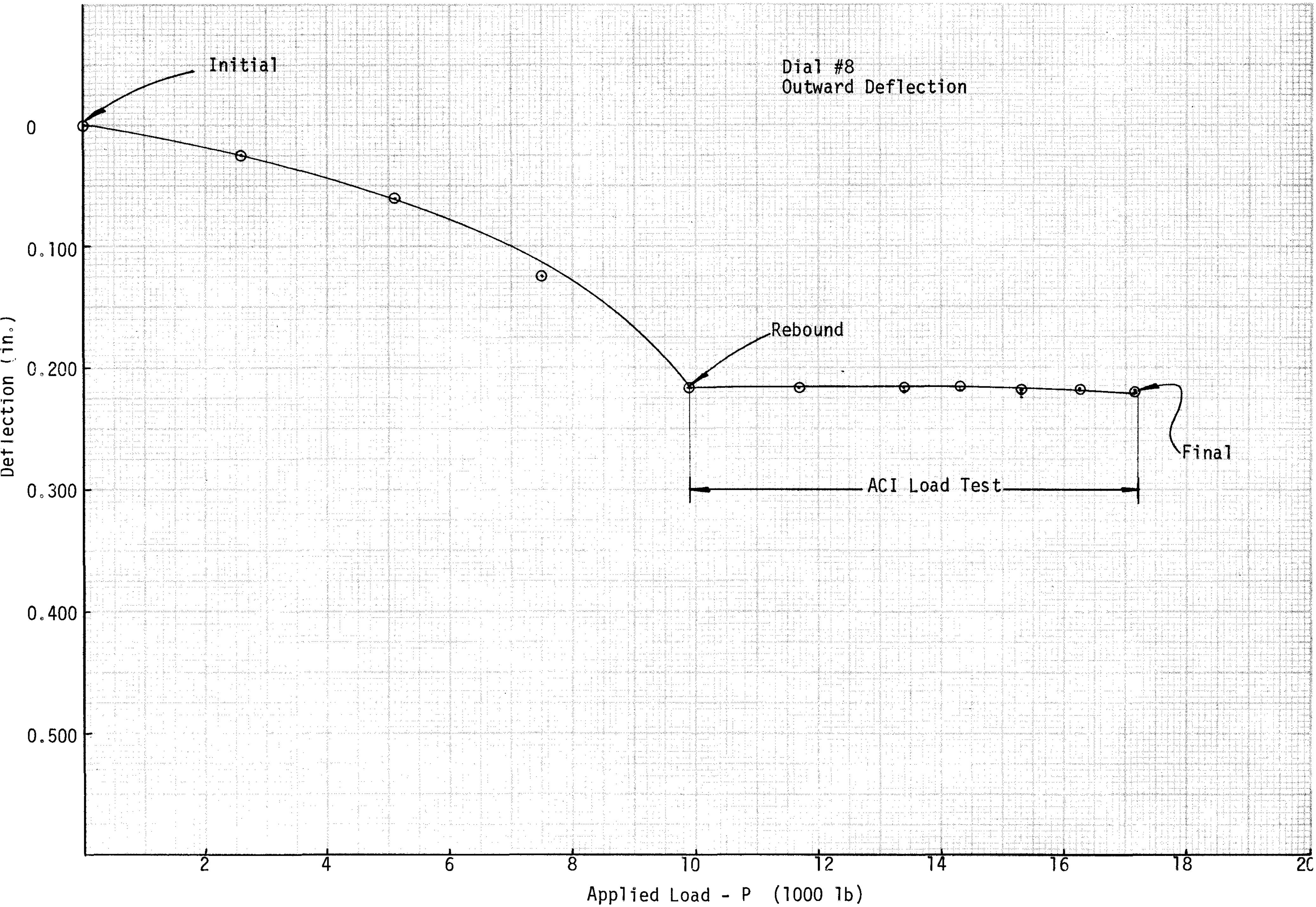


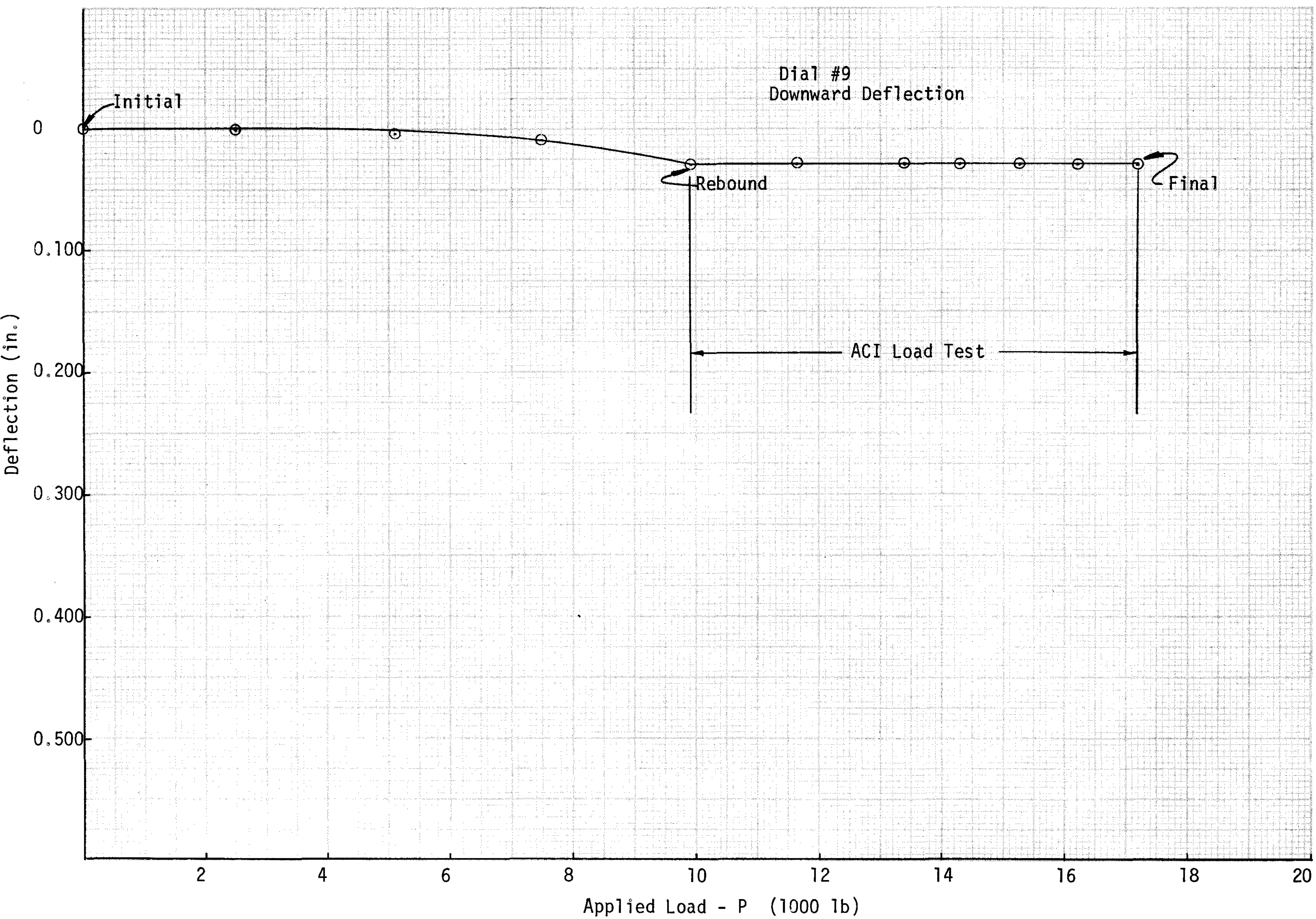
Initial
Dial #5
Outward Deflection
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
Applied Load - P (1000 lb)


Dial #6
Downward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
2
Applied Load - P (1000 lb)

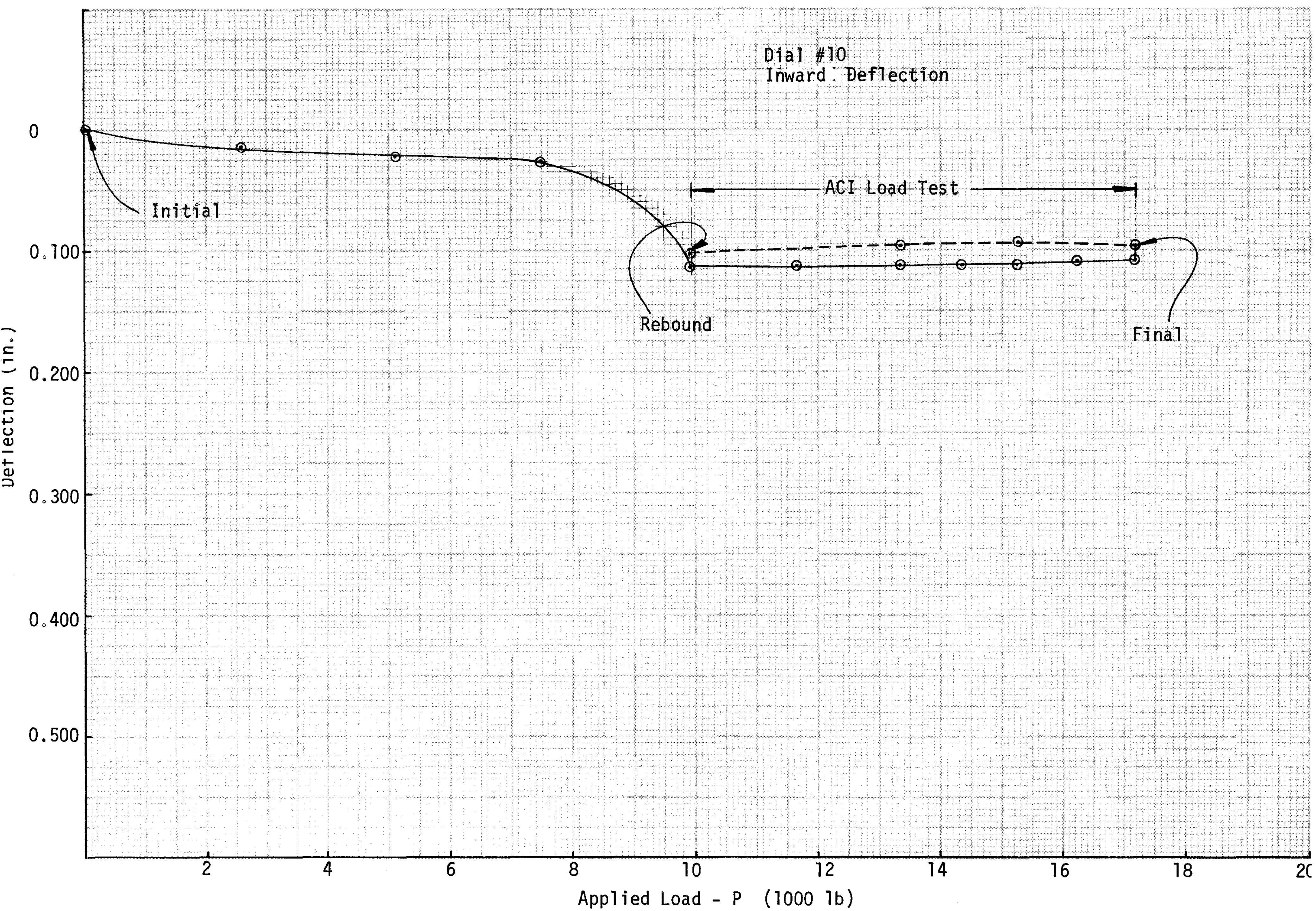


Dial #7
Inward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
Applied Load - P (1000 lb)

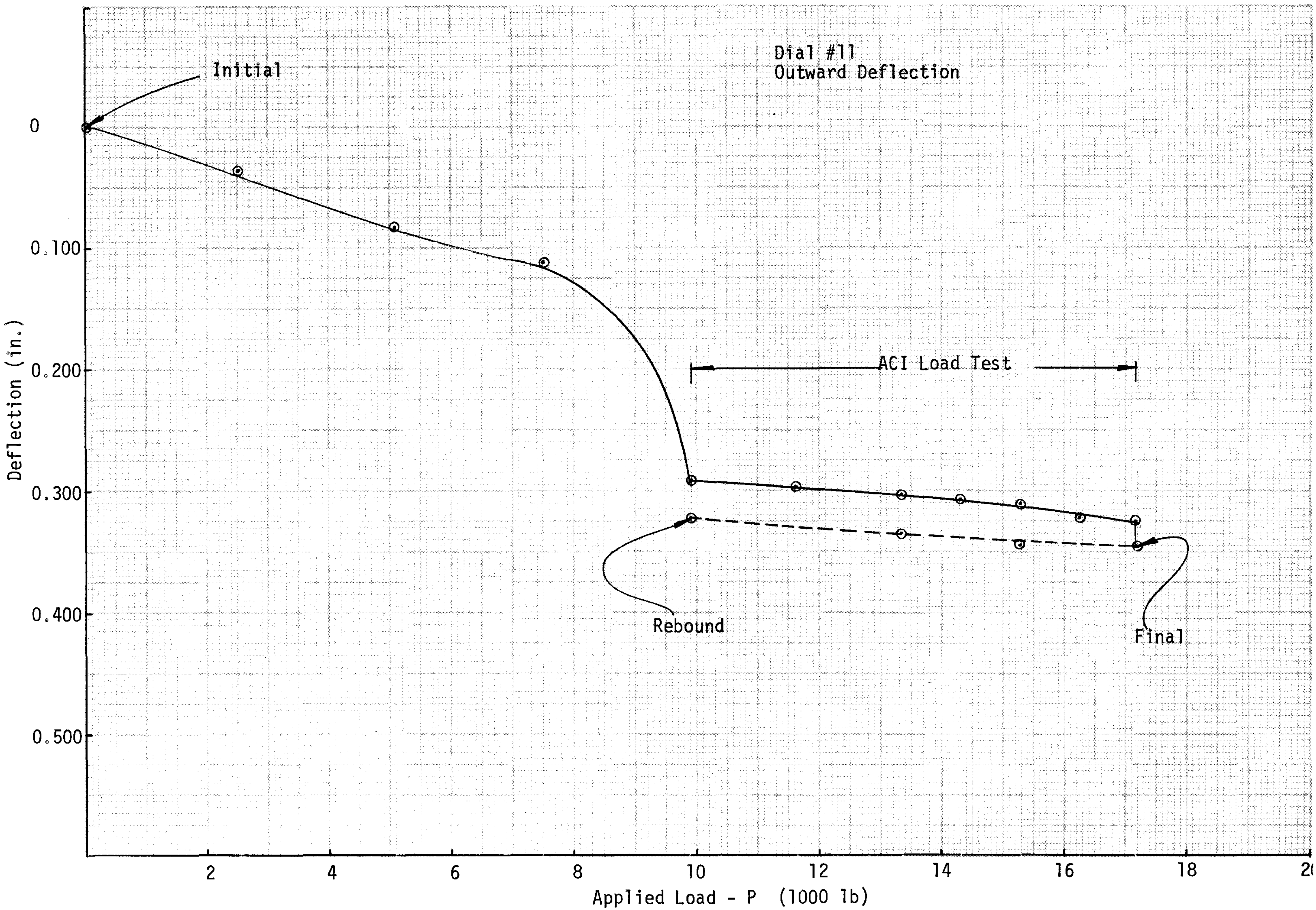

Initial
Dial #8
Outward Deflection
Rebound
Final
ACI Load Test
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)

