DUCTILITY OF REINFORCED CONCRETE BRIDGE PIERS

by

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SYNOPSIS

The design, manufacture and testing of five reinforced concrete bridge pier models under simulated seismic loading are described. Results from slow cyclic testing of four units show that adequate ductility can be obtained, but that ACI recommendations for confining steel may be inadequate to prevent buckling of vertical compression steel. Dynamic testing of a fifth model produced results which agree well with the static test results, and with predictions of an inelastic time-history computer analysis.

INTRODUCTION

Damage to bridge structures during the 1971 San Fernando earthquake (1) and other recent earthquakes has indicated the urgent need for theoretical and experimental research into the seismic response of bridge structures.

The current design philosophy for reinforced concrete frames requires energy dissipation by the formation of plastic hinges in beams. Plastic hinges in columns are held to be undesirable and are avoided by an over-capacity design approach (2). As a result of this philosophy a substantial amount of experimental research has been carried out in recent years to ascertain the available ductility of beam-column joints with beam hinges (3,4).

In general, satisfactory bridge behaviour under seismic attack will require energy dissipation by the formation of column (pier) hinges, and much of the available experimental data does not apply. Data are urgently needed for pier shapes, sizes and loads commonly adopted for bridge design. The experimental work described in this paper represents the initial stages of a continuing project designed to establish the necessary data.

DESCRIPTION OF TEST UNITS

The five pier units tested all represent variations of the same prototype; namely a single stem octagonal pier 1.5 m wide by 6 m clear height, reinforced vertically with 20 bundles of 3 - 32 mm dia. bars giving a steel content of 2.7%. This pier would satisfy New Zealand seismic design requirements (5) for a bridge superstructure of Precast I Beams 20 m long supporting an in-situ deck 10 m wide by 180 mm deep.

Units 1 to 3 were modelled to a scale of 1/3 full size. Each bundle of 3 - 32 mm dia. bars was modelled by 2 - 13 mm dia. deformed bars. (See Fig. 1.) Transverse reinforcing was designed to ACI requirements (6), and

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since the design ultimate axial load was only $0.06f'_cA_g$ the piers were designed as flexural members. Within the hinge region minimum requirements governed, namely $A_v d/s = 0.15A'_s$ or $0.15A_s$, whichever is the greater, with spacing not to exceed $d/4$ or 16 bar diameters. For Units 1 to 3 this was provided by 6.5 mm dia. welded hoops at 65 mm centres, giving a volumetric transverse steel content of 0.44%.

Units 1 to 3 differed only in the manner of load application. As is shown in Fig. 1 vertical pier load was applied by three jacks tensioning unbonded Macallow bars anchored to the pier base. The proportion of the constant total vertical load applied by each jack could be varied simultaneously with the horizontal load applied at the pier cap mid-height. Thus the ratio of base moment to shear could be altered, simulating piers of different heights. In prototype terms the effective horizontal load application points were 8.25, 5.25 and 9.75 m above the base for Units 1 to 3 respectively. Load was applied as slow cyclic load reversals of increasing displacement amplitude. Complete descriptions of Units 1 to 3 are given in ref. 7.

Unit 4 contained the same vertical steel as Units 1 to 3, but transverse steel was designed to the more stringent requirements of ref. 5, namely, that within the hinge region hoop spacing should not exceed 100 mm, and the volumetric ratio of circular reinforcing should not be less than $0.12f'_c/f_y$. In model terms this translated to 8 mm dia. hoops at 34 mm centres. A recent design document (8) results in similar transverse steel requirements. No vertical load was applied to Unit 4 but the pier cap was increased in size to allow horizontal load application at 2.75 m (i.e. 8.25 m prototype scale) above the base and to exactly model dead load stresses in Unit 5. Unit 5 represented the prototype pier at 1/6 scale, and Unit 4 at 1/2 scale. Vertical steel for Unit 5 consisted of 10 - 13 mm dia. deformed bars from the same steel batch used for Unit 4 and care was taken to obtain close agreement between concrete properties for the two units. Hoop steel was 4.4 mm dia. at 14 mm centres. Unit 5 was subjected to sinusoidal and simulated earthquake base accelerations using an MTS Electro-hydraulic system coupled to a shaking table. The purpose of these tests was to compare results with the statically tested Unit 4 in an investigation of the influence of scale and testing speed, and to compare dynamic response with results predicted using an inelastic analysis program developed by Sharpe (9). For this purpose a suitably scaled acceleration record of El Centro N-S 1940 was integrated twice by computer and applied to the shaking table as a displacement time-history. More complete details of Units 4 and 5 are given in ref. 10.

