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Progressive Collapse Assessment: A review of the current energy-based Alternate Load Path (ALP) method

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Abstract. The Alternate Load Path (ALP) is a useful method that has generated a considerable recent research interest for the assessment of progressive collapse. The outcome of the ALP analysis can be assessed either using the force-based approach or the energy-based approach. The Unified Facilities Criteria (UFC- 4- 023-03) of progressive collapse guideline – have outlined that the force-based approach can either be analysed using static or dynamic analysis. The force-based approach using static analysis is preferable as it does not require a high level of skill and experience to operate the software plus no effort is required in scrutinising the validity of the analysis results output. However, utilising the static approach will eliminate the inertial effect in capturing the actual dynamic response of the collapsed structure. In recent years, the development of the energy-based progressive collapse assessment is attracting widespread interest from researchers in the field; as the approach can produce a similar structural response with the force-based dynamic analysis by only using static analysis. Most of the current energy-based progressive collapse assessments are developed following the requirements which are given in the progressive collapse guidelines provided by the Unified Facilities Criteria. However, little attention is given to the development of the energy-based approach using the Eurocode standards as a base guideline. This article highlights the merits of utilising the energy-based approach against the force-based approach for a collapsed structure and explains the collapse mechanism of a steel frame in the perspective of the energy concept. The state of the art of energy-based progressive collapse assessment for a structural steel frame is reviewed. The comprehensive review will include insights on the development of the energy-based method, assumptions, limitations, acceptance criterion and its applicability with the European standards. Finally, potential research gaps are discussed herein.

1 Introduction

General Services Administration (GSA) document defined progressive collapse as "an extent of damage or failure that is disproportionate to the magnitude of the initiating event" [1]. By that definition, the phenomenon of progressive collapse can be divided into three stages; the local failure, the spread of damage, and the final collapse state. At the first stage, the local failure occurs due to the removal or weakening of the vertical load-bearing elements (columns, transfer girders or load-bearing wall) triggered by the abnormal loadings such as an explosion, vehicle collision, fire and seismic actions. The initial damage is then spread throughout the structure, which can either be successively or instantaneously. The final collapse stage involves the partial collapse or total collapse of the building that is disproportionate to the initial damage [2, 3]. The first phenomenon of progressive collapse can be seen in the case of the Ronan Point collapse on May 16, 1968. The building collapsed due to the removal of a precast load-bearing wall initiated by the gas explosion. The local damage has led to the partial collapse of the building as shown in **Fig. 1**. The collapse of the Ronan Point is the starting line that sparks the change in the design codes for mitigating the risk of progressive collapse [4].

The current design codes have defined several methods that are aimed to either reduce the risk of local failure or limit the spread of the initial damage. In general, the design method for progressive collapse design can be divided into two methods; direct design method and indirect design method. Under the direct design method, the resistance of a building towards progressive collapse due to abnormal loading or condition is considered explicitly in the design process [5]. Some of the design strategies are the Alternate Load Path (ALP) method and Enhanced Local Resistance (ELR) method. The indirect design method, on the other hand, does not consider the resistance of the structure against progressive collapse, but instead, the structure is designed to provide minimum strength, continuity and ductility [5] through Tying Force (TF) method. The UFC 4-023-03 [6] outlined the procedure for both of the design methods, whereas the current GSA document [1] only outlined the procedure for the ALP method. The Eurocode [7] and the UK Building Regulation [8] offer the design methods that are conceptually similar with the UFC 4-023-03 design methods, but with a different name for each method (i.e.

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notional column removal strategy and key element design). However, more detail is given on the TF method instead of the other two methods.

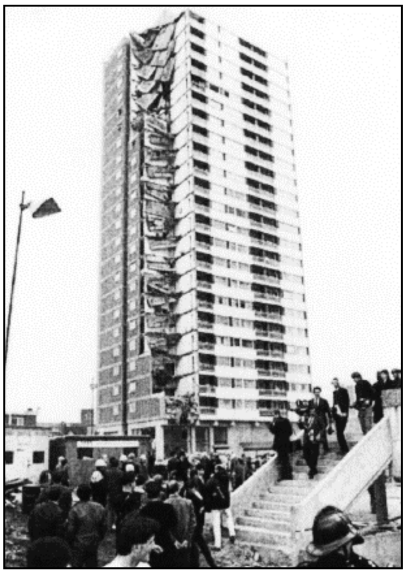


Fig. 1. Partial collapse of the Ronan Point due to the gas explosion at the 18th floor [4].

When it comes to realism, the ALP method is the best method to evaluate progressive collapse potential because it can imitate the real structural response towards localised failure. For that reason, this method is favoured by researchers in understanding the concept of progressive collapse. In this method, a load-bearing element is notionally or instantaneously removed to initiate a response, but the event that caused the removal the load-bearing element is not considered [1, 6]. The structural response is then compared with the predefined acceptance criteria to justify the vulnerability of the building against progressive collapse.

2 Approaches for the Alternate Load Path method

The ALP method can be executed based on two approaches, namely the force-based approach and the energy-based approach. Under the force-based approach, the analysis is grounded on Newton's laws, which define the relationship between the state of motion of a body and the forces that cause the motion. In the context of progressive collapse assessment, the building is assessed based on the ability of the structural elements to redistribute the unbalance gravity load throughout the structural system. The structural capacity and demand are expressed in term of forces (axial force, moment, shear force, etc.) and deformation (displacement, rotation, etc.). The implementation of the force-based ALP method can be seen in the current code of practice [1, 6]. In the energy-based approach, the analysis is based on the principle of conservation of energy. The resistance of a structure against progressive collapse is assessed based on the structure's capability to absorb and dissipate the energy generated by unstable gravity load.

