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INTRODUCTION

Failure of a concrete bridge column can be attributed to: (a) fatigue of the longitudinal reinforcing steel; (b) failure of the concrete due to either a lack of confinement or the fracture of the transverse hoop reinforcement; and/or (c) compression buckling of the longitudinal reinforcement. If sufficient transverse reinforcement is used in the potential plastic hinge zone, modes (b) and (c) can be suppressed, thus leaving mode (a) – low cycle fatigue – as the remaining failure mode. This paper explores the theoretical fatigue capacity of reinforced concrete bridge piers and validates the theory with experimental research.

Low cycle fatigue failure in the longitudinal reinforcement lends itself to the concept of using replaceable plastic hinges. Results of several experiments are presented herein that explore the use of replaceable plastic hinges. In the plastic hinge zone, specially-detailed reinforcing fuse-bars are installed. These bars have been machined down to a smaller diameter (63 percent of the original diameter) to ensure that yielding and final fracture takes place at a predefined location. Outside the plastic hinge zone all materials in the column remain elastic at all times. After a seismic event, damaged concrete and steel within the hinge zone is removed, couplers are used to connect new fuse-bars to the longitudinal reinforcement, new spiral reinforcement is provided around the column, and new quick-setting shrinkage compensated concrete is cast. One experimental column was tested in this fashion and repaired five times without undue distresses to either the foundation beam or the upper portion of the column. The variables considered include the fuse length within the hinge, the transverse spacing of the hoops in the hinge zone, and the amount of axial load. A second specimen representative of a factory-made pre-cast column was also constructed using the fuse-bar connection and repaired ten times to examine the effect of the amount of axial load and aspect ratio, and to perform several constant drift amplitude tests. For comparative purposes, a conventionally reinforced column was also constructed and tested in a similar fashion to the specimens with renewable fuse-bar plastic hinges.

For earthquake resistant design it is essential that the dependable fatigue capacity of reinforced concrete bridge piers exceeds the fatigue demand imposed by maximum credible earthquake motions. Thus, by using realistic hysteretic models that are representative of bridge piers, the cyclic loading fatigue demand is assessed for a number of naturally occurring ground motions. The results are then generalized for design purposes. By comparing hysteretic fatigue demand versus capacity the designer can choose appropriate fuse bars for dependable behavior.

THEORETICAL FATIGUE CAPACITY

Fig. 1 shows some experimental test results of recent studies by Mander et al. [2] on the low cycle fatigue performance of reinforcing steels. These results show that, regardless of the steel grade, a dependable plastic strain-life fatigue relationship is given by

$$\varepsilon_{ap} = 0.08\,(2N_f)^{-0.5} \tag{1}$$

in which N_f = number of cycles to the appearance of the first fatigue crack (see Fig. 1a); and ε_{ap} = plastic strain amplitude defined as the half-amplitude of the plastic strain range which is $\varepsilon_{ap} = 0.5(\varepsilon_{\max} - \varepsilon_{\min}) - \varepsilon_y$, where ε_y = yield strain, $\varepsilon_{\max}$ = maximum tensile strain, and $\varepsilon_{\min}$ = maximum compression strain [2].

In terms of the total strain (ε_a) an alternative fatigue relationship may be given as (Fig. 1b):

$$\varepsilon_a = 0.08\,(2N_f)^{-0.333} \tag{2}$$

where $\varepsilon_a = 0.5(\varepsilon_{\max} - \varepsilon_{\min})$ which is the half-amplitude of the total strain range for one or under one fully reversed cycle of loading.

By assuming a linear strain profile across the critical section of a concrete column, plastic strains can be related to the plastic curvature (ϕ_p) by

$$\phi_p = \frac{2\,\varepsilon_{ap}}{(D - 2d')} \tag{3}$$

where D = overall column diameter and d' = depth from the outermost concrete fiber to the center of reinforcement (Note: $D - 2d'$ = pitch circle diameter of the longitudinal steel in a circular column).

