Steel Innovations Conference 2013

Christchurch, New Zealand

21-22 February 2013

BASE CONNECTIONS SEISMIC SUSTAINABILITY AND BASE FLEXIBILITY EFFECTS

J. Borzouie[1], G. W. Rodgers[2], G.A. MacRae[3], J.G. Chase[4], C. G. Clifton[5]
A.S. Moghadam[6]

ABSTRACT

This paper presents studies to assess the effect of elastic base flexibility on the first story drift and inelastic rotation demands at the column base. Dynamic analyses of simple structures with different periods and strengths by a suite of ground motions indicated that for short period structures, the story displacement increases with increase in base flexibility whereas for longer period structures, the displacement decreases with increase in base flexibility. These aspects are explained by means of a simple kinematic model. The paper also qualitatively assesses seismic sustainability of different types of base connections. The seismic sustainability is initially assessed in terms of four parameters: seismic performance, cost, constructability, and replacement. Low damage base connections and dissipation by friction and other external dissipators are the most promising.

1. Introduction

Observations from recent earthquakes show that the performance of the base connections has varied. For example, in the Northridge (1994) and the Kobe (1995) earthquakes many brittle failures were reported at the base level of the structure. In contrast, in the recent Canterbury earthquakes (September 2010, February 2011, and June 2011) there was no significant yielding or fracture damage observed at the base of the buildings, possibly because of the rotational flexibility at the column bases as a result of soft soil. However, some of these buildings were designed assuming rigid bases, where frames expected to yield at column bases as well as over the height of the building due to inelastic mechanism as a result of lateral loading. For this reason a study to assess and develop low damage base connections is being conducted at the University of Canterbury. In the first part of this research the effect of base flexibility on demands of the structure are evaluated. Although some research has been carried out in this area (Aviram et al. (2010), Maan and Osman (2002)), they have considered only a few structures, and those results could not be easily

[1]PhD Student, Dept. Civil and Natural Resources Engineering, University of Canterbury, New Zealand.
[2]Research Associate, Dept. Mechanical Engineering, University of Canterbury, New Zealand.
[3] Associate Professor, Dept. Civil and Natural Resources Engineering, University of Canterbury, New Zealand.
[4] Professor, Dept. Mechanical Engineering, University of Canterbury, New Zealand.
[5] Associate Professor, Dept. Civil and Environmental Engineering, University of Auckland, New Zealand.
[6]. Assistant Professor, International Institution of Earthquake Engineering and Seismology, Iran.

generalized. In the next step, experimental testing will be conducted to evaluate and develop low damage base connections. Some studies have been carried out to propose low damage base connections (Midorikawa et al. (2006);Ikenaga et al. (2006);MacRae et al. (2009), MacRae et al. (2010)), but more experimental studies are needed to be undertaken to compare these base connections and also develop improved base connections.

The first part of this paper presents an analytical study for the base flexibility effect on demands of the single storey structure. The second part, presents an initial study to evaluate seismic sustainability of various base connections for low damage construction. The seismic sustainability is initially assessed in terms of four parameters: seismic performance, cost, constructability, replacement. This paper aims to answer the following questions:

1. What is the effect of base flexibility on the displacement, base rotation and the moment demand at the top of the columns for single storey structures with a specified rotational flexibility at the top and base of the column and a column base strength level for different periods based on response history analysis?

2. Can simple relationships be developed to estimate the demands level in (1)?

3. Which of the available base connections are more appropriate from aspects such as cost, constructability, seismic performance and replacement for further study to be used as low damage base connections?

2. Base flexibility considerations

2.1 Analytical model

The modelled steel frame has three degrees of freedom with storey height of 3.5m. The top and base flexibilities are varied between fully fixed to fully pinned. Mass of the frame is varied based on period of the structure. The properties of the structure are shown in Figure 1.

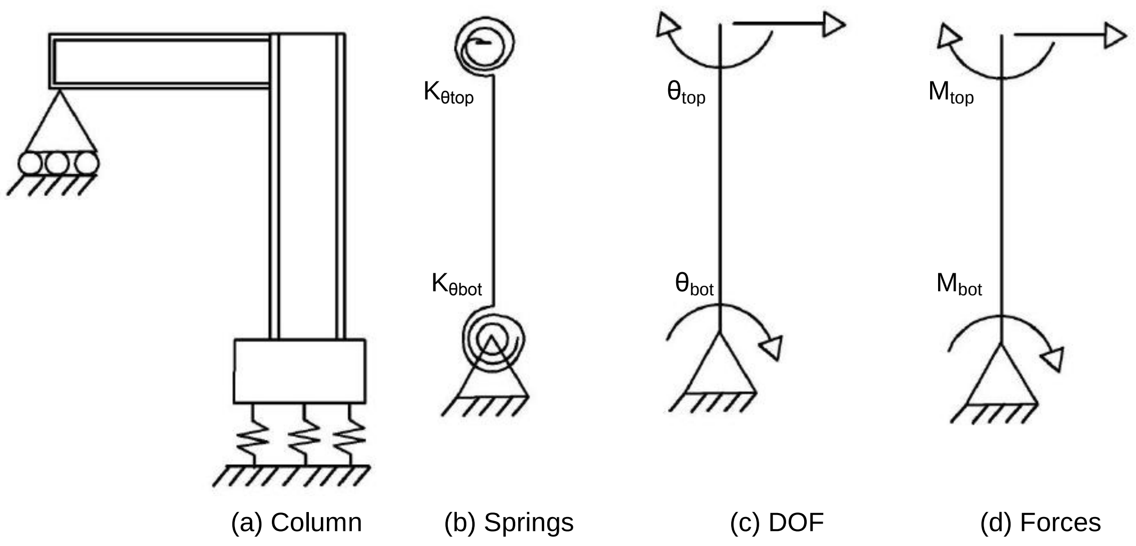
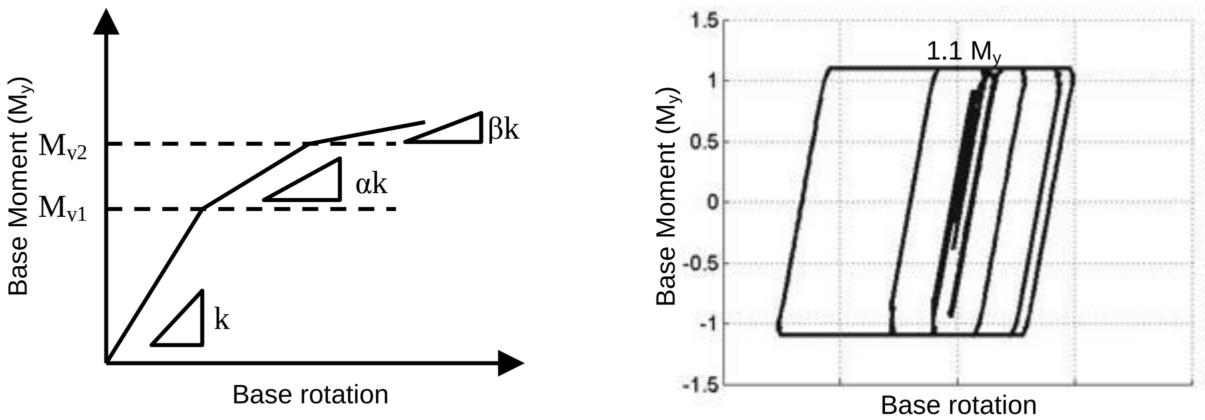