Since the progressive collapse is a dynamic phenomenon, inertial effect (the resistance of a body towards the change in state of motion) is an important element to be considered in the force-based approach. The inertial effect can only be considered if the dynamic analysis technique is being adopted. However, the requisites for this type of analysis are high computer skill and sophisticated software, both of which are not commonly acquired among the practising engineers. As an alternative to the dynamic analysis, the static analysis with the use of Dynamic Increase Factor (DIF) is introduced [1, 6]. The purpose of the DIF is to initiate static structural responses that are similar to the dynamic structural responses. Even though this type of analysis could reduce the analytical burden of the dynamic analysis, it is founded that the use of DIF in the analysis is bounded with prediction error [9, 10] and analysis output is highly conservative [11].

In recent years, many researchers have focused their effort on developing the energy-based ALP method for progressive collapse assessment [9, 10, 12-18]. It is because the energy-based approach can eliminate the inertial effect in the analysis as it is not tied to any of Newton's law, but it is only based on the principle of conservation of energy. Current researches have shown that the structural response obtained from the energy-based static analysis is approximately similar to the dynamic structural response obtained from the force-based dynamic analysis [9, 10, 17]. Thus, it is possible to eliminate the use of DIF in the analysis [10, 12]. The energy-based parameters (such as strain energy, buckling energy limit, etc.) used in the analysis are also independent to the loading rate (inertial effect), unlike most of the force-based parameters (strength, stress, etc.) [15, 19]. Therefore, the energy-based approach would be a possible alternative to the force-based dynamic analysis in providing an efficient and reliable method for progressive collapse assessment.

3 Progressive collapse and the energy concept

Before discussing progressive collapse in the viewpoint of energy, three main parameters are first introduced; those parameters are potential energy, strain energy and kinetic energy. Potential energy (E_p) is the energy stored in a body due to its position in the gravitational field Given that the body has undergone vertical downward displacement, it is said that the body has experience loss of potential energy and committing work [15]. It is estimated based on the product of the moving weight and its associated displacement [12]. Strain energy is the energy stored or dissipated by a structural element when it has undergone structural deformation. It is quite common that the strain energy of a structure is derived from the load-displacement curve (the area under the graph represent strain energy) [9, 10, 13, 16-18, 20], which can be expressed based on the following general integral:

$$U = \int P(u)\,du \tag{1}$$

In which U is strain energy, u is the vertical displacement and $P(u)$ is the load-displacement curve. This integral is only valid for single-degree-of-freedom (SDOF) i.e. the structure only experience one type of deformation which is the vertical displacement. The load-displacement curve represents the vertical resistance of the structural element [16]. Kinetic energy (E_k) is the energy stored by a body that is in motion. It is estimated based on the difference between loss of potential energy and strain energy [12].

The collapse mechanism for a steel frame is generally similar under the case of a column removal. Prior to the removal of the column (See Fig. 2a), the building is said to be in the static equilibrium state because the net energy balance is equal to zero. As soon as the column is instantaneously removed from the frame (See Fig 2b), the unbalance gravity load (from the supported weight and the structural elements) will experience loss of potential energy due to its downward movement in the gravitational pull. This loss of potential energy is then converted into kinetic energy [12] that put the structure in motion [13]. In response to that, the remaining structural elements will attempt to resist the downward motion through absorption and dissipation of the kinetic energy by means of accumulating strain energy [12]. Given that the structure has completely absorbed and dissipated the kinetic energy ($E_k \leq U$), the motion is halted and a new static equilibrium is achieved [15]. The structure is said to achieve arrested collapse [14]. However, if the structure fails to absorb nor dissipate the kinetic energy completely ($E_k > U$), the excess kinetic energy will trigger a collapse mechanism to the structure [12, 14, 15].

There are two possible modes of collapse for a planar steel frame; Contained Collapse Mode (CCM) and Propagate Collapse Mode (PCM) [21]. In the CCM as shown in Fig. 3a, the damage only occurs within the collapsed bay and does not spread to the other bays. This mode of collapse is initiated either by the failure of the beam-to-column connection or the beam itself [22]. It is also founded that this mode of collapse is most likely to occur in a planar frame with strong columns [21]. In contrast, under the PCM as shown in Fig. 3b, the damage is spread to the adjacent bays and subsequently leads to the total collapse of the frame. This collapse mode is triggered when the columns in the vicinity of the removed-column have buckled [14, 15, 21], which later induced loss-of stability to the structural frame [23, 24].

In order to avert the progression of collapse, the structure must be able to accumulate a significant amount of strain energy in order to absorb and dissipate the kinetic energy induced by the loss of potential energy. This condition can be expressed based on the following energy balance:

$$U = E_P \tag{2}$$

Eq. 2 shows that the strain energy must be equal to the loss of potential energy to prevent progressive collapse. This general concept is adopted by researchers for the development of the energy-based Alternate Load Path method.

