Report No.
UCLA/EQSE-87/01

Earthquake and Structural
Engineering Laboratory

FULL SCALE EXPERIMENTAL TESTING OF RETROFIT DEVICES USED FOR REINFORCED CONCRETE BRIDGES

by

Lawrence G. Selna

L. Javier Malvar

Report of Sponsors in Cooperation

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b. The United States of America, Department of Transportation, Federal Highway Administration
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16. Abstract

The seismic strengthening of California bridges began after the 1971 San Fernando Earthquake. Many of the bridge collapses which occurred were attributed to the loss of vertical support at the expansion joints. To ameliorate this problem, structural devices which prevent the opening of expansion joints were developed and deployed in California's highway bridges. Three of the devices which were used are: 1) the type C1 cable restrainer, 2) the bar restrainer, and 3) the deck slab bracket restrainer.

The strength and load-deflection properties of the expansion joints with the structural devices installed is evaluated with full scale experimental testing. The test results show that the bar restrainer experiences a ductile load-deflection behavior while the bars undergo yield. The type C1 and bracket devices are influenced by the cable behavior but their strength is limited by failure of the concrete anchorage.

17. Key Words
Box girder bridge; earthquake retrofit program; expansion joints; full scale experimental test; cable, bar, bracket restrainer

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This report has been prepared for, and in cooperation with: 1) The State of California, Business and Transportation Agency, Department of Transportation; 2) The United States of America, Department of Transportation, Federal Highway Administration; and 3) The United States of America, National Science Foundation.

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PREFACE

After the 1971 San Fernando Earthquake the California Department of Transportation (CALTRANS) adopted a bridge strengthening or retrofit program. Many of the collapses which occurred were attributed to loss of vertical support at the expansion joints and so the retrofit program focused on strengthening of the bridge structure at those locations. Several types of devices were developed. Three of the devices which were used are: (1) the type C1 cable restrainer used to restrain relative hinge seat movements, (2) the bar restrainer which links the hinge joint diaphragms together, and (3) the deck slab bracket restrainer which anchors cables to join the underside of deck slabs on opposite sides of the joint. The present report is concerned with determination of strength, stiffness, and cyclic load-deflection behavior of bridges which have been retrofit with type C1 cable, bar or deck slab bracket restrainers. The properties are evaluated as part of a full scale experimental test program conducted at UCLA. The test results show that the bar restrainer experiences a ductile load-deflection behavior while the bars undergo yield. The load-deflection relation type C1 and bracket devices is influenced by cable behavior and the strength is limited by the reinforced concrete.

The contract monitor at CALTRANS for the experimental test program was Ray J. Zelinski, Senior Bridge Engineer with the Office of Structures Design. His technical and administrative inputs to the project were essential for its completion.

Keywords: *box girder bridge, earthquake retrofit program; expansion joint; full scale experimental tests;* type C1 cable; high strength bar; deck slab bracket; restrainer.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

M_α = applied moment at time of bracket failure

M_r = resisting moment of the bracket

T_r = bolt tension resisting force

b_s = deck slab width between edges of fillets

d = depth of deck slab resisting punching shear

f_i = diagonal tension strength of concrete

f_i = radius of washer used on counterbored bolts

s_b = bolt spacing

x_r = distance between the bolts adjacent to the bearing plate and location of the compression resultant

α = interior angle of intersecting failure surfaces

CHAPTER 1

ORGANIZATION OF REPORT

Design considerations for bridges were radically altered after the 1971 San Fernando Earthquake. The highway bridge collapses which occurred in the San Fernando/Sylmar area demonstrated that the designs and details used were inadequate for bridges built earlier. After the earthquake it was apparent that an alteration of design procedures was required and a statewide project devoted to the strengthening of existing bridges in California was needed. The California Department of Transportation (CALTRANS) initiated a strengthening or retrofit project soon thereafter.

CALTRANS bridge designers using the retrofit devices were concerned about the strength and load - deflection behavior of bridges with the devices installed. These concerns were later expressed as a need for appropriate experimental tests. The present report gives the strength and load - deflection information obtained from a test program conducted on three of the retrofit devices which have been deployed.

The experimental research work was co-sponsored. The two sponsors and the work supported by them is as follows: (1) The National Science Foundation funded the tests on box girder bridge cable restrainers, and (2) CALTRANS with partial funding supplied by the Federal Highway Administration (FHWA) supported the work on restrainer bar and deck slab bracket restrainers. Chapter 2 of the present report is devoted to the work on cables. The work on bars and bracket restrainers is reported in Chapter 3. Both Chapters 2 and 3 are self contained each with their own introduction, summary, and reference sections. General conclusions concerning restrainer behavior are given in Chapter 4, and Chapter 5 is devoted to acknowledgements.

CHAPTER 2

BOX GIRDER BRIDGE CABLE RESTRAINERS

2.1 INTRODUCTION

2.1.1 Background - The life safety of bridge users and property damage to bridge structures in earthquakes are major considerations of bridge structure design. Earthquake damage was pushed to the top of the list of design considerations after a number of collapses occurred during the 1971 San Fernando Earthquake (2-15). An upgrading of structural details and strengthening of existing bridges was needed.

As a design option the strengthening of existing bridges has broad appeal which is created by the following factors:

- The demonstrated seismic vulnerability of existing bridges (2-15),
- The large inventory of existing structures,
- The great expense of inventory replacement,
- The paucity of new construction in effect for a period after the earthquake.

Retrofitting is a term synonymous with strengthening and rehabilitating components of bridges. The retrofitting can significantly improve the seismic performance of bridge structures, i.e., the characteristics approach those required in new construction. The cost of retrofitting is much less than new construction. Considering the low cost and the high level of performance, retrofitting is the logical choice.

During California earthquakes (2-7, -10, -15), the bridge collapses occurred after the relative movement between two adjacent spans exceeded the hinge seat widths.

Retrofit devices which are intended to overcome the seat width problem are called hinge restrainers (2-5, -7, -8, -9, -10, -13, -14, -23, -27). Among the types used is the type C-1 cable restrainer (Fig. 2-1), consisting of 7-3/4 in (19 mm) diam. galvanized cables, bearing plates, drum, and bolster.

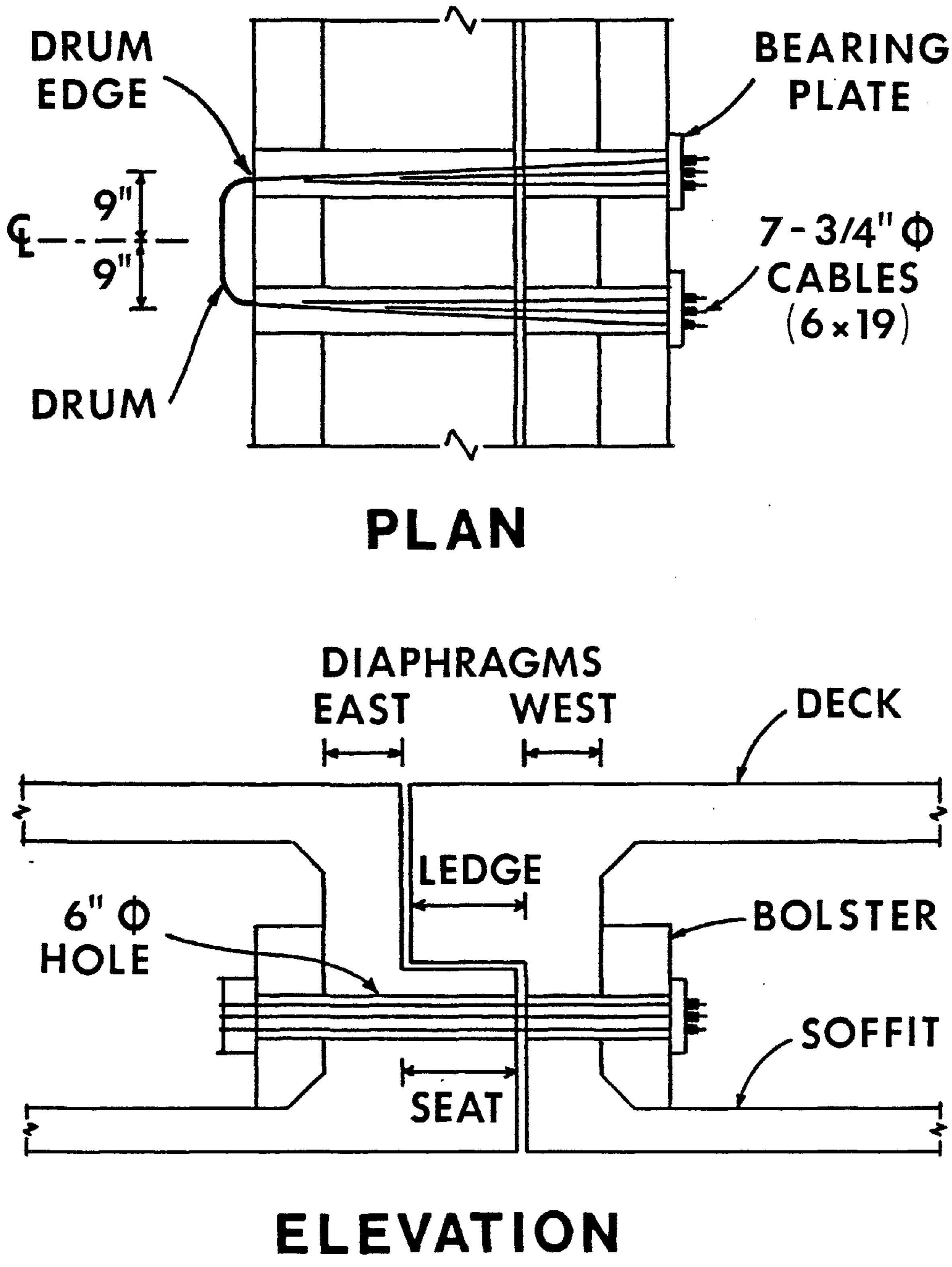

Fig. 2-1 - Type C1 Cable Restrainer
(1" = 2.54 cm)

The interaction between the tension cables and the hinge diaphragm, deck, and web elements in the box girder is a salient issue of restrainer design. When the restrainer is activated, these elements must carry forces which were not anticipated in their original design. Materials testing of the new elements, i.e., cable and bolster materials do not reveal the strength and stiffness properties in the existing diaphragm, deck and web elements. CALTRANS bridge engineers (2-16) realized that the existing elements play a major role in restrainer response and expressed their desire for an experimental research program to evaluate the extent of existing element participation. A further justification is given to this experimental research because these retrofit details are under consideration for use throughout the USA (2-18).

Structural testing is the dependable method for determining load-deflection and strength properties of the restrainer and adjacent concrete components.

2.1.2 Objective - The objective of Chapter 2 is to present the results and findings of a structural test program performed on a full scale representative portion of a box girder bridge hinge which has been retrofit with hinge diaphragm cable restrainers.

2.2 FULL SCALE STRUCTURAL TESTING

2.2.1 Specimen Characteristics - A full scale representative portion of a reinforced concrete box girder bridge which includes the hinge is constructed. The specimen (Fig. 2-2) is 4 ft. (1.22 m) high and 10 ft. (3.05 m) wide. The combined length of the specimen is 19 ft. (5.8 m). The height dimension is typical for a reinforced concrete box girder bridge with a 70 ft (21.3 m) span. The width of the specimen conforms to typical web spacings (2-7). The width and thickness dimensions of the reinforced concrete box girder elements are presented in Table 2-1. In general the height of the bolster above the top of the soffit slab is equal to 50% of the overall box girder height. Therefore, the height of the bolster used in the specimen is 24 in. (610 cm).

The reinforcement used in the specimen is similar to the bar sizes and locations of existing

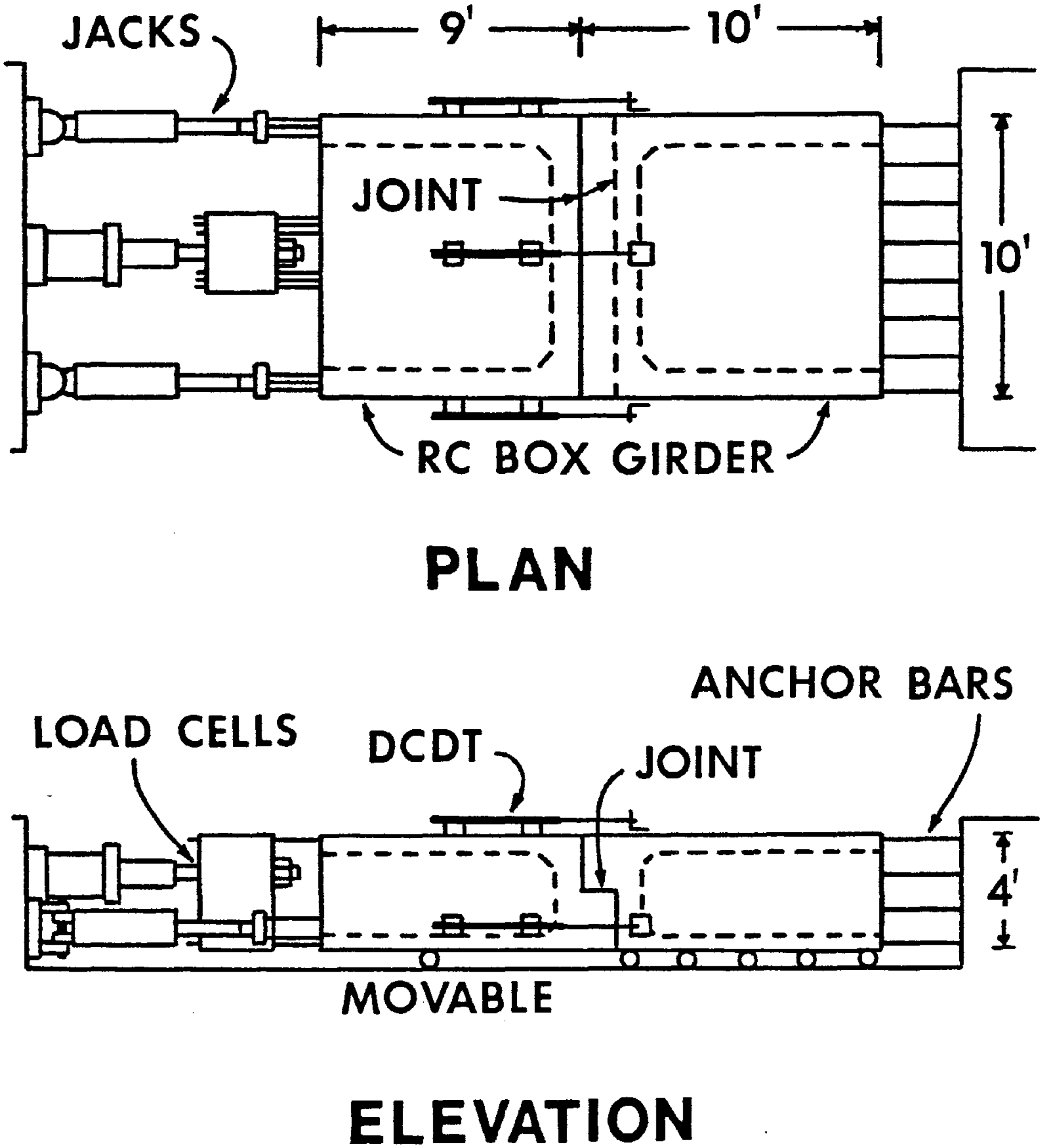

Fig. 2-2 - Test Specimen and Testing Arrangement (1 ft = 30.48 cm)

Table 2-1 - Dimensions of Reinforced Concrete Box Girder Elements and Galvanized Steel Type C1 Cable Restrainer Hardware.

