N.Z. PARLIAMENT BUILDINGS SEISMIC PROTECTION BY BASE ISOLATION

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SUMMARY

Parliament House is to be partially demolished and rebuilt, extended within the existing perimeter envelope, refurbished and replanned except for the major public spaces, seismically upgraded by means of base isolation and enhancement of existing foundations, basement walls, ground floor, upper floor walls and floors.

This paper describes the assessment of appropriate seismic loads, the structural system, the analysis and design of the retrofitted structure. Anticipated construction procedures and difficulties are also addressed.

INTRODUCTION

The New Zealand Parliament complex consists of three permanent buildings: The Beehive or Executive Wing, completed in 1977; Parliament House, completed in 1922; The Parliamentary Library, which was completed in two stages, in 1883 and 1899. A fourth building Bowen House is temporarily fulfilling the functions of Parliament House and Parliament Library. Refer to the Site Plan, Figure 1.

Selection of Consultants for Upgrading

In 1989 the N.Z.Government took the decision to strengthen and refurbish Parliament House and the Parliamentary Library because of earthquake risk and the inadequacy of the existing facilities. Eight teams of Architects and Engineers were invited to submit six schemes for each building. Essentially the brief was open ended and asked the question "What would you propose to provide the accommodation required at three different levels of conservation viz maximum, moderate and minimum. Maximum conservation involved minimum intrusion into the existing structure and refurbishment. Minimum conservation involved demolition and rebuilding within the existing walls and moderate conservation involved adding new structure to the maximum conservation option. Further, for these three different levels of conservation, a proposal was required firstly treating the building in seismic terms as a National monument and secondly at a seismic level to ensure survival of the occupants. These were interpreted by our team to be equivalent to greater than MM IX intensity and MM VIII intensity respectively. Earthquake levels were not defined in the brief, although the proximity of the Wellington Fault and its associated hazard was noted. The selection process was conducted by an independent committee of New Zealand and international experts appointed by the Parliament Services Commission.

Development of Brief

Holmes Consulting Group and Warren & Mahoney, Architects were commissioned in October 1989 to develop their moderate conservation, National monument earthquake scheme with the recommendation that base isolation of the proposed shear wall structure be investigated. Structural schemes were developed as part of the overall development of the brief for a conventional shear wall scheme and secondly a base isolated shear wall scheme. The premium attached to the base isolated scheme was approximately 3% of building cost and the client accepted our recommendation to adopt the base isolated scheme on the basis that the 3% premium was well worth paying for the superior protection achieved. Subsequent development of the design has confirmed this as the correct decision. It would have been very difficult to ensure adequate ductile behaviour in the conventional shear wall solution because of the geometry of the walls. In particular special detailing would have been necessary at the base of the "columns" to allow them to yield. Consequently this structure would have attracted very large seismic responses. Working drawings were completed in mid-1991 and construction is planned to start in mid 1992 and be completed in 1995.

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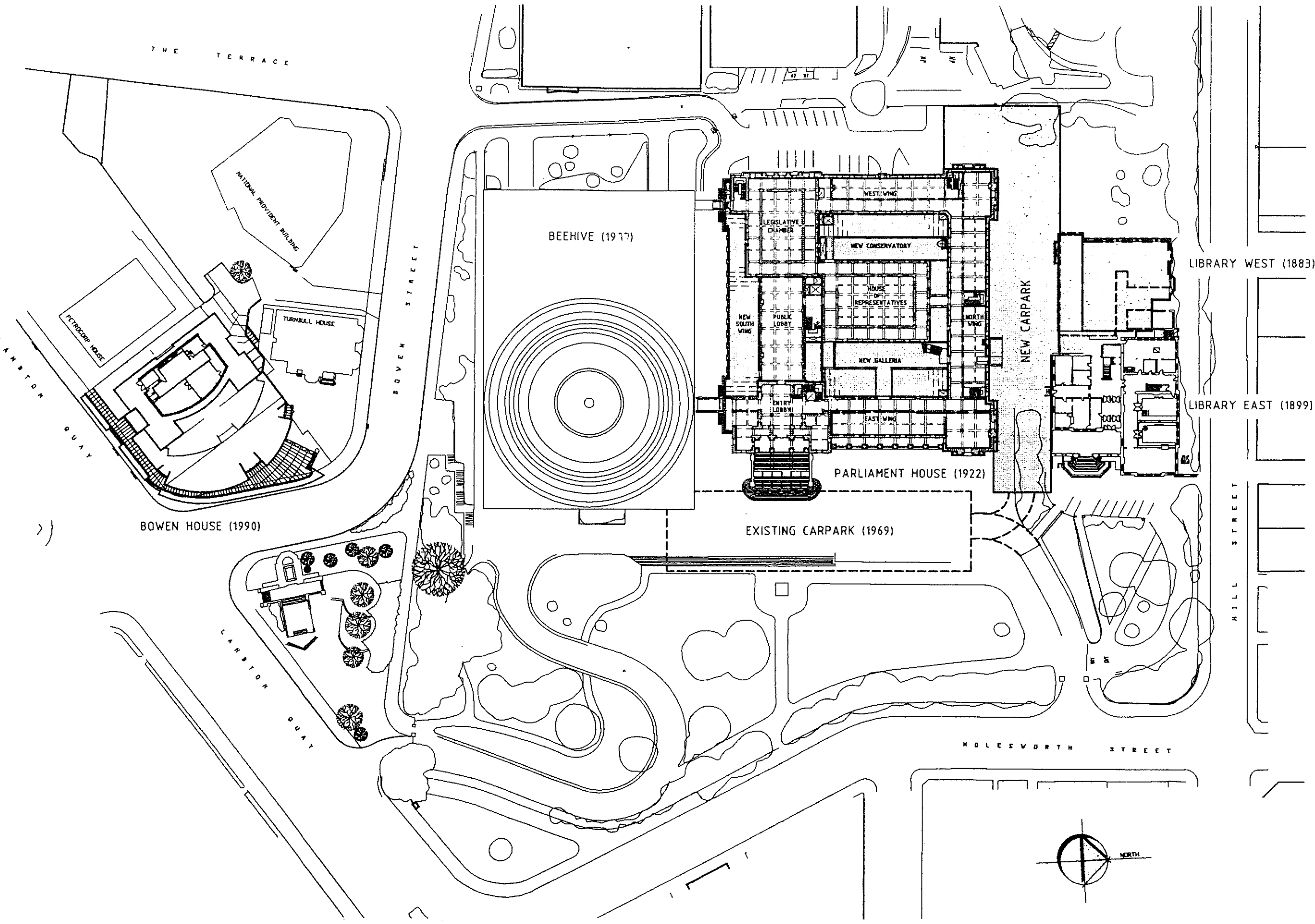

Figure 1 Site Plan of Parliament House, Wellington, New Zealand.

