SEISMIC STRENGTHENING OF INADEQUATE LENGTH LAP SPLICES

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SUMMARY

Results are summarised of full-scale field, and half scale laboratory, tests on bridge columns with longitudinal reinforcement having non-contact lap splices of inadequate length for high intensity seismic loading. The test columns had reinforcing details typical of those found in field investigations of 30-year-old columns. Some test specimens duplicated the as-built column condition while in others the lap splice was strengthened using jackets of six different types: steel bands, post-tensioned prestressing strands, hand applied and shell section fiberglass, hand applied carbon fibre, and intrusion grouted steel fibers. A procedure is described for the design of column wraps that is based on those results and is currently being used by the Illinois Department of Transportation for the strengthening of bridge columns in Illinois.

INTRODUCTION

Most of the bridges on the highway systems in the east and central USA in earthquake vulnerable areas were built prior to 1986 when modern seismic detailing requirements for bridges became mandated by the US Federal Highway Administration (Hawkins, 1999). In Illinois, ground accelerations large enough to damage existing bridges are likely south of the line of Interstate 70 that joins Indianapolis to St. Louis. Since 1986 the Illinois Department of Transportation (IDOT) has been actively mitigating earthquake hazards through emergency response planning, vulnerability assessments, retrofit of existing bridge structures, and highway reconstruction.

The earthquake vulnerability situation for bridges in Illinois is typical of that throughout much of the east and central USA. Most bridges consist of continuous steel, or made continuous for live load precast prestressed concrete, girders supported on reinforced concrete piers. The piers are supported on foundations with strengths matching those of the existing piers and the foundations are in turn often supported on piles driven into soft soils. To economically mitigate the seismic hazards associated with inadequate lap splices in the concrete piers it is essential that strengthening methods for that splice do not result in increased foundation or soil forces. Strengthening with steel jackets will typically both stiffen and increase the moment capacity of the piers and therefore increase soil and foundation forces. By strengthening with steel bands or fibre reinforced composites those increases in foundation and soil forces can in many cases be avoided.

The central part of the USA experiences large variations in temperature throughout the year, from periods of up to two weeks with temperatures in excess of 95 degrees Fahrenheit and 95% humidity in the summer to minus 20 degrees Fahrenheit in the winter. In the summer the humidity and high water tables in the soil often result in the interior of the concrete columns being moist. In the winter the extensive salting of the highways to provide ice free roadways results in a runoff during thaws that is corrosive and keeps concrete column interiors moist. Experience (Karshenas and Kaspar, 1999) has shown that if the concrete column is sealed so that moisture evaporation from its surface is prevented, then moisture and salts accumulate behind the seal, creating stresses in the jacket and resulting in freeze-thaw damage to the concrete behind the seal in winter. Seismic strengthening methods that seal moisture in the concrete can be detrimental to life of the pier and must be avoided. Retrofits with steel or composite bands allow continued breathing of the column. However, with most composites the

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column surface is coated with epoxy to bond the composite to the concrete. The composite must be applied in bands, with the concrete surface between bands epoxy free, if the freeze thaw damage is to be avoided.

FIELD TESTS

In 1992 IDOT reconstructed two mainline roadways, consisting of a series of interconnected bridges, in a large interchange in East St. Louis. However, ramps leading to those mainlines were in good condition and it was necessary only to seismically retrofit the columns supporting those ramps for their continued use. The columns supporting the mainlines were similar to those supporting ramps and demolition of the mainlines allowed testing to failure, at full scale, of columns with seismic deficiencies typical of those in the ramps. Nine columns having the properties shown in Table 1 were tested using the test setup shown in Fig.1. Test objectives were two. First, to confirm that external confinement reinforcement of the lap splice of the columns could strengthen the piers to where they met current seismic design standards without increasing column stiffness. Second, to provide a menu of acceptable splice retrofit methods which contractors could use for bidding purposes

Four columns were tested in the as-built condition and five with retrofits. Test columns were 48 or 54 inches in diameter and reinforced with No.11 bars with nominal yield stresses of 40 ksi.. Actual yield strengths were 1.12 to 1.22 times that value. Column bars were joined with 41-inch long splices to dowel bars extending out of a crash wall connecting the column bases. The columns lacked adequate ties and often the cover to those ties was insufficient, allowing them to corrode markedly. The lateral ties consisted of No.4 circular hoops spaced at 12 inches and closed by 15-inch laps. Design concrete strength for the columns and crash walls was 3,500 psi and measured strengths were 1.3 to 1.8 times that value. Shown in Fig.2 are typical measured bar locations for a 54-inch diameter column. The cover to the ties was highly variable and while the plans called for dowel and column bars to be in contact, few were. Some separations were as large as 6 inches.

