13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1888

APPLICATION OF PUSHOVER ANALYSIS TO THE DESIGN OF STRUCTURES CONTAINING DISSIPATIVE ELEMENTS

Martin S. WILLIAMS[1] and Denis E. CLÉMENT[2]

SUMMARY

Five and ten-storey steel frames incorporating dissipative knee elements are analysed using a variety of non-linear static pushover analysis procedures, in accordance with Eurocode 8, FEMA356 and ATC40. The non-linear analyses make use of a novel knee element model capable of accurately simulating the bending and shear behaviour observed in full-scale tests. The performance of the structures and of the pushover analysis methods is assessed using non-linear time-history analysis. It is shown that the knee elements can be designed to yield under small earthquakes or early in a strong one (maximising their energy dissipation) while still being able to withstand a large event without collapse. Knee elements thus have the potential to give excellent seismic performance in steel framed structures. Comparison with the time history analysis results suggests that the FEMA356 procedure, which includes a more accurate representation of the structure's significant post-yield stiffness, gives the closest approximation to the dynamic response.

INTRODUCTION

There is currently strong interest in the use of dissipative design, in which seismic energy dissipation is concentrated in discrete, sacrificial elements. One such system is knee-bracing, a concept originally proposed by Aristazabal-Ochoa [1] and subsequently developed by Bourahla [2] and Balendra et al. [3-5]. In a knee braced frame (KBF) the main cross-braces are connected to short knee elements which span diagonally across the beam-column joints, Figure 1. The knee elements are designed to remain elastic during small earthquakes. During a moderate event, all energy dissipation takes place within the knee elements, protecting the main frame from any damage. Afterwards, any damaged knee elements can be replaced, returning the structure to its original state. In large earthquakes, while some damage to the main structural members may be unavoidable, the early energy dissipation provided by the knee elements should significantly reduce the loads on the main frame.

While previous research has demonstrated the feasibility of the knee bracing concept, further work is needed (i) to finalise the detailed design of the knee element, and (ii) to produce a straightforward design procedure for knee-braced frames. Recent work at Oxford has addressed both of these issues. Williams et

[1] Lecturer, University of Oxford, Dept of Engineering Science, Oxford, UK. martin.williams@eng.ox.ac.uk
[2] Structural Engineer, Thomas Jundt Civil Engineers, Geneva, Switzerland.

al. [6] developed a knee element design based on a standard column section strengthened by web stiffeners, and showed that optimal energy dissipation can be achieved by proportioning the section so as to yield in shear rather than flexure. This paper addresses the analysis and design of knee-braced frames, focussing on the applicability of the new generation of displacement-based design procedures to dissipative structures.

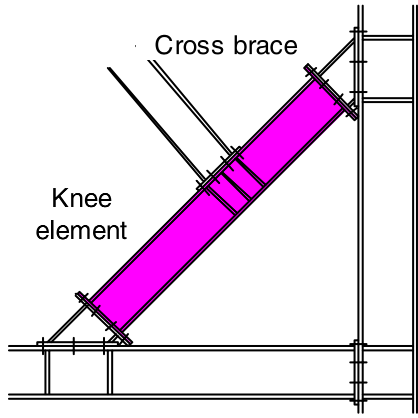


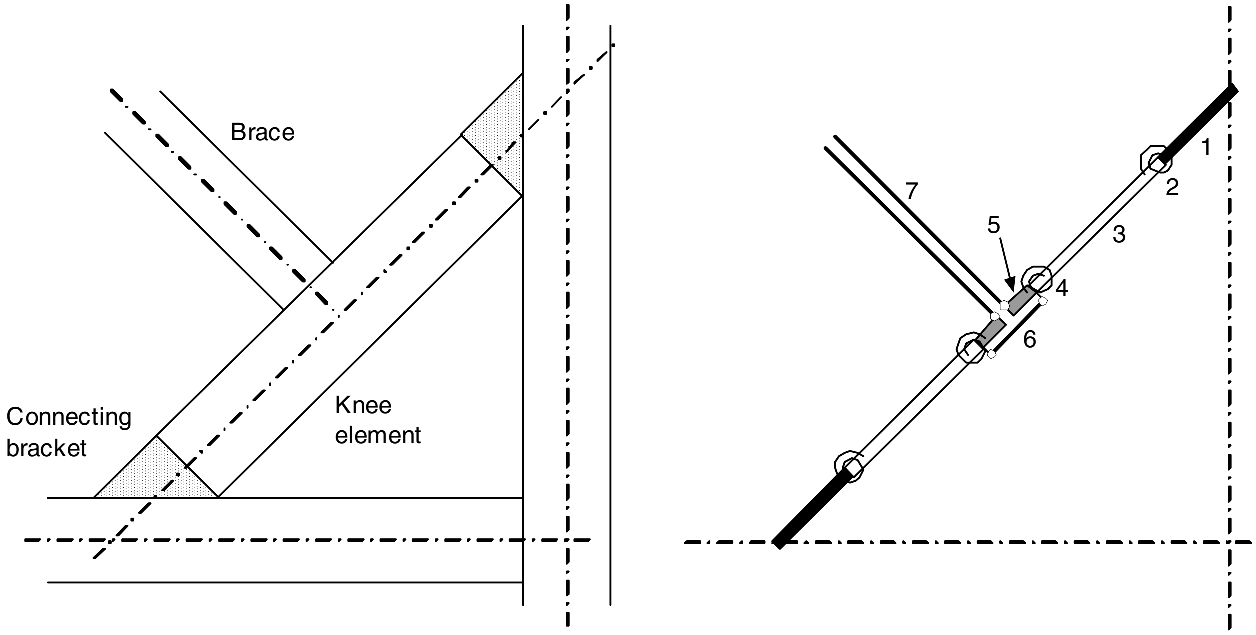
Figure 1. Typical knee element configuration

