PGC BUILDING BEFORE EQ

ROYAL COMMISSION HEARING

Nigel Priestley Presentation

BUI.CAM233.0196.2

BUI.CAM233.0196.3

PGC BUILDING AFTER EQ

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Deputy Chair, DBH "Expert Panel"

Expert Panel: Chaired by Sherwyn Williams (lawyer), and including representatives of Consulting Structural Engineers, Architects, Building Officials, Seismologists, Geotechnical Engineers, Academics.

Role: Assist and review work by Consulting Engineers appointed by DBH investigating collapse or damage to four buildings (PGC, FB, HGC and CTV), and to provide a report to DBH summarizing the consultants reports, and placing them in a wider context.

PGC STRUCTURAL FEATURES

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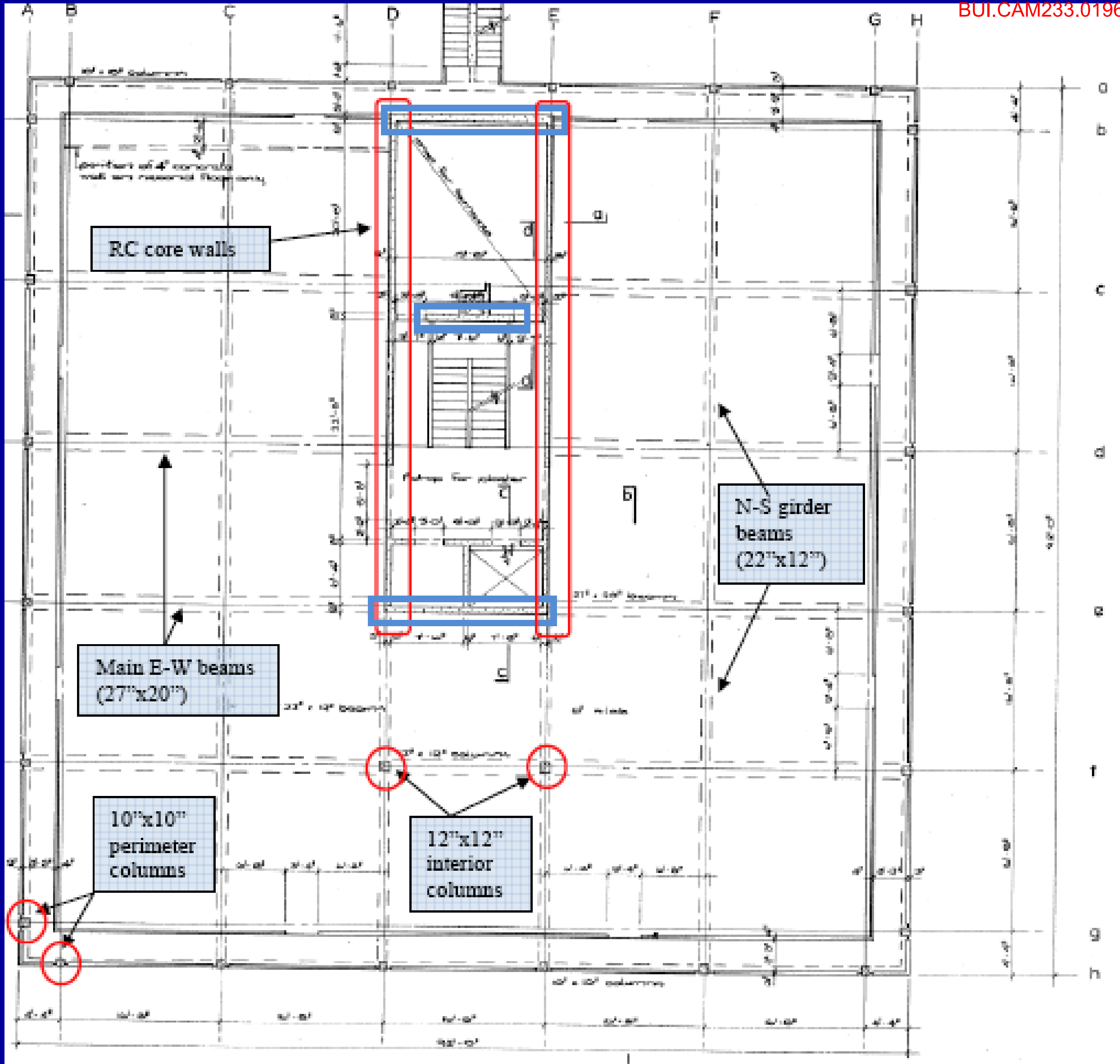
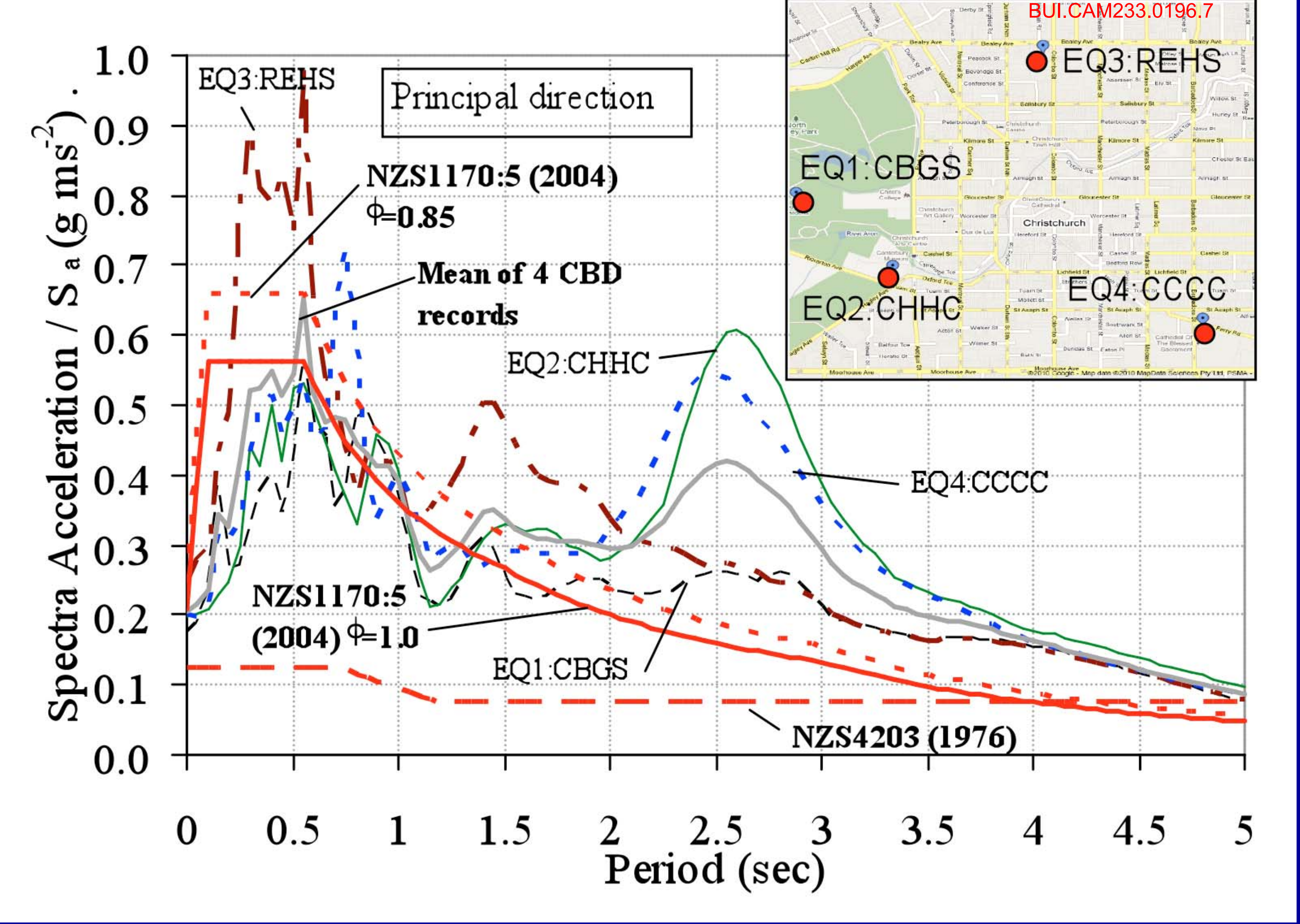
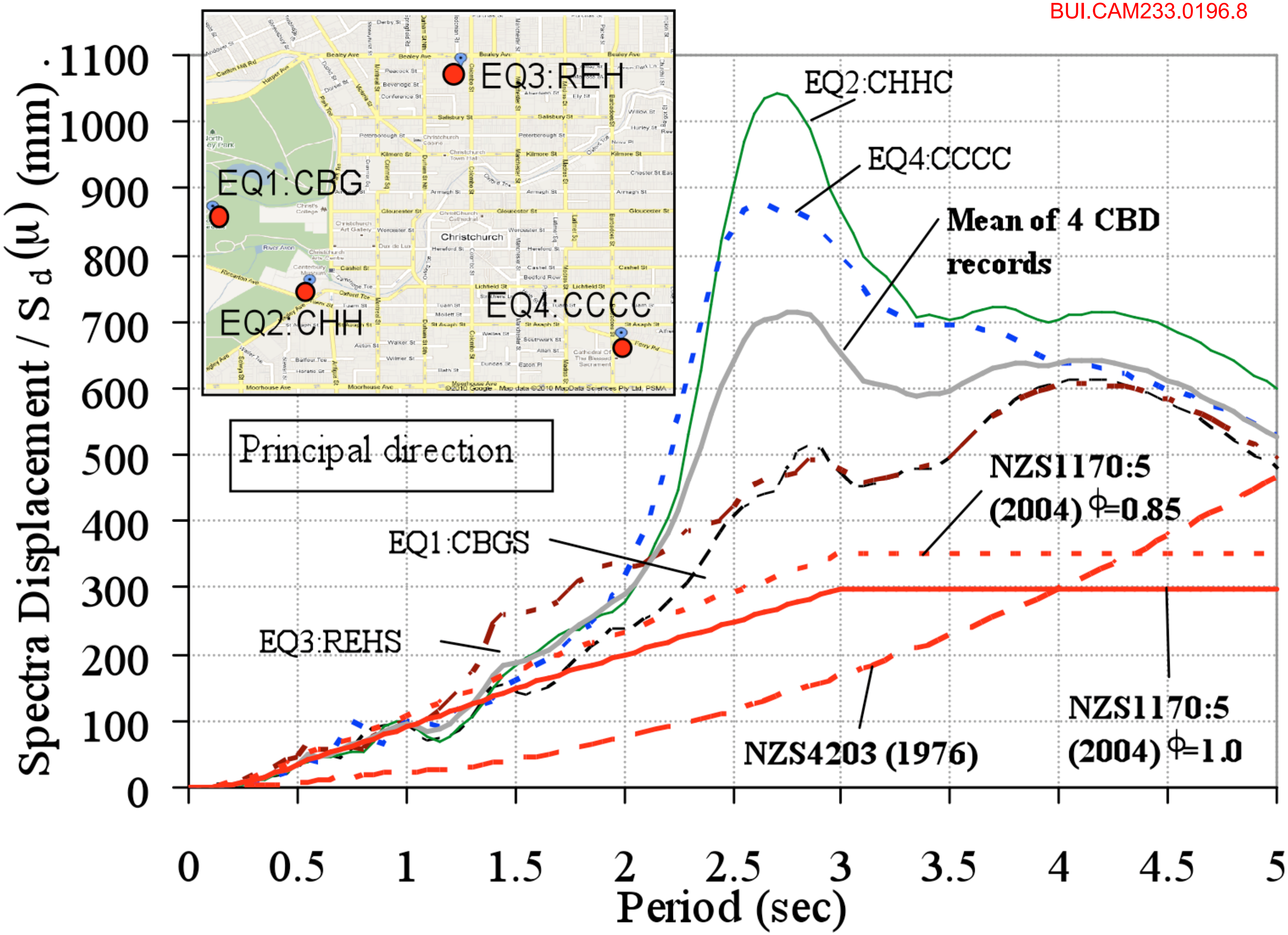


