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Conception and Justification of a New Test Setup for Assessment of the Fatigue Strength of Connections Between Precast Railway Bridge Girders

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Abstract
Important investments have been made worldwide in the modernization and construction of railway lines. Plans for new constructions, have also been laid out. Recently, precast concrete has been widely employed in the construction of railway bridges and viaducts. Several precast solutions have been applied, namely: I-shaped or U-shaped precast girders and uni-cellular or bi-cellular precast box girders. Regarding the structural scheme in the longitudinal direction, either simple span or continuous decks have been used. In this context, this work aims to contribute to the knowledge about the real performance of these structures through the development of a laboratory tests setup, for the study of this type of structure, focusing on the connection between the precast beams. The setup will be implemented at the Laboratory of Earthquake and Structural Engineering (LESE), several experimental campaigns on the cyclic behaviour of reinforced concrete elements have been carried out.

Author Keywords. Experimental Testing, Cyclic Loading, Test Setup, Load Control, Fatigue Analysis, Precast Bridge.

Type: Research Article

1. Introduction

In the study of the behaviour of high-speed railway bridges built with precast beams, fatigue strength is one of the most important design criteria (Sousa et al. 2013). Very few experimental researches can be found in the literature concerning the validation of fatigue behaviour of this type of structure, considering simultaneously the redistribution of stresses due to long-term deformations due to concrete creep and shrinkage.

In bridges with continuous spans, the connection between precast beams, which is made in the vicinity of the beam supports, is a region of strong discontinuities and high mechanical stresses. This is therefore one of the most critical regions which determines the structure's service performance (Sousa 2004).

In this context, the authors have proposed and submitted a research project application, entitled ConPBRail, for funding through the program Portugal 2020 (Compete 2020), in copromotion with a precast concrete company. The main objective of the ConPBRail project consists in the development and validation of a new solution for structural connection between precast beams, that performs well in terms of durability, mechanical resistance, economic efficiency and ease of execution. The work focuses on continuous bridge decks, with a current span ranging from 30 to 35 m, because this is a common span length in the construction on new railway infrastructures, and there is no consensus (in the technical community and infrastructure managers) about the connection type which provides the best long-term performance (Sousa et al. 2012) for the various relevant criteria (durability, structural performance, life-cycle cost).

The research project includes an extensive programme of laboratorial tests, for assessment of the fatigue behaviour of the connection between precast girders in railway bridges. Laboratorial models of the developed connection will be built at a reduced-scale of 1:2.5, which corresponds to a cross-section depth of 1.16 m according to the preliminary design. The precast beam cross-section is U-shaped, and monolithically connected to a top slab to be cast-in-place, giving rise to a unicellular box girder. A maximum of 1 million load cycles will be applied to simulate fatigue effects. No experimental research of this magnitude can be found in the literature. Therefore, it is expected to provide valuable data for calibration of numerical models, and for assessment of the fatigue strength of this type of structure.

This paper presents the first part of the study, and starts with an extensive literature survey about experimental test campaigns to assess the mechanical performance of connections between precast bridge beams. The tested specimen geometry, as well as the loading system and supports, are depicted, so that the main differences between each test campaign can be understood. After that, the model to be tested is described. Then, the description of the experimental setup is made, indicating the main criteria which determines the system design, as well as the definition of the actions to be applied. Finally, the main conclusions and further developments are summarized.

2. State-of-the-Art

In the early 1960s, the Portland Cement Association (PCA) initiated several laboratory and numerical testing campaigns aiming at to understanding the behaviour of continuous prefabricated beam bridges. Kaar, Kriz, and Hognestad (1960) performed experimental tests to assess the strength and ductility of the connection made with ordinary reinforcement, submitted to the action of monotonically increasing bending moments.

Three series of beam tests were carried out. Figure 1 represents the setup of the first and second series. The test consisted of applying a point load to one of the beam extremities, whereas the other extremity is anchored to the laboratory's reaction floor. Figure 2 represents the third series of tests. In this case, the loading was applied to the central region of the beam. The tested structure involves three precast beams segments and two connections. The structure is anchored at the extremities, so that negative bending moments develop at the connection.