Existing Parliament House

Parliament House has five floor levels designated basement, ground, first (the main public floor) second and third. Some of the upper floor areas are additions to the original building. They are generally of a poor standard of construction and are to be demolished and rebuilt. A 1960's addition on the south side of the building is also to be demolished and rebuilt. The existing plan consists of the central House of Representatives block, perimeter wings and an extensive courtyard/lightwell area between the two. This lightwell area is to be infilled. Note the first floor plan, Figure 2 and the east-west cross section, Figure 3.

Parliament House Strengthening Scheme

Parliament House is to be base isolated by inserting a system of lead rubber bearings into the basement walls and columns. This requires extensive strengthening of existing basement walls and the ground floor. The new structures in the lightwell areas and on the south side have perimeter walls and new walls on grids 4 and 8. In addition various internal walls have concrete facings added to them principally to add strength to the building. Existing floors are to be strengthened to ensure adequate diaphragm strength. This strengthening will be a mixture of concrete topping overlays and steel strips and plates. A new lightweight steel and timber roof will be constructed over the third floor.

Parliament Library

The Parliamentary Library, consists of the west wing, which was built as an extension to an early General Assembly building in 1883, and the east wing which was built in 1899. The east wing is to be strengthened and refurbished by installing a base isolation system in the basement and strengthening the upper floor walls and floors, essentially the same system as Parliament House. The west wing does not have a layout suited to a modern library and is to be demolished and replaced with a modern base isolated shear wall structure. The north west walls of the existing west wing plus part of the grand Victorian men's toilet are to be retained and integrated with the new building in the interests of conservation. This is illustrated on Figure 1.

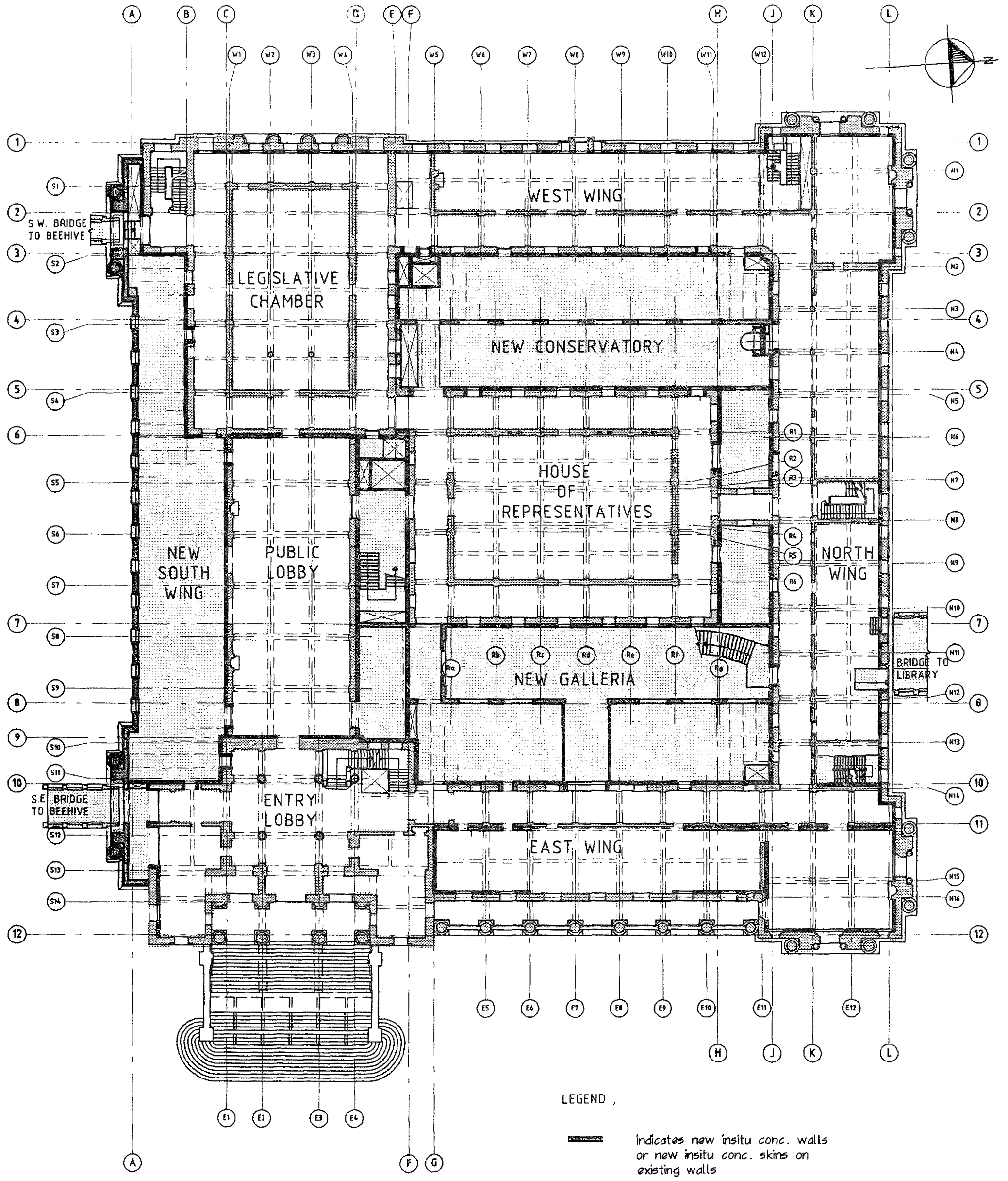

Figure 2 Parliament House, Level 1 Floor Plan.

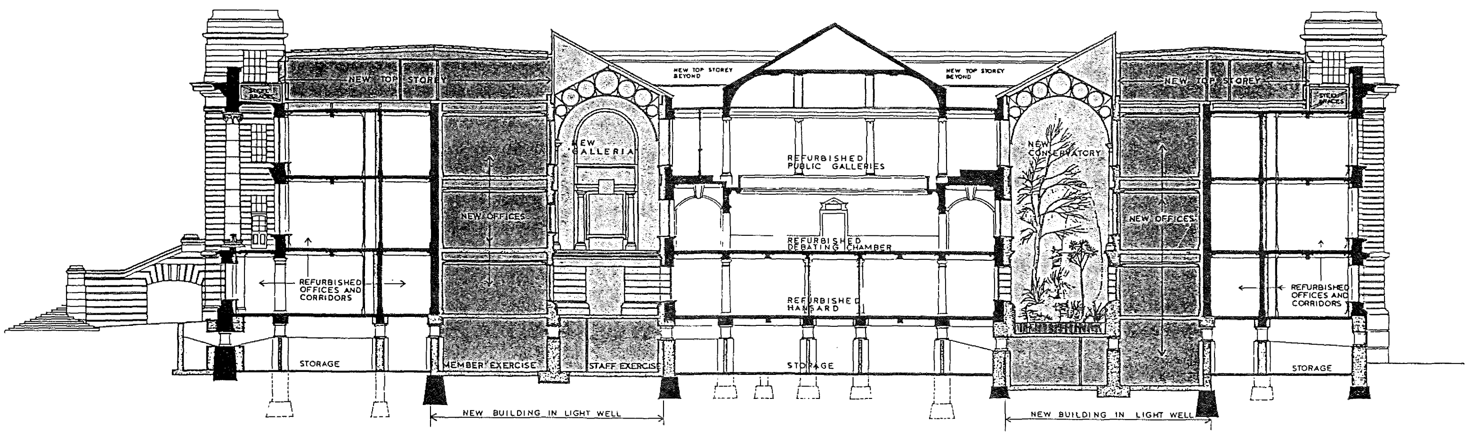

Figure 3 Section through Parliament Building showing proposed strengthening and refurbishment.