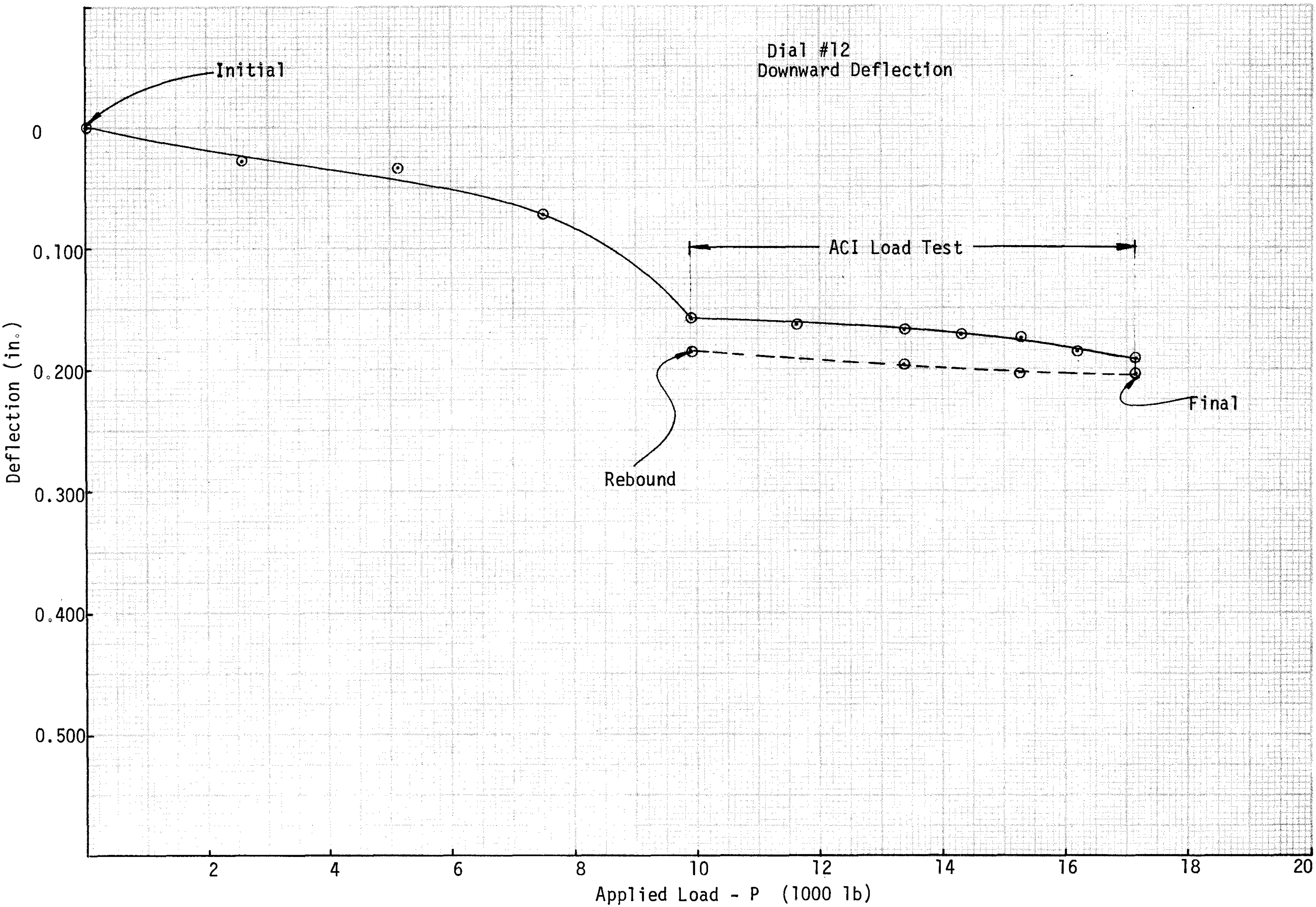


Dial #9
Downward Deflection
Initial
Rebound
Final
ACI Load Test
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)

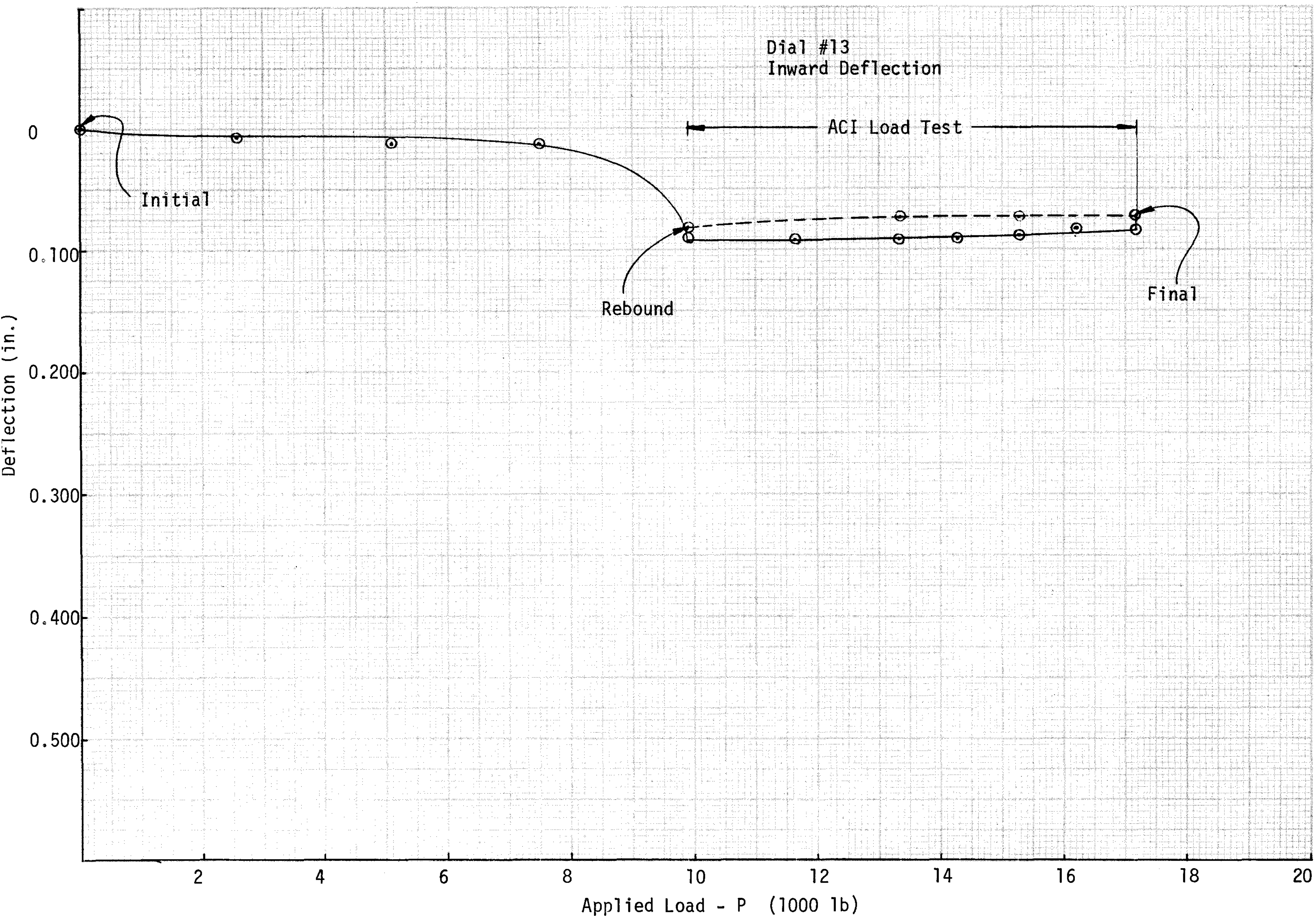


Dial #10
Inward Deflection
ACI Load Test
Initial
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)

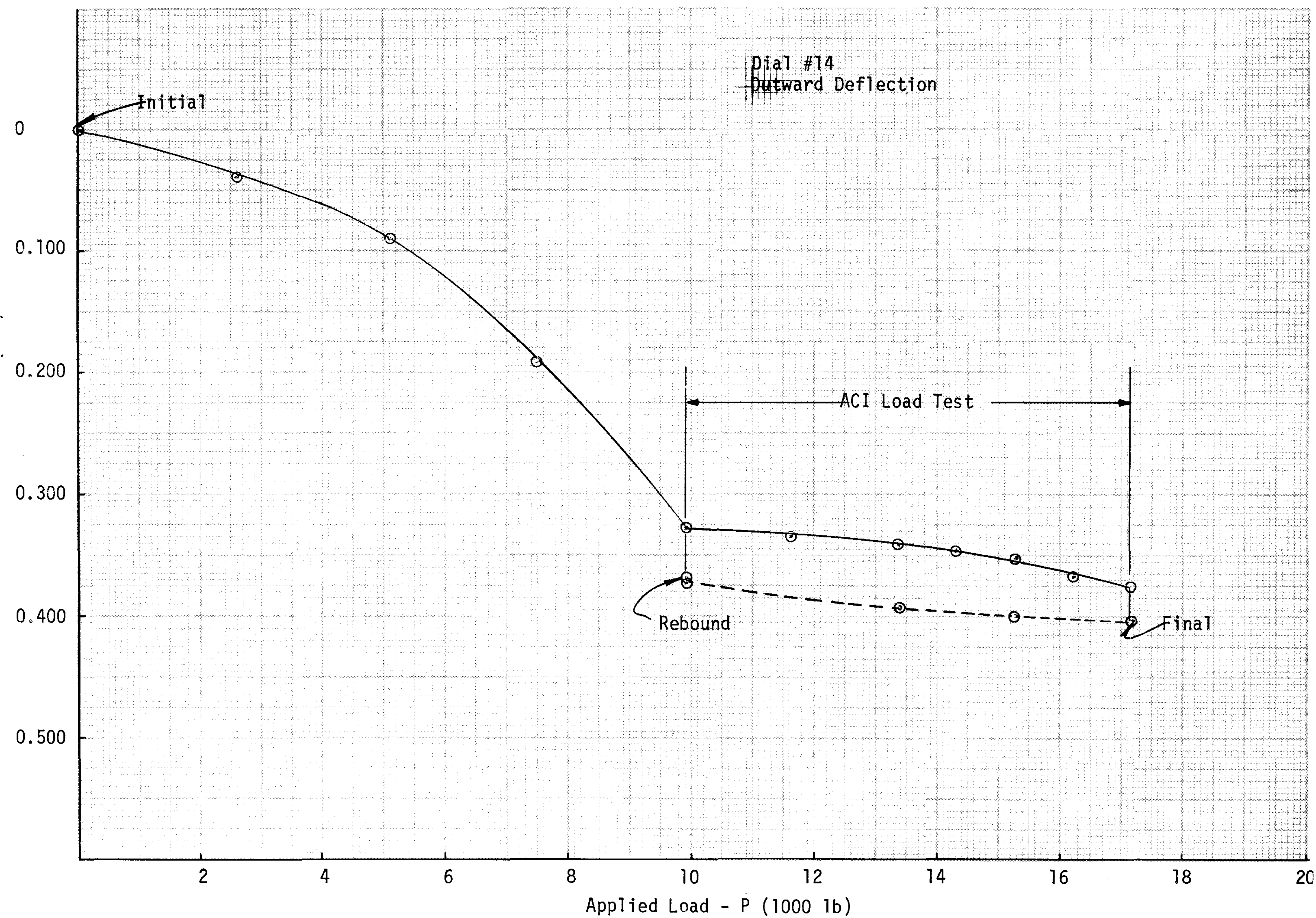

Dial #11
Outward Deflection
Initial
ACI Load Test
Rebound
Final
Deflection (in.)
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)