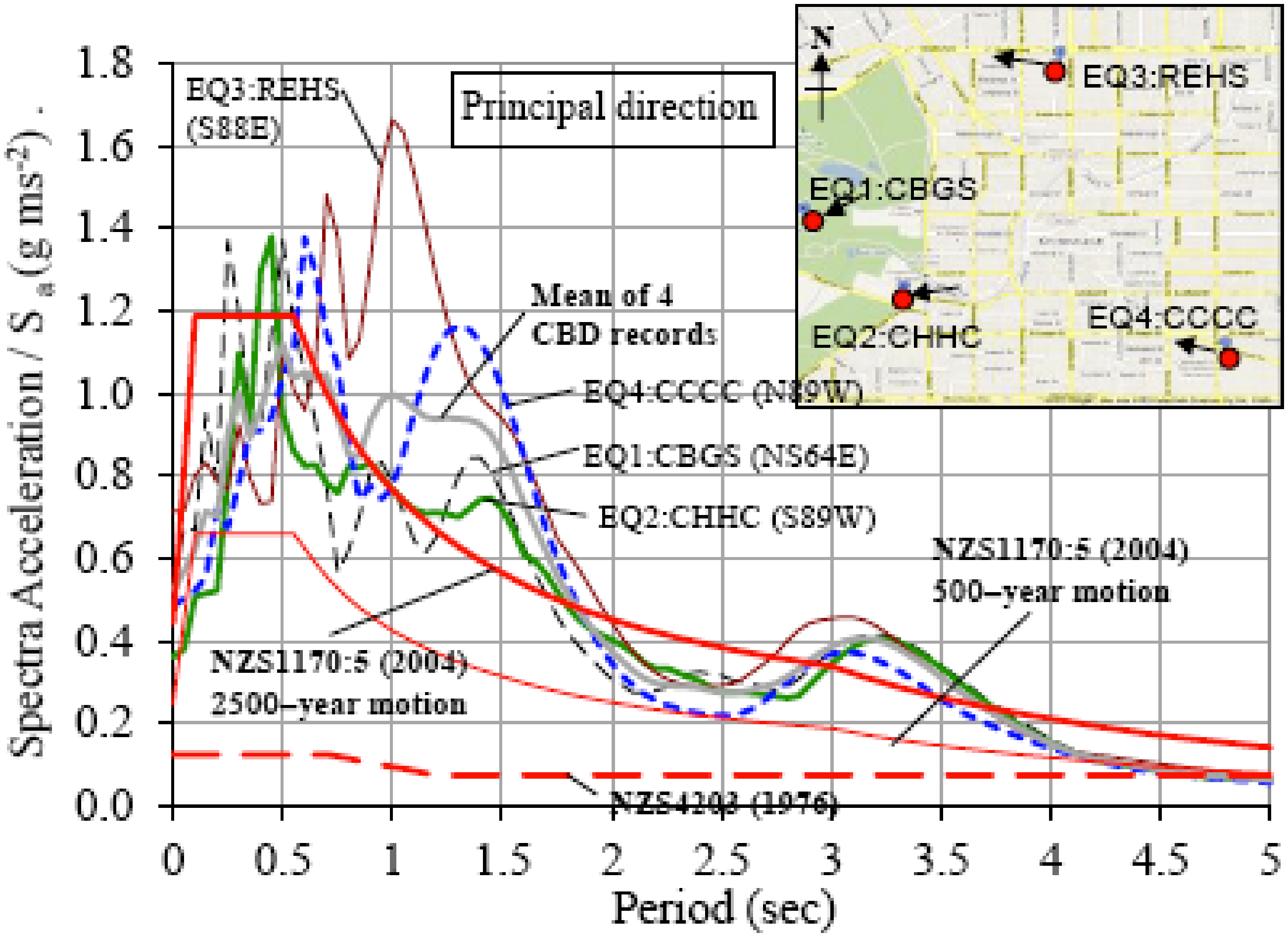


HORIZONTAL ACCELERATION RESPONSE SPECTRA FOR CBD IN SEPT. 4 EARTHQUAKE



HORIZONTAL DISPLACEMENT RESPONSE SPECTRA FOR CBD IN SEPT 4 EARTHQUAKE

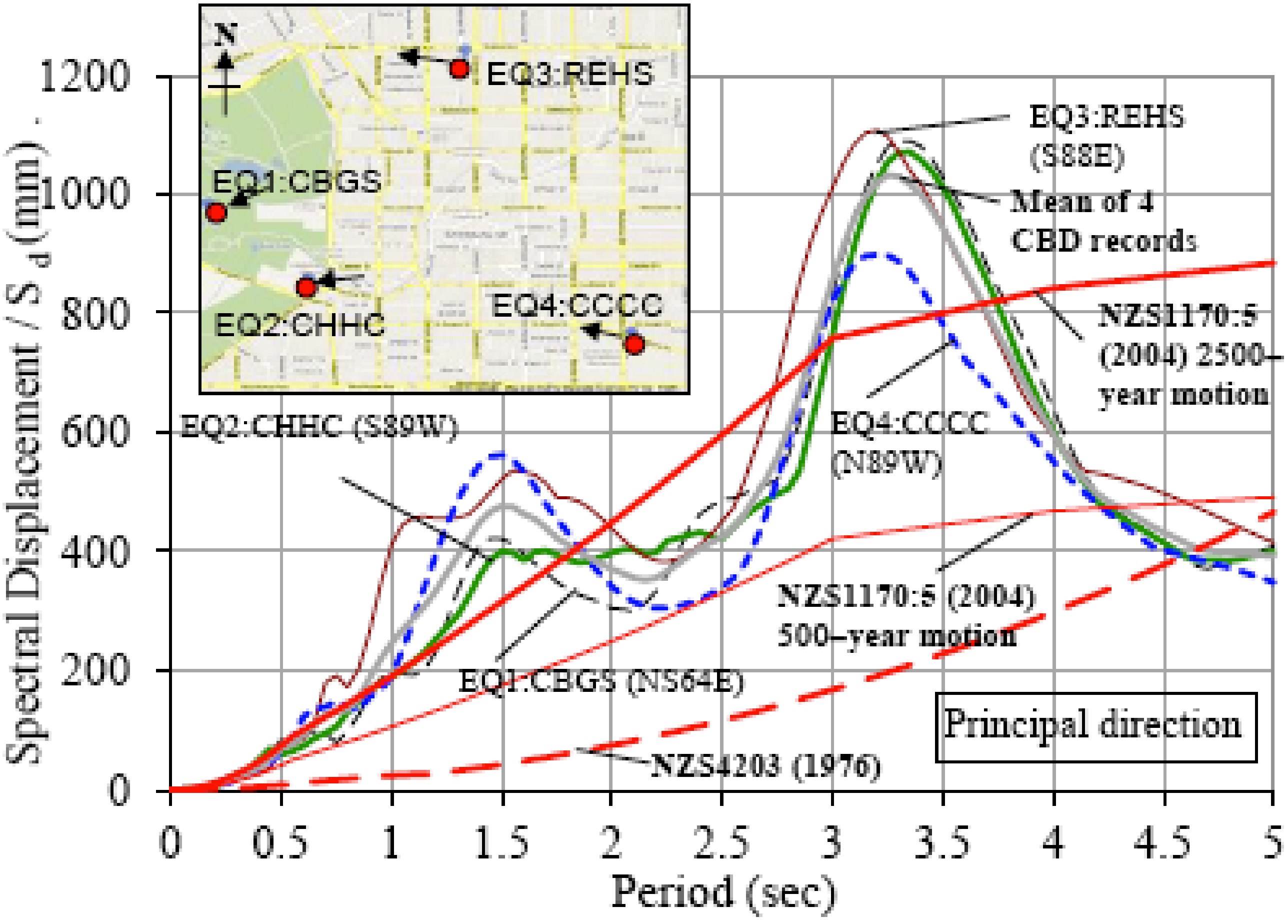
22nd Feb 2011 excitation



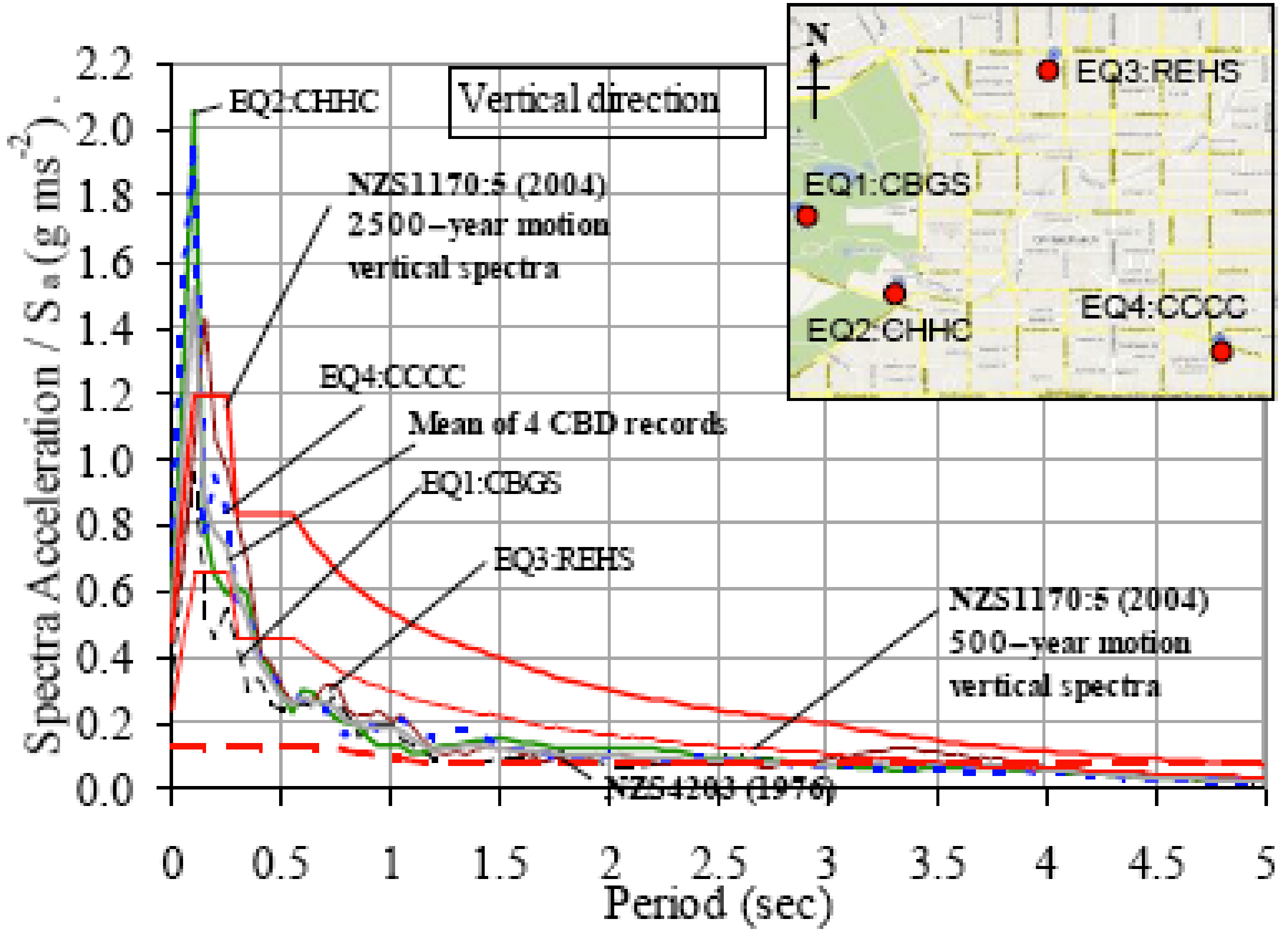
T>0.25s ~ >>500-yr design motion

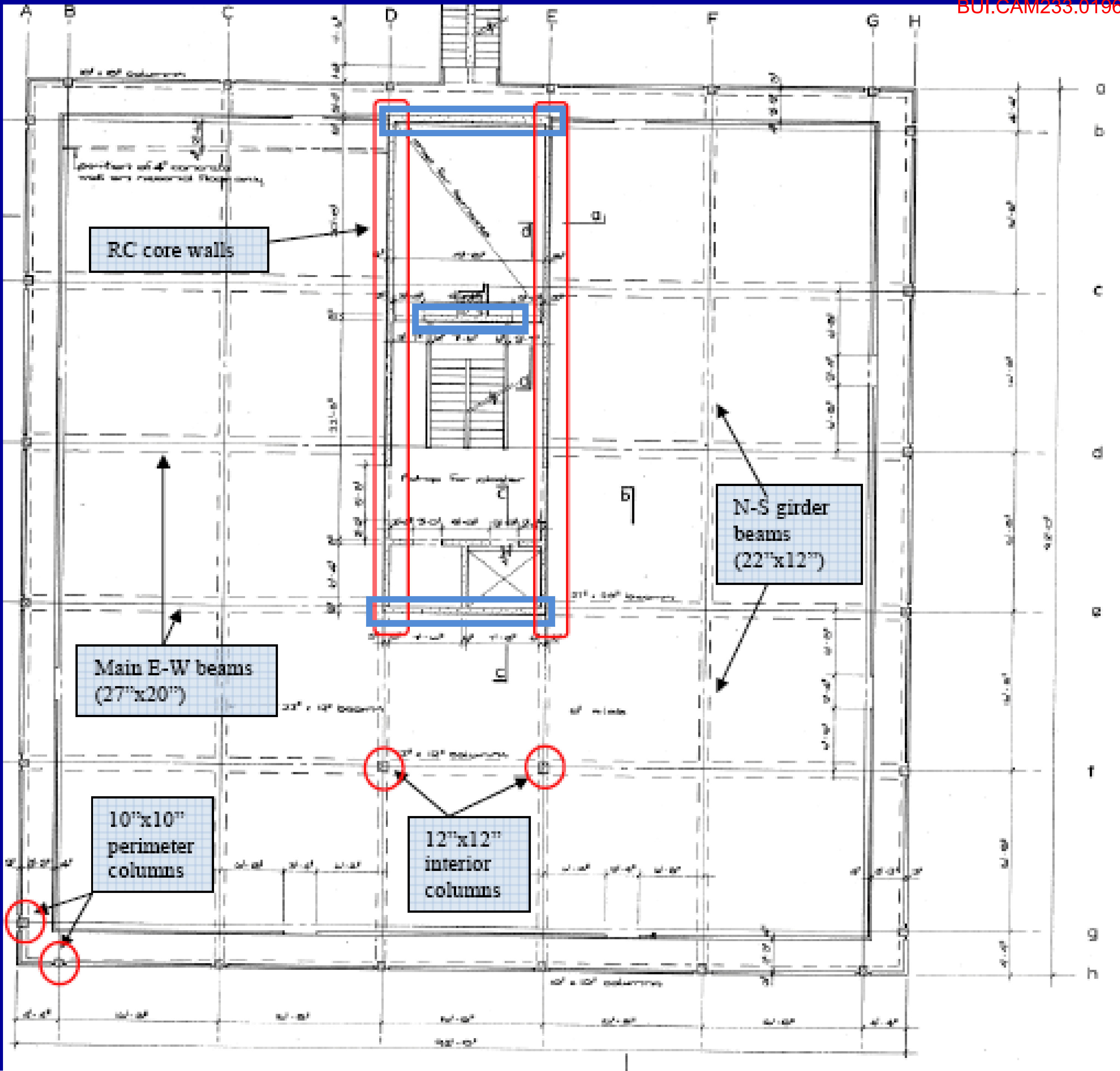
T=0.6s to 2.0s ~ 2500-yr design motion

Significant displacement demand



Vertical acceleration




RC core walls
N-S girder beams (22"x12")
Main E-W beams (27"x20")
10"x10" perimeter columns
12"x12" interior columns
A
B
C
D
E
F
G
H

BUILDING ISSUES

- Single layer of reinforcement in walls – no concrete confinement, no restraint of reinforcement buckling.
- Low reinforcement ratio in walls – strength of reinforcement less than concrete tension strength, restricting cracking (v. short plastic hinge length). Fracture of reinforcement at level 1 predicted at low displacements.
- Wall concrete compression strength rather low (25MPa)
- Poor detailing of columns above Level 1 – v. little transverse reinforcement, short lap splices of flexural reinforcement, leading to low displacement capacity.
- Capacity of shear core under simultaneous NS and EW attack very suspect.

DBH EXPERT PANEL FINDINGS

- General endorsement of Consultant's report
- Failure initiated by tensile fracture of flexural (vertical) reinforcement of West wall of shear core, followed by compression failure of East Wall at level 1.
