(SEISMIC)

Vulnerability & Retrofitting

---

of

PRECAST RC ELEMENTS & BLDGS

(precasting & ptestressing ? !)

Episkeves / Enishyseis
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Main issues

1) Non – monolithic construction …

2) Increased sensitivity regarding imperfections & deviations, deformations etc.

3) Increased vulnerability regarding:
   - Environmental actions
   - Accidental actions
     - Earthquake
     - Fire

reduced concrete cover, thin sections

increased displacements & distortions, reduced ductility

Main target

Avoiding of a characteristic (and catastrophic) local or global collapse, overturning and falling off, of a pancake type (for multi – storey bldgs), due to poor connections and lack of energy dissipation capacity.

→ … A well tied – together & robust structural system against earthquakes, of a quasi – monolithic type (for DC M as well as for DC H).

EC 8 – 1: 2004

1. BF $q = k_r \bullet (q_o \bullet k_w)$, $(q_o \bullet k_w) \geq 1{,}5$

   $k_r = 1{,}0$ or $0{,}8$

   $q_o = f$ (DC, ST) = 1,5 to 4,5 α_u/α_1, max $\alpha_u/\alpha_1 = 1{,}5$

   $k_w = (1+ \alpha_o)/3 = 0{,}5$ to $1{,}0$, $\alpha_o = \Sigma h_{wi} / \Sigma l_{wi}$

2. Special provisions for dimensioning / detailing of
   LARGE LIGHTLY REINFORCED WALLS, only for DCM (and not for DCH).

3. § 5.11: <u>PRECAST RC STRUCTURES (partly or entirely)</u>

   $q_p = k_p \bullet q$

   $k_p = 1{,}0$ (EC 8-1) or $0{,}5$ (with $1{,}0 \leq q_p \leq 1{,}5$)

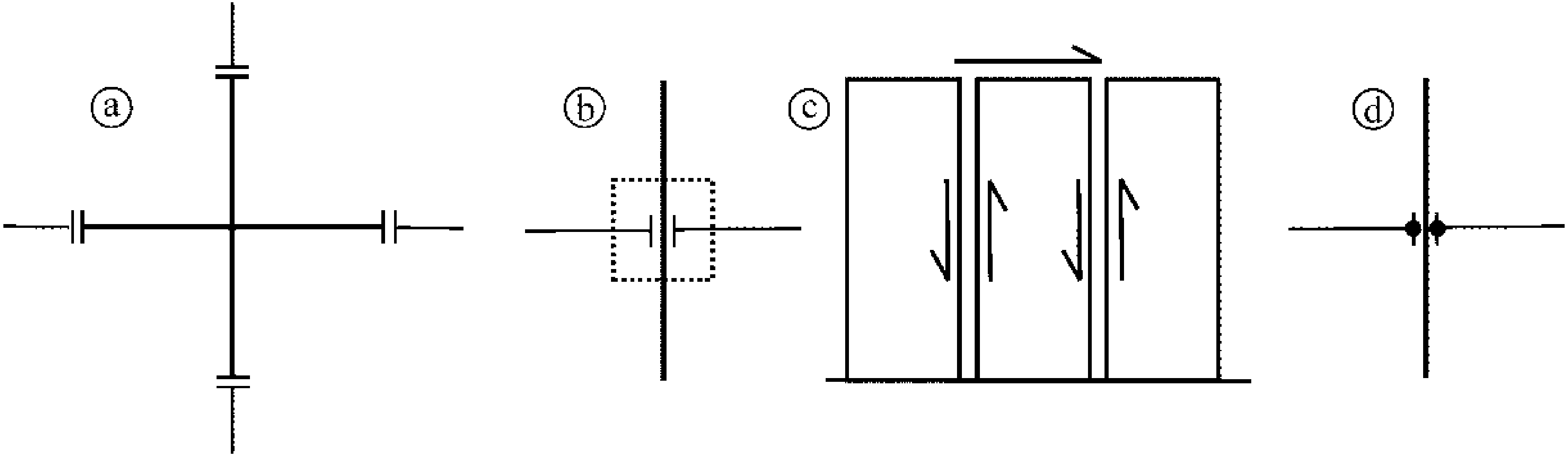


Figure 5.14: a) Connection located outside critical regions;

b) Overdesigned connection with plastic hinges shifted outside the connection;

c) Ductile shear connections of large panels located within critical regions (e.g. at ground floor); and

d) Ductile continuity connections located within critical regions of frames.

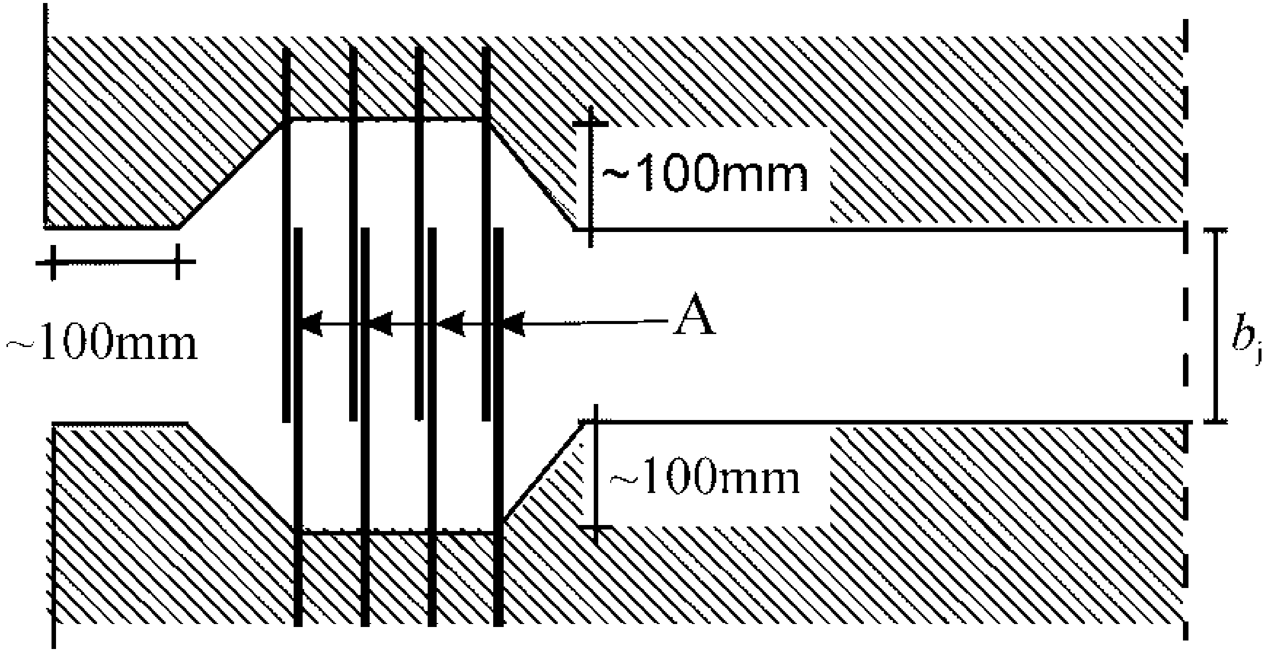
Identification of <u>the effect of the connection</u> on the energy dissipation capacity of the structure (local & global effects).

+ Additional plastic shear mechanisms (including friction) along joints,

provided that <u>both</u> of the following conditions are satisfied:

- The restoring force shall not degrade substantially during the seismic action; and
- The possible instabilities shall be appropriately avoided or taken into account.

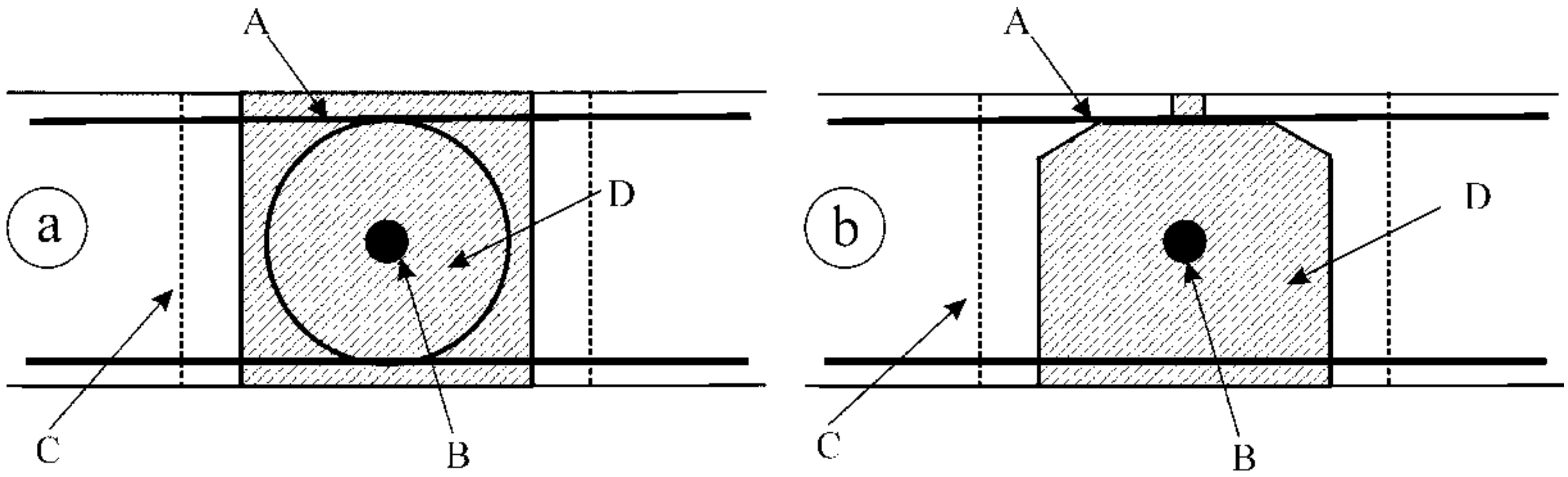
As a result of the energy dissipation capacity along the vertical (and in part along the horizontal) connections of large – panels, walls made of such precast panels are exempt from the requirements in §§ 5.4.3.4.2 and 5.5.3.4.5 regarding the confinement of boundary elements…



Key

A lap-welding of bars

Figure 5.15: Tensile reinforcement possibly needed at the edge of walls.



Key

A reinforcement protruding across connection;

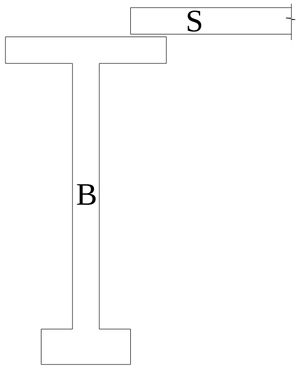
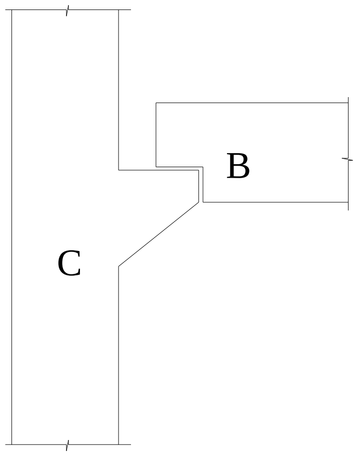
B reinforcement along connection;

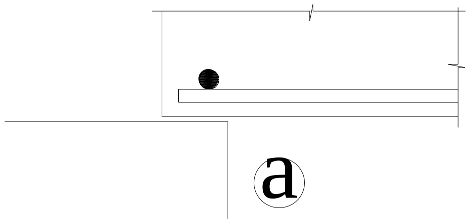
C shear keys;

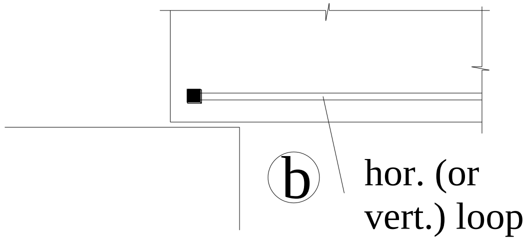
D grout filling space between panels.

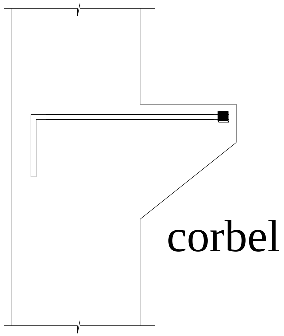
Figure 5.16: Cross-section of vertical connections between precast large-panels,

a) joint with two free faces; b) joint with one free face.

