WINDSOR FLOOD EVACUATION ROUTE

SOUTH CREEK

Environmental Impact Statement

Volume 2

Working papers: part one – chapters 1-6

AUGUST 2007

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New South Wales
Government Initiative

ENVIRONMENTAL IMPACT STATEMENT
VOLUME 2 WORKING PAPERS

Part 1

Working Paper 1 – Route Evaluation and Selection Report

Working Paper 2 – Air Quality Assessment

Working Paper 3 – Hydraulic Assessment

Working Paper 4 –South Creek Crossing Wave Analysis

Working Paper 5 – Investigation into Joint Occurrence of High Floods and Wind Waves

Working Paper 6 – Noise and Vibration Assessment

Part 2

Working Paper 7 – Flora and Fauna Assessment

Working Paper 8 – Traffic, Transport and Economic Analysis

Working Paper 9 – Non-Indigenous Heritage Assessment

Working Paper 10 – Indigenous Heritage Assessment

Working Paper 11 – Emergency Risk Management for Hawkesbury-Nepean Flooding

Working Paper 12- Urban and Landscape Design Report

Working Paper 1 – Route Evaluation and Selection Report

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Windsor Flood Evacuation Route Route Options Development and Evaluation Report

Reference 10453E/GE

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1. Introduction

1.1 Project Background

This report has been prepared by Connell Wagner for the NSW Roads and Traffic Authority (RTA) to document the route option development and evaluation process that was undertaken for the proposed Windsor flood evacuation route.

The need for the proposed Windsor flood evacuation route has arisen from the NSW Government's commitment to managing the potential effects of flooding on the Hawkesbury-Nepean floodplain and the subsequent establishment of the Hawkesbury-Nepean Flood Management Advisory Committee in 1997. Following extensive consultation with community and Government stakeholders, the Committee produced the Hawkesbury Nepean Floodplain Management Strategy (HNFMS) which in 1998 was adopted by the NSW Government. The Minister for Land and Water Conservation has overall responsibility for the implementation of the Strategy, on behalf of the NSW Government and a Steering Committee. The Steering Committee comprises representatives of DLWC (chair), State Emergency Service (SES), RTA, Department of Community Services (DoCS), the then Department of Urban Affairs and Planning (DUAP) (now Planning NSW), Sydney Catchment Authority (SCA), Western Sydney Regional Organisation of Councils (WSROC) and the community, and is overseeing the implementation of the project.

One of the recommendations of the HNFMS is the need to upgrade essential evacuation routes to help protect the lives of residents in the urbanised areas of the Hawkesbury-Nepean valley, which are subject to potentially very deep and hazardous flooding. Windsor, South Windsor, Bligh Park, Richmond, McGrath's Hill, Pitt Town and Emu Plains have additional risks from early isolation by rapidly rising floodwaters and complete inundation in severe flood events.

A Memorandum of Understanding (MOU) sets out the arrangement between DLWC and the RTA for a strategic partnership for the provision of services to achieve the improved road infrastructure in accordance with the HNFMS. DLWC is responsible for funds to implement the overall Strategy and the RTA is responsible for project management, environmental assessment, design, acquisition and construction components of the Strategy.

To coordinate and effect the implementation of the evacuation routes, a Working Group has been established. The 'Interim Regional Road Upgrade Plan' (April, 2000) prepared by this Working Group, identified the need for a high-level crossing of South Creek to be developed as a key component of the flood evacuation plan for Windsor.

The NSW Roads and Traffic Authority engaged Connell Wagner Pty Ltd to investigate route options and identify a preferred route option for the high-level crossing of South Creek. The preferred route option is the subject of an environmental impact statement (EIS) (Connell Wagner, 2001).

1.2 Route Development and Evaluation Methodology

The following activities were undertaken in the development and evaluation of route options for the proposed Windsor flood evacuation route:

- **Identification of flood contours:**

The first step in the development of route options was the identification of areas of 'high' land which would allow a route to be developed linking Windsor to land that would allow movement outside of the flood affected area.

- **Planning Constraints Analysis:**

The planning constraints analysis identified areas of environmental sensitivity in the study area with particular focus on the following issues:

- wetland areas including those areas listed under SREP 20;
- areas of ecological sensitivity including locations of known threatened species and ecological communities;
- items of indigenous and non-indigenous heritage significance;
- property effects;
- connections to surrounding road network; and
- riverine areas of scenic significance as defined in SREP 20.

Consideration of these factors led to the development of a range of preliminary route options in the study area.

- **Development and Assessment of Preliminary Route Options**

Based on the results of the planing constraints analysis, six preliminary route options were developed for further consideration (refer Figure 1). Options considered included four routes to the north of the Blacktown to Richmond railway line that would provide flood relief as well as an arterial road function and two routes to the south of the railway line that would provide flood relief as well as an alternative route for local traffic. These options were subject to preliminary assessment by the study team and were examined in further detail at a Value Management Study (VMS) workshop.

- **Development and Assessment of Shortlisted Options**

Two shortlisted options were identified based on the outcomes of the first Value Management Study and the results of stakeholder consultation. These two options (one to the north and one to the south of the railway line) were assessed by the project team in terms of the following issues:

- noise and vibration;
- traffic and transport;
- flood evacuation function;
- visual amenity and landscape;
- air quality;
- economic performance;
- flora and fauna;
- indigenous and non-indigenous heritage; and
- social and business operations.

A second Value Management Study was held and further consultation with stakeholders undertaken.

The preferred route option for the Windsor flood evacuation route was announced by the Minister for Roads in October 2001. Ongoing refinement of the preferred option was undertaken in response to the results of additional environmental investigations and stakeholder consultation. The preferred option is the subject of an EIS that has been prepared by Connell Wagner (Connell Wagner, 2001).

1.3 Structure of the Report

The Route Options Development and Evaluation Report has been structured as follows:

- Section 1 provides an introduction to the study purpose and methodology;
- Section 2 provides a discussion of the development of the preliminary corridor options;
- Section 3 provides a discussion of the findings of the initial assessment of corridor options;
- Section 4 provides a discussion of the process and outcomes of the first Value Management Study;
- Section 5 provides a discussion of the findings of the assessment of the shortlisted options;

- Section 6 provides a discussion of the process and outcomes of the second Value Management Study;
- Section 7 documents the project cost estimates and findings of the economic analysis; and
- Section 8 documents the conclusions of the study.

2. Identification of Corridor Options

The Roads and Traffic Authority (RTA) had previously undertaken preliminary environmental overview and engineering investigations to identify potential corridors for a flood evacuation route across the South Creek floodplain. The study team reviewed these reports and further investigated the study area, bound by the 17.3 metre (AHD) contour line, for constraints and opportunities. The 17.3m AHD contour was selected as the boundary of the study area as this represented the most practical level.

Six preliminary options were developed during the initial stage of the investigation to enable preliminary assessment of design issues, environmental impacts, community concerns and construction costs. Options considered included four routes north of the railway line that would provide flood evacuation as well as an arterial road function and two routes south of the railway line that would provide a route for flood evacuation and an alternative route for local traffic. Figure 2.1 shows the route options that were considered. A description of the options is provided below. For each of the options a description of how traffic would utilise the option for flood evacuation purposes and daily traffic purposes has been provided.

Route A: George Street to Windsor Road

- Flood evacuation route – along Windsor Road between George Street and Groves Avenue then along Groves Avenue, Railway Road North and a new level crossing to Railway Road South
- Daily traffic – Windsor Road

Route B: Richmond Road to Curtis Road

- Flood evacuation route – Day Street, Windsor to Railway Road North, Mulgrave and then Railway Road South via a new level crossing near Groves Avenue
- Daily traffic – Macquarie Street/Richmond Road, Windsor to Curtis Road, Mulgrave

Route C: Forbes Street to Windsor Road

- Flood evacuation route – Day Street, Windsor to Railway Road North and then Railway Road South, Mulgrave via new level crossing near Groves Avenue
- Daily traffic – Forbes Street, Windsor to Railway Road North and a deviation to Windsor Road, Vineyard

Route D: Day Street to Railway Road North

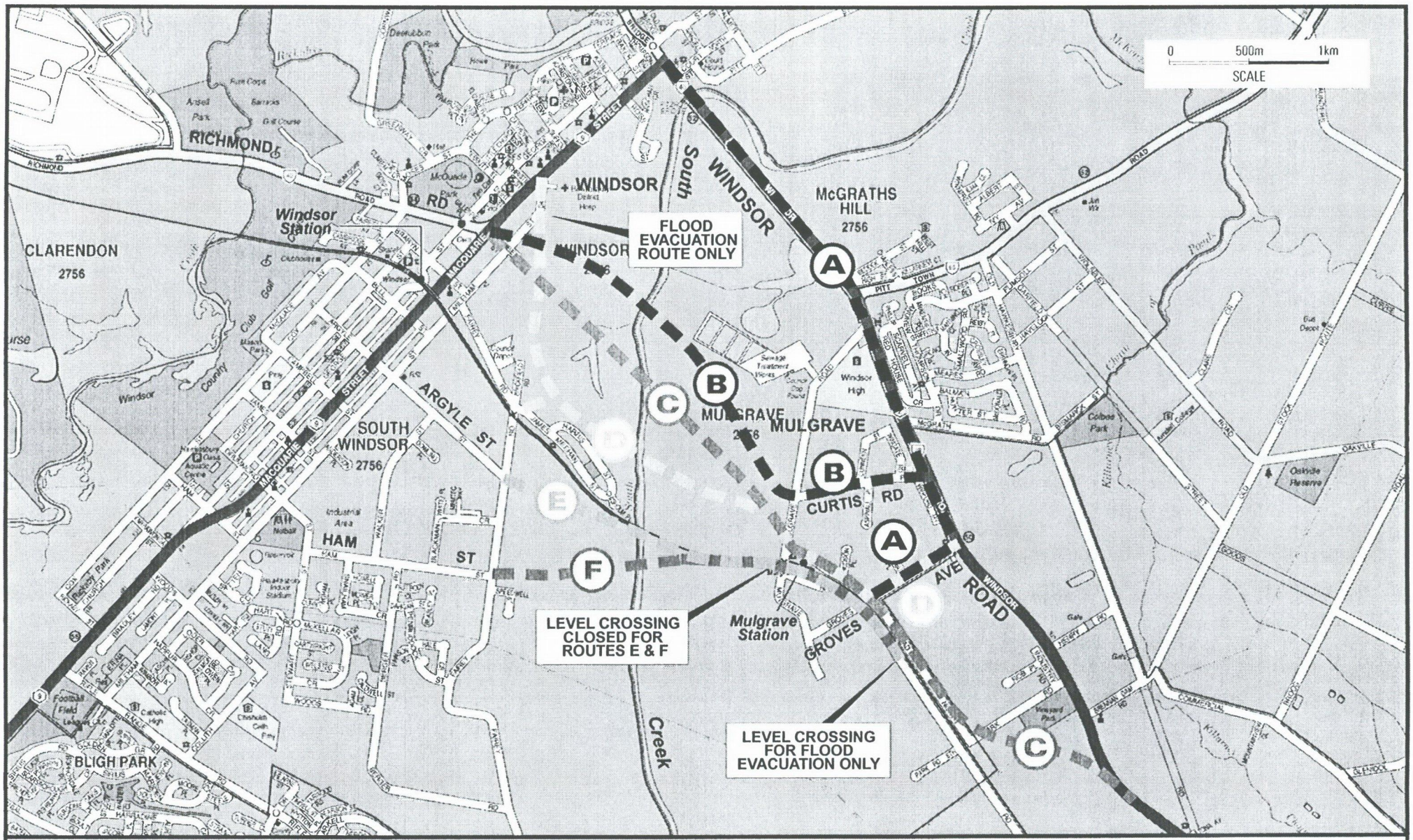
- Flood evacuation route – Day Street, Windsor to Railway Road North and then Railway Road South, Mulgrave via new level crossing near Groves Avenue
- Daily traffic – Forbes Street, Windsor to Railway Road North and Groves Avenue, Mulgrave

Route E: Argyle Street to Railway Road South

- Flood evacuation route – Argyle Street, South Windsor to Railway Road South at Mulgrave
- Daily traffic - Argyle Street at South Windsor to Railway Road South at Mulgrave

Route F: Ham Street to Railway Road South

- Flood evacuation route – Ham Street at South Windsor to Railway Road South at Mulgrave
- Daily traffic - Ham Street at South Windsor to Railway Road South at Mulgrave



WINDSOR
FLOOD EVACUATION ROUTE

FIGURE 2.1
LONG LISTED ROUTE OPTIONS

3. Preliminary Assessment of Options

3.1 Environmental and Engineering Assessment

The six options were subject to initial appraisal by the project team in relation to social and amenity impacts, visual and land use impacts, ecological constraints, traffic and engineering issues. The advantages and disadvantages of the route options are listed in Table 3.1.

Table 3.1 Comparison of Preliminary Route Options

ADVANTAGES	DISADVANTAGES
Route A	
• Utilises existing road reserve and therefore impacts confined to incremental effects in the vicinity of the existing corridor. • Avoids SREP 20 wetlands	• Unsatisfactory traffic performance through Windsor • Statutory prohibition through Environmental Protection Zone 7(a) • Property impacts at McGrath's Hill • Difficult to construct with traffic on Windsor Road • Business impacts along Windsor Rd, Mulgrave • Longest flood evacuation route • Higher cost option than 'flood evacuation only' routes • Impacts on number of heritage items in close proximity to Windsor Road
Route B	
• Avoids SREP20 Wetland • Provides arterial road network enhancement	• Property impacts between Day Street and Macquarie Street • Property access impacts along Curtis Road • Acquisition required along Railway Road North • Higher cost option than 'flood evacuation only' routes
Route C	
• Traffic benefits of extension to Windsor Road • Provides arterial road network enhancement	• Property impacts along Forbes Street • Crosses SREP20 Wetland • Acquisition required along Railway Road North • Impacts of extension to Windsor Road (eg vegetation, property) • Highest cost of all options
Route D	
• Improved access via Fairey Road to South Windsor industrial area • Provides arterial road network enhancement	• Property impacts along Forbes Street • Crosses SREP20 Wetland • Acquisition required along Railway Road North • High property & visual impacts, Harris Street • Increased traffic on Fairey Road level crossing • Higher cost than 'flood evacuation only' routes
Route E	
• Improves access via Fairey Road to South Windsor industrial area • Lower construction cost • Least visual impact on flood plain • Avoids SREP20 Wetlands	• Access and property impacts for properties on Railway Road South • Increased traffic on railway crossing at Level Crossing Road • Increased traffic along Argyle Street • Increased traffic on Fairey Road level crossing

Route F	
• Improves access via Fairey Road to South Windsor industrial area • Lower construction cost • Low visual impact on flood plain • Avoids SREP20 Wetlands	• Access and property impacts for properties along Railway Road South • Increased traffic on the railway crossing at Level Crossing Road • Increased traffic along Ham Street and fronting residential properties

3.2 Community Response

The options were presented in a March 2000 newsletter that was broadly distributed to residents and also displayed at three locations in the project area. Have Your Say Sheet Number 1 was distributed with Newsletter No. 1 in March 2000. Have Your Say Sheet No. 1 asked respondents to nominate a preferred route (from the six options presented in the newsletter) for flood evacuation purposes and for use by daily traffic, and to make any other comments about the proposal.

There was a large response to the first newsletter with 207 sheets returned. Several of these were sent in twice, and some were in the form of a submission. There were 160 surveys returned that were suitable for analysis. The preferred options nominated are detailed in Table 3.2.

Table 3.2: Preferred Routes from Have Your Say Sheet 1

OPTION[1]	FLOOD EVACUATION (%)[2]	TRAFFIC RELIEF (%)
Route A	3	3
Route B	28	33
Route C	36	40
Route D	12	10
Route E	12	9
Route F	19	19

Note:
1. Refer Figure 3.1 for route options.
2. Percentages do not total 100%, as several people preferred more than one route.

There was a general feeling amongst the community that the priority should be the improvement of the performance of the arterial road network. Particular reference was made in numerous submissions to the congestion that occurs on Windsor Road in the vicinity of McGraths Hill. The community indicated less interest in the flood evacuation function of the proposal. It is unclear whether this was due to an assumption that either route would provide for flood evacuation, a lack of knowledge about the potential effects of flooding in the area and the associated need for a flood evacuation route or a focus by respondents on the immediate issues associated with traffic congestion rather than the future possibility of a major flood event.

In terms of the six options presented in Newsletter No. 1, the majority of respondents indicated a preference for options to the north of the railway. Increased traffic relief on the arterial road network was the most common reason cited for this preference. The northern options, which involved improved performance on the arterial road network, were generally seen to provide a higher level of community and economic benefits due to the dual function of providing for flood evacuation and traffic relief.

The majority of respondents supported Option C both in terms of its flood evacuation function and provision for daily traffic movements. This option was generally preferred because of the extension to Windsor Road in the south. Many of the residents that stated a preference for either Option B or D also indicated that these options would be improved if they included this extension as it would have

additional traffic benefits. These residents raised other advantages of these routes including use of the existing road network and minimal environmental impacts of Option B, and the benefits of access to Fairey Road with Option D. This additional access point would benefit residents living in South Windsor and Bligh Park as well the industrial area near Argyle Street. Since the extension of Option C was removed from the scope of this project it can be assumed that a combination of Options B and C were the preferred routes for flood evacuation and traffic improvements.

The southern options, Options E and F, received a lower level of support from the responses to Have Your Say Sheet No. 1. However, those respondents who did indicate support for Options E or F cited reasons including lower environmental impacts particularly with respect to the effects on wetlands, fewer property impacts, better access to the South Windsor residential area and fewer impacts on the traffic performance of the road network in the Windsor town centre.

In addition to nominating preferred route options, respondents made comments in relation to a number of other issues. One of the most common issues that was raised included the need to provide four lanes on the flood evacuation route to provide adequate traffic relief on the road network. Respondents also stated that the project should be considered in the context of the need to upgrade Windsor Road along its whole length. The suitability of allowing future urban development on the Hawkesbury-Nepean floodplain was also raised as an issue by several respondents. These issues were noted and are considered in the EIS.

4. Value Management Study No. 1

4.1 Introduction

The process used to assist in the selection of the two preferred routes (one to meet flood evacuation criteria and the other to meet flood evacuation and arterial road criteria) was through a Value Management Study (VMS). This is a structured process designed to establishing a common understanding of project objectives, design problems and user requirements. It involves a group of people that have a diverse range of interests and expertise and allows a range of information to be evaluated and compared.

The first Value Management Workshop (VMS 1) was held on 30 March 2000 to undertake an evaluation of the route options based on project objectives and assessment criteria. Key stakeholders included representatives of the community and representatives of the Department of Land and Water Conservation, Roads and Traffic Authority, Hawkesbury City Council, Hawkesbury River Environment Protection Society, State Emergency Service and Connell Wagner. The workshop involved identifying selection criteria, deciding on the relative importance of the criteria, and evaluating the advantages and disadvantages of the route options.

4.2 Selection Criteria

To assist with the process of option selection the participants discussed the criteria that should be used. It was considered appropriate to develop a separate set of criteria weightings for the "flood evacuation and arterial route" and the "flood evacuation only" options. The criteria were weighted by applying a value of 10 to the criterion considered to be most important with the others then weighted relative to this. The adopted criteria weightings are shown in Table 4.1 below.

Table 4.1 Selection Criteria and Weighting used in VMS 1

Selection Criteria	Weighting	
	Flood Evacuation	Flood Evacuation Route and Arterial Road
Effectiveness for flood evacuation	10	10
Impact on traffic flow	8	10
Environmental impacts	8	9
Social and amenity impacts	6	6
Business and land use impacts	6	6
Visual impacts	4	4

Participants at VMS 1 agreed that project cost and economic returns are relevant selection criteria and acknowledged that separate assessment would be completed to supplement this multi-criteria evaluation. This issue is addressed in Working Paper 8.

4.3 Assessment of Options

Assessment of the options was conducted by examining different segments of the routes – the western or Windsor approaches, the floodplain crossing section and the eastern or Mulgrave approaches. The group listed the advantages and disadvantages of the various options for each of the three segments. Refer to the full Value Management Study 1 Report for details.

Having worked through the 'comparative' assessment process, the participants agreed on the following recommendations:

- That Option F be the preferred flood evacuation route
- That Option A, Option B and Option D not be considered further.

- That the eastern end of Option C (east of Groves Avenue linking to Windsor Road) be excluded from the current project, but recognised that it may need further consideration in the future.
- That the point of connection at Macquarie Street/Richmond Road in relation to the combined Option B & C be further considered in respect of heritage and property effects in particular. This was to include investigations to determine whether it is more appropriate for the route to go straight down Forbes St or to connect at Macquarie Street to the north of Forbes Street (ie. the new road would pass to the rear of the heritage properties).
- In terms of an assessment of those options with the primary function of a flood evacuation route (ie. Option E and Option F), Option F (Ham Street to Railway Road South) was the preferred route as it resulted in less severance of the industrial park in the vicinity of Argyle Street than Option E. This option also resulted in less property acquisition than Potion E and it would not necessitate upgrading of low spots on the surrounding road network.
- The option of upgrading Windsor Road (Option A) was deleted from further consideration as it failed to meet the project objectives of providing for an effective flood evacuation route due to its low level, difficulties of raising the level because of existing development and proximity of heritage buildings to the existing road.
- That the next stage of the study consider a redefined route across the flood plain that optimises the better features involved in Options B & C (not Curtis Road).

Based on the above, it was concluded that the next stage of the study should consider two options namely, Option F (to the south of the rail line) and a hybrid of Options B and C (to the north of the rail line).

5. Assessment of Shortlisted Options

5.1 Introduction

On 28 April 2000 the Minister for Roads and Transport announced that two routes had been selected for more detailed investigations for the proposed Windsor Flood Evacuation Route over the South Creek flood plain. These were:

- **Option 1 (North of Railway)** – flood evacuation route / arterial road between Day Street Windsor and Railway Road North, Mulgrave for flood evacuation purposes and Macquarie Street/ Richmond Road, Windsor and Groves Avenue, Mulgrave for arterial road function; and
- **Option 2 (South of Railway)** - flood evacuation route providing for local traffic between Ham Street, Windsor and Railway Road (South) in Mulgrave.

The two shortlisted route options are shown on Figure 5.1.

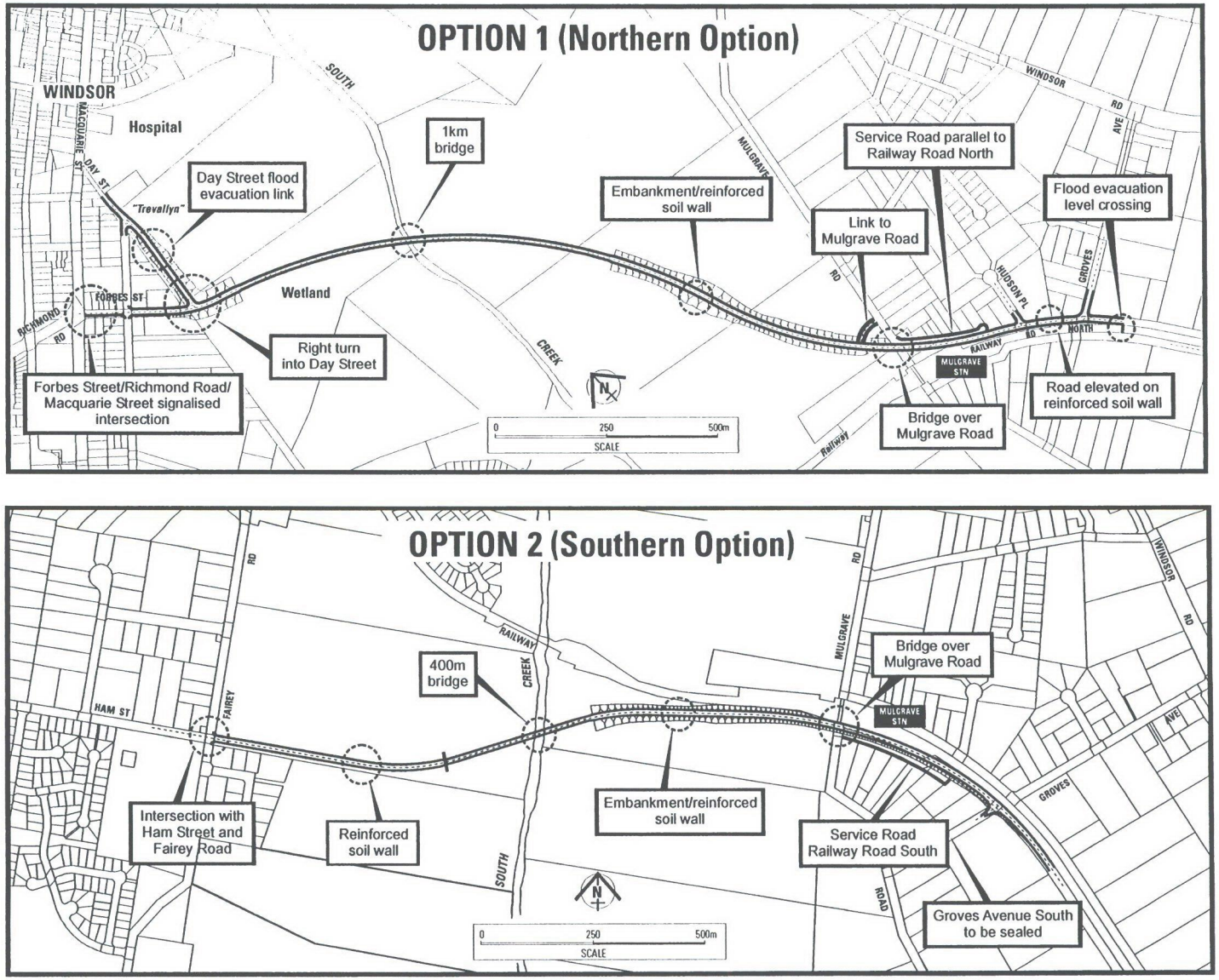
5.2 Investigation of Shortlisted Options

Further investigation of the two sub-options at the western connection of Option 1 with Macquarie Street/Richmond Road resulted in a decision to select Forbes Street as part of the alignment. This was based mainly on the significant adverse impact on heritage properties of the other (northern) sub-option.

During the period May–October 2000, the two route options were assessed in detail through various technical and environmental investigations and through further consultation with government authorities and the community. Specialist studies have included assessment of:

- flora and fauna;
- business impacts;
- non-indigenous and indigenous heritage;
- noise and vibration;
- hydrological and hydraulic effects, including wind-wave impacts;
- property impacts;
- traffic and transport;
- air quality;
- landscape and visual assessment;
- geotechnical.

Table 5.1 provides a summary assessment of the options in relation to these issues. Further details are provided in the specialist working papers.



WINDSOR
FLOOD EVACUATION ROUTE

FIGURE 5.1
SHORT LISTED ROUTE OPTIONS

Table 5.1 Summary Assessment of Route Options

FACTOR	OPTION 1 – NORTHERN OPTION	OPTION 2 – SOUTHERN OPTION
Effectiveness for Flood Evacuation	• Satisfies flood evacuation objectives	• Satisfies flood evacuation objectives
Impact on Traffic Flow	• Provides direct traffic route from Windsor Road to Richmond Road /Bells Line of Road for everyday use, effectively bypassing Windsor • Would attract 40% to 50% of traffic currently on Windsor Road between Windsor and McGrath's Hill. Congestion on the section of Windsor Road across the floodplain would be substantially reduced. • Provides level of service and operational performance improvements to Windsor Road / Pitt Town Road intersection and all signalised intersections on Macquarie Street, east of Day Street • Would serve as new arterial route providing one traffic lane in each direction with shoulders for cyclists and breakdowns. • New or reconfigured intersections at Forbes Street / Macquarie Street, connection at Day Street and Windsor Road / Groves Avenue (traffic signals) • Some potential conflict on Groves Avenue and Railway Road North with arterial road traffic and light industrial and commercial activity • Priority intersection control required at relocated Mulgrave Road and at Mileham Street/Forbes Street.	• Provides less improvement to everyday arterial road network performance due to no direct connection with the arterial road system. • Would attract approximately 30 to 35% of traffic currently on Windsor Road between Windsor and McGraths Hill. • Proposal would provide one traffic lane in each direction with shoulders for cyclists and breakdowns. • Traffic impacts on local road network with potential intersection control at Fairey Rd, Stewart St, Mileham Street., Drummond St. • Would provide improved access to South Windsor through industrial area • Potential for undesirable traffic movements from Macquarie Street to Windsor Road through Railway Road South and Level Crossing Road • Traffic signals required at Windsor Road and Level Crossing Road intersection • RIC may require a grade separated crossing of the railway line at Level Crossing Road if predicted traffic volumes indicate it (outside the study area)
Environmental impacts *Ecological*	• No threatened species listed on the TSC Act recorded • Loss of approx 0.3ha alluvial woodland at South Creek crossing, however it is in a highly disturbed form • Loss of approx 0.5ha freshwater wetlands/farm dams. This could be replaced with alternative storage to suit the owners needs. • Loss of approximately 0.5ha of Shale/Gravel Transition Forest. • No wetlands under SREP 20 are traversed by the proposal.	• Loss of approx 0.13ha of alluvial woodland at South Creek crossing, however it is in a highly disturbed form. • Loss of approx 0.06ha freshwater wetlands/farm dams. This could be replaced with alternative storage to suit the owners needs. • Cumberland Land Snail (dead specimens) recorded adjacent to study area (cnr Fairey Road and Ham Street) • *Grevillea juniperina* listed on the TSC Act as vulnerable recorded along Railway Road South in the Shale/Gravel Transition Forest (approx 12 individuals)
Heritage	• No Aboriginal sites identified along this route • Low archaeological sensitivity at creek banks and across floodplain • Crosses four potential archaeological deposits (PADs) • Crosses PAD 4, which is located south of the Hawkesbury Hospital site and east of the industrial site on Mileham Street. The potential of this site is reduced as most of the area has been developed. • Skirts the edge of PAD 5 to the west of Mulgrave Road, however the potential for an archaeological deposit is reduced by the presence of the mushroom composting business	• No Aboriginal sites identified along this route • Low archaeological sensitivity at creek banks and across floodplain • Crosses two potential archaeological deposits (PADs) • Crosses PAD 1, an area of low level previous land use impact west of South Creek. Possible stone manufacture and camping site. • Crosses PAD 2 located on elevated spur south of railway line. Likely to contain sub surface stone artefacts, however part of this area has been disturbed by a market garden. • The layout of Mulgrave Station Precinct will be affected. Lost opportunity to

Table 5.1 Summary Assessment of Route Options

FACTOR	OPTION 1 – NORTHERN OPTION	OPTION 2 – SOUTHERN OPTION
	• Small affect on PAD 6 on the west bank of South Creek • Small affect on PAD 7 on the east side of South Creek • Impact on 'Trevallyn' curtilage on Day Street (Listed on Register of the National Estate and Hawkesbury LEP 1989). • Loss of the original corner store which forms part of the Mulgrave Station Precinct • The layout of Mulgrave Station Precinct will be affected however there is an opportunity to retain and conserve the southern, eastern and western parts of the Precinct.	retain or conserve the southern, eastern and western parts of the Precinct.
Noise Impacts	• Increased noise levels near Forbes Street, would require assessment of practical and cost effective noise mitigation in consultation with affected residents • Properties along Railway Road North would be noise affected and would require assessment of practical and cost effective noise mitigation in consultation with affected residents • Near Mulgrave Railway Station and adjacent to Railway Road South a number of double-storey dwellings would experience increased noise levels and would require assessment of practical and cost effective noise mitigation in consultation with affected residents	• Increased noise levels adjacent to Railway Street South would be experienced and would require assessment of practical and cost effective noise mitigation in consultation with affected residents • Increased noise levels in the vicinity of Ham Street would be experienced and would require assessment of practical and cost effective noise mitigation in consultation with affected residents.
Business and land use impacts	• Access implications for properties on Groves Ave and Railway Road North. • Direct impact to businesses on Mileham Street and Railway Road North • Potential positive impacts for Mitre 10 and some businesses on Groves Avenue from increased exposure to passing trade • Potential loss of exposure to passing trade for several businesses on Windsor Road and Macquarie Street, especially those that rely on passing traffic such as fast food outlets • Improved traffic conditions on Windsor Road would reduce travel time for deliveries (increase productivity) • Impact on parking and turning movements for businesses on Groves Avenue and Railway Road North which require access for large trucks and semi trailers • Major severance of three rural grazing properties and Council land, although access would be provided under the proposed bridge • Land acquisition of properties on northern side of Railway Road North • Significant acquisition of property on corner of Railway Road North and Mulgrave Road	• Through traffic could be increased on local roads with consequent amenity and safety impacts • Potential loss of railway land at Mulgrave Station currently used for parking • Changed access arrangements for properties on southern side of Railway Road South • Impact on intersection movements and accesses in residential area • Some loss of exposure to passing trade for businesses on Windsor Road and Macquarie Street • Significant acquisition of one rural property (market garden) just west of Mulgrave Road • Strip acquisition required on southern side of Railway Road South for provision of service road. • Impact on approved industrial development north of Speedwell Place

Table 5.1 Summary Assessment of Route Options

FACTOR	OPTION 1 – NORTHERN OPTION	OPTION 2 – SOUTHERN OPTION
	• Groves Avenue and Railway Road North will become classified roads with some implications for granting access to frontage properties • Further impact to industrial land on Railway Road North due to service road	
Visual impacts	• Would affect visual character and urban design of Forbes Street. • Medium visual impact on 'Trevallyn' and for residents on Mileham Street and 'Trevallyn' that look over Day Street onto the floodplain. • Traverses the South Creek Floodplain and would affect views from Windsor and Mulgrave. South Creek valley is recognised in SREP 20 as having scenic quality of regional significance. • The route would interrupt and change rural views of flat open grassland to the north west of Mulgrave Station Precinct	• Medium to high level of visual impact restricting flood plain views to the south for residential properties fronting James Meehan St and Chisholm Place. • Medium level of visual loss for 8 properties fronting Railway Road South due to high embankment between Railway and properties. However, existing views of railway are not highly aesthetic. • High visual impact for residential property located on the western side of Mulgrave Rd, south of the railway level crossing. Loss of views across plain to north and north-west. • Traverses the South Creek Floodplain and would affect views from Windsor and Mulgrave. South Creek valley is recognised in SREP 20 as having scenic quality of regional significance. • Medium impact on flood plain as alignment follows existing railway, shortest crossing of flood plain and follows higher ground between Fairey Rd and South Creek. Less visually intrusive than Option 1. • Existing railway embankment provides screening when viewed from north (ie. Windsor Road / McGraths Hill direction) • The route would interrupt and change rural views of flat open grassland to the south west of Mulgrave Station Precinct

5.3 Community Response

The short listed options were presented to the community in a second newsletter and poster display that were released in August 2000. Have Your Say Sheet No. 2 was distributed with Newsletter No. 2 following the shortlisting of the two options, Option 1 and Option 2, for detailed investigation (refer Figure 5.1). Respondents were again asked to nominate their preferred route in terms of flood evacuation and provision for daily traffic. There were 127 Have Your Say Sheet No. 2 returned. Overall the responses indicated a positive feeling about the study, mostly due to the potential to improve traffic congestion in the area. Table 5.2 shows the preferred routes in terms of flood relief and traffic relief.

Table 5.2: Preferred Routes from Have Your Say Sheet 2

PREFERRED OPTION	OPTION 1	OPTION 2
Flood Relief	79%	21%
Traffic Relief	82%	18%

Option 1 was the preferred option for a Flood Evacuation Route across South Creek in terms of both its flood evacuation function and the potential to improve traffic performance. As for the responses to the first Have Your Say Sheet, the improvements to the performance of the road network were seen as the key benefit of the proposal.

Option 2 was preferred by a small proportion of respondents because of its lesser impact on visual amenity, reduced effects on areas of environmental sensitivity and heritage values and fewer property impacts. In addition a number of respondents who indicated support for Option 2 indicated that due to its lower cost it was more likely to be built sooner than Option 1 and that as such, the benefits associated with the project would realised in a shorter period.

6. Value Management Study No. 2

6.1 Introduction

A second Value Management Study (VMS 2) was held on 5 October 2000 to evaluate the two options and to recommend a preferred option for a flood evacuation route across South Creek. A multi-criteria assessment method was used to compare the performance of the corridor options against a range of planning and environmental factors. This involved refining the earlier selection criteria, assessing the relative importance of the criteria, and evaluating the options against the assessment criteria. Prior to VMS 2 taking place, the specialist investigations of the two route options had been completed to draft stage. The results of these investigations were used to assess the relative performance of the two options and the reports were finalised following the completion of VMS 2. The Value Management Study 2 Report provides more details.

6.2 Selection Criteria and Weighting

The workshop participants confirmed the range of factors on which a decision on the preferred corridor should be based. The notable refinements from the preliminary option evaluation phase were inclusion of the new factors – "Strategic Planning Benefits" and "heritage". The later criterion had previously been grouped under 'Environmental Impacts' but this was concluded to be inappropriate in terms of the very different types of issues involved with this project area. As noted in Section 4, assessment of the route options in relation to cost and economic performance is another important aspect of the route selection (refer Section 7). Working Paper 8 addresses the traffic and economic analysis.

The criteria were then weighted for their significance by applying the same method used in VMS 1 – a value of 10 to the criterion considered to be most important, and the others then weighted relative to this. To test the robustness of these weightings, a second technique was also utilised. The VMS 2 participants carried out a Paired Comparison which involves assessing the relative importance of each criterion by comparing it to all other criteria on a pairs basis. The sum of the paired comparisons yields an over-all weighting for each criterion. The weightings are shown in Table 6.1.

Table 6.1 Weighted Assessment Criteria

Selection Criteria	Weighting Method 1 (Value Allocation)	Weighting Method 2 (Paired Comparison)
Effectiveness for flood evacuation	10	10.00
Benefits to traffic flow/disadvantages to local road system	9	6.25
Environmental – ecological/water/waste	8	5.00
Strategic planning benefits	8	4.58
Social and amenity impacts	6	2.50
Business and land use impacts	5	2.08
Heritage	4	0.42
Visual impacts	4	0.00

The two different approaches to weighting the criteria produced similar results in terms of rank order. The most important criterion for the flood evacuation route selection was understandably effectiveness for flood evacuation, followed by benefits to traffic flow however, all criteria were considered to be important in terms of the assessment of the proposal.

6.3 Rating of Options

The two options were assessed as to how well they performed against the criteria and this was based on the detailed information presented in the draft specialist working papers and summarised in Table 5.1. For each criterion, the option that performed better was given a rating of 10 and the other option was rated relative to it. The ratings are summarised in Table 6.2.

Table 6.2 Rating of Options

Selection Criteria	Rating	
	Option 1	Option 2
Effectiveness for flood evacuation	9.5	10
Benefit to traffic flow and disadvantage to local system	10	3
Strategic plan benefits	10	4
Environmental ecological/water/waste	10	6
Social and amenity impacts	10	7
Business and land use impacts	9	10
Heritage	6	10
Visual impacts	7	10

From the table it is evident that Option 1 performed appreciably better in relation to four of the criteria, and performed slightly worse in terms of flood evacuation and business / land use impacts. Option 2 had a superior performance for heritage and visual impacts.

6.4 Evaluation of Options against Weighted Criteria

The next step in the evaluation brought together the weighted criteria and the comparative ratings of each option to obtain overall numerical performance scores for the two options. This process involved multiplying the criteria weighting (Table 6.1) and option rating (Table 6.2) to obtain a total score for each option was established as an overall measure of performance and suitability. The results are summarised in Table 6.3.

Table 6.3 Evaluation of Options

Assessment Criteria	Weighting Method 1		Weighting Method 2	
	Option 1	Option 2	Option 1	Option 2
Effectiveness for flood evacuation	95.0	100.0	95	100
Benefit to traffic flow / disadvantage to local system	62.5	18.8	90	27
Strategic plan benefits	50.0	20.0	80	32
Environmental ecological/water/waste	45.8	27.5	80	48
Social and amenity impacts	25.0	17.5	60	42
Business and land use impacts	18.8	20.8	54	50
Heritage	2.5	4.2	24	40
Visual impacts	0.0	0.0	28	40
TOTAL	300	209	511	379

For both weighting scenarios, the evaluation indicated that Option 1 was the superior option as it resulted in a higher overall score. The VMS participants discussed the results achieved through the scoring system described above and concluded that it reflected a fair assessment of the relative merits of the two options. The group then discussed the advantages and disadvantages of the options to confirm that Option 1 should be recommended as the preferred option. It is noted that the relative economic performance of the two options was assessed separately and is discussed in Section 7.

The advantages and disadvantages of Option 1 are listed in Table 6.4 below.

Table 6.4 Advantages and Disadvantages of Preferred Option

Advantages	Disadvantages
• Improves traffic flow • Better strategic planning option in terms of arterial road network development • Decrease in the use of Level Crossing Road • Less impact on the local area • Less environmental implications • Opens up opportunities for access to transport choice in the area • Greater community acceptance of the project given the arterial road benefits • Improved access and safety to Windsor Road from Mulgrave and Vineyard • Hawkesbury Council supports this option	• Higher Cost • Loss of amenity to residents on Forbes Street • Constraints to business area in Mulgrave • Larger visual impact • Property effects and land take are substantial • Impacts on heritage properties • Need for railway crossing east of Mulgrave for flood evacuation • Longer time to build • Would increase traffic demand on Cox Street level crossing during flood evacuation

7. Project Cost Estimates and Economic Analysis

7.1 Project Cost Estimates

Project cost estimates were prepared for both the short listed options. Cost estimates included all project cost elements (eg planning, establishment, earthworks, structure, contingencies etc) consistent with standard RTA estimating procedures. The estimates subsequently drew on the engineering concept design for both options in relation to quantities.

The cost estimates for the two options are presented in Table 7.1.

Table 7.1 Cost Estimates for Shortlisted Options

Option	Cost Estimate
Option 1	$64.0 million
Option 2	$34.7 million

7.2 Economic Analysis

A comparative economic analysis was undertaken for the two shortlisted route options. The detailed analysis is contained in Working Paper No. 7 – Traffic and Transport Assessment and summarised below.

The economic analysis undertaken was in the form of a Road User Cost Benefit Analysis (RUCBA) in accordance with the provisions of the RTA's Economic Analysis Manual (RTA, 1999). As the name suggests, the focus of the RUCBA is on those costs and benefits which relate directly to users of the new road infrastructure and the broader road network. The results of the RUCBA complemented the results of the qualitative assessments undertaken in relation to other environmental and social issues.

For the purposes of the economic analysis, a concept road and bridge design scheme was prepared. Given that the function of the two options differed (ie. Option 1 had a dual flood evacuation and arterial road function and Option 2 had a flood evacuation and local road function), an additional cost estimate was prepared for Option 2 which assumed this option was developed as an arterial route between Windsor Road and Ham Street. This costing was developed solely for the purposes of allowing a direct comparison between the two options in terms of their economic performance.

The estimated costs for the shortlisted options, including the different route standards for Option 2 are summarised in Table 7.2.

Table 7.2 Cost Estimates for Short Listed Route Options

Route Option	Length	Cost
Option 1 (Arterial Route Proposal)	2.85 km	$64.0 M
Option 2 (Flood Relief only)	2.00 km	$32.6 M
Option 2 (Arterial Route Equivalent)	3.50 km	$55.0 M

The benefit streams for the South Creek project have been calculated using the output from a strategic traffic model, NETANAL which calculates a number of travel statistics to compile a range of project benefits resulting from more efficient road network operation specifically the benefits are derived from travel time savings, reduced vehicle operating costs and accident savings. From these statistics, total savings are calculated and provided monetary values through allocation of standard rates for the various parameters as provided by the RTA. These savings are then compared, in present dollar terms, to the costs of the respective options to provide economic indicators of the merit of the project.

Benefits are calculated and aggregated over a 30 year project life cycle, commencing in the first year of operation of the completed project. These are interpolated and then extrapolated from the 2006 and 2016 reference year benefit calculations. In the case of the upgrading of Option 2 to an Arterial Route equivalent, it has been assumed that this would attract an additional 10% of benefits in relation to the basic flood relief construction.

A summary of the results of the comparative economic analysis is presented in Table 7.3. The results are presented in terms of the following parameters:

- Benefit / Cost Ratio (BCR) – which is the ratio of the present value of benefits to present value of costs. A BCR of 2 usually indicates a project of substantial economic merit.
- Net Present Value (NPV) – which is the difference between the benefits and the costs, providing a measure of the return on investment from the construction of the proposal.

Table 7.3 RUCBA Results for Short Listed Options

Route Option	Cost	RUCBA
Option 1 (Arterial Route Proposal)	$64.0 M	2.1
Option 2 (Flood Evacuation only)	$32.6 M	2.9
Option 2 (Arterial Route Equivalent)	$55.0 M	1.7

The results in Table 7.3 show that both Option 1 and Option 2 would provide a positive return on investment from any future construction, with BCR results of 2.26 and 2.93 respectively.

Option 1 as the principal arterial route option provides the most benefits in present value terms but its longer length and greater cost for bridge and retaining walls reduces its economic merit in relation to Option 2 (Flood Evacuation Only).

By contrast, consideration of Option 2 as a flood evacuation route only, gives it comparable benefits for a lower capital cost. If that option is further developed as an arterial route equivalent and increased benefits are derived, the BCR for that option reduces to 1.93.

In terms of a direct comparison of functionality and route standard it is clear that Option 1 is preferable from an economic perspective. This preference is enhanced by the greater traffic relief that Option 1 provides for Windsor Road and Macquarie Street.

8. Conclusions

The selection of a preferred option for a flood evacuation route across South Creek involved preliminary assessment of six options during the initial stage of the investigation. Options considered included four routes north of the railway line that would provide flood relief as well as an arterial road function and two routes south of the railway line that would provide a route for flood evacuation and an alternative route for local traffic.

The first Value Management Study resulted in two routes being short listed for further evaluation: one that met the flood evacuation criteria and the other that met flood evacuation and arterial road criteria. Specialist studies were conducted on the short listed options including flora and fauna, social / business, heritage, landscape and visual impacts, air quality and noise and vibration assessment.

The second Value Management Study evaluated the short listed options and found that Option 1 was the superior option and should be recommended as the preferred route.

Both Options 1 and 2 fulfilled the primary objective of providing for flood evacuation. Option 1 was subsequently recommended as the preferred option for consideration in the EIS process for a number of reasons which are summarised below:

- Based on the weighted factor analysis undertaken in the VMS which considered the results of the technical investigations, Option 1 has a better performance, with a clear distinction emerging between this option and Option 2 largely due to the superior performance of Option 1 in terms of the arterial road network.
- There is wide community support for this option, from businesses and local residents.
- The road user economic analysis indicated a positive economic return for Option 1.
- Less adverse impact on the surrounding community and stakeholders.

Option 1 was therefore recommended as the preferred route option for detailed analysis in the EIS. Th Minister for Roads announced in October 2001 that Option 1 had been selected as the preferred option for the Windsor Flood Evacuation Route.

9. References

Tierney Page Kirkland (2000). Windsor Flood Evacuation Route Across South Creek – Value Management Study Report. Prepared for Connell Wagner Pty Ltd.

Tierney Page Kirkland. (2000). Windsor Flood Evacuation Route Across South Creek – Value Management Study No. 2 Final Report. Prepared for Connell Wagner.

Working Paper 2 – Air Quality Assessment

AIR QUALITY IMPACT ASSESSMENT

WINDSOR FLOOD EVACUATION ROUTE

June 2002

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Attachment A Joint Wind Speed, Wind Direction and Stability Class Frequency Tables

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Attachment C Hydrocarbon Analysis of Vehicle Exhaust, Petrol and Petrol Vapour

1. INTRODUCTION

This report forms one of a series of working papers which have been prepared to examine the potential impacts of the construction and operation of a proposed flood evacuation route across South Creek near Windsor, NSW which is known as the Windsor Flood Evacuation Route.

This report contains the results of detailed investigations of environmental and social impacts of two route options, known as Option 1(the northern route) and Option 2 (the southern route) (refer **Figure 1**).

The findings of this report will be used as an input to the selection of the preferred route as documented in 'Windsor Flood Evacuation Route Across South Creek – Route Selection Report' (**Connell Wagner, 2001**).

The preferred option for the flood evacuation route will be the subject of an environmental impact statement (EIS) in accordance with the provisions of the *Environmental Planning and Assessment Act 1979.*

This report has been prepared by Holmes Air Sciences for Connell Wagner who are in turn acting for the Roads and Traffic Authority of New South Wales (RTA). Its purpose is to assess the air quality impacts resulting from the proposed Windsor Flood Evacuation Route. The report provides information on the following aspects of the air quality assessment:

- Vehicle emissions from Windsor Road and the proposed evacuation route involved in the proposal;
- Kerbside concentrations of roadway vehicle emissions; and
- Existing air quality in the vicinity of the route.

The assessment of the impacts of motor vehicle emissions is based on the use of a computer model to determine the dispersion of emissions and to predict ground-level concentrations of the various exhaust components in the area close to the road. It has been assumed that in the first year of assessment (2000), 85% of motor vehicles would be fitted with catalytic converters. In 2006, it is estimated that 90% of motor vehicles would be fitted with catalytic converters. By 2016, it is likely that all motor vehicles would be fitted with catalytic converters.

Since the introduction of unleaded petrol to power vehicles fitted with catalytic converters, there has been a steady and unambiguous decline in air lead levels in Australian cities. The new National Standard for Fuel Quality has led to a ban on the supply of leaded petrol containing more than 0.005 g/l of lead from January1, 2002. Lead levels in urban areas, without other local sources of lead, will therefore continue to decline to levels which are likely to be below the level of detection using current measurement technologies applied to measuring ambient air on a routine basis. Therefore, the impacts on air quality due to lead emissions have not been assessed.

The report comprises the following sections:

- A review of the dispersion meteorology of the area and existing air quality;
- A discussion of the model used to assess the impacts and a review of relevant air quality criteria;
- A description of the methods used to estimate vehicle emissions and a summary of the calculated emissions for Windsor Road and the proposed evacuation route; and
- Predictions of ground-level concentrations of emissions and an assessment of their impacts.

2. NEED FOR THE PROJECT

As part of the NSW Government's commitment to managing the potential effects of flooding on the Hawkesbury-Nepean floodplain, the Hawkesbury-Nepean Flood Management Advisory Committee was established in 1997. Following extensive consultation with community and Government stakeholders, the Committee produced a management strategy for the Hawkesbury-Nepean floodplain (**HNFMAC, 1997**) which was subsequently adopted by the NSW Government. One of the recommendations of this study was the need to develop evacuation routes for settled areas in the Hawkesbury-Nepean valley. Following adoption of the recommendations of the Strategy, a working group was established to further develop a road flood evacuation plan for the area. The resulting 'Interim Regional Road Upgrade Plan' (**HNFMSC, 2000**) identified the need for a high level crossing of South Creek to be developed as a key component of the flood evacuation plan for the Windsor township to help provide time for the affected community to evacuate.

3. DESCRIPTION OF THE PROPOSAL

As indicated, two options for the proposed flood evacuation route across South Creek were the subject of detailed investigations. These two options, Option 1 and Option 2 are shown on **Figure 1**, and described below.

- **Option 1 – Northern Route**

Option 1 has the dual function of providing a flood evacuation route and providing additional capacity on the arterial road network. The flood evacuation route extends from Macquarie Street, Windsor via Day Street to Railway Road North, Mulgrave. Option 1 also involves connections to The Macquarie Road/Richmond Road intersection via Forbes Road and to Windsor Road, Mulgrave via Groves Avenue. These connections are provided as connections to the arterial road network.

The alignment of Option 1 is to the north of the Richmond-Blacktown railway. This option is approximately 2.6 km in length and would involve a high level bridge structure of approximately 1150m across South Creek.

- **Option 2 – Southern Route**

This route generally runs between Ham Street, Windsor and Railway Road South in Mulgrave with a connection to Groves Avenue. The alignment of Option 2 is to the south of the Richmond-Blacktown railway. Option 2 provides a flood evacuation route but would not have a direct connection to the arterial road network. This option is approximately 2.1 km in length and would involve a high level bridge structure of approximately 450m across South Creek.

4. AIR QUALITY CRITERIA

The New South Wales Environment Protection Authority (EPA) has historically noted air quality goals for nitrogen dioxide, carbon monoxide and particulate matter determined by the World Health Organisation (WHO), the United States Environmental Protection Agency (US EPA) and the National Health and Medical Research Council of Australia (NHMRC). Air quality goals for hydrocarbons have been used previously, but these have been discarded because they are not specific for reactive species which are the important elements in the formation of photochemical smog.

The National Environment Protection Council of Australia (NEPC) has determined a new set of air quality goals for adoption at a national level, which are part of the National Environment Protection Measures (NEPM). In its recent publication "Action for Air" (**EPA, 1998**), the NSW EPA has proposed new air quality goals for particulate matter and nitrogen dioxide, some of which are consistent with the NEPM standards.

Table 1 lists the EPA's air quality goals for New South Wales including the historical goals and newly adopted goals. Not all of these are major emissions from motor vehicles. The goals that have been applied to this project are shown in bold in **Table 1**. The basis of these air quality goals and the safety margins which they provide are outlined below.

4.1 Carbon monoxide

Carbon monoxide can be harmful to humans because its affinity for haemoglobin is more than 200 times greater than that of oxygen. When it is inhaled it is taken up by the blood and therefore reduces the capacity of the blood to transport oxygen. This process is reversible and reducing the exposure would lead to the establishment of a new equilibrium.

Symptoms of carbon monoxide intoxication are lassitude and headaches, however these are generally not reported until the concentrations of carboxyhaemoglobin in the blood are in excess of 10% of saturation. This is approximately the equilibrium value achieved with an ambient atmospheric concentration of 70 mg/m^3 for a person engaged in light activity. However, there is evidence that there is a risk for individuals with cardiovascular disease when the carboxyhaemoglobin concentration reaches 4% and the WHO recommends that ambient concentrations be kept to values that would protect individuals from exceeding the 4% level.

The 15-minute, 1-hour and 8-hour goals noted by the EPA provide a significant margin for safety, however this is appropriate for this type of guideline, which is designed to protect a wide range of people in the community including the very young and elderly. The 8-hour goal is particularly relevant as equilibrium between ambient concentration and blood level concentration takes between 4 to 12 hours.

4.2 Oxides of nitrogen

Nitrogen oxides (NO_x) emitted by motor vehicles are comprised mainly of nitric oxide (NO, approximately 95% at the point of emission) and nitrogen dioxide (NO_2, approximately 5% at the point of emission). Nitric oxide is much less harmful to humans than nitrogen dioxide and is not generally considered a pollutant with health impacts at the concentrations normally found in urban environments. Concern with nitric oxide relates to its transformation to nitrogen dioxide and its role in the formation of photochemical smog. Nitrogen dioxide has been reported to have an effect on respiratory function although the evidence concerning effects has been mixed and conflicting. The EPA has not set any air quality goals for nitric oxide, however it has set 1-hour and annual average goals for nitrogen dioxide. It has adopted the NEPM standard of 0.12 ppm or 245 $\mu g/m^3$. It has also adopted the WHO 1-hour goal of 0.11ppm or 200 $\mu g/m^3$ as a long term reporting goal.

4.3 Hydrocarbons

Hydrocarbons alone do not generally pose a problem at the concentrations commonly experienced. However, some hydrocarbons such as benzene are known to have an adverse effect on human health, but these effects are thought to occur at concentrations higher than the levels of exposure found at roadsides from traffic emissions. Hydrocarbons do play a significant role in photochemical smog formation and until recently the air quality standards adopted by the US EPA for non-methane hydrocarbons have been applied in NSW. However it has been recognised that this goal does not distinguish the reactive species which are involved in smog formation from the total hydrocarbon concentration and this air quality goal has been abandoned by the US EPA and the NSW EPA.

There is growing concern about the amount of benzene released in motor vehicle emissions. At present NSW has no ambient air quality goal for benzene. The Victorian EPA currently has a limit of 0.10 mg/m^3 (0.033 ppm) (3-minute average). Many in the scientific community hold the view that there is no safe limit for benzene. The WHO specifies a risk factor for developing leukaemia of between $4.4x10^{-6}$ – $7.5x10^{-6}$ for a lifetime exposure to 1 $\mu g/m^3$ (**WHO, 1995**). The United Kingdom has an annual average ambient benzene goal of 5 parts

per billion (ppb) or 16 μg/m^3 to be achieved by 2005. The 5 ppb goal is based on the "No Observable Adverse Effect Level" from the findings of the UK Expert Panel on Air Quality Standards that the risk of leukaemia in workers would not be detectable when the average working lifetime exposure to benzene was less than 500 ppb. Two safety factors of 10 were then applied to derive the goal of 5 ppb.

Table 1 – NSW Air Quality Goals

Pollutant	Goal	Agency
Carbon monoxide	87 ppm or 108 mg/m^3 (15-minute maximum)	WHO
	25 ppm or 31 mg/m^3 (1-hour maximum)	WHO
	9 ppm or 10 mg/m^3 (8-hour maximum)	**NHMRC**
		NEPM
Nitrogen dioxide	0.16 ppm or 320 μg/m^3 (1-hour maximum)	NHMRC
	0.05 ppm or 103 μg/m^3 (annual mean)	US EPA
	0.12 ppm or 245 μg/m^3 (1-hour maximum)	**NEPM, NSW EPA**
	0.11ppm or 200 μg/m^3 (1-hour maximum)	WHO, NSW EPA long term reporting goal
	0.03 ppm or 60 μg/m^3 (annual mean)	NEPM, NSW EPA
Total suspended Particulate matter (TSP)	90 μg/m^3 (annual mean)	NHMRC
Particulate matter < 10 μm (PM_{10})	50 μg/m^3 (annual mean)	US EPA
	30 μg/m^3 (annual mean)	**NSW EPA**
	150 μg/m^3 (24-hour maximum)	US EPA
	50 μg/m^3 (24-hour maximum)	**NEPM, NSW EPA**
Lead	1.5 μg/m^3 (90-day average)	NHMRC
	0.5 μg/m^3 (annual average)	NEPM
Ozone	0.10 ppm or 200 μg/m^3 (1-hour maximum)	NHMRC, NEPM,
	0.08 ppm or 150 μg/m^3 (4-hour maximum)	NSW EPA
	0.08 ppm or 150 μg/m^3 (1-hour maximum)	NSW EPA long term reporting goal
	0.06 ppm or 120 μg/m^3 (4-hour average)	
Sulphur dioxide	0.25 ppm or 700 μg/m^3 (10-minute maximum)	NHMRC and NEPM
	0.20 ppm or 570 μg/m^3 (1-hour maximum)	NEPM
	0.08 ppm or 225 μg/m^3 (1 day)	NHMRC and NEPM
	0.02 ppm or 60 μg/m^3 (annual mean)	

ppm - part per million

μg/m^3 – micrograms per cubic metre

mg/m^3 – milligrams per cubic metre

4.4 Particulate matter

The presence of particulate matter in the atmosphere can have an adverse effect on health and amenity. The health effects of particles are largely related to the extent to which they

can penetrate the respiratory tract. Larger particles, that is those greater than 10 μm in aerodynamic diameter, generally adhere to the mucus in the nose, mouth, pharynx and larger bronchi and from there are removed by either swallowing or expectorating. The nature of particles in the air has an inverse relationship between the size of the particle and it's diameter. So that as a particle diameter decreases, the number of similarly sized particles increases. This relationship is a factor resulting in increased scientific concern about the effects of fine particles.

Fine particles are of concern for two principal reasons, since they have the ability to penetrate deeper into the lungs and the increased number of similarly sized particles that can reach the deep regions of the lung, like the alveolar sacs. The presence of particles can inflame tissue in this region and behind the alveolar sacs, since it is quite sensitive to foreign material. The human body does have defenses against deposition of particles in this region but due to the increased number of particles this mechanism may be unable to cope, resulting in inflammation of the alveolar sacs.

The health effects of particulate matter are further complicated by the chemical nature of the particles and by the possibility of synergistic effects with other air pollutants such as sulphur dioxide.

Much of the recent concern over the health effects of fine particulate matter is based on investigations carried out in the US, with the view to quantifying the health risks associated with both long-term and short-term exposure to airborne particulate matter. The study is colloquially referred to as "The Six Cities Study" from the original work by **Dockery et al. (1993)**, which determined a relationship between very fine particulate matter (defined as particles smaller than 2.5 μm in aerodynamic diameter) in the air and mortality in six US cities.

The basic findings of the Six Cities Study is that there is an increase in mortality with increasing concentrations of very fine particulate matter. The conclusions appear to be robust and have been supported by subsequent studies and as far as can be determined are not confounded by other known variables. The mechanism is still unknown.

These findings have not yet been fully evaluated by the scientific community. New Australian standards determined by NEPC include a goal for PM_{10} but not for $PM_{2.5}$. The US EPA has not changed its PM_{10} (particles less than 10 μm in aerodynamic diameter) goal but has introduced new goals for very fine particles ($PM_{2.5}$) with a 24-hour limit of 65 $\mu g/m^3$ and an annual limit of 15 $\mu g/m^3$. The NSW EPA has historically noted the US EPA 24-hour air quality standard of 150 $\mu g/m^3$ and annual average standard of 50 $\mu g/m^3$ for PM_{10}. It now adopts the NEPM 24-hour standard of 50 $\mu g/m^3$, and references a new annual average of 30 $\mu g/m^3$ as a long-term reporting goal. The 24-hour goal of 50 $\mu g/m^3$ corresponds to a visual distance of approximately nine kilometres (**EPA, 1998**). The NSW EPA also continues to note the NHMRC's 90 $\mu g/m^3$ annual average goal for total suspended particulate matter (TSP). This level is recommended as the maximum permissible level in urban environments.

4.5 Lead

Lead is a cumulative poison which exerts its toxic effects on the kidneys, blood and central nervous system. It is now generally agreed, that while the effects are not readily discernible on an individual basis, on a population basis, lead exposure in young children can lead to an IQ deficit of between 2 to 3 points for each 10 mg/dL increment in blood lead. As environmental lead has emerged as a public health issue, governments in Australia have developed strategies to reduce the levels of lead exposure, particularly in young children. At its November 1993 meeting, the NHMRC recommended a reduction of lead in all Australians to less than 15 mg/dL was to be achieved by the end of 1998, with the exception of occupational exposures. It is suggested in the same document that the strategies needed to

achieve this target would also result in 90% of children aged 1 to 4 having a blood lead level below 10 mg/dL, also by the end of 1998.

It has been estimated (**ABS, 1992**) that about 90% of the lead in air arises from motor vehicle emissions, apart from areas where there are significant local lead industries. However ingestion, rather than inhalation is the more significant route of lead intake for young children who absorb lead very efficiently, up to 50% of ingested lead compared to 10-15% in adults.

Most ingested lead is from contaminated soil and dust which children take in through exploratory hand to mouth activities. Some measures to reduce lead intake through ingestion include washing of children's hands and faces before meals, regular washing of outside toys and planting grass or ground cover on exposed areas of soil in the yard.

The NHMRC goal for lead is 1.5 $\mu g/m^3$ (90-day average) and is currently under consideration for revision downwards. The NEPM standard of 0.5 $\mu g/m^3$ is based on an annual average concentration. Since the introduction of unleaded petrol there has been a steady and unambiguous decline in lead emissions (**VEPA, 1991**) and in the low concentration of lead in the air in the urban environment, clearly demonstrating the effectiveness of this strategy. EPA monitoring data in NSW shows the same trend (**Figure 2**). From 1986 when unleaded petrol was introduced to 1997, the lead levels in air have been reduced in suburban Sydney, so that ambient levels are now about one tenth of the NHMRC goal.

4.6 Ozone

Ozone is a powerful oxidant, formed in the atmosphere in the presence of sunlight, nitrogen oxides and reactive hydrocarbons. It is not a primary emission from motor vehicles but a regional pollutant, being a major component of photochemical smog. Because of its highly reactive nature, ozone can combine with virtually all classes of biologically active molecules including enzymes, proteins and lipids. Cellular membranes are a target for ozone which has also been reported to have an irritant effect on the respiratory system. The air quality goal has been revised downwards from 257 $\mu g/m^3$ (0.12 ppm) to 200 $\mu g/m^3$ (0.10 ppm) for a 1-hour maximum. In addition the EPA now notes a 4-hour ozone goal of 0.08 ppm or 170 $\mu g/m^3$. The revision of the goal to a more stringent standard may result in additional exceedances of the goal without changes to air quality.

4.7 Sulphur dioxide

Sulphur dioxide is an acid gas which can have harmful effects on the respiratory system as well as on vegetation and building materials. It is however a minor component of motor vehicle emissions, due to the low sulphur content of Australian petrol, and has not been assessed quantitatively in this study. For example the Metropolitan Air Quality Study (MAQS) estimates that for the 1992 fleet, average SO_2 emissions under arterial travel conditions are 0.065 g/km compared to emissions of nitrogen oxides of 2.33 g/km for the same conditions. In addition transient emissions of above average levels of odorous sulphur compounds such as hydrogen sulphide and carbonyl sulphide (which may be smelt at concentrations as low as 5 ppb) have been noted from vehicles fitted with catalytic converters. While these compounds may produce a local short term nuisance, they do not represent significant emissions under normal running conditions.

5. DISPERSION METEOROLOGY AND AIR QUALITY ISSUES

This section describes the dispersion meteorology, general climate and air quality in the study area. As well as information on prevailing wind patterns, historical data on temperature, humidity and rainfall are presented to give a more complete picture of the local climate. Air quality issues relating to emissions from motor vehicles are also discussed.

5.1 Climate and meteorology

5.1.1 Rainfall, temperature and humidity

Table 2 presents the temperature, humidity and rainfall data for Richmond (**Bureau of Meteorology, 2000**). Temperature and humidity data consist of monthly averages of 9 am readings. Also presented are monthly averages of maximum and minimum temperatures. Rainfall data consist of mean and median monthly rainfall and the average number of raindays per month.

From temperature data recorded over 55 years, the annual average maximum and minimum temperatures experienced were 23.7 °C and 10.9 °C respectively. January was the hottest month with an average maximum temperature of 29.6 °C. July was the coldest month, with an average minimum temperature of 3.6 °C.

The annual average humidity reading from 46 years of data collected at 9 am was 74%. The month with the highest humidity on average was June with an average of 83%. 3 pm readings were not recorded. Rainfall data collected over 55 years show that February was the wettest month, with mean rainfall reading of 106 mm. The average number of raindays for February was 11. The average annual rainfall was 810 mm and the average number of raindays was 110.

5.1.2 Wind data for the Windsor area

The terrain along Windsor Road and the flood evacuation route intersects the relatively flat South Creek floodplain, making dispersion of pollutants dominated by prevailing wind patterns.

The closest meteorological station with data that can be considered as representative of the meteorological conditions along the route is at Richmond. **Figure 3** presents seasonal and annual wind roses compiled from this data. On an annual basis, the most frequent winds in the area surrounding Richmond are from the north-east sector. This pattern persists from autumn through to spring, although in winter and spring there is also a westerly component. In summer, westerlies comprise a much smaller percentage of winds.

Table 2: Temperature, humidity and rainfall data for Richmond

(Station number 067033 Latitude 33° 36' S Longitude 159° 47' E Elevation 19 m)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
9 am Mean Temperatures (°C) and Relative Humidity (%) (46 years of record)													
Dry-Bulb	22.1	21.4	20.3	16.6	12.3	9.2	8.0	10.4	14.1	17.7	19.6	21.4	16.0
Wet-Bulb	18.9	18.8	17.6	14.5	10.8	8.0	6.6	8.3	11.0	13.8	15.7	17.6	13.4
Relative Humidity	73.0	78.0	76.0	78.0	83.0	83.0	80.0	73.0	66.0	63.0	65.0	67.0	74.0
Daily Maximum Temperatures (°C) (55 Years of record)													
Mean	29.6	28.6	27.0	23.9	20.3	17.6	17.2	18.8	21.6	24.5	26.8	28.7	23.7
Daily Minimum Temperatures (°C) (57 Years of record)													
Mean	17.4	17.4	15.5	11.8	7.9	5.1	3.6	5.0	7.5	11.0	13.7	15.9	10.9
Rainfall (mm) (55 Years of record)													
Mean	93	106	92	70	59	56	36	46	40	64	76	72	810
Median	76	78	75	45	29	33	24	21	31	53	62	56	836
Raindays (Number) (58 Years of record)													
Mean	11	11	11	9	9	8	6	8	8	10	10	10	110

Source: Bureau of Meteorology (2000)

The dispersion model used for this assessment, CAL3QHC, requires data on wind speed, wind direction, temperature, mixed-layer height[1] and atmospheric stability[2] class. Hourly data for a typical year has been collected at Richmond in 1981 by the Bureau of Meteorology and forms the basis on which meteorological conditions in the area are assessed.

Table 3 shows the frequency of occurrence of the different stability categories expected in the area. The high frequency of F class stabilities indicates that dispersion conditions will be such that emissions would generally not disperse rapidly for a significant proportion of the time (39 per cent F-class). Mixing height was determined using a scheme defined by **Powell (1976)** for daytime conditions and an approach described by **Venkatram, (1980)** for night time conditions.

Joint wind speed, wind direction and stability class frequency tables derived from the Richmond data are presented in **Attachment A.**

Table 3 – Frequency of occurrence of stability classes at Richmond Island

Stability Class	Percentage frequency of occurrence
A	4.4
B	18.2
C	13.9
D	16.2
E	8.0
F	39.3
Total	**100**

5.2 Air quality issues

5.2.1 Regional air quality

The NSW government has completed a major study of air pollution in the Sydney airshed and the Newcastle and Illawarra regions, known as the *Metropolitan Air Quality Study* (referred to as MAQS). The findings of this study were released on 17 July, 1995.

MAQS upgraded and extended the Environment Protection Authority's air quality monitoring network over a wider area. It has also further refined the understanding of the present chemistry and air movements in the Sydney airshed and has updated the emissions

[1] The term mixed-layer height, refers to the height above the ground through which ground-based emissions will eventually be dispersed once a plume has been thoroughly mixed. An elevated plume, initially above the mixed-layer height will remain isolated from the ground until such time as the mixed-layer height reaches the height of the plume. In general the mixed-layer height will increase during the day as the sun causes convection to deepen the turbulent layer of the atmosphere close to the ground. Mixed-layer height will also increase if the wind speed increases because higher wind speeds will increase turbulence as the wind blows over the rough ground.

[2] In dispersion modelling, stability class is used to categorise the rate at which a plume will disperse. In the Pasquill-Gifford stability class assignment scheme (as used in this study) there are six stability classes, A through to F. Class A relates to unstable conditions, such as might be found on a sunny day with light winds. In such conditions plumes will spread rapidly. Class F relates to stable conditions, such as occur when the sky is clear, the winds are light and an inversion is present. Plume spreading is slow in these circumstances. The intermediate classes B, C, D and E relate to intermediate dispersion conditions.

inventories. Some aspects of this are discussed in the following sections. However, future air quality projections would need to take account of improved emissions controls or changes in population growth and employment patterns which are emerging. On the basis of the MAQS findings the EPA has developed a 25 year air quality management plan which is summarised in it's publication Action for Air (**EPA, 1998**). This document sets out the EPA's priorities for maintaining and improving air quality in the greater metropolitan area. The plan presents an integrated "whole of government" approach which incorporates commitment from the RTA and the NRMA through its Clean Air 2000 program which aims to reduce congestion and emissions by the year 2000 through driver education.

The Action for Air plan includes the following actions:

- Adoption of new ambient air quality goals, drawing in part from those developed at a federal level by the National Environment Protection Council (NEPC) which were discussed in **Section 3.** They have been adopted by the EPA for local use and are summarised in **Table 1**.
- Reducing emissions from motor vehicles as high priority and this will be undertaken by improved transport options including new rail links, encouraging cycling and walking as well as integrating the planning of freight movement.

Heavy duty diesel vehicles have been identified as a major source of oxides of nitrogen and particulate emissions and these will be targeted by promoting the use of cleaner fuels and alternative technologies for trucks and buses.

5.2.2 Vehicle emissions and photochemical smog

Motor vehicle emissions have the potential to contribute significantly to photochemical smog in an urban environment. Photochemical smog is formed by the reaction between nitrogen oxides and reactive hydrocarbons in the presence of sunlight. Models for the formation of photochemical smog envisage hydrocarbon emissions mostly from motor cars, facilities for the storage of hydrocarbons or spray painting operations, mixing with nitrogen oxides from either industrial sources or from motor cars. The mixture of pollution from these sources then reacts photochemically to form photochemical smog comprising mainly ozone, but also including other oxidants. At sufficient concentrations the smog can affect the eyes and respiratory system and can adversely affect plants and materials. For this project, which consists of traffic volumes under specific meteorological conditions, photochemical smog formation is not a significant issue.

5.2.3 Conversion of nitric oxide to nitrogen dioxide

As discussed previously there are no ambient air quality goals for nitric oxide, the major nitrogen oxide emission from motor vehicles. Nitric oxide is however converted to nitrogen dioxide and its rate of conversion is dependent on the presence of oxidising agents. The issue here is the rate at which nitric oxide converts to the more harmful nitrogen dioxide. Analysis of the EPA's oxides of nitrogen monitoring data reveals that the percentage of nitrogen dioxide in the air is inversely proportional to the total oxides of nitrogen concentration. **Figure 4** presents a plot of the mean total oxides of nitrogen against the mean nitrogen oxide concentration recorded at five sites in Sydney, namely Earlwood, Eagle Vale, Lidcombe, Kensington and Rozelle, from 1988 to 1992.

The trend in the graph shows an inverse correlation between total oxides of nitrogen concentration and percentage of nitrogen dioxide. Average percentages of nitrogen dioxide in the EPA monitoring data range from 5 to 50%, with a mean value of about 30%. These data however, are mostly collected at some distance from roads and while they demonstrate the inverse correlation between total oxides of nitrogen concentrations and nitrogen dioxide

concentrations, they do not necessarily reflect the proportion of nitrogen dioxide which would be present close to roadways.

Monitoring data collected by the RTA in Sydney (**RTA, 1997**) indicate that close to the roadways, nitrogen dioxide would make up from 5 to 20% by weight of the total oxides of nitrogen. A conservative value of 10% by weight at the kerbside, 15% by weight 10 m from the roadway and 20% by weight at 50 m from the roadway has been used in the impact assessment presented in **Section 8**.

5.3 Existing air quality

Local air quality with respect to ozone, nitrogen oxides, sulfur dioxide, carbon monoxide and particulate matter has been monitored at sites that represent existing concentrations currently experienced near the proposed route. These monitoring data give an indication of the likely background levels in the study area.

The two principal sites that measure pollutants and are close to the proposed route are Vineyard and Richmond. The Blacktown monitoring station was also included to provide air quality information when data from the two principal sites was not available. All these sites are maintained by the NSW EPA and form part of the Sydney network of monitors used to assess air quality across the Sydney Basin. Data consisted of continuous measurements over a 22 month period from September 1996 to June 1998. While gaseous data were reported in parts per hundred million (pphm), they have been converted to micrograms per cubic metre ($\mu g/m^3$) to ensure consistency with the modelling results (**Section 8**). Monitored levels for all pollutants, except ozone and PM_{10}, were well below their respective historical and newly adopted air quality goals.

During the warmer months, from late spring to early autumn, there is a higher concentration of ozone, which is evident in **Table 4** and **5**. This is due to the dependence of ozone formation on solar radiation as discussed in **Section 3** and **4**.

The carbon monoxide data measured at the Blacktown monitoring site has also been included in the table as Richmond and Vineyard did not measure this pollutant during the monitoring period.

Table 4 : Maximum 1-hour pollutant concentrations at EPA maintained sites near Windsor

Month & Year	Ozone ($\mu g/m^3$)		Nitrogen dioxide ($\mu g/m^3$)		Sulfur dioxide ($\mu g/m^3$)		Carbon monoxide (mg/m^3)	PM_{10} (24-hour average) ($\mu g/m^3$)	
	Richmond	Vineyard	Richmond	Vineyard	Richmond	Vineyard	Blacktown	Richmond	Blacktown
September 1996	98.4	109.1	63.6	67.7	51.5	28.6	3.3	20.0	25.0
October 1996	137.0	143.4	63.6	69.7	20.0	28.6	3.1	19.0	23.0
November 1996	160.5	164.8	49.2	73.8	17.2	17.2	3.3	33.0	30.0
December 1996	199.0	173.3	57.4	57.4	22.9	11.4	1.9	22.0	28.0
January 1997	158.4	151.9	77.9	49.2	25.7	34.3	2.4	29.0	25.0
February 1997	164.8	226.8	80.0	71.8	45.8	22.9	2.4	20.0	22.0
March 1997	201.2	192.6	75.9	57.4	20.0	25.7	4.6	25.0	24.0
April 1997	117.7	115.6	59.5	98.4	45.8	48.6	6.8	32.0	31.0
May 1997	72.8	66.3	71.8	82.0	14.3	37.2	5.4	13.0	34.0
June 1997	66.3	72.8	61.5	**	11.4	17.2	8.4	24.0	35.0
July 1997	68.5	72.8	77.9	77.9	17.2	28.6	5.8	24.0	27.0
August 1997	113.4	119.8	57.4	75.9	22.9	22.9	4.4	27.0	44.0
September 1997	102.7	104.9	65.6	75.9	22.9	45.8	3.9	16.0	20.0
October 1997	143.4	143.4	45.1	73.8	22.9	34.3	2.5	17.0	17.0
November 1997	256.8	192.6	131.2	61.5	25.7	31.5	2.9	24.0	29.0
December 1997	222.6	214.0	59.5	67.7	25.7	34.3	4.1	29.0	41.0
January 1998	199.0	258.9	57.4	47.2	**	28.6	2.0	19.0	19.0
February 1998	211.9	205.4	67.7	45.1	**	37.2	3.5	39.0	35.0
March 1998	192.6	199.0	108.7	90.2	31.5	11.4	4.6	38.0	66.0
April 1998	132.7	122.0	73.8	75.9	14.3	37.2	3.9	26.0	23.0
May 1998	77.0	79.2	61.5	69.7	11.4	28.6	8.8	24.0	35.0
June 1998	62.1	62.1	53.3	61.5	17.2	28.6	7.6	19.0	28.0
Maximum	256.8	258.9	131.2	98.4	51.5	48.6	8.8	39.0	66.0
NSW EPA Goal (1-hour)	**200**	**200**	**245**	**245**	**570**	**570**	**31**	**50**	**50**

** Data not available. *$\mu g/m^3$ – micrograms per cubic metre* *mg/m^3 – milligrams per cubic metre* Source: NSW EPA

Table 5: Summary of 1-hour average pollutant concentration at monitoring stations near the proposed route

Month & Year	Ozone ($\mu g/m^3$)		Nitrogen dioxide ($\mu g/m^3$)		Sulfur dioxide ($\mu g/m^3$)		Carbon monoxide (mg/m^3)	PM_{10} (24-hour average) ($\mu g/m^3$)	
	Richmond	Vineyard	Richmond	Vineyard	Richmond	Vineyard	Blacktown	Richmond	Blacktown
September 1996	32.10	36.38	6.2	8.2	2.9	5.7	0.4	13.0	15.0
October 1996	27.82	27.82	12.3	12.3	2.9	5.7	0.3	13.0	17.0
November 1996	32.10	29.96	6.2	6.2	2.9	5.7	0.3	19.0	18.0
December 1996	29.96	27.82	10.3	10.3	2.9	5.7	0.4	20.0	21.0
January 1997	19.26	17.12	10.3	8.2	2.9	5.7	0.4	19.0	18.0
February 1997	21.40	19.26	10.3	14.4	2.9	2.9	0.4	13.0	13.0
March 1997	23.54	23.54	10.3	12.3	2.9	5.7	0.5	16.0	15.0
April 1997	25.68	23.54	12.3	14.4	2.9	5.7	0.8	23.0	27.0
May 1997	12.84	23.54	14.4	14.4	2.9	2.9	0.5	10.0	21.0
June 1997	8.56	14.98	14.4	**	2.9	2.9	0.6	14.0	22.0
July 1997	10.70	14.98	14.4	16.4	2.9	2.9	0.5	11.0	15.0
August 1997	14.98	21.40	12.3	12.3	2.9	2.9	0.6	16.0	18.0
September 1997	21.40	25.68	12.3	10.3	2.9	5.7	0.5	13.0	15.0
October 1997	27.82	32.10	12.3	10.3	2.9	5.7	0.4	16.0	15.0
November 1997	32.10	36.38	12.3	10.3	2.9	5.7	0.5	18.0	20.0
December 1997	29.96	36.38	10.3	8.2	2.9	8.6	0.6	21.0	31.0
January 1998	19.26	25.68	14.4	8.2	**	8.6	0.4	15.0	16.0
February 1998	27.82	29.96	12.3	10.3	**	8.6	0.5	28.0	26.0
March 1998	23.54	25.68	12.3	10.3	2.9	5.7	0.6	27.0	31.0
April 1998	17.12	17.12	10.3	10.3	2.9	8.6	0.5	14.0	14.0
May 1998	14.98	19.26	10.3	12.3	2.9	8.6	1.0	14.0	21.0
June 1998	12.84	14.98	10.3	14.4	2.9	8.6	0.9	9.0	16.0
Average	21.49	24.25	11.4	11.1	2.9	5.8	0.5	16.5	19.5
NSW EPA Goal (annual)	**No long term goal**		**60**	**60**	**50**	**50**	**No long term goal**	**30**	**30**

** Data not available *$\mu g/m^3$ – micrograms per cubic metre* *mg/m^3 – milligrams per cubic metre* Source: NSW EPA

6. METHODS FOR ASSESSING IMPACTS

The computer modelling package used to assess the air quality impacts for the project was Breeze Roads. Breeze Roads incorporates Caline4 and CAL3QHC dispersion models for estimating concentrations of roadside pollutants, such as carbon monoxide, oxides of nitrogen, hydrocarbons and particulate matter from motor vehicles. In this case, the CAL3QHC dispersion model was used, since it allows for 24-hour and annual average concentrations to be determined (Caline4 does not have this feature). This was undertaken to assess the impact of particulate matter with associated long term air quality goals (**Table 1**).

CAL3QHC is a steady state Gaussian model which can determine concentrations at receptor locations downwind of "at grade", "fill", "bridge" and "cut section" roadways located in relatively uncomplicated terrain. The model is applicable for any wind direction, roadway orientation and receptor location and has been validated in a study undertaken in Sydney by **Williams et al. (1994)** for the RTA.

7. METHODS FOR ESTIMATING EMISSIONS

This section provides a brief description of the methods used to calculate the major emissions from vehicles, namely nitrogen oxides, carbon monoxide, hydrocarbons and particulate matter. This information is required as input to the dispersion model used to predict ground-level concentrations of the various pollutants.

An estimation of these emissions has been made by **Pengilley (1989)** and US EPA emission factors **(US EPA, 1985)**. These data have been used in previous roadway studies, however a comprehensive emissions inventory which relates vehicle emissions to different travel conditions in NSW has now been prepared for the Metropolitan Air Quality Study (MAQS) (**Carnovale et. al 1997**). These emission rates have been combined with traffic flow data to determine air quality impacts of the project in 2000, 2006 and 2016. **Attachment B** provides a detailed description of the calculation of vehicle emissions for the three years.

The impacts of the proposed evacuation route have been assessed according to estimated traffic volumes in 2000, 2006 and 2016. While the evacuation route will not be operational with respect to the 2000 traffic data, is included to provide an idea of the predicted change in air quality near Windsor Road and evacuation route.

7.1 Carbon monoxide

The way in which vehicle emissions vary with speed is fundamental to the understanding of the analysis presented in this report. The relationship between speed and carbon monoxide emission is shown in **Figure 5** where the estimated carbon monoxide emission rates in the years 1988 and 2006 are presented for light duty petrol vehicles (hot start). It is assumed that approximately 30% and 96% of cars are fitted with catalytic converters in 1988 and 2006 respectively. For cars without catalytic converters, there is a marked decrease of emissions with speed. Fitting cars with catalytic converters reduces the overall emissions and again the same pattern of decreasing emission rate with speed is observed.

The emissions of carbon monoxide from vehicles were determined for previous studies from these relationships (**Nigel Holmes & Associates, 1992, 1994**). The emissions inventory prepared for MAQS takes a different approach. Although similar principles apply in terms of the relationship between speed and emissions, the roads are divided into different categories and emissions from the mix of traffic on that type of road is determined. The categories are as follows:

Freeway/Highway	Major roads with relatively high average speeds (say in excess of 40 km/h), low congestion levels (say less than 5% idle time) and low proportion of heavy duty vehicles.

Arterial	Major roads with moderate average speeds (say 20-40 km/h), moderate congestion levels (say 20% idle time) and low proportion of heavy duty vehicles (say less than 7% of total fleet vehicle kilometres travelled (VKT)).
Commercial-Arterial	Major roads with moderate average speeds and congestion levels and moderate proportions of heavy duty vehicles (say greater than 7% of total fleet VKT).
Commercial-Highway	Major roads with relatively high average speeds, low congestion levels and moderate proportions of heavy duty vehicles.
Residential/Minor	Secondary roads with moderate average speeds, negligible levels of congestion and a very low proportion of heavy duty vehicles (say one half the arterial road level).

Emission rates for different vehicle categories and different roadway classifications have been determined. Details of these are presented in **Attachment B**. For this assessment it has been assumed that travel on the Windsor Flood Evacuation Route equates to arterial travel conditions.

7.2 Oxides of nitrogen

Oxides of nitrogen emissions show a different trend with speed from carbon monoxide and this is illustrated in **Figure 6**. As in the case of carbon monoxide, catalytic converters reduce the overall oxides of nitrogen emission rate, however the trend with increasing speed is reversed, that is, oxides of nitrogen increases with increasing speed although the effect is more gradual.

As for carbon monoxide, while the same trend remains, these emission factors have been replaced by those determined for MAQS (see **Attachment B**).

7.3 Hydrocarbons

Hydrocarbon emissions vary with speed in a similar way to carbon monoxide and have been determined for MAQS in the same way (see **Attachment B**).

7.4 Particulate matter

Particulate matter emission rates for the different road categories are presented in **Attachment B**. These comprise exhaust emissions as well as emissions from tyre and brake wear.

8. ESTIMATED EMISSIONS

Emission rates during different hours for each route were estimated from the traffic volume at peak and off-peak times. The estimated morning and evening peak hour traffic flows in 2000, 2006 and 2016 are summarised in **Table 6**. These data have been provided from a transport assessment undertaken by Wagner (**Connell Wagner, 2001**). The direction of traffic flow is divided as travelling either southbound, alternatively towards the city, or northbound (Windsor bound). Details of assumptions relating to the percentages of heavy vehicles and diesel fuelled vehicles are presented in **Attachment B**.

Table 6 – Estimated peak hour traffic (morning and evening) for 2000, 2006 and 2016

Roadway Option	Peak traffic time	Estimated traffic volumes – 1 hour peak (vehicles per hour)					
		2000		2006		2016	
		SB	NB	SB	NB	SB	NB
Windsor Road (without evacuation route)	am	1311	1455	1478	1124	1683	1139
	pm	1119	1545	961	1712	970	1835
Option 1							
Windsor Road	am	931	986	1096	745	1140	779
	pm	698	1056	665	1073	670	1120
Arterial and flood evacuation route	am	419	493	434	405	575	380
	pm	453	508	362	676	347	738
Option 2							
Windsor Road	am	1203	1268	1255	872	1469	896
	pm	985	1357	870	1258	885	1371
Flood evacuation route and local road	am	133	208	383	276	393	271
	pm	156	220	199	486	201	499

Source : Connell Wagner (2001)

Emission estimates for Windsor Road and the evacuation route are presented in detail in **Attachment B**, and are summarised in **Tables 7** to **9**. **Table 7** is the emission estimate calculated for the three scenarios (Windsor Road, Option 1 and Option 2) in 2000. The mix of older and newer vehicles has been taken into account when determining emission rates for the light duty vehicle fleet. Information has been taken from the MAQS emissions inventory for different years, and a new emission rate calculated from a combination of emissions from older vehicles and emissions from newer vehicles. This process is discussed further in **Attachment B**.

Table 7 – Estimated peak hour traffic emissions for 2000 (kg/km/hour)

Roadway Option	Direction of traffic flow	Carbon monoxide		Nitrogen oxides		Hydrocarbons		Particulate matter	
		am	pm	am	pm	am	pm	am	pm
Windsor Road (without evacuation route)	Southbound	15.11	12.89	3.34	2.85	1.34	1.14	0.30	0.26
	Northbound	16.76	17.80	3.71	3.94	1.48	1.57	0.34	0.36
Option 1									
Windsor Road	Southbound	10.73	8.04	2.37	1.78	0.95	0.71	0.22	0.16
	Northbound	11.36	12.17	2.51	2.69	1.00	1.08	0.23	0.24
Arterial and flood evacuation route	Southbound	4.90	5.30	1.18	1.28	0.45	0.48	0.11	0.12
	Northbound	5.77	5.94	1.39	1.43	0.53	0.54	0.13	0.14
Option 2									
Windsor Road	Southbound	13.86	11.35	3.07	2.51	1.23	1.00	0.28	0.23
	Northbound	14.61	15.64	3.23	3.46	1.29	1.38	0.29	0.31
Flood evacuation route and local road	Southbound	1.62	1.90	0.48	0.57	0.16	0.19	0.05	0.06
	Northbound	2.54	2.69	0.75	0.80	0.25	0.27	0.08	0.08

Table 8 – Estimated peak hour traffic emissions for 2006 (kg/km/hour)

Roadway Option	Direction of traffic flow	Carbon monoxide		Nitrogen oxides		Hydrocarbons		Particulate matter	
		am	pm	am	pm	am	pm	am	pm
Windsor Road (without evacuation route)	Southbound	14.30	9.30	3.64	2.37	1.26	0.82	0.34	0.22
	Northbound	10.87	16.56	2.77	4.22	0.96	1.46	0.26	0.40
Option 1									
Windsor Road	Southbound	10.60	6.43	2.70	1.64	0.94	0.57	0.25	0.15
	Northbound	7.21	10.38	1.84	2.65	0.64	0.92	0.17	0.25
Arterial and flood evacuation route	Southbound	4.29	3.58	1.19	0.99	0.39	0.33	0.12	0.10
	Northbound	4.00	6.68	1.11	1.85	0.37	0.61	0.11	0.18
Option 2									
Windsor Road	Southbound	12.14	8.42	3.09	2.15	1.07	0.74	0.29	0.20
	Northbound	8.44	12.17	2.15	3.10	0.75	1.08	0.20	0.29
Flood evacuation route and local road	Southbound	4.03	2.09	1.36	0.71	0.41	0.21	0.15	0.08
	Northbound	2.91	5.12	0.98	1.72	0.29	0.51	0.11	0.18

Table 9 – Estimated peak hour traffic emissions for 2016 (kg/km/hour)

Roadway Option	Direction of traffic flow	Carbon monoxide		Nitrogen oxides		Hydrocarbons		Particulate matter	
		am	pm	am	pm	am	pm	am	pm
Windsor Road (without evacuation route)	Southbound	14.72	8.49	4.08	2.35	1.30	0.75	0.391	0.22
	Northbound	9.97	16.05	2.76	4.45	0.88	1.42	0.26	0.43
Option 1									
Windsor Road	Southbound	9.97	5.86	2.76	1.62	0.88	0.52	0.26	0.16
	Northbound	6.82	9.80	1.89	2.71	0.60	0.87	0.18	0.26
Arterial and flood evacuation route	Southbound	5.17	3.12	1.55	0.93	0.48	0.29	0.15	0.09
	Northbound	3.41	6.63	1.02	1.99	0.31	0.61	0.10	0.20
Option 2									
Windsor Road	Southbound	12.85	7.74	3.56	2.14	1.14	0.68	0.34	0.21
	Northbound	7.84	12.00	2.17	3.32	0.69	1.06	0.21	0.32
Flood evacuation route and local road	Southbound	3.81	1.95	1.38	0.70	0.39	0.20	0.15	0.08
	Northbound	2.62	4.83	0.95	1.75	0.27	0.49	0.10	0.19

9. PREDICTION AND ASSESSMENT OF IMPACTS

9.1 Operational Impacts

This section assesses the air quality impacts of the project by comparing the predicted ground-level concentrations of roadway emissions with air quality goals or other air quality criteria where specified goals are not available.

Tables 10 to **18** present the maximum predicted 1-hour average ground-level concentrations of carbon monoxide, nitrogen dioxide and hydrocarbons at various distances from the

proposed kerb. Apart from PM_{10} which is dealt with separately, these are the major emissions from motor vehicles.

These 1-hour maximum concentrations are the levels of pollutants that are attributable to vehicles using Windsor road and the proposed evacuation route. They do not include background levels of pollutants, which can be taken from existing monitoring data. It is important to note however that the current air quality monitoring includes existing emissions from Windsor Road. Therefore adding the "background" to the predicted contributions from Windsor Road involves an element of double-counting and therefore an overestimation of impacts.

Tables 10 to **18** present the results of the air quality assessment using predicted traffic data and vehicles emission rates for the three scenarios considered. **Table 10** details the predicted 1-hour maxima for Windsor Road with no evacuation route constructed for year 2000 traffic data. **Table 11** details the predicted kerbside concentrations for Windsor Road and at various distances from Option 1 (arterial and flood evacuation route) for 2000 traffic data. **Table 12** details the predicted 1-hour maxima for kerbside concentrations along Windsor Road and at various distances from Option 2 (flood evacuation route and local road). **Tables 13** to **15** detail the concentrations of the three scenarios in 2006. **Tables 16** to **18** detail the three scenarios in 2016.

Table 10 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Windsor Road (without evacuation route) in 2000

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide* ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Macquarie Street to Pitt Town Road	0	2.88	65.9	0.26
	10	1.15	40.2	0.11
	50	0.58	27.8	0.06
Pitt Town Road to Groves Avenue	0	3.57	79.6	0.32
	10	2.19	69.7	0.18
	50	1.04	40.3	0.08
Groves Avenue to Park Road	0	2.88	64.2	0.26
	10	1.38	48.8	0.13
	50	0.81	34.3	0.06
Park Road to Level Crossing Road	0	2.30	53.6	0.21
	10	1.38	47.6	0.13
	50	0.69	31.1	0.06

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

Table 11 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Option 1 (arterial and flood evacuation route) and Windsor Road in 2000

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Windsor Road				
Macquarie Street to Pitt Town Road	0	1.27	26.9	0.11
Pitt Town Road to Groves Avenue	0	1.38	30.9	0.12
Groves Avenue to Park Road	0	1.50	33.2	0.13
Park Road to Level Crossing Road	0	1.50	34.6	0.14
Arterial and Flood Evacuation Route				
West of Bridge (i.e. from Forbes and Day Street Intersection)	0	0.69	15.3	0.06
	10	0.46	15.2	0.04
	50	0.23	11.7	0.02
Bridge Section	0	0.69	22.8	0.06
	10	0.81	25.0	0.06
	50	0.46	15.4	0.03
Bridge Section to Groves Avenue	0	0.58	15.1	0.06
	10	0.35	15.7	0.04
	50	0.23	10.4	0.02
Groves Avenue to Park Road	0	0.58	15.4	0.06
	10	0.46	16.0	0.04
	50	0.23	9.90	0.02
Park Road to Windsor Road Intersection	0	2.19	47.4	0.19
	10	1.27	40.9	0.11
	50	1.04	48.1	0.10

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

The height of the bridge in the proposed evacuation route is 17.3 metres AHD, which corresponds to the level of a 1 in 100-year flood. When modelling the impact of vehicle emissions from the bridge the maximum bridge height (allowed by the model) of 10 m was selected. There is therefore a significant height difference between the proposed and modelled bridge. The impact of a lower bridge height on ground level concentrations near the roadway would be greater than for the higher bridge which would allow more time for the pollutant to disperse. Therefore an assumed lower bridge height provides a conservative assessment.

The predicted 1-hour maxima (**Tables 11** and **12**) along Windsor road with the proposed evacuation route built are for concentrations at the edge of the road. These concentrations are the maximum that an individual in the vicinity of the road would be exposed to and they do not exceed the air quality goals outlined in **Table 1**. Note that there are no air quality goals for total hydrocarbons, and these results have been included to provide an estimate of exposure to the toxic components of hydrocarbons, in particular to benzene.

Table 12 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Option 2 (flood evacuation route and local road) and Windsor Road in 2000

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Windsor Road				
Macquarie Street to Pitt Town Road	0	2.53	55.5	0.22
Pitt Town Road to Groves Avenue	0	2.76	59.6	0.24
Groves Avenue to Park Road	0	2.99	64.5	0.26
Park Road to Level Crossing Road	0	3.22	72.6	0.29
Flood Evacuation Route				
West of Bridge	0	0.46	14.6	0.05
	10	0.23	10.5	0.02
	50	0.08	6.90	0.01
Bridge	0	0.58	17.8	0.06
	10	0.23	13.5	0.03
	50	0.08	10.8	0.02
East of Bridge to Mulgrave Road	0	0.58	16.1	0.05
	10	0.23	10.9	0.02
	50	0.08	7.59	0.01
Mulgrave road to Railway Road South	0	0.46	14.6	0.05
	10	0.23	10.7	0.02
	50	0.08	8.28	0.02

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

Table 13 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Windsor Road (without evacuation route) in 2006

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Macquarie Street to Pitt Town Road	0	2.53	64.2	0.22
	10	1.04	38.3	0.09
	50	0.58	29.0	0.05
Pitt Town Road to Groves Avenue	0	3.11	79.1	0.27
	10	1.73	65.7	0.15
	50	0.81	38.6	0.07
Groves Avenue to Park Road	0	2.53	64.2	0.22
	10	1.15	46.7	0.11
	50	0.58	33.4	0.05
Park Road to Level Crossing Road	0	2.07	53.6	0.19
	10	1.15	46.9	0.11
	50	0.58	30.4	0.05

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

Table 14 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Option 1 (arterial and flood evacuation route) and Windsor Road in 2006

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Windsor Road				
Macquarie Street to Pitt Town Road	0	1.15	28.8	0.10
Pitt Town Road to Groves Avenue	0	1.27	31.6	0.11
Groves Avenue to Park Road	0	1.27	32.8	0.11
Park Road to Level Crossing Road	0	1.27	33.4	0.12
Arterial and Flood Evacuation Route				
West of Bridge (i.e. from Forbes and Day Street Intersection)	0	0.58	17.3	0.06
	10	0.35	15.5	0.04
	50	0.12	11.7	0.02
Bridge Section	0	0.58	25.0	0.06
	10	0.69	25.9	0.06
	50	0.35	16.1	0.03
Bridge Section to Groves Avenue	0	0.46	16.2	0.05
	10	0.35	17.2	0.04
	50	0.12	10.8	0.02
Groves Avenue to Park Road	0	0.58	16.6	0.05
	10	0.46	17.4	0.04
	50	0.12	10.4	0.02
Park Road to Windsor Road Intersection	0	2.07	52.1	0.18
	10	1.04	39.8	0.09
	50	0.92	46.2	0.08

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

Table 15 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Option 2 (flood evacuation route and local road) and Windsor Road in 2006

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Windsor Road				
Macquarie Street to Pitt Town Road	0	1.70	54.1	0.19
Pitt Town Road to Groves Avenue	0	2.42	58.1	0.20
Groves Avenue to Park Road	0	2.30	59.8	0.21
Park Road to Level Crossing Road	0	2.42	64.2	0.22
Flood Evacuation Route				
West of Bridge	0	1.96	68.2	0.08
	10	0.81	48.3	0.04
	50	0.35	24.2	0.02

Bridge	0	2.07	70.9	0.10
	10	1.04	51.4	0.05
	50	0.35	22.1	0.03
East of Bridge to Mulgrave Road	0	1.04	33.0	0.09
	10	0.46	20.9	0.04
	50	0.23	15.9	0.02
Mulgrave road to Railway Road South	0	0.81	29.2	0.09
	10	0.35	18.8	0.04
	50	0.12	13.3	0.02

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

Table 16 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Windsor Road (without evacuation route) in 2016

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Macquarie Street to Pitt Town Road	0	2.65	71.2	0.30
	10	1.04	41.9	0.12
	50	0.58	32.0	0.07
Pitt Town Road to Groves Avenue	0	3.11	88.0	0.37
	10	1.73	70.7	0.20
	50	0.81	39.8	0.08
Groves Avenue to Park Road	0	2.53	71.2	0.30
	10	1.15	48.1	0.13
	50	0.58	34.7	0.06
Park Road to Level Crossing Road	0	1.96	55.7	0.23
	10	1.15	48.3	0.14
	50	0.58	31.3	0.07

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

Table 17 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Option 1 (arterial and flood evacuation route) and Windsor Road in 2016

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Windsor Road				
Macquarie Street to Pitt Town Road	0	1.04	29.6	0.09
Pitt Town Road to Groves Avenue	0	1.15	32.7	0.10
Groves Avenue to Park Road	0	1.27	33.7	0.11
Park Road to Level Crossing Road	0	1.15	33.9	0.11
Arterial and Flood Evacuation Route				
West of Bridge (i.e. from Forbes and Day Street Intersection)	0	0.58	18.1	0.05
	10	0.35	15.7	0.03
	50	0.12	12.7	0.02
Bridge Section	0	0.58	26.2	0.05
	10	0.69	26.7	0.05
	50	0.23	16.3	0.03
Bridge Section to Groves Avenue	0	0.46	16.7	0.05
	10	0.35	17.4	0.04
	50	0.12	10.8	0.02
Groves Avenue to Park Road	0	0.58	17.0	0.05

	10	0.35	17.8	0.04
	50	0.12	11.0	0.02
Park Road to Windsor Road Intersection	0	1.96	53.5	0.17
	10	0.92	40.4	0.08
	50	0.92	46.9	0.07

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

Table 18 – Predicted maximum 1-hour ground level concentrations of vehicle emissions along Option 2 (flood evacuation route and local road) and Windsor Road in 2016

Roadway Section	Distance from the kerbside (m)	Carbon monoxide (mg/m^3)	Nitrogen dioxide* ($\mu g/m^3$)	Hydrocarbons (mg/m^3)
Windsor Road				
Macquarie Street to Pitt Town Road	0	2.07	53.0	0.20
Pitt Town Road to Groves Avenue	0	2.53	57.2	0.21
Groves Avenue to Park Road	0	2.42	58.8	0.22
Park Road to Level Crossing Road	0	2.42	63.0	0.22
Flood Evacuation Route				
West of Bridge	0	0.81	67.4	0.08
	10	0.35	42.9	0.04
	50	0.12	23.9	0.02
Bridge	0	0.92	70.3	0.10
	10	0.46	50.9	0.05
	50	0.23	22.1	0.03
East of Bridge to Mulgrave Road	0	0.92	32.6	0.09
	10	0.35	20.7	0.04
	50	0.12	15.6	0.02
Mulgrave road to Railway Road South	0	0.81	28.9	0.08
	10	0.35	18.6	0.03
	50	0.12	13.3	0.02

*Assumed to be 10% by weight of total oxides of nitrogen at kerbside, 15% at 10m and 20% at 50m

9.2 Carbon monoxide

It can be seen from **Tables 10**, **11** and **12** that the highest predicted 1-hour carbon monoxide concentration 10 m from the roadway is 2.19 mg/m^3 in 2000, which occurs for the Windsor Road scenario (no evacuation route). This highest concentration occurred at Windsor Road between Pitt Town Road and Groves Avenue. The concentration at this location decreases to 1.73 mg/m^3 in 2006 and 2016. Of the two possible evacuation routes, the highest concentration are predicted to occur along Option 1 (arterial and evacuation route), between Park Road and the proposed Windsor Road Intersection in 2000. This concentration of 1.27 mg/m^3 (10 m from the roadway) decreases to 1.04 mg/m^3 and 0.92 mg/m^3 in 2006 and 2016, respectively. All predicted concentrations are well below the EPA's 1-hour goal of 31 mg/m^3, even when added to a background concentration of 8.8 mg/m^3, the maximum level measured at Blacktown from September 1996 to June 1998 (**Table 4**). The 1-hour goal is not expected to be exceeded.

9.3 Nitrogen Dioxide

Estimating nitrogen dioxide concentrations is more complicated than estimating carbon monoxide concentrations. As discussed previously, nitrogen oxides are initially emitted as a mixture of nitric oxide and other oxides of nitrogen, which are oxidised to nitrogen dioxide. At the point of emission the mixture is generally about 5% nitrogen dioxide by mass. However, while the maximum concentrations of total oxides of nitrogen generally occur during peak hour, this is not necessarily the case for nitrogen dioxide. An extensive

monitoring program undertaken by the RTA (**RTA, 1997**) indicates that during peak hour the percentage nitrogen dioxide at 10 m from the roadway edge is likely to be about 5%. The conversion rate from nitric oxide to nitrogen dioxide at other times of the day may be significantly higher than 5% although the total oxides of nitrogen levels may be significantly lower than peak hour levels. It is necessary therefore to assume some intermediate value for a worst-case assessment.

