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The potential for direct reuse of precast concrete slabs in buildings with "wet" joints

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ABSTRACT: Today, many buildings are built with precast concrete slabs each year, and as per tradition, these slabs are joined by pouring in-situ mortar in the joints between the slab elements. This is typically done to ensure the structural performance when the entire level of a building needs to perform as one rigid body. However, the consequence of the mortar-joints is that future dismantling is difficult as it requires extensive and costly use of diamond saws. Therefore, disassembly and direct reuse of concrete slab elements are rarely seen. Instead, the concrete is often crushed and used for, e.g., road filling. The global focus on reducing carbon dioxide from cement production and challenges with limited natural resources means that the production of new concrete must be reduced in the future. Direct reuse of concrete elements is essential to this transformation. More recently, methods have been employed to apply "dry" mechanical joints (e.g., steel brackets) specifically to enable a simpler disassembling process after end lifetime of buildings. Several challenges exist for the dry connections, such as more complex elements, labour-demanding assembly, higher cost, and problems with local damages, fire protection and robustness in service. Despite the trend in development and research activities in dry connection joints, the opportunity of modifying the simple, traditional "wet" mortar joints to create Design for Disassembly (DfD) has only been superficially investigated. The article unfolds the potential of a new type of wet concrete joints with a review of the challenges and limitations. The proposed solution to achieve DfD for wet connections is to reduce the mortar strength and stiffness and disassemble by pulling with a crane.

1 INTRODUCTION

Concrete elements from precast buildings are rarely reused. In developed countries, concrete buildings are crushed after the end of life to be used as a substitute for gravel in road filling or, on rare occasions, as aggregates in new concrete. But the direct reuse of entire concrete elements is almost nonexistent even though there is a global potential for enormous savings in the emission of CO_2 by doing so. Eberhardt et al. (2018) showed that 80% of the CO_2 is saved when an entire precast concrete building is reused once.

Worldwide, the construction industry accounts for 38% of the emission of CO_2 (UN, 2020), whereas cement production alone is 5 to 8 % (Figueira, 2021). As the main contributor of carbon emission from concrete, cement accounts for about 0.9 $kgCO_2e$ per kg cement, depending on the specific cement factory (Hertz and Halding, 2021).

Furthermore, the resources to produce concrete are already scarce in many areas worldwide. Continuing the current production trend will accelerate the problems related to climate change and loss of biodiversity (UN, 2019).

To lower the consumption of cement for concrete structures, researchers investigate solutions such as "Green concrete" with less cement, minimal structures with less volume of concrete, and structural elements with combinations of different grades of concrete. However, considering the direct reuse of precast concrete elements is relevant in combination with any other technological gain in concrete buildings if the required forces can be transferred via the joints during the service of the building.

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In this article, the emphasis is on the direct reuse of concrete slabs. They are often made with a relatively higher compressive strength (more cement) and constitute a large percentage of the overall volume of a concrete element building. The building industry is often considered to be conservative. Therefore, the scope is to investigate solutions within the current practice regarding erection methods – this means creating "design for disassembly" solutions with cast joints ("wet" joints) between slabs and adjacent elements. The proposed solutions are at a conceptual level. An example of a typical pouring of mortar/concrete over the reinforcement in joints between precast concrete slabs is seen in Figure 1.

Figure 1. Ordinary pouring of joints in a precast concrete building.

1.1 *Existing DfD-solutions*

The primary research in DfD revolves around mechanical "dry" joints that can typically be disassembled by loosening some bolts or similar steel connectors. Examples of proposed mechanical connections for DfD of precast concrete buildings are presented by, e.g., Kang et al. (2013), Witzany et al. (2015), Aninthaneni et al. (2017), Xiao et al. (2017), Aninthaneni et al. (2018), Ma et al. (2019) and Balineni et al. (2020). All the above research has in common that the DfD solutions rely on alterations of the concrete elements and hence require new processes at the factory and building site.

The only "wet" joint DfD-solutions available in the literature and online are based on cutting with a diamond saw around the slabs' perimeter. Such methods are cumbersome and expensive, which is why it is not already done in practice.

2 BUILDING DFD-REQUIREMENTS AND LIMITATIONS

To investigate DfD-solutions for "wet" joints, limitations for the type of precast concrete buildings and a location have been chosen as an example. The case study building is positioned in Denmark, with low seismic activity and dominating wind load conditions.