First, a knee element model suitable for use in a non-linear frame analysis program was created and calibrated against the results of physical tests and finite element analyses. Five and ten-storey knee braced frames were then designed and analysed using a variety of non-linear pushover analysis approaches. Finally, using non-linear dynamic time-history analysis, the performance of the knee braced frames and of the pushover methods was assessed.

KNEE ELEMENT MODEL FOR NON-LINEAR ANALYSIS

Non-linear analysis was performed using the program Drain-2DX [7]. Like most non-linear frame analysis programs, Drain-2DX does not include an explicit representation of yielding in shear. Since this is a fundamental aspect of knee element behaviour, an approximate method of modelling it had to be developed using the standard elements available. The knee element detail and its representation using Drain-2DX elements are shown schematically in Figure 2. In the following description, numbers in brackets are the element numbers shown in Figure 2.

(1) Rigid links simulate the offset of the knee element from the beam and column centrelines.
(2) The flexibility of connections is represented by linear rotational springs at the ends of the rigid links.
(3) The flexural response of the knee element is represented by conventional beam elements. These include the capacity for flexural hinge formation at the ends. However, since shear yielding usually dominates the behaviour these elements normally remain within their linear range.
(4) The knee element shear stiffness is represented by two bilinear rotational springs and…
(5) …two short cantilever beams. The cantilevers represent the initial shear stiffness and the first yield, while the bilinear springs allow a further stiffness reduction to be incorporated, as was observed in experimental tests [6].
(6) Axial yielding of the element is modelled by a separate bar element positioned in parallel with the two cantilevers.
(7) Lastly, because of the use of the two short cantilevers, it is necessary to split the cross-brace into two parallel elements and connect one to the tip of each cantilever.



KEY:

1 Rigid offset
2 Linear rotational spring representing connection stiffness
3 Beam element representing knee element flexural stiffness
4 Bilinear rotational spring
5 Short cantilever representing knee element shear stiffness
6 Bilinear truss element representing knee element axial stiffness
7 Truss element representing half of cross-brace

Figure 2. Drain-2DX knee element model

Properties of the various elements were chosen in a semi-empirical way, using the results from a limited number of full-scale cyclic load tests on knee elements. The procedure is fully described by Clément and Williams [8]. The general applicability of the model was then checked by comparing its predictions with the results of further experiments which had not been used in the initial parameter-fitting, and with detailed finite element analyses. A typical comparison between the model output and experimental data is shown in Figure 3, for a knee element based on a 152 mm × 152 mm × 30 kg/m universal column section.

Obviously a more analytically rigorous model could be developed. However, post-yield behaviour in shear is extremely difficult to model accurately and an empirical fit to experimental data is likely to be more reliable. The treatment of the axial load behaviour is also quite simplistic and the model could be improved by using a bending moment/axial load yield surface with inelastic deformation allowed in both the axial and rotational directions. Irrespective of these reservations, the model has been found to provide a robust and reasonably accurate representation of the knee elements' inelastic behaviour, showing good agreement with full-scale physical test data.

