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AN EMBEDDED YIELD DESIGN APPROACH WITHIN A NON-LINEAR ANALYSIS FOR STRUCTURAL MODELING OF PROGRESSIVE COLLAPSE

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Abstract

Dealing with structural robustness concept requires to investigate whether or not a structure can prevent a disproportionate collapse after the occurrence of a local failure due to an exceptional event. Numerical models can be used to simulate progressive collapse and help quantify the robustness level at a design stage. Non-linear static or dynamic finite element analyses are commonly used tools for structural performance assessment. However, computational time might be in most cases too large for complex structures where several local failure scenarios need to be investigated, and one may encounter convergence issues if the loads applied are close to the limit ones.

In this context, this study proposes a framework for studying the progressive collapse of framed structures, which combines both the yield design approach and the non-linear analysis method. This proposed framework is applied to a steel-framed multi-storey building submitted to column(s) loss.

Keywords: Structural robustness, local failure, progressive collapse, non-linear analysis, yield design approach.

1 INTRODUCTION

Many catastrophic events of structural progressive collapse highlight the importance of the structural design not to be limited to safety under normal conditions, but also to structural integrity under an exceptional event, not necessarily identified during design [1,2,3]. One of the first historical failures that led to a growing interest in structural robustness is the progressive and partial collapse of the Ronan Point tower in London (UK) in 1968. A gas explosion in a corner apartment on the 18th floor of this 22-storey precast concrete building dislodged one of the exterior walls, which led to the collapse of one entire corner of the

building [4]. Very recently, the Genoa Bridge collapse in Italy in August 2018, shed light on complex issues linked with structural robustness.

Therefore, several design codes [5,6,7] mention that structures must be sufficiently robust to prevent localized damage leading to disproportionate and unacceptable collapse. In this respect, the Eurocodes define the structural robustness as “the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause” [5].

To assess the level of robustness, a structural modeling can be used to simulate the propagation of failure. In a context of high uncertainty about the initial local failure, it is envisaged to study a large number of local failure scenarios, in order to identify the maximum capacity of the structure to withstand a local failure. The structural analysis of a large number of scenarios requires a simplified structural modeling method.

Non-linear finite element analyses are popular tools to investigate the structural capacity, but some difficulties may arise on the non-convergence of the calculation (when one reaches ultimate limit states), and on the high computation cost especially for studying a large number of scenarios.

This paper presents an original structural modeling method, which combines both the yield design approach and the non-linear analyses method, to analyze the progressive collapse of framed structures. The proposed approach is illustrated on a steel-framed multi-storey building.

2 STRUCTURAL MODELING OF PROGRESSIVE COLLAPSE

The progressive collapse analysis of structures exposed to an exceptional action involves complex phenomena, such as geometrical and material non-linearities due to large displacements and large strains, dynamic effects, and the propagation of failure.

The finite element method is a widely used method in the numerical simulation of structures. The discretization in finite element can be considered at three different levels: local, global, and semi-global [8]. Choosing the level of discretization basically depends on the dimensions of the structure and the level of precision required. In a local approach, the elements are discretized with solid elements, and each material has a specific constitutive law. This approach gives a detailed representation of the structure, with local information on the state of plastification and damage of materials. Significant computation time is requested, especially when dealing with geometrical non-linearities. In a global approach, the structure is modelled with beam/shell elements, and each element has its own constitutive law depending on its geometry and materials. This method can significantly save some computation time, but there might be some difficulties to simulate the material non-linearities and to identify the state of damage in the element sections, especially in case of heterogeneous sections. The semi-global approach is an intermediate scale of discretization between local and global approaches, where the element section is discretized on multi-layer or multi-fiber elements. The constitutive law used for each layer or fiber insures local information of materials state, and the fields of displacements calculated by formulations of classic beam element. This approach integrates the benefits of local and global approaches: saving on computation time and ability to describe geometrical and material non-linearities.

In addition to the issues linked with the choice of discretization, one needs to tackle non-linearities phenomena and dynamic effects [9]. Non-linear analyses are often very time-

consuming and vulnerable to non-convergence issues [10]. To prevent using full dynamic analyses, the structural response can be estimated from a non-linear static response under amplified gravity loading using a dynamic amplification factor [11] or a pseudo-static method [12], which estimates the non-linear dynamic response by a non-linear static analysis through the balance of energy against work done.