- Large displacements subsequent to core failure caused failure of columns and connections of beams to shear core – floor collapse.
- Strength and detailing satisfied building code in-place when PGC designed.
- Displacement capacity and detailing would not satisfy current (Feb 2011) building code seismic intensity.
- Feb 22 aftershock exceeded current (Feb 2011) building code seismic intensity.
- Site foundation conditions not instrumental in building collapse
- Inspections after Sept.4 and Boxing day shakes did not indicate significant damage to building

ISSUES REQUIRING FURTHER DISCUSSION

The Holmes analyses of 1997 came to different conclusions about critical weaknesses of PGC than did the Beca analyses. Is this a concern?

Was condition after Sept 4 2010 EQ really as good as indicated by the post-eq evaluations?

COMPARISON OF HOLMES AND BECA ANALYSES

- Both consultants used Non-linear Time-history analyses (NLTHA) and similar computer programs to form their opinions
- Beca analyses indicated that Walls of shear core were critical, and that column (and hence floor) failure would occur subsequently.
- Holmes analyses indicated that column failure would precede wall failure, which was not seen as particularly critical
- Both analyses indicated that columns were poorly detailed.
- Holmes analyses used a representation of the current (1997) code seismic intensity; Beca used accelerograms recorded near the site in the earthquake and aftershocks.
- The analyses required subjective judgement of various aspects – particularly plastic hinge lengths, and shear performance.
