Avoiding failures during building construction using structural fuses as load limiters on temporary shoring structures

Manuel Buitrago[a], Juan Sagaseta[b*], Jose M. Adam[a]

[a]*ICITECH, Universitat Politècnica de València. Camino de Vera s/n, 46022 Valencia, Spain*

[b]*Department of Civil and Environmental Engineering, University of Surrey, GU2 7XH Guildford, UK*

* Corresponding author. Tel.: +44 1483 686649; fax: +44 1483 682135.
E-mail address: j.sagaseta@surrey.ac.uk (J. Sagaseta).

Abstract

The risk of structural failure of buildings can be significant during construction. Temporary adjustable telescopic steel shores or props are commonly used in building construction. The failure of shores is sudden and therefore structural fuses as load limiters (LL) can be introduced to provide ductility in the temporary member for a specified limit failure load. Previous work by the authors showed that the design of shoring systems can be improved using LL for standard cases of imposed loads applied during construction. This paper extends this work to cases of accidental loading where the shoring system-permanent structure interaction is less known. The main principles of LLs are discussed and implemented in advanced numerical simulations of a real case RC building during construction by means of explicit nonlinear dynamic finite element analyses. Different local failure scenarios were investigated corresponding to cases observed in practice. The comparison of the numerical results obtained with and without LLs demonstrated for the first time the benefits of using LLs in terms of: a) mitigating the risk of failure of the temporary structure; and b) reducing permanent damage (cracking and short-term deflections in the slab) affecting the durability and functionality of the building.

Keywords*: Building; Damage; Load limiter; Progressive collapse; Slab; Steel Shore; Structural fuses.*

1. Introduction

Construction is one of the most critical phases in the life cycle of buildings [1–8] due to the risk of failure and the possibility of underestimating construction loads as shown in [9]. Recent review of failure reports from CROSS in 2018 [10], based on over 600 reports mainly from the UK, indicated that in 38% of the failures reported the cause was related to the construction stage, 36% to the pre-construction stage due to design, 25% during normal use and 1% during demolition. Analysing the causes of specific structural failures and proposing measures to mitigate their effects is an effective measure to reduce risks and improve safety of buildings.

A critical stage during construction is the procurement of the shoring (propping) system including the design, assembly and striking of shores. Codes of practice and guidelines on temporary works have historically focused on key aspects such as communication between the designer, supplier and Temporary Works Coordinator TWC [11], stability and overall design philosophy where members are designed for high loads for short periods of time. Considerations on progressive collapse of temporary shoring are currently being considered in guidelines for design of temporary works (e.g. [12]) with the idea of avoiding local failures that could have severe consequences. This focus follows the international concern on structural robustness [12] and the latest developments in this area captured by international codes for permanent structures [13–15]. However, some guidelines [16] suggest that formwork systems have normally sufficient built-in rigidity to distribute loads to the shores after accidental events. This was demonstrated by the authors in [17] using advanced simulations also showing that the rigidity and redundancy of the shoring system was able to prevent any dynamic amplification in the structure/shoring system.

This raises the question of whether shores should be designed to resist local failure in accidental events; some guidelines mention that this would be uneconomic [16], the results in [17] support this. However, it is recognised in [16,17] that the progressive collapse of the shoring system can occur and the permanent structure might deform excessively in accidental cases leading to cracking and permanent deflections which can affect the service life of the structure.

The main cause of temporary and permanent structural failure during construction (including progressive collapse) is the failure of the shoring system due to excessive loads on shores [9].

In order to mitigate the risk of failure or partial damage of the shoring/structure system during construction this paper investigates the use of structural fuses as load limiters (LLs) to be installed on temporary shoring systems acting as structural fuses and changing the mode of failure of the shore; patented solutions exist to LLs for example ES2636833 [18]. The main idea behind the LLs is that the working load in the shores is kept below and allowable load (See Section 3 for more details). This paper shows novel work investigating the ability of LLs to arrest the propagation of failure of the shoring system under accidental events and mitigate the potential damage on the shore/structure system.

This work focuses on the construction of RC buildings by shoring of successive floors [19–25], including a shore clearing process (SCS: Shoring/Clearing/Striking). The SCS approach was adopted in this work as the shoring loads are larger than those using other construction methods without intermediate operations (e.g. SS: Shoring/Striking), or with reshoring (e.g. SRS: Shoring/Reshoring/Striking) [19]. Section 2 of the paper contains a discussion on failures during construction highlighting trends and severity of consequences which are addressed in subsequent sections. Section 3 discusses the role of LLs in accidental events and how they could be implemented in the numerical analyses. Section 4 gives details of the numerical analysis of a real case RC building (shore/structure) including the predicted propagation of failure in cases with and without LLs installed on shores. The analysis includes also a systematic risk analysis to discuss the raw and mitigated risk using LLs. The main conclusions are drawn in Section 5.

2. Failures during construction

Many studies have analysed cases of structural failures looking at their causes and providing recommendations or mitigation measures to avoid their repetition. In many cases these reports are confidential although in some other cases the findings are filtered by international associations to alert practitioners on relevant aspects related to safety. The learning acquired from these studies (learning from failures) is at present a very active line of research [26–28]. Studies focusing on accidents during construction include [9,29,30].

In many of these reported accidents the failure occurred in the shoring system. An example is shown in Fig. 1, in which many of the shores buckled due to overloading. Shore buckling, together with pin deformation or breakage, are the most frequent mode of failure observed [31]. It is also observed that other shores in Fig. 1 that appear to be undamaged did not carry any load after the incident. In the case shown the permanent structure did not collapse, however other cases have been reported in [9] where such incident have led to the progressive collapse of the entire structure. The concept of progressive collapse is understood as the process by which local damage sets in motion a chain of failures, leading to the collapse of the entire structure or a large part of it [32].

Fig. 1. Shore buckling during the construction of a building structure.

A potential issue in cases as the one in Fig. 1 in which the integrity of the permanent structure was not affected is the consequences during the service life of the building. A serious concern in many of these cases is that the initial damage might not be detected and the structure will be put into service without any repair being considered. This scenario is plausible in cases where there is a lack of supervision. For example, an investigation in the UK commissioned by the HSE [33], which included site investigations and interviews with those involved in procurement, highlighted a concerning lack of adequate checking and erection accuracy. Allowing elements with early-age concrete to crack due to overloading during construction, as often happens, may cause excessive instant and long-term deflections and reduced strength of the structure. An example is given by Whittle [34] where a flat slab in a hotel built in the 1970s had excessive deflections due to sagging of the formwork and early striking; during some refurbishment works 20 year after it was built, it was found that the structure was unsafe and costly remedial work were needed. The work presented in this paper is aimed at avoiding such accidental situations by introducing LLs; the fundamentals of LLs are described in detailed in the following section.

3. Load limiters on shores: description and simulation

The concept of a load limiter on shores was conceived with the aim of avoiding failures during construction and reducing the risk of collapse [35,36]. The main idea behind the LLs is that the working load in the shores is kept below an allowable load. In accidental situations studied in this paper this means that the most heavily shores reach a constant load (limit load) and the excess load needed to withstand the event is redistributed to neighbouring shores acting as a group. In this way, sudden failures of the shoring system, which can often lead to severe consequences, are avoided or mitigated as demonstrated in subsequent sections.

Fig. 2 shows a shoring scheme commonly used in practice consisting of shores, joists and formwork boards as well as an example of a particular device [36] acting as LL assembled in an adjustable telescopic steel shore. The type of shore considered in this work is formed by inner and outer telescopic tubes, a fine-adjustment thread for altering shore height, and a ring to transmit the load from pin to thread. The LL shown in Fig. 2, as a particular development of LLs, is formed by a pair of connected elements installed between the shore pin and ring that interrupt and control the transmission of the shore load. The technical and economic viability of LLs was demonstrated in Buitrago et al. [35], who also showed its capacity for improving construction safety, temporary shoring system costs and structural efficiency for normal construction loads.