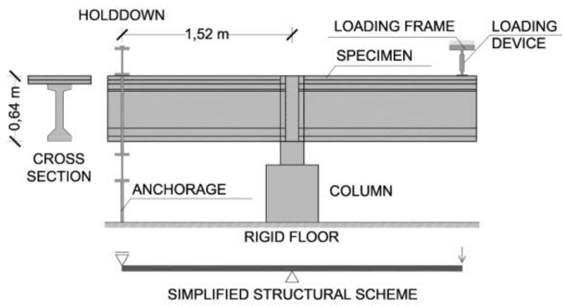


Figure 1: Setup of the first and second series of tests performed by Kaar, Kriz, and Hognestad (1960)

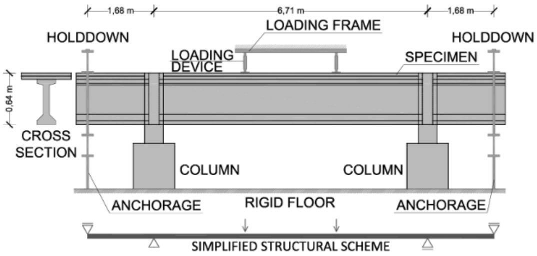


Figure 2: Setup of the third series of tests performed by Kaar, Kriz, and Hognestad (1960)

Hanson (1960) carried out studies to assess the shear strength at the interface between the prefabricated beam and the cast-in-place slab. The test, involving three series, was carried out by applying forces in the central part of the prefabricated beam and slab, as shown in Figure 3. The slip was measured using displacement transducers to record the relative displacement, in the horizontal direction, between the tob fibre of the precast element and the bottom fibre of the cast-in-place slab.

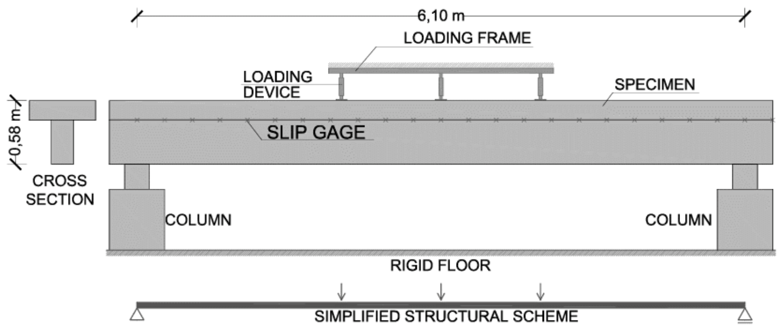


Figure 3: Setup of the tests performed by Hanson (1960)

Also in the PCA's laboratory, cyclic loading tests were carried out to analyse the resistance to the bending moment (Mattock and Kaar 1960), to determine the shear strength (Mattock and Kaar 1961b), to assess creep and shrinkage effects (Mattock 1961), and to analyse the global structural response of a bridge deck whose cross-section is composed my multiple precast beams, using a 1:2 scale model (Mattock and Kaar 1961a).

In the 1970s, under the Missouri Cooperative Highway Research Program, experimental work was carried out using full-scale models. The purpose of the study, carried out at the University of Missouri-Colombia, was to analyse the connection, realized by the continuity of the reinforcements, to resist negative bending moments (Salmons 1974).

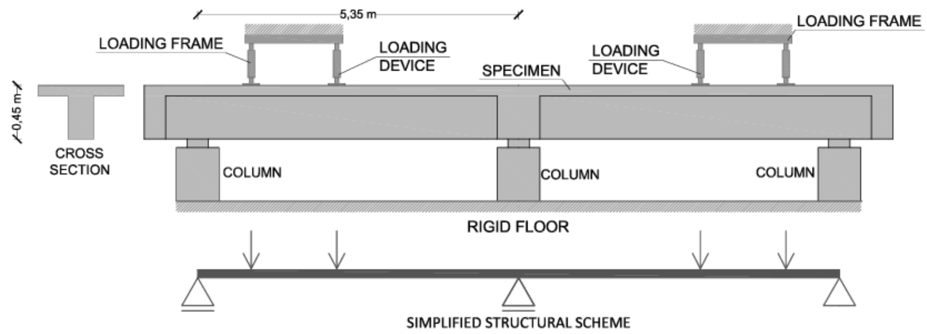


Figure 4: Setup of the tests performed by Salmons (1974)

Figure 4 shows the adopted test setup. The tests were carried out in three phases. The first phase consisted in the application of cyclic loading until cracking was reached. In the second phase, cyclic loads were applied in order to assess the increase of deformations. In the third and final phase, an increasing load was applied until rupture.