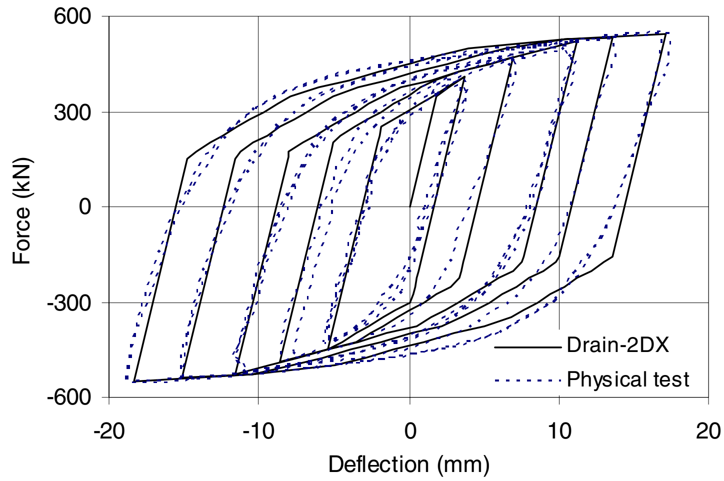


Figure 3. Comparison of response of Drain-2DX knee element model with cyclic test data

KNEE BRACED FRAME DESIGNS

Analyses of two knee-braced frames, of five and ten storeys, are presented here. The frames are shown in plan and elevation in Figure 4. Analyses presented later in this paper are for an assumed uni-directional earthquake ground motion parallel to the elevation shown.

The five-storey building was designed as a knee-braced frame using the EC8 [9] pushover analysis approach, for a peak ground acceleration of 0.35g, i.e. it was designed with the aim of limiting yielding under the design earthquake to the knee elements only. The resulting section sizes are summarised in Table 1. The fundamental period of the building was 0.57 s.

Table 1. Structural sections for the 5-storey knee braced frame

Level	Columns	Beams	Braces	Knee elements	KE yield load (kN)
5	UC 203×203×60	UB 254×146×31	SHS 120×120×8	UC 152×152×23	270
4	UC 203×203×86	UB 254×146×31	SHS 150×150×10	UC 152×152×37	430
3	UC 254×254×107	UB 254×146×31	SHS 150×150×12.5	UC 203×203×46	700
2	UC 254×254×167	UB 254×146×31	SHS 150×150×16	UC 203×203×60	930
1	UC 254×254×167	UB 254×146×43	SHS 150×150×16	UC 203×203×71	1100

Note: UK standard section designations:
UC: Universal column – x-dimension (mm) × y-dimension (mm) × mass (kg/m)
UB: Universal beam – designation as for UC
SHS: Square hollow section – x-dimension × y-dimension × wall thickness (mm)

The 10-storey frame was first designed as a ductile moment-resisting frame subjected to a PGA of 0.35g, then bracing and knee elements were added as retrofits (i.e. without adjusting the sizes of the main frame elements) with the aim of minimising damage to the main frame elements under the design earthquake. Full details of the design process are given by Williams and Albermani [10]. The section sizes are summarised in Table 2. The fundamental period of the building was 1.14 s.

Table 2. Structural sections for the 10-storey knee braced frame

Level	Columns	Beams	Brace area (mm^2)	Knee element yield load (kN)
8 – 10	310UC118	460UB67.1	2210	200
5 – 7	350WC197	530UB92.4	5520	500
1 – 4	350WC280	610UB101	6630	600

Note: Australian standard section designations:
UC: Universal column – x-dimension (mm) × mass (kg/m)
WC: Welded column – designation as for UC
UB: Universal beam – designation as for UC