Figure 1: Simple model of the structure

The damping ratio is equal to 5% and the elastic modulus of the structure is 2.1×10^8 *kN/m²*, and the second moment inertia of the section assumes 3.5×10^{-4}m^4. Furthermore, the axial stiffness of the frame was assumed high value.

Top and base rotational stiffness are normalized to the *EI/H* of the column. Three different rotational stiffnesses were considered. These were corresponding to the fully pinned condition, 5*EI/H*, *2000EI/H* approximating the fully fixed condition. In order to evaluate nonlinear base rotation, nonlinear time history analysis has been conducted by assuming the Menegotto-Pinto hysteretic curve at the base of the structure

(Figure 2, Table 1). Figure 2 shows that the yielding response due to the loading in the first half cycle of the seismic loading. According to Figure 2a, the slope *αk* only exists during this loading cycle. For subsequent cycles, the stiffness decreases from *k* to *βk*. This may be seen in the cyclic response of Figure 2b.



(a) Idealized loading response for first half cycle (b) Cyclic response

Figure 2: Menegotto-Pinto hysteretic loop

Table 1: Properties of the Menegotto-Pinto hysteretic loop

Properties	Value
α	0.1
β	10
My2	1.1My1
ϒ (curvature of loop's corners)	10

Time history analysis was carried out using the twenty medium suite earthquake records (La 10 in 50) from the SAC steel project for Los Angeles with a probability of 10% in 50 years. The elastic spectral displacement for a wide range of periods is shown in Figure 3.

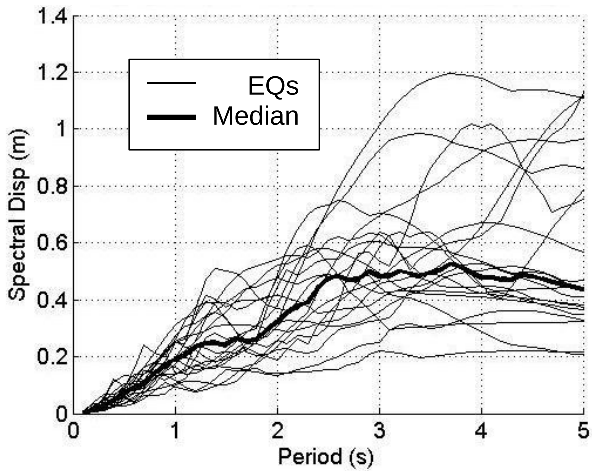


Figure 3: Median and record displacement response spectrum for SAC La 10 in 50 suite

2.2 Effects of base flexibility

2.2.1 Linear Analysis

2.2.1.1 Time history linear analysis

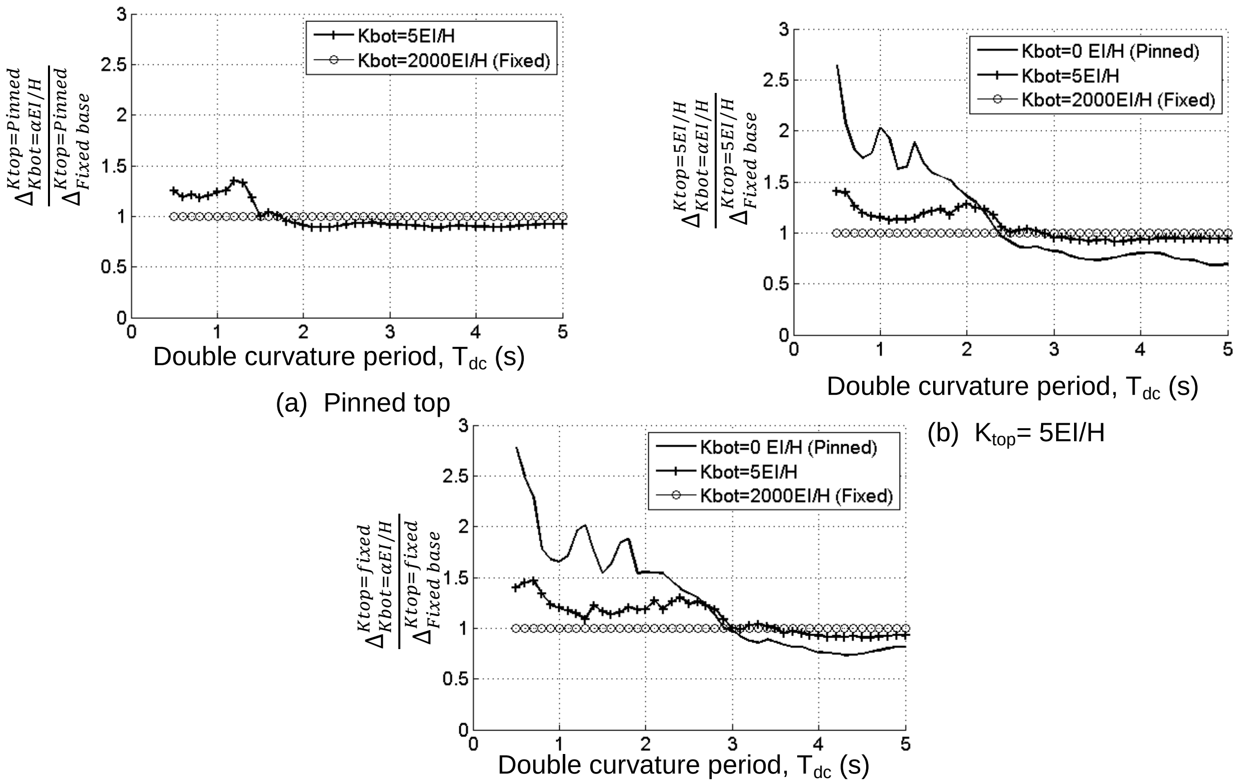
Time history linear analysis was carried out by MATLAB software. The periods of the structure given in Figure 4 were the double curvature period, T_{dc}, computed as a column lateral stiffness of $12EI/L^3$, assuming the column was fixed rotationally at the top and bottom and deflection is double curvature. The data points in Figure 4 were calculated based on the following steps:

