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Article publicat / *Published paper*:

Villalba, S., Casas, J. (2016) New structural joint by rebar looping applied to staged box girder bridge construction. Static tests. Structural Concrete . DOI: 10.1002/suco.201500117.

Technical Paper

New structural joint by rebar looping applied to staged box girder bridge construction. Static tests

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Abstract

This paper shows the design, development and experimental checking of a modified type of structural joint with limited length between concrete segments cast “in-situ”. The design concept is based on the development length of an anchorage hook stiffened by transverse reinforcement bars and is particularly suited for the case of in-situ construction of staged box girder bridges, seeking to the possibility of using lighter scaffolding.

The studies focusing on the strength, stiffness and serviceability of the proposed joint are presented. The research work comprises the bending behaviour of reinforced concrete slabs with loop joints with regard to the diameter of loop bar, loop joint width and ultimate and fatigue load. The results are compared to the behaviour of reinforced concrete slabs without joints. A total of 16 slabs were tested by static and fatigue loading tests. The present paper evaluates the flexural behaviour under static loading test. The results of fatigue tests have also shown an excellent performance.

In the static tests, crack width and crack pattern were observed at service load levels, and the ultimate behaviour was evaluated by means of up to failure tests.

From the test results, the service performance of the loop joints was confirmed similar to slabs without joints. The static loading tests confirm the good performance and effectiveness of this loop joint type under static loads. Details of loop joints design criteria are also suggested.

Keywords: Loop joint, bridge construction, box girder, staged construction, crack, static test.

This article has been accepted for publication and undergone full peer review but has not been through the copyediting, typesetting, pagination and proofreading process, which may lead to differences between this version and the Version of Record. Please cite this article as doi: 10.1002/suco.201500117.

Submitted: 03-Aug-2015
Revised: 11-Feb-2016
Accepted: 05-Mar-2016

1. Introduction

Bridge construction is in steady progress, in technological innovation and systematization of the construction process to improve execution, ensuring the safety and quality, and to achieve the optimization of means employed.

The use of precast deck systems in bridge construction requires structural joints between decks and between deck and girder too. Different studies of joints such as the "Poutre Dalle" system or the loop joint system in segmented precast deck were carried out some years ago¹, and more recently new theoretical and experimental works have been carried out^{2,3,4}. In the NCHRP report 173⁵, a summary of cast-in-place concrete connections for precast deck systems is presented.

The previous mentioned experiences deal with the joint between precast elements typical of precast construction. However, the current state of the art also offers many solutions in the construction of bridges and viaducts using cast-in-place technology. Cast-in-place construction is common in road and rail bridges but adapted to the particular characteristics and conditions imposed by the limits of deformability and the rail traffic actions. If no particular obstacles have to be crossed, the span of most bridges is limited to 30 to 60 metres. In these cases, a continuous post-tensioned concrete beam (box girder) is one of the most suitable solutions.

For long viaducts, movable span-to-span scaffolding is normally used. However, the maximum resisting capacity of this auxiliary equipment could limit the maximum achievable span-length to be built, due to the huge self-weight per unit length of the total cross-section. One possible solution is to split the longitudinal casting of the bridge in two phases (staged construction). In the first stage, the "U" drawer including bottom slab and webs is executed and when the concrete strength is reached, the post-tensioning is

introduced and the formwork advanced to the next casting position. In a second stage, the upper slab until completing the box is built (see figure 1) supported on the previous phase. In the first phase, the weight of the cross-section is less and therefore longer spans are feasible with standard formworks available. In the second phase, the already built first phase is self-supporting and able to accommodate the additional weight transmitted by the fresh concrete.

Figure 1. Typical construction of concrete box-girder in two phases (staged construction).

One of the main disadvantages of this staged construction where the casting of the cross-section is implemented in two phases (box girders) is the limited performance and handling inside the cross section due to the obstacle produced by the reinforcing bars waiting for the connection of the second phase.

This technique also implies to generate two construction joints in the cross section. The use of splices by overlapping bars (force transfer between two spliced bars) usually leads to significant lengths of the reinforcing bars. Moreover, these procedures involve construction difficulties in the withdrawal of the inner formwork through the the end of the completed section (see figure 2).

Figure 2. View of the important length of overlapping bars that difficult formwork operability.