In the early 1990s, Abdalla, Ramirez, and Lee (1993), at the Purdue University, tested connections in which the continuity is performed through prestressing strands. The objective of this work was to assess the long-term behaviour and the shear strength.

In France, in 1998, experimental fatigue tests were required before a prefabricated solution, with continuous girders, was approved for the construction of bridge decks in a high speed railway line (Chefdebien 1998). A sinusoidal cyclic loading was applied, with the load positions shown in Figure 5. The loading frequency was equal to 8 Hz, and the minimum and maximum load values in each cycle were 60 and 250 kN. The real bridge span was equal to 22.5 m, whereas the span in the experimental tests was 5.35, which corresponds to a reduced-scale of 1:4.2. "I" shaped girders were adopted, and the connection was made with ordinary reinforcement only. To the authors' knowledge, this is the only experimental research which has been made in the past to analyse the fatigue behaviour of continuous railway bridge decks made with precast concrete beams. The structures analysed by Chefdebien (1998) are therefore significantly different from the ones envisaged in the ConPBrail project: bridges built with "U" shaped, pretensioned, precast concrete beams, for bridge spans in the range of 30 to 35 m.

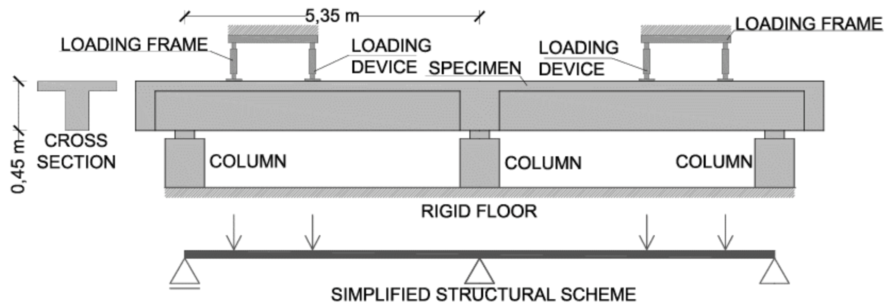


Figure 5: Schematic representation of the tests performed by Chefdebien (1998)

In 2003, under the National Cooperative Highway Research Program, tests were carried out to study the influence of the positive bending moments on the connection (Miller et al. 2003). The test setup, developed for full-scale models, consists of applying a cyclic load to the extremity of the tested structure. This model comprehends two precast beam segments and their connection at the position of the support on the bridge piers. The laboratorial model length corresponds to the extension between points of null bending moments in a continuous bridge. The supports and the loading system were designed so that load reversals could be considered, as shown in Figure 6. Thus, both positive and negative bending moment effects were assessed.

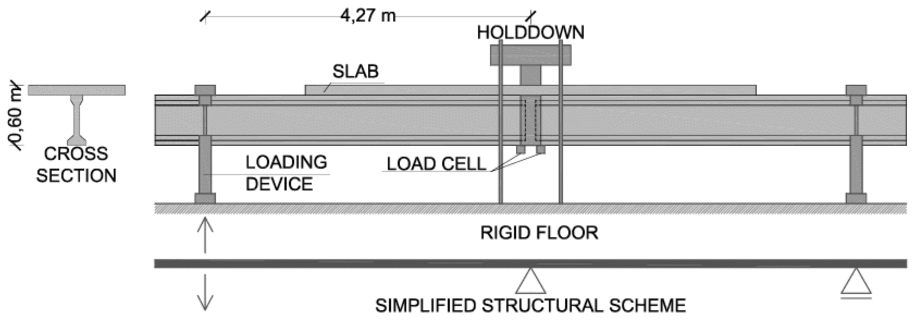


Figure 6: Setup of the tests performed by Miller et al. (2003)

3. Proposed Test Setup

3.1. Design of the full-scale structure

The design process starts with the definition of the bridge deck for the full-scale structure. An interior span of 35 m is selected, because it corresponds to the upper limit of the range under analysis. The generic shape of the adopted cross-section consists in two "U" shaped precast girders, connected by a solid deck slab, cast-in-place with resource to 7 cm thick precast planks. This is typology has been considered adequate for bridges carrying two railway tracks (Sousa 2012) which the most frequent bridge layout. The "U" shaped precast element forms, in combination with the cast-in-place deck, a cell structure which provides higher torsional stiffness than "I" shaped beams, with benefits for railway bridges.