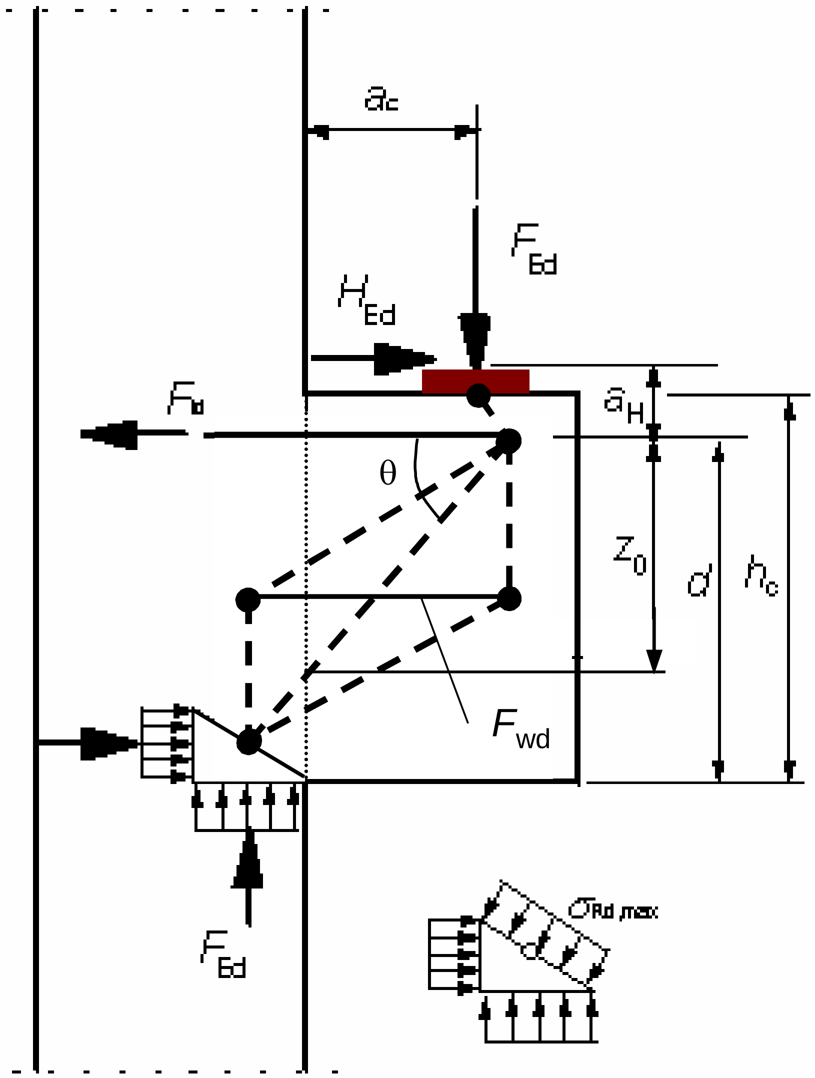
Element and/or System	Connection Type and/or Detail	Seismic Behaviour, Observed Damage	Interpretation	Structural Intervention (Repair/Strengthening)
1	**Horizontal Bearing Elements (Slabs – S, Beams – B)**			
1.1 S, B	 or 	Loss of support, falling off.	Insufficient dimensions & lengths (see, also, § 10.9 of EN 1992-1-1:2004 and §5.11 of EN 1998-1:2004).	• Inrease of dimensions (concrete or steel elements plus dowels/anchors). • Instatement of tying elements (horizontal ties at floor levels), active or passive. • Installation of special dampening/damperig systems (limitation of displacements).

<table>
<tr>
<td rowspan="2">1.2 S, B</td>
<td rowspan="2">


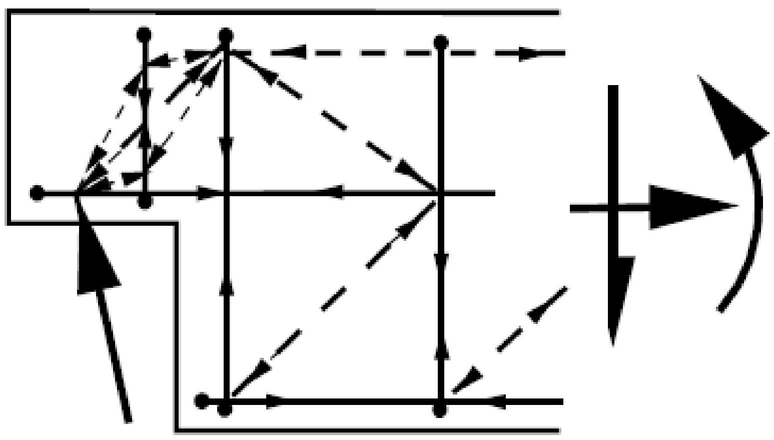
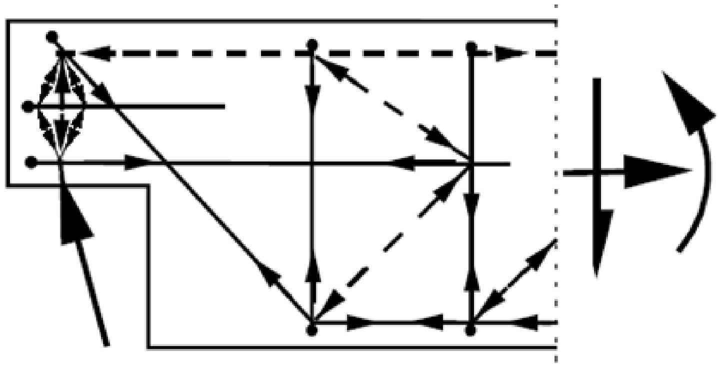
or


or


</td>
<td rowspan="2">Significant cracking, distortion & loss of support.</td>
<td>Inadequate anchorage of reinforcement (see, also, § 8.6 & Annex J.3 of EN 1992-1-1:2004 and § 5.11 of EN 1998-1:2004), under a high level of a simultaneous vertical component of the seismic action.</td>
<td rowspan="2">
• See § 1.1 here above.
• Additional anchoring (e.g. by transverse pressure or special devices).
</td>
</tr>
<tr>
<td>For cases a & b: Additional tension or, even, compression of the main reinforcement, lack of anchorage.</td>
</tr>
</table>



CORBEL STRUT – AND – TIE MODEL

<table>
<tr>
<td>1.3 S, B</td>
<td>Half joints (GERBER supports).


Indicative strut – and – fie models (see, also, §§ 6.5 and 10.9 of EN 1992-1-1:2004).</td>
<td>Cracking, distortion, loss of support (or, even, falling off).</td>
<td>See §§ 1.1 & 1.2 here above.</td>
<td>See §§ 1.1 & 1.2 here above.</td>
</tr>
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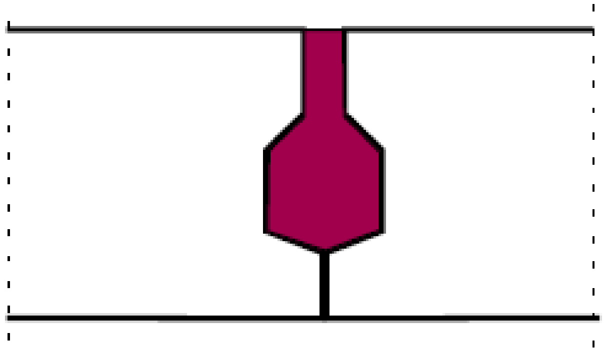
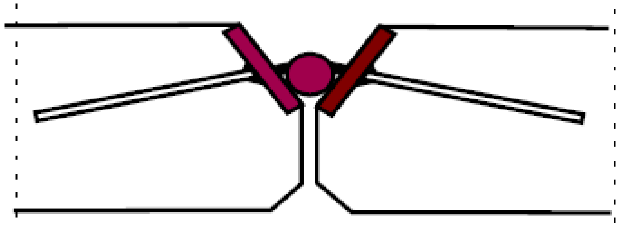
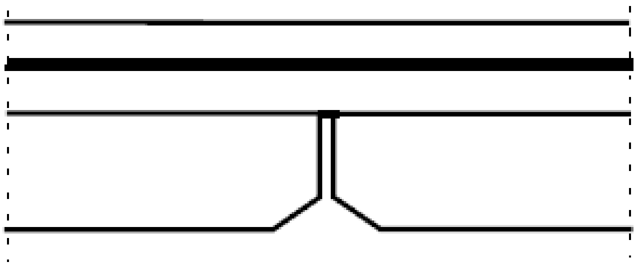
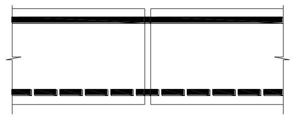
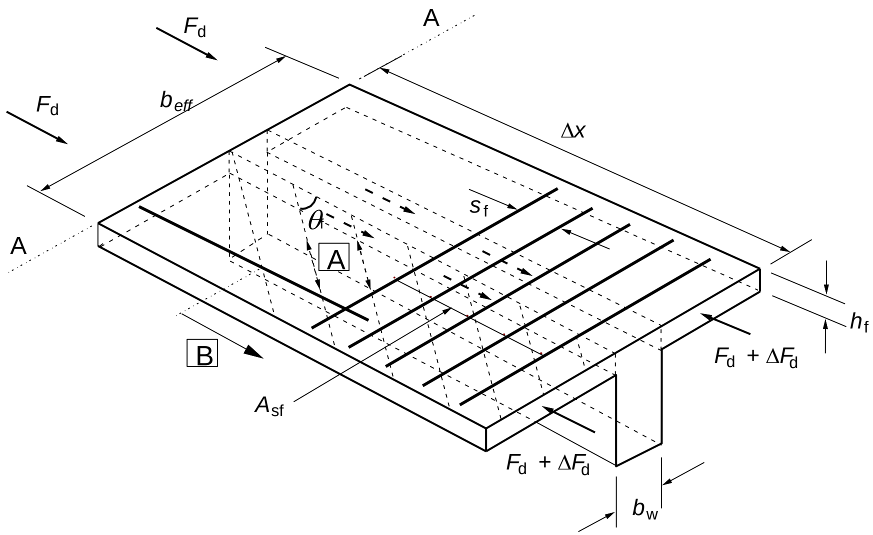
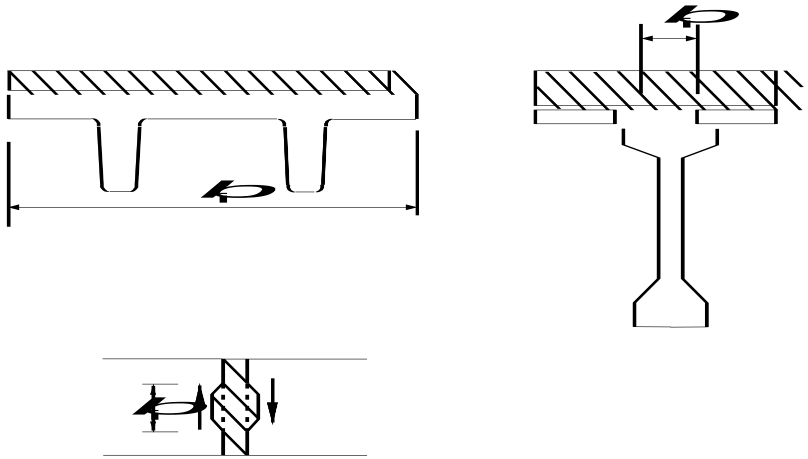
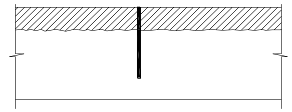
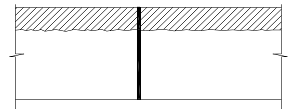
1.4 S	"Linear" connections for shear transfer.  a) Concreted or grouted connections  b) Welded or bolted connections (this shows one type of welded connection as an example)  c) Reinforced topping (vertical reinforcement connectors to topping may be required)	Significant cracking (permanent), distortion.	Inadequate connection for a combined in-plane diaphragmatic action and out-of-plane response (under the vertical component of the seismic action). Premature disorganization of connection resistances (and brittle failures).	"Stitching", see §§ 3.2 & 3.3 here below, by turns (on top and on bottom). 