The aim of the work presented here is to design, develop, and evaluate experimentally a modified type of structural joint between concrete slab segments of reduced development length that requires shorter overlap lengths. With a shorter length of the splices, the above mentioned operational difficulties can be avoided. The proposed joint may be of great interest and applicability not only in the span-by-span construction of new bridges, but also in cases of repair/strengthening solutions where connections between different concreting phases are involved.

The design concept is based on the development length of an anchorage hook stiffened by transverse reinforcement bars (see figure 3). The main difference between previous researches on joints between precast elements and the one proposed here is the positioning of the longitudinal rebar (U-bar spacing). Previous works on loops joints use a U-bar spacing “s” between longitudinal rebars, whilst in this investigation the U-bar spacing “s” between the pair of overlapping rebars is null. This difference is due to the type of construction, being cast in-situ and not precast concrete. The design concept is based on an anchorage hook of reduced development length stiffened by transverse reinforcement bars.

Figure 3. Cross section of a concrete viaduct with a loop joint between concreting phases.

The application of this type of structural joints in concrete structures presents some advantages as: 1) Ease of manoeuvrability inside the cross section, 2) Significant reduction of overlap lengths 3) Easy removal of the internal formwork 4) Reduced material in reinforcing bars 5) Increased safety at work. So, the overall improvement in the execution performance at work is increased.

The mechanical behaviour of the proposed joint in terms of stiffness and strength needs to be investigated. Moreover, the structural joint must be highly durable. Many serviceability problems such as cracking and water leakage at transverse joints can appear in bridges if these issues are not well solved. Therefore, studies focusing on the strength, stiffness and serviceability of the joints must be conducted, before the application in real structures.

The work reported in Villalba⁶ and Villalba and Casas⁷ investigates the flexural behaviour of reinforced concrete slabs with the proposed loop joints. The test variables are the diameter of the loop bars and the length of the loop joint. To investigate the stiffness and strength under service fatigue and ultimate loads, experimental tests were carried out. Crack width and crack distribution were observed at service load levels, and the ultimate capacity was evaluated by means of up to failure tests. These results were compared with the behaviour of reinforcement concrete slabs without joints.

This paper presents the main results of the static tests carried out with the objective to validate the use of the proposed loop joint. The results of the fatigue and dynamic tests have been also reported elsewhere⁸.

2. Basis of joint design

The use of splices by overlapping bars (force transfer between two spliced bars by bond) is defined in this section. The calculations for the bar anchorage and lap lengths and end hook, as defined in German Code DIN 1045^{9,10,11} are presented.

The development length concept for the splices of hook reinforcement by bond was used for the design of details of standard loop joints. Using the development length for a deformed bar in tension terminating in a standard hook according to DIN 1045, details of standard loop joints could be determined (see figure 4). The hook splices length of

overlap bars is determined by equations (1), (2) and (3). The total lengths according to the diameter of loop bars are shown in table 1.

$$T = A_s f_{yd} = \int_0^{l_b} \tau(x) \cdot u \cdot dx = \tau_{adm} \cdot u \cdot l_b \quad (1)$$

$$l_b \geq \frac{T}{u \cdot \tau_{adm}} = \frac{A_s \cdot f_{yd}}{\pi \cdot \phi \cdot \tau_{adm}} = \frac{\phi \cdot f_{yd}}{4 \cdot \tau_{adm}} \quad (2)$$

where:

τ_{adm} = Admissible bond stress ($\tau_{adm} = 2.2 \text{ N/mm}^2$)

u = Perimeter section

A_s = Area section

l_b = Basic anchorage hook length

ϕ = Bar diameter

f_{yd} = Design yield stress

T = Tensile force.

and

$$l_u \geq \alpha_u \cdot \alpha_1 \cdot \alpha_A \cdot \frac{T}{\gamma \cdot u \cdot \tau_{adm}} = \alpha_u \cdot \alpha_1 \cdot \alpha_A \cdot \frac{\phi \cdot f_{yd}}{4 \cdot \tau_{adm}} = \alpha_u \cdot \alpha_1 \cdot \alpha_A \cdot \alpha_0 \cdot \phi \quad (3)$$

where:

α_u = coefficient applied to anchorage length depending on the position of rebar, and percentage of overlapping working tension bar relative to total steel section ($\alpha_u = 2.2$).

α_1 = Reduction factor depending on the type of anchor ($\alpha_1 = 0.50$).

$$\alpha_A = \frac{A_{e \text{ required}}}{A_{e \text{ provided}}} = 1$$

l_u = Total splice hook length (overlapping length).

