DIRECT DUCTILITY DISTRIBUTION METHOD FOR TALL BUILDING DESIGN

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Abstract: In many low to moderate seismicity regions, tall and irregular buildings are common. Usually, because of their height and irregularities as defined by most current design standards, these buildings have to be designed using complex performance-based methods with non-linear dynamic analysis, or they have to be designed to be essentially elastic. This either results in a complex design procedure, or in significant overstressing of some vulnerable members. As a result this drives the design of the overall lateral stability system to become unnecessarily difficult. Based on the incremental substitution procedure (Ho & Kuang, 2014), control on the distribution of damages could be gained through reversing the evolution process (Ho & Kuang, 2012). This created the platform for a new seismic design methodology for new buildings, which is named as the direct ductility distribution method.

Introduction

Thousands of skyscrapers are being built in metropolises around the world. Ground floor spaces are demanded for the multi-purposed architectural arrangements, and structural gymnastics are frequently used to achieve irregular, open structures with maximum lettable spaces (Scott et al., 1994). This results in soft storey structures which are known to perform badly under earthquakes. Very often, these buildings with soft storeys are built in low to moderate seismicity regions, where the seismic effects are not explicitly considered in the design. With the increasing awareness of potential earthquakes (EEFIT, 1991; Pun & Ambrasey, 1992; Kuang, 1998; Park, 1998; Free et al., 2004; Pappin et al., 2008; BD, 2014), the seismic performance of such tall buildings is of interest to the government, owners, users and engineers.

In common practice for the design of tall buildings, engineers would analyse the seismic response by the response spectrum analysis method (CEN, 2004; MOHURD, 2010; ASCE, 2013) with elastic vibration modes based on gross section properties. In order to achieve an economic design, the elastic response spectrum can be reduced to the design response spectrum according to the ductility capacity of the structure using force reduction factors. The design load for some critical members may in some cases need to be further amplified using the over-strength factor, or protected by the capacity design principle. For irregular structures, a major drawback of this simplified approach using elastic vibration modes is that the resulting design could be rather qualitative, while the exact values of the reduced design load can often be debatable, especially since the actual member properties can be significantly different from the gross section properties (Priestley, 1993) that are typically used. These methods also impose significant difficulties in understanding the actual seismic behaviour of the designed building. For example the failure mode can not be visualised in general, and the concept of sacrificial elements for the life safety performance limit cannot easily be justified. As a result such buildings would have to be designed to be essentially elastic, which creates a major challenge in their design. The overstressing of some vulnerable structural members may often drive the design of the overall lateral stability system to become unnecessarily difficult. For example the lintel beams in a coupled shear wall system are often found to be significantly overstressed, while the adjoining walls could stay well within its elastic range.

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The conventional design approach of applying a global load reduction factor could either result in over-detailing the shear walls, or alternatively in over sizing the lintel beams.

Another example of the difficulties caused by current design methods would be in the application to many mixed development in low to moderate seismicity regions. In such kind of developments, a larger podium structure is often architecturally required to tie together a few high-rise towers on the same site. Because of the uncertain force distribution among the podium and the towers, engineers can usually only adopt the lowest reduction factor among the combined structures. Nonlinear time history simulations may be conducted in the above situations to better capture the more realistic seismic behaviour. However, for various practical reasons, the sophisticated simulations and performance based design methodology may often found to be unnecessarily difficult and time-consuming. This is especially true in many low to moderate seismicity regions like the U.K., Hong Kong, Macau, Singapore, Vietnam and Mumbai, where the actual usable level of ductility could be limited (Scott et al., 1994; Kuang et al., 1998; Ho, 2014; Su et al., 2014).

To capture the nonlinear multimodal dynamic seismic behaviour of tall and irregular buildings, the substitution procedure (Shibata & Sozen, 1976), which has also been frequently called as the secant method, is well recognised as a suitable method of analysis (ATC, 1996; Li, 2005; COLA, 2013; Su et al., 2014). The method is particularly useful for low to moderate seismicity regions where irregularities are often significant, where the seismic load may not necessarily be the dominated action, and where regional strong motion records are usually unavailable. However, conventional substitution procedures also have their limitations. For example Priestley (1993) has pointed out that the effects of unsymmetrical hysteresis in reinforced concrete columns cannot easily be captured, which can results in significant errors in the predicted internal forces and displacement capacities. The sequence of yielding between different members in a building during the earthquake attack is also a concern. When a part of a building start to yield, load will be redistubuted on one hand to the parts in parallel, and on the other hand the yielded part will also protects the parts in series from attracting excessive load. Yet since conventional substitution procedure does not take the sequence of yielding into consideration, the accuracy of the predicted load path is in question, and the solution process may also be unstable.