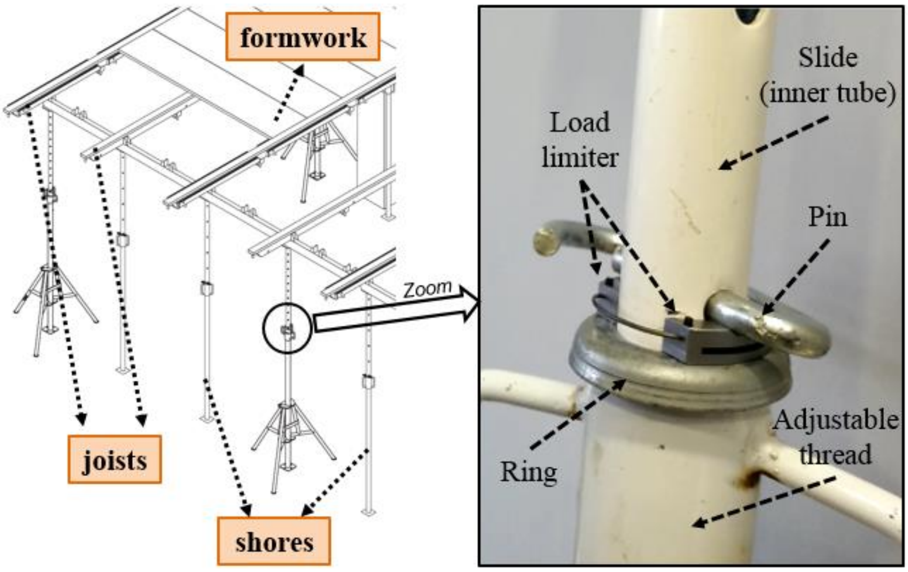

Fig. 2. Sketch of the shoring system and detail of a shore with load limiter.

The LLs investigated in this work were designed to initiate plastic behaviour at a certain limit load and allow a controlled vertical displacement (δ) of the shore with the excess load. During the controlled descent the shore remains operative avoiding the overload and the possible failure of the shore. The start of plastic behaviour involves the formation of three plastic hinges on the upper part of the LL device. Fig. 3 shows the LL behaviour before and after reaching the limit load by means of a simplified structural model. For safety and functionality reasons, plastic deformation in the LLs is limited to a maximum value, in this case this is equal to the height of the LL slot (see Figs. 2 and 3). Once the maximum plastic deformation is reached the shore is reactivated (i.e. is able to carry increasing load until it fails) following a linear elastic behaviour similarly as for loads below the limit load. Fig. 4a gives an example where the LL has reached the maximum permitted plastic displacement (i.e. contact starts between the top and bottom of the slot).

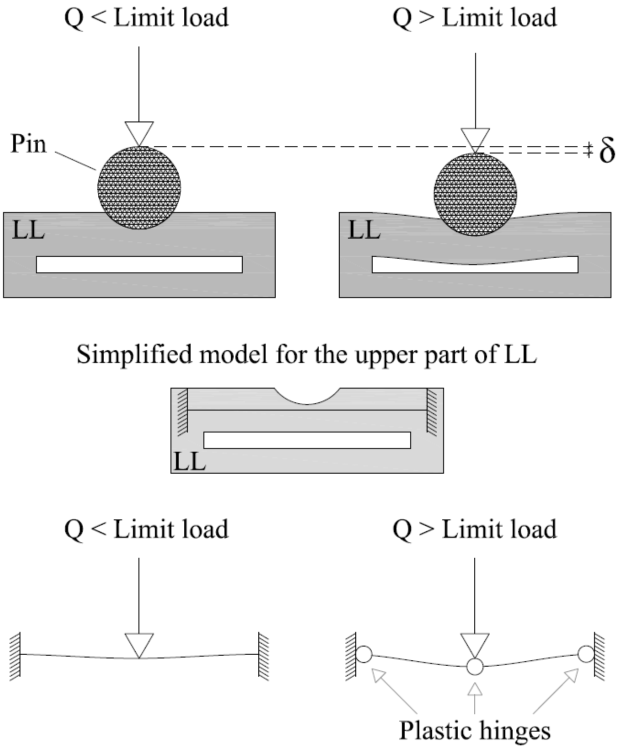

Fig. 3. LL conceptual behaviour before and after reaching the limit load.

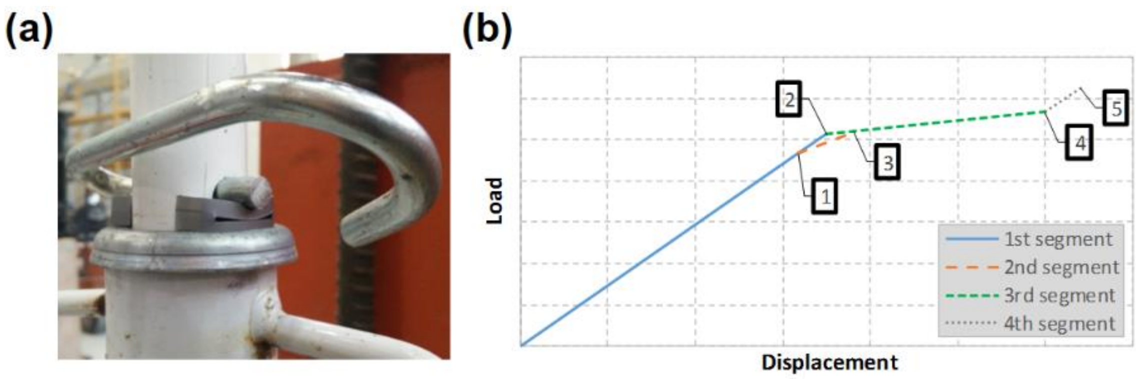

Fig. 4. a) Load limiter after reaching the maximum plastic displacement (slot height), and b) simplified LL on shore behaviour adopted for macro-scale models.

The response of LL devices can be implemented in structural analysis models. Fig. 4b shows the simplified qualitative and parameterised load-displacement behaviour of the shore-LL system. This simplified behaviour was adjusted empirically based on an extensive test campaign [36]. The model can be used for macro-scale numerical simulations. The different segments of the simplified load-displacement curve are defined systematically as follows:

- Point 2: Defines the limit load of the LL (q_y). The first segment has a slope (stiffness) of $E'A/L$ considering a small reduction of the elasticity modulus (E') from 210GPa to 190GPa in order to include the LL effect, and the area and length of the shore (A and L respectively).

- Point 4: Defined as the maximum plastic displacement in the LL. The third segment has a slope equal to 15% of the initial slope ($0.15 \cdot E'A/L$).
- Point 1: Determined by a load equal to the limit load (q_y) reduced by 1.1 ($q_y/1.1$).
- Point 3: Intersection of the second and third segments. The second segment has a slope equal to 57.5% of the initial slope ($0.575 \cdot E'A/L$).
- Point 5: Ultimate load of the shore (q_u). The fourth segment has a slope equal to the initial slope.

The load-displacement curve described above was adopted in the advanced numerical simulations in subsequent sections (path 1-3-4-5). These analyses were carried out to assess the propagation of failure of the shoring system after different accidental scenarios. The following aspects were considered in the analysis: non-linear behaviour, sudden removal of critical shoring members, contact modelling between structure and shoring system and realistic modelling of the LLs. It is worth noting that alternative LLs to the one investigated herein are expected to lead to similar conclusions assuming that the LLs share similar principles to those described in this section.

4. Failure and damage mitigation using structural fuses as load limiters on shores

This section contains a comparison of the results from structural analyses of a building during construction with and without LLs subjected to different accidental events. The authors had previously studied the effects of sudden failure of shoring elements in RC building structures under construction without LLs [17]. This previous work focused on scenarios of severe shoring system failure including: a) progressive collapse of shoring or even of the entire shoring system, and b) severe extended damage to the permanent RC structure. This section deals only with the most severe damage scenarios observed in [17]. The following subsections include: a description of the building and the shoring system used in the study, b) a description of the FE model including the LLs, c) failure scenarios considered, and d) discussion of the results.

4.1. Building structure and shoring system considered

A study was carried out on an actual building designed in accordance with Eurocode 2 [15], and described in detail in *Concrete Society* [37]. The structure consisted of 300mm thick RC flat slabs, 3.5m inter-floor height (3 floors) and 40×40cm^2 cross-section irregularly distributed columns (see Fig. 5). A full description of the building, which was previously investigated in other studies, can be found in Buitrago et al. [17] and Olmati et al. [38]. Fig. 5 contains a 3-D view of the building and a plan view of the shoring system of one of the floors.