Data from the RTA program indicates that at 10 m from the roadway a conversion rate of 15% by weight is still conservative but more realistic than the 20% assumed in previous EIS studies. At distances of 30-60 m from the kerbside the 20% conversion rate appears to be appropriate. There are no monitoring data for the kerbside location in the present study, but it is considered that a 15% conversion rate at 10 m is likely to still be conservative. Conversion rates of 10% and 15% have been used at 0 m and 10 m respectively, while a rate of 20% has been assumed for the remaining distance of 50 m.

Taking this into consideration, the highest predicted 1-hour nitrogen dioxide concentration 10 m from the two possible evacuation routes occurred along Option 2 (flood evacuation route and local road) in 2006 and 2016. In 2006, the predicted concentration was 51.4 $\mu g/m^3$ and in 2016 the predicted concentrations was 50.9 $\mu g/m^3$. All of the predicted values 10 m from the proposed evacuation route and resultant Windsor Road traffic are below the NEPM goal of 245 $\mu g/m^3$. When added to the maximum measured 1-hour average nitrogen dioxide concentration of 131.2 ug/m^3 (**Table 4**), the levels remain below the NEPM goal.

It should be noted that predictions do not take fully into account the benefits of three-way catalytic converters which are fitted to all new (fuel-injected) petrol-fuelled passenger vehicles in Australia and which substantially reduce oxides of nitrogen emissions. Nor (more significantly) do they take account of controls on diesel vehicles which are being implemented through new design rule ADR-70. Heavy vehicles comprise a significant proportion of vehicles in this assessment, so any advances in technology which help reduce emissions from diesel vehicles will significantly reduce roadside oxides of nitrogen levels.

The NEPM standard for nitrogen dioxide adopted by the NSW EPA, would currently be difficult to achieve in many areas of Australia, most notably in the vicinity of busy roads carrying a high percentage of heavy diesel vehicles. The approach developed by NEPC is that compliance with the standard will be achieved over a 10-year time frame through a raft of measures including source control. These controls are to be left to individual jurisdictions to determine and implement. This approach is reflected in the NSW EPA Air Quality Management Plan (**EPA, 1998**), which targets diesel emissions and heavy industry.

It is therefore reasonable to assume that by 2016, the fleet emissions will be substantially lower than those assumed in the modelling presented in this report. It is expected that despite the projected traffic growth, pollutant levels will not exceed acceptable levels.

9.4 Hydrocarbons

Hydrocarbon concentrations are no longer specified in the EPA's air quality goals. This is largely due to the fact that a simple hydrocarbon concentration goal is now recognised as not being useful for the purposes of assessing health impacts or identifying the need for air quality management requirements. More detailed information on specific hydrocarbons is required. As noted in **Section 3** hydrocarbons, in particular those associated with motor vehicles, are a common contaminant of urban atmospheres. Emission controls on Australian cars and equivalents since 1978 have resulted in a considerable reduction in both evaporative and exhaust emissions of hydrocarbons.

One of the components of hydrocarbons that has become a concern in the community is benzene, which is a known carcinogen (**WHO 1995**). **Attachment B** (data from **Nelson and Quigley, 1982**) shows an analysis of the hydrocarbon content of fuel and exhaust. It can be seen that benzene comprise approximately 2.6% of petrol. It can also be seen that the percentage benzene (by mass) in vehicle exhausts was approximately 5% (note these data relate to leaded petrol, but there has not been any substantial change in the benzene content with the introduction of unleaded petrol).

The Victorian EPA 3-minute design ground-level concentration for benzene is 0.10 mg/m^3 (0.033 ppm), but this goal explicitly excludes petrol and liquid mixtures containing 1% or less of benzene.

From **Tables 10** to **18** the predicted maximum 1-hour increase in total hydrocarbons is of the order of 0.29 mg/m^3 at 10 m from the kerb. Assuming a 5% benzene composition in the exhaust the benzene concentration at 10 m from the kerb would be approximately 0.0145 mg/m^3 or 14.5 $\mu g/m^3$ (1-hour average). This is only 15% of the Victorian EPA design 3-minute goal, which as noted before specifically excludes petrol. These levels are also below the proposed UK goal of 16 $\mu g/m^3$, which is a long term reporting goal.

It should be emphasized that this is a conservative prediction. The long term concentrations would be substantially less than the short-term peaks as they are averaged over conditions which include much lower traffic flows and much better atmospheric dispersion. Concentrations would of course be substantially lower at the locations of residences. It is nevertheless recognised that there may be no safe limit for benzene, but the risks to a particular individual over a lifetime is small, and on a population and individual basis would be offset by lower risks through safer roads, although to an undetermined extent.

The current understanding of long-term health risks is that they are a function of average lifetime exposure levels. As discussed in **Section 3**, WHO estimate that the risk of developing leukaemia with a lifetime exposure to ambient concentrations of 1 $\mu g/m^3$ of benzene is $4.4 \times 10^{-6} - 7.5 \times 10^{-6}$. This is referred to as a unit risk estimate.

Levels of benzene can range from 3 $\mu g/m^3$ in a rural environment to up to 160 $\mu g/m^3$ in an urban environment (higher close to service stations and storage tanks) with typical levels of about 20 $\mu g/m^3$ in residential areas. Most people would be exposed to much higher levels on occasions, for example when filling their car with petrol or in a smoke-filled indoor environment. The daily intake from ambient air in an urban environment is estimated to range from 30 to 300 μg and smoking one pack of cigarettes per day can add approximately 600 μg to this intake.

9.5 Particulate Matter

CAL3QHC was used to assess the 24-hour and annual average PM_{10} concentrations for Windsor Road alone and for Windsor Road with the two possible evacuation routes. The current particulate matter (PM_{10}) goals are based on 24-hour and annual averaging times. These goals are 50 $\mu g/m^3$ for the 24-hour goal and 30 $\mu g/m^3$ for the annual goal. The results of modelling particulate concentrations over these two averaging periods are shown in **Table 19**.

Table 19 – Predicted maximum 24-hour and annual average PM_{10} concentrations due to vehicle emissions in 2000, 2006 and 2016

Roadway Option	Distance from the kerb (m)	24-hour average ($\mu g/m^3$)			Annual average ($\mu g/m^3$)		
		2000	2006	2016	2000	2006	2016
Windsor Road (without evacuation route)	0	12.1	11.8	12.8	7.0	6.6	7.1
	10	8.5	8.0	8.6	3.9	3.7	3.9
	50	3.1	2.9	3.1	1.4	1.3	1.4
Option 1							
Windsor Road	0	7.7	7.1	7.6	3.5	3.3	3.5
Arterial and flood evacuation route	0	10.7	10.6	11.3	5.5	5.2	5.5
	10	5.6	5.6	5.9	2.8	2.6	2.8
	50	4.9	4.6	4.9	2.2	2.0	2.2
Option 2							
Windsor Road	0	13.3	11.3	12.4	5.6	4.8	5.2
Flood evacuation route and local road	0	3.9	6.8	6.8	1.4	2.4	2.5
	10	1.9	3.1	3.1	0.6	1.1	1.1
	50	0.9	1.4	1.4	0.3	0.5	0.5

The highest predicted 24-hour average for particulate matter was 13.3 $\mu g/m^3$, which occurred along Windsor Road (in Option 2) between Park Road and Level Crossing Road. This predicted concentration is approximately 27% of the 24-hour NEPM goal of 50 $\mu g/m^3$. Adding this to the maximum background concentration at Richmond of 39.0 $\mu g/m^3$ (**Table 4**) would cause an exceedance of the 24-hour goal. However, it is extremely unlikely that anyone would be exposed for 24 hiurs at this location, which is essentially kerbside. Adding maximum predicted to maximum measured is also a very conservative approach. The NHMRC 90 $\mu g/m^3$ (annual average) TSP goal is unlikely to be exceeded as is the proposed annual PM_{10} goal of 30 $\mu g/m^3$, as annual average PM_{10} levels are predicted to be less than 4 $\mu g/m^3$ at all distances from the kerb (**Table 19**).

The predictions do not take account of the newly introduced controls on diesel emissions, which include reducing the sulphur content in diesel fuel. These changes should have a significant effect on the heavy-duty fleet by 2016. It must also be remembered that the background level includes emissions from traffic already in the area, and there is therefore an element of double counting when adding these to predicted concentrations. Further, the EPA PM_{10} goal is a regional target and will not be met everywhere is Sydney, particularly next to busy roads.

Other Air Quality Impacts

The roadway impacts discussed above are the major roadside impacts that would occur from the operation of the proposed road. There are other air pollutants associated with roadways as discussed in **Section 4**, however these do not play a major role in kerbside impacts and have not been considered quantitatively in this report.

By the year 2016, the percentage of vehicles without catalytic converters will be minimal and lead will no longer be a significant emission from motor vehicles. As discussed, lead is now below 0.005 g/l in fuel for vehicles without catalytic converters. Similarly, Australian petrol has a low sulphur content and sulphur dioxide is not a major emission from motor vehicles.

The other major pollutant associated with roadways is ozone. However this (as discussed earlier) is a secondary pollutant formed by the interaction of hydrocarbons and nitrogen oxides in the presence of sunlight and is a regional air quality issue.

9.6 Construction impacts

Dust would be generated from earthworks associated with the construction of the new road. The total amount of dust generated would depend on the silt and moisture content of the soil and the types of operations being carried out.

Estimates of dust emissions from construction operations can be made using emission factors developed by the **SPCC (1983)** (now EPA) and the **US EPA (1985)**. These emission factors relate to the amount of dust generated by different types of equipment and operations associated with construction work. The most likely equipment to be used in the project includes bulldozers, backhoes, rollers, scrapers, excavators, pavers, concrete trucks, mobile cranes, truck-mounted boring rig, jackhammers and haul trucks. The major sources of dust would be the bulldozers, excavators, scrapers and wind erosion. An estimate of the amount of dust generated by each operation per day is summarised below. [The details of construction activities are not known at this stage, and so the following is given as an example of activities that are likely to take place].

- Three bulldozers, assuming ten-hours of operation per day generating dust at the rate of 2.75 kg/h, would give a total of 82.5 kg/day.
- Dust from loading of material by excavator to trucks assuming dust is generated at the rate of 0.01 kg/t and that 50 x 15 cubic metre trucks loads are removed in a ten-hour day, making a total of approximately 750 m^3 or approximately 1125 t of material removed per day gives a total dust generation of 11.25 kg/day.
- Scrapers generate dust at a rate of between 3 to 9 kg/kilometre of travel depending on soil silt content, soil moisture and the weight of the scraper. Watering the travelling surface would reduce this emission by between 50 and 80%. A scraper working for ten hours could be expected to travel approximately 70 to 100 km. Taking account of laden and unladen trips and 50% dust control the scraper would be expected to generate up to 300 kg of dust in a working day. With intensive watering of the travelling surface this could be reduced to 120 kg/day per scraper. A total of three scrapers would give 360 kg/day.
- Dust from trucks travelling on the unsealed road surface assuming a 400 m round trip distance, five movements per hour and 2 kg of dust/vehicle/km (after taking account of dust suppression by watering of the trafficked areas) would generate 40 kg during a ten-hour working shift.
- Dust from wind erosion from an exposed area of 200 m long by 30 m wide (the exposed area would be greater than this but the area which could contribute significant amounts of dust to a particular location would be unlikely to be larger) assuming an erosion rate of 0.4 kg/ha/hour gives a total of 2.4 kg in ten hours.

Thus, for this example, the total dust generated in a ten-hour working day would be expected to be approximately 500 kg. On a hot, dry, windy day the amount of dust from wind erosion could be much higher, and would have to be controlled using water sprays. It is possible that under some extreme wind conditions, construction activities would have to be stopped.

The appropriate air quality goal for determining impacts from construction work is the US EPA 24-hour goal for PM_{10} of 150 $\mu g/m^3$ (formerly noted by the NSW EPA). This is approximately equivalent to the former EPA 24-hour goal of 260 $\mu g/m^3$ for total suspended

particulate matter (TSP). It is still useful to refer to this goal as many historical measurements are for TSP. The US EPA PM_{10} goal is considered to be appropriate in this instance as the emitted dust is of crustal origin and is likely to contain only a small fraction of $PM_{2.5}$ which is the particulate fraction of concern. When assessing particulate emissions from combustion processes (such as roadway emissions) which have a high $PM_{2.5}$ content, the more stringent PM_{10} goal of 50 $\mu g/m^3$ is appropriate.

Previous dispersion modelling studies (**Stephenson, 1991**) indicate that high levels of dust generation associated with road construction work, can result in short-term dust impacts. That is, exceedances of the 24-hour air quality goal for TSP, out to 600 m or more under "worst-case" dispersion conditions and out to 300 m for typical conditions.

In addition to the goals for ambient dust concentrations it is appropriate to consider goals for dust deposition during the construction period. The EPA consider that residential areas begin to experience dust related nuisance impacts when annual average dust (insoluble solids) deposition levels exceed 4 g/m^2/month, and that dust impacts would be at unacceptable levels when they reached 10 g/m^2/month (**SPCC 1983**). In the early 1990s the EPA (**Dean et al., 1990**) refined these criteria. They are now expressed in terms of an acceptable increase in dust deposition over the existing background. For example, in residential areas with annual average deposition levels of between 0 and 2 g/m^2/month, an increase of up to 2 g/m^2/month would be permitted before it is considered that a significant degradation of air quality has occurred. **Table 20** shows the maximum acceptable increase in dust deposition over the existing dust levels.

Dust levels in the area are likely to be of the order of 1-2 g/m^2/month. Therefore an increase of 2 g/m^2/month could occur before there was a perceptible degradation in air quality.

Table 20– NSW EPA goals for dust deposition (insoluble solids)

Pollutant	Averaging period	Maximum increase in deposited dust level	Maximum total deposited dust level
Deposited dust	Annual	2 g/m^2/month	4 g/m^2/month

As construction is likely to continue over several months, it is important that exposed areas be stabilised quickly and that appropriate dust suppression methods be used to keep dust impacts to a minimum. Due to the majority of the construction taking place at significant distances from residences, (i.e. on the South Creek floodplain) monitoring of deposition rates during the construction phase would not be required.

Exhaust emissions from construction equipment are not likely to be a significant issue as the equipment would be largely spread out and not operating at all hours. They are likely to be considerably less than those from the vehicle fleet using the proposed roadway on completion.

Greenhouse issues

The temperature of the earth's atmosphere is determined by the balance between incoming solar radiation and the loss of heat energy by radiation from the earth and atmosphere to outer space. This balance is in turn affected by a complex set of processes, acting on a global scale, which control the way in which heat is transported around the earth by winds and ocean currents, and by the quantities of energy that are reflected and absorbed by the earth's surface. While the broad principles of the way in which these processes work to control the temperature of the earth's atmosphere are understood, the details, which may

well be very important in determining the final temperature that is achieved at the earth's surface, are still the subject of scientific research.

One of the important factors in determining the amount of radiant energy absorbed in the atmosphere is the concentration of carbon dioxide. Changes in this concentration are likely to cause changes in the temperature of the earth's atmosphere near the earth's surface. Increases in carbon dioxide concentration are expected to cause increases in temperature.

Australia is signatory to the "International Frame-work Convention on Climate Change" (Rio Convention), which commits Australia to programs of monitoring and reporting on greenhouse gas emissions. A target of the Rio Convention is that signatory countries should attempt to reduce greenhouse gas emissions to the levels that applied in 1990. At the Kyoto meeting the convention agreed that Australia would be allowed to reduce their emissions to 8% above the 1990 level, between 2008 and 2012.

The RTA is committed to ensuring that its environmental goals and policies are consistent with those outlined in the 1992 Intergovernmental Agreement on the Environment. This agreement addresses a number of globally important environmental issues including the greenhouse effect. This commitment is facilitated through the RTA's environmental vision which addresses greenhouse gas emissions and also energy consumption.

Approximately 14% of NSW's total carbon dioxide emissions are estimated to come from the transport sector (**EPA, 1995**). At a broad level, the RTA has been involved in and implemented several strategic initiatives to address the issue of road transport related greenhouse gas emissions. These are:

- National Greenhouse Response Strategy

  This strategy was adopted by the Council of Australian Governments in 1992 and aims to contribute to the national commitment to the National Strategy for Ecologically Sustainable Development. The RTA contributed to the development of this strategy and is the NSW representative on the Transport Working Group for the development of a greenhouse gas emissions inventory. With respect to transport, the response strategies include reducing fuel consumption in motorised transport; improving the technical and economic efficiency of operation of the road network and traffic management; and to encourage the use of bicycles. This proposal contributes to these initiatives.

- RTA Greenhouse Reduction Plan

  The RTA has prepared a plan at a strategic level to address and provide policy in relation to greenhouse gas emissions resulting from its activities. A greenhouse emissions inventory of RTA activities has been carried out (**Beer et al., 1996**) and the findings incorporated into the RTA Greenhouse Inventory.

Emissions of carbon dioxide from motor vehicles are directly proportional to fuel consumption. They cannot be reduced by emission control technologies except where they result in an improvement in fuel consumption. RTA programs which encourage better vehicle maintenance and hence better fuel economy will be beneficial.

The RTA also continues to engage in other strategies to encourage the tightening of vehicle emissions standards. These include;

- Enhancing the State's vehicle emissions enforcement resources;
- Continuing its role on MVEC (Motor Vehicle Environment Committee) to encourage the early implementation of more stringent Australian Design Rules, including the revision of

ADR 37/01 "Emission Control for Light Vehicles" and ADR70 "Exhaust Emission Control for Diesel Engined Vehicles". The new ADRs, 79/00 "Emissions Control for Light Vehicles" and 80/00 "Emissions Control for Heavy Vehicles" include for the first time, vehicles which operate on liquefied petroleum gas or natural gas. Combined with the new (1 January 2002) Fuel Standard (Petrol) Determination 2001 and the Fuel Standard (Diesel) Determination 2001 (which reduces the sulphur levels in road transport diesel fuel from the current level of 1300 parts per million (ppm) to 500 ppm by January 2003), these new ADRs will facilitate introduction of more advanced emission control technologies; and

- Playing a key role in the development of the Diesel National Environment Protection Measure (NEPM), which came into effect 29 June 2001. This set a framework for the management of emissions, enabled the development of regulations on diesel emissions testing standards, and facilitates the development and implementation of enforcement and alternative compliance strategies for vehicle emissions.

An estimate of greenhouse emissions for the project has been undertaken. An estimate for the fuel used for the construction phase of the project and the fuel saved at various stages of the project is presented in **Table 21**. The information on fuel used during construction was provided by Connell Wagner. The information on fuel saved during operation was obtained from output from the traffic model. The fuel saved is the difference between the fuel used with the bypass built and the "do-nothing" case. Also presented are associated emissions of carbon dioxide assuming an emission rate of 2714 g of CO_2 per litre of fuel burnt (**Beer, 1994**).

Table 21 - Estimate of fuel savings and greenhouse emissions for the project

	Fuel used/saved	CO_2 emitted/saved
Construction phase	0.089 Ml used	0.242 kt used
First year with evacuation route built	2.5472 Ml saved	6.91 kt saved
2006 with evacuation route built	1.8342 Ml saved	4.98 kt saved
2016 with evacuation built	2.0234 Ml saved	5.49 kt saved

On this basis, during the first year of operation, there will be a net saving of 6.668 kt of CO_2 emissions compared to the no-build case (6.91-0.242). In subsequent years there will be an average savings of about 5 kt per year of emissions.

10. CONCLUSIONS

The results of the air quality assessment undertaken for this report conclude that:

1. Due to the present emission controls on motor vehicles and the projected traffic conditions for the years 2000, 2006 and 2016, the EPA's carbon monoxide 1-hour or 8-hour goals are not expected to be exceeded for any section of Windsor Road or either of the proposed evacuation routes.
2. The historical PM_{10} annual and 24-hour maximum air quality goals are not predicted to be exceeded by the project, nor are the contemporary NEPM goals in any areas where people would spend a significant amount of time.
3. The predicted increases in concentration of nitrogen dioxide indicate that the NEPM newly adopted EPA goals (both short-term and long-term reporting) would not be exceeded at 10 m from the kerb along any section of the route. This distance is used to assess the impacts at locations where people could be reasonably expected to spend sign cant amounts of time.
4. Predicted concentrations of benzene (and other pollutants) are not at levels which, from current understanding, should pose health effects. However, it is recognised that there may be no safe level for exposure to benzene and it is prudent to minimise risk.

In terms of air quality, Option 1 (arterial and flood evacuation route) would result in lower pollutant concentrations along a large proportion of Windsor Road on a long term basis. This evacuation route, which has the additional function of serving as an arterial road, takes more traffic from Windsor Road than Option 2. When Option 1 is in operation, vehicle emissions in the Windsor area are spread over a larger area and are able to disperse into a greater volume of air resulting in lower concentrations along sections of Windsor Road.

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ATTACHMENT A
JOINT WIND SPEED, WIND DIRECTION AND STABILITY CLASS FREQUENCY TABLES

FROM 84. 2. 7 TO 84.12.31

PASQUILL STABILITY CLASS "A"

WIND SPEED CLASS (MPS)

WIND SECTOR	.51 TO 1.50	1.51 TO 3.00	3.01 TO 4.50	4.51 TO 6.00	6.01 TO 7.50	7.51 TO 9.00	9.01 TO 10.50	GREATER THAN 10.50	TOTAL	MEAN SPEED
NNE	.001723	.003133	.000000	.000000	.000000	.000000	.000000	.000000	.004856	1.81
NE	.001410	.005796	.000000	.000000	.000000	.000000	.000000	.000000	.007206	2.13
ENE	.000783	.003603	.000000	.000000	.000000	.000000	.000000	.000000	.004386	2.11
E	.000470	.002036	.000000	.000000	.000000	.000000	.000000	.000000	.002506	2.01
ESE	.001253	.003133	.000000	.000000	.000000	.000000	.000000	.000000	.004386	1.90
SE	.000627	.001253	.000000	.000000	.000000	.000000	.000000	.000000	.001880	2.02
SSE	.001096	.000940	.000000	.000000	.000000	.000000	.000000	.000000	.002036	1.66
S	.000157	.001410	.000000	.000000	.000000	.000000	.000000	.000000	.001566	2.29
SSW	.001253	.000940	.000000	.000000	.000000	.000000	.000000	.000000	.002193	1.41
SW	.000940	.000627	.000000	.000000	.000000	.000000	.000000	.000000	.001566	1.47
WSW	.000157	.000157	.000000	.000000	.000000	.000000	.000000	.000000	.000313	1.20
W	.001253	.000157	.000000	.000000	.000000	.000000	.000000	.000000	.001410	1.02
WNW	.000940	.000940	.000000	.000000	.000000	.000000	.000000	.000000	.001880	1.57
NW	.000940	.000313	.000000	.000000	.000000	.000000	.000000	.000000	.001253	1.31
NNW	.000940	.000627	.000000	.000000	.000000	.000000	.000000	.000000	.001566	1.39
N	.001410	.000157	.000000	.000000	.000000	.000000	.000000	.000000	.001566	1.12
CALM									.003133	
TOTAL	.015351	.025219	.000000	.000000	.000000	.000000	.000000	.000000	.043703	1.70

NUMBERS BELOW BASED ON ALL OBSERVATIONS
NUMBER OF INVALID OBSERVATIONS = 0
NUMBER OF VALID STABILITY DEPENDENT OBSERVATIONS = 6384

PASQUILL STABILITY CLASS "B"

WIND SPEED CLASS (MPS)

WIND SECTOR	.51 TO 1.50	1.51 TO 3.00	3.01 TO 4.50	4.51 TO 6.00	6.01 TO 7.50	7.51 TO 9.00	9.01 TO 10.50	GREATER THAN 10.50	TOTAL	MEAN SPEED
NNE	.008302	.006266	.004386	.000157	.000000	.000000	.000000	.000000	.019110	2.05
NE	.010965	.013784	.005482	.000157	.000000	.000000	.000000	.000000	.030388	2.03
ENE	.009085	.007832	.002976	.000000	.000000	.000000	.000000	.000000	.019893	1.86
E	.006736	.004699	.002350	.000000	.000000	.000000	.000000	.000000	.013784	1.90
ESE	.003759	.003289	.003289	.000313	.000000	.000000	.000000	.000000	.010652	2.32
SE	.001723	.002976	.002506	.000000	.000000	.000000	.000000	.000000	.007206	2.36
SSE	.001566	.002820	.002350	.000157	.000000	.000000	.000000	.000000	.006892	2.48
S	.002976	.004543	.005169	.000313	.000000	.000000	.000000	.000000	.013001	2.71
SSW	.002663	.002976	.002193	.000313	.000000	.000000	.000000	.000000	.008145	2.26
SW	.002350	.003446	.001410	.000157	.000000	.000000	.000000	.000000	.007362	2.14
WSW	.003446	.003603	.000783	.000157	.000000	.000000	.000000	.000000	.007989	1.92
W	.004229	.002036	.000940	.000157	.000000	.000000	.000000	.000000	.007362	1.82
WNW	.004856	.002193	.000783	.000000	.000000	.000000	.000000	.000000	.007832	1.59
NW	.003603	.001253	.001096	.000000	.000000	.000000	.000000	.000000	.005952	1.67
NNW	.002506	.000627	.000783	.000000	.000000	.000000	.000000	.000000	.003916	1.77
N	.002663	.003133	.000783	.000000	.000000	.000000	.000000	.000000	.006579	1.97
CALM									.006736	
TOTAL	.071429	.065476	.037281	.001880	.000000	.000000	.000000	.000000	.182801	2.00

NUMBERS BELOW BASED ON ALL OBSERVATIONS
NUMBER OF INVALID OBSERVATIONS = 0
NUMBER OF VALID STABILITY DEPENDENT OBSERVATIONS = 6384

PASQUILL STABILITY CLASS "C"

WIND SPEED CLASS (MPS)

WIND SECTOR	.51 TO 1.50	1.51 TO 3.00	3.01 TO 4.50	4.51 TO 6.00	6.01 TO 7.50	7.51 TO 9.00	9.01 TO 10.50	GREATER THAN 10.50	TOTAL	MEAN SPEED
NNE	.004229	.001566	.002663	.000627	.000470	.000000	.000000	.000000	.009555	2.58
NE	.004856	.004386	.002820	.002036	.000000	.000000	.000000	.000000	.014098	2.52
ENE	.004229	.004386	.002036	.001253	.000000	.000000	.000000	.000000	.011905	2.49
E	.003133	.002663	.001723	.000627	.000157	.000000	.000000	.000000	.008302	2.39
ESE	.001410	.000940	.002193	.002820	.000470	.000470	.000000	.000000	.008302	4.06
SE	.000783	.003446	.001880	.002506	.000470	.000000	.000000	.000000	.009085	3.49
SSE	.000783	.002036	.002663	.001096	.000157	.000157	.000157	.000000	.007049	3.65
S	.001723	.003759	.005013	.003916	.000313	.000157	.000157	.000000	.015038	3.67
SSW	.001723	.002193	.003603	.000627	.000157	.000157	.000000	.000000	.008459	2.93
SW	.001253	.001566	.001566	.000940	.000470	.000157	.000000	.000157	.006109	3.53
WSW	.001723	.002193	.000470	.002663	.000627	.000627	.000470	.000627	.009398	4.64
W	.002976	.001566	.001096	.001566	.000157	.000157	.000000	.000313	.007832	3.13
WNW	.002663	.000470	.001096	.001253	.000000	.000000	.000000	.000000	.005482	2.66
NW	.002036	.001566	.000627	.000470	.000000	.000627	.000000	.000000	.005326	3.01
NNW	.001253	.000470	.000627	.000470	.001096	.000157	.000157	.000313	.004543	4.59
N	.001096	.000940	.000940	.000470	.000627	.000000	.000000	.000000	.004073	3.27
CALM									.004073	
TOTAL	.035871	.034148	.031015	.023340	.005169	.002663	.000940	.001410	.138628	3.15

NUMBERS BELOW BASED ON ALL OBSERVATIONS
NUMBER OF INVALID OBSERVATIONS = 0
NUMBER OF VALID STABILITY DEPENDENT OBSERVATIONS = 6384

PASQUILL STABILITY CLASS "D"

WIND SPEED CLASS (MPS)

WIND SECTOR	.51 TO 1.50	1.51 TO 3.00	3.01 TO 4.50	4.51 TO 6.00	6.01 TO 7.50	7.51 TO 9.00	9.01 TO 10.50	GREATER THAN 10.50	TOTAL	MEAN SPEED
NNE	.000000	.000313	.001096	.000940	.000783	.000157	.000000	.000000	.003289	5.13
NE	.000000	.001566	.001096	.001566	.000470	.000313	.000000	.000000	.005013	4.44
ENE	.000000	.000940	.000940	.002506	.001880	.000627	.000000	.000000	.006892	5.52
E	.000000	.001410	.001253	.003289	.003916	.000627	.000000	.000000	.010495	5.48
ESE	.000000	.001253	.000627	.002193	.004229	.002193	.000157	.000000	.010652	6.14
SE	.000000	.000470	.000783	.002350	.005482	.001253	.000157	.000157	.010652	6.36
SSE	.000000	.002036	.001096	.001880	.001410	.000940	.000000	.000000	.007362	4.89
S	.000000	.001880	.002820	.002976	.001880	.000313	.000000	.000000	.009868	4.78
SSW	.000000	.001253	.001723	.001096	.002193	.000783	.000313	.000000	.007362	5.39
SW	.000000	.000940	.001096	.000470	.002036	.001096	.001096	.000313	.007049	6.57
WSW	.000000	.000627	.000470	.002350	.007206	.006422	.004073	.005482	.026629	8.51
W	.000000	.000783	.000783	.002663	.007362	.006579	.005326	.006736	.030232	8.60
WNW	.000000	.000470	.001096	.000940	.002506	.002193	.001566	.001880	.010652	7.98
NW	.000000	.000157	.000627	.001566	.002350	.001566	.000627	.000627	.007519	7.12
NNW	.000000	.000313	.000313	.000783	.000783	.000940	.001253	.000157	.004543	7.19
N	.000000	.000470	.000313	.001096	.001566	.000627	.000157	.000000	.004229	5.99
CALM									.000000	
TOTAL	.000000	.014881	.016134	.028665	.046053	.026629	.014724	.015351	.162437	6.90

NUMBERS BELOW BASED ON ALL OBSERVATIONS
NUMBER OF INVALID OBSERVATIONS = 0
NUMBER OF VALID STABILITY DEPENDENT OBSERVATIONS = 6384

PASQUILL STABILITY CLASS "E"

WIND SPEED CLASS (MPS)

WIND SECTOR	.51 TO 1.50	1.51 TO 3.00	3.01 TO 4.50	4.51 TO 6.00	6.01 TO 7.50	7.51 TO 9.00	9.01 TO 10.50	GREATER THAN 10.50	TOTAL	MEAN SPEED
NNE	.000000	.000313	.002663	.002036	.000000	.000000	.000000	.000000	.005013	4.35
NE	.000000	.000313	.001880	.000470	.000000	.000000	.000000	.000000	.002663	3.97
ENE	.000000	.000470	.002193	.003603	.000000	.000000	.000000	.000000	.006266	4.56
E	.000000	.000940	.002350	.002350	.000000	.000000	.000000	.000000	.005639	4.27
ESE	.000000	.000783	.002506	.002193	.000000	.000000	.000000	.000000	.005482	4.29
SE	.000000	.000313	.001410	.002663	.000000	.000000	.000000	.000000	.004386	4.51
SSE	.000000	.000783	.003133	.001566	.000000	.000000	.000000	.000000	.005482	4.04
S	.000000	.000940	.002036	.001096	.000000	.000000	.000000	.000000	.004073	3.81
SSW	.000000	.001410	.002976	.000627	.000000	.000000	.000000	.000000	.005013	3.57
SW	.000000	.001096	.001880	.000627	.000000	.000000	.000000	.000000	.003603	3.73
WSW	.000000	.001253	.003916	.002663	.000000	.000000	.000000	.000000	.007832	4.15
W	.000000	.000470	.003759	.005796	.000000	.000000	.000000	.000000	.010025	4.58
WNW	.000000	.000313	.002820	.001096	.000000	.000000	.000000	.000000	.004229	4.09
NW	.000000	.000627	.001253	.001566	.000000	.000000	.000000	.000000	.003446	4.27
NNW	.000000	.000000	.002036	.001253	.000000	.000000	.000000	.000000	.003289	4.38
N	.000000	.000313	.002036	.001096	.000000	.000000	.000000	.000000	.003446	4.18
CALM									.000000	
TOTAL	.000000	.010338	.038847	.030702	.000000	.000000	.000000	.000000	.079887	4.21

NUMBERS BELOW BASED ON ALL OBSERVATIONS
NUMBER OF INVALID OBSERVATIONS = 0
NUMBER OF VALID STABILITY DEPENDENT OBSERVATIONS = 6384

PASQUILL STABILITY CLASS "F"

WIND SPEED CLASS (MPS)

WIND SECTOR	.51 TO 1.50	1.51 TO 3.00	3.01 TO 4.50	4.51 TO 6.00	6.01 TO 7.50	7.51 TO 9.00	9.01 TO 10.50	GREATER THAN 10.50	TOTAL	MEAN SPEED
NNE	.013158	.006266	.000783	.000000	.000000	.000000	.000000	.000000	.020207	1.45
NE	.023810	.011435	.002820	.000000	.000000	.000000	.000000	.000000	.038064	1.53
ENE	.019893	.012061	.001880	.000000	.000000	.000000	.000000	.000000	.033835	1.55
E	.013001	.009555	.001566	.000000	.000000	.000000	.000000	.000000	.024123	1.65
ESE	.008302	.006109	.001880	.000000	.000000	.000000	.000000	.000000	.016291	1.74
SE	.007989	.008459	.001566	.000000	.000000	.000000	.000000	.000000	.018014	1.78
SSE	.006266	.009555	.001880	.000000	.000000	.000000	.000000	.000000	.017701	1.90
S	.007519	.013784	.003603	.000000	.000000	.000000	.000000	.000000	.024906	2.05
SSW	.010025	.015508	.003603	.000000	.000000	.000000	.000000	.000000	.029135	1.99
SW	.014411	.018484	.002976	.000000	.000000	.000000	.000000	.000000	.035871	1.81
WSW	.009712	.013941	.000940	.000000	.000000	.000000	.000000	.000000	.024593	1.71
W	.013941	.009555	.002820	.000000	.000000	.000000	.000000	.000000	.026316	1.72
WNW	.016134	.005482	.001253	.000000	.000000	.000000	.000000	.000000	.022870	1.45
NW	.010965	.004386	.001880	.000000	.000000	.000000	.000000	.000000	.017231	1.50
NNW	.008302	.005326	.000783	.000000	.000000	.000000	.000000	.000000	.014411	1.63
N	.004386	.002193	.000470	.000000	.000000	.000000	.000000	.000000	.007049	1.47
CALM									.021930	
TOTAL	.187813	.152099	.030702	.000000	.000000	.000000	.000000	.000000	.392544	1.61

NUMBERS BELOW BASED ON ALL OBSERVATIONS
NUMBER OF INVALID OBSERVATIONS = 0
NUMBER OF VALID STABILITY DEPENDENT OBSERVATIONS = 6384

ALL PASQUILL STABILITY CLASSES

WIND SPEED CLASS (MPS)

WIND SECTOR	.51 TO 1.50	1.51 TO 3.00	3.01 TO 4.50	4.51 TO 6.00	6.01 TO 7.50	7.51 TO 9.00	9.01 TO 10.50	GREATER THAN 10.50	TOTAL	MEAN SPEED
NNE	.027412	.017857	.011591	.003759	.001253	.000157	.000000	.000000	.062030	2.27
NE	.041040	.037281	.014098	.004229	.000470	.000313	.000000	.000000	.097431	2.09
ENE	.033991	.029292	.010025	.007362	.001880	.000627	.000000	.000000	.083177	2.35
E	.023340	.021303	.009242	.006266	.004073	.000627	.000000	.000000	.064850	2.66
ESE	.014724	.015508	.010495	.007519	.004699	.002663	.000157	.000000	.055764	3.30
SE	.011122	.016917	.008145	.007519	.005952	.001253	.000157	.000157	.051222	3.36
SSE	.009712	.018170	.011122	.004699	.001566	.001096	.000157	.000000	.046523	2.97
S	.012375	.026316	.018640	.008302	.002193	.000470	.000157	.000000	.068452	3.03
SSW	.015664	.024279	.014098	.002663	.002350	.000940	.000313	.000000	.060307	2.68
SW	.018954	.026159	.008929	.002193	.002506	.001253	.001096	.000470	.061560	2.67
WSW	.015038	.021773	.006579	.007832	.007832	.007049	.004543	.006109	.076754	4.70
W	.022400	.014568	.009398	.010182	.007519	.006736	.005326	.007049	.083177	4.69
WNW	.024593	.009868	.007049	.003289	.002506	.002193	.001566	.001880	.052945	3.13
NW	.017544	.008302	.005482	.003603	.002350	.002193	.000627	.000627	.040727	2.99
NNW	.013001	.007362	.004543	.002506	.001880	.001096	.001410	.000470	.032268	3.12
N	.009555	.007206	.004543	.002663	.002193	.000627	.000157	.000000	.026942	2.90
CALM									.035871	
TOTAL	.310464	.302162	.153979	.084586	.051222	.029292	.015664	.016761	1.000000	2.97

NUMBERS BELOW BASED ON ALL OBSERVATIONS
NUMBER OF INVALID OBSERVATIONS = 0
NUMBER OF VALID OBSERVATIONS = 6384

ATTACHMENT B
EMISSION CALCULATIONS

VEHICLE EMISSIONS FACTORS

The emissions from vehicles on NSW roads were assumed to all fit into three classes:

1. Light duty petrol vehicles (LDPV)
2. Heavy duty petrol vehicles (HDPV)
3. Heavy duty diesel vehicles (HDDV)

These classes of vehicles account for more than 99% of all vehicle kilometres travelled on Sydney's roads (**Pengilley 1989**). The following assumptions were made regarding the vehicles in 2000, 2006 and 2016:

- 85% of light duty passenger vehicles have been assumed to be unleaded in 2000.
- 90% of light duty passenger vehicles have been assumed to be unleaded in 2006
- 95% of petrol vehicles were using catalytic converters as emission control in 2016
- Of the 10% of heavy vehicles category travelling on Windsor Road, 90% of these are considered to be HDDV and 10% are considered to be HDPV.
- Of the 12% of heavy vehicles category travelling on Option 1, 90% of these are considered to be HDDV and 10% are considered to be HDPV.
- Of the 18% of heavy vehicles category travelling on Windsor Road, 90% of these are considered to be HDDV and 10% are considered to be HDPV.

The emissions of carbon monoxide, oxides of nitrogen, hydrocarbons and particulate matter were taken from estimates in the Sydney Metropolitan Air Quality Study (MAQS) (**Carnovale & Tilly, 1995**). It was assumed that traffic on the Windsor flood evacuation route and Windsor Road was in arterial travel mode. It was also assumed that the deterioration of catalytic converters was as in the 1992 MAQS estimates of passenger vehicle emission rates.

The emission of particulate matter from vehicles is made up of lead salts, organic and sulphate components. The total emissions comprise exhaust emission plus airborne brake wear particulate emission and airborne tyre wear particulate emissions. In the case of passenger vehicles, the PM_{10} fraction comprises 74% of the total particulate exhaust emissions. The PM_{10} fraction of HDDV and HDPV particulate exhaust emissions is 100% and 64% respectively. Brake and tyre emissions are assumed to be essentially all PM_{10}.

Calculation of vehicle emission rates

Details of emission calculations for the route sections at peak hour for 2000, 2006 and 2016 are presented in the following tables.

Emission rates for carbon monoxide, oxides of nitrogen, hydrocarbons and particulate matter corresponding to the to a given section of the route are presented for each class of vehicle and expressed as g/km/vehicle. The total emissions (tot CO etc) during the peak period have been calculated by multiplying the emission rate by the total number of vehicles estimated to be using the road in a one hour period. These values are expressed as g/km/h. Finally these values have been converted to kg/km/h and g/vehicle mile (the latter is used as input to the model).

Calculating emission rates for light duty petrol vehicles

The total emissions from the light duty fleet should take into account emissions from pre-1985 and post-1985 vehicles. This has been done for the year 2000 when an estimated 85% of the light duty petrol fleet are assumed to be using unleaded petrol and 15% are fueled by leaded petrol. The MAQS emissions inventory provides emission rates for both these types of vehicle, and these values have been used in determining the fleet emission rate. Table A1 below, shows the varying emission rates for arterial conditions, such as have been assumed for this study.

Table A1 – Vehicle emission rates for the light duty fleet, estimated for 2006 – g/km*

	1976 – 1985	Post-1985	2006**
CO	24.29	7.57	10.08
NO_x	2.95	1.51	1.73
HC	2.05	0.56	0.78

** Adapted from Table 4.15 in the Metropolitan Air Quality Study Emissions Inventory*

*** Calculated by adding 5% of the 1976 to 1985 emissions to 95% of the post-1985 emissions*

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	AM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	931								
Total Heavy	93								
LDPV	838	10.66	1.211	0.7815	0.037	8932.01	1014.70	654.82	31.00
HDDV	9	120.62	4.59	6.3	0.1472	1122.97	42.73	58.65	1.37
HDPV	84	8.01	15.72	2.8	2.09	671.16	1317.18	234.61	175.12
Brake & Tyre					0.009				8.379
Total kg/km/h						10.73	2.37	0.95	0.216
g/v-mi						18.43	4.08	1.63	0.37
Roadway Option	Option 1 - Arterial and evacuation route					Southbound	PM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	698								
Total Heavy	70								
LDPV	628	10.66	1.211	0.7815	0.037	6696.61	760.75	490.94	23.24
HDPV	7	120.62	4.59	6.3	0.1472	841.93	32.04	43.97	1.03
HDDV	63	8.01	15.72	2.8	2.09	503.19	987.53	175.90	131.29
Brake & Tyre					0.009				6.282
Total kg/km/h						8.04	1.78	0.71	0.162
g/v-mi						18.43	4.08	1.63	0.37
Roadway Option	Option 1 - Arterial and evacuation route					Northbound	AM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	986								
Total Heavy	99								
LDPV	887	10.66	1.211	0.7815	0.037	9459.68	1074.64	693.50	32.83
HDPV	10	120.62	4.59	6.3	0.1472	1189.31	45.26	62.12	1.45
HDDV	89	8.01	15.72	2.8	2.09	710.81	1394.99	248.47	185.47
Brake & Tyre					0.009				8.874
Total kg/km/h						11.36	2.51	1.00	0.229
g/v-mi						18.43	4.08	1.63	0.37
Roadway Option	Option 1 - Arterial and evacuation route					Northbound	PM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1056								
Total Heavy	106								
LDPV	950	10.66	1.211	0.7815	0.037	10131.26	1150.93	742.74	35.16
HDPV	11	120.62	4.59	6.3	0.1472	1273.75	48.47	66.53	1.55
HDDV	95	8.01	15.72	2.8	2.09	761.27	1494.03	266.11	198.63
Brake & Tyre					0.009				9.504
Total kg/km/h						12.17	2.69	1.08	0.245
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	AM		
Section	Arterial and evacuation route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	419								
Total Heavy	50								
LDPV	369	10.66	1.211	0.7815	0.037	3930.56	446.52	288.15	13.64
HDPV	5	120.62	4.59	6.3	0.1472	606.48	23.08	31.68	0.74
HDDV	45	8.01	15.72	2.8	2.09	362.47	711.36	126.71	94.58
Brake & Tyre					0.009				3.771
Total kg/km/h						4.90	1.18	0.45	0.113
g/v-mi						18.71	4.51	1.71	0.43

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	PM		
Section	Arterial and evacuation route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	453								
Total Heavy	54								
LDPV	399	10.66	1.211	0.7815	0.037	4249.50	482.75	311.54	14.75
HDPV	5	120.62	4.59	6.3	0.1472	655.69	24.95	34.25	0.80
HDDV	49	8.01	15.72	2.8	2.09	391.88	769.09	136.99	102.25
Brake & Tyre					0.009				4.077
Total kg/km/h						5.30	1.28	0.48	0.122
g/v-mi						18.71	4.51	1.71	0.43

Roadway Option	Option 1 - Arterial and evacuation route					Northbound	AM		
Section	Arterial and evacuation route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	493								
Total Heavy	59								
LDPV	434	10.66	1.211	0.7815	0.037	4624.73	525.38	339.05	16.05
HDPV	6	120.62	4.59	6.3	0.1472	713.59	27.15	37.27	0.87
HDDV	53	8.01	15.72	2.8	2.09	426.48	837.00	149.08	111.28
Brake & Tyre					0.009				4.437
Total kg/km/h						5.76	1.39	0.53	0.133
g/v-mi						18.71	4.51	1.71	0.43

Roadway Option	Option 1 - Arterial and evacuation route					Northbound	PM		
Section	Arterial and evacuation route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	508								
Total Heavy	61								
LDPV	447	10.66	1.211	0.7815	0.037	4765.45	541.37	349.36	16.54
HDPV	6	120.62	4.59	6.3	0.1472	735.30	27.98	38.40	0.90
HDDV	55	8.01	15.72	2.8	2.09	439.46	862.46	153.62	114.67
Brake & Tyre					0.009				4.572
Total kg/km/h						5.94	1.43	0.54	0.137
g/v-mi						18.71	4.51	1.71	0.43

Roadway Option	Option 2 - Evacuation route only					Southbound	AM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1203								
Total Heavy	120								
LDPV	1083	10.66	1.211	0.7815	0.037	11541.58	1311.15	846.13	40.06
HDPV	12	120.62	4.59	6.3	0.1472	1451.06	55.22	75.79	1.77
HDDV	108	8.01	15.72	2.8	2.09	867.24	1702.00	303.16	226.28
Brake & Tyre					0.009				10.827
Total kg/km/h						13.86	3.07	1.23	0.279
g/v-mi						18.43	4.65	1.86	0.42

Roadway Option	Option 2 - Evacuation route only					Southbound	PM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	985								
Total Heavy	99								
LDPV	887	10.66	1.211	0.7815	0.037	9450.09	1073.55	692.80	32.80
HDPV	10	120.62	4.59	6.3	0.1472	1188.11	45.21	62.06	1.45
HDDV	89	8.01	15.72	2.8	2.09	710.09	1393.58	248.22	185.28
Brake & Tyre					0.009				8.865
Total kg/km/h						11.35	2.51	1.00	0.228
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Option 2 - Evacuation route only					Northbound	AM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1268								
Total Heavy	127								
LDPV	1141	10.66	1.211	0.7815	0.037	12165.19	1381.99	891.85	42.22
HDPV	13	120.62	4.59	6.3	0.1472	1529.46	58.20	79.88	1.87
HDDV	114	8.01	15.72	2.8	2.09	914.10	1793.97	319.54	238.51
Brake & Tyre					0.009				11.412
Total kg/km/h						14.61	3.23	1.29	0.294
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Option 2 - Evacuation route only					Northbound	PM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1357								
Total Heavy	136								
LDPV	1221	10.66	1.211	0.7815	0.037	13019.06	1478.99	954.45	45.19
HDPV	14	120.62	4.59	6.3	0.1472	1636.81	62.29	85.49	2.00
HDDV	122	8.01	15.72	2.8	2.09	978.26	1919.88	341.96	255.25
Brake & Tyre					0.009				12.213
Total kg/km/h						15.63	3.46	1.38	0.315
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Option 2 - Evacuation route only					Southbound	AM		
Section	Evacuation Route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	133								
Total Heavy	24								
LDPV	109	10.66	1.211	0.7815	0.037	1162.58	132.07	85.23	4.04
HDPV	2	120.62	4.59	6.3	0.1472	288.76	10.99	15.08	0.35
HDDV	22	8.01	15.72	2.8	2.09	172.58	338.70	60.33	45.03
Brake & Tyre					0.009				1.197
Total kg/km/h						1.62	0.48	0.16	0.051
g/v-mi						19.54	5.80	1.93	0.61

Roadway Option	Option 2 - Evacuation route only					Southbound	PM		
Section	Evacuation Route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	156								
Total Heavy	28								
LDPV	128	10.66	1.211	0.7815	0.037	1363.63	154.91	99.97	4.73
HDPV	3	120.62	4.59	6.3	0.1472	338.70	12.89	17.69	0.41
HDDV	25	8.01	15.72	2.8	2.09	202.43	397.28	70.76	52.82
Brake & Tyre					0.009				1.404
Total kg/km/h						1.90	0.57	0.19	0.059
g/v-mi						19.54	5.80	1.93	0.61

Roadway Option	Option 2 - Evacuation route only					Northbound	AM		
Section	Evacuation Route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	208								
Total Heavy	37								
LDPV	171	10.66	1.211	0.7815	0.037	1818.17	206.55	133.29	6.31
HDPV	4	120.62	4.59	6.3	0.1472	451.60	17.18	23.59	0.55
HDDV	34	8.01	15.72	2.8	2.09	269.90	529.70	94.35	70.42
Brake & Tyre					0.009				1.872
Total kg/km/h						2.54	0.75	0.25	0.079
g/v-mi						19.54	5.80	1.93	0.61

Roadway Option	Option 2 - Evacuation route only					Northbound	PM		
Section	Evacuation Route					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	220								
Total Heavy	40								
LDPV	180	10.66	1.211	0.7815	0.037	1923.06	218.46	140.98	6.67
HDPV	4	120.62	4.59	6.3	0.1472	477.66	18.18	24.95	0.58
HDDV	36	8.01	15.72	2.8	2.09	285.48	560.26	99.79	74.49
Brake & Tyre					0.009				1.980
Total kg/km/h						2.69	0.80	0.27	0.084
g/v-mi						19.54	5.80	1.93	0.61

Roadway Option	Windsor Road					Southbound	AM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1311								
Total Heavy	131								
LDPV	1180	10.66	1.211	0.7815	0.037	12577.73	1428.86	922.09	43.66
HDPV	13	120.62	4.59	6.3	0.1472	1581.33	60.17	82.59	1.93
HDDV	118	8.01	15.72	2.8	2.09	945.10	1854.80	330.37	246.60
Brake & Tyre					0.009				11.799
Total kg/km/h						15.10	3.34	1.34	0.304
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Windsor Road					Southbound	PM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1119								
Total Heavy	112								
LDPV	1007	10.66	1.211	0.7815	0.037	10735.69	1219.60	787.05	37.26
HDPV	11	120.62	4.59	6.3	0.1472	1349.74	51.36	70.50	1.65
HDDV	101	8.01	15.72	2.8	2.09	806.69	1583.16	281.99	210.48
Brake & Tyre					0.009				10.071
Total kg/km/h						12.89	2.85	1.14	0.259
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Windsor Road					Northbound	AM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1455								
Total Heavy	146								
LDPV	1310	10.66	1.211	0.7815	0.037	13959.27	1585.80	1023.37	48.45
HDPV	15	120.62	4.59	6.3	0.1472	1755.02	66.78	91.67	2.14
HDDV	131	8.01	15.72	2.8	2.09	1048.91	2058.53	366.66	273.69
Brake & Tyre					0.009				13.095
Total kg/km/h						16.76	3.71	1.48	0.337
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Windsor Road					Northbound	PM		
Section	Windsor Road					Year	2000		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1545								
Total Heavy	155								
LDPV	1391	10.66	1.211	0.7815	0.037	14822.73	1683.90	1086.68	51.45
HDPV	15	120.62	4.59	6.3	0.1472	1863.58	70.92	97.34	2.27
HDDV	139	8.01	15.72	2.8	2.09	1113.79	2185.87	389.34	290.61
Brake & Tyre					0.009				13.905
Total kg/km/h						17.80	3.94	1.57	0.358
g/v-mi						18.43	4.08	1.63	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	AM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1096								
Total Heavy	110								
LDPV	986	8.61	1.12	0.60	0.04	7211.81	935.93	503.16	31.00
HDDV	11	120.62	4.59	6.30	0.15	1122.97	42.73	58.65	1.37
HDPV	99	8.01	15.72	2.80	2.09	671.16	1317.18	234.61	175.12
Brake & Tyre					0.01				9.864
Total kg/km/h						9.01	2.30	0.80	0.217
g/v-mi						15.48	3.95	1.37	0.37
Roadway Option	Option 1 - Arterial and evacuation route					Southbound	PM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	665								
Total Heavy	67								
LDPV	599	8.61	1.12	0.60	0.04	5406.92	701.70	377.23	23.24
HDPV	7	120.62	4.59	6.30	0.15	841.93	32.04	43.97	1.03
HDDV	60	8.01	15.72	2.80	2.09	503.19	987.53	175.90	131.29
Brake & Tyre					0.01				5.985
Total kg/km/h						6.75	1.72	0.60	0.162
g/v-mi						15.48	3.95	1.37	0.37
Roadway Option	Option 1 - Arterial and evacuation route					Northbound	AM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	745								
Total Heavy	75								
LDPV	671	8.61	1.12	0.60	0.04	7637.85	991.23	532.88	32.83
HDPV	7	120.62	4.59	6.30	0.15	1189.31	45.26	62.12	1.45
HDDV	67	8.01	15.72	2.80	2.09	710.81	1394.99	248.47	185.47
Brake & Tyre					0.01				6.705
Total kg/km/h						9.54	2.43	0.84	0.226
g/v-mi						15.48	3.95	1.37	0.37
Roadway Option	Option 1 - Arterial and evacuation route					Northbound	PM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1073								
Total Heavy	107								
LDPV	966	8.61	1.12	0.60	0.04	8180.09	1061.60	570.72	35.16
HDPV	11	120.62	4.59	6.30	0.15	1273.75	48.47	66.53	1.55
HDDV	97	8.01	15.72	2.80	2.09	761.27	1494.03	266.11	198.63
Brake & Tyre					0.01				9.657
Total kg/km/h						10.22	2.60	0.90	0.245
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	AM		
Section	Arterial and evacuation route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	434								
Total Heavy	52								
LDPV	382	8.61	1.12	0.60	0.04	3173.57	411.86	221.42	13.64
HDPV	5	120.62	4.59	6.30	0.15	606.48	23.08	31.68	0.74
HDDV	47	8.01	15.72	2.80	2.09	362.47	711.36	126.71	94.58
Brake & Tyre					0.01				3.906
Total kg/km/h						4.14	1.15	0.38	0.113
g/v-mi						15.82	4.38	1.45	0.43

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	PM		
Section	Arterial and evacuation route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	362								
Total Heavy	43								
LDPV	319	8.61	1.12	0.60	0.04	3431.09	445.28	239.38	14.75
HDPV	4	120.62	4.59	6.30	0.15	655.69	24.95	34.25	0.80
HDDV	39	8.01	15.72	2.80	2.09	391.88	769.09	136.99	102.25
Brake & Tyre					0.01				3.258
Total kg/km/h						4.48	1.24	0.41	0.121
g/v-mi						15.82	4.38	1.45	0.43

Roadway Option	Option 1 - Arterial and evacuation route					Northbound	AM		
Section	Arterial and evacuation route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	405								
Total Heavy	49								
LDPV	356	8.61	1.12	0.60	0.04	3734.06	484.60	260.52	16.05
HDPV	5	120.62	4.59	6.30	0.15	713.59	27.15	37.27	0.87
HDDV	44	8.01	15.72	2.80	2.09	426.48	837.00	149.08	111.28
Brake & Tyre					0.01				3.645
Total kg/km/h						4.87	1.35	0.45	0.132
g/v-mi						15.82	4.38	1.45	0.43

Roadway Option	Option 1 - Arterial and evacuation route					Northbound	PM		
Section	Arterial and evacuation route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	676								
Total Heavy	81								
LDPV	595	8.61	1.12	0.60	0.04	3847.67	499.34	268.45	16.54
HDPV	8	120.62	4.59	6.30	0.15	735.30	27.98	38.40	0.90
HDDV	73	8.01	15.72	2.80	2.09	439.46	862.46	153.62	114.67
Brake & Tyre					0.01				6.084
Total kg/km/h						5.02	1.39	0.46	0.138
g/v-mi						15.82	4.38	1.45	0.44

Roadway Option	Option 2 - Evacuation route only					Southbound	AM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1255								
Total Heavy	126								
LDPV	1130	8.61	1.12	0.60	0.04	9318.80	1209.38	650.16	40.06
HDPV	13	120.62	4.59	6.30	0.15	1451.06	55.22	75.79	1.77
HDDV	113	8.01	15.72	2.80	2.09	867.24	1702.00	303.16	226.28
Brake & Tyre					0.01				11.295
Total kg/km/h						11.64	2.97	1.03	0.279
g/v-mi						15.48	4.49	1.56	0.42

Roadway Option	Option 2 - Evacuation route only					Southbound	PM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	870								
Total Heavy	87								
LDPV	783	8.61	1.12	0.60	0.04	7630.11	990.22	532.34	32.80
HDPV	9	120.62	4.59	6.30	0.15	1188.11	45.21	62.06	1.45
HDDV	78	8.01	15.72	2.80	2.09	710.09	1393.58	248.22	185.28
Brake & Tyre					0.01				7.830
Total kg/km/h						9.53	2.43	0.84	0.227
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Option 2 - Evacuation route only					Northbound	AM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	872								
Total Heavy	87								
LDPV	785	8.61	1.12	0.60	0.04	9822.31	1274.72	685.29	42.22
HDPV	9	120.62	4.59	6.30	0.15	1529.46	58.20	79.88	1.87
HDDV	78	8.01	15.72	2.80	2.09	914.10	1793.97	319.54	238.51
Brake & Tyre					0.01				7.848
Total kg/km/h						12.27	3.13	1.08	0.290
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Option 2 - Evacuation route only					Northbound	PM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1258								
Total Heavy	126								
LDPV	1132	8.61	1.12	0.60	0.04	10511.73	1364.19	733.39	45.19
HDPV	13	120.62	4.59	6.30	0.15	1636.81	62.29	85.49	2.00
HDDV	113	8.01	15.72	2.80	2.09	978.26	1919.88	341.96	255.25
Brake & Tyre					0.01				11.322
Total kg/km/h						13.13	3.35	1.16	0.314
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Option 2 - Evacuation route only					Southbound	AM		
Section	Evacuation Route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	383								
Total Heavy	69								
LDPV	314	8.61	1.12	0.60	0.04	938.68	121.82	65.49	4.04
HDPV	7	120.62	4.59	6.30	0.15	288.76	10.99	15.08	0.35
HDDV	62	8.01	15.72	2.80	2.09	172.58	338.70	60.33	45.03
Brake & Tyre					0.01				3.447
Total kg/km/h						1.40	0.47	0.14	0.053
g/v-mi						16.84	5.67	1.70	0.64
Roadway Option	Option 2 - Evacuation route only					Southbound	PM		
Section	Evacuation Route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	199								
Total Heavy	36								
LDPV	163	8.61	1.12	0.60	0.04	1101.01	142.89	76.82	4.73
HDPV	4	120.62	4.59	6.30	0.15	338.70	12.89	17.69	0.41
HDDV	32	8.01	15.72	2.80	2.09	202.43	397.28	70.76	52.82
Brake & Tyre					0.01				1.791
Total kg/km/h						1.64	0.55	0.17	0.060
g/v-mi						16.84	5.67	1.70	0.61
Roadway Option	Option 2 - Evacuation route only					Northbound	AM		
Section	Evacuation Route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	276								
Total Heavy	50								
LDPV	226	8.61	1.12	0.60	0.04	1468.01	190.52	102.42	6.31
HDPV	5	120.62	4.59	6.30	0.15	451.60	17.18	23.59	0.55
HDDV	45	8.01	15.72	2.80	2.09	269.90	529.70	94.35	70.42
Brake & Tyre					0.01				2.484
Total kg/km/h						2.19	0.74	0.22	0.080
g/v-mi						16.84	5.67	1.70	0.61
Roadway Option	Option 2 - Evacuation route only					Northbound	PM		
Section	Evacuation Route					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	486								
Total Heavy	87								
LDPV	399	8.61	1.12	0.60	0.04	1552.70	201.51	108.33	6.67
HDPV	9	120.62	4.59	6.30	0.15	477.66	18.18	24.95	0.58
HDDV	79	8.01	15.72	2.80	2.09	285.48	560.26	99.79	74.49
Brake & Tyre					0.01				4.374
Total kg/km/h						2.32	0.78	0.23	0.086
g/v-mi						16.84	5.67	1.70	0.63

Roadway Option	Windsor Road					Southbound	AM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1478								
Total Heavy	148								
LDPV	1330	8.61	1.12	0.60	0.04	10155.40	1317.95	708.53	43.66
HDPV	15	120.62	4.59	6.30	0.15	1581.33	60.17	82.59	1.93
HDDV	133	8.01	15.72	2.80	2.09	945.10	1854.80	330.37	246.60
Brake & Tyre					0.01				13.302
Total kg/km/h						12.68	3.23	1.12	0.305
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Windsor Road					Southbound	PM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	961								
Total Heavy	96								
LDPV	865	8.61	1.12	0.60	0.04	8668.11	1124.93	604.76	37.26
HDPV	10	120.62	4.59	6.30	0.15	1349.74	51.36	70.50	1.65
HDDV	86	8.01	15.72	2.80	2.09	806.69	1583.16	281.99	210.48
Brake & Tyre					0.01				8.649
Total kg/km/h						10.82	2.76	0.96	0.258
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Windsor Road					Northbound	AM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1124								
Total Heavy	112								
LDPV	1012	8.61	1.12	0.60	0.04	11270.87	1462.71	786.35	48.45
HDPV	11	120.62	4.59	6.30	0.15	1755.02	66.78	91.67	2.14
HDDV	101	8.01	15.72	2.80	2.09	1048.91	2058.53	366.66	273.69
Brake & Tyre					0.01				10.116
Total kg/km/h						14.07	3.59	1.24	0.334
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Windsor Road					Northbound	PM		
Section	Windsor Road					Year	2006		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1712								
Total Heavy	171								
LDPV	1541	8.61	1.12	0.60	0.04	11968.03	1553.19	835.00	51.45
HDPV	17	120.62	4.59	6.30	0.15	1863.58	70.92	97.34	2.27
HDDV	154	8.01	15.72	2.80	2.09	1113.79	2185.87	389.34	290.61
Brake & Tyre					0.01				15.408
Total kg/km/h						14.95	3.81	1.32	0.360
g/v-mi						15.48	3.95	1.37	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	AM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1140								
Total Heavy	114								
LDPV	1026	7.58	1.07	0.51	0.04	6351.28	896.55	427.33	31.00
HDDV	11	120.62	4.59	6.30	0.15	1122.97	42.73	58.65	1.37
HDPV	103	8.01	15.72	2.80	2.09	671.16	1317.18	234.61	175.12
Brake & Tyre					0.009				10.260
Total kg/km/h						8.15	2.26	0.72	0.218
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	PM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	670								
Total Heavy	67								
LDPV	603	7.58	1.07	0.51	0.04	4761.76	672.17	320.38	23.24
HDPV	7	120.62	4.59	6.30	0.15	841.93	32.04	43.97	1.03
HDDV	60	8.01	15.72	2.80	2.09	503.19	987.53	175.90	131.29
Brake & Tyre					0.009				6.030
Total kg/km/h						6.11	1.69	0.54	0.162
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Northbound	AM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	779								
Total Heavy	78								
LDPV	701	7.58	1.07	0.51	0.04	6726.49	949.52	452.57	32.83
HDPV	8	120.62	4.59	6.30	0.15	1189.31	45.26	62.12	1.45
HDDV	70	8.01	15.72	2.80	2.09	710.81	1394.99	248.47	185.47
Brake & Tyre					0.009				7.011
Total kg/km/h						8.63	2.39	0.76	0.227
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Northbound	PM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1120								
Total Heavy	112								
LDPV	1008	7.58	1.07	0.51	0.04	7204.03	1016.93	484.70	35.16
HDPV	11	120.62	4.59	6.30	0.15	1273.75	48.47	66.53	1.55
HDDV	101	8.01	15.72	2.80	2.09	761.27	1494.03	266.11	198.63
Brake & Tyre					0.009				10.080
Total kg/km/h						9.24	2.56	0.82	0.245
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Option 1 - Arterial and evacuation route					Southbound	AM		
Section	Arterial and evacuation route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	575								
Total Heavy	69								
LDPV	506	7.58	1.07	0.51	0.04	2794.90	394.53	188.05	13.64
HDPV	7	120.62	4.59	6.30	0.15	606.48	23.08	31.68	0.74
HDDV	62	8.01	15.72	2.80	2.09	362.47	711.36	126.71	94.58
Brake & Tyre					0.009				5.175
Total kg/km/h						3.76	1.13	0.35	0.114
g/v-mi						14.37	4.31	1.32	0.44
Roadway Option	Option 1 - Arterial and evacuation route					Southbound	PM		
Section	Arterial and evacuation route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	347								
Total Heavy	42								
LDPV	305	7.58	1.07	0.51	0.04	3021.69	426.54	203.31	14.75
HDPV	4	120.62	4.59	6.30	0.15	655.69	24.95	34.25	0.80
HDDV	37	8.01	15.72	2.80	2.09	391.88	769.09	136.99	102.25
Brake & Tyre					0.009				3.123
Total kg/km/h						4.07	1.22	0.37	0.121
g/v-mi						14.37	4.31	1.32	0.43
Roadway Option	Option 1 - Arterial and evacuation route					Northbound	AM		
Section	Arterial and evacuation route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	575								
Total Heavy	69								
LDPV	506	7.58	1.07	0.51	0.04	3288.51	464.21	221.26	16.05
HDPV	7	120.62	4.59	6.30	0.15	713.59	27.15	37.27	0.87
HDDV	62	8.01	15.72	2.80	2.09	426.48	837.00	149.08	111.28
Brake & Tyre					0.009				5.175
Total kg/km/h						4.43	1.33	0.41	0.133
g/v-mi						14.37	4.31	1.32	0.43
Roadway Option	Option 1 - Arterial and evacuation route					Northbound	PM		
Section	Arterial and evacuation route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	347								
Total Heavy	42								
LDPV	305	7.58	1.07	0.51	0.04	3388.56	478.33	227.99	16.54
HDPV	4	120.62	4.59	6.30	0.15	735.30	27.98	38.40	0.90
HDDV	37	8.01	15.72	2.80	2.09	439.46	862.46	153.62	114.67
Brake & Tyre					0.009				3.123
Total kg/km/h						4.56	1.37	0.42	0.135
g/v-mi						14.37	4.31	1.32	0.43

Roadway Option	Option 2 - Evacuation route only					Southbound	AM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1469								
Total Heavy	147								
LDPV	1322	7.58	1.07	0.51	0.04	8206.87	1158.49	552.18	40.06
HDPV	15	120.62	4.59	6.30	0.15	1451.06	55.22	75.79	1.77
HDDV	132	8.01	15.72	2.80	2.09	867.24	1702.00	303.16	226.28
Brake & Tyre					0.009				13.221
Total kg/km/h						10.53	2.92	0.93	0.281
g/v-mi						14.00	4.42	1.41	0.43

Roadway Option	Option 2 - Evacuation route only					Southbound	PM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	885								
Total Heavy	89								
LDPV	797	7.58	1.07	0.51	0.04	6719.67	948.56	452.12	32.80
HDPV	9	120.62	4.59	6.30	0.15	1188.11	45.21	62.06	1.45
HDDV	80	8.01	15.72	2.80	2.09	710.09	1393.58	248.22	185.28
Brake & Tyre	1				0.009				7.965
Total kg/km/h						8.62	2.39	0.76	0.227
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Option 2 - Evacuation route only					Northbound	AM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	896								
Total Heavy	90								
LDPV	806	7.58	1.07	0.51	0.04	8650.30	1221.08	582.01	42.22
HDPV	9	120.62	4.59	6.30	0.15	1529.46	58.20	79.88	1.87
HDDV	81	8.01	15.72	2.80	2.09	914.10	1793.97	319.54	238.51
Brake & Tyre					0.009				8.064
Total kg/km/h						11.09	3.07	0.98	0.291
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Option 2 - Evacuation route only					Northbound	PM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1371								
Total Heavy	137								
LDPV	1234	7.58	1.07	0.51	0.04	9257.45	1306.79	622.86	45.19
HDPV	14	120.62	4.59	6.30	0.15	1636.81	62.29	85.49	2.00
HDDV	123	8.01	15.72	2.80	2.09	978.26	1919.88	341.96	255.25
Brake & Tyre					0.009				12.339
Total kg/km/h						11.87	3.29	1.05	0.315
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Option 2 - Evacuation route only					Southbound	AM		
Section	Evacuation Route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	393								
Total Heavy	71								
LDPV	322	7.58	1.07	0.51	0.04	826.67	116.69	55.62	4.04
HDPV	7	120.62	4.59	6.30	0.15	288.76	10.99	15.08	0.35
HDDV	64	8.01	15.72	2.80	2.09	172.58	338.70	60.33	45.03
Brake & Tyre					0.009				3.537
Total kg/km/h						1.29	0.47	0.13	0.053
g/v-mi						15.50	5.61	1.58	0.64

Roadway Option	Option 2 - Evacuation route only					Southbound	PM		
Section	Evacuation Route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	201								
Total Heavy	36								
LDPV	165	7.58	1.07	0.51	0.04	969.63	136.87	65.24	4.73
HDPV	4	120.62	4.59	6.30	0.15	338.70	12.89	17.69	0.41
HDDV	33	8.01	15.72	2.80	2.09	202.43	397.28	70.76	52.82
Brake & Tyre					0.009				1.809
Total kg/km/h						1.51	0.55	0.15	0.060
g/v-mi						15.50	5.61	1.58	0.61

Roadway Option	Option 2 - Evacuation route only					Northbound	AM		
Section	Evacuation Route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	271								
Total Heavy	49								
LDPV	222	7.58	1.07	0.51	0.04	1292.84	182.50	86.99	6.31
HDPV	5	120.62	4.59	6.30	0.15	451.60	17.18	23.59	0.55
HDDV	44	8.01	15.72	2.80	2.09	269.90	529.70	94.35	70.42
Brake & Tyre					0.009				2.439
Total kg/km/h						2.01	0.73	0.20	0.080
g/v-mi						15.50	5.61	1.58	0.61

Roadway Option	Option 2 - Evacuation route only					Northbound	PM		
Section	Evacuation Route					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	499								
Total Heavy	90								
LDPV	409	7.58	1.07	0.51	0.04	1367.43	193.03	92.00	6.67
HDPV	9	120.62	4.59	6.30	0.15	477.66	18.18	24.95	0.58
HDDV	81	8.01	15.72	2.80	2.09	285.48	560.26	99.79	74.49
Brake & Tyre					0.009				4.491
Total kg/km/h						2.13	0.77	0.22	0.086
g/v-mi						15.50	5.61	1.58	0.63

Roadway Option	Windsor Road					Southbound	AM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1683								
Total Heavy	168								
LDPV	1515	7.58	1.07	0.51	0.04	8943.64	1262.49	601.75	43.66
HDPV	17	120.62	4.59	6.30	0.15	1581.33	60.17	82.59	1.93
HDDV	151	8.01	15.72	2.80	2.09	945.10	1854.80	330.37	246.60
Brake & Tyre					0.009				15.147
Total kg/km/h						11.47	3.18	1.01	0.307
g/v-mi						14.00	3.88	1.24	0.38

Roadway Option	Windsor Road					Southbound	PM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	970								
Total Heavy	97								
LDPV	873	7.58	1.07	0.51	0.04	7633.82	1077.60	513.62	37.26
HDPV	10	120.62	4.59	6.30	0.15	1349.74	51.36	70.50	1.65
HDDV	87	8.01	15.72	2.80	2.09	806.69	1583.16	281.99	210.48
Brake & Tyre					0.009				8.730
Total kg/km/h						9.79	2.71	0.87	0.258
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Windsor Road					Northbound	AM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1139								
Total Heavy	114								
LDPV	1025	7.58	1.07	0.51	0.04	9926.01	1401.17	667.85	48.45
HDPV	11	120.62	4.59	6.30	0.15	1755.02	66.78	91.67	2.14
HDDV	103	8.01	15.72	2.80	2.09	1048.91	2058.53	366.66	273.69
Brake & Tyre					0.009				10.251
Total kg/km/h						12.73	3.53	1.13	0.335
g/v-mi						14.00	3.88	1.24	0.37

Roadway Option	Windsor Road					Northbound	PM		
Section	Windsor Road					Year	2016		
		Emission rate (g/km/vehicle)				Total Emissions g/km/h			
Vehicle	Number	CO	NOx	HC	PM10	totCO	totNOx	totHC	totPM10
Total Number	1835								
Total Heavy	184								
LDPV	1652	7.58	1.07	0.51	0.04	10539.99	1487.84	709.16	51.45
HDPV	18	120.62	4.59	6.30	0.15	1863.58	70.92	97.34	2.27
HDDV	165	8.01	15.72	2.80	2.09	1113.79	2185.87	389.34	290.61
Brake & Tyre					0.009				16.515
Total kg/km/h						13.52	3.74	1.20	0.361
g/v-mi						14.00	3.88	1.24	0.37

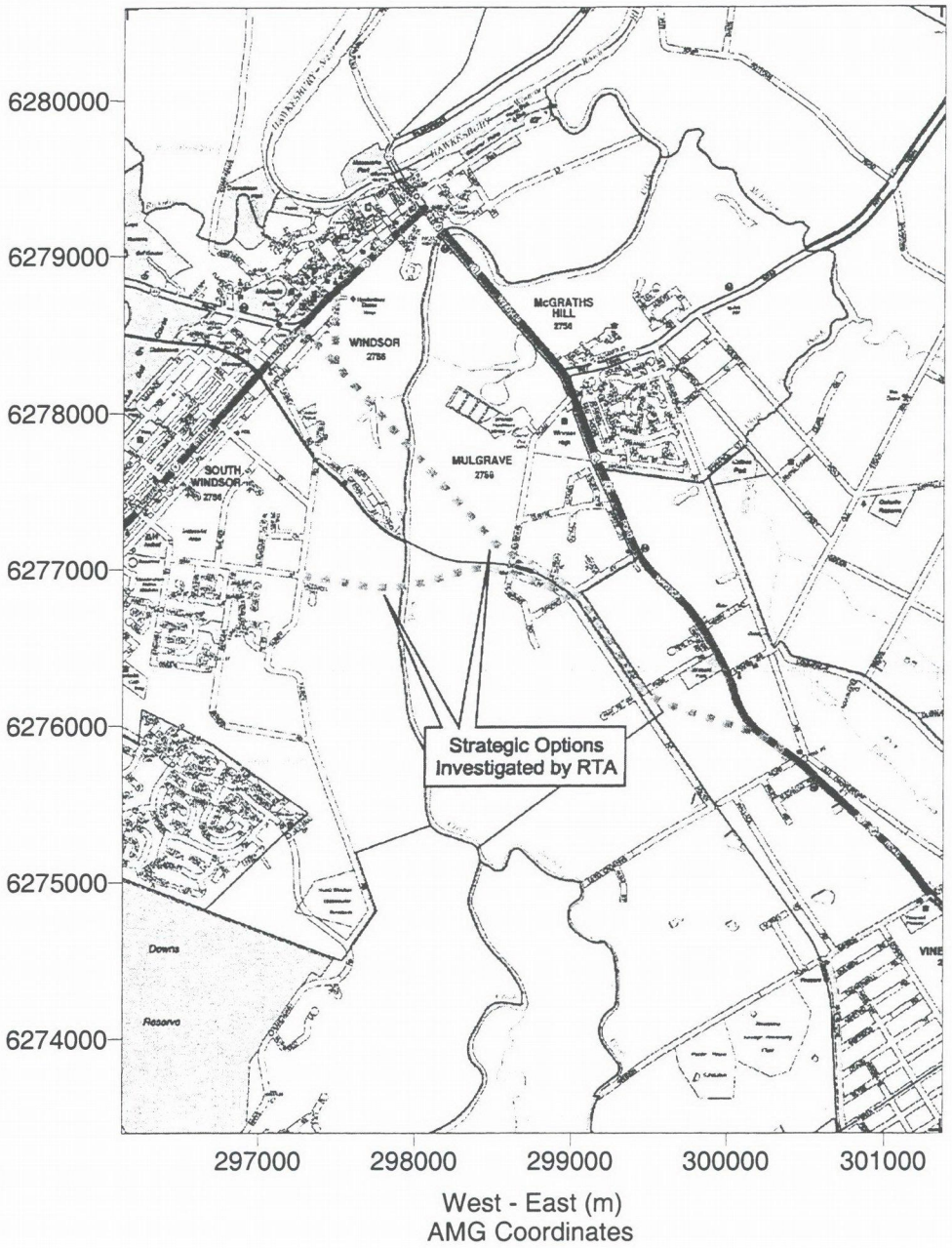
ATTACHMENT C
HYDROCARBON ANALYSIS OF VEHICLE EXHAUST, PETROL AND PETROL VAPOUR

HYDROCARBON COMPOSITION OF VEHICLE EXHAUST, PETROL AND PETROL VAPOUR

Hydrocarbon	Exhaust Average	Exhaust SD	Petrol Average	Petrol SD	Petrol Vapour
Ethane	1.4	0.5			
ethylene	11.2	3.2			
acetylene	8.7	2.7			
propane	0.1	0.1	0.1	0.1	1.5
propylene	5.0	1.6			
methylacetylene	0.4	0.3			
n-butane	2.1	0.6	2.9	0.4	18.7
i-butane	1.0	0.3	1.2	0.3	11.1
1-butene	0.9	0.3	0.2	0.1	1.6
i-butene	1.4	0.6			
trans-2-butene	0.6	0.4	0.6	0.1	3.7
cis-2-butene	0.5	0.2	0.5	0.1	2.9
n-pentane	3.0	0.7	6.0	0.6	10.7
i-pentane	4.8	0.9	10.6	0.5	25.4
cyclopentane	0.4	0.1	0.5	0.1	0.6
1-pentene	0.2	0.1	0.3	0.1	0.7
trans-2-pentene	0.3	0.2	0.8	0.1	1.5
cis-2-pentene	0.3	0.2	0.5	0.1	0.9
2-methyl-1-butene	0.3	0.2	0.6	0.1	1.3
2-methyl-2-butene	0.5	0.2	1.6	0.2	2.6
n-hexane	1.9	0.4	3.5	0.3	1.9
2-methylpentane	2.3	0.4	4.9	0.2	3.5
3-methylpentane	1.6	0.3	3.2	0.1	2.2
2,2-dimethylbutane	0.3	0.2	0.5	0.1	0.6
2,3-dimethylbutane	0.6	0.1	1.3	0.1	1.1
methylcyclopentane	1.0	0.2	1.9	0.1	0.9
cyclohexane	0.6	0.2	0.8	0.1	0.3
1-hexene	0.3	0.2	0.4	0.1	0.3
other C_6 olefins	0.7	0.2	1.6	0.1	1.0
benzene	5.0	0.7	2.6	0.2	0.9
n-heptane	0.8	0.2	1.6	0.1	0.3
2-methylhexane	1.5	0.3	2.9	0.1	0.7
3-methylhexane	1.2	0.3	2.3	0.1	0.5
2,4-dimethylpentane	0.3	0.1	0.7	0.1	0.2
methylcyclohexane	0.6	0.2	1.1	0.2	0.2
other C_7 cycloalkanes	0.3	0.2	0.6	0.1	0.1
toluene	10.2	0.9	9.6	0.6	1.0
n-octane	0.4	0.1	0.7	0.1	
2,2,4-trimethylpentane	1.0	0.4	2.1	0.5	0.4
other C_8 alkanes	3.2	0.7	7.1	0.6	0.6
ethylbenzene	1.9	0.2	1.6	0.1	0.1
m, p-xylenes	6.5	0.9	6.5	0.4	0.2
o-xylene	2.5	0.4	2.3	0.2	0.1
n-nonane	0.2	0.1	0.3	0.1	
other C_9 alkanes	1.7	0.4	2.1	0.3	
n-propylbenzene	0.4	0.1	0.4	0.1	
i-propylbenzene	0.2	0.1	0.2	0.1	
1,2,4-trimethylbenzene	1.9	0.3	1.8	0.2	
1,3,5-trimethylbenzene	0.7	0.1	0.7	0.1	
m, p-ethyltoluenes	2.0	0.3	1.8	0.2	
o-ethyltoluene	0.6	0.2	0.5	O.1	
n-decane	0.4	0.1	0.4	0.1	
other C_{10} alkanes & aromatics	0.9	0.4	1.2	0.3	
C_{11} and C_{12} alkanes & aromatics	3.6	1.1	4.2	0.7	
	100.40		99.80		100.30

Source: **Nelson & Quigley (1982)**

FIGURES



Location of Study Area

Proposed routes

FIGURE 1

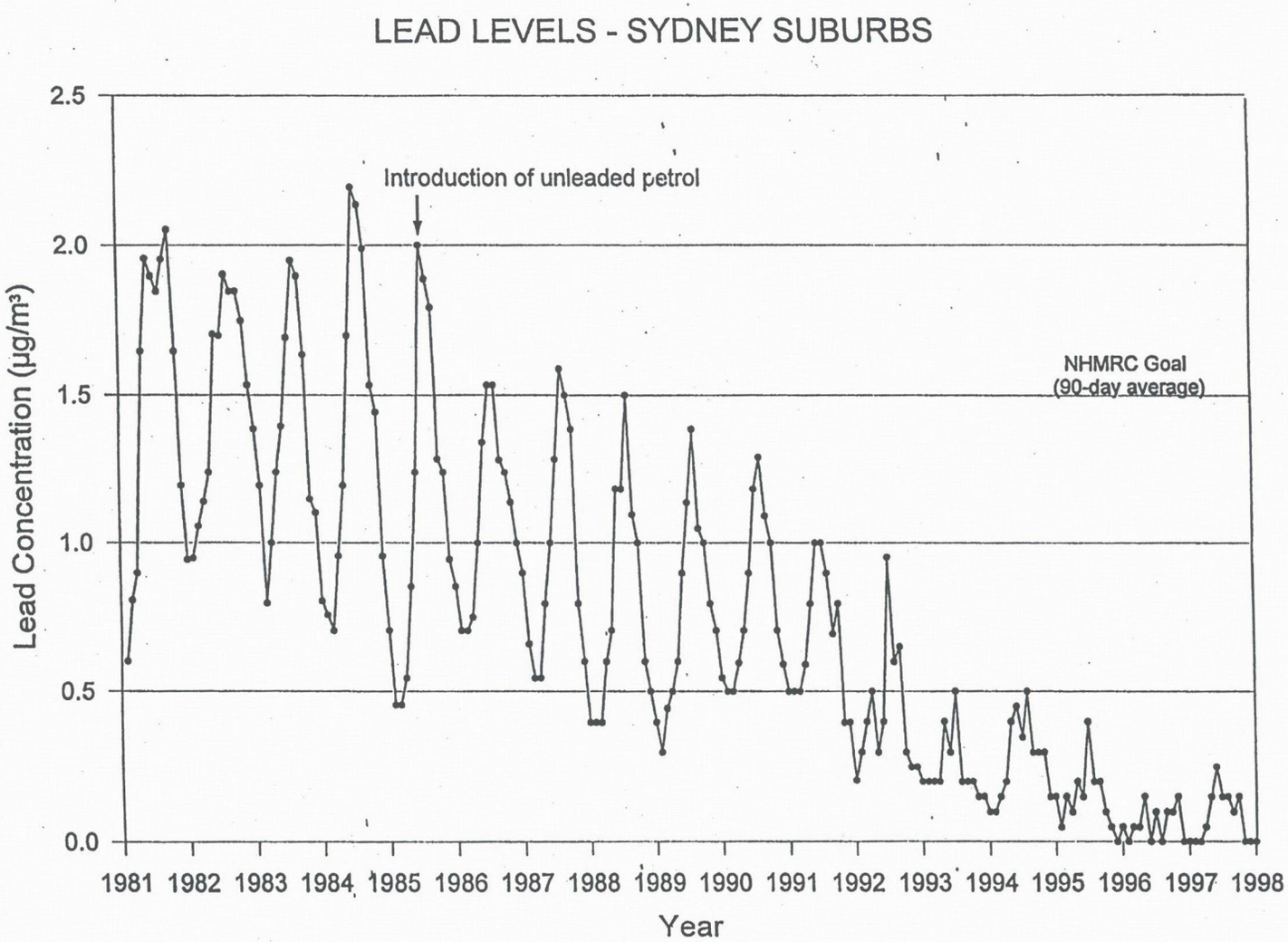
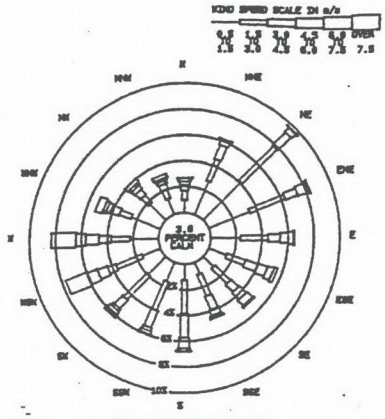
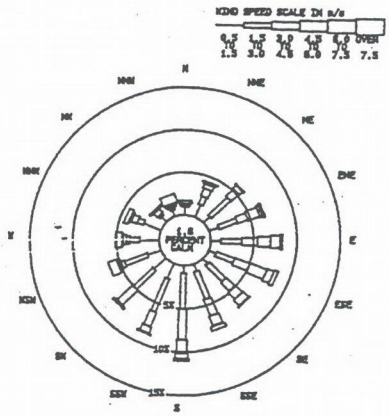

LEAD LEVELS - SYDNEY SUBURBS
Introduction of unleaded petrol
NHMRC Goal
(90-day average)
Lead Concentration (µg/m³)
2.5
2.0
1.5
1.0
0.5
0.0
1981 1982 1983 1984 1985 1986 1987 1988 1989 1990 1991 1992 1993 1994 1995 1996 1997 1998
Year

FIGURE 2

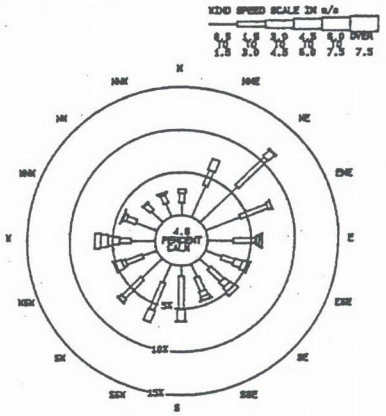
ANNUAL AND SEASONAL WINDROSES FOR RICHMOND (OVAL)



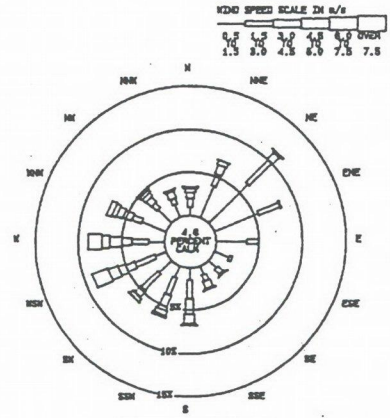
DISTRIBUTION OF WINDS
FREQUENCY OF OCCURRENCE IN PERCENT
Annual (Richmond composite 1979/80/81)



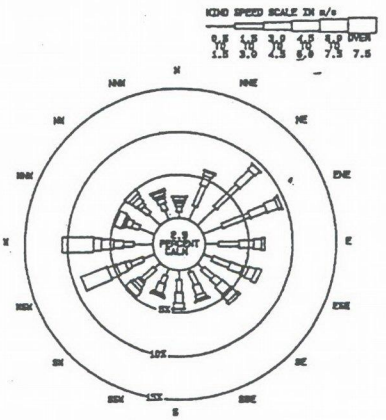
DISTRIBUTION OF WINDS
FREQUENCY OF OCCURRENCE IN PERCENT
Summer (Richmond composite 1979/80/81)



DISTRIBUTION OF WINDS
FREQUENCY OF OCCURRENCE IN PERCENT
Autumn (Richmond composite 1979/80/81)



DISTRIBUTION OF WINDS
FREQUENCY OF OCCURRENCE IN PERCENT
Winter (Richmond 1979/80/81)



DISTRIBUTION OF WINDS
FREQUENCY OF OCCURRENCE IN PERCENT
Spring (Richmond 1979/80/81)

FIGURE 3

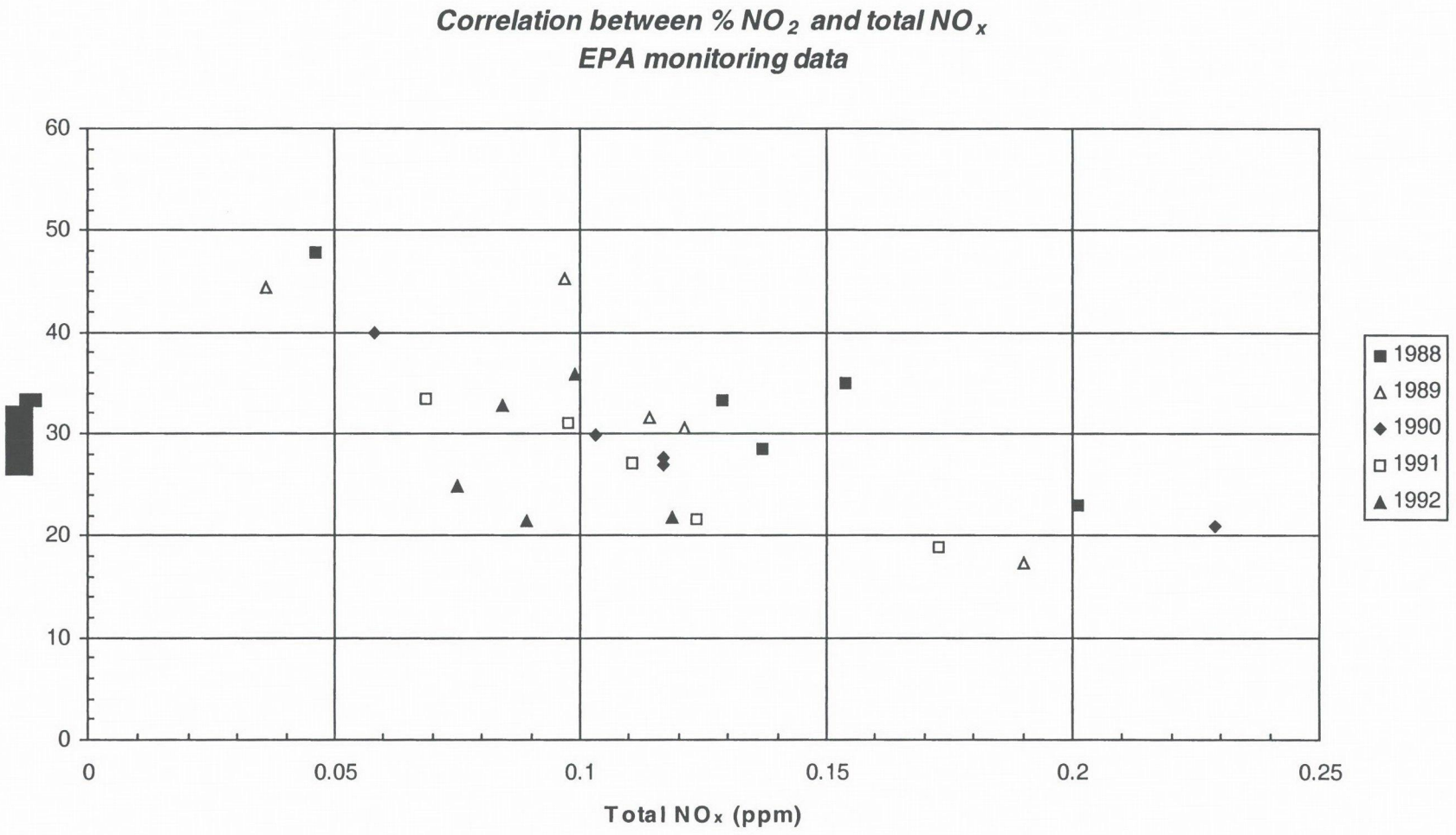

Correlation between % NO2 and total NOx
EPA monitoring data
60
50
40
30
20
10
0
0
0.05
0.1
0.15
0.2
0.25
Total NOx (ppm)
1988
1989
1990
1991
1992

FIGURE 4

EMISSION RATE OF CO vs SPEED FOR LIGHT DUTY PETROL VEHICLES - YEARS 1988 AND 2000

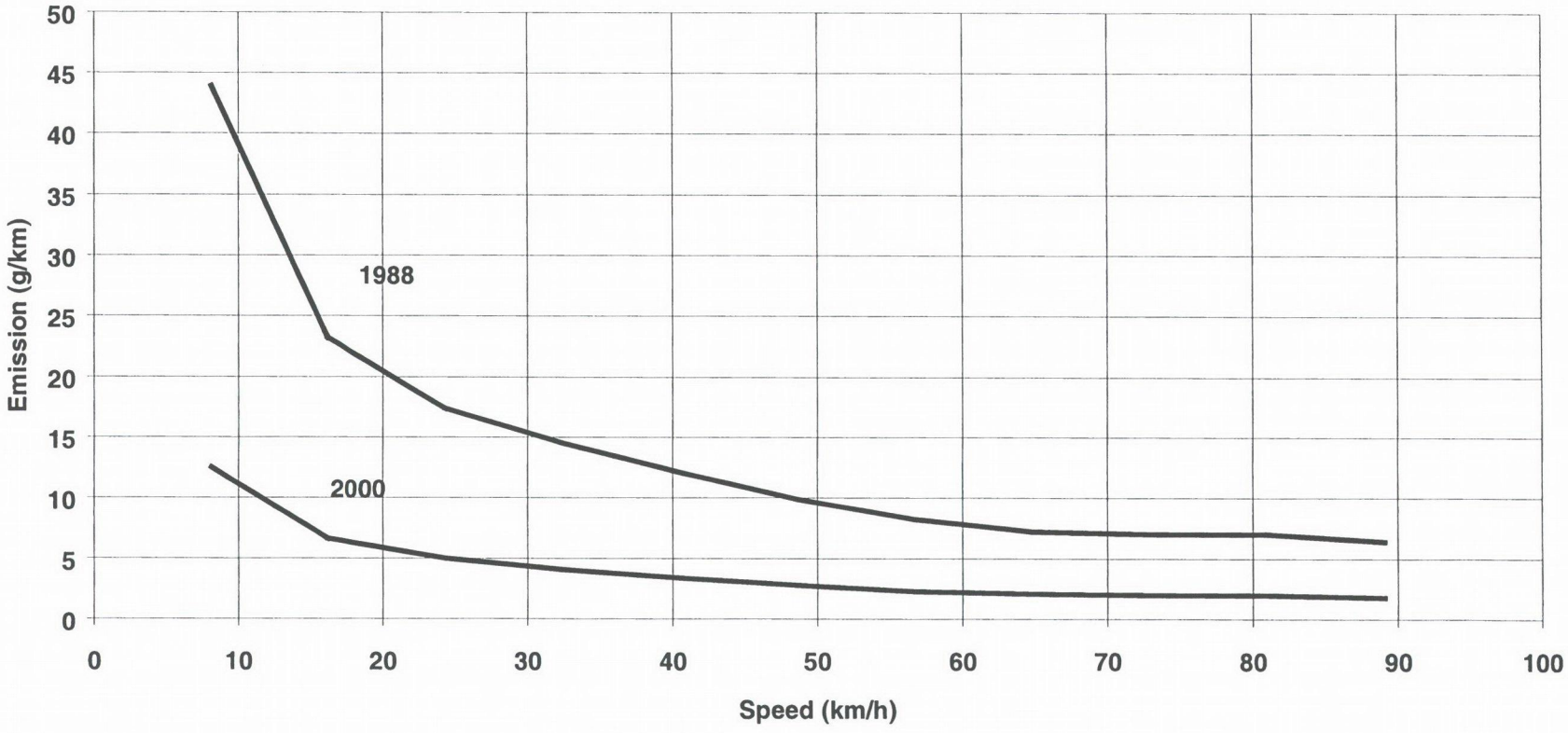


FIGURE 5

EMISSION RATE OF NO_x vs SPEED FOR LIGHT DUTY PETROL VEHICLES - YEARS 1988 AND 2000

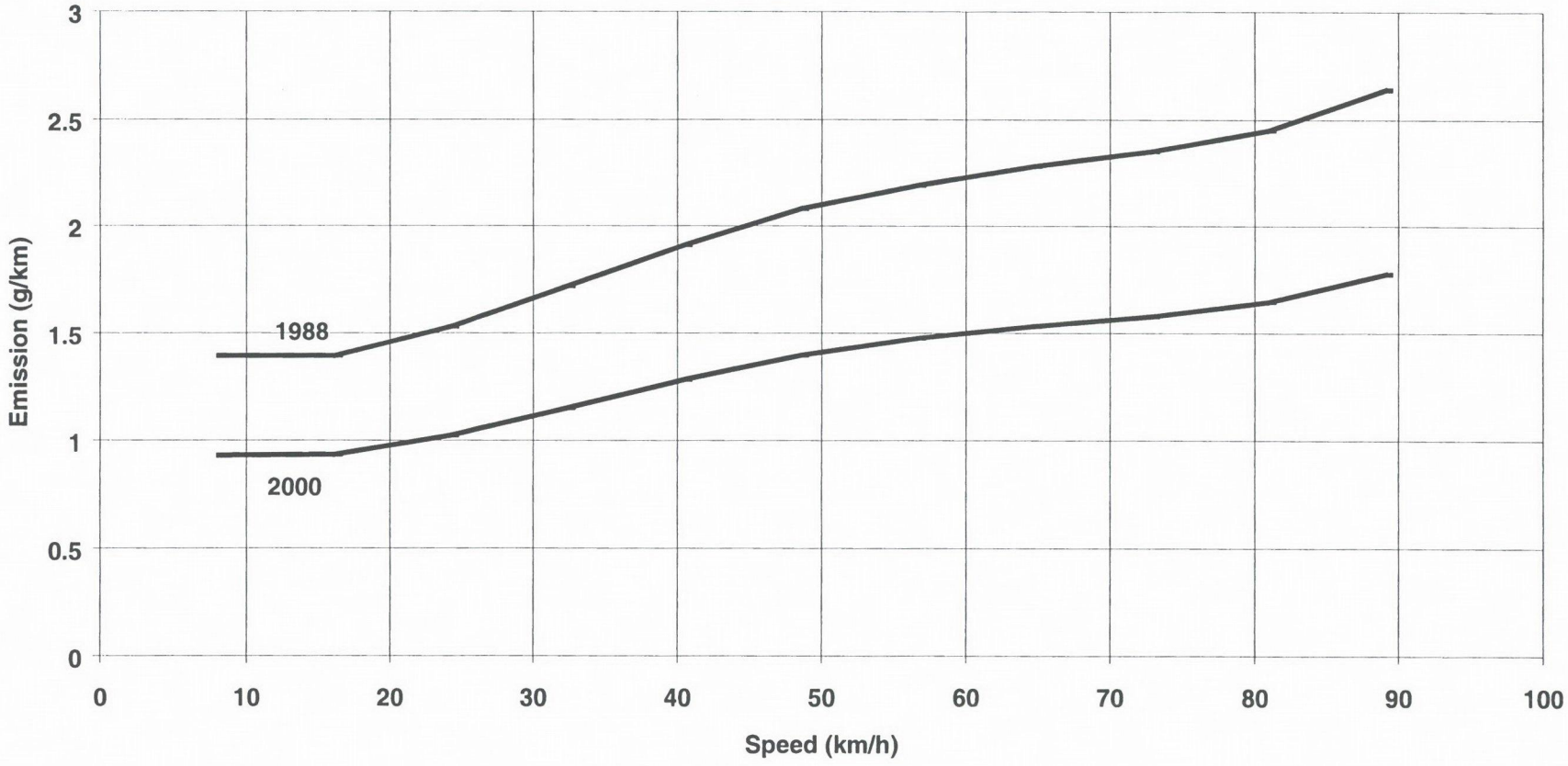


FIGURE 6

Working Paper 3 –Hydraulic Assessment

ROADS AND TRAFFIC AUTHORITY and
DEPARTMENT OF LAND AND WATER
CONSERVATION

WINDSOR FLOOD EVACUATION ROUTE
SOUTH CREEK HIGH LEVEL CROSSING
HYDRAULIC ASSESSMENT

NOVEMBER 2001

WEBB, McKEOWN & ASSOCIATES PTY LTD

ROADS AND TRAFFIC AUTHORITY and
DEPARTMENT OF LAND AND WATER CONSERVATION

WINDSOR FLOOD EVACUATION ROUTE SOUTH CREEK HIGH LEVEL CROSSING HYDRAULIC ASSESSMENT

NOVEMBER 2001

Webb, McKeown & Associates Pty Ltd
Level 2, 160 Clarence Street, SYDNEY 2000
Telephone: (02) 9299 2855
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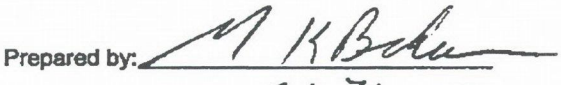
99033:WindsorEvacRoute.wpd:M6

Prepared by: 

Verified by:

WINDSOR FLOOD EVACUATION ROUTE
SOUTH CREEK HIGH LEVEL CROSSING
HYDRAULIC ASSESSMENT

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1. INTRODUCTION

1.1 General

The NSW State Government is committed to protecting the communities of Windsor and other towns in the Hawkesbury-Nepean Valley as part of an overall Floodplain Management Strategy. The Strategy is being developed with input from the community through the government/community consultation process and includes the construction of flood evacuation routes. A new high level route across South Creek is crucial to facilitate the timely evacuation of Windsor in the event of a major flood. Identifying opportunities to reduce traffic conditions on Windsor Road is also included as a beneficial aspect of the project.

Connell Wagner have been engaged by the Roads and Traffic Authority (RTA) to assess alternative routes and waterway configurations and to prepare an Environmental Impact Statement (EIS) for the preferred option. To assist in this regard, Webb McKeown & Associates (WM) were commissioned to provide specialist technical input and investigate the potential impacts on flooding of different bridge and alignment options.

1.2 Objectives

The main objective of the study was to investigate potential hydraulic impacts associated with different bridge and alignment options. Two alignment options were considered, the northern option (also referred to as Option 1B) and the southern option (Option 2).

Two possible mechanisms of flooding were considered for the analyses. The first involves backwater flooding from the Hawkesbury River in conjunction with minor flooding from South Creek, and the second, results primarily from flooding of the local South Creek catchment while minor flooding is occurring in the Hawkesbury River.

2. BACKGROUND

2.1 Catchment Description

The South Creek catchment drains a significant portion of western Sydney (refer Figure 1) which includes parts of the municipalities of Blacktown, Camden, Fairfield, Hawkesbury, Liverpool and Penrith. The main channel of South Creek is in a predominantly natural state and the major tributaries include Eastern Creek, Ropes Creek and Kemps Creek. South Creek flows into the Hawkesbury River east of Windsor. The catchment area of South Creek is approximately 570 km^2 to the railway bridge and 640 km^2 to the confluence with the Hawkesbury River at Windsor.

Land use in the catchment comprises a mixture of semi-rural, urban residential, industrial/commercial with various open space and special uses. Urban development in the catchment is concentrated in a narrow band surrounding the major western transport corridor incorporating the Main Western Railway, Great Western Highway, and the M4 Motorway.

The residential development in the catchment is relatively new with most of the housing less than 40 years old. There is a significant amount of new subdivision activity presently occurring within the catchment.

2.2 Previous Studies

A number of previous studies have dealt with flooding in the South Creek catchment. The studies most relevant to the present investigation are:

- Flood Study Report South Creek, July 1990 (Reference 1),
- South Creek Floodplain Management Study, February 1991 (Reference 2),
- Warragamba Dam Auxiliary Spillway Environmental Impact Study, Flood Study, October 1996 (Reference 3),
- Flood Study Report Eastern Creek, February 1992 (Reference 4),
- South Creek High Level Crossing Stage 1 Report, April 1999 (Reference 7).

The South Creek Flood Study (Reference 1) determined 1% AEP design flows and flood levels in South Creek and various tributaries except Eastern Creek. No other design events were examined. The RAFTS-XP hydrologic model was used to estimate flows and MIKE-11 was used for the hydraulic modelling. The limit of the study was just downstream of the confluence with Eastern Creek which is 1 km upstream of the area of interest for this study.

For the Floodplain Management Study (Reference 2), updated versions of the Flood Study models were developed and used to analyse the 5%, 2%, 1% AEP and Probable Maximum Flood (PMF) design flood events. The Floodplain Management Study (Reference 2) included these additional modelling of the South Creek catchment.