Box Girder Element	Width or Thickness in	cm
Deck Slab	7.5	19.1
Soffit Slab	5.5	14.0
Bolster	9.0	22.9
Diaphragm	10.0	25.4
Stem or Web	12.0[a]	30.5[a]
Seat	Variable[b]	Variable[b]
Restrainer Hardware	t x w x h[c] in.	t x w x h[c] cm.
Bearing Plate	2 x 10 x 10	5 x 25 x 25
Drum	4 1/4 x 16 x 10	11 x 41 x 25

Notes:

a. Normal stem thickness is 8 in (20.3 cm). 12 in. (30.5 cm) thickness is used to accommodate specimen anchor bars.

b. Seat widths vary between 6 in. (15.2 cm) and 15 in. 38.1 cm) on existing bridges. Specimen has 15 in. (38.1 cm) seat width.

c. thickness x width x height (t x w x h) bridge hinges (Fig. 2-1). The reinforcement details used in existing bridges are changing with the date of construction. A standard plan (2-24) which is most representative of retrofitted bridges is followed, but minor variations from the standard plan (2-24) do occur in the specimen in order to accommodate details used at other times. Ref. (2-2) shows box girder configurations and reinforcement details which are considered by bridge engineers. The reinforcement used in the bolster which is added during retrofit projects follows CALTRANS details (2-23).

Grade 60 reinforcement is used throughout the specimen. The material properties of the reinforcement are in compliance with CALTRANS specifications (2-22). Stone concrete used in constructing the box girder specimen conforms with CALTRANS specifications (2-21).

2.2.2 Test Arrangement - During a test, tension forces are exerted by the jacks (Fig. 2-2) through anchorage reinforcement to the movable seat side of the specimen (2-12, -19). This creates tensile forces between the jacks and anchor bars, i.e., in the restrainer and in the ledge side of the specimen which in turn is anchored through anchor bar reinforcement to a reaction block. The movable section of the bridge specimen sits on a 2 3/4 in. (7.0 cm) diameter x 10 ft. (30 m) long stainless steel roller. Forces lost to rolling friction are negligible. The sum of forces which are sensed by the load cells on the three jacks gives a true indication of force in the restrainer being tested.

The relative displacements at the joint (Fig. 2-2) between the anchored side and movable sides of the specimen are sensed by three direct current displacement transducers (DCDT's).

Out of plane horizontal displacements occur in the diaphragms due to type C1 cable forces. The displacements are measured with linear variable displacement transducers (LVDT's) mounted on backup frames which are anchored to the webs of the box girder specimen, i.e. near the DCDT joint transducers. Consequently the displacement elongation of the restrainer cable plus hardware deformation is obtained by a simple subtraction of the seat and ledge diaphragm displacements from the joint displacement.

2.3 CABLE BEHAVIOR

2.3.1 Fundamentals - CALTRANS uses 3/4 in. (19 mm) diameter cable in its guard rail end anchors and retrofit assemblies. More precisely the cable is wire rope (2-17) with the following characteristics: 1) 6 strands, 2) 19 wires per strand, 3) right regular lay (2-6), 4) independent wire rope core, 5) improved plow steel, 6) galvanized wires, and 7) area = 0.222 sq. in. (143 sq. mm) (2-20).

Uniaxial strength testing is performed on retrofit construction projects in order to confirm the compliance with Standard Specifications (2-20). Uniaxial stress-strain data for restrainer cable are also available to bridge designers. Two stress-strain curves are presented (2-4, -14); the stress-strain histories are as follows: a) Curve 1 - monotonic loading to the yield load of 46 kips followed by 14 cycles of unloading and reloading between yield and zero force, and then resumption of monotonic loading to failure, and b) Curve 2 - load to 1% strain unload to zero force, reload to 2% strain unload to zero force, etc. There are common characteristics in both stress-strain curves (2-4, -14), i.e., they both have virgin and unload-reload branches. The numerical description of the characteristics is:

1. the virgin curve can be adequately represented with a bilinear relation which has a yield point at 46 kips (205 kN) and an ultimate strength of 56 kips (245 kN);
2. the modulus of the steep segment of the virgin curve up to the yield point is 10,000 ksi (68,950 MPa) and the modulus of the flat segment is 1200 ksi (8,274 MPa);
3. the unload-reload branch has a modulus of 18,500 ksi (128,000 MPa) and is a narrow hysteresis loop; and
4. the maximum strain at failure is 5%. Detectable variation in the strength, stiffness, and maximum elongation properties is evinced in the curves (2-4, -14) due to the differences in the stress-strain histories.

In some publications (2-3, -6, -17) a "prestretched modulus" hereafter designated as E_p is recommended for design use. The prestretched modulus E_p, has a value which is in general agreement with the unload-reload branch modulus given in the stress-strain curves (2-4, -14). The modulus of the virgin curve up to yield which is approximately 50-60% of E_p is hereafter designated as E_v.

When installed in a bridge restrainer device, the cable is not prestretched. During the load-deflection experience of an earthquake, the cable will be loaded on the virgin branch and unloaded-reloaded on the prestretched branch.

2.3.2 Type C1 Restrainer Cables - do not have straight alignment (Fig 2-1). Instead the cables pass around two 90° bends on the drum. The radius of the 90° bends is 4 in. (102 mm). At these locations there are contact stresses between the cables and the drum. The distribution of the compression stresses is not uniform with the greater values occurring at the edge of the drum (Fig. 2-1). Also there are interface shear stresses between the cables and the drum of indeterminate magnitude and distribution.

The uniaxial stress-strain properties of the cables are affected by the contact stresses (2-11, -25). An empirical relation depicting the strength reduction for cables bent around pins is available (2-11). The strength efficiency, Eff., in percent is given by

$$\text{Eff.} = 100 - 50 \times R_r^{-0.5} \qquad (\%) \text{ when } R_r \leq 6 \qquad (1a)$$

$$\text{Eff.} = 100 - 76 \times R_r^{-0.73} \qquad (\%) \text{ when } R_r \geq 6 \qquad (1b)$$

in which R_r = ratio of pin diameter to wire rope diameter. For the 3/4 in. (19 mm) wire rope bent around a 4 in. (102 mm) radius the Eff. = 86.5%, i.e., the strength of the rope is 86.5% of its uniaxial value.

The reduction of maximum elongation due to contact stresses is not reported in the literature. Experimental structural testing with 3/4 in. (19 mm) 6 x 19 cable installed in a type C1 restrainer is performed in order to determine the extent of the reduction.

Single cables installed in type C1 restrainer hardware, i.e., around the drum (Fig. 2-1), are tested to failure in the full scale box girder test specimen (Fig. 2-2). Four tests on single cables are performed. The concrete components in the box girder do not crack during the tests, and the horizontal displacement of the hinge diaphragm is small. The average of joint displacements measured with the 3 DCDT's (Fig. 2-2) is practically equal to the elongation displacement of the cables. The sum of forces measured by the load cells is equal to the force carried by the cables.

The gage length used in all type C1 tests is found by taking 1/2 x length of cable plus 3 in. (7.6 cm) for extension into the swag fitting. The gage length is 60 in. (152.4 cm) for the full scale box girder test specimen.

The results are presented in Table 2-2 in the row entitled "Box Girder." The tested ultimate strength is 103 kips (458 kN) which is 9% smaller than the tested uniaxial value. The measured reduction in strength is in good agreement with the prediction (2-11) which indicates a 13.5% reduction below the uniaxial value due to bending of the cable. The tested modulus values (Table 2-2, "Box Girder") for the type C1 "U" shaped cables are somewhat higher than the uniaxial values.

The reduction of ultimate strain due to drum contact stresses is dramatic (Table 2-2). The ultimate strain is reduced from an average value of 4.9% for the uniaxial case to an average value of 2.0% in the type C1 application. The cable failure invariably occurs at the edge of the drum (Fig. 2-1), i.e., at the start of the bend. In cases where the cable is removed from the drum before failure, peening marks can be seen on strands where they press against the edge of the drum.

Additional testing is performed on type C1 cables by Thorsteinsson (2-25). He fabricated fittings so that the type C1 bearing and drum configuration could be installed in a "displacement controlled" testing machine. The purpose of the additional testing is to confirm that the type C1 cable was limited to an ultimate strain of 2%.

The results of Thorsteinsson's tests are presented in Table 2-2 in the row entitled "Testing Machine," and in Fig. 2-3. The strength results are in good agreement with those found in the box girder tests. The moduli are 10% higher than the box girder results due to diaphragm deformations in the bridge specimen. The limited ultimate strain of 2% was confirmed with these tests. Failure due to peening of strands where they cross the edge of the drum was also evident in the testing machine series.

Table 2-2 - Cable Tests and Comparison of Results
For 3/4" (19 mm) diameter,
6 x 19 Galvanized Wire Rope.

Test Program	Num. Tests	Statistic	Ult. Stress (kip)	Virgin Elas. Modulus, E_v (ksi)	Prestr. Elas. Modulus, E_p (ksi)	Ultimate Strain
Uniaxial Curve 1[a] - (2-4, -14)	-	-	111[c]	10,300[d]	18,000[d]	0.0535[e]
Uniaxial Curve 2[b] - (2-4, -14)	-	-	111[c]	10,500[d]	19,100[d]	0.0452[e]
Box Girder	4	Mean	103.0[c]	16,400	22,800	0.0203
		Std. Dev.	2.5	1,500	340	.0013
Testing Mach. (2-25)	4	Mean	105.7[c]	17,600	24,800	0.0203
		Std. Dev.	0.6	500	1,300	.0002

Notes:

a. Load to yield, then cycle 14 times between yield and zero, and then continue to failure.

b. From zero keep incrementing strain by 1% then unloading; repeat until failure.

c. Two times single cable strength.

d. Approximate values.

e. Does not include 0.25% strain which occurs during concave initial portion of virgin branch.

Units: 1) 1kip = 4.448 kN, 2) 1 ksi = 6.895 MPa

Area = 0.222 in^2 (143 sq. mm)

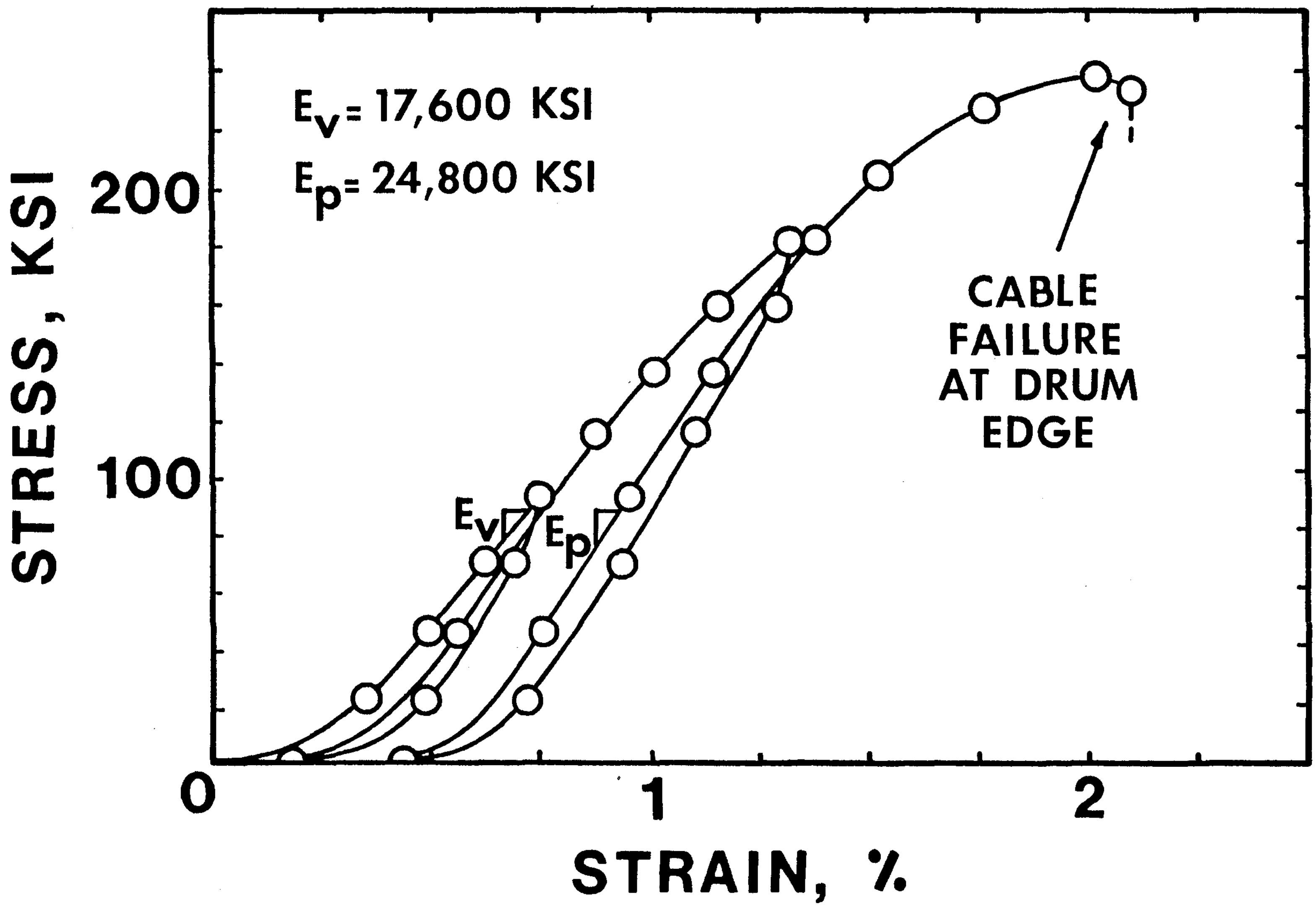

Fig. 2-3 - Stress Strain Curve for Type C1 Cable Arrangement Installed in a Testing Machine (1 ksi = 6.895 MPa)

2.4 DESIGN

2.4.1 Strength - the "weak link" considered (2-7, -14) in restrainer design is the restrainer hardware, i.e., ultimate failure is supposed to occur in the cables. The engineering information available (2-4, -14) is devoted to strength, stiffness, and ultimate elongation of uniaxial cables. Ductile design procedures are recommended, i.e., connections are to be designed to resist forces which are 25% greater than the ultimate strength of the cables. Strength data (2-14) for cable materials to be used in design are presented in Table 2-3.