Instrumentation of all units included straingauge measurements of vertical and hoop reinforcing, hinge curvature measurements and deflection profiles, and in the case of Unit 5, input and response accelerations to enable signal integrity and response inertia forces to be assessed.

Materials strengths for Units 1 to 5 are given in Table 1.

RESULTS

Figs. 2a to 2c show the moment-displacement response of Units 1 to 3, together with photographs of the plastic hinge condition at salient points in the test sequence. In each case base moments have been scaled to prototype values and displacements to prototype displacements at the effective theoretical mass centre. Theoretical ultimate moment capacities based on measured material properties are indicated by dashed lines marked M_u. Displacement ductility factors are based on the experimental yield displacement found by extrapolation of the post-cracking elastic moment-displacement curve to the theoretical ultimate moment capacity.

Behaviour as illustrated in Fig. 2 is very good, with displacement ductility factors in excess of 5 being achieved for each unit. Maximum base moments obtained exceed M_u by 12%, 25% and 10% for the three units respectively. This high excess capacity can be attributed to the short steel yield plateau (measured at 7.3 x yield strain) resulting in early strain hardening of vertical steel. Maximum shear stress at the plastic hinge was equivalent to $0.22\sqrt{f'_c}$ MPa based on the gross cross-sectional area. Despite the generally satisfactory behaviour, hoop steel strains exceeded yield at the first peaks to DF = 4.8, 8.3 and 5.3 for the three units respectively. With this yielding, and loss of cover concrete, buckling of compression steel occurred progressively over the next few cycles with subsequent moment and stiffness degradation. This was particularly noticeable for Unit 3 which formed a substantial cavity inside the reinforcing cage (see photo, Fig. 2c).

Unit 4 (Fig. 2d) with the heavier transverse steel content behaved exceptionally well. Stable loop with only minor load and stiffness degradation were obtained at all displacement levels. Maximum recorded hoop steel strains reached 92% of nominal yield based on f_y = 275 MPa and E = 200 GPa, indicating that the design requirement (5) was realistic.

Fig. 3 compares static and dynamic moment displacement curves for Units 4 and 5 scaled to prototype dimensions. In both cases the curves represent the envelope obtained by joining peaks obtained on first attaining a new maximum displacement. Note that the dynamic curve is initially stiffer than the static curve, but the difference is not great. The relative reduction in dynamic capacity at higher displacements is probably due to the small increments between successive peak displacements during dynamic testing of Unit 5 (compare with large increments in Fig. 2d), and is not thought to be significant.

Theoretical and experimental response of Unit 5 to the first 10 sec. of El Centro N-S 1940 are compared in prototype dimensions in Fig. 4. The theoretical curve is based on the experimentally observed elastic stiffness, a bilinear moment-curvature relationship and a viscous damping of 7%. Agreement is good for the 10 sec. shown and is adequate for the remainder of the earthquake record, though experimental displacements exceed theoretical values, indicating a lower viscous damping at lower levels of response.

CONCLUSIONS

The static load tests reported indicate good energy dissipation potential for circular bridge piers with low axial load. It was found that although adequate behaviour was exhibited by all units, transverse steel in accordance with ACI minimum requirements for flexural members was insufficient to restrain compression steel from buckling. A unit designed to more stringent transverse steel requirements gave substantially improved behaviour. Good agreement between moment-displacement curves for a sixth scale model tested dynamically and a third scale model tested statically, gives confidence in the continued investigation of ductility capacity using statically tested models. The adequacy of existing inelastic time-history computer analyses was confirmed by comparison with experimental dynamic results. This research has so far been confined to piers with low axial load. More research is needed to investigate the behaviour of heavily loaded piers.