Substituting Eq. (3) into Eq. (1), one obtains a plastic curvature-life fatigue relationship for reinforced concrete columns

$$\phi_p D = \frac{0.113}{1 - 2d'/D} N_f^{-0.5} \tag{4}$$

where $\phi_p D$ = a dimensionless plastic curvature amplitude. When determining $\phi_p D$ from experiments the spread of plasticity, as well as the displacement history, are important. The equivalent plastic hinge length, which is a measure of the plasticity spread, may be empirically determined from reference [3]

$$L_p = 0.08L + 4400\varepsilon_y d_b \tag{5}$$

where ε_y = yield strain of the reinforcement, d_b = diameter of the longitudinal bars and L = length of the column. Where fuse-bars are used, it will be assumed that the equivalent plastic hinge length is the same as the length of the machined down bar – the fuse length.

The plastic rotation (θ_p) is given by

$$\theta_p = \phi_p L_p = \frac{(\theta_u - \theta_y)}{(1 - 0.5L_p/L)} \tag{6}$$

where θ_u = maximum experimentally observed drift, θ_y = experimentally observed yield drift. Thus the experimentally determined plastic curvature may be determined from

$$\phi_p D = \frac{\theta_p}{L_p / D} \tag{7}$$

For a experimental tests that have a variable amplitude displacement history, it is necessary to convert the actual displacement history into an equivalent number of cycles at the maximum amplitude. Using Miner's rule coupled with Eq. (2), it can be shown

$$N_{eff} = \sum_i \left(\frac{\theta_{ui}}{\theta_{eff}} \right)^3 \tag{8}$$

where N_{eff} = effective number of cycles at a constant drift amplitude, θ_{eff}. Note Eq. (8) gives a "root-mean-cube" (RMC) relationship.

EXPERIMENTAL STUDY OF FATIGUE CAPACITY

Based on typical (900 mm diameter) prototype bridge columns, three one-third scale model pier specimens were constructed. Fig. 2 shows the details of one typical column specimen. All columns were of the same size (279 mm diameter, 1524 mm in height) and were reinforced with W2 (4.05 mm diameter) circular hoops (f_y = 317 MPa) at 50.8 mm spacing outside the plastic hinge zone. Within the plastic hinge zone the same transverse reinforcement was used for all of the columns in the form of either individual circular hoops or spirals of various spacings. The hoop spacing was based on buckling restraint and confinement requirements. The longitudinal reinforcement of the conventionally-designed column consisted of 12-D13 Grade 414 MPa rebars while the other two columns were provided with twelve 12.7 mm diameter high strength thread bars (f_{su} = 841 MPa). For the two columns that possessed the high strength steel, the bars were machined down in the hinge zone to 8 mm diameter. This ensures that the bars are unable to yield in the unmachined areas. The details at the plastic hinge zone and variables for each test are listed in Table 1.

As shown in Fig. 2(a), the connection in the pre-cast column was designed to have a 127 mm diameter concrete core reinforced with 4-D19 Grade 414 MPa rebars extended 381 mm from the top of the foundation to the upper portion of column, and with W2 spirals spaced at 12.7 mm. The lower half of the connection core was wrapped with a thin layer of plastic to simulate pre-cast conditions and avoid the bonding of the new-site cast concrete to the inner core concrete. The inner core was designed to resist axial and shear loads, but sustain no moment.

The experimental setup of the test is shown in Fig. 2(b). To prevent sliding of the specimen under lateral load, the foundation beam was anchored to the 457 mm thick laboratory strong-floor by applying a prestress of 250 kN to each 25 mm high strength thread bar. The lateral load was applied by a 250 kN servo-controlled actuator at 1245 mm above the foundation surface for all but one specimen. For the specimen "SHEAR", the actuator was located at a lower height of 641 mm to investigate the variability of the connection in zones of high shear demand coupled with high moment. The gravity load was applied by a 100 kN hydraulic actuator through a W10x77 lever beam seated on a steel bearing at the top of the column. During testing, force-control was used to hold the vertical load at a prescribed constant level. All specimens were subjected to the same amount of axial load, with the exception of specimens R5, PC-R0, and, PC-R1 which were subjected to higher levels of axial load. Forces, displacements, and column rotations were measured by load cells, and sonic and linear potentiometer displacement transducers, respectively.