The structure design has to comply with the requirements of the Eurocodes regarding loading (CEN 2003), design of reinforced and prestressed concrete (CEN 2004), specific design rules for concrete bridges, including rules for fatigue design (CEN 2005b) and design criteria concerning the safety of the railway track (CEN 2005a).

The redistribution of stresses and internal efforts caused by concrete creep and shrinkage deformations, in this type of phased construction, also needs to be taken into account. This procedure provides the bending moment due to quasi-permanent actions applied to the connection, and its evolution in the course of time. This is time evolution has to be considered for the quantification of maximum bending moments in the connection, due to the combined action of quasi-permanent and variable actions. The latter consist in the effects of railway vehicles crossing the bridge and daily temperature variations.

The calculation of stress and effort redistributions can be efficiently made by applying the age-adjusted effective modulus method (Bažant 1972), as explained by Sousa (2004).

The maximum negative and positive bending moments at the connection (sum of design action effects, including partial safety factors) were estimated as M_{Sd} = 21180 kNm and +1670 kNm, respectively. The range of variation of bending moments at the connection, due to the most severe traffic load action (characteristic value) was estimated as -8000 kNm to +1500 kNm (maximum negative and positive moments, respectively). On the other hand, the maximum negative bending moment, not including partial safety factors, was estimated as 15100 kNm. It was found that a 2.6 m high precast "U" shaped beam, complemented by a 0.30 m thick cast-in-place deck slab, fulfils the requirements for this structure. The presentation of the complete calculation process and structure geometry is out of the scope of this paper.

3.2. Design of the reduced-scale model

Fatigue tests are to be performed, in the scope of the ConPBRail project, in the LESE laboratory of the University of Porto. The main objective of this experimental campaign is the assessment of the cyclic behaviour of the connection between girders in the previously design bridge decks with spans of 25 m. For that reason, the laboratorial model to be tested is limited to the zone in the vicinity of one interior support. It comprehends two beam segments and the corresponding connection, and the model length is equal to the distance between consecutive points where the bending moment due to quasi permanent loads is equal to zero. This length can be estimated as $\approx 2\ L_s/5$, where L_s is the span length. Even though the real structure's cross-section is composed by two girders, the cross-section for the laboratorial test can be formed by a single girder, because the two girders exhibit similar structural behaviour.

After pondering about the maximum model size that could be tested in the laboratory, as well as the corresponding requirements in terms of load application (justified below in section 3.5) it was found that the reduced-scale of 1:2.5 would provide a good compromise between a large specimen size to reproduce the connection structural behaviour and a not too large size so that the laboratorial fatigue testing could be feasible. Figure 7 depicts the laboratorial model's cross section.

The geometric similitude relationship between the real-scale prototype and the laboratorial model simply consist in scaling all the geometric dimensions using the same factor 1/2.5. Therefore, the model length, measured between the axes of the extremity cross-sections where loading and supporting conditions are imposed, is equal to 35 m / 2.5 / 5 x 2 = 5.6 m.