At this juncture the restrainers are usually designed with an equivalent static analysis procedure (2-13) which represents longitudinal displacements across the joint. The bridge frames or spans on both sides of the joint are assumed to deflect away from each other. The longitudinal stiffness of the frame plus restrainer is computed by mobilizing the longitudinal stiffness of one adjacent frame or abutment in addition to the frame and restrainer under consideration, i.e., after all gaps have been closed. The acceleration-response-soil (ARS) spectrum is used to determine the loads to be applied in the analysis. A sufficient number of cables is selected so the forces in them are limited to the design yield strength (Table 3). Displacement limits are also imposed. The joint displacement is restricted to the available hinge seat width, i.e., a value less than the actual seat width. Friction forces are neglected.

2.5 TEST RESULTS AND INTERPRETATION

2.5.1 Type C1 Cable Restrainer - Two type C1 seven cable tests are performed. The load-deflection results and modes of failure are the same so that the discussion pertains to both tests. Figs. 2-4,-5, and -6 give the load-displacement data point results for the joint, restrainer, and diaphragms respectively. The joint displacement is controlled by the jacks. The sum of cable and diaphragm displacements is equal to the joint displacement. The load-joint displacement history (Fig. 2-4) imposed by the jacks in displacement control and the observed specimen behavior are as follows:

Table 2-3 - Cable Material Strength
And Stiffness Quantities to be Used in Design (2-9, -14).

Strength/Capacity			Design Value kips	kN
Galvanized wire rope 3/4 in. (19 mm) diameter, Area = 0.222 sq. in. (143 sq. mm)				
Minimum Breaking Strength			46	205
Working Capacity	0.5 x 46	=	23	102
Seismic Working Capacity	1.33 x 23	=	31	136
Yield Capacity	0.85 x 46	=	39	174
Ultimate Strength			53	236
Stiffness, i.e., Elastic Modulus			ksi	MPa
Cable, E_v / E_p			10000/18000	68950/124000
Ultimate Strain			strain	
Cable			0.044	

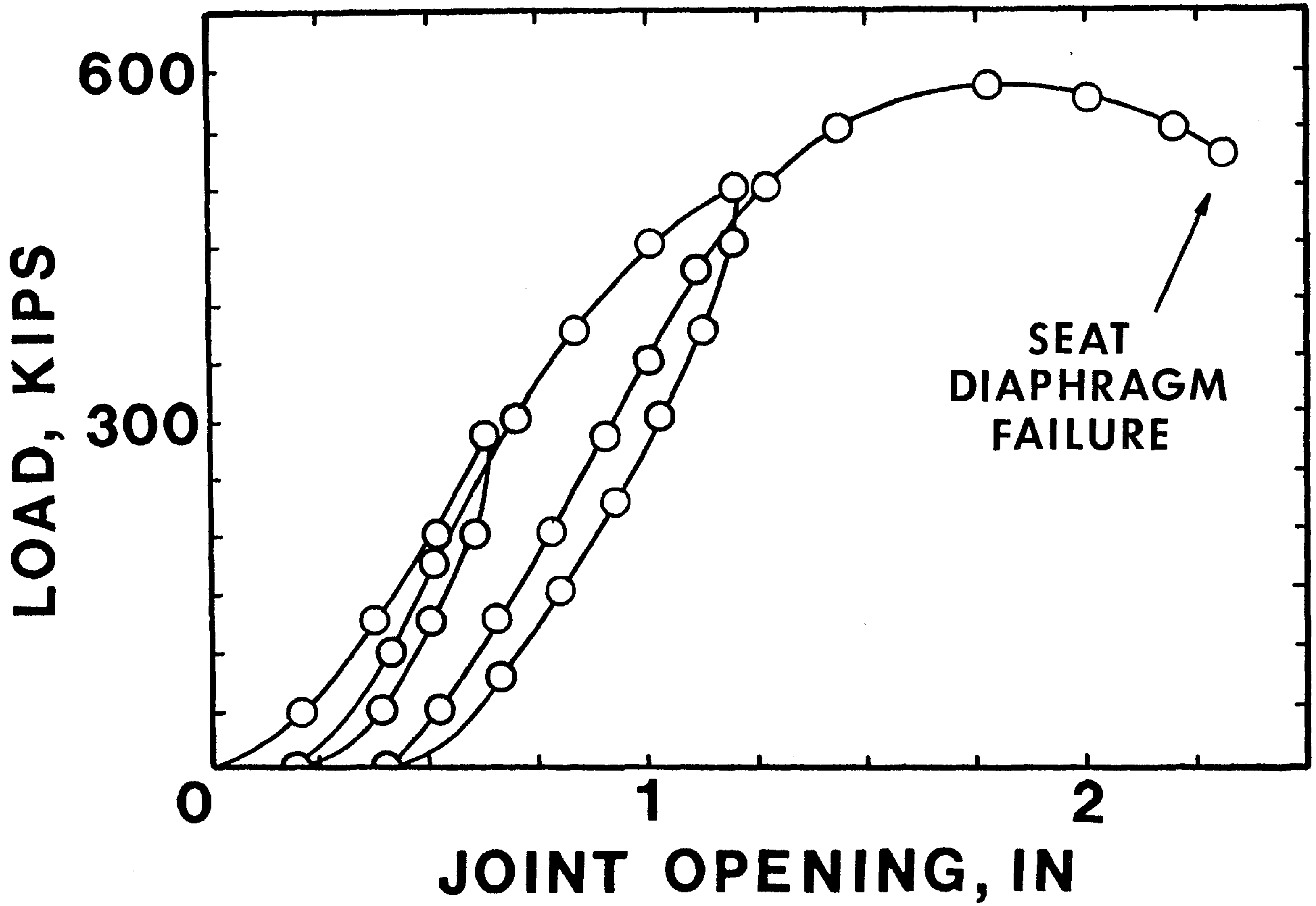

Fig. 2-4 - Load-Displacement Behavior at Joint for Type C1 Restrainer (1 kip = 4.448 KN; 1 ksi = 6.895 MPa)

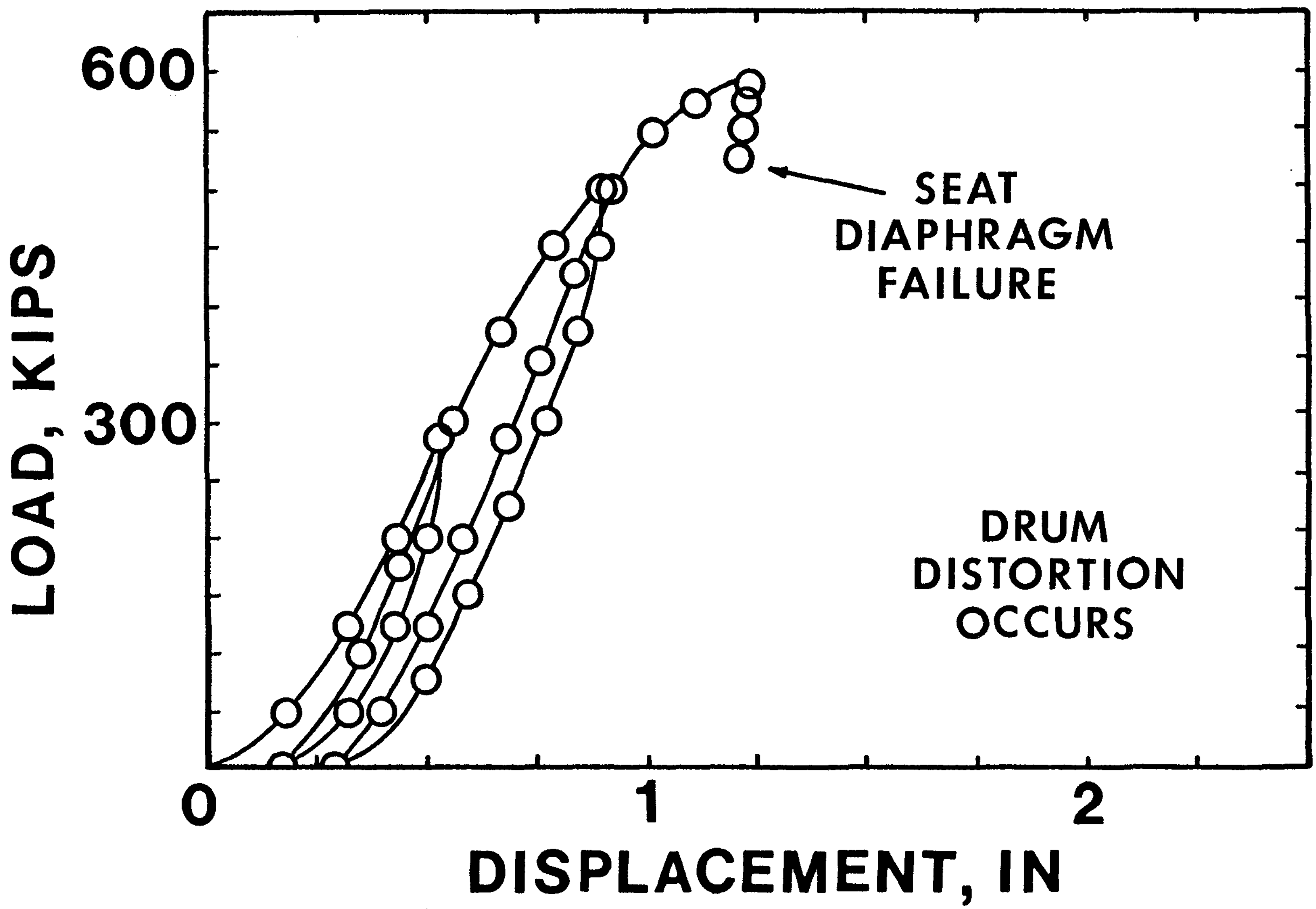

Fig. 2-5 - Restrainer Load versus Cable and Hardware Displacement, Type C1 Seven Cable Test (1 kip = 4.448 KN; 1 ksi = 6.895 MPa)

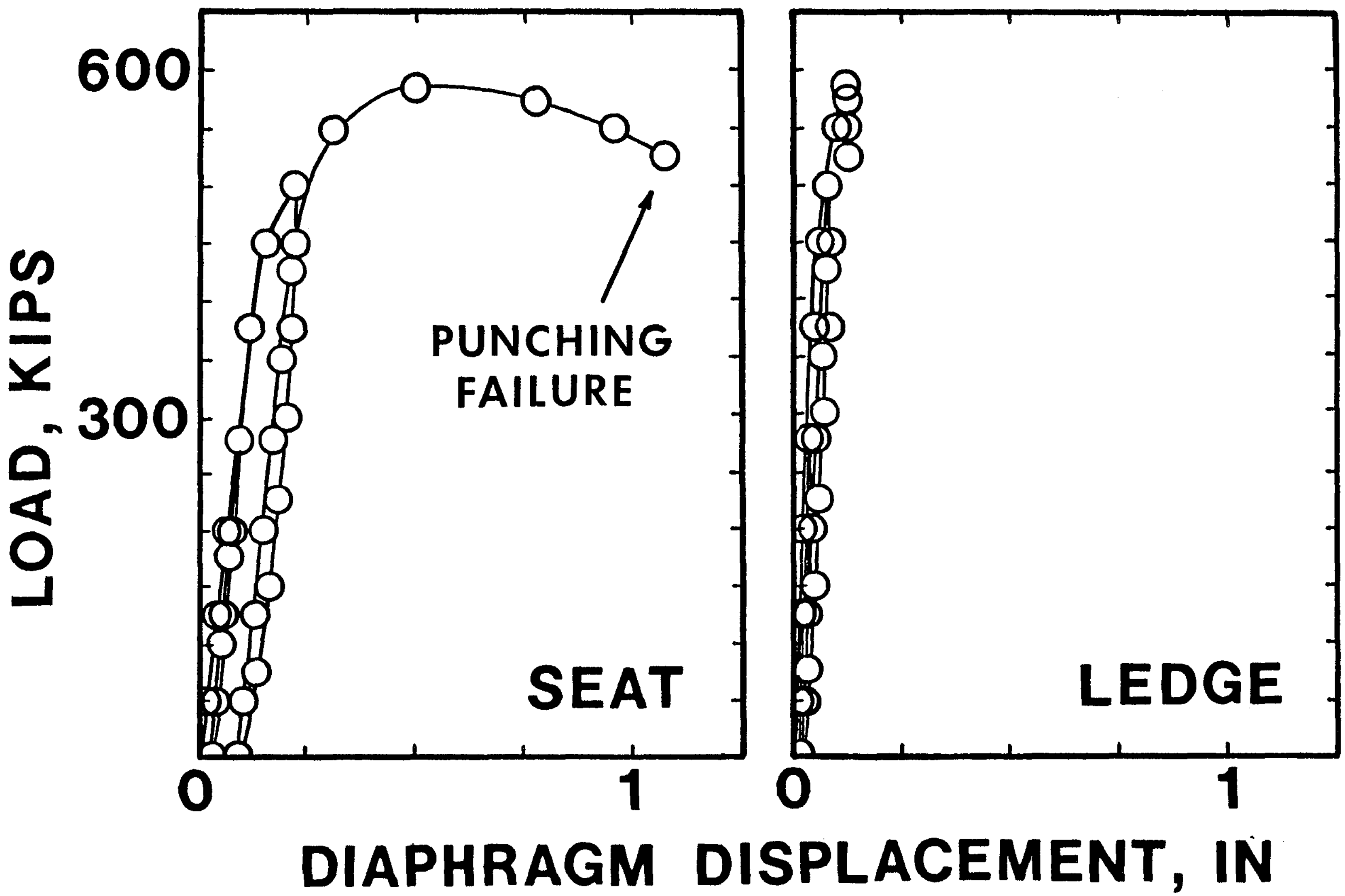

Fig. 2-6 - Load-Displacement Relations of Hinge Diaphragms,
Type C1 Restrainer (1 kip = 4.448 KN; 1 in = 2.54 cm)

1. Increase the load and displacement from zero until the load reaches 280 kips (1245 kN); the corresponding joint displacement is 0.625 in. (15.9 cm). The load level corresponds to 40 kips (178 kN) per cable, i.e., 20 kips (89 kN) per leg of the "U", and is within the working range. Cracking is not noticeable on either the seat or ledge side of the specimen.