With the aim of evaluating the capacity of structure to withstand actions, it is useful to identify the resistance capacity of the structure. In this context, the yield design approach is a good compromise, as it is a direct method, which avoids the non-linear analysis and thus the step-by-step computation of the structure along the full loading path [13]. In fact, only the compatibility between the equilibrium equations and the yield criterion is checked in every point of the structure. This method identifies the ultimate loads, as well as the failure mechanism and the most critical areas of the structure. Moreover, one can dramatically save on computing time compared to a non-linear analysis, and avoid problems of non-convergence [14]. The essential assumptions of the process of yield design approach are that the materials are elastic perfectly plastic, and the assumption of small strains. Therefore, the main challenges to use this method is to take into account the geometrical non-linearities and to simulate the progressive collapse.

3 PROPOSED STRUCTURAL MODELING STRATEGY

An iterative yield design based approach is proposed to follow the propagation of failure. Furthermore, a non-linear static analysis is applied to calculate large displacements if a second line of defence can become effective when frames devolve from a flexure dominant system to a tensile membrane or catenary dominant system. This procedure is illustrated in Figure 1, with the following steps :

- 1) yield design calculation is applied to identify the ultimate load and the failure mechanism,
- 2) ultimate and applied loads are compared,
- 3) in case the ultimate load is larger than the applied load, the current structural configuration can support the applied load, and the failure stops at this stage,
- 4) in the opposite case, the current configuration of structure cannot support the applied load, and the failure propagates,
- 5) the failure mechanism identified by the yield design calculation allows to identify the affected part, and to estimate if there is either a loss of stability or the possibility of a second line of defence,
- 6) in case the failure mechanism indicates the mechanical instability of some elements, these elements are removed for the next iteration,
- 7) in the opposite case, the failure mechanism indicates that the affected part may develop an alternative functioning stage after large displacement. A non-linear analysis is then applied to the affected part, in order to calculate the geometric displacements under the applied load. Then, a new iteration of yield design calculation is performed with the new geometric configuration,
- 8) this iterative procedure continues until the end of collapse, for which the ultimate load on remaining elements is larger than the applied load, or until total collapse of the structure.

Thus, the proposed method consists of a coupling between the yield design approach and the non-linear analysis with an iterative procedure. Also, a dynamic amplification factor is used to take into account the dynamic effect. This strategy of structural modelling enables to simulate the progressive collapse.

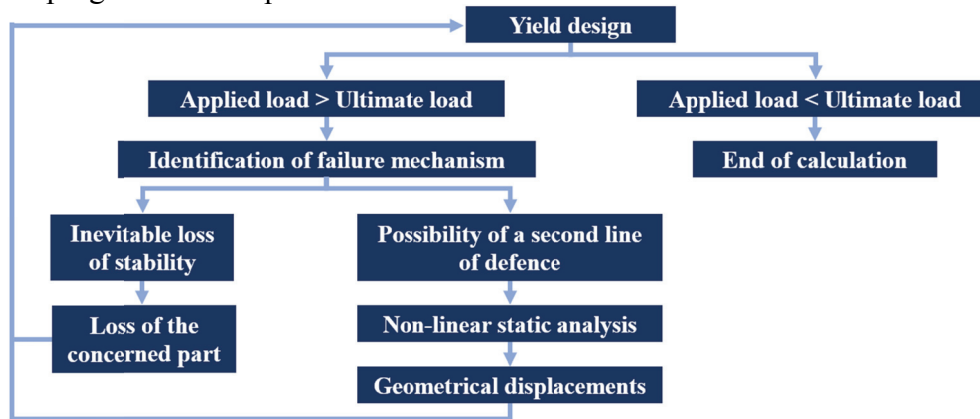


Figure 1: Proposed structural modeling strategy

4 ILLUSTRATIVE APPLICATION

This section presents the application of the proposed method on a steel-framed five-storey building, in order to study the structural response against some local failure scenarios.

4.1 Structural configuration

The structure is a 2D typical five-storey steel-framed building consisting of beams with section IPE360, and columns with section HED500, where Figure 2 presents the layout with dimensions and numbers of columns.