The Warragamba Dam Auxiliary Spillway EIS Flood Study (Reference 3) was a comprehensive study undertaken of the Hawkesbury River, completed in 1996. The study, which is also relevant to this project, was a result of extensive work by WM, which was then externally reviewed and approved by a number of experts both in government and private practice.

In this study, a RORB hydrologic model was set up to generate inflow hydrographs for a RUBICON hydraulic model of the Hawkesbury/Nepean River systems and the floodplain stretching from Camden to Broken Bay. Both models were calibrated and verified using available historical flood and rainfall data. A large range (20%, 10%, 5%, 2%, 1%, 0.5%, 0.2%, 0.1% AEP and PMF) of design events were modelled and reported.

2.3 Flood History

Significant flooding within the study area has occurred a number of times since European settlement with the first observed event occurring in March 1799. In the 1% AEP event, backwater flooding from the Hawkesbury River causes the greatest depths of inundation in the lower reaches of South Creek, to just downstream of the Ropes Creek confluence, where flood levels are controlled by local conditions. The largest event on record occurred in June 1867 when the level at Windsor rose to 19.7 mAHD and it was reported that the area was a level pool for many miles. The 1990 Flood Study (Reference 1) reported that in June 1867 the South Creek valley was affected as far upstream as the Great Western Highway.

Other significant floods for which levels were recorded include August 1990, April 1988, August 1986, March 1978, June 1964 November 1961 and February 1956. Table 1 shows historical flood levels at Windsor and Richmond Road for significant events.

Table 1: Historical Flood Levels - South Creek

Location	Year and Flood Level (mAHD)							
	1867	**1956**	**1961**	**1964**	**1978**	**1986**	**1988**	**1990**
Windsor [1]	19.7	13.8	15.0	14.8	14.5	11.2	12.7	13.5
Richmond Road [2]	NA	13.5	14.8	NA	14.5	11.4	12.8	NA

Notes: (1) Source Reference 3
(2) Source Reference 1
NA Not Available

3. DATA

3.1 Rainfall Data

The design rainfall information calculated by WM for the Warragamba Dam Auxiliary Spillway EIS Flood Study (Reference 3) was used in the present study to investigate Hawkesbury River backwater flooding scenarios. For local South Creek flooding, design rainfalls over a range of durations were estimated for the 20%, 5% and 1% AEP events using the procedures outlined in Australian Rainfall and Runoff, 1987 (AR&R87, Reference 5). The resulting design rainfall depths are presented in Table 2. Reference 2 determined PMP rainfall depths using the PMP analysis for the Nepean catchment and advice from the Bureau of Meteorology.

Table 2: Design Rainfall Depths (mm) - South Creek Catchment

Duration	Storm Frequency (AEP)		
(h)	20%	5%	1%
1	39	50	72
3	62	80	105
6	82	107	140
12	109	143	187
18	126	167	222
24	139	186	249
30	150	202	272
36	158	216	292
48	172	237	324
72	190	266	369

3.2 Survey Data

The South Creek cross-section data used in the existing RUBICON model were supplemented with nine (9) extra cross-sections. The additional cross-sections were surveyed by Connell Wagner specifically for the present study and copies of the section plots are presented in Appendix E. Details of the rail bridge over South Creek were also obtained from State Rail.

Figure 2 shows the location of the nine (9) surveyed cross-sections.

Existing hydraulic conditions for the study area were based on the surveyed cross-section information. Modelling of potential future conditions then incorporated the details proposed for the two alternative road alignments provided by the RTA. The alternative crossing alignments are indicated on Figure 2 as Option 1B (north) and Option 2 (south).

3.3 Design Flood Levels

Design flood levels at the sites of the two alternative crossings were shown in Reference 3. As mentioned earlier, Hawkesbury River flooding tends to influence or dominate flooding in the study area and the resultant flood profiles along South Creek up to the junction with Eastern Creek are relatively flat. A summary of design flood levels for a range of flood frequencies is presented in Table 3.

Table 3: Design Flood Levels at South Creek Crossing Sites

AEP (%)	Design Level (mAHD)
20	11.1
10	12.3
5	13.7
2	15.7
1	17.3
0.5	18.7
0.2 *	20.5
0.1 *	21.9
PMF *	26.4

* Note: Assuming auxiliary spilling at Warragamba Dam with 0.2% AEP fuseplug.

3.4 Description of Proposed Options

Two high level crossing options are currently being developed. Figure 2 shows the proposed route alignments. The northern option (Option 1B) crosses South Creek to the north of the railway lines while Option 2 crosses to the south.

These options were current at the time of the study and will be refined as part of the detailed design process.

3.4.1 Option 1B - Northern Option

The northern bridge consists of 26 spans of 40m width, with an overall bridge deck length of 1041m. There are 25 equally spaced piers with dimensions 2m wide by 4m deep. The highest point of obstruction on the bridge is the northern parapet at RL 18.9m. The bottom level of the bridge deck is at RL 14.9m. Details of the typical bridge section are given in Appendix A.

3.4.2 Option 2 - Southern Option

This option includes the construction of a bridge consisting of 13 spans of 34m width and an overall bridge deck length of 442.7m. There are 12 equally spaced piers with dimensions 1m wide by 5m deep. The highest point on the bridge is the northern parapet at RL 18.9m and the bottom level of the bridge deck is at RL 16m. Details of the typical bridge section are shown in Appendix B.

4. APPROACH ADOPTED

4.1 General

The potential impact of the proposed high level crossing was investigated using a combination of HEC-RAS and RUBICON hydraulic models. The HEC-RAS model contained extensive options for determining the impact of different bridge pier and alignment options. The results determined from a series of different flow combinations were then used to define a structure within the RUBICON model which then established the effect of the bridge option away from the site. Any afflux calculated by the HEC-RAS model was transferred to the RUBICON model.

The alternative crossing scenarios were analysed for Hawkesbury River backwater flooding, South Creek local flooding as well as five historical floods. Details of the approach are outlined below.

4.2 HEC-RAS Modelling

HEC-RAS was adopted as it is recognised as an industry standard for assessing the hydraulic impacts of bridges. The program incorporates several methods that were developed for the United States Federal Highway Administration. These methods have been verified against an extensive set of field observations.

A steady-state HEC-RAS model was established for the reach of South Creek containing the alternative crossing alignments. Three different models were required in order to represent:

- existing conditions,
- the northern option (Option 1B),
- the southern option (Option 2).

Each option was originally modelled using the three recommended methods in HEC-RAS (Energy, Momentum and Yarnell). For both options the Energy Method produced the greatest afflux and was adopted in accordance with the approach recommended in the software manual. Both options assume a Co-efficient of Drag (Cd) of 2 and a Pier Shape Factor (K) of 1.25. Manning's "n" values used were 0.04 and 0.06 for the channel and overbank respectively.

The hydraulic behaviour under a range of heights and flows were determined for the base condition and each option. The tables in Appendix C show the estimated head loss across the proposed route alignments for existing conditions and each of the options. For each option head loss is measured from the first natural cross-section downstream to the first upstream of the proposed bridge.

The tables show that for most flow conditions, there is very little afflux for either option. These tables (shown in Appendix C) were then inserted as a hydraulic structure in the RUBICON model.

4.3 RUBICON Modelling

A RUBICON model of the Hawkesbury/Nepean from Wallacia and Warragamba Dam to the ocean at Broken Bay was developed for the Sydney Water Auxiliary Spillway EIS Flood Study (Reference 3). This model has been used to estimate design flood levels from Penrith to Sackville, incorporating Windsor and the lower reaches of South Creek. For the purposes of this study the model was updated and refined in the South Creek area to include the additional cross-sections. A stand alone RUBICON model of South and Eastern Creeks (used the same base cross-section data) was also prepared so that different downstream boundary conditions at the junction of the Hawkesbury River could be run. Both models used Manning's "n" values of 0.05 in the overbank area as determined in Reference 3. The hydraulic characteristics of the bridge proposals were determined using HEC-RAS and incorporated into both RUBICON models.

The full RUBICON model was used for running valley-wide (Hawkesbury River) floods while the South and Eastern Creek model was used to determine the impact of local flood scenarios.

Where the HEC-RAS and RUBICON models overlapped the profiles were compared to ensure the results produced were similar.

4.4 Hawkesbury River Backwater Flooding

As part of the Auxiliary Spillway EIS Flood Study (Reference 3) a RORB model was set up for the entire Hawkesbury/Nepean catchment. A sub-set of the design storms (20%, 5%, 1%, 0.2% AEP and PMF) were run in order to obtain inflows to the hydraulic model for this study. The hydrologic conditions assumed for the Warragamba study were deemed to be still applicable to the present study. Further hydrologic modelling was therefore not required and the inflow hydrographs previously generated were adopted.

Inflows for the following historical floods were also obtained from the RORB model:

- November 1961,
- March 1978,
- August 1986,
- April/May 1988,
- August 1990.

The August 1986 and April/May 1988 floods were significant floods for both the Hawkesbury River and South Creek catchments.

During a main river (Hawkesbury) flood event the river typically reaches a peak level about a day after the peak flow into South Creek. The historical timing difference has ranged from 22 hours in June 1964 to 42 hours in November 1961. The average for the six largest events since 1961

(incorporating 1961, 1964, 1978, 1986, 1988 and 1990) is 32 hours which is similar to the 1% AEP event timing of 29 hours.

As indicated in Figure 3, the study area experiences three (3) separate stages of flooding during a typical main river (Hawkesbury River) flood event. Initially, floodwaters from the local South Creek catchment, which has a shorter response time, flow down South Creek towards the Hawkesbury River at Windsor. As flooding increases in the main Hawkesbury River, the floodwaters back up and the flow direction reverses along South Creek to fill the floodplain upstream of the proposed crossings. Once the peak level in the Hawkesbury River has been reached and floodwaters begin to recede, the flow reverses again and the third stage occurs as the floodplain storage drains back out of South Creek to the main river. It should be noted that it is also possible for South Creek to peak after the Hawkesbury River peak has passed.

The peak inflow and outflow during Hawkesbury River flooding for both design and modelled historical events under existing conditions are shown in Table 4.

Table 4: Peak Inflow/Outflow - Hawkesbury River Flooding

Event	Peak Inflow from the Hawkesbury (m^3/s)	Peak Outflow to the Hawkesbury (m^3/s)
20% AEP	60	210
5% AEP	530	470
1% AEP	880	1100
0.2% AEP	130	1850
PMF	3700	6100
Nov 1961	680	660
Mar 1978	570	630
Aug 1986	35	610
Apr/May 1988	260	320
Aug 1990	570	400

4.5 South Creek Local Flooding

A Watershed Bounded Network Model (WBNM) was developed for the local catchment comprising South and Eastern Creeks. WBNM was adopted in preference to RORB as it is much easier to setup and produce similar results. The model consisted of 9 sub-areas and had a total contributing area of 572 km^2. Design rainfall information from AR&R87 (Reference 5) was input into the model for a range of design events and durations. The WBNM model parameters were initially based on typical values recommended in AR&R87 then adjusted with a reasonable range such that the flows obtained were comparable to the results of previous studies (References 1 and 2).

The model results indicated that the critical duration for the local South Creek catchment was 18 hours at the location of the proposed crossing. Inflow hydrographs were then generated for the 20%, 5%, 1% AEP and PMF events. The PMF estimate is considered to be approximate only as Reference 2 does not provide sufficient information on how the PMF was derived for the process to be fully replicated.

The existing conditions were established for the 20%, 5%, 1% AEP and PMF events using various fixed tailwater levels as shown in Table 5. These tailwater levels were adopted as being representative from a determined assessment of peak levels due to Hawkesbury River backwater flooding at the confluence of Eastern Creek with South Creek. Based on historical behaviour the medium range of tailwater values were adopted as the most appropriate levels, while the high and low values have been included to establish the sensitivity of results obtained.

Table 5: Adopted Tailwater Levels - South Creek Local Flooding

Design Flood (AEP)	High Tailwater Levels (mAHD)	Medium Tailwater Levels (mAHD)	Low Level* (mAHD)
20%	13.7 (5%)	10.8 (20%)	8.0 (50%)
5%	13.7 (5%)	10.8 (50%)	8.0 (50%)
1%	17.3 (1%)	13.7 (5%)	10.8 (20%)
PMF	21.3 (1%)	13.7 (5%)	10.8 (20%)

* The tailwater levels are for Hawkesbury River backwater flooding at the confluence of South Creek and Hawkesbury River with the AEP shown in brackets.

It should be noted that the hydraulic performance of the proposed bridge will not be affected by any increase in urbanisation within the South Creek catchment. This is because the peak flows experienced at the proposed alignments are dominated by Hawkesbury River backwater flooding. The Hawkesbury River catchment is very large and is protected from development by the dominant land uses of water catchment, National Park and stock grazing. Any increase in development will be insignificant in relative terms and therefore have an indiscernible effect on flows.

5. HYDRAULIC IMPACTS

Although backwater flooding from the Hawkesbury River produces the highest flood levels within the study area, velocities are low and neither route has a measurable impact on the flood levels achieved. The maximum impact at any location was less than 0.01 m for the range of design floods examined. Impacts less than this are within the accuracy limits of the model.

The potential hydraulic impacts (relative change in level) of the northern (Option 1B) and southern (Option 2) crossing alignments are summarised in Tables 6, 7 and 8, for Hawkesbury River, local South Creek and historical flood events respectively.

Table 6: Hydraulic Impacts - Hawkesbury River Flooding (m)

	Relative Change in Flood Level (m)				
Location	**20%**	**5%**	**1%**	**0.2%**	**PMF**
Northern Option (Option 1B):					
Richmond Road	-	-	-	-	-
Eastern Creek Junction	-	-	-	-	-
U/S of Proposed South Crossing	-	-	-	-	-
U/S Railway Crossing	-	-	-	-	-
D/S Railway Crossing	-	-	-	-	-
D/S of Proposed North Crossing	-	-	-	-	-
Windsor Road	-	-	-	-	-
Southern Option (Option 2):					
Richmond Road	-	-	-	-	-
Eastern Creek Junction	-	-	-	-	-
U/S of Proposed South Crossing	-	-	-	-	-
U/S Railway Crossing	-	-	-	-	-
D/S Railway Crossing	-	-	-	-	-
D/S of Proposed North Crossing	-	-	-	-	-
Windsor Road	-	-	-	-	-

Table 7: Hydraulic Impacts - South Creek Local Flooding (m)

	Relative Change in Flood Level (m)				
Location	**20%**	**5%**	**1%**	**0.2%**	**PMF**
Northern Option (Option 1B):					
Richmond Road	-	-	-	-	-
Eastern Creek Junction	-	-	-	-	-
U/S of Propose South Crossing	-	-	-	-	-
U/S Railway Crossing	-	-	-	-	-
D/S Railway Crossing	-	-	-	-	-
D/S of Proposed North Crossing	-	-	-	-	-
Windsor Road	-	-	-	-	-
Southern Option (Option 2):					
Richmond Road	-	-	-	-	-
Eastern Creek Junction	-	-	-	-	-
U/S of Propose South Crossing	-	-	-	-	-
U/S Railway Crossing	-	-	-	-	-
D/S Railway Crossing	-	-	-	-	-
D/S of Proposed North Crossing	-	-	-	-	-
Windsor Road	-	-	-	-	-

Table 8: Hydraulic Impacts - Historical Flooding

	Relative Change in flood Level (m)				
Location	**November 1961**	**March 1978**	**August 1986**	**April/May 1988**	**August 1990**
Northern Option:					
Richmond Road	-	-	-	-	-
Eastern Creek Junction	-	-	-	-	-
U/S of Propose South Crossing	-	-	-	-	-
U/S Railway Crossing	-	-	-	-	-
D/S Railway Crossing	-	-	-	-	-
D/S of Proposed North Crossing	-	-	-	-	-
Windsor Road	-	-	-	-	-
Southern Option:					
Richmond Road	-	-	-	-	-
Eastern Creek Junction	-	-	-	-	-
U/S of Propose South Crossing	-	-	-	-	-
U/S Railway Crossing	-	-	-	-	-
D/S Railway Crossing	-	-	-	-	-
D/S of Proposed North Crossing	-	-	-	-	-
Windsor Road	-	-	-	-	-

For the 1% AEP and larger events, little if any changes to the flow behaviour are experienced for both of the alternative crossings once floodwaters reach the obvert of the crossing.

Away from the immediate vicinity of the alignment, less than 0.1 m/s change in velocity was found for any case considered.

6. CONCLUSION

Neither of the proposed options have any significant affect on flood levels, flows or velocities. The impacts are within the limits of accuracy of the models except at the proposed route alignments. These conclusions are based on the analysis of local South Creek and Hawkesbury River design floods as well as five historical events. The reason why neither option has any significant hydraulic impact is that the waterway area of both options is significantly greater than the waterway area of the adjacent railway bridge. At 13.5 mAHD (the top level of the railway bridge), Option 1B has approximately eight and one half (8.5) times the waterway area and Option 2 has three (3) times the waterway area. For this reason it is not unexpected that neither option has any measurable impact.

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South Creek High Level Crossing Stage 1 Report
Webb, McKeown and Associated Pty Ltd, April 1999

FIGURES

FIGURE 1
LOCALITY MAP

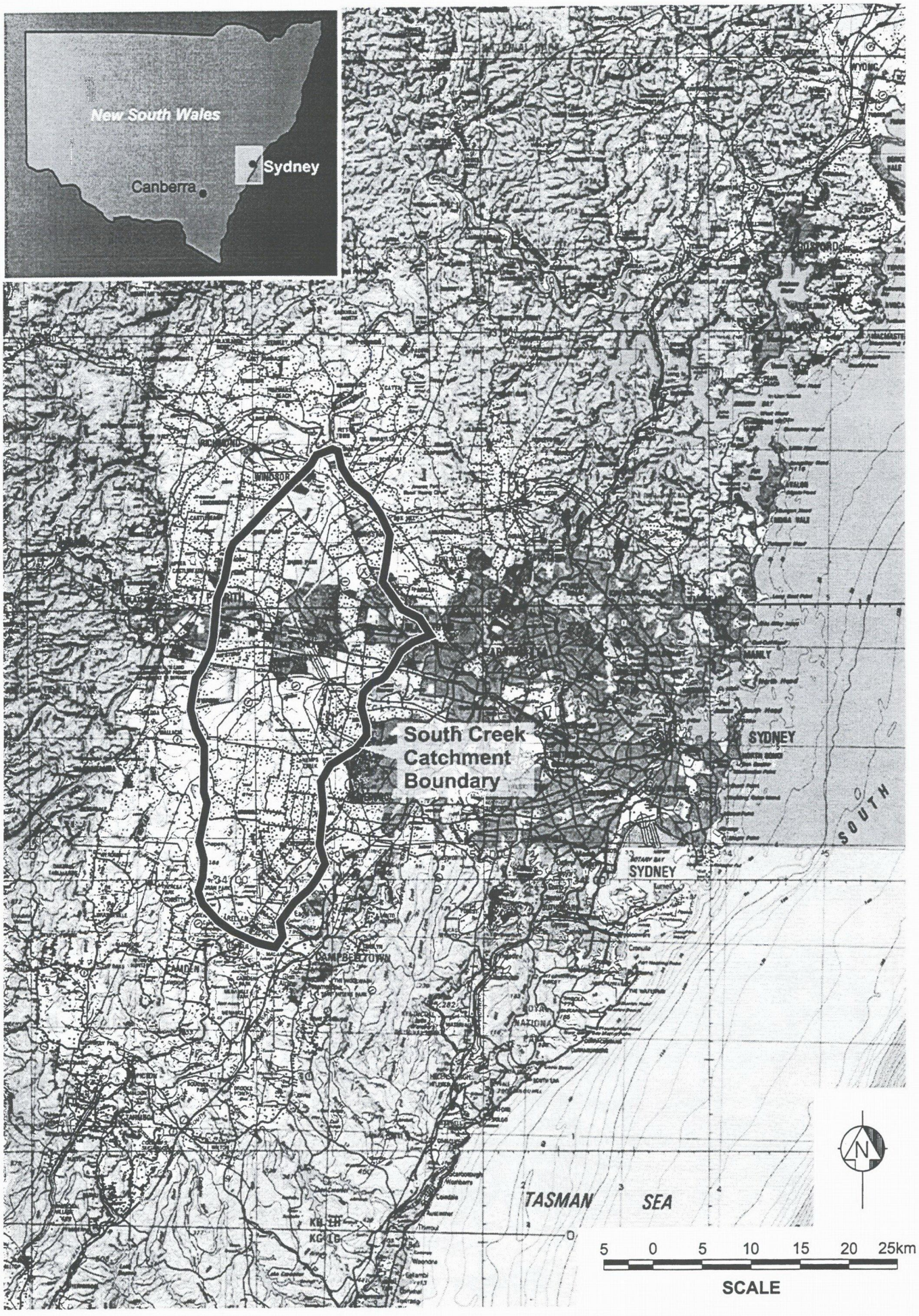

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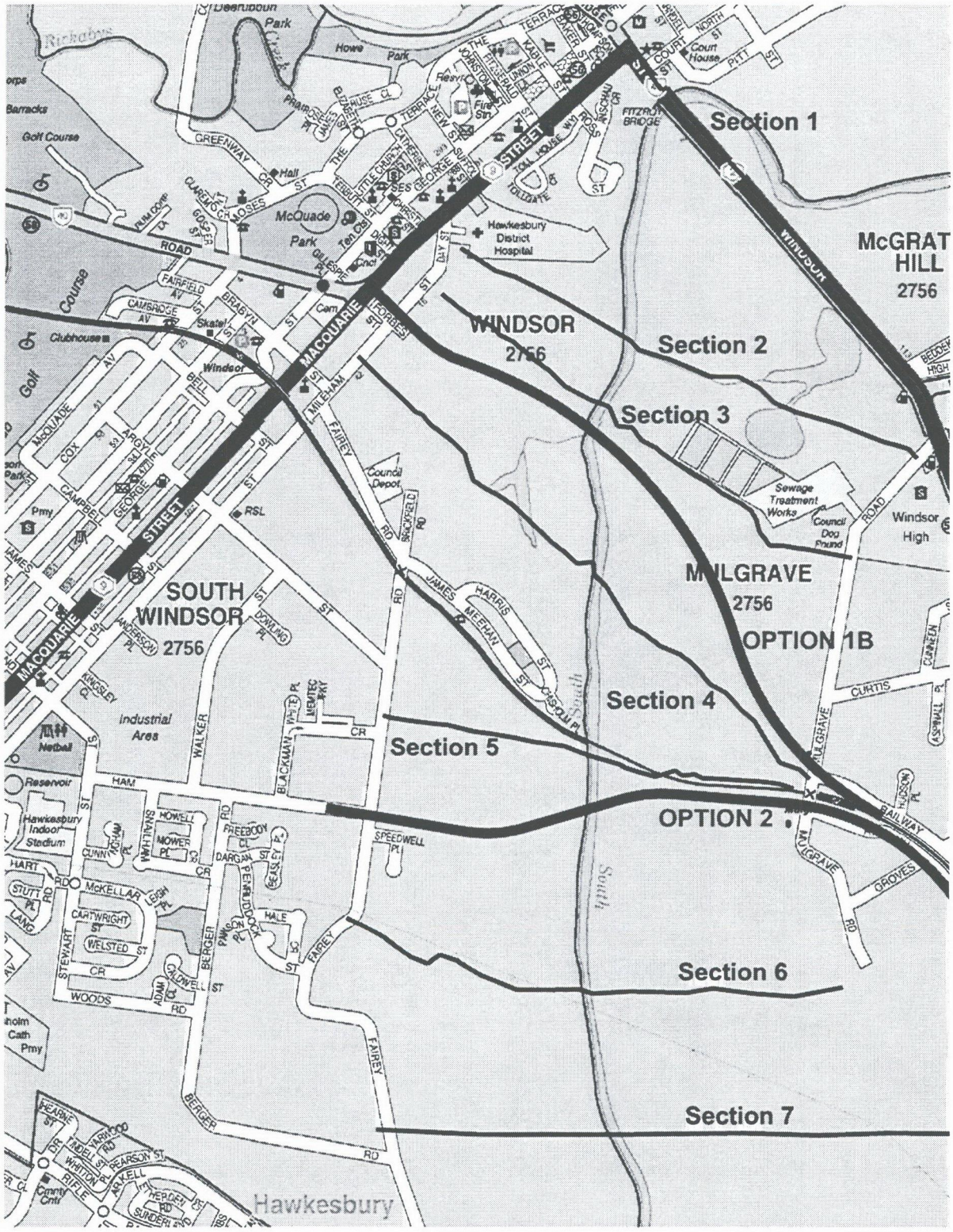
FIGURE 2

SURVEYED SECTIONS



LEGEND

Surveyed Section
Surveyed Possible Road Alignment

NOT TO SCALE

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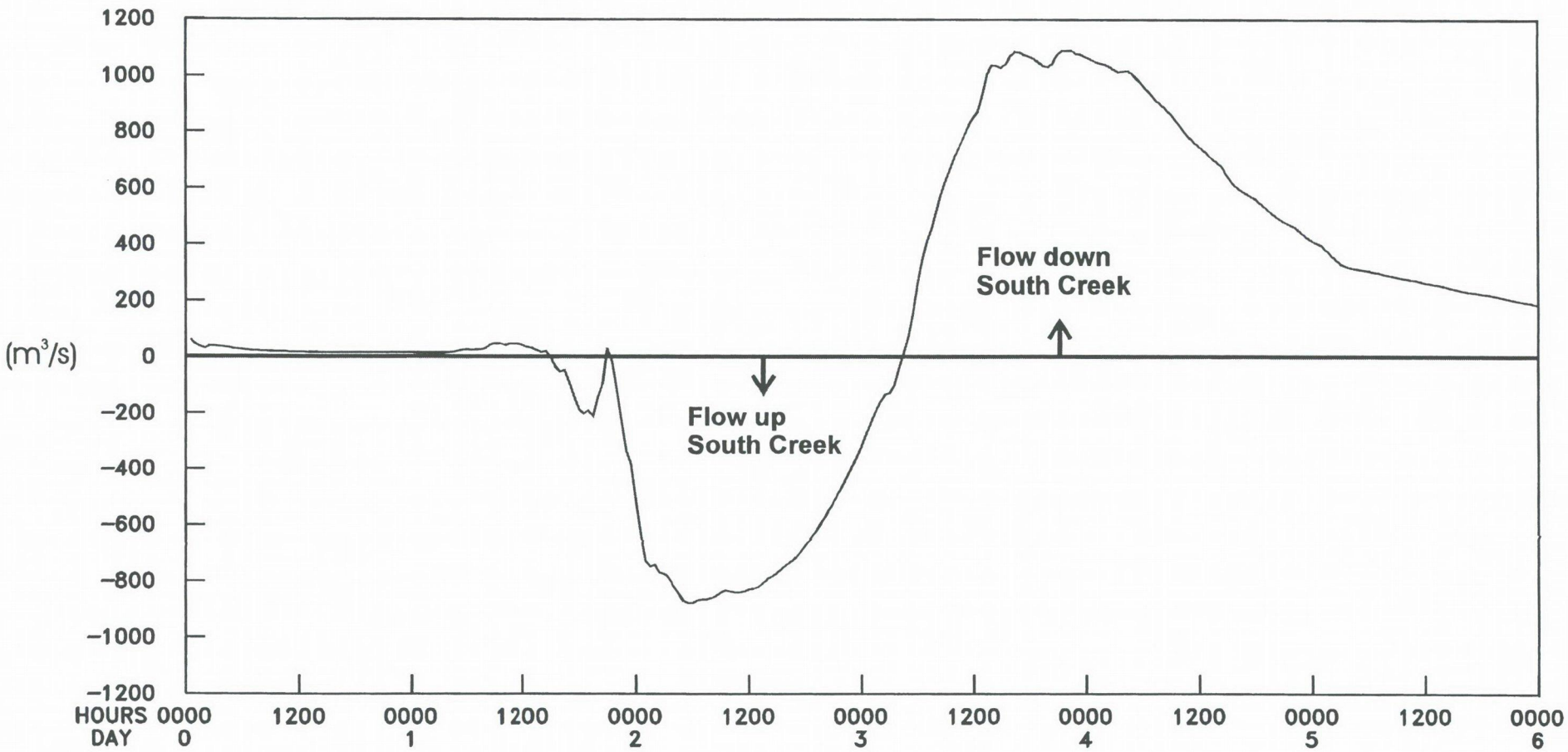

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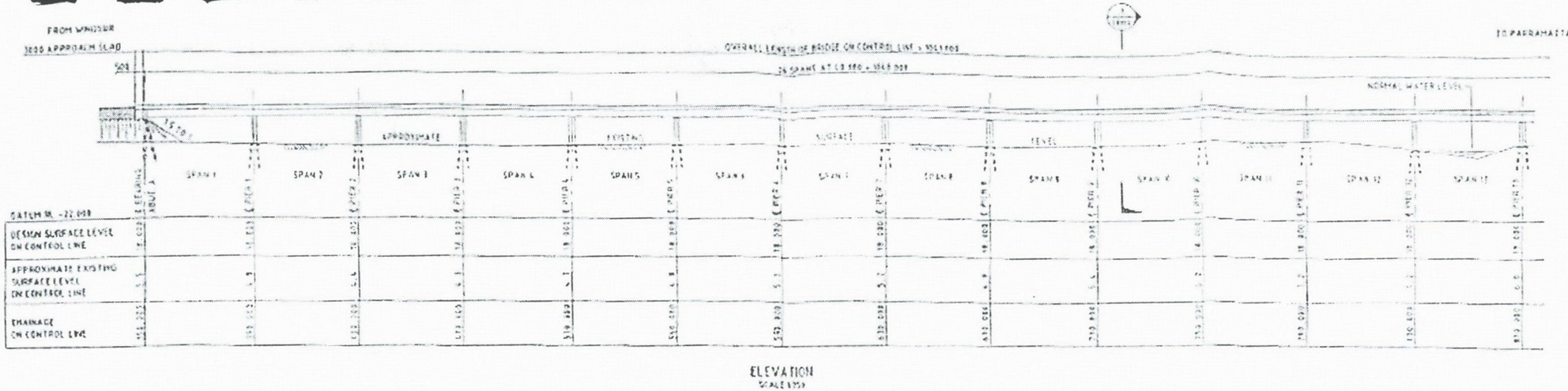
FIGURE 3

1 in 100 YEAR FLOW
HYDROGRAPH FOR SOUTH CREEK
DOWNSTREAM OF EASTERN CREEK CONFLUENCE

APPENDIX A: DETAIL OF NORTHERN OPTION (OPTION 1B)



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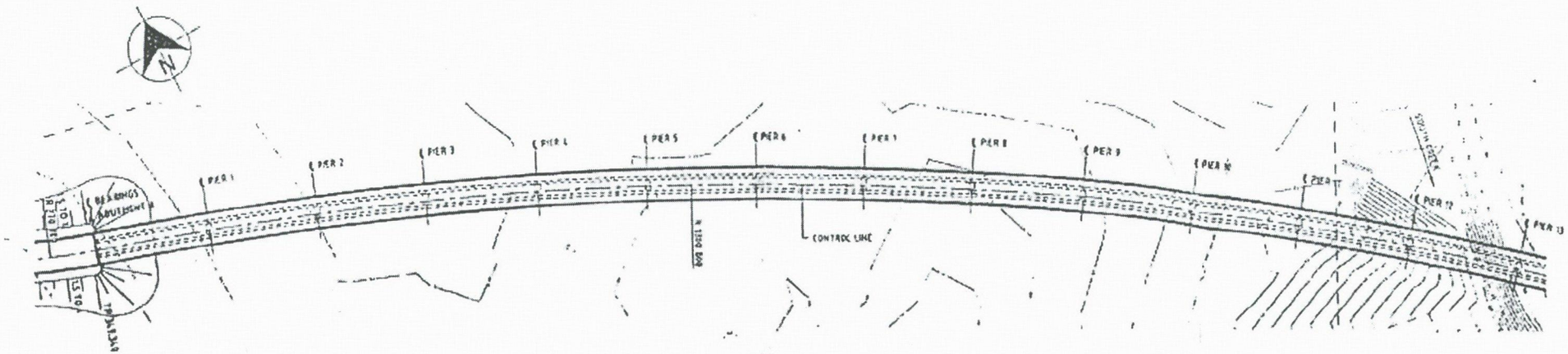


PLAN
SCALE 1:750

NOTES

DIMENSIONS ARE IN MILLIMETRES.
CHAINAGES AND REDUCED LEVELS ARE IN METRES
REDUCED LEVELS ARE RELATED TO AHD.
TP DENOTES TANGENT POINT.
SM 1600 LOADING

SCALE 1:750

PRELIMINARY

01	PRELIMINARY ISSUE	RZ			
Rev	Date	Revision Details	By	Ver	Auth

Connell Wagner

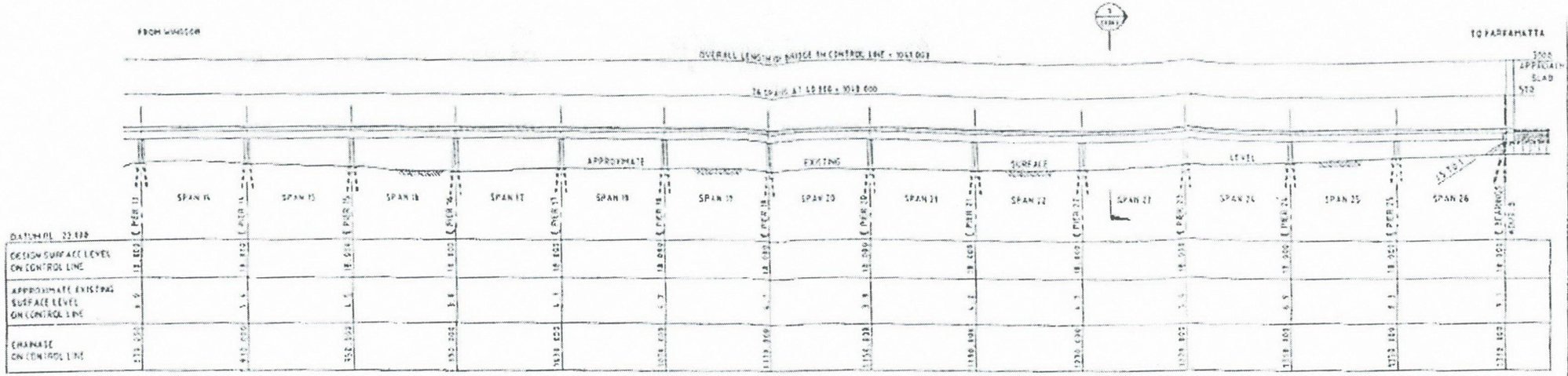
RTA

SOUTH CREEK
FLOOD EVACUATION ROUTE
OPTION 1

Drawn	RZ	Signed	Date
Designed	IDS	Signed	Date
Surveyed		Signed	Date
Approved		Signed	Date

GENERAL ARRANGEMENT
SHEET A

CW Project No	1045IBCB
Scale	AS SHOWN
Drawing No	C8001
Rev	01



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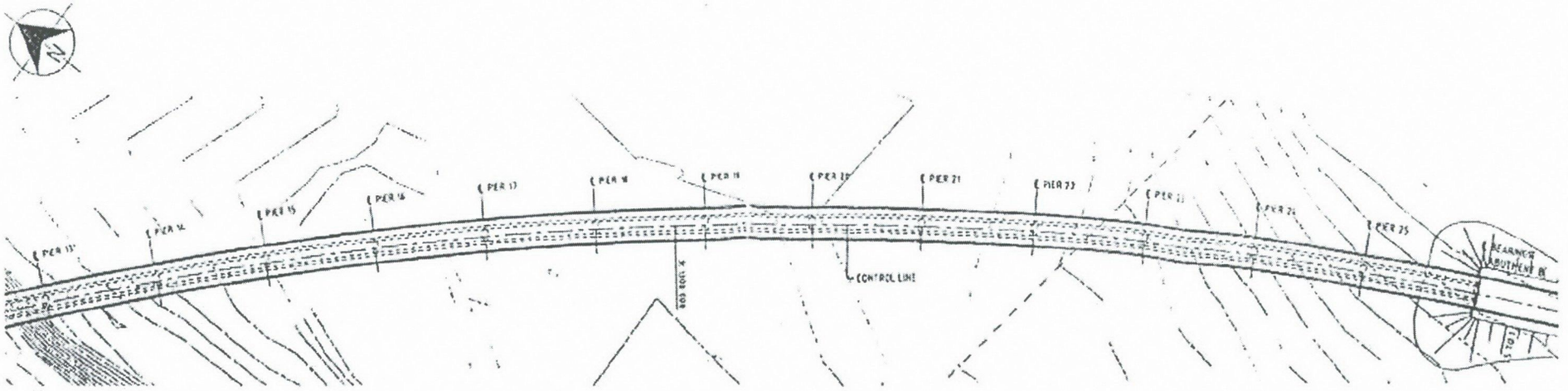

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NOTES

FOR NOTES RELATING TO THIS SHEET, REFER TO DRAWING No. CB001

PRELIMINARY

Rev	Date	Remarks	By	Chk	App
01		PRELIMINARY ISSUE	RZ		

Connell Wagner

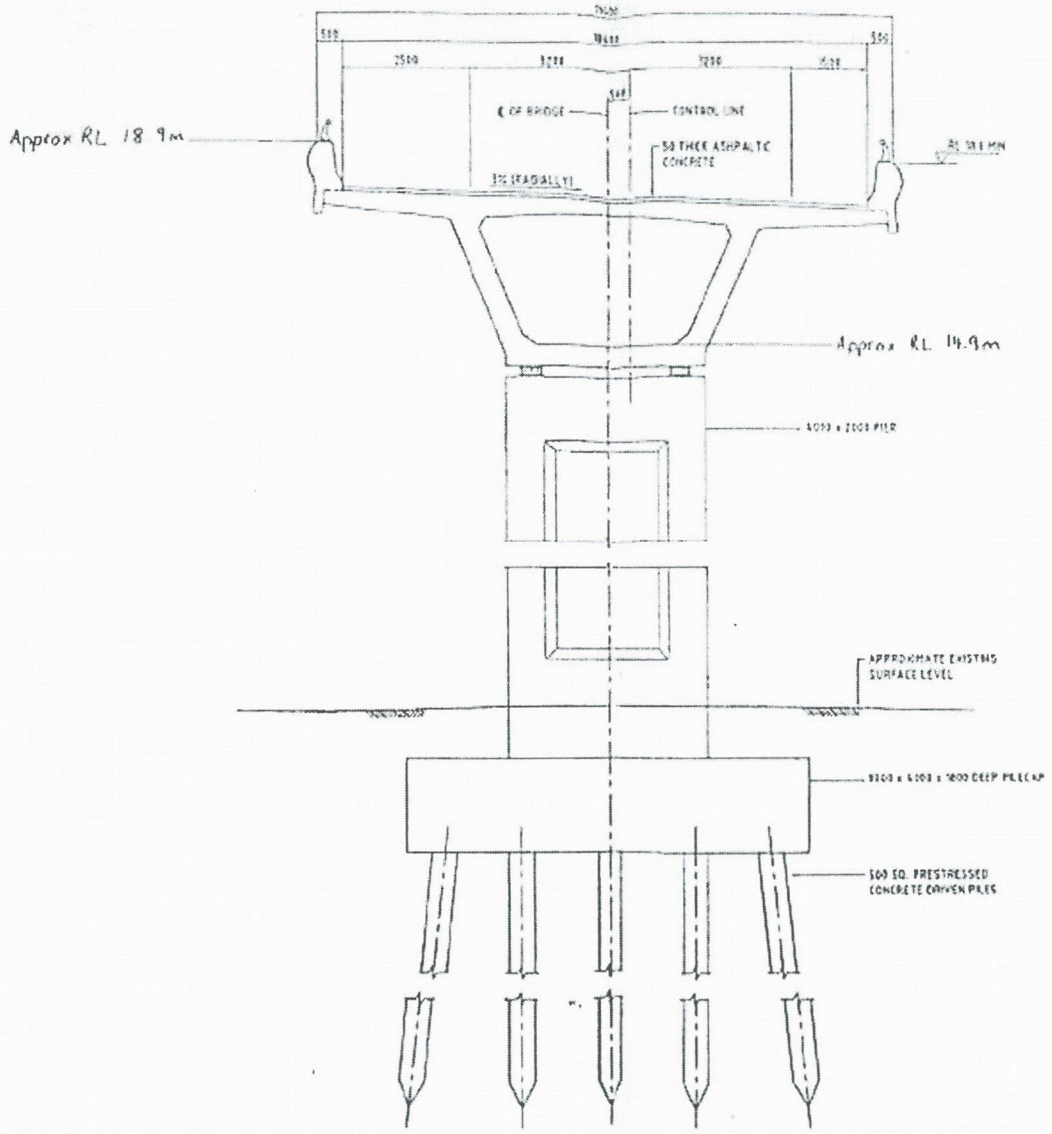
RTA

SOUTH CREEK
FLOOD EVACUATION ROUTE
OPTION 1

GENERAL ARRANGEMENT
SHEET B

Drawn	RZ	Date
Designed	RS	Date
Checked		Date
Approved		Date

Job Number	10451BC6
Size	A1
Scale	AS SHOWN
Drawing No.	CB002
Rev	01



SECTION 1
SCALE 1:50 (CB031) (CB032)

NOTES

FOR NOTES RELATING TO THIS SHEET, REFER TO DRAWING No. CB001

PRELIMINARY

Rev	Date	Revision Details	By	Chk	App
01		PRELIMINARY ISSUE	RZ		

Connell Wagner

RTA

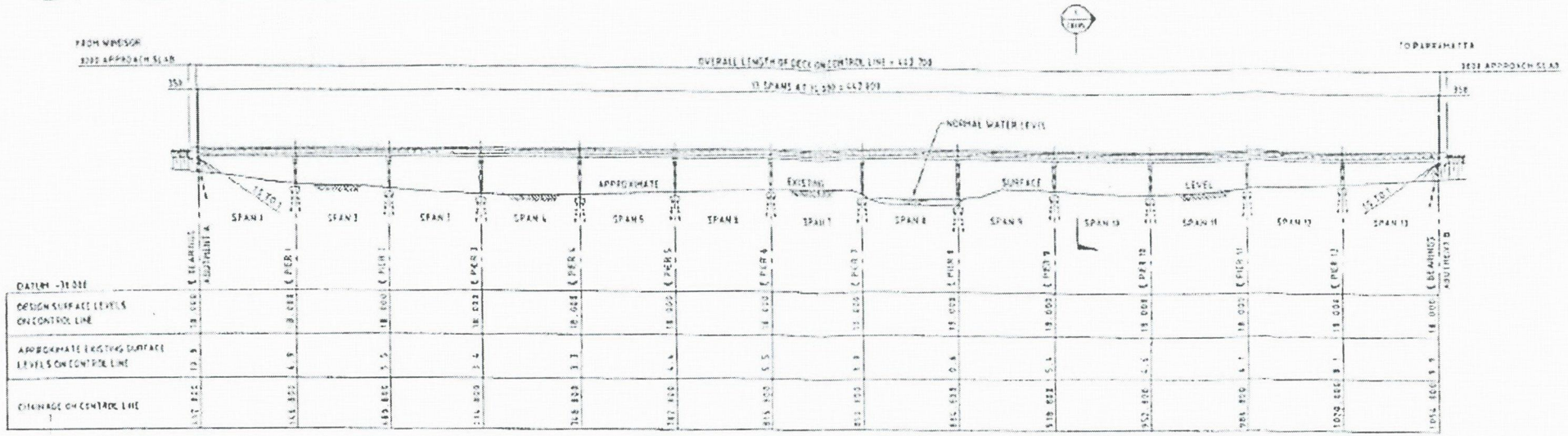
SOUTH CREEK
FLOOD EVACUATION ROUTE
OPTION 1

GENERAL ARRANGEMENT
SHEET C

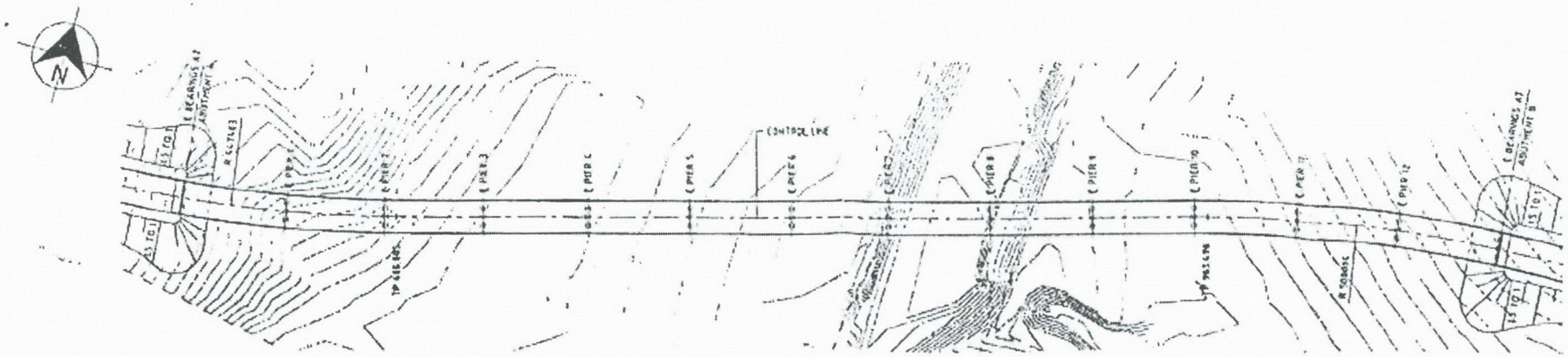
Drawn	RZ	Signed	Date
Designed		Signed	Date
Checked	IDS	Signed	Date
Verified		Signed	Date
Approved		Signed	Date

For Project No	10451BCB
Scale	AS SHOWN
Drawing No	CB003
Rev	01

APPENDIX B: DETAIL OF SOUTHERN OPTION (OPTION 2)



ELEVATION
SCALE 1:250



PLAN
SCALE 1:250

NOTES

T44 DESIGN LOADING
DIMENSIONS ARE IN MILLIMETRES
CHAINAGES AND REDUCED LEVELS ARE IN METRES
REDUCED LEVELS ARE RELATED TO AHD
TP DENOTES TANGENT POINT

SCALE 1:250

PRELIMINARY

No.	Date	Revision Details	By
01		PRELIMINARY ISSUE	RZ

Connell Wagner

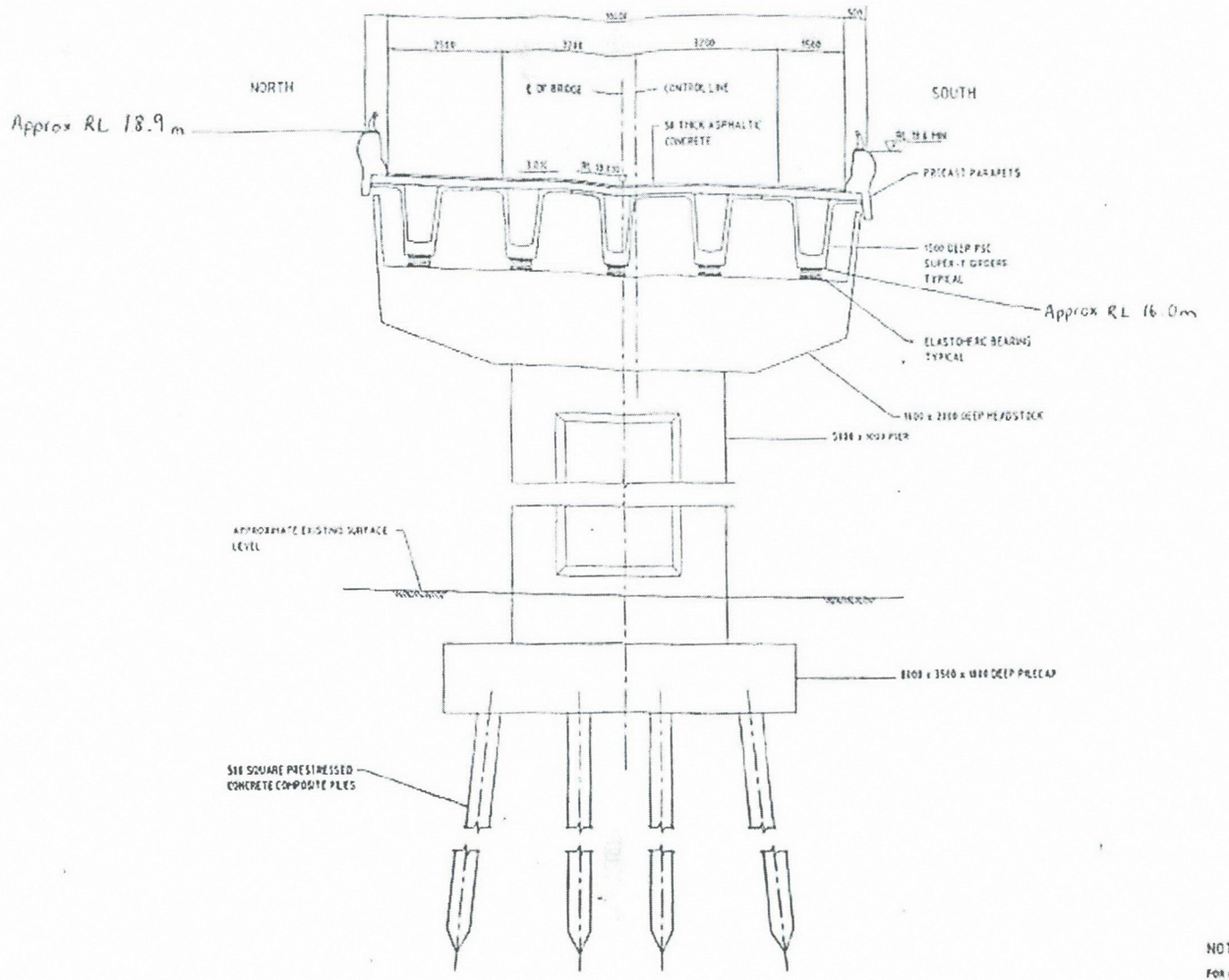
RTA

SOUTH CREEK
FLOOD EVACUATION ROUTE
OPTION 2

GENERAL ARRANGEMENT
SHEET A

Drawn	RZ	
Designed	RDS	
Verified		
Approved		

Job Project No. 10451BCB
Scale AS SHOWN
Drawing No. CB004 Rev. 01



SECTION 1 / SCALE 1:50 (A1 SHEET)

NOTES

FOR OTHER GENERAL NOTES RELATING TO THIS DRAWING REFER DRAWING No C8104

PRELIMINARY

Rev	Date	Revision Details	By	Chk	App'd
01		PRELIMINARY ISSUE	JTI		

Connell Wagner

RTA

SOUTH CREEK
FLOOD EVACUATION ROUTE
OPTION 2

Drawn JTI	Signed		Date
Designed JDS	Signed		Date
Checked	Signed		Date
Approved	Signed		Date

GENERAL ARRANGEMENT
SHEET 8

Cw Project No	104518C8
Scale	AS SHOWN
Drawing No	C8005
Rev	01

APPENDIX C: HEC-RAS RESULTS

EXISTING CASE: OPTION 1B

		Flow (m3/s)															
		0.5	1	2.5	5	10	20	50	100	200	500	1000	2000	4000	6100	20000	100000
		Upstream Water Levels															
DS Level (m)	0.00	0.00	0.00	0.00	0.02	0.08	0.30	1.61	2.43	3.11	4.37	5.25	6.02	6.82	7.43	9.78	15.67
	1.00	1.00	1.00	1.00	1.01	1.04	1.15	1.87	2.43	3.11	4.37	5.25	6.02	6.82	7.43	9.78	15.67
	2.00	2.00	2.00	2.00	2.00	2.00	2.02	2.13	2.44	3.11	4.37	5.25	6.02	6.82	7.43	9.78	15.67
	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.02	3.08	3.31	4.37	5.25	6.02	6.82	7.43	9.78	15.67
	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.02	4.06	4.42	5.25	6.02	6.82	7.43	9.78	15.67
	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.01	5.05	5.27	6.02	6.82	7.43	9.78	15.67
	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.01	6.04	6.20	6.82	7.43	9.78	15.67
	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.01	7.05	7.21	7.50	9.78	15.67
	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.01	8.02	8.07	8.16	9.78	15.67
	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.01	9.03	9.07	9.85	15.67
	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.01	10.03	10.38	15.67
	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.01	11.13	15.67
	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.06	15.67
	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.01	15.67
	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.01	15.67
	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.67
	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.09
	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.06
	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.04
	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.02
	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.01
	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.01
	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00
	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00
	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00
	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00

Note: Shading indicates range used by RUBICON model

PROPOSED CASE: OPTION 1B

DS Level (m)	Flow (m3/s)															
	0.5	1	2.5	5	10	20	50	100	200	500	1000	2000	4000	6100	20000	100000
	Upstream Water Levels															
0.00	0.00	0.00	0.00	0.02	0.08	0.30	1.61	2.44	3.12	4.44	5.46	6.39	7.32	8.05	11.32	23.12
1.00	1.00	1.00	1.00	1.01	1.04	1.15	1.87	2.44	3.12	4.44	5.46	6.39	7.32	8.05	11.32	23.13
2.00	2.00	2.00	2.00	2.00	2.00	2.02	2.13	2.44	3.12	4.44	5.46	6.39	7.32	8.05	11.32	23.13
3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.02	3.08	3.32	4.44	5.46	6.39	7.32	8.05	11.32	23.13
4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.02	4.08	4.50	5.46	6.39	7.32	8.05	11.32	23.13
5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.01	5.09	5.47	6.39	7.32	8.05	11.32	23.13
6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.01	6.02	6.10	6.43	7.32	8.05	11.32	23.13
7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.01	7.03	7.11	7.46	8.05	11.32	23.13
8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.01	8.04	8.18	8.43	11.32	23.13
9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.01	9.02	9.09	9.21	11.32	23.13
10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.01	10.05	10.12	11.43	23.13
11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.01	11.03	11.07	11.82	23.13
12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.02	12.04	12.50	23.13
13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.01	13.03	13.35	23.13
14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.01	14.02	14.26	23.13
15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.01	15.02	15.22	23.13
16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.01	16.02	16.21	23.13
17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.01	17.02	17.20	23.14
18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.01	18.02	18.19	23.58
19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.01	19.02	19.19	24.20
20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.01	20.02	20.16	24.92
21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.01	21.01	21.13	25.71
22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.01	22.11	26.29
23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.01	23.09	26.12
24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.01	24.08	26.42
25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.01	25.07	27.00

Note: Shoading indicates range used by RUBICON model

EXISTING CASE: OPTION 2

DS Level (m)	Flow (m3/s)															
	0.5	1	2.5	5	10	20	50	100	200	500	1000	2000	4000	6100	20000	100000
	Upstream Water Levels															
0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.27	1.00	2.51	5.37	6.38	7.37	8.71	9.78	14.99	22.53
1.00	1.00	1.00	1.00	1.00	1.00	1.02	1.10	1.41	2.50	5.37	6.38	7.37	8.71	9.78	14.99	22.53
2.00	2.00	2.00	2.00	2.00	2.00	2.01	2.04	2.17	2.72	5.37	6.38	7.37	8.71	9.78	14.99	22.53
3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.02	3.08	3.35	5.37	6.38	7.37	8.71	9.78	14.64	22.53
4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.01	4.05	4.22	5.37	6.38	7.37	8.71	9.78	14.75	22.53
5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.02	5.07	5.49	6.38	7.37	8.71	9.78	14.76	22.53
6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.01	6.02	6.12	6.45	7.37	8.71	9.78	14.78	22.53
7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.01	7.03	7.12	7.51	8.71	9.78	15.16	22.53
8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.01	8.04	8.18	8.79	9.78	15.16	22.53
9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.01	9.02	9.08	9.35	9.89	14.99	22.53
10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.01	10.04	10.17	10.44	14.99	22.53
11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.01	11.02	11.10	11.24	14.99	22.53
12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.01	12.06	12.14	14.77	22.53
13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.01	13.04	13.09	14.76	22.53
14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.01	14.03	14.07	15.23	22.53
15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.02	15.04	15.56	22.53
16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.01	16.02	16.29	22.53
17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.01	17.17	22.53
18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.01	18.01	18.11	22.53
19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.01	19.07	22.53
20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.01	20.05	22.53
21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.04	22.62
22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.03	23.05
23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.02	23.71
24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.02	24.51
25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.01	25.38

Note: Shading indicates range used by RUBICON model

PROPOSED CASE: OPTION 2

DS Level (m)	Flow (m3/s)															
	0.5	1	2.5	5	10	20	50	100	200	500	1000	2000	4000	6100	20000	100000
	Upstream Water Levels															
0.00	0.00	0.00	0.00	0.00	0.01	0.04	0.27	1.00	2.51	5.38	6.39	7.38	8.73	9.79	14.27	33.28
1.00	1.00	1.00	1.00	1.00	1.00	1.02	1.10	1.41	2.50	5.38	6.39	7.38	8.73	9.79	14.14	33.28
2.00	2.00	2.00	2.00	2.00	2.00	2.01	2.04	2.17	2.72	5.38	6.39	7.38	8.73	9.79	14.27	33.28
3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.02	3.08	3.35	5.38	6.39	7.38	8.73	9.79	14.27	33.28
4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.01	4.05	4.22	5.38	6.39	7.38	8.73	9.79	14.27	33.28
5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.02	5.07	5.49	6.39	7.38	8.73	9.79	14.27	33.28
6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.00	6.01	6.02	6.12	6.45	7.38	8.73	9.79	14.27	33.28
7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.01	7.03	7.13	7.52	8.73	9.79	14.27	33.28
8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.01	8.05	8.19	8.81	9.79	14.27	33.28
9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.01	9.02	9.08	9.35	9.90	14.27	33.28
10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.01	10.04	10.17	10.44	14.27	33.28
11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.00	11.01	11.02	11.10	11.23	14.27	33.28
12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.00	12.01	12.06	12.14	14.26	33.28
13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.01	13.04	13.09	14.40	33.28
14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.00	14.01	14.02	14.06	14.88	33.28
15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.00	15.02	15.04	15.58	33.28
16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.00	16.01	16.03	16.42	33.28
17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.00	17.01	17.04	17.45	33.28
18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.01	18.02	18.04	18.45	33.28
19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.01	19.02	19.04	19.45	33.28
20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.00	20.02	20.03	20.41	33.28
21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.01	21.03	21.31	33.28
22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.00	22.01	22.02	22.25	33.28
23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.00	23.01	23.02	23.20	33.28
24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.00	24.01	24.01	24.16	33.28
25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.00	25.01	25.13	33.28

Note: Shading indicates range used by RUBICON model

AFFLUX OF PROPOSED OPTION 1B

		Flow (m3/s)															
		0.5	1	2.5	5	10	20	50	100	200	500	1000	2000	4000	6100	20000	100000
		Afflux of Proposed Option 1B															
DS Level (m)	0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0700	0.2100	0.3700	0.5000	0.6200	1.5400	7.4500
	1.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0700	0.2100	0.3700	0.5000	0.6200	1.5400	7.4600
	2.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0700	0.2100	0.3700	0.5000	0.6200	1.5400	7.4600
	3.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0700	0.2100	0.3700	0.5000	0.6200	1.5400	7.4600
	4.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0200	0.0800	0.2100	0.3700	0.5000	0.6200	1.5400	7.4600
	5.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0400	0.2000	0.3700	0.5000	0.6200	1.5400	7.4600
	6.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0600	0.2300	0.5000	0.6200	1.5400	7.4600
	7.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.0600	0.2500	0.5500	1.5400	7.4600
	8.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0200	0.1100	0.2700	1.5400	7.4600
	9.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0600	0.1400	1.4700	7.4600
	10.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0400	0.0900	1.0500	7.4600
	11.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0300	0.0600	0.6900	7.4600
	12.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0200	0.0400	0.4400	7.4600
	13.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0300	0.3400	7.4600
	14.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.2500	7.4600
	15.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.2200	7.4600
	16.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.2100	7.0400
	17.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.2000	6.0800
	18.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.1900	5.5400
	19.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.1900	5.1800
	20.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.1600	4.9100
	21.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.1300	4.7000
	22.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.1100	4.2900
	23.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0900	3.1200
	24.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0800	2.4200
	25.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0700	2.0000

Note: Shading indicates measureable afflux

AFFLUX OF PROPOSED OPTION 2

DS Level (m)	Flow (m3/s)															
	0.5	1	2.5	5	10	20	50	100	200	500	1000	2000	4000	6100	20000	100000
	Afflux of Proposed Option 2															
0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0100	0.0200	0.0100	-0.7200	10.7500
1.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0100	0.0200	0.0100	-0.8500	10.7500
2.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0100	0.0200	0.0100	-0.7200	10.7500
3.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0100	0.0200	0.0100	-0.3700	10.7500
4.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0100	0.0200	0.0100	-0.4800	10.7500
5.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0200	0.0100	-0.4900	10.7500
6.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.0100	-0.5100	10.7500
7.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0200	0.0100	-0.8900	10.7500
8.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.0200	0.0100	-0.8900	10.7500
9.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	-0.7200	10.7500
10.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.7200	10.7500
11.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.0100	-0.7200	10.7500
12.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.5100	10.7500
13.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.3600	10.7500
14.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.0100	-0.0100	-0.3500	10.7500
15.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0200	10.7500
16.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.1300	10.7500
17.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0300	0.2800	10.7500
18.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0300	0.3400	10.7500
19.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.0300	0.3800	10.7500
20.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0200	0.0200	0.3600	10.7500
21.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0300	0.2700	10.6600
22.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.2200	10.2300
23.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0200	0.1800	9.5700
24.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.0100	0.1400	8.7700
25.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0100	0.1200	7.9000

Note: Shading indicates measureable afflux

APPENDIX D: SOUTH CREEK HIGH LEVEL CROSSING - STAGE 1 REPORT

WEBB, McKEOWN & ASSOCIATES PTY. LTD.

CONSULTING ENGINEERS ACN 002 592 225

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I J Tye BScEng, MEngSc, MIEAust, CPEng, NPER3

ASSOCIATES: M K Babister BE(Hons), MEngSc, GradDipMgt, MIEAust, CPEng, NPER3
G L Hurrell BSc, BE(Hons), MIEAust, CPEng
B A Withnall BE(Hons), L&ESD Cert(Hons), GradIEAust, PEng

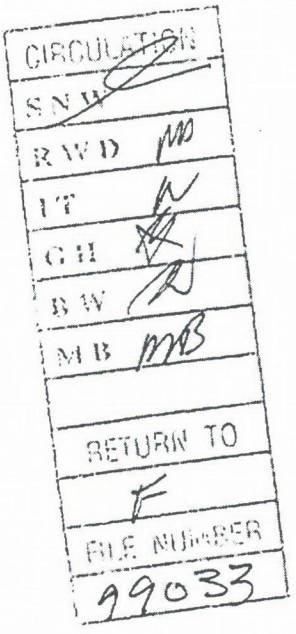
CONSULTANT: T J McKeown BSc, BE, MEngSc

OUR REF: 99033\dlwc11-99

YOUR REF:

29 April 1999

Department of Land and Water Conservation
Level 10, Signature Building
2-10 Wentworth Street
PARRAMATTA NSW 2150



Attention: **Mr A Low**
Project Manager

Dear Sir,

Re: South Creek High Level Crossing
Stage 1 Report

1. INTRODUCTION

Webb, McKeown & Associates are currently carrying out investigations to ascertain the hydraulic impact of a high level crossing over South Creek at Windsor. The work is being carried out on behalf of the Department of Land and Water Conservation (DLWC), the Roads and Transport Authority (RTA) and Hawkesbury City Council. This report describes Stage 1 of the investigation.

2. BACKGROUND

Two options have been proposed for a high level route across South Creek. The two options are shown on plans developed by the RTA {South Creek Crossing-Option 1 (Ham St to Railway Rd Sth.), Sheet 1 of 2, and South Creek Crossing-Option 2 (Forbes St to Railway Rd Nth.), Sheet 2 of 3}. Option 1 places the crossing upstream of the railway bridge. The road would run from Ham Street, Windsor to Railway Road South, Mulgrave. Option 2 places the road further north, downstream of the railway bridge. The road would connect Forbes Street, Windsor to Railway Road North, Mulgrave.

Stage 1 involved preliminary assessment of the two options shown on the aforementioned plans. For Stage 1, hydraulic modelling was carried out using the existing model with approximate contours from the plans. The aim of Stage 1 was to provide a preliminary assessment of the impacts of the options on flooding.

3. HYDRAULIC MODELLING

Hydraulic modelling was carried out using the existing Hawkesbury-Nepean RUBICON model developed by Webb McKeown on behalf of Sydney Water. The following steps were undertaken.

i. Existing cross-sections were read off the RTA plans. Five additional cross-sections were added in the local area to form the base case.

ii. Additional cross-sections were also established for the two crossing locations. For Option 1, the upstream case, three alternative waterway structures were established - a 150 m bridge, a 230 m bridge and a 350 m bridge. Three similar waterway alternatives were established for Option 2, the downstream option.

iii. The model was run with a number of design events - 5y, 10y, 20y, 50y, 100y, 200y, 500y, 1000y ARI events and the Probable Maximum Flood. Each of these events was run firstly using the base case (no crossing), and then with each of the different waterway alternatives.

iv. The different runs were then compared to the base case to establish the impacts of the crossings on flood levels.

4. RESULTS AND DISCUSSION

The modelling established that none of the options for the crossings produced changes in peak flood heights of greater than 10 mm. This is within the accuracy limits of the model. Despite the approximate nature of the analysis, a number of trends could be detected in the results on how the crossings affect flood levels. These were:

- the downstream option, Option 2, tended to have a greater impact on flood levels than Option 1,
- the greatest impacts were seen for larger events. The PMF run yielded the greatest change in flood heights and the 5 year ARI event resulted in the smallest impacts on flood heights,
- as one would expect, the greatest impacts on flood heights occurred with the shortest bridge length, i.e. where the opening under the crossing was only 150 m. The smallest impacts on flood heights occurred where the bridge was longest, i.e. 350 m.

5. STAGE 2

Stage 2 will examine the impacts of the crossing in a greater level of accuracy. The RUBICON model will be refined using detailed field survey information. Rather than approximate cross sections as used in Stage 1, five cross sections based on detailed RTA survey will be used. This will allow the model to better reflect localised flood behaviour.

Stage 2 will also consider the impacts of onsite fill recovery on flood levels and flow distribution. In Stage 2 the impacts of timing of local and main river flows will be examined together with flood level impacts during the period of hydrograph rise.

At the completion of the work, a brief report will be prepared giving a compilation of the results and discussing the hydraulic impacts of both options.

Stage 2 will commence shortly, once the necessary data are supplied.

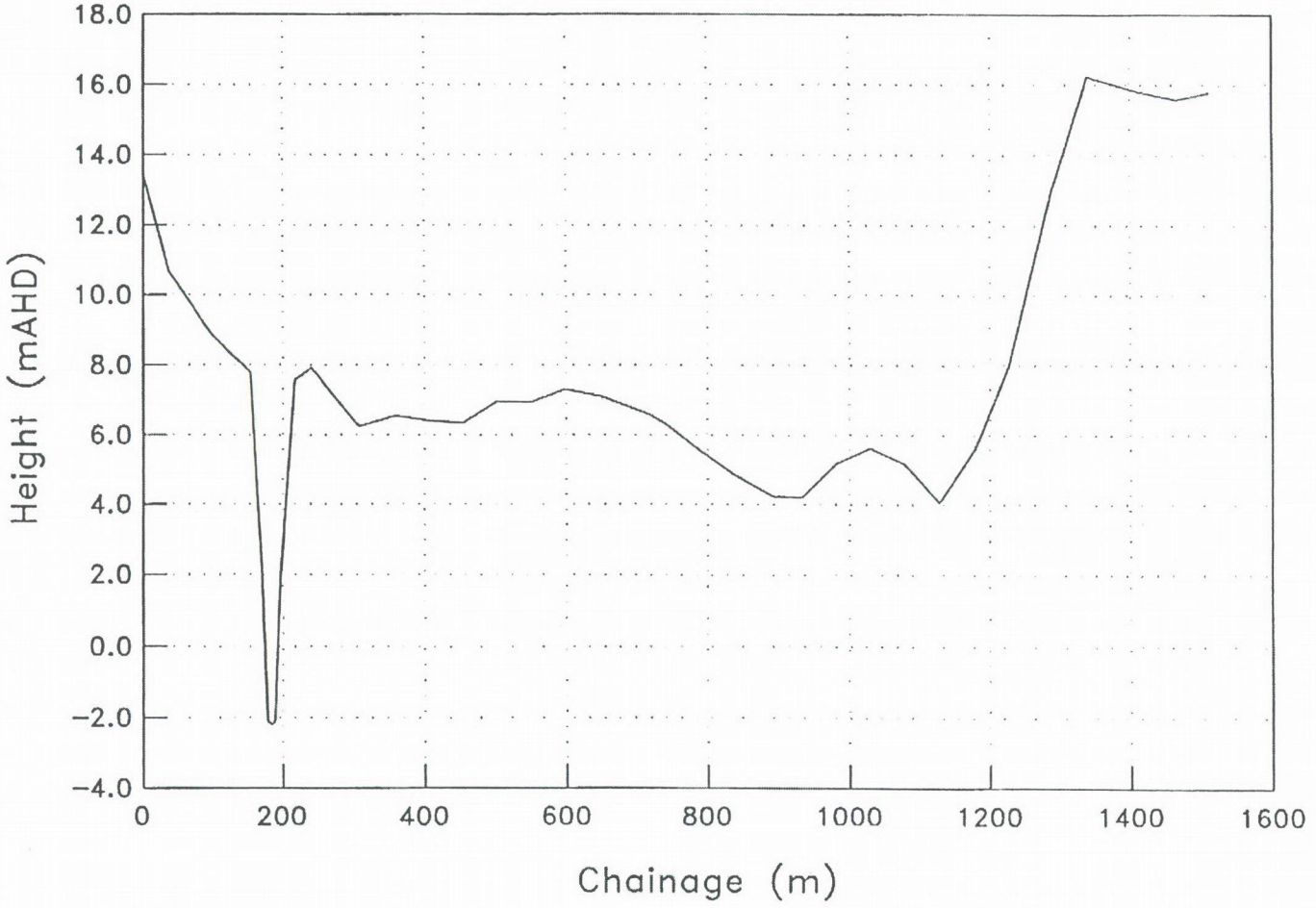
If you have require any further information please contact Mark Babister.

Yours faithfully,
WEBB, McKEOWN & ASSOCIATES PTY LTD

Dr S N Webb
Chairman

APPENDIX E: CROSS-SECTIONS SURVEYED BY CONNELL WAGNER

SECTION 1 – WINDSOR RD END



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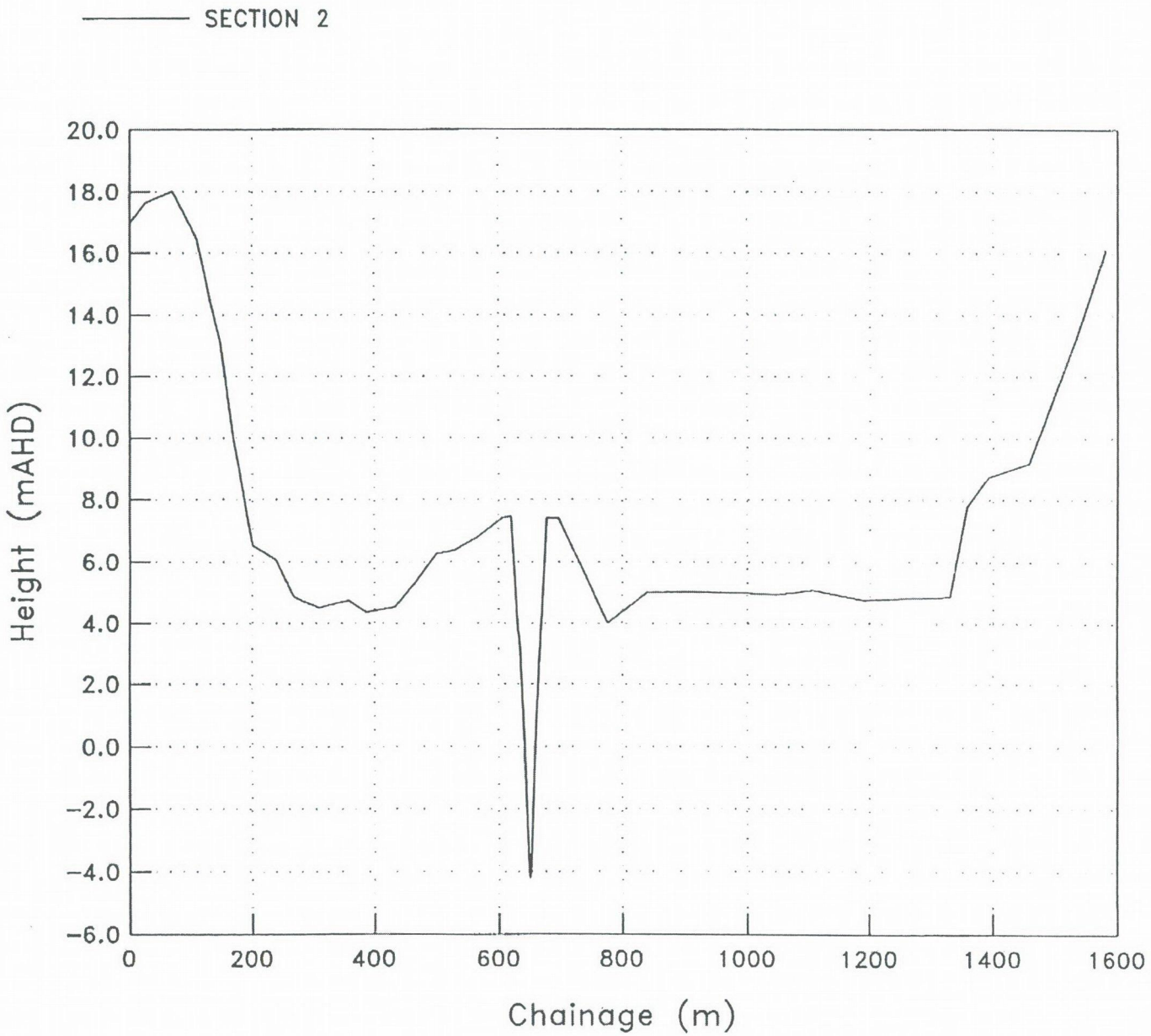
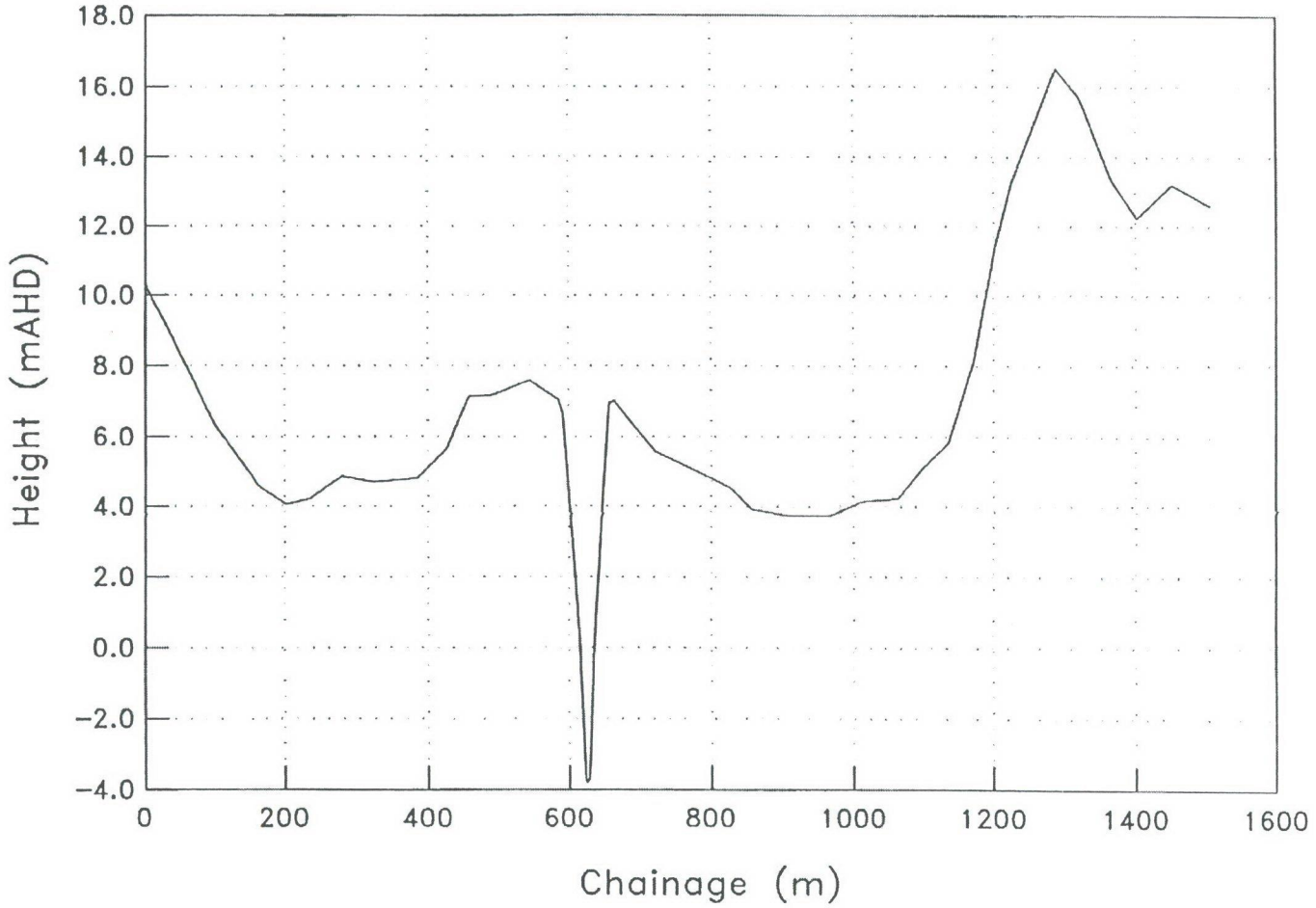
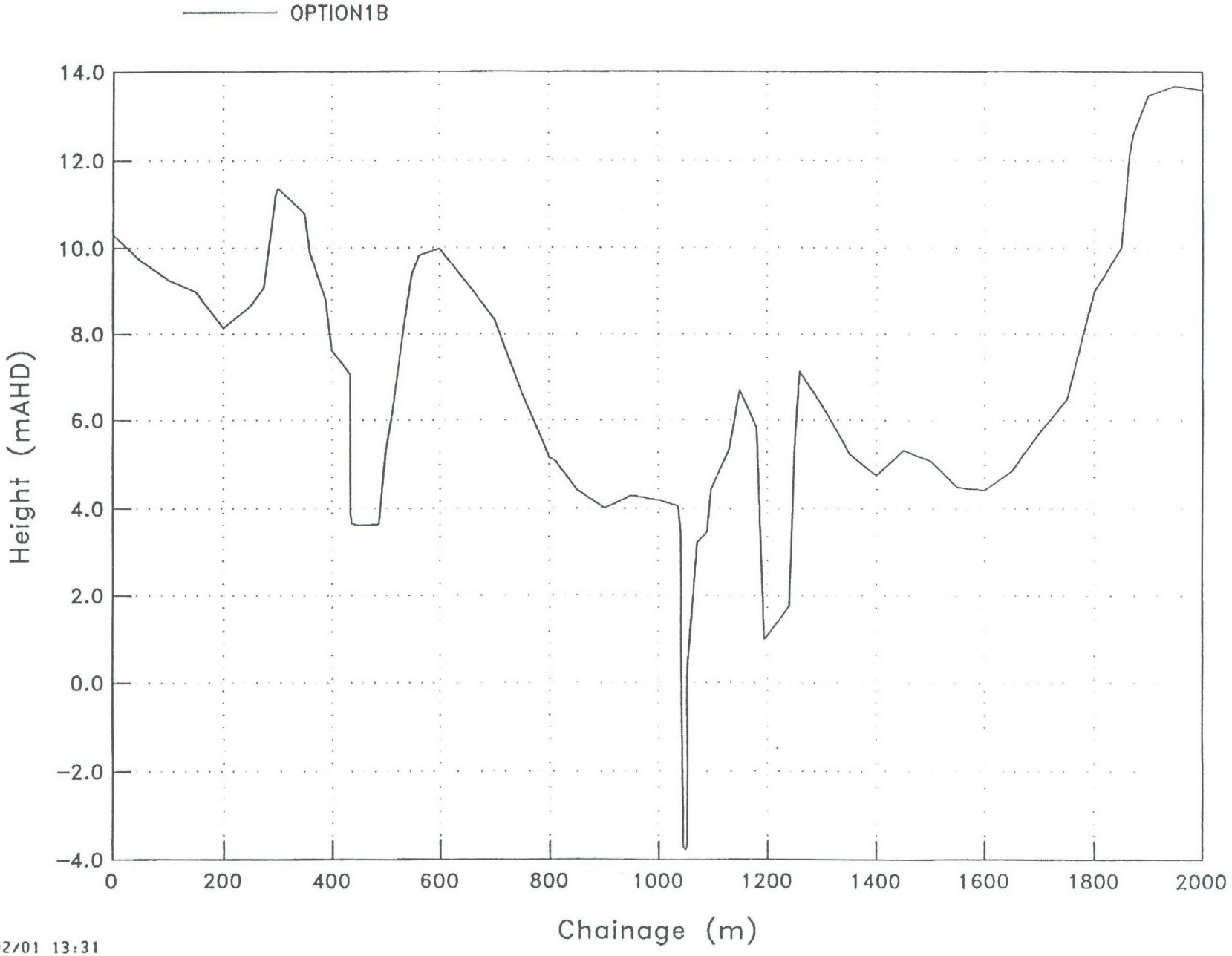

SECTION 2
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
-6.0
Height (mAHD)
0
200
400
600
800
1000
1200
1400
1600
Chainage (m)

28/02/01 13:29

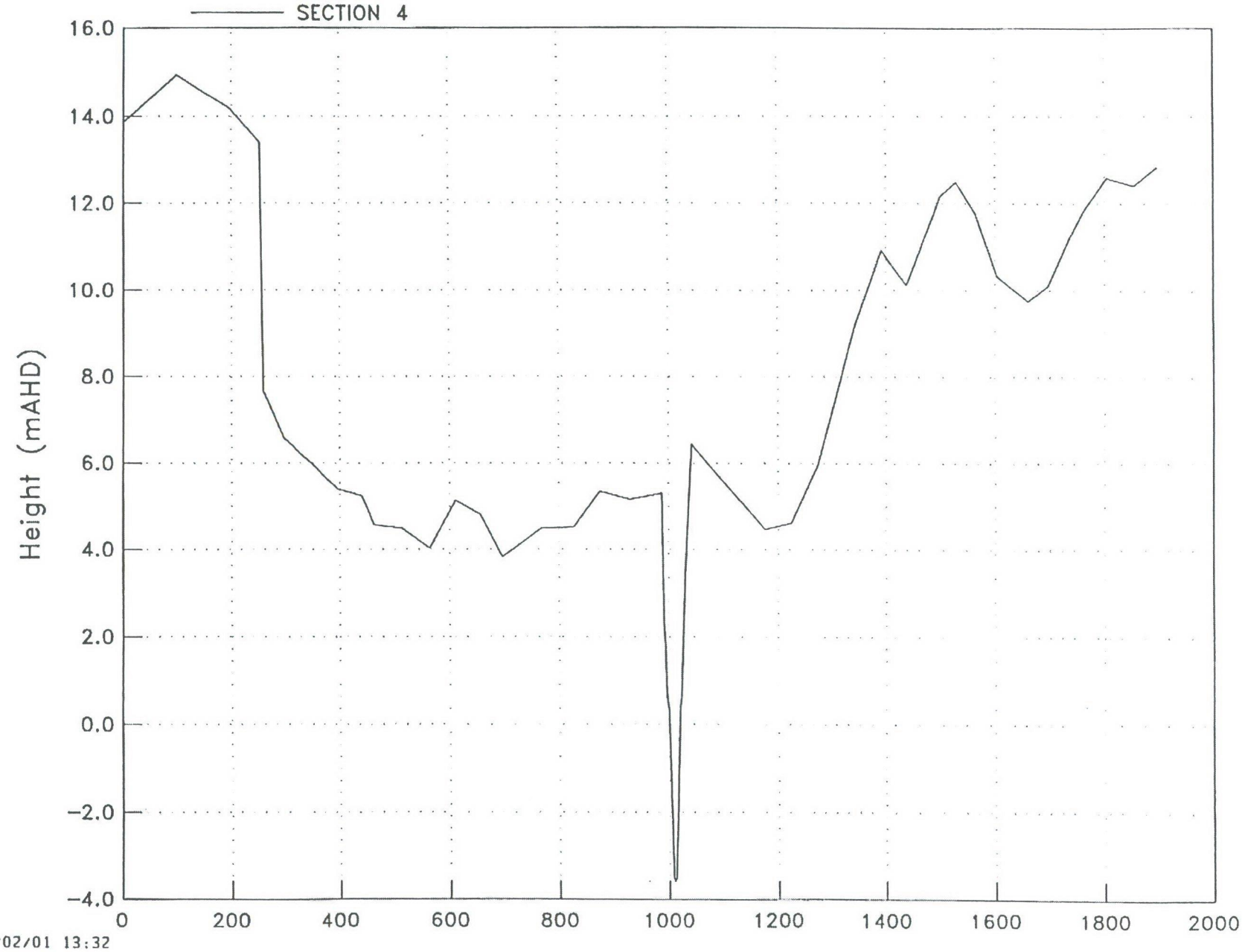
SECTION 3

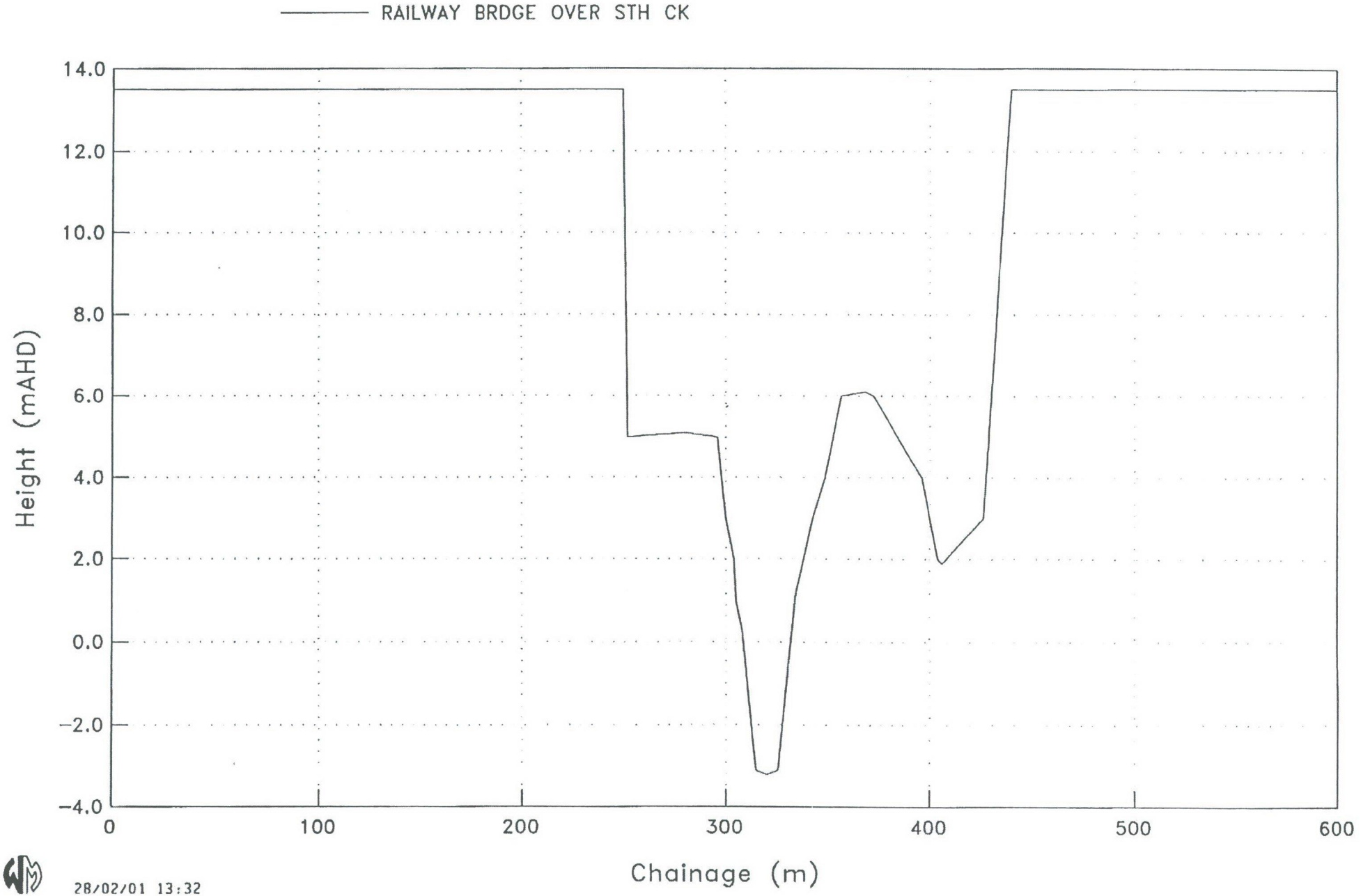


28/02/01 13:31

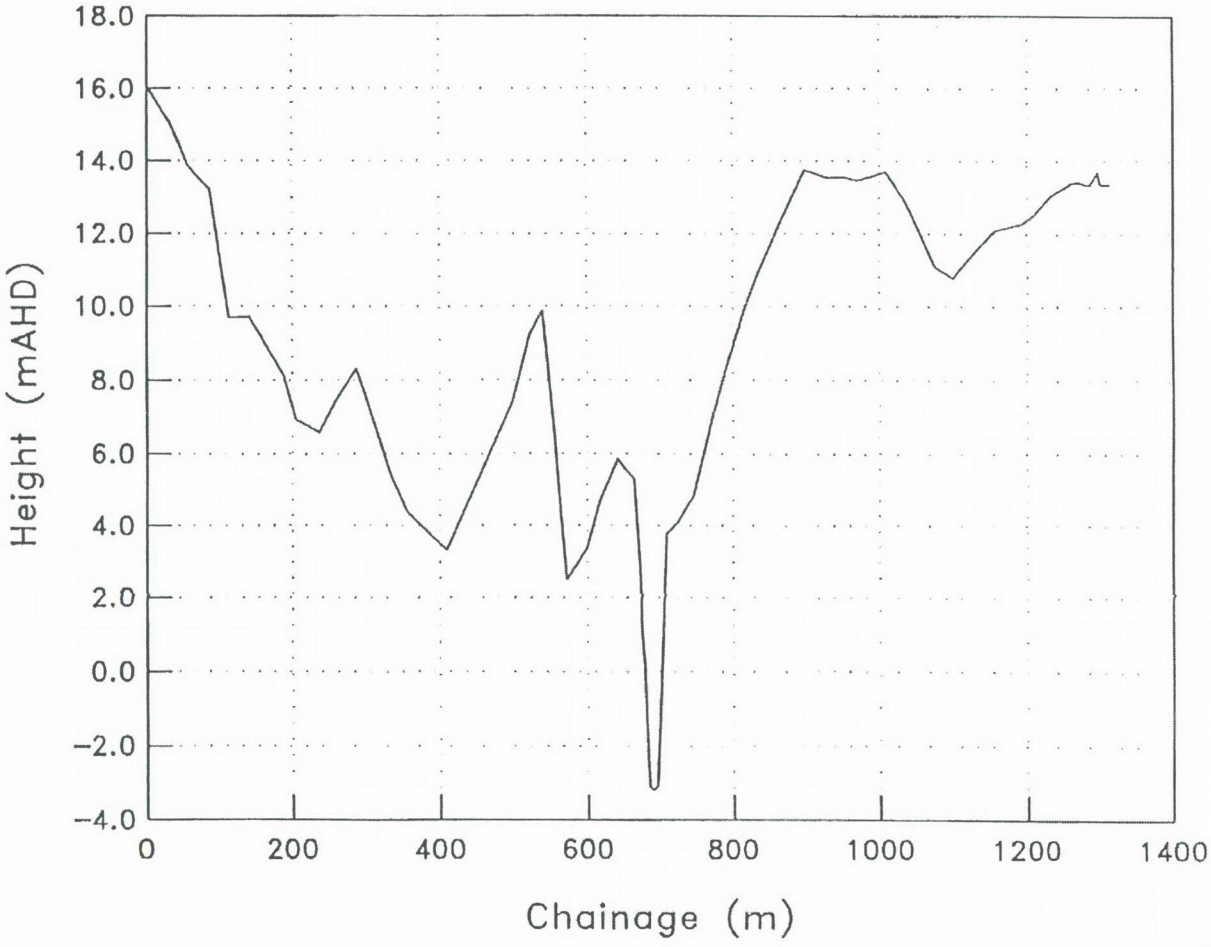

OPTION1B
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
Height (mAHD)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
Chainage (m)

28/02/01 13:31

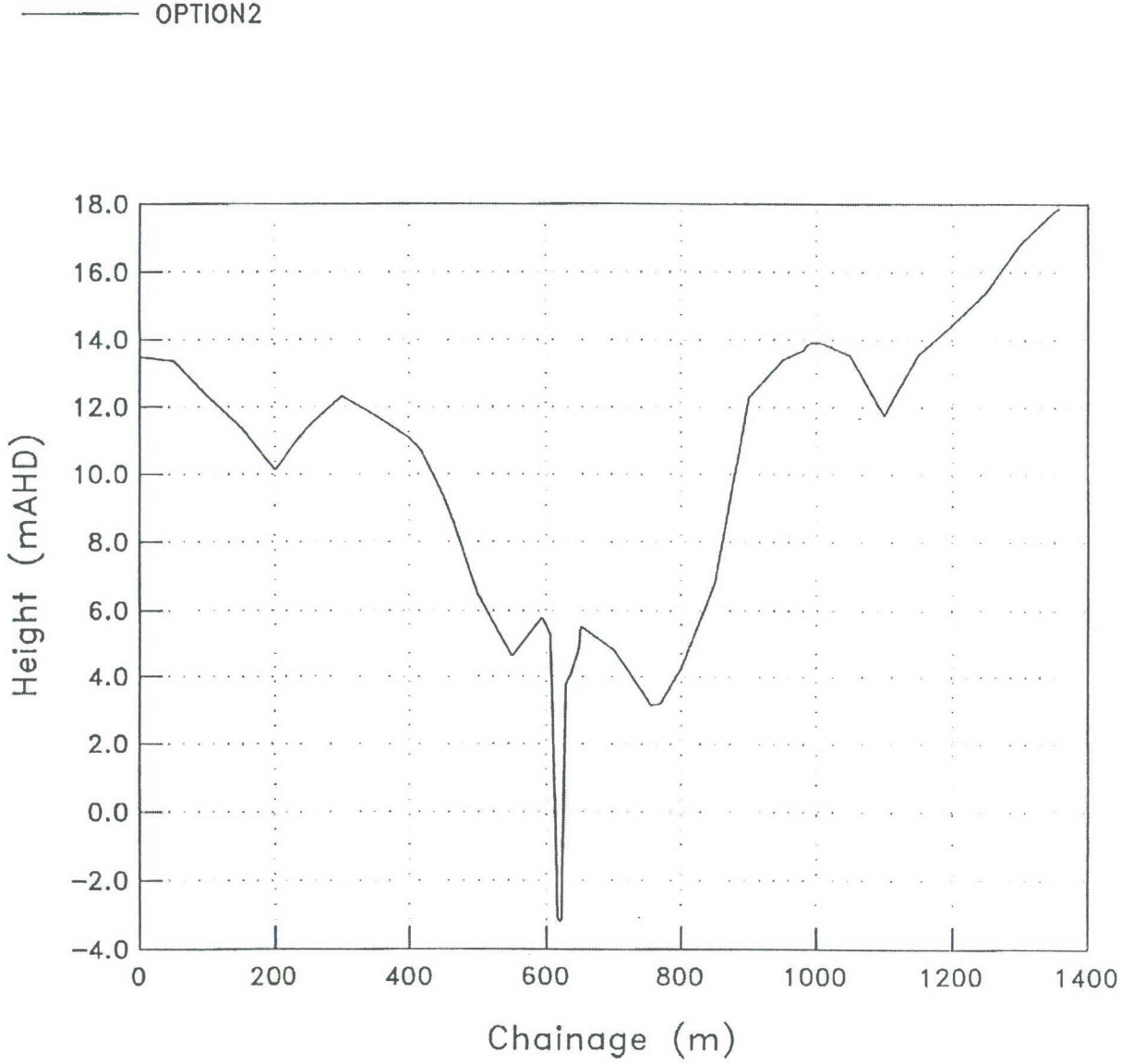

SECTION 4
Height (mAHD)
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
0
200
400
600
800
1000
1200
1400
1600
1800
2000
28/02/01 13:32


RAILWAY BRDGE OVER STH CK
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
Height (mAHD)
0
100
200
300
400
500
600
Chainage (m)
28/02/01 13:32

SECTION 5



28/02/01 13:33


OPTION2
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16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-2.0
-4.0
Height (mAHD)
0
200
400
600
800
1000
1200
1400
Chainage (m)

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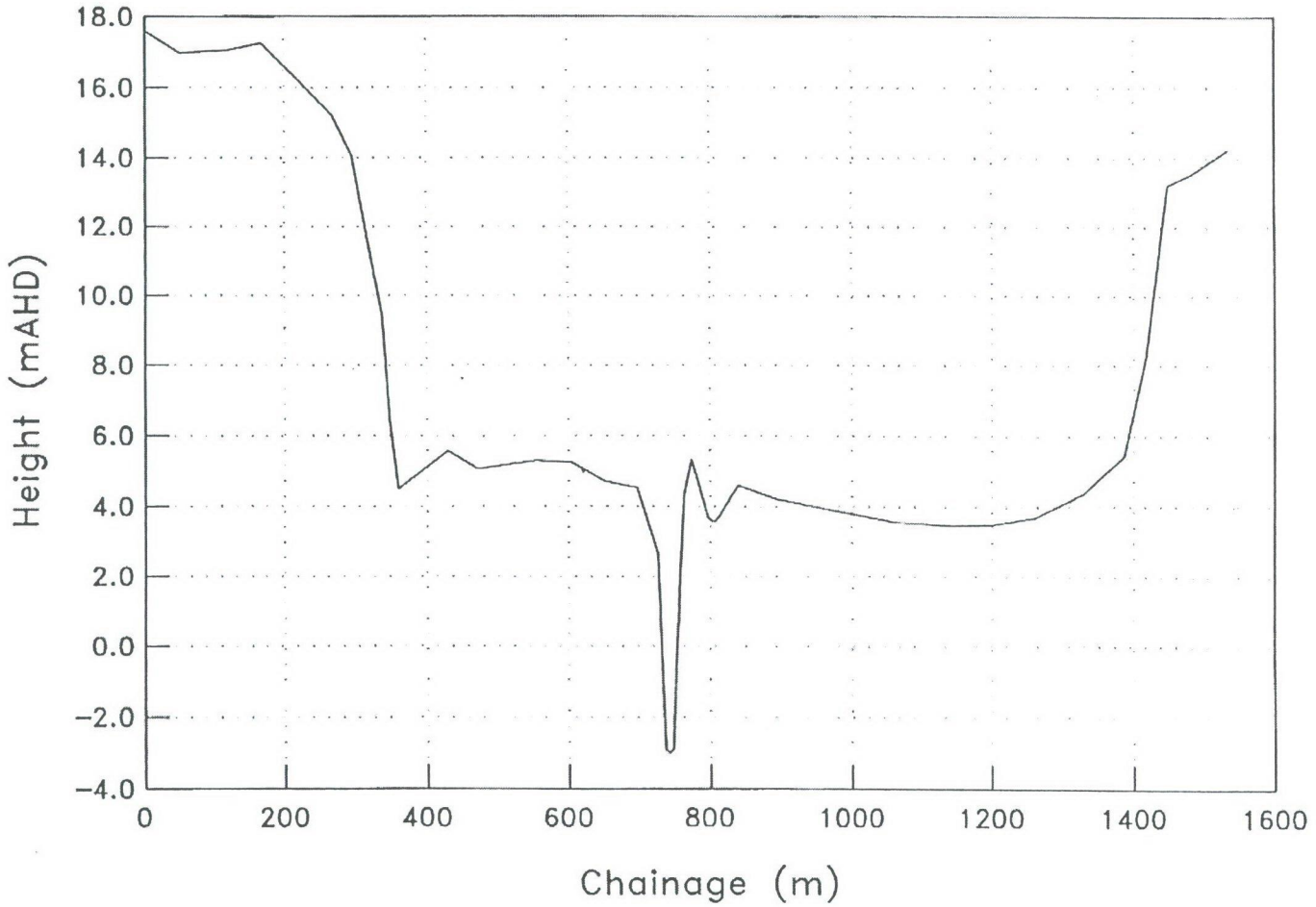
SECTION 6



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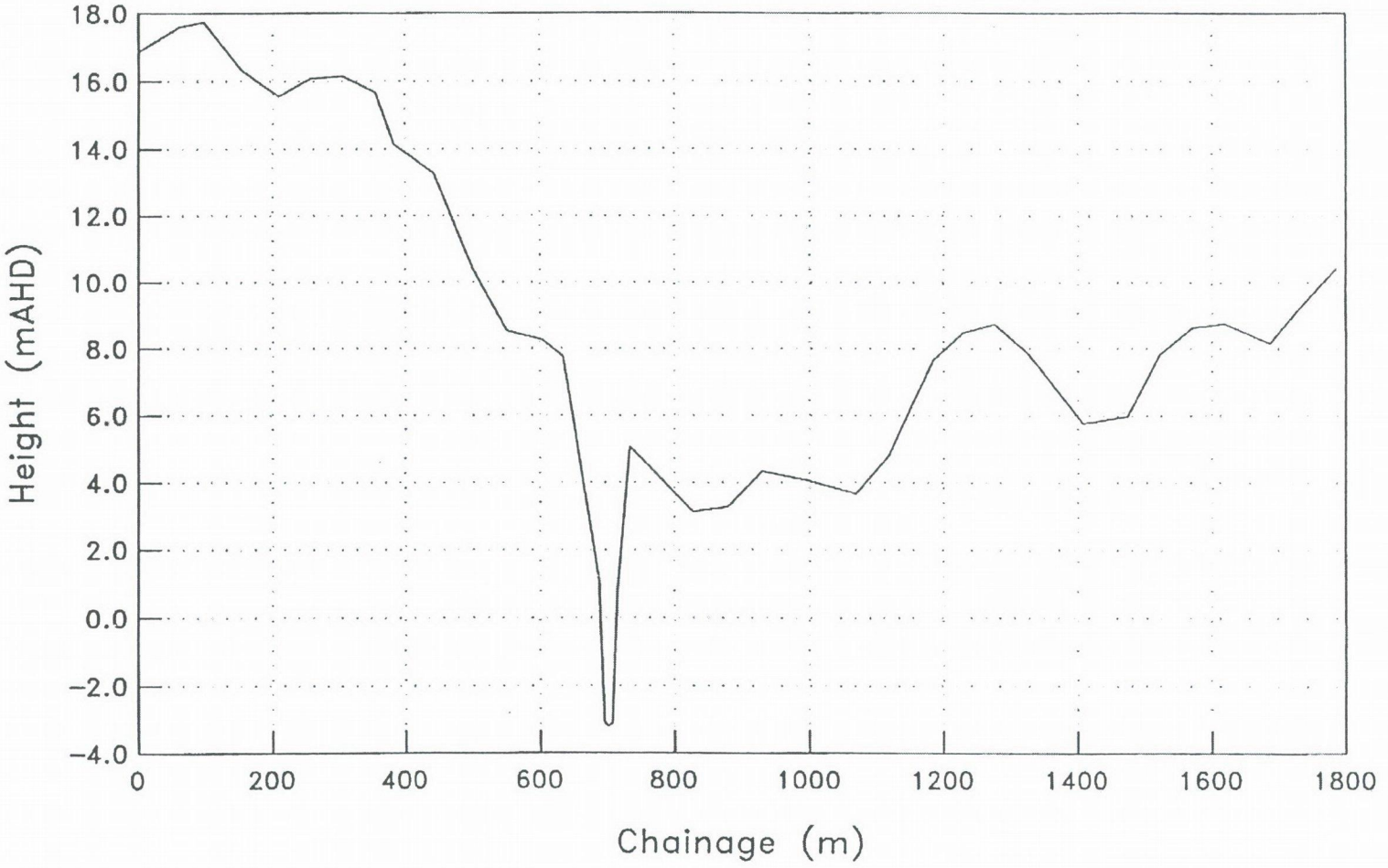

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Working Paper 4 – South Creek Crossing Wave Analysis

SOUTH CREEK CROSSING WAVE ANALYSIS

Report No. MHL1076

NSW Department of Public Works and Services
Manly Hydraulics Laboratory

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Foreword

The study was carried out by the NSW Department of Public Works and Services' Manly Hydraulics Laboratory (MHL) on behalf of the Department of Land and Water Conservation (DLWC). The study was undertaken and report prepared by I. Jayewardene. Drafting was done by Kerrie Harrison and report production by Megan Jensen.