Ho & Kuang (2014) modified the conventional substitution procedure based on the classical time domain equation of motion to become the incremental substitution procedure (ISP), which is engineered to straightforwardly capture the multimodal interacting nonlinear failure processes of tall and irregular buildings under earthquake attacks using the elastic response spectrum input defined for the conventional design approach. Major improvements have been made by introducing pseudo static offsets, ductility measures and hypothetical load histories in order to make it a complete theory. However, the procedure was primarily developed for the assessment of existing buildings, which may not be ideal for the design of new buildings. Yet according to the theory, the procedure may be reversed in some cases, as a means of controlling the distribution of damage among different parts of a building. Gaining such control has created the platform for a new seismic design methodology for new buildings, which is named as the direct ductility distribution (D3) method. In the proposed method, different levels of detailing are strategically assigned to different parts in a building according to a predefined sway mechanism. For buildings with protected vertical load paths, member forces and storey drifts may then be obtained directly by conventional response spectrum analysis without any iteration. An example building was designed by the direct ductility distribution method and is presented below. The predicted seismic performance is found to be very comparable with the results from nonlinear time history (NTH) simulations.

Analytical Derivation

The analytical derivation may begin from defining the ground motion time history. According to the incremental substitution procedure, the ground motion excitation may be assumed as a hypothetical ground motion containing infinitely many small segments of shocks which are repeating in shape, but with a non-decreasing intensity with time, as illustrated in Figure 1. The same idea is also adopted in the conventional pushover methods, shake table tests, and the more recently developed incremental dynamic analysis (Vamvatsikos & Cornell 2002).

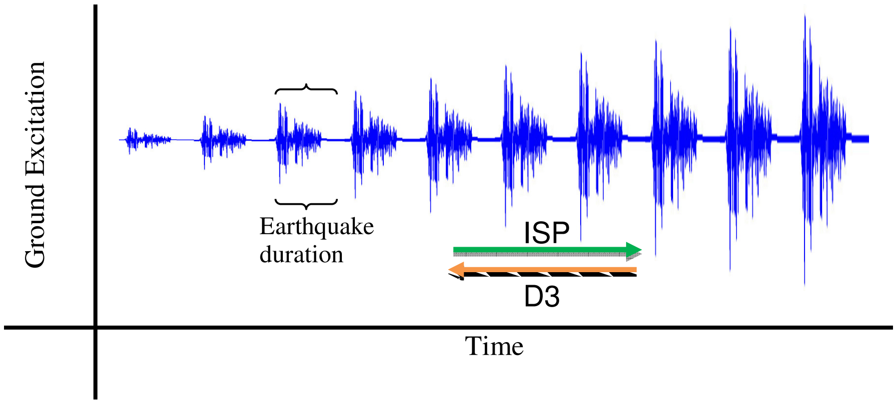


Figure 1. Illustration on hypothetical ground motion

As illustrated in Figure 1, each segment of shocks carries the ground motion of a seismic event. The major difference between the load history in conventional shake table tests and the proposed here is that the former has finite number of segments, while the latter is infinite. Adopting this ground motion is conservative as it is known that the infinitely long duration of excitation will worsen the damage to a structure and amplify the response. Yet this may still be acceptable for practical applications as a simplified alternative. According to the incremental substitution procedure, the general equation of motion at the i^{th} segment of the hypothetical ground motion can be rewritten as;

$$\mathbf{M}\ddot{\mathbf{x}}_i + \mathbf{C}_{i-1}\dot{\mathbf{x}}_i + \mathbf{F}_{i-1} + \mathbf{K}_{i-1}\mathbf{x}_i = -\mathbf{M}\mathbf{r}\ddot{x}_{g,i} + \mathbf{F}_{ext} \tag{1}$$

where $\mathbf{x}$ is the displacement of structure relative to the ground, x_g is the ground displacement, $\mathbf{M}$ is the constant seismic mass, $\mathbf{C}$ is the viscous damping, $\mathbf{F}$ is a pseudo static offset of internal forces, $\mathbf{K}$ is the unloading-reloading stiffness which is non-increasing over time, $\mathbf{r}$ is the directional vector, and $\mathbf{F}_{ext}$ is a constant external force applied to the structure. Since the method being developed here is primarily for a direct design method for new buildings, it may be difficult in practice to estimate the pseudo static internal force offset $\mathbf{F}$ without any iterations. For simplicity, the offset is assumed to be minimal in this paper. This simplification is effectively assuming that the hysteresis in the loading-unloading cycles of the structure is symmetrical at the system scale as defined according to the incremental substitution procedure. For typical building applications without significant net permanent static horizontal load, this would practically imply that the vertical gravity load paths should be typically kept essentially elastic. Exceptions could be found, but the condition of symmetrical hysteresis should be carefully examined. For example, weakly coupled walls and columns with low axial load ratio, or foundations that are designed to rock in seismic events may be allowed to be designed to enter their nonlinear range in the proposed design method in this paper. The response can be separated into the static part and the dynamic part as shown in Equation 2, which the static response $\mathbf{x}_{sta}$ and the dynamic response $\mathbf{x}_{dyn}$ can be obtained by Equation 3 and Equation 4 respectively.