- Although NLTHA is the most sophisticated analytical approach currently available, it is still an approximation to actual behaviour.

ISSUES WITH HOLMES ANALYSES

- It appears that the critical region for the shear core was incorrectly identified as the base of the wall. In fact it was at level 1, due to increased area of walls between G and 1.
- Method for modeling plasticity at wall base inappropriate
- Stiffness of columns and beams was too high
- It appear that beam-column joints were modeled as rigid elements.
- Problems with shear strength model for wall (shear flexibility included flexural components)
- Column plastic rotation capacity under-estimated. Because of steel jacket this is high between G and 1. At higher levels I estimate a total rotation capacity of at least 0.012, compared with the Holmes range of 0.002 to 0.007.
- Only one set of records used for the analyses (though these were scaled to different intensities).

ISSUES WITH BECA ANALYSES

- **Some confusion** over plastic hinge length for wall (400mm, 800mm, or 60mm reported)
- Plastic hinge length for columns is too low (40mm)
- Not clear how concrete tension capacity is dealt with in analyses.
- Displacement demand in Sept 22 EQ not clear in report, but appears to have been exceeded according to response-spectrum analysis.
- Assessed displacement capacity of "full" model seems too low.
- Some adjustment to displacement demand/capacity ratios seems appropriate
- Conclusions not affected by these issues.