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1. Introduction

1.1 Background

As part of the regional evacuation route upgrade program being carried out under the Nepean Floodplain Management Strategy it is proposed to provide a high level evacuation route over South Creek. Two evacuation routes were selected for further detailed feasibility assessment including investigation of the potential for wave inundation during a flood. These routes are:

- the northern route which is approximately 2.7 km long (Figure 1.1). This option originates north of Mulgrave Station and joins the intersection of Forbes Street and Mileham Street
- the southern route which is approximately 2.2 km long (Figure 1.2). This option originates south of Mulgrave Station and joins Ham Street.

The proposed evacuation route is expected to be a bridge spanning South Creek with the approach road on either side on suitable walls/embankments. The purpose of the route is to provide adequate time for residents of Windsor to evacuate in a severe flood event. It is therefore desirable that the route remains usable for as long as possible during a flood evacuation.

Consideration of design conditions has to date tended to focus on probable flood surface levels associated with a range of flood recurrence intervals. Selection of the optimal level for the proposed route has been based on ground level constraints at both ends of the route. During a large flood much of the area adjacent to the proposed evacuation routes will essentially become a lake and in the event that strong winds should occur at the same time then the influence of surface waves on the evacuation route must be considered. High wind waves may cause additional overtopping of roads and hence due consideration in the design of the evacuation route is required. The objective of this investigation is to estimate the likely wave climate and runup to assist in the design of the evacuation route and bridge. Estimates will be produced for the 15 m, 16 m and 17.3 m (approximately 3%, 2% and 1% AEP) flood levels.

1.2 Study Objectives and Methodology

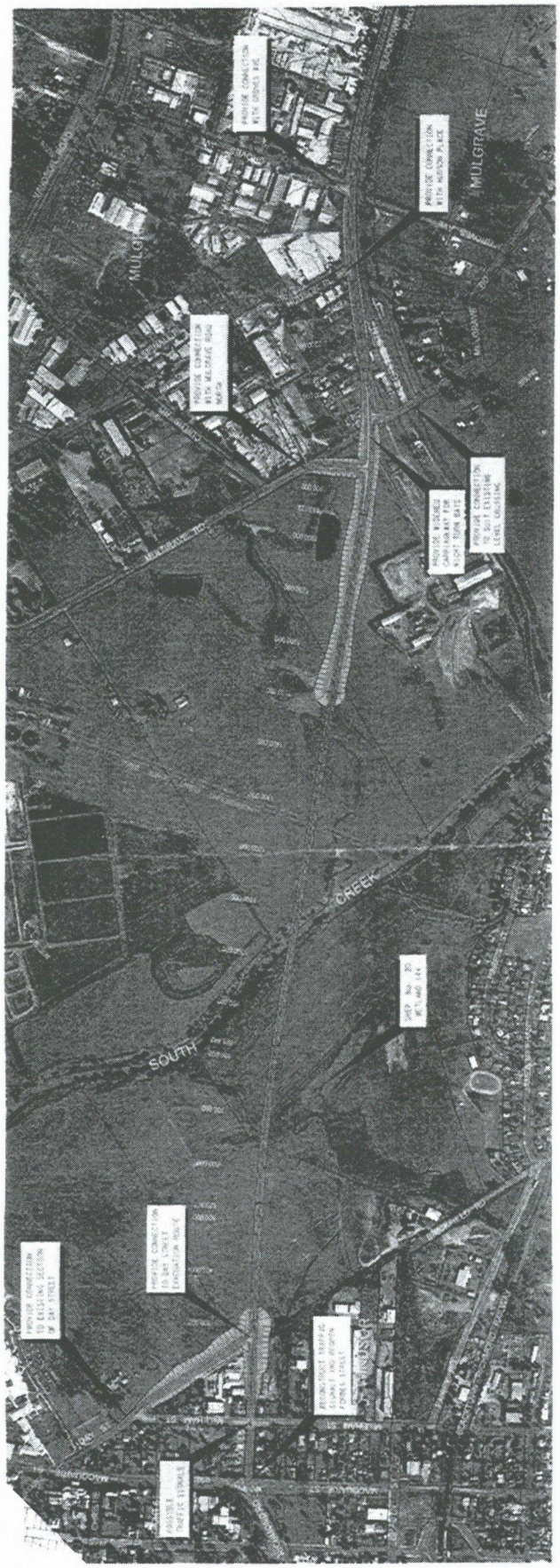
The objectives of the present study were outlined during discussions with David Avery of the Department of Land and Water Conservation. The aim was to provide conceptual design calculations to assist the program managers with an assessment of the likely effect of waves on the evacuation route.

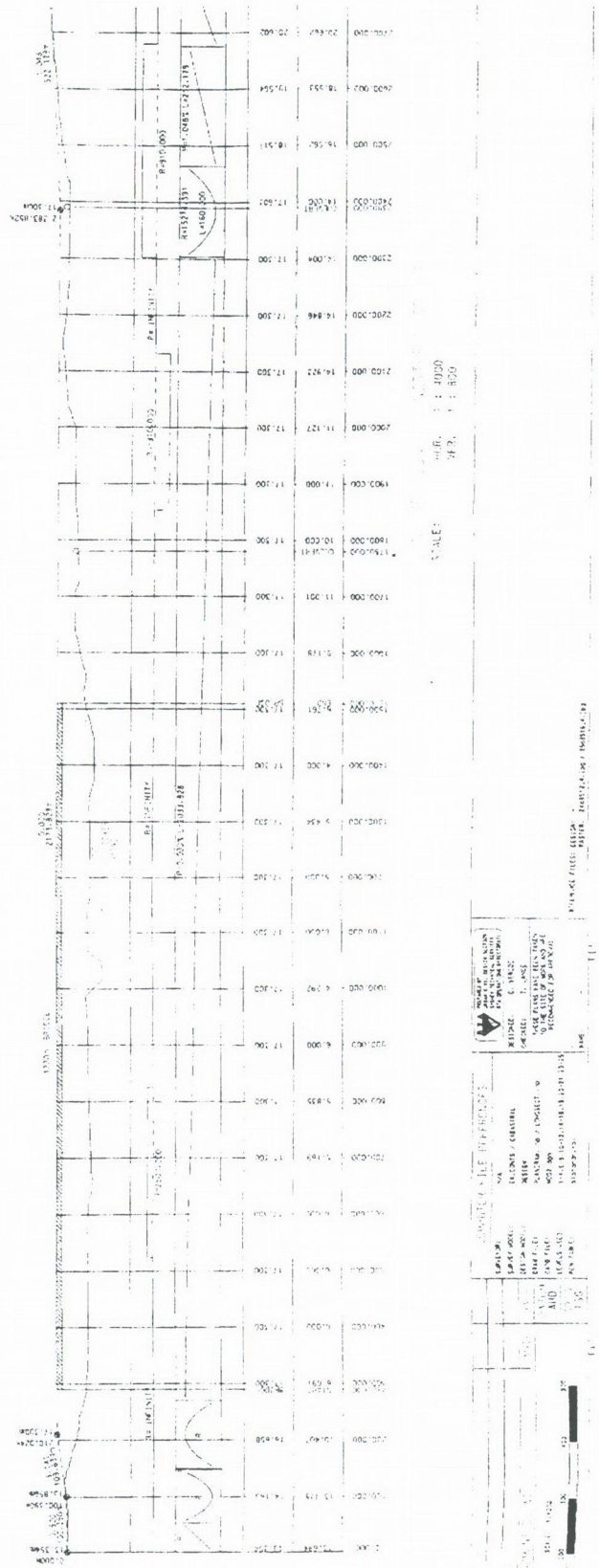
The approach taken for this investigation included the following tasks:

- A site visit to Windsor to inspect locations of interest (i.e. sites where waves may influence the flood evacuation route).
- Collate available wind data for the study area.
- Calculate fetches for the dominant wind directions using the 1-in-100,000 and 1-in-10,000 scale flood maps supplied by the Roads and Traffic Authority (RTA).
- Estimate the wind wave characteristics for a fully developed sea state using the wave hindcast methods presented in the Shore Protection Manual (CERC 1984). The waves were hindcast at designated locations along the flood evacuation route.
- Estimate wave overtopping values for the range of locations along the flood evacuation route.
- Categorise overtopping values onto roadways and breakwater crests using the criteria of Fukuda (1974). The calculated overtopping rates will be classified according to Fukuda's classification to indicate criteria for the safe evacuation of pedestrians and vehicles.
- Briefly discuss potential wave forces on the proposed structures with reference to MHL's experience with the design of wave deflectors to reduce overtopping .

1.3 Available Data

Wind data from the Bureau of Meteorology was available as monthly, 10-minute averaged wind roses and wind frequency tables for the Richmond airport based on six years (1994-2000) of data. This data was unavailable in the return period format. Therefore values of maximum gusts of 10 minutes were obtained for Sydney Harbour. The data assumes a category 2 terrain at 10 m elevation (Figure 1.3) which is an acceptable assumption for the terrain at Windsor. The wind loading code for Australia (AS 1170) defines category 2 terrain as 'open, well scattered obstructions with average obstruction height 1.5 to 10 m (example - open parklands, sparsely built up outskirts of towns)'. This wind data will be used to hindcast waves at Windsor during extreme floods.





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EVACUATION ROUTE - THE NORTHERN ROUTE

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Figure 1.1

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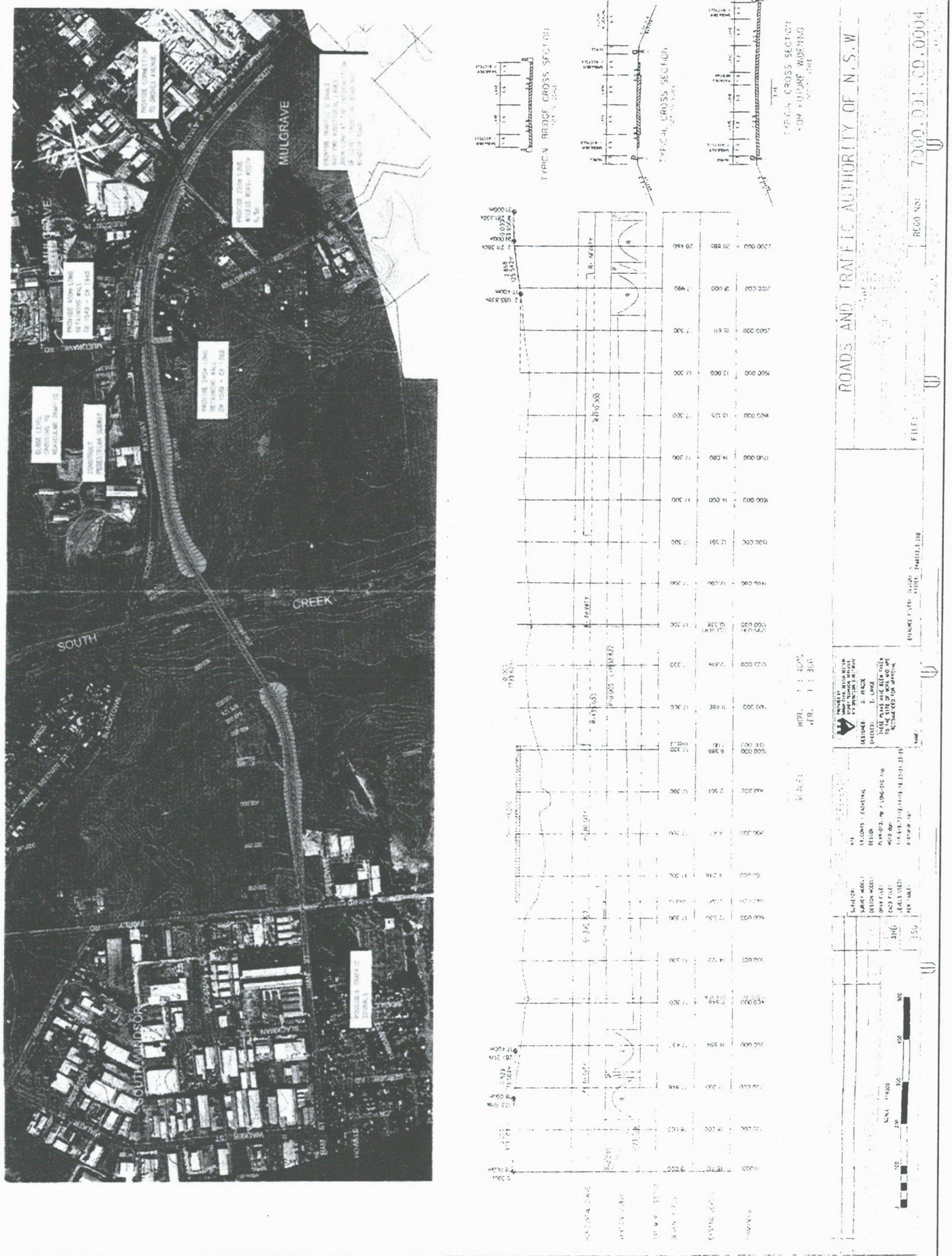

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EVACUATION ROUTE - THE SOUTHERN ROUTE

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Figure 1.2

DRAWING 1076SC12.CDR

(a) Looking from Windsor towards Mulgrave Station

(b) Looking from hospital towards railway crossing over South Creek

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FLOODPLAIN TERRAIN AT WINDSOR

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Figure 1.3

DRAWING 1076SC13.CDR

2. Brief Introduction to Wave Hindcasting

2.1 Wave Hindcasting Formulae

The first comprehensive understanding of the evolution of deepwater ocean waves under a range of wind conditions was provided by the JONSWAP experiment (Hasselmann et al. 1973). The JONSWAP experiment combined results of a number of wave and wind measurements in deep water and derived a generalised equation relating speed to the wave climate.

There is no generalised theoretical development for determining the actual growth of waves generated by winds blowing over relatively shallow and confined water bodies. The shallow water forecasting curves provided in the Shore Protection Manual (1984) represent an interim method for wave forecasting in shallow water.

A computer model employing the method for wave hindcasting presented in the Shore Protection Manual (1984) was developed by Australian Water and Coastal Studies (AWACS 89/23). This model has been applied to the Windsor area to calculate the locally generated wind waves at designated locations on the flood evacuation route.

Yarde et al. (1996) recommends the use of a simplified JONSWAP method for the prediction of wave conditions in confined and shallow water bodies. This simplification has been shown to produce small errors. The method assumes that the highest waves occur along the longest fetch with the significant wave height given by:

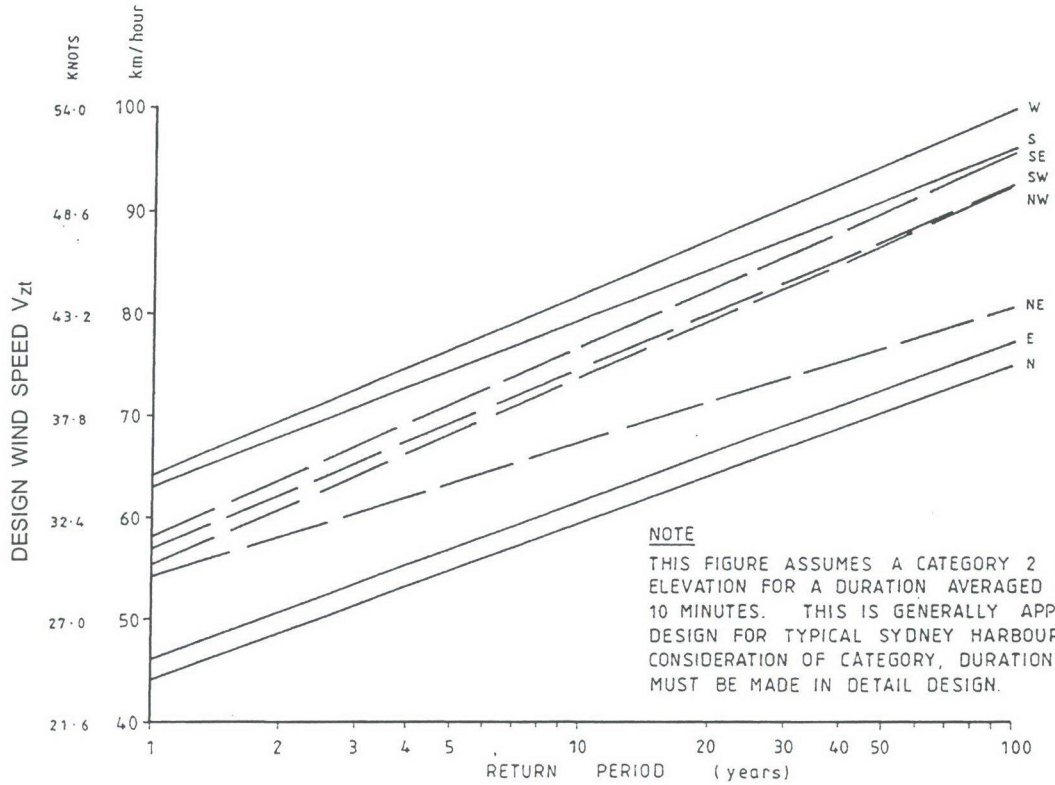
$$H_S = 0.00178\, U_D \sqrt{\frac{F}{g}} \qquad (1)$$

where H_S is the significant wave height (m)
U_D is the design wind speed (m/s)
F is the fetch length (m)
g is acceleration due to gravity (m^2/s)

The peak wave period is given by:

$$T_P = 0.07118\, F^{0.3}\, U_D{}^{0.4} \qquad (2)$$

Young (1997) provided results from a field experiment aimed at investigating the evolution of waves in shallow water. MHL has established a wind and wave database for Lake Victoria for data collected over a period of approximately four months. The value of hindcast waves at Windsor (using the SPM method) and measured waves at Lake Victoria will be compared to the results obtained by using Yarde's method in order to verify the accuracy of the SPM method.



PROBABILITY OF EXCEEDANCE OF DIRECTIONAL DAILY MAXIMUM DESIGN WIND SPEEDS FOR SYDNEY HARBOUR

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DURATION AVERAGED MAXIMUM GUSTS OF 10 MINUTES (DIRECTIONAL) RETURN PERIOD DATA

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Figure 2.1

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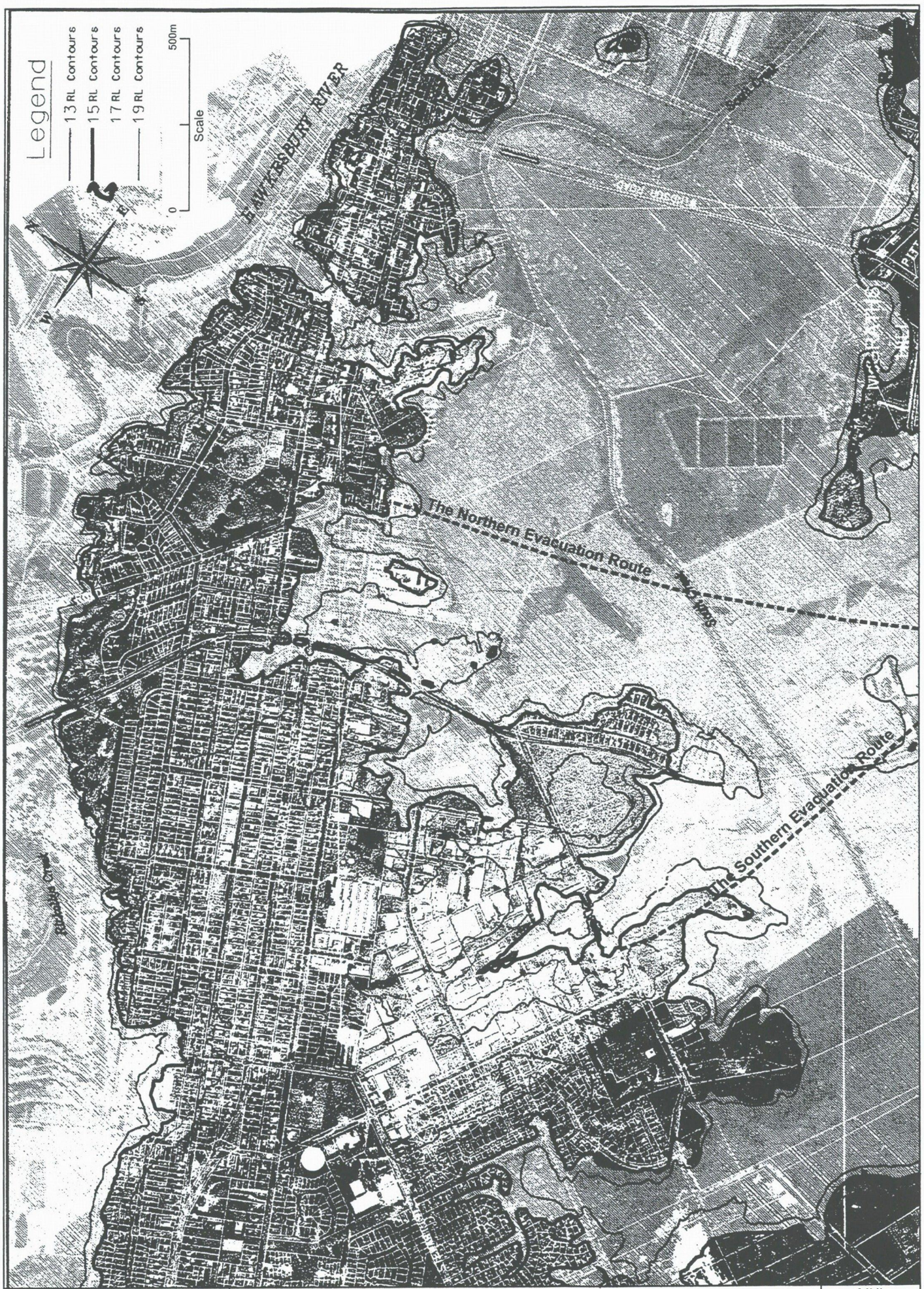

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15m FLOOD CONTOUR FOR WINDSOR

MHL Report 1076

Figure 2.2

DRAWING 1076-02-02.CDR

2.2 Wind Data for Wave Hindcasting

Long-term wind records near the study area are not readily available and information from three sources was utilised to derive best estimates of the wind conditions at the study area:

- BoM data (1994-2000) for Richmond RAAF base
- Sydney Harbour wind loading code (AS 1170)
- Penrith wind data (MHL 1999).

The Bureau of Meteorology supplied monthly, 10-minute averaged wind roses and wind frequency tables for the Richmond airport (1994-2000). This six years of data (Appendix B) was not sufficient to generate return period estimates. The wind speed return periods (Figure 2.1) for Sydney Harbour generated from 58 years (1926-1984) of data were obtained from the Wind Loading Code of Australia (Australian Standard AS 1170). These values appear to be greater than the Bureau of Meteorology data and, hence, wave heights calculated using wind loading code values would be higher than if winds from Richmond were used in the calculation. A summary of wind data collected from the MHL station at Penrith is included in Appendix C. All three data sets indicate the strongest winds are from the westerly direction.

2.3 Fetch Data

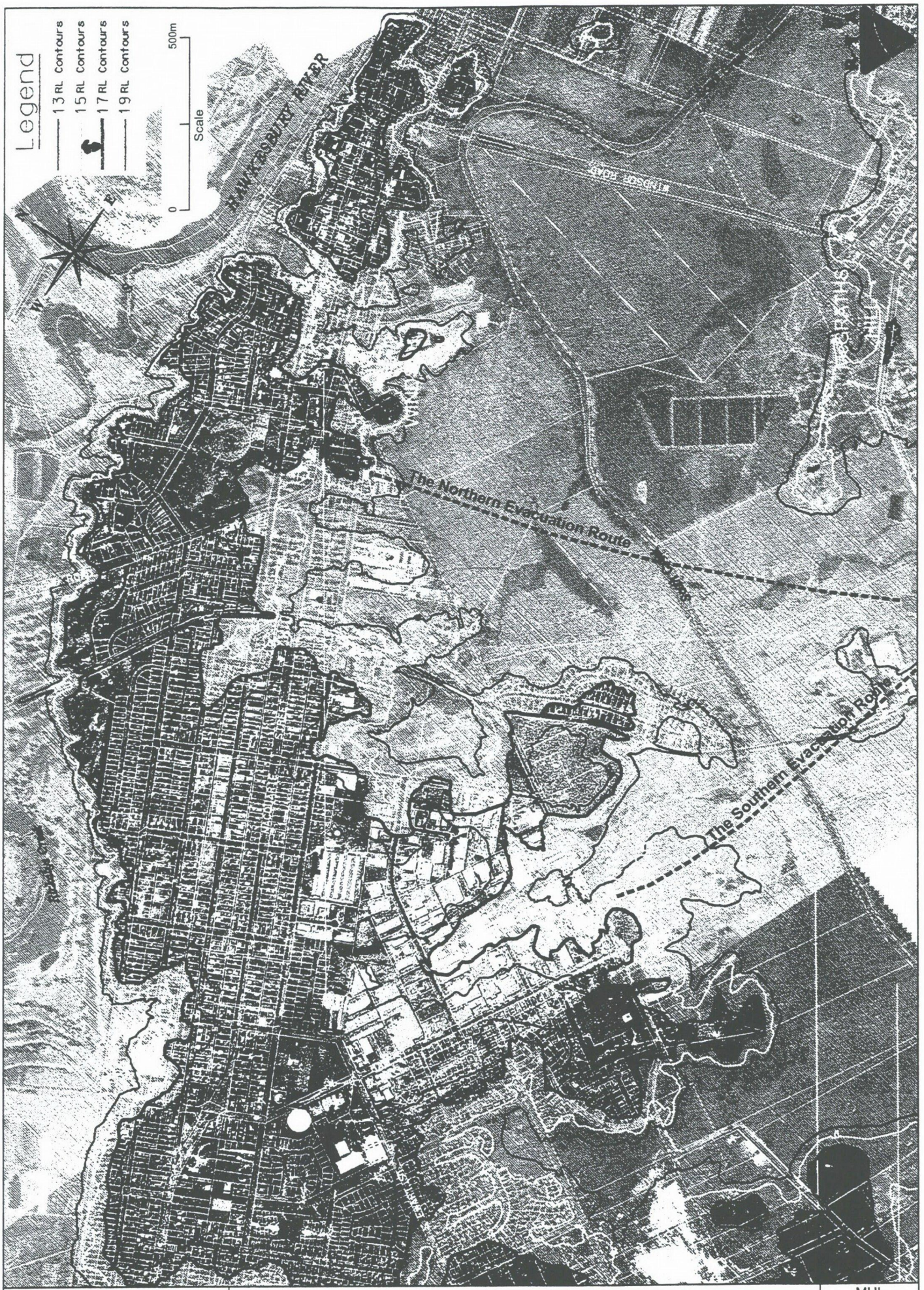
Fetch data was obtained for the length of the route from the 15 m and 17.3 m flood contours marked on the 1:10,000 maps (Figures 2.2 and 2.3) provided by the RTA. Fetch data for the 16 m flood were obtained from the 1:100,00 maps (Figure 2.4) supplied by the RTA. The fetch was estimated for 22.5 degree segments. The railway bridge was assumed not to affect the fetch.

2.4 Wave Inundation Levels

When combining water level and wave conditions the result obtained for wave inundation level is dependent on the combination chosen. A study on the joint occurrence of wind, rainfall and flood levels in Lake Macquarie (MHL 1997 in draft) concluded that it was not representative of conditions to combine a 1% AEP flood condition with the 1-in-100-year wind wave to arrive at a 1% foreshore wave inundation level. The guidelines adopted to arrive at a 1% wave inundation level in that particular study were:

- Guideline 1 - the 1% AEP and 5% AEP flood levels be combined with the 1-in-1-year wind waves to estimate the 1% and 5% foreshore inundation levels
- Guideline 2 - the 100% AEP flood level be combined with a 1-in-100-year and 1-in-20-year wind wave to estimate the 1% and 5% foreshore inundation levels
- Guideline 3 - the 1% AEP and 5% AEP foreshore inundation levels be derived as the maximum of guidelines 1 and 2.

For the Windsor area further investigation would have to be carried out on water level and wave climate combinations to arrive at a representative set of conditions for 1% and 5% foreshore flood levels. This study has combined the results from 15 m, 16 m and 17.3 m floods with 1-in-1-year, 1-in-20-year and 1-in-100-year winds to indicate the range of scenarios in which the flood evacuation route would have to function.



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17.3m FLOOD CONTOUR FOR WINDSOR

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Figure 2.3

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16m FLOOD CONTOUR FOR WINDSOR
MHL Report 1076
Figure 2.4
DRAWING 1076SC04.CDR
WINDSOR
CASTLEREAGH STATE FOREST
0
5km
Scale
NEPEAN–HAWKESBURY VALLEY
EXTENT OF FLOODING FOR EXISTING DAM
16 METRE FLOOD (at Windsor)
1867 FLOOD-ON-RECORD
PROBABLE MAXIMUM FLOOD
(assumed Auxiliary Spillway has been provided)

3. Wave Prediction for a 15 m Flood, 16 m Flood and 17.3 m Flood

3.1 General

The *Hawkesbury Gazette* has published flood levels (over 6.4 m AHD) recorded at Windsor dating back to March 1799. Based on these data and previous flood studies (Webb McKeown 1996), Annual Exceedance Probabilities have been derived and are indicated in Table 3.1. At present the proposed evacuation route is designed to the 1% AEP flood level of 17.3 m AHD.

Table 3.1 Annual Exceedance Probabilities for Flood Levels at Windsor

AEP (%)	RL (m AHD)
20	11.1
10	12.3
5	13.7
2	15.7
1	17.3
0.5	18.7
0.2	20.2
0.1	21.9
PMF	26.4

The guidelines for combining the different probability levels for flood and wave inundation (Section 2.4) were derived for an estuary where the difference between the 1% and 20% flood levels is relatively small compared with the difference of 6.3 m at Windsor. This difference is considerably larger than the wave runup estimates and hence guideline 1 will always exceed guideline 2 (Section 2.4) for the Hawkesbury River and Windsor. The most appropriate combination of wind event and flood level probabilities would need to be derived from a joint probability assessment of winds and floods, but such an analysis is beyond the scope of this study. For this investigation a 1-in-100-year wave event is taken to be the result of a 1-in-100-year wind event as determined from the wind loading code (AS1170) acting over a given floodwater level.

An example of the computation results for the 16 m flood level with the range of fetch lengths and directions is tabulated in Appendix A.

3.2 Sensitivity Tests

Sensitivity tests were carried out to investigate the influence of fetch and depth on the resultant wave heights.

- Variation of wave height at the southern and northern route. The sensitivity tests indicated that only a small variation (<5%) existed for the wave heights at the southern route when calculations were made for a 16 m flood. For this study all wave hindcast calculations were carried out for the northern route. The railway line (13 m AHD) was assumed not to have an impact on the calculations as the water depth in the 16 m level would not limit the short period waves.

- Variation in depth. The wave model used to hindcast waves required that an average depth be assumed over the floodplain. The terrain over which the flooding took place was taken to be at an average level of 6 m AHD. Sensitivity tests were carried out by varying this level by 2 m. These variations had little influence on the calculated value of the hindcast wave.

3.3 Wave Heights for 15 m and 17.3 m AHD Floods

Table 3.2 indicates the results for the variation of the 1-in-100 and 1-in-20-year wave height with distance along the northern route. The largest wave heights occur for southerly winds, which is the direction of the longest fetch.

Table 3.2 Variation of Wave Height with Distance Along the Northern Route for 15 m and 17.3 m Floods

Length Along Route	1-in-20-Year Significant Wave Height and Period Northern Option (Direction)		1-in-100-Year Significant Wave Height and Period Northern Option (Direction)	
	H_s (m)	T_p (s)	H_s (m)	T_p (s)
15 m Flood				
0 m (Windsor)	1.03 (ESE)	3.1	1.10 (ESE)	3.2
500 m	1.36 (SSE)	3.4	1.56 (SSE)	3.6
1,000 m	1.44 (S)	3.7	1.51 (S)	3.8
1,500 m	1.27 (S)	3.3	1.33 (S)	3.4
2,000 m	1.20 (S)	3.3	1.36 (S)	3.5
2,500 m (Mulgrave)	-	-	-	-
17.3 m Flood				
0 m (Windsor)	0.98 (ESE)	3.1	1.16 (ESE)	3.3
500 m	1.47 (S)	3.8	1.67 (S)	4.0
1000 m	1.44 (S)	3.8	1.64 (S)	4.0
1500 m	1.41 (S)	3.7	1.60 (S)	3.9
2000 m	1.37 (S)	3.7	1.56 (S)	3.8
2500 m (Mulgrave)	-	-	-	-

3.4 Wave Heights for a 16 m AHD Flood

Wave heights were computed for the total length of the northern route for the 15 m and 17.3 m flood levels. For the 16 m flood wave heights were calculated only at the centre of the crossing for both the northern and southern routes. The results indicate that only a small (<5%) variation exists between the wave climates experienced at the two locations. These wave heights are indicated in Table 3.3. The variation is due to the shorter fetch of the northern route as compared to the southern route.

Table 3.3 Variation of Wave Height with Return Period for a 16 m flood

Return Period	Significant Wave Height and Periods at Centre of Northern Option (Direction)		Significant Wave Height(m) and Period at Centre of Southern Option (Direction)	
	H_s (m)	T_p (s)	H_s (m)	T_p (s)
1-in-1 year event	1.0 (south)	3.3	0.96 (south)	3.2
1-in-20-year event	1.38 (south)	3.7	1.34 (south)	3.6
1-in-50-year event	1.5 (south)	3.8	1.46 (south)	3.7
1-in-100-year event	1.57 (south)	3.9	1.53 (south)	3.8

3.5 Influence of Flood Currents and Refraction on Wave Heights

3.5.1 Effect of Flood Current on Wave Height

Estimates of the velocities for both upstream and downstream of South Creek railway bridge for the 100-year event (17.3 m flood level) were provided by DLWC. Estimates of velocities for the rising limbs of the hydrograph were provided (Table 3.4).

Table 3.4 Flow Velocities Near the South Creek Railway Bridge for 100-year Event
(Source: DLWC)

Variation from Peak Level (Water Level)	Upstream 100 m	At Railway Bridge	Downstream 100 m
1 m before tidal peak (16.3 m AHD)	-0.10	-0.14	-0.10
0.5 m before tidal peak (16.8 m AHD)	-0.06	-0.10	-0.06
At tidal peak (17.3 m AHD)	0.01	0.01	0.01
0.5 m after tidal peak (16.8 m AHD)	0.08	0.14	0.08
1 m after tidal peak (16.3 mAHD)	0.11	0.19	0.11

Previous studies indicated a 30% increase in wave height due to an opposing current of around 2 m/s (Hales & Herbich 1973). The relatively small magnitude of the currents simulated at the Windsor bridge indicate that these currents will not cause an appreciable increase in the wave heights.

3.5.2 Effect of Wave Refraction on Wave Height

An increase in wave height also occurs when refraction over an irregular bed causes wave rays to converge. A wave refraction study on the waves during a flood at Windsor has not been undertaken. For the limited fetch lengths it is suggested that wave focusing would be insignificant, and this has not been included in the assessment.

3.6 Comparison of Wave Hindcast Values Using the AWACS and Yarde's Methods

In order to have confidence in the AWACS method developed using the curves given in the Shore Protection Manual (1984) a comparison of the wave hindcast was made with the simplified JONSWAP method recommended by Yarde (1996). Table 3.5 indicates a comparison of the values obtained for the 1-in-1-year waves generated on a 17.3 m flood.

Table 3.5 Comparison of Wave Height Calculations for 17.3 m Flood and 1-in-1-Year Wave for the Northern Route

Length Along Route (m) Estimated Level of Maximum Wave m (AHD)	Significant Wave Height and Period (AWACS method)		Significant Wave Height and Period (Yarde's method)		Ratio Yarde's/ AWACS/	
	H_s (m)	T_p (s)	H_s (m)	T_p (s)	H_s (m)	T_p (s)
0 m (Windsor)	0.58	2.6	0.57	2.65	0.98	1.02
500 m	1.05	2.8	0.90	3.3	0.86	1.18
1,000 m	1.03	3.2	0.87	3.3	0.84	1.06
1,500 m	1.01	3.3	0.85	3.2	0.84	0.97
2,000 m	0.98	3.0	0.83	3.2	0.85	1.06

The comparison indicates that the hindcast wave heights calculated by using the AWACS model are up to 10%-15% higher.

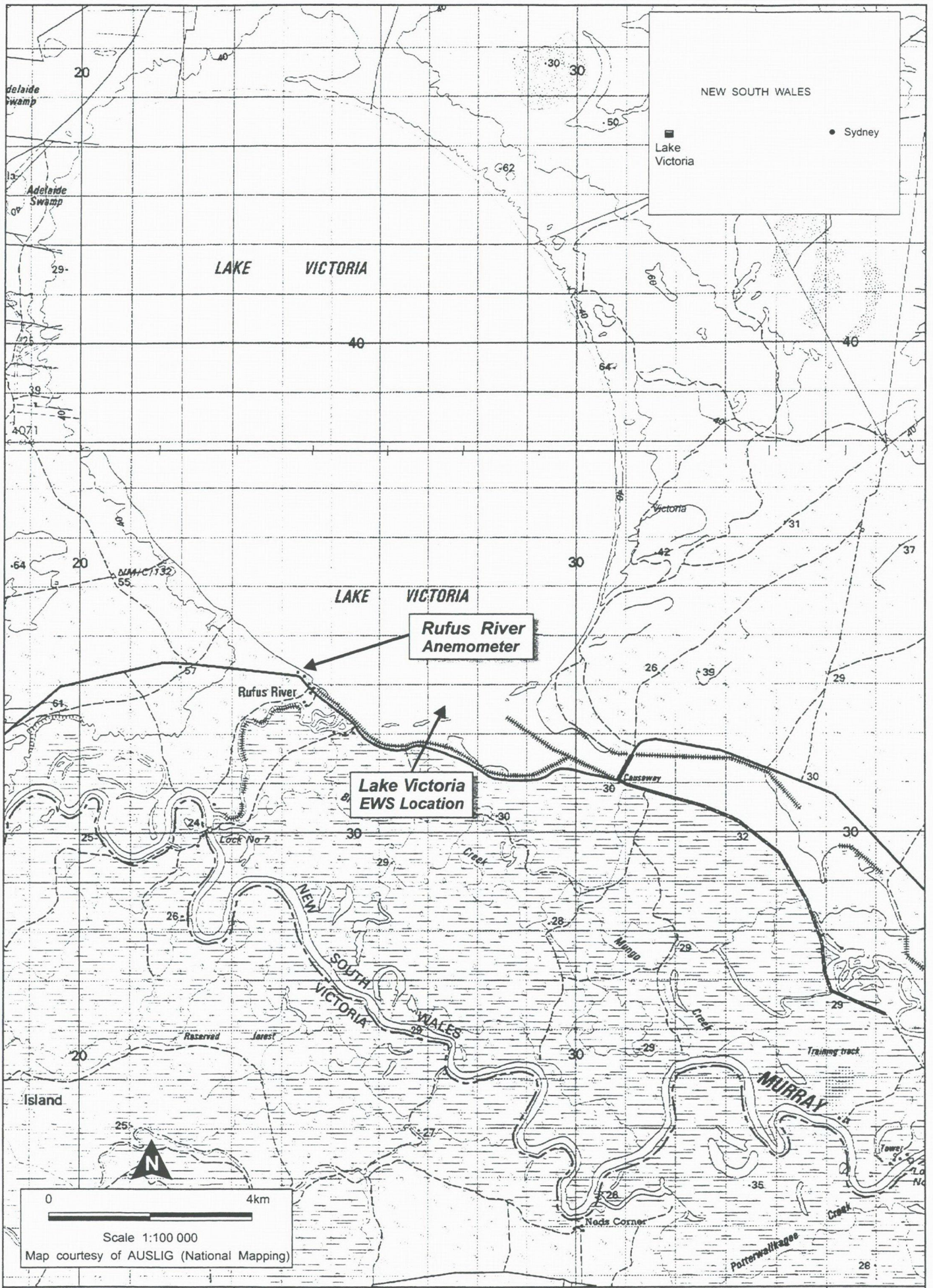
3.7 Comparison of Wave Height Hindcast Values Using Yarde's Method with Measured Values at Lake Victoria

Yarde's method was applied to waves measured at Lake Victoria. Simultaneous measurements of winds and waves were recorded at the lake (Figure 3.1) over a period of four months. Three days on which the highest waves were generated were chosen and the results are presented in Table 3.6. The AWACS model could not be used for this site as this model requires wind data in a return period format.

Table 3.6 Comparison of Measured and Hindcast Wave Heights at Lake Victoria

Date	Measured Wave Height (m)	Hindcast Wave Height (m) (Yarde's method)
	H_s (m)	H_s (m)
27/11/98	0.40	0.45
02/12/98	0.40	0.45
02/02/99	0.25	0.23

In this case Yarde's method generally tends to overestimate the wave height by about 10%. These comparisons suggest the AWACS model estimates are likely to be slightly larger than may be observed.



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SIMULTANEOUS WIND AND WAVE MEASUREMENT AT LAKE VICTORIA (NSW)

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Figure 3.1

DRAWING 1076SC31.CDR

4. Runup and Overtopping at Approaches to the Evacuation Crossing

4.1 Runup Calculations

When wind waves reach the shoreline they expend their remaining energy as wave runup. The level of wave runup is affected by the angle of wave attack, the foreshore profile, and type of foreshore structures. Wave runup is a key factor affecting the window of time available for continued use of the evacuation route at South Creek. Runup estimates were calculated for waves approaching normal to an embankment (maximum runup) as well as waves approaching at an angle. Runup and splash on vertical structures such as bridges are considered in Chapter 5. The following equation is used to predict runup on grassy dykes, smooth slopes and rubble mound breakwaters (Van der Meer 1994). These equations do not apply to sections with vertical barriers.

$$Ru_{2\%}/H_s = 1.5\,\gamma_f\,\gamma_s\,\gamma_\beta\,\xi \quad \text{(with a maximum of } 3.0\,\gamma_f\,\gamma_s\,\gamma_\beta) \qquad (3)$$

where γ_f = Roughness factor
γ_s = Shallow water factor
γ_β = Oblique wave attack factor
ξ = Surf similarity parameter = $\tan\alpha/(H_s/L_o)^{1/2}$
α = Slope
L_o = Deep water wave length
$R_{u2\%}$ = 2% runup level
H_s = Design significant wave height (average of the highest 1/3 of the waves)

The three gamma (γ) factors vary for a range of conditions. The factor for roughness (γ_f) for two layered rock and grass (3 cm) is estimated to be in the range 0.50-0.55 and 0.9-1.0, respectively. The influence of shallow water effectively reduces wave runup and varies in the range 0.88-1.0. The factor for oblique wave attack (γ_β) is given as 0.64, 0.86 and 1.0 for the angles 60°, 30° and 0°. The design requires that the soffit or undercarriage of the evacuation route be constructed at a level that reduces the impact from the highest wind-generated waves.

The estimated level of the highest wave for a return period was 1.87 x 0.75x H_S, where 1.87 H_S is the maximum wave height in a Raleigh distributed time series with a significant wave height H_S and 75% of the wave is taken to be above mean water level.

4.2 Runup During a 15 m and a 16 m Flood

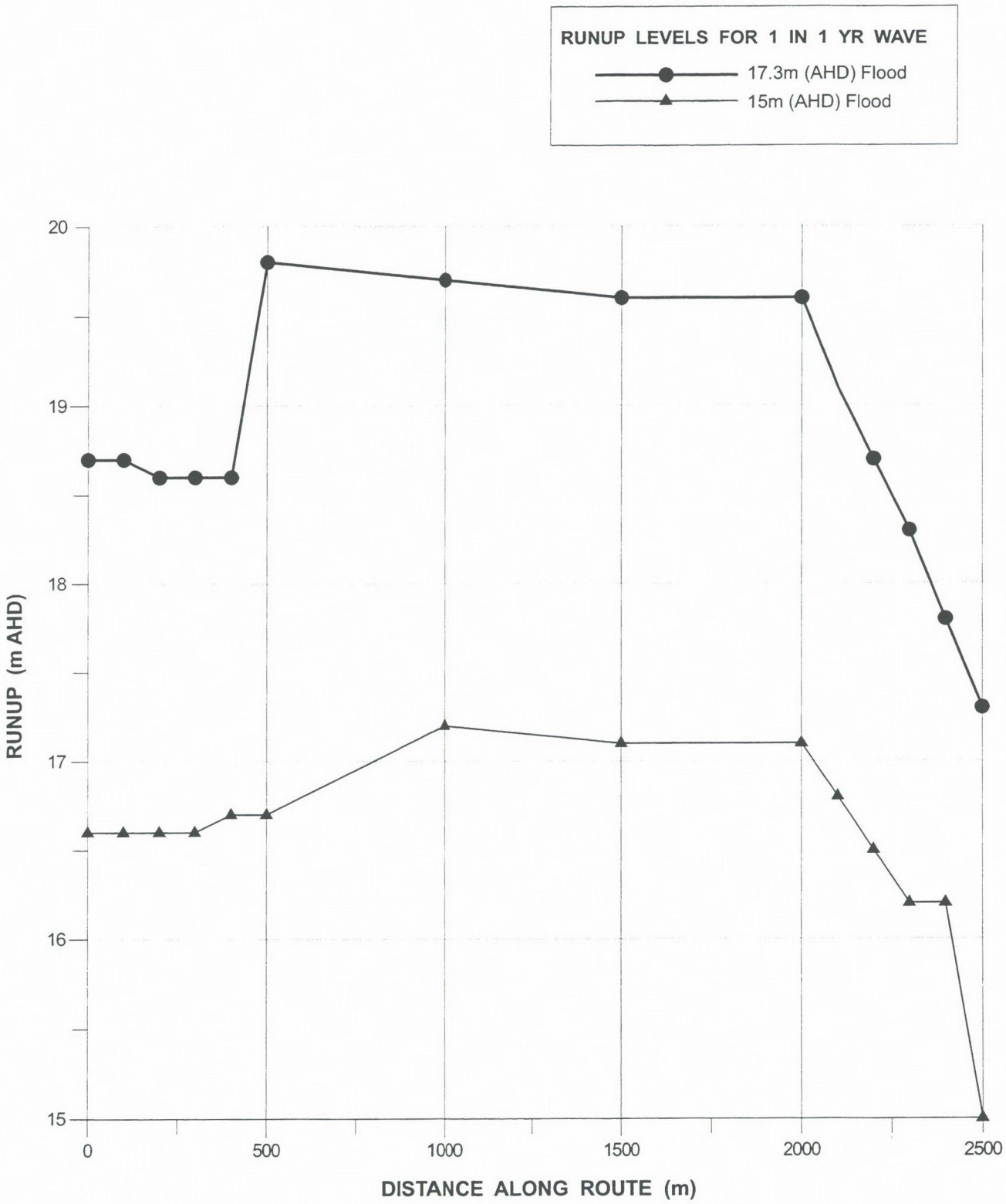
Wave runup was estimated for a 15 m flood along the total length of the evacuation route at 500 m intervals. The preliminary runup calculations for the embankment design cross-section are tabulated in Appendix D. These estimates are not suitable for vertical walls and bridges and are considered in Chapter 5 of this report. Table D1 indicates that when a 1-in-1-year wave climate jointly occurs with a 15 m flood the runup levels are estimated to reach 17.2 m. This level is below the proposed road level (17.3 m AHD) and hence may not cause disruption to the flood evacuation process.

Runup was estimated for the 16 m flood at the centre of the evacuation route. The runup levels reach 18.2 m when a 1-in-1-year wave climate jointly occurs with a 16 m flood.

Runup was estimated on a 1:2.5 slope as proposed in the evacuation route design for the 15 m and 16 m floods. Estimates were made for the 1-in-20-year wave and the 1-in-100-year wave. The runup estimates are listed in Appendix D, Tables D1, D2 and D3. The runup estimates for the centre of the route for a 16 m flood are indicated in Table D4.

4.3 Runup During a 17.3 m Flood

The maximum runup due to a 17.3 m flood and a 1-in-1-year wave was estimated to be 19.8 m. This is much higher than the proposed level of the route and would result in extensive overtopping. The estimates of 1-year, 20-year and 100-year wave runup on a 17.3 m flood are listed in Appendix D, Tables D5, D6 and D7 respectively. The runup levels for the 15 m and 17.3 m flood for a 1-in–1-year wave are shown in Figure 4.1. The figure indicates that the two ends of the route are relatively protected from wave action in comparison with the centre of the route. This occurs because the ends of the route are protected by relatively higher ground and the fetch length is longer in the middle of the route.



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RUNUP LEVELS DUE TO 1 IN 1 YEAR WAVES ON A 15m AND 17.3m FLOOD ALONG EVACUATION ROUTE

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Figure 4.1

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5. Overtopping Rates, Wave Deflectors and Wave Forces on a Vertical Structure

5.1 Overtopping Calculations

5.1.1 A Brief Introduction

The height of a flood protection structure has to be optimised so as to permit overtopping levels that are acceptable. Prototype measurements of wave overtopping are scarce. One of the most extensive sets of data is available from Fukuda (1974) who presents results of overtopping measurements carried out in Japan.

Wave overtopping is generally measured as a discharge per unit metre of structure ($m^3/m/s$ or l/m/s). In this study the overtopping estimates used the wave height estimates for the 16 m flood to indicate the severity of overtopping that may be experienced.

Wave overtopping can be expressed in two formulae (Van der Meer 1994): one for breaking waves ($\xi < 2$) and another for non-breaking waves ($\xi > 2$) where ξ is the surf similarity parameter (Section 4.1). Only breaking waves of low surf similarity ($\xi < 2$) are generated in this study. Therefore only the formula for breaking waves was utilised. The dimensionless crest height R_b (application area $0.3 < R_b < 2$) and the overtopping quantity q are given by:

$$R_b = R_c \sqrt{S_{op}} / H_s \tan\alpha \, \gamma_f \gamma_s \gamma_\beta \quad (4)$$

$$q = Q_b \sqrt{g} H_s^3 \tan\alpha / \sqrt{S_{op}} \quad (5)$$

where:

$$Q_b = 0.06 \exp(-5.2 R_b) \quad (6)$$

and

R_b = dimensionless crest height above still water level
R_c = free crest height above still water line
q = average overtopping discharge
Q_b = dimensionless discharge for breaking waves
S_{op} = wave steepness ($H_s/1.56T_p^2$)
α = structure slope

Table 5.1 indicates the average overtopping quantities calculated for a 16 m flood at the vertical structure of the proposed bridge in the centre of the northern route.

Table 5.1 Estimates of Overtopping Height and Discharge for a 16 m Flood Level with Different Return Period Wind Waves Generated from the South

Return Period	Significant Wave Height and Period at Centre of Northern Option (Direction)		Average Overtopping Rates for a Grass Slope at 30°	
	H_s (m)	T_p (s)	R_b	q (L/m/s)
1-in-1 year event	1.0 (south)	3.3	0.84	1.1
1-in-20-year event	1.38 (south)	3.7	0.64	6.7
1-in-50-year event	1.5 (south)	3.8	0.59	9.8
1-in-100-year event	1.57 (south)	3.9	0.56	12.6

5.2 Acceptable Overtopping Rates for Pedestrians and Vehicles

Criteria for acceptable overtopping rates and spray intensities have been developed by many institutions (Jensen 1987). The Dutch guidelines on river dykes (Van der Meer 1994) indicate that for relatively heavy seas and with wave heights of up to a few metres the 2% runup criterion yields an overtopping discharge of the order of 1 L/s/m. The same guideline further quotes 'In certain cases such as covered crests and slopes, sometimes 10 L/s/m can be tolerated'.

In the Dutch guidelines it is assumed that the following overtopping rates are allowable:

- 0.1 L/s/m for sandy soil with a poor turf
- 1.0 L/s/m for clayey soil with relatively good grass
- 10 L/s/m with a clay protective layer and grass according to the standards for an outer slope or with a revetment construction.

Table 5.1 suggests that overtopping due to a 1-in-50-year wind wave on a 16 m flood is allowable with proper slope construction. The hindcast wave data indicates that a 17.3 m flood would result in excessive overtopping of the 17.3 m bridge level while for the 15 m flood overtopping would not interfere with the evacuation process.

The Japanese criteria (1974) are more stringent with a value of 0.03 L/s/m being classified as 'dangerous' for a pedestrian 3 m behind the crest of a breakwater. For a car 3 m behind the breakwater the value of 0.02 L/s/m is classified as 'impassable in normal order'.

5.3 The Use of Wave Deflectors to Reduce Wave Overtopping

Over the last 15 years a considerable number of tests have been carried out to investigate irregular wave overtopping of vertical seawalls with and without deflectors. Although relationships have been established by Van der Meer et al. (1994) between a dimensionless discharge and a dimensionless freeboard (equations 4, 5 and 6), the overtopping discharge is dependent on site specific criteria such as the bathymetry, wind, wave grouping and deflector geometry. These concerns are briefly discussed in conjunction with the influence of wind and grouping on the use of deflectors in Sections 5.4 and 5.5. A typical deflector is shown in Figure 5.1.

5.4 Influence of Wind on Overtopping Rates

Water overtopping at a seawall or the spray ejected into the air at a seawall face is picked up by wind and carried further inland. The rates of overtopping depend on wind speed. Although many physical model studies of wave overtopping have been published, field data is scanty. Fukuda et al (1974) observed overtopping and horizontal distribution of spray for various conditions of wind and waves landward of an overtopping breakwater.

Jensen and Juhl (1987) made field observations of overtopping on one breakwater and model tested the same breakwater and several others. Although the model tests were carried out with wind the scale ratio is not stated. Their findings are published as equations that relate overtopping rates to distance behind the crest.

Since the trajectory of the spray is determined by both viscosity and gravity physical model scaling is expected to follow a combination of Froude (no air resistance) and Reynolds laws (no gravity) and is not clearly established in the literature. However, previous model testing was carried out at MHL (1992) at wind speeds which translated to approximately a 40 knot and a 60 knot wind. The results have to be treated with some degree of caution as the scaling laws are not reported in the literature. The results indicated that for somewhat similar wave and water level conditions as are being investigated at the South Creek crossing, spray intensity doubled at a distance of approximately 15 metres from a vertical wall and was higher than the largest reported intensity for a tropical rainfall. The largest reported intensity for a tropical rainfall in one minute is 0.5 L/s/m^2 (Jensen 1987).

5.5 Influence of Wave Grouping on Overtopping Rates

A wave train arriving at a structure will often be constituted by a number of high waves arriving consecutively after each other. These 'groups' or 'sets' have been found to have greater adverse influence on stability and overtopping than the identical individual waves. When testing a structure in a physical model a wave train having a particular wave energy spectrum should be utilised. The total energy in the spectrum generally characterises the wave climate that has been utilised for the tests. Testing of structures with spectrally identical time series having different wave grouping characteristics has been found to have considerable influence on the overtopping results. A previous study has shown that grouping effects could cause an increase of overtopping volume by two spectrally identical time series by a factor of 100 (Jayewardene 1995). With a single large swell wave overtopping rates could be several hundred times the average rate (Van der Meer 1994). Physical modelling results from tests carried out on the third runway at Sydney Airport (AWACS 1991) suggest that wave deflecting structures proved useful in reducing overtopping by a factor of 2 and permitted a water level increase of 0.5 m. Hence the use of wave deflectors along the evacuation route would likely increase the time of usage of the flood evacuation route.

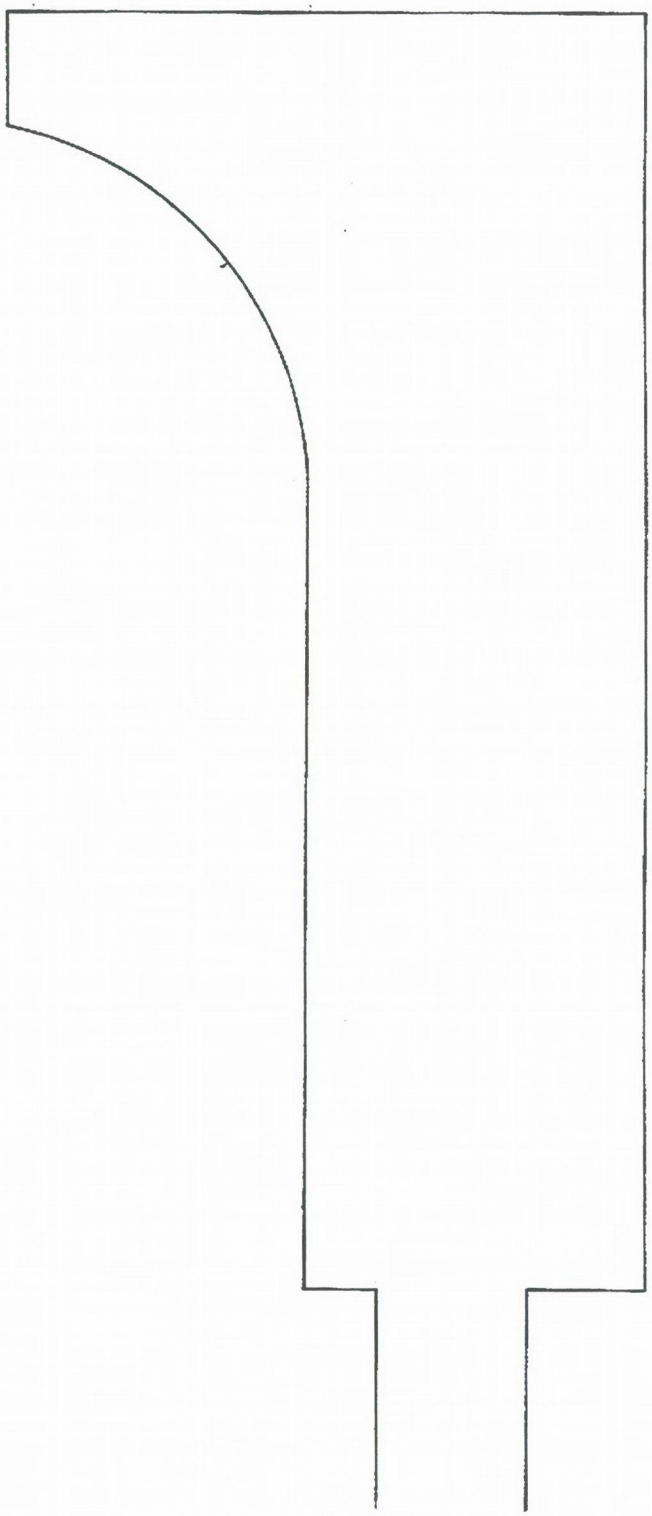
5.6 Wave Forces on a Vertical Structure

Waves breaking directly against vertical face structures exert high, short duration, dynamic pressures that act near the region where the crests hit the structure. Minikin (SPM 1984) developed a design procedure based on observations of full scale breakwaters and the results of other experimenters' studies. Minikin's method can result in forces that are extremely high, as much as 15 to 18 times those calculated for non-breaking waves. This procedure is therefore used with caution (SPM 1984). Table 5.2 presents estimates of the force resulting from waves during the 16 m flood advancing along a 1:2.5 slope and impinging on a vertical structure on the evacuation route (SPM 1984). The location of applicability is at the centre of the evacuation route and would diminish around this point.

Table 5.2 Maximum Horizontal Wave Force Acting on a Vertical Structure at 17.3 m During a 16 m Flood

Return Period	Significant Wave Height and Period at Centre of Northern Option (Direction) for a 16 m Flood		Maximum Wave Force
	H_s (m)	T_p (s)	Kn/m
1-in-1-year event	1.0 (south)	3.3	3.9
1-in-20-year event	1.38 (south)	3.7	7.4
1-in-50-year event	1.5 (south)	3.8	8.8
1-in-100-year event	1.57 (south)	3.9	9.6

The magnitude of these forces compares well with the forces obtained from physical modelling of breaking wave forces on breakwater slats of a yachting harbour for a similar wave climate (AWACS 1995).



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CONCEPTUAL DESIGN OF WAVE DEFLECTOR

MHL Report 1076

Figure 5.1

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6. Conclusions and Recommendations

Design guidelines for levels of the evacuation route over South Creek at Windsor have been derived from estimates of wave inundation levels during major flooding. Estimates were calculated for the 15 m, 16 m and 17.3 m (1% AEP flood) flood levels and a range of wind wave recurrence periods.

Calculating the probability that strong winds occur at the same time as a major flood was beyond the scope of this investigation. Therefore, a number of scenarios of major floods occurring with high winds were investigated. The values for high winds were obtained from the wind loading code for Sydney Harbour. These winds are higher than the winds measured at the Richmond airport and at Penrith and hence, the estimates of the hindcast waves are considered to be conservative.

The values for wave heights obtained by using the AWACS hindcasting model were compared with the values obtained using Yarde's method which was a simplified version of the well accepted JONSWAP method. The comparison was made for 1-in-1-year waves generated on a 16 m flood. The waves calculated by the AWACS method were 10%-15% higher. When Yarde's method was applied to short-term (four months) wind and wave data measured at Lake Victoria in inland NSW the values compared well. The conclusions may be summarised as follows:

- Guidelines for estimating the return periods for events where major flood and high wind waves occur simultaneously have been developed. These guidelines tend, however, to be site specific and should be applied only after an analysis of historical data relevant to the site.

- The return periods calculated for the waves were derived from the return periods for directional wind and hence the results are conservative. Guideline 2 (Section 2.4) has not been applied as the 100% AEP flood is below 11 m and a joint occurrence with 1% winds will not have a major infuence on runup levels.

- The 1-in-1-year wind wave event occurring from a southerly direction results in a maximum runup level estimate of 17.2 m AHD for a 15 m AHD flood.

- The 1-in-1-year wind wave event occurring from a southerly direction results in runup levels higher than the level (17.3 m AHD) of the northern crossing for a 16 m AHD flood.

- The 1-in-1-year wind wave event occurring from a southerly direction results in runup levels higher than the level (17.3 m AHD) of the northern crossing for a 17.3 m AHD flood.

- The Dutch dyke design (Van der Meer 1994) guidelines indicate that the average overtopping estimates due to 1-in-50-year wave on a 16 m flood are acceptable in the event the slopes are constructed suitably. Maximum overtopping rates could be several hundred times the average rate. The maximum overtopping rate would have significant influence on the drainage system.

- The direct influence of wind on runup and overtopping was not included in these calculations. Wind significantly increases overtopping and spray. Physical modelling would assist in establishing the influence of wind.

- Previous physical modelling results suggest that during a flood wave, deflecting structures will prove useful in reducing overtopping by a factor of about 2 and hence increase the usage time for the evacuation route.

The following recommendations result from this study:

- A more detailed investigation be carried out on the joint occurrence of extreme flood and high winds based on actual dates and duration of historical floods at Windsor.

- The installation of deflectors would reduce overtopping and reduce design levels for the evacuation route. It is recommended that a physical model be used to simulate site specific overtopping and the influence of wind and wave grouping on overtopping due to waves on a 1:2.5 embankment.

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Appendix A

Return Period Wave Calculations for 16 m Floods

HINDCAST FOR -
16m flood northern option SITE
Maximum fetch length is 7400.m to the S
Terrain Category 2 10 Min Ave windsor Design Wind Data

NOTE - H1% refers to the wave height that only 1% of waves will exceed (SPM 1984, pp3-6)

Summary For N (Fetch Length = 1000. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	12.5	.25	1.5	12.9	.39
20.000	17.5	.38	1.8	10.2	.58
50.000	18.6	.41	1.8	9.8	.63
100.000	20.0	.45	1.9	9.4	.69

Summary For NNE (Fetch Length = 1400. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	14.5	.36	1.8	15.0	.55
20.000	18.1	.47	2.0	12.9	.72
50.000	20.2	.54	2.1	11.9	.82
100.000	21.1	.57	2.1	11.6	.86

Summary For NE (Fetch Length = 4600. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.6	.75	2.8	32.4	1.14
20.000	18.8	.87	2.9	29.8	1.32
50.000	20.8	.98	3.1	27.8	1.49
100.000	22.2	1.06	3.1	26.5	1.61

Summary For ENE (Fetch Length = 3700. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	14.8	.59	2.5	30.0	.90
20.000	18.5	.77	2.7	25.8	1.17
50.000	20.0	.85	2.8	24.4	1.29
100.000	21.5	.92	2.9	23.3	1.40

Summary For E (Fetch Length = 900. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	13.0	.25	1.5	11.6	.38
20.000	18.1	.38	1.7	9.2	.58
50.000	19.2	.41	1.8	8.9	.62
100.000	20.8	.45	1.8	8.4	.68

Summary For ESE (Fetch Length = 550. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	14.5	.23	1.3	7.4	.34
20.000	20.1	.34	1.5	5.9	.51
50.000	22.0	.38	1.6	5.6	.57
100.000	23.3	.41	1.6	5.4	.62

Summary For SE (Fetch Length = 2700. m Depth = 11.0m)

.003	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.1	.56	2.3	22.6	.85
20.000	22.2	.83	2.7	18.1	1.26
50.000	24.7	.94	2.8	16.9	1.43
100.000	25.8	.99	2.8	16.4	1.50

Summary For SSE (Fetch Length = 4500. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.8	.76	2.8	31.6	1.15
20.000	27.7	1.36	3.4	22.4	2.06
50.000	24.8	1.20	3.3	24.2	1.82
100.000	26.0	1.26	3.3	23.4	1.92

Summary For S (Fetch Length = 7400. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	17.5	1.00	3.3	43.4	1.51
20.000	23.3	1.38	3.7	35.7	2.10
50.000	25.0	1.50	3.8	34.0	2.27
100.000	26.1	1.57	3.9	33.0	2.39

Summary For SSW (Fetch Length = 3500. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.5	.66	2.6	26.8	1.00
20.000	27.7	1.22	3.2	18.8	1.84
50.000	24.8	1.07	3.0	20.3	1.62
100.000	26.0	1.13	3.1	19.6	1.71

ummary For SW (Fetch Length = 1100. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	15.5	.35	1.7	12.0	.53
20.000	22.2	.54	2.0	9.3	.82
50.000	24.2	.60	2.1	8.8	.91
100.000	25.0	.62	2.1	8.6	.94

ummary For WSW (Fetch Length = 900. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.6	.34	1.7	9.8	.52
20.000	23.1	.51	1.9	7.8	.78
50.000	24.8	.56	2.0	7.5	.85
100.000	26.0	.59	2.0	7.2	.90

ummary For W (Fetch Length = 250. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	17.7	.20	1.1	3.5	.30
20.000	24.1	.29	1.3	2.9	.43
50.000	25.5	.31	1.3	2.8	.47
100.000	26.9	.33	1.3	2.7	.50

mmary For WNW (Fetch Length = 300. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)

1.000	16.4	.20	1.2	4.3	.30
20.000	23.0	.30	1.3	3.4	.45
50.000	24.6	.32	1.4	3.3	.49
100.000	26.0	.34	1.4	3.1	.52

Summary For NW (Fetch Length = 740. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	15.2	.28	1.5	9.0	.42
20.000	21.9	.44	1.8	7.0	.66
50.000	23.8	.48	1.8	6.6	.73
100.000	25.0	.51	1.9	6.4	.78

Summary For NNW (Fetch Length = 820. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	13.8	.26	1.5	10.4	.40
20.000	19.7	.40	1.7	8.1	.61
50.000	21.2	.44	1.8	7.7	.67
100.000	22.5	.47	1.8	7.4	.72

Summary for All Directions Combined

Wave Heights

RI (years)	Hsig (m)	H1% (m)	Dominant Directions
.031	.00	.00	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.038	.10	.15	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.047	.20	.30	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.070	.30	.46	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.106	.40	.61	N NNE NE ENE E ESE SE SSE S SSW SW WSW NW NNW
.133	.50	.76	NNE NE ENE SE SSE S SSW SW WSW NW
.160	.60	.91	NE ENE SE SSE S SSW SW
.222	.70	1.06	NE ENE SE SSE S SSW
.354	.80	1.21	NE ENE SE SSE S SSW
.523	.90	1.37	NE ENE SE SSE S SSW
.695	1.00	1.52	NE SSE S SSW
1.387	1.10	1.67	SSE S SSW
2.690	1.20	1.82	SSE S SSW
6.128	1.30	1.97	SSE S
22.688	1.40	2.12	S
50.305	1.50	2.28	S
100.000	1.57	2.39	S

Wave Period

RI (years)	Tp (s)	Dominant Directions
.031	.00	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.035	.30	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.039	.60	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.044	.90	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.053	1.20	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.064	1.50	N NNE NE ENE E ESE SE SSE S SSW SW WSW NW NNW
.108	1.80	N NNE NE ENE E SE SSE S SSW SW WSW NW NNW
.143	2.10	NNE NE ENE SE SSE S SSW SW
.166	2.40	NE ENE SE SSE S SSW
.265	2.70	NE ENE SE SSE S SSW
.643	3.00	NE SSE S SSW
1.058	3.30	SSE S
10.594	3.60	S
100.000	3.86	S

16m flood southern option SITE
Maximum fetch length is 6900.m to the S
Terrain Category 2 10 Min Ave windsor Design Wind Data

NOTE - H1% refers to the wave height that only 1% of waves will exceed (SPM 1984, pp3-6)

Summary For N (Fetch Length = 600. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	12.5	.20	1.3	8.8	.30
20.000	17.5	.30	1.5	7.0	.45
50.000	18.6	.32	1.5	6.7	.49
100.000	20.0	.35	1.6	6.4	.53

ummary For NNE (Fetch Length = 2200. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	14.5	.45	2.1	20.9	.68
20.000	18.1	.59	2.3	18.0	.89
50.000	20.2	.67	2.4	16.7	1.02
100.000	21.1	.71	2.5	16.2	1.07

ummary For NE (Fetch Length = 4400. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.6	.74	2.8	31.4	1.12
20.000	18.8	.85	2.9	28.8	1.30
50.000	20.8	.96	3.0	26.9	1.46
100.000	22.2	1.04	3.1	25.7	1.58

ummary For ENE (Fetch Length = 3500. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	14.8	.58	2.4	28.8	.87
20.000	18.5	.75	2.7	24.8	1.14
50.000	20.0	.83	2.8	23.5	1.25
100.000	21.5	.90	2.9	22.4	1.37

ummary For E (Fetch Length = 3600. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	13.0	.50	2.3	32.1	.76
20.000	18.1	.74	2.7	25.7	1.13
50.000	19.2	.80	2.7	24.6	1.21
100.000	20.8	.88	2.8	23.3	1.33

ummary For ESE (Fetch Length = 3200. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	14.5	.54	2.4	27.4	.82
20.000	20.1	.80	2.7	22.0	1.21
50.000	22.0	.89	2.8	20.6	1.35
100.000	23.3	.95	2.9	19.8	1.44

ummary For SE (Fetch Length = 2000. m Depth = 11.0m)

.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.1	.49	2.1	18.1	.74
20.000	22.2	.72	2.4	14.6	1.09
50.000	24.7	.82	2.5	13.5	1.24
100.000	25.8	.86	2.6	13.1	1.31

mmary For SSE (Fetch Length = 3800. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.8	.70	2.6	28.1	1.06
20.000	27.7	1.26	3.2	19.9	1.91
50.000	24.8	1.11	3.1	21.5	1.68
100.000	26.0	1.17	3.2	20.8	1.78

mmary For S (Fetch Length = 6900. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	17.5	.96	3.2	41.4	1.46
20.000	23.3	1.34	3.6	34.0	2.04
50.000	25.0	1.46	3.7	32.4	2.21
100.000	26.1	1.53	3.8	31.4	2.32

mmary For SSW (Fetch Length = 3100. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.5	.62	2.5	24.6	.94
20.000	27.7	1.15	3.0	17.2	1.75
50.000	24.8	1.01	2.9	18.6	1.53
100.000	26.0	1.07	3.0	18.0	1.62

mmary For SW (Fetch Length = 800. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	15.5	.30	1.6	9.4	.45
20.000	22.2	.46	1.8	7.4	.70
50.000	24.2	.51	1.9	6.9	.78
100.000	25.0	.53	1.9	6.8	.81

mmary For WSW (Fetch Length = 1100. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	16.6	.38	1.8	11.4	.57
20.000	23.1	.56	2.0	9.1	.86
50.000	24.8	.62	2.1	8.7	.93
100.000	26.0	.65	2.1	8.4	.99

mmary For W (Fetch Length = 600. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	17.7	.30	1.5	6.9	.46
20.000	24.1	.44	1.7	5.6	.67
50.000	25.5	.47	1.7	5.4	.72
100.000	26.9	.51	1.8	5.2	.77

mmary For WNW (Fetch Length = 850. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)

1.000	16.4	.33	1.6	9.5	.50
20.000	23.0	.50	1.9	7.5	.75
50.000	24.6	.54	1.9	7.2	.82
100.000	26.0	.57	2.0	6.9	.87

Summary For NW (Fetch Length = 1000. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	15.2	.32	1.7	11.3	.49
20.000	21.9	.51	1.9	8.8	.77
50.000	23.8	.56	2.0	8.3	.85
100.000	25.0	.59	2.0	8.0	.90

Summary For NNW (Fetch Length = 830. m Depth = 11.0m)

RI (years)	V (m/s)	Hsig (m)	Tp (s)	Duration (mins)	H1% (m)
.019	.0	.00	.0	.3	.00
.083	.0	.00	.0	.3	.00
.500	.0	.00	.0	.3	.00
1.000	13.8	.26	1.5	10.5	.40
20.000	19.7	.41	1.7	8.2	.61
50.000	21.2	.44	1.8	7.8	.67
100.000	22.5	.48	1.8	7.5	.72

Summary for All Directions Combined

Wave Heights

RI (years)	Hsig (m)	H1% (m)	Dominant Directions
.031	.00	.00	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.037	.10	.15	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.044	.20	.30	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.054	.30	.46	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.078	.40	.61	NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.104	.50	.76	NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW
.158	.60	.91	NNE NE ENE E ESE SE SSE S SSW WSW
.232	.70	1.06	NNE NE ENE E ESE SE SSE S SSW
.418	.80	1.21	NE ENE E ESE SE SSE S SSW
.603	.90	1.37	NE ENE ESE SSE S SSW
.919	1.00	1.52	NE SSE S SSW
1.896	1.10	1.67	SSE S SSW
4.447	1.20	1.82	SSE S
14.190	1.30	1.97	S
31.727	1.40	2.12	S
76.112	1.50	2.28	S
100.000	1.53	2.32	S

Wave Period

RI (years)	Tp (s)	Dominant Directions
.031	.00	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.035	.30	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.039	.60	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.043	.90	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.048	1.20	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.055	1.50	N NNE NE ENE E ESE SE SSE S SSW SW WSW W WNW NW NNW
.083	1.80	NNE NE ENE E ESE SE SSE S SSW SW WSW WNW NW NNW
.100	2.10	NNE NE ENE E ESE SE SSE S SSW WSW
.148	2.40	NNE NE ENE E ESE SE SSE S SSW
.303	2.70	NE ENE E ESE SSE S SSW
.770	3.00	NE SSE S SSW
1.921	3.30	S
18.316	3.60	S
100.000	3.78	S

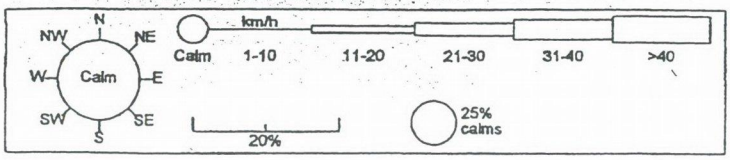
Appendix B

Bureau of Meteorology Wind Data for Richmond Airport

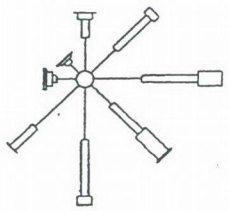
Wind Rose using available data between 1994 and 2000 for RICHMOND RAAF

9895255

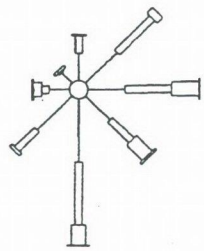
Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m



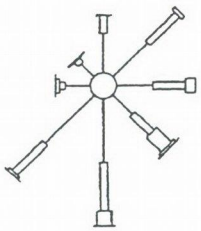
January 1559 observations



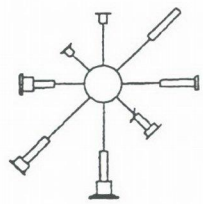
February 1428 observations



March 1555 observations



April 1500 observations



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Wind Frequency analysis using available data between 1994 and 2000 for RICHMOND RAAF

Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m

Values are percentage frequencies; * indicates the range occurred but with a frequency of less than 0.5%.

January

Calm 9					1559 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	5	5	7	4	8	8	3	2	41
11-20	3	7	8	6	8	4	1	1	37
21-30	1	1	3	5	1	*	1	*	13
31-40	*			*			*	*	1
>40									
All	8	12	18	15	17	13	5	3	100

February

Calm 10					1428 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	4	6	5	5	9	7	3	2	41
11-20	3	7	6	4	9	4	1	*	34
21-30	*	1	4	3	3	1	1	*	14
31-40			*	*	*		*		1
>40									
All	7	14	16	13	21	11	5	3	100

March

Calm 14					1555 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	6	7	5	4	9	10	3	3	46
11-20	2	6	5	4	7	6	1	1	31
21-30	*	1	1	3	2	*	1	*	8
31-40				*	*	*	*	*	1
>40				*					*
All	8	13	11	11	18	16	5	4	100

April

Calm 21					1500 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	6	7	5	4	7	8	4	3	44
11-20	1	6	5	3	4	5	3	1	27
21-30	*		*	1	2	1	2	*	6
31-40				*	*	*	*		1
>40					*				*
All	8	12	11	7	14	14	9	5	100

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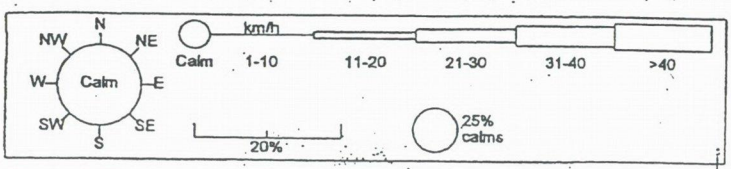
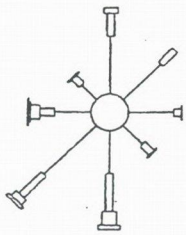
Wind Rose showing available data between 1994 and 2000 for RICHMOND RAAF

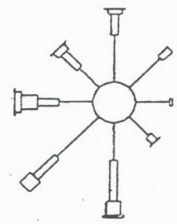
Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m



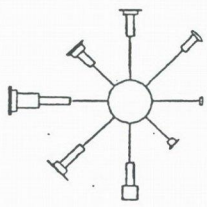
May 1446 observations



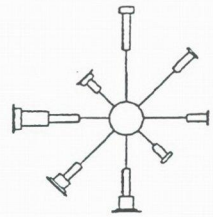
June 1419 observations



July 1457 observations



August 1464 observations



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Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m

Values are percentage frequencies; * indicates the range occurred but with a frequency of less than 0.5%.

May

Calm 21					1446 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	8	7	6	4	6	10	5	3	50
11-20	3	3	1	2	6	4	2	1	22
21-30	1		*	*	2	1	1	*	6
31-40					1	*	*		1
>40							*		*
All	12	10	8	6	14	16	9	5	100

June

Calm 24					1419 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	7	6	5	4	5	8	5	4	44
11-20	3	2	*	1	6	4	3	3	23
21-30	1			*	2	2	2	1	8
31-40	*				*		1	*	1
>40							*		*
All	11	8	5	5	14	14	11	8	100

July

Calm 24					1457 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	6	7	7	5	6	6	5	4	46
11-20	3	3	*	1	3	4	4	2	21
21-30	1	*		*	2	1	3	1	8
31-40	*						1	*	1
>40						*	*		*
All	10	10	7	6	11	11	13	8	100

August

Calm 19					1464 observations				
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	7	7	6	3	5	7	4	3	43
11-20	5	3	3	2	4	3	4	2	27
21-30	1	*	*	*	2	1	4	1	9
31-40					*	*	1	*	2
>40					*	*	*		1
All	13	11	9	6	11	11	13	6	100

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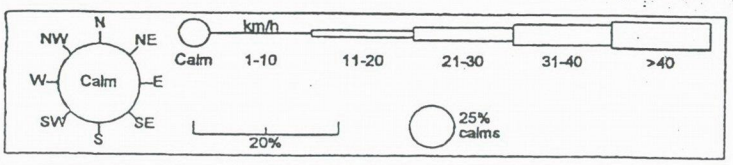
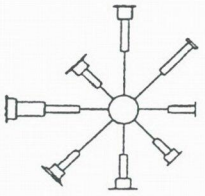
Wind Ros[illegible]g available data between 1993 and 1999 for RICHMOND RAAF

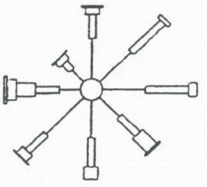
Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m



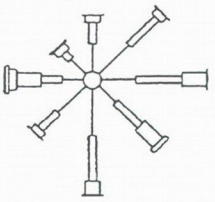
September 1388 observations



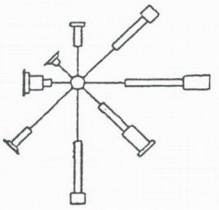
October 1506 observations



November 1522 observations



December 1581 observations



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Wind Frequency analysis using available data between 1993 and 1999 for RICHMOND RAAF

Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m

Values are percentage frequencies; * indicates the range occurred but with a frequency of less than 0.5%.

September

Calm 17	1388 observations								
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	6	5	4	4	5	7	4	3	38
11-20	5	6	4	3	3	3	5	3	32
21-30	2	*	*	1	1	1	4	1	11
31-40	*			*	*	*	1	*	2
>40					*	*	*	*	*
All	13	12	8	8	10	12	14	7	100

October

Calm 13	1506 observations								
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	6	6	6	4	5	8	3	2	41
11-20	4	6	6	4	4	4	3	2	32
21-30	1	1	1	3	2	1	3	1	12
31-40	*			*		*	1	*	2
>40							*		*
All	11	13	13	11	12	12	11	5	100

November

Calm 10	1522 observations								
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	4	6	5	3	7	5	3	3	36
11-20	3	5	7	5	7	3	3	1	34
21-30	1	1	3	4	2	1	3	1	16
31-40	*	*	*	1	*	*	1	*	2
>40							1		1
All	8	13	15	12	16	10	11	5	100

December

Calm 8	1581 observations								
km/h	N	NE	E	SE	S	SW	W	NW	All
1-10	4	6	6	4	7	9	3	3	41
11-20	3	6	8	5	8	3	1	1	35
21-30	1	2	4	4	2	*	2	*	14
31-40				1		*	1	*	1
>40							*		*
All	8	14	17	13	17	12	7	4	100

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Wind Frequency Analyses and Wind Roses

The included set of wind frequency analysis tables and/or wind roses show the frequency with which winds of various strengths are observed coming from various directions. These notes should help you to use the information.

Data

Wind speed and direction are measured by a number of means. In some cases, they are only estimated. To find out exactly which method has been used, a search of the appropriate station history file would be required.

The data are collected by the National Climate Centre in the Bureau of Meteorology's Melbourne head office. They are stored in ADAM (the Australian Data Archive for Meteorology), an extensive computer database of meteorological observations.

As the observations are stored, basic checks are performed. Any observations that fail these tests (specifically, any whose quality flag is poorer than "4 - estimated, medium certainty") are excluded from the frequency analyses that follow.

Analysis

The data are collated in a number of ways, depending on the nature of your request.

To group by hour, the observations are assigned to the closest standard three-hour reporting time. For example, all observations between 7:30 am and 10:30 am local standard time are labelled "9 am".

If a seasonal grouping has been requested, then "autumn" is March, April and May, "winter" is June, July and August, "spring" is September, October and November, and "summer" is December, January and February.

The observations are then grouped by speed. The exact number of speed ranges and their size depends on your request. When the speed ranges are labelled, "1 - 10" is used for all speeds greater than 0 but less than or equal to 10. "11 - 20" means greater than 10 but less than or equal to 20.

The data are then grouped by direction; into 8 or 16 bins as requested. When doing this, observations that fall on a bin boundary are split equally between the two bins. For example, when grouping into 8 bins, a direction of "NE" covers all observations with directions strictly between NNE and ENE; "E" covers from ENE to ESE. If the direction is exactly ENE (67.5°), then it will be placed half in the "NE" bin and half in "E" one.

Tables

If you have requested wind frequency tables, then you will get a separate table for each time group. Each table shows the time to which it applies, and the total number of observations used at that time. The percentage frequency with which calm conditions (that is, no wind) are observed are displayed at the top left of the table.

The rest of the table is laid out with directions across and speeds down. To find the frequency with which winds of a given speed and direction occur, follow down the appropriate direction column and across the speed row until they intersect. The value printed there is the frequency you require. For example, a value of "14" indicates that this speed/direction group occur about 14% of the time. "*" indicates the range has occurred but less than 0.5% of the time.

The last column, labelled "All", gives the frequency of each speed range regardless of direction. Similarly, the last row gives the frequency of each direction, regardless of speed.

Roses

Wind Roses seek to make the data in a wind frequency table easier to digest. Although not ideal for quantitative work, they are good for providing a quick visual impression of the wind regime.

Like the tables, there is one wind rose for each time group that you requested. Each rose consist of a central circle, surrounded by branches, each made up of a number of petals.

The circle represents the frequency of calm conditions. The size of the circle is proportional to the number of calms; a scale is given in the legend at the top of the page.

Each branch represents the wind coming from that direction. North is to the top, and the other directions are shown in the legend. In each case, the wind is blowing from that direction toward the calms circle. Each petal corresponds to a speed range from that direction. The length of the petal is proportional to the frequency of that wind; the scale is shown in the legend. The thickness of the petal is used to indicated which speed range it represents.

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More Information

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Daily Data for RICHMOND RAAF

Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m

Speed of Maximum Wind Gust (km/h)

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Avg	Max	Min	Total	
Nov 1994	30	81	58	50	68	93	87	54	42	50	37	37	45	41	52	30	33	33	33	87	58	54	41	41	28	45	48	42	52	48		49.9	93	28		
Dec 1994	37	39	32	33	37	37	26	48	41	41	37	37	37	42	26	37	33	45	42	37	45	41	39	32	28	37	55	42	55	33	39	38.4	55	26		
Jan 1995	68	45	35	30	48	78	42	28	26	35	30	33	39	37	33	35	32	18	37	30	35	42	33	33	37	30	28	28	33	28	50	36.7	78	18		
Feb 1995	28	32	54	37	50	42	28	41	37	42	35	33	39	37	33	33	42	41	41	48	26	35	48	32	28	28	37	78				38.8	78	26		28
Mar 1995	52	50	52	50	48	48	45	21	32	37	21	54	33	26	52	28	28	52	54	28	21	22	30	33	33	21	41	33	32	22	55	37.1	55	21		31
Apr 1995	28	37	32	26	54	35	54	58	68	59	45	28	30	18	21	26	39	18	33	26	30	28	30	58	22		28	28	26	32		34.9	68	18		29
May 1995	17	22	26	33	33	41	33	21	30	21	26	32	52	37	21	17	39	50	35	21	39	48	21	21	71	68	41	21	21	18	18	31.9	71	17		31
Jun 1995	28	22	30	22	17	17	18	15	37	42	61	45	21	11	32	63	41	48	48	18	55	35	15	26	28	21	41	55	71	68		35.0	71	11		30
Jul 1995	72	30	41	18	15	13	28	17	13	26	55	58	48	71	52	17	52	55	50	18	26	41	48	35	18	26	42	35	39	17	52	36.4	72	13		31
Aug 1995	28	50	35	41	21	59	41	39	45	26	22	21	26	28	18	21	54	26	18	17	45	21	58	26	45	39	55	28	33	76	61	36.1	76	17		31
Sep 1995	28	17	37	45	71	41	37	35	30	32	39	30	32	39	39	18	28	26	33	84	42	30	41	37	71	58	35	39	32	30		38.4	84	17		30
Oct 1995	26	28	48	33	32	22	39	48	30	30	35	39	55	42	28	35	42	48	39	54	39	18	59	41		21	30	30	39	50	35	37.2	59	18		30
Nov 1995	33	98	41	33	52	48	58	50	32	58	39	39	37	26	39	28	18	45	35	35	30	28	48	30	42	30	74	78	50	41		43.1	98	18		30
Dec 1995	50	30	33	42	41	71	41	33	37	45	48	55	32	35	28	41	33	68	71	50	45	35	35	41	39	35	52	37	35	30	32	41.9	71	28		31
Jan 1996	35	52	28	28	32	35	32	32	37	33	37	37	32	33	39	48	22	26	78	65	30	26	33	37	50	48	35	41	50	30	76	39.2	78	22		31
Feb 1996	55	48	37	35	37	33	39	55	52	39	48	48	52	50	58	42	37	33	41	35	39	41	39	37	33	30	52	65	28			42.7	65	28		29
Mar 1996	26	42	33	32	30	28	50	39	39	41	22	32	22	45	33	74	48	33	32	35	39	30	26	26	30	28	35	33	32	37	35	35.1	74	22		31
Apr 1996	42	45	18	30	42	32	52	32	41	35	35	42	30	61	26	37	50	50	35	28	26	37	30	22	26	26	41	35	26	26		35.3	61	18		30
May 1996	21	33	48	37	28	22	35	33	21	18	33	33	41	35	26	28	32	33	17	39	32	28	18	28	28		13	15	18	26	17	27.9	48	13		30
Jun 1996	32	21	18	41	18	17	28	17	15	32	35	35	30	18	28	30	15	21	18	22	11	13	65	48	22	55	37	30	45	28		28.1	65	11		30
Jul 1996	22	37	17	26	26	54	32	41	26	15	33	15	41	42	39	39	17	18	63	84		45	41	50	22	15	18	28	54	21	21	33.3	84	15		30
Aug 1996	54	42	61	68	58	37	52	52	42	26	22	63	28	71	21	26	22	61	61	30	18	30	18	26	41	26	18	42	30	52	95	41.7	95	18		31
Sep 1996	52	45	52	41	54	50	37	41	48	37		78	63	55	41	48	39	32	54	50	35	33	28	58	37	42	37	28	78	91		47.7	91	28		29
Oct 1996	61	35	30	28	37	41	28	26	28	45	45	61	32	28	35	45	32	22	45	45	32	30	30	33	28	37	21	48	37	52	28	36.1	61	21		31
Nov 1996	33	37	37	39	35	39	50	33	63	50	45	63	58	95	55	39	65	65	33	37	41	39	55	32	35	63	55	41	52	61		48.2	95	32		30
Dec 1996	42	84	33	41	35	39	71	32	45	50	52	37	41	37	41	41	32		52	41	35	37	42	39	50	63	26	58	54	26	50	44.1	84	26		30
Jan 1997	33	35	37	37	37	54	61	65	45	58	39		35		45	50	24	32	35	35	42	39	45	30	33	33	35	26	32	39	21	39.0	65	21		29
Feb 1997	41	45	35	35	28	28	33	28	35	39	32	41	35	42	28	46	33	28	28	32	39	24	42	33	30	42	32	32				34.5	46	24		28
Mar 1997	35	50	32	26	42	24	18	24	33	30	30	32	41	28	32	30	35	33	32	33	33	33	42	41	26	32	32	46	41	45	42	34.0	50	18		31
Apr 1997	28	21	50	18	21	35	42	35	28	30	26	21	24	41	33	26	28	41	22	33	28	33	32	35	28	18	21	18	18	24		28.6	50	18		30
May 1997	28	26	26	21	32	26	41	35	65	59	71	39	22	26	15	15	35	21	32	33	18	26	22	22	18	26	32	28	35	41	39	31.4	71	15		31
Jun 1997	33	13	15	11	30	58	42	21	18	11	11	18	48	28	46	46	46	45	28	22	21	11	42	24	28	37	21	18	30	37		28.7	58	11		30
Jul 1997	28	28	35	41	39	28	21	46	48	30	22	22	39	32	33	35	32	28	17	21	18	30	26	26	30	32	24	18	28	24	28	29.2	48	17		31
Aug 1997	33	32	28	24	30	24	45	54	33	21	48	48	41	18	24	42	71	41	22	28	22	22	41	59	50	46	24	32	26	26	37	35.2	71	18		31
Sep 1997	21	42	48	24	48	54	35	59	35	35	35	28	24	30	59	46	32	21	15	61	24	35	42	26	39	46	28	48	26	28		36.5	61	15		30
Oct 1997	35	59	68	35	28	41	30	46	39	30	42	59	46	45	39	46	39	50	32	46	39	32	35	35	32	30	58	30	24	30	54	40.4	68	24		31
Nov 1997	48	42	33	32	32	39	35	35	22		55	68	33	76	39	54	48	42	41	45	35	35	45		30	71	50	46	39	54		43.8	76	22		28
Dec 1997	32	68	48	41	37	39	33	24	48	52	39	45	41	48	59	41	50	46				61	52	52	37	45	32	22	39	39	45	43.4	68	22		28

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Daily Data for RICHMOND RAAF

Site Number 067105 • Locality: RICHMOND • Opened Sep 1993 • Still Open • Latitude 33°36'13"S • Longitude 150°46'35"E • Elevation 19m

Speed of Maximum Wind Gust (km/h)

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Avg	Max	Min	Total	Nb
Jan 1998	37	35		30	45	37	28	41	33	30	35	41	35	37	55	22	37	81	30	42	26	26	46	42	39	54	41	54	39	33	28	38.6	81	22		
Feb 1998	42	35	46		32	35	33	35	32	35	26	30	35	41	50	61	37	37	30	48	35	45	39	33	35	37	76	22				38.7	76	22		
Mar 1998	22	28	37	41	58	41	58	42	35	32	30				28	32	33	39	32	30	55	28	37	59	30	33	28	32	28	33	65	37.3	65	22		28
Apr 1998	35	28	35	32	28	32	30	39	35	35	24	28	50	35	26	18	42	22	28	32	42	33	32	35	54	54	21	22	21	21		32.3	54	18		30
May 1998	24	30	32	28	37	50	41	35	28	15	39	18	22	22	13	28	26	50	52	50	37	15	55	50	35	33	15		15	15	21	31.0	55	13		30
Jun 1998	15	26	22	42	45	37	42	46	33	39	32	41	32	17	21	33	37	15	22	30	33	37	68	48	61	22	24	17	21	54		33.7	68	15		30
Jul 1998	46	15	11	9	35	50	18	33	61	39	21	17	17	32	45	21	18	13	17	26	65	28	28	21	21	26	74	71	65		28	32.3	74	9		30
Aug 1998	21	42	22	22	33	50	71			15	37	18	17	21	35	24	24	41	30	26	30	48	24	30	26	39	45	35	18	21	28	30.7	71	15		29
Sep 1998	22	39	35	39	48	28	22	26	28	22	32	26	48	35	67	33	15	46	54	28	22	22	46	45	32	26	24	24	32	33		33.3	67	15		30
Oct 1998	39	41	61	39	32	76	76	35	21	42	32	32	72	32	52	33	33	72	32	61	41	37	37	37	37	37	61	72	46	37	28	52	45.1	76	21		31
Nov 1998	50	42	41	33	24	22	61		24	30	30	32	46	39	30	30	32	52	52	39	26	45	32	45	30	35	39	48	39	45		37.6	61	22		29
Dec 1998	42	32	46	42	33	28	42	30	37	28	30	37	50	67	24	24	32	50	32	41	41	37	42	35	35	39	58	41	35	42	28	38.1	67	24		31
Jan 1999	32	32	35	32	37	37	33	48	46	39	45	45	35	35	33	32	37	35	48	30	45	33	30	35	28	41	37	39	87	52	32	38.9	87	28		31
Feb 1999	35	33	41	41	37	24	48	39	50	37	32	30	21	28	55	52	32	48		42	33	37	32	52	50	41	37	28				38.3	55	21		27
Mar 1999	24	13	28	33	32	33	33	33	24	24	28	30	28	30	37	33	35	30	30	24	24	28	33	28	22	30	22	46	35	28	45	29.8	46	13		31
Apr 1999	30	24	30	28	24	37	28	22	37	30	37	24	24	32	35	22	24	21	22	28	50	52	52	45	28	18	46	63	45	45		33.4	63	18		30
May 1999	32	21	18	17	35	26	26	21	17	22	18	18	21	33	72	18	15	18	17	17	13	24	21	18	21	18	17	18	26	35	45	23.8	72	13		31
Jun 1999	26	21	30	18	21	22	32	28	17	35	46	24	67	68	50	33	30	48	39	17	13	35	42	32	24	48	26	22	21	17		31.7	68	13		30
Jul 1999	35	35	33	15	15	15	15	17		15	13	45	37	50	42	32	15	45	59	52	59	42	15	18	48	39	21	21	30	28	35	31.3	59	13		30
Aug 1999	37	22	17	22	33	22	32	39	55	18	21	21	58	55	39	28	18	21	42	55	42	24	21	22	28	37	24	35	35	28	32	31.7	58	17		31
Sep 1999	42	26	24	41	50	22	35	28	24	37	33	30	33	24	28	42	54	45	30	35	42	30	39	32	24	28	28	35		41		33.9	54	22		29
Oct 1999	32	35	21	26	28	37	22	32	30	48	61	30	32	15	28	28	26	37	35	33	28	41	33	74	54	61	28	32	30	22	37	34.6	74	15		31
Nov 1999	45	39	26	33	32	45	33	45	41	48	37	35	42	50	55	41	33	37	33	33	30	63	37	33	37	41	41	32	32	30		38.6	63	26		30
Dec 1999	32	32	32	39	28	35	35	35	46	37	46	35	37	35	35																	36.0	46	28		15

P. 10/10
May 19 '00 16:41
FROM BOM-SYDNEY-RMH TO:XP1-70-2676

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Appendix C

MHL Wind Data for Penrith

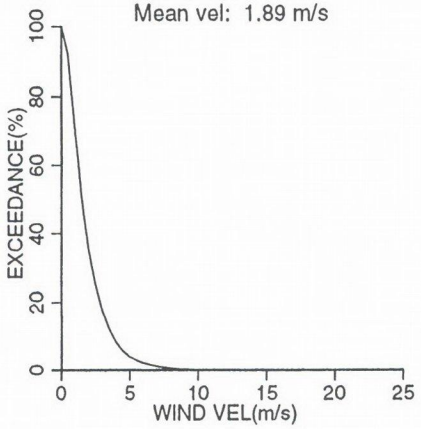
DIRECTIONAL EXCEEDANCE - WIND VELOCITIES

Site: Penrith wind

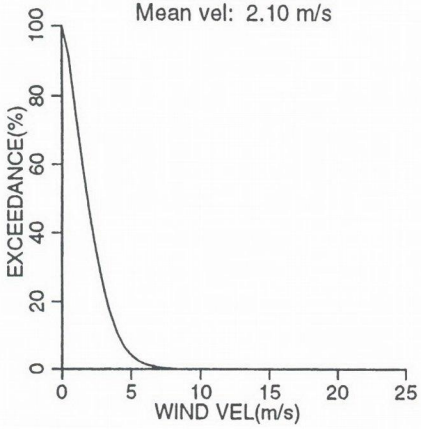
From: 01-01-1990 00:00 To: 31-12-1999 23:50

No. of records: 506828

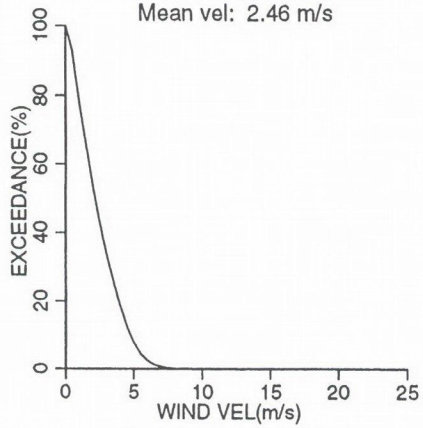
0.0°
No. of rec.: 59939
Mean vel: 1.89 m/s



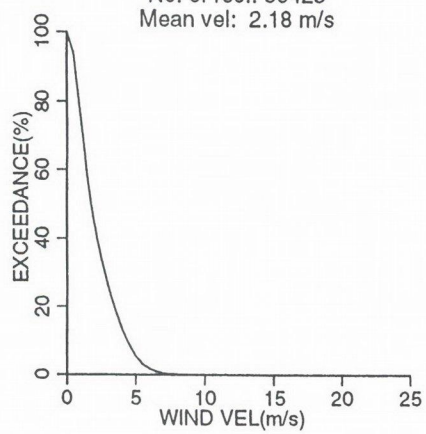
45.0°
No. of rec.: 52670
Mean vel: 2.10 m/s



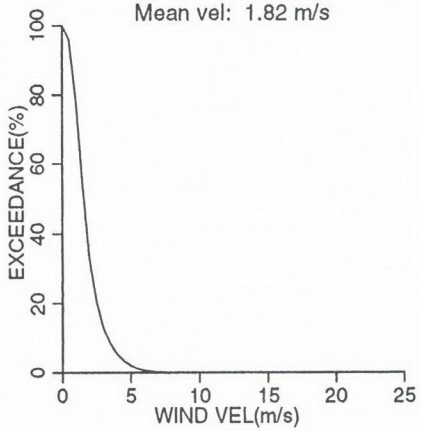
90.0°
No. of rec.: 48706
Mean vel: 2.46 m/s



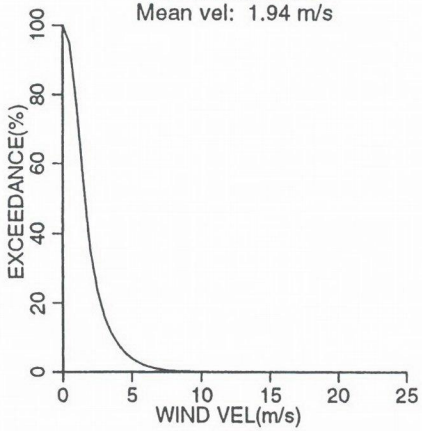
135.0°
No. of rec.: 58425
Mean vel: 2.18 m/s



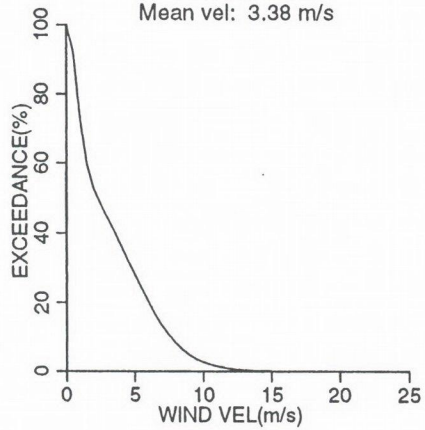
180.0°
No. of rec.: 109398
Mean vel: 1.82 m/s



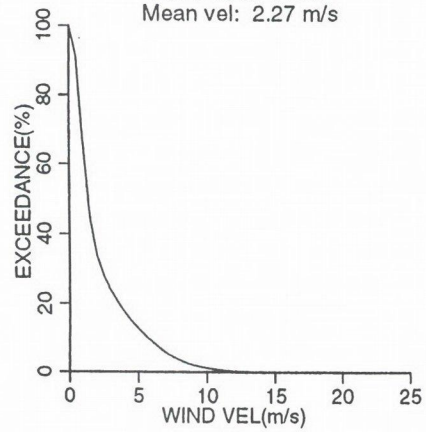
225.0°
No. of rec.: 84049
Mean vel: 1.94 m/s



270.0°
No. of rec.: 48731
Mean vel: 3.38 m/s



315.0°
No. of rec.: 44910
Mean vel: 2.27 m/s



MHL program: MET33
Time: 23-05-2000 10:45
By: MECNOTID

Appendix D

Preliminary Runup Calculations

Table D1 Runup Level Calculations for 15 m Flood and 1-in-1-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Centre of Northern Option (Direction)		Estimated Level of Maximum Wave m (AHD)	Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)		Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	0.55	2.5	15.8	16.6	16.2
100 m	0.59	2.8	15.8	16.6	16.3
200 m	0.63	2.8	15.9	16.6	16.3
300 m	0.67	2.8	16.0	16.6	16.3
400 m	0.71	2.8	16.0	16.7	16.3
500 m	0.75	2.8	16.1	16.7	16.3
1,000 m	0.96	3.2	16.3	17.2	16.7
1,500 m	0.84	3.3	16.2	17.1	16.6
2,000 m	0.86	3.0	16.2	17.1	16.6
2,100 m	0.68	3.0	15.9	16.8	16.3
2,200 m	0.51	3.0	15.6	16.5	16.1
2,300 m	0.34	3.0	15.5	16.2	15.9
2,400 m	0.17	3.0	15.2	15.9	15.7
2,500 m (Mulgrave)	0	0	15.0	15	15

A linear variation in wave height was assumed for the first and last 400 m along the route. The 2% runup level is estimated for 100 waves reaching the embankment. The levels estimated for the 1-in-1-year wave indicate that the proposed level of the crossing (17.3 m AHD) will not be exceeded. The runup due to the 1-in-20-year and 1-in-100-year wave will exceed the proposed level of the crossing. Runup was also estimated for an oblique wave on a grass slope for comparison purposes.

