ANNEX B (INFORMATIVE)

SEISMIC DESIGN OF PRECAST CONCRETE STRUCTURES

Editorial Note: The material of this annex shall be incorporated into the text of the next draft.

B1 GENERAL

B1.1 Scope

(1) This annex deals with the aseismic design of concrete structures constructed partly or entirely of precast elements.

(2) Unless otherwise specified, the provisions of section 2, of this Part and of part 1-3 of Eurocode 2 apply.

B1.2 Identification of structural types

(1) The following structural types, as described in section 2, are covered by this annex:

- frame systems
- wall systems
- dual systems (mixed precast frames and precast or monolithic walls).

(2) In addition to these systems, also covered are:

- cell structures (precast monolithic room cell systems),
- pendulum systems

  Note: Single storey industrial buildings with doubly hinged beams should be distinguished from the normal frame system.

B1.3 Evaluation of precast structures

(1) In modelling of precast structures, the following evaluations should be made:

a) Identification of the different roles of the structural elements:

- resisting only gravity loads, e.g. hinged columns around a reinforced concrete core,
- resisting both gravity and seismic loads, e.g. frames or walls,
- providing adequate connection between structural elements, e.g. floor or roof diaphragms.

b) Ability to fulfil the seismic resistance provisions of section 2:

- precast system able to satisfy all those provisions,
- precast system which deviate from those provisions and, by way of consequence, need additional design criteria and are assigned lower behaviour factors.

c) Identification of non-structural elements, which may be:

completely uncoupled from the structure,

- partially resisting the deformation of structural elements.

d) Identification of the effect of the connections on the energy dissipation capacity of the structure:

- connections located well outside critical regions (as defined in clause 2.1.2), not affecting the energy dissipation capacity of the structure (see B2.1.1 and e.g. fig. B1.a),

- connections located within critical regions but adequately overdesigned, so that inelastic behaviour is shifted to areas away from connections (see B2.1.2 and e.g. fig. B1b),

- connections located within critical regions which should exhibit substantial ductility (see B21.3 and E.g. fig. B1.c).

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Figure B1:a) connection located outside critical regions,

b) overdesigned connection with plastic hinges shifted outside the connection,

c) ductile shear connections of large panels located within critical regions.

B1.4 Design criteria

B1.4.1 Local resistance

(1) In precast elements and their connections, response degradation due to cyclic post-yield deformations should be taken into account. Therefore, at variance with the case of monolithic (cast-in-place) structures, the design resistance of precast connections under monotonic loading should not be taken as the resistance under seismic actions.

B1.4.2 Energy dissipation

(1) Ductility classes “M” and “L" will be normally considered in precast structures. However, ductility class “H" may also be used, provided an additional special study is undertaken.

(2) In addition to the plastic rotational capacity of critical regions, energy dissipation in precast structures may also be effected by means of post-yield shear displacements along joints, provided that:

a) their force response does not degrade substantially within the considered duration of the action,

and

b) possible instabilities are appropriately avoided.

Note: This shear capacity may be considered, especially in precast wall systems, by taking into account the values of the local slip-ductility factors μ_s on the choice of the overall behaviour factor q.

B1.4.3 Specific additional measures

(1) Only regular precast structures are considered (see clause 2.2 of Part 1-2). The interruption of vertical elements at any floor is not allowed.

(2) Uncertainties related to resistances are covered as in clause 2.4.7.(2). Additionally, with regard to the connections of linear precast elements, the resistances in either of the two orthogonal axes should not be smaller than 25% of the other.

(3) Uncertainties related to ductilities are covered as in clause 2.4.7.(3).

B1.5 Behaviour factors

(1) For precast-structures observing the design provisions of this annex, the value of the behaviour factor q_p may be derived as follows, unless special studies allow for deviations:

$$q_p = k_p \cdot q \tag{B1}$$

where

q behaviour factor derived according to exp.(2.2),

kp reduction factor depending on the energy dissipation capacity of the precast structure (see paragraph (2) below).

(2) The reduction factor k_p may be taken as follows:

$$k_p \begin{cases} 1{,}00 & \text{for structures with connctions located away from critical regions, see B2.1.1,} \\ 0{,}75 & \text{for structures with overdesigned or energy dissipating connections, see B2.1.2 and B2.1.3.} \end{cases} \tag{B2}$$

(3) The behaviour factors specified in table 2.2 for coupled walls, should not be used in the case of structural walls composed of precast large panels, unless special concept and design allows for adequate energy dissipation in the coupling beams.

(4) For precast structures not observing the design provisions of this annex the behaviour factor qp should be taken equal to 1,0.

B1.6 Analysis of transient situation

(1) During the erection of the structure, which should include temporary bracing, seismic actions do not have to be considered as a design situation. However, whenever the occurrence of an earthquake might produce the global collapse of parts of the structure with serious risk to human life, temporary bracings should be explicitly designed for an appropriately reduced seismic action.

(2) In the absence of special studies, this action may be taken equal to 30% of the design action as defined in clause 4.2 of Part 1-1.