$$\mathbf{x}_i = \mathbf{x}_{i,sta} + \mathbf{x}_{i,dyn}$$

$$\mathbf{K}_{i-1}\mathbf{x}_{i,sta} = \mathbf{F}_{ext}$$

$$\mathbf{M}\ddot{\mathbf{x}}_{i,dyn} + \mathbf{C}_{i-1}\dot{\mathbf{x}}_{i,dyn} + \mathbf{K}_{i-1}\mathbf{x}_{i,dyn} = -\mathbf{Mr}\ddot{x}_{g,i} \qquad \text{(2) to (4)}$$

Note that the expressions of Equation 3 and Equation 4 are very similar to the standard expressions for linear systems, thus conventional analysis techniques and software can be utilised in this design method without significant modification. This also allows Equation 4 to be solved as a classical modal analysis problem, and Equation 4 can be rewritten as;

$$\mathbf{x}_{i,dyn} = \sum q_{i,j}\boldsymbol{\phi}_{i-1,j} \qquad (5)$$

where $\boldsymbol{\phi}$ and q are the mode shape and modal response, and j is the mode number considered. The modal response q can be solved by uncoupling Equation 4 as;

$$m_{i-1,j}\ddot{q}_{i,j} + 2\xi_{i-1,j}\omega_{i-1,j}\dot{q}_{i,j} + \omega_{i-1,j}^2 q_{i,j} = -\Gamma_{i,j}\ddot{x}_{g,i} \qquad (6)$$

where m is the modal mass, ω is natural frequency, Γ is the participation factor of the vibration mode, and ξ is the equivalent modal damping as suggested in the original substitution procedure (Shibata & Sozen, 1976) and capacity spectrum method (ATC, 1996). Further references (Gulkan & Sozen, 1974; Iwan & Gates, 1979; Calvi, 1999; Miranda & Ruiz, 2002; Priestley, 2003; Ho, 2006) may be referred to in the literature. And more importantly, the modal amplitude q can thus be read directly from a response spectrum defined according to most current design standards for further modal combination. Together with a set of presumed distribution of ductility demand μ for members in different portions in the structure, the dynamic properties of the structure can be obtained in the form of equivalent stiffness and damping. Note, an assumption is made here that the set of presumed ductility demand distribution is valid, and this would need further justification, as discussed in the later part of this paper.

For practical design purposes, it would be ideal to keep the necessary information to a minimum. The simple elastic-perfectly-plastic hysteresis with no hardening may be a reasonable model to adopt. In this model, only two parameters would be necessary to define the hysteresis, which are the initial stiffness $\mathbf{K}_{ini}$ and the yield strength $\mathbf{F}_y$. Figure 2 shows the relationships between the set of load-displacement parameters in the hysteresis. Mathematically, the expression of such relationships at the yield surface can be described as;

$$\mathbf{K}_{ini}\mathbf{x}_y = \mathbf{F}_y$$

$$\mathbf{x}_i = \mu\mathbf{x}_y$$

$$\mathbf{K}_i\mathbf{x}_i = \mathbf{F}_y$$

$$\mathbf{K}_i = \frac{1}{\mu}\mathbf{K}_{ini} \qquad \text{(7) to (10)}$$

Where $\mathbf{x}_y$ is the yield displacement, and the ductility demand μ in the segment i must not be smaller than that in segment i-1.

During the design process, the damping ratios in each members are first determined by Equation 11.

$$\xi = \left(0.05 + 0.637\kappa \frac{\mu - 1}{\mu} \right) \times 100\% \tag{11}$$

Following the expression in the capacity spectrum method (ATC, 1996), the κ factor is also introduced here, as a correction factor to account for the effects of pinching and degradations in the actual hysteresis. Other form of expressions could also be adopted. Equation 11 is adopted in this paper for simplicity only.

