REPORT

TO
WALTER PROJECTS PTY LTD
ATF WALTER DEVELOPMENT TRUST

ON
GEOTECHNICAL ASSESSMENT

FOR
PROPOSAL RESIDENTIAL PLANNING PROPOSAL

AT
3-31 WALTER STREET & 462 WILLOUGHBY ROAD
WILLOUGHBY, NSW

28 February 2017
Ref: 30088Zrpt Rev1

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Date: 28 February 2017
Report No: 30088Zrpt
Revision No: 1

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TABLE OF CONTENTS

1 INTRODUCTION

This report presents the results of a geotechnical assessment of the site for the proposed residential masterplan at 3-31 Walter Street Willoughby, NSW and also includes 462 Willoughby road for purpose of assessment. The assessment was commissioned by Mr Mo Chehelnabi of Architecture Urbaneia, on behalf of Walter Projects Pty Ltd, ATF Walter Development Trust, by signed 'Acceptance of Proposal' form dated 23 December 2016. The commission was on the basis of our proposal (Ref P44091Z Willoughby) dated 23 December 2016.

We understand from the attachments to the Mo Chehelnabi emailed RFP dated 28 November 2016, that the proposed masterplan and planning proposal will include rezoning the site from R3 to R4, together with the construction of a number of five to eight storey residential buildings with surrounding integrated landscaping.

The purpose of the assessment was to obtain geotechnical information on subsurface conditions from a desktop study, as a basis for preliminary comments and recommendations on excavation conditions, shoring, retaining walls, footings, and on-grade floor slabs.

We note that our environmental division, Environmental Investigation Services (EIS), were commissioned to carry out a desktop contamination assessment of the site. The geotechnical report should be read in conjunction with the EIS report (Ref E30088KPrpt).

2 ASSESSMENT METHODOLOGY

A desktop study of previous geotechnical investigations we have carried out in the immediate vicinity was conducted. The neighbouring sites for which relevant geotechnical information was available included:

A Small Street, Willoughby.
B 2 Small Street, Willoughby.
C 79 Garland Road, Naremburn.
D 281 Willoughby Road, Naremburn.
E Channel 9, Richmond Avenue, Willoughby.
F 31 Chelmsford Road, Willoughby.

A brief site visit was also completed in order to assess the topographic and drainage setting and interface with adjoining buildings and structures.

A site description, based on our site visit, is presented in Section 3 below. The anticipated subsurface conditions, based on the above investigations, are presented in Section 4, and our preliminary comments and recommendations regarding the proposed masterplan, are discussed in Section 5.

3 SITE DESCRIPTION

The attached Figure 1 presents a site location plan. The site is located over a south-east facing hillside at the intersection between Walter Street to the south and Willoughby Road to the east, and covers a total area of approximately 1.28 hectares.

At the time of our assessment, the site was occupied by numerous one and two storey houses with associated driveways, garages and landscaping, including numerous trees, particularly along the street frontages and over the north-west. Outcropping sandstone was evident at the street frontage between No 5 and No 7 Walter Street.

Residential unit buildings were located to the north and individual houses were located across the eastern half of Walter Street to the south. Vacant land was located beyond the eastern end of the northern site boundary and beyond the western end of Walter Street to the south. The Channel 9 premises were located a short distance to the north-west and included several buildings, a telecommunication tower and dishes. The Centennial Reserve oval was located across Willoughby Road to the east.

4 SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is underlain by Hawkesbury Sandstone. Manmade fill over Hawkesbury Sandstone is indicated immediately across Willoughby Road to the east and the higher lying Ashfield Shales are indicated a short distance to the north, west and south.

Subsurface conditions established during the previous investigations in the surrounding area indicated that where sandstone bedrock was not outcropping at surface, it was present at shallow depth (up to 1.3m) and it was overlain by fill and residual clayey sand. The sandstone bedrock was generally assessed to be of at least low strength and improved to medium and high strength with depth. Groundwater was not encountered within the depths previously investigated.