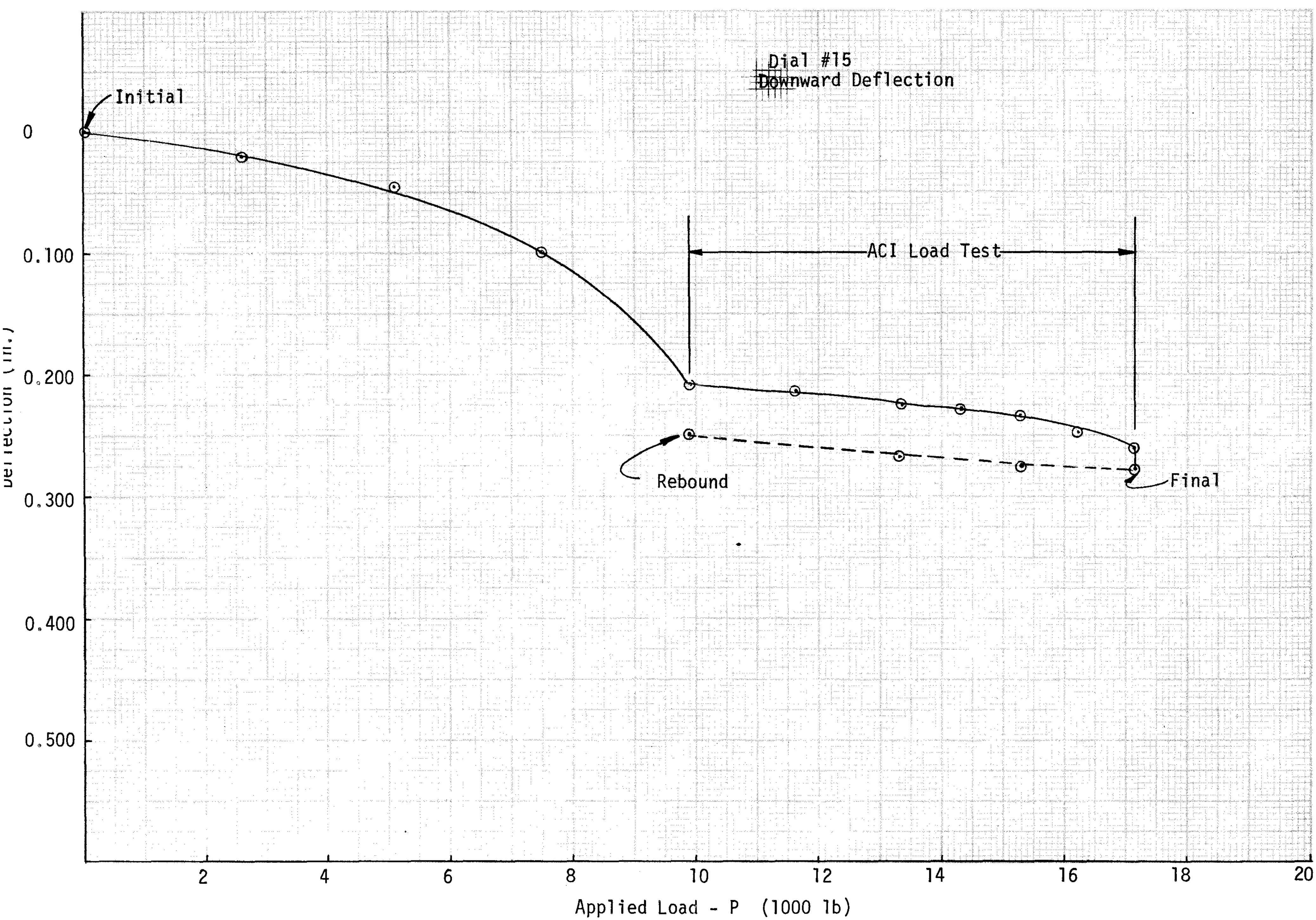
Dial #12
Downward Deflection
Initial
ACI Load Test
Rebound
Final
Deflection (in.)
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)

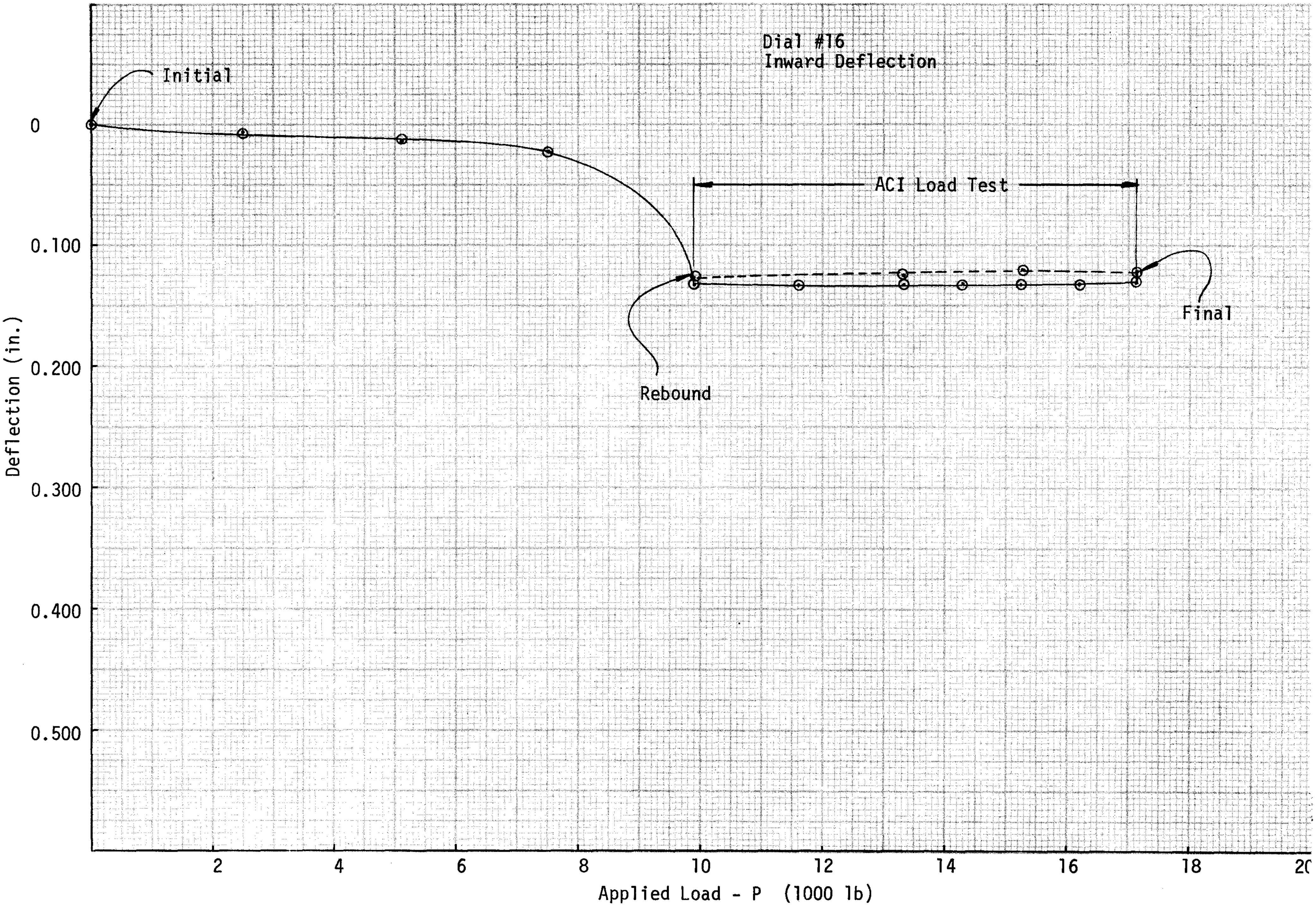


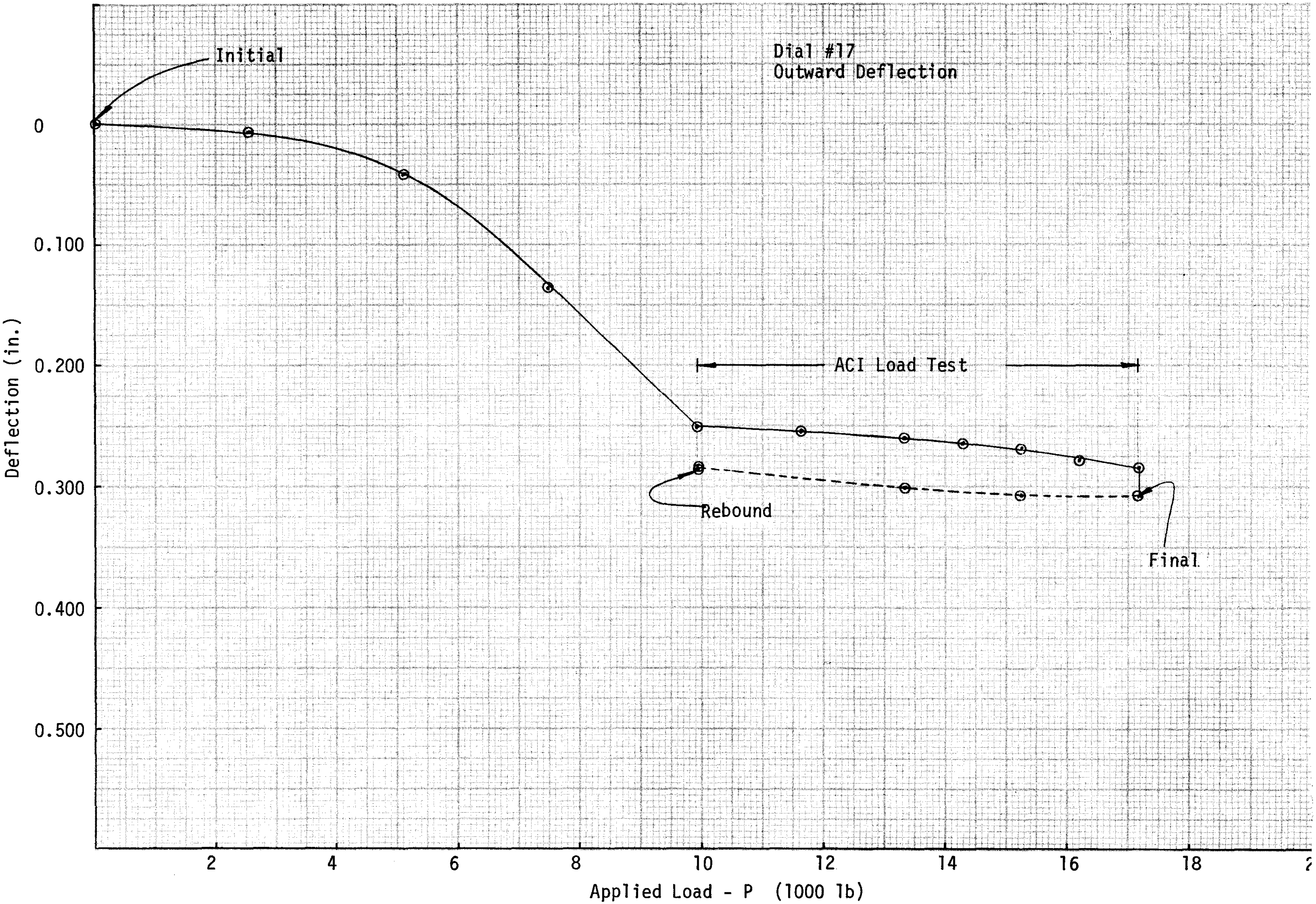
Dial #13
Inward Deflection
ACI Load Test
Initial
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)



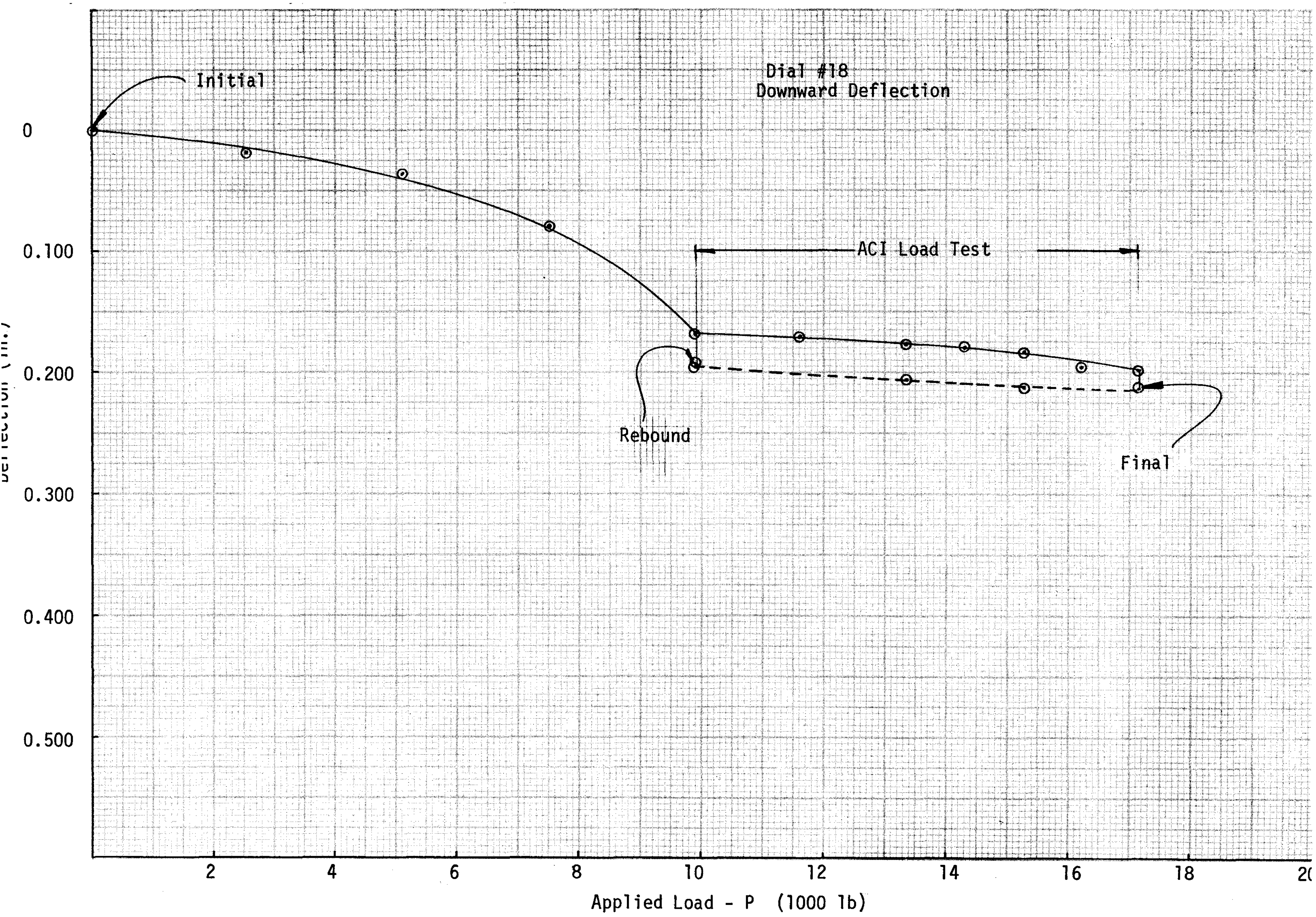
Dial #14
Outward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)