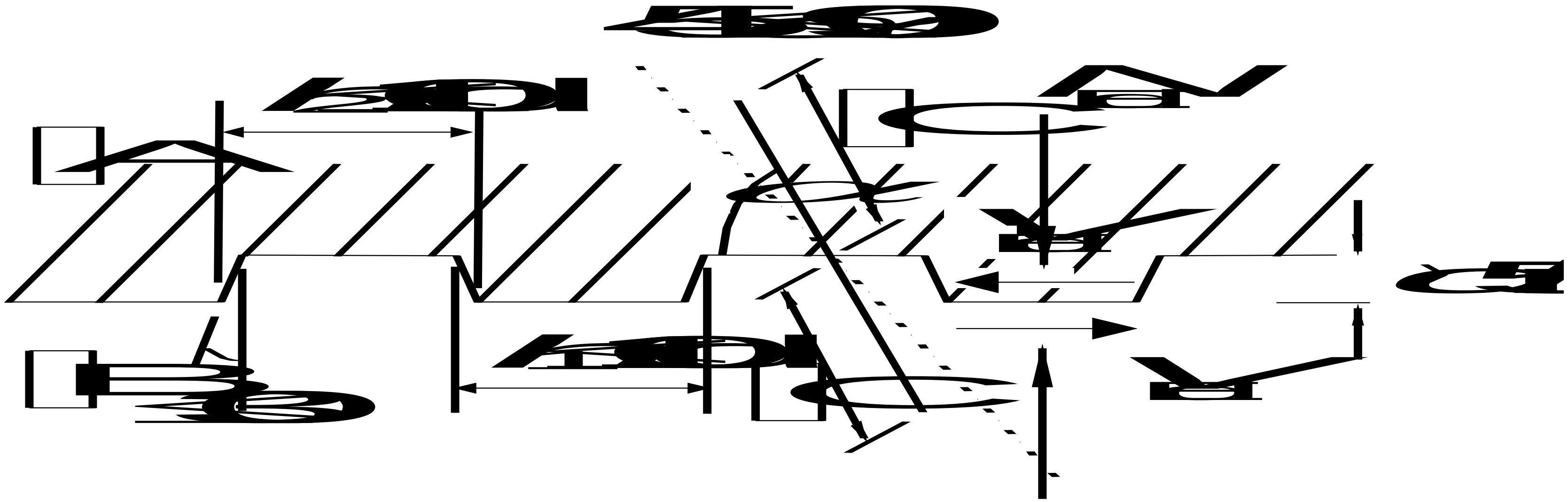
1.5 S+B	"Linear" connections between beams (webs) and flanges (slabs) under shear or / and shear at the interfaces between concrete elements casted at different times (see, also, §§ 6.2.4 & 6.2.5 of EN 1992-1-1:2004)  $\boxed{A}$: Compressive struts $\boxed{B}$: Longitudinal bar anchored beyond this projected point	See § 1.4 here above.	See § 1.4 here above.	• "Stitching", see §§ 3.2 & 3.3 here below. • "Nailing" by means of dowels, passing through or not.

	 Examples of interfaces			 "nailing" 

<u>INTERFACES (see EC 2-1-1:2004)</u>

$$v_{\mathrm{Rdi}} = c \cdot f_{\mathrm{ctd}} + \mu \cdot \sigma_{\mathrm{n}} + \rho \cdot f_{\mathrm{yd}} \, (\mu \cdot \sin \alpha + \cos \alpha) \le 0{,}5 \; \nu \cdot f_{\mathrm{cd}}$$

(very smoot*h*, smooth, rough or indented)

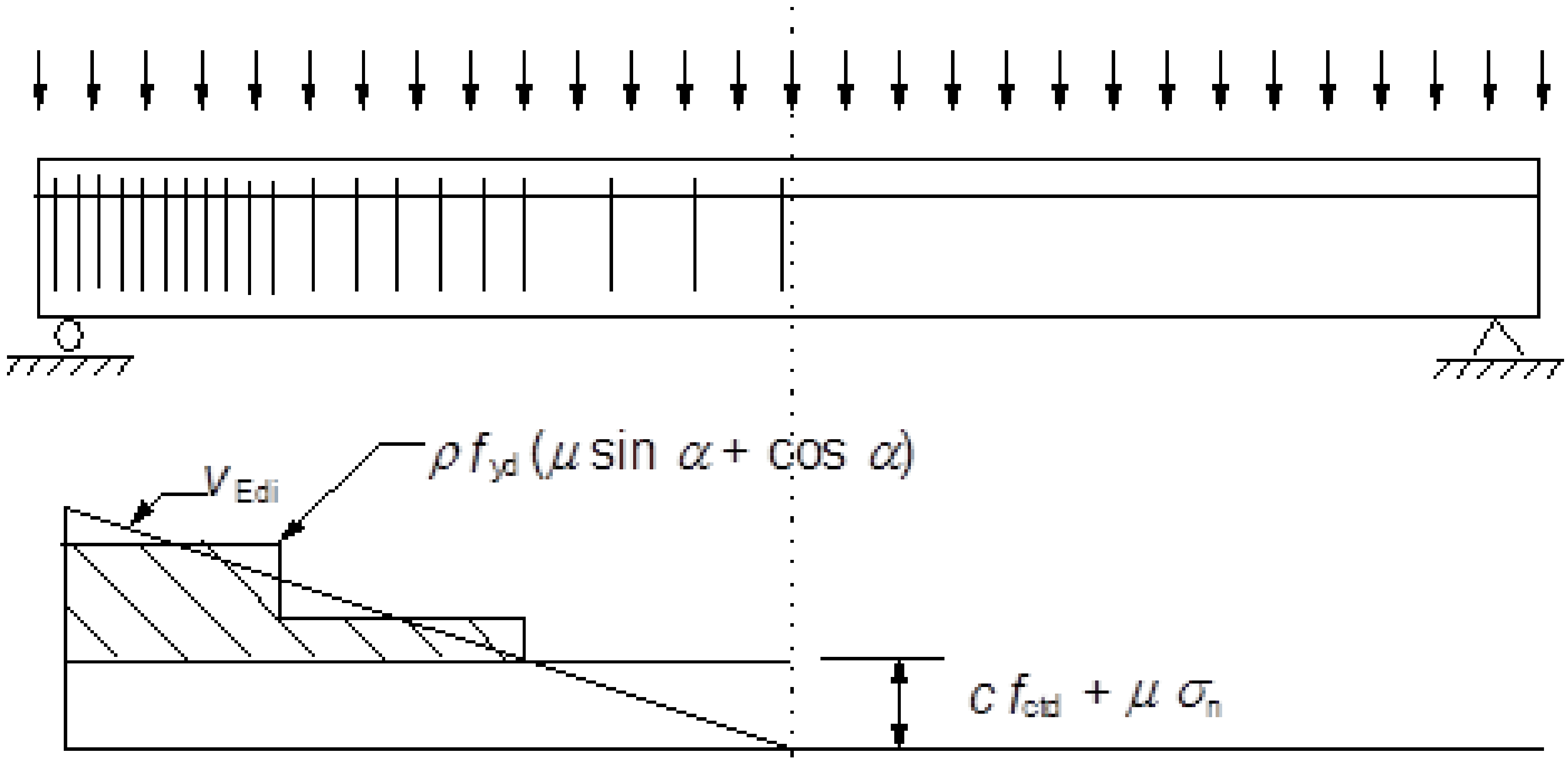


A - new concrete,

B - old concrete,

C - anchorage

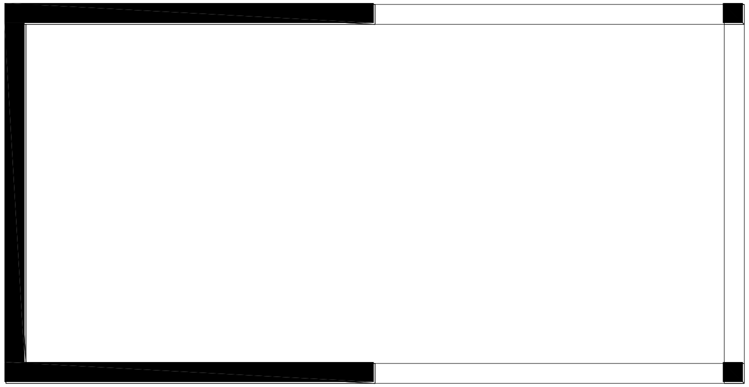
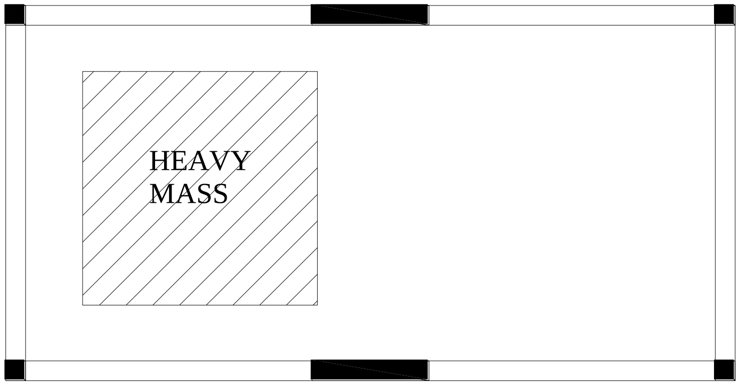
Indented construction joint

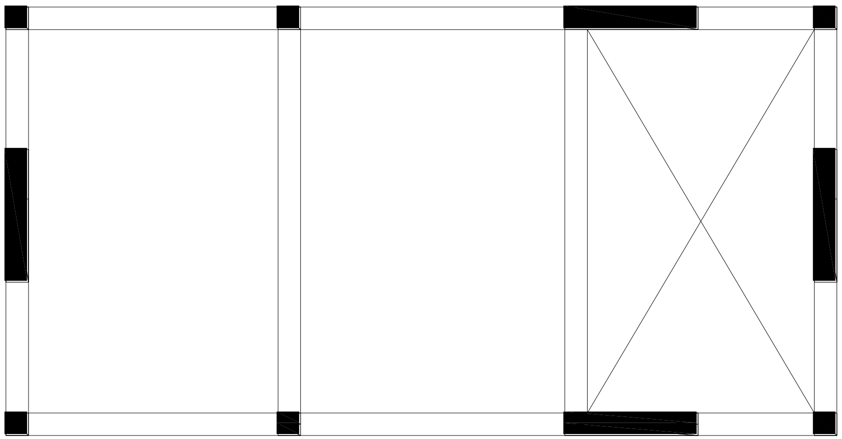
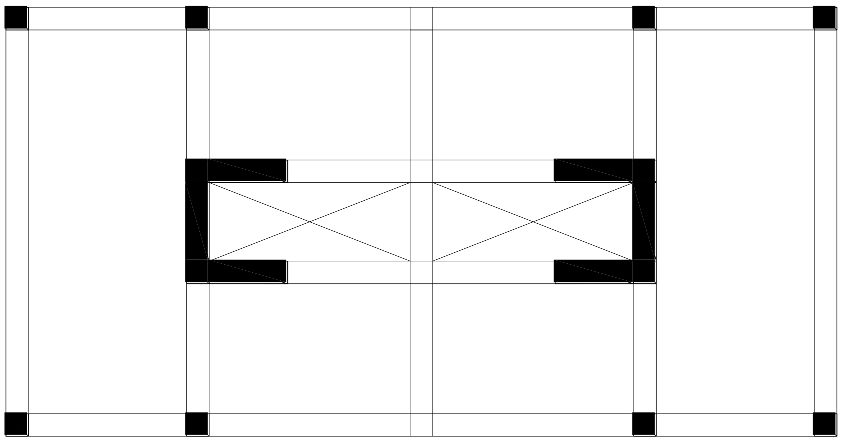