Four different retrofit methods were tested. Three columns had active retrofits provided by prestressed steel bands or strands and two had passive retrofits provided by fibreglass jackets. The “Bands” of Table 1 were interconnected 3/4 inch square steel bars bent into semicircles and stressed to about 34 kips. The “DSI” clamp-rings were two semicircular 0.6 inch, 270 ksi prestressing strands and two anchorage brackets, stressed from each bracket to about 40 kips. For both retrofits there were eight clamp-rings applied at 8 inch spacing, over a length about equal to the column diameter. Two fibreglass alternatives were tested, both on 48 inch diameter columns. The “NCF” alternative used four slit cylinder prefabricated fibreglass plies, each 8 feet high. When fitted around the column, each cylinder lacked about 2in. from closing. Resulting joints were staggered 180 degrees apart between plies. During installation the outside of the column and the inside of a cylinder were coated with a vinyl-ester resin, using paint rollers, and then the cylinder sprung open and slipped around the column. After two plies were in place, webbing clamps were tightened around the cylinders while the resin set, and the operation was repeated. Each ply was about 1/4 inch thick for a total jacket thickness of 1 inch. The fibreglass had a nominal strength of 90ksi. The “RJW” jacket consisted of five layers of fibreglass cloth laid-up on the column using an epoxy resin binder. The epoxy was first applied with paint rollers and then a fibreglass layer pressed into it. The total thickness of the final jacket was about 6mm, and it extended to about 9 feet from the column base. The fibreglass had a nominal strength of 65ksi in the circumferential direction of the column.

As shown in Fig.1, to test the columns two hydraulic rams were placed on top of a load frame bolted to the crash wall and a loading head attached to the column. One hydraulic ram pushed on the loading head, while the second pulled. Load cells installed under each ram measured forces. The first (positive) loading cycle always pulled the top of the column toward the centre of the pier, producing a closing moment on the connection between crash wall and column. Multiple fully reversed loading cycles were applied to each column with later cycles to successively higher deflections in deflection-control mode. Lateral deflections at the load point and rotations at the column base were measured, cracks marked and their widths recorded. Maximum forces applied to each column for each direction, and the accompanying deflections, are listed in Table 1. In the retrofitted columns the dowel bars yielded and none of the splices failed in spite of the large deflections imposed on those columns. Forces were continuing to increase with increasing deflections when loading became limited by rotations of the loading head. All retrofitted columns showed stable, non-pinching, hysteretic loops. In contrast, for all the as-built columns there was only limited yielding of the dowel bars and splices failed with deflections at failure decreasing with decreasing circumferential separation of the column bars. Extensive vertical splitting cracks occurred along the line of the column bars as peak loads were approached and the cover concrete delaminated extensively. For the columns with active retrofits splitting cracks along the line of the column bars at the base of the column were arrested at the level of the lowest band or strand. For the columns with fibreglass retrofit, because jackets completely covered the concrete surface, no information about concrete cracking, crushing, or spalling was obtained. The NCF jacket developed one horizontal crack near mid height and a few small cracks in the glue lines at the edges of the slit in the outer ply damage. The RJW jacket also developed one horizontal crack near midheight, in the region where the layers of fibreglass were lapped, and the epoxy eventually cracked in a few limited areas near the base of the column. With that cracking the transparent epoxy turned milky.