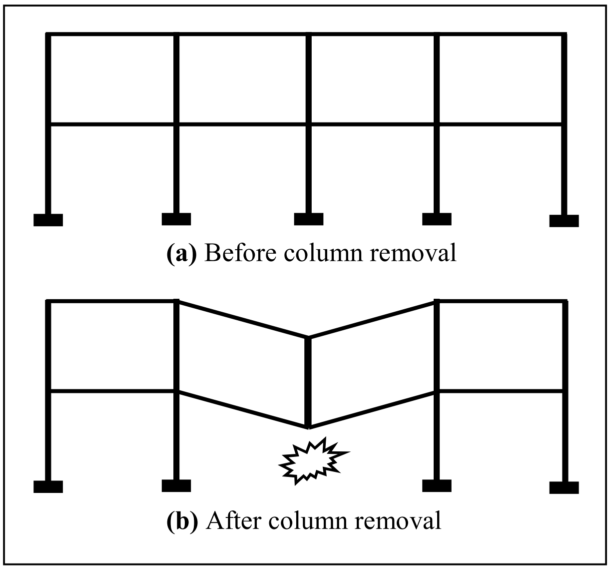


Fig. 2. Progressive collapse for planar frame

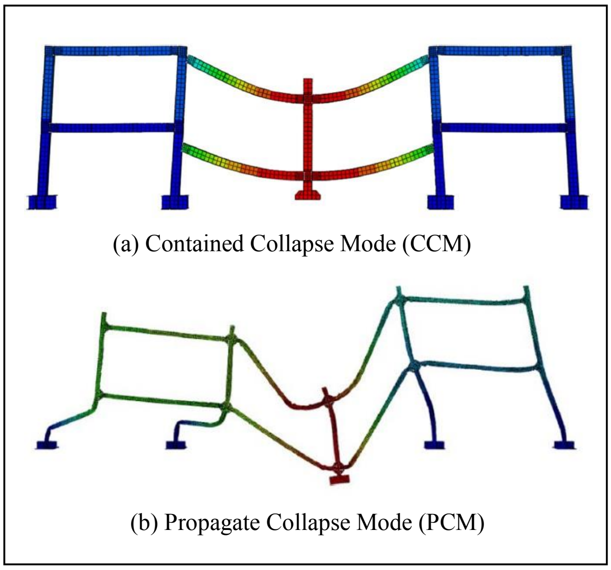


Fig. 3. The collapse mode of planar steel frame [22]

4 Established energy-based progressive collapse assessment

The current research initiative is more focused on developing an efficient yet reliable progressive collapse assessment for the industrial use. The current energy-based progressive collapse assessment is developed based on the same concept of the ALP method and the principle of conservation of energy, even though each method offers different technique of assessment. The state of the art of the energy-based progressive collapse assessment for a structural steel frame is presented herein.

4.1 Linear elastic pushdown analyses and Elastic-plastic analyses

The Linear-elastic pushdown analyses and the Elastic-plastic analyses are two energy-based progressive collapse assessments that were introduced by

Dunsenberry and Hamburger [12]. These methods use kinetic energy as an indicator for progressive collapse, in which it is quantified based on the difference between the loss of potential energy and strain energy. These methods also applicable for moment frame with full restraint.

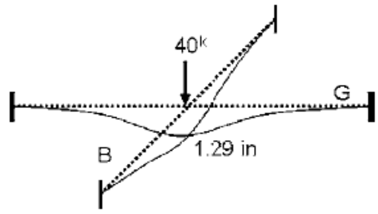
Under the Linear-elastic pushdown analyses [12], a portion of the structural system (i.e. the collapsed bay) is extracted from the frame and the original gravity load is idealised as a concentrated force acting at the location of the removed-column as shown in Fig. 4a. The loss of potential energy is induced due to the downward movement of the original gravity load, whereas the strain energy is estimated based on the load-displacement response which is obtained using linear elastic analysis on the structure. The assessment is done in phases, which include elastic phase, first plastic-hinge phase, and full plastic-hinges phase. For each phase, the energy balance is observed to detect any induction of kinetic energy. Given that the energy balance returns positive value ($E_p - U > 0$), the structure is deemed to not be able to arrest the collapse at this phase. Thus, the assessment is proceeded to the next phase by adding plastic-hinges to the structural system as shown in Fig. 4b, the previous procedure is repeated. The location of the plastic-hinges is predetermined before moving to the next phase. The assessment is halted if the energy balance returns a negative value ($E_p - U < 0$; the structure is able to arrest the collapse) or the analysis has reached the final phase assessment (structure cannot arrest the collapse). Since this method is executed using the linear-elastic analysis, the catenary behaviour cannot be captured and it is only considered the flexural behaviour of the beam only. For that reason, the Elastic-plastic analyses are introduced.

The assessment technique for the Elastic-plastic analyses is generally similar to the Linear-elastic pushdown analyses, but this method incorporates the elastic-plastic behaviour of the beam in the analysis. In another word, the flexural and catenary behaviour of the double-span beam are being considered in the assessment. Two force-displacement models are introduced in [12] that represent the flexural and catenary behaviour of the double-span beam respectively. The strain energy is estimated based on these force-displacement models. To use this method, however, the connections must have sufficient ductility in order to initiate the catenary behaviour. This requirement is not explicitly mentioned in [12].

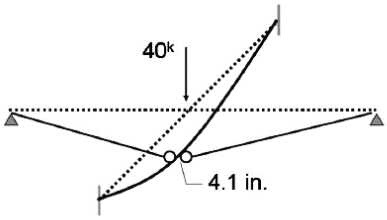
The assessment using these methods are executed on a floor-by-floor basis. It does not consider the column behaviour in the analysis. Thus, separate assessment for column survival is required [12]. These methods are also highly laborious because a series of analyses are required for a single case of column removal. It may not be practical for complex and irregular structures [17].

4.2 Pseudo-static response

The Pseudo-static response curve (Fig. 5) demonstrates the relationship between the gravity loading applied instantaneously (P) and the maximum dynamic displacement (u_d), which represents the dynamic response of the structure. Using the concept of energy, the work done by the gravity load is equated with the strain energy in order to establish the response curve. This response curve can be used for multi-level progressive collapse assessment framework under column removal case [13, 25] or floor impact case [26]. However, only the multi-level assessment framework under column removal case is introduced herein.



