GROUND MOTION CHARACTERISTIC EFFECTS ON MULTISTOREY STEEL FRAME RESPONSE

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SUMMARY

Inelastic dynamic time-history analyses of nine steel 2-D frames and one 3-D frame were analyzed using a suite of ground motions with different characteristics to anticipate the effect of ground motion characteristic on structural response. Particular ground motion characteristics studied were the effect of near-fault ground shaking, vertical shaking, and record frequency content and magnitude on 2-D frame behavior. The behavior of 3-D frames to bi-directional shaking was also investigated. It was found in the 2-D frame analyses that near-fault strike-normal records caused demands as high as 6.7%, residual displacements were approximately 52% of the peak inelastic displacements, and the SRSS method for predicting the maximum increase in column axial force for simultaneous vertical and horizontal shaking was non-conservative due to frame inelasticity. Drifts did not increase linearly with increasing earthquake ground acceleration but showed significant scatter. Changing the record frequency content while maintaining the spectral response at the fundamental period of the structure significantly altered the location of peak drift. The 3-D analyses indicated gravity column yielding. Also, drifts in one direction increased when orthogonal shaking occurred.

INTRODUCTION

Current earthquake-resistant design criteria for structures has been developed based on the two-dimensional behavior of frames subject to moderate-size earthquake shaking obtained from recordings near the fault or from large earthquake shaking recorded at a significant distance from the fault. In recent years many new ground motions have been recorded at different distances from the fault and there has been an increase in computational capability. Consequently, more sophisticated structural analyses to different types of ground motion records may now be carried out in order to improve understanding of structural behavior, and evaluate and improve existing design methods. The study described in this paper was carried out to investigate and quantify the likely response of inelastically responding moment-resisting steel 2-D and 3-D frames to horizontal bi-directional near-fault ground motions as part of the US Federal Emergency Management Agency (FEMA) funded SAC Steel Project, a joint venture partnership of the Structural Engineers Association of California (SEAOC), Applied Technology Council (ATC) and California Universities for Research in Earthquake Engineering (CUREe). In particular, a summary of effects of near-fault shaking, vertical accelerations, record magnitude and record frequency content are investigated for 2-D frames, and the effect of bi-directional horizontal loading on the behavior of steel frames is discussed. Implications of this behavior for frame design are discussed elsewhere (MacRae, 1999).

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STRUCTURES, MODELLING AND EARTHQUAKE RECORDS

Structures of 3, 9 and 20 stories in height were designed by consulting engineers for regions of low, medium and high seismicity resulting in a total of 9 structures. The low, medium and high seismicity locations were Boston, Seattle and Los Angeles respectively and have dimensions shown in Figure 1. The three buildings in Los Angeles structures are referred to as LA3, LA9, LA20 respectively. The structures represent buildings that were designed prior to the 1994 Northridge earthquake and the resulting code changes. Member sizes are given by Krawinkler (1998). Seismic moment-resisting frames are used around the perimeter of the structure and gravity frames are internal.