New Carpark

A new underground carpark building is to be built between Parliament House and the Parliamentary Library with a link to an existing underground carpark as indicated on the site plan, Figure 1. This carpark is at the level of the Parliament House basement which is 7 metres below the existing basement of Library East. This has been a challenging retaining/underpinning exercise.

This paper does not discuss these Library buildings or the Car park in detail but does refer to them with respect to aspects of particular interest which arise in the Parliament House discourse.

PARLIAMENT HOUSE STRUCTURAL SYSTEM

Description of Structure

The existing structure is essentially a masonry bearing wall structure. The floors are concrete, simply bottom reinforced with a metal lath mesh between a 2-way system of steel beams. The steel beams are supported on exterior and lightwell masonry walls and steel stanchions which are generally built into internal brick walls. The exterior walls are brick with stone facing on the east and west facades and a plastered finish on the north facade. Refer to Figure 3.

The new lightwell and south wing structures consist of proprietary precast flooring supported on reinforced concrete wall/frames.

System to Resist Earthquakes

Seismically the system is a base isolated shear wall structure. For the purposes of analysis and design existing masonry walls have been discounted. Seismic shears are accumulated in the various "shear walls" at each upper floor level and carried down to the ground floor where they are redistributed to the various base isolators which in turn transfer them to the foundations and ground. Where possible the new floors are used as the major diaphragms and existing floors, strengthened where necessary, are tied into them. Existing floor diaphragms are strengthened with concrete topping where possible. Elsewhere steel plates are used. This was provoked by conflict with the services requirements. The existing building has 75 mm false floors - an early intelligent building. For conservation reasons services cannot be exposed on the plastered ceilings of the building. Hence wiring and sprinkler pipes are to be located in the false floor rendering a concrete topping proposal useless. Hence the use of 5 mm steel plate fixed to the existing concrete with the new false floor over.

Strengthening to Accommodate Bearings

Extensive strengthening of existing basement walls, and in some cases foundations is required to redistribute foundation loads, both vertical and horizontal. Similarly the ground floor beams require major strengthening to redistribute wall loads to the bearings and to carry the moments induced by displacement of the bearings. They are also required to resist loads imposed by shear walls which are eccentrically located with respect to the bearings.

SEISMIC FORCES

In the original submissions design teams were asked to propose seismic forces appropriate to the brief described earlier. We sought the advice of Dr J Berrill of the University of Canterbury and he has assessed the risk in two ways:

Maximum Probable Event

Firstly the Uniform Risk Hazard analysis was used to assess the Most Probable Event (MPE) for the Wellington region. A return period of 350 years was proposed and confirmed by the clients consultant selection committee, the time frame being influenced by the fact that a base isolation system was proposed as opposed to a more conventional system. The maximum ground acceleration anticipated at the site is 0.5g. This is equated with ground shaking of MM IX on the Modified Mercalli scale. It is anticipated that an event of Richter Magnitude $M = 6.5$ centred within 5 km of Parliament would produce this shaking or alternatively a larger 'quake at a greater distance.

Maximum Credible Event

Secondly the site is within 400 metres of the Wellington Fault, Class I, and is therefore subject to specific risk associated with a rupture on this fault. The site is thus described as being subject to near fault effects. This risk, the Maximum Credible Event (MCE), is estimated to be an earthquake of approximate Richter Magnitude 7.0 to 7.5 and to produce ground shaking of MM X on the Modified Mercalli scale. Maximum ground accelerations of 0.85g are predicted. Dr J. Berrill and Dr Berryman of the DSIR collaborated to predict the likelihood of this event. A field investigation on the fault was completed by DSIR and the likelihood of this event, in the 150 year design life of the building, was predicted at between 10% and 50%. The recurrence interval of this event is believed to be 500 to 600 years and Geological Survey have advised that the fault has not moved for 350 years.

Response Spectra

The composite Design Response Spectra for the Maximum Probable and Maximum Credible Events are shown on Figure 4. These spectra are modified to simulate the effect of the bearings. Note the fundamental period of the base isolated structure is 2.5 seconds as opposed to the existing structure period which is assessed at 0.45 seconds.

ANALYSIS OF BASE ISOLATED STRUCTURE AND DESIGN OF BEARINGS

Factors Influencing Analytical Approach

There are a number of factors which influence the analytical approach and hence the design of the bearings in a structure such as Parliament House:

a) The bearing location and size are basically dictated by the gravity loads and geometry of the structure.

b) The bearings are very flexible compared to the foundations and superstructure and dictate behaviour. This structure is very much a stiff box on top of a "rivet group" of horizontally flexible supports.

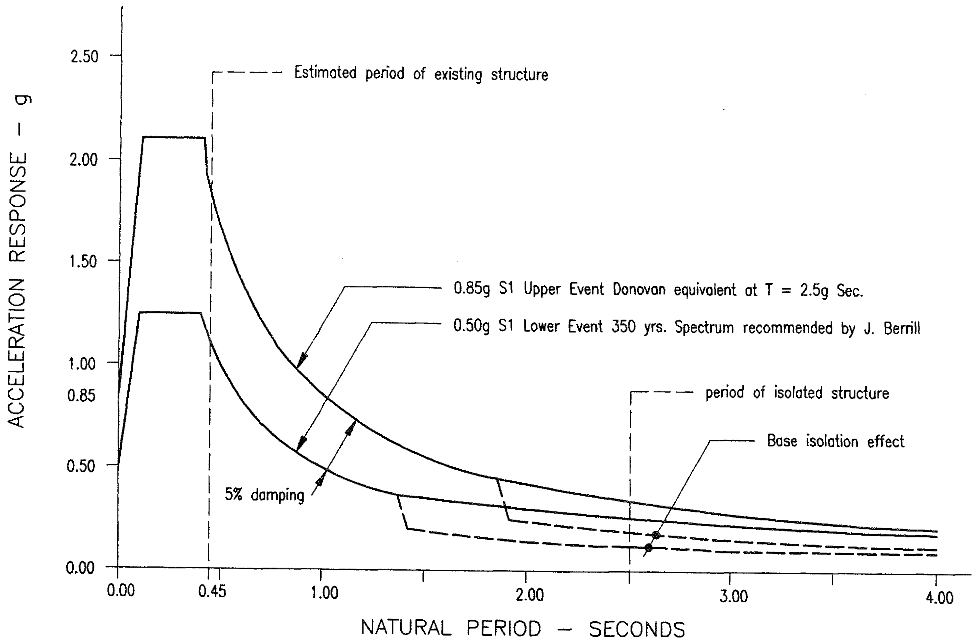

Figure 4 Composite Design Response Spectra.

c) The bearing behaviour is non-linear but for simplicity early phases of analysis assumed linear approximations of their properties.

d) This structure is torsionally sensitive. We believe the reason for this is the concentration of mass in the walls which tends to make the superstructure act like a large "flywheel". Seventy percent of the building mass is in the walls despite the concrete floors. Compare this with a modern structure where the majority of the mass is in the floor slabs. Consequently although both the superstructure and the bearings have a good disposition of elements to resist torsion, a modest eccentricity results in large driving forces and large torsional displacements. After reviewing the rationale of the 10% eccentricity allowance in NZS 4203 and noting the response of the structure is overwhelmingly dictated by the bearings, a relatively simple and known composite element, it was decided to reduce the artificial eccentricity to 5%. A brief study of the building responses using ETABS modal analyses with varying eccentricity indicated 5% eccentricity as being marginally worse than 10%. This is a result of the relative contributions of the first three modes which have very close periods at 5% eccentricity and increasing divergence with increasing eccentricity.