The exception to the above occurred to the east of the site where fill up to 7m deep was encountered over the sandstone bedrock.

5 PRELIMINARY COMMENTS AND RECOMMENDATIONS

The comments and recommendations which follow are general in nature and have been based on subsurface conditions inferred from previous geotechnical investigations carried out in the vicinity of the site. Whilst the information which follows is considered to be adequate for planning purposes, feasibility studies, and preliminary structural design, we recommend that a comprehensive geotechnical and hydrogeological investigation be carried out to confirm the subsurface conditions, as detailed in Section 5.7 below. Once the architectural details are available and the above investigation has been completed, this report should be reviewed and revised, if appropriate.

5.1 Geotechnical Issues

The principal geotechnical issue associated with the proposed R4 development of the subject site relates to the anticipated shallow depth to sandstone bedrock. Whilst the sandstone bedrock will provide suitable founding material and can generally be excavated with vertical batters, hard rock excavation conditions must be anticipated for any proposed basements, as well as the need to control the associated ground vibrations.

The above issues are discussed in further detail in the sections that follow.

However, we consider that the site is suitable for the development of buildings and structures associated with the proposed R4 rezoning. Such buildings and structures can be constructed using techniques which are used extensively and are familiar to most of the appropriate builders in the Sydney region.

5.2 Excavation Conditions

5.2.1 Excavation Methods

We are unaware of what excavations need to be undertaken on the site. However, it is evident that at least some excavations will be required for the sloping site, particularly if basements are proposed. The proposed bulk excavations will probably encounter the soil profile and extend into the sandstone bedrock.

This soil cover should be readily excavatable using conventional earthworks equipment (eg. hydraulic excavators). Some of the underlying weathered sandstone of extremely or very low strength, if encountered, may also be excavated by large bucket excavator, possibly with some ripping. However, we expect excavation of low to medium and higher strength sandstone would be most effectively excavated using hydraulic impact rock hammers. This equipment would also be required for breaking up boulders or blocks, for trimming rock excavation side slopes, and for detailed rock excavations (such as for footings or buried services).

Care is required during excavation to avoid undermining any adjacent buildings and structures.

5.2.2 Excavation Techniques

We recommend that considerable caution be taken during rock excavation on this site as there will likely be direct transmission of ground vibrations to surrounding buildings and structures. Prior to excavation commencing, detailed dilapidation reports should be compiled on buildings and structures within the zone of influence of the proposed excavation, taken as two times the excavation depth or a minimum of 15m from the excavation perimeter. The dilapidation reports should be provided to the owners of the relevant properties who should be asked to confirm that the reports present a fair record of existing conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage as a result of the works. The excavation procedures and the dilapidation reports should be carefully reviewed prior to excavation commencing, so that appropriate equipment is used.

Excavation with hydraulic rock hammers, if used, should preferably commence away from likely critical areas (ie. over the central portions of the site) using a moderately sized excavator, fitted with a relatively low energy hydraulic hammer no larger than a Krupp 900 size, or equivalent. We recommend that continuous vibration monitoring be carried out during rock excavation. Subject to review of the dilapidation reports, we recommend that vibrations, measured as Peak Particle Velocity (PPV), be limited to no higher than 5mm/sec on the surrounding residential buildings and structures within 20m of the excavation perimeter. Should higher vibrations be measured, they should be assessed against the attached Vibration Emission Design Goals, as higher vibrations may be acceptable, depending on the associated vibration frequencies. Where excessive vibrations are confirmed, it will be necessary to change to a considerably smaller rock hammer or to use alternative excavation techniques.

Alternative excavation techniques which will significantly reduce vibrations include the provision of a vertical saw cut slot along the perimeter of the excavation and then maintaining the base of the slot at a lower level than the adjoining rock excavation at all times. Also, the use of a rotary grinder or grid sawing in conjunction with rock hammering and/or ripping may be considered. When using a rock saw or rotary grinder, the resulting dust must be suppressed by spraying with water.