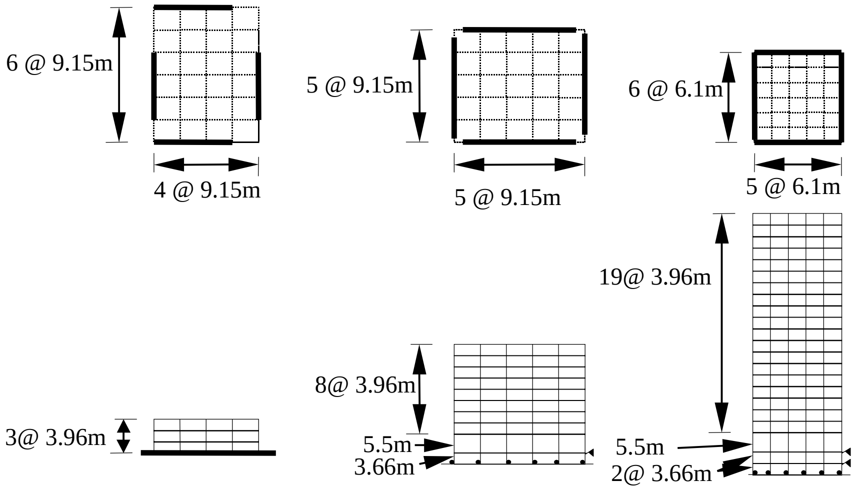


Figure 1. Frame Dimensions

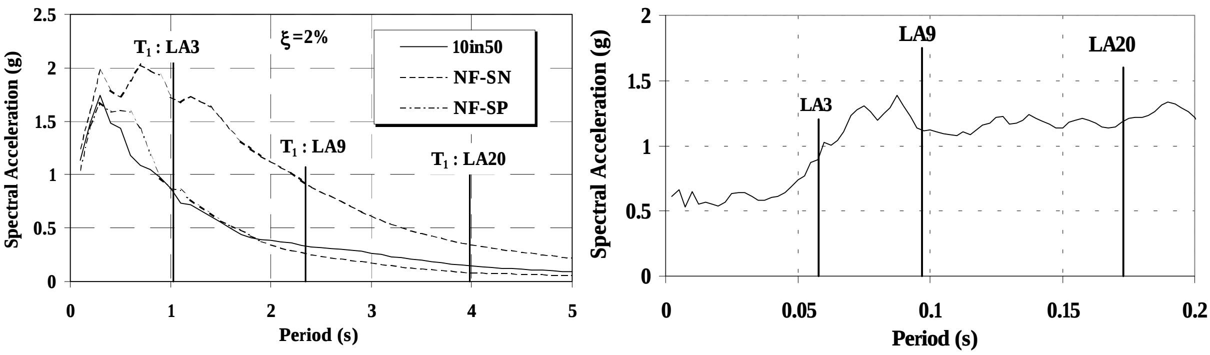
Modeling of the frames was carried out using DRAIN-2DX (Prakash et al., 1993) for two-dimensional analyses and DRAIN-3DX (Powell et al., 1994) for three-dimensional analyses (MacRae 1999). Centerline modeling was used at all beam-column connections. Floor slab contributions to beam strength and stiffness were neglected. Beams were modeled to be elastic. Inelasticity at beam ends was provided by rotational springs with a high elastic stiffness of approximately $1000EI/L$. The flexural strength of the spring was the beam plastic moment. Strain hardening was considered by giving the springs a post-yield rotational stiffness of $0.03\text{x}(6EI/L)_{beam}$. Rayleigh damping (Clough and Penzien, 1993) of 2% was used based on the initial stiffness of the structure in the first mode and at a period of 0.2s. No damping was applied to the rotational springs. Steel nominal yield strengths were 248MPa and 345MPa for Grade A36 and Grade A572 Grade 50 steel respectively. Actual likely yield strengths of 339MPa and 397MPa were generally used in the 2D analysis. Steel used in buildings generally had a yield strength of 397MPa, except the beams of the LA structures had a yield strength of 339MPa.

For the 2-D frame, column behavior was modeled with the DRAIN-2DX 2-component column model. A strain hardening factor of 0.03 was used in this model. P-delta effects on the frame were modeled using the vertical gravity forces on the half of the structure. The gravity forces that were not carried directly by the frame were carried by a P-Δ column (Krawinkler, 1999). The 2-D frame horizontal fundamental periods including P-Δ effects are 1.027s, 2.344s, 3.984s, 1.364s, 3.170s, 3.918s, 1.889s, 3.327s and 3.194s for structures LA3, LA9, LA20, SE 3, SE 9, SE 20, B03, B09 and B020 respectively.

Vertical mass was generally lumped at the nodes but it was also placed at ¼ points along the beam length as a to see the effect of distributed mass. Vertical periods for the frames with lumped nodal mass were 0.058s, 0.097s and 0.173s for LA3, LA9 and LA20 respectively.