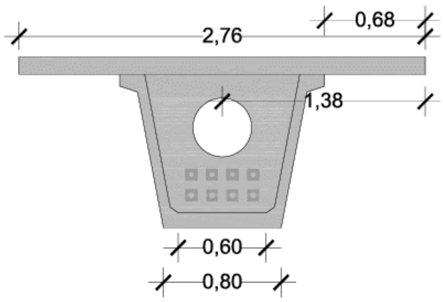


Figure 7: Cross-section of the reduced-scale laboratorial model

3.3. Determination of the force to be applied

The relationship between the magnitude of each variable in the reduced-scale model and in the real-scale prototype has to be determined by applying the Cauchy similitude relationships, as explained by Guedes (1997). For the most relevant variables in this test campaign, such relationships are shown in Table 1. Therefore, the maximum negative bending moment in the real-scale prototype of $M_{Sd,P}$ = -21180 kNm (see section 3.1) corresponds to a bending moment in the model of $M_{Sd,M}$ = -21180 / 2.5^3 = 1356 kNm, assuming, as a simplification, that the uniformly-distributed-loading scenario is applicable for the total bending moment. The flexural reinforcement in the model is therefore designed taking into account this bending moment value, $M_{Sd,M}$.

	Label	Relationship
Length	L	$L_P = 2.5\ L_M$
Area	A	$A = 2.5^2\ A$
Volume	V	$V_P = 2.5^3\ V_M$
Moment due to uniformly distributed loading	M	$M_P = 2.5^3\ M_M$
Strain	ε	$\varepsilon_P = \varepsilon_M$
Stress	σ	$\sigma_P = \sigma_M$

Table 1: Cauchy similitude relationships between the real-scale prototype (P) and the reduced-scale model (M)

The actual bending strength of the model for monotonically increasing loading, $M_{R,M}$, will be higher than $M_{Sd,M}$. It has to be estimated for definition of the requirements for the hydraulic actuator and auxiliary reaction structure. It can be estimated as:

$$M_{R,M} = \gamma_s \frac{f_{tm}}{f_{yk}} M_{Sd,M} \tag{1}$$

where the partial safety factor for the steel strength, γ_s, is 1.15 (CEN 2004) and the ratio between the mean tensile strength and the characteristic yield strength, f_{tm}/f_{yk}, can be estimated as 1.30 (Fédération Internationale du Béton 2013). It gives $M_{R,M}$ = 2027 kNm.

The fatigue loading procedure has to be determined so that the accumulated fatigue damage to be induced in the model is equivalent to the actual damage in a real structure at the end of its service life (100 years for the type of structure under analysis). According to current European design codes (CEN 2003; 2004; 2005b), the damage equivalent stress method can be used to define the constant amplitude cyclic loading which, for a number of load cycles equal to 10^6, gives rise to the same accumulates fatigue damage as the real loading in the course of the structure service life. The damage equivalent stress method is based on the hypothesis of linear damage accumulation of Palmgreen-Miner (Miner 1945), and uses a correction factor, $\lambda_s = \lambda_{s,1} \cdot \lambda_{s,2} \cdot \lambda_{s,3} \cdot \lambda_{s,4}$, to calculate the damage equivalent stress range, $\Delta\sigma_{s,equ}$, based on the stress range due to the envelope of traffic loads, $\Delta\sigma_{s,71}$, and the dynamic factor Φ:

$$\Delta\sigma_{s,equ} = \lambda_s\ \Phi\ \Delta\sigma_{s,71} \tag{2}$$

For the structure under analysis, the design codes formulae (CEN 2003; 2005b) yield λ_s = 0.85 and Φ = 1.02.

The maximum and minimum negative bending moments in a cycle, in the reduced-scale model, for a number of applied load cycles N^* = 10^6, is thus:

$$M_{max} = \frac{-15100}{2.5^3} = 966\ \text{kNm} \tag{3}$$

$$M_{min} = M_{max} - \lambda_s\ \Phi\ \frac{8000 + 1500}{2.5^3} = 438\ \text{kNm} \tag{4}$$

where the bending moment values in the real-scale prototype (-15100, 8000 and 1500) are the values quantified in section 3.1 and 2.5^3 is the similitude relationship factor for bending moments.

Based on the laboratorial facilities conditions, the frequency of load application was set as 1Hz, which gives a total test duration, for the 10^6 load cycles, of 11.6 days.

Adequate fatigue strength of the real structure is to be expected if no steel bar in the model fails due to fatigue at the end of the test. Besides that, the fatigue experiment provides relevant data for characterization of the long-time increase of deformations and crack openings due to cyclic loading. Moreover, though the experimental analysis it is possible to assess the occurrence of additional failure modes due to stress concentrations in the connection region, as well as shear effects at the interfaces between concretes cast at different times.