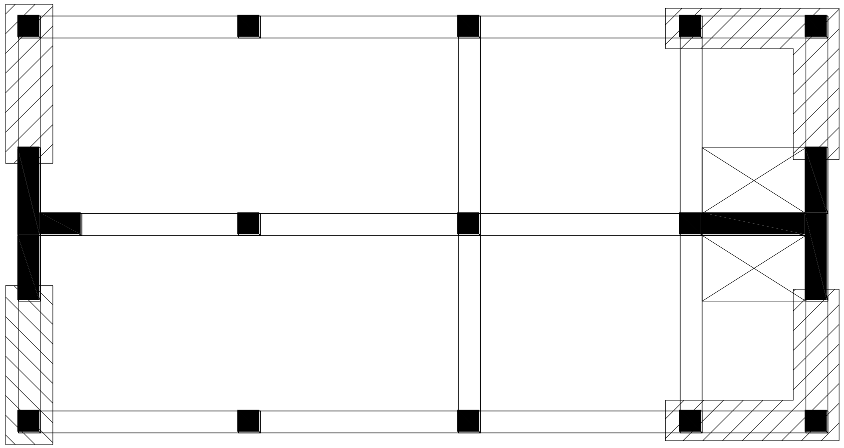
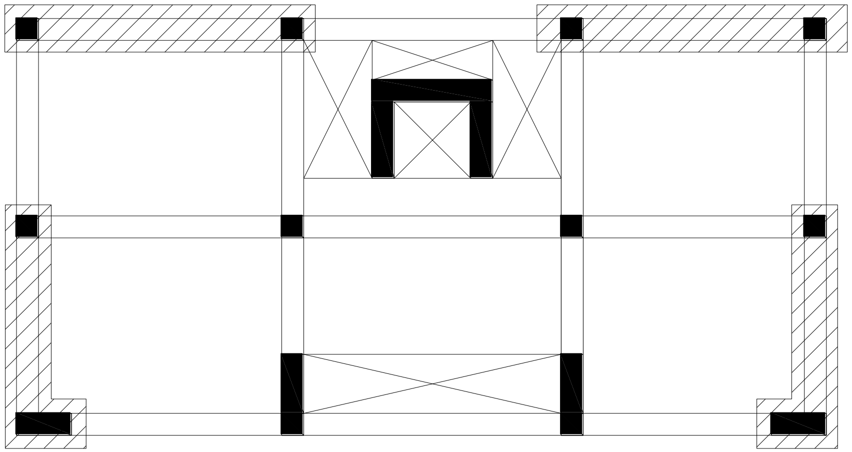
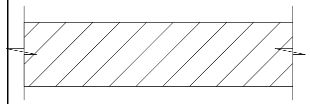
Shear diagram representing the requited interface reinforcement

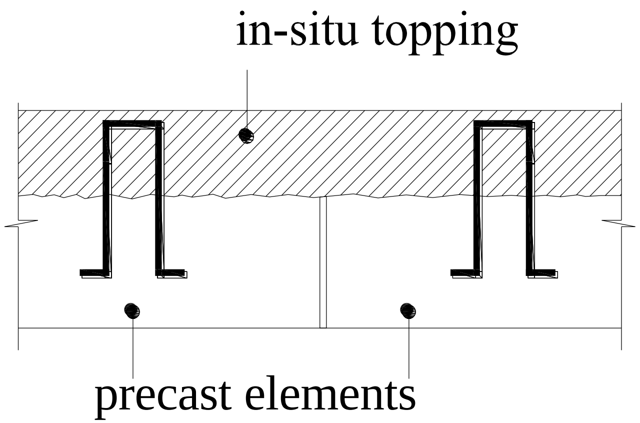
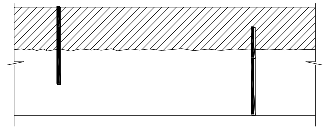
(with a stepped distribution)

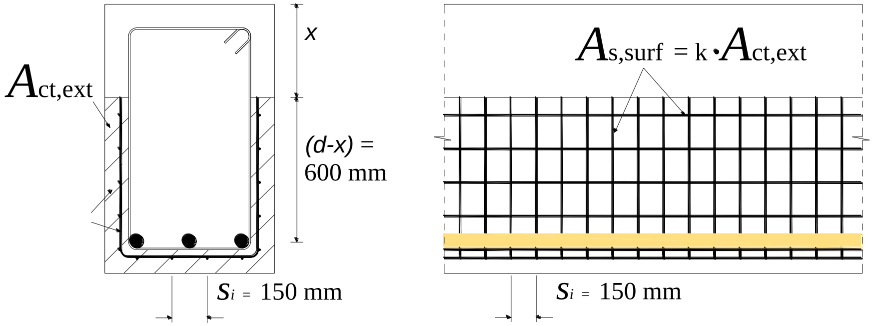
$v_{Edi} = \beta V_{Ed} / (z b_i)$

1.6 S (mainly for dual or wall systems)	Connections between slabs (and beams):  Resistance eccentricity  Mass eccentricity	Problematic diaphragmatic action and transfer of action – effects, cracking of floor systems, damages of some components of the seismic resisting system (“selectively”).	“Discontinuity” of the “load path”, premature disorganization of resistances, “imposition” of structural non – regularity (in – plan), accidental torsion etc.	• Repair of cracks (possibly combined with “stitching” or “nailing”). • Minimizing of eccentriuties and / or discontinuities, irregularities etc., e.g. by arranging new shear walls. • Instatement of ties (peripheral and / or internal ones), prevention of progressive collapse.

	DISCONTINUITY IN DIAPHRAGMATIC ACTION (strength, stiffness) or large openings close to main shear walls.  			

	 			 New shear walls (cast in-situ).

1.7 S	"Double" slabs.  	Internal cracking separation, edge distortions, deflections etc.	Combined effects of the seismic action and of insufficient detailing.	Repair (resin glue injections) and "nailing". 

1.8 B	Thin webs of beams, with or w/o surface (skin) reinforcement (see, also, Annex J.1 of EN 1992-1-1: 2004).  x is the depth of the neutral axis $k = \begin{cases} 0{,}005 \text{ where } c \geq 70 \text{ mm (for enhanced durability)} \\ 0{,}010 \text{ for bars or bundles of bars with } \Phi \text{ or } \Phi_n \geq 32 \text{ mm} \end{cases}$ **Example of surface reinforcement**	Excessive cracking, splitting & spalling near supports.	Excessive action effects due to heavy loads combined with a high level of vertical component of the seismic action.	• Local repair / strengthening. • External confinement.

RC DIAPHRAGMS (EC 8-1 : 2004)

1. MONOLITHIC

1.1 §§ 4.2.1 Basic principles, conceptual design

Adequate diaphragmatic behaviour of storey level

(among six "sister" guiding & governing principles).

1.2 § 4.2.1.5 Diaphragmatic behaviour.

(1) In buildings, floors (including the roof) play a very important role in the overall seismic behaviour of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems).

(2) Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-

plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure.

(3) Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis (e.g. rigidity of the diaphragm, see 4.3.1(4)), particularly when there are significant changes in stiffness or offsets of vertical elements above and below the diaphragm.

1.3 § 4.4.2 Ultimate limit states relative auditions.
Adequate resistance of horizontal diaphragms

(among six “sister” conditions to be met).

1.4 § 4.4.2.5 Resistance of horizontal diaphragms.

(1)P Diaphragms and bracings in horizontal planes shall be able to transmit, with sufficient overstrength, the effects of the design seismic action to the lateral load-resisting systems to which they are connected.

(2) The requirement in (1)P of this subclause is considered to be satisfied if for the relevant resistance verifications the seismic action effects in the diaphragm obtained from the analysis are multiplied by an overstrength factor γ_d greater than 1,0.

NOTE: The values to be ascribed to γ_d for use in a country may be found in its National Annex. The recommended value for brittle failure modes, such as in shear in concrete diaphragms, is 1.3, and for ductile failure is 1,1.

(3) Design provisions for concrete diaphragms are given in 5.10.

1.5 § 4.3 Structural analysis.

1.6 § 4.3.1 Modeling, floor diaphragms.

(1)P The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered. In the case of non-linear analysis, the model shall also adequately represent the distribution of strength.

(2) The model should also account for the contribution of joint regions to the

deformability of the building, e.g. the end zones in beams or columns of frame type structures. Non-structural elements, which may influence the response of the primary seismic structure, should also be accounted for.

(3) In general the structure may be considered to consist of a number of vertical and lateral load resisting systems, connected by horizontal diaphragms.

(4) When the floor diaphragms of the building may be taken as being rigid in their planes, the masses and the moments of inertia of each floor may be lumped at the centre of gravity.

NOTE: The diaphragm is taken as being rigid, if, when it is modelled with its actual in-plane flexibility, its horizontal -displacements nowhere exceed those resulting from the rigid diaphragm assumption by more than 10% of the corresponding absolute horizontal displacements in the seismic design situation.

5.10 Provisions for concrete diaphragms

(1) A solid reinforced concrete slab may be considered to serve as a diaphragm, if it has a thickness of not less than 70 mm and is reinforced in both horizontal directions with at least the minimum reinforcement specified in EN 1992-1-1:2004.

(2) A cast-in-place topping on a precast floor or roof system may be considered as a diaphragm, if: a) it meets the requirements of (1) of this subclause; b) it is designed to provide alone the required diaphragm stiffness and resistance; and c) it is cast over a clean, rough substrate, or connected to it through shear connectors.

(3)P The seismic design shall include the ULS, verification of reinforced concrete diaphragms in DCH structures with the following properties:

- irregular geometries or divided shapes in plan, diaphragms with recesses and reentrances;
- irregular and large openings in the diaphragm;
- irregular distribution of masses and/or stiffnesses (as e.g. in the case of set-backs or off-sets);
- basements with walls located only in part of the perimeter or only in part of the ground floor area;

(4) Action-effects in reinforced concrete diaphragms may be estimated by modelling the diaphragm as a deep beam or a plane truss or strut-and-tie model, on elastic supports.

(5) The design values of the action effects should be derived taking into account 4.4.2.5.

(6) The design resistances should be derived in accordance with EN 1992-1-1:2004.

(7) In cases of core or wall structural systems of DCH, it should be verified that the transfer of the horizontal forces from the diaphragms to the cores or walls has occurred. In this respect the following provisions apply:

a) the design shear stress at the interface of the diaphragm and a core or wall should be limited to $1,5f_{ctd}$, to control cracking;

b) an adequate strength to guard against shear sliding failure should be ensured, assuming that the strut inclination is 45°. Additional bars should be provided, contributing to the shear strength of the interface between diaphragms and cores or walls; anchorage of these bars should follow the provisions of 5.6 (for anchorages & lap splices).

2. **<u>NON - MONOLITHIC</u>**

§ 5.11 (partly or entirely) Precast RC structures

5.11.3.5 Diaphragms

(1) In addition to the provisions of EN 1992-1-1:2004, Section 10 relevant to slabs and to the provisions of 5.10, the following design rules also apply in the case of floor diaphragms made of precast elements.

(2) When the rigid diaphragm condition in accordance with 4.3.1(4) is not satisfied, the in-plane flexibility of the floor as well as of the connections to the vertical elements should be taken into account in the model.

(3) The rigid diaphragm behaviour is enhanced if the joints in the diaphragm are located only over its supports. An appropriate topping of in-situ reinforced concrete can drastically improve the rigidity of the diaphragm. The thickness of this topping layer should be not less than 40 mm if the span between supports is less than 8 m, or not less than 50 mm for longer spans; its mesh reinforcement should be connected to the vertical resisting elements above and below.

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

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

(6) In-plane acting shear forces along slab-to-slab or slab-to-beam connections should be computed with an overdesign factor equal to 1,30. The design resistance should be computed as in 5.11.2.2 (for the resistance of connections)

(7) Primary seismic elements, both above and below the diaphragm, should be adequately connected to the diaphragm. To this end, any horizontal joints should always be properly reinforced. Friction forces due to external compressive forces should not be relied upon.

5.11.2.2 Evaluation of the resistance of connections

(1) The design resistance of the connections between precast concrete elements should be calculated in accordance with the provisions of EN 1992-1-1:2004, 6.2.5 and of EN 1992-1-1:2004, Section 10, using the material partial factors. If those provisions do not adequately cover the connection under consideration, its resistance should be evaluated by means of appropriate experimental studies.

(2) In evaluating the resistance of a connection against sliding shear, friction resistance due to external compressive stresses (as opposed to the internal stresses due to the clamping effect of bars crossing the connection) should be neglected.

(3) Welding of steel bars in energy dissipating connections may be structurally taken into account when all of the following conditions are met:

a) only weldable steels are used;

b) welding materials, techniques and personnel ensure a loss of local ductility less than 10% of the ductility factor achieved if the connection were implemented without welding.

(4) Steel elements (sections or bars) fastened on concrete members and intended to contribute to the seismic resistance should be analytically and experimentally demonstrated to resist a cyclic loading history of imposed deformation at the target ductility level, as specified in 5.11.2.1.3(2).

… at least 3 full cycles at an amplitude corresponding to q_p

TYING SYSTEMS & ELEMENTS (EC 2-1-1: 2004)

- SECTION 9, DETAILING OF MEMBERS & PARTICULAR RULES

  § 9.10 Tying Systems

- SECTION 10, ADDITIONAL RULES FOR PRECAST ELEMENTS & STRUCTURES

  § 10.9 Particular Rules for Design & Detailing

    § 10.9.7 Tying Systems

9.10 Tying systems

9.10.1 General

(1)P Structures which are not designed to withstand accidental actions shall have a suitable tying system, to prevent progressive collapse by providing alternative load paths after local damage. The following simple rules are deemed to satisfy this requirement.