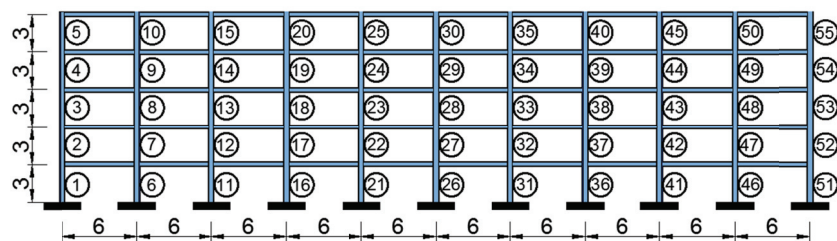


Figure 2: Layout (dimensions in meter).

The steel material of beams and columns is considered as elastic perfectly plastic, where the young modulus E and the yield strength are equal to 210 GPa and 355 MPa, respectively. The connections column/beam and column/footing are considered as rigid joints.

4.2 Applied loads

The beams are exposed to uniform loads, where the values of dead loads (DL) and live loads (LL) are 20 KN/m and 10 KN/m, respectively. The combinations of actions refer to ultimate states, according to Eurocodes EN1990 [11], as follows:

- Normal situation: $W=1.35 \text{ DL} + 1.5 \text{ LL}$ (1)
- Accidental situation: $W=\text{DL} + 0.5 \text{ LL}$ (2)

As the analysis is an accidental situation, the combination of loads is equal to 25 KN/m. The self-weight of structural members is 77 KN/m³.

According to Marchand and Alfawakhiri [15], the appropriate dynamic amplification factor for the non-linear elasto-plastic response is between 1.3 and 1.5, in this example the used value is 1.5. The load amplification is applied only on the directly affected part, which normally contains all the beams, columns and beam-to-column joints located just above lost column(s).

4.3 Numerical modeling

The model proposed by Bleyer and de Buhan [16] for yield design calculation of frame structure is used on MATLAB R2017a. This method consists of two main steps. Firstly, yield surfaces of element sections are approximated using a sum of ellipsoids, which identifies the ultimate strength domain of the section in the space of axial force (N) and bending moments (M_y , M_z) as shown in Figure 3 (this approach does not take into account the yield by shear and torsion efforts), with $n=N/N_0$, $m_y=M_y/M_{y0}$ and $m_z=M_z/M_{z0}$ where $\Sigma_0=(|\min(\Sigma)|+|\max(\Sigma)|)/2$ for $\Sigma=N$, M_y or M_z . Secondly, the structure is discretized using beam elements, and the limit load is identified by two approaches (static and kinematic).

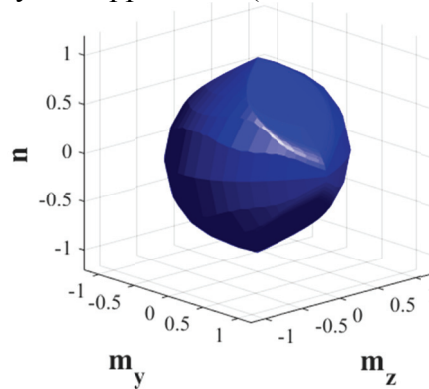


Figure 3: Yield surface in the (n, m_y, m_z) non-dimensional space.

The static approach determines a lower bound of ultimate load according to the optimisation problem (3). The applied loads are decomposed as $\lambda \cdot F + F_0$ where F_0 represents a dead load and $\lambda \cdot F$ is the multiplicative load for which we are interested in finding the limit value at failure through the multiplier λ .

$$\lambda_{static} = \max(\lambda), \text{ such that } \begin{cases} H \cdot \Sigma = \lambda \cdot F + F_0 & \text{global equilibrium} \\ \sigma \in G & \text{local yield criterion} \end{cases} \quad (3)$$

where λ_{static} is the multiplier value identified by static approach, H is the global equilibrium matrix, Σ is global vector of stress parameters, σ is the local vector of stress parameters and G represents the local yield criterion.