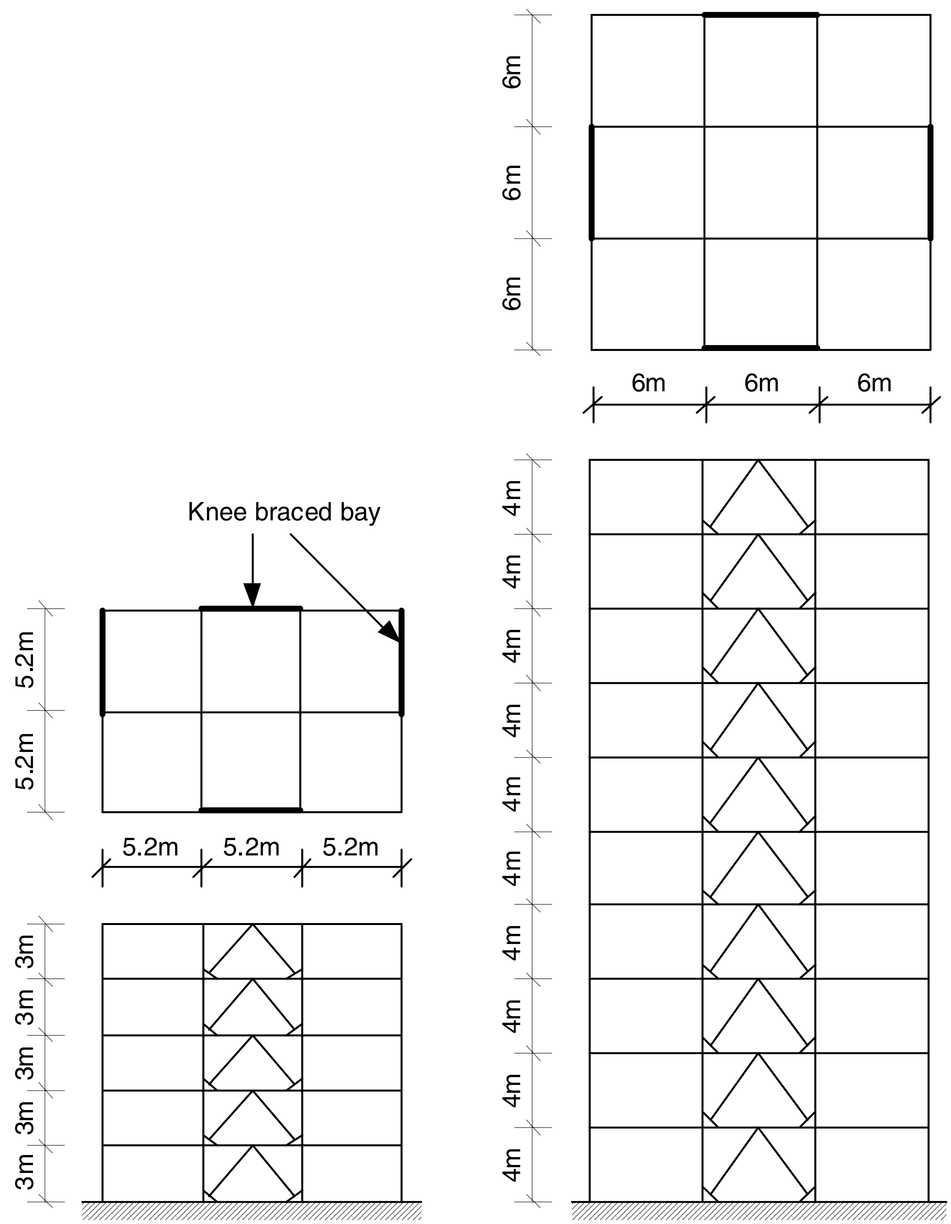


Figure 4. Plans and elevations of the two knee-braced frames

NON-LINEAR STATIC PUSHOVER ANALYSIS

The use of static pushover analysis in seismic design is growing rapidly in popularity. A non-linear static analysis is performed under a small number of pre-defined load patterns. The pushover curve is simplified to an idealised SDOF form and this is then used together with the design response spectrum to determine the peak displacement under the design earthquake – termed the *target displacement* or *performance point.* The non-linear static analysis is then revisited to determine the member forces and deformations at this point. The procedure has no rigorous theoretical basis but has been shown by Miranda [11] to provide a good approximation for buildings responding primarily in the fundamental mode. In this project three different pushover approaches were considered. The key differences between them are highlighted below.

Eurocode 8 [9] pushover method

Pushover analyses are performed under two lateral load patterns – a load distribution corresponding to the fundamental mode shape and a uniform distribution. The pushover curves are then idealised to an elastic-perfectly plastic equivalent SDOF system that dissipates the same energy for a control displacement as the multi-degree-of-freedom (MDOF) system. There is a unique formulation for such a SDOF system. For structures with significant post-yield stiffness this tends to lead to an underestimate of the initial, elastic stiffness.

FEMA 356 method [12]

FEMA 356 (a slightly revised version of the earlier FEMA 273) uses a very similar procedure to that of EC8. A slightly wider range of permissible load patterns is offered, including adaptive load patterns which allow for the redistribution of load as the structure undergoes non-uniform yielding. However, these are not conidered here.

The most significant difference is in the idealisation of the pushover curve. The elastic-perfectly plastic assumption in Eurocode 8 can result in an equivalent system whose characteristics are quite different to those of the real structure. FEMA 356 uses the more realistic alternative of a bilinear equivalent SDOF system with post-yield stiffness αK, where K is the elastic stiffness. However, unlike the elastic-perfectly plastic representation, this does not have a unique formulation and the parameters must be chosen with care. Judgement must be exercised in choosing the initial stiffness and post-yield stiffness. Although there is some margin for error here, this process tends to lead to a more realistic idealisation of the structural characteristics.