Table D2 Runup Level Calculations for a 15 m Flood and 1-in-20-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Centre of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	1.03 (ESE)	3.0	17.2	16.6
500 m	1.36 (SSE)	2.8	17.8	17.1
1,000 m	1.44 (S)	3.2	18.1	17.4
1,500 m	1.27 (S)	3.3	17.7	17.0
2,000 m	1.20 (S)	3.0	17.5	16.9
2,500 m (Mulgrave)	-	-	-	-

Table D3 Runup Level Calculations for 15 m Flood and 1-in-100-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Centre of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	1.10 (ESE)	3.2	17.3 (ESE)	16.7
500 m	1.56 (SSE)	3.6	18.1 (S)	17.3
1000 m	1.51 (S)	3.8	18.3 (S)	17.5
1500 m	1.33 (S)	3.4	17.7 (S)	17.1
2000 m	1.36 (S)	3.5	17.9 (S)	17.2
2500 m (Mulgrave)	-	-	-	-

Table D4 Runup Level Calculations for 16 m Flood for the Northern Route

Return Period (for a Southerly Wave)	Significant Wave Height and Period at Centre of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
1-in-1-year event	1.0 (south)	3.3	18.2	17.7
1-in-20-year event	1.38 (south)	3.7	19.0	18.3
1-in-50-year event	1.5 (south)	3.8	19.3	18.6
1-in-100-year Event	1.57 (south)	3.9	19.4	18.6

The runup during a 16 m flood due to a 1-in-1-year wave climate exceeds the proposed 17.3 m level of the proposed evacuation route.

Table D5 Runup Level Calculations for 17.3 m Flood and 1-in-1-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period (Direction)		Estimated Level of Maximum Wave m (AHD)	Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)		Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	0.58	2.6	18.1	18.7	18.4.
100 m	0.58	3.0	18.1	18.7	18.4
200 m	0.57	3.0	18.0	18.6	18.3
300 m	0.57	3.0	18.0	18.6	18.3
400 m	0.56	3.0	18.0	18.6	18.3
500 m	1.05	2.8	18.8	19.8	19.2
1,000 m	1.03	3.2	18.7	19.7	19.2
1,500 m	1.01	3.3	18.7	19.6	19.1
2,000 m	0.98	3.0	18.7	19.6	19.1
2,100 m	0.78	3.0	18.4	19.1	18.9
2,200 m	0.58	3.0	18.1	18.9	18.6
2,300 m	0.38	3.0	17.8	18.7	18.4
2,400 m	0.18	3.0	17.5	18.2	18.1
2,500 m (Mulgrave)	0	0	17.3	17.3	17.3

A linear variation in wave height was assumed for the last 400 m along the route.

Table D6 Runup Level Calculations for 17.3 m Flood and 1-in-20-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Centre of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	0.98 (ESE)	3.1	19.4	18.9
500 m	1.47 (S)	3.8	20.5	19.8
1000 m	1.44 (S)	3.8	20.5	19.8
1500 m	1.41 (S)	3.7	20.4	19.7
2000 m	1.37 (S)	3.7	20.3	19.6
2500 m (Mulgrave)	-	-	17.3	17.3

Table D7 Runup Level Calculations for 17.3 m Flood and a 1-in-100-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Centre of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Run up on Grass Slope at 30° (AHD)
0 m (Windsor)	1.16 (ESE)	3.3	19.8 (ESE)	19.2
500 m	1.67 (S)	4.0	20.9 (S)	20.1
1,000 m	1.64 (S)	4.0	20.9 (S)	20.1
1,500 m	1.60 (S)	3.9	20.8 (S)	20.0
2,000 m	1.56 (S)	3.8	20.7 (S)	19.9
2,500 m (Mulgrave)	1.45 (S)	3.6	20.3 (S)	19.6

Figure 4.1 indicates runup levels due to 1-in-1-year waves generated on a 15 m flood and a 17.3 m flood. The estimated runup levels indicate that the level of 17.3 m for the proposed evacuation route is not overtopped during a 15 m flood but will be heavily overtopped during a 17.3 m flood.

Appendix E Flood Glossary

annual exceedance probability	The chance or likelihood that an event of a nominated size or greater (e.g. flood discharge) will occur in any year.
Australian Height Datum (AHD)	A common national plane of level corresponding approximately to mean sea level.
bathymetry	The measurement of depths of water; also information derived from such measurements.
bed load	That portion of the total sediment load that flowing water moves along the bed by the rolling sediment particles .
catchment	The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
damage potential	The susceptibility of coastline development to damage by coastline hazards.
design floor level (DFL)	A minimum floor level specified as part of a building control program.
development	The erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land. **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power. **redevelopment:** refers to the rebuilding of an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require major extensions to urban services.
discharge	The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow which is a measure of how fast the water is moving rather than how much is moving.
ebb tide	The outflow of coastal waters from bays and estuaries caused by the falling tide.

effective warning time — Is equal to the available warning time, less the time taken to alert flood-affected people (by radio, television, loud-hailer or word of mouth), and have them commence effective evacuation procedures.

estuary — An enclosed or semi-enclosed body of water having an open or intermittently open connection to coastal waters in which water levels vary in a periodic fashion in response to ocean tides.

flood awareness — An appreciation of the likely effects of flooding and a knowledge of the relevant flood warning and evacuation procedures. In communities with a high degree of flood awareness, the response to flood warnings is prompt and efficient. In communities with a low degree of flood awareness, flood warnings are liable to be ignored and residents are often confused about when to evacuate, what to take and where it should be taken.

flood fringe — The remaining area of land affected by flooding, after floodway and flood storage areas have been defined.

flood hazard — Potential for damage to property or persons due to flooding.

flooding — The State Emergency Service uses the following definitions in flood warnings:
minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding, on the reference gauge, is the initial flood level and the upper limit is determined by local conditions.
moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic bridges may be covered. The range on the reference gauge is determined by local conditions.
major flooding: extensive rural areas are flooded with properties, villages and towns isolated and/or appreciable urban areas are flooded. The threshold for this class of flooding is the upper limit of moderate flooding.

flood-liable land — Land which would be inundated as a result of the standard flood.

flood mitigation works — Structures that are designed to manage floodwaters (eg. levees, retarding basins).

floodplain — The portion of a river valley, adjacent to the river channel, which is covered with water when the river overflows during floods.

floodplain management measures	The full range of techniques available to floodplain management.
floodplain management options	The measures which might be feasible for the management of a particular area.
flood proofing	A combination of measures incorporated in the design and/or construction and alteration of individual buildings or structures subject to flooding, for the reduction or elimination of flood damages.
flood standard (or designated flood)	should be based on an understanding of flood behaviour and the associated flood risk. It should also take into account social, economic and ecological considerations.
flood storages	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.
flood tide	The inflow of coastal waters into bays and estuaries caused by the rising tide.
floodways	Those areas where a significant volume of water flows during floods. They are often aligned with obvious naturally defined channels. Floodways are areas which, even if only partially blocked, would cause a significant redistribution of flood flow, which may in turn adversely affect other areas. They are often, but not necessarily, areas of deeper flow or the areas where higher velocities occur
fluvial	Pertaining to non-tidal flows.
foreshore	The area of shore between low and high tide marks and land adjacent thereto.
freeboard	A factor of safety usually expressed as a height above the designated flood. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects etc.
geomorphology	The study of the origin, characteristics and development of land forms.
high hazard	Possible danger to life and limb; evacuation by trucks difficult; potential for structural damage; social disruption and financial losses could be high.
hindcast wave climate	The use of historic wind climate to estimate the wave climate at a given location on a water body.

hydraulic	The term given to the study of water flow in a river, in particular the evaluation of flow parameters such as stage and velocity.
hydrograph	A graph which shows how the discharge changes with time at any particular location.
hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
low hazard	Should it be necessary, people and their possessions could be evacuated by trucks. Able-bodied adults would have little difficulty wading.
mainstream flooding	Inundation of normally dry land occurring when water conveyed to the locality from further upstream overflows the natural or artificial banks of the principal watercourse in the catchment. It generally excludes any watercourses constructed with pipes or artificial channels or considered as stormwater channels.
management plan	A document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, problems, special features and values of the area, the specific management measures which are to apply and the means and timing by which the plan will be implemented.
mathematical/computer models	The mathematical representation of the physical processes involved in runoff and streamflow. These models are often run on computers due to the complexity of the mathematical relationships. In this manual, the models referred to are mainly involved with rainfall, runoff and stream flow.
neap tides	Tides with the smallest range in a monthly cycle. Neap tides occur when the sun and moon lie at right angles relative to the earth (the gravitational effects of the moon and sun act in opposition on the ocean).
numerical model	A mathematical representation of a physical, chemical or biological process of interest. Computers are often required to solve the underlying equations.
peak discharge	The maximum discharge occurring during a flood event.
probable maximum flood	The flood calculated to be the maximum which is likely to occur.

probability	A statistical measure of the expected frequency or occurrence of flooding.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
sediment load	The quantity of sediment moved past a particular cross-section in a specified time.
significant wave height	The average height of the highest one third of waves recorded in a given monitoring period. Also referred to as $H_{1/3}$ or H_s.
spring tides	Tides with the greatest range in a monthly cycle, which occur when the sun, moon and earth are in alignment (the gravitational effects of the moon and sun act in concert on the ocean).
stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
stage hydrograph	A graph which shows how the water level changes with time. It must be referenced to a particular location and datum.
storm surge	The increase in coastal water levels caused by the barometric and wind setup effects of storms. Barometric setup refers to the increase in coastal water levels associated with the lower atmospheric pressures characteristic of storms. Wind setup refers to the increase in coastal water levels caused by an onshore wind driving water shorewards and piling it up against the coast.
tides	The regular rise and fall of sea level in response to the gravitational attraction of the sun, moon and planets. Tides along the New South Wales coastline are semi-diurnal in nature, i.e. they have a period of about 12.5 hours.
total catchment management (in the context of the Catchment Management Act, 1989)	"The coordinated and sustainable use of land, water, vegetation and other natural resources on a water catchment basis so as to balance resource utilisation and conservation".
water surface profile	A longitudinal plot showing the flood stage at any given location along a watercourse.
wave setup	The increase in water level within the surf zone above mean still water level caused by the breaking action of waves.

Working Paper 5 – Investigation into Joint Occurrence of High Floods and Wind Waves

INVESTIGATION INTO THE JOINT OCCURRENCE OF HIGH FLOODS AND WIND WAVES AT WINDSOR

Report No. MHL1085

NSW Department of Public Works and Services
Manly Hydraulics Laboratory

Report No. MHL1085
DPWS Report No. 00102
MHL File No. HS6-00052
First published August 2001
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Foreword

The study was carried out by the NSW Department of Public Works and Services' Manly Hydraulics Laboratory (MHL) on behalf of the Department of Land and Water Conservation (the Hawkesbury-Nepean Floodplain Management Strategy Group). This investigation was initiated to clarify assumptions made in a previous investigation (MHL 2000) regarding the joint occurrence of high winds and extreme floods. The study was undertaken and report prepared by Indra Jayewardene. Bronson McPherson carried out the regression analysis. Drafting was done by Kerrie Ellis and report production by Megan Jensen.

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2.4 Time Series of Flood Level (Maximum 11.63 m AHD) and Wind Data (Maximum 42 km/hr) For Event 13

A1 Flood Level Statistics
A2 Flood Level Statistics
A3 Flood Level Statistics
B1 Regression Analysis for Windsor and Sydney Winds During High (>6 m) Windsor Floods
B2 Duration-Averaged Maximum Gusts of 10 Minutes (Directional) Return Period Data
C1 Comparison of Hindcast and Measured Waves at Lake Victoria
C2 Comparison of Hindcast and Measured Waves at Lake Victoria
C3 Comparison of Hindcast and Measured Waves at Lake Victoria
C4 Comparison of Hindcast and Measured Waves at Lake Victoria
C5 Simultaneous Wind and Wave Measurement at Lake Victoria (NSW)

1. Introduction

1.1 Background

The NSW State Government is committed to evacuating the communities of Windsor and other towns in the Hawkesbury-Nepean Valley as a floodplain management strategy. As part of the regional evacuation route upgrade program being carried out under the Nepean Floodplain Management Strategy it is proposed to provide a high level evacuation route over South Creek. Figures 1.1 and 1.2 show the possible locations of the north and south routes currently under consideration. These routes predate Connell Wagner's current designs. The routes were considered sufficiently close to the final designs to estimate wave inundation levels. This study addresses inundation levels at the northern route only, as initial calculations (MHL 2000) indicated that only a small variation (<5%) existed for the wave heights at the southern route.

In an earlier investigation (MHL 2000) Manly Hydraulics Laboratory estimated wave inundation levels on the evacuation route due to 15 m, 16 m and 17.3 m (1% AEP) floods and a range of wind and wave return periods. The importance of accurately estimating the joint occurrence of wind and flood was underlined in the inundation level estimates from the previous report. The runup calculations for the 16 m and 17.3 m floods exceeded the proposed design value of the evacuation route and resulted in unacceptable levels of overtopping. In the event of high floods, however, a wave barrier is an option, resulting in possible lower embankments. The runup calculations on a proposed 1:2.5 embankment may be summarised as follows:

- The 1-in-1-year wind wave event occurring from a southerly direction results in a maximum runup level estimate of 17.2 m AHD for a 15 m AHD flood.

- The 1-in-1-year wind wave event occurring from a southerly direction results in runup levels higher than the level (17.3 m AHD) of the northern crossing for a 16 m AHD flood. The maximum runup level was estimated to be 18.2 m AHD.

- The 1-in-1-year wind wave event occurring from a southerly direction results in runup levels higher than the level (17.3 m AHD) of the northern crossing for a 17.3 m AHD flood. The maximum runup level was estimated to be 19.8 m AHD.

The overtopping discharge for a 16 m flood varied from 1.1 litres/m/s to 12.6 litres/m/s for a 17.3 m design level. These rates of overtopping are classified as unacceptable by Japanese criteria (Fukuda 1974) and are acceptable only for embankments and revetments that are built to the highest standards in Holland (Van der Meer 1994). There are no accepted overtopping standards for Australia.

There is no established joint probability relationship for wind and flood at South Creek.. The Department of Land and Water Conservation (DLWC) requested Manly Hydraulics Laboratory to investigate the joint occurrence of high flood level and high wind by analysing the time series for high (>6 m AHD) flooding events in order to obtain a better understanding of the return period of the wind events that could occur with a high flood.

Many hydrologic systems have both random and deterministic components. Whilst the deterministic component such as the flood level can be modelled mathematically, the random or stochastic components are parameters defined by means of probability distributions. The occurrence of high wind (and therefore wind waves) simultaneously with high flood is a random event which can be simulated accurately only if the joint probability relationship between the occurrence of a flood event and a wind event has been accurately estimated. The difficulty associated with establishing the dependency between winds and floods makes this a task beyond the scope of this investigation. Since no previous attempt has been made to establish such a relationship the focus of this investigation is to analyse time series of jointly occurring wind and flood level data during high floods (>6 m AHD). This is similar to the approach adopted in the Lake Macquarie flood study (MHL 1997a).

1.2 Study Objectives and Approach

The objectives of the present study were outlined during discussions with the Department of Land and Water Conservation. The broad strategy was to collate high flood (>6 m AHD) and wind data (during the flood event) for both Richmond and Sydney anemometers. A correlation between Richmond wind and Sydney wind is required so as to estimate the error that may have been introduced in using return period Sydney wind data for wave hindcasts instead of Richmond data in a previous study (MHL 2000). The Richmond data was not used in the earlier analysis (MHL 2000) due to the unavailability of return period data.

The approach taken in this investigation is as follows:

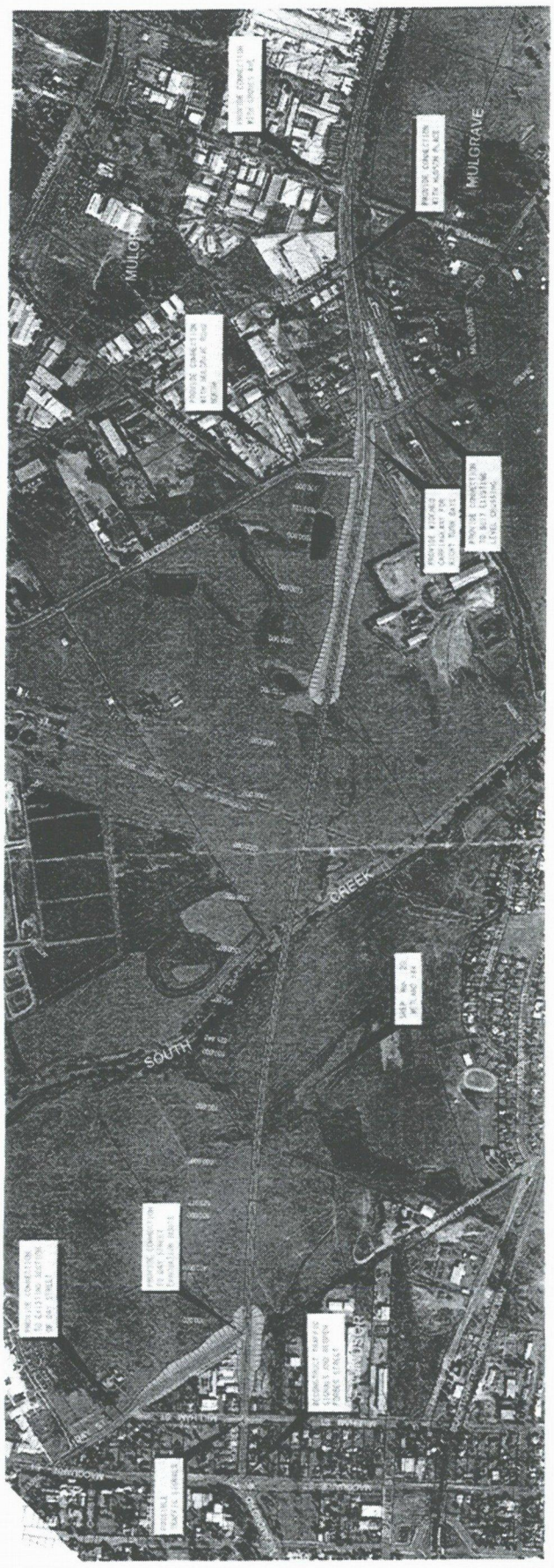
- Identify all dates and durations of floods over 6 m AHD that have been recorded for the Windsor area for the period 1949 to the present. This data was entered in an ACCESS database.
- The Bureau of Meteorology has supplied wind direction and speed (10-minute average) for Sydney Airport and Richmond for periods that coincide with the abovementioned flood events. This data was also entered in an ACCESS database.
- High winds and high water levels are not uncorrelated in the Windsor area. However the resources required to go to detailed climate modelling to assess the joint occurrence of floods and winds were not available in this project. The approach of looking at existing events provides a reasonable approach. Also there was not sufficient data to justify detailed determination of joint statistics.
- The time histories for both flood and winds for winds associated with all flood events over 13 m AHD were plotted so as to obtain an understanding into the coincidence of a major flood event and winds.
- The three highest wind events in this data set were identified to obtain an estimate of the extreme wind events that could occur during a flood at Windsor.

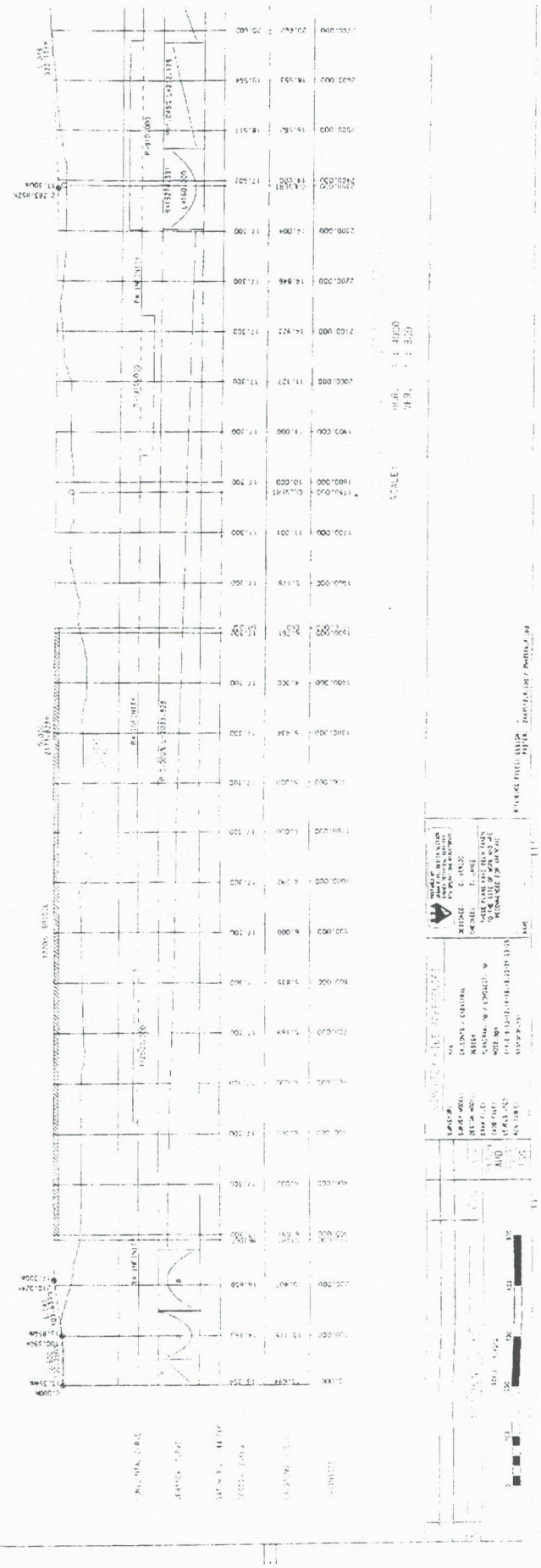
- Historical evidence was obtained of the coincidence of wave and wind events at another site, such as Lake Victoria (MHL 1997b) with similar depth and fetch. The CERC (1984) hindcasting formula was applied using the wind data and the fetch values for Lake Victoria in order to hindcast waves. The value of the hindcast wave height will be compared with actual measured heights at Lake Victoria to obtain a better appreciation for the accuracy of the CERC hindcasting formula used in the previous investigation to hindcast wind waves.
- A regression analysis was carried out on the winds measured at Windsor and Sydney for flood events exceeding 6 m AHD. This regression analysis would give a better quantification for error introduced into the hindcast wind waves when using the wind speed return periods for Sydney Harbour instead of the Richmond wind data.

1.3 Data

The data for all flood events exceeding 6 m AHD is available on the MHL computer database. Thirty flood events (1949-1999) where the peak flood level exceeded 6 m AHD were identified on the MHL database. The wind data associated with each of these events was obtained from the Bureau of Meteorology. The ACCESS database was queried to obtain information on the events such as:

- duration of a flood event above a threshold value (i.e. flood levels greater than 13 m AHD)
- the timing of the peak values for flood levels and wind speeds during a flood event
- relationships between the flood data and wind data such as the time differences between the peaks of flood and wind data.





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EVACUATION ROUTE - THE NORTHERN ROUTE

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Figure 1.1

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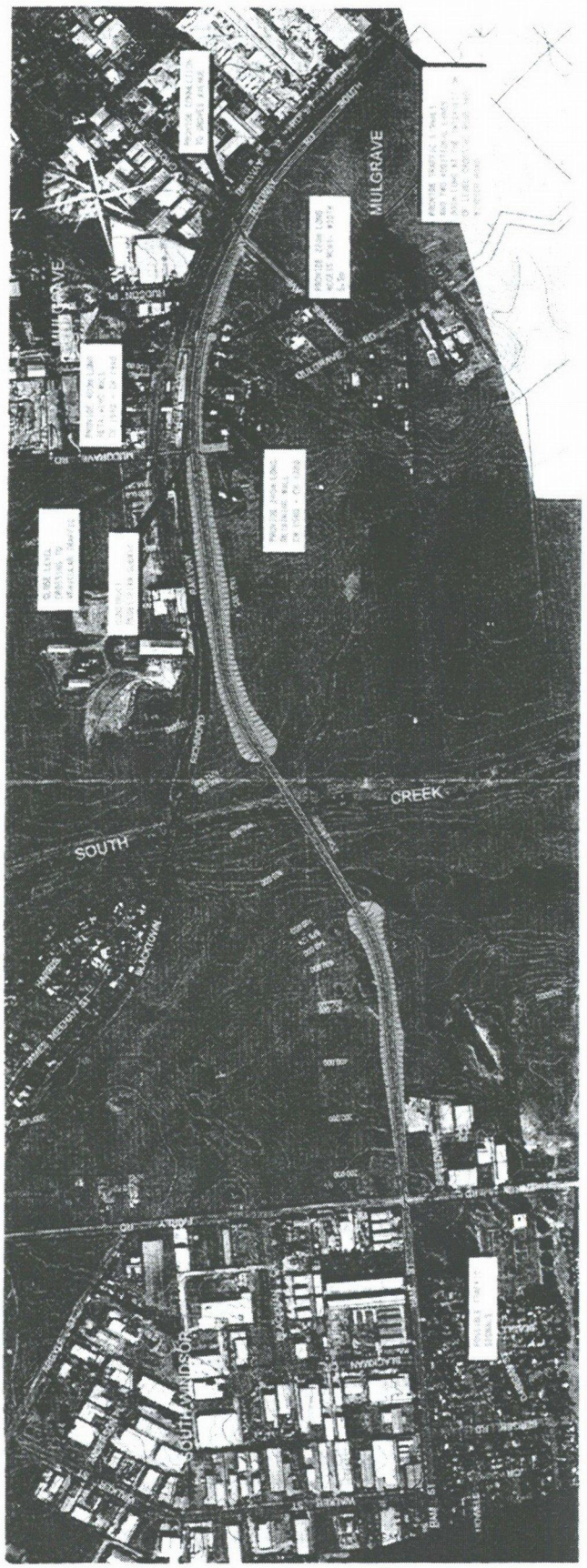

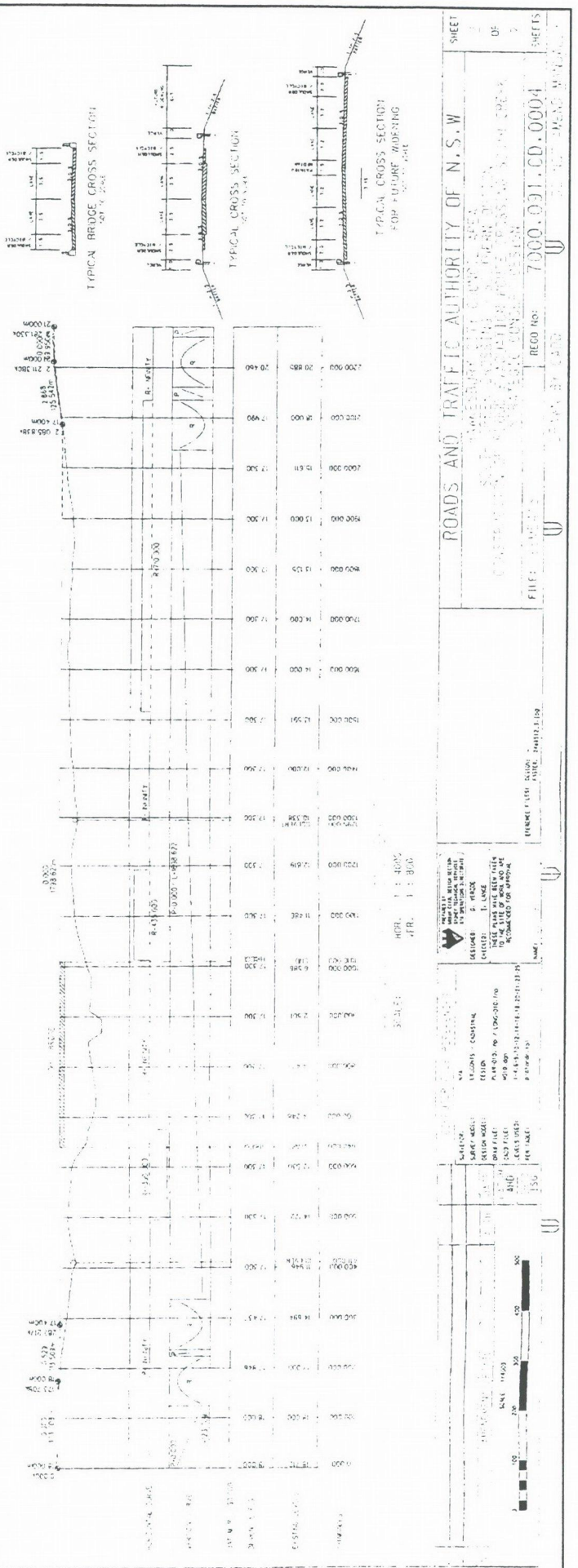

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EVACUATION ROUTE - THE SOUTHERN ROUTE

MHL Report 1085

Figure 1.2

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2. Extreme Flood Events and Corresponding Wind Data

2.1 Flood Data

All dates and durations of floods over 6 m AHD for Windsor area from 1949 to the present recorded on the MHL database are presented in Figures A1, A2 and A3 in Appendix A. The time series for the three highest recorded flood events (the 1978 event which recorded a flood level of 14.5 m AHD was not recorded on the MHL database) and jointly occurring wind events at Windsor and Sydney are shown in Figures 2.1, 2.2 and 2.3. Two of the three highest wind events (classified by the peak value for wind) occurred during the highest flood events. The highest flood levels and the date and time of these events are shown in Table 2.1.

Table 2.1 Highest Flood Levels Measured in the Windsor Richmond Area (1949-Present)

Event ID	Peak m (AHD)	Date	Time
14	14.72	13 June 1964	03:00
28	13.46	03 Aug 1990	15:00
8	13.41	11 Feb 1956	16:00

2.2 Wind Data

The wind data was obtained from the Bureau of Meteorology in the following formats for all thirty flood events recorded on the MHL database:

- 3-hourly wind direction and speed (10-minute averaged) for Richmond and Sydney.

Some of the more recent events in Sydney and Richmond have been recorded on an hourly basis.

2.3 The Coincidence of Peak Water Level and Extreme Wind During Flood Events

Table 2.2(a) indicates the highest values for wind (averaged over 10 minutes) during each of the three highest flood events. The table also includes two wind events where high values were recorded. The wind speed of 71 km/hr from the south recorded in Sydney is approximately equivalent to a 1-in-3-year wind according to return periods published for Sydney (Appendix B). The highest recorded wind on the Sydney database was 80 km/hr and on the Richmond data base was 42 km/hr. The return periods for the maximum measured Sydney value is approximately 1-in-9 years. Higher wind speeds may have been recorded at Sydney and Richmond when flood levels were below 6 m AHD.

Table 2.2(a) Highest Wind Values Measured During Each High Flood Event

Event ID	Maximum Wind Velocity During Event - Sydney		Maximum Wind Velocity During Event - Richmond	
	Wind Speed (km/hr)	Wind Direction (degrees)	Wind Speed (km/hr)	Wind Direction (degrees)
14	71	180	37	202.5 (SSW)
28	55	180	37	90 (E)
8	59	135	33	202.5 (SSW)
13	78	180	42	202.5 (SSW)
19	80	180	-	-

Table 2.2 (b) indicates that during the peak of each flood the wind values had died down significantly.

Table 2.2(b) Wind Values Measured During the Peak of Each Extreme Flood Event

Event ID	Wind Velocity During Peak of Event - Sydney		Wind Velocity During Peak of Event - Richmond	
	Wind Speed (km/hr)	Wind Direction (degrees)	Wind Speed (km/hr)	Wind Direction (degrees)
14	9	112.5	11	157.5
28	37	180	0	0
8	18	180	9	180

Table 2.2(c) indicates the flood levels measured at the time the wind reached its peak. The flood levels are significantly below the peak

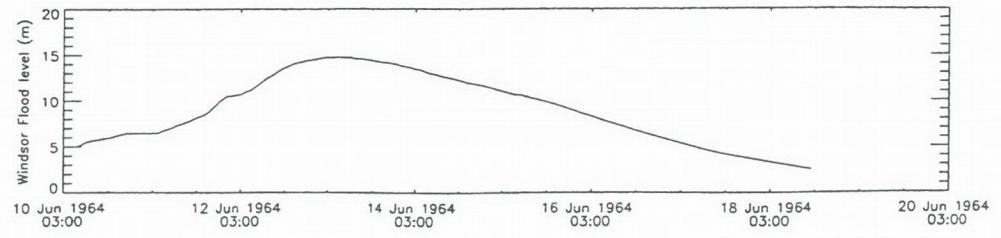
Table 2.2(c) Flood Levels Measured at Peak Wind at Richmond and Sydney for Each Extreme Flood Event

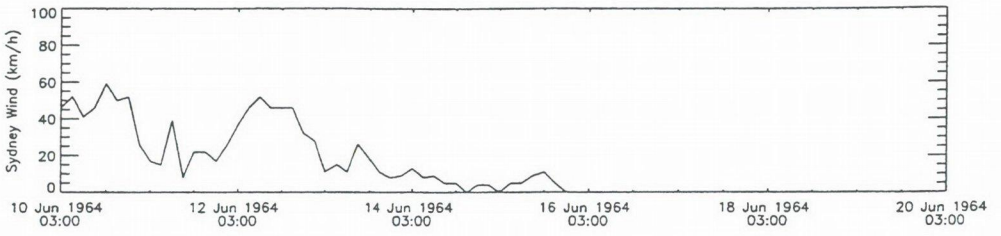
Event ID	Flood Level During Maximum Wind Velocity For Event - Sydney (m AHD)	Flood Level During Maximum Wind Velocity For Event - Windsor (m AHD)
14	13.3	13.4
28	12.2	9.7
8	5.9	12.1

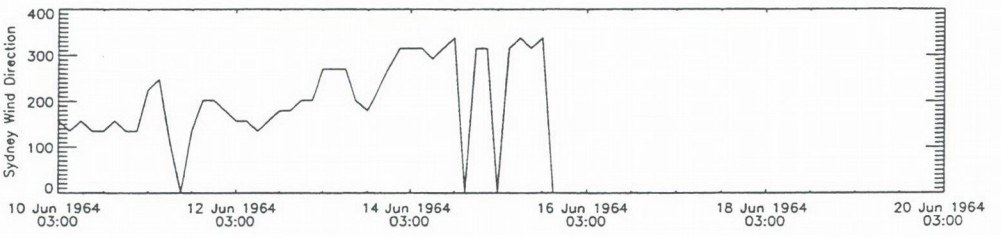
2.4 Correlation Between Sydney Wind Data and Richmond Wind Data During Extreme Flood Events

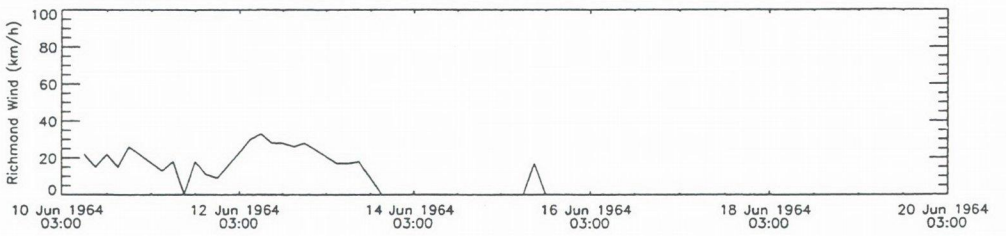
Simultaneously measured wind data exist for Sydney and Richmond since 1954. Although these two locations are significantly different it was thought it worthwhile to correlate these measurements. In order to establish a relationship between wind data measured in Sydney and Richmond a regression analysis was carried out on winds measured during high flood (>6 m). In a previous estimate of wind waves (MHL 2000) return periods for Sydney winds were used to hindcast wave climate. The regression analysis (Appendix B) developed a model to predict the wind velocity measured in Richmond given the value of wind in Sydney.

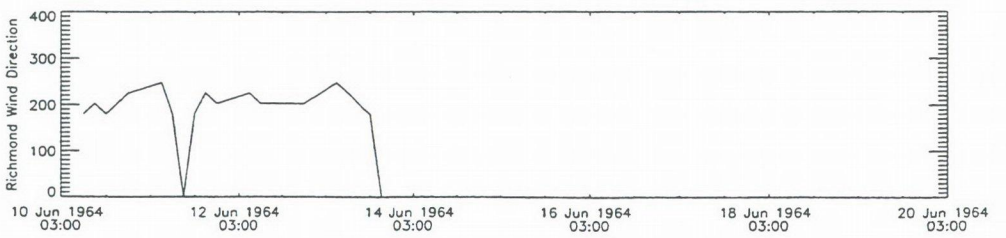
The regression analysis (Appendix B) was used to estimate the maximum wind speed in Richmond when the maximum wind speed in Sydney is 71 km/hr. The wind speed in Richmond was estimated to be 22 km/hr which is lower than the actual measured value of 37 km/hr. This suggests that the regression relationship may be biased by the noise introduced when low values of wind are recorded











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TIME SERIES OF FLOOD LEVEL (MAXIMUM 14.72m AHD) AND WIND DATA (MAXIMUM 37 km/hr) FOR EVENT 14

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Figure 2.1

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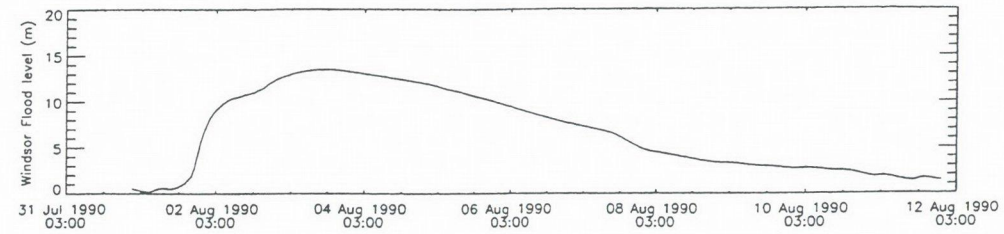

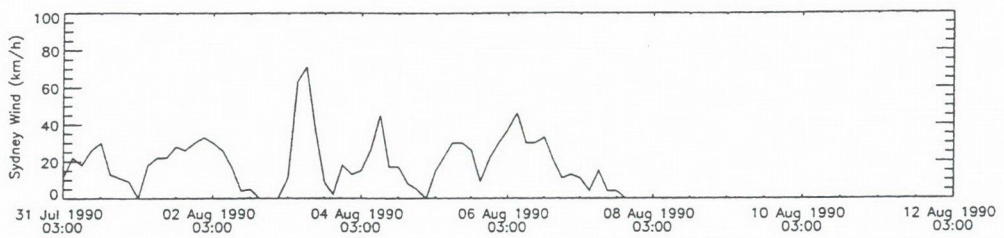

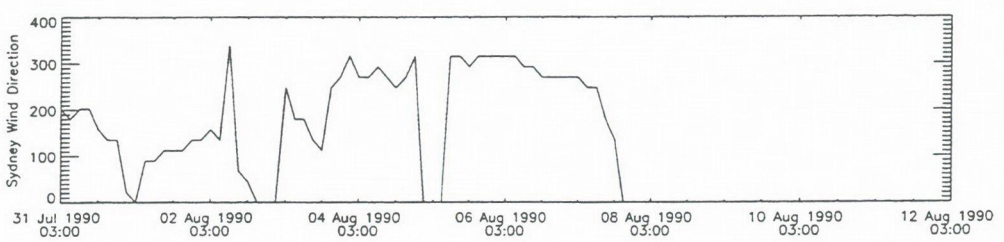

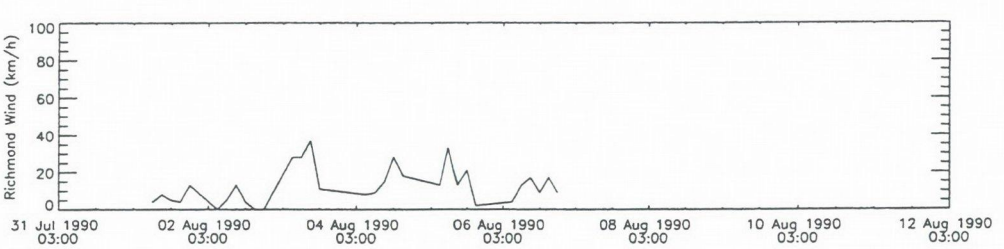

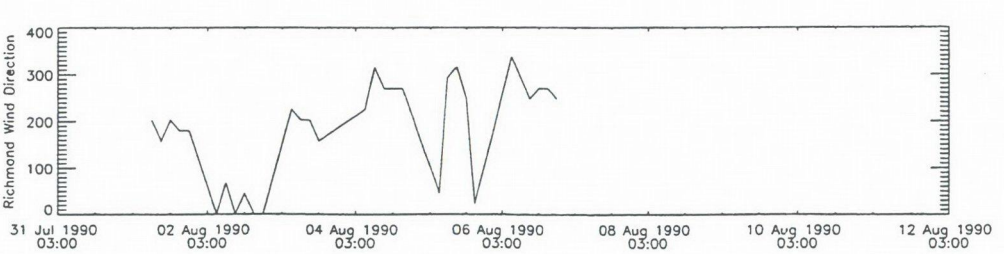

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TIME SERIES OF FLOOD LEVEL (MAXIMUM 13.46m AHD) AND WIND DATA (MAXIMUM 37 km/hr) FOR EVENT 28

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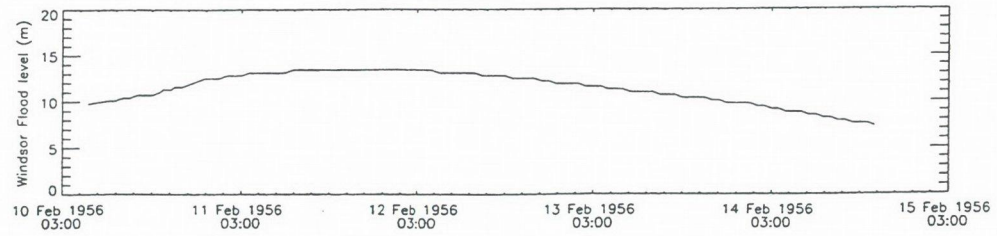
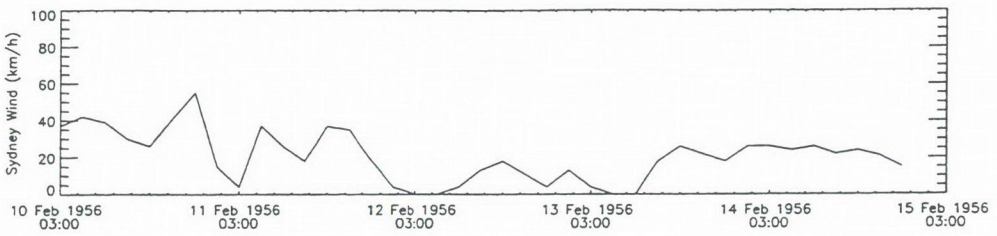
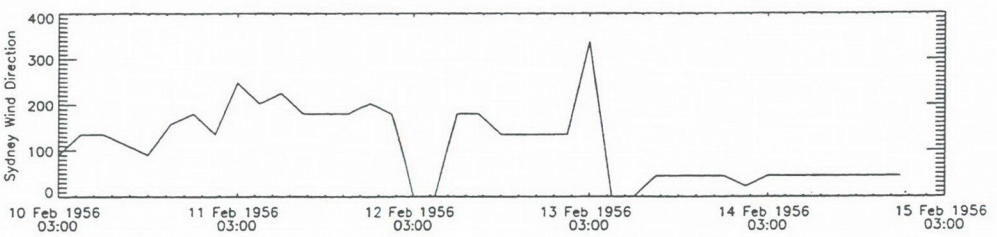
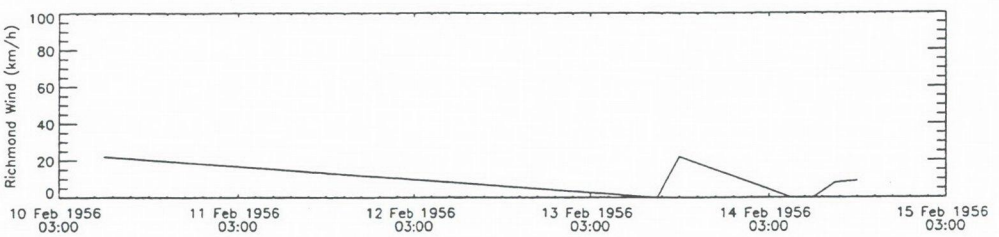
Figure 2.2

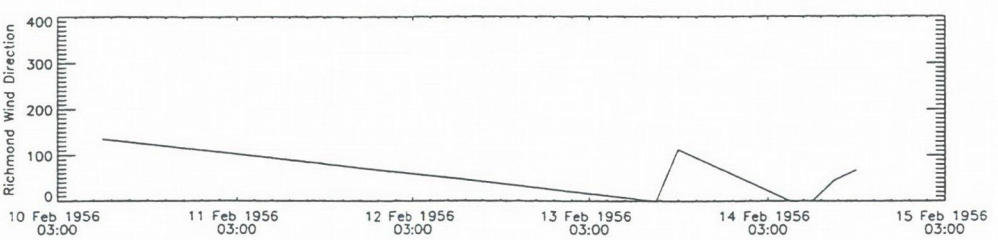
DRAWING 1085w22.CDR











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TIME SERIES OF FLOOD LEVEL (MAXIMUM 13.41m AHD) AND WIND DATA (MAXIMUM 33 km/hr) FOR EVENT 8

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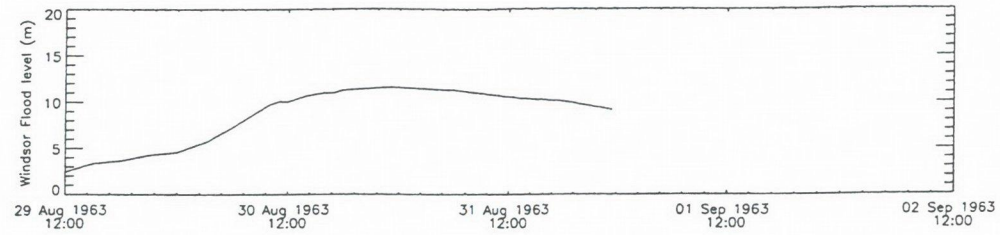
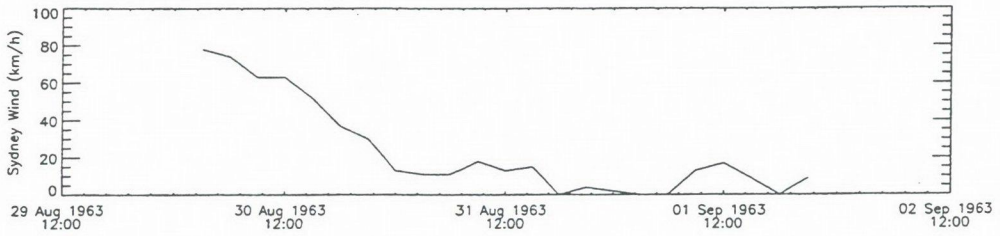
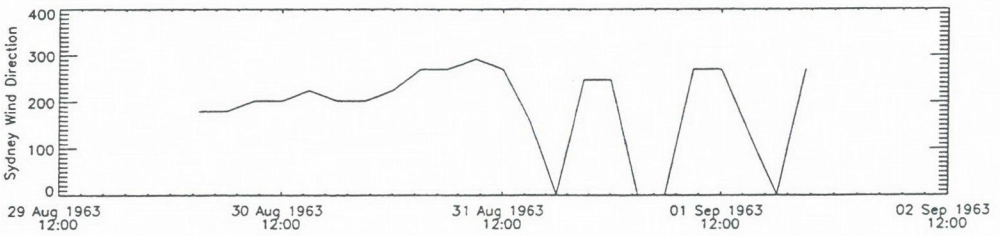
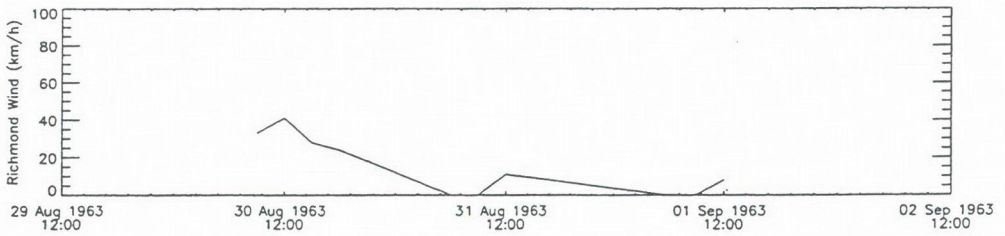
Figure 2.3

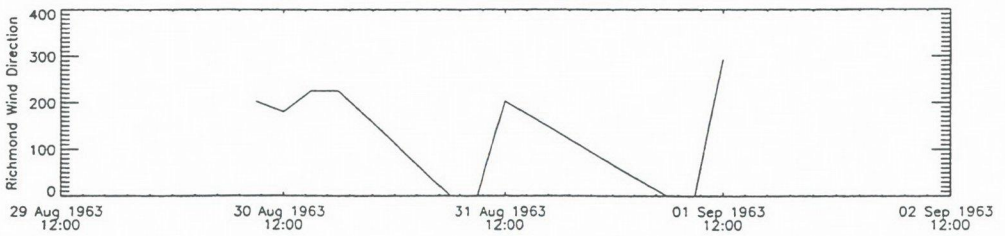
DRAWING 1085w23.CDR











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TIME SERIES OF FLOOD LEVEL (MAXIMUM 11.63m AHD) AND WIND DATA (MAXIMUM 42 km/hr) FOR EVENT 13

MHL Report 1085

Figure 2.4

DRAWING 1085w24.CDR

3. Wave Hindcasting Using Measured Extreme Wind on Extreme Water Levels

3.1 General

There is no generalised theoretical development for determining the actual growth of waves generated by winds blowing over relatively shallow and confined water bodies. The shallow water forecasting curves provided in the Shore Protection Manual (CERC 1984) represent an interim method for wave forecasting in shallow water.

A computer model employing the method for wave hindcasting presented in the Shore Protection Manual (CERC 1984) was developed by Manly Hydraulics Laboratory. This model has been applied to the Windsor area to calculate the locally-generated wind waves at designated locations on the flood evacuation route (MHL 2000).

3.2 Model Calibration

In order to verify the accuracy of the model, winds and waves recorded simultaneously (during four significant wind events) at Lake Victoria were used to compare recorded wave height with estimated wave height. Lake Victoria covers an area and has a depth of the same order as the study area at South Creek. This comparison is presented in Appendix C (Figures C1 to C4). The four events are in chronological order. The results indicate that:

- the measured wave height for a large time period of the event is smaller than the hindcast wave height. The comparison would be accurate only if approximately half the wind fetch were used.

- No time history of lake levels is available. The assumed fetches were for a lake operating at maximum depth. The fetches could reduce dramatically if the lake levels fell by a few metres.

This discrepancy between measured and calculated wave heights during the four events may be attributed to the waves being attenuated at Lake Victoria by the large number of dead trees close to the surface of the lake. Spectral analysis was not used in these wave height measurements and calculations. When relatively small wave heights (<0.3 m) are measured the noise in the signal can give rise to significant error.

3.3 Estimate of Wave Heights Generated During Coincidence of the Peak Wind and the Peak of Measured High Floods

Waves were hindcast for winds that were measured during the 14.7 m flood (Table 2.2a). Fetch data was obtained for the length of the route from the 15 m and 17.3 m flood contours marked on the 1:10,000 maps provided by the Roads and Traffic Authority (RTA). The fetch was estimated for 22.5 degree segments. To generate wave characteristics a conservative approach utilising waves generated by maximum winds during the event was adopted. This was compared with waves generated by winds measured at peak time of peak flood (Table 3.2).

Table 3.1 Highest Hindcast Wave Height During Peak of Highest Floods for Winds Measured at Windsor

Event ID	Peak m AHD	Date	Wave Height and Period H_s (m) and T_p(s)		Time
14	14.72	13 June 1964	0.34	2.1	03:00
28	13.46	03 Aug 1990	0.39	2.2	15:00
8	13.41	11 Feb 1956	0.39	2.2	16:00

Table 3.2 presents variation of hindcast wave height with distance along the northern route for the 14.7 m AHD flood and the measured maximum wind speed during that particular flood event.

Table 3.2 Variation of Wave Height with Distance Along the Northern Route for 14.7 m Flood (Event 14) and Maximum Measured Wind

Length Along Route	Hindcast Wave Height For Wind Measured at Peak of Flood Level (11 km/hr)		Hindcast Wave Height For Maximum Measured Wind During Flood Event (37 km/hr)	
	H_s (m)	T_p (s)	H_s (m)	T_p (s)
0 m (Windsor)	0.08 (SSW)	0.8	0.09 (SSW)	0.8
500 m	0.09 (SSW)	0.9	0.10 (SSW)	0.9
1,000 m	0.30 (SSW)	1.9	0.34 (SSW)	2.0
1,500 m	0.30 (SSW)	1.9	0.34 (SSW)	2.0
2,000 m	0.34 (SSW)	2.1	0.39 (SSW)	2.2
2,500 m (Mulgrave)	-	-	-	-

Wave heights were hindcast for the three highest wind events measured on the database. This technique can be considered conservative. The wave heights for the three events are presented in Table 3.3. The hindcast wave height values in Tables 3.2 and 3.3 are less than 50% of the wave heights hindcast using the Sydney 1-in-1-year return period winds. However, in reality the local wind return period could likely be greater than 1-in-1 year but there are not sufficient data to derive the local occurrence statistics.

Table 3.3 Variation of Wave Height with Distance Along the Northern Route for 17.3 m Flood for Maximum Measured Wind Events

Length Along Route	Hindcast Wave Height For Highest Wind Measured at Peak of Flood Level (Event 13) (42 km/hr)		Hindcast Wave Height For Highest Wind Measured at Peak of Flood Level (Events 28 and 14) (37 km/hr)	
	H_s (m)	T_p (s)	H_s (m)	T_p (s)
0 m (Windsor)	0.18 (SSW)	1.3	0.16 (SSW)	1.2
500 m	0.18 (SSW)	1.3	0.16 (SSW)	1.2
1,000 m	0.45 (SSW)	2.3	0.38 (SSW)	2.1
1,500 m	0.45 (SSW)	2.3	0.38 (SSW)	2.1
2,000 m	0.45 (SSW)	2.3	0.38 (SSW)	2.1
2,500 m (Mulgrave)	-	-	-	-

4. Runup and Overtopping for Combinations of Measured Flood Level and Wind Data

4.1 Runup Calculations

When wind waves reach the shoreline they expend their remaining energy as wave runup. The level of wave runup is affected by the angle of wave attack, the foreshore profile and type of foreshore structures. Wave runup is a factor affecting the window of time available for continued use of the evacuation route at South Creek. When overtopping occurs (however minor), any overtopping may be unacceptable to the wider community and/or the State Emergency Service (SES). The SES perceives that next to no overtopping could be tolerated. Runup estimates were calculated for waves approaching normal to an embankment (maximum runup) as well as waves approaching at an angle. The following equation is used to predict runup on grassy dykes, smooth slopes and rubble mound breakwaters (Van der Meer 1994):

$$Ru_{2\%}/H_s = 1.5\ \gamma_f\ \gamma_s\ \gamma_\beta\ \xi \text{ (with a maximum of } 3.0\ \gamma_f\ \gamma_s\ \gamma_\beta) \quad (3)$$

where:

$R_{u2\%}$= 2% runup level

H_s = Design significant wave height-(average of the highest 1/3 of the waves)

γ_f = Roughness factor

γ_s = Shallow water factor

γ_β = Oblique wave attack factor

ξ = Surf similarity parameter = $\tan\alpha/(H_s/L_o)^{1/2}$

α = Slope

L_o = Deep water wave length.

The three gamma (γ) factors vary for a range of conditions. The factor for roughness (γ_f) for two-layered rock and grass (3 cm) is estimated to be in the range 0.50-0.55 and 0.9-1.0 respectively. The influence of shallow water effectively reduces wave runup and varies in the range 0.88-1.0. The factor for oblique wave attack (γ_β) is given as 0.64, 0.86 and 1.0 for the angles 60°, 30° and 0°.

4.2 Runup During a 14.7 m Flood for Waves Generated During Highest Measured Winds During the Event

Wave runup was estimated for a 14.7 m AHD flood level along the total length of the evacuation route at 500 m intervals. Runup was estimated on a 1:2.5 slope as proposed in the evacuation route design as provided. The preliminary runup calculations are tabulated in Table 4.1 and show that for the hypothetical case when the maximum wind (37 km/hr) and the peak of the flood occur simultaneously the runup levels are estimated to reach 15.7 m. This level is below the proposed road level (17.3 m AHD) and hence would not be expected to cause disruption to the flood evacuation process.

Table 4.1 Predicted Runup Level Estimated for a 14.7 m Flood and Highest Measured Wind During the Event (37 km/hr) for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	0.09 (SSW)	0.8	14.9	14.9
500 m	0.10 (SSW)	0.9	14.9	14.9
1,000 m	0.34 (SSW)	2.0	15.5	15.4
1,500 m	0.34 (SSW)	2.0	15.5	15.4
2,000 m	0.39 (SSW)	2.2	15.7	15.6
2,500 m (Mulgrave)	-	-	-	-

4.3 Runup During a 17.3 m Flood For Waves Generated During the Highest Measured Wind Event

The preliminary runup calculations are tabulated in Table 4.2 and estimate runup for the hypothetical case when the highest winds for all the events (42 km/hr) and the peak of the flood occur simultaneously. Whenever runup levels are higher than the bridge level overtopping will occur. The runup levels are estimated to reach 18.4 m on a 1:2.5 smooth slope. This level is above the originally proposed road level of 17.3 m AHD. Hence overtopping estimates were produced (Table 4.5) to evaluate the safety of the evacuation route. If the crest level of the evacuation route is raised to a level of 18.4 m AHD (grass slope to 17.3 m AHD and 1.3 m high vertical structure) it is unlikely that there would be appreciable overtopping. A vertical structure greater than 1.3 m in depth which includes an appropriately designed wave deflector with a crest level of 18.4 m AHD may also mitigate overtopping to a similar extent. However there is a low probability that a freak random wave or a wave group would lead to some overtopping of either design scenario. The degree to which this could occur (if at all) can only be estimated through physical model analysis of the proposed structure.

Table 4.2 Predicted Runup Level Estimated for a 17.3 m Flood and Waves Generated by Highest Measured Wind (42 km/hr) for the Northern Route

Length Along Route	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	0.18 (SSW)	1.3	17.7	17.7
500 m	0.18 (SSW)	1.3	17.7	17.7
1,000 m	0.45 (SSW)	2.3	18.4	18.3
1,500 m	0.45 (SSW)	2.3	18.4	18.3
2,000 m	0.45 (SSW)	2.3	18.4	18.3
2,500 m (Mulgrave)	-	-	-	-

4.4 Comparison of Runup for a 1-in-1-Year Wave (Using Sydney Winds) and Waves Hindcast Using Measured Richmond Wind

Tables 4.3 and 4.4 show that runup levels for measured maximum winds during a 15 m and 17.3 m flood are 1.0 to 1.5 m less than when a 1-in-1-year Sydney wind is used.

Table 4.3 Comparison of Runup Level Calculations for a 15 m Flood and Hindcast 1-in-1-Year Wave with Wave Hindcast for Measured Maximum Wind Values

Distance Along Measured Route (m)	Runup Due to 1-in-1-Year Wave on 15 m Flood (MHL 2000)		Runup Due to Maximum Measured Flood (14.7 m) and Maximum Wind (37 km/hr)	
	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	16.6	16.2	14.9	14.9
500 m	16.7	16.3	14.9	14.9
1,000 m	17.2	16.7	15.5	15.4
1,500 m	17.1	16.6	15.5	15.4
2,000 m	17.1	16.6	15.7	15.6
2,500 m (Mulgrave)	-	-	-	-

Table 4.4 Comparison of Runup Level Calculations for a 17.3 m Flood and Waves Hindcast from Maximum Wind

Distance Along Measured Route (m)	Runup Due to 1-in-1-Year Wave (Sydney) on 17.3 m Flood		Runup Due to 17.3 m Flood and Maximum Wind (37 km/hr)	
	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	18.7	18.4	17.7	17.7
500 m	19.8	19.2	17.7	17.7
1,000 m	19.7	19.2	18.4	18.3
1,500 m	19.6	19.1	18.4	18.3
2,000 m	19.6	19.1	18.4	18.3
2,500 m (Mulgrave)	-	-	-	-

4.5 Overtopping Calculations

4.5.1 A Brief Introduction

As stated in Section 4.1 overtopping will not be acceptable to the wider community. However, for completeness overtopping scenarios have been estimated in Table 4.5. Often the height of a structure that is overtopped by waves has to be optimised so as to permit overtopping levels that are acceptable. Prototype measurements of wave overtopping are scarce. One of the most extensive sets of data is available from Fukuda (1974) who presents results of overtopping measurements carried out in Japan.

Wave overtopping is generally measured as a discharge per unit metre of structure ($m^3/m/s$ or litres/m/s). In this study the overtopping estimates used the wave height estimates for the 14.7 m flood (highest measured) and the 17.3 m flood (1-in-100 year) to indicate the severity of overtopping that may be experienced.

Wave overtopping can be expressed by two different formulae (Van der Meer 1994): one for breaking waves ($\xi < 2$) and another for non-breaking waves ($\xi > 2$) where ξ is the surf similarity parameter (Section 4.1). Only breaking waves of low surf similarity ($\xi < 2$) are generated in this study. Therefore only the formula for breaking waves was utilised. The dimensionless crest height R_b (application area $0.3 < R_b < 2$) and the overtopping quantity q are given by:

$$R_b = R_c \sqrt{S_{op}} / H_s \tan\alpha \gamma_f \gamma_s \gamma_\beta \quad (4)$$

$$q = Q_b \sqrt{g H_s^3 \tan\alpha} / \sqrt{S_{op}} \quad (5)$$

where:

$$Q_b = 0.06 \exp(-5.2 R_b) \quad (6)$$

and

R_b = dimensionless crest height above still water level

R_c = free crest height above still water line

q = average overtopping discharge

Q_b = dimensionless discharge for breaking waves

S_{op} = wave steepness ($H_s/1.56T_p^2$)

α = structure slope

To test the sensitivity of design to overtopping, a grass slope (1:2.5) to 17.3 m AHD with a vertical structure (up to 17.7 m AHD for 0 m to 500 m and 18.4 m AHD for 500 m to 2,000 m along the structure) was analysed. The overtopping figures are indicated in Table 4.5. Other factors which are not included in equations 4-6 that influence overtopping rates are:

- wind: the action of wind acting directly on the waves
- wave grouping: the action of a high single wave or wave group could increase average overtopping rates by a factor of several hundred
- wave deflectors: have to be physically modelled to predict the effectiveness of these devices.

These factors were not taken into consideration when the estimates for overtopping in Table 4.5 were calculated. The influences of the above factors are discussed in greater detail in an earlier report (MHL 2000).

4.5.2 Calculation of Overtopping Rates

Table 4.5 Estimates of Overtopping Height and Discharge for Waves Generated by the Highest Measured Wind (42 km/hr) for the Northern Route

Distance Along Length of Route	Significant Wave Height and Period at the Northern Option (Direction)		Average Overtopping Rates for a Grass Slope at 30° with a Vertical Structure	
	H_s (m)	T_p (s)	R_b	q (L/m/s)
0 m (Windsor)	0.18 (SSW)	1.3	2.0	.0018
500 m	0.18 (SSW)	1.3	2.0	.0018
1,000 m	0.45 (SSW)	2.3	1.95	.04
1,500 m	0.45 (SSW)	2.3	1.95	.04
2,000 m	0.45 (SSW)	2.3	1.95	.04
2,500 m (Mulgrave)	-	-	-	-

4.6 Acceptable Overtopping Rates for Pedestrians and Vehicles

Criteria for acceptable overtopping rates and spray intensities have been developed by many institutions (Jensen 1987). For South Creek, as discussed in Section 4.1, any overtopping would be unacceptable. The Dutch guidelines on river dykes (Van der Meer 1994) indicate that for relatively heavy seas and with wave heights of up to a few metres the 2% runup criterion yields an overtopping discharge of the order of 1 L/s/m. The same guideline further quotes 'In certain cases such as covered crests and slopes, sometimes 10 L/s/m can be tolerated'.

In the Dutch guidelines it is assumed that the following overtopping rates are allowable:

- 0.1 L/s /m for sandy soil with a poor turf
- 1.0 L/s /m for clayey soil with relatively good grass
- 10 L/s/m with a clay protective layer and grass according to the standards for an outer slope or with a revetment construction.

Table 4.5 suggests that rates of overtopping well below the least stringent Dutch criteria would be considered 'dangerous' for a pedestrian 3 m behind the crest of a 'breakwater' for the Japanese criteria (1974).

5. Conclusions and Recommendations

A previous investigation (MHL 2000) into inundation levels on the proposed flood evacuation route out of Windsor recommended that a more detailed investigation be made into the joint occurrence of high (>6 m AHD) flood levels and high winds. This subsequent investigation identified 30 high floods that had occurred in the Windsor area since 1949. Collation of data from the three highest floods and the associated high winds that occurred simultaneously with these floods indicate that:

- The highest flood level was measured at 14.7 m AHD and was classified (Webb McKeown 1997) between a 1-in-50-year (15.7 m AHD) and a 1-in-20-year (13.7 m AHD) event.

- The highest flood level and the highest winds during the flood event did not coincide for these high flood level events.

In order to utilise the large database of Sydney winds a regression analysis was carried out between Sydney winds and Richmond winds that were recorded during the period of a high flood. The regression analysis (Appendix B) suggests that the measured highest wind values for Richmond were higher than those predicted by the regression relationship for Windsor. This may be attributed to the bias due to the scatter of the data.

The wave heights obtained by using the CERC (1984) hindcasting formulae were compared with measured waves (for high wind events over a period of four months) at Lake Victoria in inland NSW where four months of simultaneous measurement of winds and waves were carried out. The hindcast waves were 30-40% higher than the measured values. The conclusions may be summarised for events C1 to C4 as follows:

- These comparisons indicate the measured wave height for a large time period of the event is smaller than the hindcast wave height. The comparison would be accurate only if approximately half the wind fetch were used.

- No time history of lake levels is available. The assumed fetches were for a lake operating at maximum depth. The fetches could reduce dramatically if the lake levels fell by a few metres.

- This discrepancy between measured and calculated wave heights during the four events may be partially attributed to the waves being attenuated at Lake Victoria by the large number of dead trees close to the surface of the lake.

- Spectral analysis was not used in these wave height measurements and calculations. When relatively small wave heights (<0.3 m) are measured the noise in the signal can give rise to significant error.

The conclusions may be summarised as follows:

- The runup levels for a 15 m flood are approximately 1 m to 1.5 m less for maximum measured Richmond wind data than for 1-in-1-year return period Sydney wind occurring during a flood.

- The runup levels for a 17.3 m flood are approximately 1 m to 1.5 m less for maximum measured Richmond wind data than for 1-in-1-year return period Sydney wind occurring during a flood.

- The influence of direct wind action on runup and overtopping was not included in these calculations. Wind may significantly increase overtopping and sprays. Physical modelling may assist in establishing the influence of wind.

- Previous physical modelling results suggest that during a flood wave, deflecting structures will prove useful in reducing overtopping by a factor of about 2 and hence increase the usage time for the evacuation route.

- The proposed crest level of 18.6 m AHD for the flood barriers should not lead to significant overtopping during a 1-in-100-year flood of 17.3 m AHD.

- Specially designed wave deflectors on top of a vertical barrier or sloping embankment should reduce the chance of any overtopping in a 17.3 m or smaller flood.

The following recommendations result from this study:

- Continue collection and collation of Sydney flood level data with Sydney and Richmond wind data in order to establish a better understanding of the joint occurrence of winds and floods at Richmond.

- The installation of wave barriers and deflectors would reduce overtopping and reduce design levels for the evacuation route. It is recommended that a physical model would assist in determining site specific overtopping and the influence of wind and wave grouping on overtopping due to waves for any proposed design scenarios. The site specific nature of the problem including wave grouping, the shape of the waves and the bathymetry in front of the structure make it amenable to a physical model investigation.

6. References

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Fukuda, N., Uno, T., and Irie, I. 1974, *Field Observations of Wave Overtopping of Wave Absorbing Revetment,* Coastal Engineering (Japan), Volume 17, 1974, pp 117-1286.

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Webb McKeown & Associates Pty Ltd 1997, *Warragamba Dam Spillway Environmental Impact Flood Study.*

Appendix A

Flood Dates and Levels of High (>6 m AHD) Floods Measured at Windsor

DEPARTMENT OF PUBLIC WORKS AND SERVICES, MANLY HYDRAULICS LABORATORY
110B KING STREET, MANLY VALE N.S.W. 2093, AUSTRALIA. TELEPHONE 02-9949-0200

FLOOD LEVEL STATISTICS

STATION NAME	:	WINDSOR BRIDGE
RIVER/STREAM/CREEK	:	HAWKESBURY RIVER
BASIN	:	HAWKESBURY
A.W.R.C. No.	:	212403
DATA START	:	17-JUN-1949
DATA FINISH	:	25-JAN-1976
TOTAL YEARS OF DATA	:	15
FLOOD THRESHOLD (METRES)	:	6.40
DATUM	:	STANDARD DATUM
NOMINATED START	:	17-JUN-1949
NOMINATED FINISH	:	25-JAN-1976
DATE OF ISSUE	:	21-JUN-2000 12:14
ANALYSIS PERFORMED BY	:	DG using INTERACTIVE program HISFLDMNU V3.0
COMMENTS	:	* after level indicates gauge reading only,
	:	levels not reduced to specified datum.
	:	All times are observer watch times.

EVENT ID	RANK	PEAK (m)	DATE	TIME	START	DURATION (days)	MAX RISE (cm/hr)	PERIOD (yrs)
1	3	12.13	19-JUN-1949	21:30	17-JUN-1949	4.85	18.8	5.3
2	12	9.36	31-MAR-1950	02:00	30-MAR-1950	9.17	45.5	1.3
3	8	9.69	17-JUN-1950	24:00	16-JUN-1950	1.71	2.5	2.0
4	15	9.14	21-JAN-1951	02:00	21-JAN-1951	0.33	N/A	1.1
4	21	7.33	22-JUN-1951	22:00	22-JUN-1951	N/A	N/A	0.8
5	11	9.52	17-JUN-1952	15:00	15-JUN-1952	2.83	27.4	1.5
6	4	11.78	28-JUL-1952	09:00	27-JUL-1952	2.96	91.0	4.0
7	14	9.19	06-AUG-1952	12:00	05-AUG-1952	1.04	2.9	1.1
8	2	13.69*	11-FEB-1956	16:00	10-FEB-1956	4.48	134.0	8.0
10	16	8.70	30-APR-1963	10:00	29-APR-1963	1.90	16.2	1.0
11	18	8.08	08-MAY-1963	16:00	08-MAY-1963	1.79	15.2	0.9
13	9	9.58	30-AUG-1963	23:00	30-AUG-1963	2.31	61.0	1.8
14	1	14.72	13-JUN-1964	03:00	10-JUN-1964	5.88	40.0	16.0
15	7	10.27	15-NOV-1969	13:00	14-NOV-1969	3.88	35.0	2.3
16	22	6.73	12-JAN-1974	21:30	12-JAN-1974	0.15	4.0	0.7
17	20	7.37	12-MAR-1974	17:30	12-MAR-1974	1.00	26.0	0.8
18	17	8.66	22-APR-1974	08:00	21-APR-1974	2.04	31.0	0.9
19	6	10.42	28-MAY-1974	08:00	26-MAY-1974	4.02	60.0	2.7
20	19	7.95	06-JUN-1974	10:00	04-JUN-1974	2.55	30.0	0.8
21	10	9.53	29-AUG-1974	16:45	29-AUG-1974	1.53	26.0	1.6
22	5	11.23	22-JUN-1975	22:59	21-JUN-1975	3.60	54.3	3.2
23	13	9.35	24-JAN-1976	20:00	23-JAN-1976	1.38	24.0	1.2

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110B KING STREET, MANLY VALE N.S.W. 2093, AUSTRALIA. TELEPHONE 02-9949-0200

FLOOD LEVEL STATISTICS

STATION NAME	:	NORTH RICHMOND BRIDGE
RIVER/STREAM/CREEK	:	HAWKESBURY RIVER
BASIN	:	HAWKESBURY
A.W.R.C. No.	:	212902
DATA START	:	18-FEB-1959
DATA FINISH	:	01-SEP-1963
TOTAL YEARS OF DATA	:	3
FLOOD THRESHOLD (METRES)	:	6.40
DATUM	:	UNKNOWN
NOMINATED START	:	18-FEB-1959
NOMINATED FINISH	:	01-SEP-1963
DATE OF ISSUE	:	21-JUN-2000 14:58
ANALYSIS PERFORMED BY	:	DG using INTERACTIVE program HISFLDMNU V3.0
COMMENTS	:	* after level indicates gauge reading only,
	:	levels not reduced to specified datum.
	:	All times are observer watch times.

EVENT ID	RANK	PEAK (m)	DATE	TIME	START	DURATION (days)	MAX RISE (cm/hr)	PERIOD (yrs)
9	5	6.76*	19-FEB-1959	02:00	19-FEB-1959	0.06	3.3	0.8
10	2	10.31*	30-APR-1963	10:00	28-APR-1963	2.41	70.7	2.0
11	3	9.47*	08-MAY-1963	13:00	08-MAY-1963	1.08	17.0	1.3
12	4	6.96*	19-MAY-1963	16:50	19-MAY-1963	0.85	13.0	1.0
13	1	11.63*	30-AUG-1963	23:00	30-AUG-1963	2.00	77.0	4.0

DEPARTMENT OF PUBLIC WORKS AND SERVICES, MANLY HYDRAULICS LABORATORY
110B KING STREET, MANLY VALE N.S.W. 2093, AUSTRALIA. TELEPHONE 02-9949-0200

TIDAL LEVEL STATISTICS

STATION NAME	:	HAWKESBURY RIVER, WINDSOR
RECORDER TYPE	:	HYDROMACE 2000
AWRC NO.	:	21242
DATA START	:	01-JUL-1988
DATA FINISH	:	21-JUN-2000
TOTAL YEARS OF DATA	:	11.97
DATABASE TIME INTERVAL (min)	:	60
THRESHOLD LEVEL (m)	:	6.40
DATUM	:	AUSTRALIAN HEIGHT DATUM
DATE OF ISSUE	:	1202:21/06/2000
ANALYSIS PERFORMED BY	:	DG using INTERACTIVE program WDBRNKX V1.0
COMMENTS	:	Recorded levels.

EVENT ID	RANK	PEAK (m)	DATE	TIME	START	DURATION (hr)	MAX RISE (cm/hr)	PERIOD (yr)
24	2	10.96	07-JUL-1988	07:00	06-JUL-1988	82.0	60.0	6.5
25	5	8.28	05-APR-1989	12:00	05-APR-1989	18.0	N/A	2.6
26	6	7.69	04-FEB-1990	14:00	04-FEB-1990	29.0	29.0	2.2
27	4	8.72	21-APR-1990	02:00	20-APR-1990	65.0	27.0	3.2
28	1	13.46	03-AUG-1990	15:00	01-AUG-1990	134.0	80.0	13.0
29	7	7.18	12-JUN-1991	10:00	12-JUN-1991	26.0	17.0	1.9
30	3	10.82	10-FEB-1992	22:00	09-FEB-1992	27.0	44.0	4.3

Appendix B

Regression Analysis for Windsor and Sydney Winds During High (>6 m) Floods

Regression Analysis for Windsor and Sydney Winds During High (>6 m) Floods

The relationship between winds at Sydney and Windsor is estimated by:

$$Y_u = -3.55 + 0.305\ X_1 + 0.118\ X_2 \quad (1)$$

$$Y_v = -3.33 - 0.125\ X_1 + 0.413\ X_2 \quad (2)$$

where:

Y_u = Windsor – wind speed in an eastward direction

Y_v = Windsor – wind speed in a northward direction

X_1 = Sydney – wind speed in an eastward direction

X_2 = Sydney – wind speed in a northward direction

The above relationship was used to estimate the maximum wind speed in Richmond when the maximum wind speed in Sydney is 71 km/hr. The wind speed in Richmond was estimated to be 22 km/hr which is lower than the actual measured value of 37 km/hr. This suggests that the regression relationship may be biased by the noise introduced when low values of wind are recorded

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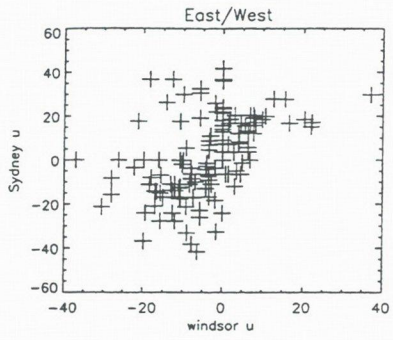

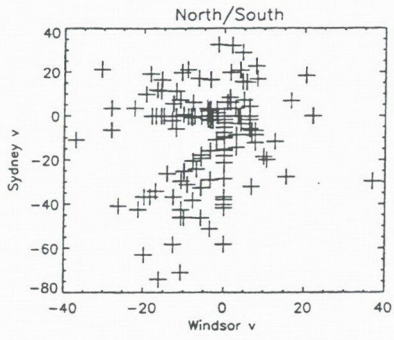

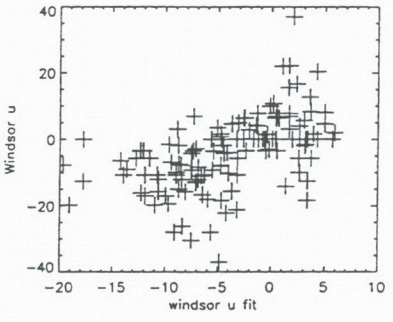

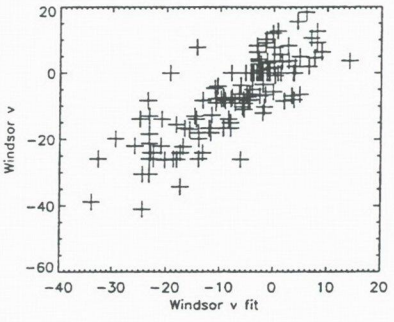

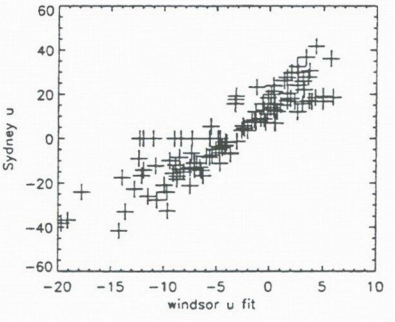

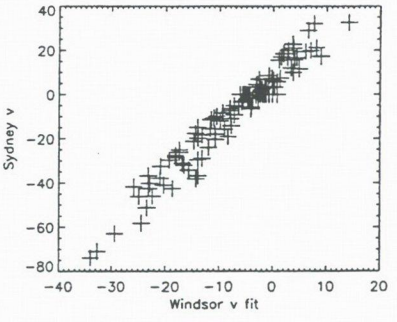

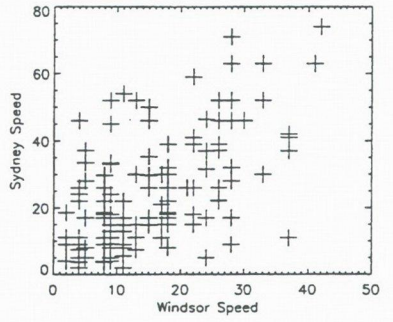

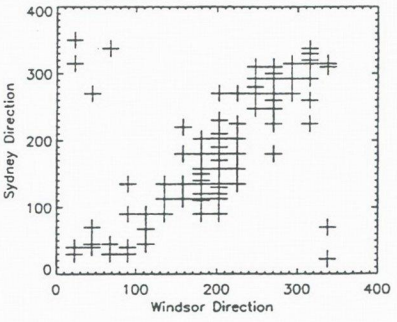

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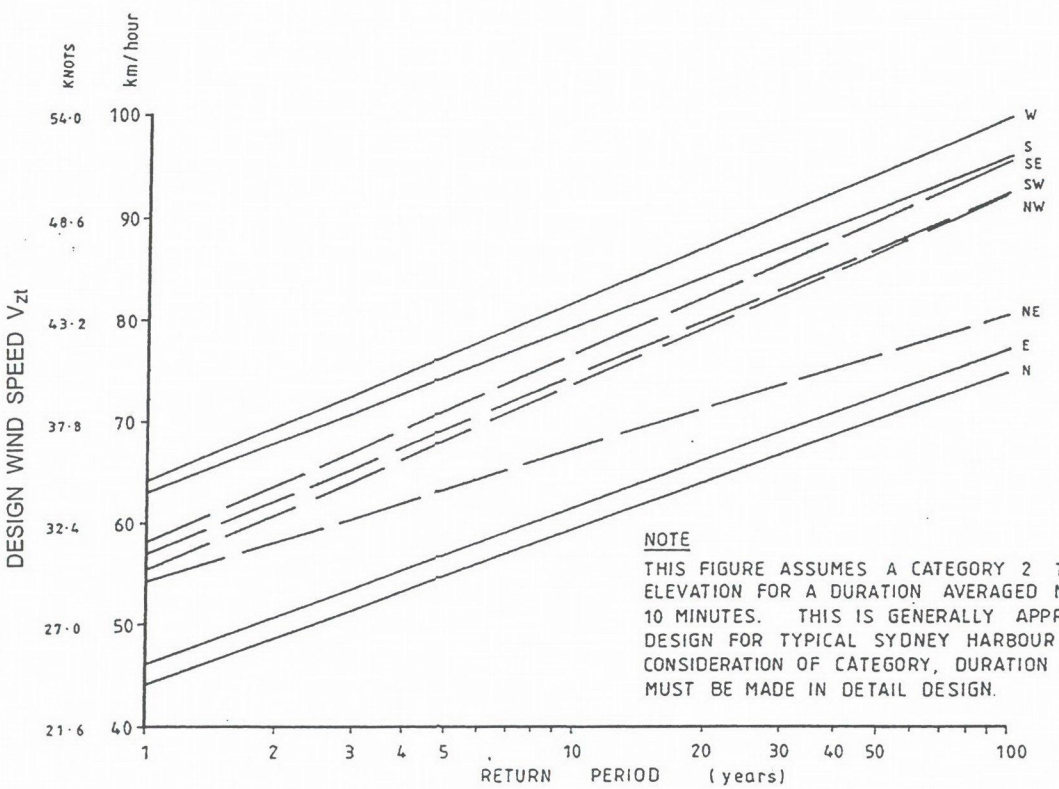
REGRESSION ANALYSIS FOR WINDSOR AND SYDNEY WINDS DURING HIGH (>6m) WINDSOR FLOODS

MHL Report 1085

Figure B1

DRAWING 1085WB1.CDR



NOTE

THIS FIGURE ASSUMES A CATEGORY 2 TERRAIN AT 10 m ELEVATION FOR A DURATION AVERAGED MAXIMUM GUST OF 10 MINUTES. THIS IS GENERALLY APPROPRIATE FOR PRELIMINARY DESIGN FOR TYPICAL SYDNEY HARBOUR FETCHES. FURTHER CONSIDERATION OF CATEGORY, DURATION AND LIMITING CONDITIONS MUST BE MADE IN DETAIL DESIGN.

PROBABILITY OF EXCEEDANCE OF DIRECTIONAL DAILY MAXIMUM DESIGN WIND SPEEDS FOR SYDNEY HARBOUR

Based on Sydney Harbour records 1926 to 1984

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DURATION AVERAGED MAXIMUM GUSTS OF 10 MINUTES (DIRECTIONAL) RETURN PERIOD DATA

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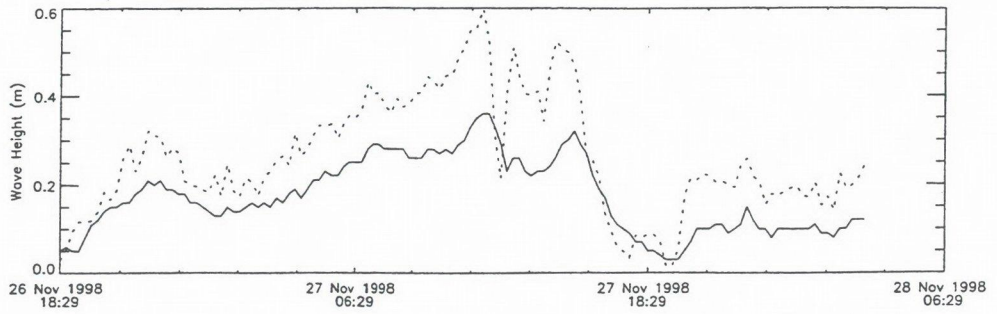
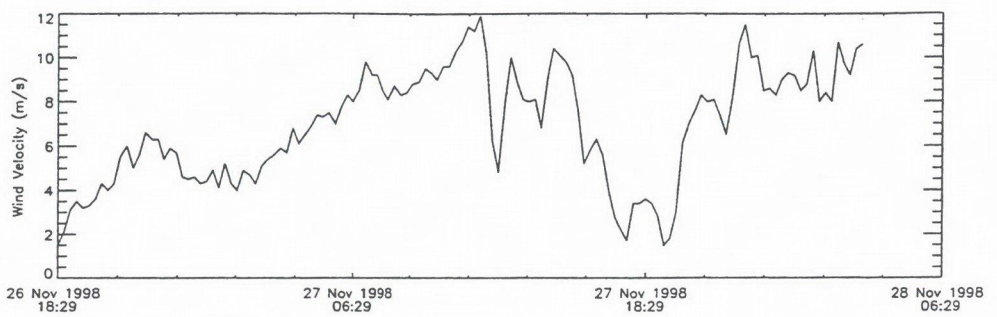
Figure B2

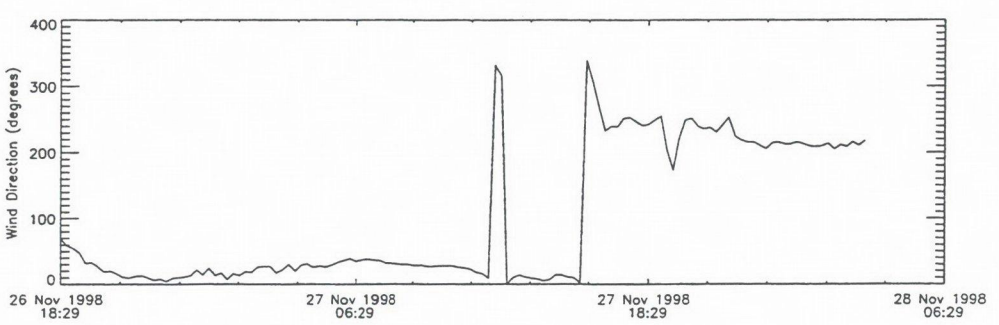
DRAWING 1085WB2.CDR

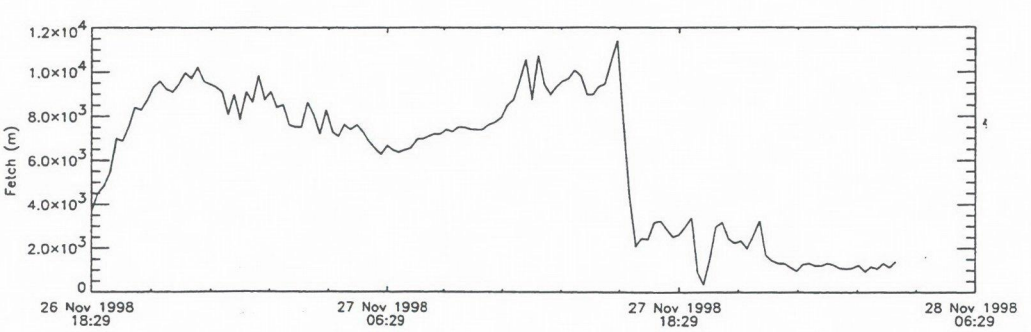
Appendix C

Comparison of Hindcast and Measured Waves at Lake Victoria









LEGEND

..... Hindcast waves

——— Measured waves

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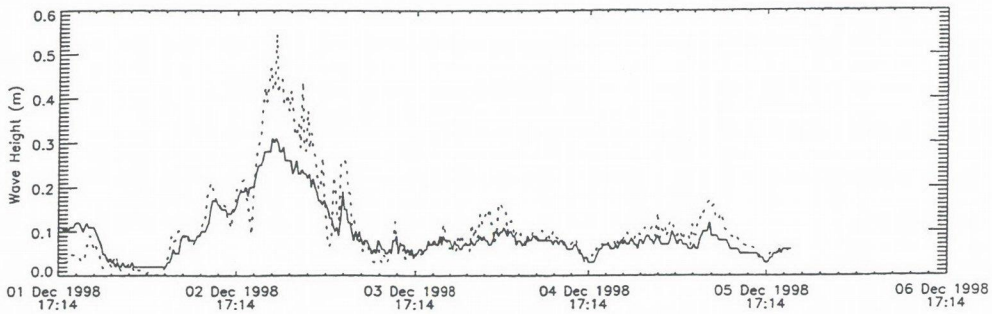
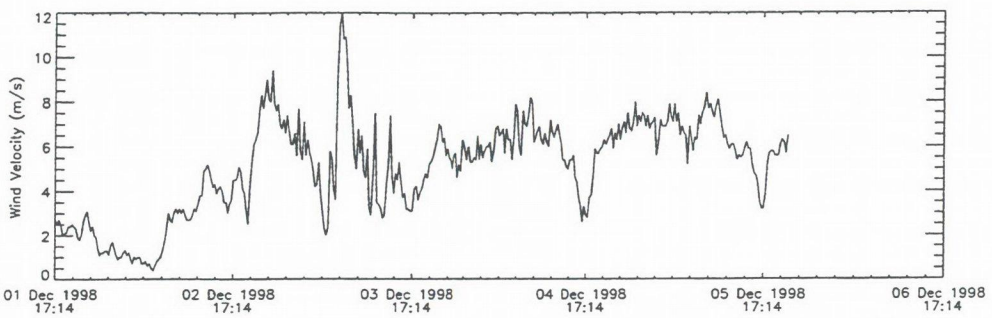
COMPARISON OF HINDCAST AND MEASURED WAVES AT LAKE VICTORIA

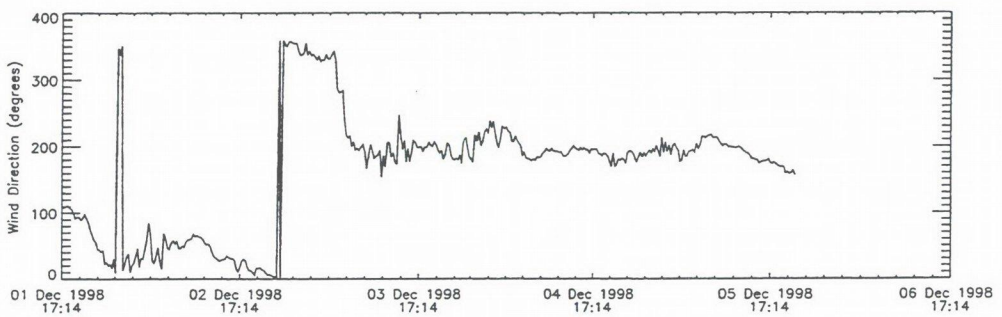
MHL Report 1085

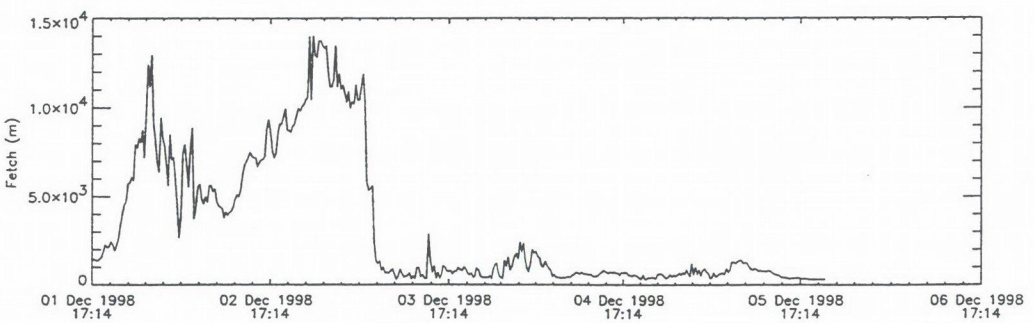
Figure C1

DRAWING 1085WC1.CDR









LEGEND

Hindcast waves

Measured waves

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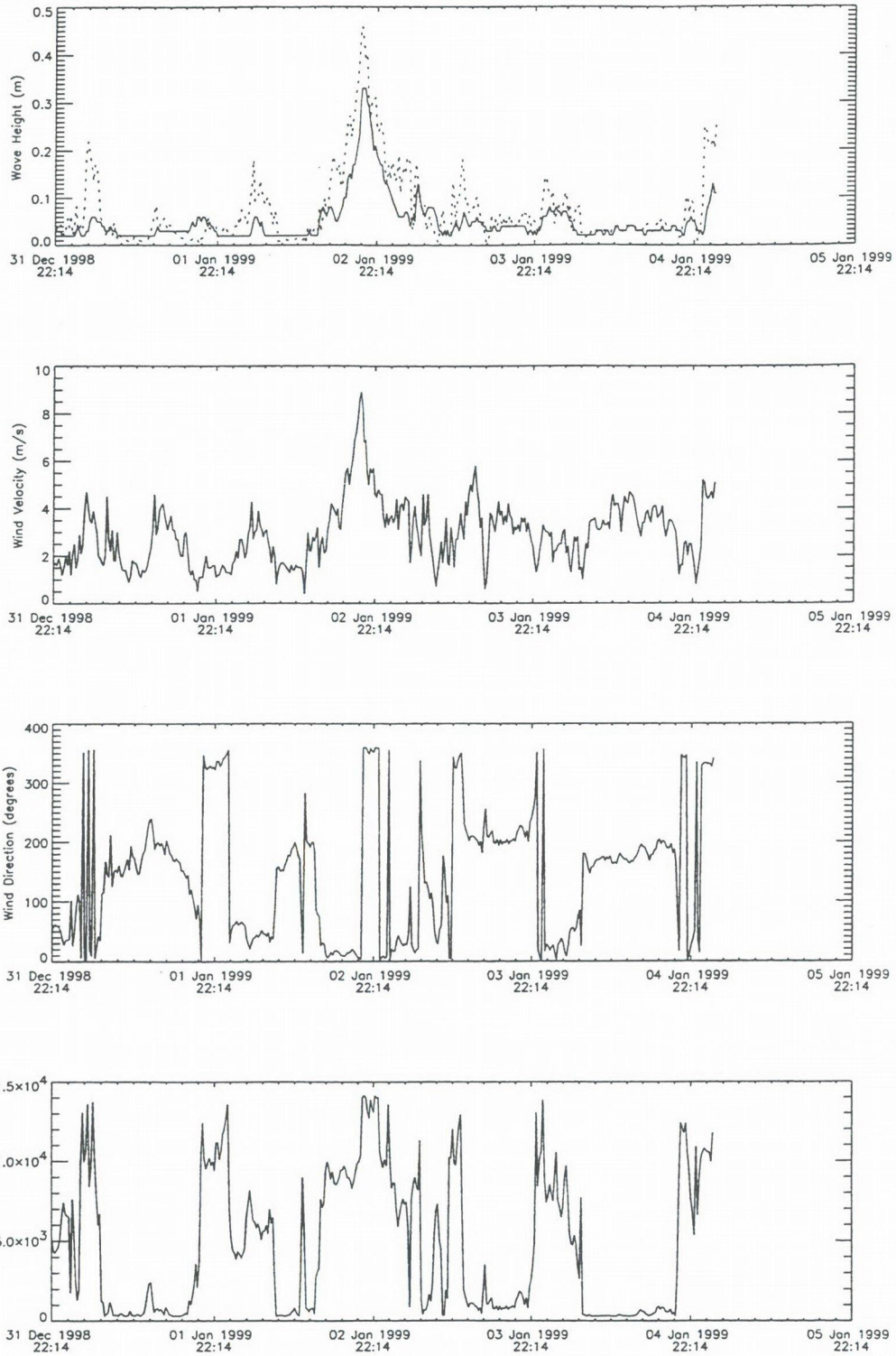
COMPARISON OF HINDCAST AND MEASURED WAVES AT LAKE VICTORIA

MHL Report 1085

Figure C2

DRAWING 1085WC2.CDR



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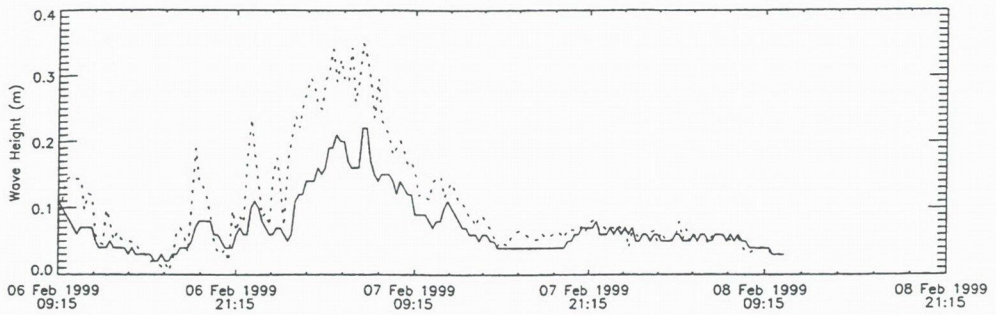
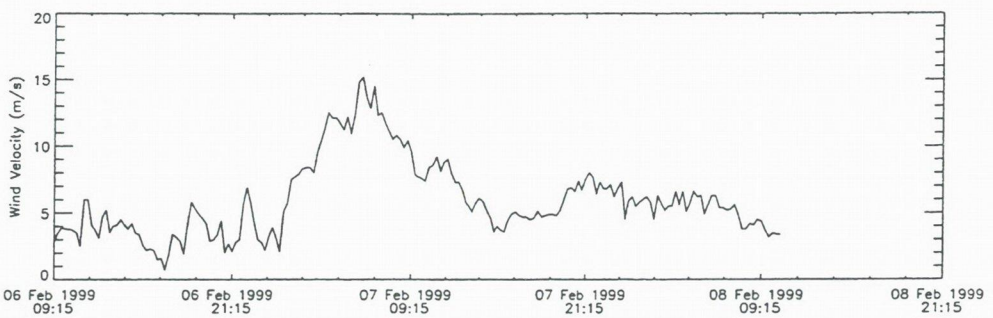
COMPARISON OF HINDCAST AND MEASURED WAVES AT LAKE VICTORIA

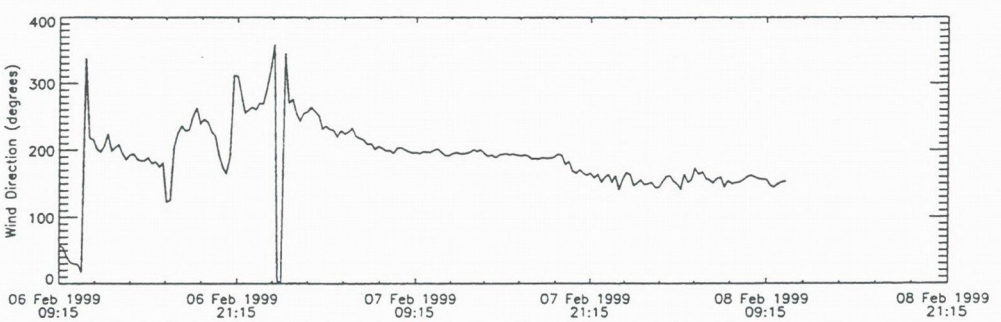
MHL Report 1085

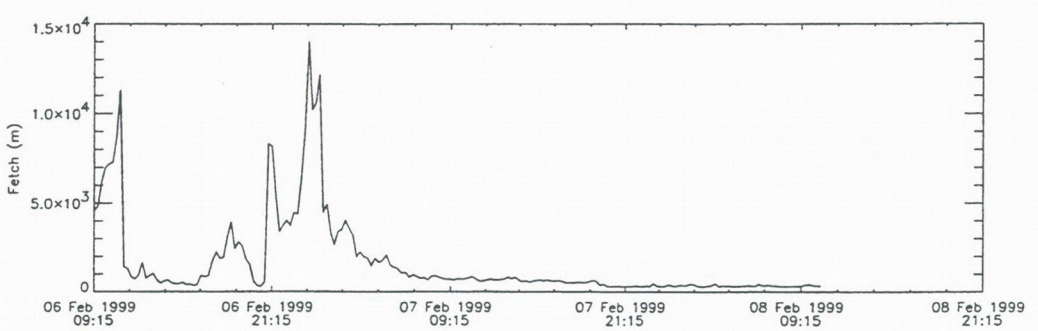
Figure C3

DRAWING 1085WC3.CDR









LEGEND

Hindcast waves

Measured waves

NSW DEPARTMENT OF PUBLIC WORKS AND SERVICES

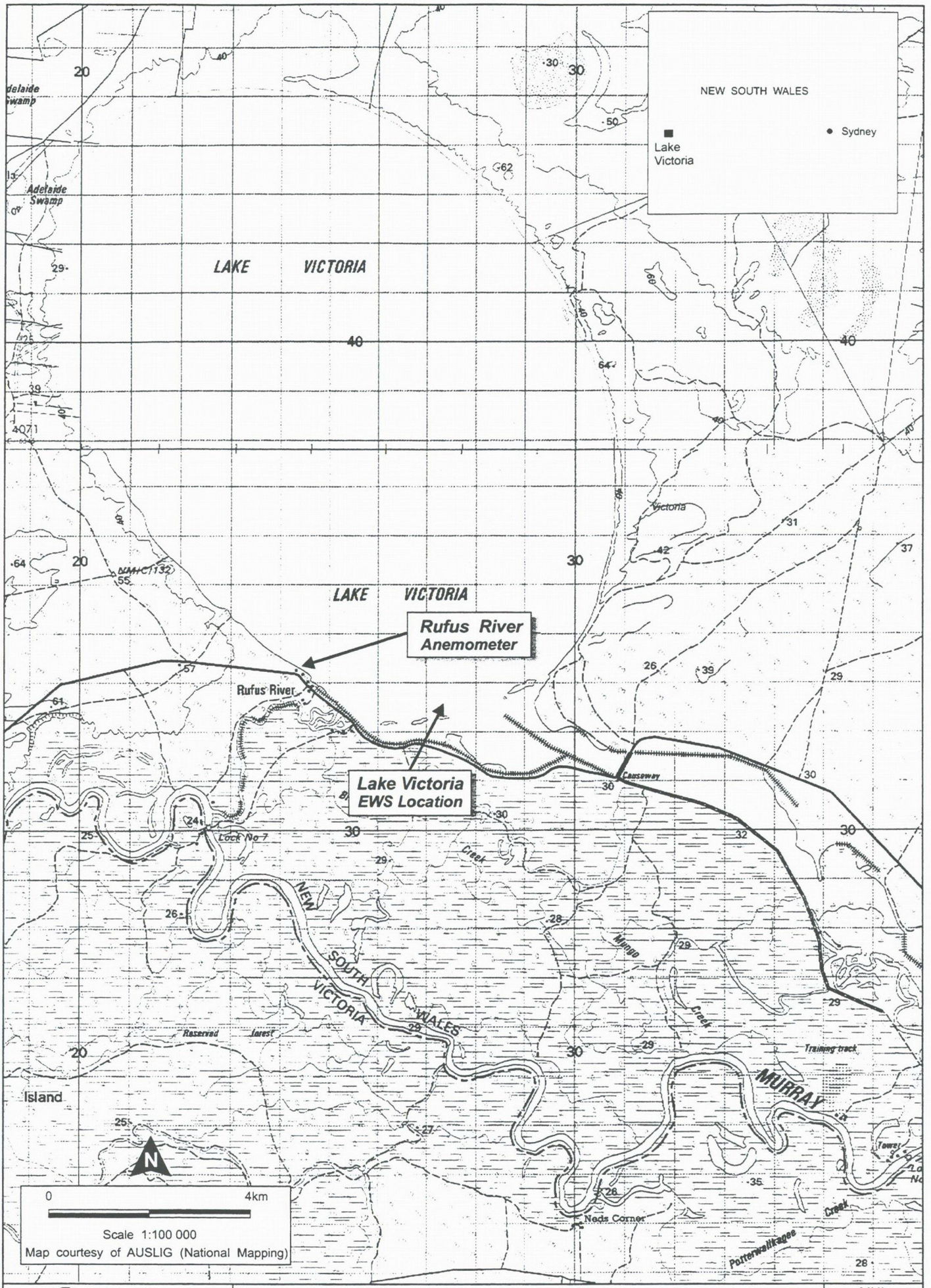
MANLY HYDRAULICS LABORATORY

COMPARISON OF HINDCAST AND MEASURED WAVES AT LAKE VICTORIA

MHL Report 1085

Figure C4

DRAWING 1085WC4.CDR



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SIMULTANEOUS WIND AND WAVE MEASUREMENT AT LAKE VICTORIA (NSW)

MHL Report 1085

Figure C5

DRAWING 1085WC5.CDR

Appendix D

Preliminary Runup Calculations (MHL 2000)

Table D1 Runup Level Calculations for 15 m Flood and 1-in-1-Year Wave for the Northern Route

Length Along Route (m) Estimated Level of Maximum Wave m (AHD)	Significant Wave Height and Period at Center of Northern Option (Direction)		Estimated Level of Maximum Wave m (AHD)	Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)		Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	0.55	2.5	15.8	16.6	16.2
100 m	0.59	2.8	15.8	16.6	16.3
200 m	0.63	2.8	15.9	16.6	16.3
300 m	0.67	2.8	16.0	16.6	16.3
400 m	0.71	2.8	16.0	16.7	16.3
500 m	0.75	2.8	16.1	16.7	16.3
1,000 m	0.96	3.2	16.3	17.2	16.7
1,500 m	0.84	3.3	16.2	17.1	16.6
2,000 m	0.86	3.0	16.2	17.1	16.6
2,100 m	0.68	3.0	15.9	16.8	16.3
2,200 m	0.51	3.0	15.6	16.5	16.1
2,300 m	0.34	3.0	15.5	16.2	15.9
2,400 m	0.17	3.0	15.2	15.9	15.7
2,500 m (Mulgrave)	0	0	15.0	15	15

A linear variation in wave height was assumed for the first and last 400 m along the route. The 2% runup level is estimated for 100 waves reaching the embankment. The levels estimated for the 1-in-1-year wave indicate that the proposed level of the crossing (17.3 m AHD) will not be exceeded. The runup due to the 1-in-20-year and 1-in-100-year wave will exceed the proposed level of the crossing. Runup was also estimated for an oblique wave on a grass slope for comparison purposes.

