Experimental investigation on the seismic behaviour of connections in precast structures

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ABSTRACT: The paper is addressed to the identification of the behaviour parameters of a type of connection between floor or roof elements and the supporting beams. This identification will allow to give a correct representation of this type of joints within the overall numerical model for structural analysis and to give practical application to the criteria of capacity design. The first results of the experimental campaign are reported with reference to monotonic (push-over) and cyclic tests. Following the results of some preliminary trial tests, a complete framing of the experimental programme has been defined, covering the general testing protocol for all types of connections. Moreover, the guidelines have been drafted for a standardized quantification of the principal parameters of the connections seismic behaviour.

1 INTRODUCTION

The present work has been developed within the scope of Reluis research project “Evaluation and reduction of seismic vulnerability of existing buildings”, with the contribution to the experimental programme of the Italian association of precast industry Assobeton. It deals with a key aspect which conditions the response of precast industrial buildings, that is the behaviour of connections under seismic actions. Very few papers are available in the literature, mainly devoted to the presentation of some specific patented devices, but a general exhaustive knowledge on the matter is still missing.

Only the connections belonging to frame systems are considered herein, neglecting those of wall panel and cell systems. Nonetheless, this choice covers in any case the great majority of existing precast structures. Moreover, only typical joining systems are treated, that is dry joints made with steel connectors generally composed of angles, plates, channel bars, anchors, fasteners, bolts, dowels, bars. Hence, Emulative joining systems consisting of wet joints with bar splices and cast-in-situ concrete are not considered. Also simple bearings working by gravity load friction are not considered, like sliding and deformable supporting devices, being all these types of connections not suitable for the transmission of seismic actions.

On the basis of the position in the overall construction and of the consequent different structural functions, the following five classes of joinings are distinguished:

1 – mutual joinings between floor (or roof) elements that, in the seismic behaviour of the structural system, concern the diaphragm action of the floor;
2 – joinings between floor elements and supporting beams that give the peripheral constraints to the floor diaphragm in its seismic behaviour;
3 – joinings between beam and column that shall ensure a hinged behaviour in the vertical plane of the beam and a full off-plane constraint;
4 – joinings between column segments or between column and foundation, made of bond anchorage of protruding bars in sleeves filled with in-situ mortar (or other devices);
5 – fastenings of cladding panels to the structure that shall ensure the stability of the panels but also permit the large drifts expected under seismic action.

2 BEHAVIOUR PARAMETERS

A connection is composed by three parts: two lateral parts A and C corresponding to the local regions of the adjacent elements close to the connector; a central part B constituted by the connector itself with its steel components (see Figure 1).

The principal parameters which characterize the seismic behaviour of the connection refer to the six properties of:

– strength: maximum value of the force which can be transferred between the parts;
– ductility: ultimate plastic deformation compared to the yielding limit;
– dissipation: specific energy dissipated through the load cycles;
Figure 1. Scheme of connection.

- **deformation**: ultimate deformation at failure limit;
- **decay**: strength loss through the load cycle compared to force level;
- **damage**: residual deformation at unloading compared to the maximum displacement.

The following points will specify the standard modes of calculation of these parameters on the basis of the experimental results.

In general the parts A and C have a non-ductile non-dissipative behaviour characterized by a brittle failure, with small displacements, due to tensile failure of concrete. A ductile dissipative behaviour of the connection can be provided by the steel connector B, if correctly shaped for a failure mode which involves flexural or tension-compression modes and not shear modes.

For a ductile connection, in addition to a ductile connector, its under-proportioning with respect to the lateral parts is necessary, with the criteria of capacity design. Also the geometric compatibility of deformations shall be checked.

Non ductile connections shall be opportunely over-proportioned by capacity design with respect to the capacity of the critical dissipative regions of the structure.

### 3 IDENTIFICATION TESTS

Generally speaking, four levels of tests are scheduled, addressed to different identification purposes as specified below:

- **particular tests** referred to the qualification of single connectors inserted between two over dimensioned blocks and subjected to the principal action expected in the structural system;
- **local tests** referred to the qualification of the connection included between two significant portions of the elements, representing the structural arrangement and subjected to the relevant components of the action;
- **tests on subassemblies** referred to groups of connections inserted in structural parts representing the current construction framing and subjected to its specific actions;
- **tests on assemblies** referred to the connection system of a complete structure subjected to the typical seismic actions.

