# Experimental research on wall-slab connections

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- Addition test: shear resistance of hcs
- Test layout
- Results
- Design recommendations

## ***Background***

- **Multi-storey concrete buildings, made of bearing wall units and pre-stressed hollow-core slabs (hcs).**
- **The ends of hcs with joint concrete transfer the loads from upper walls to lower ones.**
- **Simultaneously the slab ends are subjected to a negative bending moment until they crack**

## ***Background***

- To avoid unfavourable cracking different types of joints have been proposed and used in different countries.
- Fig 1 illustrates two alternatives with hypothetical cracking patterns

# Wall-slab connection

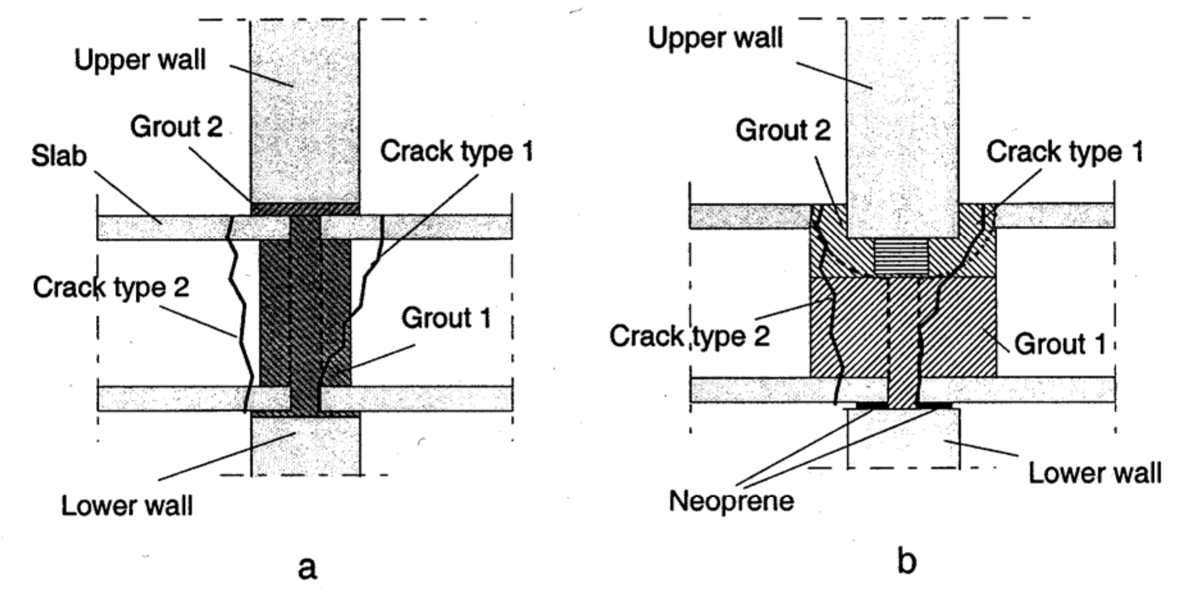



*Fig. 1. Different joints with possible favourable (type 1) and unfavourable (type 2) cracking modes. a) Finnish BES joint. b) Joint with notched slab ends.*