Eleven column specimens were tested under various drift amplitudes; the remaining seven specimens were tested under constant drift amplitude (fatigue tests). In variable drift amplitude tests, two loading stages were applied. Quasi-static lateral loading was first performed on each specimen with a cyclic frequency of 0.017 Hz. Two cycles of lateral loading were applied at each of the following drift amplitudes: ±0.25%, ±0.5%, ±1%, ±2%, ±3%, ±4%, and ±5%. After the quasi-static loading phase, testing continued at the ±5% drift amplitude at a quasi-dynamic cyclic frequency of 0.17 Hz until failure occurred – failure being defined as the first low cycle fatigue fracture of the longitudinal reinforcement. In the constant drift amplitude tests, the drift amplitude was held constant until failure occurred. During these tests the same strain rates were used for each specimen by adjusting the sine wave frequencies to give the same peak-to-peak actuator velocities. Thus cyclic frequencies ranged from 0.17 Hz to 0.5 Hz, the lower value being for the largest displacement amplitude while the higher frequency was used for the low amplitude test (Fig. 2.5).

EXPERIMENTAL FATIGUE CAPACITY RESULTS

Eighteen column specimens have been tested to date in the experimental phase of this research. Eleven of them were tested under variable drift amplitude. The relatively slender aspect ratio ($M/VD = 4.45$) of the column resulted in ductile responses from all of the specimens except test R5 which was tested under a higher level of axial load. That specimen failed prematurely due to P-delta effects that were transverse to the axis of testing. This occurred during the 4% drift amplitude and is believed to be primarily due to the initial transverse eccentricity that was locked in during the repair procedure.

The flexure-fatigue failure mode that was dominant in most tests was a result of large strain ranges induced in the longitudinal reinforcing bars. This eventually led to fatigue fracture of those bars. The hysteretic lateral load-drift behavior of this series of tests is shown in Figs. 3 and 4 for the reinforced and precast concrete columns, respectively. Note that the nominal ultimate strength computed in accordance with the flexural strength requirements of ACI-318 [1] is plotted as a straight line. With the exception of specimen R5, all of the specimens failed due to the low cycle fatigue of the longitudinal reinforcement at the 5% drift amplitude under quasi-dynamic loading as listed in Table 2.

Seven column specimens were tested under constant drift amplitudes. All of the columns failed due to low cycle fatigue of the longitudinal bars except specimens *FTG*-2.5 and *FTG*-2 which failed in high cycle fatigue under the low drift amplitudes of $\pm 2.5\%$ and $\pm 2\%$, respectively. The lateral load-drift behavior of this series of tests is shown in Fig. 5.

ANALYSIS AND DISCUSSION OF RESULTS

Using the aforementioned theory, the predicted number of loading cycles to first fracture are listed in Table 2. These results may be compared with the experimentally observed effective number of cycles at the 5% drift amplitude. According to Eq. (8), the effective number of cycles prior to the 5% drift amplitude is 1.6. The observed number of cycles at the 5% drift angle until first bar fracture were added to this value.

The experimental results of the relationship between fatigue life and plastic rotation for all columns can be plotted as shown in Fig. 6(a), in which the plastic rotation can be expressed in terms of the fatigue life by a regression analysis with a slope of -0.5. Similarly, the total drift angle can be expressed in terms of fatigue life by a slope of -0.333, as shown in Fig. 6(b). It should also be noted that the theoretical prediction for the conventional specimen (*CO*) is generally a little conservative compared to the observed number of cycles, as shown in Fig. 6(a). This is attributed to the conservatism in the assessment of the effective plastic hinge length which was believed to have spread somewhat more than that predicted by Eq. (5). As shown in Fig. 6, the fatigue life of high-cycle fatigue failure such as specimens *FTG*-2.5 and *FTG*-2 can also be conservatively predicted by this low-cycle fatigue theory. Certain results, such as specimens *FTG*-3.5 and *FTG*-4, appear to be unconservative with respect to the theoretical prediction. However, this variation is well within the -50% to +100% range of statistical variability that is commonly accepted as being reasonable for any low cycle fatigue theory.