Table D2 Runup Level Calculations for a 15 m Flood and 1-in-20-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	1.03 (ESE)	3.0	17.2	16.6
500 m	1.36 (SSE)	2.8	17.8	17.1
1,000 m	1.44 (S)	3.2	18.1	17.4
1,500 m	1.27 (S)	3.3	17.7	17.0
2,000 m	1.20 (S)	3.0	17.5	16.9
2,500 m (Mulgrave)	-	-	-	-

Table D3 Runup Level Calculations for 15 m Flood and 1-in-100-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Center of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	1.10 (ESE)	3.2	17.3 (ESE)	16.7
500 m	1.56 (SSE)	3.6	18.1 (S)	17.3
1000 m	1.51 (S)	3.8	18.3 (S)	17.5
1500 m	1.33 (S)	3.4	17.7 (S)	17.1
2000 m	1.36 (S)	3.5	17.9 (S)	17.2
2500 m (Mulgrave)	-	-	-	-

Table D4 Runup Level Calculations for 16 m Flood for the Northern Route

Return Period (for a Southerly Wave)	Significant Wave Height and Period at Center of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
			Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
	H_s (m)	T_p (s)		
1-in-1-year event	1.0 (south)	3.3	18.2	17.7
1-in-20-year event	1.38 (south)	3.7	19.0	18.3
1-in-50-year event	1.5 (south)	3.8	19.3	18.6
1-in-100-year Event	1.57 (south)	3.9	19.4	18.6

The runup during a 16 m flood due to a 1-in-1-year wave climate exceeds the proposed 17.3 m level of the proposed evacuation route.

Table D5 Runup Level Calculations for 17.3 m Flood and 1-in-1-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period (Direction)		Estimated Level of Maximum Wave (m AHD)	Runup on a 1:2.5 Embankment	
				Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
	H_s (m)	T_p (s)			
0 m (Windsor)	0.58	2.6	18.1	18.7	18.4.
100 m	0.58	3.0	18.1	18.7	18.4
200 m	0.57	3.0	18.0	18.6	18.3
300 m	0.57	3.0	18.0	18.6	18.3
400 m	0.56	3.0	18.0	18.6	18.3
500 m	1.05	2.8	18.8	19.8	19.2
1,000 m	1.03	3.2	18.7	19.7	19.2
1,500 m	1.01	3.3	18.7	19.6	19.1
2,000 m	0.98	3.0	18.7	19.6	19.1
2,100 m	0.78	3.0	18.4	19.1	18.9
2,200 m	0.58	3.0	18.1	18.9	18.6
2,300 m	0.38	3.0	17.8	18.7	18.4
2,400 m	0.18	3.0	17.5	18.2	18.1
2,500 m (Mulgrave)	0	0	17.3	17.3	17.3

A linear variation in wave height was assumed for the last 400 m along the route.

Table D6 Runup Level Calculations for 17.3 m Flood and 1-in-20-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Center of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Runup on Grass Slope at 30° (AHD)
0 m (Windsor)	0.98 (ESE)	3.1	19.4	18.9
500 m	1.47 (S)	3.8	20.5	19.8
1000 m	1.44 (S)	3.8	20.5	19.8
1500 m	1.41 (S)	3.7	20.4	19.7
2000 m	1.37 (S)	3.7	20.3	19.6
2500 m (Mulgrave)	-	-	17.3	17.3

Table D7 Runup Level Calculations for 17.3 m Flood and a 1-in-100-Year Wave for the Northern Route

Length Along Route (m)	Significant Wave Height and Period at Center of Northern Option (Direction)		Runup on a 1:2.5 Embankment	
	H_s (m)	T_p (s)	Maximum 2% Level of Runup (AHD)	Run up on Grass Slope at 30° (AHD)
0 m (Windsor)	1.16 (ESE)	3.3	19.8 (ESE)	19.2
500 m	1.67 (S)	4.0	20.9 (S)	20.1
1,000 m	1.64 (S)	4.0	20.9 (S)	20.1
1,500 m	1.60 (S)	3.9	20.8 (S)	20.0
2,000 m	1.56 (S)	3.8	20.7 (S)	19.9
2,500 m (Mulgrave)	1.45 (S)	3.6	20.3 (S)	19.6

Appendix E Flood Glossary

annual exceedance probability — The chance or likelihood that an event of a nominated size or greater (e.g. flood discharge) will occur in any year.

Australian Height Datum (AHD) — A common national plane of level corresponding approximately to mean sea level.

bathymetry — The measurement of depths of water; also information derived from such measurements.

bed load — That portion of the total sediment load that flowing water moves along the bed by the rolling sediment particles .

catchment — The area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.

damage potential — The susceptibility of coastline development to damage by coastline hazards.

design floor level (DFL) — A minimum floor level specified as part of a building control program.

development — The erection of a building or the carrying out of work; or
the use of land or of a building or work; or
the subdivision of land.
infill development: refers to the development of vacant blocks of land that are generally surrounded by developed properties.
new development: refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.
redevelopment: refers to the rebuilding of an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require major extensions to urban services.

discharge — The rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow which is a measure of how fast the water is moving rather than how much is moving.

ebb tide — The outflow of coastal waters from bays and estuaries caused by the falling tide.

effective warning time — Is equal to the available warning time, less the time taken to alert flood-affected people (by radio, television, loud-hailer or word of mouth), and have them commence effective evacuation procedures.

estuary — An enclosed or semi-enclosed body of water having an open or intermittently open connection to coastal waters in which water levels vary in a periodic fashion in response to ocean tides.

flood awareness — An appreciation of the likely effects of flooding and a knowledge of the relevant flood warning and evacuation procedures. In communities with a high degree of flood awareness, the response to flood warnings is prompt and efficient. In communities with a low degree of flood awareness, flood warnings are liable to be ignored and residents are often confused about when to evacuate, what to take and where it should be taken.

flood fringe — The remaining area of land affected by flooding, after floodway and flood storage areas have been defined.

flood hazard — Potential for damage to property or persons due to flooding.

flooding — The State Emergency Service uses the following definitions in flood warnings:
minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding, on the reference gauge, is the initial flood level and the upper limit is determined by local conditions.
moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic bridges may be covered. The range on the reference gauge is determined by local conditions.
major flooding: extensive rural areas are flooded with properties, villages and towns isolated and/or appreciable urban areas are flooded. The threshold for this class of flooding is the upper limit of moderate flooding.

flood-liable land — Land which would be inundated as a result of the standard flood.

flood mitigation works — Structures that are designed to manage floodwaters (eg. levees, retarding basins).

floodplain — The portion of a river valley, adjacent to the river channel, which is covered with water when the river overflows during floods.

floodplain management measures	The full range of techniques available to floodplain management.
floodplain management options	The measures which might be feasible for the management of a particular area.
flood proofing	A combination of measures incorporated in the design and/or construction and alteration of individual buildings or structures subject to flooding, for the reduction or elimination of flood damages.
flood standard (or designated flood)	should be based on an understanding of flood behaviour and the associated flood risk. It should also take into account social, economic and ecological considerations.
flood storages	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.
flood tide	The inflow of coastal waters into bays and estuaries caused by the rising tide.
floodways	Those areas where a significant volume of water flows during floods. They are often aligned with obvious naturally defined channels. Floodways are areas which, even if only partially blocked, would cause a significant redistribution of flood flow, which may in turn adversely affect other areas. They are often, but not necessarily, areas of deeper flow or the areas where higher velocities occur.
fluvial	Pertaining to non-tidal flows.
foreshore	The area of shore between low and high tide marks and land adjacent thereto.
freeboard	A factor of safety usually expressed as a height above the designated flood. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects etc.
geomorphology	The study of the origin, characteristics and development of land forms.
high hazard	Possible danger to life and limb; evacuation by trucks difficult; potential for structural damage; social disruption and financial losses could be high.
hindcast wave climate	The use of historic wind climate to estimate the wave climate at a given location on a water body.

hydraulic	The term given to the study of water flow in a river, in particular the evaluation of flow parameters such as stage and velocity.
hydrograph	A graph which shows how the discharge changes with time at any particular location.
hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
low hazard	Should it be necessary, people and their possessions could be evacuated by trucks. Able-bodied adults would have little difficulty wading.
mainstream flooding	Inundation of normally dry land occurring when water conveyed to the locality from further upstream overflows the natural or artificial banks of the principal watercourse in the catchment. It generally excludes any watercourses constructed with pipes or artificial channels or considered as stormwater channels.
management plan	A document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, problems, special features and values of the area, the specific management measures which are to apply and the means and timing by which the plan will be implemented.
mathematical/computer models	The mathematical representation of the physical processes involved in runoff and streamflow. These models are often run on computers due to the complexity of the mathematical relationships. In this manual, the models referred to are mainly involved with rainfall, runoff and stream flow.
neap tides	Tides with the smallest range in a monthly cycle. Neap tides occur when the sun and moon lie at right angles relative to the earth (the gravitational effects of the moon and sun act in opposition on the ocean).
numerical model	A mathematical representation of a physical, chemical or biological process of interest. Computers are often required to solve the underlying equations.
peak discharge	The maximum discharge occurring during a flood event.
probable maximum flood	The flood calculated to be the maximum which is likely to occur.

probability	A statistical measure of the expected frequency or occurrence of flooding.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
sediment load	The quantity of sediment moved past a particular cross-section in a specified time.
significant wave height	The average height of the highest one third of waves recorded in a given monitoring period. Also referred to as $H_{1/3}$ or H_s
spring tides	Tides with the greatest range in a monthly cycle, which occur when the sun, moon and earth are in alignment (the gravitational effects of the moon and sun act in concert on the ocean) .
stage	Equivalent to 'water level'. Both are measured with reference to a specified datum.
stage hydrograph	A graph which shows how the water level changes with time. It must be referenced to a particular location and datum.
storm surge	The increase in coastal water levels caused by the barometric and wind setup effects of storms. Barometric setup refers to the increase in coastal water levels associated with the lower atmospheric pressures characteristic of storms. Wind setup refers to the increase in coastal water levels caused by an onshore wind driving water shorewards and piling it up against the coast.
tides	The regular rise and fall of sea level in response to the gravitational attraction of the sun, moon and planets. Tides along the New South Wales coastline are semi-diurnal in nature, i.e. they have a period of about 12.5 hours.
total catchment management (in the context of the Catchment Management Act, 1989)	"The coordinated and sustainable use of land, water, vegetation and other natural resources on a water catchment basis so as to balance resource utilisation and conservation".
water surface profile	A longitudinal plot showing the flood stage at any given location along a watercourse.
wave setup	The increase in water level within the surf zone above mean still water level caused by the breaking action of waves.

Working Paper 6 – Noise and Vibration Assessment

NOISE AND VIBRATION ASSESSMENT
WINDSOR FLOOD EVACUATION ROUTE
SOUTH CREEK
WINDSOR

32.5030.R1:DD31
Rev 03

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June 2002

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APPENDIX 1: NOISE MONITORING RESULTS

1.0 INTRODUCTION

The NSW Road and Traffic Authority (RTA) propose to construct a flood evacuation route across South Creek, Windsor (Figure 1).

Atkins Acoustics was commissioned by Connell Wagner Pty Ltd on behalf of the RTA to conduct an assessment of road traffic noise from operation and assessment of noise and vibration from construction of the proposal.

The proposed road is located to the south-west of Windsor Road. The route will avoid the flood affected section of Windsor Road between Groves Avenue and Macquarie Street.

This report provides results of the noise assessment and advice on possible control measures to reduce noise impacts from road traffic, and noise and vibration impacts during operation and construction. The findings from the study have been prepared for the particular investigation described and no part should be used in any other context or for any other purpose.

2.0 METHODOLOGY

The methodology for the noise and vibration assessment is as follows:

- measurement of existing noise environment at receivers potentially affected by the proposal;
- establishment of project noise assessment goals, in accordance with the guidelines in the NSW EPA's Environmental Criteria for Road Traffic Noise (May 1999);
- establishment of construction noise assessment goals, in accordance with the guidelines in the EPA's Environmental Noise Control Manual (1994);
- establishment of vibration assessment goals in accordance with the EPA's Environmental Noise Control Manual (1994) guidelines, and other relevant guidelines;
- measurements and prediction of existing traffic noise levels to validate the noise prediction model;
- prediction and assessment of future road traffic noise levels from the proposal in accordance with acceptable modelling procedures;
- prediction and assessment of noise and vibration from construction activities; and
- the consideration and recommendation of possible mitigation measures to reduce noise and vibration impacts.

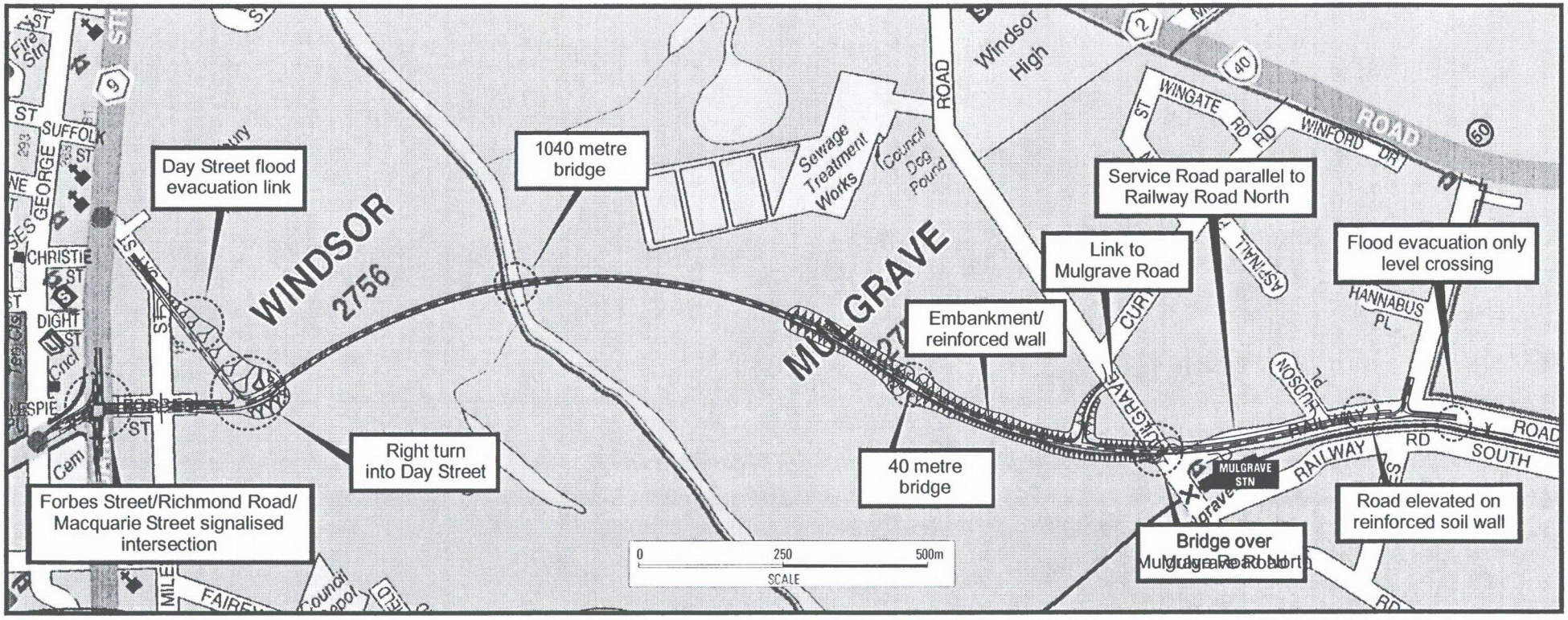
3.0 DESCRIPTION OF THE PROPOSAL

The proposal known as the South Creek Flood Evacuation Route, is approximately 2.8km long, and connects Macquarie Street, Windsor to Railway Road North, Mulgrave. The route will connect to Windsor Road via Groves Avenue at Mulgrave, and join Forbes Street and Richmond Road/Windsor Road at Windsor. The road avoids the flood-affected section of Windsor Road between Macquarie Street Windsor and Pitt Town Road McGraths Hill. Figure 1 shows the location of the proposal.

Noise impacts from the proposal are due to road traffic and plant and equipment associated with the road construction of the proposal. Other potential impacts are associated with the transmission of ground vibration from construction plant and equipment. The main construction activities will involve piling, excavation, earthworks, formation and compaction of road base, bridge construction, and construction of the road pavement.

The road proposal will mainly pass rural land, industrial and commercial land uses. There will be residential properties potentially affected by the proposal. These residences are located on Forbes Street, Day Street, Mileham Street, Macquarie Street and in the vicinity of Railway Road North, Railway Road South, Mulgrave Road, Chisholm Place, Harris Street and James Meehan Street.

Traffic on Windsor Road is projected to reduce with the opening of the proposal and hence traffic noise levels at receivers adjacent to Windsor Road between McGrath Road and Pitt Town Road, McGrath Hill is predicted to marginally reduce as a result of the development. Traffic noise on Macquarie Street, Mileham Street and Day Street is also predicted to marginally reduce following the road opening.



WINDSOR
FLOOD EVACUATION ROUTE

FIGURE 1
PROPOSED ROUTE ALIGNMENT

4.0 EXISTING AMBIENT NOISE LEVELS

The existing noise environment at representative locations was measured over seven day periods at seven residences within the study area. Instrumentation used for the measurements consisted of environmental noise data loggers. The data loggers were set to A-weighting, fast response and programmed to calculate and record statistical noise levels at every 15-minute interval. The reference calibration of each instrument was checked before and after the measurement periods using a Bruel & Kjaer sound level calibrator Type 4230 and no significant drift was detected.

Table 1 and Figure 2 provide details of the noise monitoring locations and date of monitoring. Detailed results of the noise measurements showing common statistical noise levels at every 15-minute interval are given in *Appendix 1*. *Tables 2* and *3* provide a summary of the measured $L_{Aeq,15hr}$ and $L_{Aeq,9hr}$ noise levels. It is noted that data measured on the first and last days of the measurement periods was incomplete and hence the results are not presented in *Tables 2* and *3*.

The measured noise levels adjacent to Windsor Road were used to compare with the predicted levels to validate the noise prediction model. The measured noise levels at other locations were used to establish project assessment goals for operational and construction noise.

The measured noise levels at location R5 were controlled by road traffic on Windsor Road. The measurement locations R3 and R4 were also affected by road traffic on Macquarie Street. At other locations, the measured noise levels were due to local traffic, rail traffic, occasional aircraft, local industry and domestic activities.

Table 1 Noise Monitoring Locations

Monitoring Location	Date of Monitoring	Noise Source
R2 - 2 Chisholm Place	Monday 22 May - Monday 29 May, 2000	Trains, aircraft and domestic activities
R3 - "Trevallyn" Day Street	Friday 19 May - Monday 29 May, 2000	Traffic, aircraft and domestic activities
R4 - 16 Forbes Street	Friday 19 May - Monday 29 May, 2000	Traffic, aircraft, local industry and activities
R5 - Windsor Road	Monday 22 May - Monday 29 May, 2000	Traffic and occasional aircraft
R6 - 46 Mulgrave Street	Monday 22 May - Monday 29 May, 2000	Distant and local traffic, trains, aircraft and local activities
R7 - 112 Mulgrave Street	Monday 29 May - Monday 5 June 2000	Local traffic, trains, farming equipment, aircraft, wind in trees and birds
R8 - 2 Railway Road South	Monday 29 May - Monday 5 June 2000	Local traffic, trains, farming equipment, aircraft, wind in trees, birds and domestic animals

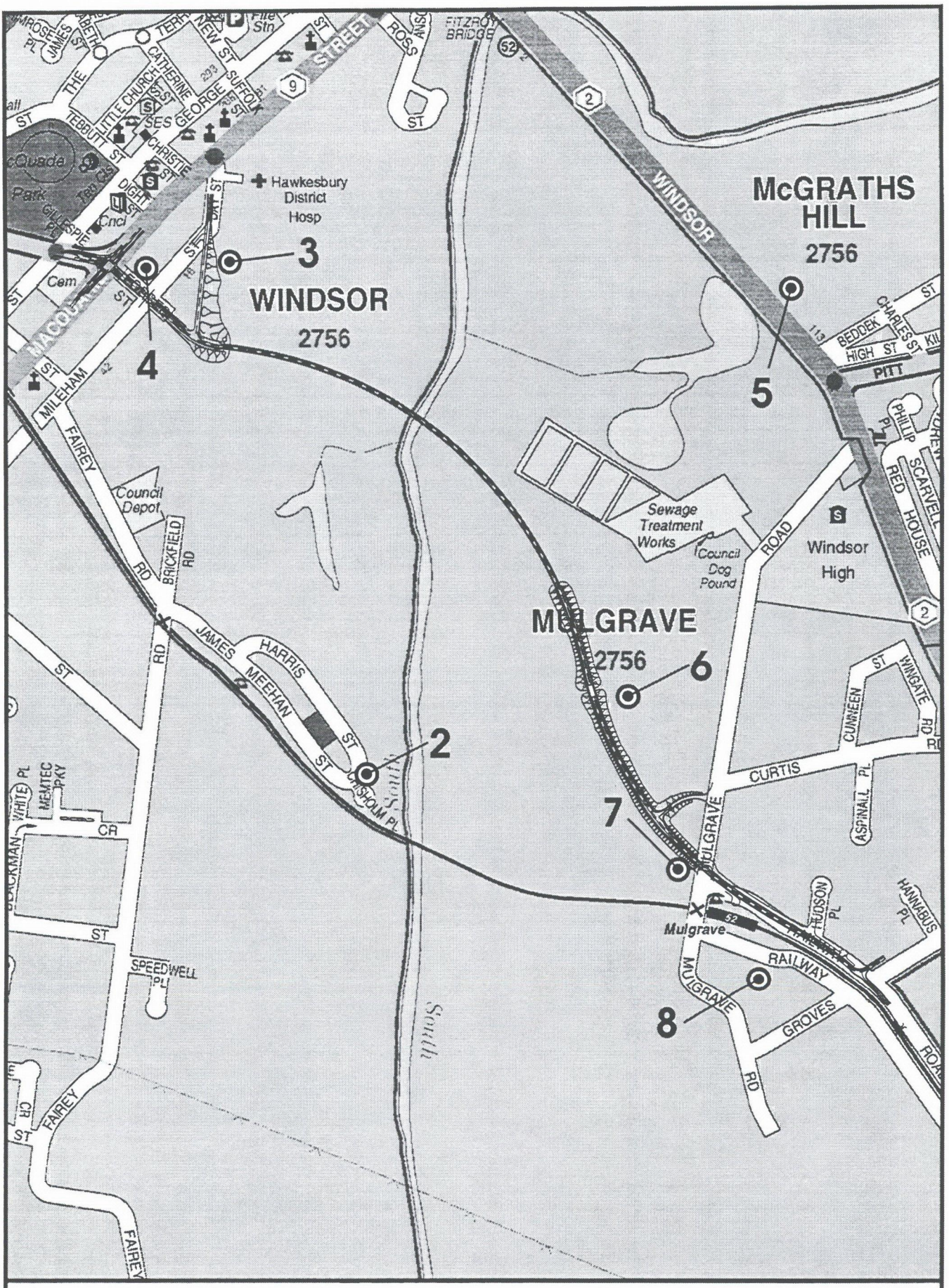

McGRATHS HILL
2756
WINDSOR
2756
MULGRAVE
2756
Hawkesbury District Hosp
Sewage Treatment Works
Council Depot
Council Dog Pound
Windsor High
Mulgrave
WINDSOR
FLOOD EVACUATION ROUTE
N
FIGURE 2
NOISE MONITORING LOCATIONS

Table 2 *Existing $L_{Aeq,15hr}$ Noise Levels*
dB(A) re 20 $\times 10^{-6}$ Pa

Date of Monitoring	Monitoring Location						
	R2	R3	R4	R5	R6	R7	R8
Saturday 20 May 2000		59.5	60.0				
Sunday 21 May 2000		58.9	60.2				
Monday 22 May 2000		61.6	63.6				
Tuesday 23 May 2000	54.7	61.9	62.7	68.8	57.5		
Wednesday 24 May 2000	57.1	61.8	63.1	68.4	57.8		
Thursday 25 May 2000	54.2	61.1	61.9	68.9	58.1		
Friday 26 May 2000	54.7	63.2	64.0	68.7	59.2		
Saturday 27 May 2000	54.9	60.1	59.9	68.1	62.7		
Sunday 28 May 2000	55.4	59.7	60.0	67.4	62.9		
Monday 29 May 2000							
Tuesday 30 May 2000						56.3	56.2
Wednesday 31 May 2000						56.9	56.2
Thursday 1 June 2000						57.3	56.7
Friday 2 June 2000						58.2	56.4
Saturday 3 June 2000						57.7	55.2
Sunday 4 June 2000						51.6	58.5
Logarithmic Average	**55**	**61**	**62**	**68**	**60**	**57**	**57**

Table 3 *Existing $L_{Aeq,9hr}$ Noise Levels*
dB(A) re 20 $\times 10^{-6}$ Pa

Date of Monitoring	Monitoring Location						
	R2	R3	R4	R5	R6	R7	R8
Friday 19 May 2000		45.6	48.6				
Saturday 20 May 2000		45.7	48.5				
Sunday 21 May 2000		51.8	53.0				
Monday 22 May 2000	51.3	52.8	52.6	64.4	50.0		
Tuesday 23 May 2000	49.9	48.6	50.1	64.2	48.9		
Wednesday 24 May 2000	48.7	45.5	49.1	64.1	48.1		
Thursday 25 May 2000	48.6	53.2	53.9	64.1	51.4		
Friday 26 May 2000	45.7	45.2	49.1	63.3	48.5		
Saturday 27 May 2000	47.2	47.6	49.1	61.0	48.7		
Sunday 28 May 2000	48.7	47.8	48.7	63.8	47.4		
Monday 29 May 2000						51.2	50.9
Tuesday 30 May 2000						52.0	49.0
Wednesday 31 May 2000						53.3	49.8
Thursday 1 June 2000						52.8	51.0
Friday 2 June 2000						50.1	48.9
Saturday 3 June 2000						47.8	48.1
Sunday 4 June 2000						52.8	50.8
Logarithmic Average	**49**	**49**	**51**	**64**	**49**	**52**	**50**

5.0 TRAFFIC NOISE ASSESSMENT GOALS

Goals for assessment of road traffic noise are nominated in the NSW EPA's Environmental Criteria for Road Traffic Noise (May 1999) (ECRTN). The goals depend on the existing noise environment, type of road development and land uses. The road proposal would be categorised as a new arterial road with collector roads. Referenced to the ECRTN the following objectives apply to traffic volumes projected 10 years post opening:

5.1 New Arterial Roads

The assessment baseline objective goals for new arterial roads are summarised below. These assessment goals apply to residential receivers along the proposed flood evacuation route.

- daytime $L_{Aeq,15hr}$ traffic noise levels between the hours from 7:00am to 10:00pm at 1m from the most exposed external facade of any residential premises should be limited to 55dB(A); and

- night-time $L_{Aeq,9hr}$ traffic noise levels between the hours from 10:00pm to 7:00am at 1m from the most exposed external facade of any residential premises should be limited to 50dB(A).

Where the existing road traffic noise already exceed the baseline objectives, the road should be designed not to increase the existing levels by more than 0.5dB(A).

5.2 New Collector Roads

Traffic noise baseline objective goals for new collector roads are referenced in the ECRTN and summarised below. These goals would apply to receivers along Mileham Street and Day Street.

- daytime $L_{Aeq,1hr}$ traffic noise levels between the hours from 7:00am to 10:00pm at 1m from the most exposed external facade of any residential premises should be limited to 60dB(A); and

- night-time $L_{Aeq,1hr}$ traffic noise levels between the hours from 10:00pm to 7:00am at 1m from the most exposed external facade of any residential premises should be limited to 55dB(A).

Where the existing road traffic noise levels already exceed the baseline objectives, the road should be designed not to increase the existing traffic noise levels by more than 0.5dB(A).

5.3 Sensitive Land Uses

The Hawkesbury District Hospital has been identified as a potential sensitive land use. Accordingly, referenced to the ECRTN an internal noise level of $L_{Aeq,1hr}$ 35dB(A) would apply to the assessment of traffic noise within hospital rooms. With windows open for natural ventilation, the accepted difference between the internal and external noise levels is 10dB(A). Hence, the noise level at 1m from the most exposed external façade of the hospital for the purpose of assessing traffic noise is $L_{Aeq,1hr}$ 45dB(A). Where the pre-development traffic noise levels exceed the recommended assessment goal, the proposed road development should not increase the existing traffic noise levels by more than 2dB(A). Our site inspections have shown that the existing traffic noise at the hospital is controlled by traffic on both Macquarie and Day Streets.

5.4 Intermittent Traffic Noise

Maximum noise levels from vehicle pass-bys during night-time hours have the potential to cause sleep disturbance to the occupants of buildings. For these situations the EPA require impacts of L_{Amax} noise levels from vehicle pass-bys to be predicted and assessed at the potentially affected buildings.

For the assessment of intermittent noise sources during night-time hours, the ECRTN concludes that for the current level of understanding, it is not possible to establish absolute noise level criteria that would correlate to acceptable level of sleep disturbance. Therefore, the assessment procedure considered for this assessment has followed the procedures documented in the "*Practice Note III – protocol for assessing maximum noise levels*" of the RTA Environmental Noise Management Manual (ENMM).

Reference to the ENMM, findings from a review of international sleep disturbance research indicate that:

- maximum internal noise levels below 50-55dB(A) are unlikely to cause awakening reactions. These levels equate to external façade L_{Amax} levels of 60-65dB(A) with bedroom windows open to normal extent for natural ventilation;

- one or two noise events per night with maximum internal noise levels of 65-70dB(A) are unlikely to affect health and well-being significantly. These equate to external façade L_{Amax} noise levels of 75-80dB(A) with bedroom windows open to provide natural ventilation; and

- for continuous traffic flow, the $L_{Aeq,9hr}$ noise goal sufficiently accounts for sleep disturbance impacts. However, where the L_{Amax} exceed the $L_{Aeq,9hr}$ noise level by 15 dB(A) or higher for intermittent traffic flow, sleep disturbance impacts may be significant.

The RTA consider that the maximum noise assessment should not be applied as a decisive criterion. However, recommend that it be used as a tool to rank and prioritise design options and noise mitigation strategies.

6.0 ROAD TRAFFIC NOISE PREDICTION MODEL

The UK Department of Environment's "Calculation of Traffic Noise" (CORTN) procedure was adopted for the prediction of traffic noise. The model has been validated under Australian conditions (*R E Saunders, S E Samuels, A Hall, A Leach (1983), "An Evaluation of the UK DoE Traffic Noise Prediction Method"*).

The CORTN procedure predicts traffic noise in terms of $L_{A10,1hr}$ and $L_{A10,18hr}$ levels. Conversions to the L_{Aeq} noise levels can be performed with the input of relevant traffic data and application of correction factors (*Technical Note No. 1 - Some Simple Transformations for Road Traffic Noise Scales, A L Brown, School of Australian Environmental Studies, Griffith University, Nathan, Queensland*).

The CORTN procedure does not take account of atmospheric absorption, hence it tends to over-predict traffic noise levels at large distances (>300m from the road). In addition, predictions of noise levels in the presence of barriers are conservative as soft ground effect is ignored. Noise attenuation due to atmospheric absorption and soft ground effect is typically 5dB(A), 7dB(A) and 9dB(A) at distances of 300m, 400m and 500m from the road respectively.

Modification to the CORTN noise prediction model was made to represent the source heights for passenger cars, heavy vehicles and heavy vehicle exhausts which are 0.5m, 1.5m and 3.6m above the road level respectively. For heavy vehicles, noise levels contributed from the exhausts are 8dB(A) lower than those from the engines (*Transportation Noise Reference Book, Paul Nelson, 1987*).

Additionally, Atkins Acoustics was advised that a dense graded asphalt road surface would be adopted and the posted road speed east of Day Street would be 80km/hr. The average traffic speed on the connecting roads was assumed to be 60km/hr. Open graded asphalt road surface was considered, however due to ongoing maintenance requirements the RTA considered the surface was not practical for this project.

The CORTN model calculates noise levels at 10m from the nearside road edge and applies corrections to account for the change in traffic speed, road gradient, road surface, distance propagation, ground type, angle of view, facade refection and screening effects from buildings, barriers, cuttings and embankments.

The noise modelling was based on digitised ground contour and road alignment maps provided by Connell Wagner.

7.0 TRAFFIC DATA

The existing and projected daytime, night-time and peak hour traffic volumes for the proposal and connecting roads for the years 2006 and 2016, ie. post-opening and 10 years post-opening, were provided by Connell Wagner. *Tables 4* to *6* present a summary of the existing and forecast traffic data for Year 2006 and Year 2016.

Atkins Acoustics was advised that the projected proportions of heavy vehicles for the proposed road are 7.5% and 9.5% during daytime and night-time, respectively.

Comparison of the existing and forecast traffic data showed that that traffic volumes on the connecting roads, ie. Macquarie Street, Mileham Street and Day Street will reduce following the road opening.

Table 4 Existing Traffic Data (Year 2000)

Road Name	Existing Traffic Volume	
	Daytime	Night-time
Macquarie Street	25,786	3,684
Mileham Street and Day Street	5,618	802

Notes Daytime is from 7:00am to 10:00pm
Night-time is from 10:00pm to 7:00am

Table 5 Forecast Traffic Data (Year 2006)

Road Name	Year 2006 Traffic Volume			
	Daytime	Night-time	AM Peak	PM Peak
Macquarie Street				
North of Day Street	19,575	2,796	1,725	1,812
Between Day Street and Flood Evacuation Route	17,074	2,439	1,498	1,655
South of Flood Evacuation Route	19,813	2,830	1,608	1,966
Mileham Street				
South of Flood Evacuation Route	4,643	663	456	498
From Flood Evacuation Route to Day Street	4,038	577	247	654
Day Street				
From Flood Evacuation Route to Mileham Street	2,189	313	243	183
From Mileham Street to Macquarie Street	2,482	406	256	211
Proposed Flood Evacuation Route				
From Macquarie Street to Mileham Street	11,314	1,616		
From Mileham Street to Day Street extension	11,375	1,625		
Flood Evacuation Route	13,563	1,938		

Table 6 Forecast Traffic Data (Year 2016)

Road Name	Year 2016 Traffic Volume			
	Daytime	Night-time	AM Peak	PM Peak
Macquarie Street				
North of Day Street	21,382	3,055	1,853	1,980
Between Day Street and bypass	18,685	2,669	1,617	1,800
South of bypass	21,089	3,013	1,679	2,050
Mileham Street				
South of bypass	5,276	754	500	609
From bypass to Day Street	4,472	639	281	718
Day Street				
From bypass to Mileham Street	2,275	325	235	191
From Mileham Street to Macquarie Street	3,049	436	264	227
Proposed Flood Evacuation Route				
From Macquarie Street to Mileham Street	12,224	1,746		
From Mileham Street to Day Street extension	12,182	1,740		
Bypass	14,438	2,063		

8.0 VALIDATION OF NOISE PREDICTION MODEL

Road traffic noise levels at the reference monitoring location on Windsor Road were compared with predicted levels to validate the noise prediction model.

Table 7 presents a summary of the measured and predicted noise levels for the existing traffic conditions. The existing daytime and night-time traffic volumes were 31,119 and 4,422; the proportions of heavy vehicles were 7.5% and 9.5% respectively.

Table 7 ***Predicted and Measured Traffic Noise Levels***
dB(A) re 20 $\times 10^{-6}$ Pa

Reference Location	$L_{Aeq,15hr}$ Traffic Noise Level		$L_{Aeq,9hr}$ Traffic Noise Level	
	Measured	Calculated	Measured	Calculated
Windsor Road (R5)	68	71	64	65

From the results summaries in *Table 7*, the predicted noise levels are 1-3 dB(A) higher than the measured results. Given the small differences between the calculated and measured noise levels, it is considered that the CORTN model provides reliable and conservative results for the prediction of traffic noise levels for the proposal.

9.0 PREDICTED TRAFFIC NOISE LEVELS

9.1 Continuous Traffic Flow

Road traffic noise levels were predicted for representative receiver locations within the subject study area and assessed against the project noise goals. The predicted traffic noise levels and identified exceedances are discussed in the following section of this report. Figure 3 identifies the referenced locations. *Figures 4 and 5* present the predicted $L_{Aeq,15hr}$ and $L_{Aeq,9hr}$ road traffic noise level contours respectively.

Based on the traffic data forecasts *(Tables 5 and 6)*, the traffic noise predictions show that $L_{Aeq,15hr}$ daytime levels are 5-6dB(A) higher than the $L_{Aeq,9hr}$ night-time levels. Hence, the daytime noise assessment goals are the controlling assessment goals for the project. The noise modelling results also confirmed that the predicted levels for year 2016 are marginally (1dB(A)) higher than those predicted for the year 2006.

9.1.1 Residences East of Day Street

Table 8 presents a summary of the predicted $L_{Aeq,15hr}$ and $L_{Aeq,9hr}$ noise levels at residential receivers east of Day Street for the years 2006 and 2016. From the results it can be seen that the levels exceed the 55dB(A) $L_{Aeq,15hr}$ and 50dB(A) $L_{Aeq,9hr}$ baseline assessment goals by 2-11dB(A) and 1-10dB(A) respectively.

Table 8 ***Predicted Road Traffic Noise Levels East of Day Street***
dB(A) re 20 $\times 10^{-6}$ Pa

Receiver Number	Predicted $L_{Aeq,15hr}$ Noise Level		Predicted $L_{Aeq,9hr}$ Noise Level	
	Year 2006	Year 2016	Year 2006	Year 2016
40	57	58	52	52
41 G/F	57	57	51	52
42 1/F	57	58	52	52
43 G/F	59	59	53	53
44 1/F	59	59	53	54

Table 8 ***Predicted Road Traffic Noise Levels East of Day Street***
dB(A) re 20 $\times 10^{-6}$ Pa (contd.)

Receiver Number	Predicted $L_{Aeq,15hr}$ Noise Level		Predicted $L_{Aeq,9hr}$ Noise Level	
	Year 2006	Year 2016	Year 2006	Year 2016
45	60	60	54	54
46	59	60	54	54
47	61	61	55	55
48	60	61	55	55
49 G/F	61	62	56	56
50 1/F	62	62	56	56
51 G/F	62	62	56	56
52 1/F	62	63	57	57
53 G/F	58	59	53	53
54 1/F	58	59	53	53
55 G/F	58	58	52	52
56 1/F	58	58	52	52
57	59	60	54	54
58 G/F	61	61	55	55
59 1/F	62	62	56	56
60 G/F	64	64	58	58
61 1/F	66	66	60	60
62 G/F	62	63	57	57
63 1/F	63	63	57	58
64	64	65	59	59
65	60	60	54	55
66	58	58	52	52

Note G/F and 1/F denote ground floor and first floor respectively for double storey residences.

9.1.2 Forbes Street Residences

Table 9 presents the predicted $L_{Aeq,15hr}$ and $L_{Aeq,9hr}$ road traffic noise levels at receivers on Forbes Street for the forecast traffic volumes.

The results summarised in *Table 9* show that the predicted levels exceed the 55dB(A) $L_{Aeq,15hr}$ and 50dB(A) $L_{Aeq,9hr}$ baseline assessment goals by 12-15dB(A) and 11-14dB(A) respectively. The allowance assessment goal is exceeded by 3-8dB(A).

Table 9 ***Predicted Road Traffic Noise Levels - Forbes Street***
dB(A) re 20 $\times 10^{-6}$ Pa

Receiver Number	Predicted $L_{Aeq,15hr}$ Noise Level			Predicted $L_{Aeq,9hr}$ Noise Level		
	Base Case*	Year 2006	Year 2016	Base Case*	Year 2006	Year 2016
67	60	68	68	55	62	62
68 G/F	62	68	69	57	63	63
69 1/F	63	69	70	58	63	64
70 G/F	63	69	70	57	63	64
71 1/F	63	70	70	58	64	64
72	63	67	67	58	61	61
73 G/F	65	69	69	59	63	64
74 1/F	66	69	70	60	64	64
91	61	67	68	55	62	62
92	61	69	69	56	63	63
93	61	69	68	56	63	63
94	63	68	68	57	62	62

Note *G/F and 1/F denote ground floor and first floor respectively for double storey residences.*
** Predicted noise levels in year 2000 (from Macquarie Street)*

9.1.3 Day Street Residences

Table 10 presents the predicted daytime $L_{Aeq,1hr}$ road traffic noise levels at receivers along Day Street. The results summarised in *Table 10* show that the predicted traffic noise levels exceed the daytime 60dB(A) $L_{Aeq,1hr}$ baseline assessment goals by 2-8dB(A). However, the predictions show that road traffic noise levels along Day Street reduce 1-2dB(A) in Years 2006 and 2016. This is because of traffic reduction projected for Macquarie Street, Mileham Street and Day Street at the opening and ten (10) years after opening of the proposed development.

Table 10 ***Predicted Road Traffic Noise Levels - Day Street***
dB(A) re 20 $\times 10^{-6}$ Pa

Receiver Number	Predicted Daytime $L_{Aeq,1hr}$ Noise Levels		
	Base Case*	Year 2006	Year 2016
82	64	63	63
83	64	62	63
84	64	63	63
85	65	63	64
86	64	63	63
87	69	67	68

* *Predicted noise levels in year 2000 (from Day Street, Mileham Street and Macquarie Street)*

9.1.4 Hawkesbury District Hospital

The predicted existing $L_{Aeq,peak\text{-}hour}$ road traffic noise levels for the external facades of the hospital buildings range between 58-66dB(A) (Base Case, Year 2000), and exceed the project assessment goal of 45dB(A). The results of the noise modelling for the proposed road works show that the existing traffic noise levels will decrease by 1-2dB(A) as a result of the reduction of traffic on Day and Macquarie Streets in Years 2006 and 2016.

9.2 Intermittent Traffic Flow

Maximum noise levels from heavy vehicles are generally associated with braking and accelerating. The most common source of brake noise is from auxiliary braking systems or engine compression brakes (Jacobs Brakes). The areas where truck acceleration and breaking are most likely to occur are near road intersections.

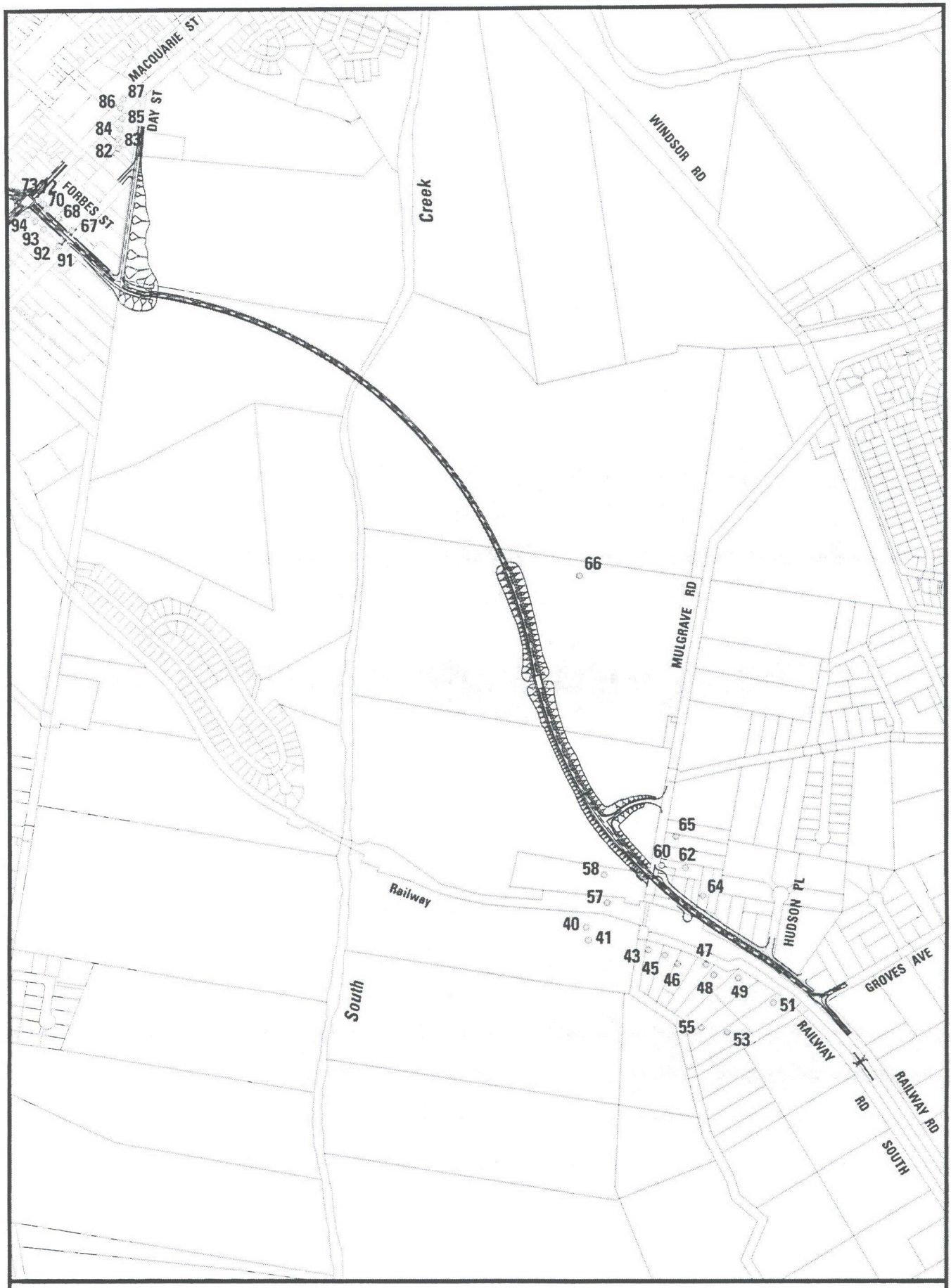
Previous assessments undertaken by Atkins Acoustics indicated that typical maximum noise levels from moving trucks accelerating is 80-87dB(A) at 7m. For a truck applying engine compression brakes, the sound level at 7m is between 85-94dB(A). *Table 11* presents the predicted single-event L_{Amax} noise levels from heavy vehicles at various distances without noise barriers.

The predicted L_{Amax} noise levels from trucks on Forbes Street, Day Street and Railway Road North range between 87-94dB(A). For the residential properties on Harris Street and James Meehan Street noise levels from truck pass-by and acceleration of 45-52dB(A) are predicted.

Table 11 Predicted L_{Amax} Noise Levels from Trucks

dB(A) re 20 × 10^{-6} Pa

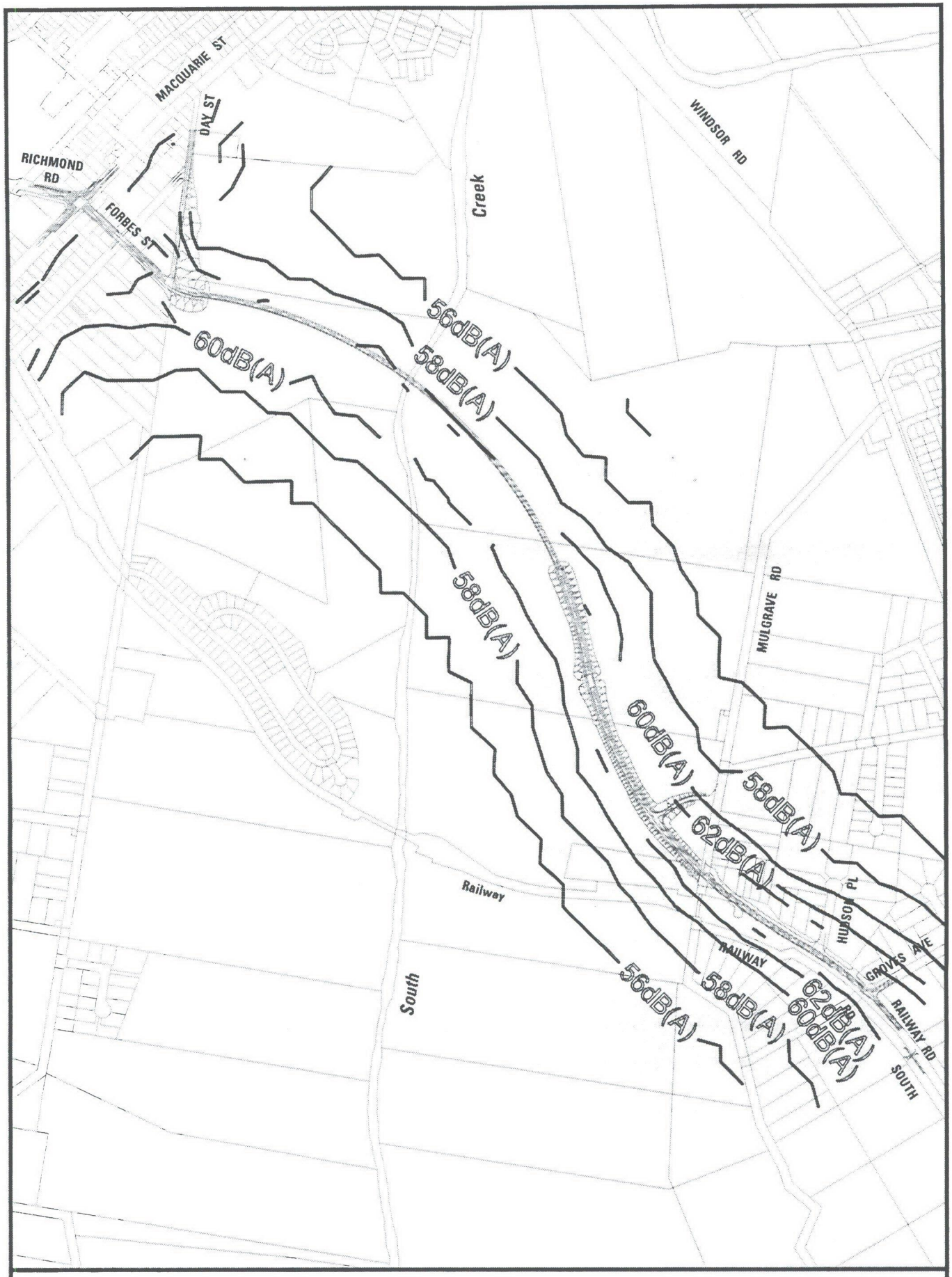
Distance m	Predicted L_{Amax} Noise Level	
	Truck pass-by and acceleration	**Truck applying engine compression brakes**
7	80 – 87	85 – 94
10	77 – 84	82 – 91
20	71 – 78	76 – 85
30	67 – 74	72 – 81
50	63 – 70	68 – 77
100	57 – 64	62 – 71
200	51 – 58	56 – 65
400	45 – 52	50 – 59



WINDSOR
FLOOD EVACUATION ROUTE

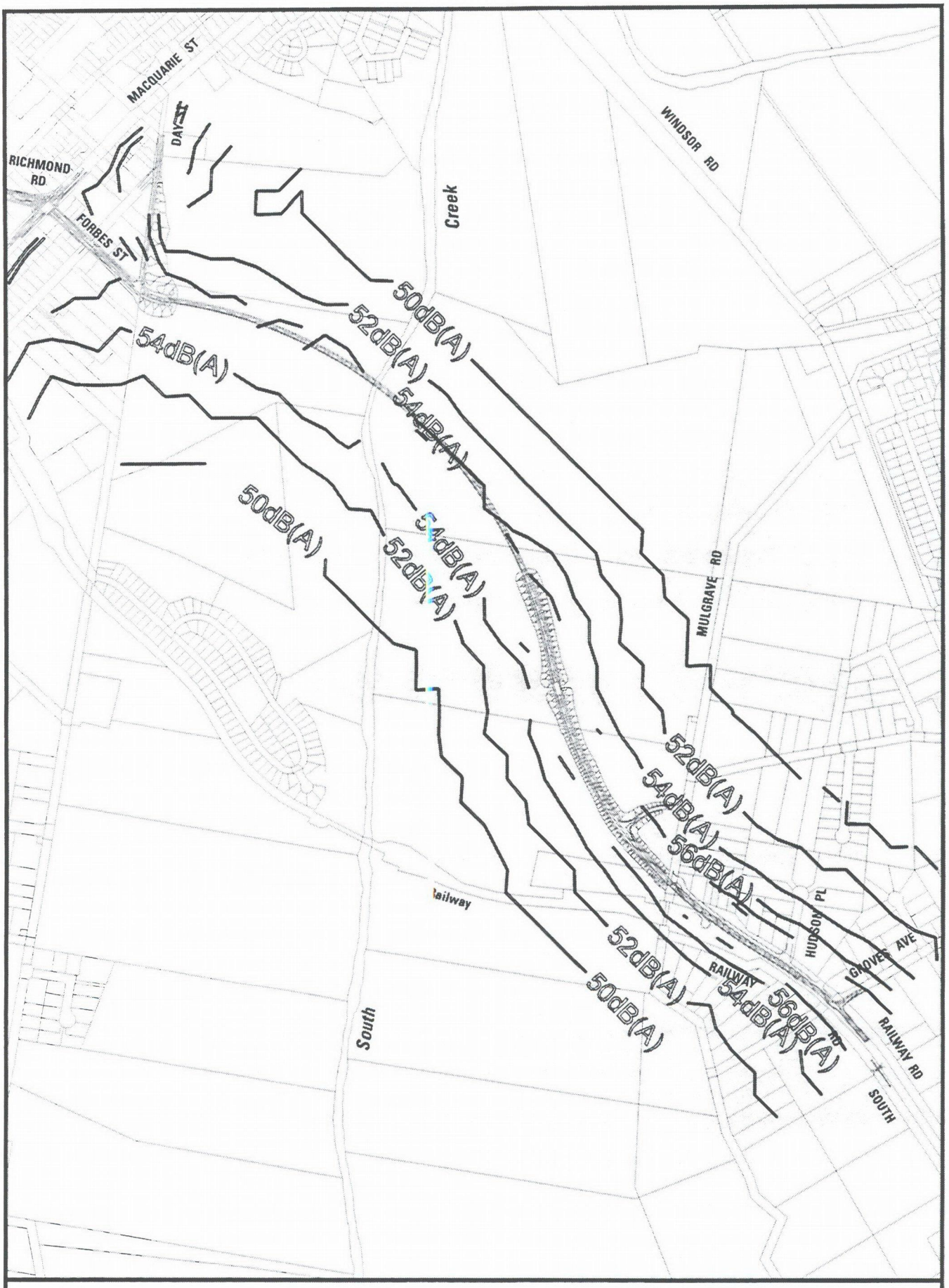
N

FIGURE 3
NOISE CRITERIA EXCEEDANCE LOCATIONS



WINDSOR
FLOOD EVACUATION ROUTE

FIGURE 4
LAeq15hr NOISE CONTOURS (2016)



WINDSOR
FLOOD EVACUATION ROUTE

FIGURE 5
LAeq9hr NOISE CONTOURS (2016)

10.0 POSSIBLE CONTROL MEASURES

The predictions in Section 9.1 show that road traffic noise at receivers along the Day Street, Mileham Street, Forbes Street and the bypass will exceed the baseline assessment goals. For the properties on Day Street and Mileham Street the noise modelling has shown that the existing traffic noise levels will decrease and the resultant noise levels satisfy the allowance goals (existing + 0.5dB(A)).

The following section of the report presents preliminary options of types of noise controls that may be appropriate for the project, providing they are shown to be practical with regard to their effectiveness, minimum secondary impacts such as safety, loss of views, over shadowing, loss of access to properties and cost effectiveness.

It should be noted that the recommended control options are preliminary and any noise control should be selected in consultation with appropriate authorities and individual land owners.

10.1 Road Side Acoustic Barriers

Noise reduction achieved by acoustic barriers depends on the locations and heights of the source, receivers and barriers and could typically be as high as 15dB(A). The barriers could be constructed in the form of concrete panels, masonry and/or earth mounds and should be absorptive to minimise the increase in traffic noise at receivers opposite the barriers. Where relevant, the barriers should be located on the embankment or same level as the finished road surface.

10.1.1 Receivers East of Day Street

Table 12 presents details of barrier heights required to achieve the noise assessment goals of 55dB(A) $L_{Aeq,15hr}$ and 50dB(A) $L_{Aeq,9hr}$ at residences east of Day Street.

Table 12 Barrier Heights and Locations - East of Day Street

Chainage	Barrier Height
Barriers along east-bound lane	
1260m – 1640m	3m
1900m – 2400m	6m
Barriers along west-bound lane	
1880m – 2260m	4.5m
2260m – 2680m	6m

10.1.2 Receivers along Forbes Street

Table 13 presents details of barrier heights required to achieve the allowance noise assessment goal (existing traffic noise levels +0.5dB(A)) at residences along Forbes Street. To satisfy the baseline assessment goals the barrier heights would need to be in excess of between six to eight (6-8) metres.

From a preliminary assessment of the barriers along Forbes Street and as a result of safety aspects, visual impact, property access, overshadowing, reliance on closing gates to maintain acoustic integrity of the barrier, generally it is not considered practical to provide barriers of 6-8m high to satisfy the assessment goals. However, where feasible lower barrier heights could be considered as they would provide some benefits.

Table 13 Barrier Heights and Locations - Forbes Street

Chainage	Barrier Height
Barriers along east-bound lane	
From Macquarie Street (with adequate return) to Chainage 240m of Bypass	5m
Barriers along west-bound lane	
From Macquarie Street (with adequate return) to Chainage 180m of Bypass	3.5m

10.2 Open Graded Asphalt

It is recognised that open graded asphalt road surfaces can provide noise reductions in the order of 2-3dB(A). This type of road surfaces is normally adopted where free flow traffic conditions occur. In regard to the proposed flood evacuation route, open graded asphalt is not considered practical due to ongoing maintenance requirement and therefore has not been recommended for the project.

10.3 Treatment to Individual Dwellings

Treatment to individual properties or dwellings could include closing exposed door and window openings, upgrading exposed building facade/glazing, sealing off openings and exposed wall vents and provisions for ventilation support.

Noise reduction achieved from a typical building façade with an open window is in the order of 10dB(A) depending on the building construction type. For closed windows, noise reductions of 20-25dB(A) are achievable. For double glazed window systems in a normal building facade, noise reductions in the order of 25-30dB(A) can be achieved with windows closed. With application of acoustic seal to windows, doors and sealing off wall openings, further outdoor to indoor noise reductions of 2-5dB(A) can be achieved. It is noted that treatments to buildings will provide no benefit to the outdoor acoustic amenity.

Acoustic treatment could be considered for a number of individual dwellings along Forbes Street where noise barriers are not considered practical due to road intersections and requirements for access to driveways, etc. However, it is noted that a number of light weight dwelling constructions may be inadequate in terms of controlling road traffic noise intrusion or have heritage listing that may restrict building alterations.

10.4 Prioritising Noise Controls

Referenced to the RTA ENMM *(Practice Note III),* where the L_{Amax} noise levels exceed the assessment protocol the locations of exceedance should be taken into consideration for prioritising control measures. The main areas identified from this assessment that require

further consideration during the design development phase of the project include Forbes

Street and Railway Street North.

10.5 Future Planning

The local Council has a role in ensuring that road traffic noise is considered when determining rezoning and development applications under the provisions of the Environmental Planning and Assessment Act, 1979. It is recommended that the Council consider planning strategies for future land zoning and development adjacent to the road.

10.6 Future Development in Traffic Noise Control

As part of the government long-term plan to reduce road traffic noise, strategies that are being considered include controlling noise emissions from individual vehicles, developing programs to monitor and control noise vehicles, controlling noise from trucks and engine brakes and reducing traffic speed. The effective implementation of these plans will assist in further reductions to road traffic noise.

11.0 CONSTRUCTION NOISE AND VIBRATION

11.1 Overview

The road construction program has not been established nor final road plans approved. Details of construction program and activities will be determined when the proposal is approved and a contractor engaged. However, it is envisaged that the construction activities would be based on established road design and construction procedures. To evaluate the likely level of noise and vibration emitted from the construction activities, the following construction phases and plant schedules have been considered.

11.1.1 Preliminary Site Works

Preliminary site works would involve relocation of services and access adjustments to local roads. At this stage it is envisaged that trucks, a grader and an excavator would be used to excavate and transport materials during the preliminary site works.

11.1.2 Earthworks

The main construction activities envisaged during the main earthworks phase of the project will involve excavation, filling and compaction. During this phase of the project, dozers, scrapers, excavators, graders and compactors would be used to excavate, transport, spread, level and compact the material to form the road base and bridge embankments

11.1.3 Sub-base Preparation

Following earthworks, the road sub-base would be prepared by a fleet of trucks and rollers.

11.1.4 Road Surfacing

Laying of the bitumen pavement would be carried out by asphalt laying machines and up to three trucks and two rollers.

11.1.5 Bridge Construction

It is envisaged that the main plant and equipment used during the construction of the bridge would consist of a pile driver, trucks, generators, pumps, cranes and compressors.

11.1.6 Ancillary Construction Facilities

Ancillary construction facilities include site compounds and offices; plant and equipment storage areas; stockpile sites; and waste management and storage areas. The locations for these facilities would be identified by the construction contractors, and normally located within the road reservation, or by prior agreement with individual property owner on private property.

At this time, suitable locations for the site compounds could include land adjacent to Forbes Street or Day Street for work west of South Creek; and land adjacent to Mulgrave Road for work east of South Creek. A bridge construction site compound could also be established on the approach embankment at the eastern side of the bridge over South Creek.

11.1.7 Construction Schedule

Table 14 presents a summary of the likely construction plant and construction schedules.

Table 14 Operations and Equipment Required

Operation	Plant Type	No. Required (Av max)
Preliminary Site Works	Grader	1
	Excavator	1
	25T trucks	2

Table 14 Operations and Equipment Required (contd)

Operation	Plant Type	No. Required (Av max)
Earthworks	25T trucks	3
	Excavator	1
	Scrapers	2
	Dozers	1
	Compactors	2
Sub-base Preparation	Trucks	3
	Asphalt Laying Machine	1
	Rollers	2
Road Paving	Asphalt trucks	3
	Asphalt Laying Machine	1
	Rollers	2
Construction of Bridge	Pile Driver	1
	Cranes	2
	Trucks	2
	Generator	2
	Truck	3
	Pumps	2

11.2 Construction Hours

To minimise noise impacts during construction, the activities would generally be restricted to between 7:00am and 6:00pm, Monday to Friday, and 8.00am to 1.00pm on Saturday. With prior approval from the EPA and Council and notification to residents construction outside these hours could be undertaken provided the construction noise did not give rise to an unreasonable nuisance or disturbance to the occupiers of nearby residential dwellings.

11.3 Assessment Goals

11.3.1 Goals for Noise Assessment

Goals for assessment of noise from construction sites are recommended in the EPA's Environmental Noise Control Manual (1994). The goals depend on the duration of the construction activities.

- for construction periods less than four weeks, the $L_{A10,15min}$ noise levels should not exceed the L_{A90} background by more than 20dB(A); and
- for construction periods between four weeks and 26 weeks, the $L_{A10,15min}$ noise levels should not exceed the L_{A90} background by more than 10dB(A).

It is noted that the EPA noise goals are generally difficult to satisfy for road construction projects. Hence, where exceedances are predicted noise management plans are developed to minimise potential impacts.

Goals for assessment of construction noise established from measured background L_{A90} noise levels are summarised in *Table 15* below.

Table 15 Goals for Assessment of Construction Noise
dB(A) re 20 $\times 10^{-6}$ Pa

Assessment Location	Daytime Background L_{A90} Noise Level		$L_{A10,15min}$ Noise Goal	
	Range	RBL	Short-term	Medium-term
Receivers near Railway Road North and Railway Road South	39 – 53	42	62	52
Receivers near Mulgrave Road	40 – 54	43	63	53
Forbes Street	48 – 56	49	69	59
Day Street	40 – 50	43	63	53
Chisholm Street	37 – 50	40	60	50

11.3.2 Goals for Vibration Assessment

i Human Comfort and Disturbance

Goals for assessment of vibration in buildings are recommended in the Environmental Noise Control Manual (1994, Chapter 174). The EPA base their goals on British Standard BS6472 (1992) and recommend that vibration levels measured on the floor of a residence should not exceed base Curve 60 (approximately 8mm/s). Where the levels are less than Curve 60 and exceed Curve 2 (approximately 0.3mm/s), the EPA recommend that the activities be restricted to between 7:00am and 6:00pm Monday to Friday and 7:00am and 1:00pm Saturday.

ii Building Occupants

British Standard BS6472 (1992) provides guidelines for assessment of potential disturbance to persons exposed to continuous and intermittent vibration in buildings. *Table 16* provides the vibration levels that may give rise to various reactions. For comparison, continuous vibration is normally felt at about 0.15mm/s and becomes noticeable at approximately 1mm/s.

Table 16 Vibration Levels for Assessment of Human Comfort

Type of Occupancy	Time	Vibration Level (mm/s)			
		Continuous Vibration		Intermittent Vibration	
		Vertical	Horizontal	Vertical	Horizontal
Residential	Day	0.3 to 0.66	0.8 to 1.6	8.4 to 12.6	24 to 36
	Night	0.2	0.6	2.8	8

Ref: British Standard BS6472

iii Building Structures

German Standard DIN4150 Part 3 (1986) provides guidelines for evaluating the effects of vibration on structures. The values recommended in the standard are summarised in *Table 17*. The values are the maximum vibration levels measured in any direction at the building foundation.

Table 17 Safety Limits for Structural Damage

Type of Structure	Vibration Level (mm/s)		
	< 10Hz	10Hz to 50Hz	50Hz to 100Hz
Commercial/industrial buildings or buildings with similar design	20	20 to 40	40 to 50
Dwellings and buildings of similar design and/or use	5	5 to 15	15 to 20
Structures of great intrinsic value (eg. buildings under preservation)	3	3 to 8	8 to 10

Ref: German Standard DIN4150

From the above, it is recommended that maximum vibration levels generated by construction equipment should not exceed of 5mm/s and 3mm/s for normal and sensitive (heritage) residential structures to minimise possible structural damage. The Standard states that this level is a "safe limit" up to which no damage due to vibration effects has been observed for the particular class of building. "Damage" is defined by DIN4150 to include minor non-structural superficial cracking in cement render, enlargement of existing cracks and separation of partitions or intermediate walls from load bearing walls.

iv Building Contents

The threshold for visible movement of susceptible building contents (eg. plants, hanging pictures, etc.) is approximately 0.5mm/s. Audible rattling of loose objects (eg. crockery) generally does not occur until levels of about 0.9mm/s.

11.4 Noise and Vibration from Construction Equipment

11.4.1 Noise Emissions from Construction Equipment

Sound power levels of equipment for the envisaged construction activities are summarised in *Table 18*. These are established from data presented in the Australian Standard AS2436-1981 and previous studies conducted by Atkins Acoustics.

Table 18 Construction plant and Sound Power Levels
dB(A) re 20 $\times 10^{-6}$ Pa

Plant Description	Plant Type	Sound Power Level

Dozer	Caterpillar D7, D9	113
Front End Loader	Wheeled	110
Scraper	Caterpillar 651	110
Grader	Caterpillar 16	110
Compactor	Caterpillar 825	110
Vibratory Roller	10-12 tonne	110
Water Cart		106
Excavator	Kato 750	107
Rock Breaker		120
Pile Driver		120
Hydraulic Drill		115
Truck		106
Crane	Truck mounted	110
Compressor	600 CFM	100
Backhoe		108
Grader	Caterpillar	106
Tip Truck		106
Concrete Paver		115
Concrete Truck		108
Concrete Pump		109
Concrete Vibrators		105

11.4.2 Vibration from Construction Equipment

Table 19 provides typical ground vibration levels generated by construction equipment.

Table 19 Vibration Levels from Construction Equipment

Plant Description	Vibration Level (mm/s)	
	5m	20m
Dozer	2	0.2
Pile Driver	10	4
Large Rock Breaker	4	0.4
Vibrating Compactor	20	2

11.5 Predicted Noise and Vibration Impacts

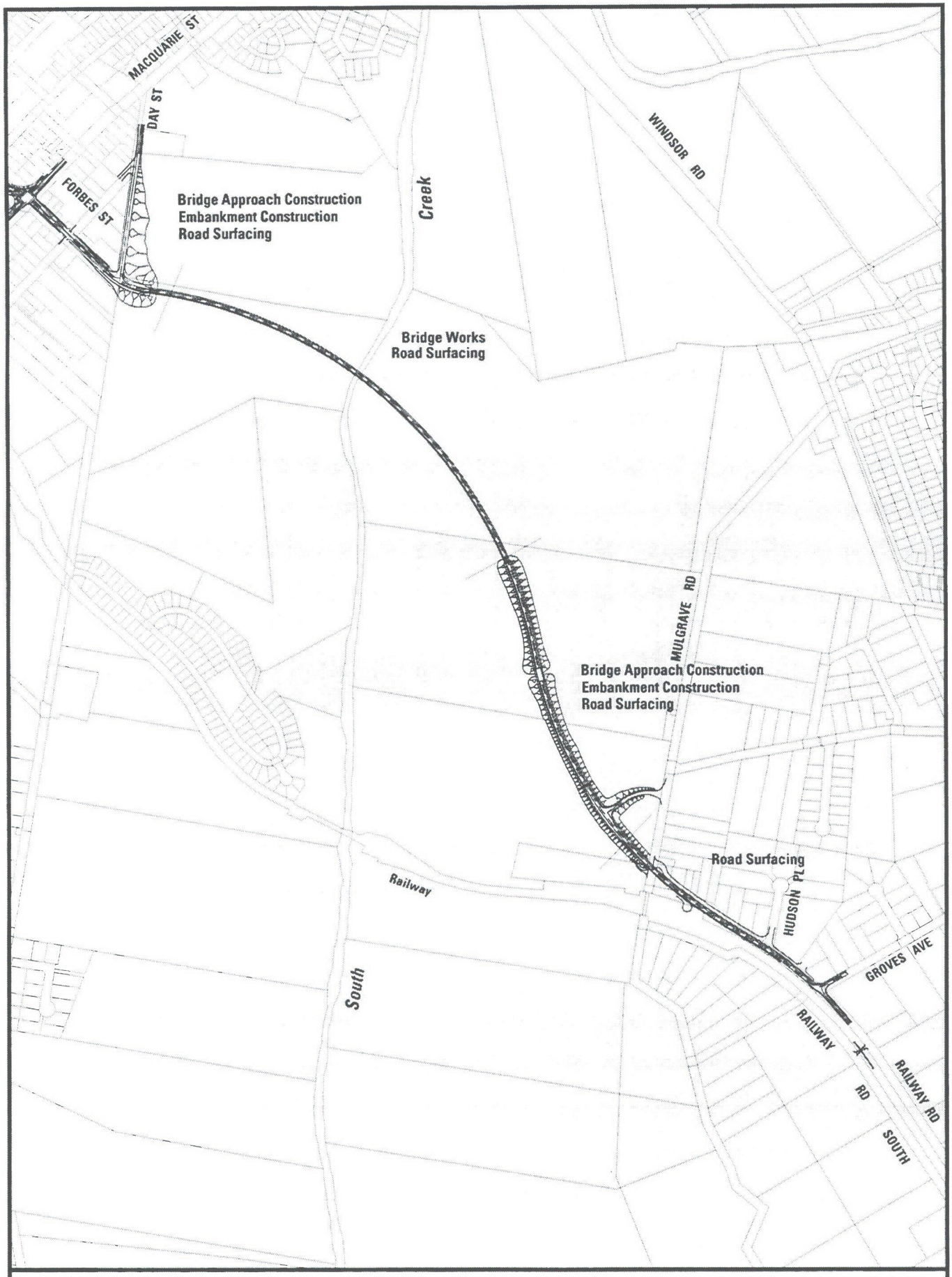
11.5.1 Noise Impacts from Construction Equipment

Figure 6 presents the locations where typical construction activities are likely to

take place. Figure 7 presents the predicted noise contours from typical construction activities, using the ENM Environmental Noise Model.

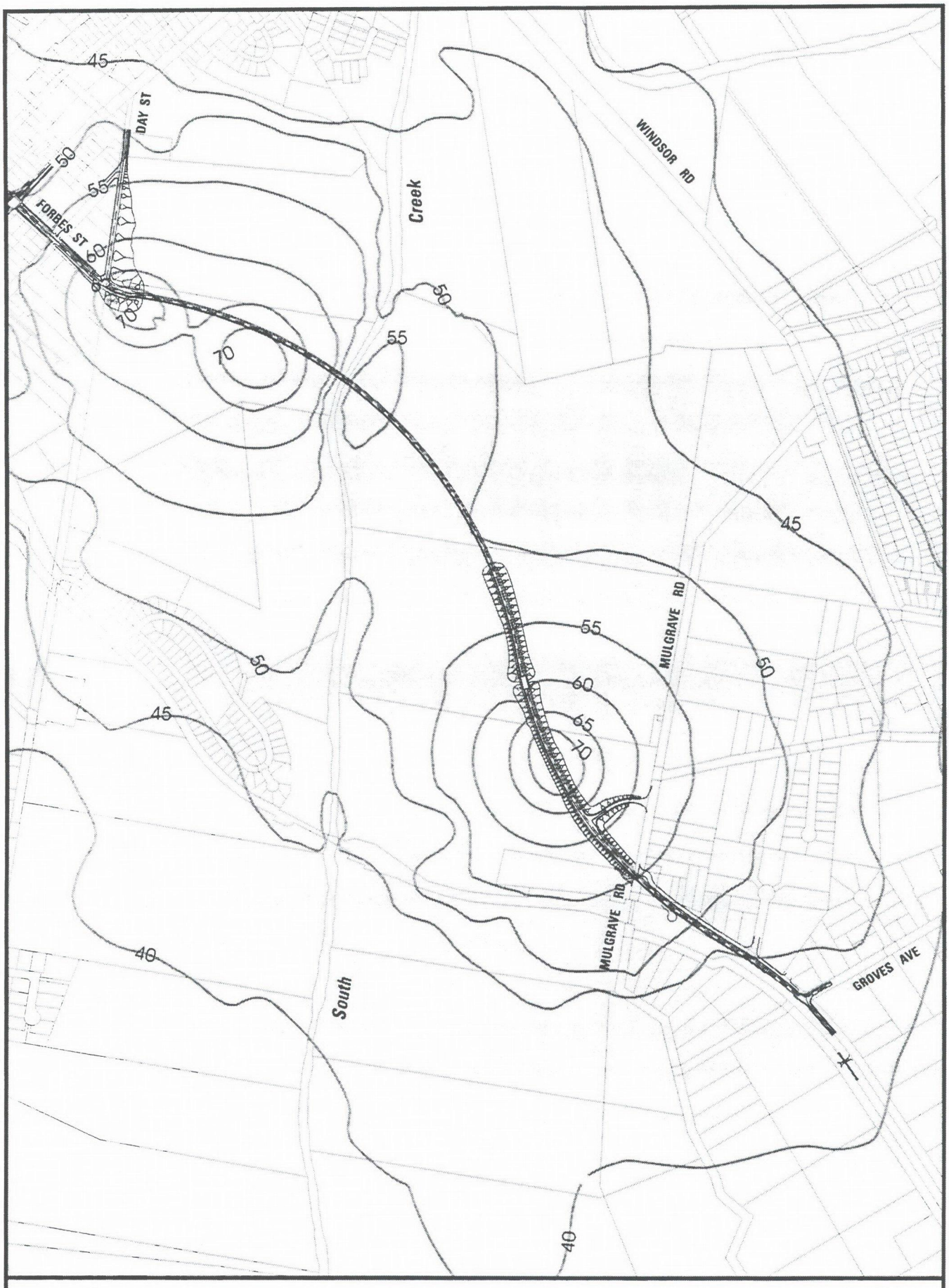
It can be seen from Figure 7 that as construction equipment moves towards the receivers along northern end (Forbes Street) and southern end of the proposed development, construction noise levels exceed the assessment goals for both the short-term and medium-term. Based on the construction scenario presented in Figure 7, the predicted noise levels at receivers in the vicinity of Forbes Street, Chisholm Place and Mulgrave Street range between 70-85dB(A), 50-65dB(A) and 60-75dB(A) respectively.

To minimise impacts during construction, a site environmental management plan (EMP) shall be prepared. Section 11.6 presents a summary of issues that should be addressed in the EMP.



WINDSOR
FLOOD EVACUATION ROUTE

FIGURE 6
LOCATION OF CONSTRUCTION ACTIVITIES



WINDSOR
FLOOD EVACUATION ROUTE

FIGURE 7
CONSTRUCTION STAGE
NOISE CONTOURS

11.5.2 Vibration Impacts from Construction Equipment

Table 20 provides a summary of predicted vibration levels that could be generated from various plant equipment.

Table 20 Typical Plant Vibration Levels

Plant Description	Vibration Level (mm/s)		
	5m	20m	40m
Dozer	2	0.2	0.02
Large Rock Breaker	4	0.4	0.15
Pile Driver	10	4	1.0
Vibratory Compactor	20	2	0.3

The main sources of ground vibration from construction equipment are associated with the rock breakers, pile drivers and compactors. Vibration levels generated by rock breakers and dozers are unlikely to exceed the "safe limit" of 3mm/s for structural damage to sensitive residential buildings. At distances within approximately 20-30m from the pile driving or compacting, the "safe limit" for structural damage to residential buildings could be exceeded.

Construction program and techniques are conceptual at this stage. It is recommended that investigation be carried out at the detailed design stage after approval is granted to ensure excessive vibration levels are not encountered. If required, control measures such as alternative construction methods should be implemented. These could include replacing impact driven piles with bored piles and lower settings on vibratory compactors.

Ground vibration levels from construction equipment would be felt by residents at some stage. In accordance with NSW EPA guidelines, the construction activities should be restricted to between 7:00am and 6:00pm Monday to Friday, 7:00am and 1:00pm Saturday where vibration levels exceed 0.3mm/s.

11.6 Recommendation

To minimise noise and vibration impacts during construction, an Environment Management Plan (EMP) should be prepared to address construction noise and vibration. As part of the EMP, the following issues should be considered:

- establishment of a construction noise and vibration control plan;
- selection of plant and equipment on acoustic performance, where practical;
- implementation of a monitoring program to ensure that construction noise and vibration are controlled and that best possible practices are implemented;
- preparation of dilapidation reports on sensitive structures within 40m of any rock breaking, piling or ground compaction; and
- implementation of an information program to inform affected residents of the construction program and periods when noise and vibration levels could exceed the recommended assessment guidelines.

12.0 SUMMARY

This report presents the results and findings of a noise and vibration assessment for the proposed Windsor Flood Evacuation Route across South Creek.

Site investigations and noise measurements were conducted to quantify the existing ambient noise levels and established goals for assessment of future traffic noise impacts, construction noise impacts and validation of noise prediction model.

The EPA's Environmental Criteria for Road Traffic Noise recommends baseline and allowance goals for assessing of road traffic noise. The baseline noise assessment goals are 55dB(A) $L_{Aeq,15hr}$ and 50dB(A) $L_{Aeq,9hr}$ for residential properties along the flood evacuation route. For residential receivers exposed to high existing traffic noise, the allowance goals require the existing road traffic noise levels not to exceed by more than 0.5dB(A) for new and redeveloped roads at opening and 10 years after the road opening. For sensitive land uses such as the hospital, the allowance assessment goals require the existing road traffic noise levels not to exceed by more than 2dB(A) following the road redevelopment.

Without abatement measures, the predicted road traffic noise levels exceed the assessment guidelines by up to 11dB(A) and 15dB(A) at receivers along the flood evacuation route and Forbes Street respectively. During the design development stage of the project, a detailed cost benefit analysis shall be conducted to establish the most effective mitigation measures which could include construction of roadside barriers and provision of acoustic treatments to dwellings.

Comparison of forecast traffic volumes to existing traffic volumes, traffic noise levels at receivers along Day Street and at Hawkesbury District Hospital are predicted to decrease by 1-2dB(A) following the proposed development. This is due to traffic reduction on connecting roads, namely Macquarie Street, Mileham Street and Day Street.

Traffic on Windsor Road is predicted to reduce following the proposed development and hence noise levels at receivers adjacent to Windsor Road between McGrath Road and Pitt Town Road, McGrath Hill are predicted to reduce as a result of the development.

Possible noise mitigation measures have been considered to control road traffic noise levels at receivers along the flood evacuation route and Forbes Street. However from a preliminary assessment of the barriers along Forbes Street and as a result of safety aspects, visual impact, property access, overshadowing, reliance on closing gates to maintain acoustic integrity of the barrier, it may not be considered practical to provide barriers to satisfy the assessment goals. Additionally acoustic treatment could be considered for a number of individual dwellings along Forbes Street where noise barriers are not practical. However, it is noted that a number of light weight residential dwelling constructions may be inadequate in terms of controlling road traffic noise intrusion or have heritage listing that may restrict building alterations.

The road design is conceptual at this stage, hence it is recommended that noise control options be reviewed during the final road design. Final noise control treatments will depend on feasibility and practicality evaluations. These will be assessed and evaluated during the final road design and selected by following the process described in the RTA Environmental Noise Management Manual, and selected in consultation with the EPA, individual property owners and the approval of Planning NSW.

Possible sleep disturbance impacts are address in the report using the *Practice Note III* in the RTA Environmental Noise Management Manual. The findings from the assessment show that predicted L_{Amax} noise levels exceed the recommended assessment protocol. The main residential areas identified from the assessment that require further consideration include Forbes Street and Railway Road North.

As part of the NSW Government long term plan to reduce road traffic noise, strategies that are being considered include controlling noise emissions from individual vehicles, developing programs to monitor and control noisy vehicles, and the control of noise from trucks and engine brakes. The progressive and effective implementation of these plans will assist in further reductions to road traffic noise from the proposal.

In regard to future planning, the Council has a role in ensuring that road traffic noise is considered when determining rezoning development and building applications under the provisions of the Environmental Planning and Assessment Act, 1979, and Local Government Act, 1993. It is recommended that Council consider planning strategies for any future residential development adjacent to the proposal.

It is recommended that following the opening of the road, post construction noise monitoring be undertaken to confirm compliance with the project assessment goals and the need for any further noise control treatments.

For noise and vibration from construction activities, the guidelines in the EPA's Environmental Noise Control Manual and British Standard BS6472 were used to assess possible disturbance to human comfort. Guidelines in the German Standard DIN4150 were used to for the assessment of possible structural damage to building structures generated by construction equipment.

At present, the proposal is conceptual and no final plan or road contractor have been established. Similarly, no final road construction program or construction techniques have been determined. The final construction details will be established when the proposal is approved and a contractor engaged.

The assessment has shown that during the construction there will be localised adverse noise impacts where road construction activities take place near residential receivers.

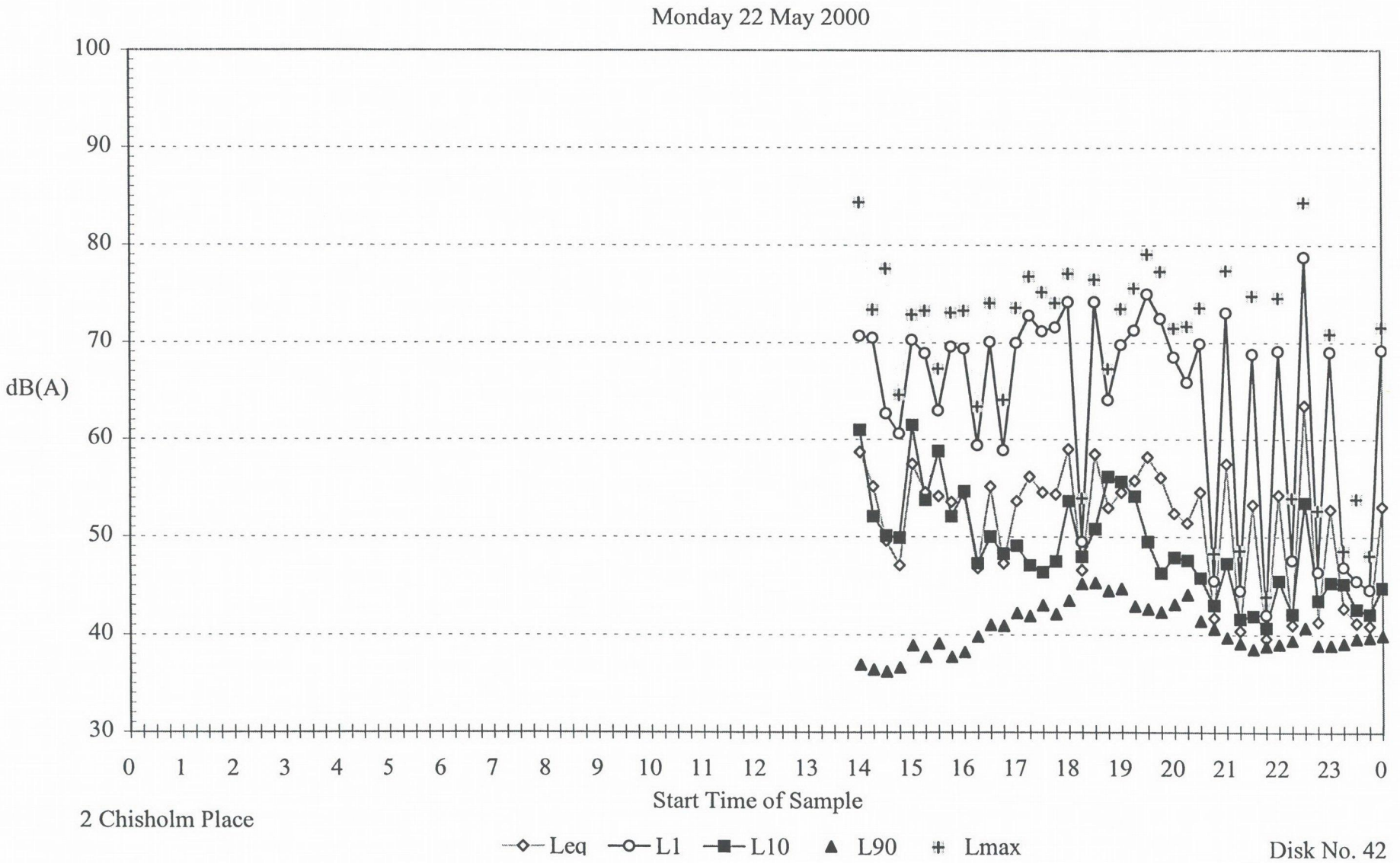
Vibration levels generated by construction equipment could exceed the "safe limit" for

structural damage to residential buildings. It is recommended that dilapidation reports on structures within 40m of any rock breaking, piling or ground compaction be prepared prior to construction activities undertaken.

Noise and vibration impacts from these activities can be minimised through planning, programming and consultation with the occupiers of the affected properties. To ensure that construction noise and vibration impacts are minimised during construction, it is recommended that as part of the construction contractors undertakings, an Environmental Management Plan be prepared to address and control construction noise and vibration.