ATC 40 [13]

ATC 40 uses the capacity spectrum method, in which the energy dissipation due to yielding is converted to an equivalent viscous damping via an iterative procedure. This method idealises the non-linear structure as an elastic SDOF system with natural period calculated using the secant stiffness and viscous damping corresponding to the hysteretic damping, both taken at the target displacement.

Modal pushover

The modal pushover approach has been proposed by Chopra and Goel [14] as a modification to the ATC 356 approach. The aim is to allow for the change in load distribution due to yielding of the structure without resorting to the complexity of an adaptive load pattern. Pushover analyses are performed according to the FEMA 356 procedure, but using lateral load patterns based on the first three modes of vibration. The results for each modal load pattern are determined and then combined by the SRSS method to give an estimate of the overall response. This approach has been applied to the ten-storey building only.

DYNAMIC ANALYSIS

The results of the various pushover methods were compared with non-linear time-history analyses. An ensemble of 30 artificial accelerograms with 20 s duration, compatible with the Eurocode 8 design spectrum, was created using the program SIMQKE [15]. A trapezoidal intensity envelope with 10 s strong motion duration was used. Non-linear dynamic time-history analysis of the building under each earthquake record was carried out using Drain-2DX [7]. Most analyses assumed a peak ground acceleration of 0.35g (i.e. the design earthquake). However, for the 5-storey frame some analyses at other magnitudes were performed, to establish (i) the minimum PGA which would cause knee elements to yield and (ii) the collapse level earthquake. In most cases, time history results are quoted as mean ± one standard deviation for the ensemble of 30 analyses performed using the different accelerograms.

RESULTS

In this section the performance of the frames and the reliability of the various pushover analysis methods are assessed in terms of plastic hinge formation, target displacements, base shear and inter-storey drifts.

Plastic hinge formation

For the five-storey frame, all the analyses showed the design objective was achieved, i.e. no plastic hinges formed in the main structural elements under the design earthquake. Pushover analyses in accordance with EC8 suggested that yielding would take place in the beams if the peak ground acceleration was increased to 0.45g. However, time history analyses indicated that a rather larger PGA of 0.56g would be needed to cause beam yield.

For the ten-storey frame, which was designed as a retrofit of a ductile MRF, it did not prove possible to eliminate all plastic hinging in the main frame under the design earthquake. However, again the various pushover methods tended to give a rather more conservative picture of plastic hinge formation than was revealed by time history analysis. The FEMA356 and ATC40 results each indicated around 6 plastic hinges forming in the main frame under the design earthquake, the EC8 approach suggested between 16 and 23 hinges (depending on the load pattern) while the 30 time history results indicated a maximum of 3 hinges forming, with none at all in most analyses.

Target displacement

Table 3 shows the target displacements (i.e. roof displacements under the design earthquake) predicted by the various pushover and time history methods for both frames.

Table 3. Target displacements (mm) estimated by pushover methods compared to maximum displacements in time history analysis

Analysis method		5-storey frame	10-storey frame
EC8	Uniform load pattern	105	298
	Modal load pattern	121	360
FEMA356	Uniform load pattern	84	258
	Modal load pattern	90	305
ATC 40	Uniform load pattern	66	248
	Modal load pattern	72	284
Multi-modal pushover		–	310
Time history	Mean – st. dev.	74	185
	Mean	82	199
	Mean + st. dev.	90	212

For both structures the three main pushover methods give the similar relative results, i.e. the EC8 approach gives the highest displacements, followed by FEMA 356 and then ATC 40. The EC8 figures are thought to be unrealistically high because the structure still possesses significant stiffness even after most of the knee elements have yielded – in EC8 this is idealised to an elastic-perfectly plastic form, with the result that the initial, elastic stiffness is significantly underestimated.

The comparison with the time history results is more complex. For the five-storey frame there is very good agreement between the FEMA 356 approach and the time history analyses, with the ATC 40 approach underestimating the displacements. For the ten-storey frame all the pushover methods give significantly higher displacements than the time history analyses. These differences can be related back to those observed in terms of plastic hinge formation, where the pushover analyses substantially overestimated the degree of plasticity in the ten-storey structure.

Base shear

Table 4 shows the maximum base shear predicted by the various pushover and time history methods for both frames. In this case the differences between the various pushover methods are not particularly large, though again all give significantly more onerous results than were achieved by time history analysis.