Method of Analysis

The analysis and design of the bearings has been a collaborative effort between Holmes Consulting Group and Trevor Kelly of Dynamic Isolation Systems Inc. of Berkeley, California.

The analysis of the isolation systems has been done in 3 stages, each increasing in order of complexity.

(1) Single Degree of Freedom Analysis:

This first stage was done by DIS to design the isolation bearings using their proprietary computer program, PC LEADER. This analysis assumes that each floor of the structure is represented by a simple column element and a single mass.

This procedure is used to design the bearings and to estimate displacements and transmitted base shear. All bearings are assumed to displace equal amounts and the effects of torsion and structure flexibility are approximated by scaling factors.

(2) Multi Degree of Freedom Analysis:

This second stage of the analysis was done by Holmes Consulting Group to determine the effects of torsion and structure flexibility on the displacements and base shears and also to obtain forces in the structure above the isolators. The computer program used was ETABS. This is a linear elastic program so the bearings were approximated as equivalent elastic elements using the "secant stiffness" for the bearing force-displacement relationship.

Hysteretic damping was accounted for by modifying the response spectrum in the long period range (beyond 2.0 secs), using the composite response spectra shown in Figure 4.

(3) Multi-Degree of Freedom Time History Analysis:

This third stage of the analysis was done by Trevor Kelly who was seconded to Holmes Consulting Group from Dynamic Isolation Systems. It is a time-history analysis.

This level of analysis incorporates the nonlinearity of the isolators directly by modelling them as yielding elements. The program used for this analysis was ANSR-II giving a full three dimensional representation of the isolators. To reduce the computational effort the superstructure was assumed to remain elastic which has proved to be reasonable. This allowed the superstructure to be modelled as a reduced "superelement" column extracted directly from the previous ETABS analysis.

As the bearings have non-linear properties, time-history analyses were run using three different input motions. This type of analysis provides more accurate displacements and forces in the individual bearings. In our analyses we found that two of the chosen input motions "Loma Prieta 1989 Corralitos x 1.38" and "Oakland City Hall 1000 year artificial" produced results lower than those obtained from the linear analysis in (2) above, while the third input option, "El Centro 1979 Array 6 x 0.8" gave higher results and became the critical analysis for bearing stability evaluation.

The El Centro 1979 Array 6 record is the strongest of the 1979 Imperial Valley accelerograms and was selected as one of our time-history records after a lot of discussion between Holmes Consulting Group and a number of Seismological Consultants. Disagreement exists as to the validity of this accelerogram due to its position near the intersection of the Imperial and Brawley Faults and directivity effects.

Value of Time-History Analyses

The use and value of time-history analyses is an interesting and important issue. The selection of the earthquake records to be used is dependent on those available and the advice of experts as to the applicability of the particular earthquake to the given situation. Our experience has been that the experts have very limited information available, apart perhaps in California, and their advice is therefore open to question by other experts. By arbitrarily selecting three earthquakes one gains some comfort that the building will behave reasonably as predicted for those three types of earthquakes. In this case our main concern was that the maximum deflection was less than that predicted by the ETABS modal analysis and hence seismic gaps were sufficient and bearings were not prone to roll-over or instability. Whilst being mindful of the non-linear behaviour of the bearings, the authors derive just as much comfort from gut engineering judgements and the statistically based ETABS modal analysis.

ANTICIPATED BUILDING BEHAVIOUR DURING EARTHQUAKES

Behaviour of Superstructure

The existing masonry walls were assessed as having an approximate shear capacity of 0.08g. The building response anticipated in the Maximum Probable Earthquake is 0.13g and in the Maximum Credible Earthquake 0.22g. It was anticipated the superstructure would yield before reaching the Maximum Credible Earthquake, the existing masonry walls would crack and lose strength and stiffness and hence should be ignored as a resisting element.

Detailed analysis of the superstructure, at an advanced stage of the design, indicated very low interstorey deflections. Further investigation indicated that due to a number of factors very little yielding of the superstructure could be anticipated. These factors included wall geometry, minimum facing wall thickness of 200 mm, minimum reinforcing and overstrength margins in the reinforcing.

Review of Facing Walls

When this excess capacity of the walls became apparent an extensive internal review was conducted to make as many savings as seemed prudent. Facing walls were removed from internal corridor walls in the west and north wings (on grids 2 and K). The corresponding wall in the east wing on grid 11 was retained in part because the east block lacks an exterior wall on grid 12 and it had been agreed to omit facing walls from the Speakers Office area in the interests of conservation. In addition the main entry lobby, south east pavilion, has no facing walls, again for conservation reasons, so walls on grids 11 and N16 directly north of this area were desirable for north-south earthquake components. Diaphragm enhancement of this entry lobby areas was also difficult.

Considerable soul searching was expended with regard to the facing walls on grids 1 and K, particularly because of conservation objectives. They were retained for two reasons. Firstly they provide a better distribution of shear walls with respect to the mass distribution and hence a more direct path for shears to the lead-rubber bearings. Secondly the more direct load path for shears reduced the demand on floor diaphragms and particularly the ground floor. Side benefits included improving the integrity of the exterior walls, improving resistance to face loads and an independent vertical means of support to the floor slabs.