1. The double curvature period was estimated.

2. The boundary conditions (top and base flexibilities) for the structure were applied. For example, when K_{top} is *5EI/H* and K_{bot} is *2000EI/H*, a top rotational spring provides this stiffness (*5EI/H*), for the fixed base structure.

3. Response history analysis (*RHA*) was conducted to determine derived response quantities. It should be noted that the fundamental period was greater than the given by T_{dc}. For the fixed top structure with the base flexibilities of *0.EI/H, 5EIH, 2000EI/H* the period was increased by *2.0*, *1.22* and *1.0* times relative to the T_{dc}.

Figure 4 shows the ratio of the lateral displacement with top and base flexibility ($\Delta^{Ktop=\beta EI/H}_{Kbot=\alpha EI/H}$) to the displacement of the fixed base structure with a given top flexibility ($\Delta^{Ktop=\beta EI/H}_{Kbot=Fixed}$). These results are presented based on the median response for the twenty earthquakes. The *P-Δ* effects were not considered in the preliminary analysis.



(a) Pinned top

(b) K_{top}= 5EI/H

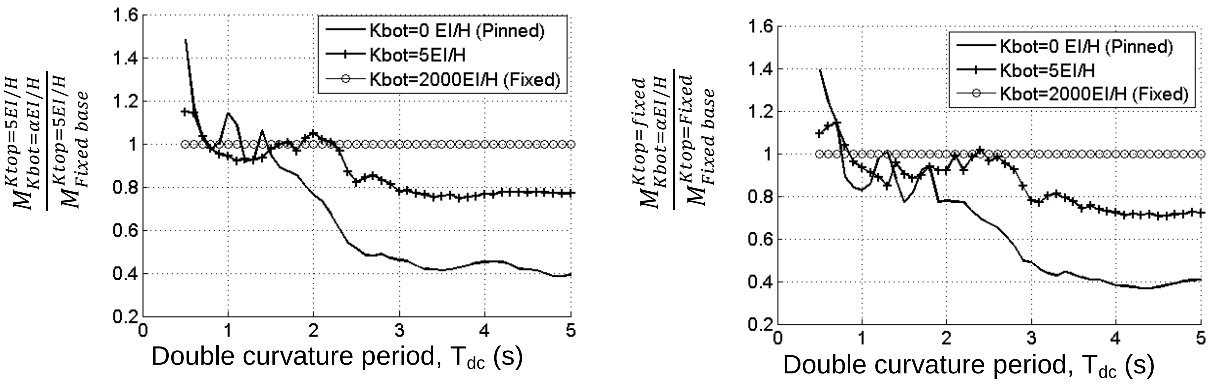
(c) Fixed top

Figure 4: Base flexibility effect on elastic top displacement of columns with top flexibilities of 0 EI/H (pinned),

5EI/H and 2000 EI/H (fixed)

It may be seen that for structures with $T_{dc} < 3s$, which is the majority of realistic structures, the lateral displacement increases due to the increase of rotational flexibility. This is consistent with Figure 3 which also indicates as average increase in displacement over this range and a smaller change in displacement for larger periods. So, designing of a structure with the short to medium period by assuming that the bases are fixed, underestimates the frame displacement.

Figure 5 shows that the moment demands at the top of the column considering base flexibility for top rotational spring flexibility of *5EI/H* ($M^{Ktop=5EI/H}_{Kbot=\,\alpha EI/H}$) and high rotational stiffness ($M^{Ktop=Fixed}_{Kbot=\,\alpha EI/H}$) respectively, are greater than those assigned the fixed base ($M^{Ktop=5EI/H}_{Kbot=\,Fixed}$), ($M^{Ktop=Fixed}_{Kbot=\,Fixed}$) respectively when the double curvature period, T_{dc}, is less than about *0.8s*. This indicates that as the top moment is increased for $T_{dc}<0.8s$, there is more likelihood of a soft-storey mechanism due to base flexibility. In contrast, for the structures with the double curvature period, T_{dc}, higher than 0.8s, the top moment demands of the column considering base flexibility for top rotational spring flexibility of *5EI/H* and high rotational stiffness respectively, are lower than those assigned the fixed base. So, assuming bases of a structure as the fixed bases, leads to conservative design for the top moment of this period range of the structure. Furthermore, the possibility of formation of the soft storey decreases due to the base flexibility in this range of period.



