EXPERIMENTAL INVESTIGATION OF DEBRIS DAMMING IN TRANSIENT FLOW CONDITIONS

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ABSTRACT

Debris entrained in tsunami bores and storms surges can form accumulations on structures, a phenomenon known as debris damming. Debris dams increase the exposed area to the flow, resulting in an increase in structural loading. Debris dams have also been known to result in a rise in water depth upstream of structures, which can cause overtopping. This study investigates the effects of debris damming through the use of idealized debris dams secured to a circular column. The idealized debris dams varied in terms of dam height and porosity. The experiments were conducted in transient flow conditions utilizing a modified dam-break wave. Results from this study illustrate that the presence of debris dams significantly increased the forces acting on the flow obstacle. Force coefficients decreased with increased impoundment depths and dam heights. Porosity reduced the loading on the column and the bulk area force coefficients.

Keywords: Debris, floods, tsunami, structures, extreme loads

1 INTRODUCTION

Extreme flooding events have resulted in catastrophic damage to coastal communities. This has been illustrated by many recent events including the 2004 Indian Ocean Tsunami, the 2005 Hurricane Katrina, and the 2011 Tōhoku Tsunami. These events have led to unexpected damage or failure to structures, highlighting the need for improved structural design standards and guidelines (Yeh et al., 2014; Nistor et al., 2017). Post-field surveys of these extreme events have concluded that these failures could be attributed to design codes not addressing extreme loading, including the loading due to debris (Yeh et al., 2014).

Debris entrained in storm surges or tsunamis can form accumulations on the face of or in-between structures, resulting in a substantial increase in loading (Yeh et al., 2014). The agglomeration of debris on structures is often referred to as debris damming. Debris dams increase the cross-sectional area exposed to the flow, resulting in an increase in the drag force acting on structures. Debris dams can also result in greater water depths upstream of structures due to increased flow obstruction, known as backwater rise. This change in water depth can cause overtopping of structures and result in greater hydrostatic pressures (Stolle et al., 2017). Lastly, floating debris dams have been known to increase vertical velocities and flow accelerations, thus resulting in increased scour surrounding structures’ foundations (Pagliara and Carnacina, 2011).

Debris damming has not been extensively investigated in transient flow conditions, however, there has been substantial research conducted in steady-state flow conditions due to the woody debris observed in fluvial systems. Through an experimental investigation, Parola et al. (2000) concluded that debris dams could form to the bottom of the flow depth, despite previous researchers considering them floating rafts (Diehl, 1997). To assess the geometry of debris dams, Zevenbergen et al. (2007) evaluated a photographic archive of woody debris dams surrounding bridge piers in the United States. This investigation also concluded that debris dams could accumulate to the bottom of the flow depth. In addition, the research concluded that two main debris dam geometries typically form: triangular or rectangular. Stolle et al. (2017) investigated the effects of debris composition and geometry on debris dam formation. It was observed that increasing the individual volume of debris resulted in an increase in dam size. The width of the debris dam was proportional to the quantity of debris supply.
The loading due to debris dams is commonly estimated using the drag force equation, as it is considered a quasi-steady force. This equation is described by FEMA (2012) as:

\[ F_D = \frac{1}{2} C_D \rho_f B h u^2 \]  

[1]

where \( C_D \) is the drag coefficient, \( \rho_f \) is the density of the fluid, \( B \) is the cross-stream width of the debris dam, \( h \) is the water depth, and \( u \) is the flow velocity.

The drag coefficient is typically used to describe the hydrodynamics of an object that is fully submerged. As this is not the case for surface piercing obstacles, a coefficient has been defined in the field of hydraulic engineering known as the resistance coefficient (Chaplin and Teigen, 2003). The resistance force is defined as:

\[ F_R = \frac{1}{2} C_R \rho_f B h u^2 \]  

[2]

where \( C_R \) is the resistance coefficient. A greater resistance coefficient implies that an obstacle is less hydrodynamically efficient, similar to the drag coefficient (Arnason, 2005). The resistance coefficient is also a function of the Froude number (Chaplin and Teigen, 2003). The use of the resistance coefficient is necessary as increasing flow velocities result in greater water depths directly upstream of the obstacle and decreased water depths directly downstream of the obstacle. This change in water depth upstream and downstream of the obstacle results in an increase in the hydrostatic pressure acting on the obstacle in the flow direction (Qi et al., 2014).