Rotations measured at column bases showed that nearly all the deflection in the retrofitted columns came from the opening and closing of one very large crack formed at the cold joint between the top of the crash wall and the bottom of the column. This crack often became 0.8 to 1.2 in. wide at the maximum deflection in one direction and then, with reversal of the load, completely closed before opening to a similar amount on the opposite face for loading in the reversed direction. Some of this opening must have resulted from slip of the dowel bars relative to the pile cap, and some from slip of the dowel bars relative to the column bars. All crash walls supporting retrofitted columns suffered significant damage by the time the tests ended. Two sets of diagonal cracks formed with one inclination for closing moment loading and another for opening moment loading. These cracks caused a diamond-shaped overall pattern. Some of the cracks were several mm wide by the end of the tests and pieces of concrete isolated by the cracks fell out. The top surface of the crash wall delaminated on the inboard side of the column and cracking on the outer end of the crash wall enabled moderate sized chunks of concrete to fall off. The combination of the side wall diagonal cracking and top surface delamination often exposed the ends of the outer top longitudinal bars in the crash wall. However, in all cases the required moment was developed. All retrofits were successful. Differences in performance were small. However, the factor of safety of associated with each retrofit was unclear. The contractor used the prestressing strand retrofit on the ramps.

LABORATORY TESTS

Three laboratory test series were conducted, (Lin et al., 1998, Shkurti, 1998, Brunnhoeffer et al., 1999). In all three series reversed cyclic lateral loading tests to failure were made on 2 foot diameter columns, each reinforced with six No.11 bars and each containing non-contact lap splices of the same length as the splices used in the field columns. With the exception of the size of the column and dowel bars, the geometry and configuration of the laboratory test specimens were approximately half scale versions of the field specimens. The overall dimensions of the specimens, including crash wall and column, were the same for all three test series. The number, diameter and spacing of the column hoop reinforcement was the same for all specimens and consisted of No.3 bars at 10 inches. The test setup and a typical specimen column cross section are shown in Figs. 3 and 4. The six tests by Lin et al. examined the effects of the geometry of the non-contact splice on as-built column performance. The eight tests by Shkurti examined the effect on column performance of the characteristics of the external confinement used to prevent lap splice failure. The three tests by Brunnhoeffer et al. were conducted on columns tested to failure in the study by Lin et al. The feasibility was examined of using jackets made from intrusion grouted steel fibre mats to repair and enhance the seismic performance of damaged as-built columns.

Results for Lin's tests are summarised in Table 2. Principal variables were the clear cover to the column ties, the clear separation of dowel and column bars, and the radial angle between bars. None of the column bars, as opposed to the dowel bars, yielded before failure. Hoop bars yielded only after displacements considerably greater than those for maximum load. Failure was initiated by opening of the vertical splitting cracks that occurred along the column longitudinal reinforcement. In some specimens, bond failure caused a marked and immediate drop in the lateral load carrying capacity of the column. In other specimens the column could continue to carry significant lateral loads with increasing deflections after the maximum capacity was reached. In Table 2, the maximum capacity and its associated deflection are listed. The ductilities achieved are those at failure, defined as the deflection at which the load had dropped to 80% of the maximum capacity. Ductilities were limited both by bond failure and by crushing of concrete cover. The deflection for yield was approximately 1.5 inches for all six specimens. The flexural and rotational characteristics of the columns were dictated by the location and confinement of the dowel bars. The force in the column bar most highly stressed in tension dictated the load for splice failure. The force for failure could be predicted based on the bond strength of that bar.

Specimen properties and results for Shkurti's tests are shown in Table 3. Principal variables were the retrofit type (none, strands or advanced composite), and the strength of the confinement as a percentage of that required by the 1995 FHWA Recommendations. For all specimens a clear cover to the hoop steel of 0.5 inch and a clear separation between column and dowel bars of 1.0 inch were used. For prestressing strands, three different spacings were used. For specimens with composite bands, spaces were left between bands equal to the width of the band with the exception of E-2 where the spacing between bands was made 1.5 times the band's width for the uppermost two spaces. All jacketed specimens performed satisfactorily with peak loads increasing with increasing displacements, and with little deterioration with cycling, until failure. The exception was D-1 with the largest strand spacing. In that case a splice failure finally occurred at a ductility ratio of 10. The performance of the specimens is evaluated further in the discussion of Illinois Design Procedures.