(a) K_{top}= 5EI/H (b) Fixed top

Figure 5: Base flexibility effect on elastic top moment of columns with top flexibilities of 5EI/H and 2000 EI/H (fixed)

2.2.1.2 Kinematic relation for estimation of demands of the structure with base flexibility

A simple relationship to estimate the change in response due to the base flexibility is as follow:

Step 1: The change in the period as a function of structural properties like base flexibility: The top rotation degree of freedom is eliminated by making the top fixed. The force- displacement relation is given in Eq. (1). In this equation *V* is lateral earthquake load, *M* is the external moment that is applied to the structure (equals to zero).

$$\begin{pmatrix} V \\ M=0 \end{pmatrix} = \begin{pmatrix} k_{11}=\dfrac{12\mathrm{EI}}{\mathrm{H}^3} & k_{12}=\dfrac{-6\mathrm{EI}}{\mathrm{H}^2} \\ k_{21}=\dfrac{-6\mathrm{EI}}{\mathrm{H}^2} & k_{22}=\dfrac{4\mathrm{EI}}{\mathrm{H}}+\mathrm{K}_\theta \end{pmatrix} \times \begin{pmatrix} \Delta \\ \theta \end{pmatrix} \quad (1)$$

So, the relation between the lateral force (V) and the displacement (Δ) can be calculated by the first row of the Eq. (1). This is given in Eqs. (2) and (3).

$$V = k_{11}\Delta + k_{12}\theta \Rightarrow V = \left[k_{11} - \frac{k_{12}k_{21}}{k_{22}}\right] \times \Delta \quad (2)$$

$$K_{lateral} = \left[12 - \left(\frac{36}{4 + \frac{H \times K_\theta}{EI}}\right)\right]\frac{EI}{H^3} \tag{3}$$

So, the modified period due to the base flexibility can be calculated by the modified stiffness of Eq. (3).

Step 2: The change in the response due to change in the period: The lateral displacement can be obtained from the elastic response spectrum of the given earthquake based on the modified period. Also, the relation between the displacement (*Δ*) and the base rotation (*θ*) can be obtained from solving the second row of the matrix in Eq. (1), that is calculated in Eqs. (4) and (5).

$$M = k_{21}\Delta + k_{22}\theta = 0 \Rightarrow \theta = -\frac{k_{21}}{k_{22}} \times \Delta \tag{4}$$

$$\theta = \frac{6}{H(4 + \frac{HK_\theta}{EI})} \times \Delta \tag{5}$$

For example, the above method is followed for the structure of Figure 6 in Kobe record from suite earthquake records (La 10 in 50) from the SAC steel project. The elastic displacement spectrum of this record is shown in Figure 7.

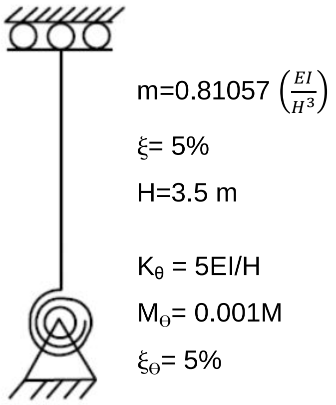


Figure 6: Sample structure

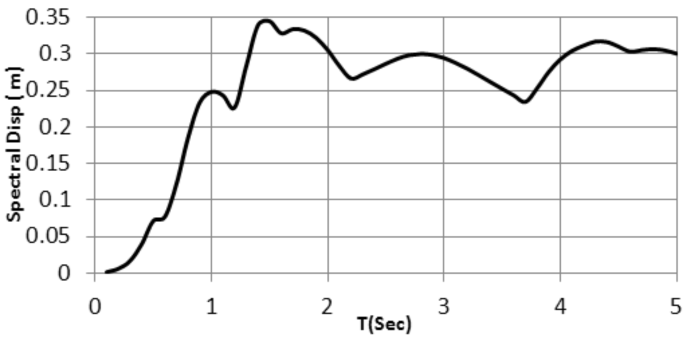
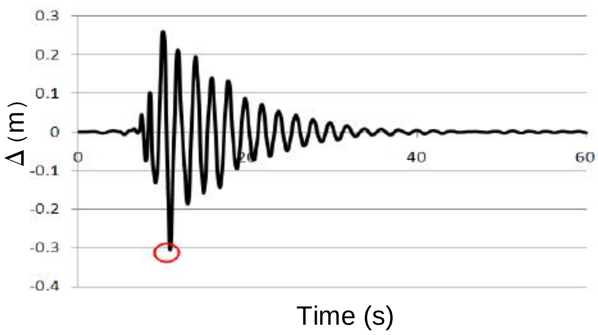


Figure 7: Elastic response spectrum of Kobe earthquake of the La 10 in 50 suite.

Step 1: For this structure the $K_{lateral}$ can be calculated from the Eq. (3). So, the period (T_1) is *2.0 s*.

Step 2: According to the elastic response spectrum of this record (Figure 7), the spectral displacement for the period of *2.0s* is *0.3m*. Also, the base rotation equals to *0.058* rad based on Eq. (5).
Figure 8 and Figure 9 are shown time history analysis of this structure with the boundary condition. So, the maximum displacement (*Δ*) and the base rotation (*θ*) that are obtained from time history analysis are consistent with the values that are computed with the kinematic method.



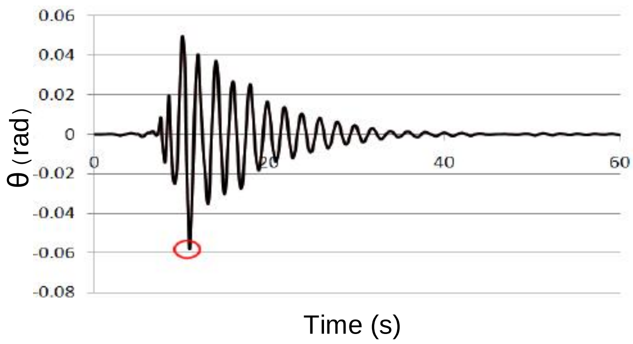


Figure 8: Time history displacement response of the structure in the Kobe earthquake

Figure 9: Time history base rotation response of the structure in the Kobe earthquake

2.2.1.3 Interpretation for demands

For a fixed-fixed structure with a period, T_f such as that shown in Figure 10a, the deformed shape and the bending moment diagrams are similar to those in
Figure 10b, and
Figure 10c respectively.