Normalized cumulative energy versus cumulative plastic drift is shown in Fig. 7. The normalized energy is obtained by dividing the energy by $(M_n^+ + M_n^-)$, where M_n^+ is the nominal moment capacity of column in the push direction, and M_n^- is the corresponding value in pull direction. The energy is obtained by integrating the area under load-drift curve in Figs 3-5. The energy absorbed by an Elasto-Perfectly-Plastic (*EPP*) material is:

$$EPP = (M_n^+ + M_n^-)(\theta^+ + \theta^-) \tag{9}$$

where θ^+ = the plastic component of drift in the positive (actuator push) direction, and θ^- in the negative (actuator pull) direction. As shown in Fig. 7, the conventional column had a slightly greater energy absorption capacity, in which the yield of the longitudinal bars spread over a larger portion of the column hinge than that of the renewable hinge columns. Also, longer fuse lengths, more transverse confinement, higher axial loads and lower aspect ratios seem to result in larger energy absorptions in the columns.

From the test results and theory, it is evident that the fatigue life is related to the length over which the fuse-bar yields. If required, improved fatigue life can be provided by lengthening the machined portion of the fuse-bar. This is at the expense of repairing a longer length of damaged column.

THEORETICAL FATIGUE DEMAND

In order to assess the hysteretic energy and cyclic loading fatigue <u>demand</u> of reinforced concrete bridge piers, reliable hysteretic models that are representative of real bridge behavior are necessary. Therefore, a rule-based smooth hysteretic model was developed that is capable of capturing the behavior of bridge piers. The model parameters are determined automatically by using a system identification routine in conjunction with either real experimental data from large scale laboratory tests, or results generated from the reversed cyclic loading Fiber Element analysis computer program UB-COLA [4].

A SDOF inelastic dynamic time-history analysis program was developed for using the new rule-based smooth model as well as more traditional hysteretic models such as the piece-wise linear Takeda model [5]. Spectral results were produced by using several different hysteretic models. An example of all the spectra generated for one earthquake using smooth hysteretic model are shown in Fig. 8. The smooth model was calibrated with full-size bridge column experimental data to determine global parameters to simulate structural force-deformation behavior. The calibration is summarized in Fig. 9. The cyclic loading demand results from several analyses are summarized in Fig. 10, and the effective dynamic magnification of displacement response depicted in Fig. 11. The cyclic fatigue demand $N(d)$ may be conservatively expressed as

$$N(d) = 7T^{-1/3} \quad ; \quad \text{but} \quad 4 \le N(d) \le 20 \qquad (10)$$

where T = natural period of vibration of the structure.

CONCLUSIONS

Based on the work completed to date, the following conclusions can be drawn:

1. Failure modes such as longitudinal bar buckling and transverse hoop fracture can be suppressed if sufficient transverse reinforcement is used. The failure mode thus becomes the low cycle fatigue capacity of the longitudinal reinforcement.

2. The fatigue failure capacity of reinforced concrete bridge columns can be predicted by the theory presented herein without modification for low cycle fatigue failure mode. For high cycle fatigue failure mode, the low cycle fatigue theory tends to be conservative.

3. The concept of a renewable plastic hinge has been introduced and validated experimentally. The fatigue life capacity can be tuned to the fatigue demand by providing an appropriate length of fuse-bar and transverse confinement. This appears to be a promising method for constructing structures that can be repaired following an earthquake. The approach is particularly useful for bridge piers as precast columns can be factory manufactured, transported to site and quickly erected using the fuse-bar approach.

4. Fuse-bars can easily be replaced after the column hinge zone has been damaged. The repaired column performs as well as the undamaged virgin columns.

5. The performance of renewable hinge columns is insensitive to changes in the axial load and the aspect ratio.

6. A methodology for the seismic design bridge piers which incorporates fatigue demand is advanced. This approach implicitly accounts for the duration effects of earthquakes.