Dial #15
Downward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)

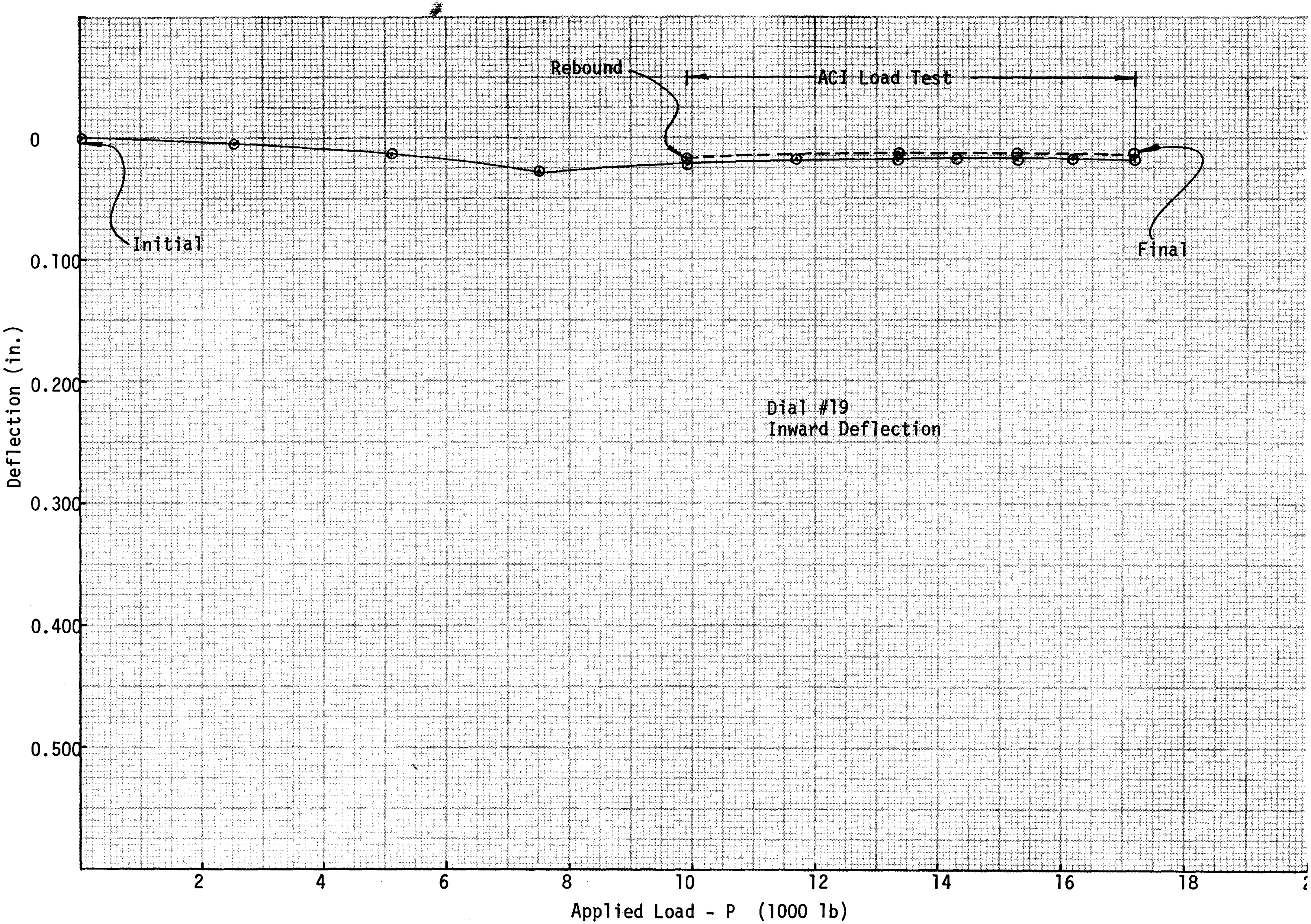

Dial #16
Inward Deflection
Initial
ACI Load Test
Rebound
Final
Deflection (in.)
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)

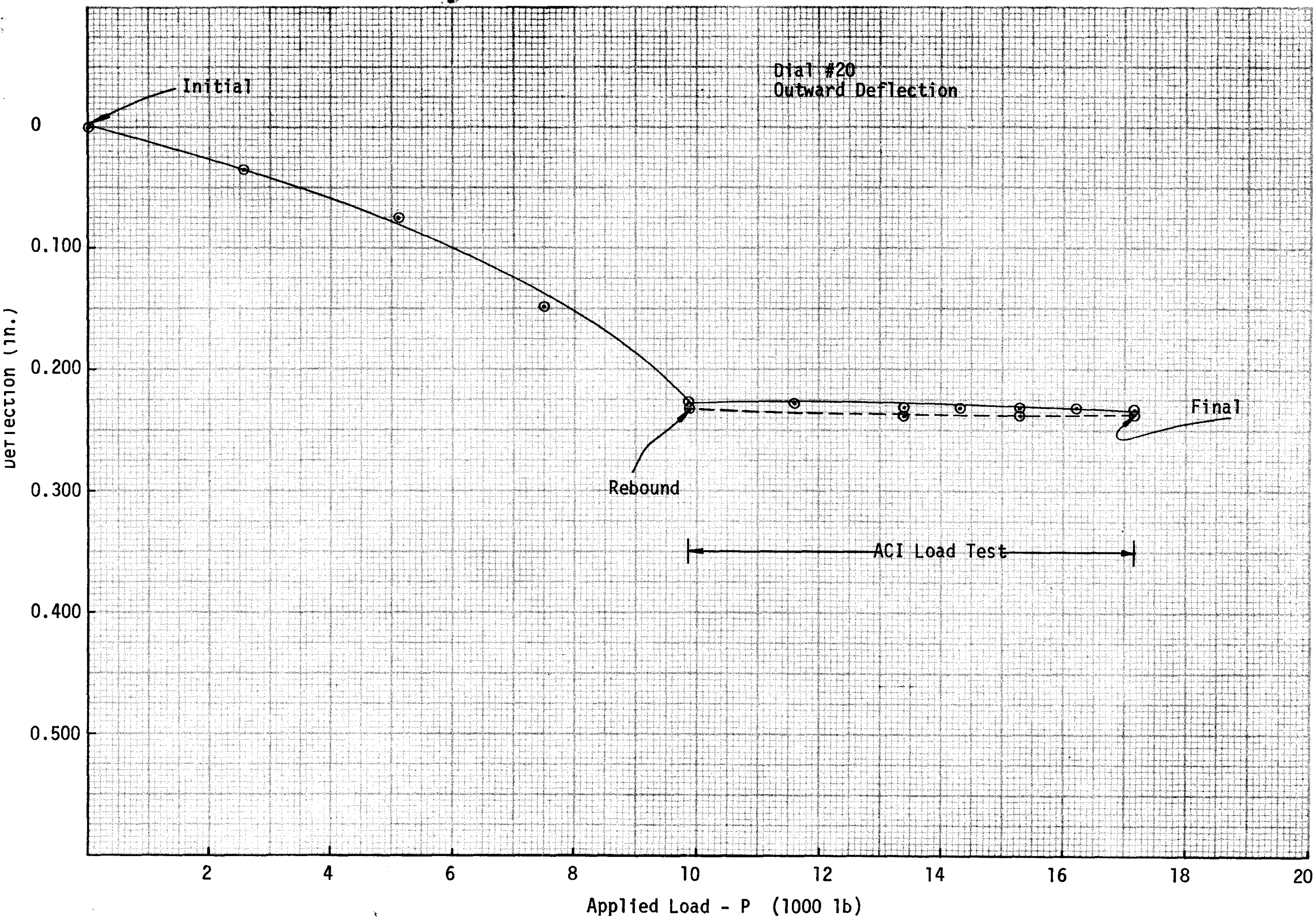


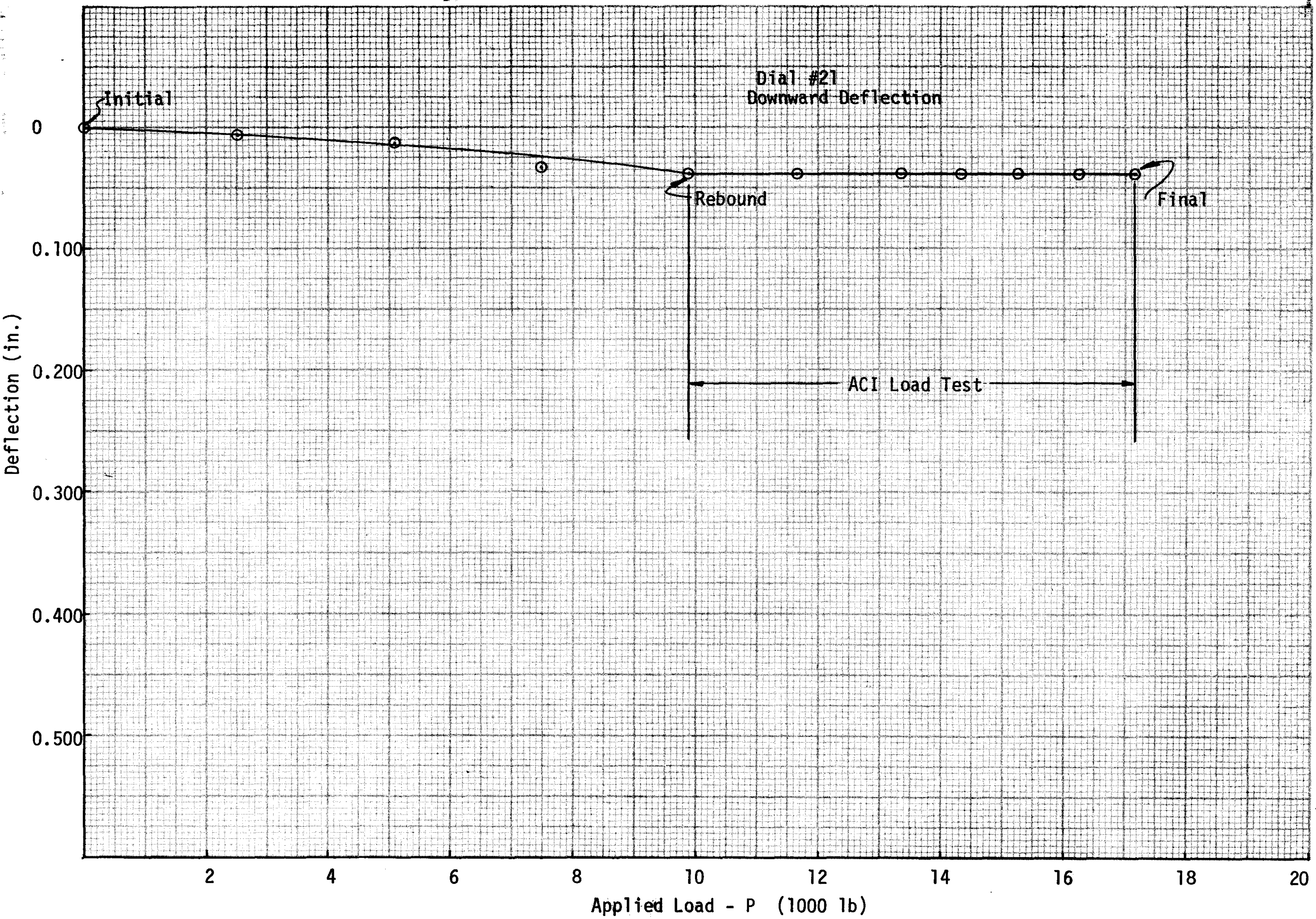
Initial
Dial #17
Outward Deflection
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
Applied Load - P (1000 lb)