APPENDIX 1: NOISE MONITORING RESULTS

Ambient Sound Pressure Levels



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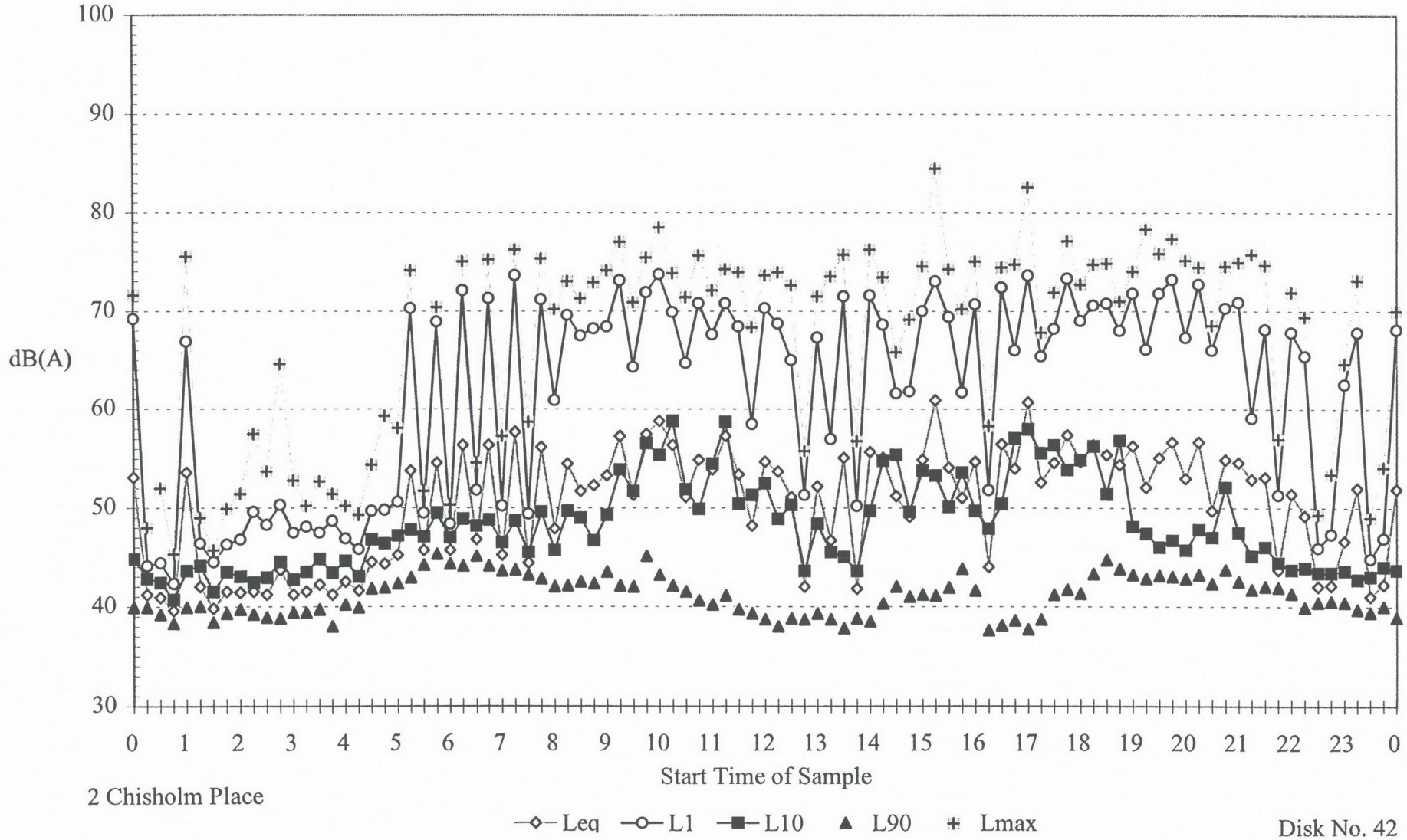
Ambient Sound Pressure Levels

Tuesday 23 May 2000



2 Chisholm Place

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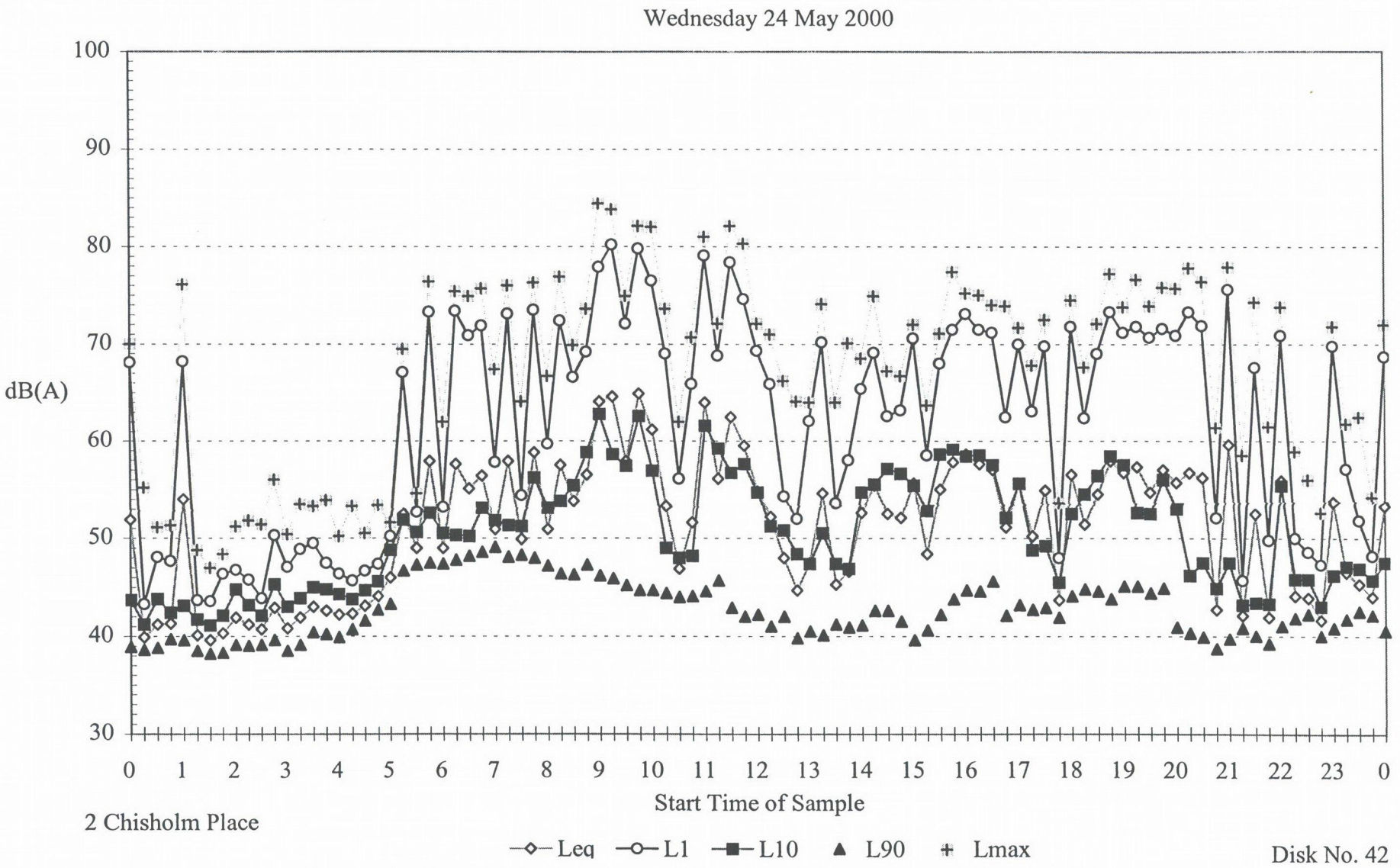
Ambient Sound Pressure Levels



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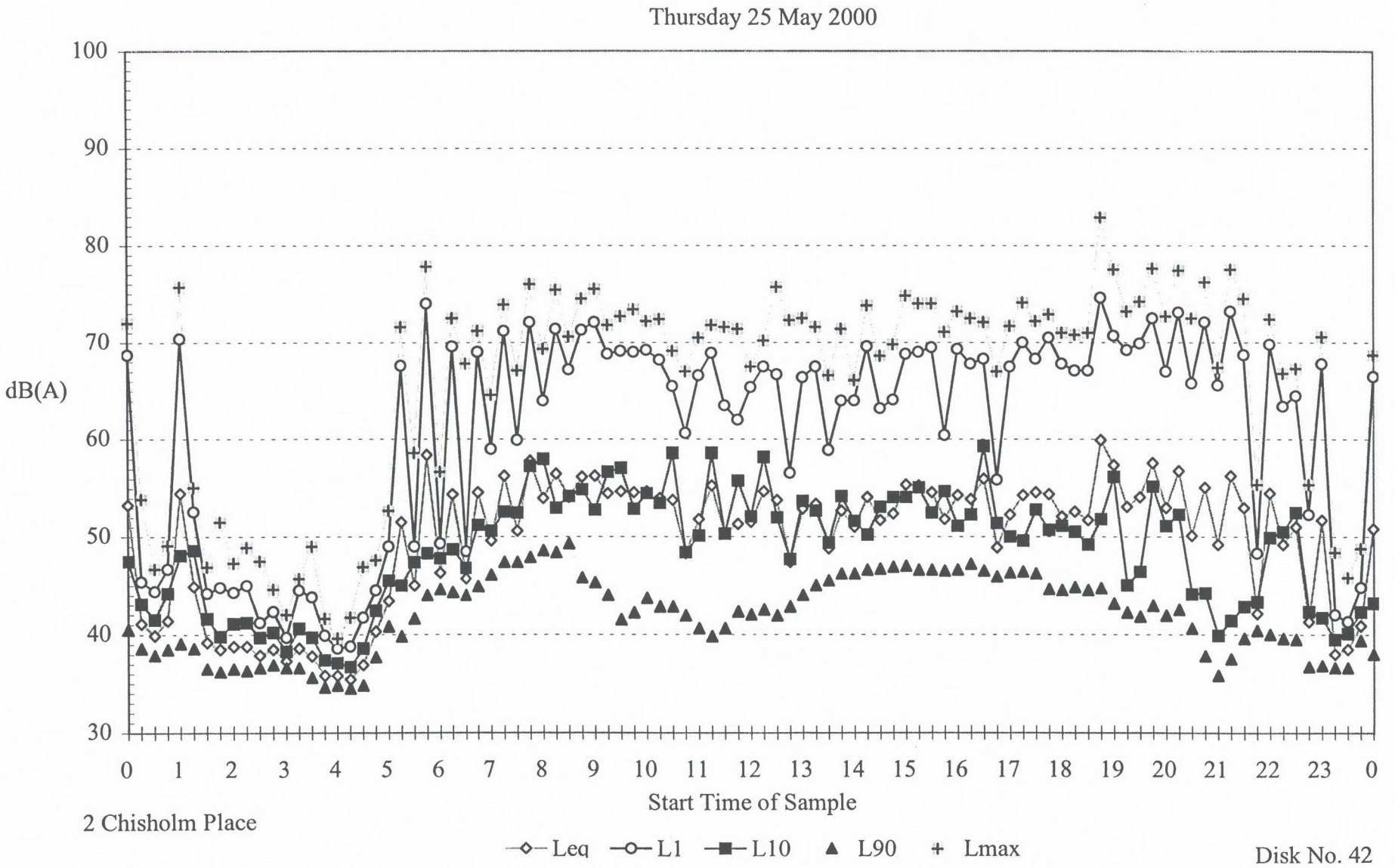
Ambient Sound Pressure Levels



2 Chisholm Place

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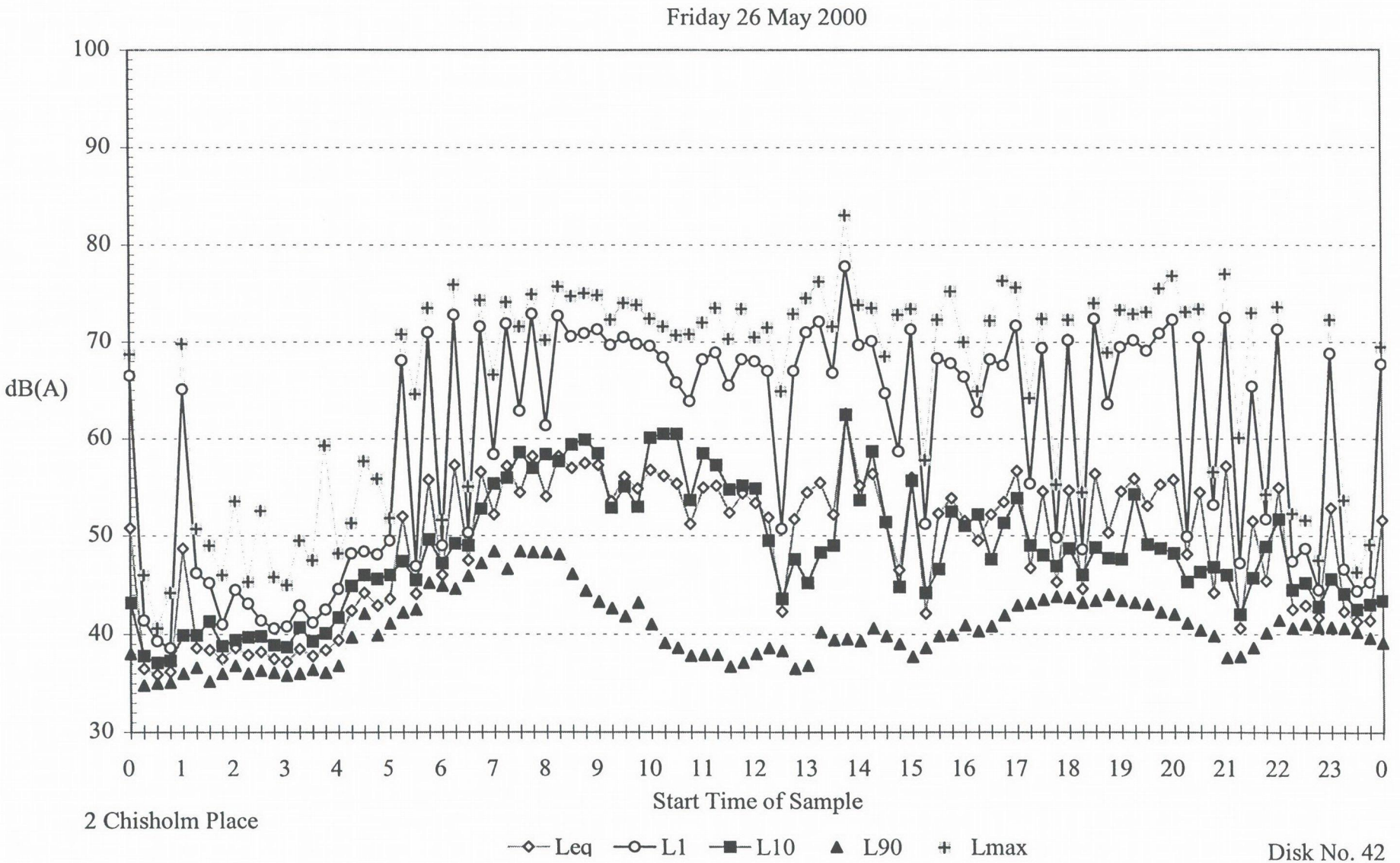
Ambient Sound Pressure Levels



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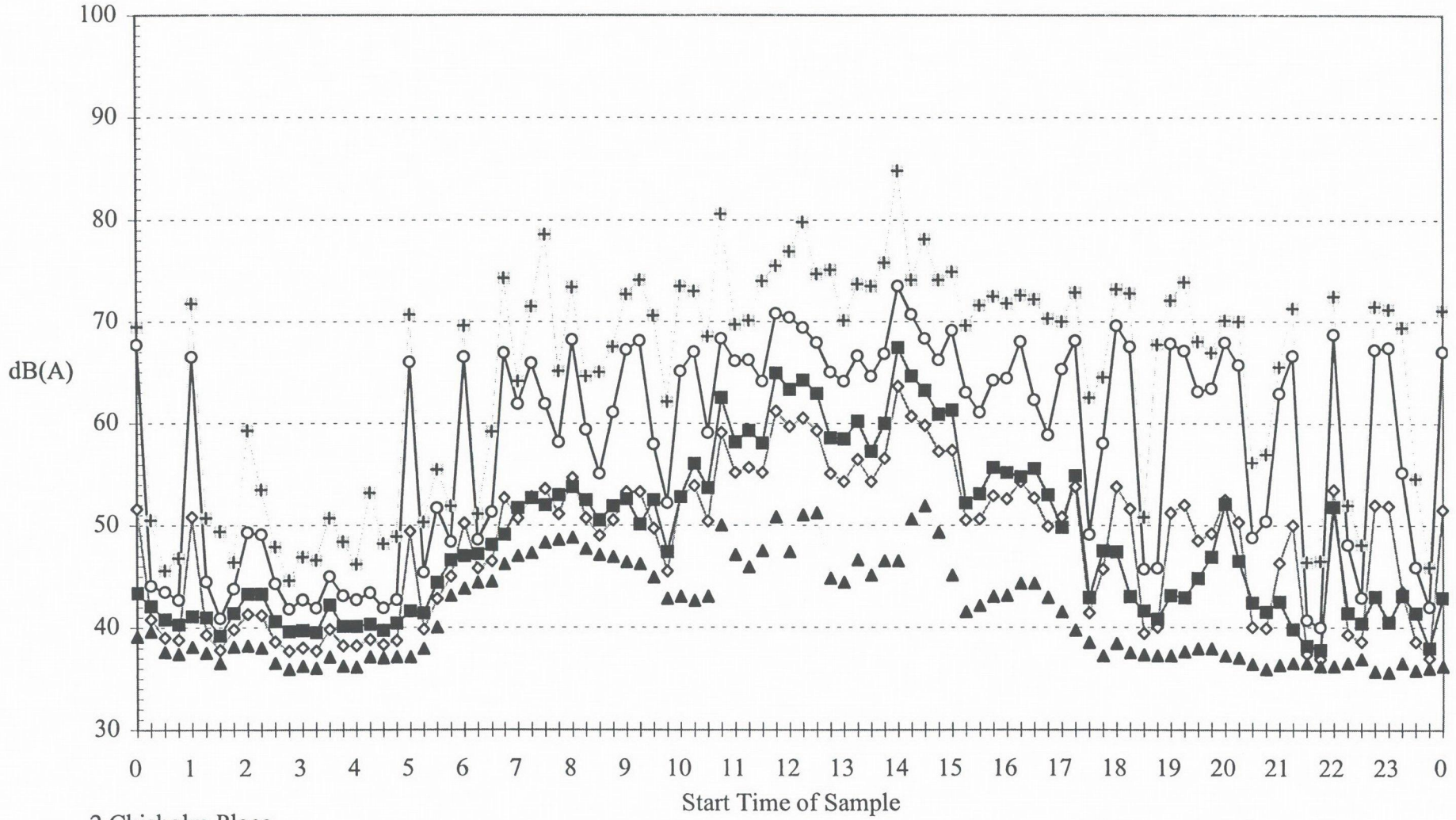
Ambient Sound Pressure Levels

Saturday 27 May 2000



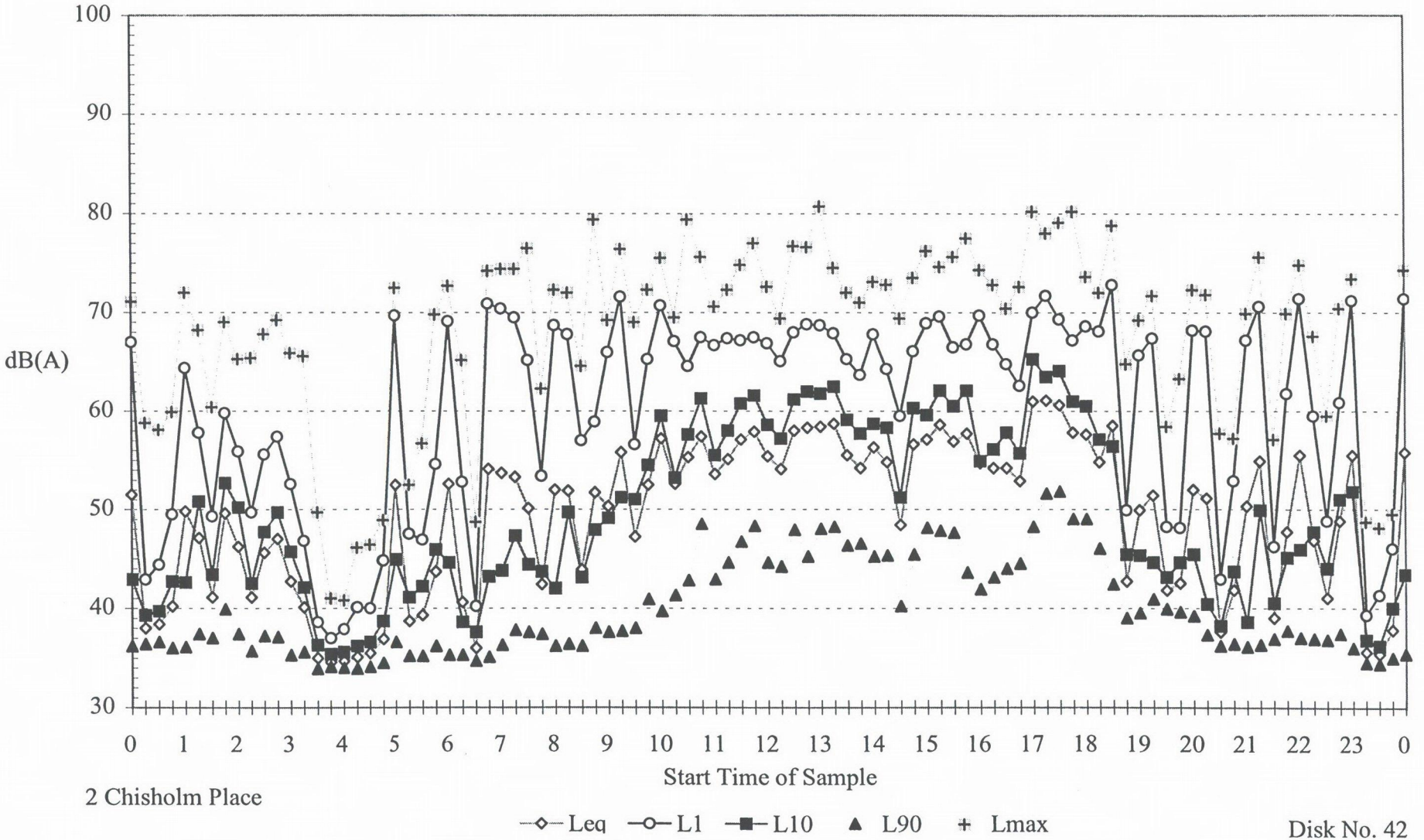
2 Chisholm Place

Disk No. 42

5030r2.xls

Ambient Sound Pressure Levels

Sunday 28 May 2000



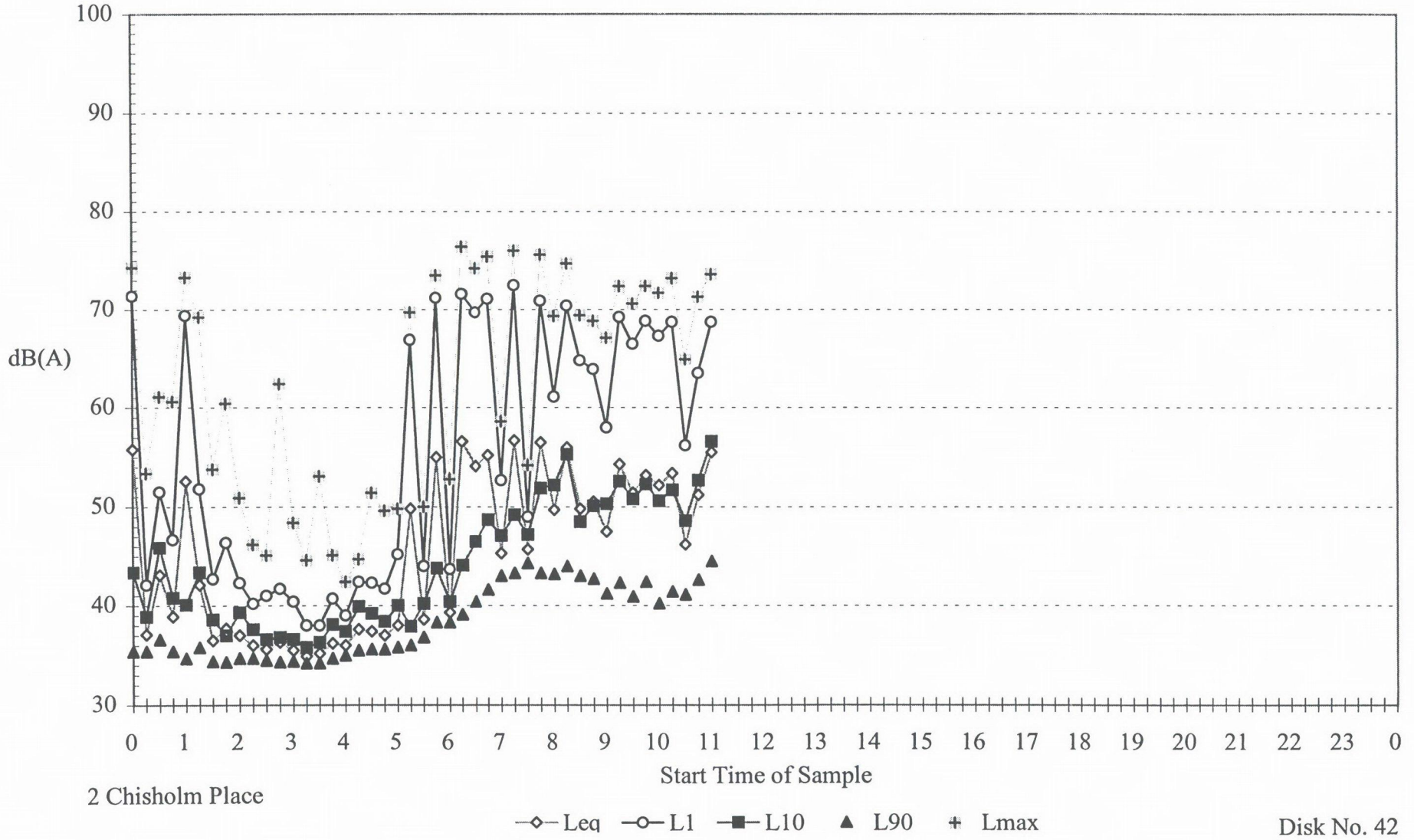
2 Chisholm Place

Disk No. 42

5030r2.xls

Ambient Sound Pressure Levels

Monday 29 May 2000

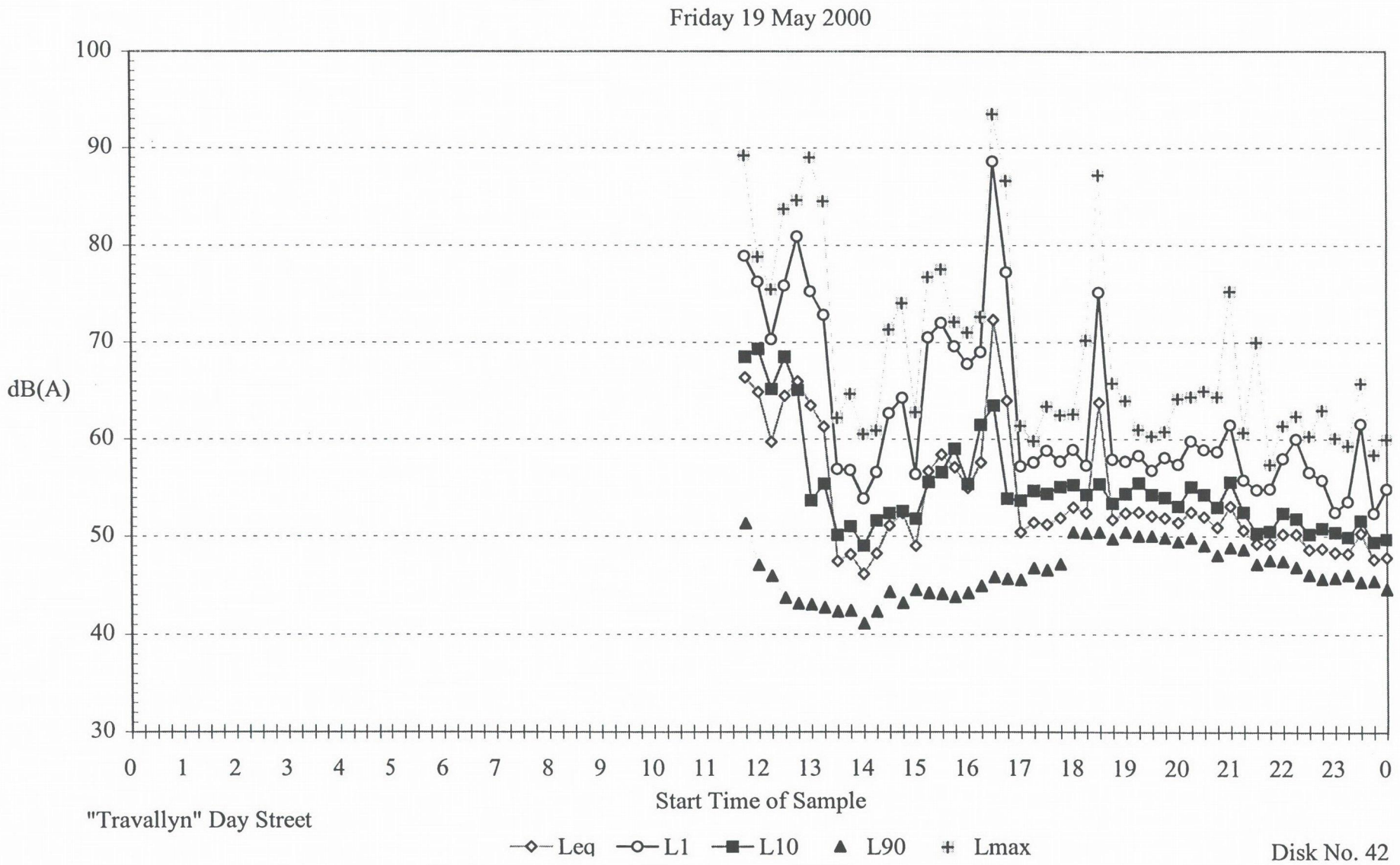


2 Chisholm Place

Disk No. 42

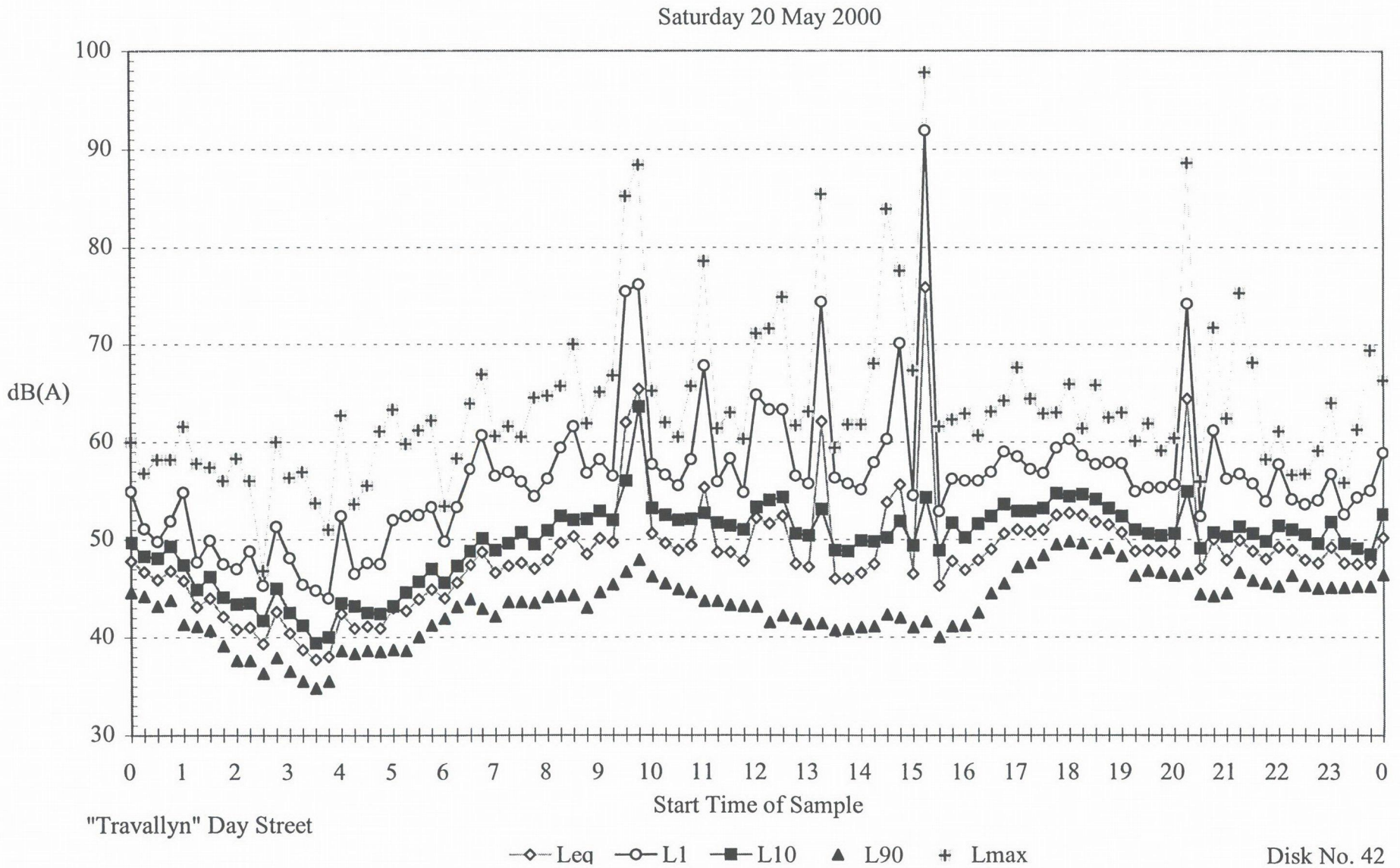
5030r2.xls

Ambient Sound Pressure Levels



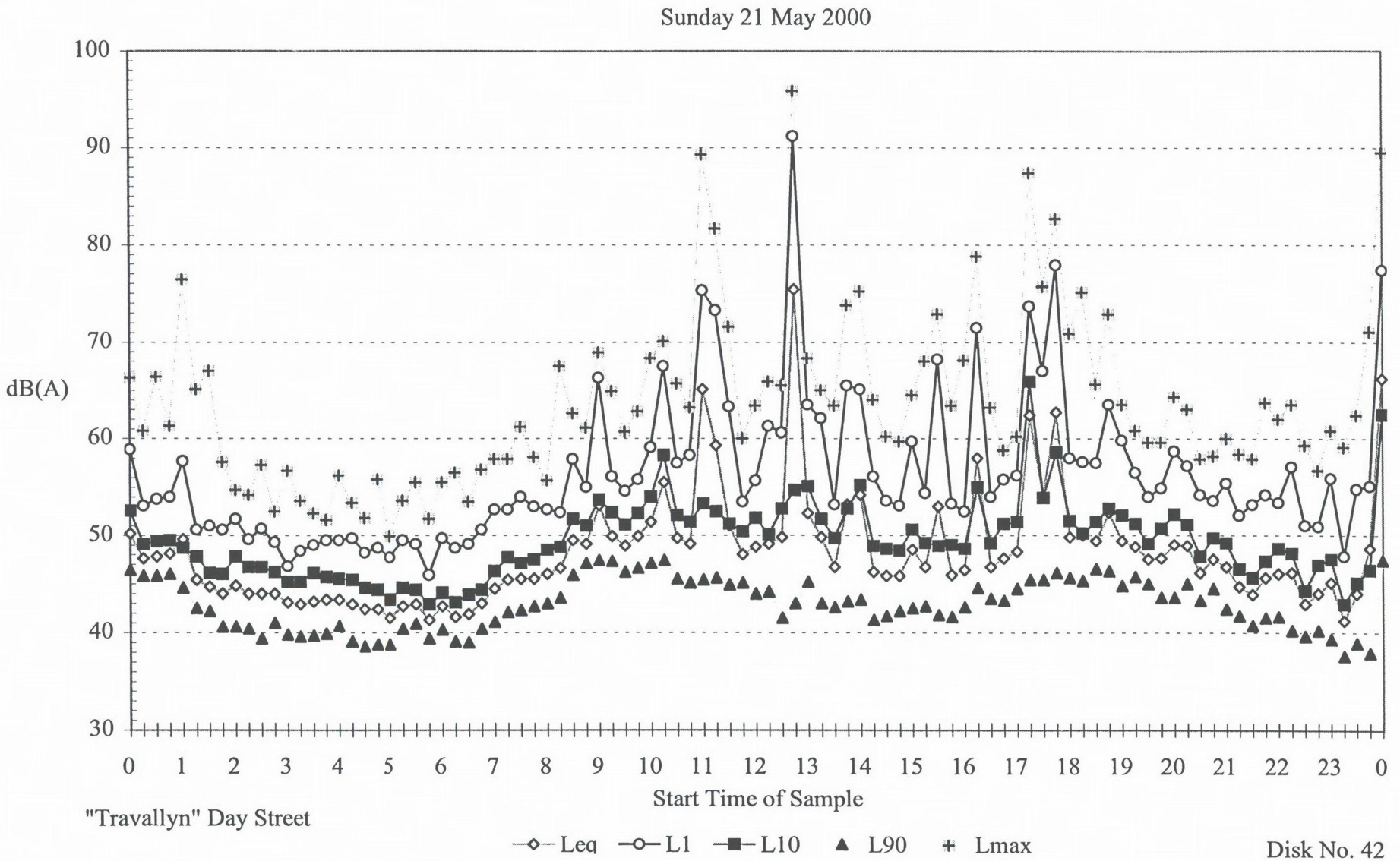
5030r3.xls

Ambient Sound Pressure Levels



5030r3.xls

Ambient Sound Pressure Levels

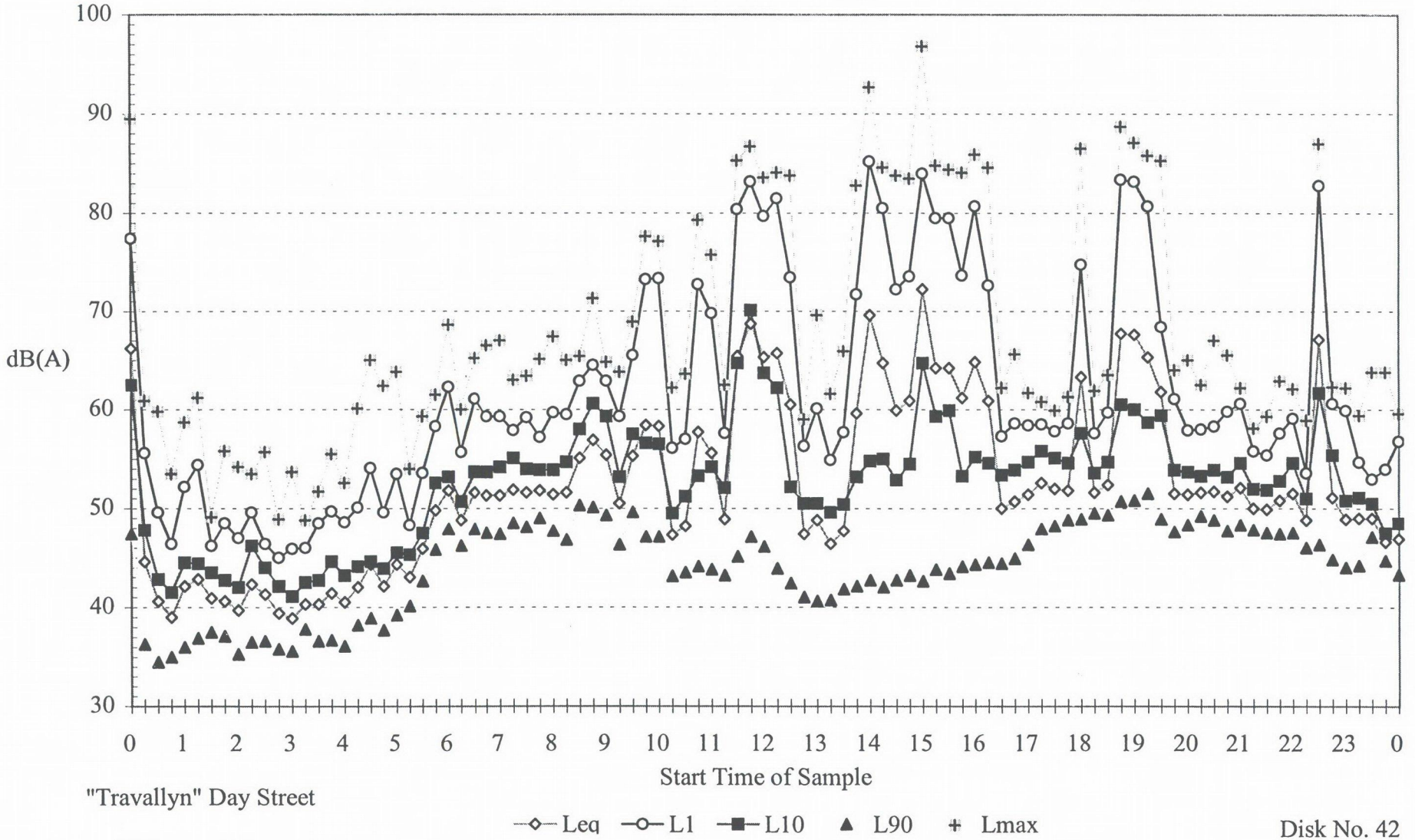


Disk No. 42

5030r3.xls

Ambient Sound Pressure Levels

Monday 22 May 2000

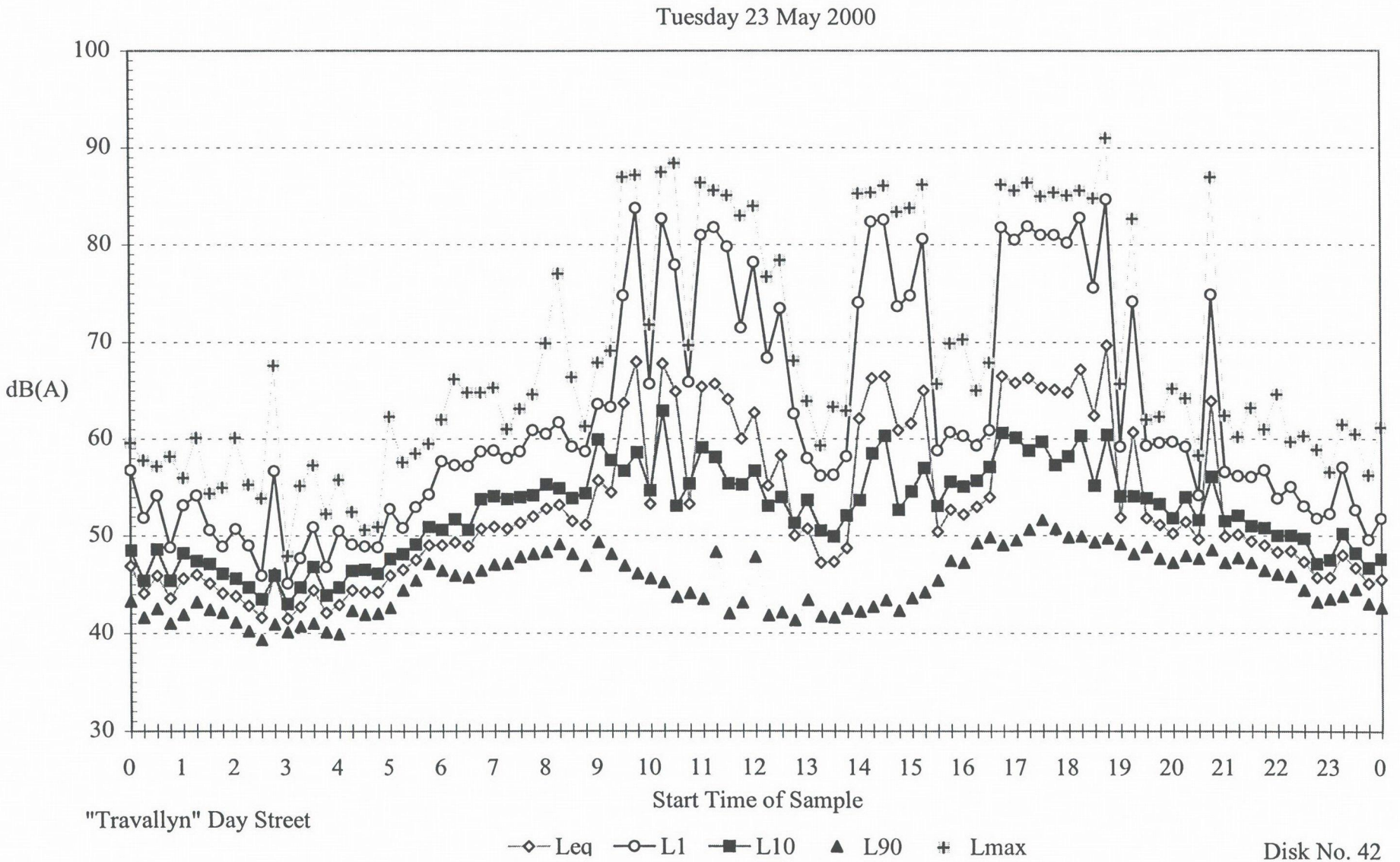


"Travallyn" Day Street

Disk No. 42

5030r3.xls

Ambient Sound Pressure Levels



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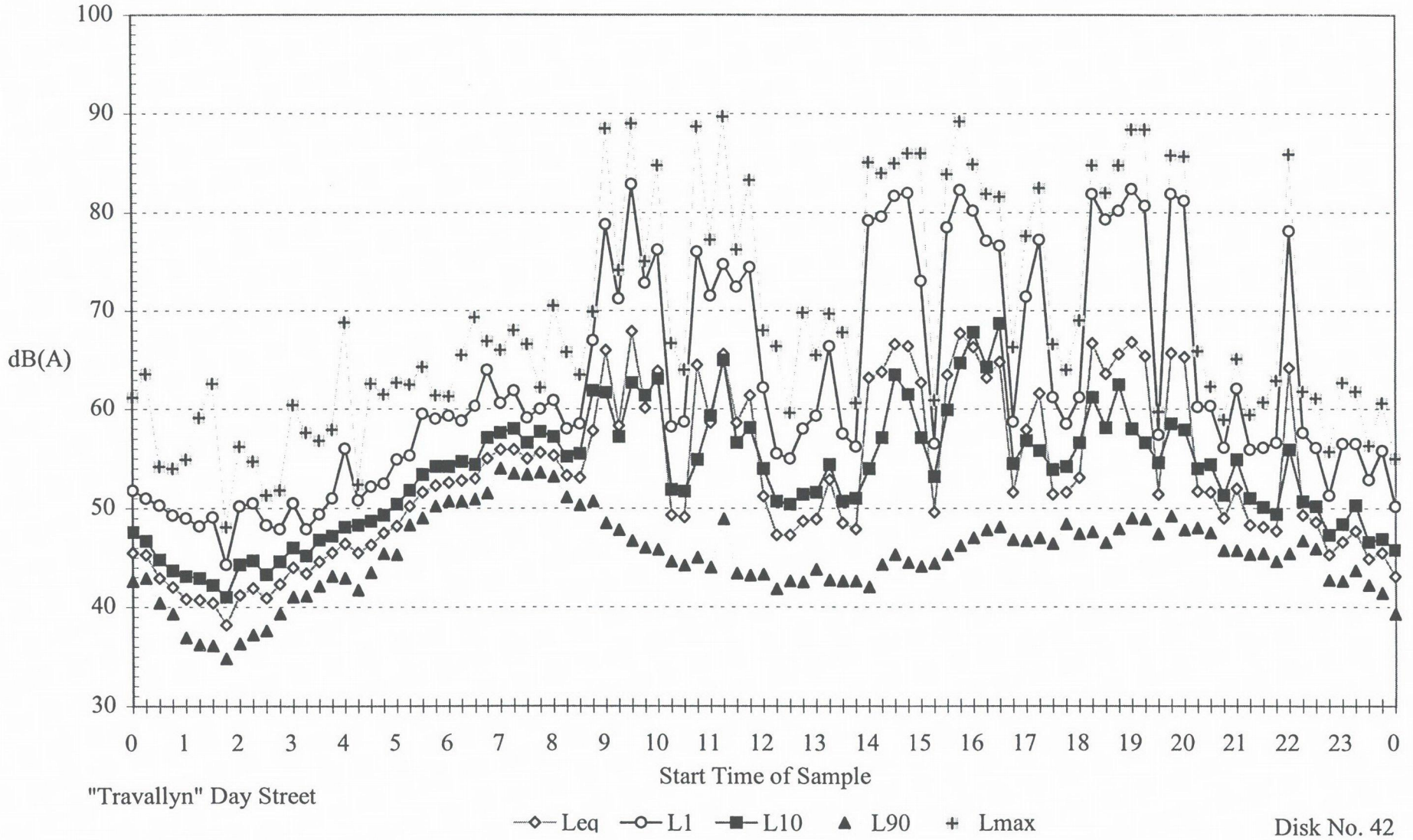
Ambient Sound Pressure Levels

Wednesday 24 May 2000

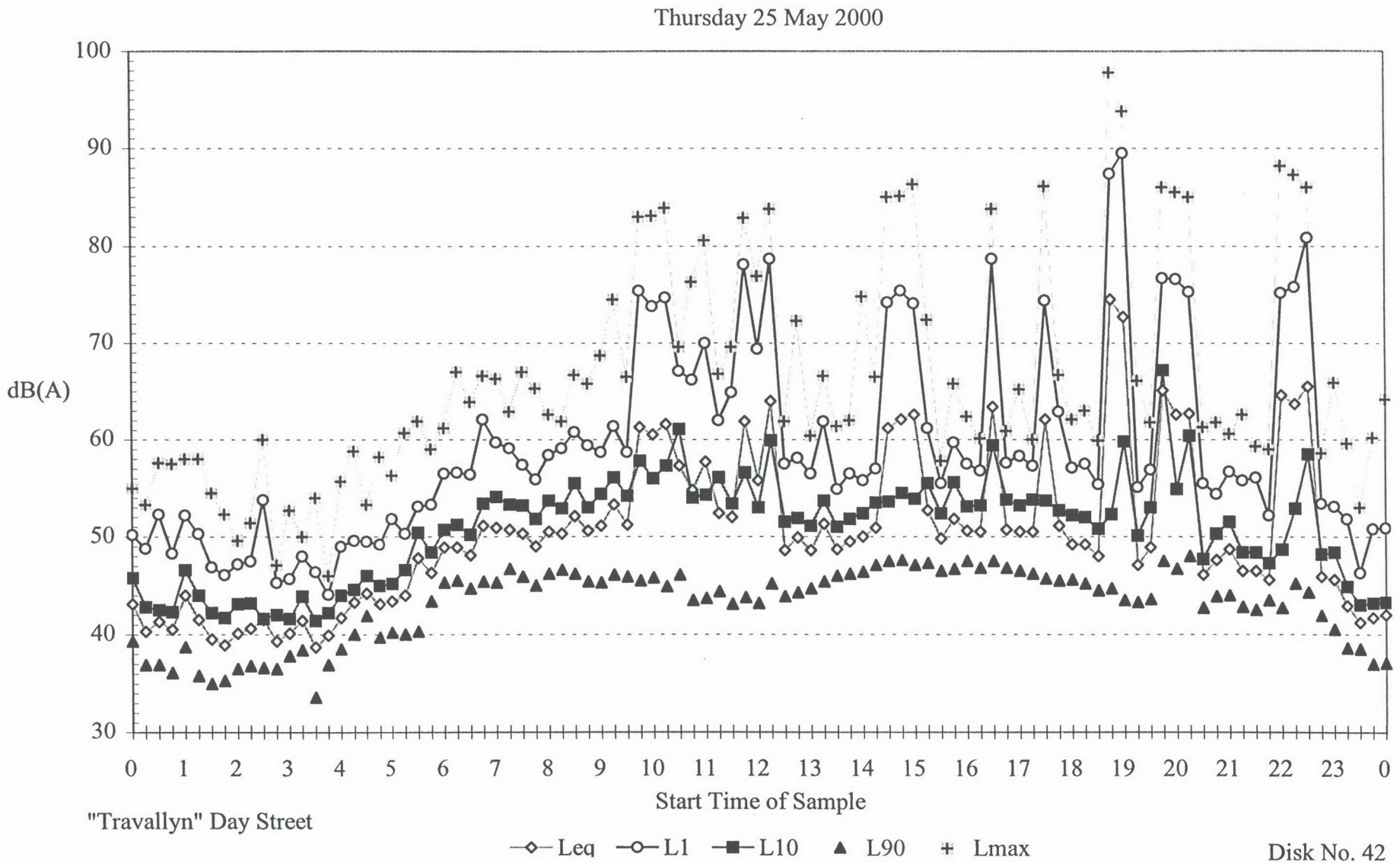


"Travallyn" Day Street

Disk No. 42

5030r3.xls

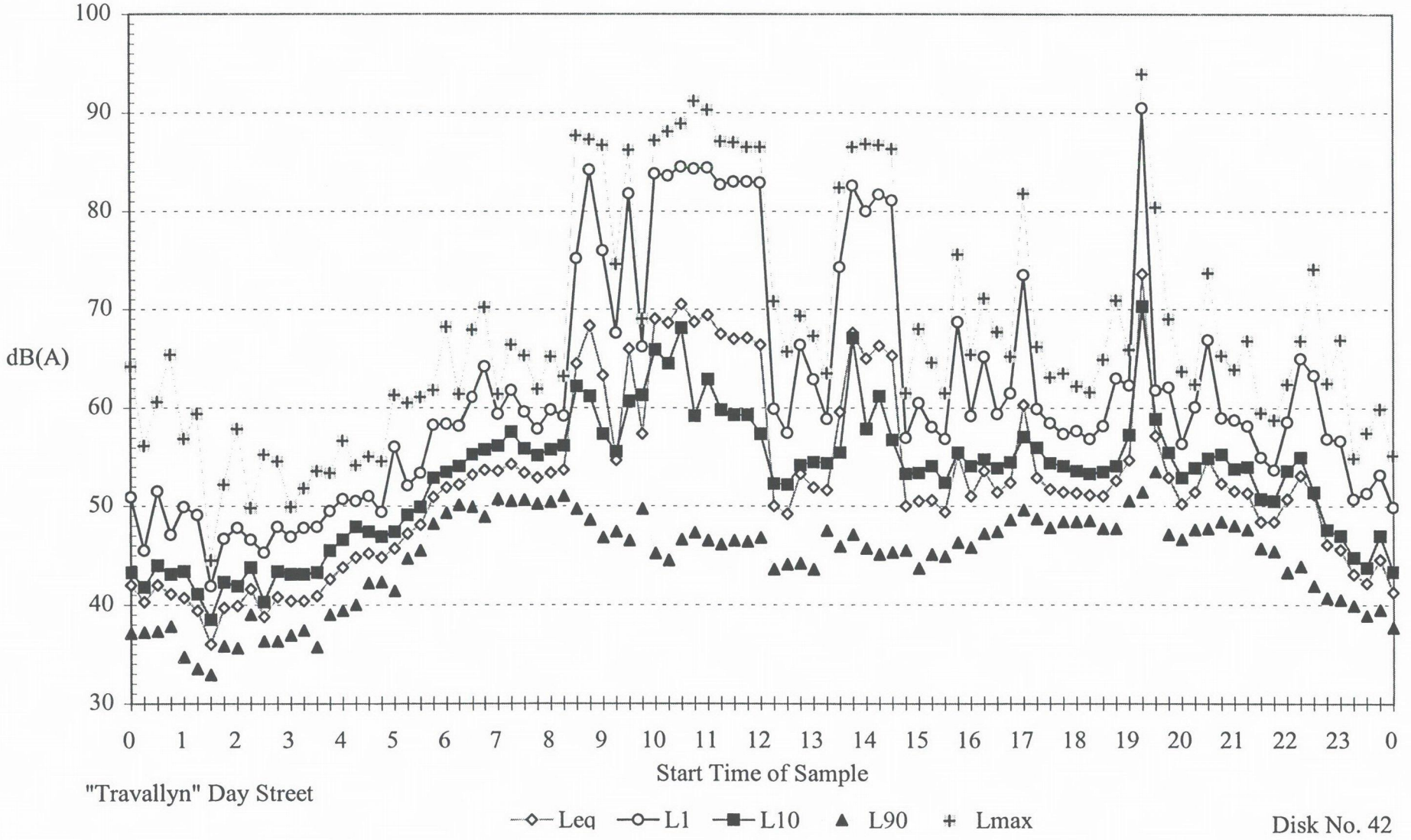
Ambient Sound Pressure Levels



5030r3.xls

Ambient Sound Pressure Levels

Friday 26 May 2000

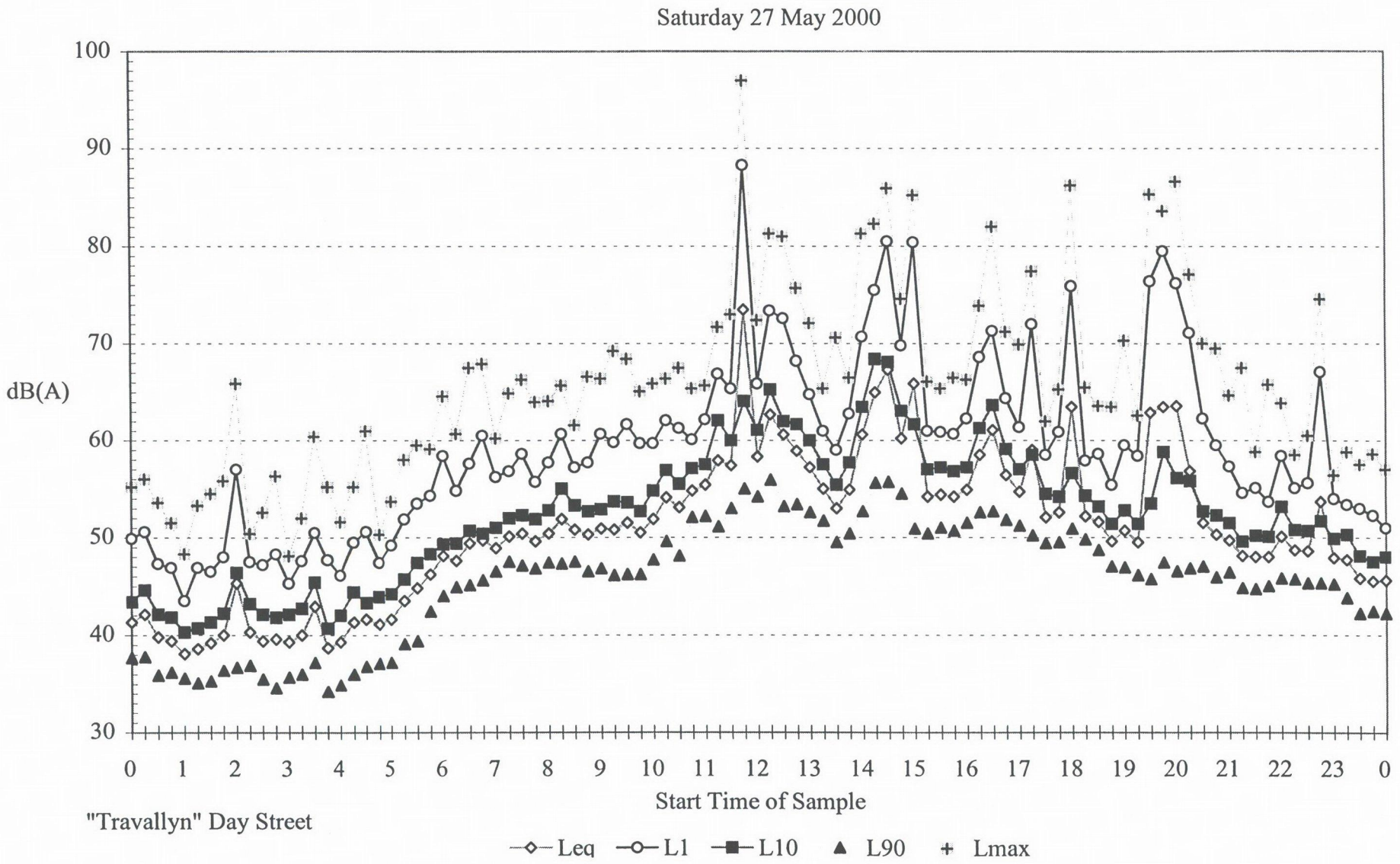


"Travallyn" Day Street

Disk No. 42

5030r3.xls

Ambient Sound Pressure Levels



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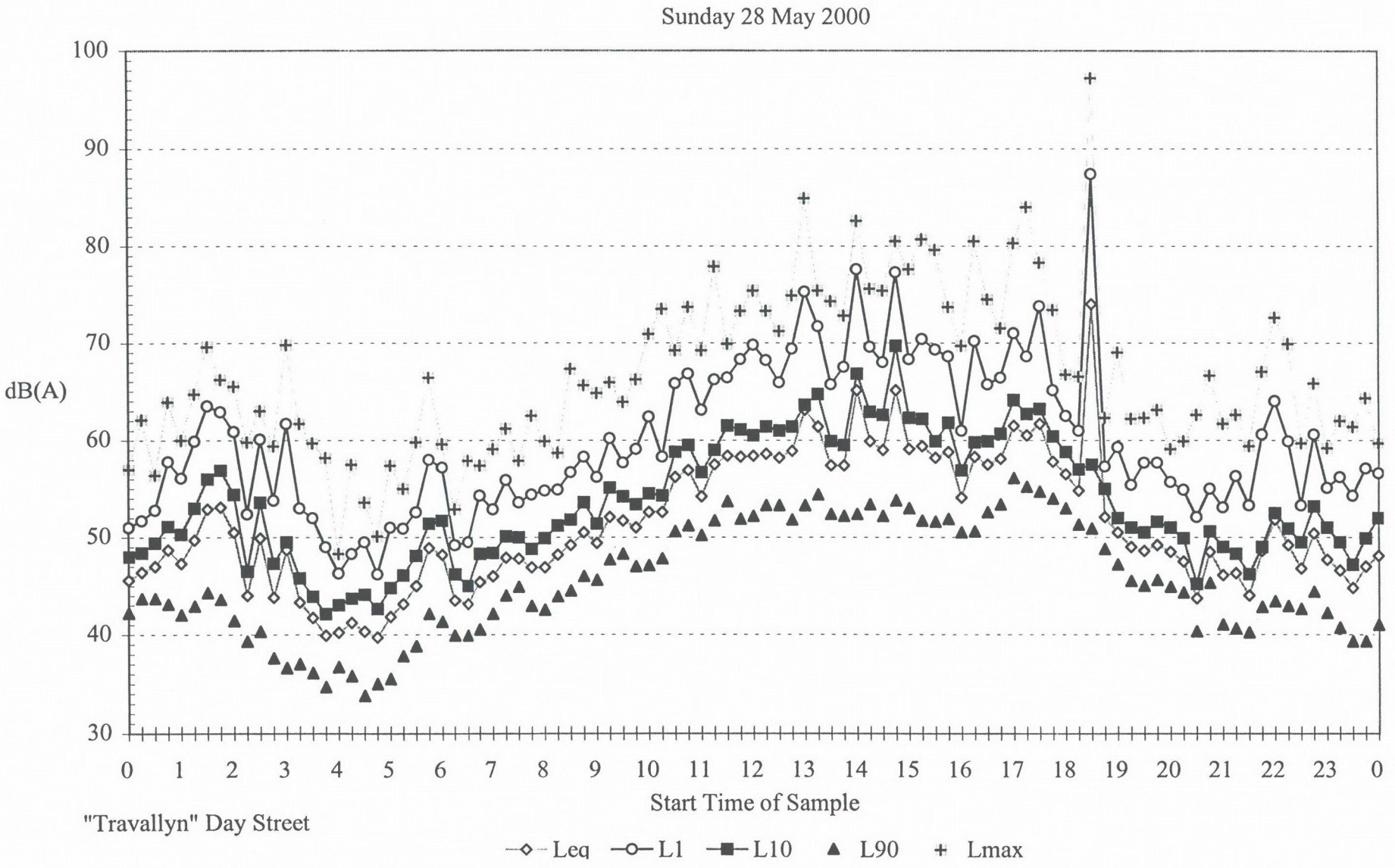
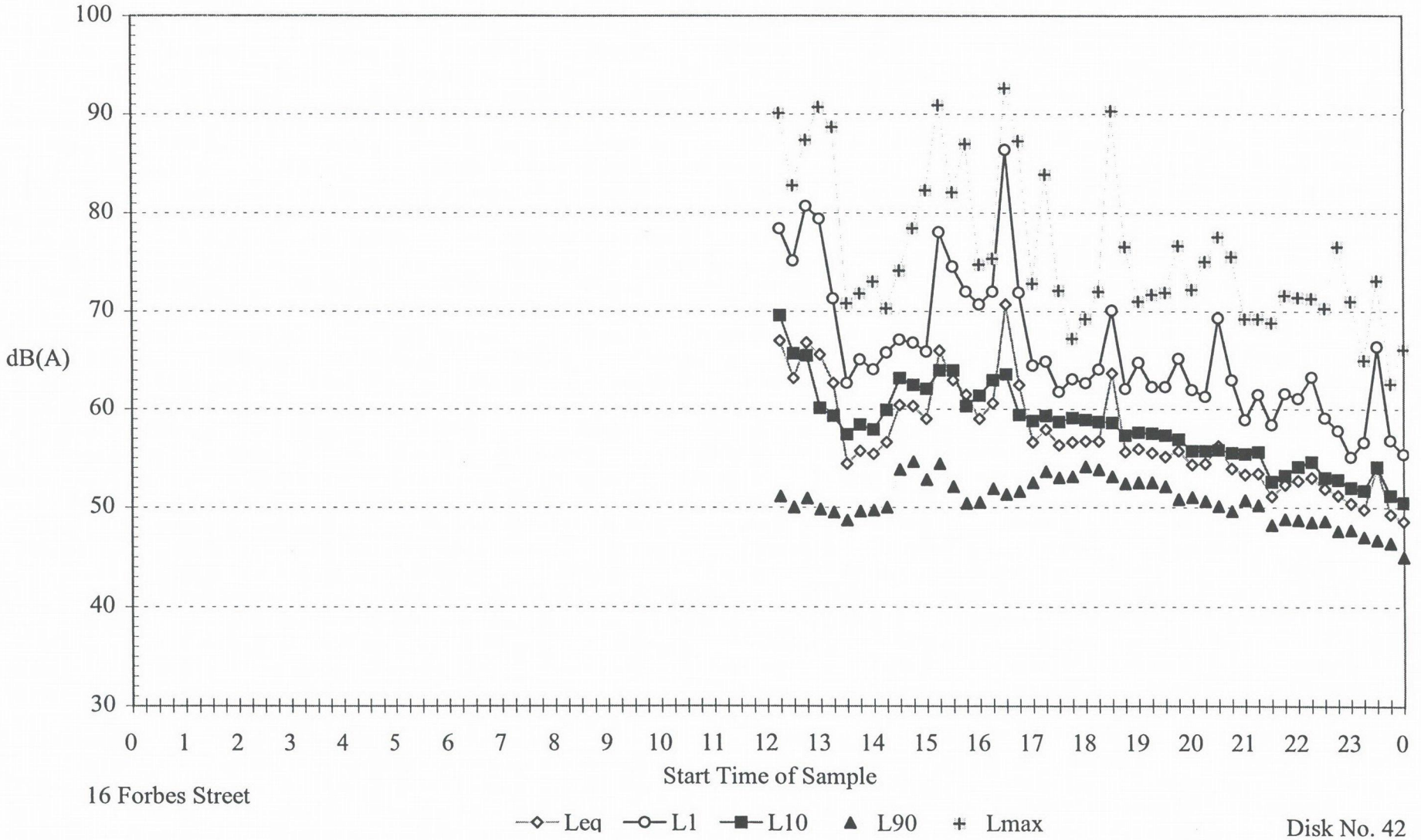
Ambient Sound Pressure Levels



5030r3.xls

Ambient Sound Pressure Levels

Friday 19 May 2000

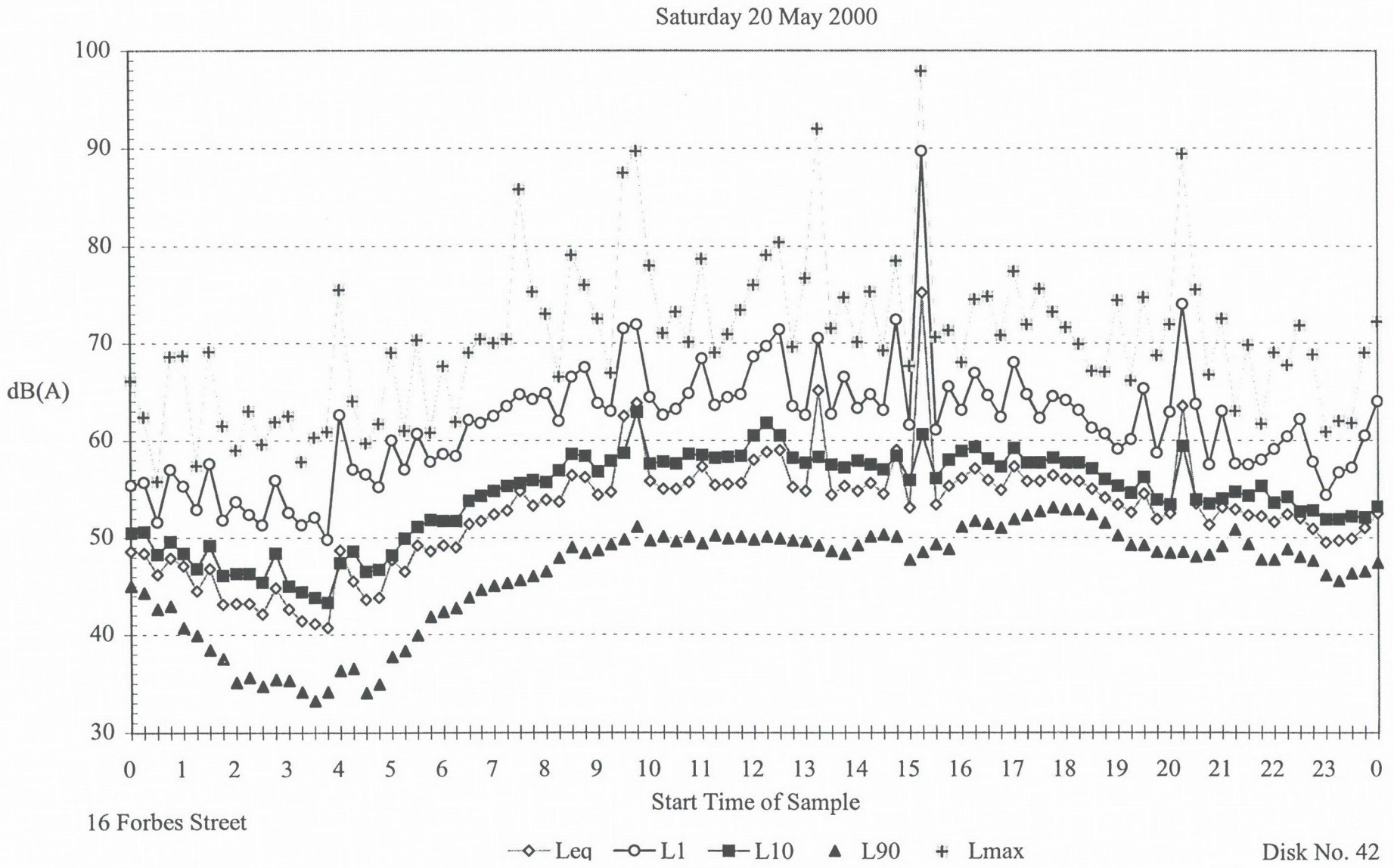


16 Forbes Street

Disk No. 42

5030r4.xls

Ambient Sound Pressure Levels



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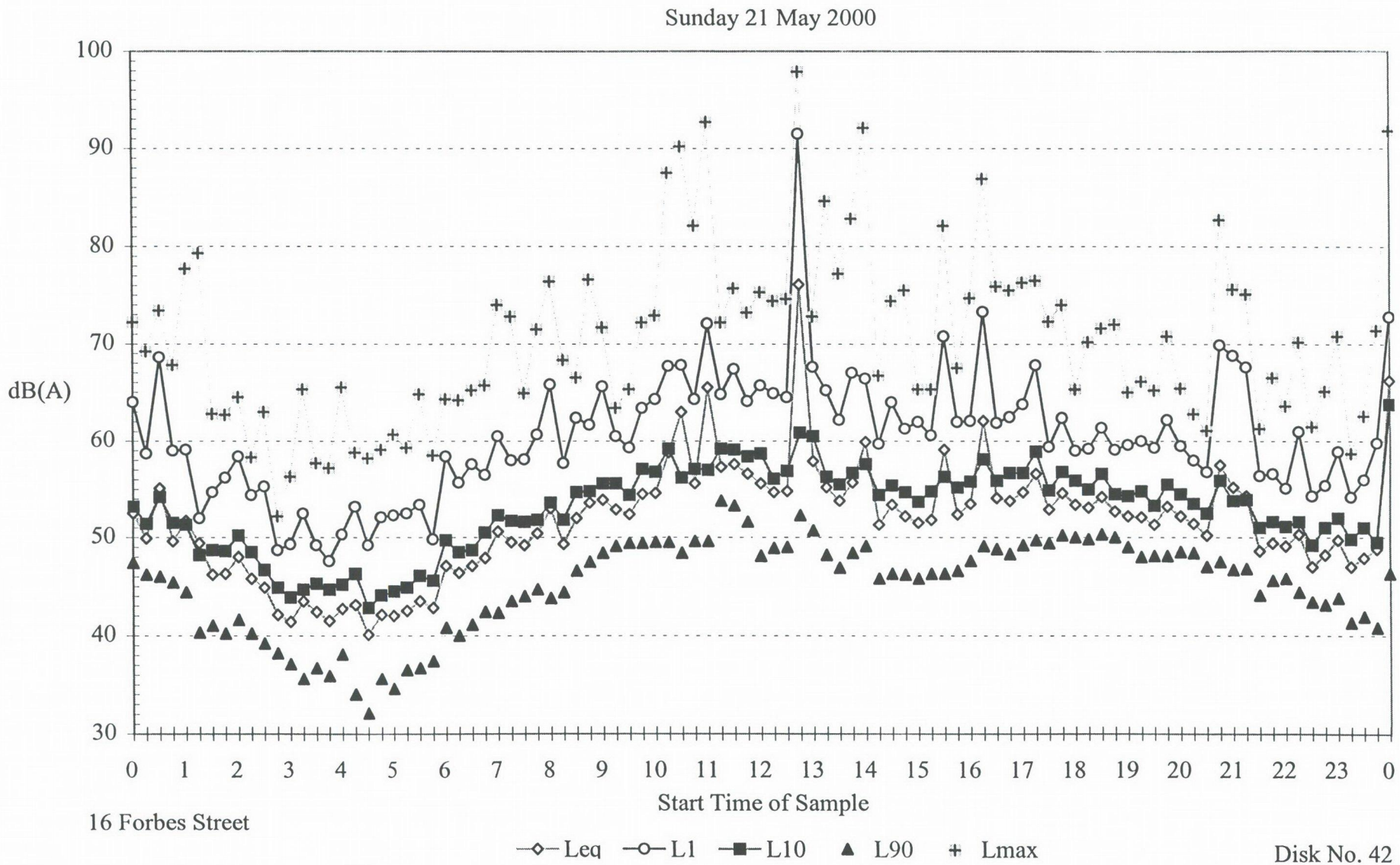
Ambient Sound Pressure Levels



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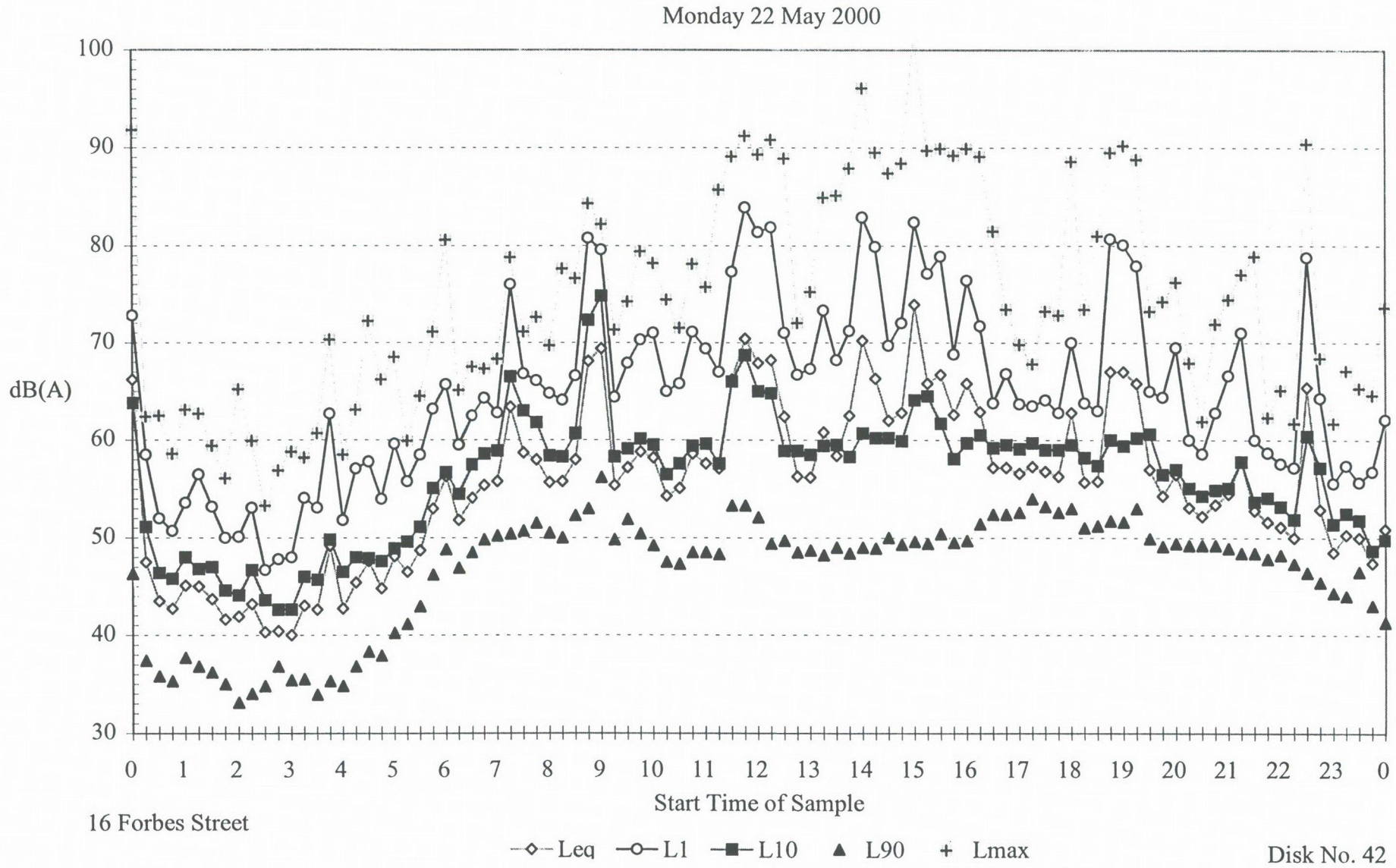
Ambient Sound Pressure Levels



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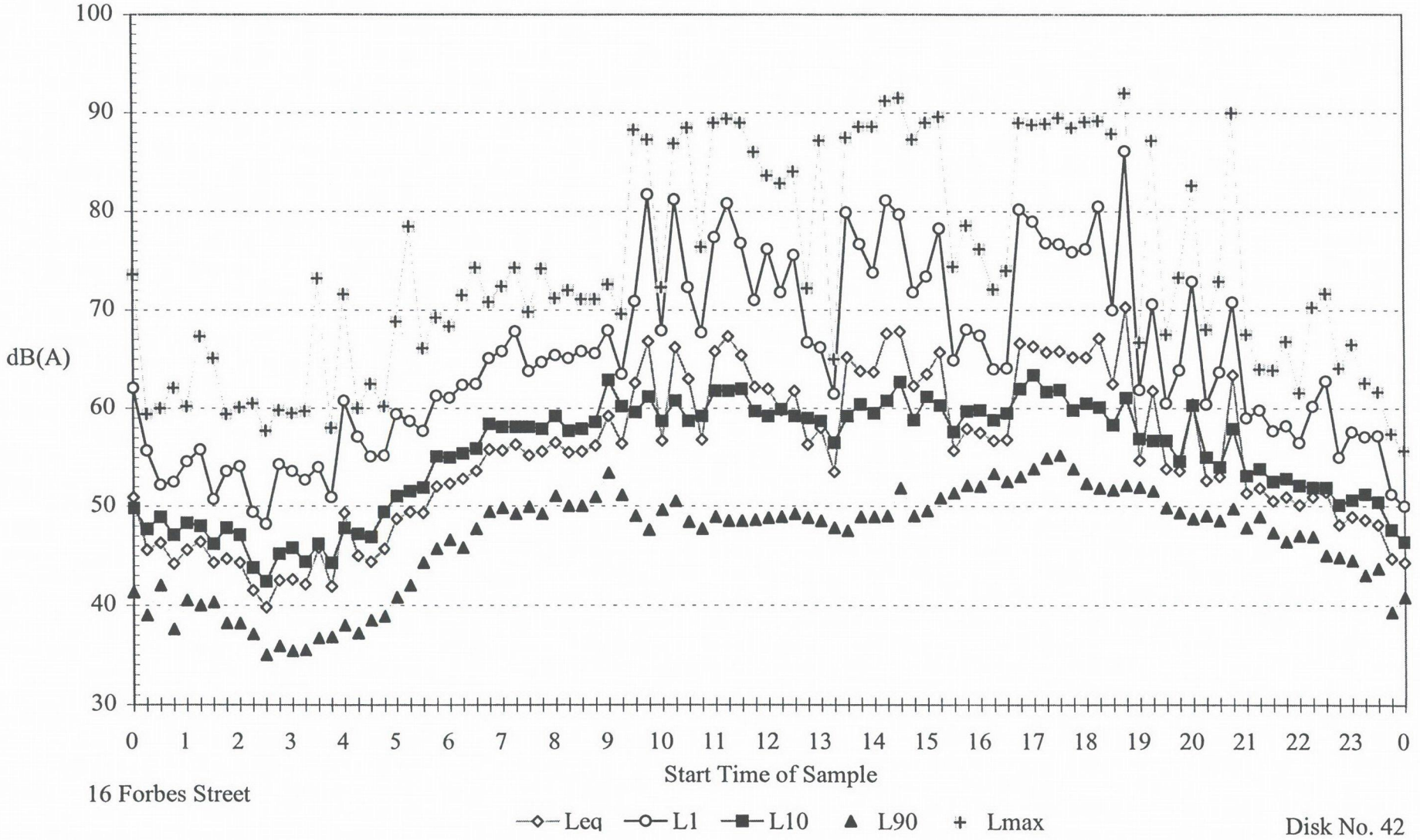
Ambient Sound Pressure Levels

Tuesday 23 May 2000

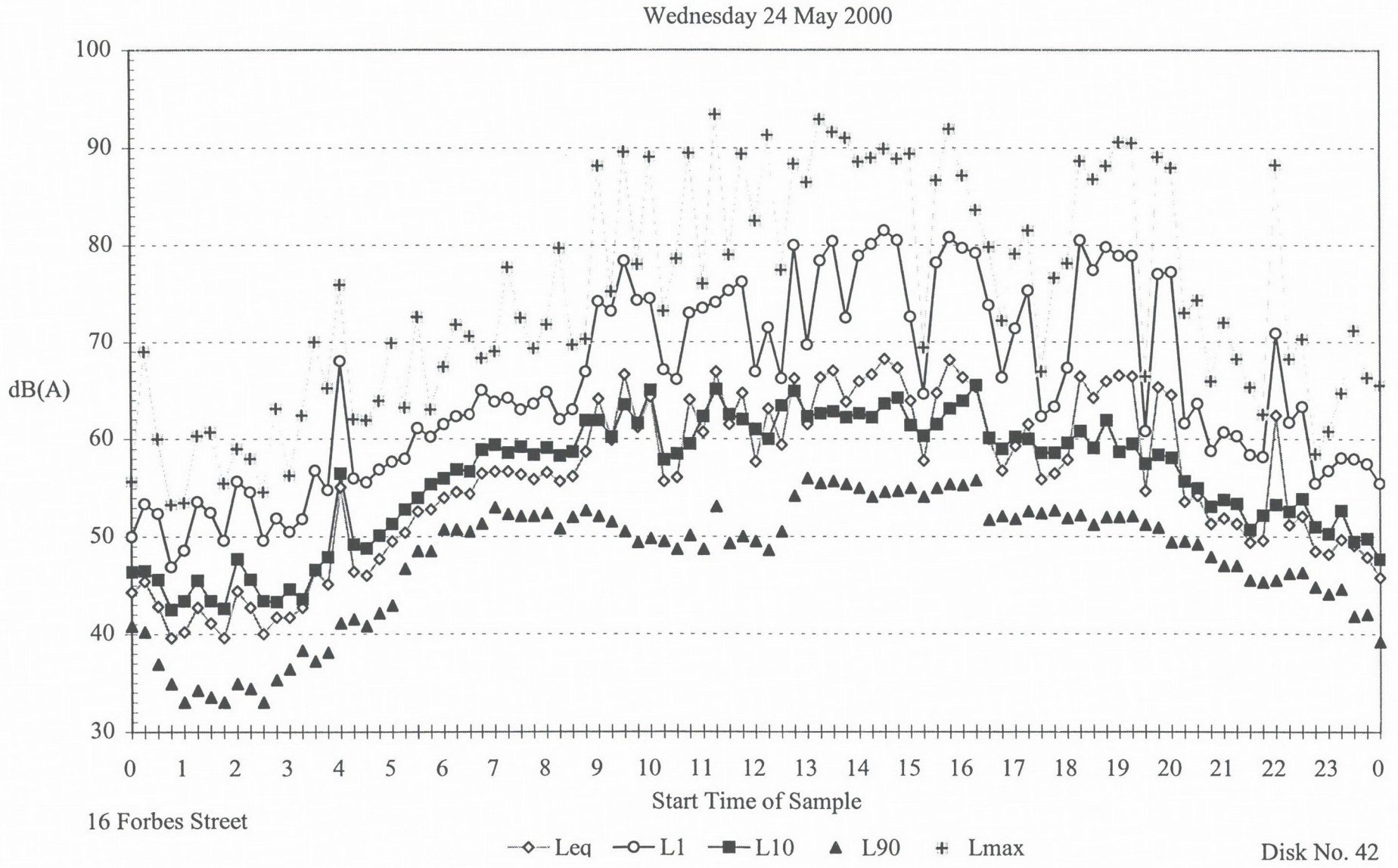


16 Forbes Street

Disk No. 42

5030r4.xls

Ambient Sound Pressure Levels



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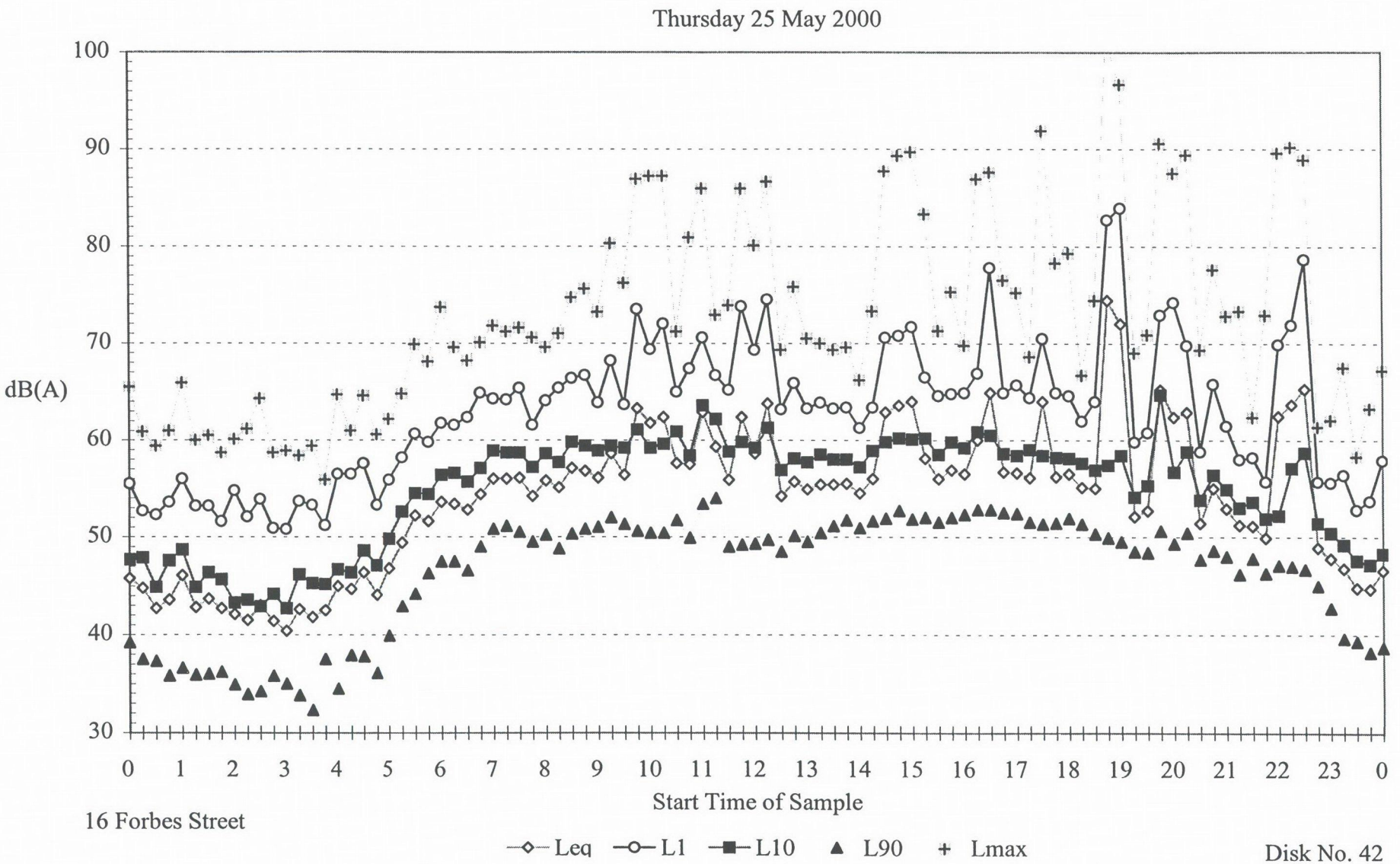
Ambient Sound Pressure Levels



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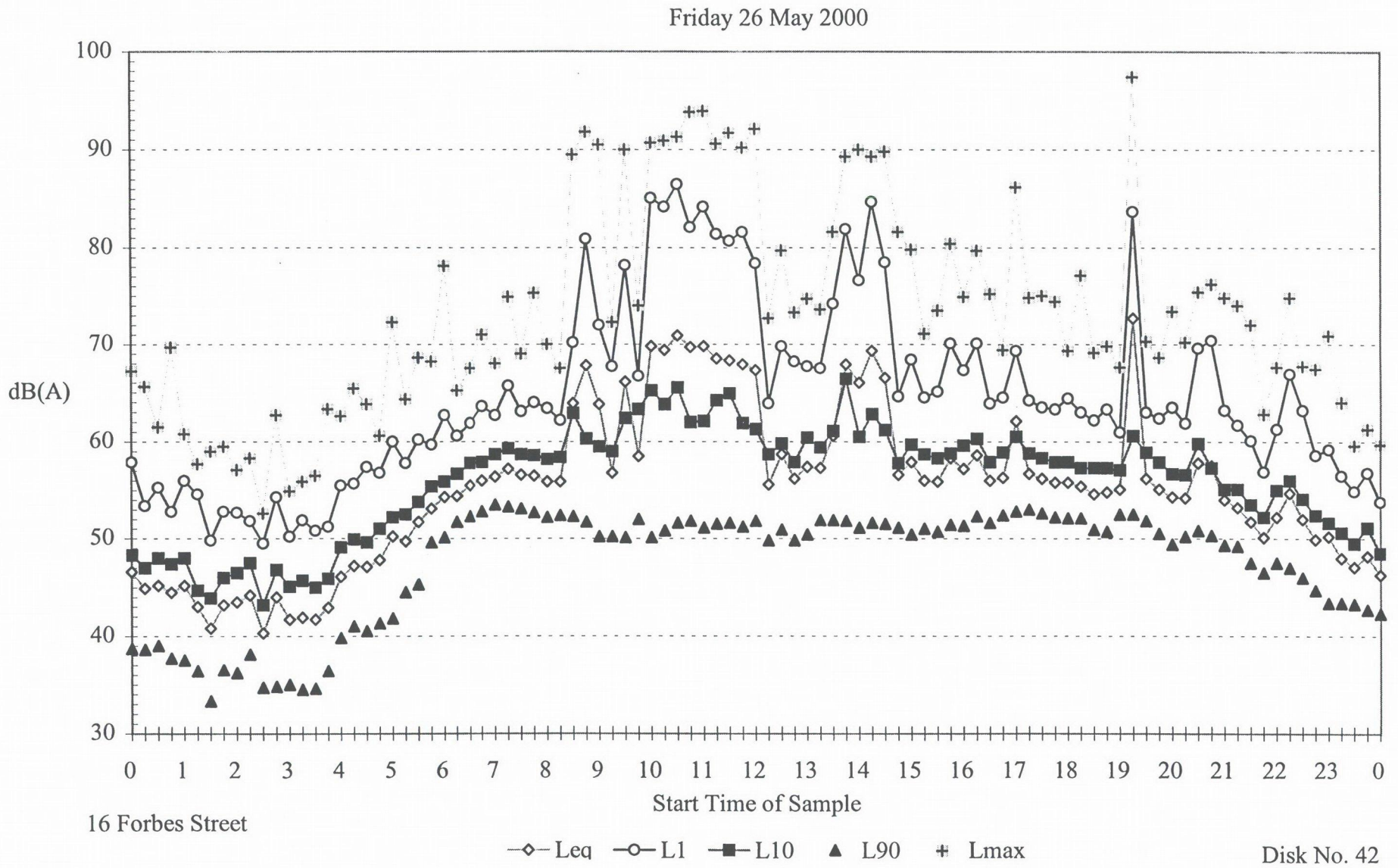
Ambient Sound Pressure Levels



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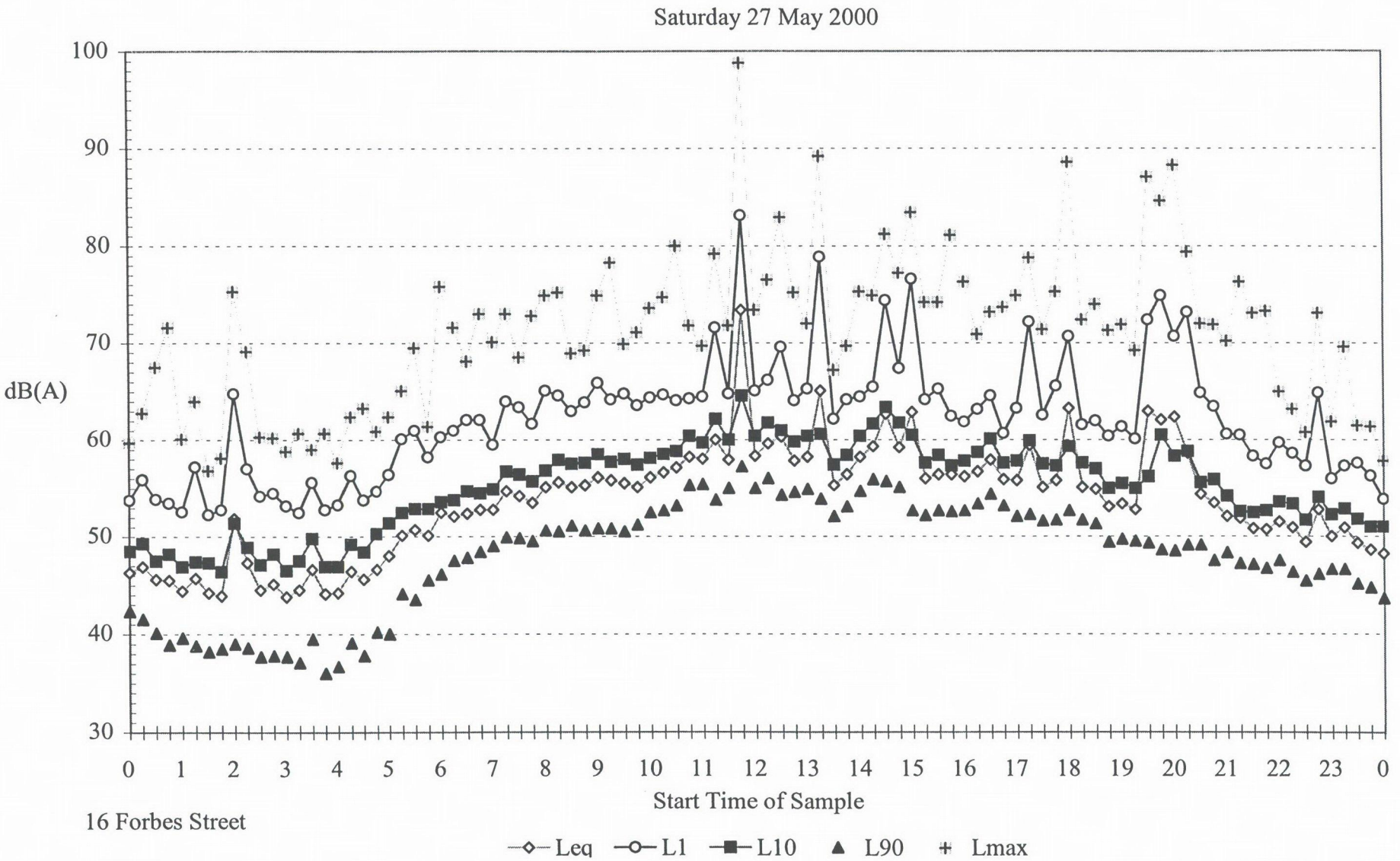
Ambient Sound Pressure Levels



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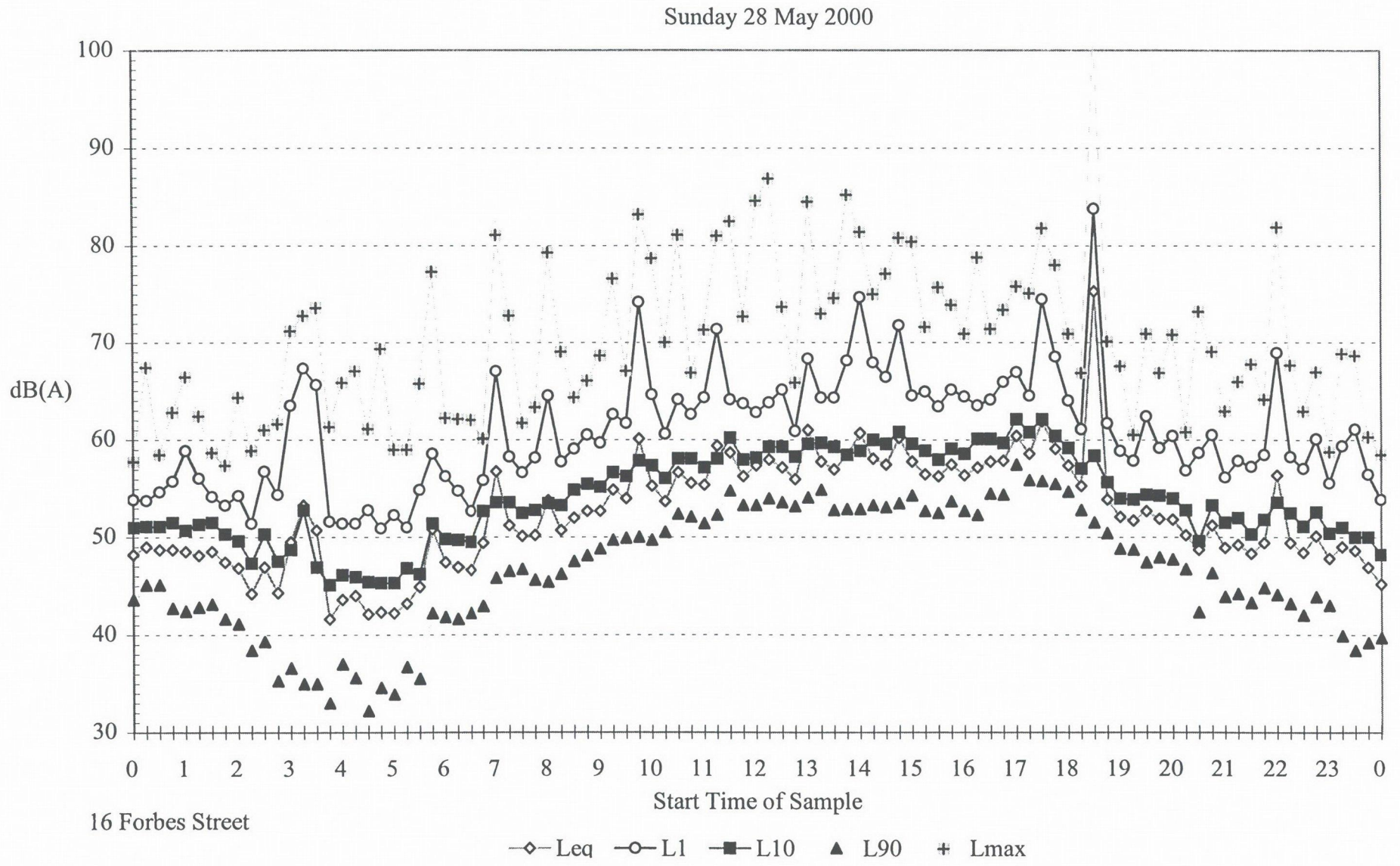
Ambient Sound Pressure Levels



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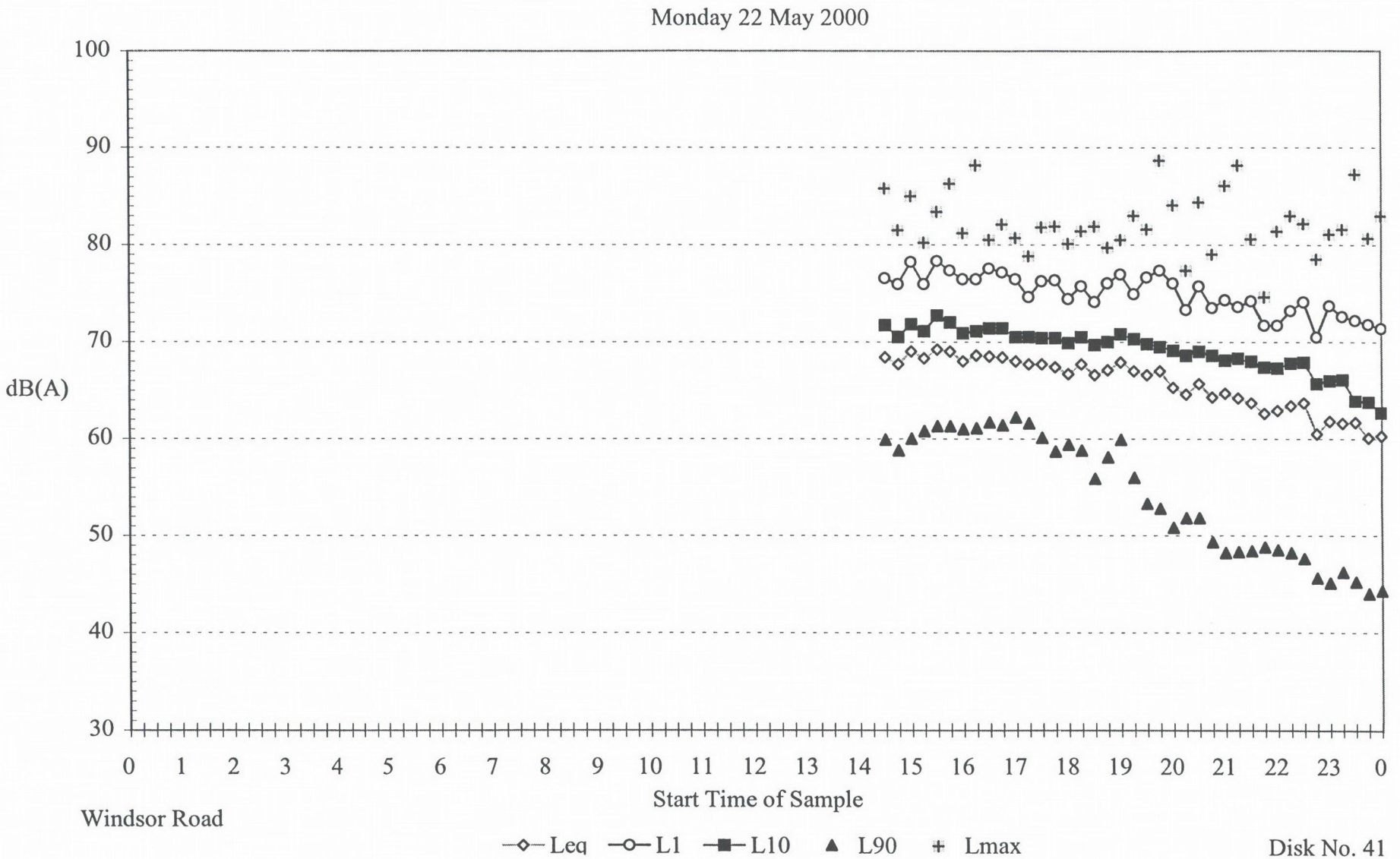
Ambient Sound Pressure Levels

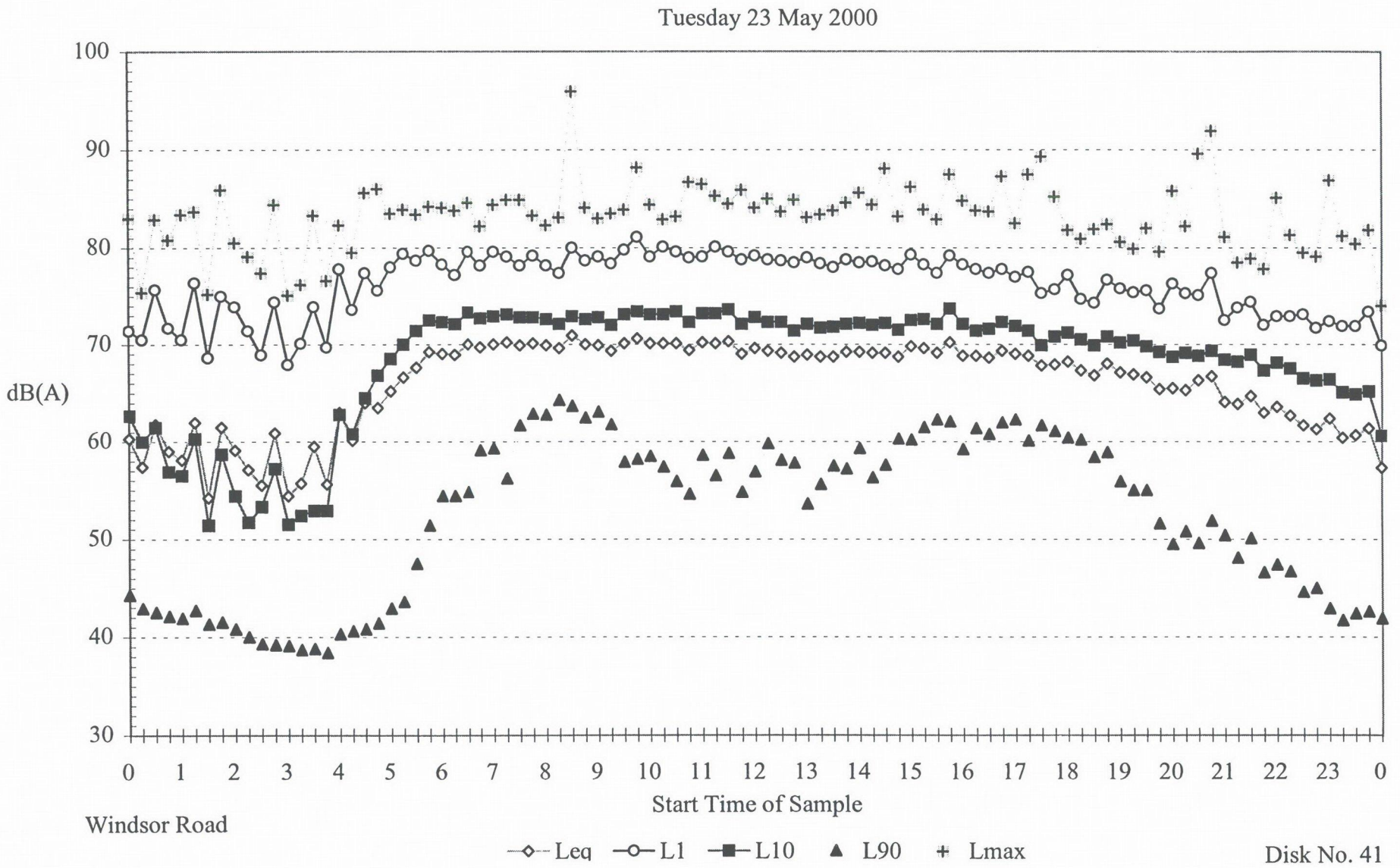


Windsor Road

Disk No. 41

5030r5.xls

Ambient Sound Pressure Levels

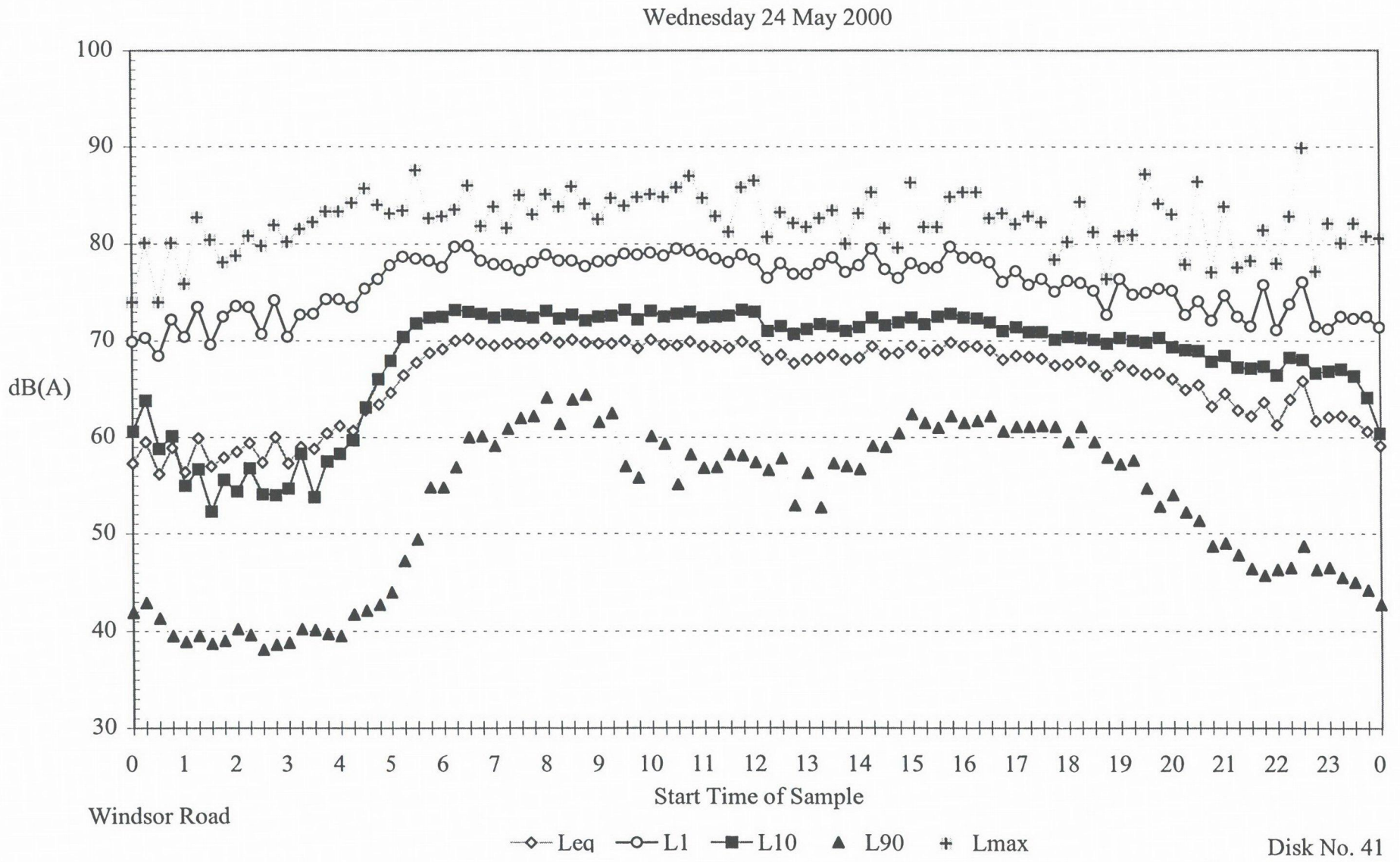


Windsor Road

Disk No. 41

5030r5.xls

Ambient Sound Pressure Levels



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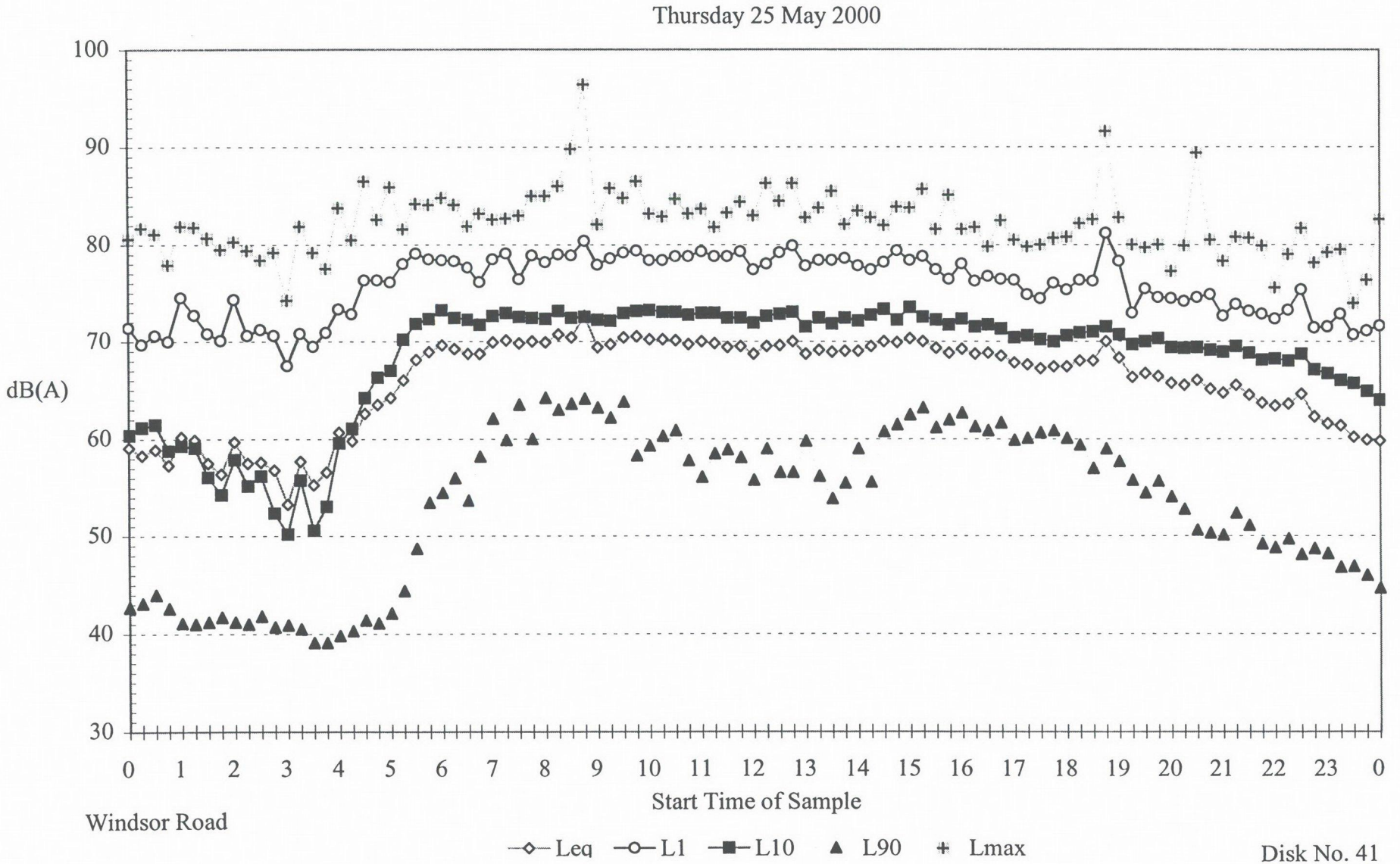
Ambient Sound Pressure Levels