Previous research regarding debris damming has focused on the formation and scour effects of debris dams, while little research has been conducted on the loading due to debris dams. In addition, research regarding debris damming has been conducted primarily in the field of river engineering and, therefore, is mainly conducted in steady-state flow conditions. Due to the difficulties associated with the analysis of debris damming loads post-disaster, experimental investigations in transient flow conditions are required to develop a further understanding of extreme loads in flooding events.

This research investigates the impacts of debris damming on the induced loads and the surrounding flow conditions. Specifically, the effects of various debris dam geometry are evaluated with respect to the resistance force and the resistance force coefficients in transient flow conditions. This is done by utilizing idealized debris dams tested in a modified dam-break wave, which is commonly used to model on-land tsunami bores (Chanson, 2006). This wave was generated using a swing gate. The various geometries that were investigated includes dam height and dam porosity. The debris dams were tested in three impoundment depths to model varying flow conditions. The overall objectives of this study are:

- To analyze the loading due to debris damming and the various impacts of debris dam geometry including dam height and dam porosity.
- To investigate the impacts of debris dam geometry on the resistance force coefficient.

To the authors’ knowledge, this is the first study conducted utilizing idealized debris dams in transient flow conditions. Idealized debris dams were used so that the loading effects of the variables can be isolated and compared. This research will provide further understanding of the impacts of debris dam properties on structural loading and force coefficients.
2 EXPERIMENTAL SETUP

2.1 Experimental Facility

Physical modelling was utilized to analyze transient flow conditions surrounding debris dams. The experiments were conducted at the University of Ottawa in the Dam-Break Flume. This flume utilizes a swing gate to simulate dam-break waves, illustrated in Figure 1. The distances of the instruments from the column are listed in Table 1. The flume is 30.0 m long, 1.5 m wide, and 0.80 m high. The reservoir of the flume extends 21.55 m and the testing area is 8.45 m long. The experimental setup was placed on a false floor that is 0.30 m in height. A smooth acrylic cylinder was used to represent a column. The column had an outer diameter of 0.09 m and an inner diameter of 0.068 m. The column was suspended to a rigid mounting bridge attached to the flume walls. Various idealized debris dams were attached to the column using a collar. Each of the debris dams were tested in three varying impoundment depths, which were 0.30 m, 0.40 m, and 0.50 m.

![Diagram of Dam-Break Flume](image)

**Figure 1.** Parameters of the Dam-Break Flume used for the experimental campaign: (a) side view of the flume; (b) plan view of the flume.

2.2 Swing Gate

Tsunami waves are commonly modeled using the dam-break phenomenon as it models the propagation of a tsunami bore over a coastal plain (Chanson, 2006). The experiments utilized a hinged swing gate to produce a dam-break wave. This gate is constructed from marine plywood attached to a steel frame. The gate is 1.40 m wide and 0.80 m high. A watertight seal was placed surrounding the gate to ensure no leakage of the reservoir occurred during testing. Due to height restrictions of the flume, the maximum impoundment depth utilized was 0.50 m. There is a locking system used when the reservoir is filling to the required impoundment depth, and once that depth is reached, the mechanism is manually released. Several tests using the same impoundment depth were ran to ensure repeatability of the swing gate, which is discussed further in Section 3.1.

**Table 1.** Location of the instruments relative to the column based on the impoundment depth.

<table>
<thead>
<tr>
<th>Impoundment Depth (m)</th>
<th>( x_1 ) (m)</th>
<th>( x_2 ) (m)</th>
<th>( x_3 ) (m)</th>
</tr>
</thead>
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<tr>
<td>0.3</td>
<td>3</td>
<td>1.5</td>
<td>1.2</td>
</tr>
<tr>
<td>0.4</td>
<td>4</td>
<td>2</td>
<td>1.6</td>
</tr>
<tr>
<td>0.5</td>
<td>5</td>
<td>2.5</td>
<td>2</td>
</tr>
</tbody>
</table>
2.3 Instrumentation

Several instruments were used to record the results of the various tests, shown in Figure 1. The data from all of the instruments were recorded to a HBM data acquisition system (DAQ).