In the present work the tests on subassemblies and assemblies are non included. In so far as particular and local tests, both monotonic loading and cyclic loading are included, specifically framed up for the joining under consideration.

#### 3.1 Monotonic behaviour

In general from push-over tests diagrams force-displacement $f-d$ such as those of Figures 2a-b-c can be deduced. These diagrams qualify the behaviour of the connector or of the connection on the base of the following definitions.

Figure 2a represents a ductile behaviour characterized by a relevant plastic deformation after the elastic phase. In particular the curve i represents a ductile hardening behaviour, the curve s represents a ductile stable behaviour, the curve d represents a ductile softening behaviour. The significant points of the diagrams are: the yielding limit $d_y$, the ultimate...
limit \( d_u - f_u \). One can add, if preceding the ultimate one, the serviceability limit \( d_a - f_a \) corresponding to the allowable deformation for the functioning of the joining.

Figure 2b represents a *brittle behaviour* without plastic deformation and with a failure before the serviceability limit. The reference point corresponds to the ultimate limit \( d_u - f_u \).

Finally, Figure 2c represents an *over-resisting behaviour* with the experimental plotting stopped after the serviceability limit but before the yielding or ultimate limit. The reference points are the serviceability limit \( d_a - f_a \) and the test limit \( d_t - f_t \).

The ductility deduced from the experimental behaviour is given mainly by the plastic resources of the steel connector with prevalent flexural deformations. Non linear effects may originate also from other phenomena like the friction, the material damaging and the geometrical changes due to the large deformations of the connector.

The standard test includes an initial cycle taken up to the serviceability limit \( d_a - f_a \), with unloading for the determination of the residual displacement \( d_r \) (see Figure 3). The final loading will follow, unless obviously an early failure occurred.

In addition to a first quantification of the constitutive parameters, the push-over test is performed also as preliminary to define the loading steps of the subsequent cyclic test.

### 3.2 Cyclic behaviour

The experimental cyclic response is obtained by applying the load history described in Figure 4, where groups of three cycles of the same amplitude are performed step by step with subsequent increments \( \Delta d \) up to the ultimate or test limit. The amplitude \( d_1 \) of the first initial group is taken as 1/4 of the lesser between \( d_u, d_a, d_t \) and \( d_r \). The amplitude increments \( \Delta d \) of the subsequent groups of cycles are taken equal to \( d_1 \). In these definitions the values are those obtained from the push over test performed on a similar prototype. The incremental loading process can be taken up to failure.

From the cyclic test one obtains diagrams force-displacement \( f - d \) like that of Figure 5. They qualify the behaviour of the connector or of the connection on the base of the following definitions.

For non perfectly elastic behaviour, from the \( f - d \) diagram the histogram of dissipated energy \( U_i \) is calculated as the area of the corresponding \( i \)-th branch of the \( f - d \) diagram (see Figure 6a). The same histogram is converted in dimensionless form (see Figure 6b) dividing any area by the one corresponding to the perfect elastic-plastic half-cycle (see Figure 7):

\[
U_i = \frac{U_i}{U_{oi}}
\]

where

\[
U_{oi} = d_{pi} f_{max}
\]

with \( d_{pi} = d_i - d_{ci} \)

and where \( d_{ci} \) is calculated on the base of the inclination \( k_1 = f_1/d_1 \) of the initial branch of \( f - d \) diagram.

\[
d_{ci} = \frac{f_{max}}{k_1}
\]

### 4 DESCRIPTION OF THE PROTOTYPE

Tests have been performed at the Laboratory of the Department of Structural Engineering of the Politecnico di Milano. Figure 8 shows the testing setup as positioned on the triangular mesh of the anchoring floor system. The specimen consists of a 1.6 m long segment of a ribbed element of ordinary production.
Its section is shown in Figure 9. This specimen is supported by two transverse beams fixed to the floor. The four bearings are provided with low-friction “Teflon” pads but only one is equipped with the connection devices subjected in the longitudinal direction to the push-over test first and to the cyclic test after. The other bearings are provided with sliding lines to prevent transverse movements.