The following procedures are recommended to reduce vibrations if rock hammers or used:

- Maintain rock hammer orientated towards the face and enlarge excavation by breaking small wedges off the faces.
- Operate one hammer at a time and in short bursts only to reduce amplification of vibrations.
- Use excavation contractors with experience in confined work with a competent supervisor who is aware of vibration damage risks, possible rock face instability issues, etc. The contractor should be provided with a copy of this report and have all appropriate statutory and public liability insurances.

5.2.3 Seepage

Although groundwater was not encountered during our previous investigations, some groundwater seepage flows may occur at the soil-rock interface and through joints and bedding planes within the completed cut faces, particularly after periods of heavy rain. Seepage, if any, during excavation is expected to be localised, of limited volume, and easily controlled by conventional sump pumping.

We recommend that a toe drain be formed at the base of all cut rock faces to collect groundwater seepage and direct it to a sump for pumped disposal. We further recommend that groundwater seepage into the excavation be monitored by a geotechnical engineer, so that any unexpected conditions can be timeously addressed.

5.3 Excavation Support

Where space permits, excavations in the soil profile and extremely weathered sandstone bedrock may be temporarily battered to a side slope no steeper than 1 Vertical (V) in 1.5 Horizontal (H) and 1V in 1H, respectively. Where temporary batters cannot be accommodated or where they are not preferred, a retention system will be required and will probably need to be installed prior to excavation commencing.

We expect that good quality sandstone of low or higher strength may be cut vertically. However, localised stabilisation measures may be required if adverse defects (such as inclined joints or bedding) are found. Treatment for zones requiring stabilisation may include rock bolting, shotcreting, underpinning, etc. Clay seams occurring in permanently exposed sandstone slopes my also require 'dental' treatment. We therefore recommend that the rock faces be progressively inspected by a geotechnical engineer/engineering geologist as excavation proceeds, to identify adverse defects and to propose appropriate stabilisation measures.

5.4 Retaining Walls

The major consideration in the selection of earth pressures for the design of retaining walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the static design of temporary and permanent retaining walls at the subject site:

- Conventional free-standing cantilever walls which support areas where movement is of little concern (ie. landscape walls), may be designed using a triangular lateral earth pressure distribution and an 'active' earth pressure coefficient, K_a, of 0.3, assuming a horizontal retained surface.
- Cantilever walls, the tops of which are restrained by the floor slabs of the permanent structure or which support movement sensitive elements, should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, K_o, of 0.6, assuming a horizontal retained surface.
- A bulk unit weight of $20kN/m^3$ should be adopted for the soil and extremely weathered sandstone profile.
- For progressively anchored or internally propped walls where minor movements can be tolerated, we recommend the use of a uniform triangular lateral earth pressure distribution of 6H kPa for the soil and extremely weathered sandstone profile, where 'H' is the retained height in metres.