ACKNOWLEDGEMENTS

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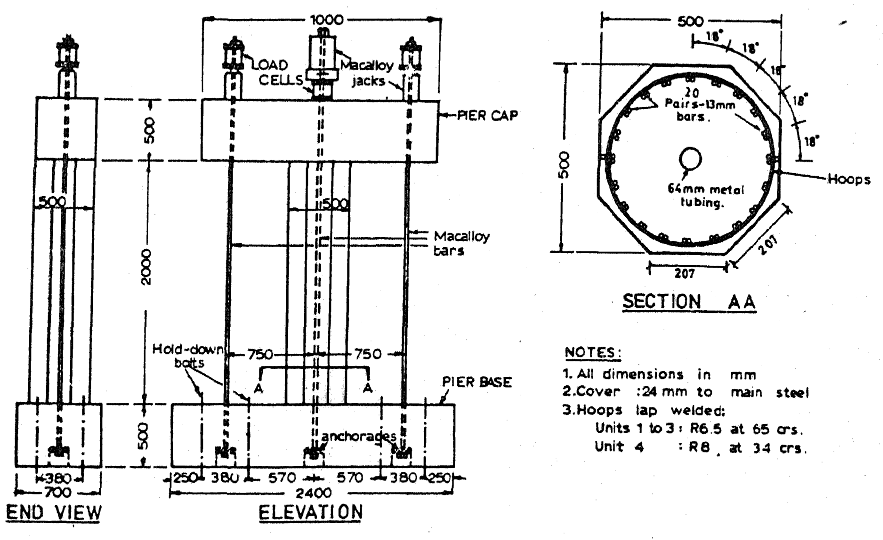
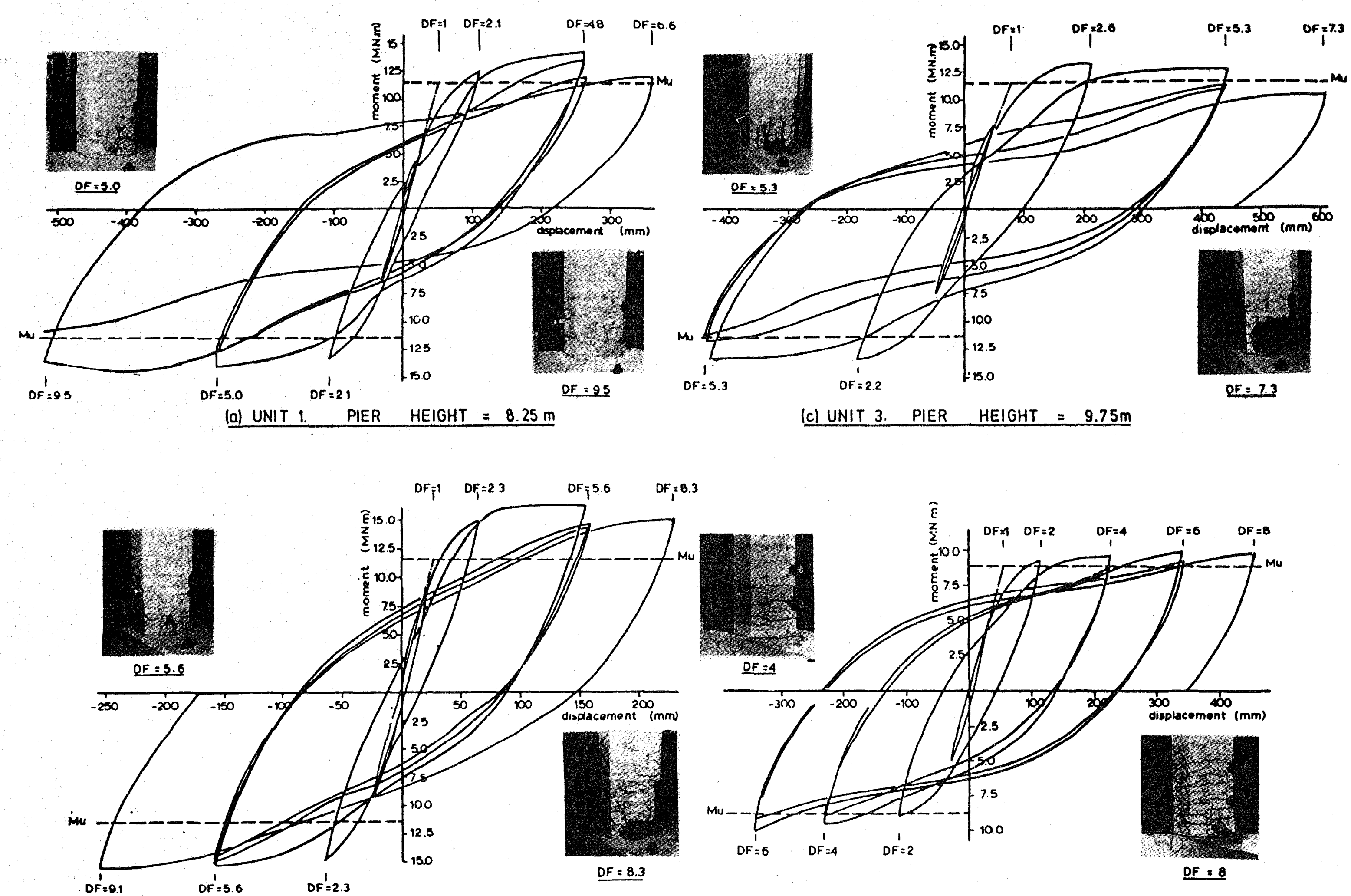