Figure 4. Methods of splice design according to DIN 1045 code

The minimum overlap lengths for loop joint were defined according to the requirements of German Code DIN 1045 and Spanish Code EHE-08¹². The minimum necessary anchorage hook length is measured from the critical section (the section of the concreting joint) to the outside end (or edge) of the hook. The total anchorage hook length shall not be less than of 1.5 times the mandrel diameter "d_B" or 200 mm. Also, the internal diameter of the reinforcement hook "D" shall be larger than 7 diameters for reinforcement diameters higher than 20 mm, as defined in the Spanish Code¹². The resulting lengths based on the DIN Code and according to the new proposal presented in this paper are shown in table 1. The criteria for this new proposal is to make the joint as short as possible. It is clear the much shorter hook-end splice length (in the order of one third) obtained with the proposed new design.

Table 1. Total hook-end splice length l_u according to DIN 1045 and as proposed in Villalba^{6,7} for two bar diameters.

ϕ	Diameter of mandrel $D \geq 7\phi$ (mm)	DIN 1045	l_u (mm)	Experimental design	l_u (new design) (mm)
20	185 ($D=9.25\phi$)	LD1	715	LE1	275
25	175 ($D=7\phi$)	LD2	900	LE2	300

3. Experimental Set-Up

3.1. Definition of tests specimens.

To investigate the structural performance of the loop joints, two experimental studies were conducted at the Structural Technology Laboratory of the Technical University of Catalonia (UPC). Firstly, static loading tests were performed in 8 slabs and fatigue loading tests were carried out in other 8 slabs with similar characteristics.

Dimensions of all the reinforced concrete specimens were: 5.60 m span length, 1.60 m width and 0.285 m thickness. The dimensions of the specimens are representative of the actual central upper slab in cast- in situ post-tensioned concrete box-girders. The slab was built up in two casting phases with a lag of 48 hours between concreting in order to represent as close as possible real concreting conditions on site.

The flexural tests were carried out through a 3-point loading tests were carried out to investigate the mechanical behaviour of loop joints loaded by a combination of bending and shear. The slabs were simply-supported on elastomeric bearing pads.

A total of 8 slabs were statically tested. The test slabs are referred to hereafter as LR1_XX_YY_ZZ, LR2_XX_YY_ZZ, LD1_XX_YY_ZZ, LD2_XX_YY_ZZ, LE1_XX_YY_ZZ and LE2_XX_YY_ZZ, where the letters LR, LD and LE indicate the different types of rebar continuity. LR indicates reference slabs (without loop joint), LD indicates those designed according to DIN 1045 (splices by overlapping bars), and finally, LE indicates slabs with experimental design loop joint as proposed in the present work (see figure 5 and table 1). The numbers 1 and 2 correspond to the different diameter of the loop bars, being 1 for 20 mm diameters and 2 for 25 mm diameters. Finally, the letters _XX_YY_ZZ refer to the day, month and year of each slab fabrication. The number of slabs of each type is shown in table 2.

Table 2. Number of slabs types for each experimental campaign

ϕ	DIN 1045 (LD)	Experimental design (LE)	Reference slab (LR)
20	1	2	1
25	1	2	1

Figure 5. Geometrical definitions and reinforcement arrangement of specimens

3.2. Loading

The slabs were simply supported at both ends and the loading was applied using a hydraulic jack of 1MN capacity at the mid-span of the slab (see figure 6). The static loading tests were performed starting with an initial preload of 5kN, followed by an increasing load “ramp” using displacement control. The velocities adopted were 0.0075 mm/s and 0.03 mm/s for all slabs.

Figure 6. Loading set-up

3.3. Instrumentation

The measurement arrangement is shown in table 3 and figure 7. Twelve strain gauges (GA) were used in LE and LD slabs. Eight of these gauges were attached to the longitudinal bars at mid-span, specifically, at the beginning of the loop anchorage. Remaining gauges were attached to the transverse reinforcement bars. Joint opening and crack widths were measured from the initiation of loading using three magnetic transducers (TEMP1, TEMP2, TEMP3 in figure 7). Deflection was measured at the mid-span of the specimens and supports using seven LVDT (3 LVDT at mid-span, 2 LVDT at 1.50 meters from supports, and 2 LVDT at supports). With this disposition of LVDT it is

possible to obtain deflections and rotations in the elastomeric bearings and obtain real deflection.