Accommodating Seismic Movements

An interesting side issue in base isolated buildings is the question of movement in less severe earthquakes and the effect this may have on architectural details and services. Predictions are as follows:

Modified Mercalli Intensity	Return Period	Description of Effects on the Community and Buildings in General	Likely Effect on the base isolated Parliament Buildings	Expected movement of the base isolated Parliament House
MM 6	5 to 10 yrs	Felt by all. People and animals alarmed. Difficulty experienced in walking steadily. Some plaster cracks or falls. Isolated cases of chimney damage. Windows, glassware and crockery broken. Heavy furniture moved.		30mm
MM 7	15 to 25 yrs	General Alarm. Difficulty experienced in standing. A few instances of damage to Ordinary non seismic masonry building. Loose brickwork and tiles dislodged. Stone walls cracked. Weak chimneys broken, usually at the roofline. Domestic water tanks burst.	No damage to Parliament Buildings, either structural or non-structural.	50mm
MM 8	50 to 80 yrs	Alarm may approach panic. Ordinary non seismic masonry buildings damaged. Chimneys, factory stacks, monuments, towers and elevated tanks twisted or down. Panel walls thrown out of frame structures. Some brick veneers damaged.	No structural damage to Parliament Buildings, onset of non structural damage in details.	70mm
MM 9	220 to 350 yrs	General panic. Ordinary non seismic masonry buildings heavily damaged, sometimes collapsing completely. Frame structures racked and distorted. Damage to foundations general. Brick veneers fall and expose frames. Cracking of the ground conspicuous. Minor damage to paths and roadways.	Maximum Probable Earthquake. Parliament survives unscathed. Minor non-structural damage within.	200mm
MM10	500 to 600yrs but see the notes.	Most masonry structures destroyed, together with their foundations. Some well built wooden buildings and bridges seriously damaged. Cement and asphalt roads and pavements badly cracked or thrown into waves.	Maximum Credible Earthquake. Minor structural damage but greater non-structural damage.	450mm

In the case of Parliament House, which is a stand alone building, moat structures which accommodate seismic movements of up to 400 mm have been built in on the north and east sides of the building. The main entry steps are to be partially rebuilt to provide an horizontal sliding joint for the same movement. Apart from sealants little damage is anticipated in these moat structures or the building itself in major or minor earthquakes. Architectural details across the isolation gap can accommodate 50 mm movement. They will be prone to damage and require repair when movements are greater, events with return periods greater than 25 years.

In the case of the Parliamentary Library the issue is more complex. Because there are two separate isolated buildings a total movement of 900 mm was anticipated in the Maximum Credible Event. This has been reduced to 550 mm after the time-history analyses but nevertheless there are still significant cumulative movements. In a number of cases the only way to accommodate these movements has been to provide sacrificial material in the movement zone. This is an acceptable approach for an infrequent major event but not for more frequent events. The authors have adopted 40 years as being a reasonable period and hence have had to design flexible material joints for movements up to 110 mm expected in this time frame.

CONSTRUCTION ISSUES

Building Bearings Into Basement Walls and Columns

The major issue and challenge is of course how to introduce the bearings into the basement walls and hence how to resist the forces generated by the seismic displacement of the bearings. Basement to Ground floor is 3.53 metres. The bearing assembly is 600 mm high and there is 1200 mm structural depth above the bearings and approximately 1750 mm structural depth below the bearings.

In general existing remnant foundation walls below the bearings require enhancement to redistribute gravity loads. These walls are on the perimeter, around the main entry lobby, the main public reception lobby and the Legislative Chamber. In many cases this has been achieved by means of sandwich beams on either side of the existing foundation. This enhancement has also proved sufficient to resist seismic actions generated by bearing displacement perpendicular to the wall. This sandwich technique has also worked in most cases above the bearings and includes a crosshead beam over the bearings. The success of this system is a function of the massive size of the existing foundations. Introduction of the bearings into these walls is comparatively easy since the demolition required is limited to the crosshead beams and a horizontal cut to separate the superstructure from the foundation. Refer to Figures 5 and 6.

In the case of many of the other walls with more modest dimensions such as the House of Representatives and the lightwell walls completely new walls incorporating piers under the bearings are required to resist seismic actions associated with bearing displacement. Ground floor sandwich beams are to be used. Foundations, piers, bearings and sandwich beams are constructed followed by the stabilising walls. Refer to Figures 7 and 8.

The most difficult walls are those where sandwich beams are not acceptable, generally for aesthetic reasons. In this case the ground floor beams have to be inserted into the wall progressively and carefully with extensive temporary propping. The wall below is then replaced as noted in the previous paragraph.

For the isolated columns, capitals are to be built into the ground floor for two reasons. Firstly they facilitate temporary propping to allow the existing columns to be cut and the bearings inserted. Secondly they enhance the existing floor and beams so that seismic forces associated with bearing displacement can be resisted. The piers are enhanced where necessary for vertical loads and they are stabilised by stub walls or foundation beams. Refer to Figures 9 and 10.

Facing Walls

These are not expected to cause any major difficulties providing pouring pressures against masonry are limited. Ties are deformed bars grouted with non-shrink cementitious grout.

Steel Diaphragms at First and Second Floors

On these floors it was not feasible to replace the existing false floor with a topping because the space was required for sprinkler pipes and various cables. Hence diaphragm enhancement is by means of a 5 mm steel plate laid over the existing concrete slab. There are a number of challenging details where the plates pass under lengths of existing brick walls and connections have to be made through brick walls to facing walls or concrete toppings.

Ground Floor Enhancement and Modification

Enhancements and modifications to the ground floor include: sandwich beams at walls, column capitals, flanking on existing beams and new beams. A new 75 mm topping over existing floors is integrated with the toppings of the new floor areas. Diaphragm stresses are greatest at this level because shears are redistributed to the bearings. A topping is feasible at this floor because services can be run beneath the floor slab.

External Moat and Stairs

Where there is external structure or adjoining ground above the bearing slot a moat with a sliding cover plate is necessary to avoid the displacing building banging into the adjoining structure or ground in an earthquake. The main entry stairs require some reconstruction to build in an horizontal joint to facilitate this movement.

CONCLUSIONS

The engineering objectives with this building are to upgrade it seismically to a level appropriate to a national monument. This level will provide ample protection for the occupants of the building. The second objective is that of conservation with the emphasis on the public spaces. It should be noted that the building also has a vital function to serve the nation as a working building and thus the conservation aspects are inevitably compromised.

The seismic forces assessed for the site for a 350 year returnperiod event using the uniform risk hazard analysis is high by current code standards. An event of MM IX intensity is predicted and this involves a maximum ground acceleration of 0.5g.