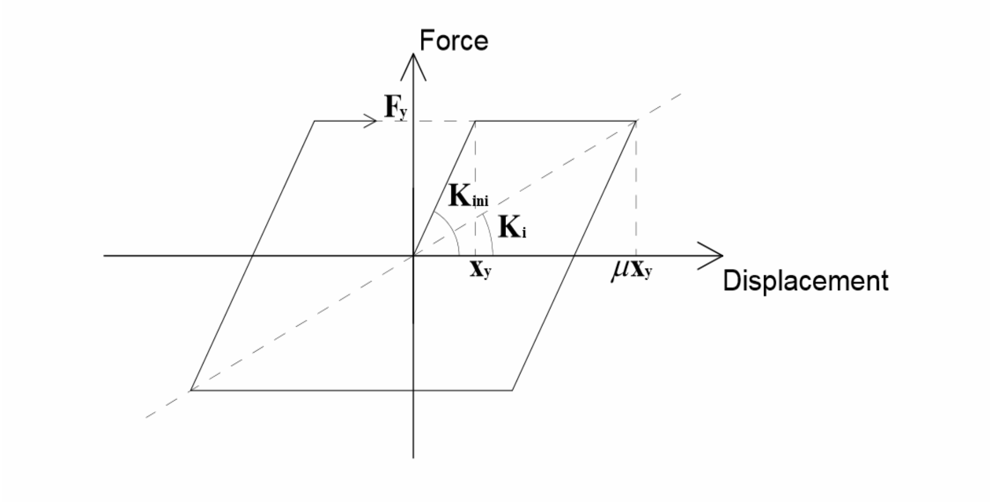


Figure 2. Illustration of elastic perfectly plastic hysteresis

The factor should be calibrated against the actual hysteresis. However, since the proposed method in this paper is for the design of new buildings, such information may not necessarily be readily available in real design practice. In this situation, the κ factor may be taken as 0.33 with an upper limit on the maximum damping ratio ξ at 20% as suggested in ATC 40 (ATC, 1996) for flexural members that are not being specially detailed for ductility in many low to moderate seismicity regions. After fixing the damping ratio in each member, the stiffness can then be determined by satisfying various drift criteria as obtained through the conventional design process. And finally, the strength parameter can be found using conventional linear analysis techniques under the assumption that the members are reaching their yield surface.

Validation of Presumed Ductility Distribution

As discussed earlier in this paper, the presumed ductility distribution may not necessarily be valid, whereby the member strengths obtained from the internal forces in the last segment of the hypothetical ground motion in Figure 1 may not necessarily result in the ductility demand presumed previously. And in general, it will not. In the incremental substitution procedure, the evolution process in Equation 6 is forward, where the dynamic properties in segment i-1 are used to evaluate the responses in segment i. However, since the dynamic properties and responses in the last segment are assumed to be known in the direct ductility distribution method, the evolution process in Equation 6 can either go forward or backward.

Consider the case of backward evolution. When using the response in segment i to evaluate the dynamic properties in segment i-1, two possible scenarios may occur. Since there is a reduction in the ground excitation intensity when changing from segment i to segment i-1, usually there will also be a reduction in the response $\mathbf{x}$. This is the preferable situation in which the conditions in Equation 7 to Equation 10 can be satisfied. However, since there will also be changes in stiffness distribution, reduction in damping and vibration periods when changing from segment i to segment i-1, sometimes the response may also increase. This will violate the non-self-healing assumption in the incremental substitution procedure, where the ductility demands μ seem to decrease with the forward evolution. Note that the ductility demand in the incremental substitution procedure is also used as a measurement in the degree of damage in a member, which should not be decreasing with time.

For a valid ductility distribution, the second scenario should be avoided, or at least practically minimised. From experience, the second scenario usually occurs when the member is designed to have a low ductility demand together with a high demand to capacity ratio (DCR) on strength. Similar to the conventional definition, the demand to capacity ratio in this paper is defined as the quotient of the induced seismic internal forces from the undamaged structure over that from the modified structure with the presumed ductility distribution. This is also sensible when considering the case of forward evolution. For members with high demand to capacity ratios, yielding of the members usually occurs at the early segments, which as a result would tend to protect other members from developing higher internal forces. And usually, this sacrificial action tends to result in higher ductility demands on the sacrificial members. Thus to minimise the occurrence of the second scenario discussed above, members presumed to have lower ductility demand should be checked such that they also have a lower demand to capacity ratio. Mathematically, this may be written as;

$$\max(DCR_s) < \min(DCR_{s+1}) \tag{12}$$

where s is the sacrificial class which represents a group of members with equal ductility demand such that;

$$\mu_s < \mu_{s+1} \tag{13}$$

The requirement in Equation 12 should be checked, and further adjustment to the presumed ductility distribution should be made if found necessary. For members with ductility demand larger than 1, this can usually be achieved easily by lowering the presumed ductility of the members with lower sacrificial class, which results in increases in stiffness, and thus attracts more forces. However difficulties may then be found for members which are already expected to stay in their elastic range. In this situation, the design load of these members should be increased such that the demand to capacity ratio is not greater than the minimum value in the higher sacrificial class. Note that since the member strength is only artificially increased to meet the minimum requirement in Equation 12, yielding is still expected to occur. Thus it may be necessary to detail the members to allow for a higher ductility demand that is not greater than that for the next sacrificial class.