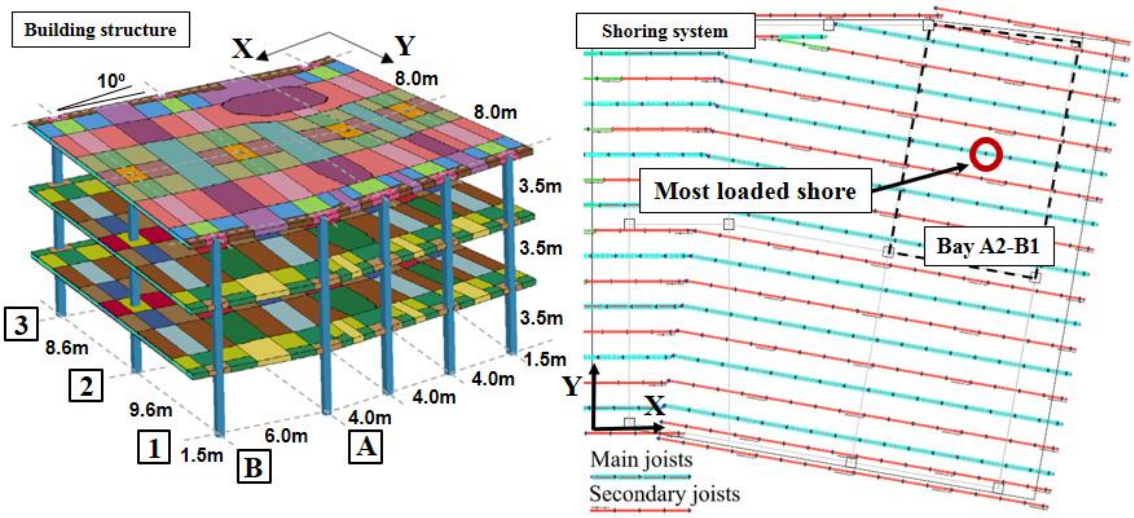

Fig. 5. Building geometry and the shoring system considered.

The temporary steel shoring system adopted in this work was identical to that designed by Buitrago et al. [17] so that a direct comparison could be made with the same case but using LLs. The construction process considered was also similar, including a clearing of 50% of the shores belonging to the secondary joists in Fig. 5 (see also Fig. 6a), with three consecutively shored floors (2 cleared and one totally shored). Joists and shores were separated by a distance of 1 m and the inter-joist distance was 2m after clearing; a new floor was poured every 7 days. The construction method adopted in the analysis was Shoring/Clearing/Striking (SCS) [19] as it was thought to be more critical compared to other approaches (i.e. the shores are more heavily loaded). In this construction process each floor has three operations: i) installing all shores or props for the concreting (shoring), ii) after few days removing only the 50% of props and all the formwork (clearing – removed props were those under the secondary joists as represented in Fig. 5), and iii) removal of the complete temporary structure of the floor as the final step (striking).

The selected shore [39] had a maximum strength of 47.7kN (slightly higher than the expected maximum load of 47.6kN) following the design approach described in [17] considering the different construction stages. The most heavily loaded shore was on the ground floor after pouring the third slab when the concrete was 14 days (compressive strength of 34.3MPa) and 7 days (compressive strength of 29.6MPa) old on the first and second floor respectively. Fig. 5 shows the position of the most heavily loaded shore placed in bay A2-B1. The maximum loads on shores are usually found on the ground floor (in contact with the foundations) at the time of placing the concrete at the highest floor [24]. Different scenarios were modelled for this most unfavourable position in the shoring system and compared with the same model with LLs installed on the shores; the results are discussed in Sections 4.2, 4.3 and 4.4.

4.2. Description of the FE model

The FE model adopted for the building is shown in Fig. 5 and Fig. 6a with two cleared levels and one totally shored. The model of the permanent structure had been previously verified against a similar FE model reported in [37]. The numerical simulations were performed using LS-DYNA [40], with a structural analysis in the time domain by means of an explicit algorithm and considering the material and geometric non-linearities. All the shores had compatibility of displacements and free rotation (as hinges) in the upper and lower nodes. The lower nodes of the shores on the ground floor also had restricted displacements. Joist-slab, joist-formwork boards and formwork board-slab connections were modelled as contacts. Shell elements were used for slabs and formwork boards, while beam elements were used for columns, shores and joists. Further details on the FE models of both the permanent RC structure and temporary shoring system can be found in Buitrago et al. [17] and Olmati et al. [38].

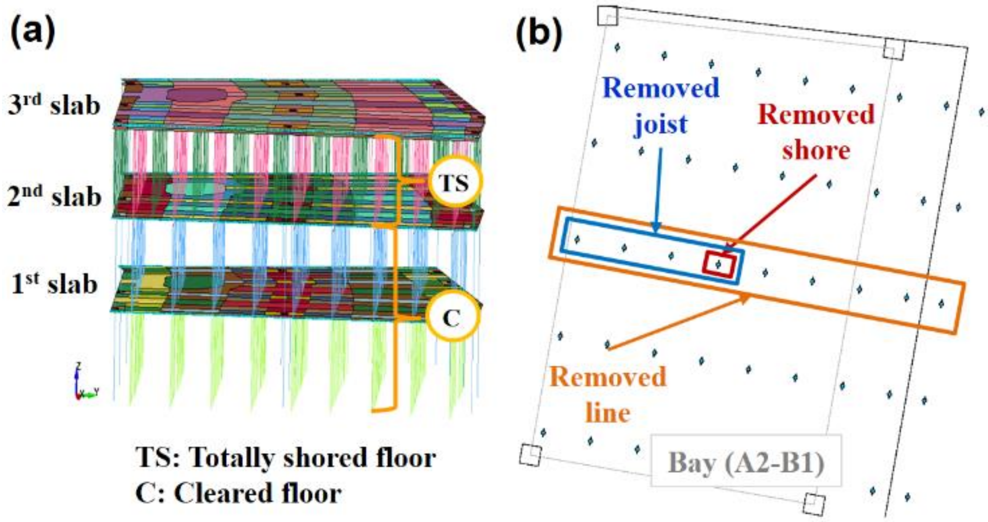

Fig. 6. FE model (a) and defined scenarios of sudden failure of ground floor shores (b).

The steel shores were modelled using Hughes-Liu beam elements with cross section integration and the piecewise linear plasticity model for the material [40]. For the shores without LLs, a linear elastic behaviour was considered up to a brittle failure defined by their strength. This captured the sudden failure of the shore when reaching the maximum load, as observed experimentally in [31]. For the shores with LLs, the material model was adapted to include the load-displacement curve defined in Fig. 4b following Section 3 with specific values according to the different local failure scenarios considered (see Section 4.3 and Fig. 6b). In the third local failure scenario in Section 4.3, with 47.7kN strength shores, the selected LLs had a limit load of 40kN (Fig. 7a) whereas in the fourth scenario, with 30.6kN strength shores, the limit load was 25kN (Fig. 7b).

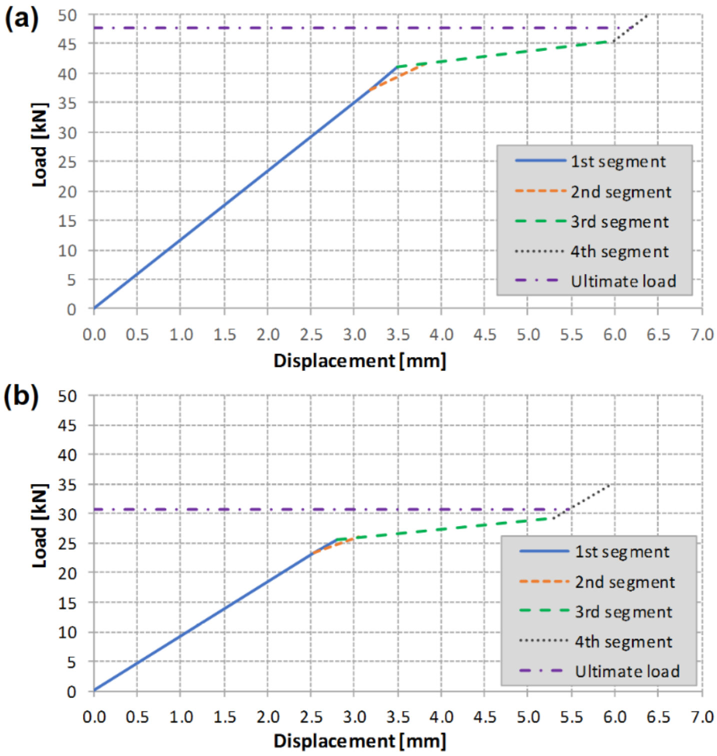

Fig. 7. Load limiter-shore behaviour using: (a) load limiters of 40kN and shores of 47.7kN strength for the third failure scenario, and (b) load limiters of 25kN and shores of 30.6kN strength for the incorrect selection of shores scenario.