The tests by Brunnhoeffer et al. used specimens A-2, B-2 and A-1 tested previously to failure by Lin. The damaged column was prepared by chipping back cracked concrete to the depth of the column bars and then clamping to its surface a stainless steel fibre mat about 1.5 inches thick with fibres of 0.5 mm diameter and variable lengths. That mat was then infiltrated with specially formulated slurry and allowed to cure for one month before testing. Construction techniques and specimen performance improved as experience was gained with the procedures that should be used. All of the columns repaired by jacketing developed comparable maximum lateral load capacities, in combination with considerably greater deflections, than the capacities and corresponding deflections of the original as-built columns. The effectiveness of the mat as a splice strengthening mechanism decreased as the degree of damage sustained by the column prior to jacketing increased.

ILLINOIS DESIGN PROCEDURES

Splice failures are inhibited if the jacket applies adequate confining pressure before large dilation strains develop. In the FHWA 1995 procedure the critical dilation strain, γ_d, is taken as 0.001. The force developed in the jacket for that strain must create a normal force such that the shearing resistance on a plane between column and dowel bars exceeds the anticipated axial force in those bars. Shearing resistance is evaluated using the shear-friction concepts of ACI 318-95, Sec. 11.7. Equivalency can be drawn between confinement requirements for steel jackets, tensioned prestressing strands, and fibre composites. However, experience from this field and laboratory testing showed that the relative performance of jackets provided by prestressing strands and by fibre reinforced composites, was not in conformity with the FHWA recommendations. While those recommendations were realistic for tensioned strands, they were unduly conservative for fibre reinforced composites. With increasing lateral load on the column, horizontal flexural cracks developed at the location of each prestressing strand. As dilation occurred the strands penetrated into the concrete, opening the flexural cracks and resulting in the maximum stress developed in the strands being no more than the effective prestress plus the stress associated with a dilation strain of 0.001. Based on this test program IDOT uses the FHWA procedure for prestressed strands, limiting the strand spacing to $6d_b$ where d_b is the diameter of the longitudinal column bar. In deriving the value of the effective prestress in the tendons IDOT requires that the effects of friction and anchor set losses occurring during stressing be carefully evaluated. In the tests, even with jacking strands from both ends to 0.7 times the strand breaking stress, it was difficult to develop more than 60,000psi effective stress in the tendons.

For composite jackets, use of dilation strains considerably greater than 0.001 is realistic. The dilation strains measured in the laboratory tests were 0.0035 and greater. Further ACI 318-95 recognises in its Commentary that the coefficient of friction defined in Sec. 11.7.4.3, and the shear stress limit of Sec. 11.7.5, result in very conservative estimates of shear friction strength. More appropriate limits on the shear force that can be developed for a given normal force are contained in the work of Vecchio and Collins (1986). To use their work a limiting crack width must be established. For the IDOT Recommendations that width was set as 0.03 inch or the value corresponding to a limiting design strain, γ_d, in the jacket equal to one third of the guaranteed ultimate strain for the jacket material but not greater than 0.003. Their procedure takes account of jacket material properties, column concrete strength and maximum aggregate size and is as follows:

1. For the potential vertical shearing plane between the column and dowel bars and the limiting design strain of the jacket calculate a resultant crack width

$$w = D'\varepsilon_d/2 \qquad (1)$$

where D' = pitch diameter of centreline of dowel bars and w is taken as not greater than 0.03 inch

2. For that value of w calculate the maximum shear strength, v_{cim}, that can be developed for unlimited normal force applied perpendicular to the shear plane. That strength in psi is given by:

$$v_{cim}=12of'c/\ (0.31+24w/\ [a+0.63]\ ) \qquad (2)$$

where f'c = concrete compressive strength and a = maximum aggregate size

3. Calculate the shear stress, v_{ci} that must act on the shear plane in order that the tensile strength of the dowel bars can be developed. That strength is given by:

$$v_{ci} = A_{bl}f'_s / \{[\prod D'/n] - d_{b1} - d_{b2} + c\}1_s\ \} \qquad (3)$$

where A_{bl} , d_{bl} = area, diameter of dowel bar, c = clear cover to surface of dowel bar, d_{b2} = diameter of column bar being spliced, f's = tensile strength of dowel bar, l_s = length of lap splice, and n = number of dowel bars

4. Calculate normal stress, f_{ci} , that must act on the shear plane to develop v_{ci} .

$$f_{ci} / v_{cim} = 1\text{-}o(1.22 - 1.22v_{ci} / v_{cim}) \qquad (4)$$

where f_{ci} should not be taken as less than 35 psi.