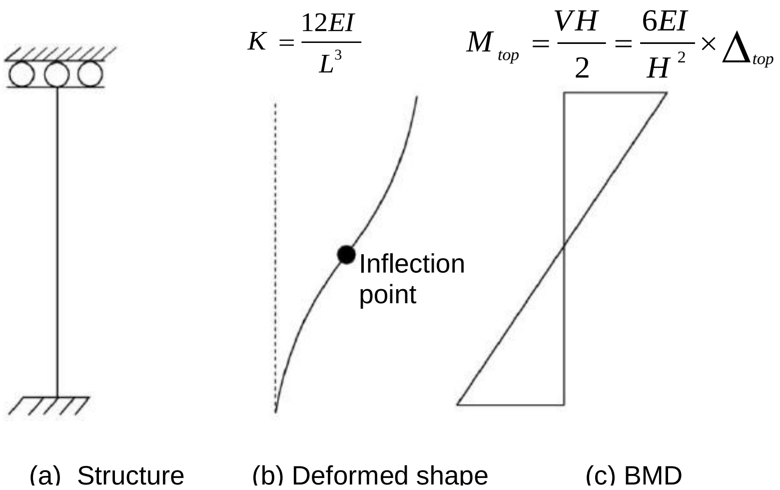


Figure 10: The fixed-fixed structure

If the base flexibility is such that the structure is fully pinned at the base, as shown in Figure 11, then the structure deformation and bending moment diagram become that shown in Figure 11b. The lateral stiffness is $3EI/L^3$, and the BMD is shown in Figure 11c. Since the lateral stiffness decreases four times, the structural period doubles.

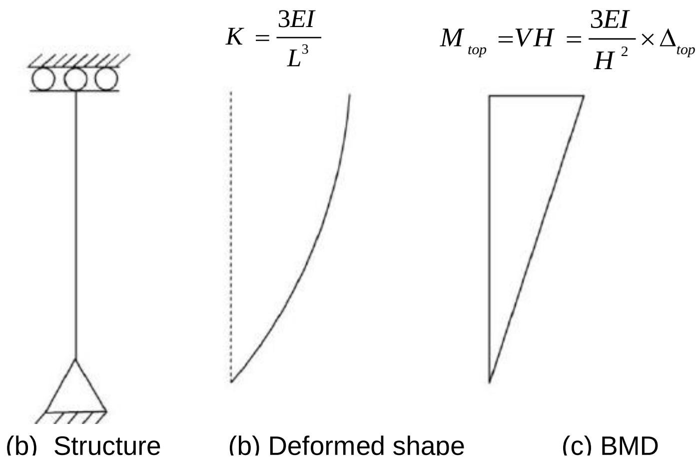


Figure 11: The pinned base-fixed top structure

According to the two above figures, the ratio of top moment of the pinned base to the fixed base structure that is fixed at top is given in Eq. (6).

$$M_{topratio} = \frac{M_{top,pinned\ base}}{M_{top,fixed\ base}} = \frac{1}{2} \times \frac{\Delta_{pinned\ base}}{\Delta_{fixed\ base}} \quad (6)$$

For example, for the structure with the period of 1.0s in Figure 4 (c) the top displacement of the pinned base structure ($\Delta_{pinned\ base}$) is 1.64 times that of the fixed base structure ($\Delta_{fixed\ base}$). According to the Eq. (6), $M_{top\ ratio}$ is 0.84. Moreover, Figure 5 (b) shows that for the period of 1s, this ratio is equal to 0.84 that is consistent with the kinematic method.

For linear increasing elastic spectral displacement diagram such as that used in many design codes for low periods, (less than approx. 1.3 seconds from Figure 7 or 0.7 seconds from the spectra in NZS 1170.5) the doubling of period corresponds to a doubling of displacement at the top of the column. This results in top moment increases due to the base flexibility and increasing the possibility of formation of the soft story. This is consistent with Figure 5 for structures with short periods. However, as it is obvious from Figure 3, linear increasing of spectral displacement only matches to the short period structures, and the spectral displacement does not change or decreases for medium and long period structures since the period increases. So, for medium to long period structures the top moment does not increase relative to the fixed base structure for increasing base flexibility, and the soft story mechanism is not possible to form except perhaps when the P-Δ effects are concluded.

2.2.2 Nonlinear analysis

For nonlinear structures, it is positive to estimate the change in inelastic rotation demand, θ, at the base of the structure as well as the response parameters such as the top displacement that affects non-structural elements, and the moment at the top of the column affects the possibility of soft storey mechanism. So, time history nonlinear analysis has been carried out by MATLAB for the structure that was shown in Figure 1. Top rotational stiffness is assumed fixed and base yielding moment is normalized to the maximum elastic base moment of the fixed base structure. Approximate inelastic demand relationship between displacement of the nonlinear structure ($\Delta_{inelastic}$) and displacement of the elastic structure ($\Delta_{elastic}$) is given in Eq. (7).