Horizontal load from the wind on the façade is transferred to the stabilizing walls via the slabs and into the foundation. The magnitude of the transferred force to each wall is directly related to the wind pressure and the geometry of the building (the façade area, the position and sizes of the stabilizing walls, etc.). The wind load must be transferred via the joints between the slabs and the stabilizing walls. Hence, a conservative design approach is to dimension the structural system with the largest possible transfer of force in the connections, meaning the following two considerations are taken:

1. Apply the largest wind load on each floor (extra-large storey heights and a very long building).
2. Have the fewest possible stabilizing walls and use only short walls (increasing the shear stress in the joints).

2.1 *Case study*

2.1.1 *Transfer of maximum shear force to stabilizing walls in service*

A case study of a "conservative" building is made to show a critical case of load transfer from slabs to walls during use. Figure 2 shows a model of the structural system.

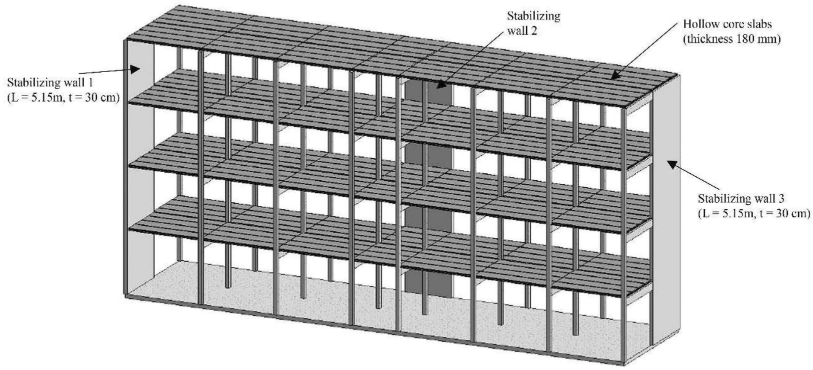

Figure 2. 3D view of the structural system of a precast concrete building with a maximum storey height of 5 m.

The building has a storey height of a maximum of 5 m, a façade length of 41.7 m, and only two transversely positioned stabilizing walls of 5.15 m in length positioned in each gable (stabilizing walls 1 and 3). This setup is more critical regarding the magnitude of transferred shear into the stabilizing walls compared to a similar building with less storey height, smaller façade length, more stabilizing walls, or with longer stabilizing walls.

Requirements for robustness (CEN, 2004) of the structural system means that reinforcement is positioned along the edges of the building and - when applying hollow core slabs - also to some extent in the joints between the slabs. Pure cement-based mortar is usually used in these reinforced joints along the perimeter of each slab, and the compressive cylinder strength is often above 40 MPa.

The slabs' size depends on the building's applied vertical loads for the case building. A maximum design load of 6 kN/m^2 is chosen, including safety factors, self-weight of slabs, services etc. It corresponds roughly to the expected imposed loads for an apartment. 180 mm hollow core slabs (HCS) are used to resist loads and deformations sufficiently, and a cross-section of the slabs and beams of the building is shown in Figure 3, including the pre-tensioning strand positions (red dots) and sizes.

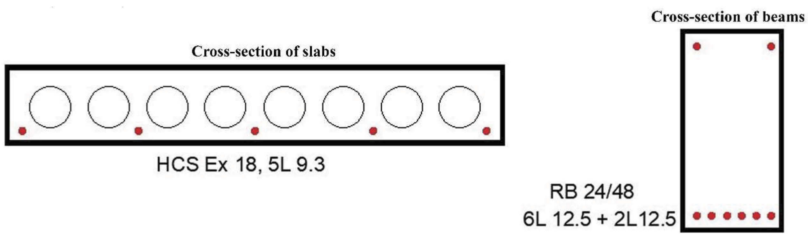

Figure 3. Cross-section of HSCs and rectangular beams in the building designed to resist the applied vertical loads, and self-weights.

According to the Eurocodes (CEN 2005), the wind load can be found as maximum positive and negative pressure on different building surface areas. When distributing the wind load on the façades as a line load on the floors, the force becomes 3.33 kN/m (except for the roof and ground floor). Since the case building façade is 41.7 m long, the total load from the wind on each floor is $3.33\ kN/m \cdot 41.7\ m = 138.9\ kN$.

This force is evenly distributed to the two transverse stabilizing walls at the gables, and each wall will receive 69.4 kN in each storey. It is now possible to investigate the occurring stresses in the local area with the mortar joint between slabs and stabilizing walls to determine the minimum requirement for the strength of the mortar.

2.1.2 *Local investigation of the required mortar strength*

FE-models were created to investigate the maximum occurring stresses in the mortar between slabs and stabilizing walls coming from the dominant wind load in combination with self-weight and imposed load.