Table 4. Maximum base shear (MN) for the 10-storey frame estimated by pushover methods and by time history analysis

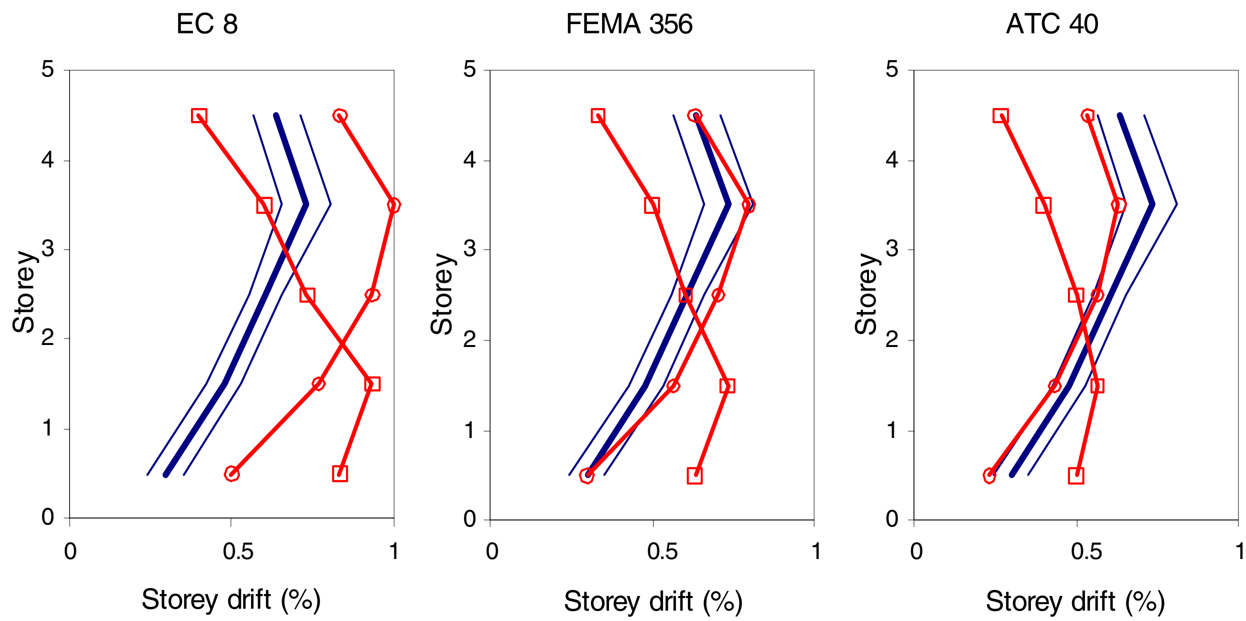
Analysis method		10-storey frame
EC8	Uniform load pattern	2.27
	Modal load pattern	1.90
FEMA356	Uniform load pattern	2.02
	Modal load pattern	1.67
ATC 40	Uniform load pattern	2.10
	Modal load pattern	1.79
Multi-modal pushover		1.92
Time history	Mean – st. dev.	1.24
	Mean	1.32
	Mean + st. dev.	1.41

Inter-storey drifts

Figures 5 and 6 show inter-storey drift comparisons for the five and ten-storey frames respectively. The general trends here are consistent with those for target displacement discussed earlier.

For the five-storey frame, the modal load pattern gives a distribution of drifts that is in accordance with the time history results, suggesting that the structure responds almost totally in its fundamental mode. The uniform load pattern gives rather high drifts near the base of the structure and low values near the top. In general the EC8 approach gives rather high drifts (serval standard deviations above the mean of the time history analyses), the FEMA 356 approach gives results that are in good agreement with the time history results, and the ATC 40 results appear non-conservative.

For the ten-storey frame, both load patterns tend to overestimate drifts near the bottom of the structure. At the top the modal load pattern comes quite close to the time history results while the uniform pattern understimates the deformations. In terms of the shape of the curves (as opposed to their magnitude) the best agreement is achieved by the modal pushover method, in which results from the first three modes are combined by the SRSS method. It thus appears that, as yielding occurs, higher mode effects become significant for this taller structure and a loading pattern based only on the fundamental mode becomes inadequate.