## ***Test Layout***

**3 tests: BES 1 simulated a BES joint; N1 and N2 simulated a joint with notched slab ends.**

**$l_{slab}$= 10 m**

**$b_{w}$= 0,2 m**

# Wall-slab connection

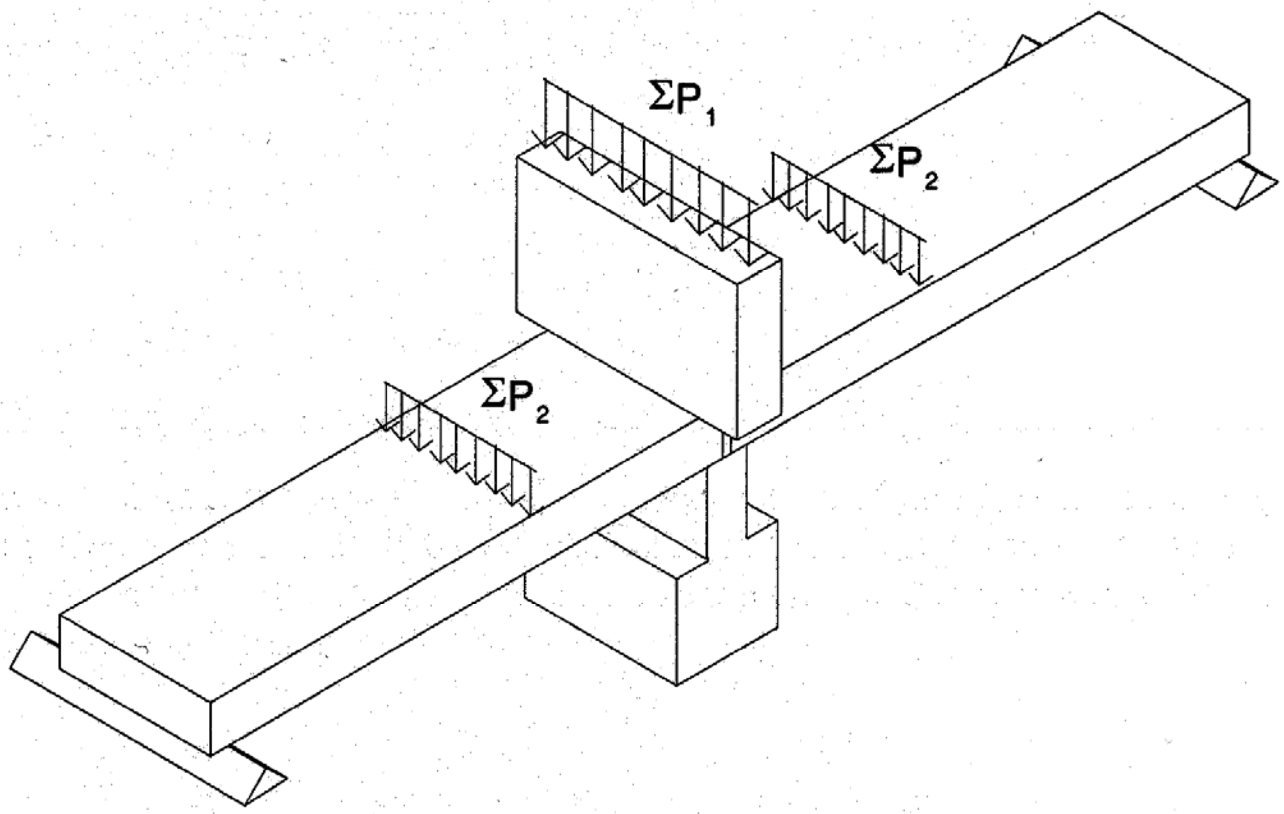



*Fig. 3. General view on test layout.*

# Loading arrangements

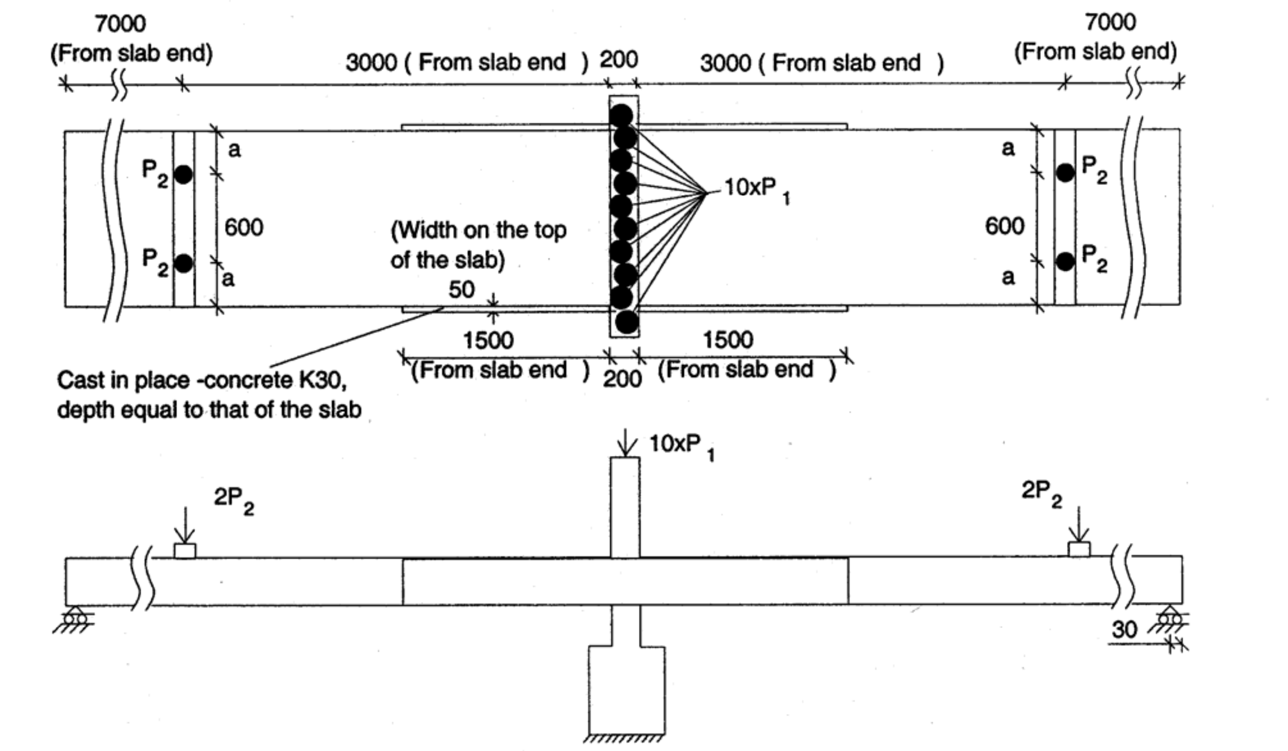


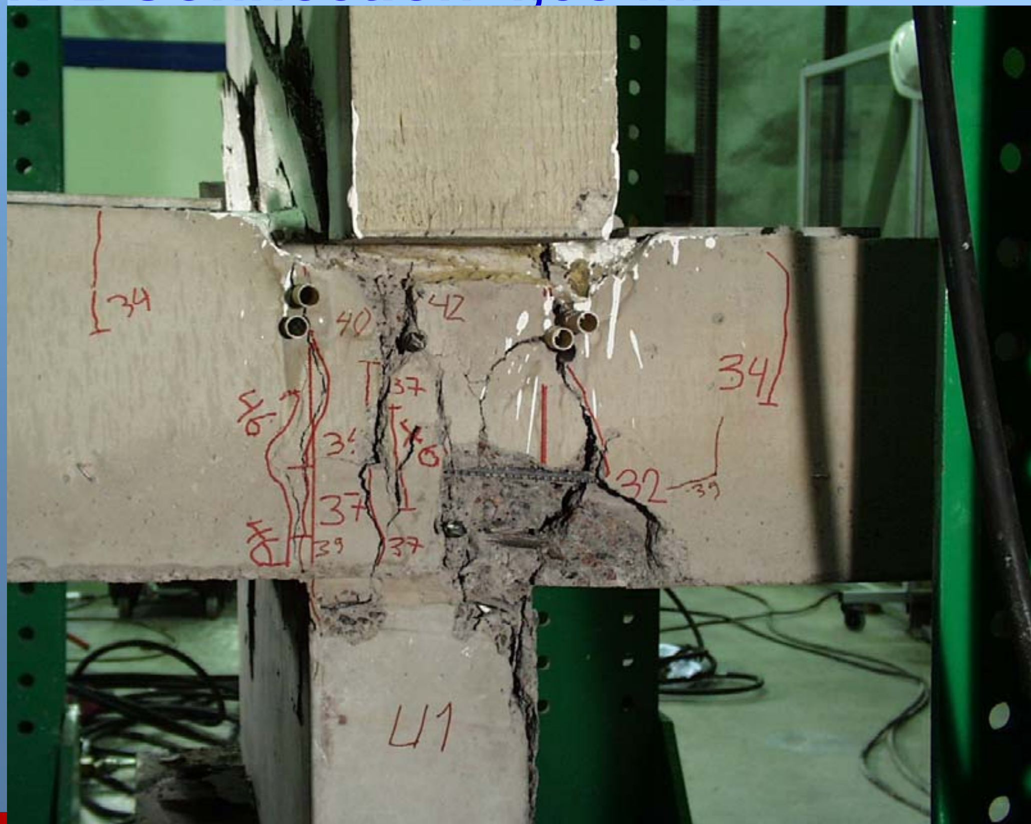

*Fig. 10. Loads.*

## *Results*

## BES 1 Connection 3,84 MN

## *Results*

## N 1 Connection 4,95 MN



## *Results*