In practical designs, there may also be critical members that are required to be protected such that they must remain elastic. This may be achieved by further applying an over-strength factor to the design load. The exact value of the factor should make reference to guidance in the local standards. From experience, such factor may not be necessary to be greater than 1.5. Finally, the calculated design load of all members with the presumed ductility demand of greater than 1 should be checked against the design load from other load cases such that the members can actually yield in the seismic event.

Example, Justifications and Discussions

An example is provided below for demonstration of the design procedure, and for justification of the proposed method. It is a 103m-tall 2D rigid frame building with a transfer at 3/F and eccentric bracings up to 19/F. Soft storeys are purposely created at the top and bottom of the tower to test on the efficiency of the proposed design method. For simplicity, flexural plastic hinges with simple elastic perfectly plastic hysteresis are assigned to the ends of all members to simulate the nonlinearity of the structure, and only the horizontal load under earthquake is considered in this study. The elastic mode shapes and periods of the first 3 modes are shown in Figure 3 to demonstrate the dynamic properties of the structure before any cracking or damage. 7 pairs of ground motion time history records are selected and scaled using the web application in the NGA ground motion database (PEER, 2013) to fit a type 1 elastic design spectrum specified in EC 8 (CEN, 2004). Comparison with the mean response spectrum of the scaled ground motion time history at 5% damping is presented with the elastic design response spectrum in Figure 4. Note that the mean spectrum from the scaled time history is in general stronger than the elastic design spectrum at smaller periods. The difference is particularly large at about 0.4s, which by coincidence matches the third mode of the structure.

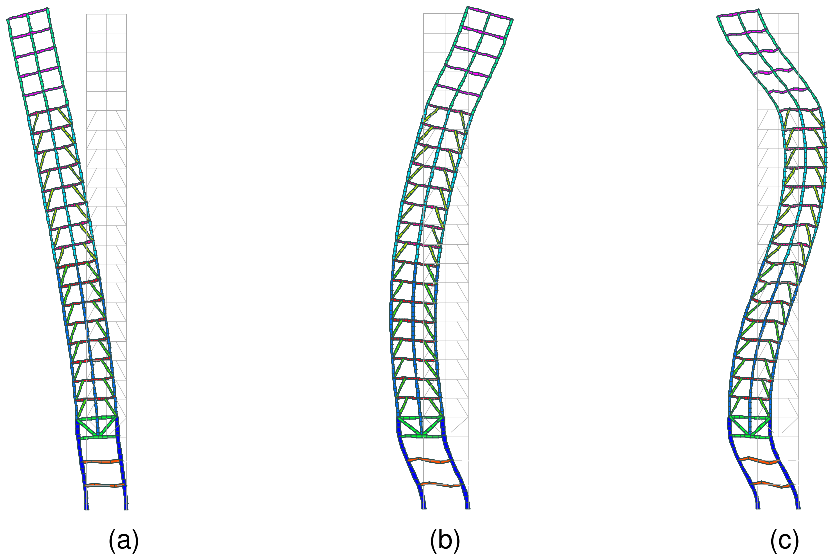


Figure 3. Mode shapes and periods;
(a) Mode 1 T=2.38s; (b) Mode 2 T=0.73s; (c) Mode 3 T=0.37s