This ensures that all of the data are synchronized with time. A 6-axis load cell was attached to the top of the column to record a time history of the forces acting on the column and debris dams at a sampling rate of 300 Hz. The force-time history was filtered using an Empirical Mode Decomposition Filter (EMD) produced by Huang et al. (1998), which removes force spikes from the data. A time varying filter was then applied to the EMD filter, developed by Li et al. (2017). This filter is known to be more robust in terms of noise removal. Wave gauges (WG) and distance sensors (DS) were used to record the water depths throughout the flume. The sampling rate of both the wave gauges and distance sensors was 300 Hz. The location of the instruments is illustrated in Figure 1. An Acoustic Doppler Velocimeter (ADV) was used to record the flow velocities. The ADV recorded the flow velocity at three various flow depths for each impoundment depth. The flow depths were 60, 50, and 40 percent of the theoretical flow depth at each impoundment depth (Chanson, 2006). The ADV had a sampling rate of 200 Hz. The ADV was placed at the location where the column would be placed for the corresponding impoundment depths. The ADV velocity data was also filtered using the EMD filter for noise removal.

The location of the instruments varied for each impoundment depth so that the results could be compared non-dimensionally. The locations of the instruments were related using a non-dimensional variable to the impoundment depth. The location of the column was a factor of 10 of the impoundment depth from the swing gate. Table 1 lists the location of each instrument relative to the column with the corresponding impoundment depth, which is illustrated in Figure 1.

2.4 Debris Dam Specimens

Idealized debris dam specimens were constructed to analyze the effects of dam height and dam porosity. Plate dams were used to represent rectangular debris dam formations. The material used for the debris dams was acrylic plastic plates. All of the debris dams had a width (B) of 0.27 m. This was chosen as it was a multiple of three of the column diameter. The thickness of the plate dams was 0.013 m (0.5 inches). Three various debris dam heights (H) were tested. The dam heights were 0.05, 0.10, and 0.15 metres. Dams of partial height were suspended so that the top of the dam was placed at the height of the full dam height (Figure 2 (a)), as debris observed in extreme events are typically buoyant.

Three various porosities were tested, which were 0, 0.2, and 0.4. The diameter of the drilled holes was 0.01 m. Smaller hole size was utilized because smaller voids typically form in debris dams (Stolle et al., 2018). The spacing of the holes is shown in Figure 2 (b). The vertical spacing, S_v, is equal to 16 mm and the horizontal spacing, S_h, is equal to 23.3 and 12.2 mm for the dams of 20 percent and 40 percent porosity, respectively. The porosity was calculated as the cross-sectional area of the drilled holes divided by the cross-sectional area of the debris dam. The porous dams had a height of 0.15 metres. Table 2 lists the variations of obstacles that were tested. Obstacle D00 represents the column with no dam present.

![Figure 2](image-url)  
Figure 2. Idealized debris dams used in experimental campaign: (a) debris dam with a dam height of 0.10 m; (b) spacing of holes used for porous dams.
2.5 Protocol

Experiments were first tested without the presence of any structure to assess the repeatability of the experiments and to determine the hydrodynamic conditions. In these experiments, the ADV measured the flow velocities at three various flow depths based on the impoundment depth. Each measurement depth was tested five times at each impoundment depth, resulting in 45 tests without a structure present. The wave gauges were also utilized for this test series. Tests were then run with only the column present. At each impoundment depth, five tests were conducted with only the column present. Each debris dam had a series of three tests conducted for each impoundment depth. The load cell and wave gauges were utilized for experiments with an obstacle present. The location of the instruments was described in Section 2.2. For all tests, the DAQ system began to record the data and then the manual swing gate was utilized. The DAQ system recorded for a minimum of 60 seconds for each test.