BECA DISPLACEMENT DEMAND/CAPACITY RATIOS

- Pushover displacement capacity should be related to ductile, not elastic spectrum. Equivalent Viscous damping estimated as 10%: reduction to demand approx 24%.
- September 4 D/C apparently related to average of horizontal spectra at 4 sites
- February 22 D/C apparently related to average of SRSS combination.
- Displacement capacity could be 30% higher than "stick" model estimate, and 80% higher than "full" model

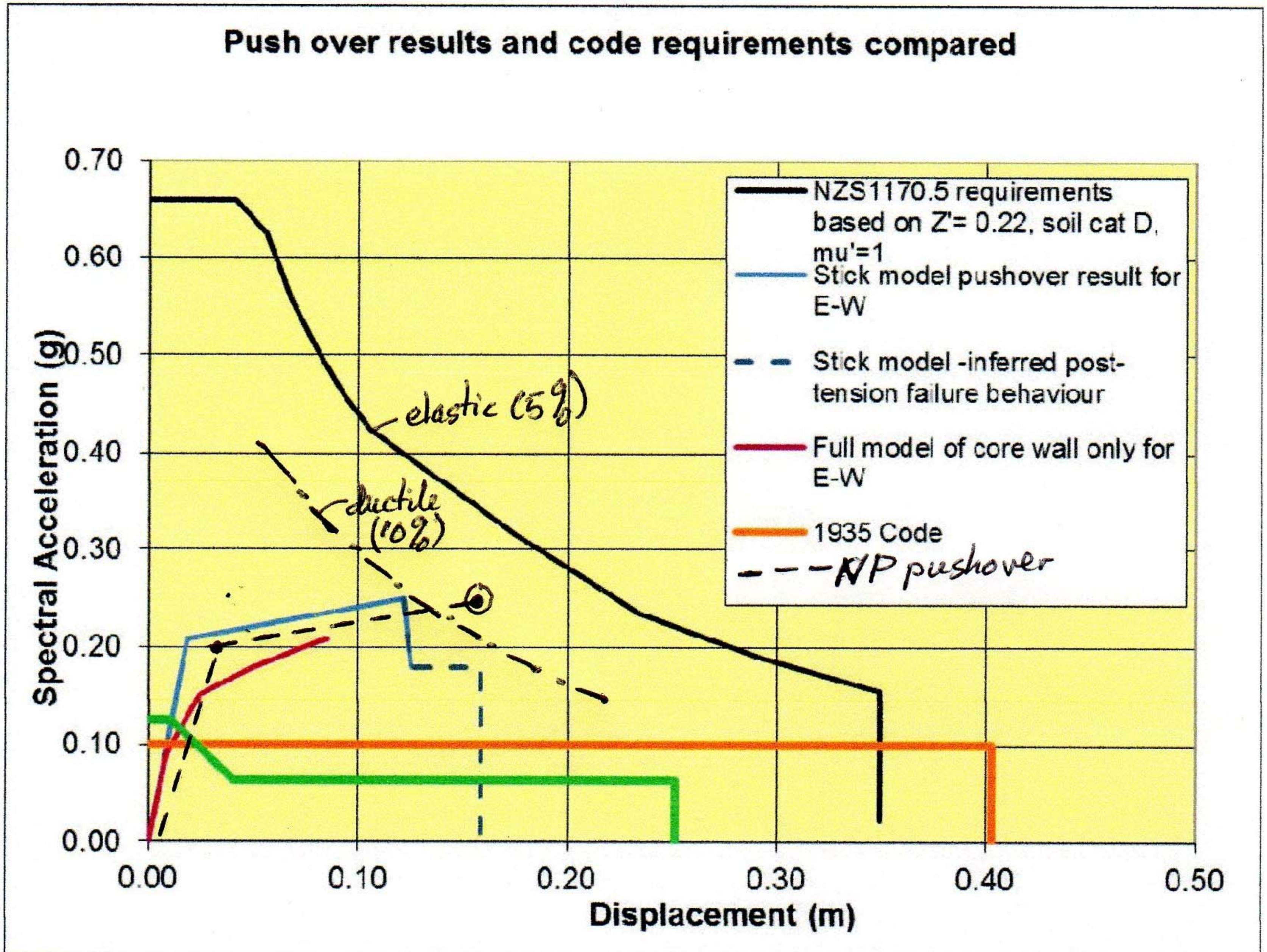


Figure A4.6.2: Pushover Analysis Results and Code Requirements Compared

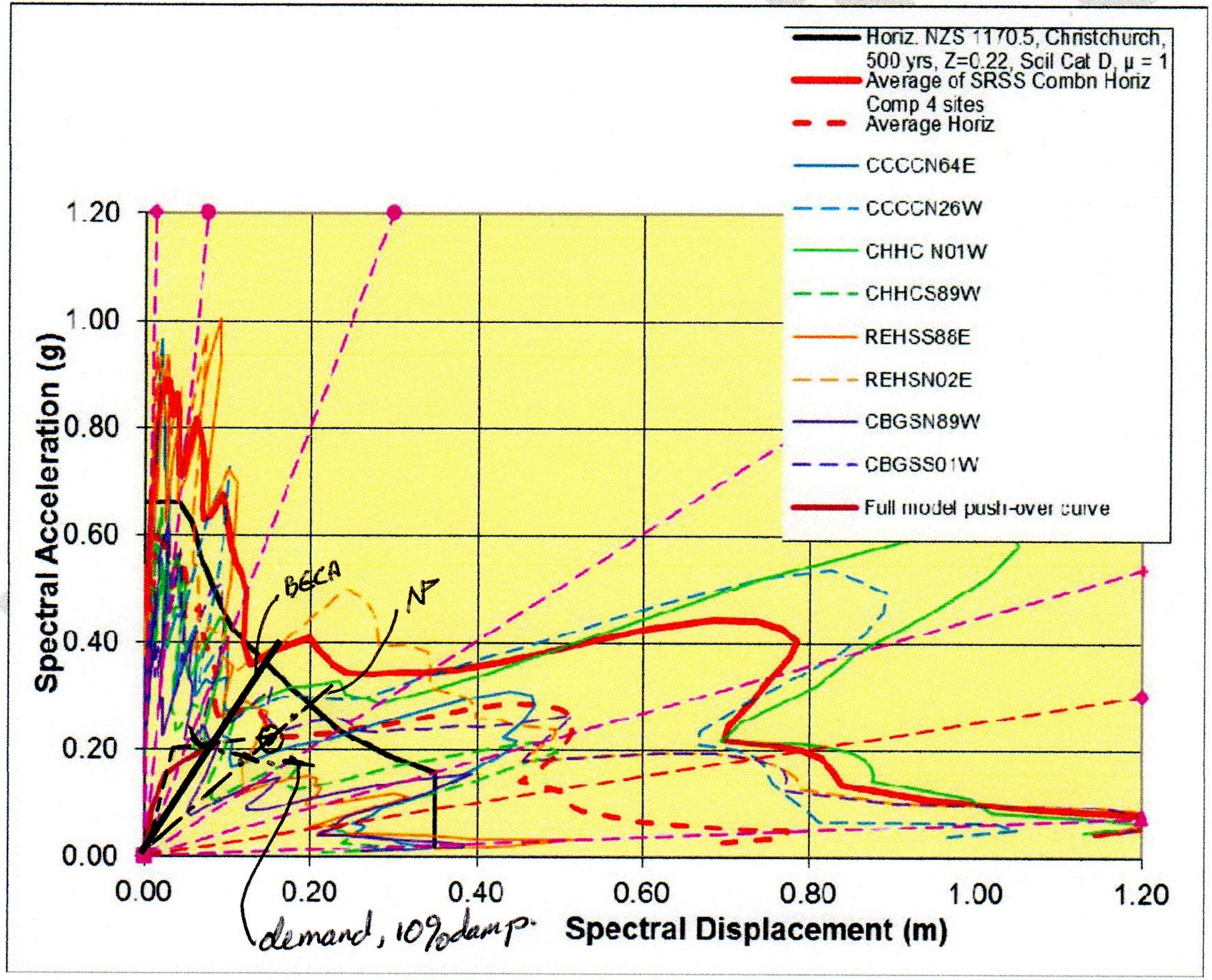


Figure A4.7.1: Push-Over Curve and Spectral Plots for 4th September 2010 Earthquake Compared

D/C = 1.3 (Beca, ave); = 1.9 Beca (SRSS) = 0.81 NP (Ave)

Feb 22 aftershock

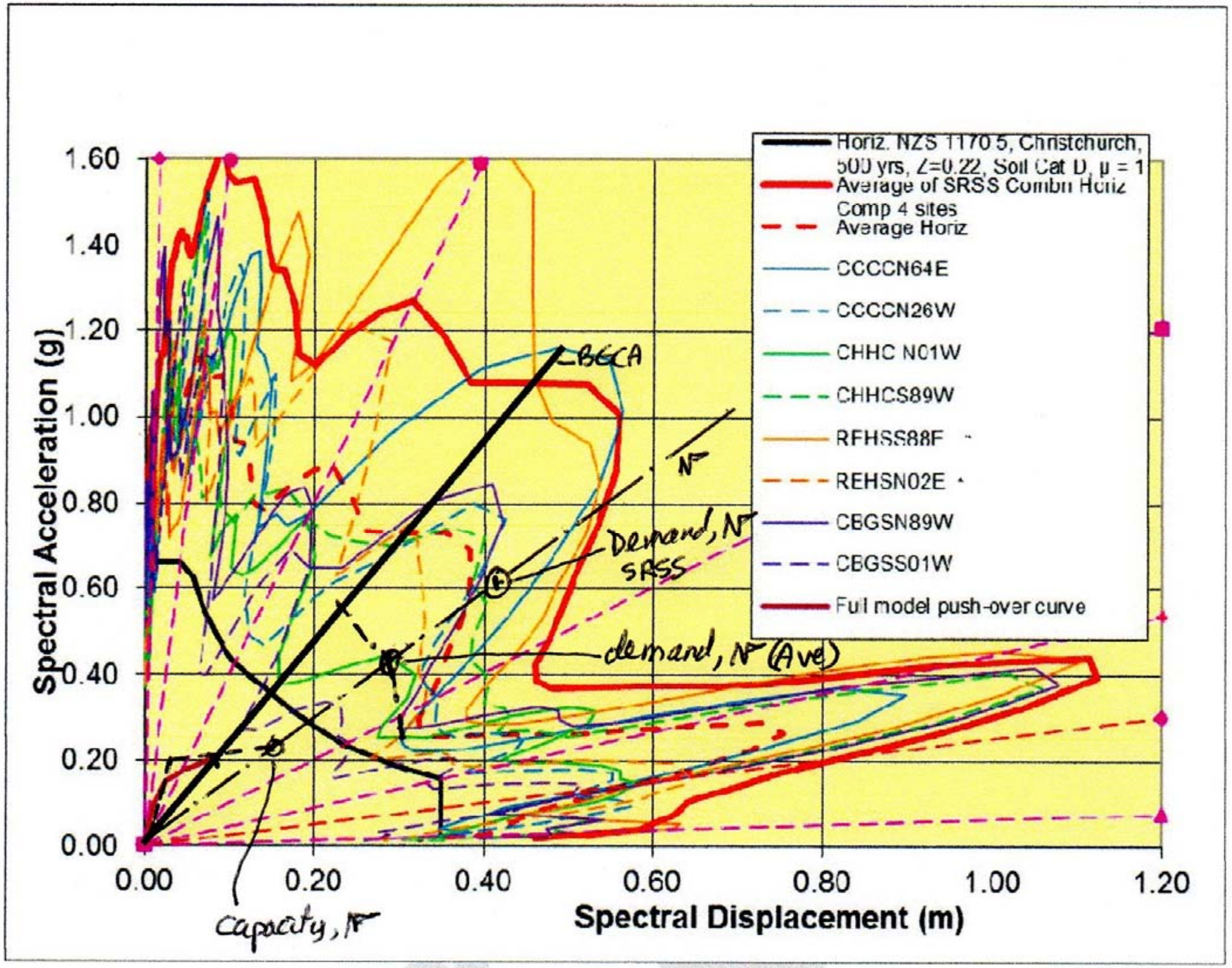


Figure A4.7.3: Push-Over Curve and Spectral Plots for 22nd February 2011 Earthquake Compared

D/C = 3.8 (Beca, ave) = 5.9 (Beca, SRSS) = 2.0 (NP, ave) = 2.7 (NP,SRSS)

PGC CONDITION AFTER SEPT 4 EQ

- Beca response spectrum analysis indicate D/C > 1 (failure), but NLTHA (and NP analysis) indicates no failure, but yield of wall.
- Eye-witness accounts indicate increased “liveliness” of building, post-Sept 4.
- Damage inspection indicates diagonal cracking and spalling in shear wall, and cracks at bottom of some columns.
- Crack widths were small after Sept 4, but reinforcement ratio in wall is so low that gravity loads would close cracks after shaking ceased, maybe giving a false impression of safety.
- Spalling of surface concrete in shear wall indicative of sliding on cracks – significant damage.
- Even if wall reinforcement had fractured in Sept (considered unlikely), this would not necessarily be apparent – again due to very low reinforcement ratio in wall (compare with Academy Towers).