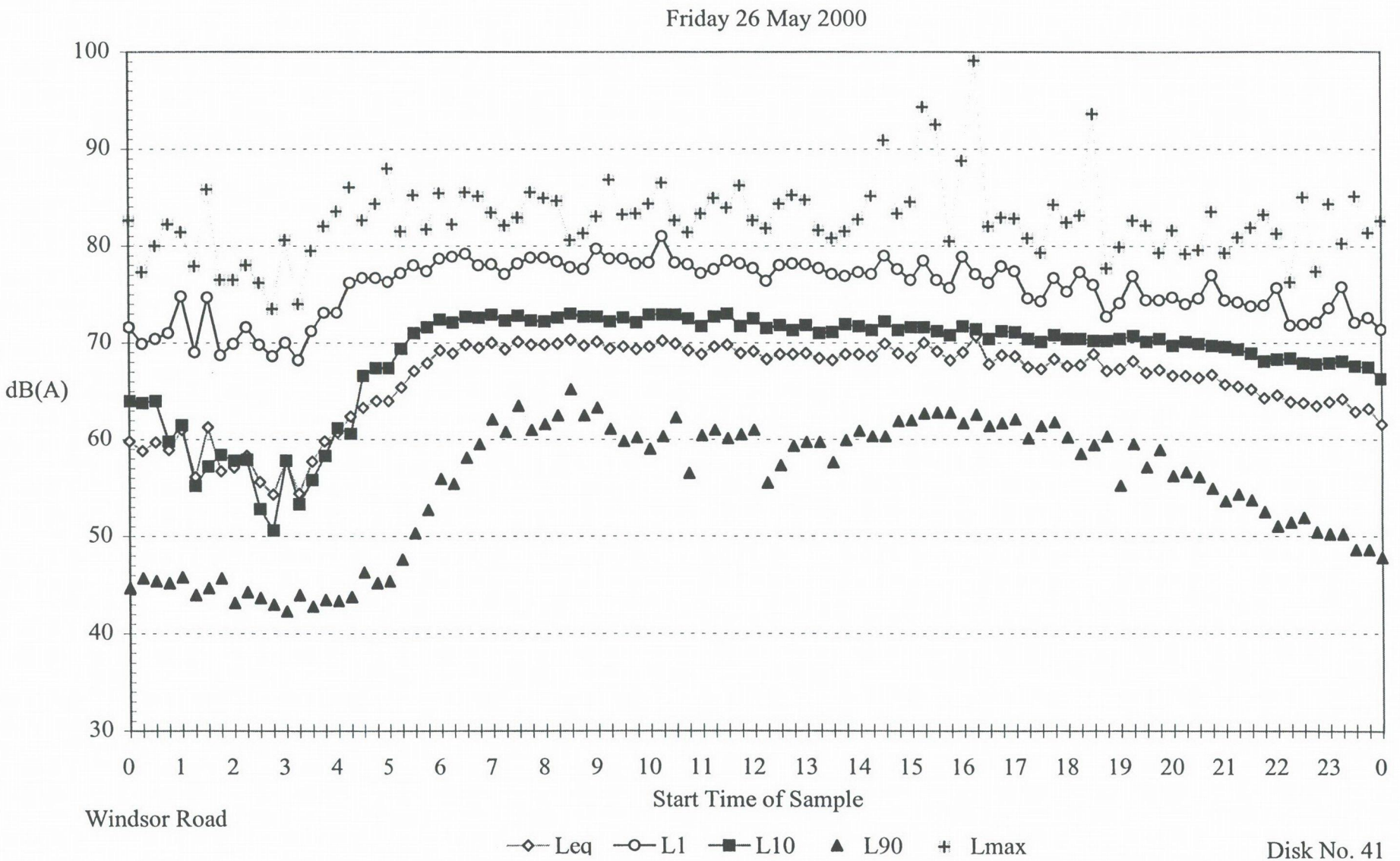


Windsor Road

Disk No. 41

5030r5.xls

Ambient Sound Pressure Levels



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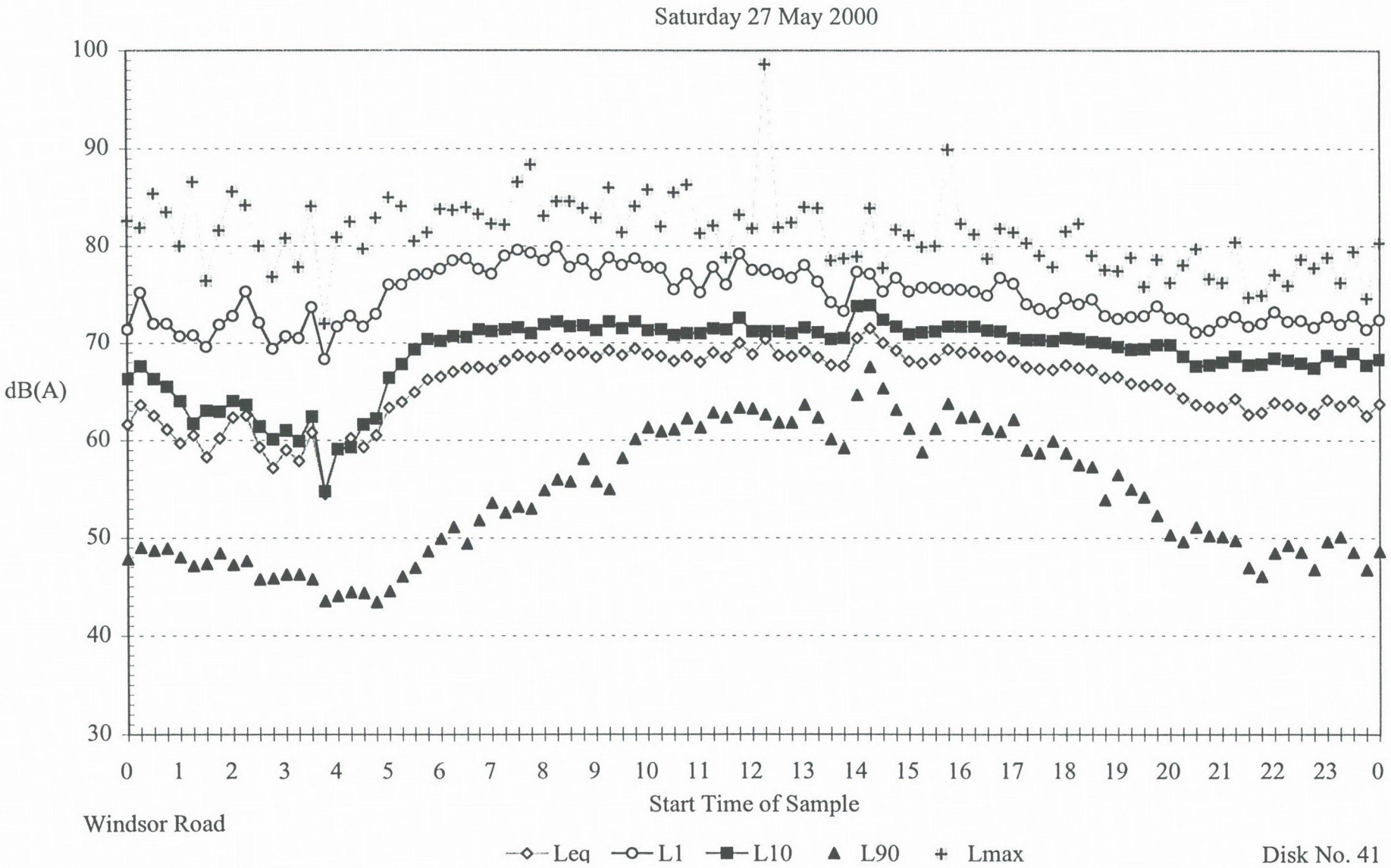
Ambient Sound Pressure Levels



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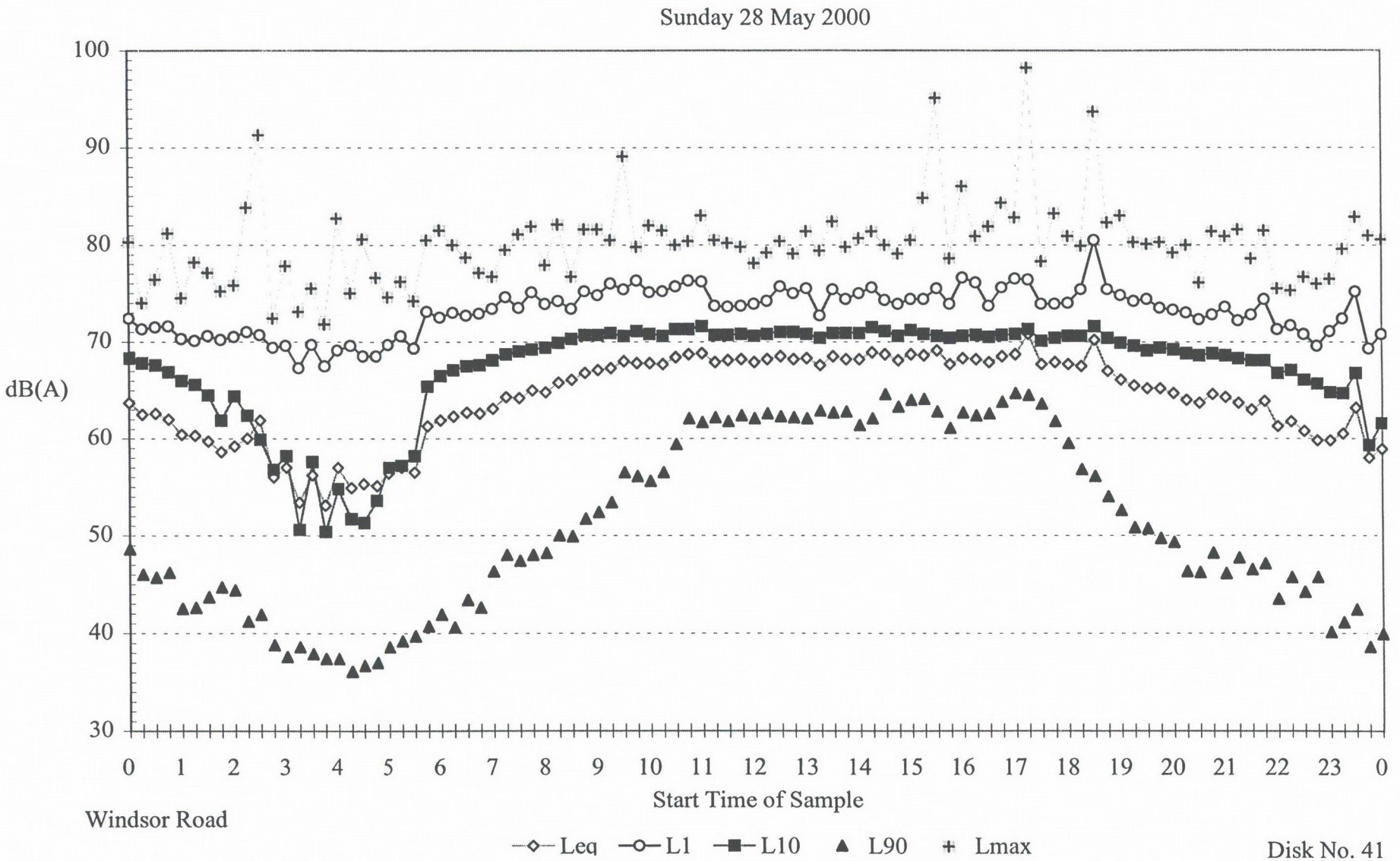
Ambient Sound Pressure Levels



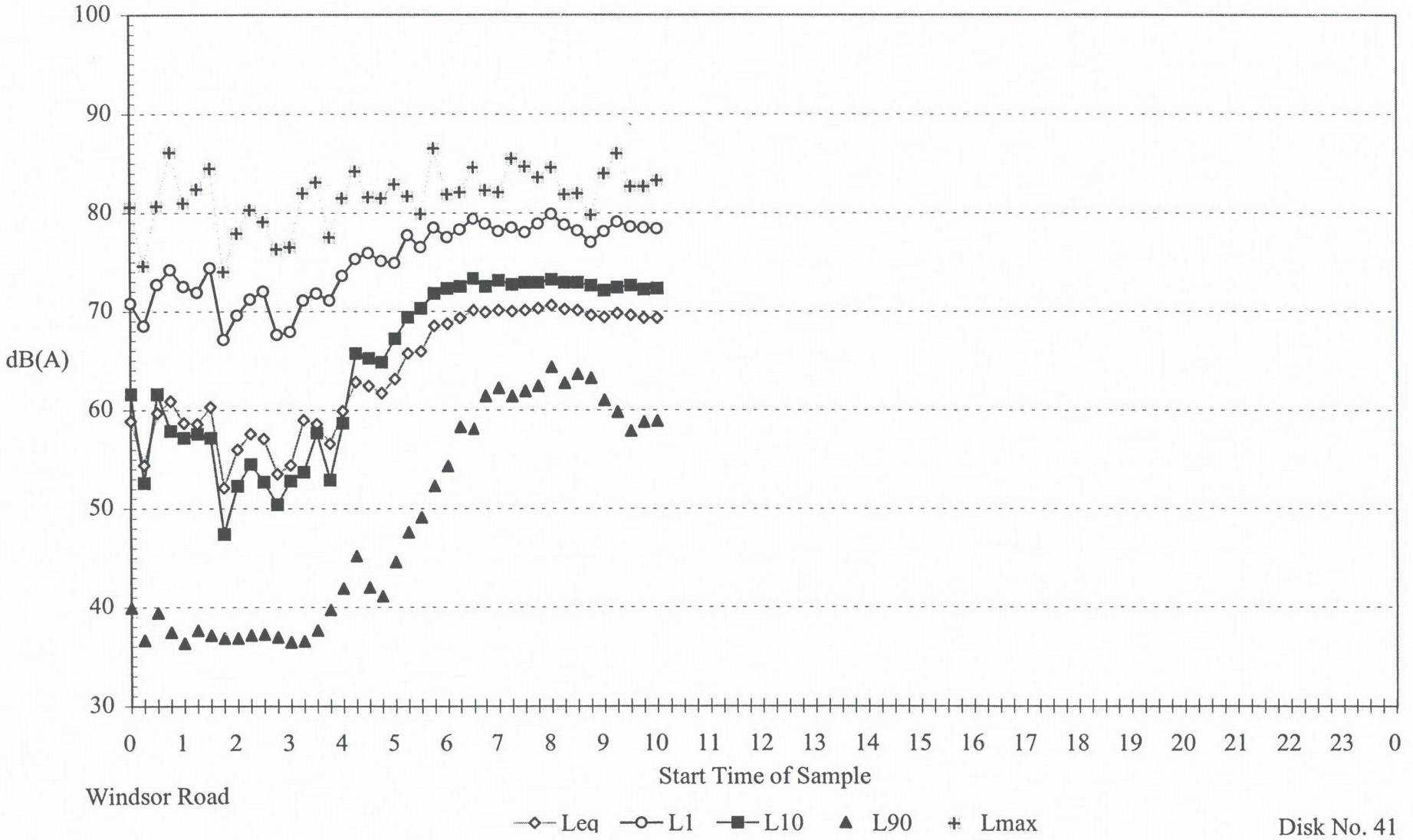
Windsor Road

Disk No. 41

5030r5.xls

Ambient Sound Pressure Levels

Monday 29 May 2000



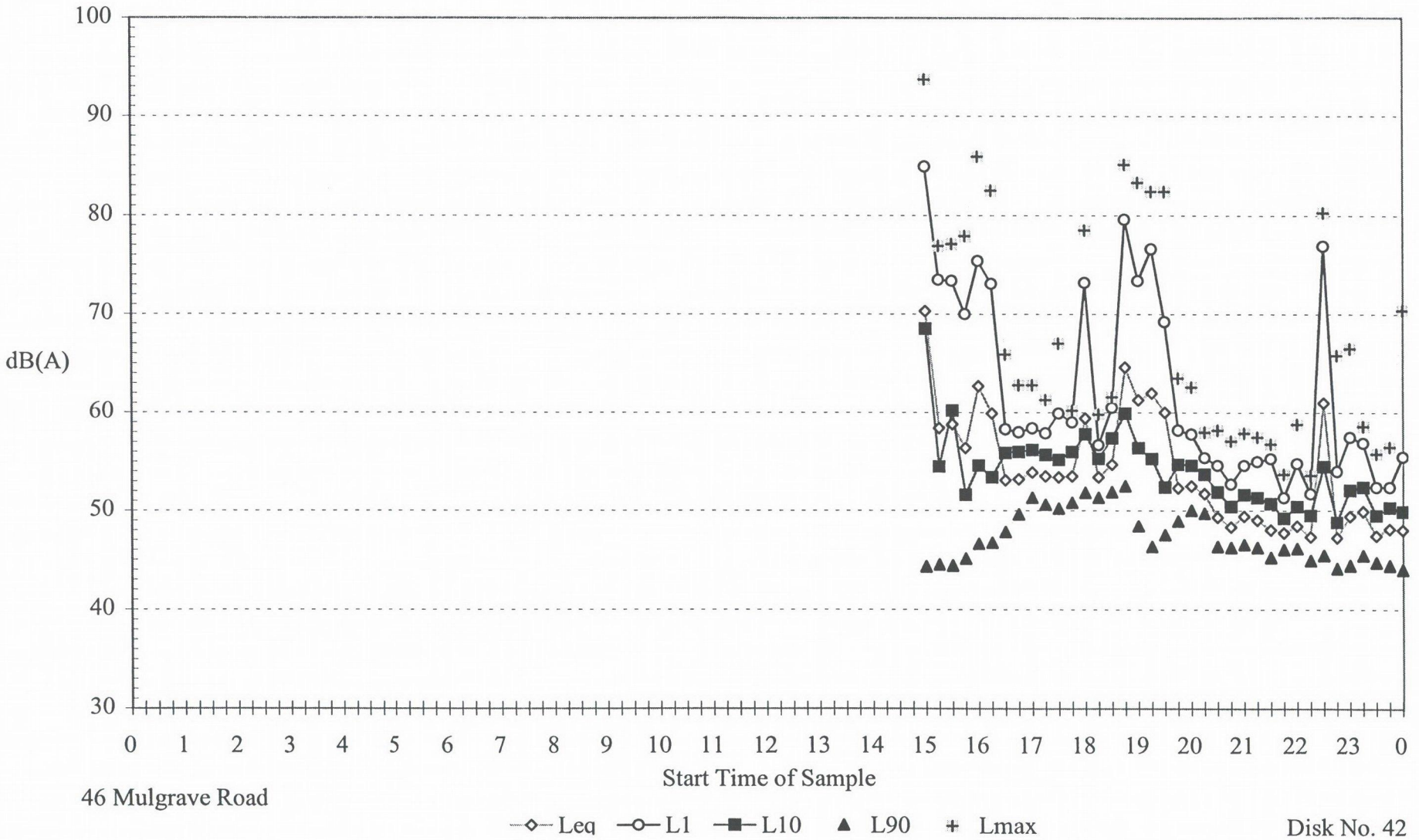
Windsor Road

Disk No. 41

5030r5.xls

Ambient Sound Pressure Levels

Monday 22 May 2000



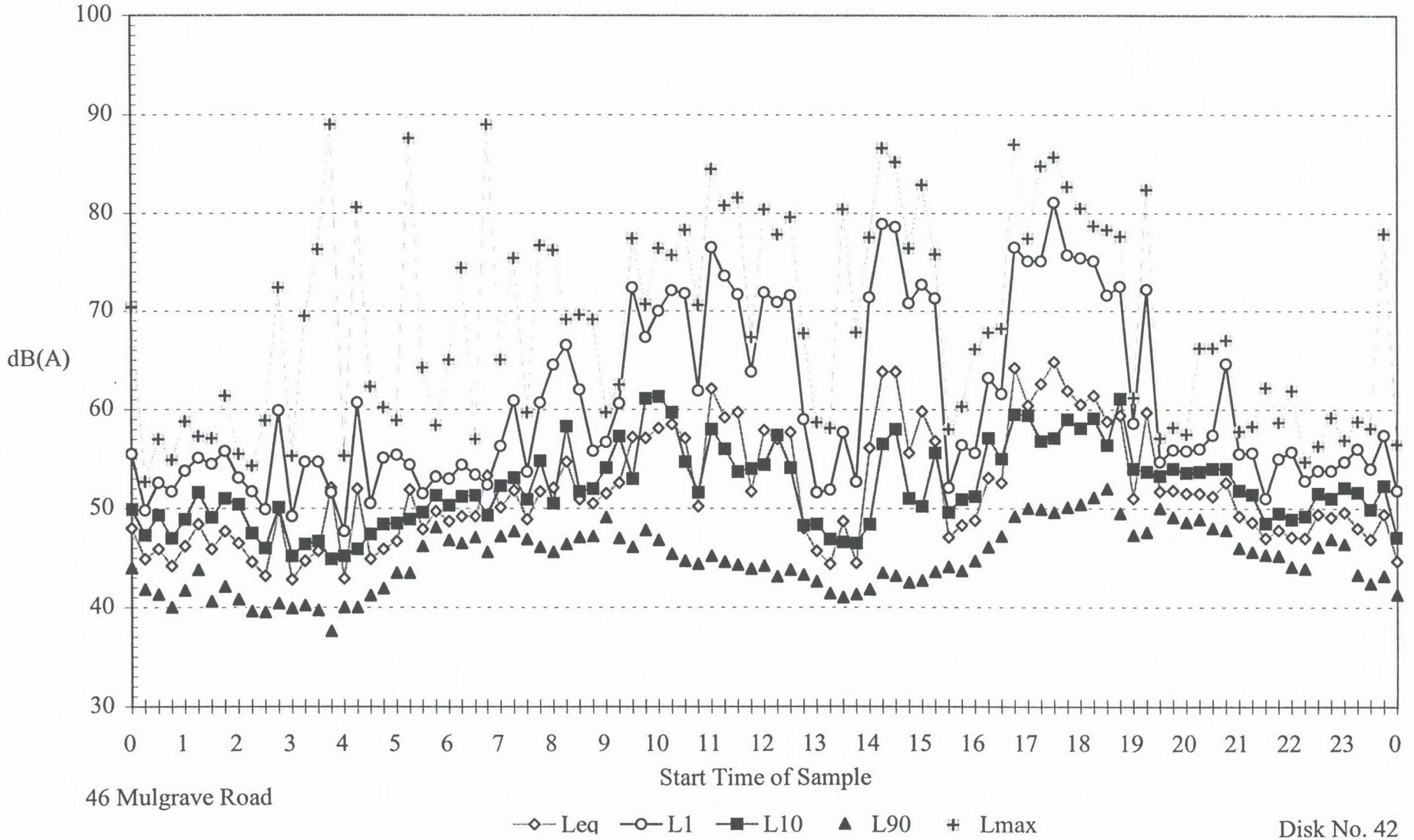
46 Mulgrave Road

Disk No. 42

5030r6.xls

Ambient Sound Pressure Levels

Tuesday 23 May 2000

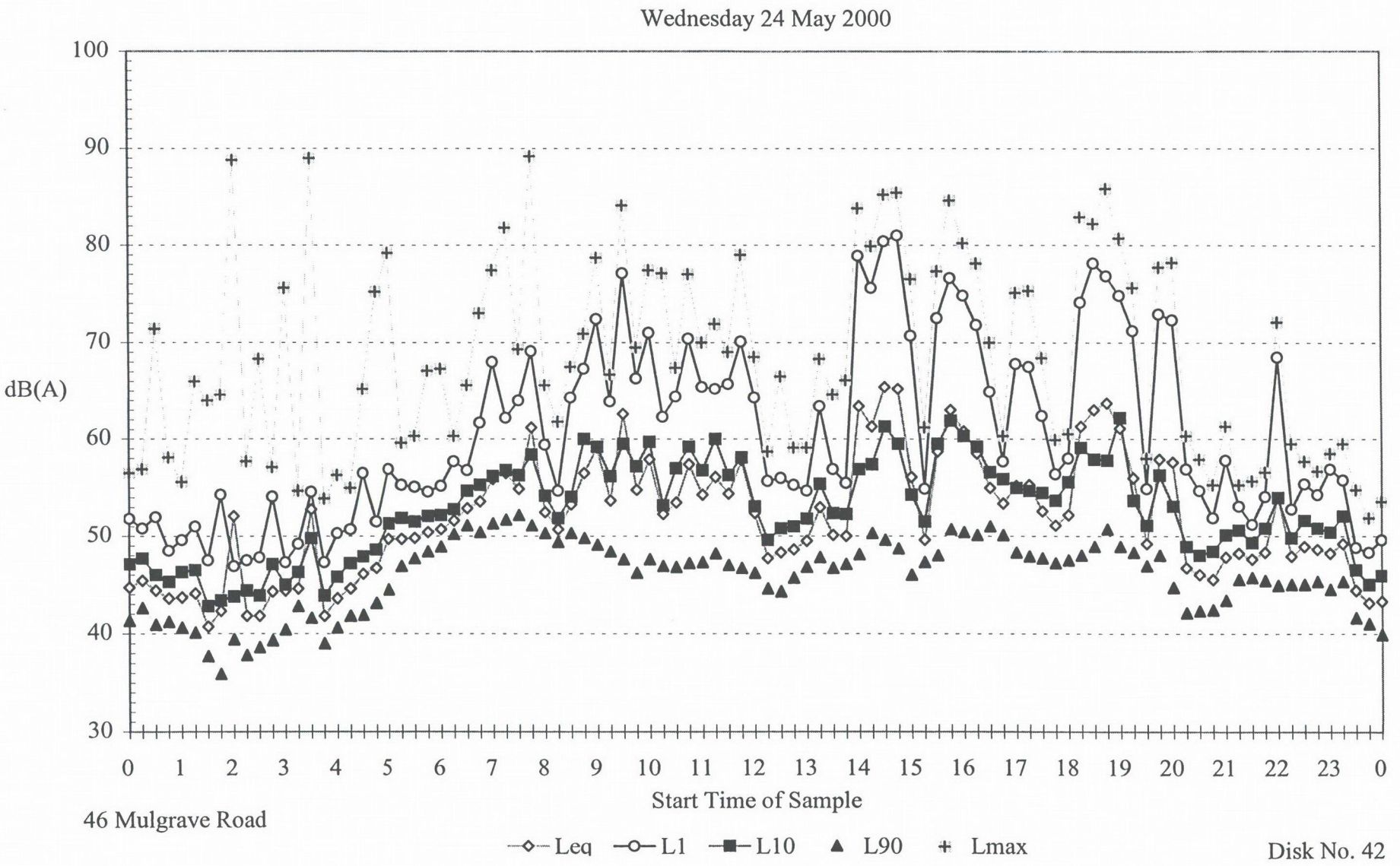


46 Mulgrave Road

Disk No. 42

5030r6.xls

Ambient Sound Pressure Levels



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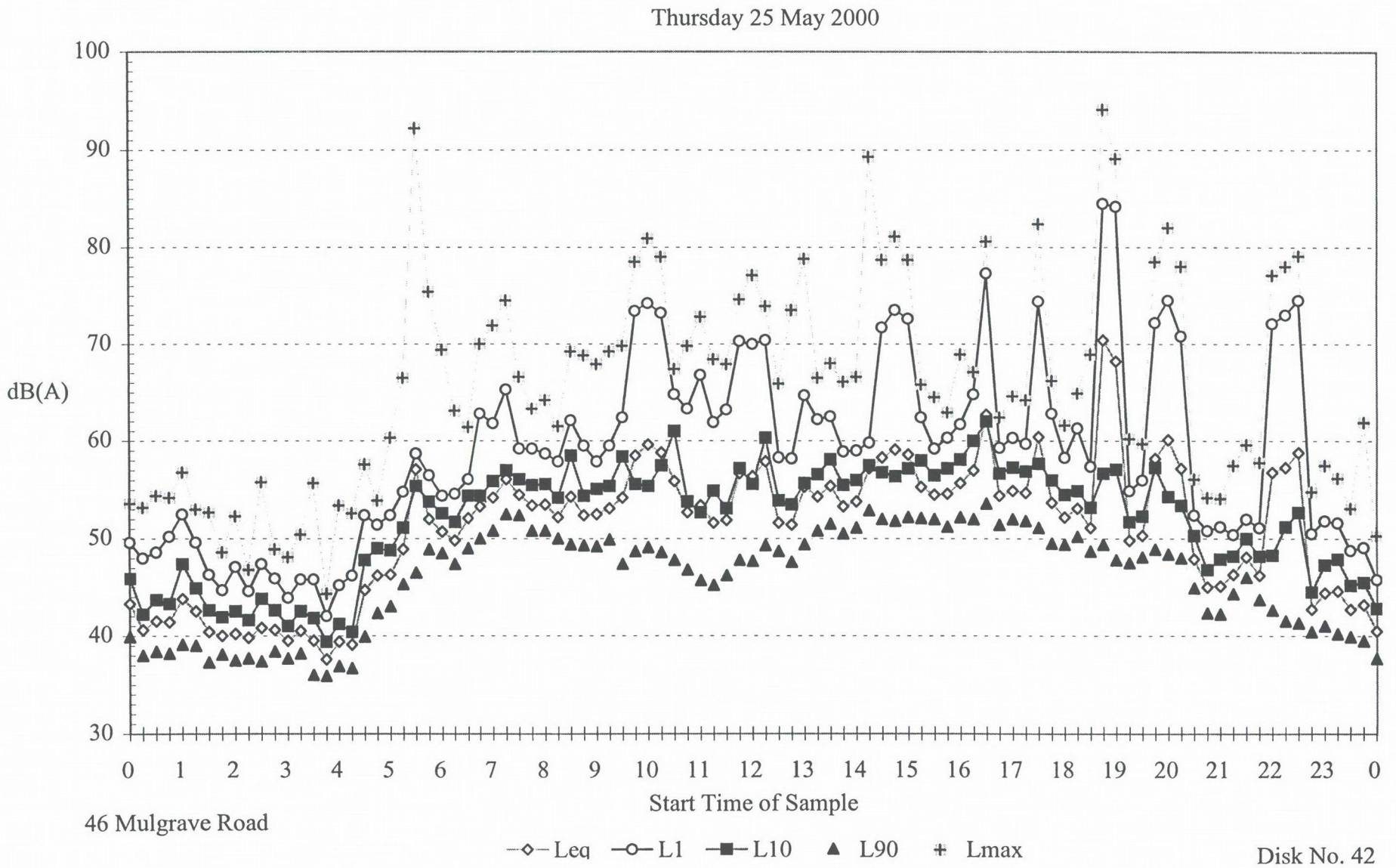
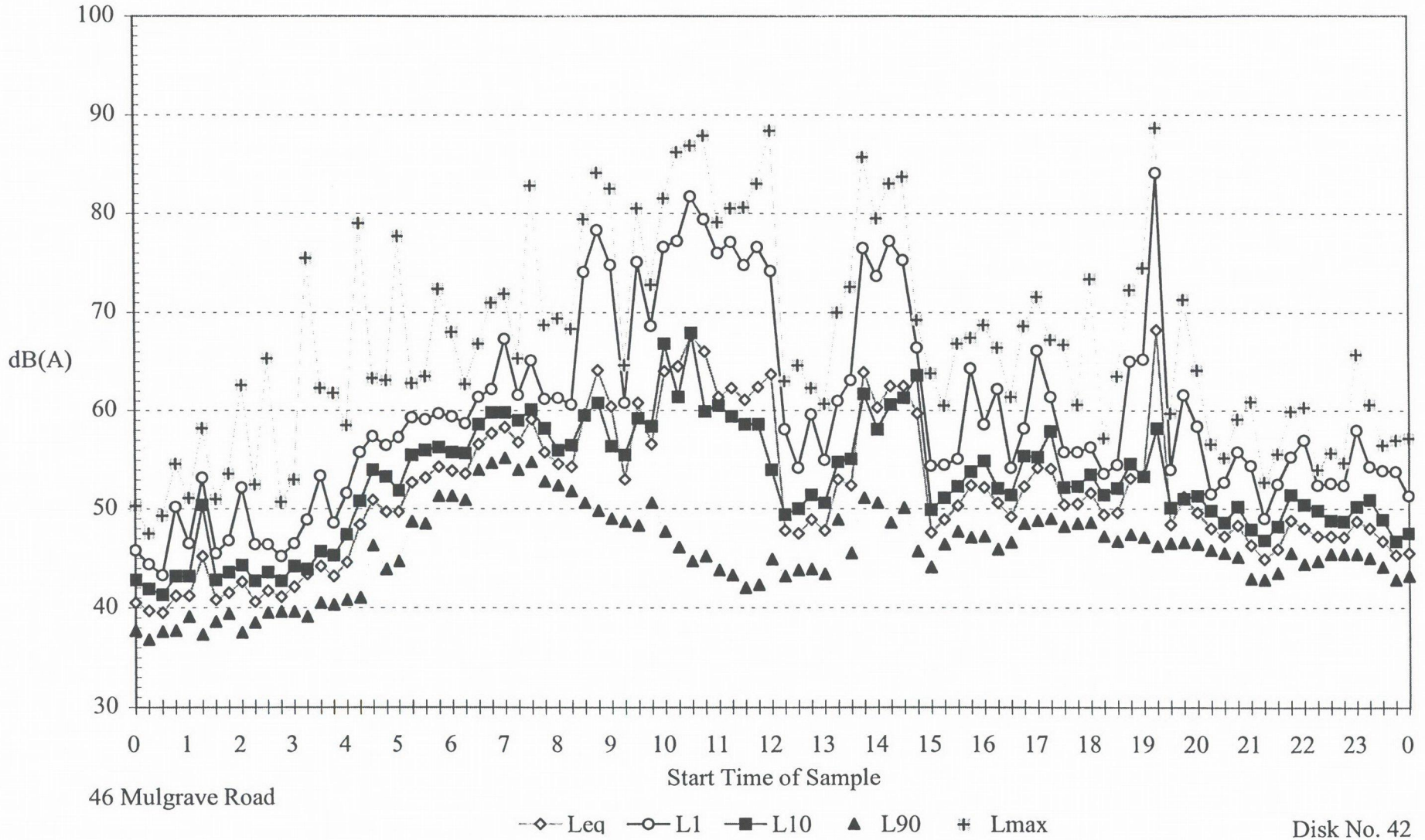
Ambient Sound Pressure Levels



5030r6.xls

Ambient Sound Pressure Levels

Friday 26 May 2000

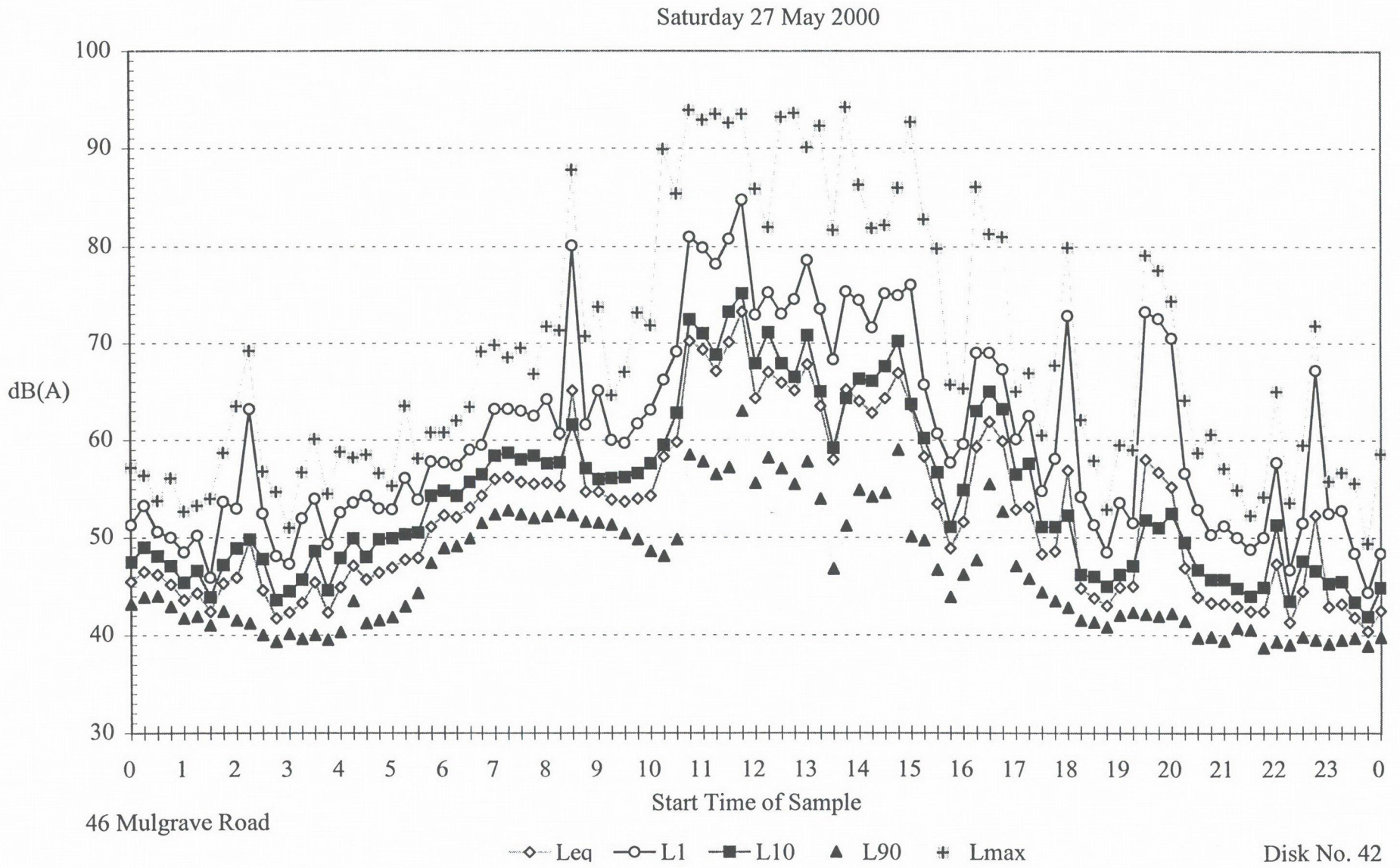


46 Mulgrave Road

Disk No. 42

5030r6.xls

Ambient Sound Pressure Levels



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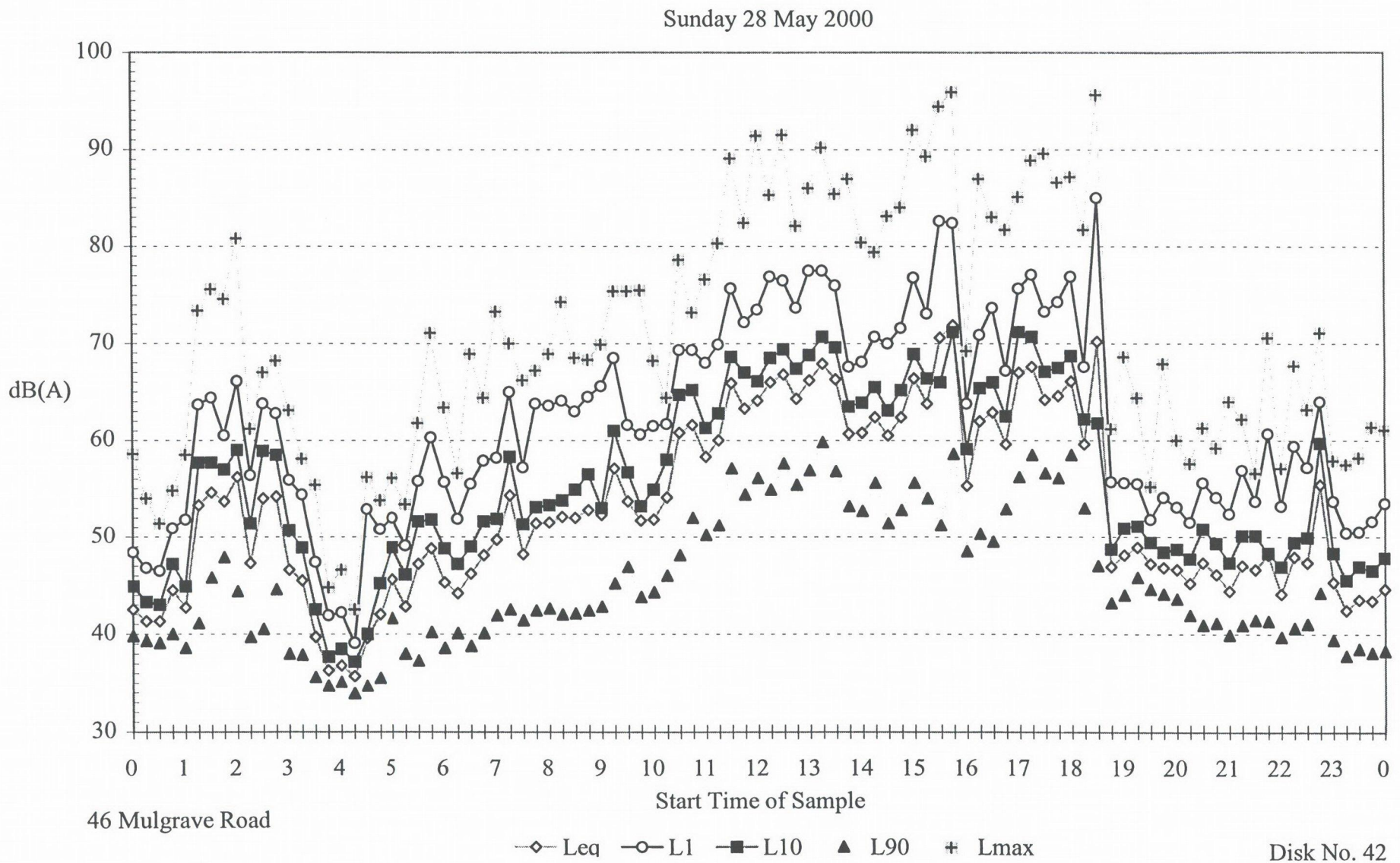
Ambient Sound Pressure Levels



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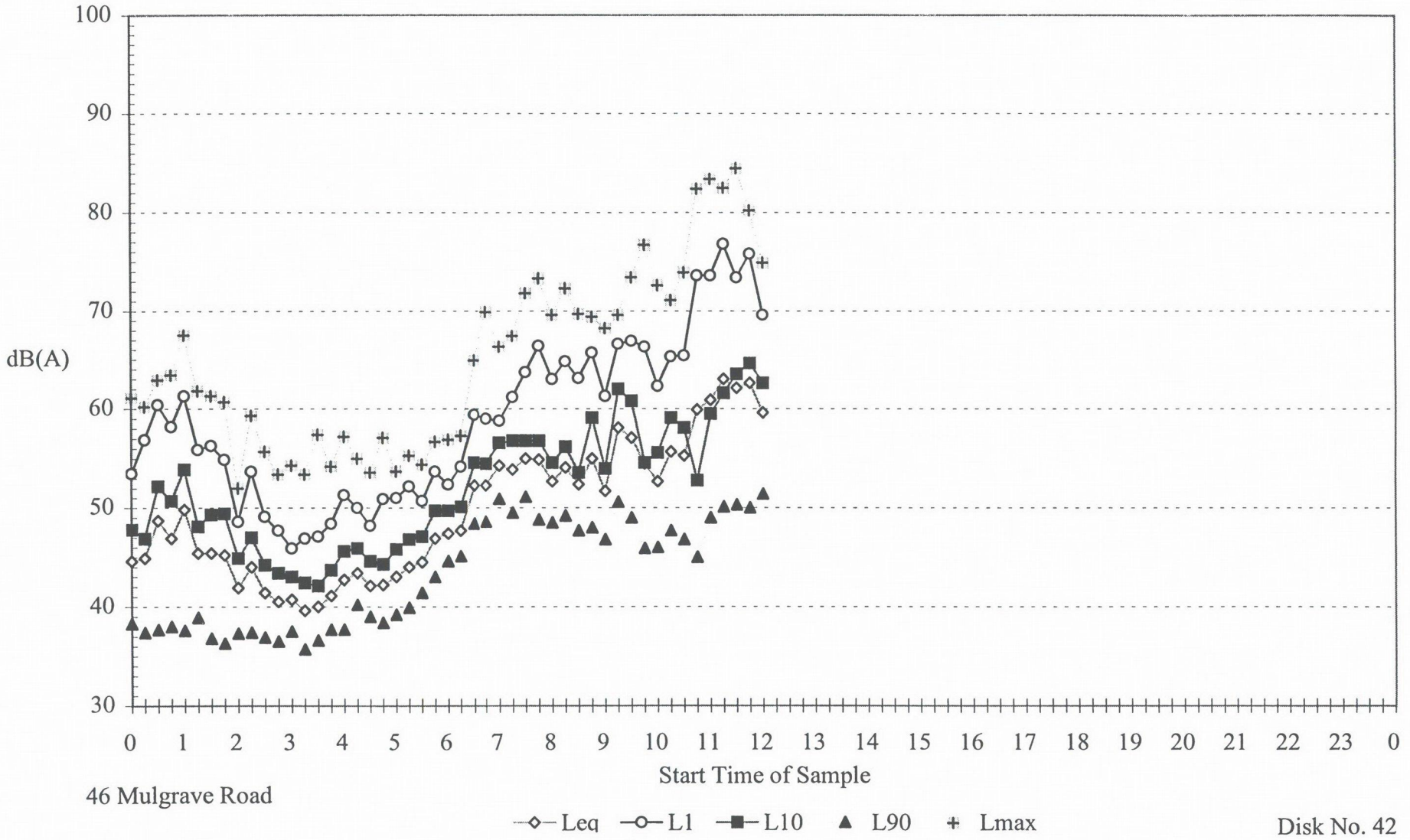
Ambient Sound Pressure Levels

Monday 29 May 2000

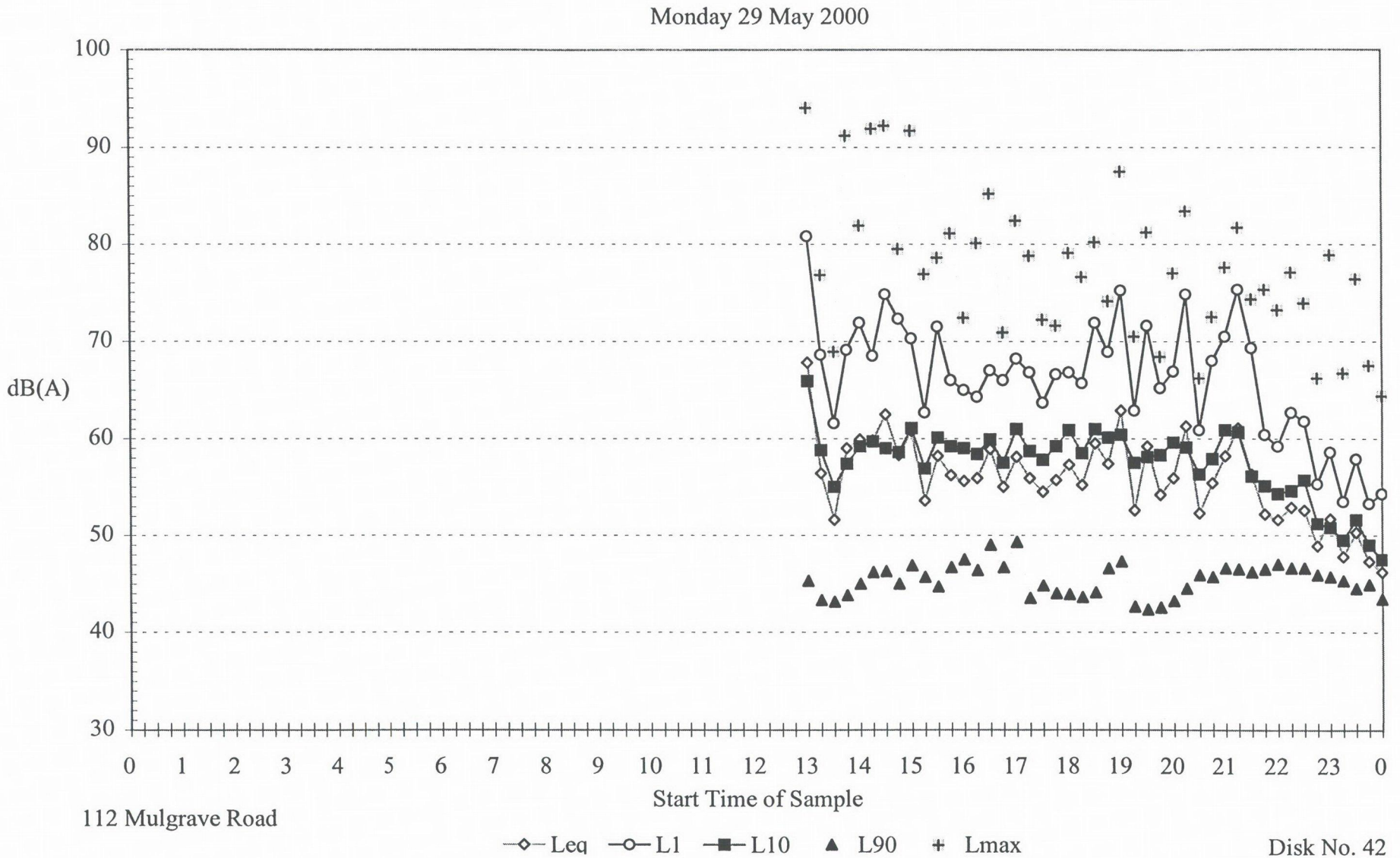


46 Mulgrave Road

Disk No. 42

5030r6.xls

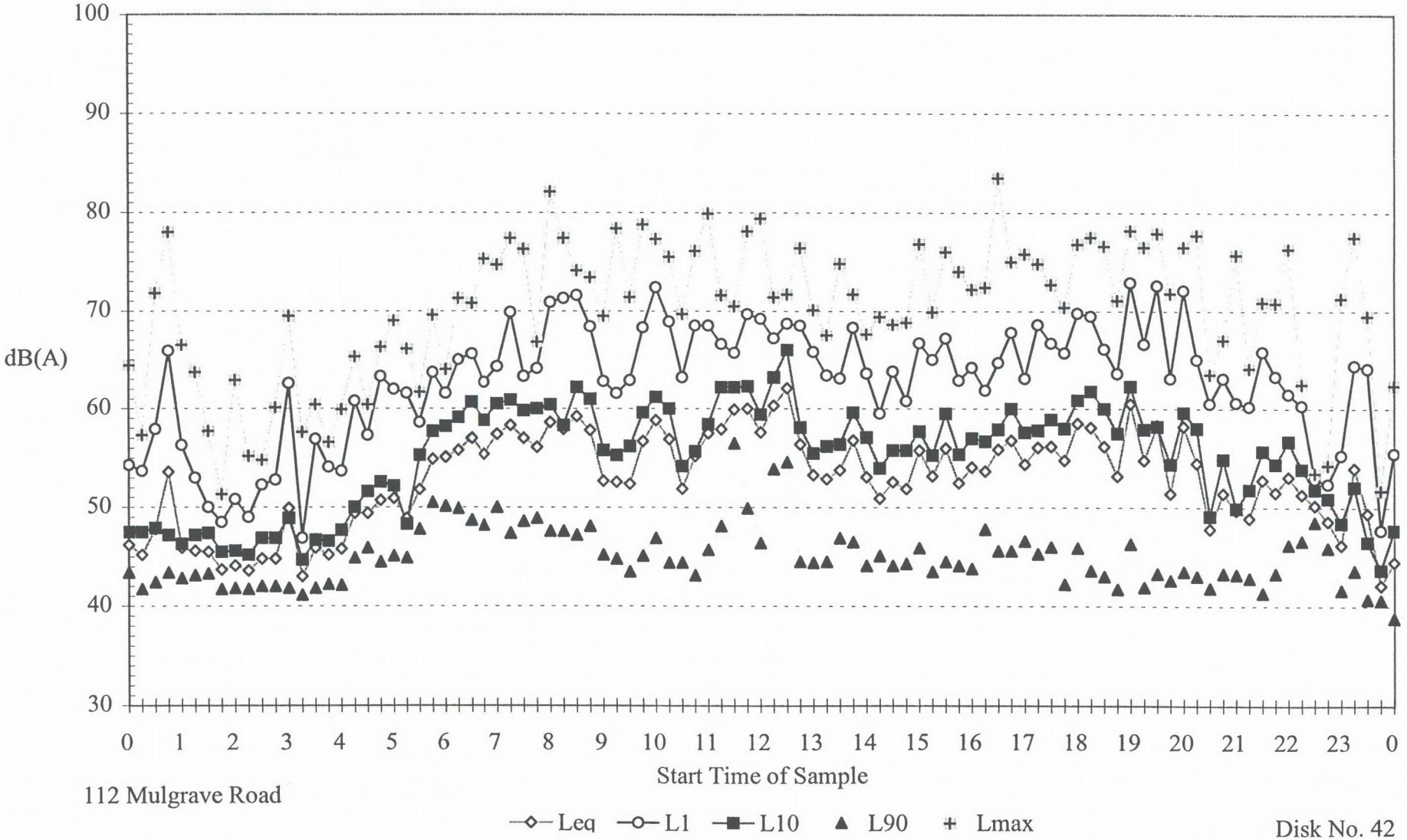
Ambient Sound Pressure Levels



5030r7.xls

Ambient Sound Pressure Levels

Tuesday 30 May 2000



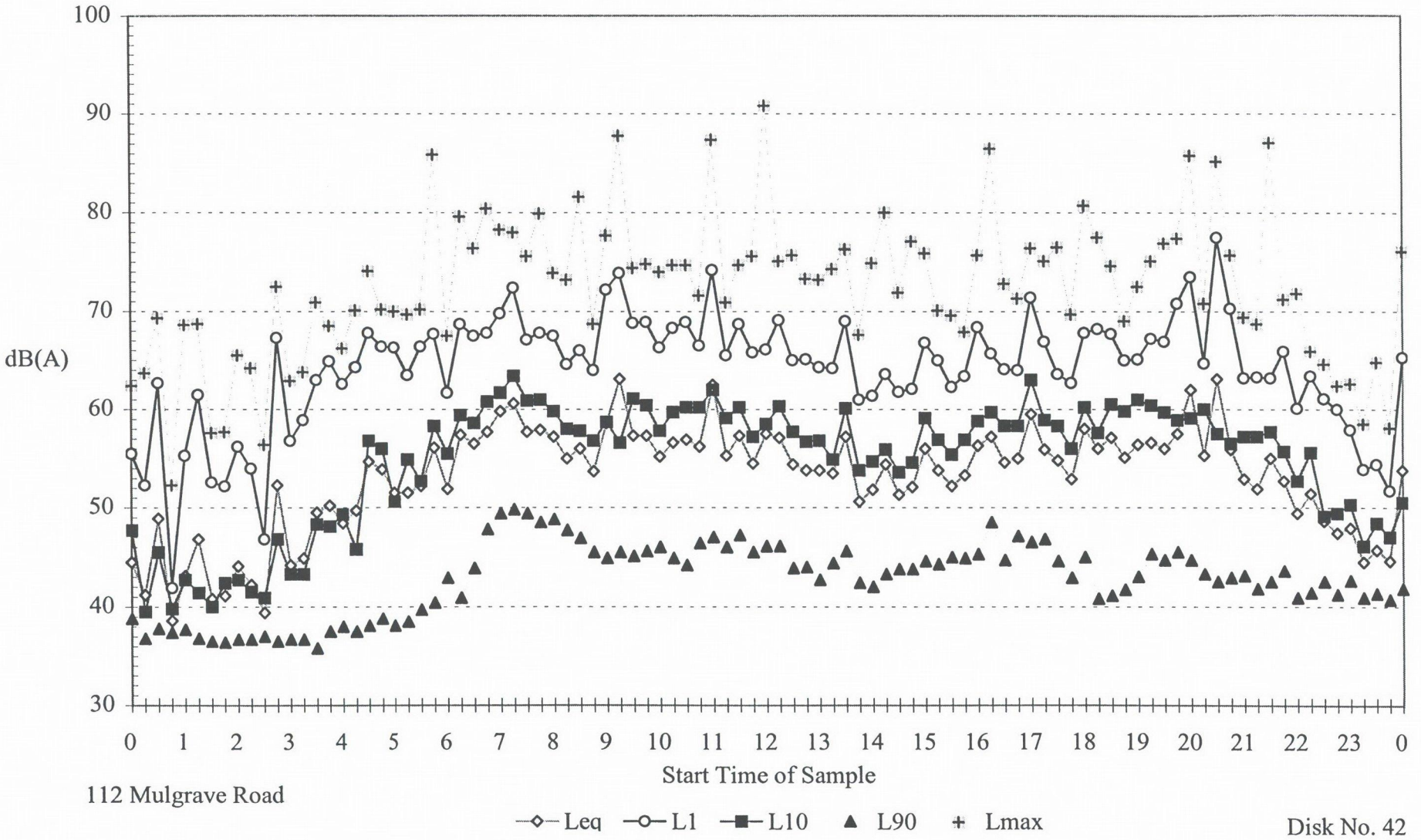
112 Mulgrave Road

Disk No. 42

5030r7.xls

Ambient Sound Pressure Levels

Wednesday 31 May 2000

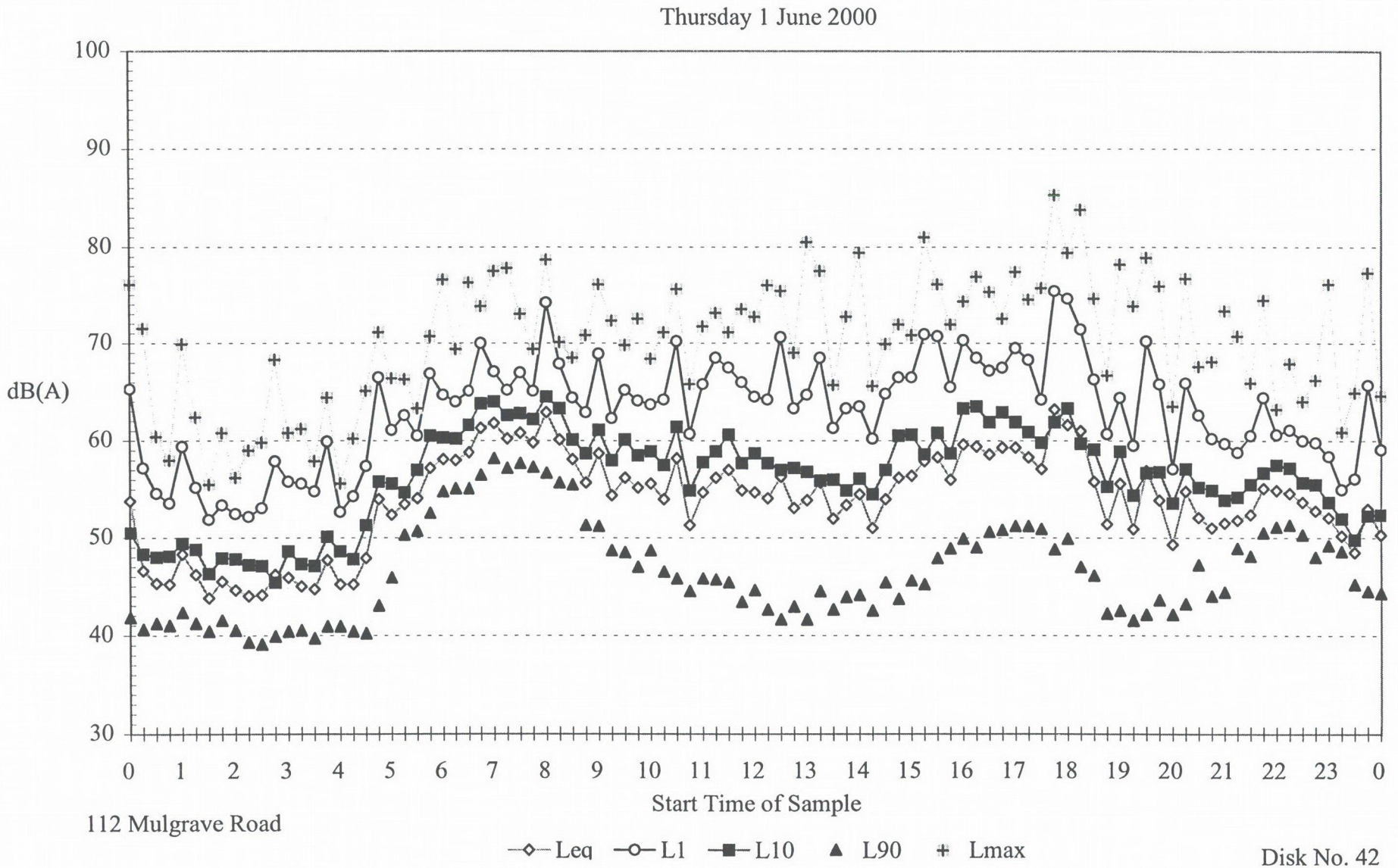


112 Mulgrave Road

Disk No. 42

5030r7.xls

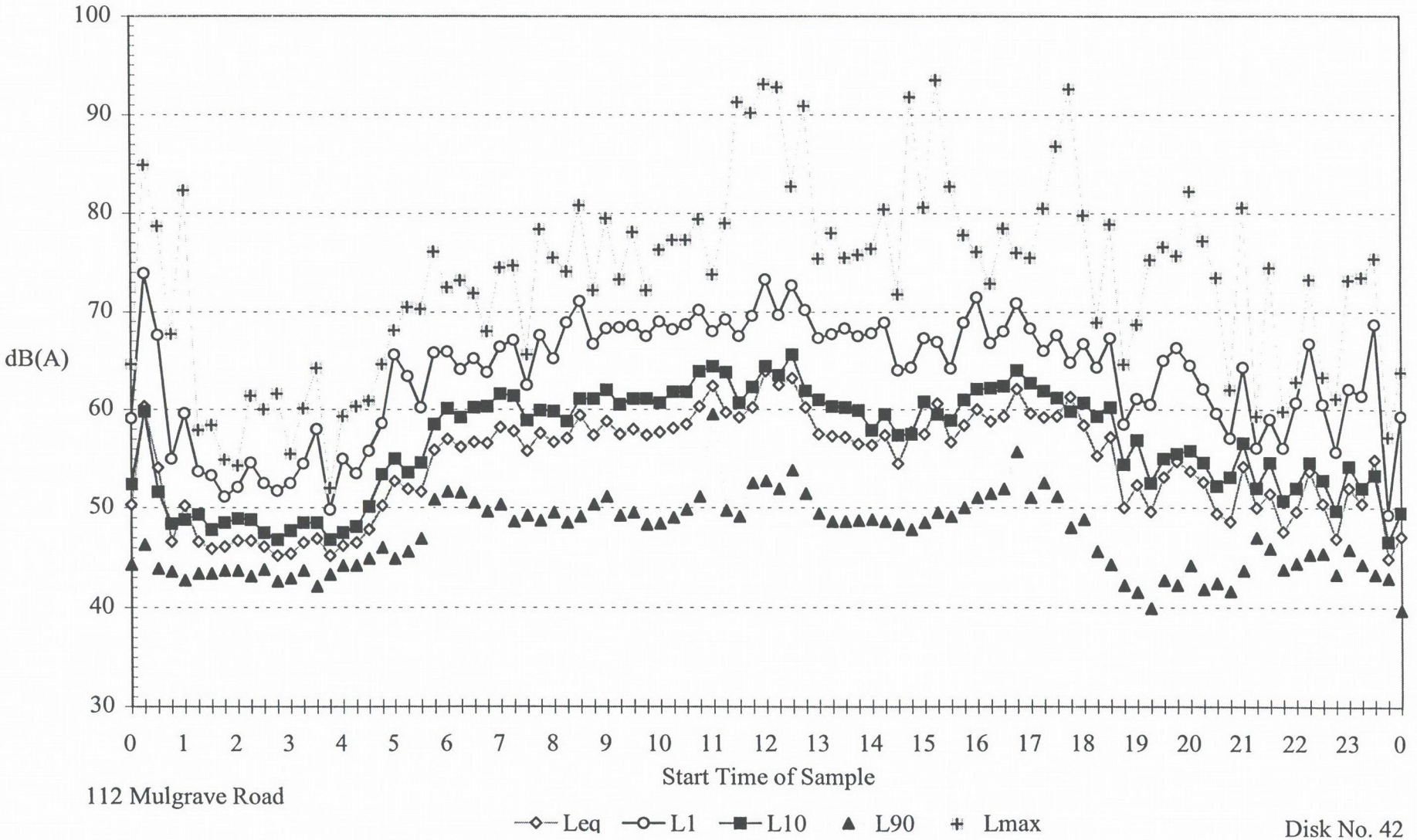
Ambient Sound Pressure Levels



5030r7.xls

Ambient Sound Pressure Levels

Friday 2 June 2000

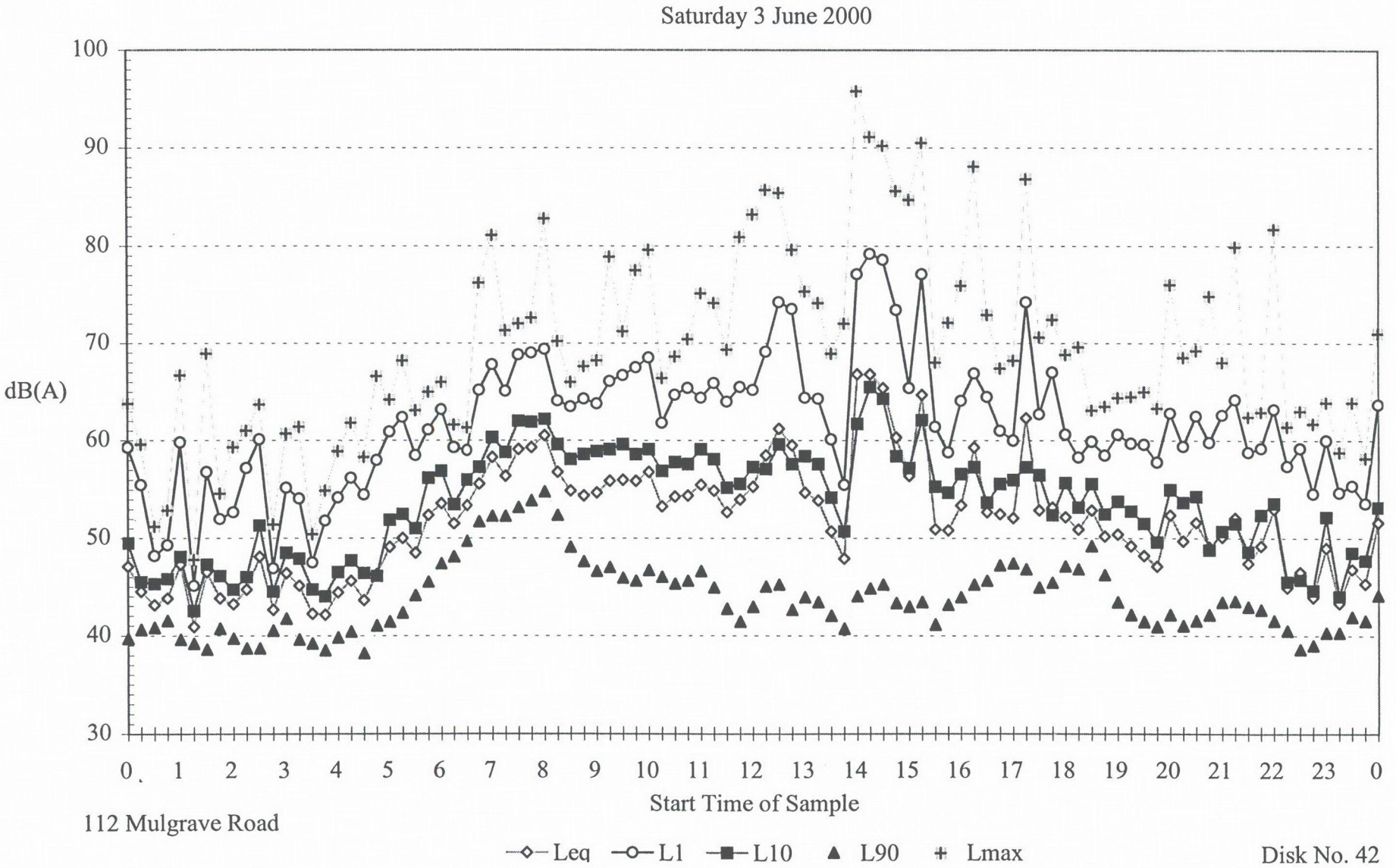


112 Mulgrave Road

Disk No. 42

5030r7.xls

Ambient Sound Pressure Levels



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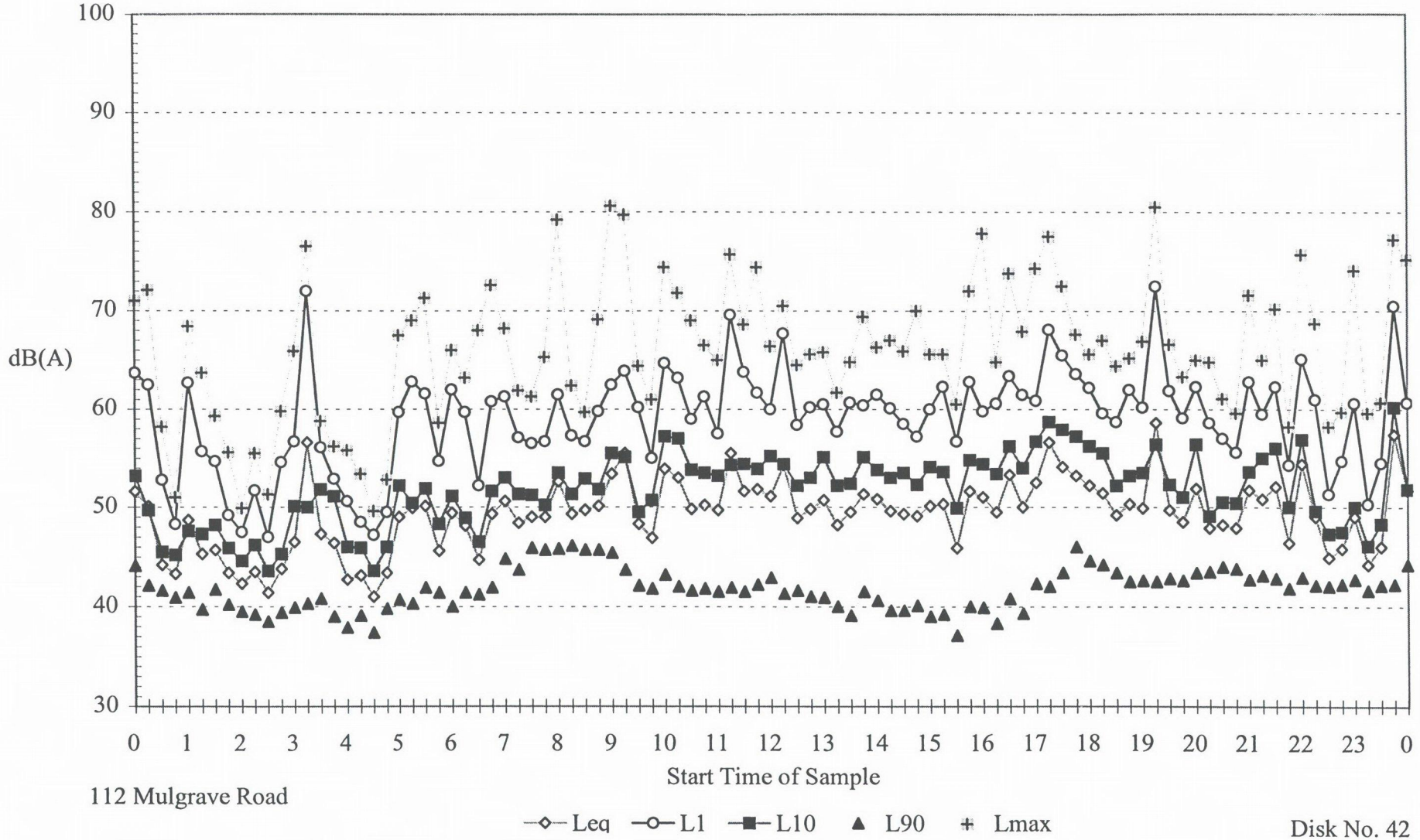
Ambient Sound Pressure Levels

Sunday 4 June 2000

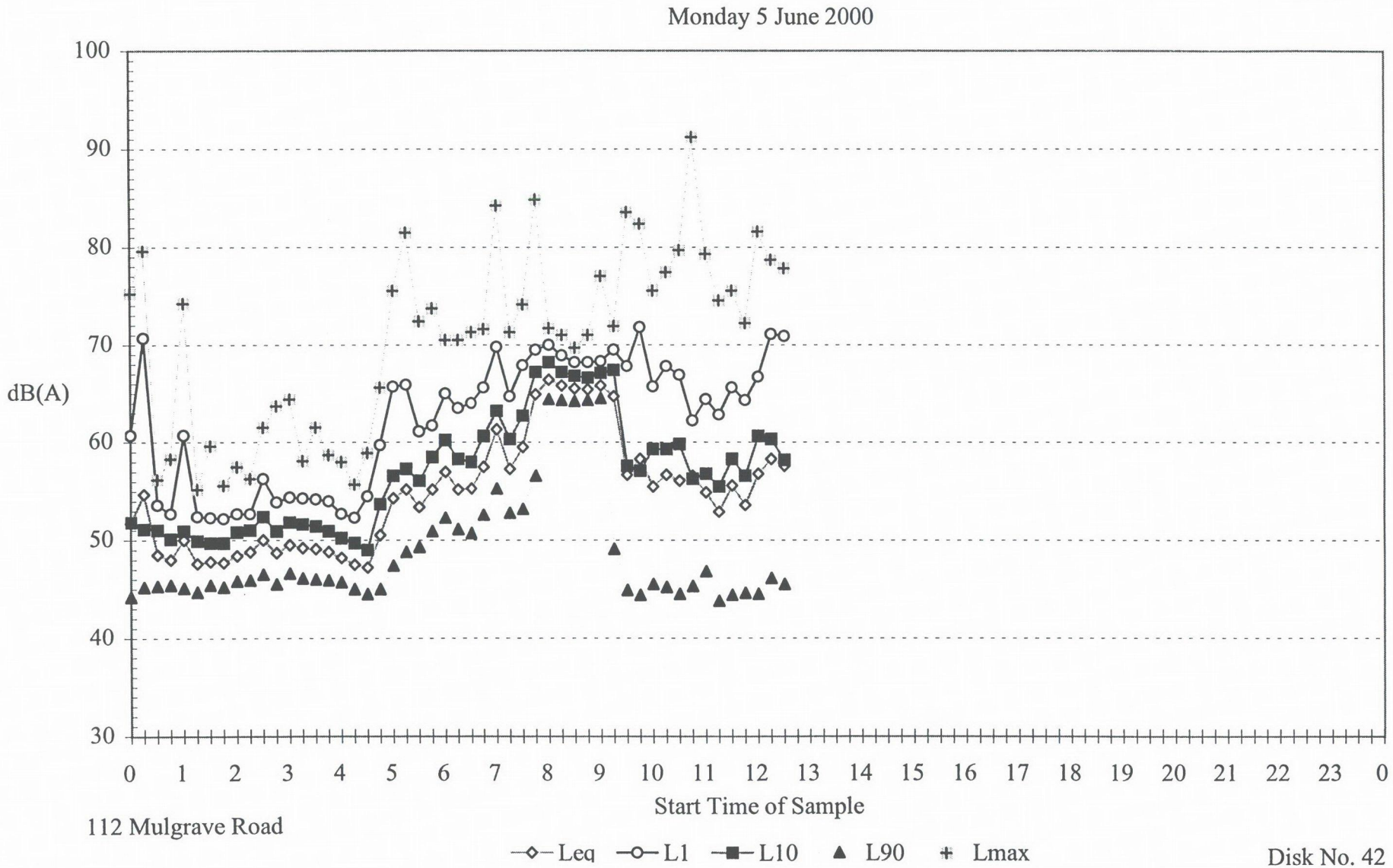


112 Mulgrave Road

Disk No. 42

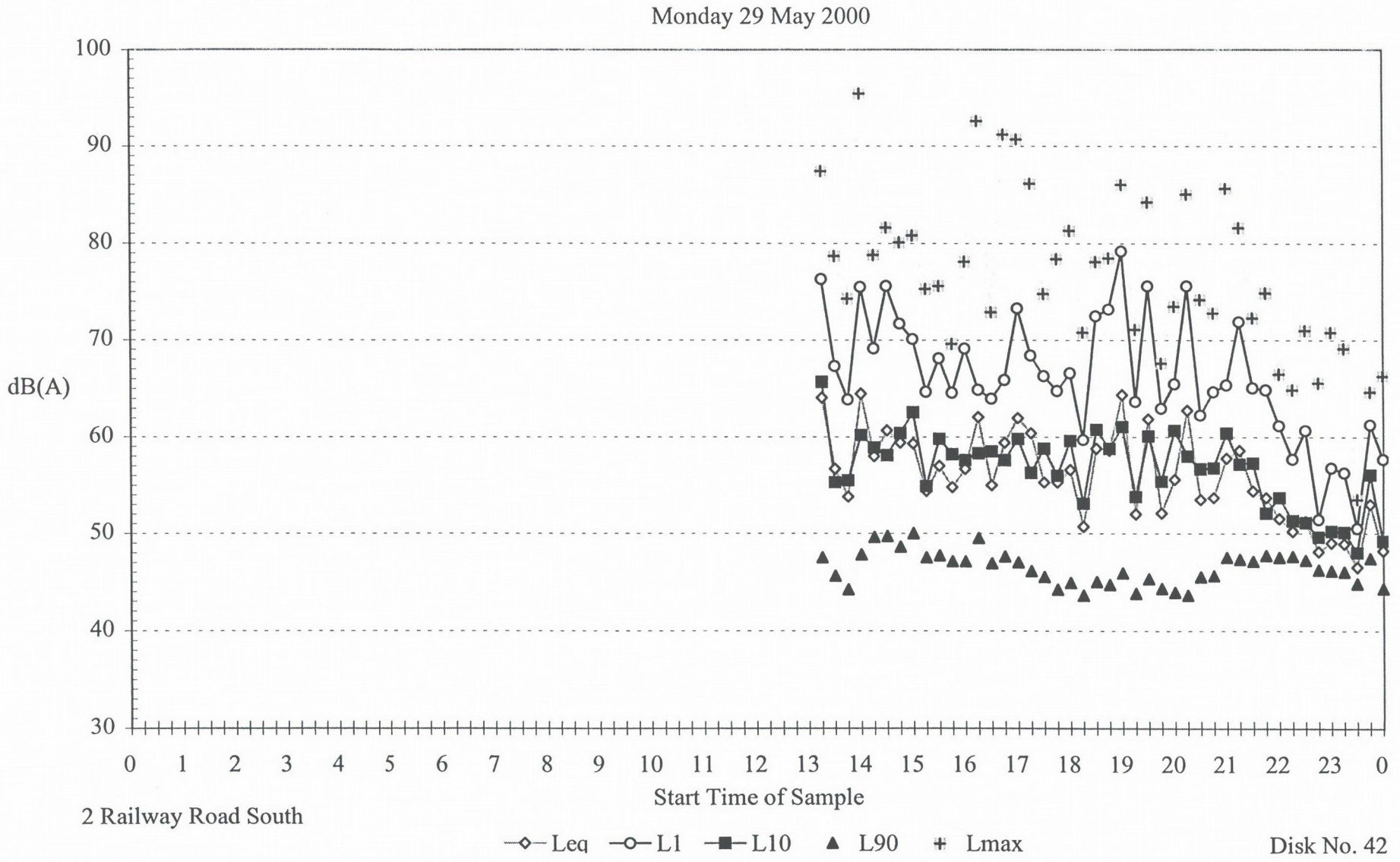
5030r7.xls

Ambient Sound Pressure Levels



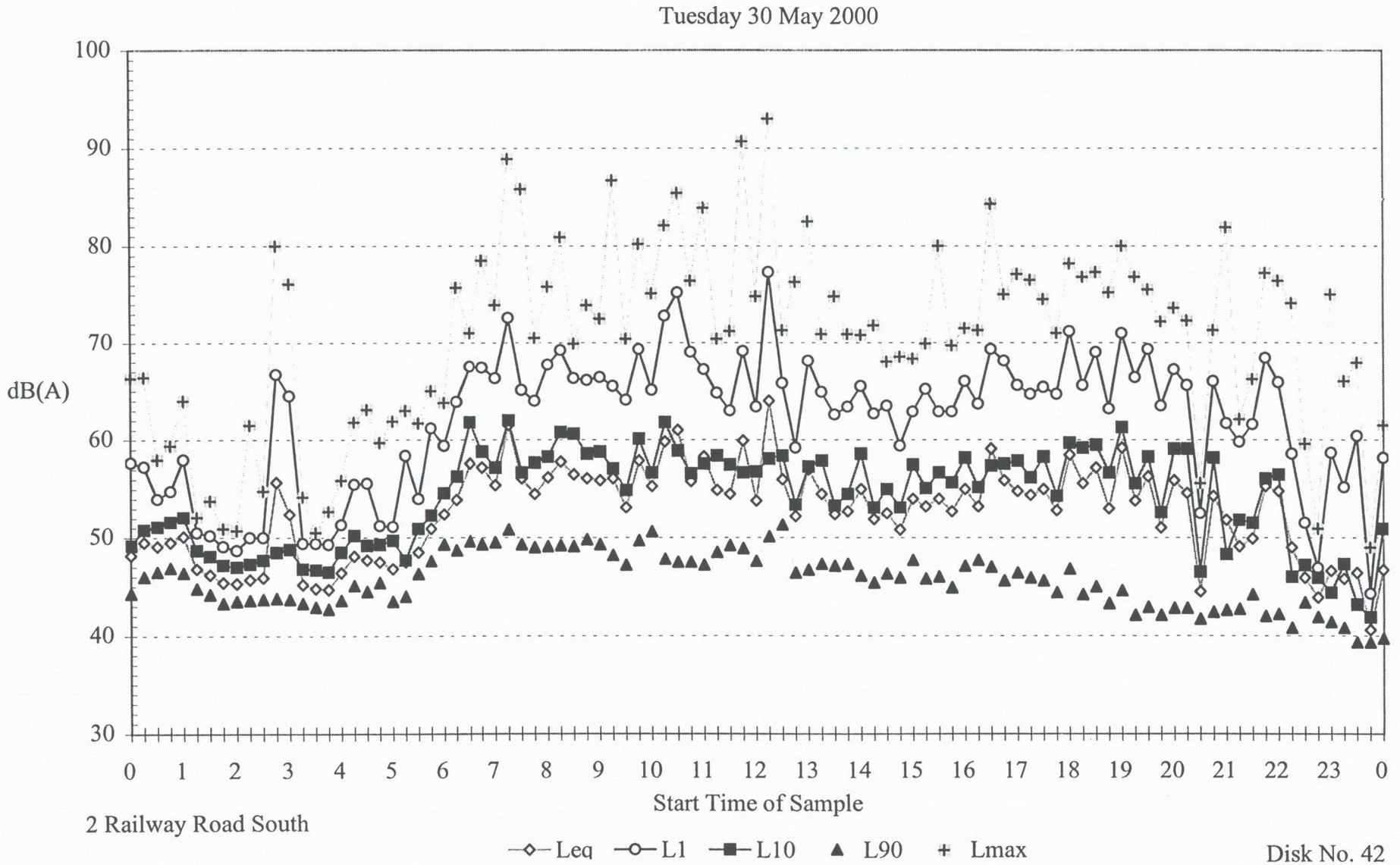
5030r7.xls

Ambient Sound Pressure Levels



5030r8.xls

Ambient Sound Pressure Levels



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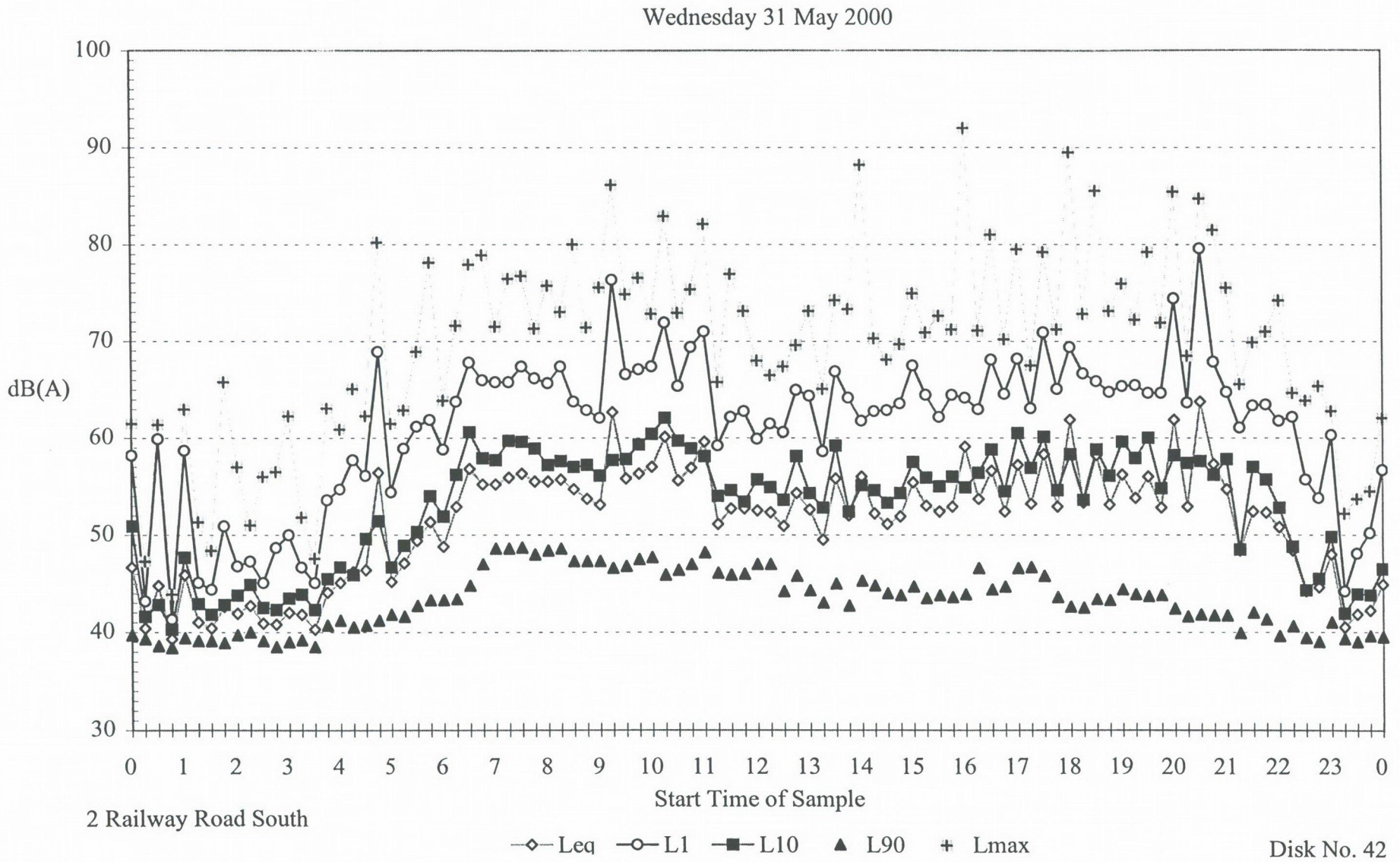
Ambient Sound Pressure Levels



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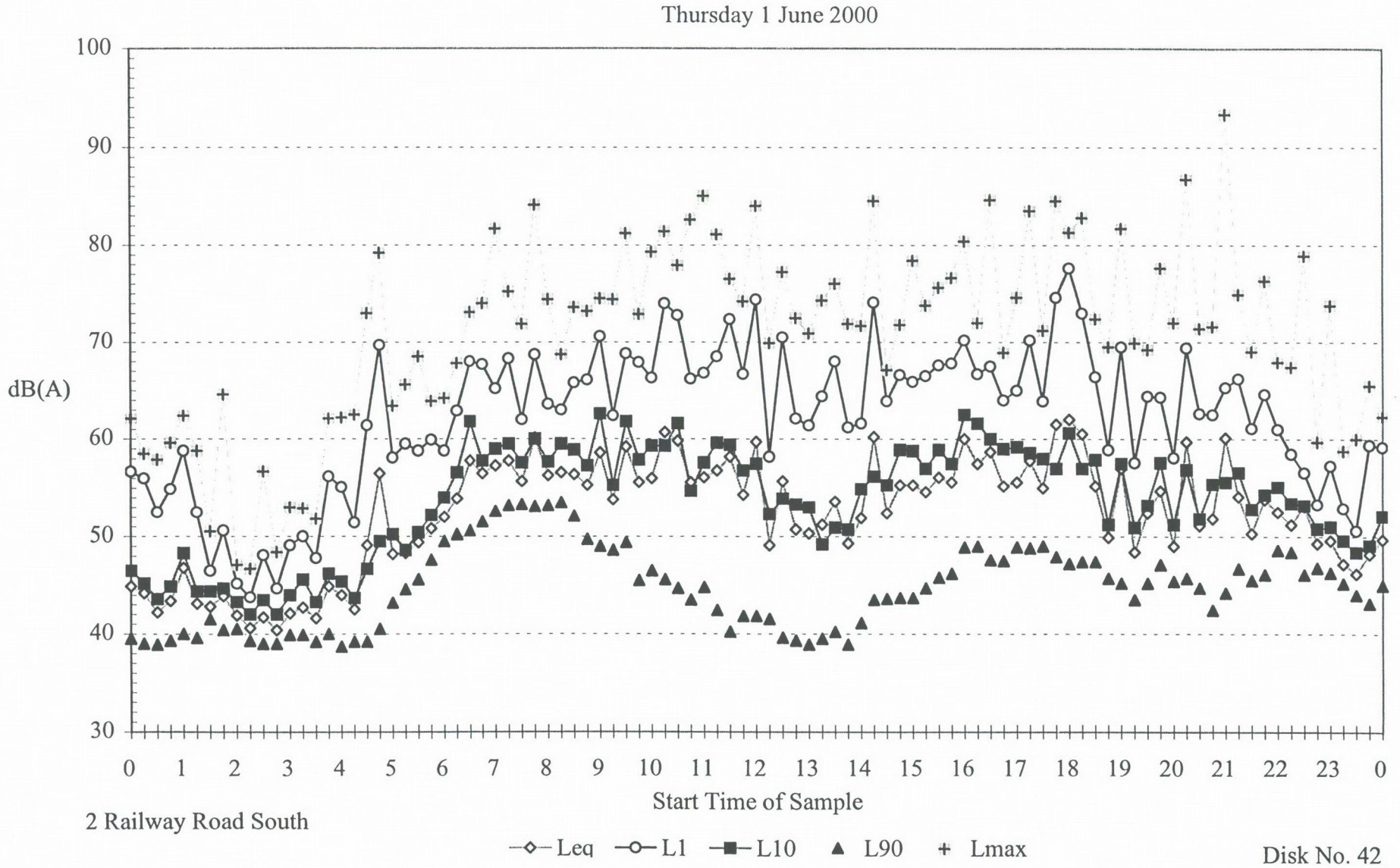
Ambient Sound Pressure Levels



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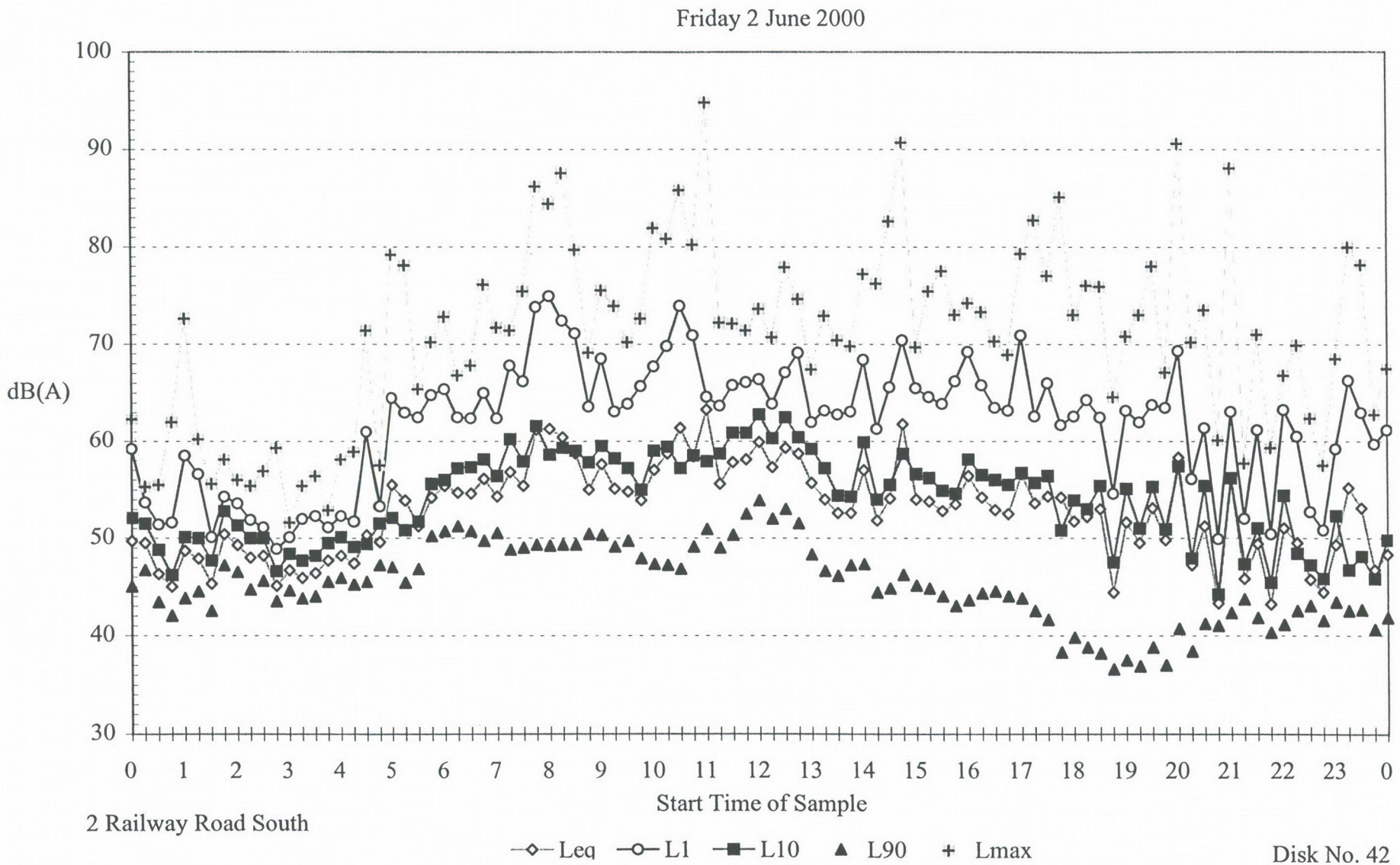
Ambient Sound Pressure Levels



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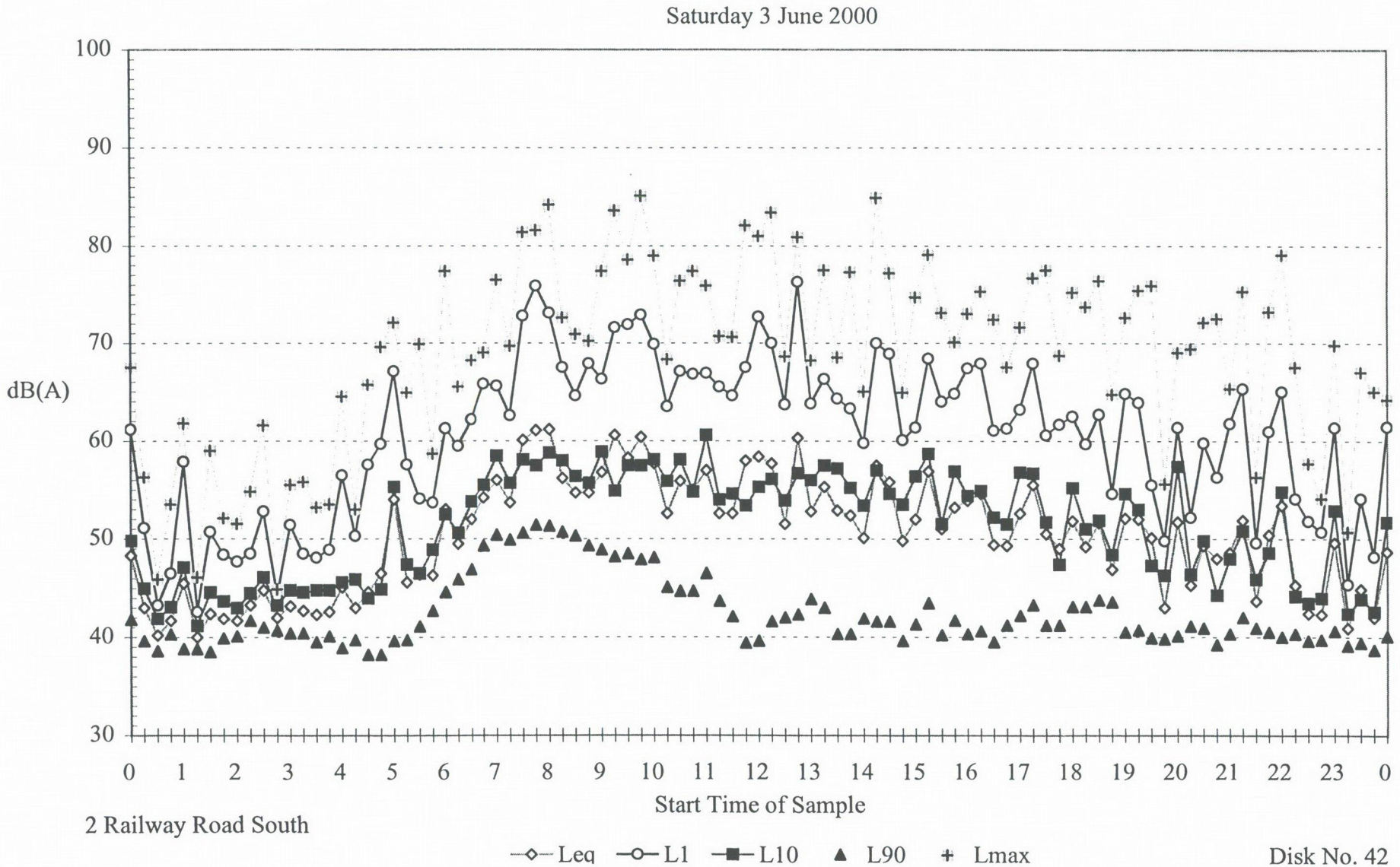
Ambient Sound Pressure Levels



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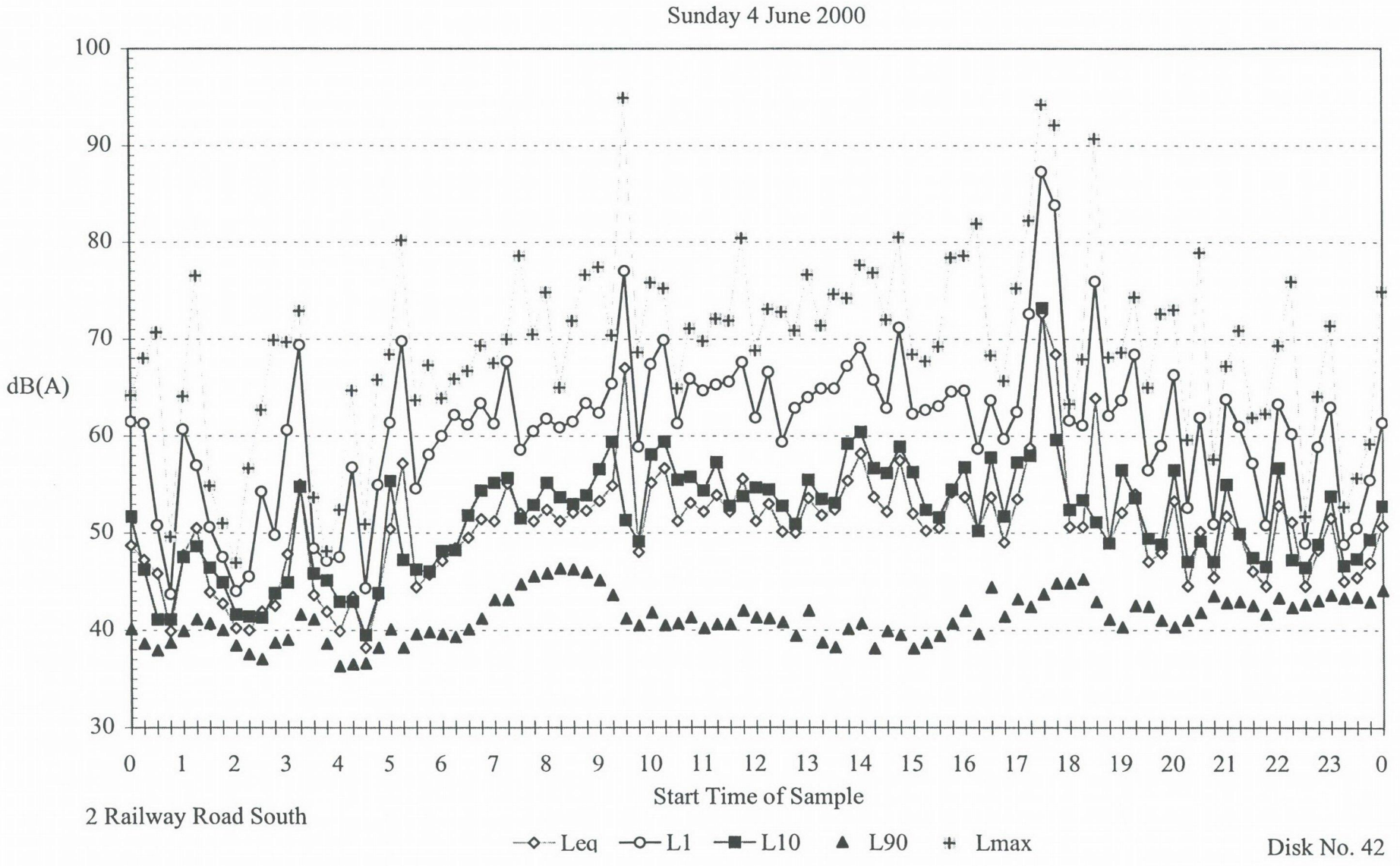
Ambient Sound Pressure Levels



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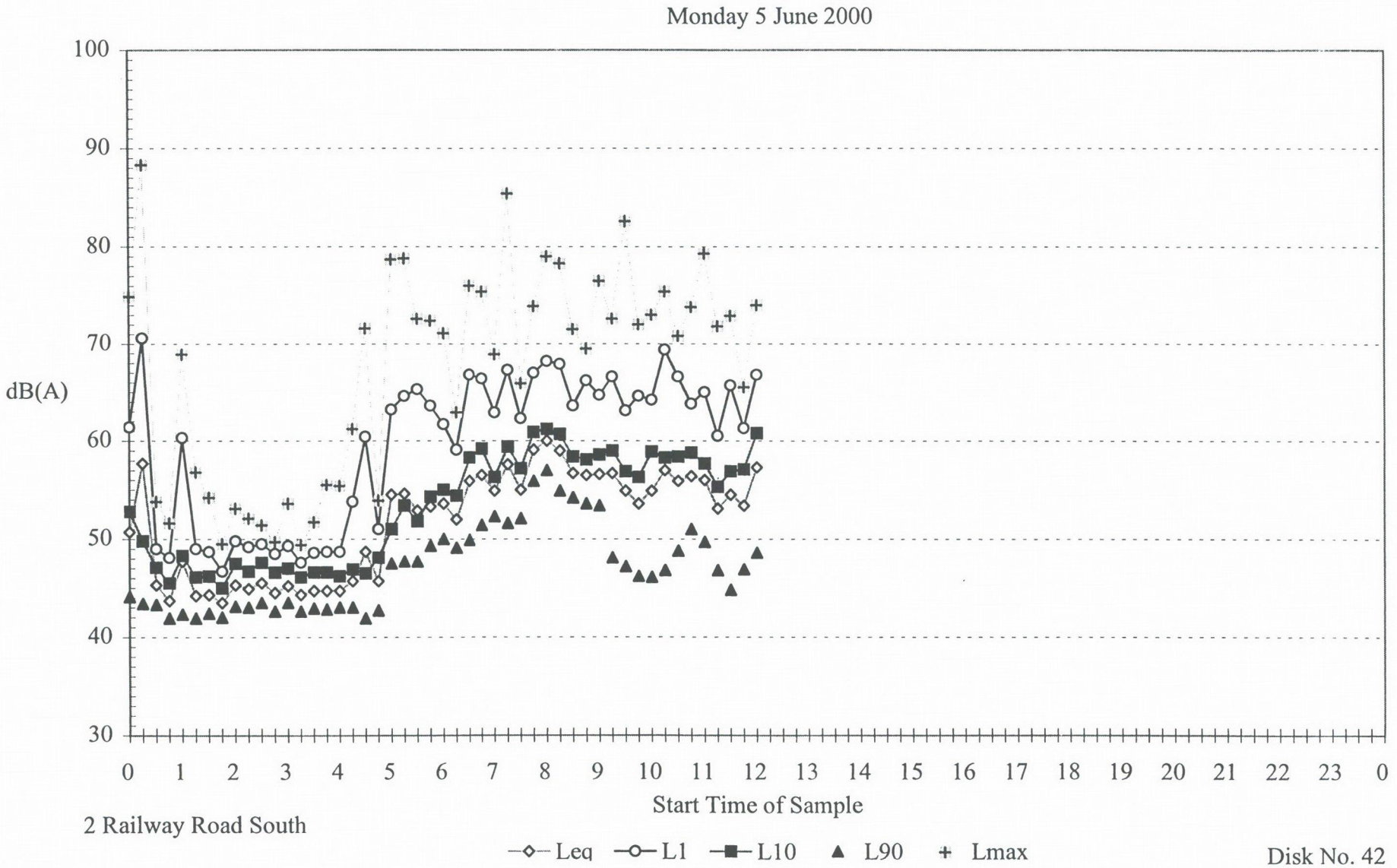
Ambient Sound Pressure Levels



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