The dead load was applied in the FE model as the self-weight of the different elements (densities of 25kN/m^3, 5.3kN/m^3 and 78.5kN/m^3 were adopted for concrete, timber and steel respectively). The live load was applied as a uniformly distributed mass on the slab, with a value of 1.0kN/m^2 representing a load due to personnel only [41]. Self-weight of the shoring system was automatically taken into account in the FE model. The load safety factors corresponding to accidental load combinations were considered using the Eurocode [42] (i.e. 1.0 and 0.5 for permanent and live loads respectively). The gravity acceleration was introduced gradually over time between t=0.0s and 0.8s, similarly as in Olmati et al. [38] and Buitrago et al. [17]. This was followed by a stabilising-time interval after which different failure scenarios were introduced in the bay investigated (A2-B1).

4.3. Failure scenarios considered

In the previous study by Buitrago et al. [17] four local failure scenarios were investigated:

1) failure of the most heavily loaded shore (Fig. 6b).

2) failure of the shores of the joist placed on the most heavily loaded shore (Fig. 6b).

3) failure of the complete shore line on the most heavily loaded shore (Fig. 6b).

4) incorrect choice of shores during design or construction.

The first three cases consider the sudden removal of shores, using the concept of notional member removal (see Fig. 6b). This approach is commonly used in design against progressive collapse and in international codes for permanent structures [13–15] as well as research [38,43–48]. The fourth scenario in [17] adopted shores with a strength (30.6kN) marginally below the strength of the shores used in the other scenarios, and well below the required strength of 47.6kN. The four failure scenarios are based on plausible design and construction situations observed in different failures during construction as described in [17].

In this paper, the third and fourth failure scenarios were considered in the analysis since it was shown in [17] that the shoring system in these cases suffered progressive collapse whereas in the first and second scenarios progressive collapse of the shoring system was arrested and only some minor damage was observed in the second case. In none of the scenarios investigated in [17] the integrity of the permanent structure was compromised although for scenarios three and four the local damage in the permanent structure was higher leading to a situation where the structural safety would need to be assessed to determine possible repairs.

4.4. Results and discussion

4.4.1. Sudden removal of a shore line (3rd failure scenario)

Fig. 8 shows a plan view of a sequence (Δt = 0.1s) of the progressive collapse of the shoring system on the ground floor in Scenario 3 (see Fig. 6b), which occurred at t=1.1s. The framed shores (highlighted in red) are those that disappeared (collapsed) in the next sequence. It can be seen that withdrawing a complete line of shores causes the progressive failure of other shores. Figs. 9a-b show the structure and its displacements before and after the accidental event respectively, for the case when LLs were used, while Fig. 9c shows the results with LLs. In the latter case the progressive collapse of the shoring system is arrested (Fig. 9c). The results show that the LLs in the shores were able to limit and keep the load below the permitted level and redistributed the excess load to the neighbouring shores (also equipped with LLs) as intended.

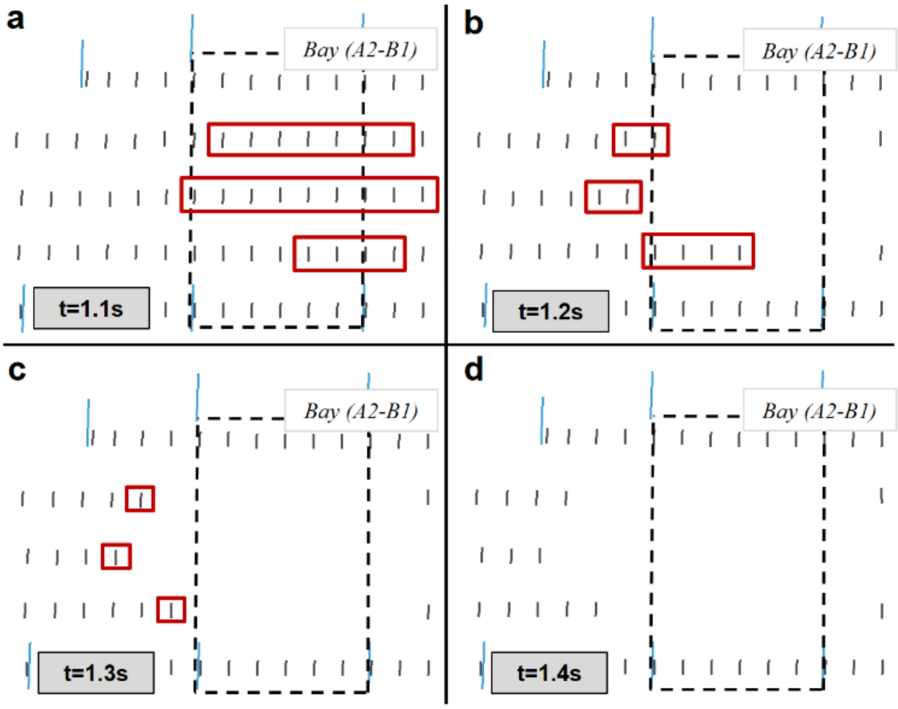

Fig. 8. Progressive collapse of the shoring system in the 3rd failure scenario without LLs.

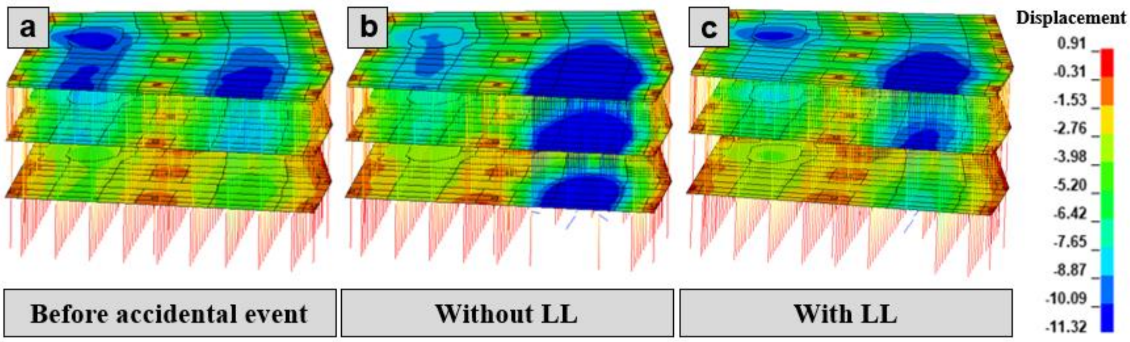

Fig. 9. Displacements and structure/shoring system: (a) before the accidental event, (b and c) after the sudden event, without (b) and with (c) load limiters (LL) on shores (units in mm).

Fig. 10 shows the time history results obtained in the slabs and shoring system during application of gravity loads (from t = 0.0s to 0.8s), the load stabilisation period (from t = 0.8s to 1.1s) and after the sudden failure of a complete line of shores (from t = 1.1s to 2.0s), without and with LLs shown on the left and right column graphs respectively.

Figs. 10a-b show, for the cases without and with LLs respectively, the shore loads below the first and second slab corresponding to the 1st and 2nd levels, in the position of the most heavily loaded shore. After the accidental event at t = 1.1s, with no LLs on shores, the load on the 2^{nd} level shore reduced significantly (more than 50%) whereas for the case with LLs this reduction was below 20%. This load reduction observed in both cases was due to the reduced stiffness of the ground floor shoring system after the sudden failure of the line of shores. This reduction was less noticeable using LLs because the progressive collapse of the ground floor shoring system was arrested and a higher number of shores were mobilised after the accidental event.

The thicker lines in Fig. 10c show that the progressive collapse in the case without LLs on shores caused a significantly larger displacement for the first (9.3mm increase) and second slab (7.8mm increase) at the position of the most heavily loaded shore under the first slab. However, the use of LLs (Fig. 10d) enabled to arrest the progressive collapse of the shoring system on the ground floor, and therefore the displacements of the slabs were smaller and only due to the sudden removal of the line of shores (2.5mm and 1.4mm increase for the first and second slab, respectively). It can also be seen in Figs. 10c-d that the accidental event had no effect on the behaviour of the adjacent bay "AB" (with or without LLs on shores).

Figs. 10e-f show, for the case without and with LLs respectively, the loads per unit surface (kN/m^2) carried by the shoring system (S) and slabs (Q) on each floor. For the case without LLs (Fig. 10e), the loads on the shores reduced significantly after the accidental event and as a result the loads on slabs increased significantly. This was not the case when using LLs where the effect of the accidental event on the loads carried on the slabs and shoring systems on each floor was reduced significantly as well as the damage which is further discussed below.