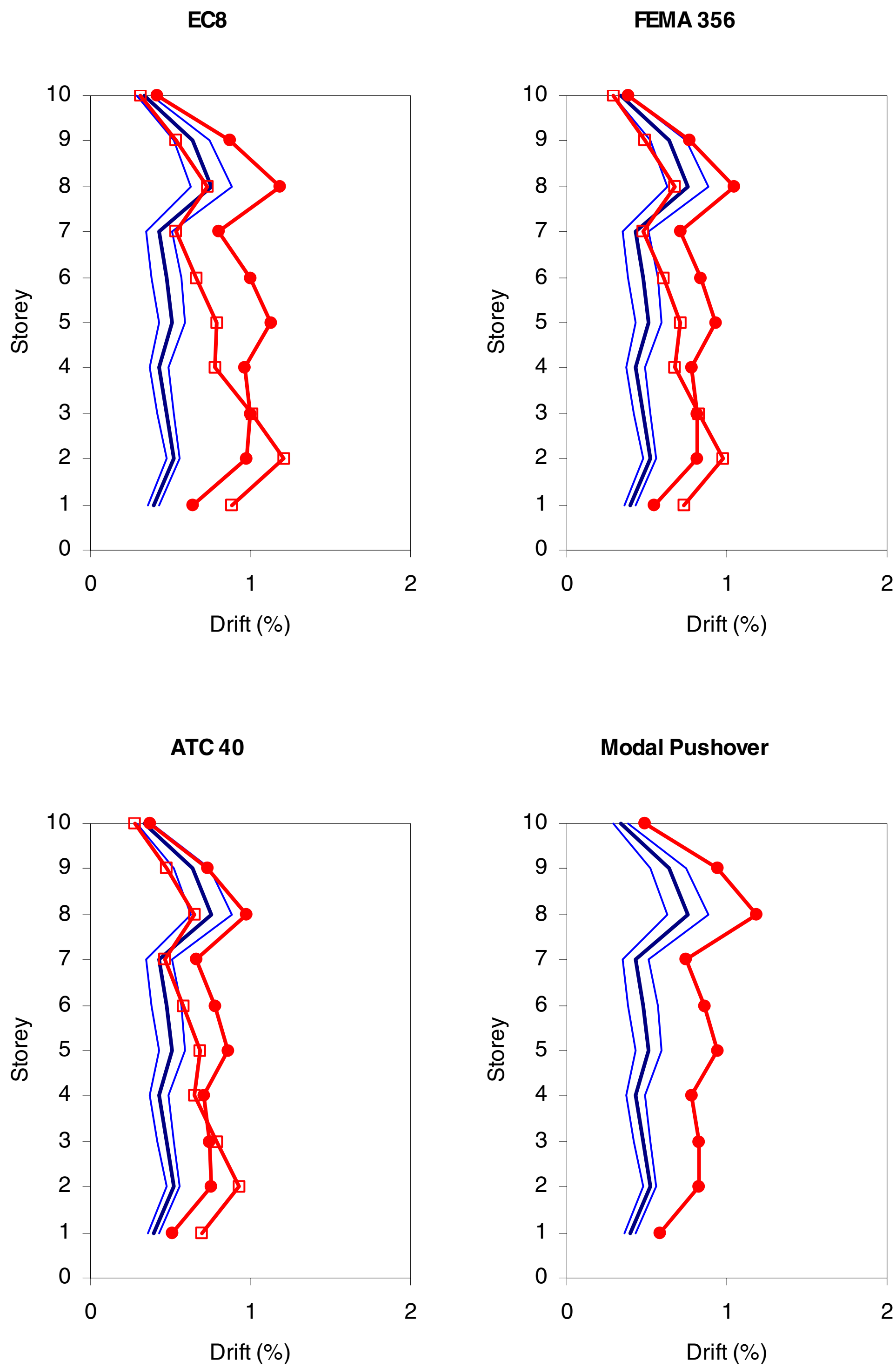
Key: *Blue lines: Time history mean and mean ± one standard deviation*
Red squares: pushover results using uniform load pattern
Red circles: pushover results using modal load pattern

Figure 5. Inter-storey drift comparisons for 5-storey knee braced frame

Other analyses

For a subset of the records, the earthquake intensities were varied to determine the lowest value of PGA at which inelastic deformation of knee elements would occur. It was consistently found that the kne elements yielded, and therefore began to affect the structural response, at ground accelerations of 7-8%g.

Lastly, for the five-storey frame, the time history analyses were repeated for higher earthquake intensities, so as to establish the levels at which drifts became excessive, and at which yielding of the main structure occurred. It was found that the drift requirements were satisfied up to PGAs of 0.54 g for all the artificial earthquake records. The main structure remained undamaged under all earthquakes up to a PGA of 0.56g, hinges forming in the third floor beams at this magnitude for some of the earthquake record.



Key: *Blue lines: Time history mean and mean ± one standard deviation*
Red squares: pushover results using uniform load pattern
Red circles: pushover results using modal load pattern

Figure 6. Inter-storey drift comparisons for 10-storey knee braced frame

CONCLUSIONS

A knee element model for non-linear static and dynamic analysis has been developed using standard element types available in Drain-2DX, and has been validated using results of full-scale physical testing and 3D finite element analysis.