(a) Elastic phase



(b) First plastic hinges phase

Fig. 4. Structural configuration for the Linear-elastic pushdown analyses [12]

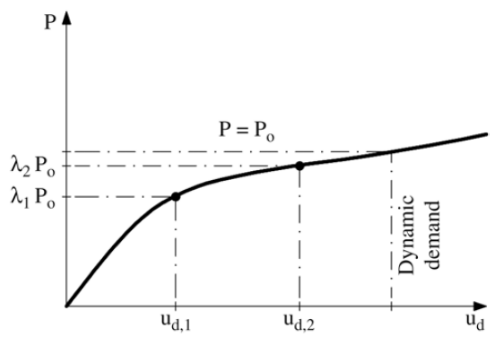


Fig. 5. The Pseudo-static response representing dynamic structural response [13]

The multi-level progressive collapse assessment as proposed by Izzuddin et al. [13] consists of three main stages; (1) generate the non-linear static structural response (load-displacement curve), (2) establish the pseudo-static response, and (3) obtain the connection ductility limit. In the first stage, the non-linear static structural response is generated either from the detailed model or simplified beam model. For the detailed model approach, the analysts are required to model the structure either at low-level sub-structural modelling (Considering individual beam or the floor system) or at high-level sub-structural modelling (Considering the sub-frame within the collapsed bay). The non-linear static pushdown analysis is then executed onto the model to generate the structural response. The accuracy of the structural response is controlled by the analysts based on their modelling techniques. The second approach for generating the non-linear structural response is by using the simplified beam model [27] as shown in Fig 6. The simplified model was first proposed by Izzuddin et al. [13] and was further developed by Stylianidis et al. [27]. The model is based on the behaviour of the double-span beam under the column-removal condition that includes the flexural action, compressive arching action and tensile catenary action developed within the double span beam. This model assumes that the beam section is very rigid in bending. Due to this assumption, the beam will not attain any compression-induced instability effect (i.e. yielding or local buckling) which will subsequently overestimate the response [27]. Attention need to be taken upon using this model, especially for the slender section as it cannot develop effective compressive arching action as compared to a compact section. The connections in the model are to assume to be sufficiently strong to transmit the compressive arching action, and the bending moment due to eccentricity is assumed to be absent during the tensile catenary stage. The material strain hardening effect is omitted in this simplified model, which can underestimate the response [27]. It is also noted that this model does not consider the composite behaviour of the concrete slab and the steel beam. Such omission may affect the overall non-linear response of the beam. A recent study [28] has shown that increase in the slab reinforcement ratio may reduce the compressive arching capacity of the composite beam, but the overall non-linear response of the composite beam would amplify due to the increase in flexural capacity at the hogging section.

Once the non-linear static response has been generated, the next stage is to establish the pseudo-static response by using the concept of energy. In this framework, the loss of potential energy is equal to the external work done by the gravity load, whereas the strain energy is derived from the load-displacement curve (non-linear static response). If the load-displacement curve is generated from a low-level sub-structural configuration (i.e. beam level), the established pseudo-static response represents the dynamic response for that particular sub-structural configuration. In order to obtain the dynamic response of a higher-level of sub-structural configuration (i.e. the floor system or sub-frame configuration), the grillage approximation technique [25] can be adopted. This technique approximates the overall response of the floor system by considering the relationship between the individual beam deformation mode and the floor system deformation mode, which is known as the deformation compatibility factor (β). There are three requisites that must be accounted for upon using this technique [29]; (1) the interaction of the adjoining structural elements with the collapsed bay must be represented with suitable boundary conditions, (2) the columns in the vicinity of the collapsed bay have sufficient strength to sustain the load redistribution, and (3) the loading and structural properties (material and geometrical properties) of the floors above the removed column are similar. If these requisites are met, a single floor system (Fig. 7) can be considered in order to establish the higher-level pseudo-static response. Under grillage approximation technique, the deformation mode of the grillage assembly (Fig. 7) is assumed to undertake the linear deformation mode and the gravity load is assumed to be distributed onto the individual beams via convectional tributary area method. Later study has founded that assuming linear deformation mode would underestimate the deformation of the secondary beam, and hence underestimate the pseudo-static response of the overall floor system [30]. It is because linear deformation mode is not suitable for the analysis of a structure with large deformation and highly non-linear [17]. It is also founded that the assumption made on the gravity load distribution has no significant effect on the overall pseudo-static response [30].

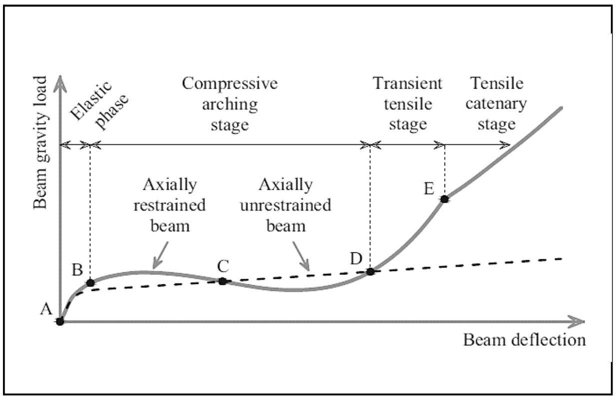


Fig. 6. Non-linear static structural response based on the simplified beam model [27]

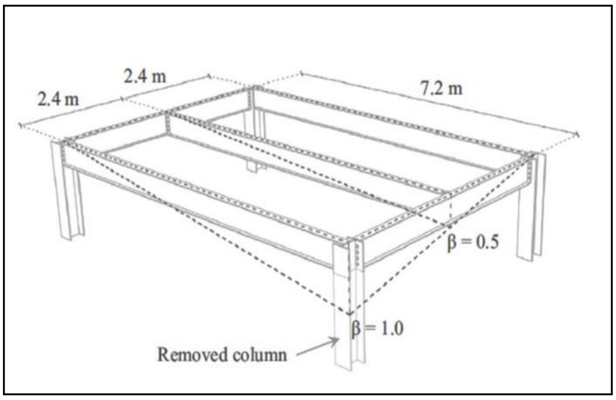


Fig. 7. Deformation mode of the grillage assembly [29]

The last stage for this framework is to obtain the connection ductility limit, which acts as the acceptance criteria for this method. The ductility limit is based on maximum connection deformation, u_{df} (ductility capacity) that is acquired either from experimental work or numerical analysis [25]. The dynamic displacement induced by the gravity load, also known as ductility demand (which obtained from the established Pseudo-static response), is compared with the ductility capacity to justify the vulnerability of the building against progressive collapse.