5. Compute jacket thickness, t_j in inches from:

$$t_j = D f_{ci}/2\varepsilon_d E_j \geq D35/2\varepsilon_d E_j \qquad (5)$$

where D = column diameter, Ej = modulus of elasticity of composite jacket, and γ_d = limiting strain for jacket of 0.003 or value calculated from Eq. 1 for w equal to 0.03 inch, whichever is less.

Dissection of the columns after testing showed that the potential shearing plane between column and dowel bar lay along the circumference of a circle passing through the centre of the dowel bars. Hence, D' in Eq. 1 is defined as the diameter of that circle. Equation 2 is the relationship developed by Vecchio and Collins. That relationship is sensitive to maximum aggregate size. In the field tests that size was 1.5 inches. In the laboratory tests it was 0.75 inches. For Eq.3 it is assumed that the dowel bars are distributed uniformly around the perimeter of the column. If not, v_{ci} values are calculated separately for each quadrant of the column with quadrants chosen so that the bars most highly stressed in tension, for a given loading direction, are in the same quadrant. When quadrants are used the number "n" in Eq.3 should be taken as four times the number of dowel bars in that quadrant. The largest value of v_{ci} should be used for subsequent calculations. The denominator of Eq. 3 is the length of cracked concrete associated with a pair of spliced bars and the numerator is the maximum force that can develop in the dowel bar. Consider Fig.3 showing the cracking pattern observed in the tests. The hatched area is the effective area of concrete associated with each column bar. Figs. 1 and 2 show the observed radial splitting crack. In the plane of the column cross section the length of the concrete portion of that crack is "c-d_{b2}". Similarly the concrete portion of the circumferential distance between dowel bars is (AD'/n-d_{b1}). The denominator is the sum of those two terms. In general d_{b1} and d_{b2} are equal. Based on the test results it was recommended that f'_s values of 75 and 110ksi be used for Grade 40 and Grade 60 bars, respectively.

For a given fibreglass material the IDOT procedure results in a jacket thickness consistent with the thickness that performed satisfactorily in both field and laboratory tests. That thickness is only 25% of the thickness required by the FHWA Recommendations. With that new procedure composite jackets, rather than strand jackets, became the jacket of choice by the contractor for a recent I-57 bridge retrofit job in Southern Illinois.

CONCLUSIONS

For circular bridge columns with lap splices at their base of inadequate length this study has shown that: (1) If the dowel bars lie inside the column bars, the cover to the column bar is the principal variable controlling the capacity of the lap splice. The lateral load capacity and rotational characteristics of the column are determined primarily by the characteristics and geometrical layout of the dowel bars. (2) Jackets formed from prestress strands, steel bands, and advanced composites can be used to strengthen lap splices without significantly increasing column stiffness or column lateral load capacity. (3) For advanced composite jackets the Illinois Procedure provides rational and cost-effective designs.

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REFERENCES

ACI Committee 318 1995. “Building Code Requirements for Structural Concrete (ACI-318-95) and Commentary,” American Concrete Institute, Farmington Hills, MI

Brunnhoeffer, G.C., M.B. Farrell, N.M. Hawkins, W.L. Gamble and F.P. Shkurti 1999. "Repair of Concrete Columns Using Steel Fiber Mats," Paper accepted for publication in SP Volume, American Concrete Institute, Farmington Hills, MI.

FHWA 1995. "Seismic Retrofitting Manual for Highway Bridges," Publication No. FHWA-RD-94-052, U.S. Department of Transportation, Federal Highway Administration , McLean, VA.

Hawkins, N.M. 1999. "MAE Center-Mid-America Departments of Transportation Workshop on Earthquake Vulnerability Considerations for Highway Systems in Mid-America," University of Illinois at Urbana-Champaign, IL.

Illinois Department of Transportation 1998. " Illinois DOT Recommended Procedure for Design of Alternate Column Wraps," Springfield, IL.

Karshenas, M., and I.I. Kaspar 1999. "Experience with Concrete Pier Column Encasement for Seismic Retrofitting," Proceedings, Mid-America Highway Seismic Conference, St. Louis, MO.