## N 2 Connection 4,41 MN

# Shear resistance of hcs cracked due to unintended bending moment

**The aim is to study experimentally the shear resistance of the slab ends when they are in high vertical compression between wall units and cracked vertically outside the bearing**

# Wall-slab connection

- **Structural design rules have been used to eliminate the cracking mode shown in fig 1**

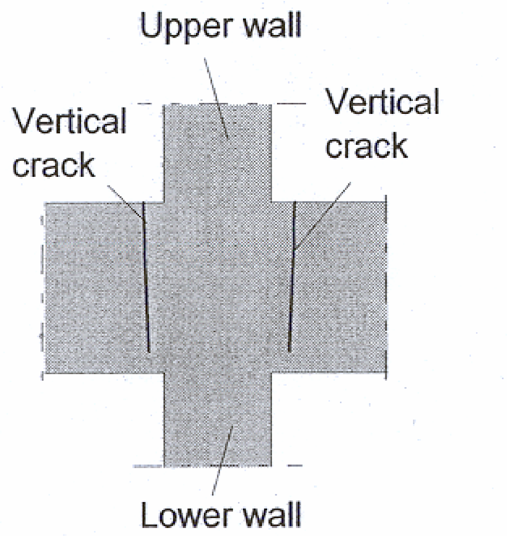



*Fig. 1. Vertical cracks in slab ends due to negative bending moment.*

## ***Background:***

- **Such cracking reduces the shear resistance of the hcs**
- **For low wall loads no risk of such a failure mode**
- **For heavy wall loads risk is obvious**

## *Test specimen, loads and measurements*

- Hcs 320, l=8000 mm, 11ø12,5, C50/60, C25/30
- Six tests were performed
- Three with typical Finnish BES joint
- Three with Swedish “ K-ended” joint
- Ducts for electrical wiring are excluded
- Vertical and horizontal displacements were measured