This framework considers the behaviour of connections in resisting progressive collapse, thus making it suitable for assessment of a frame with connections of various stiffness and restraint condition. It is noted that the behaviour of columns in resisting progressive collapse are not being considered in this framework (if the simplified beam model is being utilised). Hence, the risk of progressive collapse due to column instability [24] cannot be assessed using this method.

4.3 Collapse spectrum

The Collapse spectrum (Fig. 8) describes the relationship between the column gravity load and the dynamic chord-rotation of the double-span beam, which represents the dynamic response of a double-span beam under the column removal condition. This model was developed by Lee et al. [16] for the progressive collapse assessment of a welded moment steel frame. It can quickly predict the maximum nonlinear dynamic deformation demands of the frame with respect to the applied gravity load. This deformation demand is then compared to the acceptance criteria (deformation limits) as outlined in the UFC 04-23-03 [6] or the GSA Progressive Collapse Guideline [1]. The characteristic of the Collapse spectrum is quite similar to the Pseudo-static response [11] in term of its function, the development of the model, and the method of assessment. However, unlike the Pseudo-static response, no effort is required by the analysts to establish the dynamic response because it has been predefined in the Collapse spectrum. The dynamic response of the double-span in the Collapse spectrum is valid for a wide range of beam section and span length.

The principle of conservation of energy is used in the development of the Collapse spectrum. A simplified vertical load resistance versus chord rotation model [16] as shown in Fig. 9 is used to derive the strain energy of the structural element. This model is based on the non-linear behaviour of a double-span beam loaded statically at the mid-span with a concentrated force. The concentrated force is used to represent the axial load in the column prior to its removal. Unlike the simplified beam model proposed by Izzuddin et al. [13], this model only considers the flexural and tensile catenary behaviour of the double-span beam. No compressive arching behaviour is present in this model because the double- span beam is not assumed to be very strong in bending [27]. Other than that, the concrete slab, which contributes to the bending stiffness of the composite beam, has not been included in the numerical investigation of the double-span beam response. For that reason, the double- span beam is unable to develop the compressive arching action because of the insufficient bending stiffness. In a sense, omitting the compressive arching behaviour may indicate that the effect of compression-induced instability (i.e. yielding or local buckling) has been considered implicitly in the double-span beam, which allows the model to be applied for various section sizes. Furthermore, this model assumes the welded connections and its respective joint (the panel zone) to be stronger than the beam. Thus, any inelastic behaviour will take place in the beam only [16]. This assumption is valid only if the quality of the welded connections is controlled under stringent condition [31], which is quite impossible to achieve in practice as the weld quality may vary from one another.

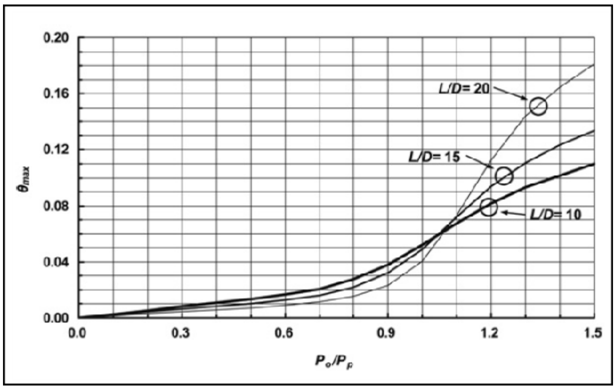


Fig. 8. The Collapse spectrum [16]

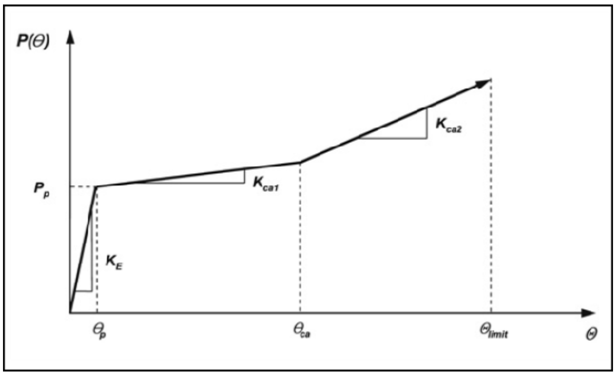


Fig. 9. The simplified vertical load resistance versus chord rotation model [16]

One of the limitations of the Collapse spectrum is that it excludes the role of columns in progressive collapse assessment as it is more focused on the behaviour of the beams in resisting the collapse. Separate analysis on the columns response must be executed in order to complete the assessment. The assessed frame must also have sufficient strength and ductility at the welded connections to allow the presumed tensile catenary action to take place [32, 21]. Nevertheless, recent research has shown that fully welded connections have insufficient ductility to develop effective tensile catenary action [33]. The Collapse spectrum also can be used for the welded planar frame assessment under the middle column-removal case (Fig. 2) only and not for edge column-removal case (Fig. 7). It is noted that the simplified model [16] does consider the effect of axial restraint provided by the neighbouring structural elements at both end of the double-span beam. The axial stiffness provided by these neighbouring structural elements is essential in developing the tensile catenary action at large deformation [32- 35]. However, under the edge column-removal case, the axial restraint is present only at one end of the double-span beam. Such consideration has not been included in the simplified model, thus limiting its application to middle-column removal case. Apart from that, the assessment of irregular frame (frame with varying beam span-length and section sizes) using the Collapse spectrum may not be a sensible approach as this model does not consider structural irregularity in its development.