In LR slabs four strain gauges were used (see figure 7). These gauges were attached to the tensile longitudinal bars at mid-span. The remaining instrumentation (LVDT and magnetic transducers) is the same as for LE and LD slabs.

Table 3. Sensors and variables measured

Reference	Type	Data measured
GA	Strain Gauges	Strain in the tension rebars at mid-span
LVDT	Linear Variable Differential Transducer	Deflection (mid-span and supports)
TEMP	Temposonics (magnetic transducers)	Joint opening

Figure7. Monitoring set-up and measurement locations. (a) LR2 slab, (b) LE2 slab, (c) LD1 slab.

3.4. Material properties

Compressive characteristic strength of the concrete specified in all slabs was 35 MPa, and the mean value of compressive strength f_{cm} associated with a specific characteristic strength f_{ck} was estimated with equation (4). The real compressive strength of the concrete measured in cylinders at the time of testing the slab is shown in table 4. The difference between the specified and actual strength is due to a higher concrete quality and also to the increase of strength with time. The reinforcement used for the slabs is a deformed bar with yield strength of 575 MPa (assuming the hypothesis to normal statistical distribution and relative standard deviation of 8%) and with diameters of, 20 and 25 mm for longitudinal bars, and 12, 16 and 20 for transversal bars.

$$f_{cm} = f_{ck} + 8 \quad (\text{in MPa}) \quad (4)$$

Table 4. Compressive strength of the concrete in each specimen

Slab	f_{ck} (N/mm ²)	f_{cm} (theoretical) (N/mm ²)	f_{cm} (experimental) (N/mm ²)	Difference (%)
LE1_12_07_08	35.00	43.00	63.27	47.14
LE1_11_08_08	35.00	43.00	51.60	20.00
LR1_03_07_08	35.00	43.00	51.31	19.33
LD1_07_07_08	35.00	43.00	47.68	10.88
LE2_14_07_08	35.00	43.00	59.94	39.40
LE2_24_07_08	35.00	43.00	53.27	23.88
LR2_21_07_08	35.00	43.00	50.77	18.07
LD2_28_08_08	35.00	43.00	54.94	27.77

4. Tests results and discussion

The experimental ultimate load capacity of each slab is shown in table 5 and compared with the theoretical value corresponding to the measured mean values for the compressive strength of concrete and yielding stress of steel. The longitudinal secant modulus of deformation (E in table 5) was estimated with equation (5). The actual capacity was always higher than the theoretically predicted. As seen in table 5, in the three-point loading tests, the ultimate response of the LE (members with loop joints) is very similar to that of ordinary LR members without joints. LD slabs have obtained the highest capacity due to the increased length of loop joints which derives on the fact that in the overlapping zone the steel amount is twice that in the outside zone.

$$E_{cm} = 8500 \cdot \sqrt[3]{(f_{cm})} \quad (\text{in MPa})$$

(5)

Table 5. Load Ratio, ultimate theoretical force vs. ultimate experimental force.

Slab	f_{cm} (N/mm ²)	E (N/mm ²)	P. ultimate theoretical strength (KN)	P. ultimate experimental strength (KN)	Difference (%)
LE1_12_07_08	63.27	33870	239.04	260.73	9.07
LE1_11_08_08	51.60	31645	237.12	242.92	2.45
LR1_03_07_08	51.31	31585	237.04	255.15	7.64
LD1_07_07_08	47.68	30822	236.32	276.86	17.15
LE2_14_07_08	59.94	33265	365.60	415.45	16.63
LE2_24_07_08	53.27	31983	363.28	401.92	10.63
LR2_21_07_08	50.77	31474	362.32	370.69	2.31
LD2_28_08_08	54.94	32313	363.92	445.50	22.42

Figure 8 presents the load versus time test curves of each specimen. Behaviour can be observed, under three different stages. It may be noted that the last part has got almost horizontal slope. In this last stage before failure there are discrete discontinuities and recoveries of the loading until the complete failure of the cross-section due to spalling of concrete in the compression zone.

Figure 8. Load vs. Time (a) 20 mm diameter, (b) 25 mm diameter.

Figure 9 plots the load versus strain in the longitudinal bars. For loads below the cracking load, a similar behavior is obtained for all specimens LR, LD and LE. The curves exhibited similar slopes regardless the diameter of the loop rebar. For higher load values, a detailed analysis has been carried out.