# Test specimen, loads and measurements

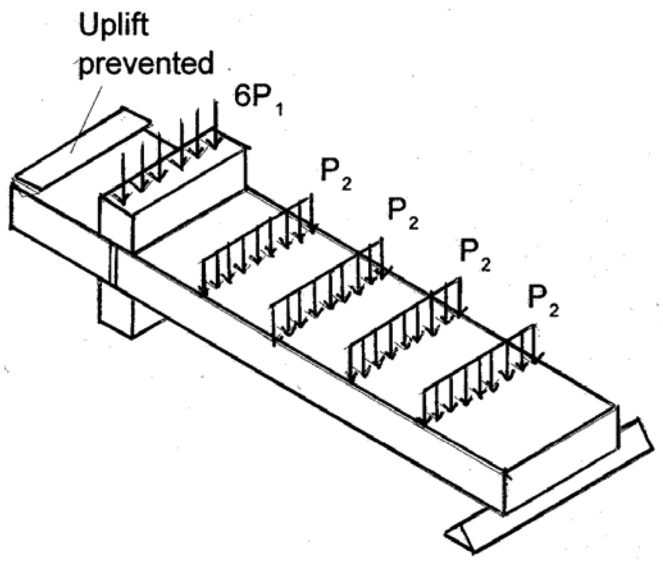



*Fig. 2. Overview on test layout. $P_1$ is kepth constant while $P_2$ is increased until failure takes place.*

## *Loading strategy*

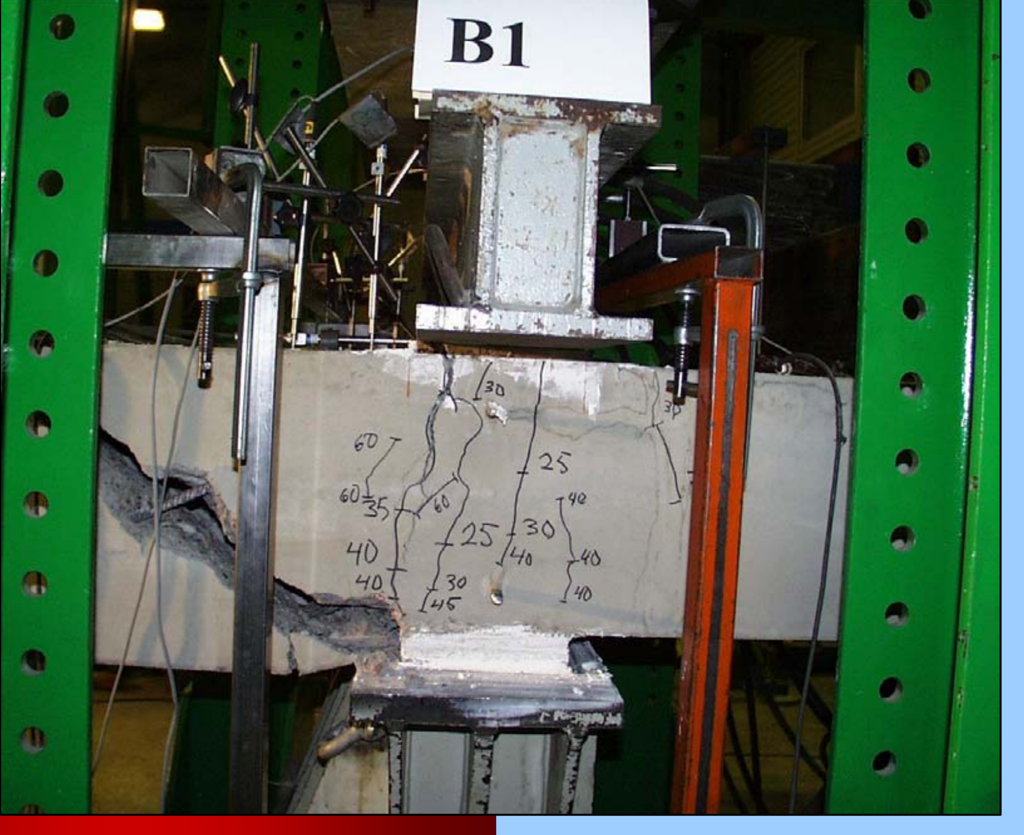
- 1. the load on the joint is increased to $6P_1$=1,6 MN. This load level is maintained throughout the test. Corresponds vertical load below 1.floor. Swedish office load (considering 17 floors+roof and snow load), assuming 10 m span and hcs 265
- 2. The loads $P_2$ on the slab are increased to $P_2=P_{crack}$