The thrust is applied by means of an hydraulic jack under displacement control with a maximum force capacity of 660 kN in compression and 460 kN in tension (see Figure 10a). This force is applied to the web of the specimen just over the experimented connection with the pression of two opposite steel plates tied to each other with four external bars. This system (see Figure 10b) avoids tension stresses in concrete preventing the possible end spalling of the loaded web.

The connection is made with a couple of steel angles fixed to the beam with fasteners φ16/25 and to the web with a passing dowel M16 tightened through the washers against the web (see Figure 11).

For this Class 2 (roof-to-beam) connection a serviceability displacement of \( d_s = \pm 24 \text{ mm} \) has been assumed, being this the limit against the loss of bearing in one direction and allowance closure in the other direction.
5 RESULTS

Following the provisions of the standard testing protocol described in Clause 3, a monotonic (push-over) test has been performed first, followed by a cyclic test with new steel angles.

5.1 Monotonic test

In Figure 12 the results of the monotonic test are represented in terms of force-displacement diagram. With respect to the standard protocol provisions, an initial reverse minor cycle has been added to compensate possible unbalances of the loading system.

From the first cycle, taken up to the serviceability limit $d_s = 24\, \text{mm}$, a residual displacement $d_r = 12\, \text{mm}$ has been measured with a high ratio: $d_r/d_s = 0.5$.

which indicates large inelastic effects due to friction slidings between steel connectors and concrete adjacent parts.

The loading has been subsequently resumed and taken up to the test limit $d_t = 40\, \text{mm}$ over which the system could seize. This limit is in any case much larger of any possible allowable displacement for the bearing preservation. An over-resisting non ductile behaviour turns out.

5.2 Cyclic test

For the cyclic test the loading history of Figure 4 has been applied with increments $\Delta d = 3\, \text{mm}$ of the imposed displacement amplitudes. The force-displacement diagram of Figure 13 has been obtained.

First of all the relevant non symmetric behaviour can be observed. This is due to the testing arrangement which has some uneven effects for large displacement. The eccentricity of the applied force with respect to
the connection induces a longitudinal moment which push down one side and lift up the other side of the supported element. The up-lifting force caused damages to one sliding line (see figure 14) which led to a friction/interlock reaction.

The envelope curve in the positive high friction direction of the diagram shows a kind of hardening trend. This is not due to the intrinsic properties of the materials, but mainly to the quoted uneven effects of friction/interlock reaction.

The test has been stopped at ±40 mm cycles amplitude with the following test limits:

\[ d_i = + 40 \text{ mm} \quad f_i = + 53 \text{ kN} \]
\[ d_i = - 40 \text{ mm} \quad f_i = - 22 \text{ kN} \]

The values read at the serviceability limit are summarized in Table 1. In particular the inclination of the initial branch is deduced from

\[ d_i = 0.434 \text{ mm} \quad f_i = 1.368 \text{ kN} \]
\[ k_i = f_i/d_i = 3.113 \text{ kN/mm} \]

Figure 13 shows the local damages on the steel devices of the sliding line. Fig. 14 shows the deformation of the steel angles of the fixed connection. In the next course of the research the testing arrangement will be modified to fit more closely with the actual functioning of the in-situ structure so to obtain cleaner data for practical applications.

6 CONCLUSIONS

These first experience on a standardized experimental definition of the behaviour parameters of the connections in precast structures give in some way confirmation to the intuitive assumptions made in seismic design. At least for the experimented type of connection, the connecting devices themselves, dimensioned more for the geometrical sizes of the elements than for the strength requirements, are largely over-resisting. For strength verification the attention should be moved to the adjacent concrete parts and to their specific reinforcement, which is a wider problem to be faced. A brittle behaviour of these parts can be expected, turning again the attention to the capacity design approach by which a reliable quantification of the over-dimensioning of connections can be made.

In any case the present research provides the basic data to which any kind of structural analysis shall refer. This approach shall be extended to all the principal types of connections used in precast structures.

REFERENCES


