Engineering Structures 150 (2017) 359–372

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Interaction of plastic hinges in prestressed concrete bridges with corrugated steel webs

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ARTICLE INFO

Article history:
Received 17 December 2015
Revised 11 July 2017
Accepted 13 July 2017
Available online 27 July 2017

Keywords:
Corrugated steel web
Critical region
External tendon
Full-range structural behaviour
Plastic hinge length
Prestressed concrete

ABSTRACT

Prestressed concrete bridges with corrugated steel webs have emerged as one of the promising bridge forms. However, the presence of prestressing tendons and shear-deformable corrugated steel webs with negligible axial stiffness complicates the formation of plastic hinges under applied loading to failure. The full-range structural behaviour of these bridges and plastic hinge formation are therefore studied experimentally and numerically. The more localised flange plastic hinge caused by both web shear deformation and local flange bending interacts with the full-depth plastic hinge of such a bridge. Tests show that the full-range structural behaviour of the bridge is mainly governed by the localised flange plastic hinges. A formula is proposed to predict the equivalent interactive plastic hinge length, which helps to predict the full-range structural behaviour taking into account the presence of external prestressing tendons. Some design recommendations are also provided.

1. Introduction

Concrete bridges with corrugated steel webs and external prestressing have emerged as one of the promising bridge forms for short to medium spans. Since the appearance of Cognac Bridge in France in 1986, this form of bridges has evolved and spread to other countries. Its lightness makes it ideal not only for girder bridges but also serving as the deck in cable-stayed bridges. The corrugated steel webs provide high shear resistance and obviate the need for extensive stiffeners. Because of the negligible axial stiffness of corrugated webs, the prestress can be introduced efficiently to the concrete flanges that need pre-compression. Prestressing is often achieved by both internal and external tendons. This structural form therefore provides excellent structural efficiency. Hereafter in this paper, this form of bridge is assumed unless otherwise stated.

Although various models [1,2] have been proposed to predict the static elastic behaviour of the bridge, few researchers have studied the formation of plastic hinges in these bridges and their effect on the full-range structural behaviour that covers both the pre-peak-strength and post-peak-strength responses. Plastic hinge formation is often used to describe the behaviour of concrete beams at failure, *e.g.* Ho and Pam [3], Au et al. [4], Du et al. [5], Chou et al. [6], Yang et al. [7], Di Ludovico et al. [8], and Lee et al. [9], but it remains a controversial issue [10]. There are three approaches in describing the extent of plastic region: (a) the actual size of plasticity zone observed, (b) the equivalent size estimated based on a representative plastic curvature, and (c) the size of region requiring effective confinement. In Approach (a), the physical length of the actual plastic hinge region, *i.e.* the physical plastic hinge length l_{pc}, is defined as the length over which actual plasticity spreads. To describe the deformation capacity in Approach (b), an equivalent plastic hinge length l_p is often defined where the plastic curvature is assumed to be constant at the peak value [11]. Approach (c) by providing effective confinement will be explained later.

Studies of plastic hinges in reinforced concrete (RC) and prestressed concrete (PC) members are mostly based on experiments. Table 1 summarises various empirical formulae proposed for the equivalent plastic hinge length l_p on one side of the critical section for conventional concrete members as elaborated below. The major factors affecting the equivalent plastic hinge length include the effective or total depth of section, concrete strength, compressive axial load level, properties and areas of longitudinal and transverse steel, and distance between the critical section and point of contraflexure. As the expressions in Table 1 are taken from various sources [12–27], the symbols are adjusted for consistent presentation. The geometric properties include the distance from the

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http://dx.doi.org/10.1016/j.engstruct.2017.07.036
0141-0296/

Table 1
Equivalent plastic hinge length l_p on one side of critical section.

Source	Equivalent plastic hinge length on one side	Applicable members
Baker [12]	$k_1k_2k_3(z/d)^{0.25}d$	Members with unconfined concrete
Baker and Amarakone [13]	$0.8k_1k_3(z/d)c$	Members confined by transverse steel
Sawyer [14]	$0.25d + 0.075z$	RC members
Corley [15]	$0.5d + 0.2\sqrt{d}(z/d)$	Beams
Mattock [16]	$0.5d + 0.05z$	Beams
Park et al. [17]	$0.5h$	Columns
Priestley and Park [18]	$0.08z + 6d_b$	Confined columns
Paulay and Priestley [19]	$0.08z + 0.022d_bf_y$	Beams and columns
Sheikh and Khoury [20]	$1.0h$	Columns under high axial loads
Bayrak and Sheikh [21]	$0.9h$ to $1.0h$	Columns
Coleman and Spacone [22]	$G_{cf}/[0.6f_c'(\varepsilon_{20} + 0.8f_c'/E_c - \varepsilon_c)]$	RC members
Panagiotakos and Fardis [23]	$0.18z + 0.021d_bf_y$	Beams and columns
Mendis [24]	$0.4h$	Columns
Bae and Bayrak [25]	$l_p/h = [0.3P/P_0 + 3A_s/A_g - 0.1]z/h + 0.25 \geqslant 0.25$	Columns
Berry et al. [26]	$0.05z + 0.1f_yd_b/\sqrt{f_c'}$	Columns
Subramanian [27]	$l_p/h = 0.9[1 + 0.5P/P_0]k_3(z/h)^{0.25}$ for $P/P_0 > 0.2$ $l_p/h = 0.25$ for $P/P_0 \leqslant 0.2$	Columns