FIG.1 DIMENSIONS OF 1/3 FULL SIZE UNITS



(a) UNIT 1. PIER HEIGHT = 8.25 m

(c) UNIT 3. PIER HEIGHT = 9.75m

(b) UNIT 2. PIER HEIGHT = 5.25m

(d) UNIT 4. PIER HEIGHT = 8.25m

FIG. 2 MOMENT - DISPLACEMENT LOOPS, UNITS 1 TO 4

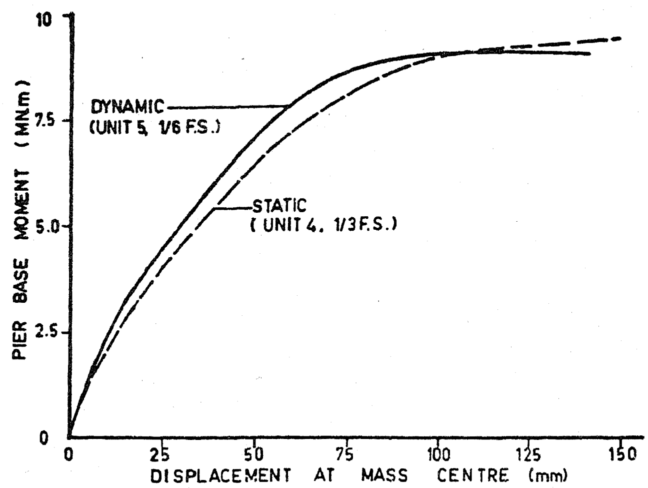


FIG. 3 STATIC + DYNAMIC MOMENT-DISPLACEMENT CURVES

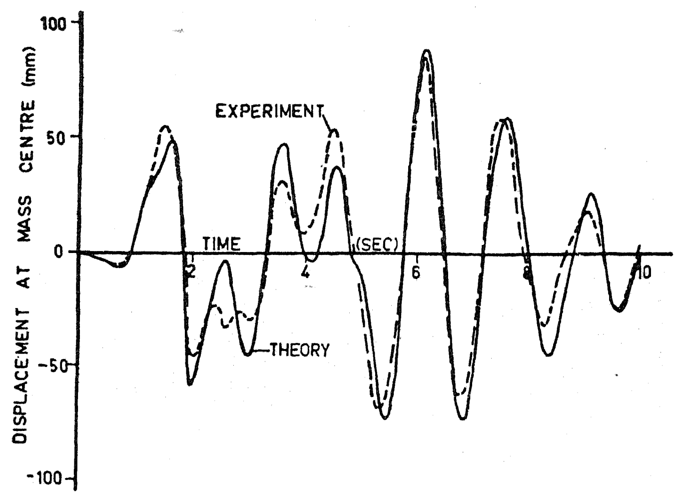


FIG. 4 RESPONSE TO EL CENTRO 1940 N-S

TABLE 1 - MATERIAL PROPERTIES

UNIT	Concrete Strength MPa	Vertical Steel Yield MPa	Vertical Steel Ultimate MPa	Hoop Steel Yield MPa	Hoop Steel Ultimate MPa
1	33.2	373	564	312	421
2	34.8	371	562	312	421
3	33.8	373	563	342	493
4	40.0	305	411	389	493
5	35.1	305	411	263	360