4.4 Energy-based nonlinear static progressive collapse analysis method (ENSM)

The ENSM is an alternative method to the Collapse spectrum for the progressive collapse assessment of a welded moment frame. This method estimates the dynamic deformation demand of the frame, which will then be compared with the deformation limits given in the UFC 04-23-03 [6] or the GSA document [1]. An energy balance equation is used in order to estimate the dynamic response. The equation is derived based on the energy equilibrium between the external work by the unbalance gravity load and the induced strain energy of the structural element, which can be expressed as follows:

$$P_o u_{max} = \sum_i \int_0^{u_{max}} R_i(u)\, du \qquad (3)$$

Given that P_o is the idealised gravity load (concentrated load), u_{max} is the maximum dynamic vertical displacement and $R_i(u)$ is the vertical resistance of each individual double-span beam. Eq. 3 was first developed by Lee et al. [16] that only considers the non-linear static response of the double-span beam under the column loss event. The simplified vertical load resistance versus chord rotation model (Fig. 9) is used to express the behaviour of the double-span beam, which has been described in Subsection 4.3. The contribution of slab in resisting progressive collapse was then incorporated into the energy balance equation by Kim et al. [18], which is expressed as follows:

$$P_o u_{max} = \sum_i \int_0^{u_{max}} R_i(u)\, du + U_{cSab} \qquad (4)$$

In which U_{cSab} is the strain energy induced by the slab. The model representing the load-resisting mechanism of the slab under the column loss scenario is introduced herein

In response to sudden or notional column removal, the slab has two important roles in resisting progressive collapse. First, it strengthens the initial stiffness of the structure through composite action between the steel beam and the concrete slab [36, 37]. It is said that the slab enhances the flexural capacity of the composite beam in the elastic stage and subsequently permits the compression arching action to progress. Second, it redistributes the load at high deformation state via tensile membrane action, which is developed within the reinforcement bars (or the welded steel wire mesh) and the metal deck [34, 38]. These actions can be mobilised only if there is sufficient shear resistance between the concrete slab and the steel beam in order to maintain the composite behaviour [28, 32, 34, 36]. In the ENSM slab model, only the tensile membrane action from the steel mesh is considered. The influence of metal deck is ignored in this model because it is claimed that the metal decks are not effectively anchored to the beam via spot welding [18]. The strain energy generated by the welded steel wire mesh is derived based on the elastic, perfectly plastic stress-strain relationship of individual steel wires (Fig. 10). It is pointed out that this model only incorporates the strain energy of the steel wires parallel to the double-span beam. The steel wires perpendicular to the double-span beam is neglected as it is not securely anchored across the span length [18].

As depicted in Fig. 10, no fracture limit is included in the material properties of the steel wires. It can be deduced that the steel wire has unlimited ductility and hence infinite strain energy capacity. Caution must be taken upon using this method, especially if the wire mesh consists of high yield strength steel (low ductility alloy), as the tensile membrane action cannot be mobilised if the steel wires have fractured. Apart from that, the steel wire must be securely fixed to the edge of the collapsed bay in order to maintain the tensile response of the slab. The fixity of the wire mesh to the edge of the collapsed bay is governed by the shear resistance provided by the shear studs [28, 32, 34, 36], the bond strength between the steel wire and concrete [39], and the continuity of the steel wire to the adjacent slab. The ENSM slab model also excludes the composite flexural action induced by the slab at the early collapse stage, which contributes significantly to the overall vertical resistance of the structure [36].

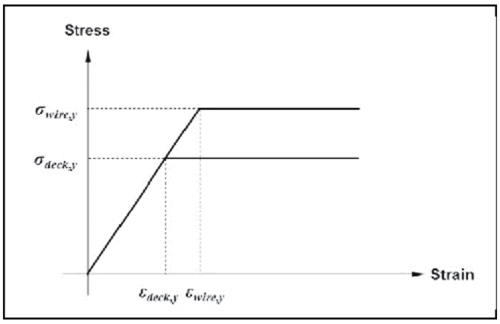


Fig. 10. The elastic, perfectly plastic stress-strain relationship of steel wires [18].

It is reported that the structural response estimated using Eq. 4 bears no significant difference than the one estimated using Eq. 3 [18]. This is because little consideration is made on the slab load-resisting mechanism in the ENSM slab model, in which it only accounts for the tensile membrane action from steel wires. Other key load-resisting mechanisms, such as composite flexural action, have been omitted. Thus, it is possible to adapt Eq. 3 without considering the ENSM slab model in order to reduce the analytical burden as solving Eq. 4 is based on trial-and-error technique.

Since the Collapse spectrum and the ENSM is built based on the same simplified beam model [16], the limitations of the Collapse spectrum (See Section 4.3) also apply to the ENSM.

4.5 Energy-based Partial Pushdown Analysis (EPP)

The pushdown analysis is the adaptation of the pushover analysis, which is a common method used in assessing the structural resistance against the seismic action. However, instead of the structure is pushover laterally with an artificial horizontal force, the pushdown analysis measures the structural resistance against column loss by applying amplified gravity load vertically to initiate a response. This method was first introduced by Khandelwal and El-Tawil [40] that implement the static force-based approach in the analysis. Later research founded that the force-based pushdown analysis overestimates the structural resistance against progressive collapse [41]. The concept of energy is then incorporated into the pushdown analysis by Xu and Ellingwood [17] that can approximate the dynamic structural response by using only static analysis technique. This method is known as Energy-based Partial Pushdown analysis (EPP). The procedure for the EPP method is basically similar with the force-based pushdown analysis, in which the structure is pushed vertically at the collapsed bay by the amplified gravity load known as *Pushdown Force* (Fig. 11a). The gravity load on the bays adjacent to the collapsed bay are maintained to their original values. The structure is then analysed under the non-linear static analysis technique. The main difference between the force-based pushdown method and the EPP method is that the EPP method monitors the energy balance of the structure throughout the loading process. As usual, strain energy and external work (loss of potential energy) are the two energy-based parameters that will be observed in the analysis. If the external work is equal to the strain energy, a new state of equilibrium is achieved and the structural responses (such as deformation, end moment, etc.) at this state represent the actual dynamic responses under the original gravity load at the collapsed bay. However, if there is no balance between the external work and the strain energy, the pushdown forces are then amplified and the procedure is repeated until energy equilibrium is achieved. This process is terminated once the predefined acceptance criteria (deformation limit or force limit) have been violated and the structure is deemed to collapse due to insufficient collapse-resisting capacity [17]. The acceptance criteria for this method is to be taken from the UFC 4-023-03 [6].