(2) The following ties should be provided:

a) peripheral ties

b) internal ties

c) horizontal column or wall ties

d) where required, vertical ties, particularly in panel buildings.

(3) Where a building is divided by expansion joints into structurally independent sections, each section should have an independent tying system.

(4) In the design of the ties the reinforcement may be assumed to be acting at its characteristic strength and capable of carrying tensile forces defined in the following clauses.

(5) Reinforcement provided for other purposes in columns, walls, beams and floors may be regarded as providing part of or the whole of these ties.

9.10.2 Proportioning of ties

9.10.2.1 General

(1) Ties are intended as a minimum and not as an additional reinforcement to that required by structural analysis.

9.10.2.2 Peripheral ties

(1) At each floor and roof level an effectively continuous peripheral tie within 1,2 m from the edge should be provided. The tie may include reinforcement used as part of the internal tie.

(2) The peripheral tie should be capable of resisting a tensile force:

$$F_{\mathrm{tie{\cdot}per}} = l_i \bullet q_1 \leq q_2 \qquad (9.15)$$

where:

$F_{\mathrm{tie{\cdot}per}}$ tie force (here: tension)

li length of the end-span

Note: Values of q_1 and q_2 for use in a Country may be found in its National Annex. The recommended value of q_1 is 10 kN/m and of q_2 is 70 kN.

(3) Structures with internal edges (e.g. atriums, courtyards, etc.) should have peripheral ties in the same way as external edges which shall be fully anchored.

9.10.2.3 Internal ties

(1) These ties should be at each floor and roof level in two directions approximately at right angles. They should be effectively continuous throughout their length and should be anchored to the peripheral ties at each end, unless continuing as horizontal ties to columns or walls.

(2) The internal ties may, in whole or in part, be spread evenly in the slabs or may be grouped at or in beams, walls or other appropriate positions. In walls they should be within 0,5 m from the top or bottom of floor slabs, see Figure 9.15.

(3) In each direction, internal ties should be capable of resisting a design value of tensile force $F_{tie,int}$ (in kN per metre width):

Note: Values of $F_{tie,int}$ for use in a Country may be found in its National Annex. The recommended value is 20 kN/m.

(4) In floors without screeds where ties cannot be distributed across the span direction, the transverse ties may be grouped along the beam lines. In this case the minimum force on an internal beam line is:

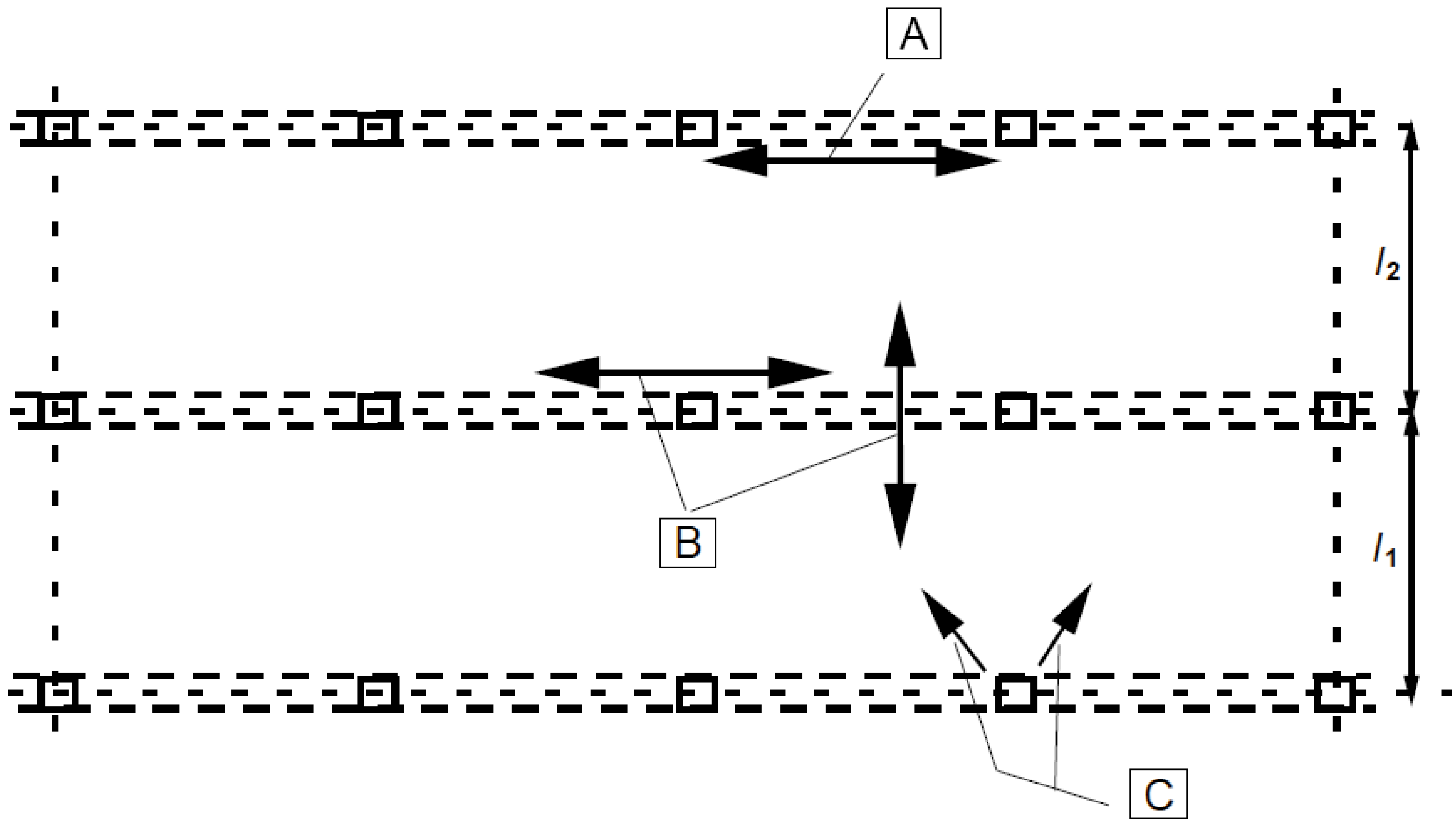
$$F_{\text{tie}} = (l_1 + l_2)/2 \bullet q_3 \leq q_4$$

where:

l_1, l_2 are the span lengths (in m) of the floor slabs on either side of the beam (see Figure 9.15)

Note: Values of q_3 and q_4 for use in a Country may be found in its National Annex. The recommended value of q_3 is 20 kN/m and of q_4 is 70 kN.

(5) Internal ties should be connected to peripheral ties such that the transfer of forces is assured.



$\boxed{A}$ - peripheral tie

$\boxed{B}$ - internal tie

$\boxed{C}$ - horizontal column or wall tie

Figure 9.15: Ties for Accidental Actions

9.10.2.4 Horizontal ties to columns and/or walls

(1) Edge columns and walls should be tied horizontally to the structure at each floor and roof level.

(2) The ties should be capable of resisting a tensile force $f_{tie,fac}$ per metre of the façade. For columns the force need not exceed $f_{\text{tie,col}}$.

Note: Values of $f_{tie,fac}$ and $f_{\text{tie,col}}$ for use in a Country may be found in its National Annex. The recommended value of $f_{tie,fac}$ is 20 kN/m and of $f_{\text{tie,col}}$ is 150 kN.

(3) Corner columns should be tied in two directions. Steel provided for the peripheral tie may be used as the horizontal tie in this case.

9.10.2.5 Vertical ties

(1) In panel buildings of 5 storeys or more, vertical ties should be provided in columns and/or walls to limit the damage of collapse of a floor in the case of accidental loss of the column or wall below. These ties should form part of a bridging system to span over the damaged area.

(2) Normally, continuous vertical ties should be provided from the lowest to the highest level, capable of carrying the load in the accidental design situation, acting

on the floor above the column/wall accidentally lost. Other solutions e.g. based on the diaphragm action of remaining wall elements and/or on membrane action in floors, may be used if equilibrium and sufficient deformation capacity can be verified.

(3) Where a column or wall is supported at its lowest level by an element other than a foundation (e.g. beam or flat slab) accidental loss of this element should be considered in the design and a suitable alternative load path should be provided.

9.10.3 Continuity and anchorage of ties

(1)P Ties in two horizontal directions shall be effectively continuous and anchored at the perimeter of the structure.

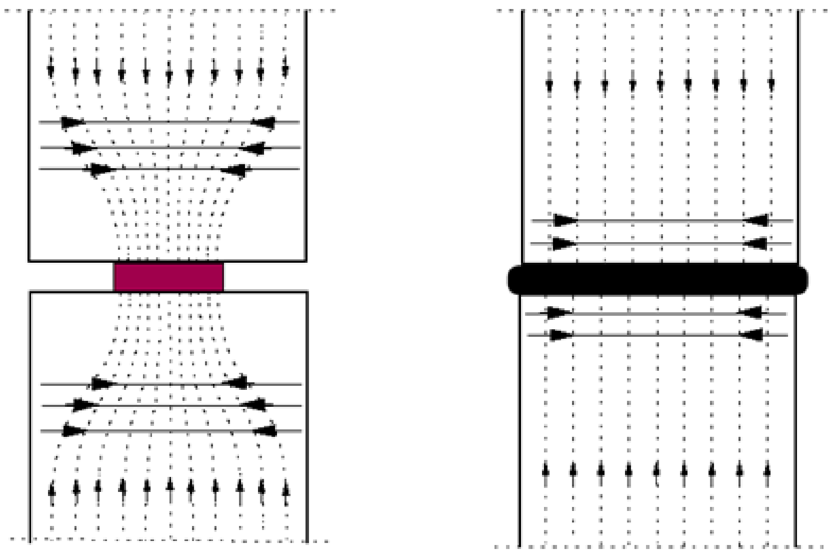
(2) Ties may be provided wholly within the in situ concrete topping or at connections of precast members. Where ties are not continuous in one plane, the bending effects resulting from the eccentricities should be considered.

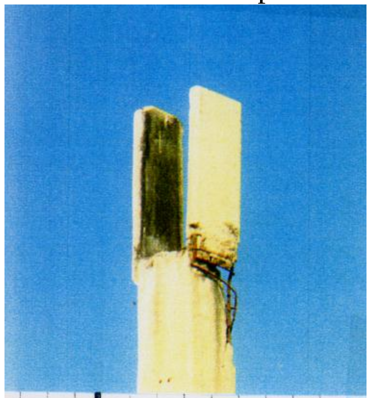
(3) Ties should not normally be lapped in narrow joints between precast units. Mechanical anchorage should be used in these cases.

10.9.7 Tying systems

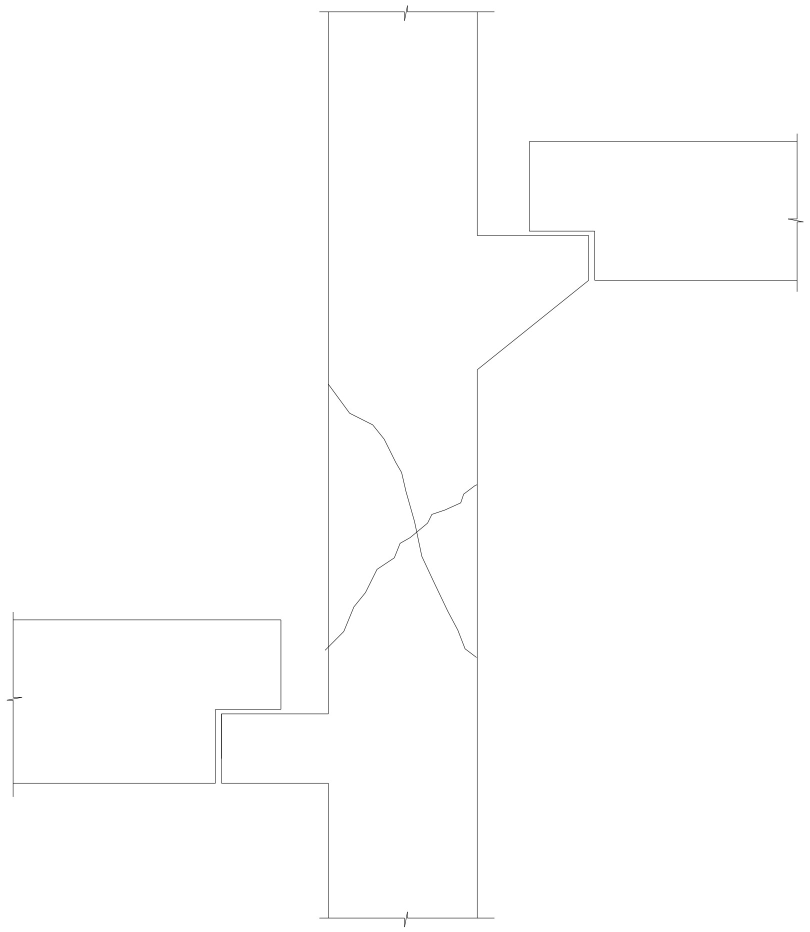
(1) For plate elements loaded in their own plane, e.g. in walls and floor diaphragms, the necessary interaction may be obtained by tying the structure together with peripheral and/or internal ties.