2. Perform three unload-reload loops, i.e., cycle three times between 0.625 in. (15.9 mm) and 0.25 in. (6.3 mm) which corresponds to zero load. After the third loop the load is 260 kips (1156 kN) and the joint displacement is 0.625 in. (15.9 mm). The hysteresis curves stay within the smaller loop shown (Fig. 2-4).

3. Increase the load to 500 kips (2224 kN) and the joint displacement is 1.118 in (28.4 mm). The load level is 71 kips (316 kN) per cable, i.e., 36 kips (160 kN) per leg of the "U". The load is 3 kips (13.3 kN) short of the design yield capacity per cable leg, i.e., 39 kips (173 kN). There is noticeable horizontal cracking on the face of the diaphragm above the seat, indicating vertical bending of the diaphragm between the deck and soffit slabs. Horizontal bending of the seat between the webs is also evident from vertical cracks which occur on the front face of the seat near to its mid-length.

4. Cycle three times between 1.118 in. (28.4 mm) displacement and zero load. The hysteresis curves stay within the larger loop shown (Fig. 2-4), i.e., the energy dissipation and the stiffness degradation are small. When the displacement is returned to 1.188 in. (30.2 mm) at the end of the third cycle, the load reaches 475 kips (2113 kN).

5. Increase the joint displacement from 1.188 in. (30.2 mm) and continue beyond that corresponding to the ultimate strength of the system. Displacement is stopped at 2.29 in. (58.2 mm)(Fig. 2-4). The load passes through a maximum of 585 kips (2602 kN) at a displacement of 1.75 in. (44.5 mm) and then descends to 525 kips (2335 kN) as 2.29 in (58.2 mm) displacement is reached. Beyond that point there is a sudden loss of resistance, a "cracking" sound, and a punching shear failure. The maximum resisting load of 585 kips (2602 kN) exceeds the design yield capacity of seven cables each with two legs, i.e., 14 x 39 = 546 kips (2,429 kN).

The load-displacement results (Fig. 2-5) for the restrainer, i.e., cables plus hardware, show a similar form in behavior as is evinced in Fig. 2-3. The unload-reload looping is conducted at similar stress levels. However, there are differences in the results between Figs. 2-3 and 2-5. The ultimate capacity of seven cables based on testing machine results (Table 2-2) is 740 kips (3292 kN) which exceeds the type C1 box girder test result by 155 kips (689 kN). Subsequent to the test the cables are removed from the box girder specimen. No wire breakage or peening is detected. The cables are in excellent condition, and the loss in resistance occurs in the seat diaphragm.

Another difference between Figs. 2-3 and 2-5 is evident in the stiffness. The representation of cable plus hardware displacement (Fig. 2-5) is more flexible than the cable alone (Fig. 2-3) because additional displacements occur in the hardware during a seven cable test. After the test the drum is examined. It is evident that cable forces distort the drum, i.e., bend the drum into the 6 in. (15.2 cm) holes (Fig. 2-1). A permanent drum distortion of 1/4 in. (6.4 mm) is present after the test.

The horizontal displacements of the seat and ledge (Fig. 2-1) diaphragms are measured. The measurements are taken at stations on the drum and bearing plates which are directly bearing on the bolsters. The load-displacement results for the two diaphragms are presented in Fig. 2-6. The ledge diaphragm remains elastic throughout the test. The seat side is more flexible and has a lower yield strength.

The bolster adds much more stiffness on the ledge side. The thin 10 in. (22.9 cm) part of the diaphragm is thickened to 19 in. (48.3 cm) with the addition of the bolster on the ledge side while the bolster adds a thickened portion on the seat side for only 1/3 of the clear height of the thin part of the diaphragm. The measured stiffness (Fig. 2-6) on the ledge side is 3 times the measured stiffness on the seat side while the load is in the elastic range.

Flexure cracks develop on the seat side after the load surpasses 450 kips (1779 kN), and the reduction in stiffness is apparent (Fig. 2-6). Three unload-reload loops at approximately 500 kips (2224 kN) show that the stiffness degradation and energy dissipation are small, but there is a noticeable permanent offset. The response at this load level is flexural and the affected bars on the front of the diaphragm and seat remain well anchored at their ends. The offset is due to local yielding and slip.

As the load is built up to the failure level, 585 kips (2602 kN), the flexure cracks deepen sufficiently so that a punching shear prism (Fig. 2-7) is formed. Thereafter the resisting load decreases while the seat diaphragm displacement increases (Fig. 2-6).

The cracking patterns indicate that flexure and shear models are appropriate for calculating the strength. A flexure yield line model (2-26) predicts a pull-out strength of 422 kips (1877 kN) when f_y = 60 ksi is used. A punching shear calculation (2-1) predicts 407 kips (1810 kN).

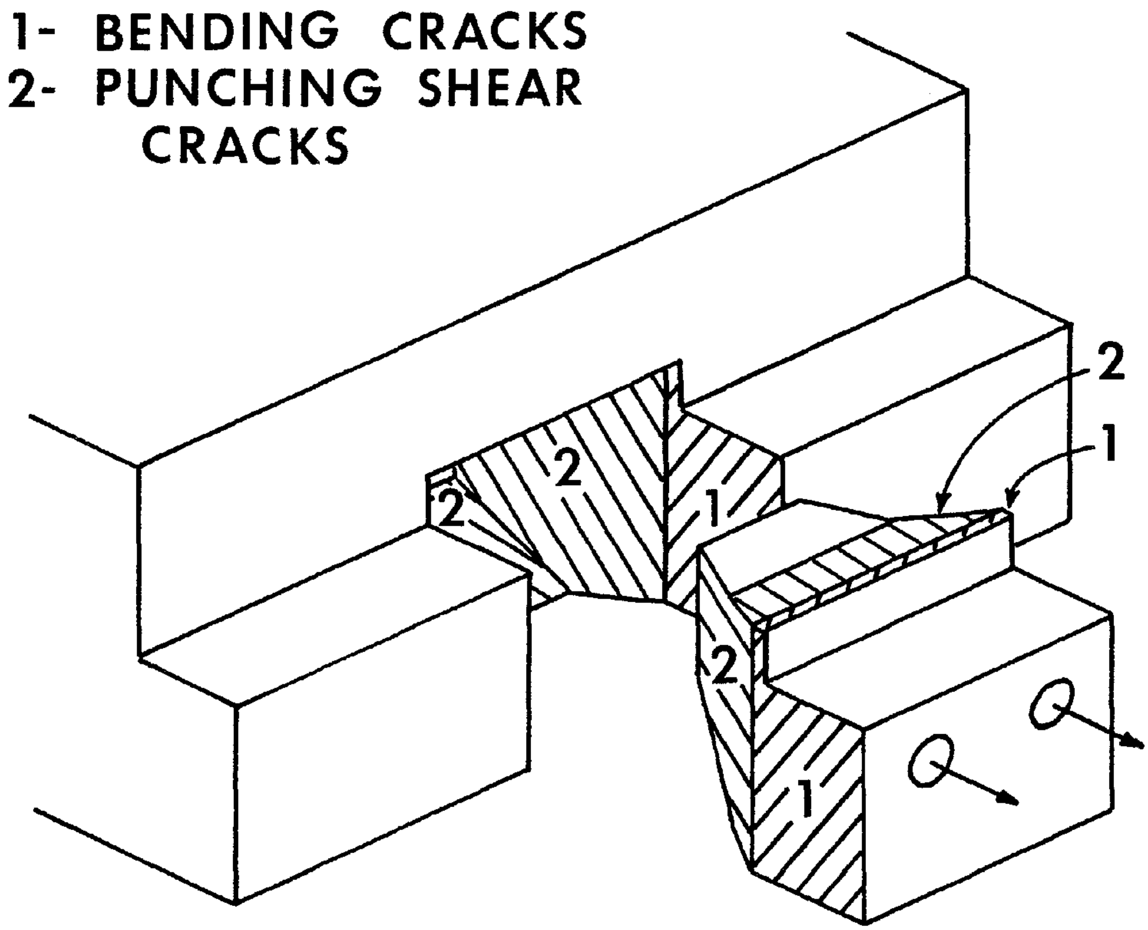

Fig. 2-7 - Seat Diaphragm Punching
Shear Failure Surface

2.6. FINDINGS AND IMPLEMENTATION

2.6.1. Findings - during the course of the experimental program a number of relevant items are revealed. They are as follows:

1. The load-deflection behavior of restrainer cables is influenced when they are bent around a radius, e.g., around the drum used in a type C1 installation. The strength is not changed significantly due to the bend in the type C1 case, but the stiffness is increased, and the ultimate strain is reduced 2.5 times when compared with a straight cable.

2. When tested to failure the type C1 installation experiences a loss of resistance due to a reinforced concrete punching shear failure in the seat side of the hinge diaphragm. A yield line flexure mechanism is well developed in the seat and diaphragm before the punching shear failure occurs. The resisting force of the installation at failure is slightly greater than the design yield strength of the cables.

3. The pull out failure near the middle of the diaphragm does not substantially reduce the vertical load carrying capacity of the hinge because the seat in the web or stem regions of the box girder remains intact. However, displacement accompanying the failure may exceed the seat width causing vertical collapse in hinges having very narrow seats.

4. Although modes of failure have been determined in the present experimental investigation, it must be borne in mind that similar failures will not necessarily occur during future ground shaking at bridge sites. Furthermore, the installation of type C1 restrainers strengthens the seismic resistance of bridge structures.

2.6.2 Implementation - there are suggested changes which could improve the performance of the type C1 installations. They are:

1. The radius of the bend on the drum can be increased and the leading edge should be rounded so that strength and, more importantly, the ultimate strain of the cable is increased.

2. Failure in the hinge diaphragm should be eliminated so that the cables become the "weak link" in the type C1 installation. The bolster strength can be increased sufficiently to prevent or dramatically reduce the likelyhood of reinforced concrete failure. The design steps to be followed are: a) select a bolster load which is greater than or equal to upper bound cable capacity force, i.e., choose a design load which is high enough so that failure will not occur in the bolster; and b) choose bolster dimensions and reinforcement detailing so that $\Phi = 0.85$ times ideal bolster strength exceeds the design load selected in "a)". For example, it would be helpful to design a full height bolster which is independent of the hinge diaphragm. There must be sufficient thickness with flexure and shear reinforcement and doweling into slabs and webs. The strength of the webs and slabs against longitudinal pull-out must also be checked.

3. For box girders with superstructure depth greater or equal to 48 in. (122 cm), hinge seat widths less than or equal to 15 in. (38.1 cm), and reinforcement in good agreement with details

followed in the specimen it is recommended that the hinge diaphragm/bolster pull-out capacity can be satisfactorily evaluated with a moment yield line procedure. The computed capacity should exceed the aggregate of cable ultimate strengths.

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CHAPTER 3

HIGH STRENGTH BAR AND DECK SLAB CABLE/ BRACKET RESTRAINER

3.1 INTRODUCTION

Reinforced concrete highway bridges experienced collapse at sites of strong ground shaking during the 1971 San Fernando Earthquake (3-4). Soon after, the California Department of Transportation (CALTRANS) began to strengthen existing bridges in the most densely populated areas with high seismicity (3-2, -3). The strengthening was accomplished by adding reinforced concrete and/or hardware elements at critical locations in the bridge structure. The process is called retrofitting.

The hinges in the superstructures tended to open during the earthquake. Before 1971 the bridges were constructed without significant longitudinal restraint through the hinges (3-2). During the seismic dynamic response, the adjacent frames moved away from each other. When the relative movement between adjacent frames equalled the hinge seat width, the suspended portions of the spans lost vertical support and collapsed.

Since 1972 approximately 1300 bridges have been retrofit. The superstructure retrofit program is essentially complete. Longitudinal hinge restrainer retrofits were the most common of all the types.

Among the types of longitudinal restrainers used (3-3, -4) were: 1) the bar restrainer (Fig. 3-1), consisting of 1 1/4 in. (32 mm) diam. galvanized threaded high strength bars satisfying ASTM A 722 (3-5), nuts, bearing plate, and reinforced concrete bolster; and 2) the deck slab bracket restrainer (Fig. 3-2), consisting of 7 - 3/4 in (19 mm) diam. galvanized cables, 2 - 12 in (30.5 cm) high by 20 in. (50.8 cm) wide by 44 in. (111.8 cm) long welded steel brackets, and 24 - 1 3/8 in. (35 mm) diam. A325 bolts. These hinge retrofit devices were used on bridges with varied geometries (3-6), e.g., curved, skewed, and sloping bridges.