Under the EPP method, the effect of damping is assumed to be absent. When damping effect is neglected, overestimation of the dynamic structural responses will occur, but the difference is not significant [17]. This method also ignores the initial displacement induced by the gravity load prior to the notional column-removal due to the fact that the initial displacement is trivial as compared to the ultimate response (deformation) [17]. Such strain energy induced by the initial deformation is also insignificant.

Unlike the other energy-based methods introduced in the previous subsections, the EPP method does take into account the overall structural stability due to column failure. In the EPP method, the structure is analysed as a whole frame instead of a reduced structural configuration (i.e. beam level). Thus, the columns behaviour in response to a notional-column removal has been indirectly included in the analysis. Other structural responses such as compression arching action, tensile catenary action and even slab tensile membrane action can also be incorporated in the analysis, which depends on how detailed does the analyst model the structural element to be. For instance, the analyst can include slabs in the skeletal steel frame using shell element in order to simulate the tensile membrane response.

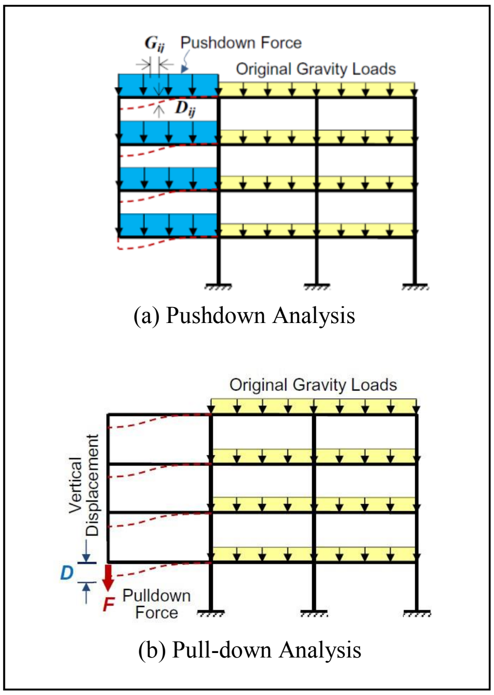


Fig. 11. The loading pattern for the Pushdown and Pull-down analysis [10].

One of the noticeable limitations of the EPP method is the iterative analysis involved for a single case of column-removal. A number of non-linear static analyses with the increasing magnitude of the *Pushdown force* are executed in order to find the energy balance within the structure. It would be a laborious task due to repetitive analysis for each column-removal case, thus making it unsuitable for a high number of column-removal cases. Other than that, this method also requires the analyst to handle a significant amount of data. For a case of column-removal, the analyst needs to obtain the load-displacement curve at each node on the collapsed bay for each floor so as to estimate the strain energy and subsequently the external work [10]. Consequently, the analytical burden for a single case of column-removal is greatly increased, especially for a multilevel frame. It is also reported that the Dynamic Increase Factor (DIF) used in the analysis to amplify the *Pushdown force* is prone to estimation error [9].

4.6 Energy-based Pull-down Analysis

The Energy-based Pull-down analysis is the innovation of the Energy-based Partial Pushdown analysis (EPP) that was first introduced by Liu [9] and further developed by

Liu and Pirmoz [10]. This method enforced the same concept and procedure as the EPP method except for the loading pattern. Instead of the amplified gravity loads applied at the collapsed bay, the *Pushdown force* is changed into a concentrated force applied at the upper node of the removed-column (Fig. 11b). This concentrated force is known as *Pull-down force*. It is noted that all the gravity loads at the bays adjacent to the collapsed bay are maintained to their original value. Even though with different loading pattern, the Pull-down analysis still able to predict the non-linear dynamic response as accurate as the EPP method [10].

The Energy-based Pull-down method was first developed along with the use of DIF [9]. It was later founded that, without the use of DIF, this method still able to estimate the dynamic structural response accurately [10]. Thus eliminating the prediction error related to the usage of DIF. In addition, the number of data to be handled by the analyst is greatly reduced with the introduction of the *Pull-down Force*. It is because only a single load-displacement curve is to be established by the analyst for each case of a collapse in order to estimate the strain energy and the external work. Nonetheless, this method still unable to eliminate the need for the iterative analysis in seeking for the energy balance. The analytical burden would be affected if the number of collapse scenarios is numerous.

5 Eurocode standard as a code of practice

Most of the researchers adopt the UFC 4-023-03 [6] and GSA document [1] as their basis for developing the energy-based ALP methods due to its comprehensive and detailed guideline. However, there is little to no consideration is given towards the requirements given in the Eurocode standard [7] as this code of practice only gives general instructions rather than specific procedures. There are two requirements that are founded to render the efficiency and reliability of the established energy-based progressive collapse assessment [9, 10, 12-18]. Those requirements are the location of column removal and the acceptance criteria.