The same ties may also act to prevent progressive collapse according to 9.10.

Element and/or System	Connection Type and/or Detail	Seismic Behaviour, Observed Damage	Interpretation	Structural Intervention (Repair/Strengthening)
2	**Vertical Bearing Elements, Columns – C**			
2.1 C	Connections transmitting compressive forces, with or w/o bedding materials (dry ones).  a) Concentrated bearing b) Expansion of soft padding Transverse tensile stresses at compression connections	Premature local damages, permanent relative displacements.	High axial (and shear) forces, due to the combined seismic action. Pre-existing problems due to inappropriate quality of workmanship, deviations and displacements (even unintentional).	• Local repair / strengthening. • External confinement. • Light steel jacketing (abolition of the “joint”)

2.2 C	Beam cases on the top of columns.   	Severe local damages or even failures & collapses.	Large displacements, insufficient geometry & detailing.	• See § 1.1 here above • External tying. • Light steel jacketing .

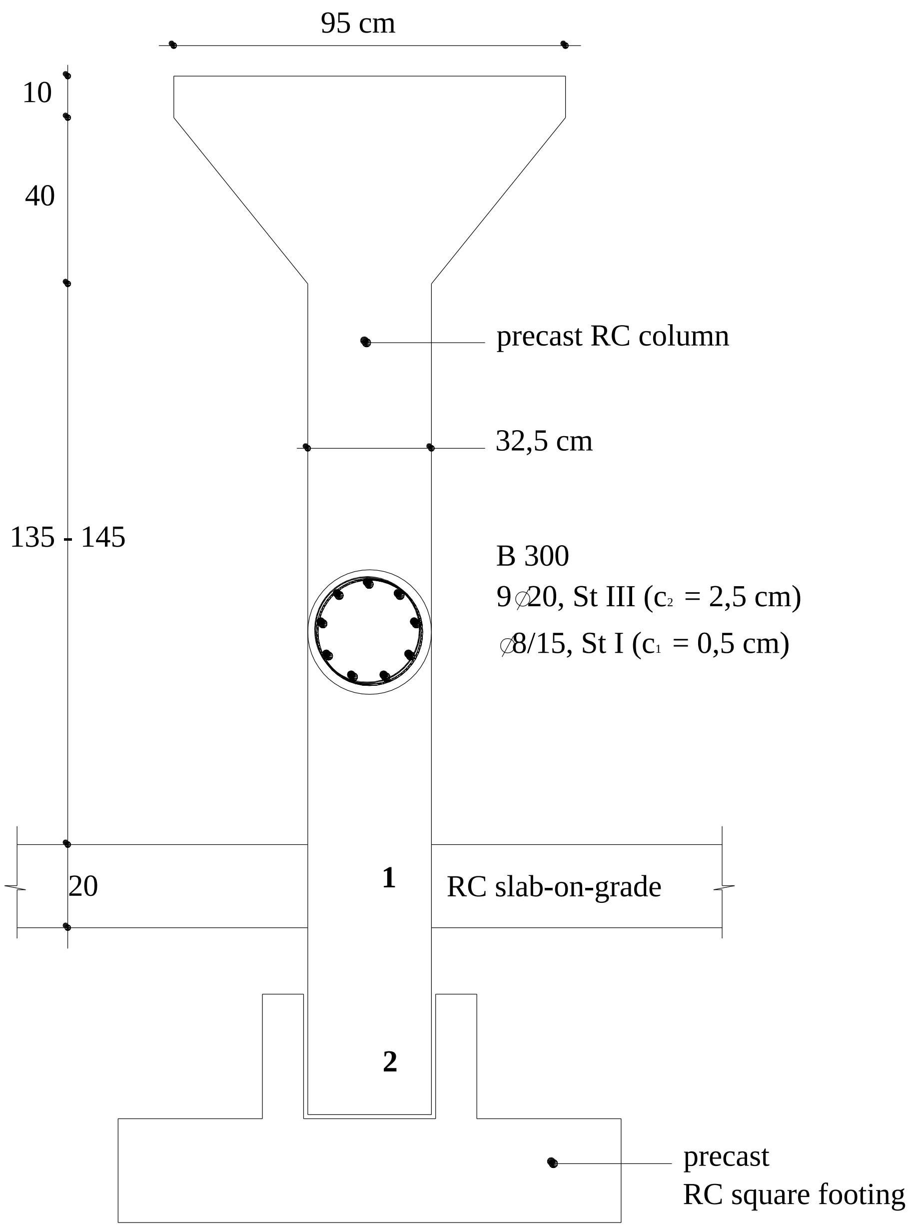
2.3 C	Cladding cases in the sides of columns. 	Severe local damages or even failures & collapses.	Large displacements, insufficient geometry & detailing, see also § 2.2 here above. Short column behaviour and premature failure, see also § 2.4 here below. Strong interaction between the skeleton and the cladding system and casing.	• See § 1.1 here above • "Isolation" of the cladding. • Arrangement of "wing" walls on the sides of the column.

2.4 C	Short columns. 	Severe local damages, permanent displacement, loss of support.	Short column behaviour and premature shear (brittle) failure. Hammering / pounding effects.	• See § 1.1 here above • External confinement. • Full height jacketing.

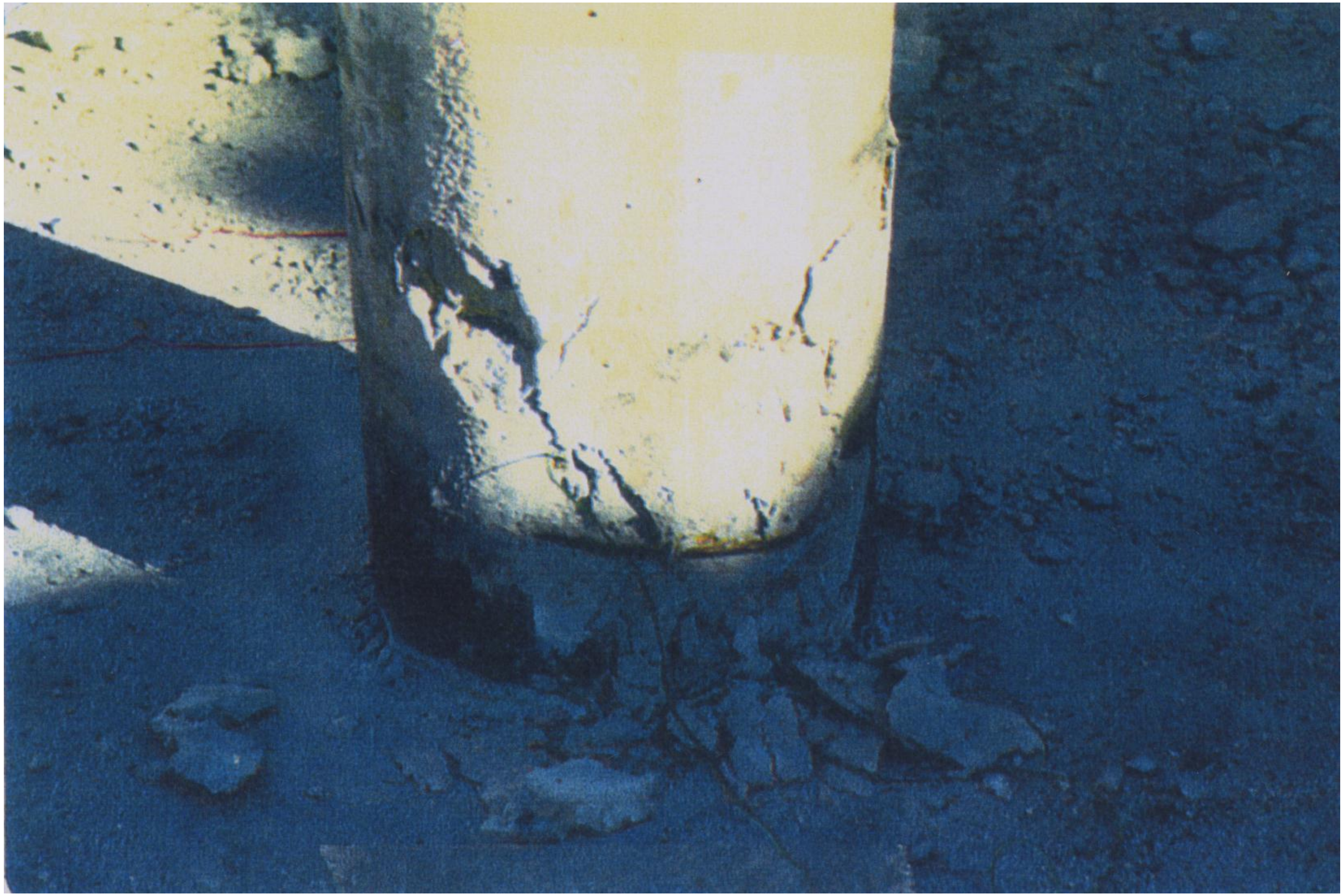
2.5 C	Stub columns (in pockets), supporting heavy oil tanks (see sketch & photos).	Severe damages & collapses.	Short column behaviour, combined with inverted pendulum behaviour.	• Simply cracked or almost intact columns: Repair and "ise-lation" between the column and the slab-on-grade. • Lightly damaged columns: "Isolation" and external confinement. • Heavily damaged columns: Demolition and reconstruction, with "identical" precast elements, "isolation".

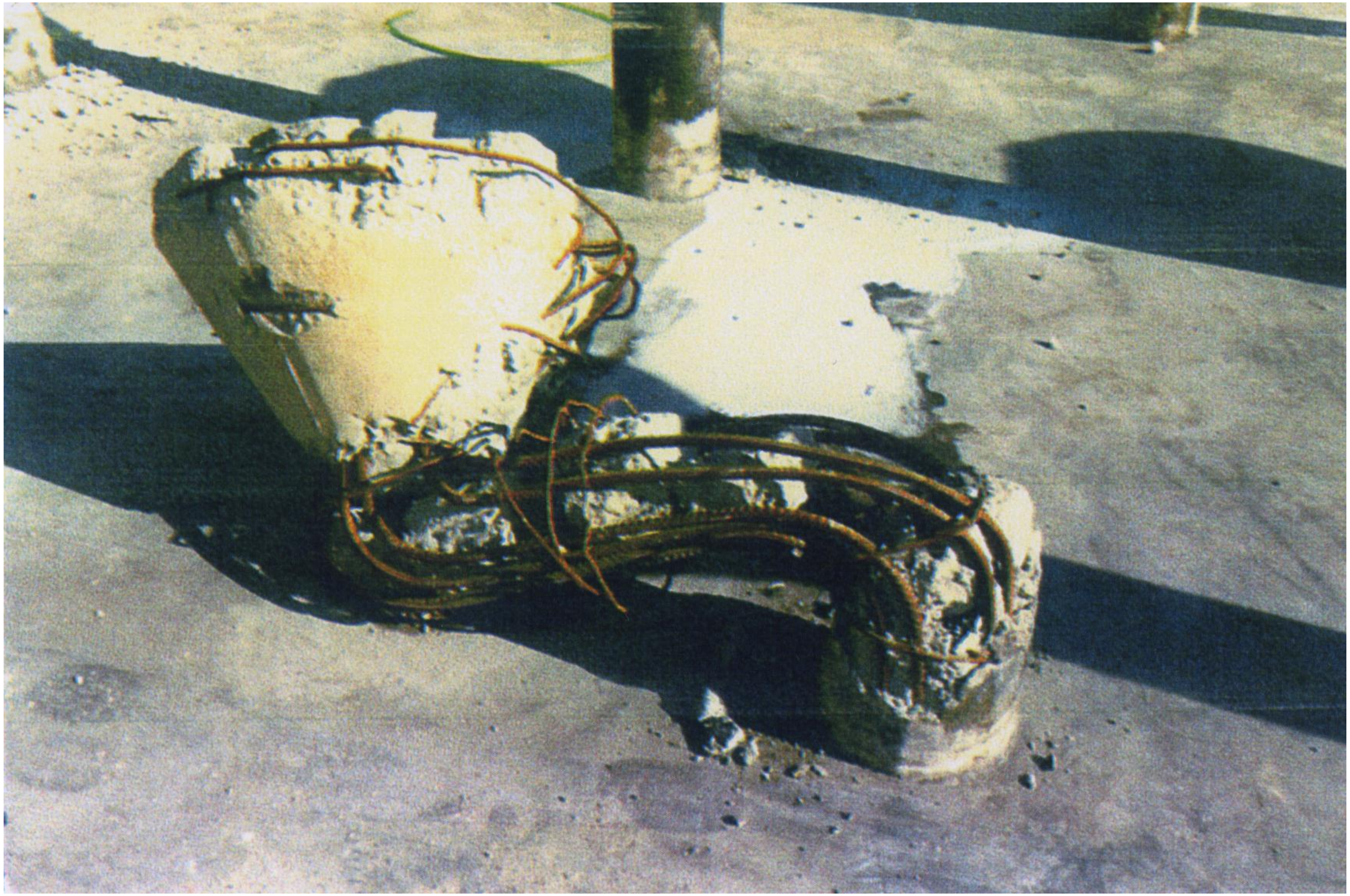
General view, after the earthquake.