Regarding the kinematic approach, one determines an upper bound limit of ultimate load according to the optimisation problem (4), where $\lambda_{kinematic}$ is the multiplier value identified

by the kinematic approach, P_{ext} is the power of external loads, U is the displacement vector, P_{rm} is the maximum resisting work, and $d[U]$ is the strain vector related to U at point χ .

$$\lambda_{kinematic} = \min(\lambda), \quad \text{such that } P_{rm}(U) \leq P_{ext}(U) \quad \left\{ \begin{array}{l} P_{ext}(U) = \lambda \cdot F \cdot U + F_0 \cdot U \\ P_{rm}(U) = \int_{\Omega} \Pi(d[U]; \chi) d\Omega \\ \Pi(d[U]; \chi) = \sup\{\sigma : d[U]; \sigma \in G\} \end{array} \right. \quad (4)$$

Concerning the static nonlinear analysis, the MATLAB toolbox FEDEASLab [17] is used, where the multilayer approach is applied to model the structural elements. This toolbox enables to take into account the geometrical non-linearities using co-rotational formulations [18].

4.4 Local failure scenarios

In this example, the local failure scenarios are limited to the total loss of one or several column(s), provided that the damaged columns are always adjacent, that the maximum extent of local failure is in two bays and that the maximum number of damaged columns is three adjacent columns. There are consequently 150 scenarios to investigate.

4.5 Structural response

The structural response is presented with two main indicators:

- directly affected zone: length of the beams within the area directly affected, represented by the affected part of the first iteration of yield design approach, this value presents the area initially affected.
- collapsed zone: length of the beams within the area that collapsed (loss of stability).

These two values enable to evaluate if the local failure has consequences, when the directly affected part is larger than zero, and if it leads to collapse, when the collapsed part is larger than zero. Moreover, these two values present the initial stage and the final stage of the progressive collapse, which allows to quantify the degree of failure propagation, and provide an effective structural robustness index.