<table>
<thead>
<tr>
<th>Obstacle</th>
<th>Dam Height, ( H ) (m)</th>
<th>Dam Width, ( B ) (m)</th>
<th>Dam Porosity, ( n (-) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>D00</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>D10</td>
<td>0.15</td>
<td>0.27</td>
<td>0</td>
</tr>
<tr>
<td>D20</td>
<td>0.10</td>
<td>0.27</td>
<td>0</td>
</tr>
<tr>
<td>D30</td>
<td>0.05</td>
<td>0.27</td>
<td>0</td>
</tr>
<tr>
<td>D12</td>
<td>0.15</td>
<td>0.27</td>
<td>0.2</td>
</tr>
<tr>
<td>D14</td>
<td>0.15</td>
<td>0.27</td>
<td>0.4</td>
</tr>
</tbody>
</table>

3 RESULTS

3.1 Hydrodynamics

The experiments were tested in transient flow conditions utilizing the dam-break phenomenon to model an on-land tsunami bore. Three impoundment depths (0.3 m, 0.4 m, and 0.5 m) were tested to obtain a range of flow parameters. Flow velocities and water depths were recorded with no obstacles present to assess the hydrodynamic conditions of the experiments, which are illustrated in Figure 3. The time history of the water depths recorded were used to assess the repeatability of the experiments. This was done by calculating the standard deviation of the water depth with time. The mean standard deviation for the experiments is 0.0031 m and the maximum standard deviation was 0.0177 m. Therefore, it was concluded that the experiments could be considered repeatable.
Due to problems associated with ADVs utilized in aerated or turbulent flow (Mori et al., 2007), the initial velocity data recorded had a low signal-to-noise ratio as well as not displaying the expected behavior. Therefore, theoretical values proposed by Leal et al. (2006) were calibrated to the recorded velocities. The theoretical values were used in the analysis until the expected flow behavior was observed. A depth-averaged mean flow velocity was calculated using the three recording depths for each impoundment depth, which was utilized in the analysis. The flow velocities obtained from the filtered ADV data ranged from 0.78 to 1.84 m/s for the first 10 seconds of recorded data after the wave arrival, to avoid downstream influences. This results in a range of Froude numbers from 0.86 to 1.60 and a range of Reynolds numbers from $7.31 \times 10^4$ to $2.57 \times 10^5$, where the characteristic length is the water depth. The experiments were scaled using Froude similitude using a 1:30 geometric scale. This results in velocities ranging from 4.7 to 8.8 m/s at prototype scale. This is comparable to flow velocities (3 to 8 m/s) observed in the 2004 Indian Ocean Tsunami by Matsutomi et al. (2006). The water depth ranged from 0 to 0.18 m with no obstacle present in the experimental campaign. At prototype scale, this is 0 to 5.4 m, which is within the range of water depth of 0 to 9 m observed by Borrelo (2005) in the 2004 Indian Ocean Tsunami.

3.2 Resistance Forces

The forces acting on the debris dam and column were recorded using a load cell. The mean of the streamwise forces for each obstacle were plotted as a function of time, illustrated in Figure 4. A large initial increase in loading can be observed upon the flow arrival. Increasing the impoundment depth results in an increase in the forces acting on the flow obstructions, which can be observed when comparing the three various subplots. This is an expected
result as the resistance force is a function of the velocity and the water depth at the flow obstruction. The presence of the dam also results in a large increase in loading compared to when there is no dam present (D00). When comparing the results of D10, D20, and D30, a significant increase in loading can be observed with increasing height. This is expected, as the resistance force is a function of the cross-sectional area exposed to the flow, as displayed in Eq. [2]. The porosity also contributes to a decrease in the resistance forces. There is a decrease in loading from D10 to D12 and D14, as porosity results in a reduction in the area exposed to flow. However, there is not a significant difference in loading between D12 and D14 for the impoundment depths of 0.3 m and 0.4 m. This is likely due to the effects of skin friction of the porous holes. The presence of the holes results in a greater decrease in blockage of the flow from D10 to D12 and D14; however, there is not a significant decrease in blockage from D12 and D14.

Figure 4. Time histories of the streamwise forces acting on the flow obstacles: impoundment depth of (a) 0.3 m; (b) 0.4 m; and (c) 0.5 m.