Five and ten-storey steel frames incorporating knee elements have been designed for a ground peak acceleration of 0.35g, and their performance was assessed using non-linear dynamic time-history analysis.

The pushover analyses showed that the knee braced frames possess high global ductility (> 6) and substantial post-yield stiffness (~16%). Under time-history analysis, the five-storey building met the strength, stability and serviceability conditions when subjected to artificial earthquakes with PGAs up to 0.54g. The time history analyses also showed that the knee elements would start yielding, and the structure benefit from their damping effect, for an earthquake with a PGA of only 0.08g. This shows that the knee elements can be designed to yield for small earthquakes or early on in a strong one and still be able to withstand the largest event for a given site without collapse. It is concluded that a knee braced frame can give excellent seismic performance.

The pushover approach does not necessarily lead to the optimal design. Optimisation has not been undertaken in the present work and better results may be expected for an optimised structure. The use of an adaptive load pattern in the pushover analysis may help in this respect. For the taller building, the use of a multi-modal pusover analysis appeared to offer some improvements.

The time history analysis results have been compared with those obtained with three different pushover analyses (Eurocode 8, FEMA 356 and ATC 40). The FEMA 356 method gave the most reliable results when compared with time history analyses, while the Eurocode 8 method, which differs from FEMA 356 only in the bilinear idealisation of the equivalent SDOF system, gave rather conservative results. This will always be the case for structures such as knee braced frames, with significant post-yield stiffness. The results obtained here appear to support the conclusions of others that the ATC 40 method can give unconservative results.

ACKNOWLEDGEMENTS

Part of this work was performed while the first author was a visiting academic at the University of Queensland, Brisbane, Australia, supported by a grant from the Royal Academy of Engineering's Global Research Awards scheme.

REFERENCES

1. Aristazabal-Ochoa JD. "Disposable knee bracing: improvement in seismic design of steel frames." J. Struct. Engng, Amer. Soc. Civ. Engrs, 1986, 112: 1544-1553.
2. Bourahla N. "Knee bracing system for earthquake resisting steel frames." PhD thesis, Department of Civil Engineering, University of Bristol, 1990.
3. Balendra T, Sam M-T, Lee C-Y. "Diagonal brace with ductile knee anchor for aseismic steel frame." Earthquake Engng Struct. Dyn., 1990, 19: 847-858.
4. Balendra T, Liaw CY. "Earthquake-resistant steel frame with energy dissipating knee element." Engineering Structures, 1995, 17: 334-343.
5. Balendra T, Lim EL, Liaw CY. "Large-scale seismic testing of knee braced frame." J. Struct. Engng, Amer. Soc. Civ. Engrs, 1997, 123: 11-19.

6. Williams MS, Blakeborough A, Clément DE, Bourahla N. “Seismic behaviour of knee braced frames.” Proc. Instn Civ. Engrs: Structs & Bldgs, 2002, 152: 147-155.
7. Prakash V, Powell GH. “DRAIN-2DX Base Program Description and User Guide (Version 1.10).” University of California, Berkeley, 1993.
8. Clément DE, Williams MS. “Seismic design and analysis of a knee braced frame building.” J. Earthquake Engineering, 2004, in press.
9. EN 1998-1: 2003. Eurocode 8: Design of Structures for Earthquake Resistance. General rules, seismic actions and rules for buildings. British Standards Institution, London, 2002.
10. Williams MS, Albermani F. “Evaluation of displacement-based analysis and design methods for steel frames with passive energy dissipators.” Civil Engineering Research Bulletin No. 24, Department of Civil Engineering, University of Queensland, Brisbane, 2003.
11. Miranda E. “Approximate seismic lateral deformation demands in multi-storey buildings.” J. Struct. Engng, Amer. Soc. Civ. Engrs, 1999, 125: 417-425.
12. FEMA 356. “Seismic Rehabilitation of Buildings.” Federal Emergency Management Agency, Washington DC, 2000.
13. ATC 40. “Seismic Evaluation and Retrofit of Concrete Buildings.” Applied Technology Council, 1996.
14. Chopra AK, Goel RK. “A modal pushover analysis procedure for estimating seismic demands for buildings.” Earthquake Engng Struct. Dyn., 2002, 31: 561-582.
15. Gasparini DA, Vanmarcke EH. “SIMQKE: A Program for Artificial Motion Generation: User's Manual and Documentation.” Report no. 2, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge MA, 1976.