Owner's engineer

Owner's engineer

(a) Cracking to Level One Shear Wall in Storeroom (inside the Southern End of the Shear-Core). Crack Widths between 0.2 and 0.6 mm

(b) Cracking to Level One Shear Wall. Typically less than 0.2 mm.

Shear core cracking after Sept.4 EQ (Beca report)

VISUAL ASSESSMENT PROCEDURES NEED REVIEW

With reinforced concrete structures, visual assessment tends to be based on crack widths, presence (or absence) of spalling of concrete) and residual displacements. The significance of these aspects depends on the quality of detailing and construction of the building.

A structure such as PGC which had poor detailing may not display significant apparent damage even if taken nearly to its capacity. Small increases in displacement may result in greatly increased damage, or even failure.

A well designed and detailed structure may be able to tolerate significant apparent damage (wide crack widths, spalling) without affecting capacity to sustain additional shaking.

Beca's suggestion for an active approach by TAs recording critical structural weaknesses of older buildings may point a way forward. Older buildings might not be given a "green" status until both visual inspection, and review of the CSWs were carried out.

PGCs CSWs: single layer rebar in walls; low reinforcement ratio in walls; poor confinement of columns, poor connection floors to walls.

COMPRESSION FAILURE OF WALLS

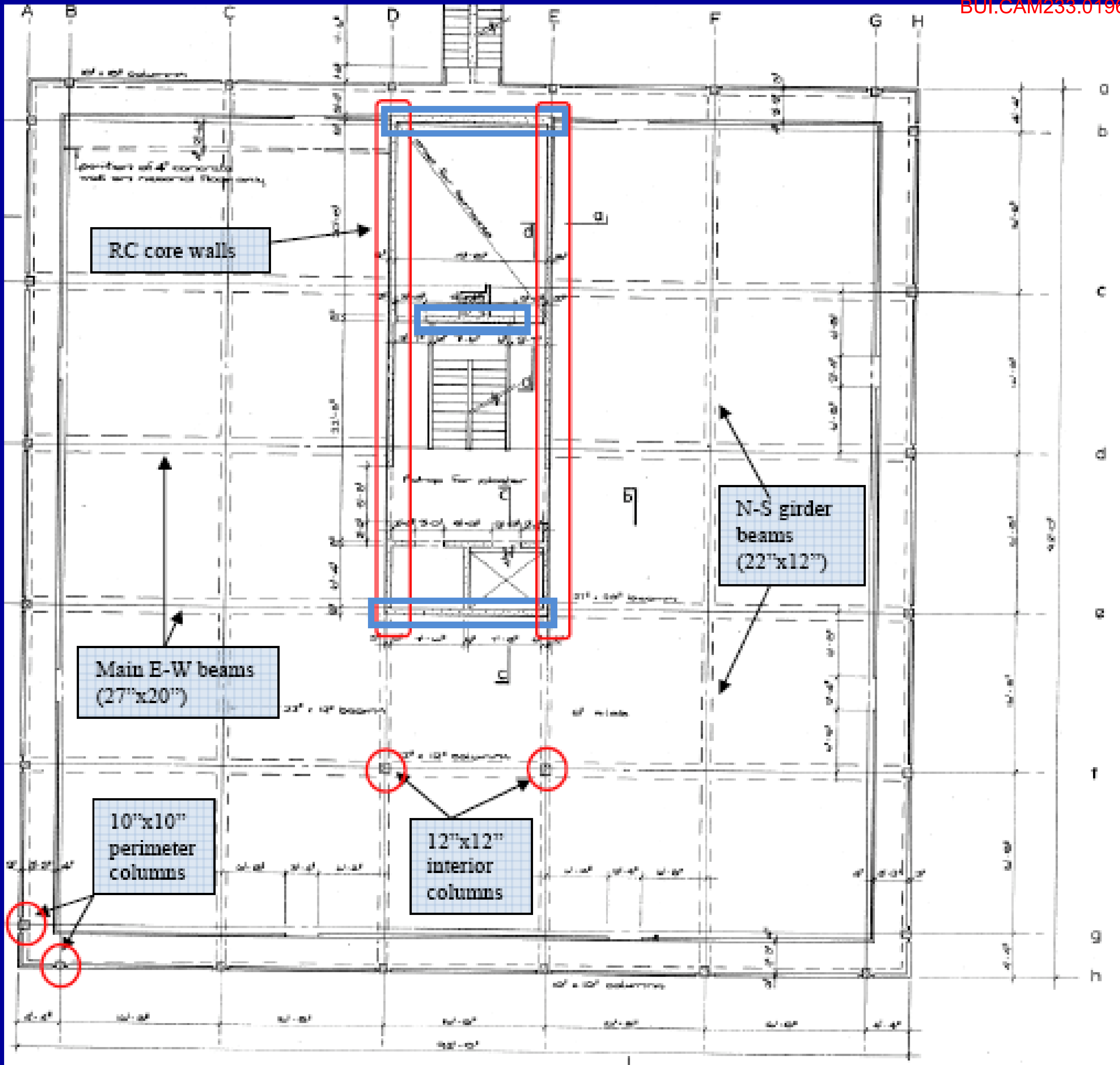
1. Compression stress on walls was not particularly high. Even if all weight and supported floor loads are considered, and this was carried by the East flange alone as West wall uplifted, compression stress would be only 12% of capacity.

2. However, East wall was not supported directly between G and 1 over full length (about 50% support). Considering a stress distribution from zero on one side of flange to maximum on the other (outside). The maximum stress would be 50% of capacity.

3. Simultaneous response in EW and NS directions would tend to concentrate the compression at one end of the wall, reducing the compression area and increasing the compression stress further.

4. If a wide crack occurred at level 1 (from fracture of vertical reinforcement, shear force would have to be transmitted through the compression zone. Calculations show the shear stress would be very high. This, combined with the high compression would cause failure of the compression zone.

5. Vertical acceleration could exacerbate this effect.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