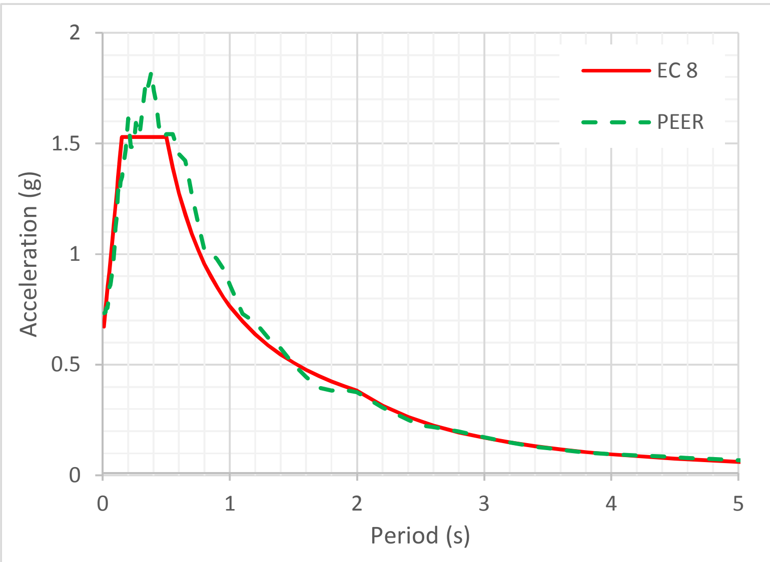


Figure 4. Comparison on response spectrum

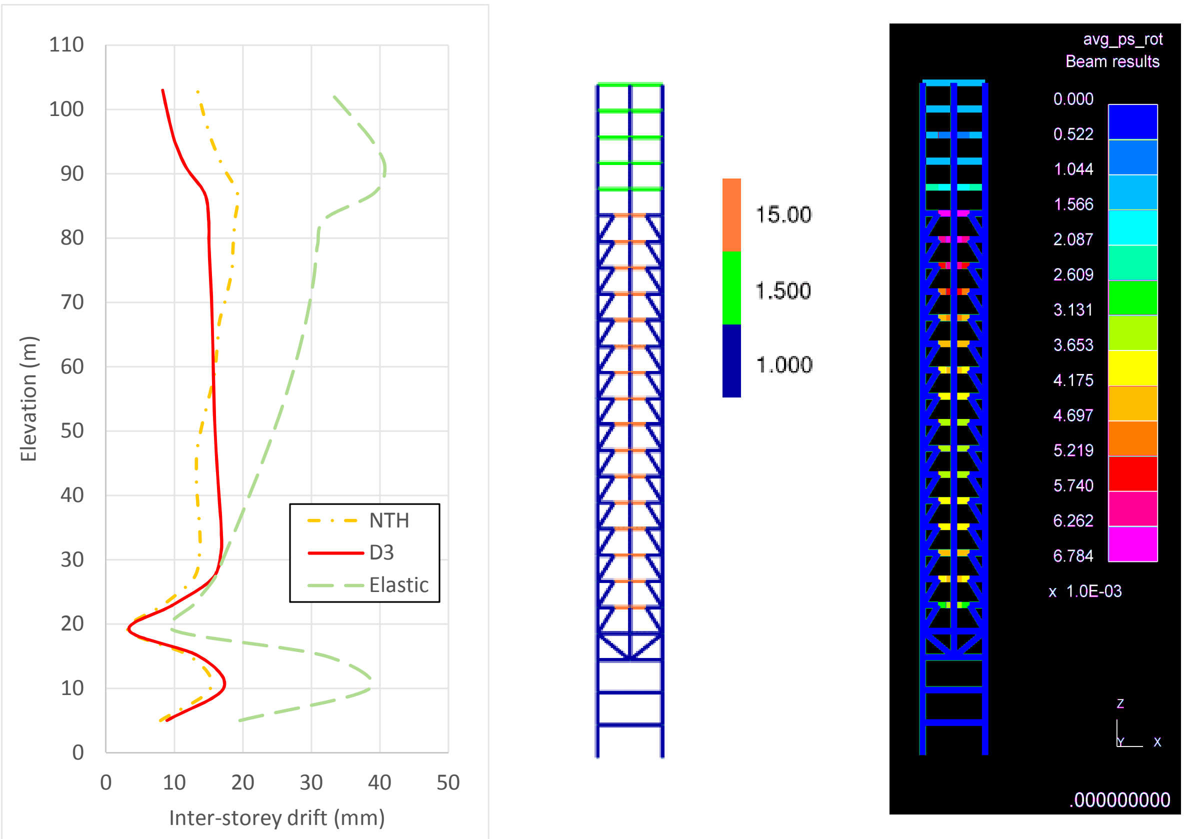
The presumed ductility distribution is shown in Table 1. Going from step (a) to step (d), table 1 also demonstrates the adjustment process of the strength demand to capacity ratio. It can be seen from the table that the soft storey at both the top and bottom of the tower are specially strengthened by presuming a lower ductility demand. However, even though the presumed ductility demand for the top 4 floors is reduced to 1, the requirement in Equation

12 is still not satisfied, and the design has to be artificially corrected by an increase of design load at those members.