The 3-D response of the LA3 frame only was studied. Column modeling was carried out using a fiber element and near-rigid struts were provided on the roof to ensure a rigid diaphragm. The structure is near-symmetric but there is a mass eccentricity for loading in the N-S direction and a stiffness eccentricity for loading in the W-E direction. Columns in the perimeter frame were modeled with a fully restrained base for bending about both the weak and strong axis directions and all internal gravity columns had pinned bases. Since 3-D analyses are more complex than 2-D analyses, the response of a 2-D frame in the 3-D program was benchmarked against 2-D program response using pushover and dynamic inelastic time-history analyses. Also, the hinge overstrength errors and time-steps were adjusted to get repeatable results and still cause convergence.

The records used were for the studies were from the SAC database. Ten sets of so-called near fault (NF) earthquake motions with strike-normal (SN), strike-parallel (SP) components and vertical (V) components were provided. A series of 20 design level records with a probability of exceedance of 10% in 50 years (10in50) were also provided for each site. Details of these records are provided by Somerville et al. (1997). "Median" design spectra, computed using the logarithmic mean (Luco and Cornell, 1998) for the LA records are given in Figure 2a. Here, it may be seen that the NF-SP records are of similar magnitude to the 10in50 records. Also, the NF-SN and NF-SP records have similar magnitude up to a period of about 0.5s. Vertical acceleration response spectra, also based on 2% critical damping, indicated spectral accelerations greater than 1g as shown in Figure 2b.



(a) Horizontal Shaking (b) NF Vertical Shaking

Figure 2. Median Response Spectra for Records used with LA Structures

2-D FRAME RESPONSE

Response to Near-Fault Records

Near fault strike normal (NF-SN) ground motions caused median peak story drifts of 6.7%, 5.7% and 4.8% for 3, 9 and 20 story frames (LA3, LA9 and LA20) designed for Los Angeles respectively. Drifts for LA9 are shown in Figure 3. For NF-SP ground motions, the median peak drifts reached 2.3%, 2.1% and 1.3%. These drifts were close to the design level earthquakes magnitude with a 10% probability of exceedance in 50 years (10in50) of 2.2%, 2.1% and 1.7% respectively. Median story drifts were 1.12, 1.63 and 2.75 times the structure roof drifts for the LA3, LA9 and LA20 frames. NF-SN median peak inelastic beam rotations reached 5.4% for LA3, 4.0% for LA9, and 3.7% for LA20 which is significantly greater than the 10in50 records where median peak inelastic beam rotations were less than 1.0% for LA3, 1.0% for LA9, and 0.75% for LA20. NF-SN median peak inelastic column rotations reached 4.5% for LA3, 3.0% for LA9, and 2.6% for LA20. This column yielding was concentrated at the base of the first story columns. The 10in50 records caused no significant column yielding. NF-SN median residual drifts were 1.8% for LA3, 2.2% for LA9, and 0.82% for LA3, LA9 and LA20 respectively. Median residual drifts using all types of record were approximately 52% of the maximum possible residual drift where the maximum possible residual drift was equal to the peak drift minus the yield drift. NF-SN median peak roof drifts in LA3 reached 6.1% compared to 2.0% for 10in50 records, 3.5% compared to 1.4% in LA9, and 1.7% compared to 0.85% in LA20. NF-SN records created significantly higher levels of hysteretic energy absorption than the 10in50 records by a factor of 5.7 for LA3, 5.4 for LA9, and 4.5 for LA20. Near fault strike parallel (NF-SP) accelerations caused as similar response to the design level (10in50) ground motions in terms of story drift, beam rotation, column rotation, global drift and hysteretic energy absorbed.