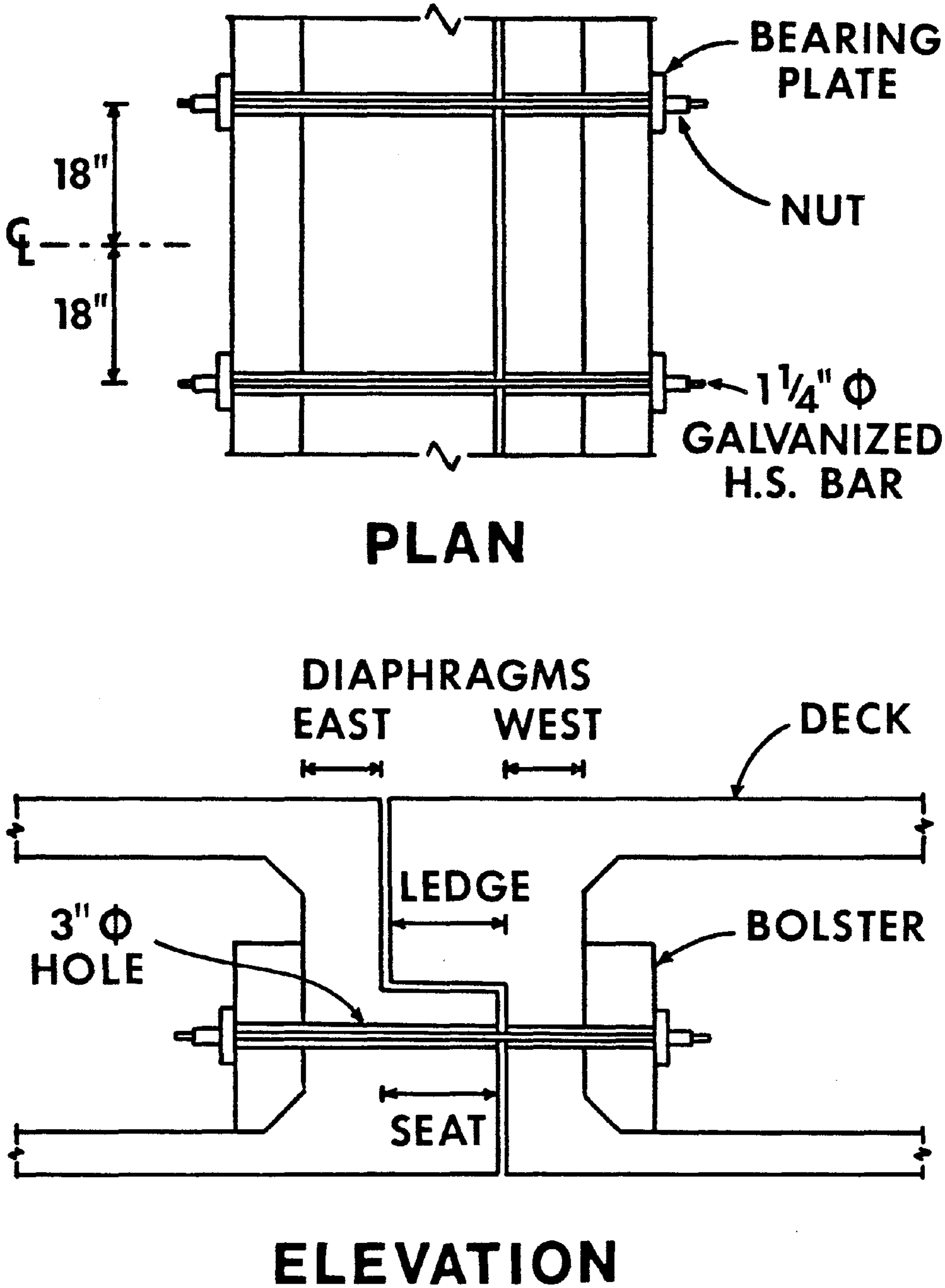

Fig. 3-1 - Hinge Strengthened with Bar Restrainer
(1 in = 2.54 cm)

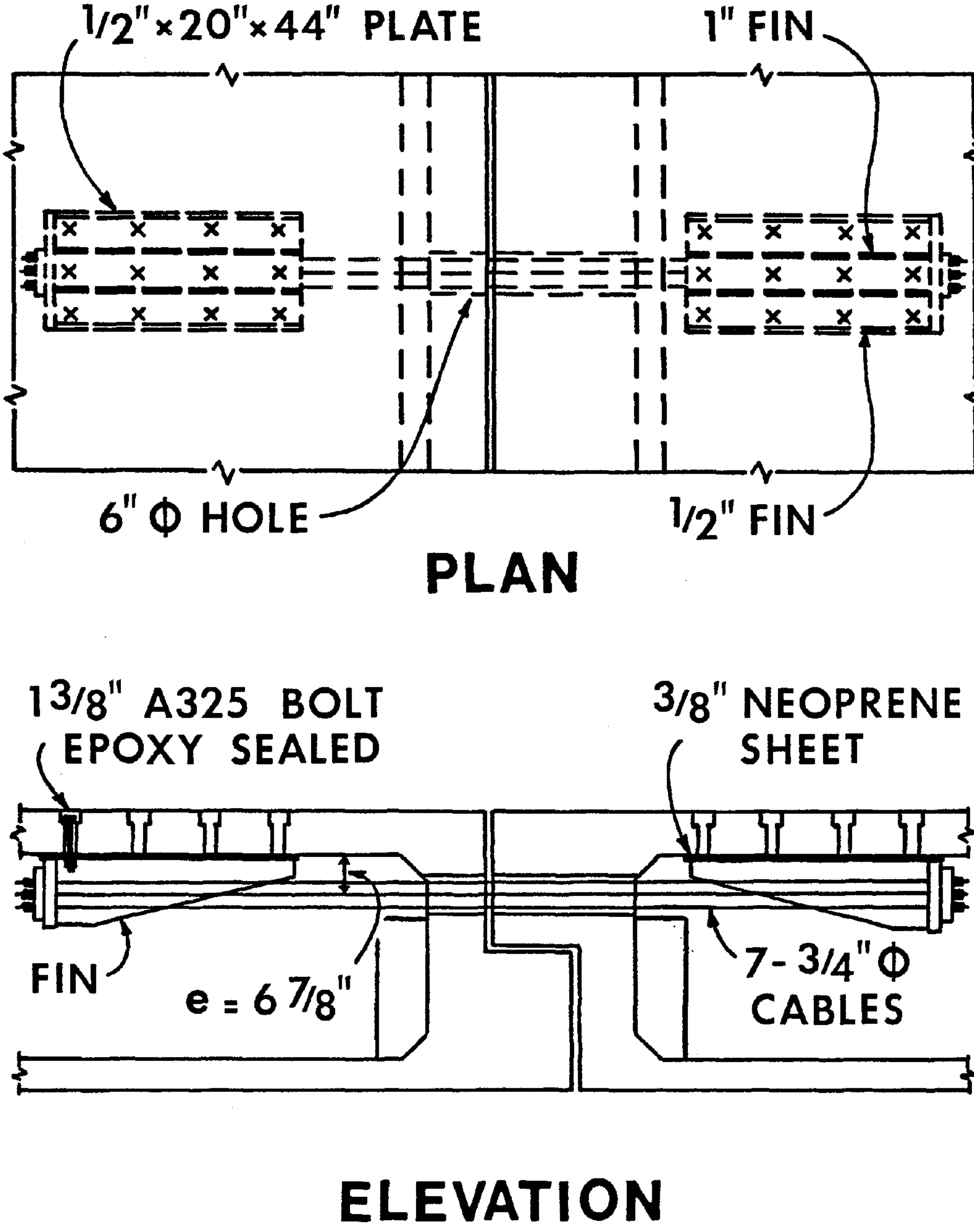

Fig. 3-2 - Deck Slab Bracket Restrainer
(1 in = 2.54 cm)

The anchorage of the restrainer hardware into existing reinforced concrete bridge components is a serious concern in retrofit design. The tension forces in the high strength bars may cause a pull out failure in the box girder hinge diaphragm (Fig. 3-1). Similarly the deck slab bracket restrainer (Fig. 3-2) may fail the deck slab where it is anchored. The forces exerted by the retrofit devices on the existing concrete components were not anticipated in the original bridge design. CALTRANS bridge engineers, (3-7) realizing that the existing elements play a major role in restrainer performance, expressed their desire for an experimental determination of the existing component strength.

Chapter 3 provides details of a full scale experimental investigation (Fig. 3-3), presents test results, and explores the use of analytical methods for predicting the strength and stiffness of the bar and bracket including the anchorage in the existing reinforced concrete bridges.

3.2 RESEARCH SIGNIFICANCE

This research demonstrates that restrainer behavior is influenced by all elements of the devices. In the case of the restrainer bar system, the bars and diaphragms collectively influence the strength and stiffness properties. Similarly the behavior of the bracket system is influenced by the cables, brackets, and anchorages. The results from the experimental testing can be used to develop experimentally verified design methods which are similar to existing procedures (3-6, -8, -9, -10, -11, -12). Also the results can be used in analytical (3-13) and code (3-14 , -15) procedures.

3.3 TEST SPECIMEN

A single specimen suffices for both tests. The full scale specimen (Fig. 3-3) is 4 ft. (1.22 m) high, 10 ft. (3.05 m) wide, and has an overall length of 19 ft (5.8 m). The thicknesses of the hinge diaphragm, deck slab, soffit slab, and other specimen elements presented in Table 3-1 are in general agreement with corresponding elements of actual bridges.

3.3.1 Bar Restrainer Tests - Bolsters (3-9) (Fig. 3-1) are added to the test specimen in

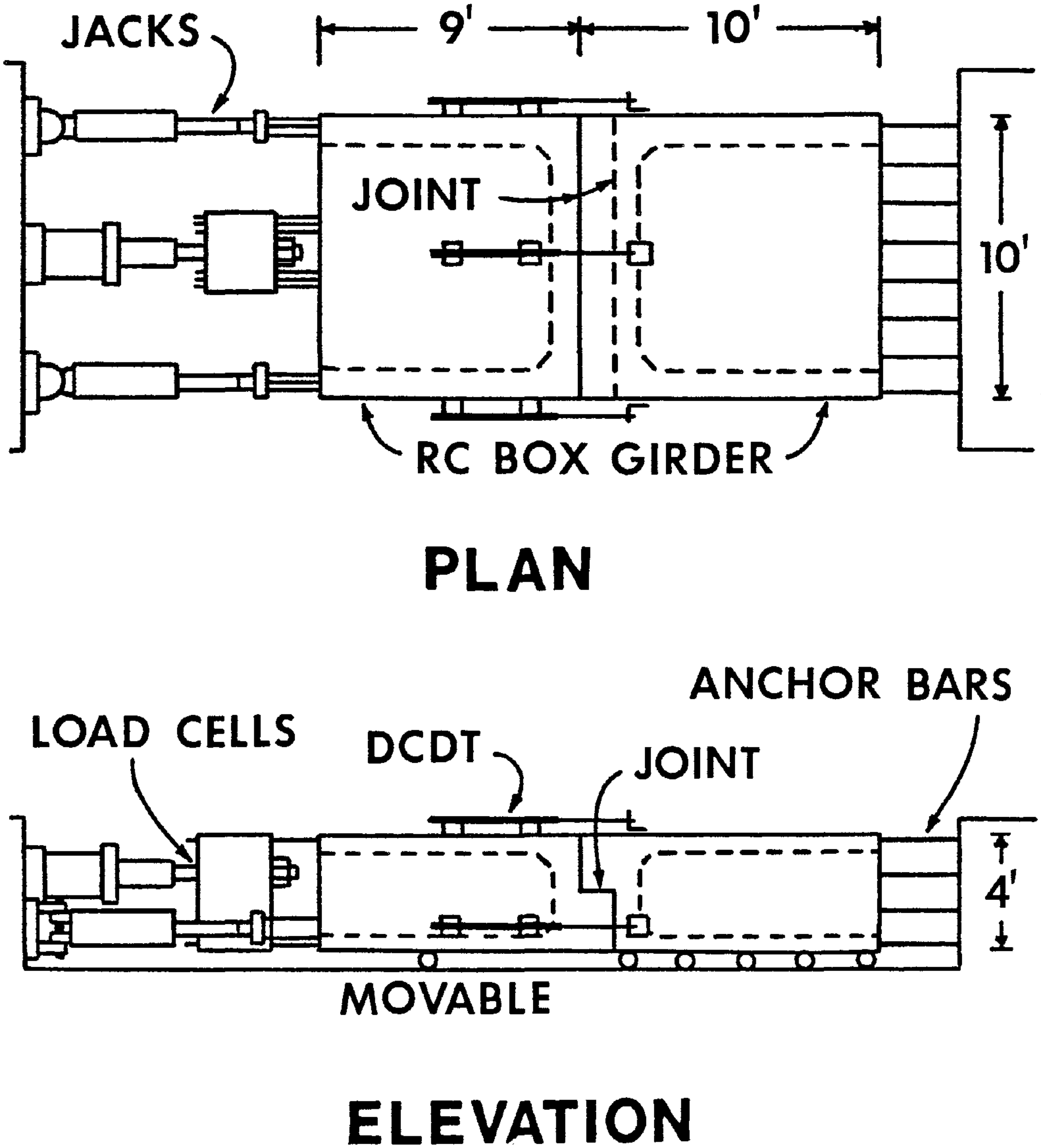

Fig. 3-3 - Test Specimen and Testing Arrangement
(1 ft = 3.48 cm)

Table 3-1 - Dimensions of Reinforced Concrete Box Girder Elements and Galvanized Bar and Bracket Restrainer Hardware

Box Girder Element	Width or Thickness	
	in.	cm
Deck Slab	7.5	19.1
Soffit Slab	5.5	14.0
Bolster	9.0	22.9
Diaphragm	10.0	25.4
Stem or Web	12.0[a.]	30.5[a.]
Seat	Variable[b.]	

Restrainer Hardware	t x w x h[c.]	
	in.	cm.
Bearing Plate	1 1/2x8x8	4x20x20
	l x w x h[d.]	
	in.	cm.
Bracket	44x20x12	112x51x30

Notes:

a. Normal stem thickness is 8 in. (20.3 cm). 12 in (30.5 cm) stem thickness is used to accommodate specimen anchorage bars.

b. Seat widths vary between 6 in. (15.2 cm) and 15 in. (38.1 cm) on existing bridges. Specimen has 15 in. (38.1 cm) seat width.

c. Thickness x width x height (t x w x h).

d. Length x width x height (l x w x h).

preparation for installation of restrainer rods just as would be done in an actual bridge. The reinforced concrete bolster which is placed with a cold joint against the inside face of the hinge diaphragm has a height dimension equal to 1/2 of the overall box girder height, i.e., 24 in. (61.0 cm) for the specimen. Two restrainer bars per cell used in the present study (Fig. 3-1) represents the majority of retrofit installations.

The inventory of bridges which have been retrofit have some variation in the details of reinforcement. A standard plan dated November 1966 (3-16) was deemed to give the most representative reinforcement details. A side elevation view of the specimen's seat diaphragm and bolster is shown in Fig. 3-4. The tension force in the restrainer bars induces bending in the hinge diaphragm and seat as follows: 1) the hinge diaphragm spans vertically between the deck slab and soffit slab; and 2) the hinge seat spans horizontally between the webs.

The reinforcement which resists the bending is as follows: 1) the outside leg of hinge diaphragm closed lapped vertical stirrups which are made from #4 (13 mm) bars spaced at 12 in. (30.5 cm); and 2) horizontal #4 (13 mm) and #6 (19 mm) straight bars located at the front face of the seat.

The bolster (Fig. 3-4) is placed against the existing hinge diaphragm to increase the shear and bending resistance. Holes are drilled on the back face of the diaphragm, and bolster dowel bars are anchored in them with epoxy. The bolster also has horizontal bending and vertical stirrup reinforcement. The bolster reinforcement used in the specimen duplicates details which were specified (3-9) on retrofit projects.