$\Delta_{inelastic} = \Delta_{elastic} \times$ Modification factor (Mf) (7)

This modification factor can be calculated as

$$Mf = \frac{\mu}{R} \quad (8)$$

Where ductility (μ) and reduction factor (R) can be obtained from the following equations:

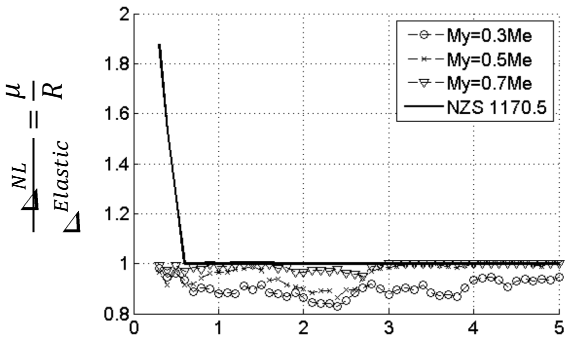
$$\mu = \frac{\Delta_{inelastic}}{\Delta_y} \quad (9)$$

$$R = \frac{F_{elastic}}{F_y} \quad (10)$$

The relation between ductility and reduction factor is mentioned in many documents such as NZS1170.5 (2004) where for stiff sites (type A,B,C and D):

$$\begin{cases} R = \mu & T_1 > 0.7\ s \\ R = \dfrac{(\mu - 1)T_1}{0.7} + 1 & T_1 < 0.7\ s \end{cases} \quad (11)$$

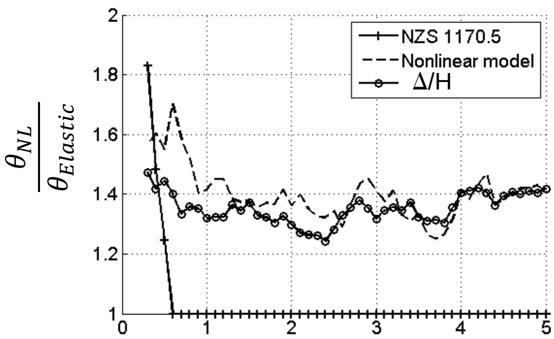
Figure 12 compares the total displacement of the nonlinear model to the displacement of elastic model for NZS method and nonlinear time history analysis. Each point represents the median results of the twenty earthquakes.



Double curvature period, T_{dc} (s)

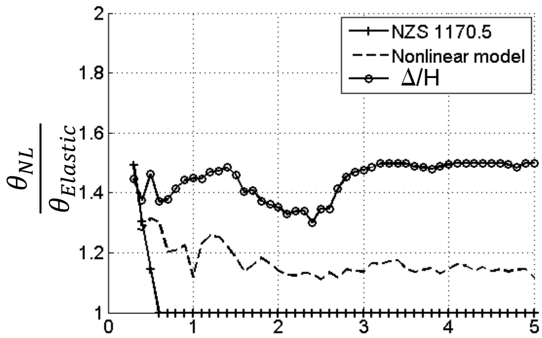
Figure 12: Total displacement of the nonlinear model to the displacement of the elastic model for the structure with $K_{\theta base} = 5EI/H\ and\ fixed\ at\ top$

This figure indicates that for different levels of the base yielding and periods of the structure, the code estimation is reasonable, except for very short period structures, that the code approach is conservative for short period structures. Total displacement results from lateral deflection of the column and the total elastic and nonlinear base rotation. So, another method that can be evaluated in parallel to the design guideline method for estimation of nonlinear base rotation is dividing of total displacement to the height of the frame. The accuracy of this method increases as the participation of the column flexural movement to the final displacement reduces. In Figure 13 the base rotation of time history nonlinear analysis is compared with methodology of NZS 1170.5 and also the second method which the total displacement is resulted from only the base rotation *(Δtop/h)* for three different yielding moment levels. Each point represents the median results of the twenty earthquakes.



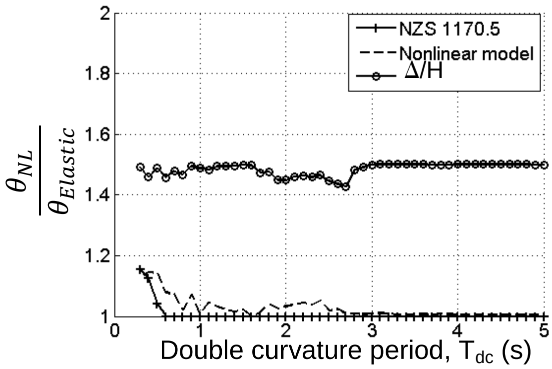
Double curvature period, T_{dc} (s)

My=0.3 Me



Double curvature period, T_{dc} (s)

My=0.5 Me



Double curvature period, T_{dc} (s)

My=0.7 Me

Figure 13: Total base rotation of the nonlinear model to the base rotation of the elastic model for the structure with $K_{\theta base} = \frac{5EI}{H}\ and\ fixed\ at\ top$

Figure 13 indicates that for the lower yielding moment (*M_y=0.3-0.5$M_{elastic}$*), due to the high nonlinear rotation at the base, the participation of lateral deflection of column in total displacement of the structure reduces and the total displacement that is divided to the height of the frame shows more exact estimation than the methodology in design guidelines. In contrast, as yielding moment increases ($M_y \geq 0.7\ M_{elastic}$), nonlinear base rotation reduces and it leads to more participation of lateral deflection of column in total displacement of the frame. For this range of base yielding, the prescribed methodology provides more precise estimation. The below step by step methodology is proposed to consider the effect of base nonlinearity to calculate demands due to base flexibility.

Step 1: Calculate the top displacement, base rotation and top moment based on the methodology for elastic section.

Step 2: Calculate ductility (μ) based on reduction factor (R), according to the relation in NZS 1170.5 (2004). (Eq. (11))

Step 3: Calculate the displacement from Eq. (7).

Step 4: The nonlinear base rotation is the elastic base rotation ($\theta_{elastic}$) multiplied by ductility (μ) for high values of the base yielding moment ($M_y > 0.7 M_{elastic}$). For the lower base yielding moment ($My < 0.7 M_{elastic}$), the base rotation equals to the total displacement from step 3, that is divided by height of the frame (H).