From the cracking load and for load values up to 60% of the ultimate load is observed a lower strain in the longitudinal rebars in type LE and LD slabs, for the same load level. This difference is around 55 to 65 %. It can be due to the fact that the steel ratio in the joint width is twice in LE and LD type slab than in LR slab.

For load values 60% of the ultimate load up to failure is observed how strain curves of longitudinal rebars in LE and LR type slabs become closer for load values

corresponding to the estimated steel yield strength ($2875 \mu\epsilon$). This is an indication of the loose of effectiveness of the experimental design loop joint (LE). The graphs of load versus strain in LE slabs show a clear trend to be represented by a logarithmic equation while the graphs of microstrains in LR type slabs, closely follow the bilinear diagram of steel with tension stiffening effect.

Figure 9. Load vs. Strain in longitudinal bars (a) 20 mm diameter, (b) 25 mm diameter.

The behavior of the transverse reinforcement bars in each slab has been examined, too. Different studies have been reviewed¹³ (see figure 10). These are based on the use of splices by overlapping bars (force transfer between two spliced bars by bond) and they show how orthogonally to the longitudinal overlapping a tensile force is induced whose resultant must be equilibrated by the transmitted force by overlapping. For this reason, the design criteria of the transverse reinforcement derived from this research has been based on: 1) The bottom cross reinforcement ratio must be higher than the longitudinal

reinforcement (overlapping rebars) and should control the width of the transverse crack, 2) The transverse reinforcement acts by means of a dowel action to the relative movement between loop longitudinal reinforcements, and 3) The reinforcement provides internal confinement to concrete at the loop joint.

Table 6 summarizes the theoretical behavior of transverse bottom rebars in LD and LE slabs. It is observed that in all cases the bottom transverse reinforcement capacity is higher than for the longitudinal reinforcement (pair of overlapping rebars), and how the theoretical strain values do not exceeded the yield strength of the material.

Figure 10. (a) Location of strain gauges in specimens LE2 and LD1. (b) Use of splices by overlapping bars. Force transfer between two spliced bars (Adopted from¹³).

Table 6. Theoretical strain at failure in bottom transverse reinforcement in LE and LD slabs.

Type Slab	Long. Rein- forcement	Trans. Rein- forcement	A_s (mm ²)	$A_{s,t}$ (mm ²)	$A_{s,t}/A_s$	U_s (kN)	U_{st} (kN)	Long. Strain ($\mu\epsilon$)	Trans. Strain ($\mu\epsilon$)
LE1	1 ϕ 20	2 ϕ 16	314	402	1.28	180.55	231.15	2875	2246
LD1	1 ϕ 20	6 ϕ 12	314	678	2.16	180.55	389.85	2875	1331
LE2	1 ϕ 25	2 ϕ 16	491	628	1.28	282.33	361.10	2875	2248
LD2	1 ϕ 25	6 ϕ 12	491	1206	2.46	282.33	693.45	2875	1171

The experimental results show that the response of transverse rebars in specimens with loop joints has been different to the theoretical behaviour as can be seen in figure 11. The LE and LD type slabs show a lower strain than expected. This behaviour seems logical because, in LE type slab, there is not enough transference length to mobilize the tensile transverse force by a strut compressed at 45°.

From the results, it is observed how the strain at transverse bars develops mainly when longitudinal bars start yielding (see figure 11). The transverse reinforcement is used as an equilibrium mechanism to the relative movement between loop longitudinal rebars that assures the correct force transfer. The effectiveness of the loop joint has a strong relation with the fact that the reinforcement provides internal confinement to the concrete at the loop joint length without cracking. The confined concrete and the transverse reinforcement provide stiffness to the joint. This stiffness is understood as a relationship between force and displacement. The stress in the transverse rebars remains always lower than the yield limit and the strain in the bars located at the bottom part of the joint is approximately twice the strain of the bars located at the middle height of the joint.

Figure11. Load vs. MicroStrain in transverse bars, (a) 25 mm, (b) 20 mm.