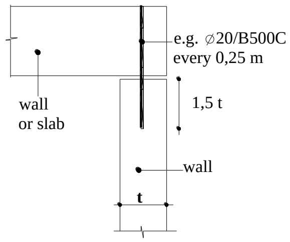
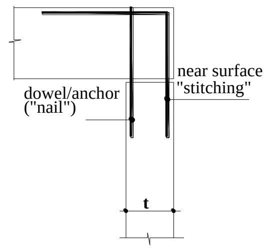
Out of a total of approx. 2000 precast RC columns, almost 2/3 of them were damaged (see photos).



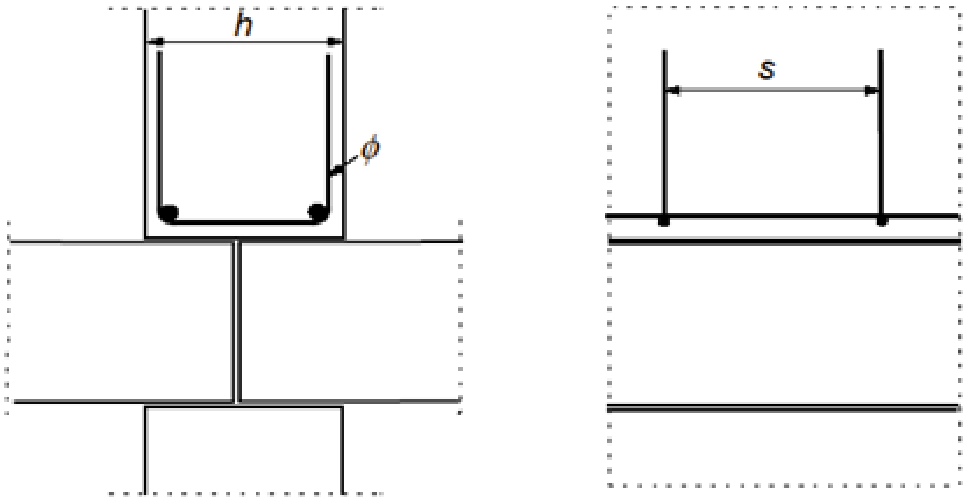
1: Full contact and restraint
2: Smooth, "dry" joint (pocket)





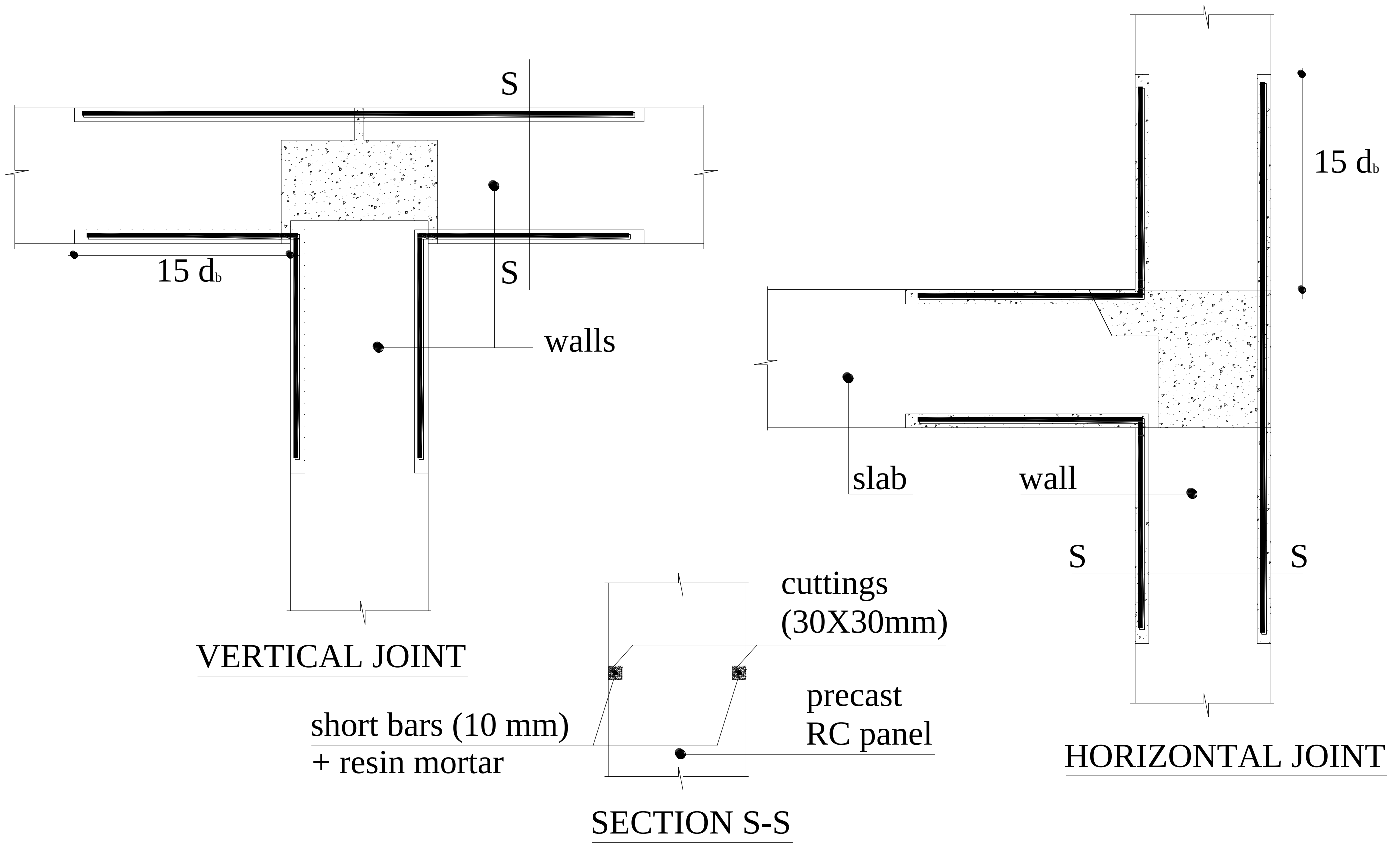
Element and/or System	Connection Type and/or Detail	Seismic Behaviour, Observed Damage	Interpretation	Structural Intervention (Repair/Strengthening)
3	**Vertical Bearing Elements, Walls - W**			
3.1 W **(or W+S)**	Vertical or horizontal joints between panels.	Cracking displacement.	Insufficient detailing against seismic action. Premature degradation of resistance mechanisms, depending on the actual connection (of various types).	Arrangement of large diameter and long dowels / anchors (“nailing”). Possible combination with “stitching” bars, see § 3.2 here below.  

3.2 W (or W+S)	Vertical or horizontal joints between panels.	Cracking displacement.	See § 3.1 here above.	“Stitching”, see details.

3.3 W	Wall elements simply arranged over floor slabs.  Example of reinforcement in a wall over a connection between two floor slabs.	Dislocation, cracking, local crushing at the ends of the walls (toes).	Eccentricities, concentrations of the action effects.	“Stitching”, see details.

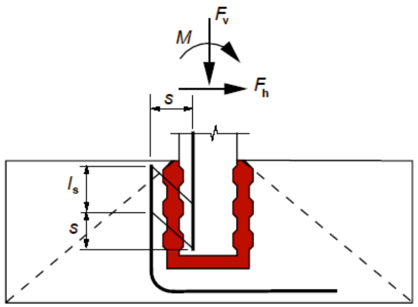
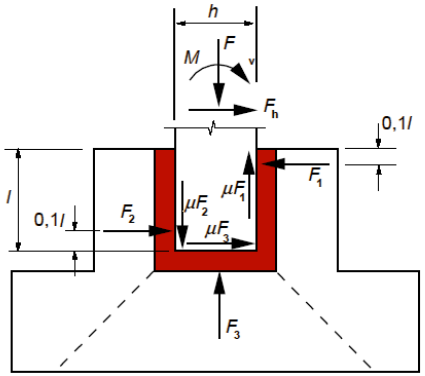
Special technique ("stitching")

In the case of damaged precast panel buildings, a special technique for repair / strengthening is reported (see CEB Bull. 162/1983); vertical as well as horizontal joints between large panels are reinforced by means of short steel bars embedded in cuttings (grooves) made in the panels and filled with resin mortar (sketch below).



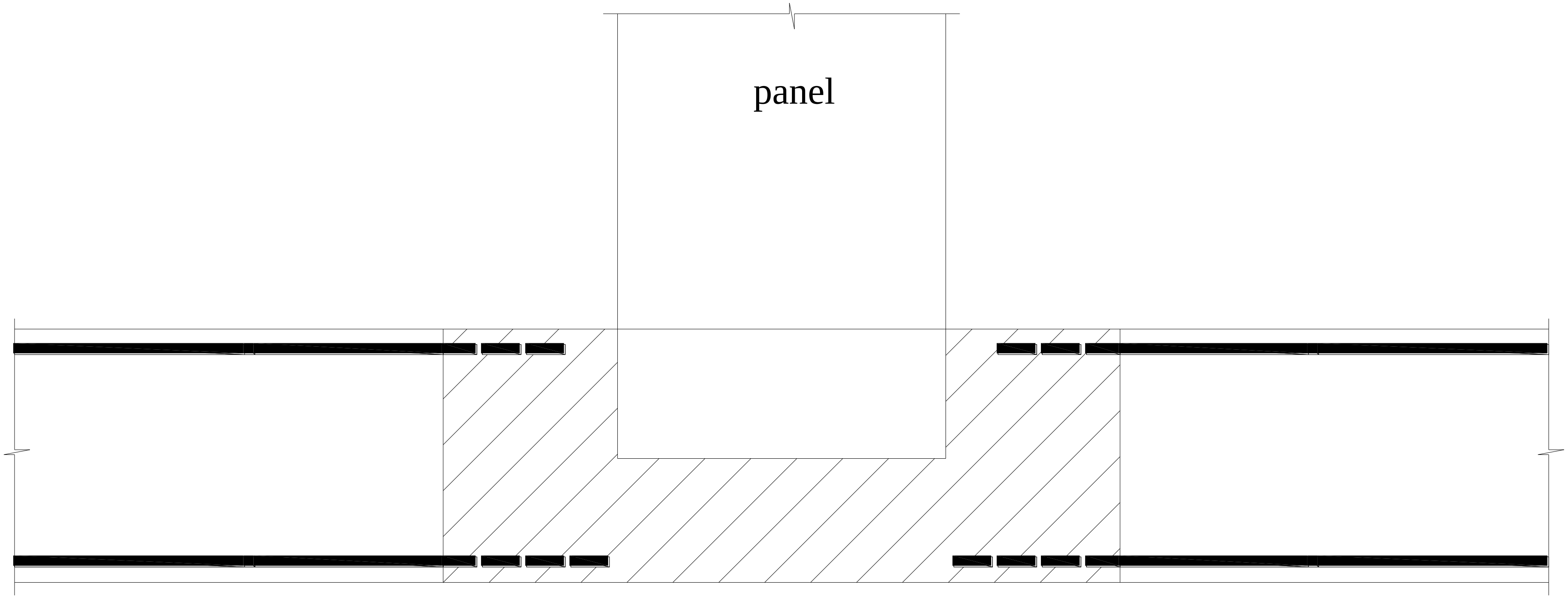
It is recommended that the cuttings should have a cross-section of approx. $3d_b$ x$3d_b$ (d_b being the diameter of the steel bar) and that the steel bars should have a minimum diameter of 10 mm; the cuttings should have a length outside the joint area at least equal to 15 d_b (d_b being the diameter of the steel bar). The spacing between steel bars can be estimated on the basis of the actions asting on the joints and on the constitutive laws of such short bars under tension and shear (bond action, dowel action and interaction between then), with a min. of 2 bars every 0,50 m (by turns).