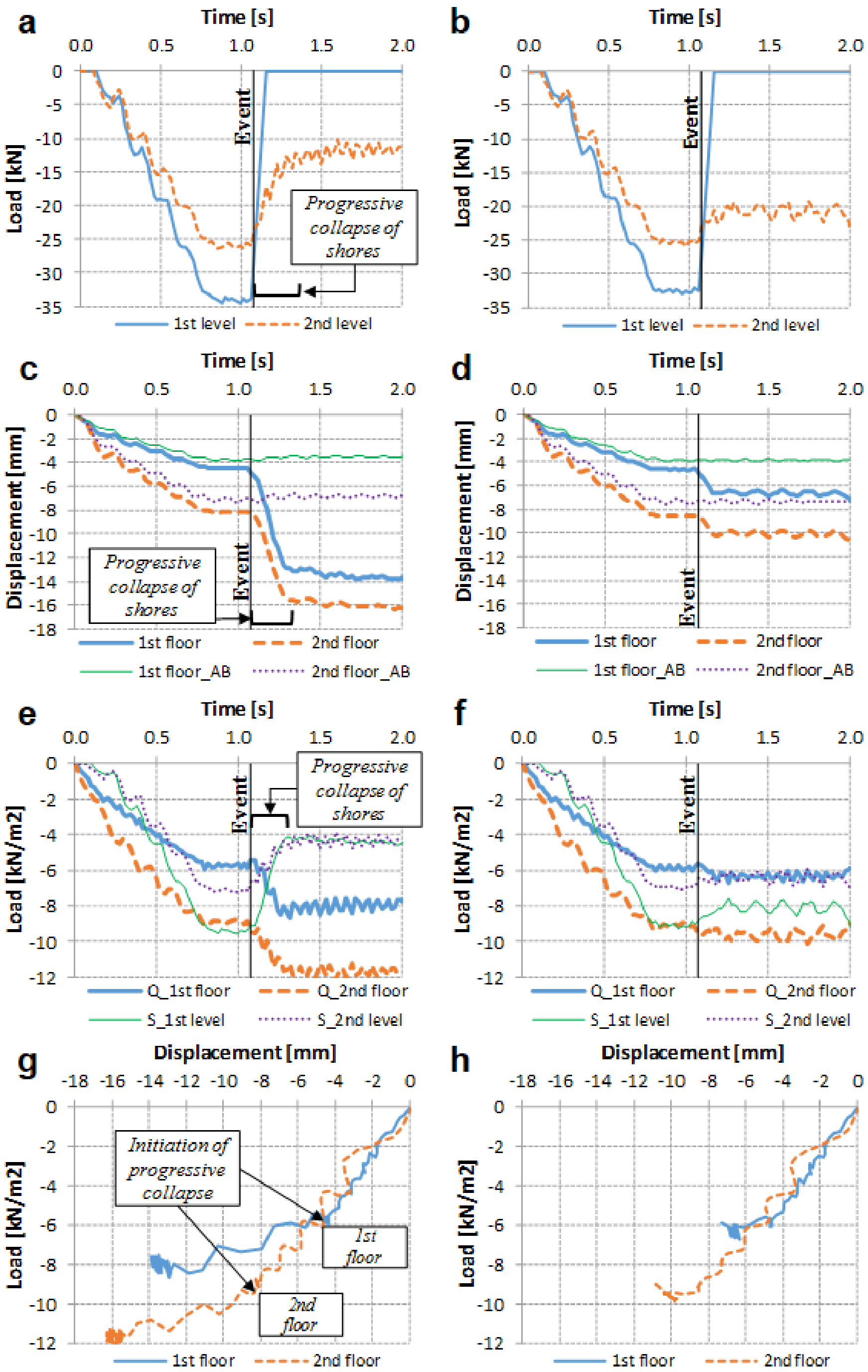

Fig. 10. Time history results of slabs and shoring for the 3rd failure scenario without LL (a, c, e, g) and with LL (b, d, f, h): a) and b) load of a single shore of level 1 and 2 for the most loaded shore under slab 1; c) and d) displacement of first and second floor for the bay under study and the adjacent bay (AB) for the position of the most loaded shore under slab 1; e) and f) slab and shoring system loads for the first and second floor and the 1st and 2nd level respectively; and g) and h) load-displacement of 1st and 2nd slabs (displacement at the position of the most loaded shore of the ground floor).

Figs. 10g-h show, for the case without and with LLs respectively, the load-displacement curves of the first and second slabs; the slab displacement corresponds to the position of the most heavily loaded shore on the ground floor. Without LLs, the slope of these curves (flexural stiffness) reduced significantly when the progressive collapse of the shoring system began. With LLs, excessive cracking was prevented and the slope remained relatively constant (linear behaviour). Fig. 11 shows the reduced cracking due to the LLs. The cracking bending moments in the slab were 51.6kN and 45.3kN for the first and second slabs respectively.

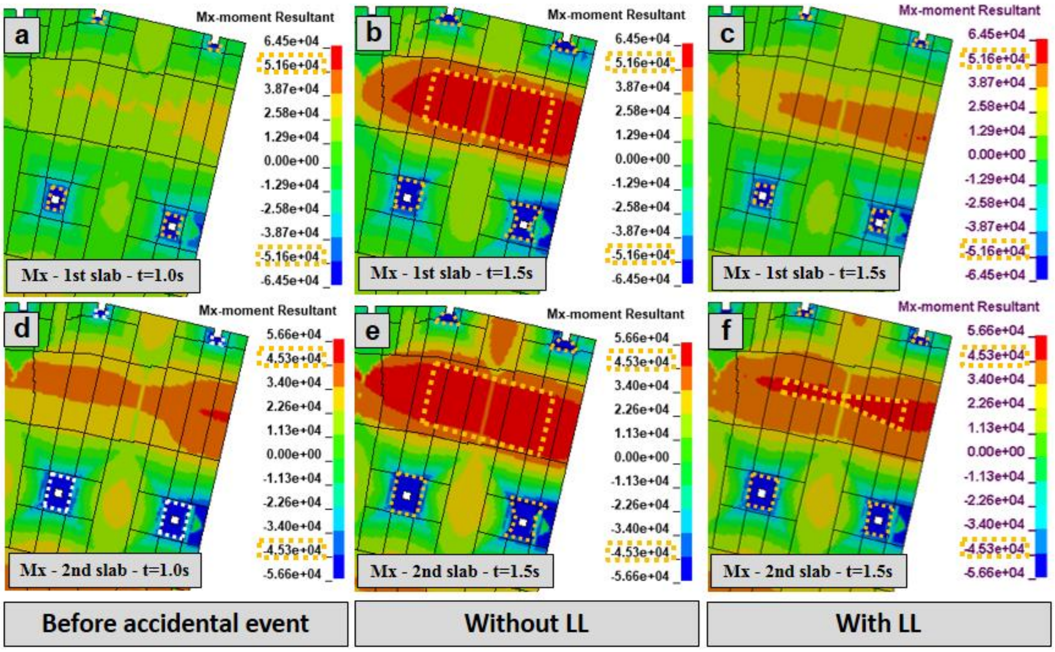

Fig. 11. Bending moments of first slab (a, b and c) and second slab (d, e and f) before (a and d) and after (b, c, e and f) the accidental event without (b and e) and with (c and f) load limiters (LL) for the 3rd failure scenario (units in N).

Fig. 12 shows the load on the ground floor shores (plan view) after the sudden removal of the complete line of shores for the case without and with LLs (white -first row- and grey -second row- background boxes respectively). Fig. 12 also gives the percentage of the use of the maximum permitted plastic displacement obtained in the LLs (third row). Without LLs, the only active joists are those at the edges of the bay, with heavy loads on the shores. However, with LLs, all the shores (except those in the failure scenario) remained active and did not reach neither their maximum strength nor maximum permitted plastic displacement. The plastic displacement was only 11% of the maximum value. LL plastic deformation began in the most heavily loaded shores at points with the highest deformation in the first slab (note that all the edges in bay A2-B1 are continuous except edge A1-B1 which is free).

Fig. 12. Ground floor shore loads without LLs (white background-first row-) and with LLs (grey background -second row-), and percentage of use of the maximum plastic displacement of LLs (orange background -third row-).

4.4.2. Incorrect choice of shore (4^{th} failure scenario)

Fig. 13 shows a plan view of the sequence (Δt = 0.1s) of the progressive collapse of the ground floor shoring system in the fourth failure scenario (incorrect selection of shores). For the case without LLs, applying gradually the full gravity load for the placing of the concrete in the third slab (from t = 0.0s to 0.8s) triggered the progressive collapse of the ground floor shoring system at t = 0.66s. The framed shores (highlighted in red) in Fig. 13 are those that disappeared (collapsed) in the following time step. This scenario resulted in the sequential overloading and failure of groups of shores leading to the progressive collapse of the entire shoring system at the ground floor affecting the upper floor levels as shown in Fig. 14a. Fig. 14b shows that the progressive failure of the shoring system did not take place when LLs were fitted to the shores.