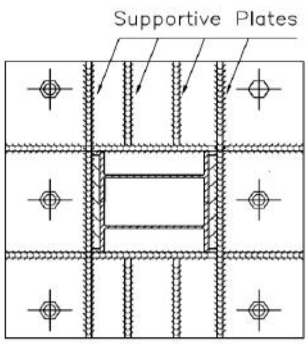
3. Base Connections Seismic Sustainability

The objective of the low damage systems is to decrease damage in all members of a building; so, if the base connections damage in earthquake, and the rest of the building does not sustain damage, this system is not low damage. This section presents the initial step of the study that is being conducted at the University of Canterbury to evaluate and develop low damage base connections. In this section, evaluation and comparison of different current details of base connections are presented and they are evaluated from seismic performance (energy dissipation and damageability), cost, constructability and replacement. Table 2 shows the grading pattern that has been used for qualitative evaluation of the base connections. All of these evaluations are carried out for each possible failure mode in each detail. In these evaluations, the columns are assumed to resist the elastic design forces without nonlinear action or buckling.

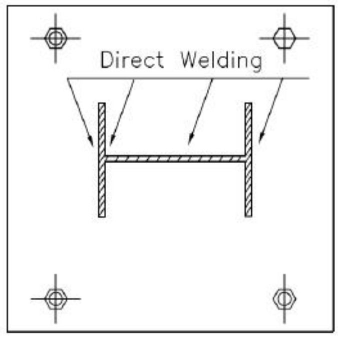
Table 2: Qualitative parameters for evaluation of the base connection detail

Items	Seismic Performance		Cost	Replaceable	Constructability	
	Damageability	Energy dissipation			Required expertise workers	Time of consuming
Min	0: damage mitigates to brittle failure mode	0: low energy dissipation	0: expensive	0: non replaceable	0: need to expertise workers	0: time consuming
Max	2: damage mitigates to ductile failure mode	2: High energy dissipation	2: Cheap	2: easy replaceable	1: easy installation	1: rapid installation
Range	[0-2]	[0-2]	[0-2]	[0-2]	[0-1]	[0-1]

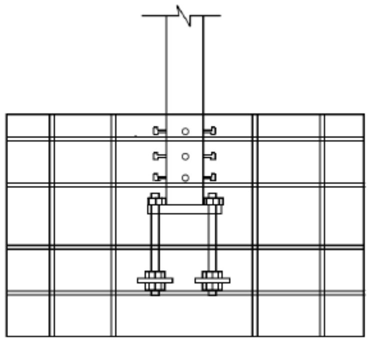
Figure 14 presents the base connection details that are evaluated in this section. Some of them are prevalent details in construction and others seem to have a potential for further investigations as a low damage base connection.



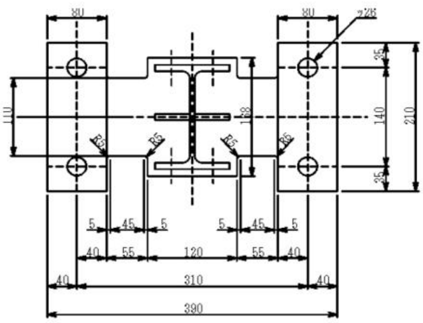
A. Use of plates to connect the column and the base plate



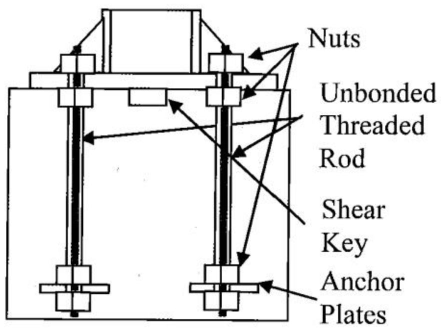
B. Direct welding of column- base plate



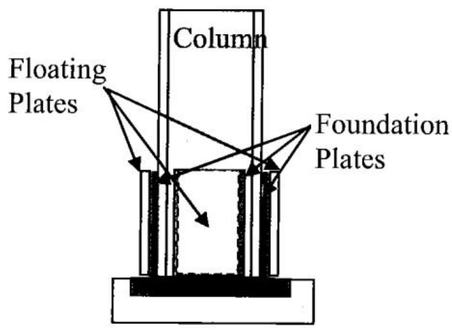
C. Embedded base plate connection detail



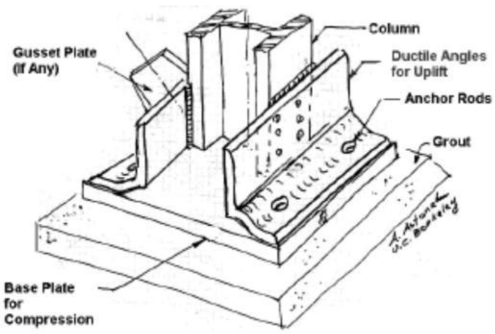
D. Base plate specifically design for yielding (Midorikawa et al. (2006))



E. Use of unbounded steel rods to act as re-centering devices (MacRae et al. (2009)



F. Using of sliding hinge joint (SHJ) in base connection (MacRae et al. (2009))



G. Welding of angles (Astaneh-Asl (2008))

Figure 14: candidate details for seismic sustainability

Simple qualitative evaluation of the above details based on Table 2 is presented in Table 3.

Table 3: Low damage assessment of fixed base connections for moment frame

(0 means unsuitable and 10 means very suitable)

Method	Applicable*	Deformation mode	Seismic Performance		Replaceable	Cost	Constructability		Total**
			Damageability	Energy dissipation			Expertise	Time	
			[0-2]	[0-2]	[0-2]	[0-2]	[0-1]	[0-1]	[0-10]
Welding of vertical plate	MRF, BF	Yielding of anchor bolts	2	1	0	2	1	1	7
		Pull out of anchor bolts	1	0	0				5
		Failure of concrete	1	0	1				6
		Fracture of column	0	0	0				4
Direct welding of column to beam	MRF, BF	Yielding of base plate	2	1	0	2	1	1	7
		Yielding of anchor bolts	2	1	0				7
		Pull out of anchor bolts	1	0	0				5
		Failure of concrete	1	0	1				6
		Fracture of column	0	0	0				4
Embedded BP	MRF	Failure of concrete	1	0	1	2	1	0	5
		Fracture of column	0	0	0				3
Yielding BP	MRF, BF, RF	Yielding of base plate	2	2	0	2	1	1	8
Unbounded steel Rods	MRF, BF, RF	Yielding of anchor bolts	2	2	0	2	1	1	8
Sliding Hinge Joint	MRF, BF, RF	Rocking	2	2	2	1	0	1	8
Welding of angles	MRF, BF, RF	Yielding of angles	2	1	2	2	1	1	9
		Pull out of anchor bolts	1	0	0				5
		Failure of concrete	1	0	1				6
		Yielding of anchor bolt	2	1	0				7