The steel fatigue life obtained in the cyclic tests of real scale models has to be compared with the analytical fatigue assessment, also based on the damage equivalent stress method (CEN 2004):

$$\gamma_{F,fat}\,\Delta\sigma_{s,equ}(N^{*}) \leq \frac{\Delta\sigma_{Rsk}(N^{*})}{\gamma_{s,fat}} \tag{5}$$

where $\gamma_{F,fat}$ is the partial safety factor for fatigue actions, equal to 1.0, $\gamma_{s,fat}$ is the partial safety factor for fatigue strength, equal to 1.15, and $\Delta\sigma_{Rsk}(N^{*})$ is the characteristic resistant stress range for N^{*} = 10^6 load cycles. Even though the design codes provide indicative values for $\Delta\sigma_{Rsk}(N^{*})$, a more accurate assessment can be performed if the S-N curve for employed steel bars is determined experimentally, this being the adopted approach for the ConPBRail project.

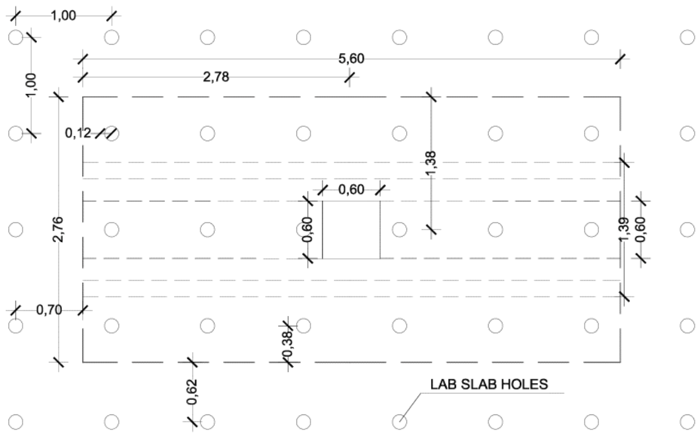
3.4. Auxiliary reaction structure

To perform the fatigue test of the connection between precast segments, it is necessary to apply negative bending moments, with a sinusoidal cyclic variation. To satisfy this condition, a dedicated test setup was conceived. It includes a servo-controlled hydraulic actuator for load application in the cross-section corresponding to the support of the real structure, and the complementary hold-down structure which restrains the structure at the extremity sections (see Figure 8). In the latter positions, the laboratorial model is fixed to the 60 cm thick, solid, reaction laboratory slab. In these conditions, the model behaves according to a three-point-bending structural scheme. The relationship between the maximum bending moment in the connection, M, and the actuator force, F, is

$$F = 2\,M/L \tag{6}$$

where L is the effective length between the axes of the hold-down structure and the actuator, equal to 2.5 m in this case. The auxiliary hold-down structure has to be designed to perform as a reaction system to the force applied to the connection. This structure must have sufficient stiffness to absorb the reactions caused by the load application with minimal deformation.

Bearing in mind the requirements for the reaction structure, two possible test setups were considered for use in the laboratory tests. Figure 8 shows hypothesis 1. The actuator should be on top of the slab´s support column. The hold-down auxiliary structure at the extremity cross-sections is based on high-strength steel ties which have to be positioned so that they cross the reaction slab in one of the existent holes for that purpose. The holes form a 1 m x 1 m grid, as shown in Figure8a. The reaction system consists of steel beams connected to the hold-down ties. The ties can be post-tensioned in order to eliminate any slack in the reaction system, and also to induce a an initial state of stress in the test system. With this test system, the model position can be adjusted in the vertical direction after it is assembled.