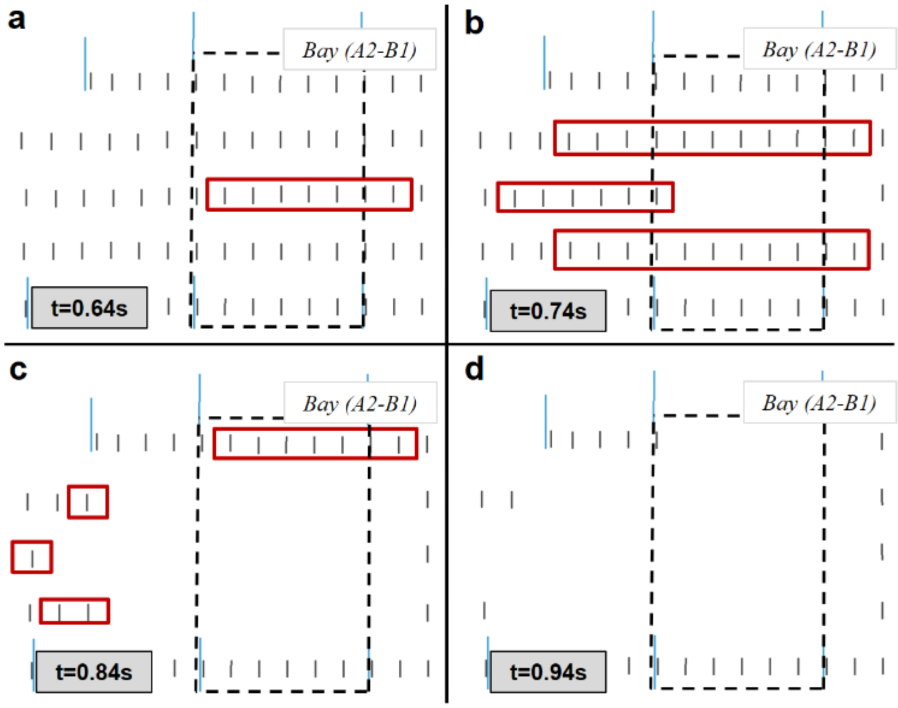

Fig. 13. Progressive collapse of the shoring system in the 4th failure scenario without LLs.

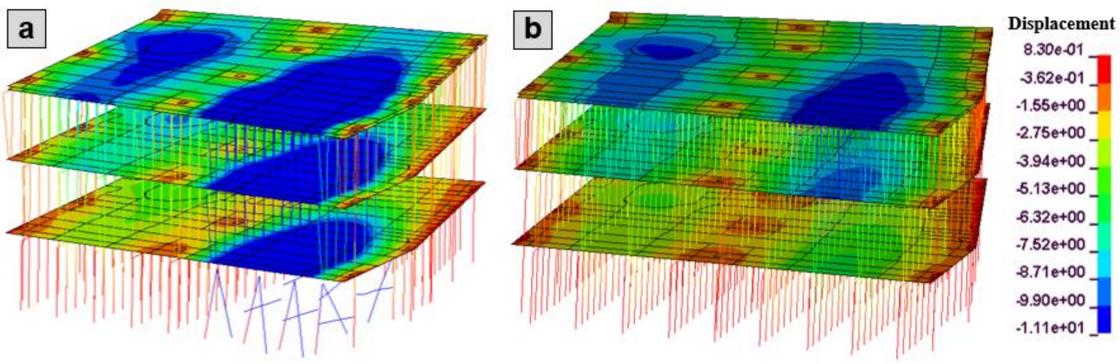

Fig. 14. Displacements and structure/shoring system after the accidental event (t = 1.5 s) for the 4th failure scenario: (a) without LLs and (b) with LLs (units in mm).

Fig. 15 shows the time history results obtained for the slabs and shoring systems during the application of the gravity loads (from t = 0.0s to 0.8s) and afterwards (until t = 2.0s); the results are shown for the case without LLs (left column graphs) and with LLs (right column graphs). Figs. 15a-b show, for the case without and with LLs respectively, the shore loads below the first and second slab corresponding to the 1st and 2nd levels at the position of the most heavily loaded shore. Without LLs, the load on the most heavily loaded shore on the ground floor dropped to zero when the shore reached its strength (30.6kN), whereas with LLs the maximum load reached values slightly over 25kN (corresponding to LL limit load). In the case where LLs were not used, the load on the corresponding shore on the second level reduced significantly (Fig. 15a) due to the reduced stiffness of the first shoring level after the accidental event. If LLs were used, the load on the shore on the second level remained constant after the accidental event (Fig. 15b).

The thicker line in Fig. 15c shows that progressive collapse without LLs caused a significant increase in the vertical displacement in the first slab (about 15.3mm) and second slab (about 14.0mm) at the position of the most heavily loaded shore below the first slab. However, with LLs (Fig. 15d) progressive collapse of the ground floor shoring system was avoided and the slab displacements remained constant. In Figs. 15c-d it can also be seen that the failure scenario did not affect the adjacent bay “AB” regardless of whether LLs were used.

Figs. 15e-f show, for the case without and with LLs respectively, the loads per unit surface (kN/m^2) carried by the shoring system (S) and slabs (Q) on each floor. In the case without LLs (Fig. 15e), after the accidental event, the loads on the shoring systems at the 1st and 2nd level reduced significantly as the loads on the slabs increased. When LLs were used, the effect of local failure due to choosing the incorrect shores was mitigated completely and the permanent RC structure remained almost undamaged; the level of damage is discussed below.

Fig. 15g-h shows the load-displacement curve of the first and second slabs for the case without and with LLs respectively; the displacements were measured at the position of the most heavily loaded shore on the ground floor. When LLs were not used, the slope of the curves reduced significantly after the start of the progressive collapse of the shoring system similarly as in Fig. 10g for the 3rd failure scenario. When LLs were used, the slope of the load-displacement curve was constant (Fig. 15h) which confirmed that cracking in the slab was minimal (linear behaviour of the slab). It can be concluded that the LLs were effective in reducing the damage in the slab after the incorrect shore was selected with strengths well below the required strength.