3.3 Resistance Coefficient

The resistance coefficient was calculated for each test using Eq. [2]. The velocity used to calculate the resistance coefficient was the mean velocity corresponding to each impoundment depth with no obstacle present, outlined in Section 3.1. The water depth used was the mean of the water depths recorded by WS1 with no obstacle present at each corresponding impoundment depth. The water depth with no obstacle present was utilized to remain consistent with the mean flow velocity. The force used in the calculation was the time histories of the mean streamwise forces outlined in Section 3.2. Two resistance coefficients were calculated, the first being the effective resistance coefficient ($C_{RE}$). This is calculated using a depth averaged cross-sectional area of the debris dam and column, which subtracts the area of the porous holes, shown in Eq. [3]. This was proposed so that the impact of porosity on the resistance coefficient could be assessed. The second coefficient is defined as the bulk resistance coefficient ($C_{RB}$), which was proposed due to the difficulties associated with determining the expected porosity of a debris dam in the design process. The bulk resistance coefficient was calculated using a depth averaged cross-sectional area based on the column and dams, shown in Eq. [4]. Both the effective and bulk resistance area vary with time as the water depth changes with time. The effective area and bulk area, respectively, are calculated as:

$$A_E = (h-H)D + (1-n)HB$$  \[3\]

$$A_B = (h-H)D + HB$$  \[4\]

where $h$ is the water depth recorded by WS1 and $D$ is the column diameter.
3.3.1 Effective Resistance Coefficient

The effective resistance coefficient and the bulk resistance coefficient are equivalent for nonporous dams as they are calculated using the same area. Therefore, a comparison of D00, D10, D20, and D30 will be discussed in only this section. A time history of the effective resistance coefficients is illustrated in Figure 5, where each plot is separated by the impoundment depth due to the volume of data. A summary of the maximum and mean effective resistance coefficients are listed in Table 3. The maximum and mean resistance coefficients were determined for the first 10 seconds after the wave arrival to avoid any downstream influences. Generally, there is an increase in the resistance coefficient with a decrease in the impoundment depth. The resistance coefficient is a function of the Froude number so this result can be expected (Chaplin and Teigen, 2003). There is generally an increase in the resistance coefficient when there is a dam present in comparison to only the column (D00). Plates have a greater drag coefficient in comparison to cylinders so this result is expected.

![Figure 5. The effective resistance coefficients as a function of time: impoundment depth of (a) 0.3 m; (b) 0.4 m; and (c) 0.5 m.](image)

Once the water depth exceeds the depth at which the dams are placed, the dams of partial height, D20 and D30, generally have a greater or equivalent resistance coefficient relative to D10. D30 particularly resulted in greater resistance coefficients relative to D10. This is an unexpected result as D10 has greater blockage relative to D20 and D30. This behavior could be attributed to the increased backwater rise that was observed with increasing dam height. It is expected that the backwater rise resulted in a reduction of flow velocities in front of the flow obstacle. The resistance force is a function of the squared velocity and the water depth. This means that the backwater rise can result in a reduction in resistance forces acting on the debris dams, as observed by Stolle et al. (2017). Therefore, the observed backwater rise could result in lesser resistance coefficients for dams of greater height. The change in dam height may also result in greater flow accelerations as the blockage of the flow depth is changed. This may contribute to the increased resistance coefficient of the dams of lesser height.

The effective resistance coefficients of D12 and D14 are generally greater than or equivalent to that of D10 and, therefore, the porosity is resulting in greater effective resistance coefficients. This is likely due to the skin friction created by the porous holes resulting in greater resistance forces compared to a dam with a reduced cross-sectional area of equivalent value. Although the loading is lesser in the presence of porous dams, the subtraction of the porous area from the calculation of the effective resistance coefficient increases the coefficient.
Particularly, D14 has a greater effective resistance coefficient in comparison to D10 due to the greater reduction in cross-sectional area. The effective resistance coefficient of D12 is lesser than that of D10 at the impoundment depth of 0.3 m and for large portions of the impoundment depth of 0.4 m.

Table 3. Maximum and mean values for the effective resistance coefficients for each dam in terms of impoundment depth.