3.3.2 Bracket Restrainer Test - The deck slab in the region surrounding the installed bracket is shown in Fig. 3-5. The coring is performed without damage to the deck reinforcement. The slab is reinforced primarily for bending between the webs. Longitudinal reinforcement is used to resist superstructure moments and for slab crack control. Typically, there is no shear reinforcement. The bracket generates moment on the underside of the slab and diagonal tension in the plane of the slab. The

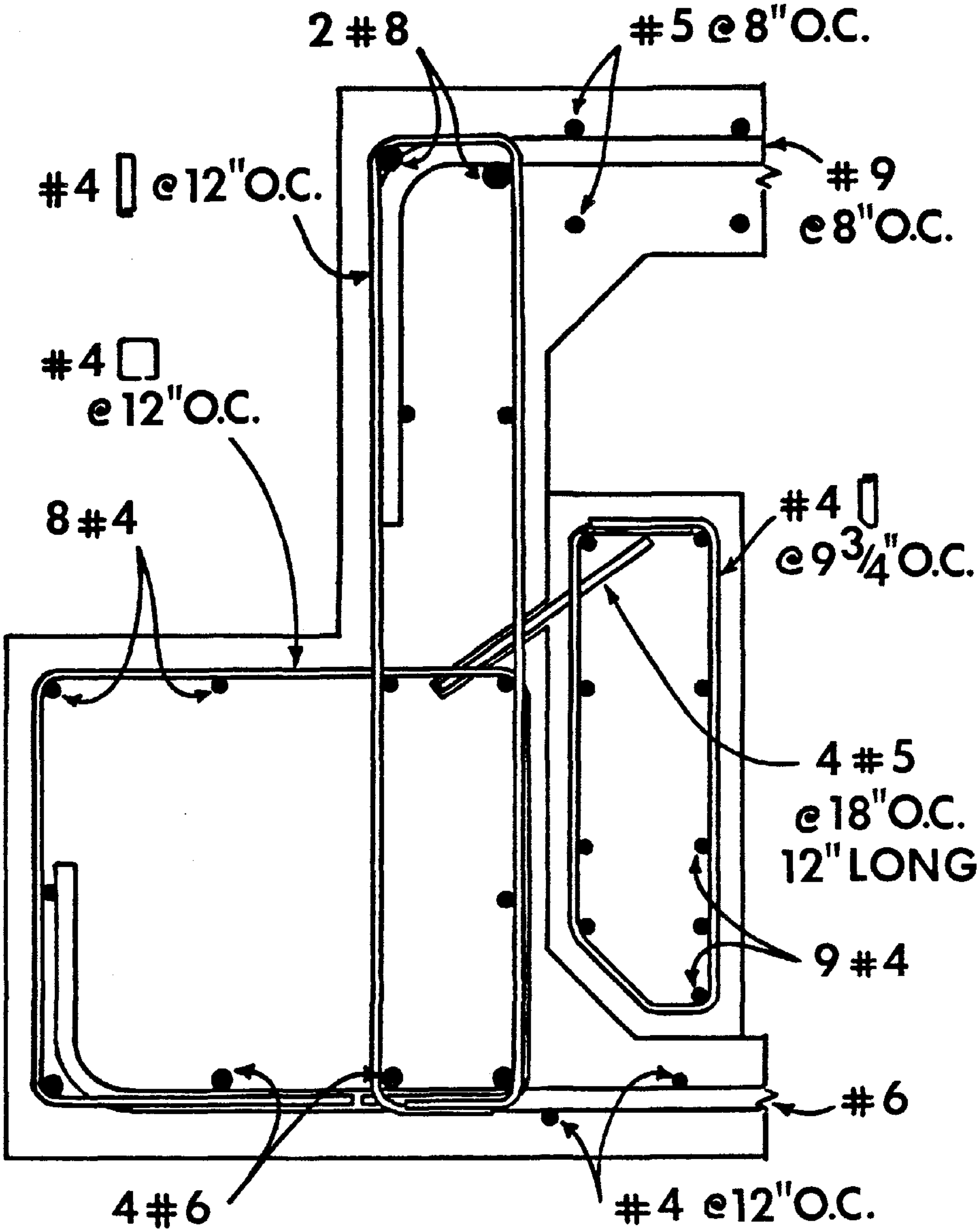

Fig. 3-4 - Seat Diaphragm with Bolster
(1 in = 2.54 cm)

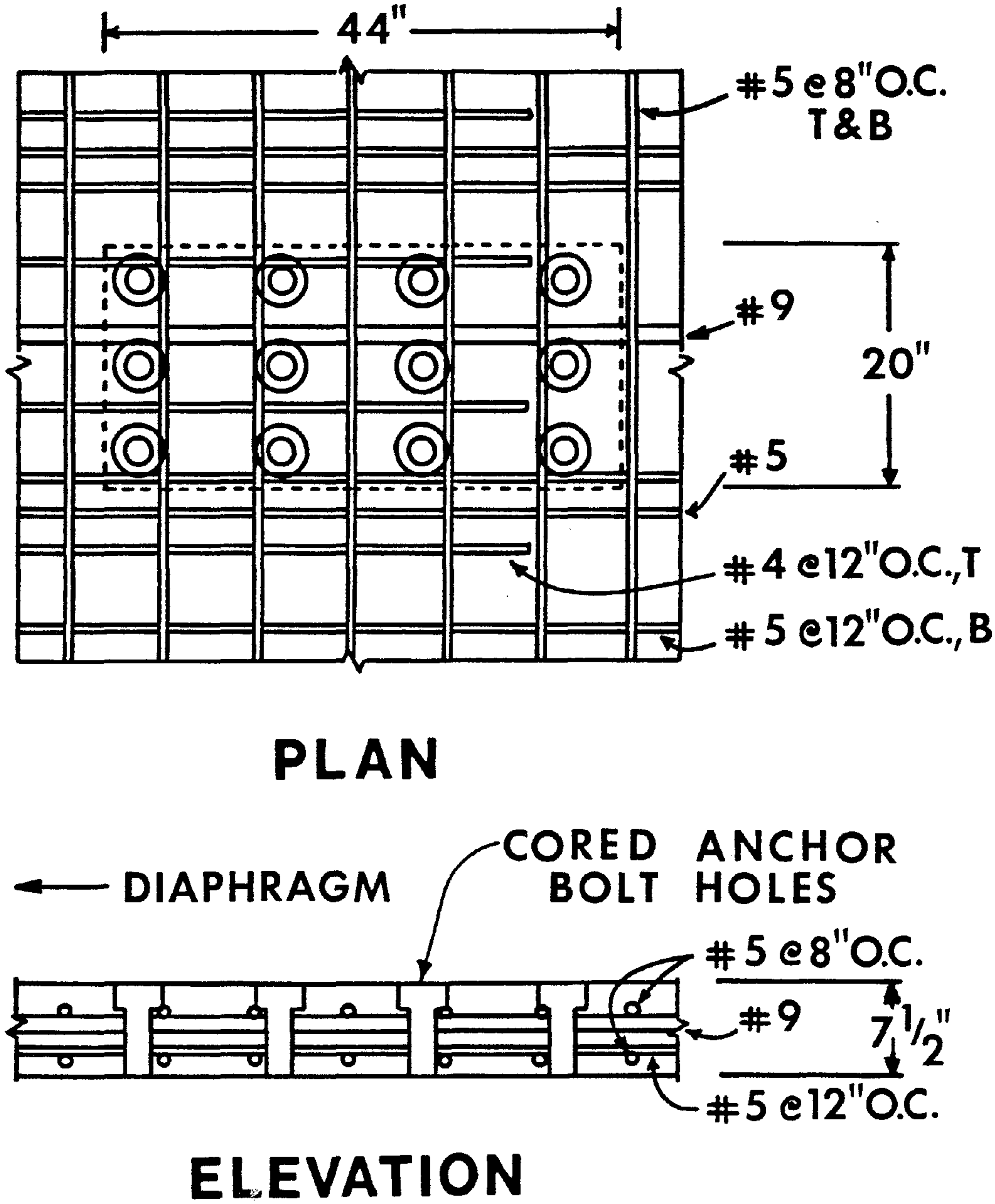

Fig. 3-5 - Ledge Diaphragm Deck Slab Reinforcement and Coring For Bracket Restrainer (1 in = 2.54 cm) moment is resisted in shear by the concrete in the deck slab.

Grade 60 reinforcement is used throughout the specimen. The material properties of the reinforcement are in compliance with the CALTRANS specifications (3-17). Structural steel used in fabrication of the bracket complies with ASTM A36 specifications (3-18).

Stone concrete used in the box girder specimen conforms with CALTRANS specifications (3-19). The cement content is 6 sacks/cu. yd. (334.6 kg/cu. m). The maximum aggregate size is 1 in. (25.4 mm) and a 4 in. (10.2 cm) slump is specified. The design strength, i.e., f_c', is 3250 psi (22.4 MPa). Six - 6 in. (15.2 cm) diameter x 12 in. (30.5 cm) high standard concrete cylinders are sampled from the bridge specimen. The tested 28 day mean strength is 5005 psi (34.5 MPa) and the standard deviation is 412 psi (2.8 MPa).

3.4 LOADING

During testing, tension forces are applied with jacks (Fig. 3-3) through anchorage reinforcement to the movable seat side of the specimen. When a restrainer is present there are tension forces throughout the system, i.e., in the restrainer and in the ledge side of the specimen which is fixed with anchor bars to a reaction block. The anchorage reinforcement used in the specimen is located so that the strength and stiffness of the hinge diaphragm and deck regions under evaluation are not affected.

The movable section of the specimen (Fig. 3-3) sits on a 2 3/4 in. (7.0 cm) diam. x 10 ft. (3.0 m) long, stainless steel roller. The movable section rolls freely so that the restrainer installed between the two sides of the specimen will carry all of the longitudinal forces exerted by the jacks. Each of the three jacks is equipped with a load cell.

3.5 DISPLACEMENT CONTROL AND INSTRUMENTATION

The relative displacement at the hinge (Fig. 3-3) between the anchored side and movable sides of the specimen are sensed by direct current displacement transducers (DCDT's). Three DCDT's are mounted (Fig. 3-3) at the joint line on top and on both sides of the box girder so that equal displacements can be maintained at the DCDT locations by the jacks. The three servovalve controllers are merged, i.e. "slaved," so that the servovalves keep the jacks moving until equal hinge joint displacements occur at the DCDT locations. The jacks are somewhat physically removed from the hinge joint, but in spite of that, the control and jacking system did maintain equal joint displacements at the DCDT locations. The maximum velocity of the movable side of the specimen is 0.5 in/min. (1.3 cm/min).

The horizontal out-of-plane displacements of the seat and ledge diaphragms are measured on the inside of the boxes with respect to the webs or slabs. Two aluminum reference frames, i.e., one on each side of the hinge joint, are inside the boxes and anchored to the webs. The linear variable displacement transducers (LVDT's) are mounted on the aluminum frames for measurement of the transverse displacement of the hinge diaphragms.

The horizontal and vertical displacement of the deck slab brackets is sensed using LVDT's. The horizontal component is measured with respect to the deck slab. The vertical component is measured from the soffit slab.

3.6 TESTING SEQUENCE

From a pretest it was learned that tensile strength of the bar restrainer (Fig. 3-1) is not sufficient to cause significant cracking or damage to the hinge diaphragm. Hairline cracking does occur, but it is limited to the inside corner of the seat. Therefore it is feasible to conduct repeated restrainer bar tests without stiffness degradation or deterioration of the hinge diaphragm. The potential damage from the bracket test is unknown. Therefore it is prudent to test the restrainer bars first.

3.7 HIGH STRENGTH BAR RESTRAINER

Ten tests are performed on pairs of bars installed in the bridge specimen. Galvanized bars which were prescribed for the bridge retrofit projects are used in five of the tests. In addition, a testing machine series is run on 4 coupons which are turned down from a threaded high strength bar which is not galvanized. The test results are summarized in Table 3-2. The bridge joint load-deflection results for the bridge specimen with galvanized bars are presented in Fig. 3-6.

A maintenance problem was confirmed during bar procurement for the test program. During fabrication threaded bars are cut to the required length and then galvanized. With galvanizing present the nuts can not be threaded onto the bars so the end zones, where the nuts travel, are subsequently sandblasted. The later process removes most or all of the galvanizing, and the bars rust in these regions.

The load-joint displacement history given in Fig. 3-6 is created by the jacks acting in displacement control. The imposed displacements and loads and observed behavior of the bridge specimen are as follows:

1. Increase the load and displacement from zero until the load reaches 333 kips (1481 kN), the mean yield strength; the corresponding yield displacement is 0.51 in. (13.0 mm). The 333 kips (1481 kN) corresponds to a yield stress of 133 ksi (918 MPa). Cracking is not detected on the seat or ledge side of the specimen.

2. Perform three unload-reload loops, i.e., cycle three times between 0.51 in. (13.0 mm) and 0.16 in. (4.2 mm) which corresponds to zero load. After the third loop the joint displacement is 0.51 in. (13.0 mm) and the load is 320 kips (1423 kN). The hysteresis curves stay within the loop shown.

3. Increase the joint displacement from 0.51 in. (13.0 mm) and continue up to the ultimate strength of the system. The displacement is stopped at 4.28 in. (10.9 cm) (Fig. 3-6). The load reaches the ultimate resisting load of 385 kips (1,712 kN) at that displacement. The corresponding ultimate strength is 154 ksi (1062 MPa). The sum of diaphragm displacements under maximum load measured on the bearing plates is 0.072 in. (1.8 mm). Hairline cracking of the diaphragm occurs. Further increase in joint displacement causes sudden necking of one of the bars, fracture, and loss of approximately 1/2 of the resisting force. Failure is assumed to occur when the first bar fails. Necking is not affected by the nuts.

The hinge diaphragm remains essentially elastic although a footprint with depth ranging between 1/64 in. (0.40 mm) and 1/32 in. (0.79 mm) is left on the bolsters by the bearing plates (Fig. 3-1). Similarly the special threadbar nut leaves a footprint less than 1/64 in. (0.40 mm) deep on the bearing plate, and some permanent deformation of outside threads on the bar and inside threads in the nut occurs.