## *Loading strategy*

- **3.The loads $P_2$ are reduced to 0**
- **4. the load cycle is repeated four more times (0==>$P_{crack}$==>0)**
- **5. Finally the loads are increased gradually until failure**

## *Results*

**[B1] $V_{obs}$>247 kN**



## *Results*

**[B2] $V_{obs}$ =299 kN**

## *Results*

**[B3] $V_{obs}$ =308 kN**

# Wall-slab connection

## ***Results***

**[N1] $V_{obs}$ =288 kN, [N2] $V_{obs}$ =291 kN**

## *Results*

## [N3] $V_{obs}$ =293 kN

## *Design recommendations*

**Vertical force capacity of the joint can be calculated with the formula (1)**

$$\boxed{N_{Rd} = k\frac{f_{ck}}{\gamma_{joint}} b_j L_j}$$

## ***Design recommendations***

**$f_{ck}$= Characteristic compressive strength of wall or joint concrete (which one is smaller)**

**$\gamma_j$ = Safety factor of the joint (1,6 in class 1 and 1,8 in class 2)**

**$L_j$ = Length of the joint in wall direction 1 m**

**k= 0,5 in Case A (BES joints)**

**$b_j \leq b_{joint}$ or $b_{wall}$**

**k= 0,6 in Case B (K-ended hcs)**

**$b_j = b_{wall}$**

## *Design recommendations*

**The shear capacity of hcs / one slab width is calculated with formulas 2 or 3 (which one is smaller)**

$$V_{uv} = V_{u1} = 0{,}3k(1+50\rho) f_{ctd} b_w d + \beta_1 A_p \frac{F_{bup}}{P_{yd}} f_{pyd}$$

$$V_{uv} = V_{u2} = \mu\left( A_s f_{yd} + \frac{x_1}{l_{bp}} P_\infty \right)$$

## ***Design recommendations***

**$b_w$=Total width of the webs**

**d= Location of the joint bars from the soffit (bars anchored to full yielding force)**

**$A_s$=Area of reinforcing steel in the joints / one slab width**

**$f_{yd}$=Design strength of joint bars**

**$\rho=A_s/b_w d$, $\rho \leq 0{,}02$**

**$f_{ctd}$=Design tensile strength of hcs**

## *Design recommendations*

**$\beta_1$=0,9**

**$A_p$= Area of pre-stressing steel in hcs**

**$F_{bup}$= Anchorage force of the bottom strands at the distance x1 from the slab end**

**x1= Distance from the slab end of the hcs to wall surface (possible wall chamfers must be subtracted)**

## *Design recommendations*

$P_{yd}$=Design value of yielding force of the strands ($A_p f_{pyd}$)

$f_{pyd}$=Design strength of the strand

$\mu$=Friction coefficient 0,8

$l_{bp}$=Transfer length of the pre-stress force (bond factor according to sudden release)

$P_{\infty}$=Prestress force after losses

## *Design recommendations*

**The amount of joint bars shall not exceed the value $A_{smax}$ (formula 4)**

$$\boxed{A_{s\max} = \left[ \frac{f_{ctk} - \left( \sigma_{cp} + \sigma_{cg} \right)}{d f_{yk}} \right] W_y}$$

**Where**

***Design recommendations***

**$\sigma_{cp}$= Stress in the concrete at the top layer of the hcs due to fully developed characteristic prestressing force at the age of 6 months**

**$\sigma_{cg}$= Stress in the concrete at the top layer of the hcs due to self weight of the jointed slab at the distance of 0,5 $l_{bp}$ from slab end**

**$W_y$=Bending resistance regarding to top layer of hcs**

## ***Design recommendations***

- **Splitting reinforcement of the walls must be looked after**
- **Structural guidelines shall be followed:**

  **min. dimensions**

  **joint reinforcement**

  **electrical wiring**

  **Concrete grades, neoprene**