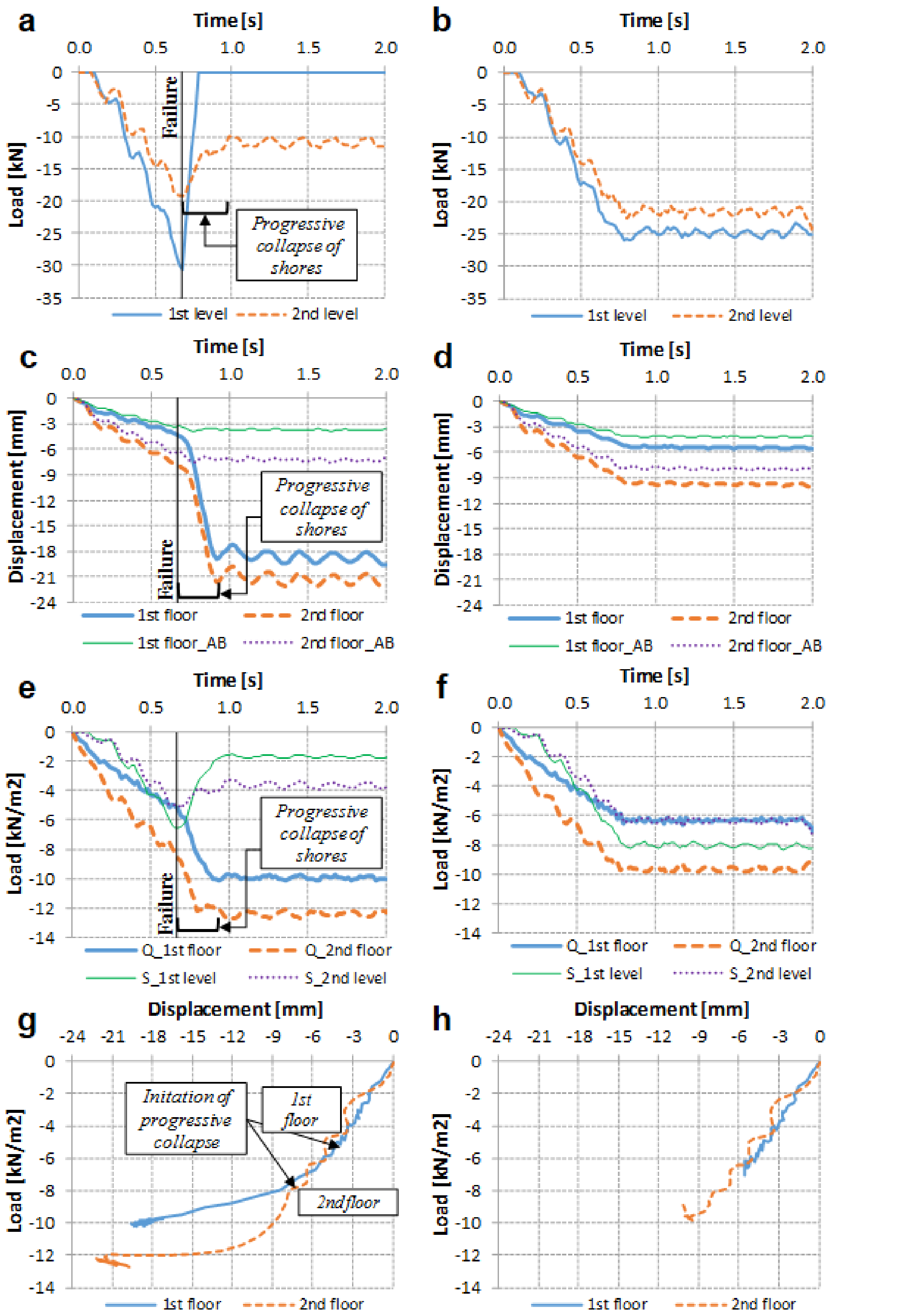

Fig. 15. Time history results of slabs and shoring for the 4th failure scenario without LL (a, c, e, g) and with LL (b, d, f, h): a) and b) load of a single shore of level 1 and level 2 for the most loaded shore under slab 1; c) and d) displacement of first and second floor for the bay under study and the adjacent bay (AB) in the position of the most loaded shore under slab 1; e) and f) slab and shoring system loads for the first and second floor and the 1st and 2nd level respectively; and g) and h) load-displacement of 1st and 2nd slabs (displacement at the position of the most loaded shore of the ground floor).

Fig. 16 shows the cracked areas in the slab in the case without LLs (enclosed by broken lines for Bay A2-B1 under study). The cracking bending moments in the slab were 51.6kN and 45.6kN for first and second slabs respectively. Fig. 17 shows the cracked areas for the case with LLs which is significantly reduced compared to the case without LLs in Fig. 16. These results show the potential of using LLs. Selecting the incorrect type of shore is not uncommon and it can also represent cases of unexpected live loads during construction for which the shores are not designed for. The LLs could act as a simple risk mitigating measure to protect against the effects of uncertainty of construction loading.

Fig. 18 shows the loads on the ground floor shores without and with LLs. Similarly, as in Fig. 12, the calculated percentage of the use of the maximum permitted LL plastic displacement is shown in Fig. 18. Without LLs, only one of the joists (at the edge of the bay) remained active whereas with LLs all the shores remained active without reaching their maximum strength or their maximum permitted plastic displacement. The shore with the largest plastic displacement reached only 30% of the maximum allowed. The plastic deformation in the LLs began in the most heavily loaded shores at the centre of the bay in the direction of the points in the slab with the highest deformation. Although many of the shores reached the limit load of the LL, the shores and LLs would be reusable. In order to reuse shores and LLs, a limit of the plastic deformation of 50% of the maximum plastic deformation is recommended by [36] based on experimental evidence.

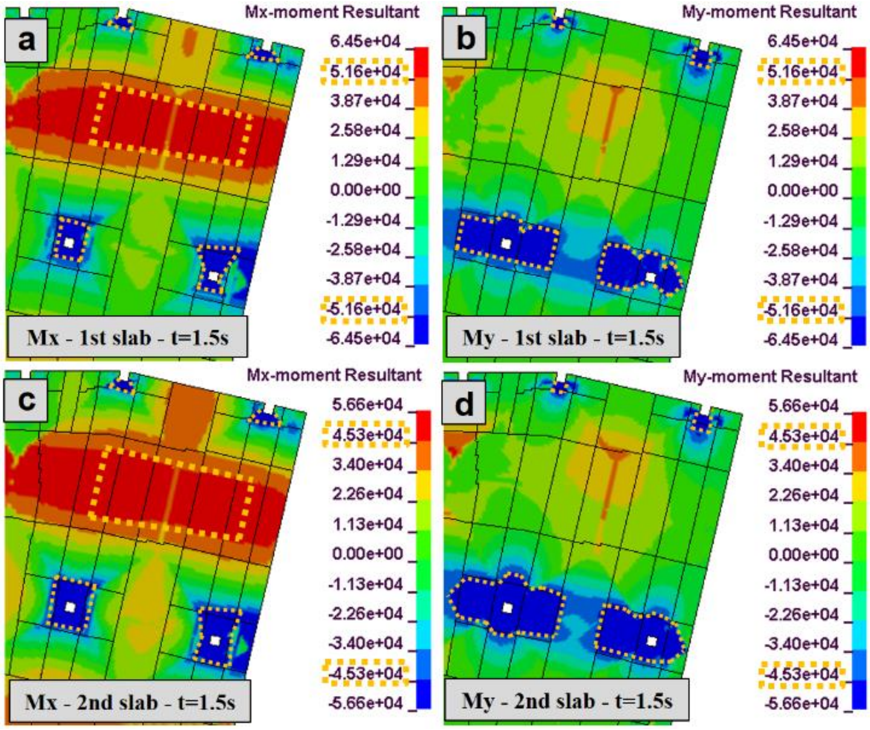

Fig. 16. Bending moments of first slab (a and b) and second slab (c and d) after the accidental event for the 4th failure scenario, without load limiters on shores (units in N).

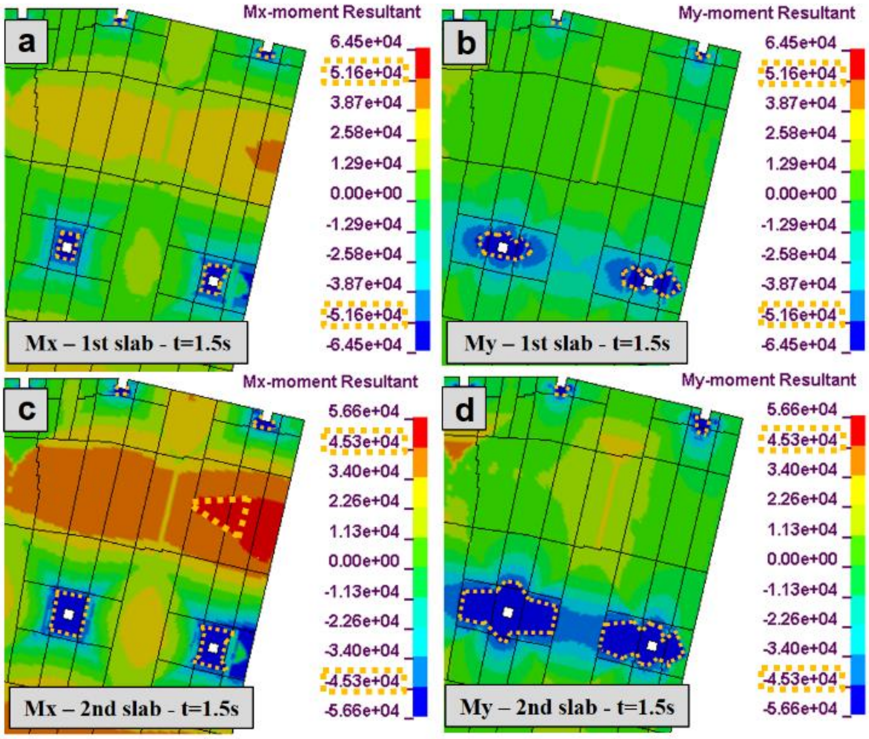

Fig. 17. Bending moments of first slab (a and b) and second slab (c and d) using load limiters (units in N).