Table 1. Demonstration of direct ductility distribution method

<table>
<tr><th rowspan="2">Members</th><th rowspan="2">Sacrificial Class</th><th>(a)</th><th>(b)</th><th>(c)</th><th>(d)</th></tr>
<tr><th>Presumed μ</th><th>Resulted DCR</th><th>Adjusted DCR</th><th>Required μ</th></tr>
<tr><td>All columns / bracings / transfer structures</td><td>S0</td><td rowspan="3">1</td><td>1.04 – 14.9</td><td>0.70 – 1.44</td><td rowspan="2">1</td></tr>
<tr><td>Beams on 1/F to 2/F</td><td>S1a</td><td colspan="2">2.16 – 2.20</td></tr>
<tr><td>Beams on 21/F to Roof</td><td>S1b</td><td>3.58 – 4.57</td><td>3.28</td><td rowspan="2">1.5</td></tr>
<tr><td>Beams on 20/F</td><td>S2</td><td>1.5</td><td colspan="2">3.28</td></tr>
<tr><td>Beams on 4/F to 19/F</td><td>S3</td><td>15</td><td colspan="2">3.40 – 5.73</td><td>15</td></tr>
</table>

Based on the design in Table 1, the structure is analysed and compared using the conventional elastic response spectrum analysis method on the undamaged model, the response spectrum analysis method on the damaged model with the presumed ductility distribution designed by the proposed method, and the mean response from the 14 sets of ground motion records simulated by nonlinear time history method. Figure 5a shows the comparison of the inter-storey drifts predicted by the 3 methods.



(a) (b) (c)

Figure 5. Comparisons on analysis results; (a) Inter-storey drift; (b) Required ductility demand by direct ductility distribution method; (c) Average plastic hinge rotations by nonlinear time history method

When compared with the results by conventional response spectrum analysis method, significant improvement in the accuracy of the predicted drifts can be obtained by introducing the presumed ductility distribution. And, as designed, the soft storey at both the top and bottom of the tower do not result any excessive response. In general, the response spectrum analysis with the presumed ductility distribution very reasonably approximates the response predicted by the nonlinear time history method. Some slight underestimation of the inter-storey drifts can be observed for the soft storey at the top. This resulted from the fact that the direct ductility distribution method can not explicitly consider the response evolution process in the incremental substitution procedure, as it is just an approximation of the nonlinear time history method. The effects is more observable with the very uneven distribution of the strength demand to capacity ratio within members under the sacrificial class S1a and S1b in Table 1. Finally, the deviation between the mean response spectrum from the ground motion records and the elastic design spectrum further overestimates the higher mode response in the nonlinear time history simulations.

Similarly, the distribution of the required ductility demands by the direct ductility distribution method well matches with the average plastic hinge rotations by nonlinear time history method as shown in Figure 5b and Figure 5c. As expected, some hotspots exist in the analysis results generated by the nonlinear time history method, but the overall distribution is in general in line with the design. Within the 14 sets of results from nonlinear time history method, none of the models shows any plastic deformation in the protected members under sacrificial class S0 in Table 1. This has further proven the effectiveness of the proposed design method in controlling the failure mechanism of a building under earthquake attacks.

Conclusion

As more tall and irregular buildings are emerging in many low to moderate seismicity regions, there is an increasing need in the industry for the development of a simplified design approach for practicing engineers to master the nonlinear dynamic behaviour under earthquake attacks. Focusing for the design of new buildings, the direct ductility distribution (D3) method is proposed in this paper as a suitable method.

The proposed method involves presuming a ductility distribution, a response spectrum analysis, verification of the presumed ductility distribution, and finally, adjustment of the seismic load and ductility demand if necessary. The simplicity of the proposed approach and the availability of the required skills in the industry is encouraging for further codification.

In the example building in this paper, the proposed design method is proven to be very effective and efficient in controlling both the drifts and the failure mechanism. By applying the direct ductility distribution method, the design engineers have gained the ability to keep failure away from the soft storey. Although the proposed method requires the primary gravity vertical load paths to be protected from plastic deformation, this is usually acceptable for tall building applications in low to moderate seismicity regions.

REFERENCES

ASCE (2013) *Minimum Design Loads for Buildings and Other Structures*, American Standard ASCE/SEI 7-10, American Society of Civil Engineers.

ATC (1996) *Seismic Evaluation and Retrofit of Concrete Buildings*, ATC 40, Applied Technology Council, Redwood City, California.

BD (2014) *Consultation on Introducing of Seismic-resistant Building Design Standards in Hong Kong*, Development Bureau, Buildings Department, The Government of the HKSAR.

Calvi GM (1999) *A Displacement-based Approach for Vulnerability Evaluation of Classes of Buildings*, Journal of Earthquake Engineering, 3(3): 411-438.

CEN (2004) *Design of Structures for Earthquake Resistance – Part 1 General Rules, Seismic Actions and Rules for Buildings*, Eurocode 8, EN 1998-1, Comité Européen de Normalisation, Brussels.

EEFIT (1991) *The Newcastle, Australia Earthquake*, The Earthquake Engineering Field Investigation Team, Institution of Structural Engineers, UK.