- For progressively anchored or propped walls which support areas which were highly sensitive to movement (such as areas where movement sensitive structures or buried services are located in close proximity), we recommend the use of a uniform rectangular earth pressure distribution of 8H kPa for the soil and extremely weathered sandstone profile, where 'H' is the retained height in metres.
- Any surcharge affecting the walls (including adjacent high level footings, traffic loads, construction loads, etc) should be allowed in the designed using the appropriate earth pressure coefficient from above.
- The retaining walls must be designed as drained and measures taken to provide permanent and effective drainage of the ground behind the walls. Subsoil drains should incorporate a non-woven geotextile fabric (eg. Bidim A34) to act as a filter against subsoil erosion.
- Lateral toe resistance of the retention system may be achieve by keying or socketing the wall footing into bedrock below bulk excavation level. For key or socket depth design, adopt an allowable lateral stress of 200kPa for bedrock of at least very low strength. Higher allowable lateral stresses may be feasible depending on the results of the geotechnical investigation detailed in Section 5.7 below.
- Anchors which extend beyond the site boundaries will require the permission of neighbours before installation. Anchors should be bonded at least 3m into sandstone bedrock behind an imaginary line which extends up from the base of the excavation at 45°. The anchors should be designed for an allowable bond stress of 200kPa. Higher allowable bond stresses may be feasible depending on the results of the geotechnical investigation detailed in Section 5.7 below. All anchors should be proof-tested to 1.3 times the working load under the direction of an experienced engineer or construction foreman, independent of the anchor contractor. Alternatively, all field records and test results must be provided to the geotechnical engineer for review. We recommend that only experienced contractors be considered for anchor installation. We have assumed that permanent lateral support of the perimeter walls will be provided by the floor slabs of the new structure. If not, permanent anchors will be required and should be designed for corrosion resistance and long term durability.

5.5 Footings

We recommend that all structures be uniformly supported in sandstone bedrock, given its anticipated shallow depth of occurrence. Conventional pad, strip or pile footings founded in sandstone bedrock may be designed for an allowable bearing pressure of 1,000kPa. Pile footings may be designed in addition for an allowable shaft adhesion value of 100kPa over the depth of rock socket below bulk excavation level. Higher allowable bearing pressures may be feasible following completion of further investigations, as indicated in Section 5.7 below.

5.6 On-Grade Floor Slabs

Basement on-grade floor slabs will probably directly overlie sandstone bedrock. Underfloor drainage should therefore be provided. The underfloor drainage should comprise a strong, durable, single size washed aggregate (eg. 'blue metal' gravel). The underfloor drainage should connect with the walls drains, when appropriate, and direct groundwater seepage to a sump for pumped disposal to the stormwater system.

Joints in the concrete on-grade floor slab should be provided with dowels or keys.

5.7 Geotechnical and Hydrogeological Investigation

We recommend that a comprehensive geotechnical and hydrogeological investigation of the site be carried out.

The geotechnical investigation should comprised the drilling or cored boreholes into the underlying bedrock to establish the rock properties. At least two boreholes must be drilled over the extreme eastern portion of the site to confirm that the manmade fill does not extend into the site area. The boreholes should extend at least 2m into bedrock or 2m below bulk excavation level, whichever is the deepest. For the hydrogeological investigation, standpipes should be installed into at least three of the boreholes to allow longer term groundwater monitoring. If significant groundwater is encountered within the proposed excavation depths, pump-out tests within the standpipes should be carried out to determine the permeability of the rock mass. The installation of data loggers will assist to provide a continuous record of groundwater levels which will allow the effects of rainfall to be assessed. Seepage analysis can then be undertaken using information from the above investigations to confirm inflow rates, external drawdown and effects on surrounding buildings, structures and groundwater uses.

Once the results of the geotechnical and hydrogeological investigation become available, this report should be reviewed, revised and amplified, as appropriate.

5.8 Further Geotechnical Input

The following summarises the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Comprehensive geotechnical and hydrogeological investigation.
- Dilapidation surveys of neighbouring buildings and structures.
- Vibration monitoring during rock excavation.
- Monitoring of groundwater seepage into excavation.
- Proof-testing of anchors.
- Geotechnical footing inspections.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

It is possible that the subsurface soil, rock or groundwater conditions encountered during construction may be found to be different (or may be interpreted to be different) from those inferred from our surface observations in preparing this report. Also, we have not had the opportunity to observe surface run-off patterns during heavy rainfall and cannot comment directly on this aspect. If conditions appear to be at variance or cause concern for any reason, then we recommend that you immediately contact this office.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