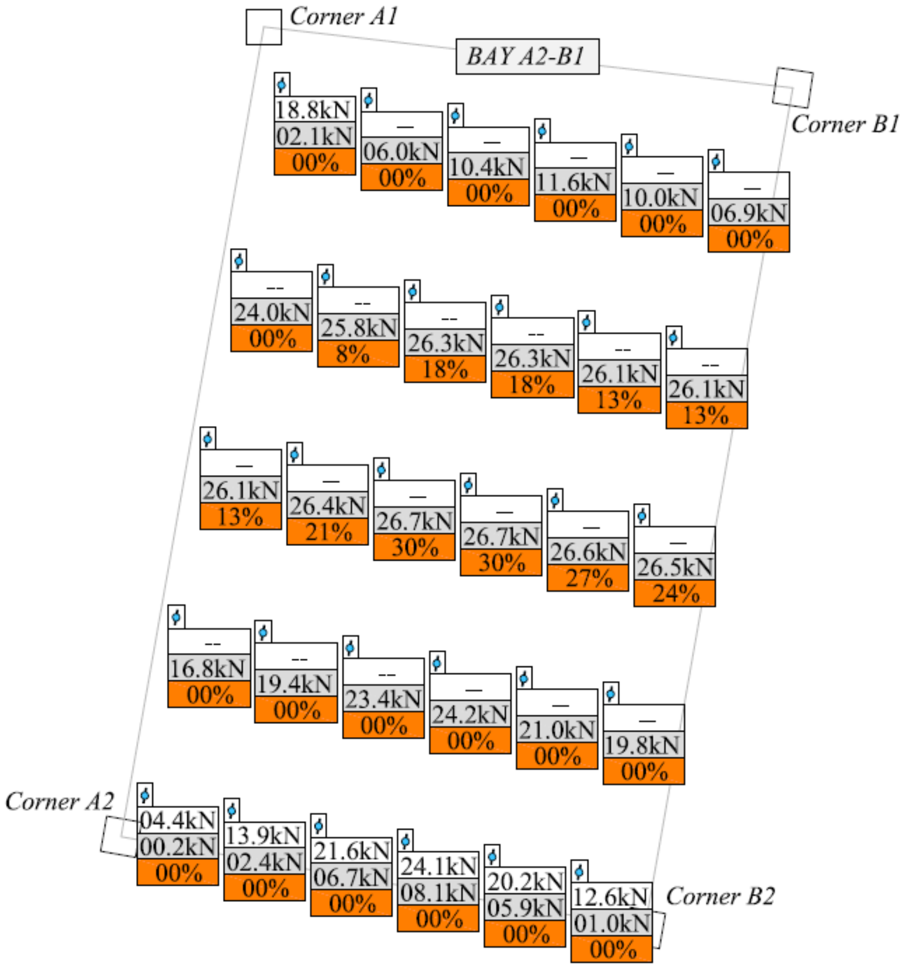

Fig. 18. Ground floor shore loads without LLs (white background -first row-) and with LLs (grey background -second row-), and percentage of use of the maximum LL plastic displacement (orange background -third row-).

4.4.3. Discussion of raw and mitigated risks

A summary of the slab loads and maximum residual displacement obtained in the analysis are shown in Table 1 for the cases without and with LLs in the two scenarios studied. These results are extracted and summarized from the dynamic analysis performed in previous sections and show the improvement achieved using LLs in reducing damage in the slabs with early-age concrete (14 days and 7 days for the first and second slab respectively) after the accidental event. In terms of consequences, the structural analysis in this work showed that LLs reduced the consequence scale from "significant/minor" to "minimal" using the IStrutE risk-assessment consequence scale [49] ("significant" means no collapse of the floor slab but potential loss of some local structural elements, “minor” means local permanent damage with minor repairs needed and “minimal” considers only some visible damage requiring only some cosmetic repairs).

Table 1. Maximum displacement and loads on slabs for the different failure scenarios without and with the use of LLs on shores.

		3rd Scenario				4th Scenario			
		Load [kN/m²]		Max. Displacement [mm]		Load [kN/m²]		Max. Displacement [mm]	
	t [days]	Without LL	With LL	Without LL	With LL	Without LL	With LL	Without LL	With LL
1st Floor	14	8.0	6.2	13.7	6.9	10.0	6.3	18.6	5.5
2nd Floor	7	11.5	9.9	16.1	10.2	12.6	9.8	22.1	9.7

A systematic risk assessment of the structure is shown in Fig. 19 using the consequence class obtained from the structural analysis and the probability of occurrence for the “unlikely” class which corresponds to a 10% probability of occurrence during the design life [49]. Probability class “likely” corresponds to 50% and “rare” is for 2% probability. The probability of occurrence of the 4th failure scenario (between 6% to 26%) can be slightly higher than for the 3rd failure scenario (between 3% to 18%) depending on the causes as discussed in more detail in [9,17]. The adopted probability class is consistent with the one used in the example building in the IStructE manual for the hazard identified as failure of temporary works during construction [49]. The black line shown in Fig. 19 shows a typical tolerable risk threshold used in the IStructE manual [49] which is roughly consistent with Annex B in EN 1991-1-7:2006 [50]. Fig. 19 shows that the raw risk of failure of the shoring system is very close to the threshold which is undesirable. The situation can worsen depending on the role of the slab where the damage takes place affecting the severity of the consequence from “minor/significant” to “significant” in Fig. 19. For example, as reported in [49], for transfer slabs in ground floors damage on the member can have significant implications.

Fig. 19 shows that introducing the LLs on the shores will shift the risk (mitigated risk) into the tolerable risk represented by the green boxes in the risk matrix. A cost-benefit analysis is generally recommended to finalise the implementation of the risk mitigating measures, followed by the review of the residual risks and carry out a check on the risk assessment [49].

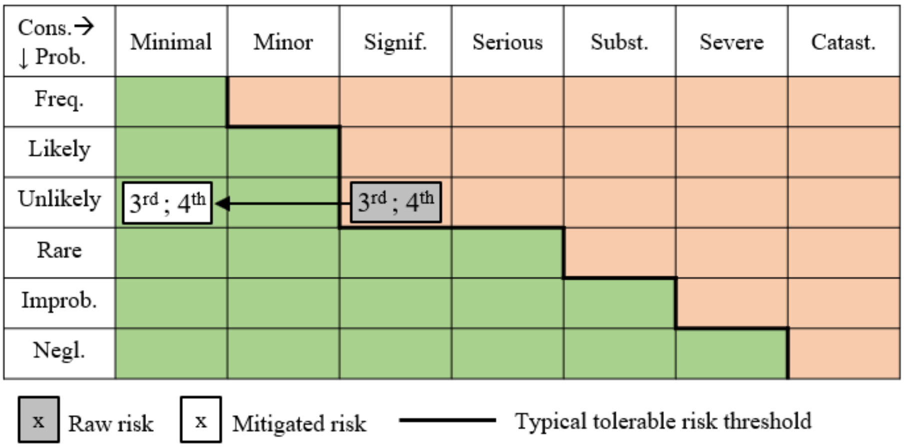

Fig. 19. Analysis of raw and mitigated risk after introducing LLs for the 3rd and 4th failure scenarios; risk matrix based on [49] **(green: tolerable risk; red: intolerable risk).**

5. Conclusions

The use of structural fuses as load limiters on shores during construction is promising in terms of improving the shoring system design and reducing costs as shown in [35]. This paper analyses the consequences and advantages of using load limiters during building construction under accidental events. The present study considers possible failure scenarios, some of them resulting into the progressive collapse of the shoring system and some structural damage in the permanent structure (concrete slabs). From the study, the following conclusions can be drawn:

- The results show that installing LLs on the shores increased safety during the construction phase, maintaining the integrity of the temporary shoring structure, preventing excessive loads and displacements being transferred into the permanent RC structure and avoiding residual damage. Using LLs on shores prevented the sudden local failure of the shoring system, which can cause progressive collapse of the structure as observed in some accidents.
- Design standards [12] are starting to consider progressive collapse of temporary shoring with the idea that local damage can trigger a more serious progressive

collapse. In this context, this work shows that LLs is a promising solution to prevent progressive collapse and mitigate residual damage (e.g. cracking and short/long term deflections) after accidental events. This is relevant towards avoiding costly structural repairs and improve the long-term performance of the structure.

- The structural analyses in this work showed that LLs were able to arrest the propagation of failure of the shoring system in the most critical scenarios investigated, viz., sudden removal of the entire shore line and incorrect shore selected during design/construction. The analysis confirmed that after the accidental event, the most heavily loaded shores reached the limit load provided by the LL with sufficient ductility to activate a larger number of shores compared to cases were LLs are not used.
- The systematic risk assessment included in this work, based on the probability of occurrence considered and the consequences obtained from the structural analysis, showed that without LLs the raw risk of temporary work failure can be near the tolerable risk threshold. The risk assessment also showed that introducing LLs in the shores was efficient as a simple risk mitigating measure.

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