<table>
<thead>
<tr>
<th></th>
<th>Maximum $C_{RE}$ [-]</th>
<th>Mean $C_{RE}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>h_o=0.3 m</td>
<td>h_o=0.4 m</td>
<td>h_o=0.5 m</td>
</tr>
<tr>
<td>D00</td>
<td>1.264</td>
<td>1.260</td>
</tr>
<tr>
<td>D10</td>
<td>2.430</td>
<td>2.050</td>
</tr>
<tr>
<td>D20</td>
<td>2.613</td>
<td>1.752</td>
</tr>
<tr>
<td>D30</td>
<td>3.234</td>
<td>1.646</td>
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<tr>
<td>D12</td>
<td>2.139</td>
<td>1.448</td>
</tr>
<tr>
<td>D14</td>
<td>3.079</td>
<td>1.549</td>
</tr>
</tbody>
</table>

3.3.2 Bulk Resistance Coefficient

The porosity of the debris dams generally results in a reduction in the bulk resistance coefficients when comparing that of D10, D12, and D14, as illustrated in Figure 6. This is likely due to the decrease in loading caused by the porous holes reducing the area exposed to the flow. This results in a lesser bulk resistance coefficient as the same cross-sectional area was used for the porous dams as the nonporous dams. A summary of the maximum and minimum values for the bulk resistance coefficient is listed in Table 4. As previously mentioned, the maximum and mean bulk resistance coefficients were calculated from the time of the wave arrival to 10 seconds after this occurrence. There is a greater difference between the mean bulk resistance coefficients of D10 and D12 compared to D12 and D14. Although there is a 20 percent decrease in the area exposed to the flow, there is likely lesser differences between D12 and D14 as the change in channel blockage is greater from D10 to D12 relative to D12 to D14.

From an engineering standpoint, the bulk resistance coefficient may present a more feasible method of estimating the resistance forces of debris dams in transient flow conditions. By subtracting the reduced porous area, the effective resistance coefficient likely overestimates the coefficient due to the increased resistance forces created by the friction of the porous holes. There are also significant difficulties associated with estimating dam porosity in extreme flooding events. In addition, the holes of the porous dams were drilled parallel to the flow direction. In reality, the voids in a debris dam would be 3-dimensional, lessening the impacts of porosity on the resistance coefficient.

Figure 6. The bulk resistance coefficients as a function of time: impoundment depth of (a) 0.3 m; (b) 0.4 m; and (c) 0.5 m.
Table 4. Maximum and mean values for the bulk resistance coefficients for each dam in terms of impoundment depth.

<table>
<thead>
<tr>
<th></th>
<th>Maximum $C_{RB}$</th>
<th>Mean $C_{RB}$</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>$h_o=0.3$ m</td>
<td>$h_o=0.4$ m</td>
</tr>
<tr>
<td>D10</td>
<td>2.430</td>
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<tr>
<td>D12</td>
<td>1.712</td>
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<tr>
<td>D14</td>
<td>1.847</td>
<td>0.929</td>
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4 CONCLUSIONS

This study investigated the loading effects of idealized debris dams in transient flow conditions. The various geometric properties including debris dam height and porosity were tested in various flow conditions to obtain a better understanding of the flow behaviour surrounding debris dams. The induced loads and the resistance coefficients were determined and compared in terms of the debris dam geometry. A summary of the conclusions stemming from this study are as follows:

- The loading due to debris dams is proportional to the height of the debris dams as the resistance force is a function of the cross-sectional area exposed to flow.
- Porosity resulted in a decrease in the forces acting on the dams and column.
- The resistance coefficients decreased with increasing impoundment depth.
- The dam height was generally inversely proportional to the resistance coefficient, which may be due to the increased backwater rise observed for dams of greater heights.
- The porosity of the debris dams resulted in a decreased bulk resistance coefficient due to the decrease in resistance forces of the porous dams and an increased effective resistance coefficient due to the skin friction of the porous holes.

This research demonstrates that debris dam properties can significantly affect the loading conditions of structures. Three variables were tested in this project including the flow conditions, debris dam height, and debris dam porosity. Further investigations are required to examine a wider range of the investigated variables and other debris dam properties such as roughness and shape.

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