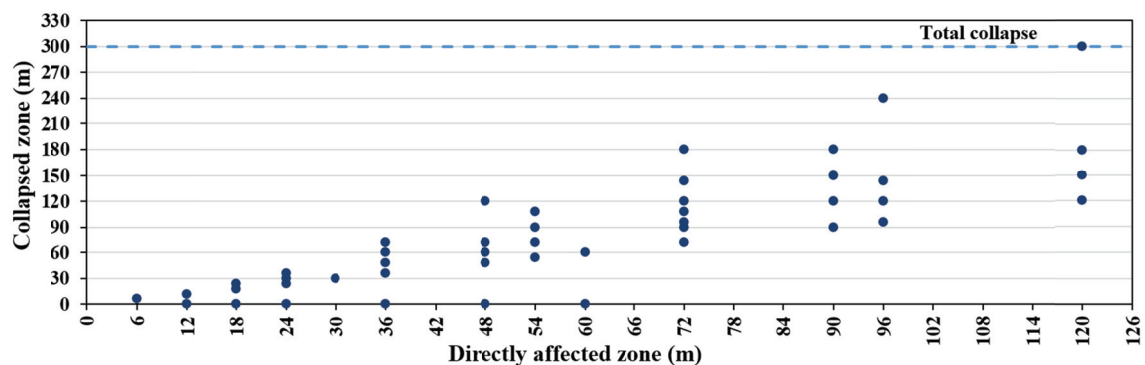


Figure 4: Results of structural response

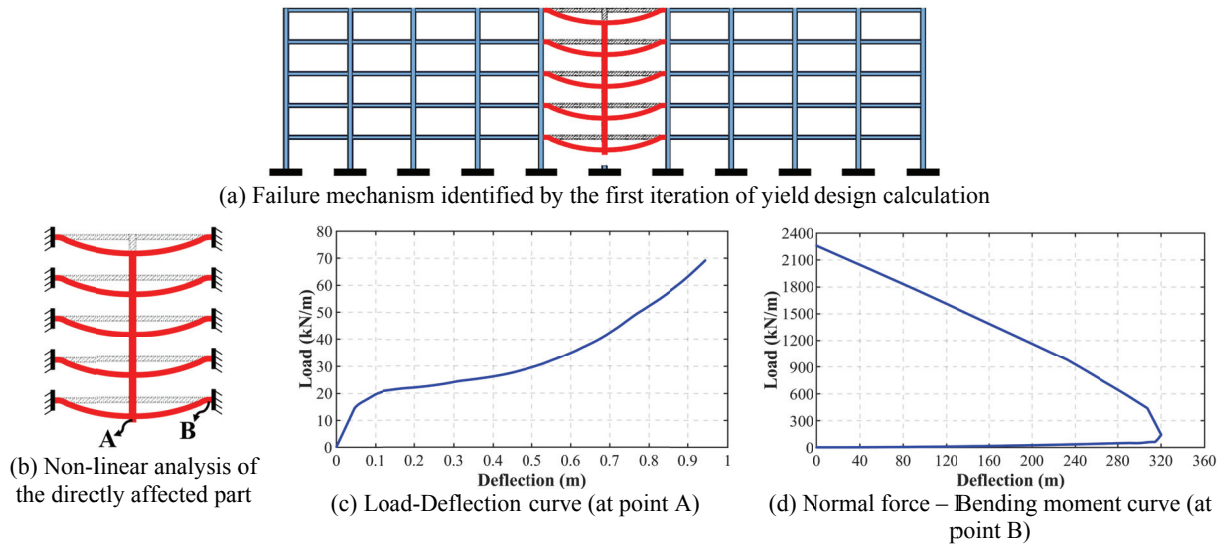


Figure 5: Catenary action (loss of column 26, see Figure 2).

Figure 4 presents the different structural responses against the local failure scenarios taken into consideration in this example. The directly affected zone is larger than zero in all cases, so there is no scenario without consequences. Some scenarios have no collapsed zone, thus the structure has succeeded to find a second line of defence by the catenary action developed in the beams of the directly affected part. Figure 5 represents the non-linear static analysis of the affected part in the case of the loss of column 26. It shows the catenary action where after large deflection there is an increase of tensile stress in beams and decrease of bending moment effort, which helps the structure reaching an alternative equilibrium configuration. Besides, there are one hundred scenarios that lead to a partial/full collapse of the structure, and the extent of failure differs from one scenario to another, but one of them leads to a total collapse of structure, which is the loss of the central columns 21, 26 and 31 (see Figure 2).

4.6 Robustness index

The goal of robustness indices is to be used as design decision support. Their validity and usefulness depend on the following general requirements: expressiveness, objectivity, simplicity, calculability, and generality [19]. In order to assess the structural robustness, the main aspect that has to be investigated is the extent of failure propagation compared to the local failure. For this purpose, a robustness propagation failure index (*RPF_I*) is proposed, which identifies the maximum degree of failure propagation among the *N* applied scenarios, where the degree of failure propagation (*DFP_i*) of each scenario *i* is the collapsed zone after propagation of failure (*CZ_i*) divided by the initial damaged zone (*IDZ_i*). This index is expressed as:

$$RPF_I = \max \{DFP_i, i \in [1, N]\} \quad (5)$$

$$\begin{cases} DFP_i = \frac{CZ_i}{IDZ_i} & \text{if } IDZ_i \neq 0 \\ DFP_i = 0 & \text{if } IDZ_i = 0 \end{cases} \quad (6)$$

Based on the applied local failure scenarios, *RPFI* is equal to 2.5, so the most critical scenarios, with the largest extent of failure propagation, are [21 26 31], [22 27 32], [23 28 33] and [24 29 34], i.e. three central columns located on a given floor, where the collapse propagates to an area 2.5 times larger than the directly affected part.

5 CONCLUSIONS

The structural modeling method proposed in this paper enables to simulate the progressive collapse with saving in computation time and mitigation convergence issues due to the adoption a direct approach by means of the yield design method. The illustrative case study shows the capability of this method to study a large number of local failure scenarios, which allows a general assessment of structural robustness, and to identify the maximum capacity of the structure to withstand exceptional events. Besides, a structural robustness index is proposed (*RPFI*) and allows to evaluate the capacity of structures to prevent the propagation of failure, which accurately responds to the definition of structural robustness.

To enhance the description of progressive collapse, further developments are still needed, to deal with aspects such as the 3D structural response, including the effects of slabs, and to adapt for a large range of materials and structures under different types of exceptional loads.

Furthermore, a strong assumption in this paper has been made, where the materials are considered as elastic perfectly plastic (yield design approach). Therefore, to provide a more realistic behavior of materials, it is important to take into account the ultimate strain of materials, as it can dramatically change the results of the analysis.

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