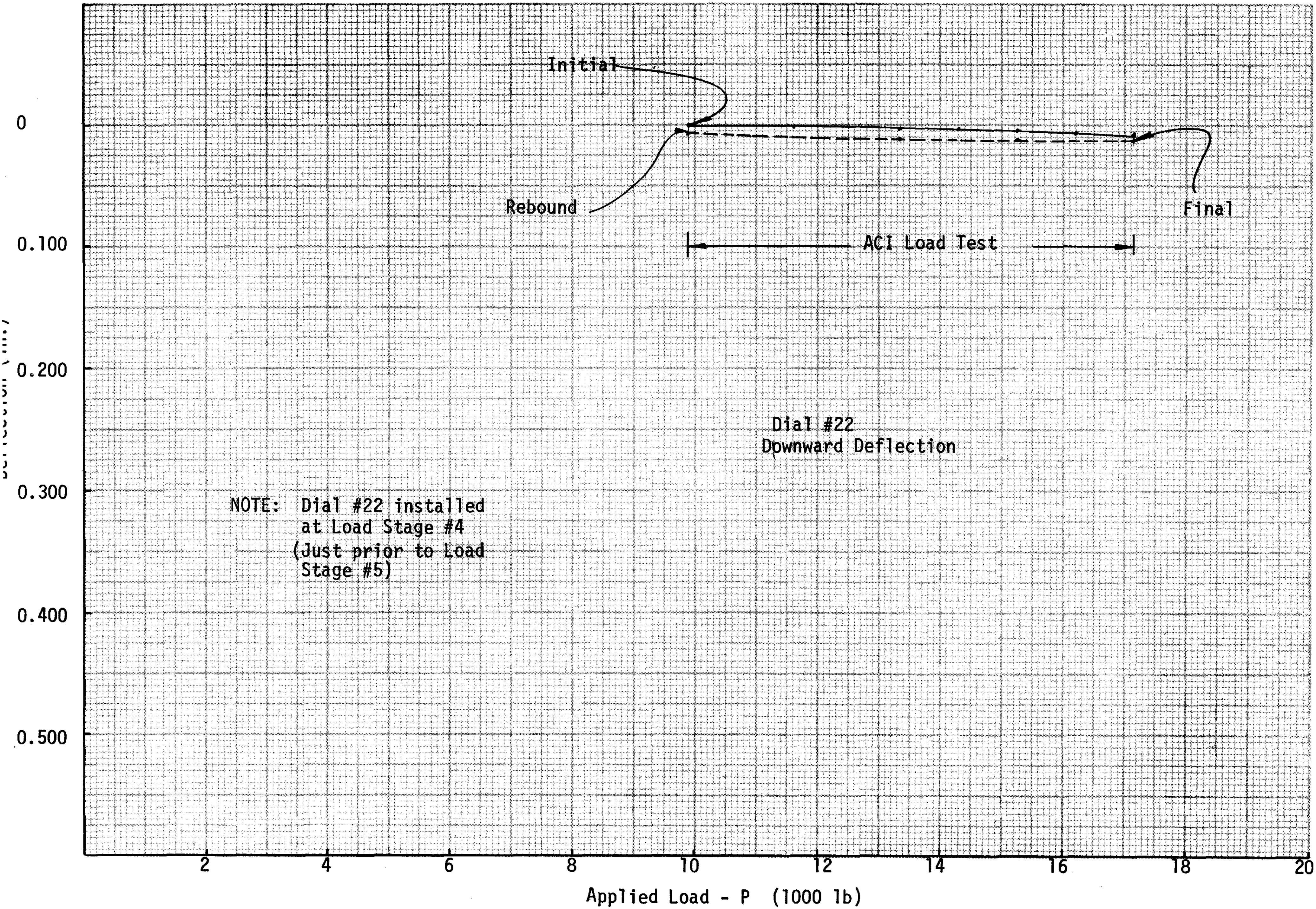


Dial #18
Downward Deflection
Initial
ACI Load Test
Rebound
Final
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)


Rebound
ACI Load Test
Initial
Final
Dial #19
Inward Deflection
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
Applied Load - P (1000 lb)

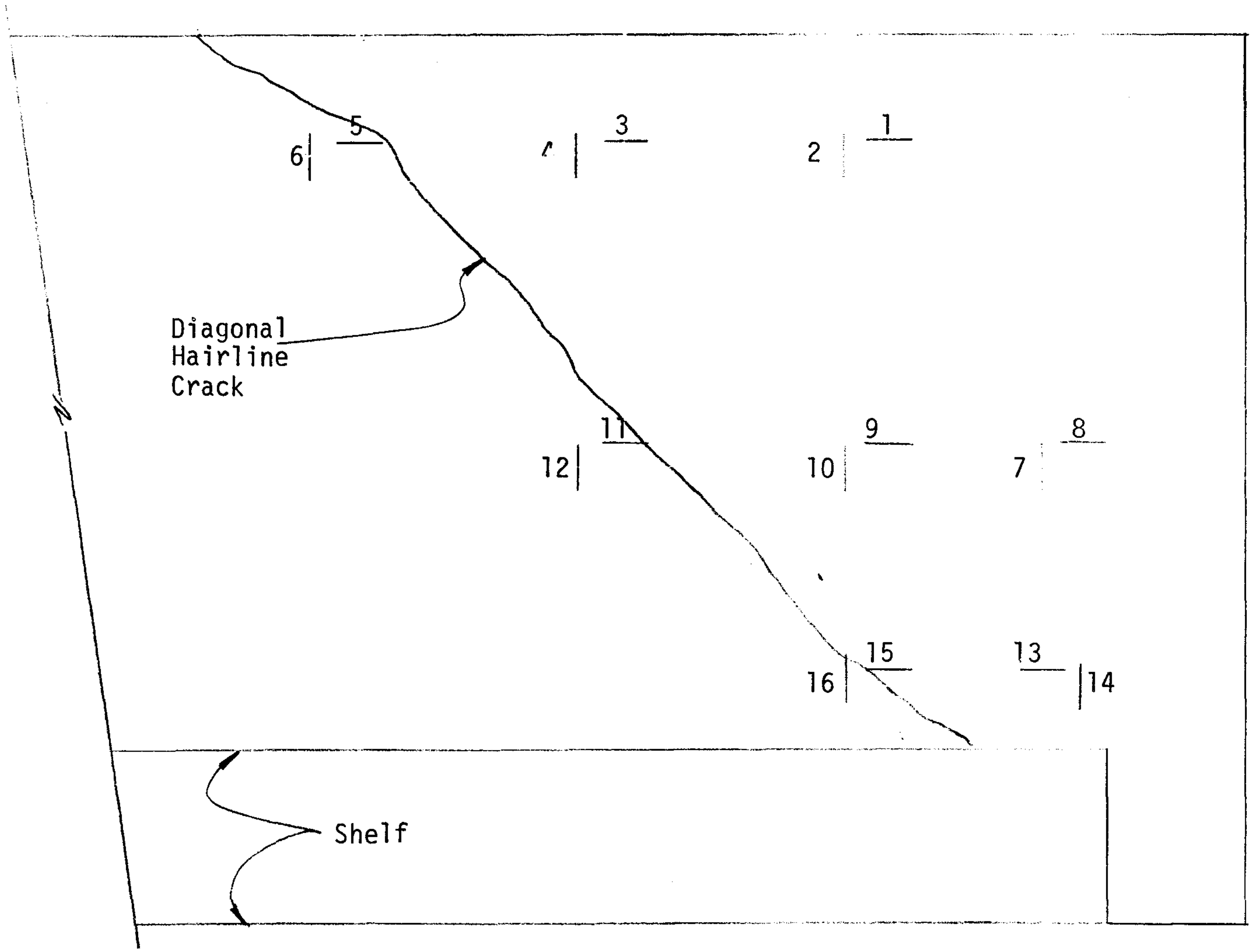

Dial #20
Outward Deflection
Initial
Rebound
Final
ACI Load Test
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)


Dial #21
Downward Deflection
Initial
Rebound
Final
ACI Load Test
0
0.100
0.200
0.300
0.400
0.500
Deflection (in.)
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)



Initial
Rebound
Final
ACI Load Test
Dial #22
Downward Deflection
NOTE: Dial #22 installed at Load Stage #4 (Just prior to Load Stage #5)
0
0.100
0.200
0.300
0.400
0.500
2
4
6
8
10
12
14
16
18
20
Applied Load - P (1000 lb)

STRAIN GAGE LOCATIONS



Elevation - 2" Strain Gage Locations - South End Reaction

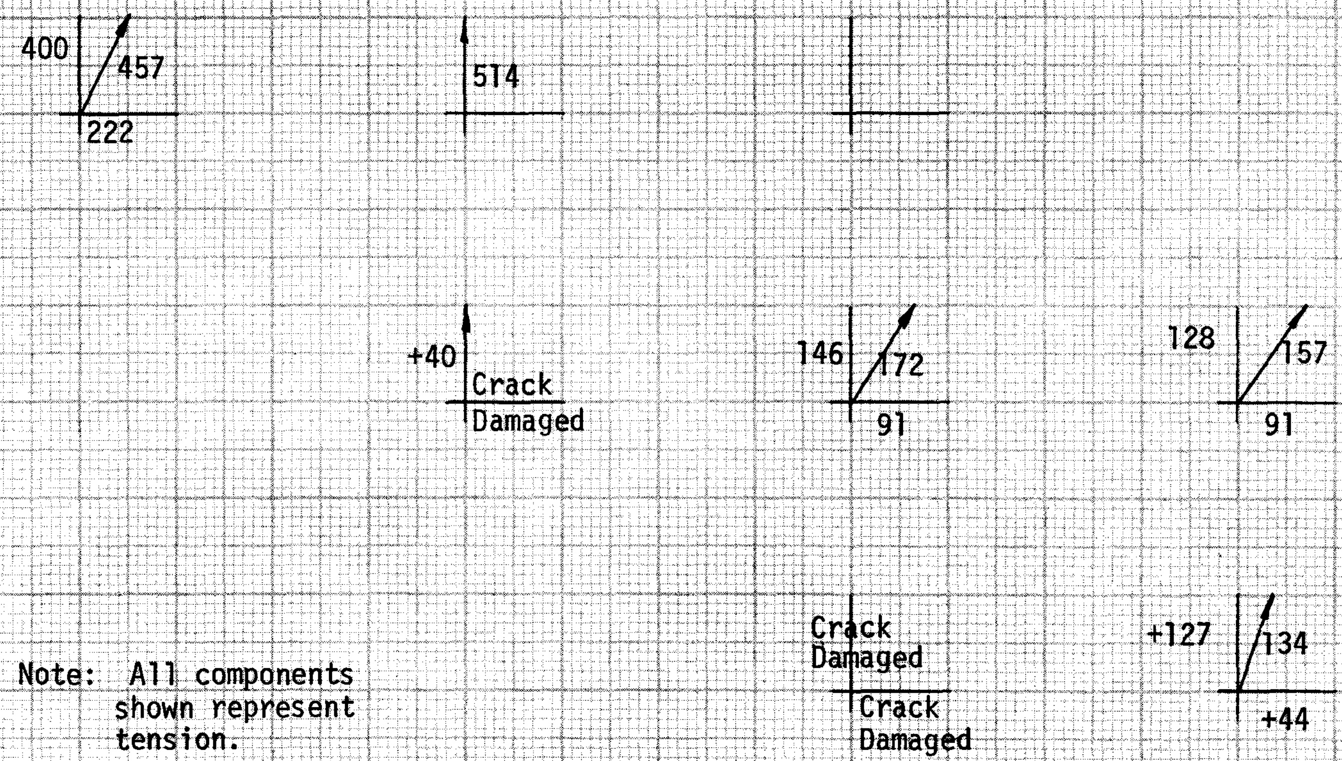
Scale; 1 1/2"=1'-0"

Laboratory Number 6-13852

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

Load Stage 4 (Just Prior to #5)

HORIZONTAL AND VERTICAL STRAIN COMPONENTS AND RESULTANTS (Micro Inches/inch)



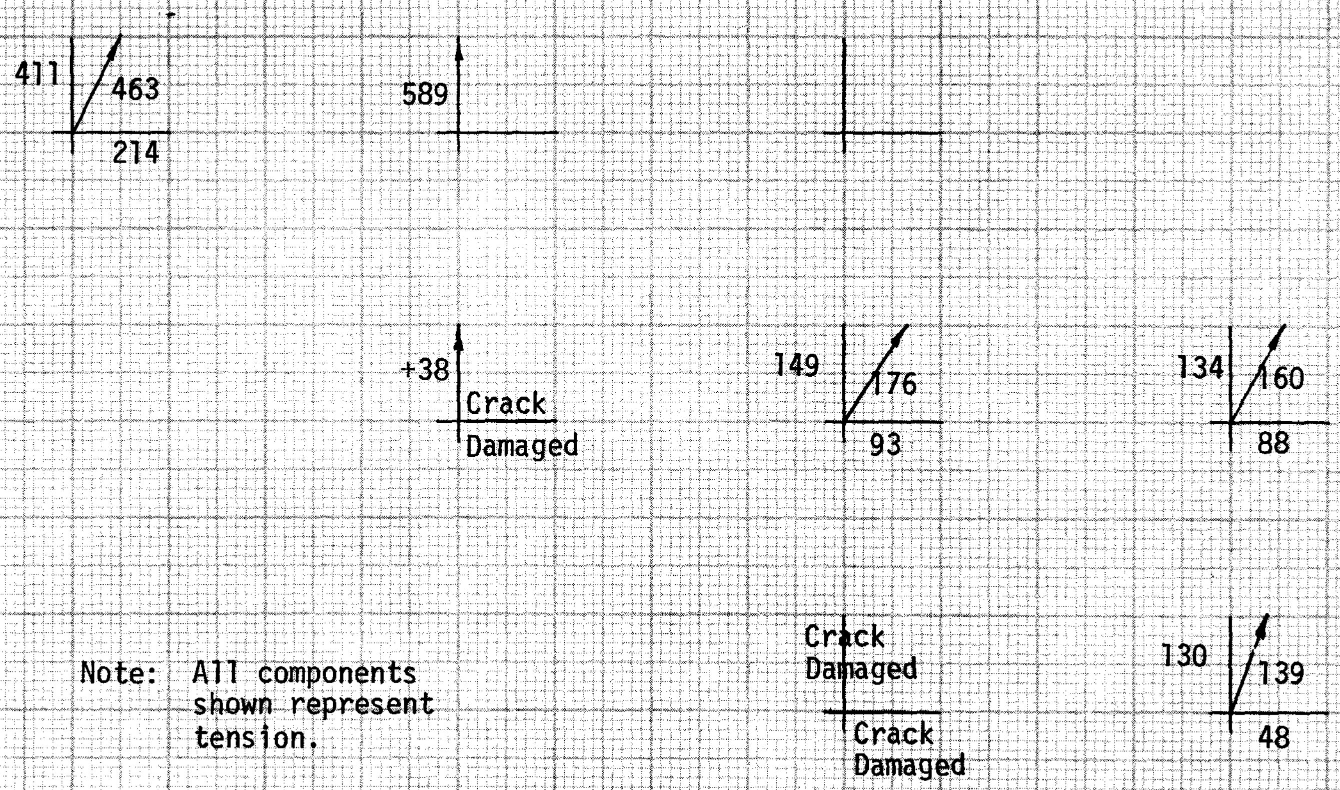
Note: All components shown represent tension.