B2 CONNECTIONS OF PRECAST ELEMENTS

B2.1 General provisions

B2.1.1 Connections located away from critical regions

(1) Such connections should be located at a distance from the end-face of the closes; critical region being larger than the largest of the cross-section dimensions of the connected elements.

(2) Their design action-effects should be further factored by 1,1 in order to cover uncertainties related to the analysis of precast elements.

B2.1.2 Overdesigned connections

(1) The design action-effects of such connections should be factored by 2,0 in ductility classes "H" and "M" and by 1,5 in ductility class "L".

(2) The same rule applies for the connected elements themselves over a length equal to 1,5· l_{cr}, where l_{cr} l_{cr} denotes the length of the critical region.

(3) The consequences of those provisions on the capacity design should be taken into account.

B2.1.3 Energy dissipating connections

(1) Such connections should comply with the local ductility criteria set forth in general in clause 2.4.4 and specifically in the relevant paragraphs of clauses 2.7, 2.8 and 2.11.

B2.2 Evaluation of the resistance of the connections

(1) The design resistance of connections between precast elements within critical regions under seismic conditions should be evaluated as:

$$R_{pd} = R_d / (\gamma_{Rd} \cdot \gamma_{cycl}) \tag{B3}$$

where

- R_d design resistance under monotonic loading (see paragraph (2) below),
- γ_{Rd} additional model-uncertainty factor (see paragraph (3) below),
- γ_{cycl} reduction factor, accounting for resistance degradation (see paragraph (4) below).

(2) The design resistance R_d should be taken as specified in clause 4.5 of Part 1-3 of Eurocode 2. In case these provisions do not sufficiently cover the type of the

envisaged connection, additional analytical and experimental studies should be carried out.

Note 1: More specifically, in evaluating the bearing capacity of a connection versus sliding shear, the following resistance mechanisms and possible interactions among them may be taken into account:

- friction resistance under external compressive stresses and additional internal stresses due to the clamping effect of bars crossing the joint,
- dowel resistances of only those bars which do not produce longitudinal splitting (i.e. only bars located near the core of the concrete element.

Note 2: The constitutive laws of the resistance mechanisms should also account for the cyclic post-yield response inherent to the seismic design concept considered in this Eurocode.

(3) The model-uncertainty factor γ_{Rd} should be taken as follows:

- for the axial resistances:

  γ_{Rd}=1,20/1,10/1,00 for DC"H"/DC"M"/DC"L" resp.,

- for shear resistances:

  γ_{Rd}=1,35/1,25/1,15 for DC"H"/DC"M"/DC"L" resp.

(4) The reduction factor γ_{cycl} accounts for resistance degradation after an adequate number of post yield reversals of imposed deformation at the target ductility level[1]. In the absence of appropriate analytical or experimental data γ_{cycl} may be taken as follows:

- for axial resistances:

  γ_{cycl} = 1,15,

- for shear resistances: γ_{cycl} = 1,20,
- for vertical connections $\gamma_{cycl} = 1+0,15\, q_p \geq 1,20$

Note: Alternatively, simplified rules predicting the resistance degradation of these connections under reversed flexural moments may also be used, if sufficient evidence of their applicability is available.

(5) The design resistances of connections outside critical regions do not need to be reduced by γ_{Rd} and γ_{cycl}.

(6) Welding of steel bars in energy dissipating connections may be structurally taken into account under the following conditions:

a) only weldable steels are used,

b) welding materials, techniques and personnel ensure a loss of local ductility lower than 10% of the ductility achieved if connections were implemented other than by means of welding.

[1] Three full cycles are considered adequate for DC"H", two full cycles for DC"M" and one full cycle for DC"L". A displacement ductility factor equal to $(q_p+1)^{2/4}$ should be imposed.

(7) Steel elements (profiles or bars) fastened on concrete members and intended to contribute to the seismic resistance should be analytically and experimentally proved to resist a loading history of imposed deformation at the target ductility level[1].

B3 BUILDING ELEMENTS

B3.1 Beams

(1) The relevant provisions of clause 2.7 apply, in addition to the rules set forth in this annex.

(2) Simply supported prefabricated beams not structurally connected to columns or walls are not part of a frame system.

(3) In addition to clause 4.5.5.3 of Part 1-3 of Eurocode 2 the tolerance and spalling allowances of the bearings should also be sufficient for the expected displacement of the supporting member (see clause 3.4 of Part 1-2).

B3.2 Columns

(1) The relevant provisions of clause 2.8 apply, in addition to the rules set forth in this annex.

(2) Column-to-column connections within critical regions are allowed only in ductility class "L".

(3) Precast columns of one storey industrial buildings not connected into frames, may be assigned behaviour factors q_O equal to 3,0 under the following conditions:

a) the column tops are connected along both main directions of the building by ties or belts (made of steel or concrete),

and

b) the total number of columns concerned is greater than six.

B3.3 Beam-column joints

(1) Monolithic beam-column joints (fig. B1.a) should follow the relevant provisions of clause 2.9.