5.1 Location of column removal

In the UFC 4-023-03 [6] and GSA document [1], the location of notional column-removal has been clearly stated herein, which includes external middle-column, external edge-column and internal column. With the definite number of column-removal cases, the progressive collapse assessment using the established energy-based methods would be a feasible and practical task. On the contrary, the EN 1991-1-7:2006 [7] requires the notional removal of all columns in the structure one at a time in order to evaluate the ability of the intact structural elements to avert progressive collapse. As a result, the analysis burden, as well as the time consumed to complete the assessment, will subsequently be increased, especially for the assessment of a tall building. Some of the energy-based ALP methods that are affected (in term of their efficiency) by this requirement are the EPP method [17] and the Energy-based Pull-down method [10]. Since the number of column-removal cases has escalated, the number of iterative analyses would substantially be increased. Likewise, the efficiency of the Pseudo-static method [13] would also be affected by this requirement as a significant amount of time is required to establish the pseudo-static response for each case of collapse. The Collapse spectrum [16] and the ENSM [18] would be the best alternative for efficient progressive collapse assessment, but these methods only applicable for external middle-column removal.

5.2 Acceptance criteria

The acceptance criteria stipulated in the EN 1991-1-7:2006 is based on collapsed area, in which the building is said to experience progressive collapse if the area of destruction is larger than the minimum of 15% of total floor area of a storey or 100 m^2 [7]. The collapse potential of a building cannot be justified by using these acceptance criteria because it does not reflect the real structural capacity of the assessed building. The UFC 4-023-03 [6] and the GSA document [1], on the other hand, use the deformation-limits and force-limits as their acceptance criteria, which are applicable for the different type of elements (beams, columns, and connections) and structures (steel structure, reinforced concrete structure, etc.). These limits are developed based on the actual structural responses that were obtained through experimental testing and numerical investigation [6]. Most of the energy-based ALP methods [10, 16-18] utilise these acceptance criteria in their assessment framework as they were developed on the basis of those two standards [1, 6]. In regards to that, since the EN 1991-1-7:2006 uses the collapsed area as the acceptance criteria, no conclusion can be drawn based on the analysis output of the established energy-based ALP methods [10, 16-18], which are mostly forces and deformations based. Nevertheless, there are a few energy-based ALP methods that have its own acceptance criteria, which are universally applicable regardless to the preferred code of practice. Those energy-based methods are the Linear elastic pushdown and Elastic-plastic analyses [12], and Pseudo-static response [13] which uses kinetic energy and ductility limit respectively as the acceptance criteria.

6 Conclusion

The energy-based Alternate Load Path (ALP) method is the best substitute to the force-based ALP method in bringing forth an efficient and reliable progressive collapse assessment. It is pointed out that the analysis under the energy-based approach is not affected by the inertial effect, unlike the force-based approach. The analytical burden can also be reduced by using the energy-

Table 1. Energy-based Alternate Load Path Method for steel frame progressive collapse assessment

Energy-Based Method	Type of analysis			Type of connection			Location of column removal			Structural Response						Acceptance criteria				
	Linea Static	Non-Linear Static	Non-linear Dynamic	Rigid	Semi-rigid	Simple	External, Middle column	External, Edge column	Internal column	Flexural action	Catenary action	Compression arching action	Composite Flexural action	Tensile membrane action	Column stability	UFC 4-023-03 [6]	GSA Document [1]	EN 1991-1-7:2006 [7]	UK Building Document [8]	Standalone
Linear elastic pushdown [12]	✓			✓			✓		✓	✓										✓
Elastic-plastic analyses [12]		✓		✓	✓		✓			✓	✓									✓
Pseudo-static response [13]		✓		✓	✓	✓	✓	✓	✓	✓	✓	✓								✓[3]
Collapse spectrum [16]		✓		✓			✓[1]			✓	✓					✓	✓			
Energy-based nonlinear static progressive collapse analysis method (ENSM) [18]		✓		✓			✓[1]			✓	✓			✓		✓	✓			
Energy-based Partial Pushdown Analysis (EPP) [17]		✓		✓	✓[2]	✓[2]	✓	✓	✓	✓	✓	✓[2]	✓[2]	✓[2]	✓	✓				
Energy-based Pull-down Analysis [10]		✓		✓	✓[2]	✓[2]	✓	✓	✓	✓	✓	✓[2]	✓[2]	✓[2]	✓	✓				

[1] Welded connection with unlimited ductility
[2] Ductility limit is based on experimental or numerical investigation
[3] Depends on the level of detail employed in the structural modelling

based approach as this approach can be executed using the non-linear static analysis instead of the time-consuming non-linear dynamic analysis.

The concept of energy and its relation to progressive collapse has been introduced in this paper. Based on the principle of conservation of energy, progressive collapse can be avoided if the induced kinetic energy is fully absorbed or dissipated by the structure. In other words, the structure must be able to accumulate sufficient strain energy in order to absorb the loss of potential energy. This concept is adopted by researchers for the development of the energy-based ALP method.

The established energy-based ALP methods for progressive collapse assessment has been introduced and reviewed in this paper, which covers the development of the method, the assumptions, limitations, and the acceptance criteria. Table 1 shows the summary of the review. From the review, it is founded that most of the energy-based ALP methods (which utilise a simplified model representing the structural response) do not consider the column instability in their assessment framework. A separate analysis of the column response is needed in order to complete the assessment. However, there is still no established method for instability-induced progressive collapse assessment based on a simplified model.

The applicability of the established energy-based ALP methods in regards to the Eurocode provisions has also been discussed. Increase in analytical burden and unreliable acceptance criteria are the two issues that affect the practicability of the energy-based ALP methods for the industrial use. It is evident that the established energy-based methods are not designed to meet with the requirements specified in the Eurocode standards. It would be a loss for end-users, especially for the countries that implement the Eurocode standards as the codes of practice, because the energy-based approach on progressive collapse assessment can be beneficial towards improving the efficiency and reliability of the force-based ALP methods. Consideration must be taken regarding the provisions given in the Eurocode standards as well as the UFC 4-023-03 and the GSA document so as to make the energy-based ALP method universally relevant to any code of practice.

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