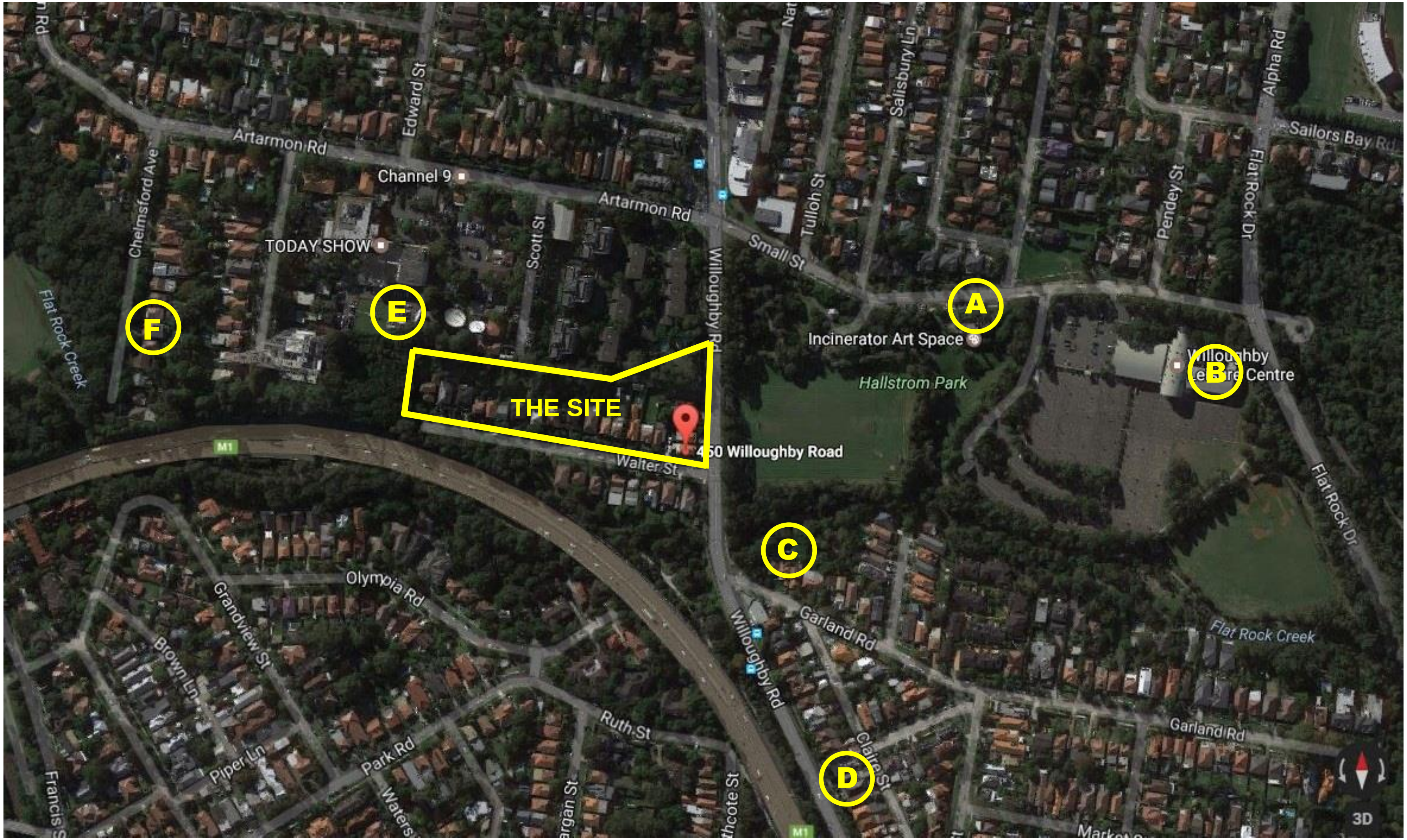

Image sourced from *Google Maps*

Site Location Plan

To be read in conjunction with text of report.

30088Zrpt • FIGURE 1

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.