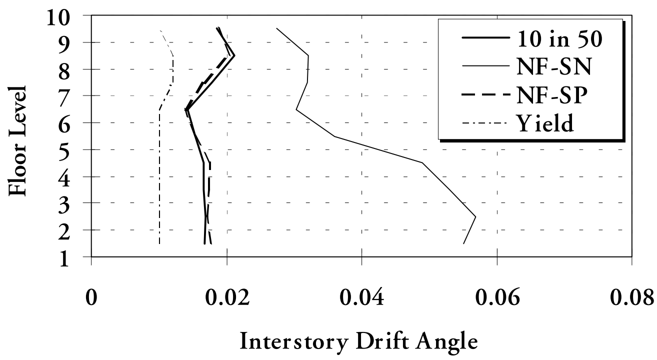


Figure 3. Median peak interstory drift angles for 9 story L.A. frame

A static inelastic analysis in which the beams yielded at each end without strain hardening, and the beam moment was distributed to the column in equal portions above and below the joint, is referred to as the "beam mechanism". Median peak column moments were on average 1.8 times, 2.2 times and 1.75 times those from the beam mechanism moments for LA3, LA9, and LA20 respectively for the NF-SN ground motions. This was a result of dynamic magnification, redistribution and strain hardening. Column median shears exceeded these beam mechanism shears by factors ranging from 1.1 to 2.4. LA3 and LA9 exterior columns were subjected to axial loads from beams at all stories yielding simultaneously. For LA20 all beams did not yield simultaneously due to higher mode type effects and the axial loads in the lower stories were less than that expected from a full mechanism.

Effect of Vertical Shaking

An integration time step of Δt = 0.001s was used in order to capture the response of the frame vertical periods. Smaller time steps produced a similar response. However, since the input record acceleration time step was as great as 0.02s the records used may not have sufficient resolution to give good accuracy for the structural periods of interest. It was found that vertical ground accelerations primarily affected the column axial loads. The effects were most prominent in the internal columns which developed no significant axial effects due to horizontal shaking alone. Internal column median peak compression with combined vertical and horizontal acceleration records (NF-SN-V and NF-SP-V), exceeded column compression from horizontal records alone (NF-SN or NF-SP) by 23% for LA3, 49% for LA9, and 27% for LA20. For external columns, these percentages reduced to 3.6%, 4.8% and 4.4%. The median peak column compression was typically about 5% greater than the SRSS load combination prediction for vertical and horizontal records analyzed separately. This was because frame plasticity which increased the likelihood of simultaneous peak column axial forces. During the combined vertical and horizontal dynamic analyses, nearly every record induced tension in the external columns. The internal columns, which were only influenced by the vertical accelerations, achieved tension in few analyses, one out of twenty possible instances for LA3, 3/20 for LA9, and 2/16 for LA20. Distributing the vertical masses along quarter points of the beam as well as beam ends changed the column axial load demands by less than 10%. Column axial loads from distributed mass analyses were less than axial loads found from analyses with vertical masses placed only at the beam-column joints in 27 out of 30 instances compared.

Effect of Record Magnitude

Incremental dynamic analyses, in which the shaking magnitude was increased in each subsequent analysis, was carried out. Ten 10in50 records were used with each structure. Acceleration scale factors were selected to cause a range in response from near-elastic to a level significantly greater than that in a major earthquake. It was found that peak story drift increased almost linearly with acceleration magnitude to a frame drift of about 3%. For higher drifts, depending on the structure and record characteristics, drifts did not change significantly for a large range of ground motion magnitude increase and for other records drifts increased rapidly.

Effect of Record Frequency

Steel moment resisting frames were subjected to the same ground motion record, but the record time-step was changed to affect the ratio of the period of peak spectral response, T_r, relative to the fundamental period of the structure, T_s. The acceleration magnitude of the records was then scaled so that the elastic spectral acceleration at the fundamental period of the frame, S_a, matched the median spectral acceleration of 10in50 records at that period. An schematic of the record response spectra is shown in Figure 4. Analyses were conducted with three 10in50 records and three artificial pulse records. A large amount of variation occurred in the peak roof displacements for frames analyzed with ground motion records that have equal first mode elastic spectral acceleration but very different spectral accelerations away from the fundamental period of the frame. Large drifts occurred in the lower stories when the period of peak spectral acceleration was greater than the fundamental period of the oscillator, and in the upper stories when there was a large spectral response at a shorter period than the fundamental period of the oscillator.