(2) Connections of adjacent beam-ends to columns (fig. B1.b and c) should be specifically checked for their resistance and ductility, as specified in B2.2.1.

B3.4 Precast large-panel walls

(1) The provisions of clause 5.4.7 of Part 1-3 of Eurocode 2 apply with the following modifications:

a) The total minimum reinforcement ratio of 0,004 should relate to the actual cross sectional area of concrete and should consider the vertical bars of the web and the boundary elements.

b) Single mesh reinforcement in the middle plane of panels is not allowed.

c) A minimum confinement should be given to the concrete near the edge of all precast panels, as specified in clause 2.8 for columns, in a section of b_w x b_w' where b_W denotes the thickness of the panel.

(2) When an opening is arranged closer than 2,5 b_W to the vertical joint of a panel, the dimensioning and detailing of the "column" left between the opening and the vertical joint should follow the provisions of clause 2.8.

(3) Force-response degradation of the resistance of the connections should be avoided.

(4) Towards this end, all vertical joints should be toothed.

(5) Horizontal joints under transverse compression along their entire length may be formed without teeth. However, if they are partly in compression and partly in tension, they should be toothed along their full length.

(6) The following additional rules apply for the verification of horizontal connections of walls made-of precast large panels:

a) The total tensile force produced by - with respect to the wall - axial action-effects should be taken by vertical reinforcement appropriately arranged along the tensile area of the panel and well anchored in the body of the upper and lower panels. Its continuity should be secured by ductile welding provided within the horizontal joint or, preferably, within special keys (teeth) purposely provided (see fig. B2).

b) In horizontal connections which are - under the seismic design situation - partly compressed and partly tensioned, the shear resistance verification (see B2.2.(1)) should be made only along the part under compression. In such a case, the value of the axial force N_{Sd} is replaced by the value of the total compressive force F_c acting on the compression area.

Figure B.2: Tensile reinforcement possibly needed at the edge of walls

(7) The following additional design rules should be observed to enhance local ductility along the vertical connections of large panels:

a) Minimum reinforcement should be provided across the connections equal to 0,10% in fully compressed, and to 0,25% in partly compressed (and partly tensioned) connections.

b) The amount of reinforcement across the connections should be limited in order to avoid abrupt post-peak force response softening. In the absence of more specific evidence, the reinforcement ratio should not exceed 2%.

c) Such reinforcement should be distributed across the entire length of the connection. In ductility class "L" this reinforcement may be concentrated in three bands (top, middle and bottom).

d) Provision should be made to ensure the continuity of reinforcement across panel-to-panel connections. Towards this end, in vertical connections, steel bars should be appropriately secured either in form of loops or (in the case of at least one face-free joints) they may be welded across the connection (see fig. B3).

e) In order to secure continuity along the length of the connection after cracking, a minimum amount of longitudinal reinforcement should be provided within the infill mortar of the connection (see fig. B3). In the absence of more specific evidence, the minimum ratio may be taken as 1% of the cross section of the connection.

Figure B3: Cross section of vertical connections between precast large-panels,

a) two faces free Joint   b) on face free joint

(8) As a result of the energy dissipation capacity along the vertical (and in part along the horizontal) connections of large-panels, walls made of such precast panels are exempt from the provisions of clauses 2.11.2.3 and 2.11.3, regarding complete confinement of boundary elements.

B3.5 Diaphragms

(1) In addition to the provisions of clause 5.5 of Part 1-3 of Eurocode 2 relevant to slabs and to the provisions of clause 2.12 the following de design rules also apply in case of floor diaphragms made of precast slab elements.

(2) The rigid diaphragm condition according to clause 2.12.(2) should be carefully evaluated.

(3) When such assumption is not valid, the in-plane deformability of the floor as well as of the connections to the vertical elements should be appropriately modelled (compatibility of displacements).

(4) The in-plane rigid-body behaviour is enhanced if room-size precast slab elements are used. However, an appropriate thin topping of in situ reinforced concrete may drastically improve such a rigid body function. The thickness of this topping layer should not be less than 50 mm and its mesh reinforcements should be connected to the vertical resisting elements above and below. The resistance of shear connectors should be calculated as in B2.2.(2).

(5) Tensile forces should be resisted by appropriate steel ties accomodated at least along the perimeter of the diaphragm, as well as along same joints of the precast slab elements. If a cast in-situ topping is used, this additional reinforcement should be located in this topping.

(6) In all cases, these ties should form a continuous system of reinforcement along and across the entire diaphragm, and they should be appropriately connected to each lateral force resisting element.

(7) In-plane design shear forces along slab-to-slab or slab-to-beam Connections should be computed with an overdesign factor equal to 1,50 and the design resistance should be computed as in B2.2.(2).

(8) Lateral force resisting elements, both above and below the diaphragm, should be adequately connected to the diaphragm. To this end, possible horizontal joints should always be appropriately reinforced. Friction forces due to external compressive forces cannot be accounted for, unless their minimum values are

conservatively calculated (considering both the vertical seismic acceleration as well as friction degradation due to reversed cyclic actions).