Autodesk Robot Structural Analysis was used to model the geometry of the structural system of the entire building (see Figure 4 left). The material stiffnesses were given as input to the different members. This was to verify the assumption that the critical wind load would be evenly distributed to the two transversely positioned stabilizing walls. It was again assumed and modelled that the columns did not participate in resisting the wind load. The model also provided the maximum shear stress's location over the stabilising wall's length, which was utilized in a local investigation of the mortar joint between slab and wall.

The program ABAQUS was used to model the local (0.6 m x 0.3 m x 0.3 m) detail where the slab transfers the wind load via the mortar to the stabilizing wall. In Figure 4 (right), the simple ABAQUS model is seen. The right part of the figure shows an example of the distribution of the stresses in the mortar when the relevant load was applied to the local detail. The slab was only supported vertically by the stabilising wall, and all critical combinations of shear, bending moment and normal force from the slab were modelled to be transferred through the mortar.

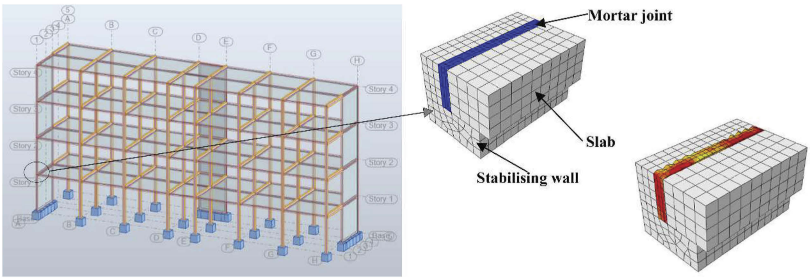

Figure 4. FE-model setup of global and local investigation to find the maximum stress in the mortar based on dominant wind load.

In the local FE-model, the stabilizing wall was fixed at the bottom.

To study the distribution of the stresses in the mortar and to find the largest stress, the model showed the numerically largest 1st and 3rd principal stresses in the mortar part of the assembly.

Based on this approach, the minimum required compressive design strength of the mortar could be determined directly when comparing different mortars with various strengths and stiffnesses, see Table 1. In all cases, the poisons ratio was 0.2, and the safety factor when calculating the design strength was 1.45.

Table 1. Mortar properties and maximum stress.

Compressive strength (char.)	Young's modulus	Tensile strength (char.)	Applied max stress	Compressive design strength
8 MPa	25.3 GPa	0.8 MPa	11.0 MPa	5.5 MPa
10 MPa	26.3 GPa	1.0 MPa	11.3 MPa	6.9 MPa
12 MPa	27.0 GPa	1.2 MPa	11.5 MPa	8.3 MPa
16 MPa	29.0 GPa	1.6 MPa	11.9 MPa	11.0 MPa

The small change in the maximum stresses was due to the small deviations in the stiffness. The goal was to determine the required mortar design strength to resist the maximum stress from the applied wind load. The setup from Figure 4 is very conservative since there will also be a contribution to the shear transfer in the surface between slab and wall, which has not been accounted for here. But with this conservative approach, the minimum limit for the strength of the mortar can be determined from Figure 5, where the “mortar design strength to max stress ratio” is plotted against the “mortar design strength”. The linear trendline crosses the ratio = 1 for a design strength of approximately 12 MPa, which corresponds to a characteristic strength of 17.5 MPa.

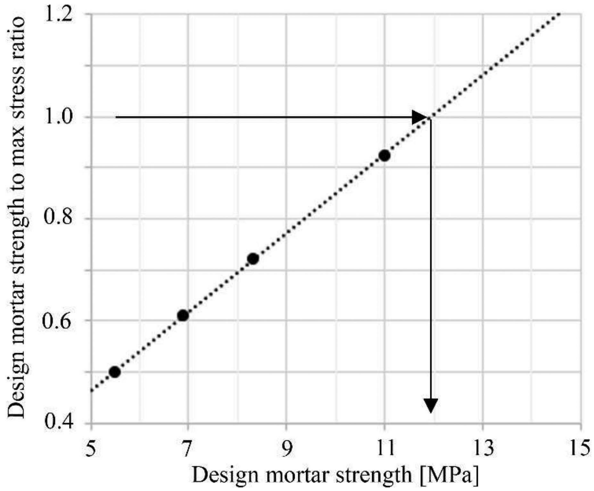

Figure 5. Mortar strength to max stress ratio with the minimum mortar strength limit location at 12 MPa.

If the slabs were higher, the maximum stress is expected to become smaller (if the mortar joint height follows the slab’s height).

The 17.5 MPa compressive strength of the mortar is less than half of what is ordinarily used for precast concrete buildings, and it enables the opportunity for DfD of slabs.