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TABLE 1—SPECIMEN DETAILS AND TEST VARIABLES.

Specimen	Steel Ratios: Long. ρ_v	Steel Ratios: Trans. ρ_s	Fuse Length L_f (mm)	Concrete Strength: Outside Hinge f'_c (MPa)	Concrete Strength: In Hinge[@] f'_c (MPa)	Axial Load: P (kN)	Axial Load: $\frac{P}{f'_c A_g}$ Outside Hinge	Axial Load: $\frac{P}{f'_c A_g}$ In Hinge
CO	0.025	0.0070[1]	--	41.9	41.9	205	0.080	0.080
R0[&]	0.01	0.0081[1]	140	34.8	34.8	205	0.096	0.096
R1[#]	0.01	0.0094[2]	140	34.8	64.4	205	0.096	0.050
R2[#]	0.01	0.0140[1]	89	34.8	48.3	205	0.096	0.070
R3[#]	0.01	0.0160[1]	140	34.8	44.7	205	0.096	0.075
R4[#]	0.01	0.0150[1]	191	34.8	59.0	205	0.096	0.058
R5[#]	0.01	0.0150[1]	191	34.8	64.1	632	0.296	0.161
Precast								
PC-R0[&]	0.01	0.0180[2]	191	46.2	46.2	632	0.224	0.224
PC-R1[#]	0.01	0.0180[2]	191	46.2	55.6	632	0.224	0.185
PC-R2[#]	0.01	0.0080[2]	191	46.2	46.5	205	0.072	0.072
FTG-6[#]	0.01	0.0100[2]	191	46.2	47.2	205	0.072	0.071
FTG-5[#]	0.01	0.0100[2]	191	46.2	46.3	205	0.072	0.072
FTG-4[#]	0.01	0.0100[2]	191	46.2	46.8	205	0.072	0.071
FTG-3.5[#]	0.01	0.0100[2]	191	46.2	40.2	205	0.072	0.083
FTG-3[#]	0.01	0.0100[2]	191	46.2	43.9	205	0.072	0.076
FTG-2.5[#]	0.01	0.0100[2]	191	46.2	47.6	205	0.072	0.070
FTG-2[#]	0.01	0.0100[2]	191	46.2	47.5	205	0.072	0.070
SHEAR[#]	0.01	0.0100[2]	191	46.2	50.7	205	0.072	0.066

[1]circular hoops, [2]spiral
[@]concrete compressive strength at test day
[&]virgin specimen of replaceable-hinge column
[#]retrofitted specimen of replaceable-hinge column

TABLE 2—PREDICTED AND OBSERVED CYCLES TO FATIGUE FAILURE.

Specimen	Hinge or Fuse Length	θ (%)	$\phi_p D$	θ_p (%)	N_f^{theory}	$N_f^{expt.}$
CO	224	5.0	0.0508	3.70	8.3	28.0
R0	140	5.0	0.0804	3.90	2.9	5.1
R1	140	5.0	0.0775	3.50	3.6	6.7
R2	89	5.0	0.1242	3.65	1.4	2.9
R3	140	5.0	0.0819	3.70	3.2	4.5
R4	191	5.0	0.0615	3.70	5.7	5.6
R5	191	5.0	0.0648	3.90	5.1	--
PC-R0	191	5.0	0.0681	4.10	4.6	5.1
PC-R1	191	5.0	0.0665	4.00	4.8	6.2
PC-R2	191	5.0	0.0615	3.70	5.7	7.3
FTG-6	191	6.2	0.0804	4.78	3.3	3.0
FTG-5	191	5.2	0.0638	3.66	5.3	5.4
FTG-4	191	4.1	0.0457	2.72	10	7.8
FTG-3.5	191	3.6	0.0332	2.11	19	13
FTG-3	191	3.1	0.0286	1.69	26	25
FTG-2.5	191	2.6	0.0166	1.12	78	107
FTG-2	191	2.0	0.0081	0.50	323	440
SHEAR	191	5.0	0.0615	3.70	5.7	6.7