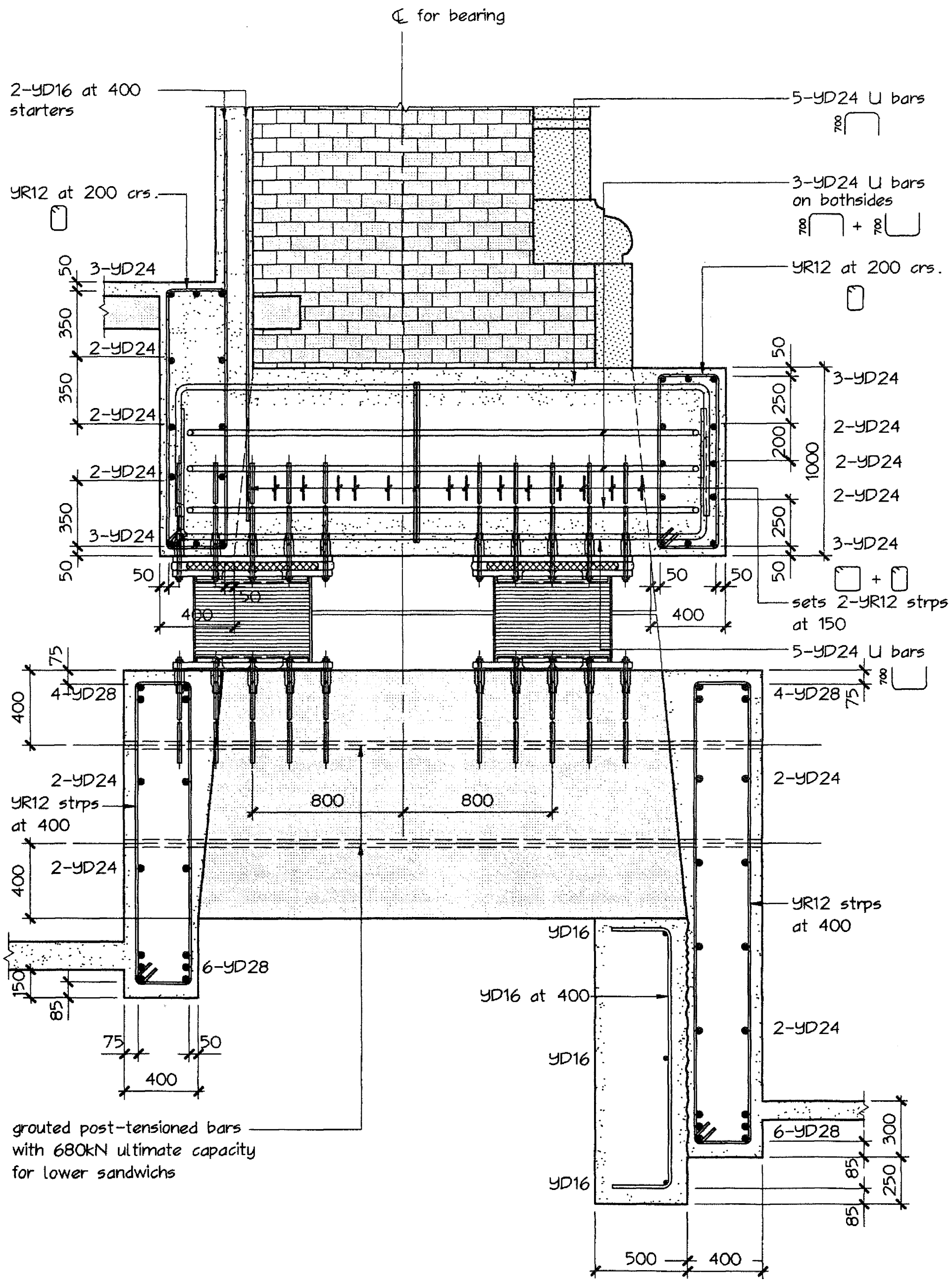

Figure 5 Section through exterior north wall at bearings.

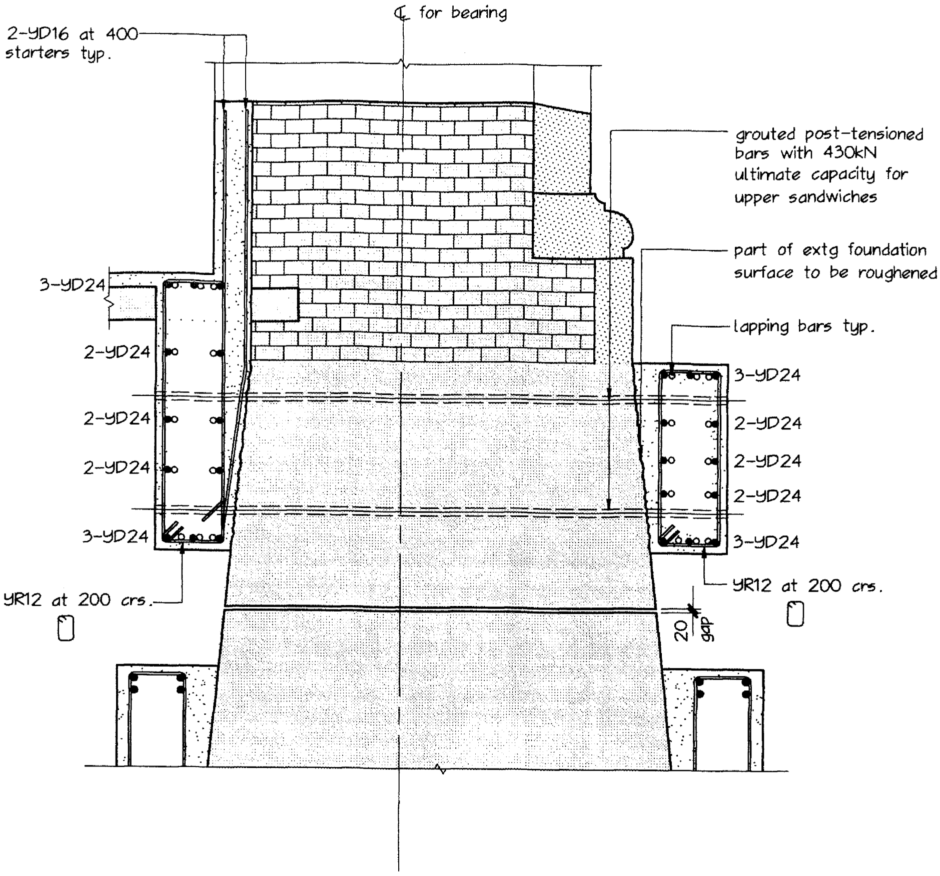

Figure 6 Section through exterior north wall between bearings.

The Wellington Fault, Class I, is 500 metres to the west of the site and poses a specific risk in the event of rupture on the fault. An event of MM X intensity is predicted with a maximum ground acceleration of 0.85g. The likelihood of this event occurring during the 150 year design life of the upgraded building is between 10% and 50%, too high a risk to be ignored.

Because of its inherent stiffness this building is not well equipped to resist these events. A conventional strengthening by means of shear walls formed by facing existing masonry walls provides strength but because of its geometry is not inherently ductile. The introduction of base isolators into the basement of the existing building dramatically reduced the seismic loads attracted to the building. Whilst not eliminating strengthening of the existing building it has provided a very high level of protection, both to the building and its occupants, against very high seismic forces.

The availability of the basement for locating lead-rubber bearings was a basic advantage facilitating the scheme. The basement is generally low priority space with little occupancy. In addition the enormous size of many existing foundations minimised strengthening requirements.

The new structures in the lightwells and on the south side of the building are enormously helpful as locations of shear walls and also provide reliable diaphragms to which existing floors are connected.

The use of concrete facing walls has proved relatively simple and effective in enhancing the shear capacity of the building. The large majority are located in conservationally inoffensive locations.

Existing concrete floors have been enhanced as diaphragms using both concrete toppings where space permits and steel plates where space is at a premium. These systems provide a reliable enhancement to existing diaphragms.

The scheme achieves the conservation objectives totally in the main public spaces, viz Entry Lobby, House of Representatives, Legislative Chamber and the main corridors. The Public Lobby has two facing walls and the perimeter walls of the west and north wings are also faced. These facings have raised some conservation objections. However, planning requirement also effect these walls. In general the conservation objectives are achieved to a high degree.