Scale: None

Laboratory Number 6-13852

Load Stage 7 (Just Prior to #8)

HORIZONTAL AND VERTICAL STRAIN COMPONENTS AND RESULTANTS (Micro inches/inch)



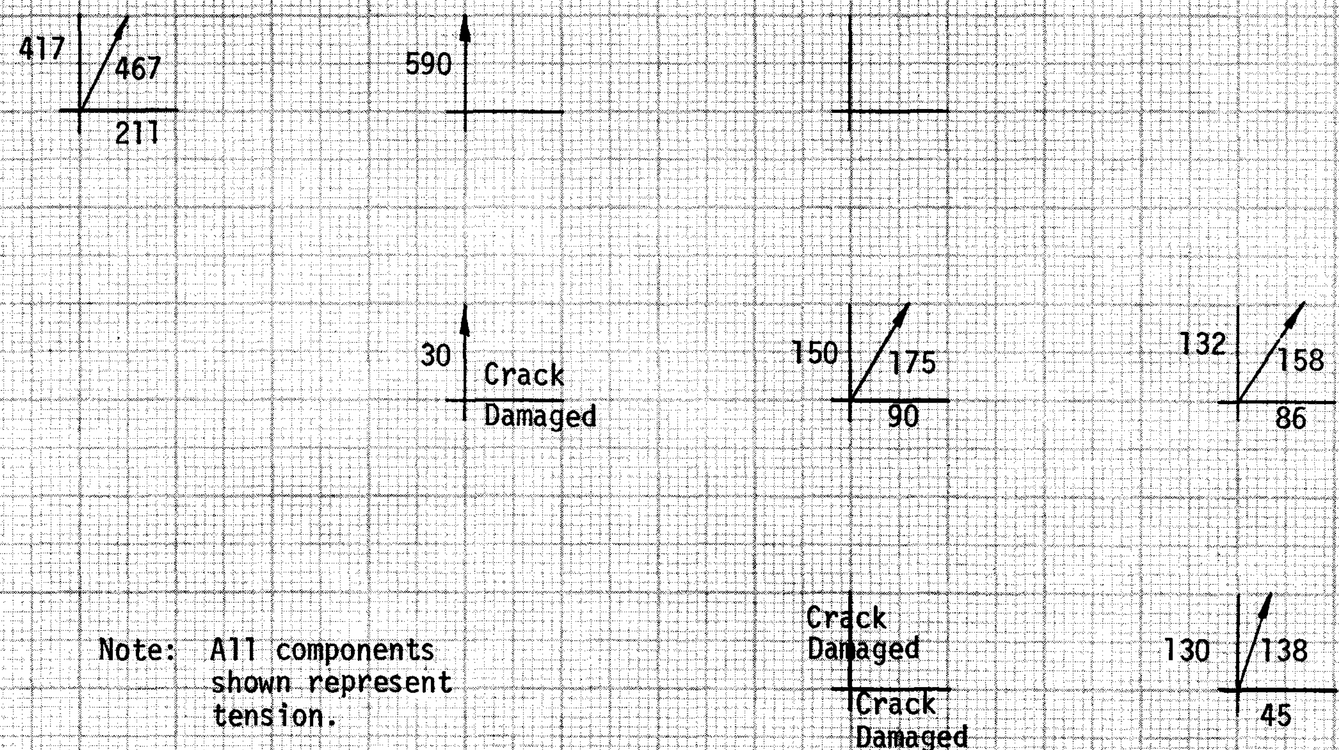
Note: All components shown represent tension.

Scale: None

Laboratory Number 6-13852

Load Stage 9 (Just Prior to #10)

HORIZONTAL AND VERTICAL STRAIN COMPONENTS AND RESULTANTS (Micro inches/inch)



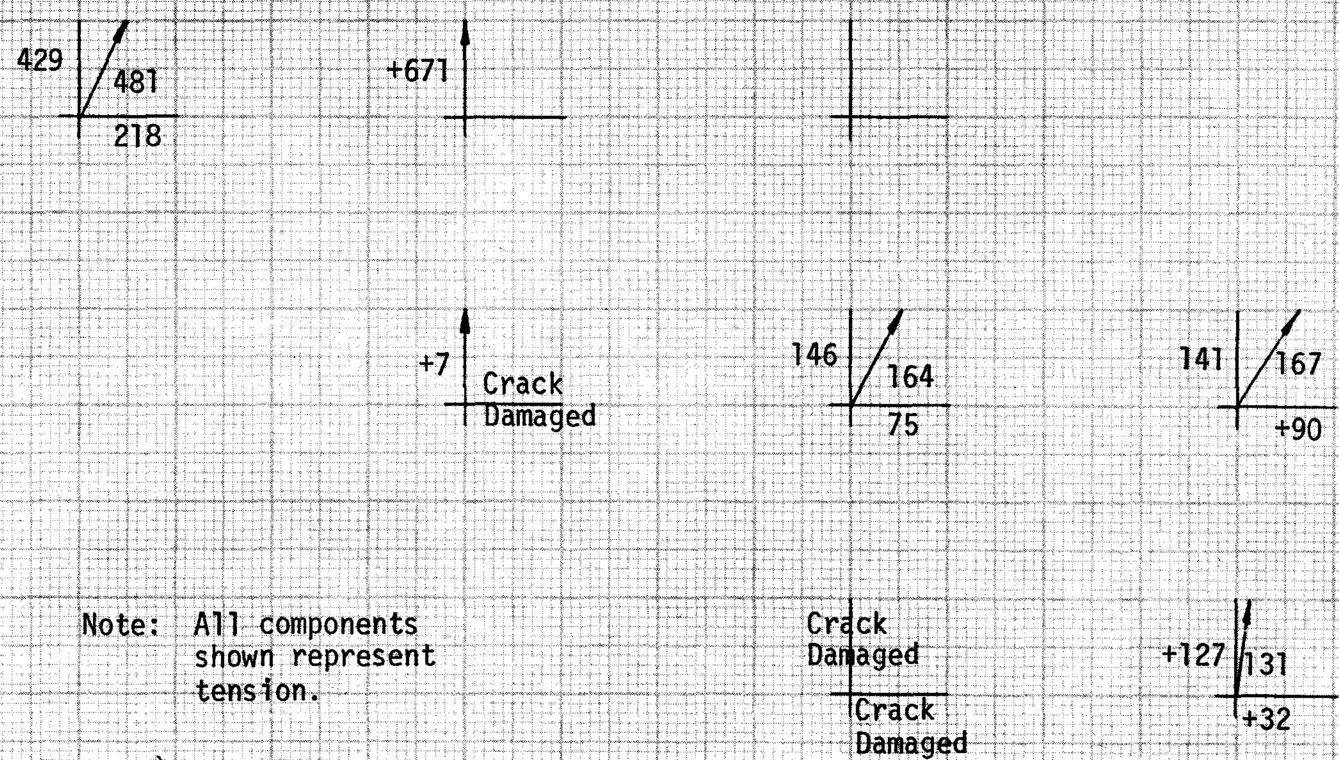
Note: All components shown represent tension.

Scale: None

Laboratory Number 6-13852

Load Stage 10 (Final)

HORIZONTAL AND VERTICAL STRAIN COMPONENTS AND RESULTANTS (Micro inches/inch)



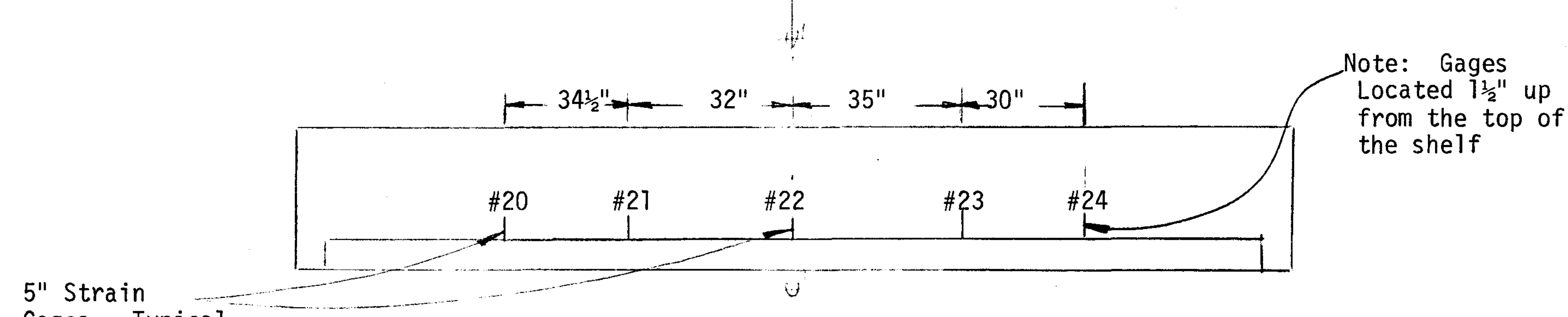
Note: All components shown represent tension.

Scale: None

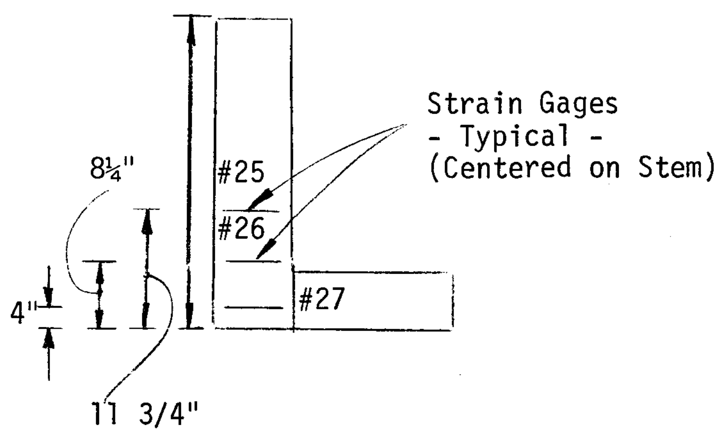
Laboratory Number 6-13852

North

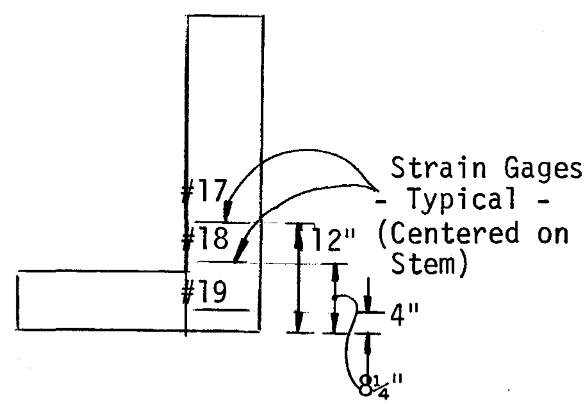
5" STRAIN GAGE LOCATIONS (Numbers 17 Through 27)



Side View - Inside Face of Concrete Spandrel Beam



End View - South End Reaction



End View - North End Reaction

Gage Type Used: BLH A9-3 5" Wire Gage

Scale: None

Laboratory Number 6-13852

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

POSITIVE STRAIN (Micro inches/inch)

Strain Gage Number	Initial Reading No Load*	Load Stage #4 (Just Prior to #5)	Load Stage #7 (Just Prior to #8)	Load Stage #10 (Just Prior to Unloading)	Load Stage #4 (End of ACI Load Test)
17	0	241	239	230	215
18	0	62	58	29	5
19	0	142	141	128	113
20	0	380	413	567	517
21	0	374	394	424	398
22	0	248	256	280	259
23	0	417	442	588	543
24	0	298	313	306	285
25	0	140	146	116	104
26	0	155	157	135	126
27	0	116	121	91	92

*Note: All strain gages were mounted with the concrete spandrel beam simply supported, and no load applied. Initial readings were then taken in this no-load condition.

The above strain gage readings are positive changes from the initial reading.

Laboratory Number 6-13852

Photograph #1 - North-One-Half of Test Setup, Front-View, Displaying Simulated Double-Tee, Loading Beams.

Photograph #2 - South-One-Half of Test Setup, Front-View.