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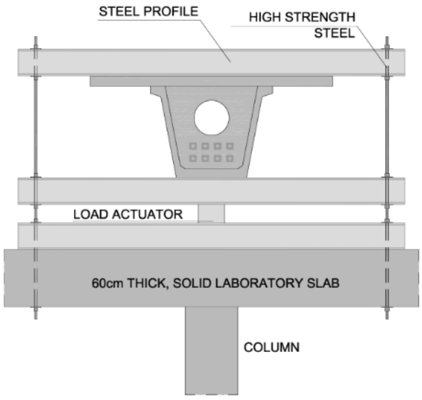

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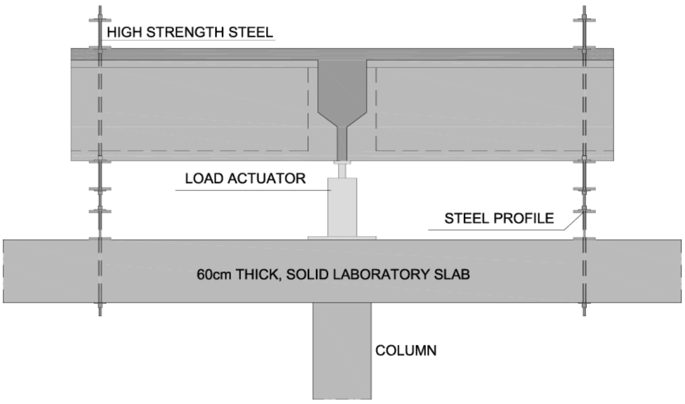
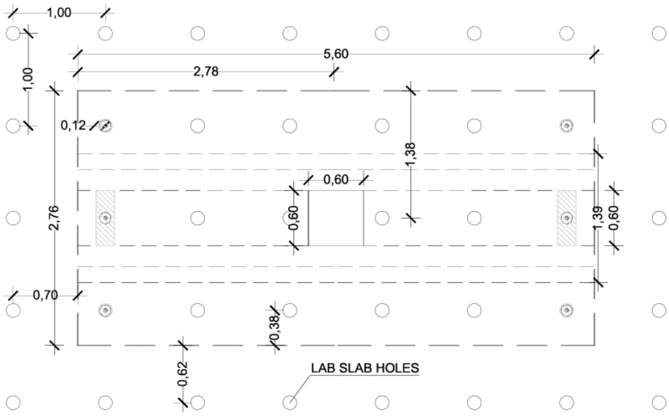


c)

Figure 8: Hypothesis 1 for the test setup: a) top view; b) front view; c) lateral view

Hypothesis 2 for the test setup is shown in Figure 9. Unlike the first one, this hypothesis uses reinforced concrete elements instead of steel profiles. These elements are provided with ducts, so that high-strength steel ties can be introduced and fix the reaction system to the 60 cm thick slab. The reaction structure is cast at the time of concreting the slab. Thus, the reaction structure and the model form a single piece, resulting in a higher stiffness for the reaction system. This is an advantage over hypothesis 1, but it has the disadvantage that the vertical position cannot be regulated after assembling the structure to be tested.



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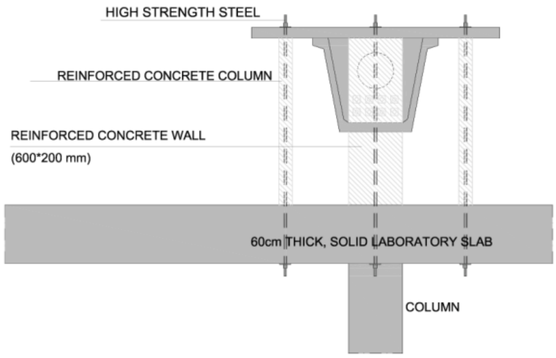

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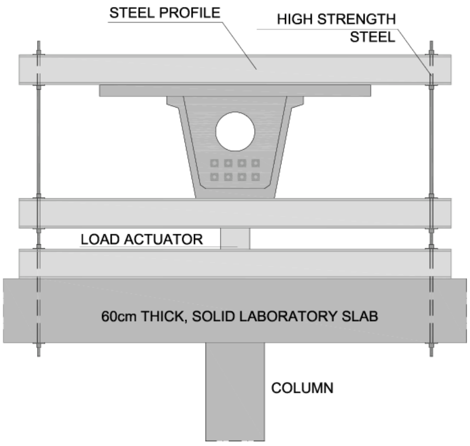

c)

Figure 9: Hypothesis 2 for the test setup: a) top view; b) front view; c) lateral view