Experimental data for panels with joints reinforced with such a technique show a considerable improvement of the behaviour of these panels under monotonic as well as cyclic actions (considerable increase in strength and ductility, without any significant increase in stiffness).

Element and/or System	Connection Type and/or Detail	Seismic Behaviour, Observed Damage	Interpretation	Structural Intervention (Repair/Strengthening)
4	**Foundations (for Columns or Walls)**			
4.1 F	Pockets and concrete filling. • Monolithic behaviour • Special design/detailing of the lap splices, with adequate horizontal / transverse reinforcement • Special design/detailing against punching shear, for full (a) or partial (b) shear transfer   a) With keyed joint surface b) With smooth joint surface	Cracking, dislocation.	Problems due to: • Detailing of reinforcement for F_1 and F_2 in top and bottom of pocket walls. • Transfer of F_1 and F_2 along the lateral walls to the footing. • Anchorage of main reinforcement in the column and the pocket walls. • Shear resistance of column within the	

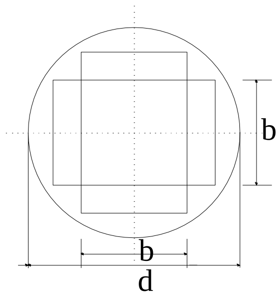
			pocket. • Punching resistance of the footing slab under the column force, the calculation for which may take into account the in situ structural concrete placed under the precast element.	

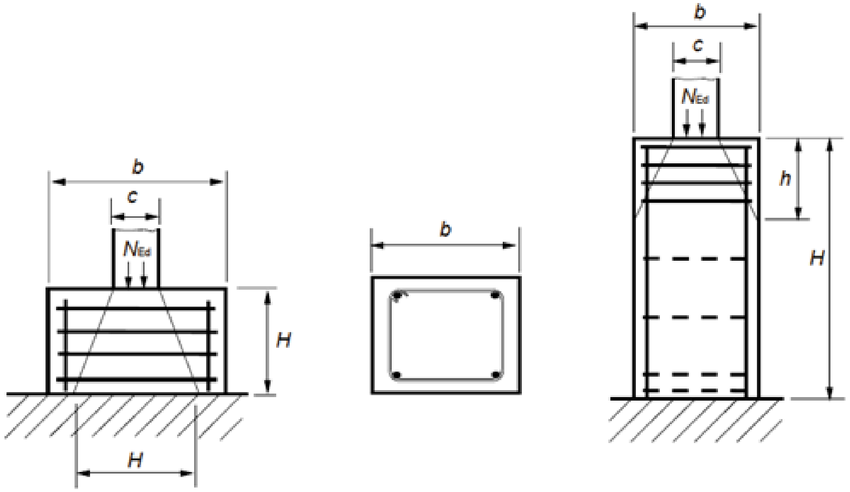
4.2 F	Isolated footing (e.g. pads).	Excessive rotation, distortion.	Sub-soil and/or foundation problems (deficient design, including conceptual one).	Arrangement of tying beams (in-situ RC and long dowels/anchors), in one or two directions, connecting adjacent footing or groups of them.



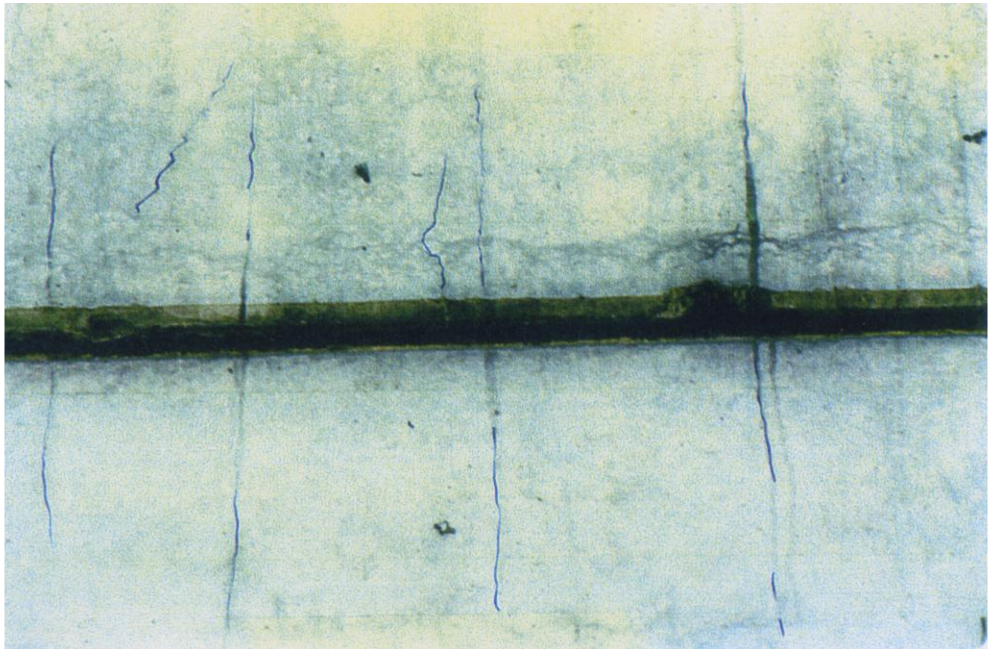
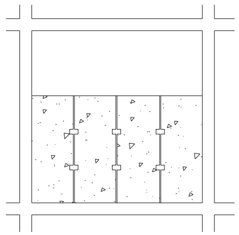
Arrangement of new, strong and stiff RC tying beams,

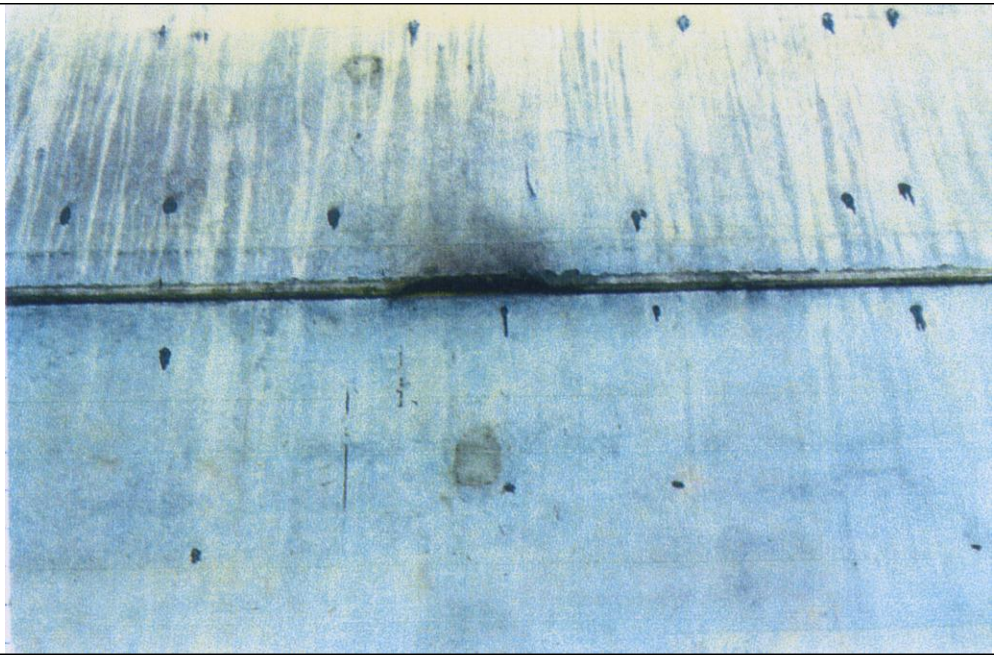
mainly to resist M & V, eliminating loading eccentricities & rotation.

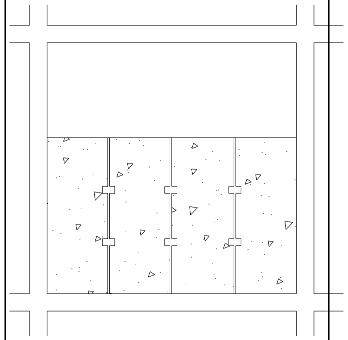
<table>
<tr>
<td>4.3 F</td>
<td>Isolated footing (e.g. pads) resting or “rock”, see, also, §§ 9.8.2 & 9.8.4 of EN 1992-1-1: 2004.


Orthogonal reinforcement in circular spread footing on rock, concentration in the middle, with b = (0,5 ± 10 %) d.


a) Footing with $h \geq H$ b) Section c) Footing with $h \leq H$
Splitting reinforcement in footing on rock, for h = min (b; H).
Splitting force: F_s = 0,25 (1-c/h) N_{Ed}.</td>
<td>Cracking, dislocation.</td>
<td>Excessive action effects, high ground pressure.
Tension at the upper or the lower part of the footing.</td>
<td>Repair and external tying or confinement.
Repair and RC jacketing.</td>
</tr>
</table>

Element and/or System	Connection Type and/or Detail	Seismic Behaviour, Observed Damage	Interpretation	Structural Intervention (Repair/Strengthening)
5	**Cladding Panels (CP’s)**			
5.1 CP	Thin and lightly reinforced panels. 	Cracking, failures, in-plane and out-of-plane.	“Self-response” as infilling panels under seismic action, or as deep (and strong) beams.	• “Isolation” (and special supports). • Repair and/or strengthening, with a detailed consideration and design (as per “infills”).  "slicing" (joints plus special connection)

5.2 CP	See § 5.1 above.	Damages of the panels and of the skeleton.	Global or local adverse influence & interaction, in plan as well as in elevation (“irregularity”).	• See § 5.1 above. • Application of special techniques, such as “slicing”.  “slicing” (joints plus special connectious)

Damaged captive column (interaction with the CP's).

EPILOGUE (or … PROLOGUE !)

Connections & Supports for Precast RC Elements

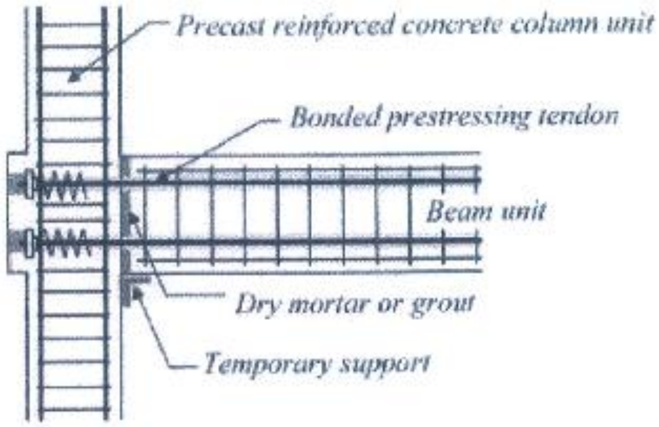
- **Materials shall be:**
  - **Stable & durable for the design working life of the structure**
  - **Protected against adverse influences**
  - **Chemically & physically compatible, and**
  - **Fire resistant to match the fire resistance of the structure.**
- **Connections shall be able to resist all combined action effects consistent with design assumptions, to accommodate the deviations and deformations, and to ensure a reliable and robust behaviour of the structure.**

  Verifications of strength, stiffness & ductility (?!) of connections & junctions shall be based on analysis & detailed design (et by means of struts-and-ties, for D-regions),possibly assisted by testing, taking into account all imperfections, eccentricities etc.

- **Premature cracking, splitting or spalling of concrete (combined with exposing of reinforcement) at the ends and edges of elements (of other regions) shall be prevented (or properly repaired, after erection), taking info account:**
  - **Relative movement between elements**
  - **Deviations & imperfections**
  - **Assembly requirements, and**
  - **Ease of execution, as well as of inspection.**

bulletin 27

state-of-art report



Seismic design of precast concrete building structures

2004

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guide to good practice

fib CEB•FIP

Structural connections for precast concrete buildings

2008

bulletin 63

guide to good practice

Design of precast concrete structures against accidental actions

2012