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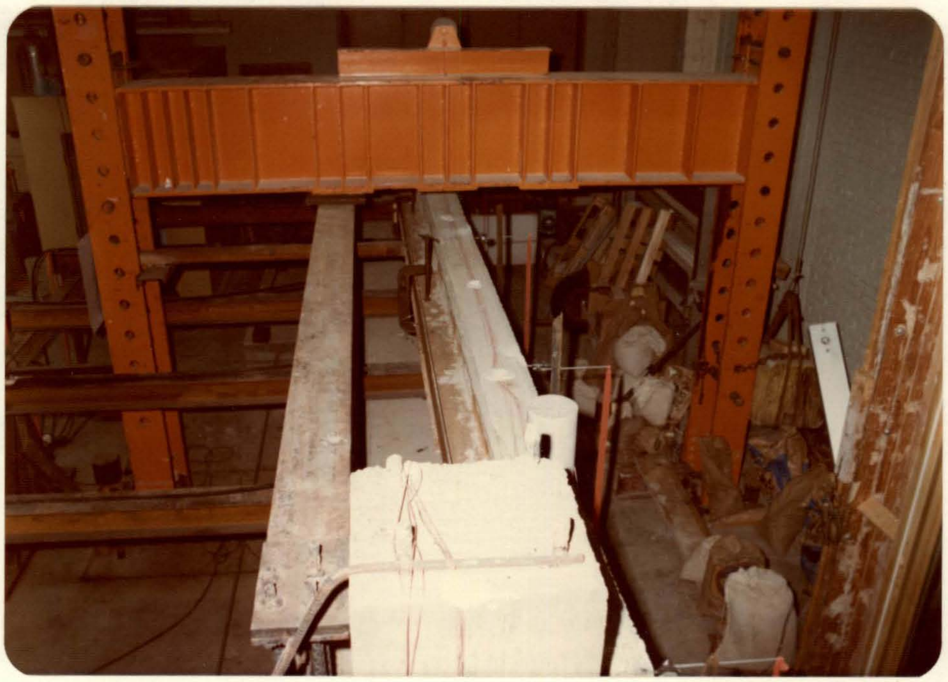
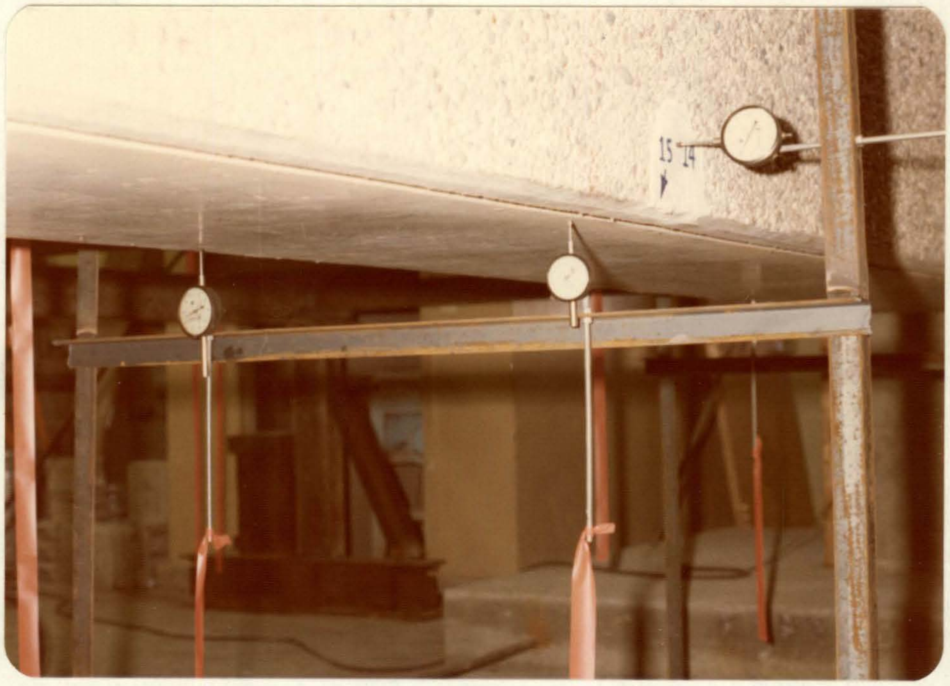
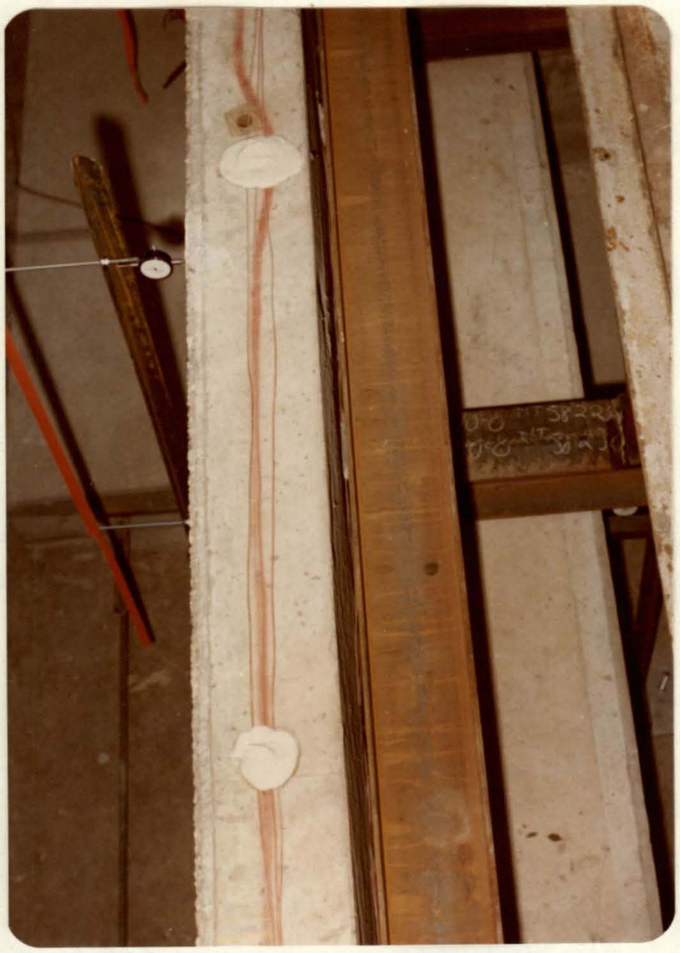
Photograph #3 - Test Setup, Side-View, Displaying Simulated Double-Tee, Loading Beams and Calibrated Hydraulic Rams.



Photograph #4 - Test Setup, Top-View, Exhibiting I-Beam, (On the Right Side of the Picture) Used for Simulation of the Concrete Topping. Note Grout Between I-Beam and Concrete Spandrel Beam.



Photograph #7 - Side-View, Mounted Dial Indicators, Measuring Outward and Downward Displacement to the 0.001 inch.



Photograph #8 -

Top-View, Facing South, Displaying I-Beam Used for the Simulation of the Concrete Topping. (Note - Grout was not yet in Place Between I-Beam and Spandrel Beam)

Photograph #9 -

Side-View of Steel Haunch Prior to Placement of 5-Star Grout. (South-End Reaction)

Photograph #10 - Side-View of Steel Haunch After Grout was Installed. (South-End Reaction)

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Photograph #11 -

Side-View of Steel Haunch
After Placement of Grout.
(North-End Reaction)

Photograph #12 -

End View, North-End Reaction,
Exhibiting Strain Gage Numbers
17, 18 and 19.

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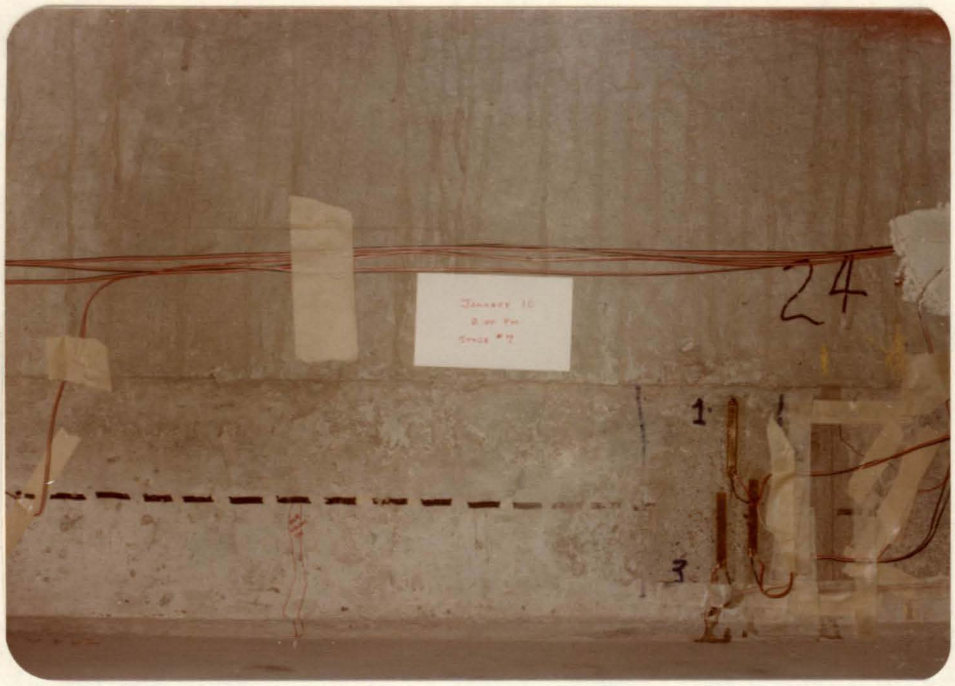
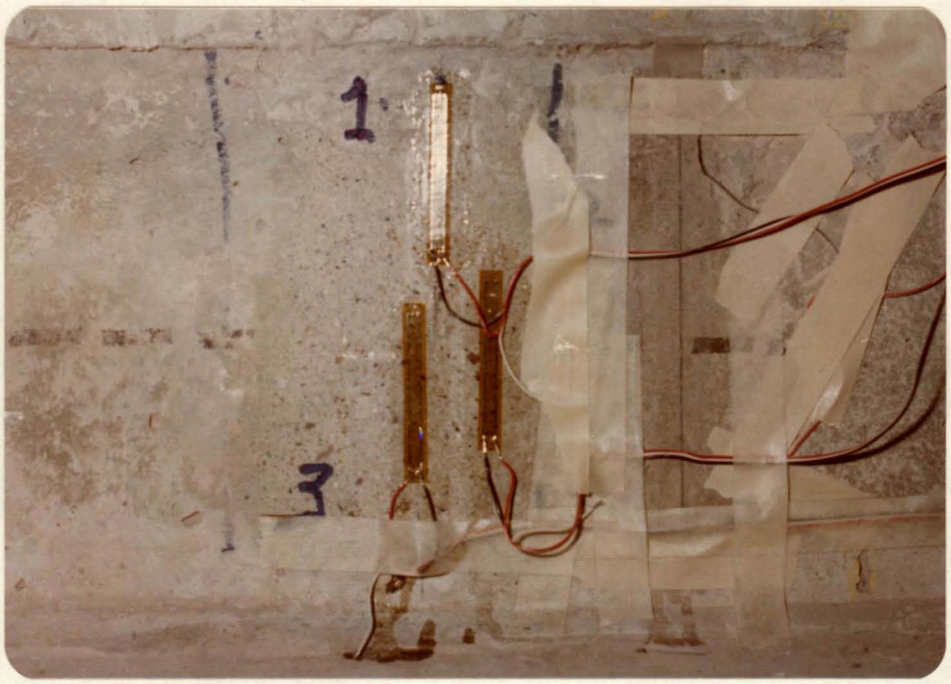
Photograph #13 - Front-View Stem, Displaying Diagonal Hairline Crack at Load Stage #4. (North-End Reaction)

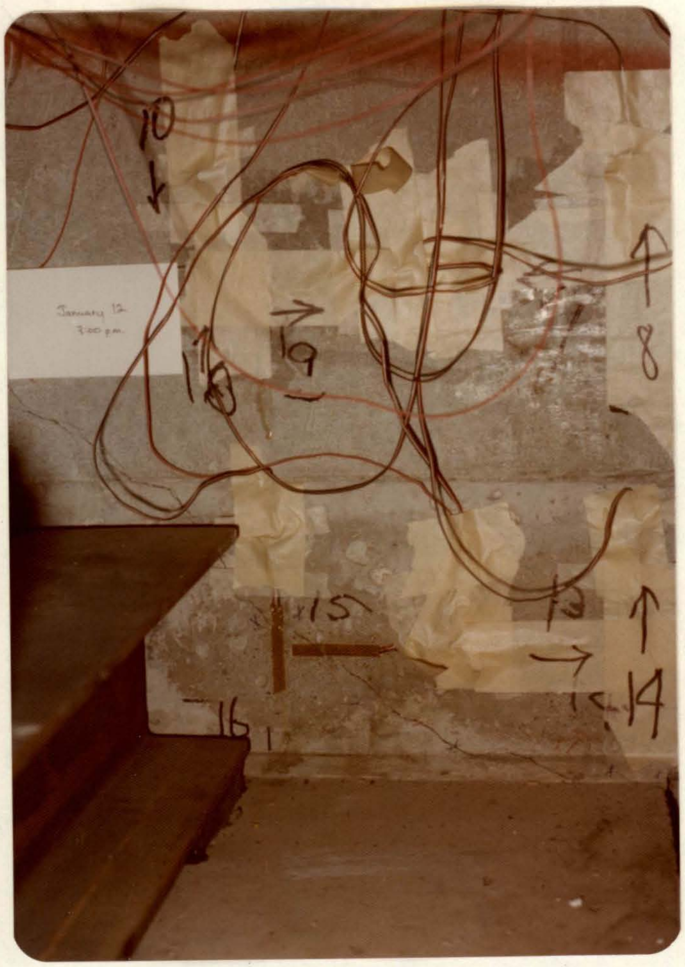
Photograph #14 - Front-View of Stem, displaying Diagonal Hairline Crack at Load Stage #4. (North-End Reaction)



Photograph #17 - Front-View of Stem, Near Shelf, Showing 5 inch and 2 inch Strain Gages (Load Stage #7)