Figure 12 presents the load versus deflection curves at mid-span. As seen, before cracking, the curves exhibit similar slopes regardless of the diameter of the loop rebar. However, after cracking, the slope of the load–deflection curves became steeper as the diameter of the loop rebar increased, and the ultimate load increased. Such a trend appeared identically for the joints type LR and LE, but not for the type LD slab. The decrease in deflection in LD type increases with the diameter of the loop rebars (25 mm). This different behavior is due to a higher stiffness in the joint length at LD type slab with crack widths controlled within the joint. The theoretical simplified analysis in the EHE Code¹² has been also applied, using Branson's equation. This method is based on the curves obtained with the effective moment of inertia, interpolating cracked and uncracked sections. This analysis has been used without concreting joint and without loop joint. The theoretical results are also plotted in figure 12.

From the results, in figure 12 it is observed that the existence of the discontinuity in slabs, induced by the concreting joints, plays a significant role in the early increase of deflection to premature loads. This trend is less significant for load level close to yield stress. Therefore, it is observed how the LD type slabs have more stiffness and their deflection is smaller. This behavior is due to a higher stiffness in the joint of 715 and 900 mm length with double steel ratio. Also, the concreting joints in LD type slabs are located at a longer distance from the acting load.

Figure 12. Load–deflection curves of LE, LD and LR specimens. Comparison with simplified analysis according to Spanish Code EHE¹².

Table 7 presents the concreting joint opening versus loading with different type of joints and rebar diameters. It should be observed that the response of the opening joint (concreting joint) has been considered as a crack width. Also, it should be observed that in this type of structures it is impossible to know where the crack may appear. The average initial crack load has been estimated to be 50.87 kN. From the results, it is observed that LE slabs experiment a larger opening joint at all load levels (see figures 13 and 14). This response and behaviour is due to poor bond because there is not enough transference length. For this reason, the transverse reinforcement develops a dowel action.

Furthermore, the crack widths (opening joint) of specimens are not bigger than the theoretical values obtained from the simplified analysis in the Code. The frequency histogram of crack widths on the surface of the slabs under constant deformation is,

approximately, logarithmic-normal. Therefore, with the simplified theoretical analysis, the maximum crack width cannot be predicted, but is possible to predict a crack width with certain probability of not being exceeded¹⁴. The calculated opening joint for different load levels is shown in table 8.

Table 7. Experimental concreting joint opening for different load levels and different slab types.

Load	LE1 12-07-08	LE1 11-08-08	LD1 07-07-08	LR1 03-07-08	LE2 14-07-08	LE2 24-07-08	LD2 28-08-08	LR2 21-07-08
(kN)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
20	0.019	0.016	0.001	0.024	0.016	0.016	0.009	0.0123
60	0.082	0.096	0.040	0.095	0.067	0.084	0.053	0.057
100	0.175	0.224	0.098	0.131	0.138	0.156	0.101	0.097
140	0.323	0.358	0.168	0.285	0.237	0.234	0.160	0.173
180	0.421	0.478	0.269	0.421	0.332	0.339	0.226	0.254
220	0.714	0.865	0.372	0.595	0.415	0.436	0.311	0.332
260	-	-	-	-	0.506	0.540	0.407	0.417
300	-	-	-	-	0.599	0.661	0.493	0.521
340	-	-	-	-	0.721	0.755	0.588	0.861

Table 8. Theoretical concreting joint opening for different load levels and different slab types.

Load	LE1 12-07-08	LE1 11-08-08	LD1 07-07-08	LR1 03-07-08	LE2 14-07-08	LE2 24-07-08	LD2 28-08-08	LR2 21-07-08
(kN)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
20	0	0	0	0	0	0	0	0
60	0.038	0.106	0.074	0.108	0.078	0.078	0.041	0.078
100	0.274	0.321	0.257	0.322	0.205	0.206	0.157	0.206
140	0.467	0.504	0.417	0.505	0.315	0.315	0.254	0.315
180	0.642	0.673	0.562	0.673	0.417	0.417	0.343	0.417
220					0.515	0.515	0.428	0.516
260					0.610	0.611	0.510	0.611
300					0.705	0.706	0.591	0.706

Figure13. Load– joint opening curves of LE, LD and LR specimens with 20 mm diameter.

Comparison with simplified analysis of width crack according to EHE .

Figure 14. Load– joint opening curves of LE, LD and LR specimens with 25 mm diameter.

Comparison with simplified analysis of width crack according to EHE .

Figures 15 to 20 present the crack pattern. Close to the loop joint, due to the proximity to the load application area, a map of cracks is developed. The cracks grow gradually as the load increases to become failure cracks.