3.5. Acquisition system and load control

A National Instruments acquisition and control system is available in LESE, which was developed with the main purpose of conducting cyclic tests of reinforced concrete columns, under plane or bi-axial bending (Arêde et al. 2017). The entire system is controlled by software developed in Labview, whether for acquisition of monitoring results, as for control of load application systems, in interaction with two different PXI systems. The data acquisition system allows direct reading of strain gauges, load cells, linear variable differential transformers, as well as other types of analogue devices or digital sensors. The quantities required to be measured in this test: are displacements in the beams and slab; strains in the steel bars used in the connection, strains in the reaction system, strains in the slab reinforcement and the applied force. The existing system will have to be adapted to the requirements of the new tests to be carried out. New subroutines will have to be implemented for Labview. Figure 10 presents a schematic layout of the mentioned systems, with a description of the information direction.

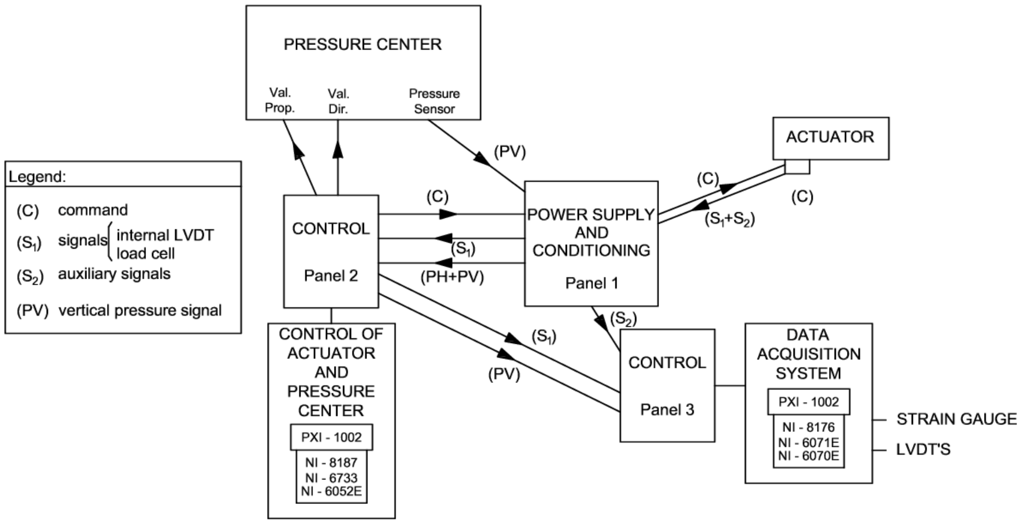

Figure 10: Schematic layout of the load and acquisition system

The nature of the tests to be carried out is different from those carried out in previous campaigns (Delgado et al. 2009; Delgado et al. 2011; 2015; 2014; Arêde et al. 2017). The load is to be applied cyclically, with a frequency of 1 Hz. The energetic group which will provide the hydraulic power to the test system has to be designed for the maximum and minimum force values in a cycle of 773 and 350 kN respectively (based on Formulas 3, 4 and 6), with a maximum displacement range estimated as 5 mm. The energetic group has to be capable of continuous operation for a minimum period of 12 days. These requirements determine that the hydraulic power has to be provided by an electric engine of regulated speed and hydraulic system of variable cylinder capacity, providing a nominal installed power of 22 kW.

4. Conclusions

The purpose of this paper is to present the first stage of the experimental campaign to be carried out within the scope of the ConPBRail project. In this stage, the overall conception of the prototype and test setup is made. In the project, fatigue tests will be made, in the LESE laboratory of the University of Porto.

The state-of-the-art for laboratorial tests of the connection between precast bridge girders was presenting, highlighting the geometric layout adopted for the various tests, the load application systems, the analyses cross-sections and the structural scheme adopted in the tests. It was found that the experimental assessment of the fatigue strength of continuity connections for precast railway bridge girders is very scarce.

The proposed test setup was described, starting with the justification of the most relevant design variables for the design of the rea-scale prototype, considered as a reference for the laboratorial tests. Then, the model design was justified presented, based on the Cauchy similitude relationships for definition of the model geometry and loading. The procedure for qualification of the fatigue loading to be applied in laboratory was presented, with resource to the damage equivalent stress method. Finally, the laboratorial test setup was described.

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