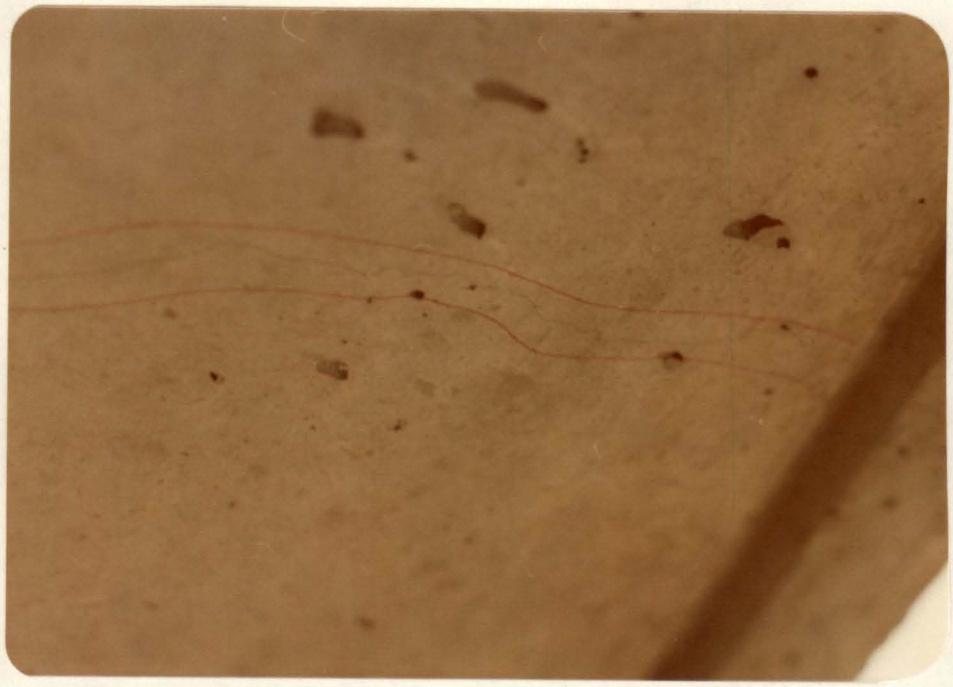


Photograph #18 - Close-up View of Above, (Load Stage #7).



Photograph #19 -

Front-View of Stem, Displaying Diagonal Hairline Crack. (South-End Reaction) Note Crack Propagation Through Strain Gage Numbers 15 and 16.



Photograph #20 - Bottom-View of Shelf Exhibiting Hairline, Flexural Crack - Typical

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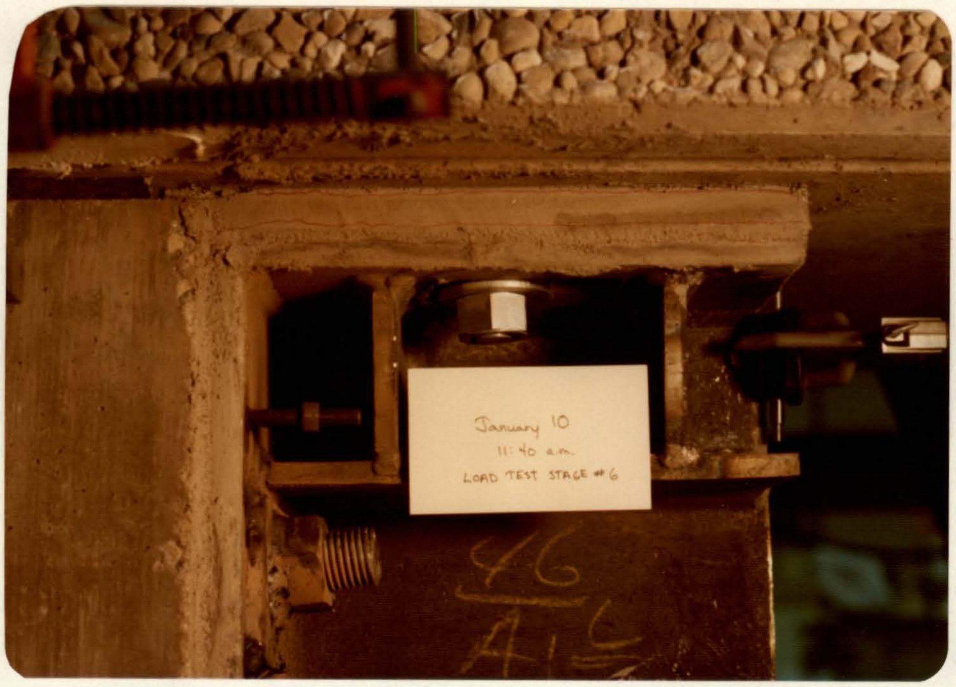
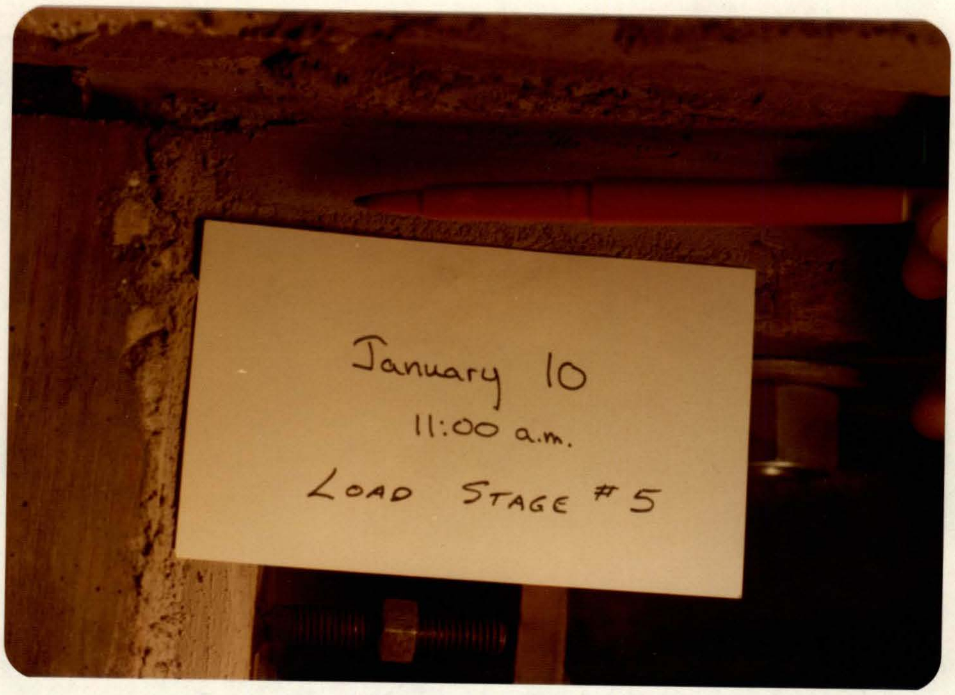
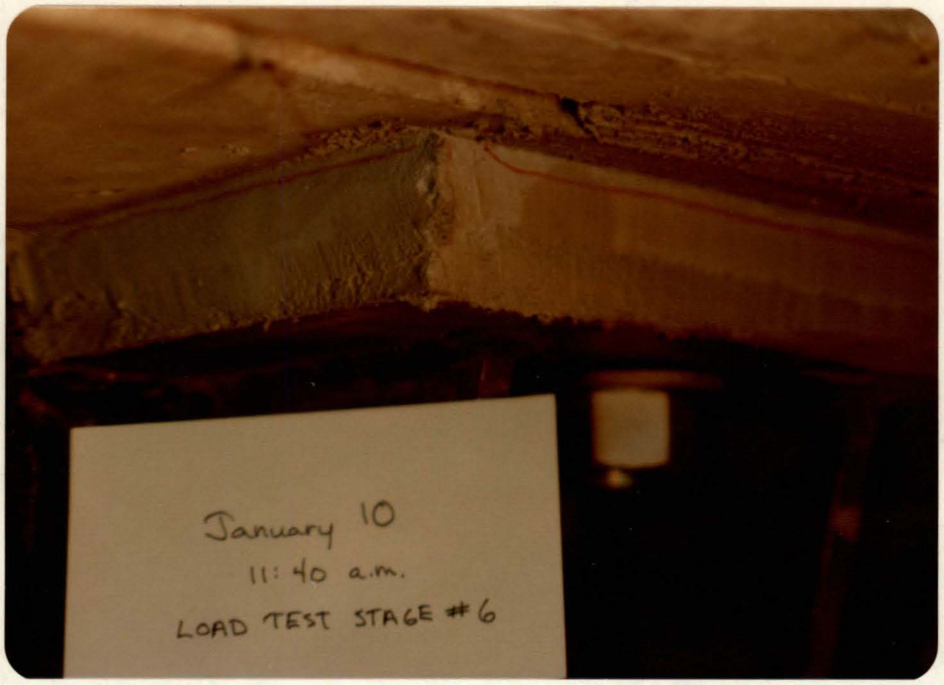


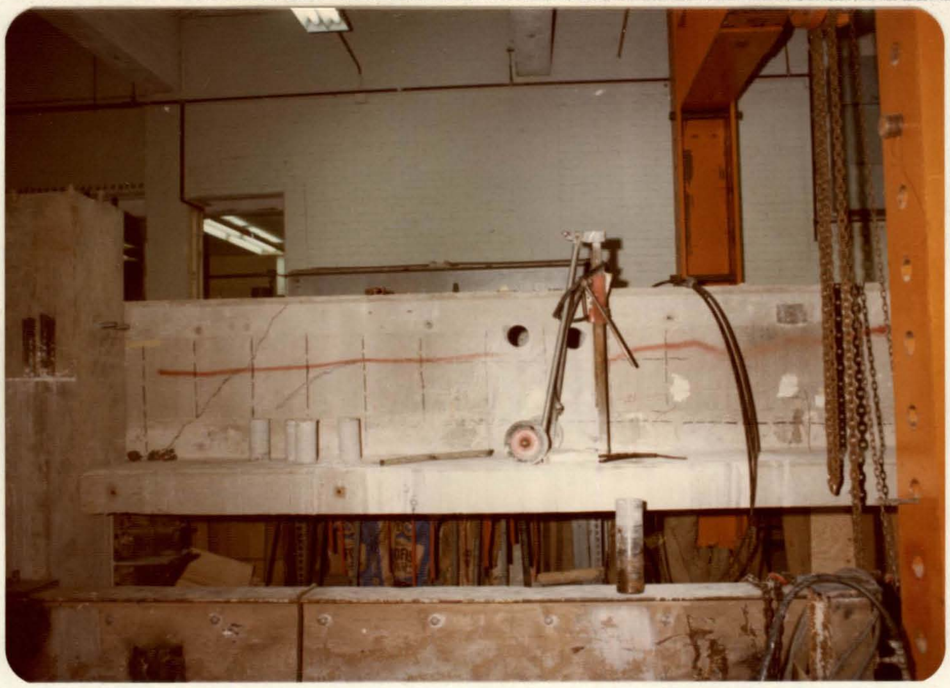
Photograph #21 - Side-View of Steel Haunch with Grout in Place. (Load Stage #6) South-End Reaction.



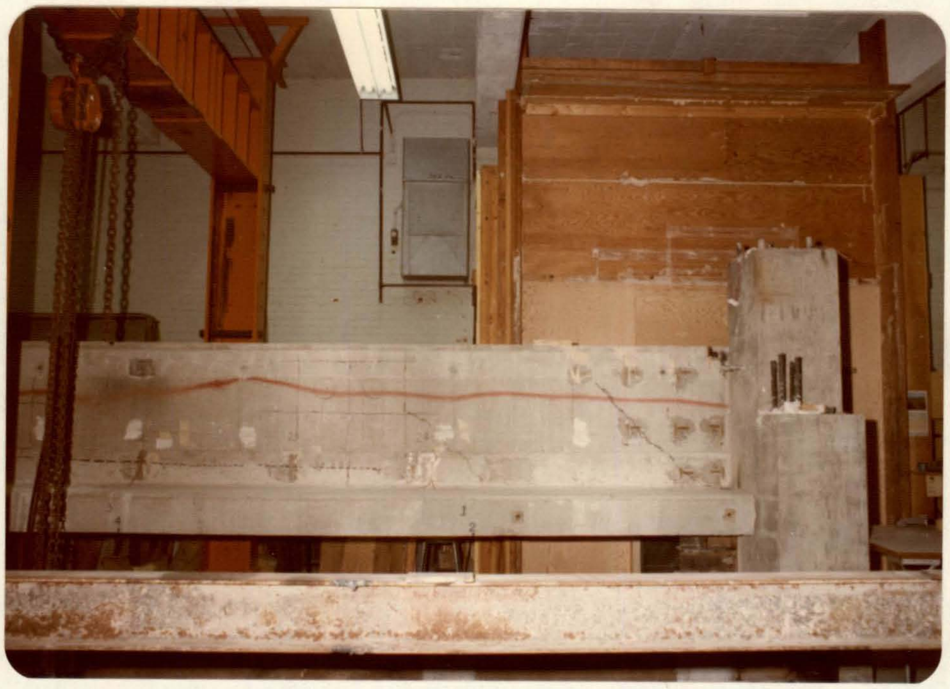
Photograph #22 - Close-up of Above, Exhibiting Horizontal Hairline Crack through Grout. South-End Reaction (Load Stage #6)



Photograph #23 - Close-up of Grout Between the Steel Haunch and Spandrel Beam, North-End Reaction (Load Stage #6)



Photograph #24 - Side-View of Concrete Spandrel Beam, During Coring Operation (North-End Reaction) Note - The Two Diagonal Cracks (near the Column) Outlined in Black.



Photograph #25 - Side-View of Concrete Spandrel Beam, South-End Reaction. Note - the Two Diagonal Cracks (near the Column) Outlined in Black.