2.1.3 *Design for disassembly of slabs with “wet” (poured) joints*

The simplest method to remove whole slabs from precast buildings after end-of-life would be to pull out the elements one by one with a crane. The requirement for such method would be:

1. The slab can resist the pulling-out loading scenario and is free to move without significantly deforming adjacent building parts.
2. The crane has the capacity to pull with the required force in a controlled and safe way.
3. The lifting chains and the anchors embedded in the decks have sufficient capacity.
4. There is access to lifting anchors on the surface of the slabs.
5. The joints are weak and will crack along the slab’s edge when pulled up.

To ensure that the slab for disassembly is free to be moved up, one side of the slab must always be free. The edge beam on the side of the first slab to be lifted must be removed first.

Now, one side edge of the slab is exposed, and the longitudinal reinforcement (if present) must also be cut by a diamond saw in the joints in the two opposite corners of the free edge (Figure 6).

The lifting process can be initiated, and if the mortar in the three attached sides is weak enough, it will start cracking around the slab periphery.

Suppose the lift is performed with ordinary lifting chains. In that case, it can be conservatively chosen to use chains with an inclination of 45 degrees vertical and four lifting anchors positioned 300 mm from the ends of the slab and 150 mm from each side of the slab.

The shear resistance can be calculated in the interface and inside each of the materials (concrete and mortar). Along the smooth surfaces of the slab, the interface will be dimensioning for the shear resistance when pulling the slab up. At the ends of the slab, where the holes are located, the shear force must overcome the shear resistance of the mortar in the location of the holes.

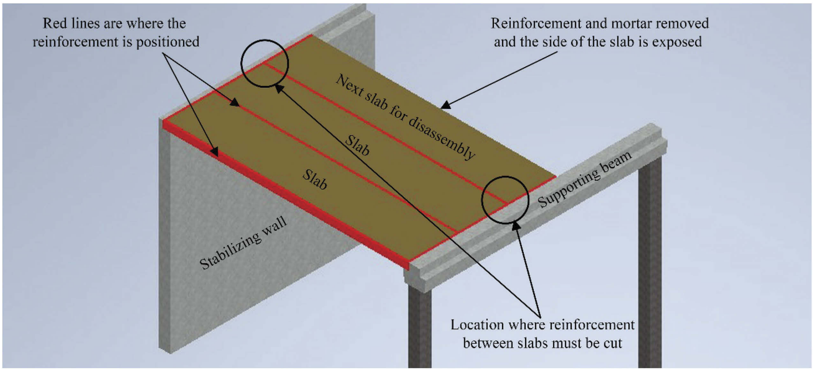

Figure 6. Disassembly setup with the location of reinforcement in mortar joints.

The shear resistance in the interface can be found by e.g., Eurocode 2 (CEN, 2004), and the calculation procedure is not shown here. For further information, the authors can be contacted. In the calculation of the shear resistance, it is assumed that there is no normal force in any of the joints (no wind during disassembly).

In the case building, the slabs have a 6.3 m span, a width of 1.2 m, and a height of 180 mm. Calculating the shear resistance and lifting force from the three sides with mortar joints of 12 MPa compressive strength (found earlier), the result becomes:

- Total shear resistance from holes at the ends of the HCS: 61.5 kN
- Total shear resistance from the remaining part of ends of the HCS: 44.9 kN
- Total shear resistance from the longitudinal joint of the HCS: 166.7 kN
- Self-weight of HCS to be lifted: 19.2 kN
- Total lifting force to overcome shear resistance and self-weight: 292.3 kN

The total load is possible to lift by a mobile crane. A solution for lifting is shown in Figure 7.

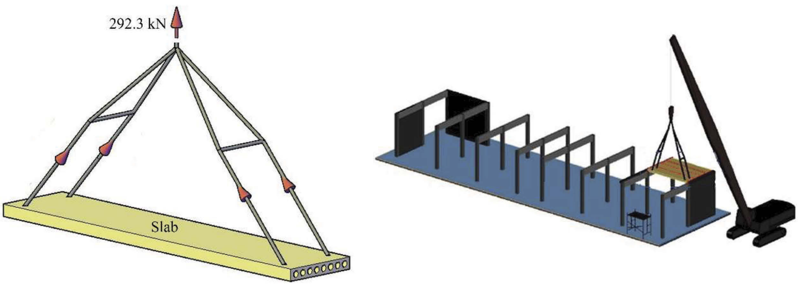

Figure 7. Disassembly setup with crane lift.