* MRF: Moment resisting frame, BF: Braced frame, RF: Rocking frame

** Total grade of sustainability

Table 3 states that due to low number of base plates in comparison with other structural member, cost and constructability do not play the main role in selection of a base plate detail. So, the seismic performance and replacement of the base connection are the main parameters to compare different base plate details. Furthermore, the performance of each detail highly depends on the deformation mode. For example, in the direct welding of a column to a base plate, total grading for column fracture deformation mode is *4* that is categorized on poor details. In contrast for the same detail with deformation mode of yielding base plate this index equals to 7 that belongs to the most appropriate details. From constructability aspect, the embedded base plate needs special consideration in the step of foundation construction. Because the base plate should be installed in inner layer of foundation and this makes some problems in construction.

The yielding base plate, unbounded steel rods, welding of angles, and sliding hinge joint are in accordance with the low damage construction concept. All of them could reduce considerable part of energy by special mechanism (yielding and friction), and the nonlinearity is mitigated to other parts than the columns. Furthermore, their main similarity is that nonlinearity happens in ductile modes, and this nonlinearity provides high energy dissipation for the building. So, these are the candidates from the above aspects for further studies to develop low damage base connections. It will be important in practice that the rotational stiffness of the proposed base detail is input into the structural analysis and then either the rotational stiffness of the final proposed column base connection is determined, the structure reanalysed and an iteration of design undertaken as required or else the proposed base detail is tuned to generate the rotational stiffness used in the structural analysis. The latter option is preferred as it does not require iterations in design,

4. Conclusion

This paper describes the base flexibility effect on demands of one story steel structure and either presents the qualitatively evaluation of different base plate details. It was shown that:

1. It may be seen that for structures with $T_{dc} < 3s$, which is the majority of realistic structures, the lateral displacement increases due to the increase of rotational flexibility. So, designing of a structure with the short to medium period by assuming that the bases are fixed, underestimates the frame displacement. The moment demands at the top of the column considering base flexibility are greater than those assigned the fixed base when the double curvature period, T_{dc}, is less than about *0.8s*. This indicates that, there is more likelihood of a soft-storey mechanism due to base flexibility of this range of period. In contrast, for the structures with the double curvature period, T_{dc}, higher than 0.8s, the top moment demands of the column considering base flexibility are lower than those assigned the fixed base. So, assuming bases of a structure as the fixed bases, leads to conservative design for the top moment of this period range of the structure. Furthermore, the possibility of formation of the soft storey decreases due to the base flexibility in this range of period.

2. The simple relations were introduced to estimate demands on the structure for the base flexibility. They could also estimate displacement of the elastic structure and the structure with nonlinear bases. Moreover, level of the base yielding moment determines the participation of lateral deflection of the column to the total displacement, so this value determines which of the described methods are more accurate to estimate the nonlinear base rotation.

3. Seven different base plate details were compared from seismic performance, cost, constructability and replacement. This comparison shows that for a single detail different deflection modes could have different qualitatively grades with wide range of variation. The yielding base plate, unbounded steel rods, Welding of angles, and sliding hinge joint can reduce considerable part of energy by special mechanism (yielding and friction), and the nonlinearity is mitigated to other parts than columns. So, these are the candidates from these aspects for further studies to develop low damage base connections.

Acknowledgments

The authors would like to acknowledge MSI Natural Hazards Research Platform (NHRP) for the financial support of this study.

References

ASTANEH-ASL, A. 2008. Seismic Behavior and Design of Base Plates in BracedFrames. *SteelTIPS Reort.* Structural Steel Educational Council.

AVIRAM, A., STOJADINOVIC, B. & DER KIUREGHIAN, A. 2010. Performance and Reliability of Exposed Column Base Plate Connections for Steel Moment-Resisting Frames. *PEER.* Pacific Earthquake Engineering Research Ccenter.

IKENAGA, M., NAGAE, T., NAKASHIMA, M. & SUITA, K. Development of Column Bases Having Self-Centering and Damping Capability. Proceedings of the Fifth International Conference on Behaviour

of Steel Structures in Seismic Areas STESSA, 2006 Yokohama, Japan.
MAAN, O. & OSMAN, A. 2002. THE INFLUENCE OF COLUMN BASES FLEXIBILITY ON THE SEISMIC RESPONSE OF STEEL FRAMED STRUCTURES. *4th Structural Specialty Conference of the Canadian Society for Civil Engineering.* Montréal, Québec, Canada.
MACRAE, G. A., CLIFTON, G. C., MACKINVEN, H., MAGO, N., BUTTERWORTH, J. & PAMPANIN, S. 2010. The Sliding Hinge Joint Moment Connection. *Bulletin of the New Zealand Society for Earthquake Engineering,* 43**,** 10.
MACRAE, G. A., URMSON, C. R., WALPOLE, W. R., MOSS, P., HYDE, K. & CLIFTON, C. 2009. AXIAL SHORTENING OF STEEL COLUMNS IN BUILDINGS SUBJECTED TO EARTHQUAKES. *Bulletin of the New Zealand Society for Earthquake Engineering,* 42**,** 275-287.
MIDORIKAWA, M., AZUHATA, T., ISHIHARA, T. & WADA, A. 2006. Shaking table tests on seismic response of steel braced frames with column uplift. *Earthquake Engineering & Structural Dynamics,* 35**,** 1767-1785.
NZS1170.5 2004. Structural design actions - Earthquake actions. Standards New Zealand,Wellington