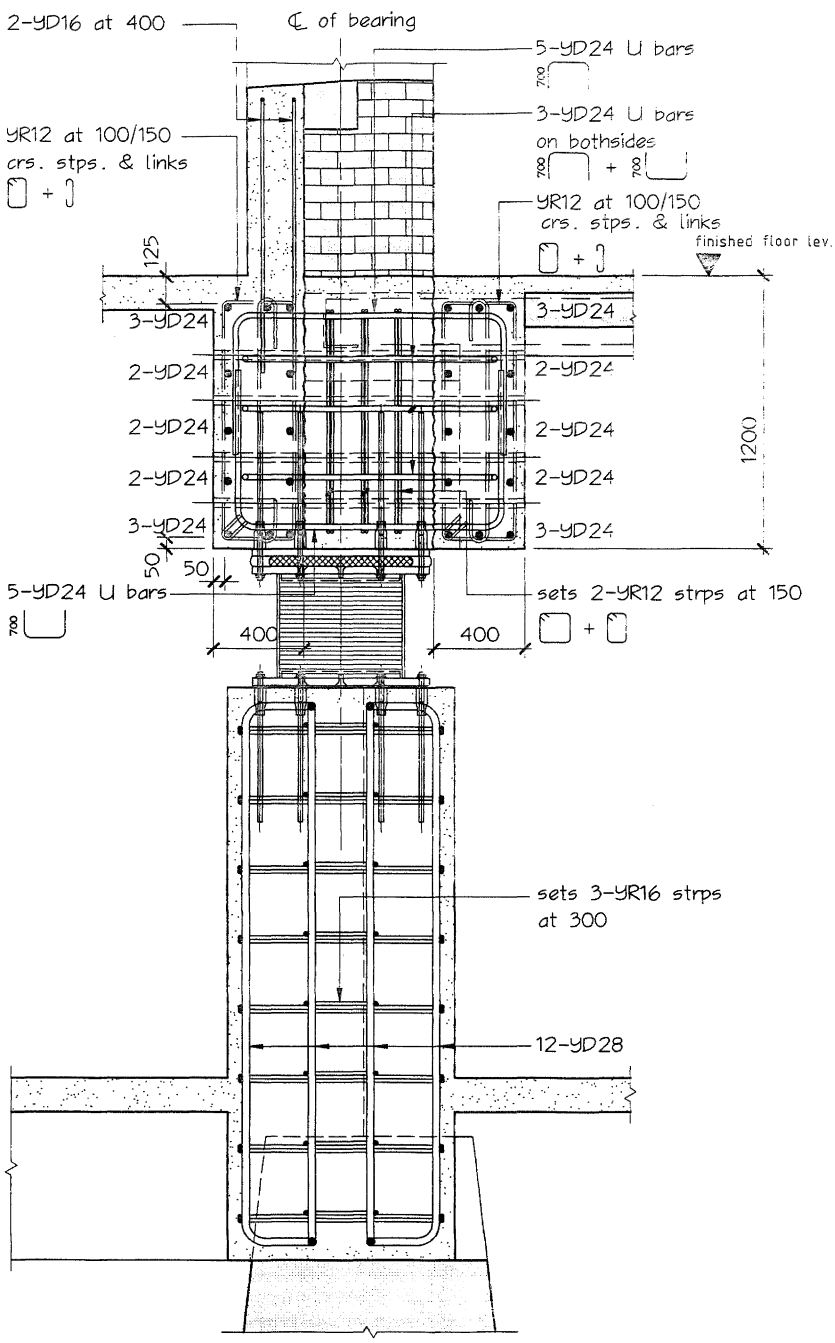

Figure 7 Section through light-well wall at bearing location.

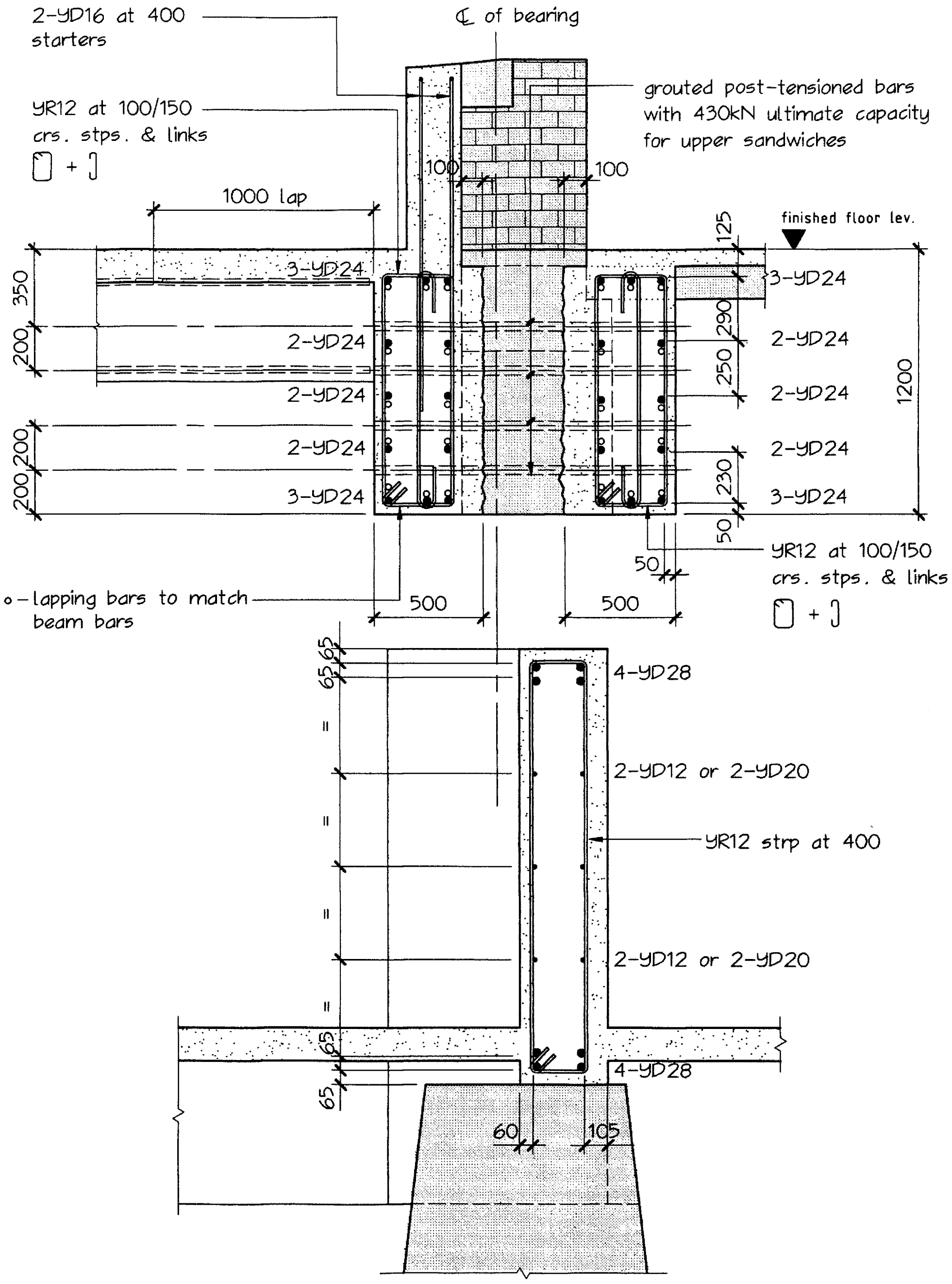

Figure 8 Section through light-well wall between bearings.