COLA (2013) *Voluntary - Earthquake Hazard Reduction in Existing Reinforced Concrete Buildings and Concrete Frame Buildings with Masonry Infills*, Division 95, Official City of Los Angeles Municipal Code, American Legal Publishing Corp.
Free M, Pappin J, Koo R (2004) *Seismic Hazard Assessment in a Moderate Seismicity Region, Hong Kong*, Proceedings of 13th World Conference on Earthquake Engineering, paper 1659.
Gulkan P and Sozen MA (1974) *Inelastic Response of Reinforced Concrete Structures to Earthquake Motions*, Journal of the American Concrete Institute, 71(12): 601-609.
Ho YB (2006) *Enhancing the Ductility of Non-seismically Designed Reinforced Concrete Shear Walls*, MPhil. Thesis, The Hong Kong University of Science and Technology.
Ho YB and Kuang JS (2012) *Damage Evolution Spectra for Seismic Analysis of High-rise Buildings*, Journal of Earthquake and Tsunami, 6(3).
Ho YB and Kuang JS (2014) *Seismic Vulnerability Assessment of Tall Buildings: Spectrum-based Nonlinear Dynamic Analysis*, International Conference on Advances in Civil and Structural Engineering - CSE 2014, Kuala Lumpur, Malaysia.
Ho G (2014) *Design of Tall Buildings in Low-to-moderate Seismicity with Strong Wind*, Proceedings of Advances in Seismic Engineering, HKIE / IStructE Joint Structural Division Annual Seminar, May, Hong Kong.
Iwan WD and Gates NC (1979) *The Effective Period and Damping of a Class of Hysteretic Structures*, Earthquake Engineering and Structural Dynamic, 7: 199-211.
Kuang JS (1998) *Why Worry About Earthquake in Regions of Moderate Seismicity: A Case-study of Hong Kong*, Journal of the Earthquake Engineering Society of Korea, 2(3): 111-119.
Kuang JS, Tso WK and Tang WH (1998) *Comparison of Wind and Seismic Effects on Hong Kong*, Proceeding of International Workshop on Earthquake Engineering for Regions of Moderate Seismicity, Hong Kong, 121-125.
Li CS (2005) *Response of Transfer Plate when Subjected to Earthquake*, Ph.D. Thesis, The Hong Kong Polytechnic University.
Miranda E and Ruiz GJ (2002) *Evaluation of the Approximate Methods to Estimate Maximum Inelastic Displacement Demands*, Earthquake Engineering and Structural Dynamics, 31: 539-560.
MOHURD (2010) *Code of Seismic Design of Buildings*, GB 50011-2010, Ministry of Housing and Urban-Rural Development, Government of the People's Republic of China.
Pappin J, Koo RCH, Free MW, Tsang HH (2008) *Seismic Hazard of Hong Kong*, EJSE Special Issue: International Earthquake Engineering in the Low and Moderate Seismic Regions of Southeast Asia and Australia.
Park R (1998) *Design Procedures for Achieving Ductile Behaviour of Reinforced Concrete Buildings*, Proceedings of the International Workshop on Earthquake Engineering for Regions of Moderate Seismicity, Hong Kong.
PEER (2013), *NGA-West 2 Ground Motion Database*, Pacific Earthquake Engineering Research Center, University of California.
Priestley MJN (1993) *Myths and Fallacies in Earthquake Engineering – Conflicts Between Design and Reality*, Bulletin of the New Zealand National Society for Earthquake Engineering, 26(3): 329-341.
Priestley MJN (2003) *Myths and Fallacies in Earthquake Engineering, Revisited*, Proceedings of the Mallet Milne Lecture, Pavia, Italy.
Pun WK and Ambrasey NN (1992) *Earthquake Data Review and Seismic Hazard Analysis for Hong Kong Region*, Earthquake Engineering and Structural Dynamic, 21: 433-443.
Scott DM, Pappin JW, Kwok MKY (1994) *Seismic Design of Buildings in Hong Kong*, HKIE Transactions, The Hong Kong Institution of Engineers, 1(2): 37-50.
Shibata A and Sozen MA (1976) *Substitute-structure Method for Seismic Design in RC*, Journal of Structural Division, ASCE, 102(1): 1-18.
Su RKL, Looi DTW, Tang TO, Law CW (2014) *Performance Based Seismic Design for Tall Buildings in Hong Kong*, Proceedings of Advances in Seismic Engineering, HKIE / IStructE Joint Structural Division Annual Seminar, May, Hong Kong.
Vamvatsikos D and Cornell CA (2002) *The Incremental Dynamic Analysis and its Application to Performance-based Earthquake Engineering*, Proceedings of the 12th European Conference on Earthquake Engineering, paper reference 479, London, UK.