If no lifting anchors are available, it may be necessary to lift by applying another method, for instance, special lifting straps around the slabs. When this method is applied, the slabs and surrounding elements should always be checked for sufficient resistance to the disassembly load.

2.1.4 *Carbon footprint comparison*

Considering only the structural system of the case building, the direct saving of CO_2 by direct reuse of the slabs can be calculated. The concrete of the slabs accounts for 72% of the consumption of concrete in the building, and the concrete grade is higher (55 MPa) than for the columns and stabilizing walls (35 MPa). If we consider only the CO_2-emission from the materials manufacturing, the slabs account for 76.5% of the total emission of the system.

Of course, the dismantling and other processes require emission of CO_2 as well, so a more precise calculation is required when such numbers are available. Still, there is a saving of about ¾ of the emission available by directly reusing the slabs in a new similar building in the future.

3 DISCUSSION AND PROSPECTS

The erection method of the building will remain the same as for an ordinary precast concrete building, with the only difference being the application of a weaker mortar in the joints between the elements. The 12 MPa compressive strength mortar could be created from either a pure cement-based recipe or a combined lime- and cement mortar. Note that the hardening of the mortar is not complete after 28 days (ordinary design assumption). This means that the mortar is slightly stronger than anticipated at the end-of-life of the building if this is not accounted for.

Most of the shear resistance comes from the longitudinal side of the slabs. Therefore, it could be a future investigation to develop a method to “pre-crack” the joints along the longitudinal side to reduce the required lifting force. This could especially be interesting for larger slabs with a bigger self-weight and longer sides. The potential of such a solution could be to develop a disassembly method for slabs from existing precast concrete buildings with stronger joint mortars. This would fast forward the CO_2-saving so that society will benefit immediately.

Another promising future development of the DfD method could be creating a “hybrid” DfD-building with a combination of dry mechanical and wet joints. By doing so, it is possible to achieve the benefits of both methods in combination and create a simpler and more inexpensive way to assemble and disassemble.

The disassembly method also requires a check of the capacity of the slabs for the pulling-out load scenario. It may be necessary to put in slightly more pre-tensioning steel in some new slabs to avoid problems with cracking during disassembly.

To verify the proposed DfD-method, full-scale tests will have to be performed. Especially the sequence of the crane pulling regarding the safety of the workers on the building site and the possible concentration of stresses in the slabs or neighbouring elements are issues that need to be proved in the lab.

Many other aspects are important before we can begin to have large numbers of direct reused concrete slabs, e.g.:

- Documentation of the carrying capacity of the slabs and the remaining component life.
- Check chemicals to ensure contamination is not transferred to new buildings via slabs.
- Where to store the elements before they are reused in a new project.
- Evaluation of the economic consequences.
- Regulations that require reuse at an element level to lower future emissions of CO_2.
- All precast concrete buildings should be designed with a solution for both the erection and disassembly methods.

4 CONCLUSIONS

A new method for Design for Disassembly (DfD) of slabs in precast concrete buildings is proposed. The method relies on the mortar in the poured joints being optimized to have the minimum required strength and stiffness to transfer the required force from the load on the building during service. With the much lower mortar strength, the building can more easily be disassembled after the end of life so that the slab elements can be directly used in a new building without being harmed and without the costly and slow process of cutting all edges with a diamond saw.

The outlined method relies on an ordinary crane to pull out the slabs one at a time. The low strength of the mortar means that the mortar is the weak point in the joined structure, and cracks and later separation failure will occur in the joints. The elements should always have one free side to ensure that the slabs are not interlocked during the crane pull. Therefore, the method relies on removing reinforcement or edge beams along the side of the building before dismantling the first slab. Furthermore, the longitudinal reinforcement in the joints between slabs must be cut locally by using a small diamond saw to ensure a restraint-free removal process. With such a setup, the only resistance towards the crane pull is the shear capacity of the interface between the slab and the mortar.

A conservative case building with a critical loading scenario is presented to show the method. An FE model of the local detail between slab and wall provides a minimum mortar compressive strength of 12 MPa. This strength is determined to be sufficient to transfer the wind load.

Subsequently, the required crane pulling force is found by calculating the shear resistance around the three sides of the slab with a mortar joint. The total required pulling force is 292 kN, which is manageable for a normal mobile crane.

The case building clearly shows that the method is a promising step towards an easier process for disassembling concrete decks in future precast concrete buildings to create CO_2 savings (up to ¾ for future buildings from slabs alone). Nevertheless, several obstacles must be overcome to apply such a method in practice. For instance, full-scale laboratory tests are necessary to prove the concept and to show that the method is safe for the workers on the building site.

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