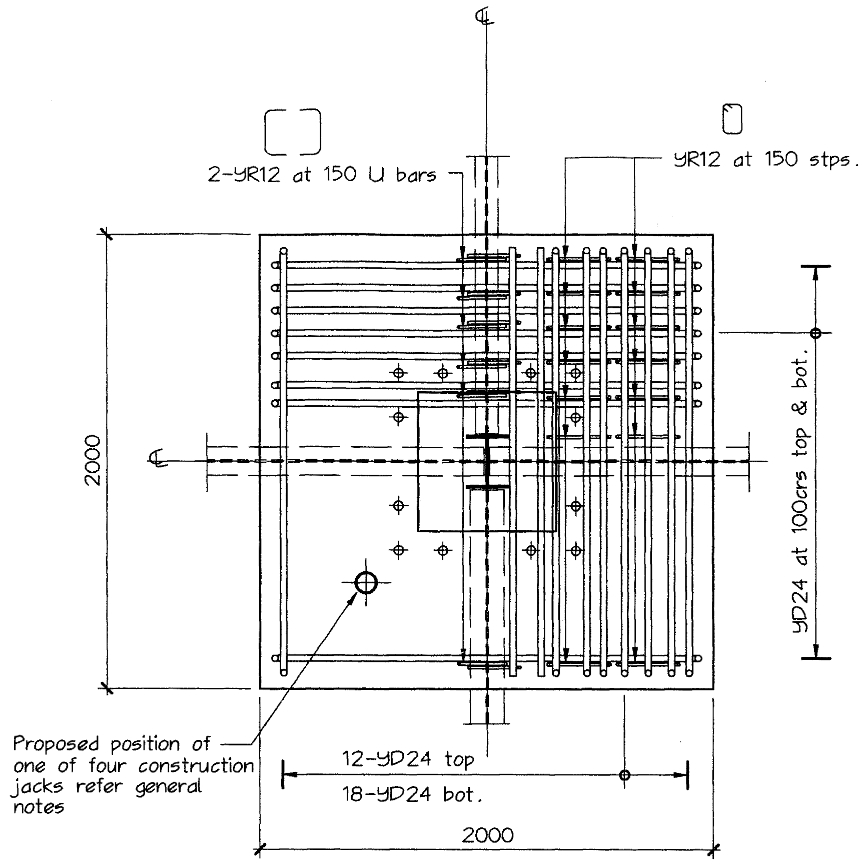

Figure 9 Plan of column capitals at grids.

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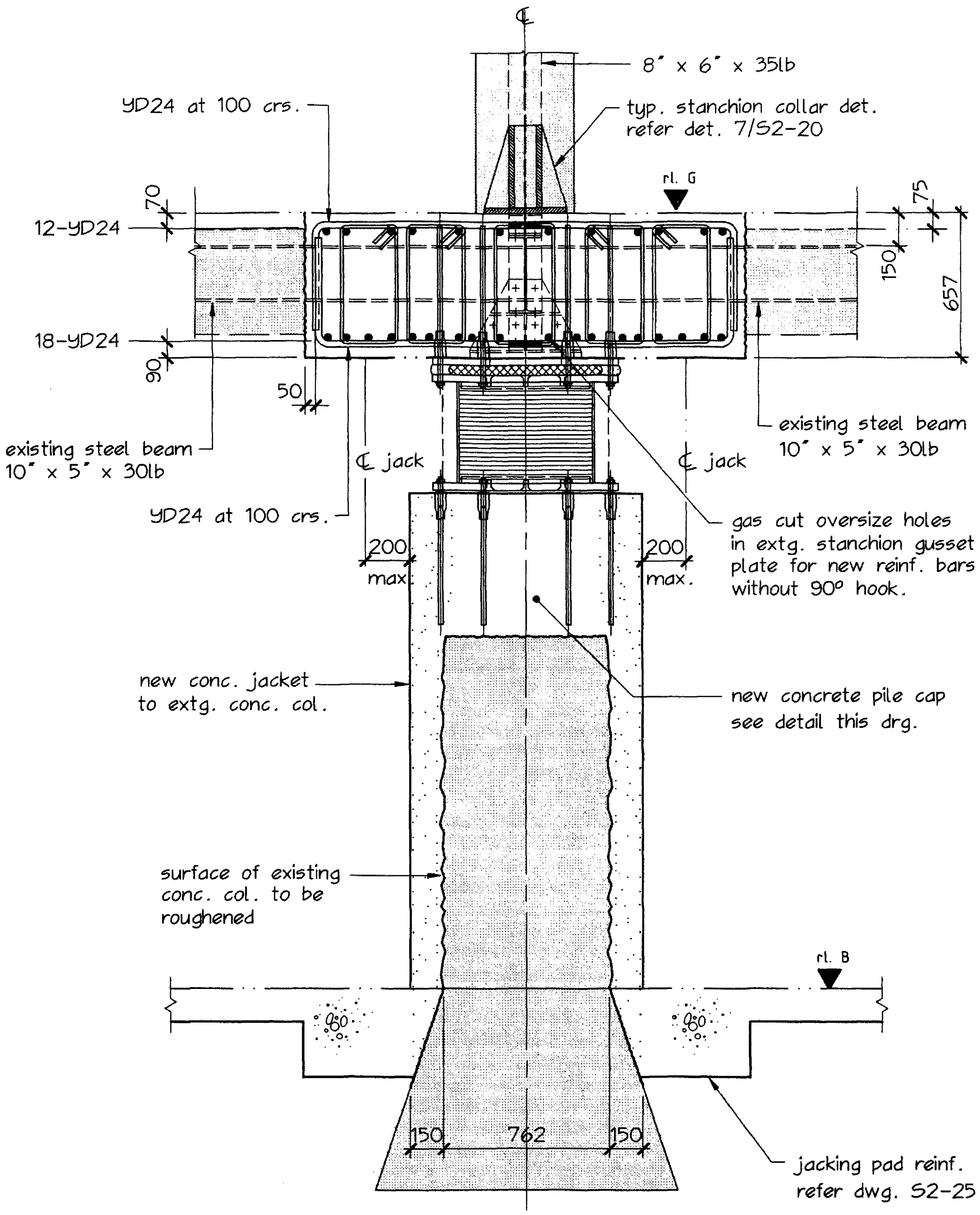

Figure 10 Section through column capital.