LE and LD type slabs show a similar crack pattern at failure. A failure crack develops around the contour to the loop joint (see figures 15, 16 and 17). This indicates the weakest points of the cross section, located at the end and the beginning of the loop joint, where there is a significant change of strength and stiffness due to the different rebar ratio.

Most of the cracks were vertical flexural cracks. They propagate vertically although these vertical cracks disappear into the compression zone. Finally a horizontal crack in the compressed zone is produced due to an excess of compressive stresses in the concrete, which propagates until it meets the vertical cracks (see figure 18). The crack pattern in LR slabs is more regular and shows a more uniform spacing than for LE and LD slabs (see figures 19 and 20)

Figure15. Crack pattern in LE1_12_07_08

Figure16. Crack pattern in LE2_14_07_08

Figure17. Crack pattern in LD2_28_08_08

Figure18. Crack pattern in LE1_11_08_08

Figure19. Crack pattern in LR1_03_07_08

Figure20. Crack pattern in LR2_21_07_08

5. Conclusions

From the results of the test carried out in the loading up to failure (static load test) of reinforced concrete slabs the following conclusions may be drawn:

1. The LE type slab is a new modified type of structural joint where the force transfer into the loop joint is due to the deviation forces in the hooks, which are equilibrated by

the compression in concrete. The transverse rebar is used to enhance the concrete confinement, equilibrating splitting perpendicular forces to the loops.

2. The member with loop joint LE exhibited a similar serviceability (deflection) and ultimate behavior (failure load) than ordinary LR members without joint in the range of the loop joint lengths (275–300 mm) and loop rebar diameters (20–25 mm) considered in this study. The LE type slab showed an opening joint larger than the other type (LR) due to less tension-stiffening effect and the poor bond due to the lack of transference length. However, the crack widths (opening joint) of specimens are not bigger than the theoretical values predicted by the Codes. Also, from the results obtained, the LE joint type proposed here is validated in the sense that loading tests confirm the correct performance and effectiveness of this loop joint type under static loads.

3. The higher load capacity obtained in LD slabs over LR and LE, is due to a higher stiffness in the joint length with cracking controlled inside it. The increase of stiffness is due to a 720 and 900 mm joint length with double steel ratio.

4. For load values up to 60% of the failure load is observed a lower strain (in the order of 60%) of the longitudinal rebars for LE and LD joints. This is because the steel ratio in the joint length is twice in LE and LD type slab than in LR slab.

5. The existence of the discontinuity in slabs, induced by the concreting joint, plays a significant role in the early increase of deflection to initial loads.

6. The weakest points (those where cracking appears first and failure occurs) in the elements are at the end and the beginning of the loop joint, where there is a significant change of strength and stiffness due to the different reinforcement ratio.

7. Based on the results of the tests carried out, the design criteria for the proposed loop joint for standard qualities of concrete and steel normally used in concrete bridges and normal thickness in the upper flange of box-girder post-tensioned bridges are as follows:

Longitudinal bars:

The total hook anchorage length for diameters 20 and 25 mm is as follows:

$$\phi = 20 \text{ mm} \rightarrow l_u = 275 \text{ mm}$$

$$\phi = 25 \text{ mm} \rightarrow l_u = 300 \text{ mm}$$

These values are much less than those adopted in the case of LD slabs

Transverse bars:

Additionally, several transverse reinforcing bars should be deployed within the joint loop with a total strength capacity equal or higher than the strength capacity of the longitudinal bars. The design criteria of the transverse reinforcement is based on:

- a) The bottom transversal reinforcement capacity must be higher than the one corresponding to the overlapping rebars (longitudinal reinforcement) and should control the width of the transverse crack.

$$U_{s, \text{ bottom transv. bars}} \geq U_{s, \text{ overlapping long. bars}}$$

U is the transversal capacity of the reinforcement, i.e., the total reinforcement area times the yielding strength.

- b) The total transverse reinforcement (bottom, middle, and top cross reinforcement) acts through a dowel action to the relative movement between loop longitudinal rebars and assures the correct force transfer (see figure21).

- c) The total transverse reinforcement provides internal confinement to concrete at the loop joint. This provides stiffness to the joint.

The fatigue behaviour and capacity of the proposed joint in front of cyclic loading has been also verified⁸.

Figure21. Dowel action effect.

Acknowledgements

The authors thank the Spanish Ministry of Education for financial support under research project BIA2010-16332.

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