critical section to the point of contra-flexure z; the effective section depth d; the neutral axis depth c at the ultimate moment; the section depth h of the column, and the gross area of member section A_g. The longitudinal reinforcement of sectional area A_s comprises bars of diameter d_b and yielding stress f_y. The concrete has compressive cylinder strength f_c', peak compressive strain ε_c, strain ε_{20} corresponding to 20% of the compressive strength, and fracture energy in compression G_{cf}. The compressive axial force in the member is P, while the axial compressive strength of member without bending moment is given by $P_0 = 0.85f_c'(A_g - A_s) + f_y\ A_s$. The parameters used in Table 1 are: $k_1 = 0.7$ for mild steel, or 0.9 for cold-worked steel; $k_2 = 1 + 0.5P/P_0$; $k_3 = 0.6$ for $f_c' = 35.2$ MPa, or 0.9 for $f_c' = 11.7$ MPa.

Some researchers, *e.g.* Youssf et al. [28], have also studied the plastic hinge of concrete columns confined by fibre-reinforced polymer (FRP). Despite various formulae suggested for the equivalent plastic hinge length, no common consensus has been reached. Another related quantity by Approach (c) is the critical region length l_c in design, defined as the extent requiring effective confinement by transverse reinforcement so as to achieve the intended performance level of flexural ductility [29]. The equivalent plastic hinge length should be a fictitious extent within the critical region of a member. Clause 10.4.5 of NZS 3101 [30] recommends the critical region length or ductile detailing length to depend only on the applied compressive axial load level $P/(\lambda_r A_g f_c)$, where the strength reduction factor λ_r is taken as 0.65 and 0.85 for unconfined and confined concrete respectively. For an RC member with $P/(\lambda_r A_g f_c) \leq 0.25$, $0.25 < P/(\lambda_r A_g f_c) \leq 0.5$ or $0.5 < P/(\lambda_r A_g f_c) \leq 0.7$, its critical region length is $1.0h$, $2.0h$ or $3.0h$ respectively; or is over a region where the moment exceeds 0.8, 0.7 or 0.6 respectively of the maximum moment, whichever is larger. Paultre et al. [31] showed the critical region length to range from $1.0h$ to $2.0h$, depending on the volumetric ratio of the transverse steel and the compressive axial load level. Pam and Ho [29] observed the critical region length to increase with the compressive axial load level and concrete strength. Hence, the critical region length increases with compressive axial load level, concrete strength, longitudinal steel ratio and strength, but reduces with the transverse steel ratio.

The presence of shear-deformable corrugated steel webs with negligible axial stiffness and prestressing tendons in this kind of bridges certainly complicates the formation of plastic hinges further. While the above formulae are useful for estimation of equivalent plastic hinge length within their respective scopes, their application to the present case should be treated with care. Hence in this study, the critical region length, performance of the plastic hinges and full-range structural behaviour of the bridges are studied.

2. Experimental programme

Some concrete beam specimens with corrugated steel webs of depth 360 mm were fabricated for testing as shown in Fig. 1. Specimens B-1 and B-2 were post-tensioned by two external 7-wire steel strands each of nominal diameter 12.7 mm and sectional area 98.7 mm^2. Specimen B-3 was not prestressed. Specimen B-4 was post-tensioned by two external aramid fibre reinforced polymer (AFRP) ropes each of area 54.5 mm^2. In Specimen B-1, the anchorages of straight strands were located at the end diaphragms at a level 110 mm above the soffit. In Specimens B-2 and B-4, the anchorages were located at the centroidal level of end diaphragms while the strands at intermediate deviator were 100 mm above the soffit. The effective prestressing forces of Specimens B-1, B-2 and B-4 were 268, 252 and 102 kN respectively. The end and intermediate diaphragms were 200 and 90 mm thick respectively. Embedment connections were provided between the concrete flanges and corrugated steel web. All specimens were simply supported over a span of 3600 mm and tested by non-reversed cyclic loading at mid-span to increasing ductility levels under displacement control. Load cells were used at the end of each tendon to monitor the variation of tendon force during tensioning and the subsequent loading test. Linear variable differential transformers (LVDTs) were used to measure displacements during the test. Strain gauges were provided on concrete surface and steel bars at selected sections for monitoring of strains. The specimens under test are shown in Fig. 2.

The material properties were determined as shown in Table 2. The Young's modulus and ultimate strength of AFRP ropes were respectively 126.5 GPa and 1248 MPa. In tests, the tendons usually broke at the anchorage.

3. Experimental and numerical investigation

3.1. Elastic behaviour

Consider for example a bridge of this type under a few point loads. While the pair of concrete flanges tends to deflect together in flexure as a smooth curve, the corrugated steel webs tend to deflect in shear as a connected series of straight line segments, thereby causing interaction between the structural components