Table 3-2 - High Strength Threaded Restrainer Bar, 1 1/4 in. (31 mm) Diam.,[a] Summary of Test Results

Test Program	Gage Length (in.)	Number of Tests	Statistic	Elastic Modulus (ksi x 1000)	Yield Strength (ksi)	Ultimate Strength (ksi)	Ultimate Strain
Box Girder, Not Galvanized	57	5[c]	Mean	-	138.8	155.2	0.0795
			Std. Dev.	-	2.3	2.8	0.0051
Box Girder, Galvanized	57	5[c]	Mean	-	133.2	154.2	0.0738
			Std. Dev.	-	1.2	1.1	0.0020
Coupon[b]	1.5	4	Mean	31.25	132.0	152.5	0.2000
			Std. Dev.	0.29	1.2	1.0	0.0540

Notes:

a. Area = 1.25 in^2 (8.06 sq. cm.)

b. Coupons are turned down from 1 1/4 in. (31 mm) diameter threaded bar. Coupon ends are threaded and have 1 in. (25 mm) diameter. Strain measurements are performed on a 3/4 in. (19 mm) diameter x 1 1/2 in. (38 mm) long midsection with an LVDT based extensometer. The cross sectional area of the tested section is 0.44 in. (3-2) (2.84 sq. cm.).

c. In each test, two bars are installed in the bridge specimen.

d. Units: 1 in. = 2.54 cm; 1 ksi = 6.895 MPa.

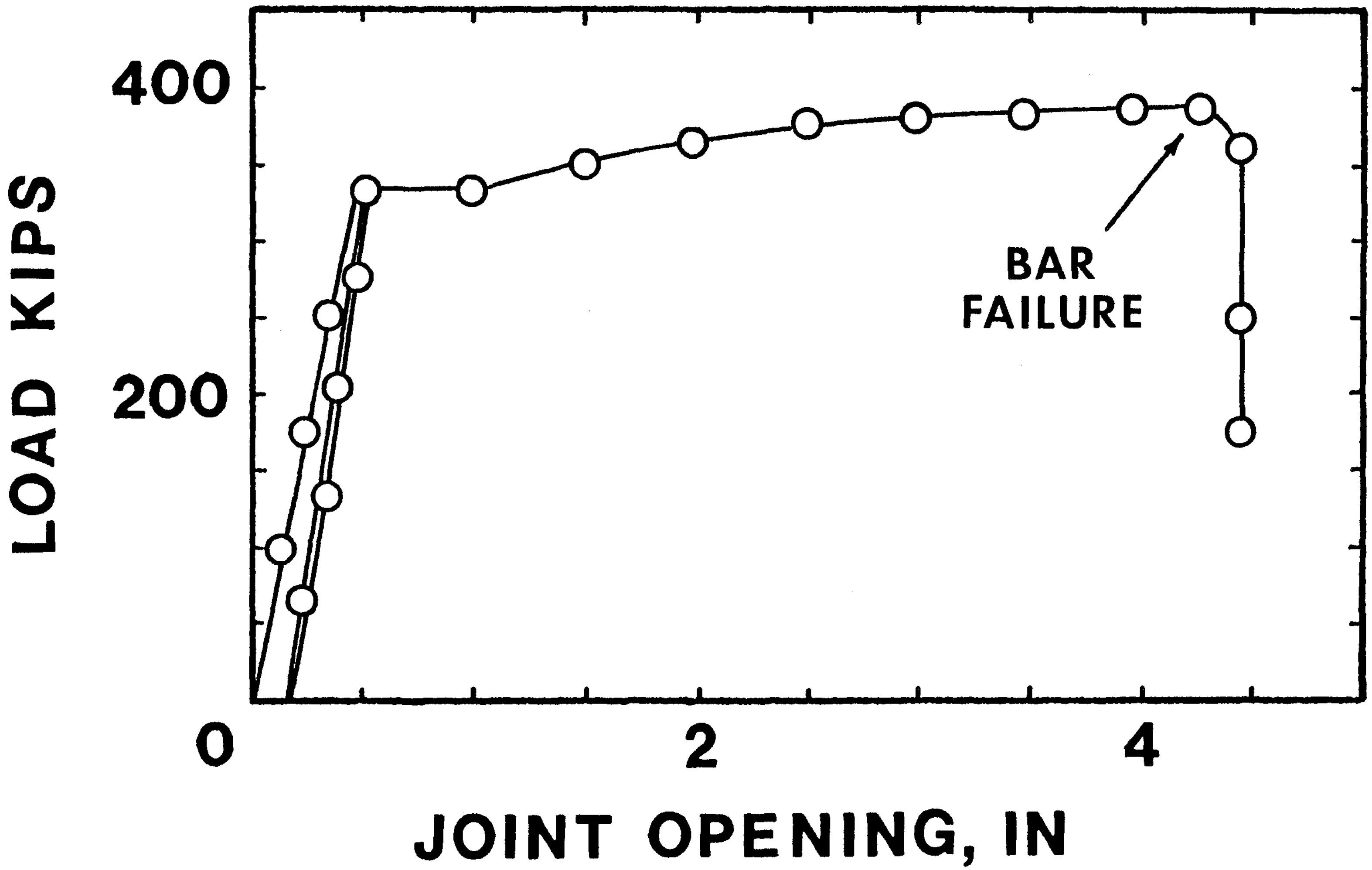

Fig. 3-6 - Global Load-Displacement Behavior at Joint for Bar Restrainer (1 kip = 4.448 KN; 1in = 2.54 cm)

The elastic stiffness of the joint is reduced by diaphragm flexibility (Fig. 3-6), i.e., when the stiffness is represented as an equivalent uniaxially loaded bar. On the virgin curve the initial elastic modulus of the equivalent bar is 15,000 ksi (103,000 MPa) while upon unloading and reloading the equivalent modulus increases to 21,700 ksi (150,000 MPa). The tested elastic modulus (Table 3-2) of the bar itself is 31,250 ksi (215,000 MPa).

The joint load-deflection relation (Fig. 3-6) resembles a stress-strain curve for high strength ductile steel. The tested yield plateau stress of 133 ksi (917 MPa) exceeds the design yield strength by 11%. The tested ultimate stress of 154 ksi (1062 MPa) exceeds the design yield strength by 28%. The overstrengths of the bars causes no problem because the hinge diaphragm has sufficient reserve strength to resist the bar forces. The tested ultimate elongation is 7.5% which exceeds the 7% in 10 bar diameters that is required (3-6).

In Table 3-2, the pertinent strength and elongation properties are compared for not galvanized, galvanized, and coupon cases. The galvanized and not galvanized results are similar. Although the average yield strength is 4% less and ultimate strain is 7% less for galvanized bars the ultimate strengths vary less than 1%.

The 3/4 in. (19.1 mm) diameter coupons are prepared from the high strength bar so that strength, elongation, and elastic modulus can be evaluated. The elastic modulus is 8% greater than that used in design for mild steel. The coupon yield and ultimate strengths agree with the full scale test results. The standard deviation for ultimate strain indicates a spread which is 27% of the mean value. This result indicates that the ultimate strain varies considerably along the bar, but it is clear from the full scale results that the requirement (3-6) for a minimum of 7% elongation in 10 bar diameters is satisfied during the present program.

3.8 DECK SLAB BRACKET RESTRAINER

One test is performed with the configuration shown (Figs. 3-2, -5). Each bracket is held up to the deck slab with 12 - 1 3/8 in. (35 mm) diameter A325 bolts. The bolt hex heads are recessed into the slab by counterboring. The shanks and heads are surrounded with epoxy. The neoprene sheet is needed to prevent leakage of freshly mixed epoxy. A 6 in. (15.2 cm) diameter hole cored through the hinge diaphragms provides for passage of the seven cables from the seat side to the ledge side of the specimen. The gage length of the cables including a 3 in. (7.6 cm) portion in each swag fitting is 137.5 in. (349 cm).

Fig. 3-7 gives the load-joint displacement results. The load-joint displacement history imposed by the jacks in displacement control and the observed specimen behavior are as follows:

1. Increase the load and displacement from zero until the load reaches 140 kips (623 kN); the corresponding joint displacement is 2.2 in. (5.6 cm). The load corresponds to 20 kips (89 kN) per cable and is within the working range. Cracking is not noticeable in the deck slab regions adjacent to the brackets.

2. Perform an unload-reload loop (Fig. 3-7) between 2.2 in. (5.6 cm) and 1.2 in. (3.0 cm). When the joint displacement is returned to 2.2 in. (5.6 cm) the load is 120 kips (534 kN).

3. Increase the load to a maximum of 300 kips (1334 kN); the corresponding joint displacement is 4 in. (10.2 cm). Beyond that point there is a sudden loss of resistance, a "cracking" sound, and downward punching shear failure at the tension bolt end of the ledge side bracket.

The maximum resisting load found in the test exceeds the design yield capacity of seven cables, i.e. 7 x 39 = 273 kips (1,214 kN). Design values used for predicting the ultimate strength of seven cables is 7 x 53 = 371 kips (1,650 kN). The equivalent ultimate cable strain, i.e., joint displacement divided by gage length, is 2.9%. Tested value (3-6) for the ultimate strain of the cable alone is 5%. Therefore, the test results show that ultimate strength and ultimate strain values are not reached because of the failure of the concrete.

The failure of the ledge side deck slab is due to bolt tension induced by the eccentricity of the cable forces acting on the bracket. The centerline of cables is 6 7/8 in. (17.5 cm) below the bottom of the deck slab (Fig. 3-2). The downward vertical and horizontal displacement of the bracket measured at the location of tension bolts, i.e., the bolts closest to the bearing end, is presented in Fig. 3-8. The horizontal and vertical displacements are comparable, and the existence of the vertical component

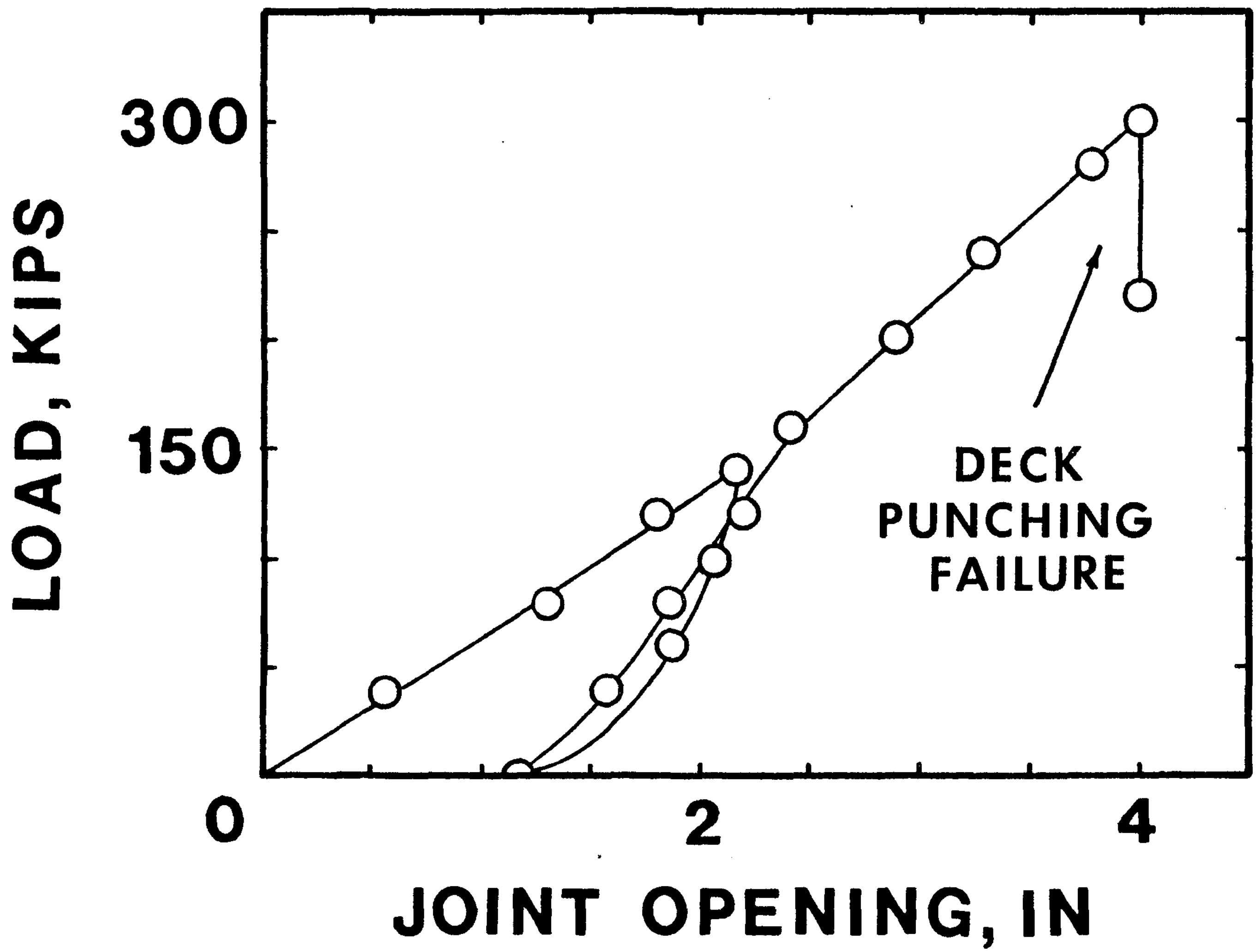

Fig. 3-7 - Global Load-Displacement Behavior at Joint for Deck Slab Bracket Restrainer (1 kip = 4.448 KN; 1 in = 2.54 cm)