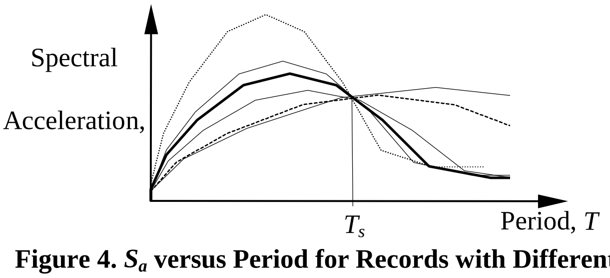


Figure 4. S_a versus Period for Records with Different Frequency Content

3-D FRAME RESPONSE

Near fault earthquake record sets with large strike normal components of shaking with respect to the strike parallel component were selected to analyze the structure. The SP component of shaking was always applied at 90^o to the SN component. Drifts in the 3-D structure subject to both SN and SP shaking along the building axes were sometimes greater and sometimes less than the displacement of the structure modeled as a 2-D frame subject to the SN component of loading depending on the records chosen. When the NF SN motions were applied at different directions to the principal axes of the building frame the peak response was generally in the direction of SN loading, or close to this angle, as a result of the smaller orthogonal SP component of loading affecting the response. Peak drifts of inelastically responding frames of up to 7% occurred with the near-fault records. Drifts due to near fault shaking at 45^o to the building axis generally were larger than due to shaking in other directions. Frame drifts in the principal shaking direction increase by up to 84% as a result of orthogonal horizontal shaking. This occurred because yielding could occur at the base of the first floor columns due to shaking in either direction making superposition of response in the orthogonal directions invalid. The fact that the columns at the base of the structure were responsible for the increase in displacements was shown by conducting further analyses where all columns at the ground floor level were analyzed with pins in their weak axis direction. Here, the magnitude of response in each direction was independent of the shaking in the orthogonal direction.

Axial forces in the seismic columns were less than $0.15f_yA$. In the gravity columns they increased from $0.30f_yA$ to $0.495f_yA$ as a result of vertical shaking. No tension occurred in any columns due to vertical shaking. The gravity columns, which were pinned at the base and which were not designed to yield, yielded at the top of the first story during the severe near-fault shaking. Torsion was not significant in the 3-D structure which was analyzed.

CONCLUSIONS

Dynamic inelastic time-history analyses of nine buildings designed with 3, 9 and 20 story heights to various types of ground motion were described. It was found that:

1. The 2-D frame analyses showed that near-fault strike-normal directionality records caused median peak story drifts as high as 6.72% and concentration of deformation generally occurred in the lower stories.
2. Median residual drifts using all types of record were approximately 52% of the maximum possible residual drift where the maximum possible residual drift was equal to the peak drift minus the yield drift.
3. Vertical acceleration effects cause increases in column axial forces. The SRSS method for predicting the maximum increase in column axial force for simultaneous vertical and horizontal shaking was non-conservative due to frame inelasticity.
4. Roof displacement and peak story drift did not increase linearly with increasing earthquake ground acceleration. In some cases a large drift increase occurred as a result of a moderate increase in ground acceleration and in other cases, large increases occurred.
5. Analyses with records with different frequency content but with the same spectra acceleration magnitude at the fundamental period of the structure showed that deformations generally increase near the base of the structure when the fundamental period of the structure is smaller than the period of peak spectral acceleration response and that the location of peak drift was further up the structure when the peak spectral acceleration response was at a period significantly less than the structure fundamental period.
6. The 3-D analyses indicated small floor torsional effects, yielding in the gravity columns at the top of the first story, and drifts in the principal direction of shaking which increased due to shaking in the orthogonal direction as a result of flexural yield interaction at the seismic column bases.

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