Lin, Y., W.L. Gamble and N.M. Hawkins 1998. "Seismic Behavior of Bridge Column Non-Contact Lap Splices," SRS No.622, Civil Engineering Studies, University of Illinois at Urbana-Champaign, IL.

Shkurti, F.P. 1998. "A Design Approach for Seismic Retrofit of Circular Bridge Columns with Lap Splices," Ph.D. Thesis, Civil Engineering, University of Illinois at Urbana-Champaign, IL.

Vecchio, F.J. and M.P. Collins 1986. "The Modified Compression Field Theory for Reinforced Concrete Elements Subjected to Shear," ACI Journal, V.83, No.2, Mar.-April, pp. 219-231.

TABLE 1 - RESULTS OF FIELD TESTS

Column	Diameter	No. of	Max. Load	Associated Deflection , and Max. Ductility				
	in.	#11 bars	Load kips	Defln. in.	Duct.	Load kips	Defln in.	Duct
B14S As-Built	48	18	100.5	7.76	4.9	-72.4	-3.50	-2.2
B18N Bands	54	28	157.8	4.80	3.0	-121.4	-2.85	-1.8
B18S As-Built	54	28	135.2	2.09	1.3	-110.2	-1.97	-2.0
C14N NCF	48	18	117.4	13.0	8.2	-108.0	-10.2	-6.5
C14S RJW	48	18	125.1	13.3	8.4	-95.0	-8.50	-5.4
C15N Bands	48	18	115.1	14.9	9.5	-110.6	-10.2	-6.5
C15S As-Built	48	18	109.2	4.74	3.0	-95.4	-4.74	-3.0
C17N DSI	54	24	180.2	14.1	9.0	-143.1	-7.91	-5.0
C17S As-Built	54	24	130.2	2.22	1.4	-129.5	-2.75	-1.7

Signs: Negative (-) load and deflection values indicate movement away from center of pier

TABLE 2-RESULTS OF AS-BUILT LABORATORY TESTS

Max. Load, Assoc. Deflection and Max. Ductility

Column	Cover in.	Separation in.	Angle deg.	Load kips	Defln. in	Duct	Load kips	Defln in	Duct
A-1	0.5	0	22	39.2	1.44	1.2	-39.2	-1.95	-1.7
A-2	0.5	2.0	28	35.3	2.05	1.8	-32.7	-1.80	-1.8
A-3	2.0	2.0	36	35.9	3.58	2.8	-34.6	-3.07	-2.4
B-1	0.5	0	36	39.3	2.80	2.5	-33.7	-0.97	-0.9
B-2	0.5	2.0	46	35.9	2.83	2.5	-35.4	-2.36	-2.1
B-3	2.0	2.0	60	33.8	3.78	2.8	-34.2	-3.78	-2.8

TABLE 3 - RESULTS OF RETROFITTED LABORATORY SPECIMENS

			Max. Load, Assoc. Deflection and Max Ductility						
Column	Retrofit Type	% FHWA	Load kips	Defln in.	Duct	Load kips	Defln in.	Duct	Typeof Failure
D-1	Strands at 8-inches	50	47.0	9.17	5.6	-47.0	-18.5	-10	Splice
D-2	Strands at 4 inches	100	50.0	9.35	5.3	-51.5	-16.9	-10	Rebar
F-1	None	0	39.4	4.99	2.5	-39.9	-4.90	-2	Splice
F-2	Strands at 6 inches	75	47.4	9.21	5.4	-46.5	-18.5	-10	Rebar
G-1	Carbon Fibre 5 inch Bands	50	50.6	9.12	5.4	-55.9	-17.6	-12	Rebar
G-2	Continous Carbon Fibre Jacket	100	49.5	9.15	5.0	-51.9	-18.3	-10	Rebar
E-1	Glass Fibre 6 inch Bands	23	49.1	9.65	5.3	-54.3	-18.9	-12	Rebar
E-2	Carbon Fibre 4 inch Bands	<50	50.7	9.49	5.7	-50.2	-18.8	-12	Rebar

Type of Failure Rebar stands for low-cycle-fatigue type of failure