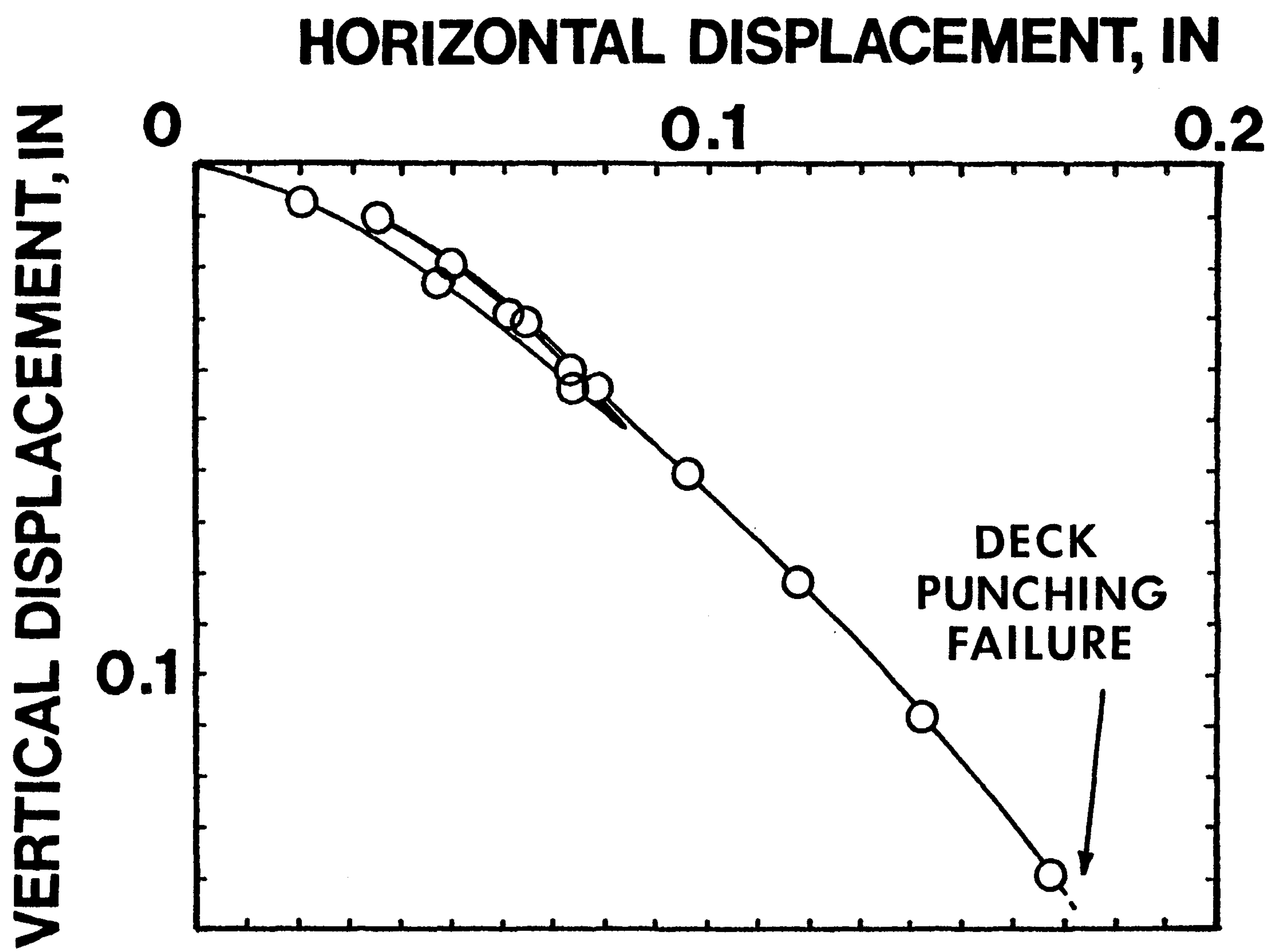

Fig. 3-8 - Vertical and Horizontal Displacements of Deck Slab Bracket Measured at Bolts Adjacent to Bearing End (1 in = 2.54 cm) confirms that bolt tension forces resulting from the eccentricity do have a noticeable effect.

The load-deflection response of the joint (Fig. 3-7) is governed by cable stretch and bracket movement until the punching shear failure surface (Fig. 3-9) is formed in the deck slab. Consider combining the cable and bracket flexibilities and then express them in terms of an equivalent cable modulus for the joint. Three equivalent cable moduli and their values obtained from Fig. 3-7 are pertinent: 1) initial tangent modulus, E_I = 5,390 ksi (37,100 MPa); 2) unload-reload modulus, E_P = 13,700 ksi (94,600 MPa); and 3) terminal tangent modulus, E_T = 8,340 ksi (57,500 MPa). Design values for cable moduli are: 1) initial modulus,(3-12) E_I = 10,000 ksi (69,000 MPa); and 2) prestretched modulus (3-6) (unloading - reloading), E_P = 18,000 ksi (124,000 MPa). A comparison of tested versus design values shows that the bracket flexibility significantly reduces the joint stiffness, particularly on the virgin curve. Bracket flexibility can be attributed to bearing deformation of the epoxy and concrete surrounding the bolts. The friction force between the bracket and slab is negligible because of the neoprene sheet between the bracket and slab.

3.8.1 Prediction of Bracket Strength - From summation of vertical forces the resisting bolt tension, T_r kips (kN), is related to the maximum diagonal tension carried across the punching shear failure surfaces (Fig. 3-9) with the relation

$$T_r = 2f_t\, d\left[(r_w + \frac{d}{2})\tan\frac{\alpha}{2} + s_b\right] + \frac{f_t\, d}{\cos\alpha}\left[\frac{b_s}{2} - (r_w + \frac{d}{2})\tan\frac{\alpha}{2} - s_b\right] \tag{3-1}$$

in which f_t = diagonal tension strength of the concrete, ksi (MPa); d = 5.5 in. (14.0 cm), depth of deck slab resisting the punching shear; r_w = 2 in. (5 cm), radius of the washer used on the counterbored bolts; α = 35°, interior angle between intersecting branches of the failure surfaces; s_b = 7 in. (17.8 cm), bolt spacing; b_s = 88 in. (223.5 cm), deck slab width between fillets.

The value of f_t will be treated as an unknown. It is assumed that the diagonal tension stress resultant varies linearly with respect to the horizontal distance along the sloping branches of the failure surface, i.e., maximum at the intersection with the transverse branch and zero at the outer ends. Also

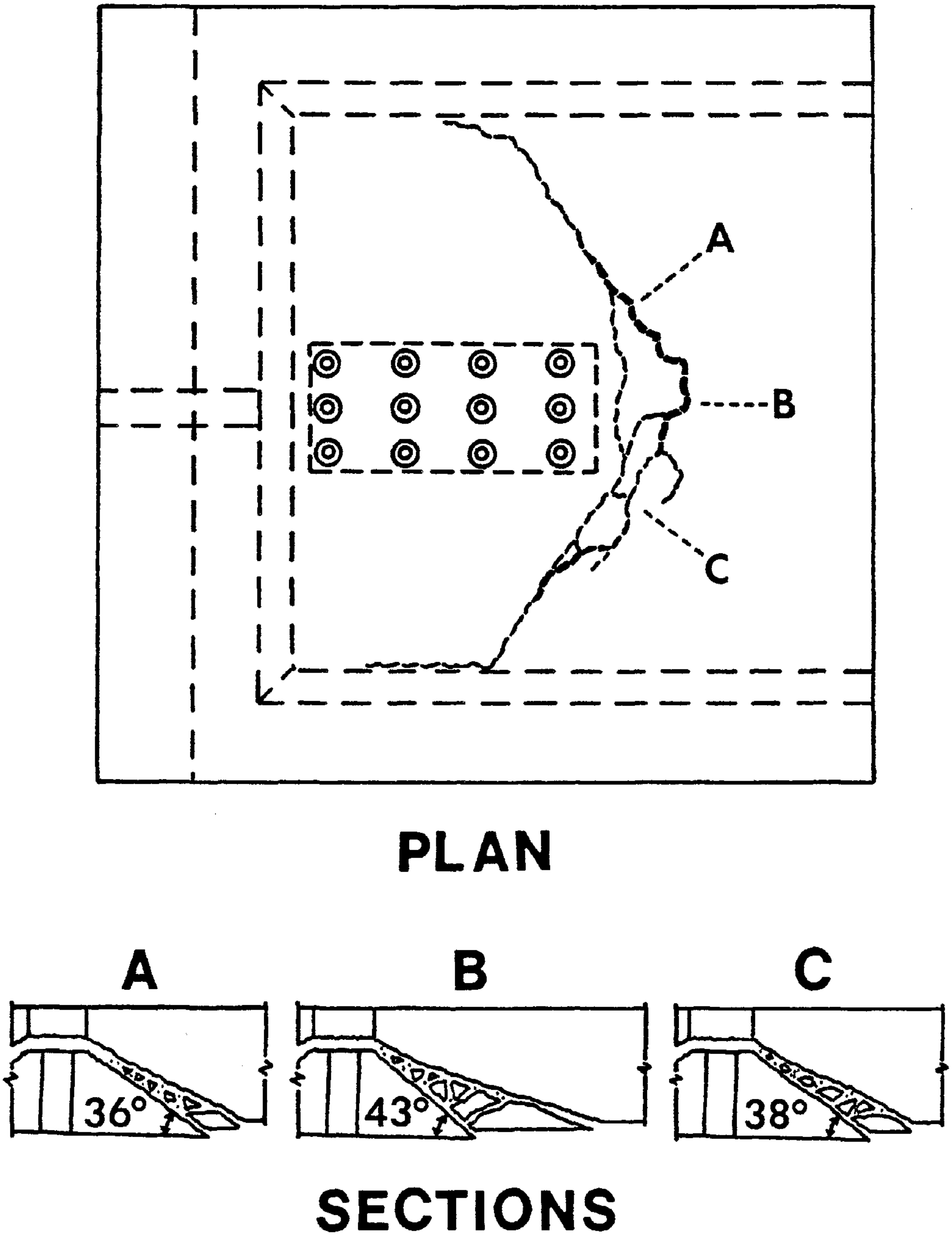

Fig. 3-9 - Deck Slab Punching Failure Surfaces, Bracket Restrainer the resultant is maximum along the transverse branch. The depth of slab resisting the punching shear is set equal to the distance from the bottom of the counterbored bolt heads to the bottom of the slab. This is a lower bound for slab thickness resisting the punching shear.

The resisting moment, M_r, can be expressed in terms of the diagonal tension resultants if moments are summed about the location of the resultant of the compression zone, i.e., area of neoprene sheet under compression. The relation is

$$M_r = 2f_t d \left[\left[\left(r_w + \frac{d}{2}\right)\tan\frac{\alpha}{2} + s_b\right]\left[r_w + \frac{d}{2} + x_r\right] + \frac{f_t d}{\cos\alpha}\left[\frac{b_s}{2} - \left(r_w + \frac{d}{2}\right)\tan\frac{\alpha}{2} - s_b\right]\right.$$

$$\left. x\left\{r_w + \frac{d}{2} + x_r - \frac{1}{3}\left[\frac{b_s}{2} - \left(r_w + \frac{d}{2}\right)\tan\frac{\alpha}{2} - s_b\right]\tan\alpha\right\}\right. \qquad (3\text{-}2)$$

in which x_r = distance from bolts next to the bearing plate to the location of the compression resultant, in (cm). The value of x_r is selected on the basis of experimental observation. Linear compression stress and strain blocks between the bracket and the slab are assumed with the neutral axis occurring at the second line of bolts from the bearing end. The resulting x_r = 30 in. (76.2 cm). Substitution of values into Eq. 3-2 gives M_r = 9,560 f_t kin (107,960 kN cm). The applied moment at failure, M_a, is the product of the measured load and distance from centerline of cables to mid-thickness of the slab. Its value is M_a = 3190 kin (36,000 kN cm). Now equate M_a to M_r and solve for f_t; the result is f_t = 0.334 ksi (2,300 MPa). The value for f_t agrees well with punching shear capacity values given in the ACI code (3-15), i.e., $4\sqrt{f_c'}$ = 283 psi (1.95 MPa). Substitution of the value of f_t = 0.334 ksi (2.3 MPa) into Eq. 3-1 gives the tension resultant, T_r, i.e., T_r = 110.8 kips (493.0 kN) are carried by the three tension bolts.

3.9 SUMMARY AND IMPLEMENTATION

3.9.1 Restrainer Bar System - The "weak link" in the bar restrainer retrofit system is the bar itself. This is highly desirable because the ductile properties of the bars can be exploited and brittle failures in the diaphragm avoided provided not more than two bars per bay are used. The tested yield and ultimate strengths, stiffness, and elongation all meet or surpass the design requirements (3-6) for box girder bridges with similar dimensions and detailing which have been retrofit with 2 bars per bay.

The hinge diaphragms experience hairline cracking only and remain elastic throughout the tests. The horizontal flexibility of the diaphragms reduces the initial elastic stiffness of the joint to 48% of the value for a rigidly anchored bar. During unloading there is stiffening associated with the arrest of local bearing failures around bearing plates and nuts. At that time the value increases to 69%.

The mechanical properties of the high strength threaded bar are not significantly diminished by hot-dip galvanizing required on retrofit projects, but the bar manufacturer emphasizes that the bars are not weldable. A rusting problem does occur because the galvanizing in the end zone, i.e., where the nuts travel, must be sandblasted sufficiently so the nuts are workable. Unless the manufacturing process for the nuts is changed the rusting should be controlled with a zinc-rich paint.

3.9.2 Deck Slab Bracket Restrainer - The "weak link" in the bracket restrainer is the punching shear mechanism in the slab due to bracket eccentricity. The tested ultimate resisting load is 300 kips (1330 kN) and equivalent ultimate cable strain including bracket movement is 2.9%. The corresponding design values are: 1) design yield load is 273 kips (1210 kN), 2) design ultimate strength is 371 kips (1650 kN), and 3) ultimate strain is 5%. The test results show that ultimate strength and ultimate strain are not reached because of failure in the concrete.

The neoprene sheet that is present reduces the friction force between the bracket and slab thereby increasing the bracket flexibility. Furthermore, the inclusion of the neoprene sheet increases the bracket eccentricity.

There are design options available for improving the bracket restrainer performance. A longer bracket would have higher strength and the strength of brackets with different dimensions can be predicted using Eqs. 1 and 2. Another option is to reduce the number of cables to five or less so that the likelihood of concrete failure is greatly diminished. The latter option is more practical because a heavier bracket would be difficult to install.

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CHAPTER 4

CONCLUSIONS

The San Fernando Earthquake of 1971 showed that bridge strengthening was needed. Logic dictated the addition of bars and cables at the expansion joints, but there was concern that these could induce failure of adjacent reinforced concrete elements. The present experimental test program evaluated the strength and stiffness of three types of restrainers, and confirmed that failure may occur in the restrainer hardware or it may occur in the adjacent concrete anchorage.

The toughness of the restrainers should be maximized so it is best to design the restrainers and their concrete anchorages so that failure is induced in the cables or bars instead of the adjacent concrete. The test program demonstrated a failure in the bars, but the type C1 and bracket restrainers failed in the adjacent concrete so it would be prudent to revise the anchorage details for these. When designing the anchorage, consideration should be given to: (1) the geometry of the box, end diaphragm, and seat, (2) the significant axial tension and moment induced in the superstructure by the restrainers, and (3) the reinforcement in the existing slabs, webs, and diaphragm.

The test program showed that the overall effect of the retrofit program will be extremely beneficial to the seismic resistance of bridges. Despite the system weakness identified by the tests, the measured strength of the tested restrainers surpassed the yield strength used in their design. The added seismic resistance provided by the restrainers will help prevent the collapse of bridges. Detail improvements developed as a result of these tests will add desirable margins of safety to future installations.

CHAPTER 5

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